

Government of India

✓ Central Waterways, Irrigation and Navigation Research Station, Poona.

Director

K. K. FRAMJI, M. B. E., I. S. E. /

RESEARCH PUBLICATION No. 12
**ANNUAL REPORT (TECHNICAL)
OF WORK DONE
DURING THE YEAR 1947**

AND INDEX FOR 1943-1947.

BY

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IWE. 1.44	9	Annual Report (Technical) of work done during the year 1944 and Index for 1940-'44 by Sir Claude Inglis, Rao Bahadur D. V. Joglekar and staff at Khadakvasla.	7	8 0	1945
IWE. 1.45	10	Annual Report (Technical) of work done during year 1945 and Index for 1937 to 1945 by Rao Bahadur D. V. Joglekar and staff at Khadakvasla.	...		In Press.
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Errata for Annual Report (Technical) 1947.

Page	Line	Incorrect	Correct
ii	21 from top.	sanction	station
3	20 from bottom.	Mr. J. J. Murray	Mr. J. B. Murray
23	Para 4, lines 1 & 2.	The width of the cill and sides is 3.5'	The width of the cill is 3.5'
25	Para 7, line 7.	'C' by current mater	'C' by current meter
35	12 from bottom	Bisnasi	Birnasi
37	heading of column 2.	Total quantity of sand collected in all branches.	Total quantity of sand collected in all branches in Cft.
	" 4.	Qs	Qs in Cft.
	" 7.	Qs	Qs in Cft.
	" 10.	Qs	Qs in Cft.
59	5 from top.	$R = \left[\frac{Q}{f} \right]^{\frac{1}{3}} = \frac{Q^{\frac{1}{3}}}{m^{\frac{1}{6}}}$	$R \propto \left[\frac{Q}{f} \right]^{\frac{1}{3}} \propto \frac{Q^{\frac{1}{3}}}{m^{\frac{1}{6}}}$
18	" "	.248	2.48
61	3 from bottom.	assumed to be 0.7 mm.	assumed to be 0.17 mm.
61	2 from bottom.	$f = \sqrt{m} \times 1.76 = 73.$	$f = \sqrt{m} \times 1.76 = 0.73.$
70	Para 3. 2, line 8.	deths	depths
77	Table IIA, Item 4.	Bansbassa	Banbassa
77	Table IIA, Item 6.	Yamuna dear	Yamuna near
77	Foot Note.	Prototope	Prototype
81	Item against Page 12.	Prototope	Prototype
101	6 from bottom.	fact that	fact but
103	9 from bottom.	$h^3e \sqrt{\frac{H}{30}} = \left(\frac{3}{20}\right)^3 \times 40 \sqrt{\frac{3}{30}}$	$h^3e \sqrt{\frac{H}{30}} = \left(\frac{3}{25}\right)^3 \times 40 \sqrt{\frac{3}{30}}$
103	3 below table 1.	$V = 1.059 \times 10^5.$	$V = 1.059 \times 10^{-5}$
117	23 from bottom.	thh	the
117	22 from bottom.	strengte	strength
118	17 from top.	respec	respect
118	18 from bottom.	Barraage	Barrage
123	Para 6. 4. 2, line 8.	occave	concave
135	5 from top.	uogus	vogue
135	8 from bottom.	oross	cross
145	column 11, Item 4.	14,0000	14,000
154	Para 4 (a), line 1.	acon	soon
155	Para 4. 3, line 14.	approved	approved
156	Para 4. 4(b), line 7.	conditons	conditions
156	Line 5 from bottom.	thr	the
156	Line 4 from bottom.	rathee	rather
166	3. 2 line 5.	0.056	0.056 = 2.02 ft.
173	9 from top.	fouling	falling
180	Para 2. 2, line 2.	60,0000 cusecs	60,000 cusecs
189	Line 2 from bottom.	A four foot wide feet-bridge	A four feet wide foot-bridge
204	14 from top.	envolved	evolved
205	8 from top.	bringed	bridge
214.	Para 4. 3, line 3.	linely	likely

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INTRODUCTION.

1. *Introductory note by Shri A. N. Khosla, I. S. E., Consulting Engineer to the Government of India for Waterways, Irrigation and Navigation.*

The Khadakwasla Research Station was first established in 1929 by the Bombay Government. The Research Station was taken over by the Government of India in the Labour Department in 1937 and named the Central Hydrodynamic Research Station. This was later renamed the Indian Waterways Experiment Station. With the creation of the post of Consulting Engineer to the Government of India for Waterways, Irrigation and Navigation, this Station was placed in control of the Consulting Engineer.

On account of the increased demand for research facilities from the Central Government and from Governments of Provinces and States and in order to cope with the special research work required in connection with the large number of river development projects under investigation, it has been found necessary to enlarge the scope of this Station. It has, therefore, been decided to reorganise the Station and raise it to the status of the Central Waterways, Irrigation and Navigation Research Station (Letter No. DW. Dy. 13975 dated the 22nd May 1947 from the Ministry of Works, Mines and Power - Appendix I). The present Station at Poona is to be expanded as far as possible and a new one set up at Okhla near Delhi. Between these two Stations it is intended to deal with all research problems connected with the schemes for the conservation and utilisation of the water resources of India. The reorganised Research Station will include the following sections :-

- (1) River and Canal Hydraulics
- (2) Navigation
- (3) Soils and Soil Mechanics
- (4) Concrete and Materials of Construction
- (5) Statistics
- (6) Chemistry
- (7) Physics
- (8) Mathematics.

New sections will be added to deal with the problems connected with high dams and appurtenant works and hydro-electric development including the designs of waterworks and generators. With the growing need for the rapid development of the many new valleys in India, it is felt that the calls on this Research Station for carrying out experiments in all aspects of its activities will be progressively on the increase and may soon outgrow the capacity of the Station. It will accordingly become necessary to expand the Station and establish specialized branches located at the sites of the various projects now under contemplation or execution.

In the present research Station, research has been mainly confined to Waterways, river control and hydraulics. There has been a certain amount of fundamental research in other directions also. Notable among the experiments carried out during the year are those on silt exclusion from the right bank canals of Sukkur Barrage, on the design of the proposed Lower Sind Barrage at Kotri, experiments with Volute Syphon Spillways, experiments on canal falls, on the training of the Katjuri river at Khanagarh and the Hoogly near the Titaghur Jute Mills.

The report also contains notes on rainfall, runoff, flood discharges, river control problems etc. These are of the nature of expression of views to stimulate thought and research.

The work of the Station was ably handled during the year by Mr. K. K. Framji, I. S. E., the Director of the Station; Rao Bahadur D. V. Joglekar, Deputy Director, the Research Officers and Staff at Khadakwasla.

APPENDIX I

Copy of letter No. DW. Dy. 13975 dated 22nd May 1948 from the Deputy Secretary to the Govt. of India, Ministry of Works, Mines and Power, to the Consulting Engineer to the Govt. of India for Waterways, Irrigation and Navigation.

Subject:- Reorganization and Expansion of Indian Waterways, Experiment Station into Central Waterways, Irrigation and Navigation Research Station.

I am directed to say that the Governor General in Council is pleased to accept the necessity of converting the present Indian Waterways Experiment Station at Poona into a strong Technical Institute under the control of the Consulting Engineer for Waterways and Irrigation to meet the largely increased demands for research in connection with the irrigation and hydro-electric schemes for the conservation and control of the Waterways of the country and sanction its expansion and reorganization on a permanent basis. The reorganized Station will henceforth be known as the Central Waterways, Irrigation and Navigation Research Station. Some portion of the Research Station as cannot be located at Poona, will be located at Delhi. The consulting Engineer will submit proposals separately for the orders of Government which of the officers of the Station will work at Poona.

2 The sanction will consist of the following research sections :-

1. River and Canal Hydraulics;
2. Navigation;
3. Concrete & materials of construction (e.g. cement, mortars, aggregates, etc);
4. Soils and Soil Mechanics;
5. Mathematics;
6. Statistics;
7. Physics; and
8. Chemistry.

A Director and two Deputy Directors will be in charge of the Station and a Chief Research Officer will be at the head of each Research Section. A statement showing the posts for the station together with the rates of pay for the posts is attached. These rates of pay will be subject to revision on the basis of the recommendations of the Pay Commission. The incumbents of the posts will also be eligible for the cost of living allowance or the dearness allowance, as the case may be.

3. The Governor-General in Council is pleased to sanction a non-recurring expenditure of Rs. 15 lakhs for buildings and equipment and a recurring expenditure of Rs. 6½ lakhs per year. I am to request you to submit detailed proposals in regard to the former in due course.

4. I am to add that the Consulting Engineer should prepare a schedule of overhead charges for the experiments conducted by the Station on behalf of the Provincial and State Governments and submit it for the orders of Government.

5. The expenditure on account of the reorganization of the station will be debitable to the "Major Head-18 - other Revenue Expenditure financed from ordinary Revenues (Central)".

ANNUAL REPORT (TECHNICAL) OF THE WORK DONE DURING THE YEAR 1947.

2. STAFF AND THEIR ACTIVITIES.

Director : Mr. K. K. Framji, M. B. E., B. A., (Bom.), B. Sc. (Lond.),
A. K. C. (Lond.), A. M. Am. S. C. E., M. I. E., I. S. E.

Deputy Director : Rao Bahadur D. V. Joglekar, B. E.

Assistant Research Officers.

1. Mr. S. T. Ghotankar, M. Sc.
2. Mr. C. V. Gole, B. E.
3. Mr. S. N. Mulekar, B. Sc. (Elec. and Mech.) from 22-5-47.
4. Mr. S. V. Chitale, B. E. from 52-5-47.
5. Mr. P. K. Kulkarni, B. Sc. (Elec. & Mech.) from 22-5-47.

Research Assistants.

1. Mr. S. N. Mulekar, B. Sc. (Elec. and Mech.) up to 22-5-47.
2. Mr. S. V. Chitale, B. E. up to 22-5-47.
3. Mr. D. P. Kulkarni, B. E. (till 22-3-47).
4. Mr. P. K. Kulkarni, B. Sc. (Elec. and Mech.) up to 22-5-47.
5. Mr. G. T. Wadekar, B. E.
6. Mr. C. G. Tamhankar, B. Sc. (Elec. and Mech.).
7. Mr. P. M. Damle, B. Sc.

Overseer.

Mr. N. K. R. Iyengar.

Computors,

Mr. S. B. Barve.

Mr. R. P. C. Varma, B. Sc.

Office Superintendent.

Mr. D. T. Deshpande from 23-5-47.

The Khadakvasla Welfare Centre, which was run by voluntary work and financed by contribution from the Red Cross, the Rotary Club, Collector's Fund and Members of the Staff of the Station, has been taken over by the Government of Bombay since 1st July 1947.

Figure 1 shows:—

- (i) the 7 places visited by the Director during the year;
- (ii) the 12 places where advice was given without model experiments;
- and (iii) the 18 places where advice was given after model experiments.

The 12 cases in which advice was given without model experiments were as under:—

1. Prevention of damage done by floods in the Auranga River, Taluka Bulsar, District Surat.
2. Protecting the Right Bank of the Luni at Balotra by means of a Repelling Spur.
3. Erection of a dam at Wahegaon, Taluka Karad, District Satara.
4. Proposed Feeder for irrigation in Cutch State from the Kotri Barrage (see item G-1-3).
5. Proposed construction of a combined road and rail bridge over the River Ganga at Patna or Mokameh Ghat. (see item G-1-2).
6. Measures to be taken to stop erosion by Janoli River in Ratnagiri District of Bombay Province and protection of the " Hind Chatralaya " Property at Kankavli.
7. Breaches on N. W. line of M. & S. M. Railway Miles 82-113.

8. Prevention of damage by floods to the downstream apron of the surplus weir in the Victoria Sagar Tank in Sambalpur District, Orissa.

9. Training of the river Ganga at Kanpur.

10 Coefficients of discharge in notch type and weir type falls on the Mahanadi Main Canal, C. P.

11. Training works for Alexandra Bridge over Chenab River.

12. Design of S. W. F. of Jawai River Project.

The 18 Cases in which advice was given after model experiments are described in detail in the body of this Report.

From 1st to 5th January 1947 the Director attended the annual session of the Indian Science Congress, held at New Delhi. During his stay there he also inspected the site at Okhla, near Delhi, where the new Central Waterways Irrigation & Navigation Research Station is proposed to be located. He then paid a visit to Cuttack from 6th to 13th January 1947 and inspected the Jobra weir across the Mahanadi, the Khannagar revetment and other important river problems on the Mahanadi system.

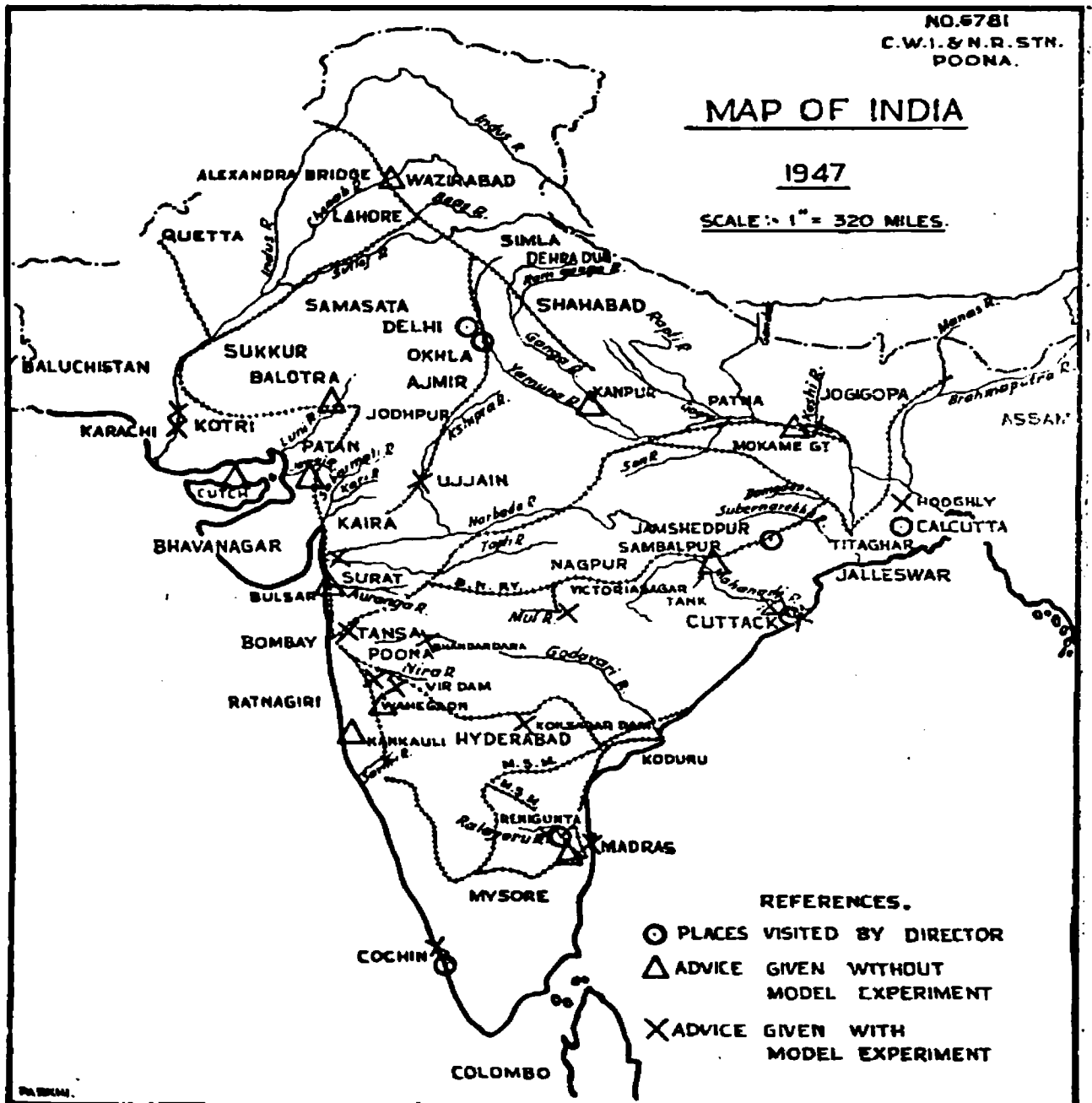


FIG 1.

At the request of the Administrative Officer, Cochin, the Director and Mr. T. M. Oag, Director of Navigation, Central Waterways Irrigation & Navigation Commission visited Cochin from 20th to 26th March 1947. They inspected the Cochin Harbour and discussed with the Administrative Officer the desirability of constructing a Tidal Model of the harbour area. A Preliminary note on the immediate ameliorative measures (see item C-2 vi-2) was furnished.

On an invitation from the Chief Engineer, M. & S. M. Railway, Madras, the Director toured the M. & S. M. Line from Renigunta to Koduru between 27th & 31st March 1947 for the inspection of the Railway Bridge over Swarnamukhi River and other bridges which were severely damaged by the 1946 floods. Remedial measures were suggested.

Between 13th to 20th May 1947, the Director visited Jamshedpur in connection with a problem of having sand-free water supply from the Subarnarekha River for the Tata Iron & Steel works and discussed with the Chief Engineer various points regarding the location of the intake, the desirability of dismantling the lower diversion dam, and the potentialities of sluicing arrangements, etc.

At the instance of the Consulting Engineer to the Govt. of India for Waterways, Irrigation & Navigation, the Director paid a visit to Calcutta in the middle of October 1947 for discussions with the Commissioners of the Port of Calcutta regarding the proposed Hooghly River Model Experiments.

The Hon'ble Mr. N. V. Gadgil, Minister for Works, Mines and Power, Mr. Khosla, Consulting Engineer to the Government of India for Waterways Irrigation & Navigation, and Sir Claude Inglis (late Director of the Station) were the distinguished visitors to the Station during the year. Other prominent visitors were:—

- (1) The Chief Engineer, Irrigation, Hyderabad (Deccan).
- (2) Rai Bahadur Brij Narain, Planning Officer of Transport Department, Govt. of India.
- (3) Mr. T. M. Oag, Director of Navigation, Central Waterways, Irrigation & Navigation Commission.
- (4) Mr. Mansingh, Director of Irrigation, Central Waterways Irrigation & Navigation Commission.
- (5) Mr. K. Green, Australia.
- (6) Mr. J. J. Murray, Chief Engineer, Port of Calcutta.
- (7) Commander Pawsey, Deputy Conservator, Port of Calcutta.
- (8) Mr. M. D. Kilford, River Surveyor, The Commissioners for the Port of Calcutta.
- (9) Lt. Col. T. Blench, Director, Punjab Irrigation Research Institute, Lahore.
- (10) The Sanitary Engineer, Gwalior.
- (11) Mr. A. R. Thomas, Executive Engineer, Bombay.
- (12) Mr. G. N. Pandit, Project Officer, Central Waterways, Irrigation & Navigation Commission, New Delhi.
- (13) Shri T. E. Sheshadri, Physics & Soil Mechanics Officer, Madras.
- (14) Mr. M. N. Venkateson, Central Technical Power Board, Simla.
- (15) Mr. T. M. Sethna, the Executive Engineer, Research Division, Karachi.
- (16) The Executive Engineer, G. I. P. Railway, Bombay.
- (17) Mr. Prasad, Assistant Engineer, Bihar.

Students and Professors from the following colleges and institutions also visited the Station during the year:—

- (1) Students of the Class of Well Inspectors, Bombay.
- (2) College of Engineering, Poona.

- (3) Kala Bhavan Technical Institute, Baroda.
- (4) Shri Jayachamarajendra Occupational Institute, Bangalore.
- (5) School of Engineering, Bangalore.
- (6) Kalyan High School, Kalyan.

The following experiments are in hand or on the Priority list:—

Priority I (Specific Experiments)

- (1) Experiments for the Kotri Barrage Design (Sind).
- (2) Coefficient of discharge of Lower Sind Barrage profile.
- (3) Kuakhai Road Bridge site CO : experiments to evolve the necessary training works (Orissa).
- (4) Katjuri at Khannagar near Cuttack, Orissa : experiments to protect the Khannagar bank.
- (5) Hooghly estuary model : experiments for comprehensive training of the estuary for navigation.
- (6) Training of the Kshipra at Ujjain (Gwalior State).
- (7) Bhandardara Waste weir experiments : design for Siphon Spillway (Bombay).
- (8) Tansa Automatic Gate Design (Bombay).
- (9) Training of the Tapti River at Surat (Bombay).
- (10) Training the Ralayeru River at M. & S. M. Rly Bridge No. 189.
- (11) Improving the Cochin Harbour.
- (12) Staunching of canals - testing P. B. S. Lining.
- (13) Testing the suitability of alternative bridge sites on the Ganga at Patna and Mokameh.
- (14) Reducing the range in Madras Harbour.
- (15) Design of a Siphon Spillway for the proposed Vir dam.
- (16) Study of pressures on the masonry floor of the proposed Vir dam.
- (17) Design of protective measures for the B. N. Railway bridge across the river Mul (Central Provinces).
- (18) Koilsagar Dam (Hyderabad Deccan): design for a siphon spillway.

Priority II (Specific Experiments)

- (1) Training of the river Bhadar (Bombay)
- (2) Design of a siphon to supply discharge into the Mutha Right Bank Canal (Bombay).
- (3) Experiments for the Kabo Head-works on the river Mu (Burma).
- (4) Protecting the M. & S. M. Railway Bridge across the Radihalla-Benihalla river.
- (5) Training of the Ganga at Kanpur.
- (6) Prevention of bar formation at the Ratnagiri Creek (Bombay).
- (7) Prevention of bar formation and erosion at the Savitri River (Bombay).
- (8) Training of the Alibag port for navigation (Bombay).
- (9) Design of the training works for the proposed bridge across the Godavari at Rajamahendry.
- (10) Design of the training works for the proposed Rly. bridge across the Kosi at Kursela (O. T. Rly).
- (11) S. W. F. design for the Jawai project canal.

PRIORITY (BASIC EXPERIMENTS.)

- (1) Sand Channel experiments : to evolve the relationship between various hydraulic factors including charge.
- (2) Meander experiments : to evolve the relations between the meander lengths, belts, etc. and charge and discharge.
- (3) Attracting and Repelling spurs and Islands—their position, orientation and dimensions in respect to meanders characteristics.
- (4) Standardisation of high coefficient weirs.
- (5) Study of the vortex formation, its profile etc. in connection with the design of the Ganesh Iyer's volute siphon.
- (6) Standardisation of Venturi flumes.
- (7) Study of water profiles in S. W. Flumes.
- (8) Study of the effect of curvature on the maximum recovery of head with different glacis slopes in Flumes & Falls.
- (9) Study of form drag on piers.
- (10) Standardisation of fluming of regulators.
- (11) Improving the wave producing machine.
- (12) Scour downstream of rigid structures.

Three Notes on " Basic " problems and 12 Notes on " Specific " problems were written during the year.

Reorganisation and expansion of the Station have been sanctioned to meet the largely increased demands for research in connection with the Irrigation and Hydro-electric schemes for the conservation and control of the waterways of the country *vide* Govt. of India Department of Works, Mines and Power No. Dw. Dy. 13975 of 22. 5. 47. Selection of staff and provision of equipment and buildings are in progress.

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

(i) *Total volume of run-off including the standardisation of hydrographs and their analysis.*

A - 1 (i) 1. SOME FEATURES OF PRECIPITATIONAL VARIATIONS IN THE UPPER INDUS CATCHMENT—

A PROBABILITY METHOD OF APPROACH.

By

G. M. PANCHANG & K. K. FRAMJI, I. S. E.

O. O. Introduction :

Precipitation has been recognized for a long time as one of Nature's most vagarious phenomena. Its year to year or place to place variations are so capricious that even approximately long term predictions cannot be made with any fair measure of certainty. All the same, instances are not wanting when rainfall of individual stations and/or catchments have exhibited peculiar characteristic features which keep on repeating in series of years. To illustrate—though there may be no apparent continuity in the total precipitation from one year to the next, yet the annual precipitation of a station (or catchment), when averaged over a sufficient number of years, keeps sensibly constant. It is thus a matter for conjecture whether the year to year departures from the long term average are not also governed by a set of precise probability laws as in the case of other natural phenomena.

A number of hypotheses have been postulated at different times for the laws governing the variations in natural phenomena. One hypothesis, for example, puts it that rainfall at any place is cyclic, being continuously above normal for a number of years and below normal for an approximately equal number of succeeding years, the total number of years together forming the full period of the cycle.

The present paper is an attempt to examine the recorded data of 75 years (1865-1940) of all the raingauge stations (of the India Mefeorological Department) included in the upper catchment of the Indus and its three principal tributaries, viz. the Sutlej, the Beas and the Chenab, for the existence of any such laws.

O. 1. Questionnaire :

The following questionnaire was, therefore, framed by the Director (when he was on Special Duty in Sind) to examine certain of the more accepted hypotheses in their applicability to the above data :

- (1) Does the year to year rainfall phenomena obey any long-term or short-term cyclic law? If so, what is the most probable period of the cycle?
- (2) If rainfall is not a periodic function of time, what is the least number of years for which the mean value of annual rainfall does not vary appreciably from that over a longer period of years? 35 years?
- (3) What is the extent of year to year variations from the normal? Do the variations fall within well defined limits?
- (4) Storage projects have been sometimes framed on the basis of supplies available in the worst year or a continuous series of bad years. What is the *probability* of the worst year and the worst 3 and worst 4 in 10 years?
- (5) What is the *percentage* of all years above and below the mean in each record?
- (6) What is the average fall for wet and dry years (i. e. above and below the mean respectively) and what are the respective ranges of variation?

- (7) What is the ratio of the rainfall in the driest (and wettest) year to the normal? What is the ratio for the average rainfall in the 2, 3 and 4 driest years to the normal?
- (8) Do the figures substantiate the hypothesis that the divergences from the normal are greater for stations with relatively smaller rainfall?

Similar questions have been answered by eminent authorities like Sir Alexander Binnie (1) and others for other Catchments but their approach to the problem was different.

1. Does the year to year rainfall phenomena obey any long-term or short-term cyclic law? If so, what is the most probable period of the cycle?

1. 1. Temperature, humidity, hours of sunshine and other natural phenomena are, generally, governed by a periodogram law—sometimes subject to a combination of more than one periodic variation; for example, temperature is subject to the solar year as well as the solar day cycles. Is rainfall a similar cyclic phenomenon?

1. 2. Rainfall is apparently irregular in frequency, duration and magnitude: rains are often early or late, it is either a dry year or a wet year; and so on; but, nevertheless, there is a current belief that rainfall variations repeat themselves after a period of 35 years. A successful estimate of the period T , if rainfall is cyclic, would be of great value for prediction of the normal expectation from the recorded data for any T successive years. For example, if R_1 and R_2 are the highest and lowest annual falls in a region subject to cyclic rainfall with period of cycle 35 years, rainfall would be a continuous function of time t with any year's fall very nearly expressed by

$$1/2 (R_1 + R_2) + 1/2 (R_1 - R_2) \sin \frac{2\pi t}{35} \dots \dots (i)$$

where t assumes all integral values from 1 to 35.

1. 3. Assuming precipitational variations are governed by a periodogram law expressed by (i) above, all intensities of rainfall between R_1 and R_2 would ordinarily occur with the same frequency in a long series of years. Previous recorded data for several raingauge stations, however, disprove the expectation and incidentally rule out the hypothesis of such periodicity. Nevertheless, 75 years' (1865-1940) actual recorded data of rainfall—*Kharif & Rabi* data considered separately—observed at 7 raingauge stations in the Sutlej Catchment, where analysed for periodicities to see if there is any such indication in these data.

The method adopted was the usual one of calculating the mean phase values at each stage for all probable values of T , the length of a complete cycle. Taking all the alternative integral values of $T = 2$ to $T = 37$ years, the meagre data of 75 years permitted only two repetitions of the individual phase values for calculating their means at each stage. A comparison between the 36 ranges of variation of these mean phase values (for what they are worth) seemed to indicate a period of 22 years as more likely than any of the other 35 alternative values examined. [This finding is based on the analysis of the data for *Kharif* rainfall at one raingauge station, namely Kasauli; but it is sufficiently indicative of the trends of other stations]

Values of T from 19 to 23 were next examined in greater detail, so as to produce at least one more repetition of the cycle from which to calculate the mean phase values. The ranges of variation between the mean phase values at Kasauli thus obtained were :-

T (Years) :	19.0	20.0	21.0	21.5	22.0	23.0
Range :	30.0	33.9	40.2	46.2	41.3	31.9

The peak value of 46.2 indicates a precipitation cycle, if one exists, of about $21\frac{1}{2}$ years duration. In view of the very scanty data and the marked lack of any uniformity in the four repetitions little weight can be attached to the reality of any long-term period (such as 22 years) as this could be due to purely fortuitous causes alone.

1.4. A search for lower periodicities—2 to 15 years—according to the more exact method of the variations of the correlation ratio, based uniformly on five repetitions of the cycle, did not yield more fruitful results for any of the seven stations in the Sutlej Catchment, for either the Kharif season's or the Rabi season's rainfall.

2. If rainfall is not a periodic function of time, what is the shortest period of rainfall from which the mean normal rainfall of any place can be reasonably estimated - i. e. what is the least number of years for which the mean value of annual rainfall does not vary appreciably from that over a longer period of years? 35 years?

2.1. While the investigation of periodicity must needs be postponed until more data of rainfall become available, it is important to know the minimum number of years over which the calculated average yearly rainfall at any locality remains sensibly constant.

2.2. For this purpose, partial means for all possible 35-years and 45-years continuous periods were computed in the first instance. The 75 years data (for 6 rain gauge stations in the Sutlej Catchment) provided, for each station, 41 successive mean values for 35-years periods, and 31 mean values for 45-years periods. The 75 years' average being taken as the "normal", it was used as the yardstick with which to compare the several partial means and their percentage deviations from the "normal". The values of the largest percentage differences of such partial means from the "normal" are tabulated below for each of the six rain gauge stations.

Table-1/ A-1 (i)-1.

Season	Rain gauge Station	"Normal" i. e. 75 years' mean rainfall	Standard Deviation.	Percentage coefficient of variation.	Percentage Deviation from 75 years' mean			
					of 35 years mean		of 45 years mean	
1	2	3	4	5	Lowest 6	Highest 6A	Lowest 7	Highest 7A
KHARIF	Kasauli	56.02	16.14	28.8	- 8.9	+ 2.3	- 5.5	+ 1.1
	Barauli	41.38	13.67	33.0	-10.1	+ 5.4	- 8.4	+ 2.3
	Simla	52.51	12.02	22.9	- 4.9	+ 4.5	- 5.9	+ 2.9
	Kot Khai	31.86	12.48	39.2	-18.6	+19.2	-17.9	+ 9.6
	Kot Garh	34.86	13.66	39.2	-19.3	+17.2	-18.0	+ 8.9
	Una	31.51	10.14	32.2	+ 3.2	+ 6.0	- 1.4	+ 5.6
RABI.	Kasauli	10.32	5.18	50.2	-14.8	+ 9.0	- 9.4	+ 7.0
	Barauli	9.61	4.80	49.9	-11.7	+ 8.8	-10.5	+ 4.5
	Simla	10.65	5.12	48.0	- 1.4	+13.1	- 1.5	+11.6
	Kot Khai	11.51	5.75	49.9	- 9.4	+ 9.2	- 7.1	+ 5.9
	Kot Garh	11.57	6.18	53.4	- 9.7	+ 8.0	- 7.1	+ 3.9
	Una	7.19	5.02	69.8	- 3.6	+16.3	- 4.1	+12.0

The 35-years' means do not generally deviate from the 75-years' means by more than about ± 10 percent for Kharif and ± 15 per cent for Rabi Season's rainfall. The two exceptions, Kot Khat and Kot Garh in Kharif, do not show any material improvement even by including 45 years' data for calculating the partial means. On the other hand, the very large values of the coefficients of variation (in Col.5 above) for the stations registering big percentage deviations perhaps indicate that dispersion is rather inherent in the Stations themselves. In a similar manner, the same explanation answers why a wider measure of latitude is necessary in the Rabi season's rainfall.

2.3. It was, therefore, considered more appropriate to attempt to fit frequency curves to the rainfall data with a view to obtain the extent of the probable deviations of the fall above and below the normal, their long-range frequencies, etc.

Starting with the *Kharif* rainfall at Kasauli a skew distribution of Pearson type I for the *percentage* probability of rainfall in any one year being equal to R was obtained as—

$$y = 11.68 \left(1 + \frac{R-56.36}{46.638} \right)^{4.002} \left[1 - \frac{R-56.36}{81.389} \right]^{8.051} \dots\dots (ii)$$

with *skewness* equal to 0.19. The curve is, therefore, not very far from being symmetrical with the total *probability* of rainfall exceeding the mean being 48 percent.

Other general indications are also evident from this curve, such as: (a) years of scanty rainfall occur more frequently than very wet years; and (b) the extent of the departures of rainfall in above-normal years is larger than in below-normal years.

This method being thus evidently promising, it was applied to study the *frequency* features of precipitation in *whole Catchments* rather than at individual stations, since it is the former that are more important in irrigation engineering.

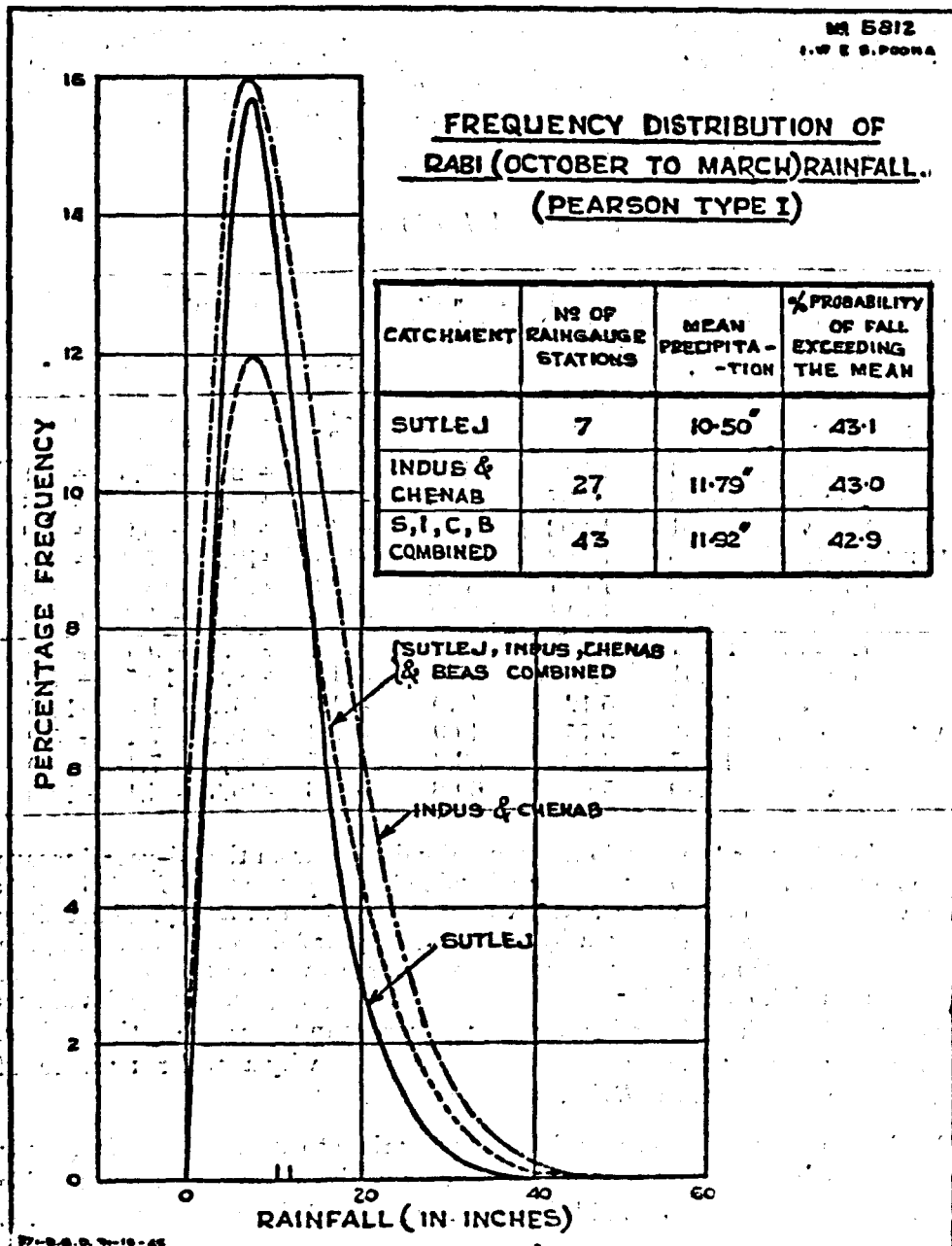


FIG. 2/A-1(i)-1

2.4. The catchments considered in this study cover the four principal rivers of the Punjab, viz. the Indus (15 stations), Sutlej (7 stations), Chenab (12 stations) and the Beas (10 stations). The units of frequency considered are *station-years*, the total rainfall recorded at any raingauge station during each season being taken to constitute one *station-year*. The names of the raingauge stations in each catchment and the number of years for which the records were available for each station are shown in Appendix I. The data for the total number of ^{Appendix I} station-years in each catchment were then analysed for the *probability* that ^{A-1(i)-1} the average rainfall in the catchment does not exceed any specified value during a season. This was done by the method of fitting appropriate Pearson type frequency curves to the observed uneven distributions. The curves turned out invariably to be of Type I with finite ends on either sides, their equations being of the form —

$$= y_0 \left(1 + \frac{R - \text{mode}}{a_1} \right)^{m_1} \left(1 - \frac{R - \text{mode}}{a_2} \right)^{m_2} dR \dots \dots \text{(iii)} \quad \begin{array}{l} \text{Figs. 1 \& 2} \\ \text{A-1(i)-1} \end{array}$$

The values of the *constants* for each catchment are as set out in Table 2.

Table 2
A - (i) - 1

The positive signs of *skewness—skewness* is a relative measure of the deviation of the *Modal* (i. e. the most frequently occurring) rainfall (Col 3) from the *mean fall* (Col. 9) in terms of the standard deviation—all show that years of deficient rainfall are more frequent than wetter years. The large values of a_2 indicate that the relative frequency of years with rainfall in excess the of *modal* tails off very gradually to the limiting value of the rainfall. These and other features (e. g. the *probability* of years with varying amounts of annual rainfall) are evident from the form of the frequency curves for the various catchments. The high degree of contact on the right implies the decreasingly small *probabilities* of very wet years, which, however, cannot be ruled out as altogether *improbable*.

2.5. An auxiliary set of curves are presented in figure 3 for the *probability* that ^{Fig. 3} the total rainfall in a season will not be less than any specified amount. From ^{A-1(i)-1} the curves, which are obtained by plotting the ratio of the definite integrals

$$\int_R^\infty \text{Prob}(R) dR \div \int_0^\infty \text{Prob}(R) dR, \text{ against } R, \text{ the probability that the average}$$

rainfall will not be less of than a prescribed figure can be directly read off; for example, the *probability* of total rainfall in any *Kharif* season being not less than 20", in the Sutlej catchment, is 85.8 percent.

3. What is the extent of the year to year variations from the normal? Do these variations fall within well defined limits?

3.1. It has already been seen, *vide* para 2.2 above, that the 35-year average provides a reasonably good estimate of long term average for any raingauge station; from the point of actual fall to be expected during any season, however, the *modal* (or the most-frequently occurring value) is a better estimate. On the other hand, for all the eight cases considered in Table 2 the modal figures are seen to be less than the mean. Though the fall most often to be expected centres somewhere round the *mode*, any rainfall value lying between

$$\text{mode} - a_1 \text{ and } \text{mode} + a_2$$

may be *equally* expected to occur in an individual year with its corresponding *probability* expressed by the right hand side of equation (iii) with the appropriate value substituted for R. These *probabilities* have a very steep rise from 0 at $R = \text{mode} - a_1$ to y_0 at $R = \text{mode}$, and a slow gradual fall again to 0 at $R = \text{mode} + a_2$, depending on the value of a_2 which is relatively large. Nevertheless, there is no year to year continuity in the values of R nor any certainty about the *probable* values in specific years.

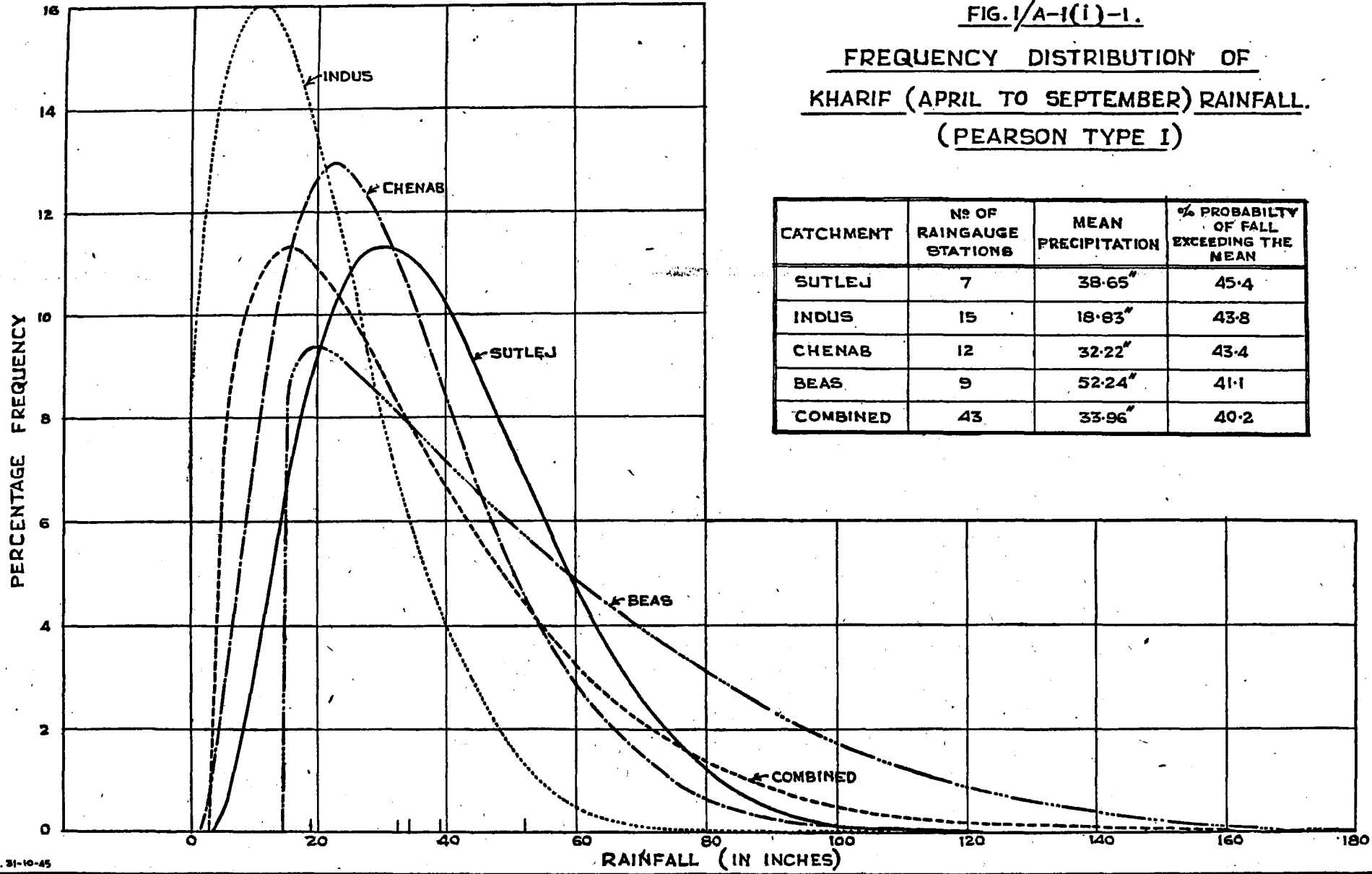
3. 2. All that it is possible to say is that the variations of a season's average rainfall in the catchments examined are ordinarily confined between two well-defined but widely different minimum and maximum limiting values.

4. What is the probability of any one year's total precipitation being in defect of the precipitation in the worst year on record? What is the probability of 3 or 4 such years occurring within a specified period of say, 10 years?

Table 2 / A - 1 (i) - 1.

Catchment	y_0	MODE (inches)	a_1	a_2	m_1	m_2	SKEW- NESS.	Mean Precipitation (inches)	PERCENTAGE PROBABILITY of fall exceeding mean	Mean fall in years (inches)	
										Below Normal	Above Normal
1	2	3	4	5	6	7	8	9	10	11	12
KHARIF											
Sutlej	11.33	30.74	26.872	107.979	1.645	6.608	0.45	38.65	45.4	25.76	54.15
Indus	16.10	10.76	14.065	97.641	1.051	7.296	0.61	18.83	43.8	9.49	30.81
Chenab	12.92	23.05	21.785	254.835	1.845	21.584	0.54	32.22	43.4	20.28	47.79
Beas	9.35	18.72	4.126	185.624	.074	3.340	1.12	52.24	41.1	31.78	81.50
All four combined	11.28	14.64	11.733	427.031	.521	18.976	0.83	33.96	40.2	18.53	56.92
RABI.											
Sutlej	15.61	7.44	7.668	221.470	2.267	65.491	0.53	10.50	43.1	7.19	14.87
Indus & Chenab	16.04	7.36	10.134	156.757	1.886	29.165	0.55	11.79	43.0	6.18	19.23
All four combined	11.88	8.01	10.106	185.614	2.212	40.626	0.52	11.92	42.9	6.70	18.88

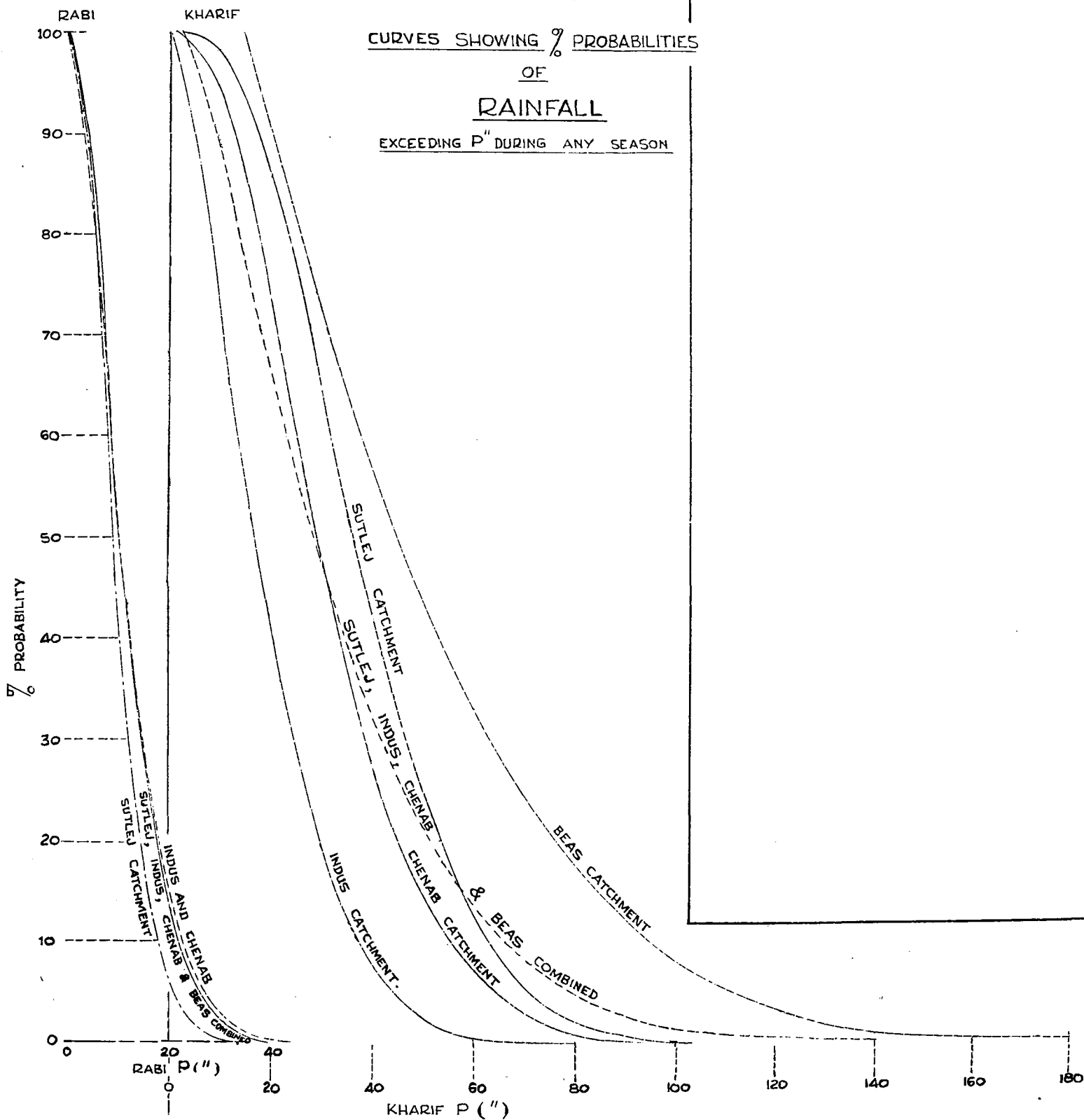
FIG. 1/A-1(i)-1.
FREQUENCY DISTRIBUTION OF
KHARIF (APRIL TO SEPTEMBER) RAINFALL.
(PEARSON TYPE I)



CATCHMENT	NO OF RAINGAUGE STATIONS	MEAN PRECIPITATION	% PROBABILITY OF FALL EXCEEDING THE MEAN
SUTLEJ	7	38.65"	45.4
INDUS	15	18.83"	43.8
CHENAB	12	32.22"	43.4
BEAS	9	52.24"	41.1
COMBINED	43	33.96"	40.2

FIG. 3/A-1(i)-1

CURVES SHOWING % PROBABILITIES
OF
RAINFALL
EXCEEDING P" DURING ANY SEASON



4.1. Figure 3 is very useful for estimating the probability that the average rainfall of the catchment during a season shall not be less than any dangerously low value, say, x'' . The probability that the rainfall will actually be in defect of x'' will be simply the complement probability, or the difference from 100 percent of the probability that the rainfall shall not be less than x'' . Thus, for instance, if 15" fall is assumed a low Kharif average in the Sutlej Catchment, the probability of the fall not exceeding 15" during any one season can be directly estimated as 6.42 percent.

But if it is desired to estimate the probability that there shall be 2, 3 or 4 such bad years with falls less than 15", say, within a specified period of 10 years, these could be further deduced by a simple application of the binomial law of probabilities. According to this law, the percentage probability of there being r bad years out of 10 will be equal to $100 (10 C_r p^r q^{10-r})$, the coefficient of p^r in the expansion of the binomial expression $100 (p + q)^{10}$ with the value of $p (= 1 - q) = .0642$. Putting $r = 0, 1, 2, 3$ or 4 so that there shall be 0, 1, 2, 3 or 4 years with rainfalls less than 15" within our specified period of 10 years, the probabilities respectively are :

	r	0	1	2	3	4
percentage probability		51.5	35.3	10.9	2.0	0.2
Multiplier for consecutive r bad years		$\frac{9}{45}$	$\frac{8}{120}$	$\frac{7}{210}$

The corresponding probabilities for a series of r ($r \geq 2$) consecutive bad years will be obtained by multiplying the above probabilities by their respective multipliers (also given above). Thus, the probability for as many as even 3 years out of 10 to record average falls below 15" remains very small indeed, being only 2 percent and for 3 consecutive bad years being $2 \times \frac{8}{120} = .133$; therefore, such contingencies apparently, need not be considered while designing large storage projects in these catchments.

4.3. By a similar application of the binomial theorem, the probabilities of the fall being below normal on r occasions out of 10 are also derivable from the successive terms of the same binomial with $p = 1 - q$ put equal to .5460 for Kharif and .5695 for Rabi. These probabilities, along with their expected annual average falls, are shown in table 3 below :-

Table 3/A-1 (i)-1.

Number of years with rainfall.		PERCENTAGE PROBABILITY		Multiplier for r below normal years to come consecutively.	Expected annual fall under the contingencies specified (in inches).	
below normal	above normal	of rainfall in r years out of 10 being below normal.			Kharif	Rabi
r	$10-r$	Kharif	Rabi		Kharif	Rabi
1	2	3	4	5	6	7
0	10	.037	.022		54.15	14.87
1	9	.45	.289		51.31	14.10
2	8	2.42	1.72	1/5	48.47	13.33
3	7	7.77	6.07	1/15	45.63	12.57
4	6	16.34	14.06	1/30	42.79	11.80
5	5	23.59	22.32	1/42	39.95	11.03
6	4	23.64	24.61	1/42	37.11	10.26
7	3	16.24	18.60	1/30	34.27	9.49
8	2	7.33	9.23	1/15	31.44	8.73
9	1	1.96	2.71	1/6	28.60	7.96
10	0	.24	.36	1	25.76	7.19

Columns 3 & 4 which contain the percentage probability values for a season's precipitation to be below normal only in 0, 1, 2, 10 years (and above normal in the remaining years of continuous period of 10 years) show that the

probability values attain their maximum for $r = 6$. This brings out that it is *more likely* for precipitation to be below normal only in six years out of every ten than in any alternative combination.

5. *What is the percentage of all years above and below the mean in each record?*

For the eight cases of frequency curves fitted to the data in this paper, the proportional frequency of years with rainfall exceeding the normal—in other words, the probability of a season's rainfall exceeding the normal—can be estimated by taking the ratio of the area on the right of the mean line to the whole area enclosed under the frequency curves. These probabilities, which are already given in column 10 of Table 2, are all seen to be *below 50 percent*. Similar figures for the *percentage* of years with above normal rainfall have been computed for each rainguage station on record in the 4 catchments considered here (vide column 4 in the Appendix). These figures also generally work out to be smaller than 50 percent with a few isolated stations recording higher percentages.

6. *What is the average fall for dry and wet years (i. e. those below and above the normal) and what are the respective ranges of variation?*

The average rainfall for all dry or below-normal years and the range of individual year's variations during such years have been computed for each rainguage station on record (vide columns 5 & 6 in the Appendix). Similar figures for wet or above-normal are shown in columns 7 & 8.

7. *What is the ratio of the rainfall in the driest (and wettest) year to the normal? What is the ratio for the average rainfall in 2, 3 and 4 driest years to the normal?*

The *percentage* ratios of each station's wettest year's rainfall as well as driest year's rainfall to its normal fall have been given in columns 9 and 10 respectively. The *percentage* ratios of the average falls in 2, 3 and 4 driest years on record to the normal fall are, similarly, tabulated for each station in columns 11, 12 & 13 respectively. An important rainfall feature shown by figures in Column 9 is that the variation of the maximum rainfall from the mean rainfall is as high as 690 percent; whereas in the driest year the ratio of $\frac{\text{minimum rainfall}}{\text{mean rainfall}}$ can, at the most, lower itself only to zero.

This shows that variations of rainfall during wet years can be exceedingly larger than similar variations during lean years, which fact is also evident from the eight elongated positive skew-type frequency curves fitted to the catchments' data.

8. *Do figures substantiate the hypothesis that the divergence from normal are greater for stations with relatively smaller rainfall?*

The *percentage* coefficient of variation which is defined as $100 \times \frac{\text{Best average measure of dispersion}}{\text{Mean rainfall}} = 100 \times \frac{\text{Standard Deviation}}{\text{Mean rainfall}}$ suggests itself as the most appropriate means of testing this hypothesis. These coefficients were, accordingly, calculated for the several stations (vide column 14 in the Appendix) and plotted against the mean rainfall of each station (Figure 4). The evidence is only qualitative but substantiates the hypothesis.

Fig
4/A-1(i)-1

9. *Resumé.*

A *questionnaire* of interest in the study of the year to year variations of rainfall in four large catchments of the Upper Indus Basin has been attempted to be answered in the body of the Note.

For simplicity, all rainguage stations have been given equal weightage in the analysis.

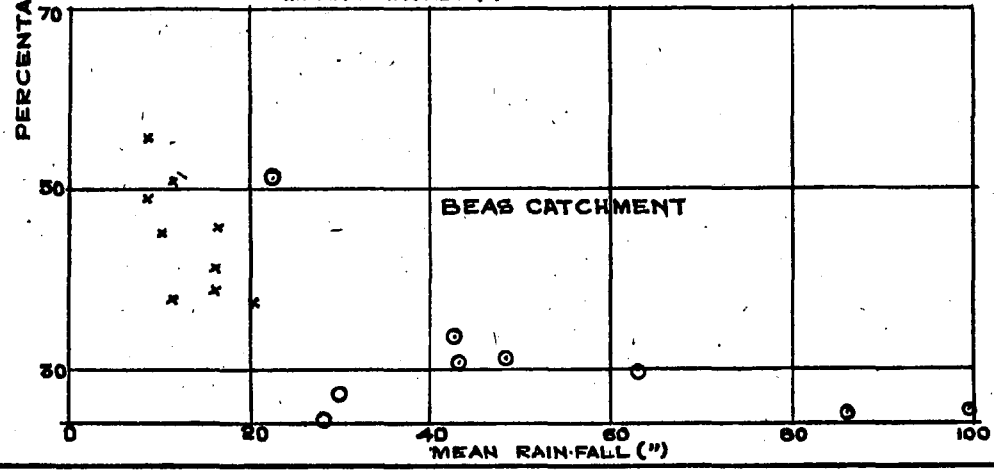
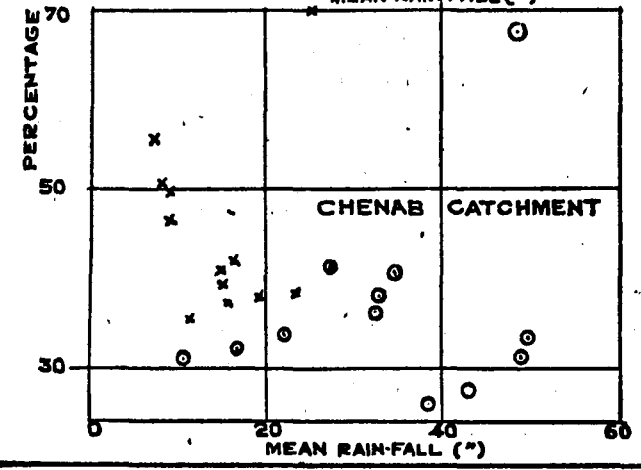
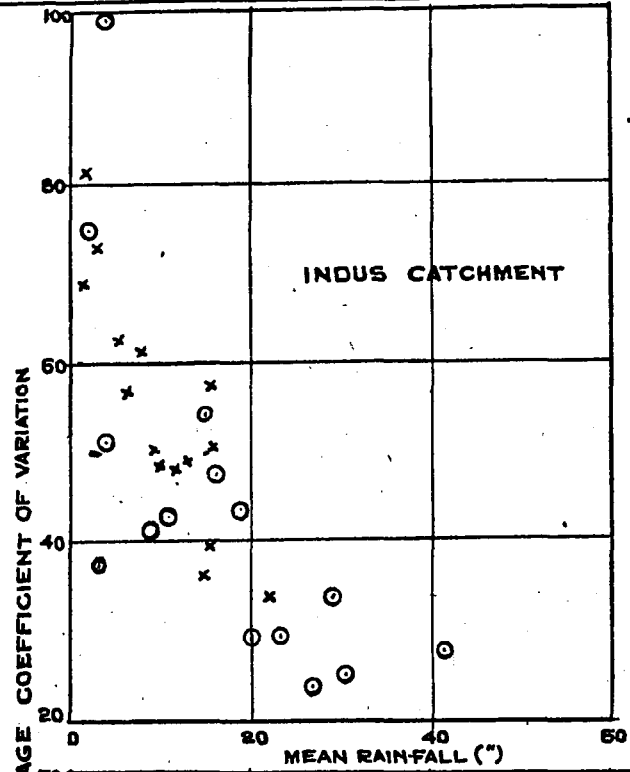
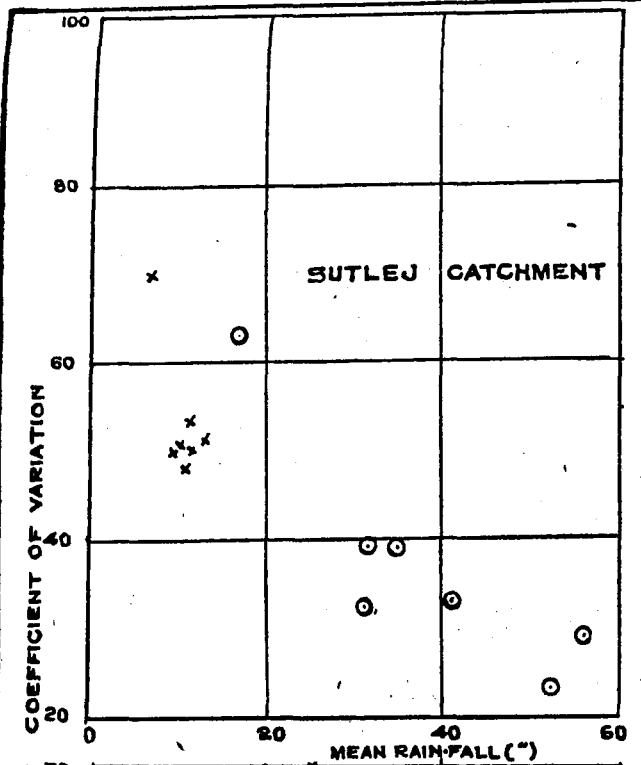
The distribution laws governing the variations, for both the *kharif* as well as the *rabi* seasons' rainfall, in these catchments, generally subscribe to *type I* of the Pearsonian system of frequency curves, the skewness being positive. By integrating these curves, probability curves for directly reading off the probability of a season's rainfall not falling short of any specified measure—have also been prepared. Together, these curves provide the answers to the *Questionnaire*—on a statistical basis.

FIG. 4/A-1(i)-1.

DISPERSION OF RAINFALL STUDY
PERCENTAGE COEFFICIENT OF VARIATION
AGAINST MEAN RAINFALL IN 4 PUNJAB RIVER
CATCHMENTS

$$\% \text{ COEFFICIENT OF VARIATION} = \frac{100 \times \sigma}{\text{MEAN RAINFALL}}$$

- KHARIF RAIN-FALL.
- × RABI RAIN-FALL.



Appendix I/A-1 (i)-1

Sheet 1

Analysis of rainfall data of Sutlej Catchment : Kharif Season.

Stations	No. of years Date avail- able for	Normal or mean rainfall in inchea	Percentage of yeass above normal	Below normal		Above normal		Percentage fall in wettest year.	Percentage average fall of the driest.				COEFF. of Variation.
				Mean fall	Range	Mean fall	Range		1 year	2 years.	3 years	4 years	
1	2	3	4	5	6	7	8	9	10	11	12	13	14
Kasauli	75	56.02	46.6	43.72	33.57	70.09	44.50	179.9	38.2	41.3	43.6	46.6	28.8
Barauli	75	41.38	44.0	32.40	31.44	52.81	43.13	204.6	21.3	31.2	40.1	44.8	33.0
Simla	75	52.51	48.0	43.40	21.90	62.38	38.77	174.3	56.4	57.1	59.7	61.9	22.9
Kot Khai	75	31.86	37.3	24.21	16.94	44.70	38.99	223.5	46.6	47.0	48.4	49.4	39.2
Kot Garh	75	34.86	38.6	26.69	18.28	47.81	53.00	252.5	47.5	50.9	53.0	54.1	39.2
Kilbe	58	17.19	32.8	11.28	13.32	29.31	36.25	315.7	20.9	22.9	27.4	30.8	62.8
Una	75	31.51	44.0	24.02	17.70	41.05	25.35	181.5	42.2	43.9	45.7	47.3	32.2

Sutlej Catchment : Rabi

Kasauli	75	10.32	41.3	6.71	7.29	15.45	13.56	232.9	29.1	31.5	32.8	34.0	50.2
Barauli	75	9.61	45.3	6.03	8.75	13.93	13.89	245.3	4.2	6.2	12.8	19.2	49.9
Simla	75	10.65	41.3	7.11	9.15	15.69	13.03	228.6	14.1	17.4	19.1	20.8	48.0
Kot Khai	75	11.51	44.0	7.35	10.78	16.80	15.80	238.1	5.0	7.7	14.1	18.5	49.9
Kot Garh	75	11.57	49.3	6.82	11.50	16.46	20.86	281.6	0.0	6.7	9.1	10.7	53.4
Kilbe	55	13.15	36.4	9.13	10.35	20.20	16.42	229.1	18.8	18.9	19.9	22.5	51.2
Una	75	7.19	38.6	4.35	6.88	11.70	29.80	515.9	1.4	9.8	14.4	16.8	69.8

RAINFALL PROBABILITY

Appendix I/A-1 (i)-1

Analysis of rainfall data of Indus Catchment : Kharif Season.

Sheet 2

Stations.	No. of years Data available for	Normal of mean rainfall in inches	Percentage of years above normal	Below normal		Above normal		Percentage falls in wettest year	Percentage average fall of the driest.				COEFF. of Variation
				Mean fall	Range	Mean fall	Range		1 year	2 years	3 years	4 years	
1	2	3	4	5	6	7	8	9	10	11	12	13	14
Swabi	69	15.43	50.7	8.80	14.90	21.88	18.83	223.4	0.0	0.0	1.5	5.0	54.0
Abbottabad	81	31.00	48.1	25.55	12.72	36.87	29.01	193.9	58.1	59.4	60.0	61.5	24.5
Kohat	79	11.30	44.3	7.91	8.16	15.56	10.43	192.5	27.4	27.7	32.8	36.8	42.2
Mansehra	75	23.64	44.0	18.86	13.86	29.73	20.34	197.2	41.0	41.7	44.3	47.0	29.1
Haripur	75	20.27	46.7	15.93	12.13	25.23	16.77	183.6	39.0	45.2	48.0	50.1	29.1
Cherat	42	16.49	38.1	11.50	11.51	24.60	19.55	231.9	30.0	35.4	40.2	40.5	47.3
Melakand	38	19.17	47.4	13.08	10.94	25.93	22.19	217.9	38.3	38.6	39.4	40.3	43.0
Oghi	39	26.95	53.8	21.58	19.08	31.56	9.24	134.8	28.5	43.1	51.6	55.9	23.5
Leh	63	2.11	41.3	1.27	1.70	3.32	9.18	538.3	18.5	18.9	19.6	20.7	74.4
Kargil	46	4.10	43.5	2.60	3.46	6.05	5.94	248.0	15.1	20.4	25.9	30.7	51.0
Dras	46	9.11	43.5	6.36	5.61	12.67	8.42	195.1	38.2	38.5	42.9	45.6	40.9
Skardu	49	3.67	30.6	2.28	3.13	6.83	21.50	690.3	14.2	14.6	17.3	18.8	99.2
Gilgit	48	3.39	47.9	2.45	2.07	4.41	3.41	203.4	37.2	37.8	41.4	43.8	37.2
Murree	73	41.77	42.5	33.86	23.80	52.48	34.34	182.6	43.0	48.9	51.1	52.4	27.4
Kabute	59	29.49	49.2	21.70	16.33	37.56	28.95	198.4	44.2	47.4	50.2	52.3	33.6

Indus Catchment : Rabi.

Swabi	69	8.06	49.3	4.11	7.90	12.13	16.67	310.1	00.0	0.0	0.0	4.0	61.1
Abbottabad	81	15.81	43.2	11.40	10.77	21.61	16.44	204.9	30.4	31.0	32.7	34.1	39.4
Kohat	78	6.33	44.9	3.93	5.60	9.23	13.90	322.2	7.9	12.6	14.8	16.9	56.5
Mansehra	74	13.12	41.9	9.19	10.00	18.56	30.83	338.8	23.6	26.7	29.5	32.0	48.7
Haripur	74	9.84	45.9	6.16	7.18	14.17	13.71	240.6	26.4	26.9	28.5	29.3	50.2
Cherat	40	11.58	50.0	7.10	10.11	16.05	12.10	205.3	6.2	17.4	22.8	25.5	47.9
Melkand	37	14.99	54.1	10.19	9.76	19.06	11.47	182.1	26.9	30.6	36.6	41.3	36.0
Oghi	38	21.96	44.7	16.76	12.34	28.38	19.54	191.0	40.9	43.4	45.7	47.4	33.6
Leh	62	1.34	35.5	0.82	1.23	2.29	3.22	340.0	8.2	9.3	9.7	17.6	68.3
Kargil	46	5.87	39.1	3.80	5.07	9.08	15.78	371.9	9.9	18.5	23.0	26.5	62.3
Dras	46	15.99	39.1	11.07	10.05	23.65	23.59	255.4	34.9	36.6	37.9	39.4	50.1
Skardu	48	3.30	39.6	2.04	2.94	5.23	12.52	482.9	7.3	14.4	17.4	19.6	72.7
Gilgit	47	1.82	34.0	1.02	1.62	3.39	5.17	384.0	8.2	11.5	12.6	14.8	81.1
Murree	72	16.04	45.8	9.92	14.82	23.29	32.69	303.9	6.3	9.5	12.6	14.4	57.6
Kabute	58	9.95	48.3	6.17	8.99	14.00	14.18	246.3	7.9	14.7	18.9	23.4	48.4

Stations	No. of years Data available for	Normal of mean rainfall in inches	Percentage of years above normal	Below normal		Above normal		Percentage falls in wettest year	Percentage average fall of the				COEFF of Variation
				Main fall	Range	Main fall	Range		1 year	2 years	3 years	4 years	
1	2	3	4	5	6	7	8	9	10	11	12	13	14

Chenab Catchment : Kharif

Jammu	50	32.97	42.0	25.01	17.41	43.95	37.93	216.2	45.3	48.6	51.0	53.1	36.1
Kishtawar	50	16.91	46.0	12.85	7.53	21.67	18.30	210.9	54.3	55.5	56.0	56.8	32.2
Ramban	50	22.23	44.0	16.90	13.46	29.02	17.12	178.4	39.3	41.3	44.3	46.1	33.7
Akhumur	41	34.70	46.3	25.33	18.08	45.54	51.93	249.7	45.9	46.8	48.3	49.3	40.4
Rampur Rajani	41	39.46	43.9	32.21	20.12	48.73	23.96	162.9	47.8	48.8	53.2	55.4	25.8
Ramnagar	40	49.75	42.5	39.21	21.82	64.00	50.98	202.7	55.2	56.2	59.2	62.0	33.3
Gulabgarh	37	48.60	29.7	32.83	30.36	85.87	134.31	380.6	36.1	37.7	41.8	44.2	67.4
Sambah	40	32.94	42.5	24.93	16.22	43.78	39.27	220.9	48.2	48.5	51.0	52.9	38.1
Riasi	40	49.20	45.0	37.76	25.26	63.17	41.40	185.1	47.1	51.3	54.1	57.6	31.6
Udampur	40	42.71	45.0	33.98	22.80	53.38	28.81	167.6	46.3	52.1	54.2	58.1	27.7
Sri Ranbir Singh Pore	40	27.50	42.5	19.93	13.83	37.74	30.45	215.3	48.8	49.8	50.1	50.4	41.0
Kailang	53	11.04	39.6	8.90	7.44	14.29	10.68	197.5	32.5	41.5	45.7	49.7	31.1

Chenab Catchment : Rabi

Jammu	49	9.05	51.0	5.64	6.63	12.33	12.64	240.4	21.1	21.2	21.4	23.1	49.4
Kishtawar	49	19.31	51.0	13.31	12.80	25.07	19.35	200.8	28.5	31.6	33.7	36.7	37.8
Ramban	49	23.71	55.1	15.87	17.88	30.11	23.14	197.8	20.3	24.9	28.3	31.0	38.1
Akhumur	40	9.35	42.5	6.27	7.87	13.53	8.40	195.1	13.2	18.1	21.7	28.4	46.2
Rampur Rajani	40	16.00	52.5	11.20	12.92	20.35	15.07	195.0	17.9	19.3	26.2	30.4	37.1
Ramnagar	39	16.51	43.6	11.90	14.09	22.49	21.92	238.4	12.4	28.8	35.5	39.2	41.8
Gulabgarh	37	25.43	35.1	16.87	19.72	41.23	74.29	393.2	21.2	23.9	25.4	26.5	69.8
Sambah	40	8.35	47.5	5.21	7.15	11.82	11.52	241.4	11.0	20.8	26.7	29.8	50.4
Riasi	40	15.21	45.0	11.24	12.60	20.05	18.23	223.3	16.8	21.0	26.6	30.8	39.2
Udampur	39	15.14	48.7	10.98	14.90	19.51	18.21	222.0	0.0	16.5	24.0	31.5	40.5
Sri Ranbir Singh Pore	40	7.50	42.5	4.85	6.77	11.09	15.75	312.0	6.1	14.4	18.8	22.0	55.5
Kailang	53	11.31	50.9	8.00	6.58	14.50	10.55	193.9	40.1	42.0	44.0	46.3	35.4

Appendix I / A-1 (i) -1

Analysis of rainfall data of Beas Catchment : Kharif Season.

Sheet 4

Stations	No of years Data available for	Normal of mean rainfall in inches.	Percentage of years above normal	Below normal		Above normal		Percentage fall in wettest year.	Percentage average fall of the driest				COEFF of Variation.
				Mean fall	Range	Mean fall	Range		1 Year	2 Years	3 Years	4 Years	
Banjar	73	29.93	48.0	24.01	24.40	36.35	20.45	168.7	18.4	23.1	32.1	39.1	27.0
Kulu	73	22.57	41.1	16.96	18.06	30.62	74.12	429.3	18.2	29.7	34.4	38.7	51.2
Nagar	57	38.08	50.9	22.36	12.14	33.59	16.14	158.0	56.6	58.1	58.7	59.9	24.1
Palampur	73	85.95	48.0	69.24	39.32	104.10	48.78	157.6	53.5	53.5	53.8	54.4	25.5
Hamirpur	73	42.66	45.2	32.54	27.78	54.93	43.80	202.8	34.7	38.6	42.2	44.2	33.7
Dera	73	43.38	41.1	34.06	25.73	56.74	33.55	182.7	40.6	40.6	43.6	46.4	30.5
Kangra	73	62.98	48.0	49.72	46.31	77.37	71.77	214.5	24.3	36.4	42.6	45.6	29.6
Dharamsala	74	99.69	47.3	81.20	53.46	120.29	80.49	181.5	45.7	46.0	46.7	49.9	25.9
Nurpur	73	48.65	50.7	36.49	24.73	60.47	47.59	198.9	48.2	49.5	49.9	50.3	31.1

Beas Catchment : Rabi.

Banjar	72	11.57	54.2	7.74	8.33	15.09	13.29	214.1	25.6	29.9	32.7	36.3	37.7
Kulu	72	16.29	55.6	10.40	10.95	21.01	14.60	190.3	31.9	32.8	33.6	35.0	38.6
Nagar	57	20.55	52.6	13.74	12.16	26.67	18.64	191.3	37.7	40.1	41.2	42.9	37.8
Palampur	72	16.47	47.2	10.82	11.78	22.78	24.94	252.0	26.7	29.4	30.6	31.6	45.7
Hamirpur	72	8.74	43.1	5.84	8.04	12.57	15.00	273.5	4.6	14.7	19.6	23.1	48.4
Dera	72	8.81	44.4	5.47	8.01	12.99	19.23	318.9	8.9	9.5	13.6	16.5	55.7
Kangra	72	10.31	43.1	7.03	6.64	14.63	12.14	219.5	34.0	34.8	35.6	37.0	44.7
Dharamsala	74	16.38	47.3	11.40	12.76	21.92	15.91	197.3	22.0	27.1	29.7	31.7	40.7
Nurpur	72	11.57	40.3	7.85	9.88	17.09	17.10	249.7	14.5	20.7	22.9	24.4	50.5

A-HYDROLOGY 1-RAINFALL AND RUNOFF.

(ii) MAXIMUM RUN-OFF DISCHARGE.

A-1 (ii)-1. Maximum Run-off formulae and their applications.

In Bombay T. P. No. 30, two formulas were derived for

- (a) Fan catchments of general application;
- and (b) Valley catchments (for particular conditions frequently found in the Bombay Presidency).

For (a) $Q = 7000 \sqrt{A}$

where A is area of fan-catchment in Sq. miles.

Q = natural flood run-off in cusecs from catchment.

and for (b) $Q = 7000 \sqrt{A} - 240 (A-100)$.

This formula is applicable for A = 100 to 600 Sq. miles. Mr. Gutmann in Engineering News-Record of October 1946 pointed out that the fan-catchment formula was suitable for large catchments (excluding the extraordinary Texas data) and that it was not suitable for small catchments. So the original

formula was modified to $q = \frac{7000}{\sqrt{A+4}}$, where q is the maximum run-off in

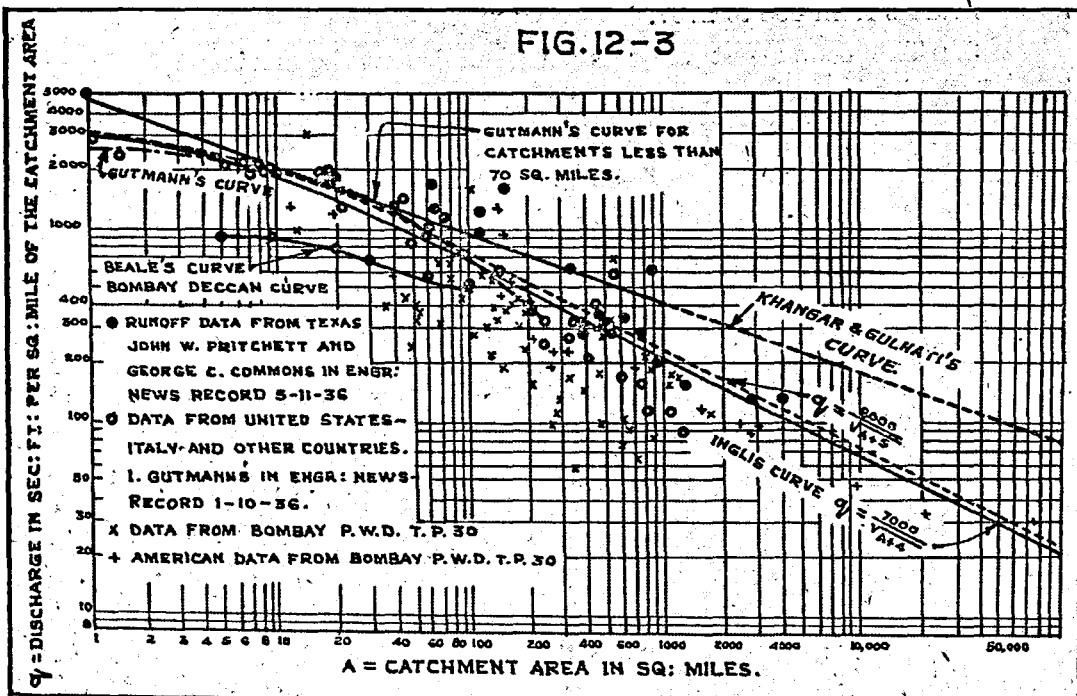
cusecs per Sq. mile of the catchment area, so that it would be applicable to small as well as large catchments. This gives the discharge at the rate of 3500 cusecs per Sq. mile for an infinitely small catchment—equivalent to

$5\frac{1}{2}$ " maximum run-off per hour. Though a curve $q = \frac{8000}{\sqrt{A+5}}$ fits the data

better than $q = \frac{7000}{\sqrt{A+4}}$, experience shows that maximum floods are nearly

always over-estimated by more than 10 percent and hence the latter formula can be stated to be more correct, vide figure 1. Mr. Beale's curve is also

Fig. 1.
A-1 (ii)-1



(1) Bombay T. P. S. 30 & 53.

A. R. (T).s	1940-41	page	6.
	1942-43	"	34.
	1943	"	44 & 67-69.
	1944	"	68-69.

shown in this figure, as Beale's formula is generally used in the Deccan. The explanation of the difference between Beale's curve and the fan-catchment curve is that Beale based his curve on floods which had passed without damaging existing canal culverts—in other words, his curve took the absorption capacity of the culverts into account. Flood absorption is considerable in the case of the Deccan Canal culverts; because when water is headed up, the upstream canal bank impounds the flood and the flood absorptive capacity increases inversely as the size of the catchment.

Fig. 2. 2. Three Empirical 'Maximum Run off' formulas and White's curve
A-1 (ii)-1 (Bombay) shown in Fig. 2 have been used to a large extent in India.

Dickens' formula (Bengal)

$$Q_{\max} = 825 A^{3/4} \quad \dots \quad (1)$$

Ryves' formula (Madras)

$$Q_{\max} = C A^{2/3} \quad \dots \quad (2)$$

Fan-catchment formula

$$Q_{\max} = \sqrt{\frac{7000 A}{A + 4}} \quad \dots \quad (3)$$

C in Ryves' formula = 450 for area within 50 miles of the sea; 560 from 50 to 100 miles, & 675 for limited areas near hills.

Fig. 3.
A-1 (ii)-1

These curves are shown in figure 3.

Ryves' figures were meant for designing floodways rather than predicting the maximum-ever floods and, consequently, were lower. Subsequently, however, various engineers have adopted higher coefficients both in Dickens' and Ryves' formulas: in Dickens' formula coefficients up to 1600 have been used or suggested; while Mr. Rhind gives a statement of 13 rivers in Southern India⁽¹⁾ which shows that the value of C in Ryves' formula varies from 308 to 2707.

3. The commonest error is the belief that the maximum floods are greater in area of high rainfall than in area of low rainfall and, consequently, the maximum run-off formulas give too high discharges for areas of low rainfall. Actually, maximum floods are *more frequent* in areas of high rainfall; but if anything, they are *greater* in areas of lower rainfall. The fan-catchment formula, gives extraordinary floods in India. It may be interesting to note that in the case of Lalpuri Tank in Rajkot State and the Erinpura Reservoir (Jawai Catchment in Jodhpur State), the maximum flood occurred in 1944 and 1941 respectively—vide points 83 & 87 in Figure 4. It was found that the ratio of actual maximum run-off to maximum run-off from the fan-catchment formula was 0.98 & 1.04 respectively. Sir M. Visveshwar ayya who was associated with these two cases—has agreed to the fan-catchment formula being used for predicting maximum flood run-offs in designing waste weirs. In many of these cases shown in figure 4, the flood has been, literally, unprecedented. For instance, the flood in the Khari River, Ajmer-Merwara, which occurred on 30th July 1943, was such a case: not even in the folk-lore of this area is there any suggestions of such a flood in the past.

Fig. 4.
A-1 (ii)-1

Fig. 4.
A-1 (ii)-1

4. An interesting note is given in para 17 'Analysis of catchment characteristics' of 1942-43 Annual Report, page 34, in which it is shown that

- (1) Shape of catchment is an important factor in the correct estimation of a maximum flood discharge and its duration.

(1) Buckley's "Irrigation Pocket Book"—Page 330 (1928 edition).

FIG. 2/A-1(ii)-1.
RELATION OF
RUN-OFF : CATCHMENT AREA
ACCORDING TO VARIOUS FORMULAS

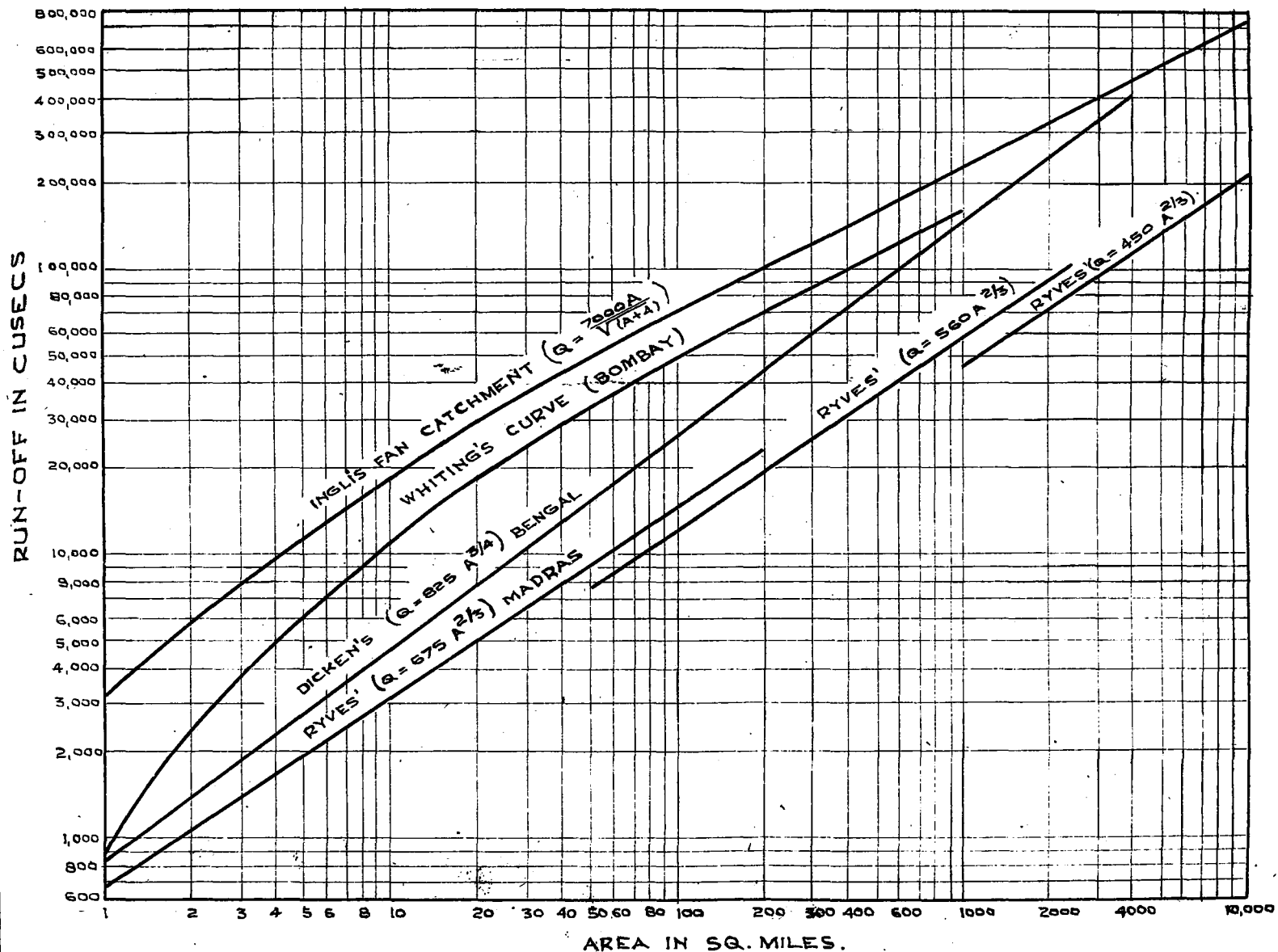
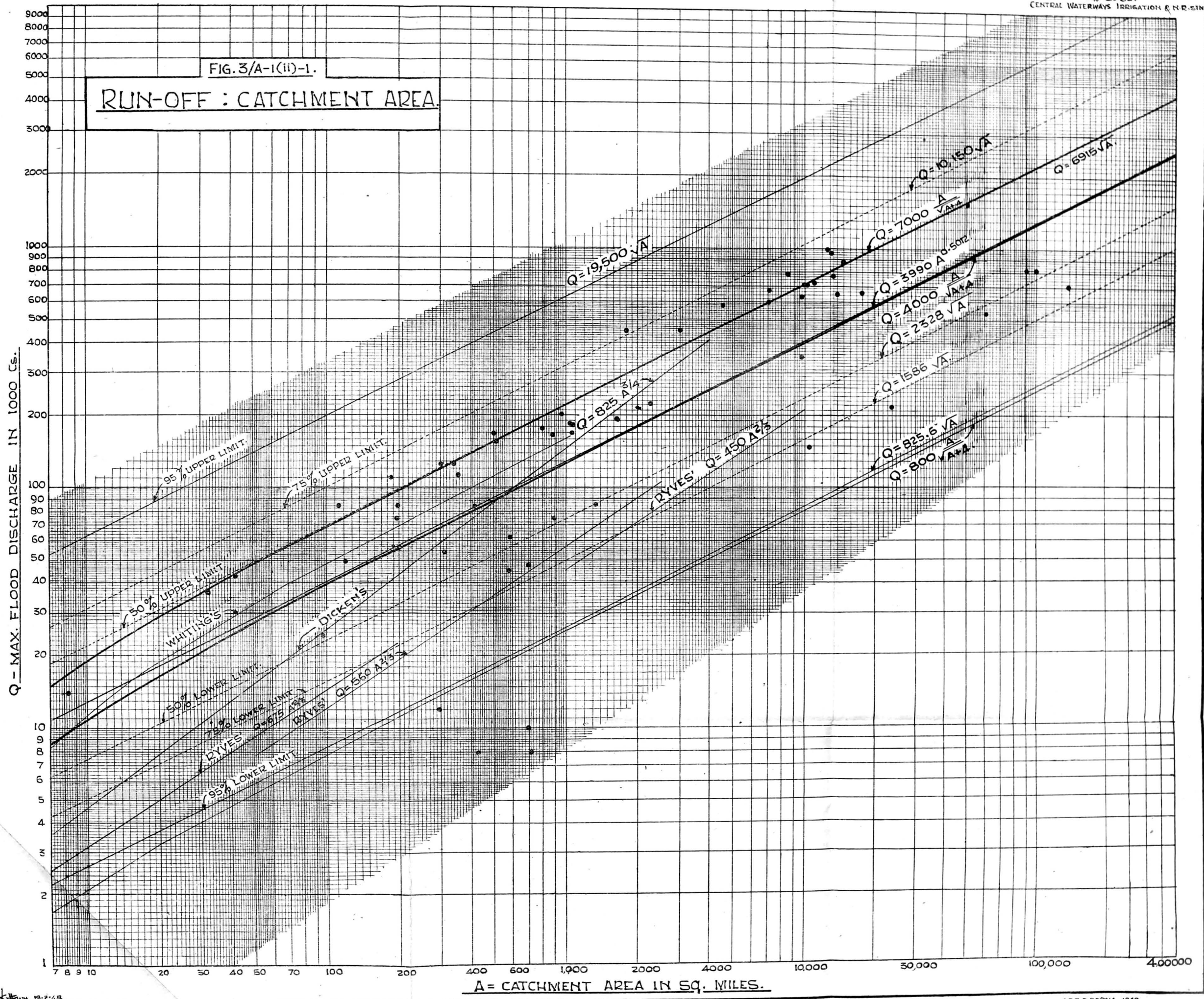


FIG. 3/A-1(ii)-1.

RUN-OFF : CATCHMENT AREA.



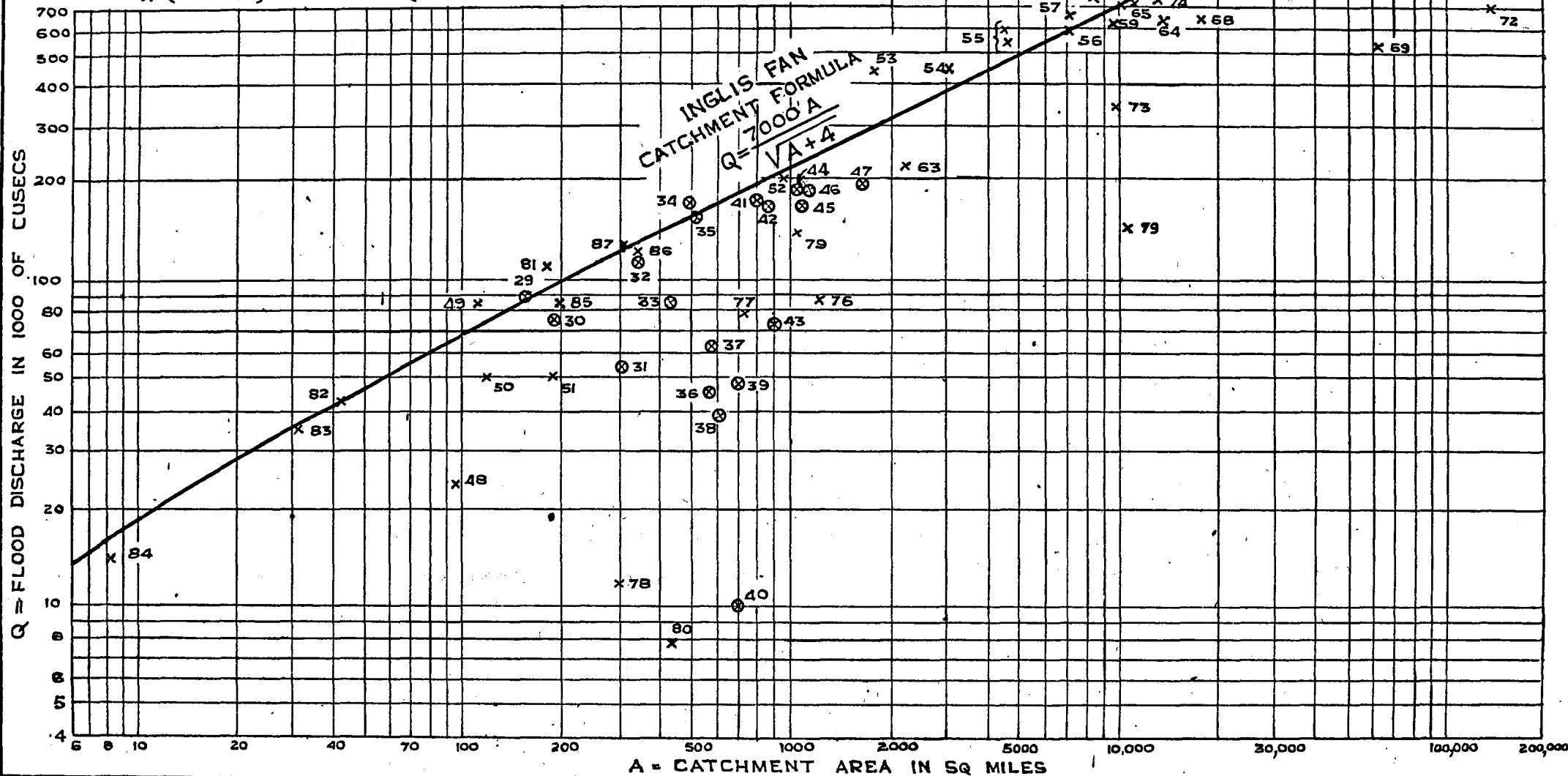
1924

FIG. 4/A -1(ii)-1

GRAPH SHOWING
EXTRAORDINARY FLOODS : CATCHMENT AREA

REFERENCES ~
⊙ (29 TO 47) BOMBAY PRESIDENCY DATA (FAN SHAPED CATCHMENT.)
X (48 TO 87) INDIAN DATA (EXCLUDING BOMBAY PRESIDENCY)

NR 1601
I.W. E. STN
POONA.



- (2) The ratio of diameter of standard catchment of same area and the actual storm length of the catchment concerned is a measure for estimating shape variation,

$$K = \frac{2}{\sqrt{\pi}} \cdot \frac{\sqrt{A}}{L}$$

- (3) For Deccan catchments

$$Q = K^{1/3} \frac{7000 A}{\sqrt{A + 4}}$$

where Q = Maximum discharge.
 A = Area of catchment.
 L = Storm length of catchment.
 K = Shape factor.

Of course, this formula deals with normal shape correction; it does not take into account the super-imposed effect of flood absorption in a long river. In this connection, an interesting case of the Karamanasa River in the U. P. is mentioned on page 68 of the 1944 A. R. (T); where it is shown that if we divide a fan-catchment into two equal parts and assume the run-off from each of the elongated half fan-catchments equal, the run-off must be 0.707 times the fan-catchment value to give the combined discharge for the whole area, thus indicating that an allowance must be made for the shape of the catchment and that it is entirely wrong to apply the fan-catchment formula to elongated catchments without affluents.

5. Though the fan-catchment formula gives a general good idea of the maximum run-off, it does not give any indication of whether a maximum flood will occur once or 50 times, in a 1000 years. Obviously, there are some areas in India like Baluchistan, where rain is so rare and storms so small that the chance of a maximum flood occurring may be said to be nil. There are other areas, like the cyclone areas of Bengal and Orissa and the Western Ghats (Bombay), where floods approaching maxima are liable to occur several times in 1000 years.

The only way to decide this question is by judgment based on past experience & records; but in examining these, one point of importance should be borne in mind that maximum storms tend to occur more frequently in *medium-sized* catchments than in very large or very small catchments. The reason for this is that in the case of very small catchments the probability of the core of the storm passing over the small catchment is remote; while in the case of very large catchments, the rain storms rarely cover the whole of the catchment.

Very large catchments should generally be looked upon as made up of several tributaries. Each of these has a small fan-catchment at its upper end, followed by long stretches in which there is little replenishment from affluents and much reduction of discharge due to flood absorption in flowing down the river. In such cases a detailed analysis is necessary. The reduction of discharge due to flood absorption in long narrow catchments is very considerable and must be allowed for.

6. Judgment must also be used as to whether a risk may be taken or not. If a dam is to be built above a town, no risks can obviously be taken; but if such damage as will occur—say once in a 100 years or more—can be easily rectified, it will often be unnecessary to design for the maximum-ever flood, or this can be done by providing a breaching section. The waterway of a bridge, for instance, may be designed according to the formula

$$Q = \frac{4000 A}{\sqrt{A + 4}} \quad \text{This is the statistical } \textit{average} \text{ formula obtained from all available data at this station, shown in Fig. 4. An allowance for afflux and free-board upstream of masonry structures should, however, be made for the } \textit{maximum possible} \text{ discharge according to the envelope curve}$$

Similarly, protective measures downstream of structures

$$Q = \frac{7000 A}{\sqrt{A + 4}}$$

should be designed for the maximum ever discharge, i. e. $Q = \frac{7000 A}{\sqrt{A + 4}}$

The Director, when he was in charge of designing the Kotri Barrage Project, actually suggested this principle namely that the waterway and crest levels should be designed for the normal maximum floods obtained, say, in 2 to 3 years in every ten—so as to obviate the serious consequences of shoaling, oblique flow, & concentrations accruing from too "loose" a waterway while providing afflux, freeboard, and downstream loose stone protections for the superflood, i. e. maximum-ever discharge likely at the site. In the case of the Sadabah River (Bihar), recently referred to the station for advice, in which the catchment area is 16.64 sq. miles, although the maximum possible discharge from the envelope curve of the fan-catchment formula is 25880 cusecs (or say 26,000 cusecs), the normal flood works out to 15,000 cusecs according to $\frac{4000 A}{\sqrt{A + 4}}$. If Dickens' formula were used with a coefficient of 1600—as later suggested, see para 2—flood discharge works out to 13,400 cusecs which is nearly equal to the normal flood of 15,000 cusecs. From Fig. 3 it will be seen that for small catchments the data obtainable at this station would not support a lower coefficient in Dickens' formula.

Fig. 3.
A-1 (ii)-1

A—HYDROLOGY.

3—Measurement of flowing water.

A-3-1 Coefficient of discharge in Notch type & Weir type falls on the Mahanadi Main Canal (C.P.) (Advice without experiment).

1. The Mahanadi Main Canal (C. P.) has 5.25 ft. falls at R.D. 2783 & 2803. The upstream fall consists of 3 notches similar to the ones described in Buckley's Irrigation Pocket Book. The lower fall is a rectangular notch. The nearest gauging site is 6 miles upstream of the falls and in the intervening distance there are a number of off-takes and the canal runs in curves, shaley cuts and fills. Estimating the actual discharge reaching the fall at R. D. 2783 is, therefore, troublesome and it was desired to use one or the other of the falls as a meter. Fig. 1a & B
A-3-1

2. The data of the two falls was analysed by Mr. G. A. D. Cochrane as shown below :

3. "The Notch (Punjab type) fall at ch : 2783 has 3 notches similar to the one described in Buckley's Irrigation Pocket Book. The formula for discharge per notch is :

$$Q = 5.35 C H^{3/2} (B + 0.4 n H)$$

where C is said to be 0.75 for $Q > 300$ cusecs
& 0.70 " " " "

From the sketch, $n = \frac{7.0 - 4.3}{6.5} = 0.415$

$$\& B = 4.3 \text{ ft.}$$

If $K_1 = 5.35 C$, then

$$Q (\text{for 3 notches}) = 3K_1 H_1^{3/2} (4.3 + 0.166 H_1)$$

$$= K_1 (E)$$

4. The rectangular notch at ch : 2803 has a cill length of 30.3 ft. The width of the cill and sides is 3.5' : the cill is horizontal and the edges are not rounded : the sides are straight and parallel to each other and to the centre line of the canal.

The usual formula for discharge is

$$Q = K_2 B H_2^{3/2}, \text{ and substituting } 30.3 \text{ for } B$$

$$Q = 30.3 K_2 H_2^{3/2}, \text{ Cusecs}$$

$$= K_2 (F)$$

5. The still-water depths, H_1 & H_2 , were read simultaneously at various values of Q . Their values, with the corresponding values of E & F , are shown in the following table. The last column gives $\frac{K_2}{K_1} (= \frac{E}{F})$.

Table No. 1/A-3-1

Fall at ch : 2783		Fall at ch : 2803		$\frac{K_2}{K_1}$ (= E/F).
$G_1 (= H_1)$	E	$G_2 (= H_2)$	F	
2.90	70	1.80	73	0.960
4.25	133	2.70	135	0.985
4.80	161	3.15	170	0.950
5.70	214	3.80	224	0.955
6.3*	254	4.00	242	1.05

* There is probably slight "submergence" at this stage despite the standing wave effect.

6. Thus allowing for practical error,

$$K_2 = K_1 \text{ (to within 5 per cent)}$$

But $K_1 = 5.35 C$ where C lies between 0.70 and 0.75

i. e. $K_1 = 3.75$ or more (up to 4.0)

and, therefore, $K_2 = 3.7$ or more.

But this value of K_2 is much higher than one would expect from such a "bastard" type notch

3. This analysis was sent to the station with the following questionnaire :-

"(1) Is K_2 (as derived from K_1) likely to be correct ?

(2) If K_2 is wrong, what would a more suitable (rough) value be ?

(3) As the reduction in K_1 demands a proportionate reduction in K_2 , is the punjab coefficient (C) incorrect, and what is the correct value ?"

4. A comprehensive series of experiments with weirs of lengths varying from 0.48' to 16.30' and heads up to 4' was done at the Cornell University and results have been given in Gibson's Hydraulics (Edition 4, page 172).

Gibson's analysis of Cornell Experiments gave results correct within 3 percent so long as the length of the weir in line of flow was greater than twice the head; and suggested that the relative error increased as the relative depth increased. These results when interpolated for length of weir (L) = 3.5', give results as tabulated in Table No. 2.

Table No. 2/A-3-1

Sr. No.	L ft.	H ₂ from Table 1 ft.	K ₂	B ft.	H ₂ ^{1.5}	Q = K ₂ BH ₂ ^{1.5} cusecs.	$\frac{L}{H_2}$	Remarks.
1	2	3	4	5	6	7	8	9
1	3.5	1.8	2.73	30.3	2.42	200	1.94	
2	"	2.7	2.72	"	4.44	365	1.30	
3	"	3.15	2.70	"	5.60	455	1.11	
4	"	3.80	2.69	"	7.40	604	0.92	
5	"	4.0	2.68	"	8.00	644	0.87	

5. It will be seen that in all the cases the value of $\frac{L}{H_2}$ is not greater than 2. In the first case of $H_2 = 1.8'$, however, the value of K_2 may be taken as a rough approximation.

6. For the trapezoidal notch fall at R. D. 2783, computing discharges by the Notch formula

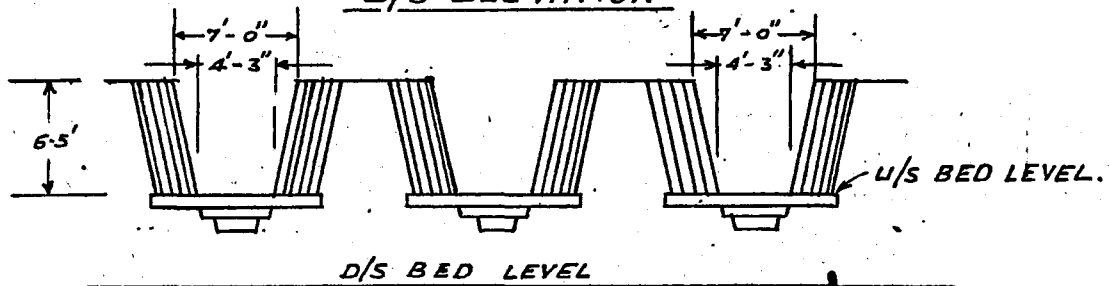
$$Q = 5.35 \times C \times H_1^{3/2} \times (B + .166 H_1)$$

MAHANADI MAIN CANAL C.P.

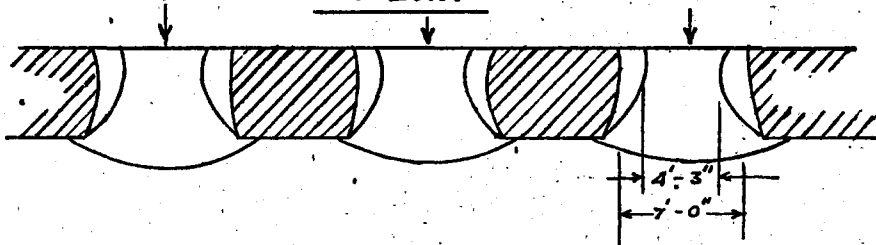
(a) PUNJAB TYPE NOTCHED FALL AT CH. 2783

SCALE 1" = 10'

D/S ELEVATION

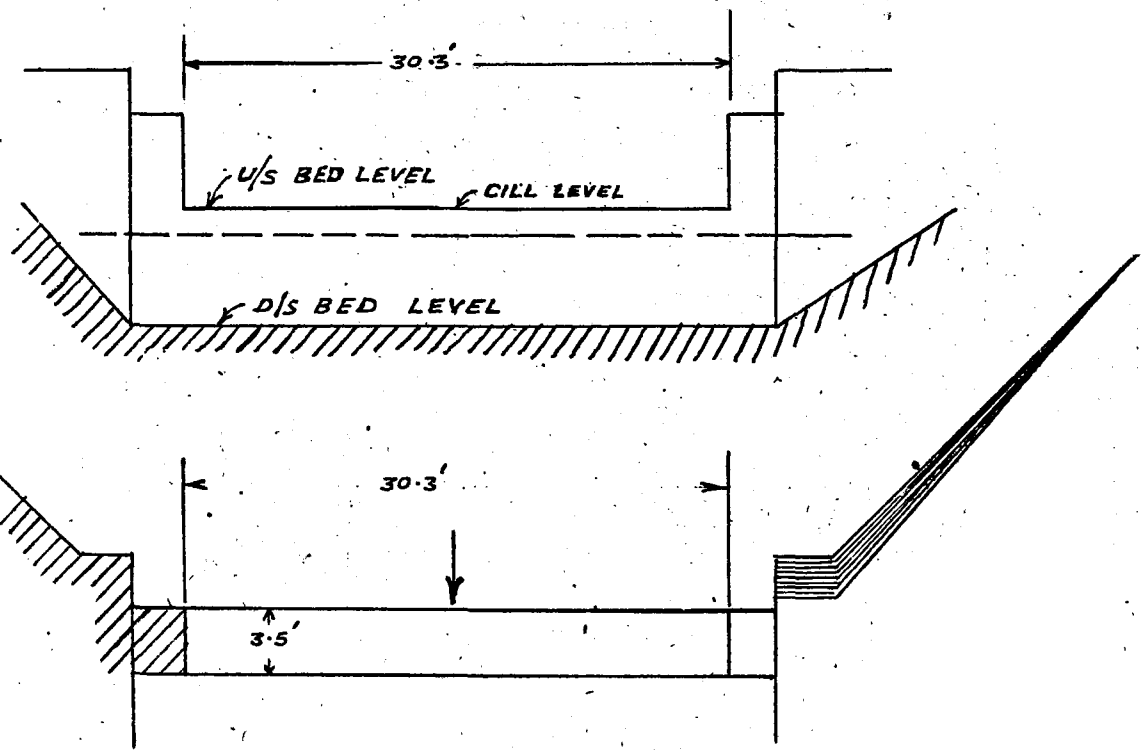


PLAN



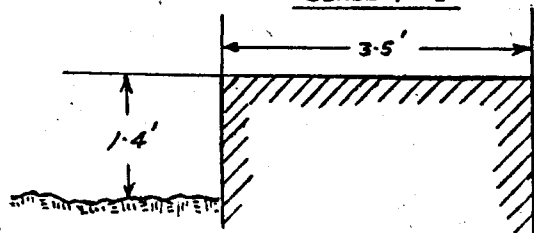
(b) RECTANGULAR NOTCHED FALL AT CH. 2803.

SCALE 1" = 10'



CROSS SECTION AT CILL

SCALE 1" = 2'



and comparing them with those obtained in Table 2 above we get results tabulated in Table 3 below :—

Table No. 3/A-3-1

Sr. No.	H ₁ Ft.	C	$\frac{3}{2}$ H	$B + .166H_1$ or $4.3 + .166H_1$ Ft.	Total of 3 notches cusecs	Q from Table 2 cusecs	Percentage varia- tion of Col. 6 from Col. 7.
1	2	3	4	5	6	7	8
1	2.9	0.70	4.94	4.78	264.5	200	+ 32
2	4.25	0.75	8.76	5.01	528	365	+ 44.5
3	4.8	„	10.92	5.13	650	455	+ 43
4	5.7	„	13.25	5.25	818	604	+ 35
5	6.3*	„	15.80	5.35	1019	644	+ 58

* Weir is stated to be drowned.

7. This *wide* variation of the discharges suggests that the discharges at R. D. 2783 have been overestimated, probably because of the approach conditions being different from those of the Punjab data or because the Punjab coefficient is too high for these notches for the reasons mentioned above.

Each weir with its individual approach conditions would have different value of C. In the particular case under reference therefore, it is better either to estimate 'C' by current meter observations or model tests, rather than estimate one from the other.

8. The answer to the three questions of Mr. Cochrane are :—

(1) K_2 should not be derived from K_1 ; but should be obtained independently either by specific model experiments or current meter observations below the fall; because the approach conditions and the nature of intermediate transitions will affect the coefficients.

(2) K_2 is apparently high and should be of the order of 2.7 (roughly) rather than 3.7: The observations made are (except probably the first) not on a broad-crested weir (where L/H is not less than 2.0 and the error increases with increase in depth); but as aptly stated by Mr. Cochrane on a "bastard" type notch where the error rapidly increases with increase in relative depth.

(3) Apart from the impropriety of correlation of coefficients of two notches with different approaches (see 1), the use of the Punjab coefficient C—perhaps never verified by accurate model tests but obtained only from rough discharge measurements with floats—since it was obtained years before the use of models and current meters was generally appreciated—is unwarranted, particularly for a series of such notches. The effect of piers being considerable the coefficient for three notches will be appreciably lower than that of a single notch, which itself appears, *prima facie*, high and needs experimental corroboration.

B—THEORY OF FLOW & DESIGN OF CHANNELS.

1. Flow in Rivers and Canals.

(ii) FLOW IN CHANNELS IN INCOHERENT & COHERENT ALLUVIUM.

B-1 (ii)-1. Comparative analysis of sand by Puri sand-meter and Air sand-meter.

Appendix I

B-1(ii)-1

Fig 13

B-1(ii)-1

Introduction:—The idea of grading sand by dropping it through a constant current of wind was apparently first evolved in the U.S.A.—vide extract from Civil Engineering for July 1939 reproduced in Appendix I. The wind tunnel classifier—as it was then called—was modified at the Poona Station for finding out the distribution of sands (.06 to 0.635 mm.). This was calibrated against the Puri 'sand-meter'—vide Appendix II.

Appendix II

B-1(ii)-1

Experiments described below were carried out to see if a similar distribution was obtained when the same sample was analysed by the two methods.

Experiments:—Two samples of sands—one coarse and one medium, from the Indus at Sukkur and the Jumna at Delhi respectively—were used for this analysis.

The coarse sand had a specific gravity 2.66 and the medium sand had a specific gravity 2.62. Each analysis was repeated three times with the same sand maintaining all the conditions of air sand-meter the same (viz. velocity, sand charge etc.). It was found that, even if the analysis was repeated several times, the air sand meter gave almost exactly the same results.

Fig 4

B-1(ii)-1

Table I

B-1(ii)-1

The distribution curves of medium sand and coarse sand show that almost all points lie on the same curve and the results in Table 1 show that there is only a slight variation between the two results.

Table I/B-1(ii)-1.

Analysis by Air sand-meter.

No. of Obs.	Air speed indicator at 31°		Air speed indicator at 32°	
	Sukkur coarse sand (Sp. Gr. =2.66) Dia. in mm.	Jumna medium sand (Sp. Gr.=2.62) Dia. in mm.	Sukkur Coarse Sand. Dia. in mm.	Jumna Medium Sand. Dia. in mm.
I	.352	.252	.341	.243
II	.353	.256	.343	.246
III	.348	.253	.340	.245
Mean of three	.351	.254	.341	.245
	± .0020	± .0018	± .003	± .0013
	Or ± 0.6 per cent.	± 0.7 per cent.	± 0.59 per cent.	± 0.53 per cent.

A similar test with the Puri sand meter was also done with the same sand samples. Each analysis was again repeated three times with special care to eliminate personal error. It was found that in the case of coarse sand there was some variation between the results; but in the case of medium sand there was practically no difference between the first two observations.

Fig 3

B-1(ii)-1

The distribution curves in Fig. 3 show that all the points overlap each other in the case of medium sand, while there is some variation in curves and also in 'm' value in the case of coarse sand.

Table 2/B-1(ii)-1.

Analysis by Puri sand-meter.

No. of observations.	Sukkur coarse sand (Specific Gravity 2.66) m in mm.	Jumna at Delhi medium sand (Specific Gravity 2.62) min mm.
I	.334	.237
II	.342	.237
III	.328	.246
Mean of three	.335	.240
	+ .0054	+ .0042
	Or ± 1.6 per cent.	Or ± 1.75 per cent.

The summation curves of these two sands show clearly the difference Figs. 1 & 2
B-1(ii)-1 between the two observations. In case of the Puri sand meter we do not get the distribution curve directly; but by drawing a mean smooth summation curve we can get the distribution percentage of sand particles with the diameter interval of the multiple of $(2)^{\frac{1}{2}}$.

In the Air sand meter, there is little chance of any personal error and any variations in the distribution are due to air eddies which can be minimized, if not altogether eliminated.

Analysis by Puri sand meter is subject to the following sources of errors:—

- (1) The whole sample taken for analysis is dropped all together from the top with the result that there may be collision of particles and some coarse particles may drag along with them some fine particles lessening their velocity; but in the case of the Air sand meter, the particles on falling through a slit in the funnel (3 mm.) get separated by air and there is no chance of collision.
- (2) The time of interval of collection of particles in the troughs down to .24 mm. dia. is very small (< 4 seconds) so that some error is inevitable in bringing the collecting troughs below the vertical brass tube within such short intervals; on the other hand, the sand in the Air sand meter, when once dropped in the funnel, is automatically graded in the various compartments.
- (3) The distribution percentage of the particles at a diameter interval of $(2)^{\frac{1}{2}}$ is obtained after drawing the mean summation curve, which may itself involve some personal bias; in the case of the Air sand meter, however, we get the distribution curves directly; thus there is no personal bias, and much time is saved.

Appendix I./B-1-(ii)-1.

Extract from Civil Engineering for July 1939, Vol. 9, No. 7
(Page 414-15).

Wind Tunnel Classifier for Sand and Silt.

Inexpensive Apparatus sorts Granular materials for Laboratory use.

By George H. Otto & Hunter Rouse, ASSOC. M. AM. SOC. C.E.

Respectively Assistant Geologist & Associate Hydraulic Engineer, Cooperative Laboratory, U.S. Soil Conservation Service, California Institute of Technology Pasadena, Calif.

Methods of sorting granular material according to particle size have long claimed the interest of a number of professions. Sieves are widely used as a means of laboratory analysis, and on a large scale are well adapted to commercial sizing of gravel and crushed stone. However, sieving becomes less satisfactory for quantity separation as the material decreases in size, and it is particularly difficult to obtain well sorted separates with sieves finer than 200 mesh; furthermore, large variations in the shape or density of particles may cause undesirable complications. These difficulties are largely overcome in hydraulic separation, either air or water being used as the distributing medium. Such methods have reached their highest development in the abrasive and mining industries, but the various patented devices developed by them are generally designed for specific separations on a commercial scale.

The increasing need for a simple, dependable, and inexpensive classifier has become particularly apparent in the sediment-research laboratory, where a wide assortment of sand and silt separates is required in sufficient quantity for experimental investigation. As has been demonstrated at the Pasadena Laboratory of the Soil Conservation Service, U.S. Department of Agriculture, artificial mixtures of any desired type can readily be prepared from a series of well-sorted separates differing in average diameter by a small increment. Since the material required by this laboratory includes sizes finer than 200 mesh, and since it is used exclusively in hydraulic experiments, a method of sorting based on settling velocities has been developed, with equipment constructed at low cost in the laboratory shop.

The Principle and Design of the Classifier.

This method utilizes the fact that sand and silt particles dropped in a horizontal stream of air are distributed approximately in accordance with their "hydraulic size"—that is, according to the diameter of spheres having the same density and the same velocity of fall. That the resulting distribution is not exact is due in part to the presence of eddies in the air stream. The latter influence has been reduced to a minimum through the use of a tunnel having a carefully rounded bell entrance, a fan at the downstream end drawing a steady flow of air through the structure (Fig. 6).

Fig. 6
B-1(ii)-1

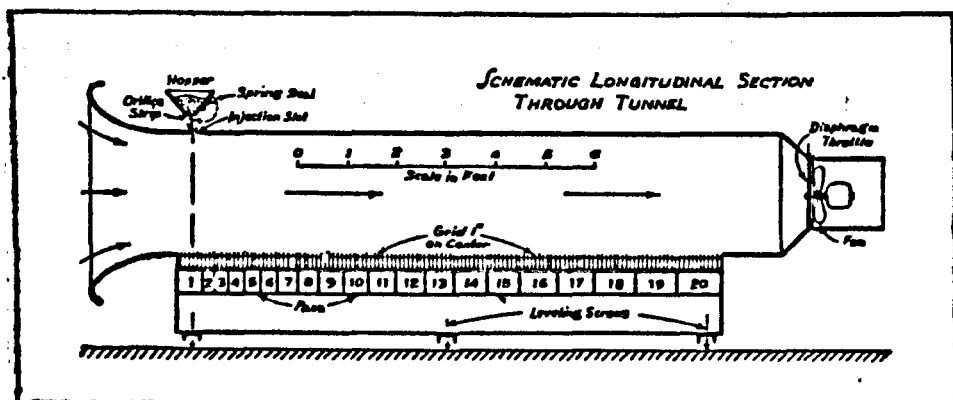
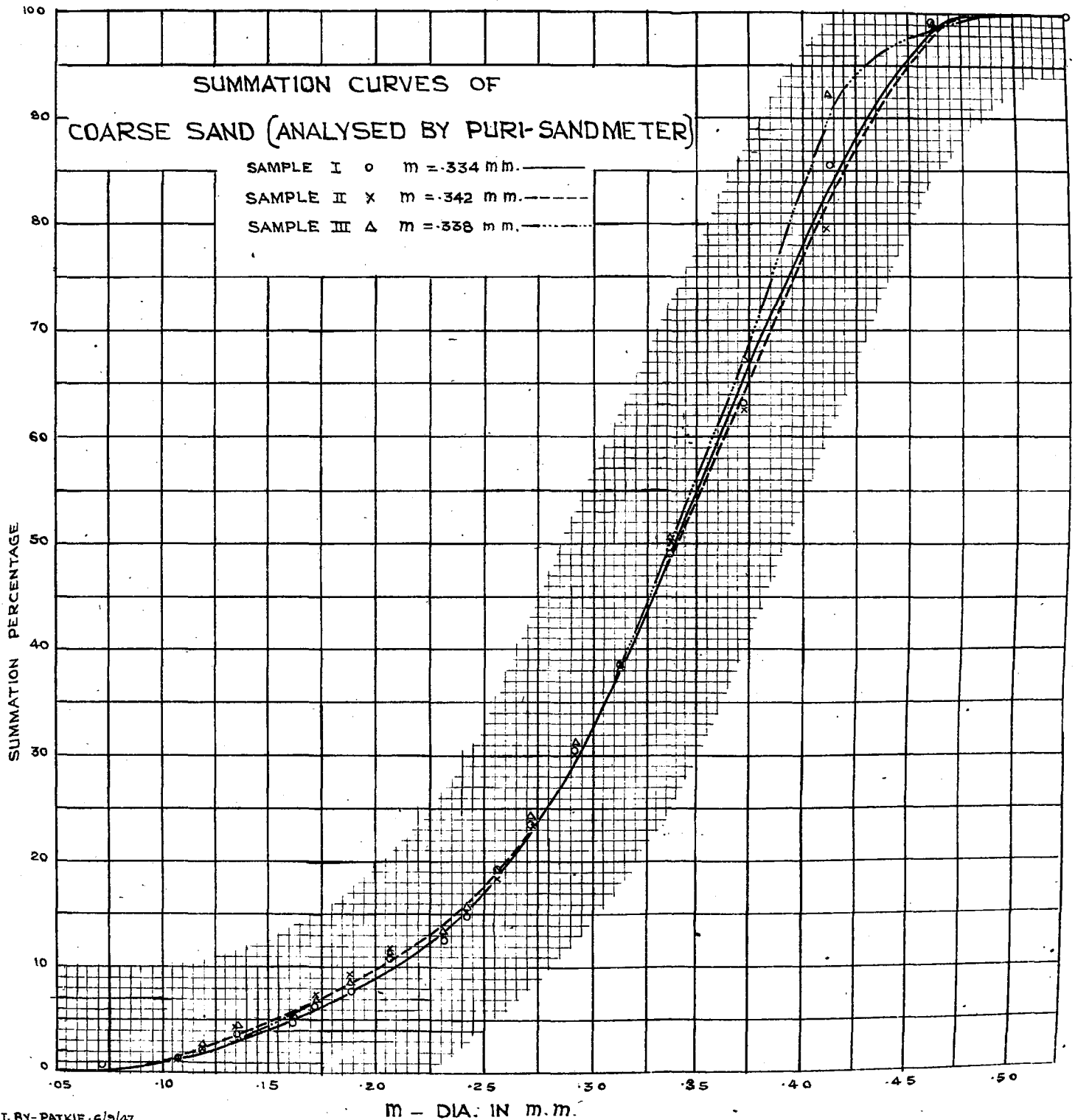


FIG. 6/B-1(ii)-1

FIG. I/B-1(ii)-1.



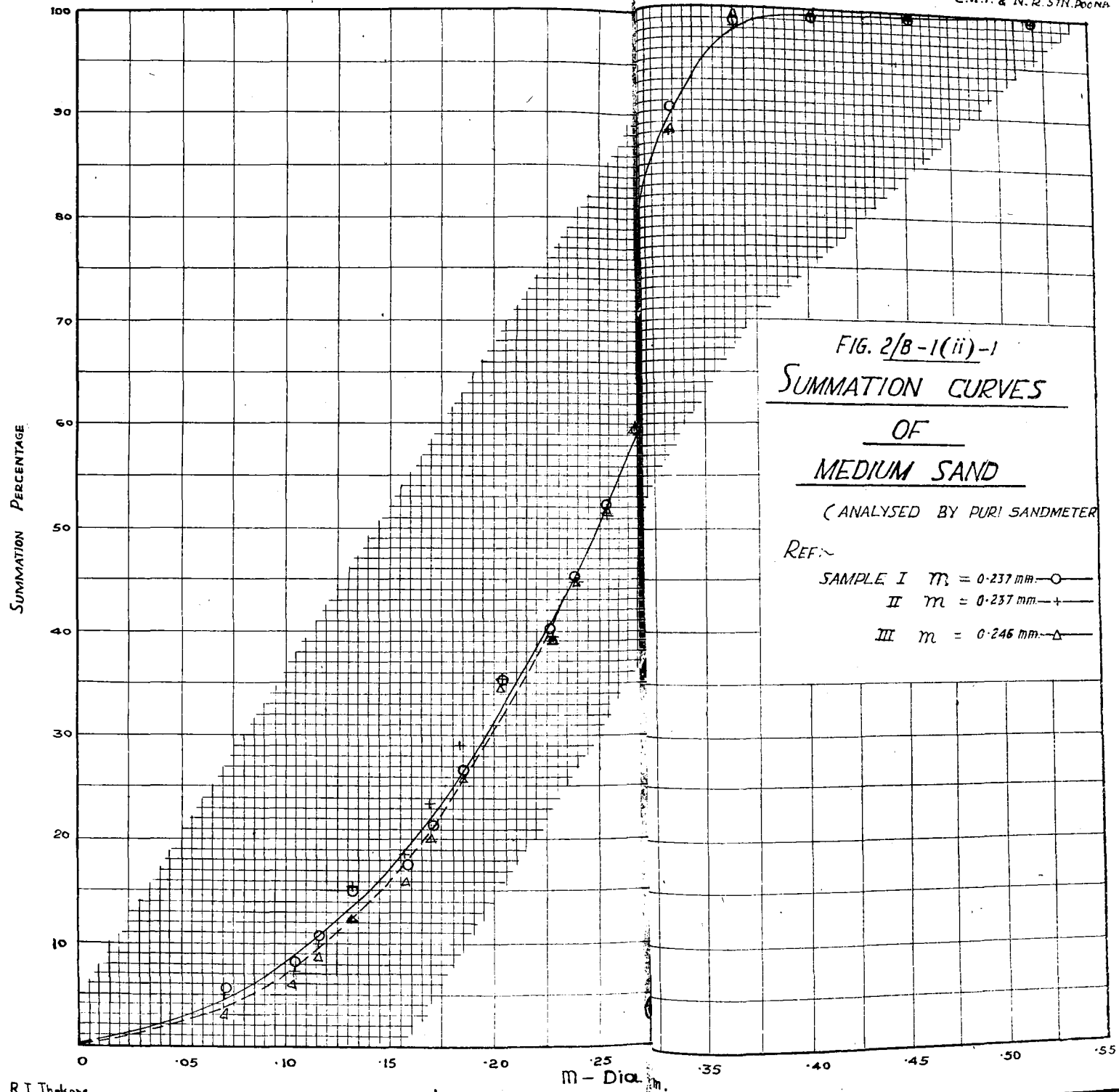


FIG. 2/B-1(ii)-1
SUMMATION CURVES
OF
MEDIUM SAND

(ANALYSED BY PURI SANDMETER)

REF.~

- SAMPLE I $m = 0.237 \text{ mm}$ - ○ -
- II $m = 0.237 \text{ mm}$ - + -
- III $m = 0.246 \text{ mm}$ - △ -

FIG. 3/B-1(ii)-1
SIZE DISTRIBUTION CURVES OF
COARSE & MEDIUM SANDS.
(BY PURI SANDMETER.)

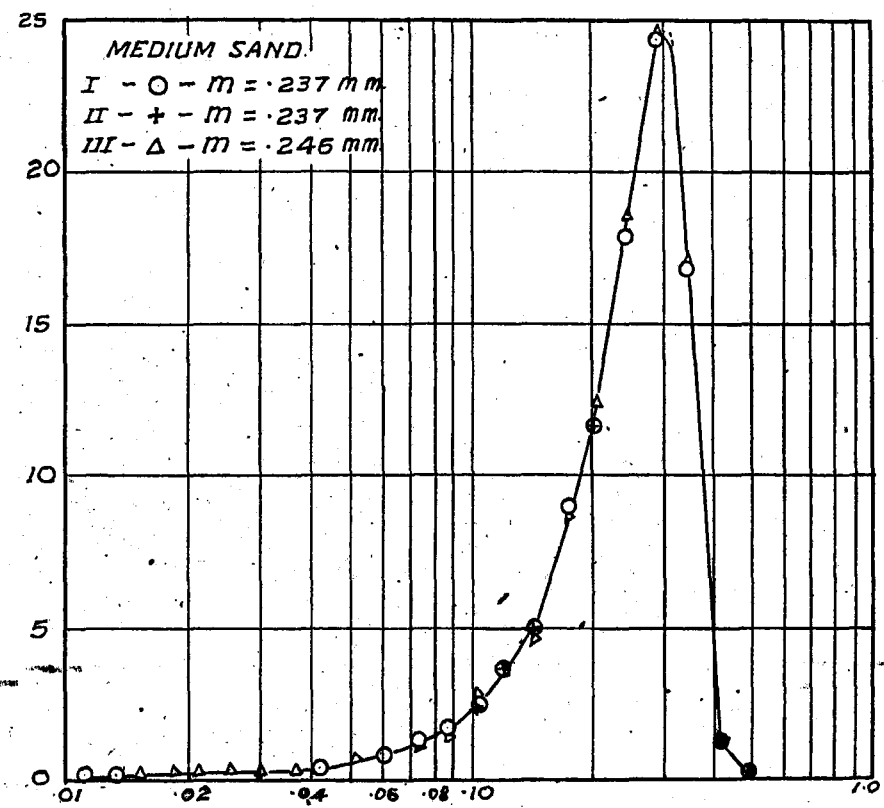
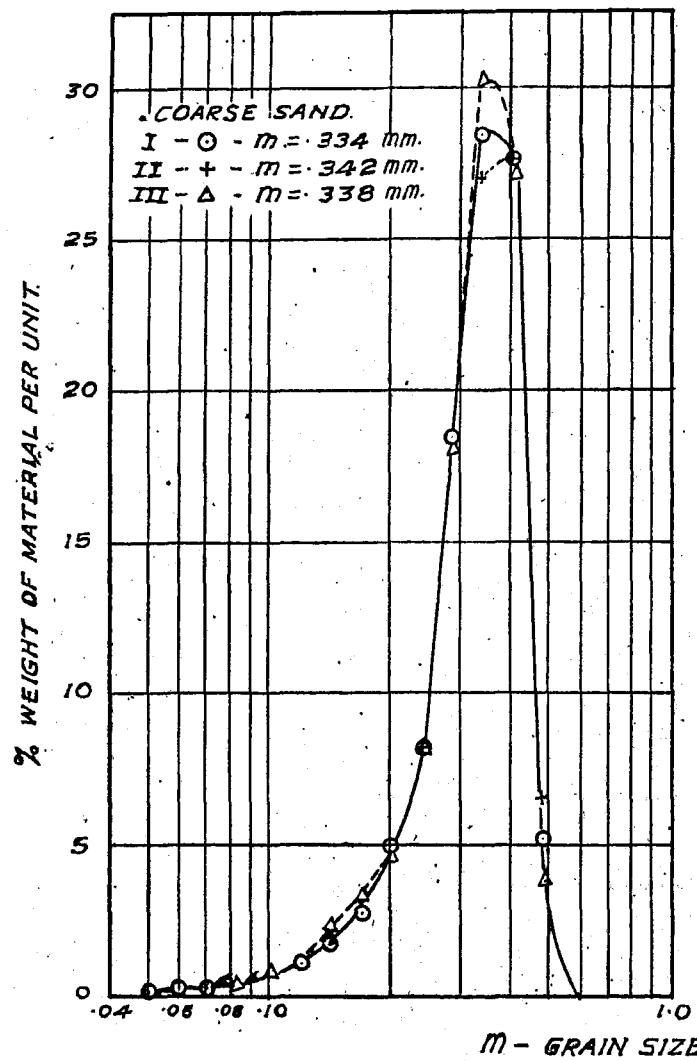
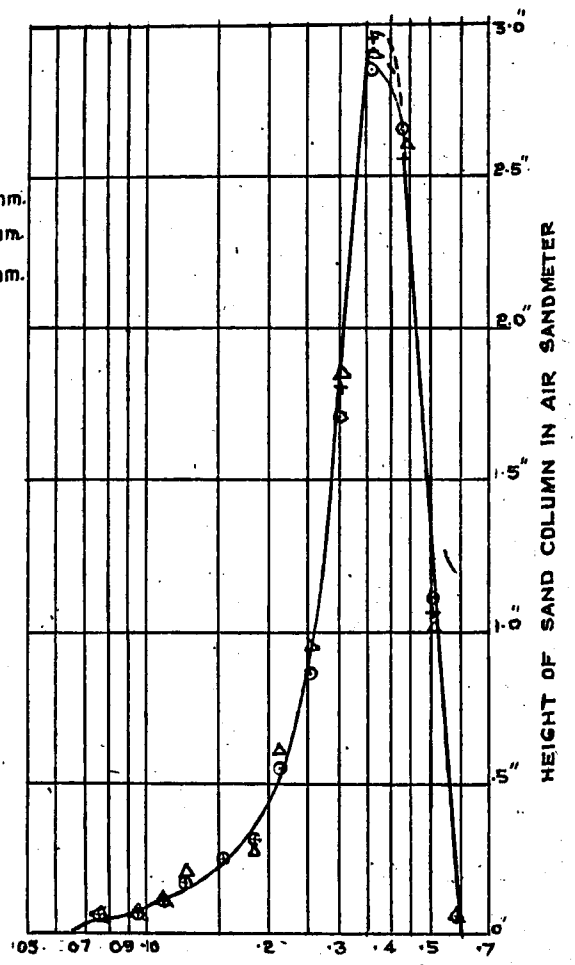
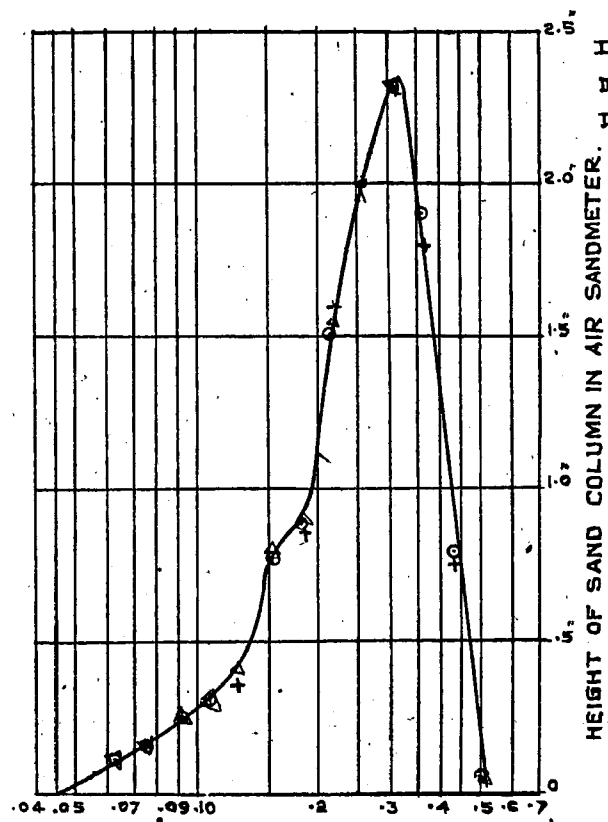


FIG. 4/B-1 (ii) - I.
SIZE DISTRIBUTION CURVES OF
COARSE AND MEDIUM SANDS
(BY AIR SAND METER)
(AIR SPEED INDICATOR AT 31°)

COARSE SAND
I. O. m. .352 mm.
II. +. m. .353 mm.
III. Δ. m. .348 mm.



MEDIUM SAND.
I. O. m. .252 mm.
II. +. m. .256 mm.
III. Δ. m. .253 mm.

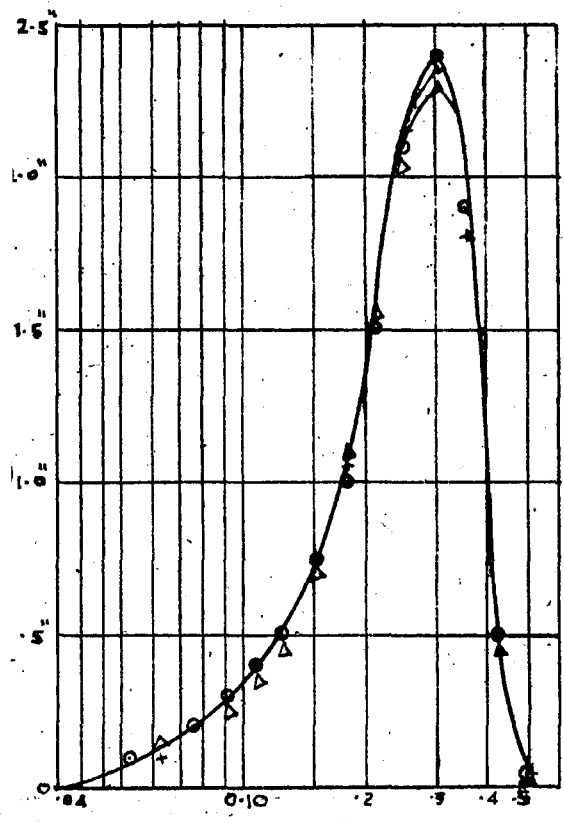
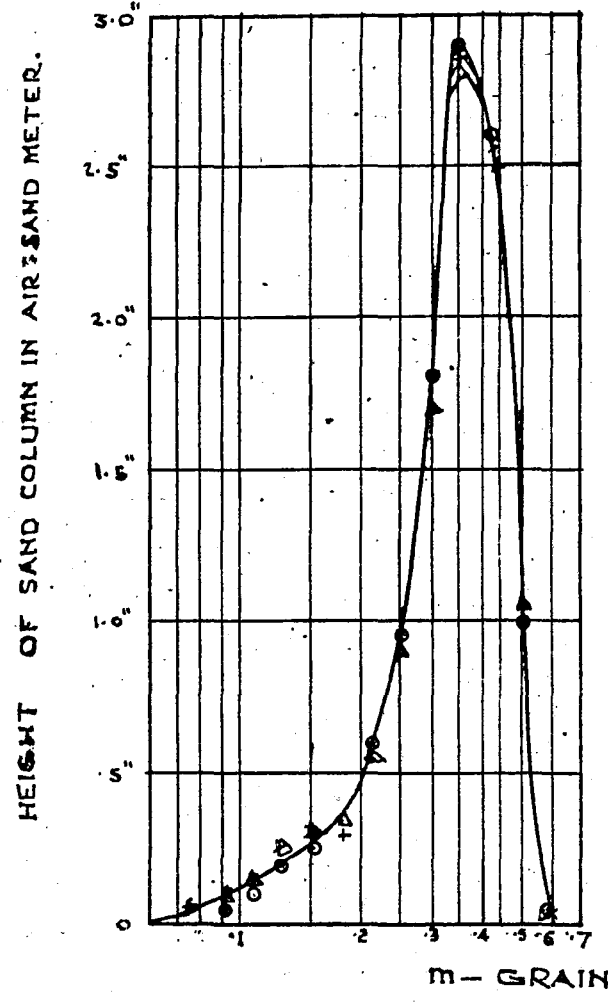


R.k.k.

FIG. 5/B-1(ii)-I. SIZE DISTRIBUTION CURVES OF COARSE & MEDIUM SANDS (BY AIR SAND METER)

(air speed indicator at 32°)

coarse sand
I O $m = .341$ mm.
II + $m = .343$ mm.
III Δ $m = .340$ mm.
mean $m = .341$ mm.



medium sand
I O $m = .243$ mm.
II + $m = .246$ mm.
III Δ $m = .245$ mm.
mean $m = .245$ mm.

As shown in Fig. 7, the tunnel is rectangular in section, the inner surfaces of the sides and top being smoothly joined and enameled. A gravity-

Fig. 7
B-1(ii)-1

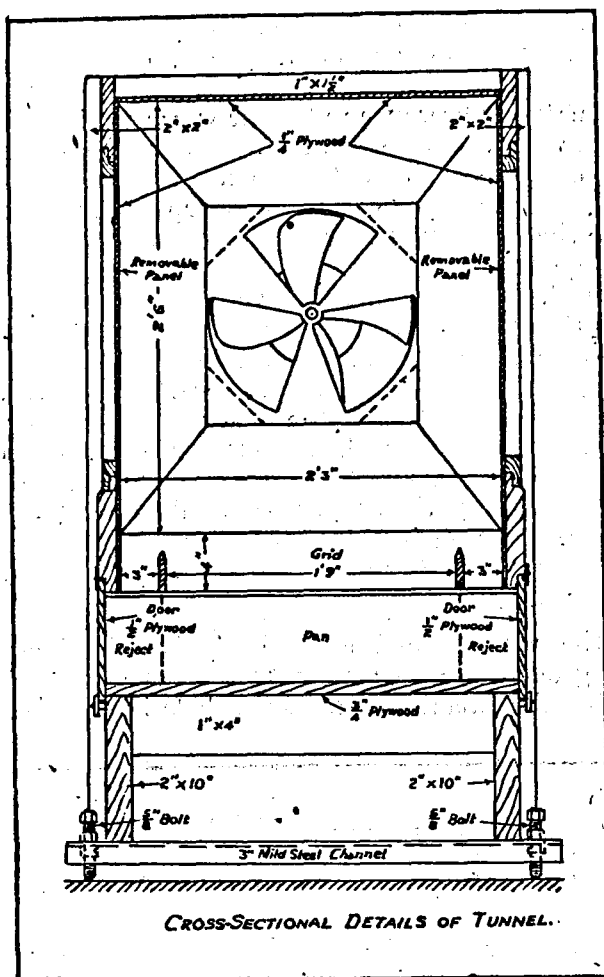


FIG. 7/B-1(ii)-1

feed hopper, located near the beginning of parallel flow, discharges the sediment in a thin, uniform sheet extending nearly across the section. The flow from the hopper can be started or stopped abruptly by means of an external spring seal, while interchangeable orifice strips permit regulation of the rate of feed between the limits of 60 and 100 lb. per hr. The entire bottom of the tunnel consists of a grid so shaped and finished as to prevent the accumulation of sediment on the upper edges. Grains falling to the floor therefore pass at once through the grid to the collecting pans. In order to avoid the effect of wall influence on the separation, partitions are so located in the grid and in the pans that all material falling within 3 in. of either wall can be discarded. Doors at pan level seal the tunnel sides during operation. A 15-in. duct fan, driven by a sealed, rubber-mounted, 1/10-h. p. motor, provides a maximum air velocity of approximately 2.5 ft. per sec. This can be reduced to any desired magnitude by a diaphragm throttle, made of three pivoted sheet-metal sections.

A white Nevada foundry sand has been selected for the quantity production separates, because of its uniformity of density, mineral composition, and shape of grain. This sand contains 99 per cent silica, has well-rounded grains in the coarser sizes, and varies in nominal diameter from essentially 0.02 to 0.6 mm. Prior to the construction of the wind-tunnel classifier, 5 tons of the original material had been partially sorted by passing through a power-operated sizing device containing screens of 28, 35 and 48 mesh, preliminary screening being necessary, of course, to remove foreign matter that might clog the hopper orifices. Subsequent experience has shown that the present classifier will sort material as large as that passing the 28-mesh.

At present the routine procedure of classification is as follows: Thoroughly air-dried screened material is first passed through the tunnel with the throttle nearly closed, in order to separate the silt and dust from the sand. The coarse material collects directly beneath the feed slot, the fines being sufficiently well distributed to permit choice of separates from all but the first few pans. (Most of the dust settles outside the machine; although the amount is very small, its presence in the air warrants the use of a respirator by the operator to avoid the danger of silicosis.) The equipment is next cleaned, and the contents of the first few pans are mixed and rerun with the diaphragm set at maximum opening. Fairly sharp breaks in the amount of sand collected in the pans will now be found near both ends of the tunnel; the position of the upstream break depends upon the screen used in cleaning the original material, and that of the downstream break upon the number of separates removed after the run with the throttle nearly closed. The contents of all pans before the first break and beyond the second are now rejected, separates being obtained from the remaining pans. Material caught in the wall sections is added to the other rejected material, which may be mixed and passed again through the classifier when a sufficient quantity has accumulated. In this manner 25 or more separate grades are obtained, each differing in average size but possessing almost the same degree of sorting as the next finer or coarser grade. The separates are stored in 5-gal. cans, samples for control analysis being taken from each can by means of a Jones sample splitter.

Fig. 8
B-1(ii)-1

Although the wind tunnel method of classification yields hydraulic rather than true geometric separations, the shape and density of the sediment in use by the laboratory are such that sieve analysis furnish more-

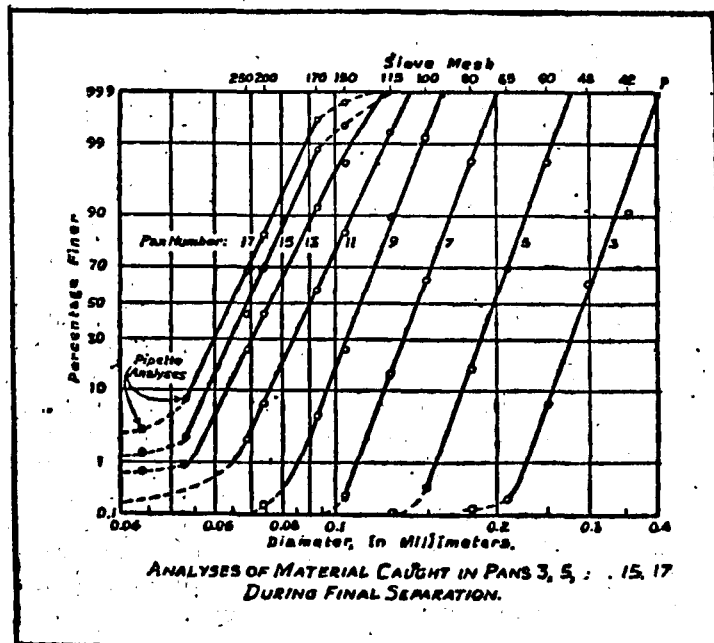


FIG. 8/B-1(ii)-1.

than a qualitative measure of the degree of sorting obtained. In Fig. 8, for instance, will be found a typical set of cumulative distribution curves for the final separation with open throttle; these correspond to the usual "per cent finer" curves, but are plotted on logarithmic probability paper in order to yield a linear graph for the logarithmic form of the normal error law. It will be seen that the material caught in each pan has an essentially normal distribution, except for a very small percentage of dust tails, under which circumstances the median and the geometric mean diameters for every separate coincide at the 50 per cent line. Since a vertical line represents perfect sorting, the deviation of each curve from the vertical is a measure of the size dispersion about the geometric mean. The dispersion will be seen to

increase slightly with distance from the point of feed, owing in part to an increase in turbulence along the tunnel boundaries and in part to the fact that the smaller sizes are more angular than the larger and hence show greater effect of shape in hydraulic versus sieve classification. A plot of the geometric mean diameter as a function of the horizontal distance from the feed slot to the middle of each pan (Fig. 9) is evidence of the smooth progression in size secured by the classifier.

Fig 9
B-1(ii)-1

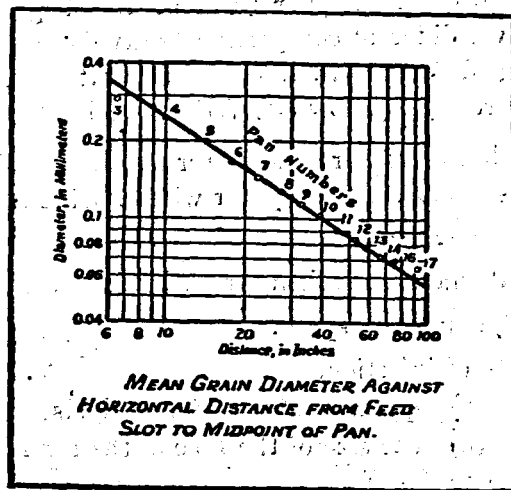


FIG. 9/B-1(ii)-1.

For precise results, the room in which the work is performed should be at least twice as long and several times as wide as the tunnel itself, to permit quiet circulation of air. Drafts, needless to say, must be avoided. Best performance will result if the tunnel is placed symmetrically in the room, with as much free space as possible around the bell entrance. The temperature should be reasonably constant for all runs, although variation in temperature will affect only the very fine sand and silt sizes; indeed, test runs with a 35° F difference in temperatures showed no measurable displacement of material coarser than 0.1 mm., while in the finer range the greatest change in mean diameter in any one pan was about 5 per cent, or some four times the probable error of mechanical analysis. Control analysis of the material caught in the various pans over a six-month period of operation indicate that with ordinary care the variation in mean diameter and degree of sorting will be not greater than two or three times the probable error of analysis.

Aside from the dependability of results, the great advantage of this form of classifier lies in the economy of the construction and operation. Material for the unit now in use cost approximately \$50, and it has been operated by one labourer for the past year. Needless to say, this design can readily be adapted to other requirements by modifying dimensions or fan capacity.

Appendix II/B-1(ii)-1.

Note on Air Sand Meter as designed and prepared by the Central Irrigation and H. R. Station, Poona and its standardisation (Drawing No. 4728) dated 7th Feb. 1944.

Fig. 13
B-1(ii)-1

Air-sand meter is based on the principle of grading sand particles by allowing them to fall through a hopper fixed at the beginning of the wind column and passing over different compartments divided into logarithmic scale and finally collecting different fractions in separate compartments of equal space of $1/3''$, fixed vertically (below line AB shown in elevation).

In order to calibrate the Air sand meter for the distribution curve, comparable in shape and size on semi-log paper with Puri siltometer curve, four things are to be adjusted by trial and error method :

- (1) Material to be taken for analysis.
- (2) Adjustment of speed.
- (3) Rate of injection,
 - (a) quantity of sand for analysis,
 - (b) the size of the hole of the funnel.
- (4) To find out the mean size of the fraction in each compartment.

The range of sand diameter used for analysis was 0.06 to 0.635 mm. as in the Puri sandmeter. The particles below .06 were removed by sedimentation method and above .635 by 20 mesh I.M.M. sieve.

(2) Adjustment of range of speed :—To get, firstly the idea of the speed of the fan, the following device was used :—

It consisted of a fine string slightly coated with wax to make it stiff suspended to a thin bar, fixed horizontally, at the downstream of the wind column (shown in Fig. 13). Any change in the speed of the fan was recorded in angles through which the string could be moved. One regulator in series

Fig. 13
B-1(ii)-1

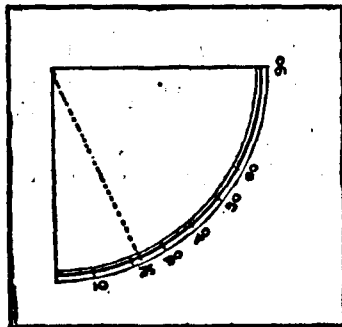
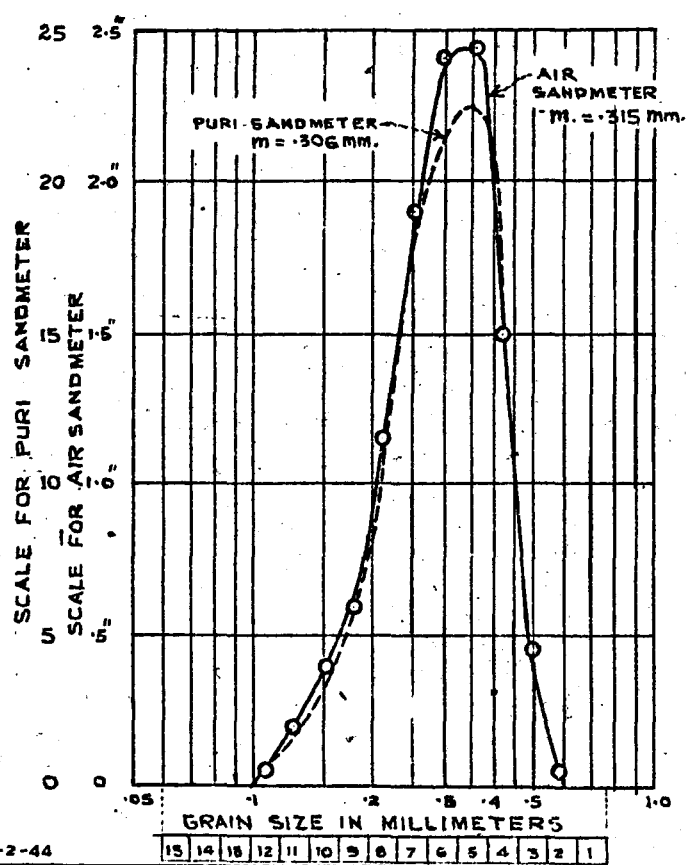


FIG. 10/B-1(ii)-1.

with 6 stops is placed between main switch and the fan of steps, so that the speed of the fan could be adjusted. To get the range of the speed, the step of the fan and the stop of the regulator were so adjusted that the finer particles in the sample were not blown away beyond the last compartment and at the same time the whole material was not collected in the first few compartments only.



B.G.D. 9-2-44

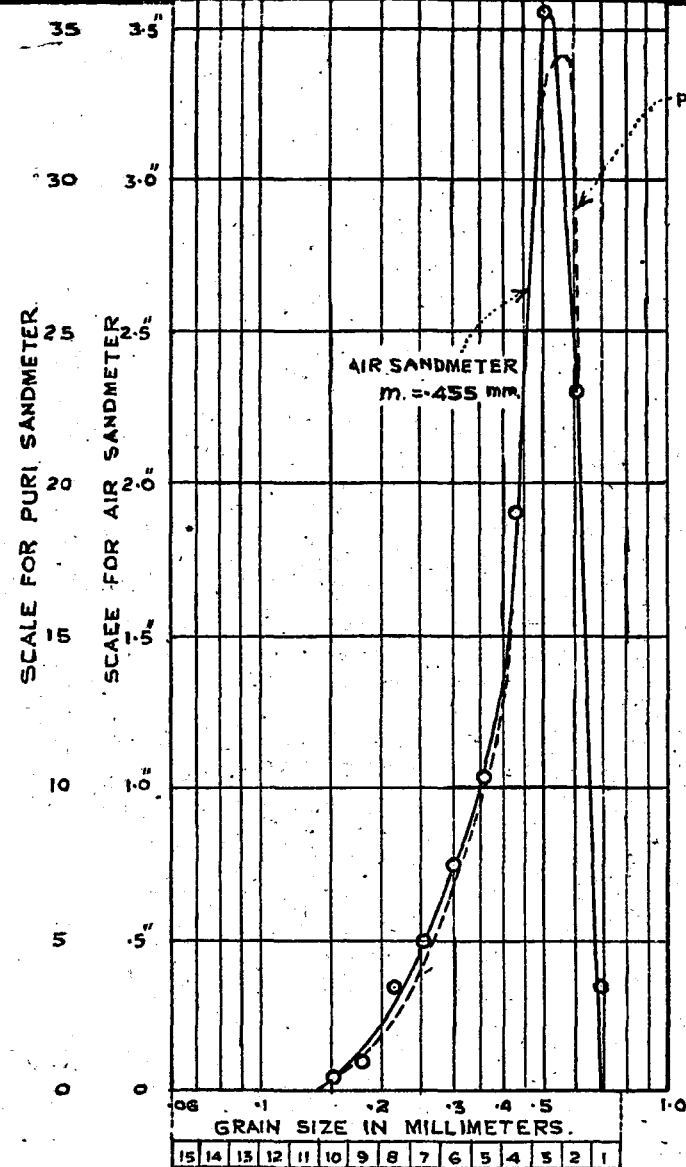


FIG. 12/B-1(ii)-1.
COMPARATIVE DISTRIBUTION
CURVES OF PURI SANDMETER
AND AIR SANDMETER

AIR SANDMETER } STANDARD SPEED 25°
INDICATED BY THE STRING.
STANDARD SAMPLE 12.0 GRMS.

- (3) Rate of injection :—Rate of injection is dependent upon the quantity of sand in the funnel and the size of the hole through which the sand is falling. Three different sizes of funnels were taken for trial 1.5 mm. ; 3 mm. ; and 4 mm. ; (as shown in figure 11), taking 15 gms. of sand each time and adjusting the speed so that the whole sample was collected in the compartments. The height in inches of each column was noted.

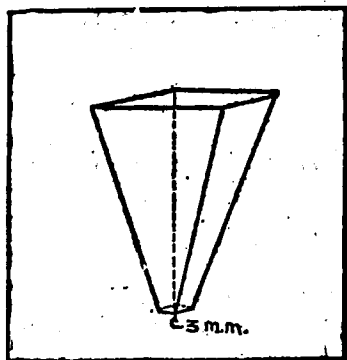


FIG. 11/B-1(ii)-1.

- (4) Lastly, finding out the size of fraction in each compartment : 13 equal compartments were drawn on semi-log graph paper starting with .07 mm. and plotted the heights of sand columns in Air sand meter according to Nos. of the compartments. On semi-log paper, the distance between .07 mm. and .635 mm. was divided equally into 13 spaces and one space on both sides was left spare for particles above .635 mm. and less than .07 mm. respectively, if there be any.

The quantity of sand, the size of the hole in the funnel and the velocity of the wind were so adjusted as stated above by trial to get the curve approximately agreeing with the 'Puri sand meter' curve.

Methods of calculation and Graphic representation : The diameter for each compartment was fixed and the height of each column of sand collected in Air sand meter could be measured in inches directly. Taking the sum of the products of height and the diameter of each fraction and dividing the same by the total height, weighted mean diameter was calculated. Plotting the heights of sand column against diameters on semi-log paper, the distribution curve is drawn.

Standard speed = 23° indicated by string shown in Fig. 10.

Standard weight of sample = 12 gms.

Standard size of funnel for injection = 3 mm.

Fig. 10
B-1(ii)-1

The Air sand meter was standardized with the velocity pointer at 23° , at the beginning. As the air sand meter was of wood and the wood was not seasoned, we had to adjust again the speed of the fan at 28° to get the distribution curve as before. Due to warping and drying of the wood in different seasons, we used to adjust the speed of the fan but such changes are very small now as the wood is fully seasoned. The change in temperature also does not affect the distribution as the speed of the fan is controlled. The air speed is checked and adjusted from time to time, to get the same results as with the Puri sand meter. The speed at present (May 1947) is 31° to 32° .

C—HYDRAULIC WORKS

1. Limitation of model experiments and comparison of models and prototype.

C-1-1. Protecting the right bank of the Luni river at Balotra by means of a repelling spur.

(Advice without model experiments).

In consequence of the abnormal rainfall in the Aravali Ranges, the main line of the Jodhpur Railway between Parlu & Gole was severely damaged. The river spilled over the banks between Balotra & Tilwara causing extensive breaches. In addition to other recommendations for the protection of the Railway embankment,⁽¹⁾ to prevent attack at Balotra a *repelling spur* was recommended in 1944. Since then, the floods in the river were very low and hence the spur has not been tested under high flood conditions; but even with continued low floods the spur, according to the local officers, is working satisfactorily. The main channel of the river, which—even when the rainfall was normal—used to run along the side of the spur, has been repelled by the spur and now flows at a distance of 300' from it.

Fig. 1

C-1-1

C-1-2. Scour below the Central Sluices of the Mahanadi Anicut at Cuttack—Orissa.

Figs. 2 & 3

C-1-2

Figures 2 & 3 show the scour patterns below the Central Sluices of the Mahanadi Anicut, respectively before & after the modifications suggested by the Station were carried out at site⁽²⁾. Figure 4 shows the comparative section ABCD of deep scour; it also shows the deepest bed cross section after modifications which does not wholly coincide with that before the modifications.

Fig 4

C-1-2

It will be seen that, while under the old conditions, the deep scour-hole (down to R. L. 25) was almost abutting the pavement with imminent danger of undermining & collapse of the pavement, after the modifications there has been silting at this point to the extent of 20 ft.

The deepest bed contour, 263' away from the dwarf wall No. 4, was previously R. L. 15; while after the modifications it has accreted by 20' or more.

Further, at the left corner between walls 3 & 4, the apron had collapsed leaving deep scour-hole, but after the modifications—See R. L. 50 contour—at this point the bed has silted by more than 5 feet.

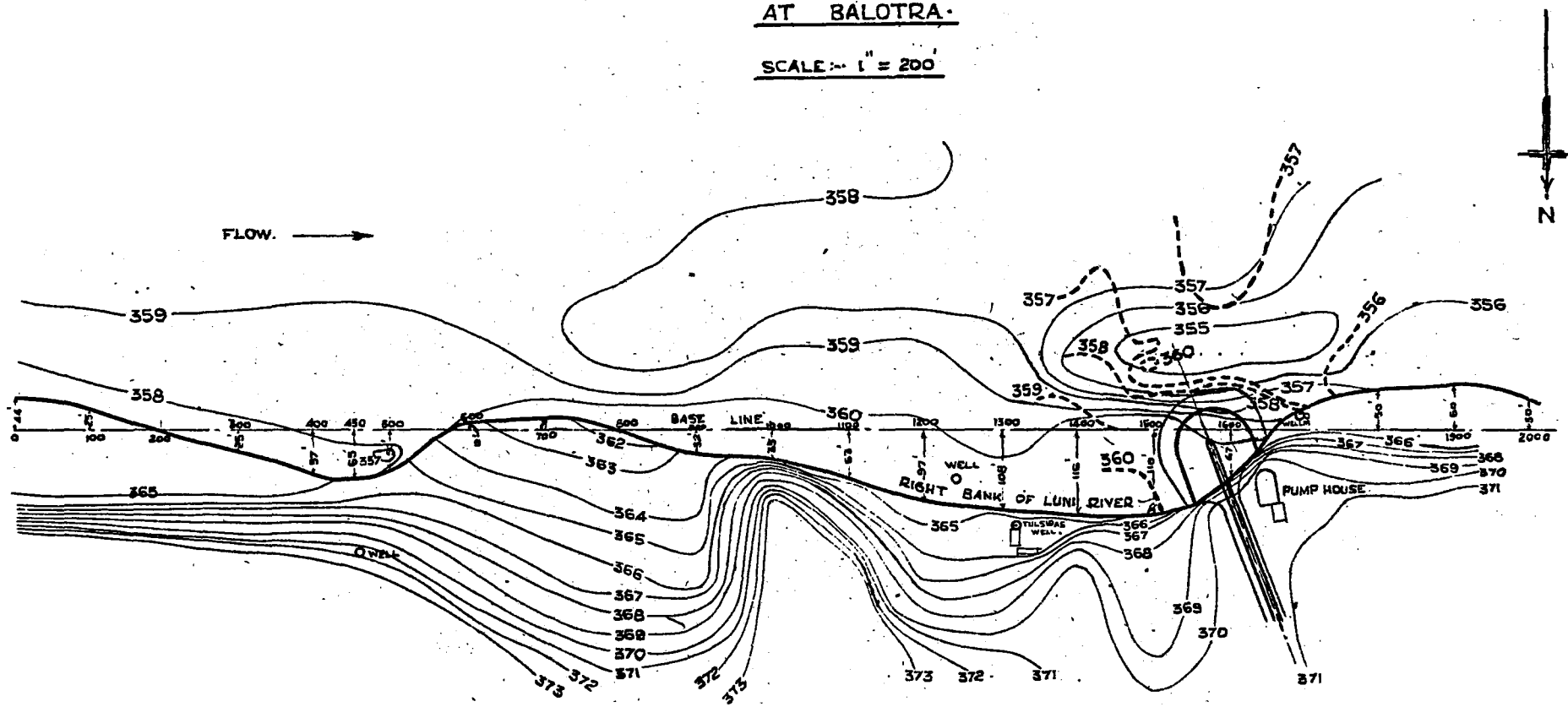
It is clear that the conditions have markedly improved since the construction of the energy dissipating device recommended by this Station.

(1) A. R. T. 1945 (item no. 12).

(2) A. R. T. for 1946 P. 5 & 27.

JODHPUR RLY
CONTOUR PLAN OF LUNI RIVER
NEAR PUMP HOUSE
AT BALOTRA.
SCALE: 1" = 200'

FIG. 1/C-1-1.





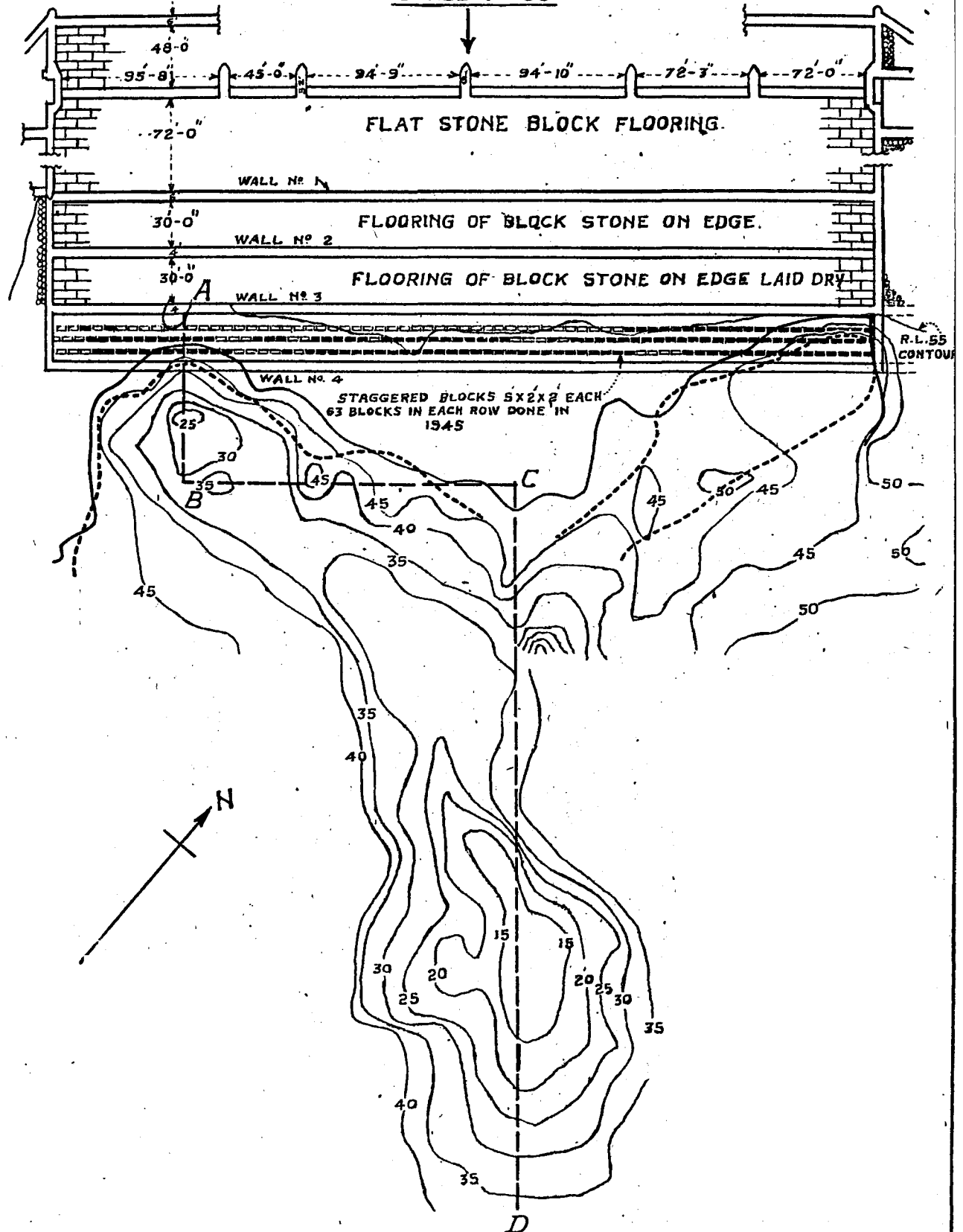
REF. :-
 CONTOURS AFTER 1944 FLOOD. 
 " " 1946 " 

FIG. 2/C-1-2
PLAN OF CENTRE SLUICES OF
THE MAHANADI ANICUT
SHOWING THE SCOUR AFTER FLOOD OF
1945
BEFORE MODIFICATIONS.

SCALE = 1" = 80'



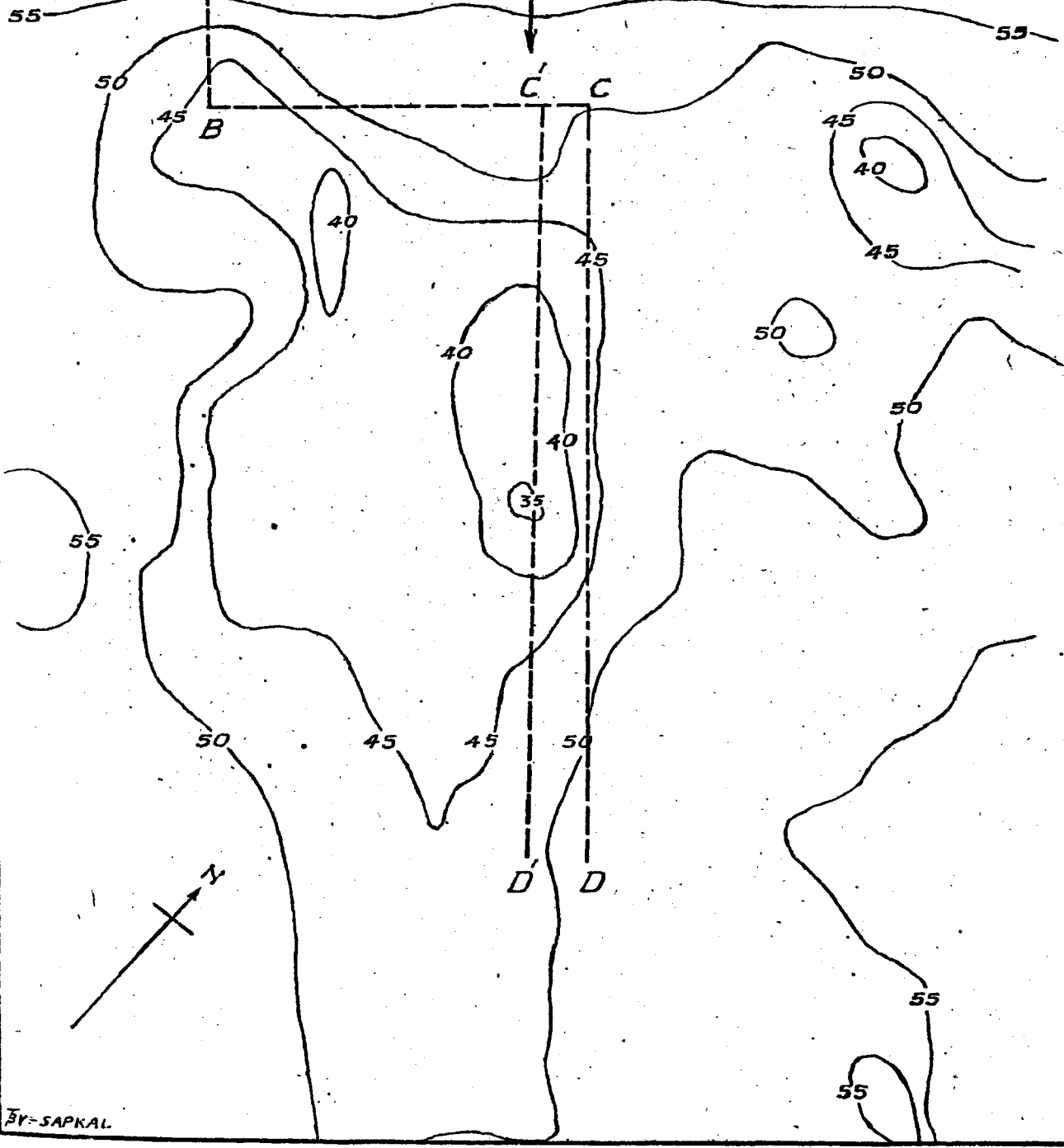
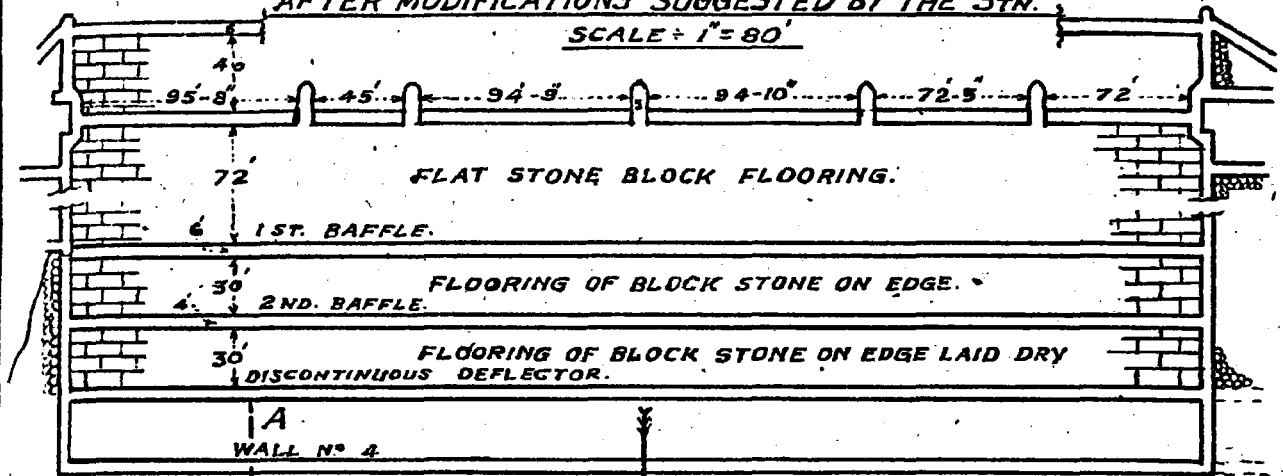
Staggered blocks disturbed after 1945 flood ■
Scour line after 1944 flood - - - - -
" " 1945 " - · - · - ·

FIG. 3/C-1-2.

SHOWING THE BED SCOUR BELOW THE CENTRE
SLUICES OF THE MAHANADI ANICUT
AFTER FLOODS OF 1947

AFTER MODIFICATIONS SUGGESTED BY THE STN.

SCALE = 1" = 80'



BY SARKAL

WATER LEVEL 62.00

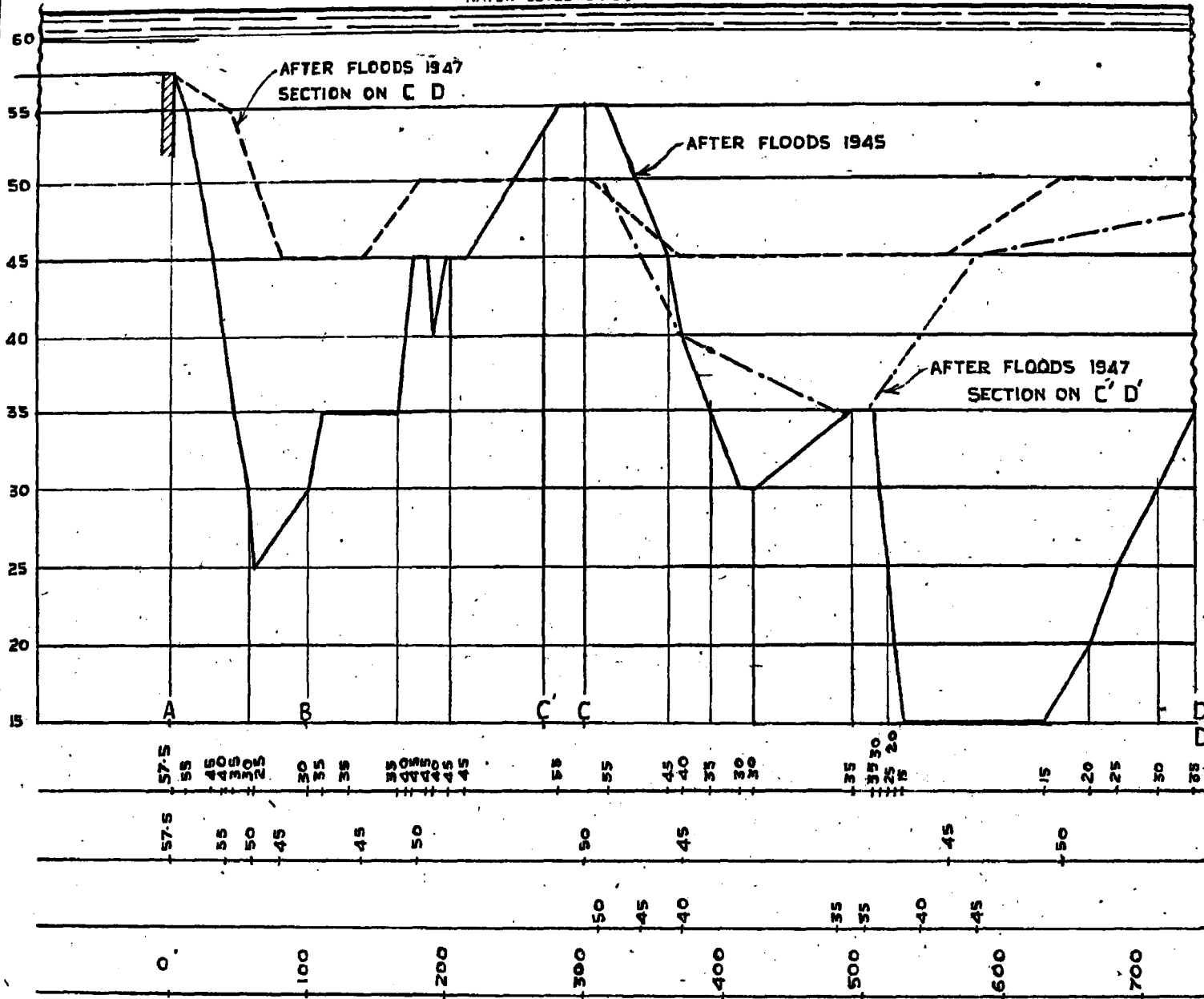


FIG. 4/C-1-2.
SHOWING SCOUR BELOW CENTRAL SLICES OF THE MAHANADI ANICUT AT CUTTACK.
FOR LOCATION OF C.S. A B C & D PLEASE SEE FIG. 2 & 3/C-1-2.

SCALE:- HOR. 1"=100'
VER. 1"=10'

- Ref.
- C.S. ABC and D before modification.
 - - - C.S. ABC and D after modification.
 - · - · C.S. ABC and D after modification showing deepest scour.

R.Ls. AFTER FLOODS OF 1945.

R.Ls. AFTER FLOOD OF 1947.

R.Ls. AFTER MODIFICATION OF 1947.

DISTANCE IN FEET.

C-HYDRAULIC WORKS

2. River Control

(i) FLOOD PROTECTION WORK.

C-2 (i)-1. Redistribution of sand load between the Mahanadi & Katjuri systems at Cuttack—Orissa.

1. *Introduction*.—The Katjuri river takes off from the Mahanadi just below the Naraj gorge, 5 miles upstream of Cuttack. At peak flood 56% (8.4 lakhs cusecs) discharge flows down the Katjuri at Naraj, whereas the rest of the discharge flows down the Mahanadi. In addition to the two spills, namely Birnasi Spill ($Q = 70,000$ cusecs at peak flood) on the left bank and Banki Road spill ($Q = 1,00,000$ cusecs at peak flood) on the right bank, the Katjuri bifurcates into Kuakhai, Surua and Katjuri loop in the neighbourhood of Cuttack.

As already mentioned in Annual Report (Technical) 1945⁽¹⁾, the fundamental source of trouble has been the excess sand charge entering the Katjuri river at Naraj. A series of experiments done with a view to improve the distribution of sand load between the Mahanadi and Katjuri river have been already reported⁽²⁾.

2. Experiments.

In continuation of those experiments the following experiments were carried out:—

Series II

Experiment 2 : (Repeated) With Banki Road spill discharge at peak flood reduced to 33,000 cusecs, Birnasi gap fully open and Naraj waterway reduced by 16.85%.

Experiment 4 : (Repeated) With Banki Road spill discharge at peak flood reduced to 33,000 cusecs, Birnasi gap fully closed by embankment ending in a spur and Naraj weir waterway reduced by 28.5%.

Experiment 5 : With Banki Road spill discharge reduced to 50,000 cusecs at peak flood; Birnasi gap closed by embankment ending in a spur but provided with high level escapes at Birnasi spill channel crossings; and Naraj weir waterway reduced by 20%.

In all the experiments the procedure followed was identical so as to enable comparison of results. In experiment 5, the high level openings in the embankment were so adjusted that with the Naraj weir waterway reduced by 20%, it gave no afflux at Kazipatana or Collectorate gauges. The sill level of the openings was kept at R. L. 82.00.

With H. F. level in the river below the weir at Naraj (R. L. 90) and at Bellevue gauge (R. L. 82.0), at the offtake of the Birnasi channel, about midway between them, the H. F. level will be about R. L. 86.0 and the river bed is at R. L. 75. This gives a flow depth of about 11 ft. With the high level openings in the Birnasi embankment at R. L. 82, it will be seen that the top one-third the depth of relatively sand-free water was admitted into the Mahanadi through the Birnasi gap.

Due to reduced waterway of the Naraj weir in experiment 5, about 85,000 cusecs, which otherwise would have flowed into the Katjuri below Naraj, were diverted into the Mahanadi⁽³⁾.

3. *Discussion of Results*.—Table 1 shows, as before, the ratios of sand drawn by the Mahanadi and its branches at Cuttack, the time average discharge, relative sand charge etc. Table 1
C-2 (i)-1

(1) Annual Report Tech. 1945, item 2.

(2) Annual Report (Tech.) for 1946—pages 44 to 52.

(3) Fig. 6/II (V) of Annual Report 1946.

For ready reference results of Series I, expt. 1 with existing conditions and of series II, expt. 1 which gave optimum results in the series of experiments done last year are also reproduced.

The experiments 2 & 4 of series II which were repeated this year were continued for 4 more rising and falling flood cycles. It was observed that in both these experiments, the relative sand mix in Mahanadi become richer; and on the Katjuri system, though the Katjuri loop and the Surua were better off, yet the Kuakhai was not. It will be seen that, except in experiments 1 and 5 of series II, the relative sand charge of the Mahanadi was slightly worse than under existing conditions. Both the Katjuri loop and the Surua improved but the Kuakhai branch drew 6.45% more sand in expt. 1 and 9.83% more in expt. 5; and still more in expts 2 & 4.

4. *Conclusions*:—The results of expt. series II—i. e. with Banki spill Q reduced to 50,000 cusecs, Birnasi gap embankment having high level openings and Naraj waterway reduced by 20%—gave the desired results for all the branches except the Kuakhai and the difference to Kuakhai would be small.

A short embankment without obstructing the Birnasi gap channels, would possibly give the optimum solution, provided the Banki spill discharge is reduced to 50,000 cusecs and Naraj weir waterway reducee by 15.5%; because it would, without materially vitiating results of expt. 1, series II, give a water front to Cuttack for which the embankment is primarily intended.

5. *Further experiments*:—While these experiments were being carried out, a point arose for consideration whether the Khannagar spur—*vide* item C-2 (ii)-2 of this report—would still be necessary to protect the Khannagar Bank in view of the Katjuri embankment developing a channel along the Cuttack bank. These experiments, together with experiments to evolve the shortest Katjuri Embankment, were in progress at the end of the year.

Table 1 / C - 2 (i) - 1
Showing the ratios of sand drawn by the Mahanadi & branches and the time average discharge.

Details of experiment	Total quantity of sand collected in all branches.	MAHANADI			KATJURI LOOP			SURUA			KUAKHAL			Remarks.
		$\sum \frac{QT}{T}$	Q_s	X	$\sum \frac{QT}{T}$	Q_s	X	$\sum \frac{QT}{T}$	Q_s	X	$\sum \frac{QT}{T}$	Q_s	X	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
<i>Series I</i> Experiment 1 : with existing conditions.	14.000	1.390	6.000	0.820	0.769	3.000	0.740	1.526	4.000	0.498	0.516	1.000	0.370	<p>'(a)' Figures denote quantity of sand and the ratios observed in the model.</p> <p>'(b)' Figures denote quantity of sand and ratios after correction to make sand collected = sand injected.</p> <p>T = Time in minutes X = Charge in parts per 1000 by weight</p> <p>= Ratio of Q_s to $\sum \frac{QT}{T}$; where $\sum \frac{QT}{T}$ is the weighted mean discharge with respect to time computed by taking the sum of the products of each stage discharge and its time of duration divided by the total time of the flood cycle.</p>
<i>Series II</i> Experiment 1: Birnasi Gap fully open; Banki Road spill at peak flood reduced to 50,000 cusecs; Naraj Weir Waterway reduced by 15.5 %	(a) 15.000 (b) 14.000	1.5000	7.000 6.550	0.880 0.825	0.750	3.000 2.80	0.760 0.710	1.420 3.720	4.000 3.720	0.535 0.499	0.450	1.000 0.934	0.423 0.394	
Experiment 2 (Repeated) : with Birnasi Gap fully open; Banki Road spill Q at peak reduced to 33,000 cusecs; Naraj Weir Waterway reduced by 16.85 %.	(a) 16.000 (b) 14.000	1.58	8.000 7.000	0.962 0.842	0.770	3.250 2.850	0.802 0.702	1.320 3.060	3.500 3.060	0.504 0.440	0.490	1.250 1.09	0.485 0.424	
Experiment 4 (Repeated) : with Birnasi gap closed; Banki Road spill at peak flood reduced to 33,000 cusecs; and Naraj weir waterway reduced by 28.5 %	(a) 16.25 (b) 14.000	1.60	8.50 7.32	1.01 0.869	0.720	3.25 2.80	0.858 0.738	1.410 2.83	3.250 2.83	0.438 0.376	0.44	1.25 1.08	0.540 0.465	
Experiment 5 : with Birnasi embankment ending in spur and provided with high level escapes; Banki Road spill Q reduced to 50,000 cusecs at peak flood and Naraj Weir Waterway reduced by 20 %.	(a) 16.045 (b) 14.000	1.55	7.75 6.77	0.95 0.830	0.730	3.25 2.84	0.845 0.737	1.43 3.42	3.92 3.42	0.521 0.455	0.46	1.125 0.986	0.465 0.405	

C. HYDRAULIC WORKS

2. River Control

(ii) - River Training works

C - 2 (ii) - 1. Protecting the right bank of the Kuakhai River above the B. N. Railway bridge Near Cuttack (Orissa).

1. Introduction :—

1. 1. Fig. 1 shows the Kuakhai which takes off from the Katjuri river on the southern side of Cuttack. The B. N. Railway crosses the Mahanadi, the Katjuri below Kuakhai head, and the Kuakhai. These railway bridges are not provided with guide banks. Upstream of the Kuakhai head, the Katjuri spills over a vast stretch of low area known as the Banki Road spill. This flood spill begins to flow when the total Mahanadi flood above Naraj is about 10 lakhs cusecs and finds its way mainly into the Kuakhai above the Railway bridge and to a very small extent flows downstream of the railway embankment through culverts. Exact discharge figures of the relative proportion of the discharge spilling into the Kuakhai above the bridge and flowing downstream of the railway embankment through the culvert are not available. It is, however, learnt that almost all the spill water of the Banki Road spill discharges into the Kuakhai above the railway bridge.

Fig. 1
C-2 (ii)-1

1. 2. The Orissa P. W. D. had observed the discharges of the Kuakhai at the railway bridge in 1939 and 1940, when the floods of the parent Mahanadi had risen to 13.61 lakhs cusecs and 14.545 lakhs cusecs, respectively. Based on these, the discharge of the Kuakhai, with a 15.5 lakhs flood in Mahanadi works out to 2.66 lakhs cusecs, which is the same as indicated by the Mahanadi model. Rhind had computed discharge figures of the various branches of the rivers for the 1872 flood conditions, but these figures may not now be applicable (a) because of the various 1872 breaches in the embankments in the Naraj gorge, and also at the head of the Katjuri; (b) because the Barang Nala taking off from the right bank below Naraj no longer exists; and (c) because of the aggradation of bed levels and changes in channel sections that have occurred since.

1. 3. Though the history of the scour along the right bank of Kuakhai, upstream of the railway bridge, is not available, yet it appears that the trouble is a chronic one and was met in the past by providing short term protection in the shape of stone pitching as and when required. There now exist two discontinuous pitched banks A and B—A being at right angles to the railway bridge and the second curved bank B projecting beyond the normal alignment of a guide Bank with a gap in between the two, probably to admit water from the small nala. This nala, as seen from the topo sheet, has no regular catchment and appears to have formed by the scour caused by the Banki spill waters. In recent years there has been serious erosion at the upstream end of the curved, detached bank B of the stone protection; and the railway officers, being apprehensive of danger to the embankment, referred the case to the station for advice.

Fig. 2.
& Photo
5166
C-2 (ii)-1

2. The model:—

2. 1. After extending the Katjuri on its right bank to the railway embankment, the existing, 1/400:1/66 vertically-exaggerated model of the Mahanadi and its branches was used for these experiments. This model was a ready asset in the speedy solution of the problem, because the model had correct entry and exit conditions.

2. 2. At the beginning of each experiment, the model was relaid to 1943 conditions and flood cycles rising from 6 lakhs to 15 lakhs and falling back to 6 lakhs above Naraj were repeated till stable conditions were obtained

3. Experiments and discussion of results.

3.1. Experiment 1 with existing conditions :—

In this experiment, the discontinuous stone pitchings on the right bank at the bridge were reproduced. The curved projecting portion of the pitching acted like an attracting spur as expected. It was originally considered that the Banki spill, flowing into the Kuakhai, might be the cause of eddies and turbulence and thus be responsible for the scour. Visual observations, however, indicated that the attack at the nose was severe when the Banki spill was not flowing. The "attracting" effect of the discontinuous portion of the projecting pitching is clearly seen in Fig. 2, where surface lines after the 2nd cycle were seen to have further approached the right bank than after the first cycle.

Fig. 2
C-2(ii)-1

3.2. Having reproduced the prototype conditions in a preliminary run, 'repelling' spurs 1200 ft. long (where river width = about 3000 ft.) with their shanks raised above H.F.L. and joined to the railway embankment were tested, 2500' to 4000' upstream of the bridge, but were not seen to be satisfactorily effective in diverting the flow away from the right bank at the bridge.

A set of two spurs, viz. one 'attracting' spur on the right bank and the other 'repelling spur' on the left bank at the offtake of Kuakhai, were also tested to centralise the flow through the bridge. The lengths of the spurs were varied from 400' to 1200'; but they, too, did not prove satisfactory in that the 'attracting' spur being on the convex (i.e. sanding) side did not act as an attracting spur, and, also, in that the conditions at the bridge did not improve.

3.3. In Experiment 2, therefore, a continuous stone pitched guide bank was tested.

Fig. 3 &
photo 5194
C-2(ii)-1

Length of the guide bank : Generally, according to the widely accepted practice, the length of a guide bank is made equal to the length of the bridge. In the present case, however, the railway bridge was considered to be much too long for the following reasons :—

Table 1 shows the data of the railway bridges across the Katjuri and the Kuakhai.

Table 1/C-2(ii)-1

	Katjuri Railway Bridge	Kuakhai Railway Bridge
1. Q (discharge in cusecs)	5.616 lakhs cusecs (extrapolated from observed figures of 1939 and 1940).	2.66 lakhs cusecs (extrapolated from observed figures of 1939 and 1940).
2. W = (Length of the bridge)	2700 ft.	3000 ft.
3. C in $w = c / \sqrt{Q}$	3.60	5.82
4. Lacey w i.e. $w = 2.67 \sqrt{Q}$	2000 ft.	1373 ft.

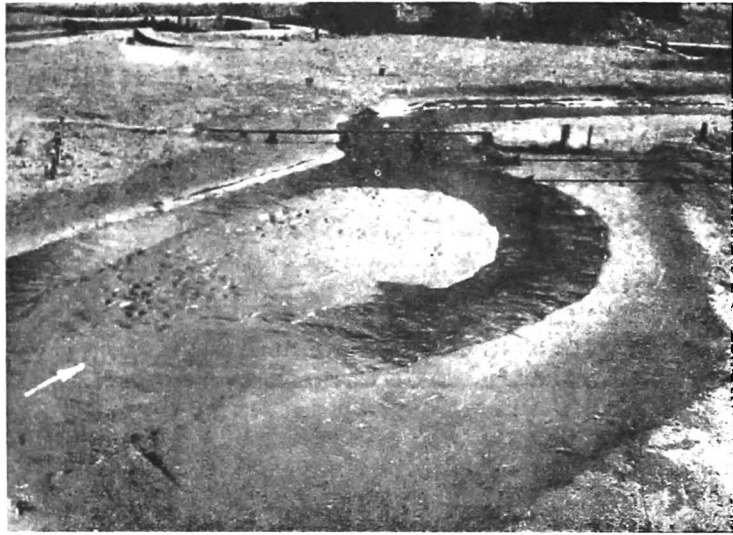
Comparing the 'c' values in item 3 of table 1, it is considered that, compared to the Katjuri Railway Bridge, the Kuakhai Railway Bridge is 1.62 $\left(= \frac{5.82}{3.60} \right)$ times too long for the discharge it has to pass.

Even allowing for the fact that the Kuakhai lies on the convex arm of the branch and would therefore be wider by virtue of "sanding", it would appear sufficient to assume the Kuakhai bridge 20% longer relatively, i.e. having a C value of 4.4 in $w = C \sqrt{Q}$ against 3.6 for the Katjuri and 5.82 for the existing, inordinately long Kuakhai bridge. The guide bank was thus kept 2270 ft. long.

The guide bank was connected to the railway embankment after rounding off its nose with 1000 ft. radius, 90° curve.

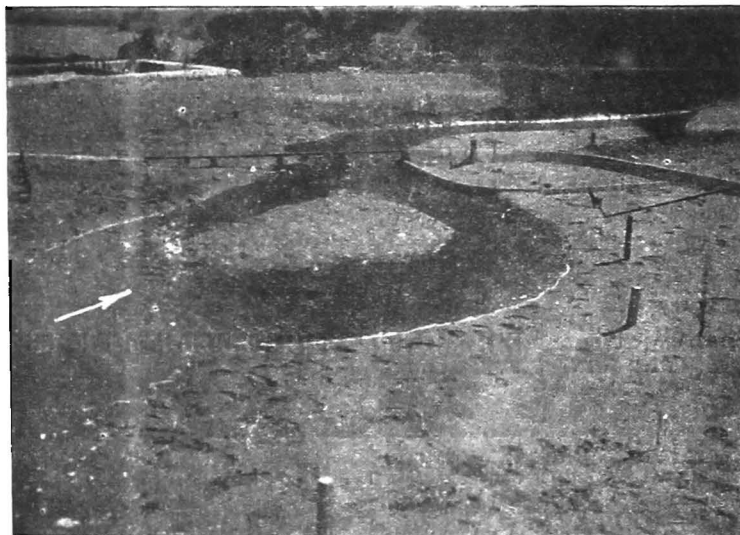
1/400 : 1/66 model of the Kuakhai River and the Railway Bridge at Cuttack—Orissa.

5166 : Photo looking downstream showing the right bank pitching as it exists in the river. Note that the u/s end of the pitched bank projects further into the river than pitching of the abutment.

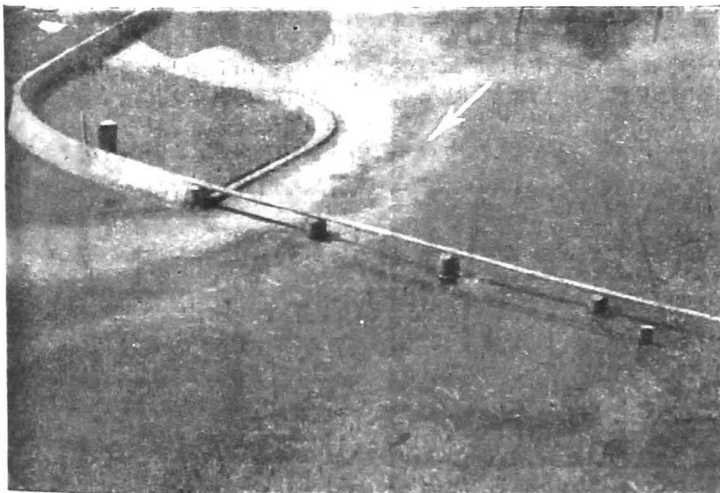


Pitched bank 'A' at abutment.
Pitched bank 'B'

5194 : Photo looking downstream showing the pitched guide bank as tested in expt. 2



Stone pitched guide bank.



5182 : Photo looking u/s showing the flow conditions with 12 lakhs cusecs flood. Note the Banki Road Spill discharge turning smoothly round the guide bank.

FIG. 2/C-2(ii)-1.

№ 6463
C.W.I.N.R.S. POONA

SHOWING SURFACE LINES OF FLOW NEAR
THE RIGHT BANK WITH A FLOW OF 6 LAKHS
IN THE MAHANADI ABOVE NARAJ IN EXPT. I.

SCALE 1" = 800'

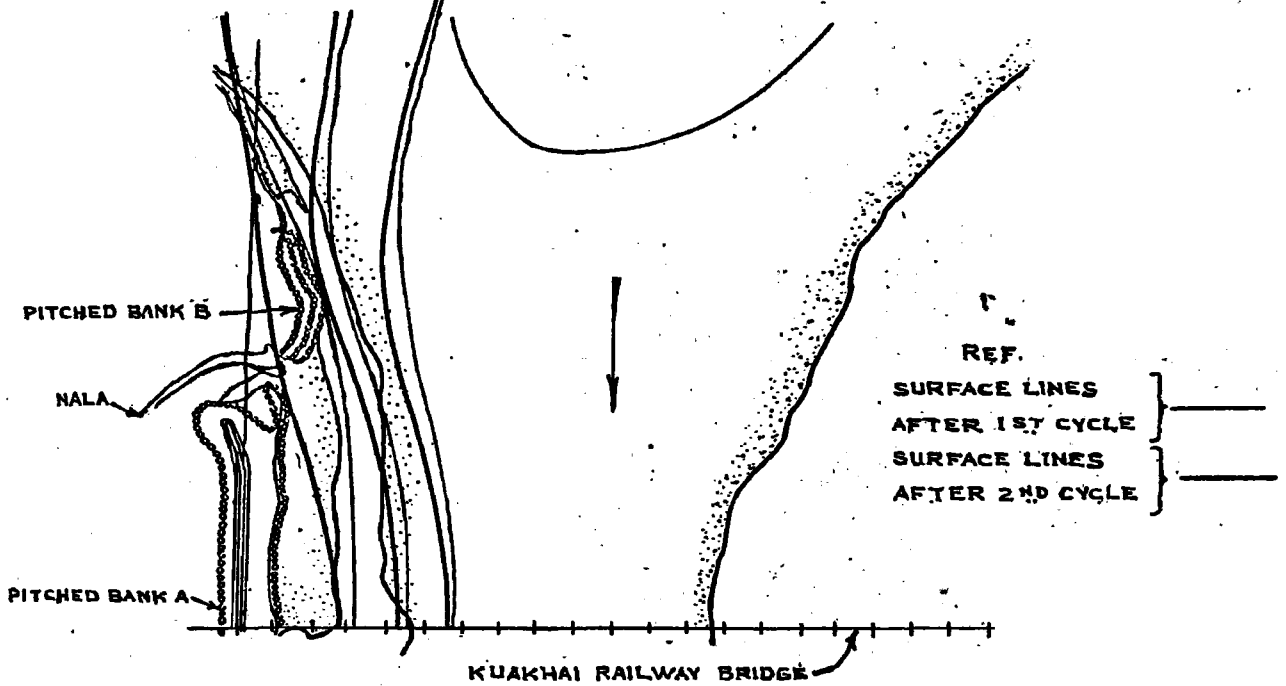
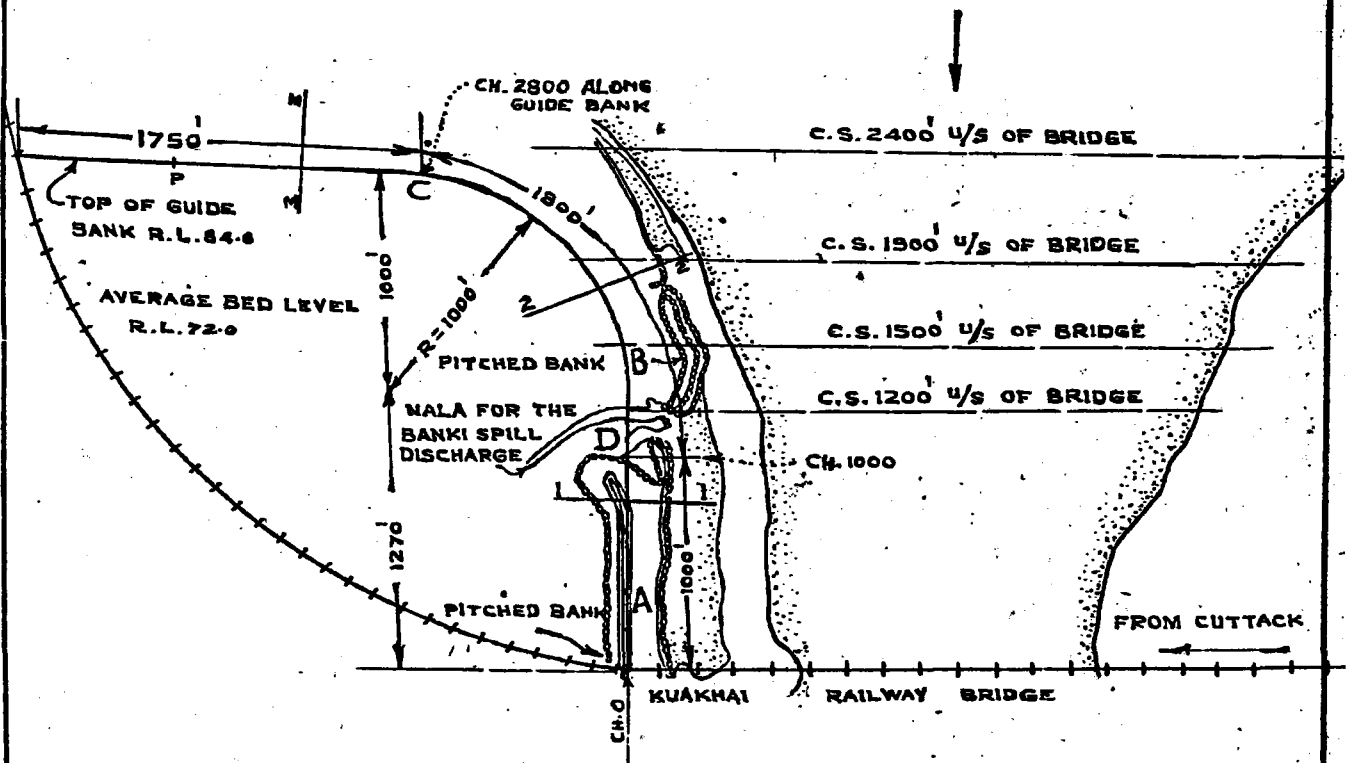


FIG. 3/C-2(ii)-1.

SHOWING THE ALIGNMENT & LENGTHS OF
THE PITCHED GUIDE BANK OF EXPT 2

SCALE 1" = 800'



3.4. *Flow conditions*:—Flow conditions with this guide bank were markedly better than under existing conditions.

Though with low floods, the flow hugged the guide bank, yet the high velocity filaments were away from it; and when the Banki Road spill began to flow with floods higher than about 10 lakhs (above Naraj), the spill discharge was smoothly guided along the shank of the guide bank joining the curved head to the Railway embankment. With the peak flood, the high velocity filament, which was near the curved head, gradually moved away from the guide bank so that at the bridge it was nearly 400' away from the guide bank. Fig. 4 shows the model cross sections, as obtained after experiments 1 and 2; superimposed on one another. It was observed that round the curved nose of the embankment, the scour was less than under existing conditions; because the flow was smoothly guided along the 1000 ft. radius curve. Between chainages 1000 ft. to about 1800 ft., however, though the scour depth remained the same yet a deep channel formed along the toe of the embankment. This, however, is quite natural due to the attracting effect of a guide bank; and with a properly designed falling apron, the condition will be well under control. It is for this reason that the reach between chainages 1000 and 2800 ft. is proposed to be provided with a launching apron sufficient for a scour depth of $2 D_{Lacey}$.

Fig. 4
C-2(ii)-1

4.0. *Design of the guide bank*—

4.1. As plenty of sand is available at site, the guide bank may be made with a sand core—width at the top = 6 ft. and having 3 ft. thick, one-man stone pitching on the side slopes.

Fig 5
C-2(ii)-1

4.2. As a measure of safety, it is essential to lay a launching apron sufficiently thick and long to launch and line scour-holes that may develop along the guide bank.

According to the model observations, scour-hole is expected to be deep in portion CD from Ch. 1000 ft. to Ch. 2800 ft. of the embankment, being shallower beyond chainage 2800. Hence, the launching apron may be provided to be adequate for a maximum depth of—

Fig. 3
C-2(ii)-1

$1.5 D_{Lacey}$ or 35.05', say 35', from chainage 0 to 1000

$2 D_{Lacey}$ or 46.74', say 47', from chainage 1000 to 2800

D_{Lacey} or 23.37', say 25', beyond chainage 2800

(where $D_{Lacey} = 0.47 (Q/f)^{1/3}$; $f = 1.76 \sqrt{m}$ and $Q = 2.66$ lakhs cusecs. and $f = 1.96$).

With H. F. L. 79.4, the deepest bed will be at

R. L. 44.4 between chainage 0 and 1000

R. L. 32.4 betwen ,, 1000 and 2800

R. L. 56.0 beyond ,, 2800

Table 2/C-2(ii)-1

Chainage from Bridge.	Average bed R. L.	Length along the sloping face of the scour-hole from R. L. in col. 2 to bottom (with 2 : 1 slope)	Quantity of stone required i.e. col. 3 x thickness of launched apron (= 3')	Size of apron required i.e. width x thickness allowing 10% for some stones being washed away etc.
1	2	3	4	5
0 to 1000	60.0	35 ft.	105 cft.	25' x 4.5'
1000 to 2800	70	84 ft.	252 cft.	35' x 8'
Beyond ch. 2800	72	35.7 ft. Say 36 ft.	108 cft.	25' x 4.5'

The aprons, as shown in table 2, when laid to the average bed level, the toe of the pitched embankment, should provide adequate protection to the guide bank.

4.3. The top of the apron on the high ground will be at a level higher than R.L. 72; whereas around the curved head, it will be 2' lower and that at chainage 1000' will be still lower. It should, however, be noted that the apron top should have a gradual slope along its length so as not to present sudden drops at the points of change-over of levels.

5.0. Double-acting sluice :

Unless the guide bank is given a designed section to withstand about 12' head on one side only, it will be necessary to provide for an adequate opening to fill in the compartment formed between the railway embankment and the guide bank.

5.1. The data for the design of such a two-way sluice are as follows :-

- (1) Area of compartment = 496×10^4 sq. ft.
- (2) Average bed R.L. = 72.0
- (3) H. F. L. to which it should be filled up = 79.4
- (4) Volume of water to be impounded in the compartment = $496 \times 10^4 \times (79.4 - 72.0)$
= 3.66×10^7 cft.
- (5) Average rise of water upstream of the guide bank = 2.81 ft. per day.
- (6) Average time in which the compartment should be filled = 2.78 days.

Working on this data, the average discharge the sluice has to pass so that the water level in the compartment will be the same as that on the upstream side of the guide bank, will be

$$= \frac{3.66 \times 10^7}{2.78 \times 24 \times 3600}$$

$$= 152 \text{ cusecs.}$$

The head available to pass this discharge varies from zero to 7.4 ft. (R.L. 79.4 - 72). Hence working on an average head of $7.4/2$ i.e. 3.7 ft. and assuming that under drowned conditions and a flat sill, the coefficient of discharge will be of the order of 2.0, the length of the opening required will be

$$= \frac{Q}{c \times D^{1.5}}$$

$$= \frac{152}{2 \times 3.7^{1.5}}$$

$$= 10.67 \text{ i.e. say 11 ft.}$$

As the discharge to be passed through the sluice is small, a simple masonry wall 5' to 6' deep with its top at the average bed R.L. and with its upstream and downstream sides pitched with 3 ft. thick stone pitching is expected to serve the conditions.

6.0 Conclusions :—

Under existing conditions, the erosion is due to the curved flow and is accentuated by the stone pitching which probably was not designed as a permanent measure. A guide bank, as suggested in the Note, appears to be a promising measure.

SHOWING CROSS SECTIONS OF THE KUAKHAI RIVER
ABOVE THE RLY BRIDGE

REFERENCES:-

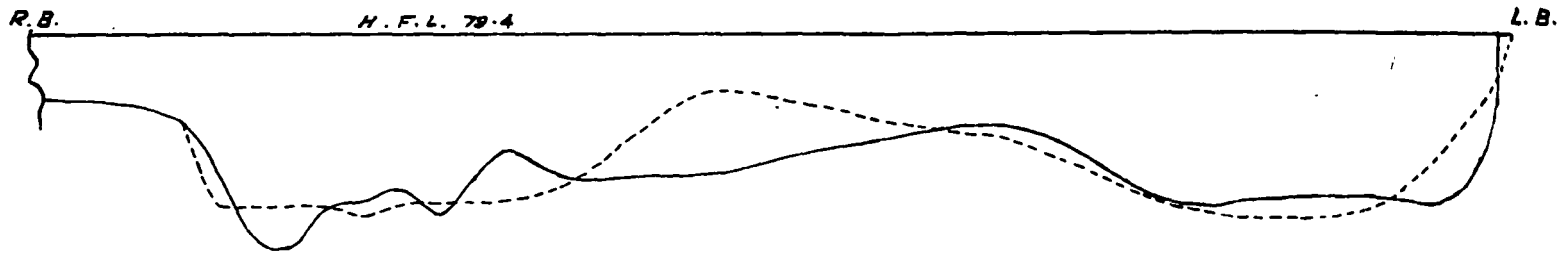
BED AS OBTAINED IN EXPT. 1. ———

BED AS OBTAINED IN EXPT. 2. - - - - -

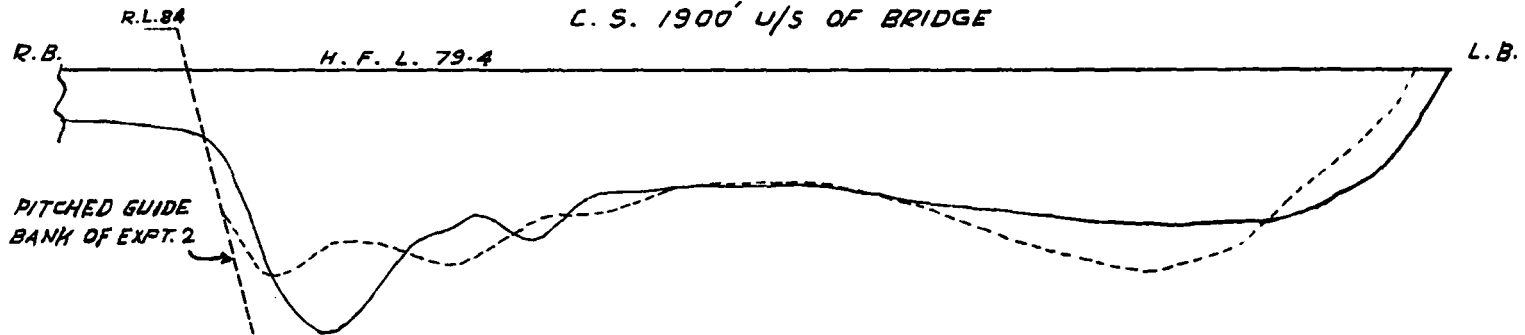
SCALE HOR. $i = 1$ IN MODEL
= 400' IN PROTO
VER. $i = 0.25$ IN MODEL
= 16.5' IN PROTO

(FOR LOCATION OF C.S. PL. SEE FIG. 3/C-2(ii)-1)

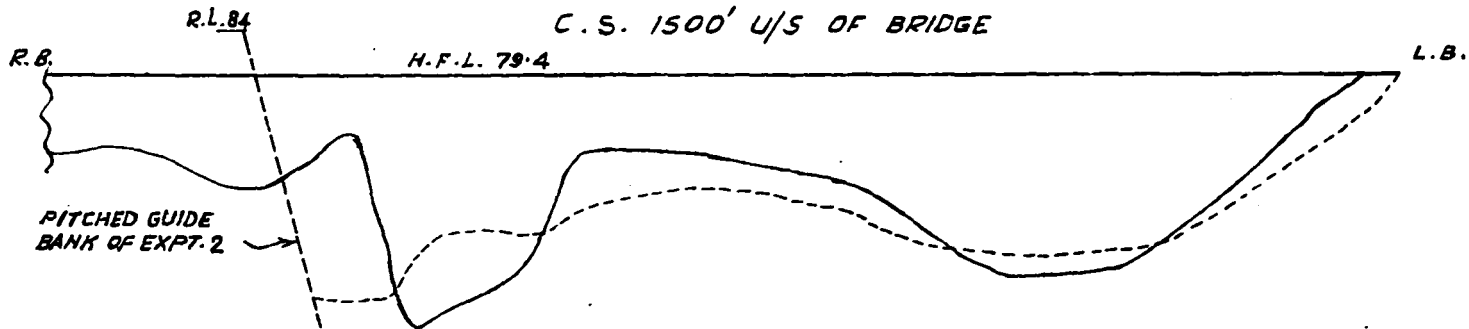
C.S. 2400' U/S OF BRIDGE



C.S. 1900' U/S OF BRIDGE



C.S. 1500' U/S OF BRIDGE



C.S. 1200' U/S OF BRIDGE

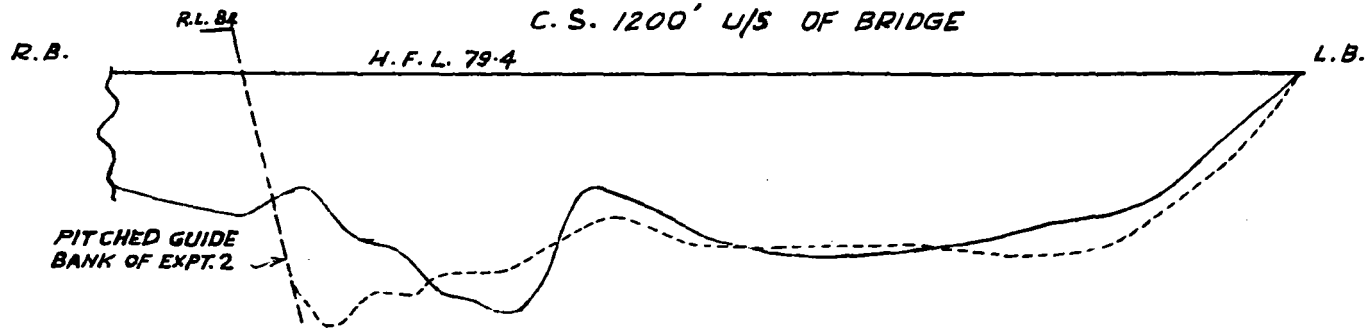


FIG. 5/C-2(ii)-1.

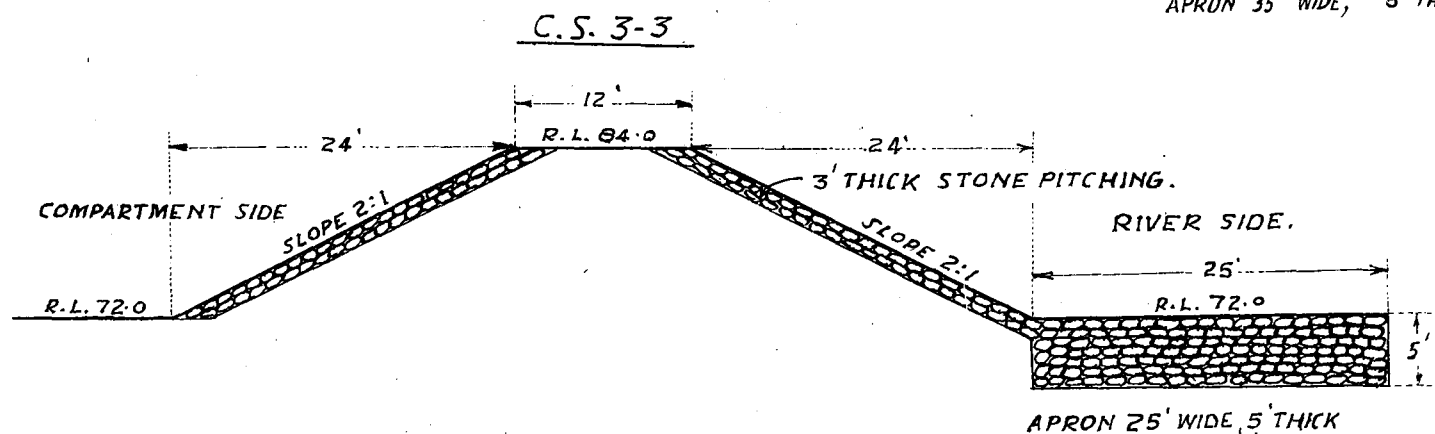
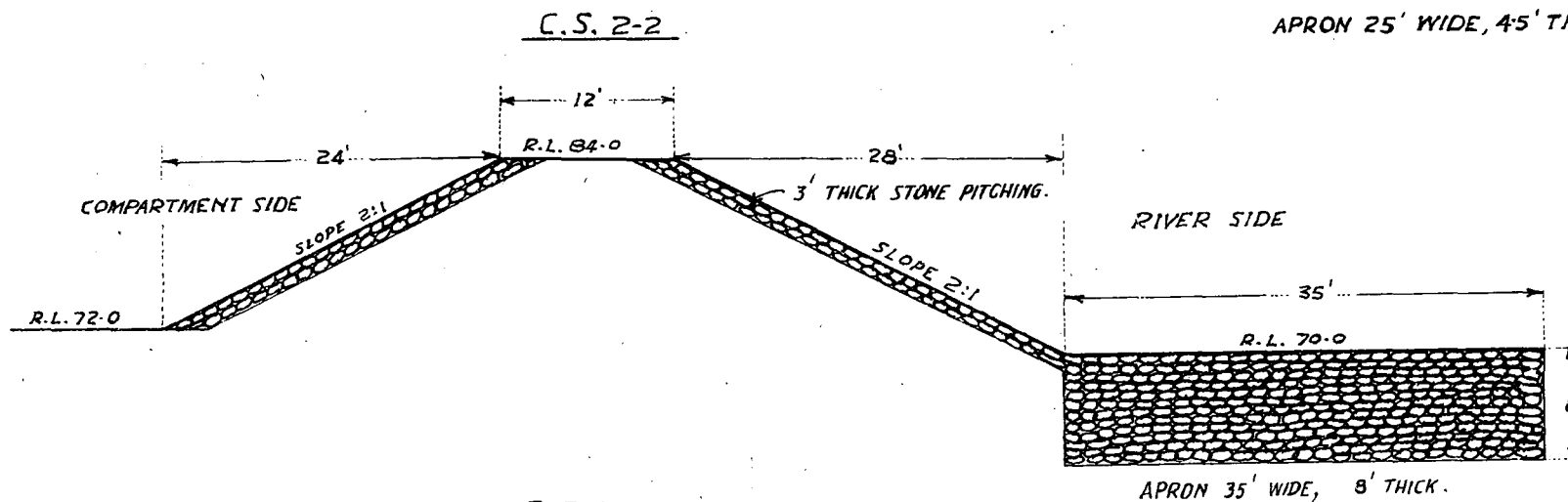
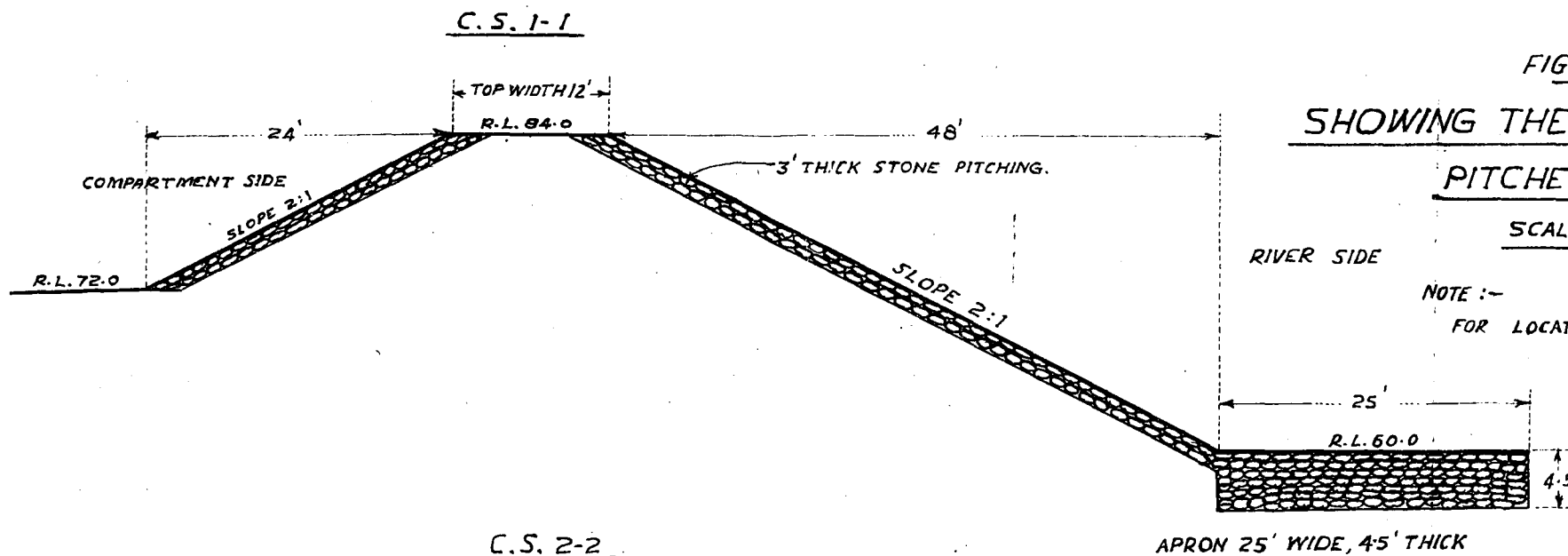
SHOWING THE CROSS SECTION OF
PITCHED GUIDE BANK

SCALE: 1" = 10'

NOTE :-

FOR LOCATION OF C.S. PLEASE

SEE FIG. NR 3/C-2(ii)-1.



C-2 (ii)-2. Protection of the left bank of the Katjuri River at Khannagar upstream of the Katjuri Railway Bridge—Orissa.

1. *Introduction:*—During the 1946 floods, chiefly owing to the sustained flood of about 12 lakhs cusecs above Naraj flowing for 5 days at or about a stage 2' below the maximum, the attack at the right bank of the Katjuri upstream of the Kuakhai head moved downstream causing concentration of flow which scoured the right bank at Kazipatana, just downstream of the Kuakhai off-take; the flow then was sharply deflected from the right bank at Kazipatana to the left bank near the Burning Ghat downstream of Cuttack. Further downstream, the flow hugged the left bank, causing severe damage between the Burning Ghat and the Khannagar revetment and also along the revetment for a length of 2500'. Extensive emergent protection work with sand bags, brush wood and temporary piles was done to prevent the embankment breaching.

1.1 Permanent protective works appeared necessary before the 1947 floods for which model tests were carried out.

1.2 When he inspected the site, the Director suggested modifications regarding the proposed retired embankment and also recommended a repelling groyne near the Burning Ghat; the optimum alignment and length of which were, however, to be determined by model tests.

2. *The model:*—These tests were carried out in the 1/400 : 1/66 vertically-exaggerated model of the Mahanadi and its branches which was being successfully operated to study the redistribution of sand and discharge between the Mahanadi and Katjuri at Cuttack. The entry and exit conditions having been already proved, this model was a ready asset in the quick solution of the new problem.

3. *Experiments :* The following qualitative tests were done to arrive at the best results:—

Experiment 1 : with existing conditions.

Experiment 2 : with 1200' long spur near the Burning Ghat, pointing 45° upstream.

Experiment 3 : with 1200' long spur near the Burning Ghat; pointing 60° upstream.

Experiment 4 : with 1200' long spur near the Burning Ghat, pointing 90° upstream, i. e. at right angles to the bank.

Fig 6
C-2 (ii)-2

Experiment 5 : with 1400' long spur near the Burning Ghat, pointing 60° upstream.

In each experiment a flood discharge of 6 lakhs cusecs above Naraj— which evidently gave bad conditions at Khannagar, specially with a spur, because the throw-off due to a spur increased with increased discharges (*vide para 4.3*)—, was run for 3 hours during which time observations of velocities, lines of flow and regions of scour and shoaling etc. were made.

3.1 Finally, the two most promising alternatives were tested under flood cycles rising from 6 lakhs cusecs to 15 lakhs and falling back to 6 lakhs cusecs (above Naraj).

4. *Discussion of results :* In preliminary tests, different lengths of spurs were tested at different sites. Spurs shorter than 1200' at and upstream of the Burning Ghat did not appreciably shift the high velocity flow from the Khannagar bank. Spurs sited further upstream than shown in Figure 6 were also inefficient because, after local deflection, the flow again swung back to the left bank at Khannagar. Shorter spurs might probably have worked fairly satisfactorily, if located further downstream; but this was not practicable due to great depth of water. The best position of a spur was found to be near the temple between cross section 7 and cross section 8, as had been suggested by the Director on site.

Fig. 6
C-2(ii)-2

Fig. 7
C-2 (ii)-2

4.1 *Velocities* : Figure 7 shows the velocity distribution—ratio of mean velocity on vertical to mean velocity of cross section observed at a C. S. 1000' upstream of and parallel to the Railway Bridge. In experiment 1 (existing conditions), the highest velocity was near the left bank, which gradually fell off to a minimum near the right bank. The effect of putting in a spur was to reduce the velocities at the left bank and increase those at the centre and a little near the right bank. Optimum normal distribution,—i. e. high in the mid-channel and low at the sides—was obtained with a 1400' long spur pointing 60° upstream. The next best distribution was obtained with a 1200' long spur pointing 60° upstream.

Fig. 8
C-2 (ii)-2

4.2 *Lines of flow* : Figure 8 shows the bed and surface lines of flow in experiments 1 to 5 observed with a flood discharge of 6 lakhs cusecs above Naraj. It will be seen that, under existing conditions the surface flow concentrated and dived near the Khannagar bank, the bed water being deflected away from the bank, indicating a likely tendency to undermine the bank.

With a 45° spur, the throw-off of the high velocity flow was not sufficient. The 60° spur gave a better throw-off of surface flow and the return flow downstream of the spur was also less severe. A 90° spur deflected the bed flow below it away from the left bank—which was undesirable for inducing silting and the surface flow throw-off was also less than with the 60° spur. The optimum results were, thus, obtained with the 60° spur.

4.3 Having observed the lines of flow and velocities, experiments 3 and 5 were repeated with cycles rising from 6 lakhs cusecs to 15 lakhs cusecs and falling back to 6 lakhs cusecs above Naraj to observe the flow conditions with higher flood discharges. An interesting common feature of both these experiments was that the throw-off due to the spur increased with increased discharge. It was also observed that with higher floods, though both 1400' and 1200' long spurs worked satisfactorily as regards surface flow, yet the bed flow tended to approach the left bank below the spur much earlier with the 1200' spur than with the 1400' long spur—a feature which indicated that with the 1200' long spur the bed near the left bank would build up more quickly.

The velocities near the right bank also increased causing a little scour of the extensive shoal near it.

Fig. 9
C-2 (ii)-2

4.4 Figure 9 shows the cross sections 9 and 10, observed after 3 hours' run of the 6 lakhs cusecs flood. C. S. 8 is too near the spur and is affected by the scour-hole at the nose of the spur and so, although observed, is of little value and is omitted.

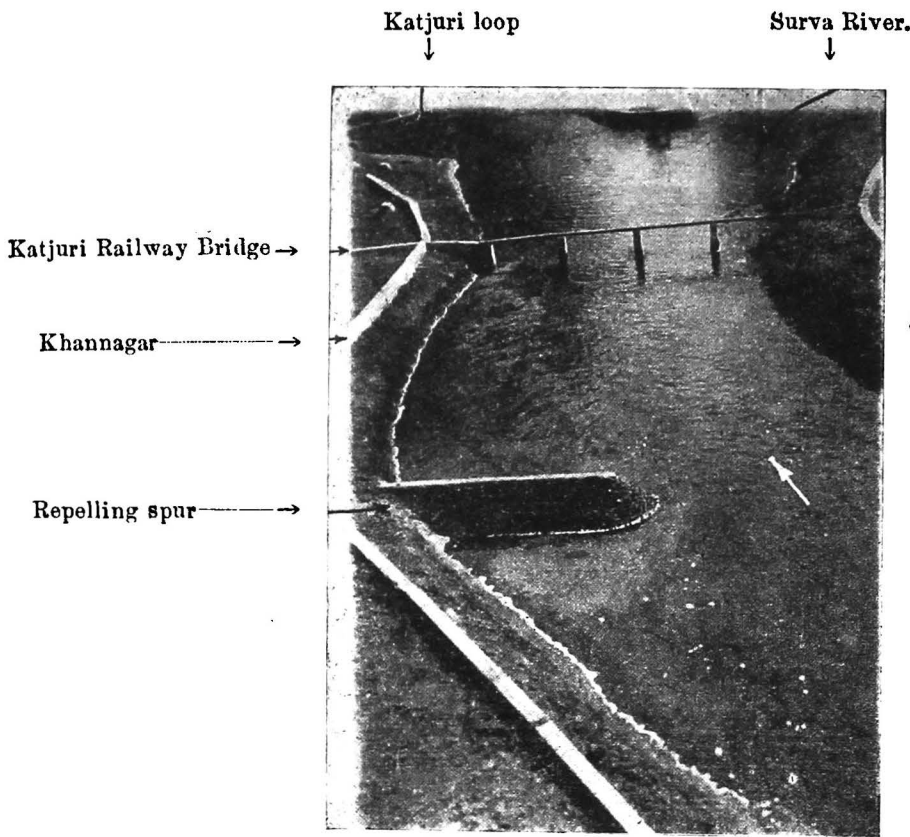
It will be seen that at C. S. 9 the deepest channel is at mid-stream with the 1200' spur pointing 60° upstream. This effect is not so pronounced, with this period of flow at C. S. 10 where the deepest portion is still near the left bank.

The fact that, inspite of the throw-off, a deep channel is observed near the left bank is mainly due to model limitations. A model has all its sand rolling on the bed and, unlike the prototype, has very little of suspended silt to deposit. Further, the period of run was also not sufficient for the model to attain its final configuration from bed sand movement alone, conforming to the conditions imposed upon it by the spurs. In the river, except in the vicinity of the revetment, the channel along the left bank must eventually silt up, consequent upon the surface flow throw-off of the spur, the inducement of the bed flow along the bank, and the deposition of suspended silt with lowered velocities along the bank.

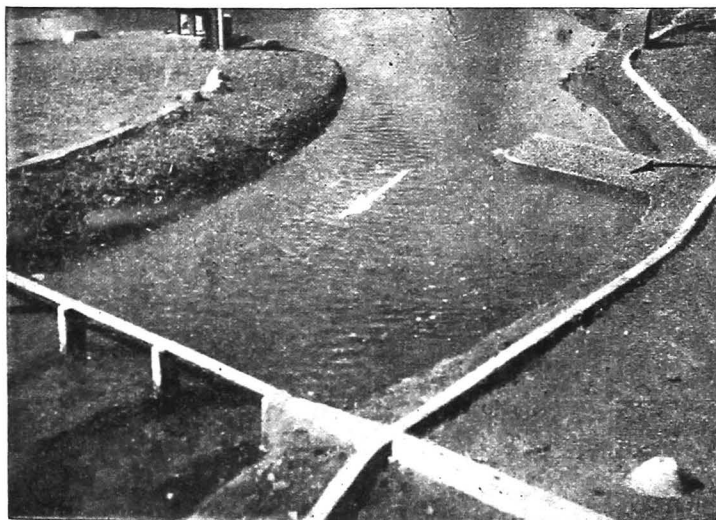
(5) *Scour at the nose of the spur ; design of launching apron.*

In order to get quick results the model spurs described above were made of a W. I. Sheet having a slope of $\cong 2\frac{1}{2}$ to 1 at the nose. The sides were vertical. The maximum scour depth obtained at the nose with this spur went

1/400 : 1/66 Model of the Katjuri river at Khannagar—Orissa.



5193 : Photo looking downstream, showing the flow conditions at Khannagar with 1200 ft. long repelling groyne, pointing 60° upstream.



←Repelling spur.

5192 : Same as 5193 but looking upstream.

CUTTACK

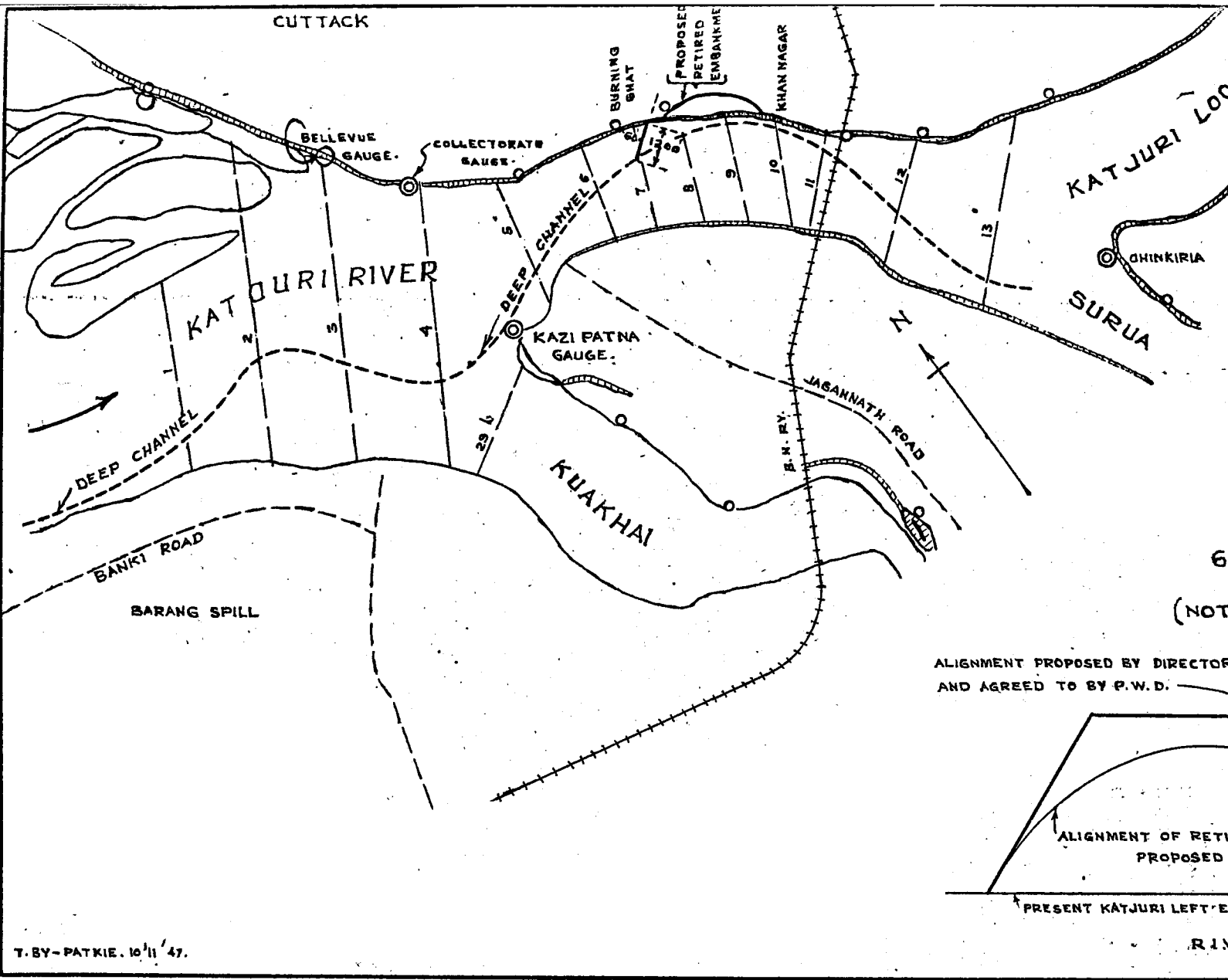


FIG. 6/C-2(ii)-2

SHOWING THE PROPOSED 1200 FT LONG REPELLING SPUR IN THE KATJURI RIVER

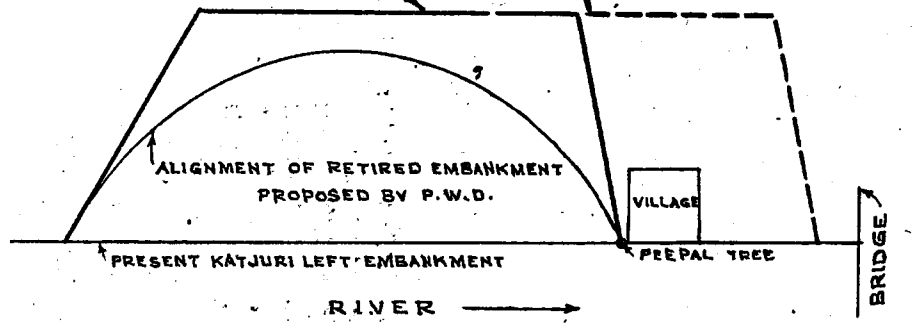
SCALE 1" = 4000 FT.

6(a).

(NOT TO SCALE)

ALIGNMENT PROPOSED BY DIRECTOR (I.W.EYPT. STN.) AND AGREED TO BY P.W.D. →

← FURTHER ALIGNMENT PROPOSED BY DIRECTOR (L.W.EYPT. STN.)



T. BY - PATKIE. 10/11/47.

FIG. 7/C -2(ii)-2.

SHOWING VELOCITY DISTRIBUTIONS AT
CS. 2.5' 4/3 OF KUAKHAI BRIDGE & PARALLEL TO IT.

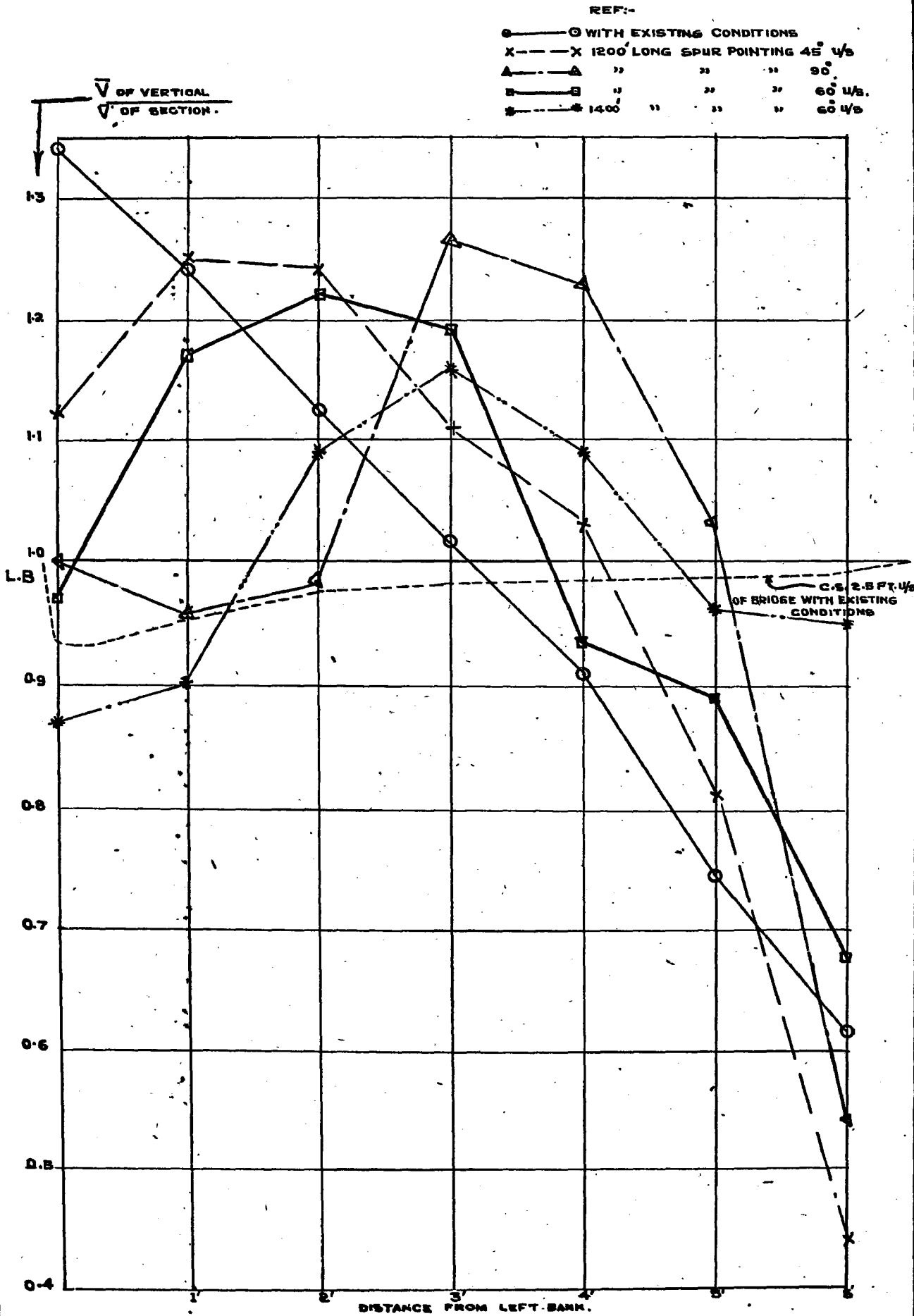


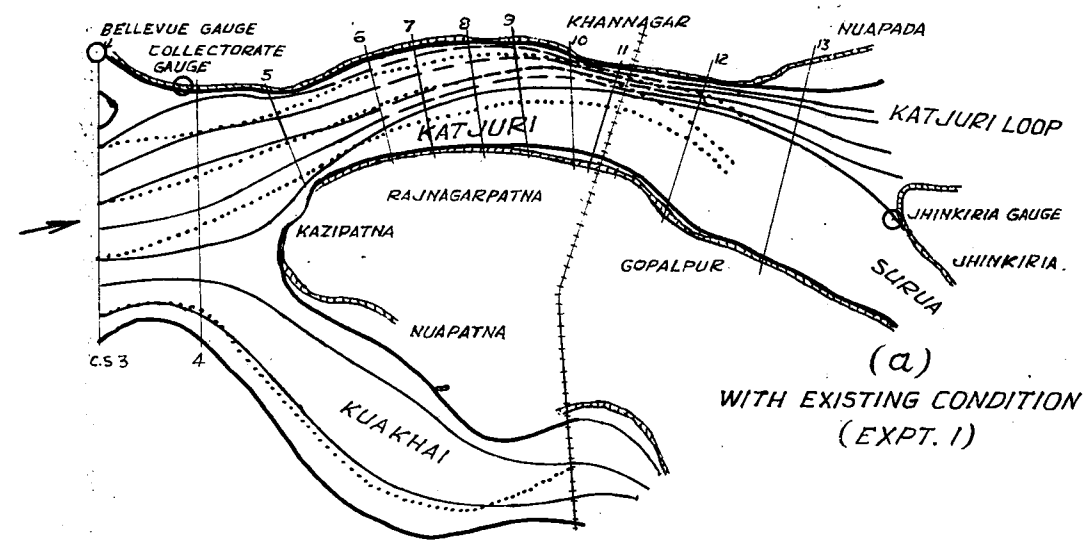
FIG. 8/C-2 (ii)-2

LINES OF FLOW IN THE KATJURI
BELOW KUAKHAI HEAD

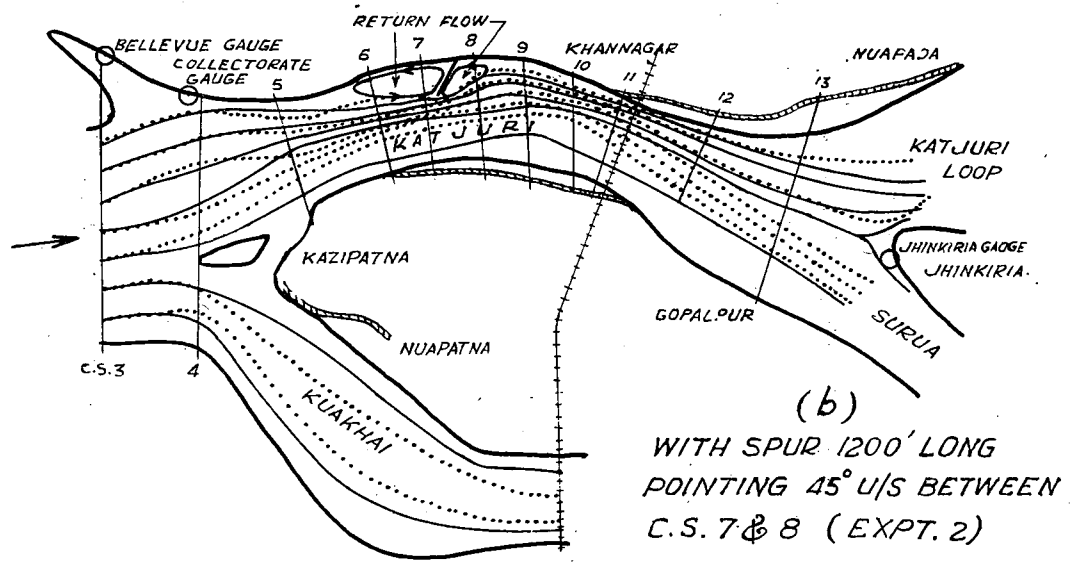
SCALE 1" = 4000' = 10' (MODEL)

REFERENCES:~

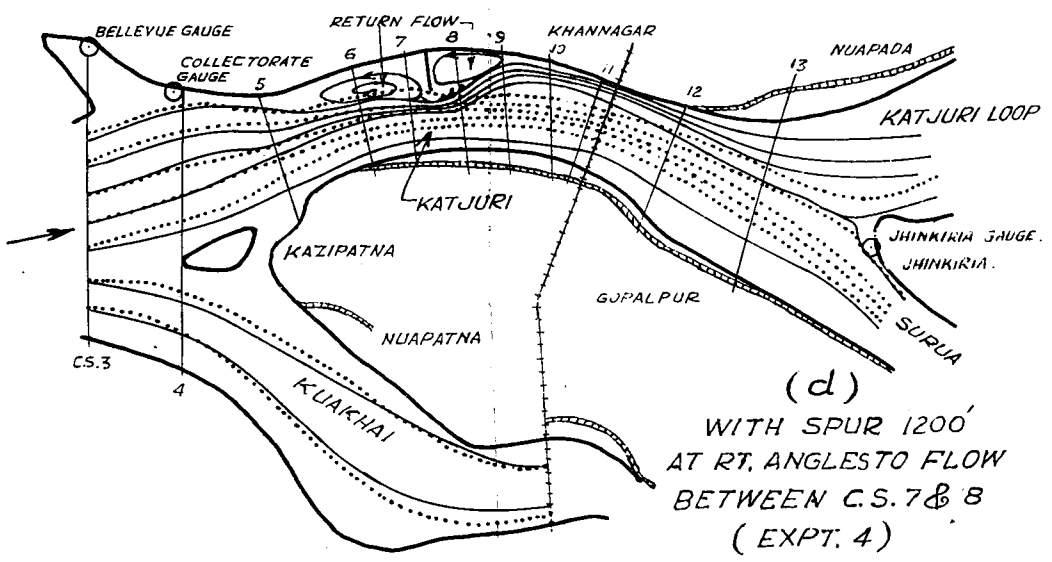
- BED LINES OF FLOW.
- SURFACE LINES OF FLOW.
- MID-DEPTH " " "
- WATER EDGE.



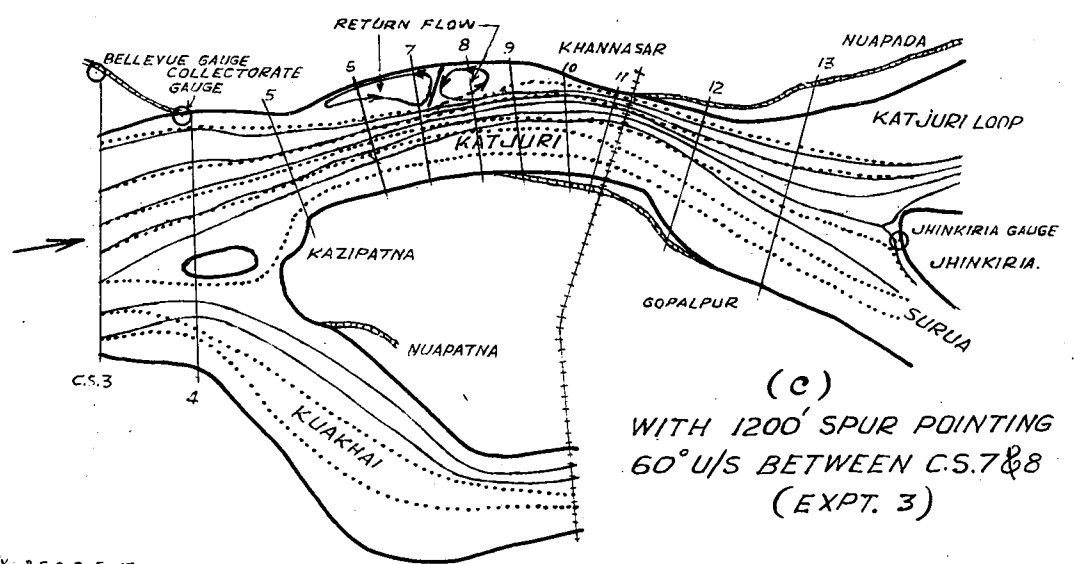
(a)
WITH EXISTING CONDITION
(EXPT. 1)



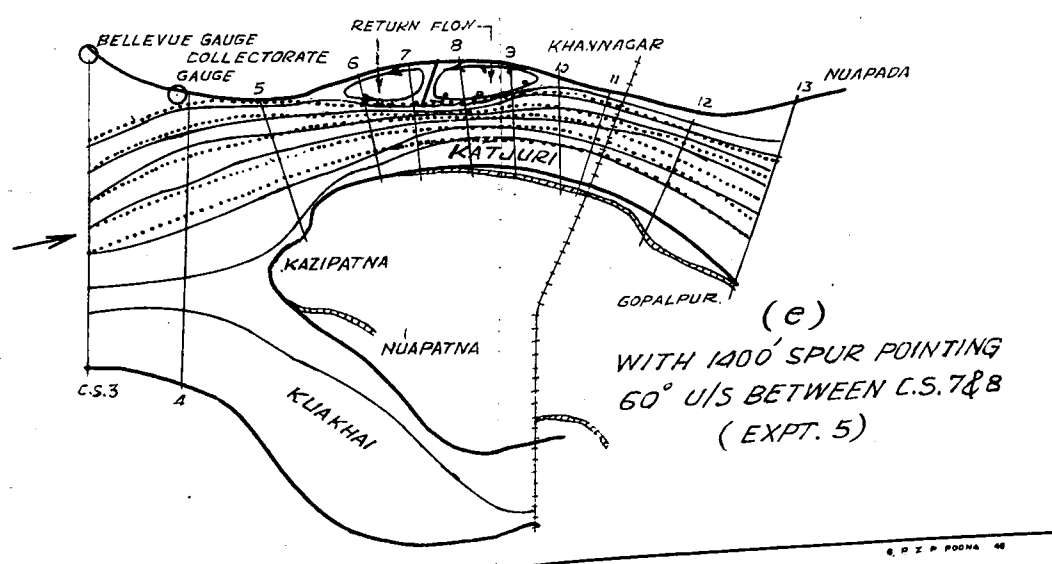
(b)
WITH SPUR 1200' LONG
POINTING 45° U/S BETWEEN
C.S. 7 & 8 (EXPT. 2)



(d)
WITH SPUR 1200'
AT RT. ANGLE TO FLOW
BETWEEN C.S. 7 & 8
(EXPT. 4)



(c)
WITH 1200' SPUR POINTING
60° U/S BETWEEN C.S. 7 & 8
(EXPT. 3)



(e)
WITH 1400' SPUR POINTING
60° U/S BETWEEN C.S. 7 & 8
(EXPT. 5)

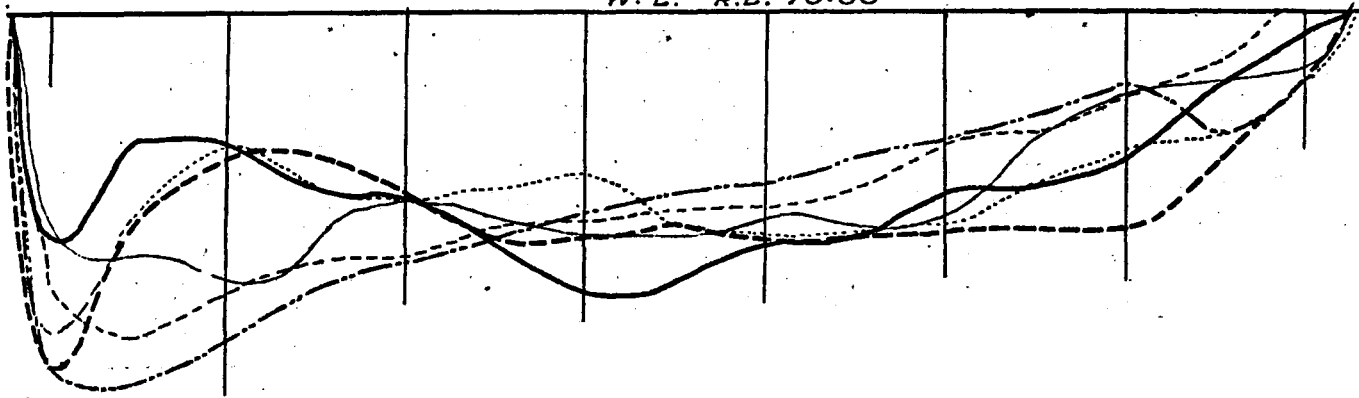
FIG. 9/C-2(ii)-2.

SCALE.
HORIZONTAL 1"=100'
VERTICAL 1"=0.25'
R.B.

DIST. FROM
BASELINE L. B.

4' 5' 6' 7' 8' 9' 10' 11'

W. L. R. L. 70.00

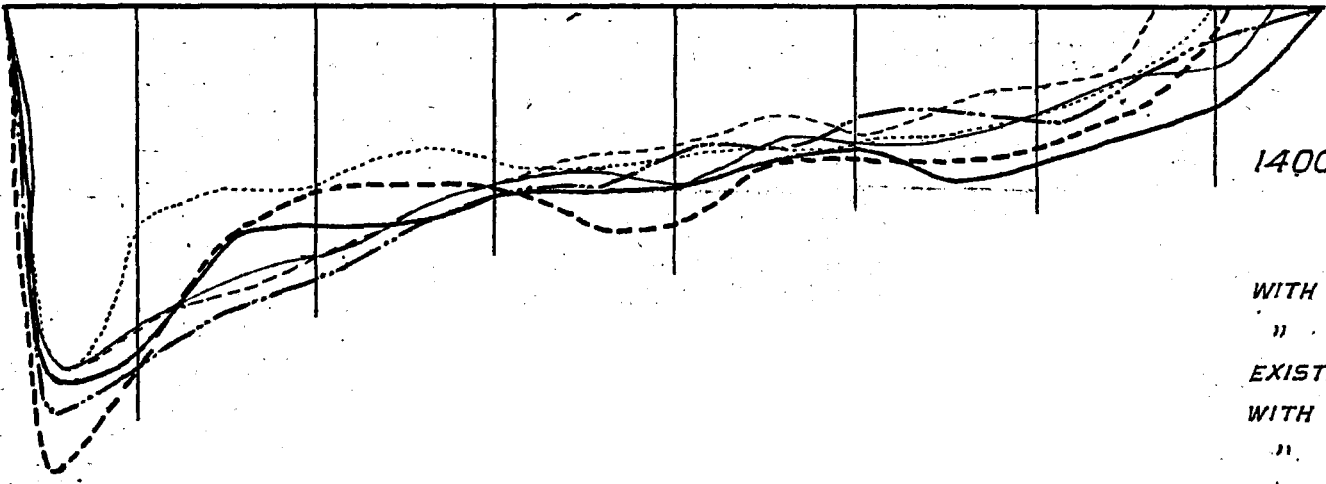


C. S. 9
2200' U/S OF THE KATJURI
Rly. BRIDGE.

DIST. FROM
BASELINE L. B.

7' 8' 9' 10' 11' 12' 13' 14'

W. L. R. L. 70.00



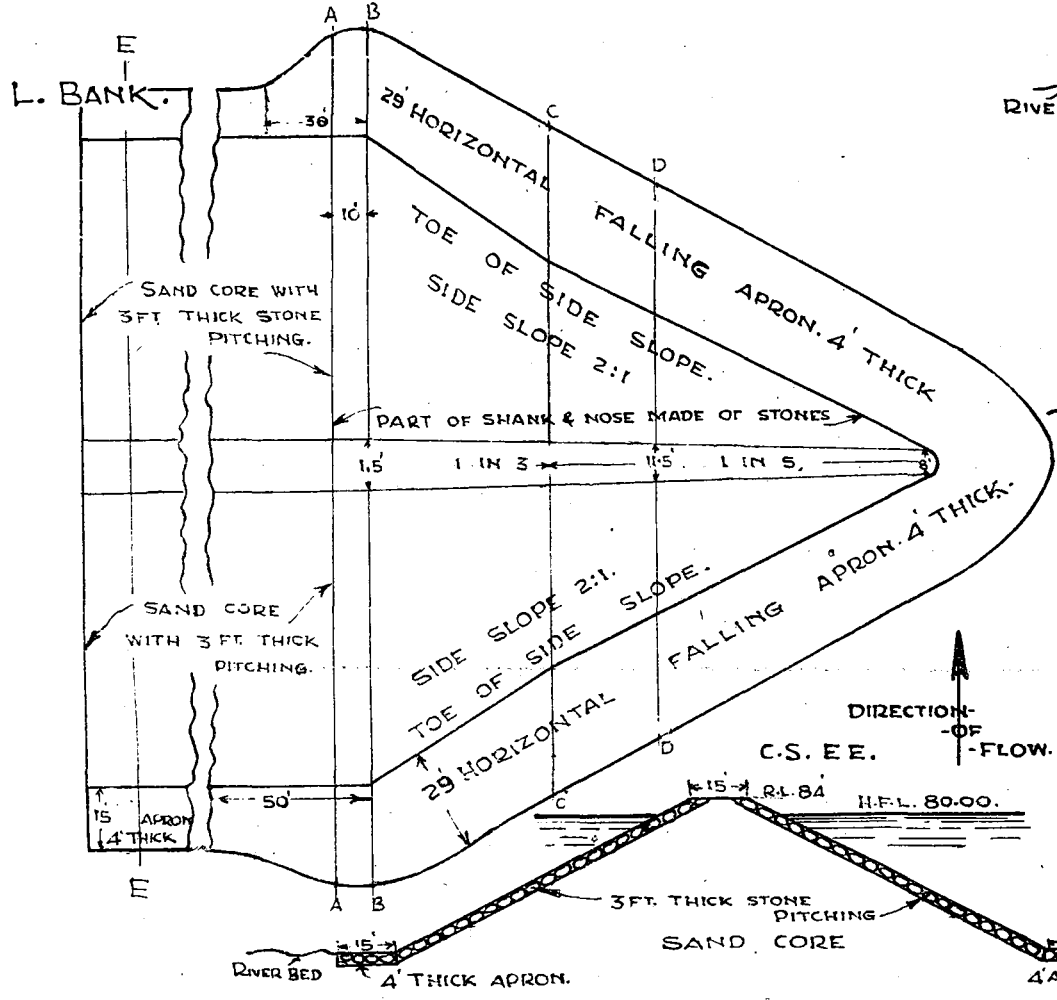
C. S. 10
1400' U/S OF THE KATJURI Rly. BRIDGE.

REFERENCE

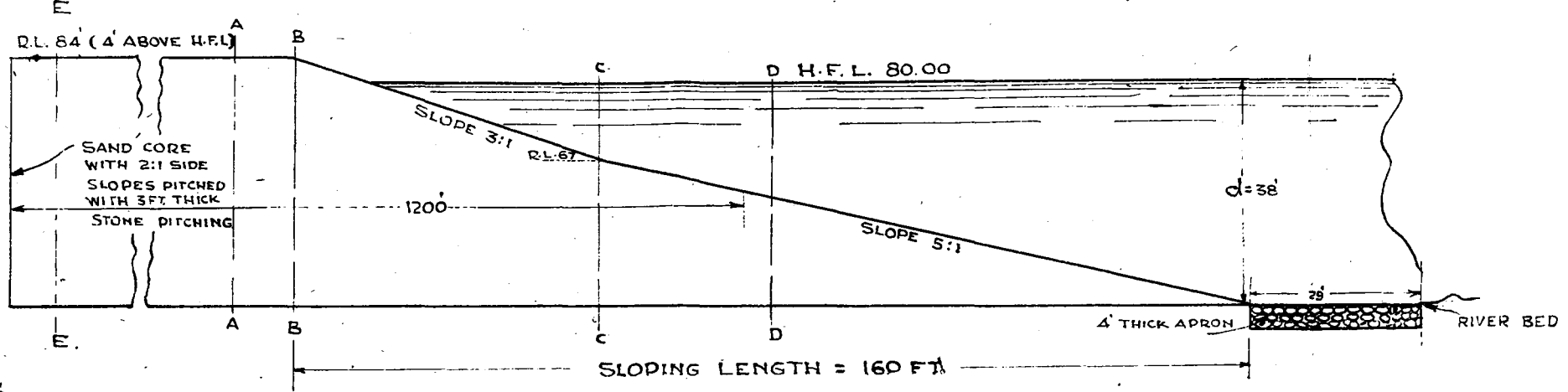
- WITH 1200' SPUR 60° U/S ———— } AFTER COMPLETE FLOOD CYCLES
- " " 1400' " 60° " - - - - - }
- EXISTING CONDITIONS - - - - -
- WITH SPUR POINTING 45° U/S - - - - - } WITH 5 LAKHS C.S. CONDITIONS AFTER 2 HRS. RUN.
- " " " 60° U/S - - - - - }
- " " AT 90° TO FLOW - - - - - }

7. BY SAPKAL.

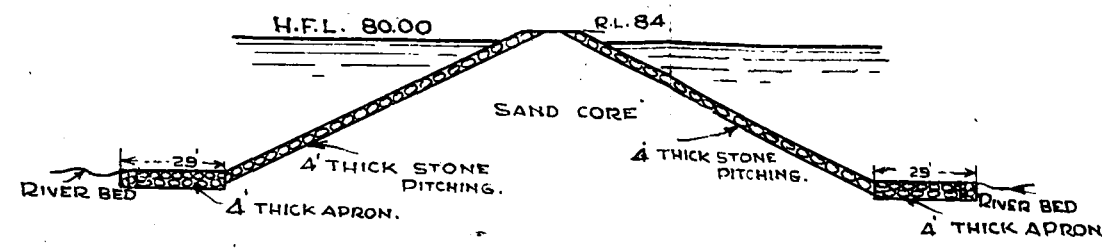
PLAN OF THE REPELLING SPUR.
Scale: - 1" = 40'



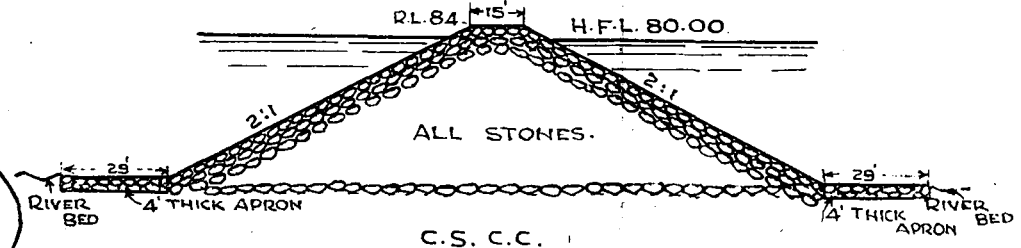
L. SECTION OF THE PROPOSED REPELLING SPUR.
Scale: - 1" = 20 FT.



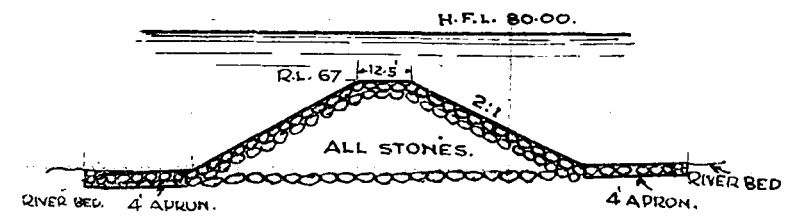
CROSS SECTION AA.



C.S. BB.



C.S. C.C.



C.S. DD.

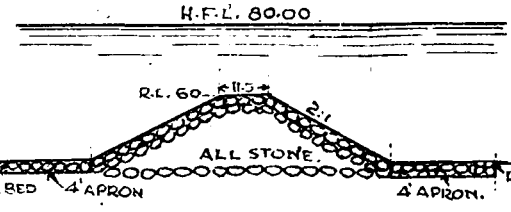


FIG. 10/c-2(ii)-2.

SHOWING PLAN, L. SECTION,
& CROSS SECTIONS OF THE
PROPOSED REPELLING -
SPUR IN THE KATJURI
RIVER AT -
CUTTACK.

Scale: - FOR PLAN & C.S = 1" = 40'
Scale: - " L. SECTION = 1" = 20'

down to R. L. 14. Having done these quick tests, a proper spur, with \cong 5:1 slope of the nose and representative side slopes, was constructed and the lowest scoured bed observed was only R. L. 28 (i. e. maximum depth \cong 52') which is approximately $2D_{\text{Lacey}}$ as shown below: Photos 5192 and 5193.

$$D_{\text{Lacey}} = 0.473 (Q/f)^{\frac{1}{3}}$$

where Q is the maximum discharge and f = the Lacey silt factor $= 1.76 \sqrt{m}$.

In this case $Q = 4.5 \times 10^5$ cusecs;

$$\text{and } f = 1.76 \sqrt{1.115}$$

$$= 1.86$$

$$\therefore D_{\text{Lacey}} = 0.473 \times \left(\frac{4.5 \times 10^5}{1.86} \right)^{\frac{1}{3}}$$

$$= 29.47 \text{ ft.}$$

$$\therefore 2 D_{\text{Lacey}} = 58.94 \text{ against observed depth in the scour pit } \cong 52 \text{ ft.}$$

With scour equivalent to $2 D_{\text{Lacey}}$ to be on the safe side, the maximum depth in the scour pit will be about 59 ft. If, therefore, a launching apron 29 ft. wide \times 4' thick is laid round the spur nose, it will launch at a natural angle of repose forming a slope of about 2:1 down to the bottom of the scour pit. This apron provides an allowance of 25 percent for stone likely to be washed away; but some more stone was recommended to be kept in reserve for emergencies.

5. 1 The nose of the spur and the first 10' length of the shank should be entirely of stone, the latter with a view to provide a good junction of the nose with the shank. The shank also will have to be protected by an apron 15' wide and 4' thick, both on its downstream side as well as on the upstream side.

5. 2 The design of the spur is shown in Figure 10. The nose of the spur is designed to slope down at 1 in 3 for top 1/3rd depth of flow in order to prevent "gulleting", the remaining 2/3rds bottom portion sloping down at 1 in 5. Fig. 10
C-2 (ii)-2

6 *Conclusions*:—As has already been observed in comparison of field and model results of the Rupnarain repelling spur, the model does not give the rate and depth of accretion below the spur. It only gives an indication of the extent of accretion. This is because the bed changes in a model occur mostly on account of rolling bed sand; but, in the river, it occurs more rapidly due to the enormous quantity of suspended load; and hence the rate of accretion is very much slower in the model than in the river.

During the time the bed near Khannagar is building up to suit new conditions a certain amount of diving flow—certainly less severe than under existing conditions—may be anticipated and taken care of; but this will disappear as soon as the bed gets built up.

In the neighbourhood of the spur due to the restricted waterway, the extensive shoal on the right bank will scour and also the right bank may erode; but this bank erosion is considered by the local officers to be of little consequence.

To counteract sufficiently the present severe conditions and avert the impending danger to the left embankment, the Khannagar revetment and the village, the optimum length of the spur should be 1200'. However, later when the deep channel has shifted to midstream at Khannagar, after analysing the post-spur conditions, it may be possible to reduce with advantage the length of the 1200' spur now recommended, if necessary for reducing action along the right bank.

C-2 (ii)-3 Training of the Ganga River At Kanpur

(Advice without model experiments.)

Fig 11
C-2 (ii)-3

Since 1900, the Municipal Board had been drawing water from the Ganga for the unfiltered water supply of Kanpur, situated on the south bank of the river. Up to 1912, the river course used to hug the south bank; but since then it has been shifting northwards, necessitating the dredging of a channel to feed the pumps. During the last few years, the main current of the river has been flowing $1\frac{1}{2}$ miles to the north of the Municipal Board's Pump House.

Fig. 11
C-2 (ii)-3

On his inspection of the river in 1941, Sir Claude Inglis had stated that the unfavourable meander conditions of the river indicated that no permanent remedy could be evolved. After the 1944 floods the river conditions appear to have been improved and, accordingly, the measures suggested by this Station, without model experiments, consist of a pitched "attracting" island about $1\frac{1}{2}$ miles upstream in the river and a pitched curved bank at the Pump House provided with an adequate launching apron.

The island was suggested, because a part island—called a bund when constructed in May 1947 by the Kanpur Municipal Board—had shown signs of improving the river curvature; and the pitched embankment followed from the analogy of the Yamuna at Delhi where a similar design was shown by model tests to be effective in "capturing" the river near the Delhi Gate Pumping Station.

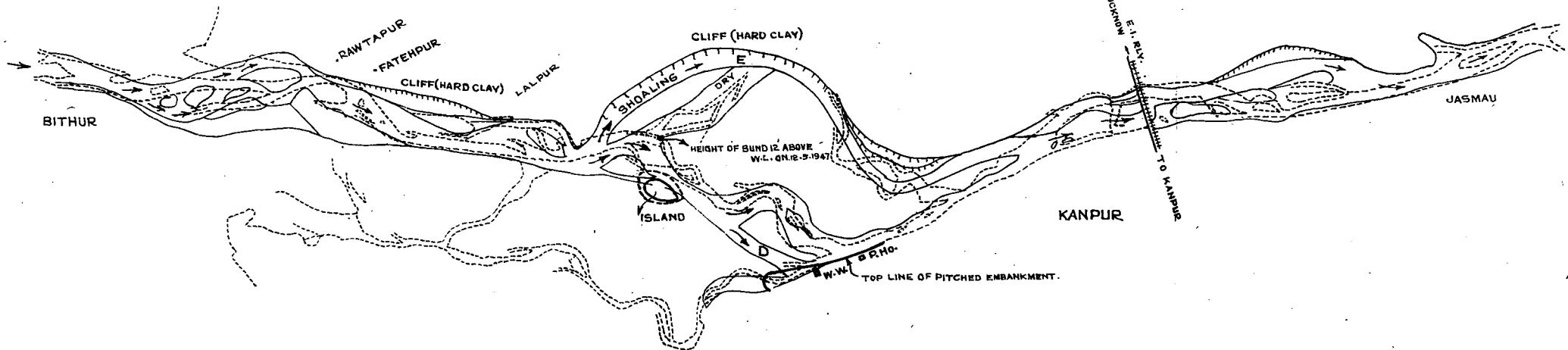
KANPUR DIVISION LOWER GANGA CANAL
TRACE PHOTOGRAPH MOSAIC OF GANGA RIVER.

FIG. II/C-2(ii)-3.

TAKEN IN JUNE 1945

SCALE:-17.2" TO A MILE APPROXIMATELY.

NOTE:-
APPROXIMATE CONDITION OF RIVER COURSE IN MAY 1947 SHOWN IN FULL *Line*
W.W. WATER WORKS.
P.HO. POWER HOUSE KANPUR: ELECTRIC SUPPLY CORPORATION.
D. NEW CHANNEL DEVELOPED
E. CHANNEL AT E EXISTED IN JUNE 1945, & PREVIOUSLY AS ENQUIRIES SHOW, ALTHOUGH IT DOES NOT APPEAR CLEAR IN THE PHOTO MOSAIC, PROBABLY DUE TO ITS HAVING BEEN THEN LESS WELL DEFINED.
COURSE TAKEN IN JUNE 1945 SHOWN IN DOTTED *Line*



C - 2 (ii) - 4. Kshipra River at Ujjain — Gwalior State.
Preliminary Note and Design of Model Scales.

1. History

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' in the years 1901-1903, and due to the increased demand of water, as a result of the tripling of the town population, the length of the gallery was increased to 600'. The gallery is constructed on rock foundations; it is rectangular in section (4' x 7') with an arched pucca top. Dry stone masonry is built between side pillars spaced 6' apart to collect the filtrate from the 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' of water by means of a double row of 6" high kurries, stanchioned on the upstream face with mud to prevent leakage, and has only two scouring sluices at present. During the floods only the top 3' of kurries can be removed, thus having a permanent ponding of 11' 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, ponding is inevitable; while, because the infiltration head immediately after the floods, exceeds the critical head of infiltration the gallery has often got clogged with fine silt.

Fig. 12
C-2(ii)-4
Photo
4027

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 also called for 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 pitching. To protect the right bank from scour and to push the current over the infiltration gallery, a series of small spurs were proposed by the local officers and advice about their efficacy was sought from this Station in November 1943.

Photo 4026
Fig. 12
C-2(ii)-4
Photos 4024
and 4025

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 Nos. 7 and 8 on the right bank. The local officers however, thought that this single spur may take a long time to act and may worsen conditions at Jantar Mantar. So they constructed, before the floods of 1944, repelling spurs No. 1 and 2 on the right bank as well as spur No. 3 on the left bank. This *attracting* spur No. 3 was, however, not given sufficient time or a fair chance to come into action because after the first low floods of August 1944, it was cut down by a length of 150' from its nose, while after the peak floods it was further cut down to 230'. Later, spurs 4, 5, 6 and 7 were constructed.

Fig. 12
C-2(ii)-4

1.4. The Government of 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 and the reach AB of the river has, therefore, to be protected from erosion. This scheme may take some time to materialize, and in the interim period the infiltration gallery has to be kept clear from getting clogged with fine silt.

Fig. 12
C-2(ii)-4

1.5. With this twofold object in view, (a) of protecting the right bank, and (b) of preventing fine silt deposition on the infiltration gallery, a model of the Kshipra River has been recently constructed at this Station. The model extends $3\frac{1}{2}$ miles upstream and $\frac{1}{2}$ mile downstream of the Barrage. The model reach is shown in figure 12.

Fig. 12
C-2(ii)-4

(2) *Hydraulic Data*—As in the majority of cases, the available hydraulic data is very meagre.

(i) *Bed sand*—Representative samples of bed sand collected from the bed near the infiltration gallery and a quarter mile upstream of the barrage were analysed and gave $m = 3.8$ mm. and 5.75 mm. respectively.

Fig. 13
C-2(ii)-4

(ii) *Bed and surface slopes*—The river bed slope, in the reach under investigation, is 1 in 3500 (approx.) and the surface slope is 1 in 4400.

(iii) *Maximum flood discharge*—There are no records of velocity and discharge observations. The maximum flood velocity, as estimated by eye by an officer, is reported to be 15 ft. per sec. which appears much exaggerated, as will be seen from the discharge computations below. Similarly, the maximum flood discharge is reported to be 2.98 lacs cusecs — presumably based on the overestimated flood velocity — and is almost double the Station estimate. As will be seen from Fig. 14, this is a flashy type of river, and the floods rise and fall quickly.

Fig. 14.
C-2 (ii)-4

3 Computed estimates of Flood Discharge :

3.1 From fig. 12 it will be seen that the meander length of the river is about 9860 ft. which gives,

from the Station formula $M_1 = 30 \sqrt{Q_{\max}}$

$$Q_{\max} = \left(\frac{9860}{30} \right)^2$$

$$= 1.08 \text{ lakhs cusecs ;}$$

whereas the meander length based on dominant discharge is given by the formula $M_1 = 40 \sqrt{Q_{\text{dom}}}$

from which $Q_{\text{dom}} = 61000$ cusecs.

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

3.2 Q_{\max} from Lacey's general flow formula

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

$R = 15.05$ ft. and $S = 1$ in 4400 at the barrage section.

$$V = 16 \times 15.05^{2/3} \times (1/4400)^{1/3}$$

$$= 5.95 \text{ ft./sec.}$$

Fig. 15.
C-2(ii)-4

It has been observed that the cross-section of the river 100' downstream of the railway bridge up to R. L. 1585.0 is quite stable and can be used for computing area of cross section.

Up to R. L. 1585.0 :

Area A = 1.685×10^4 sq. ft.

Area B = 0.73×10^4 sq. ft.

Area C = 0.305×10^4 sq. ft.

Area D = 0.46×10^4 sq. ft.

Area E = 0.29×10^4 sq. ft.

Total Area = (A + B + C + D + E)

$$= [1.685 + 0.73 + 0.305 + 0.46 + 0.29] \times 10^4$$

$$= 3.47 \times 10^4 \text{ sq. ft.}$$

Areas C, D and E are overflow portions and so a coefficient value of 0.1 is assumed in working out effective velocity, while a coefficient value of 0.75 is assumed for area B.

$$Q_{\max} = \epsilon A \cdot V$$

$$= V \cdot A + 0.75 V \cdot B + .1 V(C+D+E)$$

$$= [(5.95 \times 1.685) + (.75 \times 5.95 \times 0.73) +$$

$$0.1 \times 5.95 \times 1.055] \times 10^4 \text{ cusecs.}$$

$$= 1.39 \text{ lacs cusecs.}$$

Similarly, with area of section up to R. L. 1575.5, i. e. bankful stage,

$$Q_{\text{dom}} = 0.70 \text{ lacs cusecs.}$$

3.3 Q_{\max} with the Mississippi formula :

$$V = [(3.0966 R \sqrt{S})^{1/4} - 0.0388]^2$$

Kshipra River at Ujjain.



Photo No. 4026 : Kshipra river at Ujjain. Photo looking downstream showing the concrete apron at Jantar Mantar, the Shiva's Temple and the barrage in the background.

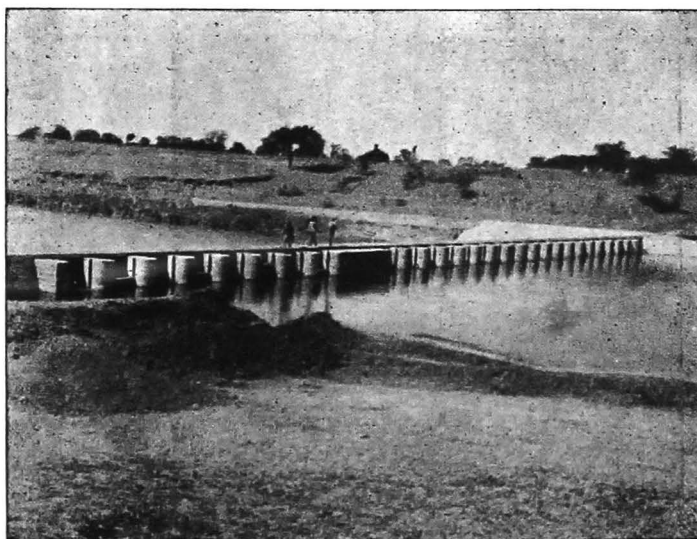


Photo No 4027 : Kshipra River at Ujjain. Showing the Barrage. Photo taken from the left bank. Two scouring sluices are in the middle foreground.

The Kshipra river at Ujjain.



Photo No. 4024 : Kshipra river at Ujjain. Photo looking upstream showing the stone groynes and the Pump House in the back ground.



Photo No. 4025 : Kshipra at Ujjain. Photo looking upstream, showing the spur on right bank and fine silt deposit being removed from over the infiltration gallery. The concrete pillar seen in the foreground is constructed to locate the position of the gallery underneath.

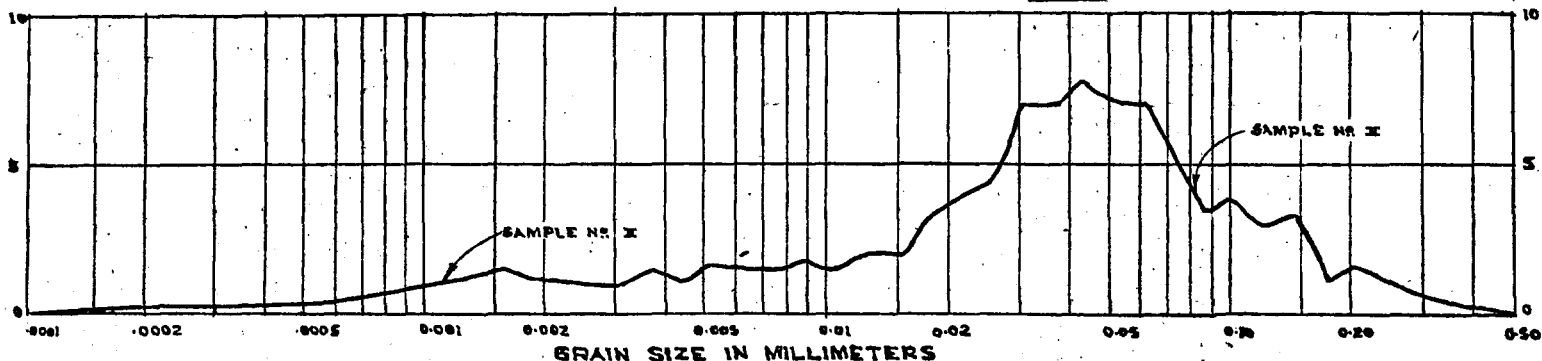
FIG. 13/C-2 (ii)-4.

SHOWING SIZE DISTRIBUTION CURVES OF
BED SAND SAMPLES COLLECTED IN KSHIPRA RIVER (UJJAIN)

No 6789
CENTRAL WATERWAYS, IRRIGATION
& NAVIGATION RESEARCH STATION
POONA

PERCENT WEIGHT OF MATERIAL PER UNIT

4077 (LAB.) SAMPLE NO. II OBTAINED FROM U/S OF THE BARRAGE
IN FRONT OF THE OBSERVATORY. $m = .047$ mm.



o 4076 (LAB.) SAMPLE NO. I OBTAINED FROM D/S OF THE BARRAGE
 $m = 4.69$ mm.

PERCENT WEIGHT OF MATERIAL PER UNIT

+ 4078 (LAB.) SAMPLE NO. III OBTAINED FROM THE BED BETWEEN
THE GALLERY AND WATER WORKS $m = 3.78$ mm.

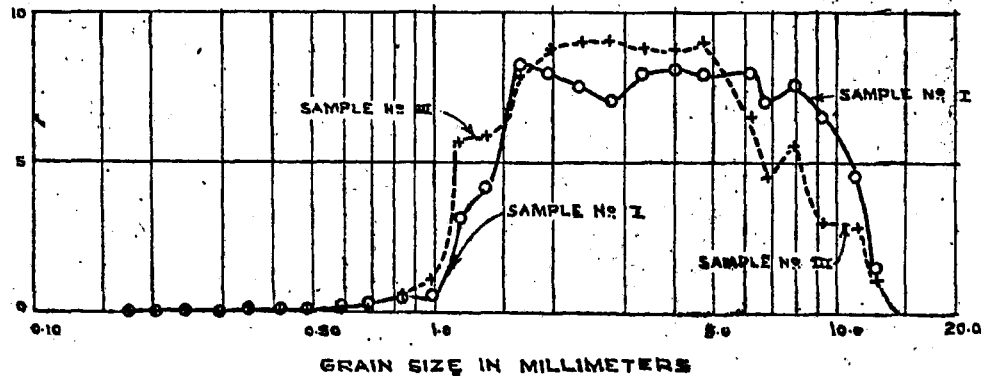
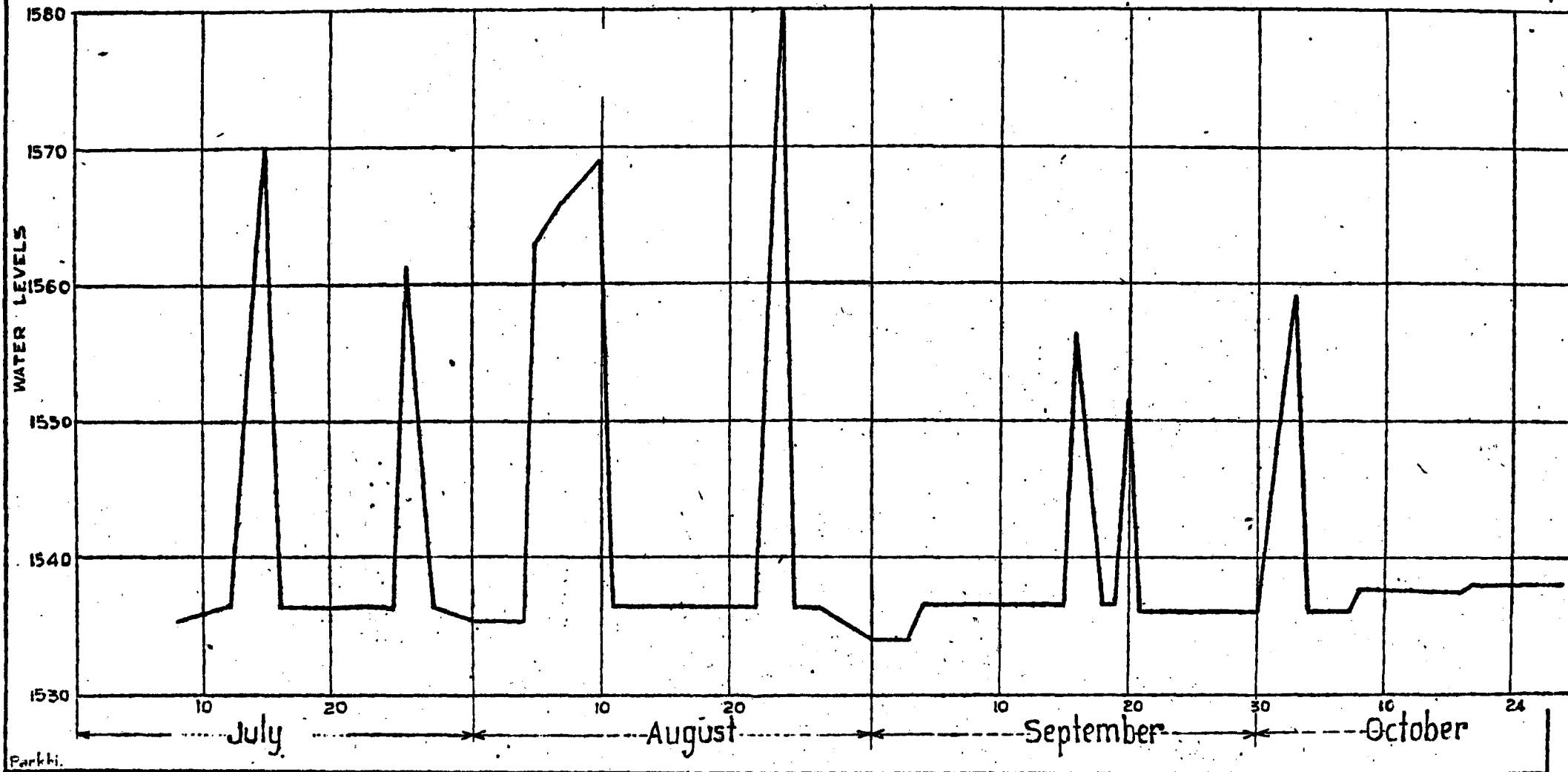


FIG. 14/C-2 (ii)-4.

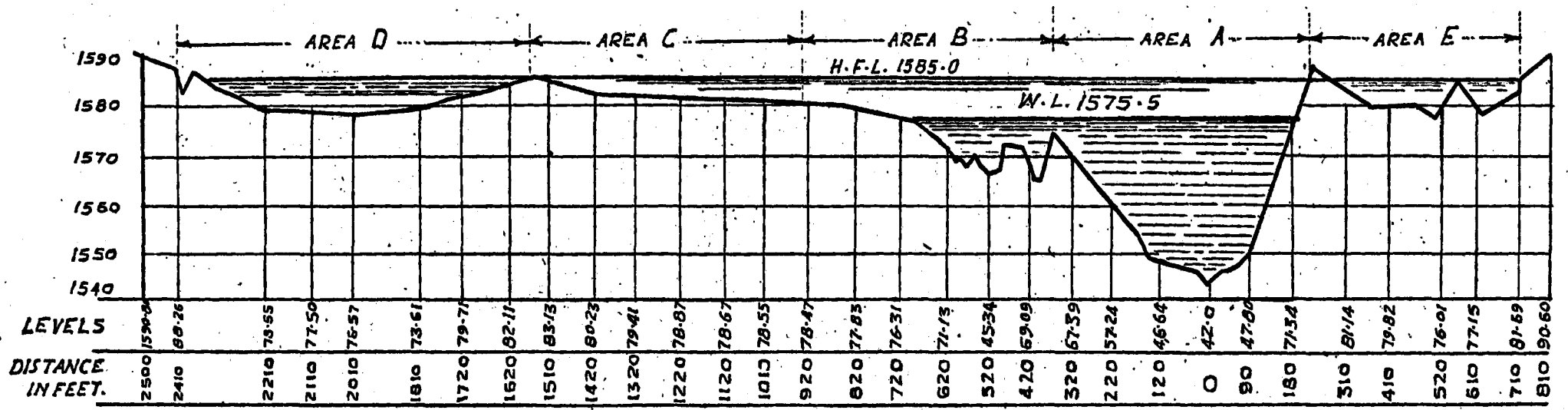
SHOWING THE WATER LEVELS IN THE
KSHIPRA RIVER DURING FLOODS OF 1944.



Parkhi.

FIG. 15/C-2(ii)-4.
KSHIPRA RIVER
SHOWING CROSS SECTION 100 FT. D/s
OF RAILWAY BRIDGE

SCALE:- HOR. 1" = 300'
VER. 1" = 30'



BY:- BHINGARE.

where

$$R = \frac{\text{total area}}{\text{wetted perimeter} + \text{surface width}}$$

$$S = \text{slope per mile.}$$

$$V = \left[\left[3.0966 \left(\frac{5.07 \times 10^4}{6120} \right) \times 1.086 \right]^{1/4} - 0.0388 \right]^2$$

$$= 5.10 \text{ ft./sec.}$$

$$Q_{\text{max.}} = A \times V$$

$$= 2.34 \times 10^4 \times 5.10$$

$$= 2.19 \text{ lacs cusecs.}$$

3.4 The Station estimate of Q_{max} is 1.39 lacs cusecs, as shown in para 3.2.

4 Choice of Model Scales :

(i) *Choice of length scale*—On considering the available and suitable space and discharge it is proposed to adopt a length scale of 1/170, giving

$$Q_{\text{max}} = 1.39 \text{ lacs cusecs} \equiv 4.81 \text{ cusecs in model,}$$

and $Q_{\text{dom.}} = 0.70 \text{ lacs cusecs} \equiv 2.42 \text{ cusecs, in model.}$

(ii) *Choice of depth scale* —

(A) Applying Lacey's formula with $m_m = 0.5 \text{ mm.}$ and

$$m_p = 3.78 \text{ mm.}$$

$$V.E. = (Q_p/Q_m)^{1/6} (m_p/m_m)^{1/6}$$

$$= (170^2)^{1/6} (7.56)^{1/6}$$

$$= 5.54 \times 1.40$$

$$= 7.76$$

\therefore d scale = 1/21.9 or say 1/22.

(B) The vertical exaggeration was also worked out from the empirical Station formula (obtained from models of the range of discharge and size of the Kshipra model) :

$d = 0.68q^{0.71}$; which is suitable for conditions, where the bed charge is only moderate, as in the case of the Kshipra which is relatively, narrow and deep, indicating low charge of bed material. The size of bed material was nearly the same as used in experiments from which the station formula was obtained.

$w_p = 900 \text{ ft.}$ and $d_p = 16.44 \text{ ft.}$ for dominant discharge at the Railway Bridge.

$$w_p/d_p = 54.7$$

$$w_m = 5.3' \text{ and } q_m = 0.775 \text{ cusec/ft.}$$

$$0.71$$

$$\text{So } d = 0.68 (0.775)^{0.71} = 0.57 \text{ ft.}$$

$$\therefore w_m/d_m = 9.6$$

or vertical exaggeration = $\frac{54.6}{9.6} = 5.67$ (as compared with vertical exaggeration according to Lacey formula = 7.76).

$$d \text{ scale} = \frac{170}{5.67} \text{ or } \frac{1}{29.9} \text{ or say } 1/30$$

A depth scale of 1/30 was adopted as a first approximation.

(iii) *Slope scale ratio* : Again applying Lacey's formula,

$$\begin{aligned} \text{S. E.} &= \left(\frac{m_m}{m_p} \right)^{5/6} \left(\frac{Q_p}{Q_m} \right)^{1/6} \\ &= (0.50/3.78)^{5/6} (170)^{1/6} \\ &= 0.185 \times 5.55 \\ &= 1.027 \end{aligned}$$

5. *Proving the Model.*

According to Lacey figures, vertical exaggeration = 7.76 and slope exaggeration = 1.027; when the model was run for proving it, it was noted that it did not reproduce prototype conditions of flow correctly till it was given a bed slope exaggeration of 2.0. Thus the model is tilted and has now vertical exaggeration = 5.67 and bed slope exaggeration = 2.0

Apart from the opinions of experienced local officers (the Sanitary Engineer and the Executive Engineer) who have satisfied themselves that the model reproduction of existing conditions is in close conformity with the prototype, there is factual evidence of the regions of attack and high velocities and the flow pattern being the same in the model as in the prototype.

6. Experiments for training measures are in hand. These will be described in next year's Annual Report.

C - 2 (ii) - 5. Measures for arresting bank erosion near Bhadeli & Bhagda Villages on Auranga River - Bombay.
(Advice with model experiments.)

1. - *Introduction* :-

Both the banks of the Auranga river near Bhadeli and Bhagada villages in Bulsar Taluka, Surat District, were heavily eroded due to combined action of the tides and floods. Investigations revealed that extensive measures for general training of the river would be prohibitive in cost and hence only local works designed to protect the important properties along the banks were possible. With this view, seven groynes on the left bank and three groynes on the right bank were constructed by local officers from time to time till 1941. The groynes on the Bulsar side were fairly successful in pegging the river to the left bank. The erosion, however, was moving downstream, as erosion generally does, and in doing this the natural swing of the river shifted to the opposite right bank. To check the consequent erosion, three groynes on the left and two groynes on the right bank downstream of the existing ones were proposed to be constructed; and the Director's views on these proposals were requested.

Fig 16
C-2 (ii)-5

2. *Recommendations* :-

2. 1. Repelling groynes for local protection should not be so long as to cause damage on the opposite bank nor so short that action (turbulence) or scour—which inevitably occurs at the nose—will reach the bank which they are intended to protect. On the left bank, 100 feet groynes were therefore recommended, with the first groyne 60 feet long, to avoid the likelihood of intensifying the attack on the right side. The distance between two repelling groynes should be such that the repelling action is maintained—the throw-off of the one not having dissipated before reaching the second. The first groyne being shorter the distance between the first and second was suggested as 330 ft. and between the second and third, 440 feet.

2. 2 It was difficult to suggest the exact position of the groynes in the absence of full information regarding the relative values of the area most to be protected. It was, therefore, pointed out that the proposed three groynes on the Bulsar side, would protect only the hatched portion in figure 16. The uppermost groyne (as proposed by the local officers) was, therefore, advised to be omitted [(a) because erosion would move downstream and (b) in order to avoid deflecting the current into the Bhadeli arm]; the second and third groynes would then become the first and second groynes (60 ft. and 100 ft. long) respectively ; and a new third groyne, also 100 ft. long, was proposed by the Station to be constructed, 440 ft. away from the third.

Fig 16
C-(ii)-5

2. 3 As the Station was given to understand that the major part of erosion on this bank was caused by river floods (*not* tides), all these three groynes were recommended to face *upstream* relative to river flow, the angles being 45°, 60° and 60° to the bank, respectively.

2. 4 To avoid turbulence near the bank which a submerged spur causes, the spurs were to have as flatly sloping noses as possible, not less than 1 in 3. The noses of the groynes were also to be adequately protected by loose stone apron around them of sufficient radius to cover the likely scour-hole.

2. 5 On the Bhadeli side, it was believed that the *tidal* action must have equal effect, and so the groynes were suggested to be oriented at right angles to the bank. In other respects, they were similar to the Bulsar groynes. Erosion below these groynes being likely, a third groyne below the proposed two, and inclined upstream at 60° to the bank to " throw off " the flood water away from the bank, was also suggested. These groynes are expected ultimately to assist in opening out the old river course when the swing upstream of village Bhagda has shifted to the opposite bank.

2. 6 The recommendations are, it is learnt, shortly to be taken up.

**C-2 (ii)-6 : Protective measures for several railway bridges on
Madras-Bombay Line of M. & S. M. Railway, between
miles 82 & 113.**

1. Introduction :

A number of Bridges between Pudi and Urampadu—miles 82 to 113 on the Madras-Bombay line of M. & S. M. Railway, were severely damaged in the December floods of 1946, regarding which advice and recommendations of the Station were sought. The advice on points common to the design of several structures is recapitulated below:—

2. Estimates of maximum flood discharges :

The data obtainable in respect of flood discharges was inadequate. The estimates furnished, as based on catchment areas by Ryves' and Burges' empirical formulae, could not be taken as satisfactory as much depends on the correct assumptions of the widely-varying coefficients. For instance, Rhind's analysis has shown that C in Ryves' formula can vary from $C = 308$ to $C = 2707^{(1)}$. Figures obtained according to these formulae were significantly lower than those given by the Station formula duly corrected for shape of catchment. The Station formula is based on extensive data and checks up with several proven formulae—Ryves' formula with $C = 2707$, in certain cases, gives about the same figures as by this formula. The Station recommendations were, therefore, based on discharges computed by the Station formula of normal floods—see A-1 (ii)-1 of the report as—it does not appear necessary to provide waterway for the maximum-ever flood, in view of the disadvantages following the formation of inerodible shoals and islands since the maximum-ever flood occurs but rarely.

3. Bridge Waterways :

The waterways proposed for the bridges had been calculated on certain assumptions of velocity which were not altogether warrantable.

The Station practice—pending the results of certain basic investigations in hand—is to design the bridge-waterway for the natural stable width of waterway ($P = C \sqrt{Q}$) for the conditions obtaining on the river. C has a value of 2.67 for Lacey conditions of "minimum charge with a fully active bed" and is modified in light of experience for divergence from regime—steep and shallow rivers indicating more bed load requiring wider waterways than flatter and deeper rivers.

The Station recommendations were made accordingly.

4. Depth of Pier foundations :—

Most of the bridges have open foundations carried to depths, which bear no relation to the conditions obtaining at the bridges.

The rational method of designing pier foundations—adopted in the Station recommendations—is to estimate the maximum probable scour and to allow for sufficient "grip" below the anticipated scour level.

Maximum probable scour round piers has been observed to be about $2R$ (Lacey)⁽²⁾ and a minimum grip of R is regarded as necessary to insure safety of the structure, where $R = .47 \left[\frac{Q}{f} \right]^{\frac{1}{3}}$ and the Lacey factor f is (roughly) equal to $1.76 \sqrt{m}$. m being the sand grade in mms.

Data are still wanting to confirm the applicability of the Lacey formulae to boulder rivers and for their modification in cases of divergence from the implicit Lacey conditions of normal charge. In the meantime the large factor of safety usually adopted is inevitable.

5. High level pavements and pitching round piers:—

At several bridges bed pavements have been provided. Experience and experiments have both shown that high level pavements and pitching are




(1) Page 330 of Buckley's 'Irrigation Pocket Book'.

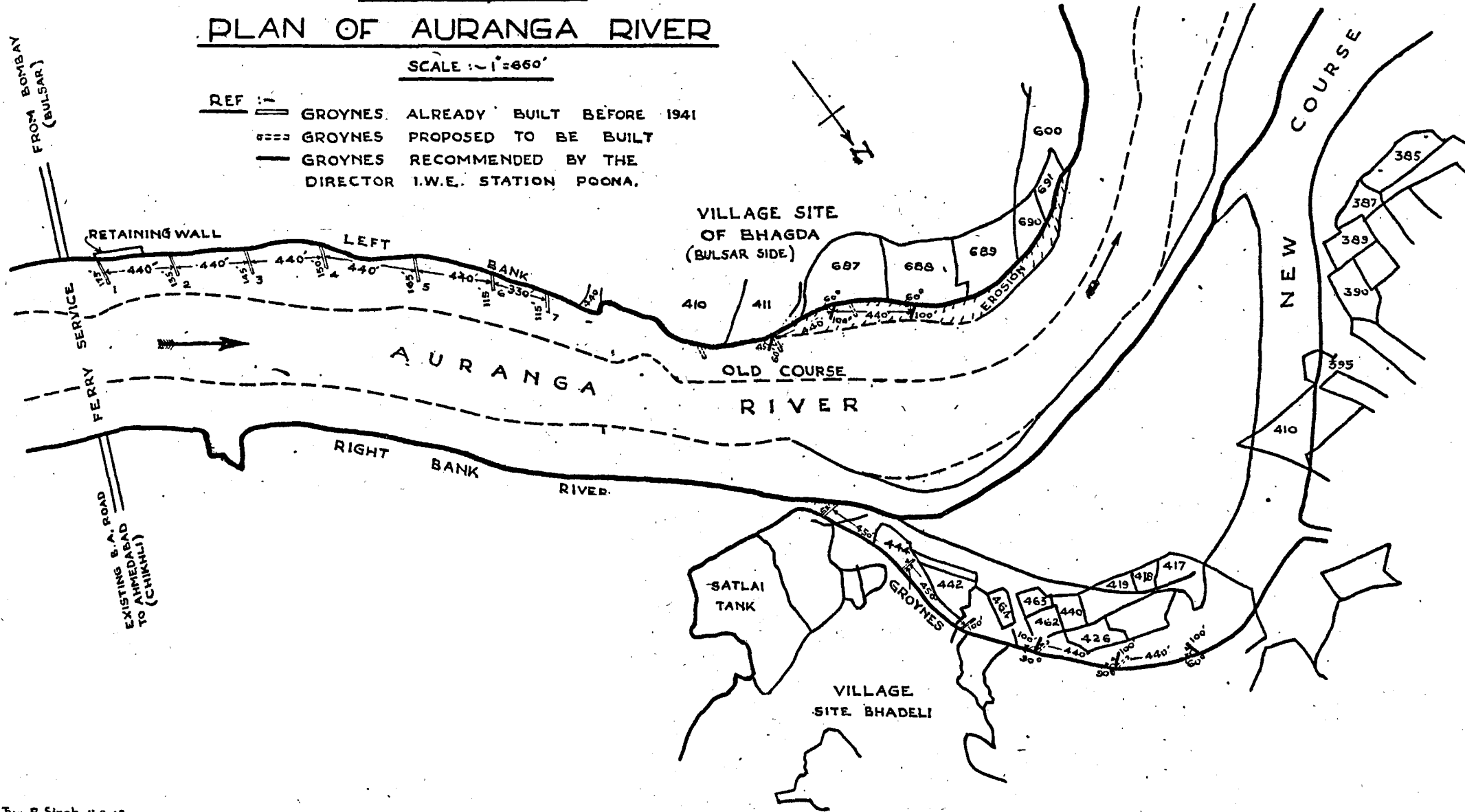
(2) Page 73 of Annual Report (Tech.) for 1944 of L. W. E. Str. Poona.

FIG. 16/c-2(ii)-5.

PLAN OF AURANGA RIVER

SCALE 1" = 660'

- REF :
-  GROYNES ALREADY BUILT BEFORE 1941
 -  GROYNES PROPOSED TO BE BUILT
 -  GROYNES RECOMMENDED BY THE DIRECTOR I.W.E. STATION POONA.



The position of repelling spur is as recommended by
The Director, Indian Waterways Experiments Stn. Poona.

FIG. 17/C-2(ii)-6

NO. 6781.
C.W.I. & N.R. STN. POONA.

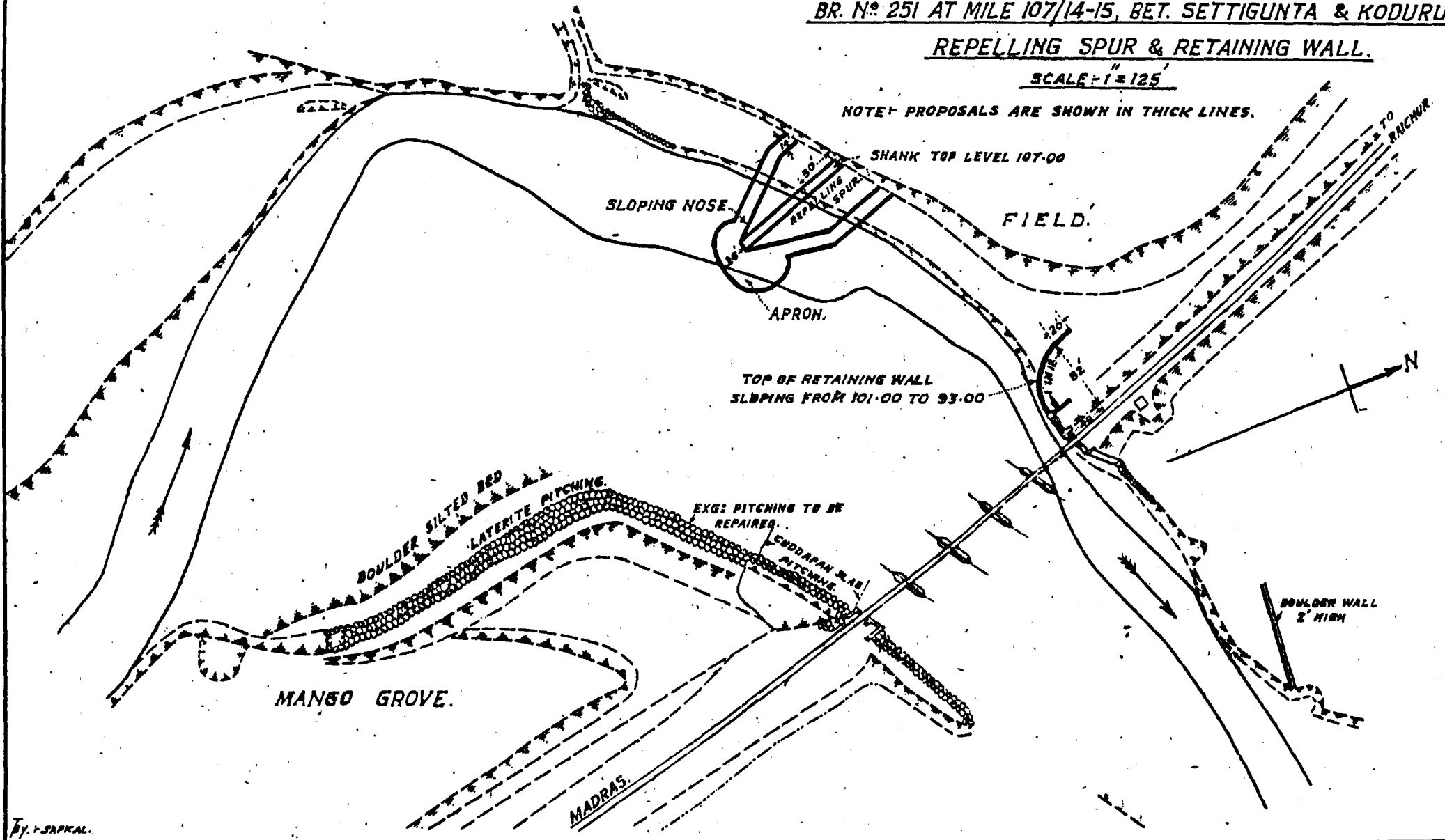
KOU/115/1. M. & S. M. Ry. N.W.L.

BR. NO. 251 AT MILE 107/14-15, BET. SETTIGUNTA & KODURU

REPELLING SPUR & RETAINING WALL.

SCALE = 1" = 125'

NOTE: PROPOSALS ARE SHOWN IN THICK LINES.



By: SARKAL.

APR 1906

not only unnecessary when sufficient grip is provided but are also undesirable, because they result in excessive scour below the bridge and round the piers.^{(1) (2)}

It was, therefore, suggested that high level pavements and pitching should be avoided as far as possible, and if pitching is provided it should be laid at as low a level as constructionally feasible, being best placed at 85 percent of the maximum scour depth without piers.⁽³⁾

6. *Cut-and ease-waters:--*

It was pointed out that the square-ended cut-and ease-waters, proposed to be adopted in some of the new piers, would cause appreciable obstruction to flow and increased afflux and turbulence. Right-angled, curved noses, being economical and easier to construct and not significantly less efficient than the design with 'equilateral arcs of a circle', were recommended where flow was expected to be axial; and, where flow might be skew, semi-circular cut-and ease-waters were suggested.

7. *Provision of adequate free-board and location of borrow pits:--*

A minimum freeboard of 2 feet above the maximum anticipated H. F. L., after making due allowance for accretion, was recommended to be invariably provided.

All borrow pits on the downstream side, particularly in the case of high banks, and borrow pits within 200 feet of the embankments on the upstream side, were suggested to be avoided. Sufficient ridges were advised to be kept between the borrow pits on the upstream side to avoid parallel flow along the toe of the embankment.

8. In addition to these general observations, advice was given in particular cases. In some cases, river training measures were suggested without model experiments and in one case model experiment was recommended. One case is described below:--

Bridge No. 251.

The bridge being located immediately below an S Curve, there was heavy attack on the left abutment and the end pier.

To relieve this attack a repelling spur as in Fig. 17 was suggested.

Besides ameliorating the conditions at the bridge the proposed spur was designed to straighten the river upstream of it and in doing this it would cause a recession of the river away from the railway embankment in the embayment where it has approached dangerously close to the railway line.

In addition to the spur the following measures were recommended :

- (i) Return walls round the north abutment.
- (ii) Adequate strengthening of the collapsed right (South) guide bank and pitching with sufficient apron, in front as well as at the back; the latter to afford protection against breaches from spills running parallel to the guide bank at its rear slope.
- (iii) Adequate waterway for the maximum discharge.

To avoid the inherent defects of the present structure, viz. high pavement, uncertain shallow foundations, inadequate waterway, unsatisfactory location below an S curve, the lack of suitable cut-and ease-waters etc., and to overcome the doubts which must remain when a bridge is repaired at the same site where a pier has twice given serious trouble. it was proposed that the bridge should be relocated in the straight portion of the river.

Fig. 17
C-2(ii)-6

(1) pp. 33-40 of Annual Report (Tech) for 1939-40 of C. I. & H. R. S.

(2) p. 12 of Annual Report (Tech) for 1941-42 of I. W. E. S.

(3) p. 38 of Annual Report (Tech) for 1939-40 of C. I. & H. R. S.

C-2 (ii)-7: Training of the Tapti River at Surat (Bombay.)

1. Introduction :-

1. 1. The detailed history of the case is given in the 1944 Annual Report (Tech) of the Station. (1) Only a brief summary is recapitulated below :

At Surat, the river Tapti takes a nearly right-angled turn and the curvature is too sudden and acute for the river to follow—especially in high floods. In the floods of 1937, the left abutment of the Hope bridge at Surat and the approach road to Rander were severely damaged.

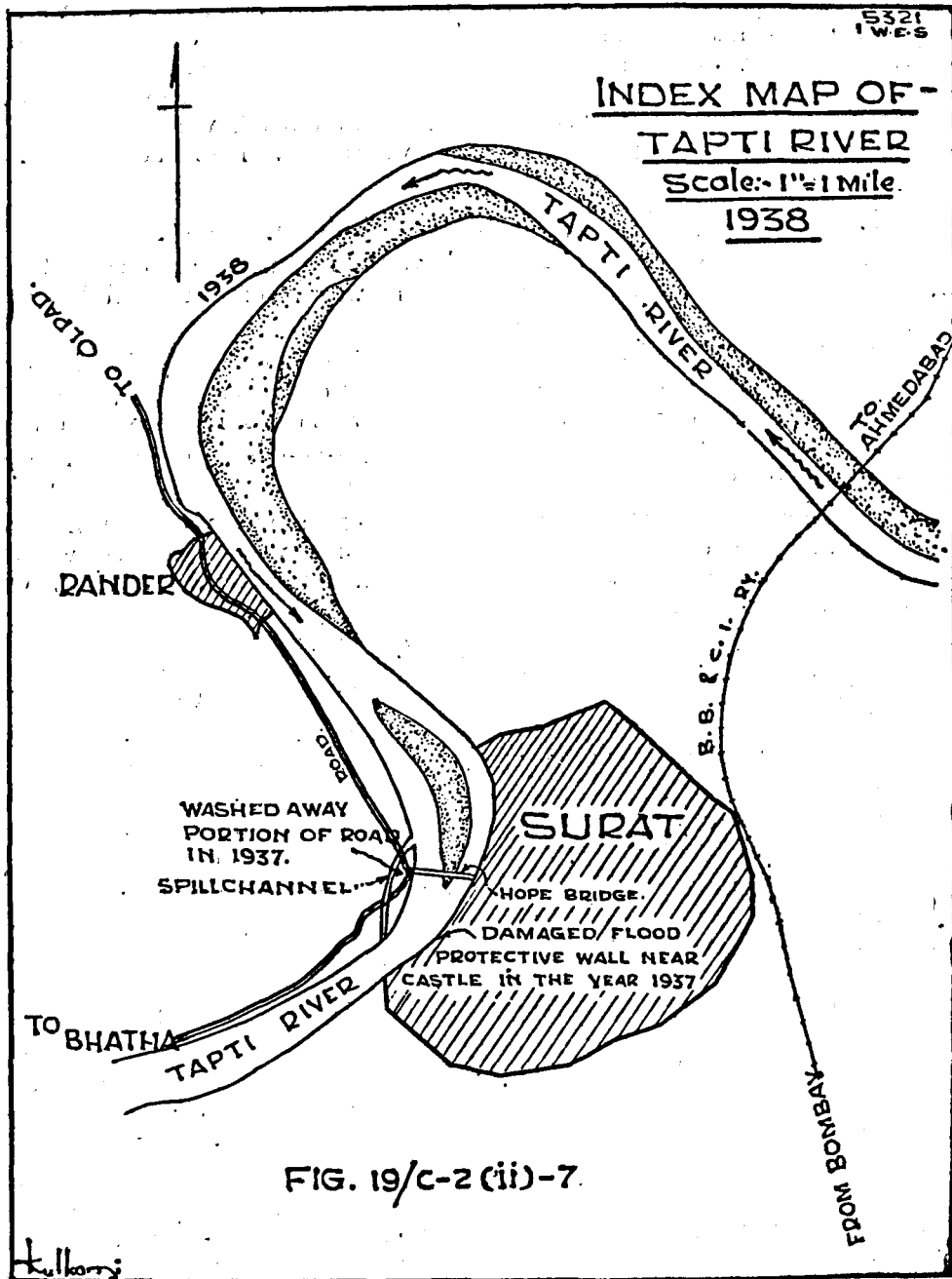


FIG. 19/c-2 (ii)-7.

Subsequently, in order to improve flow conditions and to accelerate erosion of the central island just upstream of the Hope bridge the Superintending Engineer, Northern Circle recommended blocking of the spill channel on the right bank behind the bridge abutment and raising of the loose stone floor in the left bank spans where it was deeper than the average bed level.

1. 2. The Director strongly cautioned against these proposals for the reasons that the blocking of the spill channel would cause more afflux and raising the bed of the left bank spans would distort the natural shape of the bed

Fig. 19 & 20
C-2 (ii)-7.

(1) Annual Report (Tech) of Indian Waterways Experiment Stn. Poona, 1944, Pages 87 to 90.

and flow pattern at the curve and would thus permanently worsen conditions, which were already bad.

In spite of this advice, however, a costly embankment was constructed across the spill channel in 1939. According to Superintending Engineer, Northern Circle's orders, the top of the pitching round the bridge piers of the whole bridge was also maintained at the uniform level (R. L. 65.0) and the river bed in-between piers was pitched to R. L. 53.0.

1.3. These measures were, as had been predicted, far from successful in preventing the damage in the subsequent floods and the Director was again requested to give advice after the 1944 floods. He recommended that the spill channel near the right bank of the bridge should be reopened by building 3 or 4 spans, each of 100 ft., and the top of pitching round piers should be lowered or allowed to scour naturally from R. L. 65.0 to R. L. 53.00.

The spill channel has had to be ultimately reopened by removing the embankment and the practice of making good the pitching-stone with a view to maintain the bed level uniformly at R. L. 65 has also been discontinued. This has eased conditions a bit; but more relief from flood damage consequent upon these measures owing to reduction of afflux will be apparent only in the next high floods. (Since this has been done, there have been no high floods in the Tapti).

The Director also advised model experiments as essential to evolve the optimum protective measures for arresting the bank erosion from Rander Town to Hope bridge on the right bank and the severe attack on the Town Walls below the Hope bridge on the left bank.

1.4. Government approved of this proposal and a model extending from Kathor bridge, about 16½ miles upstream of the Hope bridge, to 7½ miles downstream of it,—vide figure 18 and Photo—has been constructed at the Poona Station.

Fig. 18
C-2 (ii)-7
Photograph
No. 5218

1.5. Two other points have also to be investigated on the models:—

(1) At Bhatpur, in the right on the right bank below Surat, a strip of land, about 2 furlongs wide, has been eroded—in a length of about 1½ miles. Measures to prevent further erosion of these fertile and valuable lands are required.

(2) A new steam power station is proposed to be constructed at Umra near Surat. The Electrical Commissioner, Government of Bombay, sought advice regarding the correct location of the Pump House for the Power Station to avoid trouble due to erosion or sanding in future.

1.6. *Flood control on the Tapti.*

Parts of Surat City have frequently been inundated during abnormal floods in the last decade. Spills and overflowing of the banks are not, however, limited to the areas in the vicinity of Surat. The river inundates its banks at several places between Kathor and the outfall to the sea, and local measures—designed with or without model aid—will not ameliorate these conditions. On the general flood control problem of the Tapti, the Director recommended that nothing less than a comprehensive scheme incorporating a number of multipurpose storage reservoirs would be required and suggested a reference to the Central Waterways Irrigation and Navigation Commission, specially set up by the Central Government for the investigation of such interprovincial schemes.

1.7. The main problems to be solved with the aid of the model experiments, thus, are :-

- (1), (2) to arrest erosion on the right bank upstream of the Hope bridge and the attack on the City Walls on the left bank downstream of it;

(3) to determine the best site for the Pump House which would be free from sand trouble for the Intakes and well protected from erosion and, at the same time, not too far removed from the railway line ;

and (4) to prevent further loss of fertile lands in the Bhatpur embayment.

As far as possible the river training measures should be mutually helpful, each keying in with the other.

2. River Model :

The model extends from Kathor bridge down to sea end, a length of about 24 miles—vide figure 18. It is laid, in 0.4 mms. Theur sand, to a horizontal scale of 1/330 and vertical scale of 1/72. To obtain satisfactory bed movement and reproduce flow conditions exactly as on the prototype, a tilted model with V. E. = 4.58 and S. E. about 2.5 has been adopted, after preliminary trial runs.

Fig 18
C-2 (ii)-7

Appendix L
C-2 (ii)-7

The river being of the incised type, with meander imposed and maintained by the local Kunkar and hard clay formations, it was found that to simulate gauges the discharge scale required is $Ld\sqrt{d} = 20,200$. The bed width to depth ratio is small for the discharge, indicating little sand charge moving on bed. This also justified the adoption of discharge scale = $Ld\sqrt{d}$.

Time scale based on the assumption that tractive force in the model is equivalent to that in the river worked out to 1/1118 (Appendix I)

3. 1. Flow lines near Hope bridge at low stages of the river—at W. L. 80.00 (when the island above Hope bridge was just wholly submerged) and at W. L. 70.25 (when it was partially exposed)—were visually observed in the river.

Fig 21
C-2 (ii)-7

In the model, after stabilisation, the flow lines were observed at water level equivalent to B. L. 80.00—shown in Figure 21 ; but at water level 70.25 the depth of water was so low and flow so little that observations could not be done.

In comparing flow lines in the river with those in the model, it is to be remembered that

(1) observations in the river could only be made from the bridge and near the sides and, in between these sections, the flow lines have been drawn in by judgment ;

and (2) in the narrow section below the bridge the river is so deep that the V. E. model would require side slopes steeper than the natural angle of repose of the bed material, and with natural side slopes a relatively wider section is obtained at the bottom in the model. The concentration of flow occurring along the left bank in the river is, consequently, not reproduced to the same extent in the model. This is one of the model limitations which has to be given due consideration while interpreting results obtained with training measures.

When due allowance is made for these factors, it will be apparent that there is close conformity of flow lines in the model and prototype.

3. 2. Flow lines starting from Rander and down to Bhatpur end, and velocities at typical sections, with the dominant discharge of 2.43 cusecs ≡ 5 lakhs cusecs, were also observed.

Fig 22
C-2 (ii)-7

The high velocity flow closely hugs the bank throughout the Rander bend and the Bhatpur bight. This is evidently the root cause of the bank erosion at Rander and Bhatpur.

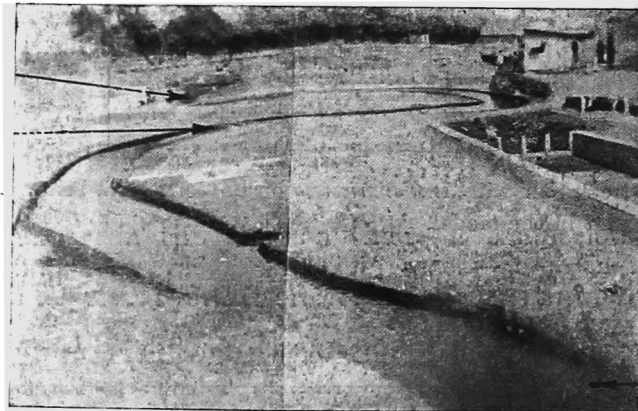
3-3. It was also seen in the model that, below Hope bridge, the flow squarely attacks the left bank between Makhi Khadi & Nanpura Post Office and undermines the buildings abutting against the bank due to diving flow.

Photos 5154
and 5187

$\frac{1}{330}, \frac{1}{72}$ Tapti Model Experiments.

Hope Bridge at
Surat _____

B. B. & C. I. _____
Railway Bridge.



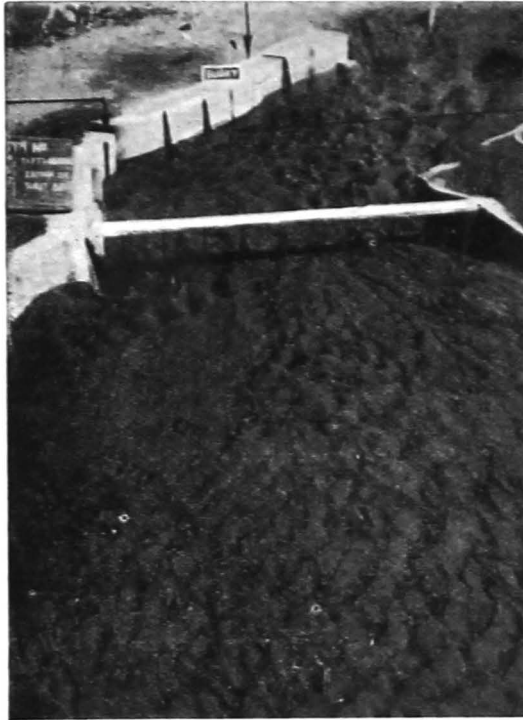
_____ u/s end of
model.

Photo 5218 : shows whole of the river reach modelled at the Station, with upstream river portion in the foreground.

1/330, 1/72 Tapti Model Experiments.

Nanpura P. O.

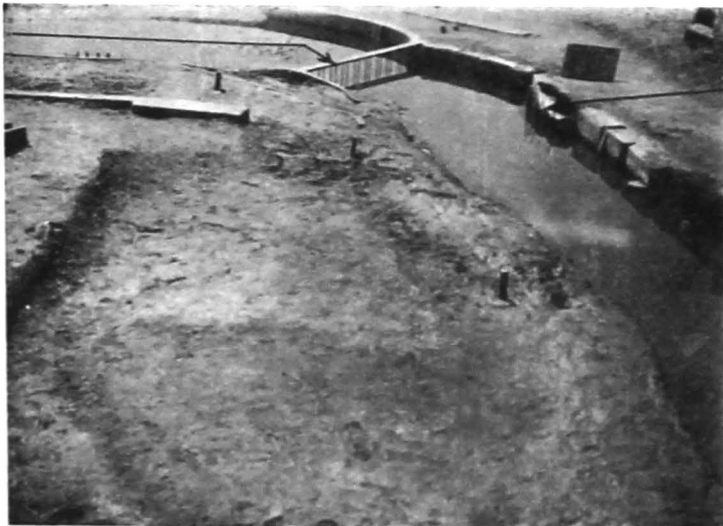
Makhi Khadi



Undermining on left bank in model.

Photo 5154 : Looking downstream, showing *undermining* on the left bank below Hope bridge as *correctly simulated* in the model.

Hope Bridge

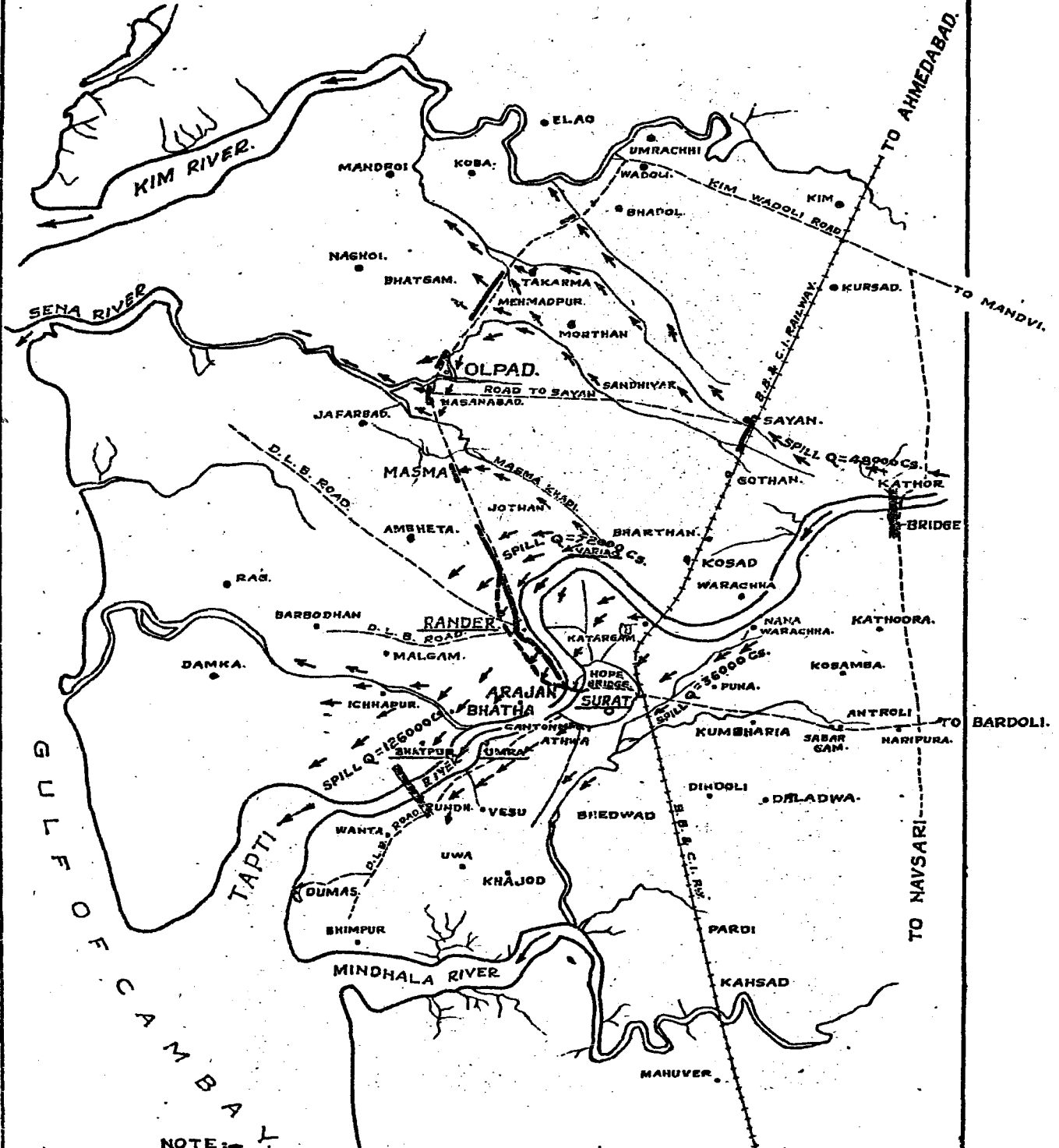


River bank repeatedly cracked and collapsed due to undermining.

Photo 5187 : Looking upstream showing the collapse due to undermining of a part of the puoca bank in the model, representing the masonry walls abutting against the river bank in Surat city.

FIG. 18/C-2(ii)-7
PLAN SHOWING THE OVERFLOW
OF
TAPTI RIVER NEAR SURAT.
DURING FLOOD OF AUGUST 1944
1944-1945

SCALE - 1 INCH = 4 MILES.



NOTE: -

THICK LINE MODEL REACH EXTENDS FROM KATHOR ABOUT 16 1/2 MILES U/S OF SURAT TOWN TO BELOW BHATHPUR 7 1/2 MILES D/S.

ARROW THICK LINES ALONG THE ROAD & RAILWAY & ARROWS SHOW HEAVY SPILLS.

DASHED LINE PROPOSED DIVERSION OF BOMBAY AHMEDABAD ROAD BETWEEN SURAT & RANDEK FROM N. 214/5 TO 217/6

FIG. 20/C-2(ii)-7.

HOPE BRIDGE AT SURAT
ELEVATION

SCALE: } VER. 1" = 50 FT.
 } HOR. 1" = 200 FT.

FROM RANER
RIGHT BANK.

TO SURAT
LEFT BANK.

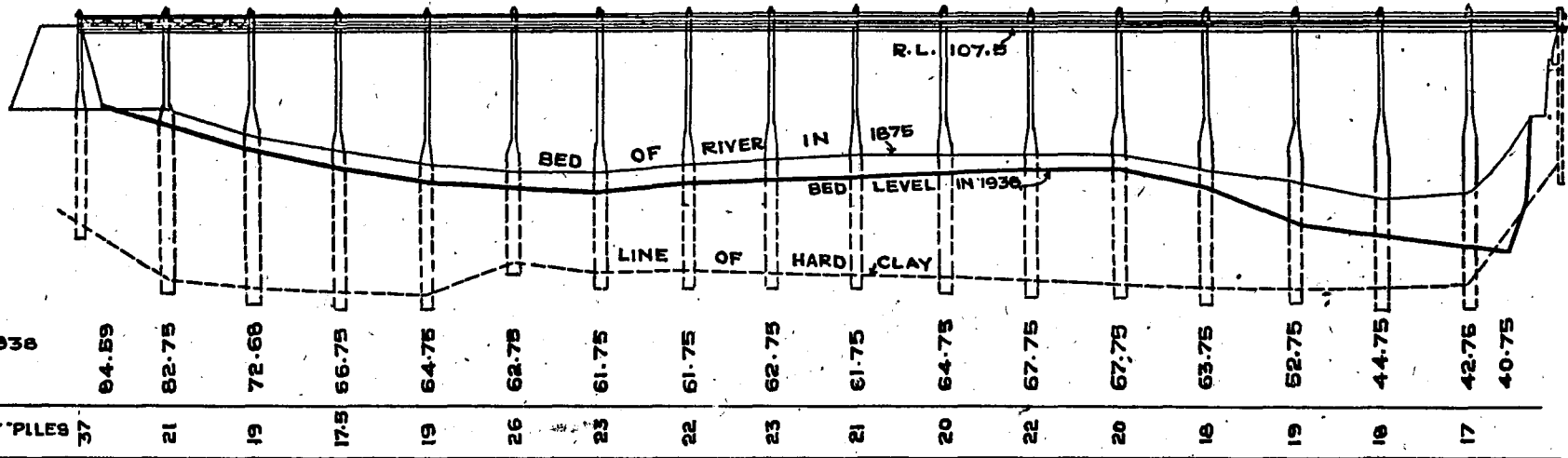


FIG. 21/C-2(ii)-7

PLAN OF TAPTI RIVER NEAR SURAT
SHOWING SURFACE FLOW LINES OBSERVED
IN RIVER AND MODEL AT W.L. 80.00 i.e.

REFERENCES:~

- SURFACE FLOW LINES IN RIVER SHOWN THUS 
- SURFACE FLOW LINES IN MODEL SHOWN THUS 

WHEN ISLAND WAS A
LITTLE SUBMERGED

SCALE: 1" = 800'

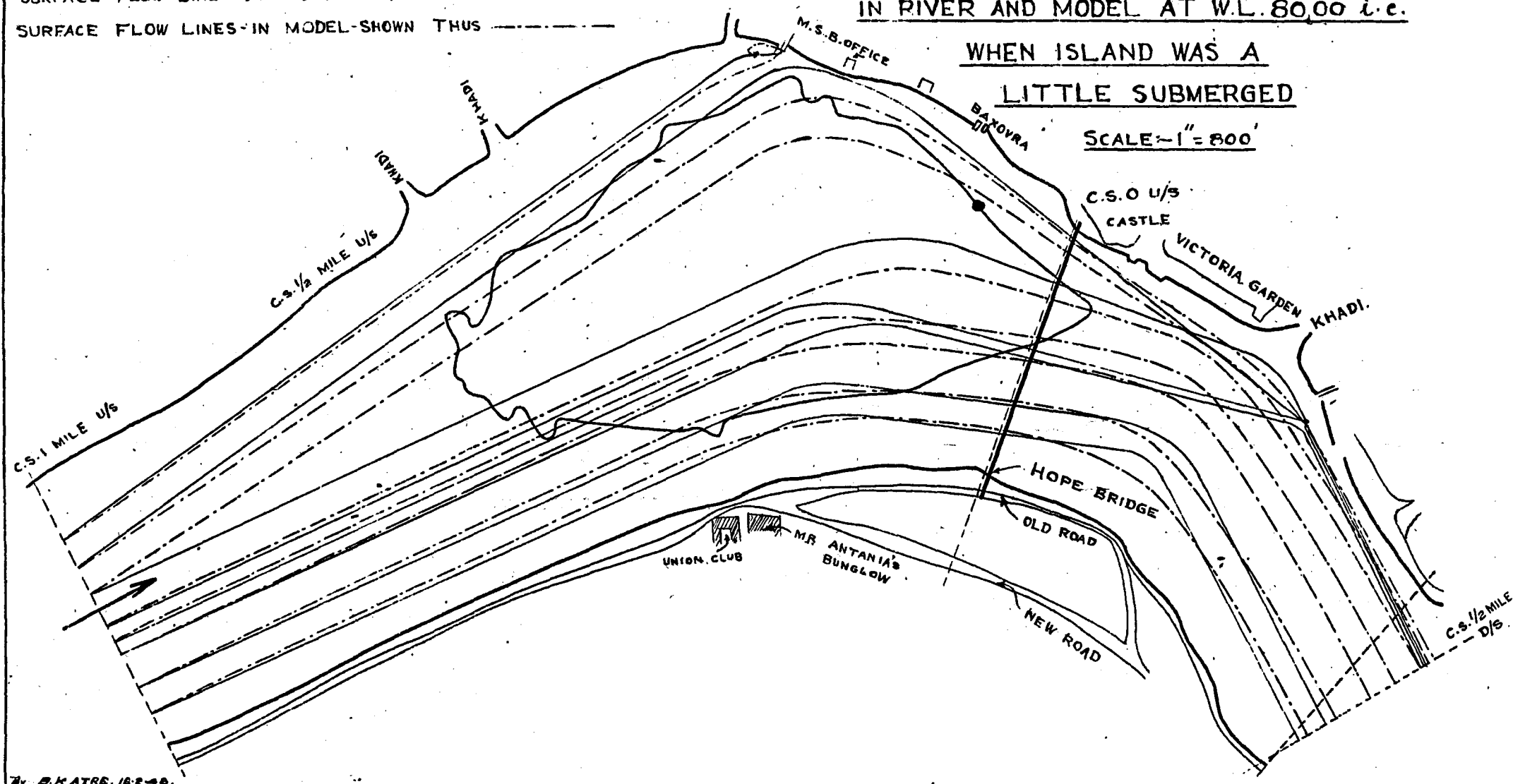
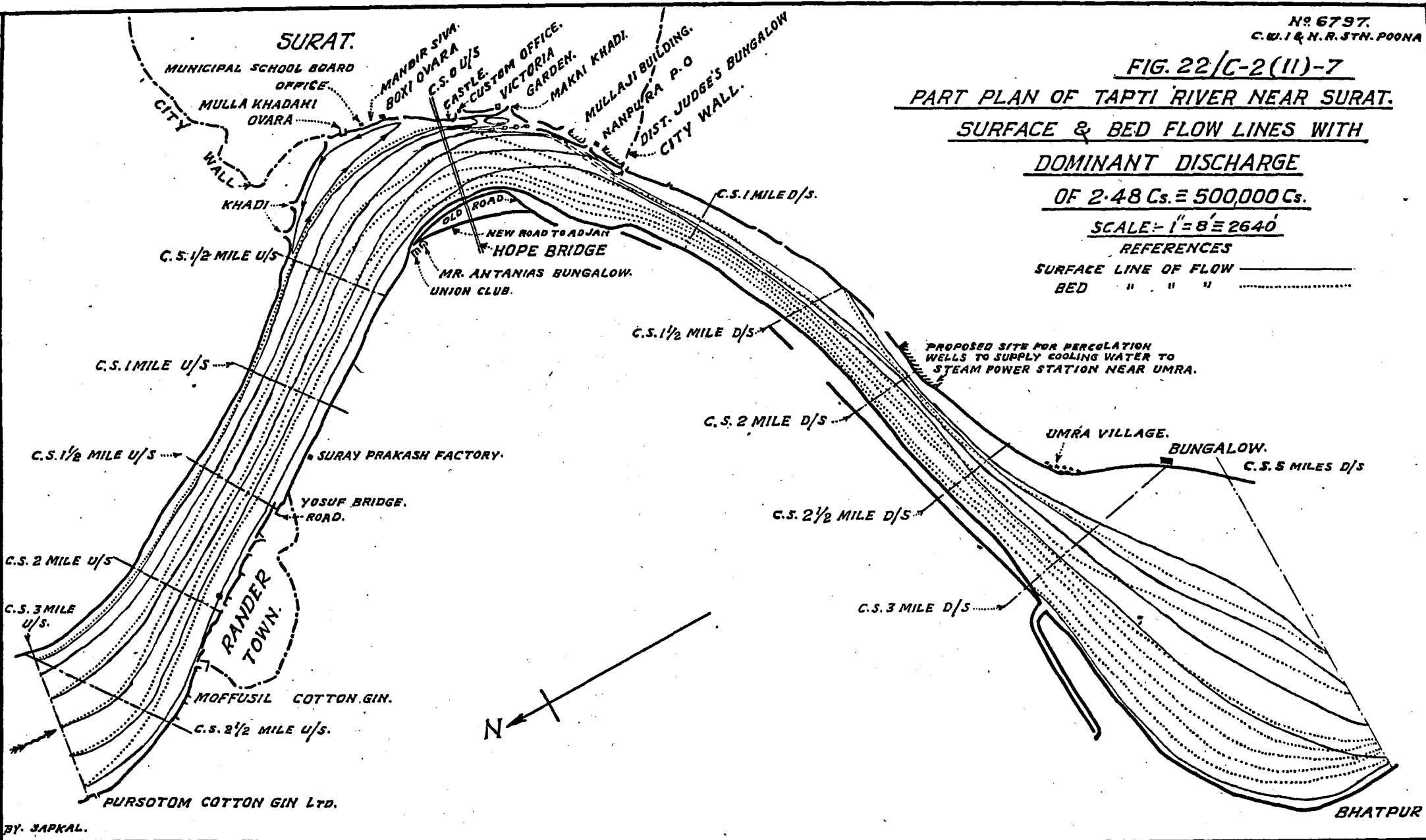


FIG. 22/C-2(11)-7

PART PLAN OF TAPTI RIVER NEAR SURAT.
SURFACE & BED FLOW LINES WITH
DOMINANT DISCHARGE
OF 2.48 Cs. \approx 500,000 Cs.

SCALE - 1" = 8' \approx 2640'

REFERENCES
SURFACE LINE OF FLOW —————
BED " " "



3. 4. At the proposed location of the Pump House near Umra, on the left bank about $2\frac{1}{4}$ miles downstream of Surat, under existing conditions the model indicated that a perennial deep water channel would not be ensured, the location being not sufficiently upstream of the transition of the main current from the left to the right bank.

3.5. Exploratory tests were carried out to devise suitable training works to solve the problems mentioned above. All these problems being inter-linked, in the sense that the consequences of one will be felt by the next, it is decided that the desired results should be obtained by a combination of a minimum number of mutually beneficial training works. Measures suggested by these exploratory tests for more detailed examination on the model were :—

- (1) a repelling groyne on the right bank near Rander;
 - (2) a similar groyne on the left bank or a pitched island in mid-river somewhere between Makhi Khadi and District Judge's bungalow; the same measure will, if possible, be made to peg the deep water channel at the proposed site of Pumping Station;
- and (3) a pitched island or other training works for protecting Bhatpur bank.

Alternative training works e. g. a pitched island upstream of the Bridge, did not hold out the same degree of promise.

4. Detailed Experiments for the best position, orientation and dimensions of the repelling groyne for the protection of the Rander bank are in progress. The other measures, tentatively obtained in the exploratory tests, may require modification as a result of detailed tests to follow.

APPENDIX I/C-2 (ii)-7.

Design of Model Scales.

1. River data:—

Maximum discharge above Hope bridge including all spills— Q_{max}	=	10 Lakhs cusecs.
Maximum discharge through Hope bridge	=	7.5 Lakhs cusecs.
Dominant discharge through Hope bridge— Q_{dom}	=	5 Lakhs cusecs.
Waterway of the bridge—A	=	64,500 Sq. ft.
Width at Hope bridge—W	=	1,800 Ft.
Average depth = $d = \frac{A}{W}$	=	35.8 Ft.
$\frac{W}{d}$ ratio	=	50.3
Average slope of river between Mandvi and Surat—S	=	0.391 per 1000
Mean velocity through bridge for maximum Q	=	$\frac{750,000}{67,500}$
	=	11.1 Ft/Sec.
Mean diameter of bed sand = m	=	0.33 mms.

2. Model : —

River reach of about 24 miles, from Kathor bridge down to below Bhatpur, is modelled at the Station. Incoherent Theur sand of 0.40 mms. diameter is used for laying the bed, while the semi-erodible banks are moulded in cohesive puddle made of mixed red and black clay to simulate conditions at site.

(i) Horizontal Scale : —

A suitable horizontal Scale of 1/330 was decided to be adopted.

(ii) Discharge Scale : —

The meander law—meander length $\propto Q^{1/2}$ in general—is fairly correctly applicable to alluvial rivers flowing in flood plains in tortuous courses with naturally formed meanders and hence $Q_{scale} = (L_{scale})^2$ is adopted in the models of such rivers. Tapti is, however, an incised river with bed width to depth ratio, only 50.3, i. e. small for the discharge, indicating little sand charge moving along the bed. Also, unlike an alluvial river, Tapti near Surat has cut through tough and hard formations of clay and the peculiar rectangular meander is solely due to occurrence of local Kunkar beds. Under such conditions velocities would generally be nearly proportional to \sqrt{d} and hence $Q_{scale} = L d \sqrt{d}$ would be more correct. With discharges according to $L d \sqrt{d}$ scale, flow conditions and river gauges were correctly simulated.

(iii) Depth Scale and Vertical Exaggeration : —

(a) According to empirical Station Formula : —

It was observed that the relation $d = 0.688 q^{0.71}$ gave correct depths for the Lower Sind Barrage, Gandak and other models under analogous conditions of model discharge only. Using the same relation, the following results are obtained : —

	Q_{dom}	W(ft.)	q (cusecs/ft.)	d(ft)	$\frac{W}{d}$	D_{scale}	V. E.
Proto.	5 Lakhs cusecs	1800	278	35.8	50.3		
Model	2.48 cusecs	5.45	0.46	0.41	13.30	1/87.3	373

These results apply to wide and shallow rivers carrying relatively more charge than the Tapti, which is an incised, narrow and deep river with a $\frac{W}{d}$ ratio of only 50.3 although discharge is of the order of 10,000,000 cusecs.

(b) According to Lacey formula:—

$$R = \left[\frac{Q}{f} \right]^{1/3} = \frac{Q^{1/3}}{m^{1/6}}$$

$$\therefore D_{\text{scale}} = \frac{D_m}{D_p} = \left[\frac{Q_m}{Q_p} \right]^{1/3} \left[\frac{m_p}{m_m} \right]^{1/6} = 27.2 \times \frac{1.847}{1.79} = 28.0$$

$$\therefore V. E. = \frac{330}{28} = 11.77$$

Lacey's relation (b) is applicable to rivers with little bed movement.

It was, therefore, decided to lay the model initially to a V. E. of 4.58 which was found suitable on running.

(iv) Slope Scale : —

(a) According to Station experience & practice : —

In Jumna, Lower Sind Barrage, Gandak & Mahanadi models, it was observed that with Theur sand of 0.4 mms. diameter and with 4 cusecs discharge, when charge injected was roughly equal to material collected at the downstream end of the model, the stable slope was of the order of 1.1 to 1.2 ft. per 1000.

Sand in Tapti is also 0.4 mms. and dominant model discharge .248 cusecs. With slope of 1.2 ft/1000, S. E. = $\frac{1.2}{0.391} = 3.07$ (average slope in the Tapti being 0.391 per 1000).

Unlike the rivers mentioned above, Tapti is, however, an incised river, relatively narrow and deep, indicating little bed charge and hence a flatter slope is appropriate. The slope required for the model would, therefore, be flatter than 1.2 ft/1000.

(b) According to Lacey Formula:—

$$S. E. = \left[\frac{f_m}{f_p} \right]^{5/3} \left[\frac{Q_p}{Q_m} \right]^{1/6} = \left[\frac{1.11}{1.01} \right]^{5/3} \times 20200^{1/6}$$

$$= 1.1^{5/3} \times 5.2 = 1.172 \times 5.2 = 6.1$$

Scale ratios worked out above are tabulated below:—

Scales:	According to Lacey formula.	Based on experience of model experimentation at the Poona Station.
(1) Horizontal	—	$\frac{1}{330}$
(2) Discharge	—	$\frac{1}{20200}$
(3) Depth Scale	$\frac{1}{28}$	$\frac{1}{87.3}$
(4) V. E.	11.77	3.78
(5) Slope Exaggeration	6.1	3.07

As there is still unfortunately some prejudice against the use of tilted models, to start with the model was laid with V. E. = S. E. = 4.58, giving depth scale $\frac{1}{72}$. Trial runs were made to test whether model behaviour was satisfactory with these scales. Velocities in the model were excessive, especially

at the sharp bends at Variao and Rander as indicated by severe action along the bank and rapid bed movement. Upstream of Surat in the left embayment the return and slack flow regions were also disproportionately bigger in the model than in the river.

As an improvement, S. E. was made = 3.07 keeping V. E. = 4.58 as before ; but the conditions were still unsatisfactory.

The difference between model and river conditions was attributed to excessive slope given to model—*vide* IV (a) *supra*.

Guided by the behaviour of the model, the slope was flattened to about 1 ft/1000 giving S. E. = 2.5 against V. E. = 4.58. A tilted model with these scales behaved satisfactorily in respect of conditions of flow and movement of bed sand.

(v) *Time Scale* : —

According to the tractive force method :

$$\begin{aligned} \frac{T_p}{T_m} &= \frac{L_p}{L_m} \times \frac{S_p}{S_m} \times \left[\frac{d_p}{d_m} \right]^{1/2} \\ &= 330 \times \frac{1}{2.5} \times 72^{1/2} = \frac{330}{2.5} \times 8.48 = 1118 \end{aligned}$$

i. e. 1 hour in river = 3.20 seconds in the model.

(vi) *Sand Charge* : —

Quantity of sand injected at the upstream end of the model was adjusted till the material collected at the downstream end was about equal to the injected charge. This was found to be 0.04 part per 1000 by weight, at the dominant discharge of 2.48 cusecs \equiv 500,000 Cusecs.

C. HYDRAULIC WORKS

2. River Control

(iii) - Scour & erosion.

C-2 (iii) - 1 Mid-season scouring operations of the Right Pocket of Sukkur Barrage.

1. The Sukkur Barrage authorities have had under consideration the proposal of having additional scouring operations⁽¹⁾ of the Right Pocket of Sukkur Barrage in the months of June, July and August without closing the Canals, in the hope that these flushings will keep the pocket clear and thus provide a silt trap where silt may deposit instead of being carried into the canals.

2. The following note discusses whether such scouring operations are necessary or likely to be useful.

Analysis of data.

Lines of flow in the Right pocket were observed in the 1/80:1/40 part-width model for the following conditions:—

Fig. 1
C-2(iii)-1

Bed laid according to soundings on 10.9.46, i. e. before scouring operations.

Approach channel	Q	=	45,000	cusecs.
Tail channel	... Q	=	27,000	cusecs.
Pocket	... Q	=	18,000	cusecs.
N. W. Canal	... Q	=	4,500	cusecs.
Rice Canal	... Q	=	10,000	cusecs.
Dadu Canal	... Q	=	3,500	cusecs.

Fig. 1 (a)
C-2 (iii)-1

3. In figure 1 (a), a, b, c, d and f are sections in the pocket at right angles to the lines of flow observed in the model at the corresponding positions. Depths along these cross-sections are shown in figures 2 and 3 for years 1945 and 1946, before and after the scouring operations were carried out in September. The approximate extent of return flow is shown at each section and this area is excluded in calculating A, the effective area of the section.

Fig. 2 & 3
C-2(iii)-1

Table 1
col 2

C-2 (iii)-1

4. In fig. 4, the change in depths at sections 11, 12, 13, 14 and 15 are shown for one cycle i. e. 'after scouring operation' on 19.9.45 to 'after scouring operation' on 10.9.46. From fig. 4 it is seen that appreciable silting took place below section 13 (i. e. section passing through the middle of the Rice Canal Head Regulator) and it was considerable at sections 14 and 15 (sections 14 at upstream end of Dadu Canal Head Regulator and section 15 near the downstream end). Very little silting, however, took place at sections 12 and 11. From this, it is clear that in spite of the increase in sand and silt charge in the river due to floods after 4.6.46, practically little silting took place down to the middle of the Rice Canal Head Regulator. Further downstream, deposition took place; but this is to be expected as the pocket channel must adjust itself to the decreased discharge after withdrawals by canals. Near the Dadu Canal entrance silting was maximum; but this was also natural, as the balance discharge for this canal is only 3500 cusecs.

Fig. 1 (a) &
4
C-2(iii)-1

In table No. 1, it is shown that the stable section finally attained still provides sufficient cross-sectional area as assessed from Lacey's formula for the particular discharge, thus:—

Table 1
C-2 (iii)-1

P_L , the wetted perimeter, was obtained from Lacey's formula

$$P_L = 2.668 Q^{1/2}$$

Mean diameter of the sand in the pocket was assumed to be 0.7 mm. (it actually varied between 0.15 to 0.20 mm.) which gave $f = \sqrt{m} \times 1.76 = .73$. R.L. = Lacey's hydraulic depth was obtained from $R_L = 0.473 \left(\frac{Q}{f} \right)^{1/3}$.

(1) Letter from Executive Engineer, Barrage Division, Sukkur No. D. 2, Sukkur dated 14.6.47

6. From the comparison of actual areas, at sections a, b, c, d and f, below R.L. 195, before and after scouring operations, and those obtained according to Lacey formulae for the corresponding discharges, it is seen that at section (a) the actual area was much more than the Lacey area both before and after scouring; at section (b), the actual area was slightly less before scouring operation but more after the scouring operation; at section (c) it was slightly less both before and after the scouring operation, while at sections d and f it was nearly the same after scouring as before scouring. On the whole, the available section of the channel can be considered to be *commensurate* to that required for carrying the desired discharge, even before the September scouring operations. Experience has shown that any surplus capacity is soon lost, as is only to be expected.

7. The usual scouring operation after Abkalani effectively flushes the pocket and the original bed levels are restored, as will be seen from fig. 4. There is *no progressive silting* apparent in the pocket such as would endanger the operation of the Right Bank canals. So additional scouring operations in the months of June to August seem to be unnecessary and of little value. In fact, the present practice of one scouring operation after Abkalani is apparently sufficient.

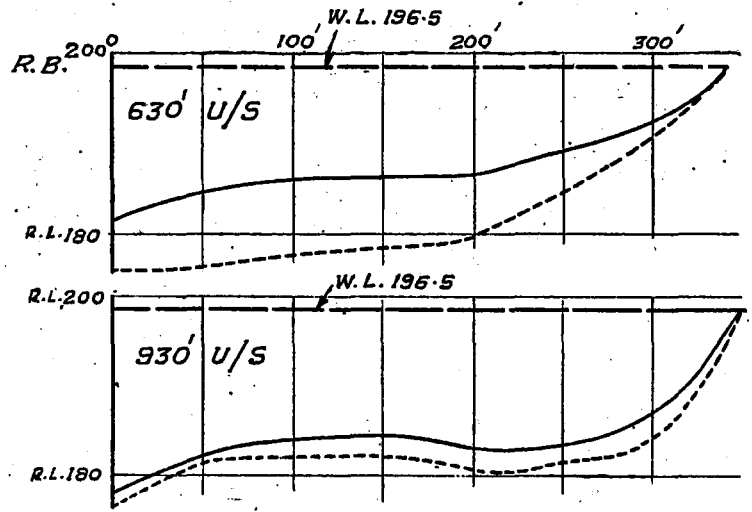
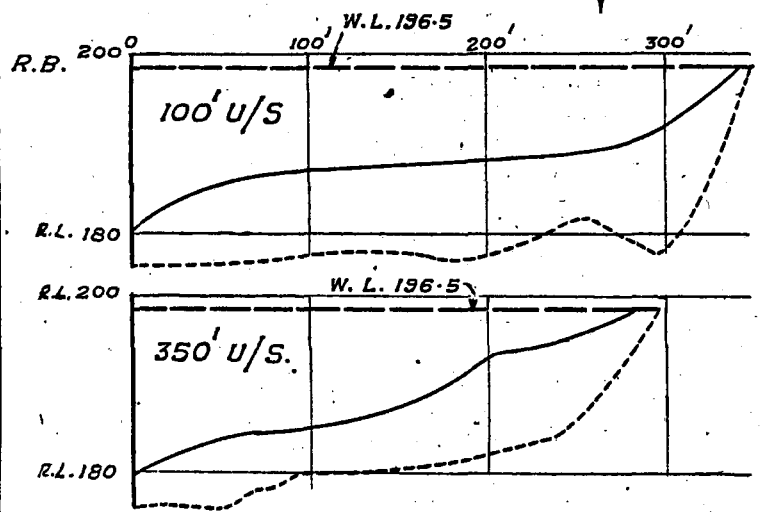
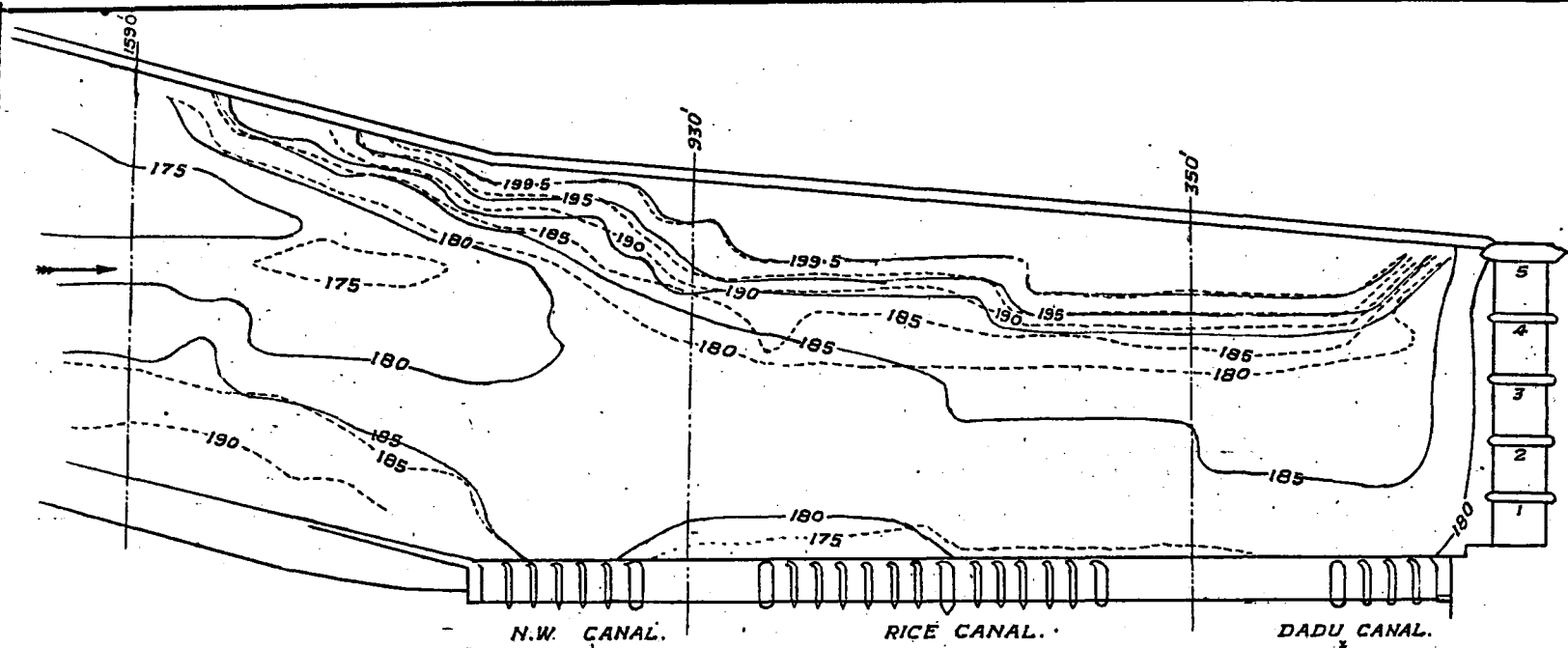
8. The second suggestion of the local officers of having scouring operations of short durations with canals working with reversal of the ratio of tail channel discharge to pocket discharge, is clearly undesirable, as it would reverse the favourable curvature at the raised sill and attract very heavy sand charge into the pocket, as shown by model experiments, described in item (C-10-1).

FIG. 1/C-2(iii)-1.
PLAN SHOWING BED CONTOURS
IN THE RIGHT POCKET
BEFORE & AFTER SCOURING
OPERATIONS.

SCALE - 1" = 200 FT.

REFERENCES.

SILTED BED ON 10-9-46 ———
SCOURED BED ON 12-9-46 - - - -



CROSS SECTIONS U/S OF THE
POCKET SLUICES IN THE RIGHT POCKET
BEFORE & AFTER SCOURING.

SCALE -
HOR. 1" = 100'; VER. 1" = 20'

REFERENCE
BEFORE SCOURING 12-9-46 ———
AFTER " 14-9-46 - - - -

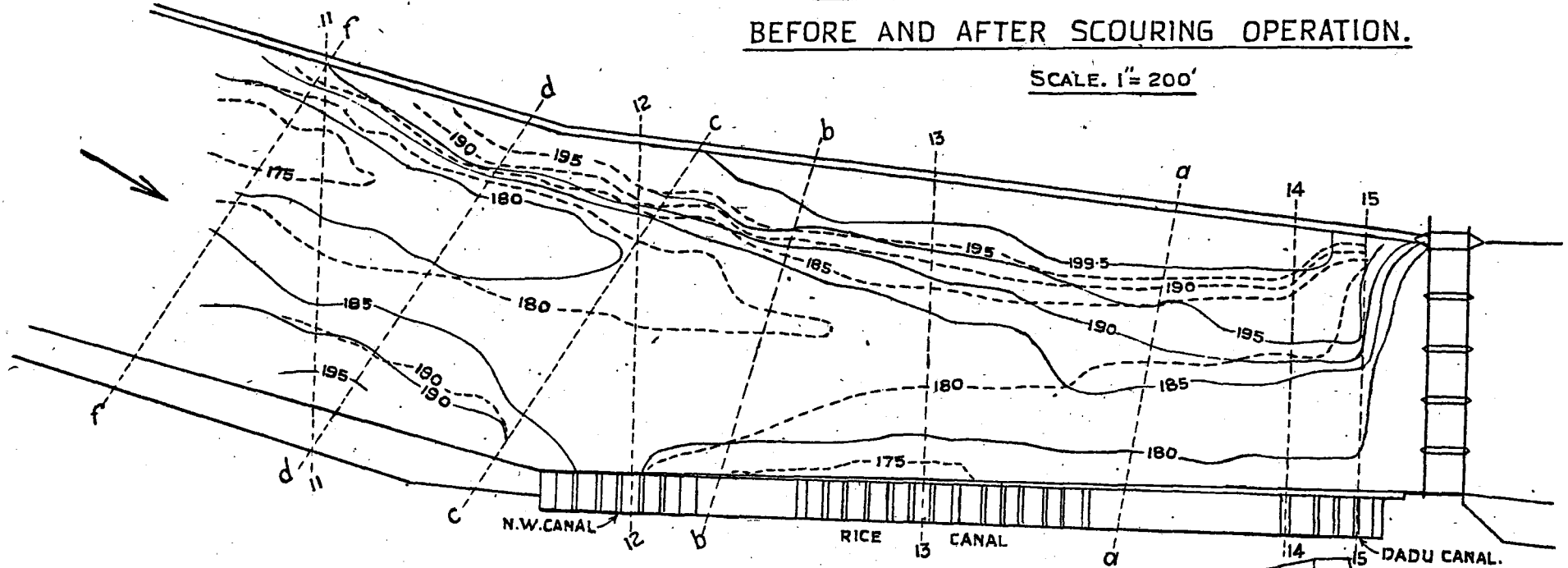
Reference:-

- R.L. OF SILTED BED ON 15-9-45 SHOWN IN FULL LINE.
- - - R.L. OF SCoured BED ON 19-9-45 SHOWN IN DASH LINE.

FIG. 1(a)/C-2 (iii)-1.

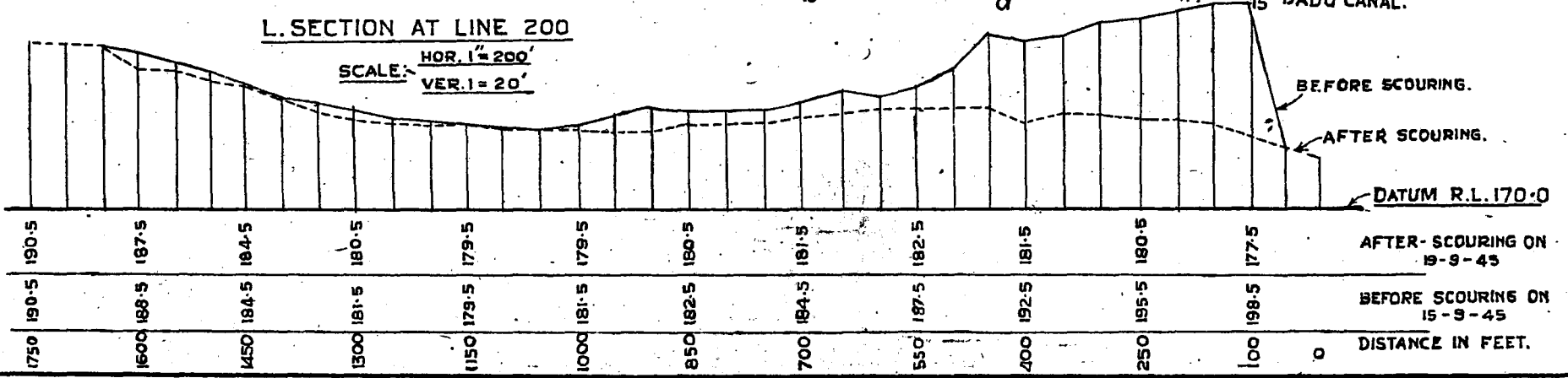
PLAN SHOWING BED CONTOURS
BEFORE AND AFTER SCOURING OPERATION.

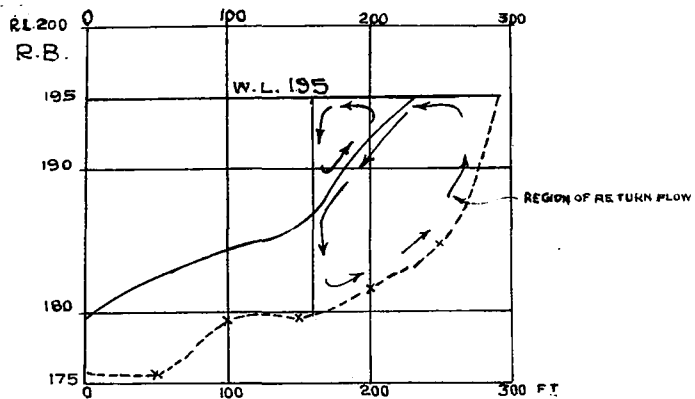
SCALE. 1" = 200'



L. SECTION AT LINE 200

SCALE: HOR. 1" = 200'
VER. 1" = 20'

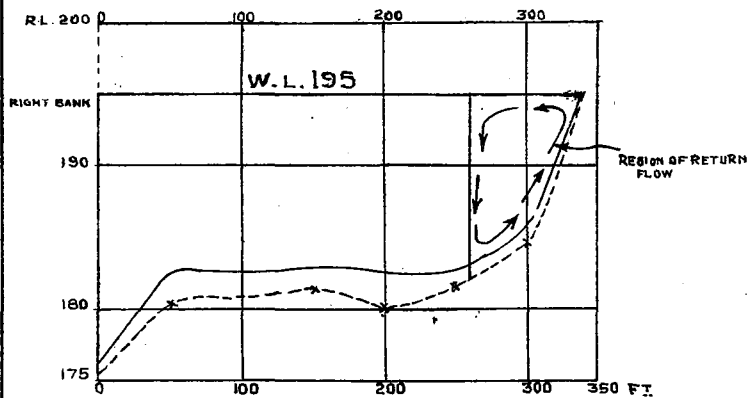




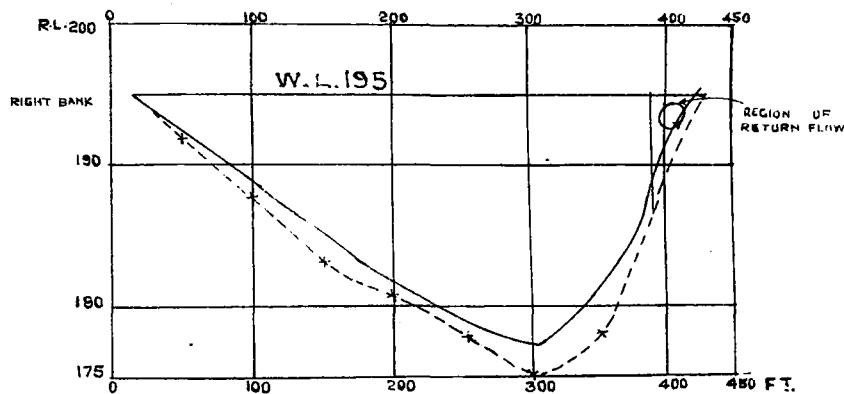
C. S. (a) — 1870 SQ. FT.
- - - 2800 SQ. FT.

FIG. 2/C-2(iii)-I.
SHOWING DEPTHS AT SECTIONS
(a) (b) (c) (d) (f) BEFORE & AFTER
SCOURING OPERATION IN SEPT. 1945

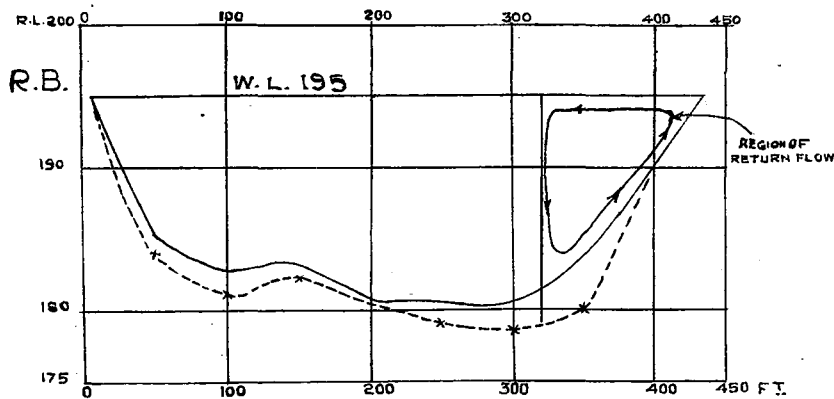
— BEFORE SCOURING. ON. 15-9-45
- - - AFTER SCOURING. ON. 19-9-45
SCALE: } HOR. 1" = 100'
VER. 1" = 10'



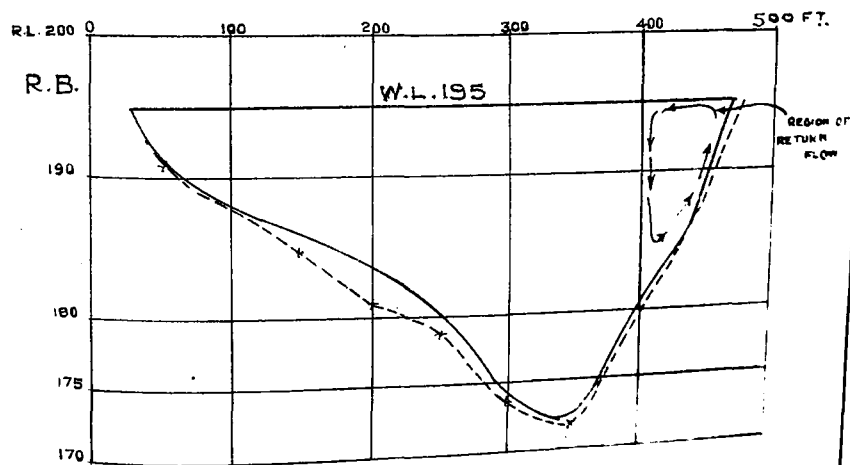
C. S. (b) — 3550 SQ. FT.
- - - 4100 SQ. FT.



C. S. (d) — 4200 SQ. FT.
- - - 4780 SQ. FT.



C. S. (c) — 4030 SQ. FT.
- - - 4470 SQ. FT.



C. S. (f) — 5100 SQ. FT.
- - - 5400 SQ. FT.

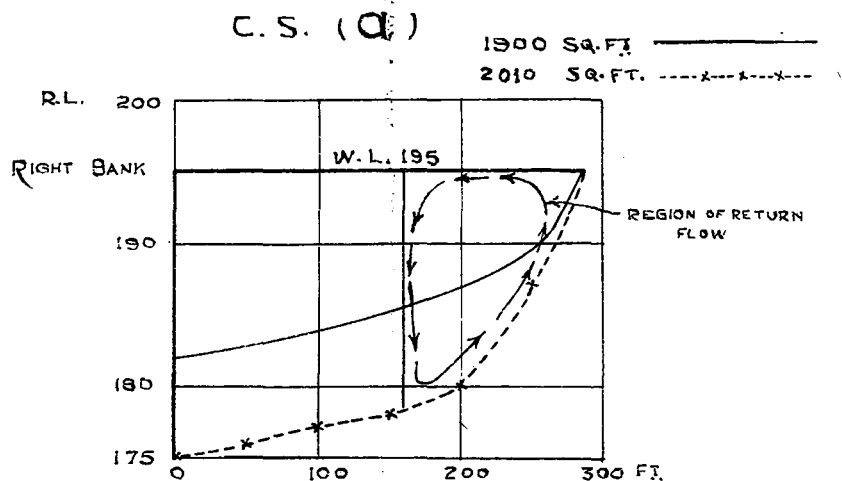
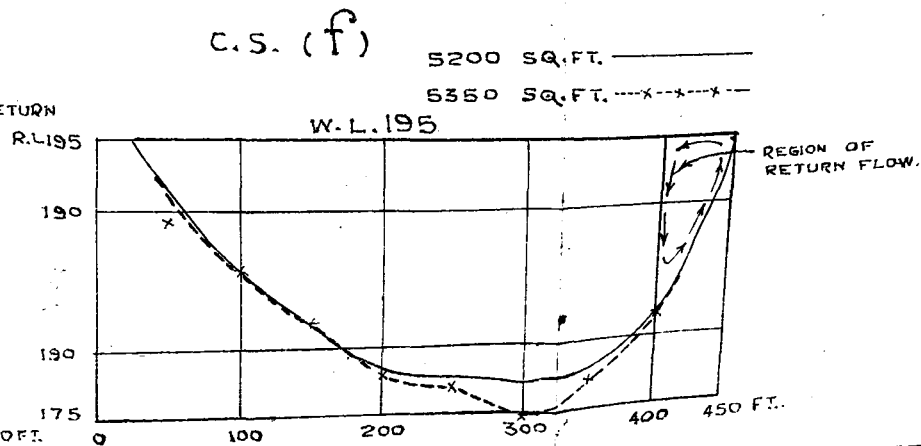
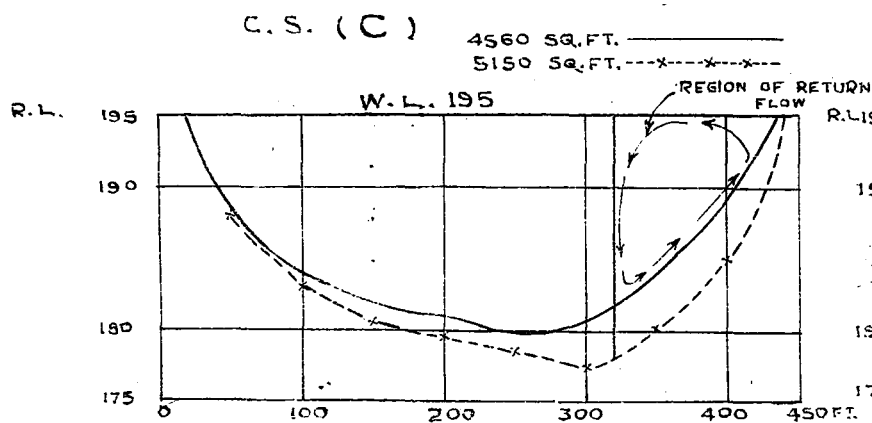
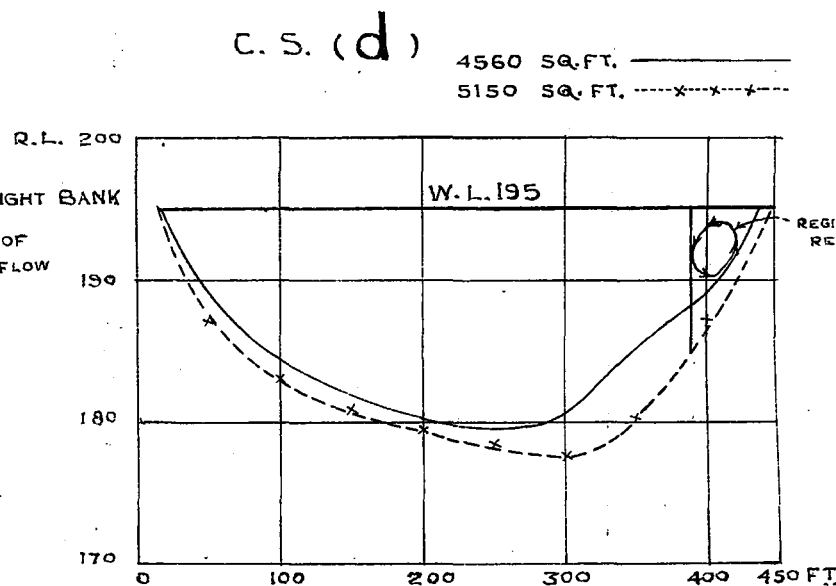
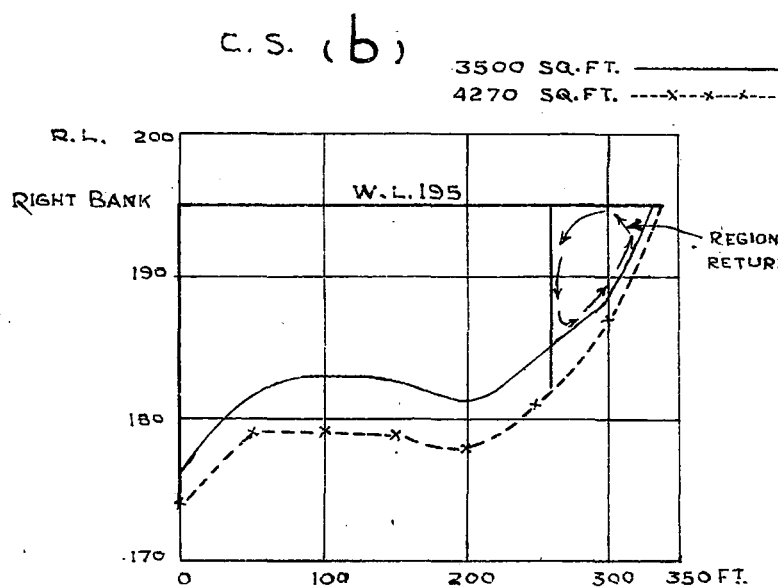


FIG. 3/C-2(iii)-1
SHOWING DEPTHS AT SECTIONS
(a) (b) (c) (d) (f) BEFORE AND
AFTER SCOURING OPERATION
IN SEPTEMBER 1946

BEFORE SCOURING ON 10-9-46. —————
AFTER SCOURING ON 12-9-46. - - - - -



RIGHT BANK.

C.S. 11

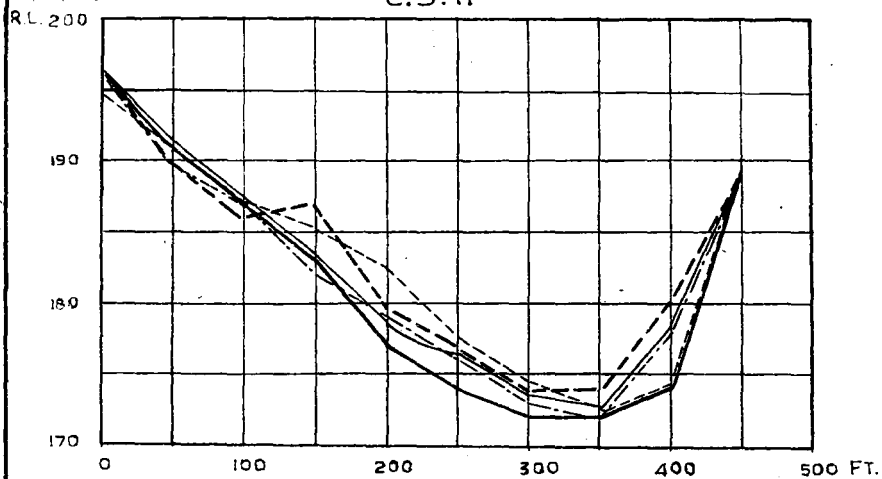
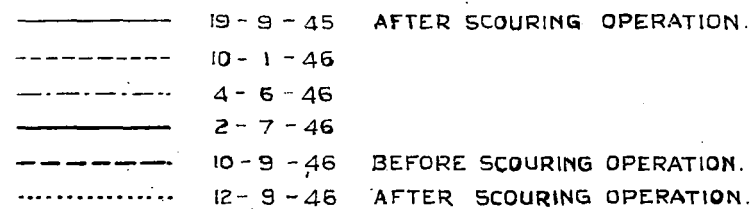


FIG. 4/C-2(iii)-1.

SHOWING DEPTHS AT CROSS SECTIONS

11, 12, 13, 14 & 15 ON



SCALE:— {
 HOR. 1" = 100'
 VER. 1" = 10'

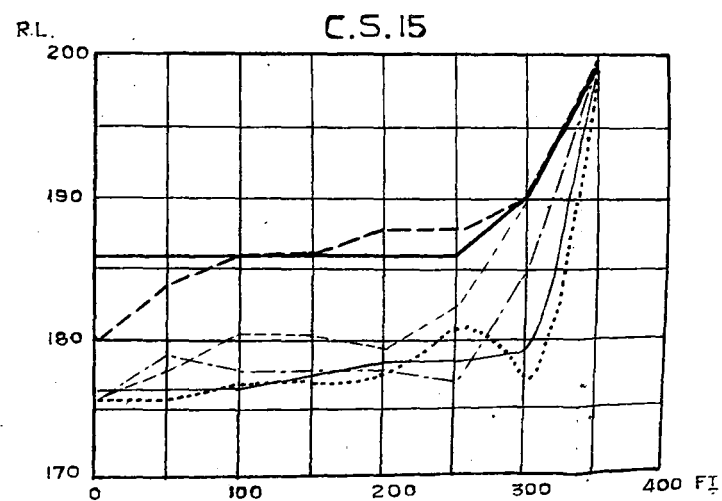
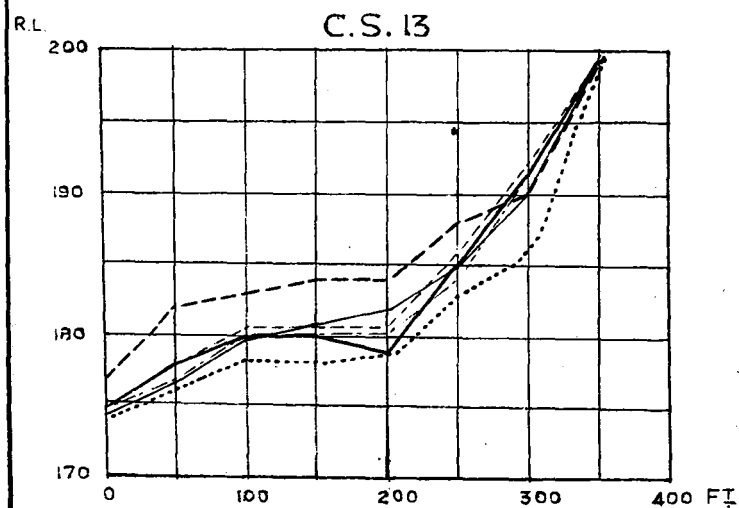
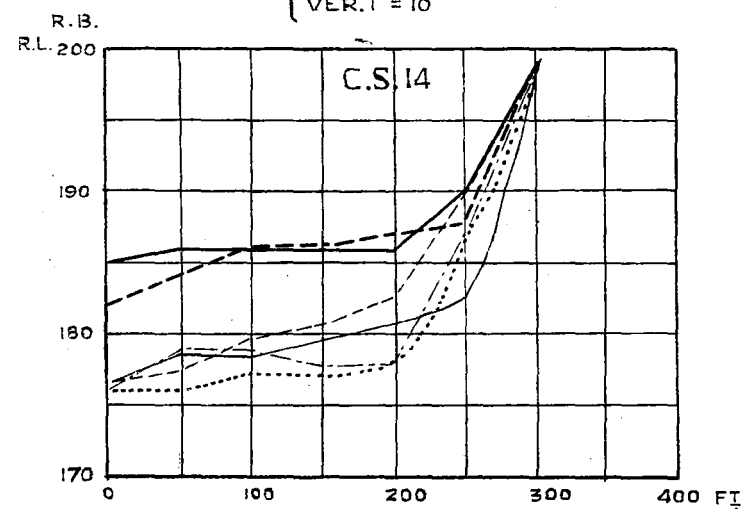
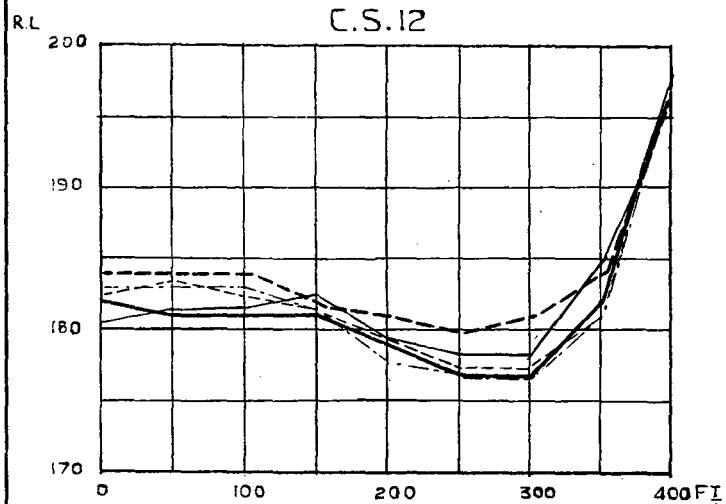


Table 1/C-2 (iii) - 1

17	Existing conditions in the Rt. pocket				Dimensions of channel calculated according to Lacey for Q in column 5.				$\frac{A}{A_L} = \frac{\text{col. 2}}{\text{col. 9}}$
	A = effective area of cross section below R. L. 195.0 (sq. ft.)	P wetted perimeter (ft.)	R hydraulic Radius = A/P (ft.)	Q at the section (cusecs)	P_L Lacey's = $2.668 Q^{1/2}$ (ft.)	f Lacey's = $1.76 \sqrt{V_m}$	R_L Lacey's = $0.473 (Q/f)^{1/3}$ (ft.)	A_L Lacey's = $R_L \times P_L$ (sq. ft.)	
	2	3	4	5	6	7	8	9	10
Year 1945									
Section (a)	1870	195	9.6	} 3500	158.0	0.73	7.9	1250	1.5
Before scouring	2800	205	13.6						
After "									
Section (b)	3550	300	11.8	} 13500	310.0		12.4	3850	0.923
Before scouring	4100	310	13.2						
After "									
Section (c)	4030	330	12.2	}					0.815
Before scouring	4470	335	13.3						
After "									
Section (d)	4200	390	10.8	} 18000	360.0		13.7	4925	0.85
Before scouring	4780	400	11.9						
After "									
Section (f)	5100	400	12.7	}					1.035
Before scouring	5400	410	13.2						
After "									

(Mean diameter of sand in pocket = 0.17 mm.)

**C-2 (iii)-2. Apron design for the South Guide Bank of the Alexandra
Bridge on the Chenab, N. W. Railway.**

The cold weather survey of 1947 of the Chenab river at Alexandra Bridge revealed that the apron of the left guide bank between chainages 6 to 9 and at chainage 12 had launched to a slope steeper than 2 : 1 for which it was originally designed. The Director's opinion regarding the necessity of providing additional stone to ensure adequate protection to the guide bank was, therefore sought.

Fig. 5.
C-2 (iii)-2

With parallel flow conditions, the stone pitching, generally, is expected to assume a side slope of 2 : 1. In the present case however, the main current after leaving the Pindi Spur nose heads directly to the south bank, thus creating severe curved flow conditions. The profile of the guide bank generally associated with such conditions is shown in figures 6 and 7. This in itself, should not be a source of anxiety unless accompanied by other signs of weakness or instability, such as insufficiency of stone thickness of the launched apron.

Fig. 6 and 7
C-2 (iii)-2

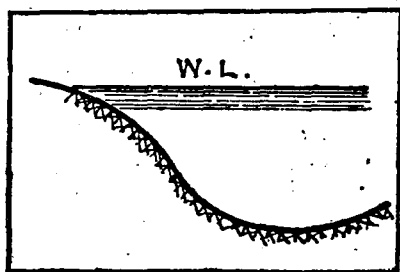


FIG. 7/C-2 (iii)-2.

With curved flow, conditions similar to those at the head of a large-radius guide bank are obtained and as maximum scour in such a case is observed to be of the order of $2R$ (Lacey), the apron should be designed for scour below $H. F. L. = 2R$, (R being the Lacey normal depth)—and which in this case = 45 ft. below L. W. L.

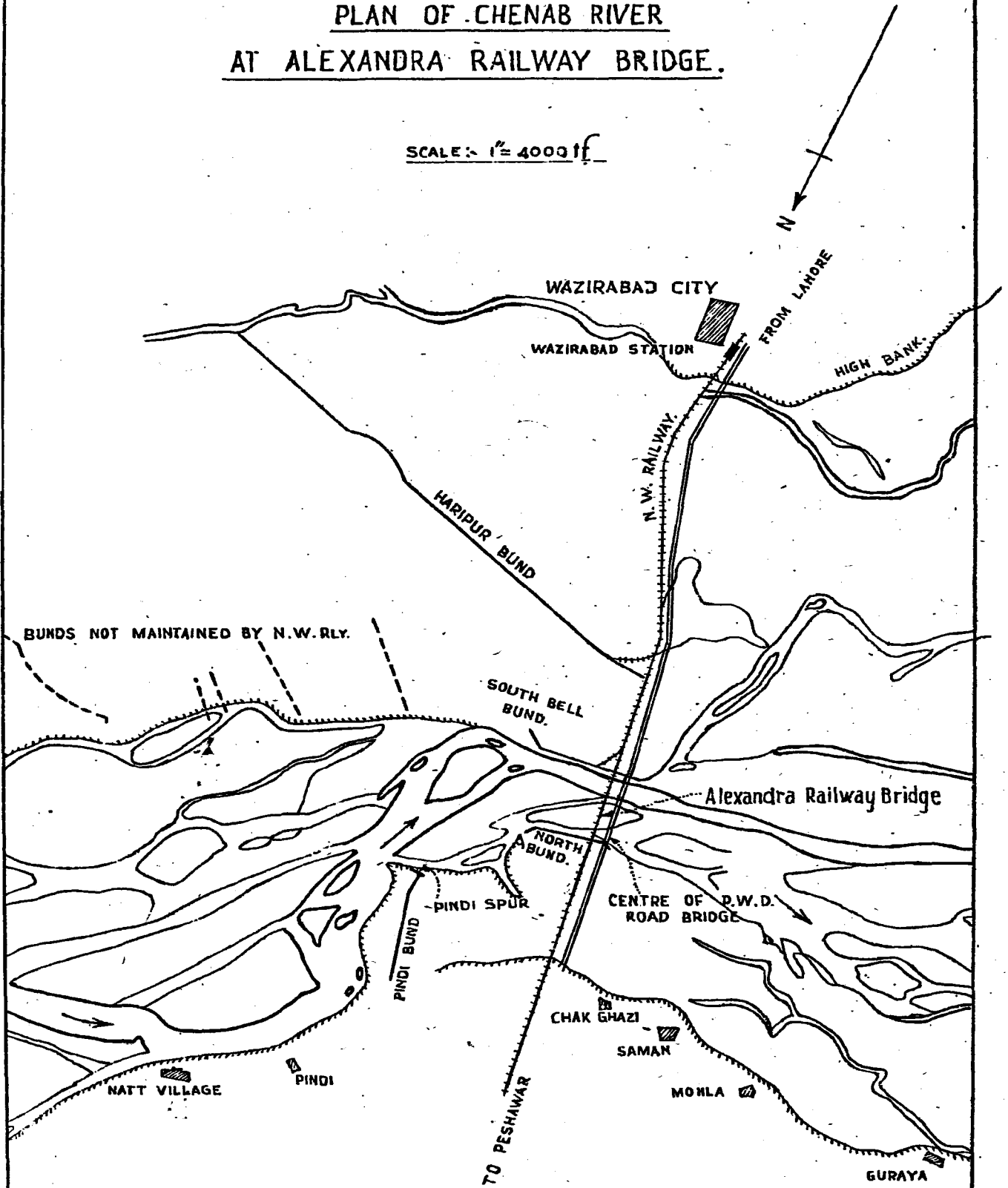
It was, therefore, recommended that—

- (1) provided there are no other signs of weakness, it is not essential to endeavour to maintain the launched apron to the unnatural slope of 2:1 in continuation of the permanent slope;
- but (2) additional stone apron should be provided up to a distance of (a) 68 ft. (1.5×45) from the present toe of the launched apron or (b) to the bottom of the scour if it be deeper than 26' 8" below L. W. L.— $1.5 R$ (Lacey)—whichever is more.

FIG. 5/C-2(iii)-2

PLAN OF CHENAB RIVER
AT ALEXANDRA RAILWAY BRIDGE.

SCALE:- 1" = 4000 ft



SECTION ON CH. 8

FIG. 6/C-2 (iii) - 2

SECTION OF SOUTH BELL BUND

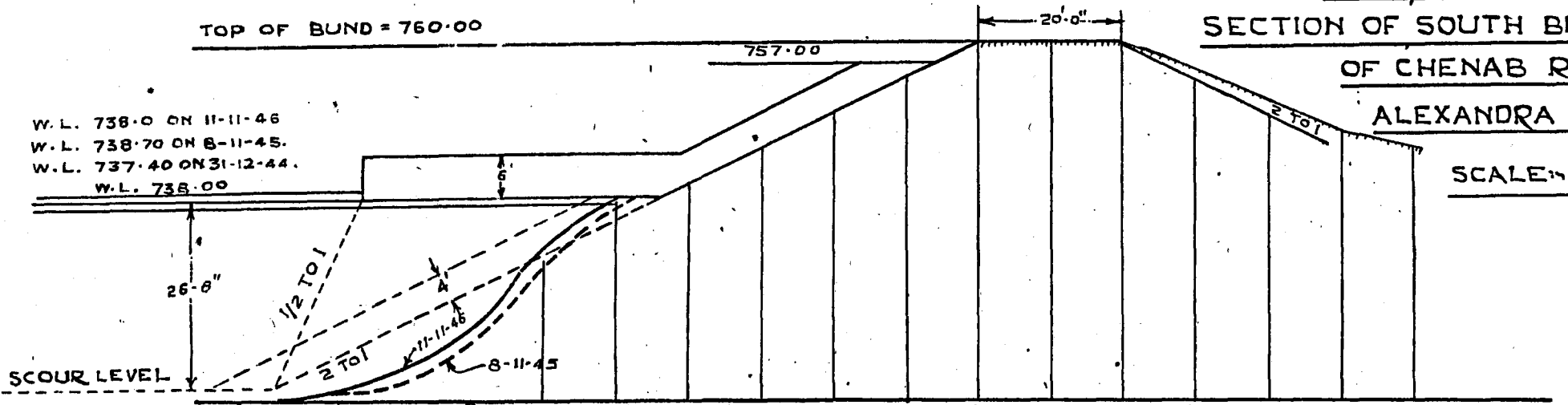
OF CHENAB RIVER

ALEXANDRA BRIDGE

SCALE: 1" = 20'

TOP OF BUND = 760.00

W. L. 738.0 ON 11-11-46
W. L. 738.70 ON 8-11-45.
W. L. 737.40 ON 31-12-44.
W. L. 738.00



DATUM R.L. 710
31-12-44

710.00	711.00	715.00	720.00	729.00	737.10	738.56	747.00	751.80	756.30	759.40	759.74	759.61	756.27	751.61	747.52	744.30
--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------

8-11-45

713.50 718.70 ← SAME AS IN 1944 →

11-11-46

715.00 723.00 731.60 738.90 ← SAME AS IN 1944 →

DISTANCE IN FEET.

110	100	90	80	70	60	50	40	30	20	10	0	10	20	30	40	50
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**Central Waterways, Irrigation & Navigation Research
Station, Poona 1.**

C-2 (iii)-3. SCOUR BELOW WEIRS.

By

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Synopsis :

Prototype and model data of depths of scour below weirs and their ancillary works, are analysed with a view to having a rational basis for the design of apron protections. Limitations of models used for the purpose and the pitfalls to be avoided in interpreting their results are indicated. Two illuminating model investigations, one for scour below a bridge over the Ganga and the second for scour below a Barrage on the Indus, are briefly set out. A close comparison of the results of model tests against prototype behaviour shows conformity to a degree, when the model is correctly operated and interpreted, but it also reveals the need for more prototype data in relation to the results of model investigations and independent basic experiments on scour.

1.0 Introduction:

1.1 In the case of weirs, deep scour occurs—

- (i) downstream of the rigid structure due to eddies shed from the high pavements, even when the energy of the hypercritical jet has been adequately dissipated by a glacis and a cistern;
- and (ii) along and at ends of Divide Walls, Guide Banks and Spurs due to concentrated flow swinging and diving round their extremities.

In all these cases, the scour is due to the bed velocity being far in excess of what is natural for the depth— V^2/d which is related to turbulence being very high—so that the results of violent eddies are superimposed over diving, curved flow.

1.2 The current practice in India is to estimate the maximum likely scour under abnormal conditions from the normal depth, D , for *regime* conditions in a straight channel with a level bed, as obtained from Lacey's⁽¹⁾ formula :—

$$D = 0.473 (Q/f)^{1/3} \quad \dots \quad \dots \quad \dots \quad (i)$$

$$\text{or } D = 0.9 (q^2/f)^{1/3} \quad \dots \quad \dots \quad \dots \quad (ii)$$

where

Q = aggregate discharge,

q = discharge intensity in cusecs/ft.,

f = Lacey's silt factor which for regime conditions approximates to $1.76 \sqrt{m}$, where m is the weighted mean diameter of the bed material in mms.

For design purposes Lacey defined the values to be adopted for various classes of scour, thus :—

Class A - Straight reach (e. g. anywhere just below flexible aprons.)	1.27 D
Class B - Moderate bend (e. g. along aprons of guide banks in a straight reach)	1.50 D
Class C - Severe bend (e. g. at and below noses of guide banks or where vortices are anticipated)	1.75 D

(1) Lacey's Stable channels in Alluvium, Paper No. 4736 (1930), Inst. of Civil Engineers, London.

Class D - Right angled bend (e. g. at and below noses of guide banks or where vortices are anticipated.) ... 2.0 D
(N. B.- D is, generally, measured below maximum water level).

1.3. The main purpose of this note is to examine the actual maximum depths of scour in relation to the corresponding maximum discharges, in nature and in the model.

In the 1944 Annual Report (Tech.) (pp. 78-83) of the Station, similar data have been examined for—

- (1) scour round piers of bridges,
 - (2) scour at noses of guide banks, groynes and sharp bends,
- and (3) scour downstream of bridges.

It was found that scour may be *much greater* than estimated by Lacey.

2. Scour downstream of Weirs :

2.1. Scour downstream of weirs occurs due to one or more of the following reasons :—

- (a) excess energy of hypercritical flow when insufficiently dissipated by a standing wave ;
 - (b) an unstable standing-wave forming below the toe of the glacis and shifting beyond the rigid floor;
- and/or (c) the pavement level being too high, making the bed velocity abnormally high and causing violent eddies to be shed from the downstream end of the pavement.

Scour due to cause (c), *ante*, may be deeper than that due to either **Statement I** (a) or (b). Deep scour *downstream of bridges* occurs for precisely the same reason as (c), and data for scour below bridges is thus of great value and has been included in Statement I. It must, however, be remembered that in a weir, waterway may differ materially from Lacey's "minimum stable waterway", P_w (where $P_w = 2.67 \sqrt{Q}$) and, therefore, *formula (i) is not applicable to weirs but (ii) is* : while at bridges narrowing can be done to nearly the minimum stable width, such narrowing in the case of a weir may expose it to serious risks.

2.2. Though Lacey brought grade of material into his formula to the $1/6$ th power — see para 1.2 — recently he, and others in India, have argued that the depth of scour downstream of rigid structures is unaffected by bed material, being dominantly due to flow pattern; and as the range of bed material in practice is generally small (0.25 to .4 mms.) data could not be taken either to support or disprove this belief; but it has been found at Poona that even though flow pattern is the *dominant* factor, f is undoubtedly a factor affecting maximum scour and that $f^{1/3}$ is probably of the right order, within the range of grade— .06 to .37 mms.—examined. On the other hand, in recent experiments on scour below siphon spillways, we have also found that maximum scour depths were sensibly the same with grades of material 17 mms. and 6 mms. in diameter.⁽²⁾

It will be recalled that Dr. Ing. Theoder Rehbock who found the stable final form of the model-bed, within wide limits, much the same with different sizes of bed material has, however, stated⁽³⁾ that the similarity of the scour-holes would be lost " if the mean diameter of the particles forming the model-bed were to become sensibly smaller than 0.5 mm. "

This is understandable because the settling velocity of sand particles = $(\text{dia.})^2$ for m up to 0.10 to 0.12 $\frac{1}{2}$ mms., and = (dia.) for m up to 0.5 mm., and = $\sqrt{\text{dia.}}$, for m exceeding 0.5 mms.; so that depth of scour must depend little

(2) See Annual Report (Tech.) of L. W. E. Stn., Poona, for 1946, p. 91.

(3) Rehbock : in discussion of Paper by Furno & White, Jour. Inst. C. E., October, 1939, p.260 also in discussion of Paper by Eutcher & Airkinson Proc. Inst. C. E. Vol. 235(1934), p. 256

on diameter if its grade exceeds 0.5 mms., the effect due to viscosity becoming negligible.

2.3. Comparing the scours below the Mahanadi Central Sluices and the Sarda Barrage, in spite of the fact that Sarda is a *boulder* river its relative scour depth (d_s/D_{Lacey}) was much *greater*; in addition to the doubtful applicability of the Lacey formulae to boulder rivers, this is explained by the fact that in the Sarda the flow upstream was very *oblique* and the downstream flexible apron was very disturbed, both inordinately augmenting the turbulence and scour depth: while the scour below the Mahanadi Central sluices was merely due to shooting flow and want of dissipation of energy by a correctly positioned Standing Wave.

2.4. The predominant factors, determining the maximum depth of scour downstream of structures like barrages and bridges *over wide rivers* are the discharge intensity and the turbulence (as indicated by v^2/d), the effect of sides being negligible: other important factors are the flow pattern and the grade of bed material, as mentioned in para 1 above. Whenever a rigid structure constricting the waterway is built across a natural channel, the velocity and discharge intensity being greater than in the channel, when the water leaves the structure and flows over the bed of the natural channel the depth of flow gradually *increases* downstream of the masonry floor inducing a scour-hole and then gradually *decreases* till the stable depth is reached again.

In *narrow* channels, however, embayments ("bellies") also occur downstream of structures, as observed in the case of bridges and regulators over the Rice Canal, the North Western Canal and the Rohri Canal in Sind, causing *return flow* on the sides and *jetting* in the middle. The bellied sides have an appreciable effect on the maximum depth of scour reached in canals where the width of channel is not large compared to the depth; model experiments confirmed that the deep scour was due to jetting consequent upon return flow along the banks which caused the embayments and when the embayments were prevented much less scour occurred. (See Annual Report (Tech) 1944, pages 23-25).

2.5. It is thus apparent that scour below structures in *narrow* channels is complicated precluding basic treatment, as it is affected by the width to depth ratio and, therefore, requires individual treatment. Scour below weirs or bridges in a *wide* river is of a more simple type being unaffected by side effects. So basic experiments are now in progress at the Station to determine, in very wide channels, the correlation between various factors, principally the discharge intensity and V^2/d at the structure and the maximum depth of scour below it. (Some of the experiments already carried out are described in the Annual Report of 1945). These experiments will give not only the *maximum depth* of scour but also the *position* of relative scour depths, about which little is known: although for obvious reasons a lesser scour in the immediate vicinity of a rigid structure would be more harmful than maximum scour occurring further downstream.

2.6. The *maximum* scour downstream of bridges is of the order of $4 \times D_{\text{Lacey}} = 4 \times 0.473 (Q/f)^{1/3}$, with high level pavements or pitchings.

Since the waterway provided in the weirs is larger, the maximum scour downstream of weirs and barrages is of the order of $2 \times 0.9 (q^2/f)^{1/3}$; but in the latter case, *q must include an allowance for the concentration of flow*. As concentration of flow is a most important factor, specific experiments on a river model should be used, wherever possible, to obtain the discharge intensity, if not the scour.

3.0 Scour at guide-banks :

3.1. The usual factors,* adopted in design of flexible (loose stone) aprons for guide banks, in terms of the Lacey regims depth D , are :—

At	Range	Mean
Noses of guide banks.	2.0 to 2.5 D	2.25 D
Transition from nose to straight.	1.25 D to 1.75 D	1.5 D
Straight reaches of guide banks.	1.0 to 1.5 D	1.25 D

Experiments and experience, however, show that the actual scours are much deeper: and Statement II shows a wide variation depending upon the severity of the conditions.

Scour at the noses of guide banks projecting straight upstream is very deep, up to $3.8 D_{Lacey}$: while guide banks which swing round as arcs of large radius give lower values = $2.75 D_{Lacey}$.

Spurs projecting from banks give widely varying values of scour depth, ranging from 1.7 to $3.8 D_{Lacey}$, depending upon the severity of river curvature induced, which in turn depends upon the length, orientation and position of the spur relative to the attack, as well as the slope of the nose. While the scour in the Yamuna river at Groyne 5 near Delhi where flow merely brushes past was $1.7 D_{Lacey}$, the scour at the noses of the spurs on right bank of the Ravi upstream of Belloki weir was of the order of 3.5 to $3.74 D_{Lacey}$.

3.2 Statement II shows, in addition to maximum scour observed in rivers, *inter alia*, the corresponding scours obtained in the vertically-exaggerated models of the Ganga River at the Hardinge Bridge and the Yamuna near Delhi Gate Pumping Station at Spur 5. In the former, d_s was 2.3 ft. for a total discharge of 8 cusecs and was consistent with the depth observed in the Ganga; in the Yamuna model, d_s was 1.0 ft at Spur No. 5 for $Q = 2.8$ cusecs, which was relatively slightly less than in the river. Thus, vertically-exaggerated models may give nearly correct scour depths but not the "scour factors" or extent of scour—see para 4.1 below.

3.3. The models have clearly demonstrated what has been generally observed that the deep scour at the noses of guide banks and divide walls, projecting straight upstream, is caused by vertical vortices, i. e. vortices having a vertical axis; but scour depths in the model are relatively much less than in the prototype. Such vortices or swirls have a very deep boring action in the prototype and can dig out even very heavy concrete blocks to a great depth. Such vortices are caused by the slightest obliquity of flow. The failures of the upstream nose of the river guide bank at Suleimanki in 1925 and of the Rasul Weir in 1929 are attributed to more than anticipated action caused by vertical vortices.

4.0. Model Experiments on Scour :

A very large number of experiments have been done on scour below rigid structures at the Khadakwasla Station and these are reported in the Annual Reports of the Station—see Appendix I. Space only permits brief recapitulation of one important investigation on scour below the Hardinge Bridge and the experiments on the model of the Kotri Barrage which are not yet published.

* These values are offered as provisional guide which may have to be modified subsequently as a result of more exact knowledge, on page 155, C. P. I. Publication No. 12, Khosla's "Design of Weirs on Permeable Foundations."

4.1. Scour downstream of a rigid structure depends principally on four factors :-

- (1) Curvature of flow upstream and consequential concentration of intensity of discharge;
 - (2) flow pattern, particularly at noses;
 - (3) depth:velocity relation, upstream and downstream;
- and (4) material of bed.

(1), (3) and (4) require *vertical exaggeration* while (2) requires *geometrical similarity*, so a close compromise is necessary.

A vertically-exaggerated model gives, in general, nearly correct maximum depth of scour like a geometrically-similar model, but owing to the fixed angle of repose of the bed material used in the model the exaggerated model gives too large an extent of scour: on the other hand, a geometrically-similar model would give quite incorrect intensity of discharge unless entry conditions are imposed to reproduce curvature correctly. Further, the exaggerated extent of scour increases the "pull" of the scour-hole and the intensity of discharge, which otherwise would be correct in a distorted model, which is more natural for curvature.

In experiments where flow pattern is the prime consideration a geometrically-similar model with a rigid scour-hole reproduces the flow pattern: but a geometrically-similar discharge could only maintain the correct equivalent depth of scour if the sand used in the model were very much finer or lighter than that in the prototype; and with the same diameter of sand in the model, a discharge which is much higher than a geometrically similar discharge must be run to maintain the same depth of scour. This, however, would bring in additional factors, such as increased impact losses at piers and distortion of flow pattern, which can only be neutralised to a limited extent. Entry and exit conditions, must, therefore, be enforced and existing embayments may have to be maintained by using scour-resisting materials.

Other important *limitations* of models used for estimating maximum scour are :

(a) There are *different* time scales for scour and silting; while it has been observed that over two-thirds of the maximum scour takes place in 5% of the time, silting in a model is relatively slower than in the prototype and has to be exaggerated precisely as the *extent* of scour has to be reduced.

(b) The distribution of sand charge in nature cannot be reproduced in the model: while, frequently, more than 90% of the charge in nature is moved in suspension, the very reverse is generally the case in models.

(c) In a canal or a narrow channel, the bed-sand movement is maximum in midstream and becomes less and less near the banks: the bed has to be '*bowed*' in cross-section by depressing the bed in midstream to reproduce this condition in a model.

One further important point which must be borne in mind is that even though a model will, when correctly constructed and interpreted, give the maximum depth of scour, it will only give the *scour factor* (Col.9 of Statements I and II, viz. $\frac{d_s}{D_{Lacey}}$) correctly when the grade and charge used in the model are such as would simulate the required width: depth ratio in the model channel. Lacey's formulae are only applicable to channels carrying "minimum charge consistent with a fully active bed": with higher charges, as the Poona Station has stressed, channels tend to become shallower and wider. Unless, therefore, the charge and grade used in the model reproduce the width: depth ratio, it is more correct to obtain the scour factor comparing the equivalent maximum scour depth obtained from the model with the Lacey depth for the grade of material on the prototype. This is clear from the alternative figures (a) and (b) of scour factors worked out in Statement I(a) for the Sarda Barrage. For this reason, in Statement III, the scour factors for the

Kotri Barrage have been based on the Lacey depths on the prototype, calculated from the intensity of discharge q as observed on the model and grade of material on the prototype, and compared to equivalent maximum depths of scour observed in the model.

The Khadakvasla Station has evolved model techniques to counteract some of the important limitations; what cannot be achieved by suitable technique is allowed for in the interpretation of the results, from long experience gained by a close study of the conformity of the model results with prototype behaviour.

4.2. Model of the Hardinge Bridge.

In the model of the Hardinge Bridge, the extent of scour was obtained using a vertical exaggeration of 1.5 (compared with natural exaggeration of the river model = 7)—based on experience of models—and the depth observed on the prototype was very closely obtained in the Model.

Fig 8.
C-2 (iii)-3.

The contours of the Ganga near the Hardinge Bridge are shown in Figure 8. In 1938, when the maximum discharge was 1,675,000 cusecs, (water level R.L.246.4), with marked curvature of flow resulting from the throw-off at the nose of the Right Guide Bank, the deep scour-hole downstream of piers 2 and 3 was 172 ft. deep, 400 ft. downstream, on 31-7-38 (when the slip occurred), but increased to 186 ft., 500ft. downstream, on 5-9-38, showing that with a high level pitching scour was much further upstream, and more dangerous than a deeper scour-hole further away, because the slope, towards the scour-hole was steeper.

The model used was a part-model of the Ganga River at Hardinge Bridge to a scale of 1/400: 1/266. The bed upstream and downstream was laid in sand ($m = 0.22$ mm.) according to river contours of 5-9-38. Three vanes were fixed to create correct entry conditions and divert the flow towards the Damukdia Bank whence it was deflected towards the head of the Right Guide Bank, reproducing the same severe conditions as in the river. Experiments were carried out with the equivalent discharge of 1,675,000 cusecs, water level R.L. 246.4, and with stone pitching round piers laid high at the bridge; when it was found that the scour-hole downstream was reproduced and stone was depleted from downstream of the pier No. 3 *as actually happened* below the Hardinge Bridge. When the pitching round the pier was laid at a low level (at R.L.150), the depth of scour was reduced by 22%. These experiments conclusively proved that—

- (i) high level pitching at piers 2 and 3 was responsible for the deep scour-hole downstream of span No. 2;
 - (ii) a deep scour-hole in the vicinity of a bridge is dangerous, as it causes slips of stone protection;
- and (iii) the safe level at which to maintain stone protection round pier No. 3 of the Hardinge Bridge is R. L. 150, as against R. L. 167 at its nose, at which level the stone was maintained by replenishment.

4.3. Kotri Barrage Model :

Fig 9
C-2 (iii)-3

An intensive series of experiments has been done on the Kotri Barrage model, *inter alia* to determine the scour (a) below the barrage (with and without friction blocks), (b) along and at the nose of the Divide walls, and (c) at different sections of the Guide Banks and the Central Dividing Island, whose alignments are shown in line sketch (Fig. 9). The Kotri Barrage is designed to take advantage of the available favourable curvature on both banks, by armouring and perpetuating the existing natural island, so as to be able to exclude sand efficiently from canals on either bank.

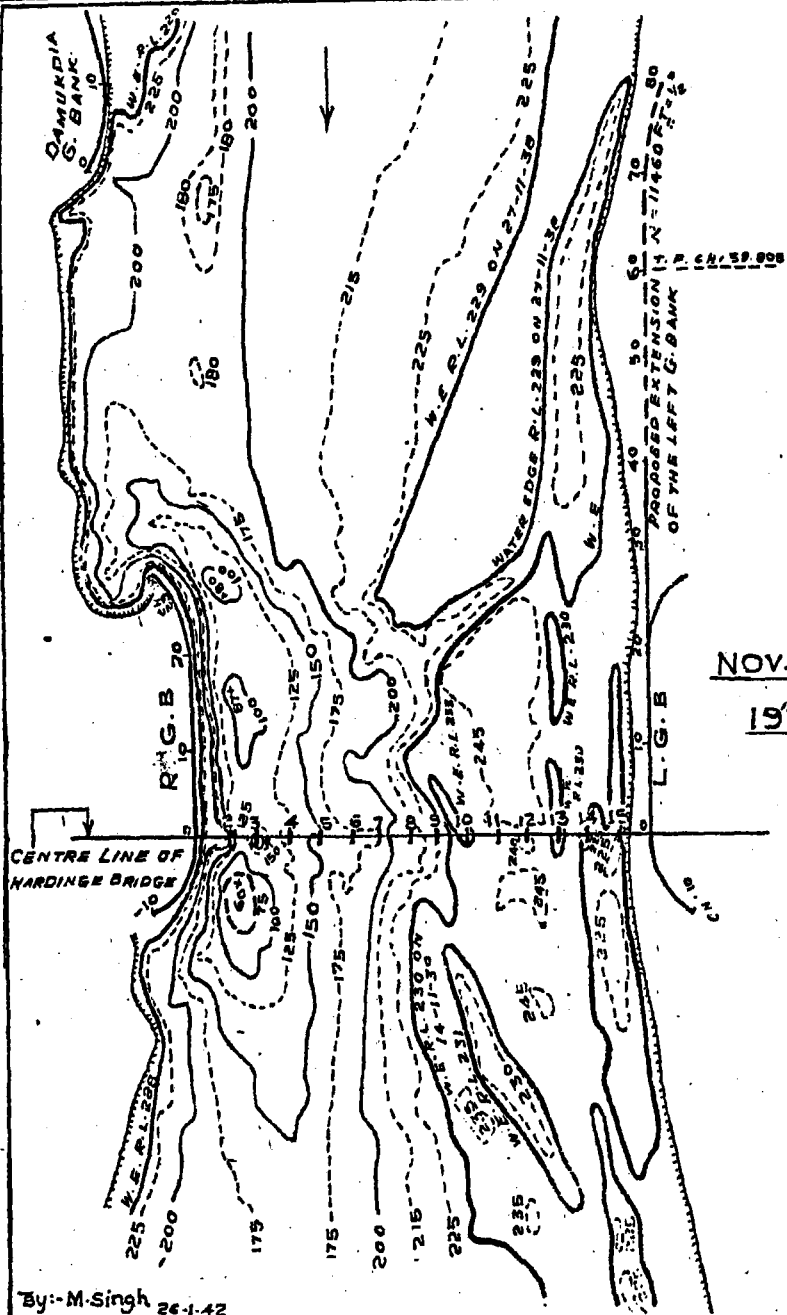
Models of different sizes and exaggerations have been used for the different requirements.

4.3.1. For scour downstream of the Barrage and the design of the flexible apron and friction blocks, experiments were first carried out in the

FIG. 8/C-2 (iii)-3.

SHOWING
CONTOURED PLAN OF THE
BED OF THE GANGA
UPSTREAM & DOWNSTREAM
OF HARDINGE BRIDGE.

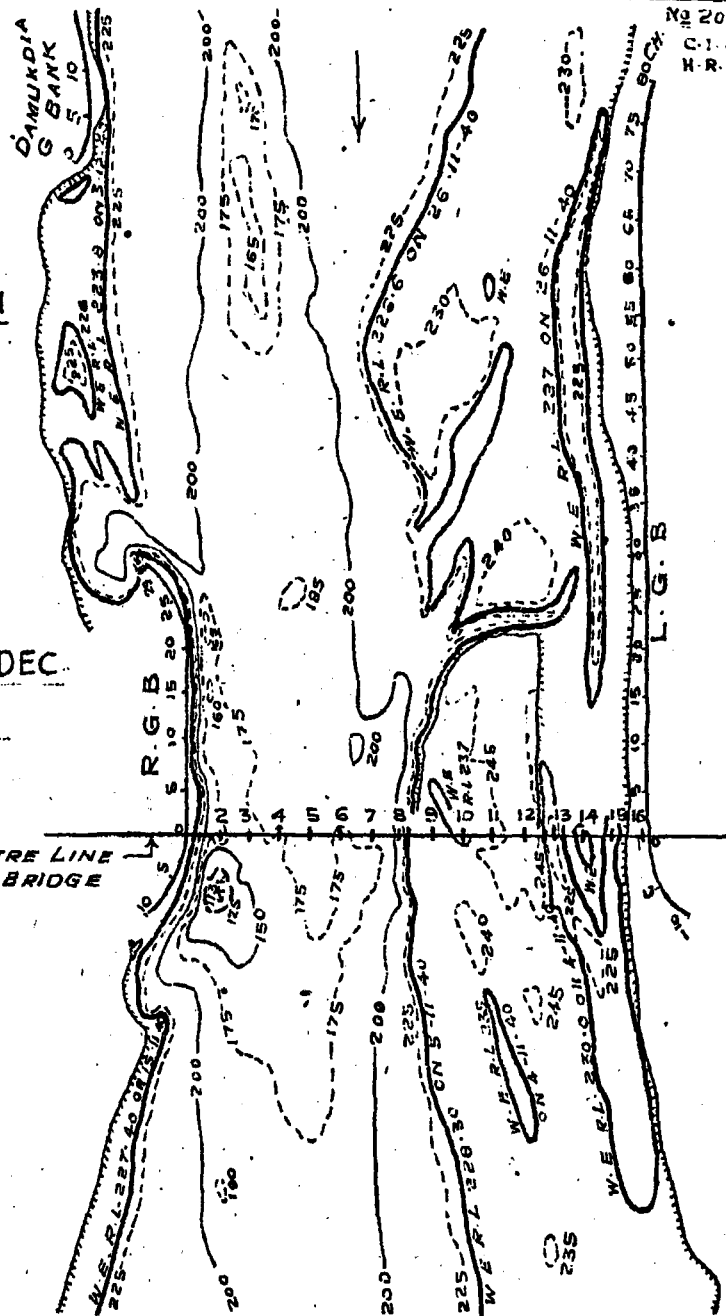
SCALE 1" = 2,000'



NOV. & DEC.

1940

CENTRE LINE OF H. BRIDGE

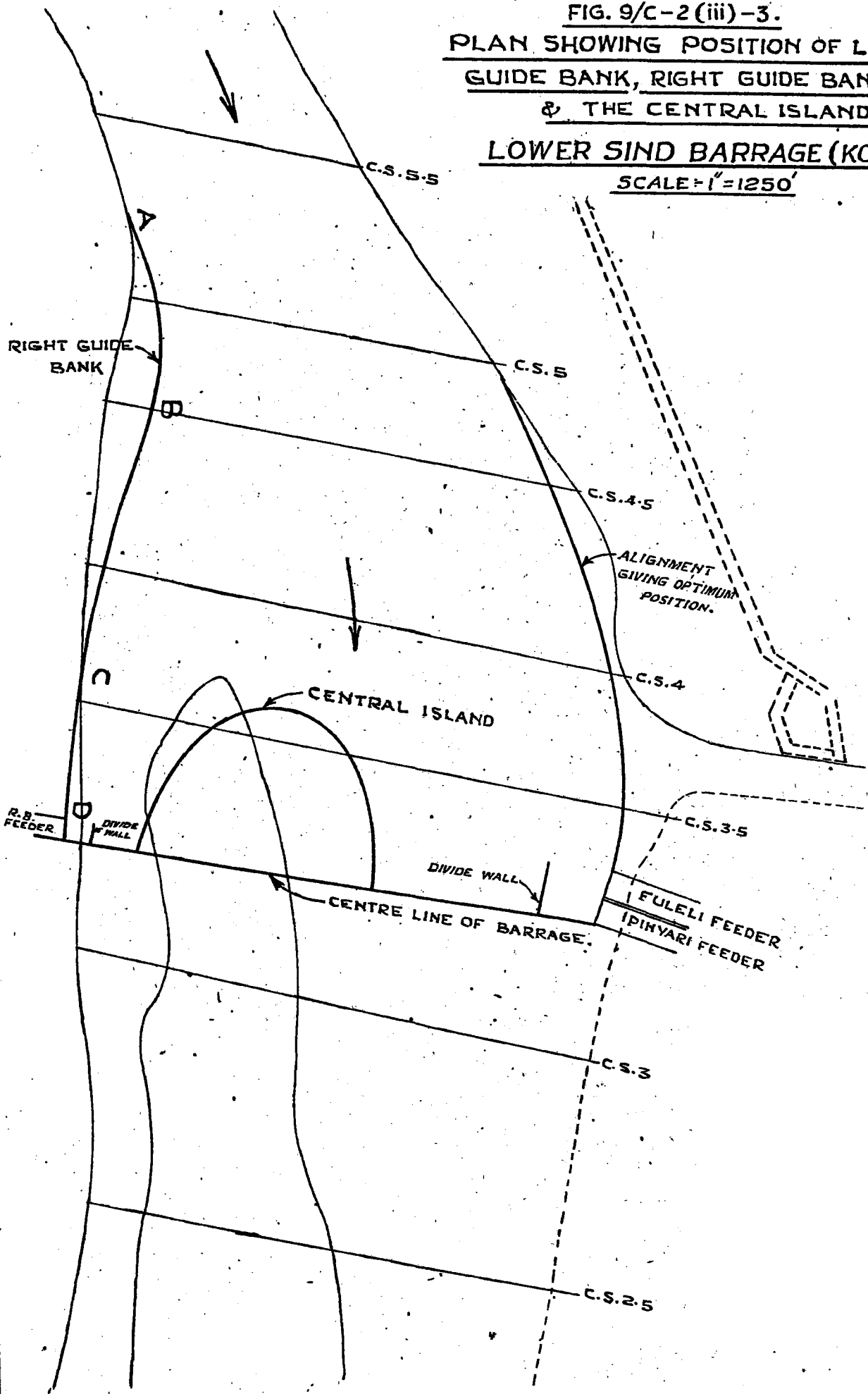


By: M. Singh 26-1-42

No 2088
C.I.P.
H.R.S.
SH

FIG. 9/C-2 (iii) - 3.
PLAN SHOWING POSITION OF LEFT
GUIDE BANK, RIGHT GUIDE BANK
& THE CENTRAL ISLAND
LOWER SIND BARRAGE (KOTRI)

SCALE = 1" = 1250'



V. E. model (1/250 : 1/50)—see also para 4.3.4. — laid in clean sand 0.30 mms. in diameter, the weir portion being treated as a geometrically-similar model to a scale of 1/50. Discharges equivalent to 2,00,000 to 5,50,000 cusecs, with necessary sand injection, were run in the model till stable depths were obtained downstream, the tail water levels maintained being the *minimum* ever observed less retrogression 5 to 1.5 ft, respectively expected downstream.

Experiments showed that *without* the baffle and deflector blocks there would be undermining below the masonry floor with maximum depth of scour down to R. L. + 9. 0, i. e. 55.5 ft. below the minimum downstream water level after retrogression ($d_s/D_{Lacey} = 1.23$): while *with* the two staggered rows of trapezoidal *baffle* blocks just downstream of the glacis and two staggered rows of rectangular *deflector* blocks at the end of the rigid pavement there was no undermining downstream, sand piled up against the face of the curtain wall at a slope of 1 in 3, and the maximum depth of scour went down to R. L. 23.5 only, i. e. 41 ft. below the minimum (after retrogression) downstream water level, ($d_s/D_{Lacey} = 0.91$)—see statement III. No apron would be strictly necessary if deflector blocks are provided; but, if for additional security a stone apron is considered desirable, it should be laid to a slope of 1 in 3 so that there is minimum disturbance and an increase of water depth. *Disturbed blocks* were found in the case of the Sarda Barrage to increase the scour inordinately by causing greater turbulence.

Fig. 10
C-2 (iii)-3

Stt. III
C-2 (iii)-3

4.3.2. Scour round Divide Wall noses :

Divide Walls are provided between the Pockets (at either end of the Barrage) and the Barrage proper to enable "still-pond" regulation (i.e. with pocket gates shut and Barrage gates partly or wholly open), which improves sand-exclusion from canals already off-taking from favourable curvature of flow. The worst river stage for depth and extent of scour at the Divide Wall nose is when the Barrage gates are just fully open and the Pocket gates shut—called the "still-pond limit"—which occurred (with pond level R.L.70.0) when the river discharge was 4,52,000 cusecs till the scour-holes stabilized at R.L.15 (depth = 55ft.) and R.L. 25 (depth = 45ft.), respectively for the left and right Divide Walls. As models do not reproduce the vortex action *quantitatively*—compare in Statement II the depth of scour from the model and prototype at the nose of the outer bank for the Sukkur Barrage—it is necessary to provide for a larger depth of scour based on prototype experience.

Statement II
C-2 (iii)-3

Due allowance has to be made to counteract the exaggeration of extent of scour occurring due to the distortion of the model scales (1/250:1/50). From past experience and comparison of scours in prototype and model, the extent of scour in this case had to be adjusted by measuring it at longitudinal and latitudinal scales of 1/125. Fig. 11 shows the prototype contours of the scour-hole thus derived.

Fig. 11
C-2 (iii)-3

Contrary to the usual practice of laying the flexible apron flat around the Divide Wall nose, it has been suggested that the apron should be laid to the natural cup-shaped profile to conform to the depth and extent of scour, *vide* Fig. 11. This would cause least loss of stone, least disturbance and consequentially least scour, and has, in addition, the merit of further aiding sand-exclusion by imposing a pull and a favourable twist on rolling material, as the model has shown.

Fig. 11
C-2 (iii)-3

4.3.3 (a) Scour along left Guide Bank.

Due to concentration of flow towards the centre, as a result of the attraction of the Central Island and the tendency of the larger discharges to flow straighter, scour less than that expected in the design was observed in the model.

Experience of the Sukkur Barrage, where berms have formed in spite of pronounced curvature on the left bank, supports these model results.

Statement
III
C-2 (iii)-3

A scour factor of $1.25 D_{Lacey}$ (against $1.5 D_{Lacey}$ provided in the design) is all that appears necessary—see item 2(c) of Statement III

(b) *Scour along right Guide Bank.*

Fig. 9
C-2 (iii)-3

Due to the attraction near the nose, concentration of flow occurs in the head reach AB—see fig.9—and the scour factor of $1.5 D$ was justified by the model—see item 2(a) of Statement III.

Statement
III
C-2 (iii)-3

In the middle reach BC, the flow was feeble due to change over of curvature; the model gave scour nearly equivalent to D_{Lacey} —see item 2(b) of statement—as already anticipated in the design.

(c) *Scour along Central Island :*

Experiments carried out in the river model (1/250:1/50) showed a *deep* scour-hole around the Central Island and *intense* turbulence which would throw bed sand into suspension to be carried into the Right Bank Canal because of its proximity to the Central Island.

Model experiments were, therefore, undertaken in a flume to evolve the optimum shape of the Island which would give minimum depth and extent of scour, particularly in the right arm, and which would have a larger scour depth and extent on the left side of the nose so as to improve sand-exclusion by pulling the bed sand away from the right arm.

It was observed that running a rising and falling flood cycle gave *less depth and extent* of scour than running the super-flood discharge; and that 80 to 90% of the scour occurred during the first few minutes; so the provision of an apron for the super-flood discharge—though that magnitude of discharge will occur for a very short duration in an exceptional season—is necessary.

4.3.4 All the above results for scour are being verified in a large river model with very low $V. E. = 2 (1/100 : 1/50)$ but with entry conditions imposed in reference to the 1/250 : 1/50 model.

5.0. The data so far available show, in respect of bridges and weirs and their ancillary works, that—

(a) the maximum scour downstream of bridges is of the order of $4 D_{Lacey}$; while below weirs, having greater waterway than Lacey's minimum stable waterway, the maximum scour will vary from $1.25 D_{Lacey}$ to $2 D_{Lacey}$ depending on concentration and intensity of turbulence (in the case of the Sarda, a scour factor of 3.14 has been obtained for special reasons);

(b) due to vertical vortices, the scour at the noses of Divide Walls facing upstream with steeply sloping heads, will be of the same order as for spurs facing upstream, say $3.8 D_{Lacey}$ —even though models may yield a lesser depth of scour, because models fail to reproduce the full action of vertical vortices;

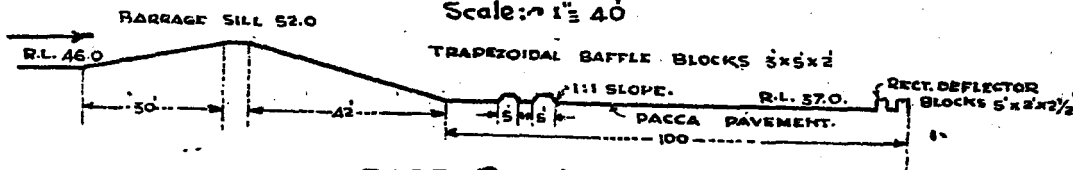
and (c) scour at large radius Guide banks is of the order of $2.75 D_{Lacey}$ at transitions $1.5 D_{Lacey}$ and at straights $1.25 D_{Lacey}$; but so much depends on the intensity of discharge and sharpness of curvature that it is only with the aid of model experiments, correctly made and interpreted, that the depths can be estimated with reasonable security or without waste.

This note, above all, shows the necessity for more data from prototypes and models, in specific cases, as well as fundamental research on scour below rigid structures.

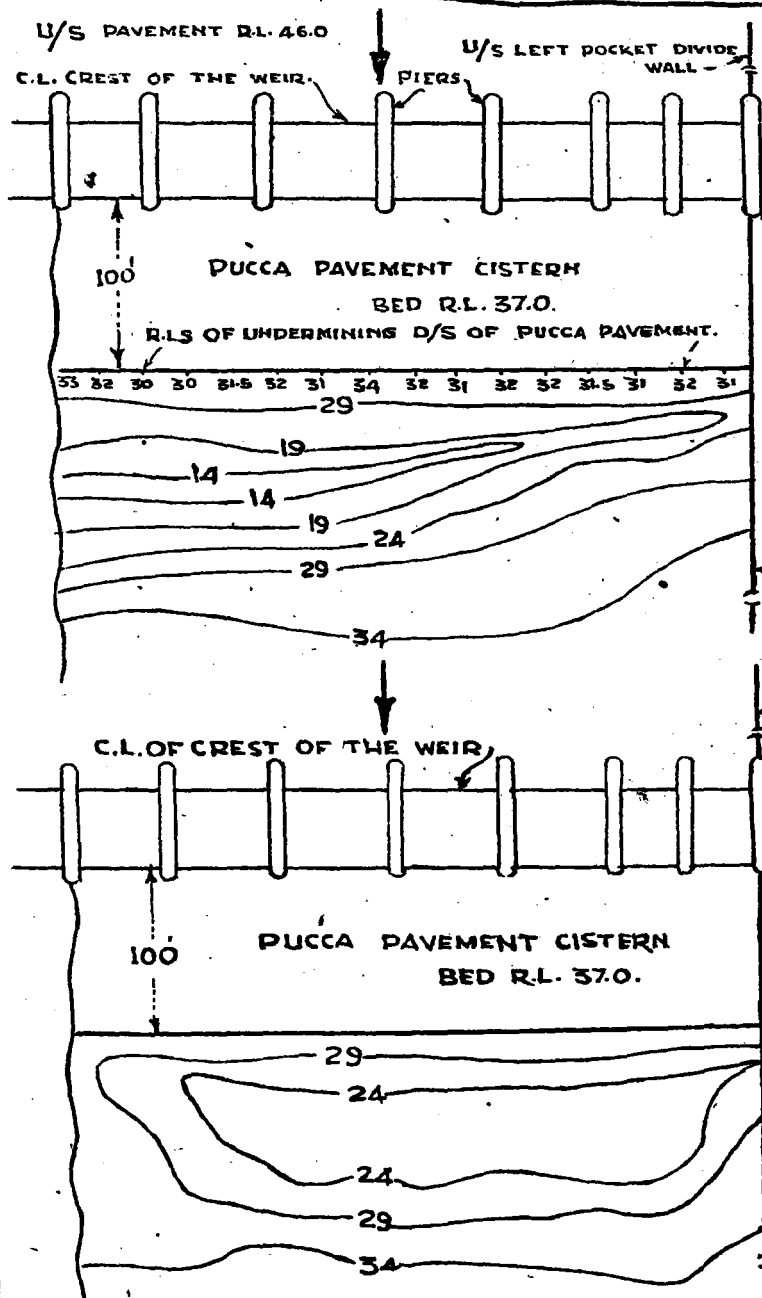
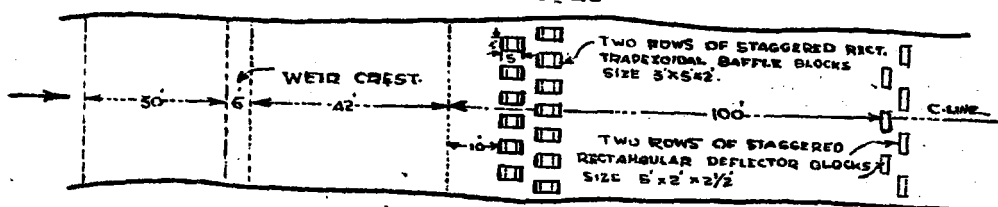
LOWER SIND BARRAGE (KOTRI)

**MODEL EXPTS. FOR PREVENTING SCOUR D/S
OF BARRAGE.**

SECTION.
Scale: $1'' = 40'$



PART PLAN.
Scale: $1'' = 40'$

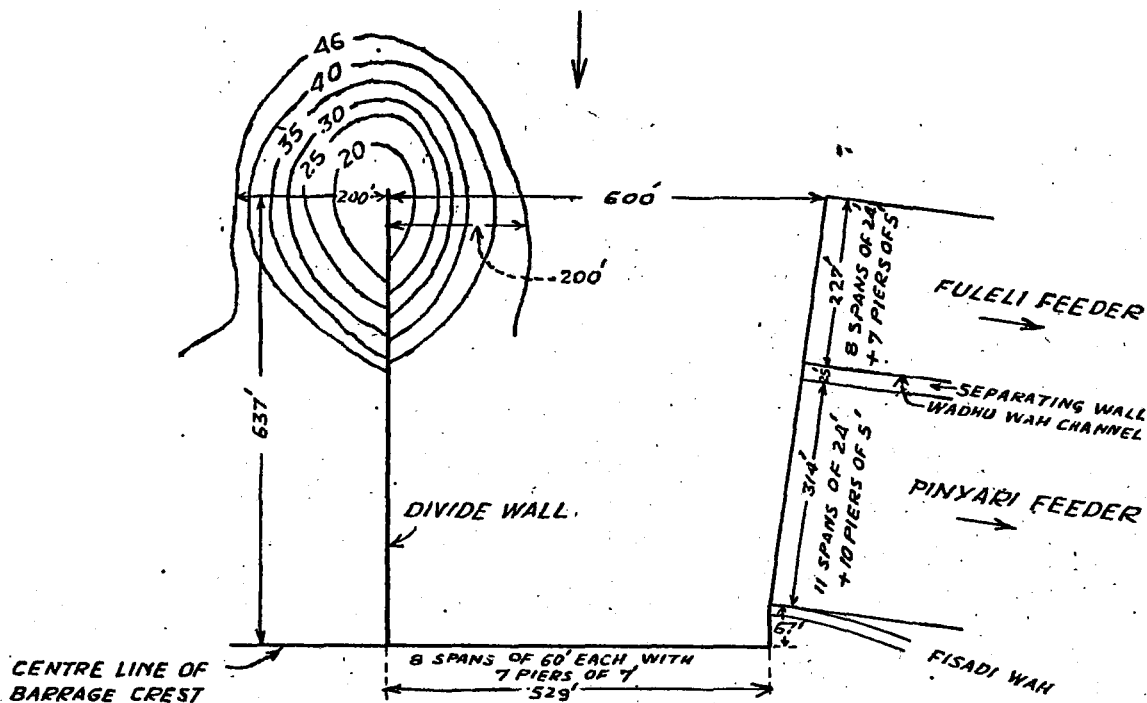


(a)
WITHOUT FRICTION
BLOCKS.
Scale: $1'' = 100'$

(b)
WITH FRICTION
BLOCKS.
AS PROPOSED IN THE
DESIGN.
Scale: $1'' = 100'$

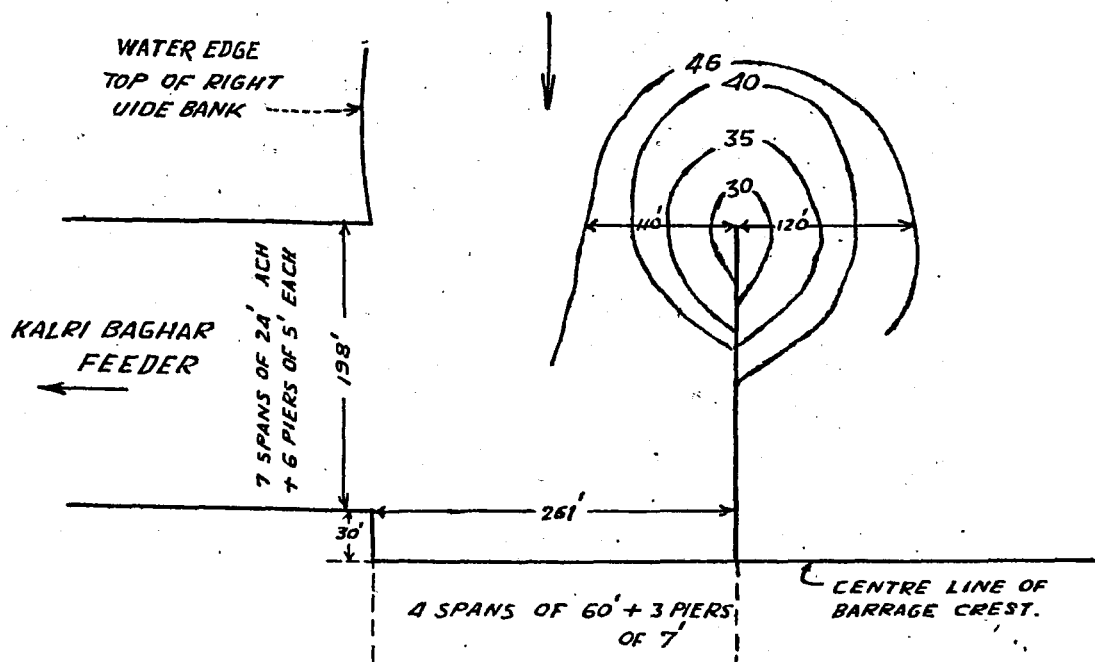
FIG. II/C-2(iii)-3
PLAN SHOWING CONTOURS
ROUND THE LEFT DIVIDE WALL NOSE
UNDER NATURAL CONDITIONS.

SCALE. 1" = 250'



PLAN SHOWING CONTOURS
ROUND THE RIGHT DIVIDE WALL NOSE.

SCALE. 1" = 125'



Statement 1/C-2 (iii)-3

Table I - showing maximum depth of scour downstream of Weirs, Bridges and Regulators.

S. No.	Site	Year of observation	Q cusecs (max. discharge) or q cs/ft. (max. intensity of discharge)	d_s (ft) = maximum depth of scour (ft.) observed below corresponding flood levels	m = mean grade of bed material in mms.	$f = 1.76 \sqrt{m}$ (Approx.) Lacey's silt factor (Approx.)	Lacey's Regime depth = D Lacey = $.473 (Q/f)^{1/3}$ or $0.9 (q^2/f)^{1/3}$	$\frac{d_s}{DL_{Lacey}}$ = scour factor	$d_s \times f^{1/3}$	Remarks.
1	2	3	4	5	6	7	8	9	10	11
1. - Prototype data of scour below Weirs, etc.,										
1.	Sarda Barrage U. P.	1938	236 cs/ft.	40	129.0	20	12.74	3.14	108.5	Boulder river. Excess scour due to disturbed talus.
2.	Mahanadi below Central sluices (Orissa) before modification.	1946	105.3 "	26.3	0.70	1.47	19.63	1.34	29.9	
3.	Hardinge Bridge on Ganga.	1938	1,670,000 cusecs	216	0.37	1.07	54.93	3.84	221	Annual Report (Tech) 1944. App. I -do-
4.	Rohri Canal downstream of head Regulator.	...	10,900 "	47	0.07	.47	13.49	3.48	36.55	
5.	Rohri Canal downstream of Multan Bridge.	10,900 "	56	0.056	.42	14.01	4.0	41.94	App. I -do-
6.	Rice Canal d/s of Rly. Bridge at R. D. 75,000	...	10,050 "	45	0.32	1.0	10.21	4.41	45.0	Annual Report (Tech) 1942-43, p. 21. P. 133, C B, I. Publication No. 12 "Design of Weirs on permeable Foundations"
7.	Khanki Weir.	145 cs/ft.	31.6	...	1.0	85.0	1.25	31.6	
8.	Merala Weir.	1928	160 "	34.8	1.0	26.6	.31	34.8	
1 (a) Model Data of scour below weirs etc.,										
1.	(a) Sarda Barrage, U.P. (a) (geometrically-similar 1/60) (Boulder River)	1940	0.99 "	0.6	0.6	1.36	0.81	0.75	0.67	(a) Scour factor based on model DL
	(b) " (b)	1940	0.99 "	0.6 ($\equiv 36$ feet)	129 (Proto)	20 (Proto)	12.74 (Proto)	2.83	(b) Scour factor corrected for proto sand
2.	Mahanadi Central sluices remodelling (1/25 G. S. Model)	(a) 1946	0.29 "	0.58 ($\equiv 14.3$)	0.3	0.96	0.29	1.97	0.57	(a) Scour factor based on model DL
	(b)	1946	0.29 "		0.7 (Proto)	1.47 (Proto)	8.65 (Proto)	1.68	...	(b) Scour factor corrected for proto sand.
3.	Sone Anicut (Bihar) Central Sluices. (G. S. 1/40) without protection.	1940	0.42 "	0.96	0.4	1.11	0.49	1.98	1.0	
	(a) -do- with blocks, deflectors etc.	1940	0.42 "	0.50	0.4	1.11	0.49	1.04	1.0	
4.	Sone Anicut under-sluices (G. S. 1/40) (without protection.)	1940	0.99 "	1.0	0.4	1.11	0.85	1.17	1.04	Annual Report (Tech.) 1944.
5.	(a) Merala Weir (without protection)	(a)	1.62 "	1.6	0.25	0.88	1.29	1.24	...	(a) Scour factor based on model DL
	(b) -do-	(b) ...	1.62 "	1.6	0.32 (Proto)	1.0	1.1	1.45	...	(b) Scour factor corrected for prototype grade of bed material.
6.	Kotri Barrage over (a) Indus (1/250: 1/50) without protection.	1947	0.86 "	1.11 ($\equiv 55.5$)	0.18 (Proto)	0.75	48.5 (Proto)	1.15	...	
	(b) -do- with protection.	1947	0.86 "	0.83 ($\equiv 41.5$)	0.18 (Proto)	0.75	48.5 (Proto)	0.85	...	

STATEMENT II/C-2 (iii)-3.

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Statement II—showing maximum depth of scour at noses of Guide Banks, Divide Walls, Groynes, sharp bends etc.

S. No.	Site	Year of observation	Q cusecs (maximum discharge) or q cs/ft. (max. intensity of discharge.)	d_s =maximum depth of scour observed below corresponding water level.	m=mean grade of bed material in mms.	($= 1.76\sqrt{m}$) (Approx) Lacey's silt factor (Approx.)	Lacey's Regime depth = $D_{Lacey} = 473(Q/f)^{1/3}$ or $0.9(q^2/f)^{1/3}$	$\frac{d_s}{D_{Lacey}}$ = scour factor.	$d_s \times f^{1/3}$	Remarks.
1	2	3	4	5	6	7	8	9	10	11
<i>II Scour at Noses of Guide Banks, Divide Walls etc.—Prototype data.</i>										
1 (a)	The Ganga, at Sara Guide Bank, 3 miles upstream of Hardinge Bridge.	1933-35	1,700,000 cusecs.	206	0.39	1.10	54.69	3.77	212.6	See Bombay Tech. Paper 55 and Annual Report (Tech.) for 1937, 1938 & 1939.
(b)	The Ganga near the nose of the Right Guide Bank of Hardinge Bridge.	1933	1,700,000 cusecs.	150	0.37	1.07	55.2	2.72	153.4	
2 (a)	Sukkur Barrage, at nose of outer bank. Approach channel side.	1940	50,000	66	0.27	0.92	16.9	3.9	64.18	A. R. (T) for 1940-41, p. 12 Assuming 3rd concentration along Outer-Bank.
		1940-41	45,000	70	0.17	0.725	28.4	2.48	63.0	
(b)	„ on river side	„	(q=150) 4,00,000	40	0.17	0.725	23.7	1.7	36.0	
3 (a)	Yamuna near Delhi Gate Pumping Station at spur 5	1938	48,000	30	0.32	1.0	17.27	1.77	30.0	Annual Report (Tech.) for 1937-38, page 65.
(b)	„ „	1939	31,030	24	0.32	1.0	14.86	1.62	24.0	
4	Sutlej upstream of Railway bridge at Ferozpur—									
(a)	At nose of spur 4,	1944	2,09,883	65	0.32 (Assumed)	1.0	27.9	2.33	65.0	A. R. T. (1944). Page 83.
(b)	At nose of old Right Rly. Guide Bank.	1944	2,09,883	68	0.32 „	1.0	27.9	2.44	68.0	
5	The Ravi upstream of Balloki Weir—									
(a)	At nose of spur 5 on right bank,	1935	50,700	65	0.32	1.0	17.4	3.74	65	A. R. T. (1944), page 83; & appendix I of 1944 A. R. T.
(b)	At nose of spur 6 on right bank.	1932	40,000	56.4	0.32	1.0	16.1	3.5	56.4	
6	Sarda Barrage (Banbassa) spur.	1939	2,38,000*	27.3	129 mms.	30.0	10.55	2.59	74	* 45% of total discharge (5,22,000 cusecs) along spur (see A.R. T. of 1939-40).

II A - Scour at nose of Guide Banks, Divide Walls etc.—Model Data.

1	Divide Wall of Sone Anicut (1/40 G. S. model) with—									
(i)	Conical stone pitching ...	1940	0.99 cs/ft.	1.02	0.4 mms.	1.11	0.86	1.185	1.04	Annual Report (Tech) 1940-41, pp. 19-22.
(ii)	Sloping 1 in 10 nose ...	1940	0.99 cs/ft.	0.8	0.4	1.11	0.86	0.835	1.04	
(iii)	Arched openings and 1 in 10 nose	1940	0.99 cs/ft.	0.36	0.4	1.11	0.86	0.41	1.04	
2	Curved Guide Bank of Sone Anicut (1/40 G. S. Model)	1943	1.92 cs/ft.	1.16	0.4	1.11	1.35	0.86	1.21	A. R. T. (1944), page 22.
3 (i)	Sukkur Barrage Outer Bank Nose (sloping at 1 in 2) of loose stone—									
(a)	Approach channel side,	1941	(0.87 (≅30,000) cs. $q_m = 0.125$ cs/ft.)	0.37	0.34	1.02	0.222	1.67	.124	A. R. T. 1941-42 pp. 16-17.
(b)	River side (G. S. Model 1/65).	1941	7.5 (≅9,00,000) cs. $q_m = 0.94$ cs/ft.)	1.0	0.24	1.02	0.86	1.17	.935	
3 (ii)	Sukkur Barrage Outer Bank, elliptical nose 1 in 2½ of loose stone—	1941	0.87 (≅30,000) cs. $q_m = 0.125$ cs/ft.)	0.23	0.34	1.02	0.222	1.04	.124	— Do —
(a)	Approach channel side ...	1941	7.5 (≅9,00,000) cs. $q_m = 0.94$ cs/ft.)	0.85	0.34	1.02	0.86	0.99	.935	
(b)	River Side (G. S. Model 1/65)	7.45 cs. (45% of this along spur)	0.6 (≅30 ft.)	129 (Proto)	20 (Proto)	10.55 (Proto)	2.82	...	(45% of total model discharge assumed along spur) (see ART of 1939-40).
4	Sarda Barrage (Banbassa Spur) (V. E. 1/150 : 1/50)	...	8.0 cusecs	2.3	0.6	1.25	0.88	2.61	2.48	See A. R. (T) 1944 p. 83.
5	The Ganga near right Guide bank of the Hardinge Bridge (V. E. 1/500 : 1/66)	...	2.8 cusecs	1.0	0.24	0.86	0.70	1.43	0.95	
6	Yamuna near Delhi Gate Pumping Station at spur 5 (V. E. 1/200 : 1/27)	...								

Statement III/C-2 (iii)-3

Showing the scour downstream of Kotri Barrage *with* and *without* baffle and deflector blocks. Scour along the left and right guide banks, nose and left and right shanks of the central island, and round the nose of the left pocket divide wall.

Item	Q Prototype (cusecs)	q in model (cusecs/ ft.)	m in mms. <u>Model</u> Prototype	f = 1.76 \sqrt{m} <u>Model</u> Prototype	Depth of scour observed below corresponding W.L.		DLacey for prototype conditions = $0.9 (q^2/l)^{1/3}$	Proto- type d _s DLacey	Remarks.	
					Model	equivalent depth in prototype				
1. Scour downstream of Kotri barrage <i>with</i> and <i>without</i> baffle and deflector blocks.	5,50,000 cusecs.	0.86	$\frac{0.32}{0.18}$	$\frac{1.0}{0.75}$	1. 11'	55.5'	45.0'	1.23	Without blocks } Depth of scour observed for minimum downstream water level corresponding to discharge after retrogression. With blocks }	
	5,50,000 cusecs.	0.86	$\frac{0.32}{0.18}$	"	0.826'	41.3'	45.0'	0.91		
2. Scour along Guide Banks	7,50,000 cusecs. (Super-flood discharge)	1.20	$\frac{0.32}{0.18}$	"	1.234'	61.70'	45.0'	1.37	with 33% concentration at the nose of the island.	
(a) Reach A B of the right guide bank.		0.923	"	"	0.774'	38.70'	37.60'	1.03		
(b) Reach B C of the right guide bank		0.923	"	"	0.974'	48.70'	37.60'	1.25		
(c) Upstream reach of the left guide bank		0.80	"	"	1. 27'	63.70'	34.0'	1.84		
(d) Nose of the Central island.		0.90	"	"	1.274'	63.70'	37.0'	1.72		
(e) Left arm of the central island.		0.60	"	"	0.974'	48.70'	28.20'	1.71		
(f) Right arm of the central island.		0.60	"	"	0.774'	38.70'	28.20'	1.34		
(g) Reach CD of the right guide bank	3. Scour round the nose of the left pocket divide wall nose.	4,52,000 cusecs.	0.76	"	"	1. 06'	55'	32'	-1.72	Depth R. L. 70.0 for Q = 4,52,000 cusecs (Still pond conditions.)

SCOUR BELOW WEIRS.

APPENDIX I. C-2(iii)-3.

Showing cases of scour at and below rigid structures dealt with in various Annual Reports of the Central Waterways, Irrigation & Navigation Research Station, Khadakvasla (Poona)

<i>A.R. (T) for 1937-38</i>	<i>Page</i>
Note on experiments with a model of the Waste Weir Channel of Spin Karez Reservoir, Baluchistan, regarding prevention of erosion.	46
Dissipating energy below the masonry waste weir, Timba Tank at Harsol. Taluka Prantij, Dist. Ahmedabad (Bombay).	38
Protection of bridge piers against scour. (See also page 33 of A.R. (T) 1939-40).	39
Durani outfall Channel, Quetta (Baluchistan) (Also see page 22 of A.R.(T) for 1940-41)	33
<i>A.R. (T) for 1939-40</i>	
Prevention of scour below Girna Canal Aqueduct, Bombay.	24
Estimated scour at proposed Danagar bridge at Eurawan, Bengal.	23
<i>A.R.(T) for 1940-41</i>	
Scour at nose of the Outer Bank of the New Approach Channel at Sukkur in prototype and models.	12
Remodelling Son Anicut:	
Scour at noses at Divide Walls upstream and downstream of the Anicut & Under-sluices.	19
Scour downstream of Sarda Barrage at Banbassa, U. P.	24
<i>A.R. (T) for 1941-42</i>	
Protection of bridge piers against scour and minimum safe level at which to lay stone around piers.	12
The effect of high level pitching laid round bridge piers on the depth of downstream scour.	31
Scour round piers—influence of upstream bed level and material of bed on maximum scour at pier.	32
<i>A.R.(T) for 1942-43</i>	
Maximum scour profile between guide banks of the Ganga at the Hardinge Bridge.	4
Alternative designs of baffle platform downstream of the Jamshedpur Siphon Spillway for dissipating energy.	20
Experiments with a model of the Rice Canal in connection with protection of the Railway Bridge at R.D. 75,000.	21
<i>A.R.(T) for 1943</i>	
Protection of scour on the Jamshoro Bund of the Left Bank of the Indus near Hyderabad (Sind).	39
<i>A.R.(T) for 1944</i>	
Son at Dehri (Bihar):	
Protective measures below the New Undersluices.	20
Deep scour downstream of N.W. Railway Bridge over the N. W. Canal at mile 18/13-4 near Ruk (Sind). Experiments carried out to evolve a suitable design of baffle and side expansion to induce accretion.	23
Dissipation of energy below standing Wave Flume Meter Baffle Falls.	29
Maximum depth of scour at heads of guide banks and groynes, piers noses and downstream of bridges.	74

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Damage to the Old Fort Wall & buildings downstream of the Hope Bridge on the Tapti at Surat, due to high level pitching at the bridge.	74
Prevention of erosion of the Left Bank of the Rupnarain River above Kolaghat Railway bridge (B. N. Railway).	13
Bank scour of the Hooghly near Danbar Cotton Mills leading to a knuckle formation and how to improve condition.	7
<i>A.R. (T) for 1945</i> (Report in the press).	
Prevention of scour downstream of the proposed Lower Sind Barrage at Hajipur.	
Scour downstream of rigid structures in wide channels. (See also page 114 of 1946 A.R. (T).)	
<i>A.R. (T) for 1946</i>	
Emergent measures for preventing serious damage to the Central Sluices of the Mahanadi Anicut at Cuttack - Orissa.	27
Training of river Indus above the Sukkur Barrage to prevent slips of the right Bank below nose of the Approach Channel.	27
Minimum waterway required for the Sagi, Khara & Sukri Bridges, Jodhpur.	37
Design of Vent approaches and exists of road culverts with recommendations for preventing scour downstream.	37
Scour at the Rly. Bridge No.3 of the Luni River, Jodhpur Rly.	52
Preventing scour below Ganesh Iyer Volute Siphon Spillways by mutual impact of jets issuing from adjacent siphon outlets.	91
Optimum shape of the nose of Central Island at Kotri for the Lower Sind Barrage for optimum scour & sand exclusion.	110

C—HYDRAULIC WORKS

2. River Control.

(Vi) Tidal action including that in estuaries.

C — (Vi) 1. Measures adopted to reduce turbulence in the Hooghly River downstream of the knuckle at the Titaghar Jute Mill.

Synopsis — This note describes the experiments carried out with the 1/300 : 1/60 scale model and the 1/150 : 1/80 part-length model of the Hooghly to test the effects on the flow conditions at the knuckle at the Titaghar Jute Mills, of—

- (a) a series of Islands,
and (b) a continuous separating wall.

The experiments showed that the conditions are so set that no practicable solution at reasonable cost appears feasible.

PART I.

1. Experiments with islands in 1/300 : 1/60 Model;

Fig. 1.
C-2 (vi)-1

1. 1. Introduction:—Figure I shows the model reaches of the 1/300:1/60 model and 1/150 : 1/80 model of the Hooghly river. In the case of the knuckle at Titaghar Jute Mill, the “change-over” of the flow occurs immediately upstream of the knuckle leading to more turbulence downstream and in addition to this, there is a throttling of the width of the river channel immediately downstream of the knuckle. These peculiar conditions made the action of a ‘row of piles(1) found successful in the case of the Dunbar Cotton Mill knuckle (1) ineffective at Titaghar in reducing the turbulence and inducing silting in the bight.

The previous experiments,(2) with islands had been done with the object of getting maximum improvement of flow downstream of the knuckle with the least number of islands. It had been found that a single island ‘B’ 3/4 mile upstream of the knuckle, attracted the flow on to the right bank, and no island upstream of B could be effective in assisting the action of this island. It was, therefore, decided to try a series of islands, downstream of island B, for attracting flow to the right face of the islands and thereby relieving the concentration of flow at the knuckle.

1. 2. The Model:—The model was laid to the 1939-40 survey and freshet spring tide curves as observed in the river on 23rd September 1941 were reproduced and repeated ten times; in each experiment ‘V’ sand (m = 0.20 m. m.) was used as bed material. Lines of flow were observed (after completion of 10 tides) under the worst condition of ebb tide when the water level at Konnagar was 13.5ft. above K. O. D. S. (Kidderepore Old Dock Sill) and the discharge was \cong 1,87,000 cusecs. Besides, these observations could not obviously be made under tidal conditions.

1. 3. Experiments:—The main requirement of these experiments was to reproduce freshet spring tides as observed in the river on 23rd September 1941. For the correct reproduction of these tides three recorders carrying the model tidal curves were fixed at the upstream (i. e. Damurdah) and downstream (i. e. Cossipore) ends of the model and at Konnagar. The observations made in the experiment consisted of lines of flow and velocities at C. S. 6 under conditions mentioned above.

1. 3- (I) Experiment No. 1:—In this experiment, an island C was constructed in addition to island B tried in the previous year's experiment.

The island C was placed just on the edge of the shoal and slightly upstream of the knuckle. After running 10 tides it was found that this island only slightly attracted the flow on the shoal. It was also observed that when the water level at Konnagar was 11.5 above K. O. D. S. the flow upstream of this island, con-

Fig. 2.
C-2 (vi)-1

(1) Annual Report (Tech) for 1946, page 40.

(2) " " " " 1945.

centrated on the left bank and made the conditions of flow severe. This was presumed to be due to the long reach between the two islands and so it was decided to add two small islands between B and C

Fig. 3.
C-2 (vi)-1

1. 3. (ii) Experiment No. 2— Two small islands D and E were placed first as shown in figure 3. After running 10 tides it was observed that movement on the shoal increased as was indicated by increased ripples. It was, however, noted, as in experiment 1, that with reducing water levels during ebb tide, the flow between the islands again concentrated on the left bank resulting in severe conditions.

Various positions and alignments of islands D and E were tried to see if the concentration of flow described above could be reduced. The results, however, were not satisfactory.

1. 3. (iii) A submerged island was constructed in the straight reach upstream of Palta, its top being kept 5 ft below K. O. D. S. thus allowing 10 ft draft. It was observed that such an island had little or no action and further experimental data were, therefore, not collected.

1: 4- Discussion— The single island B was found better than any other combination of islands tested, because islands upstream of B had no effect in augmenting the action of B, in respect of attracting flow to the right bank shoal. Island B faced the strong ebb current and, being at a considerable distance from the left bank, did not permit concentration of flow on that bank. The series of islands constructed downstream of island B showed a slight increase of attraction of flow on the shoal but the conditions again deteriorated during the last quarter of ebb. When islands downstream of the island B were placed nearer to the right bank the flow there was never strong enough to bring them into sufficient action, while, when placed nearer to the left bank, they were found to do more harm than good.

In this case, therefore, no single island or a series of islands could be evolved which would have sufficient action against the unfavourable combination of existing conditions.

PART II

Experiments with a continuous wall on the shoal opposite the knuckle.

2. 1. Introduction:— A continuous wall in line with the island was constructed with a view to prevent flow occurring between two islands because such flow made the attack on the left bank severe during the last of ebb.

The top of the wall was kept = 25 ft. above K. O. D. S., i. e. 4' above high water level, so as to remain above high floods. The right side of the wall was given a slope equivalent to 1 in 3 in prototype while the left side facing the knuckle was given a slope equivalent to 1 in 5, with a view to induce more scour in the right compartment by the steeper slope. The flat slopes would also enable the wall being constructed out of dredged bed material in the vicinity.

The alignment of the wall was decided from initial experiments, the first care being to see that it would not hamper navigation.

2. 2. Experiments:—

Fig. 4.
C-2 (vi)-1

(1) Experiment No 1.— The continuous wall was constructed as shown in figure 4 with the bulbous nose pitched. The lines of flow were observed after completing 10 tides. It is clear from figure 4 that the flow was initially attracted towards the nose, followed the wall to some extent, but then got somewhat diffused over the whole width. Slack water area was observed on the right bank.

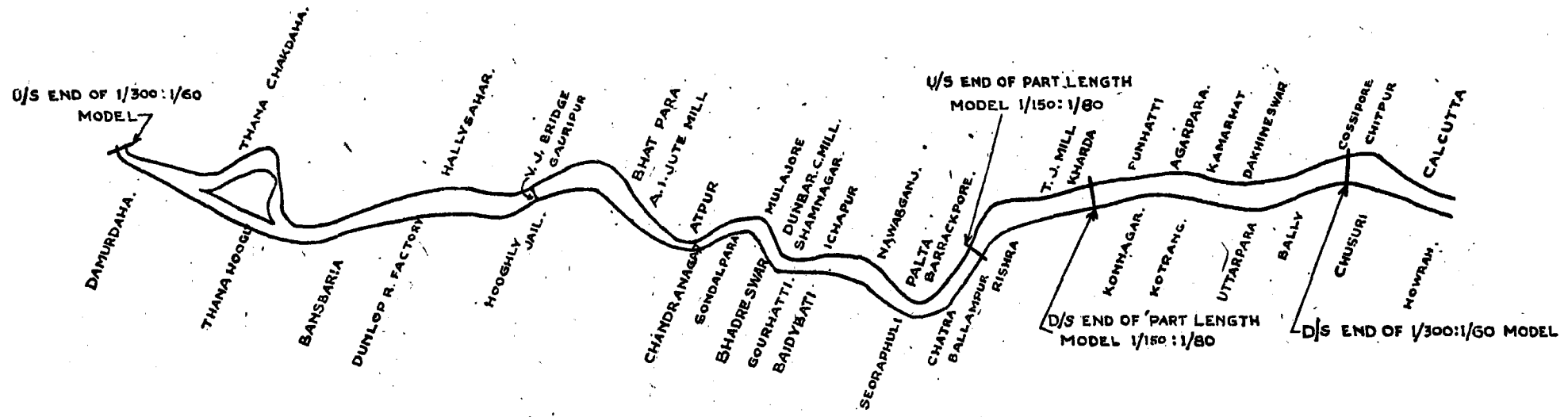
Fig. 4 (a)
C-2 (vi)-1

Velocities at C. S. 6 were observed under continuous ebb and are shown in fig. 4 (a). The discharge on the right side of the wall was observed to be 0.25 cusec out of the total of 1.34 cs., as compared with 0.20 cusec under existing conditions—see figure 10.

Fig. 10.
C-2 (vi) 1

(ii) Experiment No. 2:— In this experiment the alignment of the wall was kept the same as in experiment 1 but the right face of the wall was pitched throughout, its length with the view to attract more flow to this side.

FIG.1/C-2(VI)-1
PLAN SHOWING COMPLETE MODEL REACH.
1/300:1/60 HOOGHLY MODEL FROM DAMURDAHA TO COSSIPORE.
1/150:1/80 HOOGHLY MODEL.
SCALE :- 1"=18,000'



1/300: 1/60 SCALE HOOGHLY RIVER MODEL

(PART I) CONTINUOUS EBB FLOW

$Q = 1.34 \text{ CS.} \approx 187000 \text{ CS.}$

W. L. AT KONNAGAR $\approx 13.5 \text{ FT.}$

ABOVE H.O.D.S.

SCALE: $1" = 4' \approx 1200'$

FIG:- 2

EXPT NO. 1

LINES OF FLOW WITH ISLAND B & C

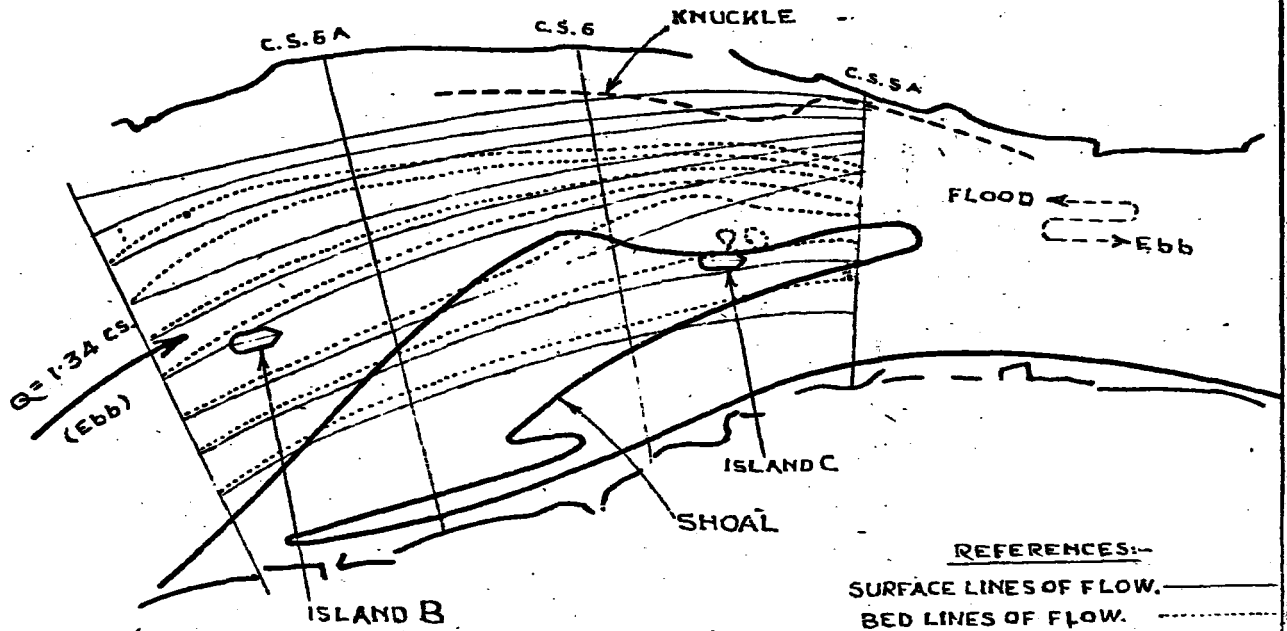


FIG:- 3

EXPT. NO.

LINES OF FLOW WITH 4 ISLANDS

ISLANDS B, C & TWO SMALL

ISLANDS BETWEEN B & C

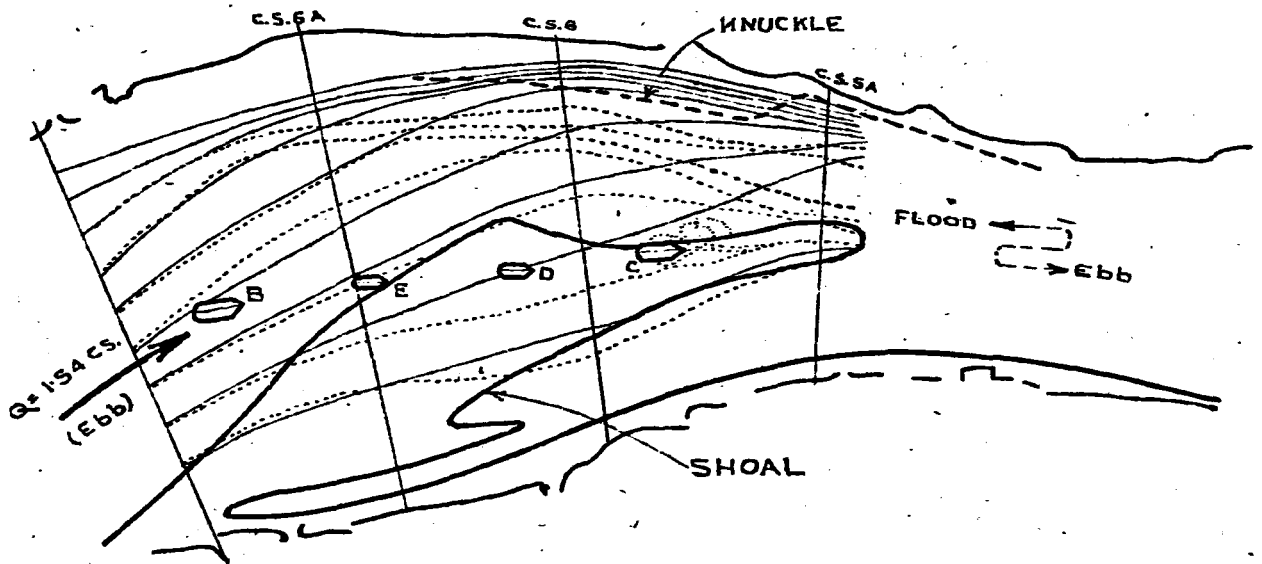


Figure 5 shows the lines of flow after 10 tides. It is clear that the flow followed the wall. The distribution of discharge between the left and right sides of the wall was, however, found to be the same as in experiment, 1. Fig. 5
 $\frac{C-2(vi)-1}{C-2(vi)-1}$

2.2. (iii) Experiment No. 3— In this experiment, the pitched wall was extended upstream and the nose was constructed nearer to the right bank as shown in figure 6. Lines of flow after 10 tides showed that the flow followed the wall and the slack water area was reduced compared to the area obtained in the previous experiment. Fig. 6.
 $\frac{C-2(vi)-1}{C-2(vi)-1}$

The discharge distribution at C. S. 6, however, showed that the discharge on the right side of the wall decreased, compared with the corresponding discharge in experiment No. 1 (see para 2. 2. (i)). Fig. 6 (a)
 $\frac{C-2(vi)-1}{C-2(vi)-1}$

2. 2. (iv) Experiment No- 4— In this experiment, the nose of the pitched wall was shifted away from the right bank, as shown in figure 7. The lines of flow showed uniform distribution of flow on the right side of the wall. As the turbulence at the nose extended up to the navigable channel, there would be unwarrantable interference to shipping and it was decided to shift the nose downstream. Discharge distribution was not, therefore, observed. Fig. 7.
 $\frac{C-2(vi)-1}{C-2(vi)-1}$

2. 2. (v) Experiment No- 5— In this experiment the wall was constructed parallel to the left bank as shown in fig. 8. Lines of flow showed that the flow did not follow the wall and there was a considerable slack water area. The distribution of discharge, however, showed that 0. 26 cusec out of 1. 34 cusecs passed on the right side of the wall. Fig. 8.
 $\frac{C-2(vi)-1}{C-2(vi)-1}$
Fig. 8. (a)
 $\frac{C-2(vi)-1}{C-2(vi)-1}$

2. 2. (vi) Experiment No 6 — The nose of the wall was shifted downstream and kept away from the right bank as shown in figure 9. The lines of flow showed a uniform distribution of flow on the right side of the wall. The distribution of discharge showed that 0. 31 cusec out of 1. 34 cusecs passed on the right side; this was the *maximum* discharge on right side of all the alignments tried. The navigable channel was well upstream of, and away from, the region of turbulence at the nose. Fig. 9.
 $\frac{C-2(vi)-1}{C-2(vi)-1}$
Fig. 9 (a)
 $\frac{C-2(vi)-1}{C-2(vi)-1}$

Figure 9 (b) shows the formation of a channel by the side of the wall and the position of bars with this optimum alignment of wall after a run of 10 tides. Fig 9 (b).
 $\frac{C-2(vi)-1}{C-2(vi)-1}$

2. 2. (vii) Experiment No, 7— In this experiment, the wall was removed and 10 tides were run to see the discharge that was passing over the shoal under similar existing conditions; the lines of flow observed under these conditions are shown in fig. 10. The discharge was found to be 0.20 cusec on the right side of the alignment of the wall. Velocity distribution is given in figure 10 (a). Fig 10. 1.
 $\frac{C-2(vi)-1}{C-2(vi)-1}$
Fig. 10 (a)
 $\frac{C-2(vi)-1}{C-2(vi)-1}$

2.2. (viii) Experiment No, 8— In order to find out the distribution of discharge with island B only, after 10 tides the velocity observation was made at C. S 6. The distribution of discharge was found to be 0. 24 cusec on the right side. (of the wall). Fig. 10 (b).
 $\frac{C-2(vi)-1}{C-2(vi)-1}$

2.3. Discussion: —The construction of the continuous wall not only stopped the flow between islands but it increased the discharge on the shoal, as will be clear from the following table.

Discharge at C. S. 6 after 10 identical tides.

Expt.	Discharge in Model		Per Centage of the total discharge on the right side.
	Right Side Cs.	Left Side Cs	
(1) Existing conditions (see vii above)	0.20	1.14	15
(2) With island B only (see viii above)	0.24	1.10	18
(3) With continuous wall as in Expt. 6. (see vi above).	0.31	1.03	24

The increase in discharge on the right side appears to be the result of the curvature induced by the alignment of the wall. No more curvature could be given to the wall with advantage as the flow did not follow sharper curvature, leaving an intermediate region of slack water near the wall, as is clear by comparing experiments 5 and 6.

Fig. 8 & 9.
C-2(vi)-1.

The more the wall is moved upstream and the wider the entrance on the right side, the more the discharge that is likely to be drawn towards the right bank; but both these factors are limited by the existing navigable channel.

It will be evident that the aim of all these experiments was to reduce the left bank discharge and create conditions favourable for silting downstream of the knuckle. The optimum alignment of the wall as in experiment 6 (para 2. 2. (vi)) was, therefore, retested in a large, part-length 1:150:1:180 model, to study the effects in greater detail.

Assuming the wall to be constructed out of the dredged material with slopes of 1 in 5 and 1 in 3, the percentage obstruction will be as shown in the table below:—

(Model data.)

C. S.	Total area of section, sft.	Area of obstruction due to wall (sft)	percentage obstruction.
7	6.6	0.50	7.6
6	6.0	0.40	6.7

Even though the percentage obstruction is 7.6 and 6.7 respectively, it will be seen that the wall has been constructed on an existing high shoal and the flow is almost wholly on the left side of the wall, so the effect on the general regime of the river would be negligible.

PART III.

3. Experiments In 1/150 ; 1/80 Model.

3. 1. The Model— The model was laid with Londha sand ($m = 0.30$ mms) to the 1939-40 survey. Discharge of 1.7 cusecs \equiv 187,000 cusecs was run continuously for 24 hours (\equiv 16 days) in all experiments. Water level at C. S. 4A was maintained corresponding to W. L. at Konnagar \equiv 13.5 ft above K. O. D. S. The bed was adjusted at the beginning of each experiment to enable fair comparison. Continuous ebb was run to expedite results, as almost all the effect was observed to occur during ebbs; this was done after careful comparison of the effects with those obtained with tidal flow.

3. 2. Preliminary experiments to determine the rate of sand injection—

Ebb tide, as described in para 3.1 above, was run for 24 hours, and the sand that was moving on bed was collected at the downstream end during this period. The sand thus collected was found to be 4 cft.

In the river, there is, generally, about 10 % of bed load and 90% suspended load. To start with, the same distribution was assumed in the model, the total load in model, during 24 hours would be 40 cft. A mixture of 36 cft. of red silt ($m = 0.076$ m. m.) and 4 cft. of Londha sand ($m = .30$ mm) was injected in 24 hours; i. e. $1\frac{2}{3}$ cft/hour.

3. 3. Experiments—

(1) Experiment No I— The wall was constructed to the alignment of experiment No. 6 (para 2. 2. (vi)) in the 1:1300:1:160 model and ebb flow of \equiv 187,000 cusecs discharge was run continuously for 24 hours. Injection was done at $1\frac{2}{3}$ cft/hour at C. S. 7A. After one hour of injection it was found

1/300 : 1/60 SCALE HOUGHY RIVER MODEL

Q = 1.34 ≅ 187,000 Cs.

CONTINUOUS EBB

W. L. AT KONNAGAR ≅ 13.5 FT ABOVE K.O.D.S.

SCALE 1" = 4' ≅ 1200'

FIG. 4
EXPT. NO. 1 (PART II)
LINES OF FLOW WITH UNSUBMERGED WALL

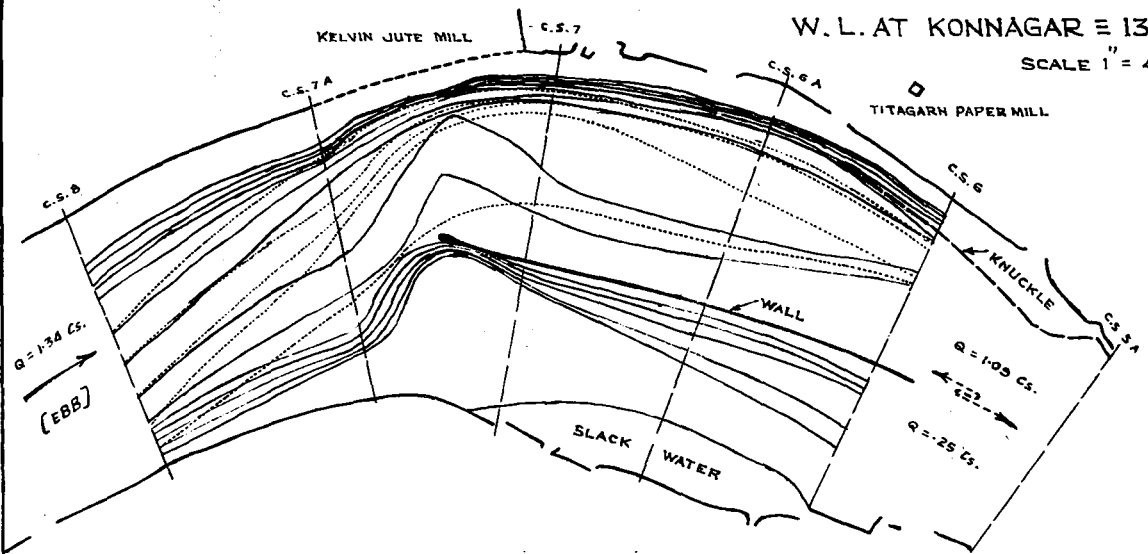


FIG. 6 EXPT. NO. 3 (PART II)
UNSUBMERGED WALL EXTENDED 4/3 & THE NOSE OF THE WALL BROUGHT TOWARDS THE RIGHT BANK
RIGHT SIDE OF WALL PITCHED

FIGS. 4-7/C-2(VI)H

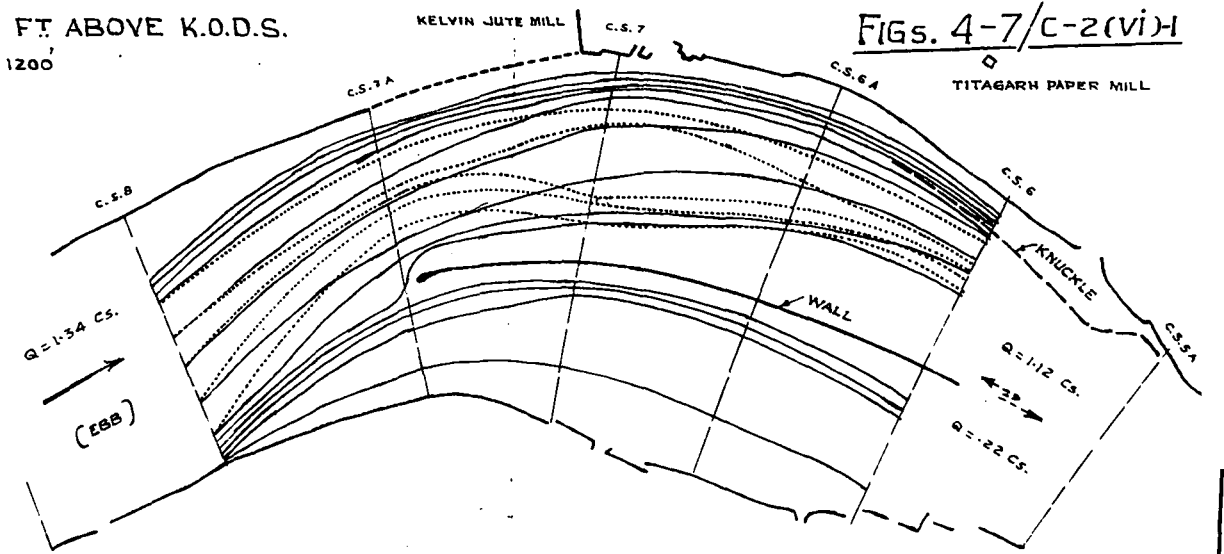


FIG. 4 (O)
EXPT. NO. 1 (PART I)
ISOVELS AT CROSS SECTION NO. 6

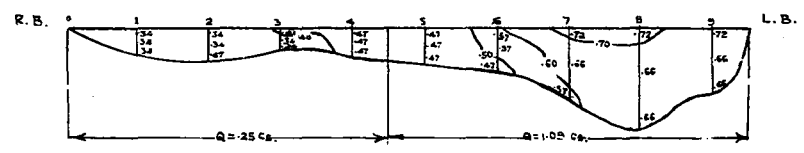
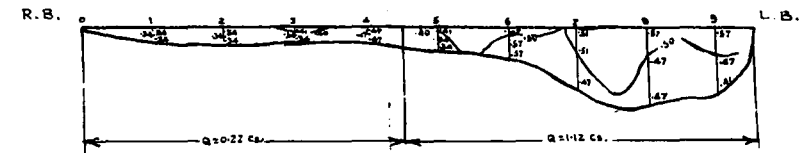


FIG. 6 (O)
EXPT. NO. 3 (PART II)
ISOVELS AT CROSS SECTION NO. 6



REF.
— SURFACE LINES OF FLOW
- - - - - BED LINES OF FLOW

FIG. 5
EXPT. NO. 2 (PART II)
LINES OF FLOW WITH UNSUBMERGED WALL
RIGHT SIDE PITCHED THROUGHOUT

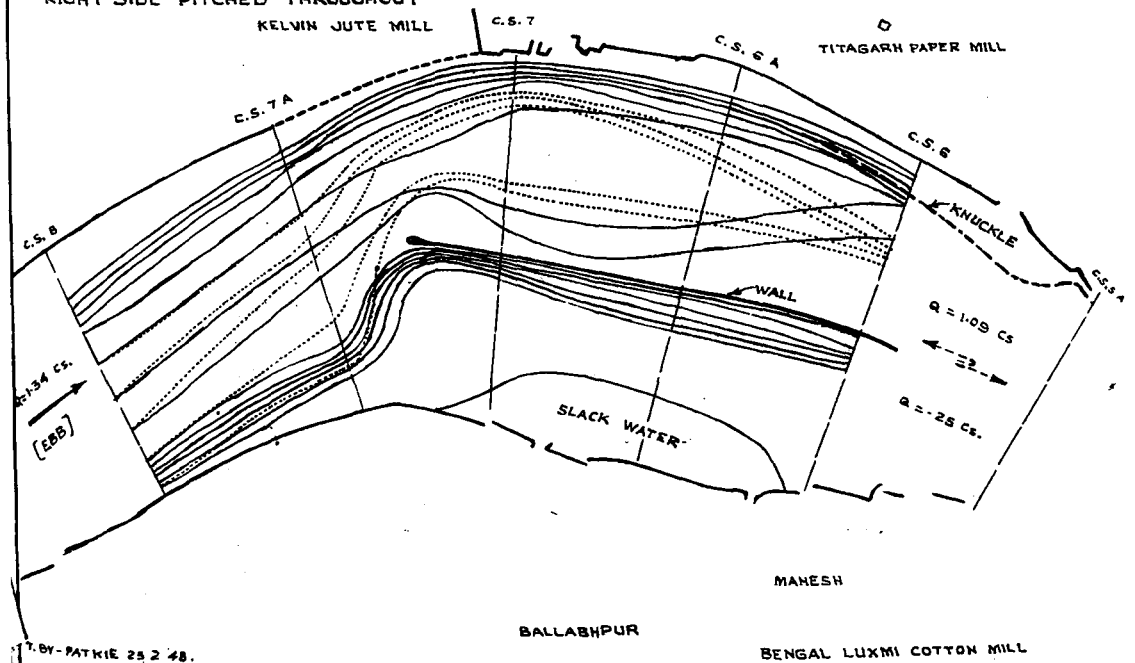


FIG. 7 EXPT. NO. 4 (PART II)
THE NOSE OF THE PITCHED WALL
MOVED TOWARDS LEFT BANK

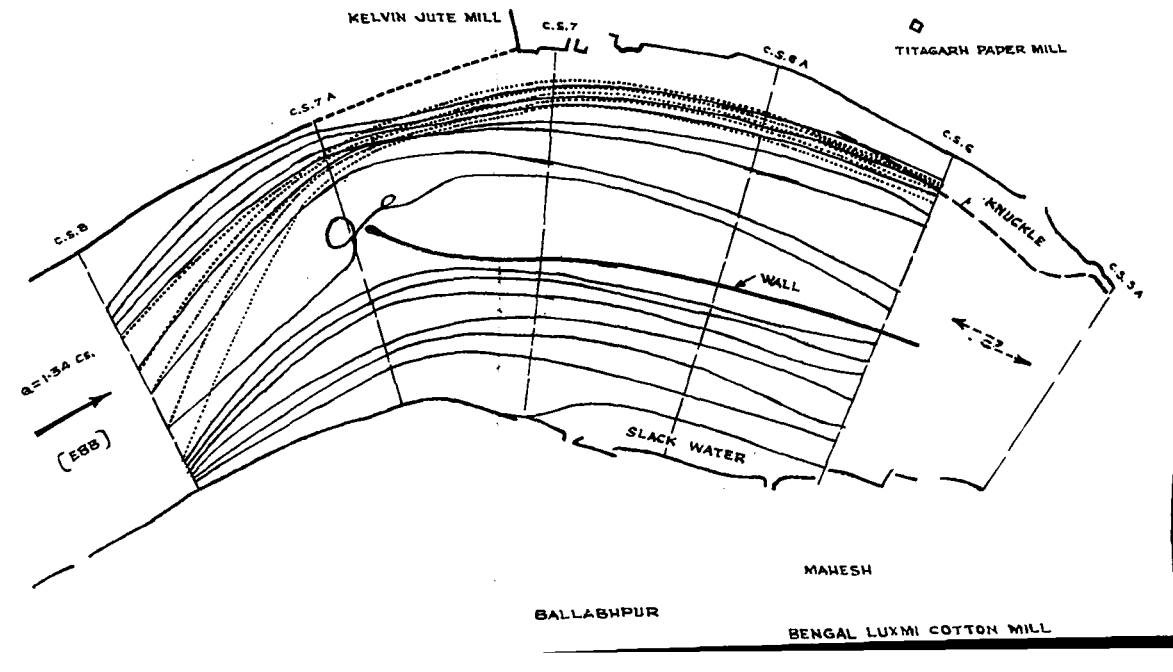


FIG. 8 EXPT. NO 5 (PART II)
LINES OF FLOW WITH WALL
PARALLEL TO LEFT BANK

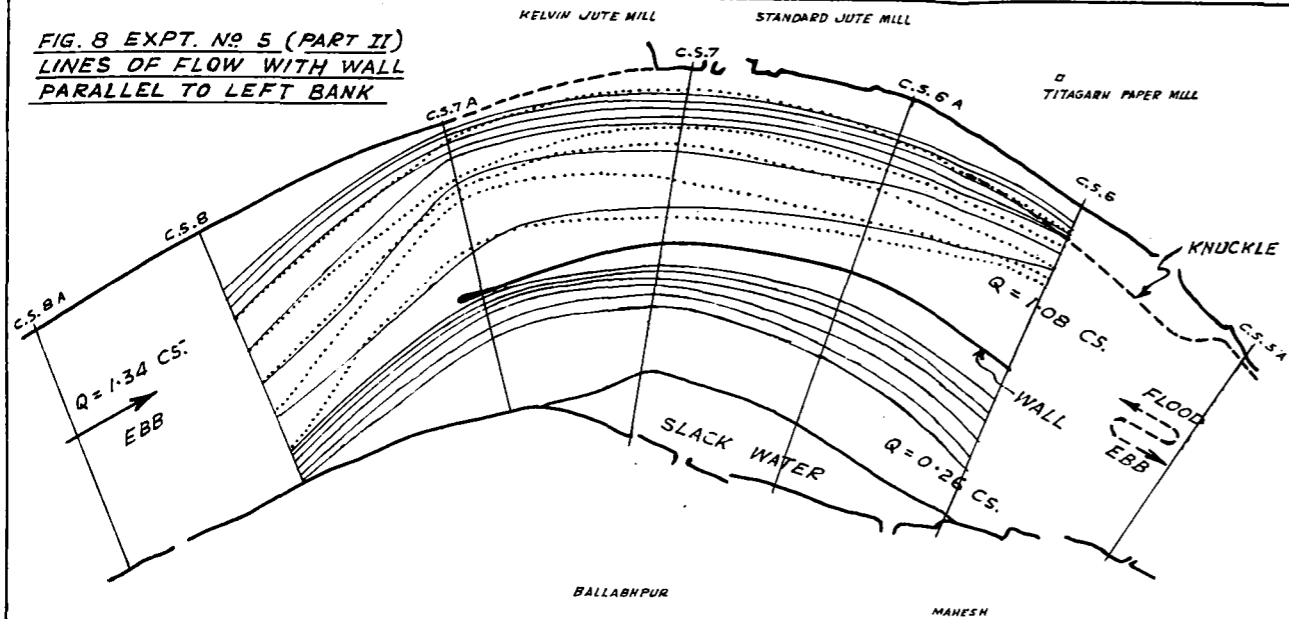


FIG. 8 (a) EXPT. 5 (PART II)

ISOVELS AT C.S. 6

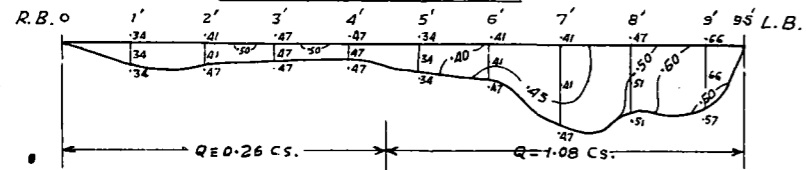


FIG. 9 EXPT. NO 6 (PART II)
LINES OF FLOW WITH THE
NOSE OF WALL SHIFTED D/S
AND MOVED AWAY FROM
THE RIGHT BANK

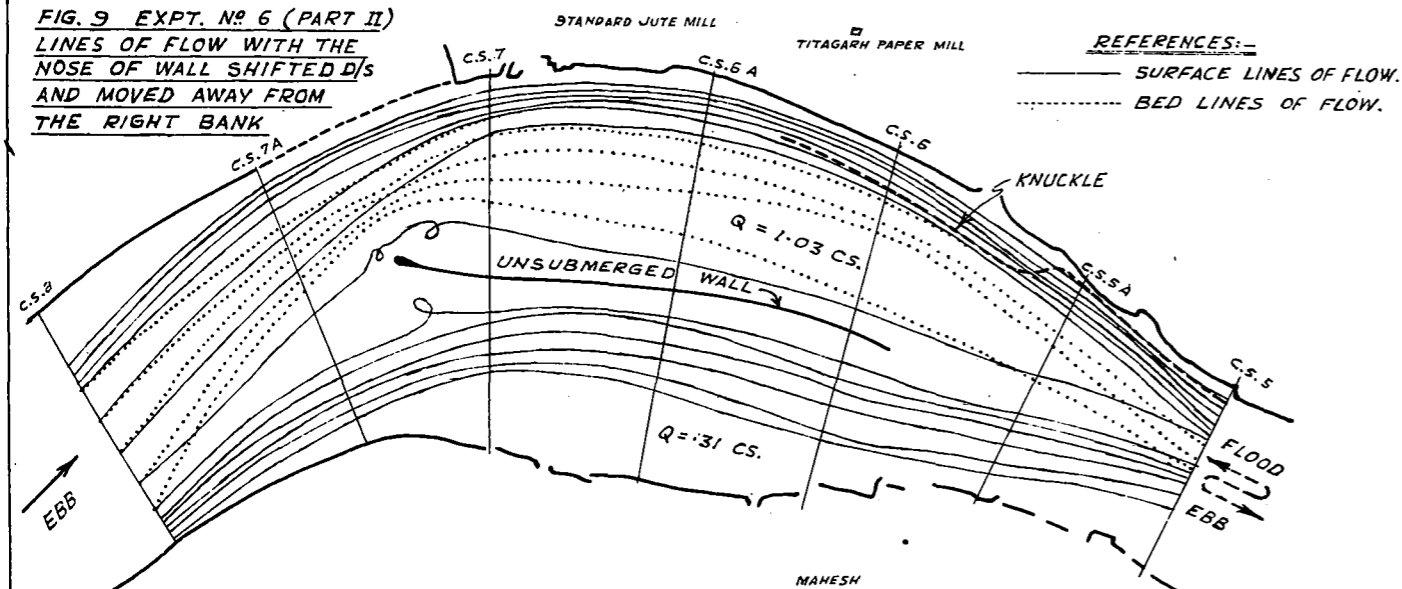


FIG. 9 (a) EXPT. 6 (PART II)

ISOVELS AT C.S. 6

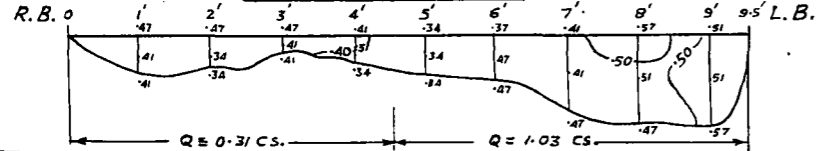


FIG. 9 (b) EXPT. NO 6 (PART II)
SHOWING THE FORMATION
OF CHANNEL AND BAR AFTER
10 TIDES WITH THE
CONSTRUCTION OF WALL.

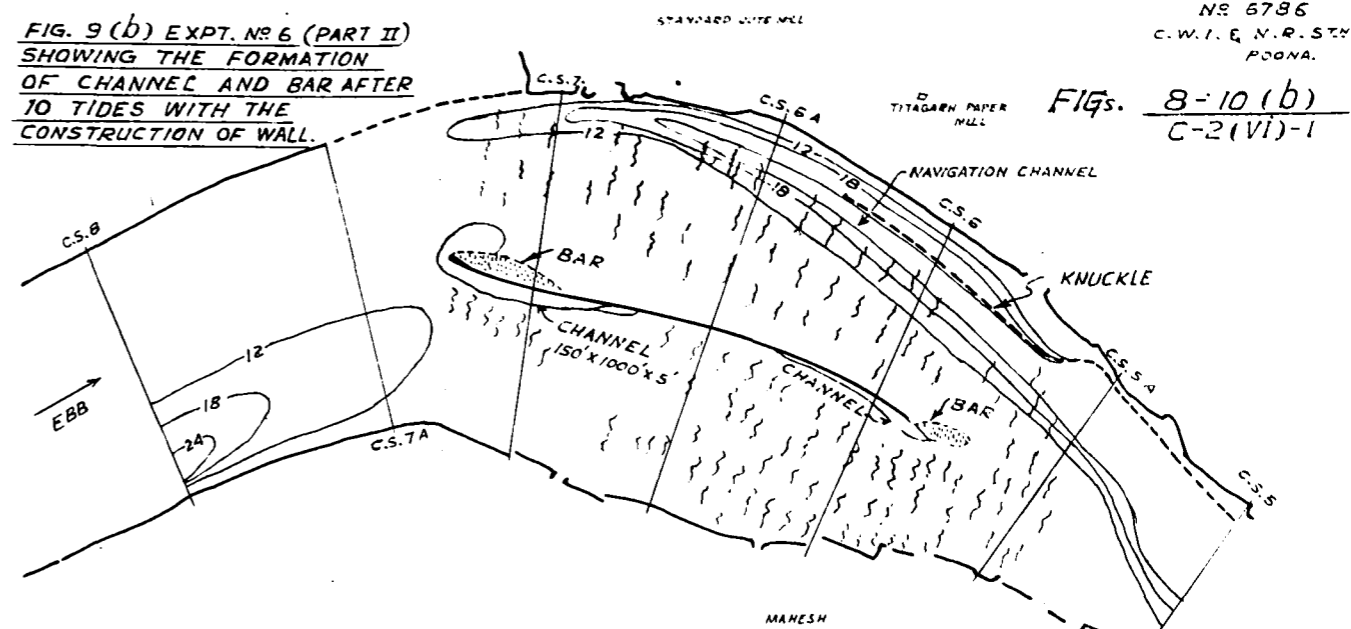


FIG. 10 EXPT. 7 (PART II)
LINES OF FLOW WITHOUT
WALL i.e. CONDITIONS
AS IN RIVER.

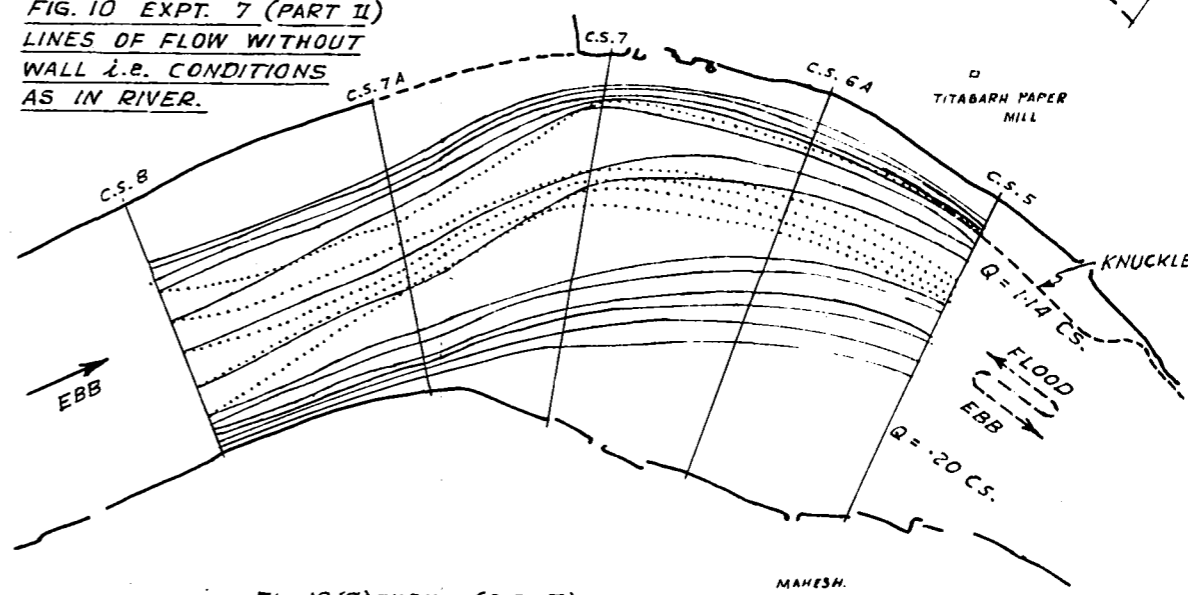


FIG. 10 (a) EXPT. 7 (PART II)

ISOVELS AT C.S. 6

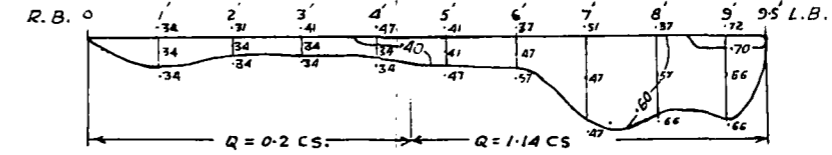
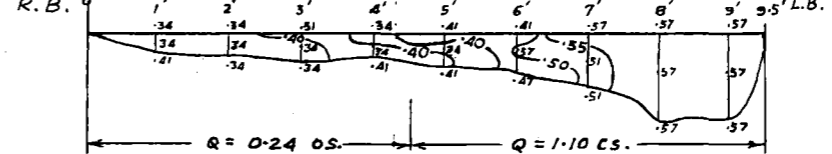


FIG. 10 (b) EXPT. 8 (PART II)

ISOVELS WITH ISLAND B ONLY AT C.S. 6.



1/300 = 1/60 SCALE
HOOGLY RIVER MODEL
SHOWING LINES OF FLOW,
FORMATION OF CHANNEL
AND ISOVELS.
CONTINUOUS EBB = 1.34 = 187,000 CS.
W.L. AT KONNAGAR = 13.5 FT.
ABOVE K.O.D.S.
SCALE 1" = 4' = 1200'

that undue deposition of injected silt took place at C.S. 7 A making the conditions unnatural. The excess deposit (measured to be 213 cft) was, therefore, removed. The charge was then reduced to 1 cft/hour for the remaining period of the experiment and in all the other experiments. This charge was found suitable by a comparison of model behaviour with prototype data of existing conditions.

Lines of flow were observed after running the equivalent of 24 hours and these are shown in figure 11. Comparing with figure 9, it will be seen that the lines of flow are nearly similar except that they are drawn only a little closer to the wall on the right side due to the continuous ebb action. With this differentiation it is possible to obtain results much quicker on the larger model by running continuous ebb tides.

Fig. 11.
C-2 (vi)-1

Fig. 12 shows the area of silting and scour in this larger model. Such portions, where silting had occurred without ripples, indicate regions of very low velocity which are likely to silt in the river; where silting had occurred in ripple formation, no silting in river is likely to occur but the bed will be just in a state of flux; where ripples had occurred without further silting, scour is likely in the river. This figure, therefore, indicates that the shoal immediately on the right of the wall is likely to be scoured.

Fig. 12.
C-2 (vi)-1

Fig. 12 and Fig. 9 (b) cannot be expected to give even approximately the same silt-scour conditions; because Fig. 9 (b) was obtained under tidal conditions in the 1/308 : 1/60 model without injection of silt while Fig. 12 was obtained in the less exaggerated 1/150 : 1/80 model (with injection) (i. e. of least silting below knuckle) with maximum continuous ebb discharge of 1,87,000 cusecs and Konnagar W.L. 13.5ft. above K. O. D. S.

Soundings were taken at C. S. 5A, 5B, 5C and 5D. These show the amount of silting downstream of the knuckle. As silting was equivalent to 2' to 2.5' in the prototype, it was decided to see whether accretion could be expected by constructing piles in addition to the wall.

Fig. 13.
C-2 (vi)-1

3. 3. (ii) Experiment No. 2— A longitudinal row of piles was constructed in position No. 1 (1) and cross row of piles was constructed as described in experiment 1 A. (2)

Fig. 14.
C-2 (vi)-1

Ebb tide representing 1,87,000 cusecs discharge condition was run continuously for 24 hours with the injection at 1 cft/hour (i.e. 0.3 part per 1000).

Fig. 14 shows the lines of flow after 24 hours of run. At C. S. 6 the flow was drawn towards the left bank evidently due to the piles.

Fig. 14.
C-2 (vi)-1

Area of silting and scour is shown in figure 15. Compared with figure 12, it indicates that the area between C. S. 6 and 7 A would silt up as a result of slack flow along the pitched wall; because area near the wall between C.S. 6 and 6 A silted up as a result of flow being drawn towards the bank.

Fig. 15.
C-2 (vi)-1

Soundings were taken downstream of the knuckle which showed an increase in silting equivalent only to 6" to 9" in the prototype, following the addition of piles.

Fig. 16.
C-2 (vi)-1

As some silting was indicated downstream of the knuckle it was decided to fill the scour-hole up to the top of the knuckle (i. e. 21 ft. below K. O. D. S) and continue the experiment. Indications of scour or silting would be evidence of the likelihood or otherwise of the eventual filling up of the scour below the knuckle. It was found, after a further 24 hours run, that a deposition of silt took place on the top of this artificially filled-up scour-hole indicating thereby that the scour-hole would silt up, at least to this depth, in due course.

Figure 17 shows the scouring and silting obtained in this experiment.

Fig. 17.
C-2 (vi)-1

3. 3. (iii) Experiment No. 3— In this experiment, the wall was removed, the bed was adjusted to 1939-40 survey and the experiment was run with continuous ebb with injection at 1 cft hour to compare the existing conditions in the river with those obtained with a wall and piles.

(1) See fig. 3/iii. iv opposite page. 40, Annual Report (Tech) 1946.

(2) See page 39 of Annual Report (Tech) for 1946.

Fig. 18.
C-2 (vi)-1

It was found that deposition of silt downstream of the knuckle did not take place. Ripples without silting were obtained downstream of the knuckle indicating a scouring tendency of the bed; Deposition of silt took place on the Right side shoal.

3. 4. Discussion of experiments — Experiments in 1/150 : 1/80 model showed that with the construction of the continuous wall deposition of the silt downstream of the knuckle was possible. Construction of piles near the knuckle in addition to the wall, were found to help the action of the wall and accelerate silting. In all the experiments with the wall when injection of red silt was done, it was found that there was a tendency of silting on the left side of the wall facing the knuckle, it was therefore, necessary to see whether this part near the wall remains silted under tidal action of two-way flow. This series of experiments are described in part IV.

PART IV.

4. 0. Introduction — In these experiments the effect of the wall was mainly tested under tidal conditions as against under continuous ebb in the previous series. The tidal curves for the upstream and downstream end of the model and at the knuckle were obtained from the simultaneous curves observed in the river on 23 rd September 1941 and in all experiments these curves were repeated 10 times before taking observations.

Fig. 19.
C-2 (vi)-1

4. 1. Correct bed movement cannot be expected in a model of so low a vertical exaggeration as 1/150 : 1/80 which is unnatural for reproducing curvature; slight difference in flow conditions and general bed movement observed in the model, by comparison with the conditions obtained in the 1/300 : 1/60 model, were eliminated by small adjustment in the tidal curves at entry and exit (keeping the conditions and tidal curve in the observation reach near the knuckle the same) — as shown in figure 19. As the observation and change in water levels at the knuckle caused by the wall is insignificant, it is reasonable to assume that the same small adjustments of tidal curves at entry and exit (vide fig. 19) are sufficient and necessary, with as without the wall.

4. 2, Experiments,

4.2 (i) Experiment No, 1— In this experiment, the wall was constructed to the alignment adopted in experiment No. 6 in the 1/300 : 1/60 model and the modified tidal curves were repeated 10 times with injection of red silt at 1 cft/hr during the ebb tide only. In freshet springs the river carries a greater silt charge and the action of ebb tide is stronger and the flood tide is weak. The Titagarh knuckle site, again, is 88 miles upstream of the mouth of the river and the flood tide, if it is carrying any silt or sand, is not likely to reach as far upstream as Titagarh. The injection of red silt during ebb tide was, therefore, similar in action to the suspended silt in the river; and a part of the same silt then moves upstream during flood tide — as would occur in the river because of influx of comparatively silt-free flood-water— and is deposited during the slack water period of the tide or during the intermixing of the ebb and flood tides, which is favourable for deposition.

Fig 20.
C-2 (vi)-1

Figure 20 shows the silting and scouring after 10 tides. There was the same tendency to silt on the left side of the wall as in previous experiments, which showed that the flood tide was not effective in scouring the silt deposited during the ebb.

Fig. 21,
C-2 (vi)-1

4. 2, (ii) Experiment No, 2— It was, therefore, decided to test such curvature — see fig. 21 — to the end of the wall facing the flood tide as would create sufficient turbulence to remove the deposited silt.

After completing 10 tides it was found that deposition of red silt on the left side of the wall was obtained in the same way as in experiment 1, part III. There was still not sufficient turbulence to remove the deposited silt on flood tide.

Fig. 22.
C-2 (vi)-1

Figure 22 shows the lines of flow under flood tide with this alignment of the wall.

Fig. 23.
C-2 (vi)-1

4.2. (iii) Experiment No, 3— It was decided to try still sharper curvature at the downstream end (facing flood tide) of the wall — vide fig. 23. Under continuous ebb condition, the flow had a tendency to concentrate on the left bank.

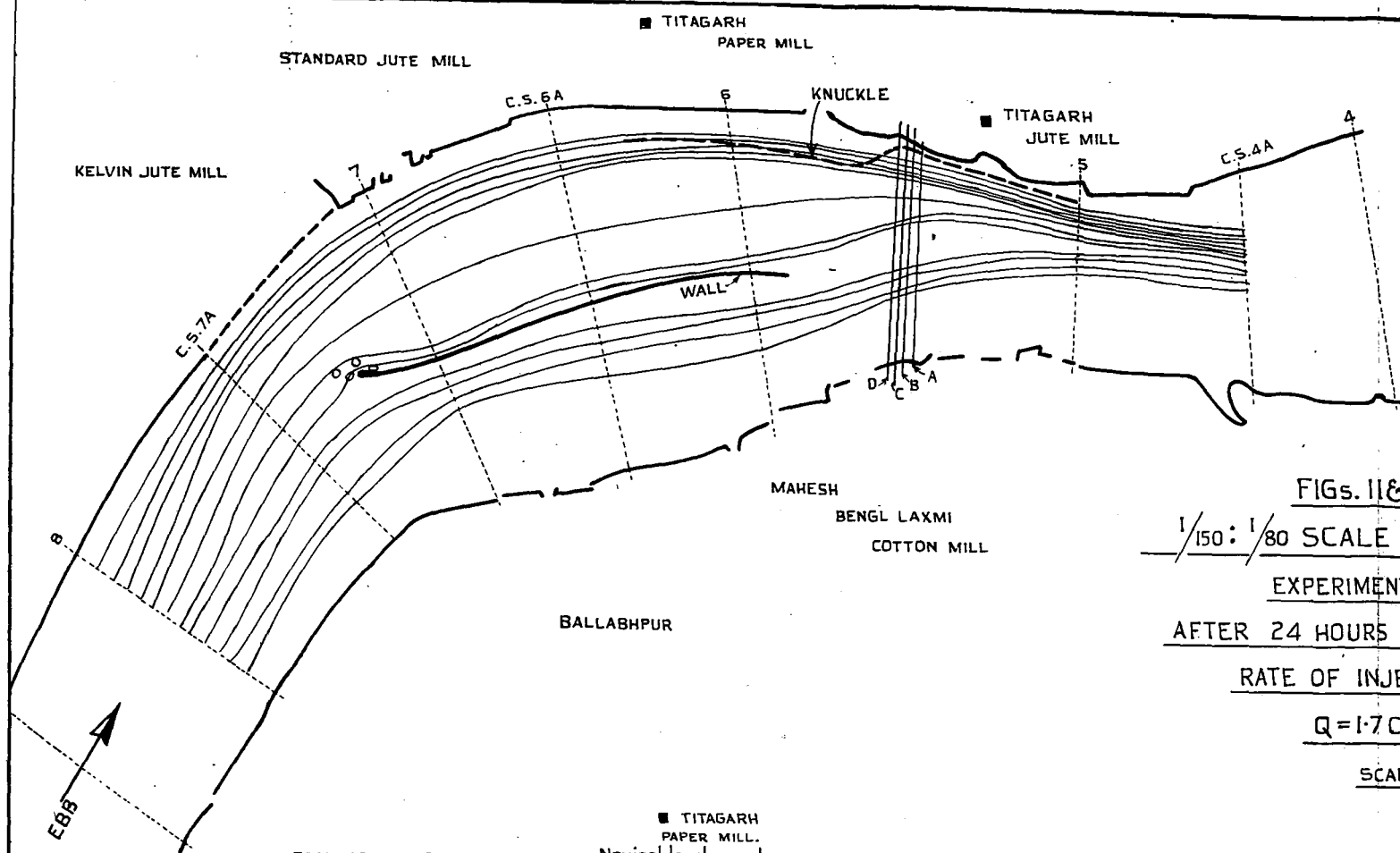


FIG. 11. SHOWING
LINES OF FLOW.

FIGS. 11 & 12 / C-2(VI)-1

$1/150 : 1/80$ SCALE HOOGHLY RIVER MODEL.

EXPERIMENT I. (PART III)

AFTER 24 HOURS OF CONTINUOUS EBB AND

RATE OF INJECTION 1 C.F.T./HR. (0.3 PART PER 1000)

$Q = 1.7 \text{ Cs.} \approx 187,000 \text{ Cs.}$

SCALE $1" = 8' \approx 1200'$

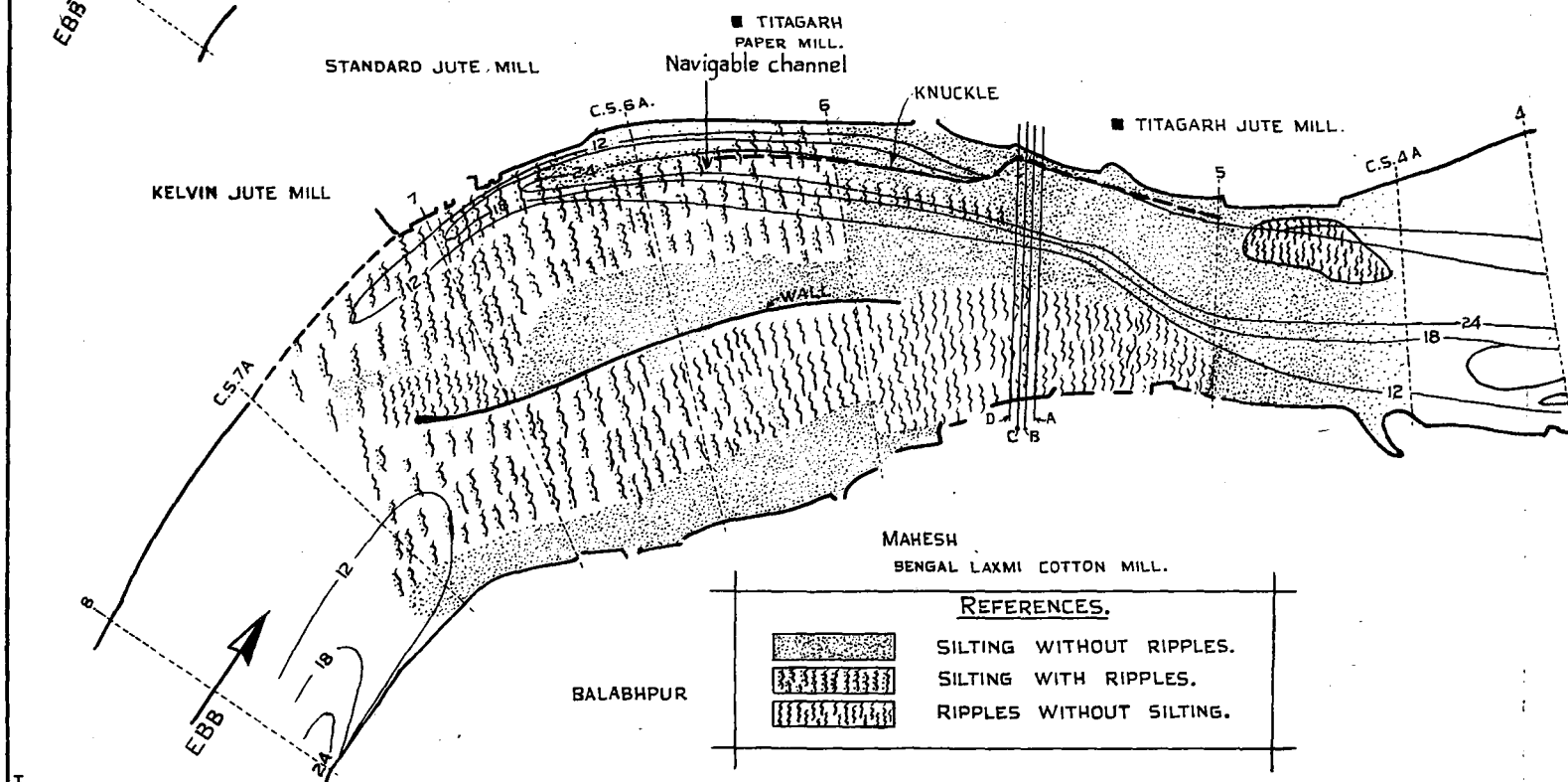


FIG. 12. SHOWING
SILTING AND SCOURING.

REFERENCES.

	SILTING WITHOUT RIPPLES.
	SILTING WITH RIPPLES.
	RIPPLES WITHOUT SILTING.

FIG 13 SHOWING SHORT C. S.

NO:-6775
C.W.I. & N.R. STN. POONA.

C-2(VI)-1

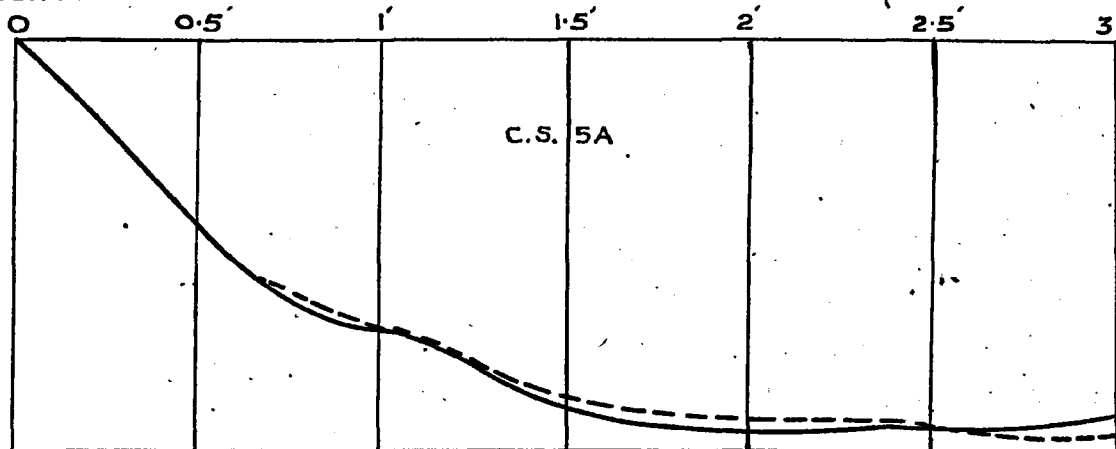
SCALE:- HOR. 1" = 5' = 75"
VER. 1" = 5' = 40"

REF:-

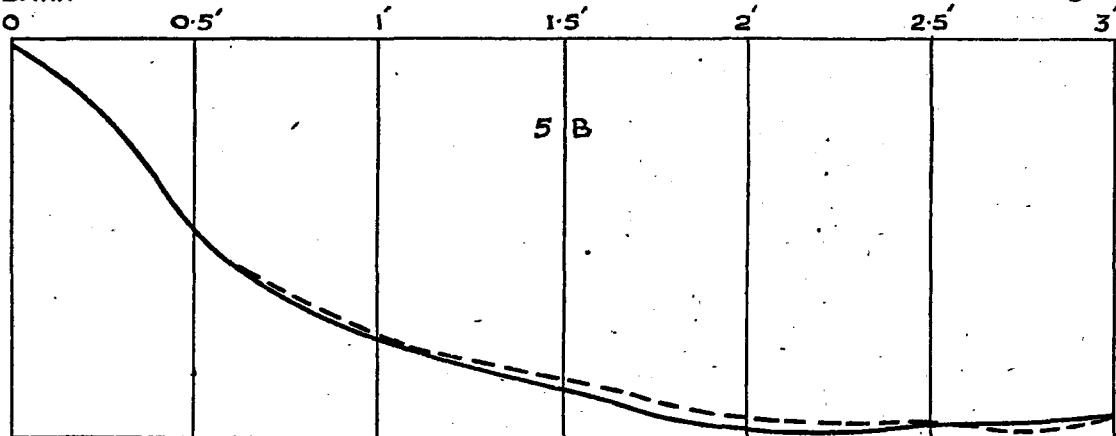
BED AS LAID. ———

SILTING WITH WALL ONLY. - - -

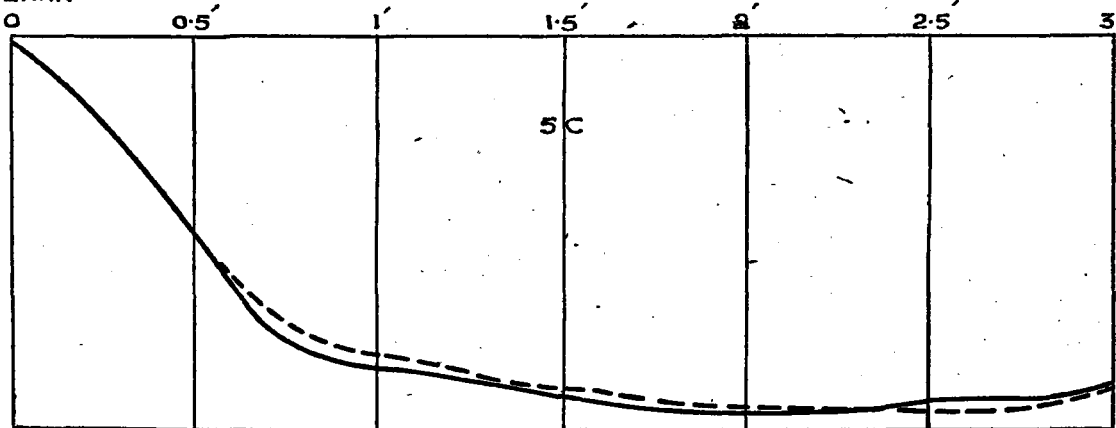
L. BANK



L. BANK



L. BANK



L. BANK

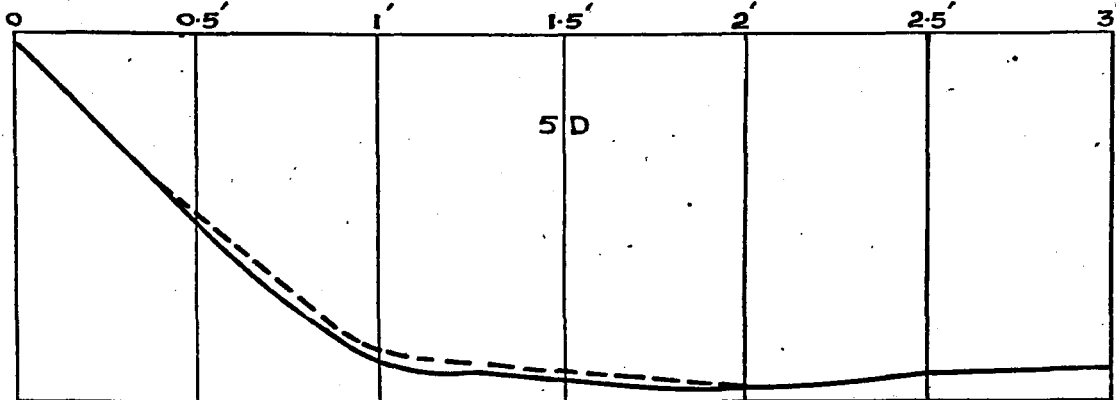


FIG. 14
1/150 : 1/80 SCALE HOOGHLY RIVER MODEL
EXPT. 2, (PART III)
LINES OF FLOW WITH WALL & PILES, CONTINUOUS EBB.
Q = 1.7 Cs. = 187,000 Cs.
RATE OF INJECTION 1 GFT/HR.
(i.e. 0.3 PART / 1000.)
SCALE: 1" = 8' = 12000

FIGS. 14 & 15
C-2 (VI)-1

REF:-
 SURFACE LINES. ————
 BED ""

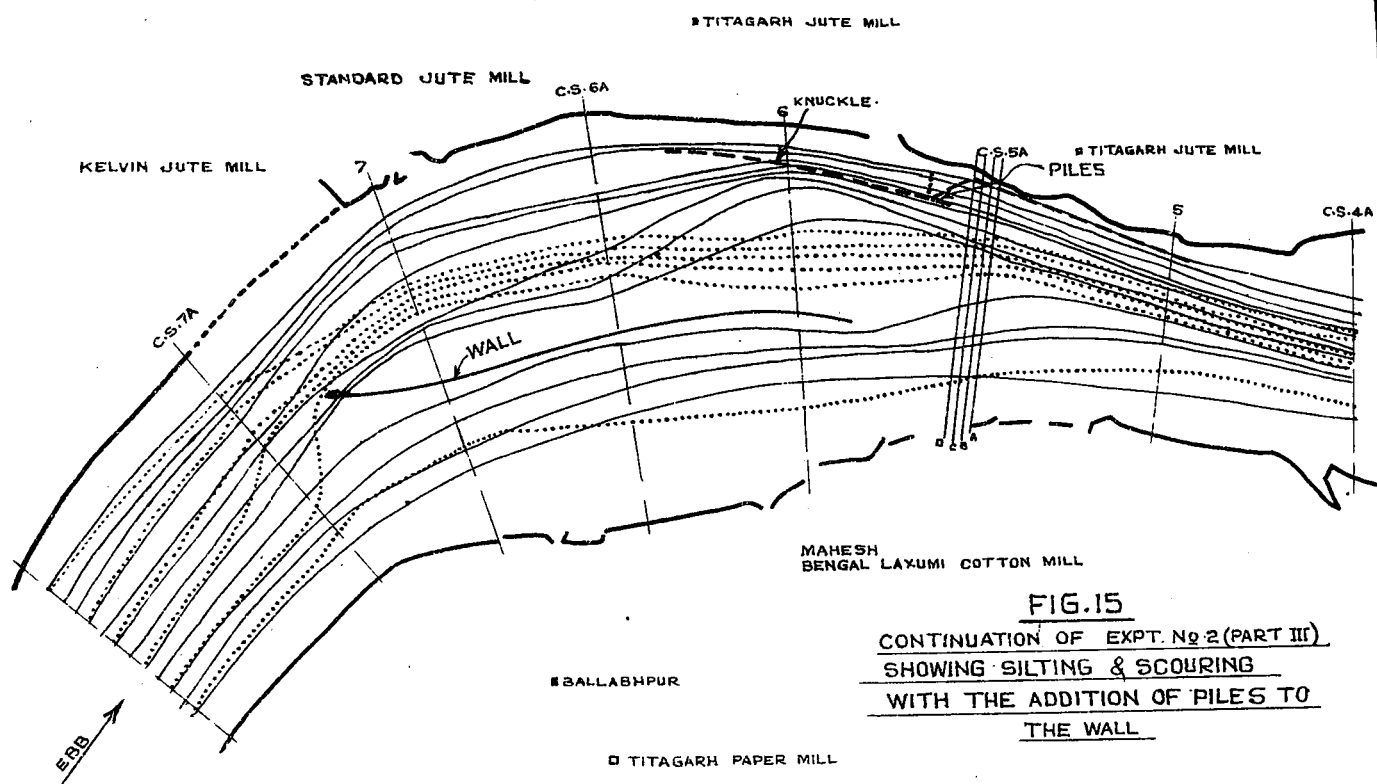
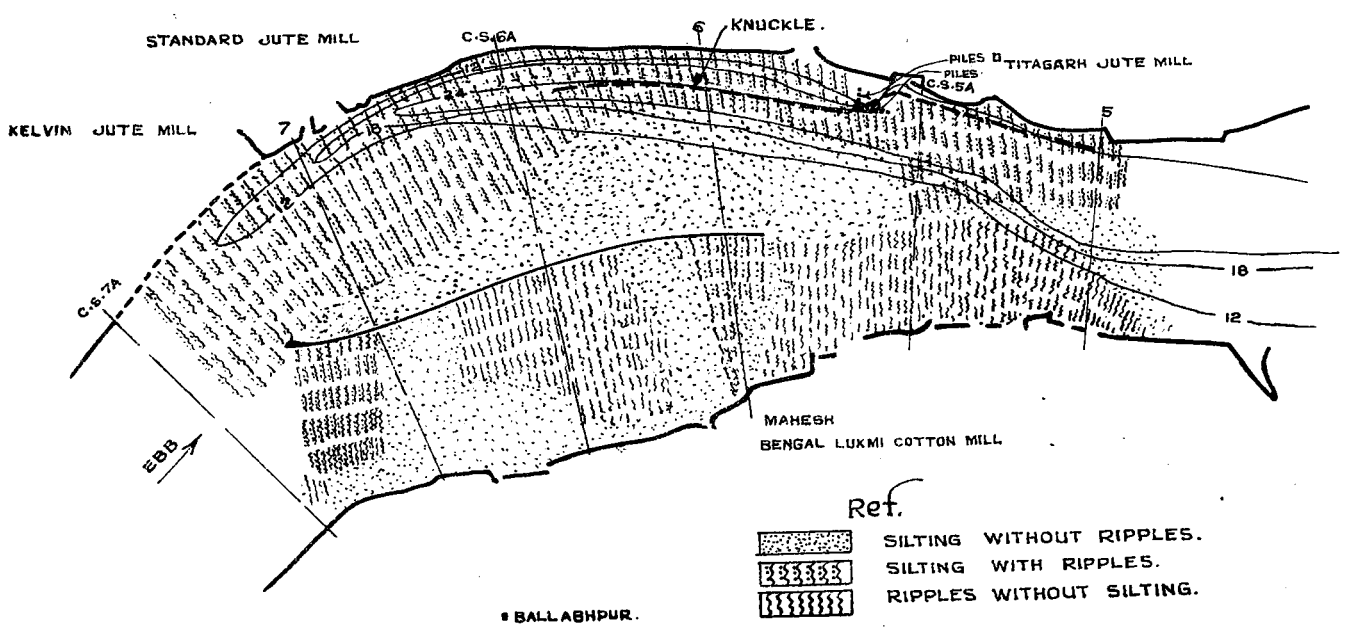


FIG. 15
CONTINUATION OF EXPT. No 2 (PART III)
SHOWING SILTING & SCOURING
WITH THE ADDITION OF PILES TO
THE WALL



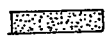
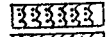

Ref.
 SILTING WITHOUT RIPPLES.
 SILTING WITH RIPPLES.
 RIPPLES WITHOUT SILTING.

FIG. 16. SHOWING SHORT C. S.

NO- 6774
C.W.I. & N.R. STN.
POONA.

C-2(VI)-1

EXPT 2 PART III

SCALE:~ HOR. 1" = 5' = 75'
VER. 1" = 5' = 40'

REF.:~
— BED AS LAID.
- - - SITTING WITH WALL ONLY.
- · - · - SILTING WITH WALL & PILES

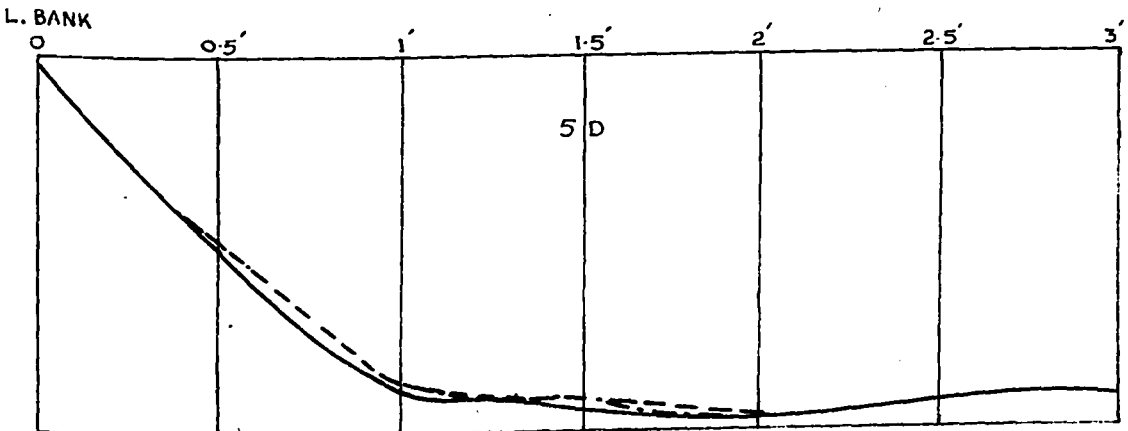
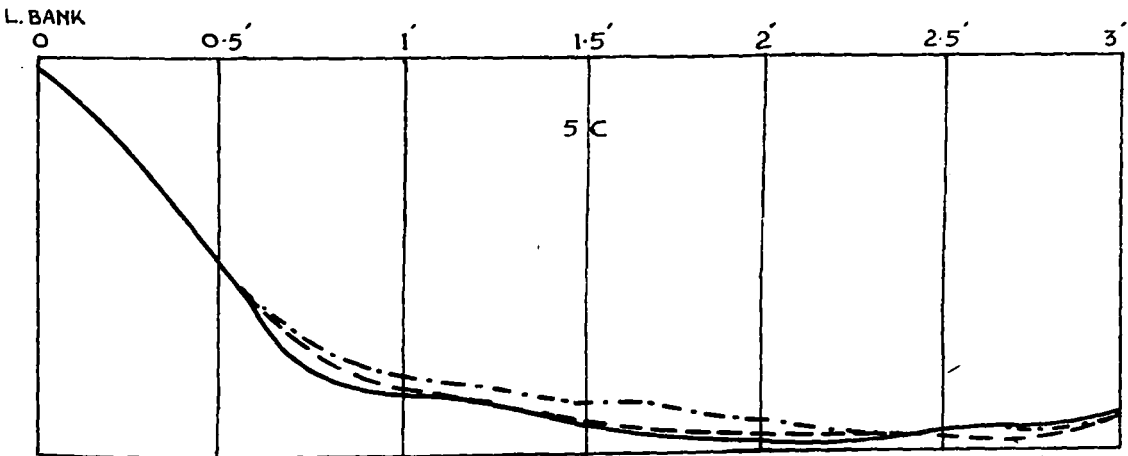
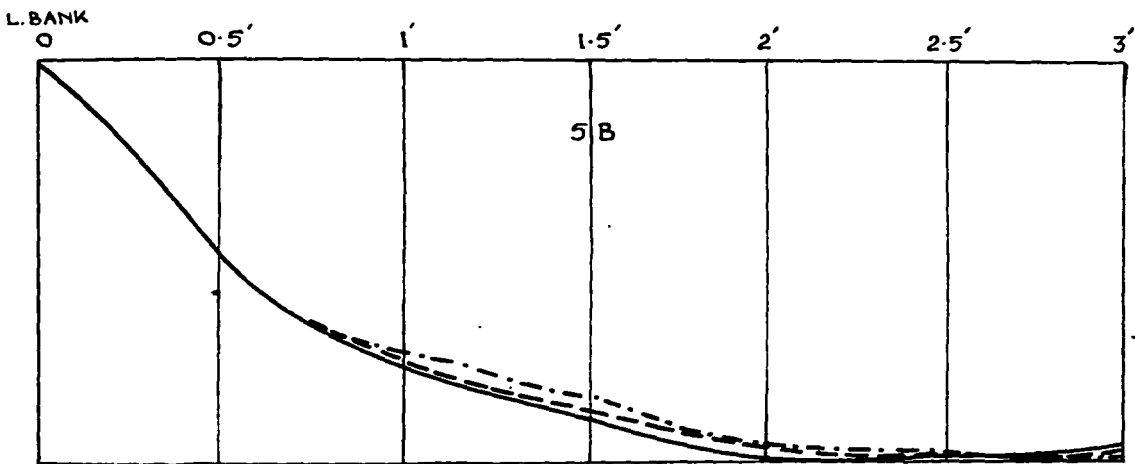
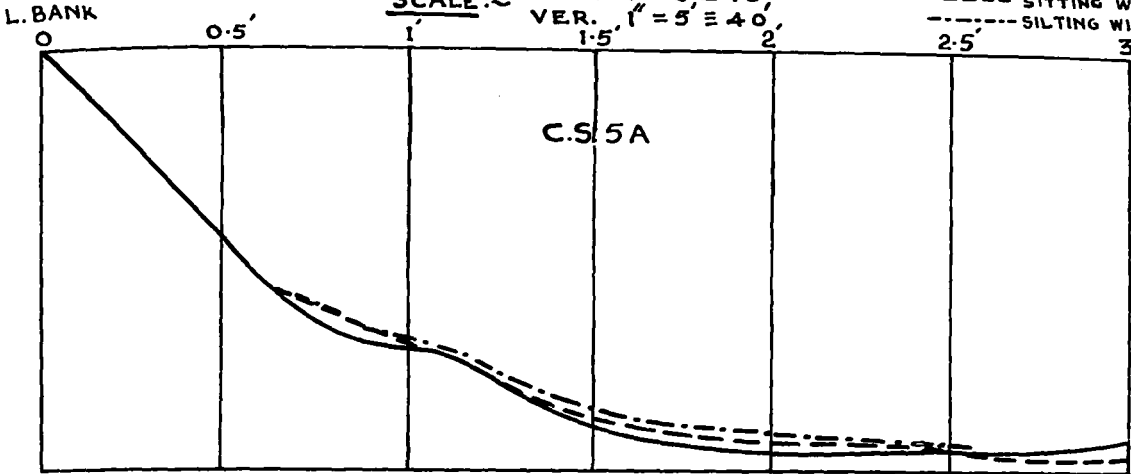


FIG:- 17

EXPT. 2

SHOWING SILTING & SCOURING AFTER 24 HOURS
 THE SCOUR-HOLE $\frac{0}{5}$ THE KNUCKLE FILLED
 UP TO THE TOP OF THE KNUCKLE

RATE OF INJECTION [c.f.t./hr. (0.3 PART / 1000)]

$Q = 1.7 \text{ cs.} \approx 187,000 \text{ c.s.}$

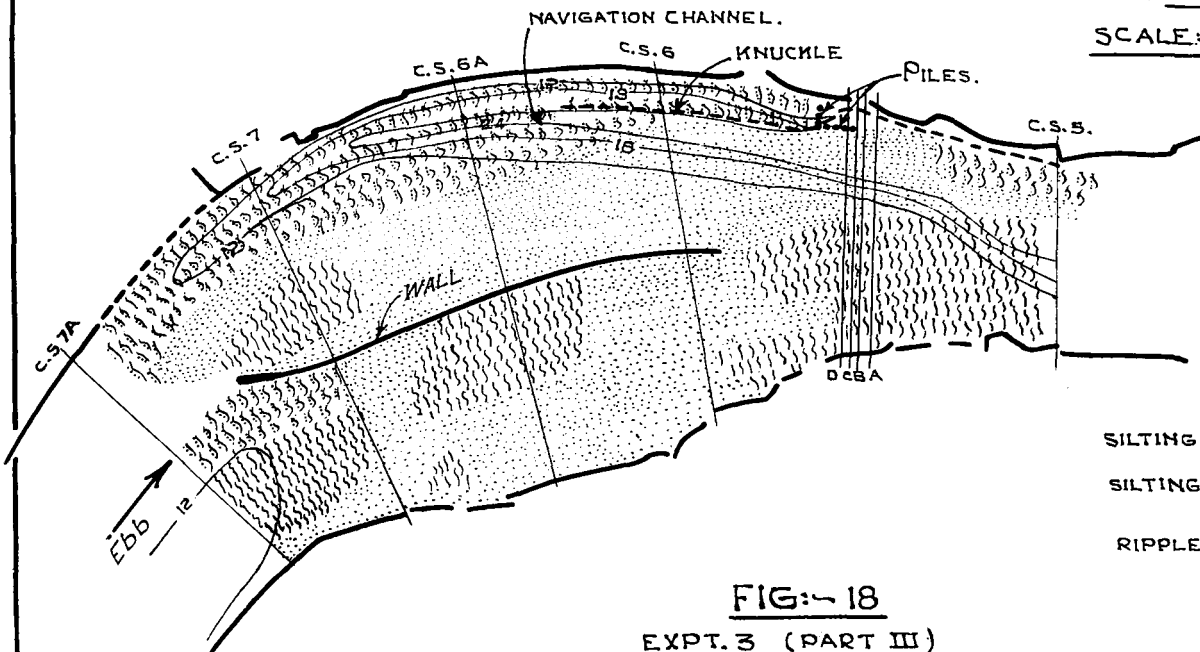
NR 6777
 C.W.I. & N.R. STN
 POONA.

FIGS. 17 & 18 / C-2(VI)-1

$\frac{1}{150} : \frac{1}{80}$ SCALE HOOGHLY RIVER MODEL

(PART III)

SCALE:- 1" = 8' = 1200'



REFERENCES:-


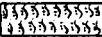
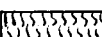
- SILTING WITHOUT RIPPLES. 
- SILTING WITH RIPPLES. 
- RIPPLES WITHOUT SILTING. 

FIG:- 18

EXPT. 3 (PART III)

SHOWING SCOURING & SILTING WITH INJECTION

AFTER 24 HOURS AS IN PROTOTYPE

RATE OF INJECTION [c.f.t./hr. (0.3 PART / 1000.)

$Q = 1.7 \text{ cs.} \approx 187,000 \text{ cs.}$

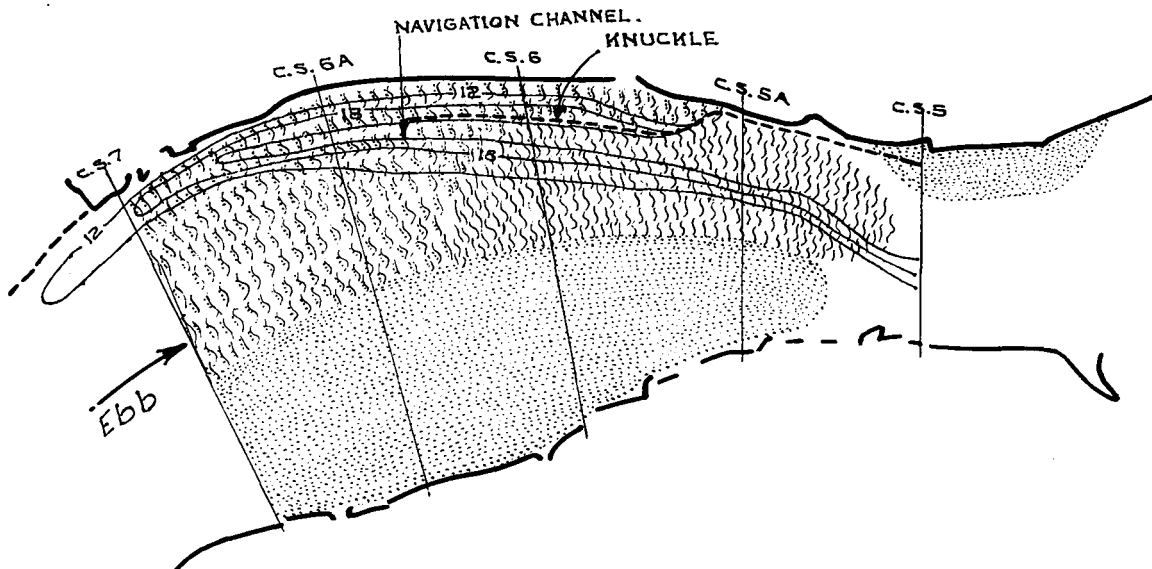
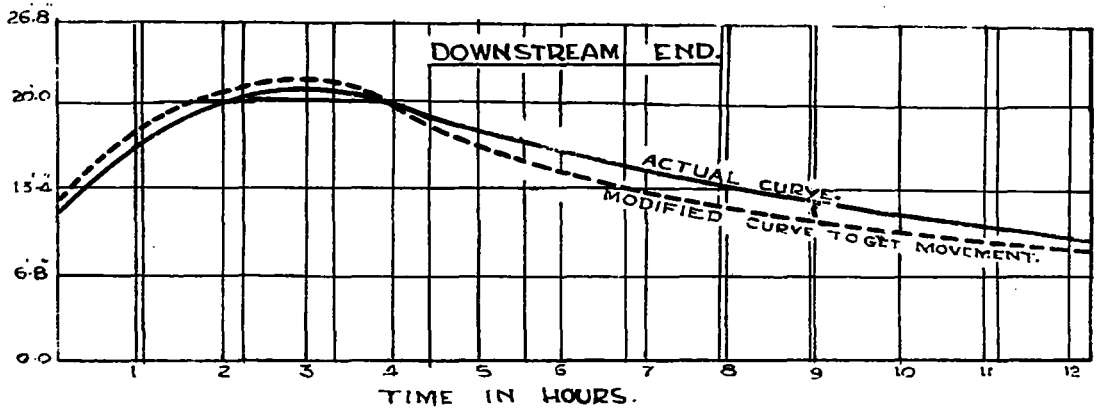


FIG. 19/C-2(vi)-1

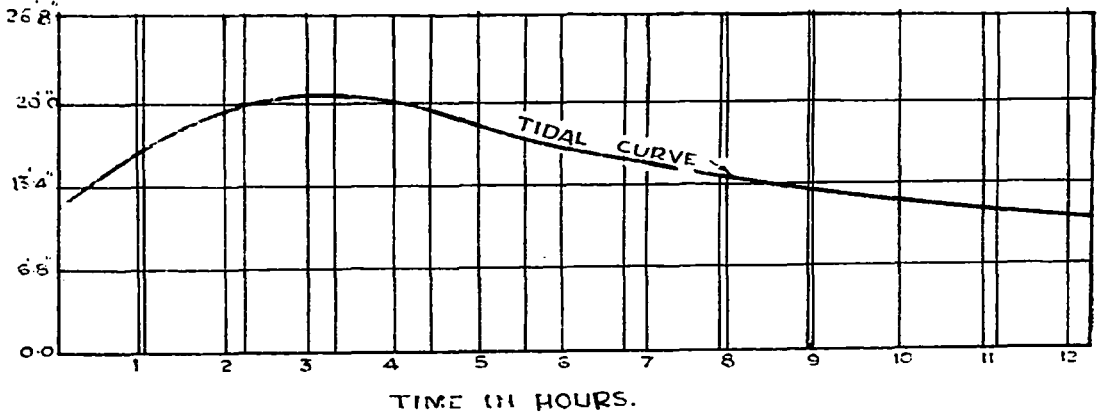
SHOWING MODEL TIDAL CURVES ACCORDING
TO SIMULTANEOUS CURVES OF-

-23RD. SEPTEMBER 1941.

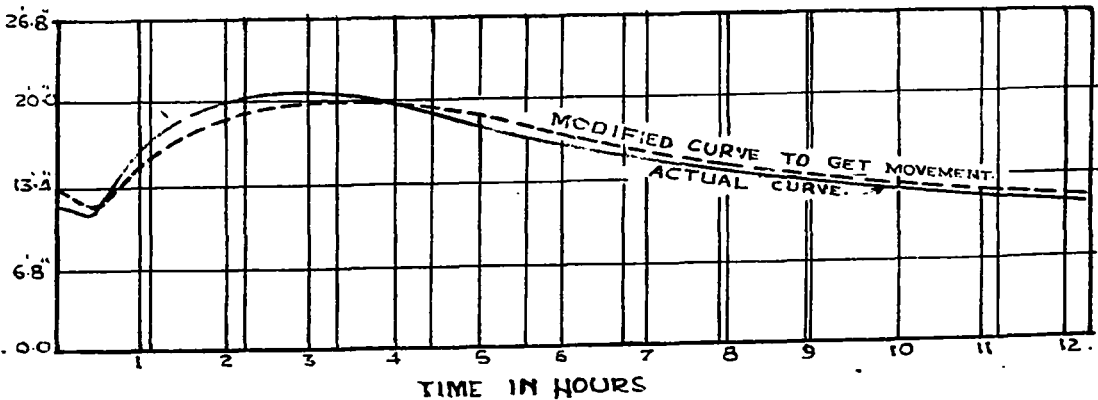
1 HOUR = 3.6 MIN. (T. SCALE $\frac{1}{16.7}$ IN MODEL.)



TIDAL CURVE AT C.S. 7 NEAR
THE KNUCKLE.



UPSTREAM END.



↑
FEET ABOVE K.O.D.S.
↓

K.O.D.S.

FIG. 20
EXPT. 1.

SHOWING SILTING AFTER 10 TIDES.
RATE OF INJECTION 1 C.F./HR. (0.3 PART PER 1000)

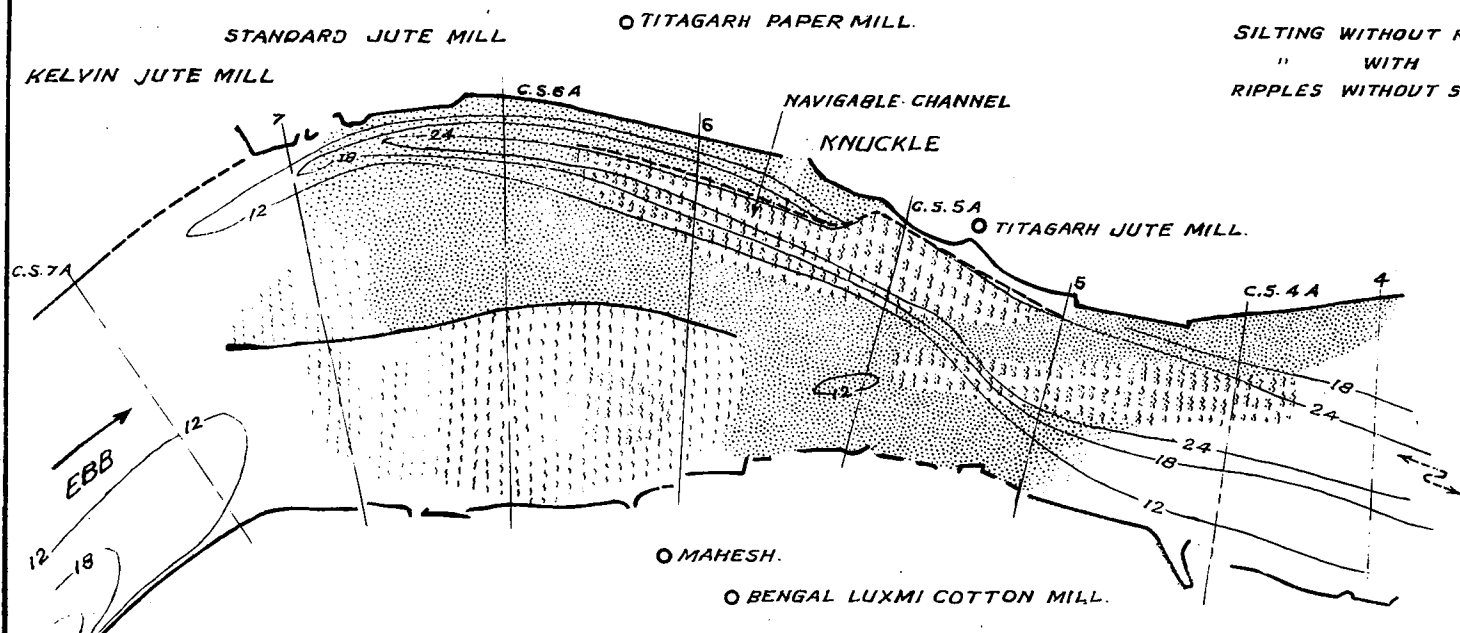
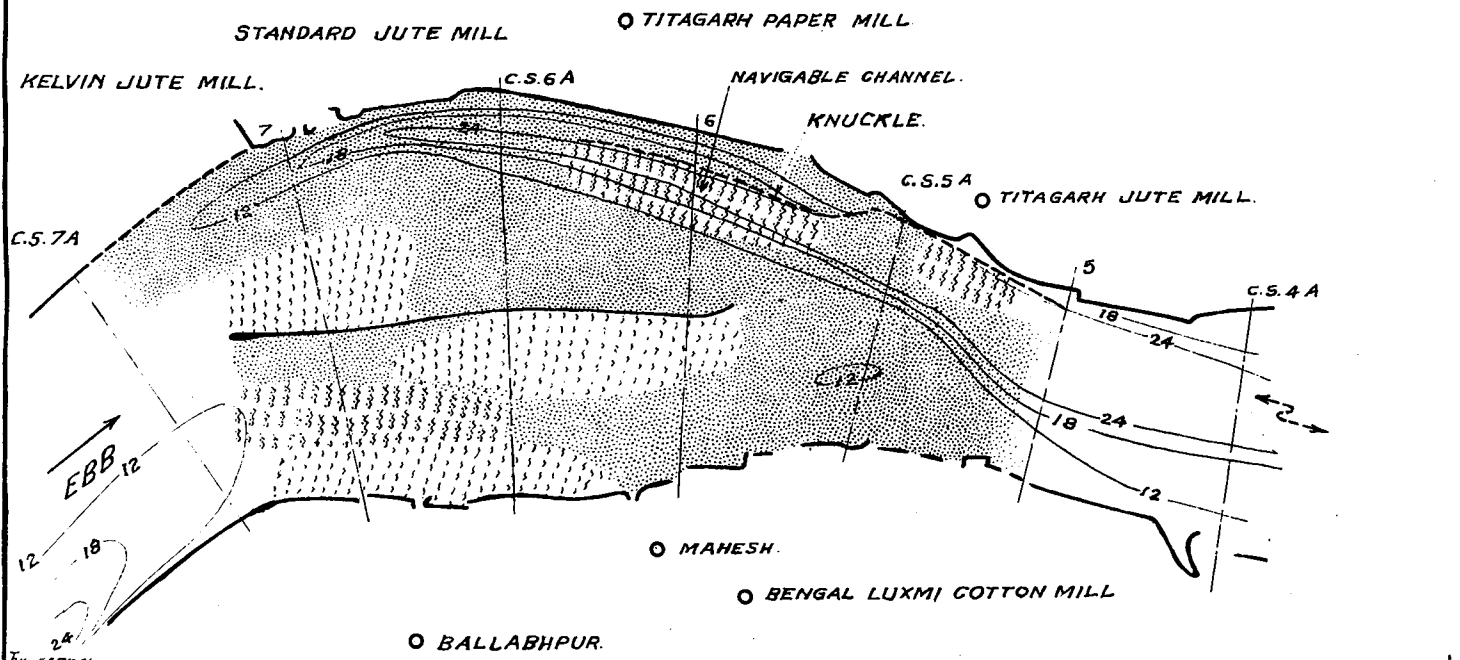


FIG. 21
EXPT. 2

SHOWING SILTING AFTER 10 TIDES.
RATE OF INJECTION 1 C.F./HR. (1 C. 0.3 PART PER 1000)



BY SAPKAL

FIGS. 20-23 / C-2 (VI)-1

1/150:1/80 SCALE HOOGLY RIVER MODEL.

PART. IV.
SCALE = 1" = 8' = 1200'

SILTING WITHOUT RIPPLES. [Pattern]
" WITH " [Pattern]
RIPPLES WITHOUT SILTING. [Pattern]

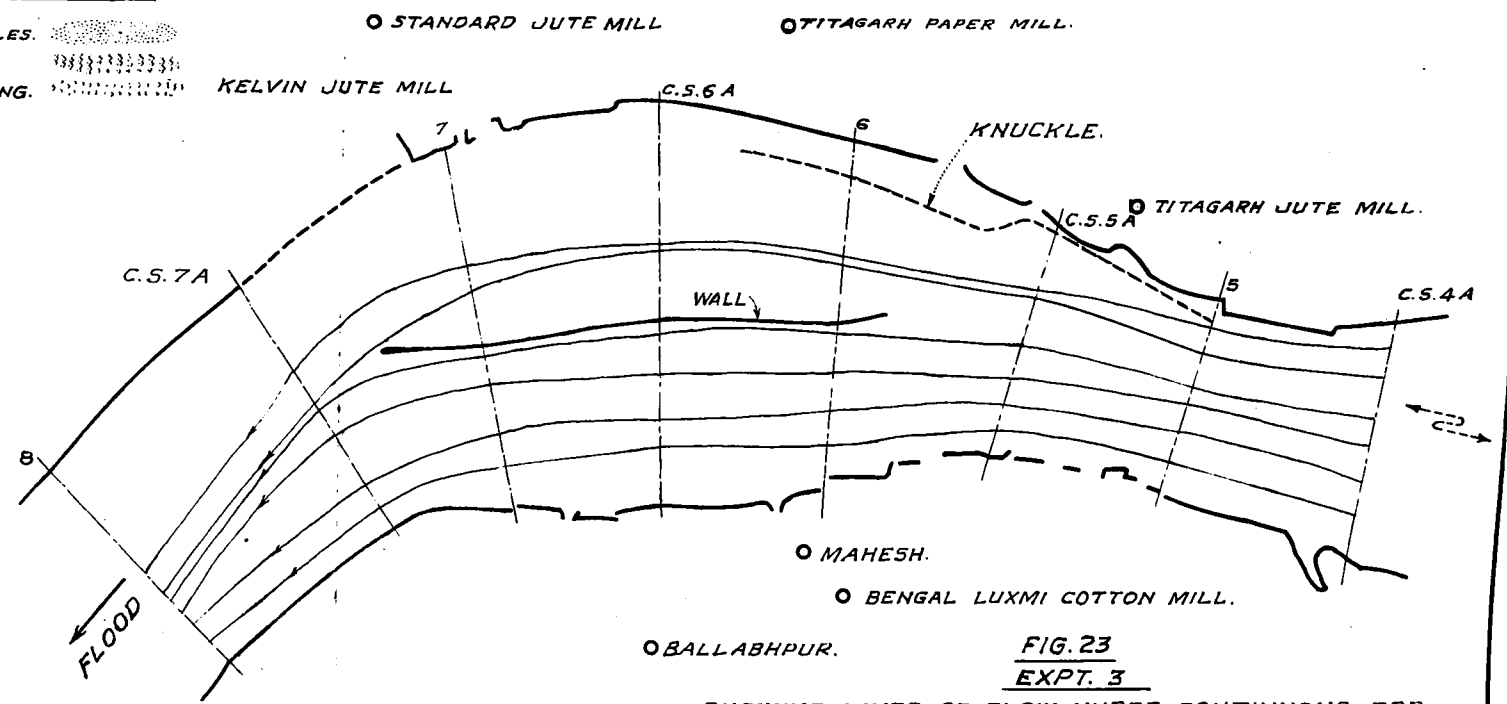


FIG. 23
EXPT. 3

SHOWING LINES OF FLOW UNDER CONTINUOUS EBB,
AFTER 10 TIDES, W.L. AT KONNAGAR = 13.5 ABOVE K.O.D.S.

Q = 1.7 Cs. = 187000 Cs.

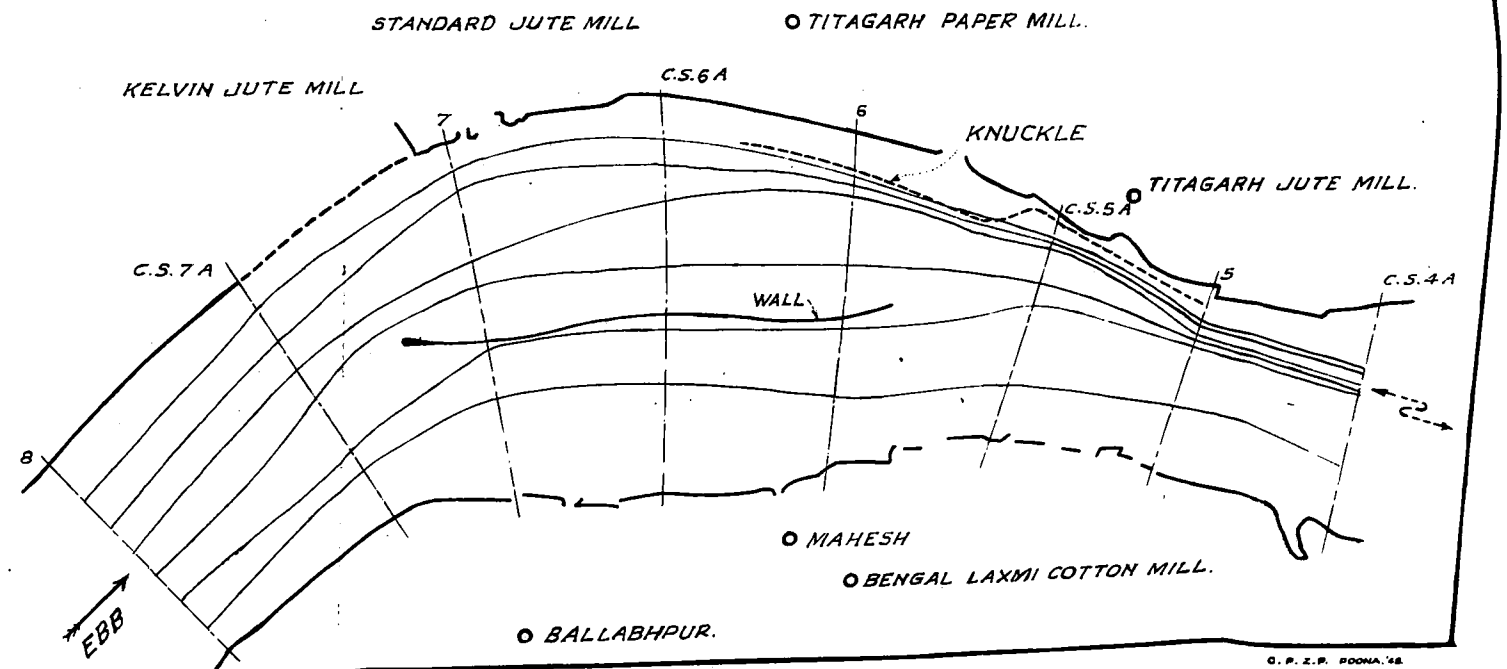


Figure 24 shows the silting after 10 tides. It was found that with the increased curvature at the downstream end the turbulence was still not sufficient to scour the deposited silt on the left side of the wall.

Fig. 24.
C-2 (vi)-1

Experiments 2 and 3 show that the curvature given to the downstream end of the wall does not help to scour the deposited silt and relieve the concentrated flow on the left bank.

4.2. (iv) Experiment No. 4 — To determine the maximum likely accretion, the bed on the left side of the wall, which showed a tendency to silt, was raised by one inch equivalent to 6'-8" of the prototype. The alignment of the wall was as in experiment No. 1 part III. This is a useful model technique to expedite results on maximum accretion, which would otherwise take considerable time in the model.

The model showed that silting would occur on the left side of the wall up to 7 ft. over the existing bed — vide figure 25.

Fig. 25.
C-2 (vi)-1

Figure 26 shows the lines of flow under ebb after 10 tides; concentration of flow was observed at C.S. 5 A and 5 as a result presumably of the shoaling along the left side of the wall.

Fig. 26.
C-2 (vi)-1

4.3. Conclusion — These experiments showed that after accretion the conditions would again deteriorate, resulting in concentration of flow near the knuckle following the shoaling along the left side of the wall which is likely to occur. Thus, though initially the wall may induce some silting below the knuckle, this is likely to be of little lasting good. The conditions at the knuckle are so set that no practicable solution at reasonable cost appears feasible.

PART V.

5. As a purely basic experiment, it was decided to test whether the contraction of the river section following the curve was primarily responsible for the formation of the knuckle, vide para 1. 1 above.

5.1. Experiments — To test whether the 'bottle-neck' downstream of the existing knuckle, contributed (1) towards the formation of the knuckle the following experiments were done:—

5.1. (i) Experiment No. 1 — The 24' contour of the knuckle was continued up to C. S. 5 where the contraction has taken place, to represent the bank under the pre-knuckle conditions — vide Figure 27. The bank in the model, 5' upstream and 5' downstream of the knuckle, was constructed in alternating layers of sand and clay similar to the Mulajore borings.

Fig. 27.
C-2 (vi)-1

Figure 27 shows the lines of flow under continuous ebb when the experiment was started showing the concentration of flow at the knuckle site.

After 12 hours of continuous ebb, a slip of the bank occurred at the site of the knuckle due to undermining of the sand layers. The material of the slip was removed, as the velocities in model were not sufficient to remove it.

After 42 hours of continuous ebb a shape of the bank similar to actual knuckle was formed showing clearly the tendency of the bank to cave at the knuckle site

5.1. (ii) Experiment No. 2—The object of this experiment was to see whether the knuckle at C. S. 5A is reformed if the existing contraction downstream of the knuckle is removed. The left bank was, therefore, cut back, as shown in figure 28, keeping the same bank slope as before.

Fig. 28.
C-2 (vi)-1

To get the desired width at C. S. 5, the width of the channel was found out from the Lacey formula $W = 2.67 \sqrt{Q} = 2.67 \sqrt{187,000} = 1160$ ft., as a first approximation, where W is the Width of the channel and Q the maximum discharge.

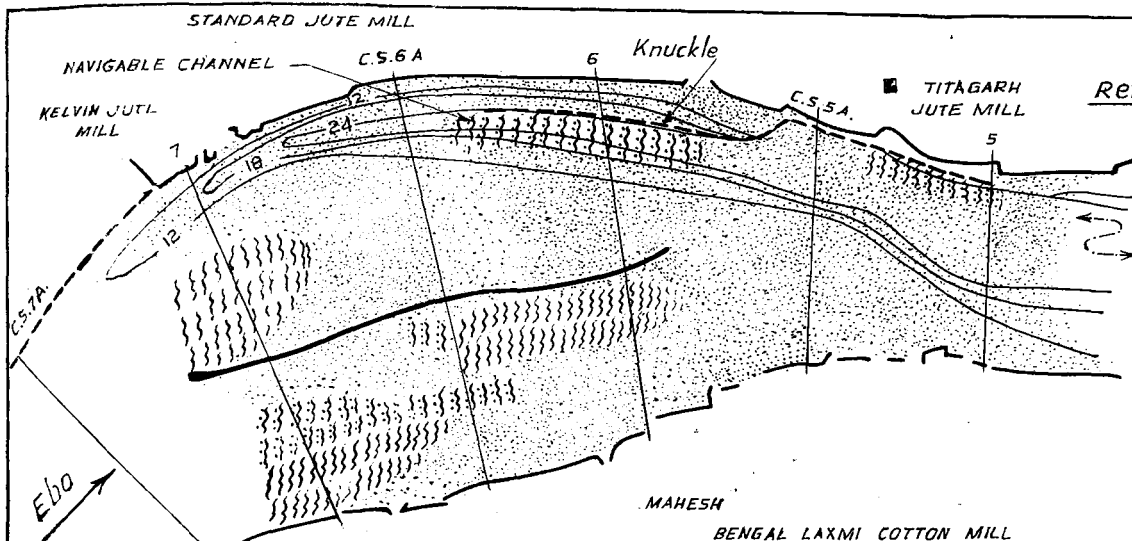
The existing width of the channel is 900 ft. and so the bank was receded by 260' at C. S. 5 to get the necessary width of the channel.

Continuous ebb flow was then reproduced as in experiment No. 1 (part V). It was observed that there was much less concentration of flow at C. S. 5A—the knuckle site—compared to that in the previous experiment. The experiment was continued for 42 hours and no bank slips were observed during this period.

It may be, therefore, concluded that the 'bottle-neck' downstream of the present knuckle has contributed towards the formation of the knuckle.

The experiments indicated that no simple remedy is possible to ameliorate the conditions at the Titaghar mill; drastic measures such as the removal of the 'bottle-neck' contraction downstream of the wall are not practicable, having regard to the small likely good, and the not inconsiderable expenditure.

(1) See page 40 of 1946 Annual Report.



Ref:
 [Stippling] Silting without ripples.
 [Wavy lines] Silting with ripples.
 [Vertical lines] Ripples without silting.

FIG. 24
EXPERIMENT 3.
 SHOWING SILTING AFTER 10 TIDES WITH THE ALIGNMENT OF THE WALL AS SHOWN.
 RATE OF INJECTION 1 cft./hr.
 (0.3 PARTS PER 1000)

FIGS. 24-26/C-2(VI)-1.
1/150:1/80 SCALE HOOGHLY RIVER MODEL

PART IV

W.L. AT KONNAGAP = 13.5 FT ABOVE K.O.D.S.

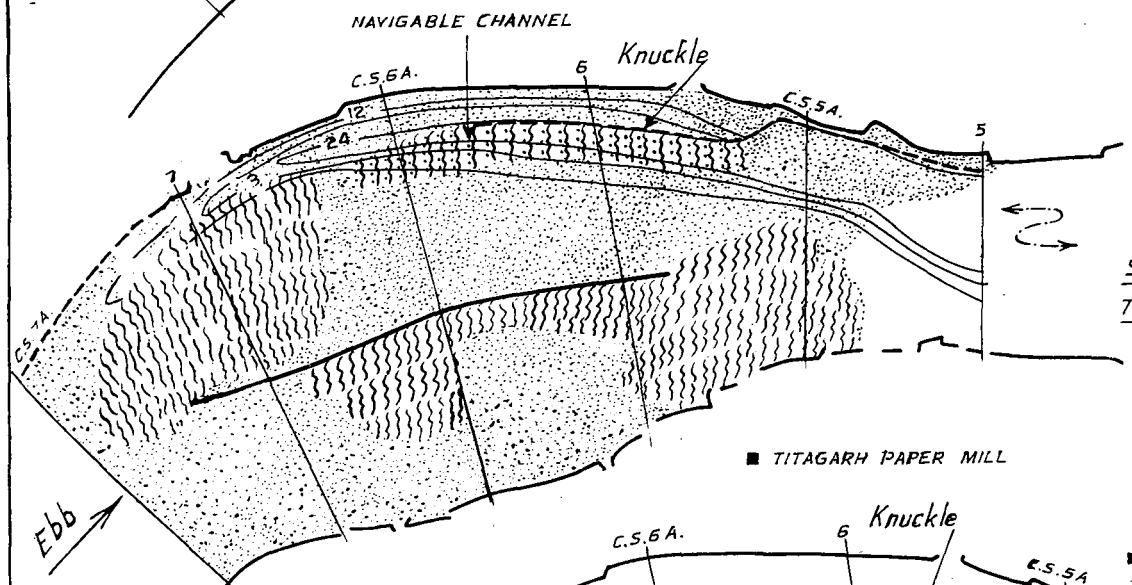


FIG. 25 $Q = 1.7 = 187,000 \text{ CS.}$
EXPERIMENT 4. $\text{scale: } 1" = 8' = 120'$
 SHOWING SILTING AFTER 10 TIDES WITH THE ALIGNMENT OF THE WALL AS SHOWN.
 RATE OF INJECTION 1 cft./hr.

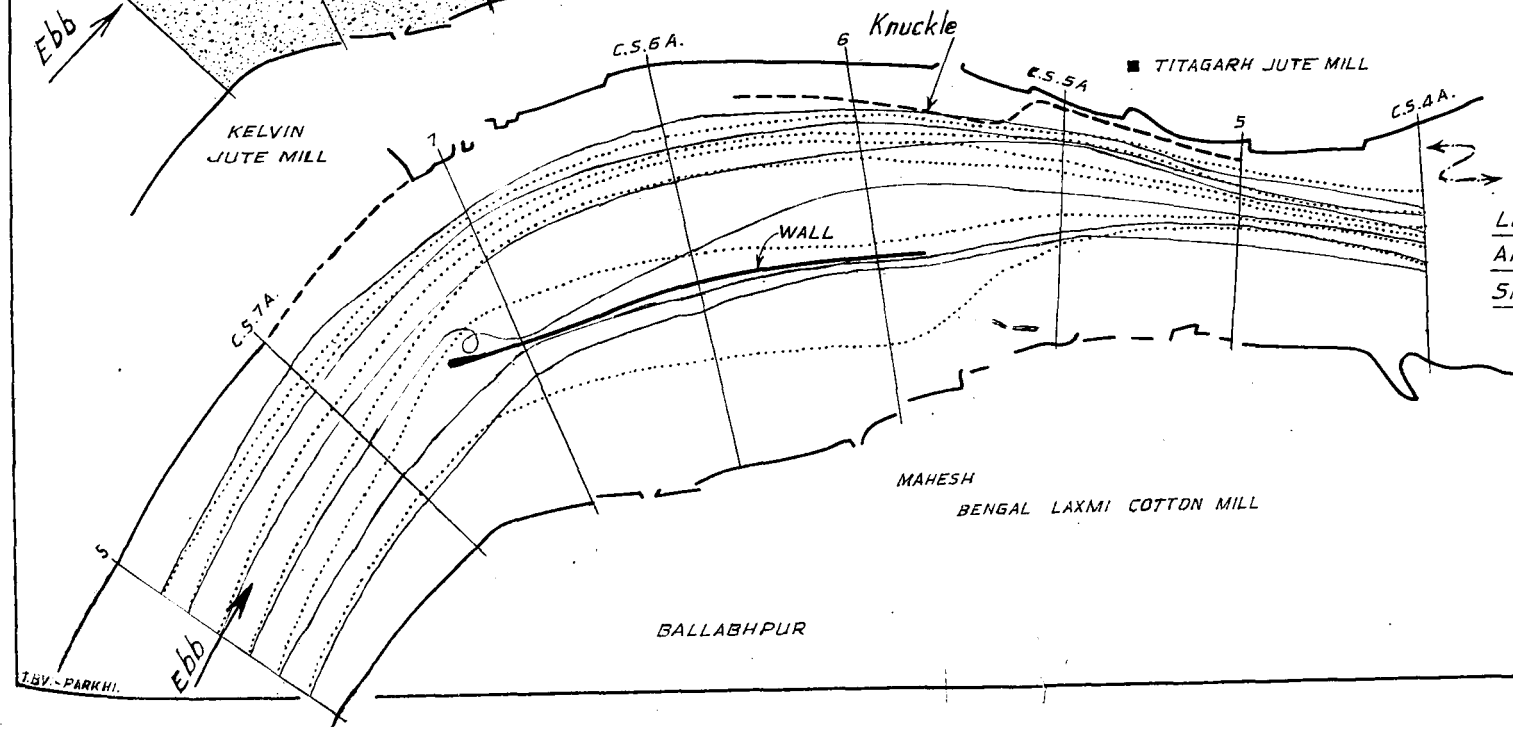


FIG. 26
EXPERIMENT 4
 LINES OF FLOW UNDER CONTINUOUS EBB AFTER 10 TIDES, WITH THE BED ON LEFT SIDE OF THE WALL RAISED BY 1" = 6'-8"

References:~
 ————— surface flow.
 Bed "

FIGS. 27 & 28/C-2(VI)-1

$\frac{1}{150} : \frac{1}{80}$ SCALE
HOOGHLY RIVER MODEL

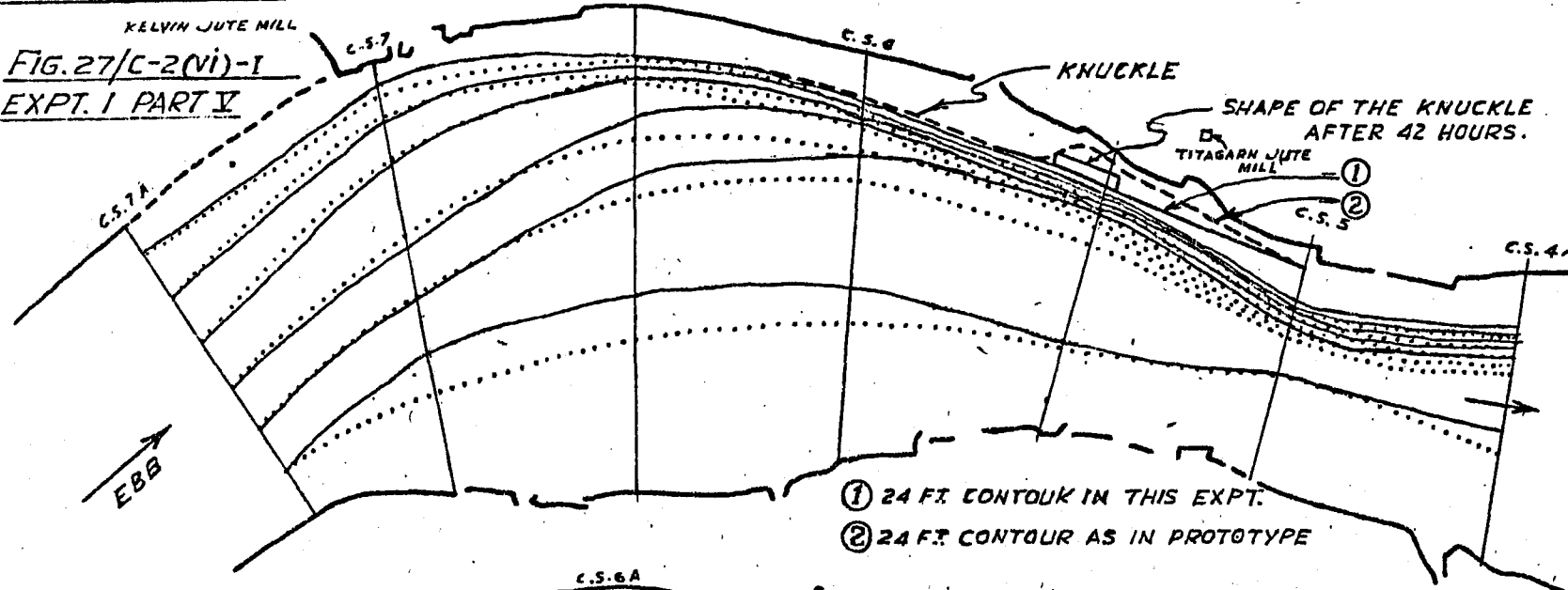
SHOWING LINES OF FLOW
AFTER 42 HOURS OF
CONTINUOUS EBB
Q = 1.7 \approx 187,000 CS.
W. L. AT KONNAGAR
 \approx 13.5 FT. ABOVE K.O.D.S.

SCALE $1'' = 4' \approx 1200'$

REFERENCES:—

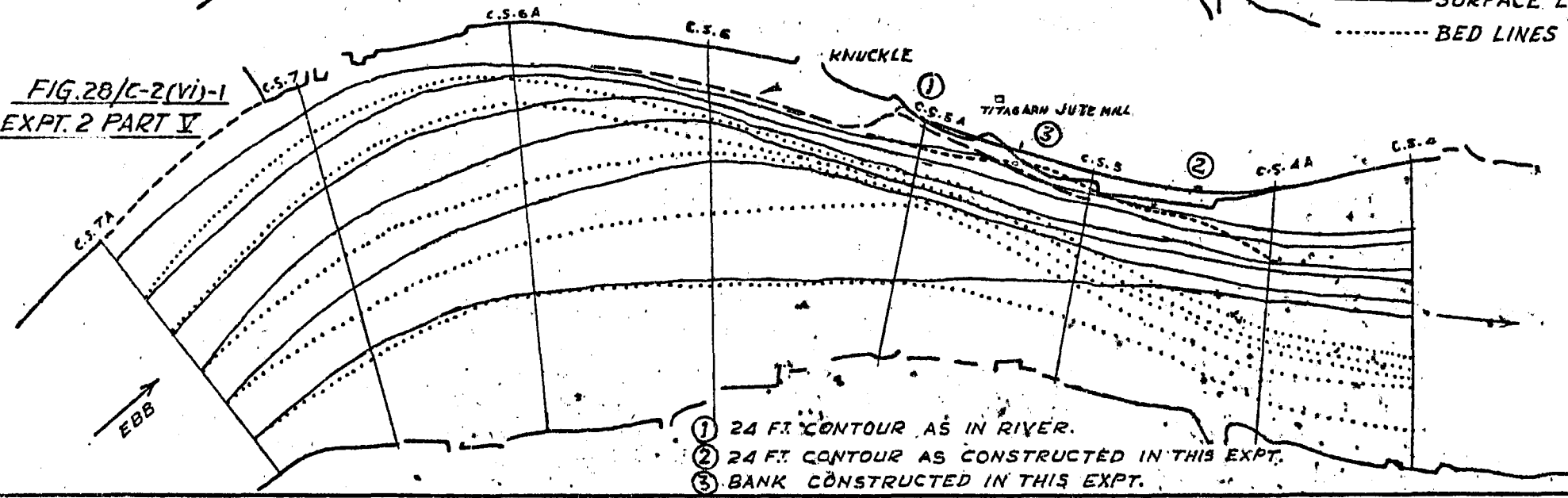
———— SURFACE LINES OF FLOW.
..... BED LINES OF FLOW.

KELVIN JUTE MILL
FIG. 27/C-2(VI)-1
EXPT. 1 PART V



- ① 24 FT. CONTOUR IN THIS EXPT.
- ② 24 FT. CONTOUR AS IN PROTOTYPE

FIG. 28/C-2(VI)-1
EXPT. 2 PART V



- ① 24 FT. CONTOUR AS IN RIVER.
- ② 24 FT. CONTOUR AS CONSTRUCTED IN THIS EXPT.
- ③ BANK CONSTRUCTED IN THIS EXPT.

C-HYDRAULIC WORKS.

2. River Control

(vi) Tidal action including that in estuaries.

C-2 (vi)-2 : Preliminary Note on the Cochin Harbour Model.

1. 0. Introduction:—

1. 1. One of the strong reasons advanced in the early days (in 1880) for opening up the Cochin Harbour was that dredging maintenance would be light. Being a natural harbour with a copious back water (extending over 100 miles N - S in length, $1\frac{1}{2}$ to 15 miles broad, and of great depth) into which some considerable rivers from the distant Travancore and Cochin hill debouch, it was considered that the silt would be wholly deposited in the back-water reservoir so that, even in the height of the monsoon, the water flowing out of the principal passage into the sea at Cochin would be practically free of silt, precluding the formation of a bar (vide page 94 - "History of Proposals").

1. 2. Later (1912-1914) in his final report on the Cochin Harbour Investigations, Cambell remarked that "Silt observations made in the Cochin gut showed that more silt was brought into the harbour during the monsoon than was discharged". ("History of Proposals" page 163).

1. 3. In 1927-29, a sea approach channel to the Port, 16000 ft. long (of which 4000 ft. was on the east or landward side of the bar), and 450 ft. wide, was dredged to a depth of 37 ft. at Low Water Ordinary Spring Tide, starting from the 32 ft. contour at the western end to the same contour at the eastern end of the bar. In the inner harbour, the Mattancherry Channel, mooring areas and the Ernakulam Channel were dredged.

Fig. 29
C-2 (vi)-2

1. 4. There has been, since 1933, much greater silting. As there were two very low annual rainfalls and one low one, all in successions, since 1933, see Fig. 30, it was thought at first that this low rainfall reduced the scouring effect of the monsoon freshets, which are usually regarded as the principal force in keeping the channel clear from the mud which the combined sea forces (the swell, the flood-tide etc.) push into the channel. It has been observed, however, that the entrance channel silts up as much as 9 ft. during the monsoon, see Fig. 31. When, despite twice the normal rainfall in July 1937 there was more than average silting in the approach channel—this was, in fact, the year of maximum silting—it became necessary to discover the cause of this heavy silting so that the Port may know what to expect in the future.

Fig. 30
C-2 (vi)-2

Fig. 31
C-2 (vi)-2

1. 5. It was believed that the two most important contributory causes were (1) a known and measured movement Southward of the Narakal "mud-bank" entering the approach channel and causing inordinate silting; and (2) an observed rising of the sea bed to the North of the harbour since 1934, which may or may not be a partial swelling and lifting of the bank, or a movement to the east of the sea bed to the North-west of the approach channel.

With regard to (1), it has been noticed that though the prevailing drift for sand and shells is unquestionably to the North East and that the coast line north of Vypeen was strengthened and widened by the hard material pumped up from the channel, the indisputable evidence of the resultant movement of the Narakal mud bank southwards shows that another force operates from the North sufficient to move the mud *but not sand*. (The southern fringe of the mudbank was found, by actual survey on 20th August 1937, about a mile south of the Cochin entrance while it had been definitely north of it before).

1. 6. A special committee was, therefore, appointed in 1938, charged with the following Terms of Reference:—

- (i) What is the force or the combination of forces, which starts the Narakal mudbank moving north or south, and continues it in that movement after the original cause has apparently disappeared?

- (ii) Is there evidence enough to justify an opinion being given as to the cause of the rising of the mud bottom originating north of Cochin Harbour in 1934? Could it have been caused by the Bihar earthquake of 15 January, 1934?
- (iii) Why is there not a similar rising originating south of the harbour mouth?
- (iv) Can it be stated, from the answers to the above, and from other evidence available, what was the immediate cause of the unusual silting in the Cochin channel in July 1937?
- (v) Can any suggestion be made as to the possibility of keeping the mudbank in its present position, or near it, by a careful programme of dredging operations, maintenance or otherwise?

Various theories in explanation of the silting in the Approach Channel and in the inner harbour are discussed in the "Blue Books" by Sir Robert Bristow on the History of Malabar Mud Banks and in the Expert Committee's Report. Some observations on the nature, origin, behaviour and possible causes of mudbanks formation, as a result of the Expert inquiry, are of interest to future investigations and are extracted below:—

1. 7. *Report of the Special Committee 1938. Extracts from —*

1. *Nature of the Muds :*

Statement 1
C-2 (vi)-2

"It was suggested in some of the early records that the "off-shore" mud is of a different nature to the "on-shore" banks which produce calm water. We have no samples from a greater depth than 7 fathoms, but it will be seen from Statement I-C-2 (vi)-2 that there is no evidence of any progressive or other marked change in properties of the mud samples either from N. to S., or from the low-water line across the mud-bank to the 7 fathom contour. On the contrary, the results suggest that all the mud samples examined have a common nature and origin. These samples cover the area bounded by an E.-W. line 2½ miles N. of the harbour, the shore-line, an E.-W. line 1 mile S. of the harbour, and the 7 fathom line. Hence we are satisfied that at least to the 7 fathom line the mud of the sea bed and that of the banks is similar."

"It will be seen from Table I that the mud is a fine grained one. About three-quarters of each sample is composed of particles having an effective diameter of 0.002 millimetres or less (the so-called "clay" fraction of a mechanical analysis), the remaining particles lying between the diameter range of 0.02-0.002 millimetres (the "silt" fraction)."

"These samples consist predominantly of clay and silt fractions in the ratio by weight of 3 to 1. There is a very small amount of fine sand and a negligible amount of coarse sand."

"The sea-bed off this part of the coast is also mud; but this mud does not give the same quiescence to the water (see (3) below)."

"The composition of the mudbanks is, however, quite different from that of the back-water mud or silt and is believed to have derived from an older source."

2. *Origin of the muds.*

"Borings at Cochin show that these are deposits of alluvial material some hundreds of feet deep overlying the rocks. This can only have been brought down over a long period by the rivers and gradually deposited on the sea bed."

FIG:- 29/c-2(vi)-2

SHOWING THE MODEL REACH

SCALE:- 1" = 8000'

NOTE:-
SEA INCLUDED UP TO 60' CONTOUR
IN MODEL.

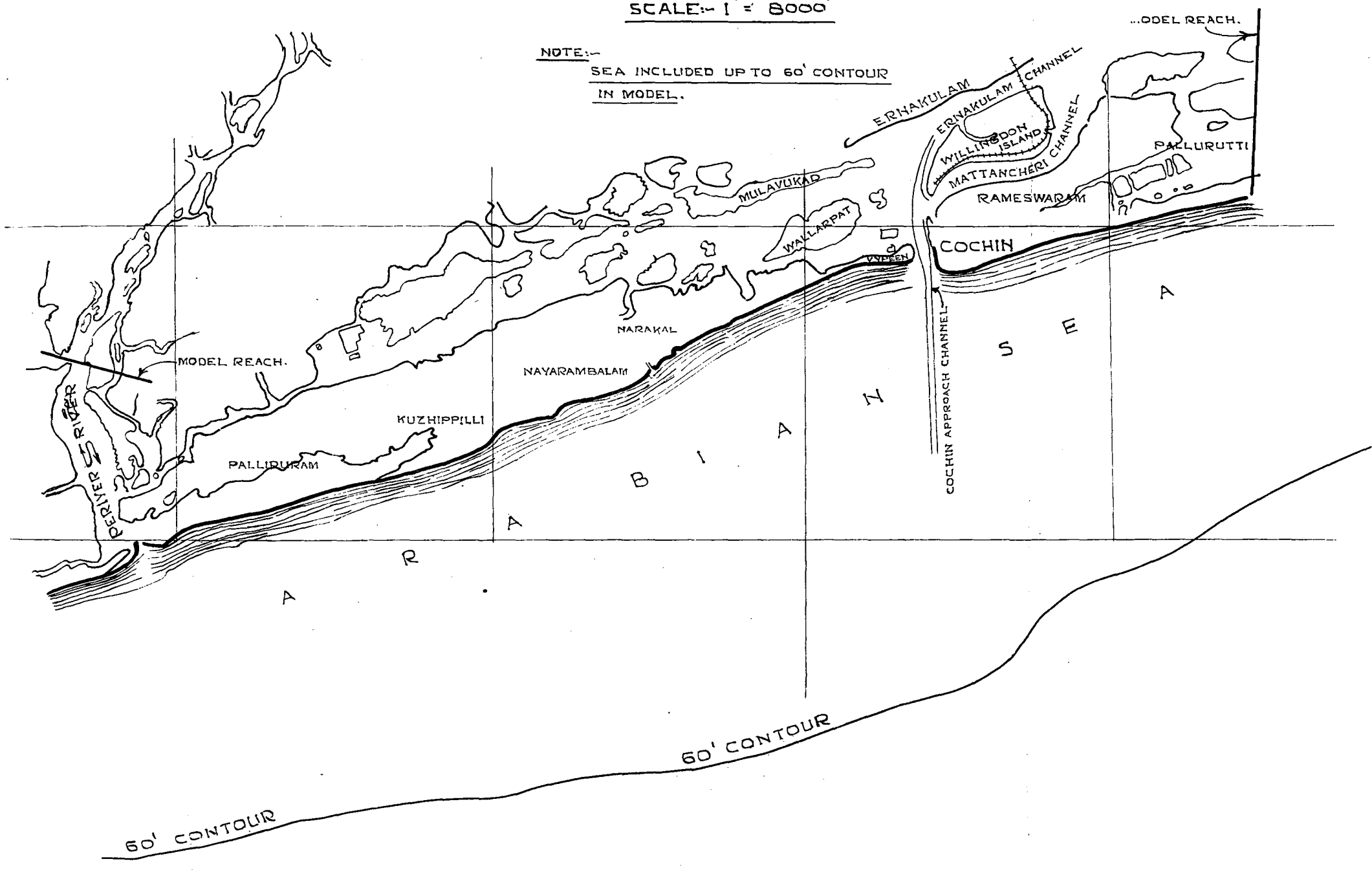
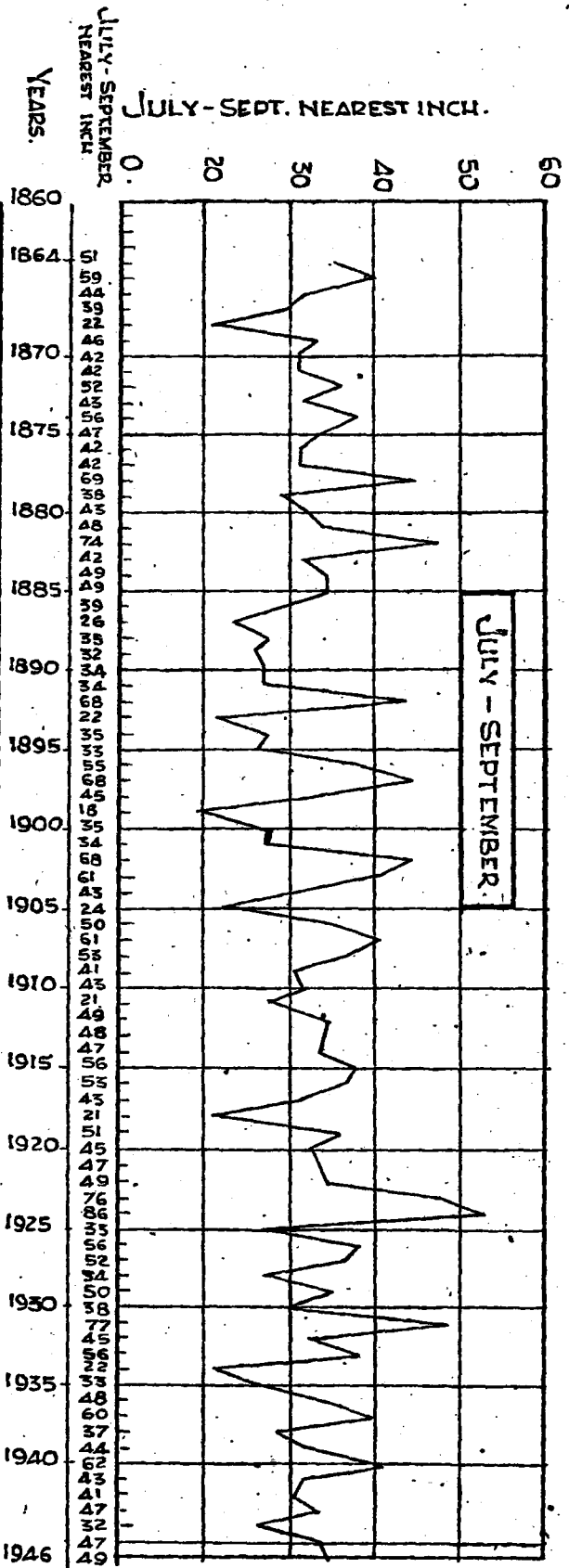
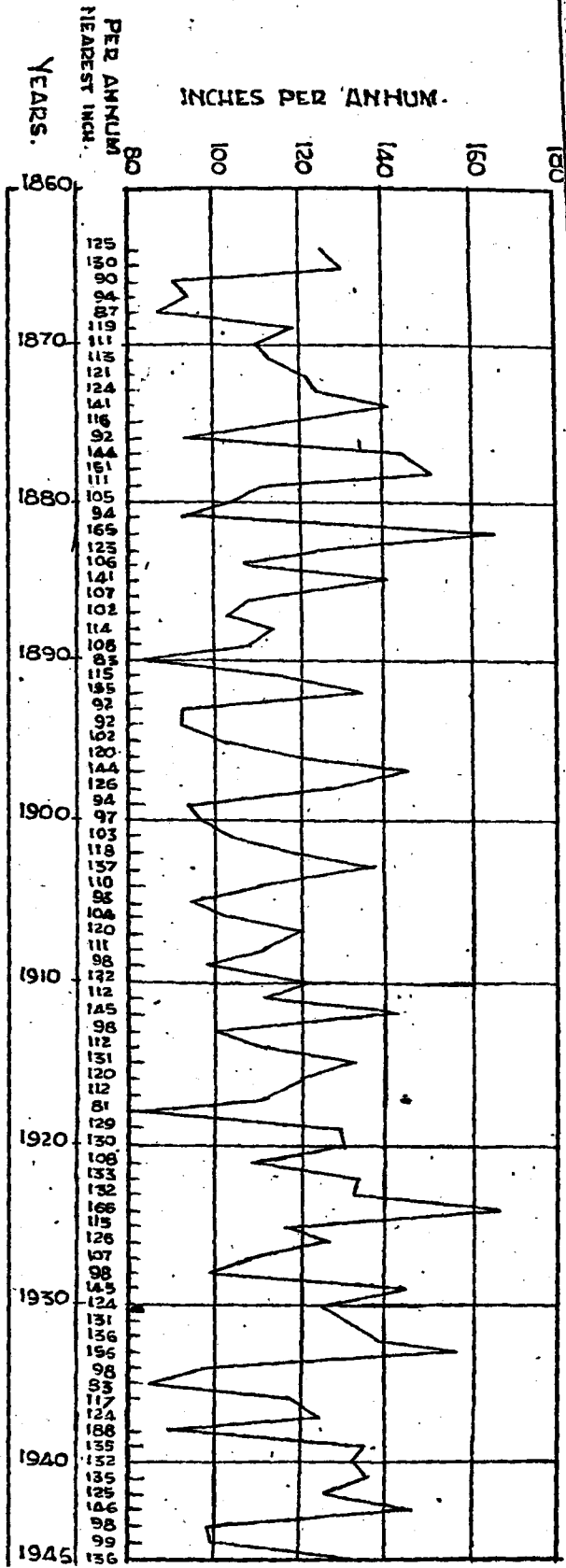


FIG. 30
RAINFALL
AT
COCHIN.
FOR THE YEARS,
1864 TO 1946,



- REF.
—— BEFORE DREDGING DECEMBER 1938.
- - - - SOUNDINGS IMMEDIATELY AFTER DREDGING
JANUARY 1939.
—— SOUNDINGS IN SEPTEMBER
▨ SILTING TO END OF SEPTEMBER

FIG. 31/C-2(VI)-2
SILTING IN THE APPROACH CHANNEL
DURING MONSOON 1939

(AVERAGE SOUNDINGS FOR EVERY 1000 FEET ARE PLOTTED.)

SCALE | HOR. 1" = 1500'
VER. 1" = 15'

SEA SIDE

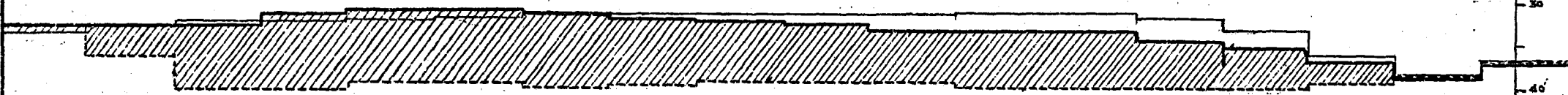
T. P.

SHORE SIDE

APPROACH CHANNEL

LOW WATER ORDINARY SPRING TIDE

0'
10'
20'
30'
40'



T. BY. PATKIE. 3/3/49.

Statement 1/C-2 (vi)-2

*Mechanical Analysis of Cochin Harbour Muds**2 1/2 Miles North of Harbour*

	Depth at which sample was taken					
	L. W.	18-Ft.	24-Ft.	30-Ft.	36-Ft.	42-Ft.
Coarse Sand	0.0	0.0	0.0	0.0	0.0	0.0
Fine Sand	1.1	3.1	3.0	3.4	1.1	3.0
Silt	22.6	22.3	25.2	23.4	25.5	26.2
Clay	74.9	71.1	68.5	70.2	70.4	68.0
Carbonates	1.4	3.5	3.3	3.0	3.0	2.8

1 Mile N. of Harbour

	Depth at which sample was taken.					
	L. W.	18-Ft.	24-Ft.	30-Ft.	36-Ft.	42-Ft.
Coarse Sand	0.1	0.5	0.3	0.4	0.1	0.2
Fine Sand	1.5	4.0	3.5	3.4	2.4	1.8
Silt	22.3	21.4	25.2	24.8	23.5	23.2
Clay	74.5	70.7	67.8	68.1	70.7	71.6
Carbonates	2.6	3.4	3.2	3.3	3.3	3.2

Taken in Channel.

	Distance from Turing Point					
	Feet East	Feet East	Feet W.	Feet W.	Feet W.	Feet W.
	4,000	1,000	2,000	5,000	8,000	11,000
Coarse Sand	0.0	4.9	0.0	0.0	0.0	0.0
Fine Sand	3.7	14.9	1.5	0.8	0.9	2.7
Silt	20.7	16.9	23.6	22.4	21.9	22.5
Clay	72.8	58.7	72.4	74.1	74.6	71.8
Carbonates	2.8	4.6	2.5	2.7	2.6	3.0

1 Mile S. of Harbour

	Depth at which sample was taken.					
	L. W.	18-Ft.	24-Ft.	30-Ft.	36-Ft.	42-Ft.
Coarse Sand	11.4	0.3	0.3	0.1	0.5	0.0
Fine Sand	44.2	2.7	13.6	4.2	4.8	6.4
Silt	10.2	24.2	21.9	23.5	21.8	22.3
Clay	29.2	69.7	60.0	68.9	68.9	67.3
Carbonates	5.0	3.1	4.2	3.3	4.0	4.0

3. Behaviour of the Mud Banks.

By the term "Mud-bank", as distinct from the general area of mud on the sea bed, is meant "those in-shore areas over all or some portion of which the special property is found to exist of reducing wave action during the S. W. Monsoons and producing areas of calm weather in the roughest weather" (There are two such areas near Cochin, one at Narakal and one at Alleppey.)

"During the 1938 monsoon special observations have been kept on the Narakal and Alleppey banks and their positions have been fixed at frequent intervals."

"The following tables show the movements of the centre of these banks and their lengths at various dates from August 1937 to September 1938."

ALLEPPEY BANK.

Date	Length of bank*	Position of centre of bank relative to Alleppey Pier	Amount of movement of centre	Direction of movement
Aug. 1937	4½ miles	3½ miles south	2½ miles	North
Nov. 1937	6 miles	1 mile "	2¾ "	South
April 1938	3½ miles	3¾ miles "	1/4 "	
July 1938	5 miles	4 " "	1¾ "	South
Sept. 1938	5 miles	5¾ " "		

NARAKAL BANK.

Date	Length of Bank*	Position of centre of bank relative to Cochin Light house.	Amount of movement of centre	Direction of movement
Aug. 1937	5½ miles	2½ miles north	1/2 mile	South
7th June 1938	5½ "	2 " "	1 "	South
23rd Sept. 1938	5¾ "	1 mile "		

As indicated by fringe of calm water.

"From the above tables it can be seen that considerable variations occur within short periods both in the area and position of these two mudbanks."

4. Possible causes of Mudbank formation.

"We have already stated our conclusion that the source of the mud is detritus brought down by the rivers and we have now to consider how the mudbanks themselves may be formed."

"As to this, we are to a considerable extent in the realms of conjecture owing to lack of evidence. We are of opinion; however, subject to this qualification, that the banks are due to the interaction of forces of a varied nature, such as currents, river discharges, tides, wave action, ground swell, etc., with occasional effects due to cyclones or exceptional storms and seismic

disturbances. The variations due to the constant interplay of these forces are probably too complicated for cause and effect to be fully traced, even after a long series of observations."

"It is probable that one cause, whether of the more persistent or temporary type of bank, is similar to that which causes ordinary shoaling in a river estuary, viz., the existence of areas where the water is nearly still, or where slow back-eddies occur due to the bed contours or the meeting and combination of currents."

"It can be readily conceived that such areas, favourable for shoal formations, exist on the Malabar coast where littoral currents come in contact with vast quantities of water pouring into and out of the rivers and backwaters, particularly in times of flood."

"It is to be noted that the areas where mudbanks occur are within a moderate distance of such rivers except at Alleppey and even here we are inclined to believe that an opening once existed. It is, in fact, shown on one of the old maps recently obtained from the British Museum (and referred to at page 57 (Appendix D) not reproduced.)"

"In the case of the Narakal bank the predominating factor is probably (in so far as ordinary shoaling is concerned) the silt brought down by the Periyar river, but further evidence is required before this can be considered conclusive."

"A further cause of accretions to the banks or the formation of temporary banks is in our opinion the action of the sea on the "off-shore" mud. It is known that sea disturbances can cause movement at considerable depths and cases have been recorded in many places of materials being transported along the sea bed at depths of 10 fathoms or more."

"When exceptionally heavy storms occur, or an unusually severe ground swell, it is reasonable to suppose that the sea bed may be disturbed at greater depths than in normal monsoon weather, similar effects would be produced either by the heavy swell caused by cyclones or by seismic waves."

"Summarizing our views on the formation of the banks, we believe, in so far as the incomplete evidence before us will permit of conclusions being drawn, that

(i) the banks are formed in two ways, acting separately or together.
viz. :—

(a) by the depositing of material directly derived from detritus brought down by the rivers;

(b) by the throwing up and redepositing of areas of similar mud from the sea-bed during rough seas;

(ii) there are processes going on, particularly during heavy monsoons and cyclones, of fretting, stirring up and redepositing of mud. These cause changes in the sizes and shapes of the mudbanks;

(iii) banks can only be formed where the resultant of all the forces acting is such as to produce conditions favourable for shoaling at the time of their formation;

(iv) where the resultant of such forces is constantly favourable to shoaling the banks will be of the more persistent type;

(v) where conditions are only temporarily favourable the banks will be more or less rapidly dissipated and may in some cases be reformed elsewhere."

5, Travel by Mudbanks.

"As can be seen from the charts in the Blue Books the banks travel considerable distances along the coast as well as undergoing changes in areas and shapes."

"The movements of the Narakal bank, so far as they can be traced, have been as follows. The most northerly position recorded was in 1775 as shown on Dalrymple's map, obtained from the British Museum, when it was just south of the Periyar river. Up to 1934 its centre had never been further south than 5 miles north of the entrance, at least as far as is known."

"For many years it was stated to be nearly stationary off Narakal and Malipuram and was indeed considered to be fairly stable between 1860 and 1870."

"Since 1934 its progress southward has been more rapid and in September 1938 the centre of the calm water nearly coincided with the line of the entrance channel, a total southward movement of over 5 miles in this period."

"We should perhaps refer here to the area of calm water which appeared off Cochin in about 1911 or 1912. This bank was non-existent, as we are informed by Capt. Leverett, a retired Port Officer of Cochin, in 1910 when he left Cochin and also in 1913 when he returned there. It appears possible therefore that this was a temporary bank possibly thrown up as a result of the cyclone of 1911 or that of 1912 (which passed actually over Cochin) and dissipated shortly afterwards. We do not necessarily regard it as evidence of the Narakal bank having been south of the entrance channel at that time."

"We think it probable that the changes of size and shape of this bank from time to time have been influenced, as suggested in the case of the Alleppey bank, by mud thrown up in storms, cyclones, etc., and also by the deposit of silt brought down by the Periyar river—possibly also by silt from the Cochin backwater."

"We believe, however, that its gradual creep southwards which seems to have been, on balance, continuous since about 1920 has been more directly influenced by the currents. Evidence as to the littoral currents as distinct from the tidal flow near the shore is not available, but it is noteworthy that in the last few years increased current from north to south has been reported by ship masters on many occasions and this increase would seem to coincide with the more rapid southward advance of the bank in recent years."

"We have plotted on the plan attached hereto as Appendix F (not reproduced) the sections along the centre of the approach channel at the end of May for each year since 1933 when the channel was first deepened to about 40 feet. These sections (which are plotted to a greatly exaggerated scale) show the silting which has taken place in the different years between the time of dredging the channel and the break of the monsoon. They therefore show the silting during the fair weather period when there is little rough sea. They indicate clearly that there was a movement of mud during the months January to May and that the amount of the pre-monsoon silting increased from year to year as the bank advanced south and the channel became closer and closer under its lee. They further indicate that as the bank actually crossed the channel the yearly increase became less and has now probably about reached its maximum."

"We may summarize our tentative opinion as to the causes of the southward movement of the banks as follows. We think it is due:

- (a) To soft mud from the sea bed being moved or brought into suspension by the action of waves or ground swell.
- (b) To the action of the currents in carrying or rolling this material and depositing it, or part of it eventually on the lee side of the banks.
- (c) To the action of the currents in carrying or rolling fresh detritus or silt derived from other sources and depositing it similarly on the banks.
- (d) To the preponderance of the currents being from north to south thus causing the accretion, on balance, to occur on the south side."

6. *Calming effect of the Mud Banks.*

After dismissing the theories that the presence of oil and the elastic nature of the mud are responsible for the reduction of waves and swell, Dr. Keen, D.Sc., F. R. S., explains that the calming effect on wave-motions is due principally to the mud-particles in suspension in the water.

"It is known from hydrodynamics that waves will die away more rapidly if by any means the ratio of the viscosity of the medium to its density is increased. The smaller the size of particles in suspension the more likely is this ratio to increase. Appendix C (not reproduced) gives results to show that the Narakal mud brought into suspension by the forces of wave motion or other means will produce a reduction in the amplitude of the incoming waves. But the experimental work discloses the further significant fact that the effect is a cumulative one because, as the shearing stresses within the suspension become reduced, the suspension itself behaves somewhat like a jelly and the remaining stresses are rapidly dissipated."

"Thus the transition belt is the seat of the reduction of incoming wave motion because it is here that the waves themselves bring the mud into suspension."

7. *Travel of Sand.*

"It has been proved beyond doubt that the sand on the Cochin-fore-shore travels in a northerly direction."

"This has been demonstrated by Mr. Bristow's experiments south of the harbour entrance where broken brick of various sizes has been spread on the foreshore and observed. Its travel has been invariably northward."

"It has been noticed that after the monsoon a patch of sand appears on the north side of the entrance channel near the shore. The position of this sand has been marked with stakes and it has been observed to move northward."

"Further, when the channel was first dredged, a large quantity of sand was pumped to the north of the channel. This sand which was at that time well south of the Narakal mudbank travelled northward and shoreward forming an appreciable accretion south of the parallel groynes."

"This northward travel of sand introduces an apparent anomaly in that it is directly opposed to the travel of the mudbanks which is southward. There is, however, in our opinion no difficulty in reconciling these two facts. We have already stated that we believe the southward movement of the bank is due to the preponderance of current from the north, but such currents are not the predominant cause of the movement of the sand. The sand travel has taken place and has been observed in the shallow water close in shore where it is subjected to direct wave action; and it is the breaking of waves or swell from the south-west which carries the sand northward."

"Undoubtedly mud will be carried northward too by this action, but ultimately it will be carried into the sphere of influence of the currents from the north and will thus be carried southward again."

8. *Cochin Entrance Channel.*

"In 1929 and 1930 it was dredged to about 36 feet for a length of 16,000 feet and in 1931 and 1932 to about 36½ feet below low water. In 1933 and 1934 the dredging was carried to a substantially greater depth, viz. about 39½ feet and in 1936 and 1937, the depth remaining after dredging was 38½ feet. From 1933 onwards it has, therefore, been dredged about 3 feet

deeper than in any of the earlier years. The width of channel after dredging has been 450 feet from 1929 onwards. There has been some natural falling in of the sides after dredging and in all cases where depths are referred to from May to October they apply to the central third of the channel which is ordinarily used by vessels going to and from the harbour. "

" 'Silting shelves' were dredged in the neighbourhood of the old bar to about 30 feet at low water on the north & south sides of the channel in 1927, 1928 and 1929. The object of these was to assist in keeping the sand and silt from the channel by allowing as much of it as possible to settle on the shelves."

(i) *Pre-monsoon Silting :*

" In 1933 there was little pre-monsoon silting west of the bar but some 2 feet on the east."

" In 1934 the silting increased west of the bar, varying from zero at the outer end to a maximum of 2' 9". There was little silting east of the bar in 1935. The results were little different to those of 1934, except that the silt extended further eastward."

" In 1936 the pre-monsoon silting was on the whole heavier than in any year from 1933-1938."

" In 1937 and 1938, between which there is little difference, there has been slightly less silting to the west of the bar than in 1936, but markedly less to the east."

" In the period January to May inclusive there are comparatively few heavy seas or other causes of disturbance and the rivers are not in flood. It is reasonable therefore to infer that changes in siltation during these years have been directly influenced by the approach of the mudbank to the channel. The maximum effect is shown in 1936 both east and west of the bar, since when there has been a tendency to decrease. The bank was known to lie across the channel in 1937, and was probably close to it in 1936 and it is, we believe, fair to conclude that no greater pre-monsoon silting is likely to be caused by the presence of the bank across the channel than has occurred in the past."

(ii) *Monsoon Silting :*

" During the monsoon period, many other factors come into play besides the natural tendency (which has been shown to exist during recent years) of the bank to creep southwards. As previously indicated there must be large quantities of mud not only brought down by the river floods but brought into suspension or otherwise moved by sea-action on the fringes of the bank and on the sea bed."

" Study of the section showing the depths of water indicates that the amount of silting in the channel during the monsoon months varies not only from year to year but from month to month in any one year. For instance since the channel was completed (in March 1929) the maximum depth of silting west of the bar occurred in June in 1929, in August in 1930, in September in 1931 and in July in 1934. Similar variations appear in the channel east of the bar."

" There appears, however, to be no direct link between the amount of rainfall in any month with the amount of silting in that month. Nevertheless the rainfall must have some effect over a whole season since a high rainfall must increase the outflow from the Cochin Gut and increase the period and speed of the ebb tide, thus providing greater scouring power. At the same time the force of the current diminishes as the distance westward from the Gut increases and its value as a scouring agent becomes less. It is therefore probable that the main factor in causing siltation is the stirring up of mud during the monsoon and its transfer by currents and flood tides into the channel."

“Another factor which has not yet been fully explored, but which may have a greater influence than has up to now been appreciated is the silt brought down by the Periyar river in times of heavy rainfall travelling down the coast with the current. It would be desirable to make some further observations in connection with this river, the catchment area of which is stated to be some 5,000 square miles, although the extent to which the run-off has been affected by the construction of the Periyar dam is not known.”

“Another feature which has been noticed in each year from 1933 onwards is the constant tendency of the silt to extend further east of the bar towards the Gut month by month. It is open to doubt whether this result is produced wholly by the transfer of silt from the sea further into the channel or in part by the deposition of silt carried out from the backwaters on the ebb tide. It has been established that there is considerable movement of silt in the Gut particularly near the bottom, but further observations are required before any definite conclusions can be drawn as to the extent of siltation derived from the backwaters.”

(iii) *Silting at the Gut.*

“It is a well established fact that silt, which remains in suspension in fresh water may be rapidly deposited when it comes in contact with salt water. Experiments made indicated that in a solution of common salt of concentration not exceeding 0.25 percent (Sp. Gr. 1.002) the mud does not settle; complete and rapid settling occurs when the concentration of salt is 2.0 percent or greater (Specific Gr. 1.013); at intermediate concentrations the bulk of the mud settles out but the rest remains indefinitely in suspension.”

“We have also had some tests made of the salinity of the water in the channel and these show that from the Gut to about the bar the lighter fresh water from the backwaters tends to flow over the heavier salt water. In view of this fact and the experiments referred to in the preceding paragraph it is not improbable that the siltation east of, and in the neighbourhood of, the bar may be due in part to the fresh water intermingling with the salt and thus causing its silt to be coagulated and deposited on the bottom and the limits of this action will depend on the volume of fresh water flowing out of the Gut.”

“The sections show that the year of *greatest silting during the monsoon* period was 1937. Most of the silting occurred in June and July, the additions in August and September being small. The greatest deposit was in July, west of the bar and in June east of the bar. It was in August 1937 that the fringe of the calm water was first seen to the south of the entrance channel and a survey was carried out by Mr. Bristow on 20th August, the result of which is shown on the plan at page 31 Vol. II of the Blue Books.”

“We have no doubt whatever that the exceptional silting in June and July was caused by the disturbed belt of mud surrounding the calm area crossing the entrance channel during these months.”

“Apart from the movement of the mudbank the variations from year to year are, we believe, caused by such varying circumstances as the rainfall, the strength of the monsoons and storms affecting the amount of mud in suspension and the area over which it is deposited.”

“The amount of silting in the entrance channel has varied from a minimum of 650,000 cubic yards in 1931 and 1932 to a maximum of 2,370,000 cubic yards in 1937. The quantity of silt rose abruptly from the low level of 650,000 in 1932 to over 1,500,000 in 1933 and from 1934 to 1936 was in the neighbourhood of 2,000,000 cubic yards. This was followed by the maximum figure of 1937. In 1938 the silting has amounted to 1,700,000 cubic yards.”

“These actual figures afford a further confirmation of what has already been said as to the effect on the channel of the southward travel of the bank.”

1. 8. Appendix I gives, in *extenso*, Sir Robert Bristow's appreciation of the summary of the Report and Evidence on the Movement of Mud Banks. Appendix I
C-2 (vi)-2

1. 9. The present position in respect of silting and maintenance dredging is best described in the words of the present Administrative Officer, Cochin Port, Mr. Milne, C. I. E., M. I. C. E.,:—

"The Mattencherry channel was dredged to a width of 1000 ft. and a turning basin opposite the wharf to a width of 1400 ft. The width of the turning basin was reduced about four years ago to 1200 ft. as siltation on the western side was very rapid, necessitating dredging three times a year. The silting which takes place in the approach channel amounts to between $1\frac{1}{2}$ and 2 millions c. yds. per annum and that in the inner harbour is about 1½ millions c. yds.

The maintenance dredging of the approach channel is done by the Suction Dredger "Lord Willingdon" and pipeline in a period of about 4 weeks, in January-February, the most favourable months for pipe-line dredging in the sea. The silt which is *fine mud and clay* is pumped through a pipeline 3000 ft. long and deposited in the sea at a distance of approximately 2000 ft. south of the approach channel. It costs about Rs. 2 lakhs a year to do this dredging.

The maintenance dredging in the inner channel is done by the bucket dredger "Lady Willingdon" and three attendant hopper barges. The silt dealt with here also is *fine mud and clay* which takes a very long time to settle in the hoppers. Weights of three samples taken from the Mattencherry channel when the design of the hopper barges was being considered were 73.8 lbs., 75.5 lbs. and 83.5 lbs. per c. ft. In order to obtain as concentrated a hopper load as possible in depositing the spoil at sea the hopper doors are not dropped as is the usual practice. The hopper doors are kept sealed and the material is pumped out of the hoppers. The dumping ground at sea is about 2 miles south of the approach channel and about 2 miles west of the shore where the spoil is deposited in 30 ft. depth of water. It takes about 8 months in the year to do this maintenance dredging and the cost is about 8 lakhs of rupees. The 10 lakhs of rupees spent annually on inner and outer channel maintenance dredging absorbs an unduly large proportion of the total revenue of the Port which is approximately 36 lakhs of rupees.

The areas in the inner harbour where regular maintenance dredging is required are :—

- (a) A width of 250 ft. at the wharf frontage where dredging is necessary at least twice a year and often three times.
- (b) The western half of the turning basin necessitating dredging twice a year.
- (c) The rest of the area forming the turning basin which has to be dredged at least once a year.
- (d) The mooring areas on the western side of the Mattencherry channel where dredging is necessary at least once a year.

The silting takes place very largely in the monsoon months and although it remains to be proved conclusively, evidence and observation strongly favours the conclusion that the silt comes mainly if not wholly from outside the harbour.

Silt observations made in recent years confirm this.

There can be little doubt that the fine silt in the sea bed around the approaches to the Port being easily stirred up and set in motion in the monsoon conditions is carried in suspension on the flood tide inside the harbour and deposited in quantity. It also seems a reasonable surmise that the bar which existed before the Port was opened up excluded a large proportion of the silt which finds easy entrance now."

2. 0. In March 1947, the Director and Mr. Oag, Director Navigation, Central Waterways Irrigation & Navigation Commission, paid a visit to the Cochin Harbour, at the request of the Administrative Officer, Cochin, for preliminary investigation and made certain immediate suggestions with a view to reduce the cost of maintenance dredging and ameliorating the conditions, pending further recommendations as may result from model studies—see Appx. II. These recommendations are based on the diagnosis that the Matancherry

channel, being on the *convex* side of the entrance channel, is the natural intake and repository for inborne silt on the flood tide,—silt by as much as 5 ft. depth during a year, see Fig 32—particularly when it is dredged so much in excess of the critical section as to preclude it being a self-scouring agency. A reduction in the width of the dredged channel—not impracticable from point of navigation—was, therefore, recommended. It was also suggested that from model experiments it might be possible to suggest a realignment and resectioning of the Approach Channel, which would minimize dredging, both in it as well as in the inner harbour.

Fig 32
C-2 (vi)-2

2.1. The Administrative Officer has explained why such a great width was provided to the Matancherry Channel in the first instance : —

“The original proposal for the Mattancherry Channel was a channel 600 to 800 feet wide with fore and aft moorings and it was realised from the start that the increased width dredged to give the required space for the centre moorings or “swinging berths” which were provided in the first instance mitigated against and reduced the possibility of a good channel from the maintenance point of view. In a recent article on Cochin Port in the “Dock and Harbour Authority” Sir Robert Bristow discusses the reasons for the change from the original proposal and comments as follows : —

‘There falls into this period the design and laying of the cargo ships’ moorings which, of course, were put down immediately the inside channels had been prepared. I select this as illustrating a problem in which a Dock and Harbour Authority has sometimes to choose between conflicting expert advice. In our first designs we had shown a series of fore and aft moorings alongside a channel 600 to 800 feet wide, and a few “swinging berths” (one buoy) in a part of the harbour where existing depths over a large area permitted their use. These swinging berths were found so convenient to the ships and pilots that when the time came to put down the fore and aft berths, the Port Officer objected on the ground that the ships after lockers contained no chains, and it would become necessary to drag them aft along the deck out of the fore lockers. When asked why wire ropes could not be used he said it would not be safe and as he was responsible he would not agree to it. The fact that the “Lord Willingdon” used wires on all her moorings which were daily exposed to stresses greatly in excess of any that would occur in an aft mooring, whether on the ebb or flood made no appeal to the Port Officer whose views were upheld by the then Presidency Port Officer and who, of course, was directly responsible to the Government. Now the provision of swinging berths for ordinary purposes meant having a channel 1000 feet wide and the reservation in the channel of 1000 feet space fore and aft for each mooring; and this, in turn, apart from the expense of dredging, meant widening the channel beyond its critical width as a self scouring or partly self scouring agency. However, knowing that the Madras Government were bound to accept the view of the navigational authority, we decided not to force the issue and against our better judgment consented to the arrangement for the time being. Later, of course, the pilots found out for themselves that it was better in every way not to have a row of ships swinging in all directions when the tide turned, and they had to take a ship in or out, and were only too pleased to adopt the so called “dangerous” practice of mooring fore and aft with chain and wire ropes respectively.’

Sir Robert does not mention the fact that it has to be remembered of course that in those early days the Port had no powerful tug to assist in the handling of ships. The tug “Cochin” was acquired later.”

Incidentally, the moorings on the western side of the channel have *now* been moved 200 feet *eastwards*, reducing the width of the navigation channel to 800 ft. and the width of the turning basin has been reduced to 1000 ft.

3.0. *The object of the model investigations.*

The model is mainly required to answer the following questions, suggested in the Administrative Officer's letter no. 3427/47-1 dated 3rd Oct. 1947:-

(I) Is the assumption correct that the main source of silting in the inner channels and particularly in the Mattancherry Channel is fine silt carried into the harbour on flood tides and deposited?

(II) What steps can be suggested to reduce the volume of silting? In this connection it is appreciated that the width of the Mattancherry Channel is too great to encourage a good scouring action.

(III) The model experiments are required to show whether the required depth can be maintained alongside and northwards of the wharves in the Mattancherry Channel (where it is the intention to extend the wharf facilities) without frequent dredging and what alignment and width of channel is necessary to ensure this and to give a satisfactory channel generally from the point of view of maintenance.

(IV) It requires to be confirmed that the present site (south of the approach channel) where the spoil arising from the dredging is pumped and deposited is the best site to deposit the arisings and also as a dumping ground for the hopper barges from the point of view of the possibility of the material being carried into the inner harbour.

(V) If it is established that the silting comes from outside, would a realignment of the outer approach channel have any effect in reducing the amount of silt carried into the inner harbour?

(VI) Dredging is to be done shortly at the southern end of the Ernakulam Channel and the tankers berth proposed to be sited as shown on Fig. 34. It is required to know whether these berths are sited to the best advantage and whether the tide will run true in this position.

4. *Model Scales :*

4.1. When selecting various scales of a model it is necessary to see that the flow in the model is fully turbulent, and there is sufficient and similar bed movement in the model. The scales of the model are interdependent and their implications on various factors will now be considered.

4.2. On considerations of the available space, suitable reproduction of all details and facility of observation in various reaches etc., a length scale of 1/1000 was adopted for the pilot model. From the data of Station No. 2 in the Mattancherry channel—where silting is known to be occurring—the values of Reynold's Number obtained with alternative proposals of vertical exaggerations, are given in Table No. 1 :

$$\text{Mean depth } \frac{A}{w} = \frac{36960}{1488} = 24.84 \text{ ft.}$$

Lowest maximum discharge observed on 1-3-1940 = 14,881 cusecs.

FIG. $\frac{32}{C-2(VI)-2}$

SILTING IN MATTANCHERRY
CHANNEL

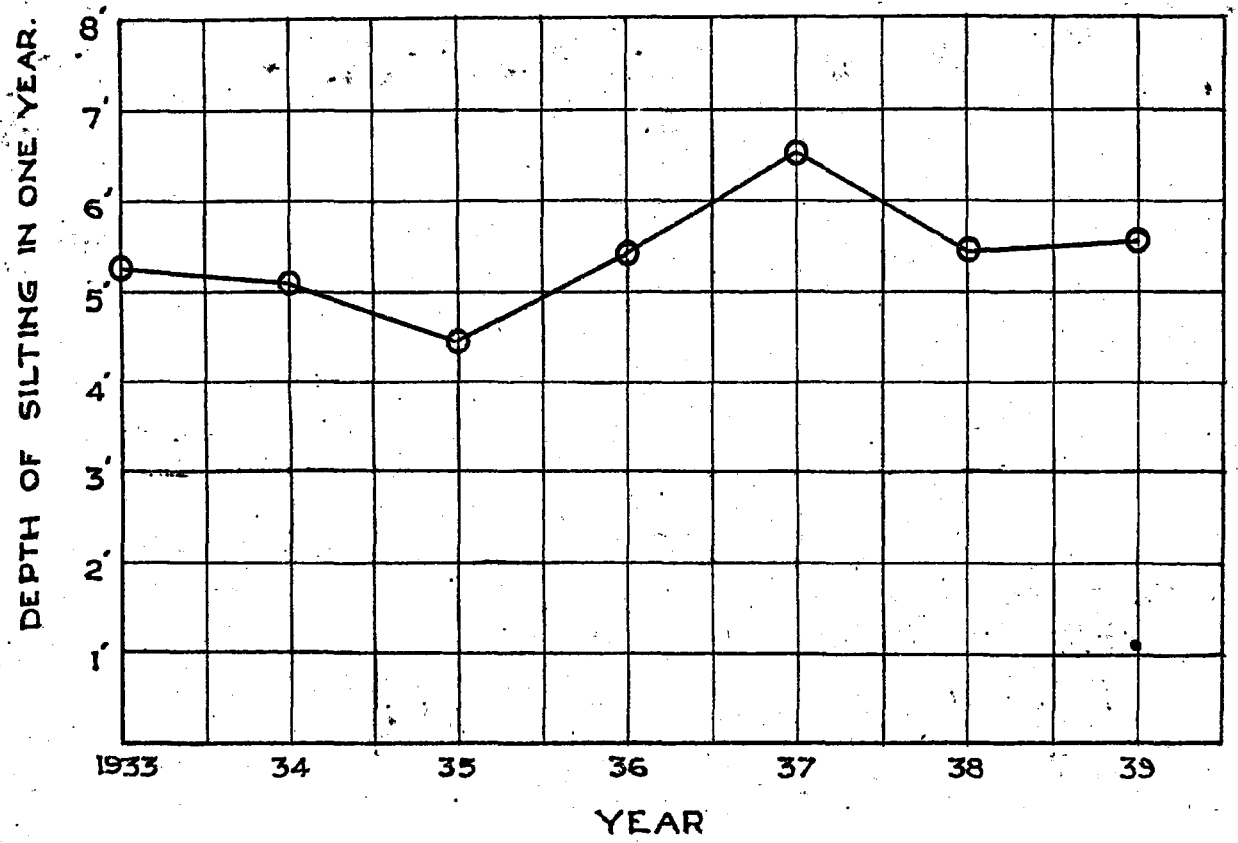


FIG. 33/C-2(VI)-2
COCHIN PORT
CONTOURS & MUDBANKS

SCALE: 4000 = 1"

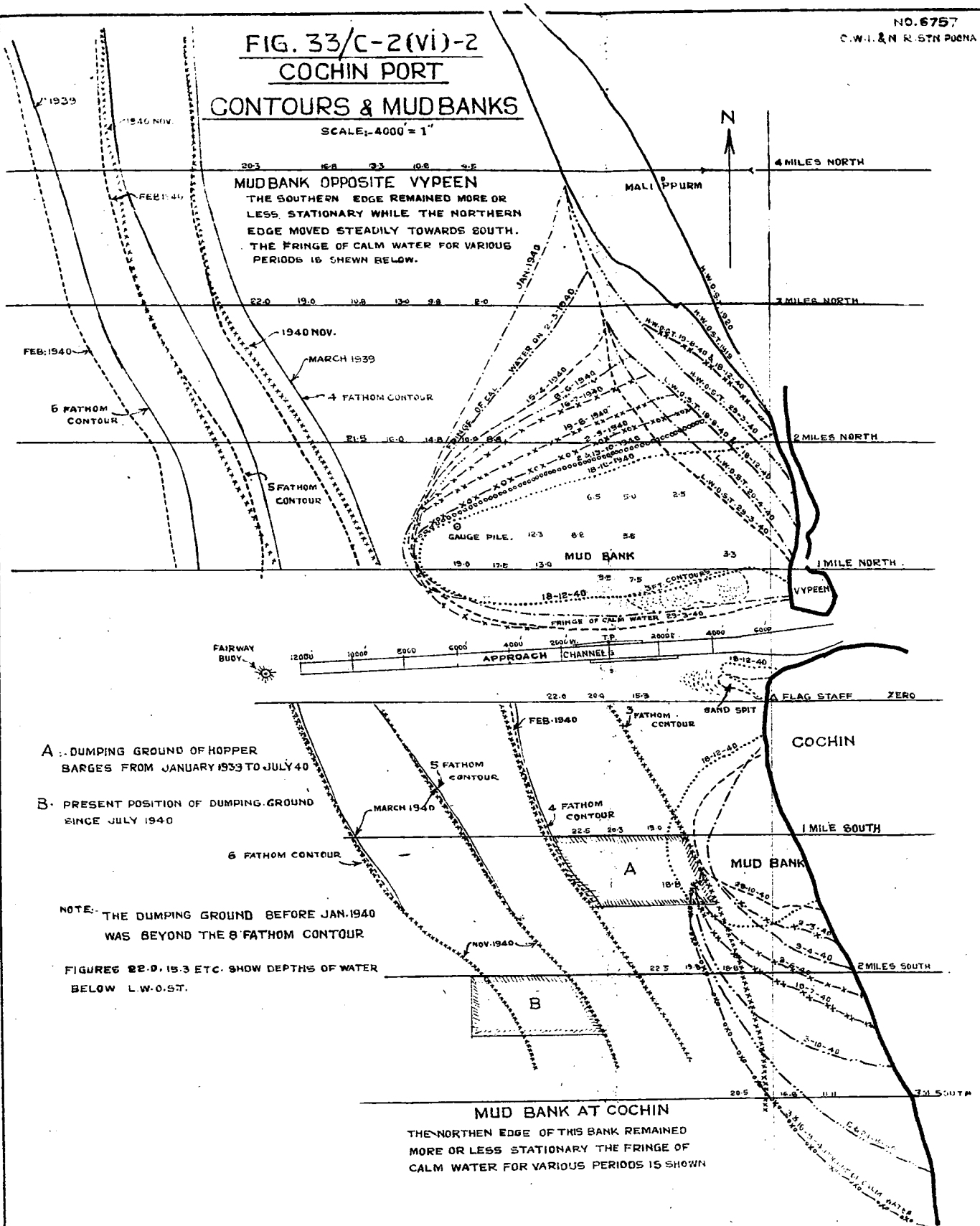


FIG. 34/C-2(VI)-2

COCHIN PORT

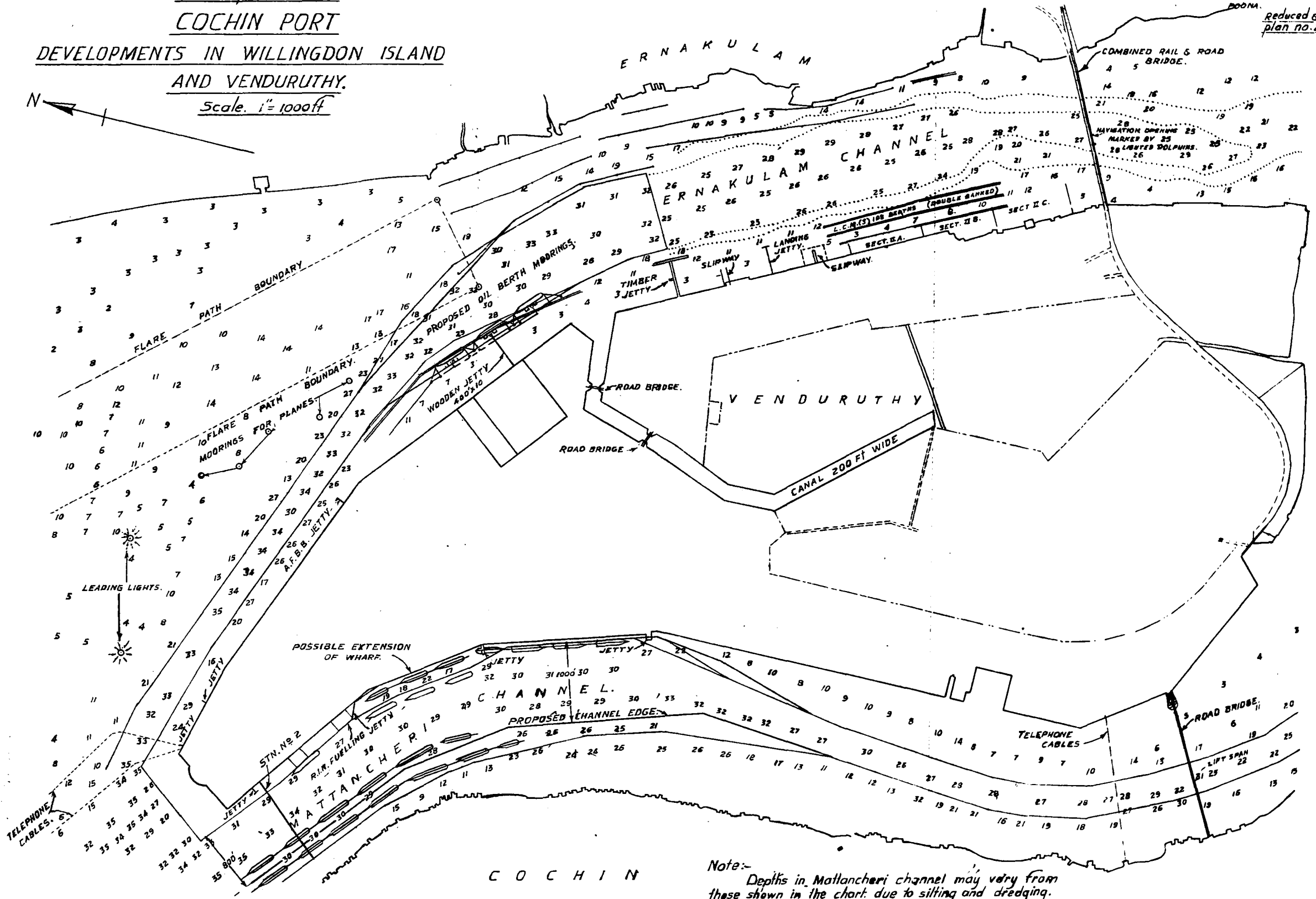
DEVELOPMENTS IN WILLINGDON ISLAND

AND VENDURUTHY.

Scale. 1"=1000ft

NO. 6750
CENTRAL W. IRRIGATION & NAVIGATION
RESEARCH STN.
POONA.

Reduced & traced
plan no. 2873.



Note:-
Depths in Mattancheri channel may vary from
those shown in the chart due to silting and dredging.

Table No. 1 / C-2 (vi)-2.

V. E.	Depth Scale	Mean Depth (ft.)	Mean Width (ft.)	Q_m cusecs	$q_m = Vd$ cusecs/ft.	Reynold's Number
1	2	3	4	5	6	7
100	10	2.484	1.488	0.47	0.316	30,000
50	20	1.242	1.488	0.17	0.115	10,800
40	25	0.993	1.488	0.12	0.08	7,600
20	50	0.496	1.488	0.042	0.028	2,650

Note : q_m in col. 6 is the discharge per ft. run in the model. Col. 7 (Reynold's number) = $\frac{Vd^2}{\nu} = \frac{q}{\nu}$ was evaluated assuming kinematic viscosity $\nu = 1.059 \times 10^5$. From Table 1 it is clear that a minimum V. E. of 40 is required to have a sufficiently high Reynold's number. It was, therefore, decided to investigate further the alternative merits of V. E. of 40 and 50 in respect of their effects on other scales, bed movement etc., the depth scales with these exaggerations being, in both cases, reasonably large enough to simulate the prevailing tidal range which is 3 ft. only.

4.4. To ensure turbulence, Reynold's criterion requires $h^3 e \sqrt{\frac{H}{30}}$ to be greater than 0.09, where h is the spring tidal range in ft. at the mouth of the model, H is the spring tidal range at the mouth of the prototype = 3 ft., and e is the vertical exaggeration.

Thus, with V. E. = 40

$$h^3 e \sqrt{\frac{H}{30}} = \left(\frac{3}{20}\right)^3 \times 40 \sqrt{\frac{3}{30}} = 0.021 \text{ which is less than } 0.09,$$

and with V. E. = 50

$$h^3 e \sqrt{\frac{H}{30}} = \left(\frac{3}{20}\right)^3 \times 50 \sqrt{\frac{3}{30}} = 0.052 \text{ which is also less than } 0.09.$$

It will thus be seen that, even with V. E. = 50, though the Reynold's number is much > 1800, the value of $h^3 e \sqrt{\frac{H}{30}}$ is still less than 0.09. However, Jack Allen⁽¹⁾ has confirmed the Station experience that "it is highly possible to succeed with much lower values than 0.09 in the average case of the irregular and unsymmetrical estuaries found in Nature."

Table 2 shows the values of $h^3 e \sqrt{\frac{H}{30}}$ obtained in various model investigations : —

(1) 'Scale models in Hydraulic Engineering' by J. Allen 1947. (Publishers: Longmans, Greens & Co.) page 136.

Table No. 2 / C - 2 (vi) - 2.

S. No.	Name of Model	Horizontal scale x	Vertical scale y	Vertical exaggeration e	Spring tidal range in prototype	$h^3 e \sqrt{\frac{H}{30}}$
1	(a) Reynold's own model of the Mersey.	31800	960	33	30'	0.001
	(b) Second model of Mersey	10560	396	26.7		0.0125
2	Thames Model by Mr. G. E. W. Grutwell	10560	384	27.5	14' 6"	0.00103
3	Seine model by Vernon Harcourt	40000	400	100	23' 7"	0.018
4	Severn model by Prof. A. H. Gibson	8500	100	85	35'	3.8
		8500	200	47.5		0.27
5	Bombay Harbour by Mc. Clure	7296	96	73	14'	0.162
6	Rangoon Model	8060	192	42	16'	0.017

Except in the case of the Severn and the Bombay Harbour models, it will be observed, the value of $h^3 e \sqrt{\frac{H}{30}}$ was less than 0.09: but the results of all these models are stated to be quite satisfactory.

It was, therefore, decided to test the effect of adopting a V. E. of 50 on other factors.

4.5. With V. E. = 50, other scales work out to be as follows -

$$\text{Depth Scale} = \frac{1}{20}$$

$$\text{Discharge Scale} = \frac{1}{1000 \times 20 \times \sqrt{20}}$$

$$= \frac{1}{89500}$$

$$\begin{aligned} \text{Time Scale} &= \frac{L - \text{Scale}}{V - \text{Scale}} \\ &= \frac{L - \text{Scale}}{\sqrt{D} \text{Scale}} \\ &= \frac{1}{\frac{1000}{\sqrt{20}}} \\ &= \frac{1}{223} \end{aligned}$$

i. e., 1 year in prototype = 38.8 hours in the model; and the normal tidal period will take 3.3 minutes in the model.

5. Dominant Discharge of the Periyar River

5. 1. In letter No. 1485/47-1 dated 22-4-47 from the Administrative Officer it has been stated that the maximum discharge of the Periyar River beyond the tidal reach, is 40,000 cusecs. In the year 1924 the maximum discharge ever was, however, about 80,000 cusecs. The dominant discharge may thus be assumed = $\frac{2}{3} \times 40,000 = 26,666 = 27,000$ cusecs (approx) not considering the abnormally high discharge.

5. 2. It has also been stated in report No. V on the special observations made in the Monsoon of 1939, in accordance with the recommendations of the Special Committee on the movement of Mudbanks, that the average discharges of the Periyar through the Periyar mouth is 20,000 cusecs.

6. Silt contents of the Periyar.

6. 1. The following Table gives the total discharge in Cft. and the quantity of silt in Cft. as observed at the mouth of the Periyar in 1940.

Table No. 3/C-2 (vi)-2.

Serial No.	Date of hours of observation	Total ebb discharge in Cft.	Quantity of silt in Cft.	Average silt Charge in Cft. per Cft. of water.
1	3 P. M. on 5. 7. 40 to 5. P. M. on 6. 7. 40	4380×10^6	874800	0.000199
2	11 A. M. on 22. 7. 40. to 1 P. M. on 23. 7. 40	3824×10^6	135000	0.00035
3	12 noon on 2. 8. 40 to 1 P. M. on 3. 8. 40	4995×10^6	523800	0.000105
4	11-30 A. M. on 23. 8. 40 to 1-30 P. M. on 24. 8. 40	4239×10^6	315900	0.00074
5	11-30 P. M. on 6. 9. 40 to 12-30 P. M. on 7. 9. 40.	1359×10^6	24300	0.0000179
6	11-30 A. M. on 4. 10. 40 to 1-30 P. M. on 5. 10. 40	2004×10^6	410400	0.000206 Mean=0.000106

The weight of 1 Cft. of Periar silt = 80 lbs. The silt contents of the Periyar will, therefore, be

$$\frac{80 \times 0.000106}{62.4} \times 1000 = 0.13 \text{ part per 1000 by weight.}$$

Assuming the same silt charge in the model, the quantity of silt required for 6 hours working - day in the model

$$= 0.224 \times 6 \times 3600 \times 62.4 \times 0.000106$$

$$= 32 \text{ lbs.} = 0.4 \text{ Cft.}$$

7. As an indication whether the motion of water in the model will be turbulent, J. Allen (on P. 279 of his book⁽¹⁾) suggests the criterion viz.,

γ Should not be greater than $20 (\nu H)^{2/3}$, where ν is the maximum current velocity in knots during flood tide (usually occurring at about half-tide),

(1) 'Scale Models in Hydraulic Engineering' by J. Allen 1947, page 279.

If the *mean* depth in the channel and y is the vertical scale. This offers, according to him, a reasonably safe check upon the chosen vertical scale from the aspect of turbulence, especially if v and H relate to neap tides.

7. 1. At the mouth of the estuary at C. S. 1 the following observations are available regarding water level and discharges during Non-monsoon and Monsoon Neap tides.

Non-monsoon neap tide on 15-3-40:

On half flood tide, water level = 2.1 ft. above low water ordinary spring tide.

Discharge = 10,000 cusecs.

Area of cross-section 1 at L. W. O. S. T. on 15-3-40 is not available, but on 22-4-40 it was 51,980 Sq. ft.

Assuming practically the same area as on 15-3-40 the area of section up to 2.1 ft. above L. W. O. S. T. will be

$$51980 + 2.1 \times 1300 = 54710 \text{ Sq. ft.}$$

$$\therefore \text{Velocity} = \frac{10000}{54710} = 0.18 \text{ ft./sec.} = 0.11 \text{ knot.}$$

$$H = 38.5 + 2.1 = 40.6 \text{ ft.}$$

$$\therefore 20 (\sqrt{vH})^{2/3} = 20 (0.11 \times 40.6)^{2/3} = 20 \times 1.95 = 39$$

$\therefore y = 20$ is *not greater* than $20 (\sqrt{vH})^{2/3}$ and thus satisfies the criterion.

7. 2. Similarly, for monsoon neap tides on 25-9-40:
On half flood tide

Water Level = 1.6 ft. above L. W. O. S. T.

Discharge = 30,000 cusecs.

Area of C. S. 1 on 27-11-40 was 54210 Sq. ft. Assuming this area on 25-9-40 the area of C. S. up to Water Level 1.6 ft. above L. W. O. S. T. will be

$$54210 + 1.6 \times 1300 = 56290 \text{ Sq. ft.}$$

$$\therefore v = \frac{30000}{56290} = 0.53 \text{ ft./sec.} = 0.31 \text{ knot.}$$

$$\therefore 20 (\sqrt{vH})^{2/3} = 20 (0.31 \times 40.1)^{2/3} = 108.$$

$$\therefore y = 20 \text{ is not greater than } 20 (\sqrt{vH})^{2/3}$$

8. An analysis of experiments (p. 280*) made in straight-sided channels of rectangular cross-section indicated that the mean velocity of the stream \bar{v} ft. / per sec. required to move grains forming an initially flat bed is given by

$$\bar{v} = K \left(\frac{L+B}{21} \right)^{0.44} (\sigma' - \sigma)^{0.22} \frac{1}{M} \frac{0.22}{h} \frac{0.06}{m} \frac{0.22}{m}$$

where $K = 1.59$ for movement of many particles.

$\frac{L+B}{2}$ = mean dimension of the grains as viewed under a microscope (inches).

l = length of side of a cube having the same weight as the average grains (inches);

σ' = specific gravity of material;

σ = specific gravity of water (say unity);

M = uniformity modulus of material;

h = depth of water (inches);

and m = hydraulic mean depth of stream (inches).

8.1. The following are the observations of L and B under a microscope of sea sand sieved through 20 mesh and Londha sand sieved through 30 mesh. These sands were considered to be suitable for these experiments and were, therefore, analysed in detail as shown below :—

* See footnote (1), Page 103.

Table No. 4/C-2 (iv)-2.

No. of Particles	Length of particle		Breadth of particle	
	Sea-sand	Londha sand	Sea-sand	Londha sand
	mm.	mm.	mm.	mm.
1	0.04	0.024	0.03	0.023
2	0.09	0.046	0.045	0.045
3	0.02	0.043	0.017	0.030
4	0.024	0.030	0.013	0.030
5	0.042	0.050	0.018	0.040
6	0.055	0.055	0.018	0.035
7	0.031	0.035	0.025	0.024
8	0.020	0.040	0.012	0.023
9	0.025	0.055	0.017	0.045
10	0.065	0.051	0.040	0.039
11		0.019		0.015
12		0.013		0.012
Mean	0.041	0.038	0.023	0.030

$$\therefore \frac{L+B}{2} = 0.032 \text{ mm.}$$

$$= 0.0012 \text{ inch for sea sand ; and}$$

$$\frac{L+B}{2} = 0.034 \text{ mm.}$$

$$= 0.00133 \text{ inch for Londha sand.}$$

8.2. If $\frac{L+B}{2}$ is less than 0.015 inch, as in this case, then may be computed sufficiently accurately from the formula

$$1 = \left(\frac{L+B}{2} \right) \cdot \frac{B}{L}$$

$$= 0.015 \times \frac{0.023}{0.041}$$

$$= 0.00067 \text{ inch for sea sand ; \&}$$

$$1 = 0.00133 \times \frac{0.03}{0.038}$$

$$= 0.00089 \text{ inch for Londha sand.}$$

For sea sand $\sigma' = 2.62$ and
 for Londha sand $\sigma' = 2.64$
 M for both sands $= 0.75$

The depth of water in prototype
 $= 40 \text{ ft.}$
 $= 2 \text{ ft. in model.}$

$$n \text{ proto} = \frac{A}{P} = \frac{A}{W} = \frac{52000}{1300} = 40$$

$n \text{ model} = 5.0 \text{ inches (found out by plotting the section).}$

$$\therefore \bar{V} \text{ sea sand} = 1.59 \left(\frac{0.0012}{0.00067} \right)^{0.44} \times (2.62 - 1)^{0.22} \times (0.00067)^{0.22}$$

$$\times (0.75)^{-0.22} \times (24.0)^{0.06} \times (5.0)^{0.22}$$

$$= \frac{1.59 \times 1.29 \times 1.11 \times 0.2 \times 1.21 \times 1.425}{0.88}$$

$$= 0.9 \text{ ft./sec.}$$

$$\begin{aligned} \bar{v} \text{ Londha sand} &= 1.59 \left(\frac{0.00133}{0.00089} \right)^{0.44} \times (2.64-1)^{0.22} \times (0.00089)^{0.22} \\ &\times (0.75)^{-0.22} \times (24.0)^{0.06} \times (5.0)^{0.22} \\ &= \frac{1.59 \times 1.19 \times 1.11 \times 0.212 \times 1.21 \times 1.425}{0.88} \\ &= 0.87 \text{ ft/sec.} \end{aligned}$$

8. 3. Considering, now, the Mound formula (2), (given on p. 280 of Allen's Book); for the first disturbance of the particles, forming a mound of material, namely

$$\begin{aligned} \bar{v} &= 1.30 (\sigma' - \sigma)^{0.27} m^{0.27} h^{0.23} \\ \bar{v} \text{ (sea sand)} &= \frac{1.3 (1.62)^{0.27} (0.00067)^{0.27} (24.0)^{0.23}}{(0.75)^{0.27}} \\ &= \frac{1.3 \times 1.139 \times 0.14 \times 2.07}{0.88} \\ &= 0.49 \text{ ft sec.} \end{aligned}$$

$$\begin{aligned} \bar{v} \text{ (Londha sand)} &= \frac{1.3 (1.64)^{0.27} (0.00089)^{0.27} (24.0)^{0.23}}{(0.75)^{0.27}} \\ &= \frac{1.3 \times 1.143 \times 0.15}{0.88} \times 2.07 \\ &= 0.525 \text{ ft/sec} \end{aligned}$$

8. 4. Maximum flood velocity in model

$$= \frac{2.53 \times 1.69}{4.49} = 0.95 \text{ ft/sec.}$$

Maximum ebb velocity in model

$$= \frac{2.93 \times 1.69}{4.49} = 1.1. \text{ ft/sec.}$$

8. 5. The following Table gives the velocity to move many particles, velocity for the first disturbance of particles forming a 'mound,' and the model velocities corresponding to the maximum ebb and flood velocities in the prototype:—

Table No. 5/C - 2 (vi) - 2.

Type of sand	\bar{v} for many particles to move:	\bar{v} according to Mound formula	Model Velocities	
			Maximum Flood	Maximum ebb.
Sea-Sand (Sieved through 30 mesh); m = 0.191 mm.	0.9 ft Sec.	0.49 ft/Sec.	0.95 ft/Sec.	1.1 ft/Sec.
Londha Sand sieved through 30 mesh; mean dia. = 0.24 mm.	0.87 ft/Sec.	0.525 ft/Sec.	0.95 ft/Sec	1.1. ft. Sec.

The above Table (No.5), therefore, shows that the velocities in the model will be sufficient to move the bed particles and the vertical scale of 1/20 will be satisfactory from the aspect of bed movement. Both the sands analysed are suitable in this respect but sea-sand has been preferred being finer. For the mudbanks and the mud bed, the *original* mudbanks material from Cochin is proposed to be used.

9. Adopting the V. E. = 50 and the depth scale = $1/20$, the silt in the model should have a vertical motion $\frac{1000}{3/2} = 11.1$ times as fast as in the prototype.

The following Table shows this factor in other models:-

Table No. 6 / C - 2 (vi) - 2.

Name of the model	Horizontal Scale x	Vertical Scale y	$\frac{x}{y^{3/2}}$
1. Mersey (Manchester University)	7040	190	2.68
2 Rangoon River	8060	192	3.04
3 River Parrett	3000	260	0.716
4 Severn	8500	200	3.0
5 Dee	(a) 5000 (b) 40000	25 100	40.0 40.0
6 Great Ouse & Wash	2500	41.7	8.95

9. 1. In comparison with the first four models, it will be seen that a faster rate of fall of water-borne silt is required in this model. The V. E., however, cannot be reduced to get the rate of fall equal to 3 times that in the proto-type as the important scales like depth scale, discharge scale would then become unsuitable. Alum will, of course, be essential and arrangements are made for recirculation to economise in cost of alum.

10. The following scales are, therefore, tentatively, decided to be adopted for the pilot model

Length Scale	=	$\frac{1}{1000}$
Depth Scale	=	$\frac{1}{20}$
V. E.	=	50
Discharge Scale	=	$\frac{1}{89100}$
Time Scale	=	$\frac{1}{223}$
Velocity Scale	=	$\frac{1}{4.49}$

11. Adopting these scales the tidal curves obtained for the model, corresponding to the tidal curves observed in the harbour in 1940, will be as shown in Fig. 35. These tidal curves are for Stn. No. 1 at the Gut. The curves have been plotted assuming the speed of the recorder as 1 inch of the periphery in 4 minutes.

Table 7 / C - 2 (vi) - 2.

Water Level for Station No. 1.

Non-monsoon Spring Tide		Monsoon Spring Tide	
Time	W. L. above low W. O. S. T.	Time	W. L. above L. W. O. S. T.
	Ft.		Ft.
7	0.6	9	1.7
9	1.1	11	2.9
11	2.7	13	3.3
13	3.0	15	2.8
15	3.3	17	1.7
17	2.5	19	1.4
19	1.6		

Non-Monsoon Neap Tide		Monsoon Neap Tide	
Time	W. L. above L. W. O. S. T.	Time	W. L. Above L. W. O. S. T.
	Ft.		Ft.
10	0.1	23	0.7
12	1.0	1	1.0
14	2.3	3	1.7
16	2.9	5	2.3
18	2.7	7	2.4
20	2.2	9	2.2
22	1.5	11	2.0

The tide data is being yet analysed to study the representative tides which must be reproduced to simulate in magnitude, order and frequency the integrated tidal effect.

11. 1. In response to a request from the Director, the President Survey Research Institute, Survey of India, furnished a note on the main features of the tides at Cochin. These notes are reproduced in Appendix III.

12. Other Relevant Features :

1. The coastal currents are supposed to run south for 8 months in the year, to be more or less stationary for a month and to run north for the remainder. The Meteorological Atlas shows varying speeds, roughly in proportion with wind velocity. There is practically complete agreement among sailors that the north to south drift has been much stronger since 1934 or 1935. Notwithstanding this, the sand and shells continue to move north with the ground swell.

The opposing force is the south-west swell set up by wind waves before and at the beginning of the S. W. Monsoon.

2. Soundings off Cochin.

(a) From 1835 to 1913, the contours may be reckoned either as fairly stable or returning to about their normal position if temporarily displaced between the dates of charts;

(b) at some time unknown between 1913 and 1920, there was a shallowing but of no material importance ;

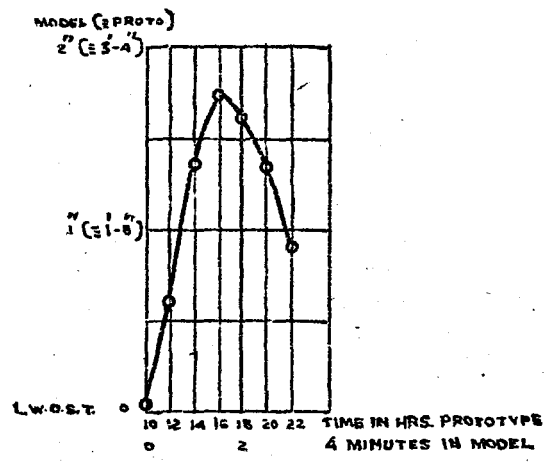
FIG: 35/C-2 (VI)-2

TIDAL CURVES AT STN NO 1

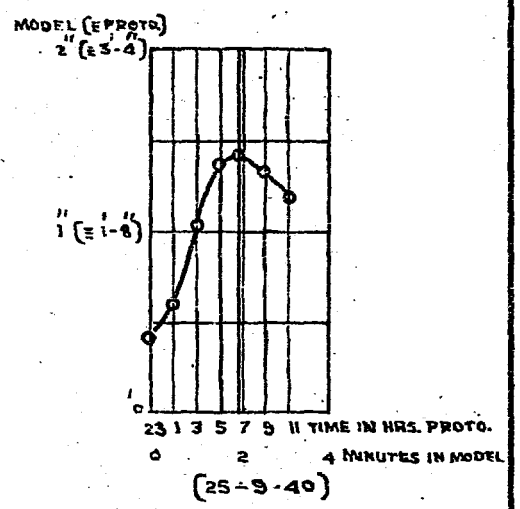
COCHIN HARBOUR MODEL

SCALE {
HOR. 1" = 4 MIN. = 14 HRS. 52 MIN.
VER. 1" = 1" = 1'-8"

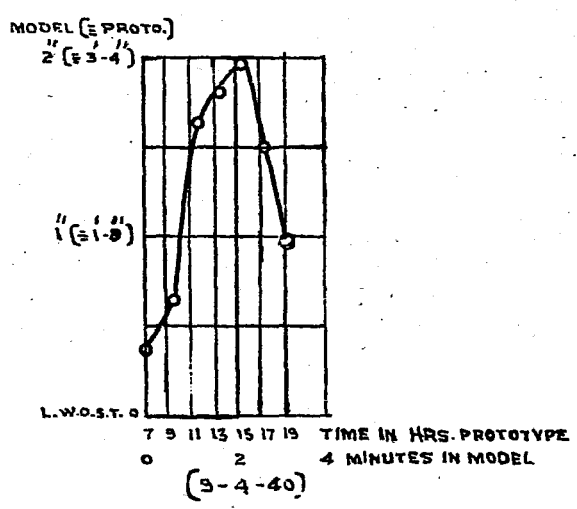
NON MONSOON NEAP TIDE



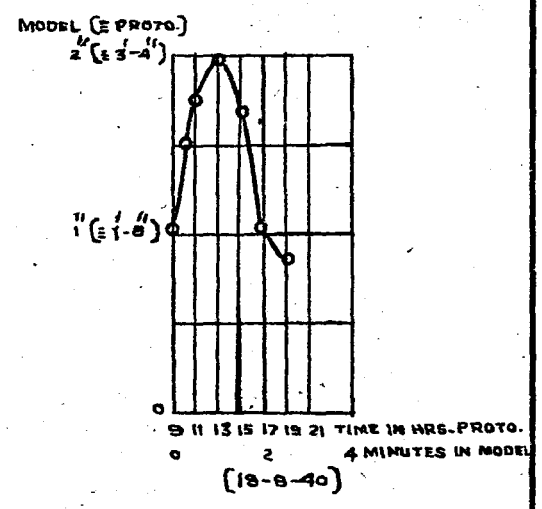
MONSOON NEAP TIDE



NON MONSOON SPRING TIDE



MONSOON SPRING TIDE



Note:- With the existing recorder the movement of the drum along the periphery is 1 inch in 4 minutes.

(c) between 1920 and 1931, the shallowing became obvious specially to the north of the harbour entrance;

(d) after 1933 or 1934 the shallowing to the north increased, and rapidly so in 1936-38; and

(e) on the contrary, $1\frac{1}{2}$ miles south of the Flag-staff, the contours returned to their original line of 100 years ago, or near it.

It is of interest that the rate of shallowing coincided with the approach of the Narakal bank towards the harbour entrance; the nearer the bank the quicker the shallowing.

3. Soundings in Outer Channel.

The depths in the channel before and after the monsoon are given in the appended table. The most marked difference occurs in 1937 when the average sounding for the whole channel was the lowest on record except in 1928.

Table 8 / C-2 (vi) -2.

Year	Measured depths in outer channel.				Silting in millions cubic yards.	
	After dredging average		After monsoon average.			Least depth
	Feet		Feet			
L. W. O. S. T.	L. W. O. S. T.	L. W. O. S. T.	L. W. O. S. T.			
1928	34.0	27	24		1.26	
1929	35.9	32.7	29.3		.87	
1930	35.9	32.1	28.8		1.06	
1931	36.5	34.1	31.6		.65	
1932	36.5	34.1	31.0		.65	
1933	39.8	33.5	30.5		1.54	
1934	39.6	31.6	29.5		2.02	
1935	38.4	31.3	29.3		1.89	
1936	38.5	31.4	29.8		1.95	
1937	38.5	29.9	28.8		2.37	
1938	38.2	33.1	30.8		1.7	
1939	38.25	33.4	30.2		1.65	
1940	38.2	33.1	31.5		1.5	

13. The data are still under further investigation. The construction of the model is started. It is fully realised that this model investigation will be very greatly complicated by the unique phenomena of the formation, movement and the calming effect of the mudbanks. The difficulties of simulation of these effects in the model will be great but they can be surmounted, it is believed, by suitable model technique and by making due allowances for model limitations.



Appendix I/C--2 (vi)-2

MOVEMENT OF MUDBANKS, COCHIN

SUMMARY OF THE REPORT AND EVIDENCE.

1. Behind the Malabar Coast there lies a strip of laterite which has a petrological content unique in geology. On disintegration by weather or flood it forms an exceedingly fine mud and is carried out to sea through the two main river openings at Cochin and Periyar, about 15 miles apart. Although the silt content in the water is a very small percentage, yet the catchment area is so great (over 3,000 square miles) and the fluvial discharge so high, that in the aggregate, and over a long period of time, there is either what is called a secular rise in the sea bottom, or an extension of the mud fringe farther out to sea, perhaps a little of both.

2. The slow and regular deposition of the mud, however, is affected in three ways; first, by the meeting of the fluvial discharge with the seawater, which accelerates the deposition of silt; second, by the opposing ground swell which drives the material in course of deposition, or part of it, back shorewards; and third, by what has been called the encircling arms of the Cochin and Periyar discharges. (The natural flow of the Periyar discharge is to the south, and that of Cochin chiefly to the north).

3. The net results of these opposing forces are that outside the littoral sand, consisting of the larger grains of gneiss etc, brought down by the rivers and deposited more quickly than the mud, there exists a long belt of fine mud, with occasional temporary or permanent banks held in place by tidal and fluvial currents. The "toes" of these banks protrude beyond the regular contours off the seacoast, which ordinarily run parallel with the shore and when a storm occurs these "toes" are immediately attacked by the ground swell in its effort to restore normality of contours.

4. In this action lies the secret of the calm areas. The water over the "toes" is thrown into a highly viscous state by the stirred-up mud, and this forms a medium in which the ocean swell is rapidly dissipated. This phenomenon is not observable elsewhere in the world to anything like the same extent simply because the fineness and composition of the laterite mud are unique.

5. Apart from the varying forces engaged upon the formation of the banks there are two other which affect their stability after formation, and these are the sea current and the sea waves produced by longshore wind. There seems no doubt that taken over the whole year the preponderating effect of these is southward, and while their movements are not strong enough to affect sand, they will certainly move this fine mud. Consequently, the Narrakal bank which was charted as immediately south of the Periyar opening in 1775 has moved south to the Cochin opening in a period of about 162 years. It may be assumed that the bank will continue to cross the Cochin opening, and afterwards move south much more rapidly, because it will have no "encircling arm" coming up from the south to keep it in position at any point. Meanwhile another bank will probably form between Cochin and Periyar, and is in fact believed to be accreting now.

6. From the above statement of the sequence of events and other points in the Committee's report the causes of silting both in the outer channel and in the inner harbour may be readily diagnosed.

(a) The charts show that from 1835 to 1913 the secular rise of the sea bed or the extension seaward of the mud contours was so slow as to be of no practical consequence.

(b) After 1913, and possibly owing largely to the severe and successive cyclones in 1911 and 1912, the mudbank began to move south, and from this time onwards the growth of the mud contours seaward, (with accelerating rapidity), began to be noted off Cochin.

(c) The five cyclones in the years 1920 to 1922 inclusive, the two cyclones in 1931/32 and the earthquake in 1934, all combined towards this acceleration, with the result that the outer channel became closer and closer under the lee of the advancing bank and was crossed by it in 1937.

(d) In monsoon time there is an oscillation of silt along the bed of the outer channel caused by the ebb and flow of tides, and a resultant movement towards the harbour caused by the ocean ground swell.

(e) Inside the harbour the depth and volume of water flowing through the Ernakulam channel keep it clear, with the result that the Mattancherry channel which has been widened beyond its "critical" width becomes a silting trap for the mud in suspension in the harbour. This was probably inevitable even if the channel had not been widened beyond 600 feet.

(f) No doubt there will be silting in the outer channel even if the mudbank moves completely away from it. The silt would be stirred up in the open sea and driven into the channel. But the fact of the mudbank being alongside naturally increased the quantity of silting, especially when each year's silt was pumped to the north instead of the south.

7. It is thus apparent the system of dredging in force now, with both outer and inner silting being discharged south of the harbour, is in accordance with the natural forces at play along this coast; and although occasional and abnormal phenomena may give rise to other questions in the future, the port can only budget for what appears to be the average and normal condition. This does not give rise to any fear as to the future of the Port.

The above remarks are based on the recent Committee's report, and may be of some use as a non-technical introduction to the closer study which the report requires.

APPENDIX II / C-2 (vi)- 2

' Preliminary Report on the improvement of Cochin Harbour '
dated June 1947, by K. K. FRAMJI, Director Central
Waterways Irrigation & Navigation Research Stn.
& T. M. OAG, Director, Navigation, Central
Waterways Irrigation & Navigation Commission.

(1) The harbour area west of Willingdon Island approaching the deep sea berths being an offtake channel on the *convex* side of the originally dredged channel leading to Ernakulam basin is the natural intake for inborne silt and detritus on the flood tide.

(2) Being oversize in section, it acts readily as a reservoir for the deposition of this inborne material : because there is an optimum section to which a natural channel will adjust itself (and at which it will neither silt nor scour) for passing a given discharge, sand charge and grade of material at a given slope, and excessive waterway will lend itself readily to inordinate silting.

For equal discharge, the harbour (Mattanchery) channel carrying a heavier charge or grade or both requires a somewhat wider and shallower section than the Ernakulam channel which, offtaking on the *concave* side, draws relatively sand-free top water. The Ernakulam channel will also, however, carry relatively more discharge on the flood tide current, being on the concave arm of the bifurcation. The Ernakulam channel is more or less stable at about 30/32 ft. depth over a bed width of 500/600 ft. Since the harbour channel will carry relatively less discharge and more bed charge, and since minimum depths (30/32 ft.) required for navigation must be provided, it may be assumed to require about the same dimensions as the Ernakulam channel, for minimum silting. It must, however, be stated clearly that, though this will reduce silting, the section cannot be stable until the inherent defects of the adverse physical configuration which will always attract inborne material are countered with the aid of model tests.

(For this reason, pending results of model experiments, the channel may be dredged some 25 per cent more in width than the minimum width considered essential for manoeuvring ships in the harbour channel or 600 ft. wide, whichever, is more, to allow for silting which may occur between dredging operations.)

(3) If the proposed reduction in width is accepted, the best results could be attained by dredging on the *concave* side adjacent to the Willingdon Island where there would be least risk of silting as it would then be reasonable to expect that:—

(a) the channel would gradually accrete on the *convex* side, that is off British Cochin;

and (b) as the channel contracts to the desired width the dredging visualized at the offtake may be reduced or become less frequent.

(4) In any case, the proposal to accept a channel with the minimum practicable width, as suggested in (2) *ante*, would reduce dredging by more than 50 per cent immediately.

(5) It is suggested that the bed width of 1600 ft. is unjustified as a navigational requirement. This width is out of all proportion to existing widths of channel in other major river ports, namely Calcutta, Chittagong and Rangoon, where restricted manoeuvring space only is available, yet Calcutta port handles almost half the port tonnage for all India. It is felt that a channel of the width specified above should impose no difficulty whatsoever in handling the largest vessels likely to visit the port of Cochin. It is suggested that vessels could be berthed on the flood tide by turning them on an anchor before reaching the harbour channel and then back up to the allotted berth, be it to the mooring buoys or wharves. It is emphasized that backing vessels in this manner with an anchor " underfoot " is the accepted daily practice in all riverine ports. In Calcutta the channel is a winding one over 6 miles in length and the width seldom exceeds 350 ft.

(6) Should pilots consider this proposed evolution impracticable in a channel of the width proposed, in the port of Cochin, it is suggested that practice should be initiated immediately in the wide channel so that the art may be acquired to work in restricted channels with confidence.

It seems unreasonable that the existing width namely 1600 feet should be imposed upon the port of Cochin where the channel is comparatively straight, sheltered, and the current velocity is ideal for manoeuvring.

Appendix III/C—2 (vi)—2

NOTES ON THE TIDES NEAR COCHIN

1. Tide gauge observations were taken at this port during the years 1886-1892, and were subjected to harmonic analysis. The amplitudes and phases of the constituents are:—

Constituents	H (ft)	g° (I. S. T.)	constituents	H (ft)	g° (I. S. T.)	Constituents	H (ft)	g° (I. S. T.)
A ₀	1.91	—	—	—	—	—	—	—
2M ₂ K ₁	0.019	114	M ₂	0.011	169	M ₄	0.026	089
2S M ₂	0.004	311	K ₂	0.080	036	M ₆	0.009	107.
M S ₄	0.020	159	P ₁	0.166	056	S ₁	0.029	195
J ₁	0.038	073	N ₂	0.156	307	M ₂	0.727	339
Q ₁	0.075	065	M ₂ N	0.020	114	S ₂	0.261	042
T ₂	0.036	042	M ₂ K ₁	0.030	144	K ₁	0.591	058
L ₂	0.030	005	Sa	0.337	302	O ₁	0.309	059
Y ₂	0.039	329	Ssa	0.118	151	2N ₂	0.019	252

2. From the above, it will be seen that the tide at this port is a combination of the semi-diurnal and diurnal waves, the two species having more or less an equal range.

3. Two high and two low waters occur during each tidal day, but with marked diurnal inequality. The difference in height between the morning and evening tides amounts at times (when the moon's declination is high) to as much as 90% of the range.

4. This diurnal inequality in heights is featured prominently in both the high and low waters. The inequality in the latter is, however, comparatively greater.

5. The sequence of the (tropic) tide is "lower low" to "higher high".

6. The inequality in time, too, is considerable, especially in the case of the high waters. The time intervals between successive high waters differ at times by as much as 10 hours.

7. The tide is of the *synodic* type, though the declination effect is strikingly predominant. The *anomalous* influence is also present, but comparatively in a much lesser degree.

8. Spring and neap tides follow full/new moon and 1st/3rd quarters of the moon respectively by about 2 days. Perigean and Apogean tides follow moon's perigee and apogee by about a day.

9. The mean range of the tide at Cochin is 1.7 ft. The mean range of greatest ordinary springs is 3.2 ft. The Establishment of the port is 11°-26'.

10. The highest high water so far recorded (since 1886) occurred in December 1887 and was 4.3 ft. above the datum of soundings. The lowest low water occurred in May 1889 and was 0.6 ft. below the datum.

11. The monsoon effects during the months of December to February and July to September generally raise the tide level by about a third of a foot.

12. On certain occasions when the moon's quarters happen to combine with the moon's (and also sun's) maximum south declination the water remains nearly "slack" for a considerable period of time (about 6-8 hours) between the higher low water and the lower high water. During such occasions, generally, the former height is slightly greater than the latter.

13. There occur occasional very, rapid changes in the level of water, to the extent of a foot, due to squalls.

14. The stream, flowing into the back water, is sometimes constant for about 20 hours. The regular but inconsiderable rise and fall of the water is, however, noticeable on a tide-gauge record.

15. The compound tide at Cochin has admitted of the usual theoretical treatment and the ordinary harmonic analysis. Tidal predictions for this port by the harmonic method using 24 constituents only (including a few shallow water and over tides components) have been found to be fairly satisfactory.

C—HYDRAULIC WORKS.

4—DAMS AND WEIRS

(ii)—DESIGNS—

C-4 (ii) 1 Proposed Design of Lower Sind Barrage at Kotri for effective sand exclusion from canals Sind.

(1) Introduction

The experiments reported in Annual Report (Tech.) 1946 dealt with (a) preliminary experiments without the barrage for proving the model and (b) experiments with the barrage mainly for testing the design as proposed by Sind.

1. 1 The former, (a), indicated that the survey between C. S. miles $5\frac{1}{2}$ to 7 was either incorrect or that there was a hard, in-erodible bank in this reach or both. When resurveyed, it showed that the survey was incorrect and that in-erodible material exists in this reach. Experiments also revealed the unsuitability of the position of the Fuleli Offtake at Jamshoro with respect to the existing river bank.

The latter, (b), showed that there was very little difficulty in excluding sand from the Pinyari Feeder offtaking from the pocket at the Barrage; but exclusion from the Fuleli head at Jamshoro gave trouble due to

- (a) counteraction of curvature by the Barrage;
- (b) unsuitable position of the existing Fuleli Head at Jamshoro in respect of the river curvature;
- (c) excessive draw-off of discharge into the canal at the low stage of river discharge (200,000 cusecs); and
- (d) attraction of flow by the central island.

1. 2 In order to restore curvature and improve sand exclusion from the Fuleli, different measures, such as (a) concave guide bank all along the left bank, (b) a raised crest at the entrance of the approach channel to the Fuleli, (c) change in the shape and size of the central island, (d) a divide wall extending right up to the upstream abutment of the Fuleli Canal, (e) wedge-shaped regulation of the Barrage, and (f) a submerged, curved vane located sufficiently upstream in the gorge, were tested but proved unsuccessful for an "all time" solution.

1. 3 While these experiments were in full swing, a discrepancy in the plotting of the 'Mukam' (burial ground) was reported. Hence a new alignment of the Barrage was suggested by the Director so as to make the barrage normal to the flow and reduce the length of the central island and the afflux between the two arms of the river.

The experiments repeated with the new alignment (which was accepted by Sind) showed no perceptible improvement in the sand exclusion from the Fuleli, thus confirming the unsuitability of measures mentioned above.

1. 4 It was only when the Fuleli Head Regulator was shifted alongside the Pinyari Head Regulator in the left pocket that there was sufficient promise of satisfactory sand exclusion from the canals. The design was modified accordingly and recommended to Sind who accepted it.

1. 5 The central island of the double Barrage design was found eminently successful in creating favourable curvature on both banks and in securing the desired control over distribution of discharge in the two arms of the river—the two purposes for which it was designed. The experiments also gave valuable information about the main factors governing the design of a barrage particularly in respect of the principal requirement of efficient exclusion of sand from its offtakes.

1. 6 This year's work mainly covers the experiments carried out for determining the salient features in the design of the barrage, such as:—

(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 the pocket gates higher than the other barrage gates; and (i) necessity or otherwise of paving the bed of the pockets.

[Note :— For (h) and (i) see item c—4 (ii) — 2 of the Report]

(2) The Model.

The model, which represented a reach of $8\frac{1}{2}$ and $6\frac{1}{2}$ miles upstream and downstream of the proposed barrage, respectively, had the following scale ratios:—

Length scale ratio	...	1/250
Depth scale ratio	...	1/50
Discharge scale ratio	...	1/62,500
Vertical exaggeration	...	5.0
Surface slope exaggeration		7.5 :to 8.5 respectively
		for Q \equiv 200,000 cusecs.
		to Q \equiv 5,30,000 cusecs.

(3) Sill levels of head regulators —

After carrying out experiments with alternative sill levels for the Fuleli and Pinyari Head Regulators, i. e. at R. L. 49.0 (3' lower than the crest of the barrage pocket spans and 3' higher than the upstream pavement level of R. L. 46) and at R. L. 52.0 (sill flush with the pocket crest), it was observed that there was very little or no improvement in exclusion consequent on high regulator crests.

Hence, in the design proposed by the Director, the low sill levels were favoured for the following reasons :—

- (a) Low sills would allow *minimum* pond levels at low river stages.
- (b) High sills would make the Head Regulator designs less economical requiring an increase of 3 spans of 24'.
- (c) Experience on some of the existing barrages has already shown that the relative difference between the crest of the Barrage and sills of the head regulators is not effective in arresting the excess influx of bed sand into the canals; but, it is mainly favourable curvature, associated with favourable V_R/V_P ratio, which governs the bed sand entry into the canals.

There is one school of thought which considers that a step between the regulator sill and pocket level provides a silt trap. But there is very little strength in this argument as it would take little time to fill this reserve; and there is no support to this belief from model experiments which show that any bed sand which rolls into the Pocket finds its way into the Canals regardless of the height of sill above Pocket Level.

(d) Experiments have also shown that the divide wall nose should extend up to the upstream abutment of the first feeder (i. e. Fuleli Feeder) to get satisfactory sand exclusion from the canals. With higher sill, the divide wall nose will have to be pushed further upstream of the barrage by 87 ft. — thus shifting the control point, which, when nearer the barrage, is beneficial at the time when the wedge-shaped regulation is required (see para 5. 2 below), namely at low stages of river discharge.

(e) Higher sills necessitate widening the Head Regulator to such an extent that it becomes wider than the canal width downstream of the Head Regulator. Experience at Sukkur has shown that the carrying capacity of end spans is considerably reduced under such conditions — sometimes throwing the end spans completely out of action.

Notwithstanding these advantages, it was decided by Sind to maintain sills at R. L. 52.0 and subsequent model tests were made with Head Regulator Sills at R. L. 52.

(4). Design of the left and right pocket and the optimum number of spans.

4. 1 After shifting the Fuleli Head Regulator into the Pocket alongside the Pinyari Head Regulator, experiments were carried out to ascertain the efficacy of 8 and 4 spans in the left and right pockets, respectively, as tentatively proposed by

the Director in the note on revised design suggested by him; and also to determine the *optimum* width at the entrance to the pocket so as to get as efficient sand exclusion as possible.

The width of the left pocket was changed by giving the necessary splay to the head regulator face line by maintaining 8 spans in the pocket as proposed. It was observed that a minimum width of 600' was essential at the entrance to the pocket to ensure satisfactory sand exclusion from the canals by lowering the V_P/V_R ratio sufficiently.

[Correctly speaking, the crucial ratio is $V_P/V_{P_0} \div V_R/V_{R_0}$; but depths inside and outside the pocket being nearly equal V_P/V_R is as good a criterion.]

Alternatively, it would have been possible to get a uniform width of 600' in the pocket by accommodating nine spans of 60' each. Though one additional span in the pocket, the aggregate width remaining unchanged, would have made the river section of the barrage slightly tighter (thus lowering the " Still pond " limit)— a splayed Head Regulator face with 8 spans in the left pocket was considered to be a better proposal for the following reasons:—

(a) A tapering pocket is better than a pocket of uniform width in respect of scouring operations as V/V_0 (V being scouring velocity and V_0 the critical velocity for the depth of flow) increases progressively at sections from the nose of divide wall to the Barrage; so that any shoaling against the divide wall in the pocket (due to curved flow) will be more easily scoured with a tapering pocket.

(b) It would slightly ease the sharp curve otherwise to be negotiated between the roadways over the barrage and the head regulator.

(c) There is also a very sharp curve in the Pinyari Canal just below the head regulator; and had there been no splay to the face of the head regulator, this curve, too, would have been more difficult to negotiate.

4.2 As the length of the right pocket is small, a similar splay has insignificant effect. Moreover, the same width of 261' at the entrance to the pocket as at the Barrage — i. e. 4 spans of 60' each — was quite satisfactory for sand exclusion from the Kalri Baghar Feeder.

4.3 Subsequently, on a request from Sind, model tests were made to see whether — without creating other complications — the number of spans in the left pocket could be reduced from 8 to 7 or even 6, and in the right pocket from 4 to 3, keeping the widths at entry the same (i. e. , 600' and 261' in the left and right pocket, respectively) by increasing the splay of the face of the head regulators.

These model tests were made to compare pockets with different number of spans for—

(a) the bed and surface lines of flow to assess the relative efficiency of exclusion;

(b) the regions of return or slack flow and still water; and

(c) velocities in the pocket and in the river section, particularly at points in the vicinity of the divide wall, for the relative values of $V_R : V_P$ and $V_P : V_0$.

All these comparisons were made for the *worst* river discharge $\cong 2,00,000$ cusecs; but at least four complete cycles of discharges were run before these observations were made for $Q \cong 2,00,000$ cusecs in order to get the correct formation of shoals and deeps as affecting the rising river discharges.

RIVER INDUS

NO-6634
D.W-I & N.R.STN-POONA

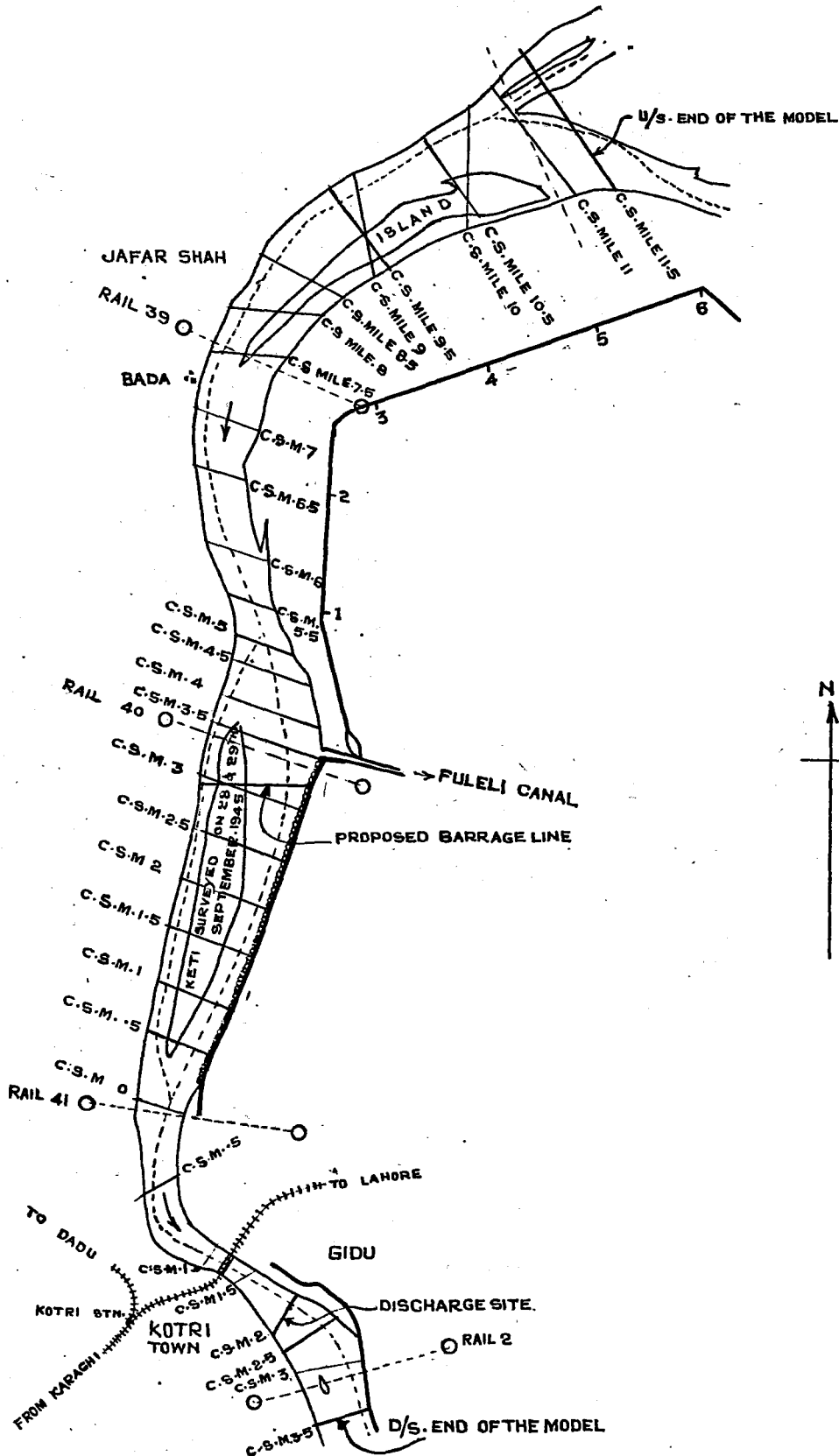
PLAN SHOWING THE MODEL REACH

$6\frac{1}{2}$ & $6\frac{1}{2}$ MILES U/S. & D/S. OF THE

PROPOSED BARRAGE LINE.

SCALE:-1"=30'≐7500'

FIG.1/C-4(ii)-1



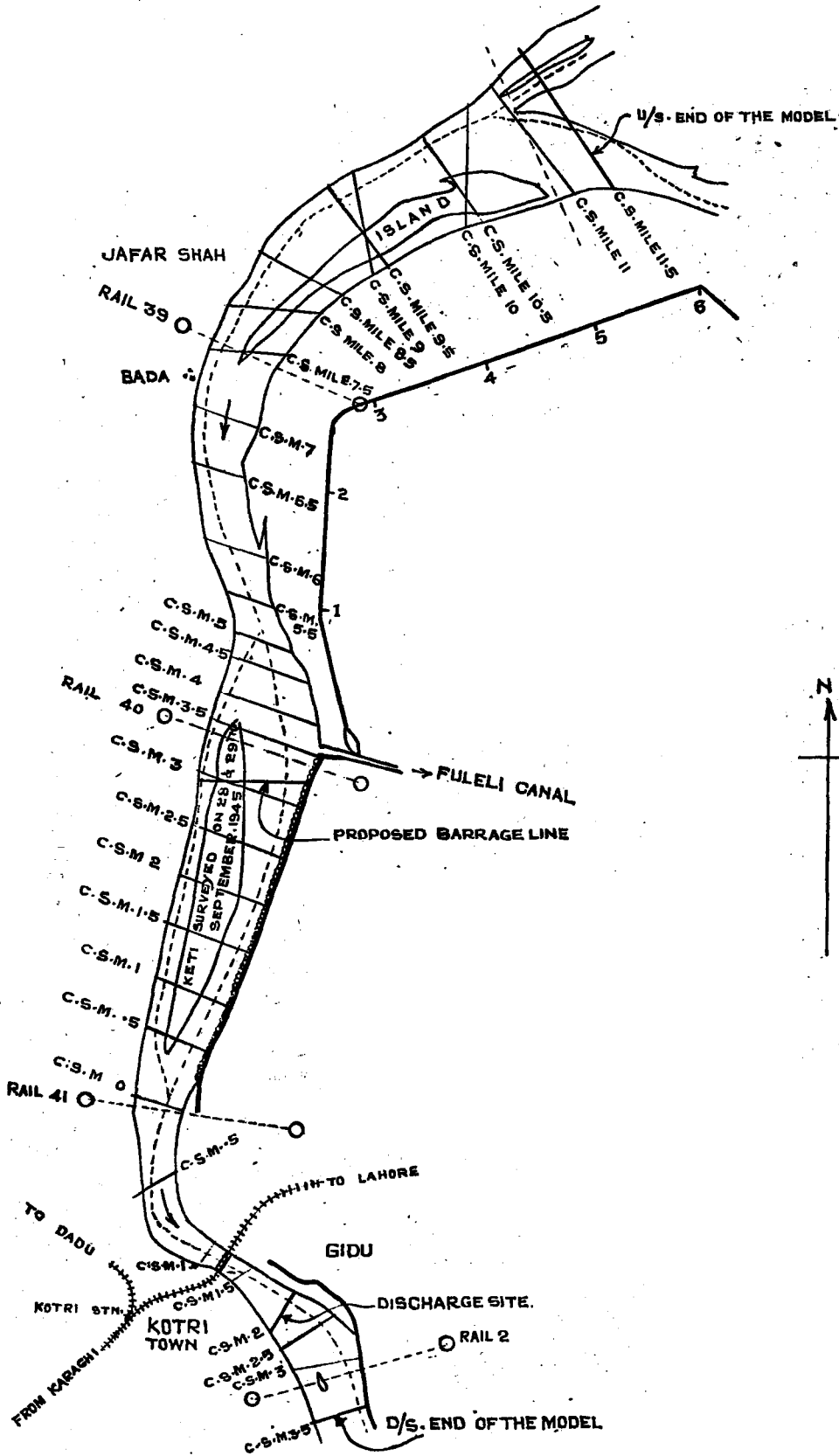
RIVER INDUS

NO-6634
C.W.I. & N.R. STN. POONA

PLAN SHOWING THE MODEL REACH 8 1/2 & 6 1/2 MILES U/S. & D/S. OF THE PROPOSED BARRAGE LINE.

SCALE: 1" = 30' = 7500'

FIG.1/C-4(II)-1



[Note :-2,00,000 cusecs is the lowest river discharge at which sufficient bed movement occurs in the river; of all the discharges that can be reproduced in the model, this is the discharge at the rising stage, at which exclusion is most difficult—i. e. at which there is maximum entry of bed material into the canals.]

4. 4 It was observed that the original proposal of the Station, as recommended in the Revised Design—namely 8 spans in the left pocket and 4 spans in the right pocket—was better than 7 or 6 in the former and 3 in the latter, for the following reasons:—

(a) As figures 2 (a), 2 (b) and 2 (c) for the left pocket and 3 (a) and 3 (b) for the right pocket will show, far from the area of shoal against the divide wall being larger in a wider pocket, as one would expect, the very reverse was true: with a reduction in the number of spans not only was the region of return or slack flow greater in area *relative* to the area of the pocket, but even in *absolute* area. The explanation for this behaviour apparently is that in a narrower pocket—with its attendant greater splay to the face of the Head Regulators — the current direction in the pocket (particularly in the vicinity of the Divide Wall) becomes more oblique, while in a wider pocket the direction of the current (particularly adjacent to the Divide Wall) is more or less parallel to the Divide Wall. The wider pocket is, therefore, to be preferred as less shoaling is desirable. Figs. 2 and 3
C-4 (ii)-1

(b) With given shape and dimensions of the Central Island — which depend on other factors, such as distribution of discharge in the two arms, afflux between the arms, scour at the nose of the island etc., — the ratio V_R/V_P governing the curvature of flow and the sand exclusion at the divide wall is greater for the largest number of pocket spans. In Appendix I, the ratios have been calculated for the various alternatives; from the velocities measured in the model these ratios (V_R/V_P) similarly decrease with decrease of number of pocket spans as shown in Table. I. Appendix I
C-4 (ii)-1

Table 1 /c —4 (ii)-1

	8 spans.	7 spans	6 spans
Equivalent mean V_P (ft/sec			
" at entrance.	3.10	3.10	3.10
" " V_R "	3.60	3.50	3.40
" " V_P/V_P (model).	1.15	1.12	1.09
" " V_R/V_P (App. I)	1.12	1.07	1.03

Hence, for this reason, too, the Station proposal of 8 spans was preferable to the other alternatives.

(c) The respective lines of flow for each case showed that with a greater number of spans in the pocket more surface water was tapped by the pocket, with a corresponding reduction in the bed flow.

(d) Moreover, it has been observed in the model tests that exclusion becomes less efficient after shoaling; so that, in the narrower pockets, where the area of shoaling is both *relatively* (to the area of the pocket,) and *absolutely* larger, the loss of efficiency in sand exclusion consequent on shoaling will become greater.

(e) These observations for relative efficiency of exclusion were made while maintaining the same scoured bed level at the nose of the divide wall; in actual fact, the differential scour between the river and pocket will be greater with 8 spans

than with 6 or 7 and hence the twist at the nose and the exclusion would be correspondingly better with 8 spans. In other words, the tests were made under unfavourable hypothesis for the larger number of spans.

(f) The greater the number of spans in the pocket, with consequent reduction in the river spans — the total length of the Barrage remaining unchanged — the lower will be the " still pond " limit. This is a great advantage as it enables keeping the upstream water level relatively lower, thus tending to reduce accretion.

Appendix I
C-4 (ii)-1

(g) As Appendix II clearly shows, without lowering the cistern levels scouring operations would not be equally feasible with 6 or 7 spans as with 8 spans in the left pocket; and, even after lowering the cistern levels — which is undesirable on grounds of extra expense and greater construction difficulties — the V/V_0 ratio, which denotes the scouring potential, would be lower; while with 8 spans, it should be possible to scour reasonably efficiently with the given cistern level at the intensity of $q = 115$ cusecs/ft.

For all these reasons, 8 and 4 spans were again recommended for the left and right pockets, respectively, and are accepted by Sind.

(5) Long pocket versus short pocket.

5.1 The following two proposals were made by the Superintending Engineer, Project Circle, for layouts of the left pocket head regulators :—

(a) To put the Pinyari head regulator on a skew at an angle of 60° to the river, with or without the inclusion of Wadhu Wah and Fisadi Wah head regulators inside the pockets, with a view to flatten the very sharp curve of 5,000' radius in the Pinyari lower down to a radius of 15,000 ft.

(b) To introduce head regulators for the two small channels, Fisadi Wah and Wadhu Wah (44 and 16 cusecs respectively), required for local irrigation of lands near Hyderabad, which are very high and cannot be irrigated properly from the Pinyari, specially in the beginning and end of the season when the Pinyari will be running low, without introducing a cross regulator in the head reach of the Pinyari, which is very undesirable. These head regulators were to be included, *without skewing* the Pinyari Head Regulator.

In the proposal (b), the Left Pocket would have increased in length from 588' to 812', while in proposal (a) — viz. skewing Pinyari as well as including the Wadhu Wah and Fisadi Wah head regulators in the pocket — the length would have increased to 850' i. e. the proposed length would have been about half as long again as the originally proposed length of the Pocket.

5.2 Experiments with the layout as proposed in (a) above showed that the short pocket (without skew head regulator for the Pinyari and with length of pocket $\equiv 588'$) was superior to the long pocket in many respects:—

(a) The advantages obtained from shifting the Fuleli Head Regulator into the pocket were largely lost.

(b) While experimenting with different numbers of pocket spans it was found that a greater splay to the face of the left bank head regulator than 1 in 8 (obtained with 8 pocket spans) was undesirable. A skew Head Regulator to the Pinyari has an identical effect and is subject to the same objection. The area of the still water region in the pocket increases and hence, after shoaling, the flow conditions would be worse, for sand exclusion from the canals.

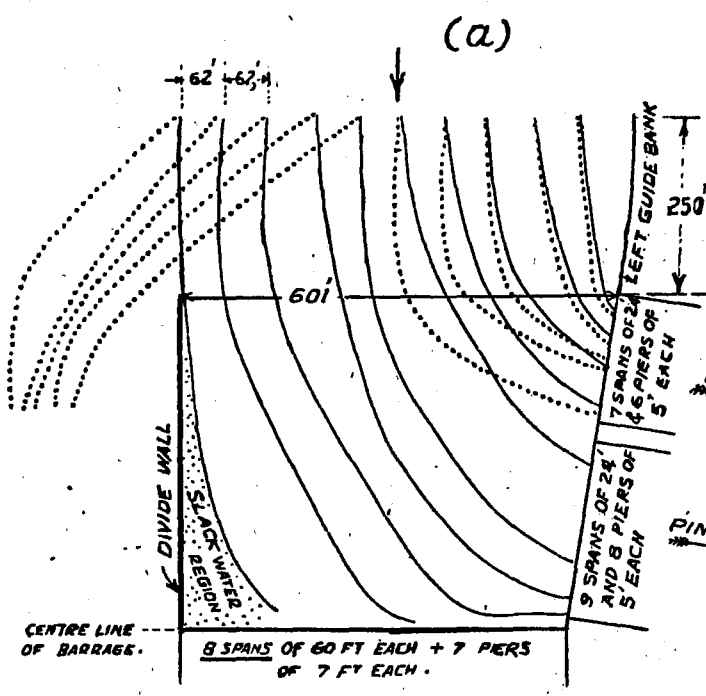
FIG. 2/C-4(II)-1

KOTRI BARRAGE MODEL.
PLAN SHOWING SURFACE
& BED LINES OF FLOW
IN THE LEFT POCKET

$Q_R \approx 200,000$ Cs.

$Q_P \approx 37257$ Cs.

SCALE $V' = 250$

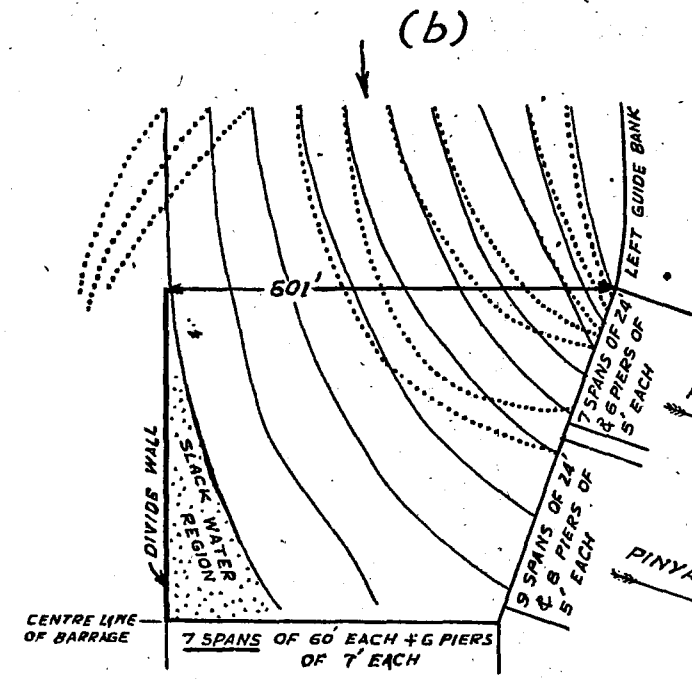


NOTE: BED WATER-OBSERVED AT 250 FT U/S OF FULELI IS MID-DEPTH WATER HIGHER UP DIVING TOWARDS BED AT THIS POINT.

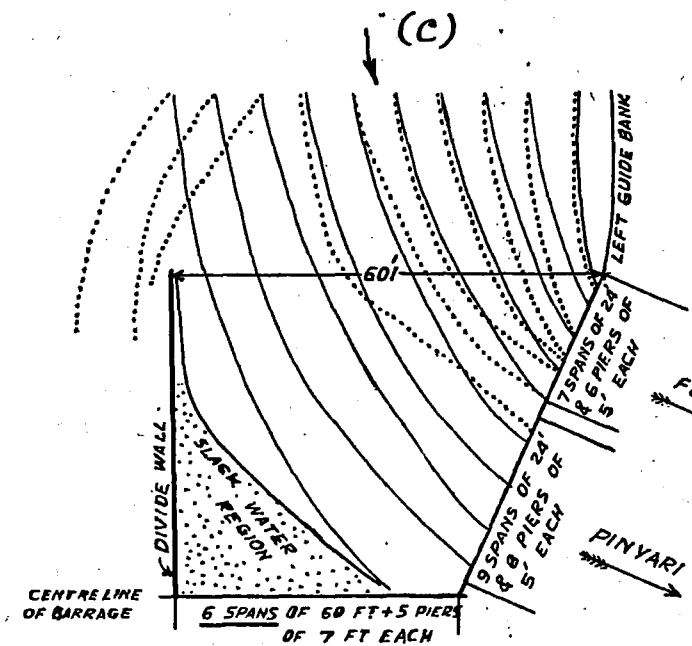
REFERENCES:-

SURFACE LINES OF FLOW. ———

BED LINES OF FLOW. ·····



COMPARISON OF FLOW CONDITION WITH 8, 7 & 6 SPANS IN THE LEFT POCKET.



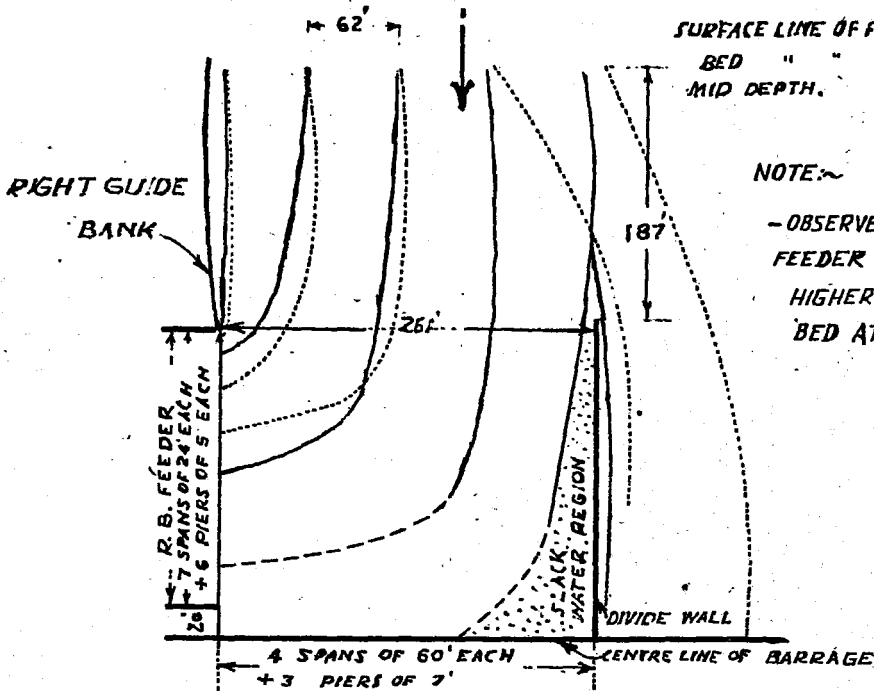
PLAN SHOWING SURFACE AND BED

LINES OF FLOW IN THE

RIGHT POCKET (For $Q_R = 200,000$ Cs.)

SCALE:- 1" = 125'

(a)



REFERENCES:-

SURFACE LINE OF FLOW. _____
BED " " " _____
MID DEPTH. _____

NOTE:- BED WATER -
- OBSERVED AT 187 FT U/S OF R.B. FEEDER IS MID-DEPTH WATER, HIGHER UP, DIVING TOWARDS BED AT THIS POINT.

(b)

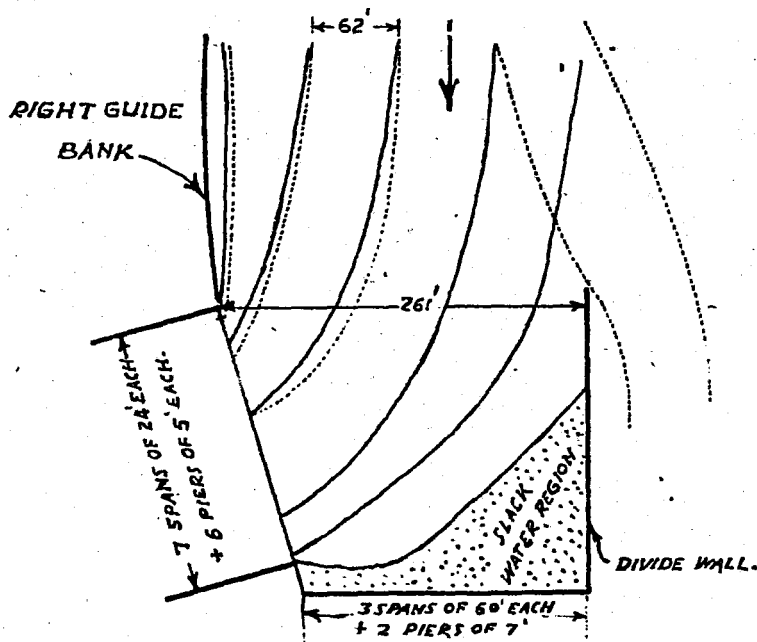


FIG. 4(a)/c-4(ii)-1

PLAN SHOWING THE LEFT POCKET ARRANGEMENT PROPOSED
BY STATION.

Scale :- 1" = 300 feet.

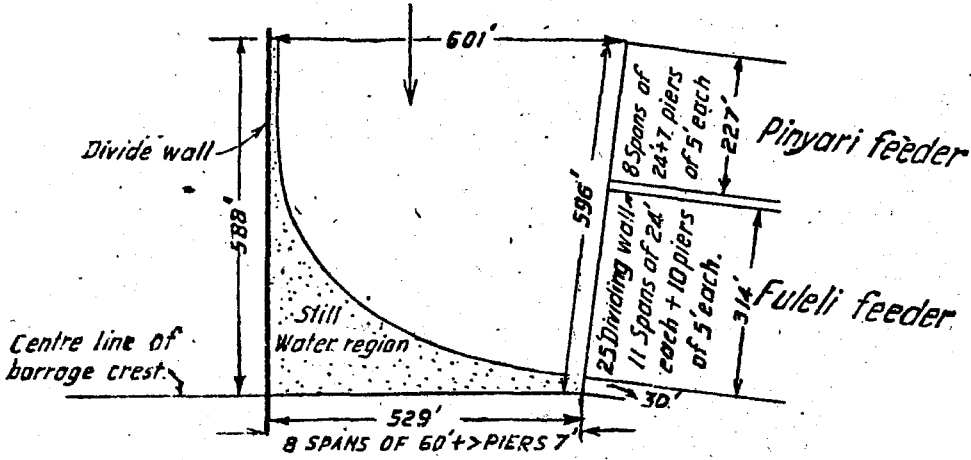
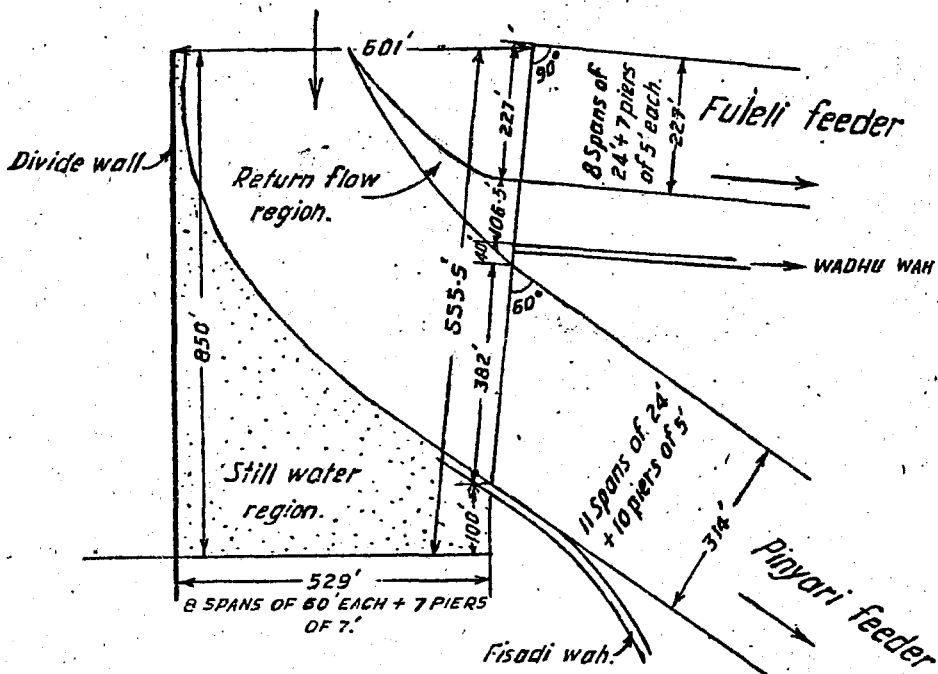


FIG. 4 (b)/c-4 (ii)-1

PLAN SHOWING THE LEFT POCKET ARRANGEMENT PROPOSED

BY SIND.

Scale :- 1" = 300 feet.



(c) It is obvious that a short pocket has less sand to scour away, and as the velocity with the same head available varies approximately as $\sqrt{L/L_1}$ while the weight of solids which a channel can move varies theoretically as V^6 the rate of scour should theoretically vary as $(L/L_1)^3$ and the time required to scour the pocket as $(L/L_1)^4$. Model experiments done in another connection have, however, shown that the theoretical approximation of time required to scour, varying as $(L/L_1)^4$ is on the low side. Thus, a short pocket can be scoured enormously more quickly than a long one. With the original length of pocket (= 2/3rd the length of the proposed pocket) the time required to scour would be less than $(2/3)^4$, i.e. less than 1/5th of the time required to scour the pocket as proposed by Sind.

(d) With the cistern level R.L.37.0 and even the short pocket design, there are already serious limitations to the scouring operations that can be safely and usefully done. For a sufficiently high scouring velocity, during the initial years when retrogression of downstream water levels persists, the scouring operations can only be done in the middle of the Abkalani, when the downstream water level is sufficiently high so as to ensure the formation of a standing wave on the pacca pavement, with a river discharge of 1,50,000 cusecs and above. Later, when the downstream water level rises up to mean specific level, or higher after accretion, scouring operations may be possible at river discharges of 1,00,000 cusecs, or possibly even lower. Even then—since such river discharges will, after the new Punjab Projects, be only obtainable between June and September—the scouring operations will cause undesirable interference with the irrigation operations. Thus, the proposal of lengthening the pocket would increase the duration and difficulties and limitations of efficient scouring operations.

(e) If even frequent and 'long period' scouring operations (which must be made in mid-season) fail to scour the silt at the entrance of the pocket, the velocity in the pocket would increase and cause a relative drop in the V_R/V_P ratio, thereby affecting sand exclusion adversely.

(f) Moreover, the further the nose of the divide wall upstream of the Barrage, the less will be the effect of the beneficial wedge-shaped regulation at the barrage on the curvature of flow at the nose, which helps so greatly in getting more efficient sand exclusion, particularly at certain low stages of the river⁽¹⁾.

(g) Apart from cost, the short pocket reduces the work of cofferdam construction which is already heavy in the first season.

5.3. With a view to make the pocket as short as possible, while providing, at the same time, a high water level to these two small channels, it was suggested by the Director to take of a combined head regulator for the two, upstream of the new head regulator off Fuleli and outside the pocket, and then cross the combined channel over the Fuleli by means of a siphon; the Wadhu Wah would then take off and the balance supply would be siphoned again below the Pinyari for Fisadi Wah. Thus, the channels would get the same water levels as when offtaking from the Pocket, less only the loss of head in siphons, without affecting the Fuleli and Pinyari Canal Systems.

Alternatively, the Director suggested a 5' diameter Hume Pipe Line for Wadhu Wah over or through the 25' wide separating wall between Fuleli and Pinyari to be extended up to the toe of the berm of the Fuleli Feeder, whence it could take off as an open channel, and the Fisadi Wah to be taken off between the Pinyari and the Barrage.

5.4. The alternative proposal of the Station was accepted by Sind, except that in place of the Hume Pipe Line they proposed to carry an open channel on top of the 25 ft. wall between the Pinyari and Fuleli Head Regulators.

They also agreed to drop the proposal of taking the Pinyari Head Regulator at a skew.

With the proposals as accepted, the length of the Pocket becomes 637' as against 588' originally proposed.

6. Design of guide banks :—

6.1 Experiments were carried out to determine the optimum position, alignment and length of the left and right guide banks, with left abutment at 500' from the toe of the Jamshoro bund.

6.2 Necessity of the guide banks :—

When a full rising and falling cycle was run without any guide banks, it was observed that there was a considerable region of slack and/or return flow against both the pucca banks of the river, as a result of which the favourable curvature, which is already partly counteracted by the barrage, was reduced. Hence, it was considered that guide banks on both the banks were essential to lead the water gently to the barrage with proper curvature imposed from a considerable length upstream of the barrage.

6.3 Type of guide banks.

The practice followed so far in aligning the guide banks of a weir is to put in either *parallel* or *converging* (towards the weir) type of guide banks, based on Bell's guide bank system, so as to ensure leading water gently to the barrage or fluming gradually from the noses of guide banks to the Barrage, with a view to give protection against outflanking by the river and to protect as long a length of the marginal bunds as possible.

Of these, the more recent *converging* type has not proved generally successful in that it leads to undue shoaling and oblique entry. The *parallel* type is not suitable for the double barrage design with a central island in between the two arms of the river.

It would have been possible to reduce the length of the guide banks by putting in *converging* type of guide banks, such as at Panjnad, Triumu and Kalabagh, symmetrical with the shape of the central island; but this was considered less suitable as (a) water after leaving the head of the guide bank in a tangential direction approaches the barrage spans with a very high velocity, thus increasing the concentration of discharge in certain spans, and (b) there still remains a slack water region against the pocket and the guide bank. This necessarily distorts the natural favourable conditions of curvature existing at the barrage site.

Moreover, it has been observed that both these type of guide banks create unpredictable, sometimes even dangerous, conditions by attracting flow and pinning it to their heads and upset the normal expectations of their working unless associated with other training works such as spurs or islands as correctives; and these ancillary training works are generally undesirable as they create turbulence just where smooth flow is the prime essential. Hence, it was decided to have a type of guide bank that would *maintain* the existing favourable conditions at the site.

6.4 Alignment of the guide banks.

The following were the main considerations in fixing the alignment:—

(A) The overriding consideration is to have an alignment that will distort existing flow conditions to the least extent and make the best use of the river energy to develop further suitable conditions—such as a deep channel along the guide bank so—as to maintain the favourable curvature even under the worst post-barrage conditions.

(b) It was also important to see that water followed the guide banks at all river stages under different conditions of regulation of the barrage,

especially at the high stages of discharge when it is more difficult for water to follow too sharp a curve.

At high river stages, with the gates fully open, there is little ponding over the natural water levels and, thus, there is sufficient latitude for the river to move freely and follow the same flat curvature as it does under pre-barrage conditions. On the other hand, at a low stage of discharge, though the natural tendency of the river without a barrage is to have a sharp curve, with the barrage unnatural conditions occur—such as ponding over the natural river water levels—and water can only follow the same flat curve as at high stage discharge conditions. Thus, the range of curvature which water can follow is *reduced* under post-Barrage conditions. The best alignment generally suitable for all stages was evolved by trial and error and is shown in fig. 5.

(c) Finally, the alignment of the guide bank should, as far as possible, avoid the deep water channel, with a view to economy.

6.4.1. *Left Guide Bank* :— Accordingly, an alignment of the left guide bank, concave throughout its length, was fixed in the model by demarcating the boundary of the still water region along the pucca edge of the river and then putting the guide bank along this line. The final alignment of the guide bank, with abutment at 325' from the toe of the Jamshoro bund, is shown in Fig. 5.

Fig. 5
C-4 (ii)-1

6.4.2 *Right Guide Bank* :— It will be seen, from Fig. 5, that there is a sharp change of curvature on the right bank between cross sections miles $4\frac{1}{2}$ to $5\frac{1}{2}$ under the existing conditions of the river. It was observed that curvature, somewhat counteracted by the barrage, became considerably flatter and water did not follow the existing change of curvature between these cross sections.

Fig. 5
C-4 (ii)-1

The alignment of the right guide bank was determined by the same trial method as that adopted for the left guide bank, the transition from convex to concave curve being made as gradual as possible so that water followed the curve all along its length. Moreover, it was observed that it was desirable to have a guide bank sufficiently long, extending to midway between C. S. mile 5 and $5\frac{1}{2}$, so as to enable it to attract with the aid of its pitching some more flow towards the right arm channel before it is "captured" by the central island. This would secure or improve the conditions of the right arm channel by as natural a method as possible without the undesirable necessity of forcing down by regulation of gates a higher percentage of discharge, which is generally attended with greater turbulence and throwing of sand into suspension to be drawn by the canal. The right guide bank is also taken sufficiently (about 250 ft.) riverwards to facilitate flattening of the transition curve from convex to concave, and to counteract to some extent the damping effect on velocities caused by the ponding at the barrage, thus minimising the formation of undesirable shoals along the guide bank and in the middle of the river.

6.5. *Length of the guide bank* :— The lengths of the left and right guide banks with the left abutment at 325' from the toe of the Jamshoro bund, would be 5900' and 6650', respectively.

6.6. *Position of the left guide bank* :— With a view to counteract the "attracting" effect of the Central Island at low discharge stages, it was believed possible to improve exclusion by positioning the left guide bank well inside the river (as shown in Fig 9/c-2 (iii)-3). But it was observed that, contrary to expectations, this had an adverse effect on exclusion, as this position of the guide bank twisted the flow and made it considerably oblique to the pocket. This oblique flow increased the crucial velocity ratio of the velocities just inside and outside the pocket impairing the efficiency of exclusion.

Fig. 9 of item
C-2 (iii)-3

6.7. *Side pitching for guide banks* :— The preliminary experiments for evolving the best alignment of the guide bank were made with G. I. sheets to

facilitate quick changes in the model. Subsequently, the guide banks were constructed to these alignments in mud with pitching to scale. It was observed from the indications of the formation of ripples along the bed of the river near the guide banks that pitched guide banks would enable maintaining a deep channel on the left bank and, similarly, attracting the flow along the right bank and would thus reduce the possibility of the formation of shoals along the guide banks on either side.

7. *Location of the left abutment* :— From the original plan and cross-sections, a position of the left abutment, 500 ft. riverwards from the toe of the Jamshoro bund, was selected with a view to avoid encroachment on the 'Mukam' area (burial ground) as well as to prevent the flow impinging on the Jamshoro Bund downstream of the Barrage. The initial experiments were made with this position of the abutment. Later, when a new cross section was observed along the barrage line, after the recession of the floods of 1946, it was observed that the abutment at 500' would be located in very deep water and thus would increase the cost and constructional difficulties. It was, therefore, proposed by the Station that the abutment be located at 275' from the toe of the Jamshoro bund and Government sanction be obtained for the submergence of the old and disused Mukam.

Comparative tests made with the left abutment at 275' and 500' from the bund showed the following additional advantages in favour of the former :—

(a) A much longer left guide bank — actually 2520' longer, i. e. about half as long again — was required with abutment at 500' from the toe of the Jamshoro bund.

(b) With the left abutment set further riverwards into deep water, the left guide bank also had to be in deep water. For the optimum alignment of the guide bank, designed to give the most satisfactory sand exclusion and minimum region of return flow along the guide bank, a fairly long length of the longer guide bank would then have to be located in deep water.

These would have caused serious difficulties during construction and maintenance as well as increased the initial and recurring costs.

(c) The curvature imposed by the guide bank with the nearer position of the abutment followed the existing natural curvature of the left bank more closely with consequent advantages.

With the best of efforts, sand exclusion from the left bank oftakes was not as satisfactory when the abutment and guide bank were set so far into the river.

7.1 In addition to the above-mentioned advantages evident from the model tests, there are the following other equally important considerations :—

(a) The location of the abutment further from the bank will reduce the available waterway during the first season's cofferdam by 225 ft. with adverse effects on the river course, increasing construction difficulties by way of greater scour, attack at key points, and possible damage to the existing river bunds.

(b) This might also lead to some increase in the afflux during construction and so add to the cost of the cofferdam work itself.

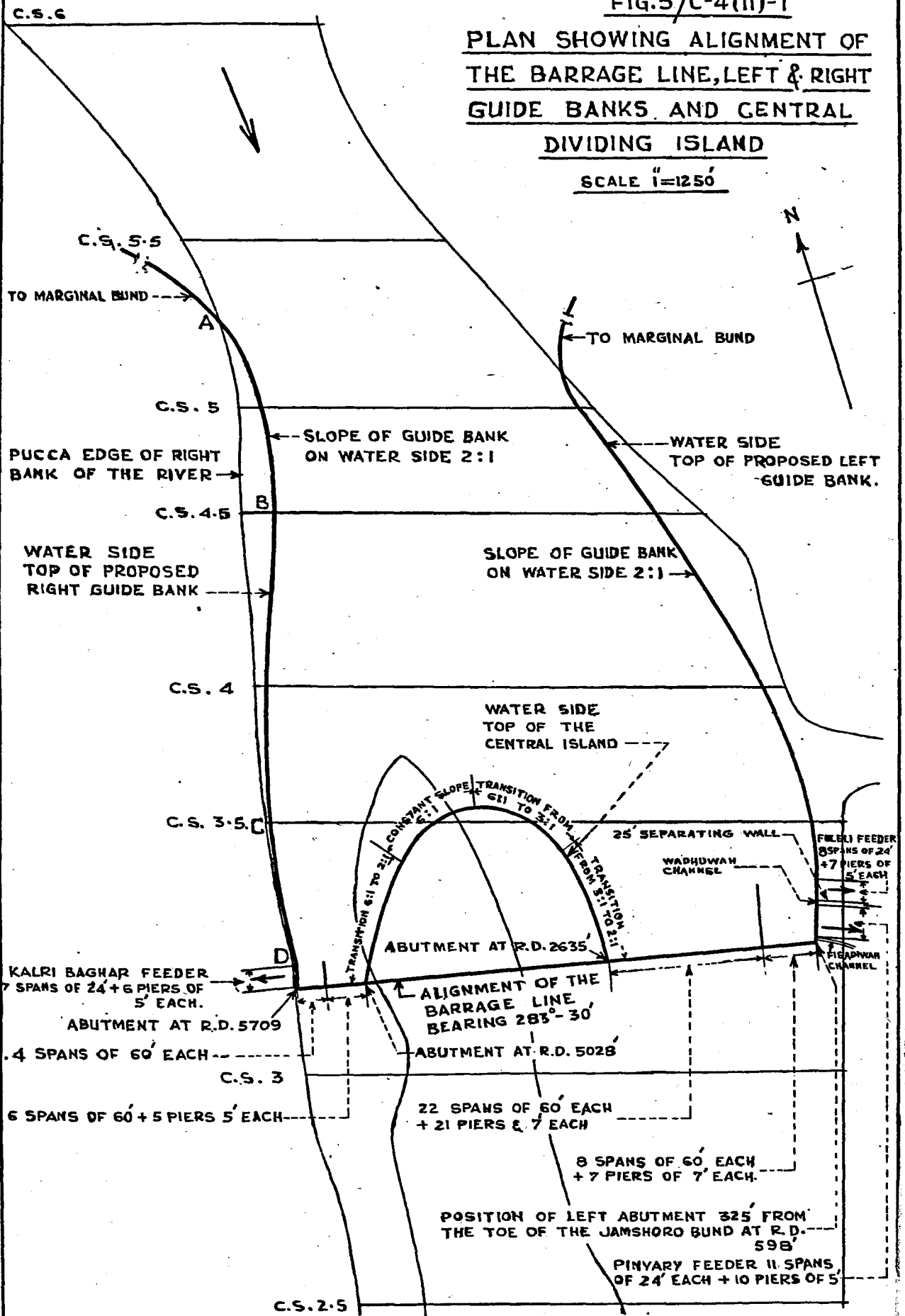
The constructional difficulties following the location of the abutment in deep water would be serious and might necessitate a reduction in the first season's work and an increase in the period of construction, causing additional overhead charges (interest charges, working expenses, maintenance during construction) during one more year and, at the same time, a delay in the revenue returns by a year.

7.2 For all these reasons, the Director strongly recommended that Government should be apprised of the consequences of letting the Mukam interfere with the best design and construction plan of the Barrage.

FIG.5/C-4(ii)-1

PLAN SHOWING ALIGNMENT OF
THE BARRAGE LINE, LEFT & RIGHT
GUIDE BANKS AND CENTRAL
DIVIDING ISLAND

SCALE 1"=1250'



7.3 It has since been decided to shift the main tomb in the 'Mukam', with due ceremony, to another site; with the proviso that there will be no excavation in the 'Mukam' area, though it may be covered. The left abutment is thus finally approved to be located 325 ft. from the toe of the Jamshoro Bund.

8. *Left and right pocket divide walls.*

8.1 *Left Pocket Divide Wall.*

Experiments were carried out with the left Divide Wall extending up to the upstream abutment of the Fuleli Canal as well as with length = $2\frac{1}{3} \times$ the length of the pocket; a shorter Divide Wall brings the control point (i. e. Divide Wall nose) nearer the barrage and would be certainly better from the point of scouring operations; and had it not been for the large draw-off of the uppermost canal (i. e. Fuleli Feeder, withdrawing 16,600 cusecs), the shorter Divide Wall would have been preferred to the longer one. It was, however, observed that the longer Divide Wall was, in this case, far better in respect of sand exclusion, especially at the low, crucial stages of river discharge: with it practically all the bed seeds dropped, even at cross section 5, were excluded from the canals, at all stages of the rising and falling cycle, under 'still pond' conditions. The longer Divide Wall is better because of the larger 'sphere of influence' of the scour round the nose of the divide wall capturing the bed flow from about 200 ft. beyond the upstream abutment of the uppermost canal, leaving little opportunity for the Fuleli Feeder to draw bed sand into it: while with a shorter Divide Wall, the scour hardly extended up to the upstream abutment of the Fuleli and thus failed to deflect the bed material away from the Fuleli in to the barrage spans. Hence, the left Divide Wall should extend up to the upstream abutment of the Fuleli Feeder.

8.2. *Right Pocket Divide Wall.*

On the right side there is only one canal of 8,484 cusecs capacity

There is thus little to choose between a short or long Divide Wall on the right side from the point of having the Divide Wall nose as near to the barrage as possible for efficient scouring operations, as the length of the Head regulator is only 198 ft. (7 spans of 24' + 6 piers of 5') —hardly one third the aggregate length of the left Head Regulators.

On the other hand, a full length Divide Wall will be better on the right side, because

(a) With a shorter Divide Wall the loose stone protection upstream of the pacca pavement of the barrage extends up to the nose of the Divide Wall and may retard the free development of the scour-hole round its nose, which (the scour-hole) helps to deflect the bed sand away from the pocket, particularly at low stages of discharge.

(b) On the other hand, heavy action at the nose of the shorter divide wall is *likely* to disturb and distort the upstream protection of the pocket and the barrage spans. From this point of view even a longer wall is desirable.

(c) A longer divide wall provides a better shield against the entry of bed material that is likely to be thrown into suspension by the turbulence at the nose of the central pitched island.

8.3. *Depth and extent of free scour-holes round the noses of Divide Walls.*

Rising and falling cycles were run, this time up to $Q \equiv 5,50,000$ cusecs, under 'still pond' condition (which is the *worst* condition for depth and

extent of scour) till the scour-holes stabilized. The equivalent R. L. s of bottom of scour were observed to be R. L. 15.0 and R. L. 25.0 in the left and the right bank scour-holes, respectively.

As regards their shape and extent, allowance has to be made for the vertical exaggeration of the model (= 5.0). Previous experiments carried out at the station have shown that depths of scour obtained in a vertically-exaggerated model are proportionally correct; while the extent of scour has to be adjusted between that obtained in a geometrically-similar model and in the vertically-exaggerated model. In this case, accordingly, the latitudinal and longitudinal scales will approximately be 1/125.

Fig. 11
C-2 (ii)-3

Figure 11/C-2 (iii)-3 shows the prototype contours of the scour-holes for the left and right pocket, respectively, derived with due allowance for V. E. from the scour-holes obtained in the model.

8. 4 Apron protection round the noses of Divide Walls and along their shanks.

In these cases, the design is still more complex unless assisted by specific model experiments for each case, as the shape of the scour would also vary according to the flow conditions existing round the nose and along the shank of the divide wall.

(a) Top of apron protection.

So far, the practice has been to lay a flat apron protection round the nose of the Divide Walls. But as with such protection many of the stones are likely to be washed away long before the apron stabilises to suit the extent and shape of the scour-hole, it was proposed to lay the apron according to the shape of the scour-hole formed at the nose. Accordingly, in the Kotri Barrage design, it was first proposed to lay the top of concrete block protection at R. L. 23. 0. As this would entail deep excavation and, consequently, great constructional difficulties and increased pumping costs, it is proposed to keep the top at R. L. 37. 0 (the depth up to which excavation has to be done, in any case, for the downstream cistern):

Experiments were carried out to see whether there was a substantial difference in exclusion with apron top laid alternatively at R. L. 37. 0 and R. L. 23. 0.

Figs. 6 (a)
(b) & (c)
C-4 (ii)-1

Figures 6 (a), (b) and (c) show the lines of flow at the nose observed under free scour, and with apron tops at R. L. 37. 0 and at R. L. 23. 0, respectively. It is evident from the bed line divide tapped by the Pocket that the exclusion is best under free scour and that there is little or no difference in exclusion with apron tops at R. L. 37.0 or R. L. 23.0.

Moreover, it was observed that for scouring operations of the pocket, due to greater concentration of flow with apron top at R. L. 23.0—the apron top being low—scouring action was feeble along the upstream face of the Fuleli Head Regulator : whereas with the apron top at R. L. 37, the scouring of the pocket was more or less uniform throughout its length. Hence, the apron top may be kept at R. L. 37.0.

Fig. 7
C-4 (ii)-1

(b) Extent of protection.

Figs. 8 (a)
& (b)
C-4 (ii)-1

Figures 8 (a) and (b) show the extents of protections required for the left and right Pockets, respectively, for the *worst* conditions to which they would be subject (e. g. the Pocket side protection is designed for scouring operations when the discharge intensity would be greater). The protection should slope from contour R. L. 46. 0 to R. L. 37.0 at the nose.

Statement
I.
C-4 (ii) 1

Detailed calculations for the quantities of protection are given in statement I attached.

Experiments carried out after laying the protection according to the final shape showed the necessity of laying the apron protection along the shanks of the Divide walls gradually sloping from R. L. 46. 0 of the stone protection of the floor to R. L. 37. 0 of the top of protection at the nose of the Divide walls, the width of the protection being 36 ft.

(c) Nature of protection.

It had been proposed to use 4' x 4' x 3' concrete blocks for this

FIG. 6/C-4(ii)-1.

(a)

PLAN SHOWING LINES OF FLOW WITH NATURAL SCOUR ROUND THE NOSE OF THE LEFT DIVIDE WALL. SCALE. 1" = 250'

NOTE :- BED WATER - OBSERVED AT 250 FT U/S OF FULELI IS MID-DEPTH WATER HIGHER UP, DIVING TOWERS BED AT THIS POINT.

(b)

PLAN SHOWING LINES OF FLOW WITH APRON ROUND THE NOSE OF THE DIVIDE WALL LAID IN A CUP SHAPE WITH TOP AT R.L. 37.0

(c)

PLAN SHOWING LINES OF FLOW WITH APRON ROUND THE NOSE OF THE DIVIDE WALL LAID IN A CUP SHAPE WITH TOP AT R.L. 23

REFERENCE :-

..... BED FLOW LINES
—— SURFACE FLOW LINES

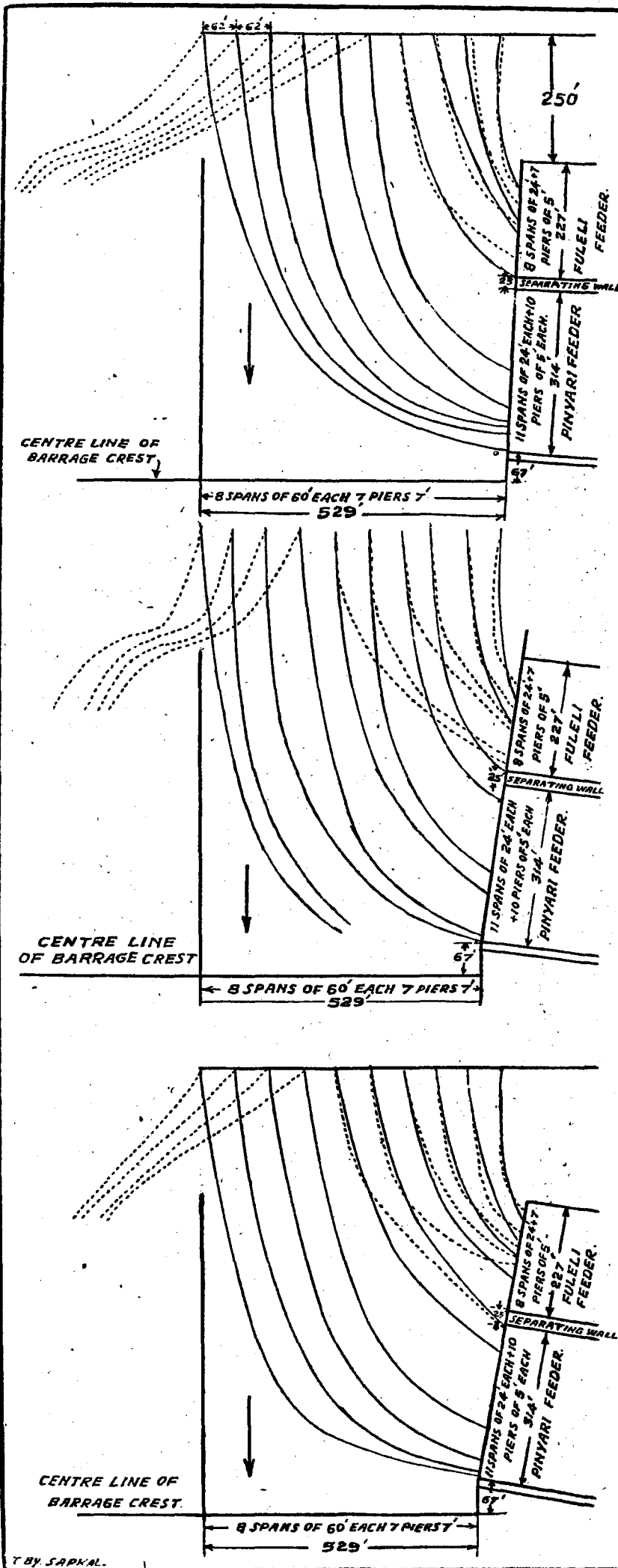
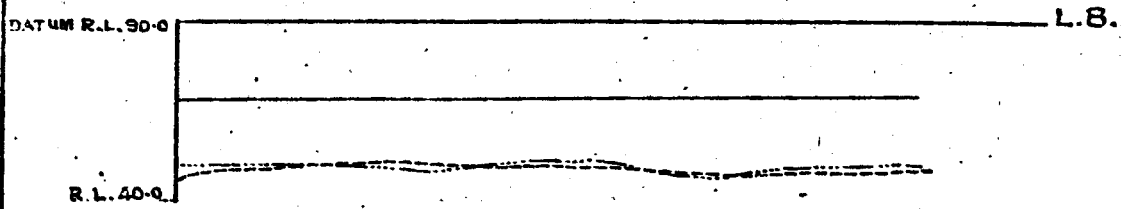
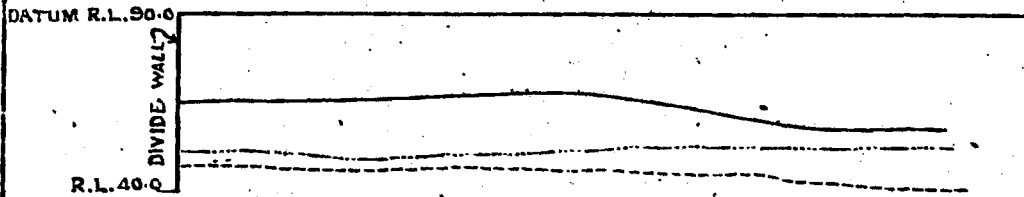


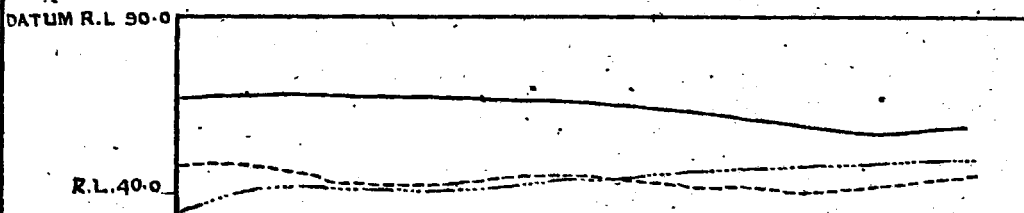
FIG. 7/C-4(ii)-1.
SECTION A-A



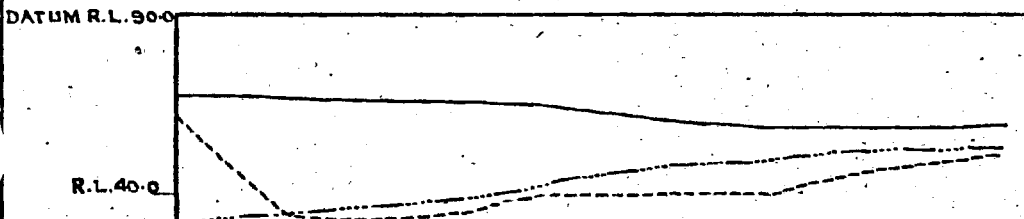
SECTION B-B



SECTION C-C



SECTION D-D



SECTION E-E

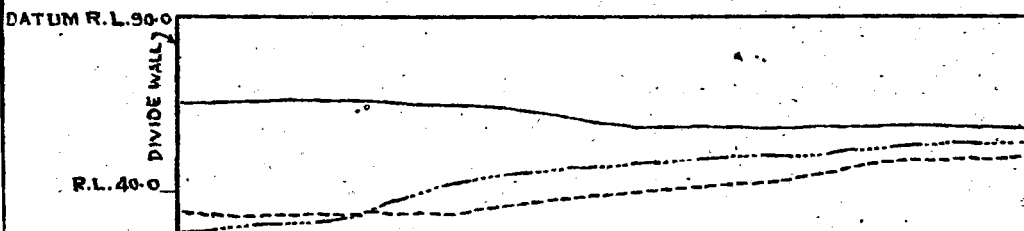
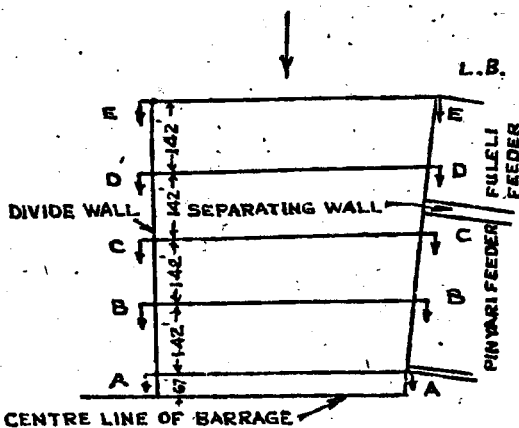


FIG. 7

LEFT POCKET SHOWING
THE POSITION OF CROSS SECTIONS
SCALE 1" = 375'



SHOWING ADVANTAGE OF
SCOURING OPERATIONS
WITH TOP OF DIVIDE WALL
APRON AT R.L. 37.0
REF.

- POCKET BED AS LAID BEFORE SCOURING OPERATION.
- - - POCKET BED AS OBSERVED AFTER SCOURING OPERATIONS WITH TOP OF APRON R.L. 37.0
- POCKET BED AS OBSERVED AFTER SCOURING OPERATIONS WITH TOP OF APRON R.L. 23.0

FIG. 8(a)/C-4(ii)-1
PLAN SHOWING THE SHAPE OF APRON
TO BE LAID ROUND THE NOSE OF THE
LEFT DIVIDE WALL.

Scale. 1" = 250'

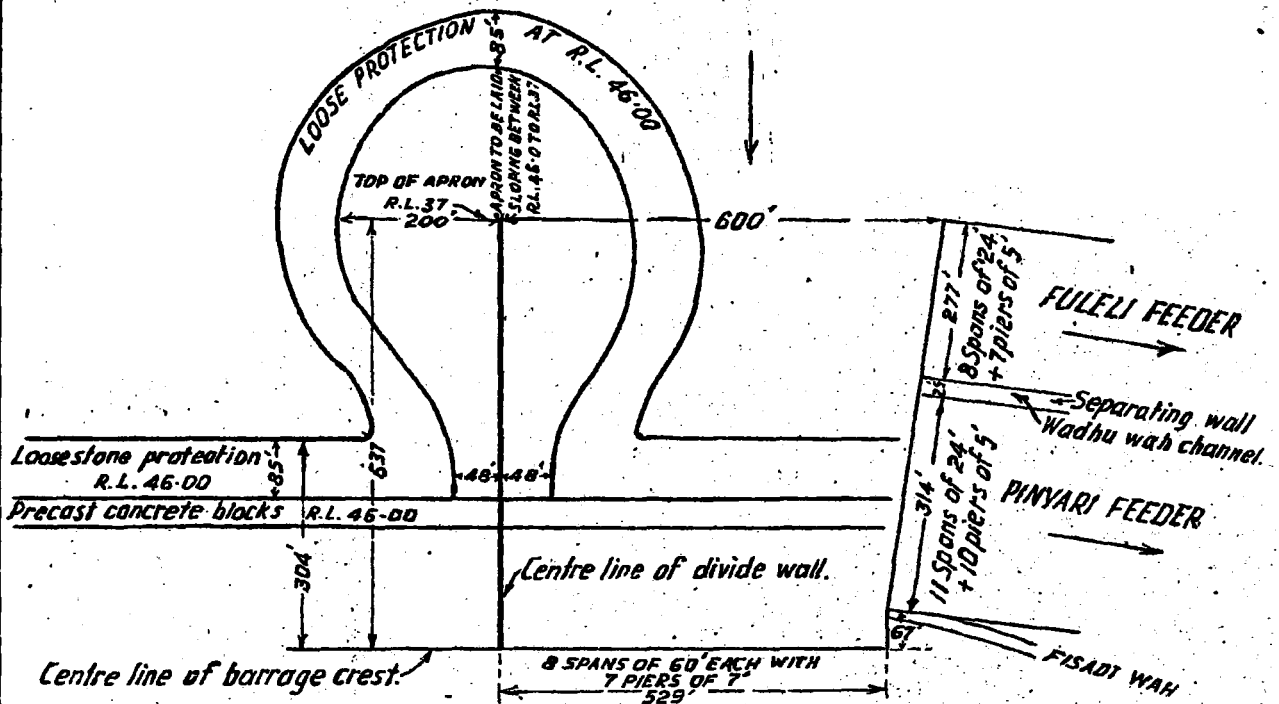
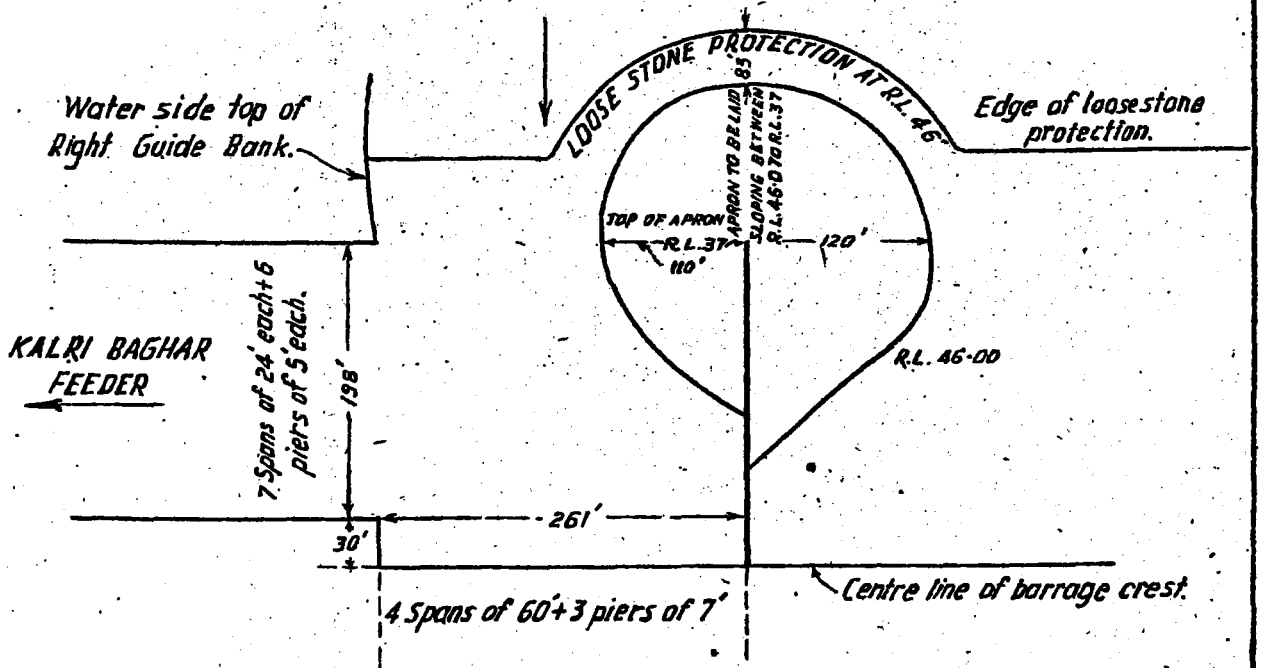


FIG. 8(b)/C-4(ii)-1
PLAN SHOWING THE SHAPE OF APRON
TO BE LAID ROUND THE NOSE OF THE
RIGHT DIVIDE WALL

Scale. 1" = 125'



protection. Each block will weigh about 3 tons. Handling and laying these blocks will be difficult; what is more important, the blocks will not settle uniformly in the way a launching apron made of ununiform stones does; in view of this it appears better to lay this protection with two to three-men-stone (each stone 200-300 lbs.), i. e. a heavier protection than the usual one made up of one-man-stone (90-100 lbs.) for lining the scour reasonably uniformly against the severe action at the nose.

9. Tests for Exclusion under the worst post-Barrage conditions.

It was observed that under the post-barrage conditions, water did not abruptly turn round the corner G shown in Fig. 9. but showed a tendency to straighten. The brunt of the attack was borne by the nose of the Central Island.

Fig 9
C-4 (ii)-1

Figs. 9 & 10 show, respectively, the lines of flow and the contour plan of the river reach upstream of the barrage after running four consecutive rising and falling cycles with a very heavy charge of fine red silt (6 parts per 1000). Under these extreme super-charge conditions, silting was indicated along the left and right Guide Banks; with normal charge, although silting is very unlikely on the left bank, nevertheless the model was tested for the worst possible conditions, and so the one bed line entering the Fuleli (because of greater concentration)—shown in figure 11—may be disregarded.

Figs. 9 & 10
C-4 (ii)-1

Fig. 11
C-4 (ii)-1

It is expected that the pitched right Guide Bank will tend to attract the flow and thus somewhat reduce the natural tendency of berm formation near the right bank. Model limitations preclude clear indications whether the attracting effect of the pitching will dominate over the natural tendency for berm formation. Even under the worst hypothesis of the attraction of the pitched right bank being wholly ineffective and the flow being more attracted to the nose of the Central Island, the model showed that a channel will persist round the nose towards the right pocket with favourable curvature for satisfactory sand exclusion. This was tested in the model by laying a berm of fine silt up to R. L. 68.0, along the edge of R. L. 55 contour, which did not cause a deterioration in exclusion.

Fig. 11
C-4 (ii)-1

10 Conclusions :-

(a) Sills of the Head Regulators (para 3).

The sills of the Pinyari and Fuleli Head Regulators are to be maintained at R. L. 52.0 i. e. flush with the pocket crest or 6' higher than the upstream pavement level (R. L. 46.0).

(b) Design of the left and right pocket and the optimum number of spans (para 4).

8 spans and 4 spans of 60' each were found suitable for the left and right pocket, respectively, from considerations of optimum scouring velocities, downstream cistern levels, shoaling in the pocket, and satisfactory V_R/V_P ratio which governs the curvature of flow.

(c) Long pocket versus short pocket (para 5)

The short pocket 637 ft. long was found to be superior to the long pocket 850 ft. long in respect of scouring operations, quantity of sand to be scoured and less time for scouring, reduction of cofferdam work, and the control point (i. e. divide wall nose) being nearer the barrage.

It is finally decided to have the left pocket 637' long with an open channel for Wadhuwah through the 25' common wall between the Fuleli and Pinyari Head Regulators, Fisadiwah offtaking upstream of the Pinyari Head Regulator through two hume pipes, 5' diameter each.

(d) Design of guide banks (para 6).

The left guide bank as recommended is 5900' long, concave throughout its length, so that it does not distort the existing conditions, avoids the deep-water channel and allows water to follow it at all the river stages.

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The Right Guide Bank is recommended to be 6650' long, convex-concave in alignment.

(e) *Location of the left abutment (para 7).*

The left abutment is decided to be 325' from the toe of the Jamshoro bund, so as to have a short guide bank, less distortion of natural curvature, minimum reduction of waterway during the first season's cofferdam work and less difficulties during construction of the abutment.

(f) *Left and right pocket divide walls (para 8).*

(i) The left and right pocket divide walls extending up to the upstream abutment of the Fuleli Feeder and the right bank Feeder respectively, were found better than the shorter divide walls.

(2) It is also recommended that the apron protection round the noses of divide walls be laid in a cup-shaped pattern conforming with the extent and depth of the natural scour-holes.

(3) As the concrete blocks 4' x 4' x 3' (proposed by Sind) are not expected to settle uniformly, it was recommended that the apron be laid with 'two- to three-men' stones.

FIG. 9/C-4(ii)-1
PLAN SHOWING LINES OF FLOW
FOR Q = 200,000 Cs.
AFTER RUNNING FOUR CONSECUTIVE
RISING AND FALLING CYCLES.
SCALE - 1" = 2500 FT.

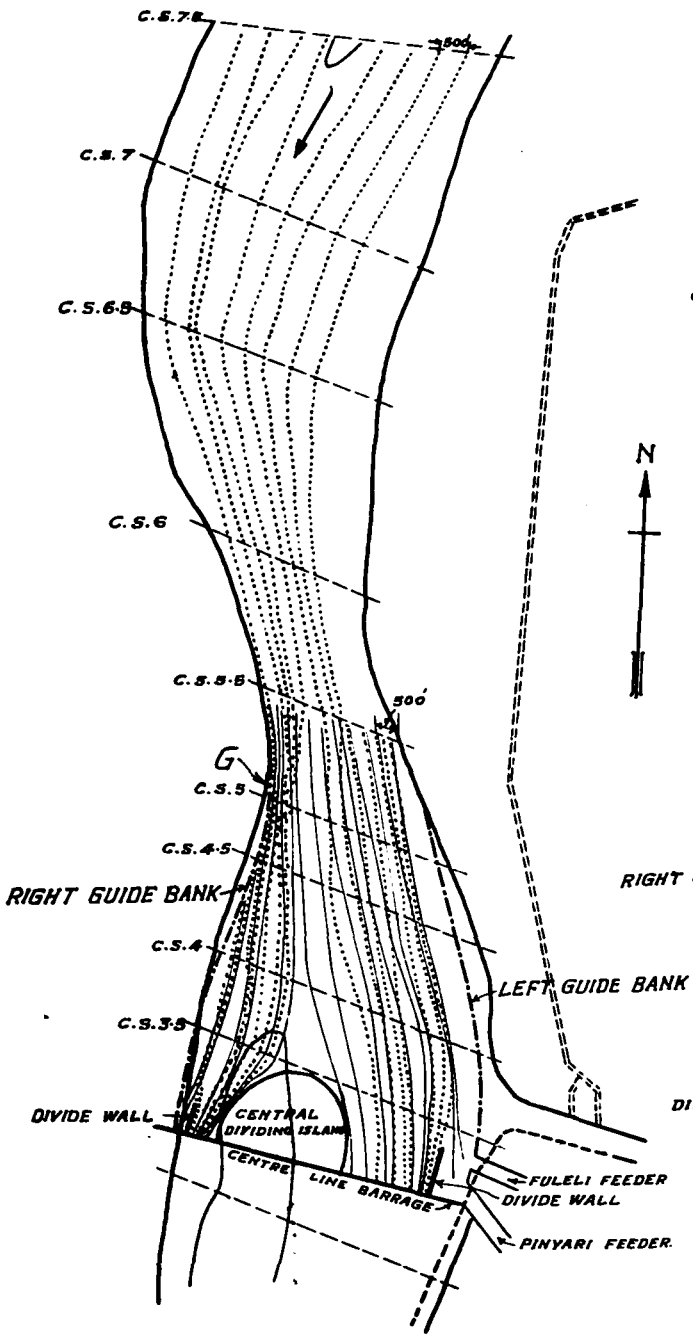


FIG. 10/C-4(ii)-1
PLAN SHOWING CONTOURS AS OBSERVED
AFTER RUNNING FOUR CONSECUTIVE RISING
AND FALLING CYCLES WITH HEAVY
CHARGE OF FINE RED SILT.
SCALE - 1" = 2500

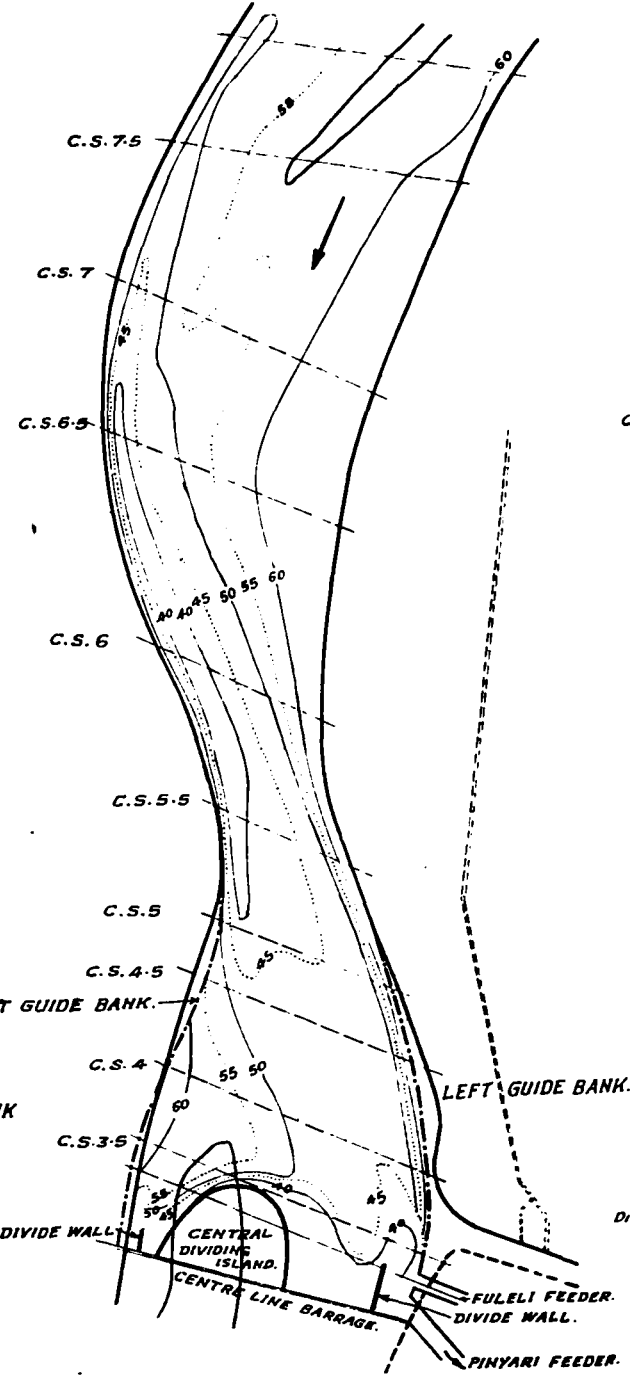
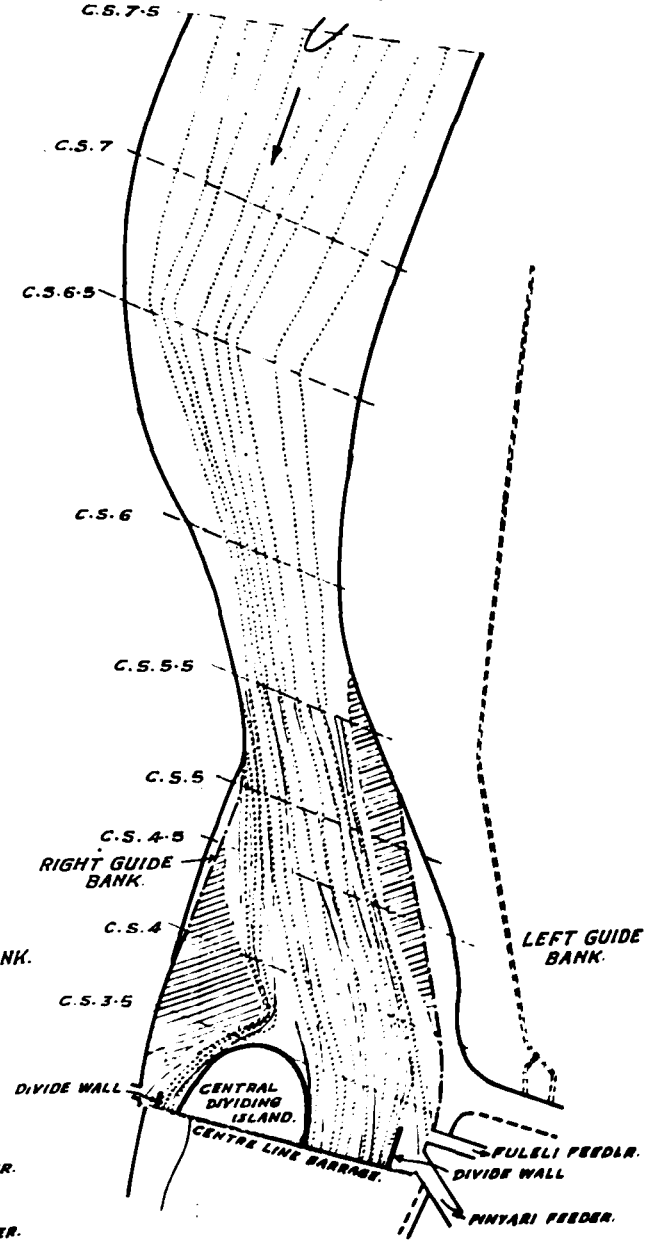


FIG. 11/C-4(ii)-1
PLAN SHOWING LINES OF FLOW
FOR Q = 200,000 Cs.
AFTER REPRODUCING SILTING ALONG
THE LEFT & RIGHT GUIDE BANKS.
(Shown in Hatched lines)
SCALE - 1" = 2500'



Reference:-
Surface lines of flow. —————
Bed lines of flow.

APPENDIX I / C-4 (ii)-1

Calculations for V_R / V_P ratios for different arrangement of spans in the left pocket

As width at entrance to the pocket is kept constant (viz. 601 ft.), V_P will be constant in all cases.

$$V_P = \frac{37,287}{601 \times 22} \quad \text{at Pond R. L. 68.0}$$

$$= 2.82 \text{ ft. / sec.}$$

(a) 8 left pocket spans :-

Final width at the nose of the left divide wall = 1625'. The model showed 80 : 20 distribution in the left and right arms of the river, respectively, for satisfactory sand exclusion.

∴ discharge in the left arm = 1,60,000 cusecs.

∴ The discharge through the left arm weir spans = 1,12,734 cusecs.

$$∴ V_R = \frac{1,12,734}{1625 \times 22} = 3.15 \text{ ft./Sec. with Pond R. L. 68.0}$$

$$∴ V_R / V_P = 1.12$$

(b) 7 left pocket spans :-

River width at the nose as one more span is added to river spans, will be increased to 1625' + 67' = 1692'

$$∴ V_R = \frac{1,12,734}{1692 \times 22} \quad \text{with pond R. L. 68.0}$$

$$= 3.02 \text{ ft./sec.} \quad ∴ V_R / V_P = \frac{3.02}{2.82} = 1.07$$

(c) 6 left pocket spans :-

River width at the nose of the divide wall = 1692' + 67' = 1759'

$$∴ V_R = \frac{1,12,734}{1759 \times 22} = 2.92 \text{ ft./sec. with Pond R. L. 68.0}$$

$$∴ V_R / V_P = 2.92 / 2.82 = 1.03$$

APPENDIX II/C-4 (ii)-1

Comparison of velocities during scouring operations with different number of left pocket spans, 8, 7 and 6, respectively.

1. The optimum intensity of discharge with $Q_R \equiv 1,00,000$ cs.—the only practicable discharge which would be available at the time. scouring operations would be done—with 8 left pocket spans, and the cistern level not lower than R. L. 37.0 is 115 cusecs, V/V_0 at the entrance to the pocket being $5.47/3.83 = 1.43$. The discharge in the left pocket during the scouring operations would, therefore be 55,200 cusecs.

2. Scouring velocity for 6 left pocket spans :-

(i) If the number of pocket spans be reduced to six spans only, either the cistern level would require to be lowered to get the same scouring velocity or the scouring will have to be carried out with a low V/V_0 ratio, as under:

Assuming 55,200 cusecs be passed down the 6 left pocket spans—

$$q = \frac{115 \times 480}{360} = 153.33 \text{ cusecs/ft.}$$

Under worst conditions, i. e. with maximum retrogression, and scouring operations done at the river discharge of 1,00,000 cusecs,

$$\text{Downstream W. L.} = 51.30$$

$$c \text{ (in } q = cH^{1.5}) = 3 \text{ for low submergence, } \therefore q/c = 51.11$$

$$\therefore H = (q/c)^{\frac{2}{3}} = 13.80$$

$$\therefore \text{Upstream T. E. L.} = 52 + 13.80 = 65.80$$

$$\therefore V_a = \frac{55,200}{601 \times (65.45 - 46)} = \frac{55,200}{601 \times 19.45} = 4.72 \text{ ft/sec.}$$

$$\therefore V_a^2/2g = h_a = 0.35 \text{ ft. Therefore, upstream W. L.} = 65.45$$

$$V_r = \frac{55,200}{(6 \times 60' + 7' \times 5) (61.3 - 37)} = \frac{55,200}{396 \times 14.30} = 9.80' \text{ /sec.}$$

$$\therefore h_r = V_r^2/2g = 1.50$$

$$\therefore \text{T. E. L. downstream} = 51.30 + 1.50 = 52.80$$

$$\therefore H_L = 65.80 - 52.80 = 13.0 \text{ ft.}$$

$$\therefore Ef_2 = 19.60 \text{ ft.}$$

$$\therefore \text{Cistern level} = 52.80 - 19.60 = 33.20 \text{ R. L.}$$

$$V_0 \text{ in the pocket} = 0.63 \times (19.45)^{0.64} = 4.25 \text{ ft./sec.}$$

$$\therefore V/V_0 = 4.72/4.25 = 1.11 \text{ (Very poor).}$$

Hence, even were it possible to lower the cistern to R. L. 33 (which it is not), the V/V_0 ratio, would be too low for efficient operations.

(ii) To find the scouring velocity ratio to suit the pre-determined cistern level of R. L. 37.0 with 6 left pocket spans :-

Assume $q = 100$ cusecs.

$$\therefore q/c = 100/3 = 33.33$$

$$\therefore H = (q/c)^{\frac{2}{3}} = (33.33)^{\frac{2}{3}} = 10.40 \text{ ft.}$$

$$\therefore \text{T. E. L. upstream} = 62.40$$

$$V_a = \frac{36,000}{601 \times 16.15} = 3.70 \text{ ft/sec.}$$

$$h_a = 3.7^2/2g = 0.22 \text{ ft.}$$

$$\therefore \text{Upstream W. L.} = 62.18 \text{ ft.}$$

$$\therefore V_r = \frac{36,000}{395 \times 14.30} = 6.4 \text{ ft/sec.}$$

$$\therefore \text{T. E. L. downstream} = 51.30 + 0.64 = 51.94$$

$$\therefore H_L = 62.40 - 51.94 = 10.46 \text{ ft.}$$

$$\therefore Ef_2 = 14.90 \text{ ft.}$$

$$\therefore \text{Downstream cistern level} = 51.94 - 14.90 \\ = 37.04$$

Hence, with a discharge intensity of $q = 100$ cusecs/ft the cistern level is suitable; at this intensity $V_a = 3.70$ ft/sec.

$$V_o = 0.63 (16.18)^{.64}$$

$$= 3.78 \text{ ft./sec.}$$

$$\therefore V/V_o = 3.70/3.78 = 0.97.$$

which is useless for effective scouring operations.

Thus, six spans are *unsuitable* as either requiring a lower cistern, or giving too low a velocity, to enable scouring operations to be successful.

2. *Scouring velocity for 7 left pocket spans :-*

(i) Assuming same river discharge during scouring operations as for 8 spans,

$$q = \frac{480 \times 115}{420} = 132 \text{ cusecs/ft.}$$

$$\therefore q/c = 132/3 = 44.0$$

$$\therefore H = (q/c)^{.75} = 12.50 \text{ ft.}$$

$$\therefore \text{T. E. L. upstream} = 52 + 12.50 = 64.50.$$

$$\therefore V_a = \frac{55,200}{601 \times (64.10 - 46)} = \frac{55,200}{601 \times 18.10} = 5.06 \text{ ft/sec.}$$

$$\therefore V_a^2/2g = h_a = 0.40 \text{ ft.}$$

$$V_r = \frac{55,200}{462 \times 14.30} = 8.36 \text{ ft/sec.}$$

$$\therefore h_r = 1.10 \text{ ft.}$$

$$\therefore \text{T. E. L. downstream} = 51.30 + 1.10 = 52.40$$

$$\therefore H_L = 64.50 - 52.40 = 12.10 \text{ ft.}$$

$$\therefore Ef_2 = 17.80 \text{ ft.}$$

$$\therefore \text{Downstream cistern level} = 52.40 - 17.80 \\ = \text{R. L. } 34.60.$$

$$V_o = 0.63 \times (18.10)^{.64} = 4.03 \text{ ft/sec.}$$

$$\therefore V/V_o = 5.06/4.03 = 1.26$$

But, the cistern level cannot be lowered to R. L. 34.60.

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(ii) To work out the scouring velocity to suit the pre-determined cistern level of R. L. 37.0 with seven pocket spans : -

Assume $q = 105$ cusecs/ft.

$$q/c = 105/3 = 35.$$

$$\therefore H = (35)^{\frac{2}{3}} = 10.60 \text{ ft.}$$

$$\therefore \text{T. E. L. upstream} = 62.60$$

$$V_a = \frac{44,000}{601 \times 16.30} = 4.50 \text{ ft./sec.}$$

$$\therefore V_a^2 / 2g = h_a = 0.31 \text{ ft.}$$

$$\therefore \text{W. L. upstream} = 62.29$$

$$V_r = \frac{44,000}{462 \times 19.30} = 6.65 \text{ ft./sec.}$$

$$\therefore h_r = 0.69 \text{ ft.}$$

$$\therefore \text{T. E. L. downstream} = 51.99$$

$$\text{H. L.} = 62.60 - 51.99 = 10.61 \text{ ft.}$$

$$E_f_2 = 15.30$$

$$\therefore \text{Downstream cistern level} = 51.99 - 15.30 \\ = 36.69, \text{ say R. L. } 37.0$$

$$V_o = 0.63 \times (16.29)^{.64} = 3.78 \text{ ft./sec.}$$

$$\therefore V/V_o = 4.50/3.78 = 1.19$$

Thus, neither 6 nor 7 spans are suitable from point of efficient scouring operations with fixed cistern level of R. L. 37.0. Hence, for this reason, too, 8 spans are to be preferred in the left Pocket.

- Statement 1/C-4 (ii)-1.

Showing calculations for protection at U/S noscs of divide walls.

Discharge	Barrage spans (left side.)	Left pocket side	Barrage spans (right side)	Right pocket side
	3,42,000 cusecs. (84 : 16 distribution; $Q_R = 4,52,000$ cusecs " Still Pond ").	55,200 cusecs discharge under scouring operations.	1,04,516 cusecs (75 : 25 distribution; $Q_R = 4,52,000$ cusecs).	27,600 discharge under scouring operations.
Width at nose of divide wall	1497'	600	413'	261'
Q " " "	229	322	250	110
$R = 0.96 q^{2/3}$	35.50'	18.65'	38.40'	23.0'
Water level	70.0	62.80	69.50	62.80
Scour factor (X in XR)	1.30	2.0	1.25	1.50
Depth of Scour below water level (feet)	46.15	39.30	48.0	34.50
R. L. of scour	23.85	22.50	21.50	28.80
River bed level	46.0	46.0	46.0	46.0
Depth of scour below bed level (feet)	22.15	22.50	24.50	17.70
Length of spread (feet)	200	200	120	110
Scour slope	1 in 9	1 in 9	1 in 5	1 in 7

C-4 (ii)-2. Regulation of the Kotri Barrage — "still pond" or "semi-open flow" — and consequent height of Pocket gates.

1. 1 As a result of model experiments⁽¹⁾ and working experience of the Sukkur Barrage, in accordance with the Director's suggestions the design of the Kotri Barrage was based on the assumption of "still pond" regulation up to the highest likely river discharge of 5,65,000 cusecs requiring the Pocket gates to be 4 ft. higher than the other Barrage gates.

1. 2 A few quick tests indicated that "semi-open" flow regulation was quite as good as "still pond" in the Kotri Barrage design, in respect of sand exclusion, for discharges higher than the "still pond limit" (3,84,000 cusecs) — i. e. the discharge at which the Barrage gates would be just out of water with "still pond regulation" and pond level of R. L. 68. 0. The chief merit of "still pond" regulation is that it assists naturally favourable curvature of flow in improving exclusion: where, for any reason, "semi-open" flow is equally effective, there are several good reasons to prefer it. For example 'semi-open' flow causes less heading up of natural river levels thus reducing accretion; the differential head across the divide wall and the depth and extent of scour-hole around its nose are less; it leads to less shoaling in and downstream of the Pockets requiring scouring operations less frequently and of shorter duration; it keeps flow more active along the guide banks thus retarding shoaling along them which partly neutralizes the beneficial curvature effect; and it requires a relatively higher downstream cistern level. (Appendix I shows that the cistern level of the Left Bank river spans would be R. L. 34.0 with 'still-pond' regulation up to upstream W. L. of R. L. 72.0 against R. L. 37.0 desired and obtained under 'semi-open' flow).

Appendix I
C-4 (ii)-2

1. 3 Further consideration and more detailed model tests were, however, necessary in view of the experience at the Sukkur Barrage where it was found necessary to raise the right Pocket gates by adding pieces at top in spite of the Pocket gates being originally 3 ft. higher than the river gates; because it is considered inadvisable at Sukkur to open the Pocket gates at any stage.

A detailed comparison of the conditions obtaining at Kotri and Sukkur Barrage explains largely the reason why restricted 'semi-open' flow regulation is suitable at Kotri *though not* at Sukkur (right bank), thus :-

(a) Assuming curvature effect the same exclusion is governed by the ratio $V_R/V_{R_0} \div V_P/V_{P_0}$ or, for equal depths, by the ratio V_R/V_P (where V_R, V_P are the actual velocities and V_{R_0}, V_{P_0} the regime critical velocities

Statement I
C-4 (ii)-2

on the river and Pocket sides of the Divide Wall). It will be seen from Statement I, that this ratio for the Kotri Barrage is 1.74 times that at Sukkur, showing exclusion at Kotri is far superior to that at Sukkur even if curvature were the same at both.

(b) Actually, however, curvature is more pronounced at Kotri than at Sukkur, being intensified by the central dividing island at Kotri.

(c) The length of the Pocket at Sukkur is *over three times* as long as at Kotri. The longer pocket is worse for sand-exclusion (the control, i. e. the Divide wall nose, being pocket further away) as well as for scouring at its entrance; during scouring operations with a higher Pocket bed at the entrance at Sukkur, the actual V_R/V_P ratio may be less than the calculated value (1.67) and the relative advantage of Kotri design would be greater.

(d) At the Sukkur Barrage, the long separating walls between adjacent Head Regulators provide a considerable inactive region within the Pocket causing shoals. These shoals and still-water pockets within the Pocket adversely affect exclusion making 'semi-open' regulation inefficient.

In spite of these comparatively unfavourable conditions, sand exclusion in the left Pocket at Sukkur is effective with 'semi-open' flow (restricted up to 4 ft. gate openings) as both model experiments and experience have shown. In fact, the shoaling in the Pocket has reduced considerably since 'semi-open' flow regulation is in vogue at Sukkur at river discharges exceeding 3,50,000 cusecs, since the year 1943 at the repeated suggestion of this Station.

If restricted 'semi-open flow' regulation is by experiments and experience found adequate for the Left Bank Canals at Sukkur the same should also prove efficient at Kotri.

1.4 The Consulting Engineer to the Government of India, when consulted on this issue, expressed the opinion that he would favour keeping the Pocket spans higher by a couple of feet *but no more*.

This proposal has been carefully investigated on the model with a view to answer the following questions—

(i) Should the design be based on 'still-pond regulation' for all river stages; if not, to what river stage will still-pond regulation be necessary and sufficient?

(ii) Is it desirable to keep the Pocket gates 2' (or more) higher than the river spans gates or can this be avoided in any way?

Obviously, the higher Pocket gates would be most needed after accretion upstream and downstream of the Barrage. The first experiments, directed to determine the 'still-pond' limit with Pond level R. L. 68.0 and 4 ft. accretion upstream and downstream, showed that all the Barrage gates would be just open, pocket gates shut, for $\approx 3,00,000$ cusecs. This agreed with the discharge worked out by calculations (see Appendix III) and so the lower limit of the range of discharges to be tested for optimum regulation was fixed at 3,00,000 cs. The upper limit was fixed at 5,00,000 cusecs from a study of the discharges expected after the contemplated further withdrawals in the Punjab take place. The maximum discharge for any five-day period, in the eleven years (1931-41) examined, after full development of the Indus Basin, ranges from 2,85,000 to 4,16,000 cusecs, with an average of 3,68,000 cusecs. Allowing for one exceptional year in every ten and for certain of the contemplated withdrawals being postponed or given up, 'semi-open' flow and 'still pond' regulation were compared over the range 3,00,000 to 5,00,000 cusecs.

Appendix III
C-4 (ii)-2

The following four phases of regulation were critically studied at the discharge stages, 3,00,000, 3,50,000, 4,00,000 and 5,00,000 cusecs, under all possible post-Barrage conditions downstream of the Barrage (i. e. with retrograded, mean and accreted specific levels, downstream of the Barrage):—

- (a) Still-pond;
- (b) *restricted* 'semi-open' flow so as not to overtop pocket gates of given heights;
- (c) unrestricted semi-open flow so as to get as low a water level upstream as possible; and
- (d) free flow.

The following Model data were collected in each case:—

(i) For each of these discharges, exclusion was tested by the usual method of bed seeds. In the quick tests carried out previously, bed seeds were dropped at cross section mile 5½; but this time seeds were dropped at cross section mile 5 (i. e. half a mile lower down) so as to have a higher factor of safety in the results showing 'semi-open' to be as good as 'still pond' even for the bed material very near the left bank of the river at cross section 5. (If bed seeds were thrown from a higher section, more were excluded under any regulation). The procedure followed was to drop a fixed number of seeds three times and then find the mean percentage of seeds entering the canals for conditions (a) to (d) as set out above.

- (ii) Percentage of discharge in the left and right arm channels.
- (iii) V_R / V_P ratio at the nose of the divide wall.
- (iv) Difference in water level between the left and right arm channels with equal gate opening.

(v) Afflux in the left and right channels i. e. difference in water levels upstream and downstream of the Barrage.

Table 1 to 4
C-4 (ii)-2

The results of these observations are shown in Tables 1, 2, 3, and 4.
[N. B. — In these experiments the left and right arm river span gates were kept equally opened— i. e. there was no regulation between the arms of the river.]

1.5 Discussion of results.

App. II to V
C-4 (ii)-2

The results of these observations (as shown in tables 1, 2, 3 and 4 and Appendices II to V) are recapitulated below in respect of the effects of different types of regulation or exclusion :—

(i) $Q = 3,00,000$ cusecs. (Table 1).

App. III
C-4 (ii)-2

(a) Barrage gates will be just out with upstream water level at R. L. 68.0 for $Q = 3,00,000$ cusecs, even after 4' accretion over the mean water level of R. L. 63.0 downstream. Thus, the barrage can be worked under 'still pond' with retrograded, mean, or accreted conditions, downstream of the barrage.

(b) Restricted 'semi-open' flow, i. e. pocket gates higher by 2' only, is as good as 'still pond' in point of sand exclusion for the left bank canals. (Table 1).

(c) Conditions on the right bank are more favourable for 'semi-open' flow. Right pocket gates can be opened even up to 4' without disturbing the exclusion of sand from the right bank feeder. The explanations for this behaviour are that the right pocket is wider than the left pocket *relative* to the draw-offs of these two pockets, and the cut-off in the right arm channel being less than in the left arm channel there is greater margin for greater openings of the right pocket gates.

(ii) $Q = 3,50,000$ cusecs — (Table 2).

(a) Restricted 'semi-open' flow — i. e. pocket gates opened by 2' only — is as good as 'still pond' after 4' accretion.

(b) If the pocket gates are kept 2' higher than the Barrage gates, the barrage can be worked under 'still pond' even after 4 ft. accretion upstream and downstream of the barrage.

(iii) $Q = 4,00,000$ cusecs — (Table 3).

Appendix II
C-4 (ii)-2

(a) It will be seen from Table 3 that the water level upstream would be higher than R. L. 68.0 with only mean water level downstream and hence the necessity of keeping the top of pocket gates higher would arise above this stage of discharge; the calculations shown in Appendix II will also show that all the gates will be completely out with upstream water at R. L. 68.0 above $Q = 3,84,000$ cs.

(b) Even if the gates are kept 2' higher, the gates are likely to be overtopped after 4' accretion as the water level obtained upstream is R. L. 71.20 under 'still pond' — i. e. 1.20' higher than the permissible water level of R. L. 70.0 (with the pocket gates 2' higher than the weir gates the 'still pond limit' being $Q = 3,75,000$ cusecs).

(c) But, at this stage of discharge, restricted 'semi-open' flow — i. e. pocket gates opened by 2' only — is as good as 'still pond' and there will be no danger of the gates being overtopped even after accretion.

(iv) $Q = 5,00,000$ cusecs. (Table 4).

Appendix V
C-4 (ii)-2

(a) Appendix V and the water level of R. L. 72.20 obtained upstream for mean water level downstream show that it is not possible to regulate the barrage under 'still pond' conditions unless the top of pocket gates be kept 4' higher than the barrage span gates.

(b) For mean water level downstream, restricted 'semi-open' flow—i. e. pocket gates opened by 2' only — is as good as 'still pond'.

(c) After 4' accretion of downstream water levels, the pocket gates can be opened by 4' so as not to overtop the gates. Exclusion obtained under these conditions is as good as under 'still pond' regulation. A glance at table 4 will show that the permissible opening of the pocket gates is more under accreted conditions of downstream water level than under the mean or retrograded water level conditions downstream of the barrage; the reason being that the cut-off (difference in upstream and downstream water levels) decreases with accretion and, thus, the discharge passing through the pocket spans is less than that under mean or retrograded water level conditions for the same opening of the pocket gates.

Table 4
C-4 (ii)-2

(d) The exclusion from the canals was not so satisfactory with pocket and Barrage gates fully open (i. e. free flow).

1.6 Summarising, the experiments showed that the conditions at Kotri were better than at Sukkur and the results obtained in the model confirmed that *restricted* 'semi-open' flow is certainly as good as 'still pond'. The model, however, showed that even with restricted 'semi-open' flow regulation, it is necessary to keep the pocket gates at least 2' higher than the Barrage gates, i. e. top at R. L. 71.0.

1.7 The model showed that, though designed for 25%, less than 20% of the total river discharge passes down the right arm channel *without regulation* between the two arms, but this is more than sufficient for efficient exclusion of sand from the Right Bank Feeder; and the more the discharge going into the left arm, the easier becomes the exclusion from the left Bank Feeders. It may, however, become necessary to force the designed 25% down the right arm by regulation, for a time, so as to flush out the right arm channel of the Barrage. Such regulation would require a considerable difference in the water levels between the two arms of the river and, at that stage, there is the possibility of the Left Pocket gates being overtopped, even if they are 2 ft. higher than the Barrage gates. Scouring the right arm upstream of the Barrage may, however, result in sanding the right arm channel on its downstream side; because the increase in sand charge is likely to be disproportionately greater than the increase in the discharge. It will be seen from the graph of distribution of discharges that the natural percentages of discharge in the right arm do not exceed 14% and 20% in the pre-Barrage and post-Barrage conditions, respectively, and it is advisable to depart artificially as little as possible from these natural conditions.

Figure 10 of item C-4 (ii)-1, showing the contours obtained in the Model after running four consecutive rising and falling cycles, indicates that silting is likely to occur near the right bank if the right bank pitching is more or less ineffective. Due to model limitations the model is not expected to show the silting *quantitatively* but it is apparent that silting would occur in the triangular portion upstream of the right arm barrage somewhere along the contour R. L. 55, but then a narrow channel would continue to be alive around the central island by virtue of the scour developed by the flow attracted by the island. At such a stage, it may be necessary to flush this channel; the gates must be such that they are not overtopped by reason of the high afflux required to flush the channel. Experiments carried out with $Q \equiv 5$ lacs cusecs showed that an afflux of 1.8 ft. with upstream left pond water level of R. L. 75.4 was required (under the *worst* conditions obtained after accretion) to pass 26% of the discharge down the right arm. As, at this stage, the left Barrage arm remained ponded, which counteracted the curvature and increased the cut-off in the left arm gates it was not possible to open the left Pocket gates by more than 2 ft. without adversely affecting exclusion. Under such conditions, the necessity of having the left pocket gates 4 ft. higher would be felt.

Fig 10 of item
C.4 (ii)-1

There is not the same necessity for having the Right Pocket Gates more than 2 ft. higher than the Barrage gates.

1. 8 Recommendations.

(a) Pocket Gates.

If the top of pocket gates are kept 4 ft. higher from the start, there is the danger of the Pond level being unnecessarily raised or ' still pond ' being maintained where ' semi-open ' flow would be just as good, with the attendant disadvantages mentioned in para 1, 2 above. Hence the top of pocket gates should be initially kept only 2' higher *with provision* to enable the Left Pocket gates being subsequently raised, if required, by another 2 ft.

(b) Experience at Sukkur and experiments at Poona have shown that unfavourable curvature cannot be reversed by ' still-pond ' regulation; but where there is favourable curvature, still-pond regulation assists exclusion, particularly at low stages of discharges — when the curvature is counteracted by the heading up over the natural water levels and when the crucial ratio V_R/V_P governing exclusion is low. The recommended regulation is, therefore, based on the following *desiderata*:

- (i) " Still-pond " regulation for as long as possible for low and medium discharges during the season; and
- (ii) the upstream water level to be maintained as low as possible by avoiding unnecessary heading up.

The experiments have, however, shown that it is necessary to specify different regulations for different discharges in relation to downstream as well as upstream water levels. Table 5 gives the recommended regulation at a glance.

Table 5
C-4 (ii)-1

2. Necessity or otherwise of paving the bed of the Pockets.

Bed sand from the Pockets was not scooped out even under ' semi-open flow ' conditions in the Kotri Barrage design. Even the deposit of a thin layer of silt, which is inevitable, would render the paved pocket virtually the same as an unpaved pocket from point of facility of scouring operations. Hence, there appears no necessity for paving the beds of the pockets.

APPENDIX I. / C-4 (ii)-2

With " Still Pond " level up to R. L. 72.0, river discharge 5,65,000 cusecs, the distribution between the two arms of higher discharges being 80 : 20, which may even increase to 84 : 16 as indicated by the model, the required cistern levels would be as under :—

Distribution (Left:right)	84:16	80:20
Left arm discharge (cusecs)	4,74,000	4,52,000
Left weir span discharge (cusecs)	4,36,700	4,18,700
q (with 20% concentration) (cusec/ft.)	397	379
q_R at cross-section at Divide wall nose (cusecs/ft.)	202	192
$V_a = q_R / 72.46$ (ft/sec)	7.8ft/sec.	7.4ft/sec.
h_a (ft.)	1.0	0.9
Upstream water level (R. L.)	72.0	72.0
Downstream water level (minimum) (R. L.)	64.50	64.50
Upstream T. E. L. (R. L.)	73.0	72.9
Downstream T. E. L. (R. L.)	65.5	65.4
H_L (ft.)	7.50	7.50
E_f	31.7	30.9
D/s Cistern level	33.8	34.50
			Say R. L. 34.0	

Appendix II. / C-4 (ii)-2

Calculations showing the "Still Pond" limit discharge with Pond R. L. 63.0 (after retrogression downstream of the Barrage.).

Assuming river $Q = 3,80,000$ cusecs.

Downstream W. L. = R. L. 60.0

	Left arm	Right arm.
Upstream discharge (cusecs) (distribution assumed 80:20)	= 3,04,000	76,000
Upstream W. L. (R. L.)	= 68.0	67.50
Depth over the upstream pavement R. L. 46.0 (in ft.)	= 22.0	21.50
V_a at section across the nose of divide wall. (ft./sec.)	= $\frac{3,04,000-37,287}{1650 \times 22.0}$	$\frac{76,000-8484}{420 \times 21.50}$
	= 2,66,713	67,516
	= $\frac{1650 \times 22.0}{7.35}$	$\frac{420 \times 21.50}{7.4}$
	= 0.84	0.87
T. E. L. (R. L.) upstream	= 68.84	68.37
q required. (cusecs/ft.)	= 202	188
C (assumed)	3.0	3.0
$q = CH^{3/2}$ (cusecs/ft.)	= $3 \times (16.84)^{3/2}$	$3 \times (16.37)^{3/2}$
	= 204	195
Q (River) (cusecs/ft.)	= 3,84,000	

After retrogression, the "still pond" limit with Pond R. L. 68.0 (i. e. discharge at which river gates just open) will be $Q = 3,84,000$ cusecs (with gates *unincreased* in height).

Appendix III/C-4 (ii)-2

Calculations showing the "still Pond" limit with Pond R. L. 68.0 after 4 ft. accretion (over the M. W. L.) downstream of the barrage.

Assuming river $Q = 3,00,000$ cusecs.

Downstream W. L. = R. L. 67.0 (4-ft accretion over the mean W. L.).

	Left arm.	Right arm.
Upstream Water level (R. L.)	= 68.0	67.50
Upstream discharge (cusecs) ... (19% distribution as observed) in model.	= 2,43,000	57,000
Depth over u/s pavement (ft) ...	= 22.0	21.50
V_a at cross-section across the nose of the Divide Wall (ft./sec.)	= $\frac{2,43,000-37,287}{22 \times 1650}$	$\frac{57,000-8484}{21.50 \times 420}$
	= 2,05,713	48,516
	= $\frac{22 \times 1650}{5.80}$	$\frac{420 \times 21.50}{5.40}$
	= 0.50	0.45
$\therefore V_a^2/2g = h_a$ (ft.) ...		
\therefore T. E. L. upstream (R. L.)	= 68.50	67.95
Drowning ratio ...	= 15/16 = 94%	15/15.50 = 96.50%
\therefore C assumed		
(according to model tests) ...	= 2.35	2.0
q required. (cusecs / ft.)	= 156	135
q obtained (cusecs / ft.) ...	= $2.35 (76.50)^{3/2}$	$2 \times (15.95)^{3/2}$
	= 156	126

\therefore "Still pond" limit after 4' accretion over mean W. L. downstream is 300,000 cusecs with pond R. L. 68.0 (with gates *unincreased* in height.)

APPENDIX IV /C - 4 (II) - 2

Calculations showing "still pond" limit for pond R. L. 70.0 after 4 ft. accretion over the mean W. L. downstream of the barrage.

Assume river $Q = 3,75,000$ cusecs.
downstream W. L. = R. L. 68.55.

	<i>Left arm</i>	<i>Right arm</i>
Upstream W. L. (R. L.)	= 70.0	69.50
Upstream discharge (cusecs) (80 : 20 distribution)	= 300,000	75,000
Depth over upstream pavement (ft.)	= 24.0	23.50
V_a at section across the nose of the Divide Wall (ft./sec.)	$= \frac{300,000 - 37,287}{1650 \times 24}$	$\frac{75,000 - 8484}{420 \times 23.5}$
	= $\frac{2,62,713}{1650 \times 24}$	$\frac{66,516}{420 \times 23.5}$
	= 6.64	6.77
$\therefore V_a^2 / 2g = h_a$ (ft.)	= 0.70	0.72
\therefore T. E. L. upstream (R. L.)	= 70.70	70.22
q reqd. (cusecs/ft.)	= 198	186
Drowning ratio	= $\frac{16.85}{18}$	$\frac{16.55}{17.50}$
	= 92%	95%
C assumed (from model tests)	= 2.5	2.3
$CH^{3/2}$ (cusecs/ft.)	= 200	180

\therefore "Still Pond" limit after 4 ft. accretion over mean downstream water level will be $Q = 3,75,000$ cusecs with Pond R. L. 70.0, i. e. after increasing the height of the gates by 2'.

APPENDIX V / C-4 (ii) - 2

Calculations showing the "Still Pond" limit with Pond R. L. 70.0 (top of gates 2' higher than the weir gates i. e. @ R. L. 71.0) after retrogression downstream of the barrage.

Assume river Q = 4,50,000 cusecs.

Downstream W. L. = 61.55

	<i>Left arm.</i>	<i>Right arm.</i>
Upstream discharge (cusecs) (distribution assumed) 80:20	= 3,60,000	90,000
Upstream water level (R. L.)	= 70.0	69.50
Depth over the upstream pavement. R. L. 46.0 (ft.)	= 24.0	23.50
V_a at the nose of the Divide wall (ft./sec.)	= $\frac{3,60,000 - 37,287}{16.50 \times 24}$	$\frac{90,000 - 8484}{420 \times 23.50}$
	= $\frac{3,22,713}{1650 \times 24}$	$\frac{81,516}{420 \times 23.50}$
	= 8.14	8.27
$V_a^2 / 2g = h_a$ (ft.)	= 1.02	1.06
\therefore T. E. L. upstream (R. L.)	= 71.02	70.56
q reqd. (cusecs / ft.)	= 244	228
C (assumed)	= 3.0	3.0
q obtained (cusecs/ft.)	= $3 \times (19.02)^{1.5}$	$3 \times (18.56)^{1.5}$
	= 243	236 (\equiv 4,52,000 cusecs.)

\therefore After retrogression, if gates be raised by 2 ft. only, the "still pond" limit will be Q = 4,52,000 cusecs for Pond R. L. 70.0.

STATEMENT I/C-4(ii)-2

Comparison of the Sukkur Barrage with the Lower Sind Barrage at Kotri for $Q = 400,000$ cusecs (Still Pond conditions) for the sand exclusion and the potentialities of " Semi-Open " flow Regulation.

	Sukkur Barrage	Lower Sind Barrage at Kotri
(1) Left Pocket withdrawals	29,600 cusecs	37,300 cusecs.
(2) Number of spans in the left pocket.	7 spans of 60' each =420'	8 Spans of 60' each =480'
(3) Width of the left pocket at the nose of the divide wall.	500'	600'
(4) Length of the pocket up to the nose of the divide wall	2020'	637' (As finally proposed).
(5) Width between the nose of the divide wall and the outer bank of the approach channel in case of the former and the Central Island in case of the latter	2325'	1650'
(6) Upstream Pavement level in the pocket (R. L.)	176.0	46.0
(7) Upstream Pavement level in weir portion (R. L.)	171.0	46.0
(8) Barrage sill level R. L.	177.0	52.0
(9) Upstream Pond level, for $Q = 400,000$ cusecs, (Still Pond Conditions), (R. L.)	196.84	71.20 } After 4' accretion
(10) Downstream W. L. (R. L.)	196.10 (Accreted W. L.),	62.30
(11) (a) V_R (ft./sec.)	$= \frac{3,25,400^*}{2,325 \times 25.84}$ = 5.4	$\frac{2,97,500}{1,650 \times 25.20}$ = 7.15 .
(b) V_{R_0} (ft./sec.)	$= .64 d^{.64} = .64 (25.84)^{.64}$ = 5.12	
(12) (a) V_P (ft./sec.)	$= \frac{29,600}{500 \times 20.80} = 2.84$	2.46
(b) V_{P_0} (ft./sec.)	$= .64 (20.84)^{.64} = 4.48$	
(13) $\frac{V_R/V_{R_0}}{V_P/V_{P_0}}$	= 1.67	2.90

* $Q = 3,25,400 = 4,00,000 - (\text{left pocket withdrawals} + \text{approach channel } Q)$
 $= 4,00,000 - (29,600 + 45,000) \text{ cusecs.}$

(14) Intensity of river discharge at the nose of the divide wall. (cusecs/ft.) $\frac{3,25,400}{2325} = 140$ $\frac{2,97,500}{1650} = 180$

Conclusions :-

(a) As V_R/V_P ratio in case of the Kotri Barrage is (= 2.9/1.67) 1.74 times that for the Sukkur Barrage under the same conditions (still pond conditions in Pockets with barrage gates fully open) conditions are *more favourable* for semi-open flow at Kotri Barrage.

Had there been no Approach Channel, which necessitated closing of eight river spans of 60 ft. each, $V_R : V_P$ ratio would have been *still less favourable* in the case of the Sukkur Barrage.

(b) This can be also seen from the comparison of the intensity of discharge at the nose of the divide wall (item 14).

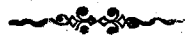


Table No. 1 / C-4 (ii)-2

Table No. 1—Showing, for Q = 300,000 cusecs, the exclusion under different types of regulation tested.

Regulation	Upstream left arm pond gauge	Upstream right arm pond gauge	Afflux between left & right arms (ft.)	Downstream water level (R. L.)	Cut-off left arm channel (ft.)	Cut-off right arm channel (ft.)	% Discharge right arm	Q Left Pocket (cusecs)	Q Left barrage spans (cusecs)	Q Right Pocket (cusecs)	Q Right Barrage spans (cs)	V_R/V_P Left arm	V_R/V_P Right arm	Exclusion left arm	Exclusion right arm	Remarks
"Still pond".	68.0	67.60	0.40	63.0 mean W. L.	5.0	4.60	19 %	37,300	2,05,700	8,400	48,600	$\frac{5.66}{2.82} = 2.0$	$\frac{5.37}{1.49} = 3.6$	All seeds excluded.	All seeds excluded.	Left & right arm barrage gates equally opened.
"Still-pond".	68.15	67.80	0.35	67.0 W. L. after 4' accretion over M. W. L.	1.15	0.80	"	37,300	2,05,700	8,400	48,600	$\frac{5.60}{2.80} = 2.0$	$\frac{5.32}{1.47} = 3.62$	"	"	"
Pocket gates opened by 2'.	"	"	"	"	"	"	"	43,800	1,99,200	11,000	46,000	$\frac{5.45}{3.30} = 1.65$	$\frac{5.05}{1.94} = 2.60$	"	"	"
" 4'.	"	"	"	"	"	"	"	50,500	1,92,500	14,000	43,000	$\frac{5.45}{3.82} = 1.37$	$\frac{4.70}{2.38} = 2.0$	30% into canals.	"	"

Table No. 2 — showing, for Q = 350,000 cusecs, exclusion under different types of regulation tested.

"Still pond".	68.0	67.60	0.40	64.20 Mean W.L. d/s.	3.80	3.40	17 %	37,300	2,53,200	8,400	51,100	$\frac{7.0}{2.82} = 2.48$	$\frac{5.64}{1.44} = 3.78$	All seeds excluded.	All seeds excluded.	Left & right arm barrage gates equally opened.
"Still pond".	69.55	69.25	0.30	68.20	1.35	1.05	17 %	"	"	"	"	$\frac{6.6}{2.67} = 2.48$	$\frac{5.23}{1.39} = 3.76$	"	"	Barrage gates fully opened.
Pocket gates opened by 2'.	69.50	69.20	0.30	68.20	1.30	1.00	17 %	44,000	2,46,500	11,500	48,000	$\frac{6.4}{3.1} = 2.07$	$\frac{4.92}{2.08} = 2.37$	"	"	"
Pocket gates opened by 4'.	69.45	69.15	0.30	68.20	1.25	.95	"	50,600	2,39,900	14,500	45,000	$\frac{6.18}{3.6} = 1.72$	$\frac{4.62}{2.40} = 1.92$	5%	"	"
- do -6'.	69.40	69.10	0.30	68.20	1.20	0.90	"	57,500	2,33,000	17,100	42,000	$\frac{6.0}{4.1} = 1.46$	$\frac{4.55}{2.85} = 1.56$	10% into canals	5% into canals.	"

Table No. 3—showing, for $Q = 400,000$ cusecs, the exclusion obtained under different types of regulation tested.

Regulation	Upstream left arm pond gauge	Upstream right arm pond gauge	Afflux between left & right arms (ft.)	Downstream water level (R. L.)	Cut-off left arm channel (ft.)	Cut-off right arm (ft.)	% discharge in the right arm	Q Left pocket (cs.)	Q Left barrage spans (cs.)	Q Right pocket (cs.)	Q Right barrage spans (cs.)	V_R/V_P Left arm	V_R/V_P Right arm	Exclusion left arm	Exclusion right arm	REMARKS
"Still pond."	68.90	68.75	0.15	65.30	3.60	3.45	16.30	37,300	2,97,500	8,400	56,800	$\frac{7.88}{2.71} = 2.90$	$\frac{5.95}{1.42} = 4.18$	All seeds excluded	All seeds excluded	Barrage gates left & right arm fully opened.
Left and right arm pocket gates opened by 2'.	68.85	68.65	0.20	"	3.55	3.35	"	48,900	2,85,900	14,000	51,200	$\frac{7.55}{3.55} = 2.13$	$\frac{5.40}{2.37} = 2.28$	"	"	"
Pocket gates opened by 4'.	68.80	68.60	0.20	"	3.50	3.30	"	60,400	2,74,400	19,600	45,600	$\frac{7.30}{4.40} = 1.66$	$\frac{4.80}{3.32} = 1.45$	10% seeds entered the canals	5% seeds into the canals	"
"Still pond".	71.20	70.85	0.35	69.30 (4' accretion over mean W. L.)	1.90	1.45	"	37,300	2,97,500	8,400	56,800	$\frac{7.15}{2.46} = 2.90$	$\frac{5.45}{1.30} = 4.20$	All seeds excluded	All seeds excluded	"
Pocket gates opened by 2'.	71.10	70.65	0.45	"	1.80	1.35	"	45,600	2,89,200	12,000	53,200	$\frac{7.0}{3.02} = 2.32$	$\frac{5.14}{1.86} = 2.76$	"	"	"
Pocket gates opened by 4'.	71.0	70.55	0.45	"	1.70	1.25	"	53,400	2,81,400	15,300	49,900	$\frac{6.80}{3.66} = 1.90$	$\frac{4.84}{2.33} = 2.08$	5% into the canals,	"	"
Pocket gates opened by 6'.	70.90	70.40	0.50	"	1.60	1.10	"	60,700	2,74,100	18,200	47,000	$\frac{6.66}{4.06} = 1.64$	$\frac{4.60}{2.87} = 1.60$	15% into the left bank canals.	5% into the right bank canals	"
Free flow.	69.95	69.85	0.10	"	0.62	0.55	15.00	1,19,800	2,22,300	29,000	31,000	$\frac{5.38}{8.17} = .66$	$\frac{3.1}{4.64} = .67$	60% into the left bank canals	35% into canals	"

Table No. 4-showing, for Q=500,000 cusecs, the exclusion obtained under different types of regulation tested.

Regulation	Upstream left arm pond gauge	Upstream right arm pond gauge	Afflex between left & right arms (ft)	Downstream water level (R. L.)	Cut-off left arm channel (ft.)	Cut-off right arm channel (ft.)	% Discharge right arm	Q Left Pocket (cs.)	Q Left barrage spans (cs.)	Q Right Pocket (cs.)	Q Right Barrage spans (cs.)	V_R / V_P Left arm	V_R / V_P Right arm	Exclusion left arm	Exclusion Right arm	Remarks.
" Still pond "	74.00	73.68	0.35	71.50 (max.W.L. with 4' accretion over the mean)	2.50	2.15	16.0 %	37,300	3,82,700	8,400	71,600	$\frac{8.30}{2.22} = 3.73$	$\frac{6.16}{1.16} = 5.30$	All seeds excluded	All seeds exclude	Barrage gates left & right arms fully opened
Pocket gates opened by 2'.	73.95	73.65	0.30	"	2.45	2.15	"	47,000	3,73,000	12,900	67,100	$\frac{8.08}{2.81} = 2.88$	$\frac{5.78}{1.79} = 3.22$	"	"	"
Pocket gates opened by 4'.	73.85	73.60	0.25	"	2.35	2.10	"	56,000	3,64,000	17,400	62,400	$\frac{7.92}{3.35} = 2.37$	$\frac{5.42}{2.42} = 2.24$	"	"	"
" 6'.	73.60	73.35	0.25	"	2.10	1.85	"	64,000	3,56,000	21,000	59,000	$\frac{7.81}{3.86} = 2.03$	$\frac{5.15}{2.95} = 1.74$	10 % into the canals	"	"
" 8'.	73.50	73.25	0.30	"	2.00	1.75	"	71,800	3,48,200	24,700	55,300	$\frac{7.69}{4.35} = 1.76$	$\frac{4.80}{3.48} = 1.3$	20 % "	10% in the canals.	"
Free flow.	72.50	72.40	0.10	"	1.00	0.90	14 %	1,80,300	2,49,700	33,000	37,000	$\frac{5.70}{11.30} = 0.51$	$\frac{3.34}{4.80} = 0.70$	10 % into the canals. Rest of the seeds though excluded through the first four spans of the the pocket.	10% in the canals rest of the seeds went through the pocket.	"
" Still pond "	72.20	71.85	0.35	67.50 (mean W. L. downstream,)	4.70	4.35	17%	37,300	3,77,900	8,400	76,600	$\frac{8.74}{2.38} = 3.66$	$\frac{7.06}{1.25} = 5.65$	All seeds excluded.	All seeds excluded.	"
Pocket gates opened by 2'.	72.10	71.60	0.50	"	4.60	4.10	"	50,500	3,64,500	14,600	70,400	$\frac{8.44}{3.22} = 2.60$	$\frac{6.53}{2.20} = 2.99$	"	"	"
" 4'.	71.65	71.20	0.45	"	4.15	3.70	"	62,300	3,52,700	20,300	64,700	$\frac{8.30}{4.06} = 2.04$	$\frac{6.12}{3.12} = 1.96$	10 % into canals	"	"
" 6'.	71.25	70.85	0.40	"	3.75	3.35	"	73,300	3,41,700	25,400	59,600	$\frac{8.20}{4.85} = 1.69$	$\frac{5.73}{3.92} = 1.45$	20 % "	10 % in canals	"
Free flow.	69.40	69.25	0.15	"	1.90	1.75	15 %	1,77,300	2,87,700	35,000	40,000	$\frac{7.40}{12.60} = 0.59$	$\frac{4.10}{5.80} = 0.70$	Same as free flow for accreted water level.	Same as free flow for accreted water level.	"

TABLE 5./C-4(ii)-2

Showing phases of regulation with pocket gates 2' higher than the river spur gates.

151

Phase.	River Discharge. (cs.)		Water levels (R.L.)		Regulation Phase	R e m a r k s .
	U/S	D/S	U/S	D/S		
I.	3,84,000	3,38,200	68.0	For water levels ranging from 60.0 to 65.0	" Still Pond "	For retrograded and mean water levels downstream of the barrage.
II.	4,52,000	4,06,200	70.0	For water levels from 61.5 to 66.3	" Still Pond "	" " "
III.	5,00,000	4,54,200	72.10	For water levels from 62.55 to 67.30	Restricted "Semi-open" Pocket gates opened by 2'.	" " "
IV.	6,00,000 & above up to 7,50,000				Open Flow	Canals closed.
V.	3,75,000	3,29,200	70.0	68.55	" Still Pond "	After 4' accretion
VI.	5,00,000	4,54,200	73.86	71.50	Restricted " Semi-Open ", Pocket gates opened by 4' so as not to overtop the pocket gates	" " "
VII.	6,00,000 and above up to 7,50,000				Open Flow	Canals closed.

C. - HYDRAULIC WORKS.

S P I L L W A Y S

C-6-1. *Automatic Gates on Tansa Waste Weir (Bombay) - Experiments with half size model.*

1. Introduction.

In order to provide partial relief against the acute water-shortage facing the Bombay city, it was decided to increase the storage capacity of the Tansa Lake by raising the retention level — and, therefore, the crest level of the waste weir — from R. L. 420.00 to R. L. 422.00 T.H.D (Town Hall Datum) by installing automatic gates on the weir wall. Experiments with 1/8 and 1/10 scale geometrically-similar models were conducted in 1945 at the Poona Station to collect data necessary for the design of the automatic gates. (1) On the basis of these data the automatic gates were tentatively designed. Experiments contemplated with a full scale gate not being feasible at the Station, experiments with half size geometrically-similar model of the weir and the gate were made in the 8 ft. wide flume. The length of the gate was, however, equivalent to 16 ft. only as against 50 ft. in the prototype.

These experiments were made on behalf of, and at the cost of, Messrs. Duncan Stratton and Co., Bombay.

2. The Model.

The height of the Tansa Weir is 130 ft. In the model only the top 20 ft. were reproduced. As the ratio of height of weir to depth of water in the model was not less than 10, the velocity of approach was negligible.

The 8 ft. wide flume had direct connection with the Lake, thus allowing any desired discharge to be drawn. Discharges up to 65 cusecs were run in several tests. The discharge passing over the gate was measured over a 5 ft. wide standing wave flume, 50 ft. below the Gate. To even out the distribution of water and damp down the turbulence, elaborate arrangements consisting of a combination of baffles, grids and cross-wall were provided with marked success.

Accurate recording of water levels was done in a gauge chamber, 11½ ft. upstream of the gate.

Air vents were provided in the side walls to be open to atmosphere through vertical shafts connected with the vents.

3 Requirements and programme of experiments.

3. 1. The gate was designed to operate as soon as the water level rose/fell just above/below the *retention* level (R. L. 422.00) and, in its final position, to lie at an angle of about 4° with the horizontal at the highest *permissible* water level of R. L. 422.25. Actually, the dam section is designed to be stable at the *maximum* water level of R. L. 422.50; but a margin of 0.25 ft. between the *permissible* and *maximum* water levels was provided in the interests of safety.

The gate was required to escape the flood discharge equal to about 6.45 ≡ 18.25 cs/ft. at the *permissible* water level 422.25 T. H. D.

An important requirement was that the gate should be sufficiently sensitive to close completely at the *retention* level so that there would be no loss of storage capacity below retention level.

3. 2 The following programme of experiments was fixed to test whether the gate behaves correctly according to design:—

Experiment No: 1 To ascertain the exact water levels at which—

- (a) the gate starts opening;
- (b) the gate is fully down;
- (c) the gate starts returning;

and (d) the gate is completely shut.

Experiment No. 2. To ascertain the discharge passing over the gate when the lake level in the prototype is at—

- (1) R. L. 422.125;
- (2) R. L. 422.250;
- (3) R. L. 422.375;

and (4) R. L. 422.500.

These experiments were to be done *with* and *without* the anti-vibration fins.

(1) Item 11 of Annual Report (Technical) 1945 of the Indian Waterways Experiment Station, Poona.

Fig 1
C-6-1

Photo Nos.
5144 & 5145

Photo 5236

Experiment No. 3 To determine and tabulate the water pressure curve on the deck plating at three different lines on the gate and for the following inclinations of the gates with the horizontal :—

- (a) 52° - 30'
- (b) 50° - 00'
- (c) 45° - 00'
- (d) 40° - 00'
- (e) 35° - 00'
- (f) 30° - 00'
- (g) 25° - 00'
- (h) 20° - 00'
- (i) 15° - 00'
- (j) 10° - 00'
- (k) 5° - 00'
- (l) 0° - 00'

Experiment No 4. To study the effect on the discharge and operational behaviour of the gate of different weir profiles with a view to obtain *optimum* conditions in respect of discharge and aeration.

4. Experiments and their results.

The results of Experiments Nos. 1, 2 and 4 (according to programme) are set out below. Experiment No. 3 is in progress and will be reported next year.

The gate with its counterweight, bearings etc. was designed and supplied by Messrs. Duncan Stratton & Co., Bombay.

Experiments Nos. 1 & 2. (Series I).

These are grouped in Series I.

Experiment No 1.

(a) The gate started opening just as soon as the water level rose to R.L. 422.00.

(b) Design calculations indicated that at upstream water level R. L. 422.25, the gate would subtend an angle of 4° with the horizontal. Actually, however, at this water level, the inclination of the gate with the horizontal was 11°. Even at the maximum water level (R.L. 422.50) the gate remained inclined at 8° 10'. In both cases the nappe was *unaerated* contrary to the design hypothesis and.

(c) In absence of aeration with falling water level the gate gradually came up till it subtended an angle of 21° with the horizontal. It then stopped moving and as soon as the upstream water level \cong R. L. 421.44 was reached, it suddenly rose up to close completely.

Experiment No 2.

Discharges over the gate at *various* water levels in the lake were ascertained and are set out in Statement I — appended at the end. Discharge at water level 422.25 was \cong 14.50 cs/ft. against 18.24 cs/ft.

Further Experiments.

In the model the gate did not go down sufficiently to pass the necessary discharge at water level R.L. 422.25. The nappe over the gate was also *not aerated* in the model owing to back pressure exerted by water which piled up under the gate up to the underside of the nappe; the air vent consequently got sealed up with water and did not function.

Anti-vibration fins, 2"x2"x4" and 4" high, were suggested by Messrs. Duncan Stratton & Co. for preventing "*chattering*" of the gate. Such *chattering* was not observed in the model. Fins of this size were also not useful for providing aeration by breaking the nappe. The model showed that the fins would cause efficient aeration only when about 3 ft. high but such high fins are objectionable as they will entangle floating debris and are also liable to be damaged, vitiating the working of the gates.

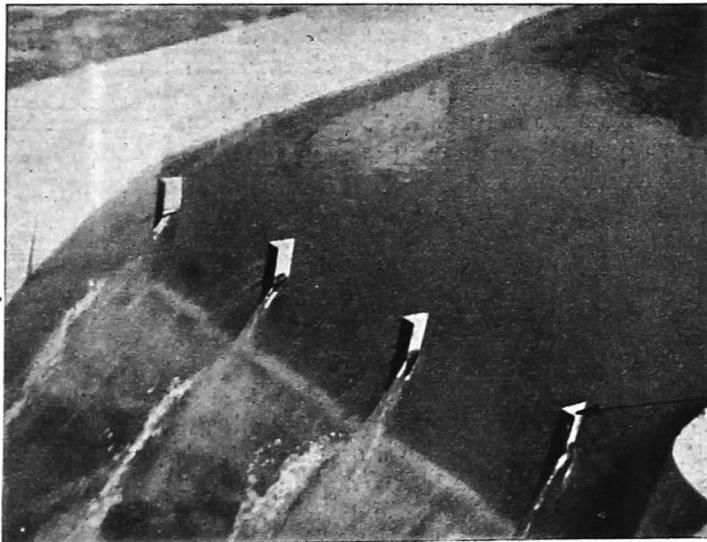
The gate has to fulfil two prime requirements: it should be able to pass 6.45 \cong 18.24 cs/ft. at water level R.L. 422.25 and the nappe should be *fully aerated at all stages*, at least up to water level R.L. 422.25 and preferably higher. The model results can only be depended upon when the nappes of *both model and prototype are fully aerated*. With an *unaerated* nappe in the model and an *aerated* nappe over the full-size gate, the full-size discharge will naturally be *less* than shown by the model; because the forces responsible for aeration in the model and the proto are not proportional and so it is not possible — and it is even dangerous — to predict the discharging

Photo No
5147

Statement I
C-6-1

Photo No.
5146

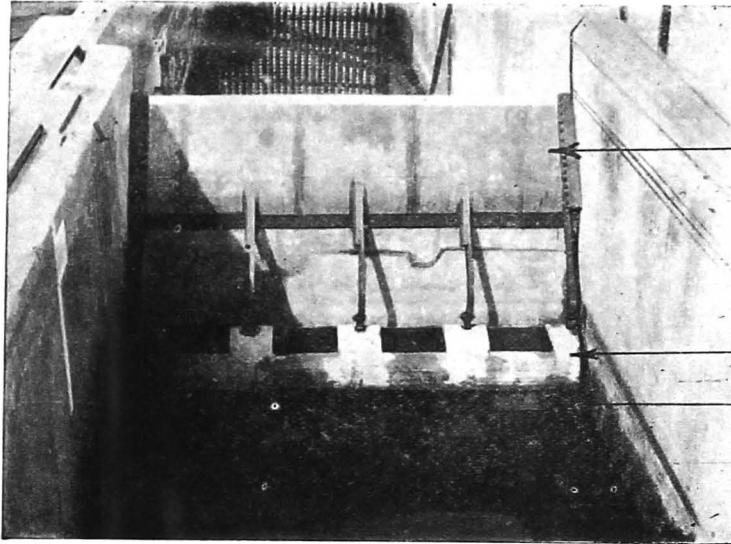
Half size model of the automatic gate proposed for the Tansa Waste Weir (Bombay)



—— Large fins

Photo 5146 : (Looking upstream).
Shows fins which caused fairly satisfactory aeration below the nappe.

Half size model of the automatic gate proposed for the Tansa Waste Weir (Bombay).

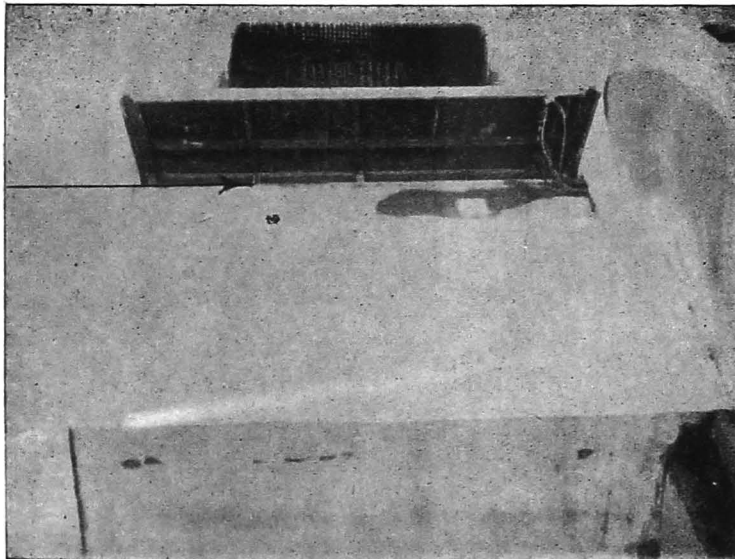


____ Gate.

____ Counterweight.

____ Brackets for supporting counterweight.

Photo 5144 : Model gate seen from *upstream* side.



Bearings _____

Photo 5145 : Model gate seen from *downstream* side.

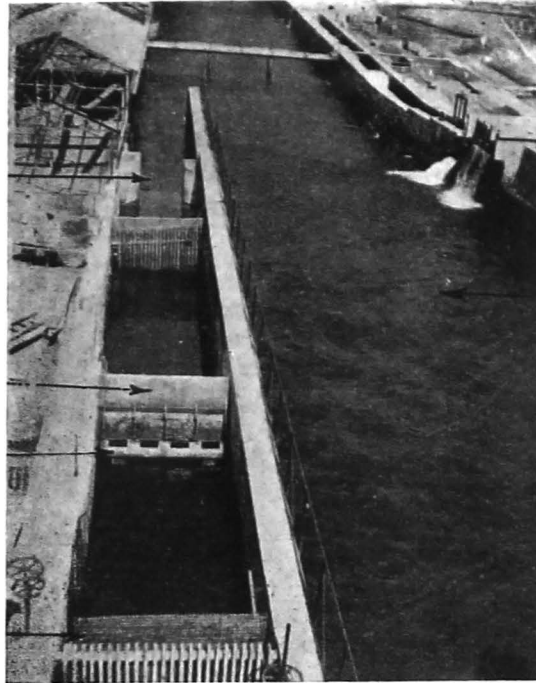
Half size model of the automatic gate proposed for the Tansa Waste Weir (Bombay).

**5 ft. Standing Wave _____
Flume for measuring discharges.**

**Tansa $\frac{1}{2}$ size spillway _____
Gate.**

Counterweight. _____

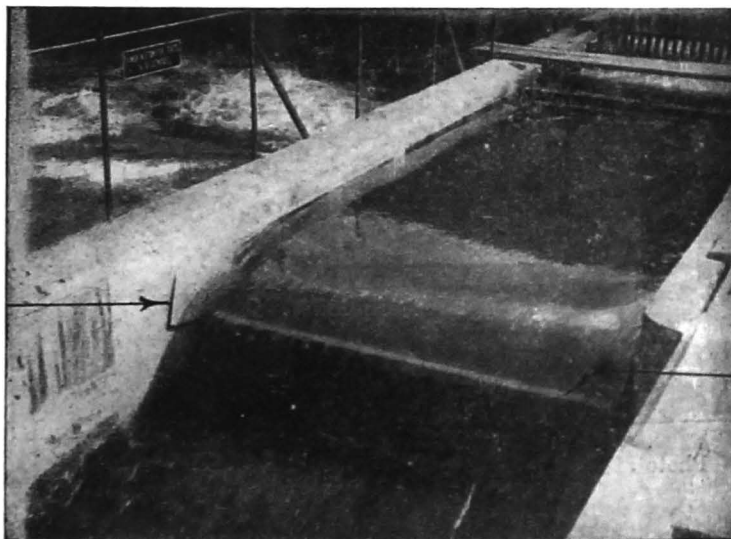
**Grides to damp down turbulence and even out _____
velocity across the entry channel.**



**_____ Mutha Right Bank
Canal,**

Photo 5236 : Showing Bird's-eye view of Tansa Gate Model. (Looking downstream).

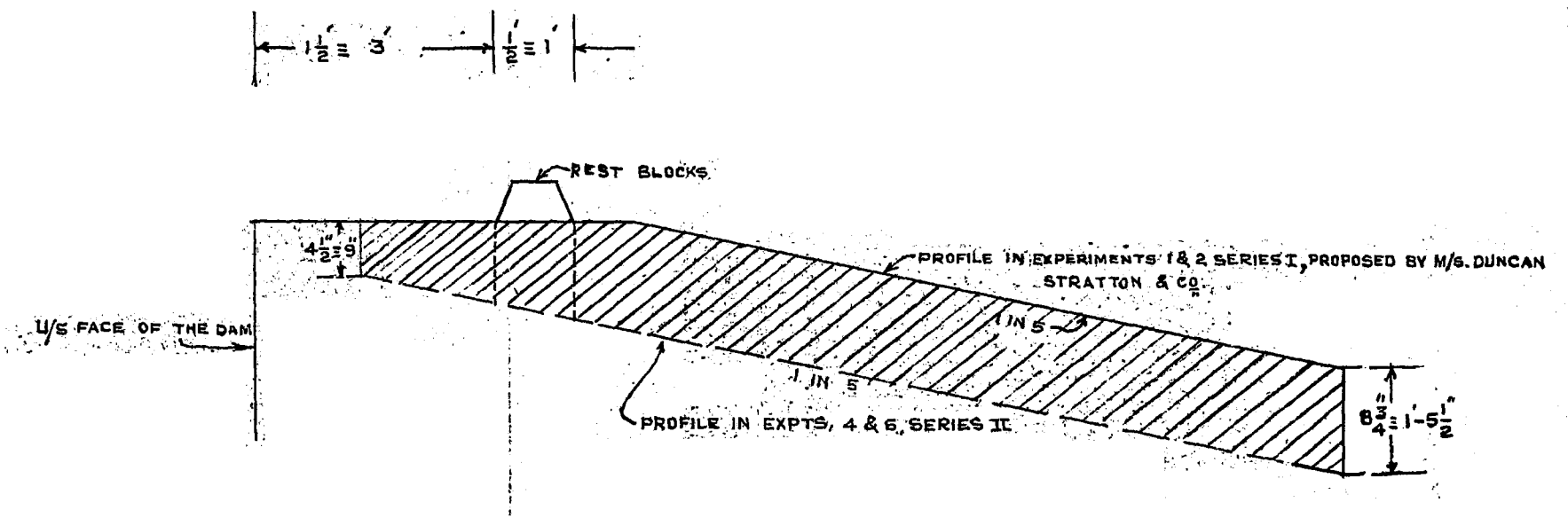
**Pointer fitted _____
in model for directly
reading the inclina-
tion of the gate with
the horizontal.**



**_____ Gate in tilted
position.**

Photo 5147 : Showing gate in operation. (Looking upstream).

FIG: 1/C-6-1
PROFILE OF TANSA WEIR TESTED
IN EXPERIMENTS 4 & 6, SERIES II.
SCALE: 1=1



capacity of the prototype from the model, unless the nappe is fully aerated in the model. Experiment 3 according to the original programme was, therefore, deferred to first evolve a suitable weir profile capable of passing the required discharge under fully aerated conditions.

4. 2. Experiment No. 4 (Experiments in Series II.)

Experiments were, accordingly, carried out with different profiles of the weir. The one found to give results satisfactory both in respect of aeration and discharge is shown in figure 1.

Statement
I/C-6-1
Fig.1/C-6-1

Stanching bars of iron were provided at both sides of the gate to minimize leakage. After working the gate for several times, it was noticed that the side friction between the stanching bar and the side of pier, against which it presses, reduced with time.

For getting comparative results it is essential that side friction should be substantially the same in all the experiments — a difficult requirement in practice.

In the prototype, the gate being 50 ft. wide, the effect of the side friction, on the whole, will be negligible; on the other hand, in the model the gate width was only 8 feet and, consequently, this effect was disproportionately exaggerated. The model *without* the stanching bars will, therefore, yield more accurate results corresponding to conditions in prototype than *with* the stanching bars.

Experiments 3 and 4 of Series II were, therefore, repeated after receding the stanching bars clear of the pier sides. All further experiments were carried out, similarly, with stanching bars receded.

4. 3. Experiments in Series III.

The profile of experiments 4 and 6, Series II, would have been recommended for adoption, had the original weir section been sufficiently stable to allow the necessary cutting. Stability calculations, however, disclosed that with the existing Tansa weir section, the resultant already fell outside the outer middle third by 0.17 ft. at the critical section at R. L. 365.00, with water level in the lake at R.L. 422.50 T. H. D.; and in the profile originally approved by the Bombay Municipality, the resultant was further out by 0.08 ft. ; while with the profile of Experiments Nos. 4 or 6 of Series II — which gave the best results as mentioned above — the resultant fell outside the middle third by 0.49 ft. which might be considered objectionable. It was also not desirable to cut the profile under the gate, as the concrete slab, 4' 9" wide with top at R. L. 418.00., was already constructed at site.

Series III experiments were designed to evolve a profile in which the resultant would not fall outside the middle third by more than 0.25 ft (as in the originally approved profile) and wherein the slab already cast would not have to be disturbed; the necessary modifications were made by adding masonry on top of the present slab and by altering the downstream glacis portion.

The optimum profile thus obtained is shown in Fig. 2 and has been accepted with very slight alteration by Messrs. Duncan Stratton & Co. and the Special Engineer, Bombay Municipality, for adoption at site.

During these experiments, however, aeration was mostly wanting; and so the discharging capacity obtained from model tests could *not* be accepted as applicable to the prototype for reasons already mentioned.

4.4. Experiments in Series IV.

The shape of the profile having been fixed, means for creating aeration at all stages, at least up to water level R.L. 422.25 if not at higher levels, were devised in experiments of Series IV.

Experiment 1.

The semi-circular ease-water was removed so as to allow the falling nappe to flow clear of the side instead of clinging to the curved side of the ease-water.

This slight change was very effective in creating aeration at all rising flood stages except at W. L. 422.50 when the aeration totally disappeared. In the falling flood, aeration progressively developed from piers towards the centre. The model gate is 8 ft. wide and the pier is reproduced only on one side. Under these conditions aeration never failed to re-establish before water level fell to 422.160 T. H. D.

Aeration was anticipated, the discharge over the gate *reduced* from $6.56 \cong 18.60$ cs/ft. (in Experiment No 7 of Series III) to $6.30 \cong 17.70$ cs/ft.

Experiment No. 2.

The piers with ease-waters were already constructed at site, so cutting of ease-waters, though successful in creating aeration, was impracticable, owing to existing reinforcement bars. Other devices, such as a projecting ledge, a slotted recess, a

Statement
1.C-6-1
Fig.2/C-6-1.

niche in the side of the pier etc, were, therefore, tested for obtaining aeration. The niche in the pier wall — tested in Experiment 2 of Series IV — appeared to be the only promising device, though not as good as the rectangular pier-end.

The results of this final experiment are arranged in two sets, (1) under *rising* stage conditions, and (2) under *falling* conditions. Also, unlike the previous experiments in which observations under falling stage conditions alone were made in this experiment observations were taken both under rising as well as falling flood conditions. The discharge was observed to be *more* under *falling* flood conditions than under rising flood conditions, as also under *unaerated* conditions than under *aerated* conditions.

(a) Rising stage conditions.

The discharge intensity was *less* under the rising stage conditions (with the angle of gate correspondingly greater) than under falling conditions. The reasons for this behaviour appear to be as under ;—

(i) Inertia.

In the model, in a rising flood, at W. L. 422.25 T. H. D., the gate attained the position at an inclination of 13° with corresponding discharge of 4.7 cs/ft. \equiv 13.30 cs/ft. When upstream water level was slightly raised the gate went down a little. With the increase in discharge following the falling of the gates the upstream water level dropped a little; but due to *inertia*, the gate did not rise correspondingly.

Thus, one of the reasons was the relative insensitivity of the gate.

(ii) In the gate design, the forces on the deck plate of the gate resulting from the change in momentum are not explicitly accounted for.

Actually, these appear to have some effect, the more so when the gate is inclined.

(b) Falling stage conditions.

The reappearance of aeration in the falling flood depended on the position of the gate at W. L. 422.50 T. H. D. If the gate did not go down below 3° , the aeration did not altogether disappear and with falling flood it easily re-established. On the other hand, once the gate went below 3° — apparently the critical angle — aeration completely vanished and could not fully develop till water level fell to the retention level.

When *unaerated conditions* were obtained in the falling flood, following the disappearance of aeration at W. L. 422.50 T.H.D., discharges were *higher* than corresponding discharges under *aerated* conditions.

When *aerated conditions* were obtained in the falling flood, the discharges were within the range given in Statement 1. These are higher than at corresponding rising stages, but lower than under *unaerated* falling stage conditions.

5. Conclusions.

Fig.2/C-6-1

(i) The final design now adopted — shown in Fig. 2— will pass a minimum of 4.7 \equiv 13.3 cs/ft. discharge in the *rising* flood under *fully aerated* conditions, against 6.45 \equiv 18.24 cs/ft. required.

Under the *falling* stage conditions, with *aerated* nappe, the discharge will be more than under rising stage conditions; but still less than the necessary discharge at W. L. 422.25; while with *unaerated* nappe the discharge will be in excess of the minimum necessary.

To pass the required discharge of 18.24 cs/ft. on *rising* flood the upstream water level has to be higher, say, R. L 422. 40 T H. D.

(ii) In the profile of the final design, the resultant falls outside the middle third at section at R. L. 365.00 with W.L. 422. 25 by 0. 3 ft., i. e. nearly by the same amount as in the original profile accepted by the Bombay Municipality.

(iii) It is not possible to say with certainty that the device which gives aeration in the model will give equally good aeration in the prototype (for reasons already mentioned), or *vice versa*.

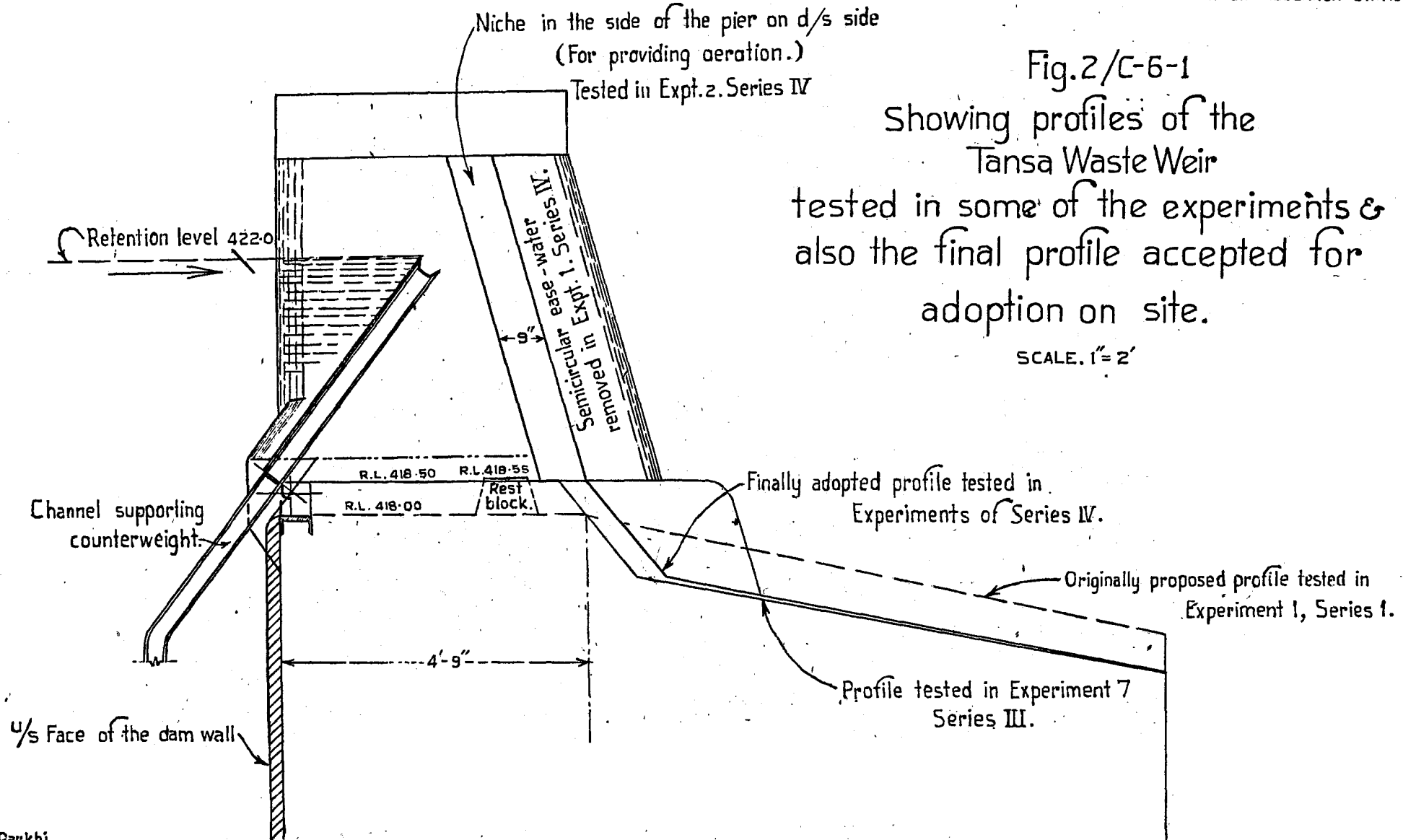
(iv) It has been observed that aeration is important for sensitivity, i. e. thr smooth working of the gates; as, inter alia, the counterweight otherwise falls rather suddenly against the brackets.

(v) On the other hand, *unaerated* conditions give a somewhat *higher* discharge.

Niche in the side of the pier on d/s side
(For providing aeration.)
Tested in Expt. 2. Series IV

Fig. 2/C-6-1
Showing profiles of the
Tansa Waste Weir
tested in some of the experiments &
also the final profile accepted for
adoption on site.

SCALE. 1" = 2'

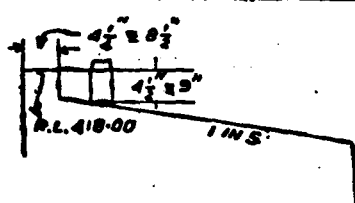
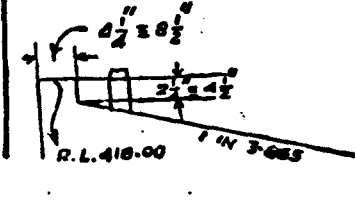


STATEMENT I/C-6-1.

Summary of Results of Experiments.

S. No.	Shape of the Profile	Experiment	U/S W. L.	Angle of gate with horizontal.	Discharge over the gate in model in cusecs.	Discharge per foot in model in cusecs	Discharge per foot in proto in cusecs	Average depth of water above R.L. 418.00 under-side of the gate. (ft.)	Conditions of aeration underneath the gate.	Remarks
1	2	3	4	5	6	7	8	9	10	11
Series I Experiment 2.		Original proposed design as in Fig. 1/C-6-1 No. 604/45 received from Messrs. Duncan Stratton & Co. Ltd.	422.250	11°-0'	41	5.13	14.50	.68	Intermittent in left half. no aeration. no aeration.	
			422.375	8°-50'	47	5.88	16.65	.68		
			422.500	8°-10'	51	6.37	18.05	.68		
Series II Experiment 1.		Glacis lowered by 4 1/2" ≡ 9" at the D/S end of rest blocks and from thence kept horizontal.	422.250	13°-55'	37	4.625	13.09	.72	Intermittent, sporadic, poor No aeration. No aeration. No aeration.	
			422.584	13°-10'	45	5.625	15.90	.82		
			422.774	13°-0'	50	6.250	17.70	.89		
			422.924	12°-50'	55	6.875	19.46	.95		
Series II Experiment 2.		Glacis lowered by 4 1/2" ≡ 9" at the D/S end of the rest blocks, glacis slope 1 in 10	422.024	7°-30'	40	5.0	14.15	.46	Satisfactory. Satisfactory. Intermittent, poor in L. half. No aeration. No aeration. No aeration.	
			422.216	6°-30'	45	5.63	15.90	.48		
			422.250	6°-15'	47	5.85	16.55	.48		
			422.390	5°-20'	50.5	6.31	17.90	.54		
			422.500	5°-10'	54.5	6.81	19.30	.54		
Series II Experiment 3.		Glacis lowered by 4.5" ≡ 9" glacis slope 1 in 5.	422.686	3°-45'	60.0	7.50	21.20	.57	No aeration. Satisfactory. Satisfactory. Satisfactory. Reduced, especially in Rt. half.	
			422.130	9°-20'	40.0	5.00	14.15	.18		
			422.126	6°-20'	45.0	5.63	15.90	.22		
			422.250	1°-55'	51.0	6.84	17.90	.14		
			422.508	1°-30'	57.0	7.13	20.20	.32		
			422.586	1°-20'	60.0	7.50	21.20	.32		

The gate is required to pass 6.45 ≡ 18.24 cs/ft. at W. L. 422.25.

S. No.	Shape of the Profile	Experiment	U/S W. L.	Angle of gate with horizontal.	Discharge over the gate in model in cusecs	Discharge per foot in model in cusecs.	Discharge per foot in proto in cusecs	Average depth of water above R. L. 418.00 under-side of the gate.	Conditions of aeration underneath the gate.	Remarks.
1	2	3	4	5	6	7	8	9	10	11
Series II Expt. 4.		Glacis, lowered by $4\frac{1}{2}'' \equiv 9''$ at $4\frac{1}{2}'' \equiv 8\frac{1}{2}''$ from U/S face of dam; glacis slope 1 in 5.	422.174	8°-15'	42.0	5.25	14.85	.06	Satisfactory	
			422.214	7°-0'	45.0	5.63	15.90	.07	Satisfactory	
			422.230	3°-45'	49.5	6.19	17.50	.07	Satisfactory	
			422.272	3°-0'	50.0	6.25	17.60	.07	Satisfactory	
			422.308	0°-30'	55.0	6.88	19.45	.07	Satisfactory	
422.500	0°-30'	60.0	7.50	21.20	.08	Satisfactory				
Series II Expt. 5.	Sketch same as for Expt. 3 above.	Same as experiment No. 3 in series II, above, but without side friction between gate and pier wall.	422.124	8°-15'	40.0	5.00	14.15	.18	Satisfactory	With side staunching bars receded clear of pier sides.
			422.250	1°-0'	54.0	6.75	19.10	.23	Intermittent sporadic in L. half only.	
			422.500	1°-30'	60.0	7.50	21.20	.32	Nil.	
Series II Expt. 6.	Sketch same for Expt. 4 above	Same as experiment No 4 in series II, but without side friction between dam gate and pier wall	422.174	9°-15'	40.0	5.00	14.15	.10	Satisfactory.	" "
			422.208	8°-45'	45.0	5.63	15.90	.12	Satisfactory.	
			422.250	2°-45'	51.0	6.38	18.00	.11	Satisfactory.	
			422.308	0°-45'	55.0	6.88	19.50	.08	Satisfactory.	
			422.464	1°-15'	60.0	7.50	21.20	.07	Satisfactory.	
Series II Expt. 7.		Glacis lowered $2\frac{1}{4}'' \equiv 4\frac{1}{2}''$ at $4\frac{1}{2}'' \equiv 8\frac{1}{2}''$ from U/S face of the dam, glacis slope 1 in 3.865 requiring cutting of same volume as in experiment 4 (series II).	422.092	6°-30'	45.0	5.63	15.90	.10	Satisfactory.	" "
			422.120	0°-0'	50.0	6.25	17.70	.20	Fairly satisfactory.	
			422.250	0°-45'	55.0	6.88	19.50	.22	No aeration.	
			422.500	1°-15'	60.5	7.56	21.40	.30	No aeration.	

S. No.	Shape of the profile	Experiment.	U/S W. L.	Angle of gate with horizontal.	Discharge over the gate in model in cusecs.	Discharge per foot in model in cusecs.	Discharge per foot in proto in cusecs.	Average depth of water above R. L. 418.00 under-side of the gate. (ft.)	Conditions of aeration underneath the gate.	Remarks
1	2	3	4	5	6	7	8	9	10	11
Series III Expt. 1 A.		Palair type profile according to Drg. No.767/47 of M/S Duncan, Stratton & Co.	422.25	0°	55	8.88	19.50	.23	Slight tendency in left half. Absent	With rest blocks.
			422.50	0°	62	7.75	22.00	.28		- do -
Series III Expt. 1 B.		— do —	422.25	-(0°-30')	56	7.0	19.80	.21	Absent	Without rest blocks. - do -
			422.50	-(0°-45')	63	7.88	22.30	.31		
Series III Expt. 2.		Palair type profile but with downstream glacis raised by 3" ≡ 6" at upstream end below the step	422.25	-(0°-30')	55	6.88	19.50	.22	"	"
			422.50	-(1°-15')	62	7.75	22.00	.30		
Series III Expt. 3.		Palair type profile but with d/s glacis raised by 6" ≡ 1' at u/s end below the step.	422.25	-(2°-0')	53	6.63	18.75	.27	"	"
			422.50	1°-30'	53	7.25	20.50	.35		

S.No.	Shape of the Profile.	Experiment.	U/S.W.L.	Angle of gate with horizontal	Discharge over the gate in model in cusecs.	Discharge per foot in model in cusecs.	Discharge per foot in proto in cusecs.	Average depth of water above R. L. 418.00 under-side the gate (ft.)	Conditions of aeration underneath the gate.	Remarks.
Series III. Expt.4.		Same as Expt. 3 above but with rest blocks and sill under the gate at R.L. 418. 375.	422.124	2° - 0'	50	6.25	17.70	.32	Absent	With rest blocks. -do- -do-
			422.250	1° - 0'	54	6.75	19.10	.30		
			422.500	0° - 0'	60.5	7.56	21.40	.35		
Series III. Expt.5.		Profile same as in Expt. 4 above, but with d/s glacis raised by 9" ≡ 1.5' at u/s end below the step.	422.00 to	5° - 30' to	unsteady	5.0	14.15	.33	Varying Satisfactory in left half but intermittent in right half.	← The critical stage in respect of aeration was reached.
			422.054	8° - 30'	but about 40	5.63	16.90	.33		
			422.104	5° - 30'	45	5.63	16.90	.33		
Series III. Expt.6.		Profile same as in Expt.4 above, but with d/st glacis raised by 12" ≡ 2' at u/s end below the step.	422.144	2° - 0'	50	6.25	17.70	.33	Absent	← Discharge less than required.
			422.250	1° - 14'	54	6.75	19.10	.35		
			422.500	1° - 15'	60	7.5	21.20	.41		
			422.064	7° - 30'	40	5.0	14.15	.42		
			422.124	6° - 30'	45	5.63	15.90	.38		
Series III. Expt.7.		Profile same as in Expt. 5 above, but with sill under the gate raised to R. L. 418.55 i. e. up to top of rest blocks.	422.044	3° - 30'	45	5.63	15.90	.41	Intermittent	Absent
			422.250	2° - 15'	52.5	5.56	18.60	.41		
			422.364	2° - 10'	55	6.88	19.45	.42		
			422.500	1° - 45'	59	7.38	20.90	...		

(Statement 1 / C.-6-1.) continued.

S. No.	Shape of the Profile	Experiment	U/S W. L.	Angle of gate with horizontal.	Discharge over the gate in model in cusecs.	Discharge per foot in model (cs/ft.)	Discharge per foot in proto. (cs/ft.)	Average depth of water above R. L. 418.00 under side of the gate (ft.)	Conditions of aeration underneath the gate.	Remarks
1	2	3	4	5	6	7	8	9	10	11
Series IV Expt.		Weir profile same as in experiment No.7 series III, i. e. with adopted profile & rectangular ease water.	422.144	9°-15'	40	5.0	14.15	0.36	Full	The gate is required to pass 6.45 ≡ 18.24 cs/ft. at W. L. 422.25 T.H.D.
			422.174	7°-30'	47	5.9	16.70	0.36	„	
			422.250	4°-30'	50	6.3	17.70	0.37	„	
			422.324	2°- 0'	55	6.9	19.50	0.37	Slightly reduced	
			422.500	1°- 0'	59	7.4	20.94	0.38	Only in the vicinity of the pier.	

S.No.	Shape of the Profile.	Experiment	U/S W.L.	Angle of gate with horizontal.	Discharge over the gate in model in cusecs.	Discharge per foot in model (cs/ft.)	Discharge per foot in proto (cs / ft.)	Average depth of water above R. L 418,00 under side of the gate.(ft.)	Conditions of aeration underneath the gate.	Remarks.
1	2	3	4	5	6	7	8	9	10	11
Series IV Expt 2.		With adopted profile and 1 niche in the pier wall. (See figure No. 3 2/C-6-1 in the note.)	422.00	52°-30'	—	—	—	—	—	Retention level.
		422.144	17°-0'	30.50	3.81	10.8	0.20	Full aeration		
		422.174	14°-30'	34.00	4.25	12.0	0.33	"		
		422.250	13°-0'	37.50	4.68	13.30	0.44	"		
		422.324	8°-30'	46.00	5.75	16.30	0.46	"		
		422.400	5°-30'	52.00	6.50	18.40	0.51	"		
		422.500'	—	—	—	—	—	—	Not possible to observe under aerated conditions. See foot note (1).	

Foot Note—(1) It was not possible to observe under aerated conditions. With attempts to introduce aeration by high fins, discharge measured to 21.20 cusecs, but this is not strictly comparable, as the fins introduced other opposing features, e. g. decrease in waterway and increase in back pressure decreasing q and the fins trying to press the gate down increasing q.

S. No.	Shape of the Profile	Experiment	U/S W.L.	Angle of gate with horizontal.	Discharge over the gate in model in cusecs.	Discharge per foot in model cs/ft.	Discharge per foot in proto cs/ft.	Average depth of water above R.L. 418.00 under-side of the gate.(ft.)	Conditions of aeration underneath the gate.	Remarks.
1	2	3	4	5	6	7	8	9	10	11
Series IV Experiment 2 continued		with adopted profile & a niche in the pier wall.	422.500	—	—	—	—	—	A little reduced at the non-pier end but on the whole satisfactory.	Not possible to observe under conditions. See foot note (1) at the end of page 162.
		422.400	4°-0'	54.50	6.81	19.27	Not observed.			
		422.324	5°-0' to 8°-15'	51 47.50	6.40 5.94	18.10 to 16.80	.36 to .35	Full aeration at 8°-15' but reduced at 5°-0'		
		425.250	6°-0' to 9°-30'	47 43.50	5.87 5.44	16.60 15.40	.37 to .35	Full aeration.		
		422.174	7°-30' to 11°-0'	45 40	5.62 5.00	15.90 to 14.15	0.20	"		
		422.144	8°-10' to 13°-30'	42 35	5.25 4.37	14.90 to 12.85	0.15	"		
		422.000	—	—	—	—	—	—	Retention level gate completely closes.	

Falling Stage Conditions with unaerated nappe (Statement 1 / C-6-1) continued

S. No.	Shape of the Profile	Experiment	U/SW. L	Angle of gate with horizontal	Discharge over the gate in model in cusecs.	Discharge per foot in model cs/ft.	Discharge per foot in proto cs/ft	Average depth of water above 418.00 R. L. under-side of the gate(ft.)	Conditions of aeration underneath the gate.	Remarks
1	2	3	4	5	6	7	8	9	10	11
Series IV Expt. 2. contd.		With adopted profile & a niche in the pier Wall.	422.50	3°-0'	53 to 60	7.25 to 7.5	20.50 to 21.20	Not observed to 0.46	At 3°-0' aeration is slight, intermittent & only in the vicinity of the pier. At 0°-30' aeration nil.	
			422.40	1°-0'	53	7.25	20.50	0.42	Little and intermittent and near the pier only.	
			422.324	1°-20'	55	6.87	19.50	0.43	Nil.	
			422.250	1°-10'	54	6.75	19.10	0.41	"	
			422.174	1°-30'	51	6.38	18.0	0.39	"	
	422.144	1°-45'	50	6.25	17.70	0.39	Slight aeration near the pier.			

C-6-2 VOLUTE SIPHON SPILLWAYS.

Preventing scour below Volute Siphon Spillways by fanning out jets with dispersers and ledges.

1. *Synopsis* : Experiments were carried out last year in which attempts were made to prevent scour below Volute Siphons by making the high velocity jets hit each other at an angle. ⁽¹⁾ This method of dispersion was not found satisfactory as the maximum depth of scour under these conditions was more than obtained with siphon jets issuing straight or acting singly.

In experiments carried out this year, jet dispersers, *with and without* a tilting ledge, were fixed at the siphon outlets so that the jets were spread out evenly, and at the same time deflected upwards at various angles, before impinging. This resulted in reducing the intensity of the jet and in lesser scour downstream. As anticipated, there was a small reduction in the coefficient of discharge.

2. *Model and Procedure of experiments* :

Two volute siphon models, with operating head of 2.064 feet, were used. The diameter of the vertical shaft was 4 ins. The bed downstream of the siphons was laid to a depth of 2 ft. in incoherent sand of mean dia. 0.4 m.m. In all the experiments, a constant water level was maintained upstream of the siphons and also over the sand bed downstream of the siphons. The siphons were run for a period of 10 hours which was found quite sufficient to give comparative results with the different dispersers tested though not sufficient to give final stable scour-holes; because 90 to 95 percent of the ultimate scour depth is reached within 5 to 10 percent of the run required for the final scour and the rate of scour thereafter is very slow.

3. *Experiments* :

3.1. *Scour depths* :

In Experiment No. 1 the maximum depth of scour downstream of siphons, *without* dispersers or expansions was observed; this formed the Yardstick for comparison with scours obtained with dispersers in position. In experiments 2, 3 and 4, expansions or dispersers were fixed just outside the outlets of the siphons. Fig. 4, shows the dispersers tested.

Fig. 5 shows the longitudinal profile of scour-holes developed, at the end of the run under different outlet conditions. Thus, the maximum depths of scour observed were : -

	Maximum depth of scour downstream below original bed level. (ft.)	Maximum depth of scour below water level. (ft.)	Ref. to Fig.
(1) Siphons <i>without</i> dispersers.	0.526	1.272	
(2) Expansion below outlet; length 0.208 ft.	0.440	1.186	Fig.4 (a)
(3) Disperser below outlet; length 0.208 ft. Side expansion 4:10	0.229	0.975	Fig.4 (b)
(4) Disperser below outlet; length 0.208 ft. Side expansion 8:10	0.175	0.921	Fig.4 (c)

(1) I. W. E. Stn. A. R. T. for 1946 (P. 91).

Photo No.
5243

The velocity of the jet of the siphons *without* disperser or expansion was 9.1 ft./sec. and gave the greatest maximum depth of scour; though the least depth of maximum scour was obtained with disperser as in Fig 4 (c), the flow was more evenly distributed with disperser as in Fig 4 (b) and the difference in maximum depth of scour in these two cases was only 0.054 ft. (\approx 1.94 ft.). This disperser was, therefore, adopted for further experiments. Photo No. 5243 shows a close view of the disperser.

3.2. Coefficient of discharge :

With the dispersers the coefficient of discharge was reduced, as is to be expected, due to back pressure caused by impact.

Fig. 6
C-6-2

The actual coefficient of discharge in each case was verified by running various discharges and noting the depth of water upstream over the crest of bellmouth of the siphon in each case. Thus, for a depth of 0.056 ft. upstream, the under-mentioned coefficients were observed; these coefficients are only to be taken to give a *comparative idea* of the relative reductions :-

(1) Siphons <i>without</i> adjuncts	0.773
(2) Siphons with expansion below outlet as in Fig. 4 (a).	0.766
(3) Siphons with disperser as in Fig. 4 (b).	0.765
(4) Siphons with disperser as in Fig. 4 (c).	0.763

3.3. Further experiments with a ledge :

Fig 7
C-6-2

In further experiments, in order to fan out the jet a horizontal ledge (length = .166 ft.) was added at the end of the disperser. The angle of the ledge could be varied, giving the desired upward tilt to the jet issuing from the disperser, as shown in Fig. 4 (b). To study the effect of the addition of a ledge on the maximum depth of downstream scour, the following angles were tried and the corresponding maximum depths of scour observed in each case were as under :-

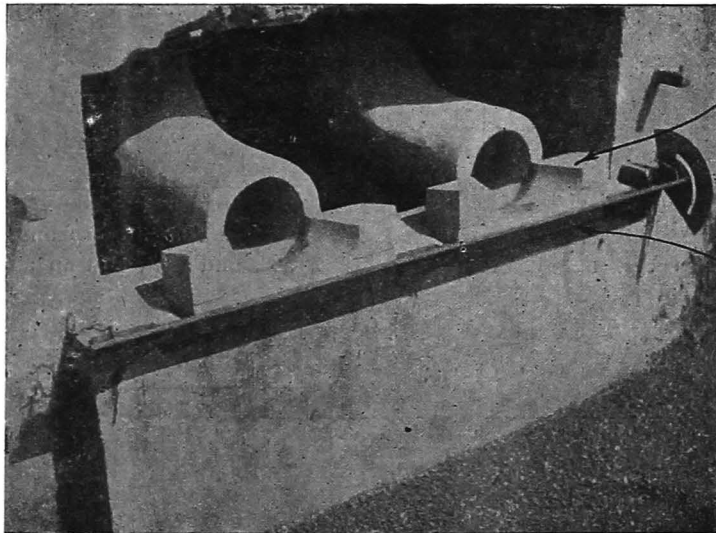
	Maximum depth of scour downstream below original bed level.
(1) Siphon with disperser as in Fig. 4 (b) but <i>no</i> ledge.	.229 ft.
(2) Siphon with disperser as in Fig. 4 (b) but with ledge; angle 10°.	.190 "
(3) Siphon with disperser as in Fig. 4 (b) but ledge angle 20°.	.136 "
(4) Siphon with disperser as in Fig. 4 (b) but ledge angle 35°.	.114 "
(5) Siphon with disperser as in Fig. 4 (b) but ledge angle 45°.	.106 "

Thus, with a ledge at 45° tilt, the maximum depth of scour was *further* reduced to 1/3 rd the depth obtained with a disperser but without the ledge and to 1/5 th depth obtained without the disperser and the ledge. The coefficient of discharge decreased to 0.75 with ledge angle of 45° against 0.763 without ledge, i. e. a further reduction of only 3 percent.

4.0. Conclusion :-

Photos Nos
5205, 5198,
5199 and
5217.

Experiments have shown that the arrangement as tried in the above experiments would substantially reduce the depth of scour and would make costly protective works on river beds unnecessary. Photos nos. 5205, 5198, 5199 and 5217 are attached showing fanning out of jets obtained with dispersers and a ledge.



— Disperser.

— Ledge.

5243 : Volute Siphon Experiments. Outlet of Siphon with disperser and ledge.

Dissipating Energy downstream of Volute Siphons by fanning out jets with dispersers and ledges.



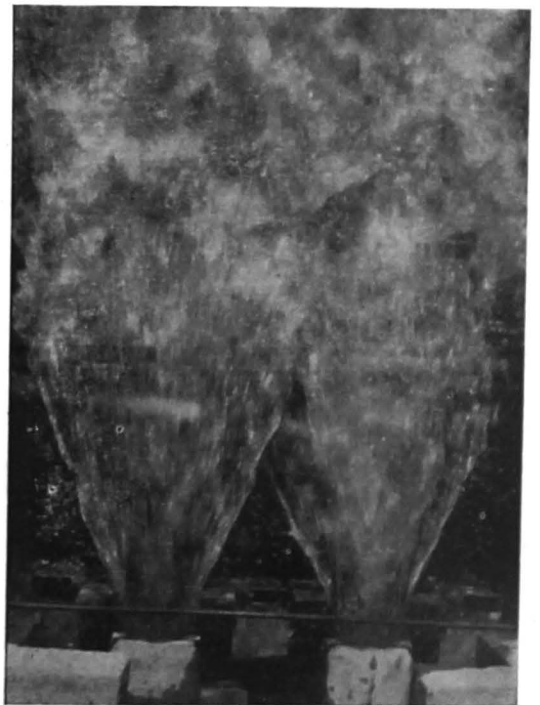
No. 5205. Jets in Siphons with disperser; and a ledge at an angle 45° .



No. 5198. Jets in Siphons with disperser; and a ledge at an angle 20° .



No. 5199, Showing downstream bed after 10 hours run with disperser and a ledge [angle 45°].



No. 5217, Siphons with disperser; side expansion 4: 10 (with ledge not in action).

FIG. 3/C-6-2 SHOWING SECTION OF VOLUTE SIPHON.

SPACING 1.111 Ft. ϕ TO ϕ

SCALE: 1" = 0.5 FT.

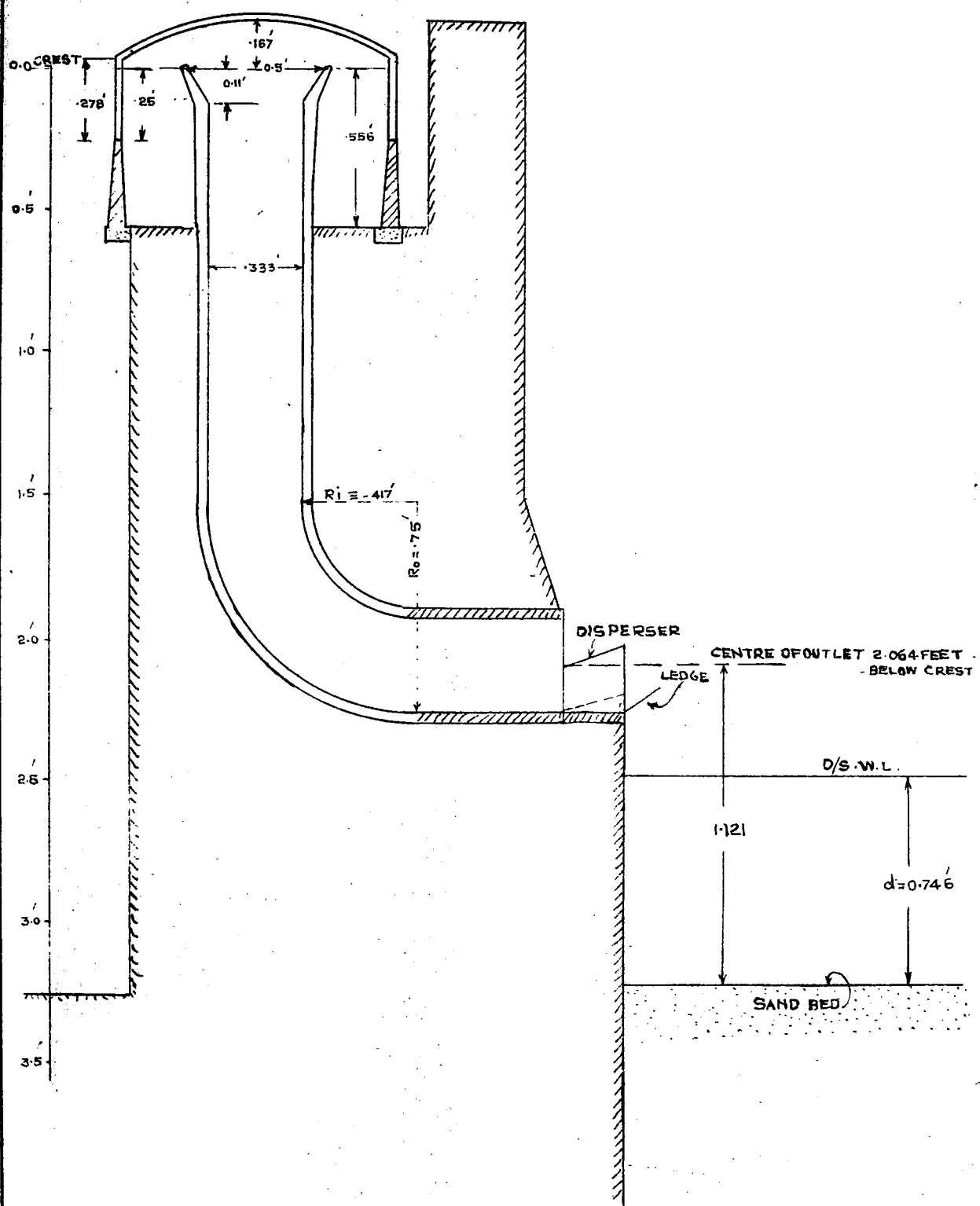
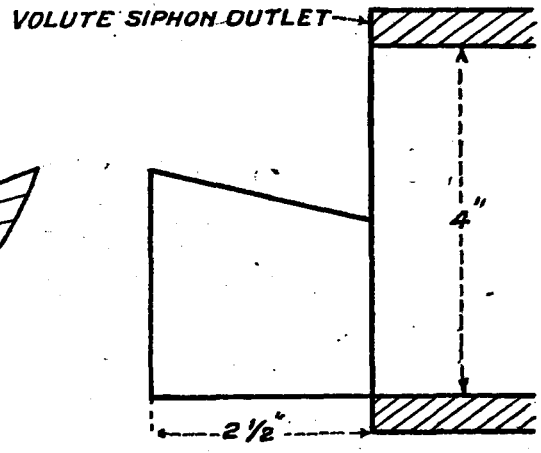
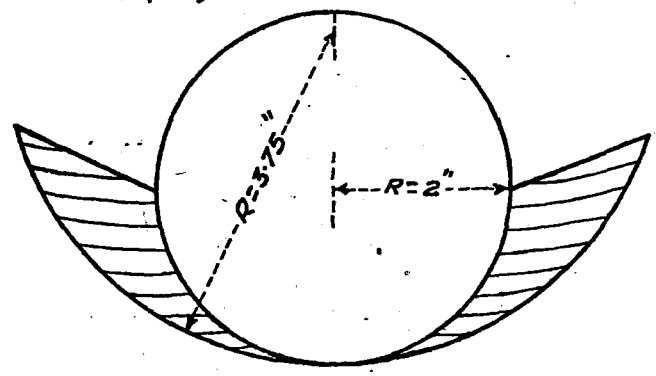


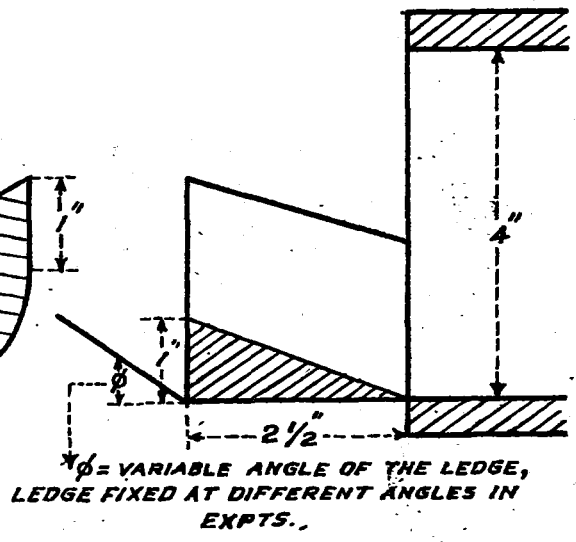
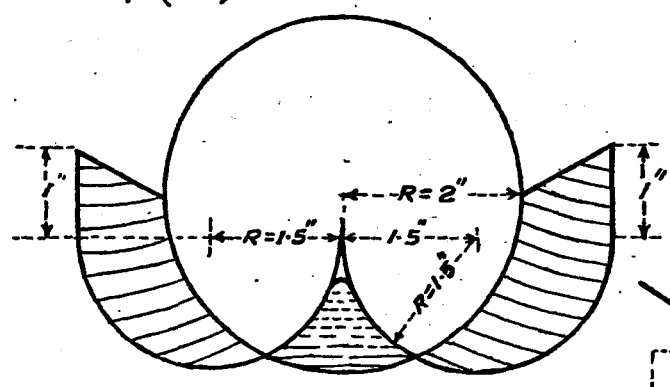
FIG. 4/C-5-2
JET DISPERSERS TESTED.

SCALE - 1" = 2 Ft.

4 (a)



4 (b)



4 (c)

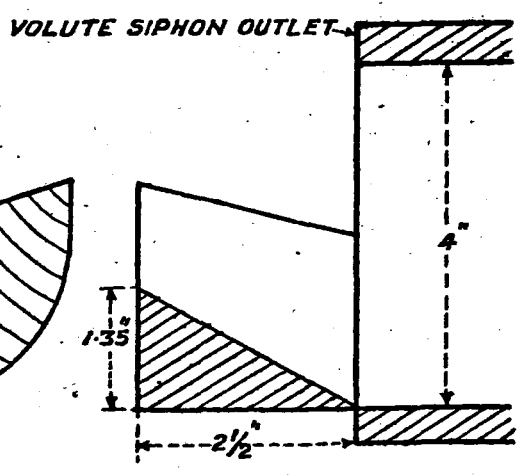
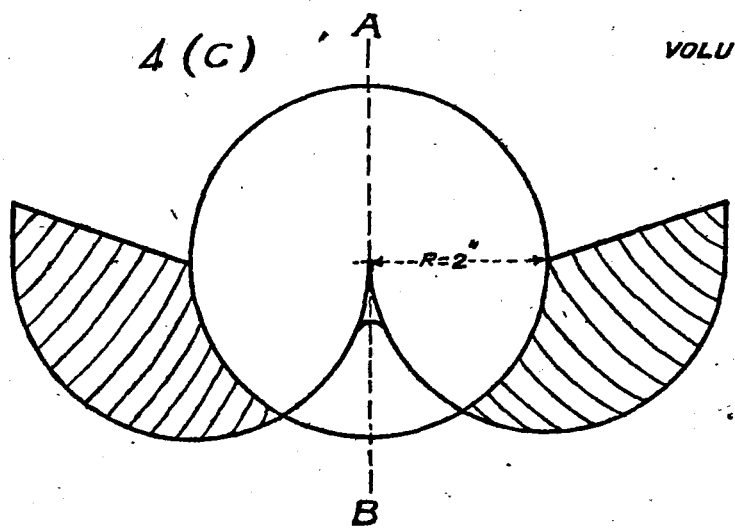
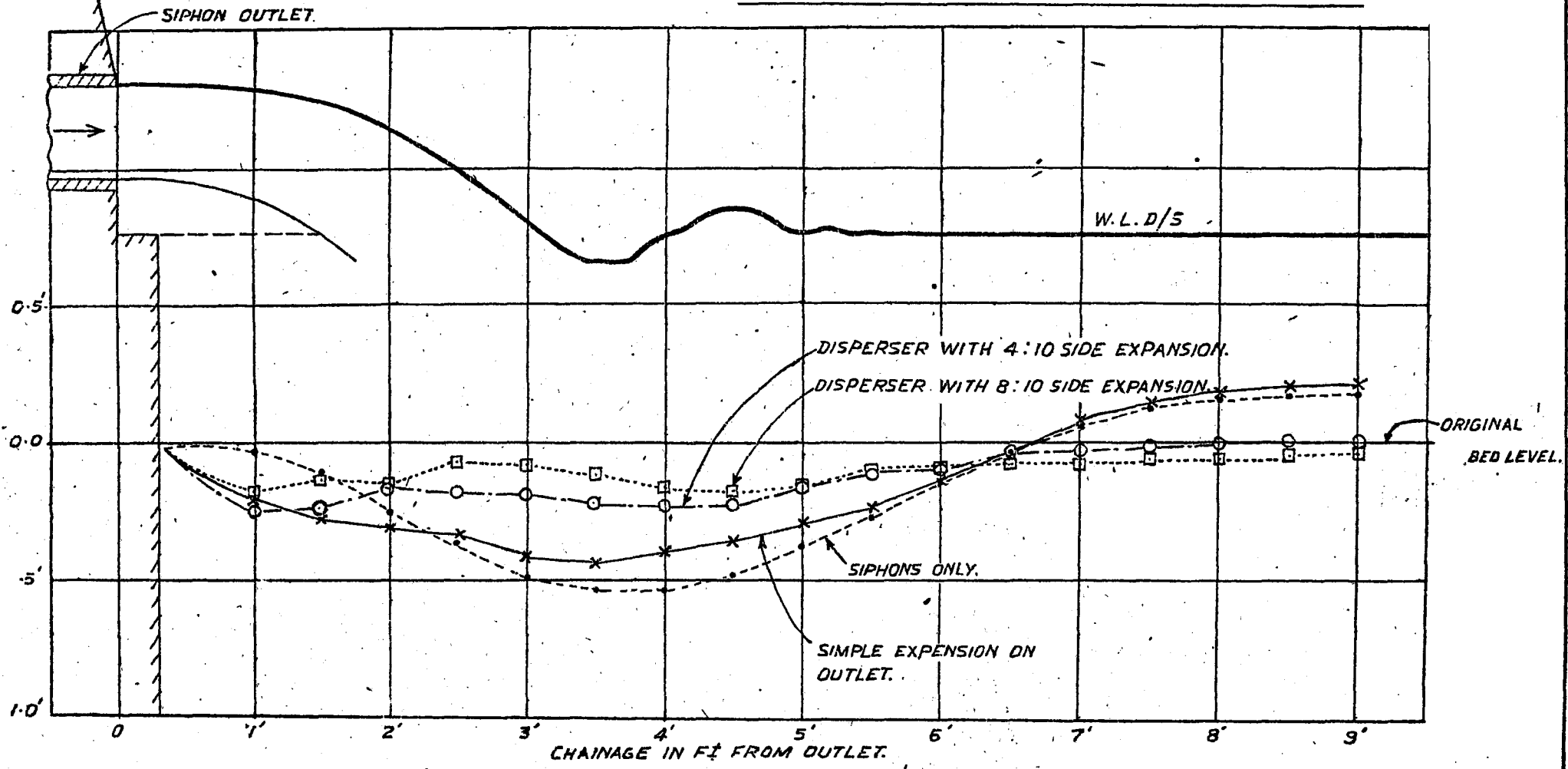


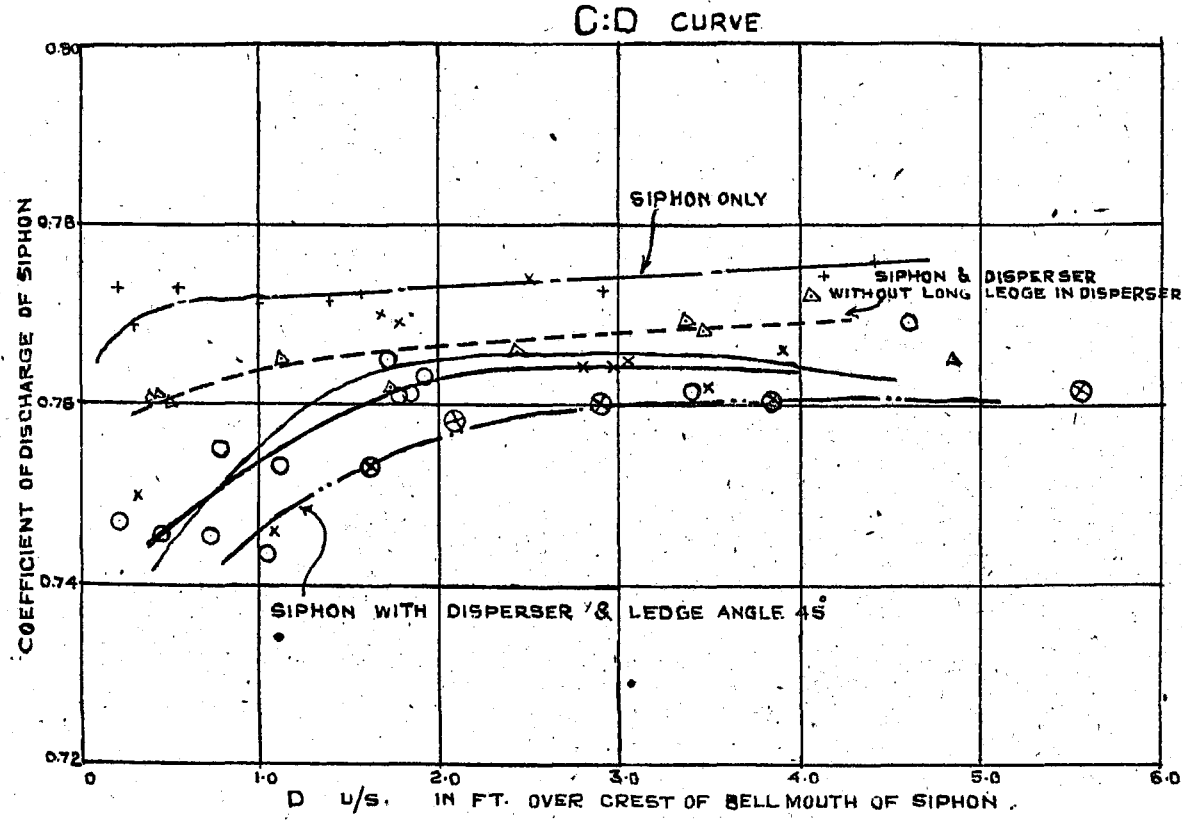
FIG. 5/C-5-2
PREVENTION OF SCOUR BELOW
VOLUTE SIPHONS
SHOWING LONG. PROFILE OF SCOUR.

- References:
- SIPHONS WITHOUT DISPERSER
 - x—x—x— EXPANSION ON OUTLET
 - DISPERSER 8:10 SIDE EXPANSION.
 - " 4:10 " "



PARKHI.

FIG. 6/C-6-2
EXPERIMENTS WITH VOLUTE SIPHONS
EFFECT OF ADDING DISPERSER TO FAN OUTJET
AT THE SIPHON OUTLET ON COEFFICIENT OF DISCHARGE

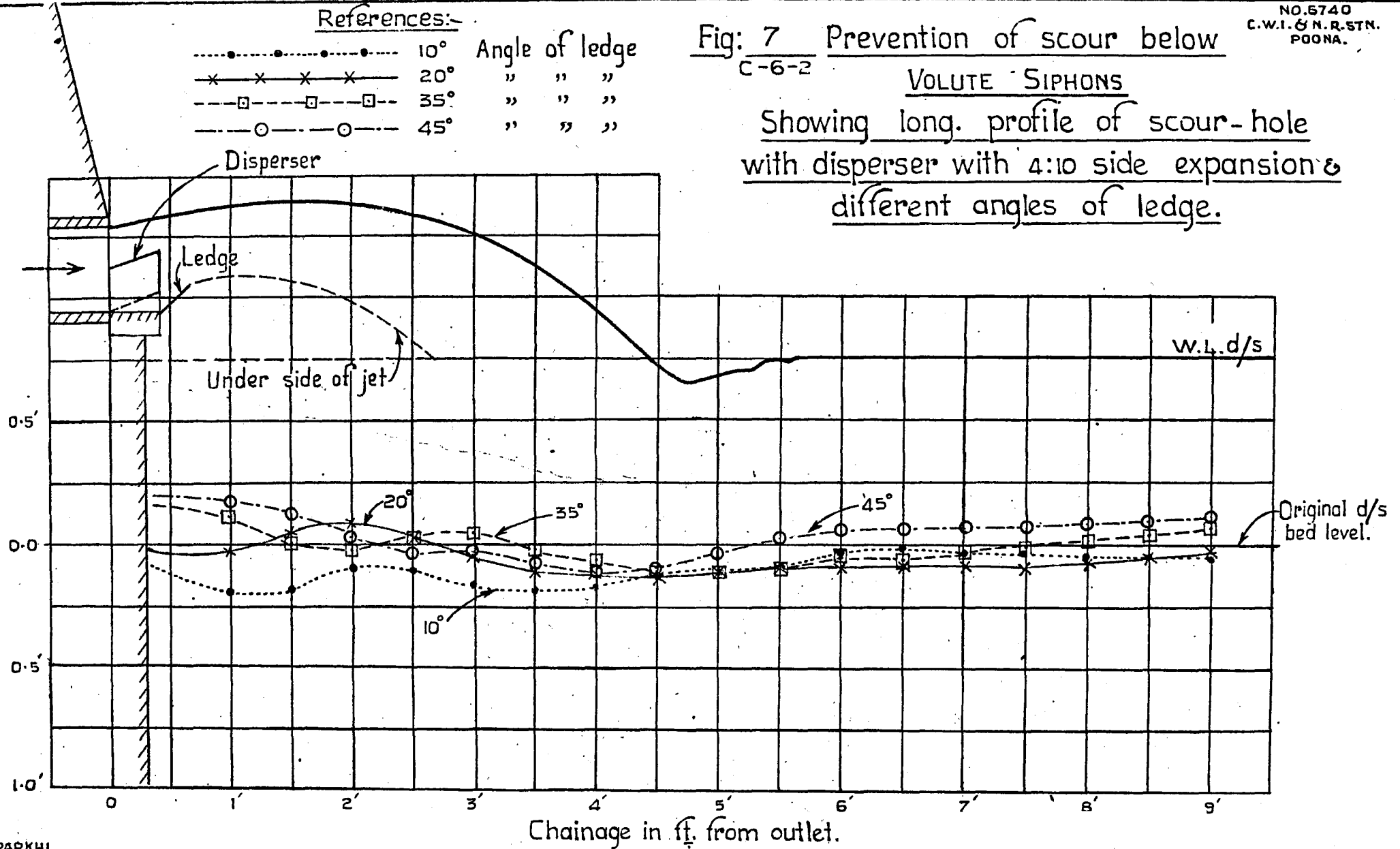


- + SIPHON WITHOUT DISPERSER
- △ DISPERSER WITHOUT LONG RIDGE IN CENTRE
- x 8:10 SIDE EXPANSION DISPERSER WITH LONG RIDGE IN CENTRE HT. AT END = 0.112'
- 4:10 SIDE EXPANSION DISPERSER WITH LONG RIDGE IN CENTRE HT AT END 0.083'
- ⊗ 4:10 SIDE EXPANSION DISPERSER & LEDGE ANGLE 45°

Fig. 7 Prevention of scour below
C-6-2

VOLUTE SIPHONS

Showing long. profile of scour-hole
with disperser with 4:10 side expansion &
different angles of ledge.



C - HYDRAULIC WORKS

8 - Canal Falls

C-8-1. Dissipation of energy below flumed vertical falls and non-flumed S. W. baffle falls.

1. *Introduction* : - At the 17th Annual Meeting of Research Officers of the Central Board of Irrigation, while discussing Canal Falls, Col. T. Blench, Director of the Punjab Irrigation Research Institute, suggested that, in *Flumed Falls*, instead of expanding the subcritical flow below the Fall, it should be better to expand the hypercritical flow above it, since the latter can be fanned out to *any* sharp divergence, thereby reducing the discharge intensity and leading to a cheaper design.

It is also known that vertical falls, particularly of the flumed category, have a tendency of "bellying" unless corrected by suitable devices.

There is a misguided tendency of comparing costs of *flumed* and *unflumed* Falls regardless of the additional purposes served by, and the greater intensity imposed on, the former. It would be useful to show that the same type of Fall e. g. the Central Station Baffle Fall — could be considerably cheapened if fluming is not imposed.

The present series of basic experiments was, thus, started at the Poona Research Station, with a view to—

- (a) study the effects of expanding the hypercritical jet above a fall;
- (b) devise means of preventing the tendency to erode the banks which cause embayments downstream of flumed vertical falls; and
- (c) evolve an economical station design of a *non-flumed*, baffle type Fall.

2. *Models* : 1/6 scale, geometrically-similar models of the flumed vertical and the non-flumed baffle type falls were designed to the following data :-

$$Q = 100 \text{ cusecs.}$$

$$H = 8 \text{ ft.}$$

$$d_1 = d_3 = 3.58 \text{ ft.}$$

$$B = 16.20 \text{ ft.}$$

Figs. 1 & 2
C-8-1

the data being the same as that of the flumed Fall designs reported in the Annual Report (Tech.) 1944. (1)

The model bed below the fall was laid in 0.34 mm. Koregaon sand and the tests were done without any sand charge. The scour pattern obtained in the sand-bed was observed and the general flow conditions were noted and photographed; to compare the designs with different modifications incorporated in them.

PART I

3. Discussion of Results of Tests on Flumed Vertical Falls.

3.1 Initial observations :-

3.1.1 It was at once evident from the model that it is not quite correct to say that a hypercritical jet can be made to follow *any* degree of expansion, leading to uniformly reduced intensity of discharge. For example, it was observed that, though the hypercritical jet followed 2 : 1 side expansions, yet the distribution of discharge across the section was not uniform and with sharper expansions the distribution was made worse. To study this factor in detail, observations of water level elevations were made with different side splays below the parallel-sided control section.

3.1.2 Figures 3, 4 & 5 show the water surface contours in the throat and the extended flat bed below it, but above the fall. In figure 3, the side expansions below the parallel sides were 1 in 2; in Fig. 4 the sides were receded so that water could freely follow its own course; and in fig. 5 the extreme flow lines obtained in fig. 4 were adopted as the natural side expansions.

Figs. 3, 4 & 5
C-8-1

Though in erodible channels, it is natural to have the discharge intensity in the centre greater than that at the side, yet just above a fall, it is desirable to get uniform distribution all across the section because this will

give uniform pressure - plus - momentum required to balance the standing wave and will ultimately prevent any jetting tendency which otherwise persists due to curvature of hypercritical flow, induced by non-uniform distribution of discharge intensity.

3.1.3 With too low a discharge intensity at the sides above a fall, optimum dissipation of energy cannot be expected; and, as seen from figures 3 to 5, the ratio of central to side depths at the end cross section worked out to 1.74 in fig. 3, 9.14 in fig. 4, and 3.86 in fig. 5; and, though the velocities in very low hypercritical depths could not be observed, yet it was noted that the discharge intensity at the sides was less than in the centre.

Table 1
C-8-1

3.1.4 Velocities were, however, observed in the central portion at a cross section 9' ($\equiv 4\frac{1}{2}'$) below the end of the parallel throat, Table 1 below shows the computations of discharge intensities on this cross section.

3.1.5 Comparing the discharge intensities at the centre and 6" ($\equiv 3'$) from the centre line on either side in the three cases, it will be seen that the effect of expansions sharper than 1 in 2 was to reduce the central intensity and *increase it up to 1 ft. from centre*; though this happened in the central 2' strip below the throat, yet further away from the centre the discharge intensity was feeble. Generally, however, the result was that the discharge intensity all across the section was not uniform; but it was rendered less un-uniform in the *central portion* when the sides below the parallel-sided throat were made to expand along the extreme line of free flow.

3.1.6. On theoretical grounds, expanding sides require a rising bed along the line of flow: this was also tested but uniform discharge intensity was still not obtained.

3.1.7 These tests—to make the discharge intensity uniform above the fall.—being of fundamental interest, are being still pursued.

3.2 *Improving the performance of flumed vertical fall (fig. 1) :-*

Starting with the design shown in fig. 1, the following modifications were tested to improve the scour pattern and to reduce the side embayments downstream.

(i) *Effect of length of cistern floor :-*

3.2.1 According to the Central Board of Irrigation publication No. 12 - "Design of Weirs on Permeable Foundations" — the length in which the disturbances below a standing wave are smoothed out is $5(D_2 - D_1)$, where D_2 = Subcritical depth *below* the standing wave and D_1 = hypercritical depth *above* the standing wave.

This would require a length of the cistern floor of $5(D_2 - D_1)$ *below* the position of the standing wave; but further experimental verification of this length is desirable.

In the present case, $D_2 = 3.04$ ft. and $D_1 = 0.50$ ft.

$$\therefore D_2 - D_1 = 2.54 \text{ ft.}$$

In the first design, the length of the downstream floor was 8' = 3.14 ($D_2 - D_1$) as measured from the upstream end of the cistern, but it was only 2 ft. = 0.78 ($D_2 - D_1$) measured from the point where the nappe dropped into the cistern. This experiment was abandoned after only half an hour's run because intensely severe scour occurred in spite of the deflector, which did *not* work. Visual observations of the turbulence below the nappe indicated that the cistern length should be $\equiv 16'$, i. e. 3.94 ($D_2 - D_1$) beyond the point where the nappe fell. The effect of increased cistern length from 0.78 ($D_2 - D_1$) to 3.94 ($D_2 - D_1$) was very marked; the scour depths were markedly less, but the results were not satisfactory because of the scour-holes near the sides which tended to undermine the pavement—see fig. 6. This was due to jetting near sides and surface return flow in midstream.

Photo 5105
Sheet 1

Fig. 6
C-8-1

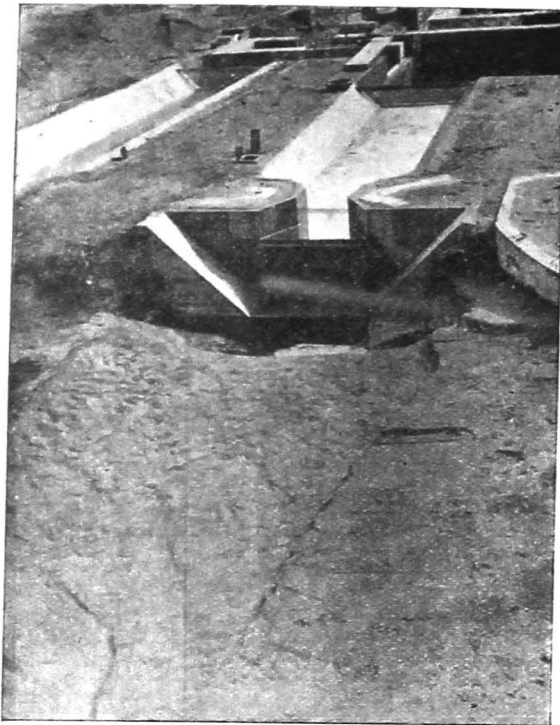


Photo 5105 : Photo showing scoured bed under conditions of design of Flumed Vertical Fall given in Fig. 1/C-8-1.

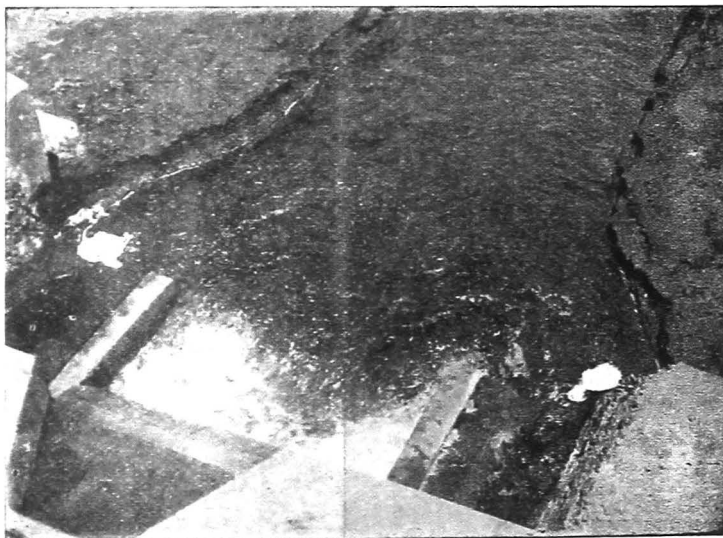


Photo 5159 : Photo showing surface turbulence with 25 percent *retrogression* in d/s water level; cistern \cong 21.34' wide and 16 ft. long with bed at R. L. \cong 89.12. and deflector 0.36 ft. high fixed on its bed. Note the side jetting & scour.

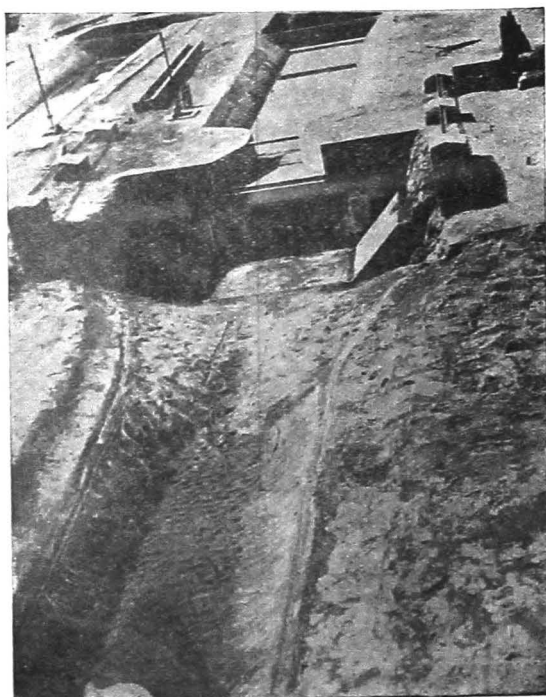


Photo 5168 : Photo showing scour condition with 25 percent *retrogression* in downstream water level; cistern bed \cong R. L. 89.12 with sloping sides. Deflector top was R. L. \cong 89.48. Pavement length \cong 16 feet.

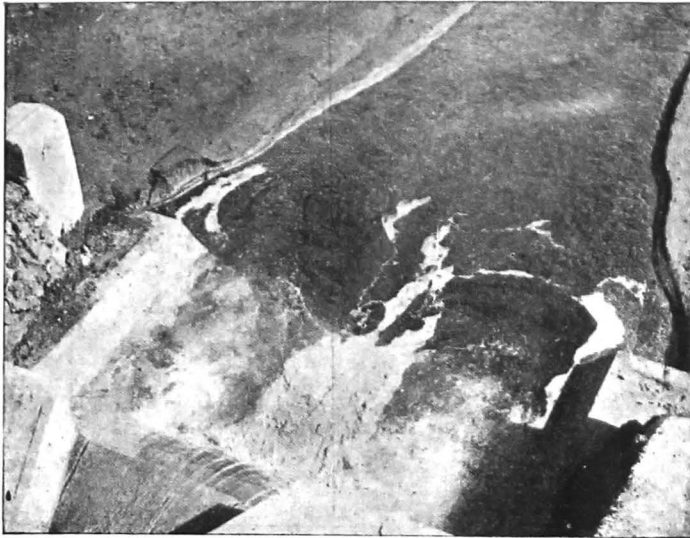


Photo 5173 : Showing surface flow conditions with 25 per cent *retrogression* in downstream bed \equiv R. L. 89.12 with sloping sides; deflector top at \equiv R. L. 89.48; the cistern length was increased to \equiv 25 ft. Notice the two forward flow jets at the sides.



Photo 5179 : Photo showing final scour with 25 per cent *retrogression* in d/s water level; the design was the same as in photo 5173.



Photo 5112 : Photo Showing scour after 1 hour's run with normal W. L. d/s and cistern at \equiv R. L. 91.76. Note scour-holes at sides and bank erosion and undermining tendency at sides but not in the centre.



Photo 5116 : Photo showing scour after one hour's run with normal W. L. Cistern at \equiv R. L. 89.12, and curtain wall and a deflector, top of deflector being at C. B. L. \equiv R. L. 92.0.

Flumed Vertical Falls.

Sheet No. 4.

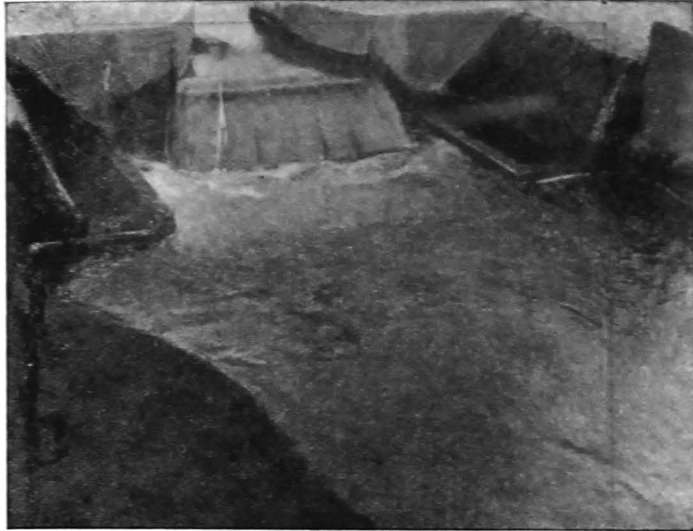


Photo 5127 : Photo showing flow conditions with \approx 26.94 ft. wide cistern with bed at \approx R. L. 89.12; deflector top at C. B. L. \approx R. L. 92.0. The sides of the cistern were made vertical.

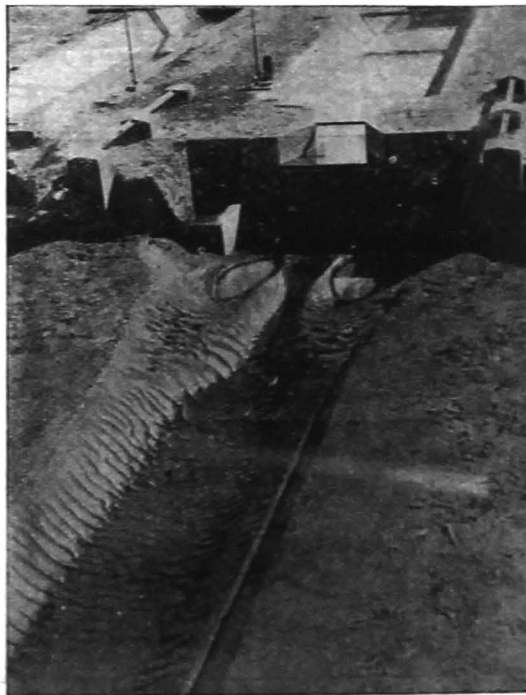


Photo 5126 : Showing scour under conditions of photo 5127.

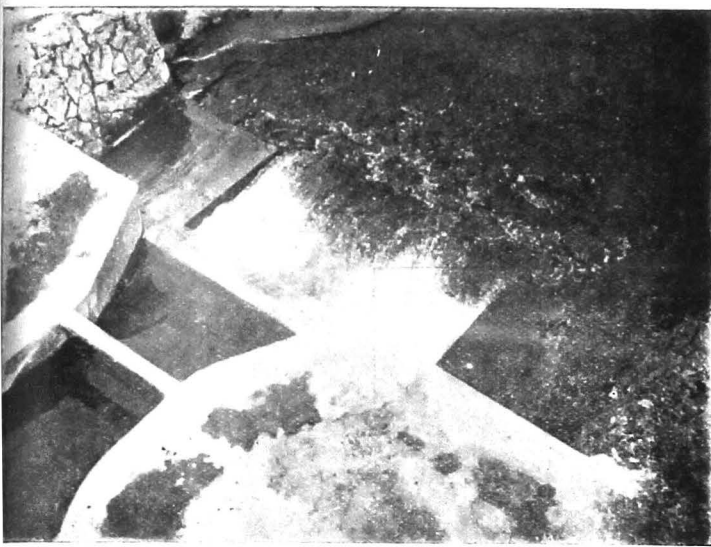


Photo 5136 : Photo showing *surface* turbulence with normal water levels; cistern \equiv 21.34' wide; bed R. L. \equiv 89.12 and deflector top at R. L. \equiv 92.

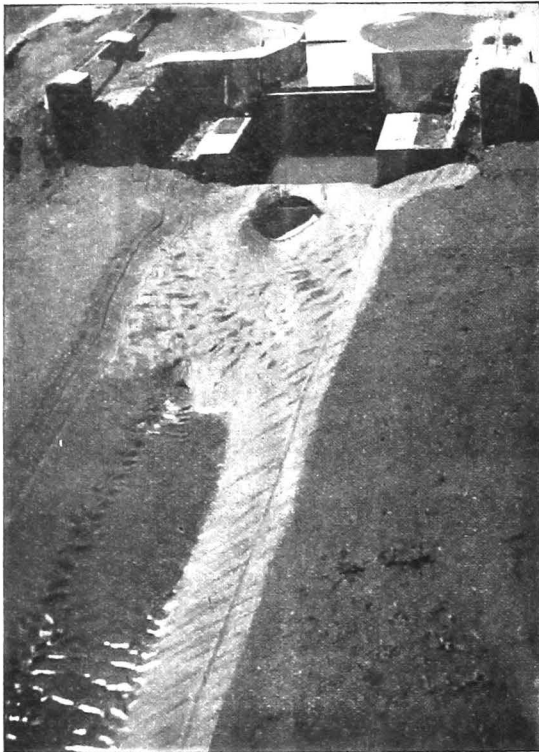


Photo 5129 : showing scour after one hour's run under conditions of photo 5136.



Photo 5134 : Same as photo 5129 but showing *final* scour.



Photo 5142 : Photo showing *final* scour conditions when design in photo 5129 was tested with 25 per cent *retrogression* of downstream water-level. (The design consisted of a cistern \cong 21.34' wide; bed R. L. \cong 89.12 and deflector top at R. L. \cong 92.).

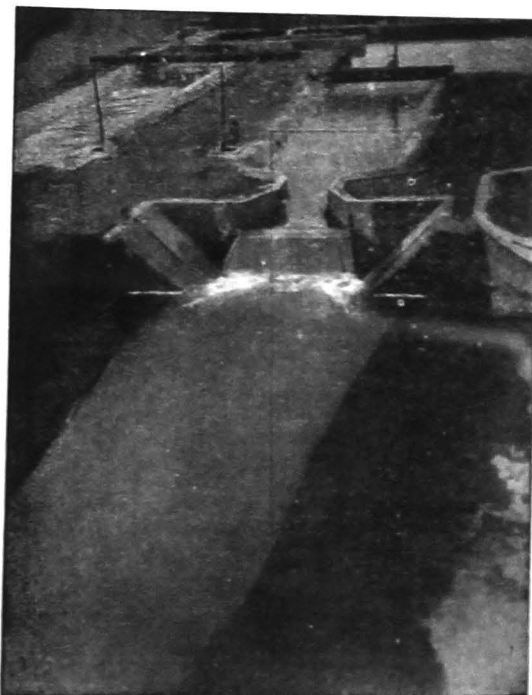


Photo 5108 : Photo showing flow conditions with *normal* water levels d/s. in the experiment with length of the cistern \cong 16 ft. Note side jetting and erosion.



Photo 5113 : Photo looking u/s showing no scour in Baffle Fall with 1 : 1 glacis and pavement length 5.80 ($D_2 - D_1$). Fall working under *normal* water level downstream.



Photo 5143 : Showing *final* scour with 2 : 3 (i. e. 2 horizontal : 3 vertical) glacis and pavement length 7 ($D_2 - D_1$). Fall working under 25 per cent *retrograded* d/s water level.

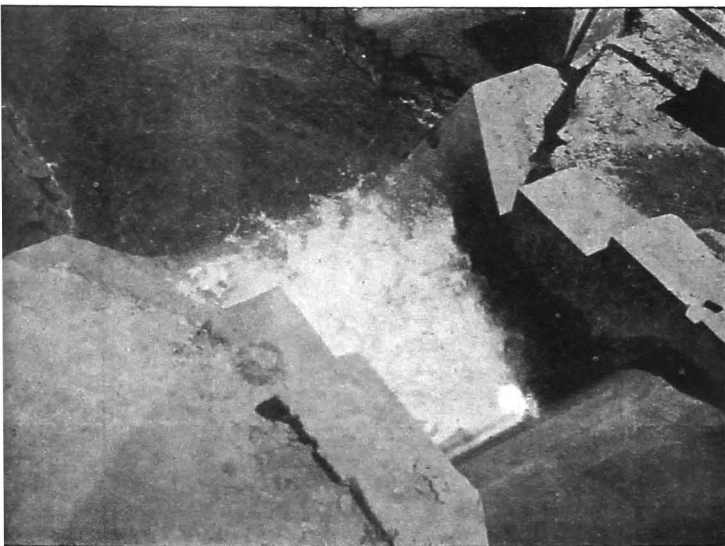


Photo 5160 : Showing satisfactory dissipation of energy with 1/2 : 1 glacis and length of pavement 7 ($D_2 - D_1$); 25 per cent *retrogression* in downstream water level.

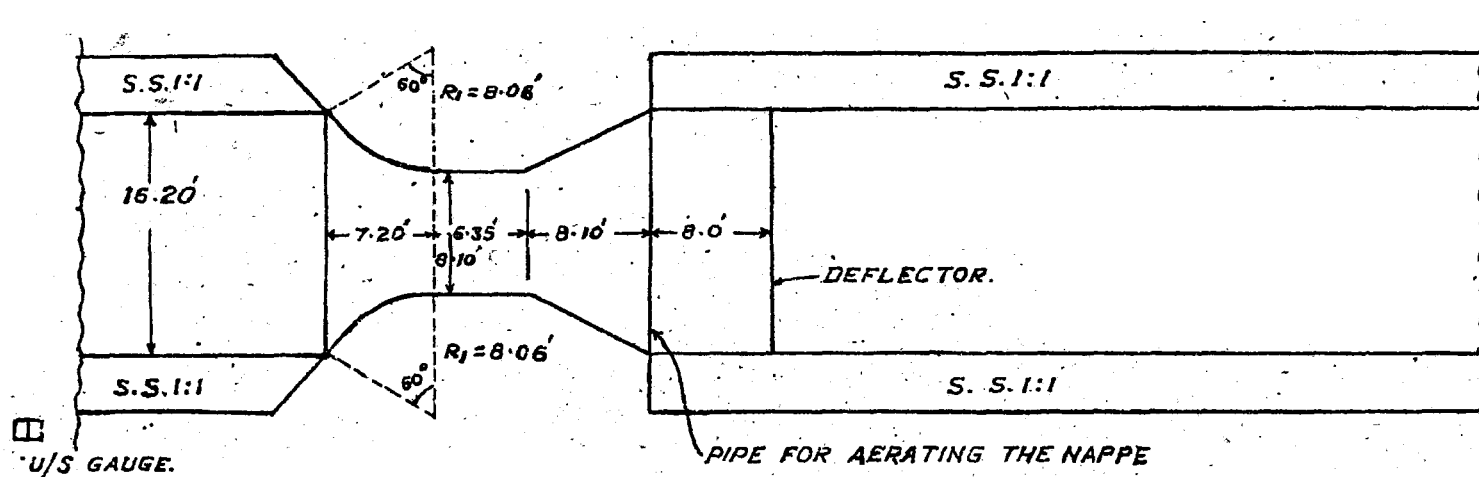
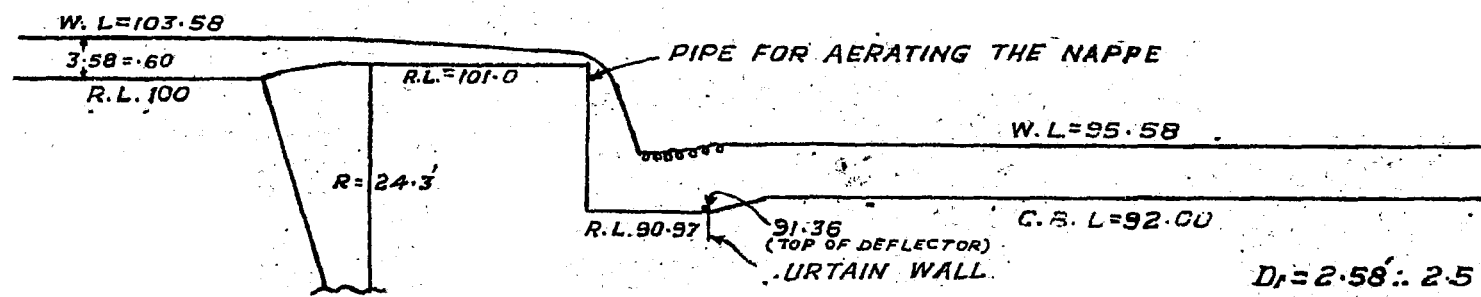


FIG. 1/C-8-1
DESIGN OF 100 CUSECS.
8' VERTICAL FALL
SCALE 1"=12'
Q=100, B=16.20', d=3.58'
FLUMING RATIO=2.0

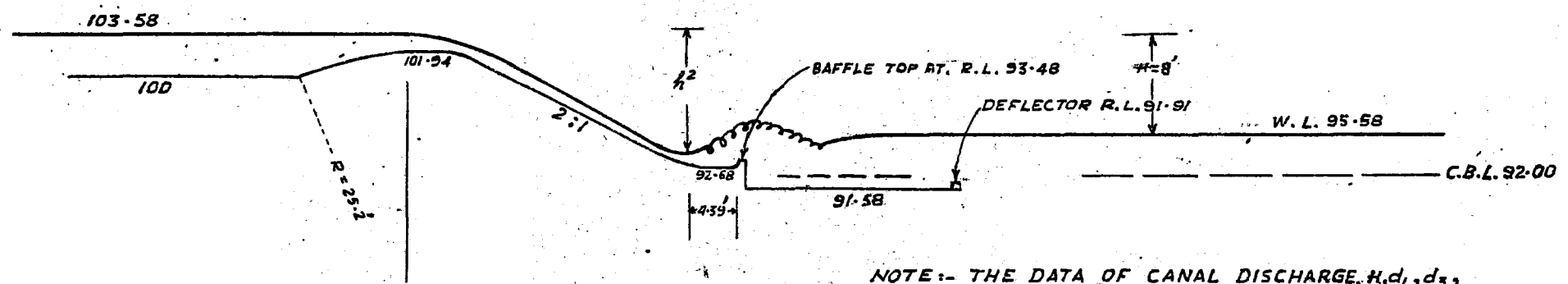
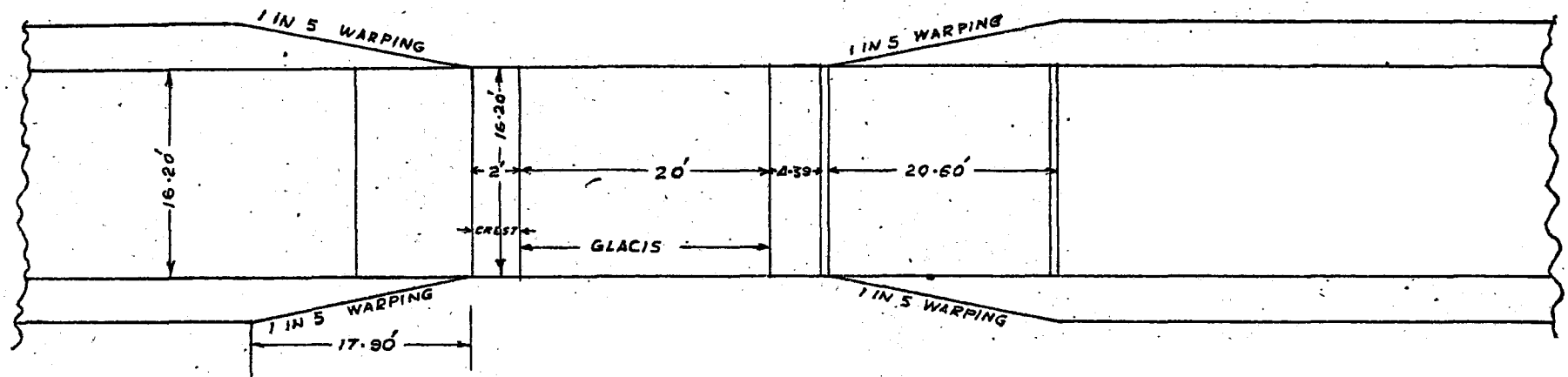
U/S GAUGE.



$D_f = 2.58 \therefore 2.5 D_f^{1.5} = 10.36 = \text{DISTANCE OF U/S GAUGE FROM U/S END OF THROAT.}$

FIG. 2/C-8-1
SHOWING BAFFLE FALL DESIGN
WITHOUT ANY FLUMING FOR Q=100 Cs.

$H = 8', d_1 = d_3 = 3.58'$
SCALE $1'' = 12'$ PROTO
 $= 2'$ MODEL.

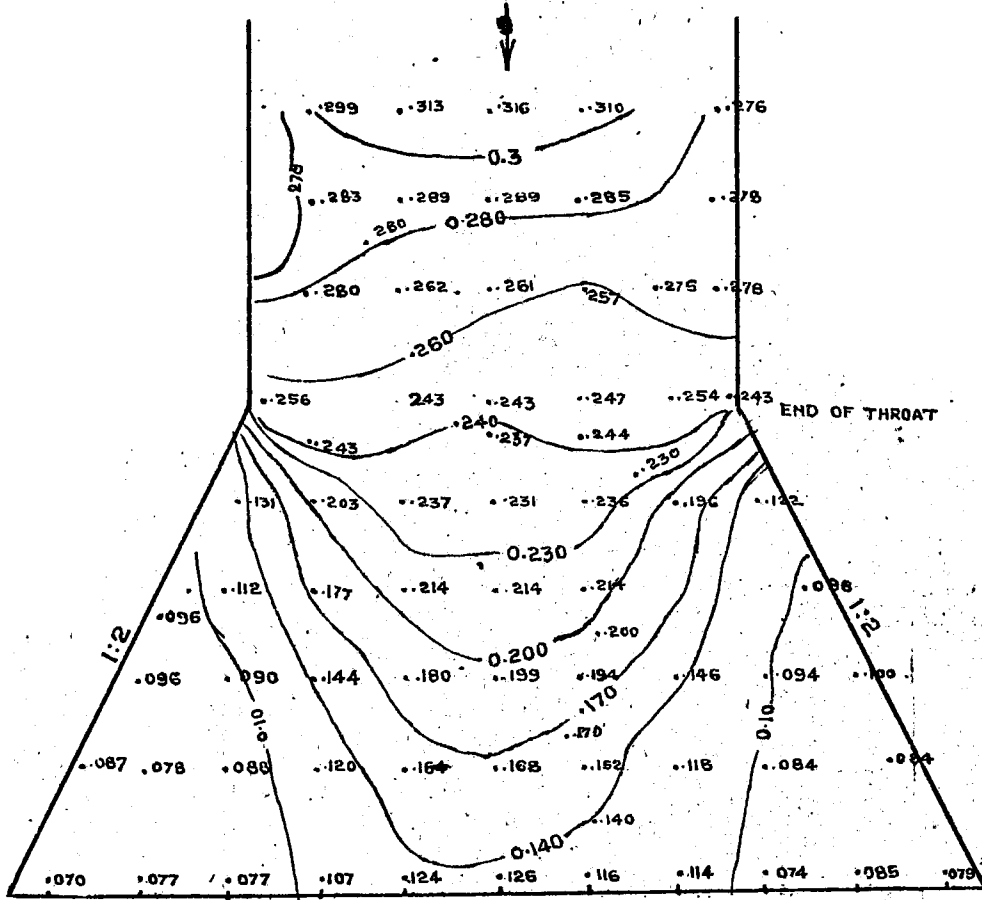


NOTE:- THE DATA OF CANAL DISCHARGE, $H, d_1, d_3,$
ETC. IS THE SAME AS USED FOR THE MODEL.
REPORTED IN A. R. T. 1944.

1/6 MODEL OF VERTICAL FALL
SHOWING WATER SURFACE CONTOURS
WITH SIDES BELOW THE THROAT EXPANDING

AT 2:1
SCALE:- 2" = 1'

PLAN



L.S. ON CENTRE

SCALE:- } HDR = 2" = 1'
 } VER = 1" = 1'

H.F.L. 100.60 ± 103.58

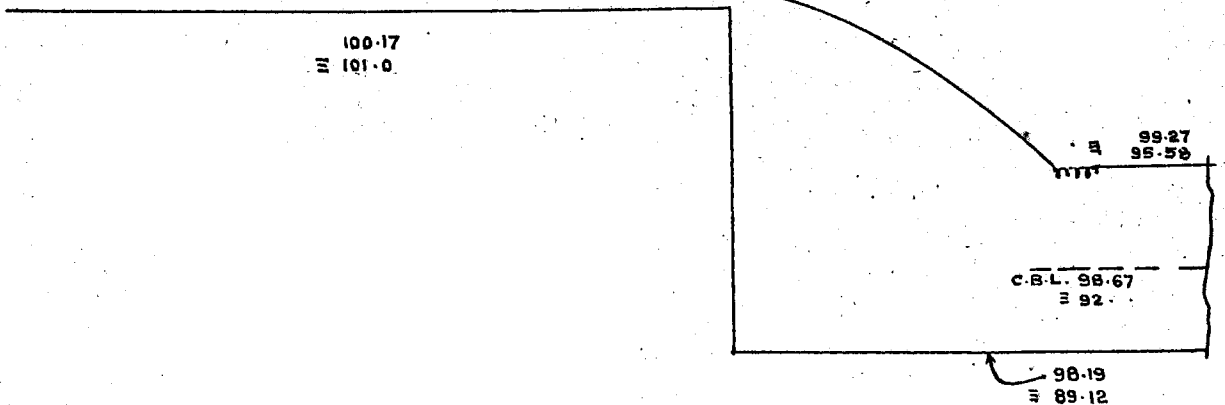
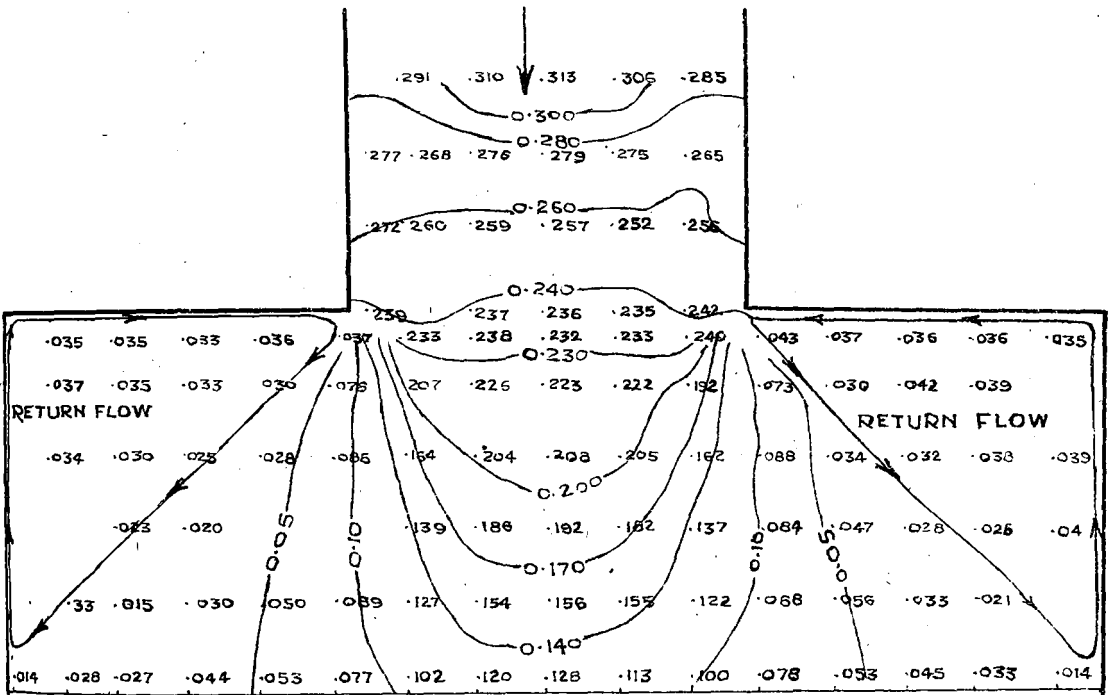


FIG:- 4/C-8-1

1/8 MODEL OF 100 CUSECS VERTICAL FALL
SHOWING WATER SURFACE CONTOURS
WITH FREE EXPANSION BELOW
PARALLEL THROAT

SCALE:- 1" = 0.5'

PLAN



L. SECTION

SCALE:- HOR. 2" = 1'
VER. 1" = 1'

H.F.L. \equiv 103.58

R.L. 100-17
 \equiv 101.0

W.L. 99.27
 \equiv 95.58

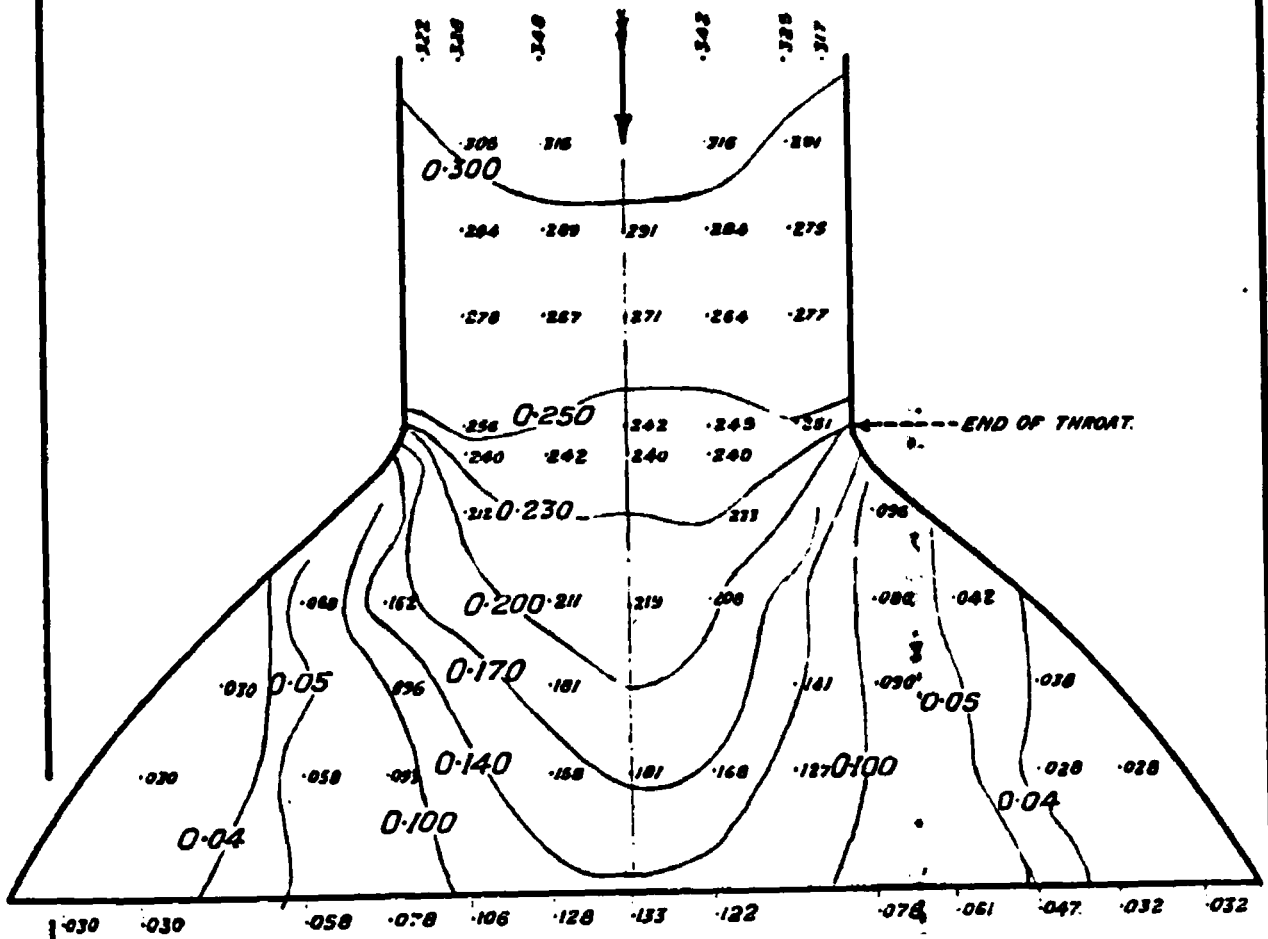
$d_3 = 0.6 \equiv 3.58$

D/S C.B.L. \equiv 92
 \equiv 98.67

CISTERN BED R.L. 98.19
 \equiv 89.12

FIG. 5/C-8-1

1/6 THE MODEL OF VERTICAL FALL
SHOWING WATER SURFACE CONTOUR IN THE THROAT
OF THE VERTICAL FALL WITH D/S SIDES
BELOW PARALLEL THROAT EXPANDING
ALONG THE EXTREME FLOW LINES.
PLAN SCALE - 2" = 1 FT.



L. SECTION.
SCALE { HOR. 2" = 1 FT.
VER. 1" = 1 FT.

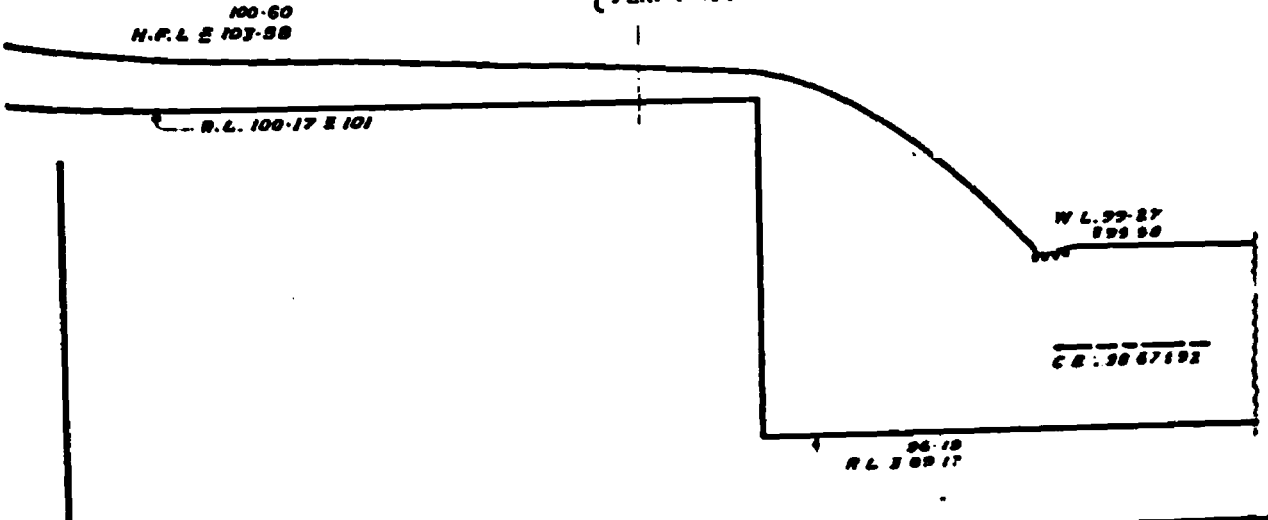


FIG. 6/C-8-1.

1/6 G.S. MODEL OF VERTICAL FALL
SCOUR OBTAINED AFTER ONE HOUR IN
EXPT. WITH NORMAL W.L. D/S;
CISTERN FLOOR LENGTH
INCREASED TO 16 FT.

$Q = 100 \text{ cs}, H. = 8'; d_1 = d_3 = 3.58'$

SCALE: ~ 1" = 2' (MODEL)
= 12' (PROTO.)

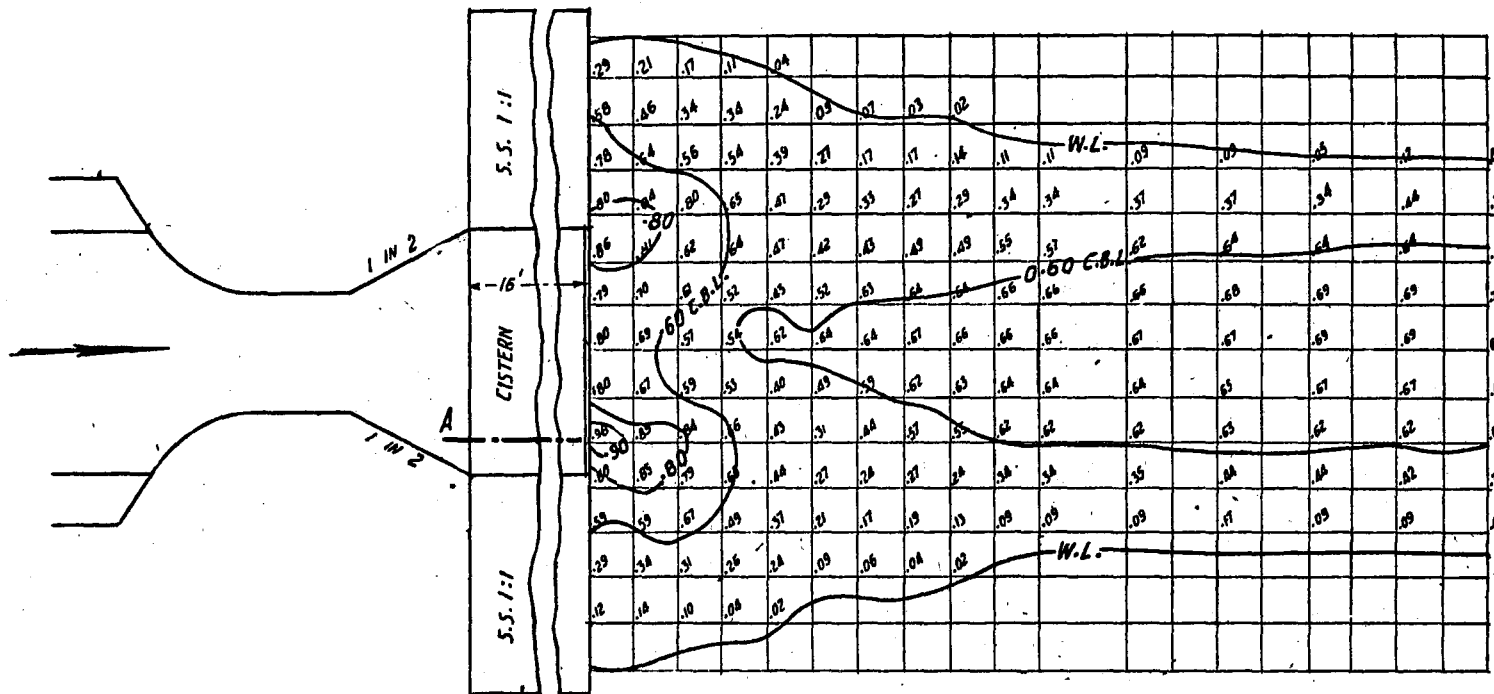
NOTE.

DATUM FOR SOUNDINGS = DESIGNED W.L. D/S.

0.60 \equiv C.B.L. CONTOUR R.L. 92.00.

0.80 \equiv 1.2' BELOW C.B.L. i.e. R.L. 90.8.

0.90 \equiv 1.8' " " " R.L. 90.2



U/S W.L. \equiv 103.58

LONGITUDINAL SECTION ON A-B.

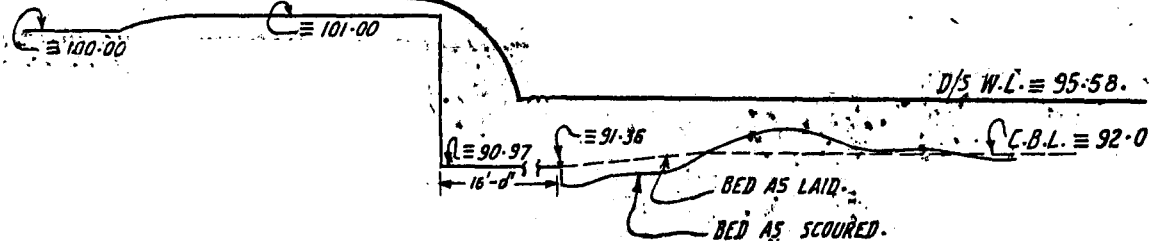


Table 1/C-8-1

1 in 2 side expansion below the throat						Free expansion, i. e. no restriction below parallel throat					Sides below parallel throat expanding along extreme lines of flow obtained under free expansion.					
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
Model chainage across section from centre as starting point.	1' (L)	6"	0	6"	1' (R)	1' (L)	6"	0	6"	1' (R)	1' (L)	6"	0	6"	1' (R)	
\bar{V} ft/sec.	3.95	3.97	3.61	3.99	3.83	4.08	3.89	3.66	3.95	4.10	4.01	3.78	3.62	3.81	4.06	
d (ft)	0.196	0.236	0.231	0.237	0.203	0.192	0.222	0.223	0.226	0.207	0.200	0.233	0.230	0.235	0.212	
q cusecs/ft.	0.774	0.936	0.833	0.945	0.790	0.785	0.864	0.816	0.895	0.850	0.800	0.880	0.833	0.896	0.860	

Note - Zero chainage was along the centre line of the throat.

Even with a deeper cistern, though a cistern length 3.94 ($D_2 - D_1$) ft. gave satisfactory scour conditions under *normal* water levels, yet when the experiment was run with 25% *retrogression* of tail water level the surface turbulence—see photo 5159 — indicated the necessity of a greater length of cistern.

Photo 5159
Sheet 1

Increasing the floor length to 25ft., i.e. 7.48 ($D_2 - D_1$) beyond where the nappe fell, reduced the intensity of return flow at sides, and the conditions of surface turbulence were the best of all observed — see photo 5173. The final scour pattern (fig.8 and photo 5179), though not ideal, was considered better than any previously obtained because the deep scour was central and away from the pukka work, i. e. not near the deflector — see longitudinal section of fig.8.

Photo 5173
Sheet 2
Fig.8/C-8-1
Photo 5179
Sheet 2

Thus, the experiments made during the year indicate that unless measures are taken to make the flow uniform and natural at the exit section, a cistern length of about 7.5 ($D_2 - D_1$) is required in this type of Fall.

In his paper "Development of Hydraulic Design, Saint Anthony Falls' Stilling Basin" Mr. Fred W. Blaisdell has given the following equation for the length of 'stilling basin' (2)

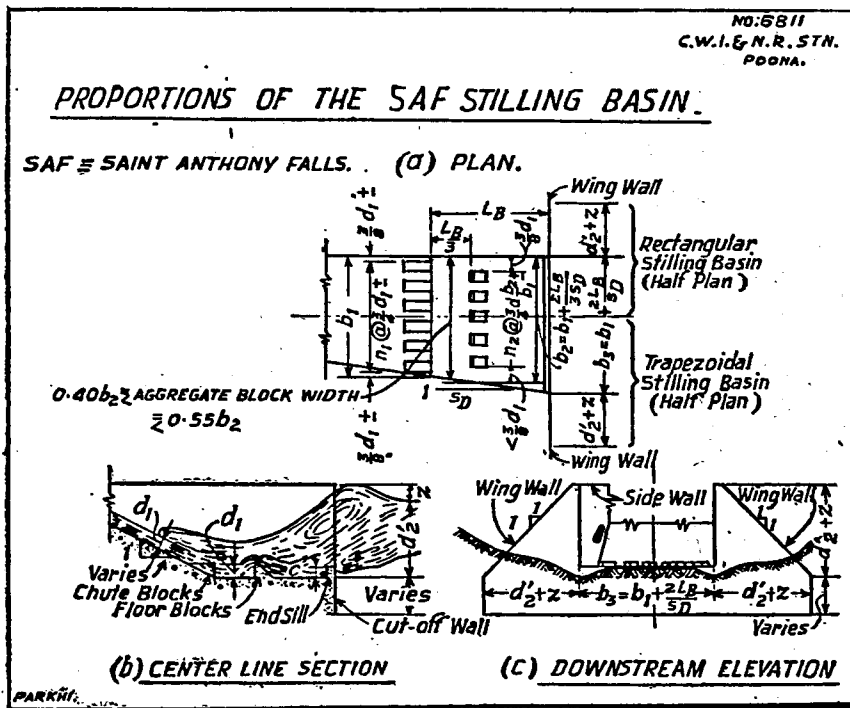


FIG. 6 (a) / C-8-1.

$$\frac{L_B}{D_2} = \frac{4.5}{(Fr.)^{0.38}}$$

where L_B = Length of cistern floor,
 D_1 = Hypercritical depth above standing wave,
 D_2 = Subcritical depth below standing wave,
 Fr = Froude No. above the standing wave = V_1^2 / gD_1

This relation is applicable to S. A. F. (Saint Anthony Falls) designs, which have a glacis of varying slope; chute blocks at the toe of the glacis and floor blocks at one-third the length of stilling basin from toe of glacis (chute blocks and floor blocks of height equal to the depth D_1 of hypercritical jet); and an end sill (i. e.

Fig-6 (a)
C-8-1

(2) Proceedings of the Am. Soc. of Civil Engineers, vol. 73, Feb. 1947, page 134.

deflector)— see fig. 6 (a). The length of the Stilling Basin is suggested to be conservative without being wasteful; and to apply over a range of Froude numbers from 3 to 288.

In the present case, this equation gives :

$$\begin{aligned} \text{Length of cistern floor} &= \frac{4.5 \times D_2}{(Fr)^{0.38}} \\ &= \frac{4.5 \times 3.04}{(9.52)^{0.38}} \\ &= \underline{5.81 \text{ ft.}} \quad \left(\because Fr = \frac{V_1^2}{gD_1} = \frac{12.35^2}{32 \times 0.5} = 9.52 \right) \end{aligned}$$

i. e. 2.29 ($D_2 - D_1$) beyond the point where the nappe falls, against 7.48 ($D_2 - D_1$) necessary according to the model.

In discussing Blaisdell's paper in the December 1947 issue of the Proceedings, American Society of Civil Engineers, J. H. Douma, Senior Hydraulic Engineer, Office, Chief of Engineers, Washington D. C. has given the criterion—

$$L_B = 3 D_2$$

from the U. S. B. R. tests for Stilling Basins of flood control and multipurpose dams, Fourteen cases are cited of the U. S. B. R. Stilling Basins, which are said to be of the horizontal-apron type with baffle blocks and end-sills; all are said to have been model-tested for satisfactory operation under maximum design conditions (range of Kinetic flow factor = V_1^2/gD_1 from 29 to 151); and those that have been constructed in prototype are reported to have operated in a fully satisfactory manner within the range of experienced discharge. This would give, in our case, a length of stilling basin = $3 \times 3.04 = 9.12$ ft beyond the point where the nappe fell, i. e. less than half that indicated by our model tests.

It must be remembered that both Blaisdell's and Douma's results are for lengths of stilling basins with baffle blocks and end sills while, in the case under discussion, there was no dissipation of energy by friction blocks. Douma also gives the results of the Bakhmeteff and Metze tests, (3) & (4) which are for lengths based on the formation of a jump on a horizontal apron without baffle blocks or an end sill. These give for $F = 9.5$, $L_B/D_2 = 5.2$, i. e. $L_B = 5.2 \times 3.04 = 15.8$ ft. which is still a little shorter than indicated as desirable by our model.

Thus, it appears that Vertical Falls require a relatively longer cistern floor; while the Blaisdell's figures of the S. A. F. stilling basin are even too short compared to the usual requirements of glaciis falls with friction blocks. Further experiments are proposed to relate cistern lengths to D_2 and/or Fr in case of Vertical Falls.

(ii) Depth of cistern:

3.2.2. The design with 16 ft. length of cistern with bed at R. L. 91.76 —
 photo 5112 i. e. 0.24 ft. below C. B. L. so as to give 25% more energy of flow in the cistern—
 sheet 3 did not give satisfactory scour pattern; because scour-holes occurred near the sides right against the deflector, thus showing a marked tendency for undermining at the sides. The deflector was, however, not exposed in the central portion—photo 5112.

(3) Discussions on (2) by J. H. Douma, Proceedings Am. S. C. E., vol. 73, No. 10 (1947)

(4) The Hydraulic Jump in terms of Dynamic Similarity by B. A. Bakhmeteff and A. E. Metze, Transactions of American Society of Civil Engineers, vol. 101, 1936, page 660.

Deepening the cistern bed to R. L. 89.12—designed on the Bombay P. W. D. Hand Book formulae (5)—improved the scour pattern considerably (see Photo 5116) under *designed* conditions of downstream water level in that the scour shifted further downstream away from the floor, and was central; yet “bellying” was not eliminated, as apparent from the shape of the canal bed level contour (R. L. 92.0) in fig. 7 and photo 5116.

Photo 5116
sheet 3
Fig. 7
C-81

(iii) Width of Cistern and Height of deflector.

3.2.3. Widening the cistern from $\cong 16.20$ ft to 27 ft. to easily accommodate the whole nappe (thd end parts of which were fouling against the sloping sides), and making the sides vertical, worsened scour conditions—see photo 5126—though, to the eye, the surface turbulence did not appear more severe, see photo 5127.

Photos 5126
& 5127
sheet 4

This photo also shows that the cistern was a little too wide and when reduced (to $\cong 21$ ft.) to be just sufficient for the falling nappe, the scour pattern was satisfactory — see Photos 5129, 5134 & 5136—though the surface turbulence was still severe, see photo 5136.

Though the final scour with normal downstream water level was satisfactory (vide Photo 5134) — except for “bellying” of the sides common to all these experiments — when the same design was tested with 25% retrogression of downstream water level the deflector was undermined in the centre, as seen in photo 5142.

Photos 5129
5134 & 5136
sheet 5

This shows how necessary it is always to test the final design under retrograded downstream water levels to avoid resting under a false sense of security.

Photo 5142
sheet 6

In the above experiments, very excessive turbulence was observed over the deflector which, in effect, was 2.88 ft. above cistern bed ($= d_z/2.24$, since d_z , the normal depth of water in the cistern, was 6.46 ft.). The bottom forward flow in the cistern, after impinging against the deflector, shot up partly to flow downstream and partly to flow back upstream along the surface, causing a surface wave, the water surface over the deflector being higher than the main water level.

The next experiment was also done with 25 % retrogression of downstream water levels but with the deflector only 0.36 ft. high over cistern bed $= \frac{d_z}{15.5}$ where d_z , was the depth in the cistern with retrograded downstream water level), the top of the deflector being at R. L. 89.48 against R. L. 92.0 in the previous experiment. This improved surface turbulence noticeably — see photo 5159— though surface return flow at sides persisted. The deflector height tallies closely with the optimum height of end sill given by Douma (see Footnote 3) $= .125 \times D_2 = .125 \times 3.04 = .38$ ft. and is rather higher than Blaisdell's(2), suggestion of $0.07 D_2 = 0.21$ ft.

Photo 5159
sheet 1

To minimize the side return flow, with the same bottom width ($= 21.34$ ft.) of cistern the sides were made to slope at 1:1 and the experiments repeated under 25 % retrograded water levels. Surface turbulence appeared much smoother and return flow at the sides was also noticeably reduced in extent; bed scour was less; the deflector was not undermined; but side scour below the cistern persisted — see photo 5168 The characteristic flow pattern was still the same: at the deflector section the surface velocity was slacker in the centre and higher at about quarter width away on either side of the centre.

Photo 5168
sheet 1

(5) Bombay P. W. D. Hand Book, page 522 (1925 edition) gives the following formulae for design of water cushion — see Fig. 7 (a).

$$(1) x = H + \sqrt[3]{H} \sqrt{d}$$

$$(2) y = 4/3 \sqrt{z} \times H$$

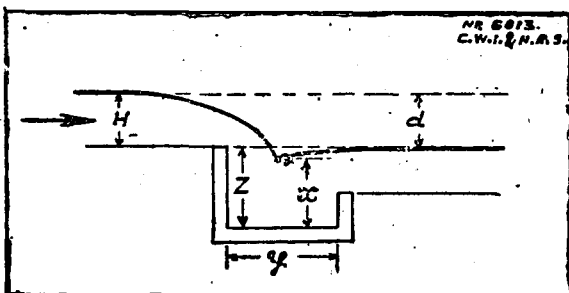


FIG. 7 (a)/C-8-1.

where H = Depth of water over top of weir.
d = Height of fall.
z = Difference of levels of the crest & the bed of water cushion.

With the same design, when the floor length was increased to 25 ft. (i e. 7.48 ($D_2 - D_1$) beyond the point where the nappe fell) return flow at the sides also reduced — see photo 5173 — and the scour pattern was the best of any obtained in this series, vide para 3.2.1 *supra*.

Photo 5173
sheet 2

3.3 Review.

The condition of flow in the cistern in all the tests was as seen in photo 5108; the surface flow near the sides was forward but heading towards the bank, while in the centre it was backwards from the deflector towards the nappe and, for the lower filaments, the flow indicator showed that there was forward flow in the bottom 1 ft. depth only (out of the total depth of 4.61 ft.) and that, too, occurred in intermittent spurts of fast current.

Photo 5108
sheet 6

3.3.1 A feature, but for which the scour pattern would have been wholly satisfactory, was the bellying of the scour contour lines at the sides. This is due to the nature of flow conditions, even in the best design so far tested, as seen in photo 5173, where two forward flow jets near the sides can be clearly seen. Under these conditions of flow, the best scour conditions obtainable appear to be those of fig. 8.

Photo 5173
sheet 2
Fig. 8
C-8-1

3.3.2 To ensure an ideal pattern of scour it is necessary to produce natural forward flow at the exit section of the deflector; and after a number of exploratory tests it was found that the addition of a central pier gave the desired results.

Figs. 9 (a)
to 9 (d)
C-8-1

3.3.3 Figures 9(a) to 9(d) show the surface and bed lines of flow in the throat, without and with a pier. The side divergence downstream of the parallel-sided throat were :-

(a) 1 in 2 — [figures 9 (a) & 9 (b)]

(b) along extreme line of flow line of unrestricted flow below the throat — (fig 9(c) and 9(d)).

It will be observed that the effect of a central pier is to pull back the flow lines, which otherwise tend to flow away from the centre, towards it. This result may be explained by the fact that due to the form drag of the pier sides, the velocities just near the pier face decrease relatively more than at quarter widths of the throat, deflecting the flow lines towards the centre.

3.3.4 These studies on *flumed* Vertical Falls are still in progress:

(a) in respect of the effect of the central stabilizing pier on scour and bellying;

(b) for obtaining the optimum and cheapest design;

and (c) relating the dimensions to flow parameters and dimensionless numbers, i. e. standardizing the design.

4. PART II—NON-FLUMED BAFFLE FALLS.

4.1 The design shown in fig. 2 worked very well in that there was satisfactory dissipation of energy in the primary and secondary standing waves resulting in no scour downstream. From preliminary tests the pavement length was reduced to 5.58 ft. = 2.19 ($D_2 - D_1$) and, even so, the results as regards scour and flow pattern were satisfactory.

Fig. 2
C-8-1

With the glacis sloping at 1:1, and a pavement length of 14.74' = 5.8 ($D_2 - D_1$) (so that the deflector remained at the same distance relative to the crest of fall) also gave satisfactory scour conditions — see photo 5113.

Photo 5113
sheet 7

FIG. 7/C-8-1

1/6 G. S. MODEL OF VERTICAL FALL
SCOUR WITH CISTERN DEEPENED TO
R.L. 89.12 ACCORDING TO BOMBAY,
P.W.D. HAND BOOK AND DEFLECTOR

TOP AT R.L. 92 i.e. AT C.B.L.

$Q = 100 \text{ C.S.}, H = 8', d_1 = d_3 = 3.58'$

SCALE:- 1" = 2' (MODEL)
1" = 12' (PROTO.)

NOTE:- DATUM FOR CONTOURS
WAS THE DESIGNED W.L. D/S

- 0.60 ≡ C.B.L. CONTOUR R.L. 92.00
- 0.80 ≡ 1.2' BELOW C.B.L. i.e. R.L. 90.8'
- 0.85 ≡ 1.5' " " " R.L. 90.2
- 1.00 ≡ 2.4' " " " R.L. 89.6

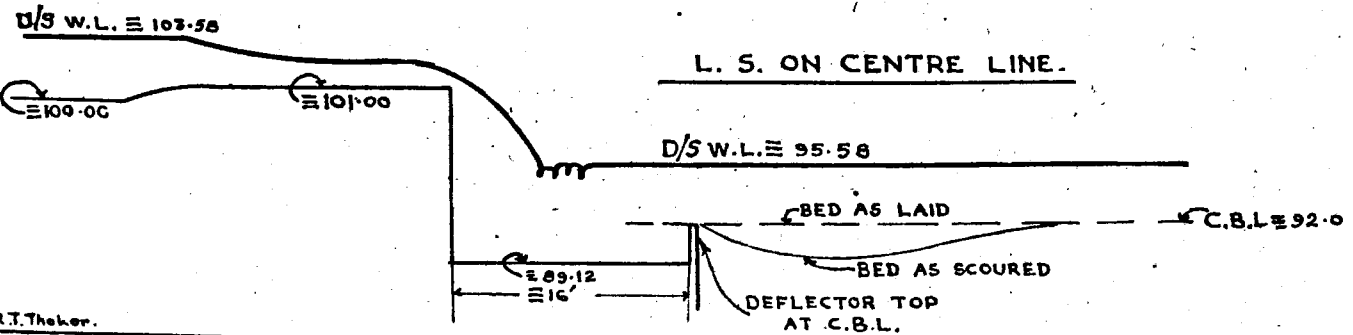
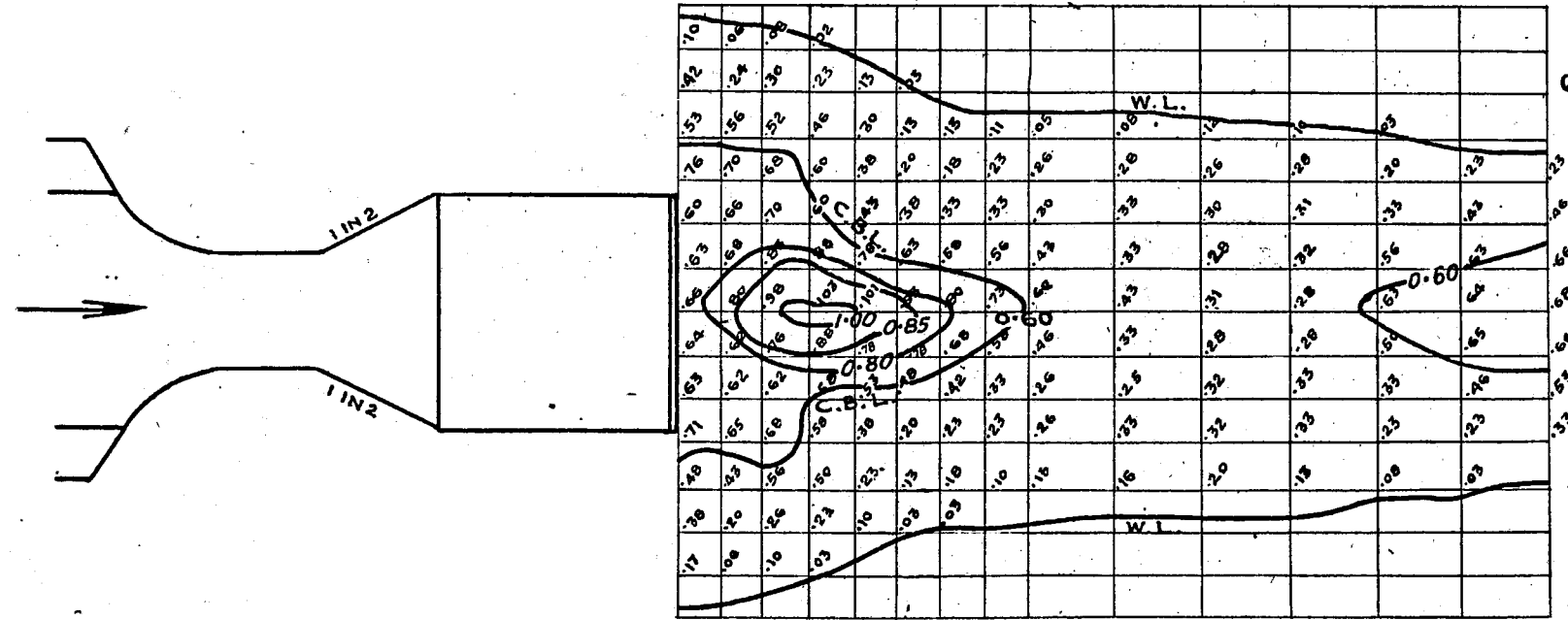
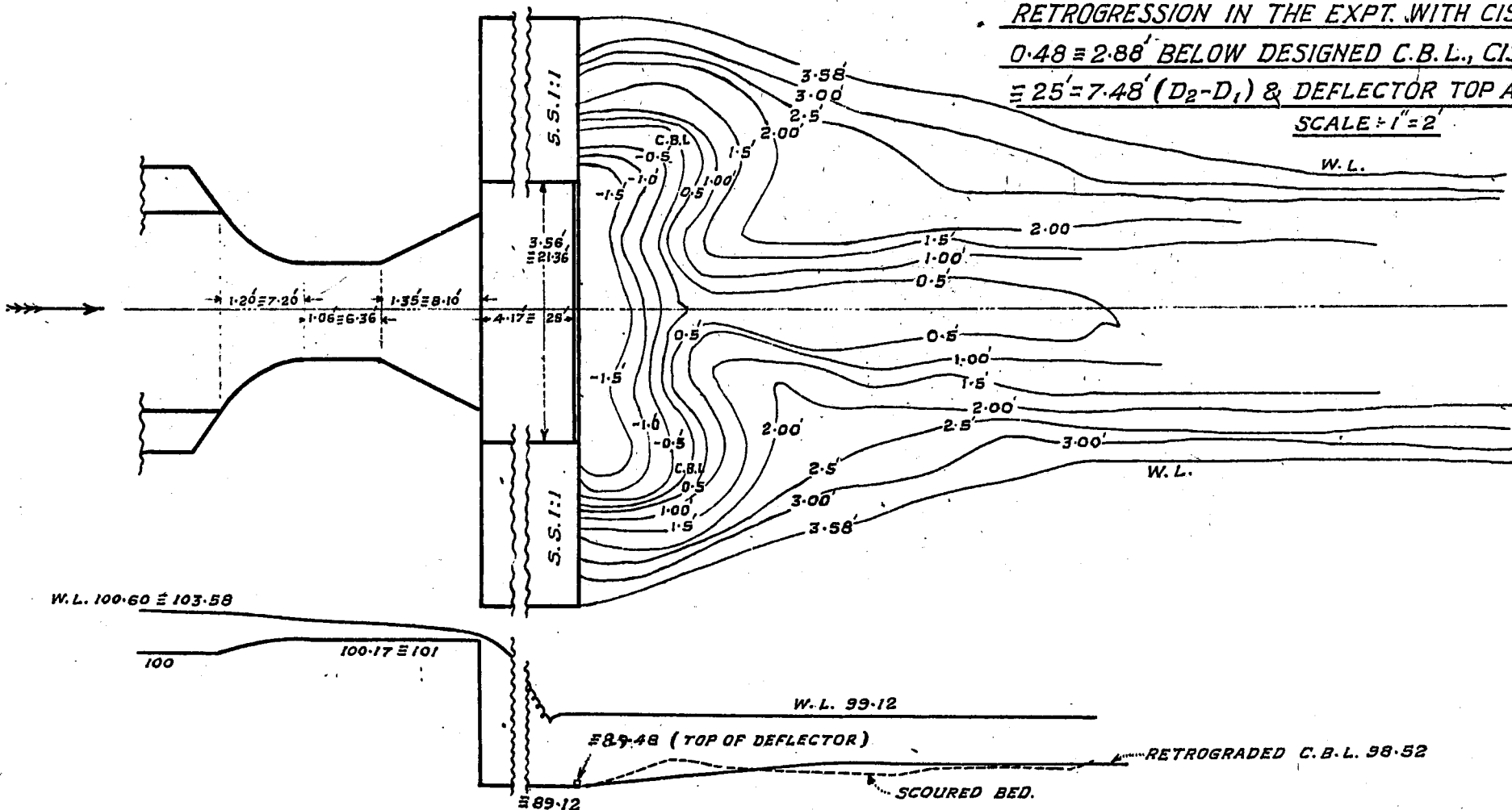


FIG. 8/C-8-1

1/6 G.S. MODEL OF 8 Ft. 100 Cs. VERTICAL FALL
SHOWING FINAL SCOUR AFTER 12 Hours. FLOW WITH 25%
RETROGRESSION IN THE EXPT. WITH CISTERN AT R.L. = 89.12,
0.48 = 2.88' BELOW DESIGNED C.B.L., CISTERN LENGTH
= 25' = 7.48' ($D_2 - D_1$) & DEFLECTOR TOP AT R.L. = 89.48

SCALE $\pm 1" = 2'$



FIGS. 9 (a) TO 9 (d).

C-8-1

NO 5817
C. W. I. & N. R. STN

1/8 MODEL OF 100 CUSECS VERTICAL FALL
SHOWING LINES OF FLOW

SCALE 1"=1'

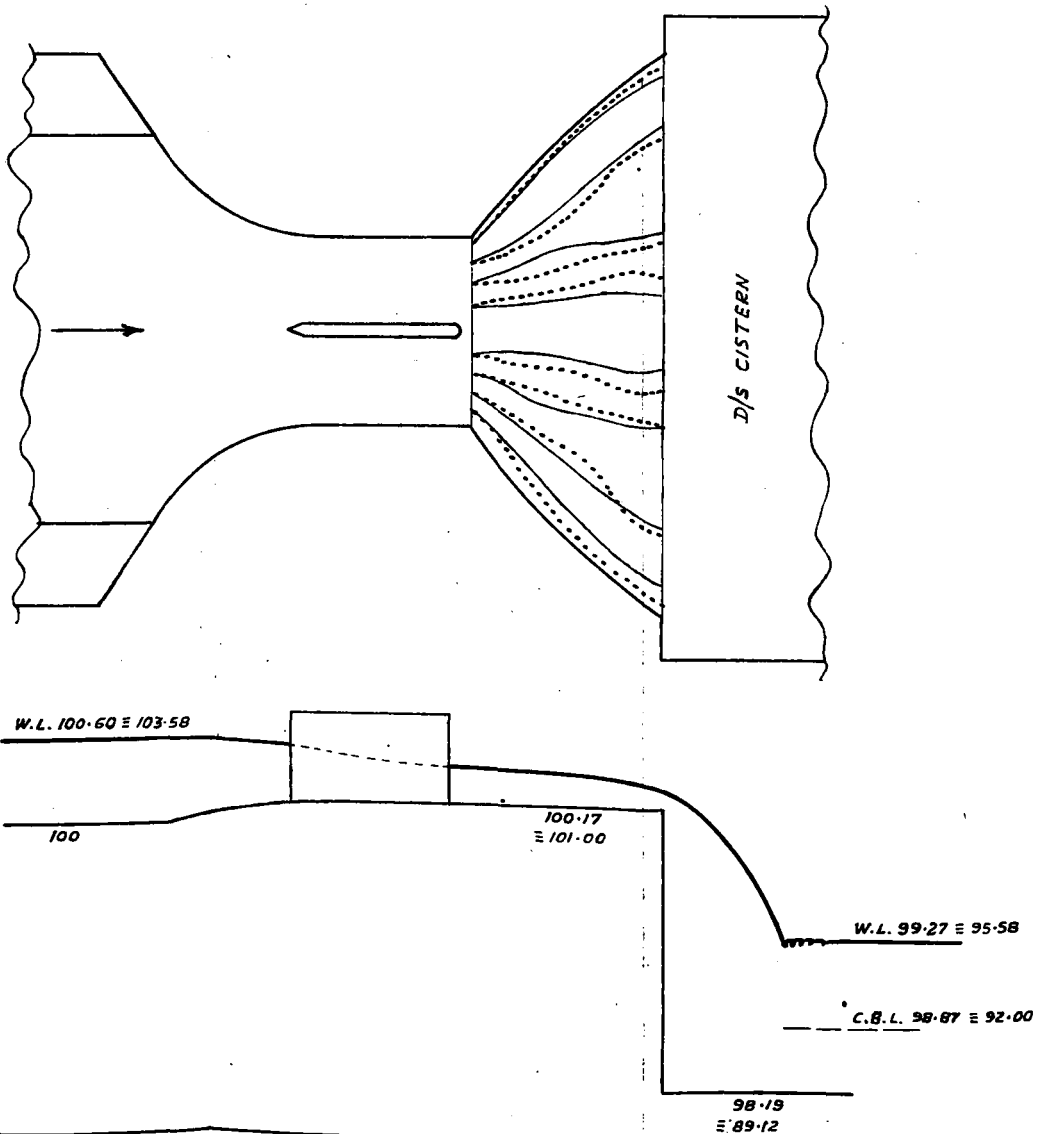
OF FLOW.
W.

FIG. 9(d) D/S OF THE THROAT WITH SIDES EXPANDING

C-8-1

ALONG EXTREME LINE OF UNRESTRICTED

FLOW BUT WITH A CENTRAL PIER



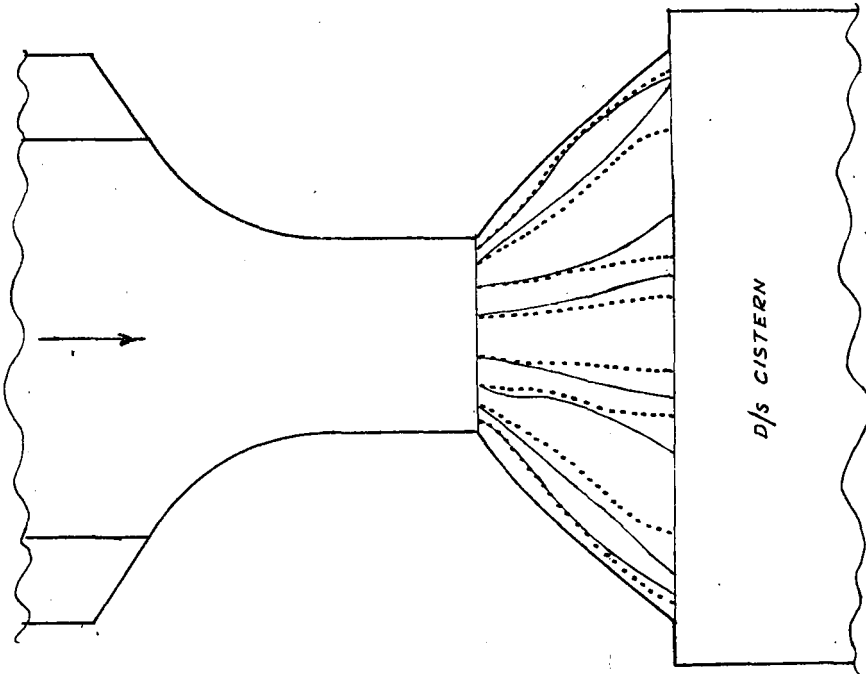
92.00

REFERENCE

———— SURFACE LINES

..... BED LINES OF FLOW

FIG. 9(c) D/s THROAT WITH SIDES EXPANDING
C-8-1 ALONG EXTREME LINE OF UNRESTRICTED
FLOW AND WITHOUT CENTRAL PIER.



W. L. 100-60 \approx 103-58

100
100-17
 \approx 101-00

W. L. 89-27 \approx 95-58

C. B. L. 98-67

98-19 \approx 89-12

FIG. 9(a) WITH 2:1 SIDES BELOW PARALLEL
C-8-1 THROAT & WITHOUT PIER

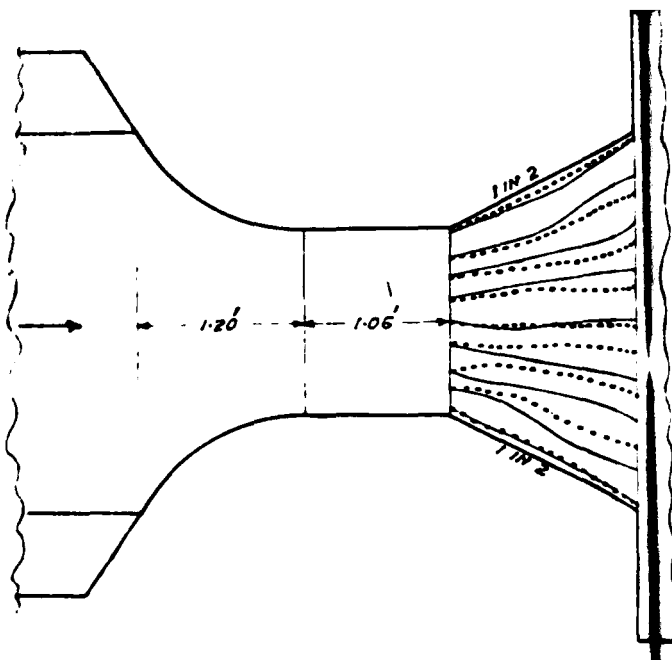
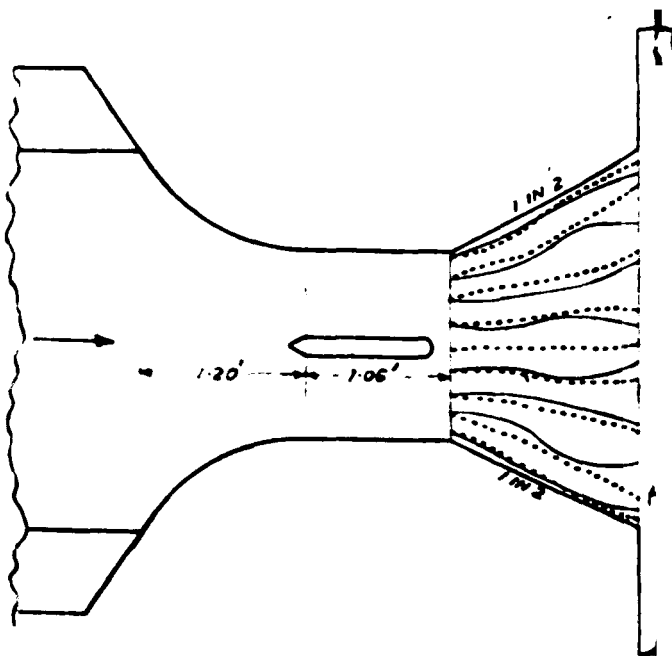
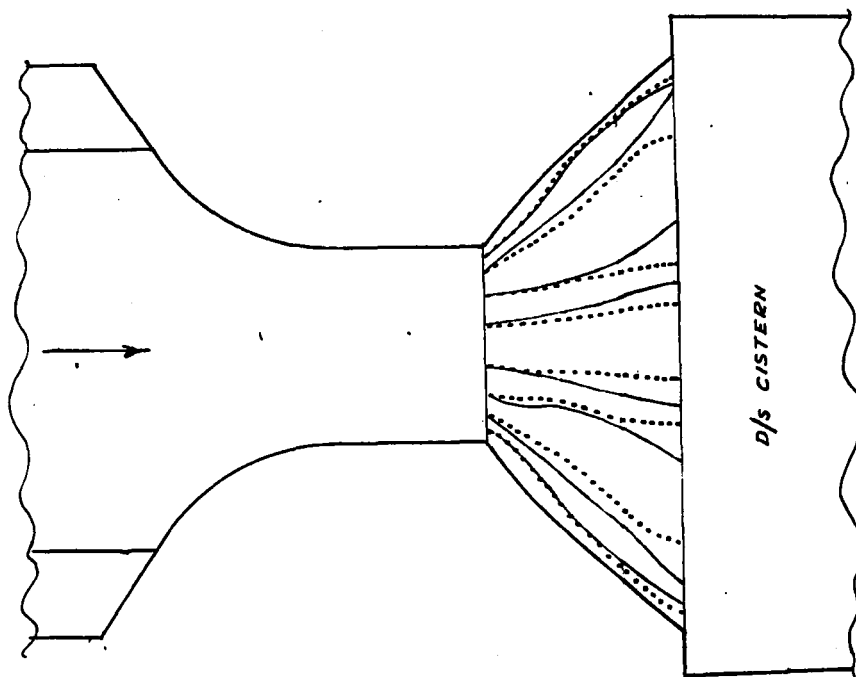


FIG. 9(b) WITH 2:1 SIDES BELOW PARALLEL
C-8-1 AND WITH CENTRAL PIER.



REFERENCE
 ——— SURFACE LINES
 BED LINES OF FLOW

FIG. 9(c) D/S THROAT WITH SIDES EXPANDING
C-8-1 ALONG EXTREME LINE OF UNRESTRICTED
FLOW AND WITHOUT CENTRAL PIER.



W.L. 100.60 = 103.58

100

100.17
 = 101.00

W.L. 89.27 = 93.60

C.S.L. 93.60

88.19 = 89.12

Making the glacis slope 2 : 3 (i. e. 2 horizontal: 3 vertical) so that the pavement length became 7 (D_2-D_1) gave negligible scour, the maximum final depth of scour with 25 per cent retrogression of downstream water level being only $\cong 9''$ in the centre, 6' away from the deflector — see photo 5143; also with glacis slope of $\frac{1}{2} : 1$ and the pavement length = $1.5' = 5.9$ (D_2-D_1) the energy dissipation was very effective (photo 5160) and there was no scour downstream as seen from fig. 10.

Photo 5143
sheet 7

Photo 5160
sheet 7

Fig. 10

C-8-1

Finally, with a view to study the minimum length of pukka floor required, when the bed immediately below the baffle was made erodible (i. e. there was no cistern), the maximum depth of scour obtained was 1.2' below the retrograded canal bed level; the scour pattern was central and surface velocity distribution (as tested by floats) was also normal, being high in the centre and low at sides.

It must also be emphasised that, with 25 % retrogression of downstream water levels, the hydraulic jump is swept out of the Stilling Basins in all types of Falls tested at the Station *except the Station type Baffle Fall, which works equally satisfactorily with or without retrogression.*

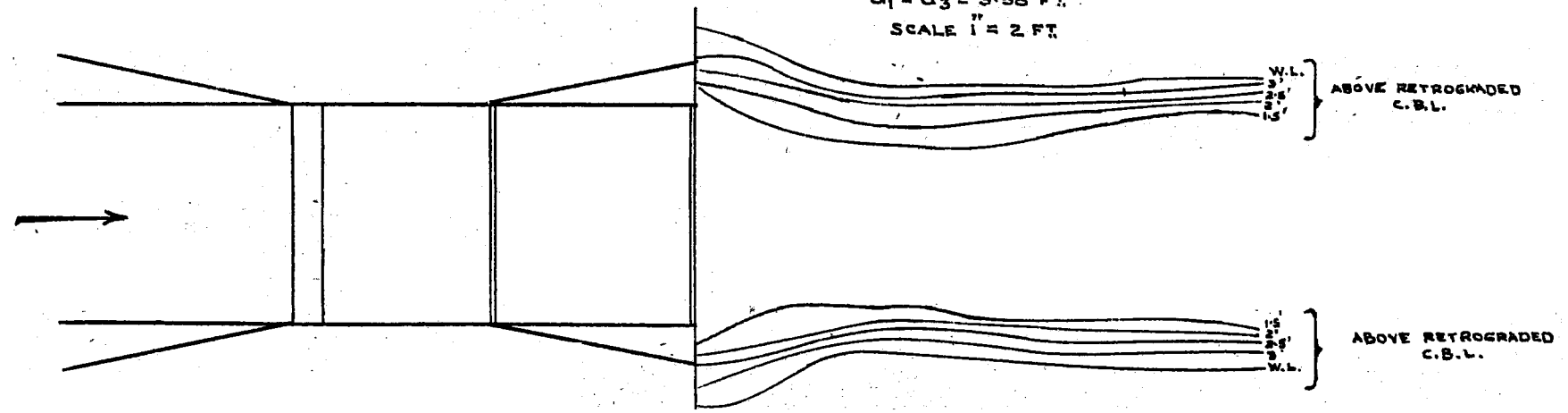
Thus, exploratory experiments made so far with the Station type design for a non-proportional, *unflumed*, Baffle-type fall have shown that 1/2:1 glacis and about 2.2 (D_2-D_1) feet long cistern below the baffle gives sufficient length in which turbulence is sufficiently damped down. Thus, the preliminary tests have confirmed, as is only to be expected, that where Fluming is not required the Station type design can be substantially cheapened.

The difference in cost obtained by adopting a steeper glacis and shorter pavement in large falls will be appreciable.

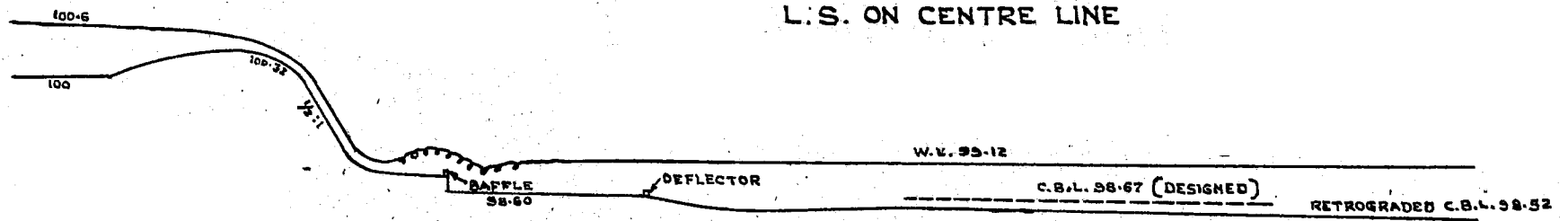
Further experiments are in progress to evolve the most economical *unflumed* design of the Baffle type Fall.

FIG. 10/C-8-1
 $\frac{1}{6}$ G. S. BAFFLE FALL MODEL
 SHOWING SCOUR AFTER 1 HOUR'S FLOW
 WITH $\frac{1}{2} : 1$ GLACIS AND 25% RETROGRESSION

Q = 100 CUSECS
 H = 8.9 FEET
 $d_1 = d_3 = 3.58$ FT.
 SCALE 1" = 2 FT.



L.S. ON CENTRE LINE



T. BY - PATKIE, 13/3/48.

FIG 11/C-8-1

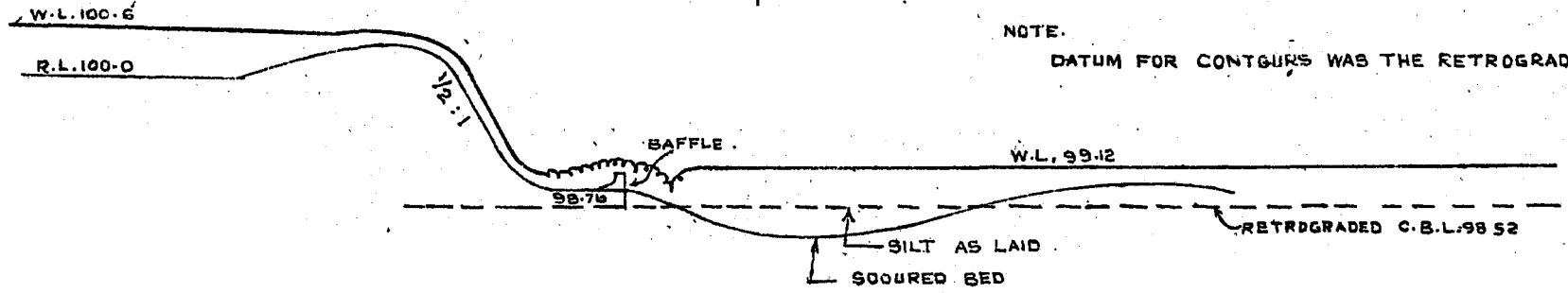
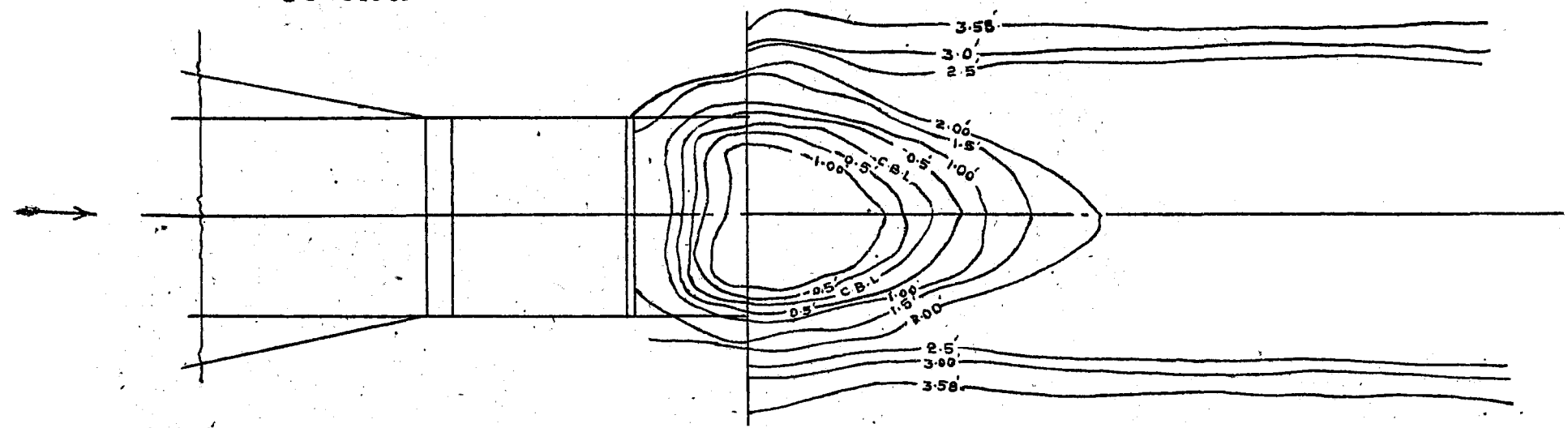
1/6 G. S. MODEL OF BAFFLE FALL

FINAL SCOUR AFTER 12 HRS. FLOW WITH 25% RETROGRESSION
IN EXPT. WITH BED BELOW BAFFLE MADE ERODIBLE;
SIDES WERE HELD FOR A LENGTH 1.5 D/S OF BAFFLE

GLACIS SLOPE 1/2:1

SCALE: - 1" = 2 FT.

Q = 100 CUSECS
H = 8.9 FT
d = d3 = 3.58 FT.



NOTE.
DATUM FOR CONTOURS WAS THE RETROGRADED D/S W.L.

C—HYDRAULIC WORKS.

(10) EXCLUDERS & EJECTORS.

C-10-1. Comparison of sand-exclusion at the nose of the Outer Bank and at the raised sill of the Approach Channel, under present conditions and with Outer Bank extended to position Q—Experiments in 1/80 : 1/40 part-width, Sukkur Barrage Model.

1. Introduction:—

At present, at the entrance of the Approach Channel, there is a deep scour-hole reaching to R. L. 120, i. e. about 75 ft. below Pond level. There is, therefore, concentrated flow at the entrance of the Approach Channel in the middle with return flow regions along the nose and the right bank, the extent of which varies in accordance with upstream flow conditions.

Fig. 1
C-10-1

Due to slack return flow along the Right bank, berms had formed at sections 3 and 4 which slipped when either they became too steep or the water level in the Barrage was suddenly lowered during the December closure. Two such slips occurred in 1944 and 1945 damaging the Bunder wall and the Sukkur Road.

Fig. 1
C-10-1

It was found from model experiments, described in last year's Annual Report⁽¹⁾, that, to start with, the construction of a truncated nose about 500 ft. upstream of the present nose and the subsequent continuous extension of the Outer bank to the truncated nose would reduce scour at the entrance of the Approach Channel due to the flared entrance and the flat slope of the nose (5 : 1 slope of the truncated nose as against 2½ : 1 of the present nose).

In addition to the improvement in local conditions, it was also expected that the general flow conditions would improve and insure better sand exclusion at the nose and at the raised sill.

2. The following experiments were carried out to test the effect of the extended bank on sand-exclusion:—

Experiment (1) : -

*Exclusion at the nose and at the raised sill
with the existing unextended Outer Bank.*

The 1/80 : 1/40 part-width model was laid in Koregaon sand of 0.34 m. m. dia. to the 1947 soundings.

The following discharges and gauges were maintained :

Q River	2,75,000 cs.
Q App. Channel	45,000 cs.
Q Tail Channel	27,000 cs.
Q Pocket	18,000 cs.
Outfall gauge	R. L. 196.8

Water level downstream of Barrage R. L. 193.3

Surface and bed lines of flow were observed by wool thread in the Approach Channel.

Fig. 2
C-10-1

Experiment (2) : -

The Outer bank of the Approach Channel was extended to position Q and the scour-hole at the present nose was filled up to R. L. 175 as obtained in previous experiments. Same gauges and discharges as in Experiment 1 were maintained.

Fig. 3
C-10-1

Experiment (3) : -

Though actually no appreciable scour occurred on the Approach Channel side of the nose in Experiment 2 the bed was laid down to R. L. 160 (as shown in Fig 4) to impose the worst conditions, assuming a scour-hole would form, though of less depth than under existing conditions due to the flat slope of the extended nose.

Fig. 4
C-10-1

(1) Annual Report, 1946. Indian Waterways Experiment Station Poona. pp. 17-27.

3. Results

For efficient sand exclusion at the raised sill, the flow in the Approach Channel should follow the right bank; actually under the present conditions it hugs the Outer Bank. This partly neutralises the curvature of the Approach Channel intended to exclude bed material at the raised sill. The exclusion at the nose has also deteriorated since the bed material is attracted into the deeper scour-hole on the Approach Channel side.

With continuous extension of the Outer Bank to the position Q, the flow conditions were appreciably improved, both at the entrance of the Approach Channel and at the raised sill.

There was no return flow at the entrance of the Approach Channel and the surface flow followed the Right bank, thus establishing the desired favourable curvature for sand exclusion.

Table 1 shows distances of bed divides from Right bank at Outfall and CD sections; i. e. the widths of bed material which entered the Approach Channel and the right pocket, respectively.

In spite of the wider entrance of the Approach Channel when extended to position Q, the width of bed material drawn by the Approach Channel was less, being only 450 ft. against 530 ft. under present conditions. Exclusion at the raised sill was also noticeably better in Experiment 2, as the distance of the divide from Right bank was only 110 ft. as against 160 ft. under present conditions.

With scour-hole laid down to R. L. 160 on the Approach Channel side of the nose in Experiment 3—a severe assumed condition—slightly more bed material was attracted into the Approach Channel as compared to Experiment 2; but this was still better than with the present unextended nose. Exclusion at the raised sill remained as good as in Experiment 2.

Table 1. C-10-1

Expt. No.	Conditions of Experiments	Equivalent Distance of divide at Outfall section. (ft.)	Equivalent Distance of divide at CD section. (ft.)	Remarks.
1	2	3	4	5
Expt. 1	Model laid according to existing Approach Channel conditions.	530	160	
Expt. 2	Continuous extension of Outer bank to truncated nose Q with present scour-hole filled up	450	110	
Expt. 3	Same as Expt. 2 with scour-hole on the Approach Channel side laid to R. L. 160.	490	110	

4. Conclusions

With the extension of the Outer Bank, as already recommended, the deep scour-hole near the nose in the Approach Channel will accrete and slips in the Right bank will be prevented.

Sand Exclusion at the nose and at the raised sill will be materially improved and will remain so even if a scour-hole develops at the entrance of the Approach Channel.

FIG. 1/C-10-1
SHOWING BED CONTOURS AND
SURFACE LINES OF FLOW IN THE APPROACH CHANNEL
(AS EXISTING)
SCALE. 1"=100'

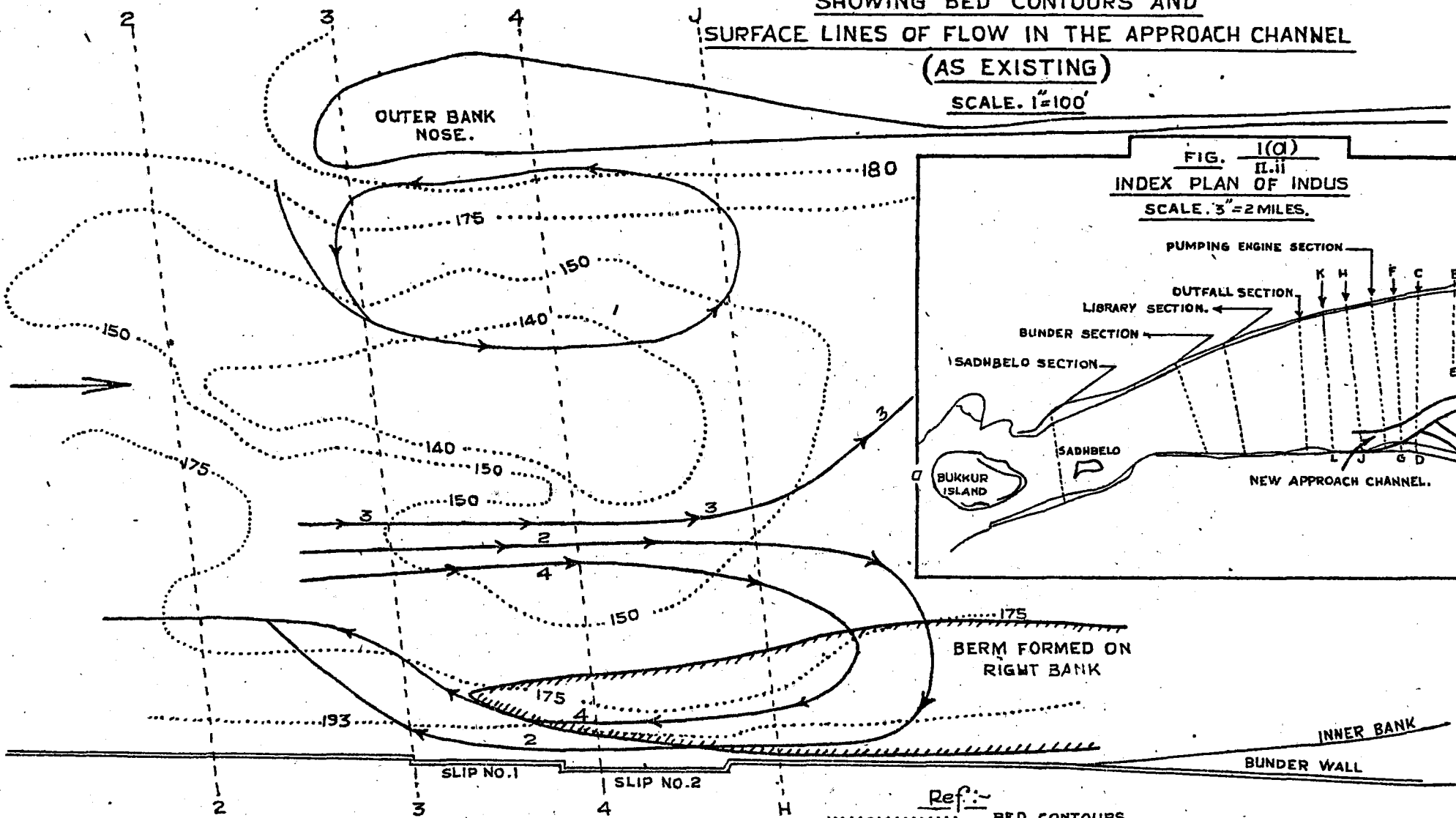
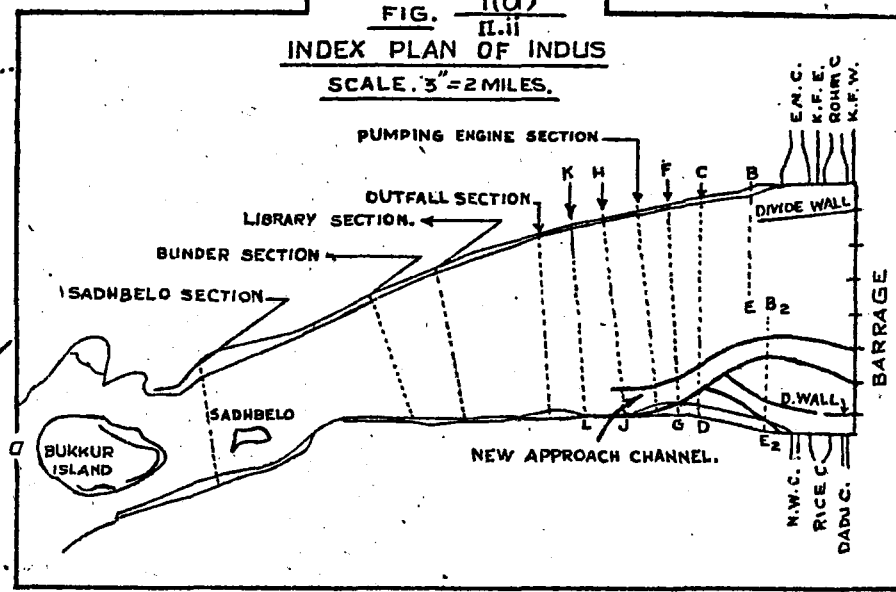


FIG. 1(C)
II.ii
INDEX PLAN OF INDUS
SCALE. 3"=2 MILES.



Ref:~
..... BED CONTOURS.
———— SURFACE LINES OF FLOW.

BY PARKH.I.R.D.

FIG:- 2/C-10-1

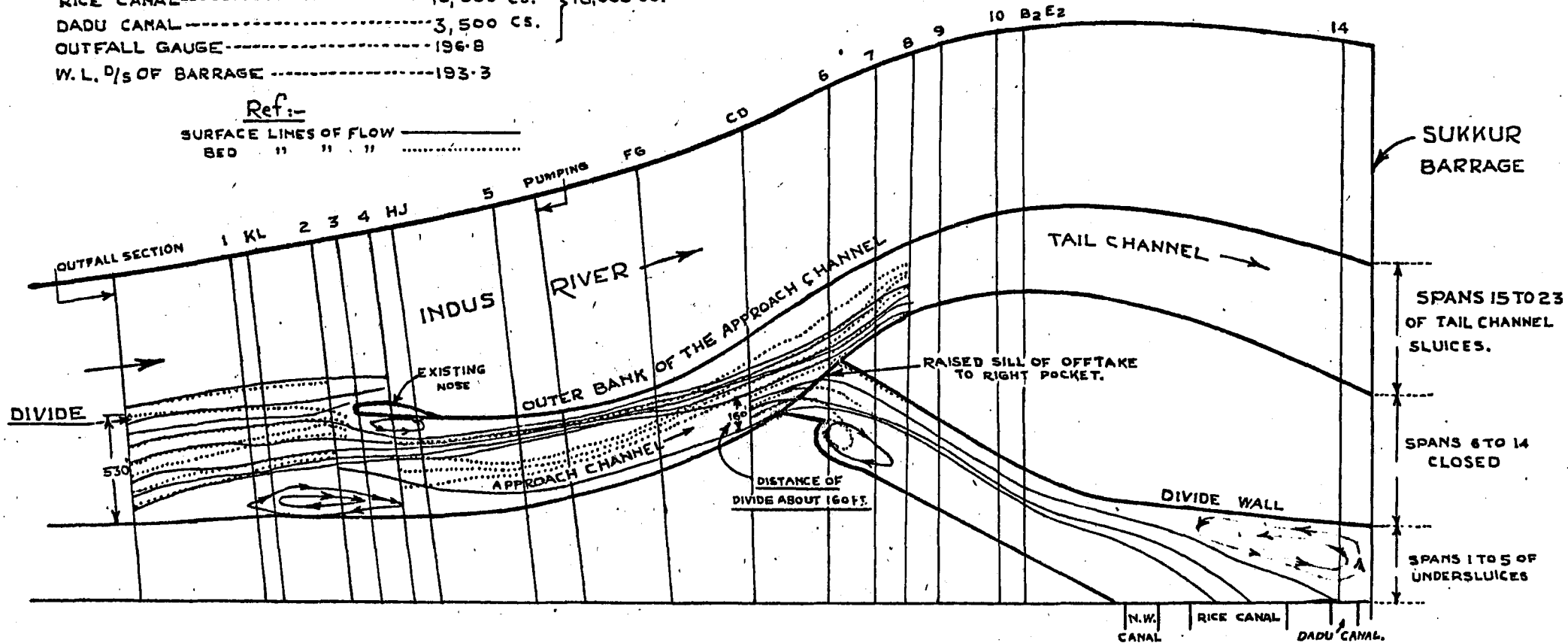
1/80: 1/40 PART WIDTH SUKKUR BARRAGE MODEL
LINES OF FLOW UNDER EXISTING
CONDITION OF THE APPROACH CHANNEL

SCALE 1" = 8' = 640'

EXPT. No. 1

Q RIVER-----	275,000 CS.	
Q APPROACH CHANNEL-----	45,000 CS.	
Q TAIL CHANNEL-----	27,000 CS.	
Q POCKET-----	18,000 CS.	
N. W. CANAL-----	4,500 CS.	} 18,000 CS.
RICE CANAL-----	10,000 CS.	
DADU CANAL-----	3,500 CS.	
OUTFALL GAUGE-----	196.8	
W. L. ^D / _S OF BARRAGE-----	193.3	

Ref:-
SURFACE LINES OF FLOW _____
BED " " " " _____



1/80:1/40 PART WIDTH SUKKUR BARRAGE MODEL.

NO. 5729
C.W.I. & N.R. STR. POONA.

SCALE. 1" = 8' = 640'

FIG. 3/C-10-1

LINES OF FLOW WITH NOSE EXTENDED TO POSITION Q
AND THE SCOUR-HOLE AT PRESENT NOSE ACCRETED.

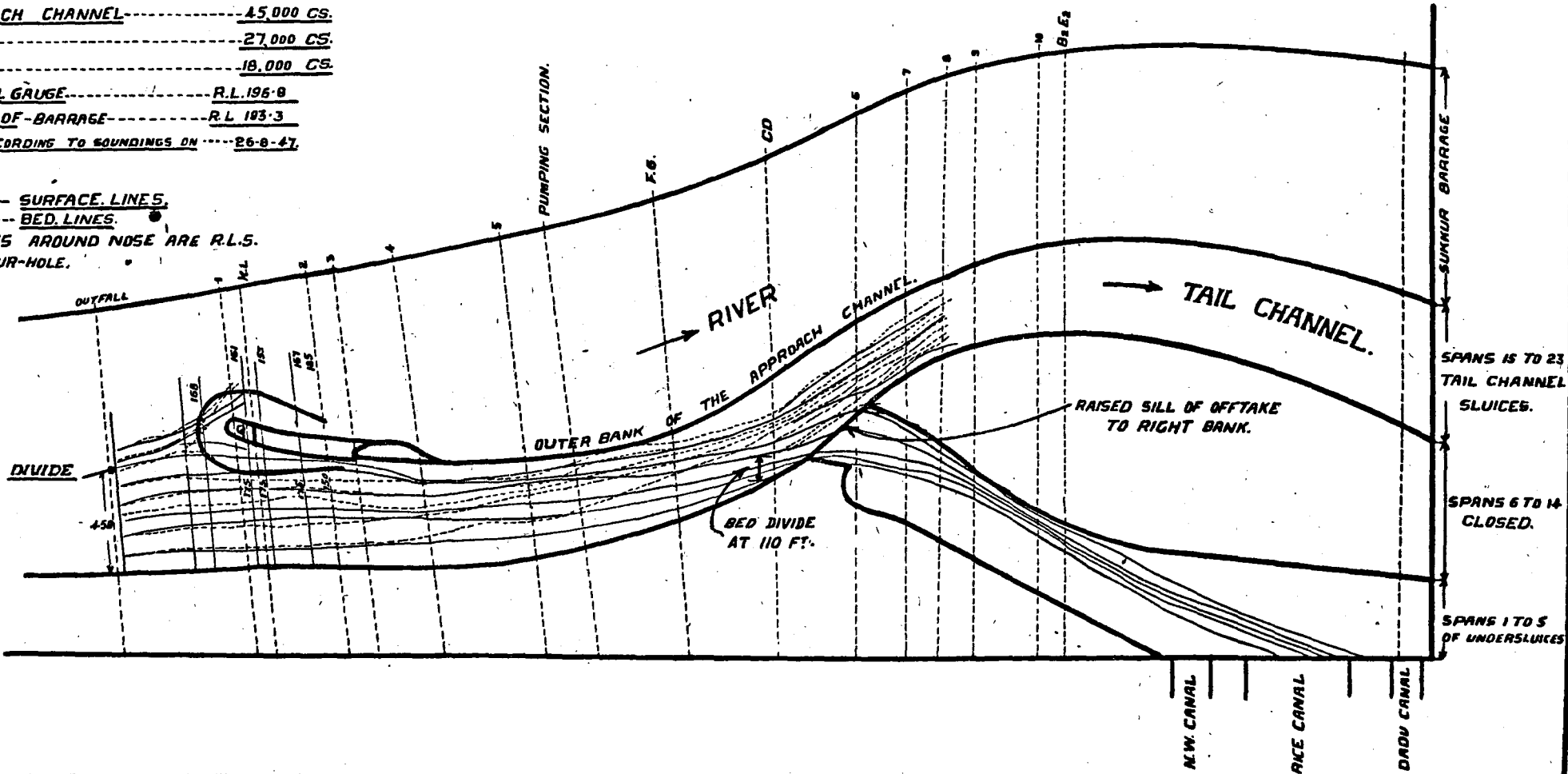
EXPT. 2

Q RIVER	-----	2,75,000 CS.
Q APPROACH CHANNEL	-----	45,000 CS.
Q TAIL	-----	27,000 CS.
Q POCKET	-----	18,000 CS.
OUTFALL GAUGE	-----	R.L. 196.0
W.L. DIS OF BARRAGE	-----	R.L. 193.3
BED LAID ACCORDING TO SOUNDINGS ON	-----	26-8-47.

REF:

SURFACE LINES.
BED LINES.

FIGURES AROUND NOSE ARE R.L.S.
OF SCOUR-HOLE.



BY S. SABNIS.

FIG. 4/C-10-1

NO. 6726

CENTRAL W. IRRIGATION & NAVIGATION
RESEARCH STATION POONA.

EXPERIMENT 3

1/80:1/40 PART-WIDTH SUKKUR BARRAGE MODEL.

LINES OF FLOW WITH NOSE EXTENDED AND SCOUR-HOLE
ON THE APPROACH CHANNEL SIDE LAID TO R.L. 160.0

Q RIVER 2,75,000 Cs.

Q APPROACH CHANNEL 45,000 Cs.

Q TAIL CHANNEL 27,000 Cs.

Q POCKET 18,000 Cs.

OUTFALL GAUGE R.L. 196.8

W.L. D/S OF BARRAGE R.L. 193.3

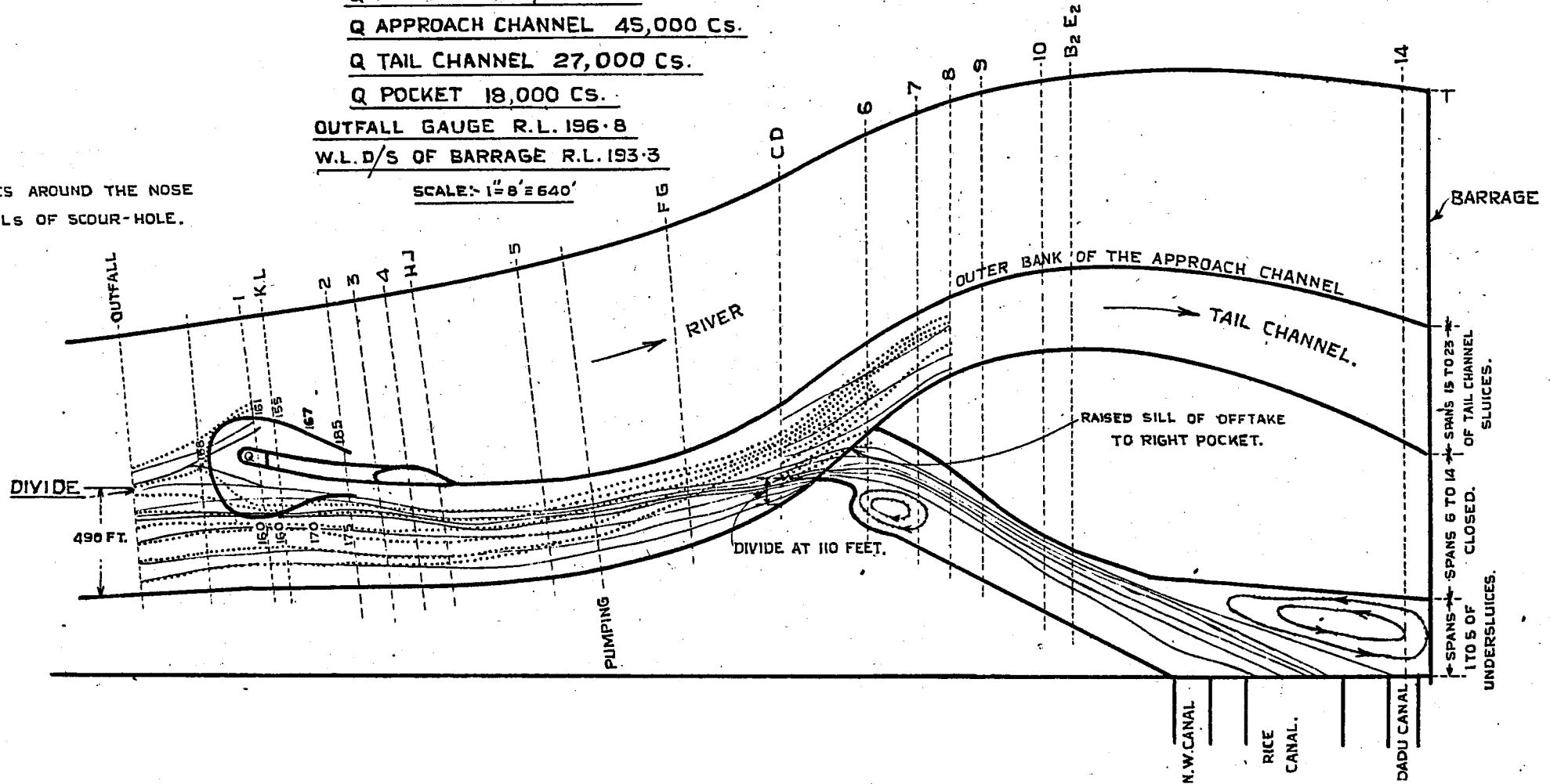
SCALE: 1" = 8' = 640'

Reference:-

Surface lines. _____
Bed lines. _____

Note:-

FIGURES AROUND THE NOSE
ARE R.L.s OF SCOUR-HOLE.



C-10-2 Effect of change in regulation operations of the Right Pocket Of the Sukkur Barrage on sand exclusion—Experiments in 1/80:1/40 part-width Sukkur Barrage Model.

1. Introduction :

Since the construction of the Approach Channel, it has been usual to carry out the annual *scouring operations* of the Right Pocket towards the end of Abkalani, in the middle of September. The Pocket thus flushed remains clear till the next floods. With river discharge above 3 lakhs cusecs, there is excessive silt in suspension and the pocket gets silted fairly quickly, especially in the reach up to 900 ft. upstream of the barrage. A stable channel section sufficient to carry the designed discharge, however, remains and there is no progressive reduction in area of the pocket as is generally supposed—vide item C-2(iii)-1. Nor can the quantity of the deposited silt, when scoured later, make a significant addition to the silt charge entering the canals.

Figs. I &
1(a) of item
C-2 (iii)-1

The September scouring operation is carried out *with canals closed* and a maximum discharge of 27,000 cusecs is run in the Pocket and 18,000 cusecs in the Tail Channel, thus making a total of 45,000 cs. in the Approach Channel. In previous years, the canals were kept closed during abkalani scouring in June, July or August; but this naturally seriously interfered with irrigation operations. So, the Executive Engineer, Barrage Division⁽¹⁾ required information about the following points : -

“(i) Can any short scouring operation in the right bank pocket be done in the months of June to September while the canals are made to flow without admitting any harmful silt in the canals by increasing the discharge in the Approach Channel from 50,000 cs. to 60,000 cs. for short period of two days in a month without creating any slips near the Bunder wall or the outer guide bank when cut-off on upstream and downstream (naturally or by ponding) i. e. difference of water levels upstream and downstream of undersluices—permits scouring operation? If so, can tail channel be run then with 24,000 cs., and pocket 36,000 cs. for at least two days utilizing 16,000 cs. for running canals and 20,000 cs. for scouring operation (scouring 2 sluices out of five at a time) without admitting any harmful silt in the Right Bank Canals? ”

“(ii) If it is risky to run approach channel with 60,000 cs., it could run with 50,000 cs. and the pocket may draw 29,000 cs. and the rest in tail channel. In this case, canals would be run with the designed discharge of 18,000 cs. and one sluice be scoured at a time with a discharge of about 11,000 cs. without admitting any harmful silt in the Right Bank Canals. ”

2.0. The following experiments were carried out to compare sand exclusion at the raised sill of the offtake into the Right Pocket under existing regulation operations and under those suggested by the Executive Engineer.

2.1. Experiment 1

The 1/80 : 1/40 part - width Sukkur Barrage model was laid in Koregaon sand (mean diameter = 0.34 mm) to the river configuration observed at 4 lakhs cusecs discharge except for the Approach Channel (including tail channel and pocket) which was laid according to soundings on 12-9-1946, i. e. before the scouring operations were carried out.

(1) Letter from Executive Engineer, Barrage Division D. O. No. D-2, Sukkur, dated 14-6-1947.

The following discharges and gauges were maintained :-

River discharge	=	4,00,000	cusecs
Approach Channel discharge	=	45,000	„
Tail Channel	=	27,000	„
Pocket discharge	=	18,000	„
$Q_{T.C.} / Q_{Pocket}$	=	1.5	
N. W. Canal discharge	=	4,500	cs.
Rice Canal discharge	=	10,000	„
Dadu „ „	=	3,500	„
Water level of Outfall	=	R. L. 196.8	
Water level downstream of River	=	R. L. 196.0	

The model was run under these conditions for 2 hours so that ripples formed on the bed in the pocket.

The following observations were made:—

(i) Lines of flow:—Lines of flow, bed and surface, downstream of Pumping Engine section in the Approach Channel, were observed by wool thread.
Fig. 5
C-10-2

(ii) Injection of seeds :—Seeds were dropped, at intervals, at Section FG in the Approach Channel. The % of seeds, which were observed to enter the Pocket and Tail Channel, are given in table 2. This percentage was arrived at from a large number (40 to 50) of seeds at each point.
Table 2
C-10-2

10 seeds were also dropped at a time at section 8 in the pocket, well distributed over the section at each time, and the numbers entering the North Western, Rice Canal and Dadu canals were observed. The results given in Table 3 are average of about 100 seeds dropped at section 8. Bracketed figures in table 3 are obtained by multiplying the percentage by 100 and dividing it by the respective discharge so as to give a rough comparative indication of sand charge, with Table 4.
Table 3
C-10-2

(iii) Injection of sand :—Koregaon sand was injected for 18 hours at section 8 in the pocket at the rate of 0.75 cft/hr (0.33 parts per thousand), i. e. a high charge, to give quick results. Sand was collected downstream of the canals. Sand charges obtained from this experiment are also given in Table 4.
Table 4
C-10-2

2.2. Experiment 2.

With other conditions similar to those in Experiment 1 the discharge in the Approach Channel was increased to 60,000 cusecs.

This was distributed as follows:—

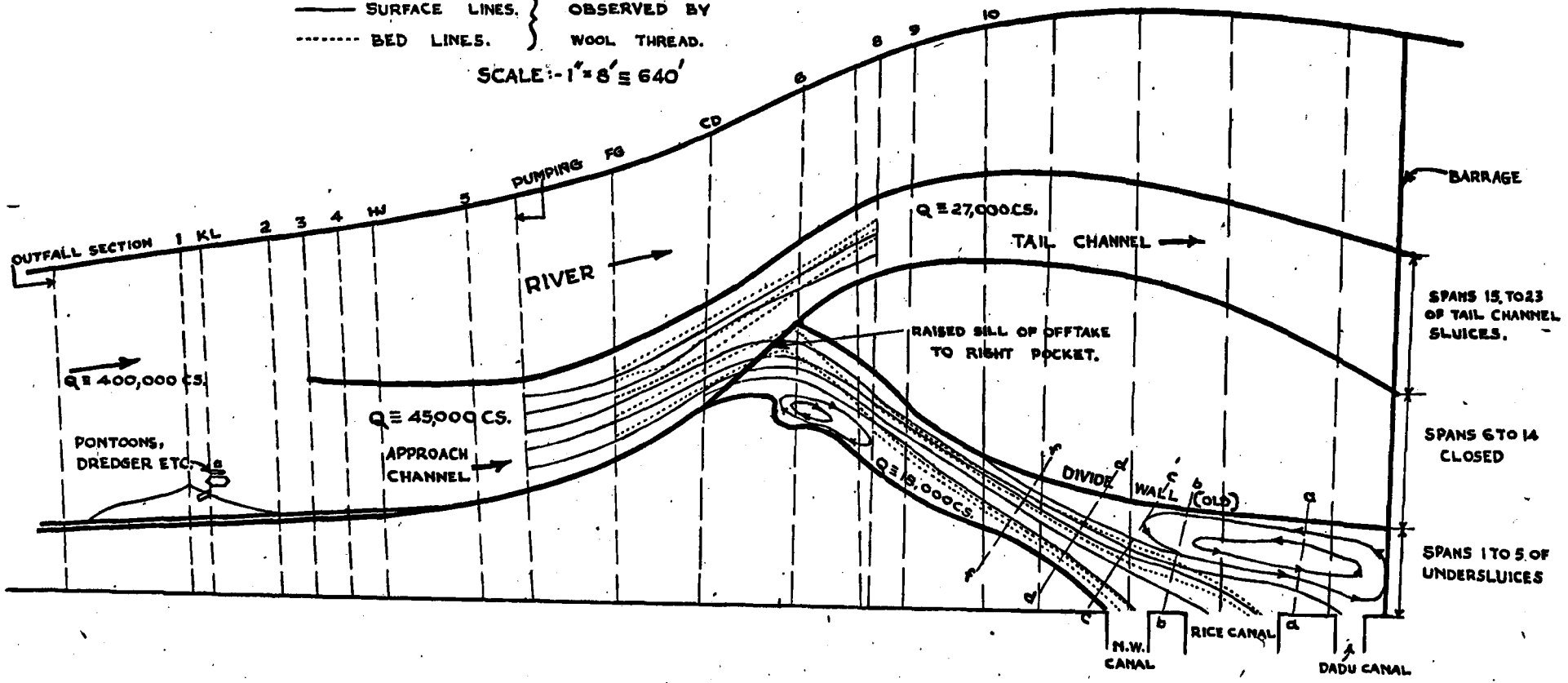
Approach channel discharge	=	60,000	cs.
Tail Channel discharge	=	36,000	„
Pocket discharge	=	24,000	„
$Q_{T.C.} / Q_{Pocket}$	=	1.5	
Canal discharge	=	18,000	cusecs.
Undersluices discharge	=	6,000	„

Water level in the river downstream of the Tail Channel and Pocket was maintained at R. L. 192.7 to simulate the usual 4.0 ft. heading up under scouring operations.

FIG- 5/C-10-2

1/80 : 1/40 SCALE, PART-WIDTH SUKKUR BARRAGE MODEL
LINES OF FLOW IN THE APPROACH CHANNEL
UNDER PRESENT CONDITIONS
RIVER DISCHARGE \approx 4,00,000 CS.
APPROACH CHANNEL DISCHARGE \approx 45,000 CS.
TAIL CHANNEL DISCHARGE \approx 27,000 CS.

— SURFACE LINES. } OBSERVED BY
 BED LINES. } WOOL THREAD.
 SCALE: -1" = 8' \approx 640'



In the model, three gates were provided in the pocket representing five undersluices of the Sukkur Barrage, 6000 cusecs were passed through gate No. 3 in the model which was roughly equivalent to sluices Nos. 4 and 5 in the barrage (sluice No. 1 abuts the right abutment of the Pocket).

Lines of flow were observed by wool thread. Distribution of seeds was tested as in experiment No. 1.

Fig. 6
C-10-2

2. 3. Experiment 3.

With water level R. L. 196.8 at Outfall Section and R. L. 192.7 of Tail Channel, the maximum discharge which could be passed over the raised sill into the Pocket was 34,600 cusecs instead of 36,000 cusecs required :-

Approach Channel discharge	=	56,500 cusecs.
Tail Channel	=	21,900 "
Pocket discharge	=	34,600 "
$Q_{T.C.}/Q_{Pocket}$	=	0.63
Canal discharge	=	18,000 cusecs.
Undersluices discharge	=	16,600 "

Gates Nos. 2 and 3 in the model (equivalent to sluices Nos. 3,4 and 5 in the barrage) were opened equally to pass 16,600 cusecs.

Lines of flow and distribution of seeds were observed as in the previous experiments.

Fig. 7
C-10-2

Koregaon sand was injected in the pocket at section 8 at the rate of 1.5 cft/hr (0.34 parts per thousand) for 7 hours. Sand charge is given in table 4.

Tables 2 & 3
C-10-2

3. 0. Results :

Table 4
C-10-2

(i) *Effect on sand exclusion at the raised sill.*

(a) *Lines of flow* :- In experiment 1 bed lines of flow up to a divide of 2 ft. \equiv 160' from right bank at section CD entered the pocket. This increased to 2.75 ft. \equiv 220 ft. and 3.5 ft. \equiv 280 ft., respectively, in experiments 2 and 3.

Figs. 5, 6 & 7
C-10-2

(b) *Bed seeds* :- Table 2 shows the increase in % of seeds (dropped at Section FG) entering the pocket in experiments 2 and 3 as compared to the existing conditions in experiment 1.

Table 2
C-10-2

(ii) *Distribution of sand entering the pocket.*

The results obtained from observations of lines of flow, injection of seeds and sand varied. This was due to the limitation and drawbacks of each method of observation. The observation of bed lines of flow by wool thread indicated that insignificant bed material would enter either the Rice or Dadu Canal; this was due to the high forward velocity at these sections, created by considerable discharge passed through the undersluices. Seeds dropped to observe distribution travelled along the bed only, while sand injected was thrown in suspension, in experiments 2 and 3, near the Dadu Canal due to turbulence.

Fig. 7
C-10-2

In spite of these quantitative differences, the qualitative inferences are clear. The results given in Tables 3 and 4 *must not* be taken, therefore, to give the quantities of sand charges under the three different conditions of regulation but merely as a comparative indication of how the material entering the pocket would be shared by the canals under each type of operation.

Under existing operations (experiment 1), the N. W. Canal and the Rice Canal draw most of the sand entering the pocket. With increase in pocket discharge to 34,600 cusecs sand charge in Dadu Canal increased considerably. (Compare results of experiment 1 to those of experiment 3).

4. *Analysis of the results :*

Increase in the pocket discharge to 24,000 cusecs maintaining ratio $Q_T. C. / Q_{\text{pocket}} = 1.5$ *deteriorated* the exclusion at the raised sill. Reversal of this ratio (experiment 3) apparently reverses the favourable curvature at the raised sill. This is only to be expected. So, either an increase in pocket discharge maintaining the discharge ratio of 1.5 or altering this ratio even for a short period is likely to draw more sand into the pocket.

Thus, with the scouring operations, as suggested by the Regulation Officer, there would be a higher sand charge drawn by the pocket and, consequently, the N. W. Canal and the Rice Canal would draw more sand than at present. The Dadu Canal is likely to be the worst affected compared to existing conditions.

By admitting more water in the Pocket, as compared with the discharge in the Tail Channel, in order to scour the pocket (while keeping the canals running), the favourable curvature at the raised sill is reversed and large quantities of sand will be drawn by the Pocket, which would subsequently find their way into all these canals.

FIG. 6/C-10-2

EXPT. 2

1/80 : 1/40 SCALE PART-WIDTH
 SUKKUR BARRAGE MODEL

LINES OF FLOW IN THE POCKET
 WITH THIRD UNDER-SLUICE OPEN
 RIVER DISCHARGE 4,00,000 CUSECS

APPROACH CHANNEL DISCHARGE 60,000 Cs.
 POCKET DISCHARGE 24,000 Cs.

[CANALS 18,000 Cs.
 UNDER-SLUICES 6,000 Cs.

TAIL CHANNEL DISCHARGE 36,000 Cs.

SCALE 1" = 8 FT. = 640 FT.

REF.

LINES OF FLOW BY WOOL THREAD

———— SURFACE LINES OF FLOW

----- BED LINES OF FLOW

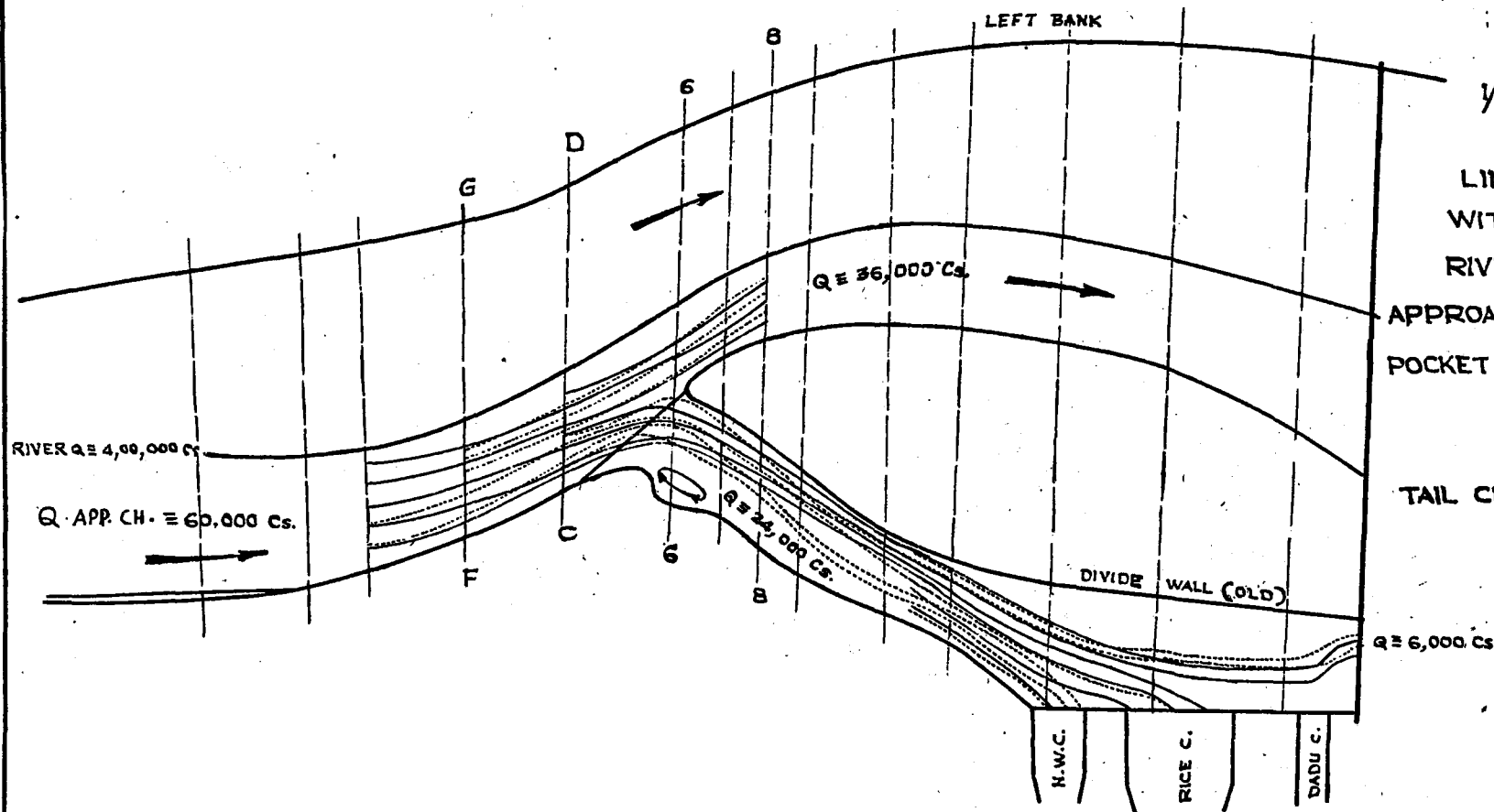


FIG. 7/C-10-2

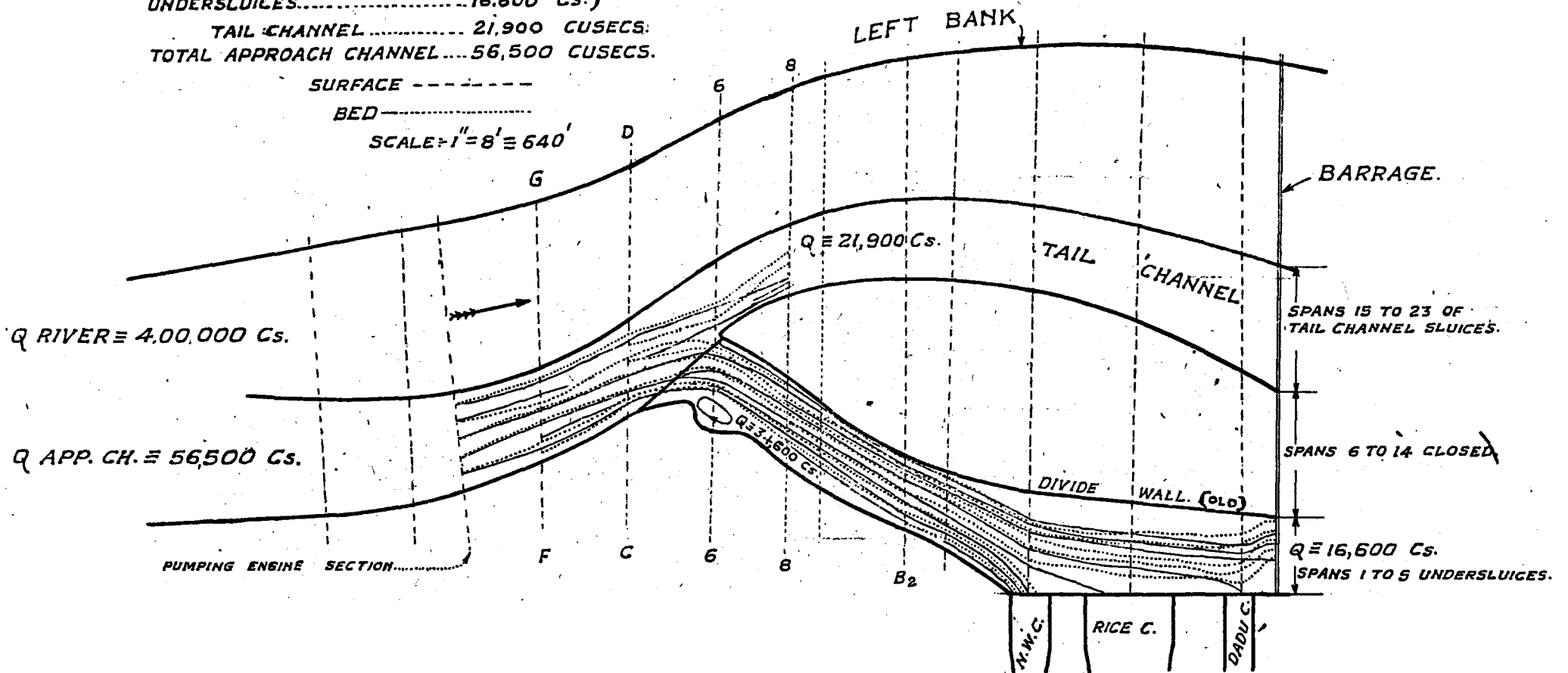
1/80:1/40 SCALE PART-WIDTH SUKKUR BARRAGE MODEL

LINES OF FLOW IN THE APPROACH CHANNEL

WITH RIVER DISCHARGE 400,000 CUSECS.

CANALS.....18,000 Cs. }
UNDERSLUICES.....16,600 Cs. } 34600 Cs.

TAIL CHANNEL.....21,900 CUSECS.
TOTAL APPROACH CHANNEL.....56,500 CUSECS.



By. SAPKAL.

Table No. 2 / C-10-2

Giving the percentage of seeds entering the pocket and tail channel when dropped at section FG.

Distance from Right Bank at Section FG. (ft.)	1.0		1.5		2.0		2.5		3.0		3.5		4.0		4.5		5.0	
	% in tail channel	% in pocket	% in tail channel	% in pocket	% in tail channel	% in Pocket	% in tail channel	% in pocket	% in tail channel	% in pocket	% in tail channel	% in pocket	% in tail channel	% in pocket	% in tail channel	% in pocket	% in tail channel	% in pocket
1	2	3	4	5	6	7	8	9	10	11	13	13	14	15	16	17	18	19

Experiment 1— $Q_{T.C.}/Q_{Pocket} = 1.5$.

Expt. 1: App. Ch. discharge = 45,000 cs. Canal dis. = 18,000 " Tail Ch. " = 27,000 " Undersluices	10	90	30	70	90	10	100	0											
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Experiment 2— $Q_{T.C.}/Q_{Pocket} = 1.5$.

Expt. 2: App. Ch. dis. = 60,000 cs. Canals dis. = 18,000 " Undersluices = 6,000 " Tail Ch. dis. = 36,000 "	10	90	10	90	50	50	80	20	100	0									
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Experiment 3— $Q_{T.C.}/Q_{Pocket} = 0.63$.

Experiment 3: App. Ch. dis. = 56,500 cs. Canals " = 18,000 " Undersluices = 16,600 " Tail Ch. dis. = 21,900 "	0	100	0	100	0	100	0	100	0	100			10	90				20	80
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Percentage of seeds dropped at Section 8 entering the canals and undersluices.

	% of seeds entering N. W. Canal.	% of seeds entering Rice Canal.	% of seeds entering Dadu Canal	% of seeds passing through the Undersluices
1	2	3	4	5
Expt. 1	40 (.9)	60 (.6)	0 (0)	Closed (0)
Expt. 2	30 (.67)	50 (.5)	10 (.286)	10 (.167)
Expt. 3	15 (.33)	25 (.25)	10 (.286)	50 (.3)

Note:—Figures in bracket give values of $\frac{\text{percentage} \times 100}{\text{Discharge}}$. This affords a comparison with Table 4 giving parts per thousand.

Table 4/C-10-2.

Giving the sand charge in canals in experiments 1 and 3.

	Parts per 1000 in N. W. Canal.	Parts per 1000 in Rice Canal.	Parts per 1000 in Dadu Canal	Parts per 1000 through the Undersluices.
1	2	3	4	5
Experiment No. 1 Rate of injection 0.75 cft/hr.	0.026	0.0105	0.008	Under sluices closed.
Experiment 3 Rate of injection 1.5 cft/hr.	0.0195	0.0232	0.195	0.101

C—HYDRAULIC WORKS

12—Other works.

C-12-1 Supply of sand-free water to Jamshedpur Pump House.

1. Introduction :—

Both the Jamshedpur Steel Works and the town were previously served by water pumped from the Subarnarekha river. Recently, the Dimna nala has been dammed some six miles north east of Jamshedpur and water supply for the town is tapped from this Dimna reservoir; but water for the Steel Works continues to be pumped from the river just upstream of the pick-up weir, shown in figure 2. The existing pumping installations have a total capacity of 31,850 gallons per minute. The existing suction main is a 4 ft. diameter, cast iron pipe with its open end located midstream, some distance above the weir, at a point where bed water heavily charged with bed sand is evidently sucked in without hindrance. Figs. 1 & 2
C-12-1

The pumps can be worked continuously all the year round except for the danger of the heavy charge of sand being drawn during the flood season, which causes heavy wear and tear of pump casings and impellers, thus reducing their life and efficiency. Hence, during floods, the pumps are being worked at present to meet only the minimum requirements. In spite of this precaution, in the monsoon about 5000 cubic feet of sand collects daily in the main common suction chamber inside the Pump House and has to be promptly removed, and, as a result, the pump casings have been severely pitted and have slowly disintegrated by attrition.

The case was referred to the Station for its recommendations and suggestions as to how to ensure supply of relatively sand-free water to the Pump House at all river stages.

2. Station recommendations :—

2.1 Location of the Pump House —

The first requirement, was to design a practically sand-excluding Intake.

Various methods have been tried and tested, in the past, for sand exclusion, mainly in connection with the design of canal headworks. None has been found as satisfactory, in general, as locating the intake on the *concave* (i. e. outer) bank of a curve, where the bed water containing sand is naturally deflected away and only sand-free, top water enters the intake works.

The *first* essential was, therefore, to locate the Intake near the downstream end of the available concave bend, though well upstream of the cross-over of the deep-water river channel, so as to obtain the maximum benefit of the favourable river curvature. A suitable point near the existing Pump House was suggested by the Station for the Intake, on these considerations.

Two counteracting features, reducing the efficiency of sand exclusion, must, however, be borne in mind :—

(1) The effect of curvature is diminished in periods of low river by ponding of water caused by the weir; and,

(2) in high floods, in spite of repositioning of the Intake, the bed sand thrown into suspension by turbulence and intermixing of the water at different depths may still be sucked in, to a certain extent, unless arrangements are made to tap only the top filaments and to insure minimum agitation.

In addition to correctly relocating the Intake, some additional arrangement to prevent the entry of sand in the suction pipe was, therefore, considered necessary.

2.2 Intake Tower :—

An Intake Tower, well streamlined in plan to minimise turbulence and with an arrangement for tapping *surface* water at all river stages, as shown in figures 3 and 4, was, therefore, recommended by the Station. Figs. 3 & 4
C-12-1

(A) Salient features of the proposed Intake Tower :—

In dry weather, when the water in the river is low and there is little or no detritus, water enters through vents from all sides and is collected in the central chambers, from where it is carried to the Pump House by water mains. When the water level rises, the bottom water from the side vents is cut-off; top water then enters the central chambers by spilling over a set of moving gates which can be operated to maintain the top of the gates at any desired level from the operating platform. If the top of the Tower is kept sufficiently high with a reasonable free-board, gates can be operated all the year round, even in the highest floods.

(B) Types of Intake Towers:—

Two designs were suggested:—

Fig. 3
C-12-1

(a) Figure 3 shows a design of the Tower similar to the one supplying water to the Wazirabad Pump House at Delhi, but modified to suit the Jamshedpur conditions.

The water after entering the bell-mouthed vents flows down the Intake pipes and thence to the inner chambers. When the water level rises, the pipes are closed at the bell-mouthed ends by covers, the outer most gates on both sides lowered, and connection between the outer and inner chambers completely cut-off; the water then passes only over the top of one of the gates, depending on the river level at the time.

Fig. 4
C-12-1

(b) The alternative design of the Tower in fig.4 was evolved at the Station and has some special features different from those of the Wazirabad type. A short-crested weir in the Station type replaces the intake pipes. This does not require any covering slab as in the other design and hence is perhaps more economical; also, in lieu of peripheral pipes rectangular vents with bell-mouth ends are provided around the periphery.

This design may require slightly more head, say 0.5 feet as against 0.2 ft. in the Wazirabad type.

Both designs are expected to work efficiently and either can be adopted, depending on the suitability to local conditions, construction facilities etc.

(C) Design details:—**(1) Tower design—****(a) Wazirabad type:—****(i) Shape, orientation, size and height:—**

It is essential that the Intake Towers should cause as little obstruction to flow as possible; otherwise, besides the danger of undermining, the bed sand would be thrown into suspension due to turbulence and the design would thus reduce in efficiency. The Tower was, therefore, provided with streamlined ends, and so oriented, in respect to the direction of river flow, that the flow lines would be least distorted.

The size of the Tower is governed by the intake capacity required and the maximum H. F. L., and so, by the size of gates, intake pipes and vents, the designs of which are given below—

It is desirable that it should be possible to operate the gates up to a level as near the highest probable high flood level in the river as possible; unless the difference between the frequent flood level and the highest H. F. L. is so large that raising the Tower above the highest H. F. L. to provide reasonable free-board becomes uneconomical. In the present case, the river being of the *flashy type* with the highest high floods occurring for an inconsiderable duration, the Tower can be allowed to be submerged; as even then, the functioning of the Tower will not in any way be adversely affected because the sluice valves at the Pump House end can, nevertheless, still be operated as desired.

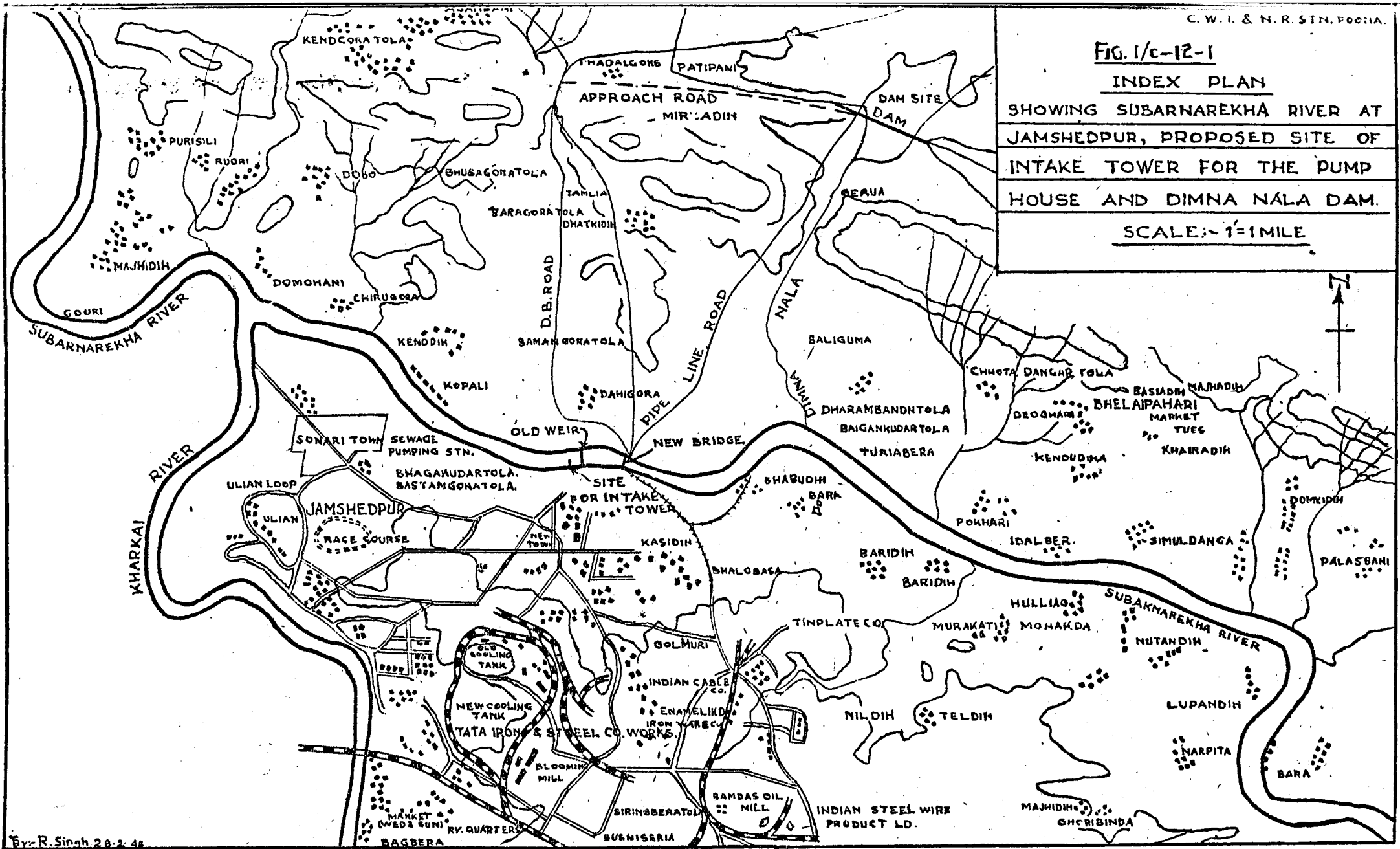
From the analysis of available data of flood water levels of the Subarnrekha river at the Pump House, it is seen that, with the Tower top at R. L. 410.00, the best compromise was obtained between constructional cost and the operational or control period. Even under worst conditions as of the 1942

FIG. 1/c-12-1

INDEX PLAN

SHOWING SUBARNAREKHA RIVER AT JAMSHEDPUR, PROPOSED SITE OF INTAKE TOWER FOR THE PUMP HOUSE AND DIMNA NALA DAM.

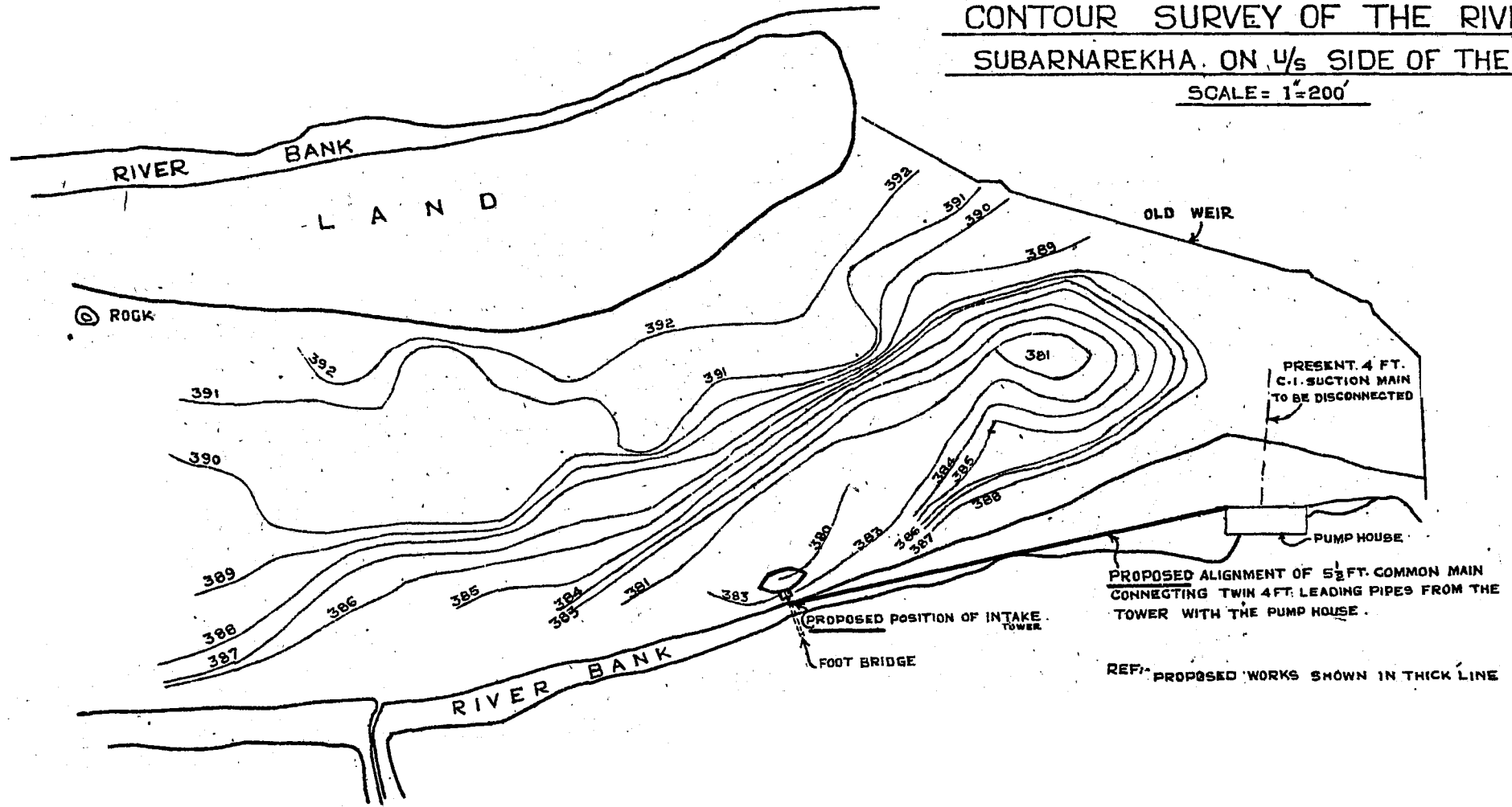
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REF:- REDUCED & TRACED FROM PLAN No CES No 103
(THE TATA IRON & STEEL Co. LTD. JAMSHEDPUR)

NO. 6/58
C.W.I. & N.R. STN. POONA

FIG.2/C-12-1.
CONTOUR SURVEY OF THE RIVER
SUBARNAREKHA. ON $\frac{1}{4}$ S SIDE OF THE DAM
SCALE = 1"=200'



R.K.K.

NR. 6791

CENTRAL WATERWAYS IRRIGATION AND
FOUNDATION RESEARCH STATION,
MADRAS.

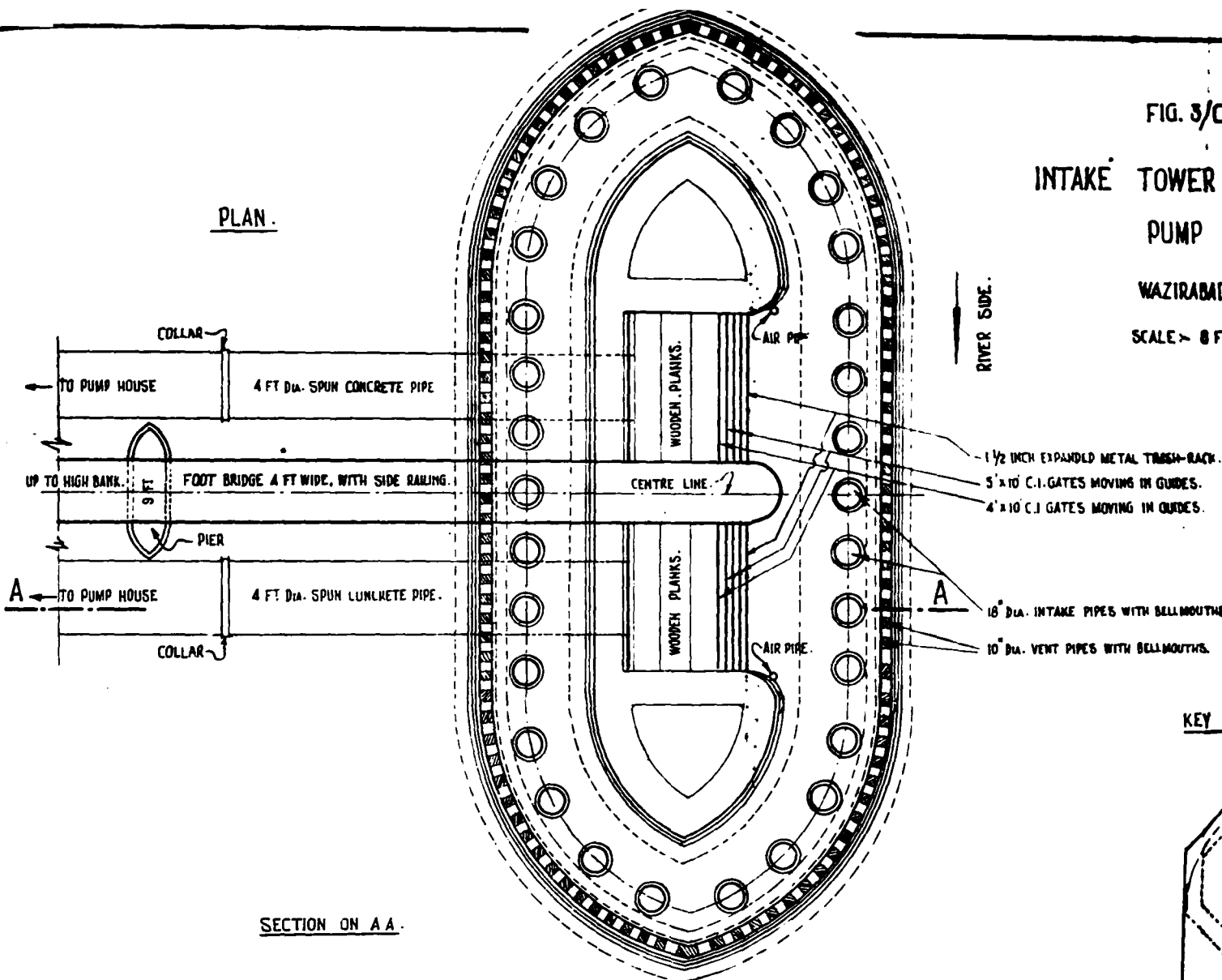
FIG. 3/C-12-1.

INTAKE TOWER FOR JAMSHEDPUR
PUMP HOUSE.

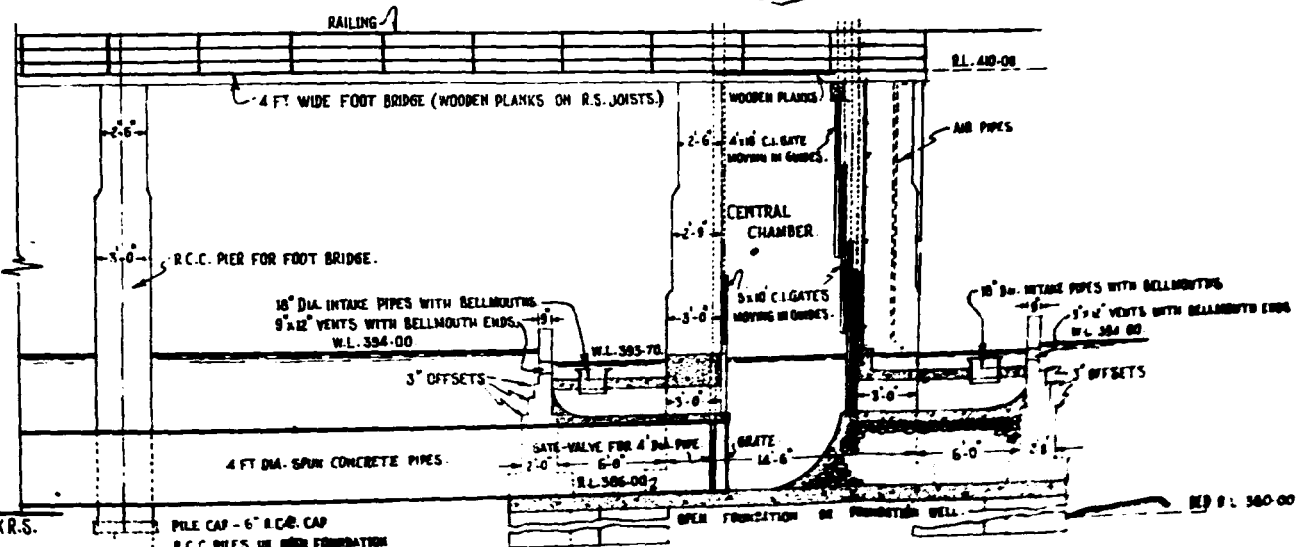
WAZIRABAD TYPE.

SCALE - 8 FT TO AN INCH.

PLAN.



SECTION ON A A.



KEY PLAN OF FOUNDATION

SCALE - 1" = 20'

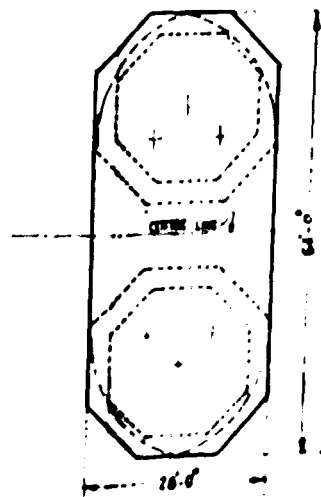


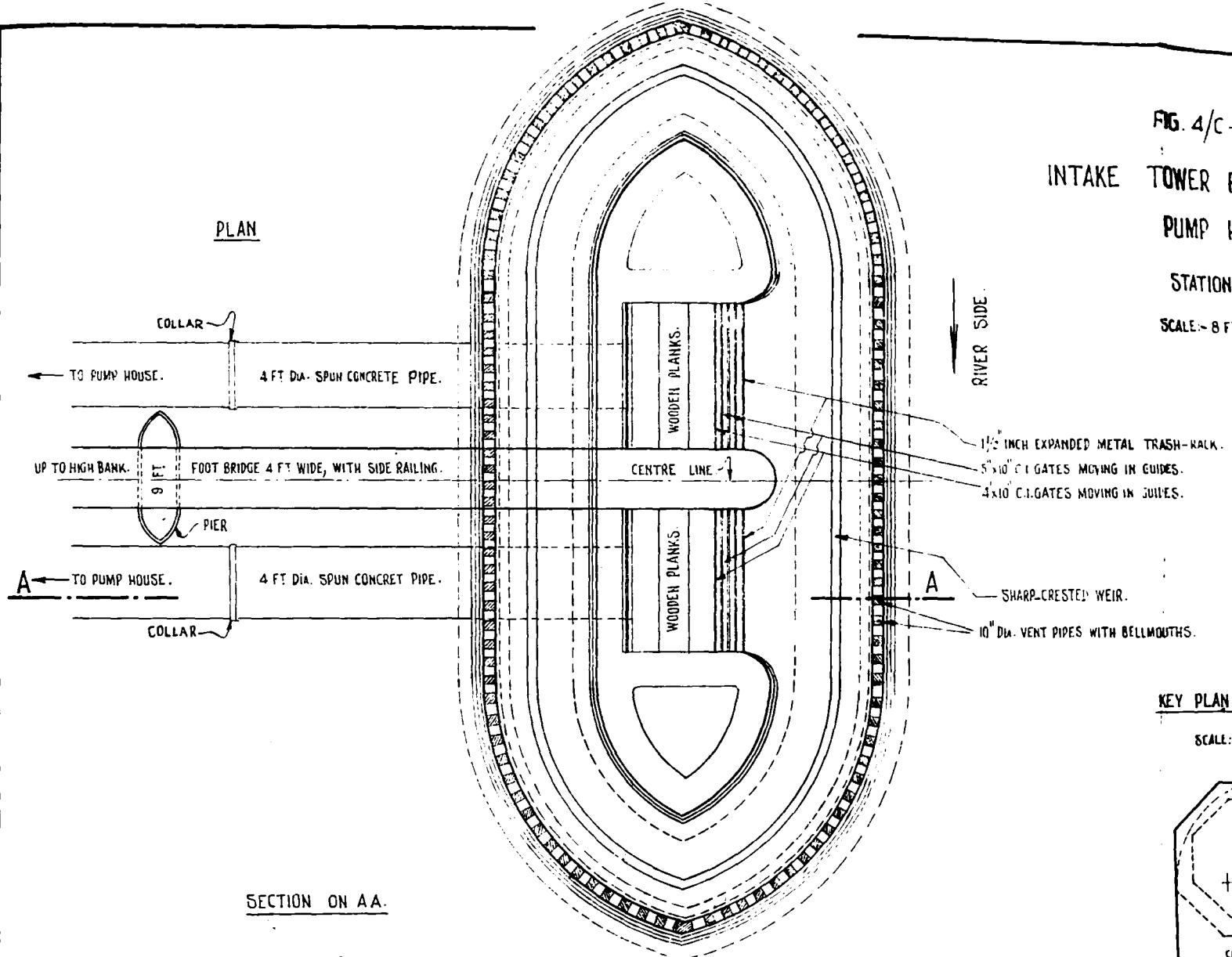
FIG. 4/C-12-1

INTAKE TOWER FOR JAMSHEDPUR
PUMP HOUSE

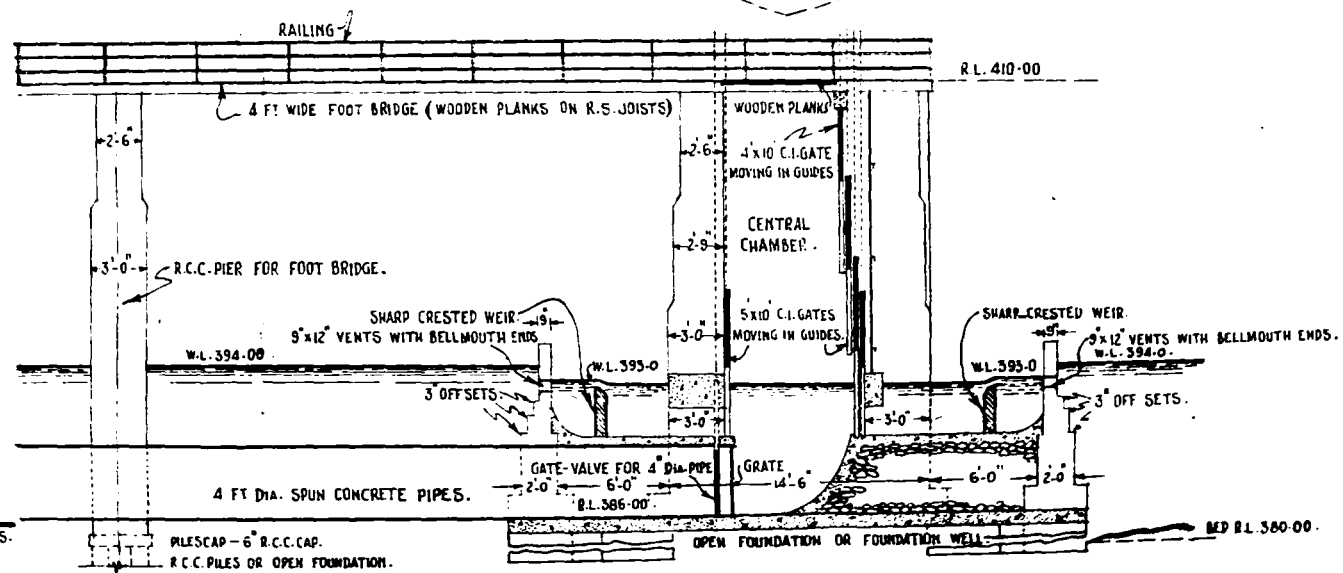
STATION DESIGN

SCALE: - 8 FT TO AN INCH.

PLAN

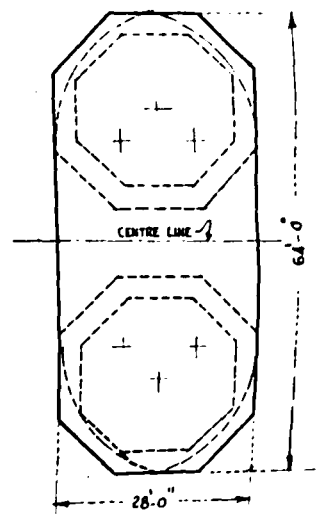


SECTION ON AA.



KEY PLAN OF FOUNDATION

SCALE: - 1" = 20'



floods—and such abnormal floods are generally rare—regulation of the Tower with its top at R. L. 410.00 will be permissible throughout the year, except for 3 consecutive days, or 10 days in the aggregate with intervening periods of low water levels.

Any higher level does not give corresponding advantage in the control period commensurate to the cost. For example, raising the Tower to R.L. 415.00 would only reduce the aggregate period by about 3 days, while the consecutive period would remain the same. Additional control for 3 more days in a season does not appear to justify the extra cost involved, and hence was not recommended; while raising the Tower above H.H.F.L (viz. R. L. 440), obtainable perhaps only for a single day at several years' interval, was out of consideration.

The top of the Tower, therefore, was suggested to be kept at R. L. 410.00. In the short and infrequent periods when the river level is above R. L. 410.0 the supply can be taken from the Dimna reservoir, or from the Work's Supply Balancing Tanks, if drawing it above the topmost gate at these river stages is found to draw in sand with it.

The old weir with crest at R. L. 394.34, being meant only as a diversion weir not intended to impound water, the Tower was designed to supply the necessary discharge at water level R. L. 394.00.

The width of the inside chamber as provided was designed to ensure that, even with river water level at R. L. 418.50 and the inside water level at its lowest (viz. at R. L. 394.00), the nappe falling over the moving gate would not hit against the opposite wall.

(ii) Foundations :—

The nature of the bed at the proposed position of the Intake pier was not exactly known. The scattered out-crops of rock on the right bank suggested that sound dependable rock may be available for foundations near the bed of the proposed site.

Well or pile foundations were suggested in case rock is not struck at a reasonable depth.

(iii) Peripheral vents :—

All along the periphery, 102 vent pipes, 10" diameter, with bell-mouth ends were provided. The number of vents and their sizes were so designed that, even with the lowest obtainable W. L., more than the required discharge will be passed.

(iv) Intake pipes with bell-mouths :—

30 pipes, each 18" diameter, were found to be necessary to pass the required discharge with a liberal margin of safety.

V. Other details :—

Gates :

Two sets of 10 ft. wide C. I. vertical gates, moving in guides were suggested. Excepting the top gate which may be 4 ft. high, it was suggested that the other gates may be 5 ft. high, with an overlap of 6" between two adjacent gates. The outermost bottom gates were provided on both sides for cutting off the connection between the outer and inner chambers in high floods, or whenever required for cleaning, repairs, inspection etc.

Trash rack :—

For preventing floating debris finding access into the Tower, trash rack of expanded metal are proposed to be fitted in front of the gates.

Foot-bridge :—

A four foot wide foot-bridge from the operational platform to the bank was suggested to be provided.

Minor details, such as gate valves for 4 ft. pipe mains, aeration pipes, foot-rests, wooden planks for covering the central chambers etc. are shown in fig.3

Fig 3.
C-12-1

2.3 Pipe lines from Intake Tower to Pump House :—

Fig.2.
C-12-1

Alignment:— The best alignment can only be determined from a contoured plan, or else from cross-sections and L. section, which were not available. The tentative alignment, as proposed, is shown in fig.2; but this may be required to be modified to suit local contours and conditions.

Size:—

It is always desirable to maintain as high a water level in the Tower as possible in keeping with the cut-off necessary to pass the required discharge into the chambers over the movable gates. It is, however, preferable that pipe diameters should be designed for the *lowest* water level inside the Tower. The size of the pipe being thus determined, further reduction of pipe diameter merely to increase the scouring velocity—to keep the pipe flushed—is not permissible as the capacity of the pipe will then be inadequate at low water level in the chamber. Moreover, for flushings, high velocities can be generated by proper control of the Tower gates and gate valves in the intake pipes. These considerations led to the adoption of 4 ft. spun-concrete pipes as leading mains from the twin inlet chambers and 5 ft. diameter pipe as common main connecting the 4 ft. pipes with the Pump House sump.

3. Regulation.

In low river, all the gates excepting the bottom gates are to be closed so that water will *only* enter through the peripheral vents, flow down the intake pipes and collect in the middle chamber, from where it is carried to the Pump House by the mains.

With the oncome of floods, the W.L. will first sufficiently rise to permit taking water over the top of the *lowest* gate. The intake pipes are then to be closed with the covers and the outer gates fully lowered.

Depth of water for passing 133.55 cusecs over the pair of gates is estimated to be about 1 ft.

The gates are *always* to be so operated, as far as possible, that only about one foot layer of surface water is tapped

The inner chambers of the Tower can easily be closed—one at a time or both together—by lowering all the gates and draining the chambers by opening the sluice valves in the water mains. During this time—which will not be frequent or long—water for works can be supplied from the second chamber of the Tower, if one is closed down; or, if both chambers are simultaneously closed, from the Cooling Pond or else from the Dimna reservoir.

Constructional difficulties :—

As water supply can be taken from the Dimna reservoir during the period of construction, no serious difficulty is anticipated in constructing the proposed Tower.

As soon as the water level has gone sufficiently low, the existing diversion weir below Pump House can be cut to drain off the water. The foundations can then be built in the dry without having recourse to coffer-dams or a ring bund.

After the work has sufficiently progressed and is carried above foundations, the breach in the weir can be made good.

5. Station type Tower design :—

Excepting for the vents in the side walls and the short-crested weir, this design is similar to Wazirabad type.

(1) Vents :—

106 vents, each 1' x 9", were suggested.

(2) Short-crested weir :—

Head required to pass 133.55 cusecs over the short-crested weir of 112 ft. length was estimated to be = 6"

C-12-2 EXPERIMENTS TO TEST THE SUITABILITY OF THE POSITION OF THE LOCK SPAN PROPOSED IN THE SIND DESIGN.

It is contemplated in the design of the lower Sind Barrage to site the Lock span just *outside* the left pocket divide wall with a Fender Lock wall 210' upstream of the weir; the length being fixed from the consideration of the biggest craft that is likely to use the Lock. Length of the biggest craft is assumed to be 200' and an extra 10' is provided in the length of the approach or Fender Lock wall.

Fig. 5
C-12-2

(2) Experiments were carried out in the 1/250 : 1/50 vertically-exaggerated model with the proposed position and arrangement of the Lock :-

- (a) to test the suitability of the position proposed in the design; or otherwise to determine the best location of the Lock span from the point of view of affording a safe approach;
- and (b) to determine the optimum length of the approach wall from the point of view of least suction caused by the adjacent river spans, drawing the craft towards them during floods.

(3) Experiments revealed the *unsuitability* of the proposed position from the following points of view :-

(a) Due to curved, and persistently turbulent flow, associated with the diving flow at the nose of the left pocket divide wall, this position left much to be desired in respect of satisfactory and safe approach conditions.

(b) Even if there were perfect conditions of parallel flow, the velocity of 10'/sec. to 12'/sec. (or sometimes even more) in this zone would certainly make the berthing of a vessel along the approach quay to the Lock in this position a difficult task, and one of considerable risk both to the vessel as well as to the quay.

(c) Though the Fender Lock wall produced the desired effect of creating slacker water within the area bounded by it and the left pocket divide wall, a ship model showed that the manoeuvring of the vessel would still be difficult, even after it crossed the first hurdle at the nose of the left pocket divide wall, in view of the secondary turbulent region formed just outside the nose of the Fender wall.

(d) A number of tests were also carried out with this Fender Wall varying in length from 150' to 400'; but neither a short nor a long wall could reduce the hazards of turbulence.

(4) At this stage, the following alternative positions for the Lock were considered :-

4.1. As there is only one canal on the right bank the Lock could be constructed in it with two approach channels, one to the upstream and the other to the downstream side of the barrage, with the following possible advantages :-

(a) economically, the cost of the approaches might be more than offset against the cost of the Fender Wall and much lower costs of Lock gates, Lock bridge, etc.;

(b) hydraulically, it would avoid the hazards of turbulent and diving flow and afford easy and safe approach conditions for vessels; it would also largely be unaffected by the operation of the Barrage gates.

The following disadvantages, however, more than counterweighed the advantages :-

(a) this arrangement would interfere with the operation of the canal based on considerations of irrigation demand; the excessive heading-up,

caused in the head reach above the first cross regulator, at periods when the canal would be running with a low capacity factor, would lead to periodic silting in this reach;

(b) and it is also likely that the right arm channel may build up its bed and may not provide sufficient draft at the entrance and/ or exit of the approach channels linking the Lock to the river.

4. 2. The second proposal was to site the Lock at the extreme right end of the left arm barrage to take advantage of the Central Island as an approach quay for the vessels.

But this position was also ruled out after model tests for the following reasons : -

- (a) it would have made the approach conditions even more difficult than if located just outside the Pocket, due to greater turbulence and higher velocity along the shank of the island;
- (b) there would also be less draft available on the downstream side of the Lock as it lies on the convex, and so the *sanding*, side of the curve in the left arm channel;
- and (c) it would have been hazardous to manoeuvre the crafts along the submerged slopes of the left arm shank of the Central Island.

4.3. The third and *best* alternative site considered was just *inside* the left pocket having the following merits : -

(a) as the discharge in the pocket is limited, the maximum velocity is never likely to exceed 5 ft./sec (and under 'still pond' up to river discharge \equiv 452000 cusecs, only 2.82 ft./sec.) with the regulation phases recommended (see item C-4 (ii).2) and so would never be subjected to such turbulence as would be objectionable to the safe manoeuvring of ships;

(b) while working the ship model, it was observed that there is no draw of current *away* from the divide wall in its vicinity, thereby producing unfavourable conditions for berthing a vessel alongside of it; besides there is likely to be a deep water channel from the left guide bank, upstream of the pocket, to the Divide wall nose;

(c) it would also enable handling a large volume of traffic at a time, due to sufficient space being available inside the pocket;

(d) the twist and scour at the nose of the Fender lock wall could also partly aid exclusion of sand rolling along the narrow strip besides the pocket divide wall.

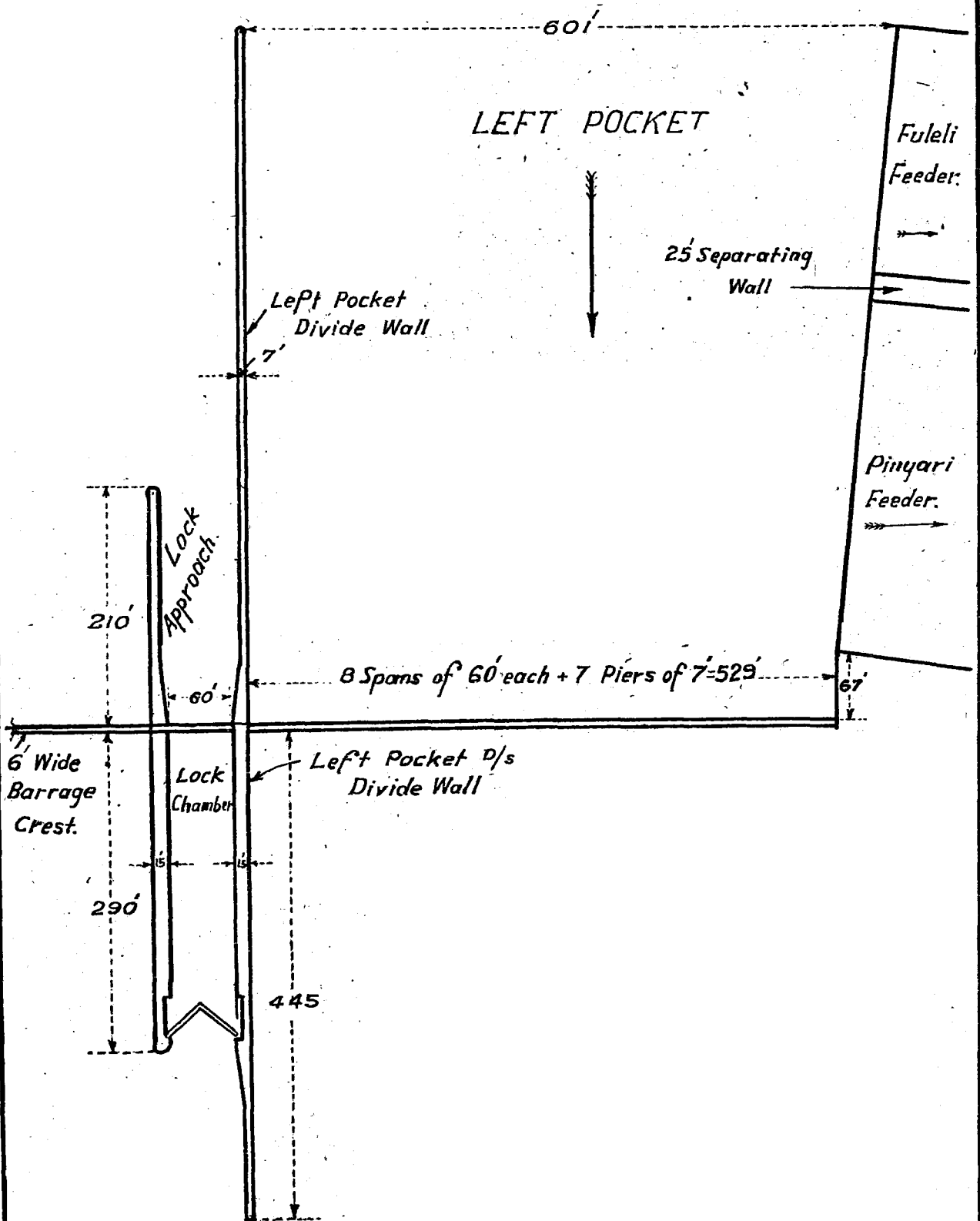
5. All doubts whether the requisite draft will still be available in case the pocket gets badly silted along the divide wall, as in the case of the Left pocket of the Sukkur Barrage, were disproved by rigorous model tests made for this purpose. These tests showed that, under the worst conditions, sufficient depth would still be maintained to give the required draft and scouring operations could quickly and efficiently increase depths, if and when necessary.

5.1. An appreciable reduction in silting of the left pocket of the Sukkur Barrage, since it has been run under "semi-open" flow conditions, for discharges higher than 3,50,000 cusecs, is convincing of the superiority of the "semi-open" flow recommended; if similar regulation is enforced, there will be no excessive silting in the left pocket at the Kotri Barrage.

5.2. Moreover, the three left bank canals at Sukkur, nearest to the barrage line, (i. e. Khairpur Feeder East, Khairpur Feeder West and the Eastern Nara Canal), drawing a maximum aggregate discharge of 16,000 cusecs, are accommodated in a width of 750' along the Head Regulator face line, as a result of which the mean velocity in the zone of the left pocket contribu-

FIG. 5/C-12-2
PLAN SHOWING POSITION OF THE
LOCK SPAN AS CONTEMPLATED IN THE
DESIGN OF THE LOWER SIND BARRAGE.

SCALE: 1" = 125 FT.



ting this discharge to these three canals is hardly more than 1.25 ft./sec., against 3 ft./sec. in the case of the Pinyari Feeder of the Kotri Barrage drawing 20,600 cusecs and accommodated in a restricted width of hardly 314'. Thus, the higher velocity obtainable in the Kotri Barrage design will always maintain a minimum depth of 11' (against 3' in the Sukkur Barrage) below the pond level, based on V/V_0 considerations in these zones.

5.3. The question of the *optimum* position of the Lock from the point of navigation was discussed with Col. T. M. Oag, Director of Navigation, Central Waterways, Irrigation and Navigation Commission. Col. Oag also saw the experiments with different positions of the Lock and the attached note (see Appendix I) is the result of his observations.

App. I
C-12-2

6.1. One series of experiments were aimed at working out the discharge that can be boosted in the pocket and pushed through the lock approach channel to keep it clear of silt, with the optimum permissible opening of the Lock span gate, without vitiating the sand exclusion from the pocket or affecting the regulation phases.

Statement I shows the discharge in the Lock span for the optimum opening of the Lock span gate, V/V_0 in the Lock approach channel, and the theoretical maximum depth up to which it will be possible to scour the silt deposited in the Lock approach channel.

Stt. I
C-12-2

6.2. Though the V/V_0 ratios worked out in Statement I are somewhat disappointing, it will be possible to improve these ratios by pushing in the difference of discharge between the full and actual capacity factors at which the canals will be run during the abkalani; this only means that improvement of the lock approach channel will be suitably timed when the canals are drawing low discharges.

If the balance discharge then available is diverted through the Lock span and the spans adjacent to it, it will maintain satisfactory conditions in the pocket by preventing silting which would otherwise take place at a low capacity factor of the canals. The model showed that even 10% difference in the capacity factors of the canals, forced through the Lock span, will substantially improve the Lock approach channel.

6.3. If despite this the approach channel shows signs of undue silting, with the pocket gates kept 3' higher than the barrage gates (instead of 2' recommended), more discharge can be forced through the lock span and the spans adjacent to it under semi-open flow conditions than through the spans nearer the face of the Head Regulator, by throttling the latter as required without the pocket gates being overtopped.

7. 1. After deciding upon the best position of the Lock, experiments were carried out for determining the optimum length of the approach or Fender wall with the following main considerations in view : -

(a) to avoid danger to the crafts by the suction caused by adjacent spans;

and (b) to provide satisfactory approach conditions by ensuring sufficient draft at the entrance to and in the Approach.

7.2. As the pocket gates adjacent to the Lock span will not be kept open to the same extent as the barrage gates, suction effect was hardly observed to extend 100' upstream of the weir line and thus did not influence the length of the approach wall.

7.3. For consideration (b) above, four different lengths \equiv 337', 275', 230' and 210' upstream of the weir, were tested in the model. After running rising and falling cycles with the optimum permissible openings of the Lock span as worked out in Statement I—and with silting in the dead water region of the left pocket artificially reproduced right up to the pond level (i. e. the *worst* hypo-

thesis)—it was observed that the longer wall was better than the shorter wall for the purpose of clearing the silt from, and in front of, the Lock approach channel (see table 1 below).

Table 1
C-12-2

Figure 2 shows the R. Ls. of silt laid in the pocket and the R. Ls. of the scour as obtained for the 230' Long Lock wall—finally adopted for reasons given subsequently.

Fig. 6
C-12-2

Table 1. C-12-2.

Length of the Fender wall	Minimum R. L. down to which sand was scoured in front of the wall	REMARKS
337'	R. L. 50.0	Lock span gate opened as shown in Statement I.
275'	R. L. 53.0	
230'	R. L. 56.0	
210'	R. L. 60.0	

Had it not been for the diving mid-depth flow at the nose of the pocket divide wall extending 150' below it—under the worst conditions of the regulation—the longest Lock wall would have been, therefore, preferred. With the Lock wall 337' in length, the noses of the two walls will be 293' apart and crafts would have steady water for a length of hardly 143' (obtained by deducting 150' from 293'); which is less than the length of the biggest craft (200') likely to use the Lock. For satisfactory approach conditions there must remain a distance of at least 200'—preferably 250'—essential to avoid vessels being directly drawn to, and impinging against, the nose of the Lock wall. To meet this requirement, the Approach Wall would have to be not more than 230 ft. long.

Fig. 7
C-12-2

8. Length of downstream Lock Wall :-

In the Project design, the downstream Lock wall is proposed to be 290' in length measured from the weir crest, the length being fixed by the maximum length of the craft and the minimum additional distance required for operating the flap gates of the lock.

Model experiments showed that this length was suitable from the point of view of approach and exit to the lock as the nose of the downstream divide wall of the Left Pocket, i.e. the point of the maximum turbulence, is sufficiently away (by about 155') from the nose of the downstream Lock wall.

The maximum draft of 10' required for the craft is also likely to be insured by the regulation of the Lock span as proposed in Statement I.

Hence, the length of the downstream Lock wall as contemplated in the design may be retained.

9. Recommendations :-

(1) The Lock should be constructed in the Left Pocket, i. e. immediately to the left of the Left Pocket divide wall.

(2) The length of the upstream Fender Lock wall should be 230 ft.

(3) The left pocket divide wall should be used as the approach to enter the lock with bollards fitted to it at intervals of 100' from the upstream end of the Lock.

(4) Through the Lock span and spans adjacent to it supplies available owing to the difference between the actual and capacity discharge drawn by the canals should be utilized to minimize sanding of the Lock span.

(5) The left pocket gates may be kept 3' higher than the other barrage gates (instead of 2' previously recommended) to enable flushing of the Lock span.

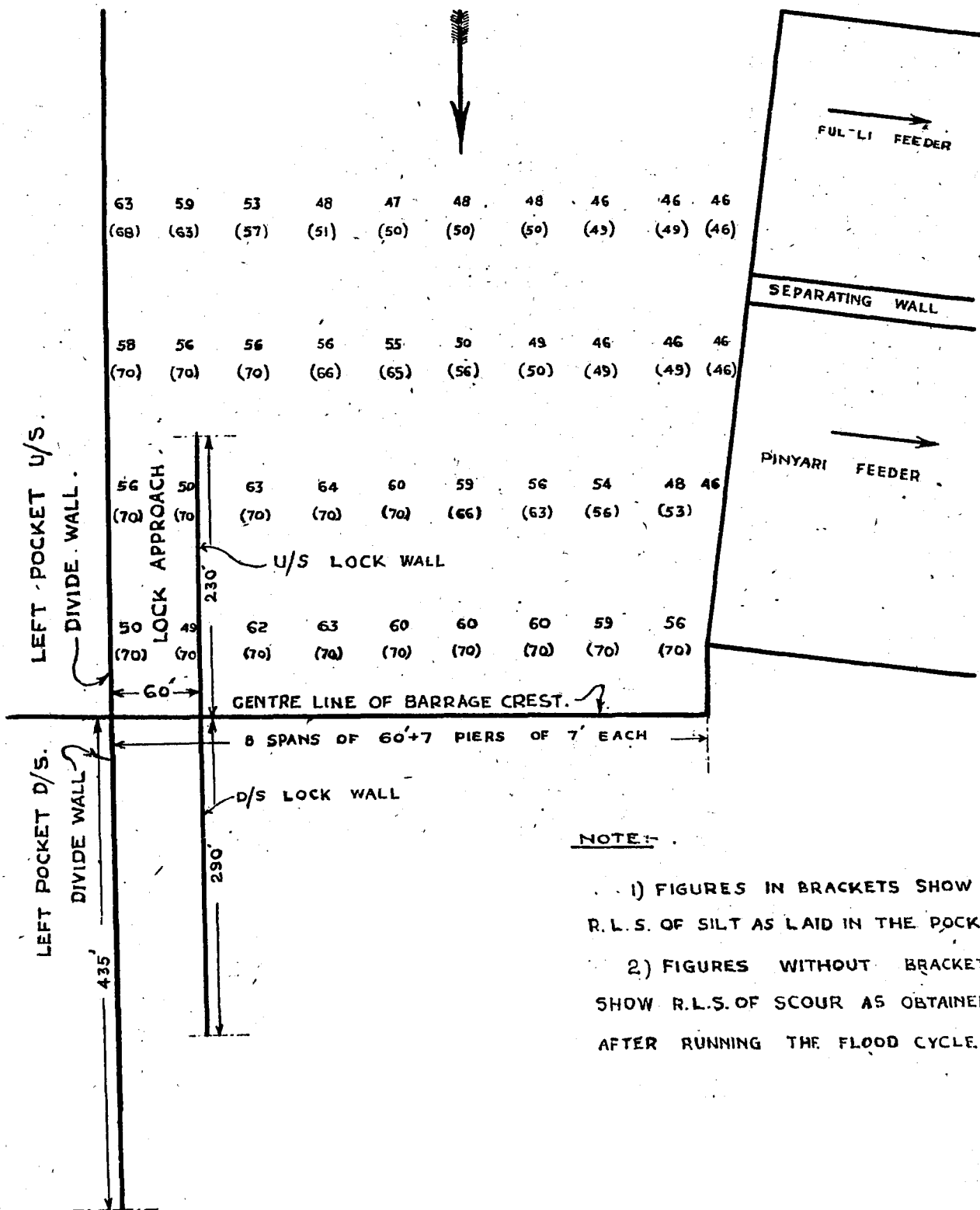
(6) The length of the downstream Lock wall may be kept 290 ft. as contemplated in the design.

FIG. 6/C-12-2.

SHOWING R.L.S. OF SCOUR OBTAINED IN THE LEFT POCKET AFTER RUNNING RISING AND FALLING CYCLE WITH OPTIMUM POCKET & LOCK GATE, OPENINGS.

(LOCK APPROACH WALL 230' LONG.)

SCALE:- 1" = 125'



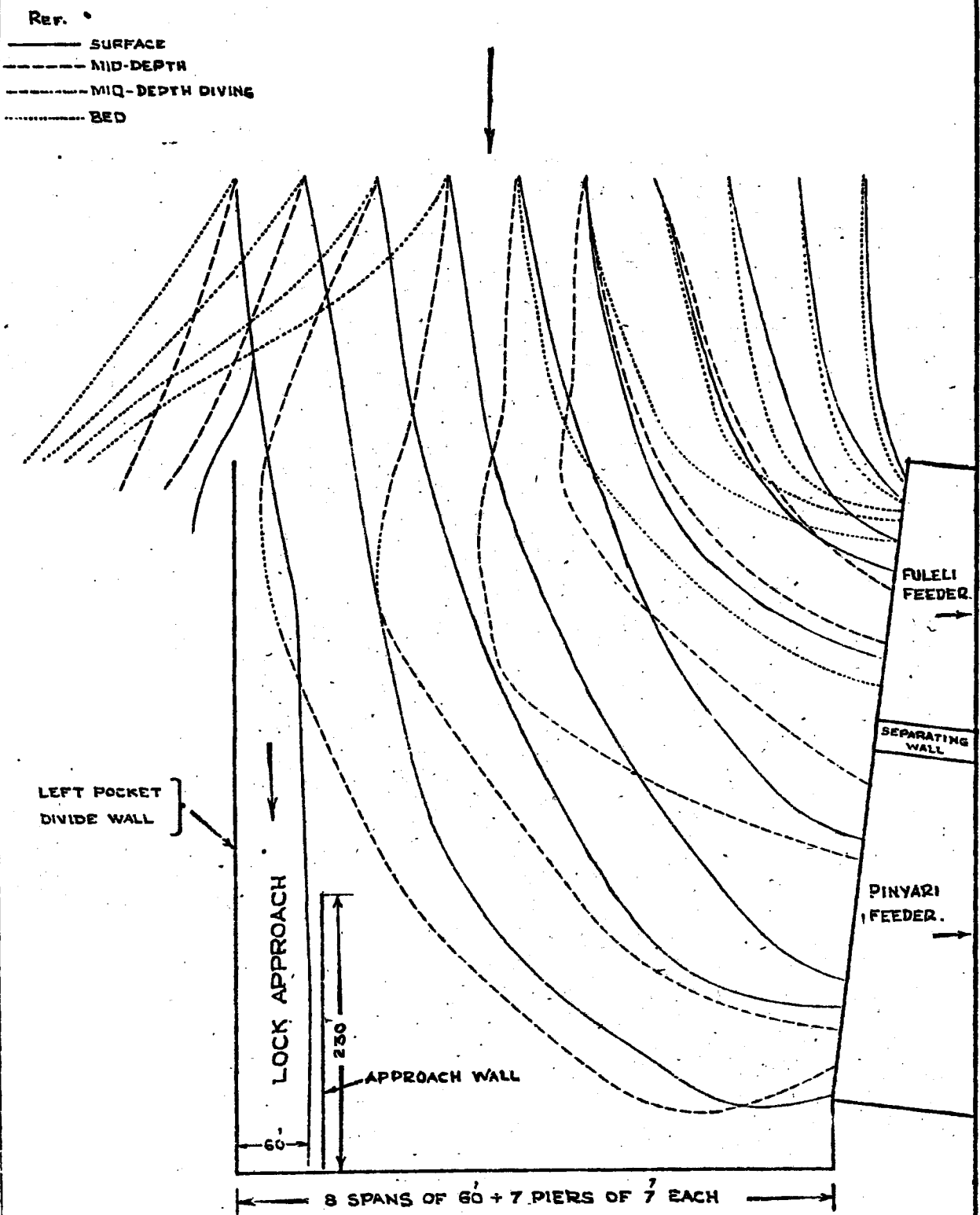
NOTE:-

- 1) FIGURES IN BRACKETS SHOW R.L.S. OF SILT AS LAID IN THE POCKET
- 2) FIGURES WITHOUT BRACKETS SHOW R.L.S. OF SCOUR AS OBTAINED AFTER RUNNING THE FLOOD CYCLE.

FIG. 7/C-12-2.

SHOWING LINES OF FLOW IN THE LEFT POCKET
FOR Q RIVER = 4,52,000 CUSECS (STILL POND)
[LOCK GATE OPENED BY 12 FT.]

SCALE $\frac{1}{125}$



Showing the optimum permissible opening and discharge through the Lock span.

Discharge in the river u/s of the barrage. (Cs.)	Water level u/s	Water level d/s	Cut-off = col. 2 minus col. 3 (ft.)	Maximum permissible opening of the lock span. (ft.)	Permissible opening of the other pocket gates.	Discharge through the Lock approach channel (Cs.)	Velocity in the Lock approach channel (ft./sec.)	V_0 in the Lock app. ch. ft./sec.	V/V_0 = $\frac{\text{col. 8}}{\text{col. 9}}$	Theoretical maxi- mum depth down to which silt can be scoured for velocity in Col. 8. (ft.)	Remarks.
1	2	3	4	5	6	7	8	9	10	11	12
200,000	68.0	63.40 (accreted w. 1.)	4.60	2	Closed	1620	1.25	4.62	0.27	2.85	Sand exclusion from left bank canals just satis- factory.
300,000	68.0	67.0 „	1.0	10	„	3840	2.90	4.62	0.63	10.80	Exclusion very good.
350,000	68.0	64.20 (Mean w. 1.)	3.80	8	„	6000	4.55	4.62	0.99	22	- do -
350,000	69.55	68.20 (accreted w. 1.)	1.35	12	„	5350	4.05	4.85	0.835	18	- do -
400,000	68.90	65.30 (Mean w. 1.)	3.60	12	„	8750	6.45	4.74	1.340	Approach channel completely scoured.	- do -
400,000	71.20	69.30 (accreted w. 1.)	1.90	12	The pocket gates opened by 2'	6350	4.20	5.05	0.83	19	- do -
500,000	72.20	67.50 (Mean w. 1.)	4.70	8	- do -	6700	4.25	5.18	0.82	20	- do -
500,000	73.85	71.50 (accreted w. 1.)	2.35	12	The pocket gates opened by 4'	7200	4.33	5.37	0.81	20.5	- do -

APPENDIX I/C-12-2.

NOTE ON

THE LOCK SPAN OF THE KOTRI BARRAGE

by

T. M. Oag Esq., Director (Navigation),
Central Waterpower, Irrigation & Navigation Commission.

The navigational problem at the Kotri Barrage is one of siting the lock in a position to afford the most practicable and easy approach for craft to pass through. This was seen on the model to be contemplated in the Project on the right side of the Divide Wall with an additional wall parallel to the Divide Wall, at a distance equivalent to the width of the lock away from it, to create easy approach conditions.

The area on the right of the left pocket Divide Wall is subject to turbulent flow varying in intensity as the current velocity increases. At the maximum velocity of 10 to 12 feet per second, even under perfect conditions of parallel flow, berthing a vessel alongside any approach quay to a lock would be attended with considerable risk both to vessel and quay and should be avoided.

A number of experiments were tried with the approach wall placed in relative prototype positions varying in length from 150 to 210 feet. An approach wall, as visualised in the Project certainly produced the desired effect of creating slacker water but only within the enclosed area (though not at its entrance) and in the matter of practicable ship manoeuvring the hazard was only shifted further upstream so that, in practice, the precise conditions which the wall intended to eliminate still existed, in the area then to be used to approach the Divide Wall to enter the approach channel to the locks, by warping under control of ropes and hawsers from the Divide Wall.

Accordingly, I agree with the Director, Central Waterways, Irrigation & Navigation Research, that no method of placing a parallel stilling wall could ameliorate the fundamental hazard of turbulent flow at high velocity to obtain an easy approach to a lock entrance on the right or west side of the divide wall. Mr. Framji, therefore, desired my opinion on the placing of the stilling wall within the Pocket.

The area on the left of the left pocket Divide Wall was then tried, and, in several experiments it was seen that a parallel wall in this area would answer the purpose of creating workable conditions.

The maximum velocity in the area would never exceed 5 feet per second in the prototype and it was observed while working a ship model that there was a slight draw of current *towards* the Divide Wall, thereby producing an ideal condition for berthing a vessel alongside of it.

Several lengths of approach wall were tried on the left side of the Divide Wall and it seems to me that its final length should be determined as the minimum necessary to eliminate draw-down near the sluices. With this exception, manoeuvring conditions are ideal and the Divide Wall itself can be used as approach to the lock.

It is therefore recommended that :

- (a) The lock should be constructed in the barrage immediately left of the Divide Wall,

and (b) the experiments at present in hand should be continued to determine the minimum length of parallel wall necessary to eliminate the turbulence created by the draw-down at the barrage gates.

(c) The Divide Wall should be used as the approach wall to enter the lock and, therefore, it should be fitted with bollards from the upstream end to the lock entrance at intervals of 100 feet.

At Kut Barrage on the River Tigris, as far as I can recollect, the parallel walls to approach the lock on either side were short ones, not more than 70 feet in length. In the case of the Kotri Barrage, even a shorter length may be found satisfactory, and it is for consideration whether it should be fitted with sluices which could be opened to keep the approach clear of silt when the lock is not in use. Whether or not it will be necessary to construct a stilling wall on the downstream end should also be tested in the model, as I doubt if an extension of the Divide Wall will be sufficient to completely eliminate turbulence within the unlocking area for vessels.

Although the question was not raised, it is understood that the lock dimensions at Kotri Barrage are to be something of the order of 60' x 250'.

The factors affecting the considerations of these dimensions are not known. But it is suggested for consideration that for practical ship manoeuvring, to pass through a lock the margins of safety accepted are usually of the order of 2 feet on the beam, 5 feet on the length and 3 inches under the bottom at the maximum draft.

If for any other reasons it is necessary to standardize the dimensions of the width of the lock to be in keeping with the other Barrage gates it is for consideration whether a longer lock might not prove more economical in the practical operation of shipping. It is usually accepted that the length of a lock is from 10 to 12 times the width, and thus in this case a lock 60' x 300' would have the same capacity as one of 30' x 600' within certain craft dimensional limits.

G — GENERAL

1. PROJECTS EXISTING AND PROPOSED—

G-1-1 *The Sakri River Project (Bihar).*

1. Introduction :—

1.1 The project provides for an inundation canal taking off from the Sakri River in Bihar. The principal function of this canal is to fill tanks etc. so as to enable the irrigators to lift water at their convenience.

The project is suggested to be developed in three stages as under :—

1st Stage :—

The Canal is to draw up to a maximum of 1,000 cusecs and must depend for sand exclusion mainly on the proper positioning and alignment of the head regulator.

2nd Stage :—

Should the sand exclusion by natural curvature be found inadequate after 3 to 4 years, a sand ejector is to be constructed.

3rd Stage :—

The Canal section and Head Regulator are to be remodelled for increased demand of water up to maximum of 2,000 cusecs.

1.2 The case was referred to the Station for general advice regarding the design of a sand-excluding Head Regulator, of the canal and allied works in the project.

As the project is to be developed in three stages, the design has been dealt with, accordingly, in three parts.

PART I.

Project design for conditions in Stage 1.

2.1 Requirements :

The gross requirements including losses worked out to 9,650 cusecs days.

2.2 Head Regulator :

(A) *Positioning of Head Regulator :*

The first requirement governing the choice of site for the Head Regulator is sufficiently developed, favourable natural curvature in the parent channel. Experience and experiments made at the Poona Station have shown that, in alluvial rivers, reliable and effective silt- or, more correctly, sand-exclusion can be obtained by correctly positioning the off-take of a canal towards the lower end of a suitably concave bank so as only to draw off the relatively sand-free, surface water, the bottom bed water being deflected into the river. This is Nature's way of exclusion and experiments with the Sarda Barrage⁽¹⁾ and the Son Anicut⁽²⁾ have shown this open-type excluder to be efficient even with relatively coarse material. A site was suggested which appeared suitable for natural exclusion in this manner, see Fig. 1.

Fig. 1.
G-1-1

The selection of the site was also guided by other considerations such as stability of the river reach in the vicinity, availability of hard and reliable strata at the proper level for foundations, adequate command of the area proposed to be flow irrigated, *et cetera*.

(B) *Alignment of the Head Regulator :*

It was suggested that the Regulator should be located on the curved right bank facing fairly square to the current and the canal made to take off tangentially to the river curve—as shown in Fig. 2.

Fig. 2
G-1-1

(1) Page 19. A. R. (T) of I. W. E. Station, Poona for 1941-42.

(2) Page 20.

This would result in the optimum sand exclusion possible under the existing conditions; and incidentally help in obtaining a high velocity of approach—practically equal to the river velocity—which will enable some shortening of the regulator width.

(C) *Clear waterway of the Head Regulator :*

The only hydrograph available was for the year 1946. After detailed study and analysis of this hydrograph, the sill of the Head Regulator was proposed to be at R. L. 263.00, i. e. 6" above the river bed. The clear lineal waterway required to pass the necessary discharges, at all stages, will be proportionate to the river discharge, as desired.

(D) *Details of the Head Regulator :*

It was suggested that the Head Regulator should consist of 10 spans, with piers 2½ ft. wide provided with semicircular cut-and-ease-waters. The width between abutments will then be 122½ ft., i. e. equal to the width of the canal, designed for a *high* bed width/depth ratio.

In order to enable the canal to tap only the surface, sand-free water at low and medium river stages, a set of sliding gates was suggested by the Station. The design of the regulator gates proposed by the Station is shown in Figure 3.

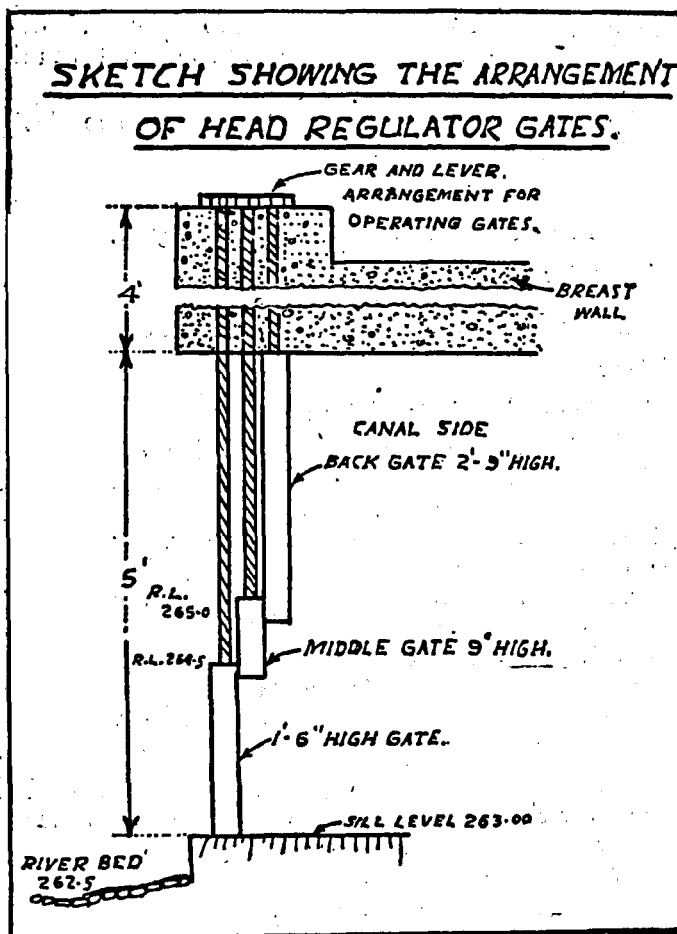


FIG. 3/G-1-1.

2.3. *Canal design : -*

(A) *Capacity : -*

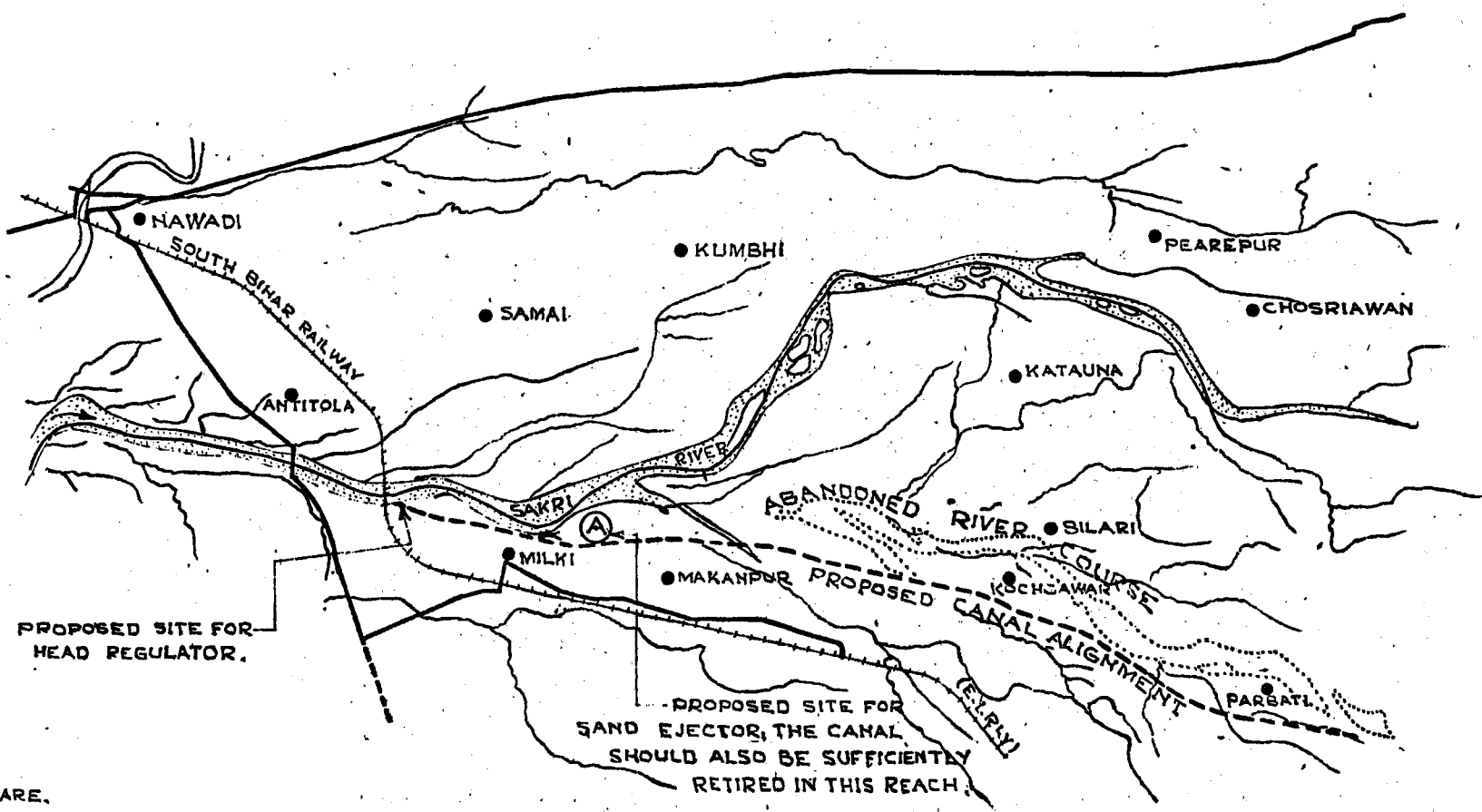
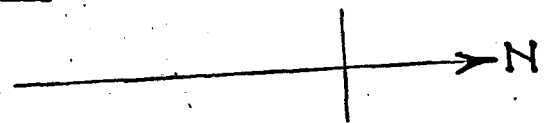
The design was made for 1,000 cusecs capacity with scope for increase, and for transporting lower discharges without undue risk of silting.

FIG:- 1/G-1-1.

INDEX PLAN

SHOWING PROPOSED CANAL HEAD REGULATOR
& SITE FOR SAND EJECTOR IN THE
SAKRI RIVER PROJECT

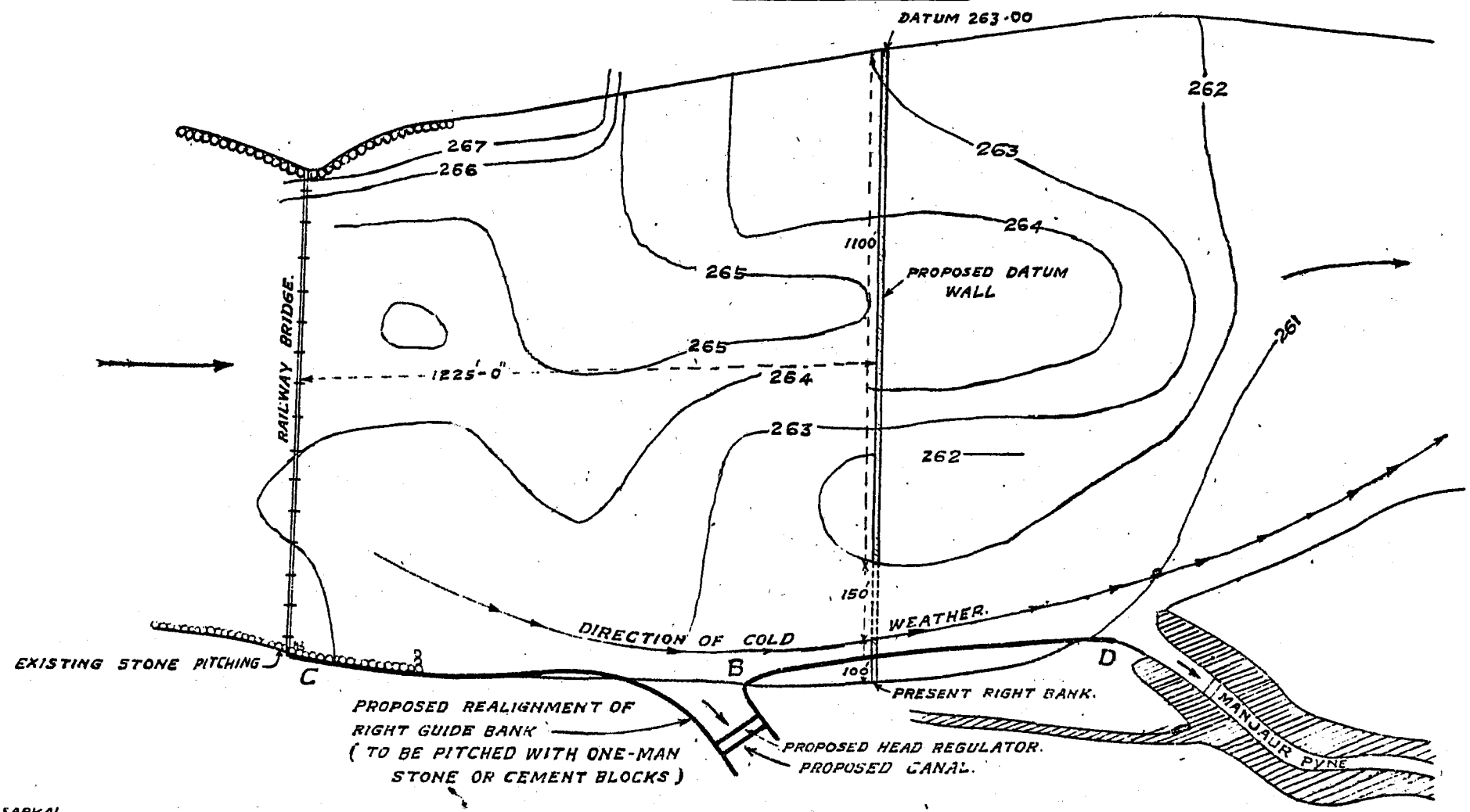
SCALE:- 1" = 2 MILES.



BY:- BHINGARE.

FIG. 2/G-1-1
PLAN SHOWING DATUM WALL & PROPOSED
HEAD REGULATOR FOR
SAKRI PROJECT CANAL.

SCALE: 1" = 300 FT



(B) Conditions guiding the canal design : -

The F. S. L. of the canal, in this case, will mainly depend on the river water level available for passing 1,000 cusecs through the Head Regulator; and has to be as low as possible with non-silting slope so that it will run in low cutting avoiding heavy losses.

The B/d ratio of the river=180, indicating a heavy bed charge in the river. A wider, shallower and steeper canal than according to the Lacey dimensions was, therefore, suggested. A wide initial section was also desirable to allow the deepening of the canal for passing increased discharges, in future, rather than widening it as the latter would be more expensive entailing alterations to cross-drainage works, crossings etc. The steeper slope provided would avoid the necessity of subsequent raising of the banks following sanding due to deficient slope.

The canal was designed for 1/2 : 1 side slopes but was recommended to be constructed to flatter side slopes so as to allow sufficient capacity even after berming to 1/2 : 1 slope.

(C) Canal section and slope adopted : -

Various alternative designs, with different slopes and with a width equal to or wider than the width between abutments of the Head Regulator, were prepared according to Lacey and Kennedy formulae.

Conditions guiding the design—mentioned in 2.3 (B) above suggested the choice of the following design :—

Bed width	= 120 ft.
Slope	= 0.0004, i. e. 1 in 2,500.
S. S.	= 1/2 : 1 (to be constructed at 1 : 1).
Depth for 1,000 cusecs	= 3.22 ft.
Velocity for 1,000 cusecs	= 2.61 ft./sec.
F. S. L. for 1,000 cusecs	= R. L. 262.03.

(D) Canal alignment : -

The canal alignment proposed by the local Project Engineer approached dangerously close to the river at the point, marked (A) in figure No. 1. The river in this reach is flowing along a sharp bend, and the possibility of an inroad into the canal must not be overlooked. It was, therefore, suggested that the canal should, from the start, be sufficiently retired in this reach, to avoid the necessity of a subsequent diversion.

Fig. 1
G-1-1

2.4. Pucca Datum.

A " pucca datum " across the river designed to maintain its bed at the same level as the Regulator sill was proposed by the Project Engineer.

In order to protect the Datum wall from scour and subsequent undercutting, loose stone pitching—as in Fig. 4—was suggested. This will also prevent any retrogression of levels downstream of the datum affecting the bed levels upstream.

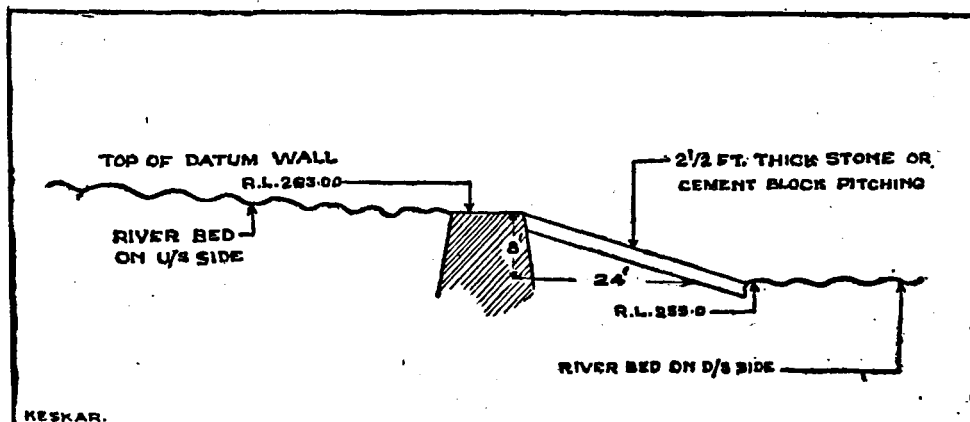


FIG. 4/G-1-1 .

2. 5. Training Works :

To hold the river between its present high banks, to provide a smooth approach and to stabilise the existing favourable concave curvature along the right bank for a suitable offtake, it was recommended that the right bank should be pitched and sufficient apron be provided against the probable scour.

PART II.

3. Project design for conditions in Stage 2:

There is scope in the proposed design for the subsequent addition, in the future, of an Ejector near point A in Fig. 1 if at all found necessary.

PART III

4. Project design for conditions in Stage 3.

In future, the requirements are expected to increase to double the present requirements.

The modifications then required will be as follows:—

- (1) The Head Regulator will have to be sufficiently widened, or flumed and widened, to double the discharging capacity.
- (2) The canal will have to be remodelled and/or lined, the choice being governed by the considerations of economy and efficiency.

The design suggested, while meeting the present requirements, allows full scope for such alterations and modifications as are likely to be necessary.

G- GENERAL

1 Projects existing and proposed.

1. Introduction

The relevant portions of the 1947 Report (prepared by Mr. Fenton, the Dy. Chief Engineer, Ganga Bridge Project of the E. I. Railway) on the proposal for the construction of a combined road and rail bridge over the Ganga river were forwarded to the Director for his opinion.

The field work had included investigations at both the alternative sites of Patna and Mokameh—Fig. 5. The results of Mr. Fenton's investigations showed that the Patna Site, at the present time and in the near future, is unfavourable from an engineering point of view and that the construction of a bridge in that neighbourhood is *not* a feasible proposition.

Fig. 5
G-1-2

A traffic survey, undertaken with a view to determine the site to be preferred from the commercial point of view, resulted in the conclusion that, given comparable capital expenditure on a bridge at each of the two sites, the financial justification would be roughly the same.

It has been stated that the Govt. of Bihar is extremely keen on the proposed bridge being located at Patna and not at Mokameh and, if the river condition at Patna made the construction of a bridge at that site a feasible undertaking, it is probable that, even if the cost of the bridge at Patna were appreciably greater than the one at Mokameh, the Patna Site would be the one decided upon.

The Director's opinion was requested to be shaped in reply to the following questions :-

- (i) (a) Do you agree with the conclusions in the Report that a bridge at Patna at the present time or in the immediate future is not a feasible engineering undertaking ?
- (b) Is it your view that by conducting model experiments vital information, not apparent in the course of ordinary investigations, may be obtained and a clearer picture of the worthiness or otherwise of the site produced ?
- (ii) (a) Do you agree that the site suggested for a bridge at Mokameh is one offering the security with which the Report credits it ?
- (b) Do you consider that any model experiments are necessary to support or modify the conclusions of the Report in this respect ?
- (iii) If your reply to question (i) (a) above is to the effect that in your view the Patna site is not to be ruled out of court but is one which offers the possibility of a reasonably secure bridge, how would you compare its worthiness and security with those of the suggested bridge at the Mokameh site ?
- (iv) Do you agree with those matters relating to scour, depths of foundations, design of training bunds etc., dealt with in the Report in respect of both the alternative sites of Patna and Mokameh, or do you consider that they need modification in any respect ?
- (v) Have you any other general criticisms or suggestions to offer in respect of any of the above issues or any others that may occur to you ?

2. At very short notice, the Director submitted the following criticism (para 3 below), making it clear that some of the opinions were *tentative* and subject to modification on maturer and more detailed examination of the data supplied.

As desired, the Director's opinions were furnished in the shape of replies to the question raised, though this involves some pardonable repetition and lack of logical sequence.

- 3.1 (a) *Do you agree with the conclusions in the Report that a bridge at Patna at the present time or in the immediate future is not a feasible engineering undertaking ?*
- (b) *Is it your view that by conducting model experiments vital information not apparent in the course of ordinary investigations, may be obtained and a clearer picture of the worthiness or otherwise of the site produced ?*

3.1.1 These two questions will be answered simultaneously. The Director cannot wholly endorse the view "that no bridge at Digha (Patna), short of one spanning the major portion of the 5 mile Khadir, ever will be found to be a feasible undertaking"; because with suitable training measures, evolved from model experiments, the existing river conditions at Patna do not summarily preclude the practicability of a bridge in this vicinity, although, the conditions not being favourable, the bridge together with the training works may prove expensive, both in first cost and maintenance. Undoubtedly, the site at Mokameh appears, on the face of it, more suitable from hydrological and other considerations and a bridge there is also likely to be cheaper and better as an all-time solution.

3.1.2. The following *vital* information, which it is not possible to obtain otherwise, can be obtained from a model :-

(i) Bed and surface flow lines, without and with a bridge at the different proposed sites (or at other sites which appear to be more promising from the model), would indicate the nature and extent of the optimum training works required and the protection they would afford. For example, whether an upstream guide bund only 6000 ft. long would be sufficient in the case of the Patna Project alignment, and its repercussions on the south unprotected bank, can only be conclusively ascertained from model experiments.

(ii) Velocity and discharge distribution in various spans, under varying river courses and under the worst hypothesis of flow, is of vital importance in deciding the relative merits of the various sites and its design details (e. g. training works, waterway, depth of foundations); because concentration of intensity of discharge and obliquity of approach and attack are closely associated.

(iii) The model can give useful indications in support or contradiction of certain conjectures in the Report; for example, about the probable behaviour of the river upstream of the Patna site, upon which the rejection of the Patna site and the recommendation of the Mokameh site have been mainly based.

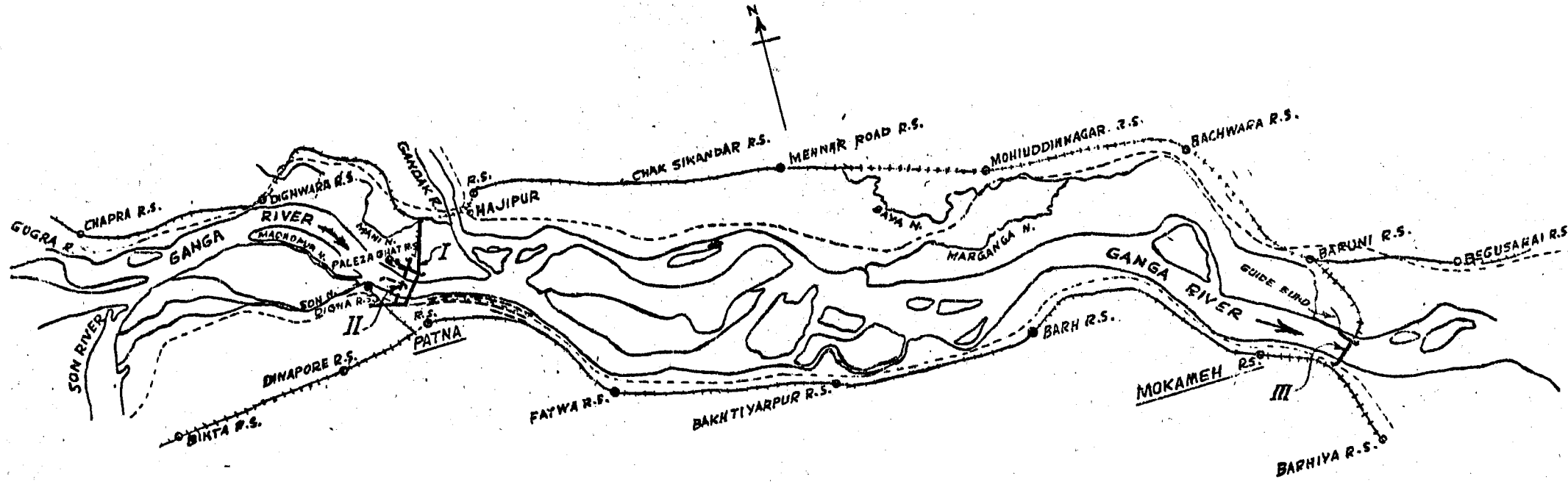
And, (iv) the probable positions of shoals (accretion), deep channels, and scour and erosion can be satisfactorily indicated by a model.

3.1.3 Although the Mokameh site apparently offers the better engineering prospects, if the bridge must, for some other reasons, be constructed at Patna, the Director cannot agree that the Project alignment of the Patna site is necessarily the best; on the other hand, subject to verification on a model, he is inclined to favour the alternative alignment (B) half mile upstream of the Project alignment, shown in fig. 6 in dotted lines. His reasons for this preference are that while the bridge at this upper site, even if constructed now, would require to be no longer than the proposed bridge on the Project alignment at Patna, the upper site would appear to have the following material advantages over the other :-

FIG. 5/G-1-2.

PLAN OF THE GANGA RIVER
SHOWING THE PROPOSED ALTERNATIVE SITES FOR THE
RAILWAY-CUM-ROAD BRIDGE NEAR PATNA & MOKAMEH.

SCALE 1" = 8 MILES.



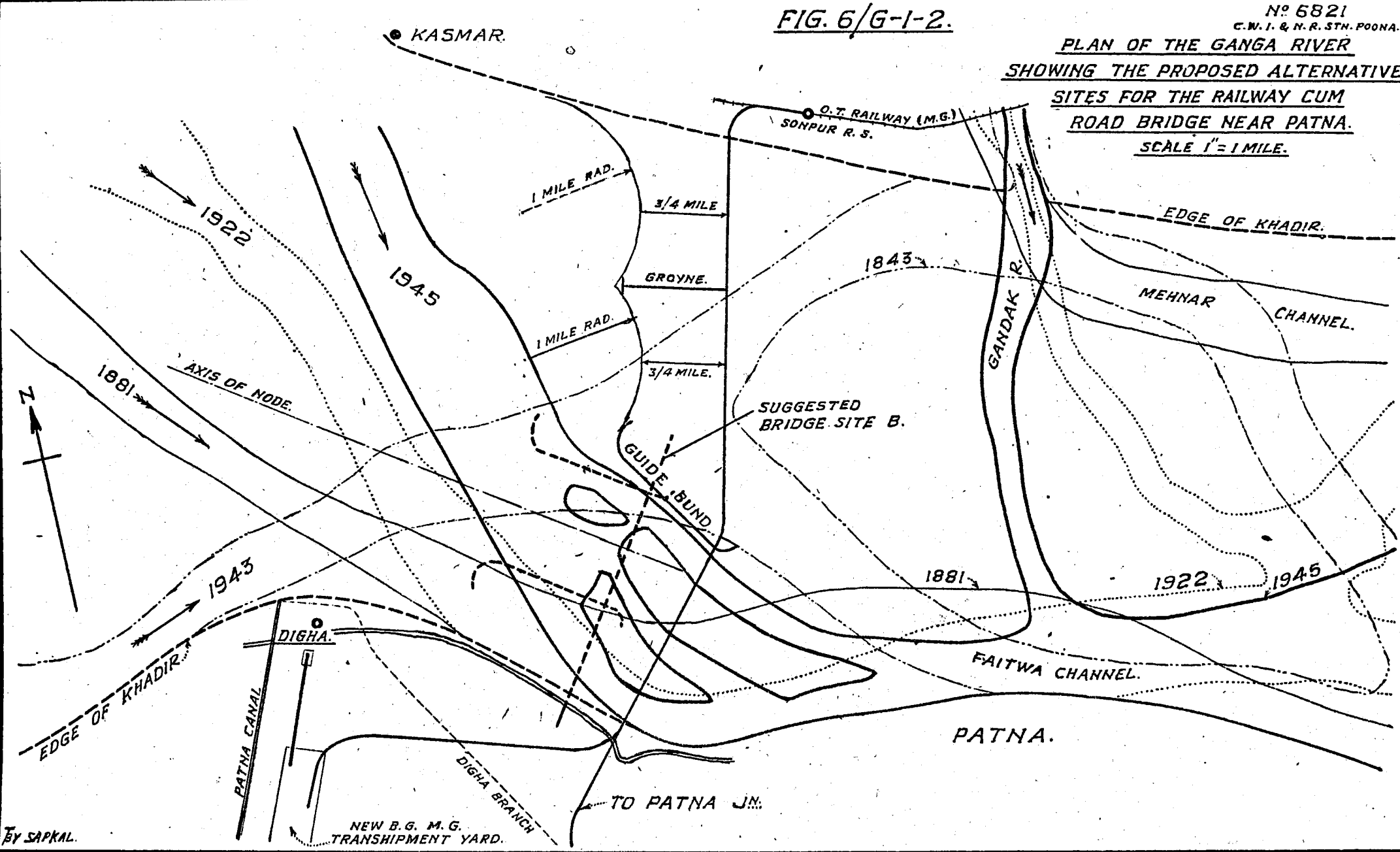
- I. BRIDGE SITE AT PATNA.
- II. ALTERNATIVE SITE AT PATNA.
- III. PROPOSED MOKAMEH SITE.

FIG. 6/G-1-2.

No 6821
C.W.I. & N.R. STN. POONA.

PLAN OF THE GANGA RIVER
SHOWING THE PROPOSED ALTERNATIVE
SITES FOR THE RAILWAY CUM
ROAD BRIDGE NEAR PATNA.

SCALE 1" = 1 MILE.



BY SAPKAL.

(i) Normal flow, and even more so oblique flow from north to south, would have better exit conditions downstream of the bridge, without necessarily impinging on the bank adjacent to the Patna town, to quite the same extent as a bridge along the proposed Project alignment would.

(ii) The backward swing of the Gandak which is likely to persist so long as the Mehnar Channel is the main river channel downstream—and this is likely to be so in the near future as the meander upstream of the Kasmar is progressively pressing downstream—requires that the bridge site should be further upstream than the Project alignment to avoid the endangering effects thus created. At the same time, the alternative alignment (half mile upstream of the Project alignment) is well downstream of the centre of the node.

(iii) At this alternative site, which is nearer to the pegging point on the north bank (Kasmar), no groyne is likely to be required in addition to the left guide bund, which at most may have to be somewhat longer. This is based on the Director's opinion that the curvature of embayments assumed in the Report appears too sharp for a river of the size of the Ganga. Actual calculations of the average radius of curvature of the sharpest meander—from the measured data of meander characteristics—have given a minimum value of 4 miles; therefore, when the channel is in contact with the Khadir, or near the apex of the meander where the radius is smaller, the minimum radius may be taken as two miles, *not one mile* as assumed in the Report.

Without justification by model experiments the Director considers the association of groynes with training bunds unwarranted; because such groynes or spurs are likely to attract flow and pin it to their heads and create unpredictable, sometimes even dangerous, conditions which would upset the normal expectations of the working of a guide bund.

(iv) While borings have shown that the south bank at (Digha) Patna is equally favourable to both the alignments being "extremely worthy" as a natural training bund (page 4, Chapter 5 of the Report), it is only the alternative site which will permit *symmetrical* as well as *parallel* guide bunds, the south natural training bund being the edge of the Khadir and the north guide bund being constructed symmetrically where river conditions permit.

The Director is, therefore, unable to find himself in entire agreement either with the Project alignment as being the best at Patna or with the view that in future the best site would be the site downstream of the Gandak confluence and at the easternmost outskirts of Patna on the south arm of the Diara island (the Fatwah channel). In addition to the reasons given in the Report, pages 10 and 11, Chapter 4, his objections to the downstream alignment is that a site in such close proximity to an unstable confluence will be an interminable source of uncertainty, trouble and expense.

3. (2) (a) *Do you agree that the site suggested for a bridge at Mokameh is one offering the security with which the Report credits it?*

(b) *Do you consider that any model experiments are necessary to support or modify the conclusions of the Report in this respect?*

Except that the south bank is apparently not so "worthy" as at Patna, the Director agrees that the site suggested for a bridge at Mokameh—see fig. 7—offers reasonable security as contended in the Report and is probably the best site at Mokameh, so far as can be judged from surveys. A model alone, in the Director's view, can, however, *conclusively* show whether the Project alignment at Mokameh site is better than the one about 2500 ft. upstream of the present alignment, the one presumably in view in the B. & N. W. Railway proposals made sometime about 1917; so also, in view of the less dependable nature of the south bank, whether one guide bank on the north bank would be

sufficient without any protection on the south bank would need to be elucidated by a model. The Director agrees that the present conditions are especially favourable but, as has been mentioned in the Report, these conditions are likely to persist for a few years and so, though the construction of the bridge should be expedited as much as possible, in the Director's considered opinion, model investigations *must precede* the construction. For instance, he cannot accept unreservedly that the bridge as proposed will be completely "secure against any possible embayment consequent on the path of the river upstream taking the most disadvantageous direction, even though it might eventually assume the path now followed by the Bya Nadi" (page 17, Chapter IV of the Report); should this come to pass, it is not improbable that there would be direct frontal attack on the head of the proposed guide bund. The model would indicate what changes, if any, are required in the orientation of the site, in the alignment and length of the proposed guide bund, and in the apron protection proposed for it.

3. (3) *If your reply to question (1) (a) above is to the effect that in your view the Patna site is not to be ruled out of court but is one which offers the possibility of a reasonably secure bridge, how will you compare its worthiness and security with those of the suggested bridge at the Mokameh site ?*

3.3.1 Without any hesitation or reserve, the worthiness, security and prospects of the Mokameh site are, on the evidence before the Director, much superior to the Patna site; but the Director would not go so far as to say that no favourable site can ever exist at Patna because model experiments might perhaps indicate how it would be possible to insure a reasonably secure site by means of adequate training measures at unprohibitive expense.

3.3.2. Admittedly, there are obvious and serious objections to the Patna site, the most important of which, in the Director's opinion, are the following :-

- (a) all the Patna sites are above or below the confluence of two or more rivers—which cause unforeseeable complications due to variations in the flood cycles, gradients, and sediment load, changes in river courses of the individual rivers, synchronization or otherwise of their peaks and dominant discharges; etc;
 - (b) the left guide bank would be resting on somewhat treacherous ground composed of sand interspersed with clay layers, which experience as well as experiments have proved to be unsuitable for the stability of guide bunds and the efficient functioning of falling aprons;
- and (c) the left guide bund will be splayed (while the right natural Khadir edge would be normal) adding to the obliquity and unsymmetrical incidence of approach imposed by the river conditions, endangering itself as well as the bridge.

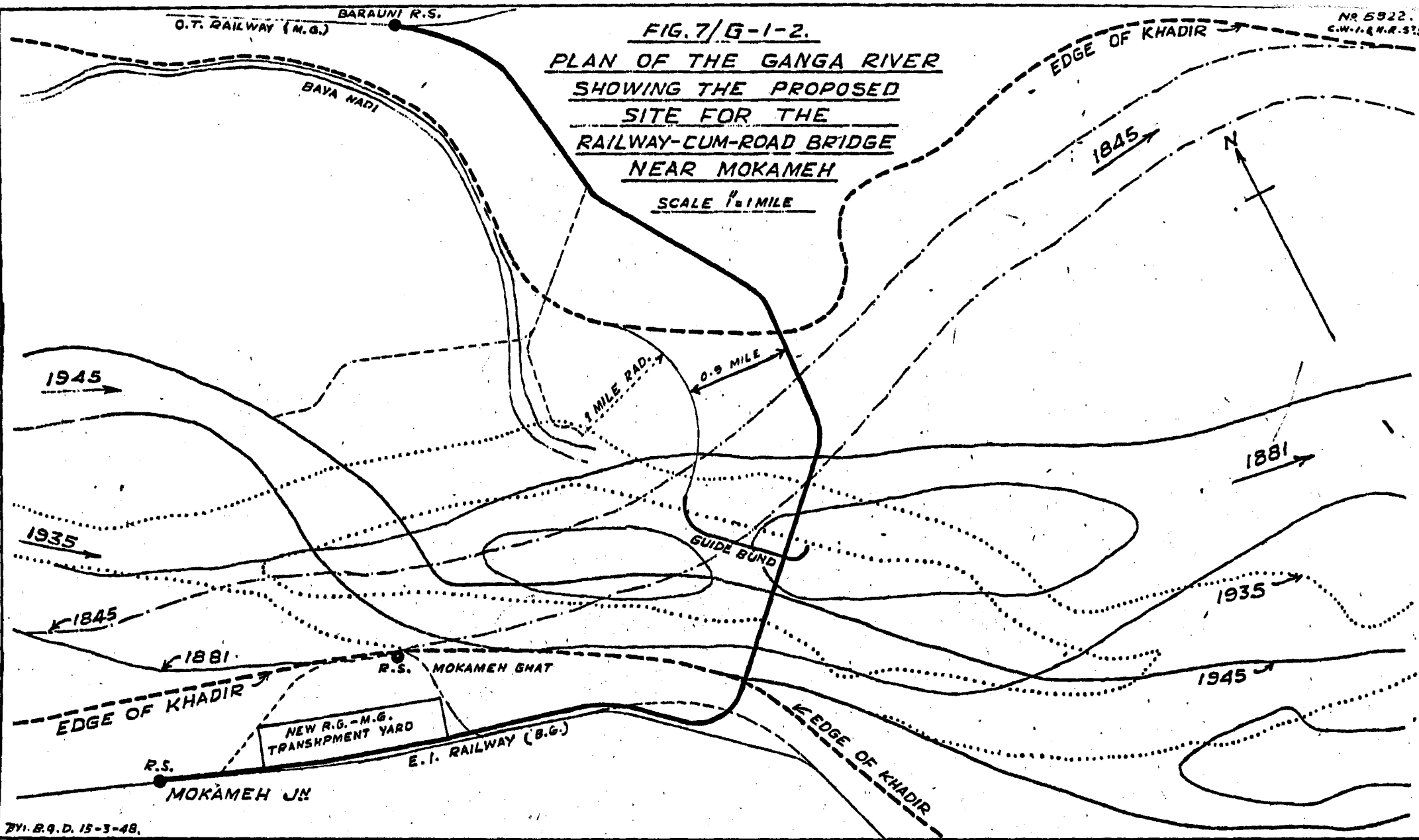
3.3.3. Similarly, the Mokameh site has obvious merits as a bridge site for reasons already emphasized in the Report and, in the Director's opinion, particularly because -

- (a) the borings show that the guide bund and falling apron would be founded on uninterrupted sand, securing its stability if properly designed;
- (b) of the relatively narrow meander belt, a comparatively straight approach, the narrow width of the Khadir and of the cold weather channel, and the suitability of the present river course to an early undertaking of the bridge construction;
- (c) while the river is at present pressing hard against the south edge of the Khadir, the north artificial guide bund can be constructed normal to the bridge on a comparatively high *chur*; and
- (d) the fanning out on the downstream enables free movement of meanders there to permit the river to adjust itself to the restrictions in lateral movement upstream of the gorge, which secure its approach.

BARAUNI R.S.
O.T. RAILWAY (M.G.)

FIG. 7/G-1-2.
PLAN OF THE GANGA RIVER
SHOWING THE PROPOSED
SITE FOR THE
RAILWAY-CUM-ROAD BRIDGE
NEAR MOKAMEH

SCALE 1/2 MILE



3.3.4. Thus, although there are clear advantages in favour of Mokameh and equally strong objections against Patna, owing to certain divergent or unforeseeable factors and the unsubstantiable conjectures which have to be made in respect of the future behaviour of the river, it is impossible to be unequivocal and reject the Patna site completely out of court without model experiments in support thereof. Even if the Mokameh site is forthwith accepted in view of its evident greater advantages, model experiments appear desirable to find out the best method of keeping the river course stable along the right (south) bank of the river at the bridge. The Director must make it very clear that year to year changes cannot be wholly reproduced in a model due to model limitations in respect of quantitatively reproducing accretion and bank erosion: nevertheless, there would be sufficient indications from the model with respect to river behaviour under the worst hypothesis to give far greater confidence in the final selection of the site and the proposed training works.

3. Do you agree with those matters relating to scour, depth of foundations, design of training bunds etc., dealt with in the Report in respect of both the alternative sites Patna and Mokameh, or do you consider that they need modification in any respect?

3. 4. 1. Site

In respect of the precise local position of the bridge alignment at Mokameh or Patna, it has already been stated above that model experiments would be necessary to elucidate whether the Project alignment or an alignment somewhat upstream or any other would be more suitable.

In the case of the Mokameh site, the reasons stated for adopting the Project alignment (vide Page 2, Chapter 5 of the Report) appear strong; on the other hand, the security given by the clay on the south bank when related to the maximum anticipated scour is questionable. In spite of this, the Director agrees that because revetment and apron may only tend to attract and concentrate the flow along this bank no artificial protection should be provided to the south bank unless and until experiments or experience warrant their necessity; but he is not so sure as the writer of the Report that no protective work will ever be required there. While agreeing with the preference in general of parallel guide banks, or in the case of one being a natural inerodible bank an artificial guide bund parallel to the general direction of the Khadir, the Director would stress the advantages of having *symmetrical* guide bunds.

3.4. (2) Peak Discharges.

Logarithmic plotting masks the larger errors at higher stages and this may in extrapolation give a somewhat inaccurate figure: on the other hand, arithmetic plotting with its large and erratic scatter, and the fact that the gauge-discharge relationship at the higher stages is not normally linear, makes extrapolation even more undependable, unless done by statistical methods of regression. In any case, extrapolation should extend only over a small range beyond the measured data: while the extrapolation has been made from discharges measured up to 1,835,000 cusecs to 3,000,000 cusecs for the Patna site, and from measured discharges up to 1,870,000 cusecs to 2,500,000 cusecs for the Mokameh site. It is suggested that higher discharges should be measured, if at all possible, in the ensuing seasons to obtain a nearer approximation to the peak discharge.

Secondly, measurements of surface velocities with a coefficient of .85 applied for conversion to the mean velocity along the vertical—as is stated by Mr. Varma, Ex. Engr. to have been done—requires sufficient data in support of the coefficient, as the coefficient varies widely from river to river, and on the same river, and even from vertical to vertical at the same site, from .85 to .9, and is more generally of the order of .89. This variation is especially marked when there is curved flow in the river.

However, as these corrections are of an opposite nature counteracting each other, pending further measurements the Director is inclined to accept the discharges as being reasonably safe. He would add that any comparison of the peak discharge at Patna and Mokameh with that at Hardinge bridge can at best be qualitative because the valley storage (flood absorption), spill, and tributary in-flow discharges are indefinite.

3.4. (3) Waterway :—

The coefficient K in Lacey's formula $P_w = KQ^{1/2}$ is 2.667 only under the hypothesis to which the Lacey formulae are applicable, namely "with minimum transported load consistent with a fully active bed" (1); where the load transported is in excess of the Lacey load, the channel is wider and shallower than the Lacey section. defect narrower and deeper

It is understood from discussions with Mr. Varma that the shape W/D or P/R of the Ganga at Mokameh is of the order of $5000/40 = 125$ at dominant discharge, as against Lacey's value for the appropriate conditions ($Q = 2$ million cusecs, $f = 1.15$) of 75. It is true that in a bridge with adequate training works and without bed protection or pitching round piers, the bed is free to scour unrestrictedly under maximum flood; and it is better not to have excessive waterway which shoals up under normal high floods, and gets covered with 'pilchy' and jungle and coherent, inerodible clay so that when a super-flood comes down there is greater obliquity of flow and concentration of discharge. At the same time, some, though not perhaps full, allowance is necessary for the divergence from the Lacey regime conditions; while about 10 per cent allowance already exists over Lacey $P = 2.667 \sqrt{Q}$ in that in arriving at the number of spans an amount equal to twice the width of the wells deducted from the distance between centers of piers to obtain the effective waterway per span. The adequacy of waterway for the Mokameh site, therefore, needs to be reviewed from this angle of the divergency of shape from the Lacey shape.

A comparison with the Hardinge Bridge waterway is not, *prima facie*, justifiable because at Hardinge Bridge the flow is known to be oblique and concentrated in a few spans which it is neither desirable to simulate nor reasonable to expect at Mokameh.

3.4. (4) Scour and depth of piers, scour at shank and head of guide bank.

While the maximum depth of scour at piers has been worked out in the Report according to Lacey's formula — see appendix I — the anticipated scour at head and shank of guide banks can also be similarly computed and is furnished below, together with the computed maximum likely scour according to Spring's and Gales' methods. As the width : depth ratio of the Ganga is higher than of the appropriate Lacey section, the Lacey formula should give scour depths on the safe side: the maximum scour anticipated at the curved heads of guide banks will be $2.75^{(2)}$; at shanks of guide banks = $1.5 R^{(3)}$; at bridge piers = $2 R^{(2)}$; where R is the Lacey regime depth in the formula, $R = 0.472 (Q/f)^{1/3}$.

(i) According to Spring:

(a) At the bridge piers and shanks of guide banks

Free scour observed at —

Patna = 53 feet.

Mokameh = 38 feet below L.W.L.

Allowing 33% for arriving at equivalent scour at bends with inerodible banks,

(1) Instt. of C. E. Paper 5518 'A General Theory of Flow in Alluvium' by G. Lacey.

(2) I. W. E. S. Annual Report (T) for 1944, page 78.

(3) C. B. I. Publication No. 12, 'Design of Weirs on Permeable Foundations' by Rai Bahadur Khosla.

greatest discoverable scour at the piers and shanks,

$$\begin{aligned} \text{at Patna} &= .53 + 18 = 71 \text{ ft.} && \text{and } 104 \text{ ft.} \\ \text{at Mokameh} &= .38 + 13 = 51 \text{ ft.} && \text{and } 83 \text{ ft.} \\ &&& \text{below L. W. L.} \quad \text{below H. F. L.} \end{aligned}$$

(b) At the heads of guide bunds —

Spring recommends that at the heads the thickness of apron should be doubled and the breadth be kept twice the depth of scour instead of 1.5 x depth of scour. This means that the design should be based on 1.7 times 'the greatest discoverable scour below low water level.'

	At Patna.	At Mokameh.
Greatest discoverable scour as above (ft.)	71	51
Add 70% for heads (ft.)	50	36
Deepest scour at heads below L. W. L. (ft.)	121	87
or below H. F. L. (ft.)	154	119

(ii) According to Gales:—

	At shanks and bridge piers.		At heads of guide banks.	
	At Patna	At Mokameh	At Patna	At Mokameh
Maximum depth of scour below L. W. L. (ft.)	53	38	53	38
Add 33% for hard bank (ft.)	18	13	18	13
Deepest known scour (ft.)	71	51	71	51
Allowance for contingencies for C class rivers at 45 and 90% respectively (ft.)	32	23	64	46
Add difference between H. F. L. & L. W. L. (ft.)	33	32	33	32
Deepest anticipated scour below H. F. L. (ft.)	136	106	168	129

(iii) According to Lacey:

$$R \text{ (Lacey)} = .472 (Q/f)^{1/3}$$

where R = Lacey regime depth in ft.,

Q = Maximum discharge in cusecs; here maximum discharges = 3×10^6 cusecs and 2.75×10^6 cusecs at Patna & Mokameh respectively.,

and f = silt factor = 1.15

(a) At bridge piers:

Maximum depth of scour = 2 R below H. F. L.

$$\text{at Patna} = 2 \times .472 \left\{ \frac{(3 \times 10^6)}{1.15} \right\}^{1/3} = 130 \text{ ft.}$$

$$\text{and at Mokameh} = 2 \times .472 \left\{ \frac{(2.75 \times 10^6)}{1.15} \right\}^{1/3} = 126 \text{ ft.}$$

(b) At *shanks* of guide bunds :

Vide C. B. I. Publication No. 12, page 155, deepest scour = 1.5 R. below H. F. L.

$$\text{At Patna} = \underline{98' \text{ ft.}}$$

$$\text{At Mokameh} = \underline{95' \text{ ft.}}$$

and (c) at *heads* of guide bunds ; deepest scour = 2.0 to 2.5 R (vide C. B. I. Publication No. 12, page 155) the smaller depth being for the larger radius curves. According to examination of existing bridges, however, the scour at the head of larger radius noses of guide banks is found to be 2.75 R. (1)

Accordingly, maximum scour below H. F. L. —

$$\begin{aligned} \text{at Patna} &= \underline{179 \text{ ft}} \\ \text{and at Mokameh} &= \underline{173 \text{ ft}} \end{aligned}$$

Hence, recapitulating, according to various authorities, the *maximum* scour in feet below H. F. L. would be :—

TABLE 1 G-1-2

	At shanks of guidebanks (A)		At heads of guide banks (B)		At piers (C)	
	Patna site (ft.)	Mokameh site (ft.)	Patna site (ft.)	Mokameh site (ft.)	Patna site (ft.)	Mokameh site (ft.)
	Adopted in Report	Not calculated.		Not calculated.		130
(i) According to Spring ...	104	83	154	119	104	83
,, ,, Gales ...	136	106	168	129	136	106
(iii) ,, ,, Lacey ...	98	95	179	173	130	126
as modified by the Station observations.						
(iv) Recommended by the Director (ft)	100 (a) 136 (b)	95 (a) 106 (b)	180 175		130	125

Note (a) Further Upstream, (b) Near abutments.

The deepest "free" scour is stated in the Report to be observed in the *dry* season, while deeper scours are known to occur during the falling floods at about 2/3 to 3/4 flood; nevertheless, the anticipated maximum scour according to the various methods appear to agree reasonably well.

3. 4- (5) Grip of pier foundations.

According to current practice a grip of R (See Major McIntyre's Paper on "Well foundations for Bridges" Railway Board Committee Tech. Bulletin No. 78) should be provided for bridge piers. With maximum scour = 2 R (Lacey) and a grip of R, the depth of foundations below H.H.F.L. should be—

	At Patna site	At Mokameh site
Maximum depth of Scour (ft.) = 2 R	130	126
Add grip (ft.) = R	<u>65</u>	<u>63</u>
	195	189

(1) Page 78, Annual Report (Tech) of the Indian waterways Experiment Station, Poona, for 1944.

These figures are practically the same as recommended in the Report based on Gales' grip so that the latter may be adopted. It would be better still to calculate the depth of pier foundations for each pier equal to $3R$ (Lacey) for the f appropriate to the pier. The piers next to the training bund or Khadir should be carried 10' deeper for additional safety, as already proposed.

Although not in keeping with the established practice, the Director agrees that no pitching should be provided at the piers and he is glad to see that no provision for these has been made. This is in accordance with the findings and recommendations of the Poona Station. In this respect, reference is invited to page 12 of the I. W. E. S. Annual Report (Tech) for 1941-42, where the conclusions arrived earlier in 1939-40⁽¹⁾ on protection of bridge piers against scour are summarised. (2)

"(ii) The stone at the nose remains undisturbed until the attack reaches a critical stage, when the stone suddenly begins to be carried away and scour pit rapidly forms, reaching a depth nearly as great as had there been no protective stone."

At any rate, if pitching is provided round the pier it should be laid at as low a level as practicable, being optimum at 85 per cent of the maximum scour depth without piers. (3)

3.4. (6) Training works.

- (i) As has been stated above, it is extremely difficult and costly to protect guide bunds with falling aprons when the sand bed contains interspersed layers of clay—as was evident in the Hardinge Bridge experiments. In such cases, it is better to construct a new sand-bank and then pitch it with stone; but this would perhaps be too expensive.
- (ii) The optimum length of the guide banks can only be conclusively settled, in cases where the full length as required by Gales is not provided, by model experiments; in particular, the guide bank proposed for the Patna Project alignment appears to the Director to be a little on the small side, specially when it is splayed; while the length proposed for the Mokameh appears sufficient relative to its waterway. Whether groynes—in association with the guide banks—will enable the length to be suitably curtailed can be ascertained from a model; but without such testing the Director is averse to the provision of such spurs for reasons already stated.
- (iii) The radius of the noses of the guide banks is proposed to be smaller than recommended by Gales; while this would be sufficient for negotiations by meter gauge track it is likely to cause greater scour and full apron protection, as recommended above for an anticipated scour depth of $2.75R$ should at least be provided.
- (iv) The anticipated scour depths having been computed as above, the apron widths can be determined for the depths of apron adopted on page 22, Chapter 5, of the Report in the usual manner.

3.4. (7) Piers.

The Director agrees that monolithic, semi-circular-ended wells are much to be preferred for sinking in sand to either the double-octagonal or the twin-circular

(1)	C. I. H. R.	Annual	Report (Tech)	for 1939-40	pp. 33-40
(2)	I. W. E. S.	"	"	" 1941-42	p. 12.
(3)	"	"	"	" 1939-40	p. 38

type; but he is averse to making the super-structure above a certain level—R. L. 125 (at Mokameh) — of twin octagonal reinforced concrete piers. Cut-and-embankment waters of the semi-circular type should extend to sufficiently above the highest high flood level.

3.4. (8) Provision against earthquakes.

Being in an area of considerable seismic activity and very severe earthquakes, it is desirable to make reasonably sufficient, if not full, provision against forces due to earthquake even though it may lead to rather expensive piers. The Director suggests that this point may be reconsidered, and suitable reinforcements provided to cope with horizontal forces due to accelerations *at least* up to $g/5$ (against $g/8$ provided) in combination with longitudinal forces resulting from braking and traction.

Q. (5) *have you any other general criticisms or suggestions to offer in respect of any of the above issues or any other that may occur to you?*

In the short time available, the Director cannot be expected to give exhaustive general criticisms in respect of the issues raised or other issues that may occur to him. One or two points of general interest are, however, suggested for consideration.

(i) The post-bridge action of the river on the south bank need not necessarily remain the same as under existing conditions before bridging. It has been observed by the Poona Station⁽¹⁾ that the meander belt above a rigid structure or a fixed exit, e.g. a bridge, weir or inerodible gorge, is generally twice the meander belt elsewhere. In the case of Patna or Mokameh sites, even before bridging, the conditions are somewhat analogous to those of a fixed exit and such large changes in meander characteristics may not occur consequent upon bridging; at the same time as borings indicate that stratification of layers, i.e. mixed sand and clay layers, exist, severer action on the south bank due to additional constraint imposed on the north bank must be anticipated. Whether and when protection will be required for this bank would only be clear from model tests; but it must *not* be assumed that the edge of a Khadir is necessarily immune from erosion even under altered conditions following the construction of the bridge.

(ii) Though under favourable river conditions, guide bunds have proved sufficient and satisfactory as river training measures, and they are certainly an improvement over the earlier system of training the river by a number of side groynes parallel to the river, it will be conceded by even the strongest supporter of the guide bund system that it is not always an unmixed blessing and, often, it has been known to give significant trouble and to cost inordinately in maintenance. Though not a critic of guide bunds the Director is equally convinced that other river training measures — for example, a long 'repelling' spur at a suitable position upstream on the concave side, or an 'attracting' spur on the opposite shoaling bank, or an 'attracting' island or islands—may prove better alternatives, both in first cost and in all time maintenance, for centering the river and leading it smoothly through the bridge, under conditions favourable to such measures. In important or difficult cases the optimum training works, their best position, orientation and design, can only be conclusively settled by model tests.

(1) Annual Report (Tech) of I. W. E. S., Poona for 1943, p. 13.

4. In the *final* Report, in "Chapter X, Summary and Conclusions", Mr. Fenton's remarks on the points raised by the Director—vide para 3, *supra*—are set out. With a view to present a comprehensive picture it is proposed to give, in the following paragraphs, Mr. Fenton's remarks (in his own words) and the Director's rejoinders thereto.

4.1 "The writer feels that the Director does not wish to convey this impression, that the technique that has been evolved since the days of Spring's paper on River Control is not capable of being applied without the assistance of model experiments, to produce decisions on which a high degree of dependence may be placed. Model experiments, valuable though they are, have limitations; they can indicate trends towards the establishment of deep channels and accretions but are unable to predict with any degree of certainty the worthiness of indurated deposits and the probable rate of their erosion. The quantitative predictions of models are greatly limited and the Director admits that year to year changes cannot be wholly forecast."

Director's reply—

Springs' paper was written 45 years back—in the days when the art of model experiments was non-existent. As a result of over twenty years' work in model experimentation, it can be authoritatively said that so much depends upon unforeseeable movements of the river in the post-Bridge conditions, that it would be unwise to embark to-day on such a major undertaking without taking advantage of model experiments, which, for a nominal cost, enables studies of river behaviour to be made under all possible conditions and which yields dependable results. Model technique under expert interpretation have been evolved at the Station fully to meet model limitations, such as the difficulty of wholly reproducing year to year conditions due to the rate of shoaling being different from the rate of scour, or that of reproducing bank erosion. It was made clear in the Director's first note—see para 3.3.4—that "nevertheless, there would be sufficient indications from the model with respect to river behaviour under the worst hypothesis to give far greater confidence in the final selection of the site."

For this reason, the Director recommended, and still recommends, that, though the present conditions at Mokameh are more favourable than at Patna and the construction of a bridge should be expedited as much as possible to avail of the existing favourable conditions, *model investigations must precede the construction* to test the feasibility or otherwise of a bridge at Patna at unprohibitive additional expense. It is interesting to observe that the opinions on the site issue of Mr. G. Lacey and Messrs. Randel, Palmer and Tritton, Consulting Engineers, independently sought and obtained, generally confirm those of the Director : —

"The Patna site has few natural advantages but is practicable and could be rendered reasonably secure." (Mr. G. Lacey).

We do not consider that the project site at Digha (Patna) can be ruled out as being an unfeasible engineering undertaking. . . " (Messrs. Randel, Palmer and Tritton)

Mr. Fenton has suggested as a compromise, that the bridge be constructed forthwith at Mokameh and with industrial development there would in the future be justification and means to again consider a bridge at Patna. An additional major bridge, only 60 miles upstream of an existing bridge, would require more than this justification in the foreseeable future and the Director advises that the matter should be decided *now* on less speculative considerations.

4.2 Mr. Fenton : " The Director states that the suggested future bridge site (at Patna) about half a mile upstream of the project alignment is to be preferred to that of the project alignment itself. This is, in some measure, also the writer's view as expressed in Chapter IV, paras 14 and 15. The benefits

of this site cannot, however, be secured at the present time. From the right bank Khadir the channel is at present over 10,000 ft. wide and would require 24 or 25 spans as compared with the 22 spans proposed in the Project. The shoaling at the right bank does not to-day, and will not for sometime, provide conditions suitable for the construction of an artificial right guide bank and the benefit of the symmetrical guide bank cannot be secured at the present time."

Director's reply—

Mr. Fenton's argument about the longer bridge are based on the presumption that it is necessary to span the river from khadir to khadir. As is well-known, the minimum necessary length of bridge, equal to the natural stable waterway, depends upon the maximum discharge—see para 3.4.3 above. From Mr. Lacey's paper in the Institute of Civil Engineers Vol. No. 229, the illuminating case of the Alexandra Bridge on the Chenab may be cited. "When the bridge was constructed in 1876, there was given no less than 64 spans of 132 ft. clear, the old bridging principal being that of bridging very nearly everything that presented the appearance of a river bed. Some of the spans were reduced in 1881, 1888 and 1890 in an attempt to confine the river to one stable waterway. In 1917, it was realised that this river when in flood had never made effective use of more than 17 spans." It might even be possible to reduce further the waterway at Patna from model tests on account of the better conditions obtainable at this alternative site.

The Director did not suggest the construction of an artificial right guide bank for merely securing the benefits of symmetrical guide banks. It has been clearly stated (see para 3.1.3 iv) that it is only this alternative site (i. e. the one $\frac{1}{2}$ mile above the Project site) which will permit symmetrical as well as parallel guide banks, the south (or right) natural training bund being the edge of the Khadir and the north guide bund being constructed where river conditions permit", i.e. leaving sufficient minimum stable waterway from the right bank as obtained from model experiments.

4.3 Mr. Fenton : "The Director objects to the groyne proposed at the north approach embankment at the Patna site. He considers that no groyne is likely to be required in addition to the north guide bank which at most may have to be somewhat longer. Whether a groyne be required or not can only be established, he states, by model experiments and he points out how groynes, by attracting flow and pinning it to their heads, are apt to create unpredictable and even dangerous conditions.

The writer does not consider these arguments valid in the present instance. Heads of guide bunds are equally guilty of tendencies towards pinning flow once the river channel is in contact with them and the head of the north guide bund must come under attack before the groyne can function. The river, in the writer's opinion, would easily overcome the "attraction" of the groyne once it showed a tendency to recede from its embayment particularly, as the Director points out, because a limiting curvature of 1 mile radius has been assumed for the embaying current rather than the more probable curvature of 2 miles radius. Once the bridge is constructed the presence of the bluff at Kasmar is calculated to offer considerable protection to the area between it and the north approach embankment, and it may well be proved by model experiments, as the Director forecasts, that the proposed groyne would rarely be subjected to any serious attack. As however its cost is small when compared with any reasonable lengthening of the north guide bund, its provision may be regarded as an insurance against trouble, provided the designed length of the guide bund provides, as the writer considers it does, an acceptable degree of obliquity of the bed and surface flow lines through the bridge."

Director's reply—

The possibility of the groyne attracting the flow with a sharper embayment than the normal limiting curvature of one mile radius, on account of the

triple combination of the Kasmar bluff, the groyne head and the head of the north guide bank, must not be ignored. Any heavy attack on, and consequential breach of, the groyne might then result in an isolation of the head from the shank, accompanied by more intense attraction owing to the head acting as an 'attracting' island with parallel flow in between. The potential danger to the north approach under such conditions is obvious. With even an equally severe attack at the guide - bank head, similar conditions are not expected to be obtained.

In this connection, the case of the Hardinge Bridge is to the point. Earlier experiments carried out at Poona had clearly shown that the Damukdia Guide Bank (analogous to the proposed groyne at Patna), which was constructed upstream of the Hardinge Bridge on the right bank to guide the flow through the middle of the bridge, was responsible for the failure of the right guide bank in 1933. This turned the high velocity flow into violent eddies and surges to attack the head of the right guide bank further downstream, intensifying the diving flow and leading to the slip of the right guide bank. If the groyne as proposed by Mr. Fenton were once to capture the flow, it may set up similar disruptive forces.

4.4 Mr. Fenton : "With regard to the Mokameh site, the Director agrees that the Project alignment is probably the best in the neighbourhood but goes on to say that model experiments alone could decide this as they may show a site about $\frac{1}{2}$ mile further upstream to be preferable. The writer doubts this very much. Shifting the bridge further upstream would deprive the north guide bund of a degree of protection afforded by the gorge and would be calculated to increase the severity of potential attacks at its head. It is also not clear why the Director should express concern at the possibility, mentioned by the writer in chapter IV, para 27, of a limiting embayment along the Baya Nadi, and its effect on the head of the guide bund. If this should come about, and Chapter IV gives reasons why this may be a remote contingency, the head of the guide bund would receive no greater punishment than that for which it is designed. The worst conditions under an extreme embayment are indicated in Plate 4 where the limiting curvature of the embaying current is shown as 1 mile radius. With the more probable curvature of 2 miles radius the degree of potential attack at the head of the guide bund would be less than indicated, more especially because of the clay deposits on the north bank of the Baya Nadi which, it appears, extend inside the khadir to within a mile or so of the head of the guide bund."

Director's reply—

All that the Director has desired to convey was that the experience at the Station shows that model experiments sometimes indicate modifications in the site, alignment and the orientations of the bridge and the guide bank, fixed as the most obvious or correct from the plan or the site.

4.5 Mr. Fenton : "The Director agrees with the project proposals in which the guide bund is made parallel to the general direction of the khadir but stresses the desirability of symmetrical guide bunds. Where the inerodible bank of the khadir is utilised as a natural guide bund, symmetry cannot be achieved, as the incidence and obliquity of attack, as discussed in Chapter V para 39, must necessarily differ greatly at the two flanks of the bridge."

Director's reply—

An artificial left guide bund, *parallel and symmetrical* to the natural right bank, appears possible at the Mokameh site. If, having due regard to the incidence and obliquity of attack obtaining at the site, under the worst flow conditions, symmetrical guide banks are found suitable on the model, they are to be preferred.

4.6 Mr. Fenton : " The river levels reached during the 1946 monsoon were higher than normal and the writer was able to conduct discharge measurements at both the Patna and Mokameh sites at almost the peak of the flood. It is improbable that conditions as favourable as those met during 1946 will recur for some time and even if they do, any change in the extrapolated peak discharge is unlikely to be large enough to materially affect the design.

Twice the width of the wells has been deducted from the distance between well centres to give the effective waterway per span (Chapter V para .26). This is in accordance with railway practice. The Director feels that this is excessive but agrees that some, though not perhaps full, compensation is provided by arriving at the waterway by the application of Lacey's unmodified formula and not allowing any divergence from it because of a shallower natural section."

Director's reply—

For reasons already given and because of the importance of avoiding an underestimate of the maximum discharge at Patna and Mokameh, the Director still feels that there is need for measurement of higher discharges and a closer estimate. In view of the uncertain quantities of spills and percolation between Patna and Makameh and of the contribution by the Gandak, as a measure of safety, it would, however, be wise to assume a peak discharge of 2.75 lacs cusecs for the Mokameh site.

In addition to what the Director has stated before, it must be observed that the Railway practice to arrive at the effective waterway per span by deducting twice the width of the wells from the distance between well centres must presume a standard design of piers and cut-and ease-waters; in other words, the afflux will vary with different designs of piers and cut-and ease-waters. Actually, the flow is rarely uniform in all spans: for example, at the Hardinge Bridge, 90% of the flow was through spans 1 to 6 from the right bank. The final word on the adequacy or otherwise of the effective waterway should, therefore, be left to the model.

4.7 Mr. Fenton : " The figures of scour given by the Director would lead to a reduction of armour at the shanks and an increase at the heads. For the length of guide bunds recommended in the Report these modifications would result in an increase of Rs. 11 lakhs in the estimate for the Patna site and Rs. 8 lakhs for the Mokameh site. The Director's figures give more consistent protection than by the adoption of Lacey's unmodified pier scour but the writer is not entirely happy at the proposed reduction in the armour at the shanks. The damage sustained by the right guide bank of the Hardinge Bridge in 1933 was at the shank and not the head. The cause was ascribed by some to a weak patch in the armour at or near water level and by others to a slipping of the launched apron. While accepting the figures given for the heads, the writer would prefer the following scour values for the shanks.

	<i>Upstream</i>	<i>Near abutments:</i>
Patna site	115'	130'
Mokameh site	110'	125'

Director's reply—

Mr. Fenton's feeling about the depths of scour at the shanks being rather low are based on misapprehension in respect of the cause of the damage to the right guide bank of the Hardings Bridge in 1933. This damage was mainly due to the Damukdia guide bank and *not* due to insufficiency of the armour along the shanks. When the river is interfered with to the extent that the Damukdia Guide bank caused, no reasonable amount of stone armouring

can give adequate protection to the guide bank against a frontal or direct attack at the shank. A study of past cases and experience of model investigations does not, however, justify the armouring proposed for the shanks of Patna and Mokameh bridges; but this can be verified by model experiments.

4.8 Mr. Fenton : " Although Gales recommends a head radius of 3 degrees (1910 feet) for class C rivers, a radius of 5 degrees (1146 feet) has been adopted. This curvature was chosen not because, as the Director surmises, it is negotiable by metre gauge stock (this curvature is also suitable for broad gauge) but because, in the writer's opinion, Gales' curve is too easy. Spring considers that a head radius of 600 to 800 feet would be sufficient, with a water velocity of 8 to 10 feet per second, to encourage the stream to cling to the curve head without forming swirls. The maximum mid-stream surface velocity does not exceed 13 feet per second and the maximum velocity adjacent to the head is unlikely to exceed 9 feet per second. An easier curve for the head of the guide bund would, of course, reduce the depth of scour and lessen the possibility of eddies being formed; but if Gales' curvature of 3 degrees were adopted the very considerable cost of the head would be increased by two-thirds. It would be preferable, in the writer's opinion, to adopt the sharper curvature and add to the armour; alternatively, as a compromise, a curvature of 4 degrees, could be assumed."

Director's reply—

Generally speaking, water finds it easier, particularly at the higher discharges, to cling to an easier curve and the easier curves lead to less depth of scour and reduction of eddies. Whether the 3 degree curve or a 4 degree curve is *optimum* can, however, best be determined on the model.

4.9. Mr. Fenton : " The Director is averse to the twin octagonal design of reinforced concrete piers included in the design and would prefer the pier design to follow that of the well and provided with semi-circular ends. The twin octagons of the pier are connected by a thick rib which extends above highest flood level and this dumb-bell shape is calculated to offer small interference with surface flow lines, and then only at high flood. The Director supports the writer's views in his preference for semi-circular ended wells but if the piers follow the same design, the relief of base pressure which results from the special design of piers, will be lost."

Director's reply—

All that the Director desires is the adoption of cut- and ease-waters of the semi-circular type instead of the octagonal, keeping the rest of the design in R. C. C. as suggested in the Project. This should involve no major structural changes; and, perhaps, relief of base pressure should also not be affected.

4.10. Mr. Fenton : "The adoption of an acceleration of at least $g/5$ (against $g/8$) for the computation of seismic forces for the design of the piers, is likely to result in an unduly expensive design. This matter may be finally decided when the detailing of the structure is undertaken."

In the 1934 Bihar earth-quake the maximum intensity was 10.7 ft./sec.^2 , i. e. $g/3$. It was, therefore, suggested that it is desirable to make sufficient provision to cope with horizontal forces at least equal to $g/5$, i. e. about 6.5 ft./sec.^2 , the lowest intensity in Zone IX on the curve Mercalli Scale.

4.11 Mr. Fenton : " In the concluding paragraph of his note the Director calls attention to alternative protective measures other than the guide bund system on which this Report is based. These alternatives—'attrac-

ting' and 'repelling' spurs and 'attracting' islands—have not so far to the writer's knowledge been employed on any major works and much experimental work and practical experience are needed before such measures could be recommended, with any degree of confidence. There must be many, however, like the writer, who would always prefer their protective works as inner defences near home, rather than as distant bastions protecting a weak and otherwise defenceless centre. The problem besides being technical, is largely economic, the outer defences would need maintenance in the same manner as training bunds and it would have to be shown that such an arrangement is not only of greater technical worthiness but of greater resulting economy."

Director's reply—

The alternative training measures suggested by the Director are just as much 'inner defences' as guide banks; and they have to be given equal consideration as the guide banks during the model tests. The fact that they have not been tried before to the same extent as guide banks is not sufficient to exclude them out of consideration, particularly in view of the serious troubles experienced from misdirected guide banks, in the past, in many instances.

5. Further examination of the data as well as remarks by Mr. Lacey will be undertaken at the time of the Model experiments.

G-1-3 Indus water to Cutch State—A note on the possibility of bringing—

Introduction :—The Consulting Engineer to the Govt. of India, Waterways, Irrigation and Navigation, forwarded to the Director a note by Brigadier Hawes on "the possibilities of bringing Indus water to Cutch State" for the Director's comments.

(2) Brigadier Hawes' Proposals :—

Brigadier Hawes first considered the feasibility of bringing the Indus water to Cutch *via* the Nara canal ex the Sukkur Barrage and the Puran Dhoro but condemned it on the grounds of prohibitive cost.

The suggestion sponsored by him was to take off a *separate Feeder* from the proposed Lower Sind Barrage, (then contemplated to be constructed near Hajipur) about 75 miles from the nearest point in Cutch Territory, alternatively for taking water to (a) Village Luna to irrigate lands near Banni or (b) the border of the Rann at mile 75 to commence irrigation for the lands thereabout. He, however, himself considered the proposal (a) to be a rather hopeless task as the Feeder from Cutch boundary to village Luna would have to run in very heavy banking and thus be expensive and constantly liable to serious breaches. He favoured the latter proposal (b) as it would enable dropping the F. S. L. from R. L. 29.80 to R. L. 16.0 through Falls at two places resulting in the reduction of the cost of the scheme.

Fig. 8
G-1-3.

The costs of these two alternative schemes as worked out by him were Rs. 1,64,20,000/- and Rs. 70,70,000/- respectively.

3. Comments by the Director on Brigadier Hawes' Proposals :—

The Director agreed with Brigadier Hawes' first conclusion that the cost of bringing Indus water to Cutch *via* the Nara ex the Sukkur Barrage and the Puran Dhoro would be prohibitive, but entirely disagreed with the proposal of an idle Feeder from the Lower Sind Barrage for reasons set out below :—

(a) Brig. Hawes' note was written in July 1943, when the Lower Sind Barrage was proposed to be sited at Hajipur, about 75 miles from the nearest point in Cutch Territory. It has since been finally decided to locate the Barrage near Kotri, some 23 miles further upstream on the river. This will make the nearest point in Cutch about 100 miles away from the Barrage in a proposal similar to that of Brig. Hawes.

(b) The costs worked out by Brig. Hawes were based on very rough figures, without surveys, quantities or proper estimates. Actually the figures used by Brig. Hawes are about only *one-third* the present estimates.

(c) The estimates would also be out as due allowance would have to be made for 33 percent additional length of the Feeder from Kotri.

(d) Brig. Hawes' proposals are, moreover, impracticable with a barrage at Kotri, because it would not be possible to give the required Full Supply Level at Luna in Cutch with a separate Feeder.

(e) Such an idle Feeder running in very heavy bank (as already anticipated by Brig. Hawes himself) would be very expensive, liable to breach, difficult and costly to maintain. The transit losses would be heavier than expected by Brig. Hawes, leading to waste of very valuable water and the risk of possible water-logging. Lining of this Feeder running in heavy bank (20 to 30 ft. above ground) has been considered impracticable by Brig. Hawes; it would certainly be a costly and hazardous undertaking. Without lining, however, there would be considerable risk of water-logging of the Sind areas through which the Feeder would pass, resulting in complications, disputes and claims for compensation by Sind, which must all be taken into account in any reasonable financial forecast. Sind would, moreover, strongly oppose any such scheme at any price.

(f) The meagre balance river supplies available (except in July and August), particularly the *essential* supplies in early and late Kharif months required for sowing and maturing, have been already fully appropriated for the new Sind and Punjab Canals. The burden of shortages would, therefore, fall on the new-comers, particularly as Cutch would find it hard to establish herself as a "*riparian*" state with a valid *claim* on the waters of the river. These shortages would be very heavy in five years out of ten, especially in May and June, and, on a conservative estimate Sind should count on 20 percent remissions, on an average, on account of shortages of supply, excessive losses, breaches etc. This makes the poor financial aspects of Brig. Hawes' schemes most gloomy.

(g) It is also doubtful whether Sind and Punjab would agree to participation by Cutch except by legal agreement that their canals would have *complete priority* in all circumstances, i. e. the Cutch Feeder would only get supplies *when and if* river supplies would be in excess of the *full* requirements of Sind and Punjab and that Sind and Punjab would not be called upon to share supplies with Cutch at such times. Under such conditions participation would be of little value to Cutch.

(4) *Proposals by the Director:—*

4.1 In the Director's opinion, the only scheme worthy of consideration would be a *separate branch* for Cutch up to Luna from the *tail* of the Fuleli Canal System, preferably with the main *lined* Fuleli lined up to its tail.

4.2 Even with the *unlined* Fuleli Canal, in addition to lower first cost and a superior financial forecast, there would be the following advantages:—

(a) Brig. Hawes' proposal of a direct Cutch Feeder from the Barrage would not give flow irrigation when the Barrage would be at Kotri while this proposal will.

(b) The Director's proposal would not entail high transit losses otherwise necessary in a highly banked separate Feeder.

(c) The duplication of canals, cross drainage works, cross regulators, bridges, protective works, service roads would be avoided.

(d) It would avoid the undesirable necessity of truncating the commands of the Sind channels and/or providing siphons.

The cost of this proposal of a branch from the tail of the *unlined* Fuleli Canal has worked out to Rs. 1.5 crores.

4.3 As the first proposal would require raising of the water level in the Fuleli by 2 ft. (which would be opposed by Sind), though more expensive the alternative proposal of a *lined* Fuleli Canal was considered more attractive. The merits claimed for it were as under:—

(a) The proposal would benefit Sind as their lands would not be damaged by water logging, while Cutch would not be required to pay heavy compensation which might otherwise be claimed by Sind.

(b) Transit losses and Pocket withdrawals from the Kotri Barrage would be reduced by 1,000 cusecs as a result of the lined Feeder and thus sand exclusion difficulties would not be enhanced.

(c) It would enable flattening of gradients to the full extent necessary to give the Cutch lands the water levels required by them.

(d) This would be the only method by which it would be possible to give supplies saved by lining to Cutch during the crucial months when river supplies are essential but would otherwise be short of Sind requirements.

The total cost of this alternative proposal has worked out to Rs. 4.5 crores.

4.4 The Director concluded that it would, in any case, be an undertaking financially very difficult for Cutch; but if the unassessable, indirect returns of giving irrigation supplies to Cutch would be of sufficient value to an otherwise arid and parched area, it would be a project worthy of Cutch's consideration.


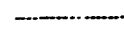
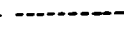
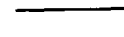
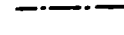




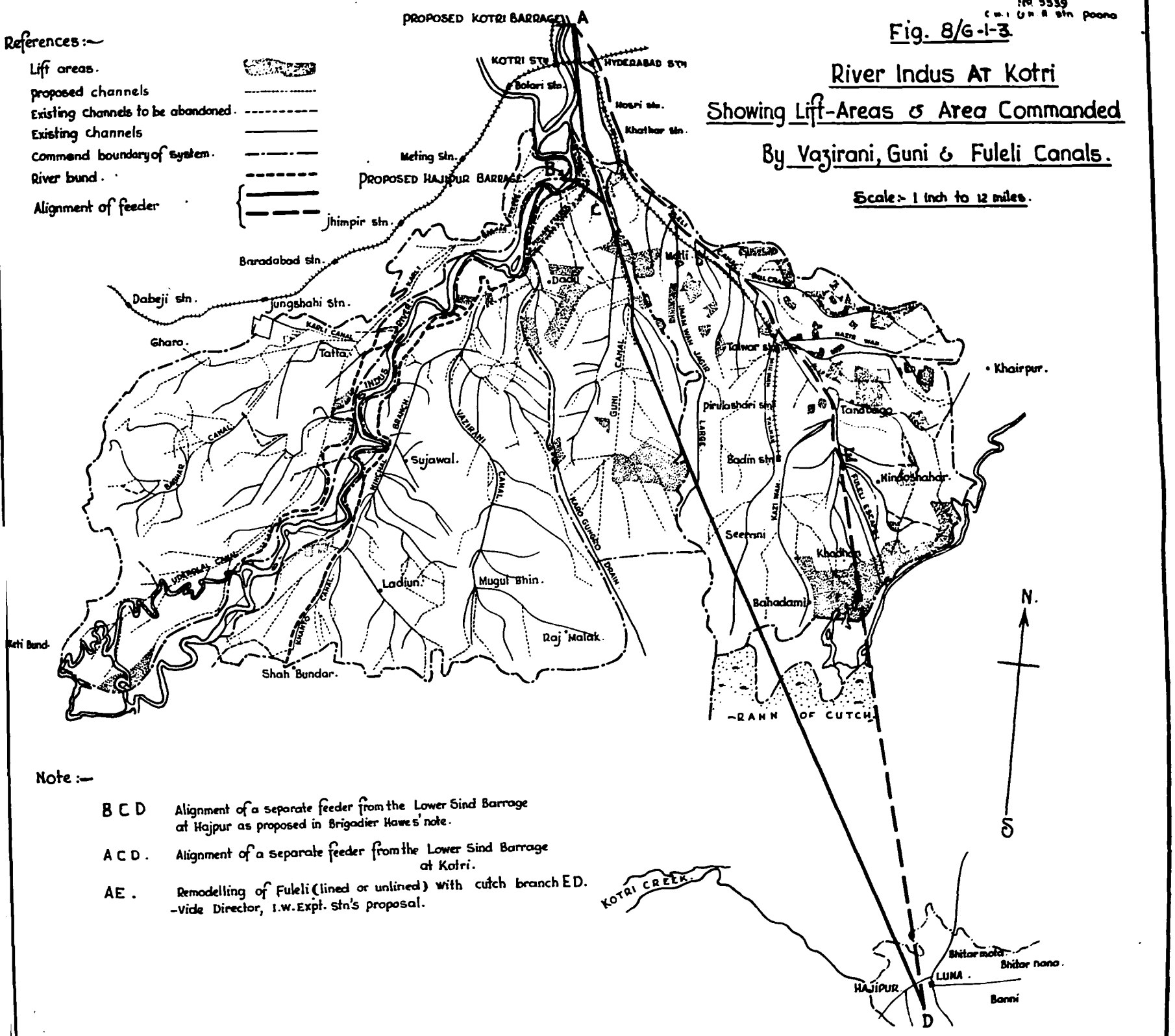
Fig. 8/G-1-3

River Indus AT Kotri
Showing Lift-Areas & Area Commanded
By Vazirani, Guni & Fuleli Canals.

Scale - 1 inch to 12 miles.

References :-

- Lift areas. 
- proposed channels 
- Existing channels to be abandoned. 
- Existing channels 
- Command boundary of system. 
- River bund. 
- Alignment of feeder 



Note :-

- B C D Alignment of a separate feeder from the Lower Sind Barrage at Hajipur as proposed in Brigadier Howe's note.
- A C D. Alignment of a separate feeder from the Lower Sind Barrage at Kotri.
- AE. Remodelling of Fuleli (lined or unlined) with catch branch ED. -vide Director, I.W. Expt. Stn's proposal.

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