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CENTRAL WATER & POWER COMMISSION

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Director :

D. V. JOGLEKAR, B.E., M.I.E. (IND.)

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	2	Fortnal	Fort Wall
	5	Plow	Flow
	6	Refergnces	References
	7a	Tor	Top
	7a	Pfching	Pitching
7	1	Canai	Canal
	1	Bachwara	Bichwara
	2	Flcw	Flow
	2	Gandar	Gandak
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	4	GaiDak	Gandak
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	18	Hannel	Channel
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	7	1/40 Scale	1/40 Scale
11	3	Chamble	Chambal
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	3	Curve	Curve
	3	Rctah	Kotah
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	3	Disharge	Discharge
	10	Sope	Slope
12	1	Gauge wall	Gauge Well
	4	Measurment	Measurement
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	5	Cofeer Dam	Coffer Dam
	5	Flood	Flood
14	7	Opfrating	Operating
	9	Grest	Crest

(Errata to Figures)—contd.

Item No.	Figure No.	For	Read	
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	1	Pressube	Pressure	
	2	Wvth	With	
19	1	Scale 1"=20	Scale 1"=20'	
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	8	Discharse	Discharge	
	14	Displaceme	Displacement	
	15	Minote	Minute	
	15	Interval	Interval	
	17	Indivivual	Individual	
	17	Gorresponding	Corresponding	
	18	Viabration	Vibration	
	18	Parellal	Parallel	
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FOREWORD

The scope of the work carried out by the Research Station has materially increased during the year and with additional equipment for which orders have been placed, a further substantial increase is anticipated.

New area has been acquired for expanding the activities in respect of River and Canal Hydraulics and Navigation. The work on leading and return channels, supply chambers, etc., is in progress in this area. In the new set up, it is proposed to circulate water up to a maximum of 100 cusecs during dry months of March to June when the water level in the lake gets low. For this purpose a high level tank, a low level tank and a pump house have been constructed. Four units of pumps out of five have been received and are being fitted in the pump house between the low level tank and the high level tank. The rating tank 500 ft. long, 12 ft. wide and 7 ft. deep has been completed and an electrically operated carriage will be obtained shortly. It will then be possible to calibrate current meters, pilot tubes and also carry out tests with models of small ships.

It is proposed to set up a fully equipped concrete laboratory, also a large Froude (Navigation) tank 850 ft. long, 30 ft. wide and 15 ft. deep. These proposals have been framed and are being forwarded to Government for approval. Modern improvements in the designs of ships have been made possible through the experimental technique evolved in testing models of ships in Froude tanks in various countries of the world. It is time that India makes a determined start in this direction.

Two Experts from UNESCO have been associated with the work at this Station, namely, Dr. R. V. Baud for Photoelasticity, and Mr. A. F. Mogensen for Navigation, harbour problems, wave studies, break-water designs, etc. These experts are also imparting training to members of staff of the Research Station. Dr. Baud has assisted in the fitting up of the photoelastic laboratory. A Case Polariscopes has now been obtained. This will enable us to carry out tests with large-scale models where body load is the main consideration for earth dams. Also, Seismic Refraction and Resistivity Equipments have been obtained during the year. Both these instruments will be useful for carrying out foundation investigation by geophysical methods. Similarly, Oscillographs have been obtained for recording wave characteristics in harbour models. It is now proposed to obtain a high-speed camera (giving pictures up to 14,000 frames per second), which will be very useful in high-speed variation phenomena such as cavitation, air entrainment, etc. In certain problems like fluctuating pressures in models of siphons and deep-seated sluices, it is very essential to record instantaneous pressures. For this, electrical cells are very suitable. Attempts are being made to obtain these from abroad and also getting them made locally.

As usual the River and Canal Hydraulics and Navigation Sections were the most busy Sections. The problem of training the Narbada River at Broach (to check the deep scour near the pier of the Railway Bridge) was successfully solved and the problem of the Watrak river at Kaira (to arrest

erosion of the left bank upstream of Kaira town and the right bank and right embankment of the road bridge downstream of the town) is under investigation.

Experiments to determine the suitability or otherwise of the different sites for road-cum-rail bridge across Ganga at Mokameh and at Patna were completed. In this connection, it is recommended that the model technique followed by this Research Station, namely, that training measures should be tested not only under existing conditions but also under conditions of the worst possible attack expected from the river, should form an essential part of any further experiments of this nature.

Important model experiments were conducted to test the designs of retaining walls of Hirakud Dam Left Channel Spillway, and to study the alternative alignments of Kakrapar weir on River Tapi. A series of tests were also made for the discharge characteristics and dissipation of energy of:—

Hirakud Dam conventional and upturned deep-seated sluices,

Subsidiary Dam Spillway of Hirakud Dam Project, and

Proposed Kotah Barrage.

These tests were carried out by Dr. S. K. Roy, Chief Research Officer, assisted by Shri P. M. Damle, Research Assistant for which they deserve credit.

Useful results were obtained for increasing the co-efficient of discharge of Weir Regulators of Ghataprabha Left Bank Canal, by means of providing bellmouth entry and smooth expansion downstream. The co-efficient was thus increased from 0.64 to 1.3.

An economic design for Falls for Hirakud Project was evolved for different discharges and different drops both for Flumed and Unflumed conditions. Further basic research on the design of Falls is in hand.

On the Navigation side, experiments were continued to evolve training measures for the improvement of navigability in some of the difficult reaches of the Hooghly like Fulta, James and Mary, Sankrail and Howrah.

The problem of Head Water Supply of the Hooghly and its effect on Navigation are being studied closely and all relevant data is being analysed. Final proposals were made to arrest silting in the Harbour Channel at Cochin by means of blocking the Mattancherry Channel. Experiments to reduce the "Range" in Madras Harbour were completed in a model with vertical exaggeration of 20. Experiments in a geometrically-similar model will now be taken in hand.

The Station gave expert advice for the improvement of water supply for Tata Iron and Steel Works, Jamshedpur and for Nagpur town.

Geophysical investigations for Foundations were made at alternative dam sites at Ukai on River Tapi—Kakrapar Project. Advantage was taken during the working of Siphon No. 7 and the undersluices at Hirebhasgar during the last monsoon season to carry out vibration studies.

In the Physics Section, the study of stress analysis of Engineering Structures by means of photoelastic method was the main feature. The assistance of Dr. Baud, the UNESCO Expert in this subject is acknowledged.

The most notable and interesting investigation carried out by the Physics Section is in connection with vibration characteristics of the Hirebhasgar Dam due to working of siphon No. 7 under cavitation conditions and due to working of undersluices. This is probably the first kind of investigation of this type carried out in India. Special mention is made of Dr. Gurdas Ram, Chief Research Officer and Shri S. K. Guha, Assistant Research Officer (Physics) for this original work.

This Soil Section was busy with the study of Shear Characteristics of Soils by triaxial compression tests, and with study of physical properties of black cotton soil. Soil analyses were also continued for exploration of suitable materials for the earth dam across the River Tapi.

Peak runoff in the Mahanadi Basin was analysed by the Unit-Hydrograph method in the Statistical Section. Statistical analysis was also made of the tidal data in connection with Head Water Supply of the river Hooghly.

Of late there has been a very great demand for carrying out model experiments at Central Water and Power Research Station. With the reorganisation of the Station into various sections which have now come into being, it has been possible to carry out intensive basic as well as applied research with good results. The programme for the next year will be still more intensive and of special importance.

A high standard of research has been maintained at the Research Station. Shri D. V. Joglekar, Director, Dr. R. C. Hoon, Deputy Director and other officers and members of the staff have contributed to the successful operation of the Section and deserve full credit for the efficient operation of the Research Station for the year.

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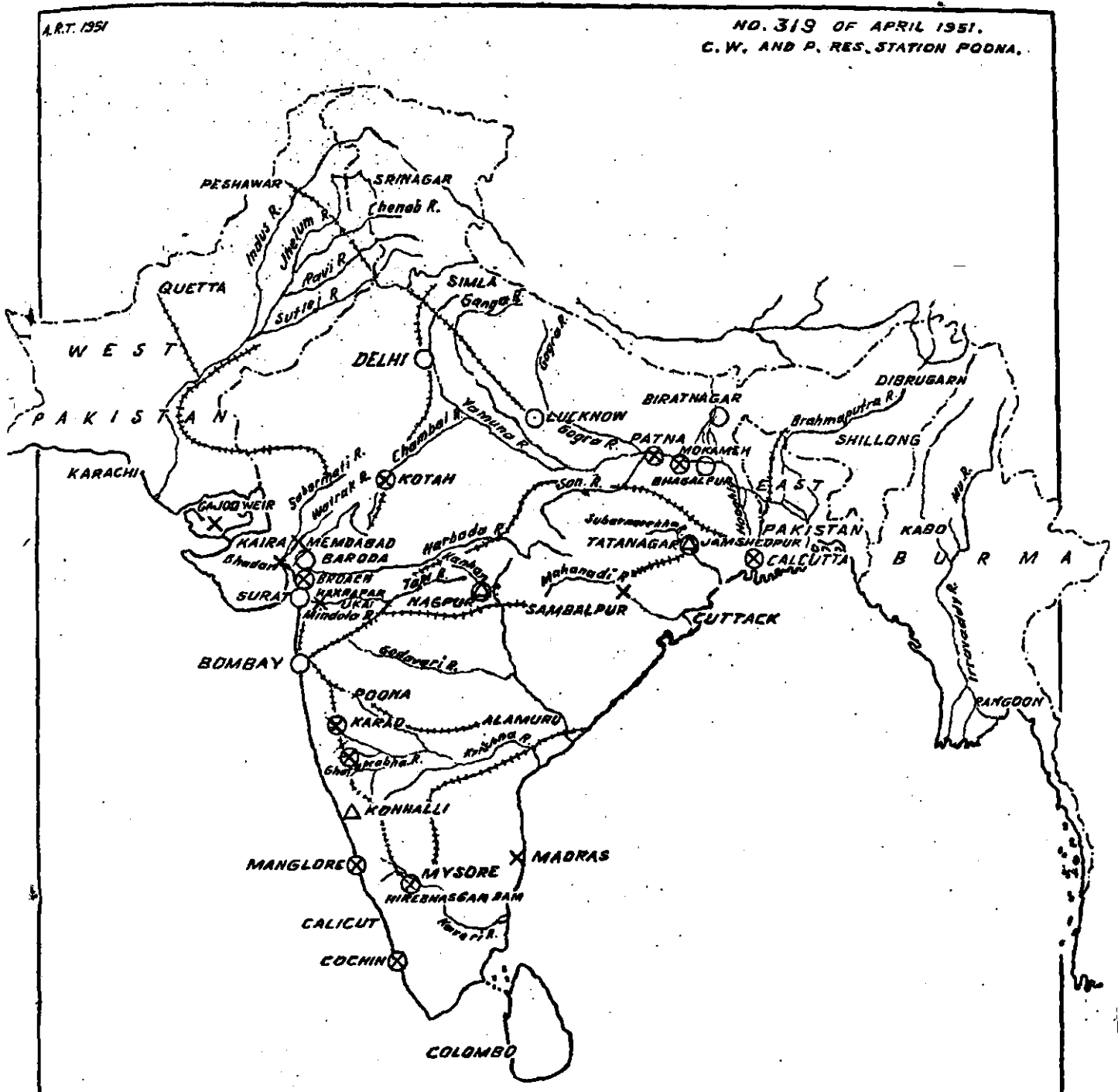
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FIG. 1.
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A D J A C E N T
C O U N T R I E S
1951

SCALE: 1" = 320 MILES

COMP. BY: *M. Singh*
TRACED BY: DESHPANDE & PARKHI.
9 APRIL 1952

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INTRODUCTION

1. With the amalgamation of Central Waterpower, Irrigation and Navigation Commission and the Central Electricity Commission, the name of this station was also changed from Central Waterpower, Irrigation and Navigation Research Station to Central Water and Power Research Station. During the year under report further expansion of the Station under the Reorganization Scheme was rather slow, both due to non-availability of suitable personnel and financial stringency.

Among the gazetted staff no more recruitment could be made except that of Shri S. S. Awatramani and Shri S. L. Ranganathan who joined as A. R. O. (Administration). The former however resigned on 10th September 1951. Shri B. Pant, Assistant Research Officer (Physics) was awarded scholarship under the Technical Co-operation Scheme for South East Asia, for higher study particularly in photo-elasticity in Canada and he left the Station on 4th August 1951.

Eleven new Research Assistants were recruited for various sections since November 1950. However 4 of them (3 engineers and one scientist) resigned later. It has been seen that the engineering graduates with first class degrees do not stick as Research Assistants due to their lower initial pay. In order to make the posts attractive, steps have been taken and Government has been moved to consider the question of raising their initial pay.

2. At Khadakwasla Hydraulic Field Station, the workshop was fitted up and it started functioning. During dry months of March to June when the water level of the lake (from which supply is obtained by gravity at present) falls down, and gravity supply is not obtainable, the models in the new area have to be worked with circulated supply. For this purpose High Level Tank, Low Level Tank and Pump House have been constructed; and the Leading and Return channels are under construction. Four units of pumps have been received and are being fitted up between the Low Level Tank and the High Level Tank.

The Rating Tank has also been constructed. Unfortunately the building estimate have still not been sanctioned and the work of constructing the field office at Khadakwasla has been postponed for another year. This work is to be done by the Central P. W. D.

Agreement has been made to get the electric energy for the Station through the National Defence Academy, and this work will start next year.

It has been finally decided now, not to shift the Soil Section Laboratory from the Station, which was originally proposed to be sent to Kakrapar. It is being fitted up at present in the office.

3. Mr. A. F. Mogensen, UNESCO Adviser in Navigation, joined the Station on 2nd December 1951 for a period of 8 months; and Dr. R. V. Baud, UNESCO Adviser in Photo-elasticity, also joined at the close of January 1952.

Their expert advice is being usefully utilised for study in their respective subjects, and they are also training the members of the staff.

4. The following important equipments were received for the Station during the year:—

- (i) Case Polariscopes.
- (ii) Seismic Refraction Equipment
- (iii) Resistivity (Geophysical) Equipment.
- (iv) Aneamograph.
- (v) Spekker Absorptionmeter.
- (vi) Oscillograph.

5. As usual the River and Canal Hydraulics and Navigation Sections were the most busy Sections. The problem of training the Narmada river at Broach (to check the deep scour near the pier of the Railway Bridge) was successfully solved and the problem of the Watrak river at Kaira (to arrest erosion of the left bank u/s of Kaira town and the right bank and right embankment of the road bridge d/s of the town) is under investigation.

Experiments to determine the suitability or otherwise of the different sites for road-cum-rail bridge across Ganga at Mokameh and at Patna were completed.

Important model experiments were conducted to test the designs of retaining walls of Hirakud Dam Left Channel Spillway, and to test the alternative alignments of Kakrapar Weir on river Tapi. A series of tests were also made for the discharge characteristics and dissipation of energy of—

- (i) Hirakud Dam conventional and upturned deep-seated sluices,
- (ii) Subsidiary Dam Spillway of Hirakud Dam Project, and
- (iii) Proposed Kotah Barrage.

Further studies were made of the performance of Volute Siphon No. 7 at Hirebhasgar. Very useful results were obtained for increasing the coefficient of discharge of Weir Regulators of Ghataprabha Left Bank Canal, by means of providing bell-mouth entry and smooth expansion downstream. The coefficient was thus increased from 0.64 to 1.3.

An economic design for Falls for Hirakud Project was evolved for different discharges and different drops, both for Flumed and Unflumed conditions. Further basic research on the design of Falls is in hand.

In the Navigation field, experiments were continued to evolve training measures for the improvement of navigability in some of the difficult reaches of the Hooghly like Fulta, James and Mary, Sankrail and Howrah.

The problem of Head Water Supply of the Hooghly and its effect on navigation are being studied closely and all relevant data is being analysed. Final proposals were made to arrest silting in the Harbour Channel at Cochin by means of blocking the Mattanchari Channel; experiments to reduce the "Range" in Madras Harbour were completed in a model with vertical exaggeration of 20. Experiments in a geometrically-similar model will now be taken in hand.

The Station gave suitable advice for the improvement of water supply for Tata Iron and Steel Works, Jamshedpur and for Nagpur town.

Geophysical investigations for Foundations were made at alternative dam sites at Ukai on river Tapi—Kakrapar Project. Advantage was taken during the working of Siphon No. 7 and the undersluices at Hirebhasgar during the last monsoon season for the vibration studies.

In the Physics Section of this Station, the study of stress analysis of the Engineering Structures by means of photo-elastic method was the main feature. These studies were very much helped by the presence of Dr. Baud, the UNESCO Expert in this subject. The Soil Section was busy with the study of Shear Characteristics of Soils by triaxial compression tests, and with study of physical properties of black cotton soil. Soil analysis were also continued for exploration of suitable materials for the earth dam across the river Tapi.

Peak runoff in the Mahanadi Basin was analysed by the Unit Hydrograph method in the Statistical Section. Statistical analyses were also made of the tidal data in connection with Head Water Supply of the river Hooghly.

6. During the year the Director and the Staff visited a large number of places in connection with experiments in progress in the Station. (See Figure 1.) In many

cases advice was given after inspection of sites only, without carrying out actual model experiments. The inspection of Volute Siphons at Hirebhasgar (Mysore) and the Hooghly river at Calcutta lasted for a number of days at a stretch.

7. The Station contributed the following papers to different Journals during the year—

(A) C. B. I. & P. Journal

1. Some Aspects of break up of liquid jets as applicable to the behaviour of bucket-type-splitters used as energy dissipators.
2. Earthquake forces on earth dam.
3. Distribution of horizontal acceleration along the height of a dam during earthquake.
4. The use of a transformation of the Laplace equation in the relaxational approach to potential flow in curved conduits.
5. The development of three-dimensional relaxation processes.
6. Parabolic liquid jets.
7. Unit hydrograph method applied to Mahanadi catchment above Sambalpur.
8. Is rainfall increasing in Tapti Basin?
9. Preliminary vibration experiment on the Khadakwasla Dam (Poona).
10. Use of frequency method for estimating rainfall intensities of rare occurrence.
11. The standardisation of relaxation treatment of systematic pressure compositions for overflow spillway discharge.
12. Design of Canal Falls.
by Shri B. S. Talwani, D. D. S. T. Gholankar, R. O.

(B) C. W. & P. Commission Journal

1. Dams and earthquakes.
2. Model studies for sand exclusion from the Ye-U Canal (Burma).
3. Training of Narmada river at Broach.
4. A refraction seismic equipment used in C. W. and P. R. Station, Poona.

(C) Indian Journal of Power and River Valley Development

During the year 24 specific notes and 2 basic notes were published.

8. As usual Khadakwasla Field Hydraulic Station was visited by a number of eminent and distinguished engineers, scientists, educationalists, public men, members of parliament, and the honourable ministers.

The notable among them are—

- (i) Hon'ble Shri Prakasa, Minister of Natural Resources and Scientific Research, Govt. of India.
- (ii) Hon'ble Jivraj Mehta, Minister for P. W. D., Govt. of Bombay.
- (iii) Mr. G. P. Alexander, Chairman, Madras Port Trust.
- (iv) Shri M. R. Venkataraman, Chief Engineer, Western Railways.
- (v) Shri H. P. Mathrani, Consulting Engineer, Govt. of India.
- (vi) Shri B. L. Gulati, Director, Geodetic Branch, Survey of India.
- (vii) Mr. J. R. D. Tata, Chairman, Tata Industries.
- (viii) Shri A. K. Chanda, Finance Commissioner of Railways.
- (ix) Shri P. C. Battacharya, Joint Secretary, Finance Ministry, Govt. of India.
- (x) Shri M. L. Behl, Chief Engineer, P. W. D., Bihar.
- (xi) Shri C. V. Venkateswaran, Chief Engineer, Cochin Harbour.

Members of the British Commonwealth University's Delegates also visited the Station and were much impressed by the work done here.

RESEARCH PUBLICATIONS*

of

Central Waterpower, Irrigation and Navigation Research Station

Publications*	Title	Price
Number		Rs. As. p.
1	Annual Report for 1937-38	7 8 0
2	Annual Report for 1938-39	7 8 0
3	Note on 'Gibb Modules, 1941	3 0 0
4	Annual Report for 1939-40	7 8 0
5	Annual Report (Technical) for 1940-41 with Index for 1937-41	7 8 0
6	Annual Report (Technical) for 1941-42	7 8 0
7	Annual Report (Technical) for 1942-43	7 8 0
8	Annual Report (Technical) for 1943	7 8 0
9	Annual Report (Technical) for 1944 with Index for 1940-44	7 8 0
10	Annual Report (Technical) for 1945 with Index for 1937-45	25 0 0
11	Annual Report (Technical) for 1946	7 8 0
12	Annual Report (Technical) for 1947 with Index for 1943-47	25 0 0
13	The Behaviour and Control of Rivers and Canals (Parts I and II), 1949, by Sir Claude C. Inglis	16 8 0
14	Annual Report (Technical) for 1948 with Index for 1944-48	10 0 0
15	Annual Report (Technical) for 1949 with Index for 1945-49	21 4 0
16	Annual Report (Technical) for 1950 with Index for 1946-50	

* Available from the Manager of Publications, Delhi or agents and the High Commissioner for India, India House, Aldwych, London, W.C. 2.

ANNUAL REPORT (TECHNICAL), 1951

SECTION A—THEORY AND PRACTICE OF MODEL EXPERIMENTS

2. PRACTICE

(i) COLLECTION OF DATA FOR MODELS

1. ANALYSIS OF MADRAS HARBOUR DATA TO DETERMINE THE PRESENCE OR OTHERWISE OF SEICHES (i.e., LONG PERIOD SURGES)

1. Introduction

A.R.T. 1951.

THE disturbance experienced in the Madras Harbour during cyclonic storms is described as 'range'. Such range in the harbour causes ships to move in longitudinal and transverse directions which makes loading and unloading operations pretty difficult and sometimes impossible. A closer examination of the trouble and the operating forces is essential for the proper appreciation of the problem.

2. Prototype Data

2.1. Knowledge of conditions in the harbour and outside, during a cyclonic storm, was essential when model experiments were undertaken. Some data was collected in the 1948 and 1949 storms by float type gauges; but, unfortunately, this was of little use due to location of gauges in shallow depths. In 1950, eight gauges were fixed at carefully selected points in deep water. Their locations are shown in Figure 1. The gauges consist of 9 inches diameter pipes immersed 60 feet below I. S. L. W. A float works in the pipe and the wave heights are observed on a graduated scale.

Fig. 1.

2.2. A cyclonic storm occurred in the Bay of Bengal on 3rd December 1950 and affected the Madras Harbour. The following is the record of the conditions at Madras:—

(i) Wind blowing from N.N.E. direction.

(ii) Waves.

(a) *North of Aga Light*—Length 200 feet; Height 2'-8"; Period 8½ seconds; Direction E.S.E.

(b) *West of Aga Light*—Length 100-150 feet; Height 1 to 3'-6"; Period 12 seconds; Direction N.E.

(c) *Outside Harbour (2,000 feet east of Aga Light)*—Length 125 feet; Height 8-10 feet; Period 8.6 seconds; Direction E.S.E.

Waves were wheeling round the Aga Light in a radius of 400 feet and making towards the harbour entrance.

(d) *Inside Harbour*—Waves generally fanned out towards the East, South and West Quays. Approximate values of waves at South Quay were—length 20 feet, height 2.4 feet, period 7 seconds. Waves were not well defined.

(e) *Ranging of Ships*—Movement of ship at South Quay S. S. Rajula, of 10,200 tons had berthed. Draft Stern 19'0 feet, Bow 17'0 feet.

Ranging { Vertical 1'-6".
Longitudinal 12'.
Transverse 2'-6".

The period of ranging of the ship was not observed.

Simultaneous observations of wave heights were made at eight gauge points for a period of 15 minutes.

3. Analysis of Results

3.1. Frequency distribution of wave heights and periods was done to get the significant wave height, average period, etc. The results are presented in tables 1 and 2. Wave heights at gauge No. 7 on East Quay are exaggerated on account of shallow depth and are, therefore, not comparable. The maximum wave height in the harbour at South Quay II was 3.75 feet, the significant wave height being 2.8 feet. The significant wave height lies between 70 to 80 per cent of the maximum wave height observed at all gauge points. The wave periods lie in a wide band from 4.5 seconds to 20.0 seconds though 80 per cent of the wave periods are below 10.5 seconds. Waves of period longer than 20 seconds were too infrequent for a satisfactory analysis.

3.2. The ranging of ships in a cyclonic storm may be due to either of two different conditions in the harbour—

(a) Reflection of waves from vertical surfaces.

(b) Generation of long period surges known as 'seiches' due to resonant motion of inside mass of water.

TABLE 1

Analysis of observed wave heights in the Madras Harbour during the storm on 3rd December 1950.

Serial No.		Gauge No.								REMARKS.
		1	2	3	4	5	6	7	8	
1	2	3	4	5	6	7	8	9	10	11
1	Maximum wave height observed at the point, (ft)	3.75	3.0	2.0	2.0	6.0	4.0	5.5	4.25	
2	Significant wave height at the point (ft.)	2.8	2.19	1.6	1.62	4.66	2.95	4.5	3.18	
3	$\frac{\text{Significant ht}}{\text{Max. ht.}} \times 100$	74.5	73.0	80.0	81.0	77.7	71.5	81.6	75.0	
4	80 per cent of waves observed below height (ft.)	2.12	1.62	1.62	0.87	4.37	2.12	3.0	2.3	
5	80 per cent of waves observed below period (seconds).	10.5	8.5	10.5	10.5	12.5	10.5	12.5	8.5	

Definition.

Significant wave height —It is the average of all wave heights greater than 2/3 of the maximum observed height.

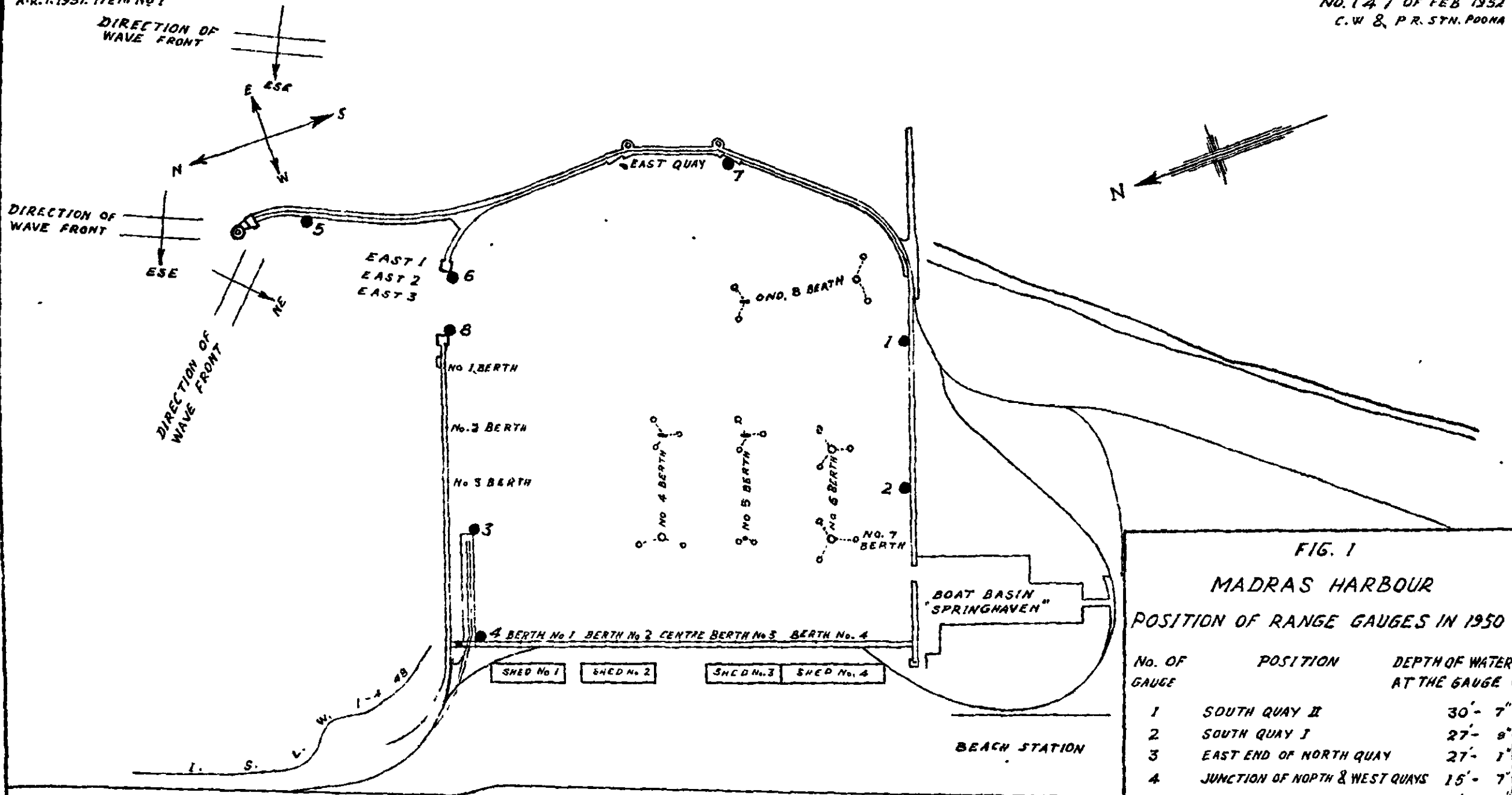


FIG. 1
MADRAS HARBOUR
POSITION OF RANGE GAUGES IN 1950

No. OF GAUGE	POSITION	DEPTH OF WATER AT THE GAUGE
1	SOUTH QUAY II	30'- 7"
2	SOUTH QUAY I	27'- 8"
3	EAST END OF NORTH QUAY	27'- 1"
4	JUNCTION OF NORTH & WEST QUAYS	15'- 7"
5	OUTER QUAY NEAR AGA LIGHT	5'- 11"
6	EAST ENTRANCE HEAD	28'- 5"
7	EAST QUAY	4'- 3"
8	WEST ENTRANCE HEAD	28'- 7"

SCALE:- 1" = 800 FT

COMP. BY. *M. S. S. S.*
TRACED BY R. T. THAKOR

TABLE 2
Percentage frequency distribution of wave periods at three typical gauge points.

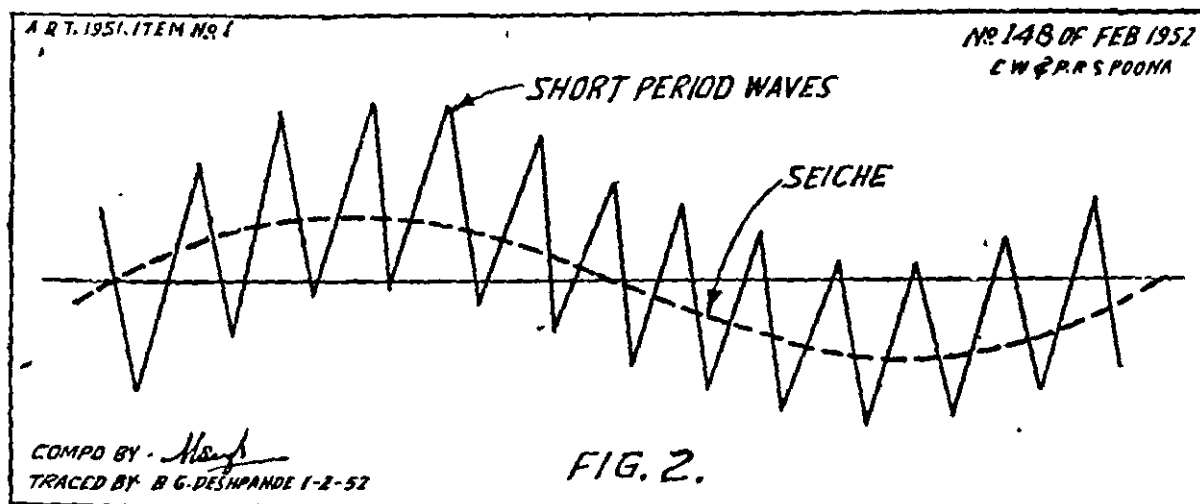
Serial No.	Wave period (Seconds)	Gauge No			REMARKS.
		1 (%)	5 (%)	8 (%)	
1	2	3	4	5	6
1	≤ 2.5	0.69	4.0	1.64	
2	2.5 ≤ 4.5	26.9	12.0	31.69	
3	4.5 ≤ 6.5	23.45	12.0	30.05	
4	6.5 ≤ 8.5	18.62	22.67	14.75	
5	8.5 ≤ 10.5	5.52	17.33	10.93	
6	10.5 ≤ 12.5	7.59	10.67	5.46	
7	12.5 ≤ 14.5	4.83	5.33	1.64	
8	14.5 ≤ 16.5	4.14	2.67	1.64	
9	16.5 ≤ 18.5	2.76	4.0	1.64	
10	18.5 ≤ 20.5	2.07	1.33	—	
11	20.5 ≤ 22.5	0.69	4.0	—	
12	22.5 ≤ 24.5	1.38	—	0.55	
13	24.5 ≤ 26.5	—	1.33	—	
14	26.5 ≤ 28.5	—	—	—	
15	28.5 ≤ 30.5	0.69	—	—	
16	30.5 ≤ 32.5	—	1.33	—	
17	32.5 ≤ 34.5	0.69	—	—	

3.3. Surges of periods between 20 seconds and 60 seconds particularly affect ships of 6,000 to 30,000 tons displacement. In this connection interesting observations were made and important conclusions drawn by Wilson in his paper "Ship Response to Range Action in Harbour Basins"—Proceedings of American Society of Civil Engineers, Volume 76, Separate No. 41 of November 1950. The following is the extract from this paper.

"These seiches which have a periodicity of about 2 minutes and less are capable of causing serious resonant motion in ships as normally moored. In particular, for most ships of commercial sizes (from 5,000 to 30,000 tons displacement) the dangerous periodicities would seem to be in the range from 1 minute to 20 seconds."

It is, therefore, vital to know if such seiches are generated in the harbour. A fuller knowledge of the characteristics of waves, such as period, amplitude, etc., is necessary for evolving proper remedial measures in order to combat the menace of ship ranging. The gauges with submerged ends open are not well suited for detection of surges as the observations in these are obscured by waves of comparatively larger amplitude—see Figure 2. An attempt was, however, made to detect their presence by periodogram analysis.

Fig. 2.



3.4. The wave data for gauge No. 6 was selected for analysis as the presence of long period surges was apparent. The short term periods range from 5 to 20 seconds. To examine the presence of any superimposed long term periodicity, the short term periodicities were ironed out, in the first instance, by a high low

mid-points average line drawn through. The very small periods having been evened out by the method, larger periods in the neighbourhood of about 20 seconds appeared more conspicuous in the new picture. Considerable variations were, nevertheless, present and the rigid mathematical search for a periodicity failed to yield one in the neighbourhood of 20 seconds.

The new waves were all, therefore, again evened out by yet another high low mid-points average line passed through. The new line brought out into prominence waves of larger period duration. Mathematical analysis was once more employed when periods ranging over 58 seconds, 65 seconds, 70 seconds, 75 seconds, etc., stood out.

Eighty per cent of the waves, as observed, were of periods less than 105 seconds. The statistical analysis, however, shows that surges of longer period do exist and their periods are of the order of 60 seconds

4. Gauges for Observation of Long Period Surges

4.1. The present gauges installed in the harbour, with submerged ends open, respond to all frequencies and are, therefore, unsuitable for observation of seiches. For this a simple device was evolved. To record long period surges, faithfully, the gauge should not respond to short period waves. It was found that this could, effectively, be done by closing the bottom end of the gauge pipe leaving only a small opening.

In the Madras Harbour, 80 per cent of the waves, as observed in the recent storm, were of periods less than 105 seconds. Maximum amplitude was 50 feet. The period of seiche (if at all present in the harbour) seems to be of the order of 60 seconds as shown above. The maximum amplitude can be roughly estimated as 15 feet from the vertical movement of the ship S. S. Rajula at the South Quay.

4.2. The gauge for observation of seiches should have, therefore, the following characteristics:—

Diameter of gauge pipe and of aperture should be so proportioned that 50 feet waves of 100 seconds period and below should cause very little variation in the gauge reading while seiches of 40 seconds period and above and 15 feet amplitude should be recorded faithfully.

Experiments were made in a wave flume, where gauge tubes with different apertures were tested. These showed that the above conditions would approximately be met by making the aperture diameter 20 per cent of that of the gauge pipe.

4.3. In the Madras Harbour, additional gauges with pipe diameter 9 inches and aperture diameter 1.75 inches have been set up, now, at all gauge points. These are expected to yield useful information about the presence or otherwise of long period surges.

5. Conclusions

- 5.1. (i) The data of maximum and significant wave heights is very useful for verification of model.
- (ii) The analysis of wave data at gauge No. 6 indicated that the probable period of surge, if present, would be in the neighbourhood of 60 seconds.

Further Studies

Behaviour of gauges with apertures at bottom for recording seiches is being studied in greater detail.

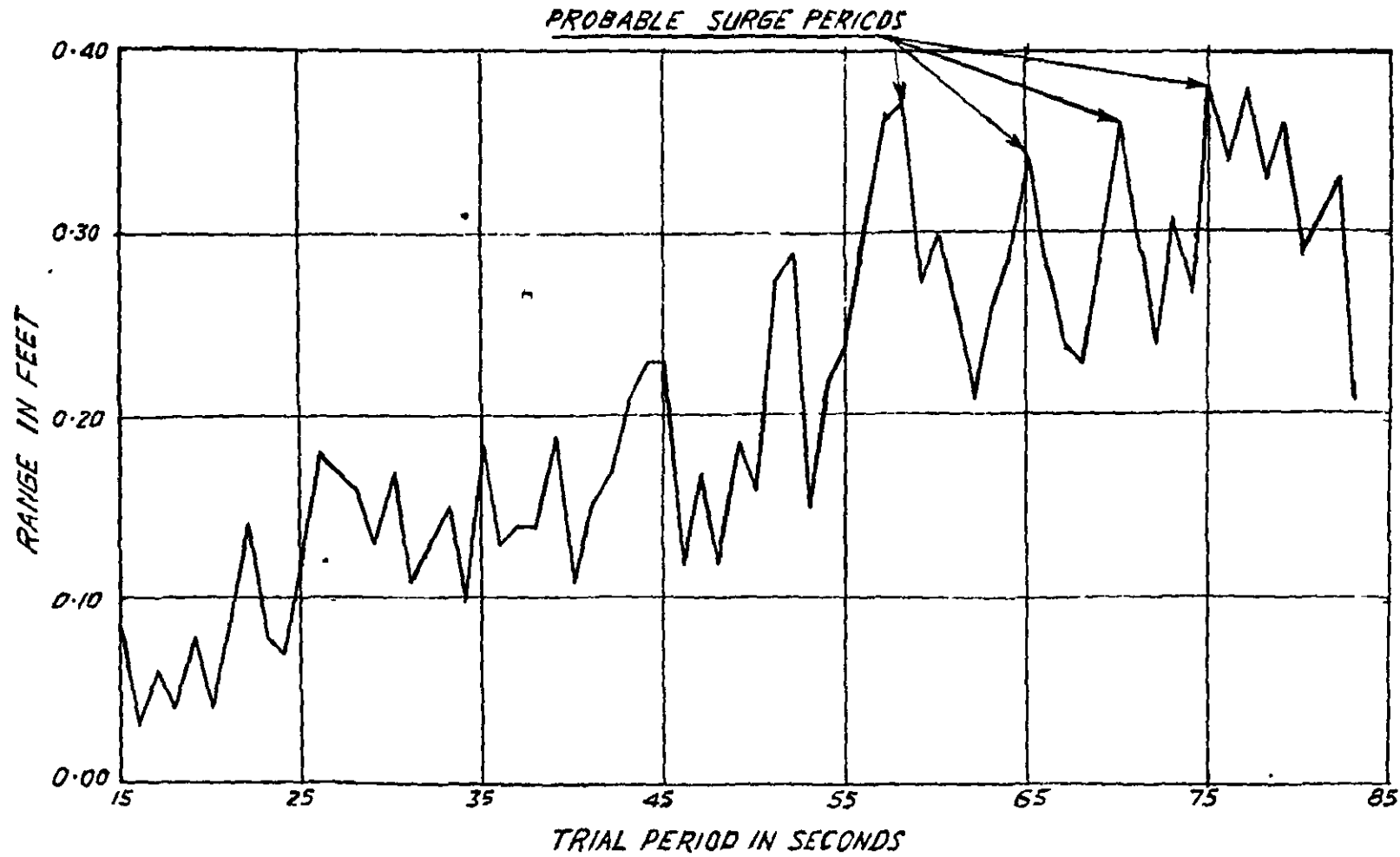


FIG. 3
PERIDDOGRAM ANALYSIS OF WAVE DATA
AT
GAUGE No. 6
IN MADRAS HARBOUR

(iv) LIMITATION OF MODEL EXPERIMENTS AND COMPARISON OF MODELS AND PROTOTYPE RESULTS

2. EFFECT OF INTERFERENCE ON DISCHARGE IN VOLUTE SIPHONS.

A. R. T. 1952.

Introductory

INTERFERENCE is unavoidable when siphons are arranged side by side in a battery. Economics and hydraulic efficiency of a project demand that they should be arranged as close to one another as possible, as also as close to the dam. Otherwise, the discharge per foot run of the dam decreases; it also means greater area of platform upstream of the dam on which the siphons are constructed, thus making the project costly. Too close a spacing, on the other hand, may cause considerable reduction in the discharge which is also an undesirable feature.

Results of interference experiments with regard to the Marconahalli siphons are indicative of the magnitude of the possible interference for the spacing there (5' clear spacing between dome and dome). About one per cent interference was indicated when the rim depth of the dome was one foot, and negligible when it was 6.5 feet¹. Thus, the increase in the drum height would decrease the interference effect. This is obviously due to prevention of air entry from outside by a deeper drum.

One interference aspect which cannot be explained by the mere theory of interference is the development of vortices. It is observed that when siphons are very close, the vortices are to some extent suppressed and when they are far apart, they are aggravated. These conflicting factors can be solved by experimental investigation alone.

Accordingly experiments were initiated to determine exactly how the various factors contributed to the loss of discharge by interference. The factors considered were—

- (1) distance from siphon to dam, x_1 ,
- (2) distance between two siphons, x_2 ;
- (3) the height (rather, the depth) of the drum x_3 ; and
- (4) the opening below the drum, x_4 .

Experimental Set-up

The experimental unit set-up was as shown in Figure 1 and photographs 6674, 323 and 6313 and the explanations of x_1 , x_2 , etc., are given in Figure 1.

Fig. 1 and
Photos Nos.
6674, 323
6313.

It can be seen that the unit contains three siphons, 6" diameter each, with side plates at 6" distances from the end siphons. This is equal to the diameter of the siphon and it was presumed that this space was sufficiently great as compared to values of x_1 , x_2 , etc., and thus would not cause any side effect.

The individual siphons had their bends with rectangular bases, so that all the three siphons could be supported on a machined girder supported on either side by wooden blocks and adjustable up or down by means of jacks. The platform for the siphons was made of properly arranged plain iron sheets reinforced at the bottom by wooden battens. A number of intermediate sheet pieces of various dimensions were got ready so that the distance between siphons could be easily changed at will by replacing some of the sheets and pushing the siphons on the girder towards or away from one another, thus varying the value of x_2 . A movable plank was used to represent the dam and the adjustment of the value of x_1 was thus quite easy.

The domes were made of 16-gauge galvanized iron sheeting and collars of any depth could be added by a simple expedient (see photograph 324). This enabled the variation of the drum height x_3 . The piers in prototype were represented by

Photo 324.

¹ Annual Report Technical 1945, page 76

a cage of 3/8" rods and the depth of the cage, i.e., the length of the rods, could be easily adjusted by pushing the cage up or down on the drum according to the opening provided. The opening, x_4 , could be manipulated by raising or lowering the siphons by the method mentioned above.

For all experiments, the annular width and the shortest distance between the siphon lip and the dome were kept the same.

Thus the set-up permitted the variation of all the four factors at will and the following was the order of experiments conducted.

Experiments—First, the effect of the values of x_1 and x_2 on C_d was determined for the following combination—

x_1 (ins) 7.75, 8.75, 9.75, 10.75, 11.75 and 12.75.

x_2 (ins) 14.5, 15.5, 16.5, 17.5, 18.5 and 19.5.

Now, the values of x_3 and x_4 were changed as follows—

For any particular platform level with respect to the lip, $x_3 + x_4 = \text{constant}$. When, say, one-inch collar was added to the drum x_3 would be increased by 1" and x_4 reduced by the same amount. When the opening was reduced to a minimum, the platform level was changed and other combinations were obtained as follows, the value of x_2 in all the cases being kept at 16.5 inches.

Opening (ins) 1, 2, 3, 4, 5 and 6.

Height of Drum (ins.) $3\frac{1}{2}$, $4\frac{1}{2}$, $5\frac{1}{2}$, $6\frac{1}{2}$, $7\frac{1}{2}$ and $8\frac{1}{2}$.

Siphon height (ft) 1.60, 2.10 and 2.60

In the above experiments a liberal use of plasticine had to be adopted to prevent any leakage of water. A combination of a standard hump and a calibrated V-notch was used for measuring discharge, the variation due to interference being generally measured on the notch only and a constant discharge being maintained on the S. W. Flume.

All the experiments were repeated for three different heights of siphons, viz., 1.604', 2.104' and 2.604'.

Each observation of discharge made was repeated twenty times, so that in all, 1,800 observations were made for the above experiments.

Results of Experiments

Table 1. (1) x_1 and x_2 —The data obtained are tabulated in Table 1.

TABLE 1

		Distance between Dome to Dome Centres, by divided diameter = $\frac{x_1}{D}$						
		2.42	2.58	2.75	2.92	3.08	3.25	Average
Distance between Dam Centre and Dome divided by diameter x_1/D	1.29	.668	.696	.687	.669	.699	.666	.681
	1.46	.674	.697	.686	.672	.704	.695	.688
	1.63	.682	.701	.686	.679	.701	.690	.690
	1.79	.684	.707	.690	.690	.704	.693	.694
	1.96	.684	.700	.677	.686	.702	.692	.690
	2.13	.676	.700	.675	.674	.703	.696	.687
	Average	.678	.700	.684	.678	.702	.689	

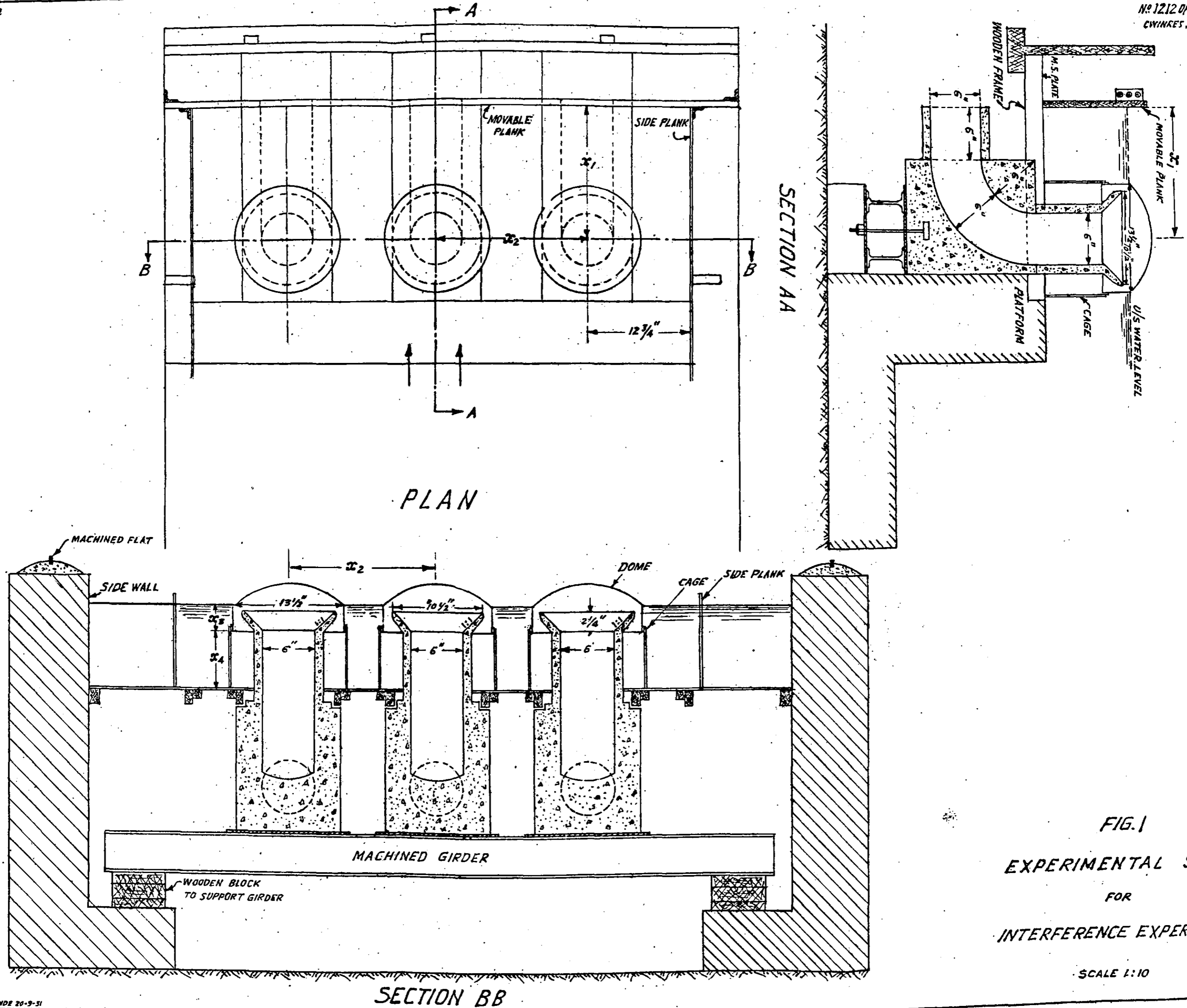


FIG. 1
EXPERIMENTAL SET-UP
FOR
INTERFERENCE EXPERIMENTS

SCALE 1:10

ITEM NO. 2.
A. R. T. 1951.



Photo 6674 : Experimental unit—Domes removed.

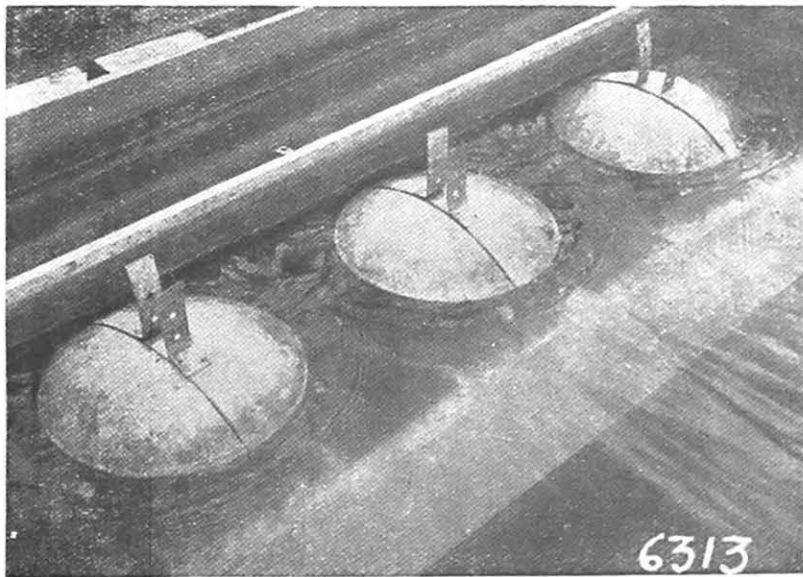


Photo 6313 : Interference Unit—working.

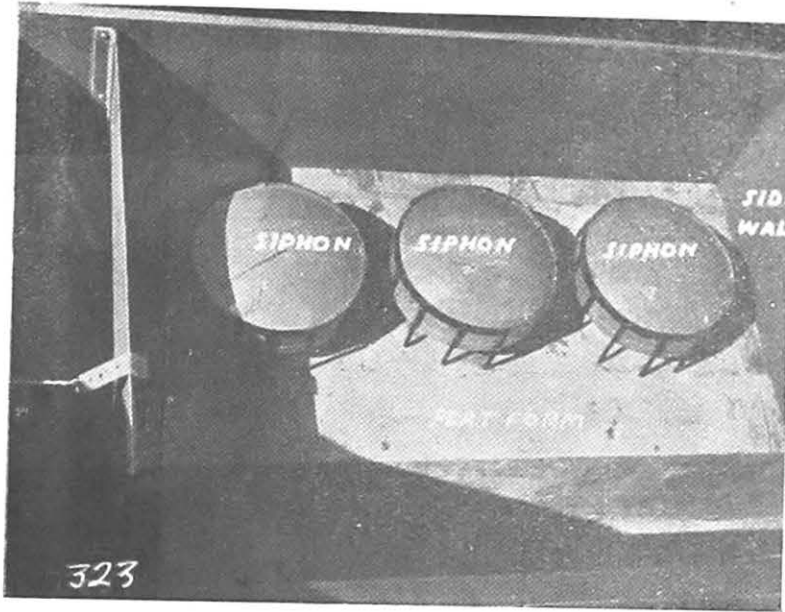


Photo 323 : Arrangement for interference experiments.

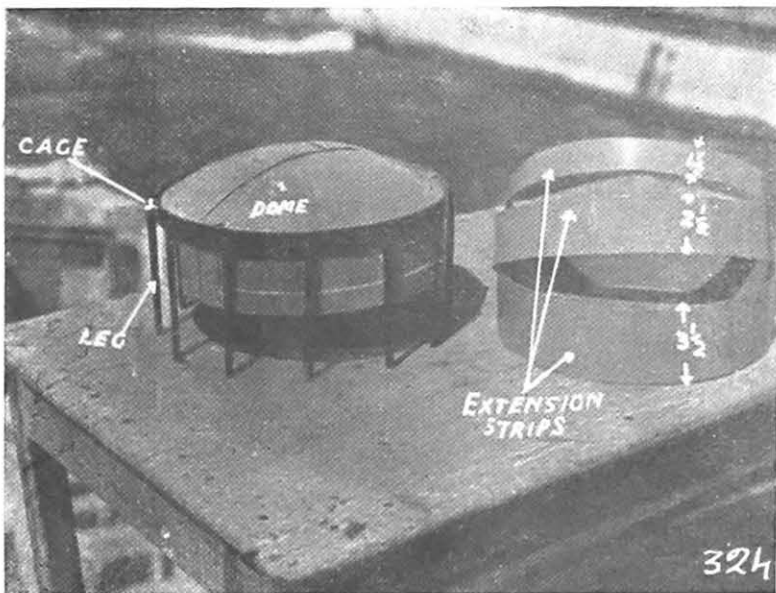
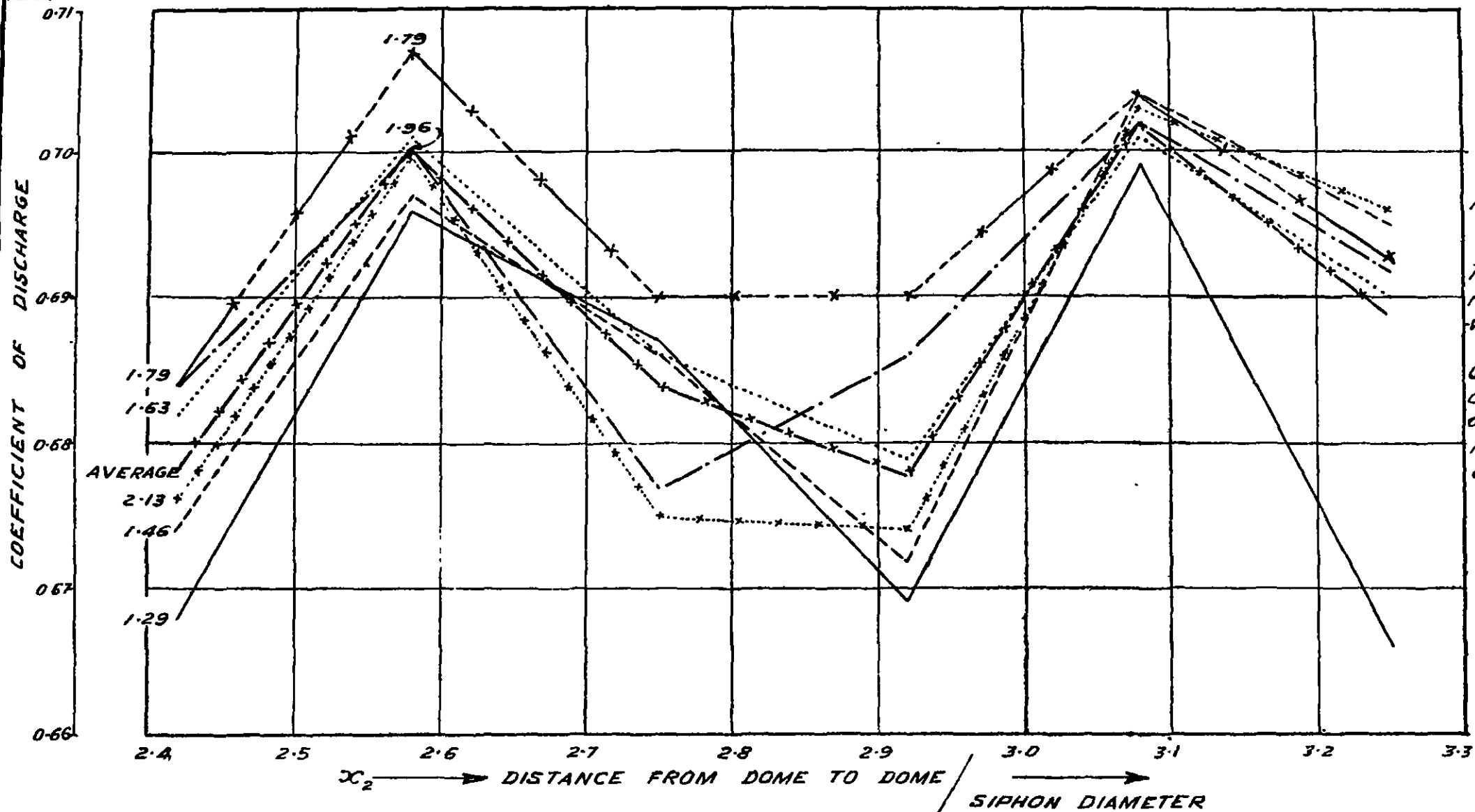


Photo 324 : The Siphon Dome, Cage and additional collars used in the interference experiments.



NOTE
Region of disturbance is
2.75 to 2.92 (x_2)
This indicates that in this
region more thorough in-
vestigations are desirable
Otherwise the results are
consistent and the coeff.
of discharge attains
Maximum at two stages
corresp. to $x_2=2.58$ AND 3.08

REFERENCES

1.29	—————
1.46	- - - - -
1.63
1.79	- x - x - x -
1.96	- . - . - . -
2.13x.....
AVERAGE	- x - x - x -

AT DIFFERENT STAGES OF
DISTANCE BETWEEN CENTRE OF DOME TO DAM
SIPHON DIAMETER

COMP BY Masrigh
TRACED BY PARNHI
S-N-1951

FIG. 2
GRAPHS SHOWING
COEFFICIENT OF DISCHARGE FOR DIFFERENT VALUES OF
DISTANCE BETWEEN CENTRE OF DOME TO DOME
SIPHON DIAMETER

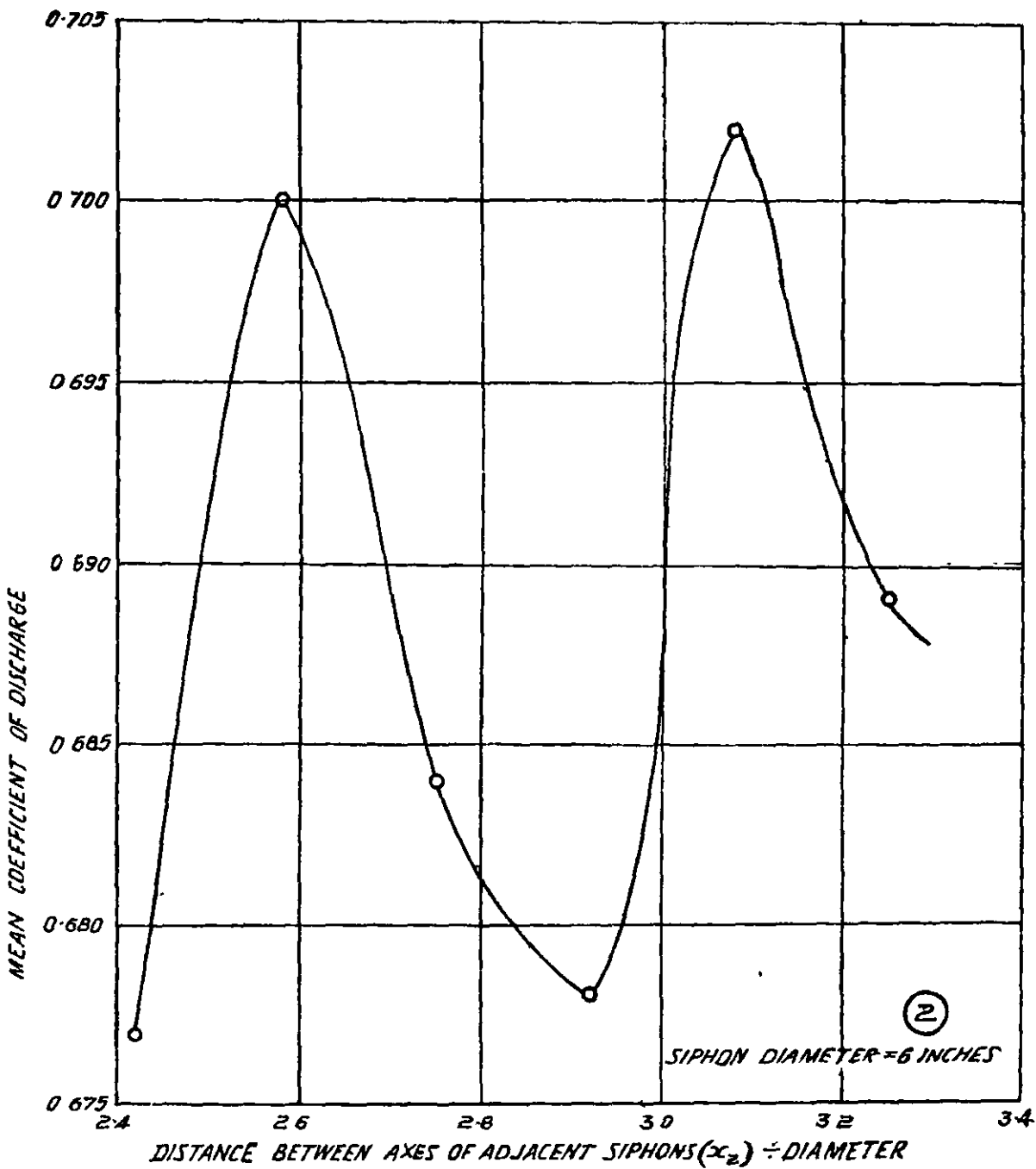
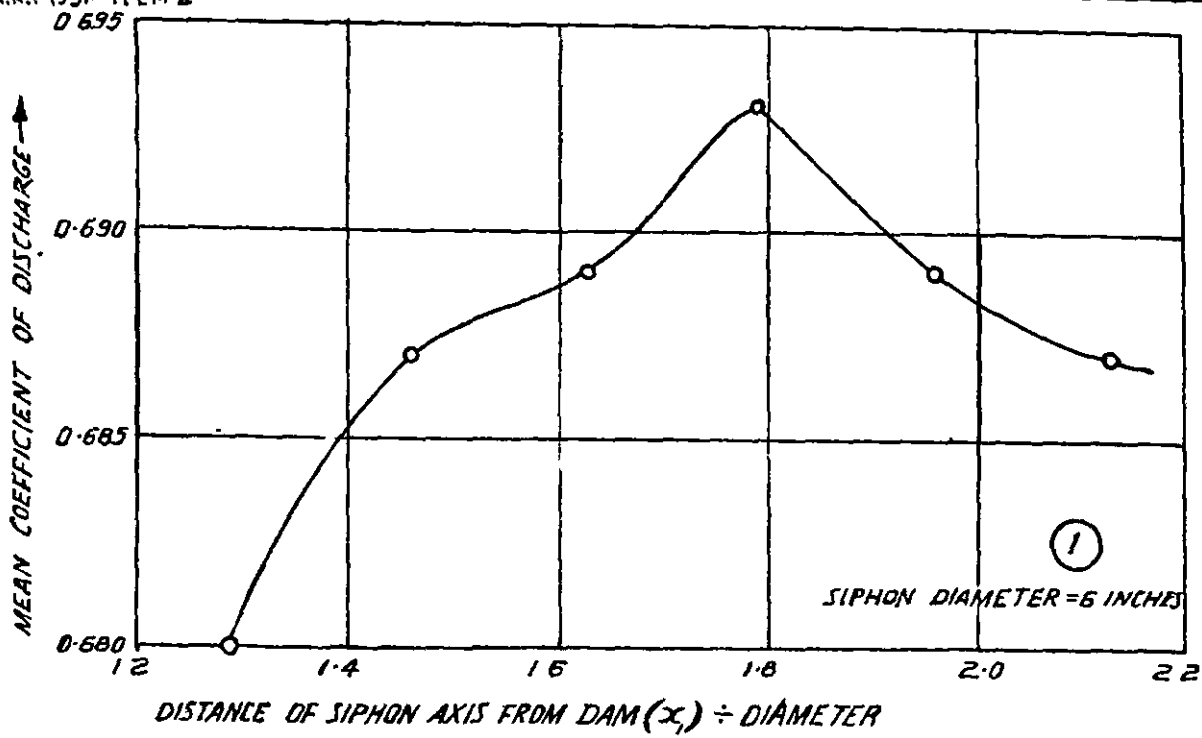


FIG: 3

VOLUTE SIPHONS - INTERFERENCE EXPERIMENTS
EFFECT ON COEFFICIENT OF DISCHARGE OF
(1) DISTANCE OF SIPHON AXIS FROM DAM, x , AND
(2) DISTANCE BETWEEN AXES OF ADJACENT SIPHONS, x_2

COMPARED BY: *M.S.*
TRACED BY: M.K. SARNIS 20-11-51

Figure 2 gives a plot of the coefficient of discharge against $\frac{x_2}{D}$ or, what may be termed the relative Dome to Dome distance, for various values of $\frac{x_1}{D}$ or, the relative Dome to Dam distance. Interaction seems to be prominent in the region $2.75 < \frac{x_2}{D} < 2.92$. It is difficult to say whether the coefficient of discharge is increasing or decreasing with an increase in x_2 in this region. Out of six distances from dome to Dam, on three occasions it is decreasing, twice it is nearly constant and once it is increasing. A superficial examination of the marked portion in Table 1 will bring out the significance of the interaction.

If the prominent region of disturbance is removed from the analysis and the examination is restricted to the rest only, the results are very consistent.

The variation of the coefficient of discharge with x_1 is on the other hand unimodal, attaining a consistently optimum value of 1.79, while that of x_2 is bimodal, reaching two peaks—see Figure 3.

Fig. 3.

While the analysis indicates that a more thorough investigation is necessary in the region of disturbance, the following pertinent points may be noted:—

- (a) The interference studied is a combined effect of the interference of streamlines and the formation of vortices.
- (b) Interference in flow causes, in effect, a suppression of vortex formation; but as vortices contribute to admission of air and hence reduction in discharge much more than the interference among the streamlines, suppression of vortices will help considerably in maintaining a high coefficient of discharge.
- (c) Where the values of x_1 and x_2 are high, there is greater scope for vortex formation and hence the reduction of discharge.
- (d) The above points show why the optimum values of x_1 and x_2 are not obtained when they are maximum, as would be normally expected.
- (e) If the interference effect among streamlines alone is to be investigated, it is necessary that vortex formation is artificially suppressed by covering the water surface—with grids or planks. This calls for the repetition of the whole experiment, but it will not be of much practical value.
- (f) It is also significant to note that the variation of coefficient of discharge is from 0.666 to 0.707, i.e., a difference of only 0.041 which meant about 6 per cent of the optimum value.

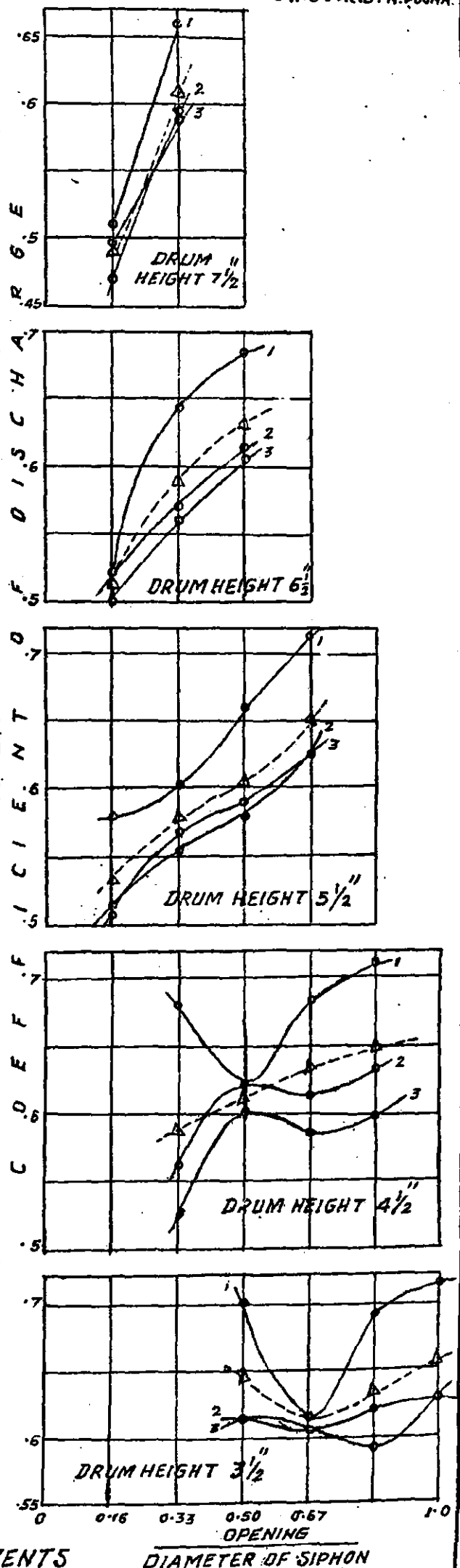
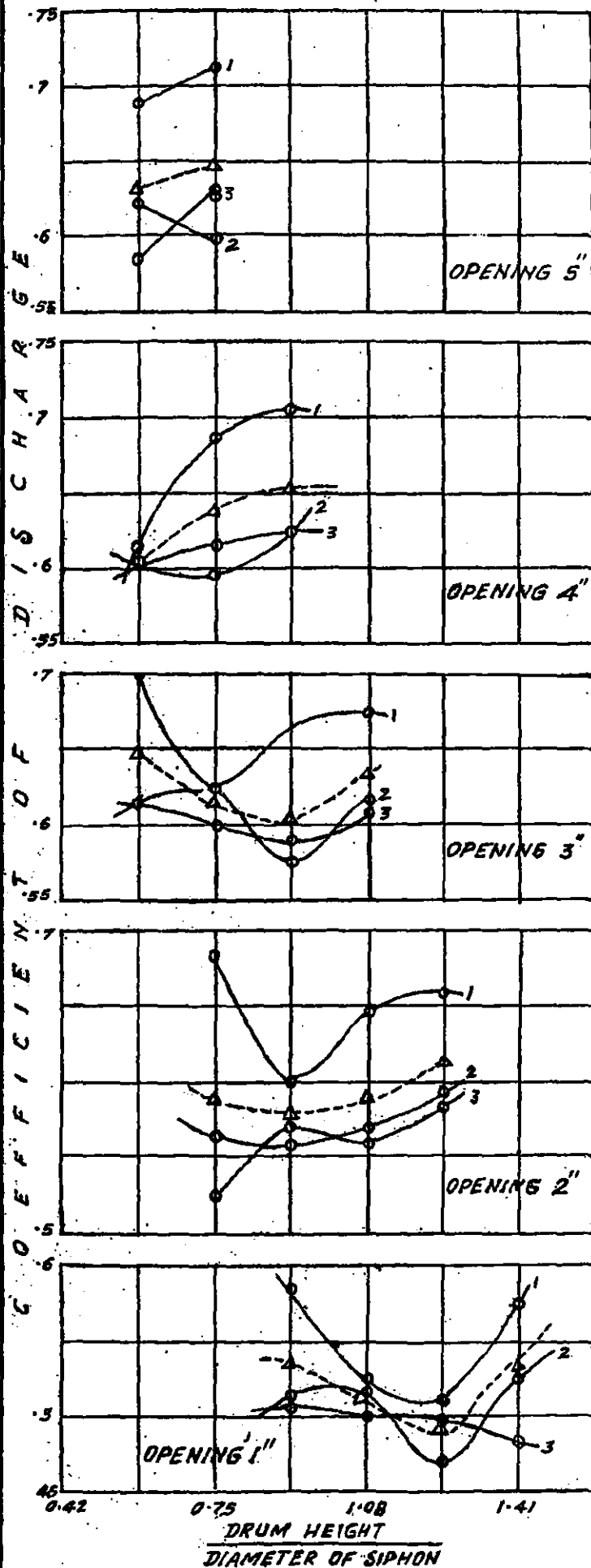
Table 2. 2. x_3 and x_4 .—The data obtained are tabulated after finding the statistical mean for each height of siphon—See Table 2.

Drum height Diameter	Opening/Diameter					
	.16	.33	.50	.67	.83	1.0
.58			0.703	0.612	0.689	0.717
			.616	.604	.621	.629
			.615	.602	.588	.632
			.645	.606	.631	.659
.75		0.682	0.623	0.686	0.713	
		.567	.625	.597	.600	
		.522	.603	.614	.632	
		.590	.617	.634	.648	
.92	0.579	0.602	0.661	0.706		
	.513	.558	.577	.625		
	.509	.569	.588	.625		
	.534	.576	.607	.651		
1.08	.522	.643	0.673			
	.515	.568	.614			
	.500	.559	.609			
	.512	.588	.632			
1.25	0.511	0.658				
	.510	.593				
	.496	.586				
	.492	.612				
1.41	0.573					
	.525					
	.486					
	.529					

The three figures in each cell indicate the values obtained respectively for the three different siphon heights 1.60, 2.10 and 2.60 ft respectively. The figures *italicized* at the bottom are pooled averages computed for the three heights of siphons.

For all drum heights, the coefficient of discharge generally increases with the opening; but the same does not appear to hold good when the variation is considered for any particular opening. However, it is apparent that the optimum result is obtained when the relative opening $X_4/D = 0.5$ and the relative drum height $X_3/D = 1.08$, both the factors being considered together. Figure 4 is a plot of the data obtained.

Fig. 4.



CURVE 1 FOR SIPHON HEIGHT = 1.50 FT.
CURVE 2 FOR SIPHON HEIGHT = 2.10 FT.
CURVE 3 FOR SIPHON HEIGHT = 2.60 FT.

FIG. - 4
VOLUTE SIPHONS INTERFERENCE EXPERIMENTS
EFFECT ON COEFFICIENT OF DISCHARGE OF
(1) DRUM HEIGHT X_3 AND
(2) OPENING BELOW THE DRUM, X_4

SECTION B—HYDRAULIC WORKS

1. RIVER CONTROL

(ii) RIVER TRAINING WORKS

3. TRAINING THE NARMADA RIVER AT BROACH (WESTERN RAILWAY)

1. History

The first railway bridge on the Narmada River at Broach, consisting of 67 spans of 60 ft. each was constructed in 1860. At that time the river was flowing in two channels, the deeper and larger channel being along the northern bank (i.e., Broach Side) and the smaller one along the southern bank with an island between the two channels. However, between 1867 and 1888, due to some major changes in the river upstream, the main stream swung towards the south bank and continues to do so even today. The bridge was considerably damaged in the floods of 1864 and 1868 while in 1876, a still higher flood caused complete collapse of the bridge due to undermining of pier foundations by heavy scour. A new bridge of 25 spans of 187.5 feet located a little further upstream of the original bridge was completed in 1881.

Fig. 1 (a).

Fig. 1 (b).

1.2. The southward trend of the deep water channel, which began in 1867, continued so much so that the channel enbayed upstream of the south abutment and caused heavy erosion of the south bank upstream and downstream of the abutment. During the high floods of 25th August 1898 pier No. 23 (3rd from south) canted 7 ft. downstream but when the flood subsided was pulled back by the eccentrically loaded superstructure of the bridge. As a result of this calamity the following protection works were carried out in successive stages:—

- (i) A pile groyne was constructed on south bank $2\frac{1}{2}$ miles upstream. Fig 1 (d).
- (ii) A small 'Z' groyne was later added between the pile groyne and the bridge. Fig. 1 (e).
- (iii) A sloping bund called 'south abutment groyne' pointing upstream from the south abutment was constructed in 1898. This is above the H. F. L. for a length of 550 ft. and is above H. W. O. S. T. for a further distance of 850 ft. and then slopes down to the river bed having a total length of 2,000 feet. Photo 7486.

1.3. In 1934, a new, second railway bridge was constructed and the old bridge was converted into a road bridge for lighter traffic. However the danger of the pot-hole, originally formed in 1898, continued to extend towards the bridges and the south abutment and at present constitutes a major threat to the safety of the bridges. Since 1940 onwards the pot-hole seems to have split up into two deep holes, one approaching the sloping head of the bund and the other moving downstream towards the bridges.

1.4. As it is very urgent to protect the south bank and the south abutments from heavy erosion and scour likely to occur in the near future, model experiments were conducted to test suitable protection works at the request of the Chief Engineer, Bombay, Baroda and Central India Railway (now Western Railway).

2. Hydraulic Data

2.1. *River Discharge*—It may be noted that Broach is only 20 miles from the sea and the bridges are within tidal limits of the river. At the mouth of the river the tidal range is from 25 to 30 feet and it extends inland to 30 miles. The high tide has, however, no influence on the gauge reading at the railway bridge during high floods and hence the model was laid as flowing in one way only. The Station estimate of highest flood discharge is 12,50,000 cusecs corresponding to the gauge

Fig. 3. reading 76 86 at the railway bridge The discharge hydrograph of 1950 is shown in Figure 3.

22. *River Slope*—The river slope in high floods is of the order of 1 8 ft. per mile.

23. *River Bed Material*—The south bank near the bridges consists of alternate layers of sand ($m=0.27$ mm.) and clay ($m=0.042$ mm.). Upstream of the bridges around shoals there are *kunker* and pebble formations having an average size of 0 59 mm. diameter.

3. Model Scales

Length Scale	1:330
Depth Scale	1:50
\therefore V. E	6.6
Discharge Scale	1.1,10,000
Time Scale=	1 day in prototype \cong 4 1 minutes in model.

4. Experiments

The following experiments were conducted:—

- Fig. 4.
- (1) *Experiment No. 1*—With existing conditions of the river and the sloping guide bund—see (a) in Figure 4.
 - (2) *Experiment No. 2*—With the present length of the sloping bund raised above H. F. L. (+76 86) and having a curved head of radius 600 ft.—(b) in Figure 4.
 - (3) *Experiment No. 3*—With the guide bund raised above H. F. L. and extended upstream along the curved high bank—(c) in Figure 4.
 - (4) *Experiment No. 4*—With the guide bund raised above H. F. L. and extended upstream by 500 feet along the existing alignment and having a curved head of 800 ft. radius—(d) in Figure 4.
 - (5) *Experiment No. 5*—With the guide bund raised and extended by 1,000 feet along the existing alignment and having a curved head of radius 800 feet—(e) in Figure 4.
 - (6) *Experiment No. 6*—With the guide bund raised and extended upstream along a curved alignment midway between alignment (c) and the extended alignment (e) and with a curved head of radius 800 feet—in Figure 4.
 - (7) *Experiment No. 7*—With the guide bund of length 1,800 feet along alignment (f) and having a curved head of radius 600 feet—(g) in Figure 4.
 - (8) *Experiment No. 8*—Same as experiment No. 2 with length of guide bund shortened by 160 feet or total length=1,640 feet—(h) in Figure 4(a).
 - (9) *Experiment No. 9*—Same as experiment No. 7 with length of guide bund shortened by 400 feet or total length=1,400 feet—(i) in Figure 4(a).

5. Experimental Results

Fig. 3.

5.1. The discharge cycle run in the model corresponded to the discharge hydrograph of 1950 which was the only one available. It was felt that the deep pot-holes formed near the sloping guide bund were mainly due to the severe eddy with a vertical axis formed during high floods when the cross flow from over the submerged part of the sloping bund met the main high velocity flow of the river. At high discharge stages the model reproduced this eddy and the resulting deep scour, thus confirming the above diagnosis.

Fig. 5.
Photos
8998 &
6996.

5.2. In experiment No 1 with the existing conditions of the river and the sloping guide bund the main river flow continued to hug the south bank upstream of the bridges. A severe eddy formed at the sloping head of the bund and the subsidiary boils and eddies continued to travel downstream creating dangerous conditions for the bridges and the south bank downstream. The velocity distribution also confirmed that the high velocity, persisting along the bund and the south bank is the main cause of the bank embayment and erosion.

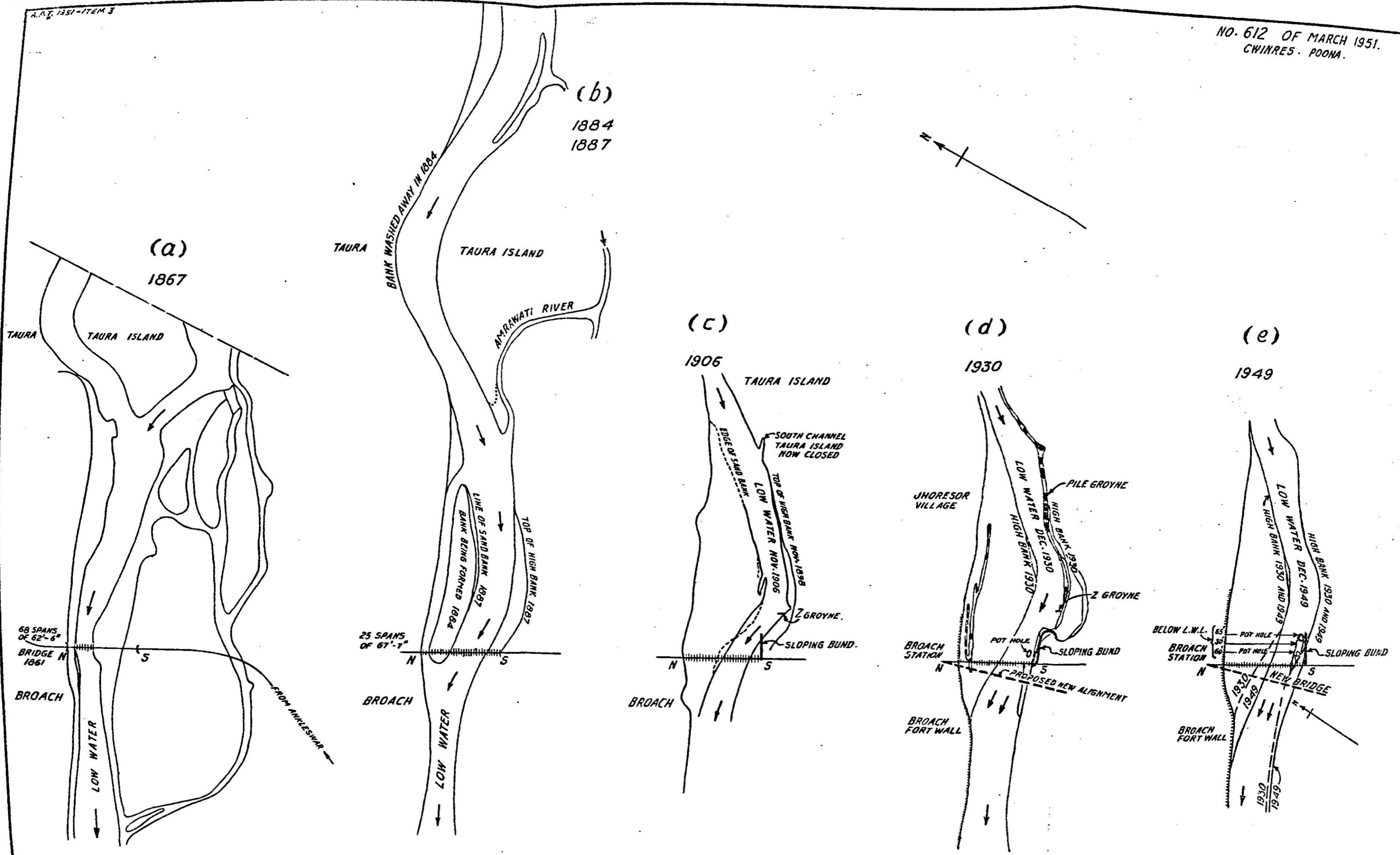


FIG. 1
 NARBADA RIVER AT BROACH
 (B.B. AND C.I. RLY.)
 PLAN SHOWING MAJOR RIVER CHANGES AND
 BRIDGE DEVELOPMENT AT BROACH SINCE 1867
 SCALE 1" = 2 MILES.

COMP. BY: Ms. A.
 TRACED BY: NAIK R.S. 8-3-51.

Narbada River at Broach, Western Railway.

1/330 : 1/50 scale vertically-exaggerated model of Narbadā River at Broach.

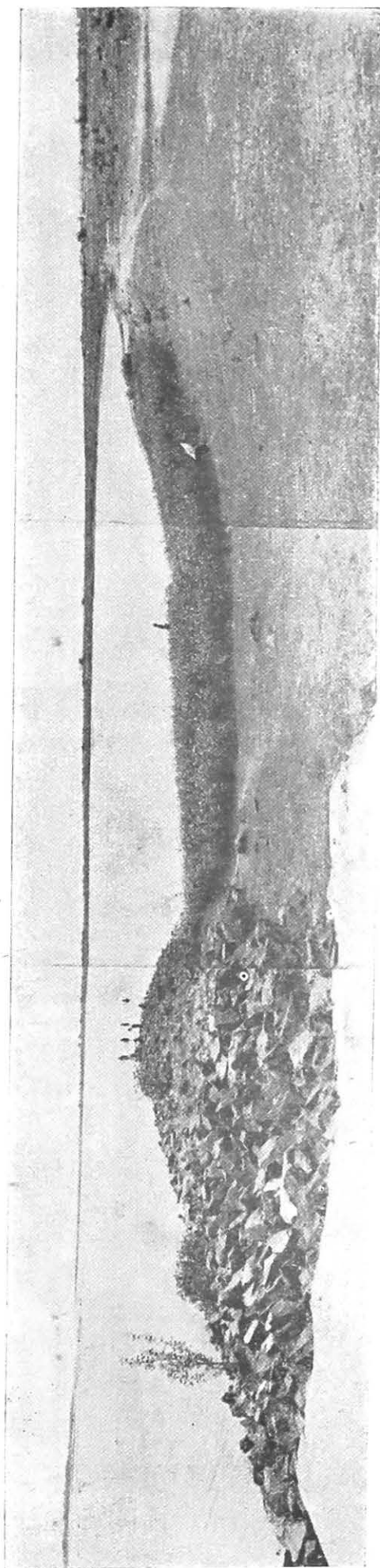


Photo No. 64-Looking down-stream showing high flood discharge with the guide bund aligned as per drawing No. 11425
24. F.

Experiment 6.

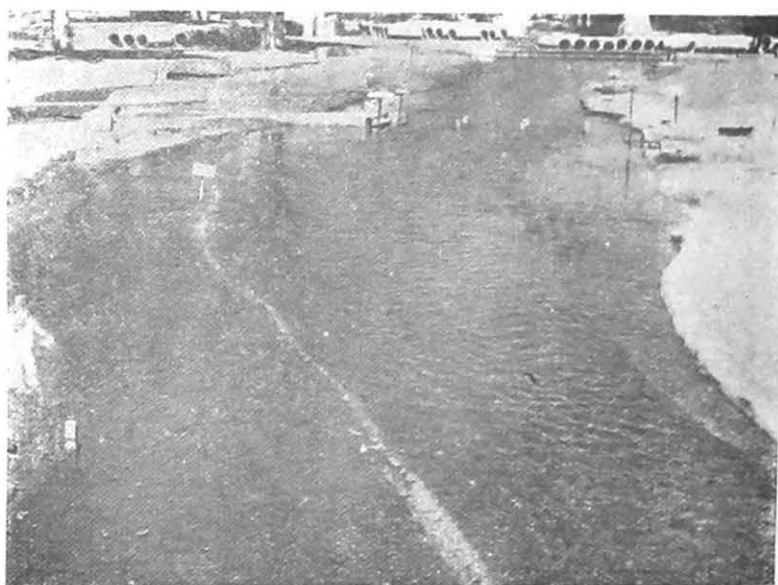


Photo No. 6998- Looking downstream showing high flood discharge running with sloping guide bund in position—Experiment 1.

Photo No. 7488 to 7486-Looking upstream showing the sloping head of the present bund and the embayed high bank upstream.

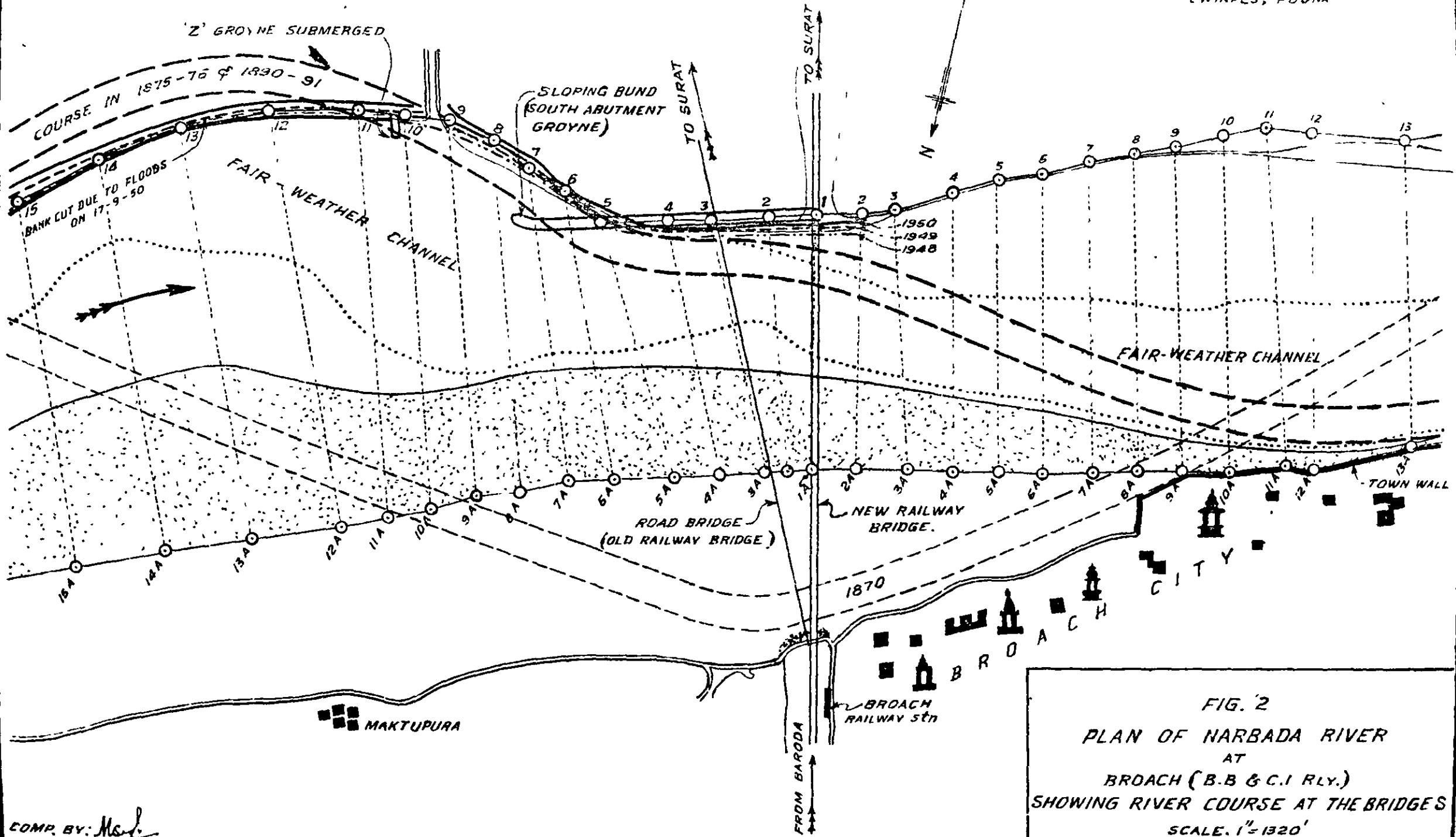
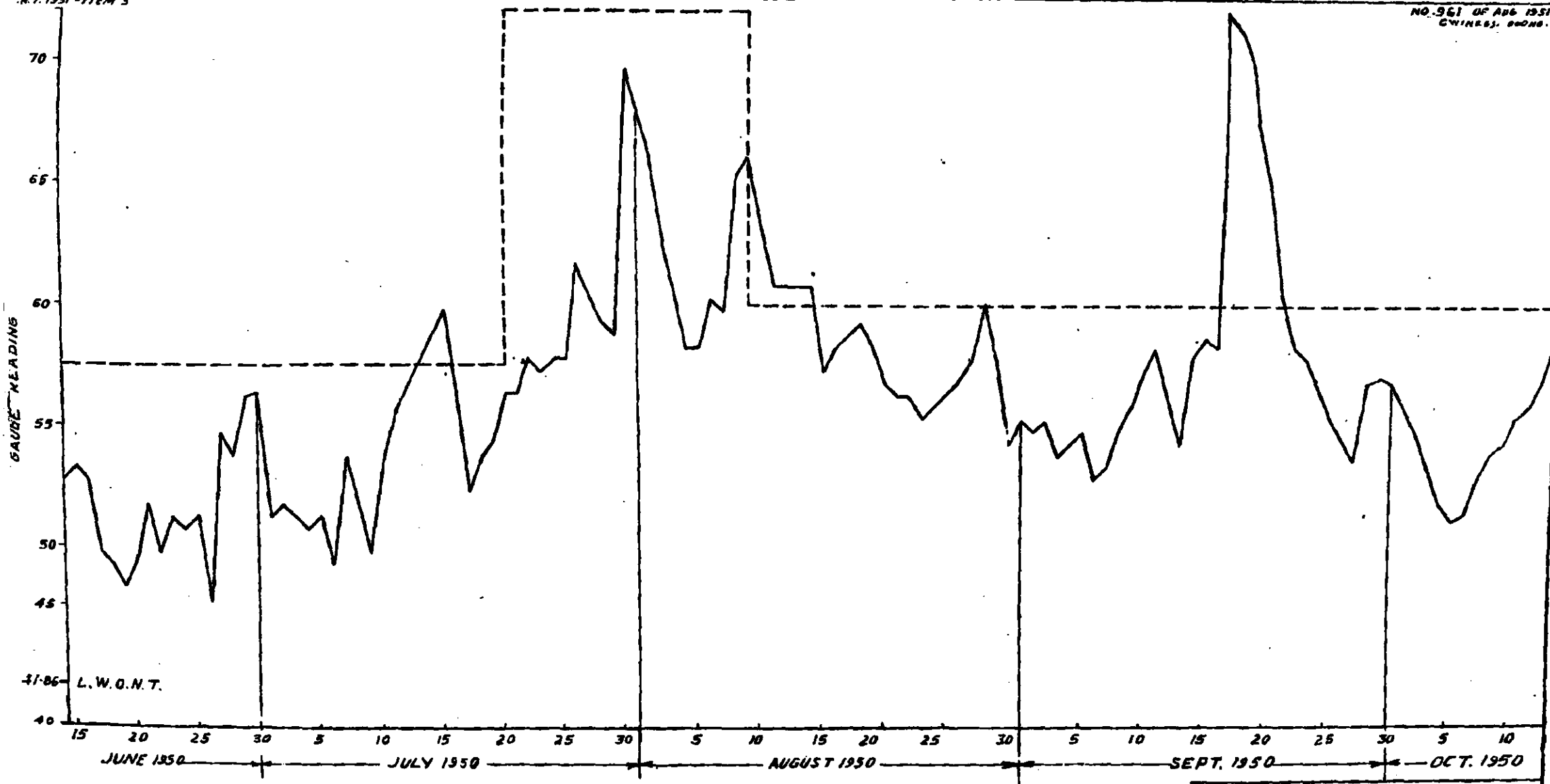


FIG. 2
 PLAN OF NARBADA RIVER
 AT
 BROACH (B.B & C.I RLY.)
 SHOWING RIVER COURSE AT THE BRIDGES
 SCALE. 1"=1320'

COMP. BY: M.S.P.
 TRACED BY PARKHI.
 8-3-51

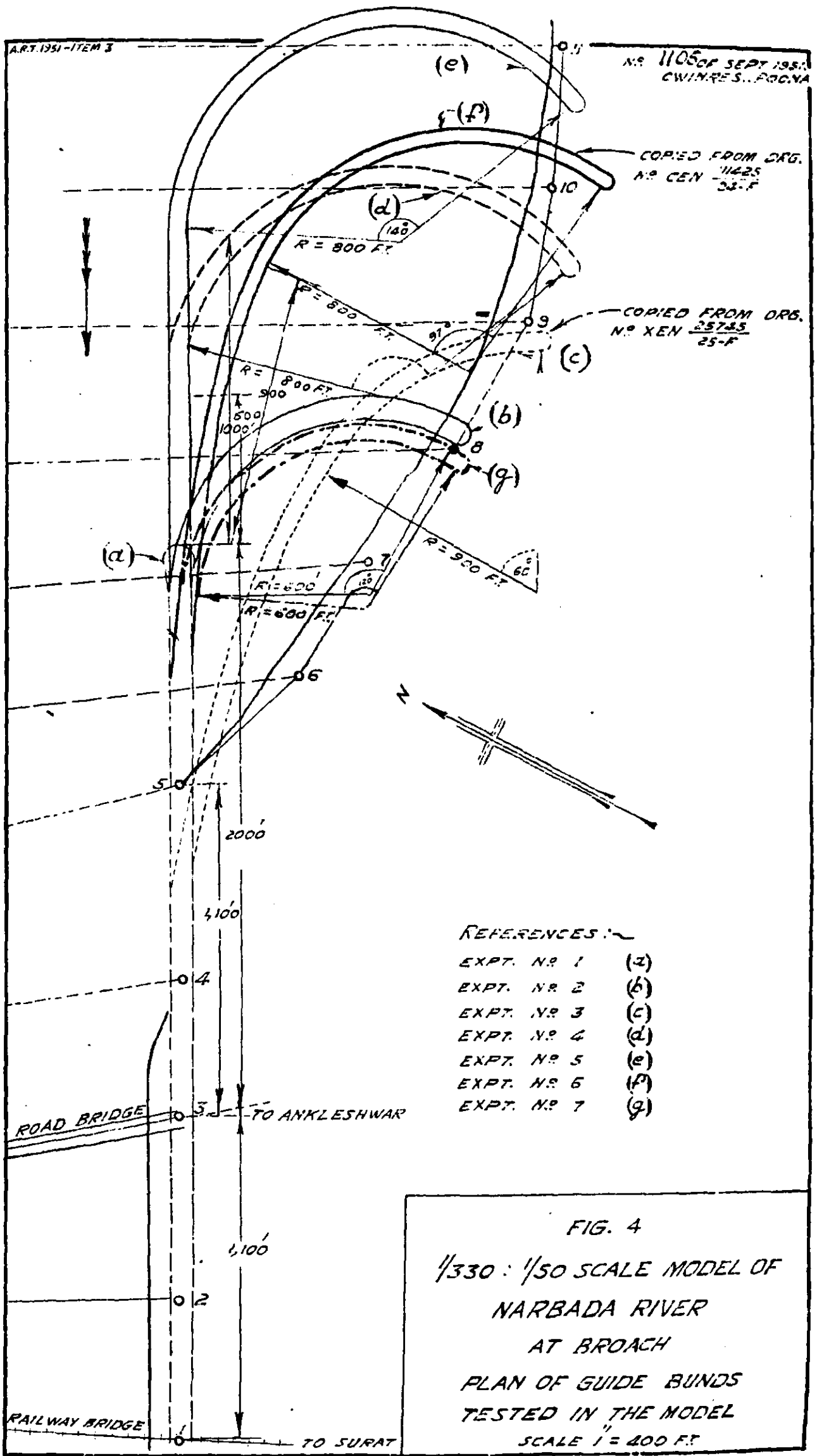


31-86 L.W.O.N.T.

REF.
 GAUGE READINGS ———
 DISCHARGE CYCLE RUN }
 IN THE MODEL. ———
 T-SCALE. 4-1 MIN. & 1 DAY

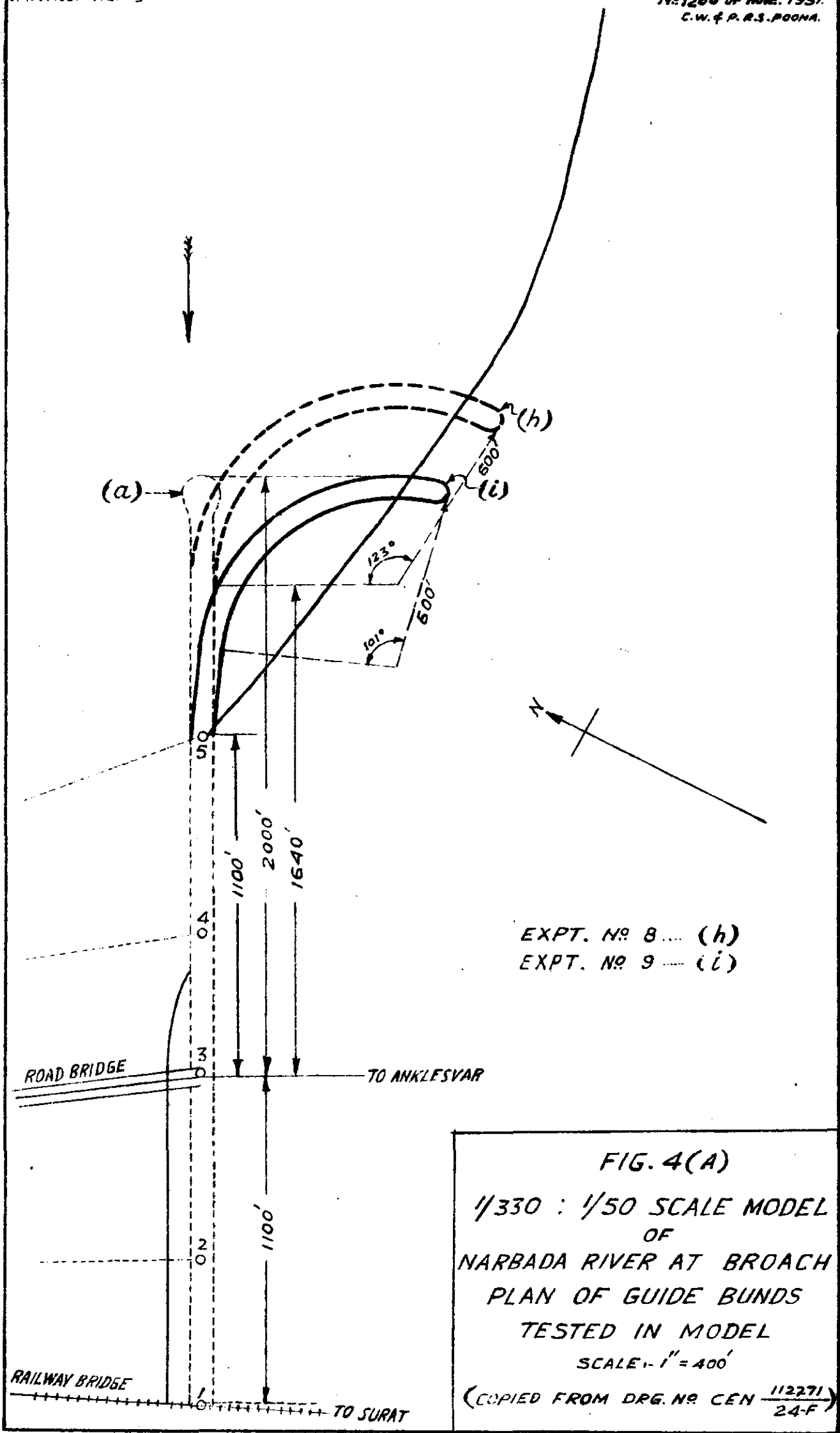
FIG. 3
 NARBADA RIVER AT BROACH
 GAUGE READINGS
 AT
 (NEW) RAILWAY BRIDGE 1950

COMP BY *M.S.*
 TRACED BY R. T. THAKUR.



- REFERENCES :-
- EXPT. NO. 1 (a)
 - EXPT. NO. 2 (b)
 - EXPT. NO. 3 (c)
 - EXPT. NO. 4 (d)
 - EXPT. NO. 5 (e)
 - EXPT. NO. 6 (f)
 - EXPT. NO. 7 (g)

FIG. 4
 1/330 : 1/50 SCALE MODEL OF
 NARBADA RIVER
 AT BROACH
 PLAN OF GUIDE BUNDS
 TESTED IN THE MODEL
 SCALE 1" = 400 FT.



EXPT. NO 8 ... (h)
EXPT. NO 9 ... (i)

FIG. 4(A)
 1/330 : 1/50 SCALE MODEL
 OF
 NARBADA RIVER AT BROACH
 PLAN OF GUIDE BUNDS
 TESTED IN MODEL
 SCALE 1" = 400'
 (COPIED FROM DRG. NO CEN 112271
 24-F)

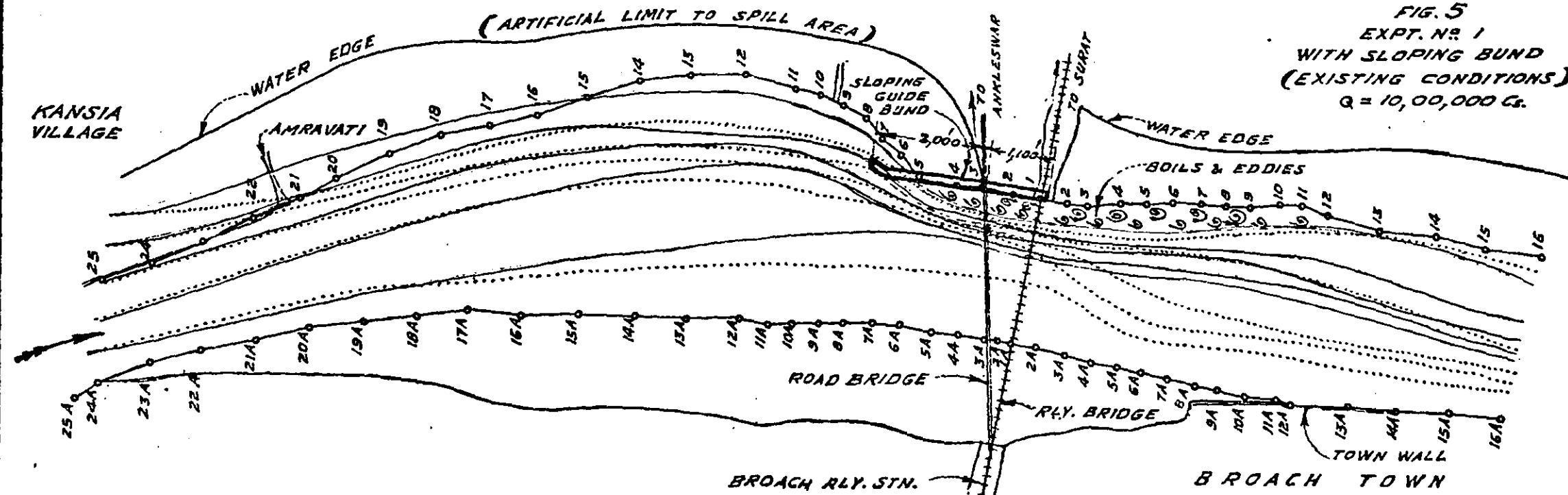


FIG. 5
EXPT. NO. 1
WITH SLOPING BUND
(EXISTING CONDITIONS)
Q = 10,00,000 C.S.

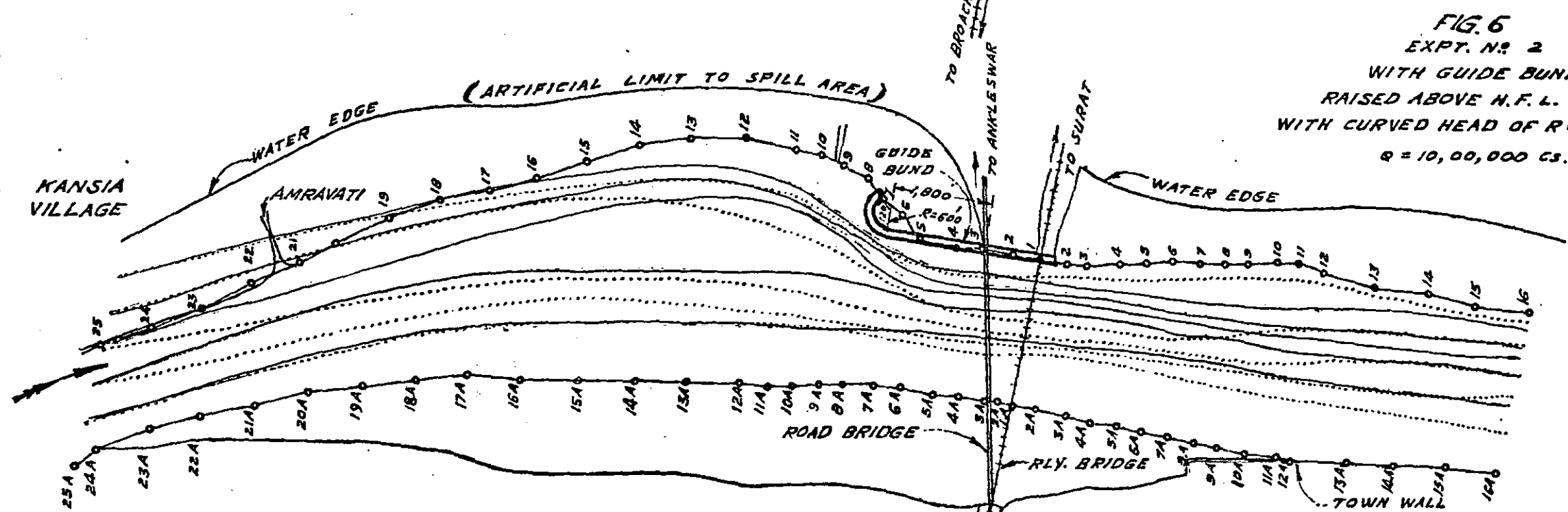


FIG. 6
EXPT. NO. 2
WITH GUIDE BUND
RAISED ABOVE H.F.L. AND
WITH CURVED HEAD OF R = 600 FT.
Q = 10,00,000 C.S.

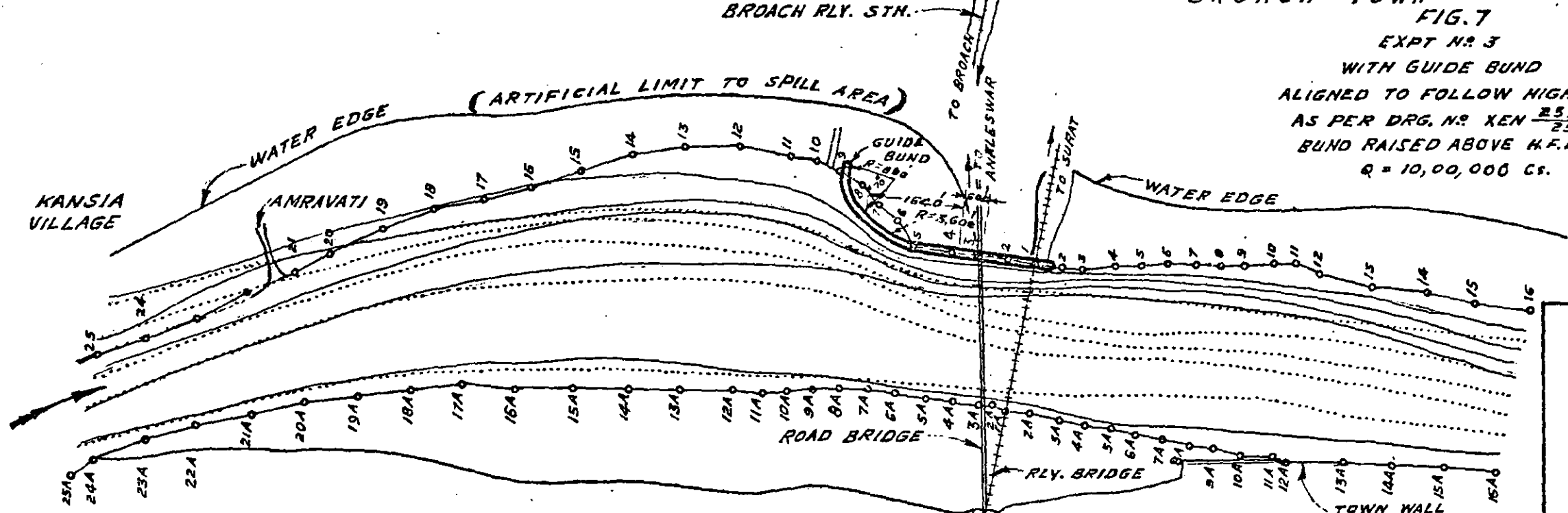


FIG. 7
EXPT. NO. 3
WITH GUIDE BUND
ALIGNED TO FOLLOW HIGH BANK
AS PER DRG. NO. XEN $\frac{25745}{25-F}$
BUND RAISED ABOVE H.F.L.
Q = 10,00,000 C.S.

REFERENCES :-
 SURFACE LINES OF FLOW —————
 BED LINES OF FLOW
 WATER EDGE —————
 SURVEY CHAIN LINE }
 AND STATIONS } ————○———

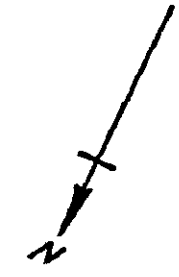


FIG. 5-7
 $\frac{1}{330}, \frac{1}{50}$ SCALE MODEL OF
 NARBADA RIVER
 AT
 BROACH
 SHOWING LINES OF FLOW
 SCALE 1" = 2,640'

COMP. BY - *Maf*
 TRACED BY - G.R. PATKIE
 1-2-1952.

ITEM NO. 3
A. R. T. 1951

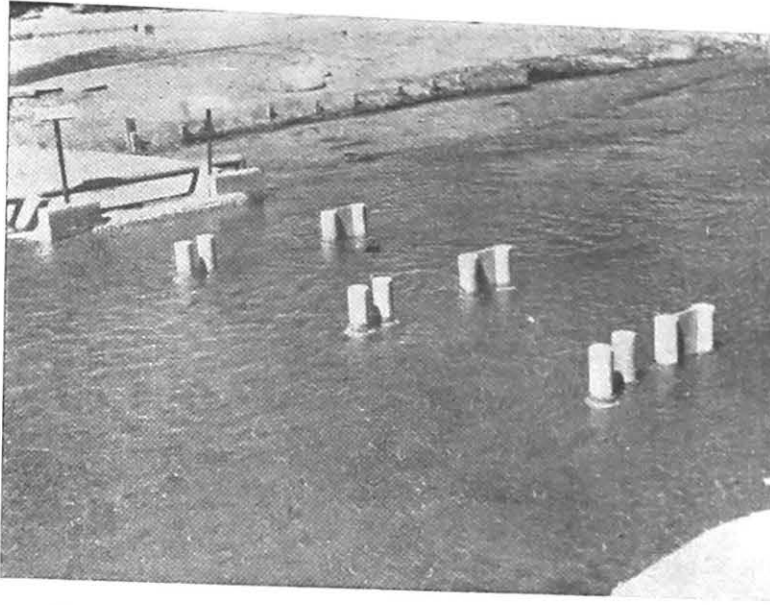


Photo No. 6997—showing the piers of road and railway bridges in the model.

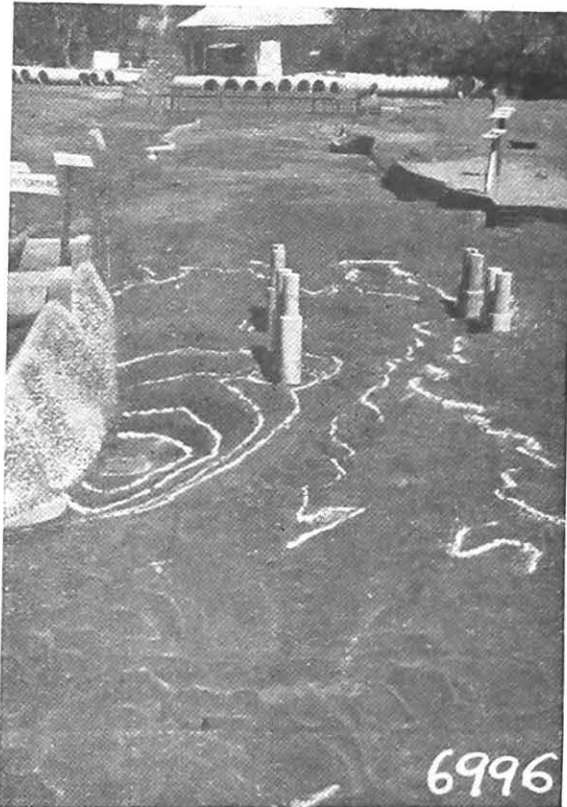


Photo No. 6996 Looking downstream showing scour and its contour's round the sloping guide bund—Experiment 1.

53. In experiment No. 2 with the present length (2,000 feet) of the guide bund raised above H. F. L. and provided with a curved head of 600 feet radius abutting the high bank, the eddies which were caused by the overflow from over the bund in experiment No. 1 were absent. The main river flow still remained along the south bank. As the major danger to the bridges and the south bank in its vicinity is due to the formation and persistence of pot-holes which deepen and consequently approach the bridge piers and the abutments, raising of the guide bund above H. F. L. would certainly minimise that danger which exists at present. But a proper design of the guide bund appeared necessary for effective control of the river at the bridges. Fig. 6 Photo 7014.

54. Experiment No. 3, with the guide bund aligned to follow the high bank for reasons of economy, showed no improvement over experiment No. 2 as the main river flow continued to attack the south bank upstream and downstream of the bridges, though it was smoothly guided along the bund at the bridges. The intention in testing this alignment was to shift the guide bund towards the high bank to avoid deep region around the sloping bund. Fig. 7.

55. Experiment No. 4 was conducted with the guide bund extended upstream by 500 feet along the existing alignment and provided with a curved head of radius 800 feet. This length was found insufficient for effecting a sufficient throw-off towards the middle of the bridge as the main flow still attacked the south bank upstream and downstream of the bridges with quiescent conditions along the bund only. Fig. 8.

56. In experiment No. 5 the guide bund was further extended upstream by 500 ft. along the present alignment total extension=1,000 feet and provided with a curved head of radius 800 feet. This design gave very satisfactory results. The main river flow was gradually deflected away from the south bank far upstream of the guide bund creating very quiescent region along the south bank. The flow was further deflected to the centre of the bridges from the curved head of the guide bund and continued to flow down centrally. The scour conditions at the bridges had very much improved and the velocity distribution tended to be uniform at the bridges. The river was thus controlled effectively at the bridges. Fig. 9.

57. The next experiment No. 6 was with a guide bund aligned as per Drawing No. CEN 111425/24-F, communicated by the Chief Engineer, Western Railway. This alignment was a compromise between alignment in experiment Nos. 3 and 5. In this case the main river flow hugged the south bank upstream of the guide bund and was suddenly deflected to the middle of the river by the curved head. It continued to flow down centrally through and downstream of the bridges. Fig. 10. Photo 64.

58. Experiment Nos. 7, 8 and 9 were made with the guide bund of length shorter than 2,000 feet, raised above H. F. L. and having suitable curved head. In all these experiments the only advantage was the disappearance of eddies and partial filling up of pot-holes. They, however, were not effective in controlling the river. As the economic aspect overruled the long-term object of fully controlling the river by a guide bund as in experiment No. 5, the short-term objective of checking the pot-holes by removing the eddies could very well be achieved by a guide bund design adopted in these tests. Figs. 11, 12, & 13.

6. Recommendations

The guide bund as in experiment No. 5, viz, extended by 1,000 feet upstream along the present alignment with a mole-head of radius 800 feet was very effective, in the model, in controlling the river at the bridges. However, if consideration of cost compels to adopt an economic design, then the guide bund as in experiment No. 8, viz, the existing bund raised above H. F. L., having a curved head of radius 600 feet and length reduced by 160 feet, i.e., to 1,640 feet, would serve the purpose. By raising the guide bund above H. F. L. and giving it a suitable mole-head, the cross-flow and subsequent eddy formation would be prevented and the pot-holes would silt up.

4. TRAINING OF WATRAK AT KAIRA (BOMBAY STATE)

1. History

Fig. 1. Photo, 421. The Watrak River flows in an alluvial bed of fine sand ($m=0.15$ mm.). The river takes a very sharp—almost re-entrant—bend from about one mile to 3 furlongs upstream of Kaira Town. For many years in the past the left bank along the bend is under erosion. The earlier erosion appears to have been slower and steadier on account of less sharp curvature than at present; and also because the river bank partially consisted of hard material of a mixture of sand and stiff clay. During the past 20 years or so the rate of erosion has been alarming and the bank has receded 300 to 400 feet at places.

Many attempts were made in the past to protect this bank. After the floods of 1927, four groynes of teak piles filled with sand bags and brushwood were constructed along the concave bank just above the town. Two of these were washed away in the floods of 1930; and the remaining two along with a substantial portion of brick pitching protecting the provincial road, together with a part of the road itself were washed away piecemeal subsequently.

In early 1938, a groyne 138 feet long consisting of R. C. C. piles filled with sand bags was constructed along the left bank just upstream of the bend. This 'Baxter' groyne afforded very little protection and was subsequently dismantled as it induced severe embayment downstream.

As the situation became serious and as there was a grave danger of the river damaging the north-west corner of the old Fort Wall, which it did later in July 1950, the case was referred to this Research Station for advice; model experiments were carried out¹ and three solutions were suggested—

- (1) A bridge—submersible or high level,—projecting from the north-west corner of the town wall pointing 30° upstream, the bridge portion starting 500 feet from the town wall with a clear linear waterway of 300 feet;
- (2) a groyne or spur 180 feet long sloping down at 1 in 5 from the north-west corner of the town wall pointing upstream at 35°, and
- (3) to deepen and strengthen the north-west corner of the town wall by pitching.

The third solution, being cheapest, was adopted.

Photos 424 to 427. From 1940 onwards further trouble cropped up. Though the pitching protected the N W Corner of the Fort Wall, it pegged down the river permanently at this point while the erosion of left bank upstream continued unabated. In 1942 a major change occurred in the river course downstream. After leaving the N. W. Corner the river swung over to the right bank downstream due to the development of a very sharp curvature upstream. At this time the construction of a R. C. C. bow string girder bridge was in progress and the newly formed right bank cold weather channel had to be blocked by an embankment, with the idea of forcing the river through the bridge. Actually, this resulted in a breach of the approach bank and the bridge was almost outflanked. The approach bank (in 1944) had then to be very strongly armoured with pitching on the slopes and a heavy apron at the toe had to be provided at a considerable cost. This approach bank distorted the river curvature considerably with the result that the right bank along Harijala Village upstream was severely eroded and an avulsion of the river into the low lying area of the village was apprehended.

Fig. 1. Photo, 6983. Once again the Research Station was approached for advice. In December 1946, in view of the urgency of the problem, a temporary solution for protecting the Harijala Bank with a series of permeable (timber) pile groynes or with tree groynes—which would not conflict with the final solution as indicated by future model experiments—was recommended. The three-tree groynes were subsequently constructed; induced silting along the Harijala Bank and thus saved the village for the time being.

¹ CHIR Annual Report for 1937-38 (p. 77) and 1938-39 (p. 16)

ITEM NO. 3.
A. R. T. 1951.

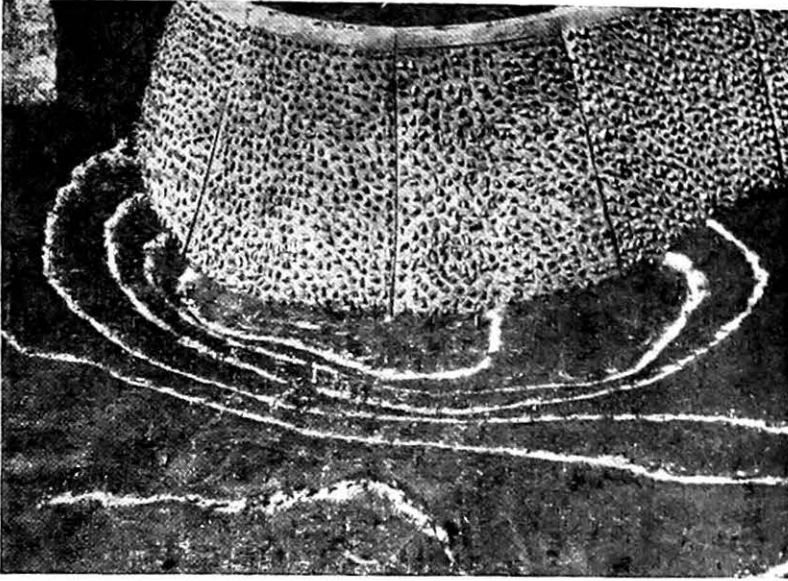


Photo No. 7014-Showing deep scour and its contours round the curved head of the guide bund raised above H. F. L. with its present length—Experiment 2.

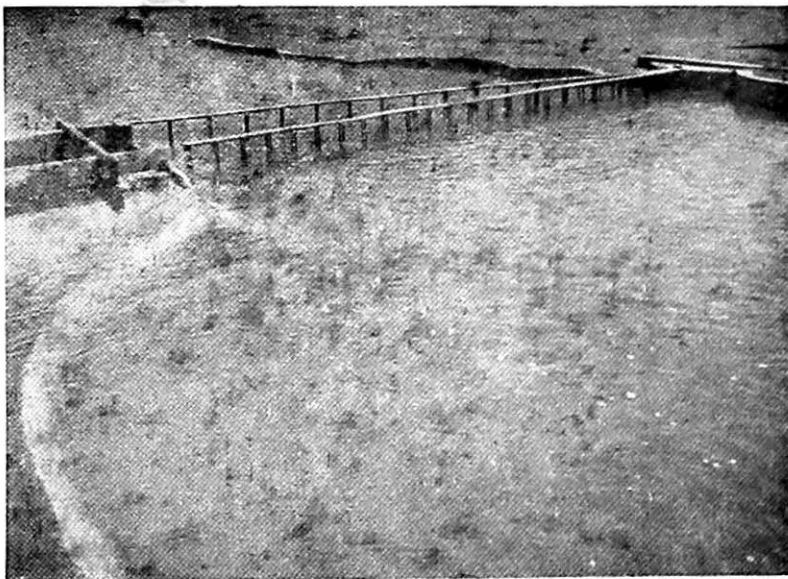


Photo No. 563-Looking downstream showing the full number of piers of road and railway bridge reproduced in the model.

FIG. 8
EXPT. NO. 4

WITH GUIDE BUND EXTENDED BY 500 FT.
AND WITH A CURVED HEAD $R = 800$ FT. $\angle 120^\circ$
BUND RAISED ABOVE H.F.L.
 $Q = 10,00,000$ Cs.

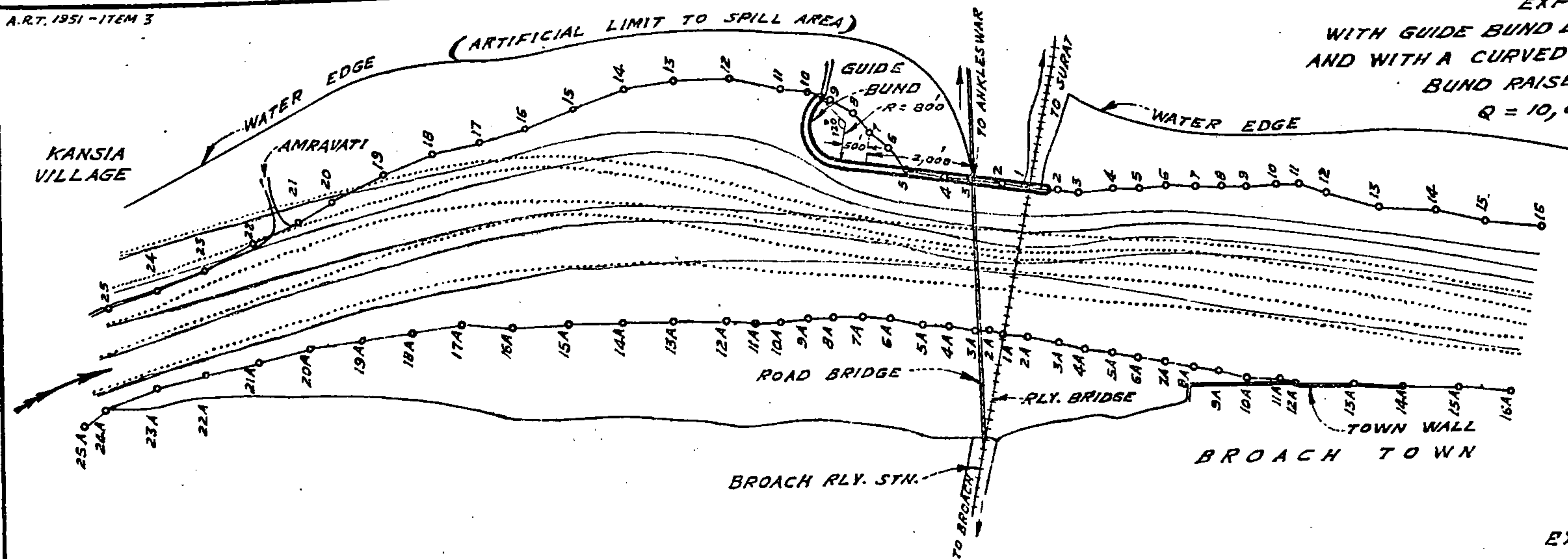
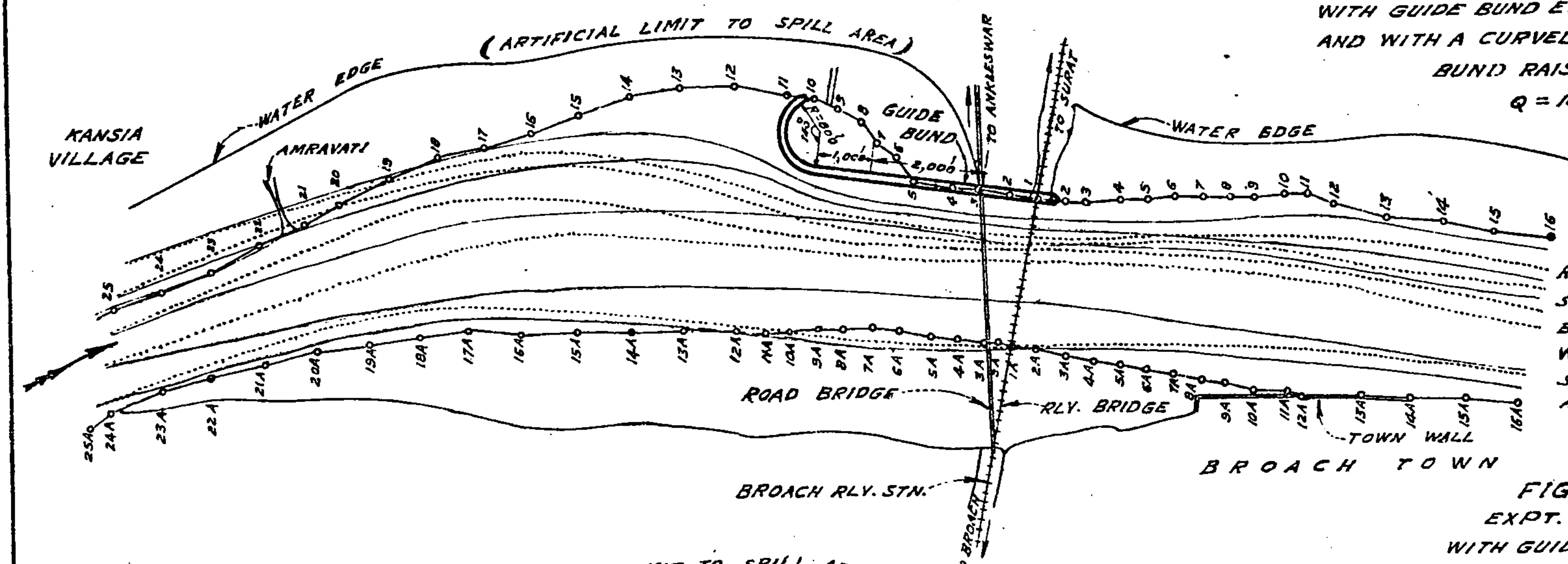


FIG. 9
EXPT. NO. 5

WITH GUIDE BUND EXTENDED BY 1,000 FEET
AND WITH A CURVED HEAD $R = 800$ FT. $\angle 140^\circ$
BUND RAISED ABOVE H.F.L.
 $Q = 10,00,000$ Cs.



- REFERENCES :-
- SURFACE LINES OF FLOW ————
 - BED LINES OF FLOW (dotted)
 - WATER EDGE ———— (dashed)
 - SURVEY CHAIN LINE AND STATIONS ———— (with dots)

FIG. 10
EXPT. NO. 6

WITH GUIDE BUND
ALIGNED AS PER DRG. NO. CEN ¹¹¹⁴²⁵/_{24-F}
BUND RAISED ABOVE H.F.L.
 $Q = 10,00,000$ Cs.

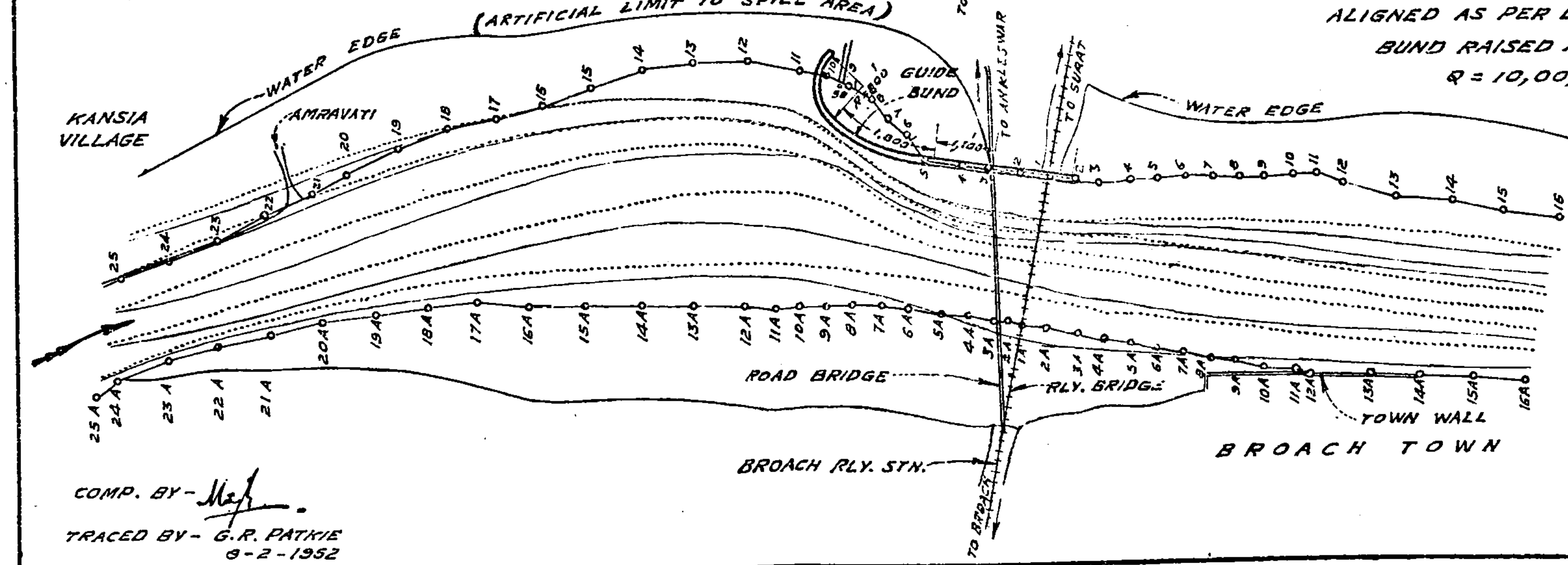
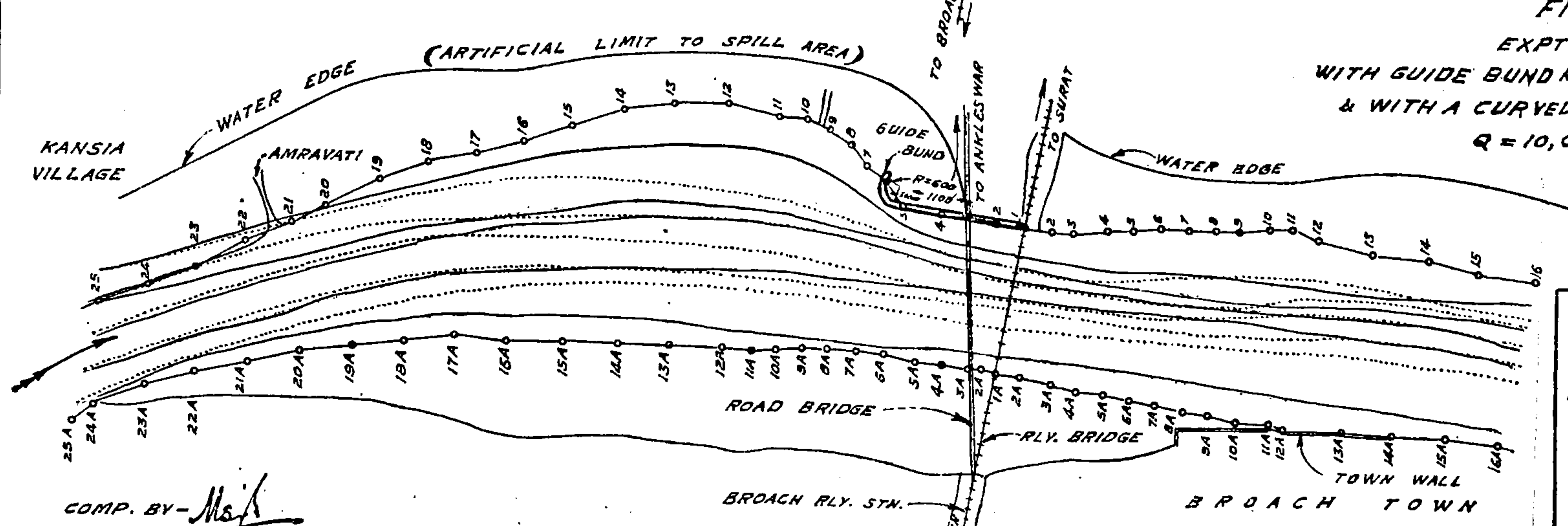
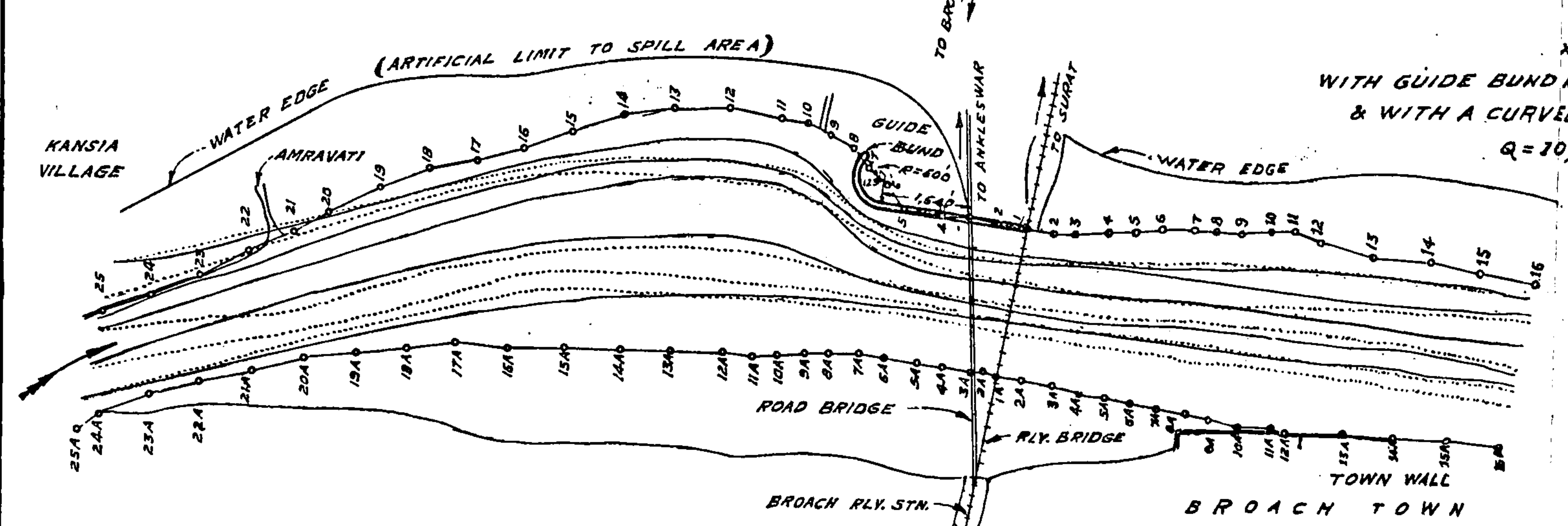
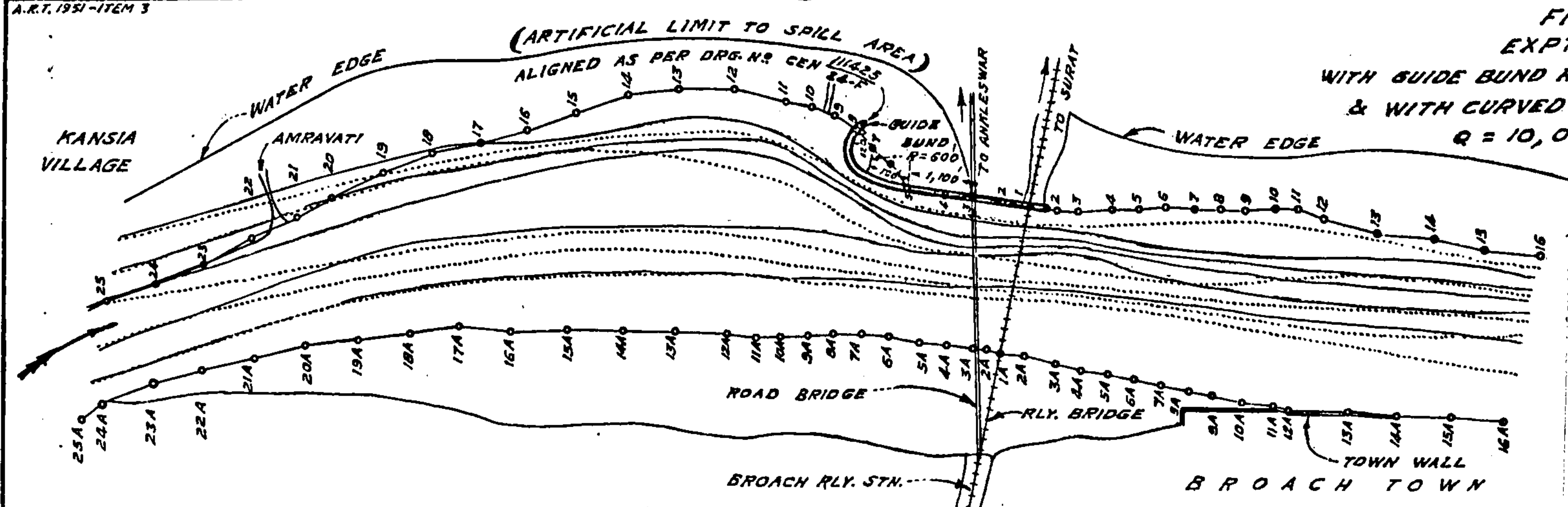


FIG. 8-10
 $1/330, 1/50$ SCALE MODEL OF
NARBADA RIVER
AT
BROACH
SHOWING LINES OF FLOW
SCALE $1" = 2,640'$

COMP. BY - *M.P.*
TRACED BY - G.R. PATKIE
3-2-1952



REFERENCES:
 SURFACE LINES OF FLOW ———
 BED LINES OF FLOW - - - - -
 WATER EDGE ———
 SURVEY CHAIN LINE }
 AND STATIONS. ———

FIG. 11-13
 1/330, 1/50 SCALE MODEL OF
 NARBADA RIVER
 AT
 BROACH
 SHOWING LINES OF FLOW
 SCALE 1" = 2,640'

COMP. BY - *Maj*
 TRACED BY - G.R. PATKIE.
 12-2-1952.

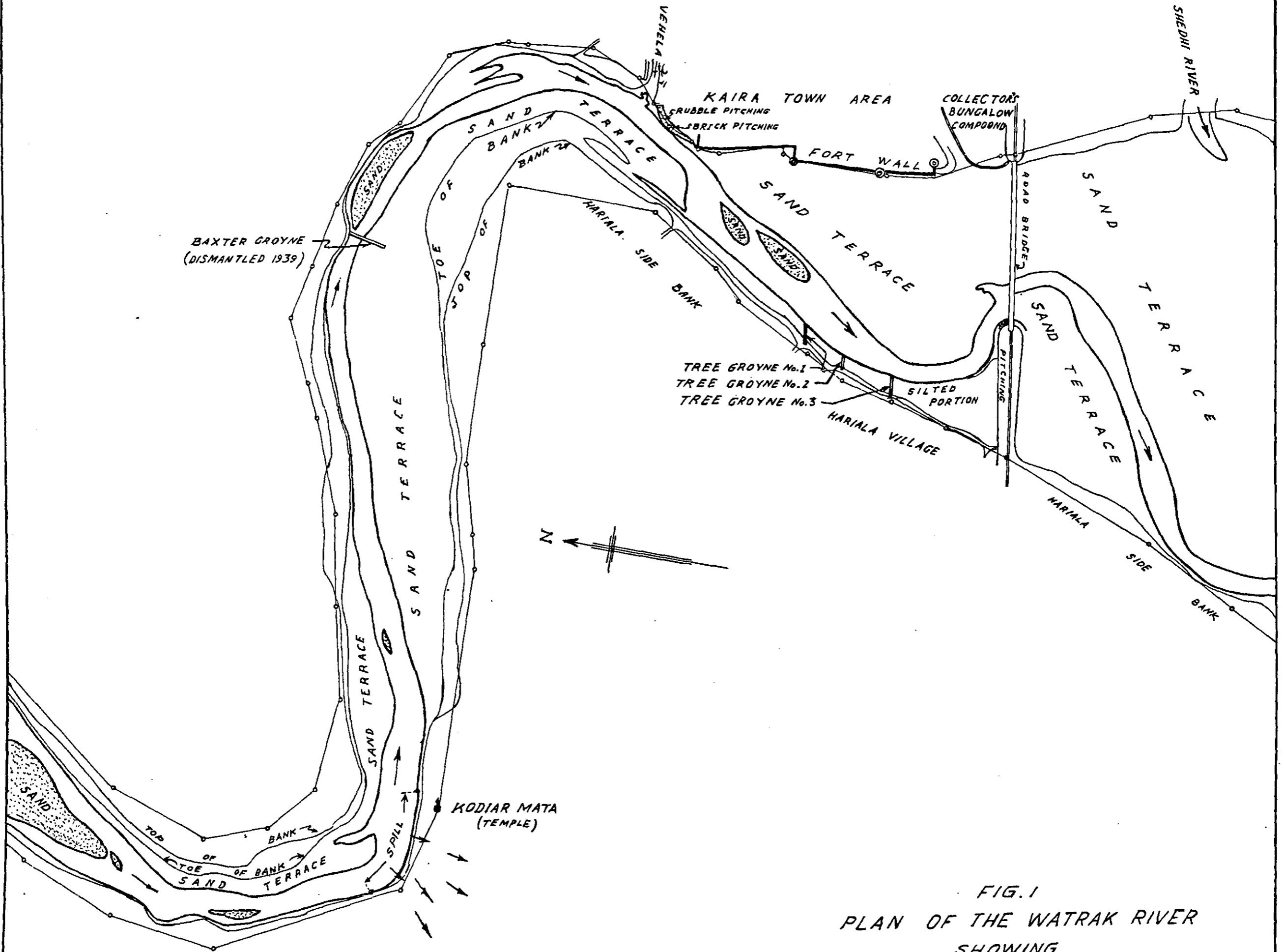


FIG. 1
 PLAN OF THE WATRAK RIVER
 SHOWING
 REACH - 2 1/2 MILES U/S AND 1/2 MILE D/S
 OF THE ROAD BRIDGE AT KAIRA TOWN
 1951
 SCALE. 1" = 750 FT.

COMP. BY: *M. S. Nigam*
 TRACED BY: R. T. THAKOR.
 19-3-1952

THE WATRAK RIVER AT KAIRA — Bombay State.

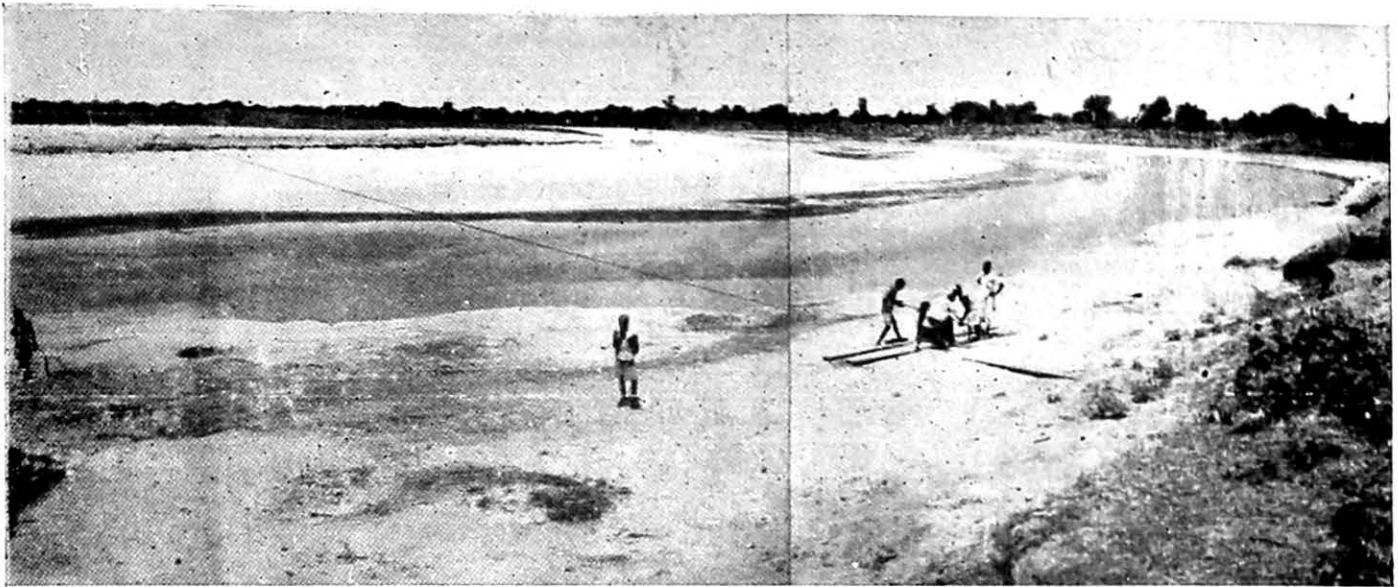


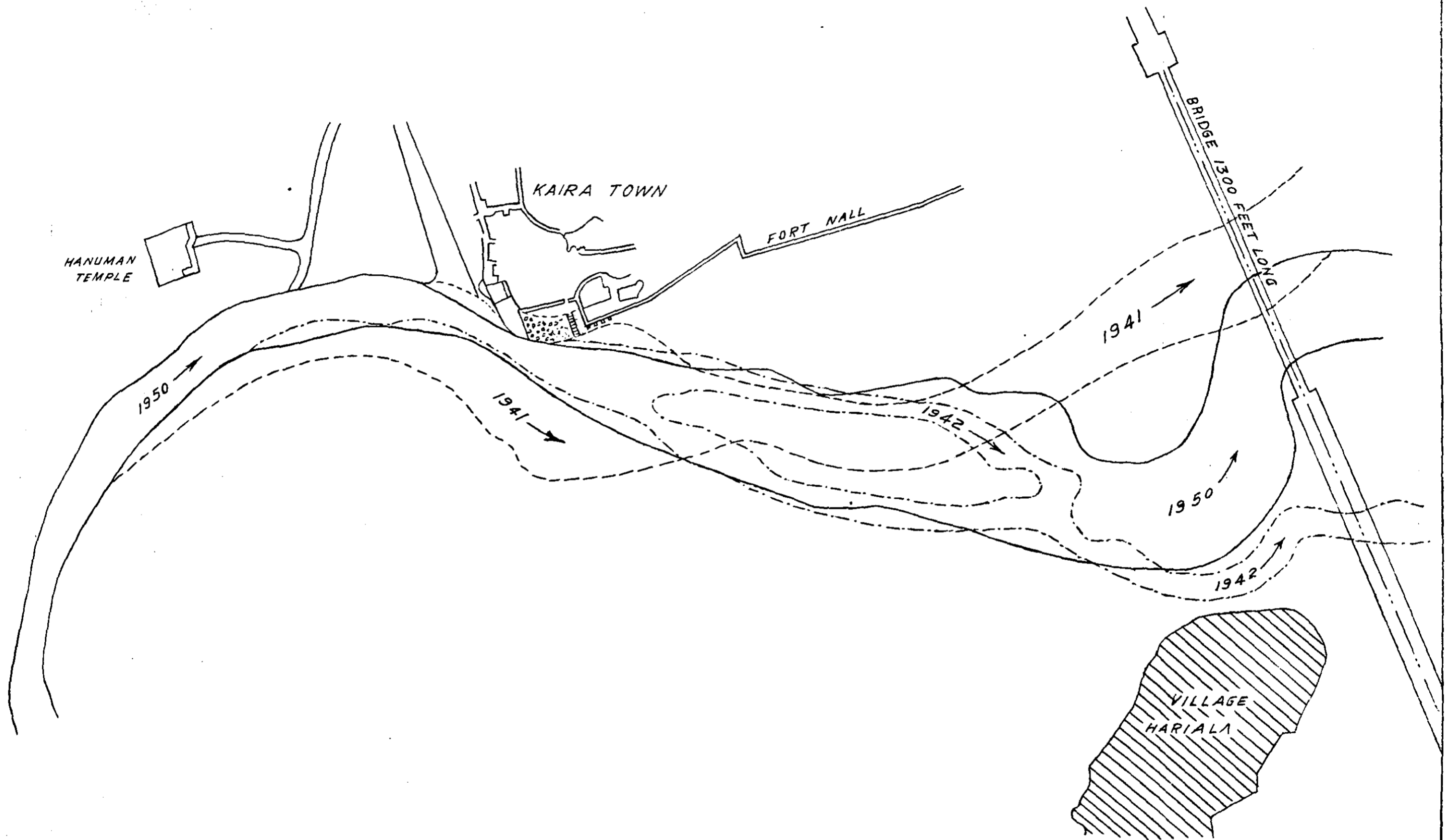
Photo Nos. 420, 421 : Looking upstream showing the sharp river bend upstream of Kaira Town.



Photo No. 422 The left bank, along the bend, under erosion with Hanuman Temple on a hillock in the background.



Photo No. 423 : The damaged part of pitching at the North West Corner of the fort wall.



REFERENCE:-

- 1941 RIVER COURSE.....
- 1942 RIVER COURSE.....
- 1950 RIVER COURSE.....

FIG: 2
 PLAN OF WATRAK RIVER
 AT KAIRA
 SHOWING
 EROSION OF BANKS
 SINCE 1931.
 SCALE: 1" = 400 FT.

COMPARED BY: *Munasaft*
 TRACED BY: M. H. SABNIS, 5-4-52

ITEM No. 4.
A. R. T 1951.

THE WATRAK RIVER AT KAIRA — Bombay State.



Photo Nos: 424 427 Looking upstream of the Road Bridge showing Hariala Village, tree groynes and the river channel along right bank; fort wall and Kaira Town on left bank.
Note parallel flow of the river along approach embankment seen in foreground.

The watrak River at Kaira (Bombay State)

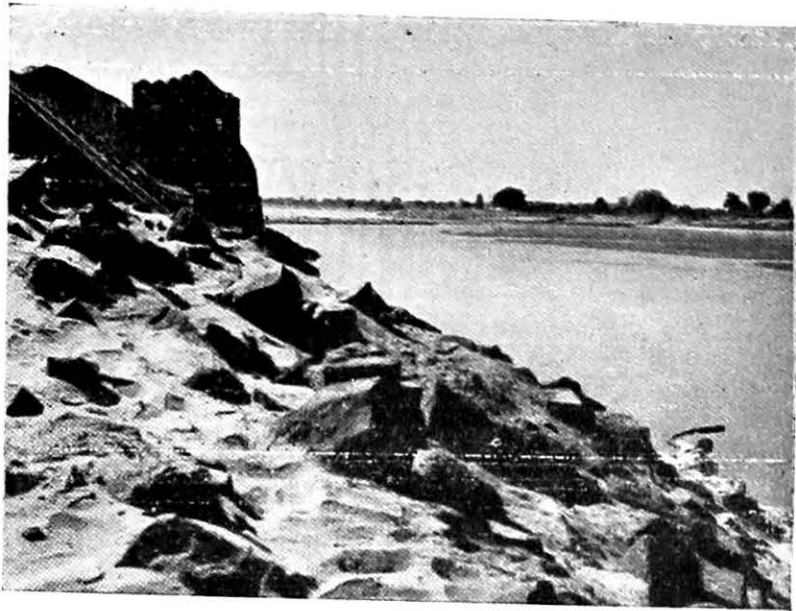


Photo No. 6847 : North - west Corner of the Fort wall showing damage to the pitching.

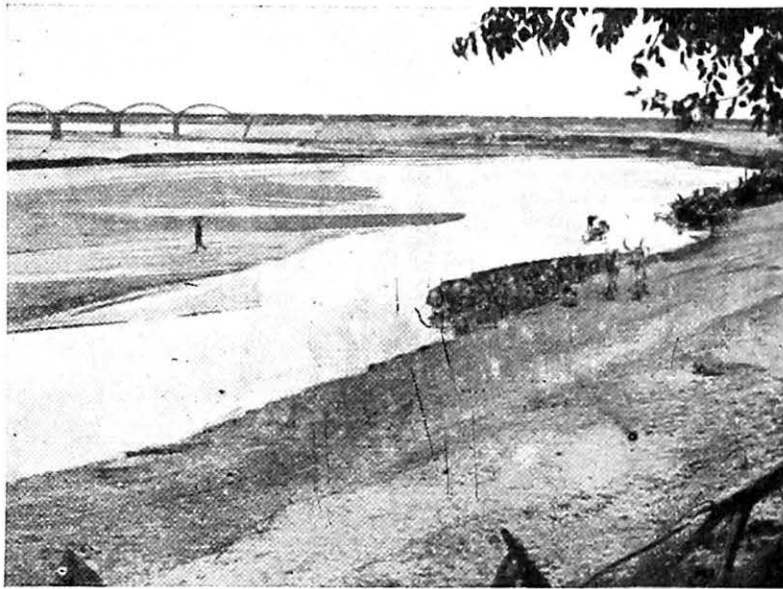


Photo No. 6983 - Looking downstream showing skew approach of the river to the Road Bridge. A tree groyne is visible in the right foreground.

1/125, 1:30 Scale Model of watrak River at Kaira.

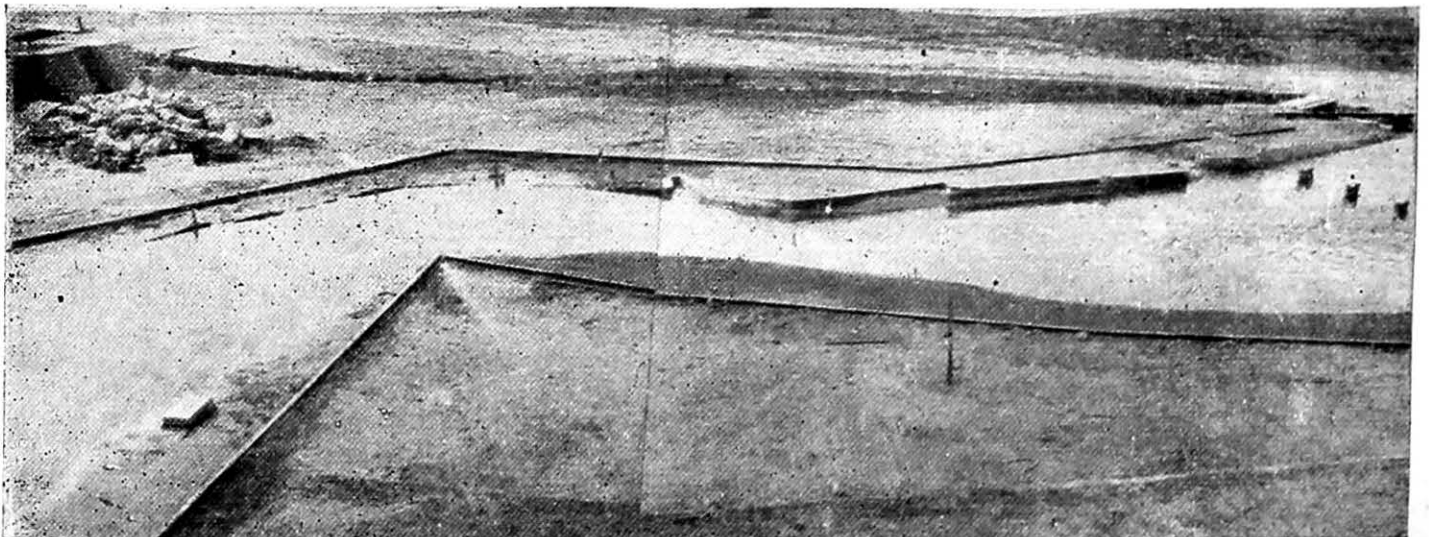


Photo Nos. 395, 396 : The acute river bend upstream of Kaira Town as reproduced in the model. The town wall is seen in the right background.

The situation today, more or less remains the same. The river attack on the left bank upstream of Kaira Town and at the N. W. Corner as also on the right bank along Hariala Village and the approach embankment continues unabated. It has, therefore, been decided to conduct model experiments for evolving long term training measures for protection of Kaira Town, Hariala Village and the Road Bridge.

2. Hydraulic Data

2.1 *River Discharge*—No flood data is available except for the yearly highest flood levels recorded at Kaira Town; information in respect of flood hydrographs also is not available.

The Station Estimate of the highest flood discharge based on the Lacey Formula $W=2.67 \sqrt{Q}$ worked out to 1,00,000 cusecs.

2.2 *River Slope*—River slopes during high floods were worked out from yearly high flood levels recorded on the Railway Bridge at Mehmedabad and the Kaira Road Bridge, at a distance of 10.7 miles along the river.

Year	W.L. at	W.L. at Kaira		Slope ft./mile
	Mehmedabad	Bridge	Site	
1927	107.81	87.04	1.93	
1941	101.01	83.50	1.73	„
1942*	95.56	81.00	1.36	„
1944	101.81	81.50	1.89	„
1950	108.81	88.50	2.07	„

Between the two bridges the Meshwa River joins the Watrak. Assuming that it is also simultaneously in floods and the backwater reaches the Railway Bridge at Mehmedabad Slope of 1.73 ft /mile has been adopted in the model for the dominant discharge.

2.3 *River Bed Material*—The Watrak flows in fine incoherent alluvial sand of average size 0.15 mm. diameter.

3. Model Scales

- Length Scale = 1:125
- Depth Scale = 1:30
- .. V. E. = 4.17
- Initially S. E. = V. E.
- Time Scale=1 day in proto=2.1 min. in model.
- Discharge Scale = 1:20,550

4. Experiments

The following experiments were conducted—

Experiment No. 1—With existing conditions of the river for ascertaining the extent of attack on the left bank along the bend and the right bank near Hariala Village and at the bridge.

Experiment No. 2—With Bell’s Bund located at the right abutment, length of the bund being 900 and 600 feet, tested in two different experiments.

Experiment No. 3—With a repelling spur on the right bank near Hariala Village, the optimum length and alignment to be determined experimentally.

* Road Bridge at Kaira was built in 1942

Experiment No. 4—Experiments with a spur at the N. W. Corner of the town wall—length of the spur 250, 150 and 100 feet, its alignment to be determined experimentally. (Preliminary experiments with a spur at various points near N. W. Corner and along the left concave bank upstream showed that only a spur at N. W. Corner would be effective in repelling the flow. The preliminary experiments also showed that—a series of repelling spurs along the bend on the left concave bank were not only ineffective, but caused considerably afflux)

5. Experimental Results

5.1. In the absence of discharge hydrographs, an arbitrary discharge cycle was run in the model. It was found that when the flood discharge was about 60,000 cusecs, viz., the dominant stage, the main flow conformed to the existing deep channel in the river. Beyond this discharge stage, downstream of the N. W. Corner, the tendency was to straighten out leaving the tortuous deep channel course. However, the river bend upstream of Kaira Town is so acute that at all stages of flood discharge the main flow followed the deep channel on the outer side of the bend. As the normal high flood range is within 60,000 and 70,000 cusecs the experimental observations were recorded for this dominant discharge.

Fig 3
Photo
395,396

Fig 3

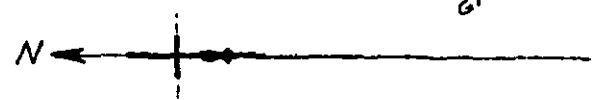
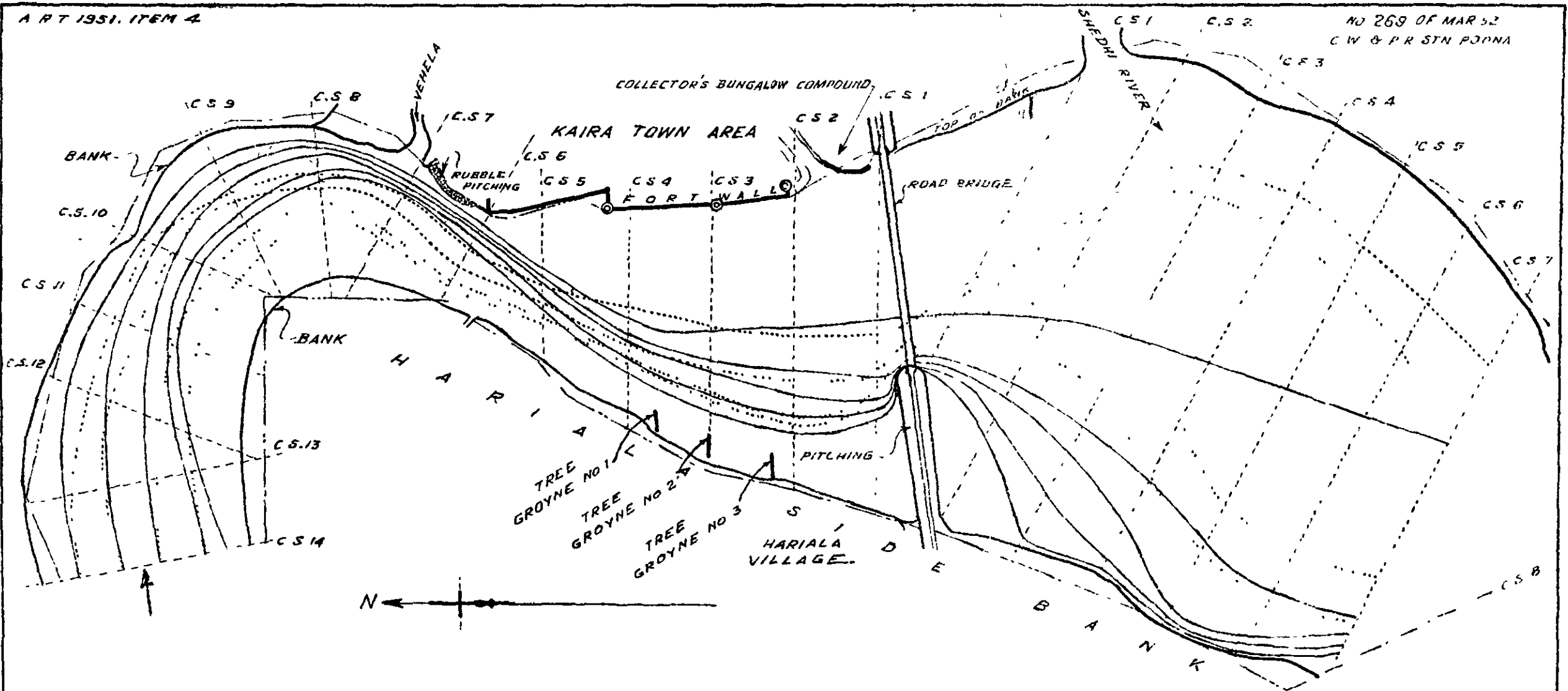
5.2 *Experiment No 1* was conducted with existing conditions of the river to test the extent of attack on the left bank along the bend upstream of Kaira Town and on the right bank along Harijala Village downstream. The lines of flow indicate how correctly the river behaviour was reproduced in the model. Along the upstream bend, the discharge concentration was along the left bank until all surface lines converged closely at the N. W. Corner pitching and bed lines were pushed out on the opposite shoal. From this point downstream the main flow suddenly swung to the right bank attacking the right bank along Harijala Village and then flowing almost parallel to the right approach embankment passed through the bridge with almost 70 per cent discharge concentrating in the first four spans of the bridge, creating heavy scour round the right abutment. Downstream of the bridge the flow again swung to the right bank. The velocity observations also showed that higher velocities occurred on the left bank along the bend as also on the right bank near Harijala Village. It may be noted that the brick pitching at the N. W. Corner and the right approach embankment are acting as attracting spurs and have the river pegged down to them. In the model, the scour-depth round the right abutment of the bridge was about 2.25 D/Lacey in the model. The extent of scour hole was thus relatively very large due to natural angle of repose of the bed material—one of the model limitations. To offset this effect, the scour hole was restricted artificially to 1.5 D/Lacey in further experiments.

Fig 4
Photo 47C.

5.3. *Experiment No. 2* was conducted with guide bund located at the right abutment. The lengths of the bund tested were (a) 900 feet and (b) 600 feet from the abutment to the tangent along the curved head. In both tests, the flow upstream of the N. W. Corner remained unaltered indicating that the guide bund had no influence on rectifying the acute river curvature so far upstream. Conditions on the right bank upstream of the mole-head improved as there was slack flow. The discharge concentrating near the mole-head was smoothly guided downstream along the guide bund; the discharge concentration still remained heavy as before in the first four spans of the bridge. This meant strong armour of boulded apron all along the length of the guide bund. In case of the shorter guide bund (600 ft) there was some return flow behind it. The guide bunds were effective in affording local protection to the right bank near Harijala but did not have the river under control at the bridge. There was also no reduction of attack on the left bank along the bend upstream of Kaira.

Fig. 5
Photo 868.

5.4. *Experiment No. 3* with a repelling spur along the right bank near Harijala Village—Two experiments were conducted with the repelling spur (375 feet long) located at tree groyne No. 1 and (450 feet long) located at tree groyne No. 2. It was expected that by repelling the flow away from the right bank the river might swing over to the shoal and distribute the discharge evenly through the bridge by reducing the existing concentration at the right abutment; besides a change in river curvature downstream would ease the acute curvature upstream. The results of these tests were, however, not satisfactory. The spur repelled the flow towards the shoal to a certain extent but again downstream there was a swing to the right abutment. There was no noticeable change in the flow around the upstream bend.



REFERENCES.-

- SURFACE LINES OF FLOW.....
- BED LINES OF FLOW.....

FIG. 3
 1/25, 1/30 SCALE MODEL OF WATRAK RIVER
 AT KAIRA
 EXPERIMENT NO. 1 - LINES OF FLOW
 WITH EXISTING CONDITIONS OF RIVER
 Q = 60,000 CU SECS
 SCALE. 1" = 750'

COMP BY *M. S. J.*
 TRACED BY JAPKAL
 13 MAR 1952

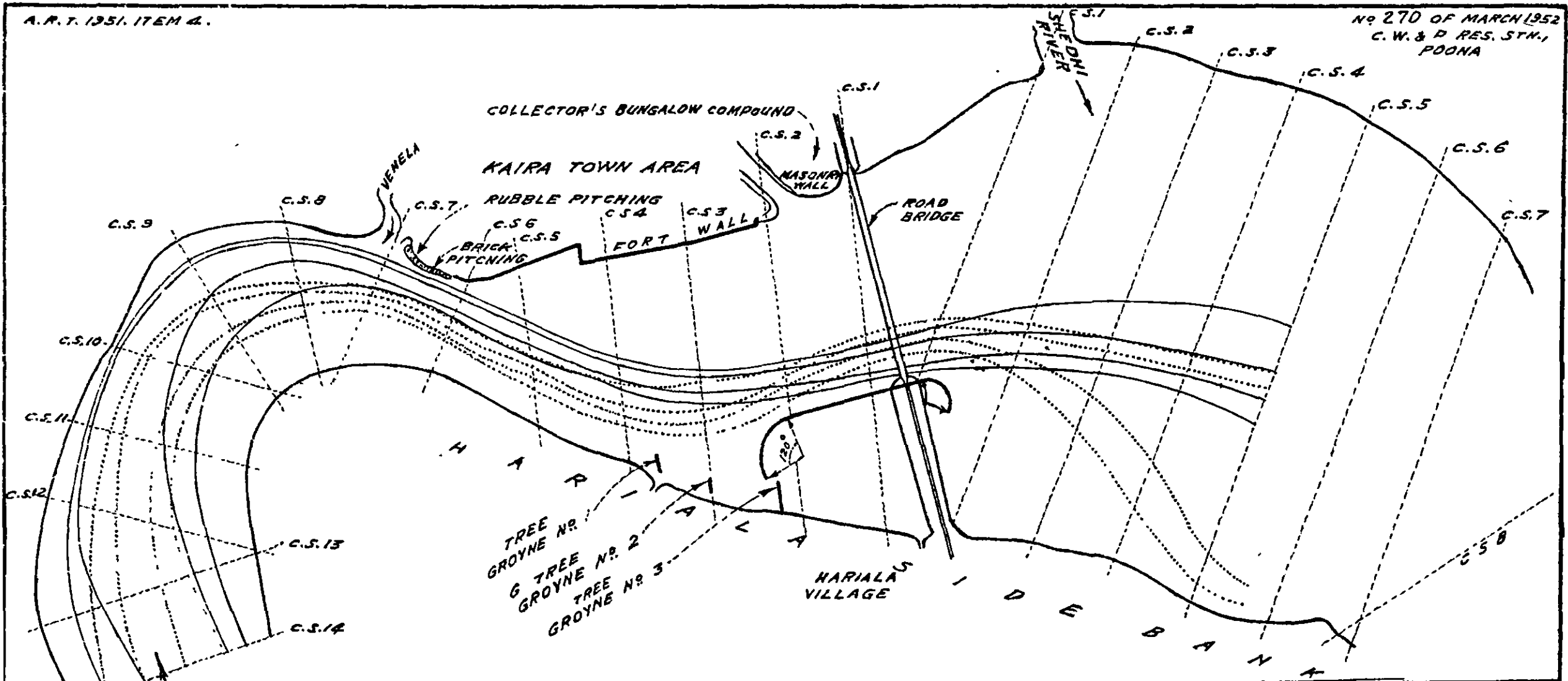


FIG. 4
 1/125, 1/30 SCALE MODEL OF WATRAK RIVER
 AT KAIRA
 EXPERIMENT No 2 (a) LINES OF FLOW
 WITH GUIDE BUND 900 FEET LONG
 Q = 60,000 cfs.
 SCALE 1" = 750 FT.

REFERENCES: —
 SURFACE LINES OF FLOW —————
 BED LINES OF FLOW - - - - -

COMP. BY - *M. Singh*

TRACED BY - G. R. PATKIE
 14-3-1952.

1/125, 1/30 Scale Model of Watrak River at Kaira.

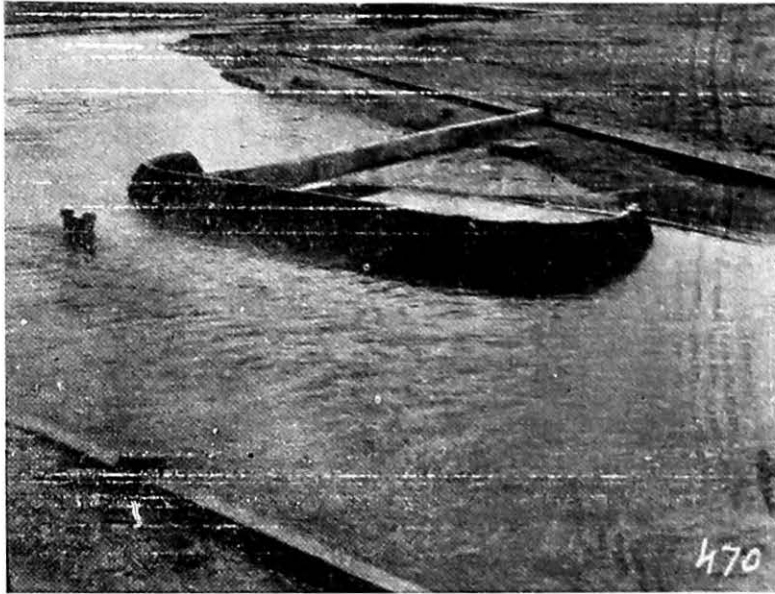


Photo No. 470 : The guide bund 600 feet long located at the right abutment.

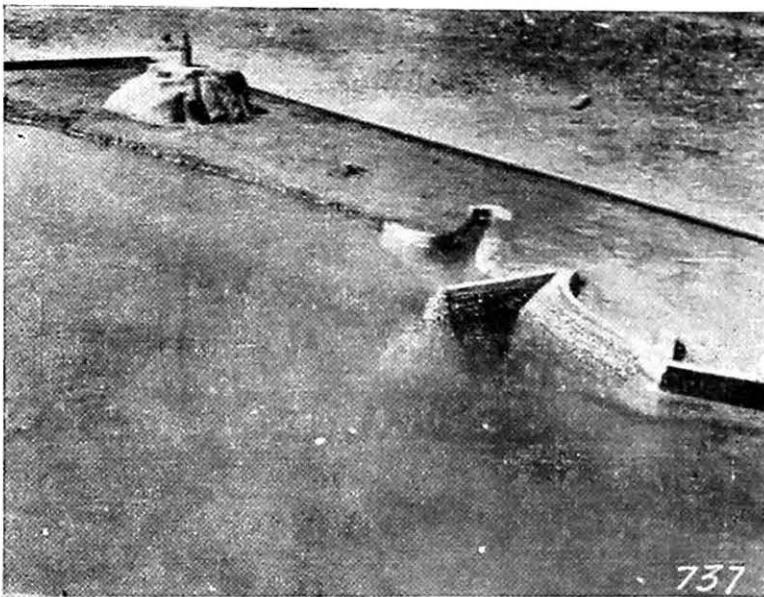


Photo No. 737 - The spur 150 ft. long located at the N. W. Corner. The Hanuman Temple is seen on the left bank upstream of the spur.

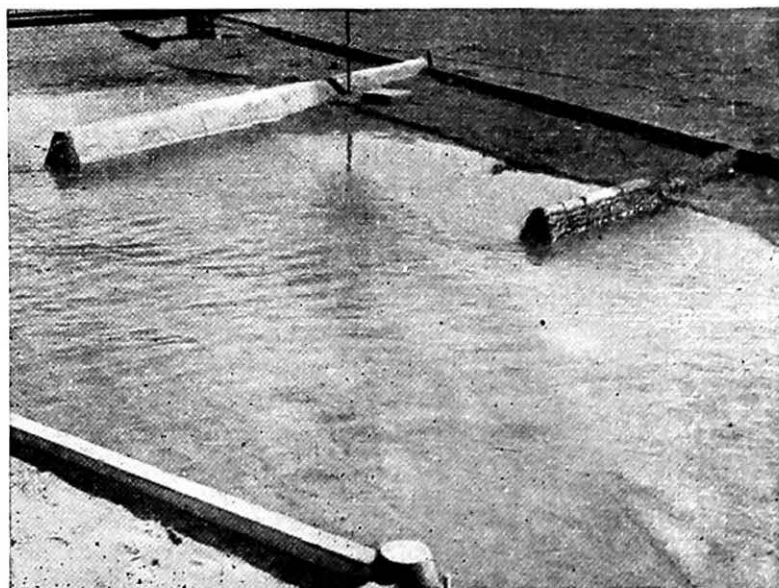


Photo No : 868 : A repelling spur near Hariala Bank seen parallel to the right side approach embankment. The fort wall is seen in the foreground.

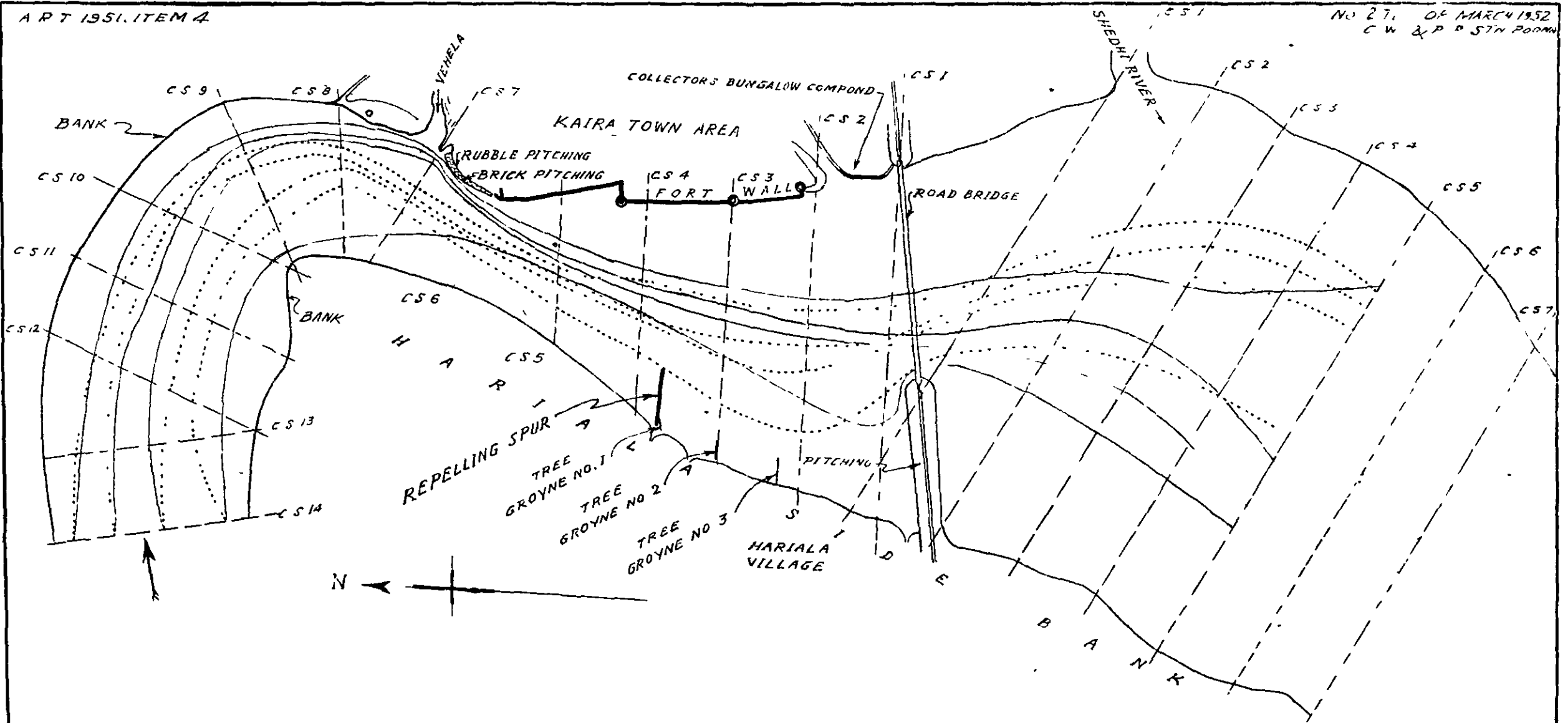


FIG. 5

1/125, 1/30 SCALE MODEL OF WATRAK RIVER
AT KAIRA

EXPERIMENT NO. 3 (U) LINES OF FLOW
WITH REPELLING SPUR ON RIGHT BANK

Q = 60.000 CU SECS
SCALE 1" = 750'

- REFERENCES --
- SURFACE LINES OF FLOW —————
 - BED LINES OF FLOW - - - - -

COMP BY. *M. S. J.*
TRACED BY THAKOR R. T.

5.5. *Experiment No. 4* with a spur near N. W. Corner of the Fort Wall, lengths 250, 150 and 100 feet.

Experiment No. 4(a) with 250 feet long spur located at the brick pitching near the N. W. Corner, the flow along the left bank of the upstream bend was slack up to about 500 feet upstream. However, all the surface lines converged at the nose of the spur and then swung downstream to the left flowing across the shoal. The bed floats moved out to the right bank indicating that the deep channel might silt up. The turbulence round the mole-head was very heavy and it was the scoured out material which was moving towards the Harijala Right Bank. Once the scour-hole was fully established there might not occur bed movement towards the right bank to the extent of silting up the deep channel. There was considerable scouring over the shoal near the fort wall. However, the afflux was equivalent to 1'4" and owing to the constriction of waterway at the N. W. Corner the tendency to swing towards the right bank in the initial stage was very great with the result that the Harijala bank was attacked and there was a danger of the village being encircled by water.

Fig. 6.

Experiment No. 4(b) with 150 feet long spur. The flow lines and the velocity distribution indicated that the conditions along the left bank upstream were slack to an extent of only about 400 feet from the spur. The surface lines concentrated at the nose of the spur and swung to the left downstream to distribute the flow uniformly on the shoal. There was even scouring on the shoal and the flow was centralised. The afflux was equivalent to 8 inches. So far this was the only test giving good results simultaneously for the left bank upstream of Kaira and the right bank downstream although the extent to which the upstream left bank was protected was not enough. The spur was located in deep waters and there was heavy scour round it necessitating a heavy armour of boulder apron.

Fig. 7.

Experiment No. 4(c) 100 feet long spur. This spur was not effective in protecting the left bank upstream of Kaira and hence the conditions downstream remained unchanged. The previous experiment showed that about 150 feet was optimum length of the spur inducing the flow to swing towards the left downstream.

6. By the time the experiments described above were completed, the Superintending Engineer Gujarat Circle reported that the river had spilled once in 1927 and again in 1950 along the outside of the bend, two miles above Kaira. As this was vitally linked with the present problem, the experiments were discontinued and the site inspected once again.

The first flood occurred during the year 1927 when water level at the Kaira Bridge was 87.04. The next big flood occurred in 1950 and inundated about 2,000 acres of valuable cultivated land as shown in Figure 9. It is now feared that the spill channel may develop, in the course of time, into a large river stream.

Fig. 9.

In spite of the water level in 1950 at the Mehmabad railway bridge, being higher by 1 ft. than in 1927, the water level at the Kaira bridge was lower by about 1.5 ft. This shows that a large part of the discharge must have flowed through the spill at Kodiar Mata temple and that the spill discharge in 1950 was larger than in 1927. The inspection of the site disclosed two distinct zones of bank scour indicating the stages in which the spills enlarged, *vide*, Photo Nos 1139-41, that for the year 1950 being very much larger. As the spill took relatively sand-free surface water, the water which flowed through the Watrak must have been heavily sand-laden resulting in enormous growth of shoals on the convex side of the bend upstream of Kaira and near the left bank downstream of Kaira. This accentuated scour on the concave left bank above Kaira and caused very extensive bank erosion. It would be interesting to note that all attempts to protect the left bank with groynes since 1923 failed. The inspection disclosed that unless more water is sent down through the Watrak during heavy floods so as to scour away the shoals, the attack on left bank upstream of Kaira and on right bank downstream of Kaira would continue. This can only be done by closing the spills. At the same time we have to be careful to keep the afflux within permissible limits, otherwise there is a danger of the river overflowing the right bank of Harijala Village. The best way how this can be achieved will be shown by the model. A spur in the north-west corner will of course be necessary for diverting the river along the fort wall so as to flow through the bridge.

Photo
1139-41.

Experiments will now be continued after incorporating the spill in the model

5. DISTRIBUTION OF BHADAR RIVER WATERS

Synopsis

1. River Bhadar is very unstable at its bifurcation near Dhanduka Town in Ahmedabad District. The North Branch which was originally a small spill has now developed into the main branch and carries about 90 per cent of the flood waters, while the parent river course (now called South Branch) has fast deteriorated. The discharges are now required to be divided into the two branches artificially in desired proportions. The experiments in a vertically-exaggerated model, were conducted for evolving a suitable measure and working out its details such that with the existing configuration in this reach, the discharge allocation made, would not alter appreciably in future.

Introduction

Fig 1.

2 *History of the Problem*—River Bhadar rises in the hills to the south of Chotila in Kathiawar and after running through the flat alluvial tracts of Saurashtra and Gujarat for a distance of about 80 miles falls in the Gulf of Cambay through Dholera Creek. The river is broad, shallow and steep and the flood water spills over the banks and carries with it the rich fertilising silt in suspension to the fields. The rainfall in this tract being low, the cultivation is very much dependent on the river waters. The land is virgin and the river is so rich in water-borne silt that the areas receiving spill waters produce the best crops of tobacco, wheat, maize, etc. Near Dhanduka Town in Ahmedabad District, the river bifurcates, the old course, now known as the South Channel meets Dholera Creek while the North Channel newly formed since 1927 joins the Bhagwo River which is a tributary of the Sabarmati. The North Branch was initially only a spill channel but developed during the abnormal high floods of 1927 and having its offtake favourably located on the outside of a sharp bend, it has opened out at such an alarming rate that about 90 per cent of the flood water is now carried by it, while most of the bed sand is diverted into the south channel, for partial removal of which a sum of about Rupees 12,000 is annually spent. Cuts have been tried in vain for increasing the supply of flood waters to the South Channel, so also a temporary bund with a low level weir for forcing more winter discharge into it. The accretion of the south channel and consequent deterioration has also affected the conditions at its out-fall in Dholera Creek, which in absence of freshet discharge, has got silted up and thus is not now navigable. The bifurcation being on the State boundaries of Bombay and Saurashtra, these Governments ultimately fixed the proportions in which the discharges should be shared by the two branches—half and half, during monsoon and two to one during cold season for south and north branches respectively and consulted the Poona Station for the design of suitable measures to implement this award.

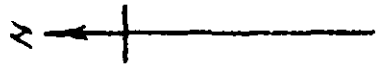
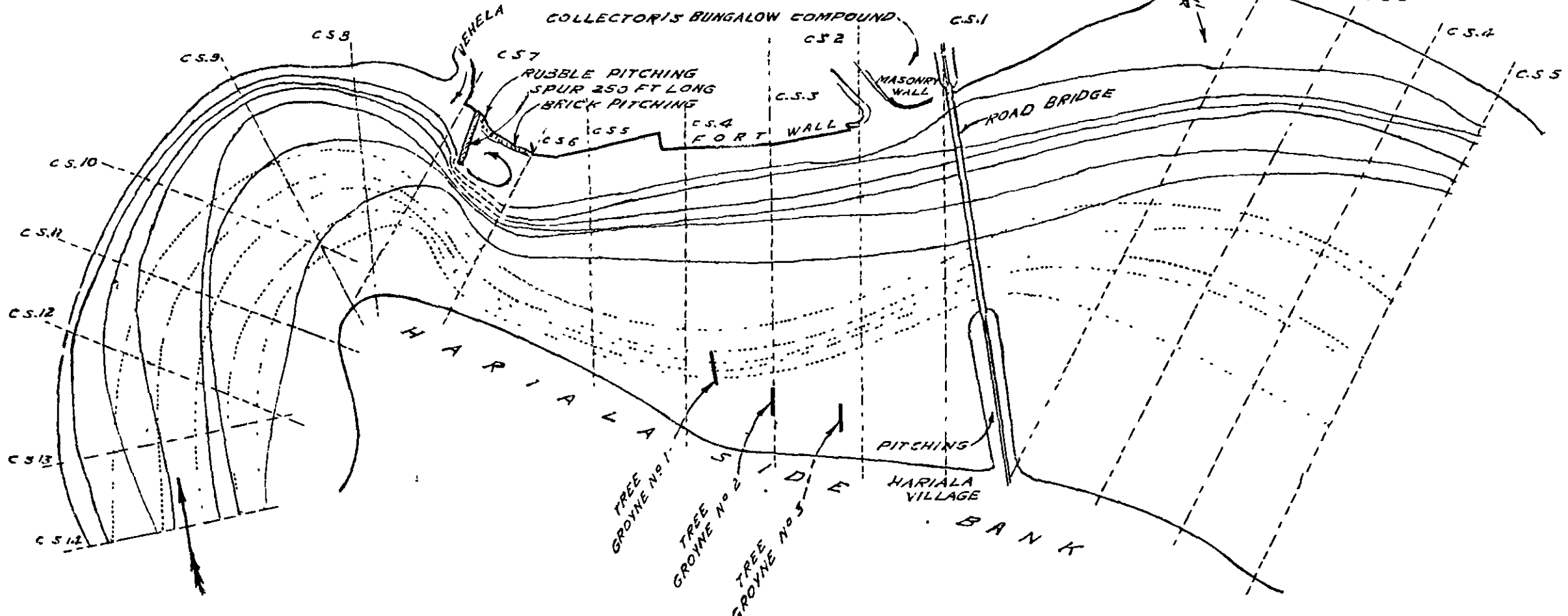
On studying the problem and available data, it was found that a gated barrage, though ideal for a rigid and efficient control, could not be adopted since,

- (a) the river rises and falls suddenly during the monsoon, frequently in a single day by as much as 16,000 cusecs which may as well be the highest flood intensity for the whole season and therefore gates cannot be operated in time, and
- (b) the cost of a gated barrage would be prohibitive.

A cheaper though less flexible solution for apportioning the flood discharge equally was ultimately decided upon in the shape of a low level weir divided in two bays by a divide wall.

Table 1.

The flood hydrograph analysis presented in Table 1 below shows that a maximum discharge of 15,000 cusecs and above is only a rare occurrence of a single day in the whole season, so also the discharges in excess of 1,000 cusecs are generally not obtained for more than a few days.

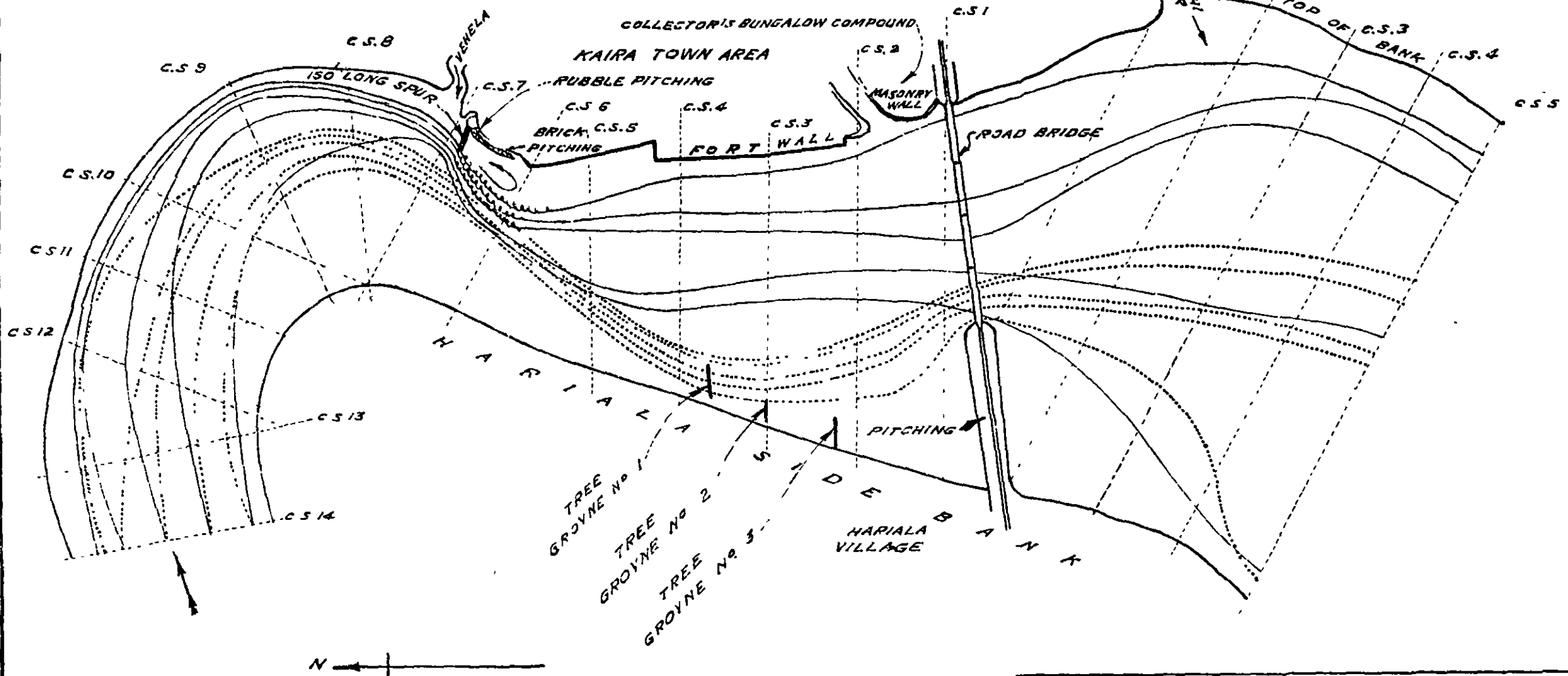


COMP. BY- Maugh

TRACED BY- G. R. PATKIE
25-3-1952.

REFERENCES
SURFACE LINES OF FLOW —————
BED LINES OF FLOW

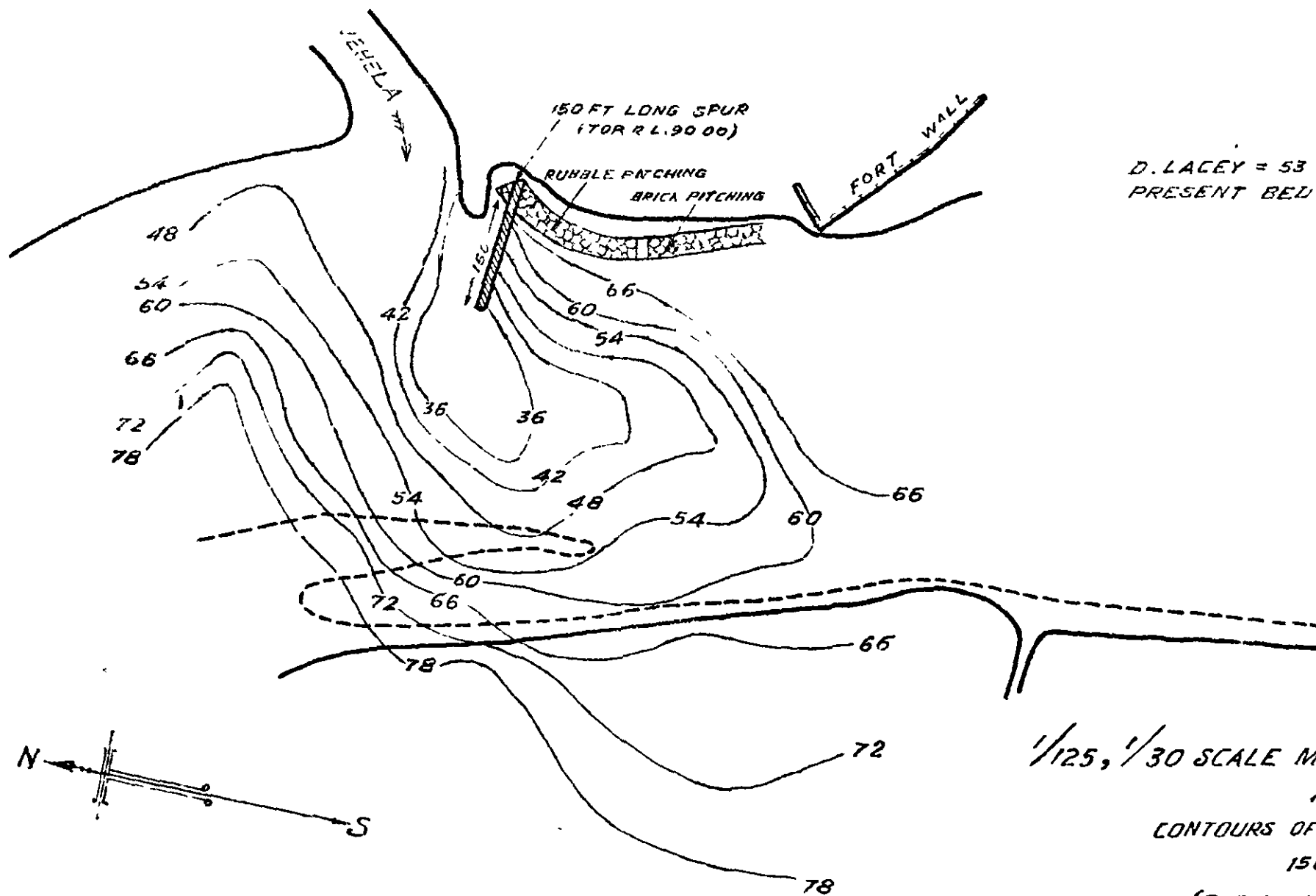
FIG. 6
 1/125, 1/30 SCALE MODEL OF WATRAK RIVER
 AT KAIRA
 EXPERIMENT NO 4 (D) LINES OF FLOW
 WITH 250 FT. LONG SPUR AT N W CORNER
 Q = 60,000 C.S.
 SCALE 1" = 7.50'



REFERENCES. :-
 SURFACE LINES OF FLOW ———
 BED LINES OF FLOW
 N ←

COMP. BY- *Mudali*
 TRACED BY- G. R. PATKIE.
 19-3-1952.

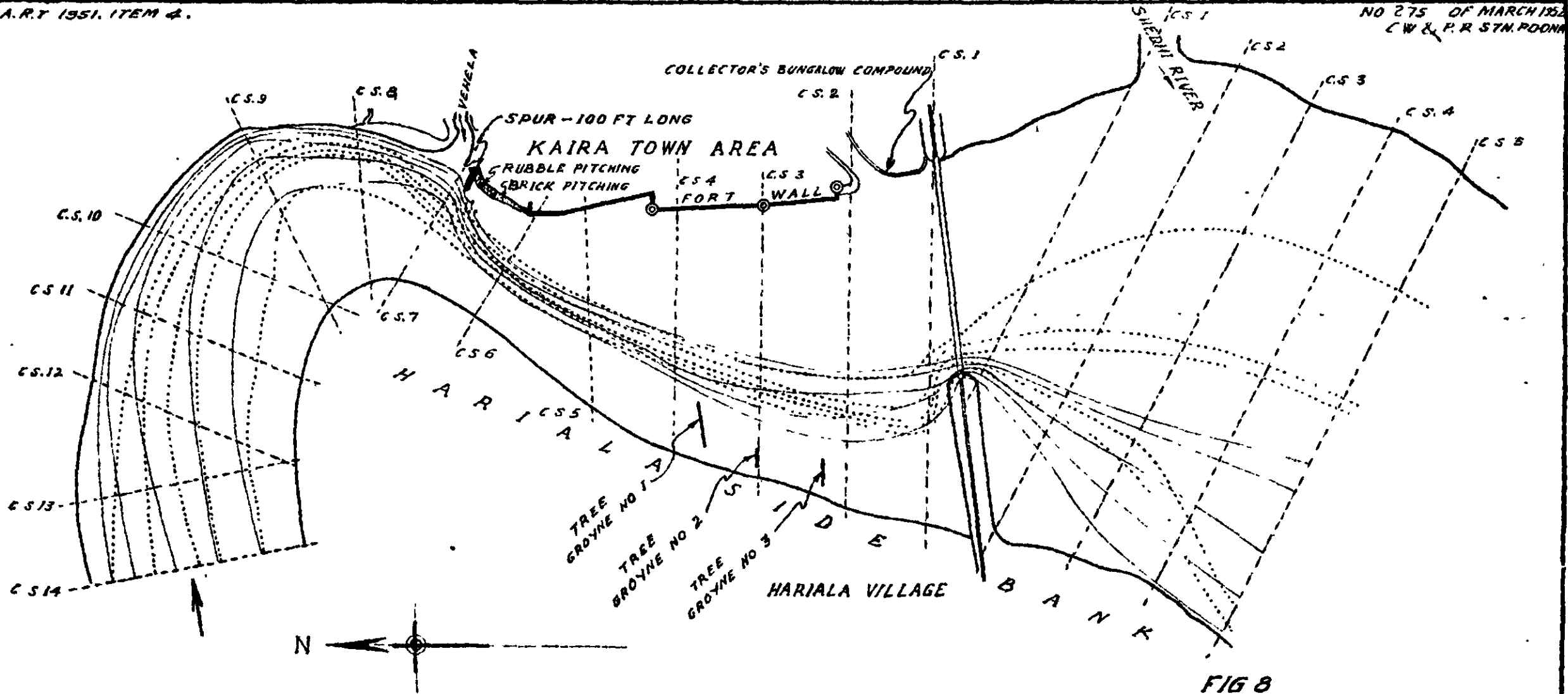
FIG. 7
 1/125, 1/30 SCALE MODEL OF WATRAK RIVER
 AT KAIRA
 EXPERIMENT Nº 4 (b) - LINES OF FLOW
 WITH 150' LONG SPUR AT N. W. CORNER
 Q = 60,000 Cs.
 SCALE 1" = 750'



D. LACEY = 53.5 FT. BELOW H.F.L. (87.5)
 PRESENT BED LEVEL AT SCOUR HOLE IS
 R.L. 53.0

FIG. 7 (a)
 1/125, 1/30 SCALE MODEL OF WATRAK RIVER
 AT KAIRA.
 CONTOURS OF SCOUR-HOLE ROUND
 150 FT. SPUR
 (TOP OF SPUR AT R.L. 90.00)
 SCALE: 1" = 200'

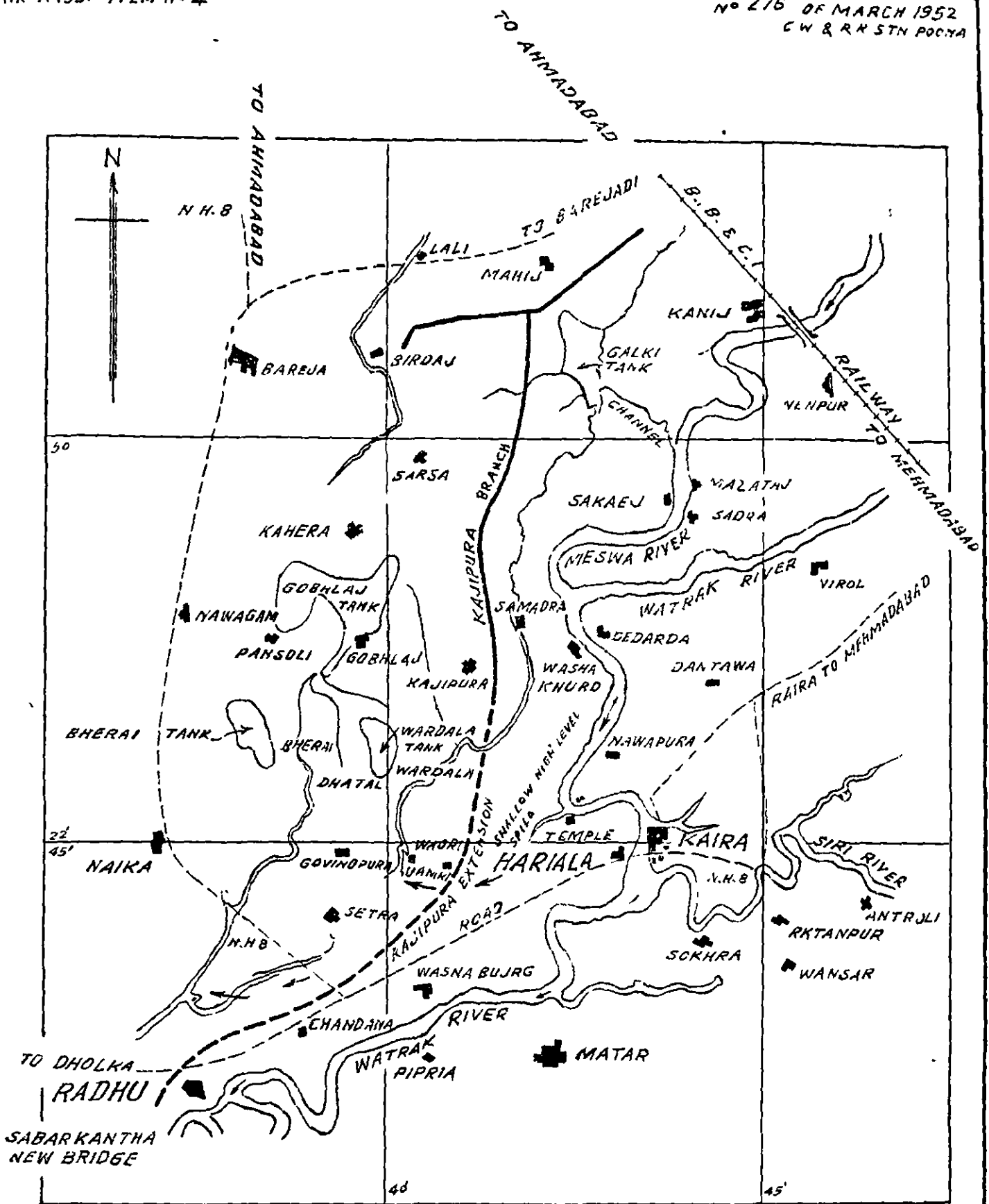
COMP BY *Munshi*
 TRACED BY S. B. SARKAL
 20-3-1952



REF.
 SURFACE LINES OF FLOW —————
 BED LINES OF FLOW
 N

FIG 8
 1/125, 1/30 SCALE MODEL OF WATRAK RIVER
 AT KAIRA
 EXPERIMENT No. 4(C) LINES OF FLOW
 WITH 100 FT LONG SPUR AT N.W. CORNER
 Q = 60,000 CUSECS
 SCALE. 1" = 750'

COMP. BY. *Munasingh*
 TRACED BY THAKOR R.T.



REF:

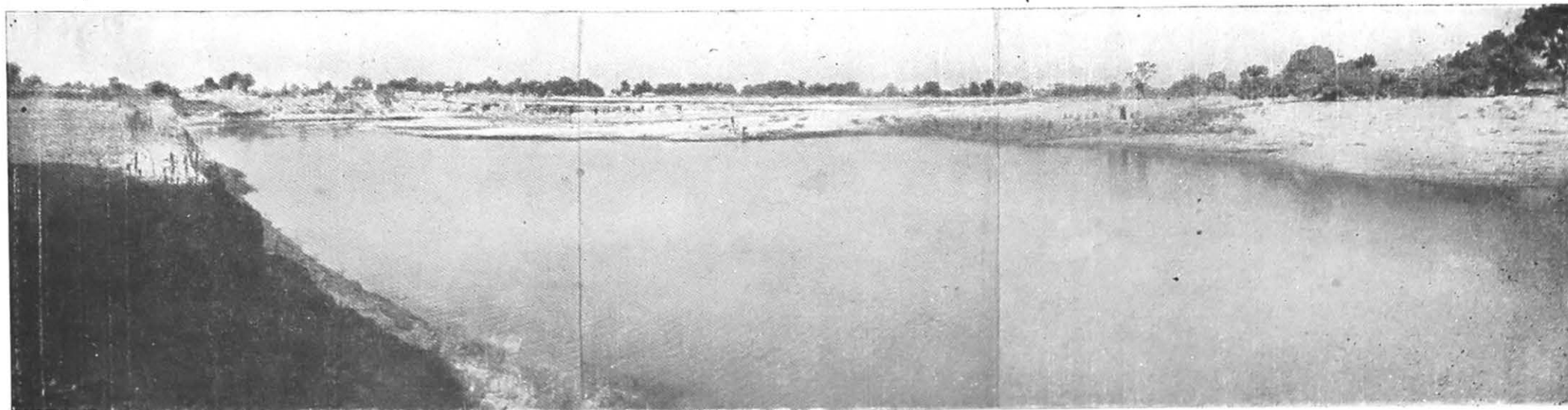
- ROADS - - - - -
- MESHWA CANAL ————
- KAJIPURA BRANCH EXTENSION - - - - -

FIG:-9
 INDEX PLAN
 SHOWING
 THE COURSE OF WATRAC RIVER
 NEAR HARIJALA VILLAGE
 SCALE - 1" = 2 MILES.

COMP
 TRACED BY SARKAL 28-3-52

No. .
A. R. T. 1951.

THE WATRAK RIVER AT KAIRA (Bombay State)



**Photos Nos: 1139 - 1141 Looking upstream showing the
low left bank over which spills occur during high floods.**

ITEM No. 4.
A. R. T, 1951

THE WATRAK RIVER AT KAIRA (Bombay State)

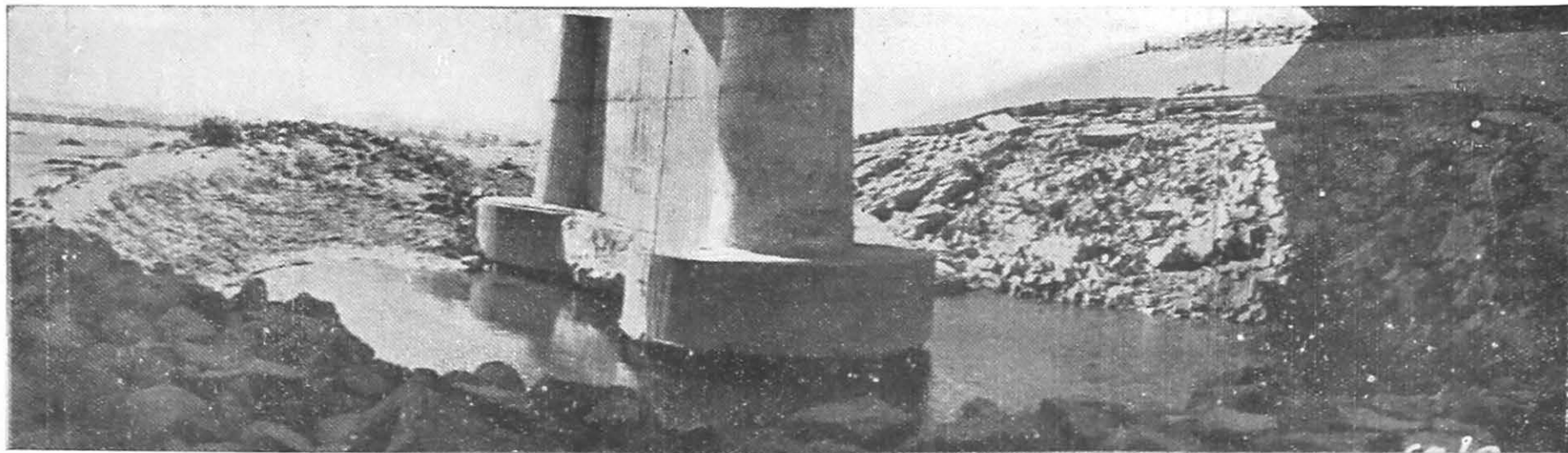


Photo Nos. 6842, 6843 and 6644 — showing deep scour round pier
No. 16 (right bank) and damage to brick pitching at the right
abutment — looking downstream.

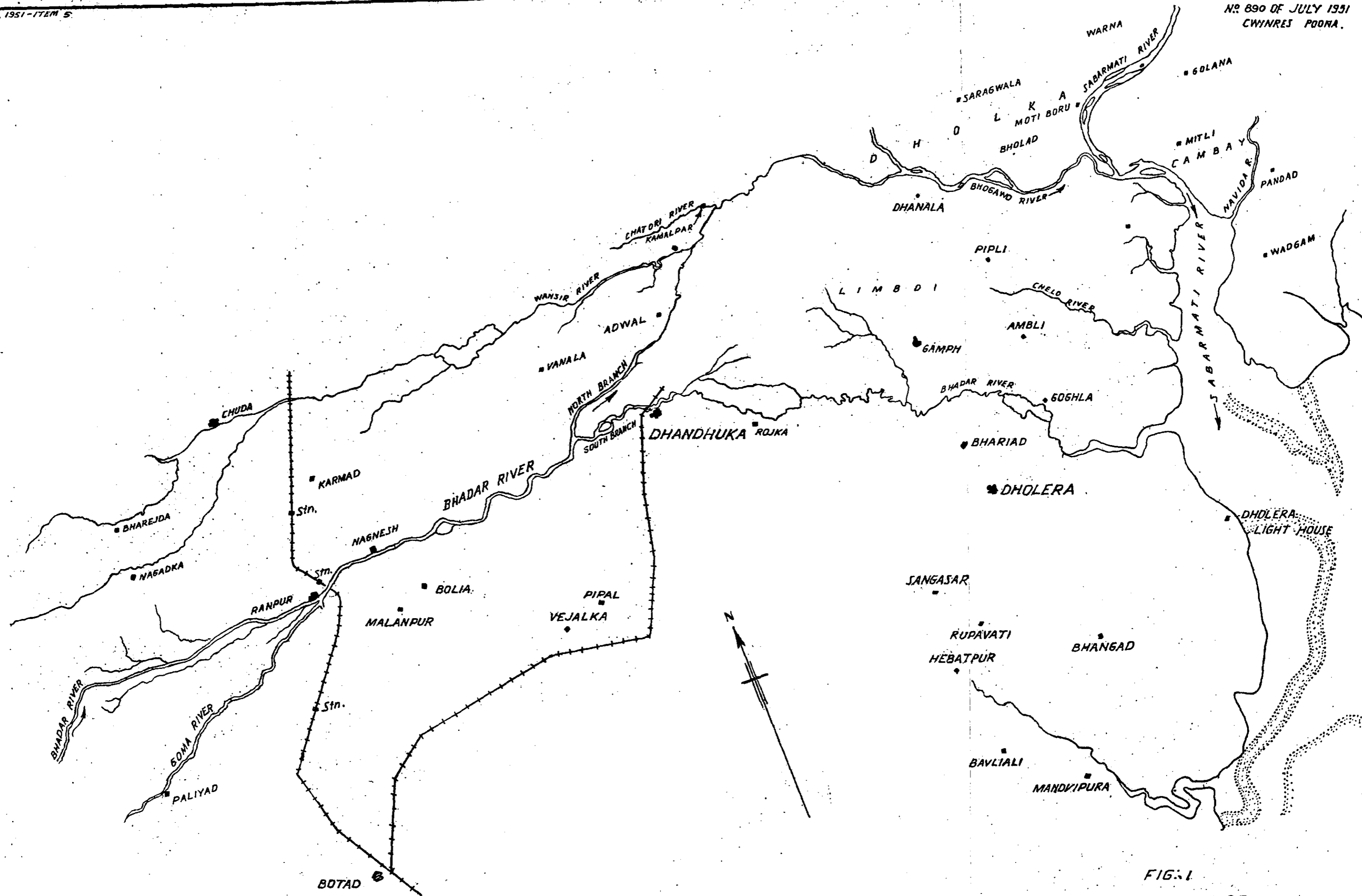


FIG. 1
 INDEX PLAN OF
 BHADAR RIVER AND ITS BRANCHES
 NEAR DHANDHUKA TOWN
 SCALE 1" = 4 MILES.

COMPARED BY: *M. Singh*
 TRACED BY: M.K. JAIN.

TABLE 1
Flood Hydrograph Data

Discharge intensity of the river in cusecs between	Number of days on which these discharges were obtained in	
	1949	1950
20,000 and 15,000	nil	1
15,000 and 10,000	3	nil
10,000 and 5,000	8	4
5,000 and 1,000	23	5
1,000 and 400	11	7
400 and 300	6	9
300 and 100	26	9
100 and 0	57	83
	118	134

Normal flow during the monsoon is below 400 cusecs, while in October the river hardly carries more than about 30 to 40 cusecs which also dwindles down before the beginning of November. The range of flood discharges being great it is necessary that the device to be proposed should, in addition to its cheapness, be sufficiently elastic and flexible to work with equal efficiency at all river stages. On the other hand the winter discharge is so small and its period so short that a provision for distribution of water in the desired proportion can be easily made by providing suitable cuts—depressed weirs—in the main weirs.

To minimise the length of the divide wall which is a costly item of the scheme, the weir should really be located near the bifurcation. This reach, however, is on an established curve and though the river could be divided arbitrarily, this distribution is likely to be altered in future unless the *bed load* is also proportionately divided. Under the circumstances it was difficult and hazardous to suggest a proper location, length and other details of the weir with respect to present river configuration without model tests. It was therefore proposed to undertake model experiments to evolve a suitable design of necessary works.

3. The River Model

The following scales worked out by trial and error were found to be satisfactory Photos 0815. and were finally adopted.

Horizontal Scale L	1/200
Vertical Scale D	1/35
Vertical Exaggeration	5 71
Slope Scale	20
Discharge Scale	1/41,500 according to Froudian Law which approximately tallies with that given by Meander formula viz., L^2 or 1/40,000.

4. Model Experiments

The problem was closely connected with bed load transportation and river discharges. The model was therefore proposed to be tested for reproduction of these factors as closely as possible. The river data available in these respects showed—

- (a) at all flood stages excess bed load is drawn by the south channel. This chokes and reduces its section while the north channel carries much less sand and is free from sand trouble; on the contrary it has caused erosion of the left bank, to the extent of about 60 ft;
- (b) in 1950, with the river discharge of about 20,000 cusecs, the south branch carried approximately only 13 per cent., i.e., 2,600 cusecs with a surface slope of 5.56 ft./mile.

To verify these occurrences in the model, a discharge equivalent to 20,000 cusecs was let in and the downstream water levels were adjusted to obtain one of the known gauges equivalent to that in the prototype. The discharge division and water surface slope observed in the model were as in Table 2.

Table 2.

TABLE 2
Model Observations with Discharge Equivalent to 20,000 cusecs

In	Discharges (cusecs)		Prototype Slope as observed.	Model Slope converted to prototype.
	Prototype discharge calculated from actual observations in the river.	Model discharge converted to prototype scale.		
Main river above bifurcation	20,000	20,000	} 5.56 ft./mile	5.45 ft./mile
South Branch	2,800	2,560		

Sand sufficiently fine to allow of its movement with the available tractive force was subsequently injected when it was noticed that almost all the sand entered the south channel, while surface sand-free water mostly flowed into the north branch.

Figs. 2 & 3.

Having thus proved the model further experiments were undertaken.

Experiments for Distribution of Flood Discharge

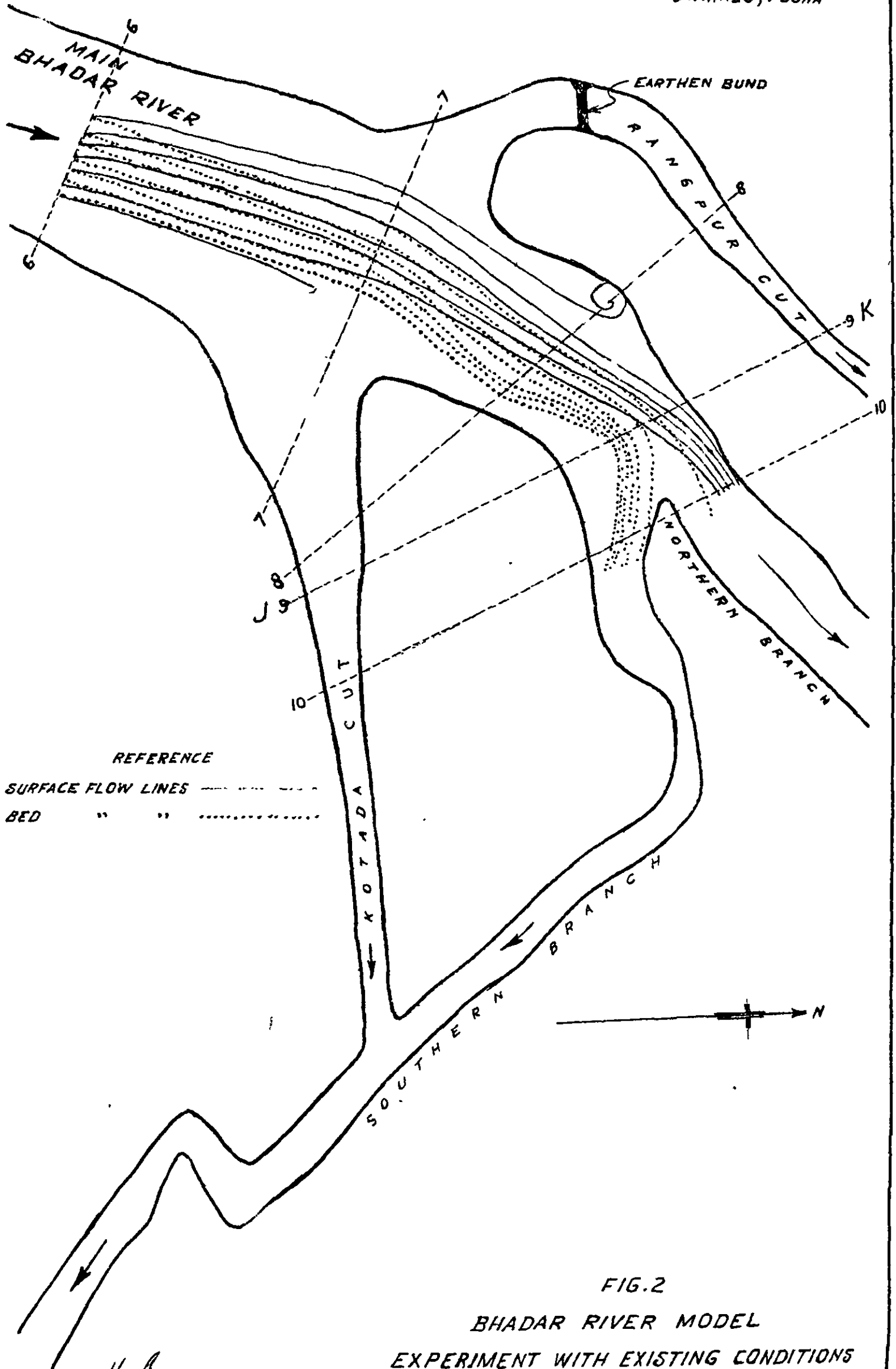
A cheap design is that of a weir at the bifurcation point. The JK line where the river is narrow and deep and is nearer to the bifurcation point appeared to be a suitable site for a weir. The weir 315 feet long with its sill at R. L. 96 00 was tested for highest flood stage of 20,000 cusecs discharge in the river. The results were unsatisfactory, because for equal discharge distribution in the two branches the weir lengths required were far too divergent, viz., in the ratio of about 1:3 (i.e., 1 for the northern branch and three for the southern branch) This was due to the existing bed slope of the North Branch being too steep as compared with that of South Branch. This caused a large draw-down effect in the northern branch and this could be neutralised only by constricting the length of the weir in that branch as stated above. Besides the weir having been located in a curved reach, the bed sand was being drawn by the south channel. The model indicated that sand would deposit near the right bank upstream of the weir and mask the right end portion of the weir (meant for the South Branch) and in the course of time, the share of water drawn by the southern branch would be adversely affected. Thus the experiments clearly indicated that mere distribution of discharge in the ratio of 1:1 without equal distribution of bed load would not solve the problem.

Fig 4
Photo 7212.

Photo 7221.

Other suitable positions for the weir were subsequently tested upstream of JK line in the straight river reach and ultimately the weir between sections 1,500 feet and 1,600 feet upstream of JK and just above the mouth of the Kotda Cut was found to be most promising. The design with the weir at this position with sill at R. L. 99 00 and with weir lengths for South and North Branches in the ratio 1 6:1—which was next tested, was however defective in that, though the discharge division was equal with peak discharge of 20,000 cusecs in the river, at lower stages the distribution was affected. The variation in discharge ratio was due to varying drowning conditions at the weir which indicated that the weir crest would have to be sufficiently raised so as not to permit the downstream water levels to affect the discharging capacity. At the same time 'raising' must not be too great in order to keep down the afflux to the minimum. Data of the river water levels at different flood stages not being available, the optimum weir crest level could best be determined by trial and error in the model.

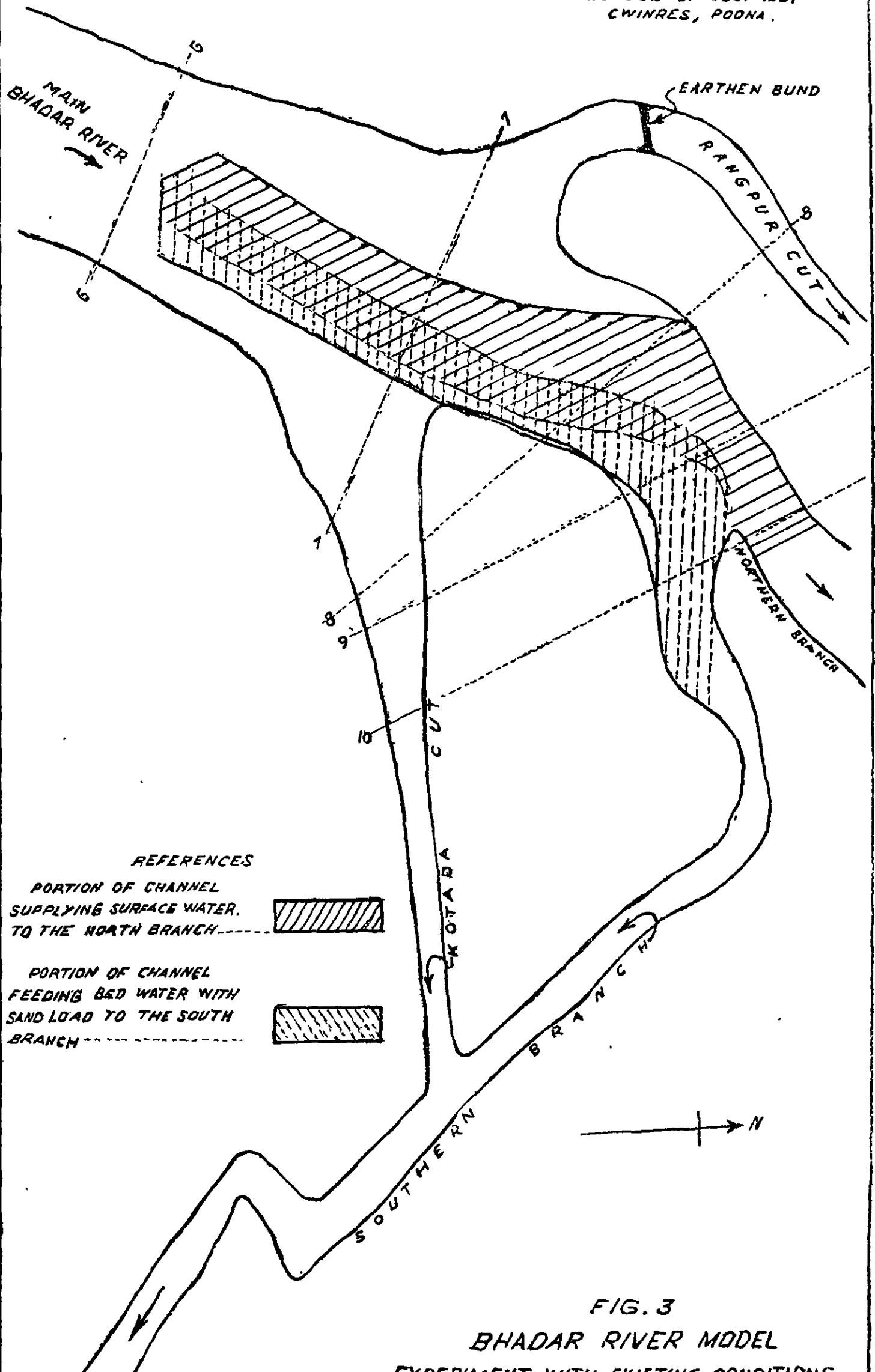
It was realised that it was not possible to pass a maximum of 10,000 cusecs through the present Kotda Channel, as its mouth is heavily accreted with sand and the bed level is as high as R L 103 00, as compared with river level at the offtake of 95 00. Widening of the Kotda Channel to a uniform width of 175 feet was indicated to be essential in the model. The widened Kotda Cut without lowering the bed meant that the weir crest should be at R. L. 103 75, which is obviously



REFERENCE
 SURFACE FLOW LINES - - - - -
 BED " "

FIG.2
 BHADAR RIVER MODEL
 EXPERIMENT WITH EXISTING CONDITIONS
 SCALE : 1" = 2' = 400'

COMP. BY. *M. S. Parkhi*
 TRACED BY: PARKHI.
 5 JULY 1951



REFERENCES

PORTION OF CHANNEL
SUPPLYING SURFACE WATER
TO THE NORTH BRANCH



PORTION OF CHANNEL
FEEDING BED WATER WITH
SAND LOAD TO THE SOUTH
BRANCH



FIG. 3
BHADAR RIVER MODEL
EXPERIMENT WITH EXISTING CONDITIONS

SCALE. 1" = 2' = 400'

COMP. BY: *Manj*
TRACED BY: MANCHUNDIA.
10TH JULY 1951

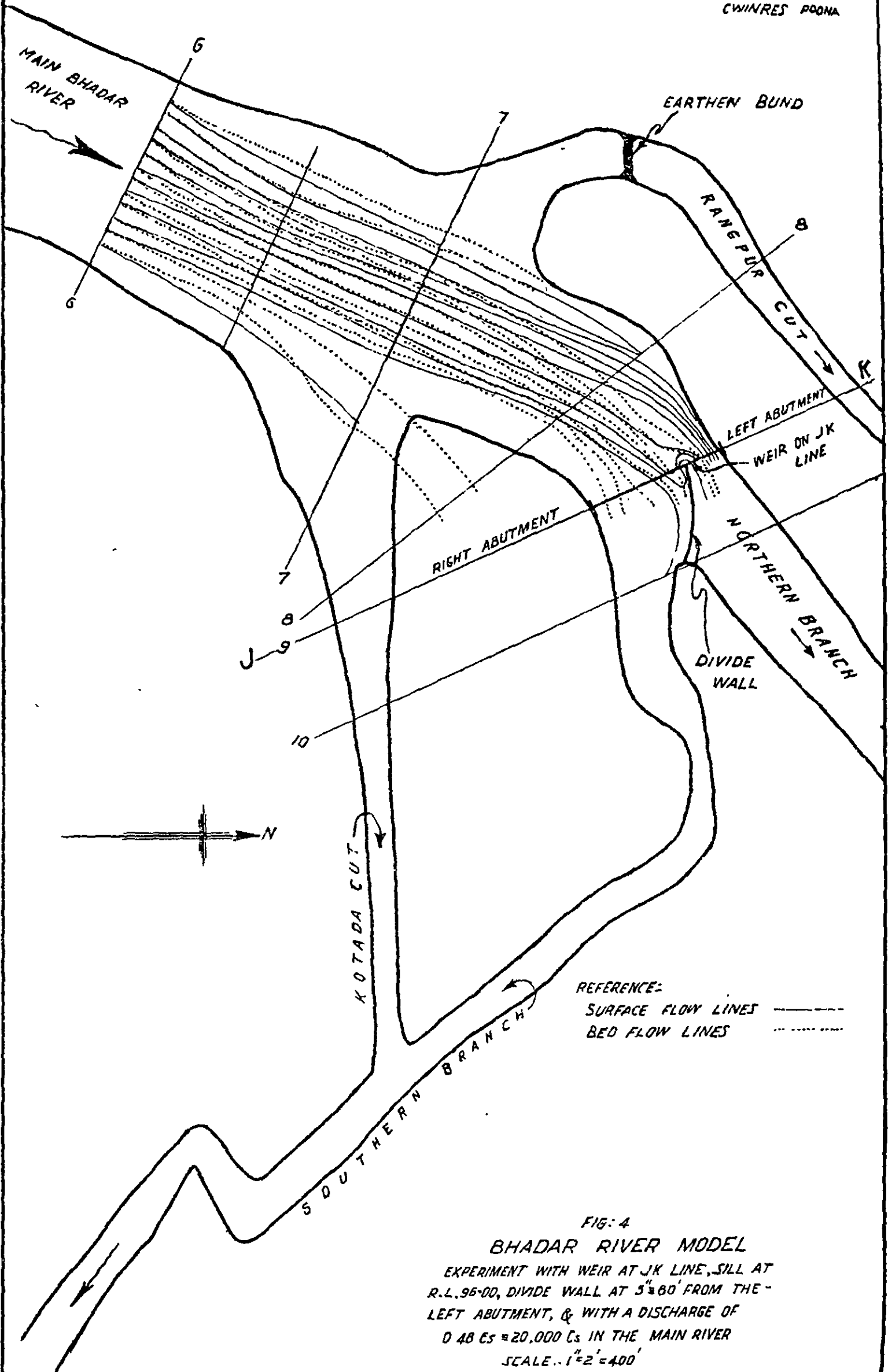
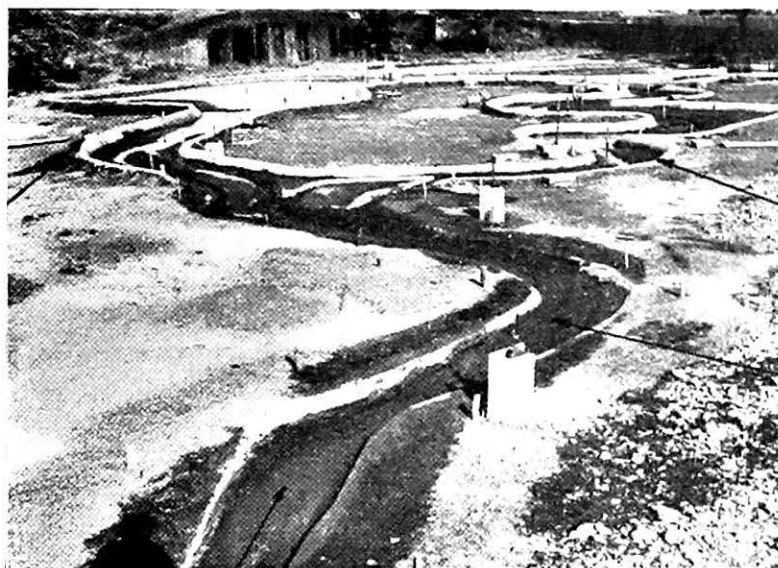


FIG. 4
BHADAR RIVER MODEL
 EXPERIMENT WITH WEIR AT JK LINE, SILL AT
 R.L. 96.00, DIVIDE WALL AT 5' 80" FROM THE
 LEFT ABUTMENT, & WITH A DISCHARGE OF
 $Q_{48.55} \approx 20,000 \text{ Cfs}$ IN THE MAIN RIVER
 SCALE - 1" = 400'

COMPARED BY: .
 TRACED BY: SABNIS

Model of the Bhadar River (District Ahmedabad).

North Branch



South Branch

Main Bhadar River.

No. 6815.

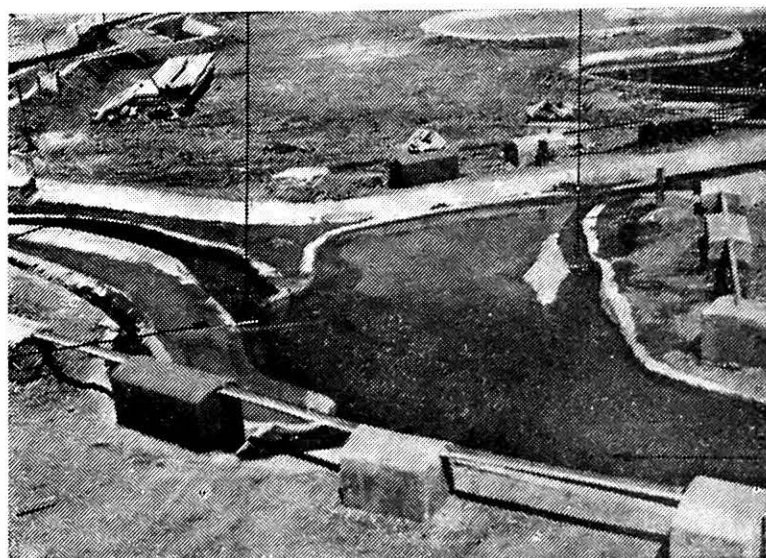
Showing general view of the Bhadar River Model.

South Branch

Kotda Cut.

North Branch

Weir on
JK line.



Main Bhadar River.

No. 7212.

Bhadar River Model with weir on JK line.
Photo Looking downstream shows the running model.

North Branch South Branch

Weir on Jkline



Photo No. 7221.

Bhadar River Model with weir on JK line. Dry model after fine sand injection showing ripple pattern on coarse sand bed. Bed load is mostly rolling along right bank and has masked the right end of the weir while north branch is clear of it.

Main Bhadar River

Kotda Cut
Remodelled.

Divide Bund
between South
& North
Channels.



Weir Line

North Branch.

Photo No. 8.

Model running with $Q \cong 10,000$ cs. with the weir on the section 1500 ft upstream of JK line. Width of weir = 150 ft. for each branch. Looking upstream.

impracticable. The only other alternative therefore is to lower water levels in the Kotda Cut deepening and regarding the channel bed to a uniform slope joining the existing bed of the Kotda Cut at the lower end with that of the river at the weir site. Most of this excavation work mainly consists of clearing sand deposits from the mouth of the Kotda Cut. The excavated material can advantageously be used for constructing the bund in continuation of the divide wall for separating the two branches. With the improved capacity of this cut, it was found in the model that the weir sill could be lowered to R. L. 100 25.

Different weir lengths were then tried keeping the weir crest at R. L. 100 25. The position of the divide wall was fixed to coincide with the section of maximum velocity which happened to be almost at the centre of the weir. In all the succeeding experiments, the position of the divide wall was unaltered. Model observations of these tests are summarised in Table 3. Photos 6, 8 and 398.

Table 3.

With a total weir length of 400 feet, which is about 85 per cent of the river width, divided equally for the two branches, the afflux obtained was smaller than with shorter weir lengths. The velocity of approach, however, influenced the discharge distribution and the proportion of the discharge in the North and South Branches varied between 1.11 and 1.0 with flood stages corresponding to 20,000 cusecs to 5,000 cusecs Fig. 5.

In general, it was seen that with a wider weir the discharge division was slightly affected with higher discharges though the afflux, on the other hand, was less. In studying Table 3 the following points were considered before selecting any of the three different widths.

- (i) With any of the three alternatives, discharge distribution for all floods below 5,000 cusecs would be equal, since the water levels downstream of the weir will be lower than the sill level and approach velocity being small, the discharge distribution will be governed by the weir lengths only. The discharges in excess of 5,000 cusecs occur for a very short period—9.35 per cent and 3.74 per cent of the total flow period in 1949 and 1950 respectively.
- (ii) Equivalent water levels obtained for three different lengths of weirs and for various river floods are shown in Table 3. The afflux for 20,000 cusecs will be—

0.32 feet for a 400 feet long weir ;

1.82 " " a 300 " " " ; and

3.38 " " a 218 " " " .

Assuming a permissible afflux of about 2.5 feet, it would appear that a 300 feet long weir will be suitable. But in the course of time, the river bed upstream and downstream of the weir would accrete to a certain extent and then stabilise. There is no data to determine the extent of accretion. It is however prudent to assume a lower coefficient of discharge under such conditions. Water levels were therefore worked out with a reduced discharge coefficient of 3.0 in items (5) and (6) of Table 3. The afflux under accreted conditions for 20,000 cusecs is expected to be

2.31 feet for a 400 feet long weir ;

3.88 " " a 300 " " " ;

5.53 " " a 218 " " " .

Thus from the afflux point of view, it is safer to adopt a 400 feet long weir shown in Figure 6, though a 300 feet long weir would give a better distribution of water Fig. 6.

TABLE 3

Water levels and discharge ratios of the Branches as observed in the model.										
Total river discharge (cusecs)	With weir on section 1,500 ft upstream of JK line and sill at 100 25 and with weir lengths.								Remarks.	
	Existing conditions in the prototype		200 ft. each for both branches.		150 ft each for both branches		109 ft each for both branches			
	$\frac{Q \text{ N. B.}}{Q \text{ S. B.}}$	W. L. at section 1,500 ft u/s of JK	$\frac{Q \text{ N. B.}}{Q \text{ S. B.}}$	W. L. u/s of the weir	$\frac{Q \text{ N. B.}}{Q \text{ S. B.}}$	W. L. u/s of the weir	$\frac{Q \text{ N. B.}}{Q \text{ S. B.}}$	W. L. u/s of the weir.		
1	2	3	4	5	6	7	8	9	10	
20,000	9.0	104.47	1.11	104.79	1.09	106.29	1.09	107.85	1	Water levels in the river with 40,000 cusecs, 20,000 cusecs at the weir site not being observed they are interpolated from available data.
15,000	not observed	..	1.11	104.05	1.08	104.85	1.08	106.52		
10,000	1.09	103.07	1.04	103.84	1.00	105.05	2	Existing bank level on the left, i.e., Rangpur Side is 104.95 and on right, i.e., Kotda Side is 107.24.
5,000	1.00	102.09	1.00	102.40	1.00	103.18		
Water levels with weir on section 1,500 ft. upstream of JK line, sill at R. L. 100 25 and with discharge coefficient of 3.00										
5.	40,000	..	108.55	..	110.28	..	113.05	..	115.75	}*
6.	20,000	..	104.47	..	106.78	..	108.35	..	110.00	

*With the accreted river bed upstream and downstream the discharge coefficient of the weir may be reduced to 3.0. Water levels are worked out with this coefficient

ITEM NO. 5.
A. R. T. 1951.

North Branch



Kotda Cut
remodelled.

Main Bhadar River
Photo No. 6

Same as Photo No 8 but looking downstream.

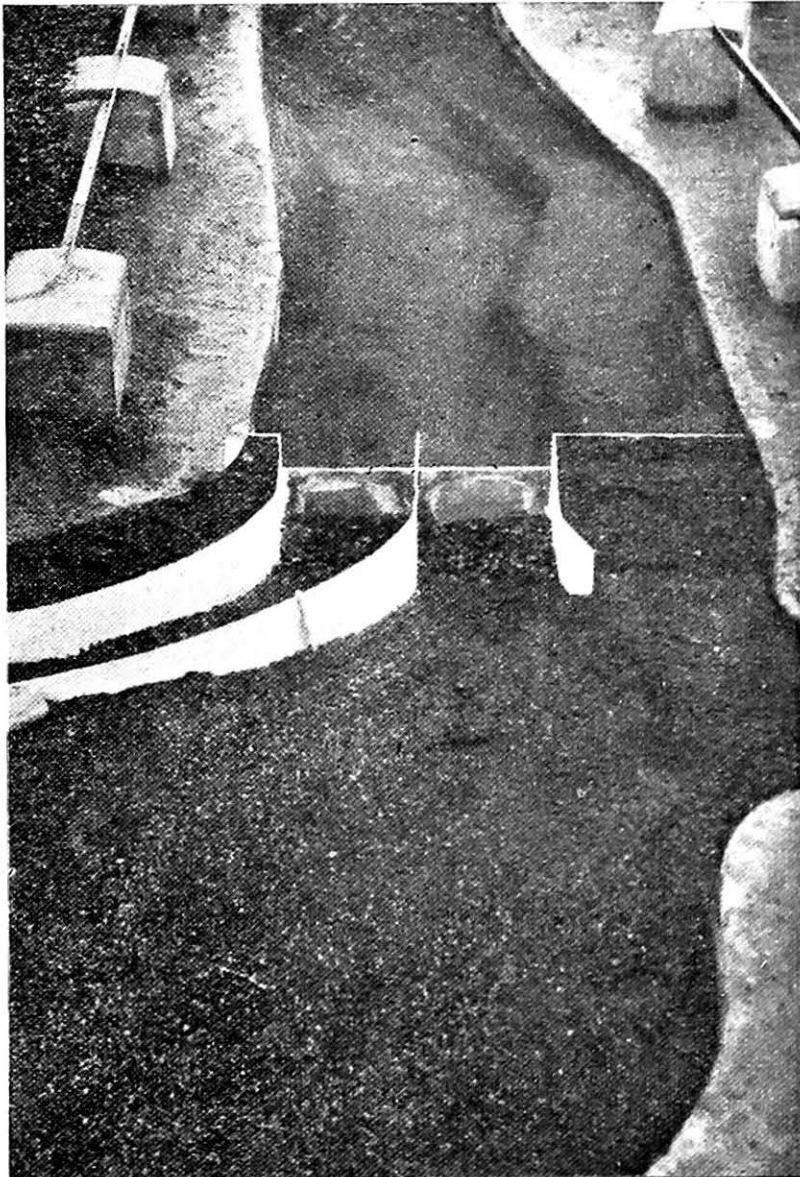
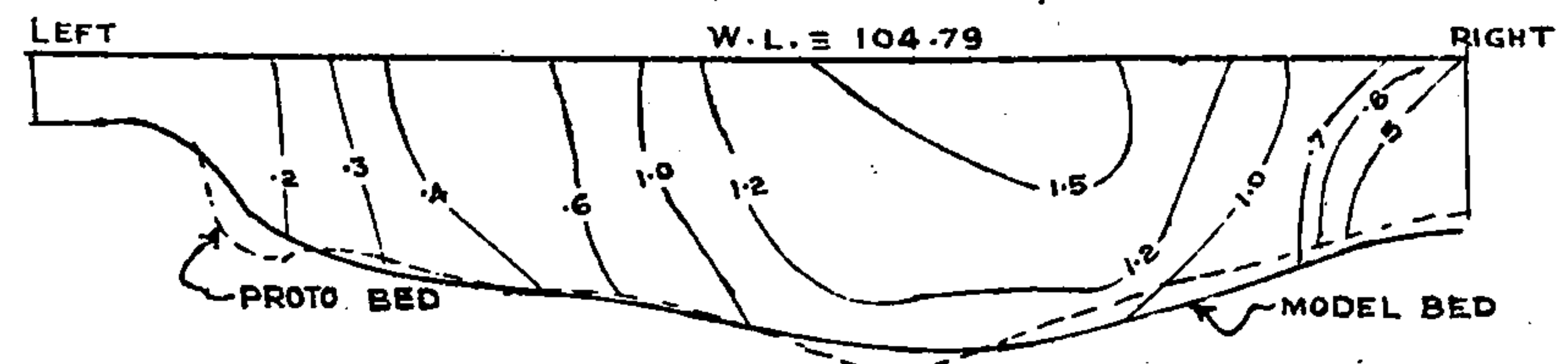
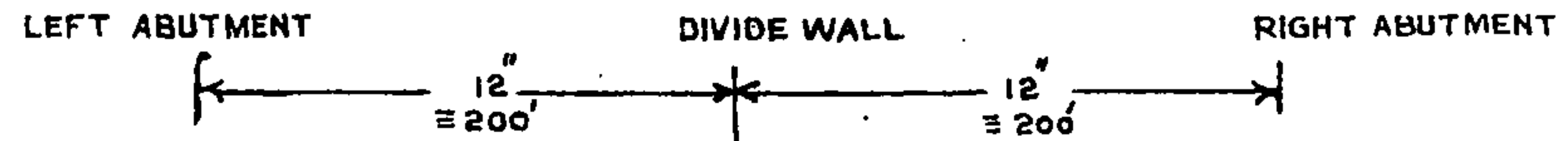
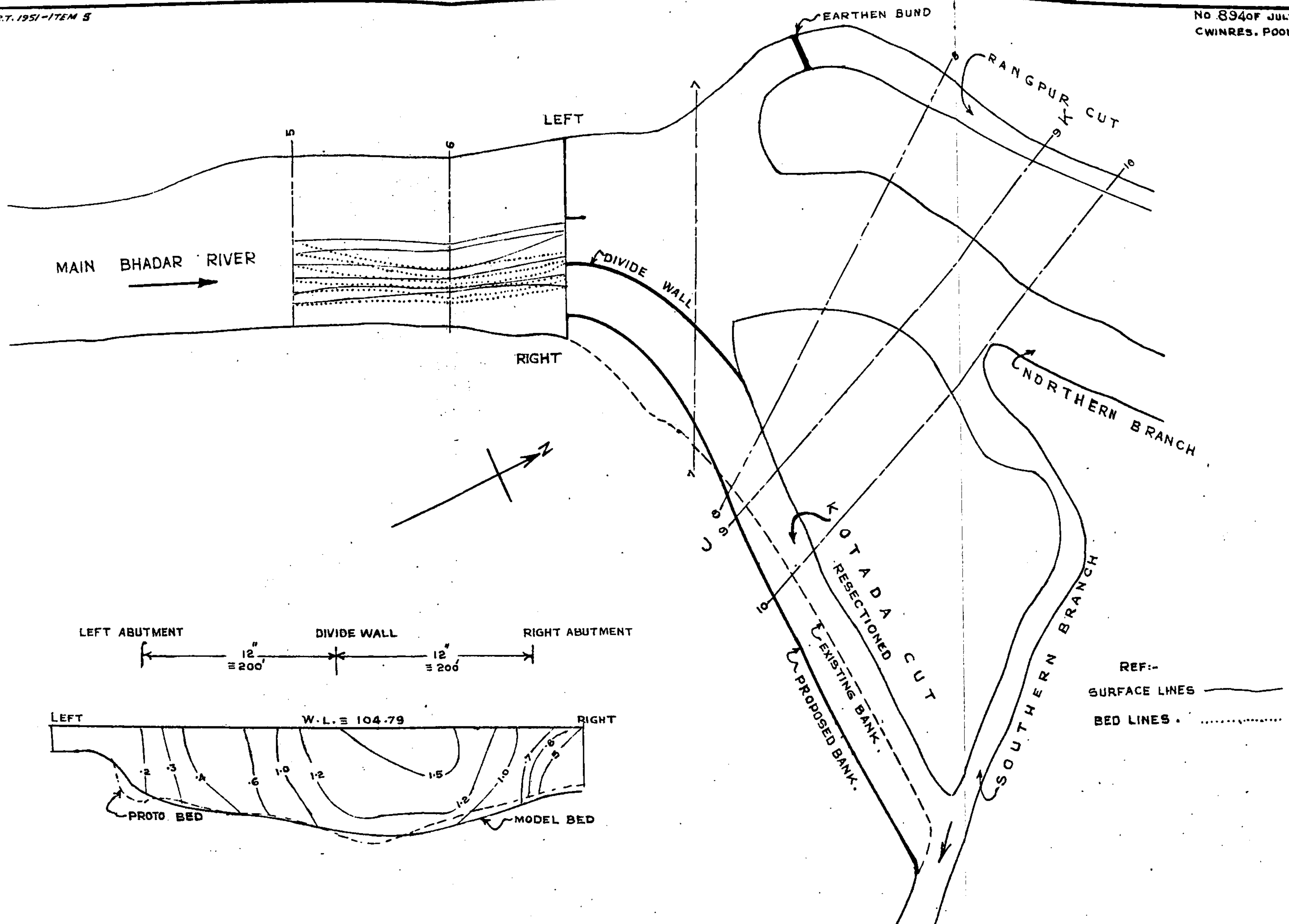


Photo No. 398

Bhadar River Model as laid showing proposed weirs
each 200 ft. long on section 1500 ft. upstream of JK
line-Looking upstream.



VELOCITY OBSERVATIONS.
ON SECTION PARALLEL TO THE WEIR
6" UP STREAM OF IT.
SCALE: - 1 FT./Sec. = 5.9 FT/Sec.

FIG:5
BHADAR RIVER MODEL
EXPERIMENT WITH WEIR ON SECTION
1500' UPSTREAM OF JK LINE.
WITH RIVER DISCHARGE OF 20,000 CS
SCALE: 1" = 2' = 400'

COM. BY: *[Signature]*
TRACED BY: R. K. KANTKAR

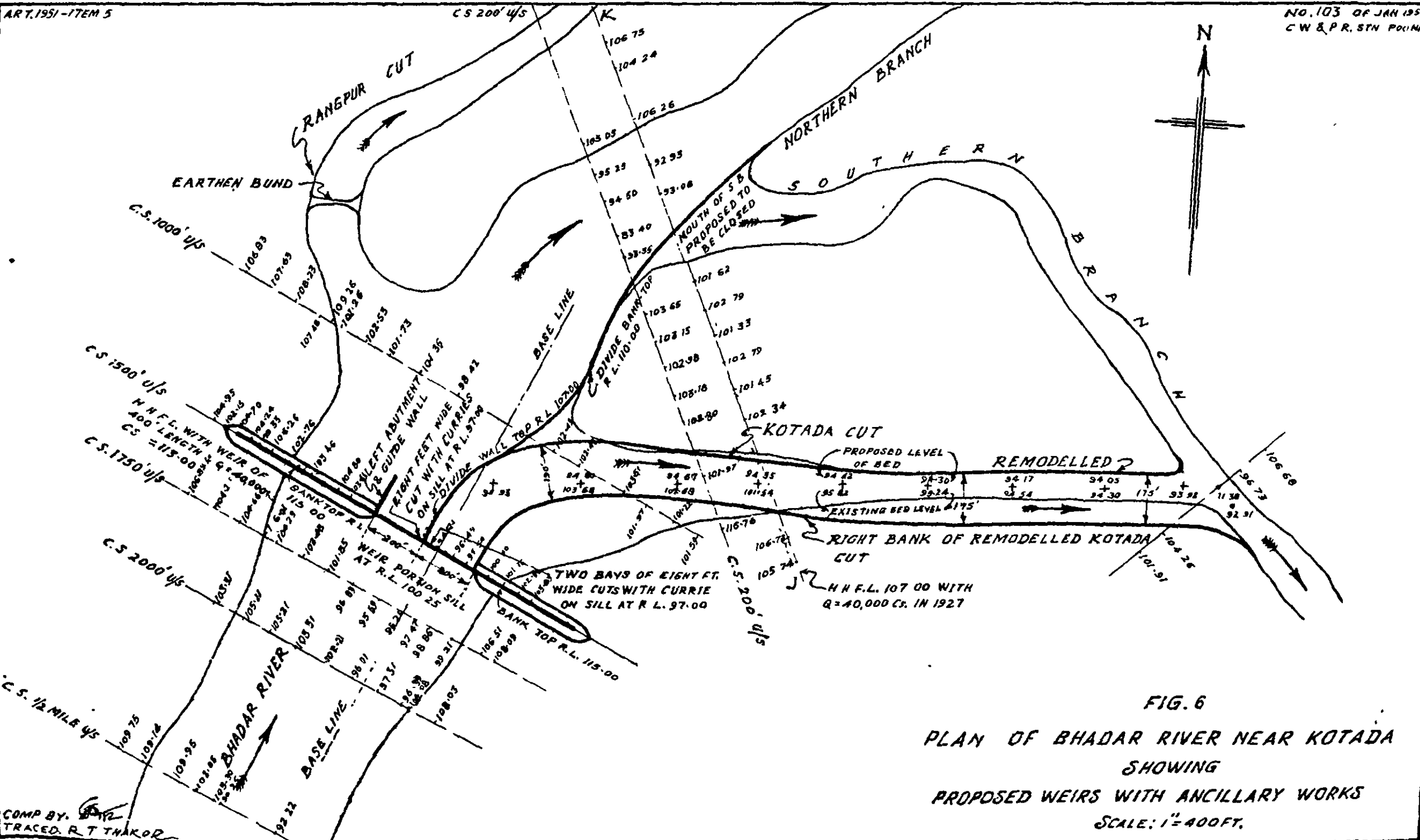
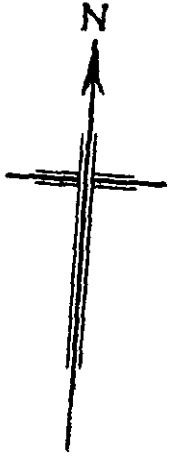


FIG. 6

PLAN OF BHADAR RIVER NEAR KOTADA
SHOWING
PROPOSED WEIRS WITH ANCILLARY WORKS
SCALE: 1" = 400 FT.

COMP BY: [Signature]
TRACED. R. T. THAKOR

over a wider range of discharges. The existing bank level on Kotda Side at the weir is at about R. L. 107.24 and on Rangpur Side at R. L. 105.00. With 20,000 cusecs, the probable H. F. L. with 400 feet weir would be 107.00 and hence would cause a spill on the left bank for a distance of about 1,000 feet only. The discharges in the North Branch are likely to be greater than in the South Branch by about 9 to 11 per cent at flood stages higher than 10,000 cusecs (*vide* Table 3). It may, however, be noted that the duration of discharges when equal proportion would not be obtained is small; considering all aspects the proposal of 200 feet long weirs for each of the branches appears to be preferable.

Besides the allocation of water distribution, the necessity of *ensuring equal bed load division* has already been emphasised. Accordingly careful observations for studying the bed movement at different flood stages were next made. The

results are presented in Table 4. The ratio of $\frac{\text{Average Velocity}}{\text{Velocity by Lacey Formula}}$ in Column 2 is a measure of likely sand movement. At the value of 1 and more, the movement in general should be expected to occur. Thus for discharges of 5,000 cusecs and below in post-weir period, it is unlikely that appreciable movement will be obtained. In calculating these ratios the grain size of sand used for working out 'f' value is 2 mm. giving $f=2.5$. Actually material on the river bed is still coarser and hence it is safer to work on the value of $f=2.5$. For discharges of 10,000 cusecs the model indicated about equal distribution of sand load in the two branches. With 15,000 and 20,000 cusecs the sand drawn by the North Branch was a little more than in the South Branch—65 per cent in north and 35 per cent in the south branches. This is a desirable feature as in the initial period the south branch requires comparatively less bed charge so that it shall be able to flush out the deposits formed already. Moreover this reduction of charge in the South Branch is likely to disappear after the North Branch accretes as the curvature caused by unequal water levels downstream of the two weirs will then not exist. The reason for this is that due to the water level in the North Channel downstream of the weir being lower than in the South Channel due to steeper slope in the former, local curvature is induced at the nose of the divide wall, deflecting more bed water into the North Channel. This is not objectionable as the North Channel, the bed of which is already scoured and the slope steeper, can absorb more bed sand. In the course of time, however, as accretion occurs in that channel, the difference of water levels on two sides of the divide wall downstream of the weir would reduce and then the division of bed material in the two branches would be more equal as in the case of water—*vide* columns 4 and 6 of Table 4. During this transition period, the South Branch would be drawing less bed material. This is a desirable feature, as this would enable it to scour its bed which has already accreted considerably.

Table 4.

It will thus be seen that after stability is reached, there is no chance of excess bed load disproportionate to the discharges—being drawn by either of the two branches. However, in the event of such a contingency arising in future, it will be easy to control sand by constructing an upstream divide wall, about 100 feet long, in continuation of the proposed divide wall.

5. Winter and Low Discharge Distribution

Winter discharges during and after the month of October are desired to be divided in the ratio of 2:1 for south and north branches respectively. Available flood data shows that the river discharge in October is generally less than 40 cusecs and that too for a very short period, the river flow practically ceasing by the end of October. The division of available discharge under these conditions is not practical since much of the water would be lost by percolation. Provided however the distribution ratio of 2:1 is ensured during September as well, the suitable discharge stage for the allocation appears to be 400 cusecs which will flow over the 400 feet long weir with a depth of 6 inches. The distribution of discharges ranging between zero and 400 cusecs can be made in the desired ratio of 2:1 by providing cuts in the weirs with sills depressed to R. L. 97.00. One bay of 8 feet in the north branch section and two similar bays each 8 feet wide in the south branch section, located near the central divide wall, would suffice. The cuts should be *closed during* floods by means of *karries* but should be opened by removing the *karries* when the depth of flow reduces to about 6 inches corresponding to a discharge of 400 cusecs.

6. Recommendations

(1) To allocate the river waters and bed load equally to the two branches during the flood season, a weir should be constructed at or within 100 feet upstream of

TABLE 4
Observation in the Model of Bed Load Distribution

Discharge Ratio	$V/V_1 = \frac{\text{Average Velocity } V \text{ with the weirs}}{\text{Velocity } V_1 \text{ by Lacey Formula in post-weir period with } f=2.5}$	Sand drawn by north branch (%)		Sand drawn by south branch (%)		Remarks
		Without accretion	With accretion	Without accretion in N. B.	With accretion in N. B.	
1	2	3	4	5	6	7
20,000 cu. . . .	1.16	60 to 65	60 to 62	40 to 35	40 to 38	For discharges of 5,000 and less, the ratio of V/V_1 was less than even 0.70 and hence no significant bed movement is likely to occur, especially in post-weir period.
15,000 ,,	1.00	50 to 65	50 to 60	50 to 35	50 to 40	
10,000 ,,	0.82	50 to 55	50	50 to 45	50	
5,000 ,,	0.62	

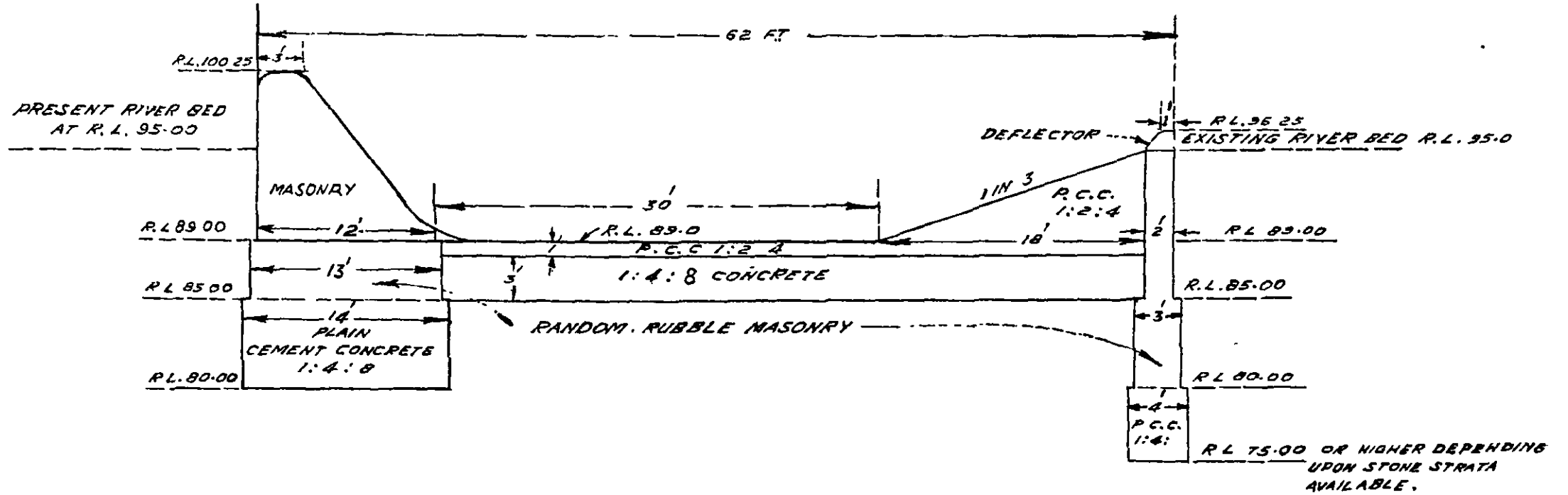


FIG. 7
SHOWING PROPOSED CISTERNS
FOR
BHADAR RIVER WEIR
SCALE 1" = 10'

COMP BY - *M.S.J.*
TRACED BY - PATRIE G.R.
11-12-51.

the section 1,500 feet upstream of JK line as shown in Figure 6. Of the alternative proposals with different lengths, the one with 200 feet long of weir for each of the branches appears to be suitable. The weir with its protective works as well as the bunds in line of the weir on both the sides should be designed for a flood intensity of 40,000 cusecs as such an extraordinary flood was once recorded in 1927. Cistern below the weir should be constructed to dissipate the energy of water falling over the weir as suggested, vide Figure 7 and Appendix I.

Fig. 6.

Fig. 7
Appendix I.

(2) It is for consideration whether the flank bunds to prevent spills upstream of the weirs should be designed for 20,000 cusecs or 40,000 cusecs. Even a flood of 20,000 cusecs is not experienced every year and then also it lasts for not more than a day. Provided, therefore, much damage is not apprehended with 40,000 cusecs it appears reasonable to try to restrict the overflowing up to the stage of 20,000 cusecs only. With 20,000 cusecs, High Flood Level with 200 feet long weirs is expected to be 107.00 and hence a bund with top R. L. 109.00 will be required on the left flank for a length of about 1,000 feet upstream of the weir. On the right bank the bank level being at about R. L. 107.00 no spill will occur. High Flood Level with 200 ft. weirs and with the extraordinary flood of 40,000 cusecs will be much higher at R. L. 110.28 and to prevent spills, much longer bunds will have to be built on both the banks.

(3) The suggested position of the weir is expected to be optimum for equal distribution of bed load. In future, however, bed sand can be easily controlled by means of a divide wall splayed either on the side of North Branch or South Branch as necessary. This should not be constructed till its necessity is actually felt which appears, however, a remote possibility.

(4) To separate the flow below the weir a divide wall should be constructed with a 2' to 3' free board. Owing to the flatter slope in the South Channel the water level downstream of the weir in that branch will be higher than in North Branch. In the model, water levels corresponding to river discharges of 40,000 cusecs and 20,000 cusecs were observed downstream of the weirs in the South Branch and were found to be equivalent to R. L. 108.00 and 104.15 respectively, and higher by about 3.20 feet than those in North Branch. Thus the divide wall will have to be designed for this unbalanced water pressure. The top of the divide wall may be kept at R. L. 108 or say at R. L. 110 to allow 2 feet free board. At 40,000 cusecs water may be allowed to spill over the wall as likelihood of such abnormal flood is very remote.

(5) The existing mouth of the South Channel at the bifurcation downstream of JK line, so also the Charodia Cut and any other spills and depressions joining the two branches, downstream of the weir should be closed. In view of this, a bund will have to be constructed at the mouth of the existing South Channel to meet the divide wall for preventing spill from the North Branch into the South Branch after the river waters are divided. The top of this bund may be kept at R. L. 110.00. The material for this bund will be available from the excavated material of the Kotda Cut. Another bund may be put at the junction of existing South Branch and Kotda Cut.

(6) The existing Kotda Cut should be widened to a uniform width of 175 feet from its mouth down to its junction with the South Branch. Also its bed should be dressed to a uniform slope as suggested under 'Experiments' above.

(7) The South Branch channel is at present heavily sanded. At several points its capacity having been reduced, spills have developed such as in the Khodol Reach and Rojka Cut. Undesirable flooding through depressions like Khasta Cart track, etc., is also at present experienced. In order to improve and maintain the South Channel to carry all the water passed over the weir down to the Dholera Creek, it is imperative that all unwanted spills are stopped by putting bunds and the reaches where the South Channel has very badly accreted are cleared.

(8) Due to the South Branch getting relatively more sandfree water than hitherto with the new proposals, it is expected that the Dholera Creek can again be put into commission with much less effort and expense.

(9) The Gunjar Groyne, about which so much criticism has been made in the past, was not found to affect adversely the proposed distribution of water and bed load. On the contrary, it was observed to have a stabilising effect on the river and also helped straightening it upstream of the proposed weir, which is desirable. Hence it should be retained. This groyne was constructed in 1909, about one mile upstream of bifurcation of the two branches.

(10) For winter discharge distribution, cuts should be provided with sills depressed to R. L. 97.00. The width for the North Branch Cut should be 8 feet and for the South Branch two such bays of 8 feet should be provided. These will have removable *karries* which should be removed when depth over the weir is reduced to 6 inches and the *karries* again restored before the next monsoon.

APPENDIX I

CISTERN DESIGN

Design Data

Max. discharge intensity=100 cusecs/ft.

Assuming 20 per cent concentration, Max. discharge=120 cusecs/ft.

Velocity of approach=8 ft/sec.

Retgression assumed=1.5 ft.

Velocity of=8 ft./sec.

Design of the Cistern

$$U/S \text{ T. E. L.} = H. F. L. + \text{Velocity head} = 110.28 + \frac{V^2}{2g} = 110.28 + 1 = 111.28$$

$$D/S \text{ T. E. L.} = H. F. L. + \text{Velocity head} - \text{retgression} = 104.85 + 1 - 1.5 = 104.35$$

$$\text{Loss of head} = U/S \text{ T. E. L.} - D/S \text{ T. E. L.} = 111.28 - 104.35 = 6.93$$

$$\text{For } q = 120 \text{ cusec/ft. and } H_L = 6.93 \text{ } E f_2 = 15.6$$

$$\therefore D/S \text{ floor level} = 104.35 - 15.60 = 88.75 \text{ Say R. L. } 89.00$$

$$\text{Length of cistern} = 5 (D_2 - D_1)$$

$$D_2 = \text{Depth of subcritical flow} = E f_2 - \text{velocity head} = 15.6 - 1 = 14.6$$

$$D_1 = \text{Depth of hypercritical flow (for } E f_1 = E f_2 + H_L = 15.60 + 6.93, \text{ i.e. } 22.53 \text{ and } q = 120 \text{ cusec/ft.)} = 3.5 \text{ ft.}$$

$$\therefore \text{Length} = (14.6 - 3.5) \times 5 = 11.1 \times 5 = 55.5 \text{ feet}$$

Actually S. W. will occur upstream of the calculated position by a distance = $d_2 - d_1$. Hence length of cistern of 50 feet is considered adequate.

This design was tested in a geometrically-similar 1/10 scale, part-width model and was found to be satisfactory.

6. RAIL-cum-ROAD BRIDGE ACROSS THE GANGA RIVER AT MOKAMEH—BIHAR PROVINCE

Introduction

1. As already stated in last year's annual report, the main channel has not in the past taken a course midway between the two banks of the river, but, when it has not selected one bank to the complete exclusion of the other, it has in general divided into two channels bifurcating around a fairly extensive chur and this bifurcation persists for a major length of the period. At the time of preparation of project for the railway bridge, in the year 1946, the Southern channel was the dominant stream, and there was very little flow in the Northern channel. Advantage was to have been taken of the configuration of the river at that time, by blocking the North channel and then to construct a bridge on the South channel, only one guide bund being then necessary on the left bank.

1.1. However since 1946, the North channel is showing definite signs of development, and by the time the construction of bridge is actually taken in hand (if Railway Proposals are approved) the North channel will have developed to an appreciable extent. Under these circumstances, it is considered desirable to find out means and ways to arrest this development of the North channel and to induce the river to silt it up, so as to facilitate the work of the Left Guide Bund and the approach embankment as already proposed.

1.2. With a view to achieve the object as stated above, the following alternatives were tried:

- (1) A short attracting spur on the South Bank near Mokameh Ghat.
- (2) A repelling spur near Madhurapur on the North Bank.
- (3) A repelling spur as above with an additional subsidiary bund below the bridge site.

2. Attracting Spur

Some rough, quick tests with an attracting short spur, near Mokameh Ghat, were made. But these tests showed that the attracting spur was not only ineffective in inducing silting in the North channel, but was definitely harmful, as the velocities near the South Bank were found to increase considerably. Hence this measure was not investigated in detail.

3. Repelling Spur

Various locations from 2½ miles to 4 miles upstream of the bridge site, were tried for this spur. The best location was found to be just below the outfall of the Baya Nadi, i.e., 2½ miles upstream of the bridge site. At this site different lengths were tried and it was found that the length of the spur required to be fully effective was 5,000 ft., at an angle of 60° pointing upstream, with the North Bank (see Figure 1). Detailed experiments were carried out with this position of the spur.

Fig. 1.

3.1. Figure 1 shows the flow lines with the 5,000 ft. long spur in position. The slack water region created by this spur extended only up to about 1¼ miles below the spur and as such it did not appear to be as much effective for the bridge site under consideration as desired. But at the same time it is not advisable to construct the spur very near the bridge site because after its removal, when the construction of the bridge is over, its remnants below the low water level, would create turbulence which might be dangerous to the bridge. Hence this spur could only be located sufficiently upstream of the bridge. As seen from the lines of flow, the effect of this spur at the bridge site was only fairly pronounced. Photos 6855 and 6857 show these flow lines and slack water region formed by this spur.

Photos 6855
& 6857.

Fig. 2. 3.2. Figure 2 shows the velocity distribution across the cross-section, 1 mile upstream of the bridge site. This shows that the North Channel was silting up, though the process was rather slow; but the high velocity flow was eroding the central chur very rapidly.

3.3. Velocities were also observed on the following cross-sections:—

- (1) 1/2 mile upstream of bridge site,
 Fig. 3. (2) at the bridge site (Figure 3),
 (3) 1 75 miles downstream of the bridge.

These showed that the velocities in the North channel had reduced with the spur, whereas they increased over the central chur. The velocities in the South Channel showed an increase, but those adjacent to the south bank were not much affected.

Fig. 4. 3.4. Figure 4 shows the contours of the scoured bed near the spur head.
 Photo 6866. Photo 6866 shows the scour pattern looking upstream.

3.5. *Conclusion*—The above observations lead to the conclusion, that by means of a repelling spur, located 2½ miles upstream of the bridge site, the North Channel was likely to silt up, though slowly. The rate of silting in the model was very low. This is due to the model containing very little material in suspension (one of the model limitations). However when more material in suspension was introduced artificially, the silting of North Channel was accelerated, which confirmed the previous conclusions.

But the effect of this spur in silting up the North Channel would be nullified by the fact, that it would also cause scour of the central chur, which separates the two channels. The Left Guide Bund is to be located on this chur, and if the latter is washed away, it will mean construction of the Guide Bund in deeper waters, at a much higher cost. This disadvantage outweighs the small advantages gained, in facilities of construction in closing the North Channel and reduction in the cost of North Approach Bank. Hence the proposal was not promising.

4. Repelling Spur with a Subsidiary Bund below the Bridge

Fig. 5. Figure 5 shows the position of this subsidiary bund. These experiments were carried out to see if the action of silting the North Channel could be accelerated by constructing a subsidiary bund below the bridge site—see Figure 5, and also if some of the disadvantages shown by the repelling spur alone could be obviated.

4.1. Figure 5 shows the flow lines in this case. This showed a definite improvement over the previous case, so far as the conditions in the North Channel in the vicinity of the bridge site were concerned. Figures 6 and 7 show the velocity distribution on cross-section 1 mile upstream of the bridge site and at the bridge site respectively. The general nature of the velocity concentration was more or less the same, though the velocities in the North Channel were reduced still further. There was also a marked improvement near the South Bank where the velocities were almost the same as in the pre-spur conditions. This experiment thus showed that the silting of the North Channel would be achieved, but at the cost of the central chur, as no signs of improvement in the conditions on the central chur were visible.

4.2. *Conclusions*—Though the subsidiary bund would accelerate silting in the North Channel as compared with only one spur located 2½ miles upstream, yet it would not improve the conditions over the central chur and hence the additional expenditure involved would not be justified.

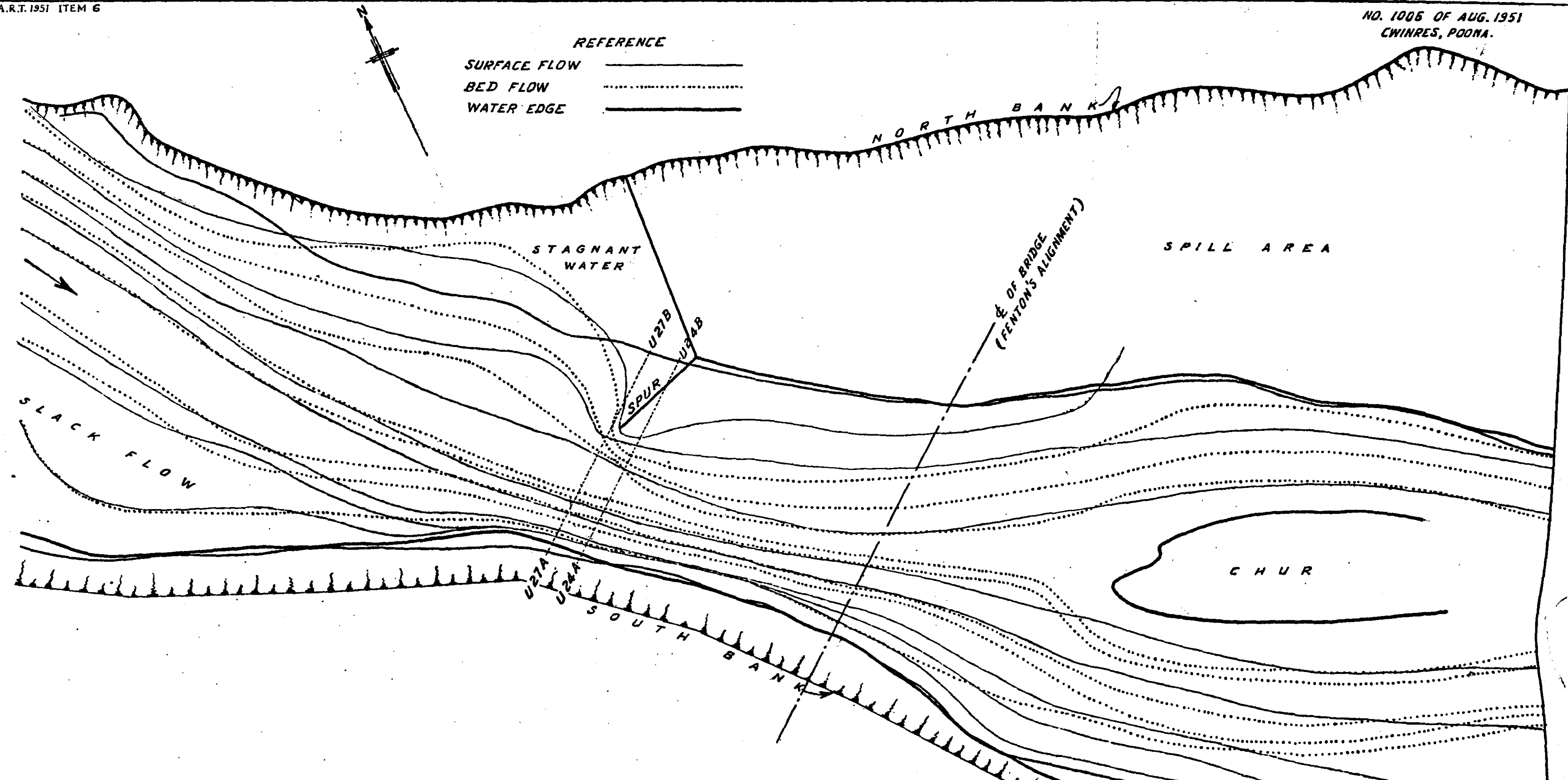


FIG. 1.
 MODEL OF RIVER GANGA - MOKAMEH BRIDGE
 SHOWING LINES OF FLOW
 RIVER COURSE 1948
 SPUR: 5000 FEET LONG; (2.50 MILES U/S OF BRIDGE SITE)
 DISCHARGE = 7.2 Cs. = 1800,000 Cs.
 MODEL SCALES: - HOR. 1/500 VER. 1/67
 DRAWING SCALE: - 1" = 4000 FEET.

COMP. *Munshi*
 TRACED BY: PARKHI. 30-8-51.

Ganga River Model for Rail-Cum-Road Bridge
at Mokameh-Bihar Province.

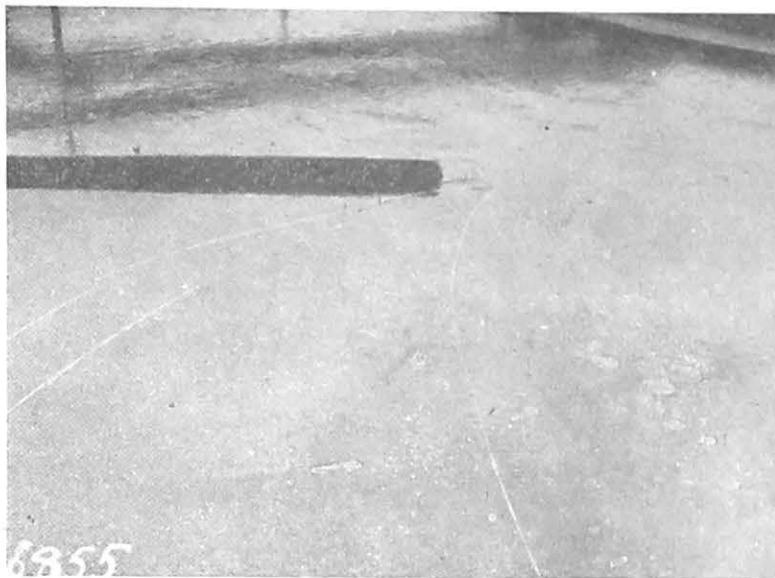


Photo No. 6855

Flow lines, looking downstream with a repelling spur 5000 ft. long. 1948 River course. $Q \cong 18,00,000$ cusecs.

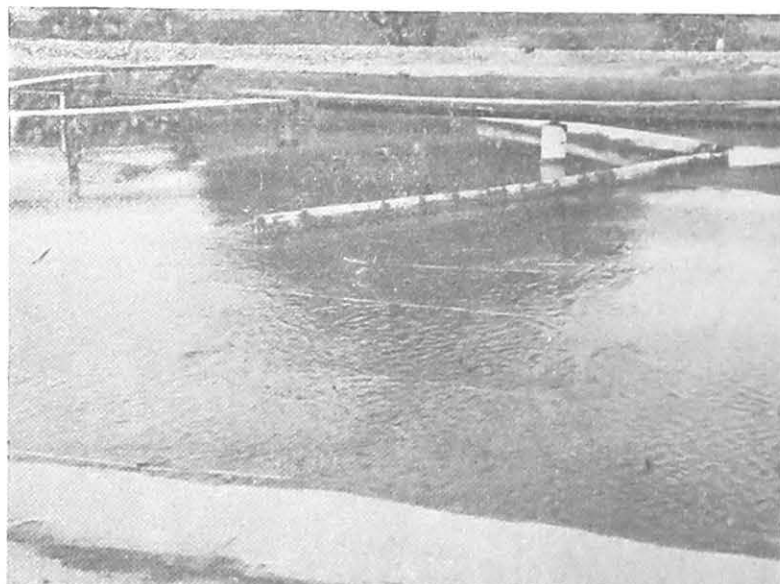


Photo No. 6857

Flow lines, looking upstream with a repelling spur 5000 ft. long. 1948 River course $Q \cong 18,00,000$ cusecs.

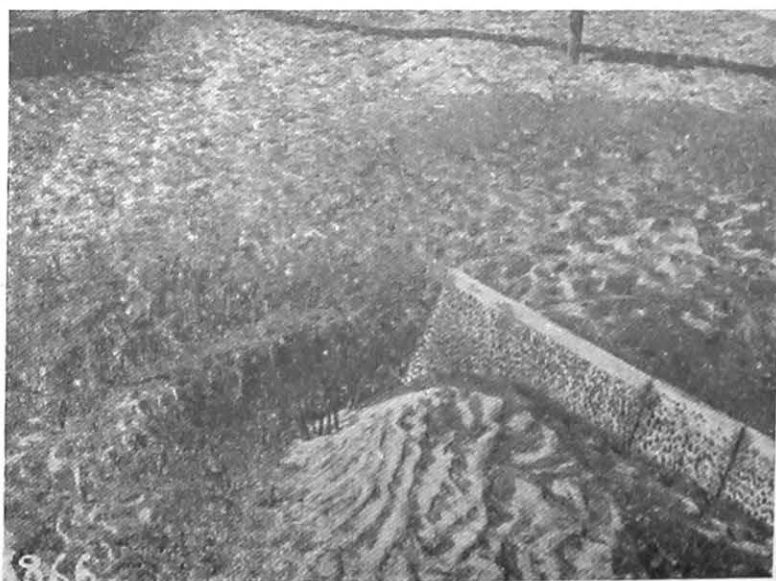
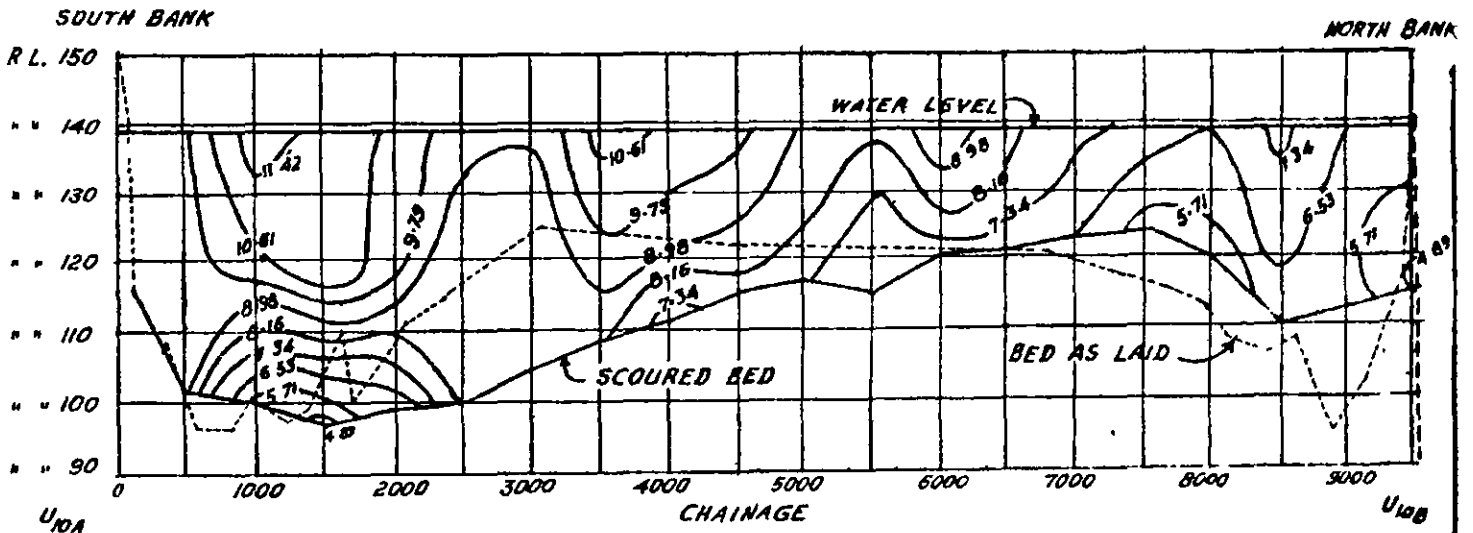
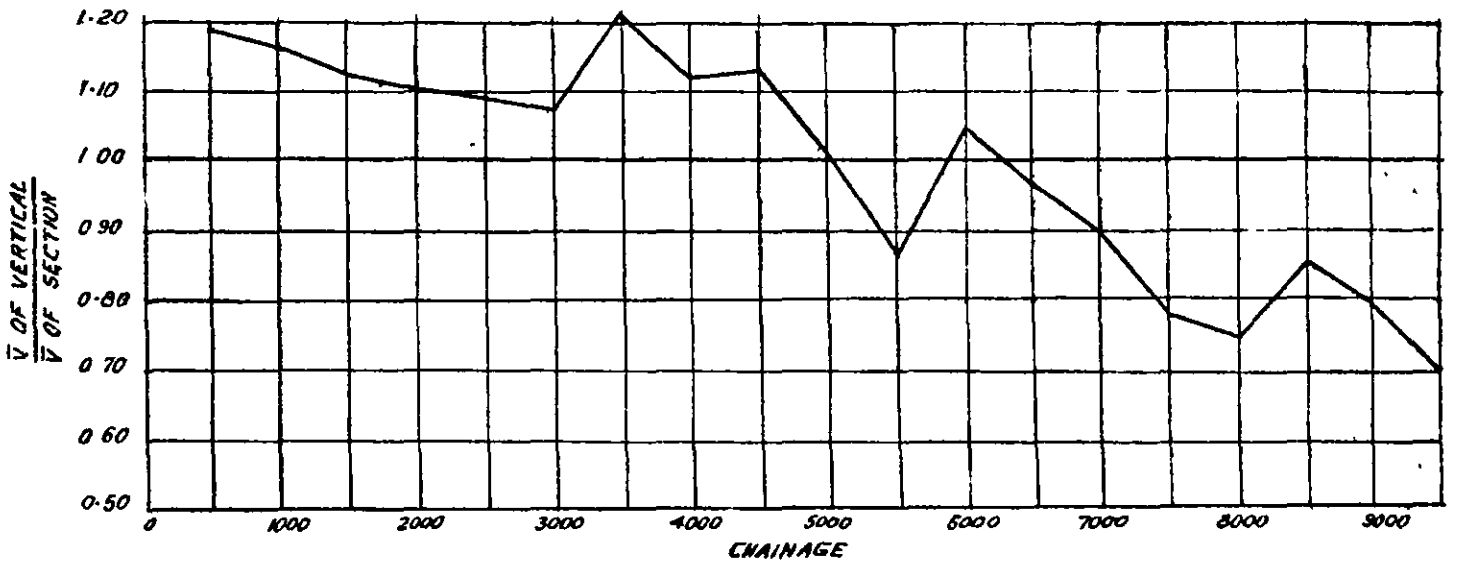


Photo No. 6866

Scour pattern near the spur head looking upstream with 5000 ft. long repelling spur. 1948 River course. $Q \cong 18,00,000$ cusecs.



REFERENCES:-

- BED AS LAID -----
- SCOURED BED —————
- RIVER COURSE 1948
- DISCHARGE $Q = 7.2 \text{ Cs.} \approx 1800,000 \text{ CUSECS.}$

FIG. 2
MODEL OF RIVER GANGA - MOKAMEH BRIDGE
VELOCITY DISTRIBUTION CURVES IN SECTION $U_{10A} - U_{10B}$
(1 MILE U/S OF BRIDGE SITE)
WITH
REPELLING SPUR 5000 FEET LONG
(2.5 MILES U/S OF BRIDGE SITE)
MODEL SCALE :- HOR. $1/500$, VER. $1/67$
DRAWING SCALE :- HOR. $1" = 1500'$ VER. $1" = 30'$

COMP BY *M. Singh*
TRACED BY *NATK R 5 10-8-51*

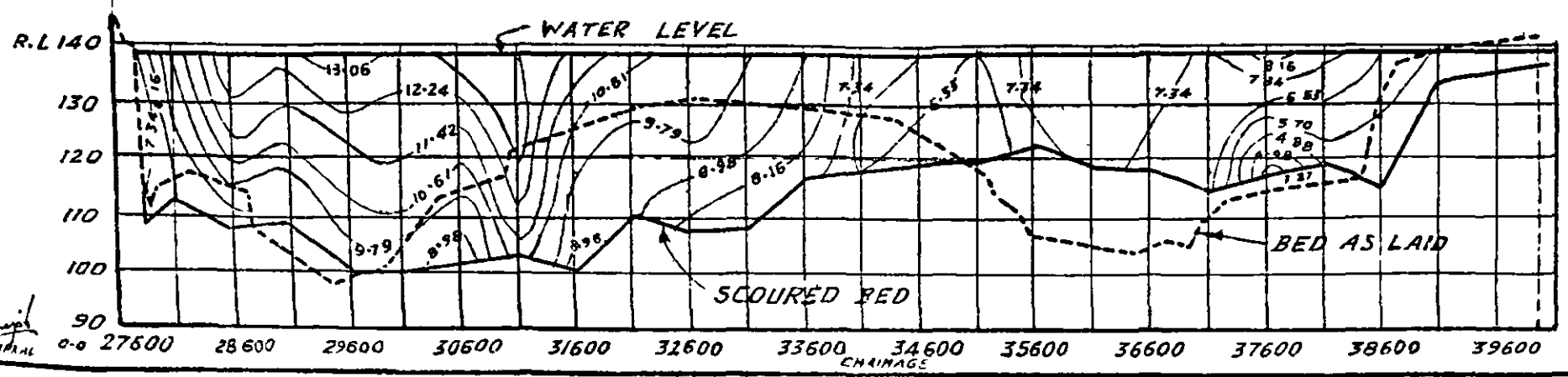
A RT 1951 ITEM 6



FIG. 3.
MODEL OF RIVER GANGA
MOKAMEH BRIDGE
VELOCITY DISTRIBUTION CURVES
BRIDGE ALIGNMENT CROSS SECTION
WITH
REPELLING SPUR 5000 FT. LONG.
(2.5 MILES U/S OF BRIDGE SITE)
MODEL SCALE - HOR. 1"=500', VER. 1"=67'
DRAWING SCALE - HOR. 1"=1500', VER. 1"=30'

SOUTH BANK.

NORTH BANK.



SCOURED BED ———
BED AS LAID - - - - -
RIVER COURSE - 1948 ·····
DISCHARGE - Q=7.2 Cs. ≈ 18,00,000 Cs.

Manoj
201 SERIAL

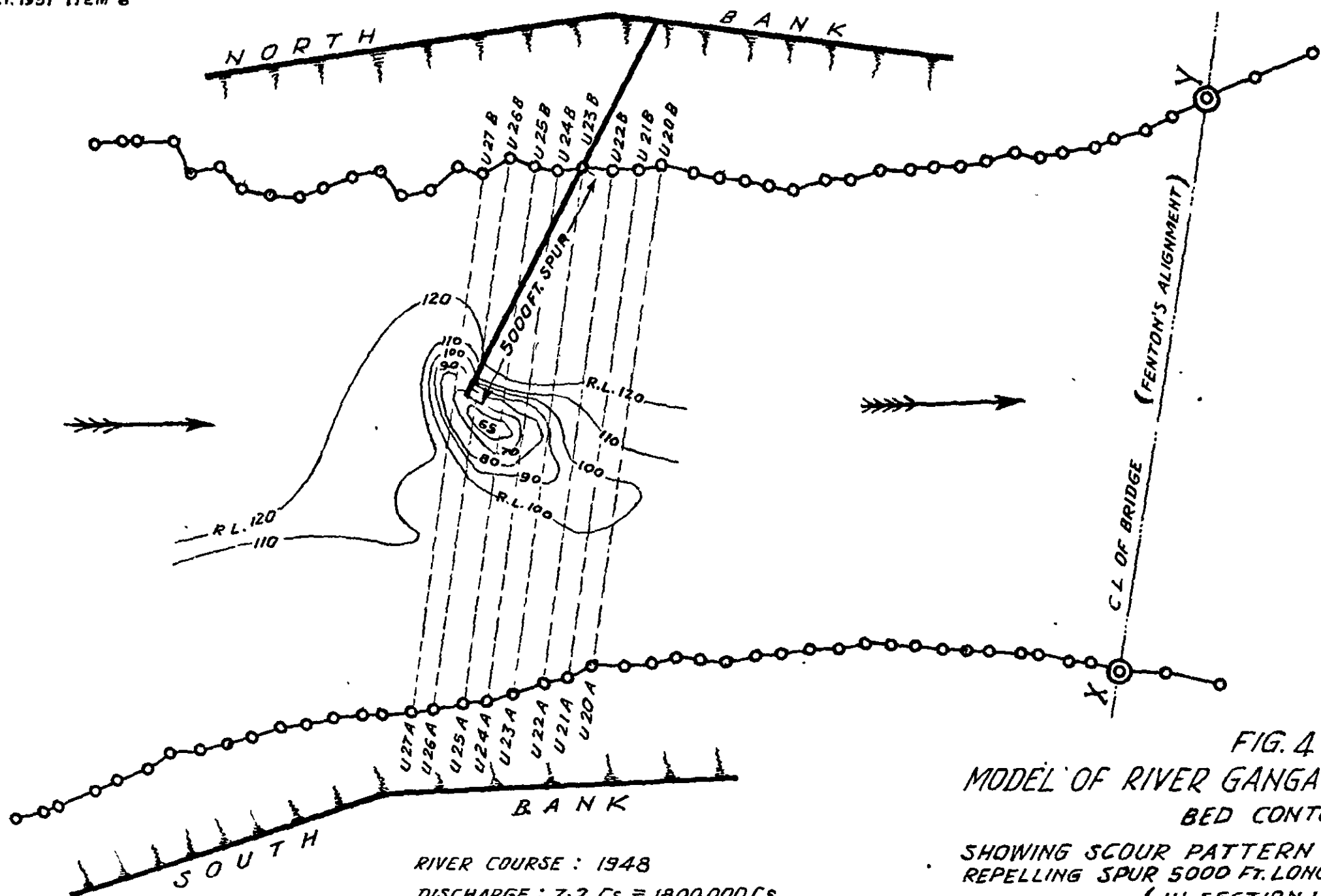


FIG. 4.
 MODEL OF RIVER GANGA-MOKAMEH BRIDGE
 BED CONTOURS
 SHOWING SCOUR PATTERN NEAR HEAD OF THE
 REPELLING SPUR 5000 FT. LONG U/S OF BRIDGE SITE
 (IN SECTION U 23 B)
 MODEL SCALES - HOR. 1/500, VER. 1/67
 DRAWING SCALE - 2" = 1 MILE

RIVER COURSE : 1948
 DISCHARGE : 7.2 Cs. \approx 18,00,000 Cs

COMP BY *M. S. K.*
 TRACED BY SARKAL 13-8-51

A.R.T. 1951 ITEM 6

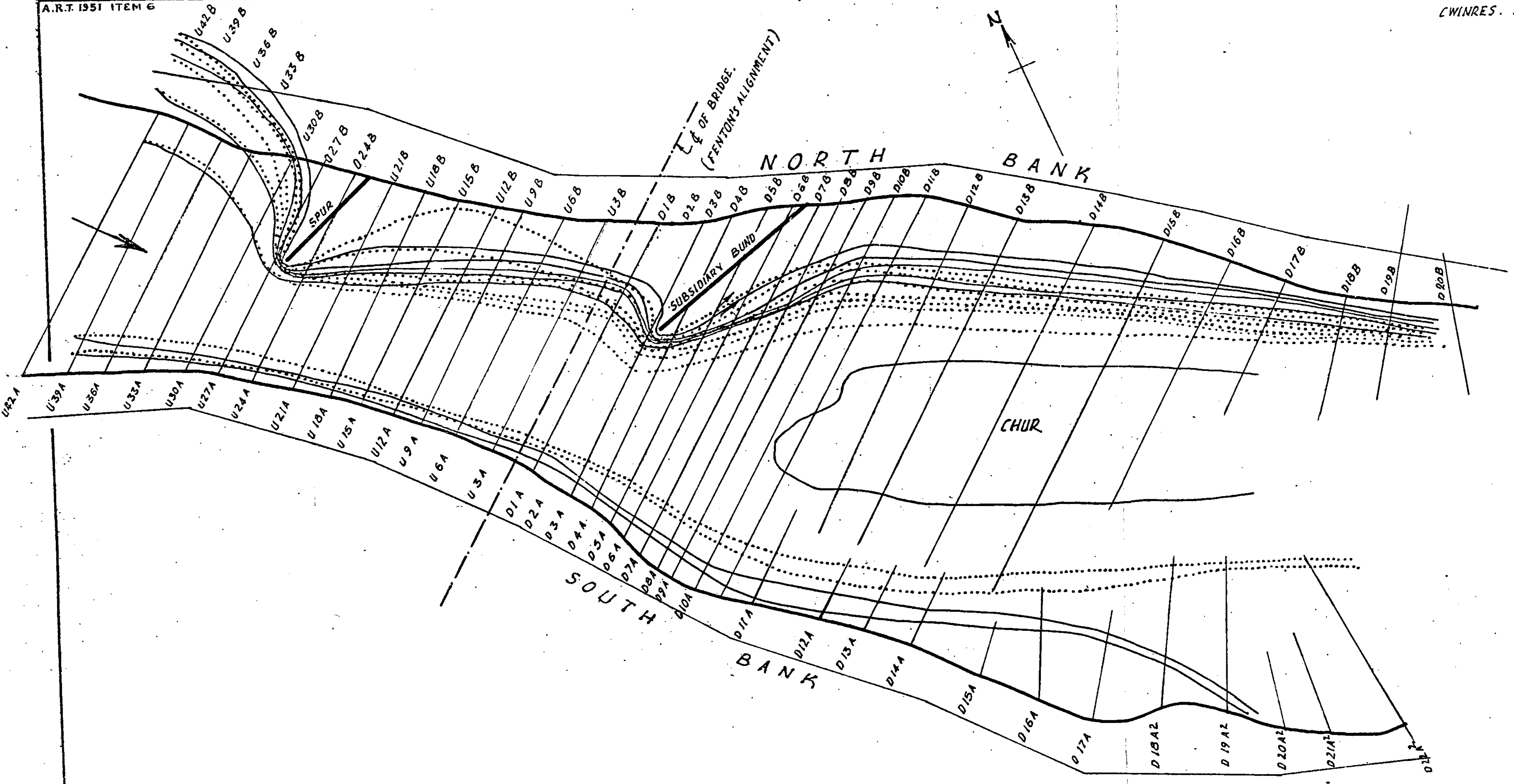


FIG.5
MODEL OF RIVER GANGA - MOKAMEH BRIDGE
SHOWING LINES OF FLOW

RIVER COURSE:- EXISTING CONDITIONS 1948.
 SPUR 5000 FT LONG (2.50 MILES U/S OF BRIDGE SITE.)
 WITH A 8750 FT. SUBSIDIARY BUND D/S OF BRIDGE SITE

REF:-
 SURFACE FLOW. —————
 BED FLOW.
 DISCHARGE: Q = 7.2 Cs. = 18,00,000 Cs.

MODEL SCALE:- HOR. 1/500 VER. 1/67
 DRAWING SCALE:- 1" = 4000'

COMP. BY:- *M. Bhangare*

BY:- BHINGARE. 14-8-1951.

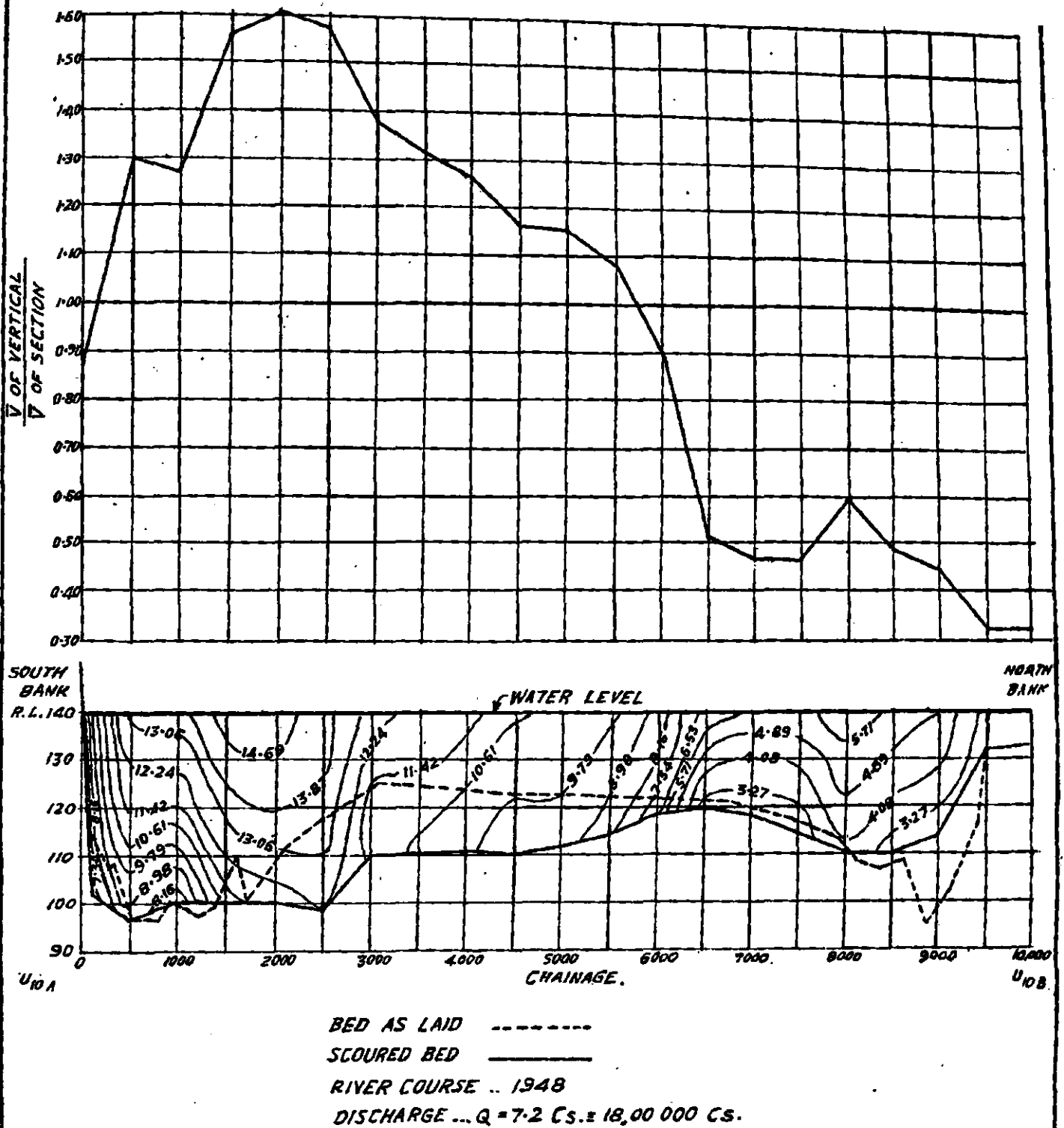
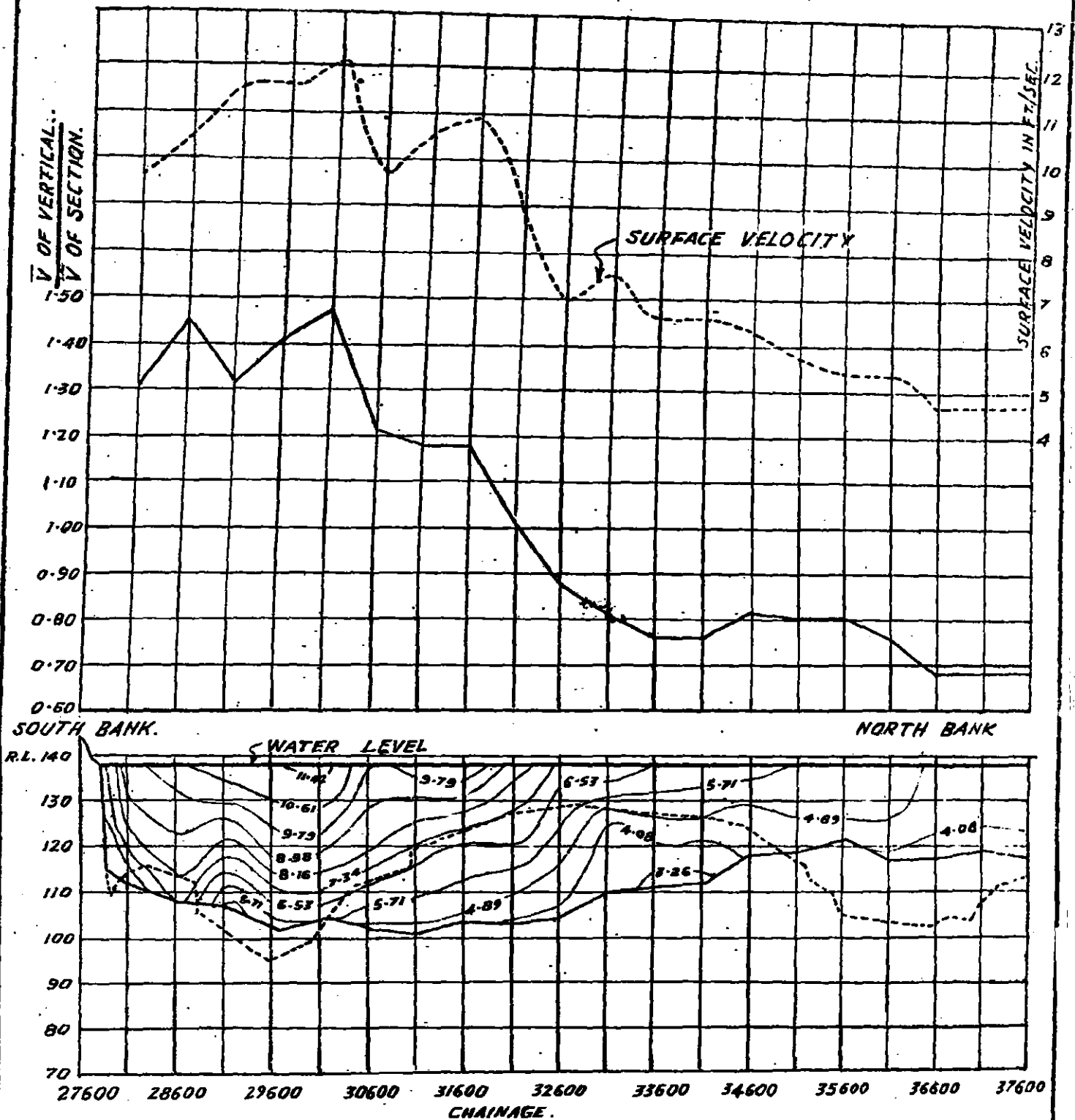


FIG. 6
 MODEL OF RIVER GANGA - MOKAMEH BRIDGE
 VELOCITY DISTRIBUTION CURVES
 CROSS SECTION U_{10A}-U_{10B} (1 MILE U_{1/5} OF BRIDGE SITE)
 WITH
 A REPELLING SPUR 5000 FT. LONG U_{1/5} OF BRIDGE SITE
 & A SUBSIDIARY BUND 8750 FT. LONG U_{1/5} OF BRIDGE SITE

MODEL SCALE - HOR. 1/500, VER. 1/67
 DRAWING SCALE: HOR. 1" = 1500' VER. 1" = 30'

COMP. BY: *M. S. Mungil*
 TRACED BY: B. G. DESHPANDE B-8-51.



REFERENCE.
 BED AS LAID -----
 SCOURED BED —————
 RIVER COURSE 1948
 DISCHARGE-Q=7.2 Cs. = 18,00,000 Cs.

FIG.-7
 MODEL OF RIVER GANGA-MOKAMEH BRIDGE
 VELOCITY DISTRIBUTION CURVES
 BRIDGE ALIGNMENT - CROSS SECTION WITH
 A REPELLING SPUR 5000 FT. LONG U/S OF BRIDGE SITE
 AND
 A SUBSIDIARY BUND 8750 FT. LONG D/S OF BRIDGE SITE
 MODEL SCALE - HOR. 1/500, VER. 1/67
 DRAWING SCALE - HOR. 1"=1500, VER. 1"=30

M. Singh
 BY - JAPKAL.

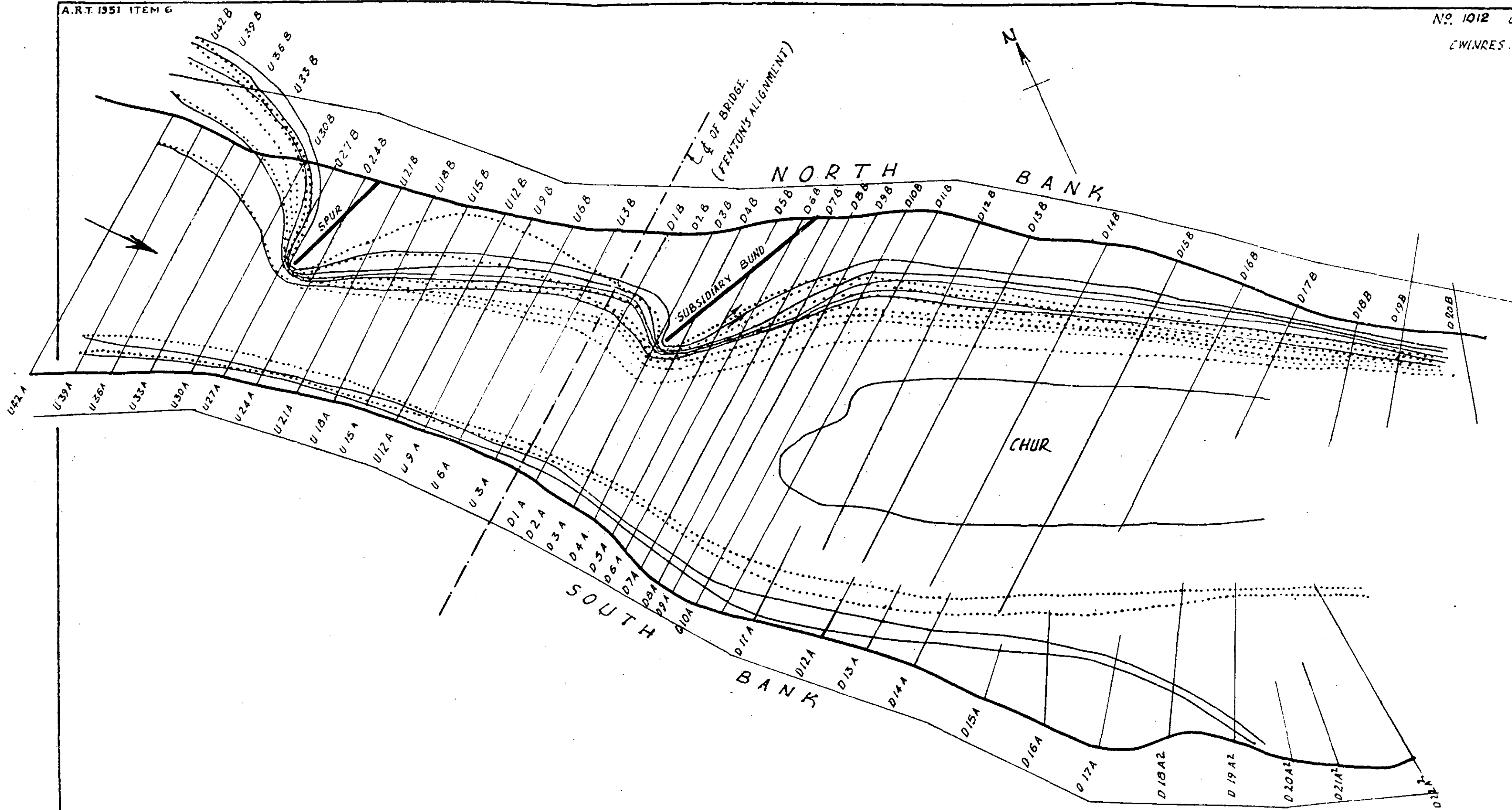


FIG.5
MODEL OF RIVER GANGA - MOKAMEH BRIDGE
SHOWING LINES OF FLOW
 RIVER COURSE:- EXISTING CONDITIONS 1948.
 SPUR 5000 FT LONG (2.50 MILES U/S OF BRIDGE SITE.)
 WITH A 8750 FT. SUBSIDIARY BUND U/S OF BRIDGE SITE

REF:-
 SURFACE FLOW. —————
 BED FLOW.
 DISCHARGE: $Q = 7.2 \text{ Cs.} \equiv 18,00,000 \text{ Cs.}$

MODEL SCALE:- HOR. 1/500 VER. 1/67
 DRAWING SCALE:- 1" = 4000'

COMP. BY:- *M. N. Singh*

DRY:- BHINGARE. 14-8-1951.

7. MODEL EXPERIMENTS FOR A ROAD-cum-RAIL BRIDGE ON GANGA AT PATNA

Introduction

The necessity for a bridge across the Ganga River in Bihar has been felt for a very long time, particularly because there is no crossing below Benares. The question of building a bridge in Bihar was first taken up in 1907 but it lingered on because the two Railway Companies on the two sides could not come to a common decision. Finally in 1945, the Railway Board directed the E. I. Railway to study the sites and prepare the project. Mr. Fenton, Dy. Chief Engineer, E. I. Railway carried out investigations for the site of a bridge both at Patna and Mokameh Ghat. He, however, as a result of his findings favoured the Mokameh site and prepared a detailed project for the same.

1.1. The State Government of Bihar, however, favour the Patna site, because they feel that a bridge at Patna would be a vital means of communication between the South and North Bihar. Hence the Bihar P. W. D. carried out detailed investigations for the Patna site.

1.2. *The River*—The river has a considerable meander belt within the large width of the Khadir between the Sone confluence and Digha. Figure 1 shows the two extreme courses of 1828 and 1949. Between Digha and Patna the width of the Khadir is about 5 miles. The effect of the main channel being pinned down to the South bank has given rise to a nodal region. Because of the presence of this nodal region above Patna the sites have been selected in its vicinity. Mr. Fenton proposed a site about $\frac{3}{4}$ th mile upstream of Patna. Here the minimum winter river width being about 9,000 ft. he proposed 9,000 ft. as the waterway though it was much in excess of the waterway required for the highest flood discharge. In the Bihar investigations they proposed two sites: (1) about one mile and (2) $1\frac{1}{2}$ miles upstream of the Fenton's site. These sites have the advantage of the waterway being reduced to the minimum required to pass the highest flood discharge. Moreover these sites being further away from the Patna Bank, it was considered that the turbulence of the water emerging out of the constricted bridge might get reduced before its reaching the Patna Bank and hence the city would remain safe. All these sites are shown in Figure 1.

Fig. 1.

Fig. 1.

2. The object of these experiments was to find out if any of the proposed sites in the vicinity of Patna were suitable. The main criterion kept in view for all upstream sites was that the river attack on Patna Bank should not be allowed to worsen by the bridge. Also the safety of the bridge itself and the protection it afforded to the Kasmar bluff were also considered. The Kasmar bluff needs some artificial protection, because the bore-hole data shows that there is clay only for the top 15 ft and below that, is all sand.

3. Survey Data Available

The river reach between Goldengunj and Futwah was surveyed by means of a number of cross-sections (1949 dry weather period) connected together by a transverse so that the same could be plotted on ground to make up the model. However as the transverse survey was done long after the cross-sections were observed, some differences were detected between the survey plan and the aerial photo. The model on stabilizing corrected these errors.

4. The Model

The model was laid to the following scales and covered a reach from Goldengunj—21 miles upstream of Fenton's Bridge Site, down to Futwah—14.5 miles below the bridge site.

- (1) Length scale— $1/500$.
- (2) Depth scale— $1/67$.
- (3) Discharge scale— $1/2,50,000$.
- (4) Time scale— $1/545$, i.e., 1 day = 264 minutes.
- (5) Vertical exaggeration—75.
- (6) The bed was laid to a slope exaggeration of 75.

The same scales were used as those adopted for the model of the Ganga river for the proposed bridge site at Mokameh.

5. Hydraulic Data

In the investigations carried out by Mr. Fenton, he had collected certain data about the water levels and corresponding discharges; a stage discharge curve was prepared, which showed a maximum discharge of 30,00,000 cusecs corresponding to the highest high flood level of 169.3 observed in the year 1923.

5.1. The Bihar P. W. D. took similar observations in 1949 and 1950 and plotted the stage discharge curves which show a maximum flood discharge of 24,00,000 and 25,70,000 cusecs respectively extrapolated from 1949 and 1950 curves, corresponding to the highest flood level of 169.3.

5.2. All the model observations were made with a discharge of 18,50,000 cusecs for all sites upstream of Patna and 20,25,000 cusecs for sites below Patna as this represents the bankful and hence the dominant stage.

5.3. The only data available for proving the model was as follows:—

- (1) Gauge values of the various gauges.
- (2) High flood cross-sections at the Fenton's site.
- (3) Visual observations showing the river attack on the Patna Bank.

6. Model Experiments

After making the preliminary runs of the hydrograph cycle to stabilize, the model was proved by the reproduction of the known conditions. Afterwards the flow lines, cross-sections and the bed contours were observed. Figure 2 shows the flow lines under existing conditions and Figure 3 shows the bed contours of the river obtained up to a flood discharge of 18,50,000 cusecs. Photos 383 to 386 show bed contours near the various bridge sites and near the Patna bank and photos 340 to 342 show the flow lines under existing conditions.

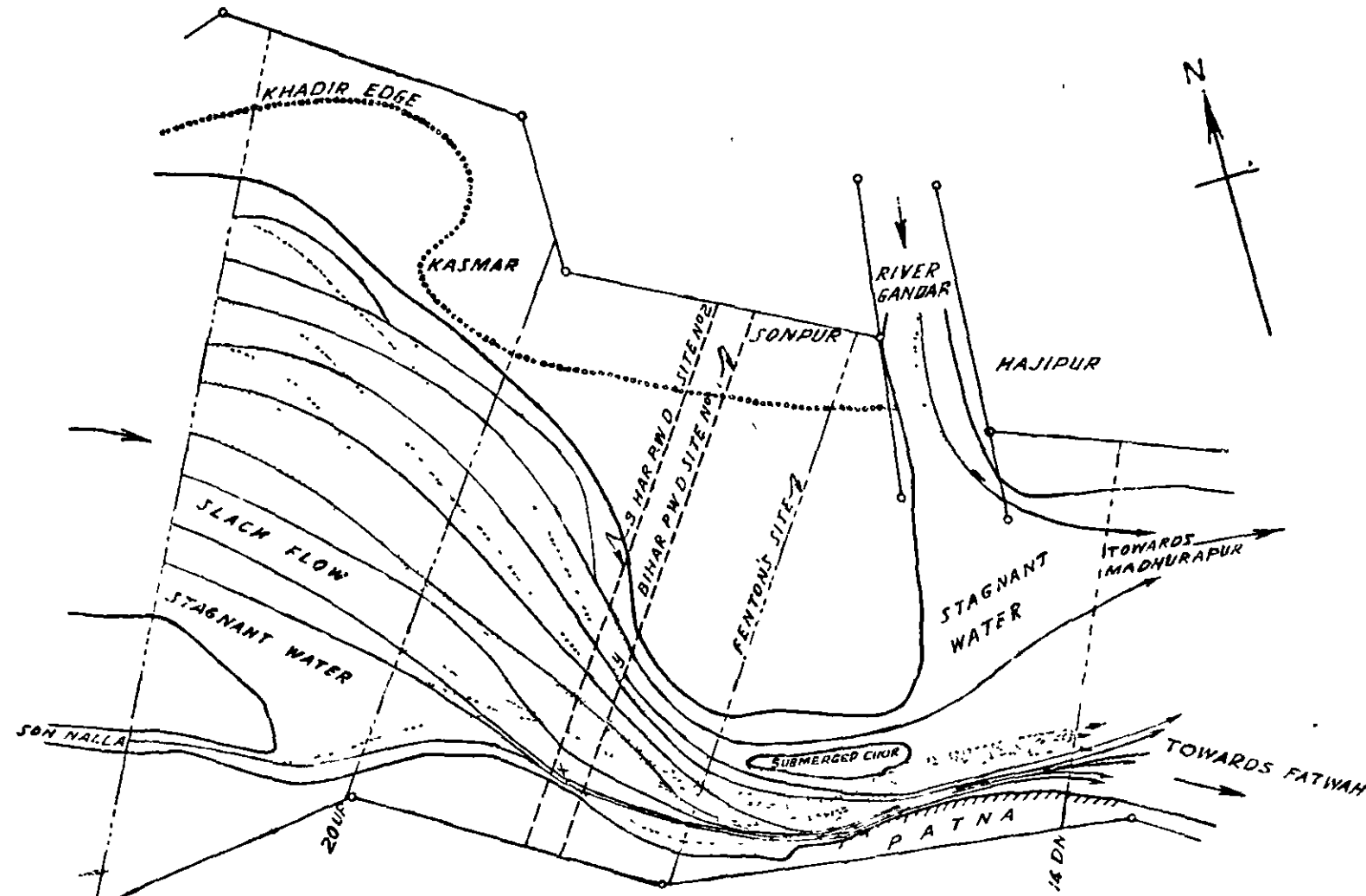
Fig. 2.
Fig. 3.
Photos 383
to 386.
Photos 340
to 342.

Fig. 1. The following bridge sites were then tested under the present (1949) river course:—

- (1) Existing conditions without a bridge.
- (2) With a bridge upstream of Patna at Bihar P. W. D., Site No. 2.
- (3) Bihar P. W. D., Site No. 1.
- (4) Fenton's site with the 1949 river course and with worst assumed river course.
- (5) Site proposed by the Director, C. W. & P. Research Station, Poona.
- (6) With a bridge below Patna at Bihar P. W. D., Site No. 3.
- (7) Fenton's alternative site across Futwah channel.

In all experiments the Guide Bund head was made up with a 3° curve up to 90° sweep and a 5° curve from 90° to 120°, which gave the best results in the Mokameh bridge experiments. The tail head had a radius of 800 ft.

Table 1 below shows the bridge waterway and the Guide Bund length tested in each experiment along with the deepest scour obtained near the Patna Bank between the Mahendru Ghat and the Rani Ghat.



RIVER COURSE: 1949
DISCHARGE. Q = 7.4 CS = 18,50,000 CS

FIG: 2
MODEL FOR A ROAD-LUM-RAIL BRIDGE
ON THE GANGA AT PATNA
PLAN SHOWING FLOW LINES
UNDER EXISTING CONDITIONS.
MODEL SCALES: (HOR. 1/500
VER. 1/67.
DRAWING SCALE 1"=10,000 FT.

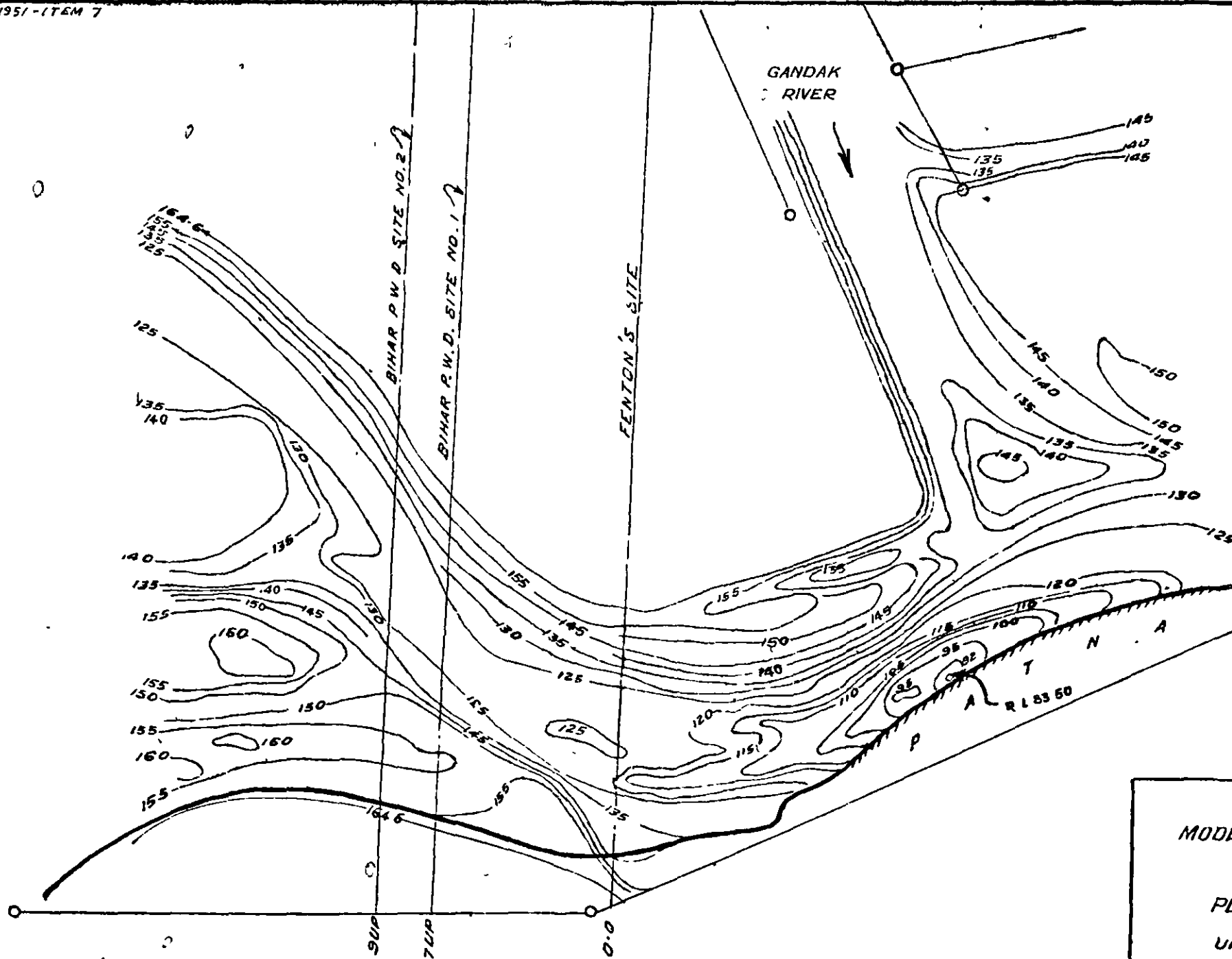
REFERENCE--

SURFACE FLOW —————

BED FLOW - - - - -

PITCHED BANK |||||

COMPARED BY: *M.S.*
TRACED BY: M. K. JAIN'S.



RIVER COURSE - 1949
DISCHARGE - 74 Cs. = 18,50,000 Cs

FIG:3
 MODEL FOR A ROAD-CUM-RAIL BRIDGE
 ON THE GANGA AT PATNA
 PLAN SHOWING BED CONTOURS
 UNDER EXISTING CONDITIONS
 MODEL SCALES:-HOR:1/500 VER:1/67
 DRAWING SCALE: 1" = 5000 FT

COMP. BY: *M. Singh*
 TRACED BY: KANETKAR, R. K.
 21-1-52

ITEM No. 7.
A. R. T. 1951.

Road-Cum-Rail Bridge on the Ganga at Patna.

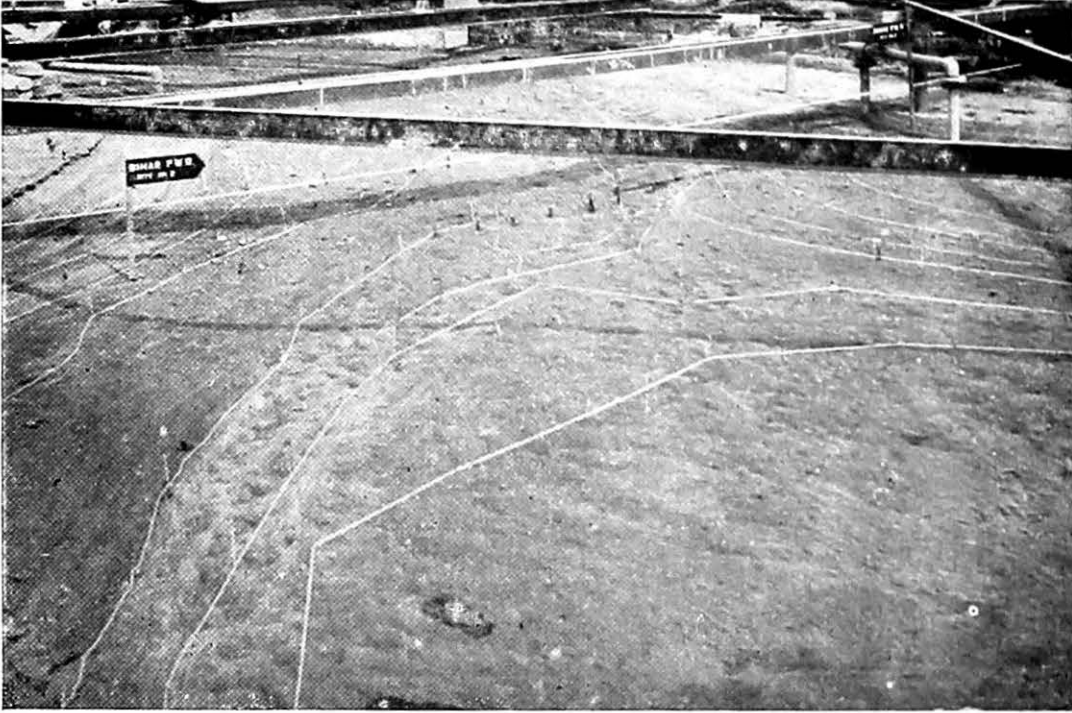


Photo 383

Bed contours under existing conditions, near Bihar
P. W. D. site No 2. Looking downstream



Photo 384.

Bed contours under existing conditions near Bihar
P. W. D Site No. 1 looking upstream.

Road-Cum Rail Bridge on the Ganga at Patna.

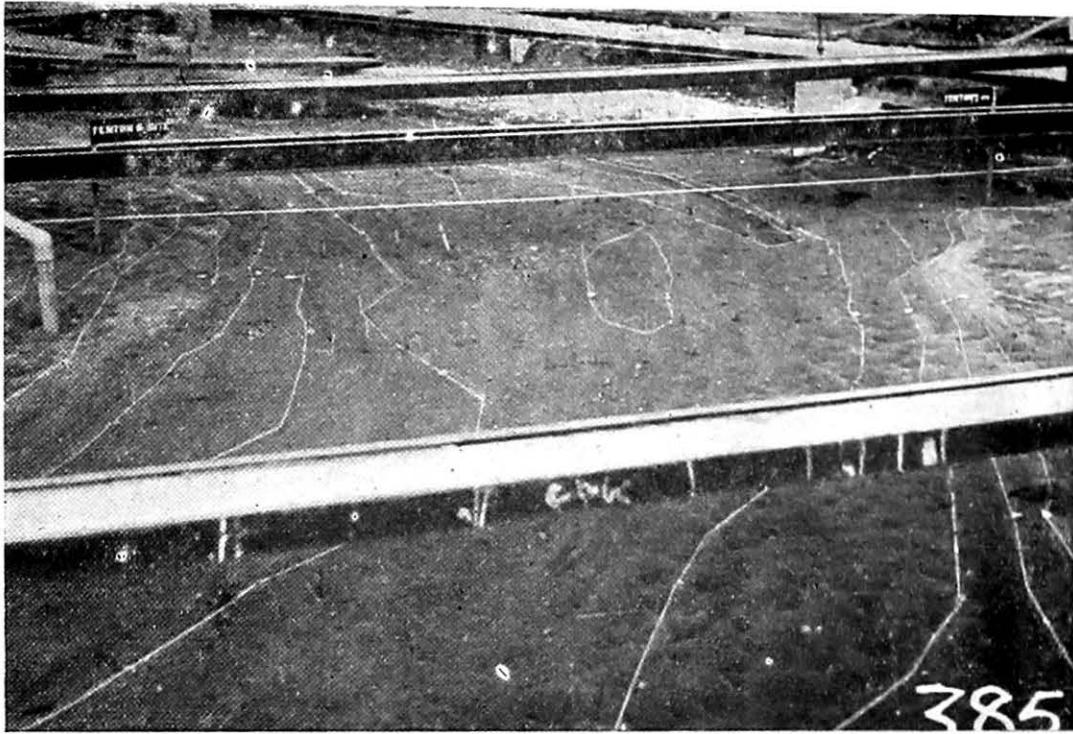


Photo 385.
Bed contours under existing conditions near Fenton's site
looking downstream.

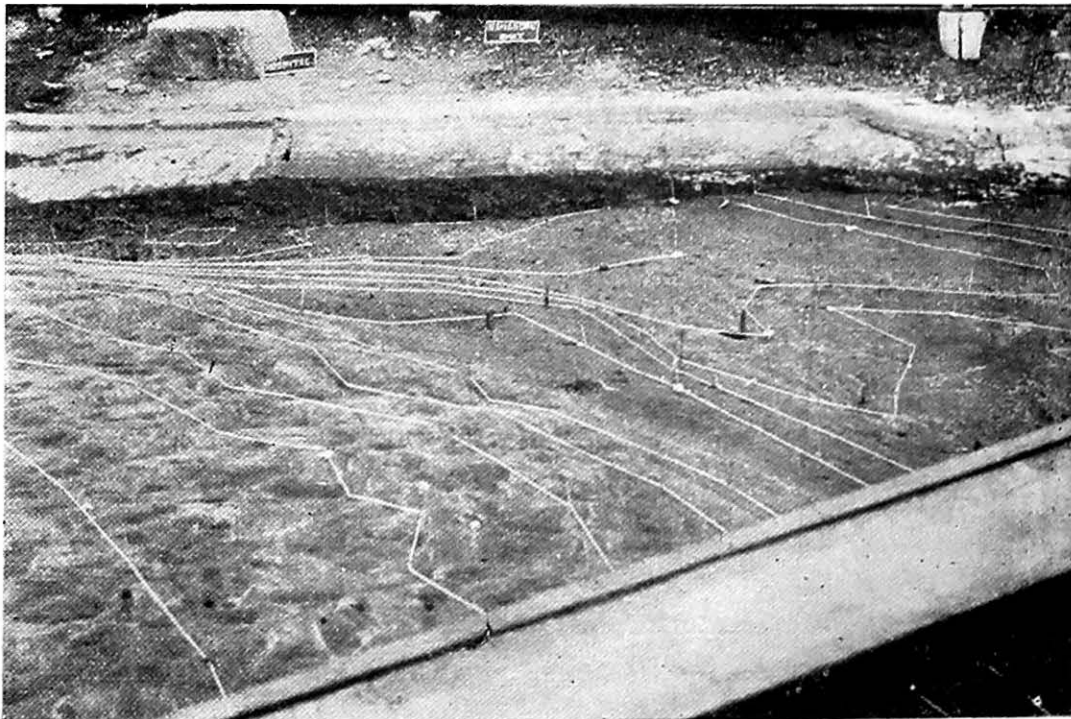


Photo 386
Bed contours under the existing conditions
near the Patna - Bank, showing deep scour.

Model of the Ganga at Patna for a Road cum Rail Bridge

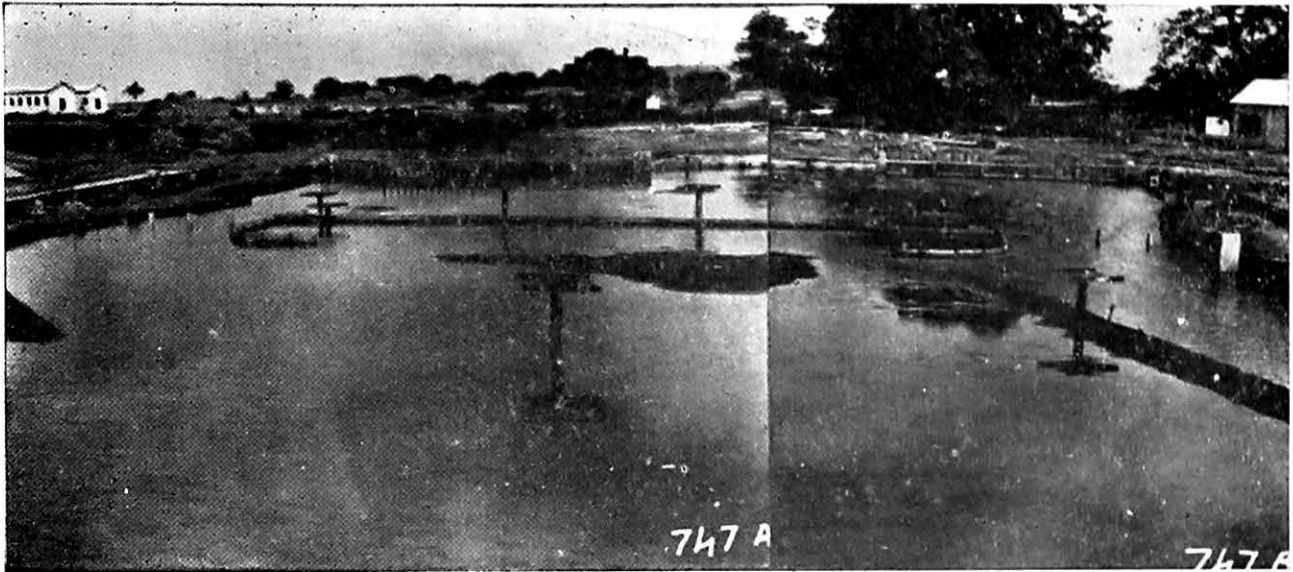


Photo 747 A B.

View of the two bridges across Fatwah and Mehnar channels. Looking downstream.

$Q \equiv 20, 25,000$ cusecs



Photo 340.

Flow line near Fenton's Site. Existing conditions
Looking downstream

$Q \equiv 7.4$ cusecs $\equiv 18,50,000$ cs

ITEM No. 7.
A. R. T, 1951

Road-Cum Rail Bridge on the Ganga at Patna.

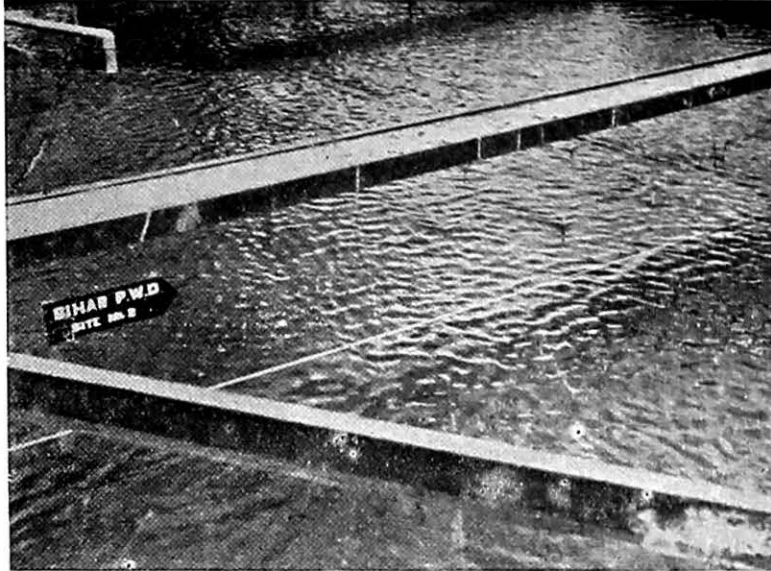


Photo 341

Flow lines near Bihar P. W. D. site No 2
Existing conditions Looking downstream.

$$Q = 7.4 \text{ cs.} \equiv 18,50,000 \text{ cs.}$$



Photo 342.

Flow lines near the Patna Bank. Looking from North Bank.

$$Q = 7.4 \text{ cs.} \equiv 18,50,000 \text{ cs.}$$

TABLE 1—Results of experiments on sites upstream of Patna

Experiment No.	Bridge site.	River Course.	Waterway. (Feet)	Length of Guide Bund. (Feet)	Level of deepest scour R. L.	Scour depth below average flood level near Patna Bank. (Feet)
1.	Existing conditions . . .	1949	—	—	83.50	80.00
2.	Bihar P. W. D. Site No. 2 . . .	„	6,000	5,000	56.50	107.00
3.	Bihar P. W. D. Site No. 1 ..	„	6,000	5,000	66.50	97.00
4.	Fenton's Site . . .	„	9,240	5,000	86.50	77.00
	Do.	„	9,240	10,000	82.00	81.50
5.	CWINRES Site	„	6,000	6,500	84.50	70.50
	Do.	„	6,000	10,000	70.00	93.50
6.	Fenton's Site	Worst assumed course.	9,240	6,000	73.50	90.00

7. Sites Upstream of Patna

Figures 4 to 10 show the flow lines with the proposed bridge at different sites and the Figures 11 to 17 show bed contours near the Patna Bank with a flood of 18,50,000 cusecs. Under the present conditions there was strong diving flow near the Patna Bank with the surface flow concentrated near the bank and the bed flow deflected away from the bank and the resultant scour had reached a bed level of R. L. 83.50 as can be seen from Figure 3. With the bridge (waterway 6,000 ft.) at Bihar P. W. D., Site No. 2, the strength of the diving flow increased further and consequently the bed scour reached a level of R. L. 56.50 as seen from Figure 11. The experiment clearly showed that the water emerging out of the bridge had not enough space for expansion below the bridge; on the other hand it concentrated near the Patna Bank near Mahendra Ghat thereby endangering the town. It was however, noted that this site had one advantage, viz., it had a tendency to straighten the upstream current thus affording some natural protection to the Kasmar bluff.

Figs. 4 to 10.
Figs 11 to 17.

Fig. 3.

Fig. 11.

7.1. A bridge at Bihar P. W. D., Site No 1 (waterway 6,000 ft.) slightly improved the conditions near the Patna Bank, the lowest scour produced reaching R. L. 66.50 (Figure 12), i.e, 10 ft. higher than with Site No 2. But in this case the Guide Bund head being further downstream as compared to Bihar P. W. D., Site No. 2, the attracting influence under the present course was less and the Kasmar bluff would thus be liable to be attacked.

Fig. 5.

Fig. 12

7.2. The next site to be tried was the Fenton's site with the bridge waterway of 9,240 ft. and left Guide Bund 5,000 ft. long. A bridge at this site was found to maintain the status quo in the river; actually the model showed a slight improvement as the deepest scour near Patna Bank was only down to R. L. 86.50 (Figure 13). But the splayed Guide Bund was only 5,000 ft. long and hence was insufficient for protecting the North approach embankment; the length was therefore increased to 10,000 ft. This increase in the length of the Guide Bund slightly worsened the conditions near the Patna Bank, producing the lowest scour down to R. L. 82.00 (Figure 14). As will be seen from the flow lines in Figures 6 and 7, there was a lesser divergence between the surface and bed flow lines showing a reduction in the strength of the diving flow and hence reduced attack with 5,000 ft. long guide bund.

Fig. 6.

Fig. 13.

Fig. 7.

Fig. 14.

7.3 Next the site proposed by the Director, CWIN Research Station, Poona, which was midway between Bihar P. W. D., Site No. 1 and Fenton's site was tested, with the Guide Bunds 6,500 ft long. In this case the location and alignment of the left Guide Bund was the same as in the previous case (with 10,000 ft. length of Guide Bund of Fenton's design) and the position of the mole head was also kept the same. It was expected to give results similar to those with Fenton's site with the extended length of the left Guide Bund of 10,000 ft. This was well corroborated by the model, as the deepest scour reached a level of 84.50 (Figure 15). The flow lines (Figure 8) also were quite similar to those under existing conditions. But this Guide Bund length of 6,500 ft. was not expected to give enough protection to the North approach embankment. Hence a length of 10,000 ft. was further tried. With the advance of the mole head, upstream, more water was deflected to the South bank thus intensifying the diving flow and the scour near the Patna Bank reached a value of R.L. 70.00 (Figure 16).

Fig. 8.

Fig. 15.

Fig. 8.

Fig. 9.

Fig. 16.

Fig. 1.
Figs. 10
& 17.

7.4. As the bridge at Fenton's site maintained *status quo* at the Patna Bank under the present river course (1949), it was decided to test it under a worst possible river course as shown in Figure 1. The results of this test are shown in Figures 10 and 17. In this case the mole head was fully under action of the strong current which embayed behind it. As seen from Figure 10 the surface currents behind the Guide Bund head were very active and showed a tendency to increase the embayment still further. The radius of the embayment curve was about 18 miles and the embayment already touched the approach embayment. Also because of the severe action on the mole head, the main flow was deflected to the South bank and the conditions at the Patna Bank worsened, the scour reaching a bed level of R. L. 73 00.

7.5 Instead of extending the guide bund, a groyne projecting from the North approach embankment as suggested by Fenton was tested. It was seen that the nose of this groyne was subject to a very severe attack; the high velocity flow after leaving the groyne attacked the head of the Guide Bund, which in turn, swung the flow with large boils and eddies towards the Patna Bank, thus increasing the attack there. As these conditions were decidedly bad, the experiment was discontinued.

8. Conclusion

The present protection works of the Patna Bank consist of brick pitching above the low water level and this pitching is anchored to vertical bullies driven into the bank at that level. Even under the existing conditions, the attack on the Patna bank is very active and as the protection works are not very strong, it is doubtful if they can hold against a more severe attack of the river.

- 8.1. A glance at Table 1 and Figure 1 will show that under the present river course, if the Guide Bund head is advanced more and more upstream, it would be brought into greater action and as a result thereof it would deflect the flow to the South bank endangering its stability. On the other hand bearing in mind the necessity of protecting the Kasmar bluff and also the North approach embankment of the bridge, a longer Guide Bund is necessary. Providing special protection for the Kasmar bluff a compromise between these diverse requirements would be obtained with a bridge at the Fenton's site; but the bridge would be safe under the present river course only, because with the worst assumed river course the embayment with a curve of 18 miles radius had touched the North approach embankment and attack on Patna bank also increased. Thus with this length of the Guide Bund it would not be possible to protect the North approach bank against an embayment of one mile radius under the worst assumed conditions of the river. If the Guide Bund length were to be increased to remedy this defect, it would increase the attack on the Patna bank as already seen.

8.2 With any proposal for a bridge site upstream of Patna, it is necessary to protect the Patna bank downstream as well as the North approach embankment upstream. Any proposal for a bridge site which gives better protection to the Patna bank reduces the safety of the approach bank which is liable to be attacked causing very heavy expenditure for maintenance every year; and *vice versa*. Under the circumstance no suitable site can be recommended upstream of Patna, keeping in view the safety of the town and the bridge.

9. Sites below Patna

Downstream of Patna there exists a large diara island some 16 miles long and 5 miles wide which has formed in the last 100 years or so, the river bifurcating into two channels around it. Up to 1944, the river steamer services ever since their inception 105 years ago, had used the South or Futwah Channel, but in 1944 the main channel of the river established itself along the North or Mehnar channel and subsequently the latter channel has further developed, the Futwah channel in the dry weather being now of meagre proportions. The past history shows that the Futwah channel is the main river course for a major period of the regime cycle. Taking this into consideration Fenton expected the river to swing back to Futwah channel within a few years and hence proposed an alternative bridge site as shown in Figure 1.

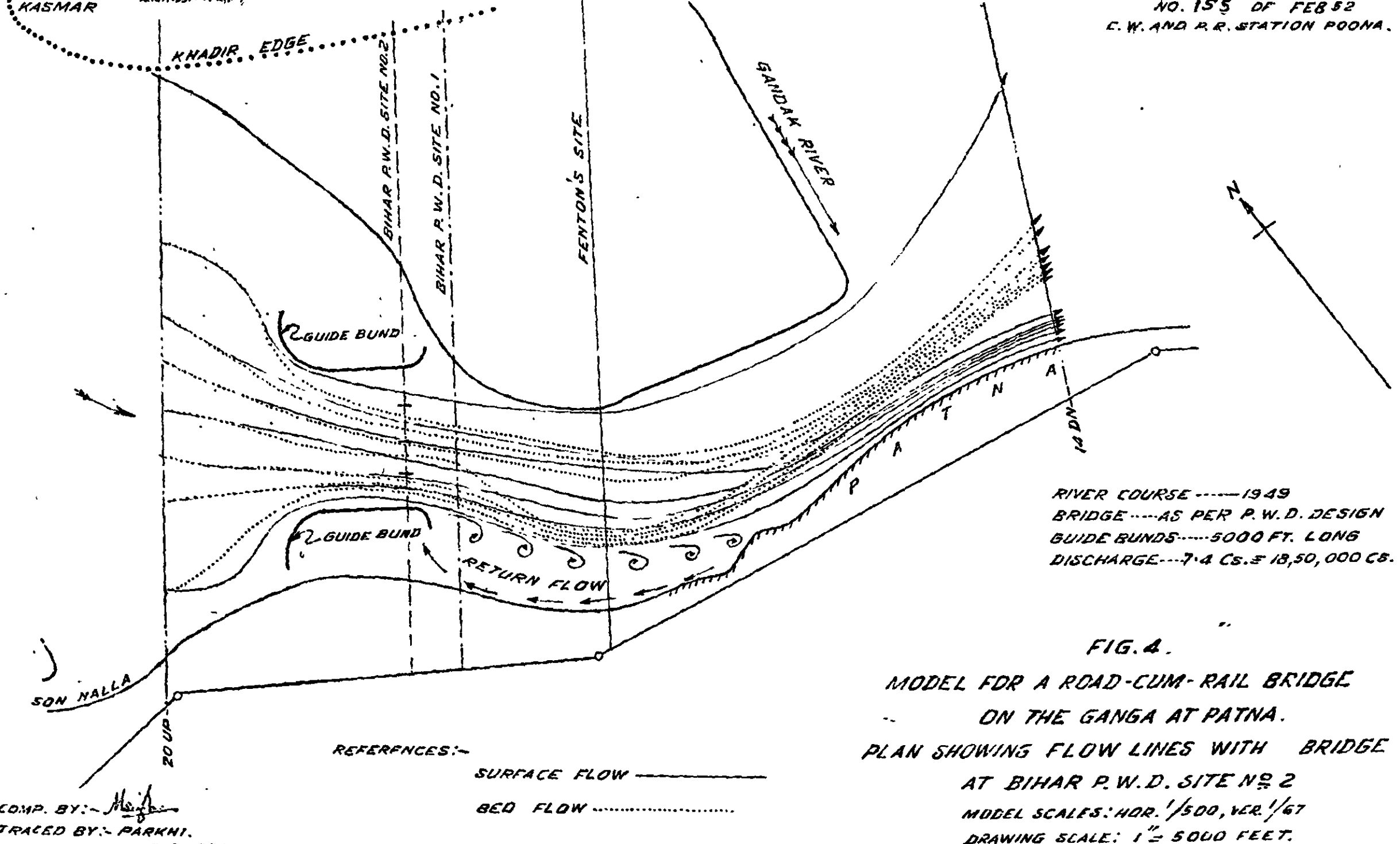
Fig 1.

The Bihar P. W. D., taking into account the present configuration of the river and considering the long periods that elapse before a major change takes place in the river, proposed a site across the Mehnar Channel.

KASMAR

ART. 151-ITEM 7

NO. 155 OF FEB 52
C.W. AND R.R. STATION POONA.



RIVER COURSE ----- 1949
 BRIDGE ----- AS PER P.W.D. DESIGN
 GUIDE BUNDS ----- 5000 FT. LONG
 DISCHARGE --- 7.4 Cs. ± 18,50,000 Cs.

FIG. 4.

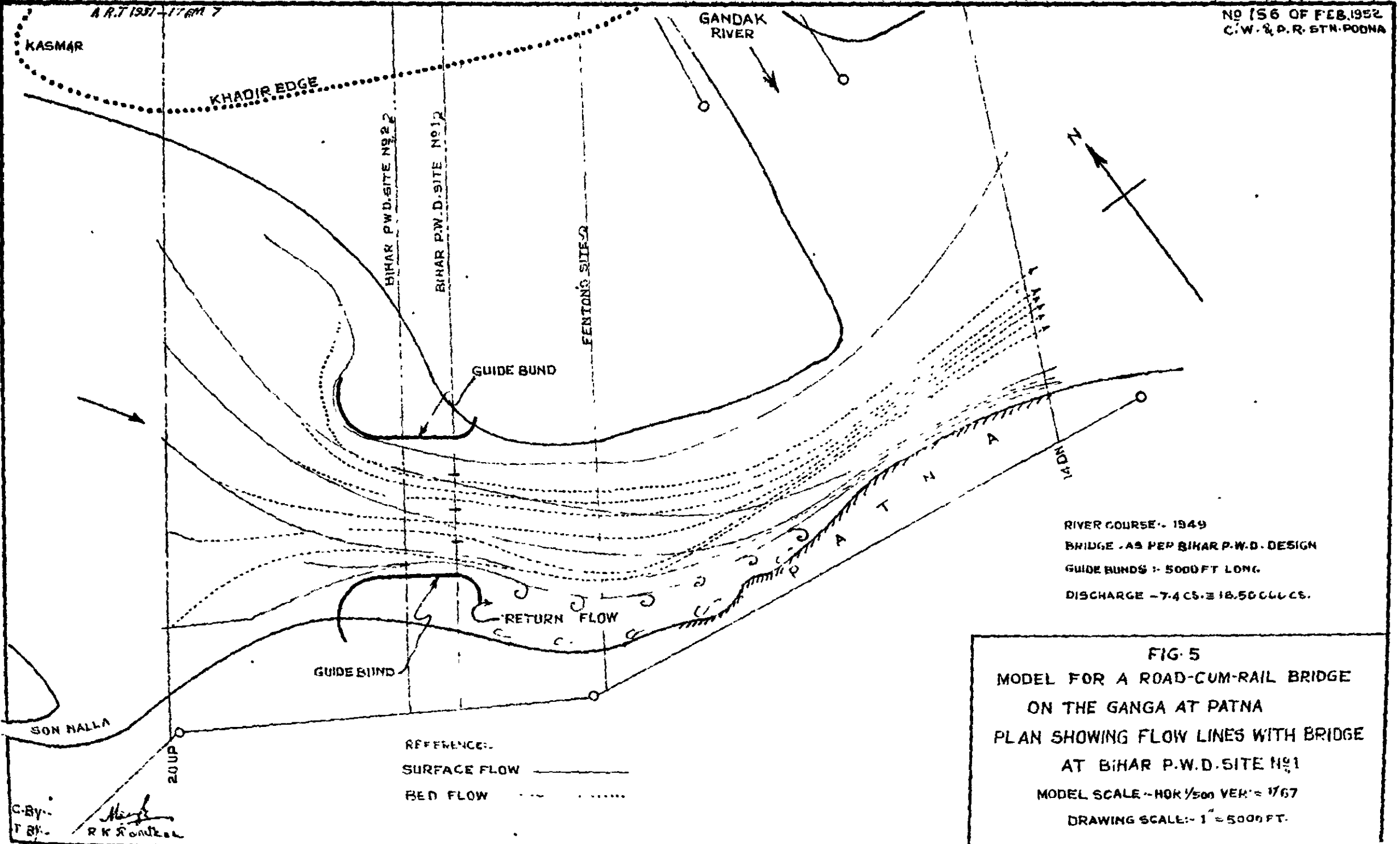
MODEL FOR A ROAD-CUM-RAIL BRIDGE
 ON THE GANGA AT PATNA.
 PLAN SHOWING FLOW LINES WITH BRIDGE
 AT BIHAR P.W.D. SITE NO 2
 MODEL SCALES: HOR. 1"/500, VER. 1"/67
 DRAWING SCALE: 1" = 500 FEET.

REFERENCES:-
 SURFACE FLOW -----
 BED FLOW

COMP. BY: - *M. S. Parkhi*
 TRACED BY: - PARKHI.
 17-2-1952

A.P.T 1957-17 AM 7

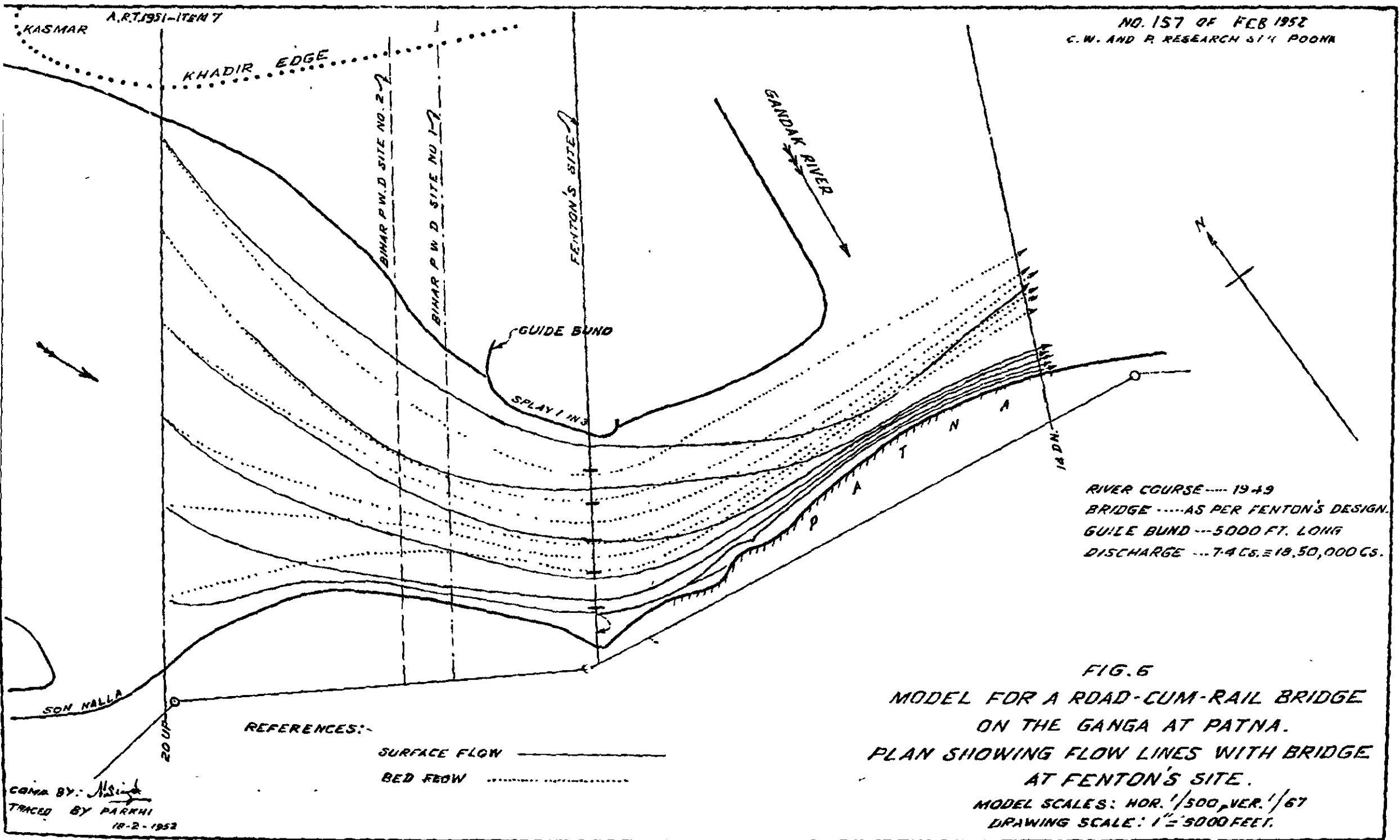
No 156 OF FEB. 1952
C.W. & P.R. STN. PONDIA



RIVER COURSE - 1949
 BRIDGE AS PER BIHAR P.W.D. DESIGN
 GUIDE BUNDS - 5000 FT LONG
 DISCHARGE - 7.4 CS. = 18.50 CUM CS.

FIG 5
 MODEL FOR A ROAD-CUM-RAIL BRIDGE
 ON THE GANGA AT PATNA
 PLAN SHOWING FLOW LINES WITH BRIDGE
 AT BIHAR P.W.D. SITE NO 1
 MODEL SCALE - HOR $\frac{1}{500}$ VER $\frac{1}{167}$
 DRAWING SCALE - 1" = 5000 FT.

C. By -
 T. By -
R K Roudra



A.R.T. 1951-ITEM 7

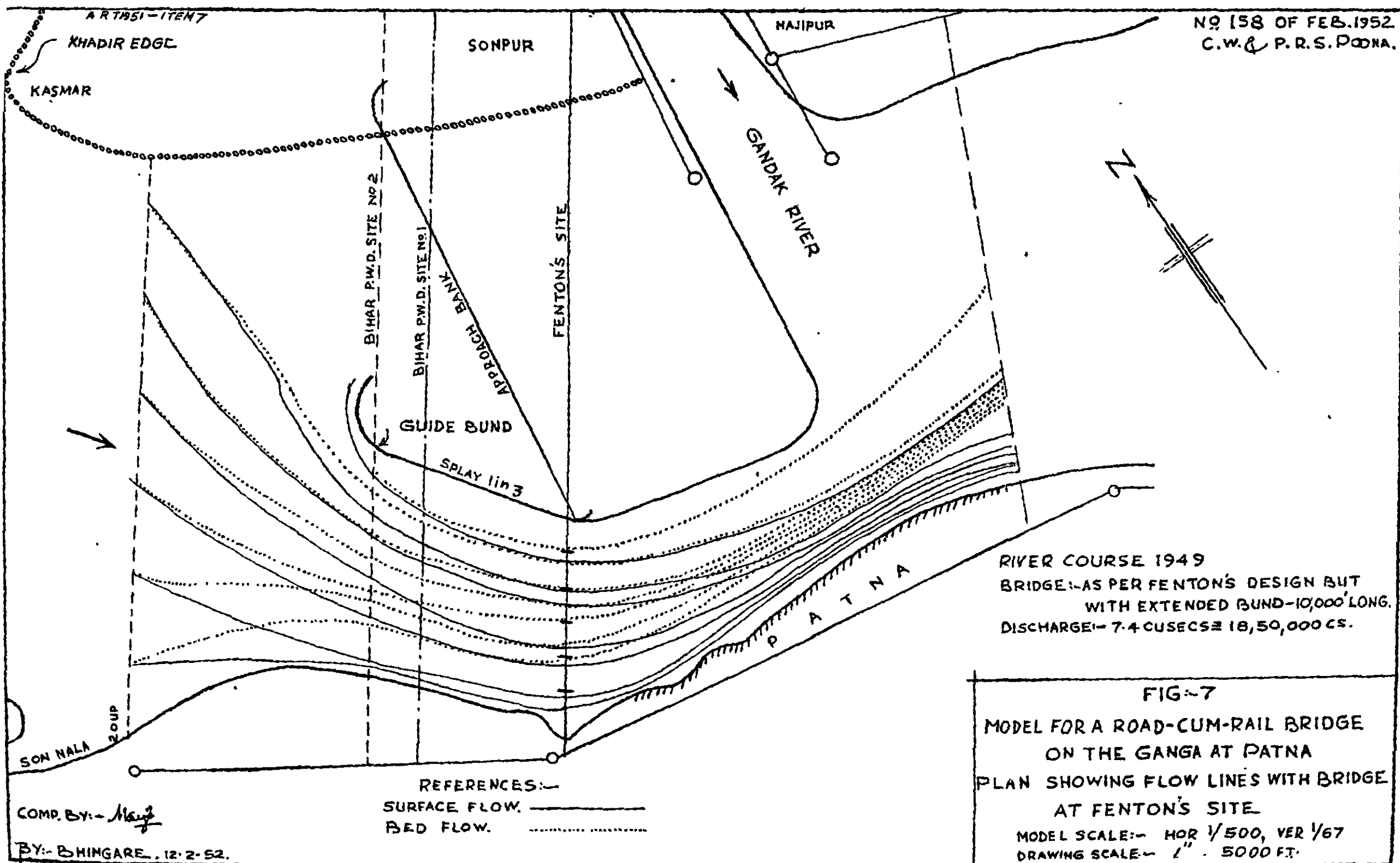
NO. 157 OF FEB 1952
C.W. AND R. RESEARCH STN POONA

RIVER COURSE ---- 1949
BRIDGE ---- AS PER FENTON'S DESIGN.
GUIDE BUND --- 5000 FT. LONG
DISCHARGE --- 7.4 C.S. = 18,50,000 C.S.

FIG. 6
MODEL FOR A ROAD-CUM-RAIL BRIDGE
ON THE GANGA AT PATNA.
PLAN SHOWING FLOW LINES WITH BRIDGE
AT FENTON'S SITE.
MODEL SCALES: HOR. 1/500, VER. 1/67
DRAWING SCALE: 1" = 500 FEET.

REFERENCES:-
SURFACE FLOW —————
BED FLOW
14 DN.

CONC BY: *M. Singh*
TRACED BY PARRHI
18-2-1952

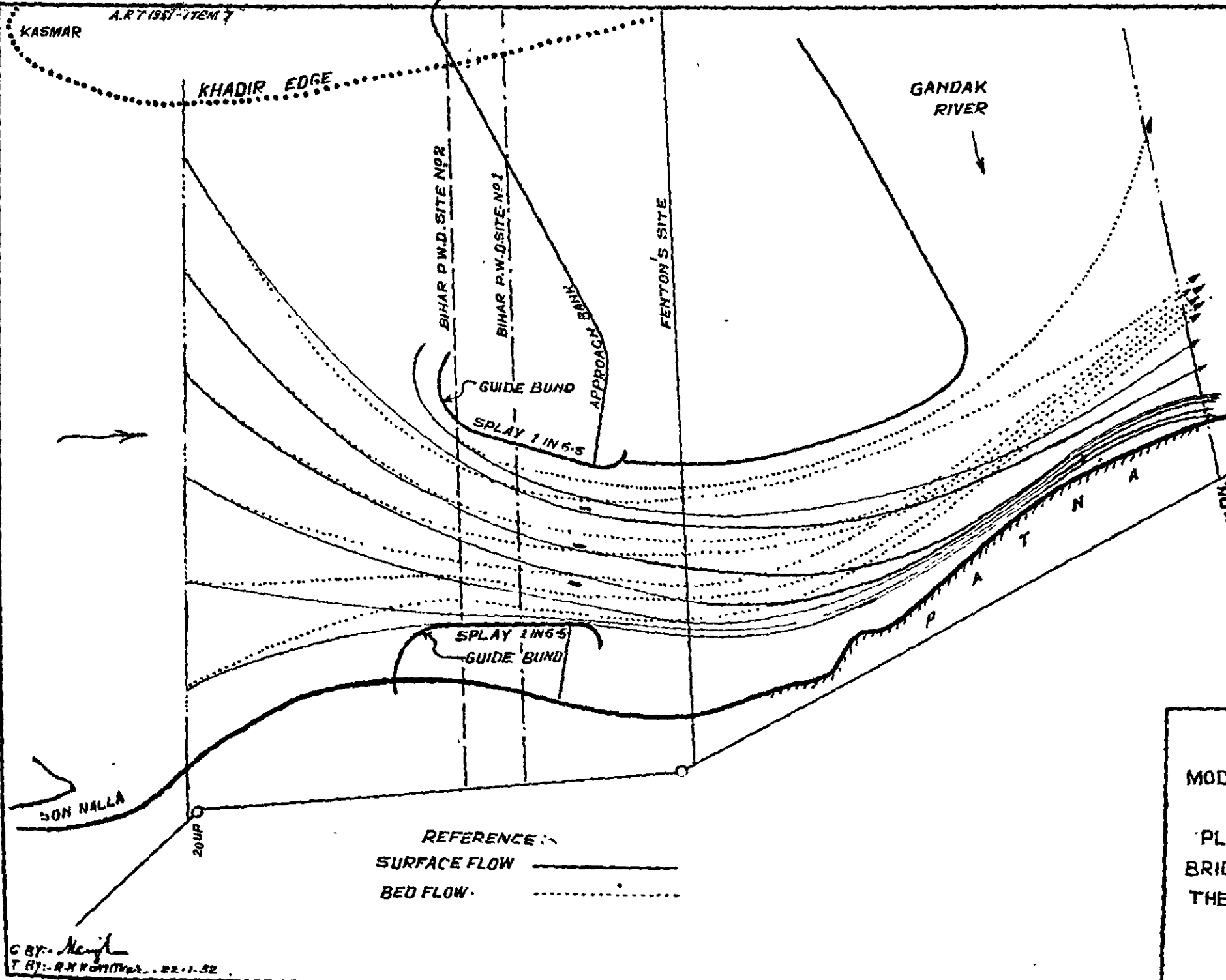


RIVER COURSE 1949
BRIDGE:- AS PER FENTON'S DESIGN BUT
WITH EXTENDED BUND-10,000' LONG.
DISCHARGE:- 7.4 CUSECS \approx 18,50,000 CS.

FIG:-7
MODEL FOR A ROAD-CUM-RAIL BRIDGE
ON THE GANGA AT PATNA
PLAN SHOWING FLOW LINES WITH BRIDGE
AT FENTON'S SITE
MODEL SCALE:- HOR 1/500, VER 1/67
DRAWING SCALE:- 1" = 5000 FT.

REFERENCES:-
SURFACE FLOW. —————
BED FLOW.

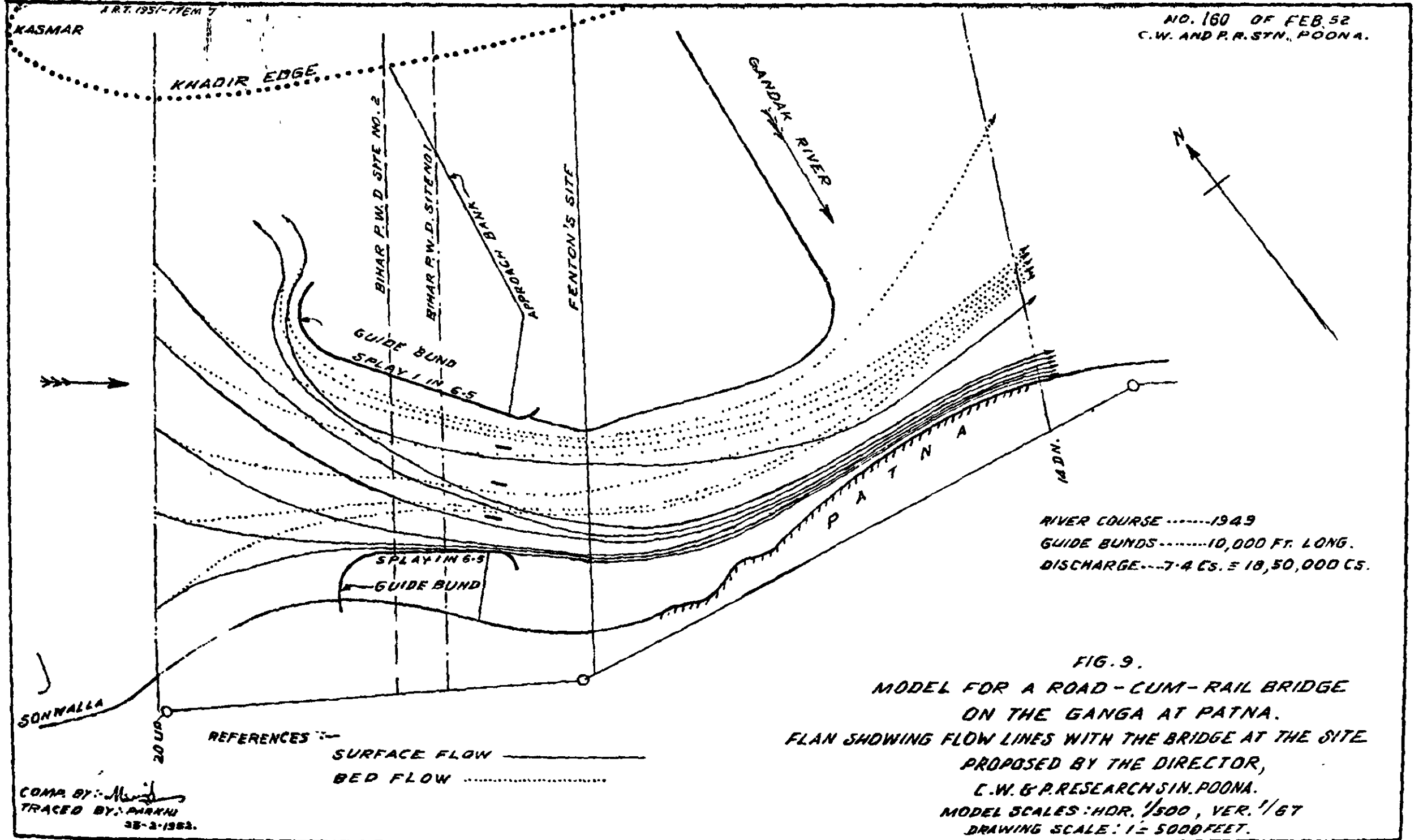
COMP. BY:- *M. J.*
BY:- BHINGARE. 12.2.52.



RIVER COURSE:- 1949
 GUIDE BUNDS:- 6500 FT. LONG.
 DISCHARGE:- 7.4 CS TO 18,50,000 CS.

FIG:- 8
 MODEL FOR A ROAD-CUM-RAIL BRIDGE
 ON GANGA AT PATNA
 PLAN SHOWING FLOW LINES WITH
 BRIDGE AT THE SITE PROPOSED BY
 THE DIRECTOR C.W.&P.R STN POONA
 MODEL SCALES: HOR: 1/500, VER: 1/67
 DRAWING SCALES:- 1 = 5000 FT

C BY: *Manjiv*
 T BY: R.N. KUMAR... 22-1-52



KASMAR

KHADIR EDGE

GANDAK RIVER

BIMAR P.W.D. SITE NO. 2

BIMAR P.W.D. SITE NO. 1

APPROACH BANK

FENTON'S SITE

GUIDE BUND
SPLAY IN 6.5

SPLAY IN 6.5
GUIDE BUND

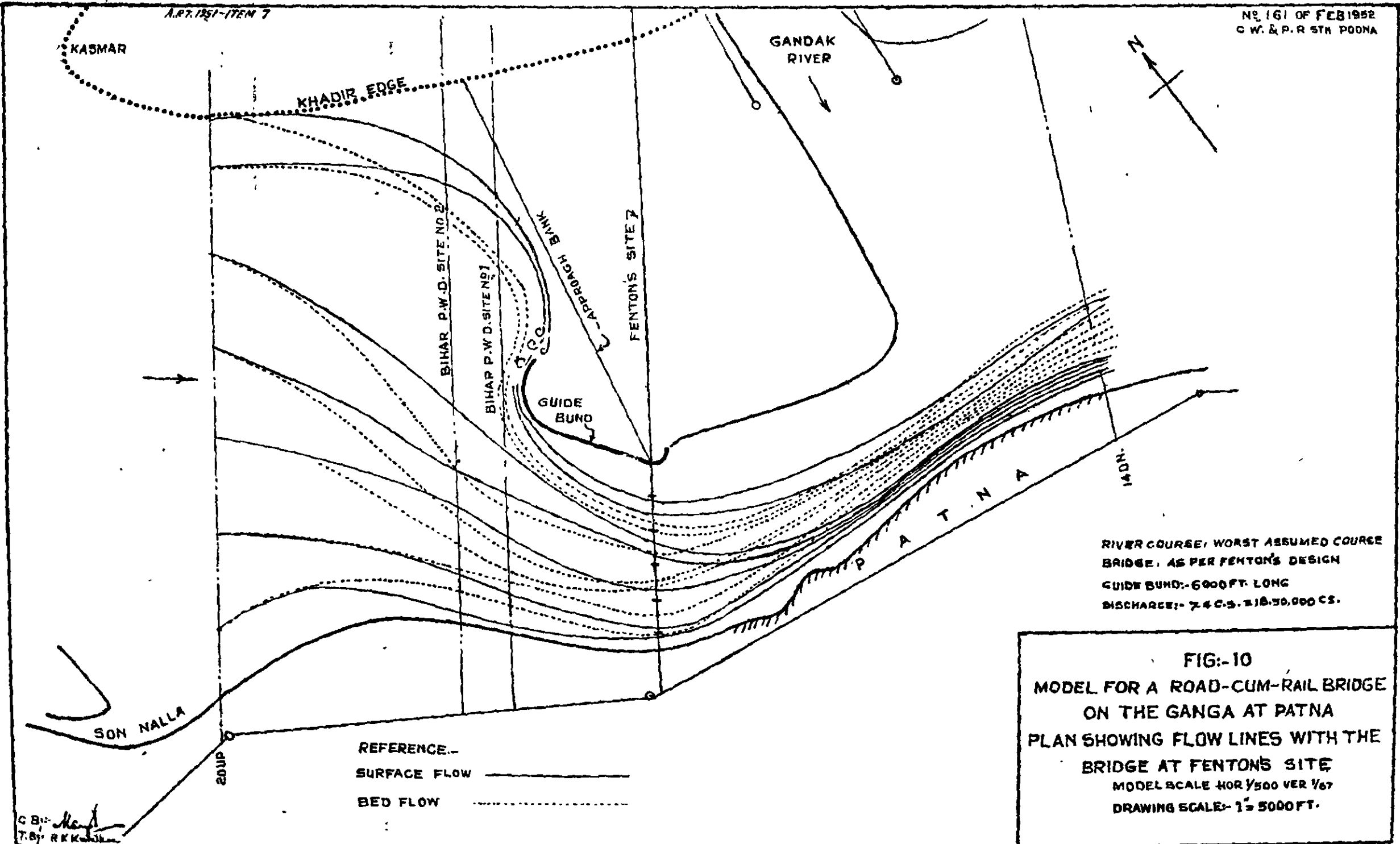
PATNA
M.D.N.

RIVER COURSE 1949
GUIDE BUNDS 10,000 FT. LONG.
DISCHARGE 7.4 C.S. ± 18,50,000 C.S.

REFERENCES :-
SURFACE FLOW —————
BED FLOW (dotted line)

FIG. 9.
MODEL FOR A ROAD-CUM-RAIL BRIDGE
ON THE GANGA AT PATNA.
PLAN SHOWING FLOW LINES WITH THE BRIDGE AT THE SITE
PROPOSED BY THE DIRECTOR,
C.W. & P. RESEARCH SIN. POONA.
MODEL SCALES: HDR. 1/500, VER. 1/67
DRAWING SCALE: 1" = 500 FEET.

COMP. BY: *[Signature]*
TRACED BY: *[Signature]*
38-2-1962.



RIVER COURSE: WORST ASSUMED COURSE
 BRIDGE: AS PER FENTON'S DESIGN
 GUIDE BUND: 6000 FT. LONG
 DISCHARGE: 7.4 C.S. 318.50,000 CS.

FIG:-10
MODEL FOR A ROAD-CUM-RAIL BRIDGE
ON THE GANGA AT PATNA
PLAN SHOWING FLOW LINES WITH THE
BRIDGE AT FENTON'S SITE
 MODEL SCALE HOR $\frac{1}{500}$ VER $\frac{1}{67}$
 DRAWING SCALE: $1" = 5000$ FT.

C.B.: *[Signature]*
 T.S.: R.K.K. *[Signature]*

FIG. 11
BRIDGE AT BIHAR P.W.D. SITE NO 2

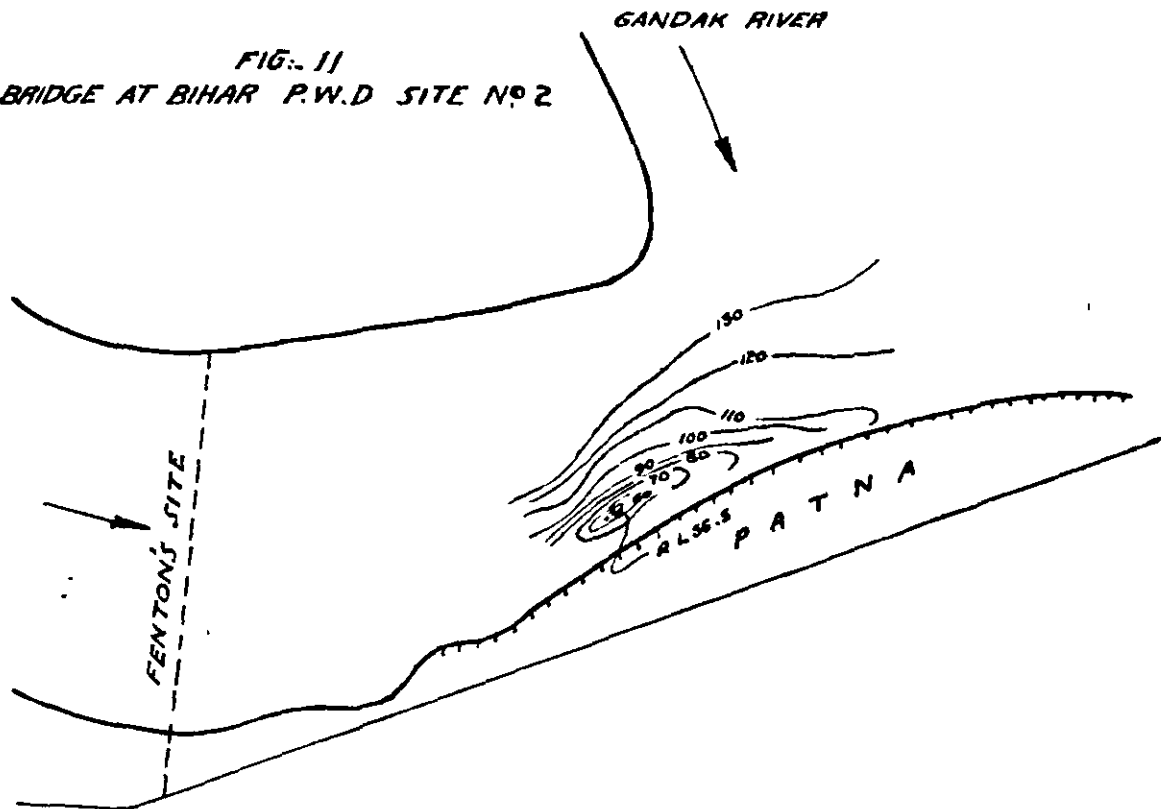
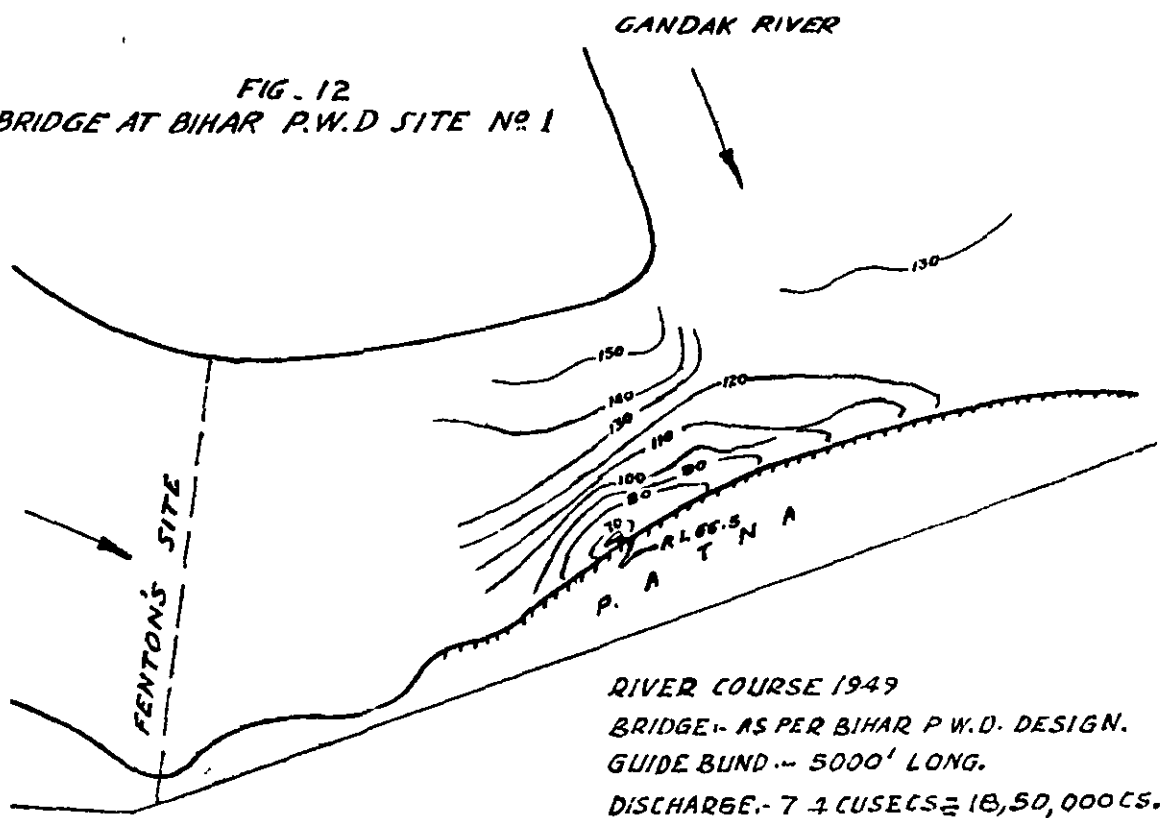


FIG. 12
BRIDGE AT BIHAR P.W.D. SITE NO 1



RIVER COURSE 1949
BRIDGE - AS PER BIHAR P.W.D. DESIGN.
GUIDE BUND - 5000' LONG.
DISCHARGE - 7.4 CUSECS = 16,50,000 CS.

FIG:-11 & 12
MODEL FOR A ROAD-CUM-RAIL BRIDGE
ON THE GANGA AT PATNA
PLAN SHOWING BED CONTOURS NEAR PATNA BANK
WITH BRIDGE AT BIHAR P.W.D. SITE 1 & 2
MODEL SCALE - HOR: 1/500, VER: 1/67
DRAWING SCALE - 1" = 5000 FT.

COMP. BY: - M.S.L.

T. BY: - MANCHUNDIA. 22-2-52

FIG. 13
GUIDE BUND 5000 FT LONG
(SPLAYED 1 IN 3)

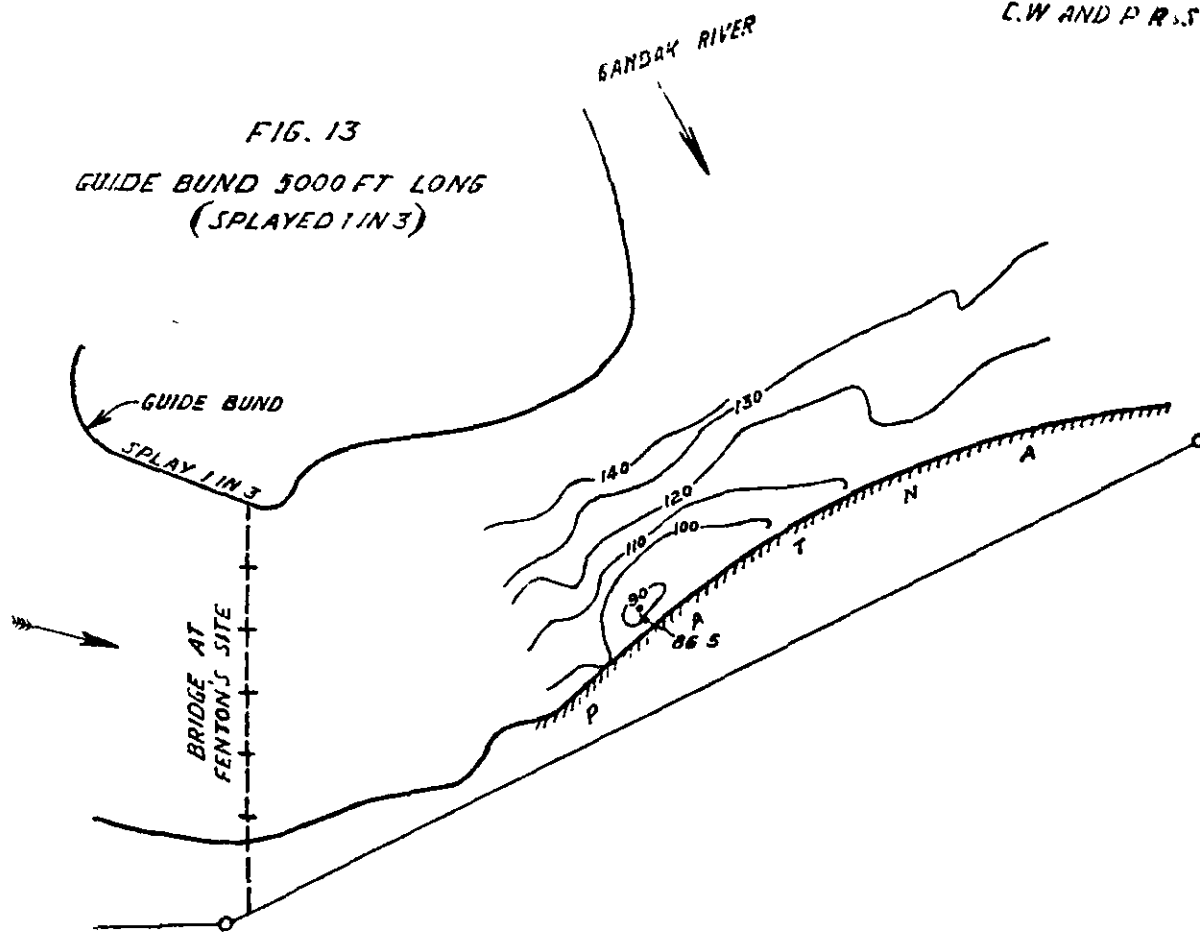
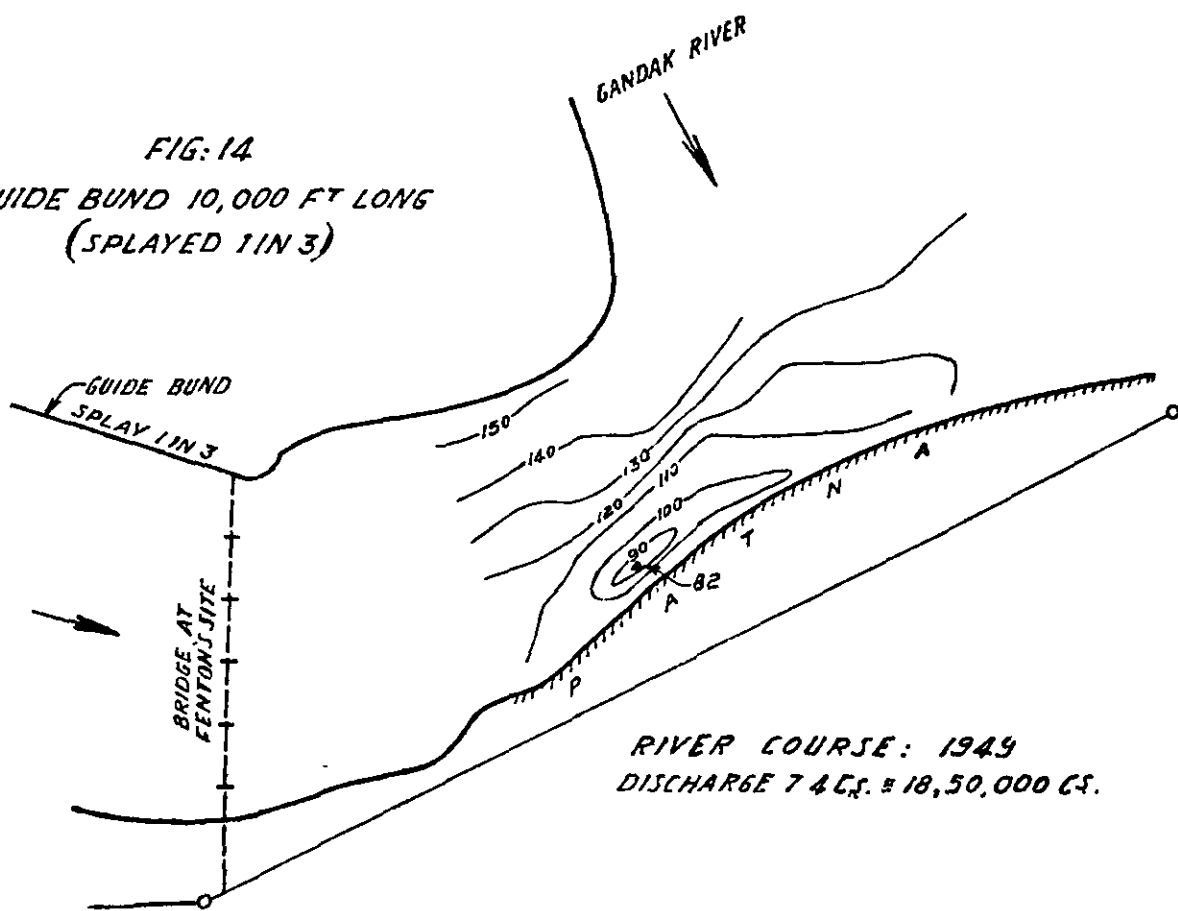


FIG: 14
GUIDE BUND 10,000 FT LONG
(SPLAYED 1 IN 3)



RIVER COURSE: 1949
DISCHARGE 74 C.S. = 18,50,000 C.S.

FIG: 13 & 14
MODEL FOR A ROAD-CUM-RAIL BRIDGE ON
THE GANGA AT PATNA
PLAN SHOWING BED CONTOURS NEAR PATNA BANK
WITH BRIDGE AT FENTON'S SITE
MODEL SCALES { HOR. 1/500
VER 1/67
DRAWING SCALE: 1" = 5000 FT.

COMPARED BY: *Maigh*
TRACED BY: M.K. SABNIS. 18-1-52

FIG. 15
GUIDE BUNDS - 6500 FT LONG
(SPLAYED 1 IN 6.5)

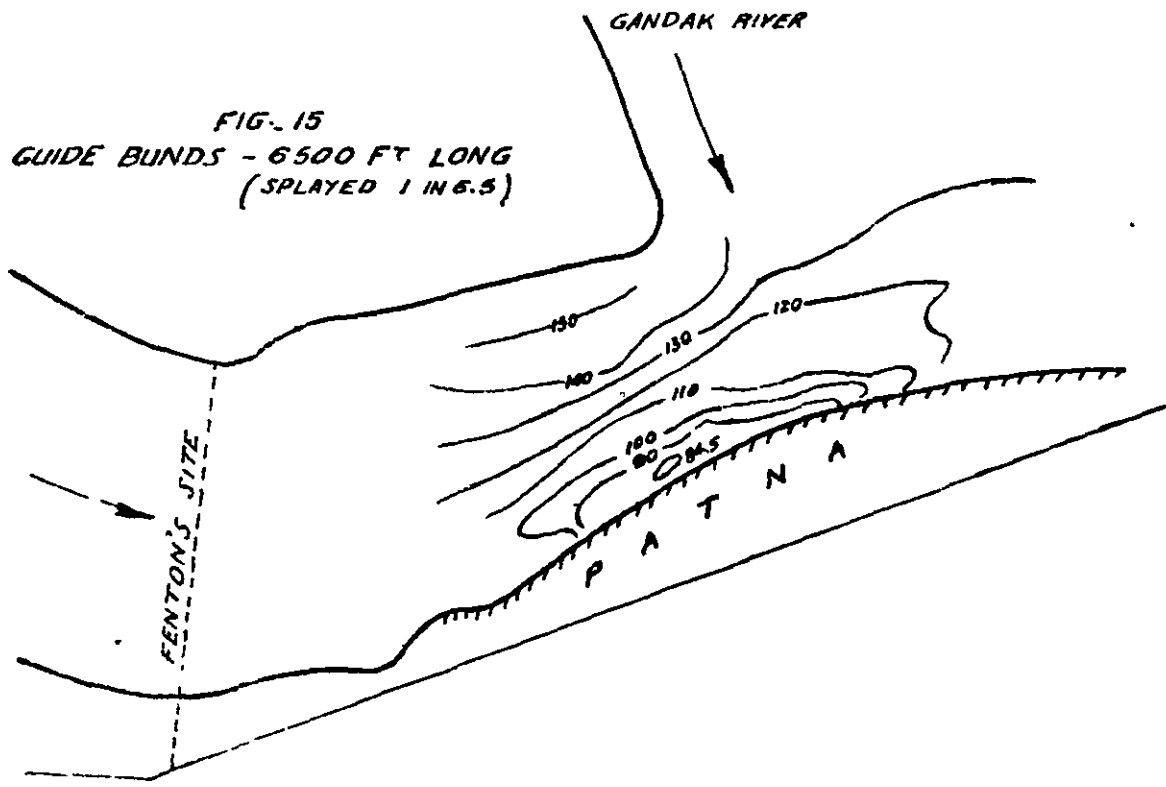
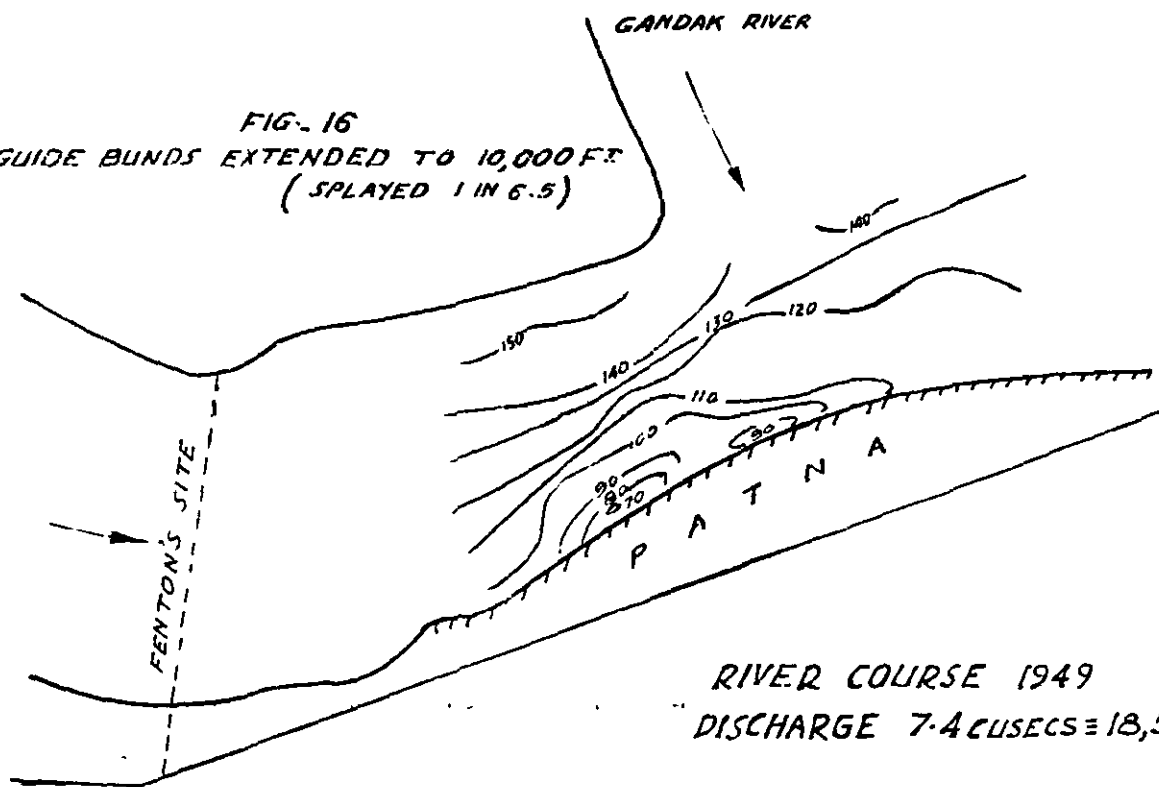


FIG. 16
GUIDE BUNDS EXTENDED TO 10,000 FT
(SPLAYED 1 IN 6.5)



RIVER COURSE 1949
DISCHARGE 7.4 CUSECS = 18,50,000 CS.

FIG:- 15 & 16
 MODEL FOR A ROAD-CUM-RAIL BRIDGE
 ON THE GANGA AT PATNA
 PLAN SHOWING BED CONTOURS NEAR PATNA BANK
 WITH BRIDGE AT THE SITE SUGGESTED BY THE
 DIRECTOR C.W. & P.R. STN. POONA
 MODEL SCALE:- HOR. 1/500 VER. 1/67
 DRAWING SCALE:- 1" = 500'

COMP. BY:- *Ms. J*

BY:- MANCHUNDIA. 20-2-51

RIVER COURSE..... WORST ASSUMED COURSE.
BRIDGE..... AS PER FENTON'S DESIGN.
GUIDE BUND... 6000 FT LONG.
DISCHARGE 7.4 CS = 18,50,000 CS.

GANDAK RIVER.

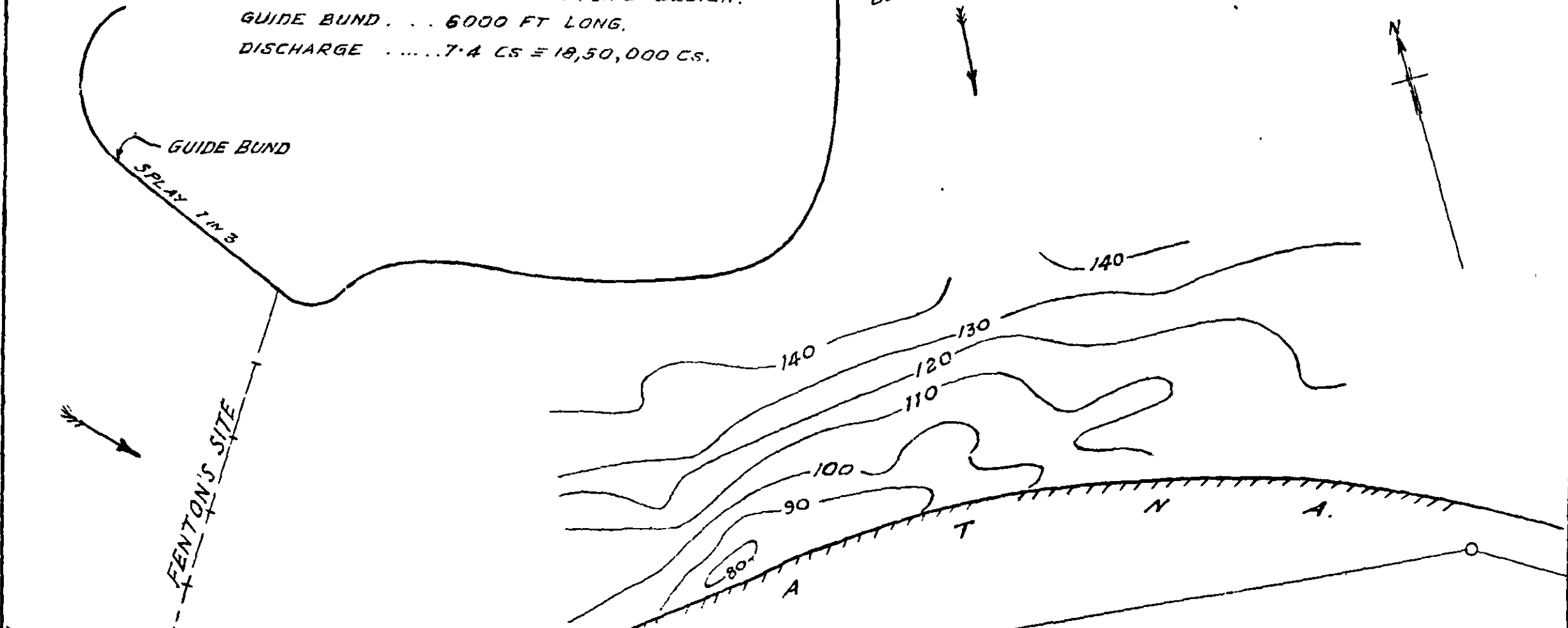


FIG.17
MODEL FOR A ROAD-CUM-RAIL BRIDGE
ON THE GANGA AT PATNA
PLAN SHOWING
BED CONTOURS NEAR PATNA BANK WITH
THE BRIDGE AT FENTON'S SITE.

MODEL SCALES: HOR. 1/500, VER. 1/67
DRAWING SCALE: 1" = 2500 FT.

Both these sites have some inherent defects of their own. They lie in the immediate vicinity of an upstream unstable confluence of the Ganga and the Gandak. A bridge across the Mehnaar channel, would not be much favoured by the river, as it would, to some extent perpetuate the awkward backward swing of the Gandak. On the other hand a bridge across the Futwah channel is unsuitable at the present time, as this is no longer the main river channel.

9.1. Taking into account the present river configuration, the following experiments were carried out—

- (7) Existing conditions without a bridge.
- (8) A bridge across the Mehnaar channel—Bihar P. W. D., site No. 3—under the present river course (1949).
- (9) A bridge across the Mehnaar channel—Bihar P. W. D., site No. 3—with Futwah channel as the main river course.
- (10) Two bridges—one across the Mehnaar channel and one across the Futwah channel—with Futwah channel as the main river course.

The Futwah channel site in this case is about 1/2 mile downstream of the Fenton's alternative site. This shifting was necessary in order to design the approach bank between the two bridges, with only one smooth curve.

9.1.1. These bridge sites not being situated in a nodal region, the bridge will not be having a full control over the river course upstream and hence the bridge had to be tested under an alternate river course as in experiment No. 9.

9.1.2. When the river splits into two channels—the Mehnaar and Futwah—the maximum discharge carried by the main channel is about 80 per cent of the total—as per verbal information received at site. Hence with two bridges, the waterway provided for each was 5,000 ft. as against a waterway of 6,000 ft. for a single bridge.

9.2. Figure 18 shows the flow lines under the existing conditions without a bridge. The model showed that about 70 per cent discharge was taken by the Mehnaar channel. The maximum surface velocity in the North channel was equivalent to 98 ft./sec. and 101 ft./sec. and covered a region between 1,000 ft. to 3,500 ft. from the North bank. The maximum surface velocity in the Futwah channel also was of the same order, even though it carried only 30 per cent discharge.

Fig. 18.

9.3. Then a bridge at Bihar P. W. D. Site No 3 was tested. It had a waterway of 6,000 ft. and the Right Guide bund was 5,000 ft. long. The flow in the Futwah channel was now closed with a bund and hence it started flowing parallel to the approach bank (Figure 19) and was deflected by the mole head to the North bank, thus increasing the intensity of flow there. The highest surface velocity was between 135 ft./sec. to 140 ft./sec. and covered a reach between 400 ft. to 3,400 ft. from the North bank. The design appears to work satisfactorily as long as only a low discharge runs in Futwah channel, except that the long approach bank will require additional protection in the form of pitching and constant maintenance against erosion by the parallel flow. But if this parallel flow develops into a well-defined channel then it will not be possible to protect the approach bank against an embayment.

Fig. 19.

9.4. Then the same bridge site was further tested with Futwah channel as the main river course. In this case a very strong flow parallel to the approach bank was developed (Figure 20) and the water was finally deflected by the mole head towards the North bank. The highest surface velocity was as much as 15.25 ft./sec. and covered a region between 1,100 ft. and 1,900 ft. from the North bank. The approach embankment at its Southern end was subjected to a differential head of about 7 ft. It will be very difficult to hold the approach bank against this differential head combined with embayment by the high velocity current flowing parallel to it.

Fig. 20.

9.5. Finally two bridges across the two channels were tested. The Guide bund length as provided was 5,000 ft. This design seems to work quite satisfactorily as seen in Figure 21. Photo No 747 shows the two bridges in position. In the Futwah channel the maximum surface velocity was about 13 ft./sec. and covered a very

Fig. 21
Photo 747.

wide region between 1,900 ft. to 5,000 ft. from the South bank. In the Mehnaar channel—which now carried less discharge—the highest surface velocity was only 7.73 ft/sec. and covered a reach between 600 ft and 1,900 ft. from the North bank. In this case the flow past the approach bank was very little—only the spill from one channel to other—and hence it will be easy to protect it with some additional light protective works.

10. Conclusion

From the above experiments it is quite clear that the only feasible site in the vicinity of Patna is Bihar P. W. D., Site No. 3 with an additional bridge across the Futwah channel. Two bridges across the two channels, each with a waterway of 5,000 ft. will provide a sufficiently safe and dependable means of communication across the Ganga. Each of these bridges can be sufficiently protected with only one Guide bund 5,000 ft. long as both the natural banks consist of good clay. This proposal compares very favourably in cost with a bridge at Fenton's site. These costs are compared below—

- (1) Bridge at Fenton's site with 9,240 ft. waterway and 10,000 ft. long left Guide bund with additional protection for the Kasmar bluff will cost—Rs. 16.38 crores.
- (2) Two bridges at Bihar P. W. D., site No. 3, waterway of each 5,000 ft. with one Guide bund 5,000 ft. long for each will cost—Rs. 16.57 crores.

Table 2. These costs have been arrived at as shown in Table 2 along with the costs of the Rail-cum-Road bridge at other sites. These have been based on rates in the Fenton's report.

The above costs are based on the rates in the Fenton's estimate, by adding 40 per cent thereto to bring the same to the present price level.

Table 3. Table 3 shows the costs of the Road bridge alone at the various Patna sites. These have been based on the rates given in Mr. Khan's report and 40 per cent added to allow for increased prices.

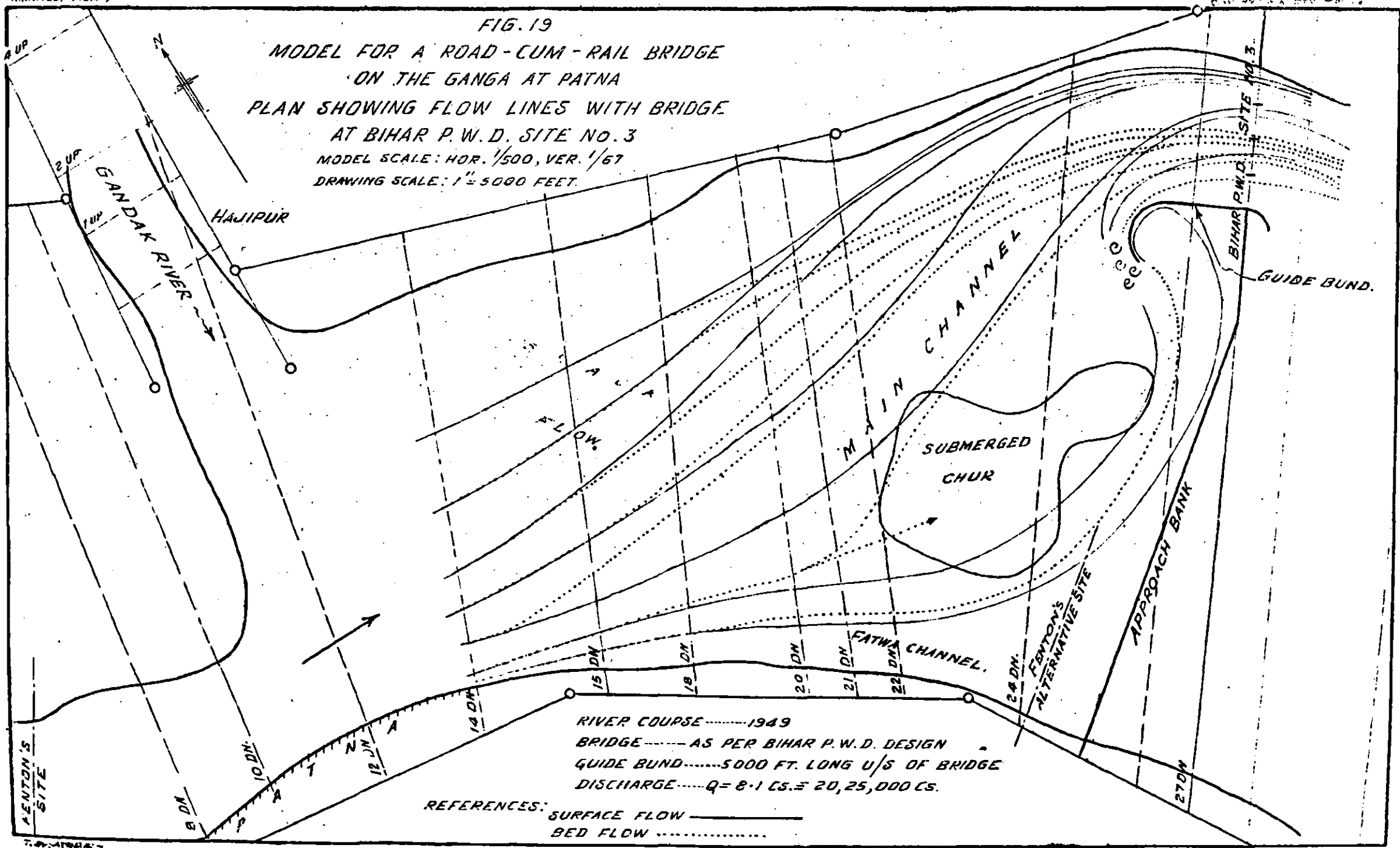
Fig. 1.
Fig. 22.

10.1. While making the above recommendations, the possibility of stabilizing the Mehnaar channel by means of either ordinary repelling spurs or T spurs was considered. But considering the 1949 river course (Figure 1) along with the rest of the river courses shown in Figure 22, it will be clear that the lengths of such spurs will be excessive as they will have to be connected to a high bank, i.e., the South bank or the bridge approach bank. As the top levels of the Raghapur Diara upstream of the bridge site are between 10 to 12 ft. below the high flood level, short spurs will be of no avail as water can very well flow behind them. Because of this excessive length of these spurs, the cost—both initial and maintenance—will be prohibitive. Hence the suggestion of constructing only one bridge across the Mehnaar channel by stabilizing that channel had to be dropped in favour of the final recommendation of constructing two bridges

Fig. 22.

10.2 The next question to be considered was, whether the bridge across the Futwah channel could be put off till a future date—till that channel shows signs of developing. Unfortunately the answer has to be in the negative. In view of the paucity of the data, it is very difficult and dangerous too, to predict the future river courses. In 1944, all of a sudden the Mehnaar channel developed at the cost of the Futwah channel. Data for a complete regime cycle is not available, but the part data about the river courses available (Figure 22) unmistakably shows that the Futwah channel is the main river course for a period, considerably more than half the regime cycle. Hence it is considered likely that the river will swing back to the Futwah channel, within a comparatively shorter period and the Gandak will also then resort to its more natural South-Easterly course. If such a catastrophe is to take place after bridging only the Mehnaar channel, it will mean break in communications for a period of at least 5 years. Hence, to avoid such an occurrence, the only time-lag which may be allowed between the construction of the two bridges should be limited to the absolute minimum required to facilitate the construction, as the same equipment and personnel can be used at both the places. This period may safely be between 2 and 3 years. It need not be emphasised that in allowing this time-lag, the year of the highest high flood, which is expected to occur every 12 to 14 years, should be avoided.

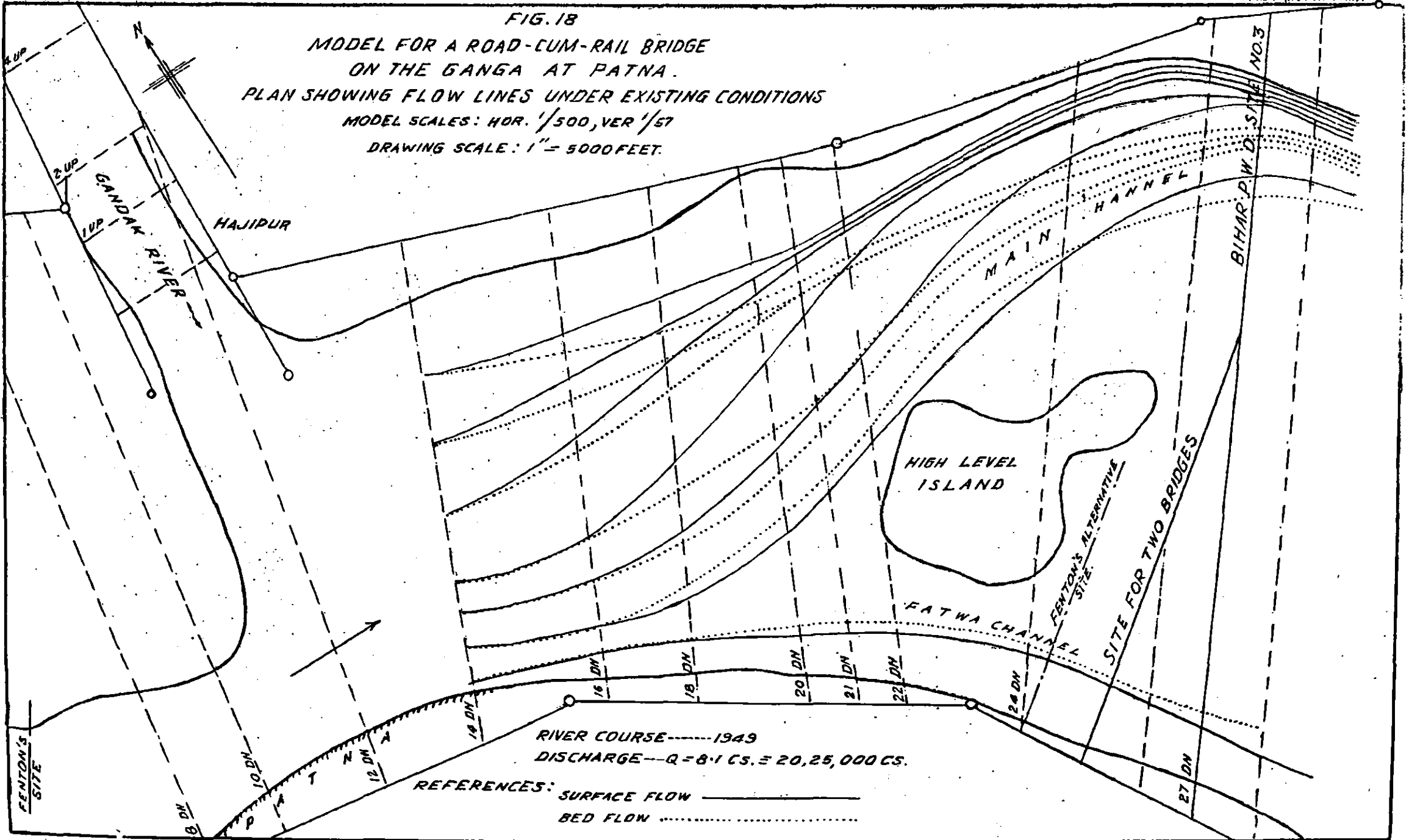
FIG. 19
 MODEL FOR A ROAD-CUM-RAIL BRIDGE
 ON THE GANGA AT PATNA
 PLAN SHOWING FLOW LINES WITH BRIDGE
 AT BIHAR P.W.D. SITE NO. 3
 MODEL SCALE: HOR. 1/500, VER. 1/67
 DRAWING SCALE: 1" = 5000 FEET.



RIVER COURSE.....1949
 BRIDGE.....AS PER BIHAR P.W.D. DESIGN
 GUIDE BUND.....5000 FT. LONG U/S OF BRIDGE
 DISCHARGE..... $Q = 8.1 \text{ Cs.} \approx 20, 25, 000 \text{ Cs.}$

REFERENCES: SURFACE FLOW —————
 BED FLOW (dotted line)

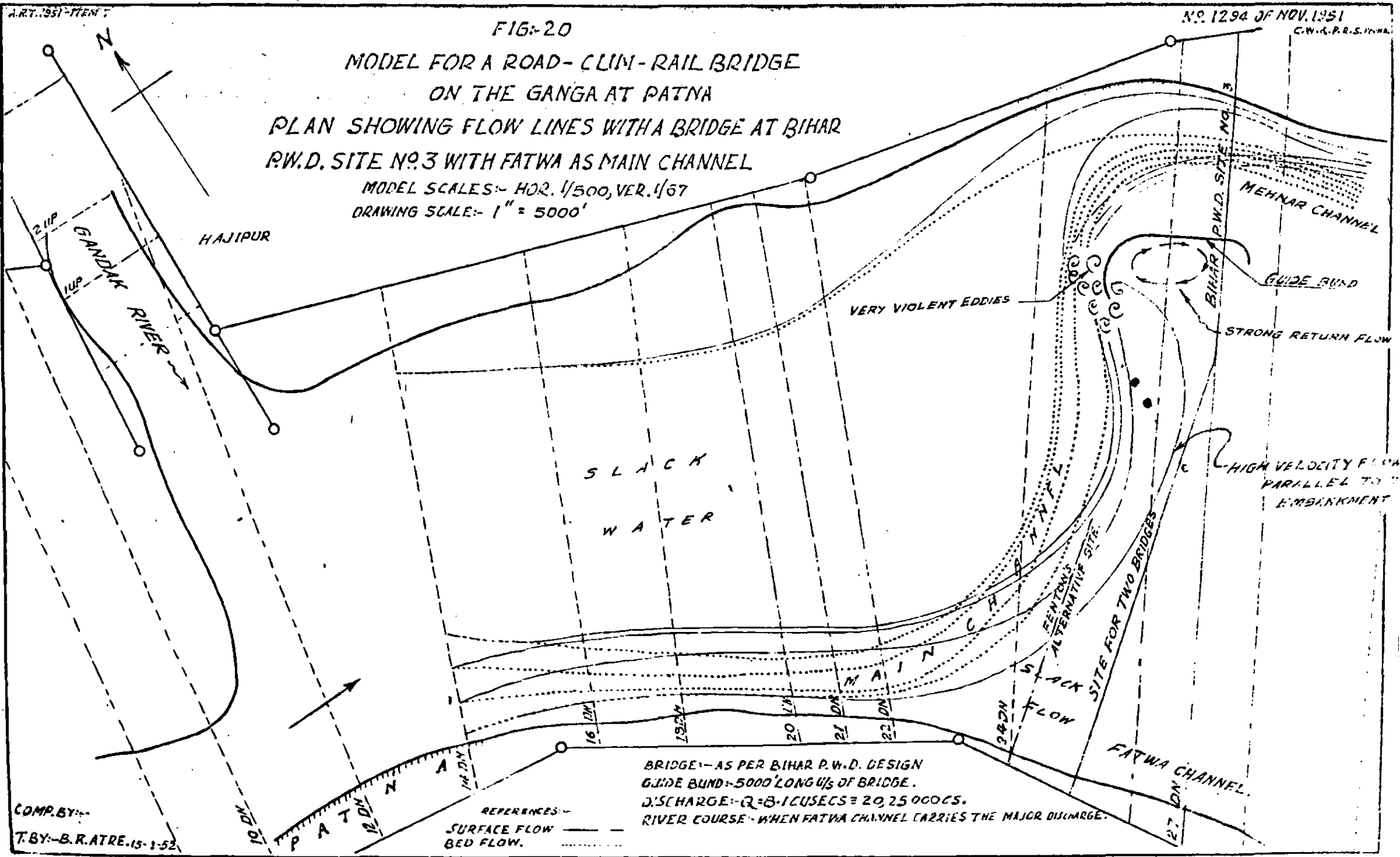
FIG. 18
 MODEL FOR A ROAD-CUM-RAIL BRIDGE
 ON THE GANGA AT PATNA.
 PLAN SHOWING FLOW LINES UNDER EXISTING CONDITIONS
 MODEL SCALES: HOR. 1/500, VER 1/57
 DRAWING SCALE: 1" = 5000 FEET.



Misup 4

FIG-20
 MODEL FOR A ROAD-CLIM-RAIL BRIDGE
 ON THE GANGA AT PATNA
 PLAN SHOWING FLOW LINES WITH A BRIDGE AT BIHAR
 P.W.D. SITE NO.3 WITH FATWA AS MAIN CHANNEL

MODEL SCALES:- HOR. 1/500, VER. 1/67
 DRAWING SCALE:- 1" = 5000'



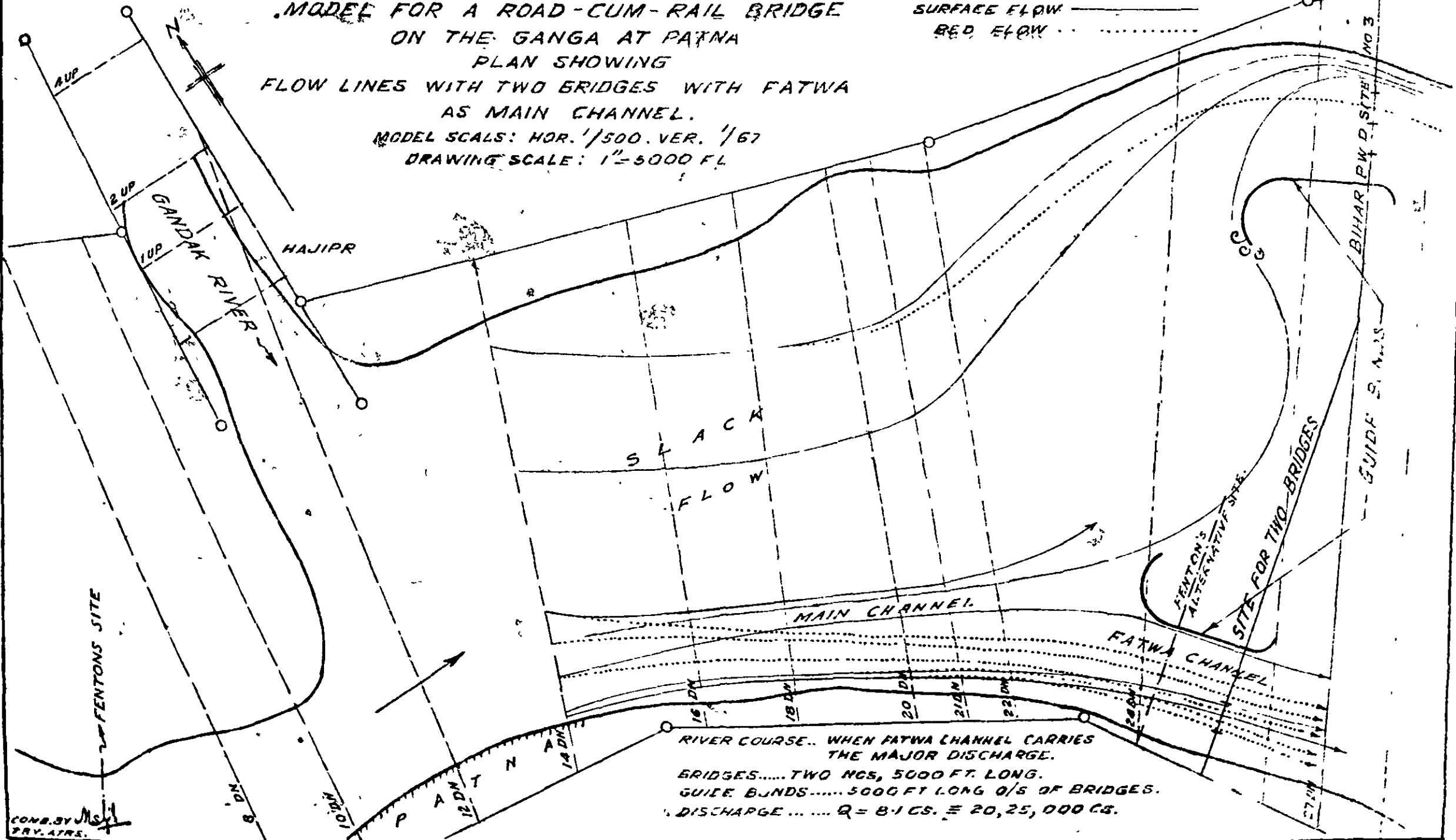
BRIDGE:- AS PER BIHAR P.W.D. DESIGN
 GUIDE BUND:- 5000' LONG U/S OF BRIDGE.
 DISCHARGE:- $Q=8.1 \text{ CUSECS} = 20,250 \text{ CFS.}$
 RIVER COURSE:- WHEN FATWA CHANNEL CARRIES THE MAJOR DISCHARGE.

REFERENCES:-
 SURFACE FLOW ————
 BED FLOW - - - - -

COMPR. BY:-
 T.BY:- B.R.ATRE.15-1-52

FIG. 21
 MODEL FOR A ROAD-CUM-RAIL BRIDGE
 ON THE GANGA AT FATWA
 PLAN SHOWING
 FLOW LINES WITH TWO BRIDGES WITH FATWA
 AS MAIN CHANNEL.
 MODEL SCALES: HOR. 1/500, VER. 1/67
 DRAWING SCALE: 1"=5000 FT.

REF. SURFACE FLOW
 BED FLOW



RIVER COURSE.. WHEN FATWA CHANNEL CARRIES
 THE MAJOR DISCHARGE.
 BRIDGES..... TWO MCS, 5000 FT. LONG.
 GUIDE BUNDS..... 5000 FT LONG O/S OF BRIDGES.
 DISCHARGE Q = 81 CS. = 20,25,000 CS.

TABLE No. 2—Statement showing cost of bridges (Rail-cum-Road) at various sites across the Ganga river based on rates given in Fenton's Estimate

Serial No	Bridge site	Waterway	Guide Bunds.		Cost of Guide Bund in Lacs.	Cost of Bridge proper in Lacs	Cost, General Plant, Office Establish-ment, quarter, etc in Lacs	Cost apprao-ches, Land acquisition etc. in Lacs	Cost of other extra works in Lacs.	Total cost in Lacs	Present day cost=cost in column No.10 +40 per cent in Lacs
			No	Length							
1	2	3	4	5	6	7	8	9	10	11	
1.	Mokameh Ghat (South channel)	6,000 ft (14 spans)	1	5,000 ft	123 5	291	130 08	220 05	—	764 63	1,070 482
2.	Mokameh Ghat (North channel)	Do	2	5,000 ft.	247	291	130 08	220 05	—	888 13	1,243 382
3.	Fenton's site at Patna with modifications	9,240 ft (22 spans)	1	10,000 ft	229 77	452 42	177 66	259 9	Kasmar protection 50	1,169 75	1,637 65
4.	Bihar P. W. D, site No 3 across Mehnar channel and another bridge across the Futwah channel	5,000 ft & 5,000 ft.	1 1	5,000 ft & 5,000 ft	247	498 96	177 66	259 9	—	1,183 52	1,656 928

TABLE No 3—Statement of cost of Road Bridges at Patna at various sites

Serial No	Position of Bridge	Nos of Span	Length of Waterway in ft.	Cost of Spans & Approach Road in Lacs. (a)	Nos of Guide Bund	Guide Bund			Total cost in Lacs. Rs. (a+b)=c	Cost of Kasmar Protection in Lacs. (d)	Grand Total in Lacs (c+d)	Total after adding 40 per cent in Lacs	Remarks
						Length of each Guide Bund. in ft.	Total length of Guide Bund in ft.	Cost of Guide Bund in Lacs (b)					
1.	P. W. D, site No. 2	14	6,000	532	2	7,200	14,400	275	807	Not required.	807	1038	See Khan's report pp 92-93
2.	P W. D, site No. 1	14	6,000	532	2	7,200	14,400	275	807	Not required.	807	1038	Do.
3.	Fenton's site	22	9 240	836	1	5,000	5,000	96	932	50	982	1 375	—
4.	Fenton's site (modified)	22	9,240	836	1	10,000	10,000	192	1,028	50	1,070	1500	Cost with Kasmar Aprn.
5.	P. W. D, site No. 3 with 2 Bridges one on Futwah channel and the other on Mehnar channel each of 12 spans of 420 ft each.	2×12	2×5,000 =10,000	910	2	5,000	10,000	192	1,102	Not required.	1,102	1540	—

(The above costs have been calculated according to the Schedule given in Mr. Khan's Report.)

Prepared by Shri Sudheswar Nath,

Assistant Engineer, Bihar, P. W. D.

(iii) SPILL

8. PREVENTING FLOODING OF PADDY FIELDS IN VILLAGE KONHALLI (DISTRICT NORTH KANARA)

1. The River Chandavar is a hilly torrent carrying about 6,000 cusecs maximum flood discharge. The history of this river records that during the unprecedented floods in the year 1924 the right bank of the river gave way in 2 places—one along ABC and the other along B₁—B₅ as shown in the plan. These breaches were small in the beginning and have gradually become wide enough to allow the flood water to flow over the whole low-lying area of about 300 acres of rice fields rendering most of the area unfit for cultivation. The major problem is, therefore, of arresting bank erosion and closing of spills at ABC and B₁—B₅ on the right bank. A subsidiary problem is to provide arrangements such as small bandharas for feeding the canals at points M and N and thus utilize the cold weather flow. At point M near the spill B₁—B₅ the cold weather channel is blocked by an earthen bund and water is diverted to a small canal taking off from the right bank. This bund before being washed away in the next monsoon freshets, creates considerable afflux which helps this overflowing. There is also an additional afflux due to the river negotiating an unnaturally sharp bend along ABC. The local cultivators informed that this bend was not severe as it exists today; during the last 15 years or so the river has eroded about 100 ft. of the right bank. With increase of curvature in this bend a huge sand shoal has formed along the inside of the bend on the left bank and a new water channel N has formed on the right bank which flows through the low-lying fields causing submergence. The cultivators take advantage of this new channel by damming up the cold weather channel and diverting water to this channel. The conditions leading to the sharpening of this river bend and the consequent formation of the sand bar could not be ascertained but it is considered that a stone wall along ABC (Figure 1) guarding the property along the left bank had some influence in shifting the river course more towards the right. The river banks consist of soft red soil mixed with fine sand and are easily erodible. The bed material ranges from 1.0 mm. to 7.0 mm. diameter.

Fig. 1.

Photo 761.

Photo 758.

Photo 756.

The highest flood level of 1924 has been given as R. L. 509.52 while the normal floods rise up to R. L. 507.0. The frequency of submergence over the breached portion at N and M is about 5 in a normal year. Seedling operations even though repeated several times a year yield no crops as they are swept away whenever the floods inundate these lands. Only the right bank from M to N is high and does not allow water to spill in a normal flood year.

2. Recommendations

The following measures are suggested to bring under cultivation the lands damaged by flooding.

Figs. 1 & 2.

- (1) An earthen embankment retired 10 feet behind the damaged right bank marked as B₁, B₂, B₃—B₅ with stone pitching on the river side and a stone apron up to the present river edge. A rough sketch showing the proposed construction is enclosed. The top level of this embankment may be fixed at R. L. 511.0, i.e., a foot and a half higher than the H. F. L. of 1924, viz., 509.52. The extremities of the embankment should be well abutting the high bank on the sides so as to leave no weak joints likely to breach. The exact length of the embankment may be determined on the field.

It is observed that the right bank at places between the points N and M is lower than the H. F. L. of 1924. It is, therefore, very essential to raise the entire right bank, wherever low, in a similar manner.

- (ii) Along the bend ABC it is proposed to close the gap from where the small canal takes off by a stone wall as an earthen bund may not stand the river action in floods. It may be provided with regulated openings to allow water in the canal if desired. The stone wall should be of the retaining type.
- (iii) For stopping further erosion of the right bank along this bend, the river course has to be straightened by bypassing this bend altogether. It is, therefore, proposed to have a leading cut, 50 feet wide and 10 feet deep with sides at $\frac{1}{2}$: 1 slope, joining the points X and Y. The nature of the river banks and the high velocity during floods indicate that this cut

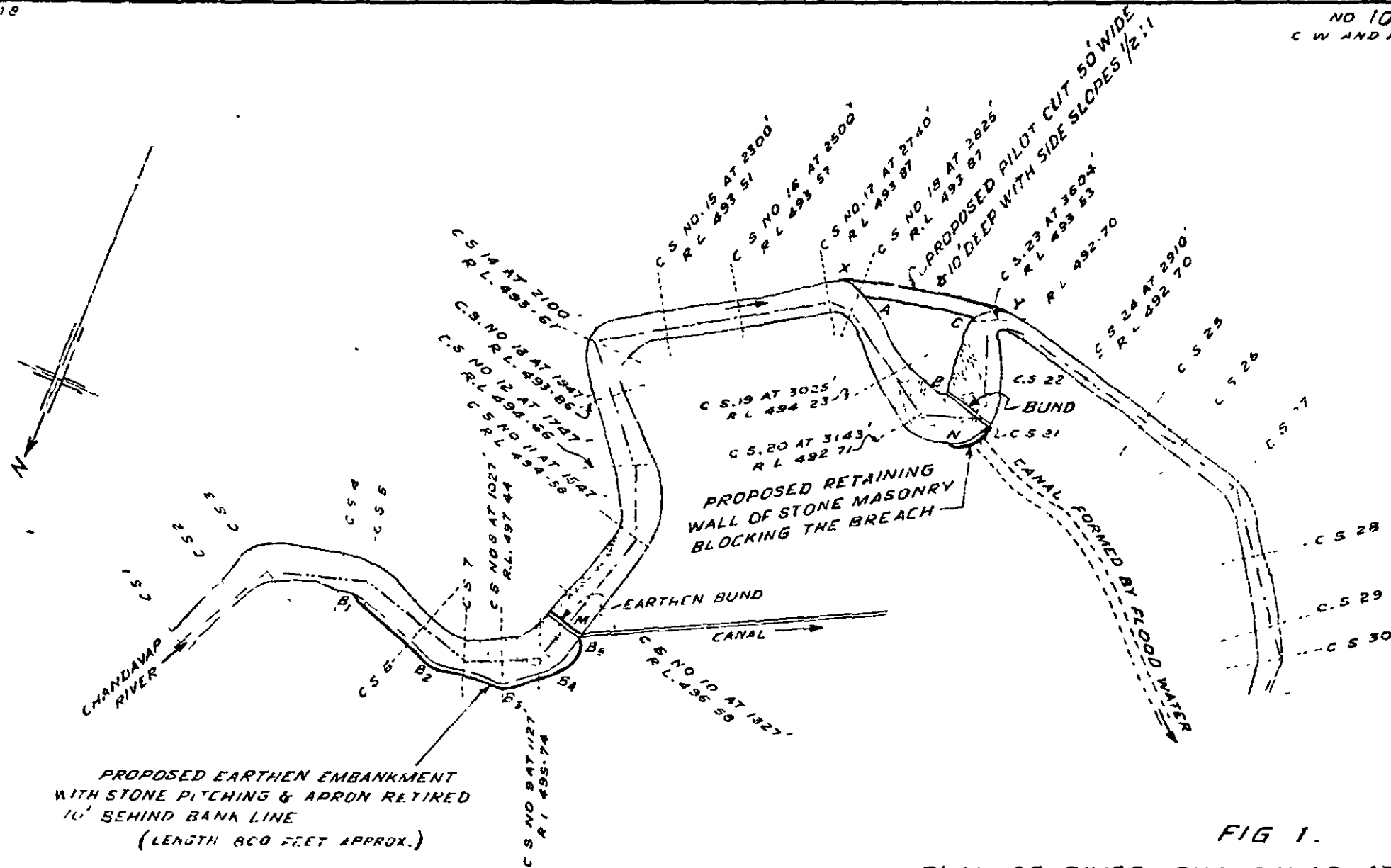
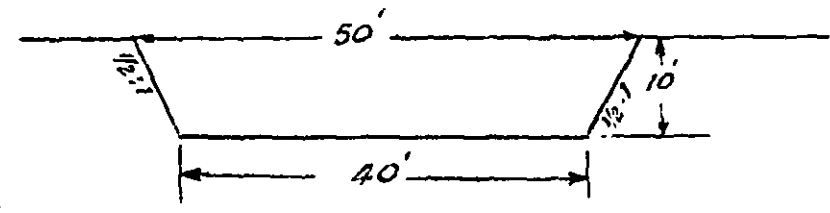


FIG 1.
PLAN OF RIVER CHANDAVAR AT KONHALL!
SHOWING PROPOSED WORKS FOR
FLOOD CONTROL AND RIVER TRAINING.
SCALE: 1" = 400

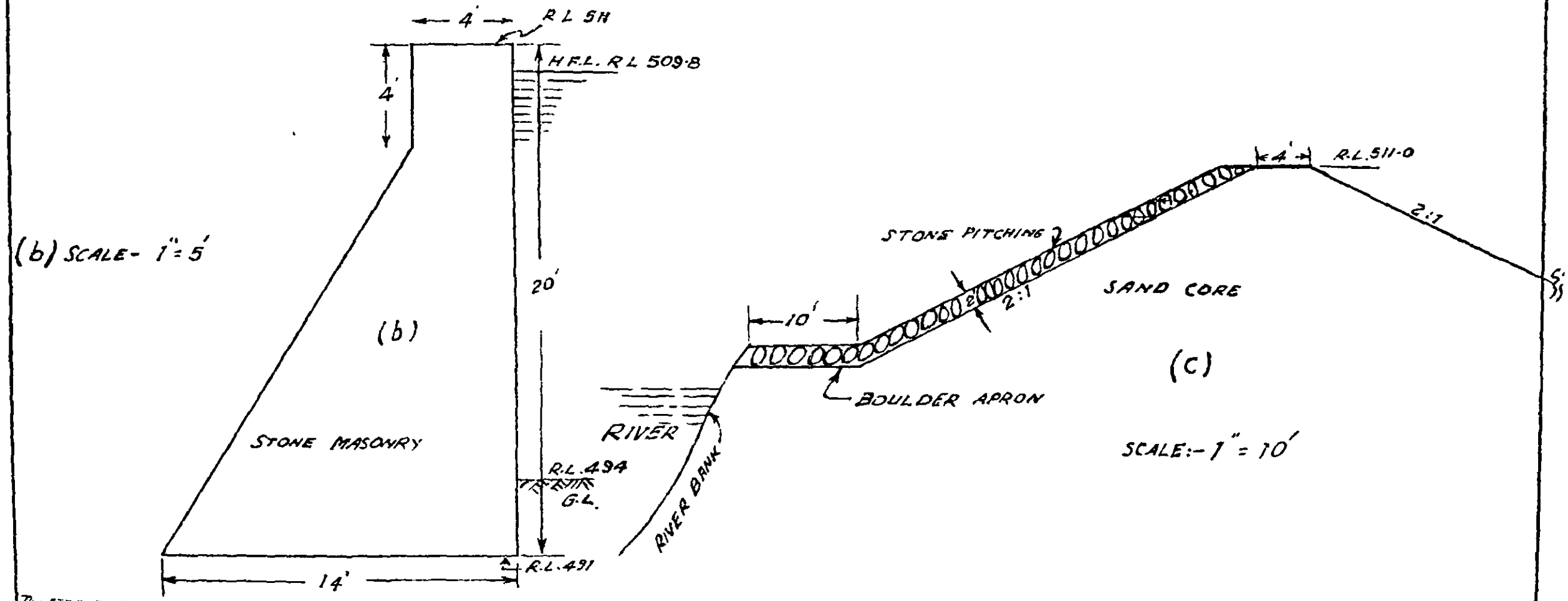
COMP BY *M. Singh*
TRACED BY PARMHI
R-2 952



(a) SCALE - 1" = 20'

FIG. 2.

- TYPICAL SECTIONS OF
- (a) PILOT CUT (X Y)
- (b) MASONRY WALL ACROSS THE BREACH (AT N)
- (c) EARTHEN EMBANKMENT ALONG B1.....B5



(b) SCALE - 1" = 5'

(c) SCALE - 1" = 10'

RIVER CHANDAVAR AT VILLAGE KONHALLI
near Kumta (North Kanara District)
(Vide Drawing No. 10 of Jan. 1952)

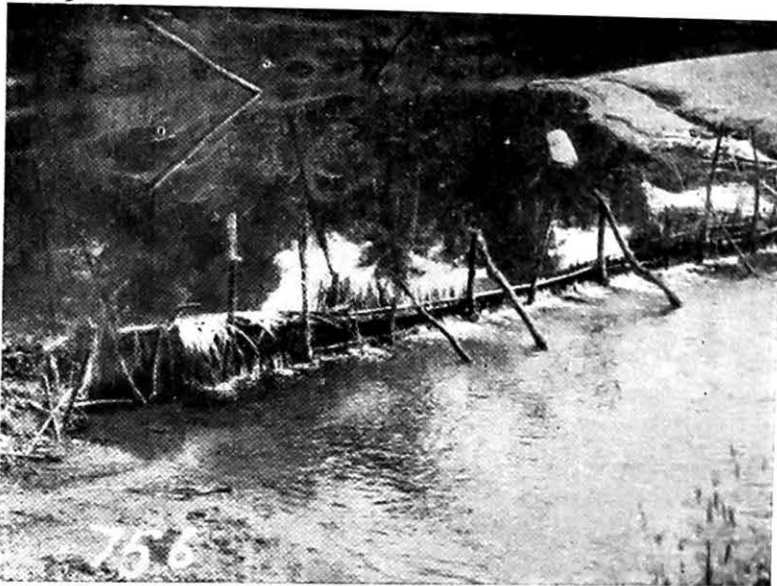


Photo 756-The crude structure of a bund of bamboo and matting which is enclosed in earth, after the floods subside-near point N.



Photo No. 758-The sand shoal along the inside of the bend on the left bank (Same as 756).



Photo No. 761-The site for an earthen bund at M which is constructed across the cold weather channel to divert water to a small canal

would soon develop into a parent channel. This would mean that there would be very little discharge flowing along the bend which area may gradually be reclaimed for cultivation. As this is a very sharp bend it would be almost impossible to make the river flow through the mound of sand by dredging as it would surely sand up again.

- (iv) The earthen bunds should be removed immediately before the monsoon begins so as to avoid high afflux occurring before the bunds are totally washed away. In fact instead of adopting such crude and temporary methods, the object of utilizing cold weather flow could be better achieved by constructing a bandhara with a *pakka* regulator at the offtake for controlling the supply in the canal. This bandhara should work only during winter to head up the supply according to requirements and should be fitted with wooden *karries* which may be removed when flood warnings are received.

3. DAMS AND WEIRS

(ii) Design

9. HIRAKUD DAM LEFT CHANNEL SPILLWAY—

TESTING THE DESIGNS OF RETAINING AND TRAINING WALLS

- (A) PRELIMINARY TESTS WITH PART-WIDTH MODEL WITH SLUICES OF THE UPTURNED TYPE
—G. S. SCALE 1:80.
- (B) FINAL TESTS OF FULL WIDTH MODEL OF LEFT CHANNEL SPILLWAY WITH SLUICES BOTH CONVENTIONAL AND OF UPTURNED TYPE AS IN THE FINAL LAY-OUT PLAN—G. S. SCALE 1:100.

(A) *Part-width model of left channel spillway*—This report refers to preliminary tests on designs of retaining and training walls in the left channel spillway when it was proposed to have all sluices of the upturned type. Based on subsequent readjustment in some major design features, the left channel spillway in the final lay-out now consists of sluiceways of both the upturned type as well as the conventional type with the outside bucket, tests of which are reported elsewhere here.

The general elevation of the bed of site is EL. 500. The maximum design flood level for the reservoir is EL. 630. The normal full supply level is EL. 625. The maximum tail-water elevation is EL. 535. In the arrangement tested in the part model, the lay-out was on the basis of 44 undersluices on the left spillway, and 28 on the right spillway so that the 60 per cent of the discharge for the lower floods of a minimum of 7 lakh cusecs could be disposed of by the left and 40 per cent by the right spillways. For the higher floods of a maximum of 12 lakh cusecs, additional overflow discharge was provided, with the spillway crest kept at EL. 616. The spillway blocks were of 50 ft. width. For the hydraulic model tests, a model to G. S. scale 1:80 was used consisting of 11 blocks of the left channel spillway with a portion of the earth dam on the left. The main terms of reference were the following—

- (a) To determine the tendency of the current which on its way to the spillway may erode the top portions of the earth dam and to take measures to keep the material safe; and
- (b) to determine the suitable design of the downstream divide wall, i.e., shape, height, and length which may afford adequate protection of the downstream portion of the earth dam.

The model lay-out in elevation and plan is shown in Figure 1 and photo 6854. 1,400 ft of the left channel spillway was represented in the model. Half the left channel spillway consisting of 22 undersluices and spillway crest at EL. 616 was constructed in a channel 17.5 ft. wide. The divide wall at the junction of concrete and earth dam both upstream and downstream were constructed as per C. D. O. design. A 5-ft. parapet wall was provided on the upstream divide wall. The earth dam was made up of semi-pervious red silt properly rammed with a covering of granular material $\frac{1}{8}$ inch to $\frac{3}{8}$ inch to represent riprap.

Fig. 1.
Photo 6854

The bed upstream of the dam was laid to contours in cement plaster. Downstream it was laid in $\frac{1}{8}$ inch to $\frac{3}{8}$ inch gravel. Gauges were fixed both upstream and downstream. The model discharge was measured in a 2 ft. standing wave flume.

Tests with maximum reservoir level EL. 630 were made with combined sluice and overflow crest discharges. For pool R.L. 625 and 590 tests were made with only sluices discharging. The following were the test sequences.

- (1) (a) A 5-ft. high parapet wall on the upstream divide wall with downstream divide wall No. 1 as per Int. Eng. Co's design.
- (b) The 5-ft. parapet wall removed.

Fig. 2. It was found that on the upstream side, the velocity of flow was quite small, and the parapet wall was unnecessary (Figure 2). On the downstream side there was an eddy region in the pocket between the end sluice and downstream divide wall, causing deep scour along the wall. There was considerable surging action of the eddy when undersluice jets were unaerated due to the overflow discharge. Beyond the curved wall, the water spilled over the 3.1 sloping protection wall to the earth dam, but the current was quite feeble, and material here was uneroded.

The height of the downstream divide wall up to EL. 560 as provided was more than appeared necessary.

- (ii) No parapet wall, upstream divide wall as originally designed. Downstream divide wall No. 2 as per C. D. O. design. The wall was taken straight up to 295 ft from the axis, i.e., where the earth dam cuts the river bed at EL. 510, and then curved at 50 ft radius towards the earth dam.

In this case the conditions in the downstream pocket were violent for the combined discharge. The height of the wall (up to EL. 540) was found to be too small. Due to the sharp 50 ft R. curvature, the eroded material from the bed downstream of the spillway got deposited towards toe of the earth dam to a level beyond EL. 520, along with the formation of a clear cut channel between this deposited material and the earth dam, involving a serious condition of a washing away of downstream portion of the earth dam.

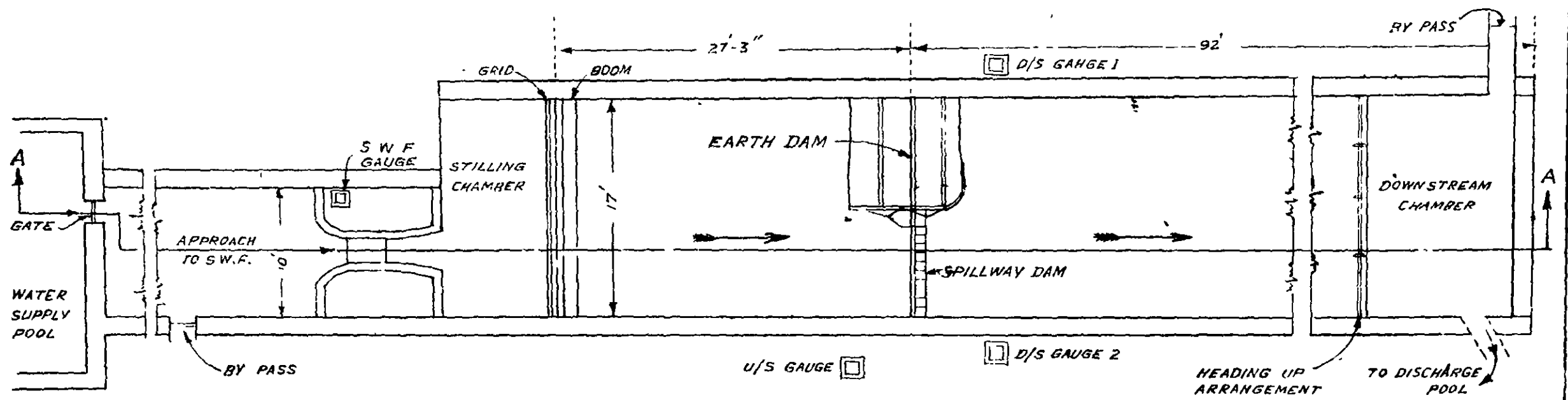
- (iii) Design 2 being found unsuitable, downstream divide wall No. 3 was tested. This was a straight wall taken at EL. 545 up to 345 ft from the axis and then sloping at 2.1 downstream.

Figs. 3 & 4. In this case the conditions in the downstream pocket improved (Figures 3 and 4) due to the increased length of wall; the return eddy was however still strong. The height of the divide wall before full scour took place appeared to be slightly insufficient, but after the initial stages of the scour, the height appeared sufficient. The scoured material from the river bed deposited about 20 ft. away from the toe of the earth dam and the flow conditions between the toe and the deposited material were satisfactory.

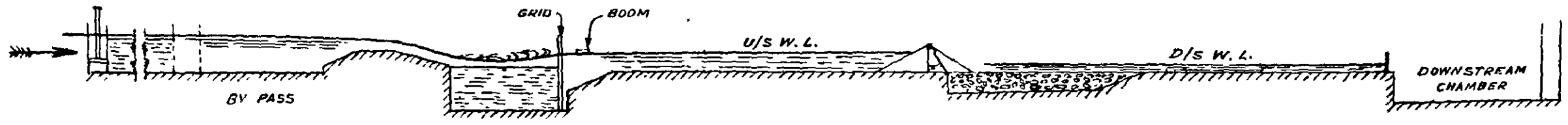
- (iv) Downstream divide wall No. 4: This was curved away from the earth dam with a radius of 150 ft from beyond a straight length of 295 ft., the length of the wall however being the same as in divide wall No. 3.

With this design and with reservoir W.L. 630, the conditions in the pocket due to constriction became very violent and the height of the wall was insufficient. The flow on the earth dam side was however satisfactory. The results, however, were better with divide wall No. 3. The principal results with this preliminary model were therefore—

- (i) The current on the upstream portion of the earth dam was not strong enough to erode the top portions of the earth dam and hence the parapet wall on the upstream divide wall was not necessary.
- (ii) With the undersluices of the upturned barrel type, the height of the downstream wall up to EL. 560 was more than sufficient, but up to EL. 540 was quite insufficient. The wall up to EL. 545 was found to be just sufficient.
- (iii) Out of all the divide walls tested, the long straight wall No. 3 was the most satisfactory.



PLAN



SECTION AA

FIG. 1
 HIRAKUD DAM PROJECT
 1/80 G.S. PART MODEL LAYOUT
 OF
 LEFT CHANNEL SPILLWAY
 AND EARTH DAM
 MODEL SCALE
 0 10 20 FT

COMP. BY: *Meij*
 TRACED BY B G DESHPANDE 15-1-52

Hirakud Dam Left Spillway.

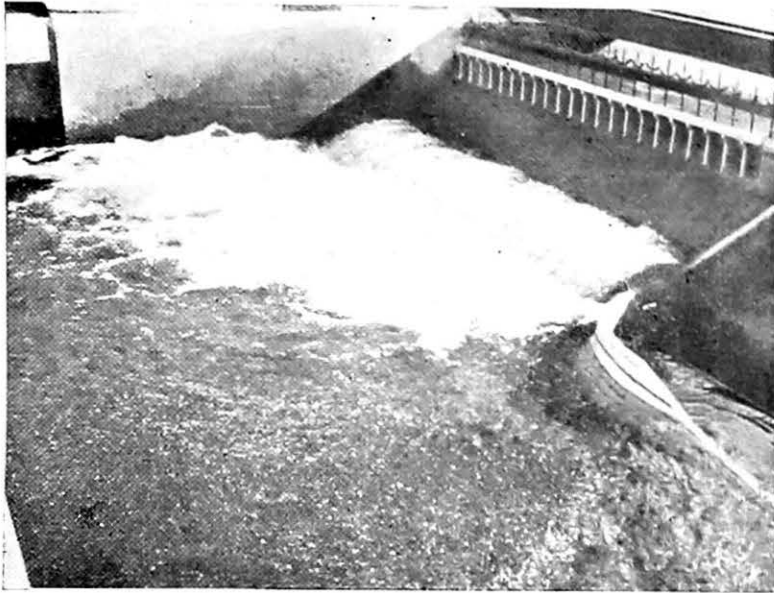


Photo 6854

Flow with preliminary Model - All Upturned sluiceways.
Downstream divide wall No 1. Upstream W. L. 630, downstream
W. L. 535.

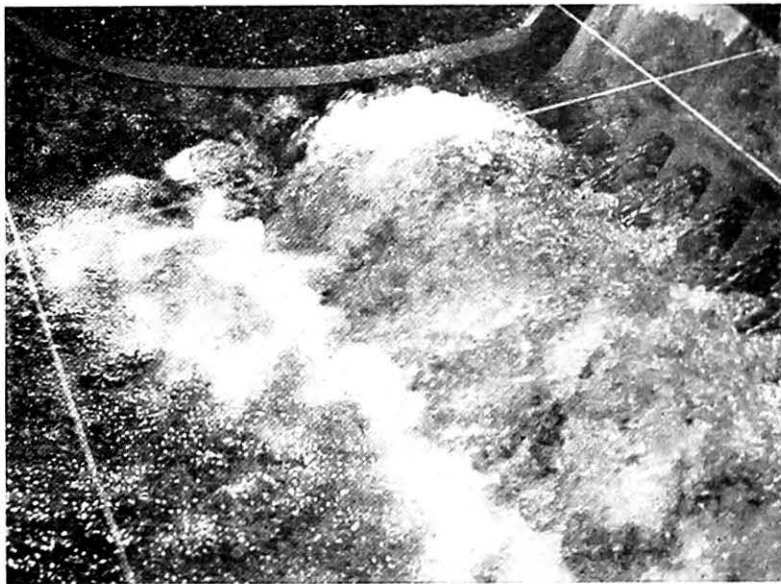


Photo 7459

Flow near Right Training wall downstream. Final Layout
Upstream W. L. 630, Downstream W. L. 535.

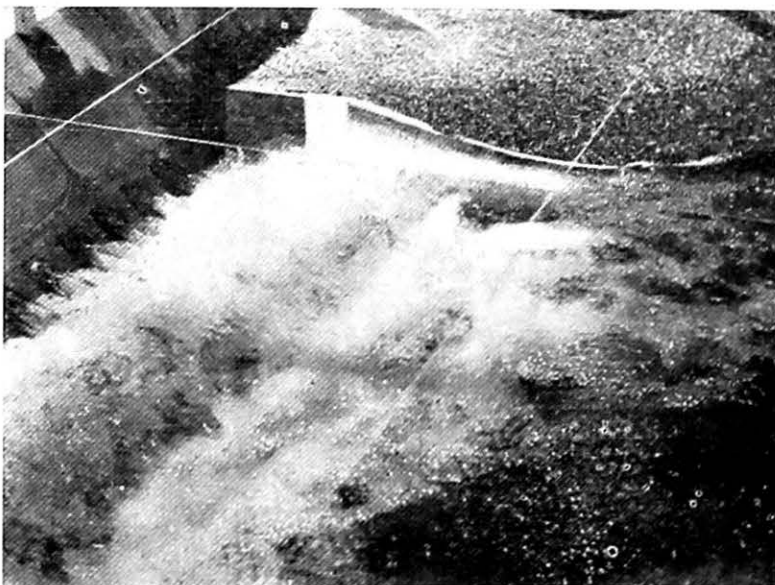
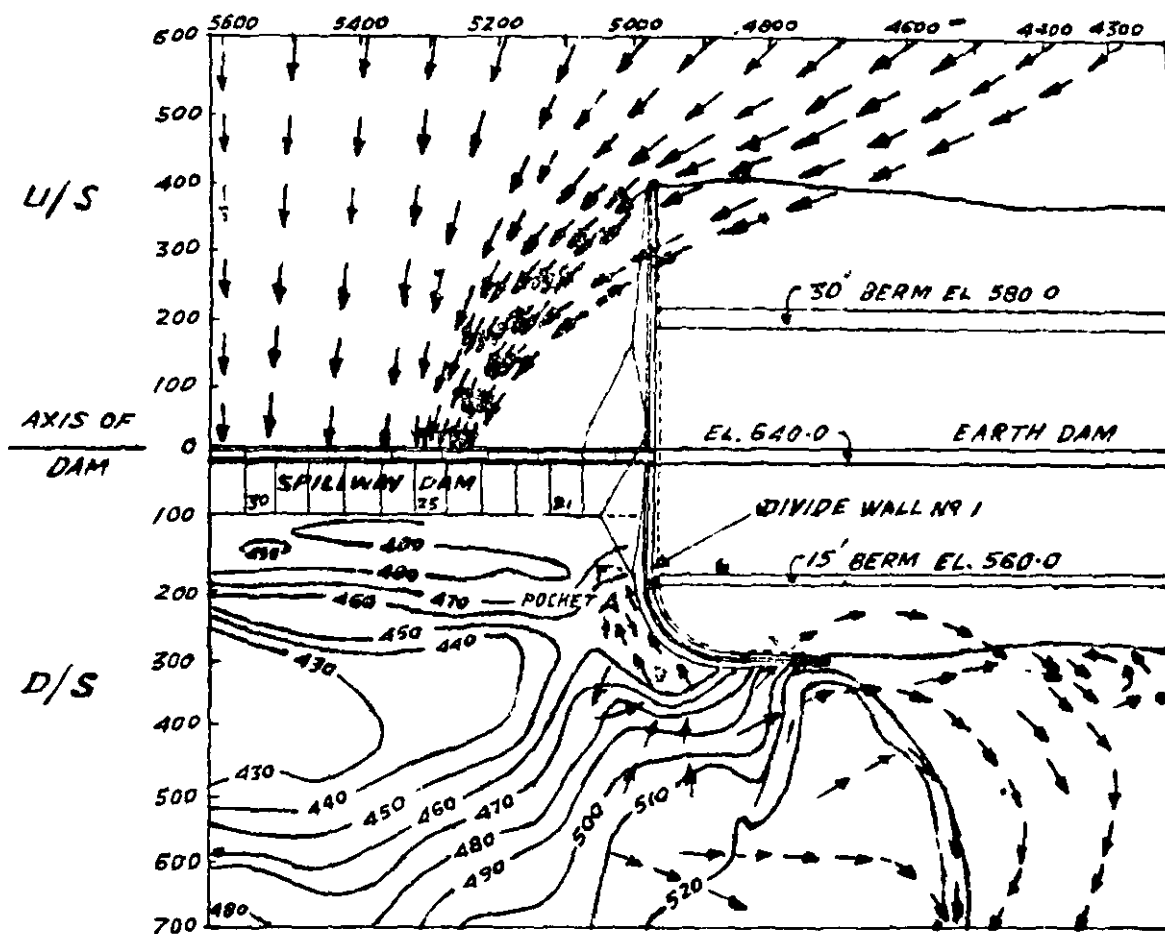


Photo 7460

Flow near Left Training wall downstream. Final layout. Upstream
W. L. 630, Downstream W. L. 535.



PLAN

D/S DIVIDE WALL NR. 1.

U/S DIVIDE WALL ORIGINAL, NO PARAPET WALL.

CONDITIONS:—

U/S W.L. 630.0

D/S W.L. 535.0

ALL SLICES AND OVERFLOW CRESTS OPERATING.

Q PER BLOCK OF 50 FT = 40,000 CUSECS

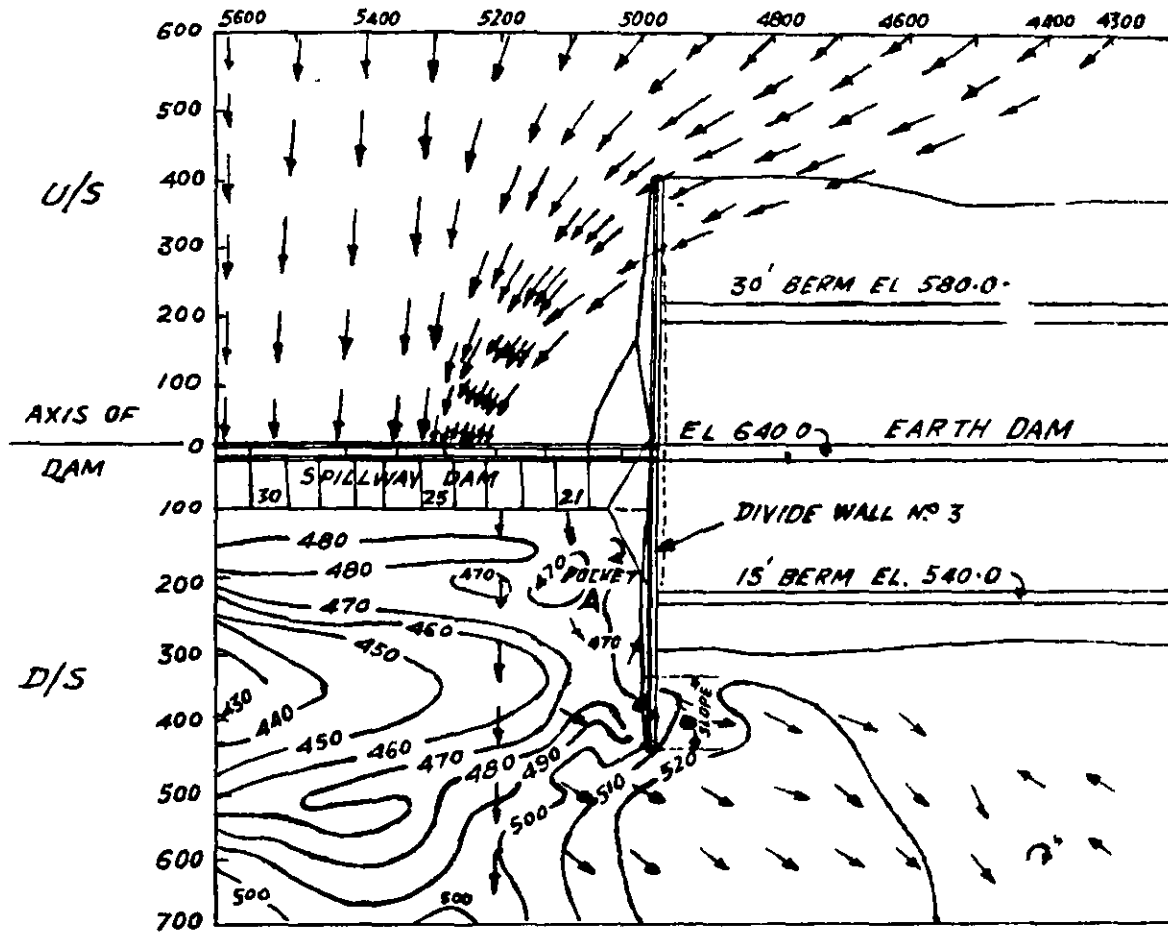
FLOW DIRECTIONS →

CONTOURS AFTER SCOUR ○ 480

REMARKS:—

- 1 RETURN FLOW IN POCKET "A" DUE TO STRONG EDDY
- 2 TOE OF EARTH DAM SAFE
- 3 HEIGHT OF D/S DIVIDE WALL AT EL. 560 IS MORE THAN NECESSARY
- 4 SCOUR NEAR D/S DIVIDE WALL DOWN TO EL. 460.0

FIG. 2
HIRAKUD DAM PROJECT
1/80 G. S. PART MODEL
OF
LEFT CHANNEL SPILLWAY
AND EARTH DAM
SCALE 1"=300'



PLAN

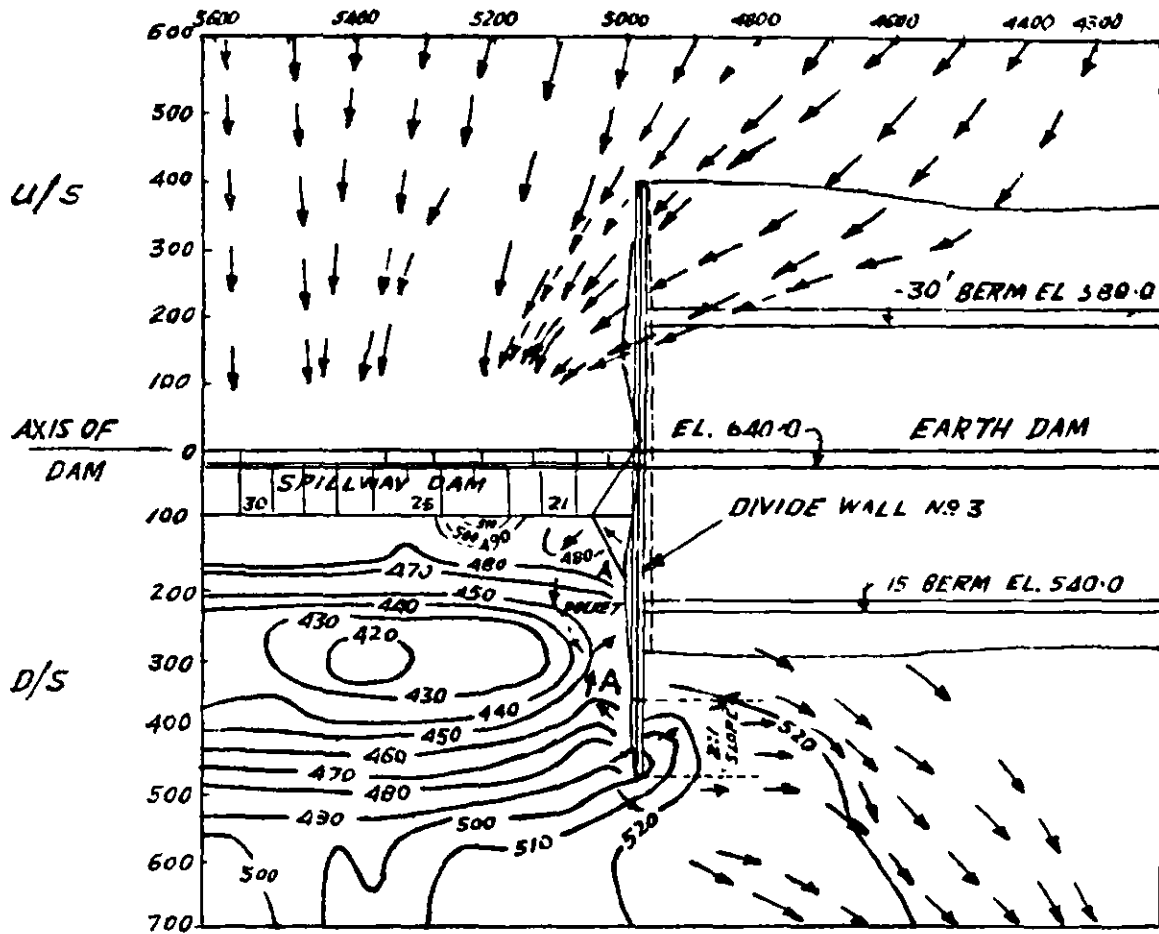
D/S DIVIDE WALL NO. 3.
 U/S DIVIDE WALL ORIGINAL NO PARAPET WALL
 CONDITIONS: —
 U/S W.L. 630.0
 D/S W.L. 535.0
 ALL SLUICES AND ALL OVERFLOW CRESTS OPERATING.
 Q PER BLOCK OF 50 FT. = 39,700 CUSECS
 FLOW DIRECTIONS →
 CONTOURS AFTER SCOUR (470)

REMARKS:—

- 1 FLOW IN POCKET "A" SATISFACTORY
- 2 TOE OF EARTH DAM SAFE
SLIGHT WAVES OCCUR BEHIND THE WALL
- 3 HEIGHT OF D/S DIVIDE WALL AT EL. 545 JUST SUFFICIENT
- 4 SCOUR NEAR D/S DIVIDE WALL DOWN TO EL. 470.0

FIG. 3
 HIRAKUD DAM PROJECT
 1/80 G.S. PART MODEL
 OF
 LEFT CHANNEL SPILLWAY
 AND EARTH DAM
 SCALE 1"=300'

COMP'D BY: — *M. S. Deshpande*
 TRACED BY: B.G. DESHPANDE 5-2-52



PLAN

D/S DIVIDE WALL NO. 3.
 U/S DIVIDE WALL ORIGINAL; NO PARAPET WALL.
 CONDITIONS: —
 U/S W.L. 625.0
 D/S W.L. 530.0
 ALL SLICES AND NO OVERFLOW CRESTS OPERATING.
 Q PER BLOCK OF 50 FT. = 30,900 CUSECS
 FLOW CONDITIONS →
 CONTOURS AFTER SCOUR (420)

REMARKS:—
 1 FLOW IN POCKET "A" SATISFACTORY
 2 TOE OF EARTH DAM SAFE
 3 HEIGHT OF D/S DIVIDE WALL AT EL. 545.0 JUST SUFFICIENT
 4 SCOUR NEAR D/S DIVIDE WALL DOWN TO EL. 460.0

FIG. 4
 HIRAKUD DAM PROJECT
 1/80 G. S. PART MODEL
 OF
 LEFT CHANNEL SPILLWAY
 AND EARTH DAM
 SCALE 1"=300'

COMPD BY:— *M. S. J.*
 TRACED BY:— B. G. DESHPANDE 8-2-52

(B) *Full width model of left channel spillway*—In the lay-out tested for the full model to a G. S. scale 1:100, the arrangement is on the basis of the final design proposals of 40 undersluices on the left spillway and 24 on the right. The crest was now lowered to EL. 610 only, to provide for a larger overflow capacity. The undersluices on the left channel spillway consist of 12 of the upturned type in the central blocks, and the rest of them of conventional type provided with outside buckets. The spillway block width was increased from 50 ft. to 60 ft.

The main terms of reference for the tests were as follows—

- (i) To determine the sufficiency or otherwise of the proposed training walls.
- (ii) To determine the relative scouring tendencies of the upturned sluice jets, and those from the buckets, and to finalise a design for the upturned sluices whose dissipation efficiency would be equally satisfactory as the conventional sluices.
- (iii) To determine general flow conditions both upstream and downstream so as to correct for unsatisfactory findings, if any
- (iv) To determine the necessity or otherwise of a bucket or a short apron at the toe of the spillway blocks which contain the upturned sluices when both overflow and undersluice discharges are present.

The model lay-out in plan and elevation is shown in Figure 5 and Photo 7385. 3,200 ft. of the left spillway and 1,000 ft. of earth dam and transitions on either side were reproduced across a channel 32 ft. wide. The tests were composed of three series—

Fig. 5.
Photo 7385.

- (i) *Series I*—The central group of upturned sluices were given rectangular unflared outlets. The downstream bed was made up of erodible gravel throughout, and no attempts were made to reproduce the rock outcrops occurring in nature.
- (ii) *Series II*—The upturned sluices were given flared outlets. The downstream bed was kept erodible up to 1,000 ft. from the dam and beyond that the rock outcrops were represented by larger size stones and pebbles. The tests showed that changes in the nature of the reproduction of the downstream bed beyond 1,000 ft. had little effect on the results.
- (iii) *Series III*—Different ski-jump bucket shapes were tried for conventional sluices. Also the downstream divide wall on the left side and the approach road were rearranged according to a later modification received from the C. D. O. Tests were finalised to obtain the optimum height and alignment of this divide wall.

Test Results

(1) *Series I*—The main findings were that on the downstream side, right training wall alignment was satisfactory but height (EL. 540) was insufficient (Photo 7459), while the left training wall was satisfactory (Photo 7460) both in alignment and height (EL. 560), except for the case of "hurdlng" at maximum flood with combined undersluice and overflow discharges. This could be avoided only by avoiding the operation either of the end sluices or the end crest bays.

Photo 7459.
Photo 7460.

On the upstream side, the flow near right retaining wall was satisfactory. However for the left wing wall, the alignment of which follows the general upstream section of the earth dam (see Figure 5a), there is introduced a substantial rotary motion to the flow coming from sideways to the end section of the spillway. This resulted in the appearance of a strong eddy at the corner prominently visible under conditions of undersluice discharge only with crest gate closed. The size of the eddy was comparable to the overall size of the sluice cross-sections. It was formed, then "sucked in" in one of the end sluices and then formed again, the cycle lasting for about a minute. This feature was a little undesirable, but could not be avoided with a wing wall of the type put in. It was not serious however, as putting trash racks for the end sluices would stop any tendency of floating logs from being sucked in, or alternatively, the 3 or 4 left sluices should not be operated when the upstream W. L. is in the neighbourhood of EL. 590 to 610.

Fig. 5a.

In tests on the scouring tendencies it was found that in the absence of overflow discharge while the toe of the dam below the upturned sluices was safe due to the strong return roller, the scour downstream was more than in the case of the conventional sluices. In the case of combined sluice and crest discharge, however, the maximum scour was about the same for both these types, being about 50 ft. at a distance of 350 ft. from the toe of the dam.

For the central spillway blocks, the absence of protection at the toe caused local scour of the order of 20 ft but with the provision of a bucket no such scour Photo 257. appeared (Photo 257).

(ii) *Series II*—The tests of this series with flared outlets showed that the maximum scour could be reduced from about 50 ft. to 25 ft. only by flaring the outlet. The upturned sluices with flared outlet were found no less efficient than the conventional sluices with outside bucket, against scouring tendencies (Figure 6). Fig. 6.

(iii) *Series III*—In this series four cases of the outside bucket II-V were tested (Figure 7) in addition to tests on bucket I as given in the design supplied. The tests showed that the bucket as per supplied design was the most satisfactory under the present tailwater conditions. Fig. 7.

Water Profiles and Aeration

The aeration for the central upturned sluices was found to be very satisfactory in the absence of overflow discharge. For the conventional sluices no extra aeration is possible, except for the high velocity stream from the trajectory bucket rising very high due to the end lip angle of 40°.

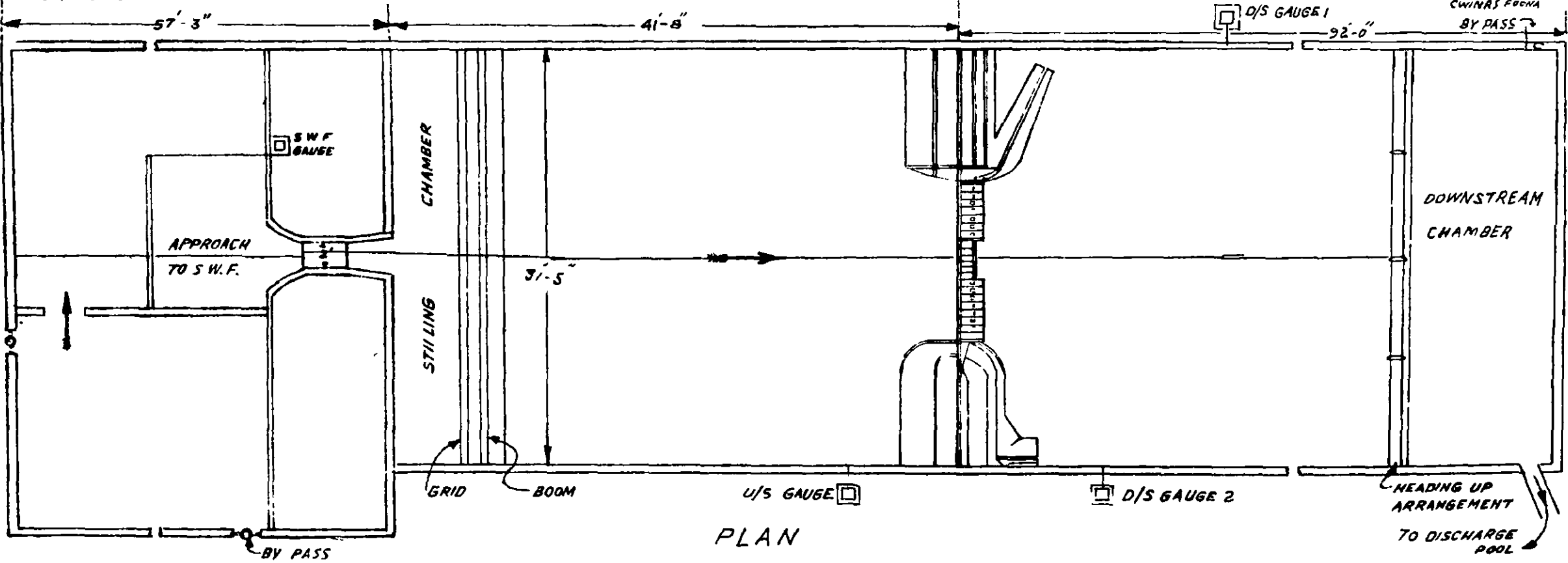
Figs. 8 & 9. On the downstream side, near the right and the left training walls, the water profiles are shown in figures 8 and 9 under combined flow conditions both when the end sluices and the end crest bays were operating separately or jointly. These show that maximum wall height of EL. 560 on the left and EL. 550 on the right would be sufficient if the end sluices are not operated under the maximum discharge conditions.

Recommendations

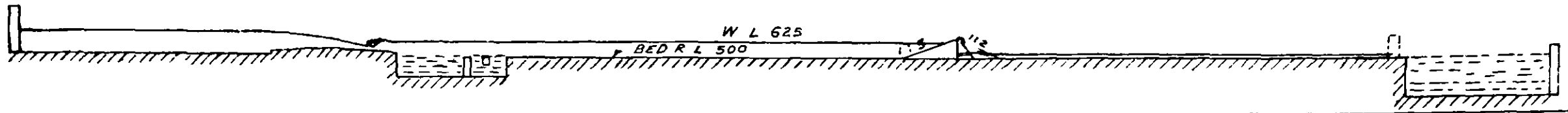
- (i) On the upstream side, the retaining wall and the flank walls are satisfactory. On the downstream side, with the end sluices closed under maximum discharge conditions, the height of the left training wall as designed (EL. 550) was sufficient. The elevation of the right training wall however may be kept at EL. 550 as against 540 designed). If the end bays and sluices were to be operated at maximum flood, the walls would have to be raised to EL. 575 on the left and EL. 570 on the right.
- (ii) For the divide walls between spillway blocks containing the two types of sluiceways, a height up to EL. 530 is recommended, the walls being extended up to the end of the bucket.
- (iii) The 50 ft. radius bucket with 40° lip angle as per C. D. O. design is sufficient from the point of view of a good 'throw' of the jet overlying a strong return roller which would pile up scoured material at the toe of the dam thus rendering it safe against undermining.
- (iv) The protective work at the toe of the central block which would be unnecessary in the absence of overflow discharge can consist of a 35 ft long apron with 40 ft. radius and an end sill with top at EL. 510. For uniformity of construction, however, the 50 ft. radius bucket as in (iii) can be accepted being on the safe side.
- (v) On the upstream side, near the left flank wall, the five end sluices can be protected by trash racks to provide against sucking action of the eddy observed or they may be kept closed for upstream water elevations between 590 and 610.

A.R.T 1951-ITEM 9

MP 1023 OF JUNE 1951
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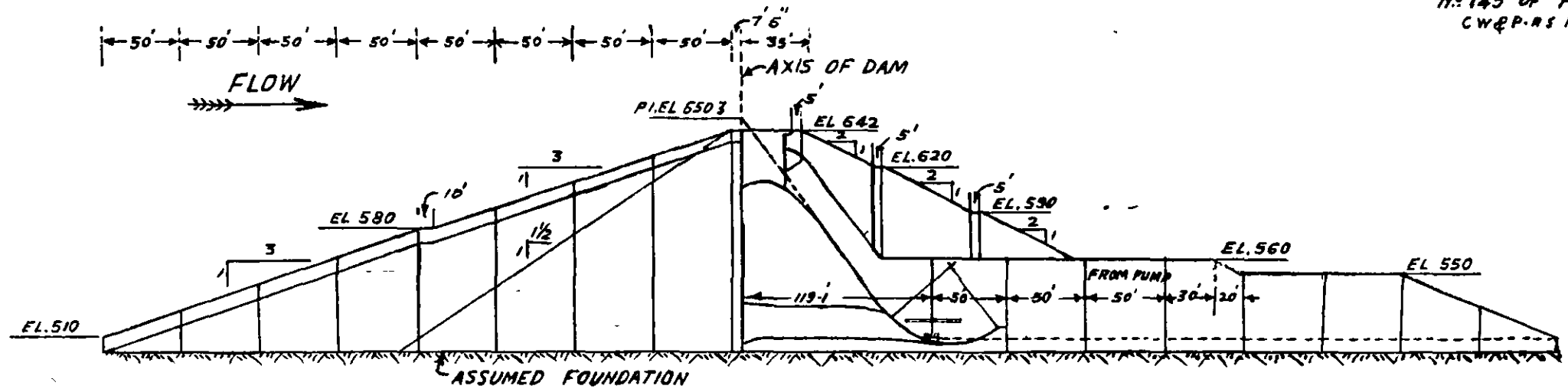
PLAN



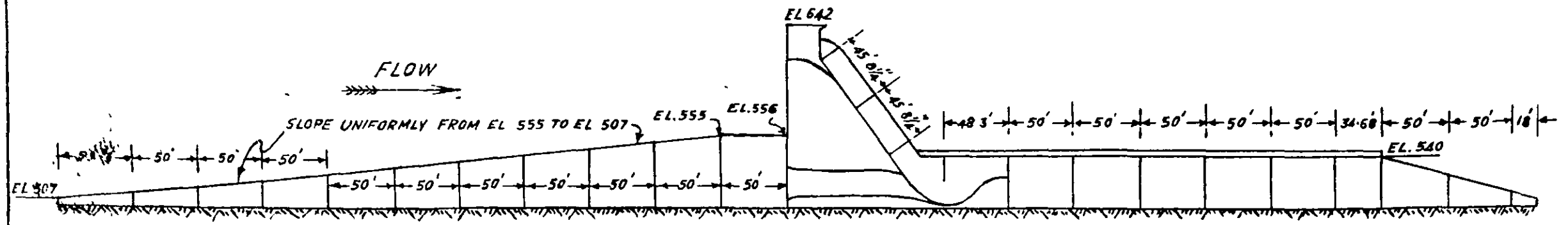
SECTION

FIG. 5
HIRAKUD DAM PROJECT
 :/100 G.S. PART MODEL LAYOUT
 OF
 LEFT CHANNEL SPILLWAY
 AND EARTH DAM
 MODEL SCALE

COMP'D BY *M.S.P.*
 TRACED BY B. S. DESHPANDE 11-1-52



LEFT CHANNEL SPILLWAY LEFT FLANK WALLS U/S AND D/S
DEVELOPED SECTION



LEFT CHANNEL SPILLWAY RIGHT FLANK WALLS U/S AND D/S
DEVELOPED SECTION

NOTE.- EDDY DEVELOPES NEAR U/S LEFT FLANK WALL AS IT IS VERY HIGH.
NO EDDY NEAR U/S RIGHT FLANK SINCE RETAINING WALL THERE IS LOW.

COMP. BY: *[Signature]*
TRACED BY: B G DESHPANDE 21-1-52

FIG. 5A
HIRAKUD DAM PROJECT
HYDRAULIC MODEL 3(B)
1/100, G. S. MODEL
OF
LEFT CHANNEL SPILLWAY
AND PARTS OF EARTH DAM
SCALE 1" = 100'

Hirakud dam Left Spillway.

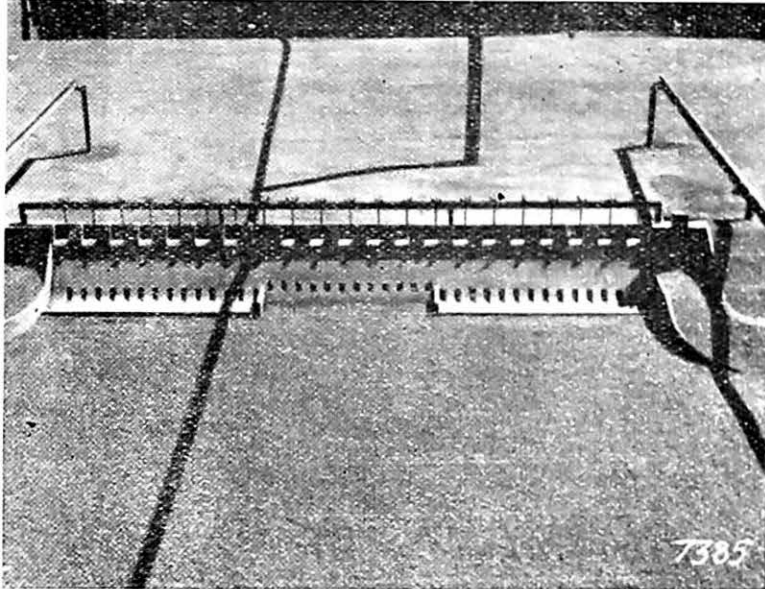


Photo 7385

Hirakud Dam – Left Channel Spillway. Final layout with central Upturned blocks. Model looking Upstream.

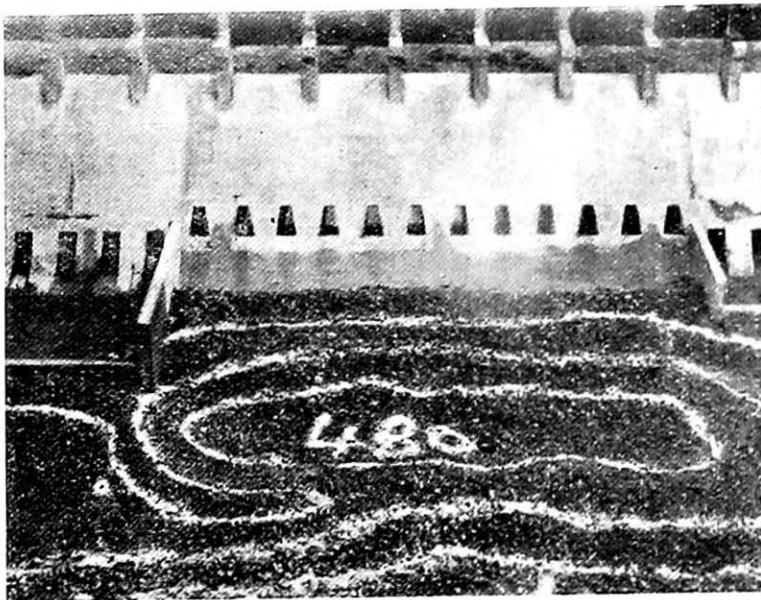


Photo 257

Control of scour below central blocks. 35 ft apron. Scour with combined discharge over crest and through sluices. Upstream W. L. 630, Downstream W. L. 535.

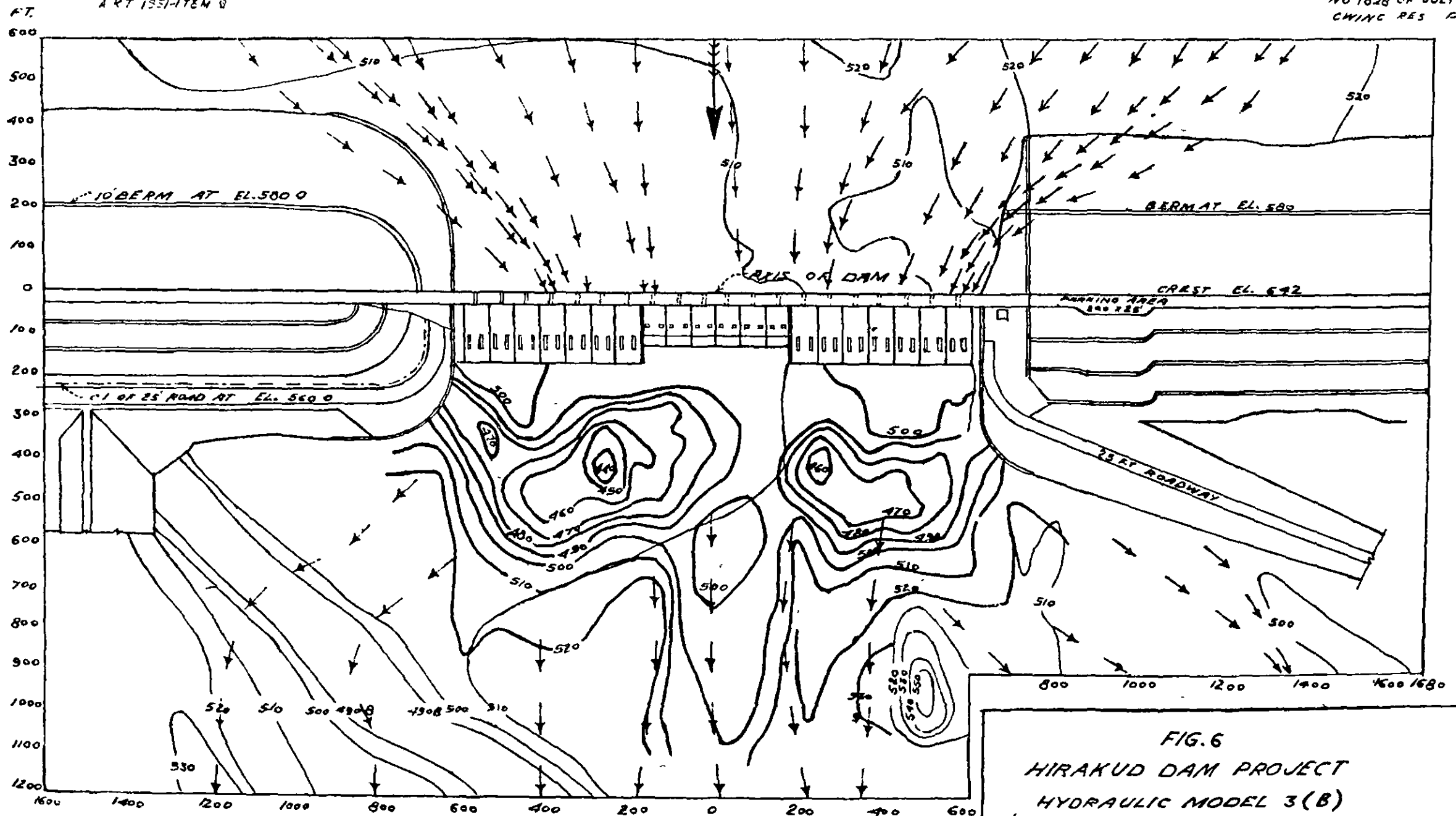


FIG. 6
 HIRAKUD DAM PROJECT
 HYDRAULIC MODEL 3(B)
 1/100 G.S. MODEL OF LEFT CHANNEL
 SPILL WAY WITH PARTS OF EARTH DAM

SCALE 300 200 100 0 100 200 300 FEET

ALL CREST OPERATING U/S WL 625 CENTRAL UPTURNED SLUICE OUTLET MODIFIED.
 COND BY [Signature] D/S WL 535 STABILISED BED CONTOURS IN THE RIVER D/S AFTER
 TRACED BY TAKALKAR 2-7-51 3 HOURS RUN WITH SLUICES AND OVERFLOW SPILLWAY DISCHARGING

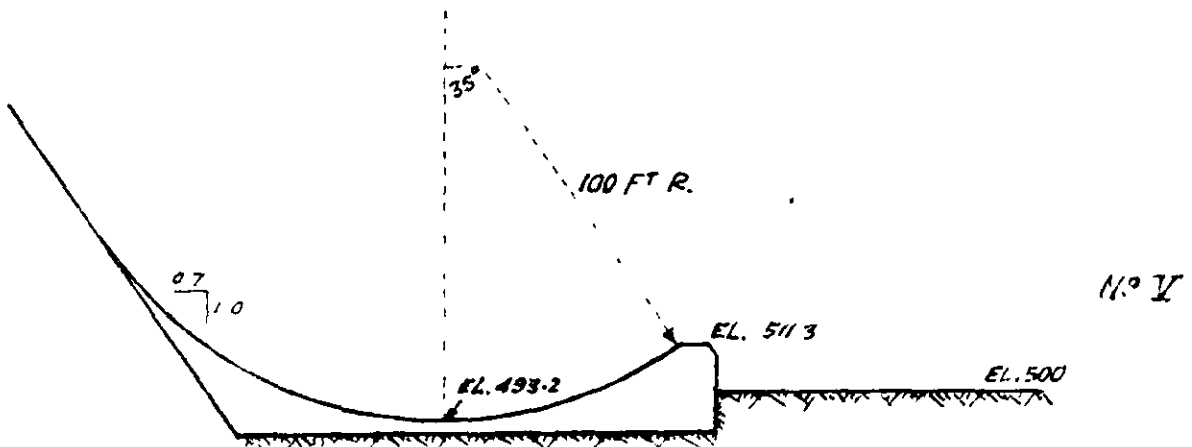
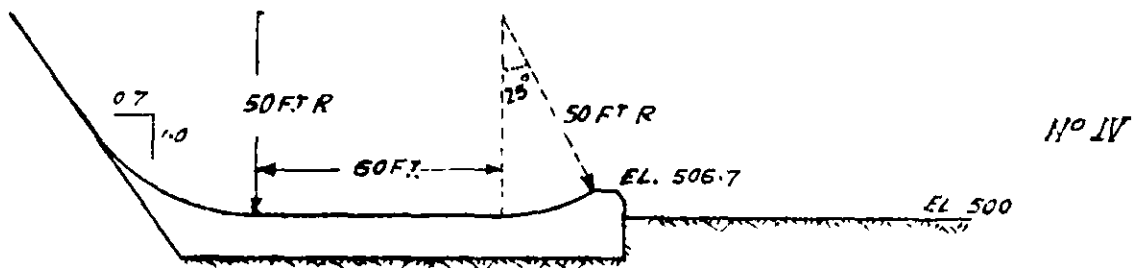
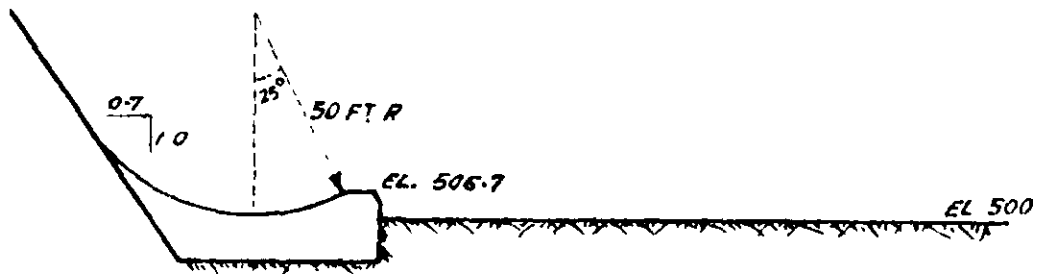
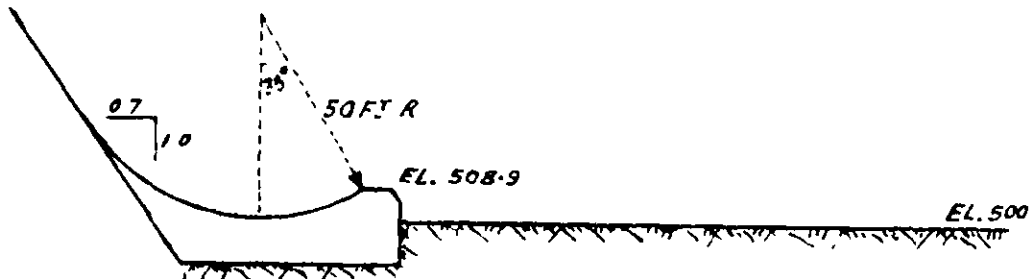
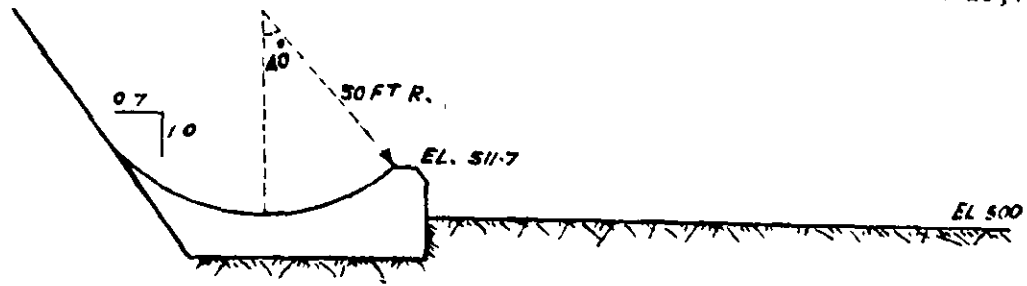
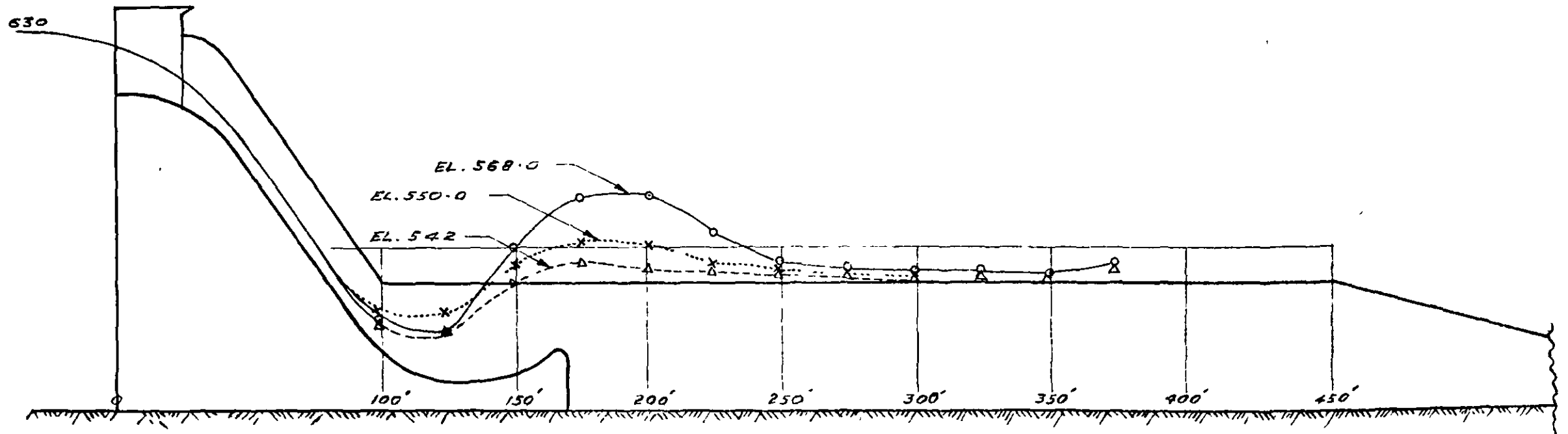


FIG. 7
BUCKETS TRIED
IN

RIGHT FLANK SLUICWAY GROUP
OF LEFT CHANNEL SPILLWAY
SCALE 1" = 50'

COMP'D BY: *M. S. ...*
TRACED BY: B G DESHPANDE 2 9-51

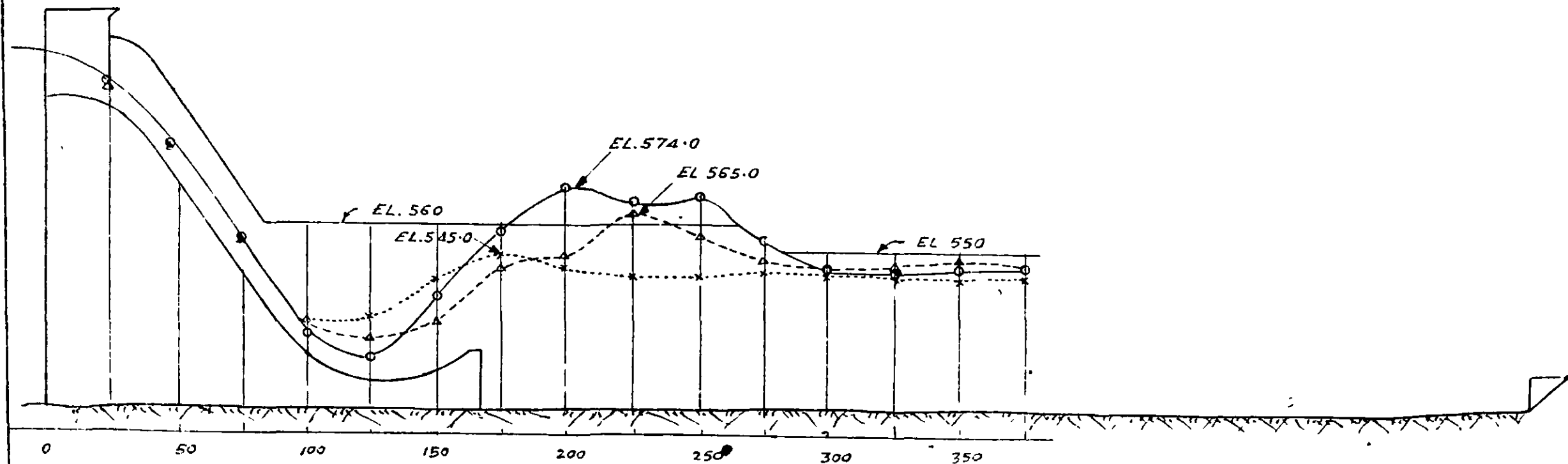


REFERENCES.

- 1. BOTH SLUICE AND OVERFLOW OPEN ———○———
- 2. CREST GATE EXTREME RIGHT CLOSED
 AND SLUICE DISCHARGING - - - - - × - - - - -
- 3. SLUICE GATE EXTREME RIGHT CLOSED
 AND OVERFLOW DISCHARGING - - - - - △ - - - - -

FIG. 8
 1/100 SCALE MODEL
 WATER PROFILES ALONG RIGHT TRAINING
 WALL (DEVELOPED) OF LEFT CHANNEL
 SPILLWAY.
 SCALE. 1" = 50'

COMP. BY: *MP*
 T. BY PARRNI.



- REFERENCES:~
- 1. BOTH SLUICE AND OVERFLOW○-----○
 - 2. CREST GATE EXTREME RIGHT CLOSED AND SLUICE DISCHARGINGx-----x
 - 3. SLUICE GATE EXTREME RIGHT CLOSED AND OVERFLOW DISCHARGING△-----△

FIG. 9
1/100 SCALE MODEL
WATER PROFILES ALONG LEFT
TRAINING WALL (DEVELOPED) OF
LEFT CHANNEL SPILLWAY.
SCALE 1" = 50'

COMPO. BY: *[Signature]*
TRACED BY: B. G. DESHPANDE. 4-9-51

10. KAKRAPAR WEIR PROJECT ON RIVER TAPI—TESTING OF ALTERNATIVE ALIGNMENTS OF WEIR, AND LOCATION OF SCOURING SLUICES AND REGULATORS

The Kakrapar weir hydraulic model tests were taken in hand in November 1951 with the terms of reference as follows:—

1. Studies of Weir Sections and Aprons

With discharges varying from 6 lakh to 12 lakh cusecs, the experiments are required to be carried out to obtain information on the following—

- (i) Freedom from subatmospheric pressures on the weir profile and the effect of reducing the radius of the arc upstream of the top of the crest.
- (ii) Sufficiency or otherwise of the length and arrangements of the apron at the rear of the weir.
- (iii) The afflux and the elevations for different river discharges.
- (iv) Necessity or otherwise of the end sill and pressures on the front face and the rear face of the sill.
- (v) General adequacy of the weir arrangements.
- (vi) Effective discharge for the different lengths of weir, i.e., the inclined and the straight reaches.

2. Studies for Protection Measures for the River Bank on the Right

The right bank is curved and has considerable overburden. Studies are to be made for protection measures which may be necessary to prevent further erosion on the right side of the weir.

3. Studies on Regulators

The left bank regulator is designed to carry 3,000 cusecs. It is located on solid rock and the channel water is to be flumed in the head reach. It is to be examined whether any energy-dissipating arrangements are necessary. The afflux caused by the regulator is to be examined, and it is to be determined whether the bottom of the breast wall could be made elliptical without increasing afflux

For the right bank regulator, rock is met with at EL. 150 and the canal discharge is proposed to be 1,000 cusecs. The design is to be tested for general adequacy.

4. Relative Location of Weir, Regulator and the Scouring Sluices

The location of the scouring sluices in the weir with respect to the location and the angle of offtakes is to be studied, with a view to satisfactory freedom of canal water from sand and silt.

5. Scouring Sluices

Experiments for pressure distribution, discharge coefficients, and protection measures downstream are to be made

The model tests were planned over a composite model to G. S. scale 1:100, the necessary sectional models of weir sections generally of G. S. scale 1:40, and regulator and scouring sluice models to G. S. scale 1:24. Herein is reported the scope of tests as made up to the middle of December 1951. Subsequent tests including those on different weir alignments will be reported later.

The river at Kakrapar weir site is about 2,500 ft. wide and the general configuration is shown in a copy of an aerial photograph (Photo 726) Upstream of the weir site, the river with its general flow westwards, has a very flat slope and very little curvature and has a sandy bed with high stiff banks on both sides. The axis of the weir is placed where the river is crossed by transverse dykes going down

the river bed. Downstream of the proposed weir site, for five miles or so, the bed consists of exposed rock outcrops of the order of 5 ft. in height. There is a very pronounced curvature of flow from about $1\frac{1}{2}$ miles below, turning the river channel almost southward, followed by another bend turning the river back to westward where the river is met by the Moticher Nala, a small weir across which will form a balancing reservoir. The entire portion of the sharp bend consists of exposed rock over a wide area of the channel which has a very rapid fall of the order of 6 ft. per mile. The river, in this stretch, forms rapids during winter.

Just below the axis of the weir, while the exposed rock dyke extends from the left bank right across the river, it dips again just near the right bank, not coming out until about 4,000 ft. northwards. On the right side, there is a local enclave, which has been caused by side erosion of the overburden of the right side high bank. This has taken place during high floods. The deepest bed level of this enclave is about R. L. 137 (as shown on contoured plan). The main channel of the river Tapi is much deeper, about R. L. 125. Both upstream and downstream of this enclave the river flows in straight reach. When the weir is built to R. L. 155, the flow over the weir during floods would be centralised as shown by model tests and the tendency of the high velocity flow to take a curve towards the right bank to attack the enclave would be much reduced. There is thus no danger to the right bank.

Fig. 1. The alignment of the weir and the location of the scouring sluices and the canal head regulator as proposed is shown in Figure 1. The general bed rock level outside the deep sections is E.L. 150. The weir crest is kept at E.L. 155. The sections of the weir are of four types depending upon the bed rock level. The main central portion of the weir is of type IV with its downstream apron level at R. L. 125. The other three types have flatter sections, type I being flattest with the crest at E.L. 155 and bed at E.L. 150, and constitutes the type for a large part of the proposed flank weirs.

Composite Model

Fig. 2. The model lay-out is shown in Figure 2 representing 3,000 ft. upstream and 4,000 ft. downstream of the weir. The discharge was measured by a 25 ft. S.W.F. which led the water into the model through stilling grids and booms.

The straight reaches of the weir of different types, as well as the scouring sluices and canal regulators were constructed of wood. The curved transitions between the straight portions were cast in cement concrete and the whole weir fixed in place with its crest at E.L. 155.

The bed was laid to contours in $\frac{1}{8}$ inch to $\frac{3}{8}$ inch gravel. The rock outcrops were represented by $\frac{1}{2}$ inch stone pieces. For some of the final tests the area round the canal regulator was made of cement plaster, according to rock levels.

In view of the fact that the downstream bend of the river is sufficiently far downstream and the river has little curvature upstream of the weir for some distance, the problem of silt exclusion is principally a problem of the weir alignment.

Effective Length of the Inclined Reaches of the Weir

Fig. 3. The total length of 4,213 ft of the weir consists of a normal reach 1,006 ft. long, and inclined reaches 1,374 ft on the right flank and 1,833 ft. on the left. Of the normal reach of 1,006 ft, 900 ft. consists of weir of type IV section with downstream apron at E.L. 125, and the rest 106 ft. consists of weir of type III section with downstream apron at E.L. 135. The discharges as measured in this model against different upstream water levels and corresponding downstream tail water levels are shown in Figure 3.

The velocity heads in different locations of the weir are different. To take a suitable mean velocity head, in order to be able to make use of the discharge coefficients obtainable from section models, the area of the river cross-section at 400 ft. upstream of the weir, for 2,200 ft. of river width, was obtained for different water levels, and the mean velocity heads were obtained on the basis of the observed discharge as shown in Table 1.

ITEM No. 10.
A. R. T, 1951

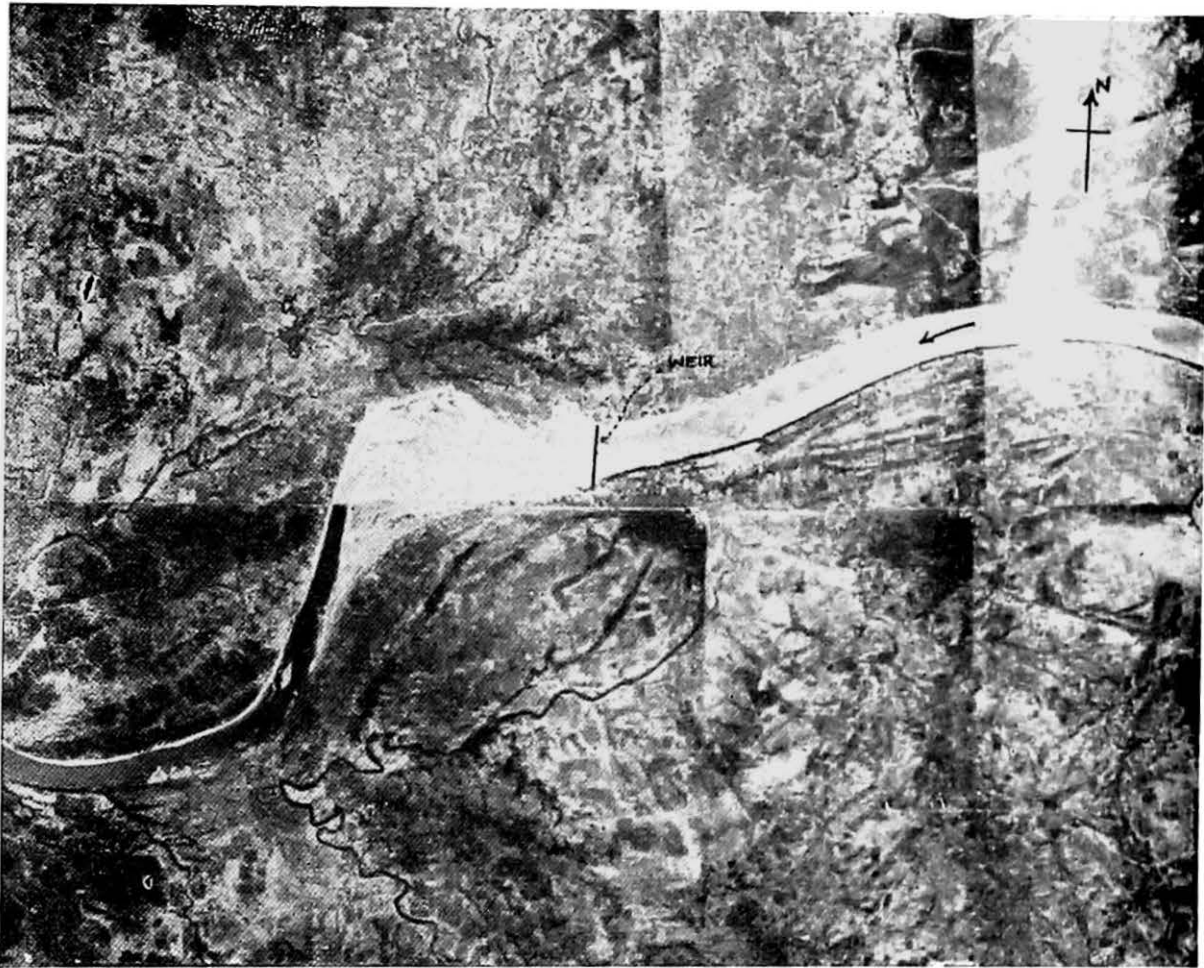


Photo No 726.
Aerial photograph of river Tapi at Kakrapar
Weir Site.

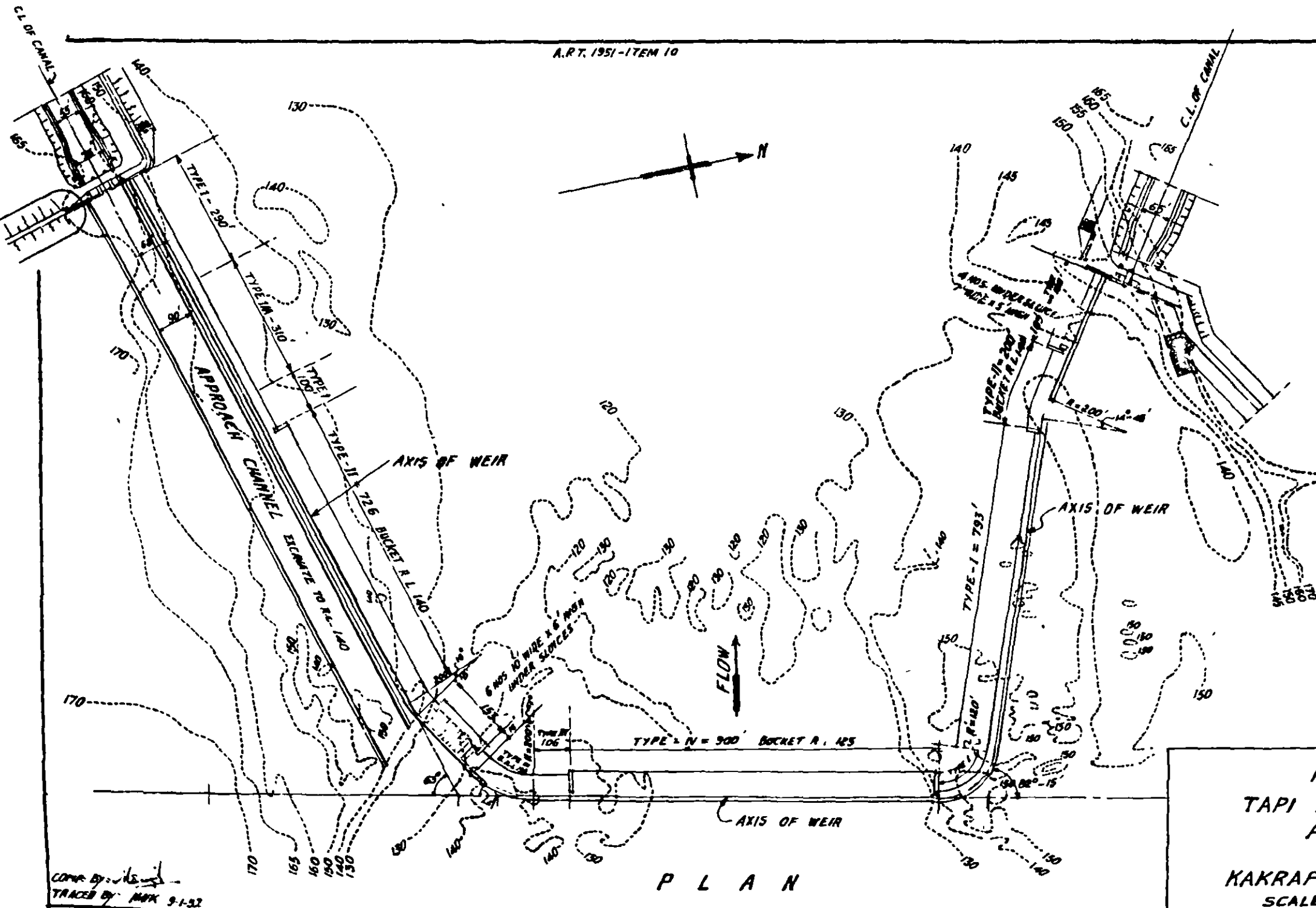
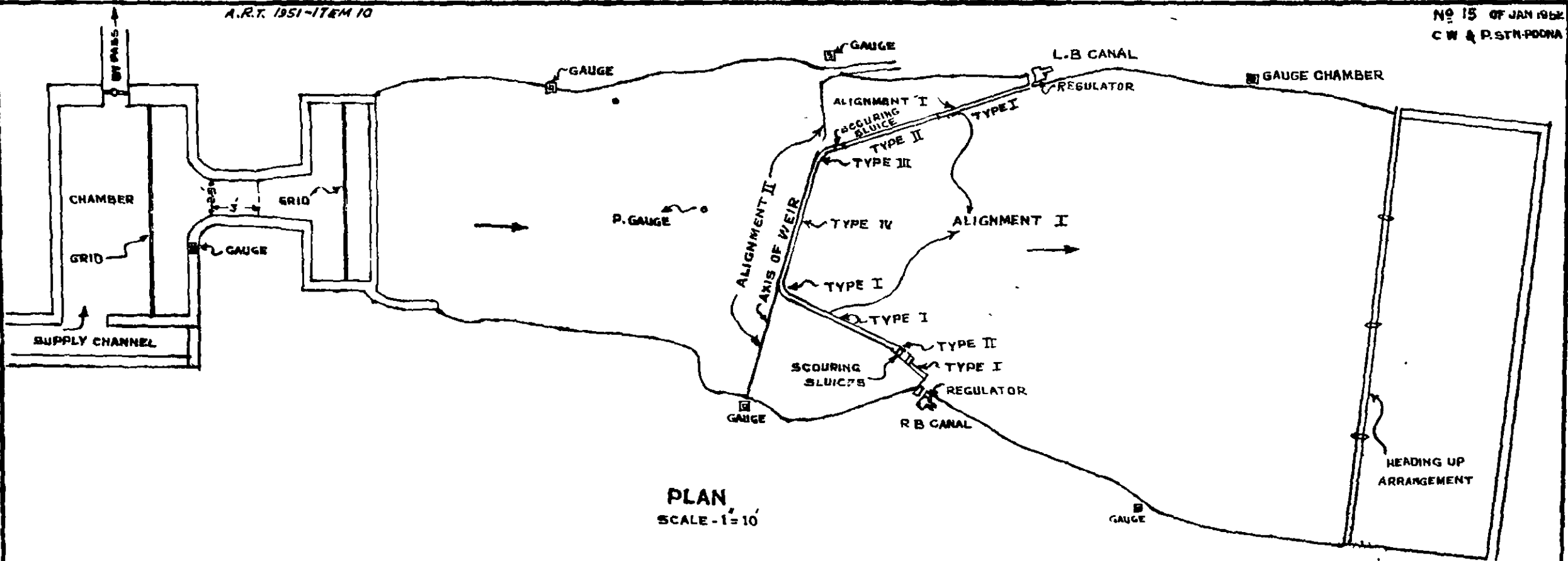
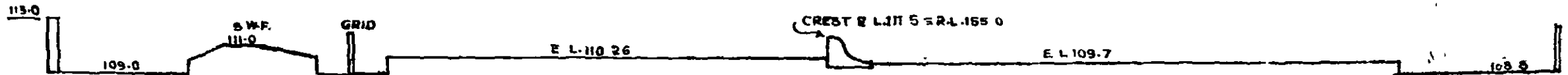


FIG. 1
TAPI PROJECT
PLAN
OF
KAKRAPAR WEIR
SCALE: 1" = 300'

CONTR. BY: [Signature]
TRACED BY: M.V.K. 9-1-52



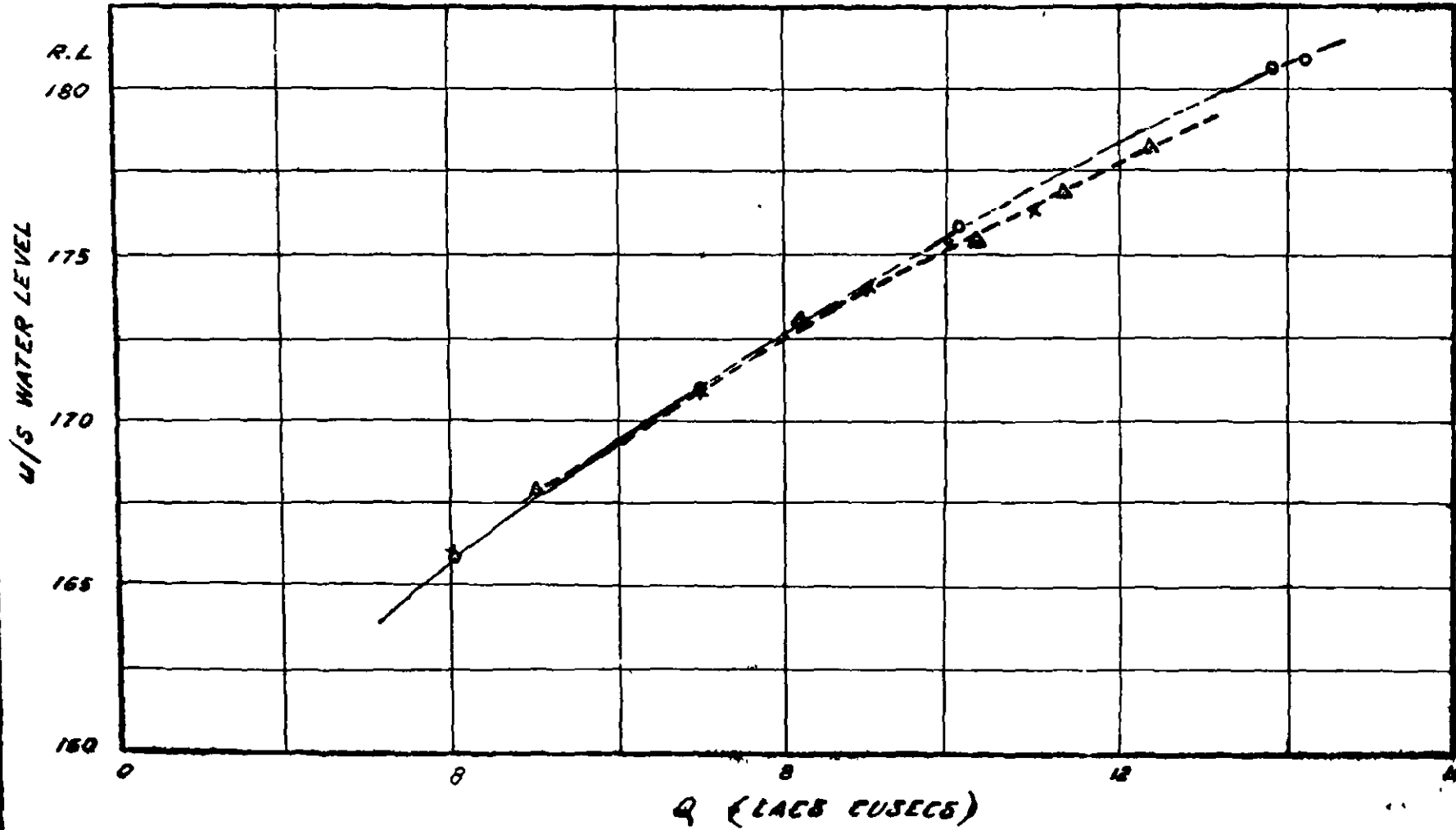
PLAN
SCALE - 1" = 10'



L. SECTION
SCALE - 1" = 10'

FIG. 2
TAPI PROJECT
LAY OUT PLAN OF
1/100 G.S MODEL
SHOWING KAKRAPAR WEIR ALIGNMENTS

C. BY - *Manglik*
T. BY - R.K. KANETKAR 9.1.52



REF:-
 O..... ALIGNMENT I
 Δ X..... ALIGNMENT II

AFFLUX FOR DIFFERENT FLOOD DISCHARGES
WEIR ALIGNMENTS I AND II.

FIG. 3
KAKRAPAR WEIR MODEL I
1/100 S. S. COMPOSITE MODEL
OF
WEIR AND RIVER.

COMP. BY:- *Mang*
 TRACED BY:- PARNI.
 5-1-1952

TABLE 1

Upstream R.L.	Area (Sq. ft)	Q (lakh cusecs)	V_m (ft./sec)	$V_m^2/2g=h_v$ (ft.):
182	1,06,700	14.2	13.3	2.75
180	1,02,300	13.3	12.9	2.60
175	91,300	9.5	10.4	1.60
170	80,300	6.4	7.9	0.95
165	69,600	3.6	5.2	0.40
160	59,000	1.5	2.55	0.10

The coefficients as obtained from type IV weir section (the deepest section) for different heads were obtained from the sectional model. Since the discharging capacity of type IV would naturally be larger than the shallower types I, II and III, the effective length of the inclined flanks of the weir is best expressed as a percentage P in term of a comparison between the following—

Compare—

- (i) The central reach of 1,006 ft., and
- (ii) the flanks totalling 3,207 ft as placed actually in the straight reach ,
with
- (i) central reach of 1,006 ft. of type IV section, and
- (ii) 3,207 ft. × P per cent of additional straight reach assumed to be of the same discharging capacity as type IV

The percentage effective length is expressed as follows—

$$P \text{ per cent} = 100 \times \frac{Q - C \times 1,006 \times (D_1 + h_v)^{3/2}}{C \times (D_1 + h_v)^{3/2} \times 3,207}$$

where Q = total discharge,

C = discharge coefficient as obtained from type IV.

The percentages are given in Table 2 and Figure 4.

TABLE 2

Upstream R.L.	Q (lakh cusecs)	C (Type IV)	h_v	P (per cent)
181.9	14.2	3.84	2.72	41
181.5	13.85	3.89	2.7	39
176.9	10.1	3.87	1.8	39
171.8	7.0	3.72	1.1	46
165.2	4.0	3.40	0.5	74
160.9	1.84	3.17	0.12	90

It will be seen that the percentage effective length on the basis explained is 40 per cent for the higher floods, rising to higher values as the flood level goes down. At small floods, the effective length is much nearer unity, i.e., as free flow conditions are approached. The high angularity of approach and the very shallow flank weirs due to which the flow to a large extent was far from normal to the flank resulted in the great reduction in the effectiveness at higher floods.

As explained above, the 40 per cent effective length at high floods is on the basis of a comparison of the flank weirs which are on shallow foundations with the

type IV section which is on the deep section of the river. When the lesser coefficients of discharge over shallow weir sections are taken account of, the effective length would be slightly more than 40 per cent. From the point of view of overall efficiency of the weir, it is not of much interest to go into this point any further.

Location of Undersluices—Right Flank

Due to the very favourable curvature induced by the already existing small channel on the right flank, the location of the undersluice at the downstream end of this channel and the canal offtake on the outer side of this bend was quite satisfactory, as the bed material was deflected away from the regulator and flowed into the river through and near the undersluices—*vide* Figures 5 (a) and (b).

Location of the Undersluices—Left Flank

On the left flank, there is a long approach channel to the regulator, running parallel to the flank weir, and the undersluices are located near the entry to this channel, and on the left flank bend (Figure 1) while the undersluices are placed at a fairly favourable location for the bed flow, the flow at the end of the approach channel just upstream of the left regulator became rather slack. This was conducive to silting due to the settlement of the finer sediment, though bed sand was likely to be excluded to some extent. This high angle for the flank weir was thus entirely incompatible with a satisfactory sand exclusion. Tests were therefore directed towards a straighter reach of the weir alignment II, keeping in view that the upstream maximum afflux should not exceed the design R.L. 182. Further tests on these lines are in progress, consequent on the subsequent proposed decision to take only one canal carrying 4,000 cusecs from the left bank.

Studies on Different Weir Sections of the Composite Weir

The discharge characteristics of the section of the composite weir of different types (Figure 6) were obtained in sectional models built to G. S. scale 1.40, under normal flow conditions. It was difficult to apply these results directly to the inclined flanks of the weir with any certainty since in the actual composite model, the flow is at a very much acute angle to the flanks, and almost parallel to the weir for a long reach. Since the effective length of the inclined reaches was low partly due to too acute an angle and partly due to the shallow approach over the type I and type II sections, these results from the sectional models were helpful in giving a correct idea of the effective length of the inclined reaches of the weir, as well as the scouring tendencies.

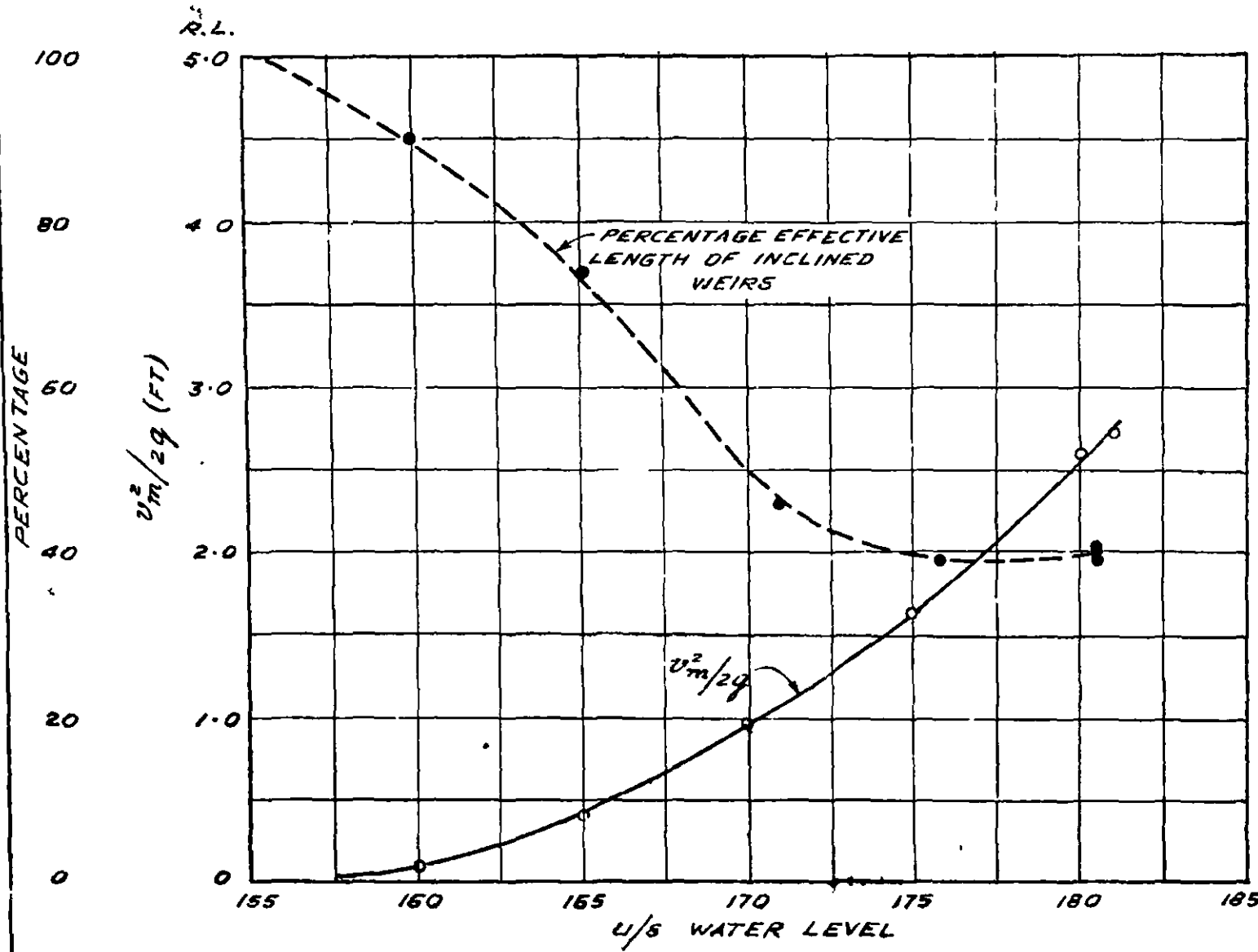
The tests were made with different upstream and downstream water levels, the maximum upstream water level R.L. 182 corresponding to the maximum downstream R.L. 169. Due to the high afflux the discharge coefficients were sufficiently independent of the tail water.

The pressures (Figure 7) were fairly satisfactory for type IV section, being nearly atmospheric, and not more than about 2 ft. below atmospheric pressure. For the shallower sections types I and III [Figures 8 and 8(a)] there existed a region of negative pressure on the crest which however would not exist in the actual composite weir due to the high angularity of flow. This region of negative pressure on the shallowest section however can be avoided by incorporating a flatter curvature on the crest.

The discharge coefficients are shown in the Table 3 and Figure 9. From the point of view of designed afflux, the discharge coefficients as obtained were satisfactory.

TABLE 3—Discharge coefficients

	U/s W. L.	D/s W. L.	$V^2/2g$ (ft.)	Total Head (ft.)	q (cu./ft.)	C
Type I	182.04	169.0	8.80	35.85	762	3.55
	181.96	169.0	8.32	35.3	739	3.52
	175.00	160.0	6.05	26.05	494	3.70
	170.2	154.8	3.62	18.8	309	3.70
	165.0	149.8	1.53	11.5	148.5	3.81
Type III	182.2	169	2.39	29.55	625	3.89
	179.95	166	1.98	26.95	544	3.89
	175.1	159.8	1.33	21.40	400	4.04
	170.2	154.8	0.62	15.80	241	3.85
	165.1	149.8	0.20	10.30	118.5	3.60



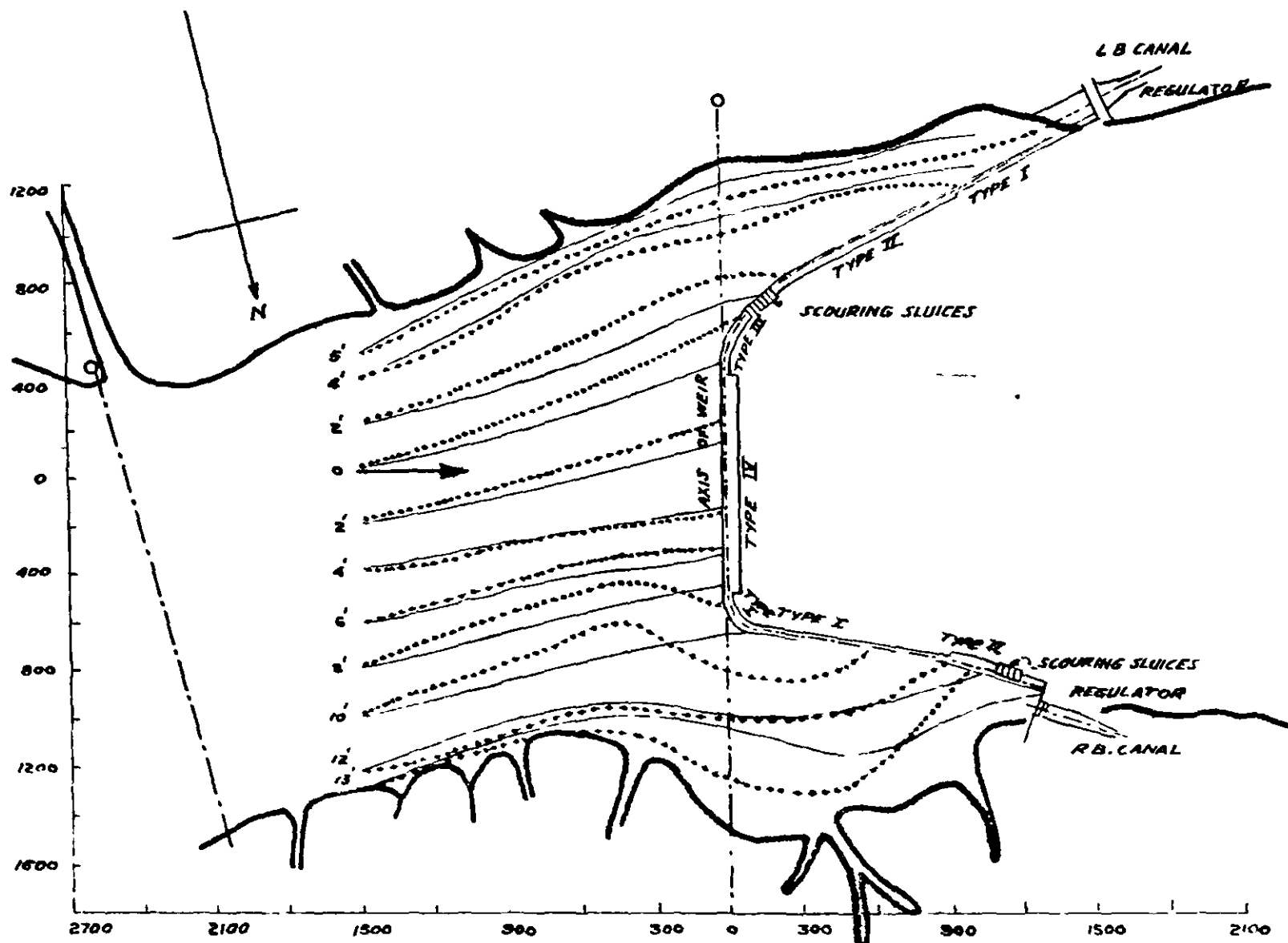
MEAN VELOCITY HEAD FOR -
DIFFERENT U/S WATER LEVELS
AND PERCENTAGE EFFECTIVE
LENGTH OF INCLINED FLANKS OF
WEIR ALIGNMENT I.

REF:-

- MEAN VELOCITY HEAD
- PERCENTAGE EFFECTIVENESS

COMP. BY: *Handwritten Signature*
TRACED BY: PARKHI.
7-1-1952

FIG. 4
KAKRAPAR WEIR MODEL I
1:100 G. S. COMPOSITE MODEL
OF
WEIR AND RIVER



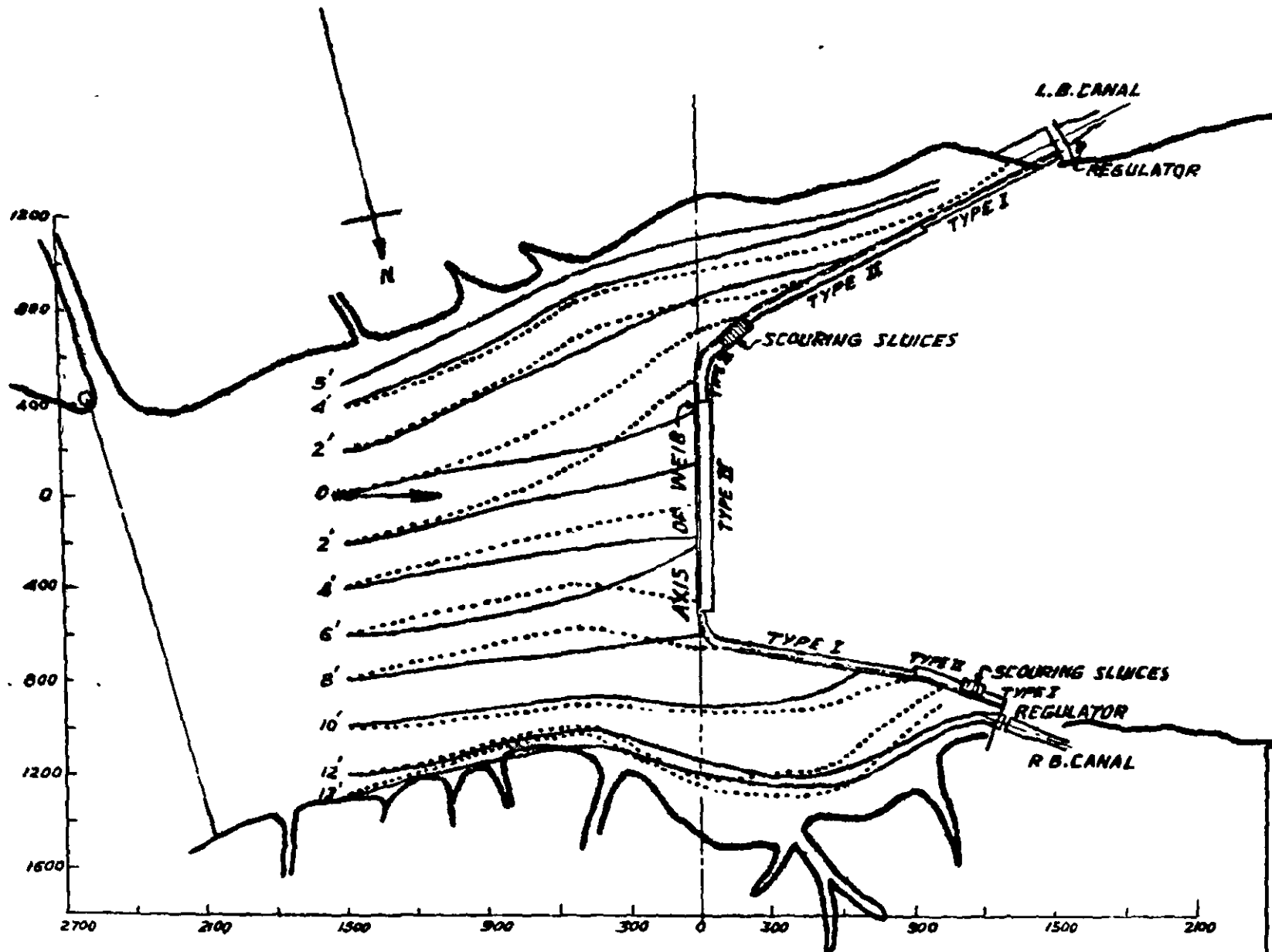
WEIR ALIGNMENT I
LINES OF FLOW FOR U/S R.L-182
 SURFACE FLOW _____
 BED FLOW

NOTE:-

- (i) VERY FAVOURABLE CURVATURE AT R.B. REGULATOR
- (ii) UNFAVOURABLE CURVATURE FOR FLOW NEAR L.B. REGULATOR.
- (iii) LONG STRAIGHT APPROACH TO L.B. REGULATOR WITH SLACK WATER NEAR L.B. REGULATOR.

FIG: 5(a)
 KAKRAPAR WEIR MODEL I
 1/100 G.S. COMPOSITE MODEL OF
 WEIR AND RIVER
 SHOWING
 LINES OF FLOW
 SCALE:- 660'-1"

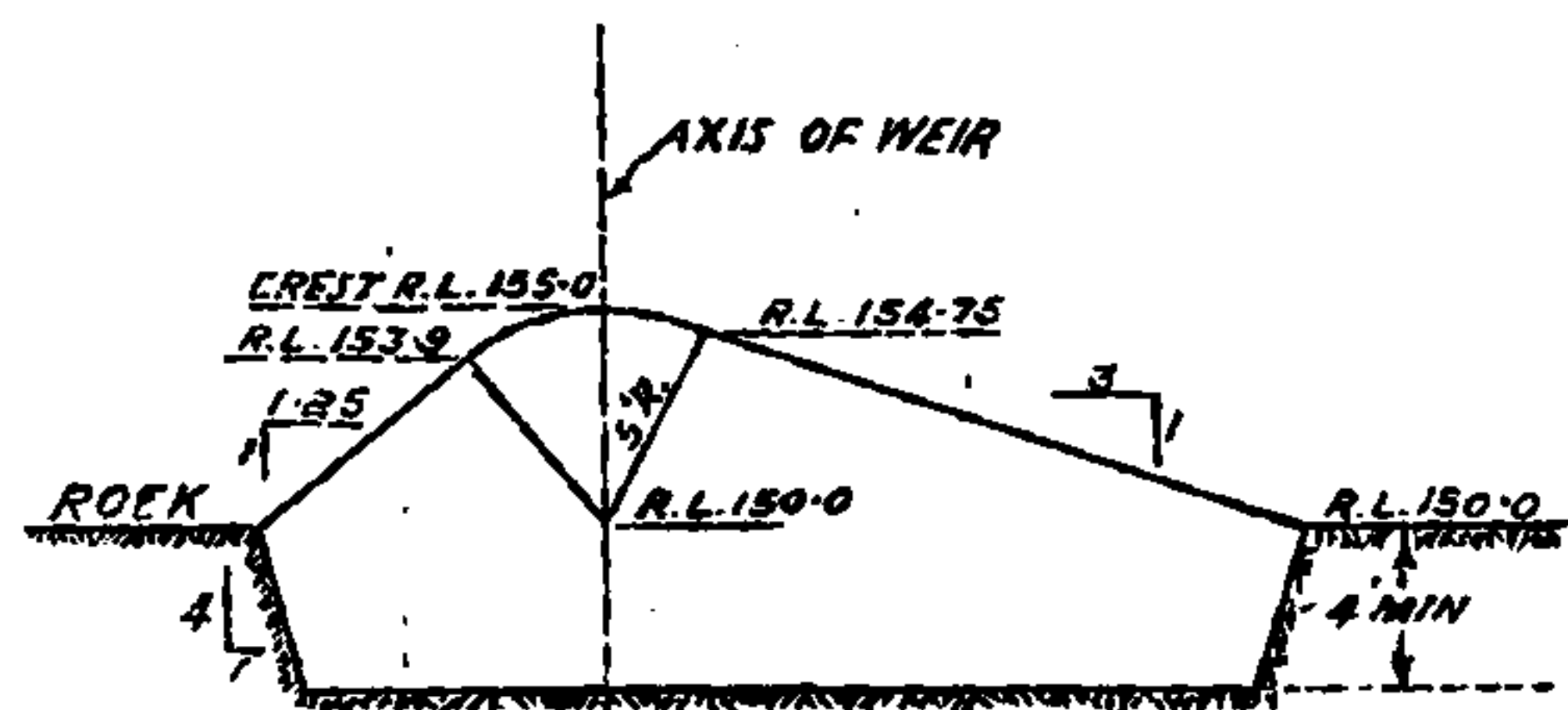
COMP BY - *[Signature]*
 TRACED BY - S A MANCUNDIA.



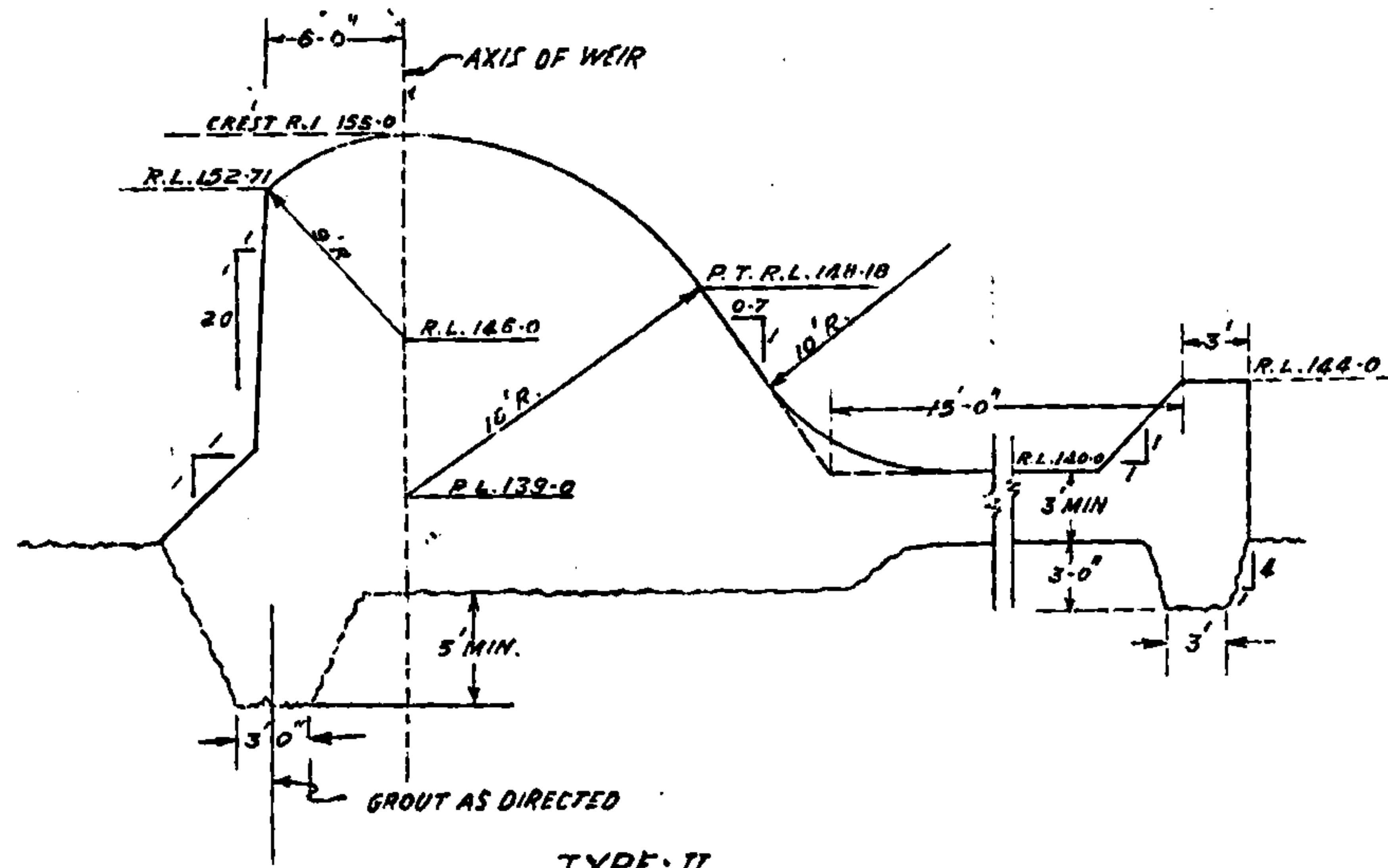
WEIR ALIGNMENT I
 LINES OF FLOW FOR U/S R.L. 170
 SURFACE FLOW _____
 BED FLOW _____

FIG. 5 (b)
 KAKRAPAR WEIR MODEL 1
 1/100 G.S. COMPOSITE MODEL OF
 WEIR AND RIVER
 SHOWING
 LINES OF FLOW
 SCALE 660' = 1"

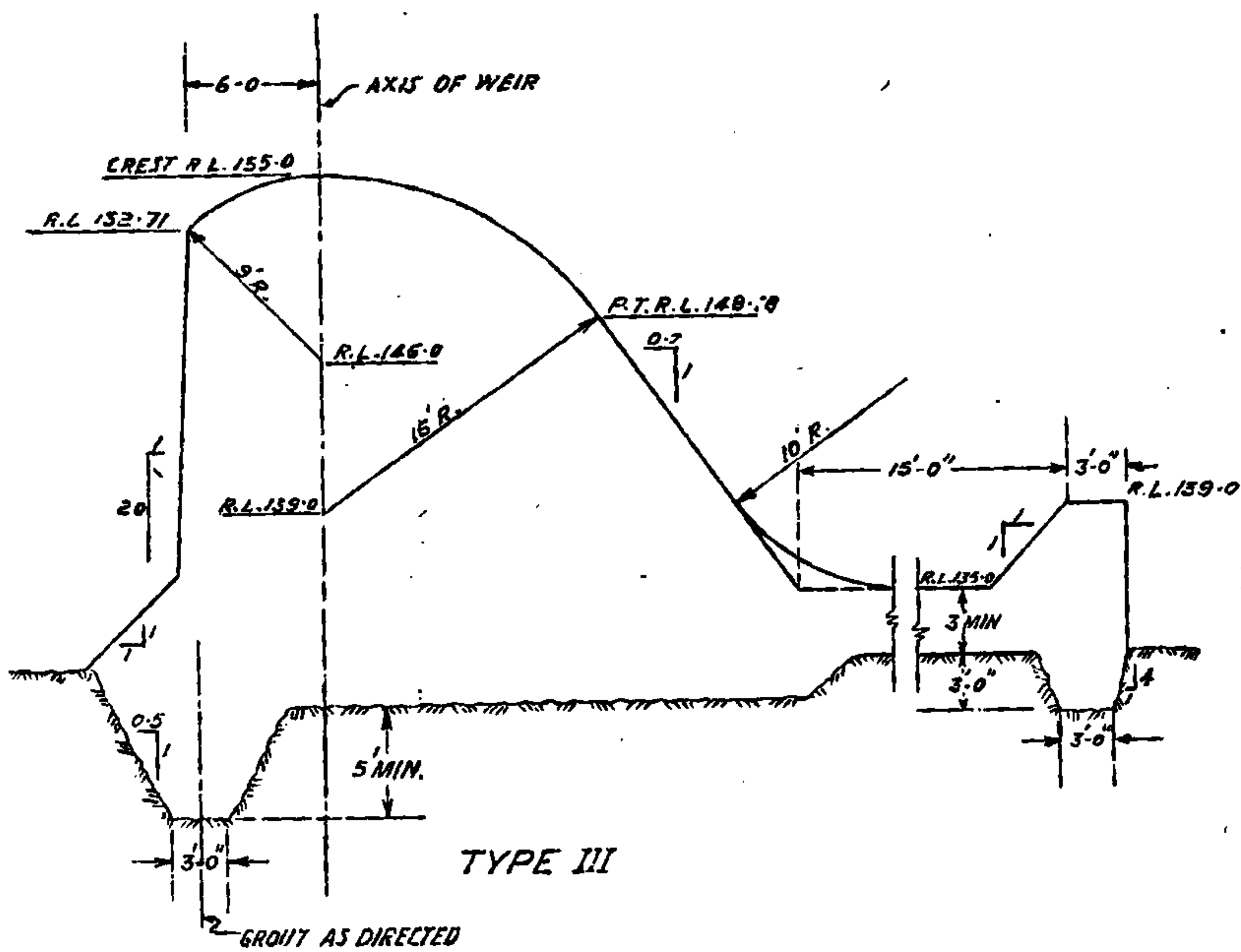
COMPD. BY: *M. S. Deshpande*
 TRACED BY: B. G. DESHPANDE 9-1-52



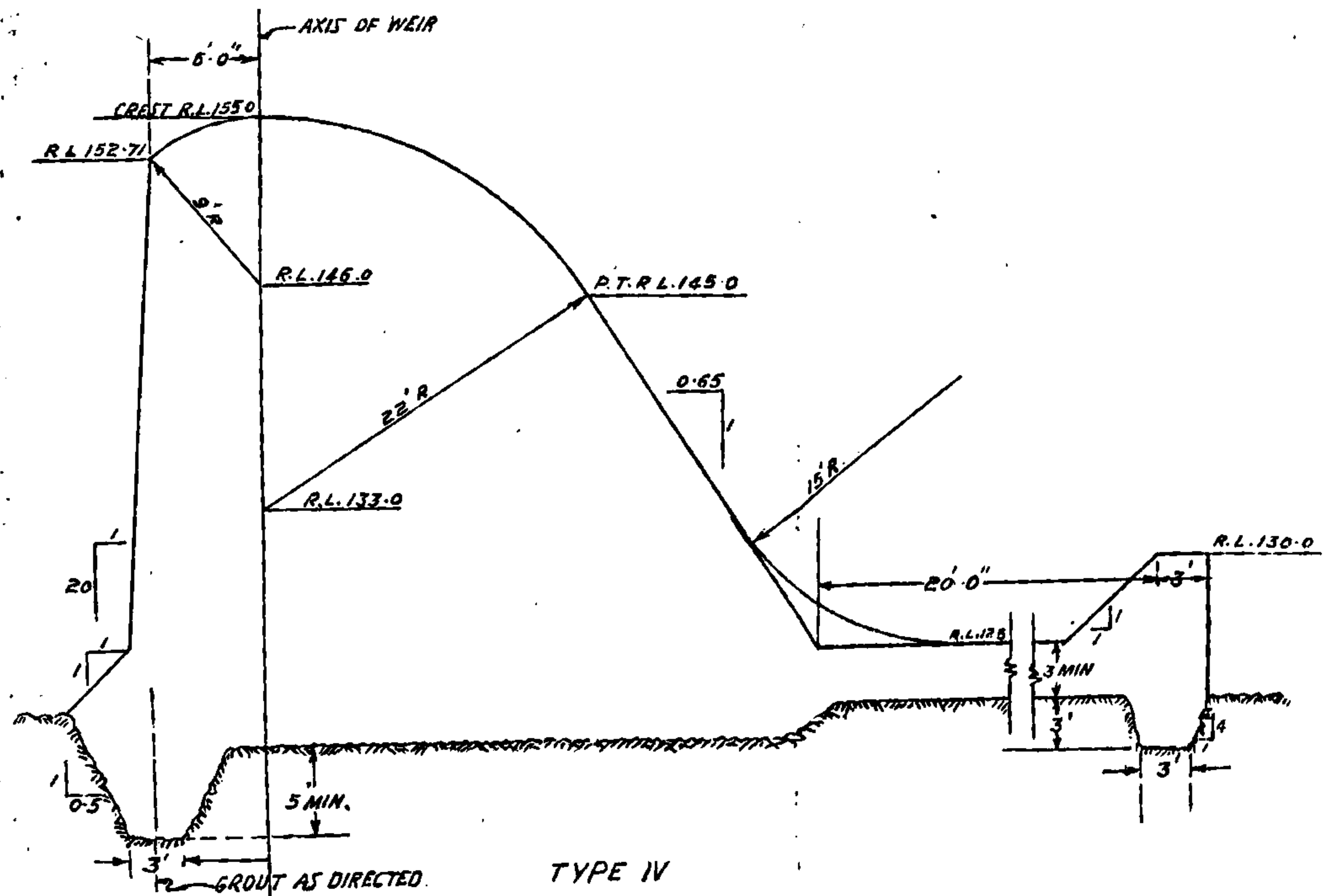
TYPE I



TYPE II



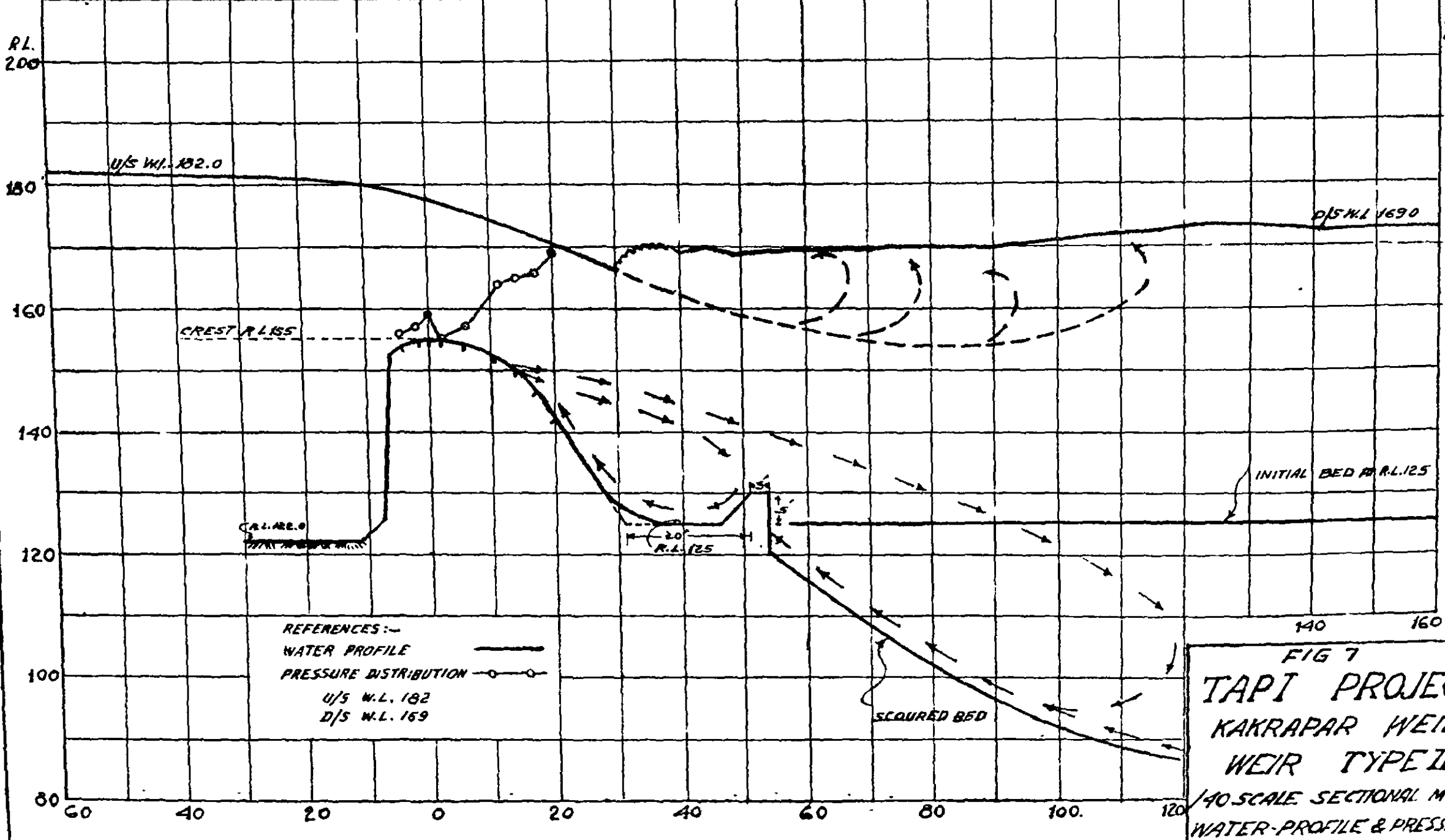
TYPE III



TYPE IV

FIG. 6
TAPI PROJECT
KAKRAPAR WEIR SECTIONS
SCALE 1"=10FT.

COMPARED BY: *Manoj*
TRACED BY: M.K. JABNIS. 9-1-51.



REFERENCES :-
 WATER PROFILE ———
 PRESSURE DISTRIBUTION ○—○
 U/S W.L. 182
 D/S W.L. 169

FIG 7
 TAPI PROJECT
 KAKRAPAR WEIR
 WEIR TYPE II
 1/40 SCALE SECTIONAL MODEL
 WATER-PROFILE & PRESSURES

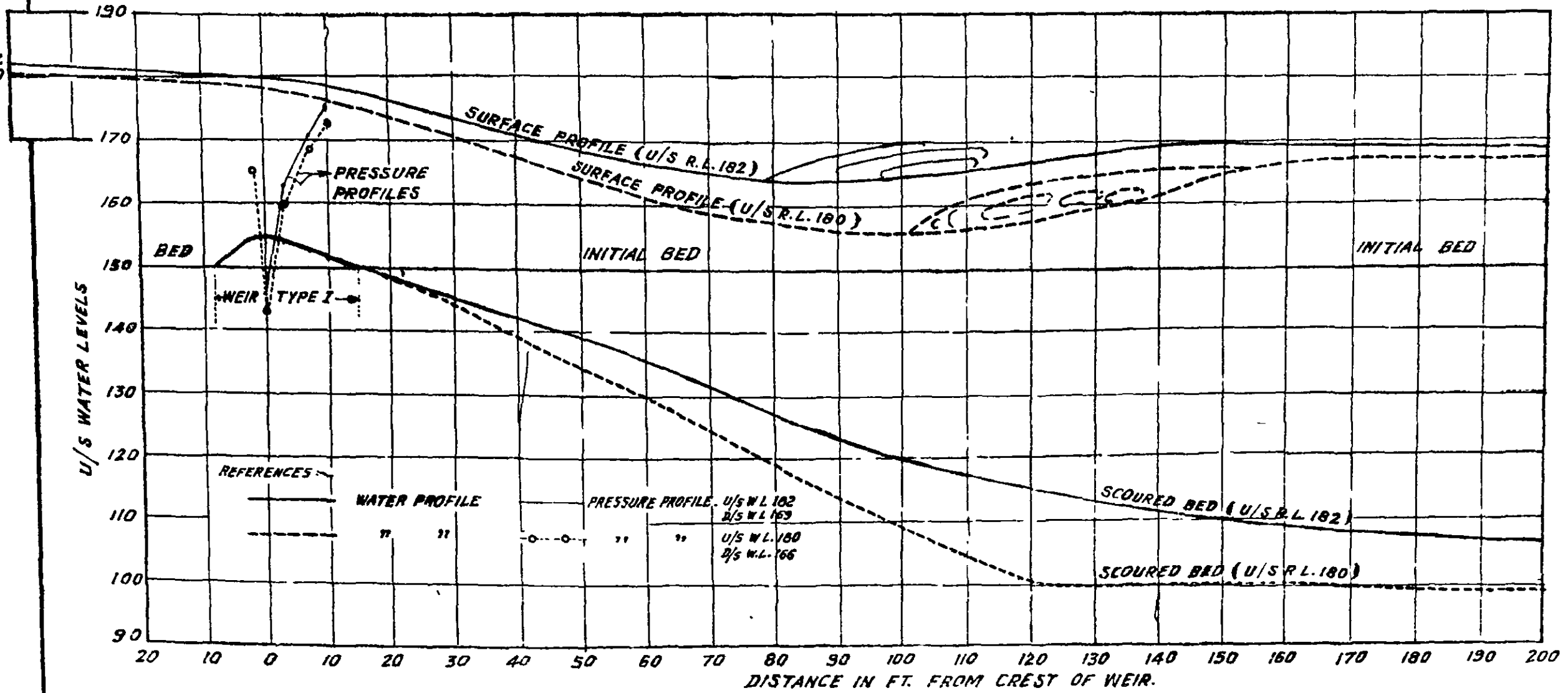
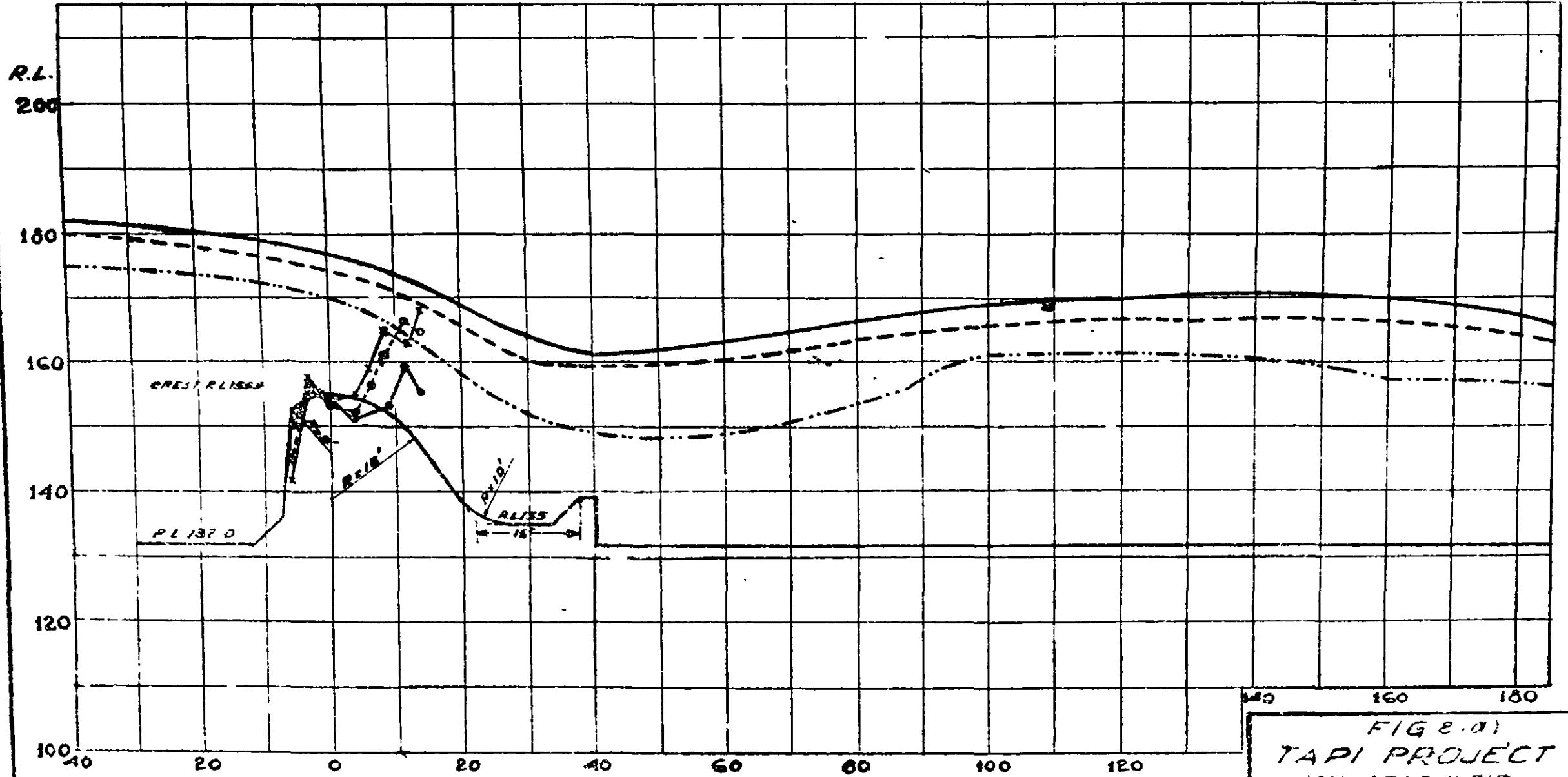


FIG. 8
TAPI PROJECT-KAKRAPAR WEIR
WEIR TYPE I
1/40 SCALE SECTIONAL MODEL.

COMP. *[Signature]*
 TRACED BY SARKAR 9-1-52.



REF: ——— WATER PROFILE x-x-x-x-x PRESSURE DISTRIBUTION U/S WL 152 D/S WL 163
 - - - - - WATER PROFILE o-o-o-o-o PRESSURE DISTRIBUTION U/S WL 150 D/S WL 156
 WATER PROFILE ——— PRESSURE DISTRIBUTION U/S WL 175 D/S WL 160

FIG 2.01
 TAPI PROJECT
 KAKKAPAR WEIR
 WEIR TYPE III
 1/40 SCALE SECTIONAL
 WATER-PROFILE & PRESSURE DISTRIBUTION

TRACED BY TAKALKAR 15-12-51

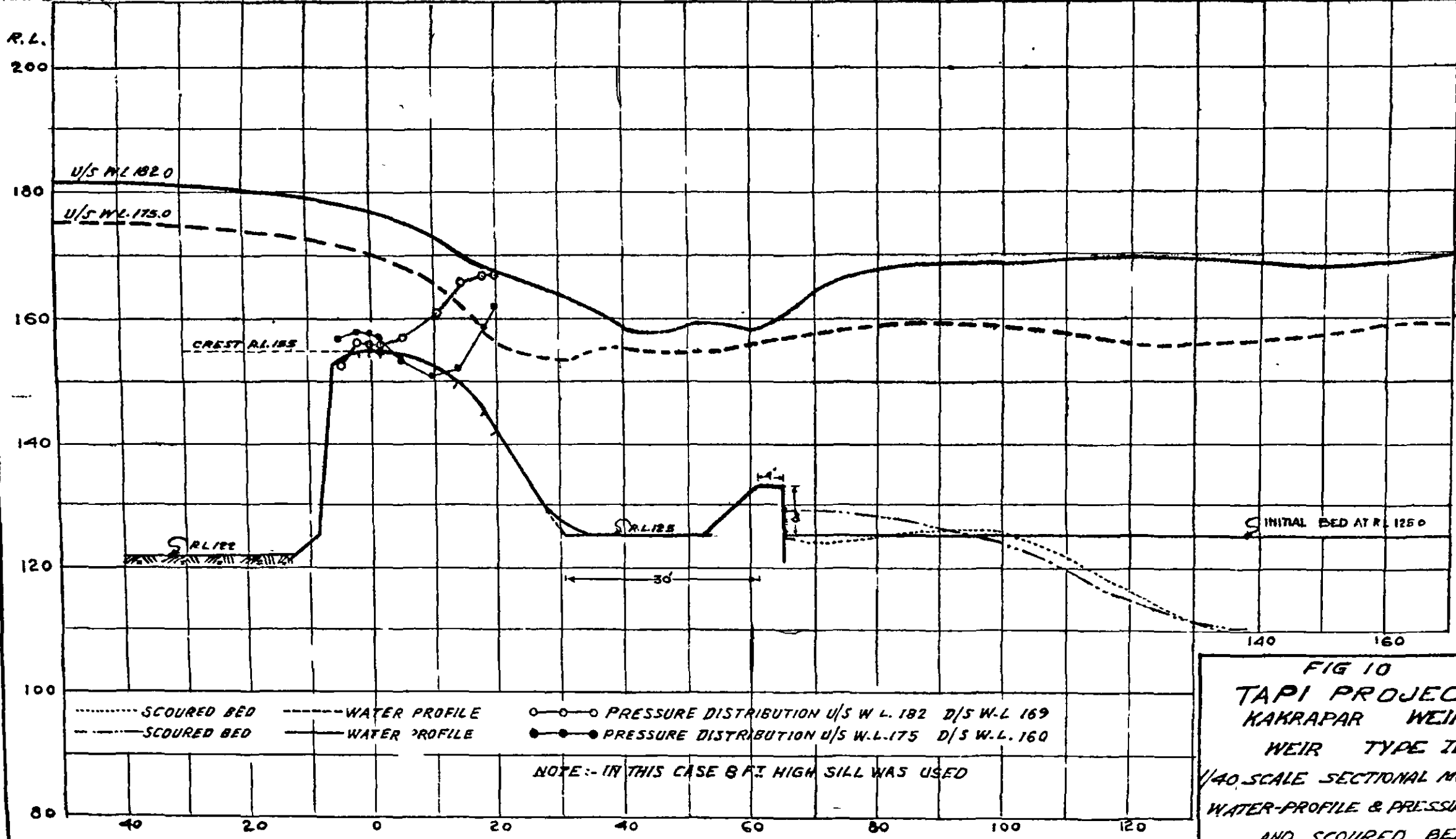
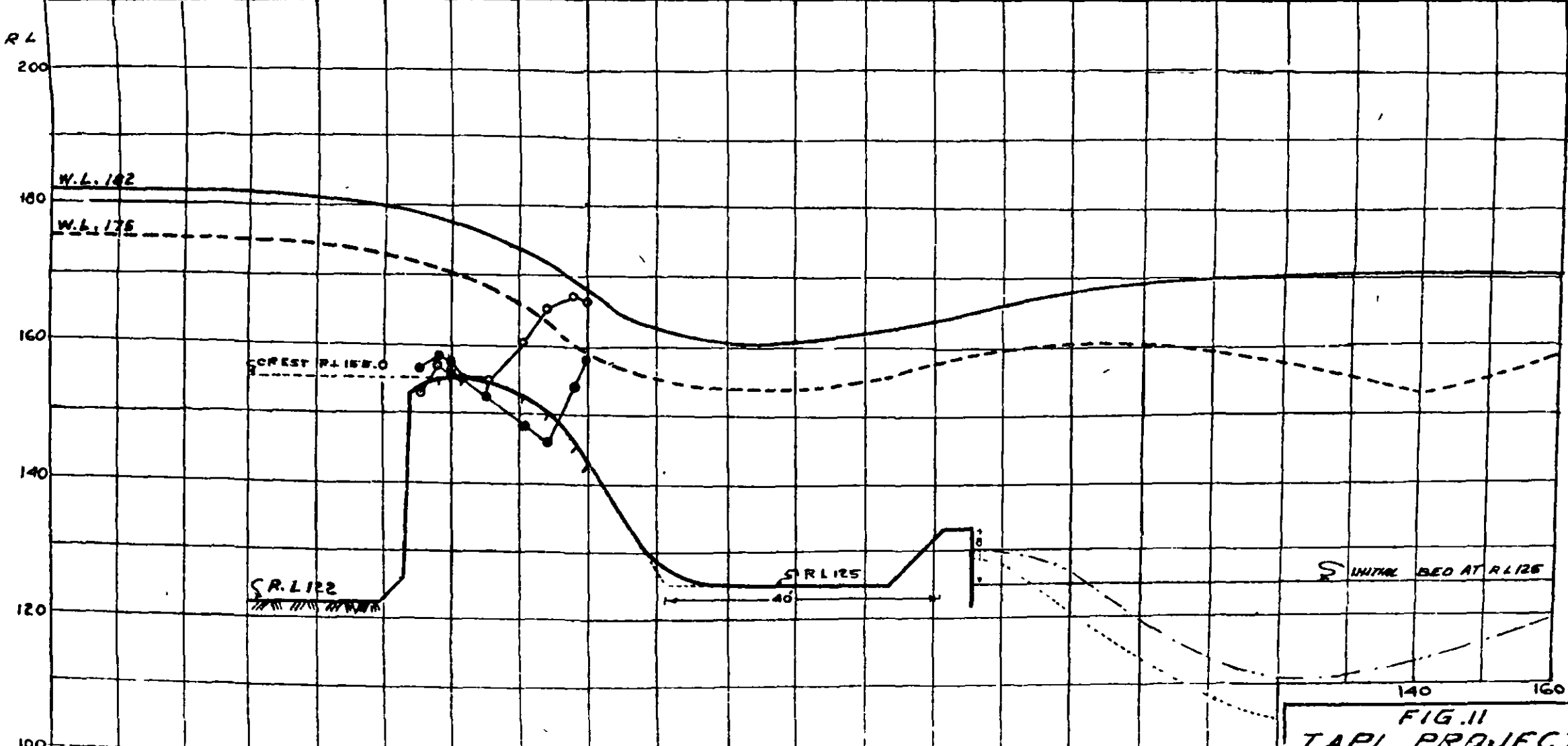


FIG 10
TAPI PROJECT
KAKRAPAR WEIR
WEIR TYPE IV
1/40 SCALE SECTIONAL MODEL
WATER-PROFILE & PRESSURES
AND SCOURED BED
30 FT. APRON

TRACED BY NDT 4-12-51



- - - SCOURED BED ——— WATER PROFILE ○—○—○ PRESSURE DISTRIBUTION U/S W.L. 182.0 D/S W.L. 169
 - - - SCOURED BED - - - WATER PROFILE ●—●—● PRESSURE DISTRIBUTION U/S W.L. 175.0 D/S W.L. 160
 NOTE - IN THIS CASE 8 FT HIGH SILL WAS USED

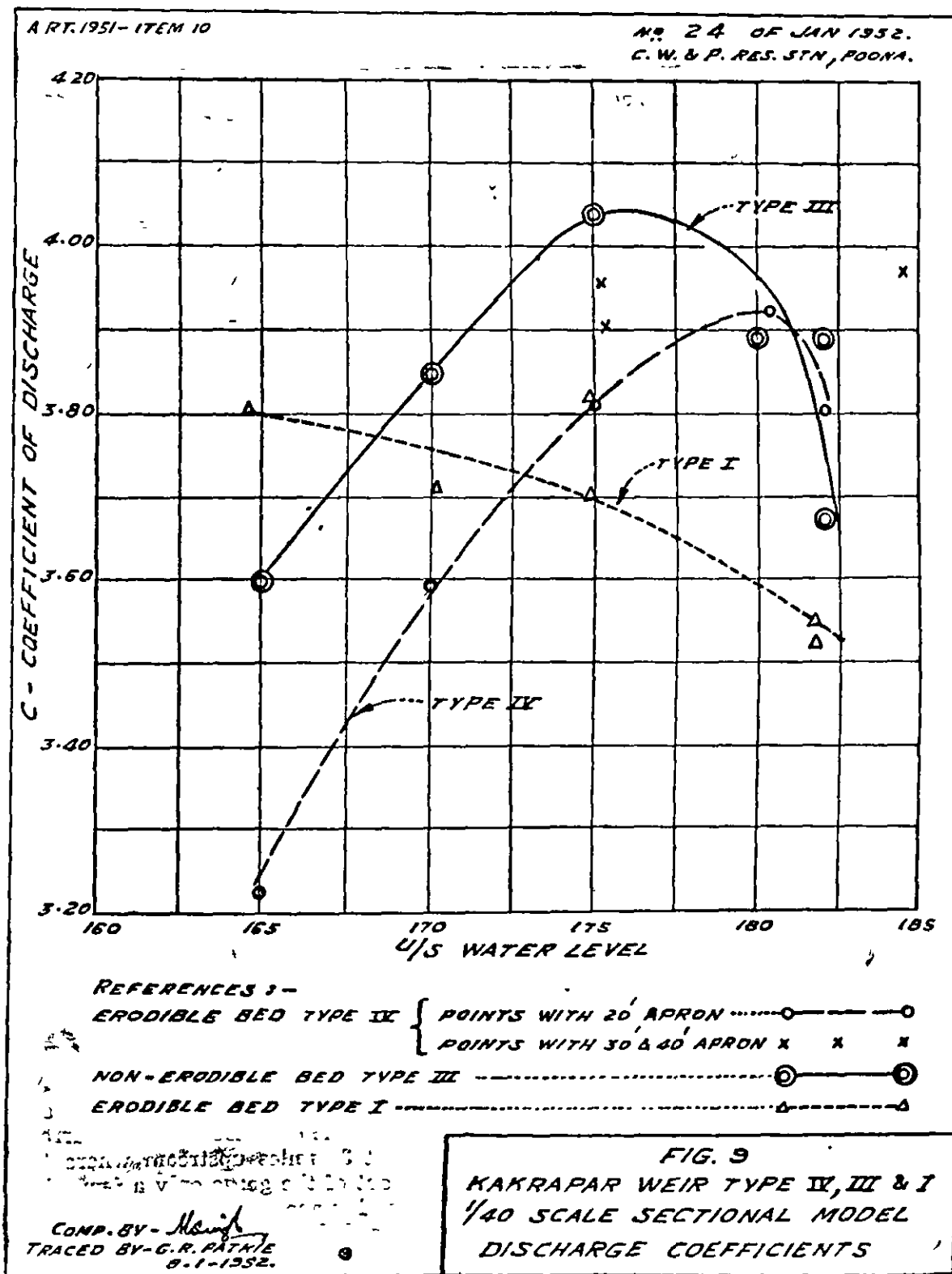
FIG. II
TAPI PROJECT
KAKRAPAR WEIR
WEIR TYPE II
 1/40 SCALE SECTIONAL MODEL.
 WATER PROFILE, PRESSURES
 AND SCOURED BED
 40 FT APRON

TABLE 3—Discharge coefficients—contd.

	U/s W. L.	D/s W. L.	$V^2/2g$ (ft.)	Total Head (ft.)	q (cu ft.)	C
Type IV	182.1	169	1.44	28.55	580	3.80
	180.45	166.3	1.32	26.75	537	3.87
	174.95	159.8	0.72	20.70	358	3.51
	170.1	154.8	0.32	15.45	215	3.59
	165.0	155	0.07	10.1	104	3.22

The 20 ft. apron length as designed was found rather short. Better results were obtained with 30 and 40 ft. long aprons (Figures 10 and 11). In the prototype however because of the hard rock, it has been proposed that the construction of the end sill would be omitted at first keeping the shorter length of apron, and the actual resistance of the bed rock under this condition in the prototype could then be observed during the floods before making the final decision on putting the end sill.

Figs. 10 & 11.



11. DISCHARGE CHARACTERISTICS OF THE DESIGN OF PROPOSED KOTAH BARRAGE ON THE CHAMBAL

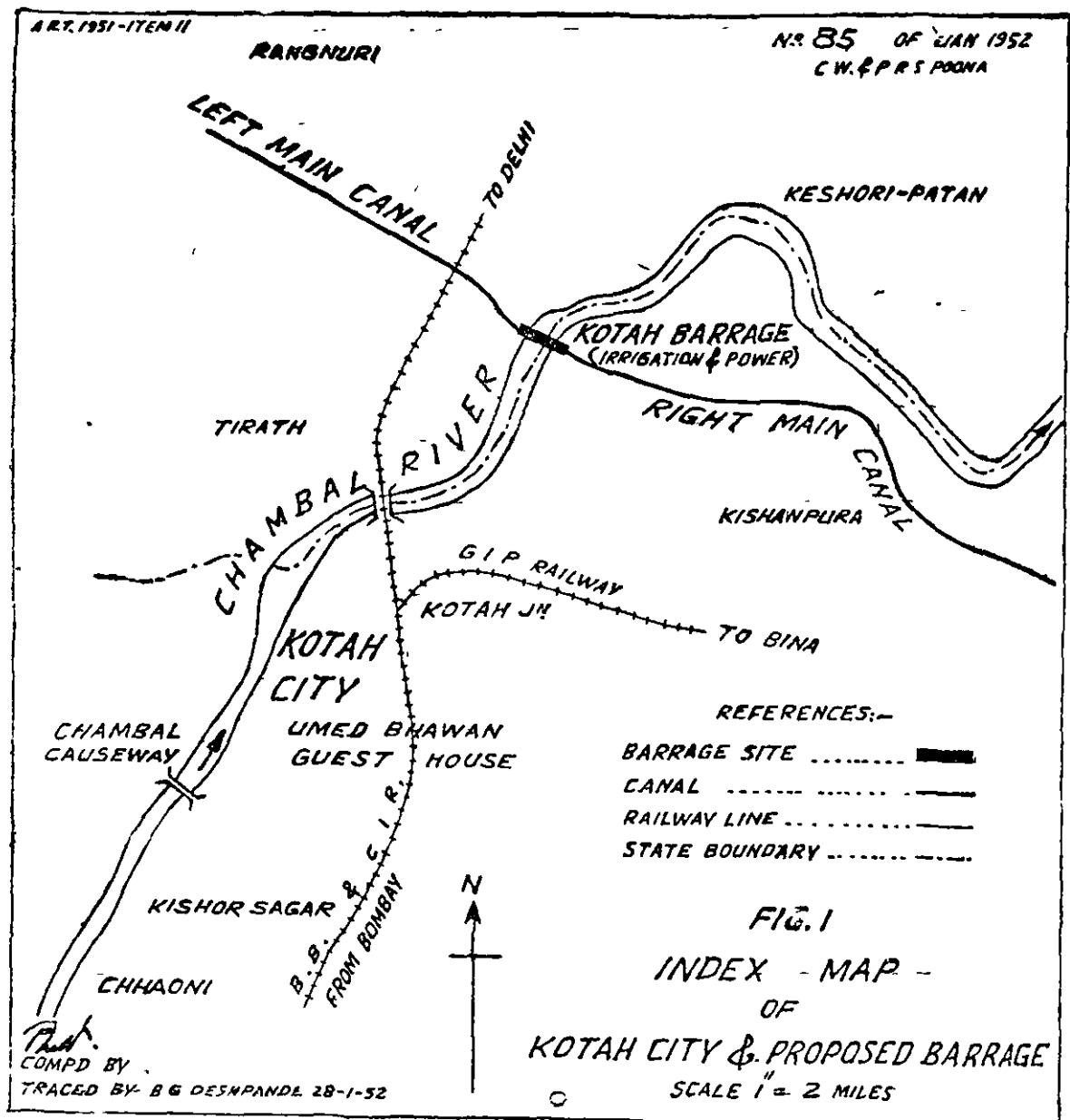
The Chambal valley development is envisaged in three stages, viz.:—

- (1) Barrage on the Chambal river near Kotah (Rajasthan),
- (2) Gandhisagar Dam (Madhya Bharat), and
- (3) a dam at Rawatbhata (Rajasthan).

Study of certain features on the 1st stage of the Project, viz., the barrage was referred to C. W and P. Research Station, Poona in October 1951. This note presents results obtained with the sectional model of the barrage built to 1/40 geometrically-similar scale.

Site of Barrage

The Kotah Barrage site is about 8 miles downstream of the Kotah City (Figure 1).



The L-profile of the bed of the river shows that deep pools in the river are interspaced with barriers which mainly consist of exposed rock. The barrage site is located at one such barrier—vide Photos 823, 824, 825 and 828. Between the barrage and the city, another barrier exists which is about $3\frac{1}{2}$ miles upstream where the railway bridge is located. As the river has come out of the gorge only a few miles upstream of Kotah, the slope of the river is steep and is about $1\frac{1}{2}$ ft. per mile. With the maximum water level upstream of the barrage at R.L. 818 it has been calculated that back-water effect would extend up to the railway bridge.

Chambal River at the Proposed Barrage Site near Kotah
(Rajasthan)



Photo Nos. 824, 825
Looking across the Chambal River towards the Left
Bank Just above Kotah Barrage Site.



Photo No 823
River bed at Barrage Site looking
towards left bank.



Photo No. 828
River bed in the vicinity of the proposed
Barrage is strewn with big rocks.

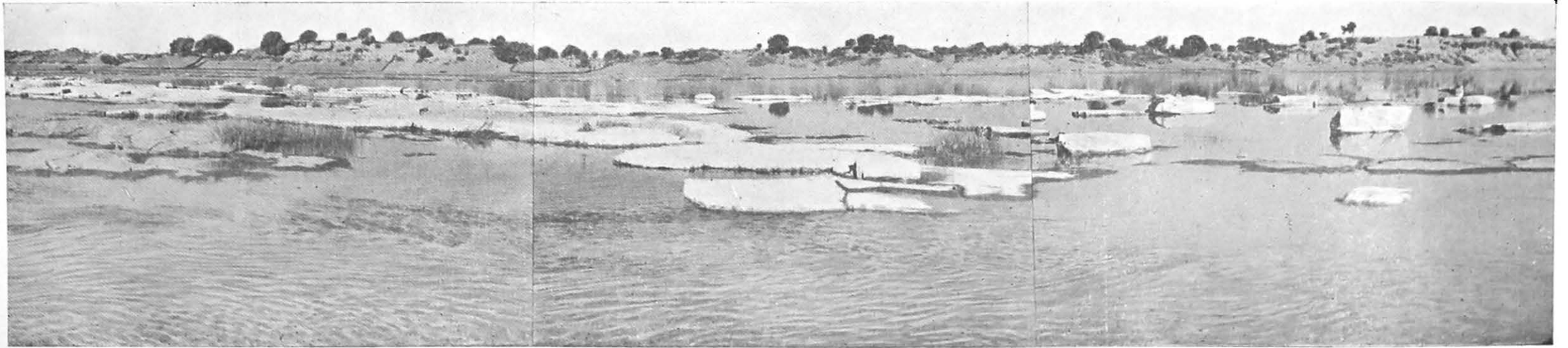


Photo Nos. 829, 830, 831.

View of the river reach at the Barrage Site. Photo looking
towards left bank. Note the bank which is sloping at
3 : 1 on the left changes to steeper slope
towards the right due to change in
the river curvature.

Salient Features of the Barrage

The catchment area of the river above the proposed barrage site is 10,670 square miles. The highest water level in the river taken is R.L. 815. With a maximum permissible afflux of 3 ft. the maximum upstream water level assumed in design is R.L. 818. The proposed barrage consists of 31 flumed vents 50 ft. \times 27 ft. separated by 10 ft. wide piers. The total length of the barrage is 2,098 ft. (including abutments). The vents are flumed at top by providing a bellmouth with its top transition, starting from E. L. 818 and gradually coming down to E. L. 806 about 8 ft. upstream of the gate. The vented design of the barrage is mainly with a view to reduce the height of gates—one of the costly items. The weir is designed to have a 19-ft. high sill at E. L. 779 with horizontal length of 20 ft. with 1:3 upstream slope and 1.6 downstream slope. The river bed is at R.L. 760. A single row of 5 ft. high stream-lined blocks is provided at a distance of 17 ft. from the downstream toe of the glacis. The *pakka* pavement is continued 50 ft. downstream of the blocks followed by 2 ft. thick rubble pitching for a length of 50 ft.

Object of the Experiments

The following points were studied in the model with a view to determine the adequacy of the design—

- (1) Afflux with maximum flood discharge and with corresponding W. L. in the river downstream of the barrage.
- (2) Discharging capacity of the barrage with different water levels upstream and downstream and the number of vents required to pass the maximum flood discharge.
- (3) Flood conditions on the downstream glacis and pavement with various gate openings.
- (4) Pressure distribution in the bellmouth entrance of the vent.
- (5) Whether the profile of the barrage weir as proposed, together with the row of blocks is adequate from the point of view of dissipation of energy of flow and elimination of scouring tendency downstream of the pavement.

The Model

The sectional model of the barrage was constructed in a 3 ft. wide and 60 ft. long flume—*vide* Figure 2. One full span and two half spans on either side were reproduced in the model. The actual width of the flume was equivalent to 119.5 ft. while the clear waterway of the weir in the model was equivalent to 99.5 ft. On the right side of the flume, a 10 ft. long glass sheet was fixed for visual observations of the flow conditions. The discharge passing through the barrage was measured over a Rehbock weir downstream. The top transition of the barrage vents was cast separately. Piezometers were located at suitable points on the top transition. The barrage gates were provided with an arrangement to obtain any pre-determined opening of the gate.

Photos 700,
804, 808, 906,
907 & 971.

Fig. 2.

The Experimental Procedure

The rating curve for the river (Figure 3) was not available before the experiments with the original design were complete. The only information available was, that, for design purposes for a maximum flood discharge of 10 lakh cusecs, a flood level of 815.0 downstream was adopted. Hence in the earlier experiments the water level upstream of the barrage was restricted to R.L. 818.0 (the designed maximum water level upstream of the barrage) and the discharging capacity of the barrage vents was observed for various water levels downstream of the barrage (corresponding to different degrees of submergence). From these observations the discharge per foot width of the barrage vent was worked out.

Fig. 3.

Similar procedure was followed with 75 per cent and 50 per cent gated flow (both with upstream water level \approx 818.0). Pressures on the transitions of the vents and water profiles were observed in almost all cases. Flow conditions on the downstream glacis and the pavement was also studied with each set of water levels.

Results

Tables 1 and 2 and Figures 4 to 6 show the results obtained.

Table 1 shows the discharge per foot width of the barrage vent with 100 per cent gate opening.

In these experiments discharges were observed with upstream water level at 818.0 and various water levels downstream of the barrage. The coefficient of discharge was 3.09 with downstream water level at 814 (drowning ratio=0.90). With downstream water level at 815.0 (the reported maximum water level in the river) and upstream water level at 818.0, the discharge of the vent was 770 cusecs/ft. The coefficient under this condition was 2.9 with the drowning ratio of 0.93. The discharging capacity rapidly decreased for water levels higher than 814.0 due to high submergence (Figure 6). Discharges obtained with upstream water level restricted to 818.0 and with 50 per cent gate openings are given in Table 2 and are plotted in Figure 5.

Figs. 9 to 11. Figures 9 to 11 show the water profiles and scour obtained with various gate openings.

Pressure Distribution in the Bellmouth Transition of the Vent

Pressure distribution was observed for two conditions—

- (1) Upstream water level 818 and downstream 815, and
- (2) upstream water level 815 and downstream 812.

Between the bellmouth entrance and the gate the pressure were all positive varying between +4.0 to 0.0 at the gate. Only just downstream of the gate slot due to separation negative pressure was observed, but it was within about -4.0 ft. (Figure 12).

Fig. 12.

Flow Conditions

Just upstream of the breast wall the water profile slightly rose towards the breast wall. At the wall itself, the extra ponding corresponded the velocity head, as usually happens with part-gate openings of sluice gates. This extra velocity head is to be taken into account in the design of the height of the breast wall.

On the downstream glacis even with 1:6 slope the flow did not follow the glacis but left the glacis at about 30 ft. downstream of the gate slot. Injected sand particles (size 1/4 inch to 3/8 inch) when caught in the return eddy rolled up the glacis from the toe. Beyond the row of blocks the flow conditions were satisfactory as the flow was forward and was deflected towards the surface.

No scour was experienced in the erodible bed downstream of the pavement under maximum water level upstream and downstream of the barrage. However with lower water levels downstream a maximum scour of 8 ft. was observed. In order to further protect the pavement end, a 3-ft. high deflector was tested; a favourable return roller was obtained; this may be adopted. At Kotah barrage site the river bed is not erodible and hence this small depth of scour obtained in the erodible bed of the model may be considered as negligible.

Discussion of Results

According to the rating curve of the river supplied later, with water level in the river at Kotah at R.L. 815.0, the corresponding flood discharge is 8.8 lakh cusecs only. It will be seen that with this water level downstream of the barrage and with 3 ft. afflux (i.e., upstream water level 818.0) a discharge of about 38,500 cusecs is obtained per vent (width 50 ft.) and only 23 vents are required to pass the flood discharge of 8.8 lakh cusecs. The barrage profile was next modified,

- (1) to observe effect of reduction of the length of the barrage,
- (2) to improve the discharging capacity of the weir if possible.

ITEM No. 11.
A. R. T. 1951.

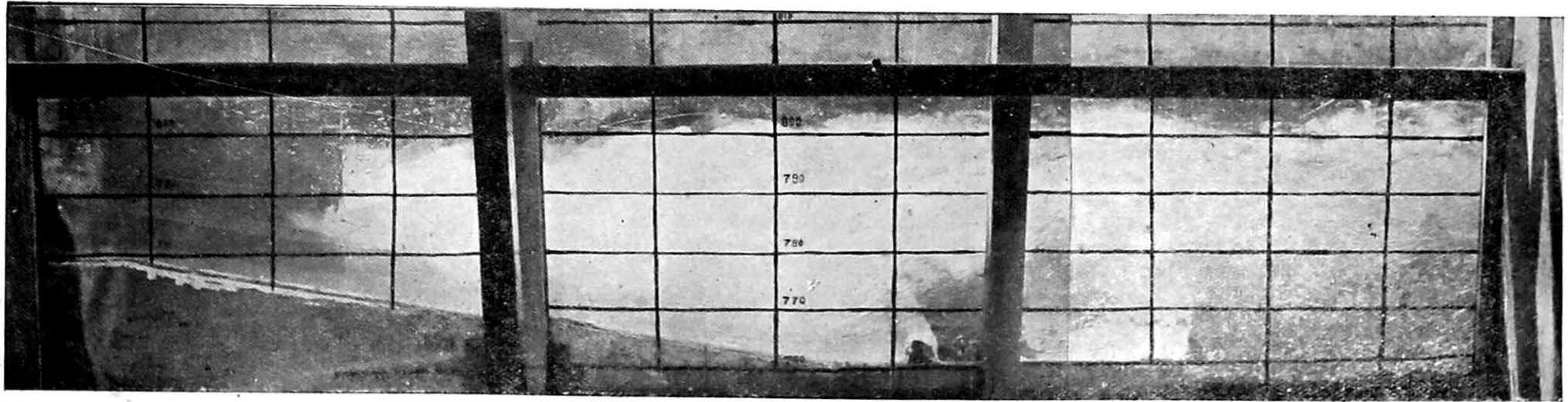


Photo Nos. 790, 791, 792

Design A. Original gated flow (50 per cent of the gate opening) U. S. W. L. = 818. 0;
D/S. W. L. = 797. 0, showing formation of standing wave on glacia and
smooth flow near bed downstream of the pavement.

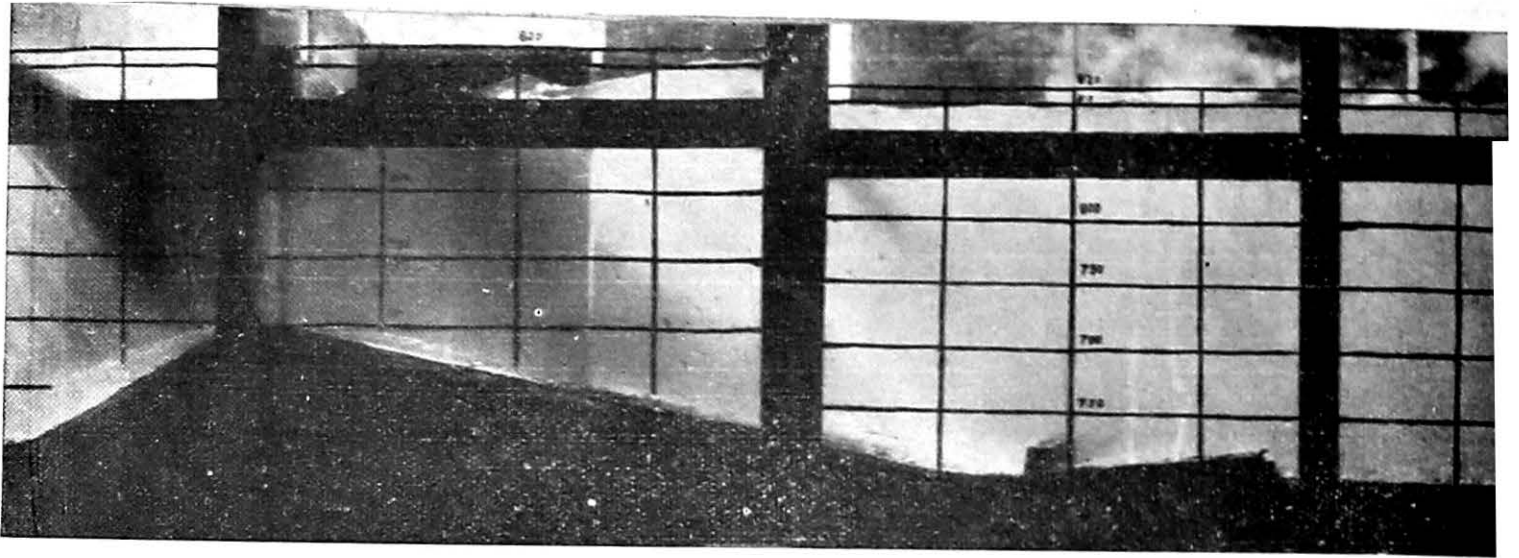


Photo No. 971 A-B
Design D. U/S W. L. = 820; D/S W. L. = 818, showing bed material rolling up the glacis near 20 feet from the gate.

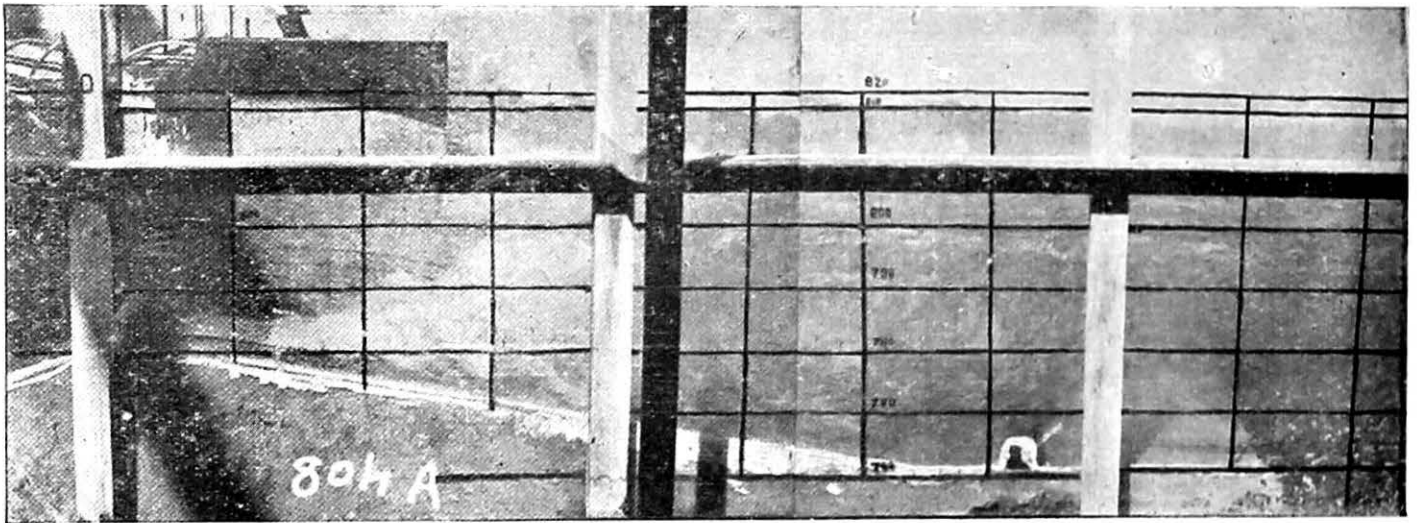


Photo Nos. 804 A. B
Design A. Original Design-Gated flow (25 per cent gate opening
U/S W. L. = 818. 0; D/S W. L. = 792. 0, shows Satisfactory flow
Downstream of the Pavement.

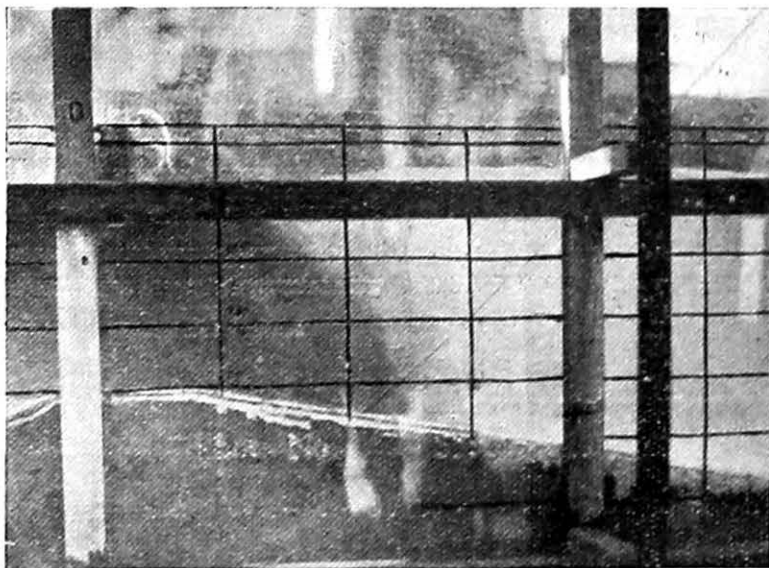


Photo No. 808
Design A. Original Design U/S W. L. = 818 0; D/S W L. = 815. 0,
showing existence of return eddy on the bed of the glacis

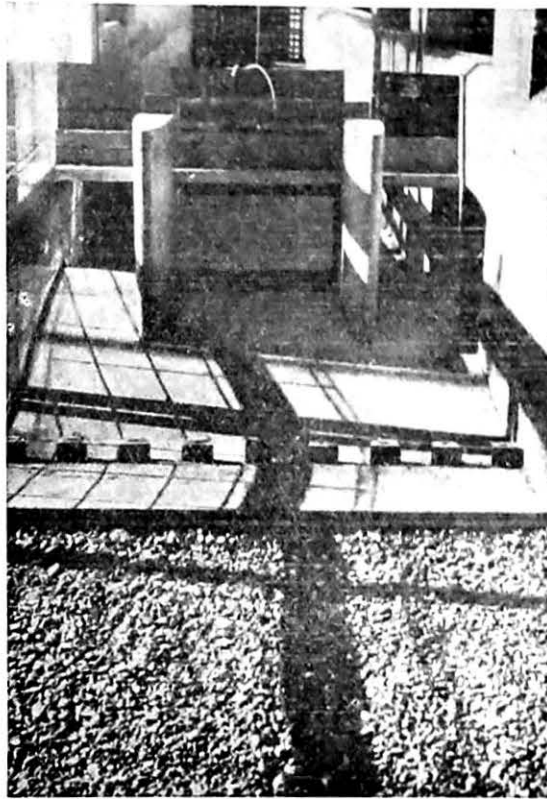


Photo No. 966
Design D showing model
looking upstream

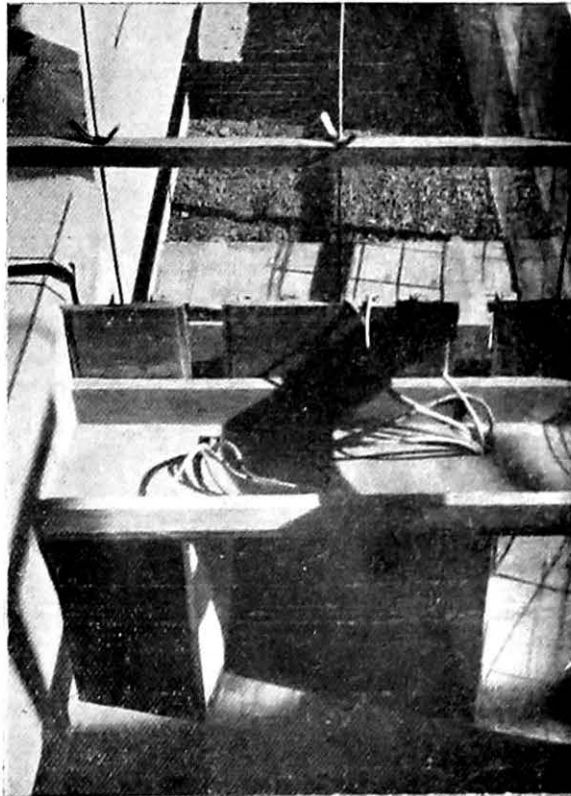


Photo No. 967
Design D - showing model looking downstream

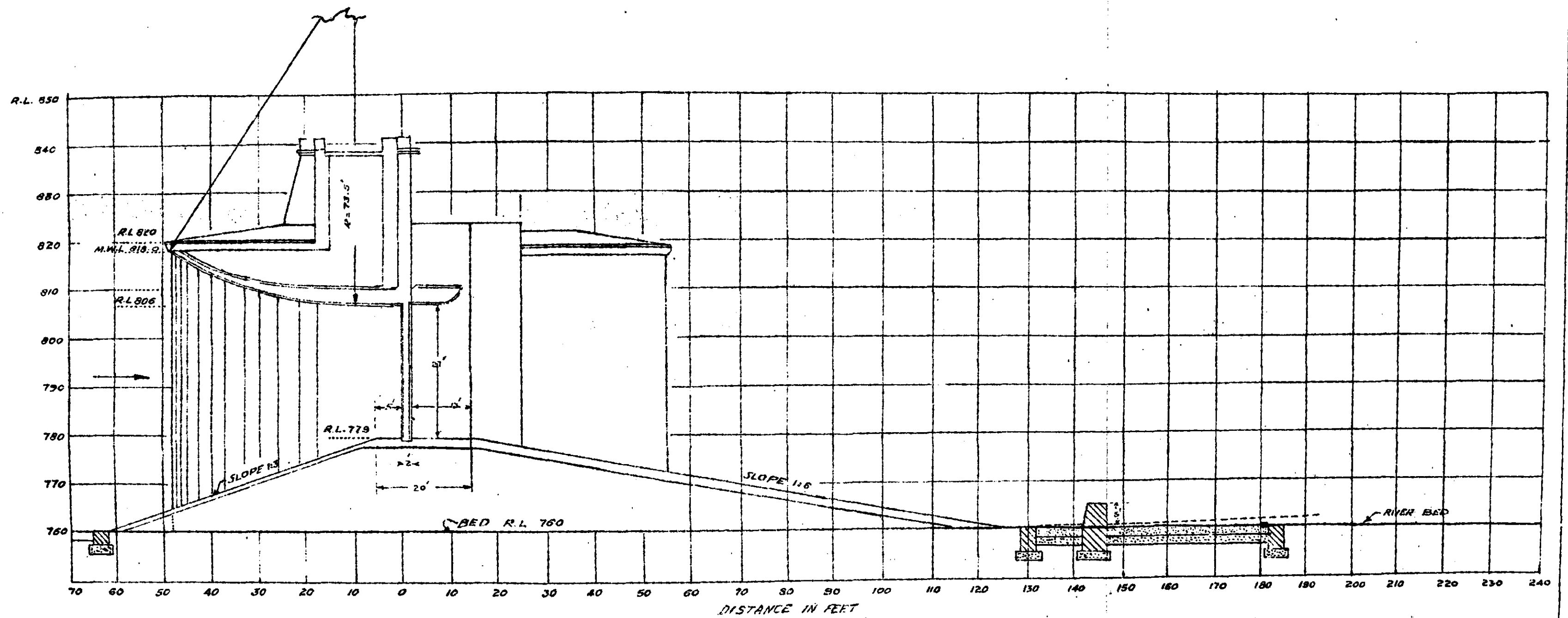
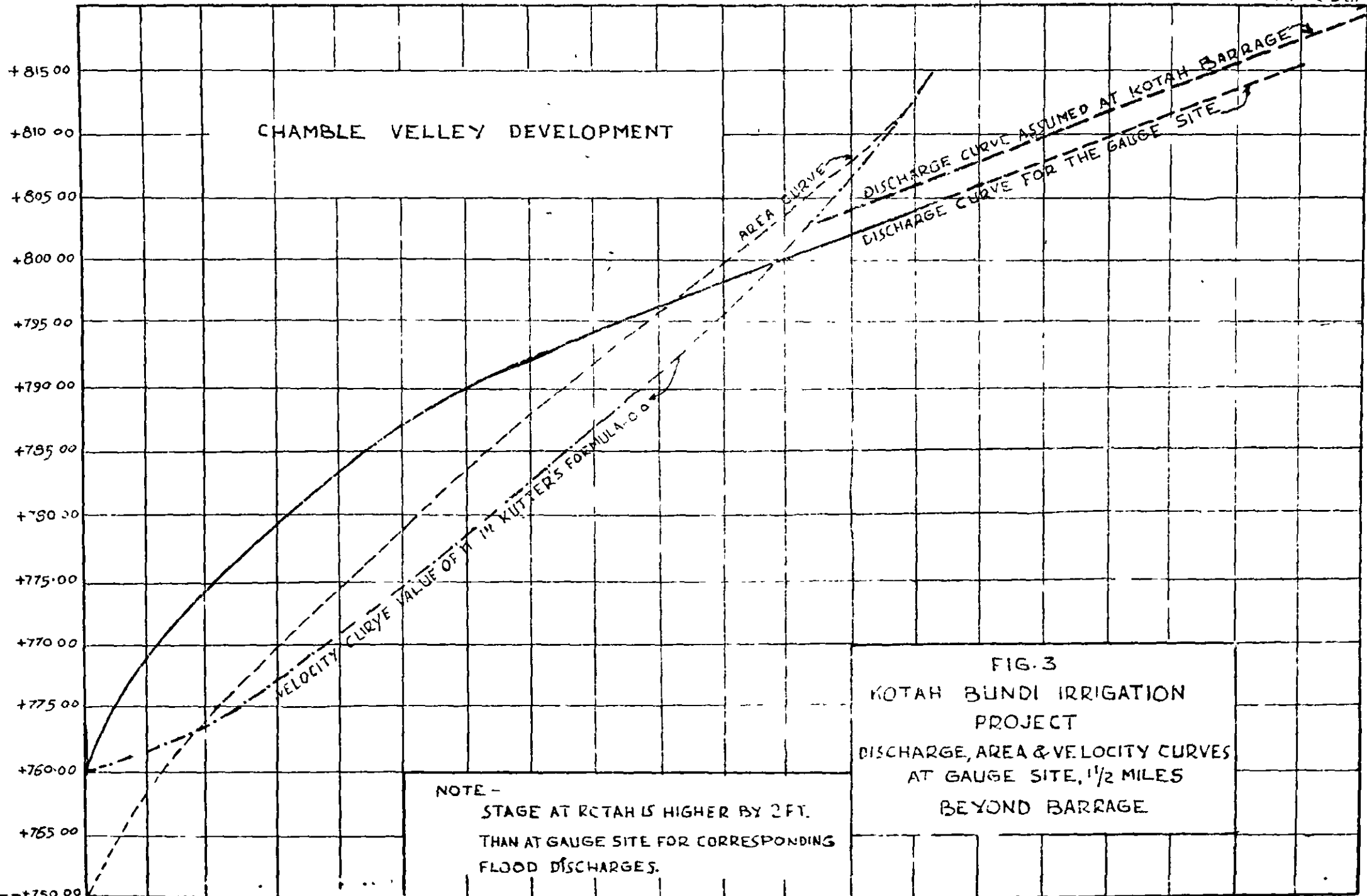


FIG. 2
KOTAH BARRAGE
IRRIGATION PROJECT
1/40 SCALE SECTIONAL MODEL

COMP BY: *[Signature]*
TRACED BY: S.A. MANCINDIA

WATER POOL GAUGE READINGS. 1/2 MILES D/S OF THE BARRAGE SITE



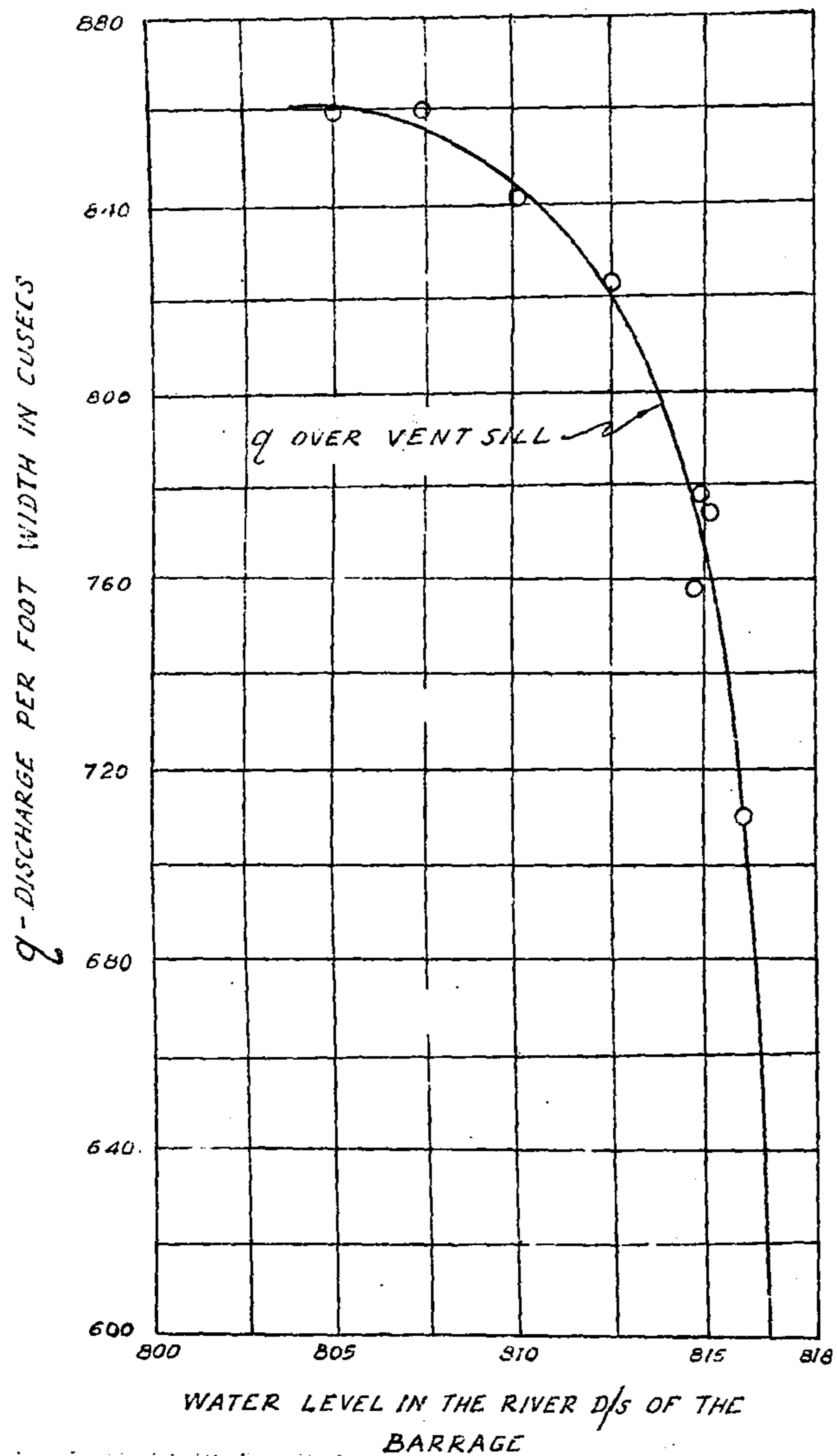
NOTE -
STAGE AT KOTAH IS HIGHER BY 2 FT.
THAN AT GAUGE SITE FOR CORRESPONDING
FLOOD DISCHARGES.

FIG. 3
KOTAH BUNDI IRRIGATION
PROJECT
DISCHARGE, AREA & VELOCITY CURVES
AT GAUGE SITE, 1/2 MILES
BEYOND BARRAGE.

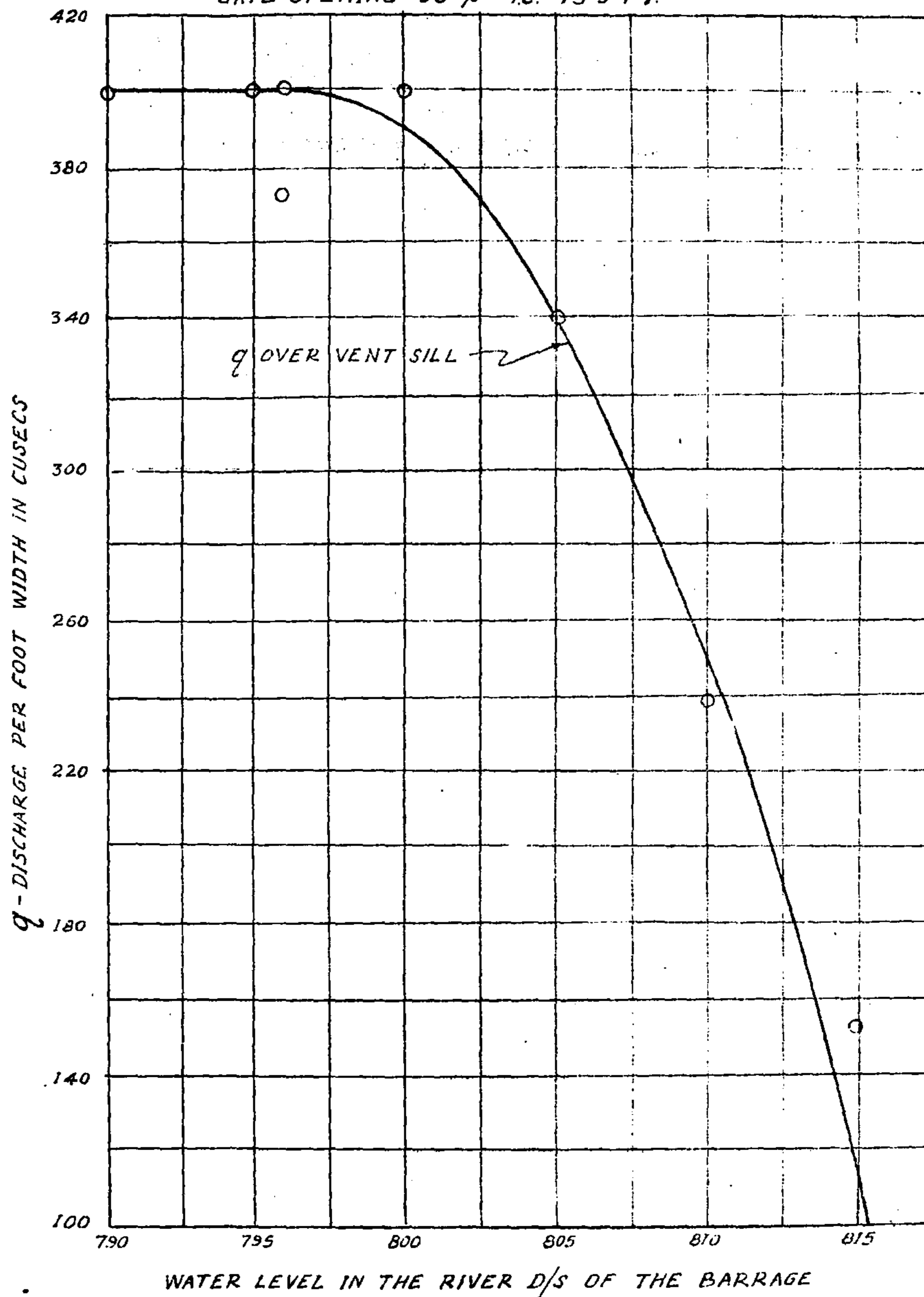
DISCHARGE IN LACS OF CFS	0.5	10	15	20	25	30	35	40	45	50	55	60	65	70	75	80	85	90	95	100
AREA IN 000'S OF SQ. FT.	10	20	30	40	50	60	70	80	90	100	110	120	130	140	150	160	170	180	190	200
VELOCITY FT/SEC	20	30	40	50	60	70	80	90	100	110	120	130	140	150	160	170	180	190	200	210

FIG. 4

GATE FULLY OPEN



GATE OPENING 50% i.e. 13.5 FT.



SSI
COMP BY
TRACED BY: R. T. THAKUR

FIG. 4 & 5
KOTAH BARRAGE
1/40 SCALE SECTIONAL MODEL
ORIGINAL DESIGN A
q/FT. OF VENT: D/S W.L.
W.L. U/S OF BARRAGE = 818.0

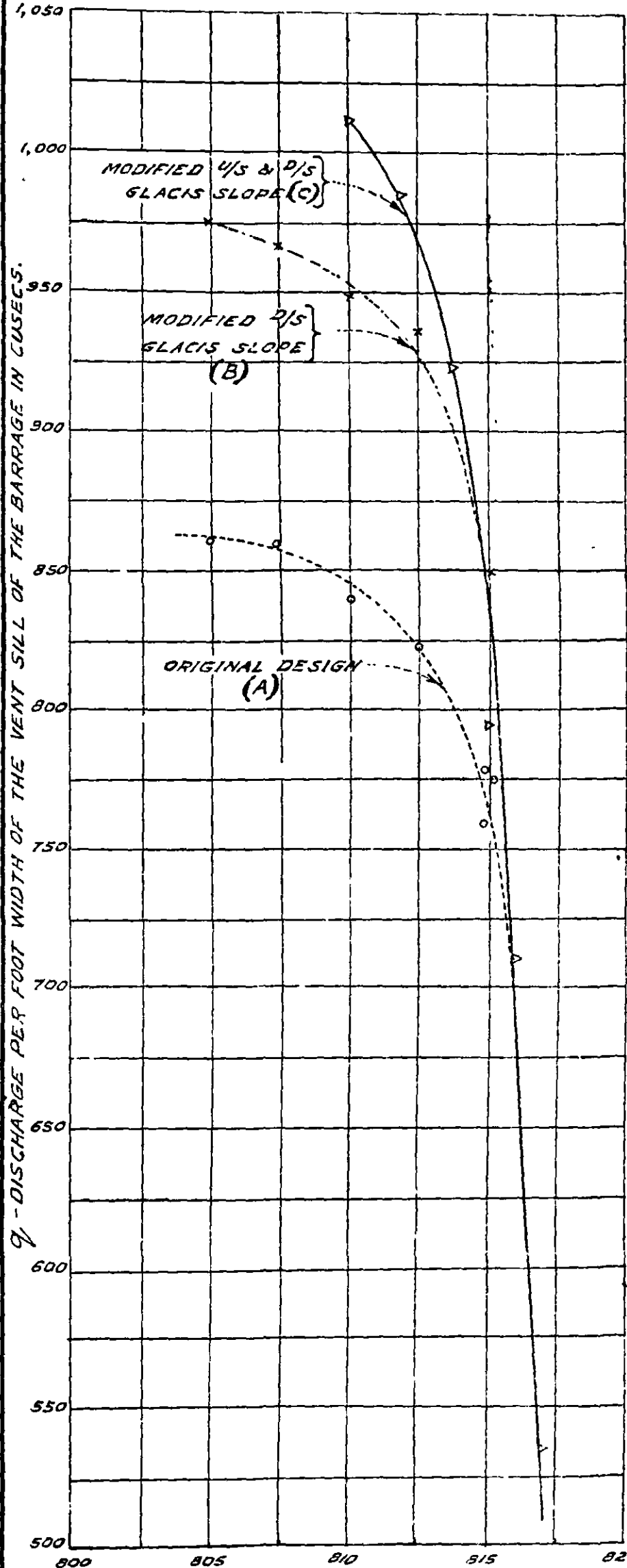


FIG. 7
KOTAH BARRAGE
ON CHAMBAL
1/40 SCALE SECTIONAL MODEL
Q - DISCHARGE PER FOOT OF
BARRAGE VENT: D/S WL.
FOR W.L. 4/5 = 818.00
WITH 100% GATE OPENING

COMP. BY -
TRACED BY - PATRIE G. R. *Pat*
29-1-1952. *Pat*

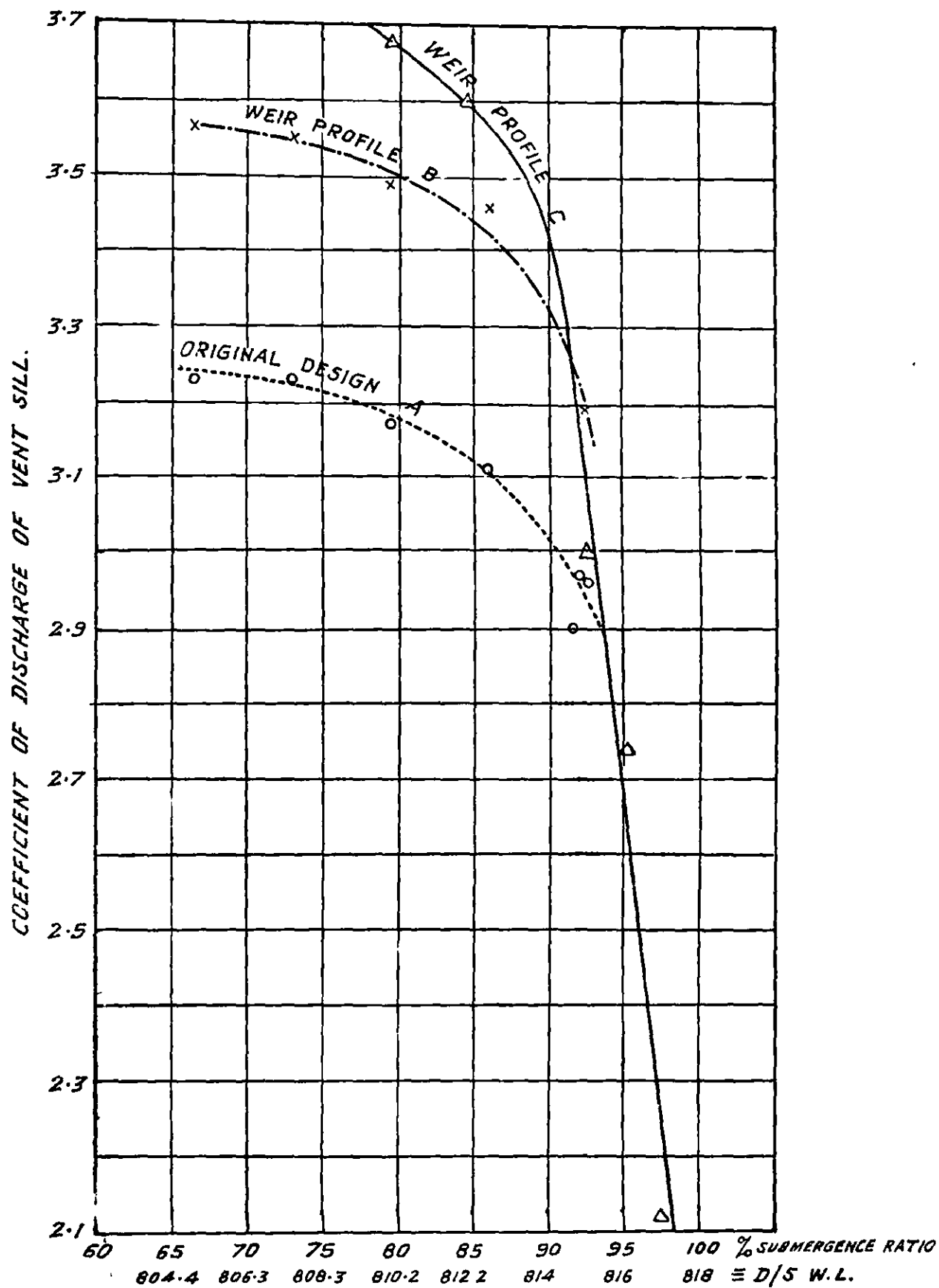


FIG. - 6
KOTAH BARRAGE PROJECT
1/40 SCALE SECTIONAL MODEL
C - COEFFICIENT OF BARRAGE VENT : D/S.W.L.
FOR
W.L. U/S OF BARRAGE \equiv 818.0
BARRAGE GATES FULLY OPEN
BARRAGE SILL AT R.L. 779.0

COMP. *PhD*
TRACED BY SAPKAL 25-1-52.

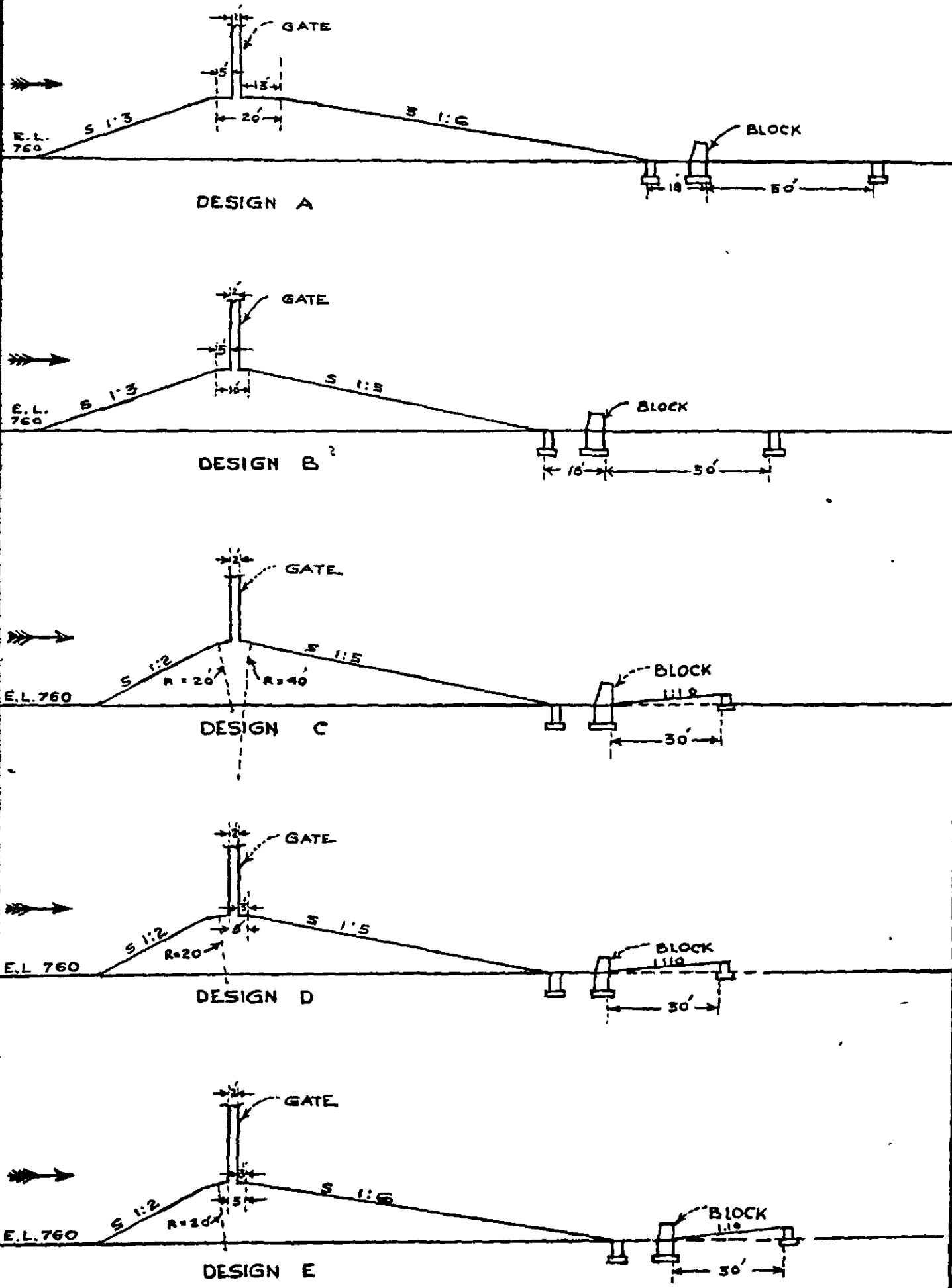
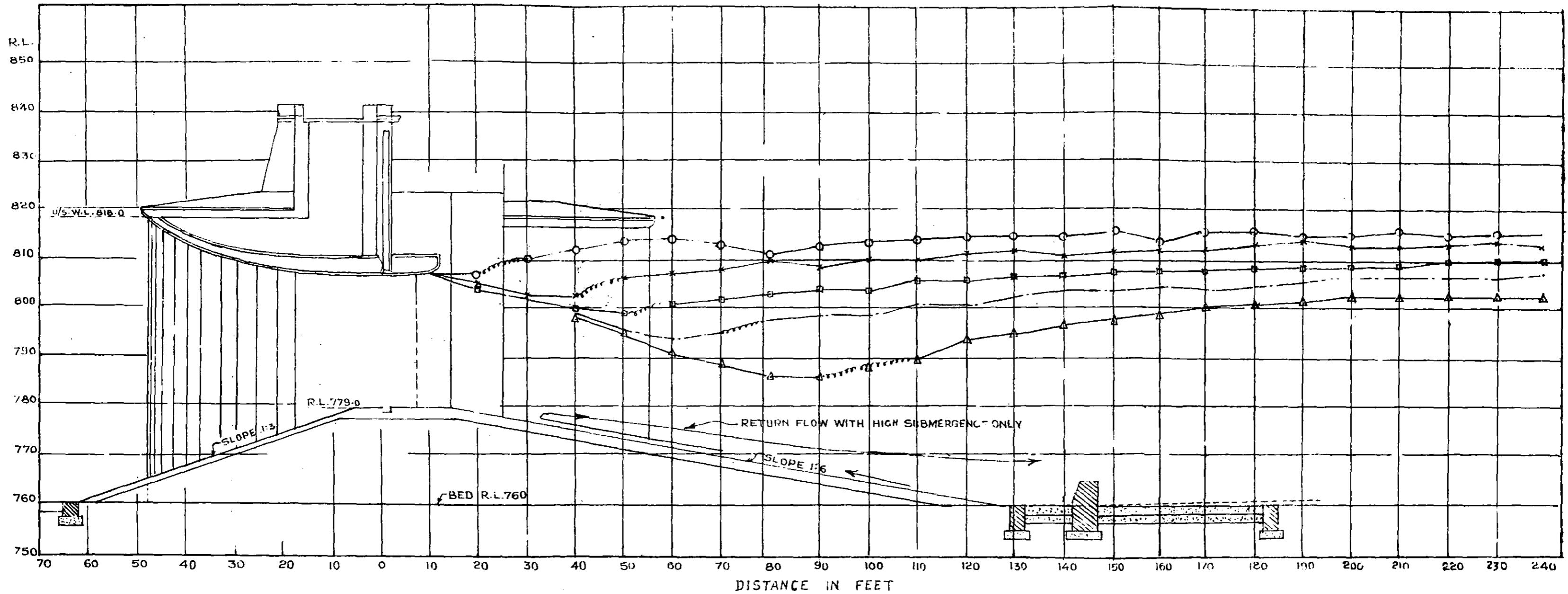


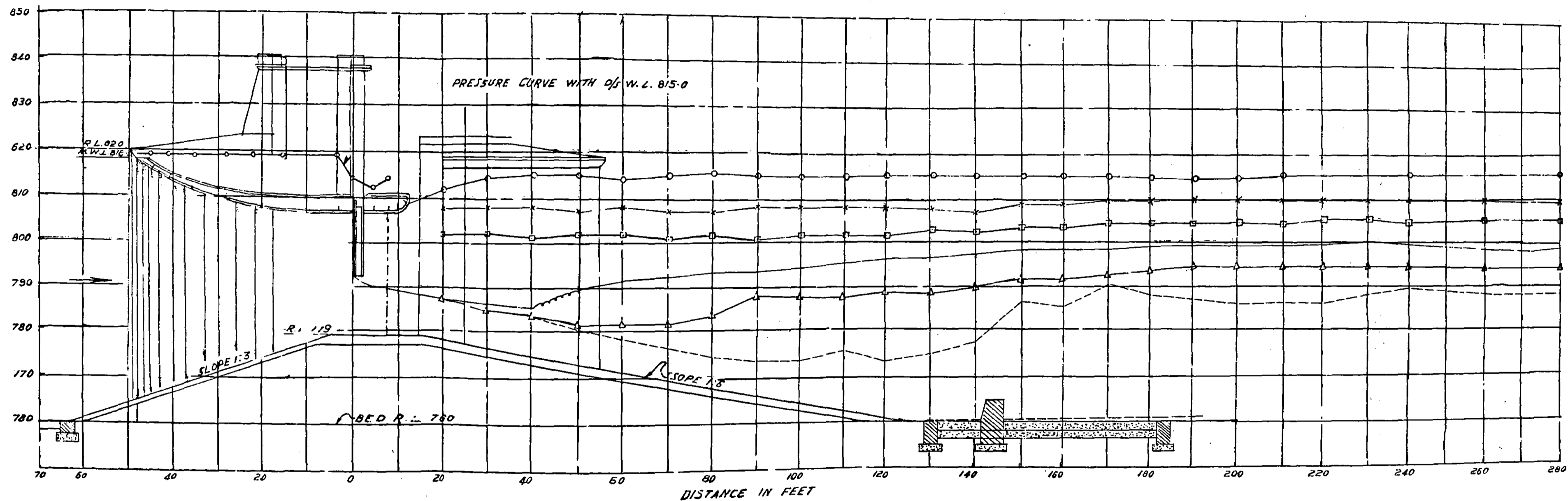
FIG. 8
KOTAH BARRAGE PROJECT
1/40 SCALE SECTIONAL MODEL
ORIGINAL DESIGN AND MODIFICATIONS TESTED
SCALE: 1" = 40'



U/S. W.L. 818.0	D/S.W.L. 815.0 ALL GATES FULLY OPEN.	○—○—○
U/S. W.L. 818.0	D/S W.L. 812.50 ALL GATES FULLY OPEN	x—x—x
U/S. W.L. 818.0	D/S W.L. 810.00 ALL GATES FULLY OPEN	□—□—□
U/S. W.L. 818.0	D/S. W.L. 807.50 ALL GATES FULLY OPEN	- - - - -
U/S. W.L. 818.0	D/S W.L. 805.00 ALL GATES FULLY OPEN	△—△—△

FIG. 9
KOTAH BARRAGE
IRRIGATION PROJECT
ORIGINAL DESIGN A
1/40 SCALE SECTIONAL MODEL
W.S. PROFILE WITH U/S W.L. 818.0
AND VARIOUS W.L. D/S

C. BY: *[Signature]*
T. BY: R. K. KAWILKAR - 28-1-52.

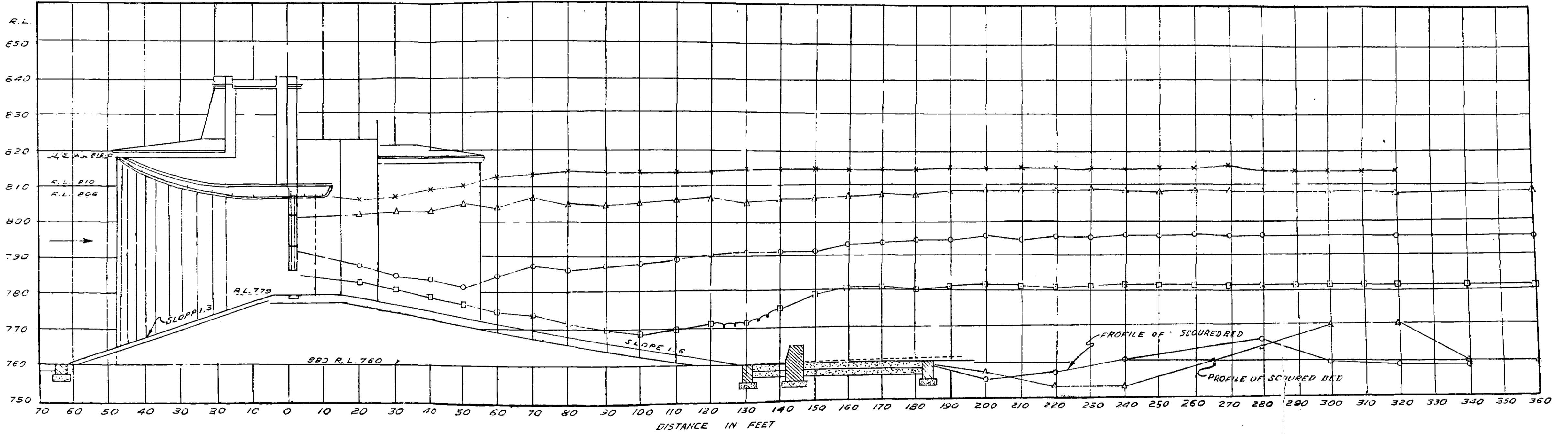


REFERENCE:

U/S W.L. 818-0	D/S W.L. 815-00	GATE OPENING 50 %	○—○—○—○—○
U/S W.L. 818-0	D/S W.L. 810-00	GATE OPENING 50 %	×—×—×—×—×
U/S W.L. 818-0	D/S W.L. 805-00	GATE OPENING 50 %	□—□—□—□—□
U/S W.L. 818-0	D/S W.L. 800-00	GATE OPENING 50 %	— — — — —
U/S W.L. 818-0	D/S W.L. 795-00	GATE OPENING 50 %	△—△—△—△—△
U/S W.L. 818-0	D/S W.L. 790-00	GATE OPENING 50 %	- - - - -

FIG: 10
KOTAH BARRAGE
IRRIGATION PROJECT
ORIGINAL DESIGN, A
1/40 SCALE SECTIONAL MODEL
W.S. PROFILE WITH 50% GATE
OPENING WITH U/S W.L. 818-0

COMPARED BY *[Signature]*
TRACED BY: M.K. SABNIS, 28-1-52.



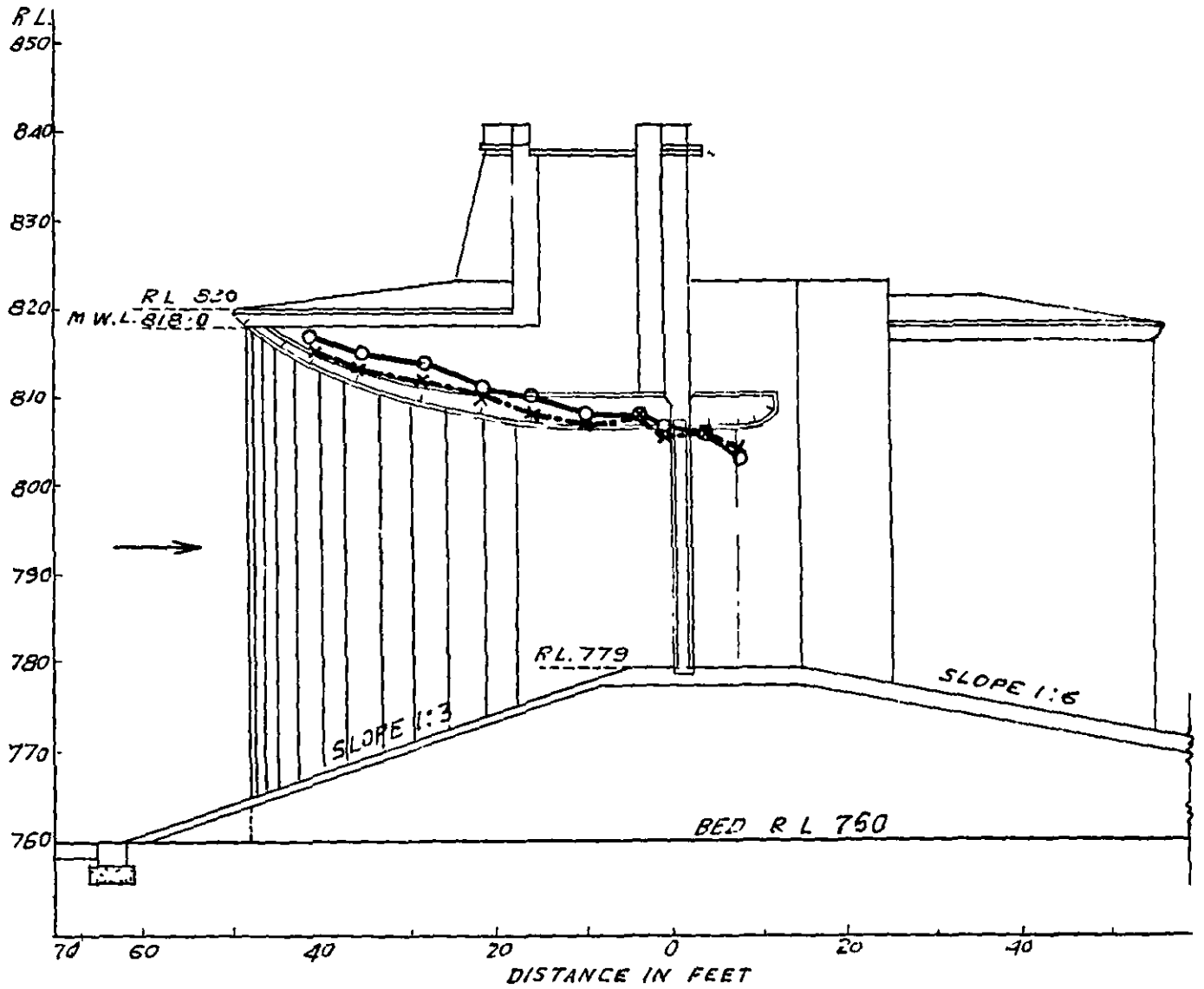
REFERENCES:-

U/S W. L. 818.0	D/S W. L. 815.00	GATE OPENING 100%	x-x-x-x-x
U/S W. L. 818.0	D/S W. L. 808.6	GATE OPENING 75%	Δ-Δ-Δ-Δ-Δ
U/S W. L. 818.0	D/S W. L. 796.00	GATE OPENING 50%	○-○-○-○-○
U/S W. L. 818.0	D/S W. L. 782.5	GATE OPENING 25%	□-□-□-□-□

FIG. 11
KOIAH BARRAGE
IRRIGATION PROJECT
ORIGINAL DESIGN

1/40 SCALE SECTIONAL MODEL
U/S W. L. 818.0 W. S. PROFILES WITH VARIOUS
GATE OPENINGS WITH CORRESPONDING D/S W. L.

Phlane
COMP BY [Signature]
TRACED BY PARKHI
15.1.1952



REF. ~

U/S W L. 818.00	D/S W.L 815.00	○ — ○ — ○	} ALL GATES FULLY OPEN
U/S W L. 815.00	D/S W L. 812.00	x - - x - - x	

FIG.12
 KOTAH BARRAGE
 IRRIGATION PROJECT
 ORIGINAL DESIGN-A
 1/40 SCALE SECTIONAL MODEL
 PRESSURE DISTRIBUTION
 IN
 BELLMOUTH

P. Deshpande

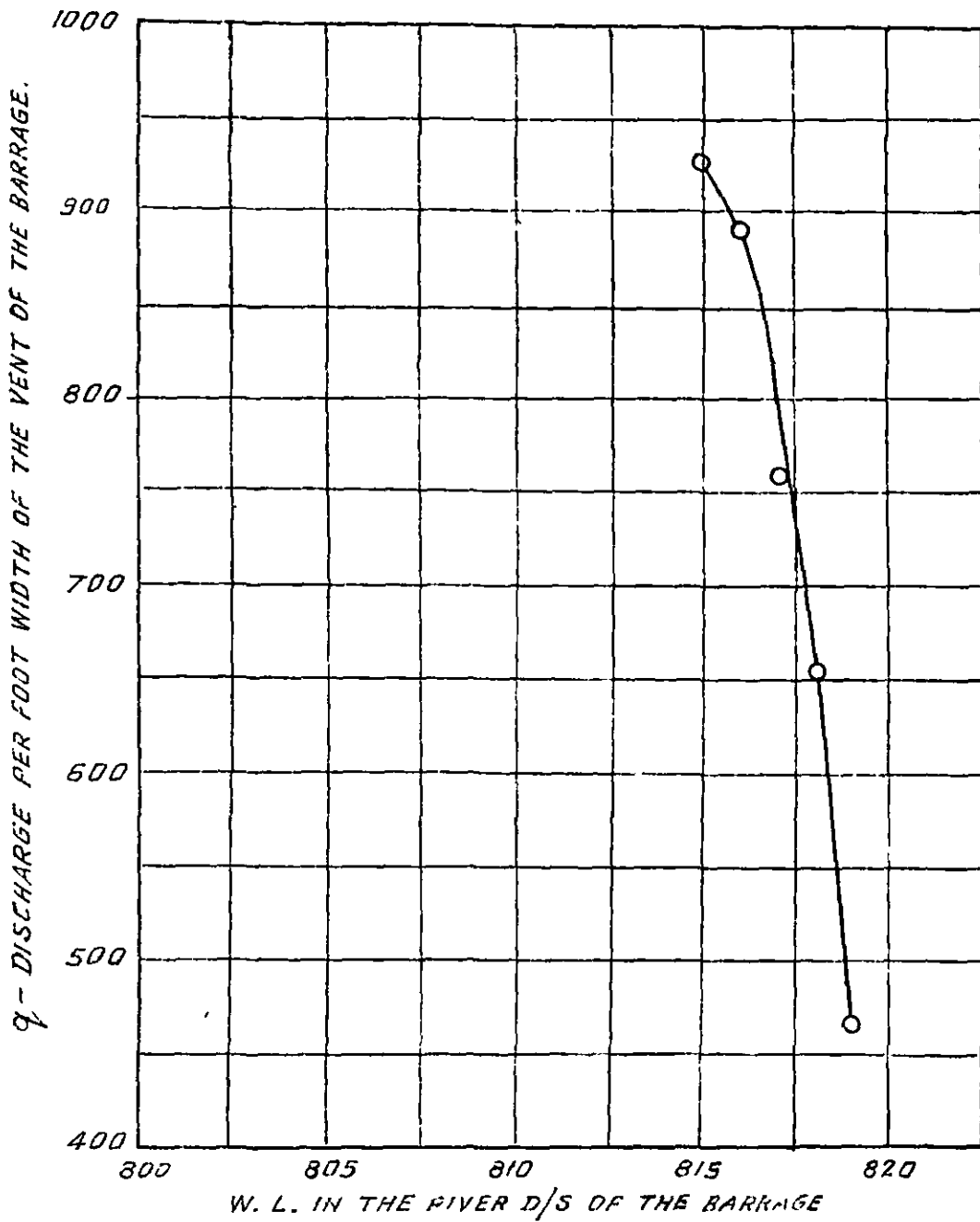


FIG.-13
 KOTAH BARRAGE PROJECT
 1/40 SCALE SECTIONAL MODEL
 q - DISCHARGE PER FOOT OF VENT : D/S W. L.
 FOR
 W. L. U/S OF BARRAGE AT R.L. \approx 820.0
 DESIGN MODIFICATION D

COMP. *[Signature]*
 TRACED BY: SAPKAL 29-1-52

Alternative Barrage Weir Profile B (Figs. 6 and 8)

Figs. 6 & 8.

In this design the horizontal length of the sill was reduced from 20 ft. in the original design to 10 ft. The downstream glacis slope was steepened from 1 in 6 (i.e., 1 vertical in 6 horizontal) to 1 in 5. Thus the total width of the barrage section was reduced by 29 ft. as compared to the original design. Results obtained are given in Table 3 with water levels 818 0 upstream and 815 0 downstream; the discharging capacity improved by only about 5 per cent. This did not give sufficient improvement. Due to steeper glacis slope in this design the back roller on the bed of the downstream glacis was more pronounced and the slight improvement obtained in the coefficient was solely due to the reduction of the sill length from 20 ft. to 10 ft.

Alternative Barrage Weir Profile C (Figs. 6 and 8)

Figs. 6 & 8.

In this design the downstream slope was kept 1 in 5 as in weir profile B; but the upstream slope was changed from 1 in 3 to 1 in 2. The sill length was further reduced to only 2 ft. (just to allow for the gate seat). The upstream and downstream slopes were joined to the sill by curves of 20 ft. and 40 ft. radius respectively.

Figure 7 shows the discharging capacities obtained in designs A, B and C. They show that with steeper upstream and downstream slopes, and shorter crest lengths the coefficient of discharge and the discharge per foot width, of the vent markedly improved, but with higher submergence over 92 per cent the differences become less prominent due to the sharp fall in the discharge coefficients. This is also accompanied by unfavourable flow conditions on the glacis downstream in that, with the steeper downstream glacis slope the bed filaments leave the glacis earlier and the return eddy region moves up the glacis. This is also accompanied with increased strength of the return eddy which will roll up stones of 1½ ft. diameter up the glacis and may cause abrasion; but in this case, the river in flood is not reported to carry large-sized boulders, which are more likely to be caught up in the numerous pools upstream.

Fig. 7.

Flow round the ease-water of the piers was improved by rounding the downstream edge of the pier with 15 ft. radius in plan.

Stage Discharge Curve of the River

From the stage discharge curve supplied, the river discharges of the channel at different water levels at Kotah are—

Stage	Discharge in Lakh cusecs
819	9 9
818	9 62
817	9 36
816	9 19
815	8 85

Hence in the design C, higher water levels downstream of the barrage than 815 0 were tested. In addition, discharging capacity of the barrage with upstream water level 819 0 and downstream water level 817 0 was also observed. Under this condition the discharge per vent obtained was (Table 4) 31,900 cusecs and about 30 vents will be required to pass a flood discharge of 9 4 lakh cusecs.

Alternative Barrage Weir Profile D (Fig. 8)

Fig. 8.

This design differed from design C in that the length of the sill was increased from 2 ft. to 5 ft. keeping the upstream and downstream glacis the same, viz., 1 in 2 and 1 in 5 respectively. Results obtained are given in Table 5 and Figure 13.

Fig. 13.

In these experiments upstream water level was restricted to R.L. 820 0 and various water levels downstream of the barrage were maintained. The discharges obtained under these conditions show that with upstream water level 820 0 and 3 ft. afflux the discharging capacity of the vent is lesser by about 45 per cent as compared to equal afflux but with upstream and downstream water levels at 818 0 and 815 0 respectively, as with higher water levels, the submergence ratio increased, so that the discharge coefficient decreased slightly.

Fig. 8. Alternative Barrage Weir Profile E (Fig. 8)

This design E differs from design D in only having its downstream glacis being 1:6 instead of 1.5 as in design D, the crest being 5 ft and upstream glacis 1:2 in both cases with same downstream protection. Table No. 5 gives the discharge values for design D and Table No. 6 those of design E.

It will be observed that for all practical purposes regarding discharge, the two designs can be considered equivalent.

The design E for the weir profile is recommended for economy and satisfactory performance. The upstream glacis of 1.2 used in this design can however be replaced by 1.3 as used in the original design, if desired, without any significant difference in the discharge characteristics.

Consideration of Results

It will be evident from the results of the foregoing experiments that upstream water level cannot be fixed at 818 0 with flood discharges of 10 lakh cusecs unless the number of vents provided is increased considerably. Or, alternatively the maximum water level upstream may be fixed at R.L. 819 0 in which case a discharge of nearly 9.5 lakh cusecs would be passed with 30 vents. This is suitable because the maximum permissible upstream water level is 820 0 at the barrage and the back-water effect will extend up to 6 miles only, whereas Kotah City is about 8 miles above barrage.

This however does not take into account the flood moderation which will take place due to construction of dam or dams upstream of the barrage, but if the construction of the barrage precedes that of the proposed dams or if their construction is deferred for a considerable period, a provision to pass the full maximum flood discharge appears necessary. With the proposed Gandhisagar Dam, the flood will be moderated to about 7.5 lakh cusecs.

Conclusions

As seen above, 23 vents will be sufficient to dispose of a flood corresponding to river R.L. 815, with an afflux only 3 ft, upstream water level being R.L. 818 where, from the rating curve the discharge appears to be 8.8 lakh cusecs. For higher floods however, water level downstream of the barrage will be higher than R.L. 815 so that the discharging capacity per vent will be lesser if the same upstream water level is assumed, viz., 818. For example, for disposal of a flood of 9.5 lakh cusecs the downstream water level will be R.L. 817 and it will be necessary to have 45 vents with 1 foot of afflux (upstream R.L. 818), 30 vents for 2 ft. afflux (upstream R.L. 819) and 25 vents only for 3 ft afflux (upstream R.L. 820).

In such a case, therefore, when a large flood of 9.5 lakh cusecs is adopted for design, the upstream permissible level also should be raised to R.L. 820. Since it is known that with a 3-ft. afflux at the barrage site the back-water effect does not reach beyond railway bridge 3 miles down from the city, the designs on the basis of permissible upstream R.L. 820 should be considered acceptable.

Serial No.	Gate Opening	U/S W. L.	D/S W. L.	q vent (cusecs/ft.)	q barrage (cusecs/ft.)	Q per vent (cusecs)	C	Q—from rating curve lakh cusecs	No. of vents required
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TABLE 1—Kotah barrage original design 1/40 scale sectional model. Discharges with different afflux—with U/S W.L. 818

1	100%	818 0	814.76	758	631	37,900	2.90	8.80	23.2
2	"	"	815.12	775	645	38,700	2.96	8.86	23.2
3	"	"	814.84	778	648	35,000	2.97	8.80	22.7
4	"	"	812.56	823	686	41,200	3.11	8.22	20.0
5	"	"	810.04	841	700	42,000	3.17	7.58	18.0
6	"	"	807.44	859	715	43,000	3.23	6.92	16.1
7	"	"	805.00	860	717	43,000	3.23	6.28	14.6

TABLE 2—Discharge with different tail water elevations with U/S W.L. 818, original design.

1	50%	818 0	814.9	151	126	.	1.24
2	"	"	810.0	238	198	..	1.94
3	"	"	805.0	338	282	..	2.74
4	"	"	800.0	400	333	..	3.22
5	"	"	795.0	400	332	..	3.22
6	"	"	790.0	400	332	..	3.22

TABLE 3—Kotah barrage. Alternative design B. 1/40 scale sectional model.

1	100%	818 0	815.00	848.0	706.0	42,400	3.19	8.85	20.8
2	"	"	812.5	936.0	779.0	46,800	3.46	8.2	..
3	"	"	810.0	948.0	789.0	47,400	3.49	7.58	..
4	"	"	807.5	966.0	804.0	48,300	3.55
5	"	"	804.9	974.0	811.0	48,700	3.57	6.29	..

TABLE 4—Kotah barrage Alternative design C. 1/40 scale sectional model.

1	100%	818 0	817.0	535	441	26,750	2.12	9.35	34.95
2	"	"	816.0	709	591	35,450	2.74	9.09	25.60
3	"	"	815.0	794	662	39,700	3.02	8.83	22.22
4	"	"	812.0	984	819	49,200	3.60	8.00	.
5	"	819 0	817.0	638	531	31,900	2.41	9.35	20.0

Serial No.	Gate Opening	U/S W. L.	D/S W. L.	q vent (cusecs/ft.)	q barrage (cusecs/ft.)	Q per vent (cusecs)	C	Q—from rating curve lakh cusecs	No. of vents required
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TABLE 5—Kotah barrage. Alternative design D. 1/40 scale sectional model.

1	100%	820 0	819 0	466	388	23,300	1.73	9.9	42.5
2	"	"	818.0	656	548	32,800	2.31	9.62	29.3
3	"	"	817.0	758	631	37,900	2.71	9.36	24.7
4	"	"	816 0	891	742	44,550	3.09	9.12	20.5
5	"	"	815 0	927	772	46,350	3.22	8.85	..
6	"	818.00	815 0	828	690	41,400	3.13	8.85	21.40
7	"	815.0	810.0	848	707	42,400	3.54
8	"	820	818.0	688	573	34,400	2.49	9.62	28.0
9	"	820	817 0	770	641	38,500	2.75	9.36	24.5
10	"	820.4	816 2	869	724	43,500	3.02	9.12	21.0
11	"	819 9	816 1	877	730	43,900	3.09	9.12	21.0
12	"	820 1	816 1	880	733	44,000	3.09	9.12	21.0
13	"	820 0	815 0	960	799	48,000	3.32	8.85	18.5
14	"	819 0	817 0	723	602	36,200	2.70	9.36	26.0
15	"	819.0	816.0	858	715	42,900	3.13	9.12	21.5
16	"	819 0	815 0	948	789	47,400	3.39	8.85	19.0
17	"	818 0	817.0	570	475	28,500	2.25	9.36	33.0
18	"	818 0	816.0	739	616	37,000	2.84	9.12	25.0
19	"	818.0	815.0	930	774	46,500	3.45	8.85	19.0

TABLE 6—Kotah barrage. Alternative design E. 1/40 scale sectional model.

1	100%	820	819.0	497	414	24,850	1.84	9.9	40.0
2	"	"	818 0	681	572	34,050	2.48	9.62	28.5
3	"	"	817 0	824	687	41,200	2.92	9.36	23.0
4	"	819	818 0	553	460	27,650	2.11	9.62	35.0
5	"	"	817.0	740	616	37,000	2.74	9.36	25.5
6	"	"	816 0	849	706	42,450	3.09	9.12	21.5
7	"	"	815 0	917	764	45,850	3.29	8.85	19.5
8	"	818	817.0	577	480	28,850	2.27	9.36	32.5
9	"	"	816.0	792	660	39,600	3.01	9.12	23.0
10	"	"	815.0	920	766	46,000	3.41	8.85	19.5
11	"	"	814 0	937	780	46,850	3.46
12	"	820	818 0	715	595	35,750	2.57	9.62	27.0
13	"	"	817.0	869	724	43,450	3.05	9.36	21.5
14	"	"	816 0	981	817	49,050	3.37	9.12	18.5
15	"	"	815.0	990	825	49,500	3.39	8.85	18.0
16	"	819	817 0	709	590	35,450	2.65	9.36	26.5
17	"	"	816.0	931	776	46,550	3.35	9.12	19.5
18	"	"	815.0	949	790	47,450	3.39	8.85	19.0
19	"	818	817.0	573	477	28,650	2.26	9.36	33.0
20	"	"	816.0	787	655	39,350	2.99	9.12	23.5
21	"	"	815 0	922	767	46,100	3.42	8.85	19.5

(iii) DIVERSION WORKS

12. DETERMINATION OF AFFLUX CAUSED BY THE PROPOSED DIVERSION OF THE RIVER MAHANADI DURING CONSTRUCTION—STAGES I AND II

Introduction

This report refers to a hydraulic model for testing diversion proposals in connection with construction stages I and II for the Hirakud Main Dam.

The Hirakud Main Dam consists of two earth sections and two concrete sections covering a total length of about 16,000 feet. The river Mahanadi in this area consists of two main channels enclosing between them the Hirakud island; the channels join about two miles downstream of the axis of the dam, which cuts through the Hirakud island and lies at an angle to the river (Figures 1 and 2).

Figs. 1 & 2.

On the left channel the concrete spillway dam is to extend over 1,200 feet between the left earth section and the central earth dam and on the right channel concrete structure consists of 720 feet of spillway section, 660 feet of power section and provision for a future navigation lock.

While certain stretches of the river channels contain a shallow layer of coarse sand, a large part of the bed consists of hard rock, and rock outcrops as high as 10 to 15 feet. The general slope of the river in this area is of the order of 4 feet to a mile. The normal maximum flood is proposed to be disposed of by 40 undersluices and 21 crest bays in the left channel spillway, and 24 undersluices and 13 crest bays in the right channel spillway.

The general bed level at site is E.L. 500; the general foundation level is E.L. 490. The spillway crest is at E.L. 610, the normal maximum flood level is R.L. 625 while the maximum flood level for which the dam is designed is R.L. 630. The top level of the earth dam is at E.L. 642. The undersluice sill is at E.L. 510, so that all the sluices are near the bed allowing the optimum silt scouring possibilities. The maximum tail water level is R.L. 535. The terms of reference for this model were as follows—

With the construction stage I at E.L. 510 for the spillway portions as well as stage II at E.L. 530, with a coffer dam round the power section, to determine:

- (i) the afflux,
- (ii) the sufficiency and satisfactory performance of the alignment of the coffer dam, and
- (iii) the nature of flow and recommendations for avoiding unsatisfactory or serious flow conditions, if any.

2. Model Lay-out and Construction

The scale chosen for the model was 1/200 horizontal and 1/60 vertical. With this vertical exaggeration of $3\frac{1}{2}$, the length of the sluiceway and the bucket in the direction of flow became correspondingly shortened. In the diversion stage I, the left channel and the right channel spillways would act as broad-crested weirs in the prototype. For a head H of the order of 30 feet and breadth B (up to the sill of the bucket) at 163.4 feet, the ratio B/D is 5.4. With the exaggeration $3\frac{1}{2}$ in the model this becomes only 1.6 which is somewhat less than that necessary for the structure in the model to act as a broad-crested weir. The afflux obtained from this scale would therefore be slightly underestimated, and hence the experiments were repeated with a G. S. model of the weir only—along the general normal to the axis—the rest being vertically exaggerated so as to ensure proper curvature.

The model lay-out is shown in Figure 1. The model area represents about 3 miles upstream and 2 miles downstream of the dam, in a length of about 150 feet. The discharge entered the model over two standing wave flumes at the upstream end of the model from where the river channel begins to bifurcate, before finally separating with two distinct ones at the nose of the Hirakud island. The downstream heading up arrangement was by means of battens across the river about

Fig. 1.

Photo 408. half a mile downstream of the toe of the Hirakud island (Photo 408) where the two channels have already joined to form the single channel going to the Mahanadi-rail-road bridge site and Sambalpur beyond.

Across the Hirakud island in the model, the earth sections were laid in semi-pervious, black cotton soil properly rammed up to E.L. 550 only. The spillway sections were made of cement concrete to the elevation required for the two stages. The coffer dam round the power section was reproduced by a vertical G. I. sheet according to plan to represent the sheet pile and connected end sections to represent the end earth sections were made in cement plaster.

A representation of the model area was made out from the 60 feet contours obtained from a 1947-48 survey which demarcated the main channels as well as the areas of extensive rock outcrops.

The permanent banks of the river were represented by a thin vertical wall of cement plaster up to E.L. 550. The bed was made up mainly of about 1/2 inch screened Khadakwasla sand overlaying crushed murum properly rammed. The islands were made up of murum, and in some areas adjacent to the earth, they were made up simply of black cotton soil. The sub-channels of the two main channels, along with the areas of the rock outcrops were properly demarcated near the spillways with the help of the 1947-48 survey map; these channels were laid with coarse gravel, while the rock outcrops up to E.L. 520 were laid with 1 inch stone and islands above E.L. 520, which normally get submerged in flood were made up of 1"-2" pebbles.

The foregoing arrangement reproduced the general nature of the bed, viz., a bed load for high flood over the general reach, larger losses through the rock outcrops and a proper subdivision of the discharge through the main channels. In a normal river model where bed friction plays a predominant part, verification of the model would be necessary and requisite adjustment of the bed friction made to reproduce main features of prototype data on stage-discharge relationships at various stations, the water slope and the currents, previous to the construction of permanent structures. In the present case, however, where the main requirement is the testing of diversion proposals to obtain the afflux, which is to a great extent influenced by the spillway areas acting as weirs with a high submergence ratio, the effect of an incomplete representation of bed friction would not influence the results appreciably. The necessary hydrological data also was not available at this stage.

Fig. 1. Gauge-wells were placed in the model in locations as shown in Figure 1. Parallel angle irons were placed in the spillway areas to carry girders for velocity and surface flow measurements.

3. Test Procedure

With a given set-up for the coffer dam and the spillway area available for flood discharge, with a given flood discharge, the tail water had to be adjusted by the tail-gate battens, to correspond to available stage-discharge relationship. However, since the given flood discharge is divided into two parts entering the two channels, adjustments had to be made assuming a reasonable division of the discharge. The simplest division which could be assumed was that of approximately in the ratio of the spillway lengths provided. The ratio on this basis taken was Q-left channel: Q-right channel=60 per cent: 40 per cent approximately. When the downstream gauge-well for the left channel was made to correspond to the tail water level answering to the assumed discharge, using the rating curve, it was found in each case that the downstream gauge-well for the right channel stood at a higher elevation than what would correspond to a 40 per cent discharge. In other words, the right channel was carrying more discharge than 40 per cent. The ratio of 55 per cent: 45 per cent gave a simultaneous correct tail water downstream of the left and right channel spillways. This was due to the fact that the upstream water level was higher at the right channel spillway than at the left channel spillway by about 5 feet.

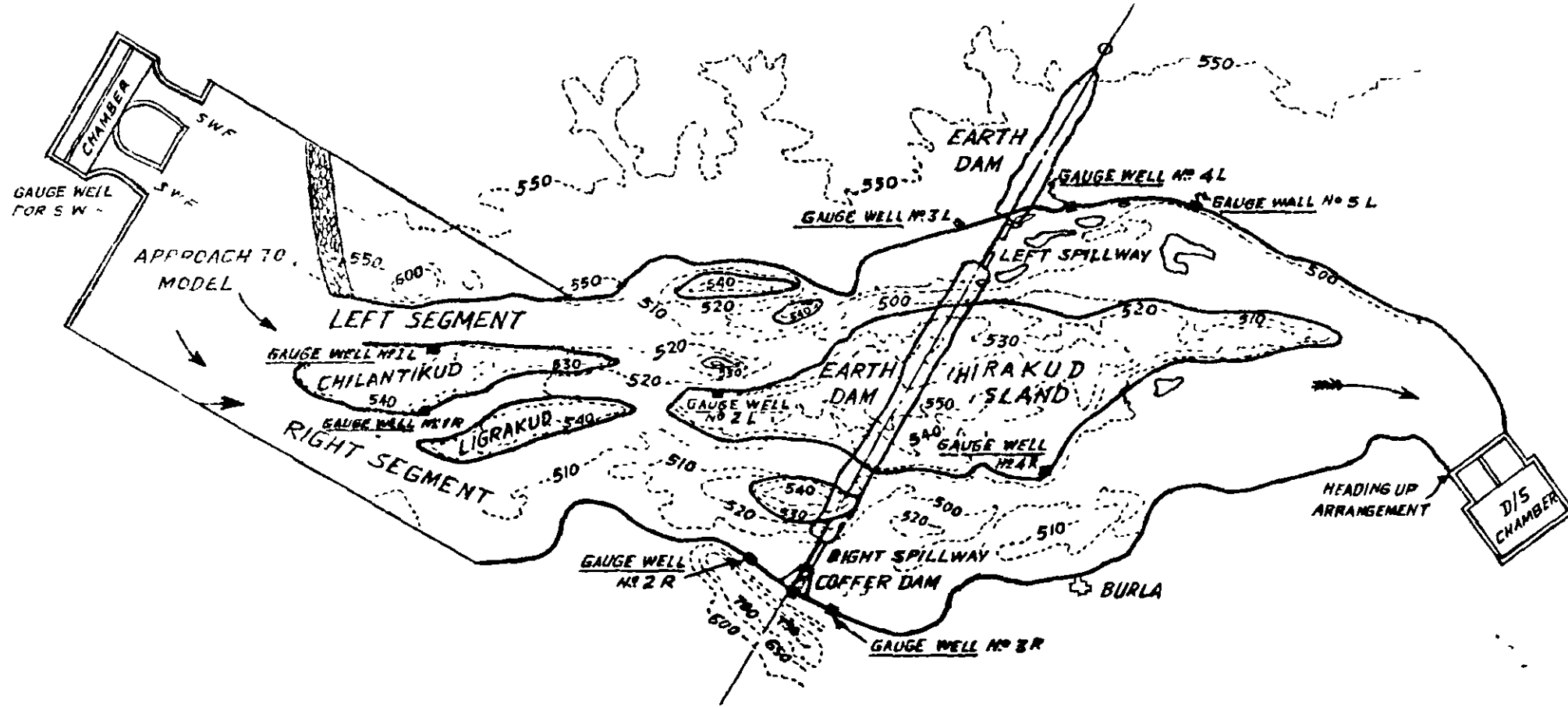
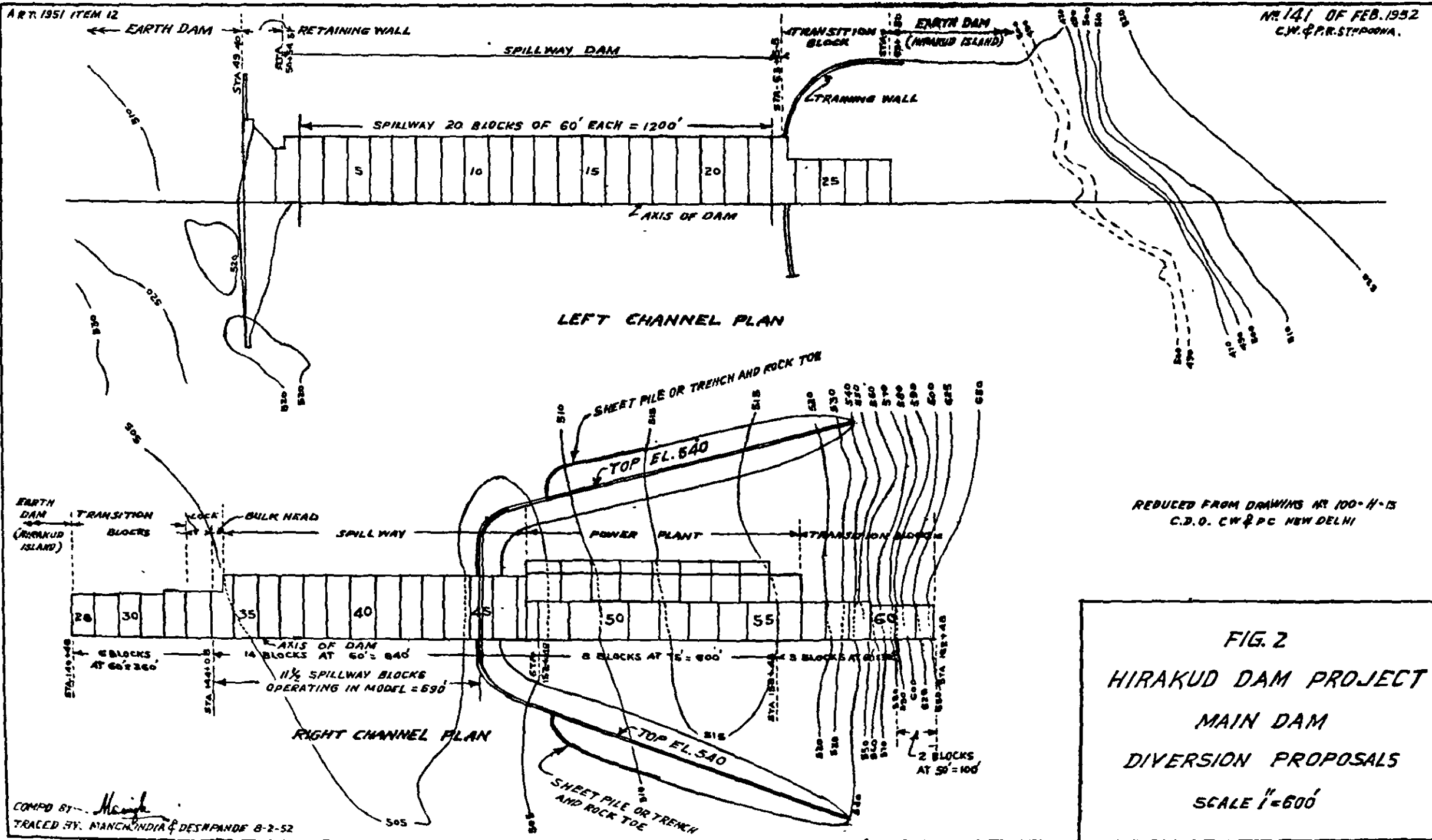


FIG.1
HIRAKUD PROJECT
MAHANADI AT HIRAKUD DAM
MODEL LAYOUT
HYDRAULIC MODEL 5
 MODEL SCALE - HORZ. = 1/200
 VERT = 1/60
 SCALE 1" = 400'

COMP'D BY *M. S. Singh*
 TRACED BY . B G DESHPANDE 23-1-52



REDUCED FROM DRAWINGS NR 100-H-15
C.D.O. CW & PC NEW DELHI

FIG. 2
HIRAKUD DAM PROJECT
MAIN DAM
DIVERSION PROPOSALS
SCALE 1"=600'

COMPD BY: *Manoj*
TRACED BY: MANCHINDIA & DESHPANDE 8-2-52

Hirakud Dam - Construction Stages I and II

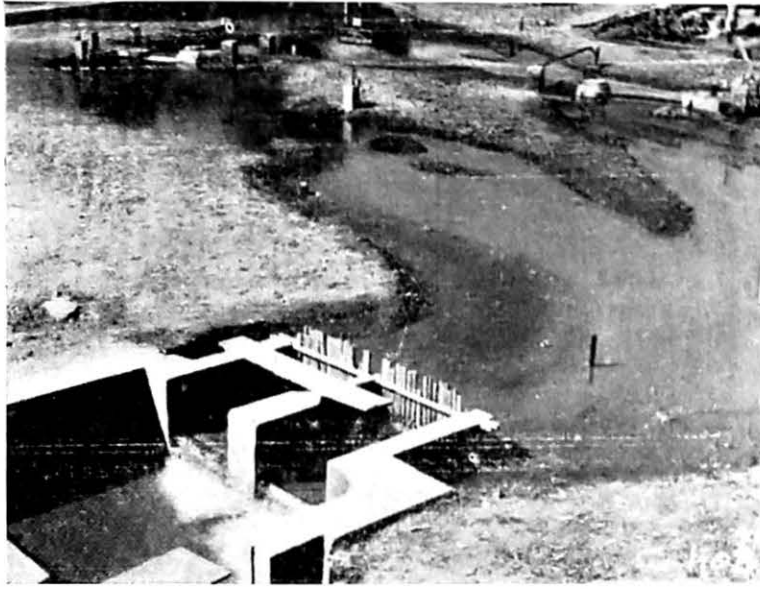


Photo 408.
General view of model flow looking upstream
from below Hirakud Island.

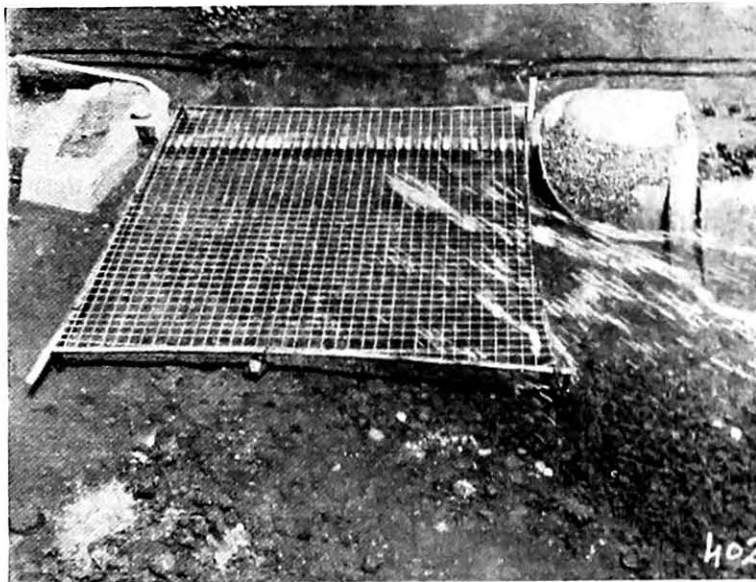


Photo 409
General view of angular approach. Left Spillway -
Construction stage II View looking downstream.

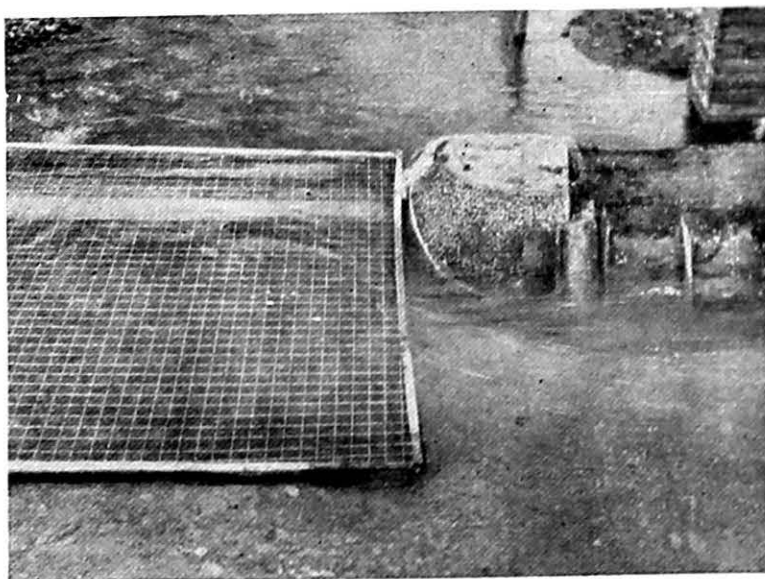


Photo 348
Deflection of flow by 4 short spurs. Left Spillway -
Construction Stage I. View looking downstream.

All gauge readings were taken for flood conditions with different discharges ranging from 4 lakh cusecs to 10 lakh cusecs flood. Velocity readings were taken along the approach flanks. Surface currents near the spillways were photographed using paper bits sprinkled over the photographed area against a 2-inch or 1-inch reference grid. One set of scour photographs was taken using very light gravel in the spillway areas.

Near the left channel spillway, upstream of the right flank, there was high velocity flow and erosion occurred due to the sharp angularity of the flow. Four small spurs, as an alternative, were found to improve flow conditions sufficiently.

To get an indication whether the distortion of the spillway section due to vertical exaggeration has an appreciable effect on the afflux, the case was taken up of an undistorted section for the spillway using the scale normal to the axis as 1:60, i.e., the same as the vertical scale.

For tests corresponding to the other construction stage, viz, when the concrete is laid up to E.L. 530, the sluiceways in plan were reproduced in the spillway blocks up to E.L. 530 using wooden blocks constructed to shape and fixed to form parallel open sluices. The spillways each then formed a parallel series of clear waterway of minimum width 0.72" each (12 feet prototype) and height 4" above the sill (20 feet prototype) (Photo 402).

Photo 402.

4. TEST RESULTS

Construction Stage I

Afflux—For the discharge range 4 lakh—10 lakh cusecs prototype, the afflux caused is shown in Table 1 and Figure 3, which gives the corresponding water surface for each flood discharge, at the four gauge-wells, two upstream and two downstream of each spillway.

Table I &
Fig. 3.

It was observed that afflux was greater on the right spillway than on the left by about 5 feet. A rough idea of the verification of this is given by the fact that from about the nose of the Hirakud island where the two main channels start, the distance to left spillway is greater than the distance to the right spillway by about a mile in the prototype. With a water slope of 4 feet per mile, this would imply afflux difference of this order.

For a flood of 9 lakh cusecs, the coffer dam top is recommended to be taken up to E.L. 543 instead of the designed E.L. 540, or with a free board of two feet, it should be E.L. 545.

Approach to Right Spillway—The alignment of the coffer dam gives a very satisfactorily smooth transition. The flow was generally normal to the spillway length.

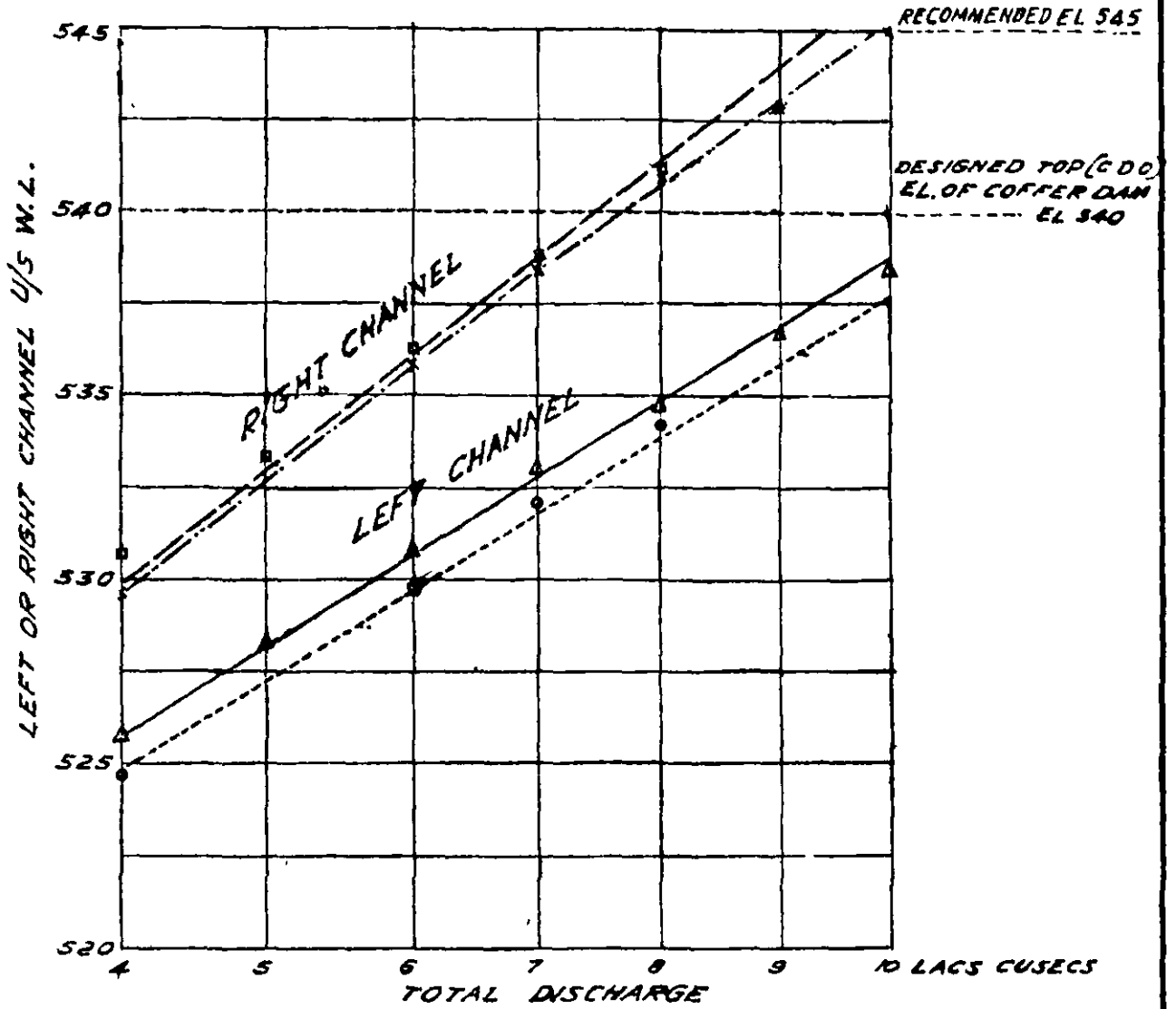
Exit from Right Spillway—Downstream of the spillway the deep channel takes a turn towards the right flank of the Hirakud island. The result of the construction of the spillway to E.L. 510 and the coffer dam is to have a stream about 700 feet wide issuing through the spillway at velocities of the order of 25 ft./sec. Before taking a turn towards the left in the general direction of the right channel, it follows a straight course to a certain extent. For the higher floods, this high velocity stream would attack the right bank about half a mile downstream. No protection is however necessary as the bed consists of exposed rock. The conditions just downstream of the spillway bucket due partly to the high velocity stream and partly to the hydraulic jump, were violent. Bed made of 1" stone pieces was scoured away to some extent in the model. With the bed made of coarse gravel, the scour was fairly deep.

Approach to the Left Spillway—Here the angularity of approach of the flow to the left spillway is very acute (Photos 348, 402 and Figure 4). This caused very strong currents on the right flank with a strong eddy just on the upstream right edge of the spillway. The structure has to be protected against this serious current either

Photos 348,
402 & Fig. 4.

TABLE NO 1—Upstream water levels with various downstream water levels and discharge in the right and left channels during diversion stage I, i.e., Spillway in the two channels formed at R.L. 510 0 in V.E. and G.S. (in line of flow) models.

Scale of the model	Total discharge in the Mahanadi river (cusecs)	Right channel spillway block			Left channel spillway block			Remarks
		U/S gauge at 1200' U/S of axis	D/S gauge at 1200' D/S of axis	Difference in U/S. W. L s in two models (i.e., B—A)	U/S gauge at 1200' U/S of axis	D/S gauge at 1200' D/S of axis	Difference in U/S. W. L s in two models (i.e., B—A)	
A 1/200 : 1/60 .	4,00,000	529 86	519 26	..	524 88	519 14	..	(1) The D/S gauge in the left channel was first adjusted assuming 60 per cent of the total discharge passing through the left spillway block and observing the downstream gauge in the right channel. (2) The D/S gauge in the left channel was adjusted assuming 55 per cent of the total discharge passing through the left channel. (3) The D/S water levels were sufficiently low as not to effect U/S water levels.
B 1/60 : 1/60 .	4,00,000	530 70	520 26	0 84	525 84	519 36	0 96	
A . . .	5,00,000	Not recorded			Not recorded			
B . . .	5,00,000	533 40	522 96	..	528 36	521 28	..	
A . . .	6,00,000	535 62	522 86	..	529 74	522 68	..	
B . . .	6,00,000	536 16	525 48	0 54	530 82	522 68	1 08	
A . . .	7,00,000	538 30	525 78	..	532 08	525 56	..	
B . . .	7,00,000	538 62	526 80	0 32	533 04	525 66	0 96	
A . . .	8,00,000	541 02	526 86	..	534 30	527 24	..	
B . . .	8,00,000	540 84	528 46	-0 18	534 72	527 28	0 42	
A . . .	9,00,000	542 22	528 72	..	536 28	527 58	..	
B . . .	9,00,000	542 82	529 92	0 60	536 64	527 28	0 36	
A . . .	10,00,000	544 02	530 34	..	538 62	529 14	..	
B . . .	10,00,000	544 80	532 26	0 78	538 28	528 90	-0 34	



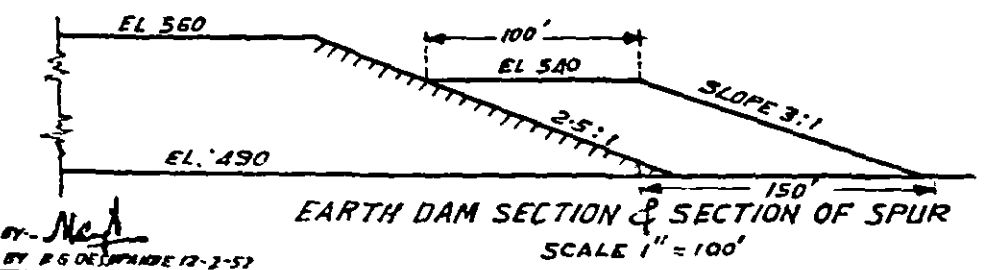
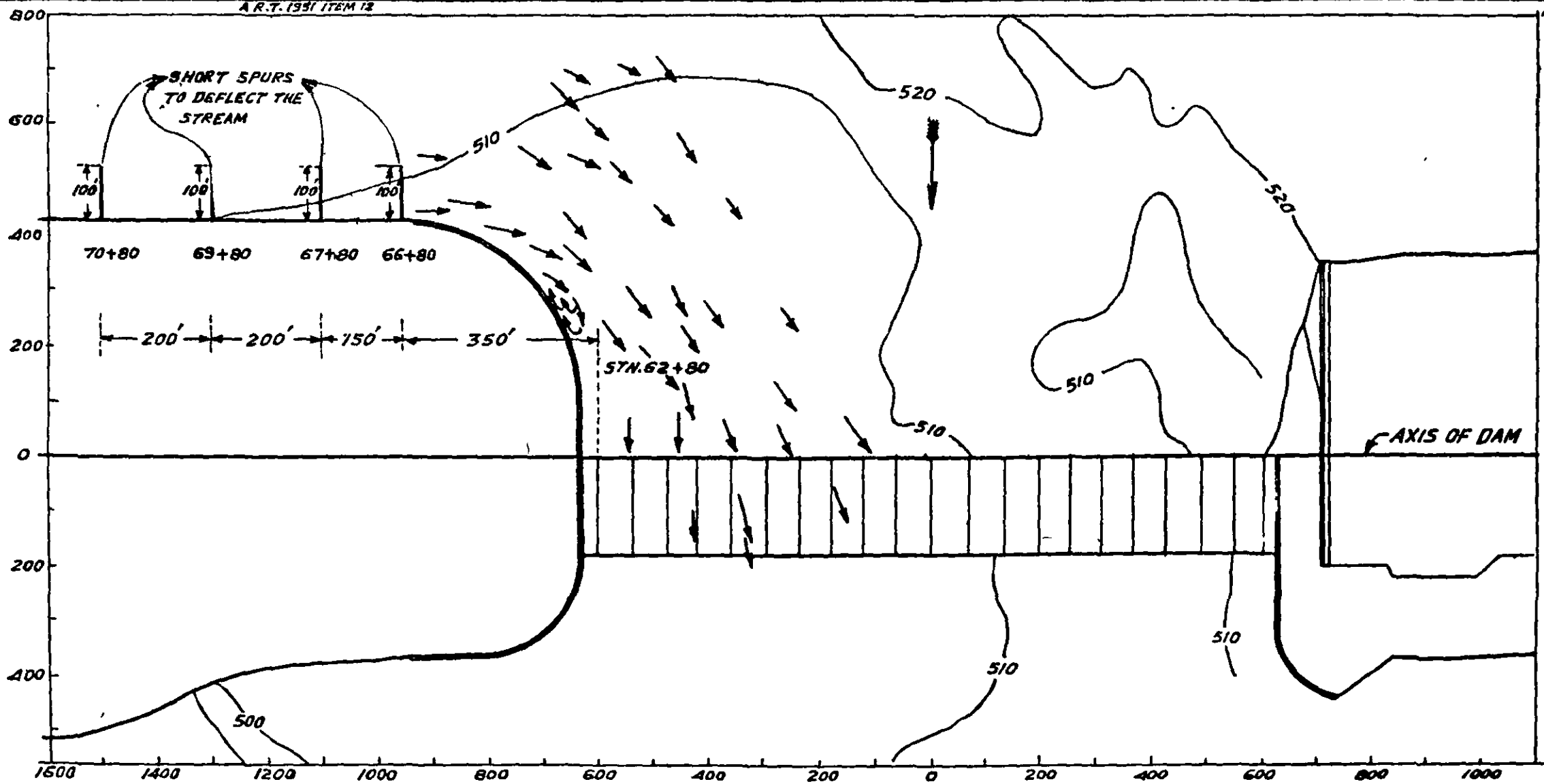
DISTRIBUTION OF DISCHARGE
BETWEEN THE RIGHT AND LEFT CHANNEL

RIGHT CHANNEL	RATIO OF Q RIGHT & LEFT CHANNEL	LEFT CHANNEL
□ GEOM: SIMILAR $Q = 4,500 H^{3/2}$ $Q_R = 2.93 L H^{3/2}$	45% : 55%	△ GEOM: SIMILAR $Q = 6,440 H^{3/2}$ $Q_L = 2.95 L H^{3/2}$
X VERTICALLY EXAG: $Q = 4,660 H^{3/2}$ $Q_R = 3.09 L H^{3/2}$	46% : 54%	○ VERTICALLY EXAG: $Q = 6,850 H^{3/2}$ $Q_L = 3.08 L H^{3/2}$

Q = TOTAL DISCHARGE
 Q_R } = DISCHARGE IN EITHER CHANNEL
 Q_L }

FIG. 3
 HIRAKUD DAM PROJECT
 1/200 : 1/60 SCALE MODEL OF MAHANADI
 FOR TESTING DIVERSION PROPOSALS
 DISCHARGE CHARACTERISTICS FOR
 SPILLWAYS DURING DIVERSION STAGE I

CORR BY - *[Signature]*
 TRACED BY - PATRICK G.P.
 11-3-51.



SCALE OF VELOCITIES
 → 10 FT./SEC
 → 20 FT./SEC.
 SURFACE VELOCITIES FROM PHOTOGRAPHIC MEASUREMENT
 7 LACS CUSECS FLOOD (WITHOUT SPURS)

FIG. 4
HIRAKUD DAM PROJECT
HYDRAULIC MODEL 5
 DIVERSION PROPOSALS-FLOW NEAR
 LEFT CHANNEL SPILLWAY BLOCKS
 SHOWING POSITION OF SPURS
 SCALE 1"=250'

COMP'D. BY - *MeA*
 TRACED BY P. G. DEWANE (12-2-57)

by heavy pitching or by a long spur or by four short spurs to deflect the stream away from the right flank. The best protection was found with four short spurs with top R.L. 540 and length about 100 feet deflecting the current away from the flank. The bed scour by using coarse gravel was significant along the curvature.

Exit from the Left Spillway—The same conditions as in the right channel including high velocity stream and hydraulic jump existed here also, but due to the straighter alignment of the left permanent bank, the stream did not attack the bank. There was, however, a considerable scouring tendency on the bed.

Afflux in a Geometrically-similar Model—With the undistorted spillway sections constructed, using geometrically-similar scale of 1·60 along the horizontal normal to the axis in the spillway portions, the afflux is given in Table 1. It will be seen that the difference with the results of the vertically exaggerated model is of an order less than one foot, the water level in the V. E model being less. This showed that the results of the vertically exaggerated model could be accepted with little modification.

Construction Stage II

In the construction stage II, the flow in the model took place partly through the constructed open spillways, and partly through the unrestricted full length of the spillway above the top level E.L. 530. The afflux was then found in the model by this combined flow. This was influenced by—

- (i) the pool elevation differences at the right and left channel spillways,
- (ii) the friction loss in the model due to the narrow sluices, and
- (iii) the extra drawdown effect due to the vertical exaggeration (1/200·1/60).

Pool Elevation Difference Between the Left and Right Channels—In terms of distance down the deep sections of the left and right channels, the left channel spillway is about 4,000 feet further down the river than the right channel spillway. For computation purposes, in this stage, assuming a velocity of the order of 15 ft./sec., with pool elevation of the order of E.L. 550, assume in Manning's formulae.

$$V = \frac{1.486}{n} R^{1/6} (RS)^{1/2}$$

$n=0.025$ suitable for the rocky bed at Hirakud. This is the value obtained for the Tapi river in flood at Kakrapar, the bed consisting of rock outcrops. With R of the order of 40 feet, slope $S=0.00047$, i.e., about half the water slope at maximum flood in the river at site as it is today. The pool elevation differences between the two spillway sites in stage II, would then be $0.00047 \times 4000 \text{ ft.} = 1.9$ feet approximately with a larger value of n , say $n=0.030$, this difference = 2.7 feet approximately.

This order of difference agrees well with the results in the test.

Order of Friction Loss in the Model—As the spillways are represented at this stage of construction, they form a parallel series of clear waterway of minimum width 0.72" each (=12 ft. prototype).

The hydraulic mean radius for each open sluiceway together with 10 feet (prototype) depth of water above, is

$$(1.8 \times 2 + 0.72 \times 4) \div (2 \times 4 + 1.8) = 0.66 = 0.055'$$

The velocity being of the order of 3 ft./sec. in the model flood the Reynolds number is of the order of

$$3 \times 0.055 / 1.2 \times 10^{-5} = 14,000$$

for flow through each open sluiceway.

This is still sufficiently above the critical Reynolds number, but there would exist some scale effect, due to which the afflux as expected in the prototype would be somewhat less than that indicated in the model.

The order of losses due to the small open sluiceways in the model would be of the order of one foot (prototype) thus:

$$\text{Take the head loss as } hf = \frac{fLV^3}{gR}$$

where f =friction coefficient, and L =length along sluiceway. For the case under consideration $V=3$ ft./sec., $R=0.055$ ft., $L=0.5$ ft. The friction coefficient in terms of Manning's 'n' is

$$f = g \left(\frac{n}{1.486 \times R^{1/4}} \right)^2 = 0.0031 \text{ or } 0.0055 \text{ with } n=0.009 \text{ and } 0.012 \text{ respectively}$$

which are good values for smooth wooden surface of the model sluiceway. Then

$$hf = \left. \begin{array}{l} 0.008 \text{ ft} \\ 0.014 \text{ ft} \end{array} \right\} R.O. \text{ model} = 0.48 \text{ ft. and } 0.84 \text{ ft prototype.}$$

The test results obtained from the model would therefore require slight modification from this aspect.

Discharge Capacity of the Combined Flow Through the Sluiceway and Above E.L. 530—The clear waterway through the open sluiceways only, as measured by the gated area is $2 \times 12 \times 20$ sq ft. per spillway block of 60 ft., i.e., 8 sq. feet per foot length of the spillway. With a depth of flow d feet of the spillway above E.L. 530 the discharge of the right channel spillway block = $690 \times (d+8) \times \sqrt{2gh}$ cusecs if $\sqrt{2gh}$ is taken as a mean velocity. The maximum value of the discharge, which correspond to the case of flow over a broad-crested weir would be given when

$$d+8 = 2h' = 2/3 (\text{E.L. of pool} - \text{E.L. 522}) \text{ ft}$$

so that the corresponding discharge would be

$$3.09 \times 690 \times (\text{E.L. of Pool} - \text{E.L. 522})^{3/2} \text{ cusecs.}$$

Taking the left channel spillway pool two feet lower, the total flood would be related to EL_R , the pool, elevation at the right channel by

$$Q = 3.09 \times 690 \times (\text{E.L.}_R - \text{E.L. 522})^{3/2} + 3.09 \times 1200 \times (\text{E.L.}_R - 2 - \text{E.L. 522})^{3/2} \text{ cusecs.}$$

The actual afflux would be slightly lower than what would be given by this relation. The reason is that the drawdown effect is present in the model as well as in the prototype, as the $B:D$ ratio is about 2.5 only in prototype, and 0.75 in model, for rough values of $B=100$ feet, $D=40$ feet prototype.

The afflux and the upstream pool elevation as obtained thus, and as obtained from the test results are given in the Table 2 and Figure 5. It will be seen that the difference between the two sets of results is two feet only, which therefore gives the estimate for the drawdown effect.

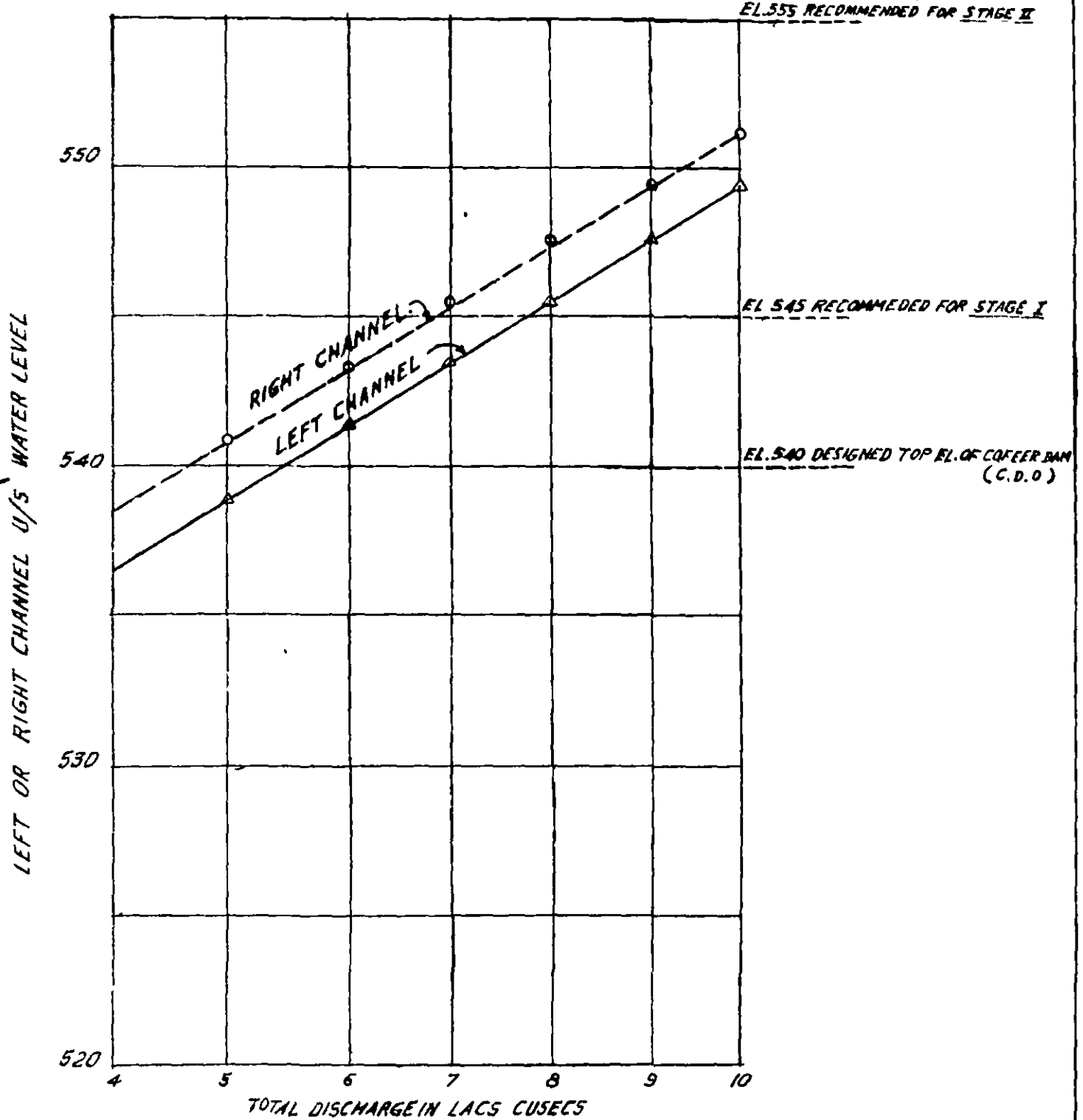
For the 9 lakh cusecs flood, upstream pool E.L. right channel = 549.8 model test, and 552 estimated without drawdown.

The coffer dam top E.L. which, during construction stage I, should be at E.L. 545 is recommended for the second construction stage to be raised to E.L. 552 or to R.L. 555 with a free-board of 3 feet.

6. Recommendations

(i) For safety against a 9 lakh cusecs flood, the coffer dam top elevation designed at E.L. 540 should be raised to E.L. 545 for first construction stage (E.L. 510). For second construction stage (construction to E.L. 530), a height up to E.L. 555 is required.

(ii) For the left spillway, the right flank would need protection by a series of four short spurs each about 100 feet long.



UPSTREAM POOL ELEVATIONS FOR
DIFFERENT FLOOD DISCHARGES
IN THE MAHANADI DURING CONSTRUCTION
STAGE, II L.B. SPILLWAY FORMED TO EL. 530

FIG. 5
HIRAKUD DAM PROJECT
1/200 : 1/60 SCALE MODEL OF THE MAHANADI
FOR
TESTING DIVERSION PROPOSALS
DISCHARGE CHARACTERISTICS DURING STAGE, II

COMP BY *Murphy*
TRACED BY: NAIK R.S 23 11 51

TABLE No. 2—Upstream water levels with various downstream water levels and discharges in the right and left channels during diversion stage II, i.e., spillway in the two channels formed up to R.L. 530 and comparison with estimated afflux.

Total discharge in the Mananadi river (cusecs)	Right Channel Spillway Block				Left Channel Spillway Block				Remarks
	U/S gauge at 1200' U/S of axis	D/S gauge at 1200' D/S of axis	U/S gauge computed without draw-down effect	Difference due to drawdown	U/S gauge at 1200' U/S of axis	D/S gauge at 1200' D/S of axis	U/S gauge computed without drawdown effect	Difference due to drawdown	
4,00,000	540.0	538.0	..	No model test was made with geometrically-similar scale
5,00,000	540.96	521.70	542.6	+1.6	538.86	521.40	540.6	+1.7	
6,00,000	553.30	524.40	545.4	2.1	541.38	523.56	543.4	+2.0	
7,00,000	545.52	525.84	547.5	2.0	543.54	525.30	545.5	+2.0	
8,00,000	547.62	527.94	549.8	2.2	545.58	526.93	547.8	+2.2	
9,00,000	549.42	528.60	552	2.6	547.62	527.46	550.0	+2.4	
10,00,000	551.22	530.04	549.48	529.02	

(iv) FOUNDATION

13. GEOPHYSICAL INVESTIGATION AT THE ALTERNATIVE DAM SITE AT UKAI ON RIVER TAPI

1. At the request of the Superintending Engineer, Tapi Construction Circle following the suggestions made by Dr. J. B. Auden, Superintending Geologist, Geological Survey of India, geophysical investigations were undertaken at the alternative dam site at Ukai, $16\frac{1}{2}$ miles upstream of Kakrapar for finding out the bed-rock level there along the proposed alignment thus supplementing bore hole results. The work was conducted during the latter half of June 1951. Due to short time that was available for work before the rains set in the proposed geophysical study for the other two alternative sites namely at Tintaria and Panchpipla could not be completed to provide a comparative study for all the three sites. The results for the Ukai site only are presented here.

2. The location map for the Ukai site (Lat. $21^{\circ} 15'$, Long. $73^{\circ} 35\frac{1}{2}'$) is given in Figure 1. The local geology for all the three sites has been described by Dr. Auden in his recent report on "Geological note on alternative dam sites on the Tapi (Tapti) river, Bombay State" of June 1951. Lava flows intersected by dykes form the bed-rock over which there is varying thickness of silts on either banks. At Ukai site no rock is exposed in the north bank for a distance of 1,470 feet up to 220 feet contour and the silt cover there was thought to be thick whereas intermittent rock exposures on the south bank would make it likely that a much thinner silt cover occurred on that bank. At the Serula saddle (see Figure 1) absence of any rock exposure for more than a half a mile of the flat ground made also a geophysical investigation desirable there. Electrical method of sounding for the bed-rock level was found to be suitable because of the existing good electrical resistivity contrast between the overburden and the bed-rock namely a resistivity value of 30 to 70 ohms ft. for silt overburden as against a value of 600 to 1,000 ohms ft. for the underlying trap rock. Figure 2 shows the contour plan and the electrical sounding stations along the alignment across the river. It also shows a 5 ft. contour map ($32''=1$ mile in original map) for the north bank which provides the reduced levels of the electrical sounding stations. Figure 3 shows the longitudinal section showing the bed-rock level as obtained by electrical soundings. It will be seen that whereas the maximum thickness of silt on the left bank is only 48 ft. and occurs near the bank, the maximum thickness of silt on the right bank which also occurs near the bank is about 110 ft. Figure 4 shows the bed-rock level as obtained by electrical soundings in the Serula saddle for which the full cross-section is shown in Figure 5. Figure 6 gives the cross-section of the river showing the results of two bore holes and that of the penetrometer tests. In the river section proper, only one electrical sounding in the river water at Station No. 4 at R. D. 700 ft. (see Figures 3 and 6) was taken near the unsuccessful bore hole and R. L. of the bed-rock from the electrical soundings is estimated at 115 ft. It is therefore presumed that the diamond drill would have met with solid rock there if it could have proceeded a few feet further. At R. D. 900 ft. the diamond drill reached solid rock at R.L. 111.2 ft. after penetrating to a depth of 25 ft. of river sand. No more bore hole results are yet available but the penetrometer tests taken up to R.D. 1,200 ft. shows the lowest tentative rock level there at R.L. 111.5 ft. with a further downward trend towards the right bank. Just on the right bank, at R.D. 1,365 ft., the rock-level is estimated at R.L. 97 ft. from electrical sounding No. 5 (see Figure 3), afterwards it rises as one proceeds northwards. For the Serula saddle Figure 4 shows the average bed rock level at R.L. 235 ft. Figure 5 shows the cross-section for the Serula saddle as also the rock-level there as estimated from electrical soundings.

3. An "Evershed and Vignoles" Geophysical Megger was employed in the above electrical soundings using a Wenner configuration for the spreading electrodes. The electrode spacing was expanded in steps from 5 ft. to 300 ft. or more where necessary in a East-West direction nearly parallel to the river flow. The apparent resistivity in ohms-ft. (which is given by $2sR$, where 's' is the electrode spacing in feet and 'R' is the resistance measured for that particular electrode spacing in ohms) is plotted versus the electrode spacing in a Logarithmic paper and the field curve so obtained is matched with the best fitting theoretical curve thus giving a most probable depth 'h' for the overburden. A theoretical representation assuming two layers—namely an electrically homogenous overburden of silt of resistivity P_1 lying over a Deccan-trap bed-rock of resistivity P_2 extending very deep satisfies most of the field curves. Resistivity of the overburden is known from the

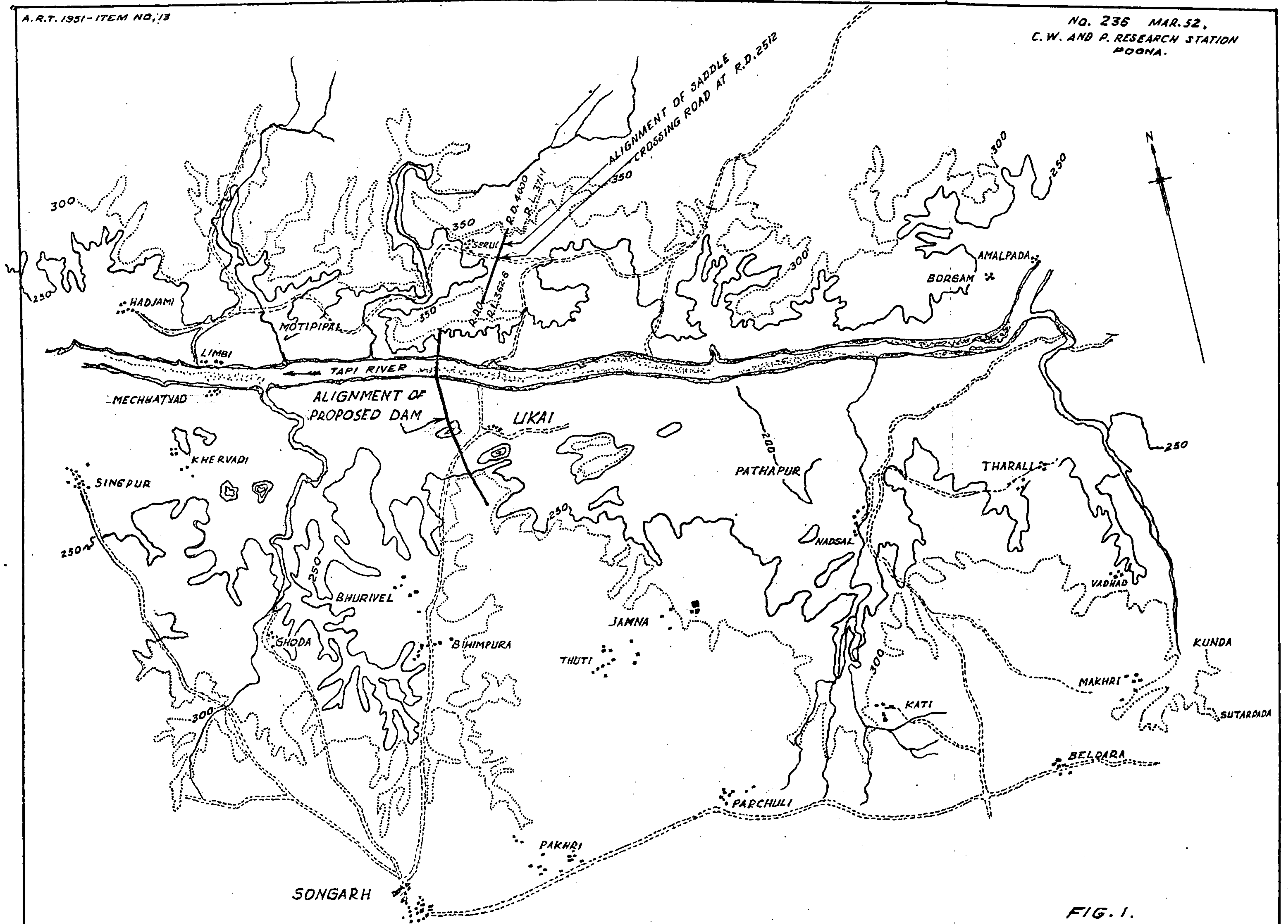


FIG. 1.
SHOWS CENTRE LINE
OF
PROPOSED DAM ACROSS TAPI
NEAR UKAI
SCALE: - 1" = 1 MILE.



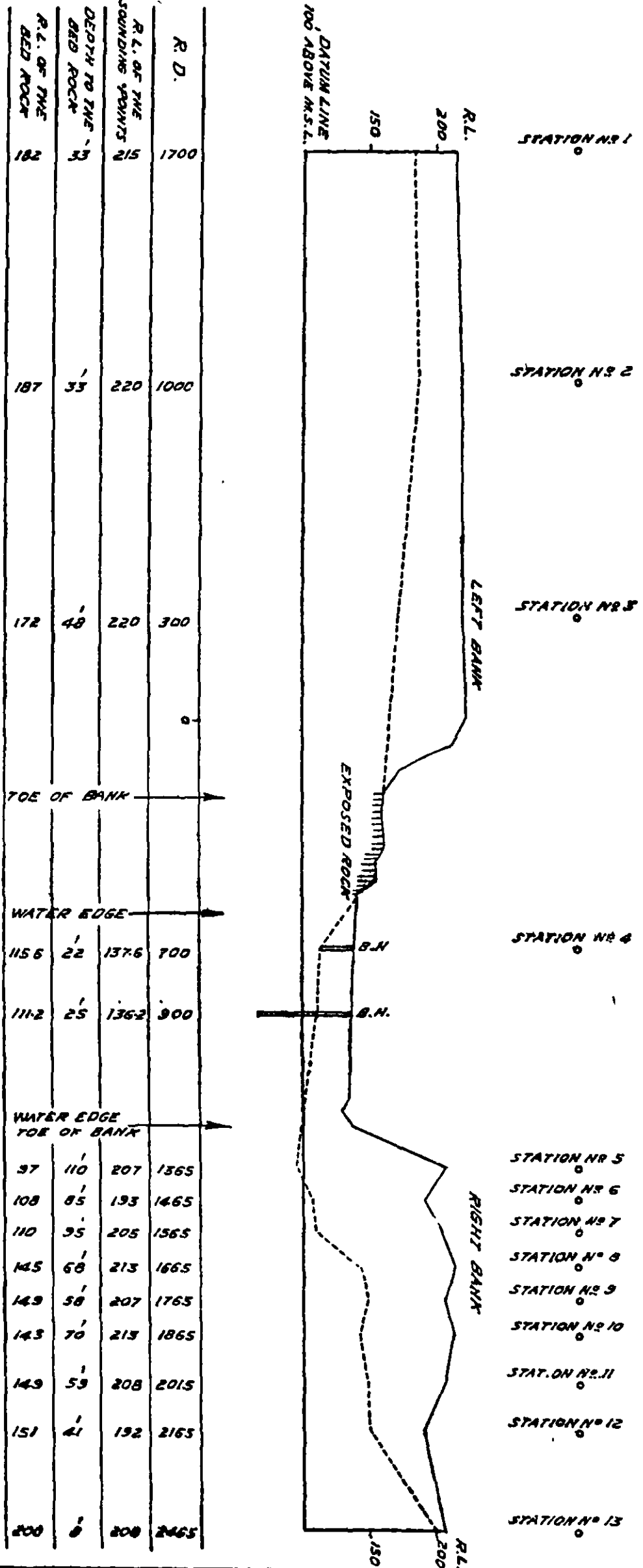


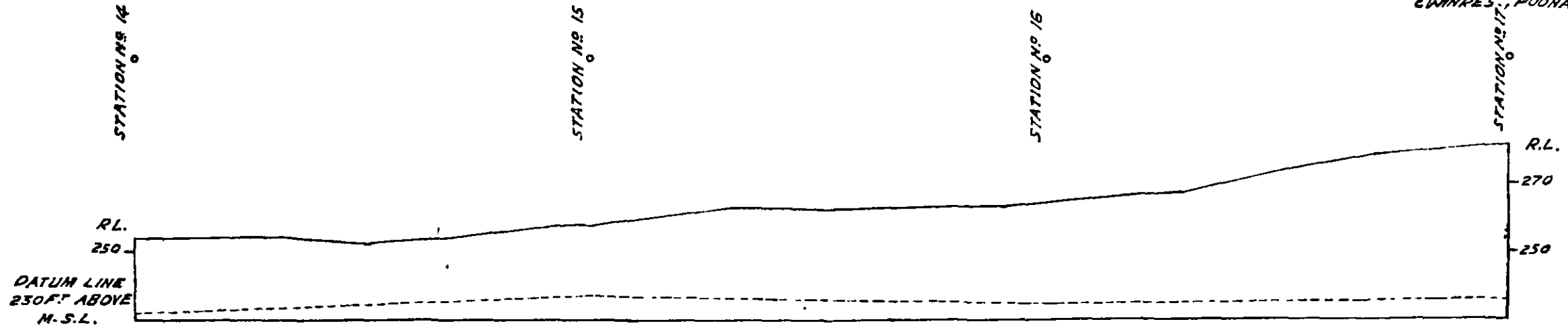
FIG. 3
LONGITUDINAL SECTION ALONG THE ALIGNMENT
ACROSS RIVER TAPI AT UKAI
SHOWING THE GEOPHYSICAL SOUNDING STATIONS
AND CORRESPONDING BED ROCK LEVELS

COMP. BY - *[Signature]*
TRACED BY - *[Signature]* G.R. 27-B-51.

SCALE { HOR. 1" = 400 FT.
VER. 1" = 100 FT.

A.R.T 1951 - ITEM NO.13

Nº 979 OF AUG 1951
 CWINRES., POONA.



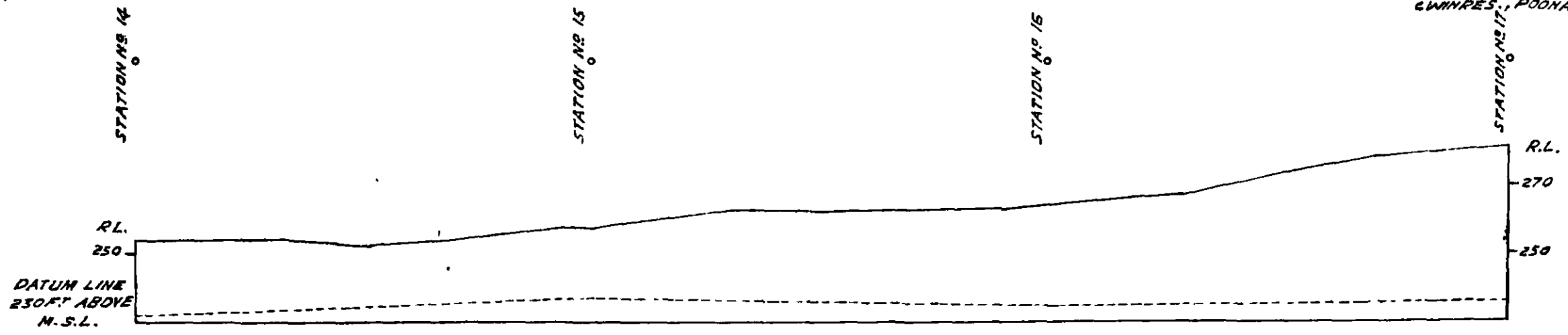
R. D.	2050	2550	3050	3550
R.L. OF THE SOUNDING POINTS	254	257	264	281
DEPTH TO THE BED ROCK	22	20	29	45
R.L. OF THE BED ROCK	232	237	235	236

FIG. 4
 LONGITUDINAL SECTION ALONG THE ALIGNMENT
 IN THE SADDLE PORTION ON THE NORTH BANK OF
 RIVER TAPI AT UKAI
 SHOWING THE GEOPHYSICAL SOUNDING STATIONS
 AND CORRESPONDING BED ROCK LEVELS
 SCALE { HOR. 1" = 150 FT.
 VER. 1" = 40 FT.

COMP. BY - *Neil*
 TRACED BY - PATRIE & R. 24-B-51.

A.R.T 1951 - ITEM NO.13

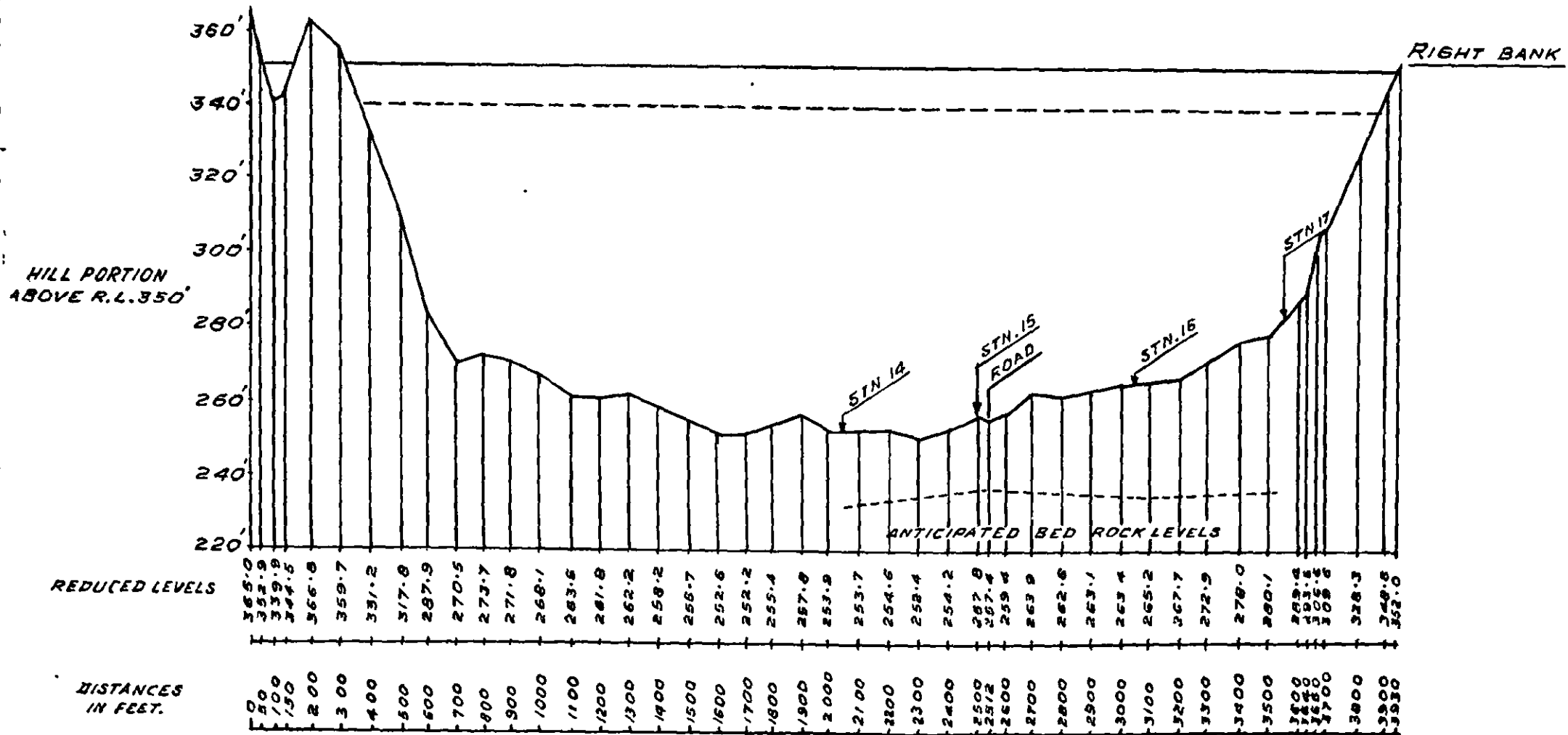
No. 979 OF AUG. 1951.
LWINPES., POONA



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 SHOWING THE GEOPHYSICAL SOUNDING STATIONS
 AND CORRESPONDING BED ROCK LEVELS
 SCALE { HOR. 1" = 150 FT.
 VER. 1" = 40 FT.

COMP. BY - *Neil*
 TRACED BY - PATKIE G.R. 26-B-51.



GAP PORTION FOR R.D. 0 TO R.D. 3900 = 3900'
 LESS HILL PORTION R.D. 0 TO R.D. 50 = 50'
 " " " 150 TO 350 = 200' } $\frac{250'}{3650'}$ NET

SADDLE PORTION ON NORTH BANK OF UKAI SITE

FIG. 5
 L. SECTION OF DAM
 AT
 UKAI VILLAGE FOR
 PROPOSED DAM SITE.
 SCALE: HOR. 1" = 500'
 VER. 1" = 40'

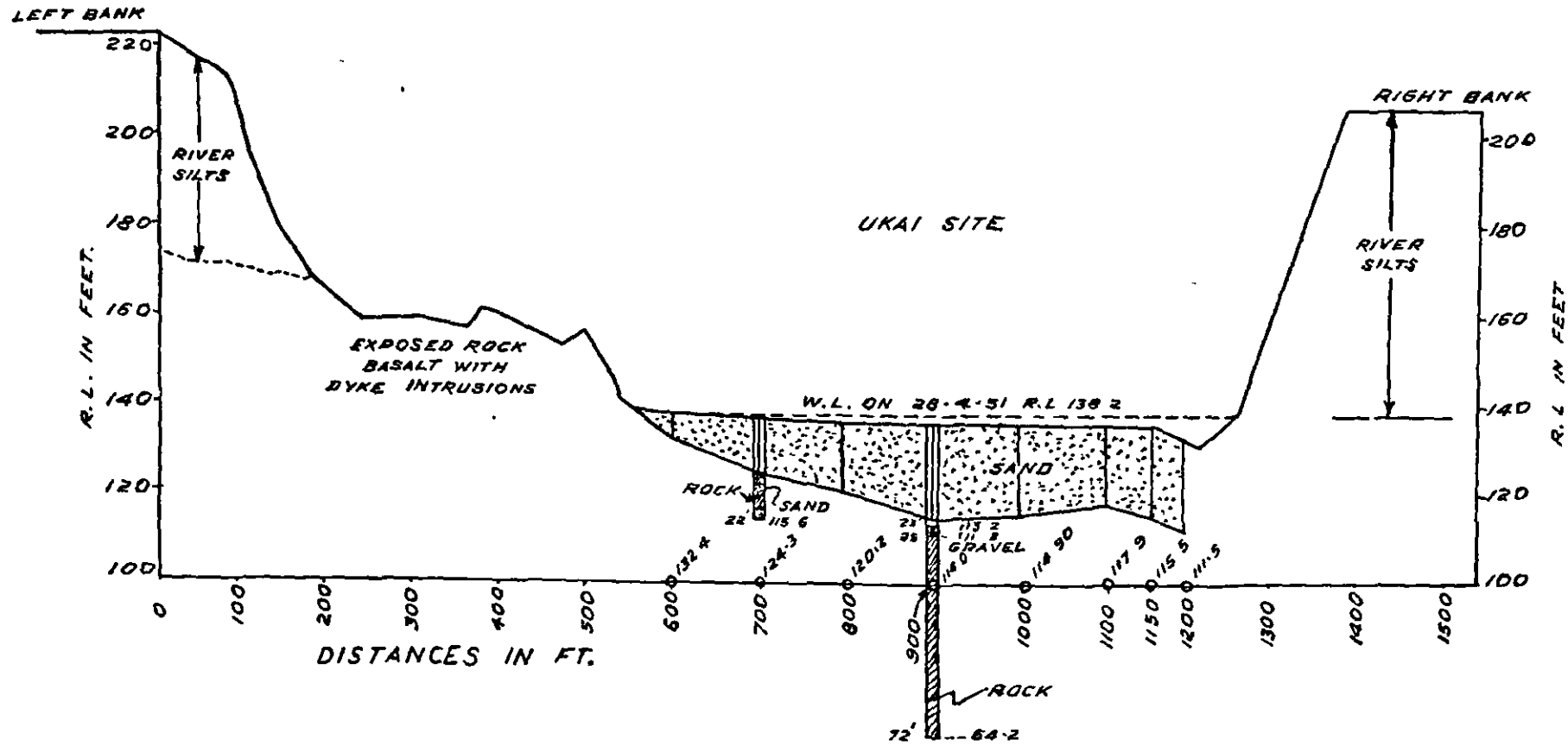
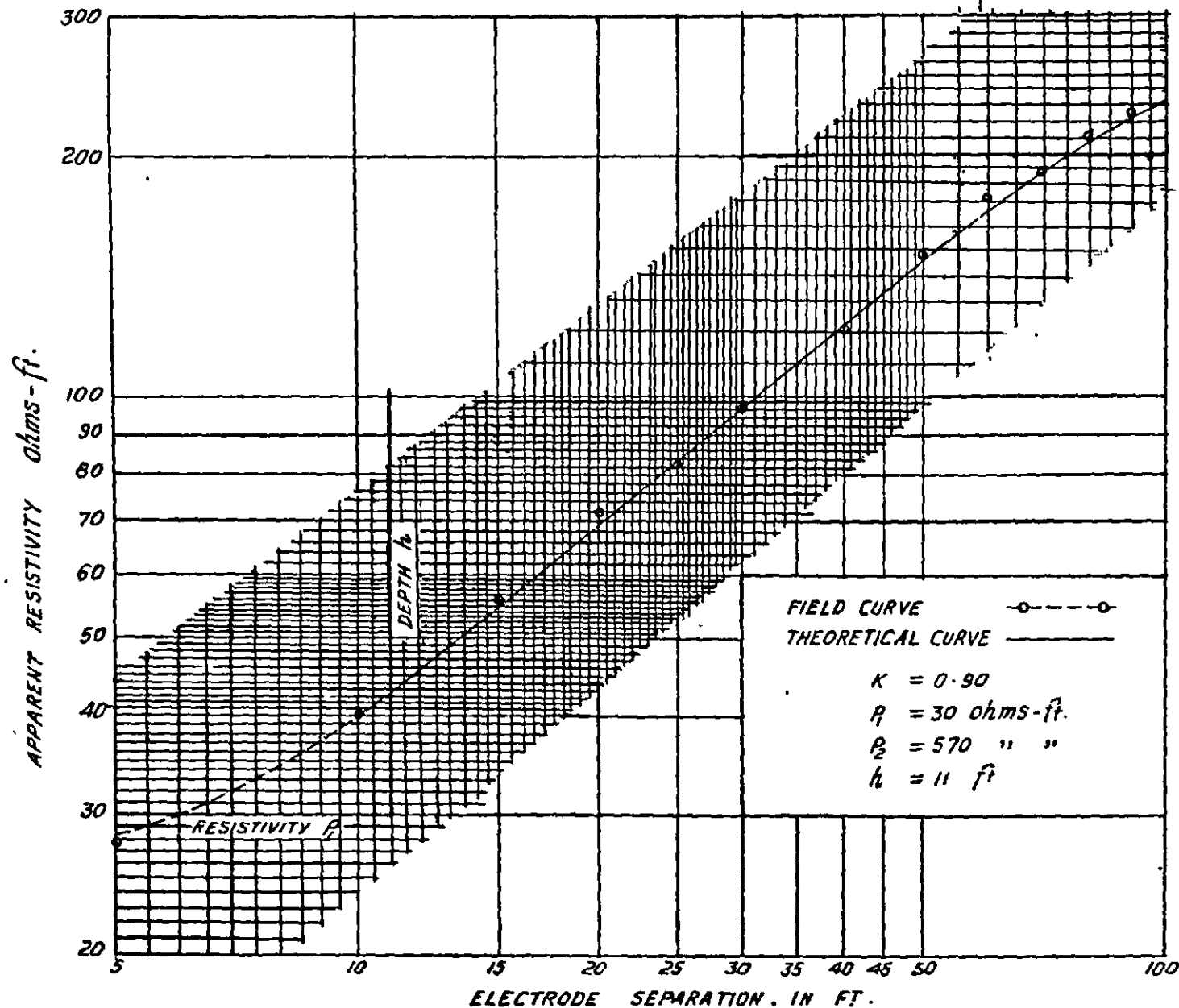


FIG. 6
RESULTS OF PENITROMETER TESTS
& OF DIAMOND DRILLS IN SANDS OF
THE TAPI RIVER.

SCALE: ~ {
HORIZ. 1" = 200'
VERT. 1" = 40'

COMP BY: *Maugh*
T BY: PARKHI
23-2-52



DESCRIPTION OF 15 FT DEEP TEST PIT AT
UKAI VILLAGE NEAR THE STATION NO. 13

BLACK COTTON SOIL	5 FT
BROWN SOIL	6 FT
BROWN SOFT MURUM	2 FT
GREY HARD MURUM	2 FT

FIG. 7
ELECTRICAL SOUNDING
STATION NO. 18

apparent resistivity obtained for short electrode spacings. Approximately known probable range for the resistivity of Deccan-trap limits the value for the reflection factor 'K' (i.e., the ratio of the difference in resistivities of the overburden and the bed-rock to their sum). The matching of the field curve with the theoretical curves for each of the 19 electrical soundings taken at this site provided the estimated depth 'h' of the overburden in each case (see Figures 3 and 4). A typical matching of a field curve with a theoretical one is shown in Figure 7 which was only a trial electrical sounding taken near a pit in a cultivated field located just east of the Ukai village. This pit was dug in connection with a soil survey at this site. The depth of the overburden here as estimated for the electrical sounding No. 18 (see Figure 7) corresponds well with the observations in the pit.

Fig. 7.

5. DAM APPURTENANCES

(iii) CONDUITS AND PENSTOCKS

14. DISCHARGE CHARACTERISTICS AND DISSIPATION OF ENERGY WITH CONVENTIONAL AND UPTURNED DEEP SEATED SLUICES—HIRAKUD DAM PROJECT (1/20 SCALE GEOMETRICALLY-SIMILAR MODEL)

(A) Discharge Characteristics and Pressure Distribution

THIS report refers to hydraulic tests in connection with some additional characteristics of design in the Hirakud Dam Project—Hirakud main dam. In connection with some major design changes in 1951, model tests additional to what have already been reported in Annual Report (Technical) 1950 were carried out during the year 1951. In the new lay-out, conventional straight type sluiceways with elliptical bellmouth transitions and outside trajectory bucket are incorporated in the whole of the right channel spillway and about two-thirds of the left channel spillway. The remaining one-third of the left channel spillway is to consist of sluiceways of the upturned barrel type (Figure 1)

Fig. 1.

Preliminary pressure tests on the design for the conventional sluices supplied by the Central Designs Organisation, C. W. & P. Commission were made in a model (scale 1:40). The final confirmatory tests were made for different types of sluiceways in the 1:20 scale model in the flume which has been used for similar tests reported in Annual Report (Technical) 1950. The sluiceways tested were (Figures 2 and 3)—

Figs. 2 & 3.

- (i) conventional sluices supplied by the C. D. O, C. W. & P. C.—Design A;
- (ii) same as (i) only with elliptical bellmouth transition replaced by I. E. C. type of bellmouth used in tests reported in A. R. T. 1950—Design B; and
- (iii) sluiceways with upturned barrel type with and without flared outlets—Design C.

The sill level of the sluices in each case was E.L. 510, the general upstream bed level being E.L. 500. The crest level of the dam was E.L. 610. The gated area 26 ft. down from the upstream face of the dam was 20 33 ft. × 12 ft. Each design was tested both with and without overflow for upstream water level ranging from E.L. 590 to E.L. 630. For pressure and discharge test in each case it was found sufficient to test under free flow conditions downstream.

The typical pressure profiles in the different sluices under different conditions are shown in Figures 4 to 8 as obtained from the 1/20 scale model. The main features of the pressures are the following.

Figs. 4 to 8.

Pressures in the Bellmouth

(i) These were generally positive; the exception was the case of design B (I. E. C. bellmouth), where a little upstream of the gate, it was just below atmospheric to the extent of about 2 ft.

(ii) The pressures in the bellmouth top in the upturned sluiceway (Design C) were higher than in the other two sluices, corresponding to raised outlet. For design A, the pressure curve fell smoothly after the first 5 ft. from entry. The pressures in the bellmouth top became uniform quite quicker in design B, so that for the most part, these pressures were lower than those corresponding to either design A or design C.

(iii) Due to the high curvature at entry, for most cases there was higher velocity just at entry which was quickly decelerated within the first few feet, followed by an acceleration thus lowering the pressure curve beyond. If we take into account

the fact that from the face of the dam up to the beginning of the transition, the stream must be accelerated, the pressure curve within the first few feet takes the shape of a rapidly falling curve and then a rising curve, which afterwards falls smoothly to the value at the gate. This characteristic is present in various degrees in most of the top and side pressure curves, the only exception being the top pressure curves for design B. To that extent, the design B of the bellmouth (I. E. C. design) is superior to the more simple elliptical transition of design A, in which this characteristic is very pronounced.

(iv) An initial retardation of flow at entry contributes to a small loss of head, the order of this extra loss, however, is negligible. As the model becomes smaller, friction becomes proportionately more important, which might lead to a slight separation, and this would lower the pressure curve within that range, and would be followed by a fluctuation leading to the rest of the pressure curve in the bellmouth. This could be a contributory cause of this nature of fluctuation observed in the model with scale 1:40 (Figure 9). Without further tests, however, the behaviour of smaller scale models cannot be examined in more detail as regards the friction effect and the degree of accuracy involved.

Pressures in Barrel

(i) For designs A and B which differ only in the bellmouth, the pressures in the barrel were nearly the same and practically all positive.

(ii) For the pressure curve in design C, a minimum of about +5 ft. has been kept in view and attained in the observed values. The pressure curve could be brought down to the same margin of positive pressures as in designs A or B, had it been desired, by requisite slight modification in the outlet.

(iii) Pressures in design C at the outlet were all positive when the outlet was flared, and were practically so when the outlet was unflared except for a slight negative pressure of the order of -2 ft. on the top just near the outlet (Figure 8).

Pressures with Overflow Discharge Present

With overflow discharge present, when upstream water level was above R.L. 610, the pressure curves were all raised corresponding to a back pressure at the outlet due to the deflection of the downcoming jet from the crest. This effective diminution of head for flow through sluiceway was of the same order as the head over the crest. The reduction of discharge due to this was about 10 per cent of the sluiceway discharge at upstream R.L. 625.

Coefficient of Discharge

The coefficients of discharge of the sluiceways were calculated against the gate area, which in all the three designs was the same. For any given reservoir level, the head H as defined by the elevation difference between the reservoir level and the centre of the outlet was the same for designs A and B but less for design C. The values of the coefficients are:

$$C_d = 0.88 \text{ for designs A and B;} \\ \text{and } C_d = 0.86 \text{ for design C;}$$

within about ± 1 per cent, when the sluices were acting singly without any overflow discharge (Figure 10). Keeping in view the fact that the upturned sluiceway was designed to have a little safer pressure, a general minimum of +5 ft., the three designs can be said to have about the same coefficient.

The overflow spillway coefficient (Figure 11) varied from about 3.5 to 4.0.

Discharges

Discharges in the three sluiceways when operated singly without overflow are shown in Figure 10. For combined overflow and undersluice discharge as we have noted before, the discharge was less than when they worked independently. Since however, the discharging capacity of the overflow spillway increases much more rapidly with head than in the case of the sluiceways, this lessening of discharge can be accommodated by a slight increase in the head only.

Recommendations

The sluiceways with design A (CDO type) as well as the upturned sluiceways design C can be accepted as satisfactory from the point of view of absence of sub-atmospheric pressures.

ART. 1951 ITEM 14

NO 136 OF FEB. 1952
C W & P S. POONA

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C W & P C

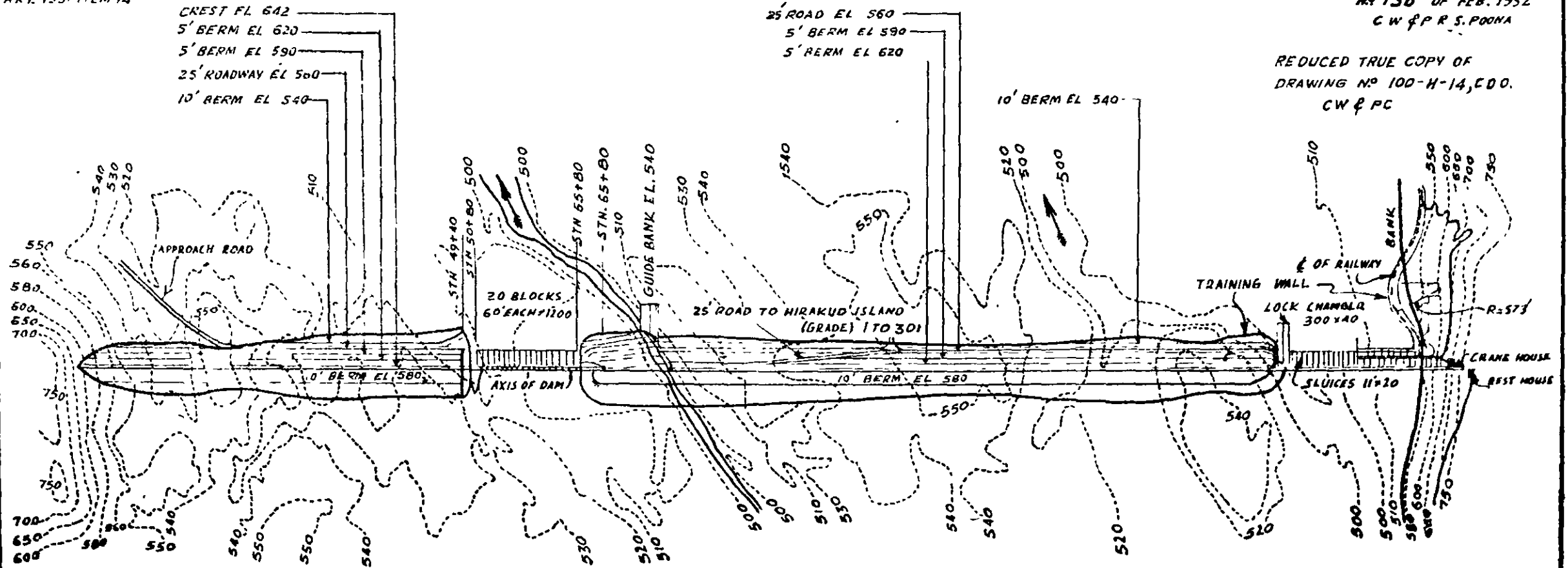
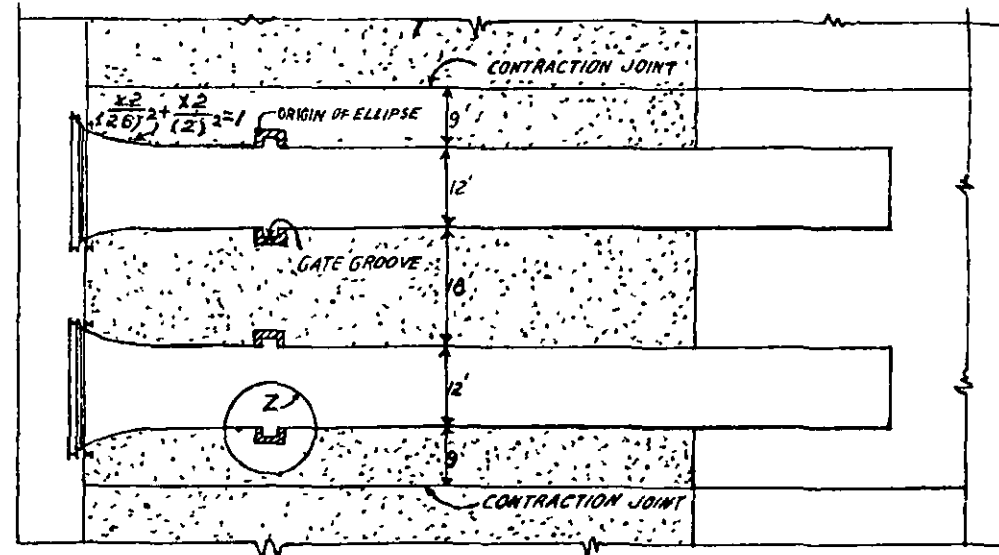
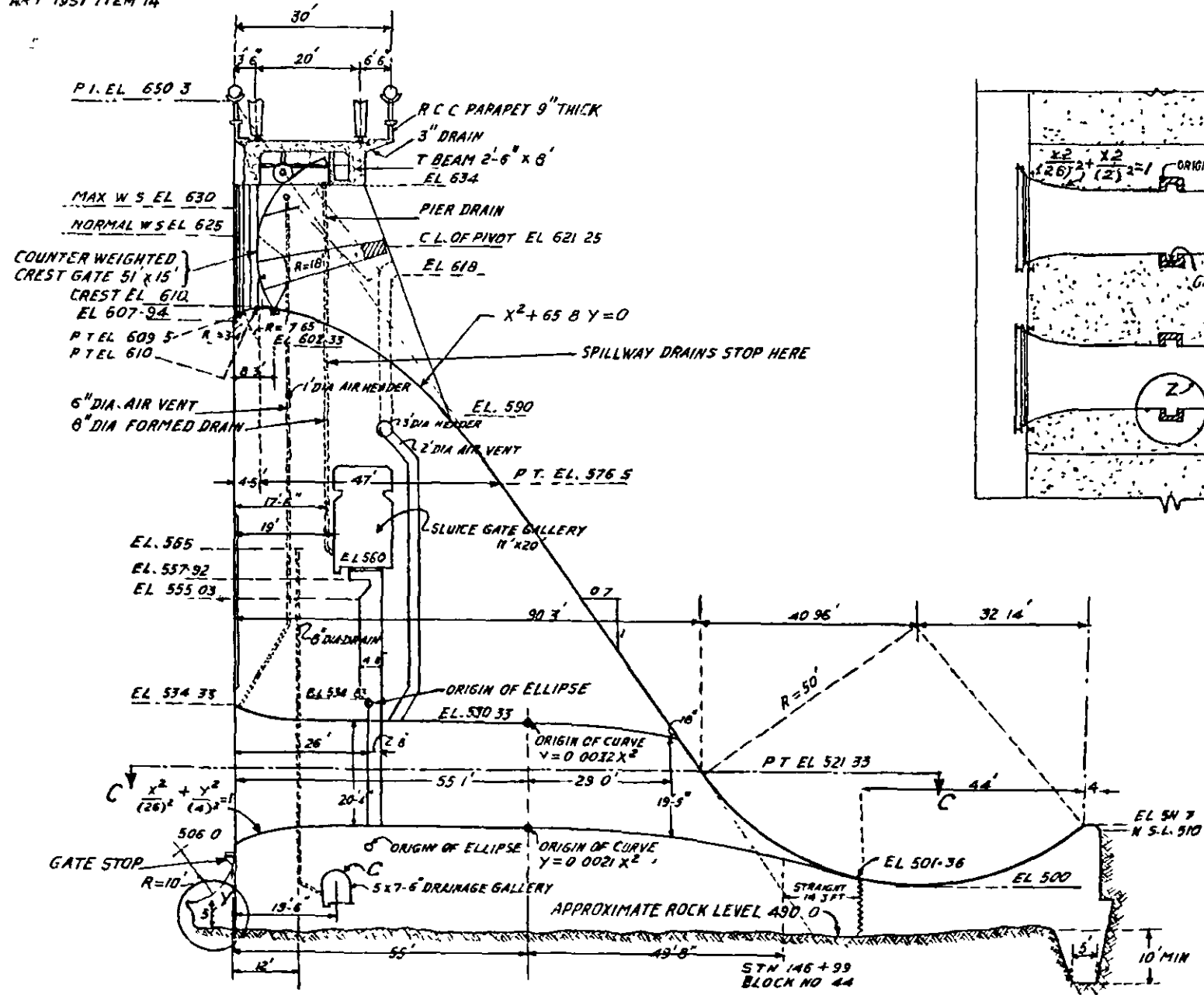
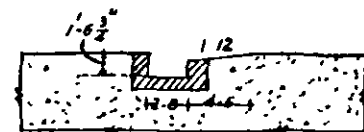


FIG. 1
HIRAKUD DAM PROJECT
GENERAL PLAN
OF
MAIN DAM
SCALE 1" = 1500'

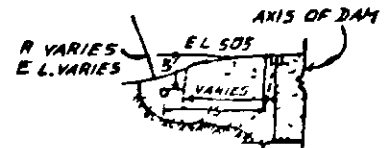
COMPD BY: *M.A.*
TRACED BY: BHINJARE & DESHPANDE S-2-51



VIEW CC



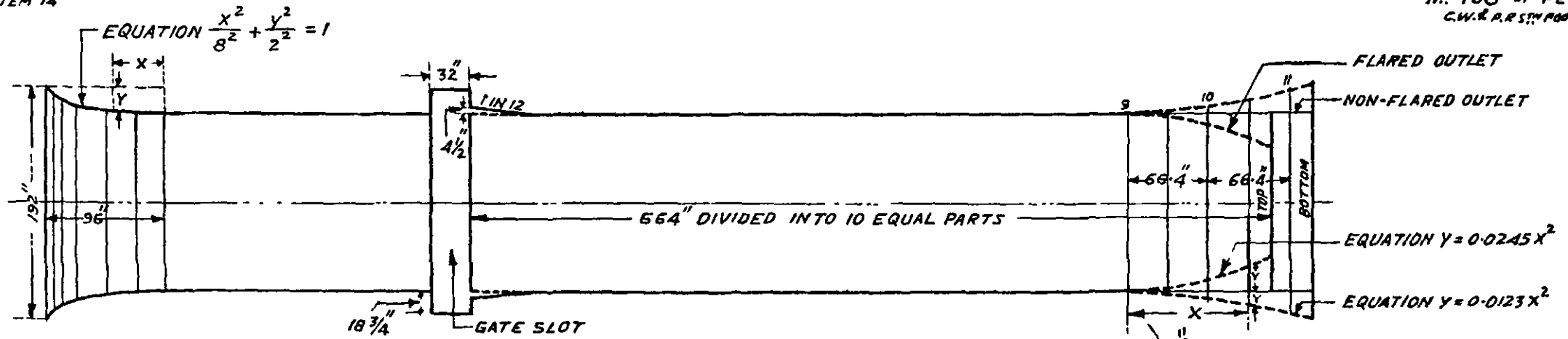
DETAIL AT Z
SCALE 1" = 15'
MANWAY FROM SLUICE BLOCK NO 9 16 44.



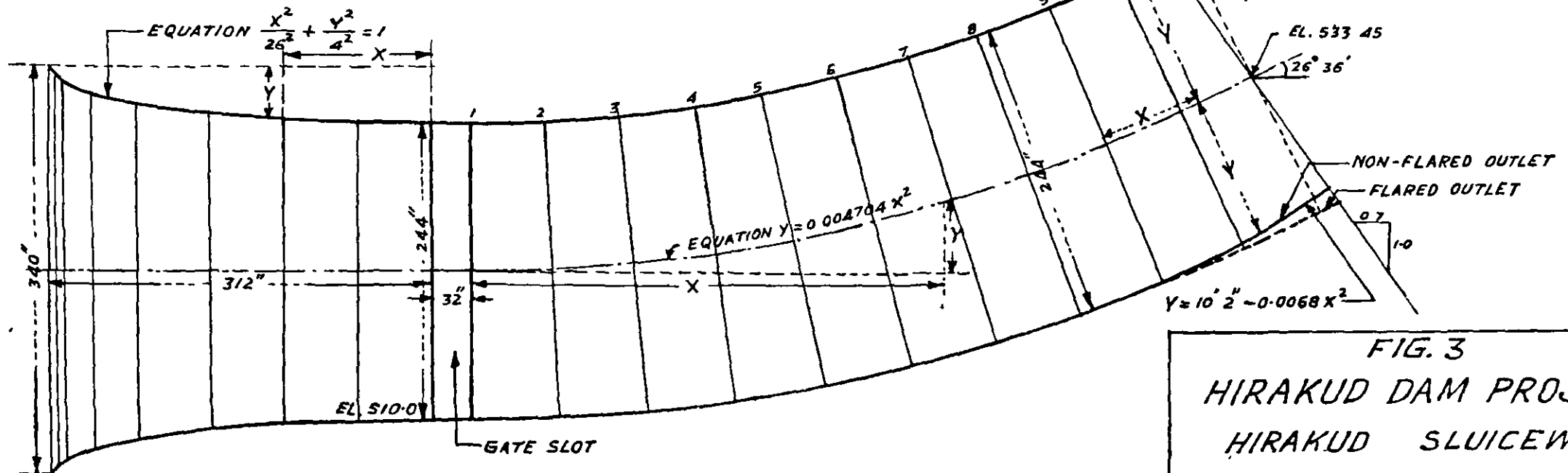
DETAIL AT Y
WHEN ROCK LEVEL IS EL 500
SCALE 1" = 30'

FIG.2
HIRAKUD DAM PROJECT
MAIN DAM
SPILLWAY SECTION
SCALE, 1" = 30'

COMP BY *M. R.*
TRACED BY MAIN R/S 4.1.52



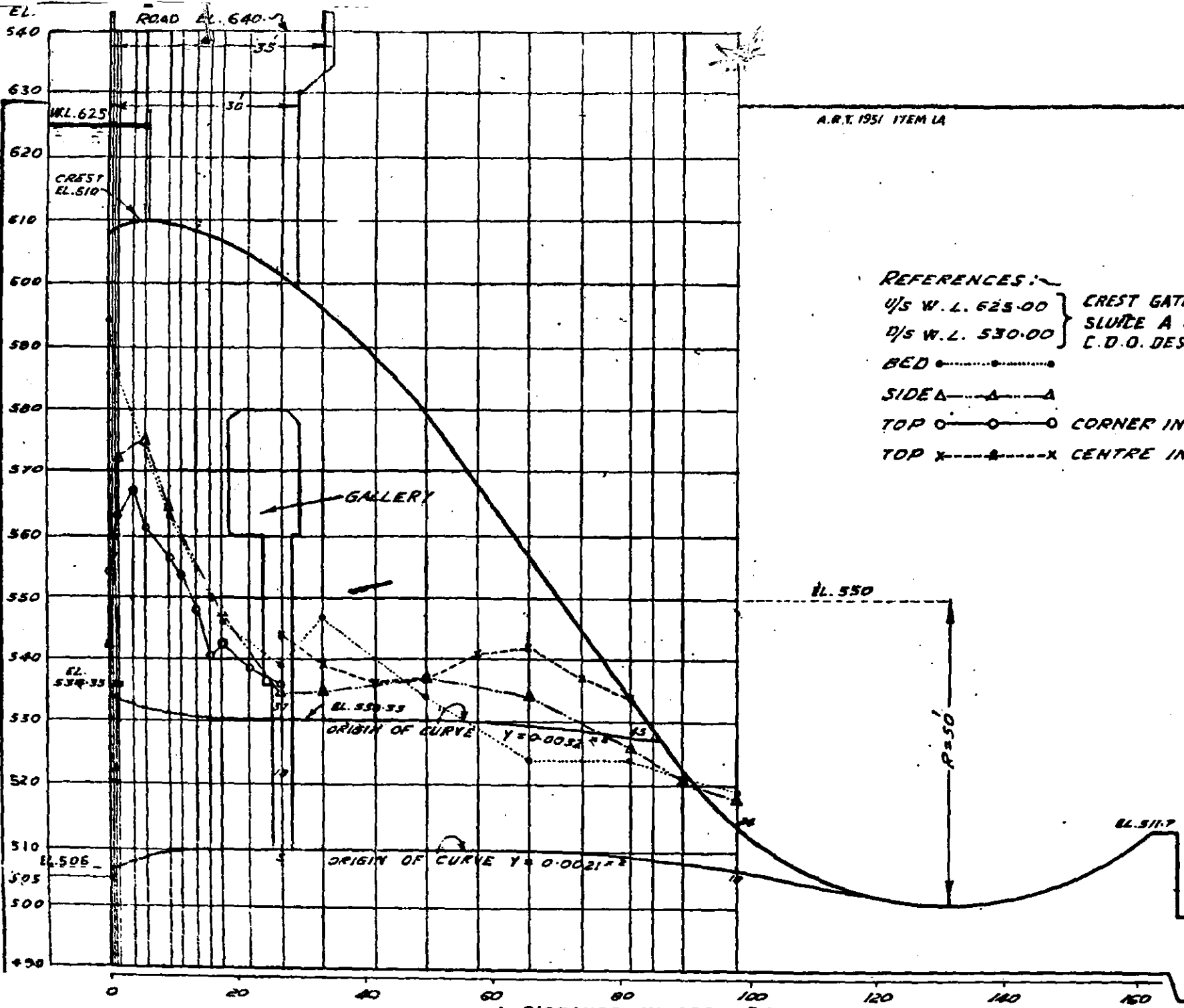
DEVELOPED PLAN



CENTRAL SECTION

FIG. 3
HIRAKUD DAM PROJECT
HIRAKUD SLUICeway
DESIGN FOR UPTURNED BARREL





A.R.T. 1951 ITEM 1A

No. 1047 OF SEPT. 1951
BWINRES., POONA.

REFERENCES:-

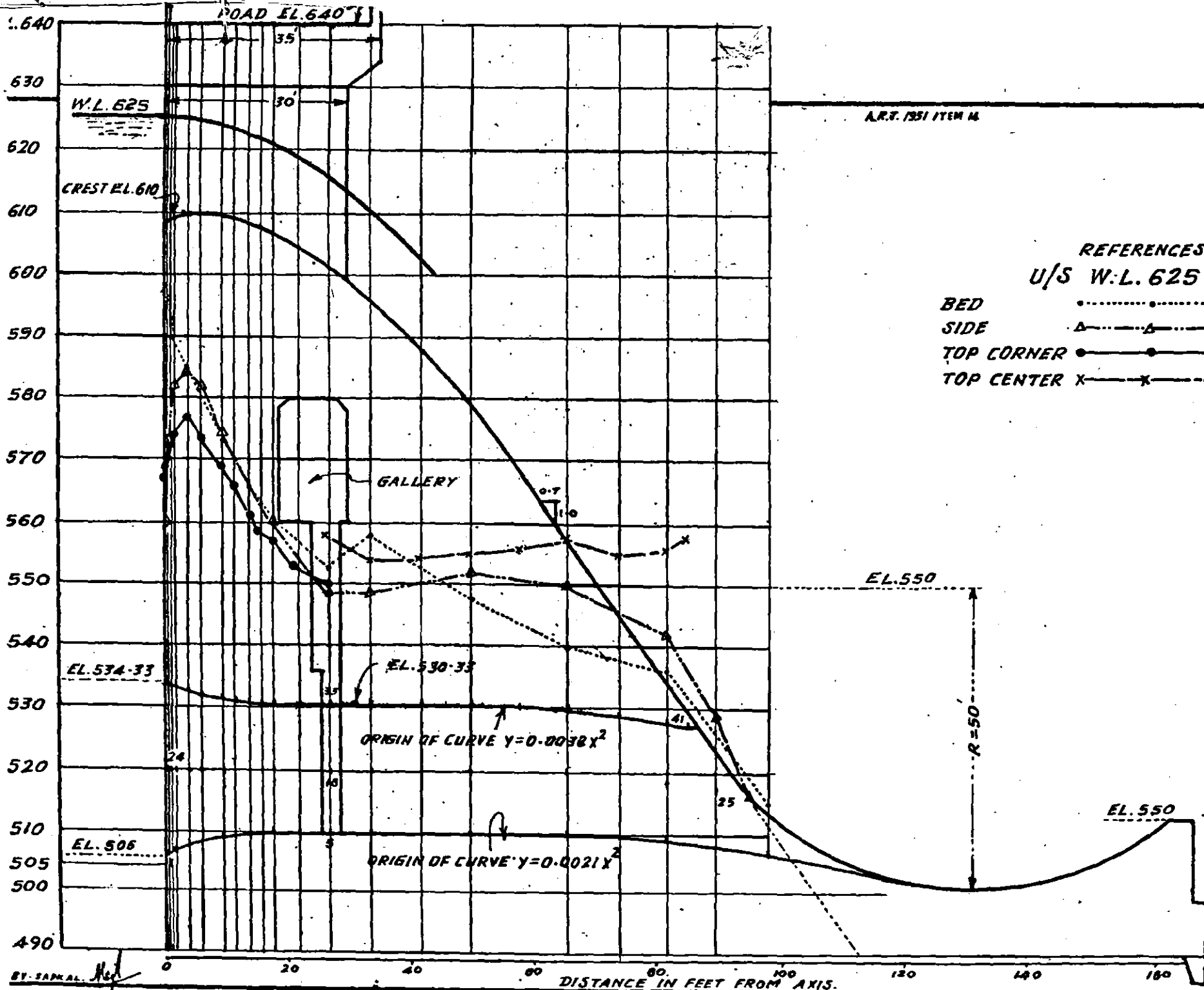
U/S W.L. 625.00 } CREST GATES CLOSED
D/S W.L. 530.00 } SLUICE A OPERATING
C.D.O. DESIGN

BED
SIDE Δ-Δ-Δ
TOP O-O-O CORNER IN BELLMOUTH
TOP X-X-X CENTRE IN BARREL

FIG. 4
HIRAKUD DAM PROJECT
HYDRAULIC MODEL I
1/20 G.S. MODEL OF SLUICeway
PRESSURE DISTRIBUTION
SCALE 10 5 0 10 20 FEET

TRACED BY - PATRIE G.R. 10-9-51. CORR BY *Naugh*

DISTANCE IN FEET FROM AXIS



A.R.T. 1951 ITEM M

No. 1048 OF SEPT. 1951
CWINRES. POONA.

REFERENCES
U/S W.L. 625.00

- BED CREST GATES OPEN
- SIDE - - - - - SLUICE A OPERATING
- TOP CORNER ——— C. D. O. DESIGN
- TOP CENTER X - - - - -

GALLERY

EL. 550

R=50'

EL. 534.33

EL. 530.33

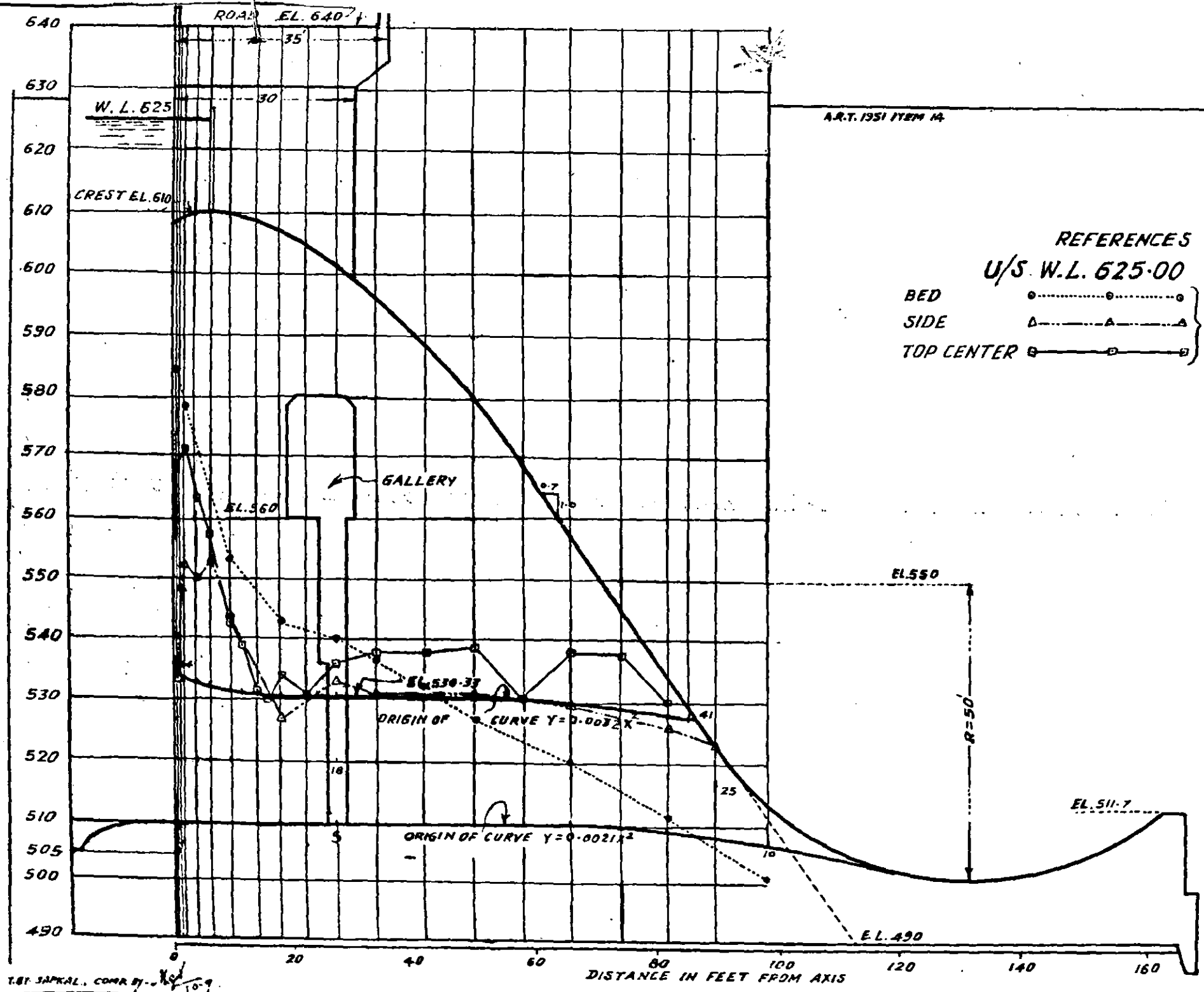
ORIGIN OF CURVE $y = 0.0038x^2$

ORIGIN OF CURVE $y = 0.0021x^2$

EL. 550

FIG. 5.
HIRAKUD DAM PROJECT
HYDRAULIC MODEL I
1/20 G.S. MODEL OF SLUICWAY
PRESSURE DISTRIBUTION

SCALE: 0 10 20 30 40



A.R.T. 1951 ITEM 14

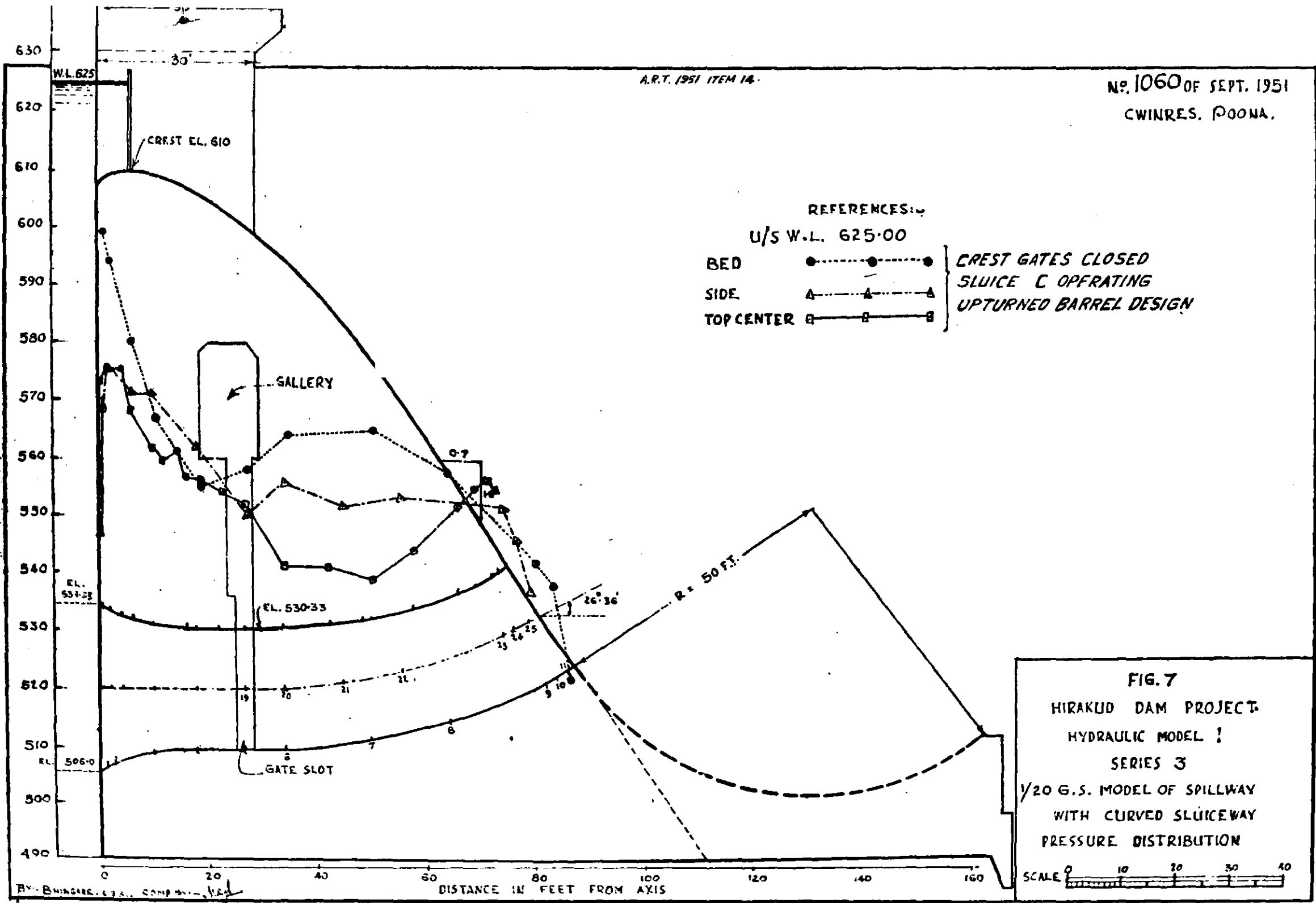
No 1052 OF SEPT. 1951
CWINRES, POONA.

REFERENCES

U/S W.L. 625.00

- BED ○-----○-----○ } CREST GATES CLOSED
- SIDE Δ-----Δ-----Δ } SLUICE B OPERATING
- TOP CENTER ◻-----◻-----◻ } I. E. C. BELLMOUTH DESIGN

FIG. 6
HIRAKUD DAM PROJECT
HYDRAULIC MODEL 1
1/20 G.S. MODEL OF SLUICeway
PRESSURE DISTRIBUTION
SCALE 0 10' 20' 30' 40'



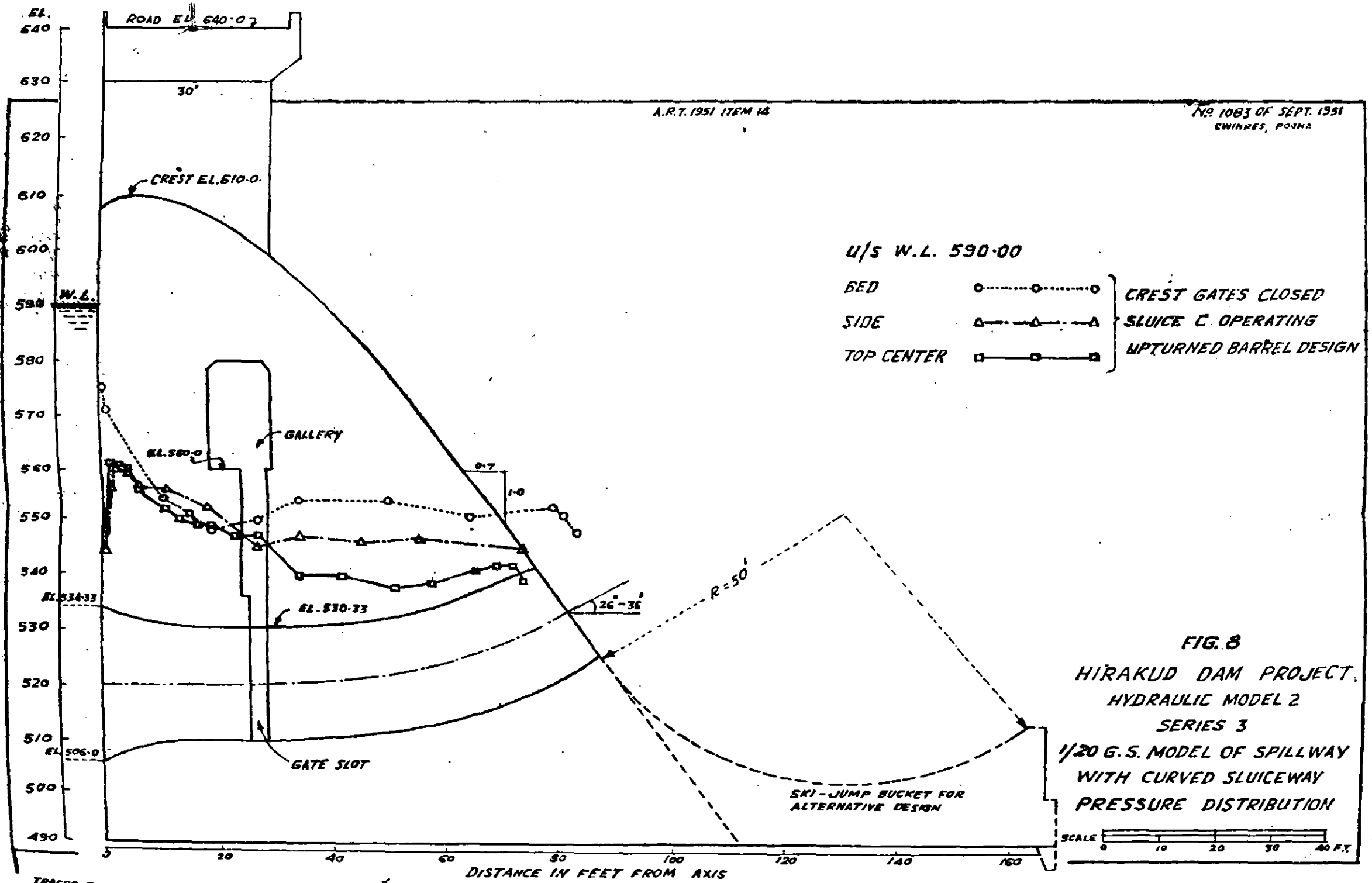
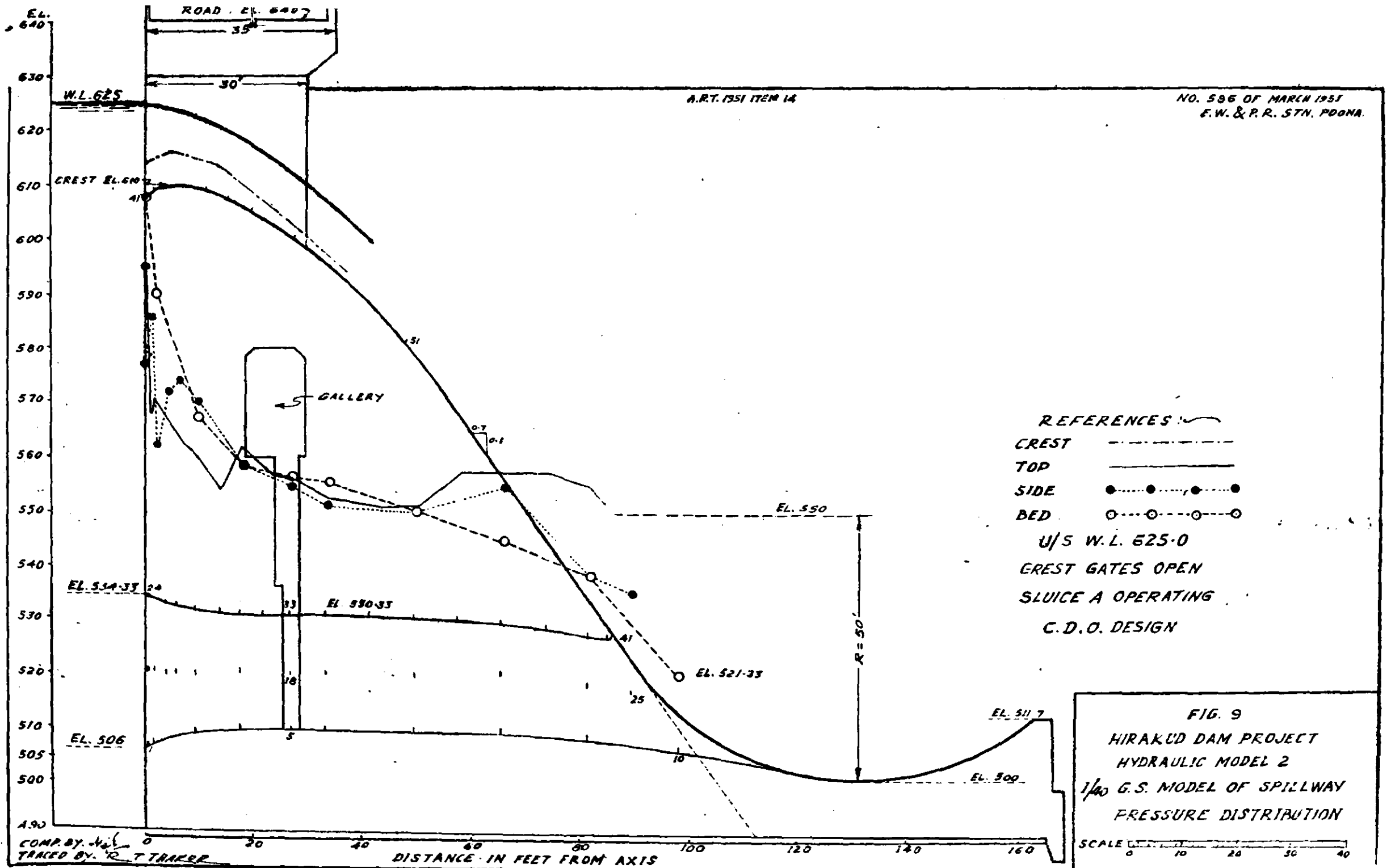


FIG. 8
HIRAKUD DAM PROJECT,
HYDRAULIC MODEL 2
SERIES 3
1/20 G. S. MODEL OF SPILLWAY
WITH CURVED SLUICEWAY
PRESSURE DISTRIBUTION

TRACED BY: B.G. DESHPANDE 15-3-51 FOMPD. 81- Jax



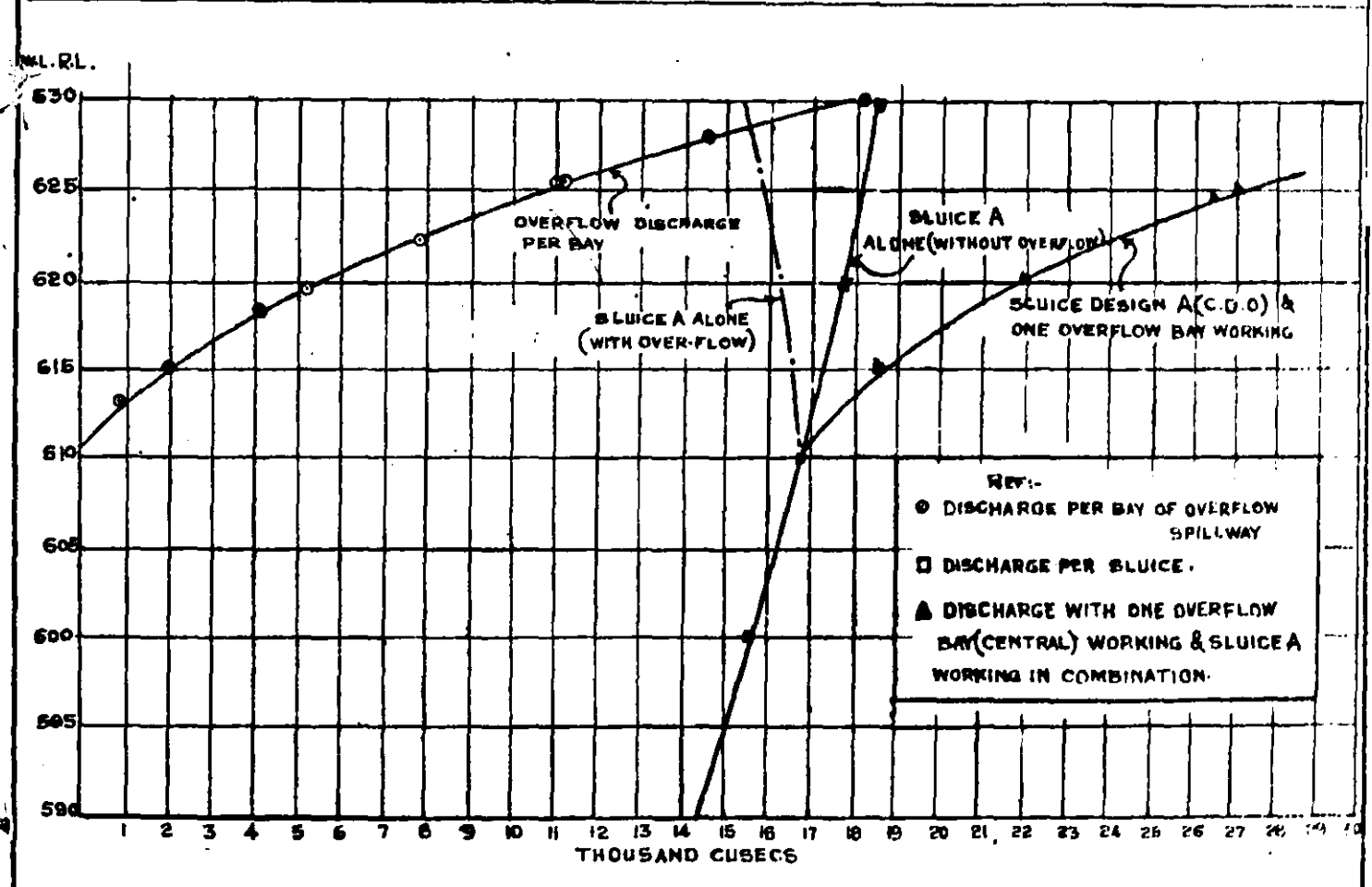
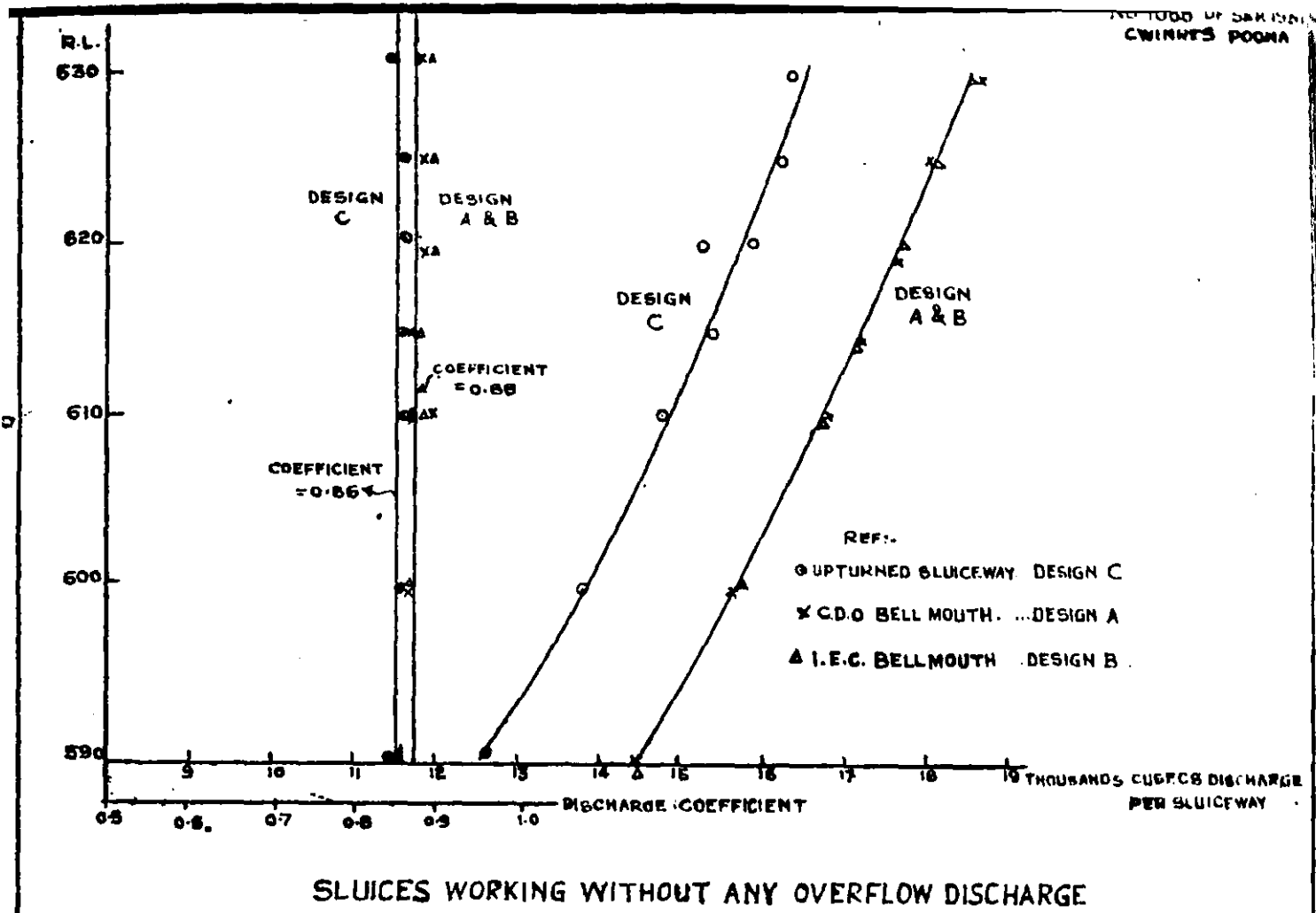


FIG. 10
HIRAKUD DAM PROJECT
1/20 G.S. MODEL OF SLUICWAY
COEFFICIENTS OF DISCHARGE & DISCHARGES
OF SLUICE & OVERFLOW BAY
WORKING INDIVIDUALLY & IN COMBINATION

C. BY: *[Signature]*
 T. BY: R.K.K. 11-2-51

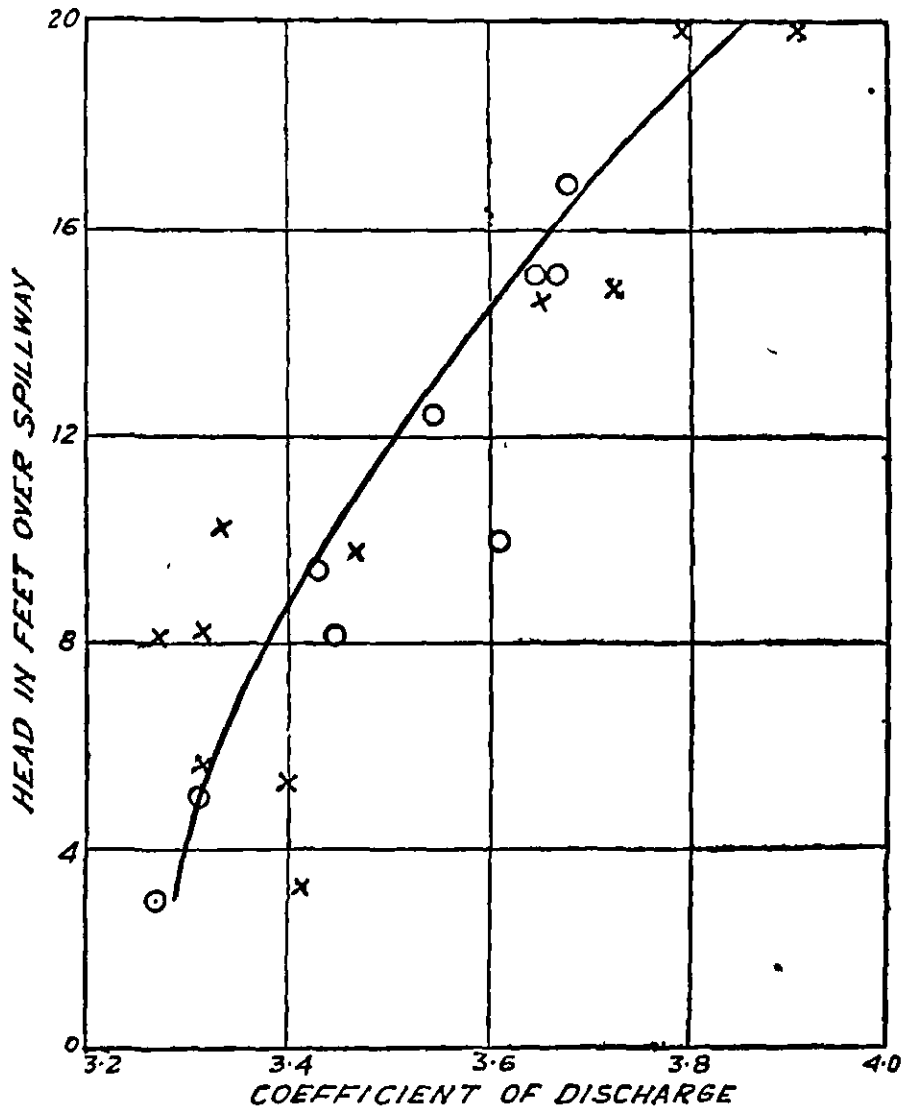


FIG. 11

HIRAKUD DAM PROJECT
1/40 G. S. MODEL - 4 1/2 SLUICES
1/20 G. S. MODEL - SINGLE SLUICE
SPILLWAY CREST DISCHARGE COEFFICIENT

REF -

OBSERVED POINTS .. X 1/40 SCALE
" " .. O 1/20 SCALE

COMP'D BY *M.S.A.*
TRACED BY B.G. DESHPANDE 4-1-52

15. DISCHARGE CHARACTERISTICS AND DISSIPATION OF ENERGY WITH CONVENTIONAL AND UPTURNED SLUICES—HIRAKUD DAM PROJECT (1/40 SCALE, GEOMETRICALLY-SIMILAR MODEL)

(B) Dissipation of Energy below the Spillway Dam

IN this report is described the work done on hydraulic models of four and a half sluiceways to geometrically-similar scale 1.40 to study interference of jets and scouring tendencies. The tests have been made with:

- (i) all conventional straight sluices with outside bucket;
- (ii) all upturned barrel type with short apron and sill as protection against the large fraction (about 30 p.c.) of discharge to be carried over the overflow crest—outlet being flared;
- (iii) same as (ii) but outlet being unflared.

The model construction and tests were carried out in the same flume which was used for similar tests reported in Annual Report (Technical) 1950, in connection with upturned sluiceways with elliptical outlets; only the width of the flume was adjusted in accordance with 30 ft. centre to centre distance of the sluiceways in the new lay-out instead of 25 ft. in the lay-out reported last year.

(i) Tests with Conventional Sluiceways

For different discharges and tailwaters the water profiles and the scours were observed. The bucket in this design is of 50 ft. radius and the lip angle=40°. While the scouring tendency as shown by the scoured bed under different conditions showed the presence of satisfactory strong return rollers, the main problematic feature in this design was the high water profile when overflow crest discharge was present, with or without undersluice discharge. The scouring tendency is shown in Figure 1 with maximum upstream water level. The water profiles with overflow discharge only are shown in Figure 2

Fig. 1.
Fig. 2.

The main problem here therefore was the height of retaining walls necessary to contain the water profiles. This could be done finally only in a composite model of the whole spillway, for which reference is to be made to another report.

(ii) Tests with Upturned Sluiceways

Flared Outlets—Tests for these were concentrated on the scour due to combined overflow and undersluice discharges, under the maximum upstream water level 630, with different aprons and end-sill. The following have been the test sequence:

- (a) *No Apron—Bed made Erodible up to the Toe of the Dam*—Tests showed that for upstream R.L. 630, the scour at the toe of the dam was down to E.L. 470 while the general downstream scour was down to about E.L. 485 only (Figure 3). With the reservoir level at R.L. 620, the characteristics were clearly seen—a ridge forming about 40 ft. from the toe of the dam where the back roller due to the sluiceway jet met the roller at the toe due to the overflow jet.
- (b) *35 feet Long Apron*—As an arc of radius 40 ft., lowest level at E.L. 500, end-sill lip angle of 20°, with end-sill elevation at R.L. 502.4. In this case the back roller was found to be not strong enough.
- (c) Same as (b) with an additional height to end-sill to bring it to E.L. 506.4 and also higher-sill up to E.L. 510.4. With the latter-sill, the material was found to pile up below the sill up to about E.L. 502.
- (d) *35 ft Long Apron*—consisting of arc of radius 40 ft., and a horizontal stretch followed by an end-sill with upstream rounded corner. The scour with this was satisfactory. The maximum scour downstream was down to about E.L. 485 only. There was no scour at the downstream face of the sill. For lower discharges also, the conditions were satisfactory (Figure 4).

Fig. 3.

Fig. 4.

Aeration of Jets—Due to the possibility of intense intermingling of undersluice jets, the capacity for air entrainment in this case is very large, if sufficient air supply can be assured. Experiments in the model show that if the overflow jet was deflected away from the face of the dam by a 3-ft (prototype) thick ledge

Fig. 1.
Photos
310 & 378.

(see Figure 1 for location) or by an extra ogee crest coming down vertical at the end of the pier (see Photos 310 and 378 for comparison) so as to fall on the undersluice jet away from the dam, a very pronounced improvement was seen in air entrainment. From the point of view of simplicity of construction, however, a flatter extra ogee crest coming down vertical at the end of the pier was found more suitable, though it involved some reduction in discharge coefficients, with safer pressures on the crest.

(iii) Comparison of Theoretical and Observed Throw-off in Upturned Sluiceways

Unflared Outlet without Overflow—Unflared outlets are more acceptable due to simplicity in construction. It helped in the correlation of test data on 'throw' with theoretical values. Tests on scour were made with a ledge present near the crest, 25 ft. wide horizontally, 2 ft. high and fixed on the dam with its downstream vertical face flush with the pier. A typical scour detail is shown in Figure 5.

Fig. 5.

In comparison to flared outlets, the scour was a little greater in the case of unflared outlets. In the former case, the undersluice jets, in the absence of overflow, flared out to intermix with jets from adjacent sluices. In the latter case, however, the jets were distinct and intermingling occurred further downstream, but still above the bucket lip. In the presence of overflow stream as deflected by the ledge near the pier, the mixing up, as observed visually was as good as before; a large fraction of the overflow jet, however, in passing through the clear space between adjacent undersluice jets fell directly on the pool in the bucket. Under the conditions of mixing in the pool in the bucket, the visual observations could not make any clear distinction between the case of the unflared and the flared outlets.

Throw of the Jet—Comparison with Theoretical Value—It can be easily deduced that when a projectile is thrown at an angle θ with a velocity due to an effective head H' , from the centre line of the outlet, it will fall on the ground at a horizontal distance from the outlet equivalent to

$$d = H' \sin^2 \theta \left[1 + \sqrt{1 + \frac{h}{H' \sin^2 \theta}} \right]$$

where h = height of the centre line of the outlet from the ground. For the upturned sluiceway, $\sin^2 \theta = 0.80$.

Fig. 5.

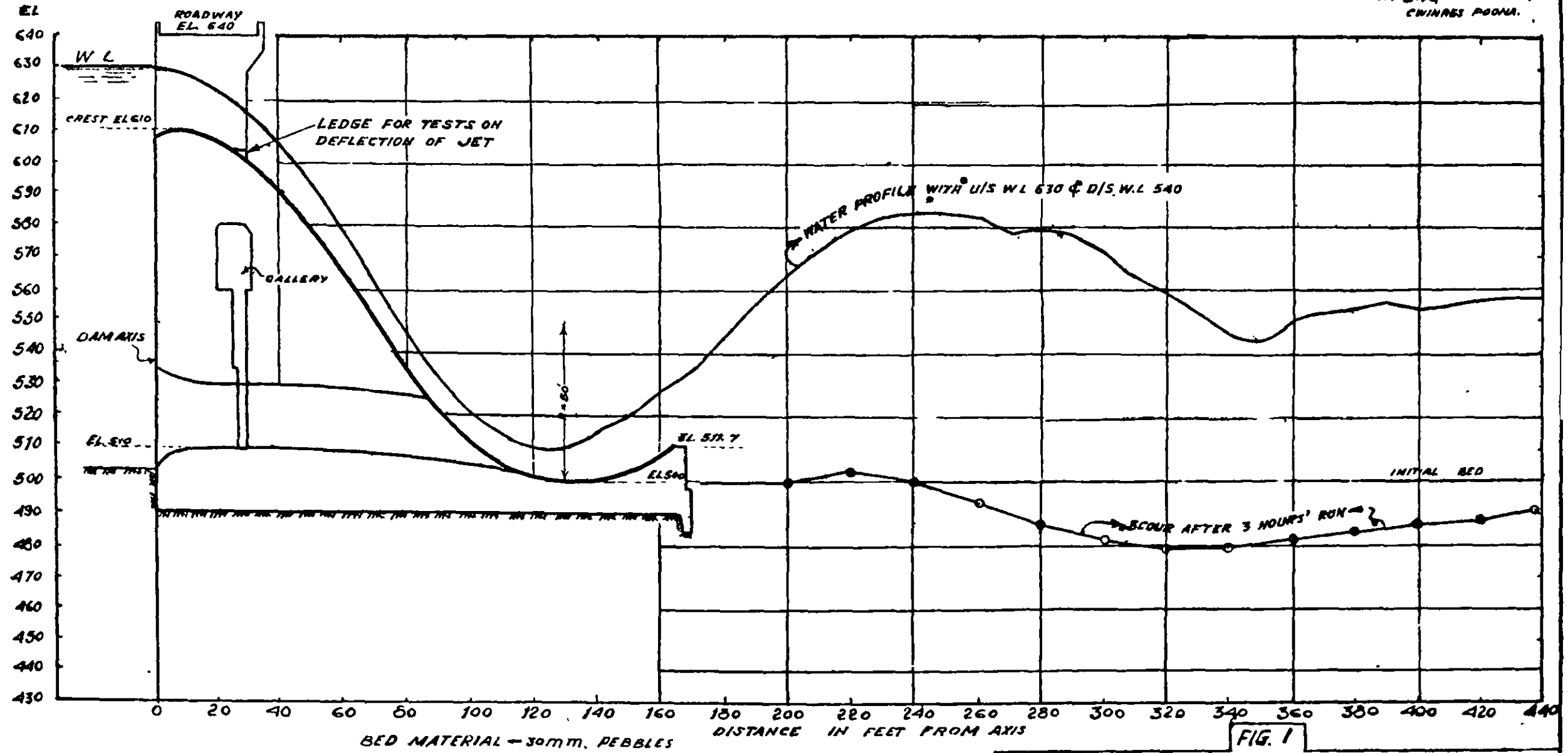
Figure 5 shows the comparison.

Table 1 gives the distance "D" from the toe to the place where the central filament struck the level E.L. 480, both when C_d is assumed, $C_d = 0.9$ and $C_d = 1.0$. This is compared with the distance D_1 , where the maximum depth of scour is formed.

TABLE 1

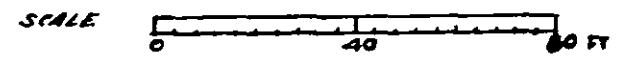
Upstream R. L.	Observed D_1 (ft.)	D calculated with	
		$C_D = 1.0$ (ft.)	$C_D = 0.9$ (ft.)
590	120—130	131	110
600	150—160	149	125
610	160—170	167	140
620	190—200	186	156
630	210—220	204	171

It will be seen from the table that the distance of maximum scour is somewhat greater than the 'throw' obtained using $C_d = 0.9$ to some extent due to the differential head created by the water levels behind and in front of the jet where it strikes the high velocity stream.



NOTE : THE WATER PROFILE AS SHOWN IS HALFWAY BETWEEN SLUICWAYS

FIG. 1
HIRAKUD DAM PROJECT
 HYDRAULIC MODEL 2 - SERIES II
 1/40 G.S. MODEL OF 2 1/4 CONCRETE SPILLWAY BLOCKS
 BED SCOUR



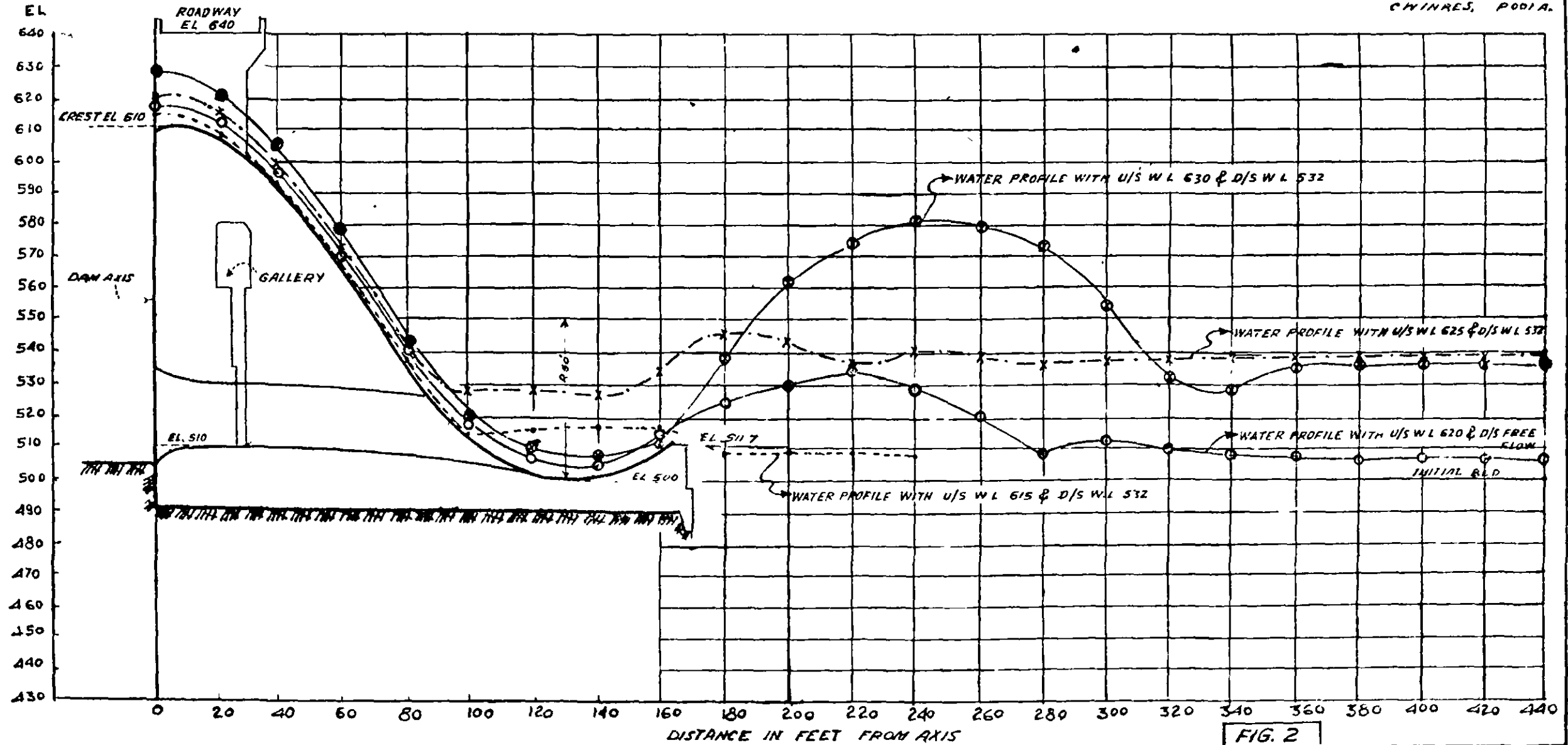
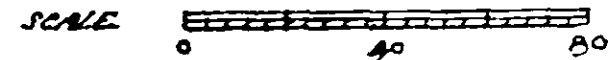


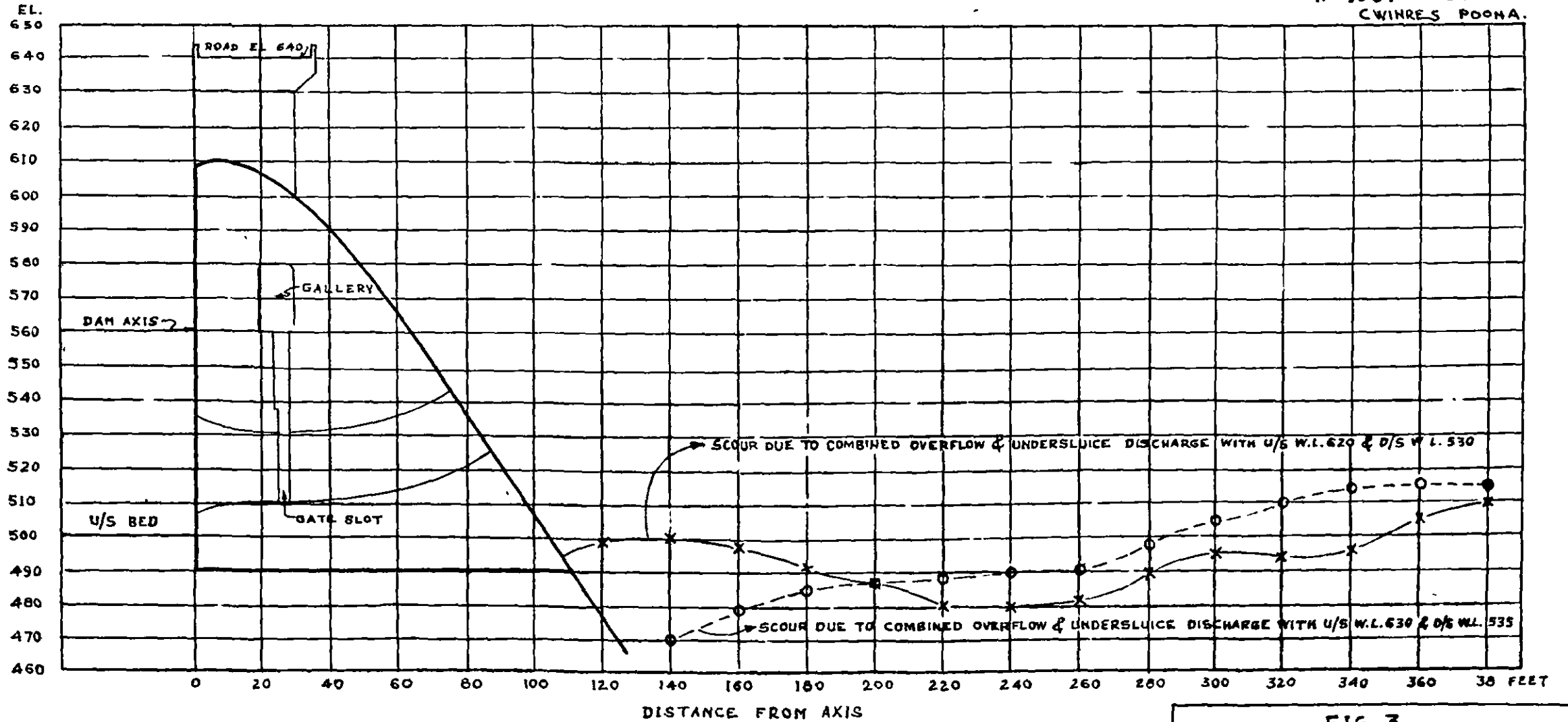
FIG. 2

**HIRAKUD DAM PROJECT
HYDRAULIC MODEL 2**

1/40 G. S MODEL OF 2 1/4 CONCRETE SPILLWAY BLOCKS
WATER PROFILES WITH OVERFLOW DISCHARGE ONLY



COND BY *M. S. J.*
PARED BY JAMALKAR 16-6-51.

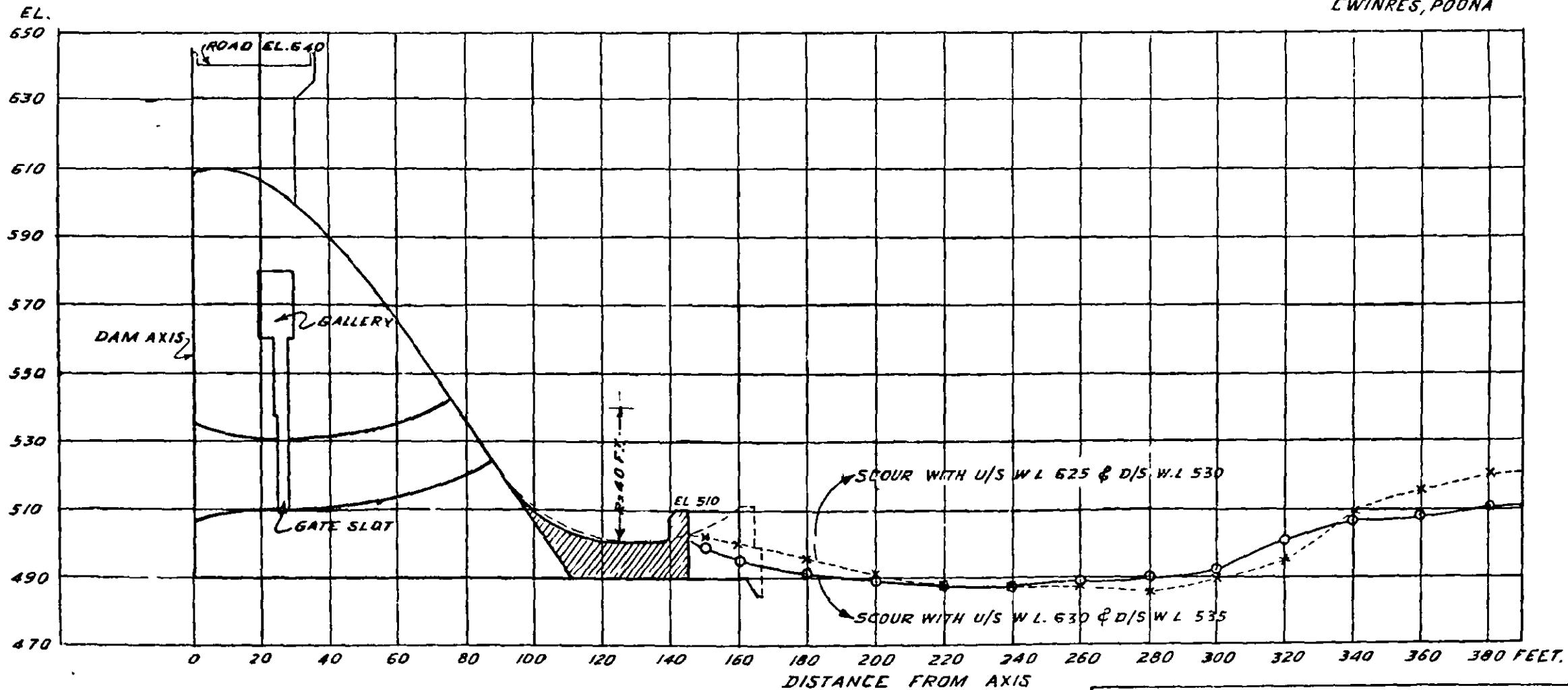


BED MATERIAL - 1" STONE AND PEBBLES
RIVER BED D/S INITIALLY LAID AT EL. 500

FIG. 3
HIRAKUD DAM PROJECT
HYDRAULIC MODEL 2 SERIES III
1/40 G.S. MODEL OF SPILLWAY
BED SCOUR WITH NO APRON

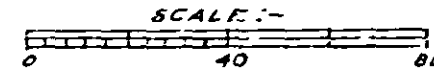
SCALE 80 FT.

COMP. BY. *K. Singh*
TRACED BY R. T. THARON



SCOUR PATTERN WITH 35 FT APRON & END SILL EL. 510

FIG 4
HIRAKUD DAM PROJECT
HYDRAULIC MODEL 2 SERIES III
1/40 G. S MODEL OF SPILLWAY
SCOUR TESTS WITH APRON



COMP BY: *[Signature]*
TRACED BY: PARKHI. 11-0-1951

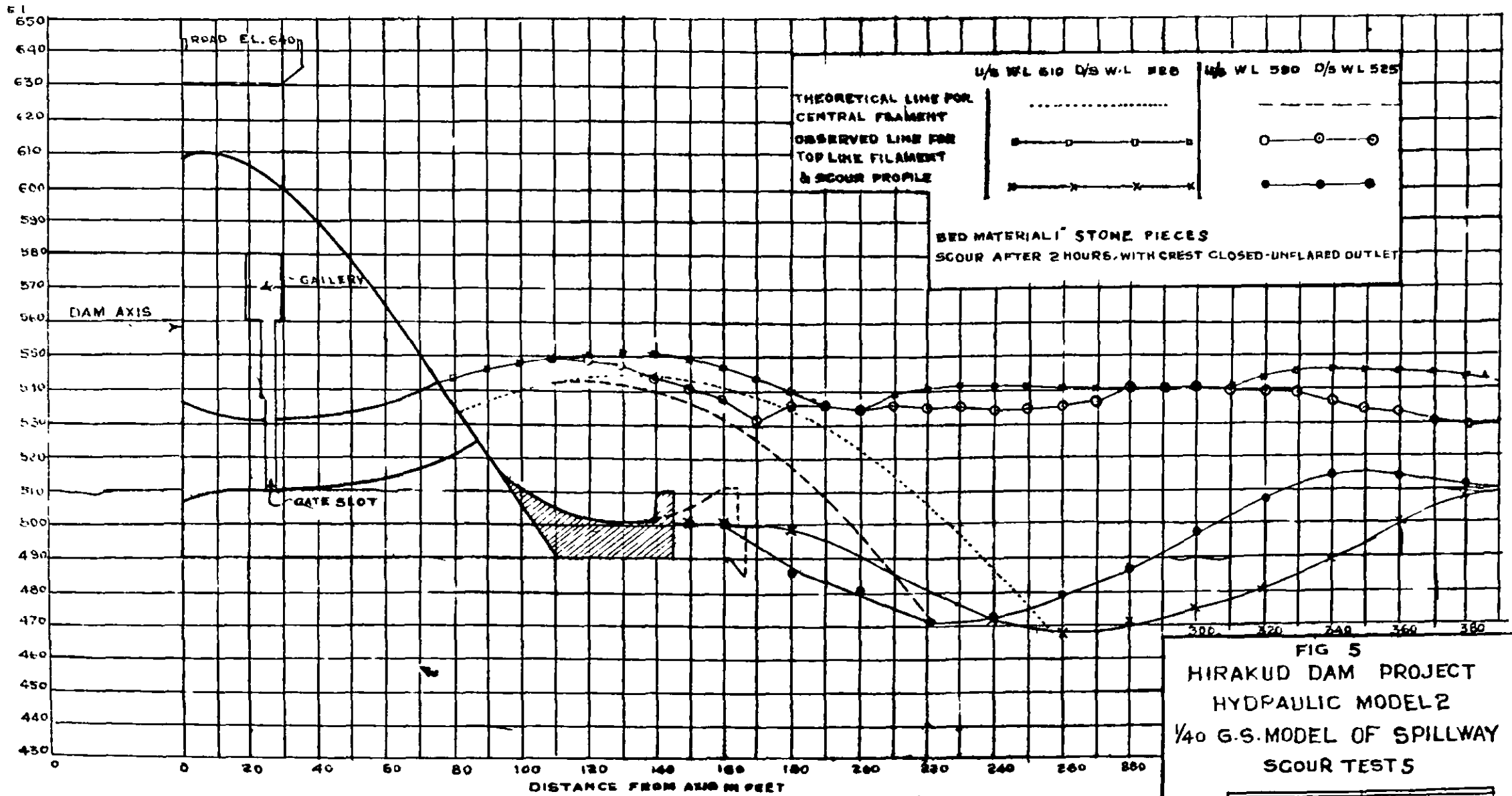
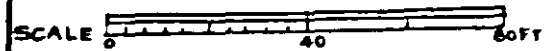


FIG 5
 HIRAKUD DAM PROJECT
 HYDRAULIC MODEL 2
 1/40 G.S. MODEL OF SPILLWAY
 SCOUR TESTS



C BY *M. K. Kulkarni*
 T BY R K Kulkarni 1579/51

ITEM No. 15.
A. R. T. 1951.

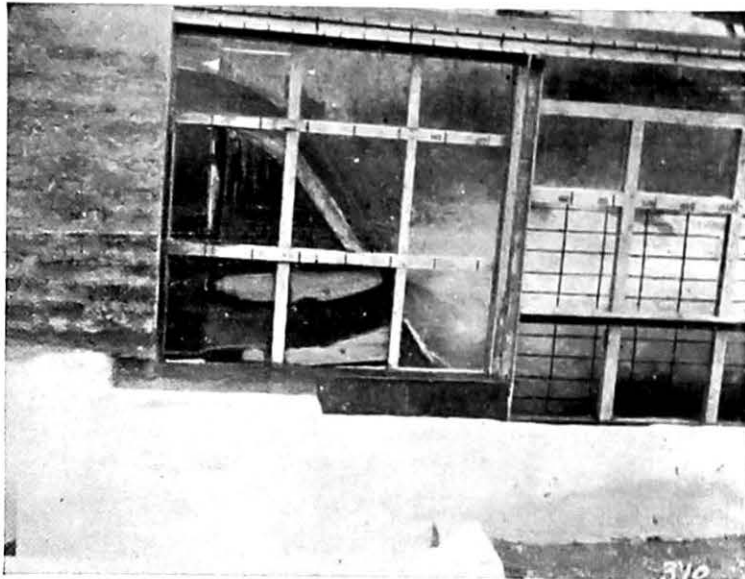


Photo 310
Interaction of overflow Jet and undersluice Jet.

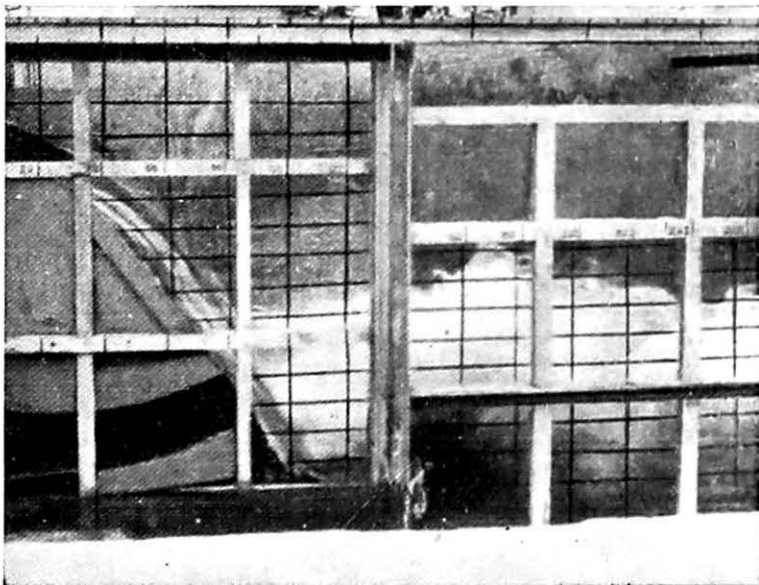


Photo 378
Interaction of overflow Jet and undersluice
Jet with a flatter extra ogee crest on top to
improve air entrainment.

Taking into consideration the further fact that air entrainment usually exercises rather a small effect on the 'throw-off', it may be concluded that the throw designed on the basis of a simple parabolic trajectory would be realised in practice.

Flood disposal capacity of Hirakud spillway with 64 sluices and 34 crest bays, under the present lay-out is sufficiently well above design requirements for the disposal of a moderated maximum flood of 12 lakh cusecs, as shown in Table 2. This should make it entirely feasible to incorporate the upturned sluiceways which have proved fully satisfactory from the hydraulic point of view in model tests.

TABLE 2

Flood disposal capacity of Hirakud Spillway—64 sluices and 34 crest bays

Crest bays along with	Upstream R.L. 630 lakh cusecs	Upstream R.L. 625 lakh cusecs	End sluices	Upturned sluices	Crest bays of upturned sluiceway blocks
No upturned sluiceways + 64 conventional sluices	16.6	14.5	Open
	16.0	13.8	Closed
6 upturned sluiceways + 58 conventional sluices	15.9	13.7	Closed	Open	Open
	15.4	13.4	Closed	Open	Closed
	15.0	12.8	Closed	Closed	Open
12 upturned sluiceways + 52 conventional sluices	15.8	13.6	Closed	Open	Open
	14.7	12.9	Closed	Open	Closed
	14.1	11.8	Closed	Closed	Open

NOTE—End sluices may have to be closed during the maximum flood only, so that water will not overtop the downstream training walls.

Recommendations

(i) The 50 ft. radius bucket may be adopted for uniformity of construction, though with present overflow a shorter bucket 35 ft. long was also proved sufficient below the upturned sluiceways.

(ii) Sluiceways of the upturned type may be incorporated in the left channel spillway consistent with effective spillway discharge capacity for maximum flood.

(iii) Aeration by means of an extra ogee crest in the case of the upturned sluiceway blocks would be highly effective.

6. SPILLWAYS**16. FAILURE OF KHODSHI WEIR (BOMBAY STATE) AND MEASURES TO PREVENT ITS RECURRENCE***Introduction*

MANY cases of broad or narrow-crested weirs arise in which during floods tail-water rises to a level when conditions in the pocket below the nappe become highly unstable. The rapid fluctuations in pressures lead to vibrations and consequent weakening of the structure in course of time. Failures such as that of the Khodshi Weir on the Krishna River near Karad (North Satara District, Bombay), have occurred in the past and so the investigations of pressure distribution—especially subatmospheric pressures and their fluctuations, over the profile of a narrow-crested weir were studied in the model of Khodshi Weir.

1. History

The Khodshi Weir across the Krishna River near Karad supplies water to the Krishna Canal. The Weir was built in 1868 with top width of 8 ft. and downstream

batter 1 in 6. During 1914-1918 it was proposed to raise its F.S.L. by 4 ft. by constructing collapsible shutters over the weir crest. As the old weir section was not strong enough for the increased head a continuous buttress was added to the lower 10 ft. of the weir. The project was subsequently dropped at this stage and the existing weir stands as shown in Figure 1. The length of the weir is 1,000 ft. and effective length for spillway is 800 ft. (approx).

2. Breach

The weir breached on 20th July 1950 at 10 p.m. The portion of the weir which breached in huge blocks is shown in Figure 1. The high flood level recorded during this flood was 7 ft above the crest level. The highest flood level recorded has been 8 ft. 6 inches over the crest in 1914, i.e., before the section was buttressed. The probable reasons for the breach appear to be, (a) weir section is weak if uplift pressures are considered, and (b) weakening of masonry joints by the dissolution of lime mortar. This was later confirmed in the weir inspection.

It was, therefore, decided to conduct model experiments to ascertain the pressure distribution over the weir profile before strengthening the weir.

3. Model Scales

L and D scale=1.10 (geometrically-similar)

Q Scale=(D^{5/2})=1:313

The maximum flood discharge in prototype,

$$Q \text{ Max.} = 3.1 \times 800 \times 8.5^{3/2} \quad (D=8'-6") \\ = 62,000 \text{ cusecs.}$$

$$Q_m = \frac{62,000}{313} = 198 \text{ cusecs for 80 ft. long weir in the model.}$$

$$\therefore q_m = \frac{198}{18} = 2.47 \text{ cusecs.}$$

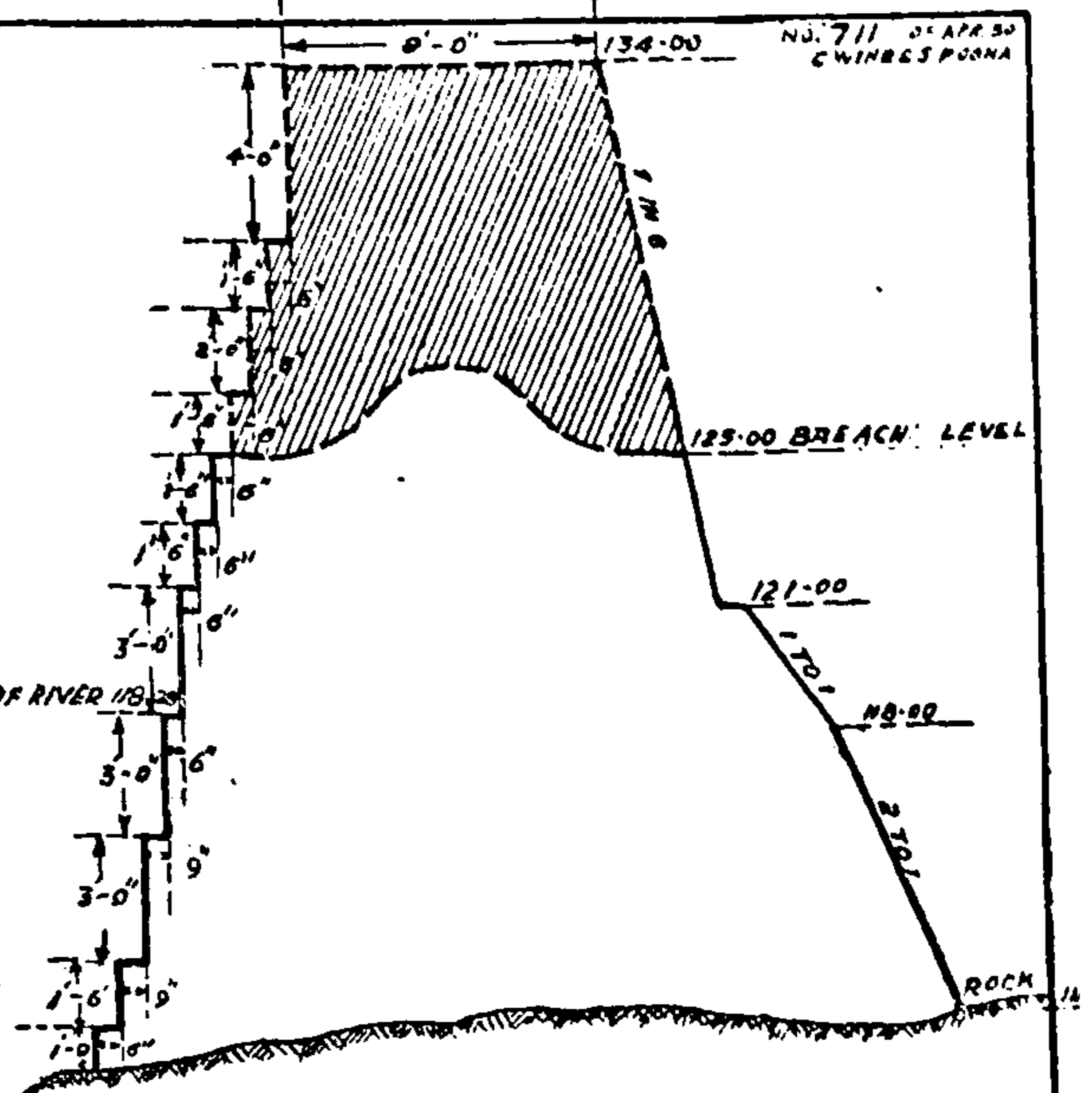
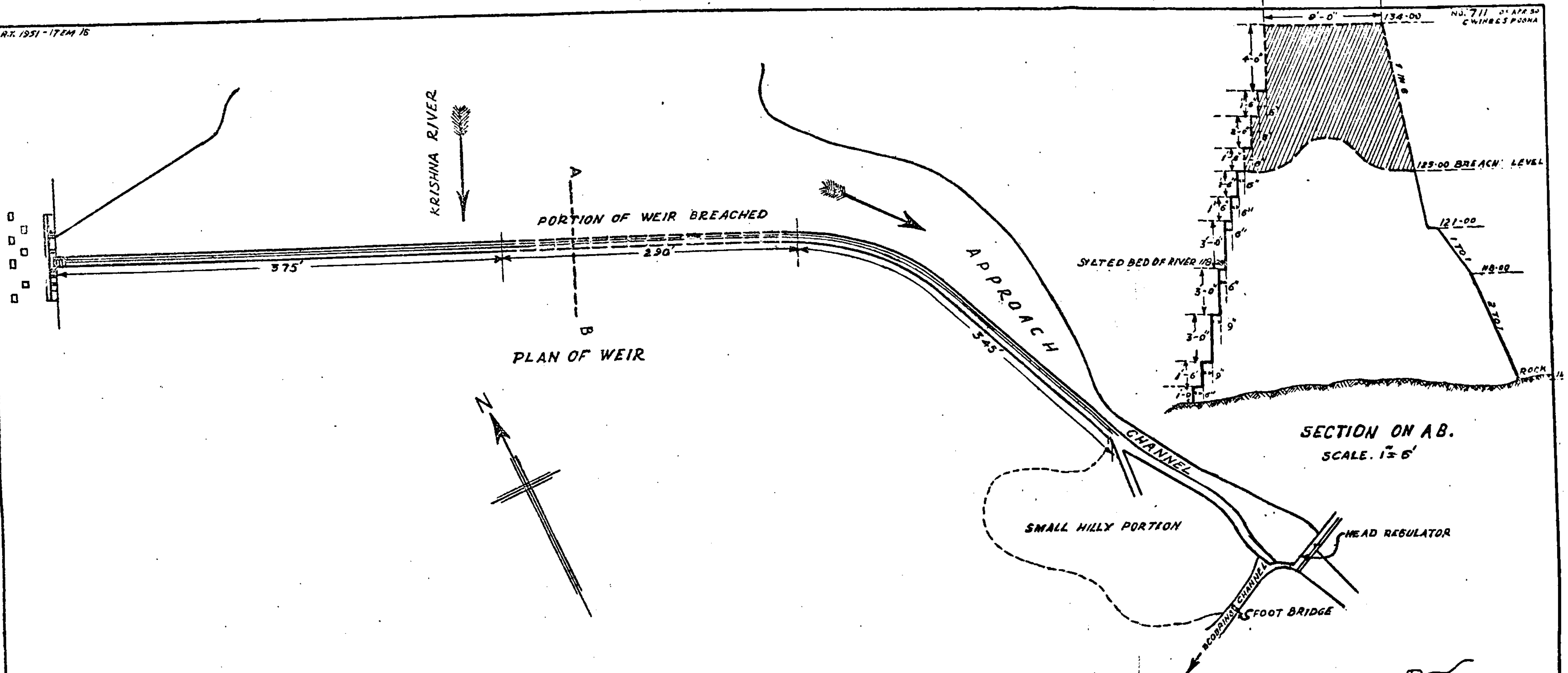
4. Experiments

4.1. *Experimental set-up*—Experiments were conducted over 3 ft. length of the weir in the model with manometers connected to pressure points No. 1 to 13 as shown in Figure 2. In all experiments maximum flood discharge corresponding to W.L. R. L. 142.5, i.e., 8 ft. 6 inches over the crest was run and pressure variations recorded. Only the downstream W.L. was changed in each experiment to ascertain the maximum negative pressures. Also effect of aerating the nappe by putting air pipes was observed on the pressure distribution.

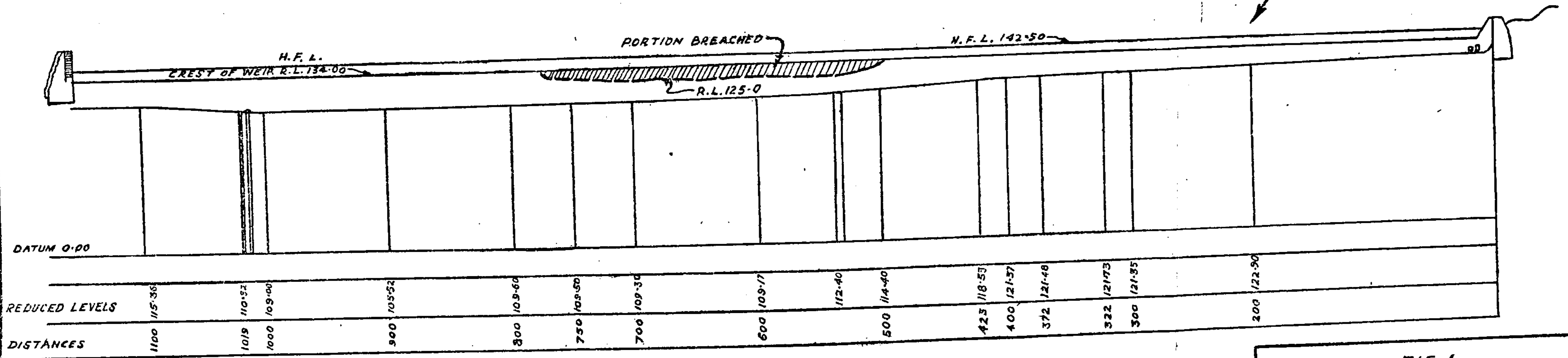
4.2. *Experiment No. 1—Downstream W.L. R.L. 121.0*—The physical phenomenon observed during the experiment was of considerable interest.

4.2. 1. *Roller on the Weir Crest*—As the stream could not follow the upstream sharp bend it separated from the weir crest just beyond the upstream edge and formed a roller as in Figure 2. The roller extended over a length of about 50 per cent of the head over the crest. This roller led to a slight contraction in the flow at about its mid-point, where due to the increase in velocities there was a drop in pressures at points 2 to 6 which however were positive, i.e., above atmospheric pressure. The pressure was restored at the end of the roller at point 7 and continued to be positive until it dropped down at the downstream edge of the crest. The value at this point fluctuated with the conditions below the lower surface of the nappe and appears to be governed by it. This roller persisted and had almost the same magnitude in all the experiments.

4.2. 2. *Conditions below the Nappe*—Initially when there was a free fall with low tailwater, an air pocket formed bounded by the nappe, the downstream face of the weir and the tailwater pool. This air pocket was sufficiently large, the pool below the pocket was fairly steady and at a level higher by 5 ft. than the tailwater level further downstream. The pressures on the downstream face of the weir were slightly subatmospheric. The maximum negative pressure was 1.5 ft. and the pressure curve fluctuated within a range of 2 ft. The effect of aeration by air pipes was to stabilise the flow conditions, eliminate the pressure fluctuations and the water level in the pocket behind the nappe became nearly equal to the tailwater level.



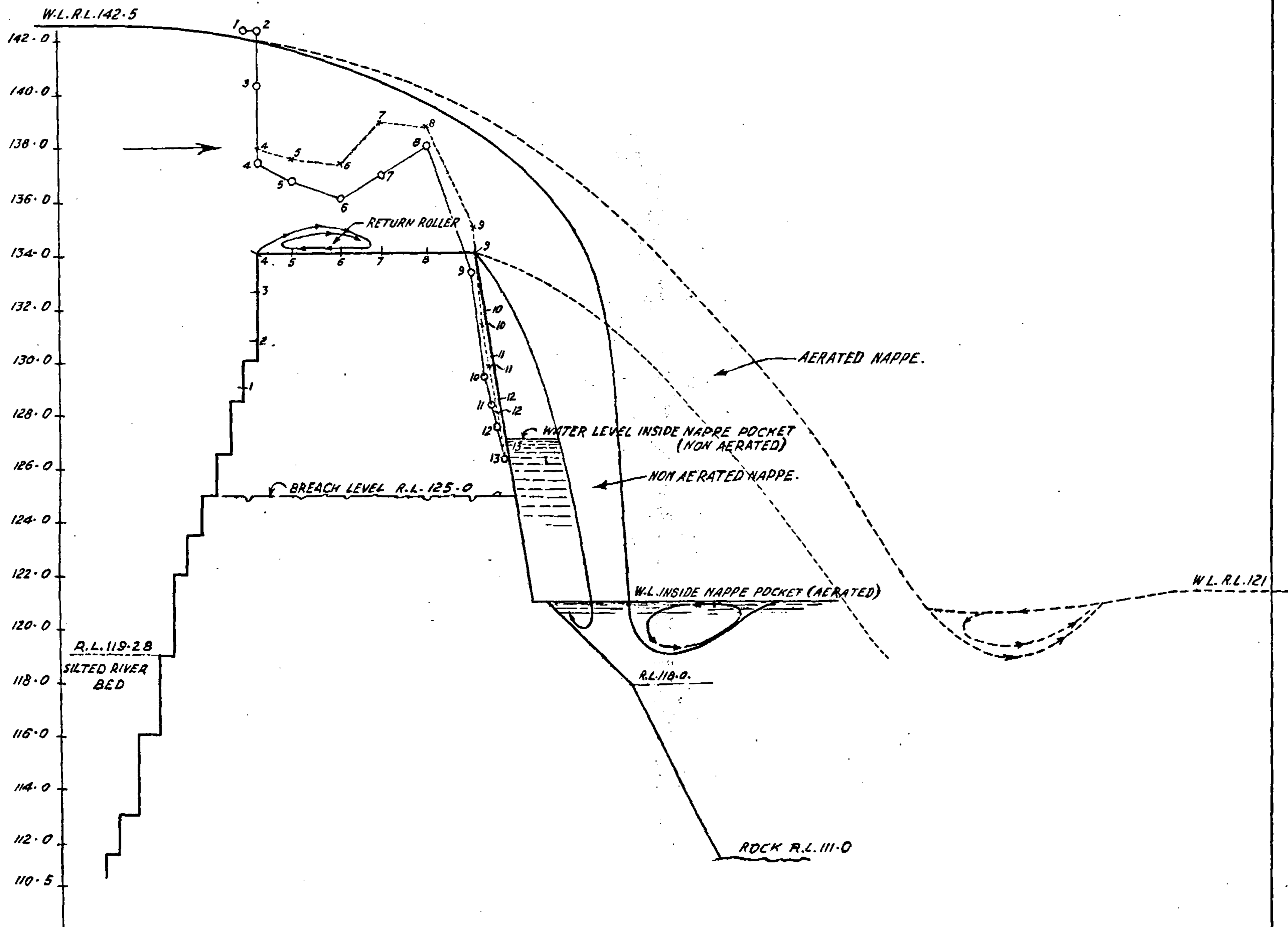
SECTION ON A-B.
SCALE. 1" = 6'



LONGITUDINAL SECTION OF WEIR

NOTE.
NORMAL FLOOD LEVEL 141.00
HIGHEST FLOOD LEVEL 142.50

FIG. 1
WEIR ACROSS THE RIVER
KRISHNA AT KHODSHI
SCALE. 1" = 80'



REFERENCES:-

- MAX. PRESSURE *-----*
- MIN. PRESSURE ○-----○
- PROFILE OF AERATED NAPPE —————
- PROFILE OF NON AERATED NAPPE. - - - - -

FIG.2
KHODSHI WEIR
ON KRISHNA RIVER AT KARAD
PRESSURE DISTRIBUTION CURVES
AND WATER SURFACE PROFILES.
SCALE, 1" = 4'

4.3. *Experiment No. 2—Downstream W.L. R.L. 126.0*—In this experiment the tailwater level was gradually raised to R.L. 126.0. The pressures on the upstream part of the crest in the roller region were little affected by the raising of the tailwater level. As the tailwater was raised in stages to R.L. 126.0 the air pocket below the nappe became smaller owing to the pool below the pocket rising higher and becoming unsteady; until at R.L. 126.0 the fluctuations were so large that the air pocket was formed intermittently and then swept away by the high velocity stream and its space filled by eddying water with entrained air. This cycle of fluctuations was so violent that it would set up vibrations in the body of the weir. The pressure fluctuations on the crest were within a range of 15 ft. the pressures being all positive except at the downstream extremity. The pressures in the air pocket fluctuated over a large range, the maximum fluctuation being equivalent to about 8 ft. The nappe also pulsated to and fro with these fluctuations. The pressure became negative and went down to a minimum of 8 ft. at point No 10. This was the worst case from the point of view of vibrations and negative pressures.

Photos
8932 & 8961.

Fig. 3.

4.4. *Experiment No. 3—Downstream W.L. R.L. 128.0*—This was a case of near submergence of the nappe. In this case the nappe, instead of falling clear with a fluctuating air pocket, clung to the downstream face of the weir. The pressures were still negative on the downstream face, but the fluctuations were less violent than previously, the maximum fluctuation being 4.2 ft. at pressure point No. 10. There was no entrained air in the eddying flow along the downstream face of the weir.

Fig. 4.

4.5. *Experiment No. 4—Downstream W.L. R.L. 134.0—(crest level)*—The downstream water level was raised to R.L. 134.0 which is the crest level. This was a case of complete submergence. The pressures were all positive and the fluctuations on the downstream face had died down. The fluctuations on the crest were insignificant.

Fig. 5.

5. Discussion of Results

5.1. As already stated the roller on the upper part of the crest formed due to the stream not following the sharp bend. This roller decreased the pressures on the crest—not to the extent of making them below atmospheric pressure. It can be got rid of by properly bevelling the corner.

5.2. The most dangerous conditions with regard to subatmospheric pressures and violent fluctuations occurred when the air pocket below the nappe was sufficiently small to be periodically swept away and filled by eddying water with entrained air and formed again. The suddenness of the breaking away of the air pocket contributed to large impact pressures which must have led to gradual weakening of the structure. When the tailwater was low there was merely a rise of water level in the pocket behind the nappe above the normal tailwater level further downstream. The pressures on the downstream face of the weir were slightly below atmospheric with small fluctuations. On the contrary all pressures were positive and there was negligible fluctuation when there was complete submergence. In fact the structure was safe only under these two conditions. The dangerous conditions arose in the intermediate stages of water level.

5.3. The cycle of sudden breakaway and reformation of the air pocket is an interesting phenomenon. When a high velocity jet flows past air medium, fluctuation in the flow leads to entrainment of air from the air-water interface. In the case of the air pocket bounded by the high velocity nappe, downstream face of the weir and the tailwater pool there was continuous evacuation of air from the pocket. The reduction in pressure and volume of the air in the pocket reached an equilibrium stage when the pocket was large enough. Otherwise when the tailwater was being raised to R.L. 126.0 a stage was reached when the pocket was small enough to be swept away bodily by the jet and its space filled by eddying water. As usual in a sudden collapse of cavity in any mass of a liquid, this implies large impact pressures. While the magnitude of negative pressures may be small the impact may be significantly high. The eddying water brought in large air bubbles leading to the restoration of the air pocket. The cycle began anew.

Conclusions

In a broad or narrow-crested weir the conditions below the nappe play an important part in determining safe pressures on the downstream face of the structure. Unless the flow is submerged there can be no safe and steady pressures except with

very low tailwater level or proper aeration. These experiments have shown that where there is a possibility of the tailwater level rising to a level where conditions below the nappe become violently unstable, proper aeration is absolutely necessary. In this particular case of the Khodshi Weir, allowing for 10 ft. of negative pressure in the design and proper aeration are recommended. Aeration pipes may be supported on piers to be constructed at suitable intervals.

The piers can be advantageously used to store more water at the end of the flood season by providing a double row of planks fixed between the piers and by filling the gap between the planks with mud. The planks will of course be removed before the next season's flood arrives.

17. DISCHARGE CHARACTERISTICS AND DISSIPATION OF ENERGY BELOW A SUBSIDIARY DAM SPILLWAY—HIRAKUD DAM PROJECT

Introduction

The object of these experiments was to determine—

- (i) whether the spillway crest as designed was free from negative pressures for a maximum overflow of 15 ft.,
- (ii) whether blocks or end-sills were best suited for energy dissipation.

The subsidiary dam is a simple overflow weir with crest at level E.L. 500, spanning 92 ft. of channel symmetrically divided by an 8-ft. pier. The end-sill 10 ft. high is placed at the end of a 50-ft. radius bucket and a horizontal apron 30 ft. long at E.L. 407. The maximum head is 15 ft. of water with upstream pool R.L. 515. The minimum T.W.R.L. is 427 and top of guide wall is at E.L. 463.

Dimensions and locations of baffle blocks were to be obtained from the test results.

For the tests, an existing flume was remodelled to a width of 2.3 ft. for model construction to geometrically-similar scale 1/40. The bed material for scour studies after a few tests was taken to be 1/8 inch to 3/8 inch gravel.

Initial tests for scour were made with bucket design as supplied. For tests with baffle blocks, wooden blocks were experimented on at different locations, with end-sill in position. With suitable block designs, the concrete end-sill was replaced by wooden sill, to test different heights and locations.

Pressure Tests

Observations were made with upstream R.L. at 505, 510, 515 and 520. The maximum discharge pool level is R.L. 515. Due to sufficient overfall, the pressure conditions tested were independent of the tailwater.

The pressures were found satisfactory and positive for the working range of pool elevation (Figure 1). The design as supplied from the crest, consisting of a straight segment and a circular arc up to the crest, followed by another circular arc and a parabolic arc up to the downstream glacis could possibly be simplified without detriment to the positive pressures on the crest, but since the pressures were safe, it was not considered necessary to change it.

Scour Tests

The experiments were made with the worst condition assumed, *viz.*, upstream pool E.L. 515. and T.W. minimum at E.L. 427. This would of course ensure the best performance, but the design might be well conservative.

1 : 10 Scale Model of Khodshi Weir on Krishna River near Karad. (Bombay State).

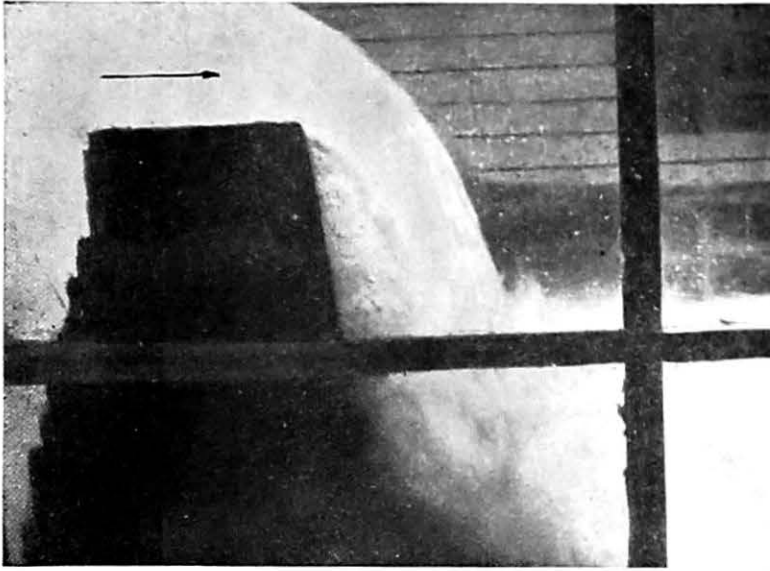


Photo No. 6952.

Showing the freely falling nappe with upstream water level 142.5 and downstream water level 126.0. The nappe pocket is fully aerated.

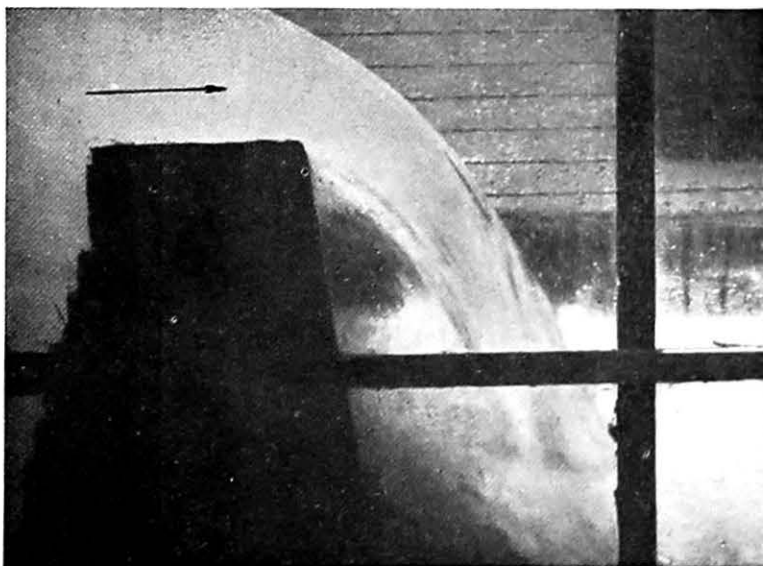
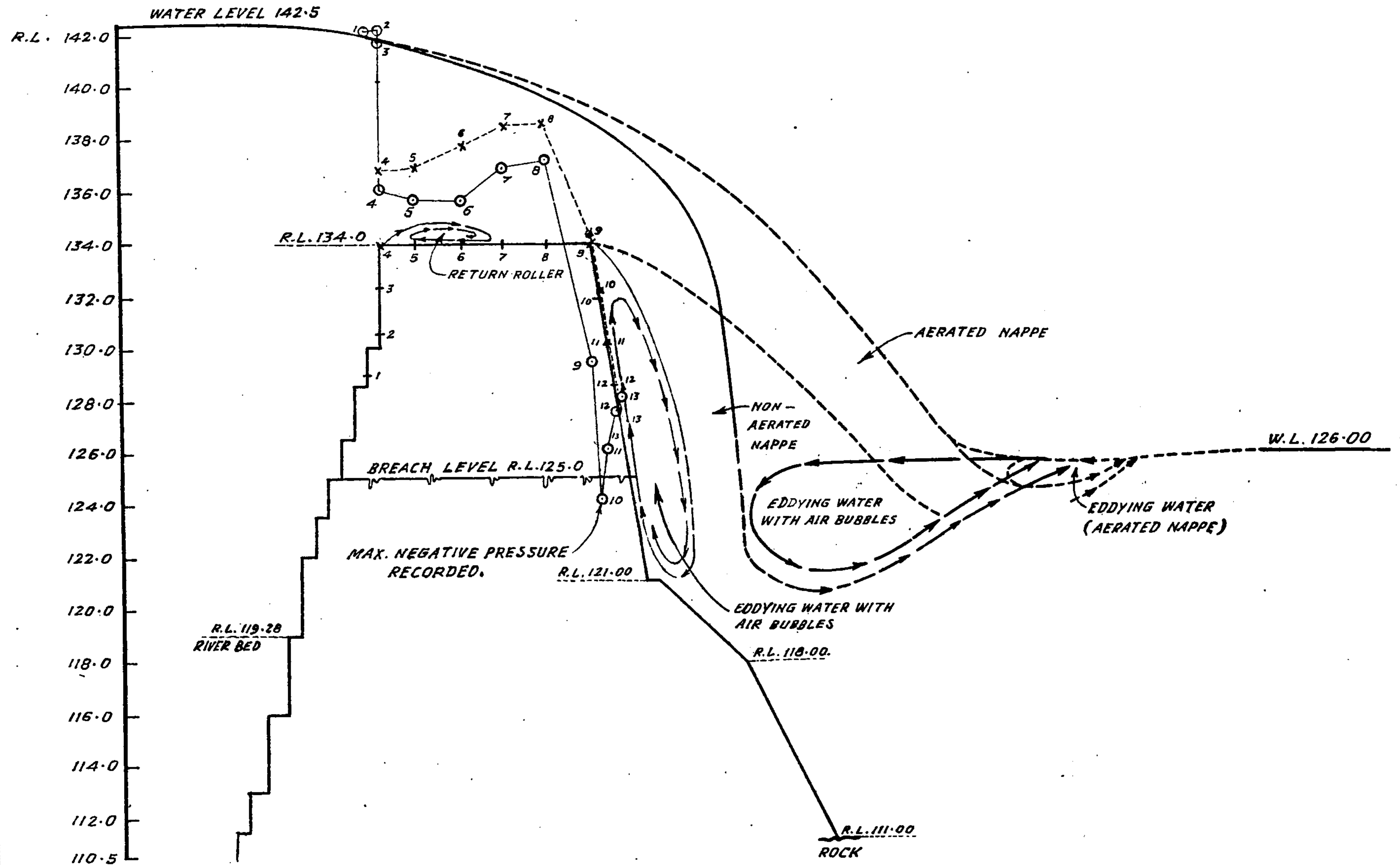


Photo No. 6953.

Downstream water level = 126.0, showing the nappe almost clinging to the downstream face of the weir. There was partial vacuum in the nappe pocket which was filled with eddying water and entrained air bubbles.

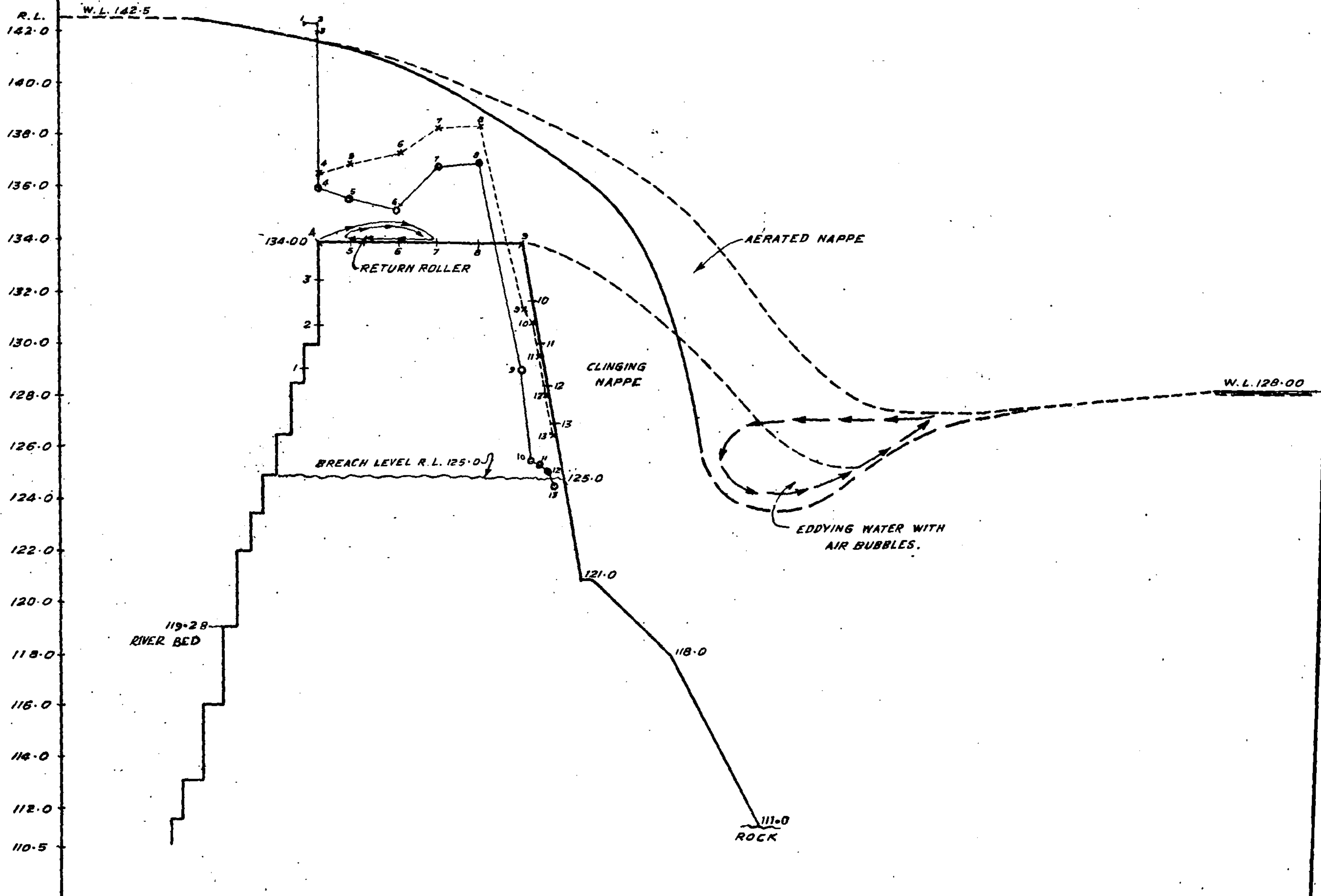


REFERENCE

- MAX. PRESSURE. -----x-----x
- MIN. PRESSURE -----o-----o
- PROFILE OF AERATED NAPPE:-----
- PROFILE OF NON AERATED NAPPE:-----

FIG-3
KHODSHI WEIR
 ON KRISHNA RIVER AT KARAD
 PRESSURE DISTRIBUTION CURVES
 AND
 WATER SURFACE PROFILES
 SCALE - 1 INCH = 4 FT.

COMP. BY *Meji*
TRACED BY SAKPAL



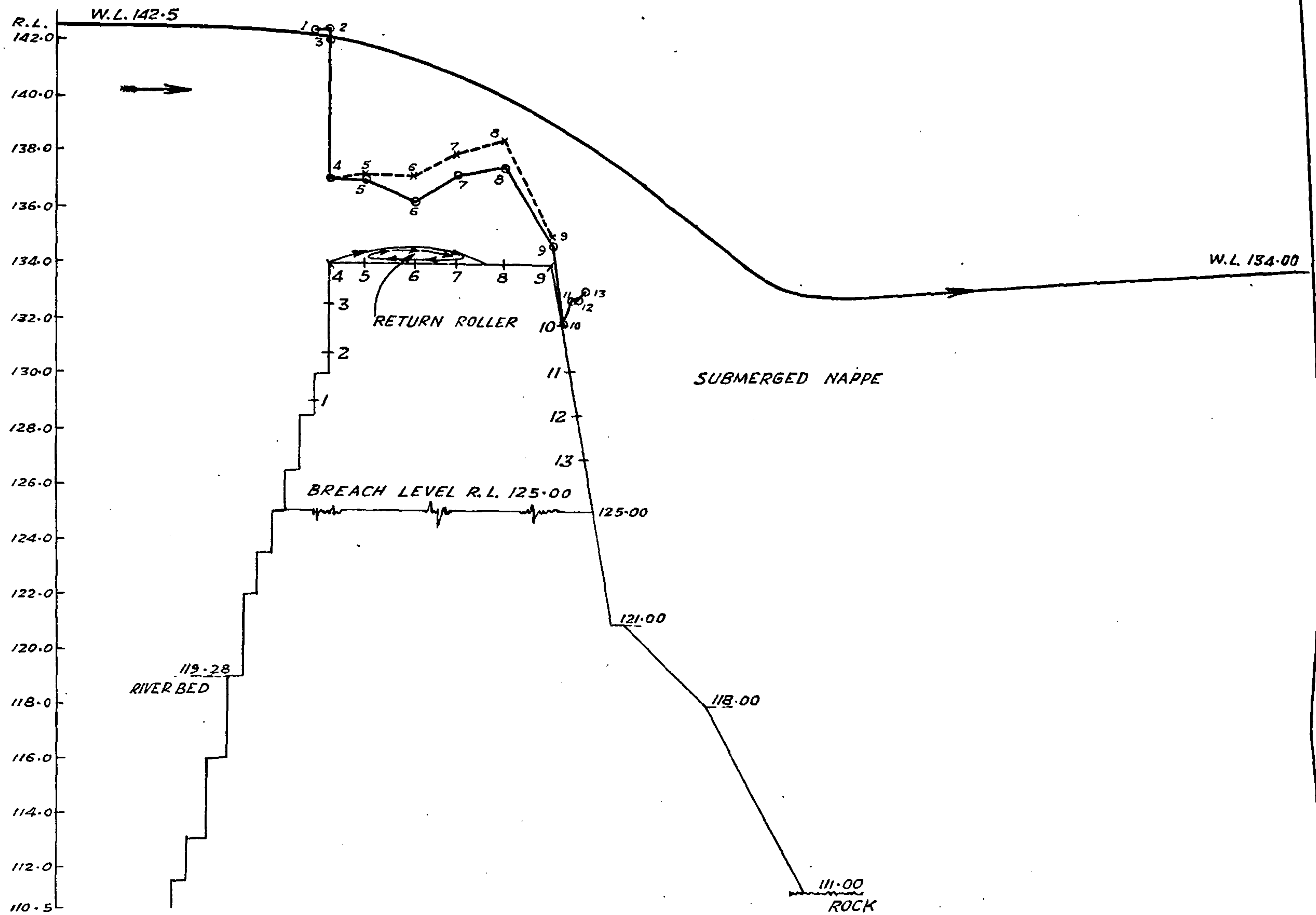
REFERENCE:-

- MAXIMUM PRESSURE - - - * - - -
- MINIMUM " - - - • - - -
- PROFILE OF AERATED NAPPE - - - - -
- PROFILE OF NON - AERATED NAPPE - - - - -

FIG. 4

KHODSHI WEIR
ON KRISHNA RIVER AT KARAD
PRESSURE DISTRIBUTION CURVES &
WATER SURFACE PROFILES
SCALE: 1"=4'

COMP. BY: *Manglik*
TRACED BY: PARKHI.
4-51



REFERENCES:-
 MAX. PRESSURE *-----*
 MIN. PRESSURE ○-----○
 PROFILE OF NAPPE —————

FIG. 5
 KHODSHI WEIR
 ON KRISHNA RIVER AT KARAD
 PRESSURE DISTRIBUTION CURVES
 AND WATER SURFACE PROFILE
 SCALE: -1" = 4'

The initial bed for test purposes has been taken at the bucket bed EL. 407; one test was however made with bed levelled up to the top of the end-sill (EL. 417 shown as the assumed line of excavation in the blue print received) though it appeared that this involved very high velocity flows.

The initial experiments for scour with 1/8 inch to 3/8 inch gravel bed were made with the bucket and end-sill as per design. It was found that the structure then acted like a ski-jump bucket as the apron length was quite insufficient; the scour given here for comparative purposes was a maximum of 25 ft. only compared with about 6 ft. obtained in the test with the final design.

Experiments with the end-sill in presence of baffle blocks of size 5 ft. x 5 ft. x 4 ft. were made for different locations and arrangement of the blocks. It was found that location of 9 blocks at 10 ft. centres with their centre line at 17 ft. upstream from the upstream face of the end-sill functioned satisfactorily in that all possibility of a clear jump over the blocks and spray formations were eliminated (Figures 2, 3, Photo 7225). The maximum scour in these tests was reduced to below 10 ft.

Fig. 2, 3,
Photo 7225.

From these tests, blocks with approximately stream-lined sides were made (Figure 4). As the blocking was thereby increased, only seven of these stream-lined blocks were used at 12 ft. centres. But they were found slightly insufficient, by this arrangement the frontage area of the blockage decreased considerably, with the result that the jet shot past the blocks at minimum tailwater with maximum discharge. Hence these were replaced by 6 ft. high blocks which proved satisfactory, the total blocking now being

Fig. 4.

$7 \times 6 \text{ ft.} \times 5 \text{ ft.} = 210 \text{ sq. ft.}$ compared to

$9 \times 5 \text{ ft.} \times 5 \text{ ft.} = 225 \text{ sq. ft.}$ in the previous test.

End-sills lower than the one per design, when tried, proved insufficient to avoid shooting flow over the blocks at minimum tailwater. The original end-sill height was therefore retained.

It was also found that spray formation for maximum flood even up to T.W.R.L. 422 was completely prevented, if the end-sill along with the blocks were receded by 5 ft., making the apron length 35 ft. instead of 30 ft. Since however the tailrace which discharges into the main river Mahanadi within a few hundred feet is hardly likely to go below R.L. 429 in the dry season, this extension is not necessary.

Tests were also made for comparison purposes with the use of splitters on the downstream glacis of the spillway. Splitters consisting of two horizontal deflectors on the downstream face of the dam, the lower one being continuous and the upper one consisting only of a number of equal segments, were found to function very satisfactorily in previous tests with Vaitarna spillway model¹ where a high over-fall was available. In this case of comparatively low fall, splitters were tested placed about 20 ft. below the crest. The throw obtained was as expected, somewhat more than 100 ft., the aeration was satisfactory, but the scour, still of the order of 20 ft., was much less satisfactory than with the apron, end-sill and friction blocks.

Recommendations

In view of the positive pressures, the spillway crest profile is considered satisfactory. For energy dissipation under the worst conditions, the apron length of 30 ft. is satisfactory for T.W. minimum of R.L. 427. Seven baffle blocks 6 ft. high, 5 ft. wide on the upstream face, curved in plan, 5 ft. long in the direction of flow and placed at 12 ft. centres with upstream face 19 ft. from the upstream face of the sill will ensure satisfactory performance. In view of the worst tailwater conditions assumed, the design is conservative.

¹ See Annual Report (Technical) 1949.

18. PERFORMANCE OF VOLUTE SIPHON NO. 7 AT HIREBHASGAR

Introductory

Of all the eleven siphons installed at Hirebhasgar, the one subject to regular, full scale experimentation is No. 7 Siphon. When it was ready to take up the floods in 1946, it had the same dimensions as the rest, viz., 18 ft. diameter and 58 ft. siphon height.

During 1947 a ledge of one foot was built at the throat, thus creating a local diameter of 16 ft. This gave only a discharge of 3,000 cusecs¹.

During 1948, this local constriction at the throat was carried downwards to the top of the bend and left at that, the lower bend and the outlet remaining 18 ft., there probably having been insufficient time to complete the work. The siphon ran full till that point and the flow separated thereafter and the discharge was about 6,300 cusecs². The R. C. C. parapet on the top of the dam was found vibrating and a metallic thud sound was audible. No serious damage was however discovered at the end of the experiment.

During 1949, a sloping ramp was added at the outlet to a depth of two ft., thus, in effect, reducing the exit area. Lack of time again came in the way of completing the construction in the bend portion. The siphon primed and apparently ran full-bore, though it indicated reduced velocity³. At the end of the experiment a big scour hole was discovered at the beginning of the outer wall of the bend which was subsequently sealed by introducing R. C. C. arched ribs and guniting. The probable reason for this damage was given in Annual Report (Technical) 1950.

In 1950, the whole length of the siphon was made 16 ft. in diameter throughout, including the bend; but as the inflow at the time of experimentation was not sufficient, it did not function.

During the same year, several experiments were conducted at the research station, including the determination of the optimum constriction of the exit diameter which would make the design cavitation-free,⁴ and hence dependable. As a result of these experiments a conical steel tip of exit diameter 16 ft. was added to Siphon No. 10 as an experimental measure—*vide* Photograph 356.

Arrangements were made for velocity and pressure observation; the performance during the 1951 floods was watched. The dome windows of all other domes except that of Siphon No. 10 were kept open in order to delay their priming; but in spite of this precaution, Siphon 7 primed, and as the inflow during experimentation was not enough for a second siphon to prime, Siphon No. 10 could not be tested as programmed.

Behaviour of Siphon No. 7

The priming of Siphon 7 earlier than Siphon 10 should be attributed to the smaller diameter obtaining with the former siphon.

As the dome window of Siphon 7 was open, the siphon was not running quite full. It was therefore closed with cement mortar.

The working of the siphon could not be considered as satisfactory. After the dome window was closed, the outlet ran full more frequently than before. Most of the time the water coming out had a frothy appearance indicating considerable air entrainment; the length of the trajectory was very much less than what one would expect from the high head under which it was acting; besides, it was shifting

¹ Hydraulic Research Station, K. R. S. ART. 1947

² Hydraulic Research Station, K. R. S. ART. 1948.

³ C. W. & P. R. Station, ART. 1950.

⁴ C. W. & P. R. Station, ART. 1950.

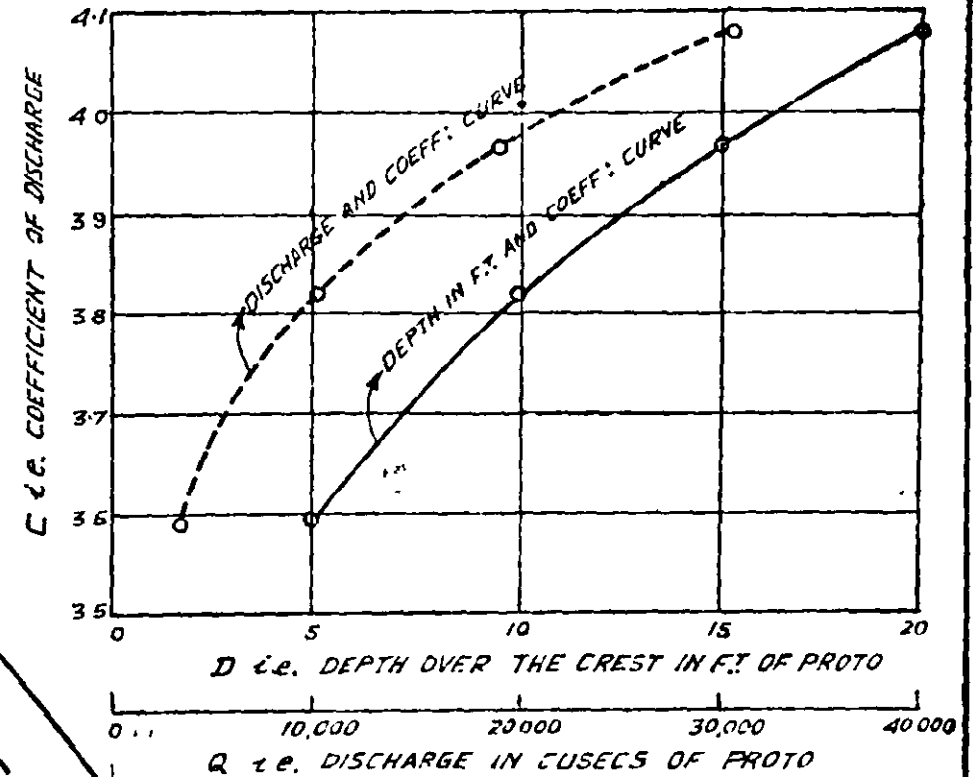
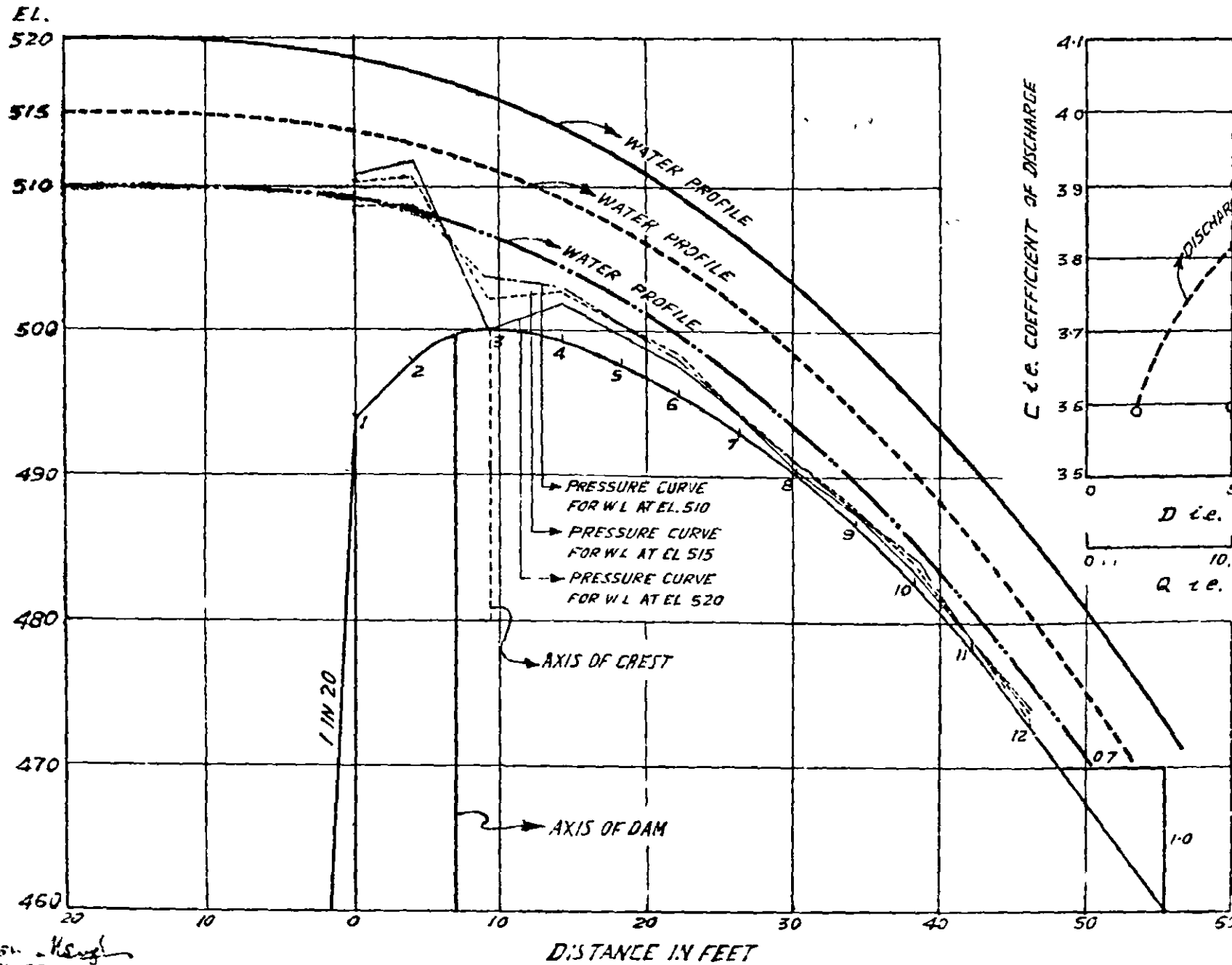
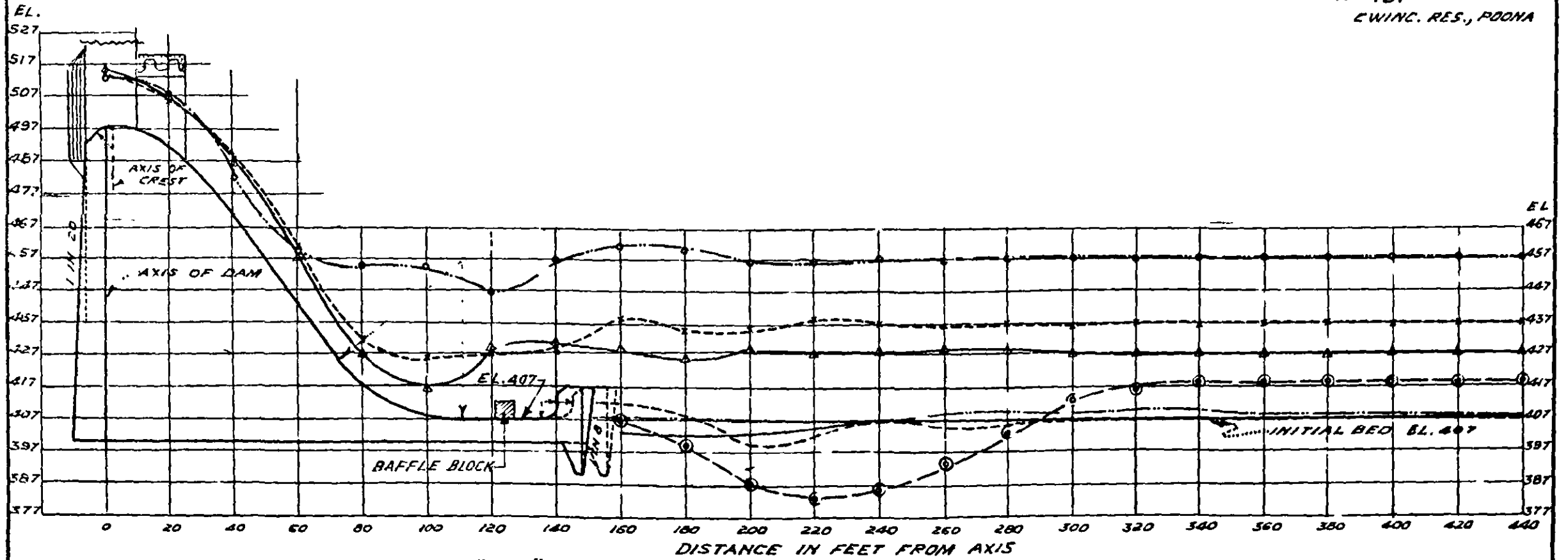


FIG. 1
HIRAKUD DAM PROJECT
HYDRAULIC MODEL 6
1/40 G S. MODEL
OF
SUBSIDIARY DAM
PRESSURE DISTRIBUTION
&
DISCHARGE COEFFICIENTS

COMPO. BY H.S. [Signature]
TRACED BY E.S. DESAI ON 11-9-52.



BED MATERIAL : 1/8" - 3/8" GRAVEL

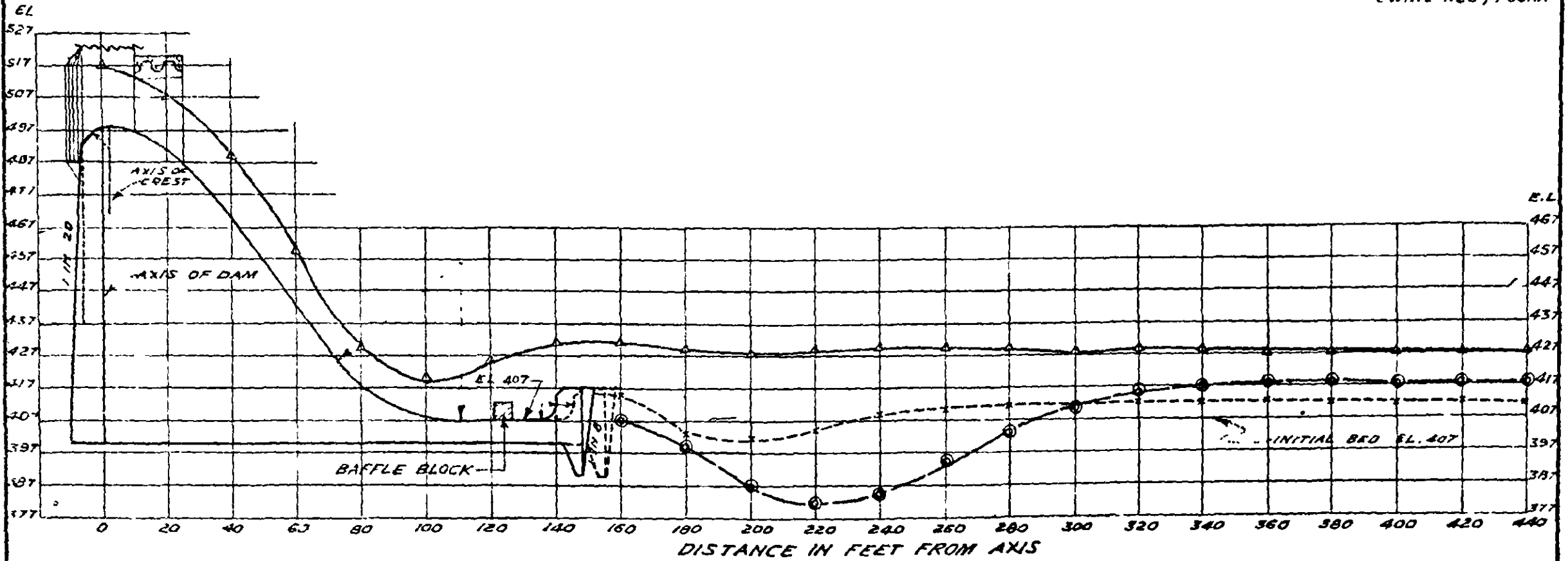
REFERENCES		WATER SURFACE		SCOURED BED	
U/S WATER EL.	D/S WATER EL.	Symbol	Symbol	Symbol	Symbol
515	427	△	△	—	—
515	437	x	x	- - -	- - -
515	457	○	○	—	—
515	427	○	○	○	○

WITH FINAL LOCATION OF
BAFFLE BLOCKS & SILL

SCOUR WITH NO BAFFLE BLOCKS

FIG. 2
HIRAKUD DAM PROJECT
HYDRAULIC MODEL 6
1/40 G. S. MODEL OF SUBSIDIARY DAM?
C. D. O. DESIGN WITH BAFFLE BLOCKS
SCOUR TESTS
40 30 20 10 0 20 40 FEET
SCALE

COMP BY -
TRACED BY - PATKIE G. R.
26-4-51.



BED MATERIAL : $\frac{1}{8}$ - $\frac{3}{8}$ GRAVEL

REFERENCES

U/S WATER EL	D/S WATER EL	WATER SURFACE	SCOURED BED
515	427	A-----B	X-----X
515	427	SCOUR WITH NO BAFFLE BLOCKS	○-----○

TESTS WITH FINAL LOCATION OF BLOCKS
WITH INITIAL BED EL. 417 AND
U/S R.L. 515. D/S R.L. 427.
D/S BED STABILISED AT EL. 413
MAX SCOUR EL. 403

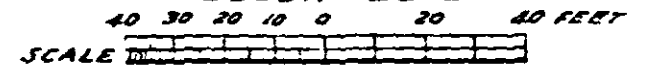
COMP BY-
TRACED BY- DATKIE G R 2-S-51.

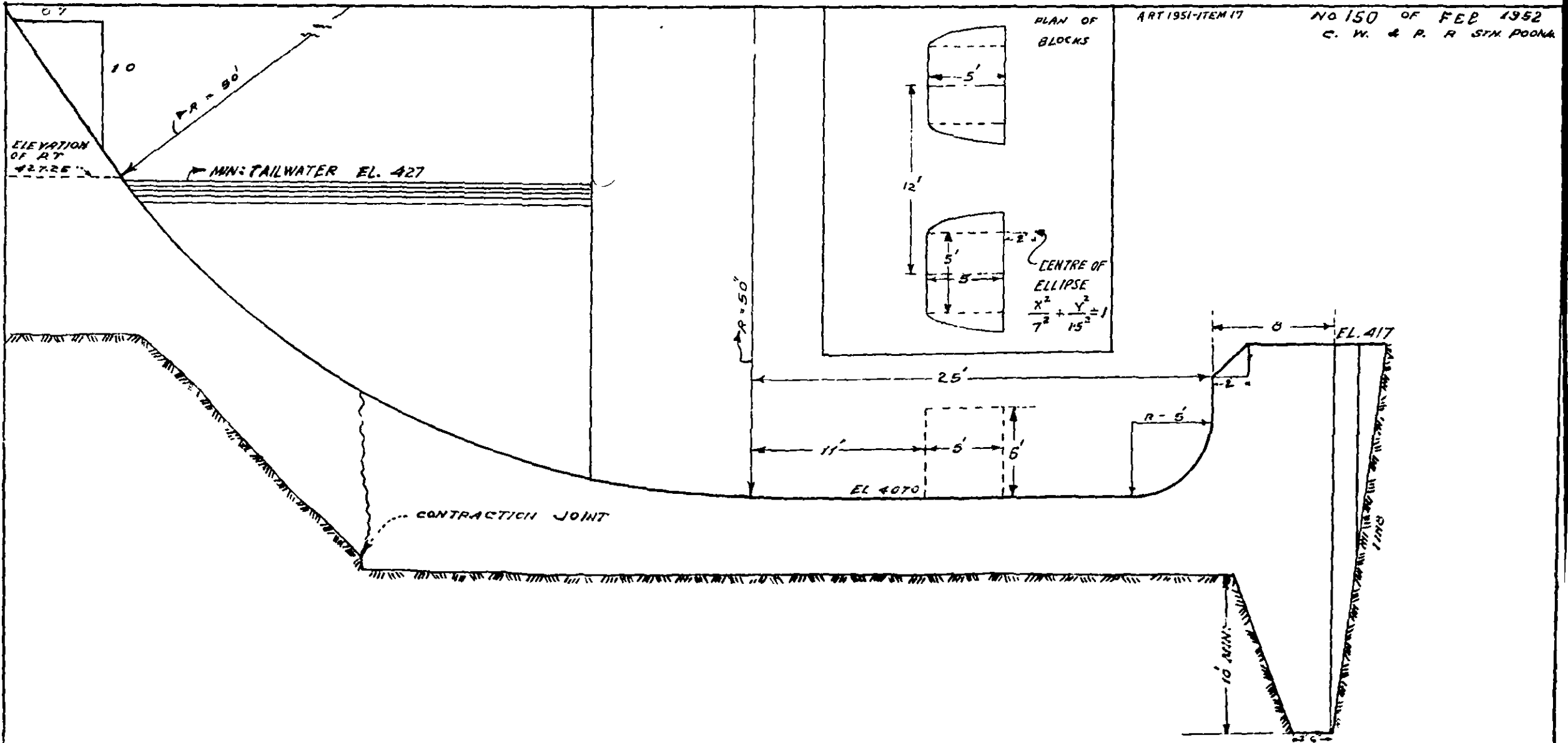
FIG. 3

HIRAKUD DAM PROJECT

HYDRAULIC MODEL 6

1/40 G.S MODEL OF SUBSIDIARY DAM
C.D.O. DESIGN WITH BAFFLE BLOCKS
SCOUR TESTS





ART 1951-ITEM 17
 NO 150 OF FEB 1952
 C. W. & P. R. STH. POONA

FIG 4
 HIRAKUD DAM PROJECT
 HYDRAULIC MODEL 6
 1/40 G.S. MODEL OF SUBSIDIARY DAM
 BAFFLE BLOCKS - RECOMMENDED DESIGN
 WITHOUT EXTENSION OF APRON LENGTH

TRACED BY TAKALKAR 10-1-52

ITEM NO. 17.
A. R. T. 1951

Hirakud Dam Project.
Subsidiary Dam Spillway

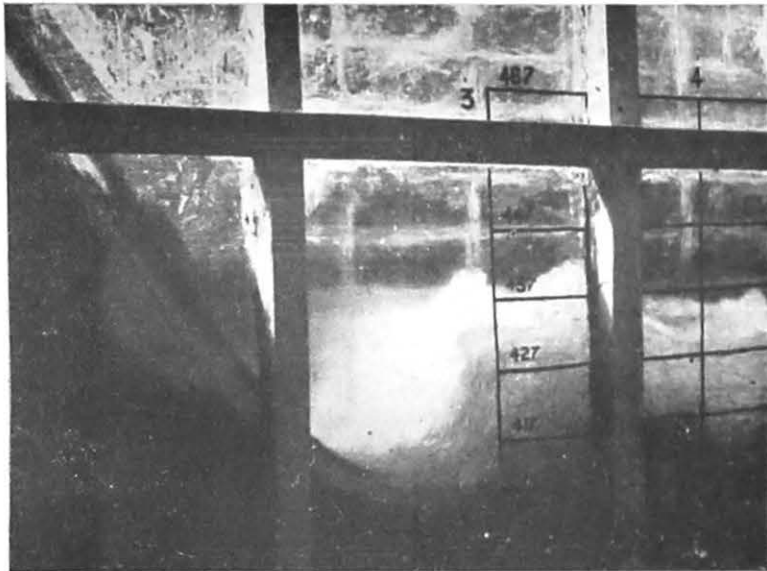


Photo No. 7225
Side view of Model. Action of blocks and end sill
for energy dissipation, and avoidance of spray
formation
Upstream W. L. 515, Downstream W. L. 427.

Volute Siphons at Hirebhasgar (Mysore)



Photo 271
Hirebhasgar dam (1951)

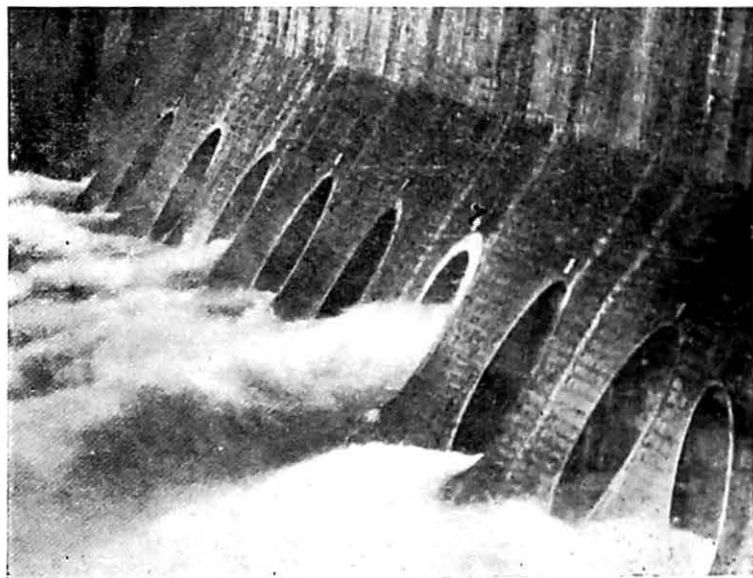


Photo 355
Hirebhasgar siphon outlets.

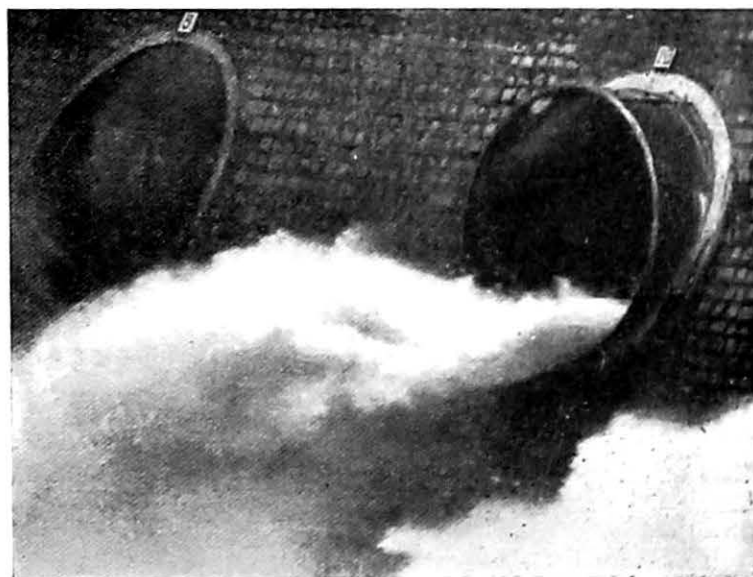


Photo 356
A close - up of siphon No. 10.

ITEM No. 18.
A. R. T. 1951

Volute siphons at Hirebhasgar (Mysore)

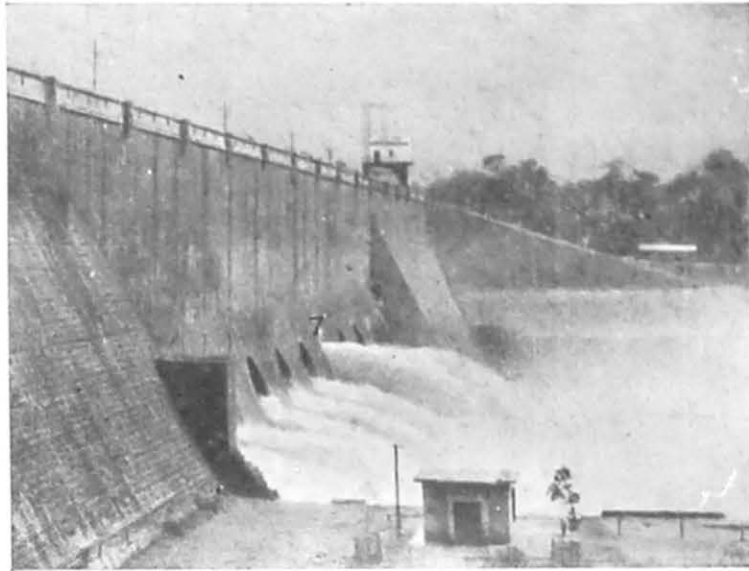


Photo 358

**Shed for recording Vibrations at ground level, seen in the foreground
View of Hirebhasgar Dam (rear) from the Right bank, Just
before siphon 7 primed. (See the shed for Vibration
experiments)**



Photo 361

Siphon 7 primed (1951) outlet erratically running.

Volute siphons at Hirebhasgar (Mysore)

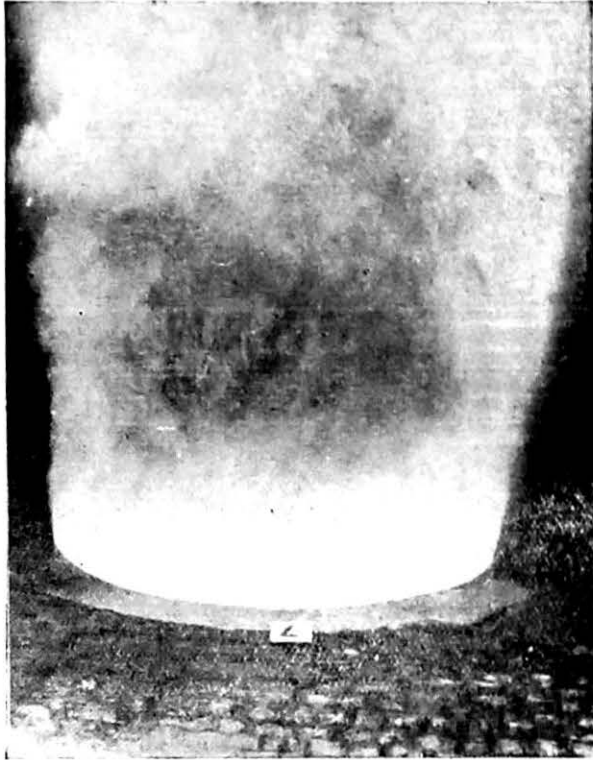


Photo 359
View of jet of siphon 7 from top of dam



Photo 360
Vortex formed near the dome of siphon 7.

in all directions, water rushing out in intermittent gushes. The flow appeared to be very unsteady. Vibrations were felt which gave the feeling of a hammering sound and the approximate rate at which this occurred was about five per second. Besides, big explosions were heard at frequent intervals. Near the dome there were anticlockwise vortices towards the dam side, the most frequent one being on the left side facing upstream (see Photograph 360). The pressure in the dome varied from -57 to -99 ft. of water. Photo 360.

The fall in the water surface from Siphon No. 1 to Siphon No. 7 was three inches and from 11 to 7 (with the undersluices working) $1\frac{1}{2}$ inch. From front to rear of the dome, the fall was three inches to the left and one inch to the right (facing upstream). All these should be expected to vary in accordance with the number of siphons functioning—a matter for consideration of the suitable design of the deprimers.

As it was feared that serious damage might occur in the body of the siphon, the undersluices were opened. The vibrations continued until, at the end of about 11 hours, the siphon began depriming.

Depriming

First, the two upstream deprimers (R.L. 1776 20) were exposed and air was intermittently sucked in; after about two hours, the downstream deprimers (R.L. 1775·12) also began to function (water level at Siphon No. 1 being R.L. 1775·40).

During depriming, the variation of pressures (one of the front deprimers working) varied from -8 to -9 ft. The vibrations during depriming were not significant and finally the siphon deprimed. During depriming, the siphon flow reduced and the outlet was running partially and the jet was clearly marked. The depriming occurred at R.L. 1775 05, the lake being depleted by 0·95 ft. below F.R.L.

Observations

Afterwards, the sluices were closed and the siphon was inspected and the following information was obtained:

Figure 1 shows the damage done to the gunited plastered surface. Some of the reinforcement bars were exposed and at some places, the depth of scour was of the order of nine inches. Curiously enough, the convex side of the bend did not show any damage. It will be noted that the region of damage was lower down in the barrel on the upstream side. On further study, it was deduced that the cavities which formed at the throat might have collapsed in this region. These collapses were heard as explosions referred to before. This behaviour has well been explained in the symposium on cavitation, ASCE Sept. 1945. Fig. 1.

It was also deduced that air released from water on the concave side travelled towards the convex side thus reducing the negative pressure and hence no pitting occurred. Another explanation for this may be that though cavities might have formed on the convex side, they might not have collapsed within the short length of outlet. These points can best be studied only in a cavitation tank.

19. VIBRATION STUDIES AT HIREBHASGAR DURING THE WORKING OF SIPHON NO. 7 AND UNDERSLUICES

1. Stations

T—Average period of individual wave (in a wave-group in secs.).

A—Corresponding maximum amplitude (absolute or relative).

P—Average period of wave-groups in seconds.

V—Maximum velocity of vibration (absolute or relative).

P_0 —Period of individual burst on the dome of Siphon No. 7.

V_0 —Corresponding (to P_0) maximum relative velocity associated with individual burst.

H—Horizontal component of vibration perpendicular to the axis of the Dam.

H_0 —Horizontal component of vibration parallel to the axis of the Dam.

W—Record with Wood-Anderson Seismograph.

G—Record with Electromagnetic Geophones.

F—Record on the foundation, in the seismograph room shown in Figures 6 *Fig. 6 & 6a* and 6A.

1, 2, are the stations of observation on the Dam (Figure 6).

D—Station of observation on the Dome as shown in Figure 6.

a—Acceleration— $\frac{4\pi^2 A}{T^2}$

V—Velocity— $\frac{2\pi A}{T}$

2. Introduction

Hirebhasgar Dam has been built across the river Sharavati on the eastern slopes of the Western Ghats as water reservoir for the Hydroelectric Power Generating Station at Jog. Figures 1 and 6 show cross-section and plan respectively of the Dam. The Dam consists of three parts, namely, the central masonry portion and the two side earth embankments. Figure 1A gives a portion of the profile of the river valley.

Fig. 6. As shown in Figure 6, the dam is provided with eleven Ganesh Iyer Type volute siphons in addition to the usual undersluices (6 in number). The entire central portion of the dam is of masonry, whereas the siphons have been built in cement concrete.

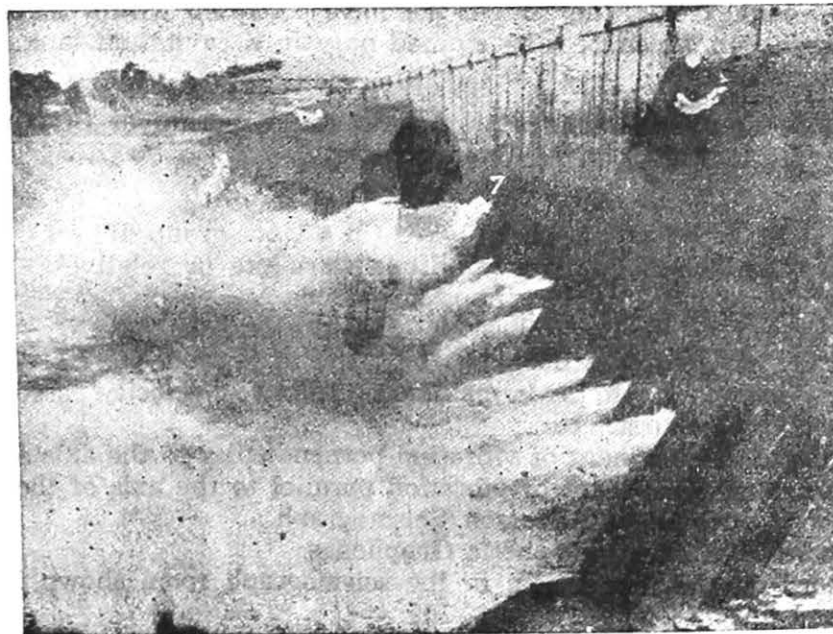
The undersluices have been designed to pass a total of 60,000 cusecs (10,000 cusecs each) discharge, whereas each one of the siphons has a diameter of 18 ft. and was designed for passing a discharge of 13,000 cusecs.

Experience has shown that these volute siphons, when allowed to function, generate considerable vibratory motion, both on the dam and on the domes of the siphons. This has given rise to doubts whether these siphons can function smoothly in conformity with the safety of the structure. Thus an investigation was undertaken regarding the vibration characteristics of the Dam [Stations 1 to 6 (Figure 6)], under various conditions of exciting force, namely—

- (1) siphons working as weirs (Discharge Total=2,177 cusecs),
- (2) the Siphon No. 7 in primed state,
- (3) the Siphon No. 7 in depriming state with undersluices, in function in both the cases, and
- (4) undersluices functioning alone.

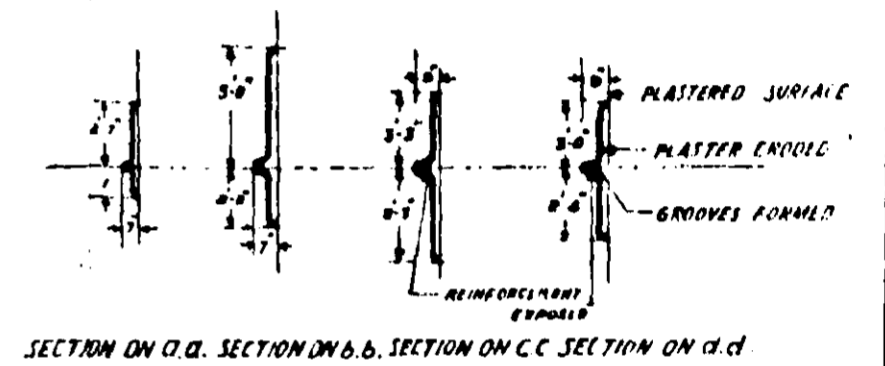
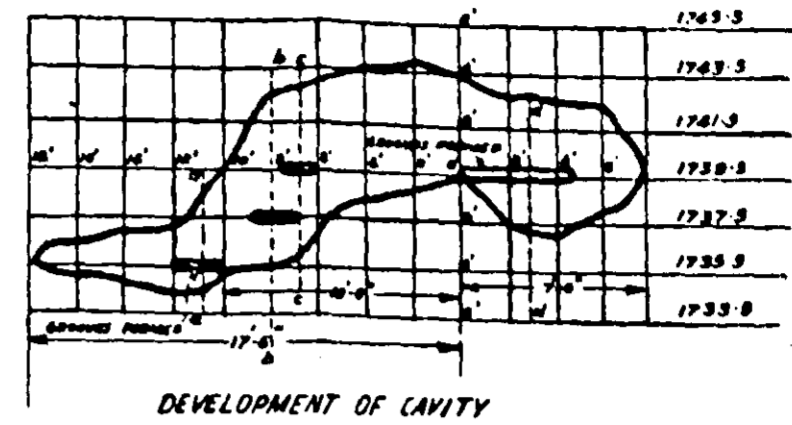
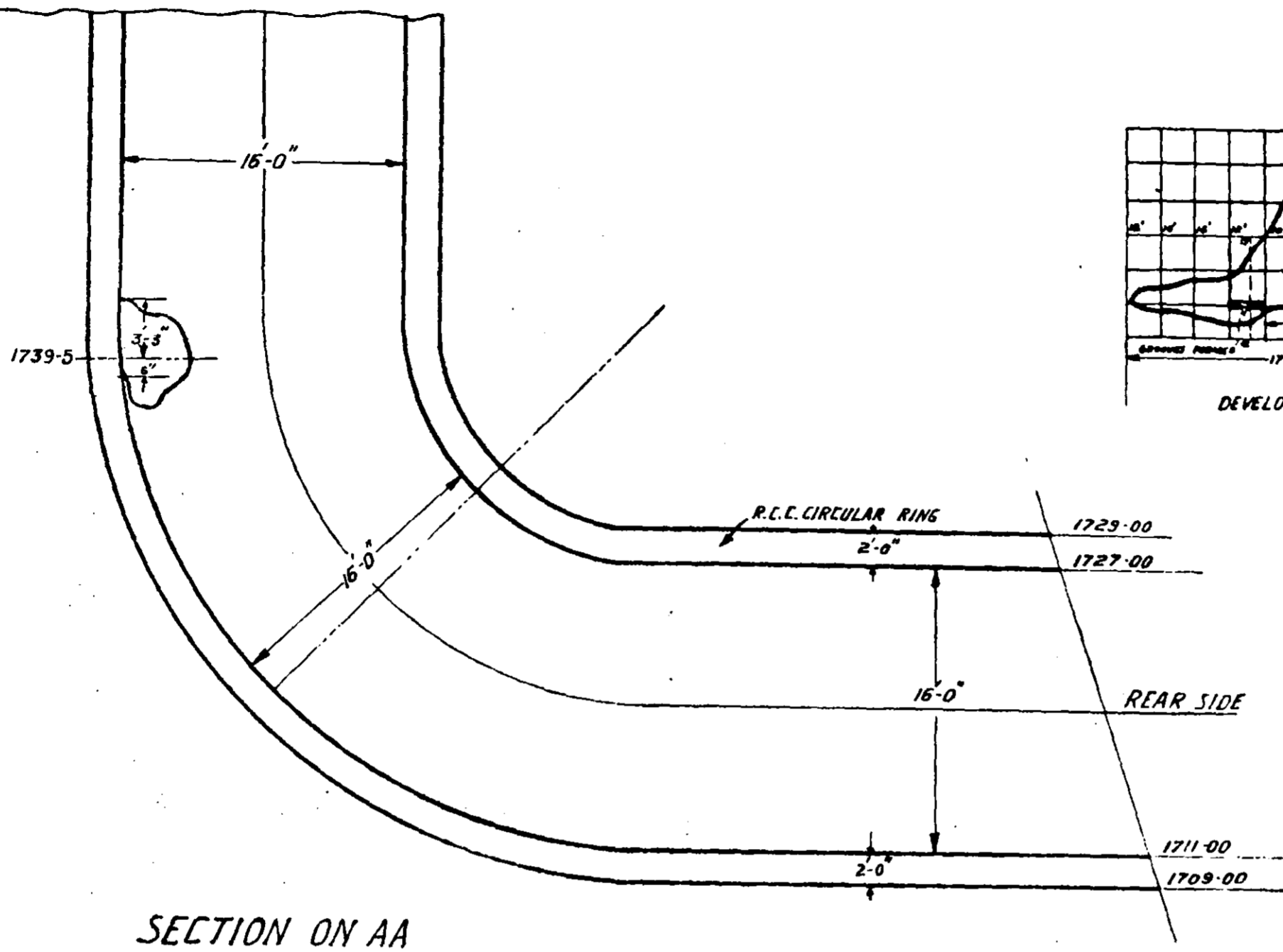
Observations were extended to the earthen embankments also.

In addition, various other fundamental aspects of the vibration characteristics of the structure and foundation were also undertaken, mainly, to help interpretation of the engineering properties in conformity with the boundary conditions existing at the site. The stations of observation are shown in Figure 6. Observations on the foundation were taken under an existing force due to siphons working only as weirs.

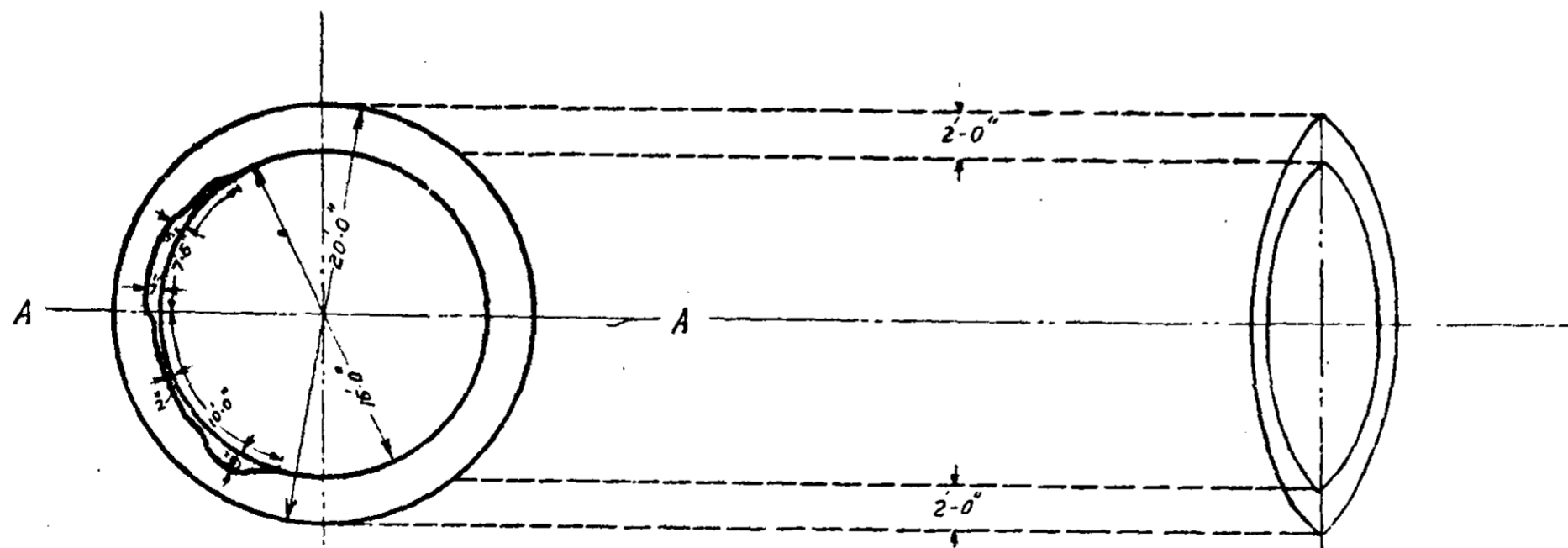


(REDUCED & COPIED FROM GOVT. OF MYSORE
M.S.E. WORKS DRG. NO. J. 1043.)

WATER SIDE



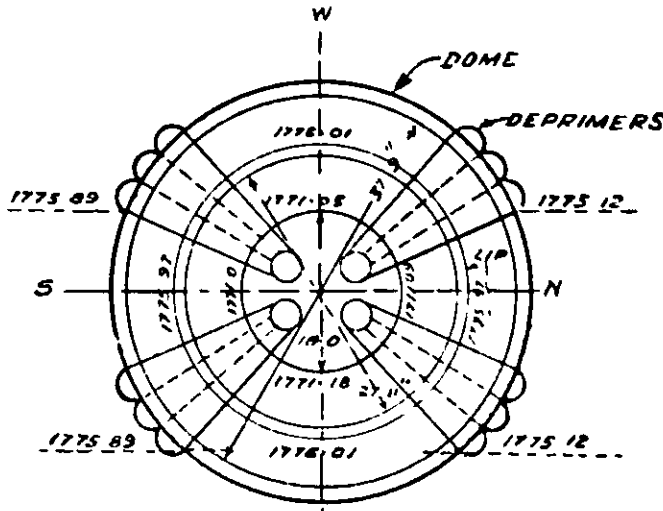
WATER SIDE



PLAN AT 1739.5

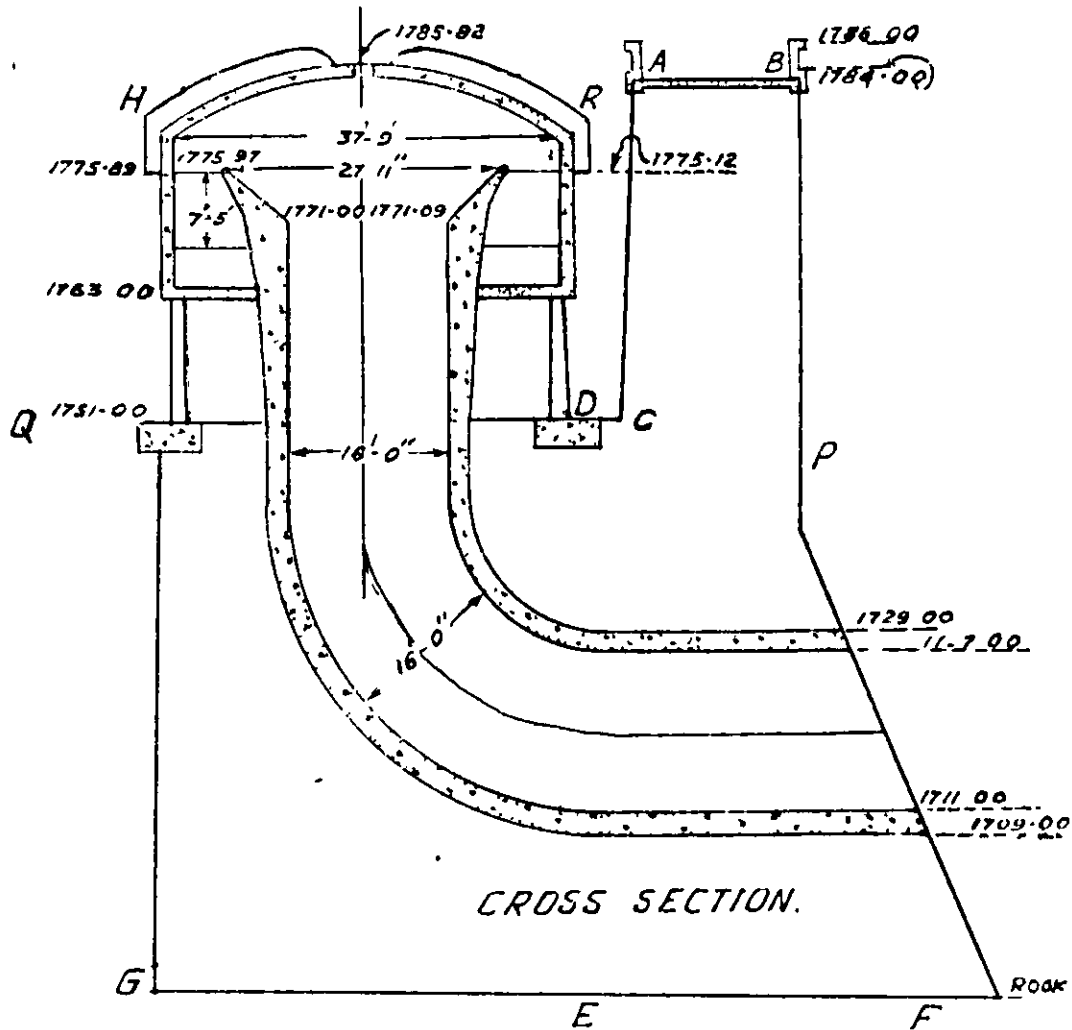
FIG.1
HIREBHASGAR DAM SIPHON No. 7
DETAILS OF DAMAGE AFTER OPERATING IT
ON
17-8-1951.
SCALE 1/4" = 2 FEET.

COMP'D BY:- M. S. S. S.
TRACED BY:- SABNIS, 10.10.51



PLAN

E



CROSS SECTION.

ROCK P.L. 1659

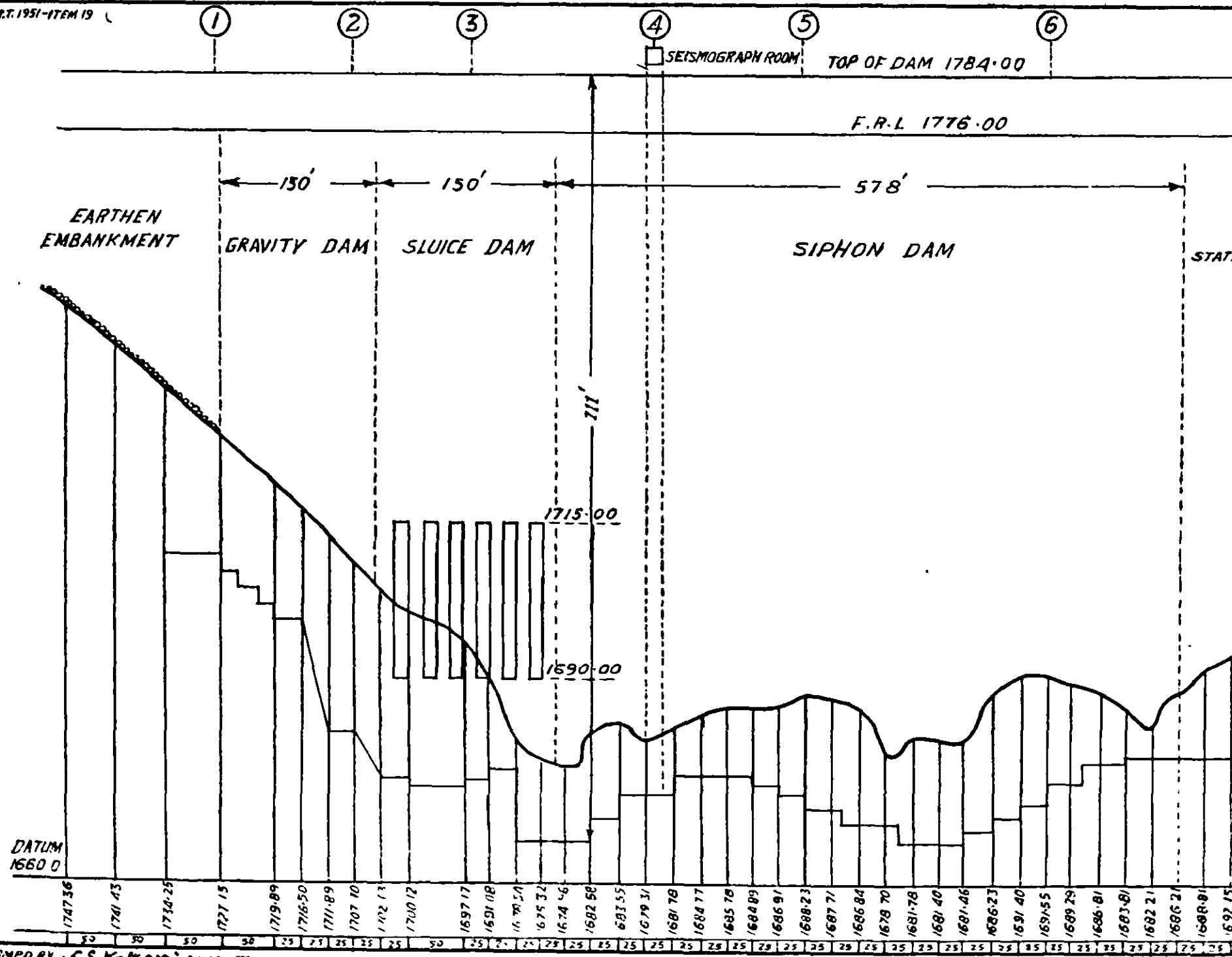
SI NO.	PARTICULARS.	
1	INSIDE DIA. OF DOME.....	37'-9"
2	INSIDE DIA. OF FUNNEL AT LIP LEVEL.....	27'-11"
3	DIA. AT THROAT.....	16'-0"
4	DIA. OF BARREL.....	16'-0"
5	AVERAGE LEVEL AT LIP OF FUNNEL.....	1775.99
6	AVERAGE LEVEL AT THROAT.....	1771.08
7	LEVEL AT TOP OF DOME.....	1785.82
8	LEVEL AT THE BOTTOM OF DEPRIMERS	
	FRONT (AV).....	1775.89
	REAR (AV).....	1775.12
9	OUTLET DIA - VERTICAL.....	16'-0"
10	OUTLET DIA - HORIZONTAL.....	16'-0"
11	REMARKS.....	

FIG 1
HIREBHAGSAR SIPHON
DETAILS OF R C C SIPHON
AS EXISTING IN JUNE 1951.

SIPHON NO.7
SCALE:-1"=20'

COMP BY
TRACED BY SAPRAL.

GUR



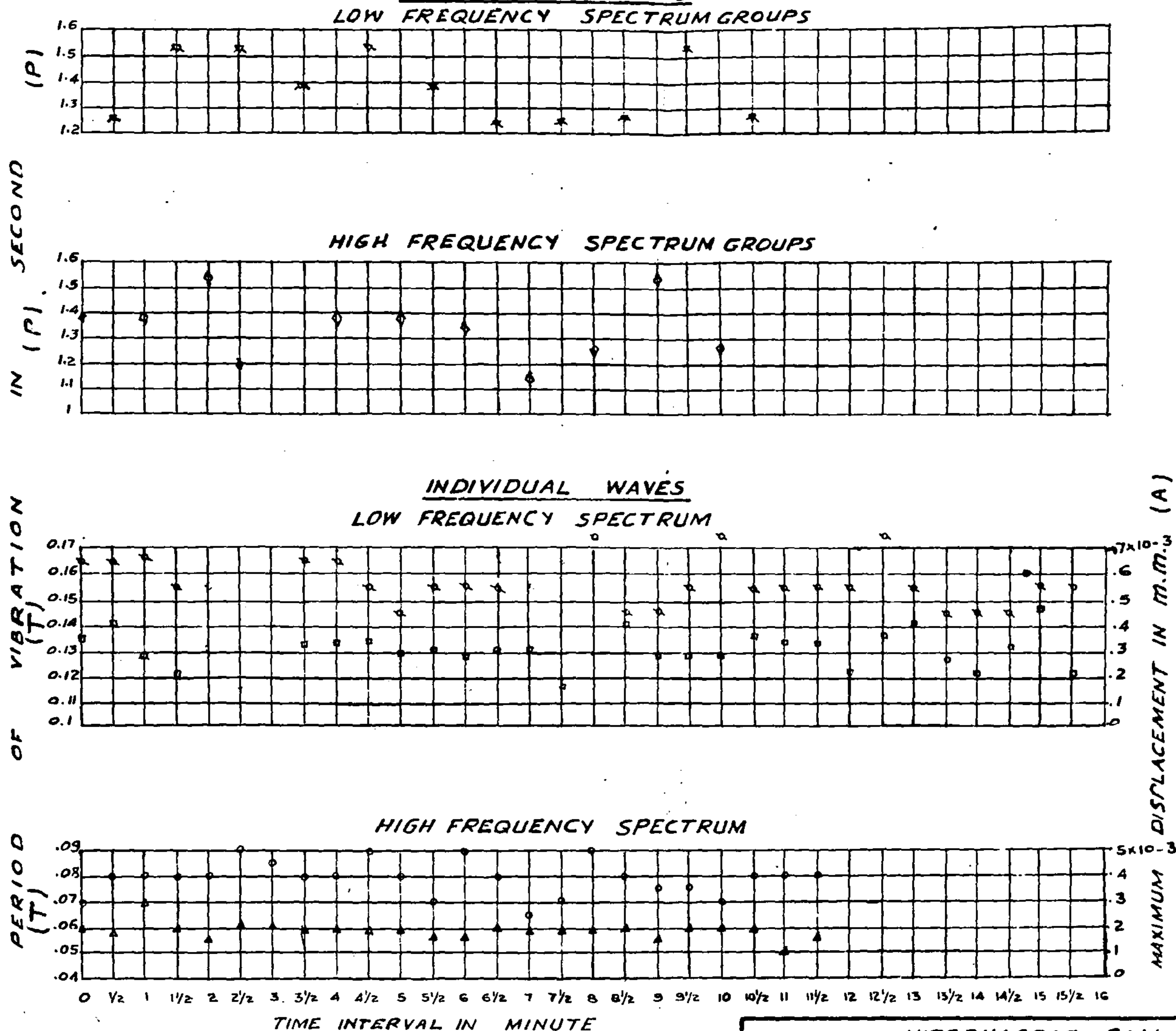
REFERENCE —
STATIONS OF OBSERVATION ON THE DAM. (1), (2) ETC

FIG. 1A.

HIREBHASGAR DAM
A PORTION OF THE PROFILE
OF
THE RIVER VALLY WITH
STATIONS OF OBSERVATION
ON THE DAM

SCALE: HOR 1" = 120'
VER 1" = 20'

FIG. 2
RECORD ON FOUNDATION WITH WOOD-ANDERSON SEISMOGRAPH PERPENDICULAR
TO THE AXIS OF DAM
MODULATED GROUPS

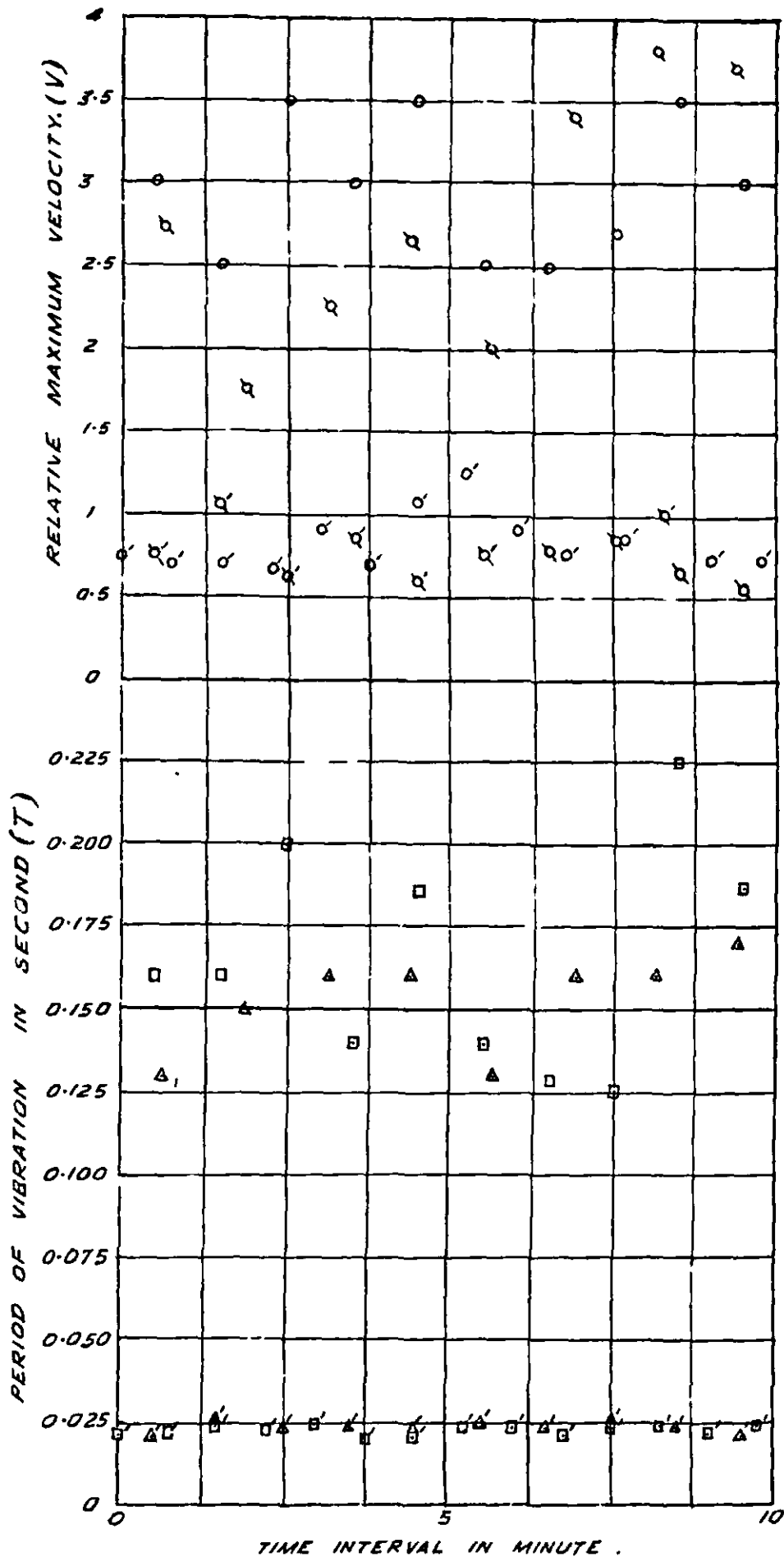


REF:-

PERIOD OF OCCURRENCE OF GROUP OF HIGH & LOW
 FREQUENCY SPECTRUM RESPECTIVELY Δ & \square
 MAXIMUM DISPLACEMENT IN WAVES OF HIGH & LOW
 FREQUENCY SPECTRUM RESPECTIVELY \circ & \square
 PERIOD OF VIBRATION OF INDIVIDUAL WAVES IN
 HIGH & LOW FREQUENCY SPECTRUM RESPECTIVELY Δ & \square

COMPIBY:-
 TRACED BY:- S.A. MANCHUNDIA.

HIREBHASGAR DAM - \perp FW
VARIATION OF PERIOD OF VIBRATION AND
OF MAXIMUM DISPLACEMENT WITH TIME ON THE
FOUNDATION ROCK WHILE SIPHONS WORKED
AS WEIRS
(TOTAL DISCHARGE 2177 CU SECS).
COMPONENT OF VIBRATION \perp TO AXIS OF DAM

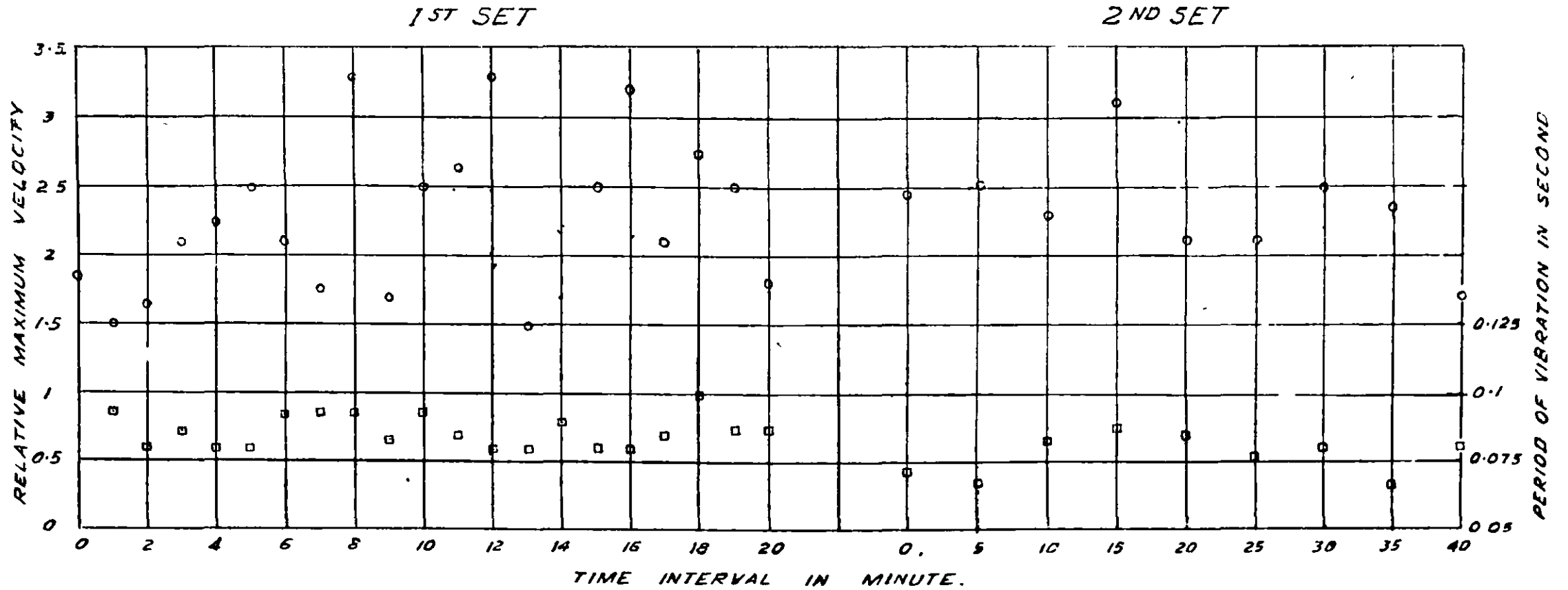


REFERENCES:~

- , □' PERIODS OF HORIZONTAL VIBRATIONS ⊥ TO AXIS OF DAM AND O, O' CORRESPONDING RELATIVE MAX VELOCITY ASSOCIATED WITH THEM
- Δ, Δ' PERIODS OF HORIZONTAL VIBRATIONS || TO AXIS OF DAM AND Δ, Δ' CORRESPONDING RELATIVE MAX. VELOCITY ASSOCIATED WITH THEM

Comp by *Mei*
By *P. G. K. H.* 27 11 51

FIG. 3
HIREBHASGAR DAM-FG
VARIATION OF PERIOD OF VIBRATION
AND ASSOCIATED RELATIVE MAXIMUM
VELOCITY ON THE FOUNDATION WITH TIME
WHILE SIPHONS WORKED AS WEIRS.
(TOTAL DISCHARGE 2177 CUSECS.)



REFERENCES:~

- PERIOD OF VIBRATION IN SECOND.
- RELATIVE MAXIMUM VELOCITY.

FIG. 4

HIREBHASGAR DAM-FG

VARIATION OF PERIOD OF VERTICAL VIBRATION &
ASSOCIATED RELATIVE MAXIMUM VELOCITY WITH TIME
ON THE FOUNDATION WHEN SIPHONS
WERE RUNNING AS WEIRS.
(DISCHARGE 2177 CUSECS)

COMD BY:- *[Signature]*
TRACED BY:- PARKHI. 30-11-51

RECORD ON DAM WITH
GEOPHONES.

FIG. 5.

HIREBHASGAR DAM-G

VARIATION OF PERIOD OF VIBRATION AND
RELATIVE VELOCITY WITH TIME AT VARIOUS POINTS
ON THE DAM AND ON THE FOUNDATION.

REFERENCES:-

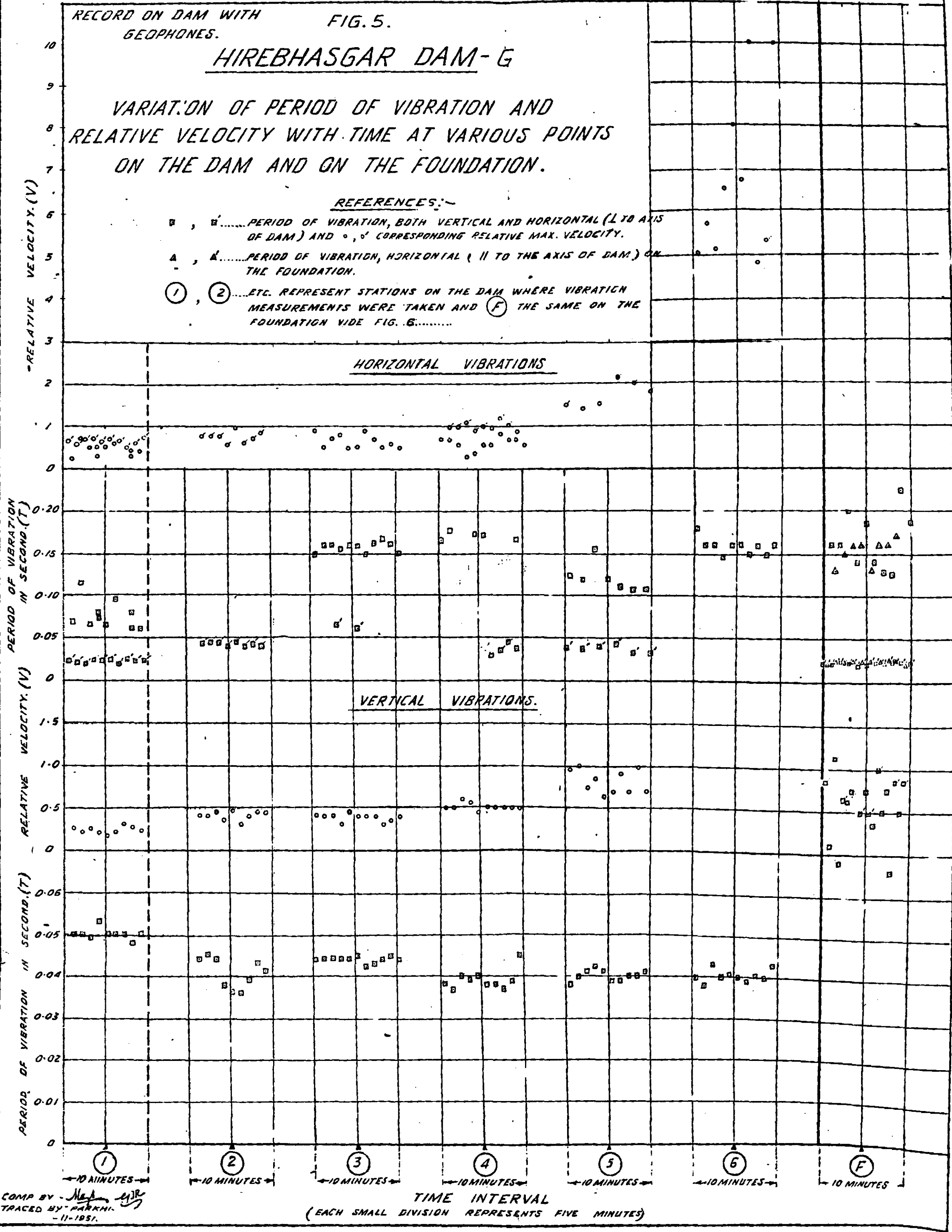
- , □ PERIOD OF VIBRATION, BOTH VERTICAL AND HORIZONTAL (⊥ TO AXIS OF DAM) AND ○ , ○' CORRESPONDING RELATIVE MAX. VELOCITY.
- △ , △' PERIOD OF VIBRATION, HORIZONTAL (|| TO THE AXIS OF DAM) ON THE FOUNDATION.
- ① , ② ETC. REPRESENT STATIONS ON THE DAM WHERE VIBRATION MEASUREMENTS WERE TAKEN AND (F) THE SAME ON THE FOUNDATION VIDE FIG. 6.....

PERIOD OF VIBRATION IN SECOND. (T)

RELATIVE VELOCITY. (V)

HORIZONTAL VIBRATIONS

VERTICAL VIBRATIONS.



COMP BY - *[Signature]*
TRACED BY - *[Signature]*
- 11-1951.

TIME INTERVAL
(EACH SMALL DIVISION REPRESENTS FIVE MINUTES)

A special investigation was done to assess the intensity of vibration on the dome of Siphon No. 7 (Station 'D' in Figure 6) itself as it is a light structure and so liable to be easily thrown into vibration. The observations were taken during the primed and depriming states of Siphon No. 7.

Fig 6

The hydraulic behaviour of the siphon is discussed elsewhere.

During this investigation, the reservoir was full up to R.L. 1777.5 with no appreciable tailwater.

3. Instruments used in the Investigation

In order to obtain comprehensive data, a number of instruments were used in the investigation as given below:

(a) *Wood-Anderson Seismograph*—This instrument was used in kind cooperation with the Meteorological Department, Government of India, Poona. It is a mechanical instrument with a static magnification nearly 1,000.

Usual expression for magnification can be shown to be

$$M = \frac{V_s}{\sqrt{\left[1 + \left(\frac{T_s}{T_v}\right)^2\right]^2 + 4\left(\frac{T_s}{T_v}\right)^2(h^2-1)}} \quad \dots (1)$$

Here M —is effective or dynamic magnification;

V_s —Static Magnification;

T_s —Period of the earth wave;

T_v —Period of the seismograph;

h —Damping constant.

From a review of the registered records, it was found that T_s is of the order of 0.1 sec. and T_v nearly 1 sec. Hence $\frac{T_s}{T_v} \approx 0.1$.

Thus neglecting higher powers of $\frac{T_s}{T_v}$, equation (1) gives $M = V_s$ for any finite value of h .

Throughout the frequency range obtained in the investigation, the dynamic magnification 'M' of the instrument was, therefore, equal to 'V_s', i.e., 1,000. The optical distance used was equal to one meter.

As the natural period of the instrument (1 sec. nearly) was much higher than that of the earth wave (0.1 sec.) the seismograph served the purpose of an ideal displacement-meter recording proportional displacements of the earth movements.

(b) *Geophones*—Rugged geophones (electromagnetic instruments) usually used in geophysical investigations, were adopted for use in this investigation in horizontal as well as vertical planes. These geophones were coupled with electronic amplifiers, and recording Galvanometer, both having flat response in the encountered frequency range and thus the electromagnetic fluctuations in the geophones are transmitted to the recorder without much distortion.

It has, however, to be noted that the geophone records represent proportional velocities as against proportional displacements by Wood-Anderson Seismograph.

(c) *Cambridge Vibrograph*—This is also a mechanical instrument with a very low magnification (nearly 5). It was, therefore, only possible to use it on the dome of Siphon No. 7, where the intensity of vibration was sufficient.

(d) *Reed Vibrometer*—This is also a mechanical instrument working on the principle of resonance. It, having a low magnification, could only be used to measure the intense vibrations of the dome of the Siphon No. 7. It has indicated some prominent frequencies.

4. Analysis of Seismograms

A close study of all the seismograms shows the existence of an over-all low frequency envelop (wave-group), over and above a high frequency spectrum (individual wave), existing throughout. The existence of these wave-groups in all the seismograms, recorded during the investigation, shows that they are probably significant in the dynamical behaviour of the structure, under examination.

In analysing the various records, it was thought best to take into consideration only the most prevalent groups occurring in the seismograms.

It is found that the average period in a group and maximum amplitude were significant and represented the dynamical behaviour of the vibrating system.

5. Vibration Study of the Dam—All the Stations on the Dam with Particular Reference to Station '4'

Figure 6 shows the stations of observation on the dam. Station No. 1 was on the earth portion whereas Nos. 2, 3, 4, 5 and 6 were on the masonry portion.

Station 3 was on the masonry portion above the undersluices, 4 was opposite siphon No. 10 which was expected to function and 5 was opposite siphon No. 7 which primed on 17th August, 1951. Accordingly, seismograph room was built at station 4 and so the observations with the Wood-Anderson Seismograph could only be taken at station 4. Observation with electro-magnetic geophones have been taken at all the stations. Comprehensive data is, therefore, available only at station 4 and value of acceleration, velocity and displacement of the dam have all been determined at this station. By comparing the observations with those at station 4, attempt has been made to get an idea of acceleration, etc., at station 5 which was nearest to the source of disturbance, i.e., siphon No. 7.

Figure 5 gives the values of periods and relative velocity from records at various stations with vertical and horizontal geophones (perpendicular to the axis of dam), when the siphons were discharging water as weirs only. Figure 6 gives average values of periods and relative velocity (as obtained from Figure 5).

Considering vertical vibrations it is seen that the period of vibration was practically uniform, on the masonry portion of the dam, with a value of nearly 0.04 sec.

It will also be seen that the value of the period is distinctly different on the earth and on the masonry portion of the dam. This difference of periods of vibration on the masonry (0.04 sec.) and the earth portion (0.05 sec.) indicates the influence of respective structures on the vibration characteristics. Considering the vertical period of the respective parts of the dam, a tentative idea of the elastic constants of the materials of construction of these parts can be obtained as follows:

The period 'T' of the fundamental vibration can be written as

$$T = \frac{4h_1}{V_1}$$

where h_1 = height of the dam and V_1 = velocity of longitudinal waves = $\sqrt{\frac{E}{\rho}}$ where E = elasticity and ρ is the density of the material. Here $\rho = 2.8$ for rock, $h_1 = 90$ ft. for masonry portion.

$$T = \sqrt{\frac{4h_1}{\rho E}} \quad \therefore 0.04 = \sqrt{\frac{4 \times 90}{2.8 E}}$$

which gives $E = 2.1 \times 10^{11}$ c.g.s. units—a usual value for masonry.

Similarly for earth portion, $T = 0.05$ sec., $\rho = 2.4$ and $h_1 = 60$ ft. and the formula gives

$$0.05 = \sqrt{\frac{4 \times 60}{2.4 E}}$$

which gives $E = 4.97 \times 10^{10}$ c.g.s. units for earth embankments.

The study of horizontal vibration (perpendicular to the axis of dam) from Figures 5 and 6 also shows a behaviour similar to that of vertical vibrations. Here also the periods show a sudden change in passing from the masonry to the earth dam portion. Whereas for vertical movement relative velocity and displacement decrease, the period increases in passing from the masonry to the earth portion; and for horizontal movement all these elements decrease. The difference in the behaviour is due to difference in elasticity in the two portions of the dam.

FIGS. 5 & 6.

It is also seen that horizontal vibration consists of two spectra, one of higher period, 0.15 sec. and the other of lower period 0.04 sec.; both these periods are present on the masonry dam, earth embankments and foundations.

It is, however, observed that whereas the periods corresponding to masonry dam, in both the components remain fairly constant, the displacements and associated relative velocities increase towards the right end of the dam (looking downstream). A study of the plan of the dam (Figure 6) reveals that towards the left end (looking downstream) the masonry dam is well embedded in earth embankments on both faces, while towards the right end it touches the earth embankment on the downstream face only and this asymmetry in the boundary conditions is perhaps responsible for the increase in relative velocity and displacements towards the right end of the dam.

FIG. 6.

Figure 7 shows the analysis of records, with the Wood-Anderson Seismograph, at station 4 when one undersluice was slowly opened to maximum discharge value (10,000 cusecs). It will be seen from the figure that the period, as well as, amplitude remain at a comparatively low value in the initial stages and rise suddenly up to a high value 0.13 sec., which remains constant.

FIG. 7.

Figure 7A shows the analysis of records at station 4 (with Wood-Anderson Seismograph) for 10 minutes, under a constant discharge of 10,000 cusecs. It can be seen that though the displacement shows some scatter, the periods of vibration remain fairly constant.

FIG. 7A.

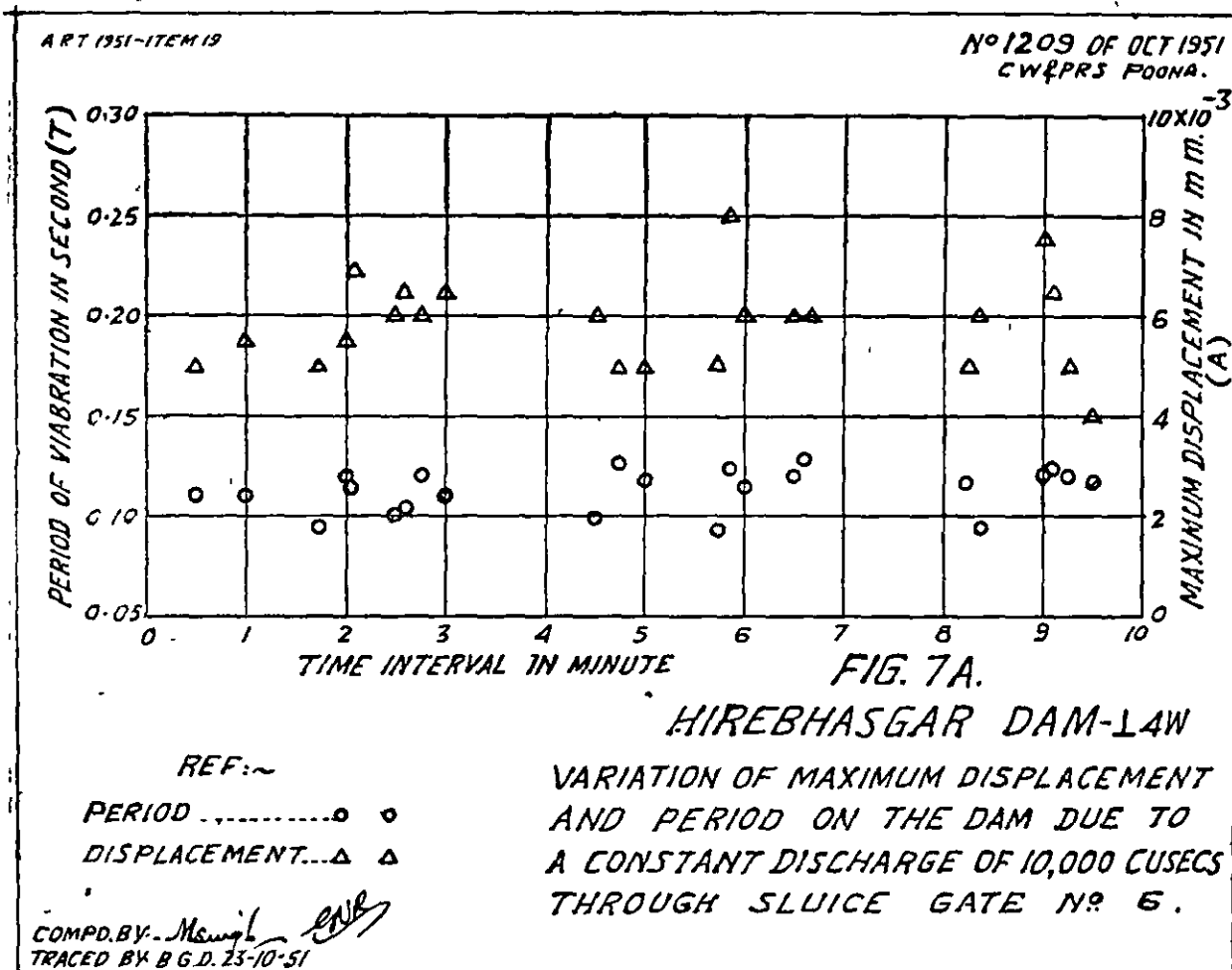


Fig. 8. Figure 8 shows the variation of period with displacement (Wood-Anderson record at station 4) when the sluice No. 6 was slowly opened to maximum discharge value. It is seen that in the initial stages the period increases with displacement attaining a constant value afterwards.

Fig. 7. Maximum velocity and acceleration at station 4, as calculated from the values of periods and corresponding maximum displacements, for various discharges (from Figure 7) are shown in Figure 9. It will be seen that acceleration as well as velocity attain a constant value, that for acceleration being nearly 0.001 g.

Fig. 9. Variation of period of vibration at station 4 (Wood-Anderson record), with various undersluice gates open, is shown in Figure 10. It is seen from a close scrutiny of the figure that though the scatter increases with increase in the number of gates open, the mean period of vibration is not much affected.

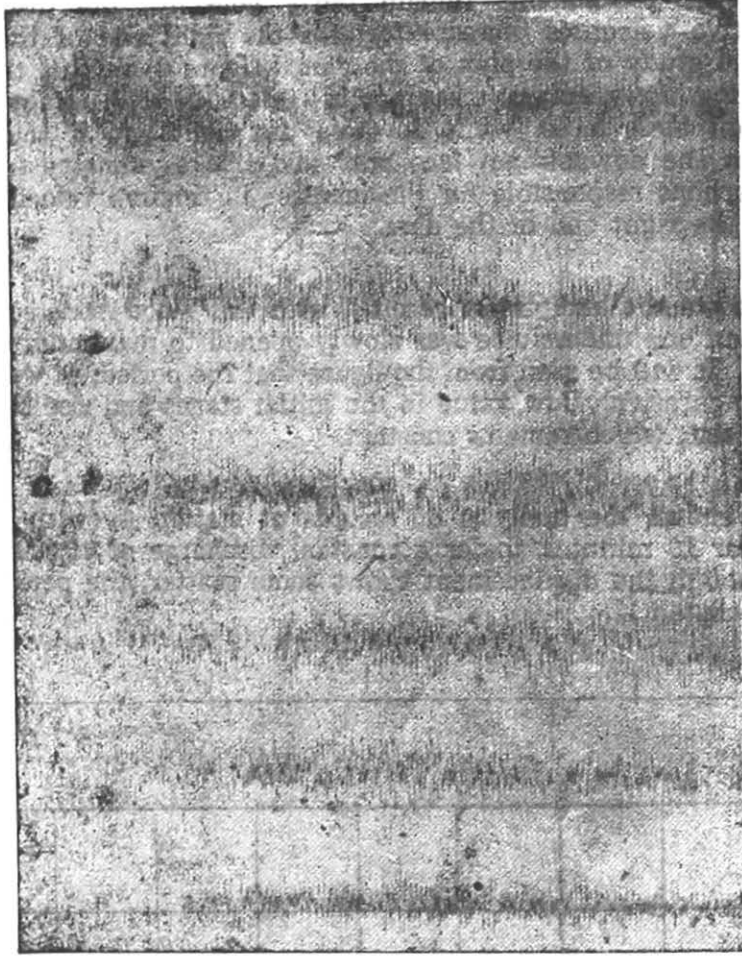
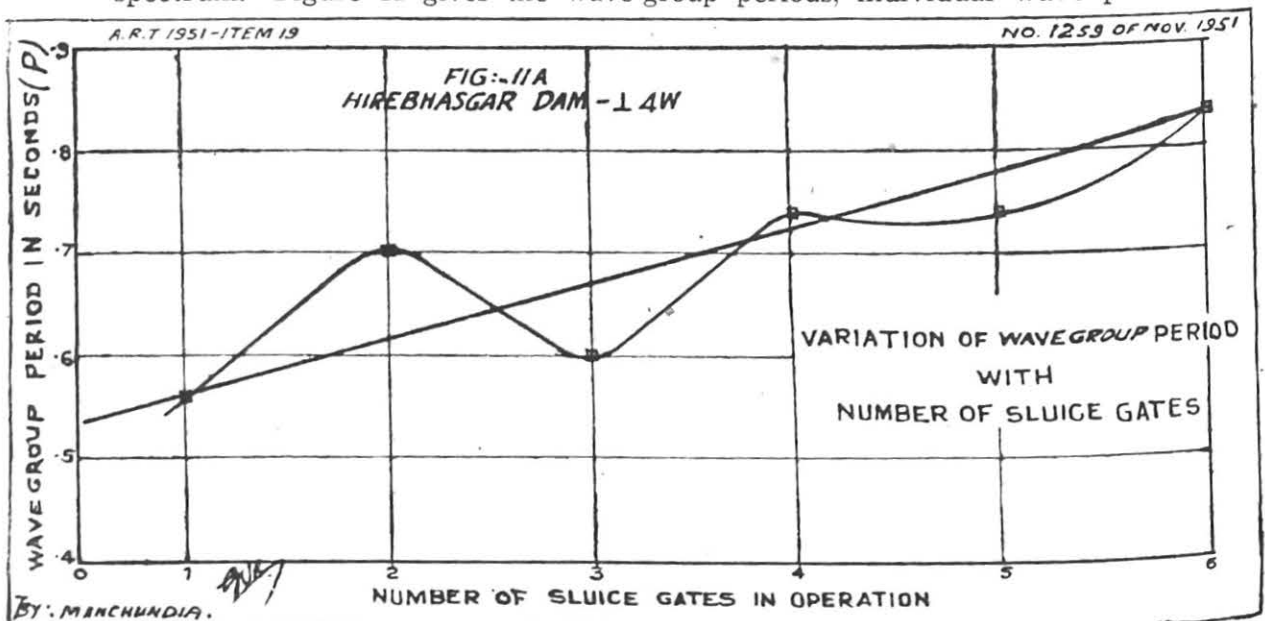


Fig. 11B. Figure 11B shows a portion of seismogram (Wood-Anderson record at station 4) with various undersluices open. It depicts existence of wave-groups, thus showing the presence of low frequency spectrum over and above the high frequency spectrum. Figure 11 gives the wave-group periods, individual wave periods and



RECORD WITH WOOD-ANDERSON SEISMOGRAPH ON DAM
STATION (4) ⊥ TO AXIS OF DAM.

№ 1208 OF OCT. 1951
C.I.V. & P.C. POONA.

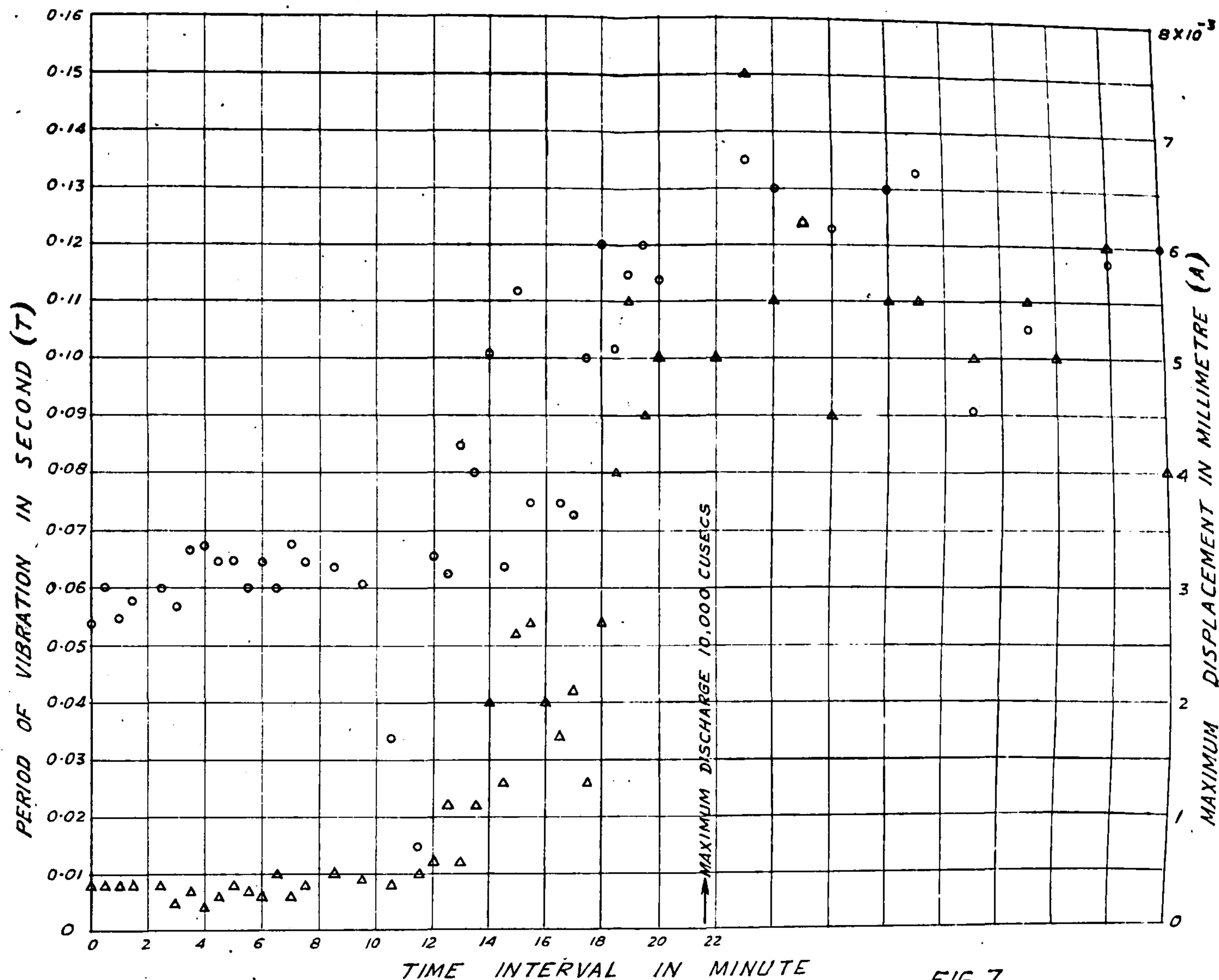


FIG. 7

DISCHARGE INCREASING →

PERIOD ○ ○ ○ ○
DISPLACEMENT △ △ △ △

HIREBHASGAR DAM-14W
VARIATION OF DISPLACEMENT AND PERIOD
OF
VIBRATION ON THE TOP OF THE DAM
AS ONE DEEP SEATED GATE (№ 6)
WAS SLOWLY OPENED. MAXIMUM DISCHARGE
WAS 10,000 CUSECS THROUGH THE GATE.

RECORD WITH WOOD-ANDERSON SEISMOGRAPH ON DAM
STATION ④ 1 TO AXIS OF DAM.

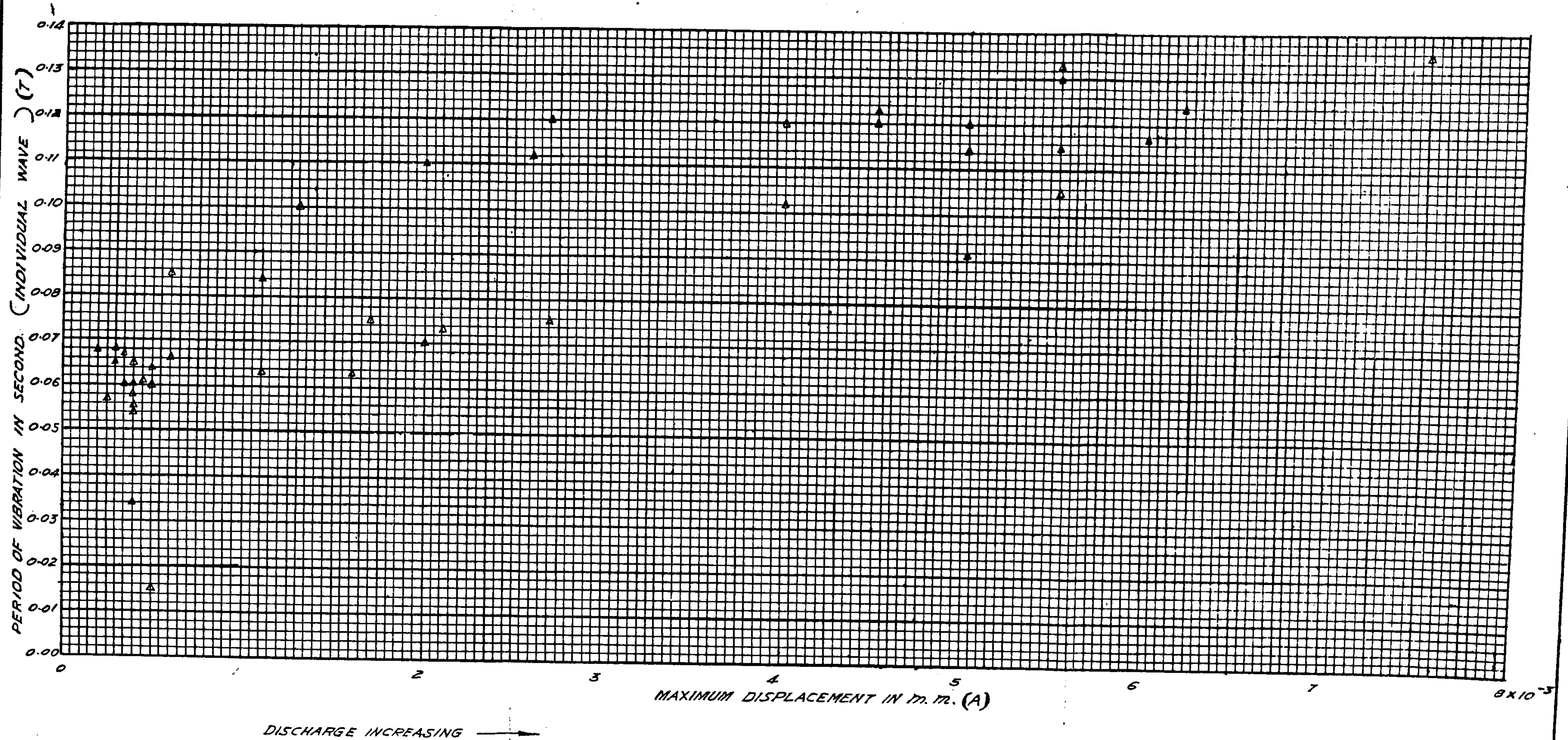
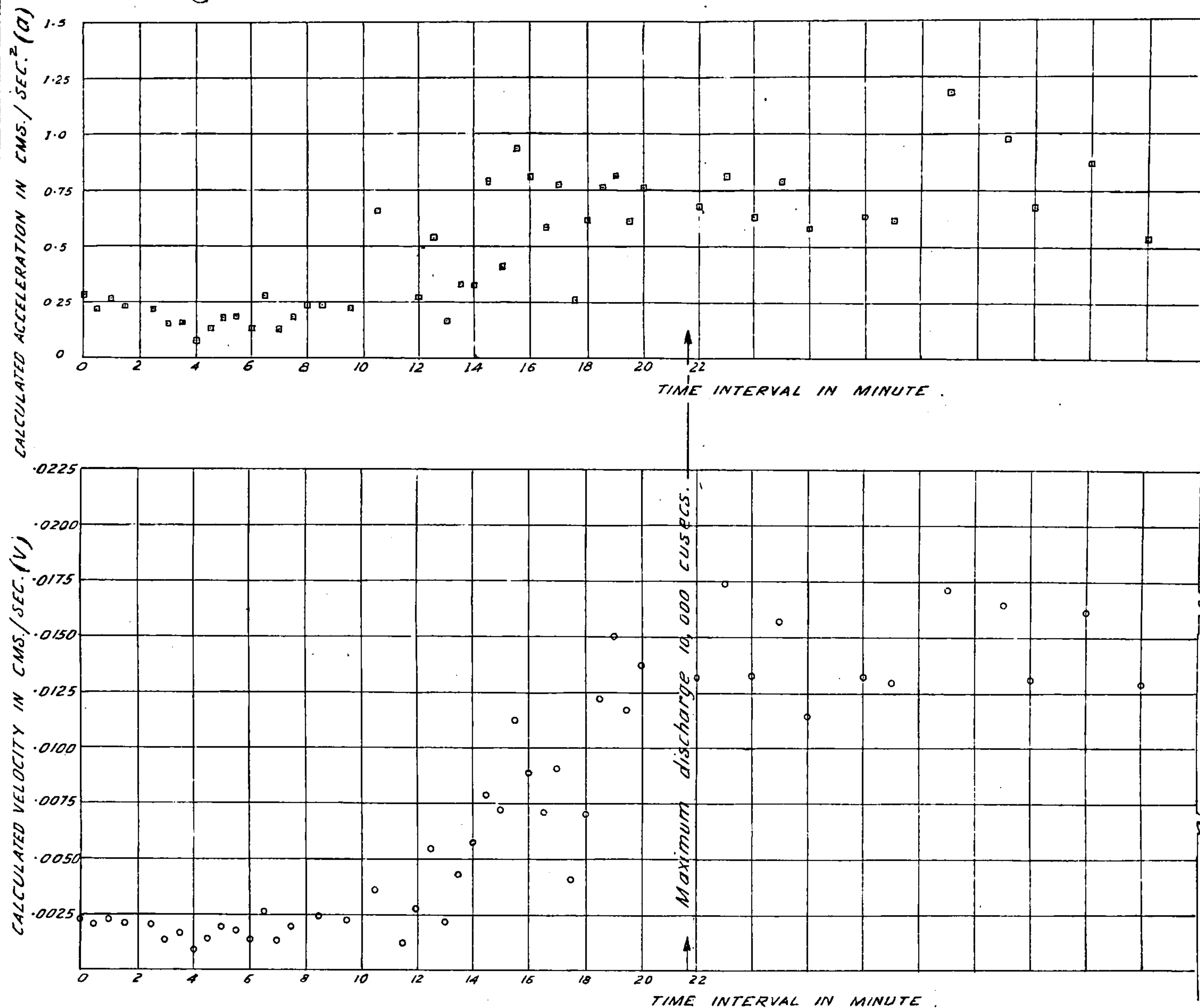


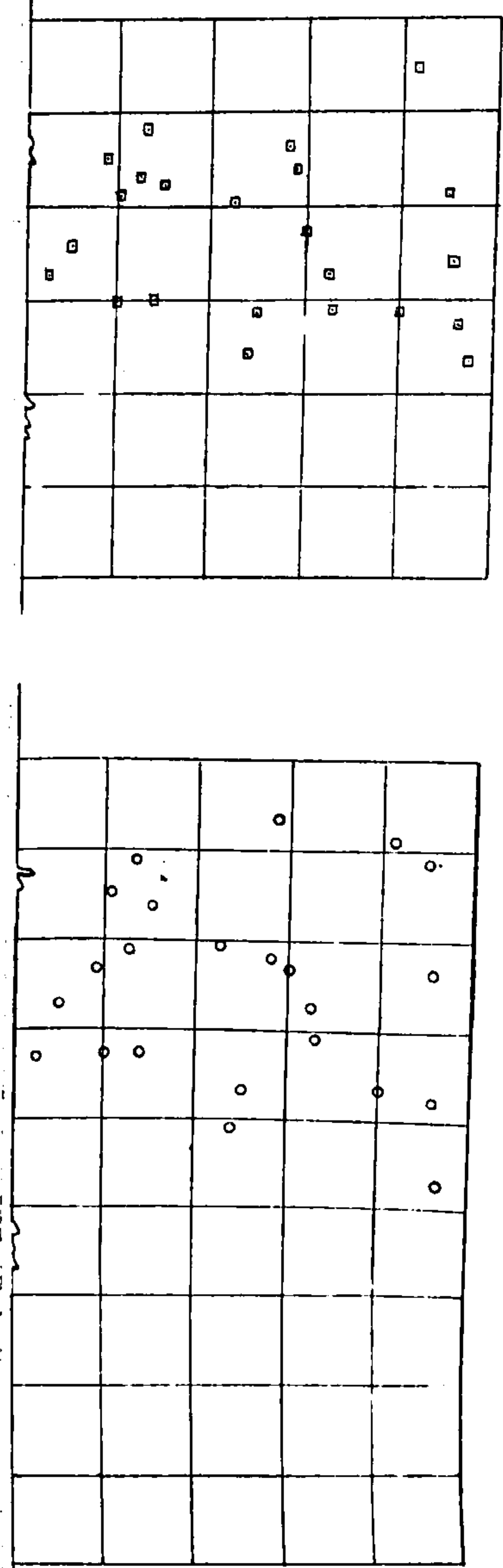
FIG. 8
 HIREBHASGAR DAM-1.4W.
 VARIATION OF PERIOD WITH MAXIMUM DISPLACEMENT
 ON THE DAM AS UNDERSLUICE NO. 6
 WAS SLOWLY OPENED TO MAXIMUM
 (MAXIMUM DISCHARGE 10,000 CS.)

COMP. BY - *Munsiff*
 TRACED BY - PATKIE G. R.
 26-10-1951.

RECORD WITH WOOD-ANDERSON SEISMOGRAPH ON DAM STATION (A) \perp TO AXIS OF DAM.



An interval of the order of 30 minutes.



DISCHARGE INCREASING. →

FIG. 9

HIREBHASGAR DAM-14W

VARIATION OF MAXIMUM ACCELERATION AND VELOCITY (ASSOCIATED WITH EACH WAVE) AT THE TOP OF THE DAM WHEN SLUICE GATE NO. 6 WAS SLOWLY RAISED UPTO MAXIMUM.

COMPONENT OF VIBRATION \perp TO THE AXIS OF THE DAM.

COMP. BY:
TRACED BY: R. T. THAKOR.
12th NOV. 1951.

gvr

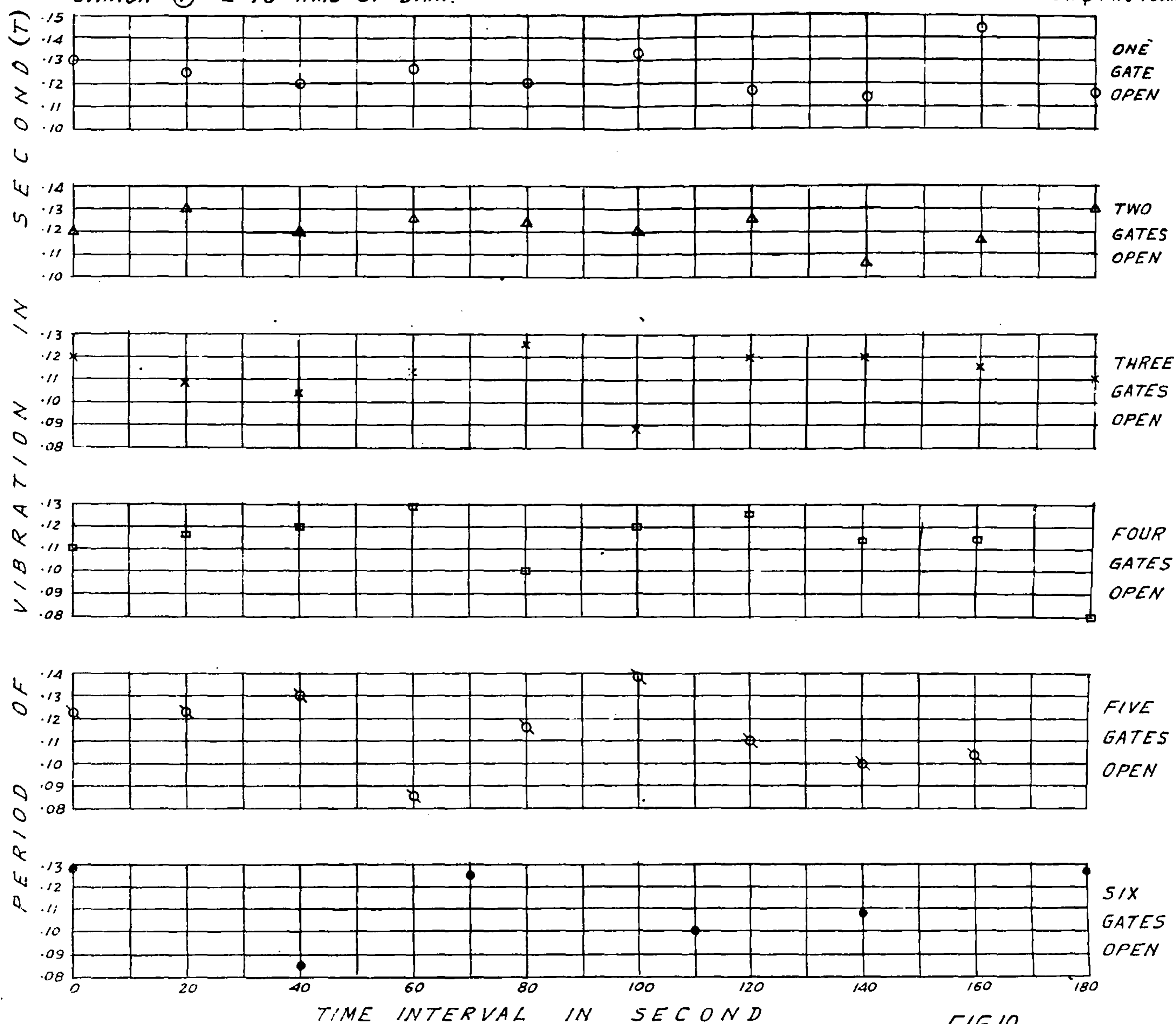


FIG. 10

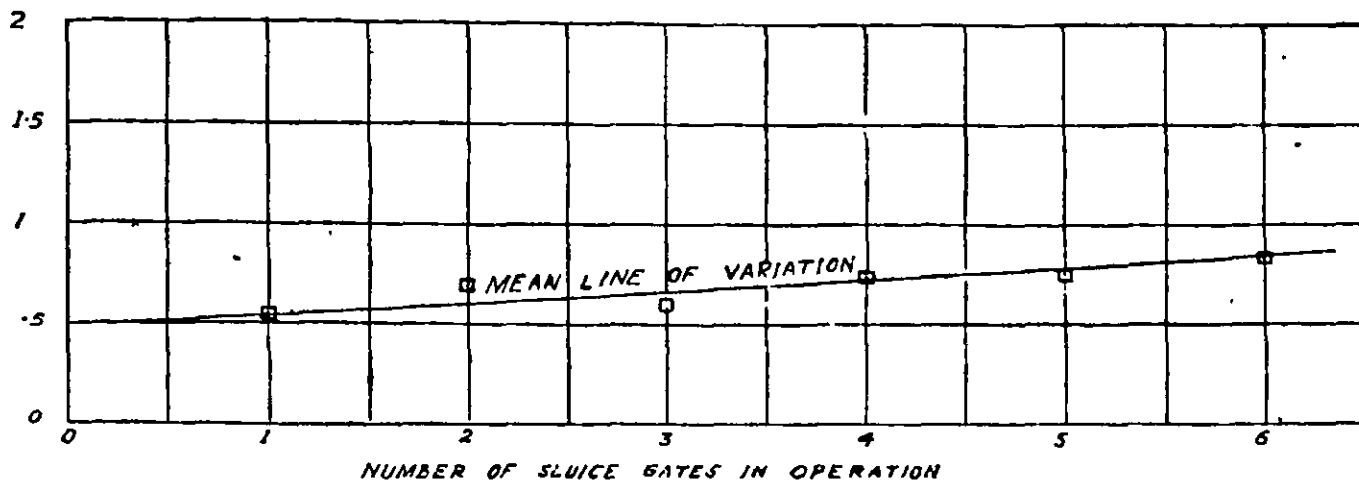
DISCHARGE THROUGH EACH GATE = 10,000 CUSECS.

HIREBHASGAR DAM-14W
 VARIATION OF PERIOD OF
 VIBRATION WITH TIME
 FOR
 VARIOUS SLUICE GATES OPEN

FIG. 11
HIREBHASGAR DAM-14W

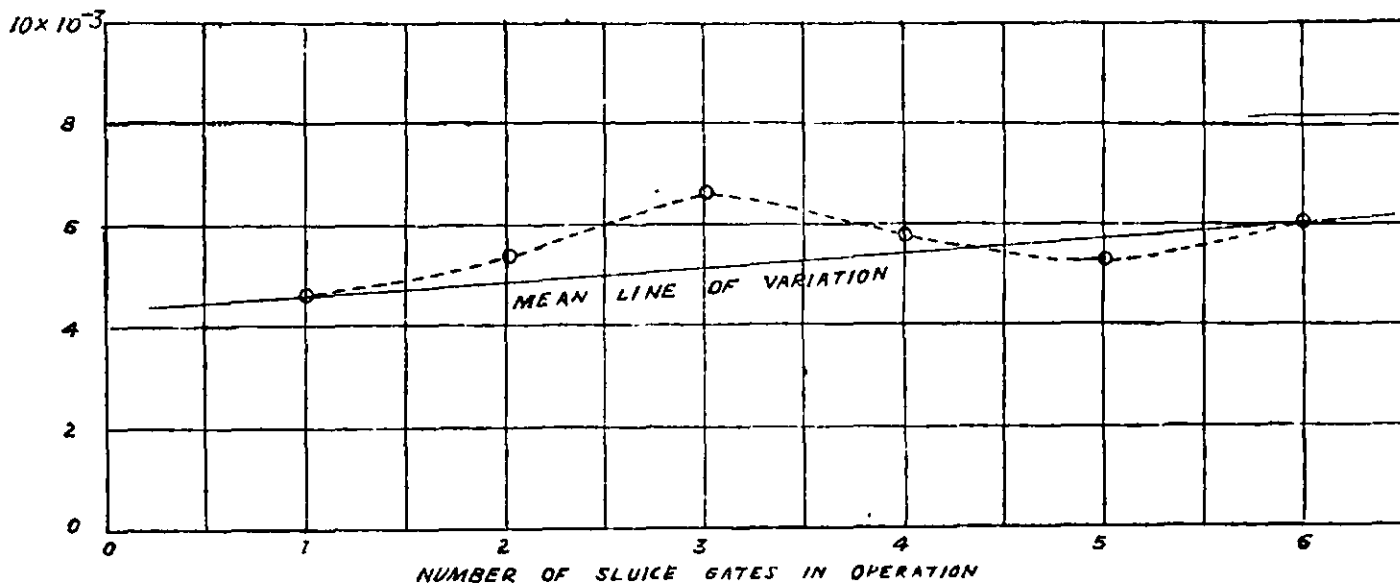
VARIATION OF AVERAGE PERIOD OF WAVE GROUP WITH
NUMBER OF SLUICE GATE

PERIOD OF WAVE GROUP IN SECOND (P)



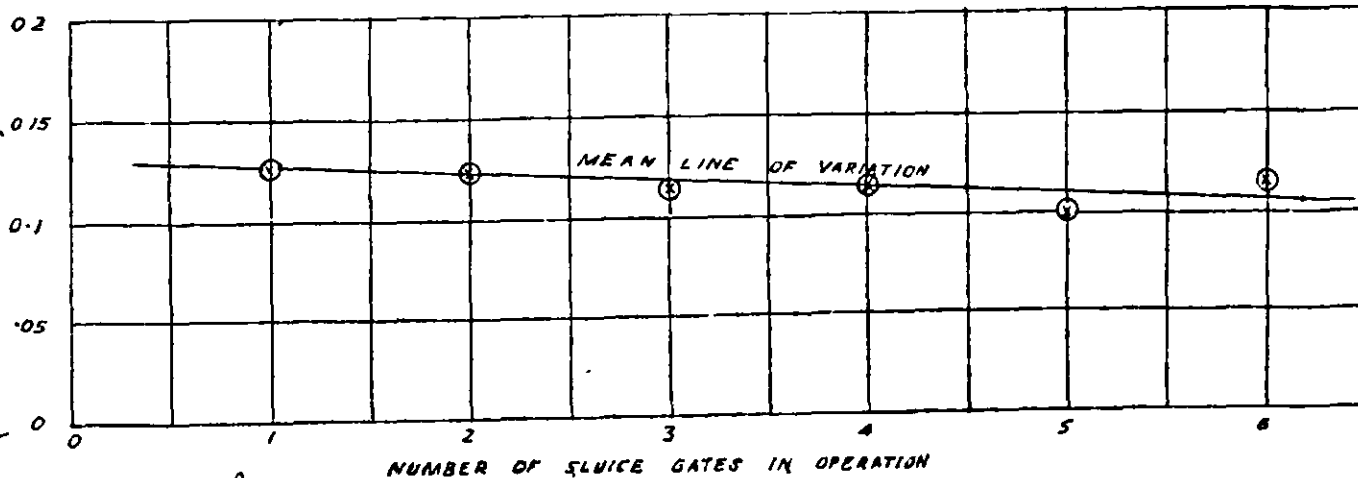
VARIATION OF AVERAGE MAXIMUM DISPLACEMENT
WITH NUMBER OF SLUICE GATE

MAXIMUM DISPLACEMENT IN MILLIMETER (A)



VARIATION OF AVERAGE PERIOD OF VIBRATION WITH NUMBER
OF SLUICE GATE

PERIOD OF VIBRATION IN SECOND (T)
(INDIVIDUAL WAVE)



displacements from the record. It is seen that the wave-group period increases with the increase in the number of open gates. Mean value of the maximum displacement also shows an increase as can be seen from the figure. Individual wave periods, though they appear to show a slight decrease with the increase in the number of open gates, can be taken as fairly constant, considering the scatter in the individual values as can be seen from Figure 10.

Fig. 10.

Analysis of records with horizontal (perpendicular to the axis of the dam) and vertical geophones at stations 3 and 4 is given in Figure 12, which gives maximum relative velocity and period of vibration for one gate open and six gates open. It is seen that the vertical vibrations represent a more monochromatic behaviour than the horizontal ones, in respect of periods of vibration as well as associated relative velocities. There is, however, a slight increase in the vertical velocity with six undersluice gates open. The period and relative velocity of horizontal vibrations show some scatter which increases with six undersluices open. Records of the vertical vibrations are thus of more sustained nature than those of the horizontal ones; and this is as one would expect considering the boundary conditions of the two cases.

Fig. 12.

The variation of period of vibration and displacement, at station 4 (Wood-Anderson record) during the primed state of siphon No 7 are shown in Figure 13 and these during its depriming state in Figure 14

Fig. 13.
Fig. 14.

It is seen from Figure 13 that both individual wave period and wave-group period are, more or less, similar in character to that observed with vibrations in case of undersluices open; the displacement, so the maximum acceleration, however, is five times more than that in the latter case (i.e., with undersluices open).

From Figure 14, when the siphon began to deprime, it is seen that though the periods of individual waves are similar to those in the primed state, the displacement in this case is about half that in the primed state (see Figure 13) and therefore, the related acceleration will also be half that in the primed state.

Fig. 14.

Fig. 13.

A summary of wave-group periods, periods of individual wave, associated displacements and calculated maximum acceleration, due to various forces of excitation, at station 4 (Wood-Anderson record) has been given in Figure 15. The maximum value of acceleration is found to be about 95 cms/sec². (i.e., 0.01 g) during the primed state of the siphon No. 7.

Fig. 15.

It may be noted that station No. 4 is at a distance of about 150 ft. from station 5, which is opposite the siphon No. 7 (which primed and as such was the source of excitation) and where the acceleration would be much higher. The behaviour of various elements, like periods of individual wave, periods of wave-groups, associated displacements and accelerations, show that, whereas the periods remain pretty constant, the displacement and so acceleration diminish as the siphon begins to deprime. This sudden change on passing from the primed to the depriming state of the siphon is clearly depicted in Figure 15

Fig. 15.

Figure 16 shows that the variation of acceleration with displacement is, more or less, linear, as one would expect from the relation $a = \frac{4\pi^2 A}{T^2}$ for simple harmonic motion of amplitude of displacement 'A', period 'T' and amplitude of acceleration 'a'.

Fig. 16.

Figure 17 (Wood-Anderson record at station 4) gives displacements and periods of individual waves within a wave-group. These would give an idea of the variation of displacements or accelerations during the period occupied by a wave-group. Maximum accelerations, which are of interest have already been given in Figure 15.

Fig. 17.

Fig. 15.

Station 3 on the Dam—This station on the dam, is just above the undersluices and so when the undersluices function, this station would be affected most. The records on this station were taken with the help of geophones. From these geophone records the value of maximum acceleration calculated at station 3 with all the undersluice gates open is only of the order of 0.003 g.

Station 5 on the Dam—This station is of special importance as it was just opposite siphon No. 7 and which primed and so was nearest to the source of disturbance. As the seismograph room had been constructed at station 4, for the reason given already, the records with the Wood-Anderson Seismograph were not possible on this station. The study was, therefore, made with the help of geophones alone.

Figs 18 & 19. The analysis of records on this station is given in Figure 18 and Figure 19 (geophone records during primed stage only).

Fig. 18. It is seen from Figure 18 that there exists a spectrum of low periods of 0.05 sec with superimposed wave-groups of periods varying from 1 to 10 secs. The wave-group spectrum is highly non-uniform and it is not possible to associate it with any natural vibration of the structural elements and further they would only produce a very much lower acceleration as compared with that associated with low period individual waves.

Fig. 18. Figure 18 also shows the relative velocity associated with the individual waves (period 0.05 sec.). It is seen that relative velocity is also uniform like the associated periods.

Maximum acceleration at this station, calculated from these geophone records is of the order of 0.02 g, as compared with 0.01 g obtained at station 4, which was 150 ft. away.

Fig. 19. Figure 19 depicts the period and amplitude of individual waves in some representative wave-groups. It is seen that the periods maintain a nearly constant value in both the components. There also exist some sub-wave-groups inside the wave-groups. The sub-wave-groups were not present in the case of Figure 17. This peculiarity may be associated with the high frequency spectrum, met with in the geophone records, with which large accelerations are associated and so these sub-wave-groups may be due to large irregular impressed forces.

6. Vibration Study on the Foundation

The records were taken in all the components, namely, vertical and horizontal (parallel and perpendicular to the axis of the dam).

Fig. 2. Figure 2 represents periods of vibration and associated maximum displacements graphically (from a Wood-Anderson record). It shows two frequencies which occur alternately on the seismogram. One has a mean period of 0.06 sec. and the other of 0.13 sec. As can be seen the periods are, more or less uniform. Associated displacements are also mostly uniform and are of a similar order in both the spectra, those for the low frequency being slightly of higher magnitude.

Fig. 3. Figure 3 gives analysis of records with geophones in two mutually perpendicular horizontal directions. It gives a low frequency similar to that of the Wood-Anderson record (Figure 2) but of a slightly higher period 0.15 sec in both the directions. In this record, however, there is a very uniform high frequency spectrum, present in both the directions, with mean period of 0.025 sec., which is much lower than the high frequency spectrum in Wood-Anderson record in Figure 2.

Fig. 2.

The figure also shows the associated relative velocity for both the spectra in the two directions. It is seen that the relative velocity associated with the higher frequency is nearly the same in two directions, viz., perpendicular and parallel to the axis of dam. Similar remarks apply for low frequency spectrum if the percentage scatter of observations is taken into account.

Wave-groups, which envelope high frequency spectra, occur alternately with those enveloping low frequency spectra and both these wave-groups have a period of 1.4 sec. as can be seen from Figure 2. This alternate recurrence would be due to predominance of one mode of vibration over the other at a particular instant. As the two modes of vibration occur throughout the record, the generating agency must have a cycle of 1.4 sec., during which there are favourable moments for the two modes to be excited

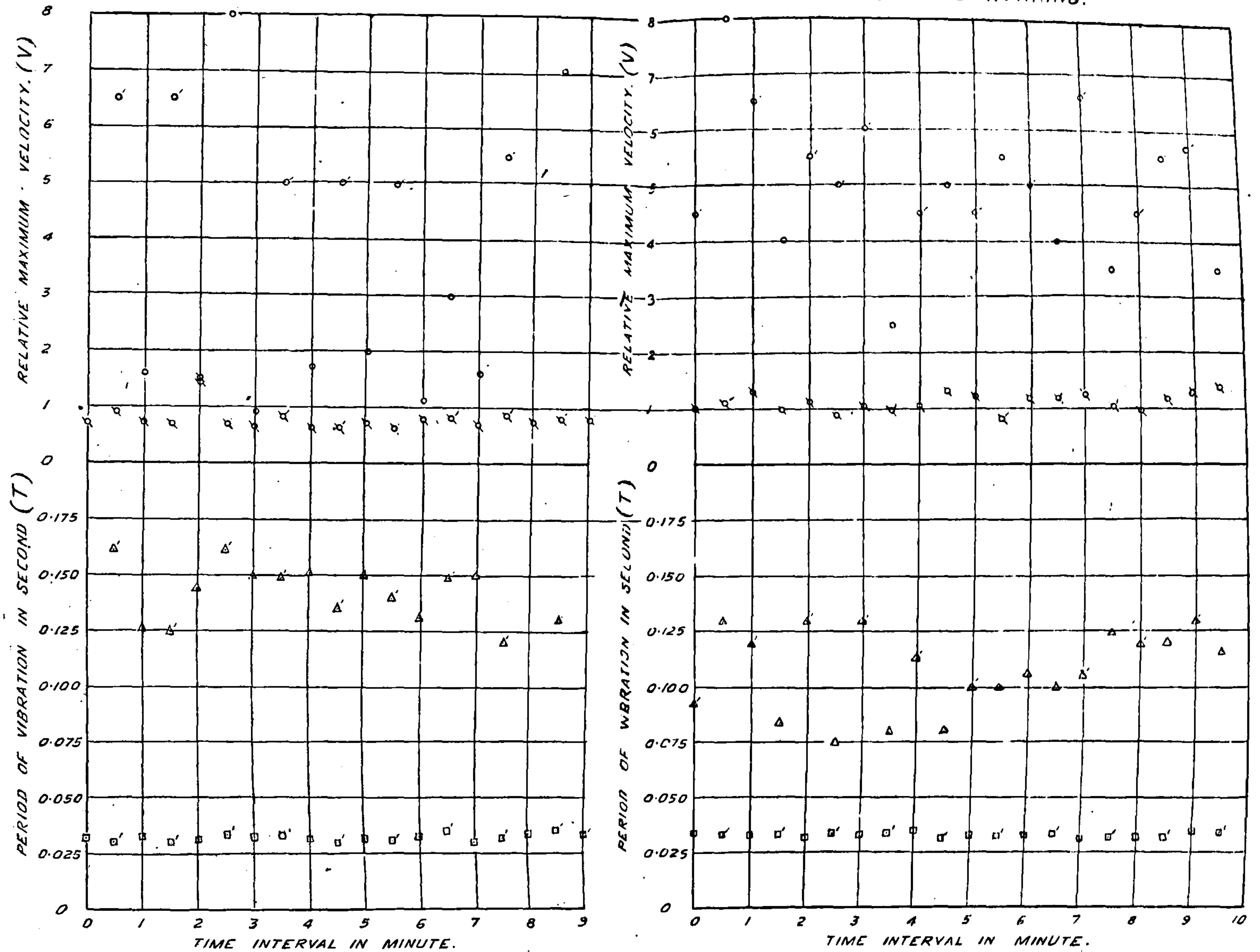
Fig. 2.

RECORD ON DAM STATIONS (3) AND (4)
WITH GEOPHONES.

NO. 1326 OF NOV. 51.
C.W. AND P.R. STATION POONA.

ONLY ONE GATE WORKING.

SIX GATES WORKING.



REFERENCES:-

Δ, Δ' REPRESENT PERIOD OF HORIZONTAL VIBRATIONS (L) AT POSITIONS (4), (3) RESPECTIVELY AND \square, \square' CORRESPONDING VALUES FOR VERTICAL VIBRATIONS VIDE FIG. 9.....

\circ, \circ' REPRESENT HORIZONTAL RELATIVE MAXIMUM VELOCITY (L) AT POSITIONS (4), (3) RESPECTIVELY AND \circ, \circ' CORRESPONDING VALUES FOR VERTICAL VIBRATIONS.

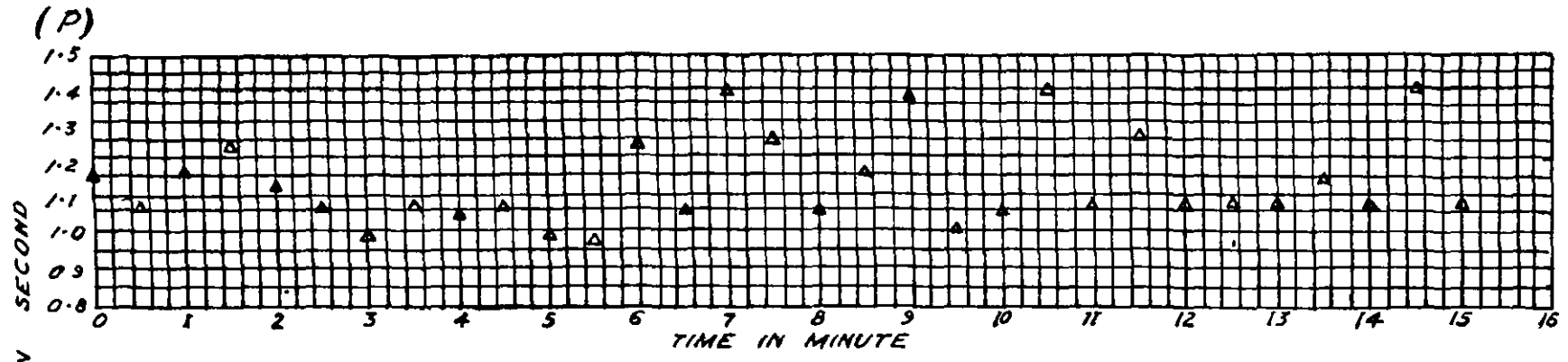
Comp. by:
T. BY: PARKHI.
28-11-51

PVR

FIG. 12

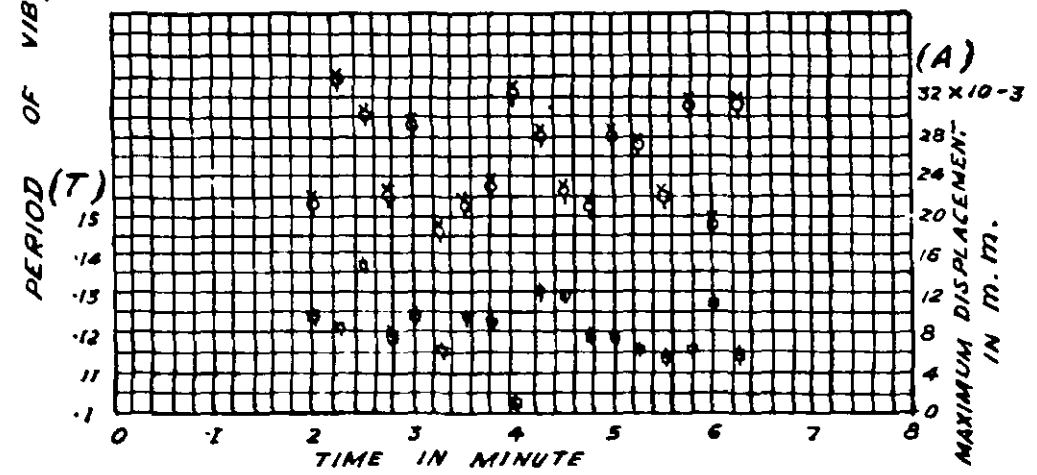
HIREBHASGAR DAM-4,3 G
COMPARISON OF VARIATION OF PERIOD AND RELATIVE
MAXIMUM VELOCITY AT (4) (3) WHEN ONE AND SIX
UNDER SLUICES WERE RUNNING.

RECORD WITH WOOD-ANDERSON SEISMOGRAPH ON DAM
STATION (4) ⊥ TO AXIS OF DAM.



VARIATION OF PERIOD OF WAVE GROUP WITH TIME

Ref.
Δ... Period of wave of group
at interval of 1/2 minute.

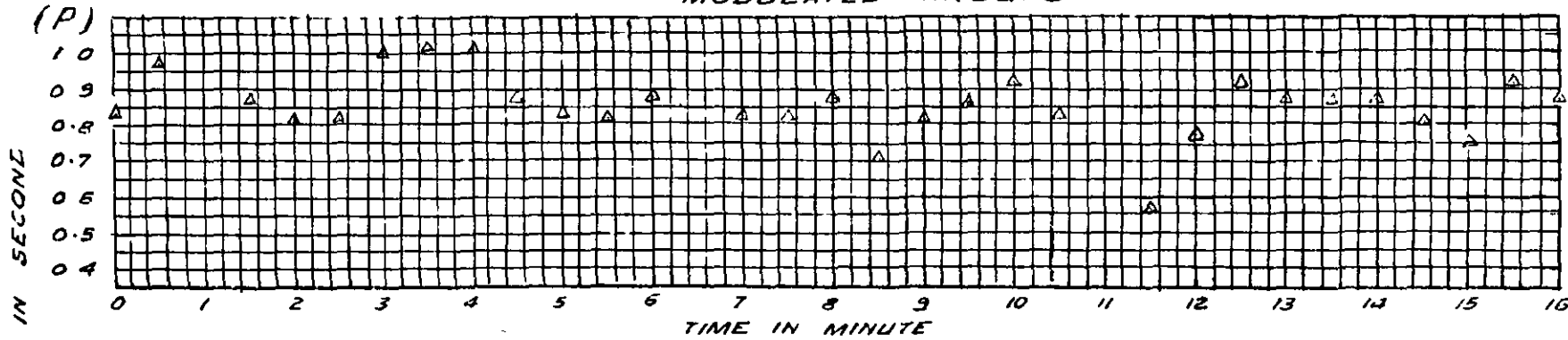


VARIATION OF AVERAGE PERIOD AND MAXIMUM DISPLACEMENT
OF INDIVIDUAL WAVE WITH TIME
TAKEN AT INTERVAL OF 15 SECONDS.

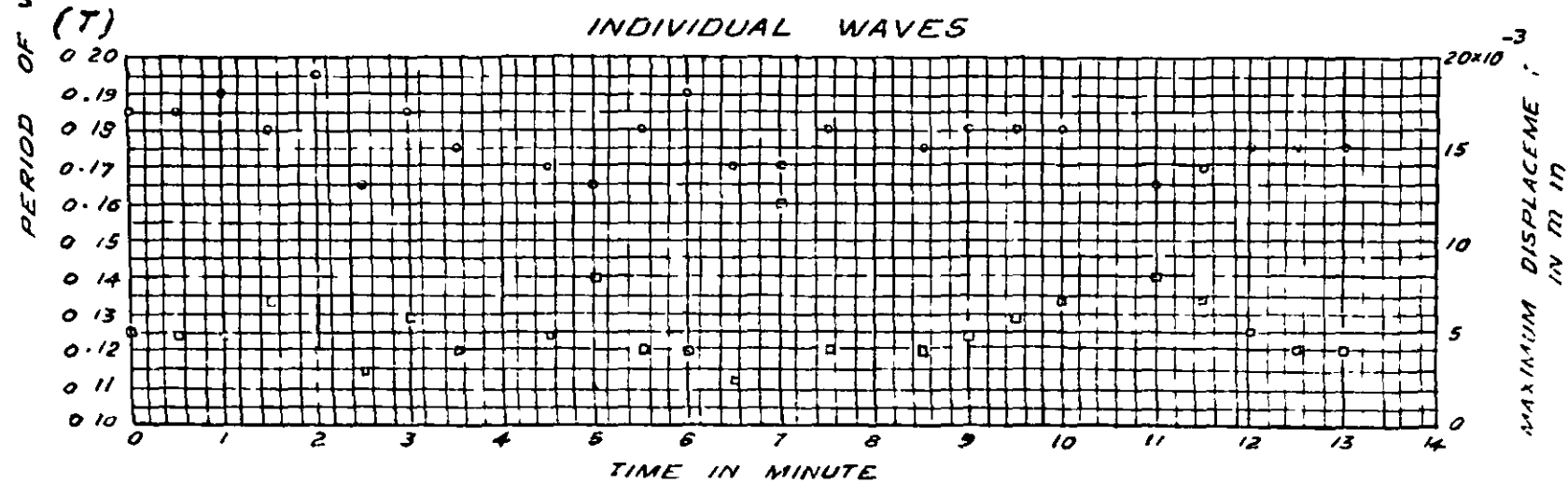
Ref
□..... Average period of individual
wave.
○..... Maximum displacement associated
with individual wave.

FIG. 13
HIREBHASGAR DAM-14W
WITH SIPHON NO 7 IN PRIMED STAGE
COMPONENT OF VIBRATIONS ⊥ TO THE
AXIS OF DAM

MODULATED GROUPS



INDIVIDUAL WAVES



Ref
 ▲ & □ Period of modulated group and individual wave respectively
 ● ... Maximum displacement.

COMP. BY. *M. J. P.*
 TRACED BY P. K. H. P. D.
 21 NOV 1951

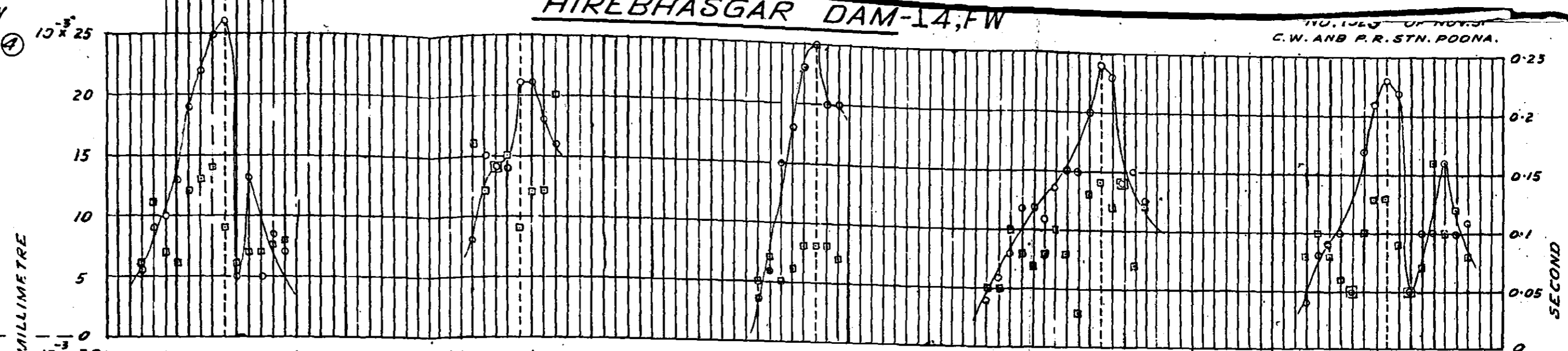
FIG. 14

HIREBHASGAR DAM-L4W
 VARIATION OF PERIOD OF VIBRATION
 AND MAXIMUM DISPLACEMENT ON THE DAM
 DURING DEPRIMING OF SIPHON NO. 7
 COMPONENT OF VIBRATIONS ⊥ TO
 THE AXIS OF DAM.

1 TO AXIS OF DAM.

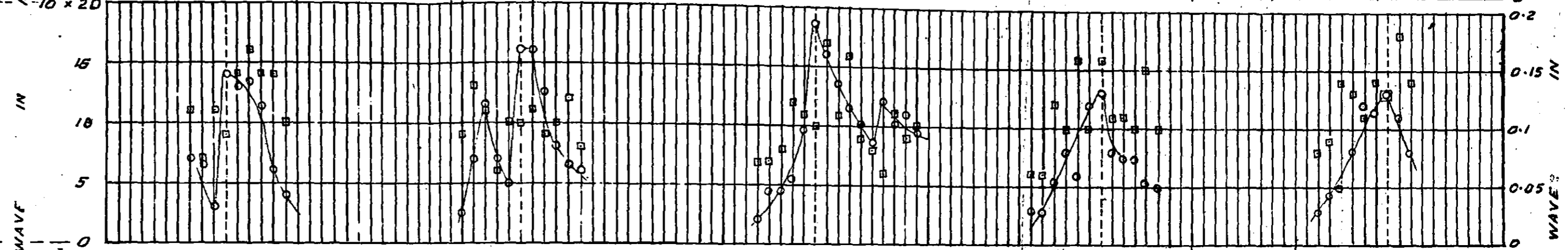
CONDITION OF EXPT.:-

SIPHON NO. 7 IN PRIMED STAGE AND SIX SLUICE GATES OPEN.



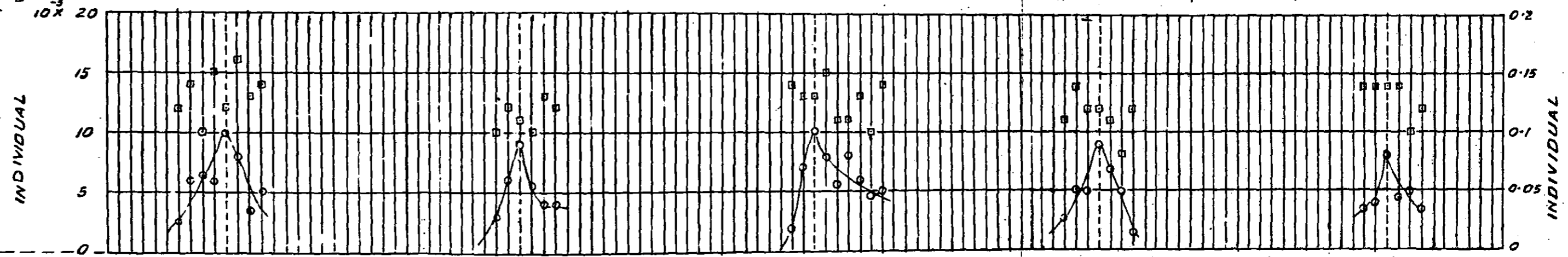
CONDITION OF EXPT.:-

SIPHON NO. 7 IN DE-PRIMING STAGE AND SIX SLUICE GATES OPEN.



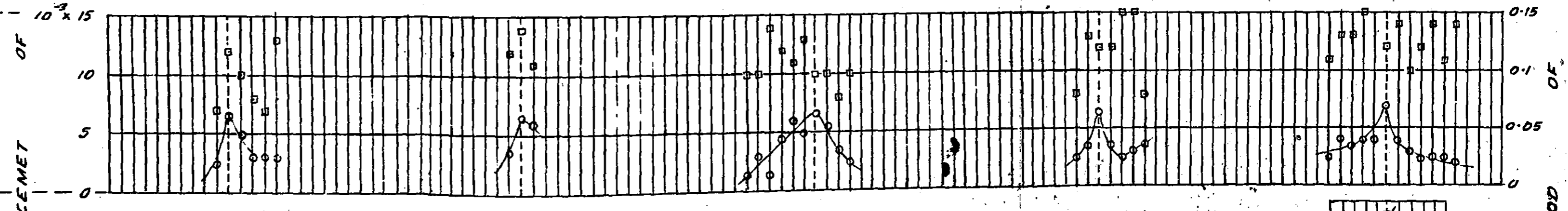
CONDITION OF EXPT.:-

SIX UNDER-SLUICES WERE OPEN.



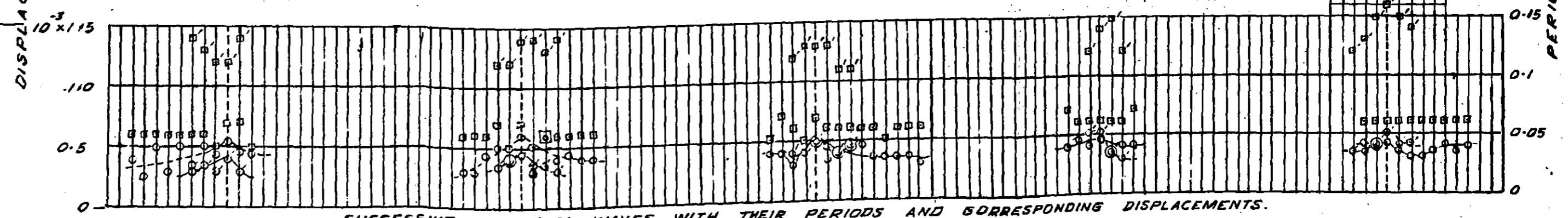
CONDITION OF EXPT.:-

ONE UNDER-SLUICE (NO. 6) WAS OPEN.



CONDITION OF EXPT.:-

RECORD ON THE FOUNDATION WHEN SIPHONS WERE RUNNING AS PIPES. (DISCHARGE 2177 CS.)



SUCCESSIVE INDIVIDUAL WAVES WITH THEIR PERIODS AND CORRESPONDING DISPLACEMENTS. (EACH SMALL DIVISION REPRESENTS ONE WAVE)

REFEREENCES:-

- , PERIODS OF INDIVIDUAL WAVE IN WAVE GROUPS AND
- , CORRESPONDING ASSOCIATED DISPLACEMENTS.

FIG.17 ANALYSIS OF STRUCTURE OF TYPICAL WAVE-GROUPS UNDER VARIOUS CONDITIONS OF EXPERIMENT.

As has already been stated the excitation in the foundation investigations was due to siphons running as weirs with a total discharge of 2,177 cusecs. A similar experiment was conducted in 1950, with a discharge varying from 10,000 to 52,000 cusecs, through the undersluices. Analysis of records of this investigation (with Wood-Anderson Seismograph) is represented in Table 1 below.

TABLE 1

Discharge in cusecs	Component.	Period in sec.	Displacement in mm. $\times 10^{-3}$
10,000	Parallel to the axis	0.122	0.495
20,000	"	0.123	0.605
30,000	"	0.121	0.705
10,000	Perpendicular to the axis	0.133	0.355
20,000	"	0.135	0.649
30,000	"	0.137	0.713
40,000	"	0.140	0.979
52,000	"	0.141	1.237
Occasional spurts of 30,000 (Total) through siphons.	"	0.126	2.745

It will be seen that the mean period is nearly the same as that of the low frequency spectrum in Figure 2. This shows that *the period does not depend so much on the force of excitation as on the structural and boundary conditions*, though the displacement increases with the force of excitation.

The analysis of vertical vibration records, with geophones is shown in Figure 4. It is seen that the period remains practically constant at a mean value of 0.08 second, though the associated relative velocity shows large scatter. Fig. 4.

Figures 5 and 6, which have been already discussed in connection with vibration study on the dam, give analysis of records on the foundation also. It is seen that the same low and high frequency spectra in the horizontal component, which are present in the vibrations of the dam, are also present in vibrations of the foundation. On the other hand the vertical spectrum on the dam is quite different from that on the foundation. This difference in behaviour in vertical and horizontal vibrations requires further study. Fig. 5 & 6

7. Vibration Study on the Dome of Siphon No. 7

Graphic representation of the investigation at this station (with geophones) is shown in Figure 18 during the primed state of the siphon. It shows that there exists a high frequency spectrum of period 0.05 sec. (low periods) with superimposed wave-groups, having widely varying periods from 1 to 10 secs (low frequency spectrum). It is seen that though the high frequency spectrum is highly uniform, the low frequency is highly non-uniform, pointing to the fact that during the primed state of the siphon, the state of affairs, as far as the low frequency spectrum is concerned, was highly disturbed. It is, however, difficult to associate this low frequency spectrum with any natural vibration of the structural elements. Fig. 18.

Period of high frequency is highly uniform and so is its associated velocity. The value of this period is same as that on the dam (station 5) and so also the associated relative velocity. It is also seen that the high frequency spectrum is uniform in all components under various conditions of excitation.

Value of maximum acceleration at the dome is found to be of the order of 0.02 g

Figure 18 shows a number of vibrations of high amplitude, but of irregular character. These vibrations may be associated with the *bursts which could be heard during the primed state of the siphon*. Fig. 18

Maximum acceleration associated with these vibrations (associated with the "Bursts") is found to be of the order of 0.03 g. This may be compared with 0.02 g on the dome in the absence of bursts and on the dam at station 5 opposite to siphon No. 7 and 0.01 g at station 4 (150 ft. away from station 5).

Fig. 18. Figure 18 also shows that these vibrations are highly irregular but once they start they continue for some time and this points to some sort of trigger action.

The vertical vibrations on the dome show a similar character to those of horizontal vibrations.

The study on the dome of siphon No. 7 was also made with Cambridge Vibrograph (magnification 5). Being a Mechanical Instrument, it was not expected to register high frequency spectrum present in the geophone records.

Fig. 20. Figure 20 shows the analysis of the record with this instrument, during the period as well as depriming states. Periods, associated displacements and calculated maximum accelerations have been shown.

It is seen that during the primed state, the periods maintain a nearly uniform character and so is the case of associated displacements, though to a slightly less degree. Calculated acceleration, however, shows a larger scatter, varying from 4.5 cms./sec². to nearly 10 cms./sec².

The figure also shows periods, associated displacements and calculated accelerations during depriming state. It is seen that the vibration during the primed state is more sustained than during the depriming state. The period here also maintains nearly same value as that in the primed state but associated displacements and calculated, maximum accelerations show much more scatter than those during the primed state. This observation shows that though the vibrations of the dome, during the depriming state, was of highly irregular nature, the average acceleration encountered is of the same order as in the primed state.

Study at dome with the Reed Vibrometer showed a number of predominant frequencies, including those met with in the Wood-Anderson and Geophone Records

8. Conclusions

The investigations show that the period of horizontal vibrations of the dam remains nearly constant, irrespective of the station of observation on the masonry dam and the force of excitation. This points to the fact that the period depends more on the structural and boundary conditions rather than on force or method of excitation. On the masonry dam two individual wave-periods, namely, 0.13 and 0.05 sec were present in horizontal vibrations whereas only one, namely, 0.04 sec. was present in the vertical vibrations.

On the earth embankments the periods for horizontal vibrations were 0.07 sec and 0.025 sec. and that for the vertical vibrations was 0.05 sec

On the foundation the corresponding periods for horizontal vibrations were 0.13 sec and 0.06 sec. and that for vertical vibrations was 0.08 sec. It has been also noted that the period of vibration is more or less independent of the force of excitation, though the displacement increases with the forces.

The investigation also points to unsteady nature of vibrations during the primed state of siphon No. 7 as compared with the state during working of undersluices.

It has also been found that during the functioning of the undersluices the calculated maximum acceleration at station 4 was of the order of only 0.001 g. whereas during the primed state of siphon No. 7 it was of the order of 0.01 g. This shows that the force of excitation due to siphon in the primed state is much more than that due to the undersluices working.

VIBROGRAPH RECORDS

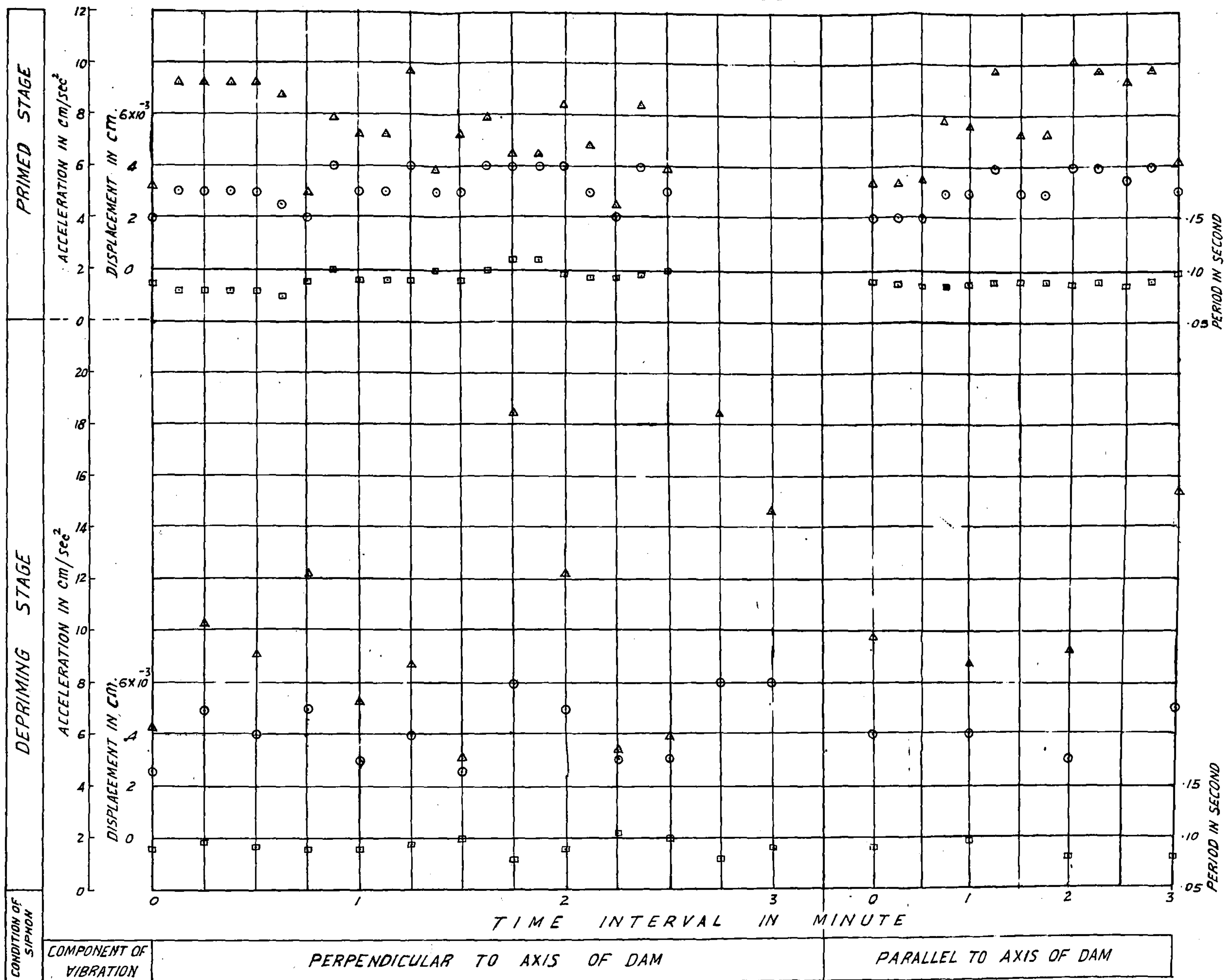


FIG. 20

HIREBHASGAR DAM

ANALYSIS OF CAMBRIDGE VIBROGRAPH RECORDS
ON THE DOME OF SIPHON NO. 7VARIATION OF PERIOD OF VIBRATION, MAXIMUM DISPLACEMENT
AND CALCULATED MAXIMUM ACCELERATION
UNDER TWO DIFFERENT CONDITIONS OF SIPHON

REF: -

PERIOD IN SECOND.....□

DISPLACEMENT IN CM.....○

ACCELERATION IN cm/sec².....△

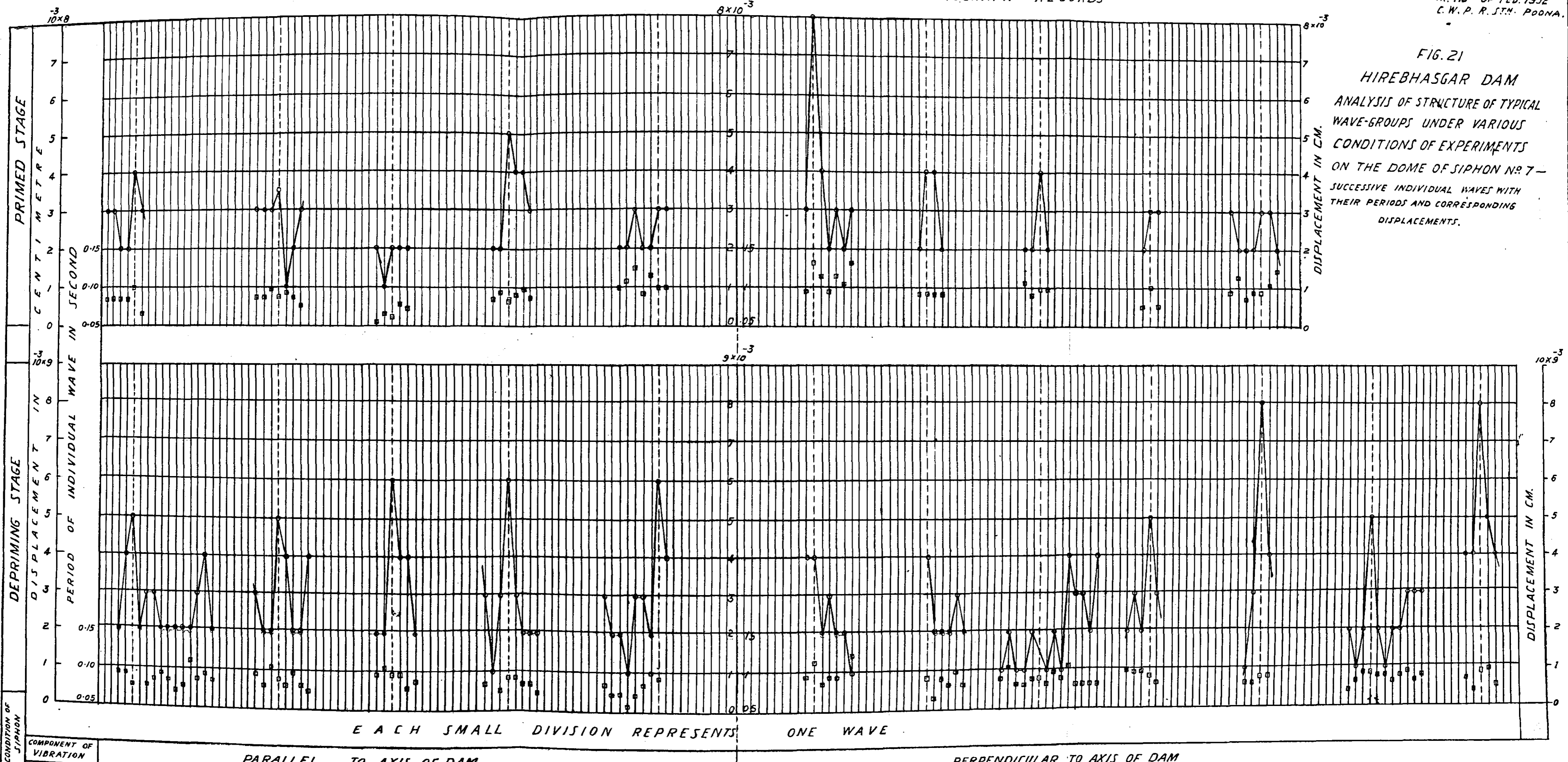


FIG. 21
 HIREBHASGAR DAM
 ANALYSIS OF STRUCTURE OF TYPICAL
 WAVE-GROUPS UNDER VARIOUS
 CONDITIONS OF EXPERIMENTS
 ON THE DOME OF SIPHON NO. 7 -
 SUCCESSIVE INDIVIDUAL WAVES WITH
 THEIR PERIODS AND CORRESPONDING
 DISPLACEMENTS.

EACH SMALL DIVISION REPRESENTS ONE WAVE

PARALLEL TO AXIS OF DAM

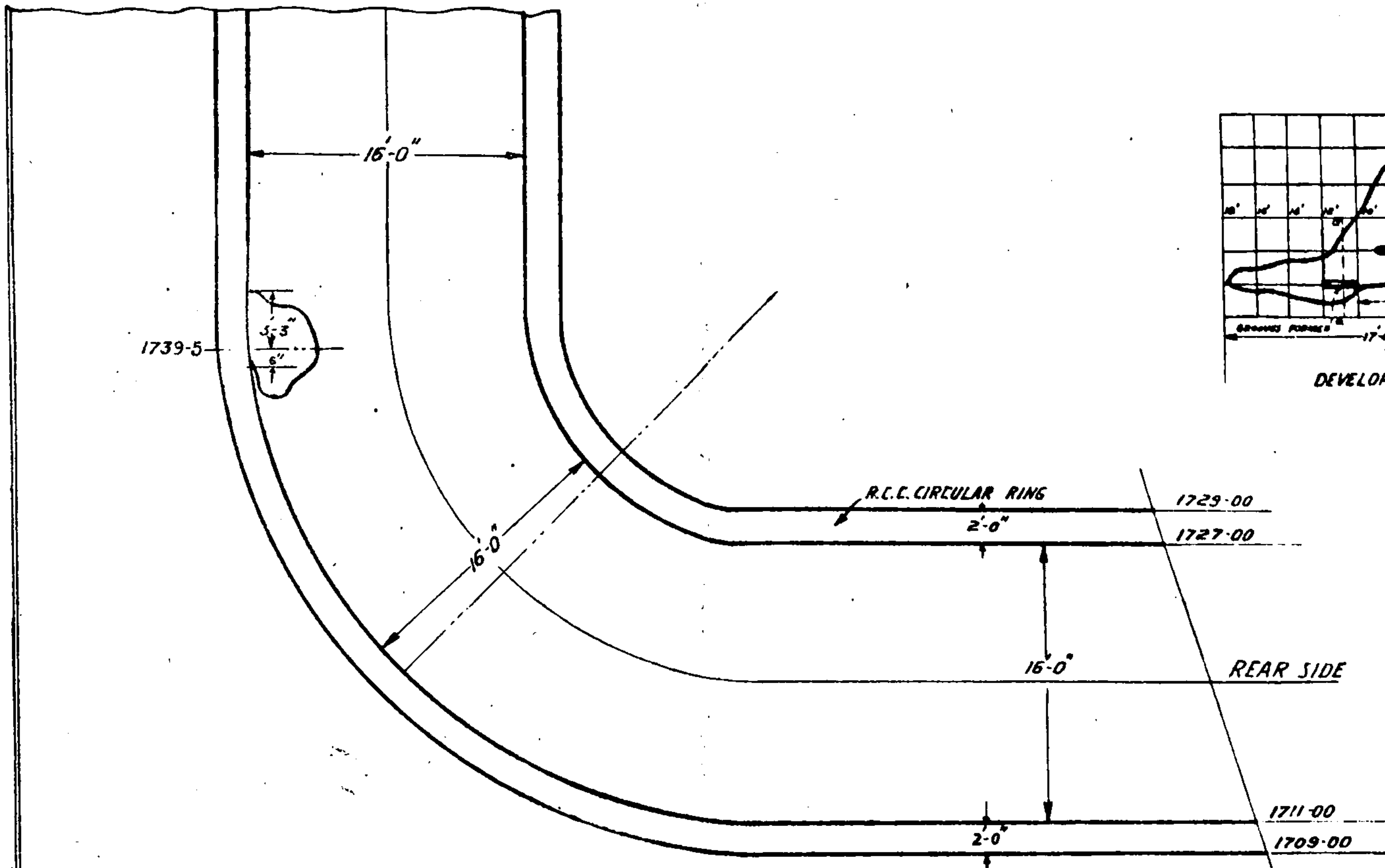
PERPENDICULAR TO AXIS OF DAM

REF: PERIOD OF INDIVIDUAL WAVE IN WAVE-GROUPS --- ■
 CORRESPONDING ASSOCIATED DISPLACEMENTS --- ○

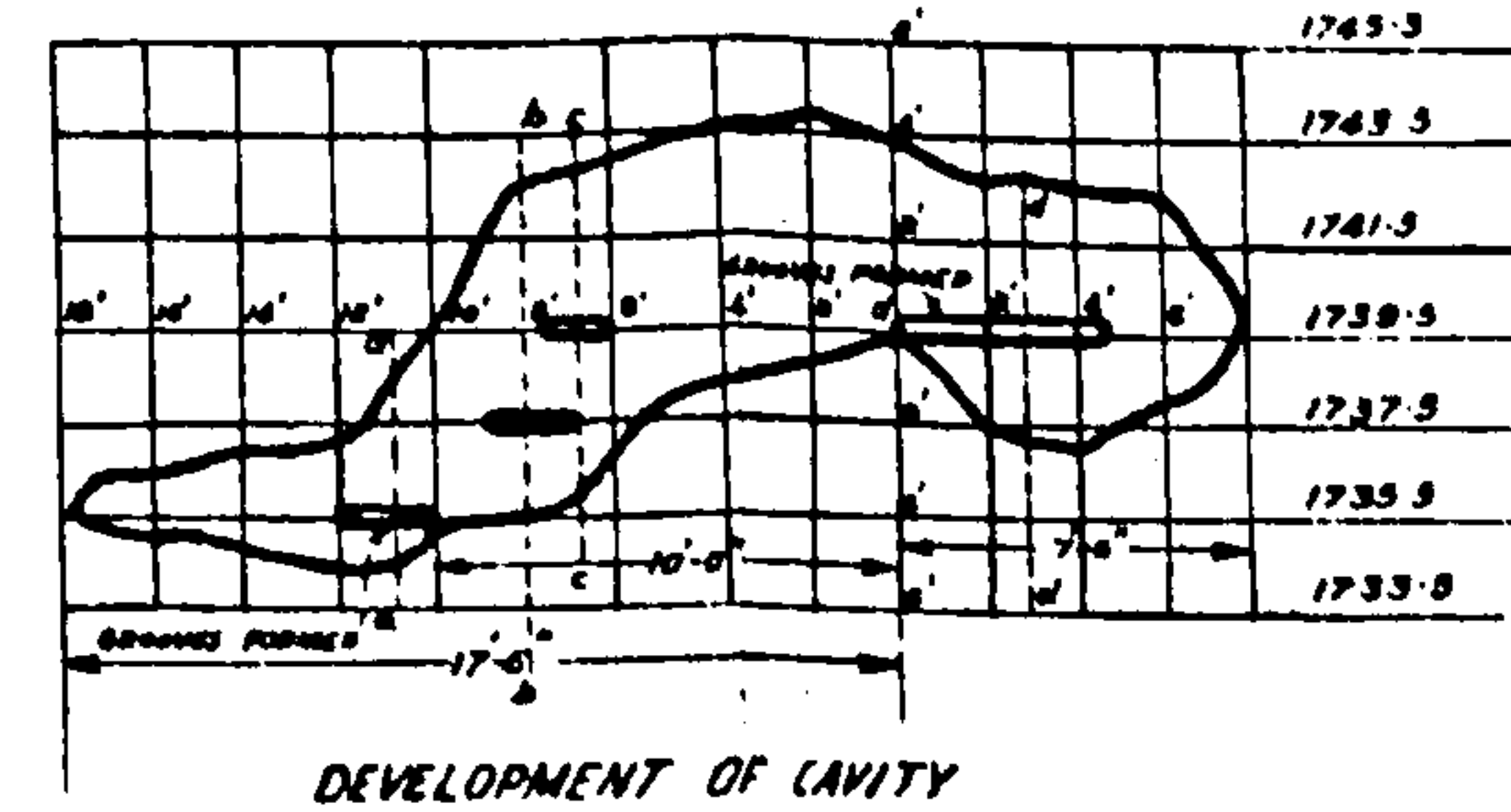
COMPARED BY:
 TRACED BY: M.K. SABNIS. *MS*

(REDUCED & COPIED FROM GOVT. OF MYSORE
M.S.N.E. WORKS DRG. No. J. 1043.)

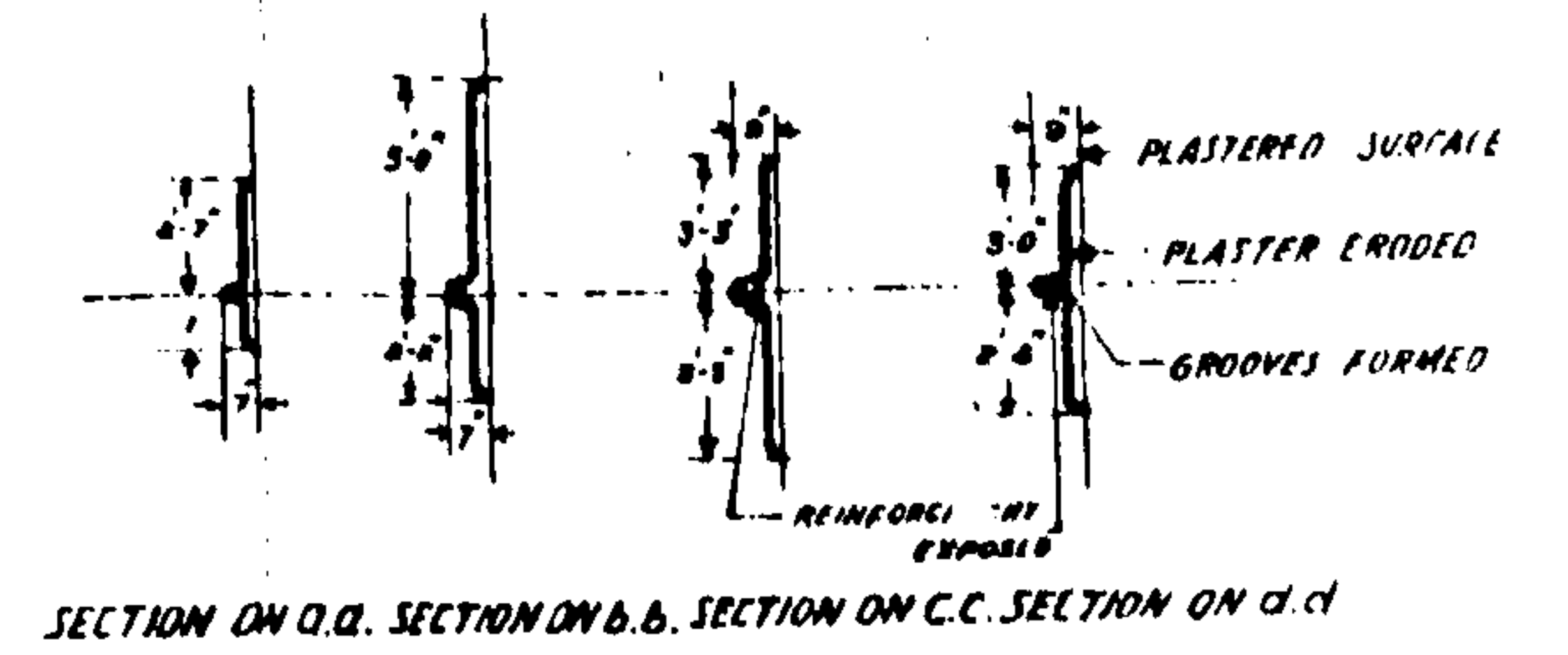
WATER SIDE



SECTION ON AA

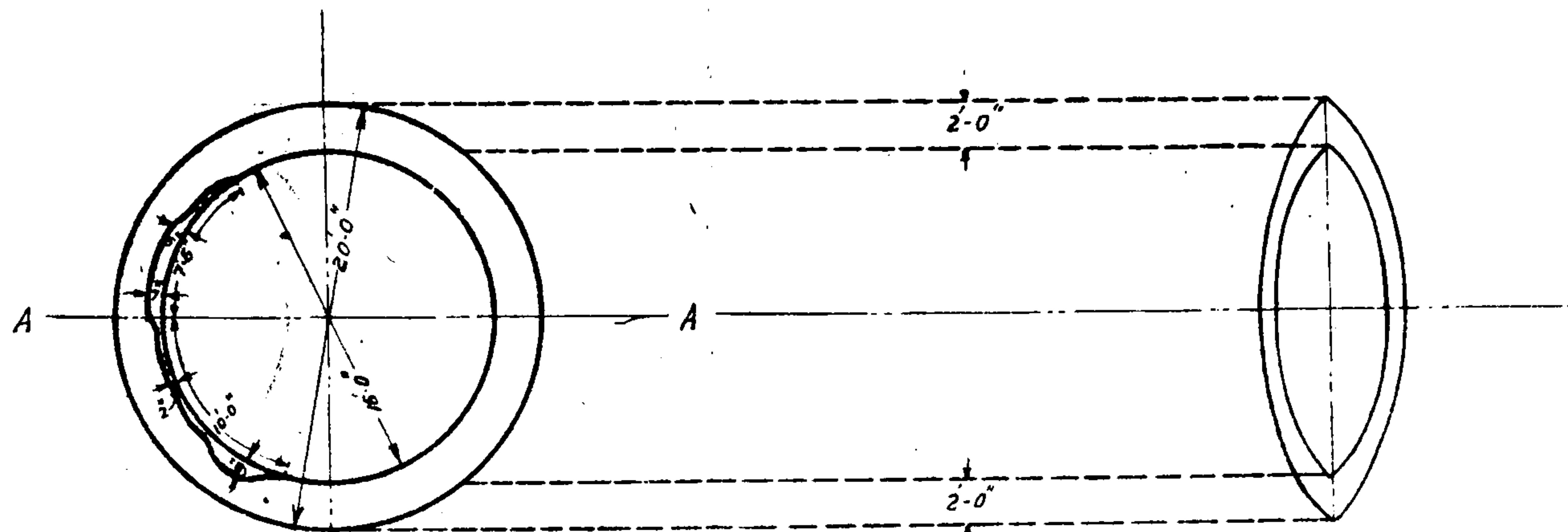


DEVELOPMENT OF CAVITY



SECTION ON B.B. SECTION ON C.C. SECTION ON D.D.

WATER SIDE



PLAN AT 1739.5

FIG: 22
HIREBHASGAR DAM SIPHON No 7
DETAILS OF DAMAGE AFTER OPERATING IT
ON
17-8-1951.

SCALE 1/4" = 2 FEET

COMP'D BY: [Signature]
TRACED BY: S.A.B.V.S. 10-10-51

Station No. 4 is at a distance of 150 ft from No. 5 which is opposite the siphon No. 7, the source of disturbance. Maximum calculated acceleration at station 5 and on the dome of siphon No. 7 was found to be of the order of 0.02 g. As compared to this the maximum acceleration which was obtained with all the undersluices open at the station 3, located just above these undersluices, was only 0.003 g.

Records also show some high amplitude vibrations, of an irregular character, on the dome of siphon No. 7, during its primed state. These vibrations might be associated with the load bursts heard during the primed state of the siphon. Maximum acceleration for these vibrations was of the order of 0.03 g.

A comparison between accelerations, produced during the primed and depriming states is available at station 4 and it shows an abrupt fall of acceleration, when the depriming starts, to a value 1/3rd during the primed state.

Acceleration on the dome, calculated from the vibrograph records had a much lower value than those obtained from the corresponding geophone records, referred to in the preceding para. This is because, this instrument does not respond to higher frequencies (obtained with geophones), with which higher accelerations are associated. It is found from vibrograph records that the average of maximum accelerations on the dome were of the same order during the primed and depriming states, but it showed more scatter during the depriming state which may perhaps be associated with air entrainments.

Dark-room arrangements for Wood-Anderson Seismograph record had been made at station 4, opposite siphon No. 10, which was expected to prime, but siphon No. 7 primed instead. Hence the investigations had to be done under somewhat unfavourable conditions namely, that absolute records were available only at station 4 and relative records with geophones at all other stations. All the same the results show that the siphon in primed and depriming states generates vibrations of much larger energy than when the undersluices are working alone and the corresponding accelerations are discussed above.

20 EXPERIMENTS WITH HIREBHASGAR VOLUTE SIPHONS

After the inspection of the Hirebhasgar Siphons during the monsoons of 1951, it was found necessary to conduct certain experiments with a 1/24 scale G. S. model, for effecting some improvements in their design so as to make their working safe. The following tests were carried out:

(I) For 18 ft. diameter siphon—

- (a) The outlet length being short, the siphon did not run full of its own accord after priming. Pressures were recorded for this condition.
- (b) The outlet was made to run full by creating a temporary back-pressure, which result could also be attained by extending the outlet. Pressures were recorded for this condition.
- (c) Sri Ganesh Iyer has proposed an extension piece so as to make the siphon run full—see Figure 1. Pressure observations were made to see whether cavitation conditions were different from the existing units as observed in tests (b) above.

Fig. 1.

Figure 1 shows the pressure curves of the siphon in cases (b) and (c) mentioned above. In case (b) the maximum equivalent negative pressures indicated cavitation. Figure 1 also shows the pressures obtained when the outlet of 18 ft. diameter siphon was constricted to 16 ft. It will be seen that in this case the pressures are within safe limits. Though various siphons have primed at Hirebhasgar, they have run only part full and their discharge was less than 10,000 cusecs. In some cases intense cavitation was indicated and hence the design could not be considered safe. It may be noted that Shri Ganesh Iyer's proposal of extending the outlet to make the siphons run full will not remove the possibility of cavitation, but on the other hand by improving the flow conditions in the bend is likely to intensify it. The equivalent discharge figures of 11,600 and 13,370 cusecs were obtained from model results where cavitation does not occur unless the experiments are done in a cavitation tank and hence are not applicable to prototype where cavitation is indicated.

Table 1 is the record of pressure observations.

(II) 16 ft. diameter siphons (Siphon 7).

Fig 2.

The performance of siphon No. 7 during 1951 floods indicated the presence of cavitation and it was considered necessary to make pressure observations in models for the existing condition and also to find out what minimum constriction is necessary to reduce the negative pressures to safe limits. Table 2 is the record of pressure observations and Figure 2 is self-explanatory. Three constrictions were tested and an outlet constriction to 15 ft. appears to serve the purpose. The discharge in prototype is indicated as 8,900 cusecs, with a coefficient of discharge of 0.815.

Allowing for fluctuations in mean pressures, a constriction to 14 ft. 6 inches giving a discharge of about 8,200 cusecs is desirable.

TABLE 1—Pressure observations with 1/24th scale model of Volute Siphon No. 4

Pressures in Siphon No. 4 where (a) it was not running full, (b) running full and (c) also with Shri Ganesh Iyer's extended outlet design.

Pressure Point Nos.	Depth of piezometric point from lip in ft.	Pressures in feet of water.		
		(a)	(b)	(c)
1	2	3	4	5
1	2	-22 50	-36 50	-38 00
2	5	-12 00	-21 25	-23 50
3	8	-24 00	-39 25	-40 00
4	12	-17 00	-30 50	-34 50
5	15	-14 50	-30 25	-32 50
6	18	-9 75	-27 25	-29 00
7	20 5	-10 50	-23 25	-24 50
8	30	-5 00	-21 50	-26 00
9	33	-16 75	-25 50	-27 50
10	37	-11 25	-34 75	-38 00
11	41	-9 25	-34 50	-39 00
12	43 5	0 00	-29 00	-36 00
13	46	0 00	-26 75	-28 00
14	47 5	0 00	-26 00	-24 50
15	48	0 00	-20 50	-19 50
16	48	0 00	-14 00	-15 00
17	48	0 00	-4 50	-5 00
18	48	0 00	-16 75	-12 00
19	47 5			-8 00
20	47 0	Discharge scale = $(24)^{5/2}$		-4 00
21	46 5	$= \sqrt{576 \times 576 \times 24}$		-8 00
22	46	$= 2822$		-4 00
23	45 5			-5 00
24	65			+14 00
25	65			+3 00
26	64 5			+8 00
27	64			+4 50
Discharge in model in cusecs		Q=3 50	Q=4 12	Q=4 74
Equivalent Discharge (proto) in cusecs		9,877	11,600	13,370
Coefficient of Discharge		0 670	0 789	0 823

FIG:- 1

LEGEND:-
ORIGINAL DESIGN ----- Δ ----- Δ ----- Δ
EXTENSION ADDED AT OUTLET.
(SRI. GANESH IYER'S PROPOSAL) ----- X ----- X ----- X
OUTLET CONSTRICTED TO 16' DIAM. O ----- O ----- O

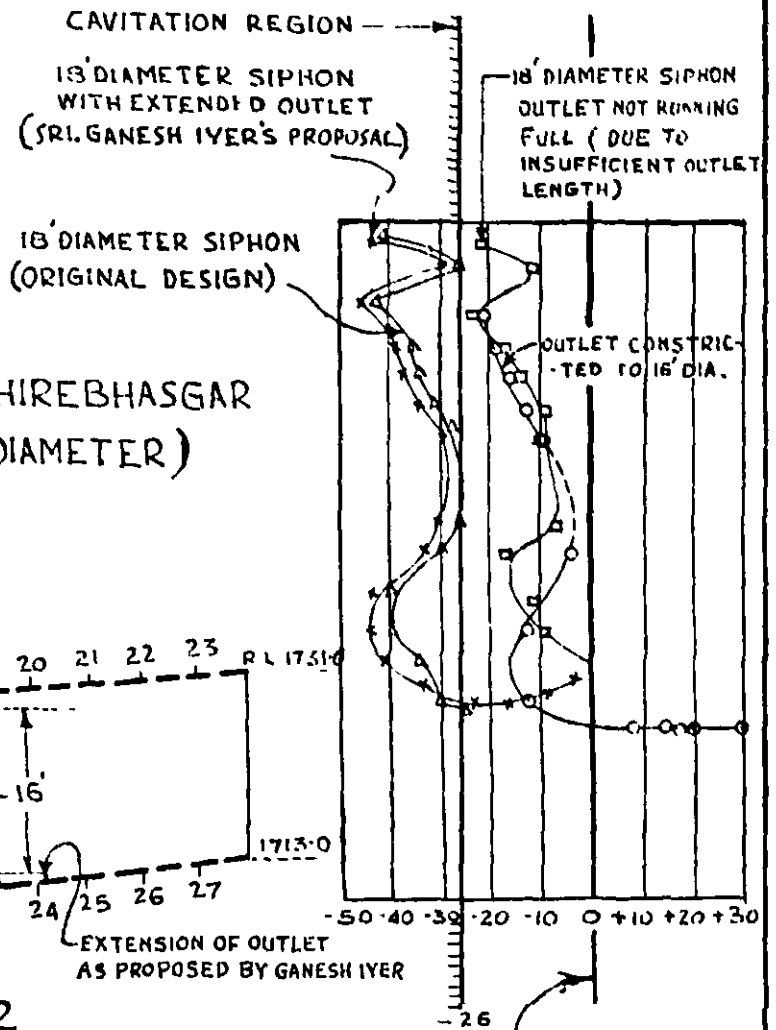
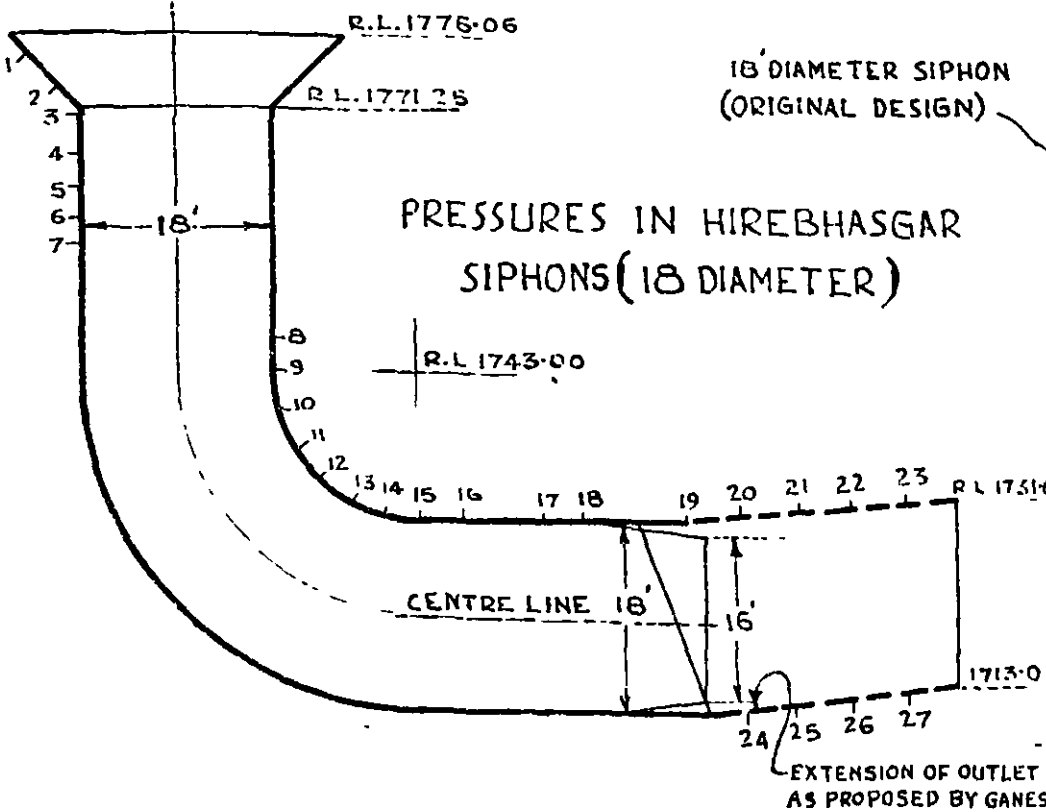
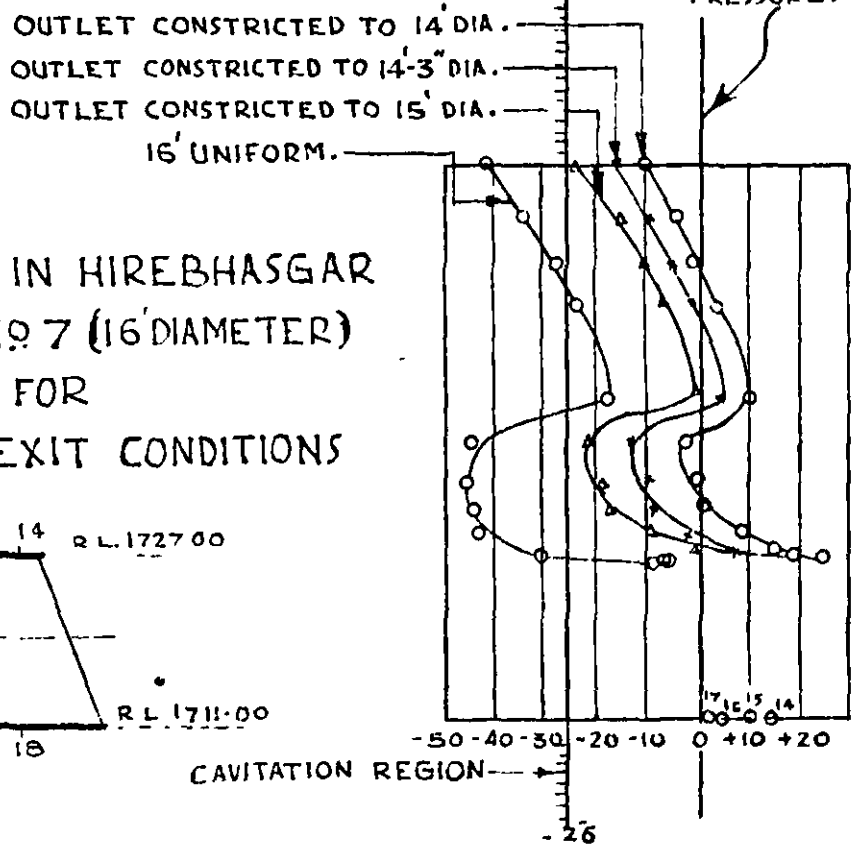
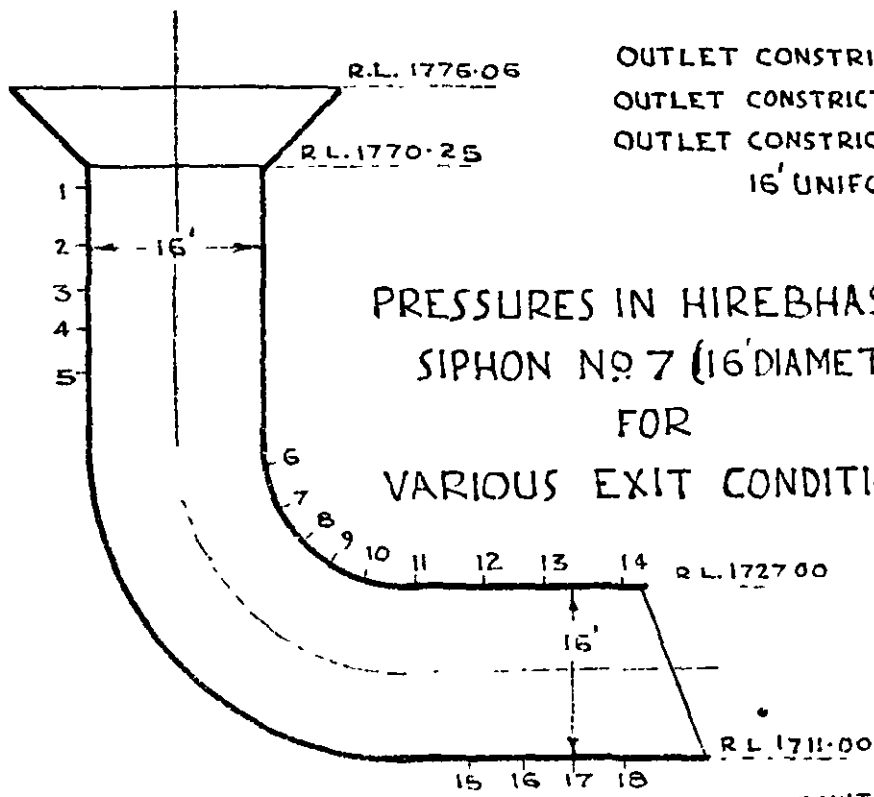


FIG:- 2



LEGEND:-
16' UNIFORM DIAMETER O ----- O ----- O
OUTLET CONSTRICTED TO 15' DIA. Δ ----- Δ ----- Δ
OUTLET CONSTRICTED TO 14.25' DIA. X ----- X ----- X
OUTLET CONSTRICTED TO 14' DIA. O ----- O ----- O

EXPERIMENTS WITH HIREBHASGAR SIPHON MODELS
PRESSURE OBSERVATIONS
SCALE:- 1" = 20'

COMP BY: *[Signature]*
BY: BHINGARE 16-1-52.

TABLE 2—Pressure observations with 1/24th scale model of Siphon No 7

Serial No.	Depth of piezometric point from lip	16' uniform Diameter	Outlet constricted to 15 ft Diameter	Outlet constricted to 14.25 ft. Diameter	Outlet constricted to 14.0 ft Diameter
1	10.0	-42.0	-25.5	-17.0	-11.0
2	15.5	-34.24	-15.25	-10.4	-4.8
3	20.0	-28.50	-10.625	-6.4	-1.2
4	23.75	-24.50	-6.375	-1.4	+3.5
5	28.0	-17.75	-0.5	+4.5	+0.2
6	37.25	-4.5	-22.0	-13.4	-2.8
7	41.0	-4.5	-18.625	-10.0	-0.5
8	44.0	-4.4	-17.25	-8.8	+0.8
9	46.0	-43.25	-9.375	-1.8	+8.12
10	48.0	-31.0	-1.375	+6.0	+14.12
11	49.0	-8.25	+13.25	+18.4	+24.12
12	49.0	-7.0	+14.375	+18.8	+25.0
13	49.0	-6.75	+14.0	+18.9	+24.8
14	49.0	-6.0	+14.75	+20.5	+25.5
15	65.0	+13.0	+32.75	+37.5	+42.8
16	65.0	+10.0	+31.25	+38	+42.12
17	65.0	+5.0	+28.75	+35	+39.9
18	65.0	+4.5	+28.75	+34.5	+40.8
Discharge (Model)		3.8 cusecs	3.16 cusecs	2.85 cusecs	2.74 cusecs
Discharge (Proto) (Equivalent)		10,720 "	8,900 "	7,900 "	7,700 "
Coefficient of Discharge (based on diameter of constricted outlet).		0.857 (cavitation).	0.815	0.815	0.810

21. STANDARDIZATION OF RELAXATION SOLUTION FOR OVERFLOW SPILLWAYS

Herein is reported a systematisation of pressure computations for overflow spillway designs using a two-dimensional relaxation treatment of the problem, which is a sufficiently good approximation to the field of flow in the case of the usual routine tests of sectional models of spillway and weir sections, where three-dimensionality and viscous effects do not exercise any important influence on the overall flow.

The method of approach to the case of free streamline flow under gravity is now well known, as included in the methods applied by Southwell and co-workers in a variety of problems in engineering and theoretical physics. In the present report, the solution for the field of flow for a standard crest profile used for any design head is given. In the normal case, any head other than the design head would require a fresh solution *ab initio*. Herein is given a method of avoiding much of the computations for the case of other heads. The solutions for the full range of head from very nearly zero to twice the design head are therefore easily systematised. These can now be used for a routine check-up of any spillway crest profile for any head with little additional labour.

In the Annual Report (Technical), 1950, the method of solution for two-dimensional sectional spillway models has been exemplified in relation to the Dimna Nala Spillway. Assuming that the stream function ψ , for the standard case of a free overfall with a large upstream depth is a solution of Laplace equation.

$$\frac{\partial^2 \psi}{\partial x^2} + \frac{\partial^2 \psi}{\partial y^2} = 0;$$

the co-ordinate axis x and y are chosen to be horizontal and vertical respectively, the y -axis being assumed to be the vertical upstream face. The velocity being defined by

$$\text{horizontal velocity } u = -\frac{\partial \psi}{\partial y}$$

$$\text{vertical velocity } v = \frac{\delta\psi}{\delta x}$$

$$\text{and tangential velocity} = -\frac{\delta\psi}{\delta n}$$

where $\delta/\delta n$ is a differentiation normal to the streamline.

If on the free streamline, $\psi = \psi_0$, and the velocity head be y_s , where y_s = depth below stagnation level, then

$$\delta y/\delta n = \sqrt{2gy_s}$$

To make all quantities non-dimensional, let the unit of length be a , unit of discharge be q , and we write

$$y = y'a, \quad n = n'a, \quad \psi = \psi'q$$

where y' , n' , ψ' are dimensionless, then

$$\delta\psi'/\delta n = \frac{\sqrt{2ga^3H'}}{q} \sqrt{y'/H'}$$

with $H = aH'$ = head as properly defined.

If from the relaxation solution, the relation obtained between y'/H' and $\delta\psi'/\delta n$ be given by

$$\delta\psi'/\delta n' = k \sqrt{y'/H'}$$

$$\text{then } k = \frac{\sqrt{2ga^3H'}}{q} \text{ and } \psi = \psi'q = \frac{\psi'}{kH'} \sqrt{2g} H^{3/2}$$

The discharge coefficient therefore becomes

$$C = \frac{q}{H^{3/2}} = \frac{\psi'}{kH'} \sqrt{2g}$$

Fig. Solution for the standard profile (Figure 1) shows that the value of C obtained from the k -value deduced, becomes

$$C = 4.06 \text{ ft. sec. units.}$$

With the k value obtained, the velocity head can be calculated at any desired point of the field of flow, and the corresponding pressures deduced. In particular, the pressure reduction on the upstream face due to the kinetic energy of flow can be obtained.

Nappe-shaped Dams—with Heads Different from Design Head

As the head is gradually raised above the design head, the crest pressures continue to have steep gradients, and negative values, until for sufficiently high head, a stage is reached when the boundary layer separates from the face of the dam and a back roller forms, beyond which state, the flow ceases to be predominantly potential flow. The solutions here have therefore been restricted to approximately twice the design head for a standard profile.

The solutions for the different heads can be considered either by changing the upper nappe and the stagnation level while keeping the lower nappe fixed, or otherwise the lower nappe can be changed keeping the stagnation level and the upstream face the same so as to give the same picture only on a reduced or magnified scale. The latter method is most suitable for a minimum of labour. Referring to Figure 2

STAGNATION LEVEL

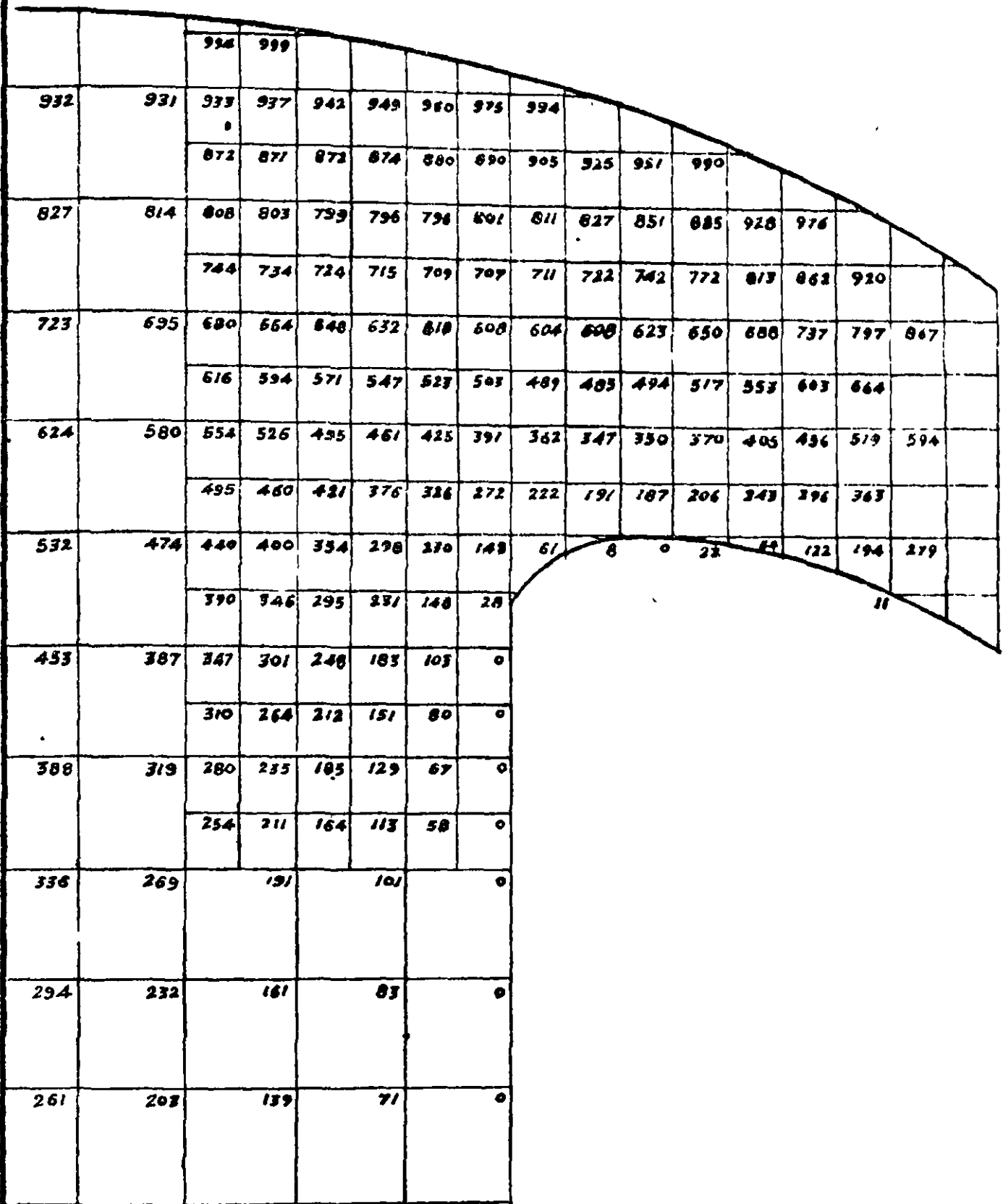


FIG.-1
SOLUTIONS CLOSE TO
THE SHARP CRESTED DAM
SPILL DEPTH OVER THE UPSTREAM SHARP CREST
IS 5:62 UNITS IN TERMS OF COARSER MESH.

COMPILED BY SANKAL.

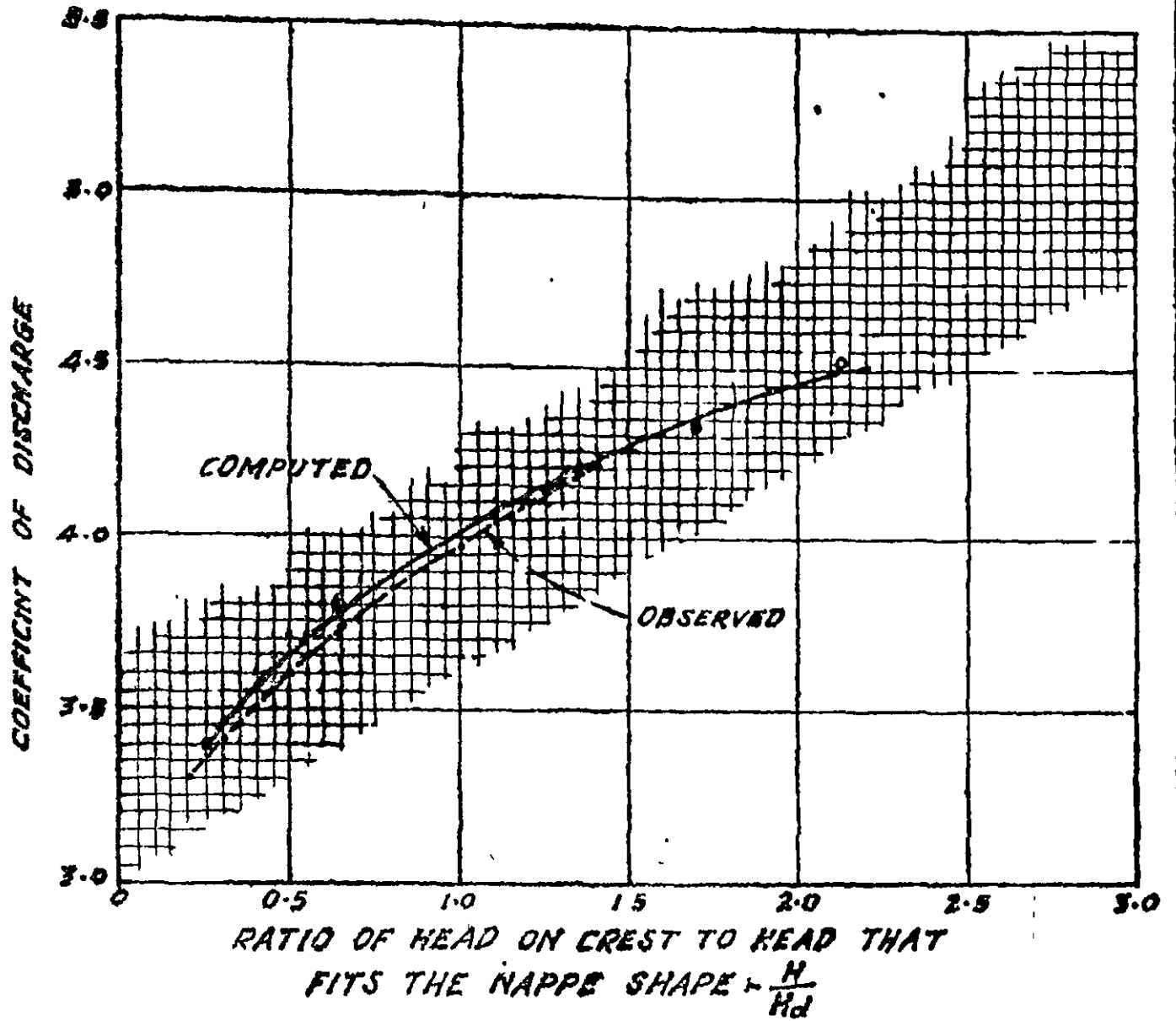


FIG 3
 COMPUTED DISCHARGE COEFFICIENTS
 ALONG WITH THE OBSERVED COEFFICIENTS
 (*THE HANDBOOK OF HYDRAULICS-DAVIS.)

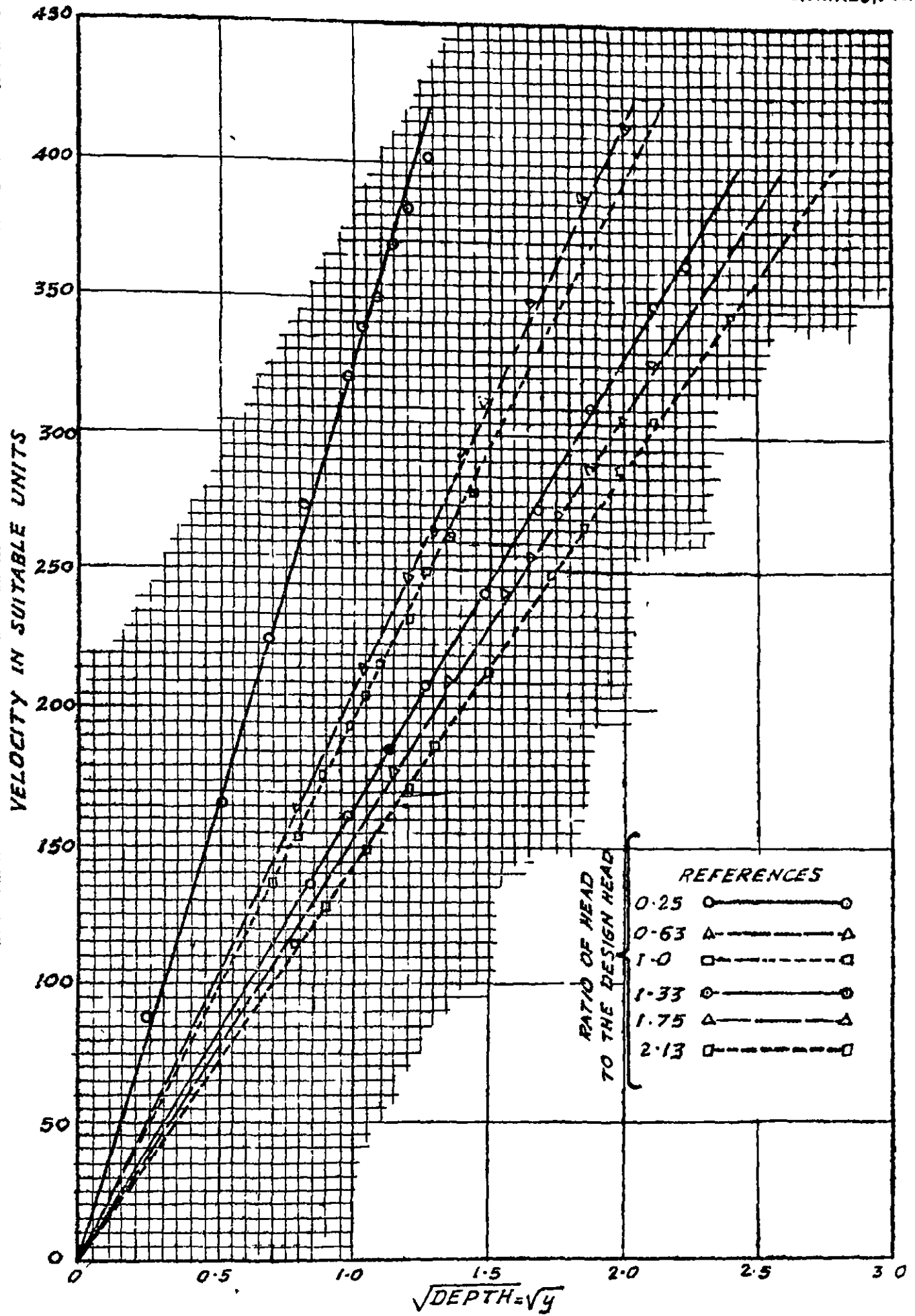
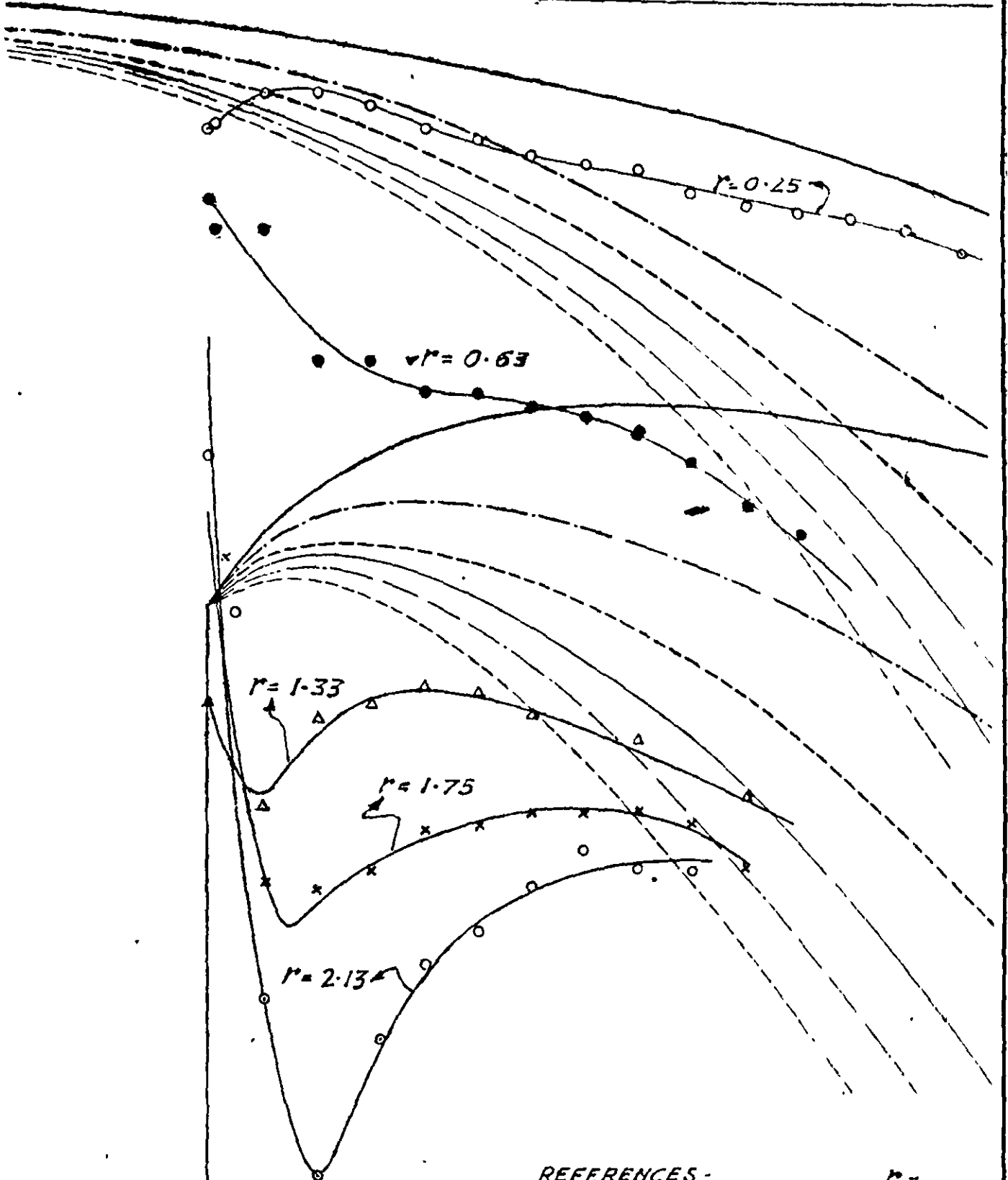


FIG-4
 RELATION BETWEEN VELOCITIES
 ON THE FREE SURFACE & THERE
 (DEPTHS)^{1/2} BELOW THE
 STAGNATION POOL

STAGNATION LEVEL



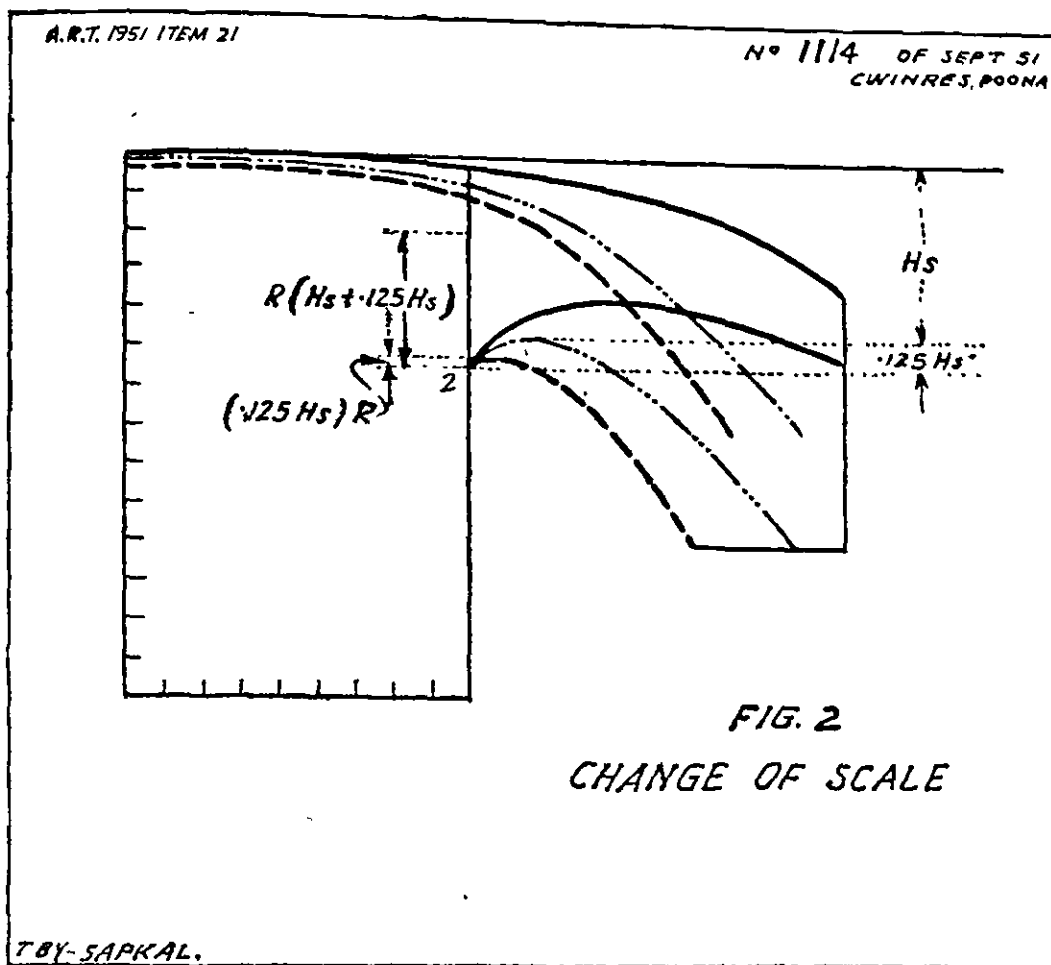
REFERENCES -		$H =$
PRESSURE CURVES FOR DIFF. HEADS	PROFILE FOR THE FREE SURFACE & THE NAPPE SHAPE	RATIO OF HEAD TO THE DESIGN HEAD
○ — ○ — ○	—————	0.25
● — ● — ●	-----	0.63
— — — — —	-----	1.00
△ — △ — △	—————	1.33
× — × — ×	-----	1.75
○ — ○ — ○	-----	2.13

FIG.-5
PRESSURE DISTRIBUTION
ON THE CREST.

COMP. BY SAKAL
TRACED BY SAKAL.

where the design head is represented by H_s , it is easily seen that for the case of solution for head $= rH_s$, the field of flow is magnified R times where

$$R = 1.125 / (r + 0.125)$$



Using this magnification, and an approximate appropriate change of the free surface, a solution for the head $= H_s$ can be obtained which can be tested for linearity of v^2 . Any further modification of the free surface usually consists of a minor smoothing out of the profile.

The discharge coefficients for a set of heads varying from $0.25 H_s$ to $2.13 H_s$, are shown in Figure 3.

Fig. 3.

The linearity relations between velocity head and the depth below the stagnation level on the free surface profile are shown in Figure 4.

Fig. 4.

The computed pressures for the spillway crest in the different cases are shown in Figure 5. It will be observed that the characteristic sharp dips in the crest pressure curves usual with high coefficient profiles are very well reproduced in the computed curves.

Fig. 5.

While the individual velocities are accurate to about 2 per cent, except very near the sharp edge where finer nets are needed for accuracy, the discharge coefficients are probably accurate up to one per cent. The slightly lower values in the observed coefficient of discharges as compared to the computed values of the order of 1—2 per cent indicate the order of turbulence losses.

It will thus be seen that for the majority of model studies which use transfer equation based on the Froudean relation only, the potential changes associated with acceleration due to gravity being predominant, the design data regarding the

pressure distribution along with other details regarding velocities at any point of the field of flow can be obtained as routine check-up from these standardised solution.

The computation details could become important in a more fundamental field of research, viz., the development of boundary layer under varying accelerated or

retarded flow. The rather complex velocity and pressure distribution associated with high velocity flow over the crest can give us further insight into the laws of variation of skin-friction in a developing boundary layer when the problem of separation becomes an important feature of the phenomenon, as in the case of high coefficient weir profiles, which however is not being considered here.

22. TESTS FOR THE HYDRAULIC BRAKE DEVICE INVENTED BY SIGNOR TAPIAS

The hydraulic brake device invented by Signor Tapias was tested in this Research Station. The device consists in providing a part cylindrical bucket at the toe of falling water as shown in Figure 1.

Fig. 1.

According to the inventor:

"The hydraulic braking device which is the object of the invention tends to stop the flow of water from a duct of a free waterfall by means of a return movement and an impact of this water as it is produced in the swell, brakes or slows down the flow of water which follows at the precise position where this slowing down must be produced, forming at this position a zone where the flow will be stopped as in the zone where the turbulence is produced as in the phenomenon of the sea swell".

Apart from the doubtful possibility of its economic design, it appeared that the device was meant for high structure. As such it was tested in the model of Gajod Weir (Kutch) and a canal fall.

Comments on the Basis of Design

The inventor's note starts with an analogy of the dissipation of a sea swell. He suggests that the energy in the forward wave, i.e., the wave approaching the shore is dissipated because of its impact with the reflected wave. Figures 1(a) to 1(e) in his note show the loss of form and energy of a forward wave when it collides with a reflected wave. This loss of form and energy does not occur, as the inventor suggests, because of the collision with the reflected wave, but it occurs because the forward wave, as it approaches the shore over decreasing depths, continuously gains in amplitude with decrease in wave length and consequently becomes unstable and begins to fall by curling down over its upper part.

Figs. 1(a) to 1(e).

This instability occurs when the ratio of wave length to amplitude reaches the value 1/7. As occurs in a general phenomenon of interference of two waves, the collision of two waves produces a standing wave form. The forward wave may dissipate its energy, as shown by the author, if only instead of a reflected wave there is a current involving mass transportation of water.

The author seems to have taken inspiration from the phenomenon of dissipation of a sea swell; but as the experiments described below have shown, this hydraulic brake device was not effective in dissipating energy below weirs or canal falls.

Tests in the Model of Gajod Weir

The Gajod Weir has a drop of 48.5 feet and 95 cusecs discharge per foot run. In a 1/30 scale G. S. model a suitable design had been previously tested for a horizontal apron, baffle blocks and an end sill for efficient energy dissipation (Figure 2). In the same model, with the cylindrical bucket as shown in Figure 3, three tests were made with slight alterations in the bucket design. In all the tests a common observed feature was that the jet skipped over the cylindrical bucket when discharge was high while the bucket functioned merely as a stilling pool for low discharges. For the intermediate range for which the bucket was designed the following observations were recorded.

Fig. 2.
Fig. 3.

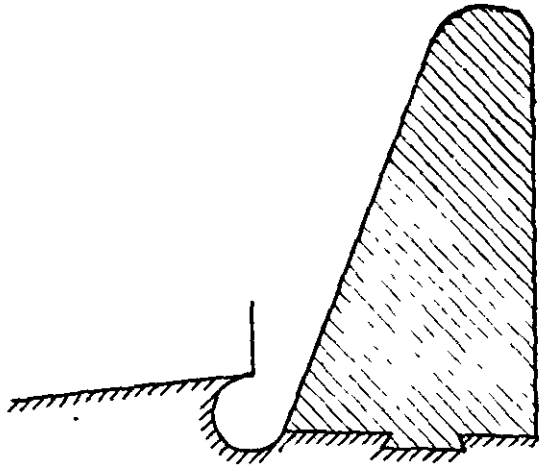


FIG 1

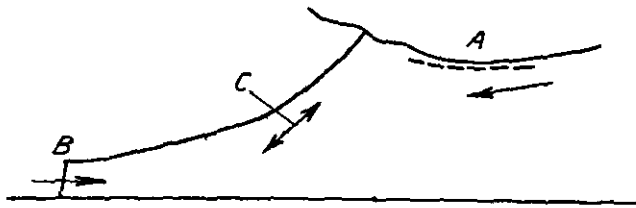


FIG 1(a)

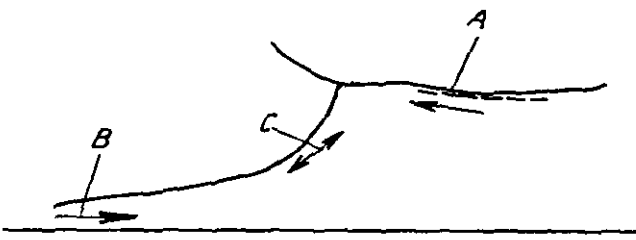


FIG 1(b)

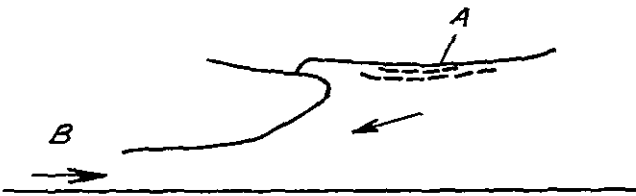


FIG 1(c)

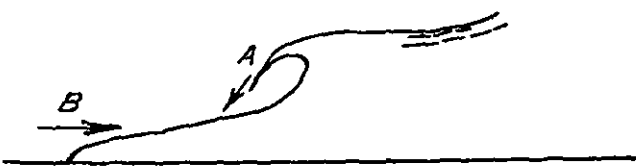


FIG 1(d)

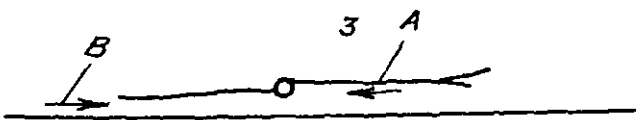
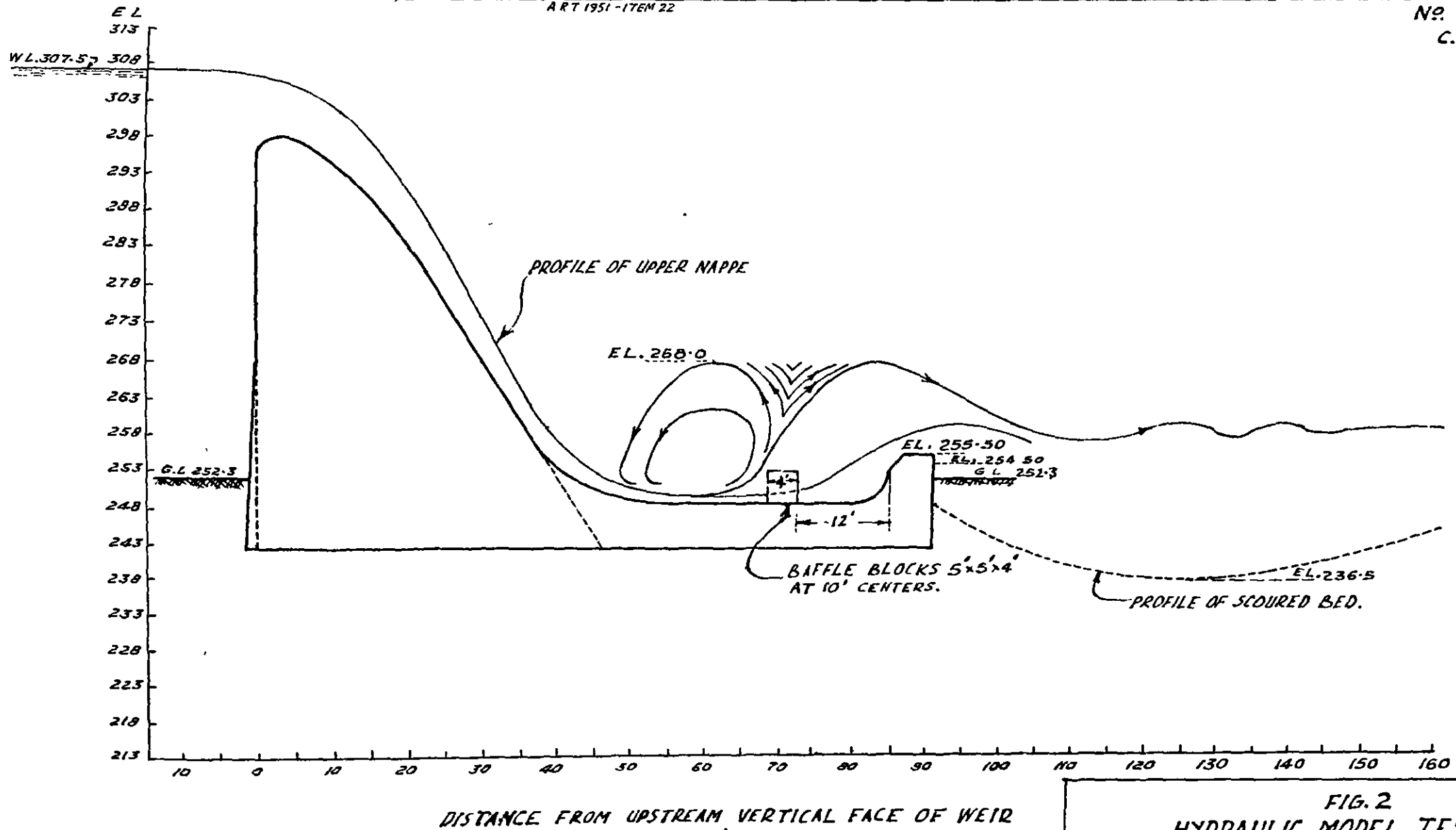


FIG 1(e)

REF.:

- A - INCOMING WAVE.
- B - REFLECTED WAVE.
- C - POINT OF EQUILIBRIUM



COMP. BY *M. M. Mungli*
BY: BHINGARE. 12-2-1952

FIG. 2
HYDRAULIC MODEL TESTS
1/30 SCALE G.S. MODEL OF GAJOD WEIR-KUTCH
TESTS FOR BED SCOUR DOWNSTREAM
SCALE:- 1" = 18.75'

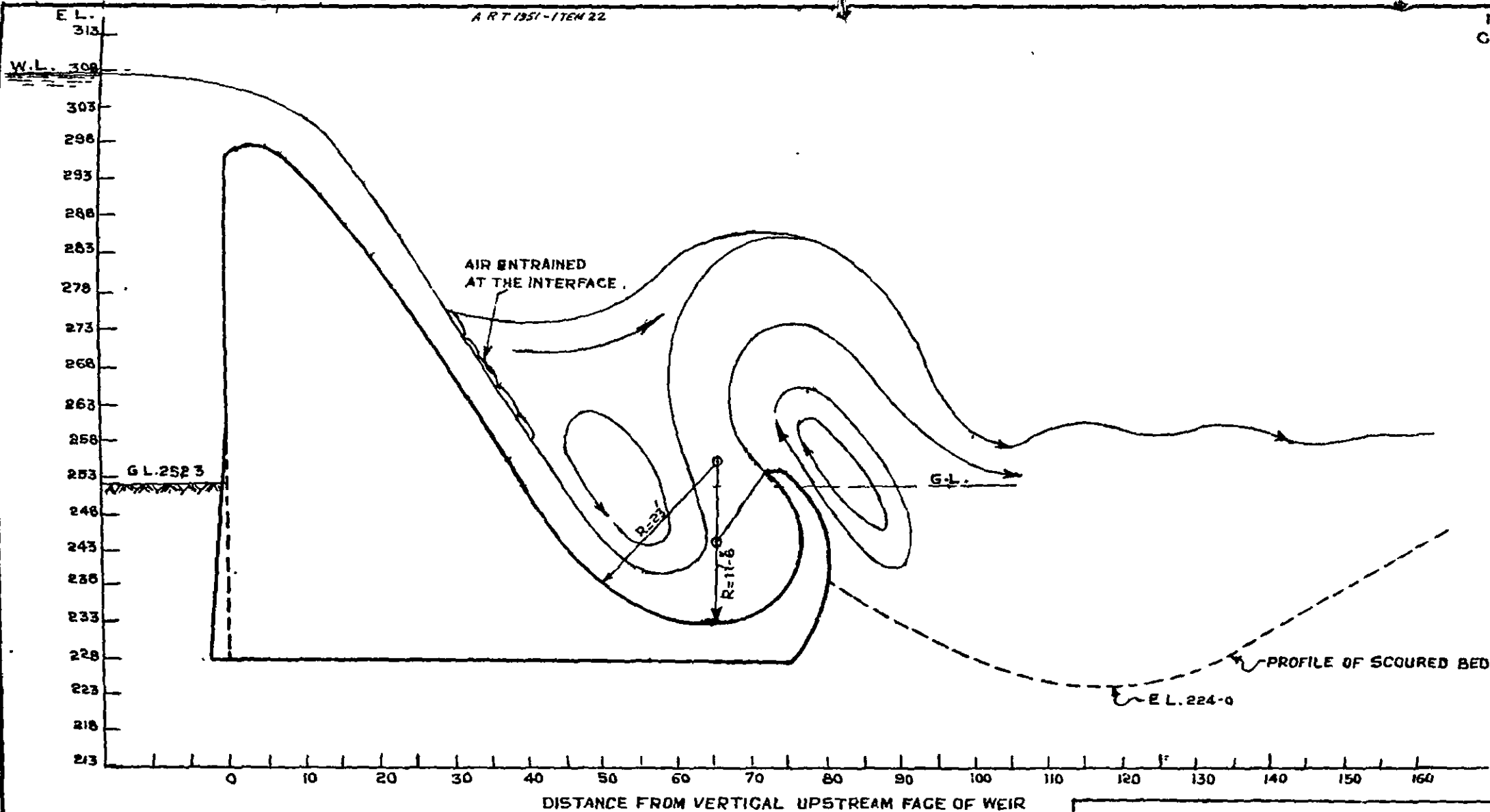


FIG-3
 HYDRAULIC MODEL TESTS
 1/30 SCALE G.S. MODEL OF GAJOD WEIR-KUTCH
 TESTS FOR HYDRAULIC BRAKE DEVICE
 INVENTED BY SIGNOR TAPIAS
 SCALE:- 1"=18'-7.5"

Ch. By: *M. S. ...*
 T By: R. K. Kamath . 14-1-52

Hydraulic Brake Device invented by Signor Tapias
tested in a fall model.

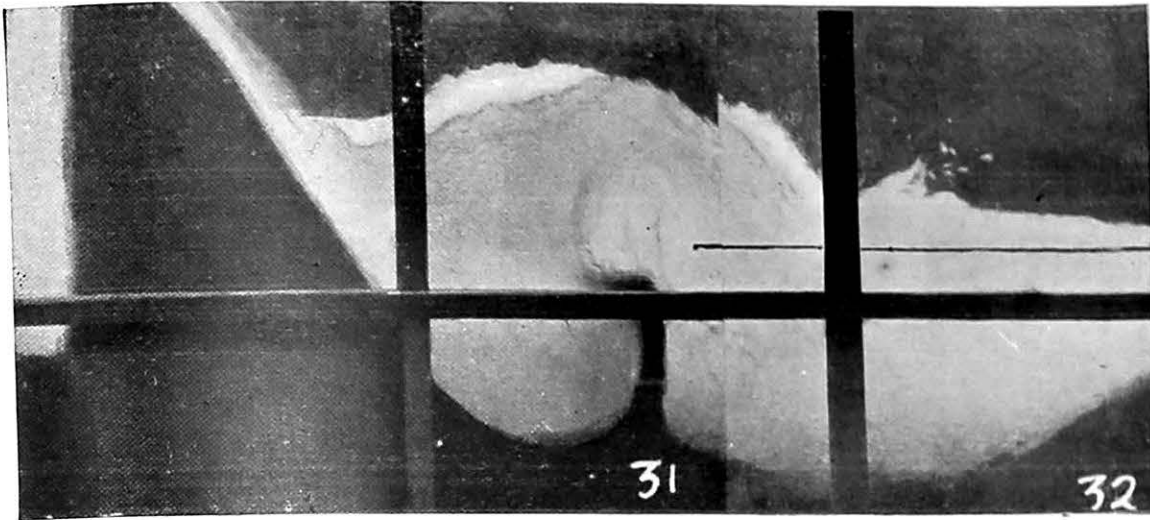


Photo No 31 - 32

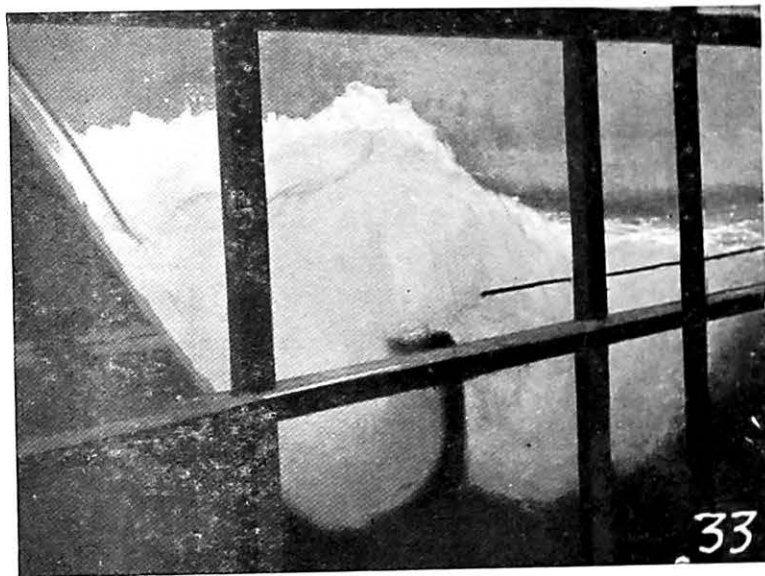


Photo No 33.

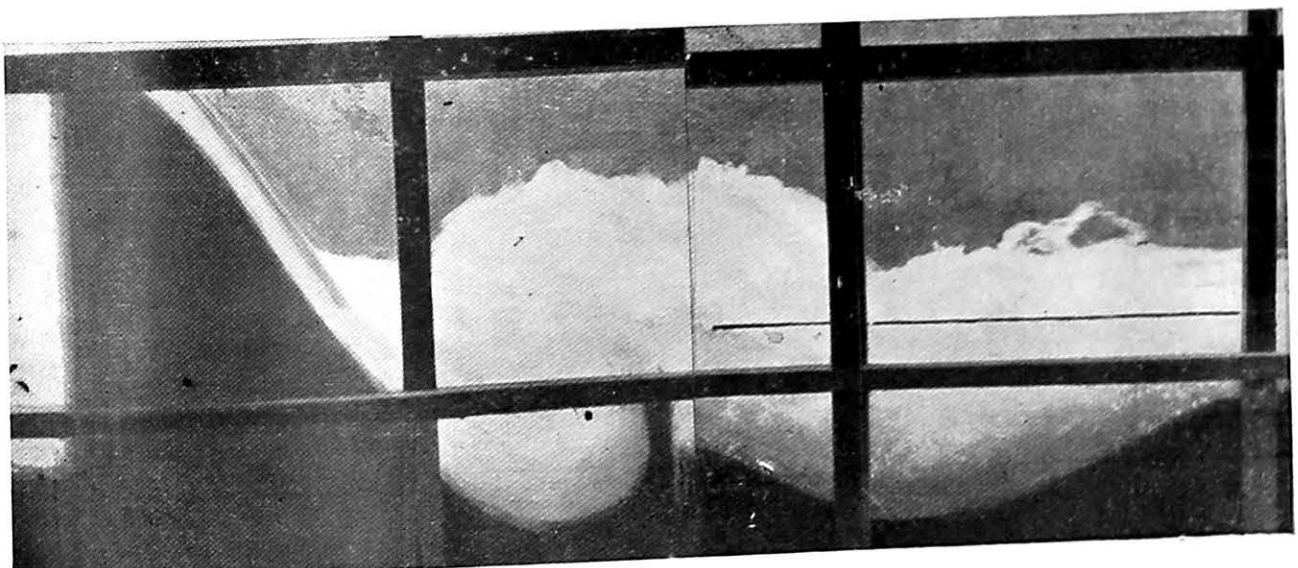


Photo No. 50 - 51.

Test No. 1—The bucket design is shown in Figure 3 and the test was made with 9.5 feet head of water over the crest—Photo 31. The high velocity jet as it entered the bucket followed the bucket profile with continuously decreasing velocity, i.e., with continuously increasing jet thickness and by the time it reached the end tip its distinctiveness was almost lost. What upsurged out of the buckets was a mass of turbulent water with considerable air entrained in it—Photo 33. The air entrainment was clearly seen at the interface of the primary jet and this upsurging mass of water. Figure 3 also shows the profile of scoured bed downstream.

Fig. 3.

Photo 31.

Photo 33.

Fig. 3.

Test No. 2—Figure 4 shows the cylindrical bucket in which the end tip was lowered by 3 feet so that it was level with the downstream ground level—Photos 50 and 52. The general flow pattern was the same as in test No. 1 except that the top level of upsurging water was lower by 5 feet. The scour was slightly less.

Fig. 4.

Photos 50

& 52.

Test No. 3—The bucket was provided with a horizontal platform of 10 feet by shifting the return face further downstream—Photos 65 and 67. The flow pattern was similar to that in the previous tests. The bed scour was less than in Test 1 and nearly similar to the scour in test 2; but still greater than with a baffle and blocks (in Figure 2).

Fig. 5.

Photos 65

& 67.

Tests on the Model of a Canal Fall

The device was tried at the foot of a canal fall model with scale 1:12. The fall carried a discharge $\cong 1,500$ cusecs and had a drop $\cong 10$ feet. Following are the principal dimensions of the design (see Figure 6).

Fig. 6.

$Q=1,500$ cusecs.

$H=10$ feet.

Canal bed width $B_1=B_0=88$ feet (i.e., the fall was not flumed).

Discharge intensity $q=17.05$ cusecs/ft.

Velocity at toe of fall $v_1 = \sqrt{2gH}=25.38$ ft./sec.

Thickness of the jet $d_j = \frac{q}{v_1} = 0.67$ ft.

Radius of the bucket $R=3.36$ ft $=5 d_j$.

The tip of the bucket was kept at the downstream bed level. A preliminary run revealed that the action of the bucket was more or less similar to that of a baffle and that, consequently because of the jet flowing over the top-sill of the bucket the downstream vertical face was exposed considerably. Hence a sloping apron of loose blocks was laid downstream of the wall, with this modification the experiment was commenced.

The jet of water flowing down the glacis began to diffuse as it reached the point A in Figure 6 and it no longer moved along the bucket. In fact the cavity downstream of the centre contained almost stagnant water hence the return face of the bucket acted as a baffle. Photos 788 and 789 show the flow conditions. Wave action was considerable and resulted in sloughing of the sides. Figure 6 and Photo 797 show the scour pattern which depicts a scooping of the bed material downstream of the blocks. Previous experiments on glacis type designs of falls with a cistern length of 18 feet gave a central scour of only 2 ft with no side-erosion.

Fig. 6.

Photos 788

& 789.

Fig. 6 and

Photo 797.

Flow conditions were better when the water level in the downstream channel was lowered. A standing wave clearly formed downstream of the bucket wall. The design was then tested under a drop of 15 ft. by lowering the downstream bed and water levels. The jet continued to retain its individuality till it reached the lowest point in the bucket—point B in Figure 7 and afterwards again diffused instead of moving along the bucket.

The effect of lowering the downstream Water Level was that the Standing Wave was no longer submerged—See Photo 814. But the contemplated action of the bucket failed to obtain and there was no dissipation by means of the upturned jet striking the original jet flowing down the glacis. That the dissipation by means of the bucket was still unsatisfactory can be seen from the scour pattern in Figure 7 and Photo 854. The extent of scour was the same as in Figure 6.

To remedy the bed scour conditions, a pavement was added downstream of the bucket. The depth of the pavement was 11 d₃ below downstream W.L. A deflector of height $\frac{d_3}{10}$ was fixed at the end of pavement; with this alteration, the previous experiment was repeated. Photo 870 shows the flow conditions which are identical with those in Photo 814. Figure 8 and Photo 869 show the scour pattern after 3 hours of flow. The scour was considerably less than shown in Figures 6 and 7. In fact with the addition of the pavement, the design became the equivalent of the usual baffle type design requiring a pavement to contain the secondary S. W. Side action, however, persisted.

The series of experiments described above showed conclusively the necessity of providing a protection downstream of the bucket which renders this design equally costly and more complicated than the usual glacis or baffle type designs. Indeed for a low drop it is unlikely that the jet after flowing over the bucket will have velocity enough to strike back at the original jet and destroy its movement in the collision. The bucket type design was thus not satisfactory in its performance.

Conclusion

The tests carried out were meant to investigate the efficacy of a half cylindrical bucket as a device for dissipation of energy at the toe of a low weir and a canal fall. The results of both the tests have shown conclusively that for low drops the cylindrical bucket does not act in the manner described by the inventor. On the contrary, the high velocity jet in trying to conform to the bucket profile sets the whole mass of water in the bucket in a rotary motion and as it happens in a general eddy phenomenon the rotary motion generates smaller and faster eddies the energy of which is essentially lost in the form of heat. For low falls, as the experiments indicated, the energy was dissipated through this mechanism only and not by the impingement of the returning jet on the primary jet.

8. CANAL HEAD REGULATORS

23. REMODELLING THE WEIR REGULATOR OF THE GHATAPRABHA LEFT BANK CANAL (BOMBAY STATE) TO PASS INCREASED DISCHARGE

1. Introduction

Ghataprabha Left Bank Canal is fed from the Dhupdal Lake in Dharwar District (Bombay State). The Weir Regulator is on the left bank, a little upstream of the weir and consists of 3 openings 4' × 6', of which one is closed with masonry. The Canal Regulator having 5 sluices each 4' × 6', of which 3 have been closed with masonry, is 3,028 ft. downstream of the weir regulator. A set of scouring sluices is also provided adjoining the Canal Regulator but only one of the four openings is working, the rest being closed down as their operation is neither necessary nor desirable as excess discharge would cause damage to the crops and land over which it runs to waste.

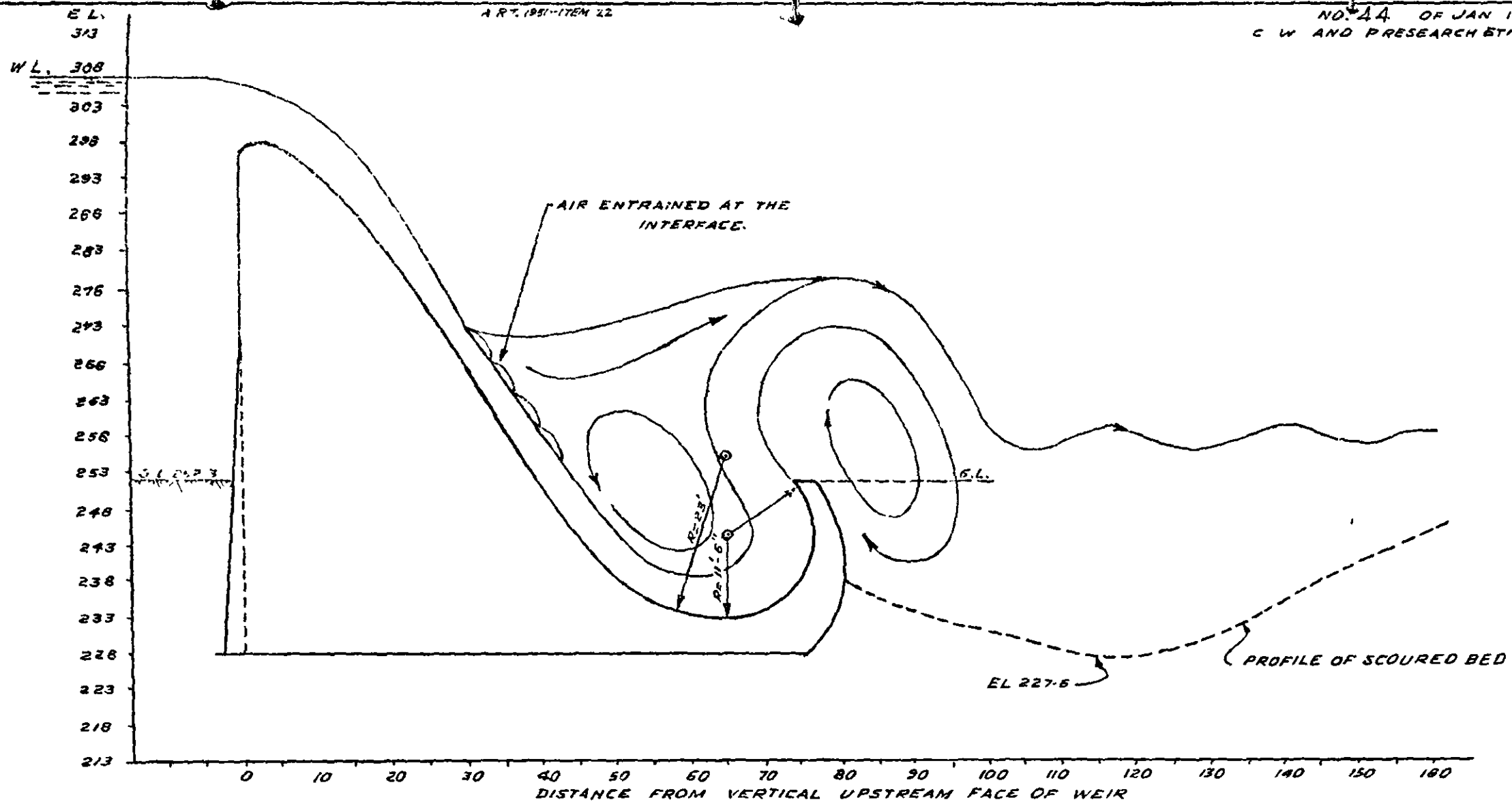


FIG. 4
 HYDRAULIC MODEL TESTS
 1/30 G.S. MODEL OF GAJOD WEIR - KUTCH
 TESTS FOR HYDRAULIC BRAKE DEVICE
 INVENTED BY SIGNOR TAPIJAS,
 END TIP LOWERED TO D/5 G.L
 SCALE. 1" = 18.75'

COMP BY *M...*
 TRCED BY:- PARKHI. 14-1-52

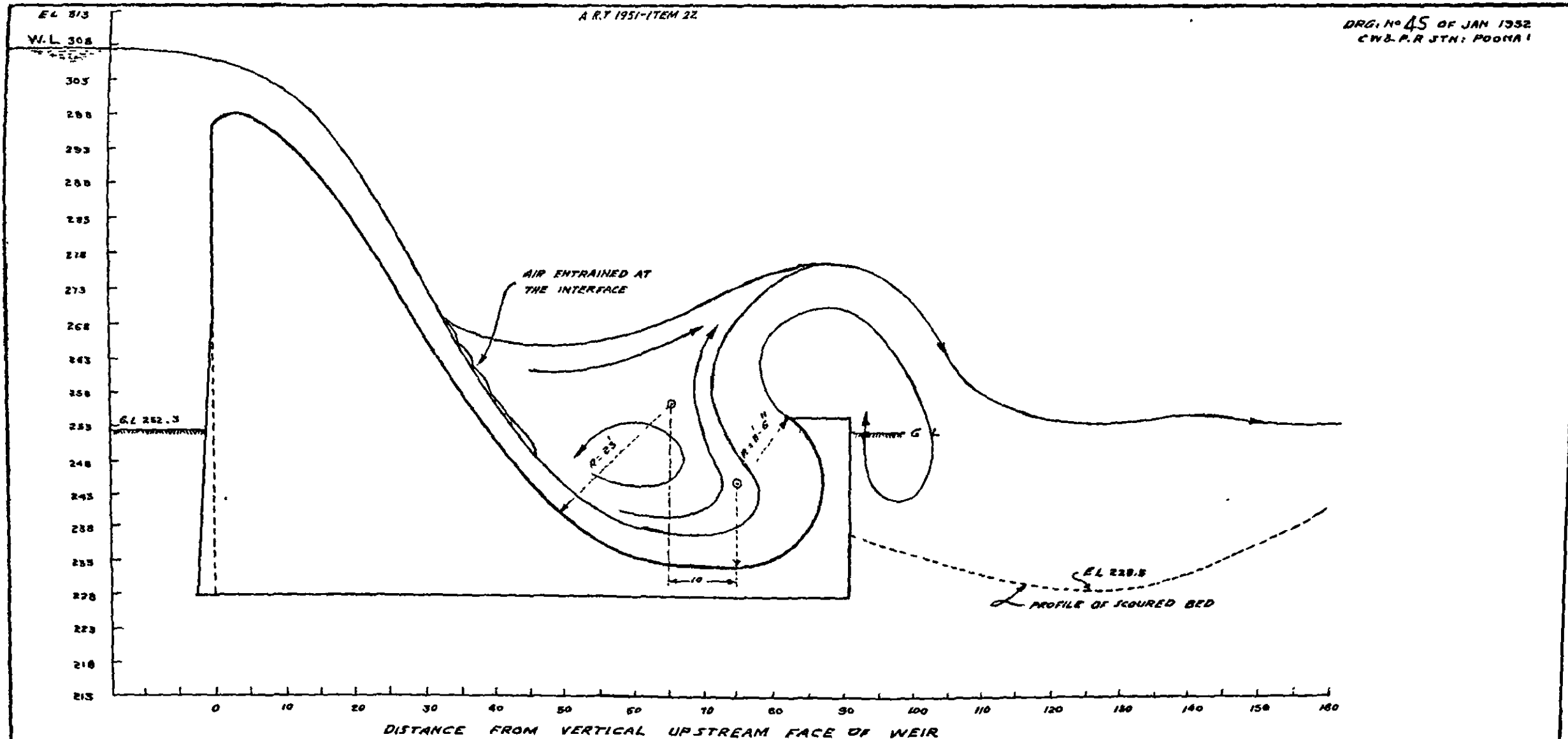


FIG. 5
 HYDRAULIC MODEL TESTS
 1/30 SCALE G.S. MODEL OF GAJOD WEIR
 KUTCH
 TESTS FOR THE HYDRAULIC BRAKE DEVICE
 INVENTED BY SIGNOR TAPIAS
 A HORIZONTAL PLATFORM 10 FT.
 LONG ADDED.
 SCALE: 1"=18.75'

COMP. BY: *M. S. Manjhi*
 TRACED BY: MANCHUNDIA.

Hydraulic Brake Device invented by Signor Tapias, Tested in a fall model.

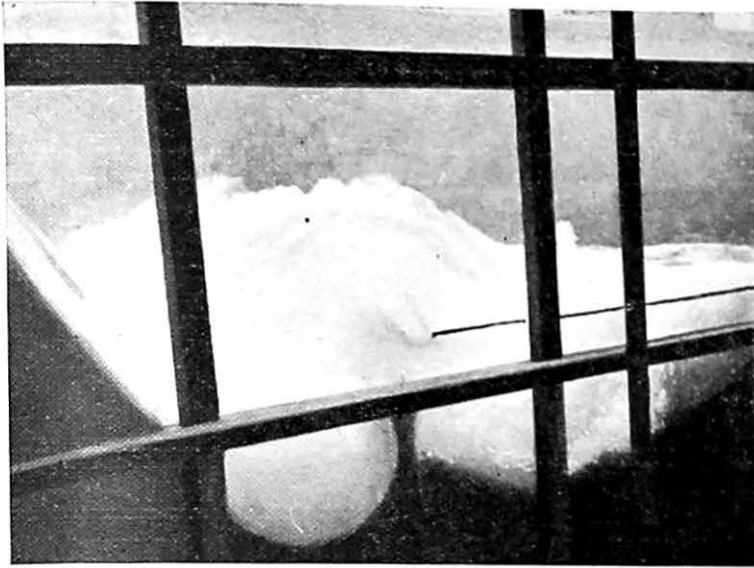


Photo No 52

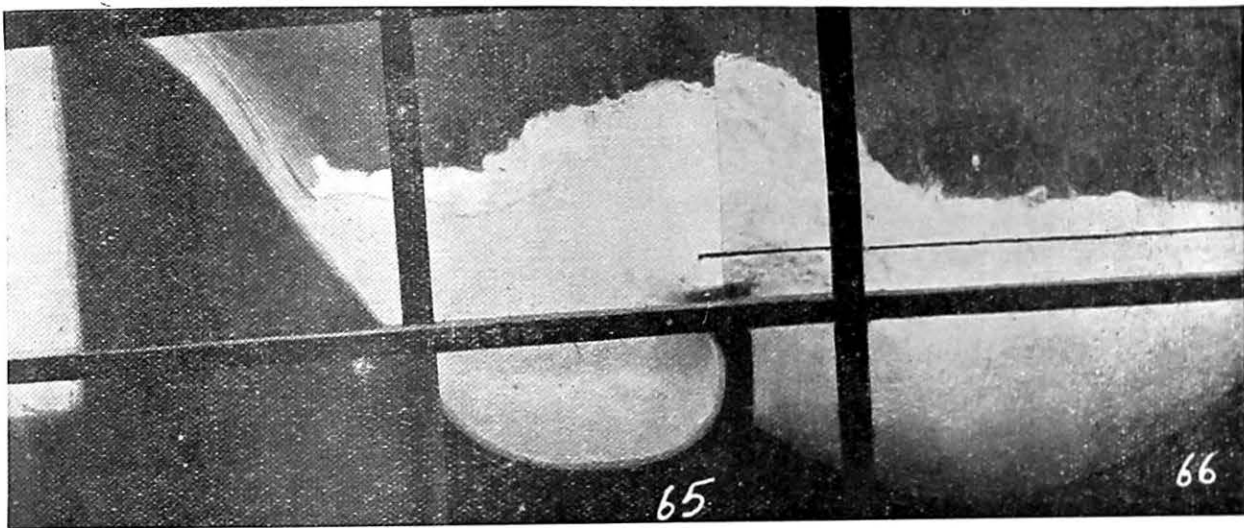


Photo Nos. 65 - 66

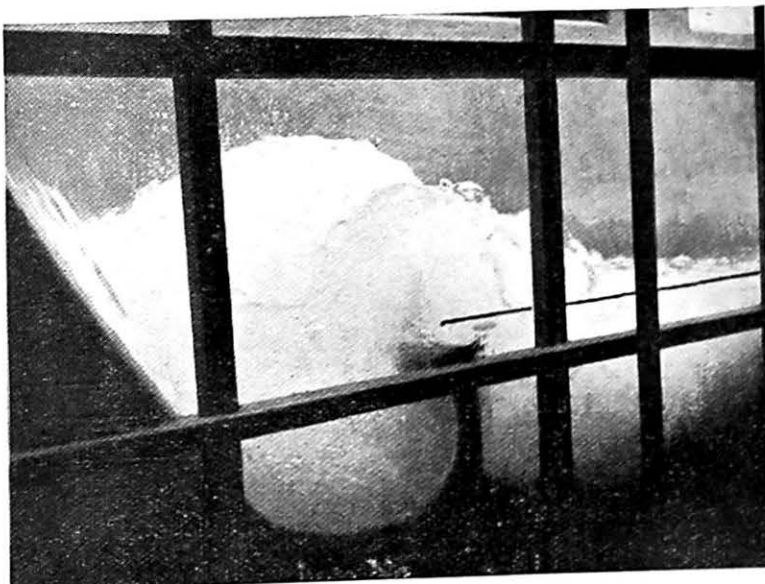
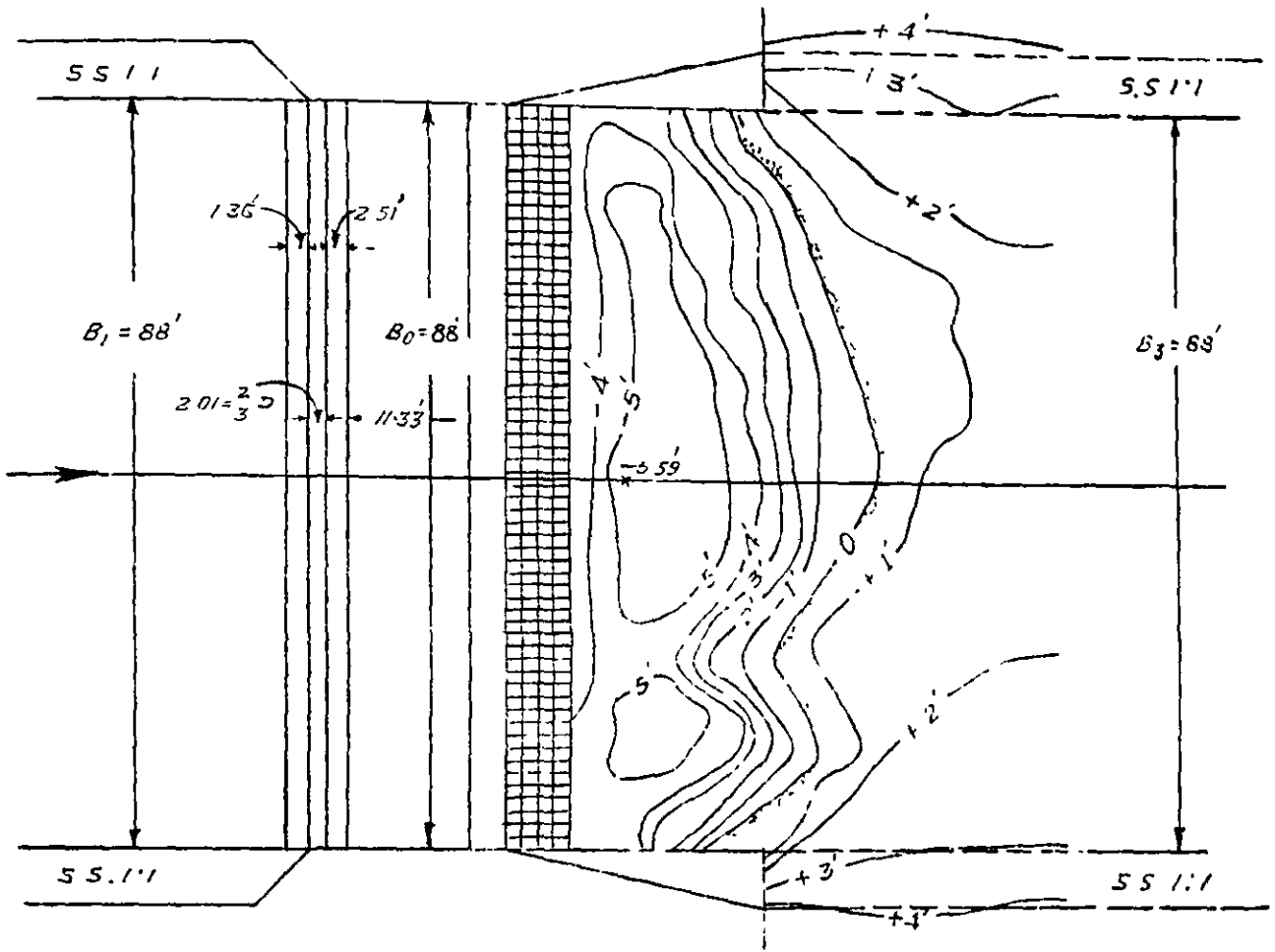
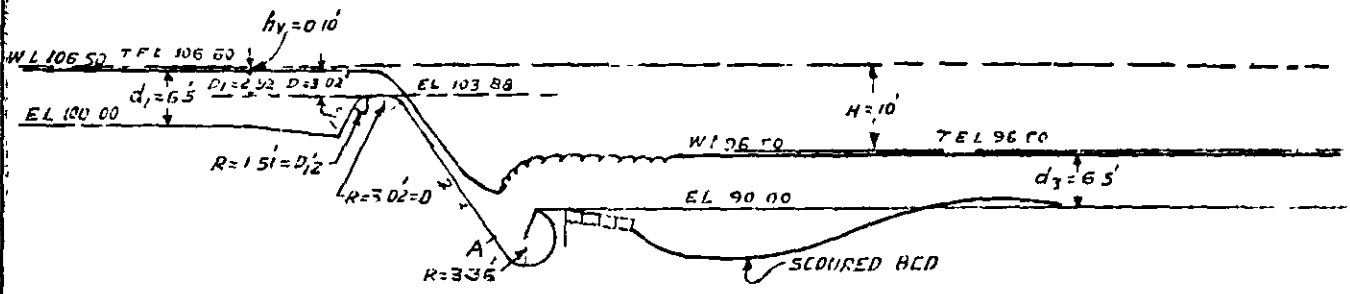


Photo No 67.



PLAN



L. SECTION

FIG. 6
 BUCKET FALL MODEL EXPERIMENT
 SCOUR PATTERN AFTER 3 HOURS OF FLOW
 $Q = 1500$ CUSECS, $H = 15'$
 SCALE $1'' = 1'$ (MODEL)
 $1'' = 24'$ (PROTO)
 DATUM OF CONTOURS WAS C.B.L. D/S
 BLOCK APRON LAID TO A SLOPE OF 1 IN 10 D/S OF BUCKET
 (BUCKET INVENTED BY SIGNOR TAPIAS)

DESIGNED BY *M. S. P.*
 TRACED BY B.G. DESHPANDE 14-1-52

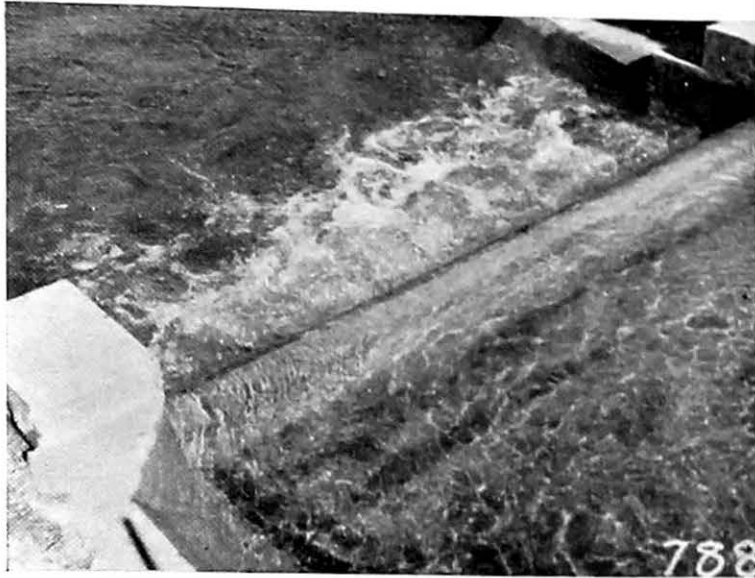


Photo No 788

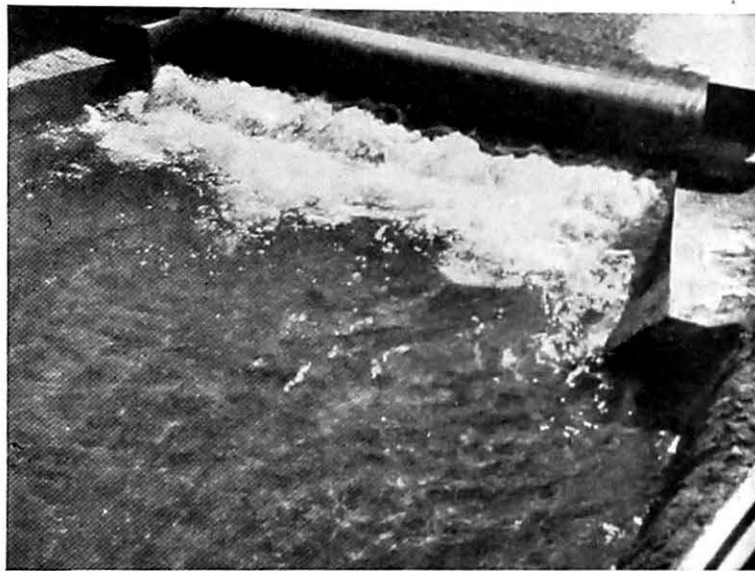
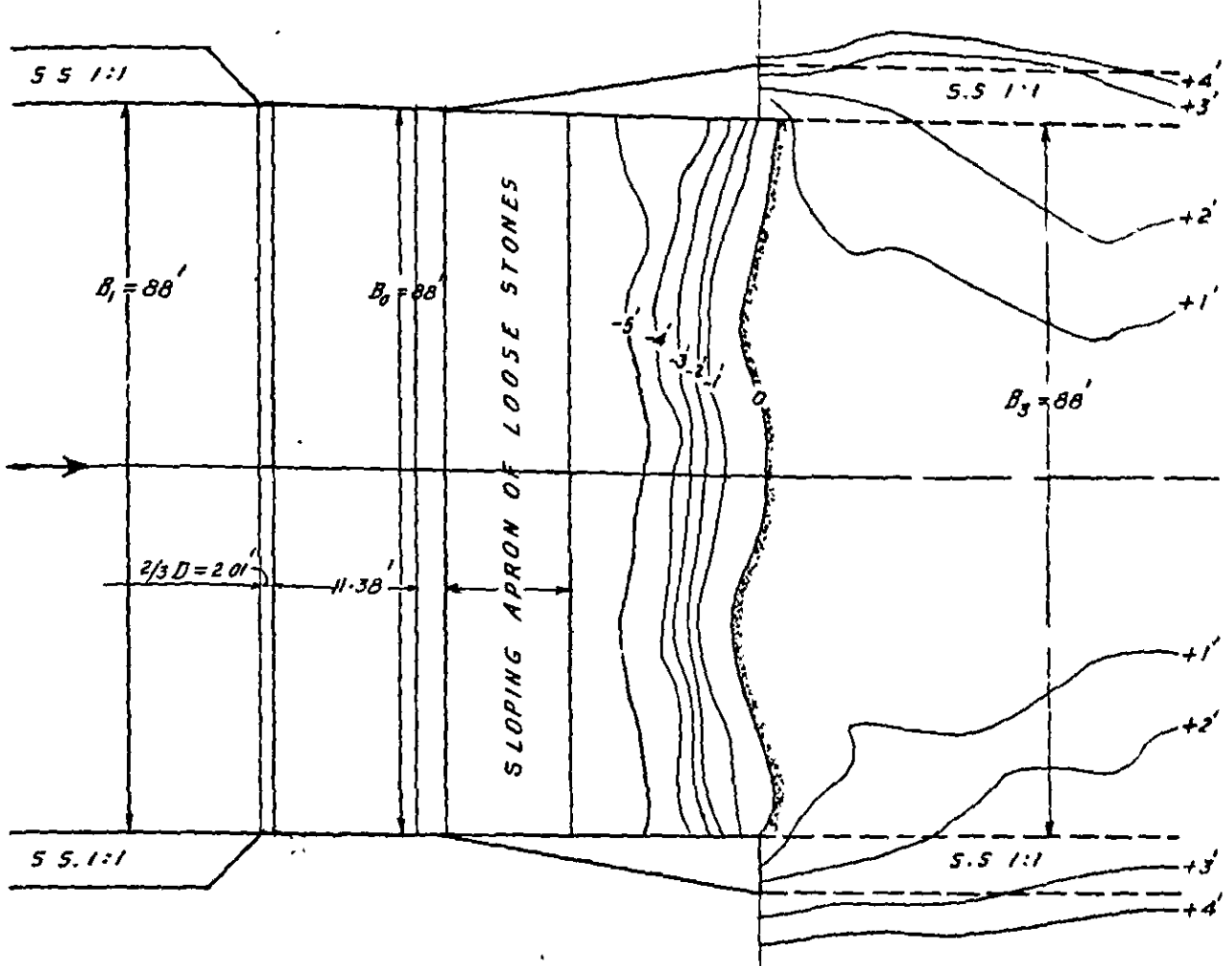


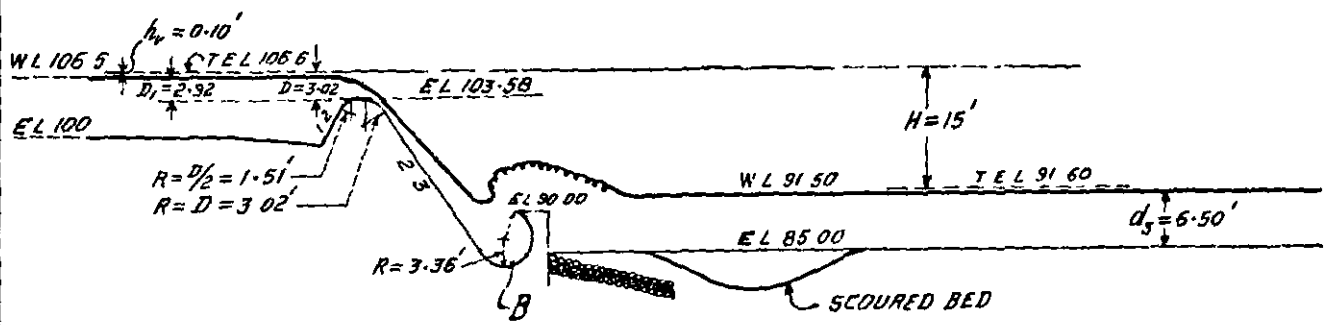
Photo No. 789



Photo No. 797



P L A N



L. SECTION

FIG. 7
 BUCKET FALL MODEL EXPERIMENT
 SCOUR PATTERN AFTER 3 HOURS OF FLOW
 $Q = 1500 C_s$, $H = 15'$
 SCALE, $1'' = 1'$ (MODEL)
 $= 24'$ (PROTO)
 DATUM OF CONTOURS WAS C.B.L D/S.
 LOOSE STONE PITCHING D/S OF BUCKET.
 (BUCKET INVENTED BY SIGNOR TAPIAS)

COMP BY: *[Signature]*
 TRACED BY: NAIK 14-1-51

ITEM No. 22.
A. R. T, 1951

Hydraulic Brake Device invented by Signor Tapias tested in Fall model.

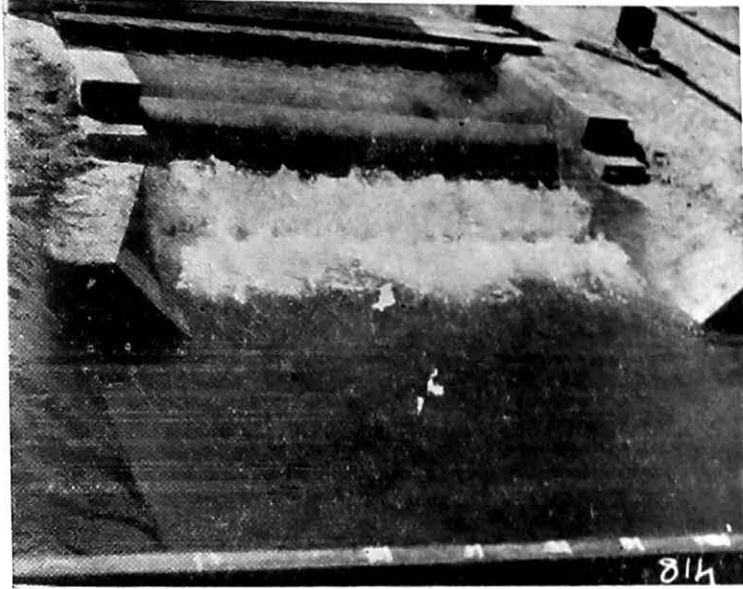


Photo No 814

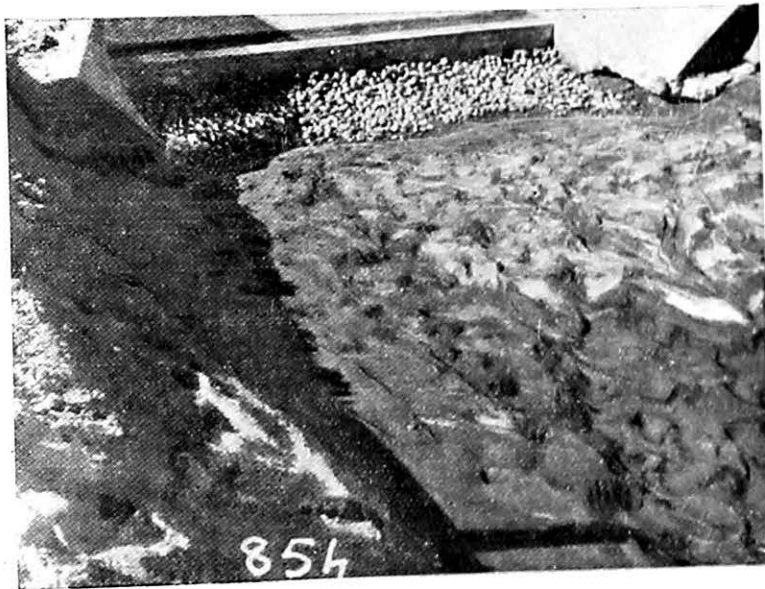
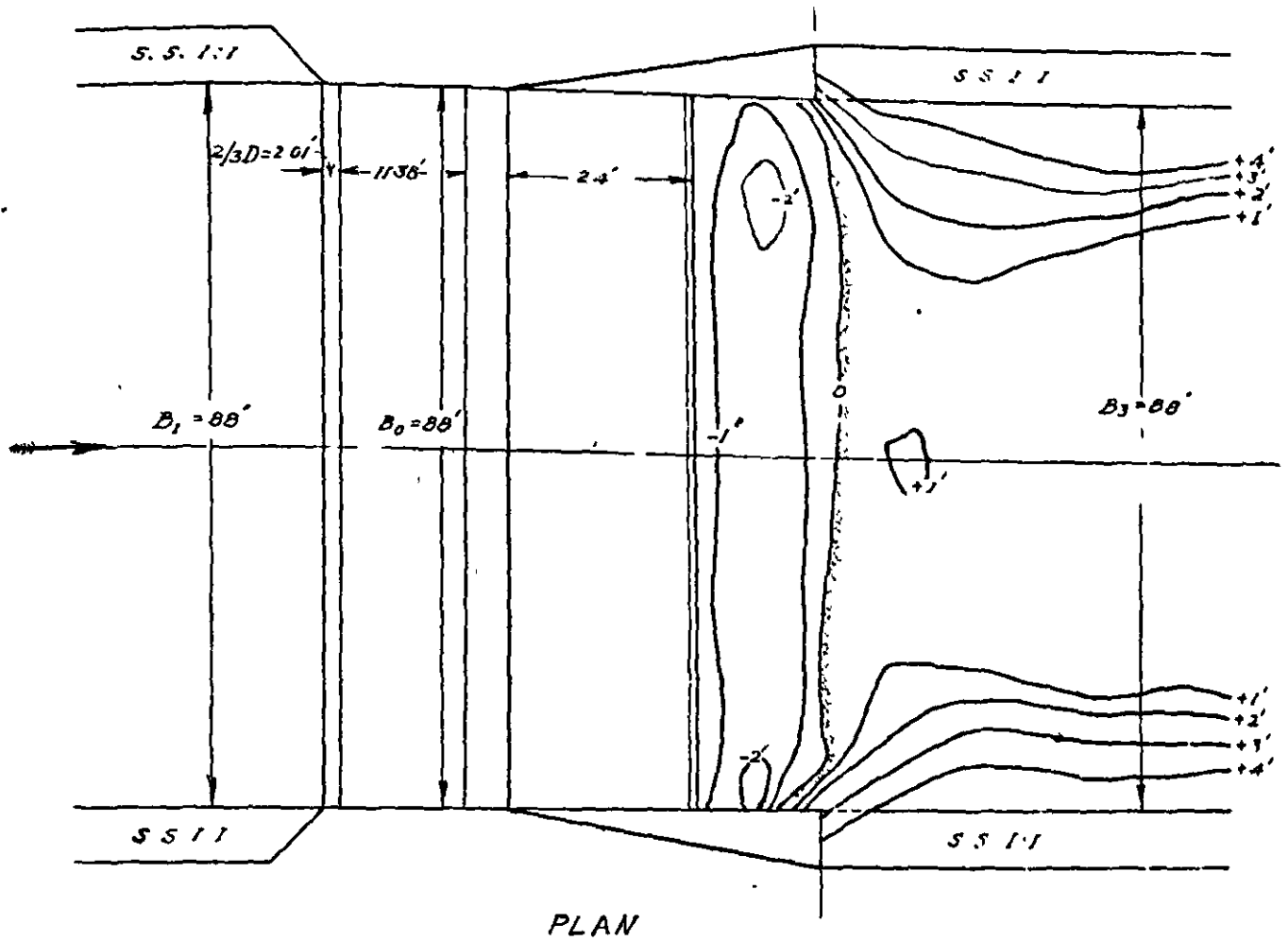
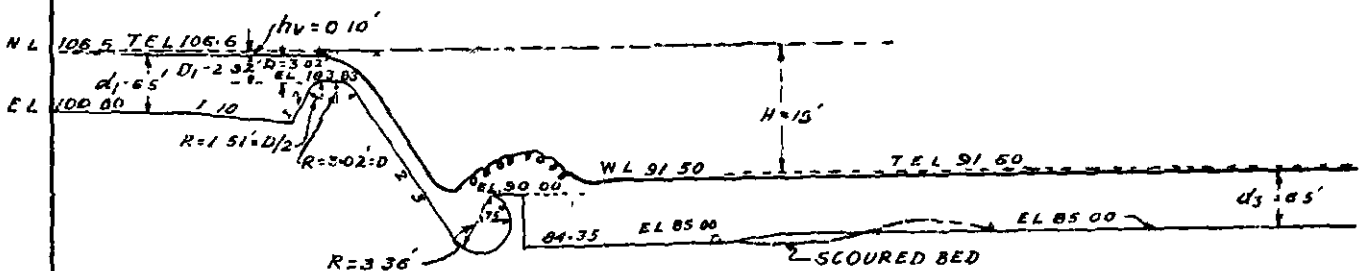


Photo No 854



PLAN



L. SECTION

FIG 8
BUCKET FALL MODEL EXPERIMENT
SCOUR PATTERN AFTER 3 HOURS OF FLOW
Q = 1500 CUSECS, H = 15'
 SCALE 1" = 1' (MODEL)
 1" = 24' (PROTO)
 DATUM OF CONTOURS WAS C.B.L. D/S
 A PAVEMENT OF LENGTH 24' WAS ADDED
 (BUCKET INVENTED BY SIGNOR TAPIAS)

COMP BY *M. Singh*
 TRACED BY R. T. THARUR 15-1-51

Hydraulic Brake Device invented by Signor Tapias Tested in a
Fall model.



Photo No. 869

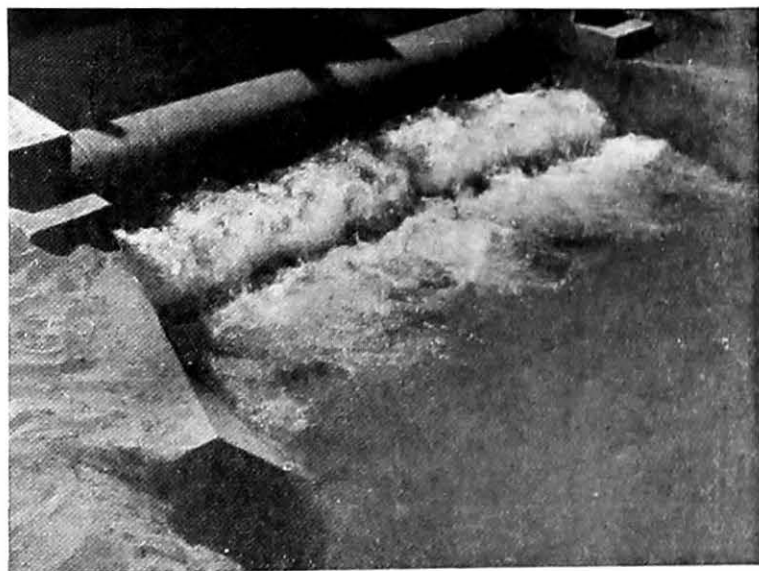


Photo No. 870

It has now been proposed to put into operation all the sluices including those closed at present at both the regulators with a view to pass a maximum discharge of 2,000 cusecs, which the canal would carry after the construction of the storage dam.

The problem of increasing the discharging capacity of the regulators to pass 2,000 cusecs was referred to the Research Station by the Superintending Engineer, Karnatak Irrigation Circle, Dharwar (Bombay State). The existing number of regulators will not be able to pass this discharge unless the number of sluices or the area of sluices or both, are increased. As this will be fairly expensive, the following alternative to increase the discharging capacity of the existing sluices was considered:—

- (1) Remodelling the existing sluices by adding bellmouth entrances and flaring divergences to improve their coefficient of discharge.

In the Ghataprabha L. B. Canal Development Project a provision of Rs. 10,800 was provided for constructing one extra masonry sluice for the Weir Regulator to pass 1,500 cusecs; it is however considered inadvisable to dismantle the masonry near the Weir Regulator to provide the extra sluice or sluices and instead, the above alternative is considered to be economical. Experiments were therefore undertaken with a geometrically-similar model of the Weir Regulator, at the Research Station.

2. Notations

Q —Total discharge in cusecs in the Prototype.

Q_m —Discharge in the model.

q —Discharge through a single sluice.

D_1 —Depth of water upstream above sill of the sluice.

D_3 —Depth of water downstream above sill of the sluice.

A —Area of waterway at upstream end of the sluices
= $4 \times 6 \times 3$ spans = 72 sq ft.

H —Difference of water levels upstream and downstream of the sluices, under submerged conditions.

X —Afflux in feet.

X_1 —Afflux permissible.

C_H —Coefficient of discharge in the formula:

$$\begin{aligned} Q &= C_H \cdot A \cdot \sqrt{2gH} \\ &= C_H \times 72 \times 8 \sqrt{H} \\ &= 576 C_H \sqrt{H} \end{aligned}$$

3. Description of the Model

The Weir Regulator consists of 3 sluices each sluice being 4 ft. wide, with gates $4' \times 6'$. The sluices are 16 ft. 11.75 inches long and have arched tops; the rise of the arch being 1 ft. and the gate frames block the arch sector when the gates are open.

A half width G S Model of the Weir Regulator was constructed to a scale of 1/10, consisting of $1\frac{1}{2}$ sluices (Sluice No. 3 full and No. 2 half)—see Figure 1(a), Fig. 1(a). incorporating a length of 75 ft. of the channel upstream and 220-ft. long channel downstream of the sluices reproducing the cross-sections and bed configurations. The data shown in Figures 1(b), (c) and (d) is that of the existing conditions. The channel cross-section No. 1 at 10 ft. on upstream side of the gates which is shown in Figure 1(a) indicates that the bed is at an elevation much higher than the upstream sill of the sluices and as such covers the lower half openings of the three

Photo 807. sluices. Photo 807 shows the cross-sections, gates and piers upstream as reproduced in the model. The weir top is at elevation R.L. 2008 0 which is the F.S.L. for the Regulator. The sill of the upstream end of the sluices is at R.L. 1986.5 while that of the downstream end is at R.L. 1986 25 with a bed slope of 3 inches.

Full supply discharge desired in the Prototype=2,000 cusecs or for $1\frac{1}{2}$ spans=1,000 cusecs.

$$\text{The equivalent discharge in the model} = \frac{1000}{10 \times 10 \times \sqrt{10}} = 3162 \text{ cusecs.}$$

For measuring water depths in the model, two gauges were fixed at a distance of 1.5 ft. upstream of the gate and about 8 ft. downstream of the gate—where natural flow first re-established. The masonry of the sluices in the prototype is of coursed rubble (II Sort). In the model, this was represented by rough cast cement plaster to obtain approximate relative roughness. The downstream channel is in rock-cutting with almost vertical sides. The corresponding roughness was reproduced in the model by fixing pebbles of $\frac{1}{4}$ to $\frac{1}{2}$ inch diameter on the banks as shown in Photo 799.

4. Experiments and Consideration of Results

The salient features of the Weir Regulator are given below—

Q max.	2,000 cusecs.
Elevation of weir top	R.L. 2008 0
U/S sill of the sluices	R.L. 1986.5
D/S sill of the sluices	R.L. 1986 25

The value of the discharge coefficient as assumed in the original project was 0.79 and taking the depth in the channel downstream of the Weir Regulator as 130 ft. and H as 8.75 ft., the discharge through the three sluices worked out to 1,346 cusecs. The information regarding the present discharging capacity of the 3 sluices when fully simultaneously opened could not be obtained as one of the gates is completely closed and it is also not possible to let down the maximum discharge which can be passed through them until the construction of the remodelled canal, is completed. An experiment was therefore carried out to determine the discharging capacity of the 3 sluices under existing conditions. The coefficient was found to be 0.64 as against 0.79 (assumed) and a discharge of only 778 cusecs could be passed through the three sluices when fully opened.

Thus the Weir Regulator will have to be remodelled to pass nearly 2.6 times the discharge (i.e., 2,000 cusecs against 778 cusecs). Also it would be necessary to remodel the Canal Regulator (which is situated at the end of a 3,028 ft. long approach channel which is completely in rock-cutting) as the coefficient of discharge for Canal Regulator, as well, was assumed to be 0.79 in the original project. With considerations to the economic feasibility of the problem various alternatives of lining or widening the approach channel were worked out and the following three proposals were considered:

	Proposal No. 1.	Proposal No. 2.	Proposal No. 3.
(1) Weir Regulator	Value of C_H to be achieved by remodelling=1.7 (at least).	C_H at least 1.2	C_H at least 1.08.
(2) Approach Channel	30 ft. unlined (i.e. existing) $n=0.04$. Width 17.3 ft. Depth.	41 ft. unlined $n=0.04$ 13.0 ft.	28 ft. lined. $n=0.02$. 11.2 ft.
(3) Canal Regulator	C_H at least = 0.75, therefore need not be remodelled.	C_H at least = 1.2	C_H at least = 1.65.
(4) Lake Level for F.S.Q. of 2,000 cusecs.	R. L. 2008 00 with Weir Regulator Coefficient of 1.7 & Canal Regulator Coefficient of 0.75.	R. L. 2004 90 with Weir Regulator Coefficient of 1.5 (or more) & Canal Regulator Coefficient of 1.2 (or more).	R. L. 2003-10 with Weir Regulator Coefficient of 1.5 (or more) & Canal Regulator Coefficient of 1.65 (or more).

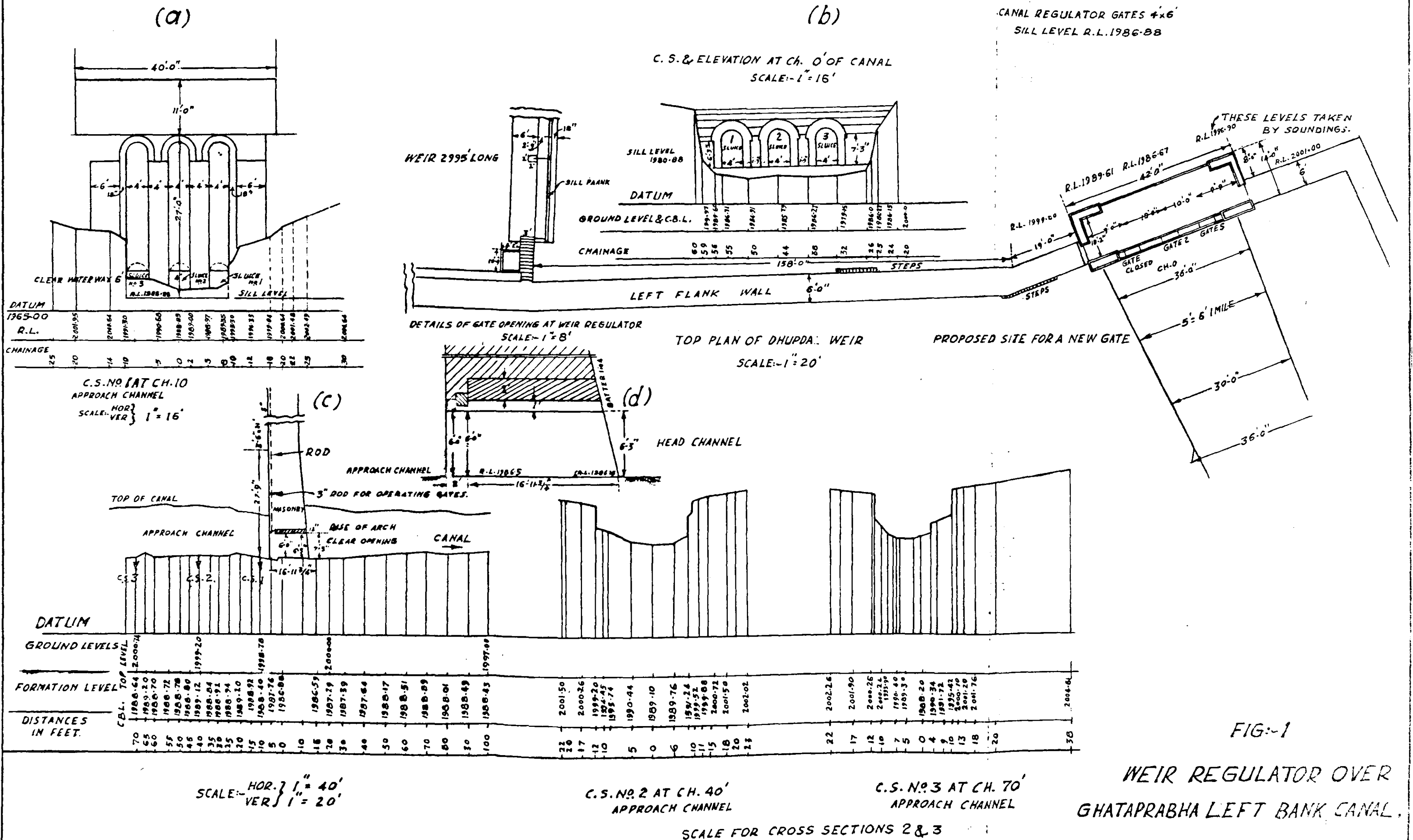


FIG:-1

WEIR REGULATOR OVER
GHATAPRABHA LEFT BANK CANAL.

COMP. BY: *Manjiv*
JBY-MANCHUNDIA-21-2-52.

COPIED FROM THE DRAWING
RECEIVED FROM THE EX. ENGINEER

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

Remodelling the Weir Regulator of the Ghataprabha Left Bank Canal

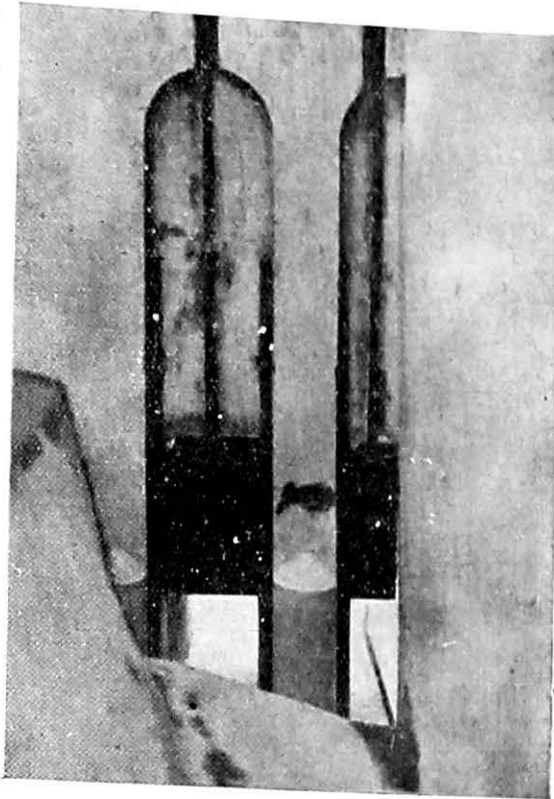


Photo 807

Showing bed cross - sections in the channel upstream of the Weir Regulator, gates, piers and openings of $1\frac{1}{2}$ sluices in the model, looking downstream.

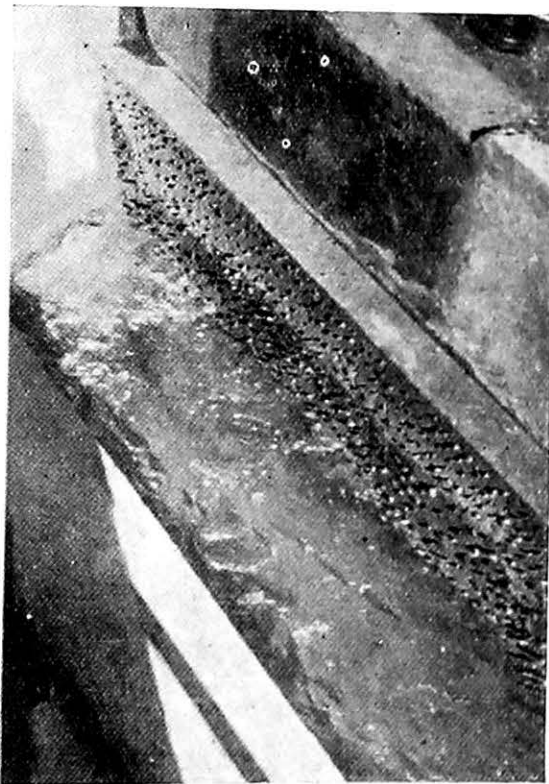


Photo 799

Showing side roughness reproduced in the the channel downstream of the Weir Regulator by pebbles in the model, looking downstream.

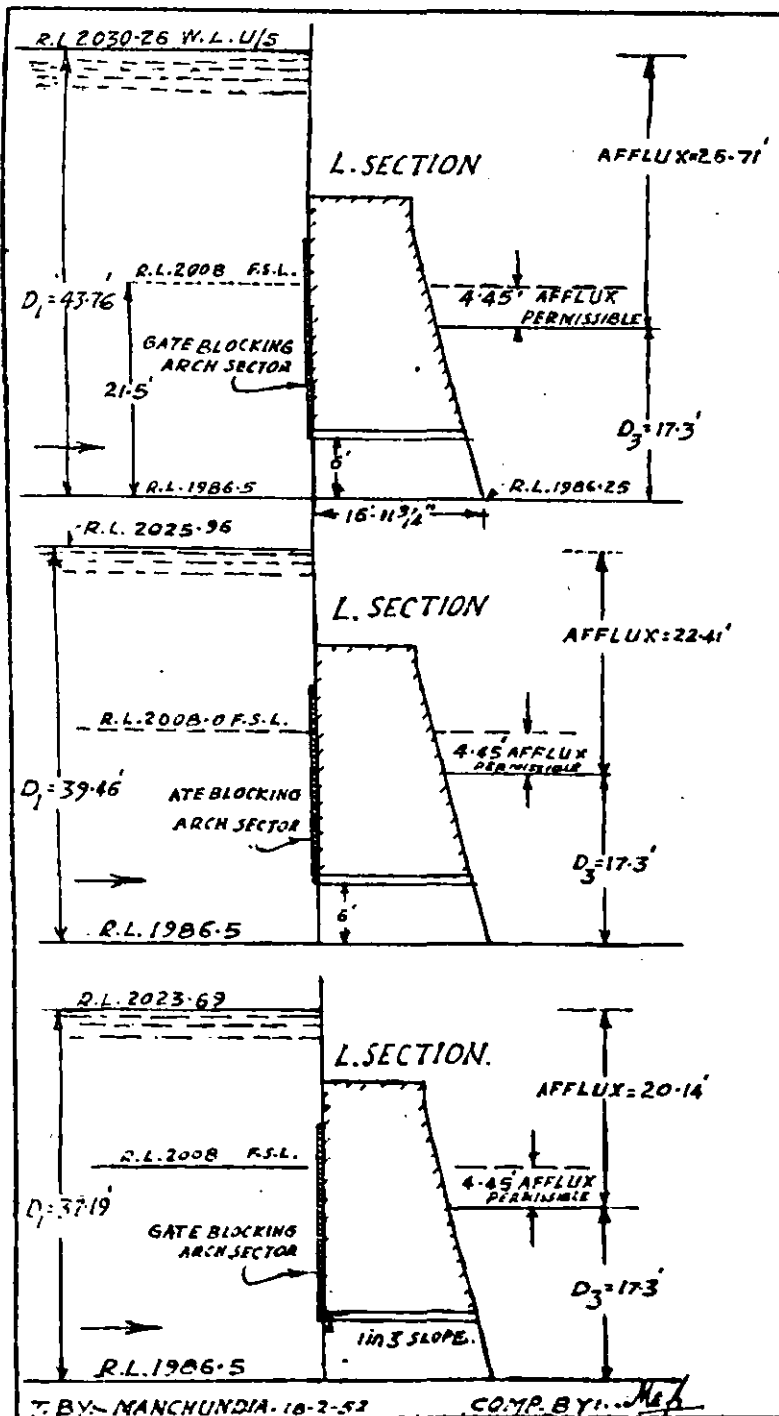


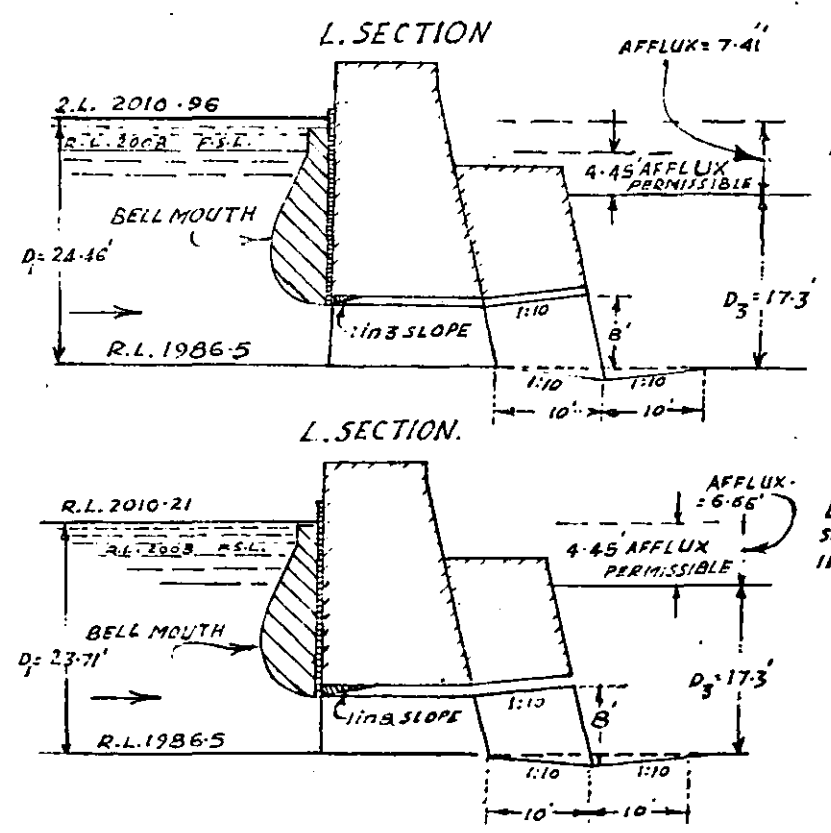
FIG. 2
SERIES A
EXPERIMENT 1
EXISTING CONDITION.

FIG. 3
EXPERIMENT 2.
WITH 1 in 5 SLOPE IN BED U/S.

FIG. 4
EXPERIMENT 3
WITH 1 in 10 SLOPES IN BED U/S
AND 1 in 3 SLOPE IN ARCH SECTOR.

Q = 2000 CUSECS.

A.R.T. 1951-ITEM 23



NO. 185 OF FEB 1952
C.W. & P.R.S. POONA

FIG. 5
EXPERIMENT 4
WITH HORIZONTAL BED U/S
BELL MOUTH U/S & 1:10 EXPANSION
U/S 1 in 3 SLOPE IN ARCH SECTOR.

FIG. 6
EXPERIMENT 5
SAME AS EXPT. 4 BUT WITH 1 in 8 SLOPE
IN ARCH SECTOR.

FIG. 2 TO 6
EXPERIMENTS WITH MODEL OF WEIR HEAD REGULATOR
OVER
GHATAPRABHA LEFT BANK CANAL
PROPOSAL I
SHOWING AFFLUX OBSERVATIONS
SCALE: 1" = 20'

FIG. 7
SERIES B₁
EXPERIMENT. 1

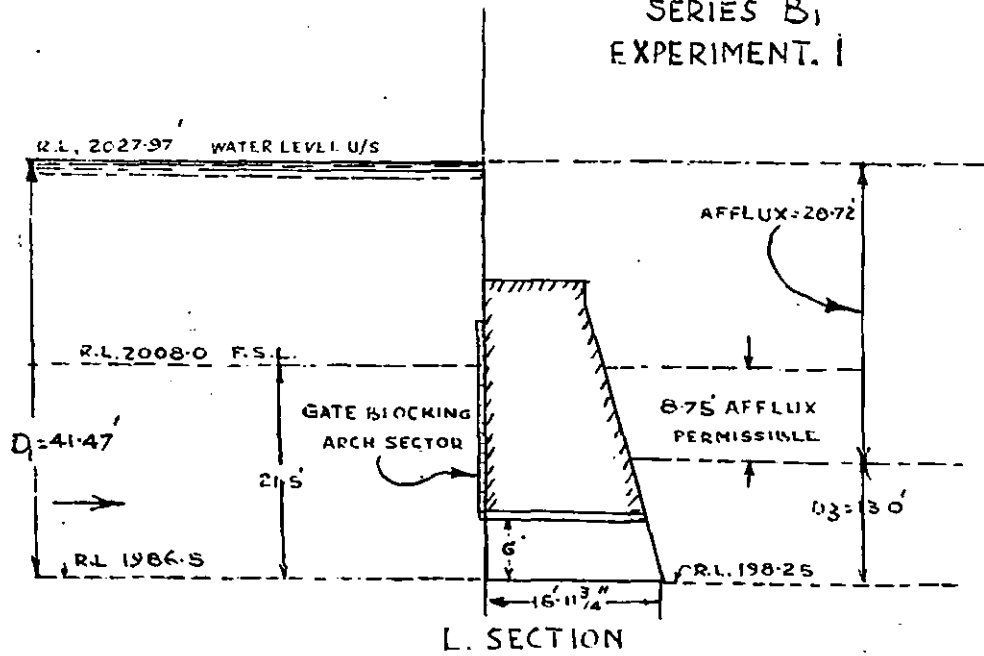


FIG. 9
EXPERIMENT. 3

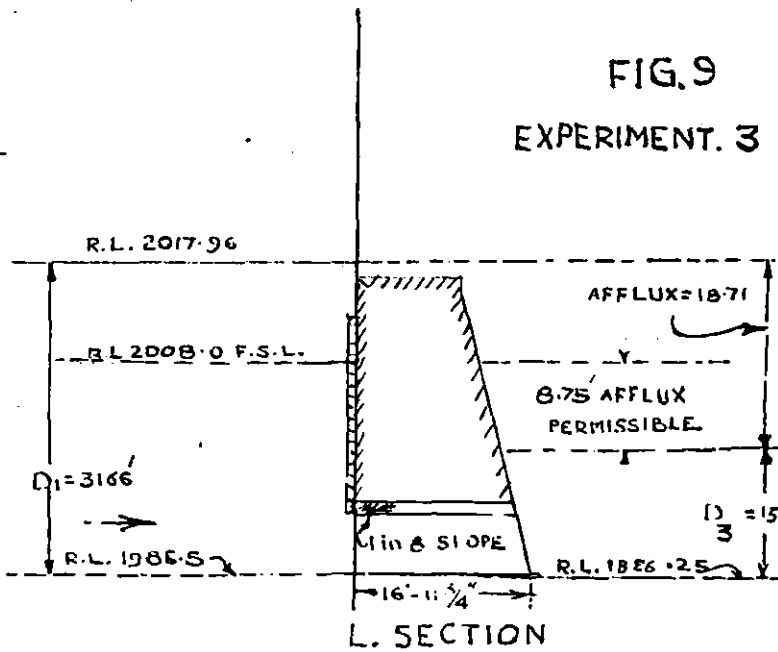


FIG. 11
EXPERIMENT. 5

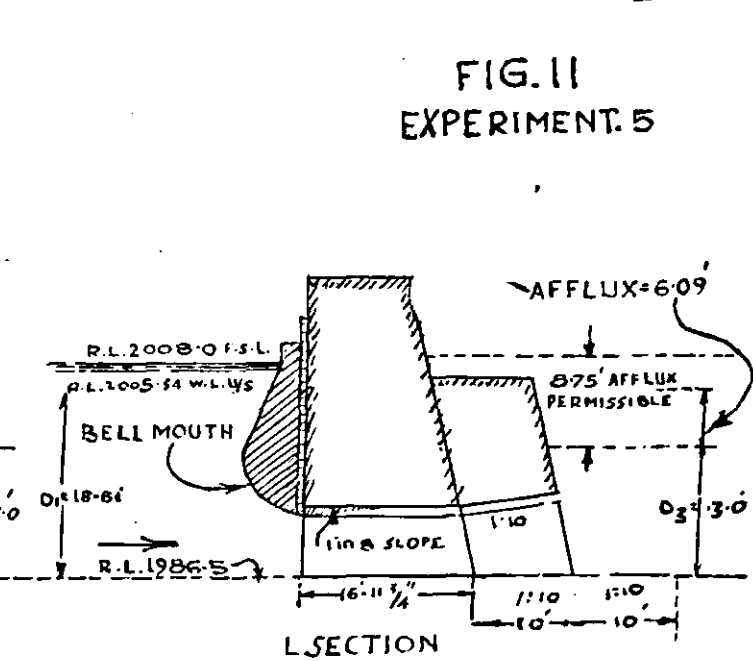


FIG 8
EXPERIMENT. 2

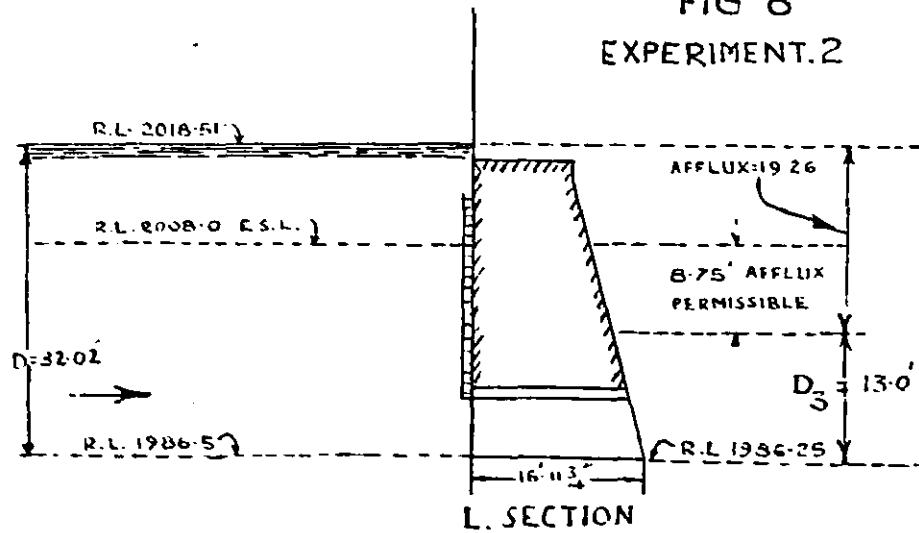


FIG. 10
EXPERIMENT 4

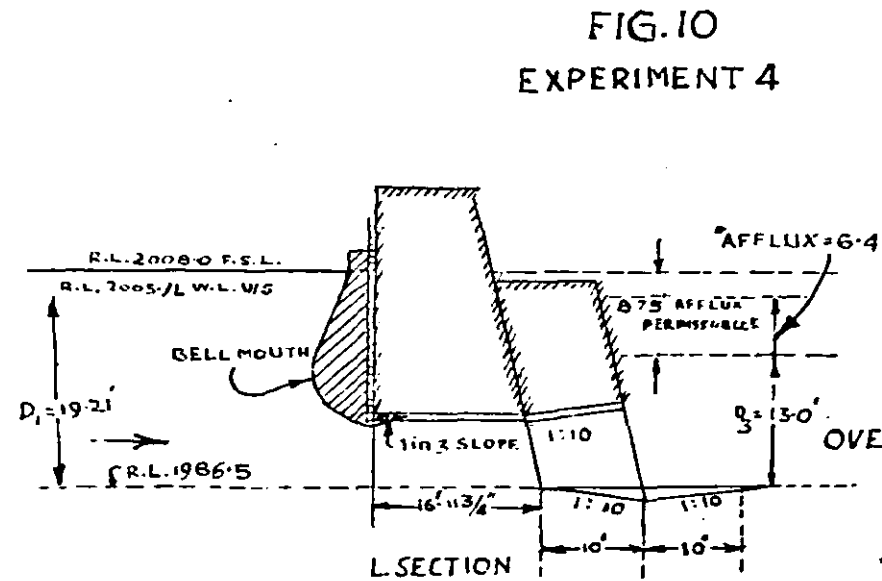


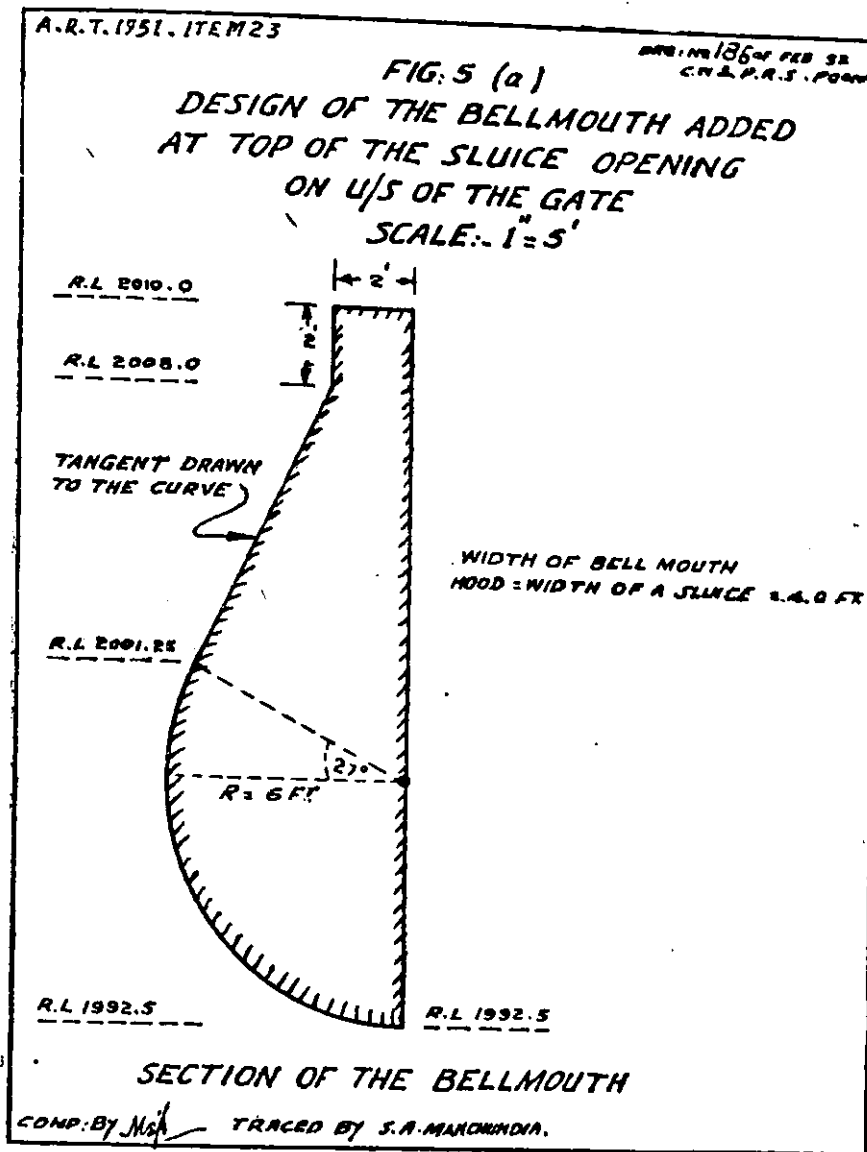
FIG. 7 TO 11
EXPERIMENTS WITH MODEL
OF WEIR HEAD REGULATOR
OVER GHATAPRABHA LEFT BANK CANAL

PROPOSAL II
SHOWING AFFLUX OBSERVATIONS
SCALE: -1"=20'

Q=2000 CUSECS.

CONP. BY: *[Signature]*

The experimental study with a model of the Canal Regulator will be carried out in a three-dimensional, geometrically-similar model separately.



A large series of experiments was conducted by providing bellmouth upstream and expansions downstream of the sluices in the Weir Regulator Model of 1/10 scale and the results are shown in Tables 1 to 3 and Figures 2 to 11. Figure 5(a) shows the design of the bellmouth tested. Sets of observations of coefficient of discharge and afflux for each of the modifications were recorded and are given in Tables 1 and 2, for the following three depths of water in the channel downstream of the Weir Regulator in accordance with the above three alternatives.

Tables 1 to 3. Figs. 2 to 11.

Series No.	Depth in Channel D/S of the Weir Regulator (i.e. depth in the Approach Channel)
I	17.3 ft.
II	13.0 "
III	11.2 "

A set of observations for a discharge of 1,500 cusecs \equiv 2,372 cusecs (in model) was also recorded as 1,500 cusecs is the immediate requirement for the Left Bank Canal which will develop to 2,000 cusecs in the final stages of the Project. The results, of the experiments, showing different values of afflux for the various modifications added to the sluices (as in above experiments) are tabulated below.

TABLE NO. 1

Showing values of " C_H " in various experiments. (Area of a sluice=24 sq. ft. No. of sluices=3)

Series No.	Test No.	Description	Q (prototype) (cusecs)	U/S W.L. R.L.	D/S W.L. R.L.	D_1 (ft.)	D_3 (ft.)	H (ft.)	C_H	Proposal No.
I	2	3	4	5	6	7	8	9	10	11
A	1	Existing design with gates blocking arch sector	777.8	2,008.0	2,003.55	21.5	17.3	4.45	0.640	I. Channel d/s of Weir Regulator <i>unlined</i> and <i>unwidened</i> , $n=.04$, width=30 ft.(existing).
	2	Same as in Test 1 but with 1 in 5 slope in bed in the Channel u/s of the Weir Regulator	880.4	2,008.0	2,003.55	21.5	17.3	4.45	0.725	
	3	Arch sector filled with 1 in 3 divergence d/s of the gate and 1 in 10 slope in u/s channel bed.	986.5	2,008.0	2,003.55	21.5	17.3	4.45	0.812	
	4	Horizontal bed in channel u/s of the Weir Regulator, Bellmouth u/s of gate, arch sector filled with 1 in 3 divergence and 1 in 10 expansion d/s of the sluices.	1,511.0	2,008.0	2,003.55	21.5	17.3	4.45	1.244	
	5	Same as Test 5, but with arch sector filled with 1 in 8 divergence d/s of the gate . . .	1,616.0	2,008.0	2,003.55	21.5	17.3	4.45	1.330	
B	1	Existing design with gates blocking arch sector	1,074.0	2,008.0	1,999.25	21.5	13.0	8.75	0.630	II. Channel d/s of Weir Regulator <i>unlined</i> but <i>widened</i> from 30 ft. to 41 ft., $n=.04$.
	2	Same as in Test 1 but with 1 in 5 slope in bed in the channel u/s of the Weir Regulator.	1,288.0	2,008.0	1,999.25	21.5	13.0	8.75	0.756	
	3	Arch sector filled with 1 in 3 divergence d/s of the gate and 1 in 10 slope in u/s channel bed.	1,462.0	2,008.0	1,999.25	21.5	13.0	8.75	0.857	
	4	Horizontal bed in channel u/s of the Weir Regulator, Bellmouth u/s of gate, arch sector filled with 1 in 3 divergence and 1 in 10 expansion d/s of sluices.	2,055.0	2,008.0	1,999.25	21.5	13.0	8.75	1.206	
	5	Same as Test 5, but with arch sector filled with 1 in 8 divergence d/s of the gate . . .	2,347.0	2,008.0	1,999.25	21.5	13.0	8.75	1.378	
C	1	Existing design with gates blocking arch sector	1,134.0	2,008.0	1,997.45	21.5	11.2	10.55	0.606	III. Channel d/s of Weir Regulator <i>lined</i> but <i>unwidened</i> , $n=.02$, width=28.0 ft.
	2	Arch sector filled with 1 in 3 divergence d/s of the gate and 1 in 10 slope in u/s channel bed.	1,495.0	2,008.0	1,997.45	21.5	11.2	10.55	0.800	
	3	Horizontal bed in channel u/s of the Weir Regulator, Bellmouth u/s of gate, arch sector filled with 1 in 3 divergence and 1 in 10 expansion d/s of sluices.	1,878.0	2,008.0	1,997.45	21.5	11.2	10.55	1.004	
	4	Same as Test 5, but with arch sector filled with 1 in 8 divergence d/s of the gate . . .	2,582.0	2,008.0	1,997.45	21.5	11.2	10.55	1.080	

TABLE No. 2

Showing values of 'Afflux' in various experiments

Series No.	Expt. No.	Description	Q (Proto-type) (cusecs)	U/S W.L. R.L.	D/S W.L. R.L.	D ₁ (ft.)	D _s (ft.)	Actual afflux X (ft.)	Permissible afflux X ₁ (ft.)	Proposal No.
1	2	3	4	5	6	7	8	9	10	11
A ₁	1	Existing design with gates blocking arch sector	2,000	2,030.26	2,003.55	43.76	17.3	26.71	4.45	I. Channel d/s of Weir Regulator <i>Unlined</i> and <i>Unwidened</i> , n=.04, width=30 ft. (existing).
	2	Same as in Expt. 1 but with 1 in 5 slope in bed in the channel u/s of the Weir Regulator.	2,000	2,025.96	2,003.55	39.46	17.3	22.41	4.45	
	3	Arch sector filled with 1 in 3 divergence d/s of the gate and 1 in 10 slope in u/s channel bed.	2,000	2,023.69	2,003.55	37.19	17.3	20.14	4.45	
	4	Horizontal bed in channel u/s of Weir Regulator, bellmouth u/s of gate, arch sector filled with 1 in 3 divergence and 1 in 10 expansion d/s of the sluices.	2,000	2,010.96	2,003.55	24.46	17.3	7.41	4.45	
	5	Same as Expt. 4, but with arch sector filled with 1 in 8 divergence d/s of the gate .	2,000	2,010.21	2,003.55	23.71	17.3	6.66	4.45	
B ₁	1	Same as Expt. 1 above	2,000	2,027.97	1,999.25	41.47	13.0	28.72	8.75	II. Channel d/s of Weir Regulator <i>Unlined</i> but <i>Widened</i> from 30 ft. to 41 ft., n=.04.
	2	Same as Expt. 2 above	2,000	2,018.51	1,999.25	32.01	13.0	19.26	8.75	
	3	Same as Expt. 3 above	2,000	2,017.96	1,999.25	31.46	13.0	18.71	8.75	
	4	Same as Expt. 4 above	2,000	2,005.71	1,999.25	19.21	13.0	6.46	8.75	
	5	Same as Expt. 5 above	2,000	2,005.34	1,999.25	18.84	13.0	6.09	8.75	
C ₁	1	Same as Expt. 1 above	2,000	2,028.4	1,997.45	41.9	11.2	30.95	10.55	III. Channel d/s of Weir Regulator <i>Lined</i> but <i>Unwidened</i> , n=.02, width = 28 ft.
	2	Same as Expt. 2 above	2,000	..	1,997.45	..	11.2	..	10.55	
	3	Same as Expt. 3 above	2,000	..	1,997.45	..	11.2	..	10.55	
	4	Same as Expt. 4 above	2,000	2,002.98	1,997.45	16.48	11.2	5.53	10.55	
	5	Same as Expt. 5 above	2,000	2,002.91	1,997.45	16.41	11.2	5.46	10.55	

TABLE NO. 3

Showing values of 'Afflux' in various experiments

Series No.	Expt. No.	Description	Q (Proto-type.) (cusecs.)	U/S W.L. R.L.	D/S W.L. R.L.	D ₁ (ft.)	D ₃ (ft.)	Actual Afflux X (ft.)	Permissible Afflux X ₁ (ft.)	Proposal No.
1	2	3	4	5	6	7	8	9	10	11
A ₁	1	Existing design with gates blocking arch sector	1,500	2,015-17	2,000-25	28-67	14-0	14-92	7-75	I. Channel d/s of Weir Regulator <i>Unlined</i> and <i>Unwidened</i> n=.04, width=30 ft. (existing).
	2	Same as in Expt. 1 but with 1 in 5 slope in bed in the channel u/s of the Weir Regulator.	1,500	2,012-45	2,000-25	25-95	14-0	12-20	7-75	
	3	Arch sector filled with 1 in 3 divergence d/s of gate and 1 in 10 slope in u/s channel bed	1,500	2,011-13	2,000-25	24-63	14-0	10-88	7-75	
	4	Horizontal bed in channel u/s of Weir Regulator, bellmouth u/s of gate, arch sector filled with 1 in 3 divergence and 1 in 10 expansion d/s of sluices.	1,500	2,004-76	2,000-25	18-26	14-0	4-51	7-75	
	5	Same as Expt. 4, but with arch sector filled with 1 in 8 divergence d/s of gate	1,500	2,003-91	2,000-25	17-41	14-0	3-66	7-75	
B ₁	1	Same as Expt. 1 above	1,500	2,010-91	1,997-75	24-41	11-5	13-16	10-25	II. Channel d/s of Weir Regulator <i>Unlined</i> but <i>Widened</i> from 30 ft. to 41 ft., n=.04.
	2	Same as Expt. 2 above	1,500	2,008-56	1,997-75	22-02	11-5	10-81	10-25	
	3	Same as Expt. 3 above	1,500	2,008-19	1,997-75	21-69	11-5	10-44	10-25	
	4	Same as Expt. 4 above	1,500	2,001-98	1,997-75	15-48	11-5	4-23	10-25	
	5	Same as Expt. 5 above	1,500	2,001-20	1,997-75	14-70	11-5	3-45	10-25	
C ₁	1	Same as Expt. 1 above	1,500	2,010-19	1,995-75	25-69	9-5	14-44	12-25	III. Channel d/s of Weir Regulator <i>Lined</i> but <i>Unwidened</i> , n=.02, width=28 ft.
	2	Same as Expt. 2 above	1,500	..	1,995-75	..	9-5	..	12-25	
	3	Same as Expt. 3 above	1,500	..	1,995-75	..	9-5	..	12-25	
	4	Same as Expt. 4 above	1,500	1,999-68	1,995-75	13-18	9-5	3-93	12-25	
	5	Same as Expt. 5 above	1,500	1,998-98	1,995-75	12-48	9-5	3-23	12-25	

Approach Channel Upstream of the Weir Regulator

On reproducing the existing conditions of the bed and side configurations of the channel upstream of the Weir Regulator in the model, it was observed that the bed of the channel is all through at an elevation higher than the upstream sill of the sluices. This channel was therefore resectioned by cutting away the rock and making the banks nearly vertical. The bed was cut to a slope of 1:5 downwards in Test (2),—see Tables 1 to 3, and a slope of 1:10 in case of test (3). The bed was further excavated and made horizontal, i.e., at R.L. 1986.5, up to a distance of 25 ft. upstream of the sluices in tests 4 and 5.

Coefficient of Discharge

Series A—(Reference Table No. 1)—From the results of Table No. 1 it will be seen that the coefficient of discharge of the sluices could be increased from 0.64 (Test No. 1) to 1.330 (Test No. 5) with the following modifications added in the model—

- (1) A bellmouth upstream of the gate to reduce head losses at entrance.
- (2) Filling the arch sector with 1 in 8 divergence downstream of the gates to minimise the losses due to sudden expansion downstream of the gate which blocks the arch sector.
- (3) Giving 1 in 10 divergence in the bed, sides and top of the arched sluices for a length of 1=10 ft.—in prototype from the existing d/s end of the sluices, to reduce the head losses due to sudden expansion of flow emerging out of the sluices.

The area of waterway at the gate, in the sluices and at the end of the 10 ft. length of expansion downstream of a sluice was 24 sq. ft., 26.79 ft. and 52.0 ft. respectively.

Series B and C—Similarly in series B and C which give results of Proposal Nos. II and III, the value of discharge coefficient could be raised to 1.378 and 1.380 respectively.

It will be seen from Tests No. 5 and 4 that a slope of 1 in 8 in arch sector as against 1 in 3 has increased the coefficient of discharge by about 7.0 per cent in the case of series A, for instance, when all other conditions in these two tests were kept the same. A comparison of results of these experiments was therefore made with those of the Nira Left Bank Canal Head Regulator¹ where the coefficient of discharge as high as 1.927 against 1.33 in series A was achieved by remodelling.

	Nira L.B. Canal Head Regulator.	Ghataprabha L.B. Canal Head Regulator at the Weir.
(1) Original discharging capacity	655 cusecs.	778 cusecs.
(2) Maximum discharge required to be passed.	720 ,,	2,000 ,,
(3) Number of sluices	7, each 4'×4'	3, 4'×6' each.
(4) Area of a single sluice	16 sq. ft.	24 sq. ft.
(5) Discharge through a sluice	$\frac{720}{7}$ = 103 cusecs approx.	$\frac{2,000}{3}$ = 667 cusecs approx.
(6) Discharge through per sq. ft. of opening	$\frac{103}{16}$ = 6.437 cusecs.	$\frac{667}{24}$ = 27.79 cusecs.
(7) Operating Head	0.174 ft.	4.45 ft.

¹Bombay P. W. D. Technical Paper No. 54. Note on experiments with a model of Head Sluices of the Nira L.B. C. by C. C. Inglis and D. V. Joglekar, 1935.

It will be seen from the above comparison that the discharge per sq. ft. in the present case is far greater (about 4 times) and consequently the velocity is greater than that in the Nira Canal Sluices, where 1 in 3 slope, out of 1:2, 1:5 and 1:10, in the arch sector had given better results. In the present model it was observed that with 1 in 8 slope the flow in the sluices was uniform and followed the slope in the arch.

Afflux

In the series A₁, B₁ and C₁ the three proposals when the depth downstream of the Weir Regulator is 17.3, 13.0 and 11.2 ft. respectively were tested. The full supply discharge of 2,000 \equiv 6.324 cusecs was actually passed in the model and keeping the depth downstream constant in each series, observations of afflux in upstream were taken and are given in Table No. 2. The afflux permissible when downstream depth is 17.3 ft. is equal to 4.45 ft.

In Test 1 the afflux measured under the existing conditions was 26.7 ft. which could be reduced to 6.66 ft. when a bellmouth upstream of the gate, 1 in 8 divergence in the arch and 1 in 10 expansion downstream of the sluices were added—*vide* Test No. 5.

The results of series B and C of Table No. 1 and series B₁ and C₁ of Table No. 2 show that if depth downstream of the Weir Regulator is 13.0 ft. (*i.e.*, Proposal II when channel is unlined but widened from 30 to 41 ft.), the full supply discharge of 2,000 cusecs can be easily passed after the modifications are introduced in the design of the sluices and also in the case when depth in downstream channel is 11.2 ft. (*i.e.*, Proposal III, Channel lined but unwidened). But as widening the 3,028 ft. long channel downstream of the Weir Regulator or lining the same would comparatively be more expensive than keeping the channel as it is, experiments are being continued to improve the design still further.

10. CANAL FALLS

24. ECONOMIC DESIGN OF FALLS FOR HIRAKUD PROJECT

1. Introduction

Over 500 Canal Falls have to be constructed in the Main Canal and Distributaries of the Hirakud Project. The range of discharge is from 50 cusecs to 2,000 cusecs, while that of the drops is from 1 ft. to 10 ft.

The Central Designs Office of the Central Water and Power Commission, New Delhi, designed three types of falls, *viz.*—

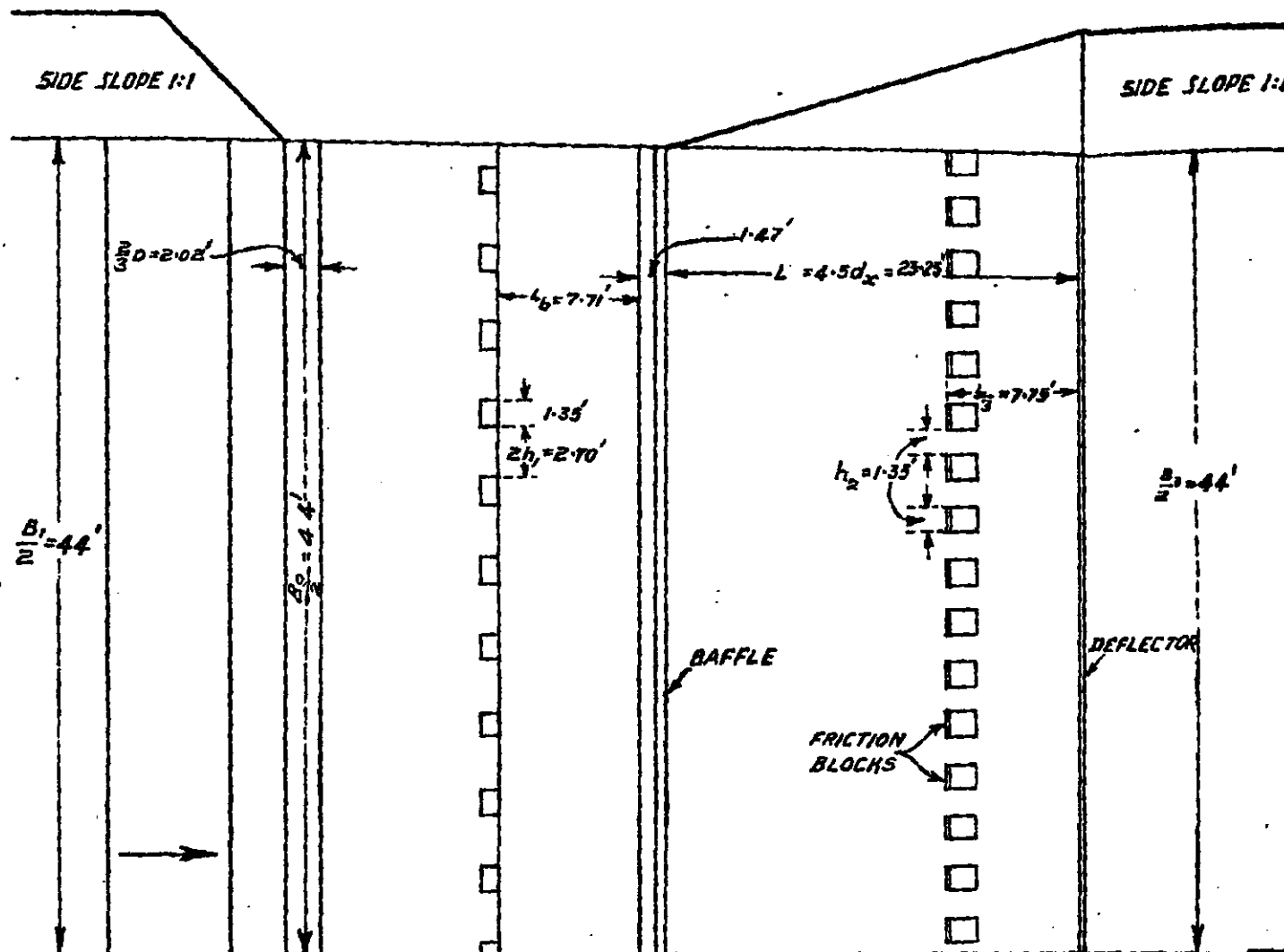
- (i) Q= 500 cusecs, drop=5 ft.—Flumed Vertical Type.
- (ii) Q=1,500 cusecs, drop=4 ft.—Flumed Glacis Type.
- (iii) Q=1,500 cusecs, drop=5 ft.—Flumed Baffle Type.

This Station was asked to carry out experiments on these types. The fundamental aim of these experiments was not merely to improve the performance of the design supplied by C. D. O., but to make any changes, howsoever radical they may be, to evolve cheap and effective designs. The reference was, though, of a specific nature ; yet model experiments were carried out on an exhaustive scale, so that general conclusions could be drawn to evolve type designs for falls under various conditions of discharges and drops. Apart from a large number of quick tests, as many as 26 specific model experiments for Unflumed Falls and 18 Specific Experiments for Flumed Falls were conducted. The results of all these experiments are compiled in Appendices I and II, with brief remarks.

2. Designs

Fundamentally there appear to be two distinct types of designs, *i.e.*—

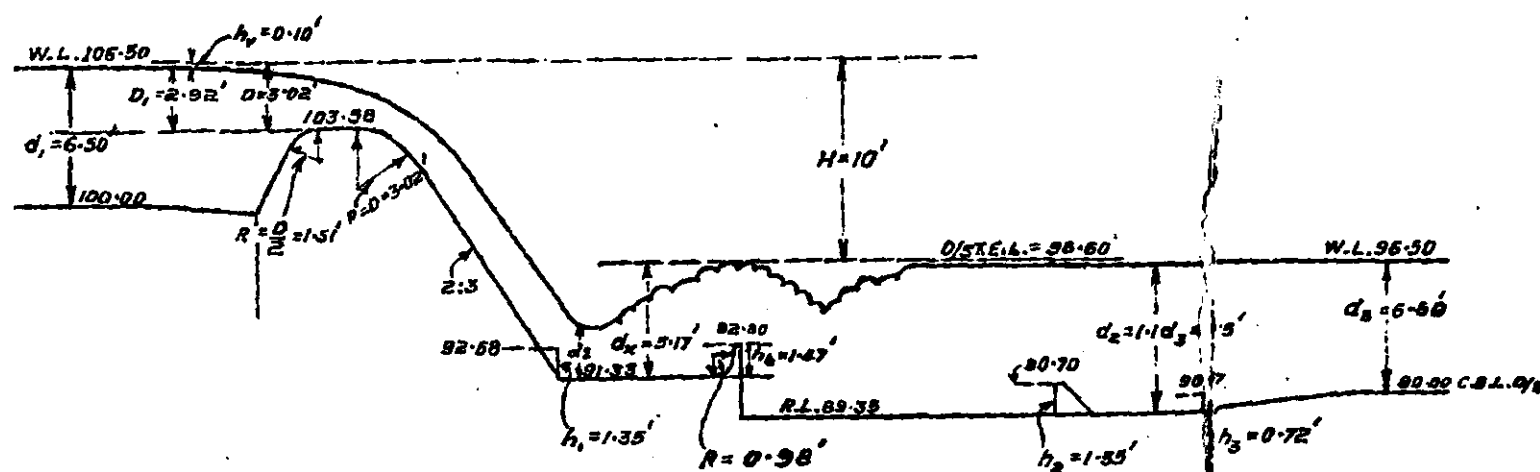
- (a) those in which the standing wave is used for dissipating excess energy, and
- (b) those in which there is no clear standing wave, but the nappe impinges on the pool downstream and the dissipation occurs in the turbulent diffusion by eddies.



HALF PLAN

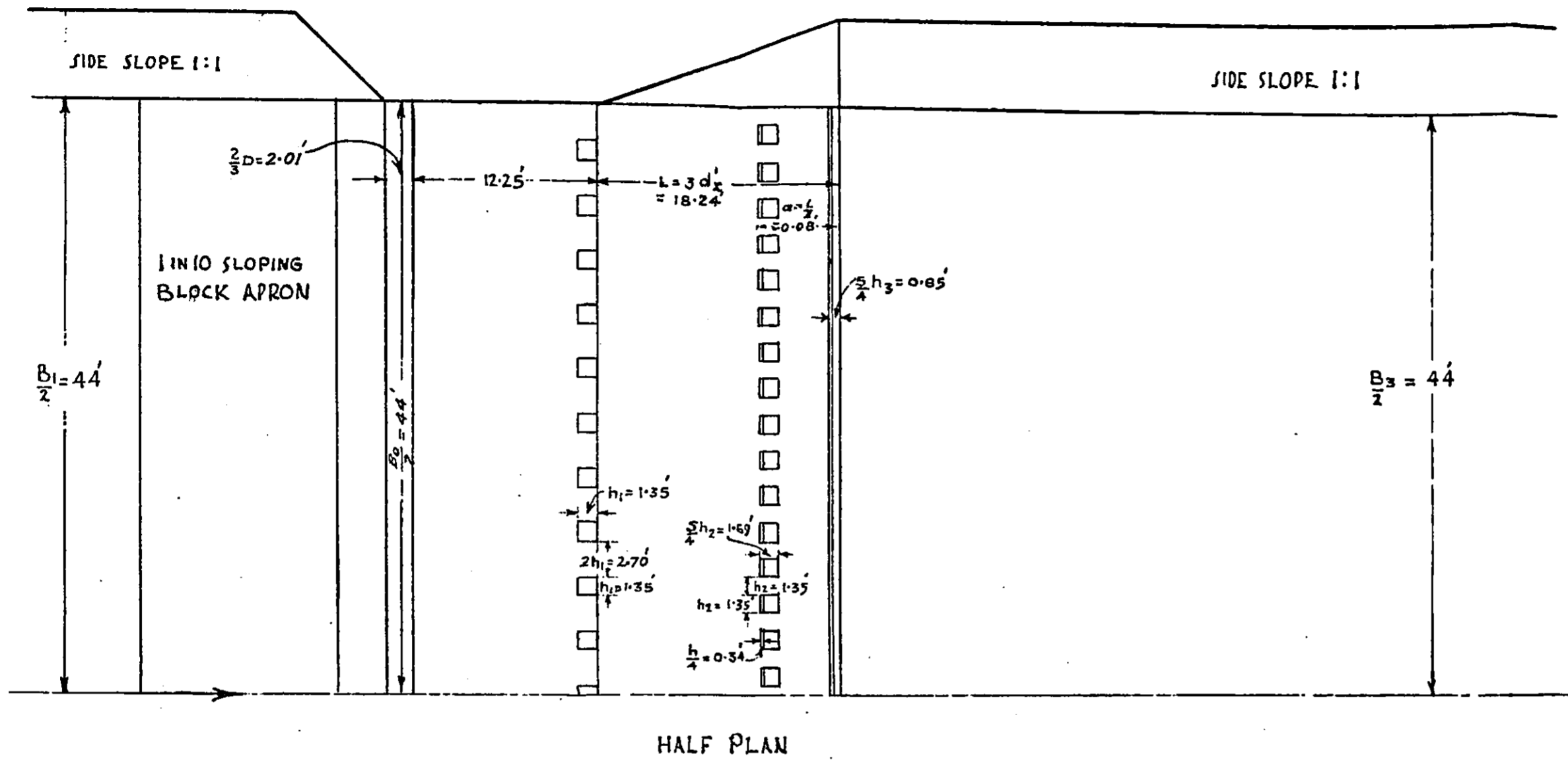
INDEX

- Q = DISCHARGE IN CANAL = 1500 CUSECS.
 H = DROP = 10'
 q = DISCHARGE PER FOOT AT THE CREST = $\frac{Q}{B_0} = 17.05 \text{ CS./FT.}$
 B_1 = $\frac{1}{2}$ WIDTH OF CANAL = 88'
 B_0 = WIDTH AT THROAT = 88'
 B_2 = $\frac{1}{2}$ WIDTH OF CANAL = 88'
 d_1 = $\frac{1}{2}$ DEPTH OF WATER = 6.5'
 d_2 = $\frac{1}{2}$ DEPTH OF WATER = 6.5'
 C = COEFFICIENT OF DISCHARGE (S-B ASSUMED)
 D = HEAD OVER WEIR. $\frac{2}{3} \sqrt[3]{17.05} = 3.02'$
 h_v = HEAD DUE TO VELOCITY OF APPROACH $\frac{v_1^2}{2g} = 0.10'$
 D_1 = ACTUAL DEPTH OVER CREST OF WEIR = $D + h_v = 2.92'$
 d_2 = DEPTH OF CISTERN = $1.1d_1 = 7.15'$
 d_2 = SUBCRITICAL DEPTH NECESSARY TO BALANCE THE STANDING WAVE = $0.73 \sqrt[3]{0.82 H} = 5.17'$
 d_c = CRITICAL DEPTH = $(\frac{2}{3}) \sqrt[3]{\frac{Q^2}{g B^3}} = 2.06'$
 d_{H^2} = HYPERCRITICAL DEPTH = $0.11 \sqrt[3]{\frac{Q^2}{g H^3}} = 0.61'$
 h_b = HEIGHT OF BAFFLE = $d_c - d_2 = 1.47'$
 L_b = LENGTH OF BAFFLE PLATFORM = $5.86 h_b = 7.71'$
 h_1 = HEIGHT OF GLACIS BLOCKS = $\frac{2d_1}{3} = \frac{2}{3} \times 6.5 = 4.33'$
 h_2 = HEIGHT OF FLOOR BLOCKS = $\frac{2d_2}{3} = \frac{2}{3} \times 7.15 = 4.77'$
 h_3 = HEIGHT OF DEFLECTOR = $\frac{d_2}{10} = 0.72'$

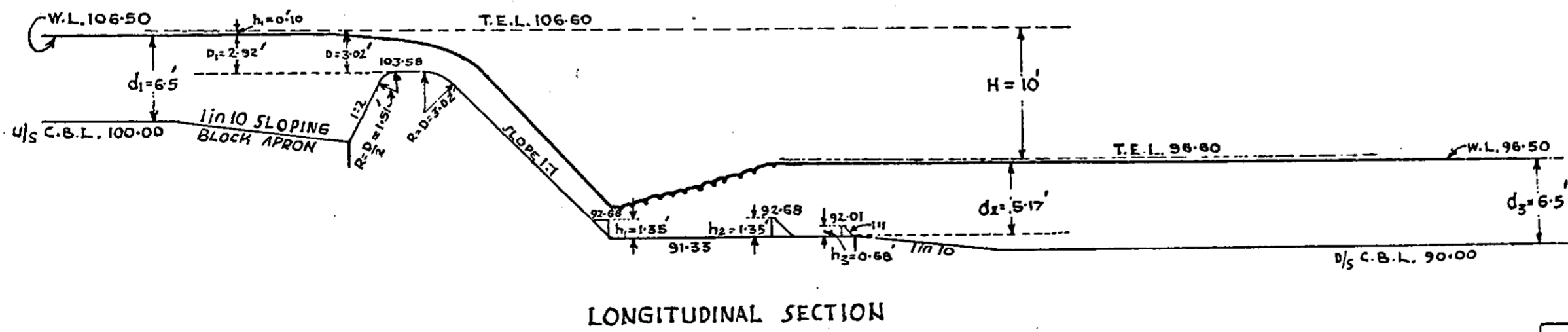


LONGITUDINAL SECTION

FIG. 1
UNFLUMED BAFFLE FALL DESIGN1500 CUSECS, 10' DROP.
SCALE: 1" = 10'COMPARED BY: *Mun Singh*
TRACED BY: M.K. SABNIS. 12-11-1951

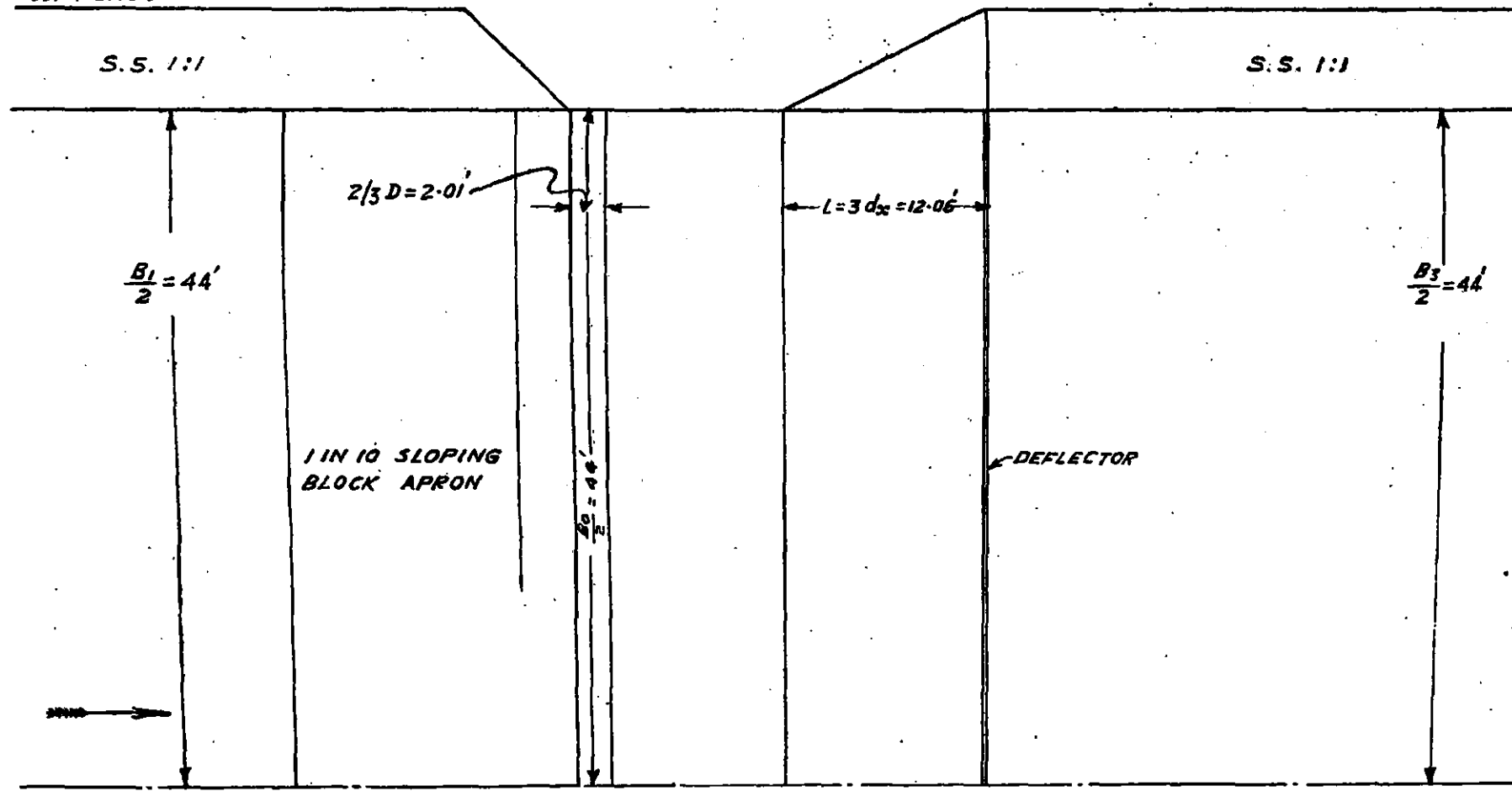


INDEX:-
 B₁ = U/S CANAL WIDTH = 88'
 B₀ = WIDTH OF THE THROAT = 88'
 B₃ = D/S CANAL WIDTH = 88'
 d₁ = U/S DEPTH OF WATER = 6.5'
 d₃ = D/S DEPTH OF WATER = 6.5'
 d_x = SUB-CRITICAL DEPTH REQUIRED TO BALANCE STANDING WAVE = $0.73 \frac{Q}{D} 0.52 H^{0.22} = 5.17'$
 D = HEAD OVER THE CREST = $(\frac{Q}{C})^{2/3} S^{0.2} = 2.92'$
 D₁ = DEPTH OF WATER OVER WEIR CREST = 2.92'
 d_x = THEORETICAL SUBCRITICAL DEPTH BALANCING THE STANDING WAVE = $(0.85)^{-1} d_x = 6.08'$
 L = LENGTH OF PAVEMENT = 3d_x = 18.24
 d₁ = DISTANCE OF U/S FACE OF FLOOR BLOCKS FROM D/S END OF PAVEMENT = $\frac{L}{3} = 6.08'$
 h₁ = HT. OF GLACIS BLOCKS = d₂ OR $\frac{2}{3} d_x$, WHICHEVER IS GREATER = $\frac{2}{3} d_x$ IN THE DESIGN = 1.35'
 h₂ = HT. OF FLOOR BLOCKS $\frac{2}{3} d_x = 1.35'$
 h₃ = HT. OF COMBINED DEFLECTOR = $\frac{d_x}{5} = 0.68'$
 SPACING OF GLACIS BLOCKS = 2h₁
 WIDTH " " " = h₁
 SPACING OF FLOOR BLOCKS = h₂



LONGITUDINAL SECTION

FIG. 2
ALL-AMERICAN CANAL TYPE FALL DESIGN
OF A 1500 CUSECS, 10' DROP
UNFLUMED FALL
SCALE = 1" = 10'

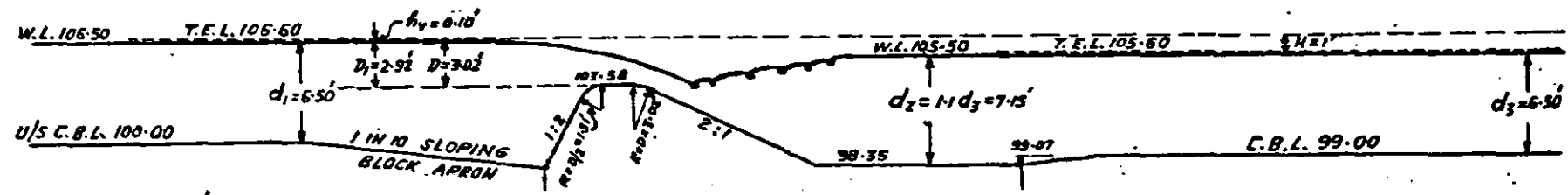


INDEX

B_1 = U/S CANAL WIDTH = 88'
 B_0 = WIDTH OF THE THROAT = 88'
 B_3 = D/S CANAL WIDTH = 88'
 d_1 = U/S DEPTH OF WATER = 6.5'
 d_3 = D/S DEPTH OF WATER = 6.5'
 D = HEAD OVER THE CREST
 $= \left(\frac{Q^2}{C}\right)^{2/3} = 3.02'$

L = LENGTH OF PAVEMENT = $3d_x = 12.06'$
 $d_x = 0.73 q_0^{0.52} H_c^{0.21} = 4.02'$
 h_2 = HT. OF DEFLECTOR = $\frac{d_2}{10} = 0.72'$
 D_1 = U/S DEPTH OF WATER OVER CREST = $D - h_2 = 2.92'$
 d_2 = DEPTH OF CISTERN BELOW D/S W.L. = $1.1 \cdot d_3$

HALF PLAN



LONGITUDINAL SECTION

FIG. 3
STRAIGHT GLACIS (2:1) TYPE
UNFLUMED FALL
 $Q = 1500$ CUSECS, $H = 1'$
 SCALE $1'' = 10'$

COMP'D. BY *Munasingh*
 TRACED BY, B.G. DEHPANDE 5-51

The following designs, viz.—

- (i) Baffle type ;
 - (ii) " All American Canal Fall " type ; and
 - (iii) Parabolic or straight flat glacis type.
- belongs to the category (b).

3. Requirements

The design of a Fall envisages the following criteria :

- (i) It should be cheap in initial cost.
- (ii) Recurring expenses on maintenance and repairs should be minimum.
- (iii) A certain amount of harmless scour may be tolerated ; i.e., a cheap design giving a little harmless scour is preferable to one giving comparatively less scour but costing appreciably more.
- (iv) The design should work safely under 10 per cent retrogression, if it were to occur subsequently.
- (v) It should be capable of being used as a meter.

Experiments in this Station were carried out to satisfy the criteria (i) to (iv) only.

4. Groups

For the purpose of experiments, falls were divided into four main groups according to discharges and drops. They are—

- Group I—High discharges and high falls, i.e., Discharge (Q) greater than 500 cusecs and head (H) greater than 3 ft.
- Group II—High discharges and low falls, i.e., Q greater than 500 cusecs and H less than or equal to 3 ft.
- Group III—Low discharges and high falls, i.e., Q less than 500 cusecs and H greater than 3 ft.
- Group IV—Low discharges and low falls, i.e., Q less than 500 cusecs and H less than or equal to 3 ft.

Each of these four main groups will obviously have two sub-divisions, viz.,—

- (a) flumed, and (b) unflumed type.

In the following paragraphs the results of experiments for each main group of falls have been discussed. The Unflumed and Flumed designs have been considered separately in Part I and Part II.

The following notation is used:—

Notation used in the figures attached.

Q=Discharge in cusecs.

q=Discharge per foot run.

q_0 =Discharge per foot run in the throat.

B_1 =Bed width of canal upstream of the fall.

B_0 =Width of the throat.

B_2 =Bed width of canal downstream of fall.

d_1 =Upstream depth of water.

d_2 =Downstream depth of water.

d_0 =Critical depth, i.e. $(q^2/g)^{1/3}$.

d_c =Depth of water in the cistern below the fall.

D_0 =Depth of cistern below downstream C. B. L.

D_1 = Upstream depth of water over weir crest.

D = Head over the weir, i.e., D_1 plus h_v .

h_v = Head due to velocity of approach.

d_1 = Sub-critical depth required to balance the standing wave after allowing for frictional losses.

$$d_1 = 0.73 q_v^{0.52} H^{0.21} \quad (\text{vide A. R. T., 1944, page 50}).$$

d_1' = Theoretical sub-critical depth required to balance the standing wave without any allowance for losses.

d_2 = Supercritical depth at the standing wave. This may be worked out by the relation

$$d_2 = 0.11 q_v^{0.89} H^{-0.35}$$

H = Height of fall.

T.E. = Total Energy.

h_1 = Height of glacis blocks.

h_2 = Height of floor blocks.

h_3 = Height of deflector.

h_b = Height of baffle.

L = Length of downstream pavement.

L_b = Length of baffle platform.

Part I—Unflumed Designs

5. Group I, i.e., High Discharges and High Falls

Fig. 1.

(i) *Baffle Type Designs*—A modified economical design of this type consists of a narrow crested weir having crest $2/3 D$ long—Figure 1—joined to the upstream $1/2:1$ steep glacis and downstream $2:3$ steep glacis by curves of radius $1/2 D$ and D respectively. The $2:3$ glacis ends in the baffle platform which is at the level at which the standing wave naturally forms under designed conditions. The baffle platform is 5.25 times the height of the baffle (h_b), the height of baffle being $d_2 - d_1$ feet. The baffle platform is followed by a cistern $4.5 d_1$ long at a level giving $d_3 = 1.10 d_1$. A deflector $d_3/10$ high at the end of the cistern helps to form the beneficial roller with horizontal axis, which piles up sand against the pavement thus protecting the *pukka* work. A set of glacis blocks at the toe of the glacis helps to reduce the wave wash.

The design described above works very well under normal designed conditions and is also equally satisfactory for 10 per cent or even more up to 25 per cent retrogression in downstream levels.

Fig. 2.

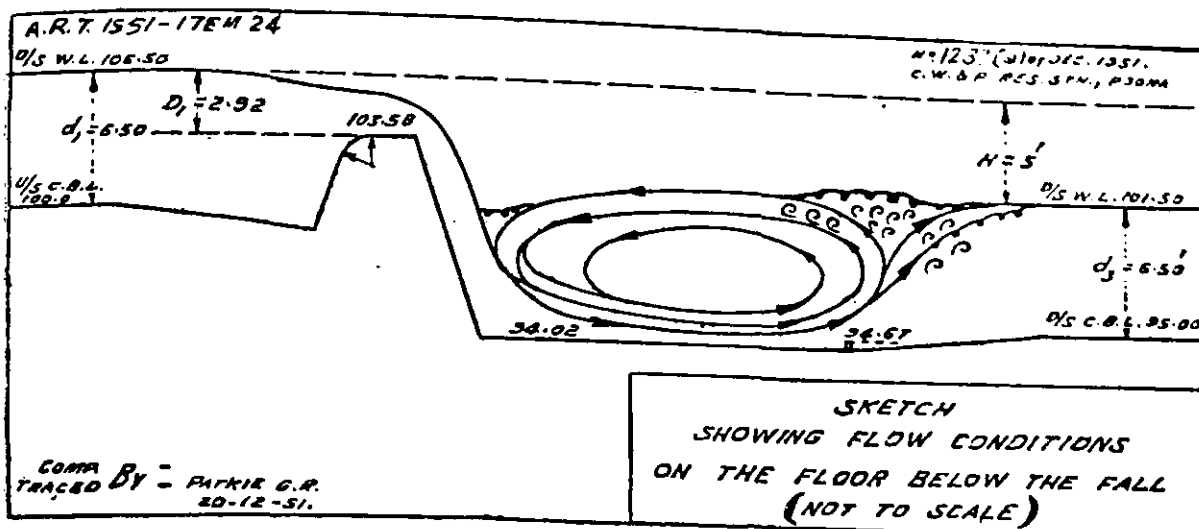
(ii) *All American Canal Fall Type Design*—This design consists of a $1:1$ glacis below the weir crest ending in a horizontal platform at the level at which the standing wave naturally forms under designed conditions. This platform which is $3 d_1'$ long—Figure 2—has a continuous deflector at its end and a row of floor blocks at d_1' feet upstream of the deflector to deflect high bed velocities upwards. Glacis blocks at the toe of the glacis are intended to stabilise the standing wave. In the American design the spacing of floor blocks is twice their height but this may be reduced to equal their height, to control the scour depths within tolerable limits.

This design works well enough both under normal and 10 per cent retrograded condition. Should, however, the soil be easily erodible a second staggered row of floor blocks fixed at a distance 2.5 times the height of the block placed below the row described above may be provided with advantage.

(iii) *Parabolic Glacis Type Design*—In this design the parabolic glacis ensures maximum horizontal component of velocity of the falling jet, which is dissipated effectively. The glacis is carried down to cistern level which is such as to give 25 per cent more energy of flow (i.e., d plus $v^2/2g$). This is followed by a pavement and a deflector at its end. This design works satisfactorily.

(iv) *Vertical Type*—For high discharges and high falls, the performance of vertical type fall designs is unsatisfactory. In this design, optimum dissipation of

energy, as obtained in a standing wave, does not take place. The nappe after impinging on the water cushion below the crest, tends to flow forward along the bed. This bed flow of high velocity is partly deflected upward by the deflector causing intense boils and eddies below it over the erodible bed; and partly forms reverse flow in the cistern. See sketch below:



This is an unsatisfactory feature. Besides this, the tendency to form side embayments below *pukka* work is an intrinsic defect in the design and the maintenance charges are therefore high.

Remarks—Both designs of Baffle type (i) and the All American Canal Fall type (ii) work satisfactorily. However, the later type is the cheaper of the two because of its short pavement. This type, therefore, may be adopted where soil is hard. In case the soil is easily erodible it is preferable to go in for a baffle type design, which, though expensive in initial cost, is cheaper in maintenance, better in performance and works even if as heavy a retrogression as 25 per cent were to occur.

A parabolic or flat glacis type design is much more expensive and is therefore, not recommended.

Vertical type is not recommended because of its unsatisfactory performance.

6. Group II—High Discharges and Low Falls

(i) *Baffle Type*—This design works only when the fall is not drowned.

(ii) *All American Canal Fall Type*—This design also, like the baffle design, does not work when the fall is drowned.

(iii) *Glacis Type*—Under drowned conditions a 2:1 glacis below the crest followed by a pavement $3 d_x$ long—Figure 3—at a level giving $d_x = 1.1 d$, works well. The 2:1 glacis helps to stabilise the flow. A deflector at the end of pavement protects the *pukka* work by deflector action.

Fig. 3.

(iv) *Vertical Type*—In this type the inherent tendency of side erosion and incomplete dissipation of excess energy renders it unsuitable.

Remarks—Under clear overfall conditions same remarks apply as in para (5). But under drowned conditions, a glacis design described in (iii) above is preferable and should be adopted.

7. Group III, i.e., Low Discharges and High Falls

In this group, for discharges below 500 cusecs and drops above 3 ft. all the four types, viz. (i) the baffle type, (ii) All American Canal type, (iii) The parabolic or flat glacis type and (iv) Vertical type, work well except that the vertical fall design which for its inherent defects is suitable only for discharges below 300 cusecs.

The Baffle and the All American type designs are shown in Figures 1 and 2.

Fig. 4.

Figure 4 shows the vertical type design in details for $Q=100$ cusecs and $H=10$ ft. The design consists of a pavement $5 (H.D_1)^{1/2}$ long.¹ The level of the pavement is lower than the downstream canal bed level by an amount D_c where $D_c=1/6 (H.D_1)^{1/2}$.

Two rows of floor blocks $\frac{d_z}{6}$ high, the upper row being at $1/3$ rd the pavement length above its downstream end and the second staggered one 2.5 times block height below it, are provided to redistribute the bed velocities and reduce bed and side scour. A deflector $\frac{d_z}{10}$ high at the end of pavement helps to protect the pavement.

Remarks—For discharges between 500 and 300 cusecs, where soil is easily erodible, a baffle type design may be adopted though it may be a little more expensive in initial cost, as due to its very efficient performance as regards energy dissipation in the primary and secondary standing waves, no scour may be expected.

Where soil is hard an All American Canal Fall type design may be adopted. It is less expensive than the other designs.

Below 300 cusecs the choice of the design may be based on cost considerations, whichever type may be found economical.

8. Group IV—Low Discharges and Low Falls

- (i) Baffle type design.
- (ii) All American type design.
- (iii) Parabolic Glacis type design.
- (iv) The vertical type design.

All designs work well when the fall is not drowned ; though vertical fall gives some side erosion which may be prevented by providing side D. B. pitching.

Under drowned conditions, in this group, a modified glacis type design works satisfactorily. The glacis slope of 1:1 helps to stabilise flow. A pavement $3d_r$ long at a level giving $d_r=1.1 d_s$, with a deflector at the end and a row of blocks $\frac{d_z}{6}$ high at $1/3$ rd pavement length above the deflector helps to redistribute velocities and check scour—Figure 5—with the glacis flattened to the slope of 2:1 the floor blocks may be omitted.

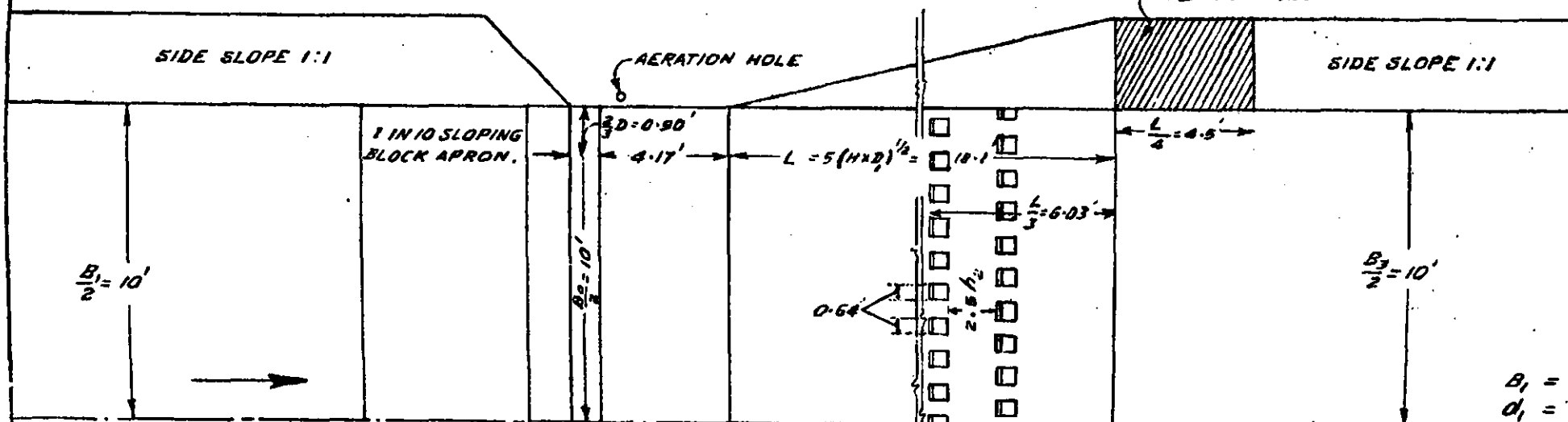
Fig. 5.

The vertical type design gives unstable conditions of flow, which can be stabilised by blocks $\frac{d_z}{6}$ high at $3d_2$ distance from the toe of fall. These blocks serve the purpose of redistribution of velocities and also dissipate a certain amount of energy by creating turbulence as the high velocity nappe impinges against them. With a discharge of 100 cusecs, with one foot drop, this design works well enough, but as the discharge increases, another row of blocks at $1/3$ rd the length of pavement from the downstream end is required to check bed and side erosion. Side D. B. pitching may be provided to guard against possible side embayments.

Remarks—Under clear overfall conditions either of the designs may be adopted. Cost may justify adopting "All American" canal design. However, where retrogression in bed levels is likely to occur, it may be preferable to adopt baffle type design.

Under drowned conditions a modified glacis design as described above may be adopted for higher discharges. For discharges below about 300 cusecs a vertical type may be preferable to glacis, the former being cheaper.

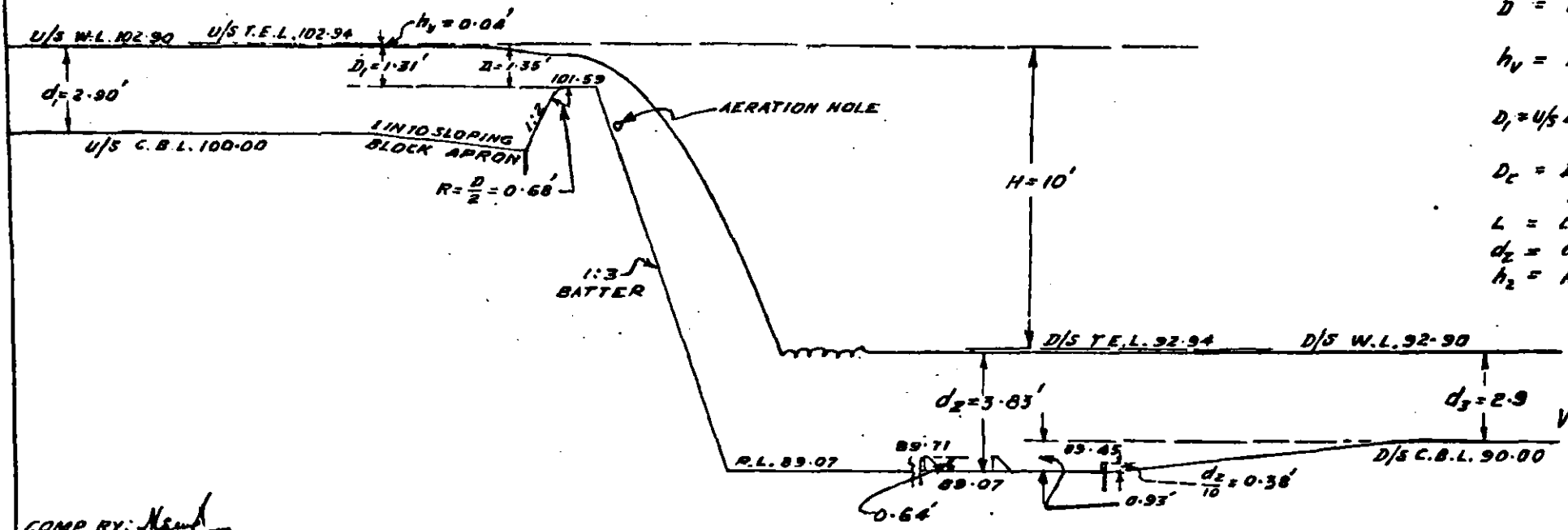
¹ U. P. Research Station Formulae.



HALF PLAN

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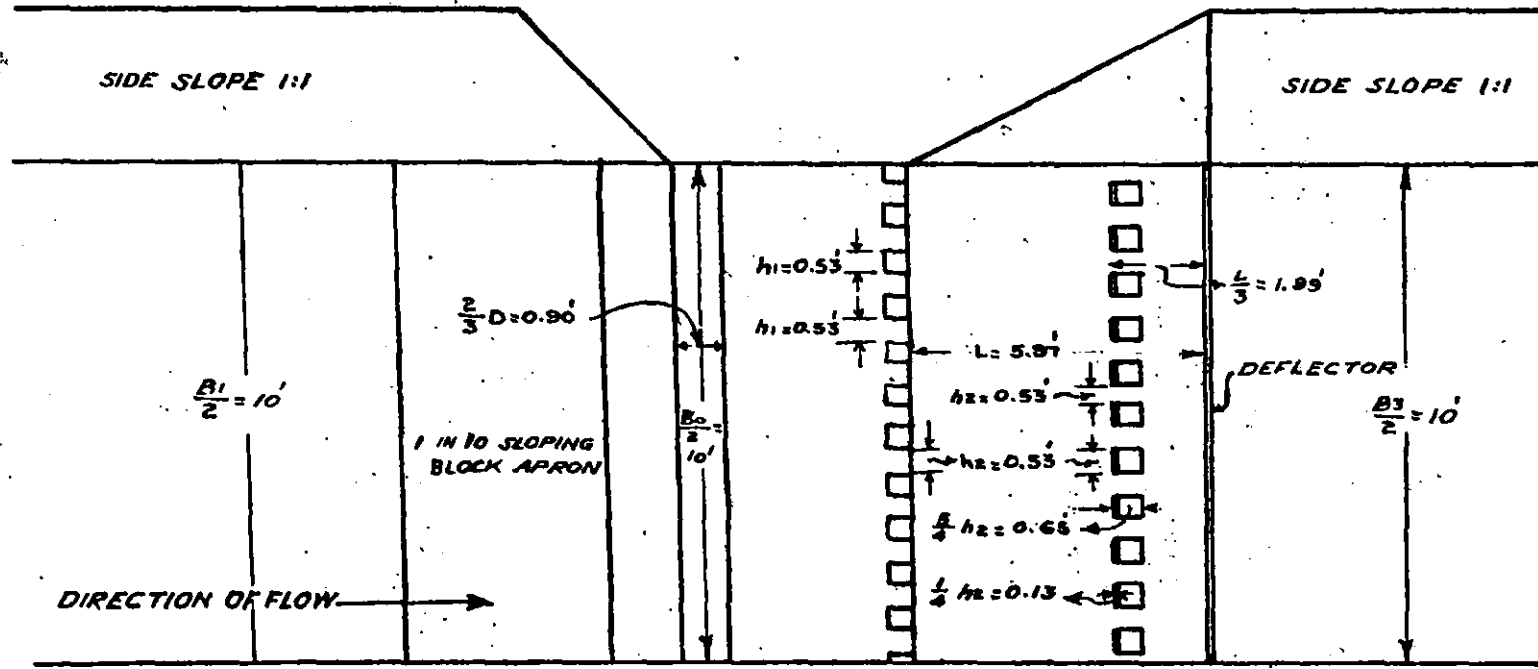
- B_1 = U/S CANAL WIDTH = 20.0'
- d_1 = U/S DEPTH OF WATER = 2.90'
- B_0 = WIDTH OF THE THROAT = 20.0'
- B_3 = D/S CANAL WIDTH = 20.0'
- d_3 = D/S DEPTH OF WATER = 2.90'
- D = HEAD OVER THE CREST = $(\frac{90}{2})^{1/2} = 1.35'$
- h_v = HEAD DUE TO APPROACH VELOCITY = $\frac{V^2}{2g} = 0.04'$
- D_1 = U/S DEPTH OF WATER OVER CREST OF WEIR = $D - h_v = 1.31'$
- D_c = DEPTH OF CISTERN BELOW D/S C.B.L. = $\frac{1}{2}(H+D)^{3/2} = 0.93'$
- L = LENGTH OF PAVEMENT = $5(H+D)^{1/2} = 18.1'$
- d_2 = $d_3 + D_c = 2.9 + 0.93 = 3.83'$
- h_2 = Ht. OF BLOCKS = $\frac{d_2}{5}$ WIDTH = SPACING



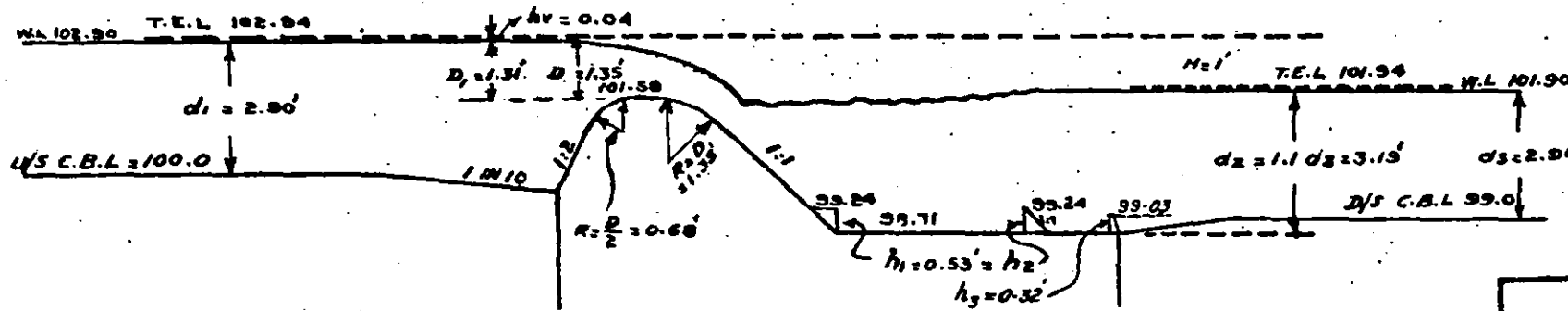
LONGITUDINAL SECTION

FIG. 4
VERTICAL DROP TYPE DESIGN
OF UNFLUMED FALL
 $Q = 100 \text{ C.S.}, H = 10'$
Scale. 1" = 5'

COMP BY: *Mend*
TRACED BY: R. T. THAKOR.
9 NOV. 51



HALF PLAN



L. SECTION

INDEX:-
 $B_1 = U/S$ CANAL WIDTH = 20';
 $d_1 = U/S$ CANAL DEPTH = 2.90'
 $B_0 =$ CREST WIDTH = 20';
 $d_3 = D/S$ CANAL DEPTH = 2.90'
 $B_3 = D/S$ CANAL WIDTH = 20';
 $d_2 =$ DEPTH OVER PAVEMENT = $1.1d_3 = 3.15'$
 $W =$ CREST LENGTH = $\frac{2}{3}D = 0.90'$
 $D =$ HEAD OVER THE CREST = $(\frac{3}{2})^{2/3} = 1.35'$
 $Q_0 =$ DISCHARGE PER FOOT RUN OF CREST
 $C =$ ASSUMED COEFFICIENT OF DISCHARGE = 3.2'
 $h_v =$ HEAD DUE TO APPROACH VELOCITY = 0.04'
 $D_1 =$ DEPTH OVER WEIR CREST = $D - h_v = 1.31'$
 $h_1 =$ HEIGHT OF GLACIS BLOCKS = h_2
 $h_2 =$ FLOOR BLOCKS = $\frac{h_1}{2} = 0.53'$
 $h_3 =$ HEIGHT OF DEFLECTOR = $\frac{h_2}{10} = 0.32'$
 $L =$ LENGTH OF PAVEMENT = $30d_{x_1} = 5.97'$
 $d_{x_1} = (0.85)^{-1} d_x$ WHERE $d_x = 0.73Q_0^{0.52} \times H^{0.21} = 1.69'$
 SPACING OF GLACIS & FLOOR BLOCKS = h_2

FIG:-5
 SHOWING DESIGN OF 100 Cs 1 DROP
 1:1 GLACIS UNFLUMED FALL
 SCALE:- 1" = 4'

COMP. BY:-
 TRACED BY:- MANCHUNDIA.

Part II—Flumed Designs

9. Introduction

It is easier to design an unflumed fall rather than a flumed one. But when a fall is combined with a bridge, in order to reduce the total cost of a separate bridge and a fall; or when it is proposed to use the fall as a regulator as well, a flumed design is adopted.

The Standard type, proportional flumed design has the following characteristics:

- (1) D. B. pitching on sides above the bellmouth followed by a *pukka* bellmouth which consists of 60° curve of radius $2D^{3/2}$ merging into the throat.
- (2) A flat *pukka* pavement upstream.
- (3) A hump or a flat glacis joining the flat upstream pavement to the crest.
- (4) A crest 2.5 D long.
- (5) A parabolic or 2:1 glacis on the downstream side of the crest joined by radius of 2 D to the crest. See Figure 5A.

Fig. 5A.

Experiments were carried out to effect economy in these transitions as a result of which the following modifications are recommended:

- (1) D. B. pitching upstream may be omitted. The side contraction can be modified as shown in Figure 6.
- (2) The *pukka* flat pavement upstream may be replaced by a loose apron. The apron may be made to slope down say at 1 in 10 to facilitate unwatering the upstream channel for repairs.
- (3) The weir crest can be joined to the upstream bed by a short steep glacis of $\frac{1}{2}:1$, slope.
- (4) Crest length may be reduced to $\frac{2}{3} D$.
- (5) The downstream glacis may be made as steep as 2:3. The steep glacis does develop little negative pressures. In the 1/7 geometrically-similar model, the observed negative pressure was only 2 ft.—Figure 7. Allowing for scale effect and side effect, etc., the maximum negative pressure head, with as high a discharge intensity as 38 cusecs, is not expected to be more than about 6 ft. Consequently the difficulty can be overcome by providing reinforced concrete in the region.

Fig. 6.

Fig. 7.

10. Downstream Transitions in Flumed Designs

In straight channel the velocities in a section, fall off from the centre to the sides, *i.e.*, the discharge intensity is higher in the centre than at sides. At the throat of a flume, the side contraction accentuates this distribution of discharge intensity. Thus a flumed fall design has to take care of the side effect also in addition to dissipating the excess energy.

A converging flow is usually stable whereas a diverging flow is not. In a flumed fall, the flow has to expand from the contracted section below the fall to the normal canal section downstream. The downstream side transitions, therefore, have to be so designed that the flow always hugs the sides. If it leaves the sides, it might swing from side to side, or develop return flow on one or both sides. This is generally harmful, because, eddies with vertical axis develop at the junction of forward and return flow causing deep scour which may undermine the downstream curtain wall.

One in five side splay with hyperbolically expanding sides is perhaps the best, since this is about the limiting sharpest splay that expanding water can follow but for simplicity of construction, straight 1 in 5 expanding walls may be adopted.

The hyperbolic expansion may be worked out by the formula evolved by the U. P. Research Station, which is reproduced below:—

$$B_x = \frac{B_0 B_3 L}{LB_3 - (B_3 - B_0)^2}$$

where L = Length of expansion along centre line.

B_0 = Overall throat width.

B_n = Normal channel width.

B_x = Width of expanding flume at distance x from the beginning of expansion.

Rather than make the sides rigid all the way down the expansion, they need be made rigid only down to the end of pavement. At this section the side slopes may be 1:1 or $\frac{1}{2}$:1, being the same as that of the normal channel just below the fall. The portion of the expanding sides below the pavement may thus be carried on at the same slope, with D. B. pitching. This will reduce the cost of the structure. If a sharper splay is adopted, floor blocks in addition have to be introduced in the design for fanning out the flow and thus off-setting any saving in sharper splay. Fluming should not generally exceed $\frac{2}{3}$; and 0.8 is to be preferred. Greater fluming leads to greater discharge intensity in the throat, which results in intense eddies and greater turbulence below, requiring long transitions and thus making the design uneconomical.

11. Group I—High Discharges and High Falls

Fig. 8. (i) *Baffle Type*—With modified upstream transitions as mentioned in paras 9 and 10 above, this design works satisfactorily. Figure 8 shows a type design for $Q=1,500$ cusecs and 10 ft. drop. The design consists of 2:3 glacis with blocks at the toe to check wave wash. This is followed by a baffle platform, a baffle and a cistern $5d_x$ long, the level of the cistern being such as to give $d_x=1.1d_3$. The side expansion which starts from the baffle, has a splay of 1 in 5. The sides at canal bed level are designed to the hyperbolic curve (para 10).

(ii) *All American Canal Fall Type Design*—This design is not suitable for flumed conditions. The side contraction accentuates the distribution of discharge intensity, which continues, as such, even below the fall, where central jetting with severe side return flow obtains. This in turn gives harmful scour, tending to undermine the pavement near the sides. Modifications such as lengthening pavement, gentler side splay, floor blocks, etc., do not improve the performance to any appreciable extent.

Fig. 9. (iii) *Parabolic or Flat Glacis Type Design*—The design of this type which can work well enough is shown in Figure 9. It consists of a 2:1 or parabolic glacis which meets the pavement at a level such as to give $d_x=1.1 d_3$. The length of pavement is $7d_x$. As many as three staggered rows of blocks $d_3/6$ high are required to fan out the flow. The uppermost row is at a distance of $3d_2$ from the toe of the glacis; the remaining two rows spaced 1.25 times the block height apart. A deflector $1/10d_x$ high serves to protect the pavement by deflector action. The blocks in a row may be spaced their height apart with advantage.

(iv) *Vertical Type*—This type is highly unsuitable for flumed conditions. All efforts to improve the performance of this type design, failed. Conditions of bed scour could be controlled but those of very disturbed surface flow and consequent wave action could not be checked. In fact four rows of blocks were necessary to check bed erosion but they increased wave action and turbulence.

Remarks—Of all the four types, the cheapest design which is recommended for its performance is the baffle type design.

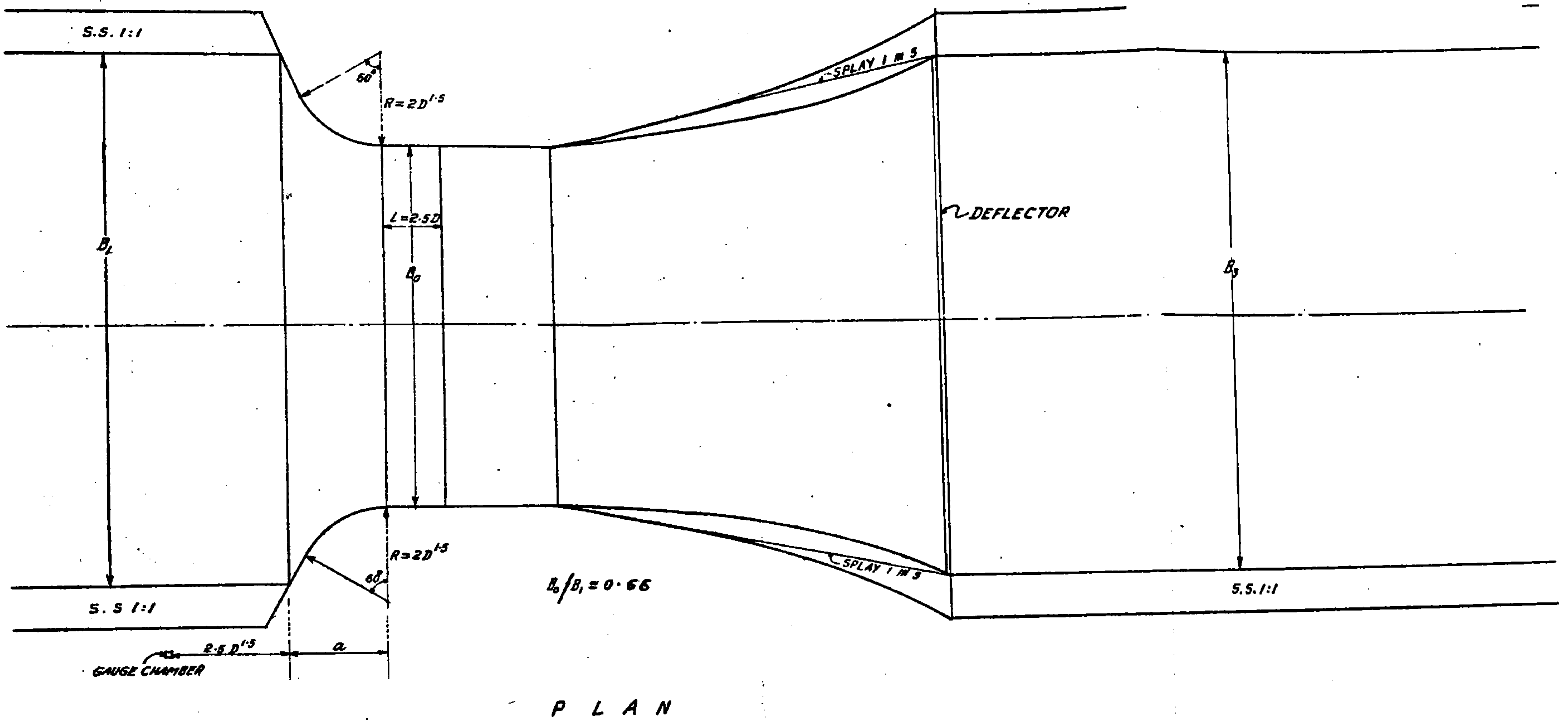
A glacis type design, though working satisfactorily is comparatively costlier in initial expense, and is next to the baffle type, in performance.

12. Group II—High Discharges and Low Falls

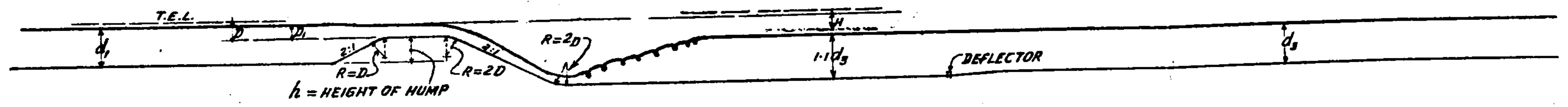
(i) *Baffle Type*—With clear overfall conditions, this type works very well.

(ii) *All American Canal Fall Type*—This, as mentioned in para 11 above, does not work satisfactorily.

Fig. 10. (iii) *Glacis Type Design*—When the fall is drowned, the baffle type fails and under these conditions this type design—Figure 10—works well enough. The design consists of a 2:1 glacis below the crest followed by a pavement $7d$ long. The level



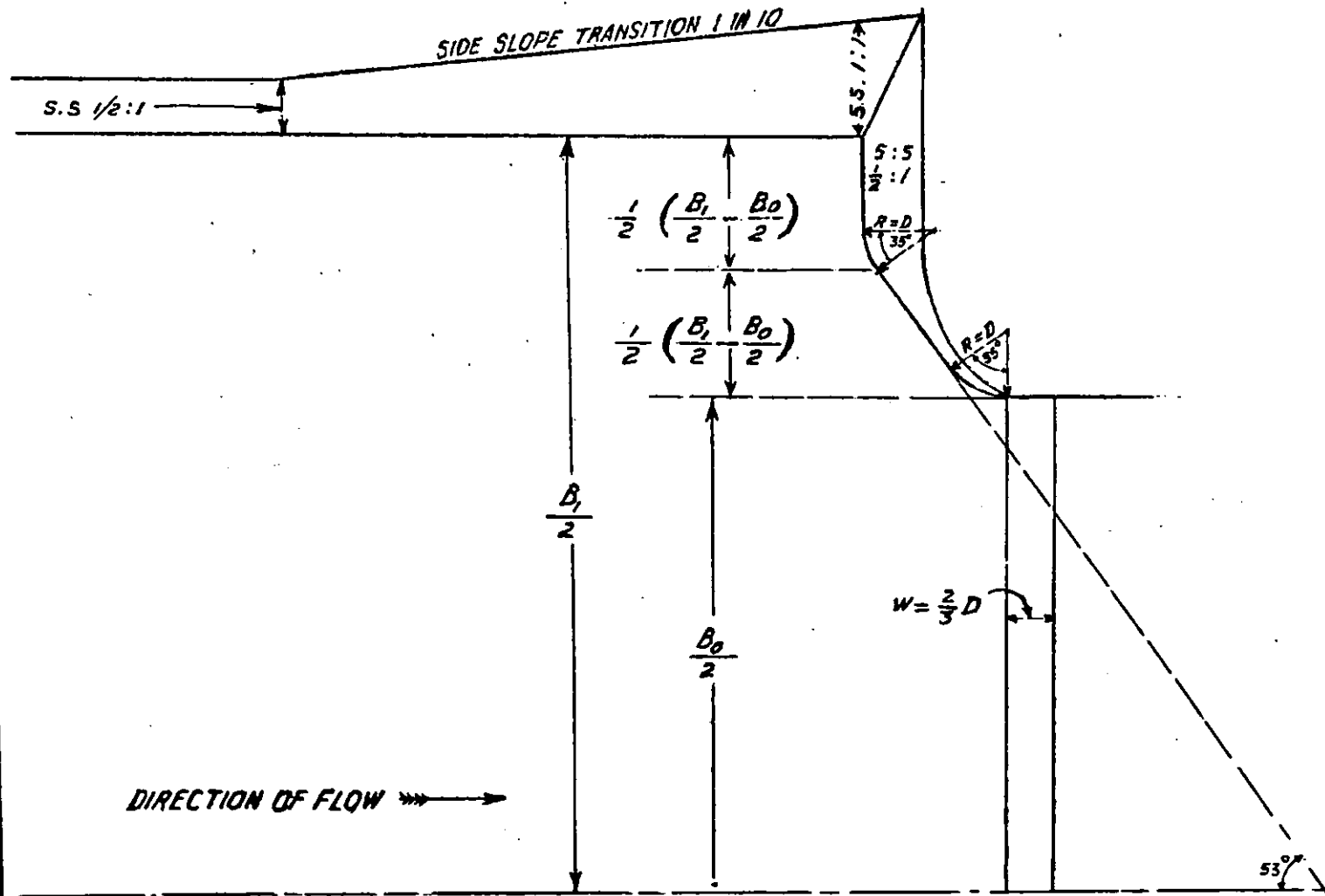
P L A N



LONGITUDINAL SECTION

FIG. 5 (A)
STANDARD TRANSITIONS OF A FLUMED METER FALL
SCALE, 1" = 20'

COMP. BY: *Handwritten signature*
TRACED BY: NAIK R.S. 26-11-51



INDEX :-

- B_1 = U/S CANAL WIDTHS
- B_0 = WIDTH OF CREST
- D = HEAD OVER THE CREST.
- W = CREST LENGTH = $\frac{2}{3} D$
- $\frac{B_0}{B_1} = 0.66$

FIG. 6
SHOWING U/S SIDE TRANSITION DESIGN
FOR A FLUMED FALL.

COMP. BY: Munesingh
TRACED BY: HAIK R.S. 5-11-51

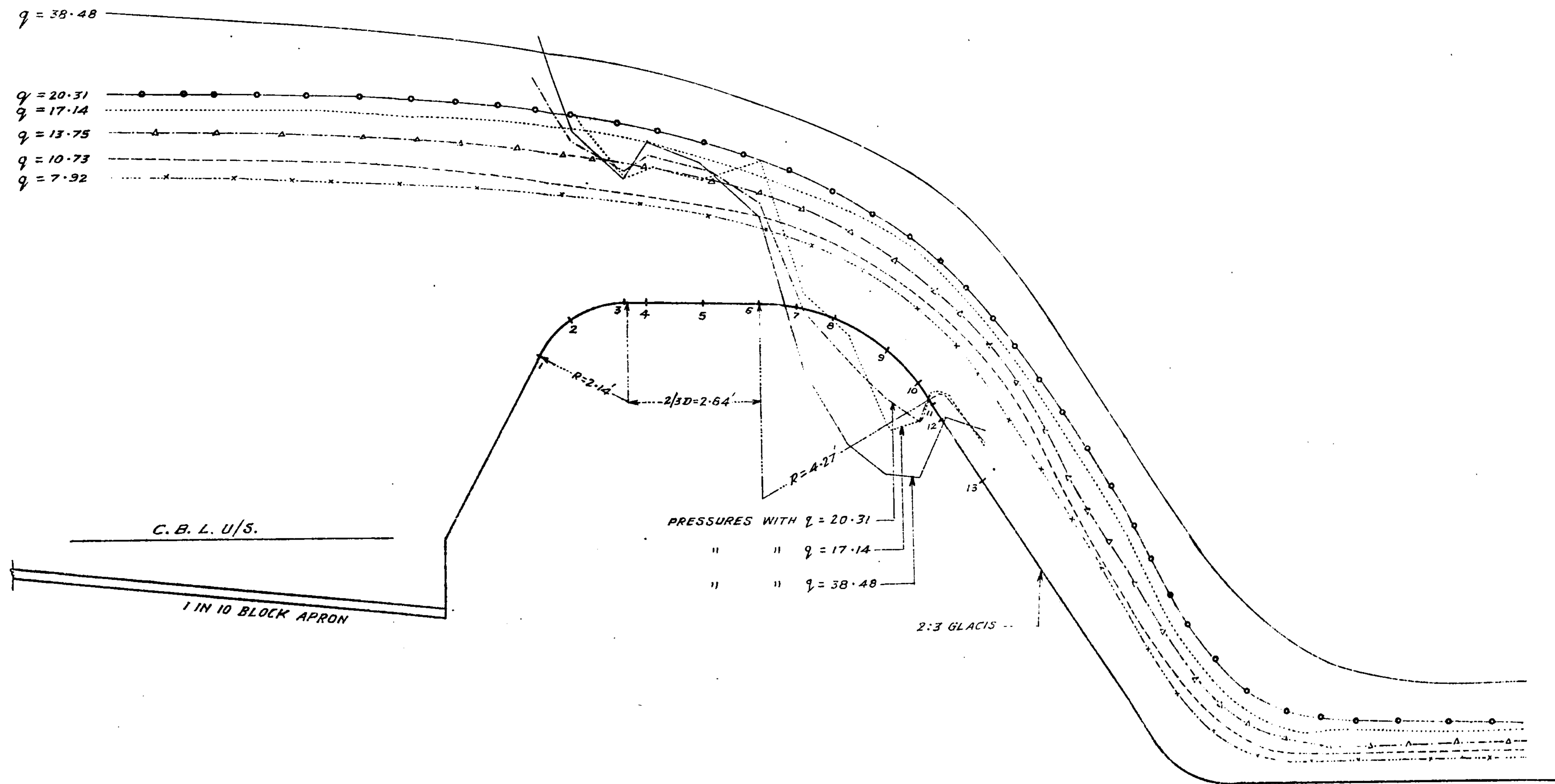
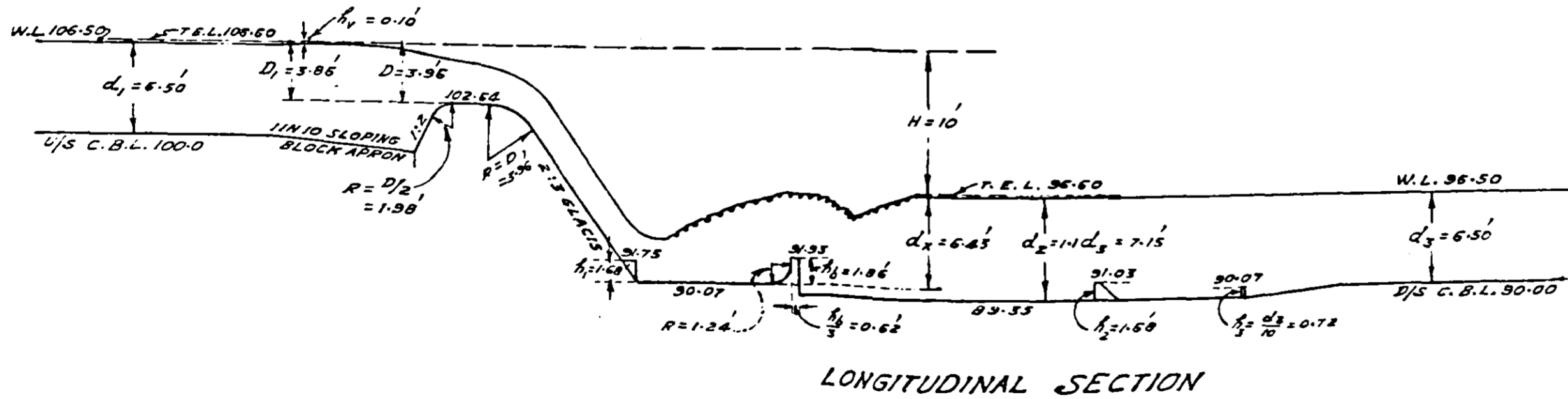
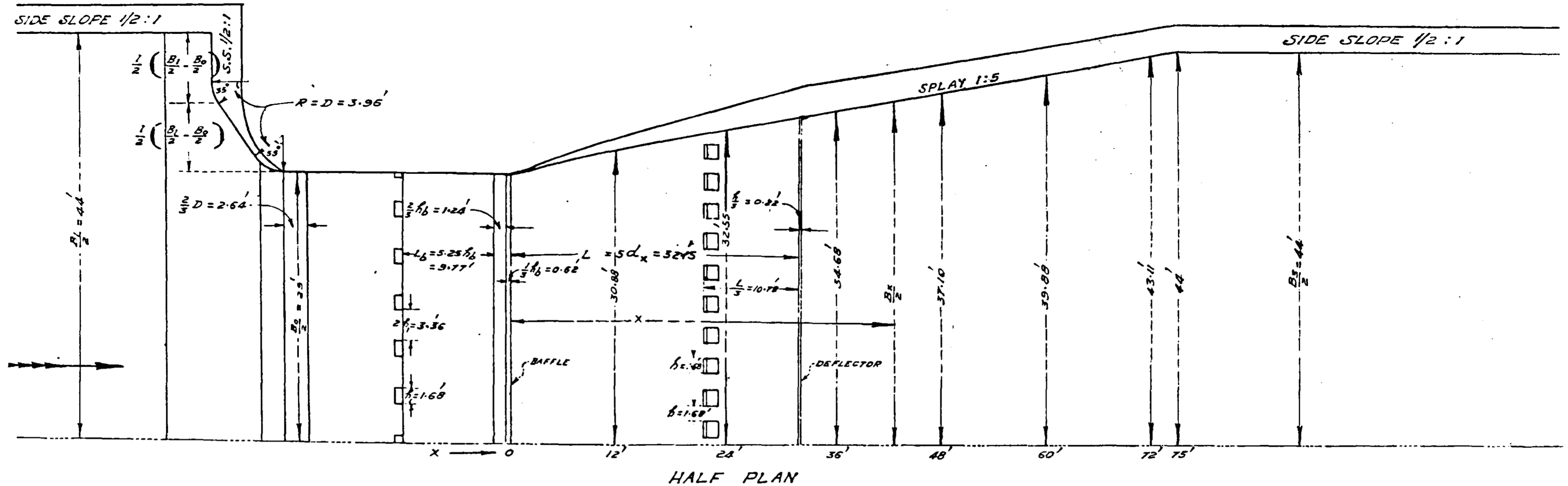


FIG.-7
 SHOWING WATER PROFILES & PRESSURES
 FOR
 DIFFERENT DISCHARGE INTENSITIES
 1/7 MODEL OF WEIR FOR CANAL FALLS.
 SCALE - 1" = 2 FT.

COMP. BY *M. S. S.*
 TRACED BY SAPHAL.

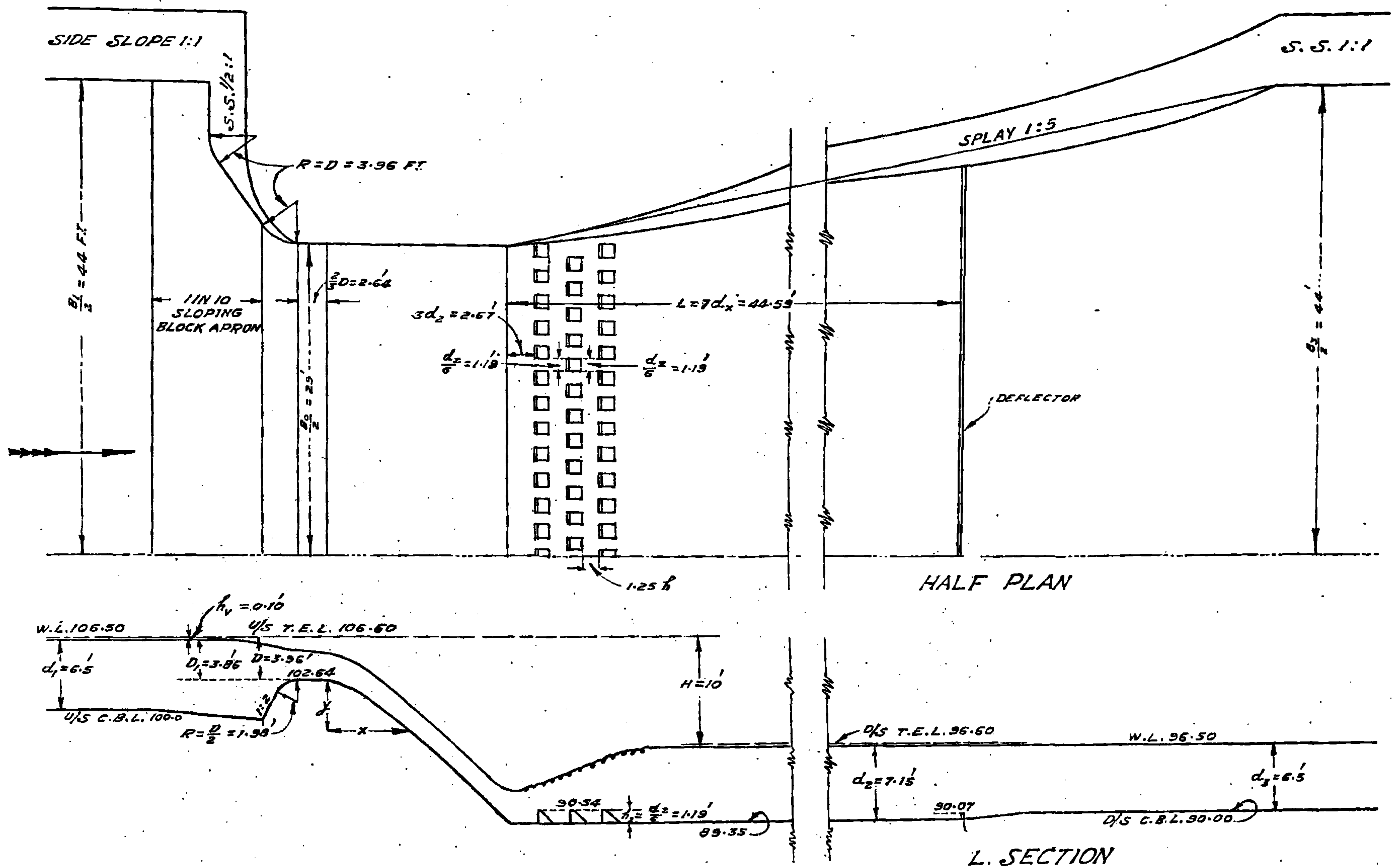


INDEX :-

- $q_0 = \frac{Q}{B_0} = 25.86 \text{ CUSECS/FT.}$
- $B_1 = 1/5 \text{ CANAL WIDTH} = 88 \text{ FT.}$
- $B_0 = \text{WIDTH OF CREST OR FLUMED WIDTH IN THROAT} = 58$
- $B_2 = 2/5 \text{ CANAL WIDTH} = 88 \text{ FT.}$
- $d_1 = 1/5 \text{ DEPTH OF WATER} = d_3 = 2/5 \text{ DEPTH OF WATER} = 6.5$
- $d_c = \text{CRITICAL DEPTH} \left(\frac{q_0^2}{g} \right)^{1/3} = 2.75 \text{ FT.}$
- $d_2 = 0.11 (q_0)^{0.89} (H_x)^{-0.38} = 0.89; h_b = \text{HT. OF BAFFLE} = d_c - d_2 = 1.86$
- $L_b = \text{LENGTH OF BAFFLE PLATFORM} = 5.25 h_b = 9.77 \text{ FT.}$
- $d_2 = \text{DEPTH OF CISTERN} = 1.1 d_3 = 7.15 \text{ FT.}$
- $d_x = \text{DEPTH OF } 2/5 \text{ W.L. ON BAFFLE PLATFORM} = 0.73 (q_0)^{0.52} (H_x)^{0.21} = 6.43$
- $L = \text{LENGTH OF PAVEMENT} = 5 d_x = 32.15 \text{ FT.}$
- $B_x = \text{WIDTH OF PAVEMENT AT DISTANCE 'X' FROM END OF BAFFLE} = \frac{B_0 B_2 L}{L B_2 - (B_2 - B_0) X}$
- $h_1 = \text{HEIGHT OF GLACIS BLOCKS} = \frac{2}{3} (0.85)^{-1} d_x = 1.68 \text{ FT.}$
- $\text{SPACING OF GLACIS BLOCKS} = 2 h_1$

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FIG. 8
BAFFLE TYPE DESIGN OF FLUMED FALL
 $Q = 1,500 \text{ CS.}; H = 10 \text{ FT.}$
SCALE $1'' = 10 \text{ FT.}$

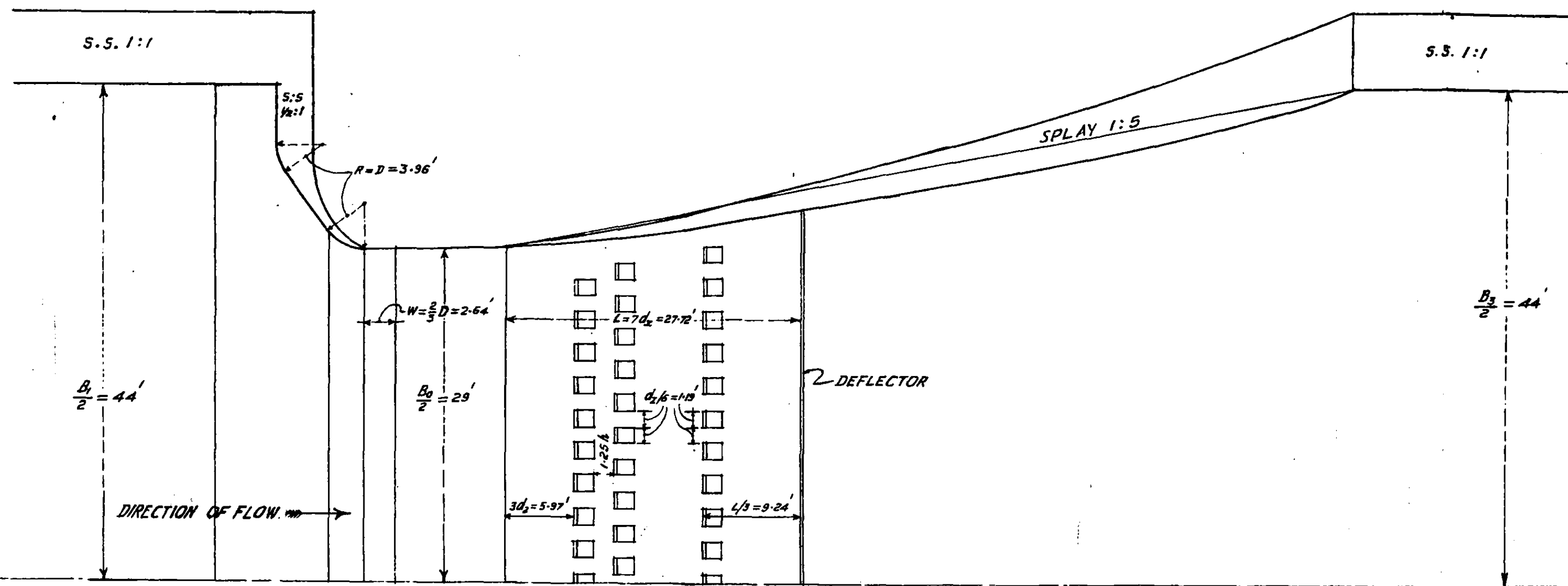


$B_1 = \text{U/S CANAL WIDTH} = 88$; $B_2 = \text{D/S CANAL WIDTH} = 88$ FT.
 $d_1 = \text{U/S DEPTH OF WATER} = 6.5$ FT.; $d_2 = \text{D/S DEPTH OF WATER} = 6.5$ FT.
 $B_0 = \text{FLUMED WIDTH OF CANAL} = 59$; C ASSUMED = 3.2
 $D = \text{HEAD OVER THE CREST} = \left(\frac{Q}{C}\right)^{1/2} = 3.96$; $q_0 = \frac{Q}{B_0} = 25.86$ CUSECS/FT.
 $h_v = \text{HEAD DUE TO VELOCITY OF APPROACH} = 0.10$ FT.
 $D_1 = D - h_v = 3.96 - 0.10 = 3.86$
 $d_2 = \text{D/S DEPTH OVER PAVEMENT} = 1.1 d_1 = 7.15$ FT.
 $W = \text{CREST LENGTH} = \frac{2}{3} D = 2.64$
 GLACIS FORMULA AS $x = v_c \sqrt{\frac{z}{2}} \cdot \sqrt{y} + y$
 $L = \text{LENGTH OF PAVEMENT} = 7 d_x = 44.59$
 $d_x = 0.73 q_0^{0.52} H^{0.21} = 6.37$ FT.
 HEIGHT OF BLOCKS: $h = \frac{d_x}{6} = \text{SPACING BETWEEN BLOCKS} = \text{WIDTH OF BLOCKS}$
 DISTANCE BETWEEN ROWS = 1.25 ft
 $h_3 = \text{HEIGHT OF DEFLECTOR} = \frac{d_x}{10}$

FIG. 9
GLACIS TYPE DESIGN (FLUMED)

$Q = 1,500$ CS., $H = 10$ FT.
 SCALE $\frac{1}{4}'' = 10$ FT.

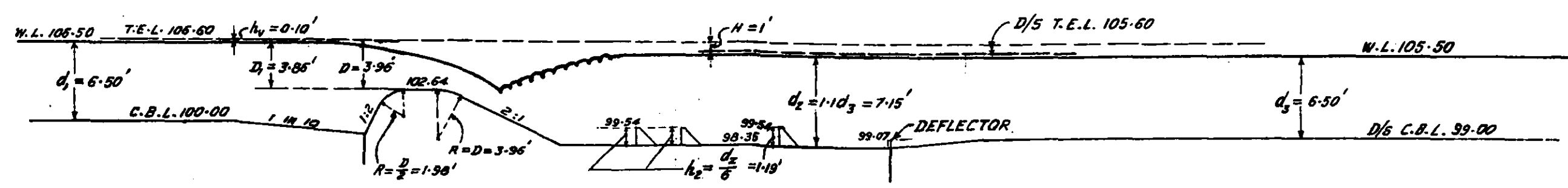
COMP. BY - Munsingh
 TRACED BY - PATRIE G.R.
 12-11-51.



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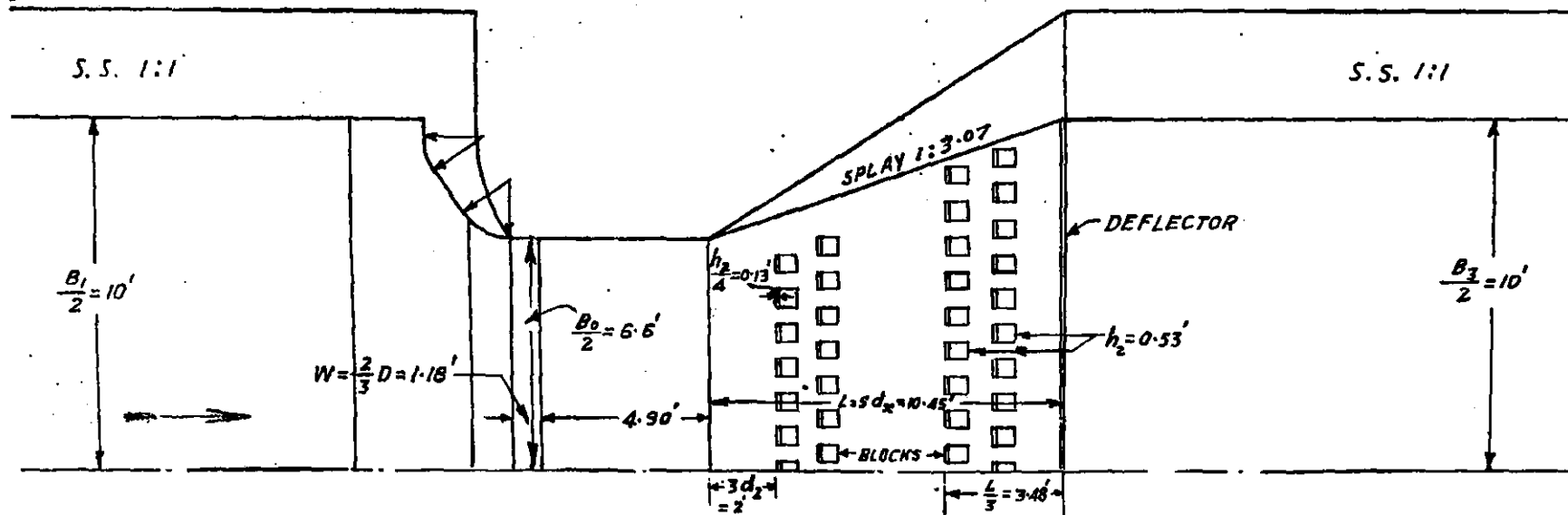
- B_1 = U/S CANAL WIDTH, d_1 = U/S DEPTH OF WATER
- B_0 = WIDTH OF THE THROAT
- B_2 = D/S CANAL WIDTH, d_2 = D/S DEPTH OF WATER
- Q_0 = DISCHARGE PER FT OF WEIR = Q_0/B_0
- D = HEAD OVER THE CREST = $(9.1/c)^{2/3}$
- h_v = HEAD DUE TO APPROACH VELOCITY = $\frac{v_a^2}{2g}$
- D_1 = ACTUAL HEAD OVER WEIR = $D - h_v$
- d_2 = D/S DEPTH OVER PAVEMENT = $1.1 d_2$
- $d_x = 0.73 Q_0^{0.52} H^{0.21}$
- L = LENGTH OF PAVEMENT = $7 d_x$
- W = WEIR CREST LENGTH = $2/3 D$
- h_2 = HEIGHT OF BLOCKS = $d_2/6$ = WIDTH = SPACING BETWEEN BLOCKS
- SPACING BETWEEN ROWS = $1.25 h_2$
- $d_2 = 0.11 Q_0^{0.89} H^{-0.55} = 1.99'$

HALF PLAN.



LONGITUDINAL SECTION.

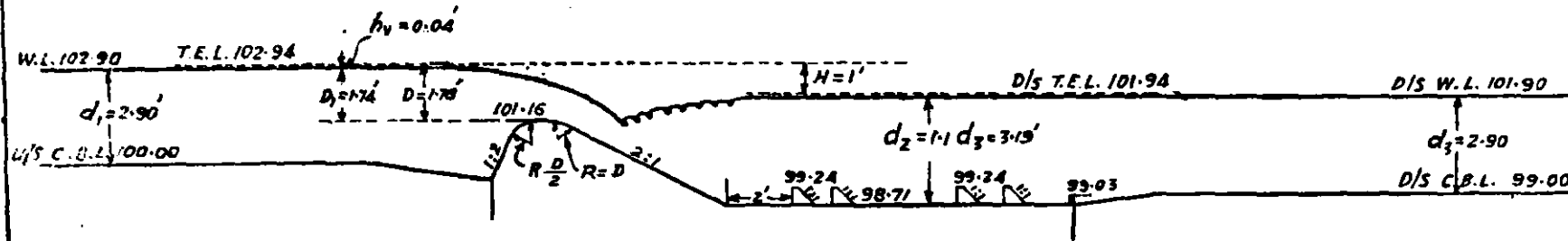
FIG. 10
STRAIGHT GLACIS (2:1) TYPE FLUMED FALL
Q = 1500 C.S., H = 1 FT.
SCALE. 1" = 10'



HALF PLAN

INDEX

$Q_0 = \frac{Q}{B_0} = \frac{100}{13.2} = 7.58$ CUSECS/FT.
 $B_1 =$ U/S CANAL WIDTH = 20'
 $d_1 =$ U/S DEPTH OF WATER = 2.9'
 $B_0 =$ FLUMED WIDTH OF CANAL = 13.2'
 $B_3 =$ D/S CANAL WIDTH = 20'
 $d_3 =$ D/S DEPTH OF WATER = 2.9'
 $W =$ CREST WIDTH = $\frac{2}{3} D = 1.18$ '
 $D =$ HEAD OVER THE CREST = $(\frac{Q_0^2}{g})^{2/3} = 1.76$ '
 $D_1 = D - h_v =$ DEPTH OF FLOW ABOVE WEIR CREST = 1.74'
 $h_v =$ HEAD DUE TO APPROACH VELOCITY = 0.04
 $d_x =$ D/S W. L. - PAVEMENT LEVEL = $1.1 d_3 = 3.19$ '
 $d_x =$ DEPTH ON PAVEMENT NECESSARY TO BALANCE THE S.W. = $0.73 Q_0^{0.52} H^{0.21} = 2.09$ '
 $d_2 = 0.11 (Q_0^{0.89}) H^{0.35} = 3.66$ '
 $h_2 =$ HEIGHT OF BLOCKS = $\frac{d_x}{6} = 0.53$ = WIDTH = SPACING BETWEEN BLOCKS
 $L =$ LENGTH OF PAVEMENT = $5 d_x = 10.45$ '
 SPACING BETWEEN TWO ROWS OF BLOCKS = $1.25 h_2$



LONGITUDINAL SECTION

FIG. II
 FLUMED STRAIGHT GLACIS TYPE FALL
 $Q = 100$ CUSECS, $H = 1'$
 SCALE: 1" = 5'

of the pavement is such as to give $d_x = 1.1 d_s$. Two staggered rows of floor blocks $\frac{d_x}{5}$ high fixed below the toe of fall, starting at a distance of $3d_x$ from it, help to fan out the flow. Spacing of rows is 1.25 times block height, whereas spacing of blocks in a row is a block height. A third row of floor blocks at $1/3$ rd the length of pavement, i.e., $7/3 d_x$ from the deflector is required to check the spirality of the roller below the deflector.

(iv) *Vertical Type*—For same reasons as explained in para 11, this type is unsuitable.

Remarks—Under clear overfall conditions a baffle type design is recommended whereas under drowned conditions the glacis design of the type shown in Figure 10 may be adopted.

Fig. 10.

13. Group III—Low Discharges and High Falls

(i) *Baffle Type*—This design works satisfactorily and may be further cheapened by reducing the length of the pavement below the baffle to $3d_x$ in cases of low discharges. This pavement length is adequate to hold the secondary standing wave on *pukka* bed.

(ii) *All American Canal Type*—Does not work.

(iii) and (iv) Same remarks as in para 12 (iii) and (iv) apply in this case.

Remarks—A baffle type design may be adopted.

14. Group IV—Low Discharges and Low Falls

(i) *Baffle Type*—When clear overfall exists this design is satisfactory.

(ii) *All American Canal Fall Type*—Does not work.

(iii) *Glacis type*—Under both 'clear overfall' and 'drowned' conditions the design as shown in Figure 11 works well.

Fig. 11.

The design consists of a 2:1 glacis followed by a pavement $5d_x$ long. Four rows of blocks $\frac{d_x}{6}$ high are intended to fan out flow. A deflector at the end of pavement is provided to create the beneficial bed roller.

(iv) *Vertical Type*—Does not work under flumed conditions.

Remarks—The glacis design and the baffle design, are well suited from performance point of view. However, from cost considerations, the glacis type as shown in Figure 11 may be adopted irrespective of whether the fall is clear or drowned.

15. Conclusion

(i) Under unflumed conditions, the All American Canal Fall type and the Baffle type designs are well suited under clear overfall conditions. The latter design is more expensive, but where the soil is easily erodible, it may be advisable to adopt the Baffle type.

Under drowned conditions a flat Glacis type design is recommended.

With low discharges below about 300 cusecs a vertical type design may be more economical irrespective of drop. The pavement may be kept $5(BD_x)^{1/2}$ with two rows of floor blocks and side pitching may also be provided.

(ii) Under flumed conditions the choice of design is limited only to the Baffle type and the flat 2:1 Glacis type. The other two designs, viz., the vertical drop type and the All American Canal type with 1:1 glacis are unsuitable. Under drowned conditions, Baffle type may be rejected and the flat Glacis type with three to four rows of blocks and a deflector adopted.

APPENDIX I

Experiments carried out with Unflumed designs of canal falls and their results

S. No.	Main Group.	Q in cusecs.	H in feet.	Design Type.	REMARKS.
1	2	3	4	5	6
1	I.e. Q 500 H 3 ft.	1,500	10	All American Canal Fall type	The performance of this design was satisfactory. By spacing the floor blocks closer (i.e., h_2 apart) as compared with the original design (which is $2h_2$ apart) scour could be reduced appreciably. Another row of the blocks fixed staggered with the one above it and spaced $2.5 h_2$ apart, reduced scour markedly. It, therefore, may be mentioned that where soil is easily erodible two rows of floor blocks may be used with advantage. This also increases the deflector action. The pavement which was $3dx'$ long was at R.L.91.33' higher. There thus appears a subsidiary drop at the end of pavement. In order to safeguard against any scour of bed downstream, when letting in water a loose stone apron sloping down at say 1 in 3 may be laid to join the pavement to C.B.L. downstream. Actually in the experiments sand piled up against the pavement even when no apron was laid and it entirely covered the apron when it was laid in another experiment.
2	Do.	1,500	10-65 with 10% retrogression	Do.	Though the scour was a little more than in experiment 1 above, yet it was within safe limits and the design thus was satisfactory.
3	Do.	1,500	5	Baffle type	The design consisted of a 2:3 glacis downstream followed by a baffle on the baffle platform, a cistern $4.5 dx$ long and a deflector $dz/10$ high. A couple of staggered rows of blocks, say $3' \times 1' \times dz/10$ high may be used in place of the deflector to facilitate unwatering. Though this worked well yet perhaps a continuous deflector may be better from construction point of view.
4	Do.	1,500	5-65 with 10% retrogression	Do.	Performance was quite satisfactory.
5	Do.	500	5	Vertical fall	This design initially consisted of a pavement $5(HD_1)^{1/2}$ long, the level being worked out by using the U. P. formula $D_c = 1/6(H.D_1)^{2/3}$. Violent eddies and boils downstream of the deflector and the severe side action were obtained. Floor blocks were tried with pavement length of $5(HD)^{1/2}$ and $7.94(H.D)^{1/2}$. But results were not satisfactory. To improve the conditions submerged longitudinal vanes were also tried but to no effect. The inherent tendency of side erosion persisted though bed scour could be checked.
6	I	500	5-65 with 10% retrogression	Do.	Side action was more severe than that in experiment 5 above.
7	Do.	500	5	Baffle type	The design was of the same type as described in expt. 3 above. This design worked very well. The length of <i>pukka</i> work below the downstream end of crest in this design was 31.22 ft. as against 30.24 ft. in the vertical fall design of expt. 5. The design therefore was almost as cheap as the vertical type design and much better in performance. This will also save recurring expenditure.
8	Do.	500	5-46 with 10% retrogression	...	Design of expt. 7 was tested under 10 per cent retrogression. It worked satisfactorily.

APPENDIX I—contd.

S. No.	Main Group.	Q in cusecs.	H in feet.	Design Type.	REMARKS.
1	2	3	4	5	6
9	II	1,500	3	All American Canal Fall type	Worked satisfactorily. Similar remarks as in expt. 1 also apply in this case as regards height of blocks, number of rows and pavement level.
10	Do.	1,500	3.65 with 10% retrogression	Do.	Design of experiment 9 was tested under 10 per cent retrograded conditions. It worked well.
11	II Q 500 H 3' (continued)	1,500	3	Baffle type	The design was prepared on the same lines as in experiment 3. It worked well.
12	Do.	1,500	3.65 with 10% retrogression	...	Ditto.
13	Do.	1,500	1	Vertical type	The design did not work well since flow conditions were unstable. Flow immediately downstream of fall swings from side to side resulting in severe side embayment. The design was abandoned.
14	Do	1,500	1	Glacis type	A 2 : 1 glacis type design with a pavement length = 3dx at a level 1.1d ₃ below downstream W. L. gave good results as regards both scour and flow conditions. Attempts to cheapen the design by adopting a 1 : 1 glacis slope gave rise to strong boils all over the surface above deflector. Performance of 2 : 1 glacis design was much better. Lengthening the pavement to 5 dx with 1 : 1 glacis did not improve flow conditions.
15	Do.	500	3	Baffle type	The design was prepared on the lines described in expt. 3. It worked well.
16	Do.	500	3.46 with 10% retrogression in d's W.L.	...	Worked well.
17	III	100	10	Vertical type	The design with pavement $5(HD_1)^{1/2}$ long worked well with two rows of staggered floor blocks dz/6 high. Upper row was at 1/3rd the pavement length above the deflector and the 2nd row $2.5 \times dz/6$ below it.
18	Do.	100	5	Do.	In this design the length of the pavement and its level were worked out by the U. P. Research Station formulae $L = 5 (H.D_1)^{1/2}$ and $D_c = 1/6 (H.D_1)^{1/2}$ It was tested with a deflector. To improve its performance two rows of staggered blocks dz/6 high were used as in experiment 18 above. The distance between the two rows was, however, 1.25 times block height. Though improved, flow conditions were not good enough being very rough. Increasing the pavement to $10(H.D_1)^{1/2}$ improved the performance.

APPENDIX I—concl'd.

S. No.	Main Group.	Q in cusecs.	H in feet.	Design type.	REMARKS.
1	2	3	4	5	6
19	III	100	5	All American Canal Fall type	The design gave results much better than in the vertical fall design of experiment 18. It is also as cheap as the vertical type design or perhaps cheaper.
20	Do.	100	5	Baffle type	Worked well with oistern length 4.5 dx below the baffle. The design is more expensive as compared with the one of experiment 19 but is much better in performance and is better suited for localities where soil is easily erodible and retrogression may be expected.
21	IV	100	3	All American Canal Fall type	Worked well with two staggered rows of floor blocks.
22	Do.	100	3.29 with 10% retrogression	Do.	Worked well.
23	Do.	100	3	Vertical type	Scour conditions were good enough with pavement $5(HD_1)^{1/2}$ and two rows of floor blocks lengthening the pavement.
24	Do.	100	3.29 with 10% retrogression	Do.	Side erosion is an inherent defect in the design, but for this group, it may be guarded against by providing D. B. pitching.
25	Do.	100	1	Do.	The design with pavement $5(H.D_1)^{1/2}$ long was tested with and without blocks at the toe of fall. Side action and intense eddies and boils over the deflector occurred. Lengthening the pavement to $10(HD_1)^{1/2}$ improved conditions. Without blocks the flow was unstable.
26	...	100	1	Glacis type	The design consisted of a 1 : 1 glacis with pavement 3 dx' long. It worked well with glacis and floor blocks and a deflector. Pavement level was such as to give $d_2=1.1 d_3$. Considering the <i>pukka</i> work length, this design is cheaper than the one of experiment 25 above.

APPENDIX II

Experiments carried out with Flumed designs of canal falls and their results

S. No.	Main Group.	Q in cusecs.	H in feet.	Design type.	REMARKS.
1	2	3	4	5	6
1	I	1,500	10	All American Canal Fall type fluming 66%	Design did not work satisfactorily as central jetting with severe return flow at sides developed on and below the rigid structure. Pavement was undermined at the junction of forward and return flow. Measures to improve the performance viz., additional floor blocks, flattening of glacis, increasing pavement length, gentler splay than 1 in 5, were tried but no improvement was obtained.
2	Do.	1,500	10	Baffle type fluming 66%	This design with modified upstream transitions, 2:3 glacis and a short oistern length of 5 dx and a deflector worked well except for the roller action below the deflector which was spiral, tending to take away the sand from the sides to the centre. This tended to expose the curtain wall near the sides. A row of floor blocks 2/9 dx' high, spaced their height apart at 5/3 dx from deflector, did away with the spirality of the roller. Results were satisfactory.
3	Do.	1,500	5	Do.	This design was a standard proportional meter design, prepared by the C.D.O. CWINC. It had long upstream transitions and a pavement 8 dx long. Floor blocks were provided in the original design. According to the experience of this type which was also confirmed by tests, no floor blocks were required. Only glacis blocks would reduce wave wash. Working of the design was satisfactory. The design was, however, very expensive and efforts were made to economise it.
4	Do.	1,500	6.25 with 25% retrogression	Do.	Worked satisfactorily.
5	Do.	1,500	4	Glacis type fluming 66%	Like the baffle design of experiments 3 & 4, this was also a standard design prepared by the C.D.O. CWINC. This design worked satisfactorily with minor modifications, such as shifting position of floor blocks, doing away with the flat loose block apron etc. The design was, however, expensive, the glacis being 3:1 and pavement 10 dx long, besides long upstream transitions. Efforts were made to economise this design. Finally with upstream sharp transitions, as explained in the text of the note, 2:1 glacis followed by pavement 7 dx long worked well with 3 staggered rows of floor blocks dx/6 high below the toe of glacis and one near the deflector.
6	Do.	1,500	5.00 with 25% retrogression	...	The original design with minor modifications was tested under retrograded conditions. It worked satisfactorily.
7	Do.	500	5	Vertical fall fluming 75%	Like the Baffle and Glacis type above this was a design prepared by the C.D.O. CWINC. It gave highly unsatisfactory performance. Being a cheap design, all efforts were made to improve the performance. Though scour could be brought under control by using floor blocks in curved alignment, yet these very blocks, caused very turbulent flow conditions and side action. The general performance was not satisfactory enough. This type design was considered unsuitable for flumed conditions.

APPENDIX II—contd.

S. No.	Main Group.	Q in cusecs.	H in feet.	Design type.	REMARKS.
1	2	3	4	5	6
8	I	500	5-46 with 10% retrogression	Vertical design	The original C.D.O. design with three curved rows of staggered blocks, which gave controlled bed scour conditions only, was tested under retrogression. A fourth row of blocks was found essential, which however added to the rough flow. Cumulative performance was unsatisfactory.
9	II	1,500	3	Baffle type	The modified baffle type design was tested. The fall was drowned. The design did not work satisfactorily. Periodic swells and wave action obtained. When the downstream water level was lowered the performance immediately improved; indicating that the design is not suited for drowned conditions. Efforts to improve its performance failed.
10	Do.	1,500	3	Glacis type	The final design that was evolved in this type had a 2:1 glacis followed by a pavement 7 dx long at a level such that $dz = 1.3 d_3$. Three rows of floor blocks $dz/6$ high were fixed at $3d_2$ from the toe of glacis in addition to glacis blocks. This design worked satisfactorily.
11	Do.	1,500	1	Do.	The final design that gave satisfactory results had a 2:1 glacis followed by a pavement 7 dx long at a level giving $dz = 1.1d_2$; two rows of staggered floor blocks $dz/6$ high starting from a distance $3d_2$ from the toe of the glacis, a row of floor blocks at $L/3$ i.e., $7/3 dx$ from deflector, and a deflector $dz/10$ high. Deflector action was quite good and scour central and within safe limits.
12	III	100	10	Baffle type	The modified design worked well with cistern 3 dx long below the baffle. This short cistern length is adequate for small discharges.
13	IV	100	3	Glacis type	The design was of the same type as that in experiment 10 above except that the pavement length of 5 dx was found insufficient. This design worked satisfactorily.
14	Do.	100	3	Vertical type	As in experiments 7 and 8 the design was found unsuitable.
15	Do.	100	1	Glacis type	The design had a 2:1 glacis followed by a pavement 5 dx long, two rows of staggered blocks $dz/6$ high starting from distance $3d_2$ from the toe of glacis, and two rows of floor blocks starting from $L/3$ i.e., $5/3 dx$ upstream of the deflector. The spacing of rows was 1.25 times block height apart, while spacing of blocks in a row was one block height apart.
16	Do.	100	1-29 with 10% retrogression	Do.	Design of experiment 15 was tested under retrograded conditions. It worked well.
17	Do.	100	1	All American Canal type	} Did not work.
18	Do.	100	1	Vertical drop	

17. OTHER WORKS

25. SERVICE WATER SUPPLIES TO THE TATA IRON AND STEEL CO., JAMSHEDPUR, FROM THE SUBERNAREKHA RIVER

Introduction

Water supplies for the Tata Iron and Steel Co., are taken entirely from the Subernarekha river. A submersible pick-up weir with top at R.L. 394.34 was constructed across the river some time in 1912. The alignment of the weir—as shown in Figure 1—follows the highest rock outcrop in the bed of the river. Water is pumped from the river just upstream of the pick-up weir by means of pumps located in the River Pump House as shown in the figure. The suction main is a 4 ft. diameter cast iron pipe with its open end facing downstream. Pukka arrangements for sluicing the silt deposits from the upstream of the weir are not provided, there being installed only 2 number 24"×18" gate type sluice valves, which serve no useful purpose. The average daily requirement of water for the Steel Plant in 1947 was round about 14 m.g.p.d. or about 26 cusecs, which has since increased to about 50 cusecs. Further increase is proposed in the water supply to about 72 m.g.p.d. or about 103.5 cusecs.

The pumps can be worked continuously all the year round, except for the danger of heavy charge of sand being drawn during the flood season, which causes heavy wear and tear of the pump casings and impellers, thus reducing their life and efficiency. Hence, the pumps are being worked at present during flood only to meet the minimum requirements. In spite of this precaution, in the monsoon, about 5,000 cft. of sand collects daily in the main common suction chamber, inside the Pump House, and has to be promptly removed.

The case was referred to this Station in 1947 for its considered opinion and recommendations, *inter alia*, on the following points.

- “(a) River Pumping Station: Can the location of the Pump House and Pumping set be considered as being satisfactory in all respects?”
- “(b) Is there any sound and practical means for a suction main which would considerably reduce the silt troubles, the pumping units are subject to?”

After careful study of the case, the Station made the following recommendations.

“The first essential is, to locate the Intake near the downstream end of the concave bend; but, well upstream of the cross-over of the river Thalweg—so as to obtain the maximum benefit of the favourable river curvature. A suitable point near the existing Pump House was selected by the Station for the Intake on these considerations, as shown in dotted lines in Figure 1. Two counter-acting features reducing the efficiency of sand-exclusion, must, however, be borne in mind.

- (i) The effect of curvature is diminished in periods of low river by ponding of water caused by the weir; and
- (ii) in high floods, in spite of repositioning of the Intake, the bed sand thrown into suspension by turbulence may still be sucked in, to a certain extent, unless arrangements are made to tap the top filaments and to insure minimum turbulence.

In addition to correctly relocating the Intake, some additional arrangement to prevent the entry of sand in the suction pipe is, therefore, necessary.”

For this purpose, it was recommended to construct an Intake Tower, well stream-lined in plan to minimise turbulence and with an arrangement for tapping surface water at all river stages, on the model of one constructed for the Wazirabad Water Works at Delhi.

In view of the appreciable cost of the Intake Tower (about 6 lakhs of rupees) and lack of availability of funds, this whole scheme was held in abeyance for over three years. In the meanwhile, the sand trouble did not abate and if anything, the trouble in the Pump House and repair charges to the pumps very much increased.

At the end of 1949, the Company consulted Mr. K. B. Ray, R.I.S.E., Chartered Engineer, Bengal, to advise and investigate the possibility of minimising the flow of sand along with water through the Intake Pipe into the Suction Pump Chamber. Mr. Ray inspected the site and after careful study of the case made recommendations, an abstract of which is given below.

“In order to lower the high flood levels of the river, the capacity of the river should be increased by—

- (a) blasting the high rock bed to general bed level both upstream and downstream of the last four spans (to the left) of the bridge downstream,
- (b) removing all the projecting rocks in the river bed by blasting, and
- (c) blocking the deep ravines in the high banks of the river.”

Regarding the measures to reduce the sand entry, he made the following remarks:—

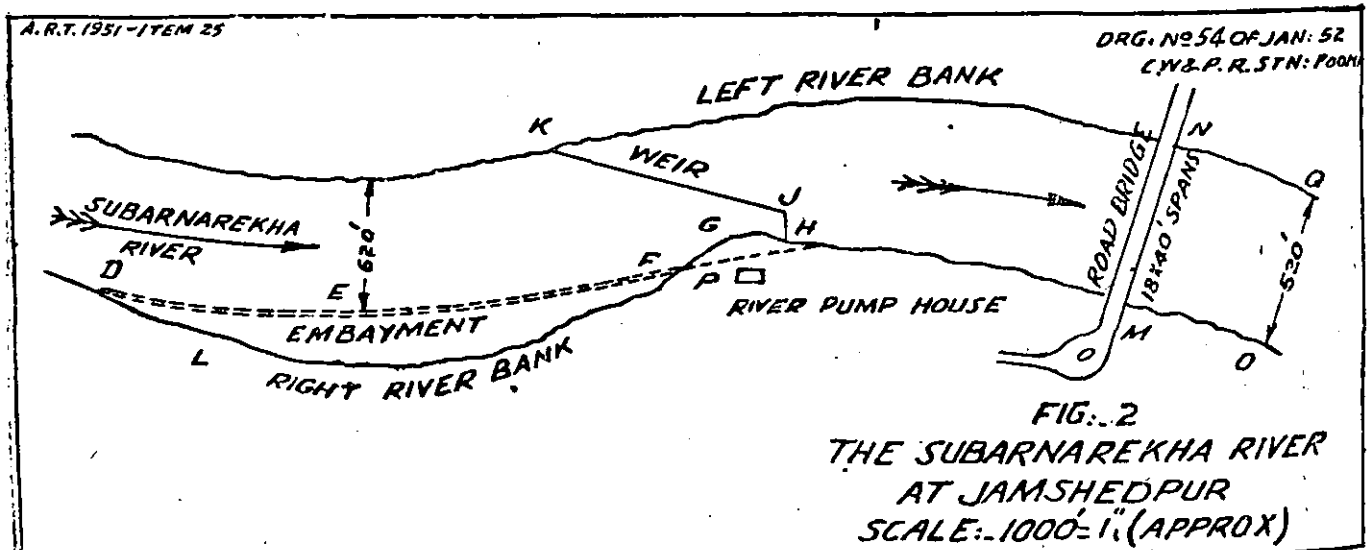
“The Management of the Steel Company, realising the gravity of the situation, were alert and they had wisely arranged for advice from Experts on the subject, as well as from the River Research Station, Poona. Suggestions received were an Intake Tower upstream of the River Pump House costing between Rs. 6 lakhs to 10 lakhs, and scouring sluices, etc., to reduce the sand at the Intake Pipes to a minimum. Engineers of the Steel Company visited the Intake Tower of the Wazirabad Pumping Station, Delhi, but this was found to be infeasible for the Subarnarekha, which carries coarser and much more sand than the Jamuna. The River Research Station, Poona, gave their opinion, that ‘sand inclusion by wrong positioning cannot be cured or mitigated by facing the inlets downstream’. Poona laid great emphasis on the fact, that ‘it is better to exclude the sand in the first place, than try to scour it away with partial or indifferent success later on’. Poona, therefore, recommended:

For a sand-free intake, the first essential is to locate the intake near the downstream end of the concave bend, so as to obtain the maximum benefit of the favourable river curvature.”

“I fully agree with this recommendation of the Poona River Research Station. My suggestions will serve to implement Poona’s recommendations.”

“By laying a lined permeable screen along a very flat curve of $1\frac{1}{2}$ followed by a straight from D to F, (Figure 2), the reduction in the velocity of flow in the portion of the embayment between the screens DEF and the existing right river bank DLF will cause deposit of silt, so that a new concave right river bank ending in a straight will form along DEF in one flood season.”

Fig. 2.



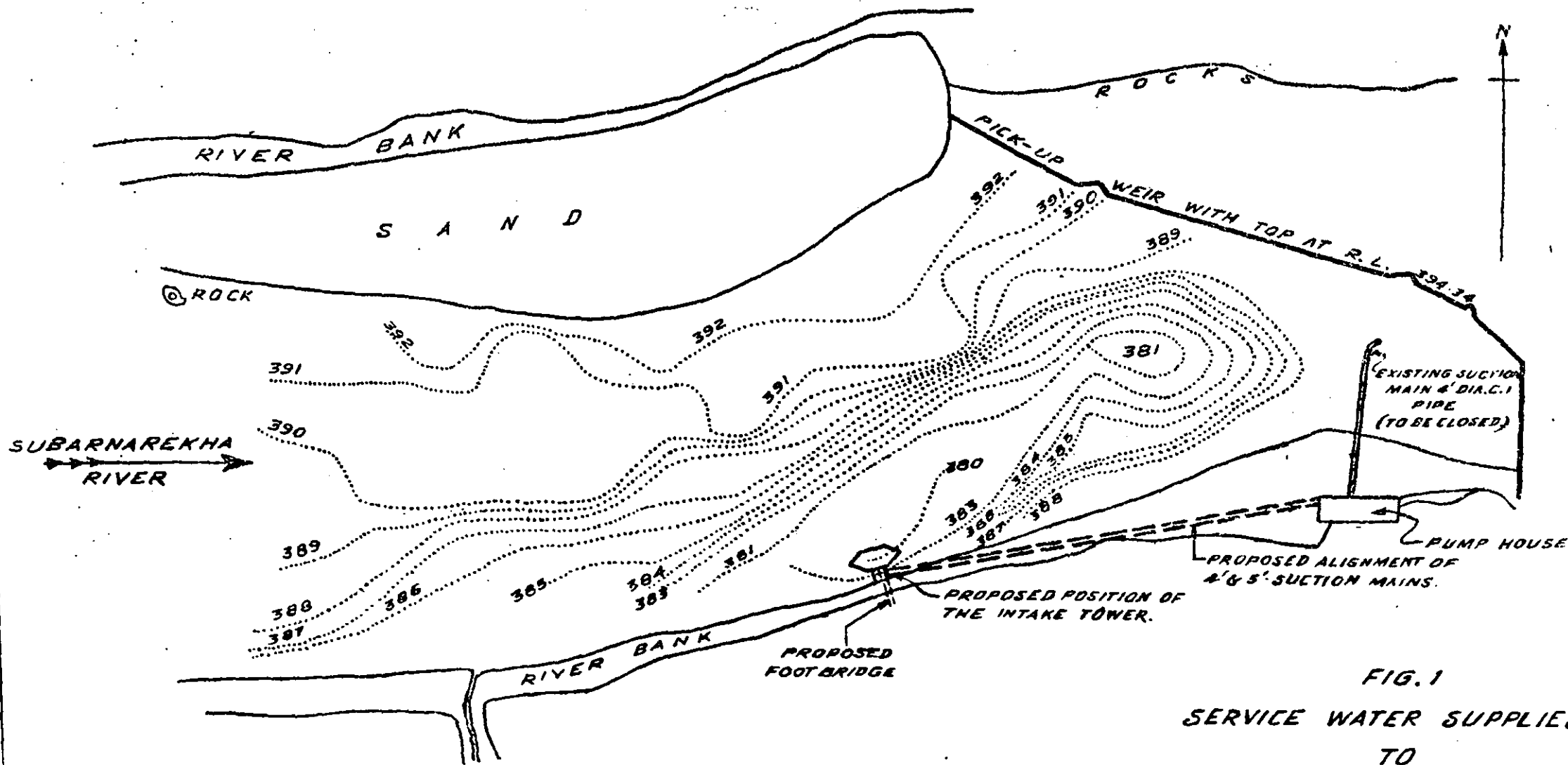


FIG. 1
 SERVICE WATER SUPPLIES
 TO
 THE TATA IRON & STEEL CO., JAMSHEDPUR
 FROM
 THE SUBARNAREKHA RIVER
 SCALE: 1" = 200'

NOTE:-
 CONTOUR SURVEY IN MAY AND JUNE 1947

COMP. BY
 TRACED BY: PARKHI
 8-1-52

"The projection FGM in the right river bank, in front of the River Pump House P, will be blasted out and made straight along FH. The River Pump House will then be located 'near the downstream end of the concave bend' and thus 'obtain the maximum benefit of the favourable river curvature', so that a 'sand-free intake' for the River Pump House will be ensured, as recommended by the Poona River Research Station."

"In addition to providing a silt-free intake for the River Pump House, there will be other advantages. River water will flow smoothly down the whole width of the channel in front of the River Pump House, so that its bed will be scoured down to a uniform low level. As a result of this the storage capacity of the Reservoir above the weir will be doubled."

The Management of the Company discussed these proposals with some of their officers and all of them were of the opinion that the scheme suggested by Mr. Ray was not going to give them sand-free intake, in view of the existence of the present weir in the vicinity of the Pump Intake. They further commented that this weir had no effective scouring arrangement and it caused the impounding storage reservoir near the Pump House to become filled up with sand. As the intake was very near the weir and at a very much lower level than the top of the weir, it naturally sucked the sand. They further believed that the conditions were not going to be altered or affected by flattening the curvature of the embayment as proposed by Mr. Ray.

With regards to Mr. Ray's recommendations for improving the discharge capacity and thereby lowering the flood levels in the river, their views were:

- (a) Removal of high bed rocks which masked the last 4 spans of the bridge downstream, was too costly.
- (b) Removal of all projecting rocks in the river bed would not improve the discharge capacity to an appreciable extent and the cost would not be commensurate with the benefit likely to be derived.
- (c) By plugging up the ravines, the spill-over would be prevented resulting in the possibility of an increased danger of flooding low lying areas further upstream.
- (d) The storage capacity of the reservoir above the weir would not be increased by any means to the extent indicated by Mr. Ray.

In March 1951, the Management once again sought advice of the Station and asked for measures to be adopted for preventing the sand entering the Pump Intake, in view of the recommendations made by Mr. Ray and their comments on the same.

Final Recommendations

In the considered opinion of the Station on Mr. Ray's recommendation, a change in the bank alignment will never affect the sand entry at the Intake in its present position as a substitute to the water tower scheme already suggested 3 years back. The present intake is so near the weir and so low near the bed that slight change of curvature effect, caused by screens and blastings would hardly alter the present conditions. The trouble of silt entry is due to two main factors, viz.,

- (i) the existing position of the Inlet Pipe relative to the convex side of the curve of the river bank; and

(ii) the existing level of the Inlet Pipe relative to the bed level of the river as shown by the following data ;

(a) 4 ft. C. I. Suction main is at R. L.	387-44
(b) Lowest bed level	383-00
(c) Low water level	390-00
(d) Crest level of the Dam	394-34.

The bed level and the low water level are pretty constant for a number of years. All the year round, the Inlet Pipe is practically at the bed level and sucks a considerable amount of sand and silt into the pumps, particularly during the period when the river is in floods and there is too much of sand and rolling material in turbulence in water.

Fig. 3. It is clear that unless the position of the Intake Pipe is changed to a site upstream of the Pumping Station so as to take off from the concave side of the bend (see Figure 3), the conditions cannot be improved ; because the present position of the pipe is such that it cannot but draw bed sand collected in the pocket along with the water as it is situated in the portion of the river downstream of the change over from the concave to convex curvature. The proposed intake proposed by the Research Station is located at a place where a deep channel has now been maintained due to diving flow which is associated with the concave curvature. There has been practically no change with bed levels near the vicinity of the proposed intake site as seen by the survey done in May and June 1951.

The main change in the site of the Intake Pipe will not however completely improve the matters. At present, the mouth of the Intake Pipe is placed at such a low position that it draws supplies from practically the bottom-most layers of the flow of water, which are impregnated with bed silt, all the year round, whatever the depth of water there may be in the river. Of course, during the flood period, there is maximum amount of silt and sand in the bottom layers and that is all drawn into the pumps. At the proposed site, the pumps will, of course, draw comparatively clearer supply as explained above. But, apart from changing the site of the Intake Pipe, it is also essential to so design the intake so that only surface water is drawn which may be free from sand throughout the year.

The design of Water Tower previously recommended in 1947, was to ensure complete freedom from sand entry into the pumps ; but as its cost has been considered exorbitant, the change in the design has now been made in order to make it economical and at the same time to be fairly efficient. The details of the design are given in Appendix I. It is expected that the arrangement now proposed will eliminate most of the sand entry into the pumps at a very reasonable cost and the present trouble will be over.

As regards the arrangement of deposited bed material, it seems to be a matter of secondary importance at present. A provision of falling shutters can be made later on, if found necessary. The first essential is to change the site of the Intake Pipe and provide Intake Tower Channel at the new site.

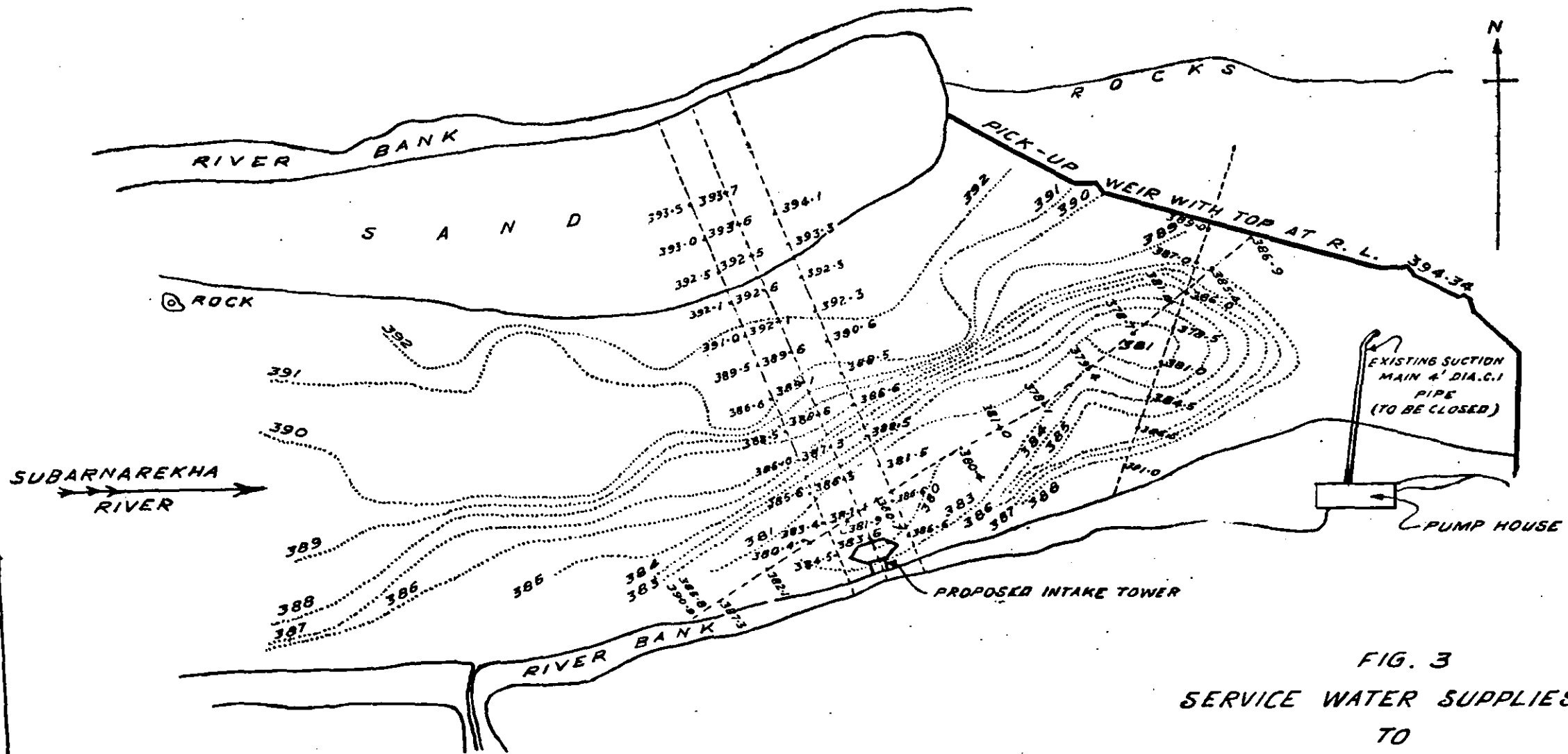


FIG. 3
SERVICE WATER SUPPLIES
TO
THE TATA IRON & STEEL CO., JAMSHEDPUR
FROM
THE SUBARNAREKHA RIVER
SCALE: 1" = 200'

- NOTE:-
- (1) ... CONTOUR SURVEY IN MAY AND JUNE 1947
 - (2) ... THE LEVELS ALONG SIDE THE SECTION LINES WERE OBSERVED FROM 30-5-51 TO 5-6-51.

COMP BY:-
TRACED BY:- PARKHI.
7-1-52

APPENDIX I.

DESIGN OF THE INTAKE TOWER FOR SUBERNAREKHA RIVER WATER WORKS AT JAMSHEDPUR

1. Introduction

Subernarekha is a flashy river and during monsoon, when there is heavy rainfall in its catchment area, the flood level at Jamshedpur rises to a very high peak, though its duration may last for a short time only. The daily gauges of the river at Jamshedpur for the last 10 years have been statistically examined and it has been brought forth clearly (Stt. No. 1 and Figure 4), Stt. 1. Fig. 4 that whereas the lowest W.L. is 391.0, the highest W.L. seldom goes above R.L. 424. In fact during the last 10 years, the water level has risen during the year on an average for—

20.75 days above R.L. 400,
15.13 days above R.L. 401,
11.45 days above R.L. 402,
8.54 days above R.L. 403,
1.66 days above R.L. 410, and
0.39 days above R.L. 421.

In case, if an attempt is made to draw sand-free supply from the river throughout the year, it will mean construction of a tower as high as R.L. 421.0, and at a very heavy cost. But as the above table shows that such peak gauges last for very short period only, it is proposed, as a compromise, to design a smaller tower which is more economical, but sacrifice the water supply for a few days during the flood period. The tower is accordingly designed with top level of 403.00 so that to draw supplies up to R.L. 402.00, which will mean to throw the tower—and consequently the waterworks—out of commission for a short period of about 11 days on an average during the flood season. During this short period the required water supply can be drawn from the alternate source of the Dimna Nala Reservoir.

The present draw-off of the river by pumps is 1.12 m.g.p.h. or about 50 cusecs. It is proposed by the Company, to increase the pumping capacity to about 3 m.g.p.h. or about 133.5 cusecs. But the river gauges show that during the dry weather the total inflow of the river is less than 133.5 and hence it is not enough to meet the full requirements of the Steel Works. It has been calculated that the inflow supply in the river reduces to less than 133.5 cusecs when the water level falls below R.L. 394.42 (i.e., about 12 ft. above the crest level of the weir). Referring to Figure 3, it will be found that for about 83.5 per cent of the period of a year, the water level is about R.L. 394.42, and for the balance 16.5 of the period or about 60 days in a year, water level drops below that level. Thus it means that this river will not be able to meet the full discharge of 133.5 cusecs during 2 months of the dry period. Either the demand should not be kept as high as 3 m.g.p.h. or some alternate arrangement, for example to draw supply from the Dimna Nala Reservoir, will have to be made. The storage capacity of Subernarekha river up to the crest level of the weir as it exists today is about 5.25 m.c.ft. which is hardly sufficient for a $\frac{1}{2}$ day's supply at the rate of 3 m.g.p.h.

2. Design of Water Tower

Under the conditions explained above, the tower has been designed to admit the required full discharge when the river W. L. is between R.L. 402 and 394.42. It has been provided with circular R. C. C. beams 6" thick with tops at R.L. 397.00 and 391.50 in order to serve as sills of the gate openings. The tower is covered with a top R. C. C. slab 6" thick. This also has not been designed for the full load of the depth of water over it during the flood time. But instead 3 openings 2' x 6" have been provided with sill at R.L. 402. These will ensure to keep the tower full when it is out of action and water has risen above its top. This provision is necessary to make an economical design of the top slab. The tower bed is kept 3'-3" below sill of the Outlet Pipe. This depth will provide space for the silt settling down from the standing water inside the tower during high floods. The settled silt can be removed afterwards in winter, when the water level is low. Fig. 5.

Structural details of the design have not been worked out as they can easily be done in your office. For the design of its foundation, the nature of the bed at the proposed location of the Intake Tower is not exactly known. However, scattered outcrops of the rock on the right bank and also in its vicinity suggest that sound dependable rock foundation will be available at this site as well. The tower can be built on such rock foundation at a depth where they are found.

3. Tower Regulation

Fig. 5 & 6.

It is proposed to draw supplies from the top 2½ ft.—comparatively sand-free water of the river. This requires total length of the opening to be 10'-6" for the required maximum discharge of 133.5 cusecs. This length of 10'-6" has been made up in 3 parts so as to design gates of suitable sizes, in order to be economical in cost and easy in hand operation. A battery of 3 gates of size 3.5'×5' and with their sill level at R.L. 391.5; and another battery of 3 gates of the same size but with their sill level at R.L. 397.00 have been provided. These gates can be operated from top by manual labour. No gearing machinery has been proposed as the tower is to remain under water during high floods and there is a danger of the gears getting out of order. The required discharge of 133.5 cusecs can be drawn by sliding down upper 3 gates or the lower 3 gates according to the river water level,—so as to have an operating head of 2'-6" over the sills of the gates. If a lower discharge is desired, gate openings or the number of gates can be adjusted accordingly.

The outlet pipe will be of the same design as previously given. It is placed with its centre at 387.50 so that the present W.L. in the sump will not be affected. A valve is provided on the pipe line immediately outside the tower so that it could be closed in order to prevent the silt entering the pipe line when the tower is out of action during high floods. The valve on the Outlet Pipe will have to be throttled so as to adjust its opening for the low discharges. This will have to be done only during summer months, when the inflow in the river gets reduced. This valve can easily be operated from the top slab of Intake Chamber.

STATEMENT NO. 1

Serial No.	Water level X	Per cent frequency of water level exceeding X	Number of days per year.	REMARKS.
1	391	100.000	365.000	Top level of existing weir = R.L. 394.30.
2	392	99.761	364.128	
3	393	99.045	361.514	
4	394	94.718	345.721	
5	395	53.571	195.534	
6	396	29.573	107.941	
7	397	17.442	63.663	
8	398	12.107	44.191	
9	399	8.604	31.405	
10	400	5.684	20.747	
11	401	4.144	15.126	
12	402	3.136	11.446	
13	403	2.339	8.537	
14	404	1.994	7.278	
15	405	1.437	5.245	
16	406	1.013	3.697	
17	407	0.774	2.825	
19	410	0.454	1.657	
22	413	0.320	1.168	
26	418	0.134	0.489	
30	421	0.107	0.391	
33	424	0.000	0.000	

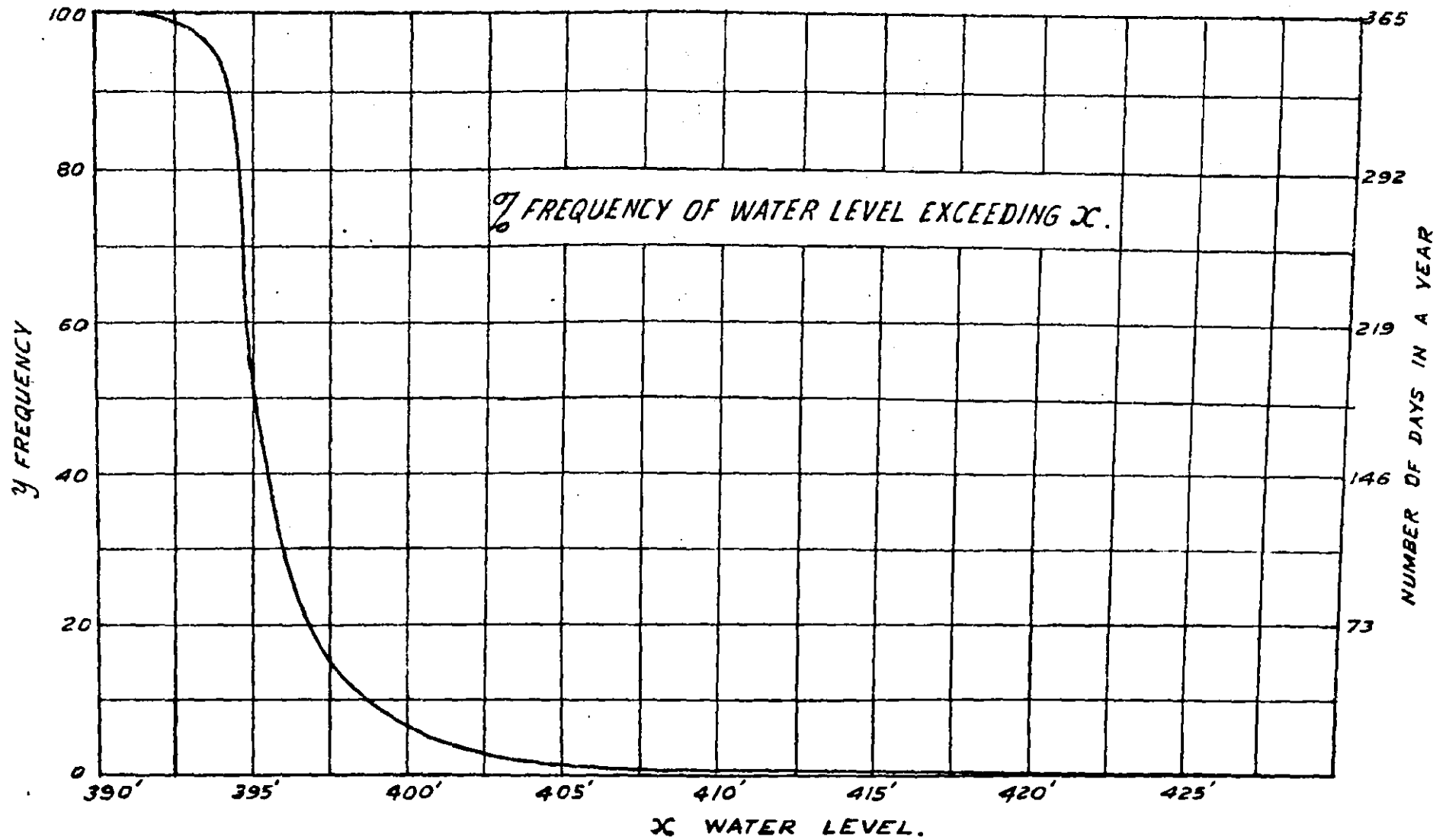
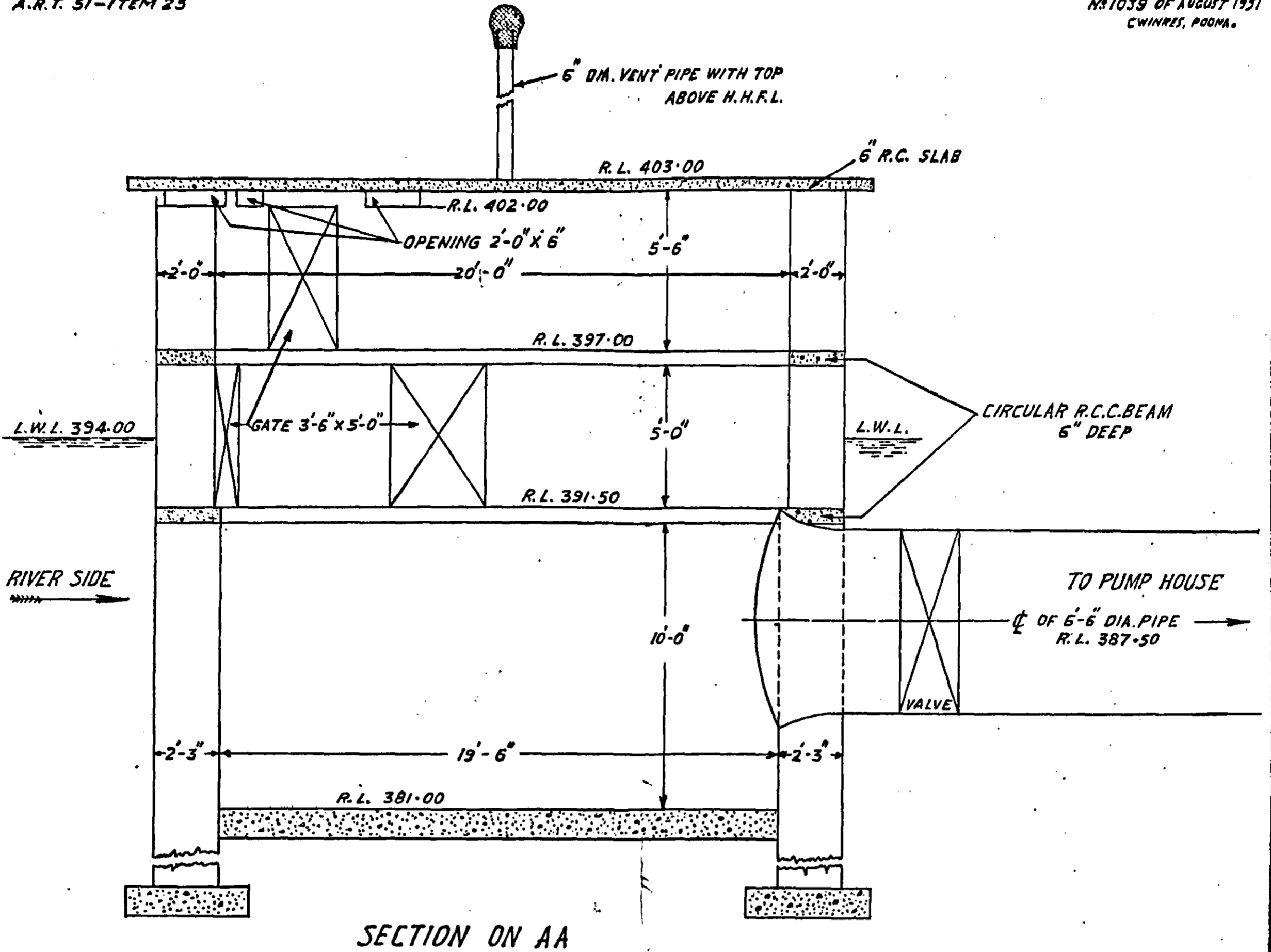


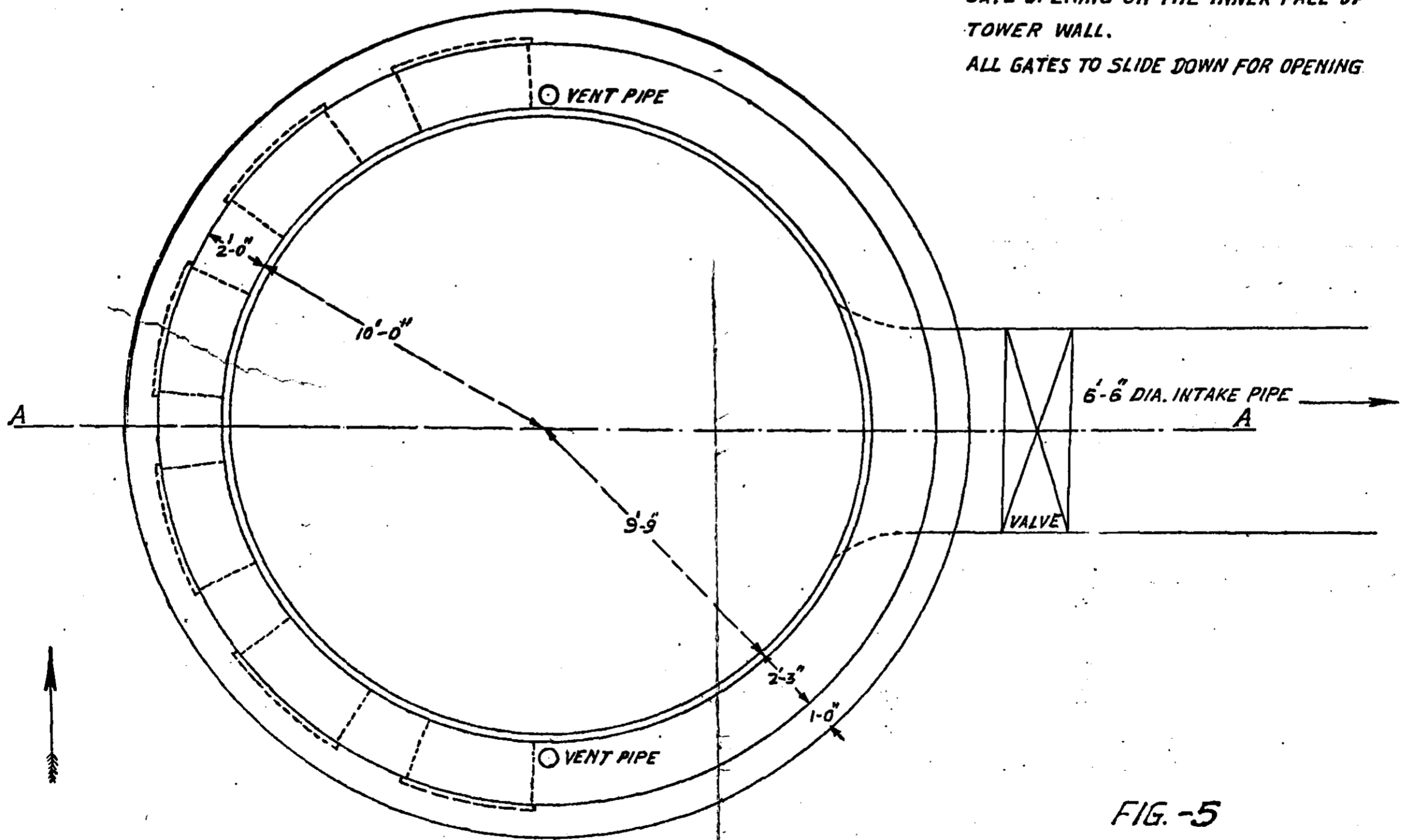
FIG.-4

COMP. BY: *M. S. Jit*
TRACED BY: PARKHI.
23-8-1951



SECTION ON AA

NOTE - 3'-6" X 5'-0" IS THE SIZE OF THE CLEAR GATE OPENING ON THE INNER FACE OF TOWER WALL.
ALL GATES TO SLIDE DOWN FOR OPENING



PLAN

FIG. -5
INTAKE TOWER
FOR
JAMSHEDPUR PUMP HOUSE
SCALE 1" = 4'

R.L. 403.00

R.C.C. SLAB.

R.L. 402.00

OPENING
2'-0" X 0'-6"

5'

2'-0"

GATE 5'-0" X 3'-6"

5'

CIRCULAR
R.C.C. BEAM .6 DEEP.

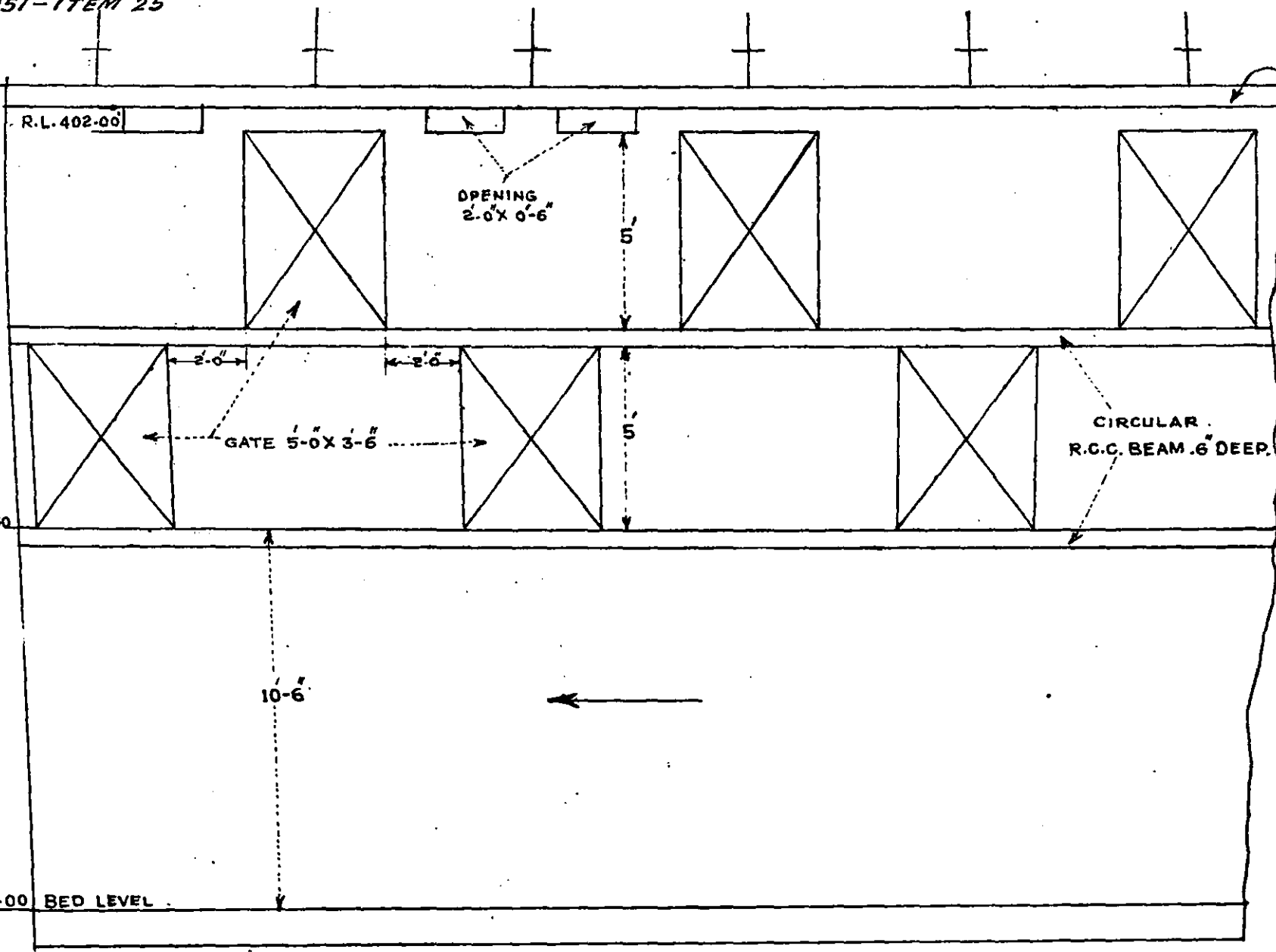
R.L. 391.50

10'-6"

R.L. 381.00 BED LEVEL

FIG:-6
DEVELOPEMENT OF
INNER FACE
SCALE:- 1" = 4'

C. BY:- *[Signature]*
TRACED BY R.K. KONDKA.



The Subernarekha River Jamshedpur.

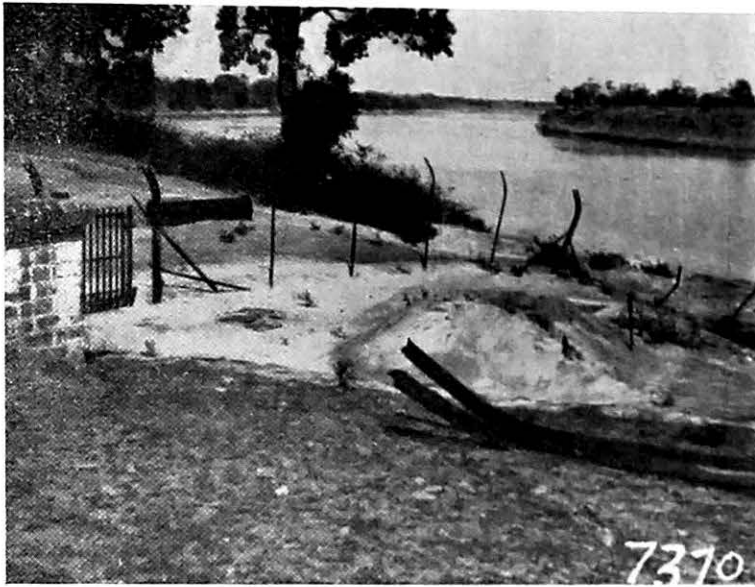


Photo No. 7370
Subernarekha river Jamshedpur. River bank
upstream of pump house.



Photo No. 7371
Subernarekha river Jamshedpur. River bank
upstream of pump house.

26. KANHAN WATER WORKS (NAGPUR WATER SUPPLY)

Introduction

1. The Kanhan Water Works of the Nagpur Corporation are situated on the banks of the Kanhan river about 1,000 ft. below the confluence of Kolar river with Kanhan (Figure 1). The Water Works consist of a well wherein raw water is collected and pumped by low-lift pump installed in another well at a distance of 90 ft. and of Patterson Mechanical type filters; where after purification and chlorination, water is stored in a balancing reservoir and then pumped again by high lift pumps to service reservoir at Nagpur. The present rate of pumping from this water works is only 7.5 m.g.p.d., but in second and third stages, it is proposed to increase it to 10 and 15 m.g.p.d. respectively.

Fig. 1.

The intake consists of a 25-ft. diameter well with R. C. C. steining, sunk 40 ft. deep below the lowest bed level passing through 12 ft. of sand, 20 ft. sand mixed with clay and 8 ft. red clay. 1,200 holes of 2" diameter are provided in the steining of this well at 1 ft. centres with a view to admit filtered water. Four penstocks have also been provided at different levels for drawing surface water during emergencies, according to the water level of the river (see Figure 2). The bottom of the well is filled in with rubble to a depth of 5 ft., i.e., R.L. 845 and from this level infiltration holes start and are located so far as R.L. 875.

Fig. 2.

2. Soon after the construction of the Intake Well, there was an unprecedented flood in the river and both the Pumping Station and the Intake Well were completely submerged under water resulting in silt and sand deposit in it to a considerable height. This was, however, removed by ordinary manual labour; but not below R.L. 868. Since then, there has been great trouble of the silt entry and reduction in water supply for the Water Works.

The main difficulty, now experienced in pumping raw water from this well, is that a large quantity of sand and silt is sucked in, finding its way partly in the infiltration and partly in the penstocks through the suction pipe and thereby corroding the impellers, bell-mouths and the low level centrifugal pumps, thus increasing the cost of pumping and filtration as well. This also chokes the suction pipe, hoppers and settling tanks. This trouble is practically throughout the year but in monsoon it is felt most as the manual labour cannot work so efficiently in removing sand, which collects in the well, as in other seasons, due to more depth of water in the well. In dry season, the water level in the river falls knee deep, while the pumping increases. The main river channel runs along the left bank. Every year, temporary bunds across the river have to be constructed in order to bring a narrow channel of water near the Intake Well (Figure 3).

Fig. 3.

3. The Municipal Committee, Nagpur, first consulted Mr. Newton, Joint Chief Engineer, C. P., P. W. D., who made the following suggestions:

- (i) Closure of the infiltration holes in the steining of the well through which sand enters partially,
- (ii) admission of water through an additional penstock above R.L. 884.00 at 45° to the flow, opening downwards to admit surface water only,
- (iii) replacing of the vertical type low lift pumps which were found unsuitable, and
- (iv) securing more safely the suction pipes which were expected to give trouble sooner or later.

The holes could not be closed, as suggested, as the well is filled with flood deposits. However, a new opening of 3 ft. x 2 ft. was provided at 45° to the flow and surface water is being taken through this in summer and winter. It is closed during the rainy season. No further suggestion was implemented.

4. In 1946 the matter was referred to a Board of Consulting Engineers to consider the difficulties experienced and to suggest remedial measures for smooth and economical working of the Water Works, which are the main source of water supply to the rapid growing industrial city of Nagpur. The Board was consulted on the following 2 main points, *viz.*,

- (a) "Whether constant dredging of the sand coming in, to keep foot valves sufficiently clear and above the top of sand bed in the Intake Well, will be useful? The dredger may even be needed for removing the sand from manholes of the infiltration gallery.
- (b) "Whether a sand pump at the Intake Well for extracting daily incoming sand will be helpful?"

The Board which consisted of—

- (1) Mr. P. C. Bose, Chief Engineer, Public Health Directorate, Calcutta,
- (2) Mr. W. J. Berry, Superintending Engineer, Health Services, Delhi Province, Delhi,
- (3) Mr. H. C. Bulsara, Chartered Engineer, The Model Mills, Nagpur, and
- (4) Mr. B. St. J. Newton, Joint Chief Engineer, C. P. and Berar, P. W. D., Nagpur,

made recommendations in the following words:

"We should have recommended the removal of all sand and silt from the Intake Well and the closure of the holes in the steining, if this could have been done at reasonable cost and without seriously interfering with the water supply; it would necessitate dewatering, alteration to the suction pipes and pumping direct from the river. We consider it very undesirable to disturb the suction pipes and we, therefore, recommend the following alterations to the existing Intake Well.

- (a) Desilting with dewatering to atleast R.L. 867.25 by air lift pumps. The sand and silt should first be loosened, if necessary, by a water jet under high pressure.
- (b) The provision of a 9" cement concrete floor made of precast blocks at about R.L. 867.25.
- (c) Provision of 3 vents each 4 ft. square in the steining on the upstream side, with sills at R.L. 881.5; each vent to be fitted with a penstock, opening downwards, so as to admit surface water.
- (d) Provision of an adjustable trough, specially designed as to prevent the entry of sand into the well.
- (e) Screens of bamboo "Trellis" should be fitted right across the dry weather channel at about 100 ft. and 200 ft. upstream of the Intake Well.

Stage II Scheme—As an increased water supply is an immediate necessity, arrangements should be made now for the construction of the stage II scheme, so that contracts may be given out and work started after the monsoon of 1946."

5. Unfortunately, no implementation of any of these recommendations were carried out. In August 1950, Mr. Nadirshah, Chief Engineer, The Concrete Association of India, Bombay, was also consulted by the Nagpur Corporation and he made the following suggestion.

"(i) *Construction of Groynes*—Mr. Chatre, Government Health Engineer, P. W. D., Madhya Pradesh, has suggested the construction of groynes in the river just on the downstream side of the confluence of the rivers Kanhan and Kolar so that all the sand and silt carried with the flow of water could be arrested at the groynes. This suggestion has not been put into effect probably on account of the cost and the doubtful benefit that would be derived from such a proposal.

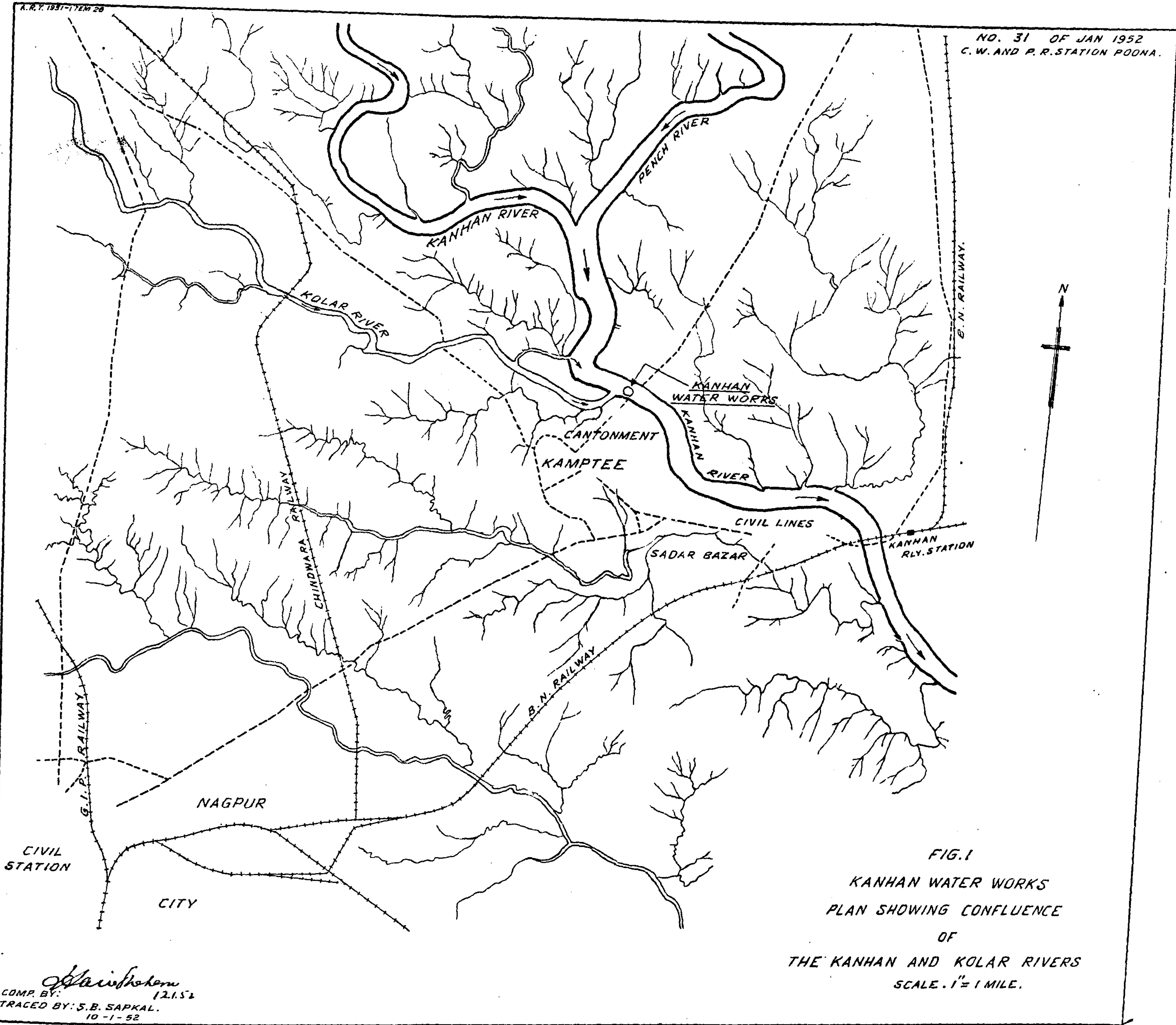
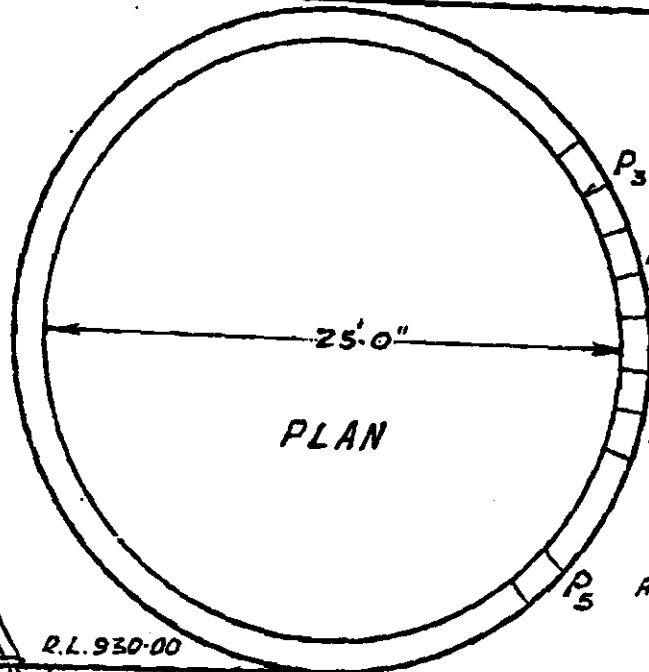


FIG. I
 KANHAN WATER WORKS
 PLAN SHOWING CONFLUENCE
 OF
 THE KANHAN AND KOLAR RIVERS
 SCALE. 1" = 1 MILE.

COMP BY: *M. S. Shinde* 12.1.52
 TRACED BY: S.B. SAPKAL.
 10-1-52



R.L. 884.0

R.L. 894.0

R.L. 880.0

R.L. 885.0

R.L. 900.0



R.L. 930.00

R.L. 925.0

SECTION

R.L. 888.05

R.L. 886.0

R.L. 885.00

R.L. 882.0

R.L. 880.00

2" dia. 1200 HOLE FOR PERCOLATION





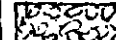
R.L. 865.00

FIG. 2
KANHAN WATER WORKS
PLAN & SECTION
OF
INTAKE WELL
SCALE 1" = 8'

R.L. 845.00

Handwritten signature
CONP. BY: 07/15/52
T. BY: 07/15/52

REFERENCE

-  SANDY BED.
-  WATER COURSE AFTER FLOODS.
-  WATER COURSE DIVERTED EVERY YEAR.
-  TEMPORARY BUNDS OF SAND STONES ETC. CONSTRUCTED TO DIVERT FLOW.
-  ROCK AVAILABLE IN PATCHES.

NOTE:

THE TEMPORARY BUNDS ACROSS THE RIVER ARE CONSTRUCTED EVERY YEAR TO ALLOW THE WATER TO PASS NEAR THE INTAKE WELL.

OLD KAMPTEE

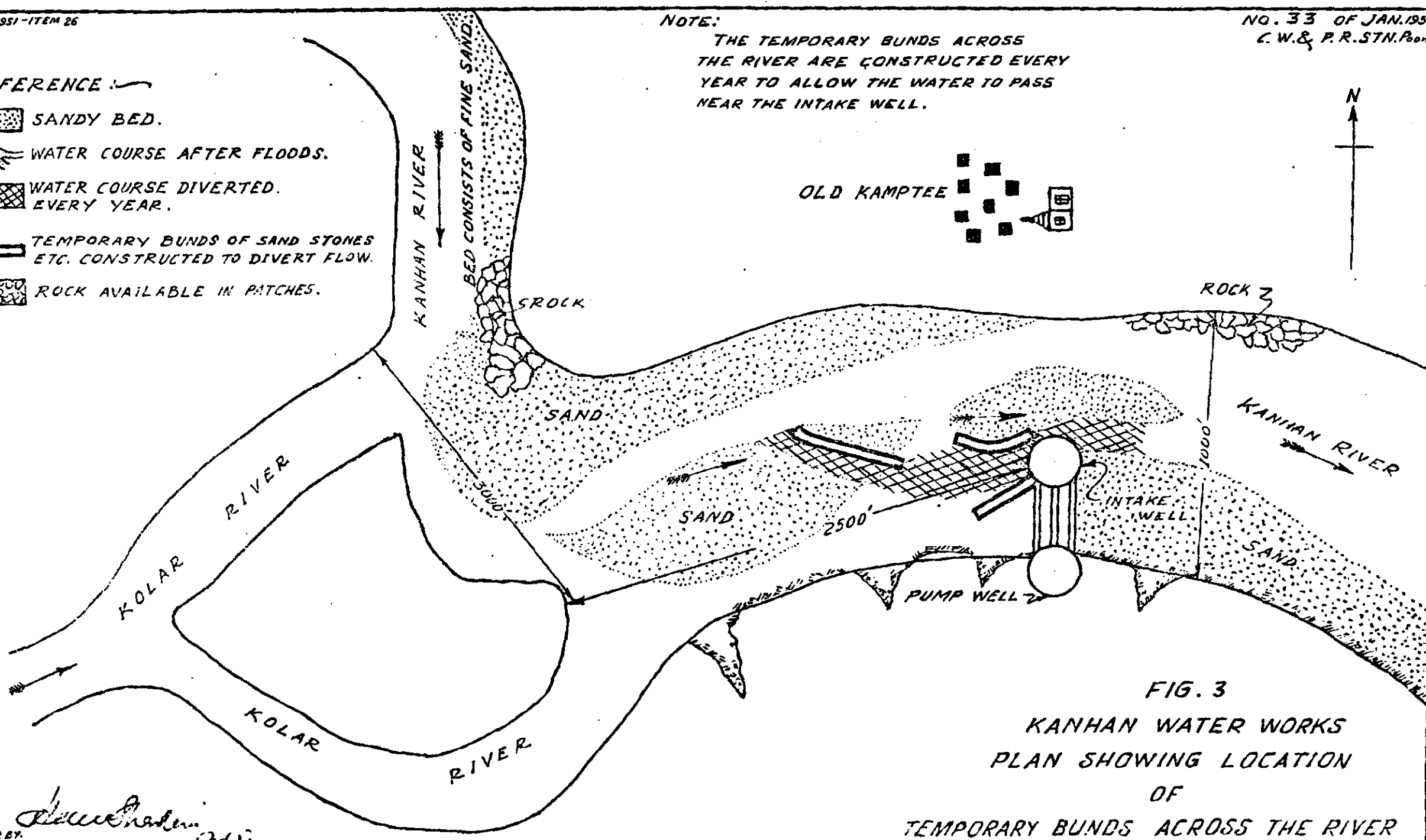
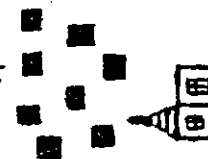


FIG. 3
KANHAN WATER WORKS
PLAN SHOWING LOCATION
OF
TEMPORARY BUNDS ACROSS THE RIVER
(NOT TO SCALE)

CONR BY. *[Signature]*
TRACED BY. R. T. THAKKAR (24.12.)

(ii) *Provision of an Inner Ring in the Existing Well*—There is also suggestion of providing a ring inside the well so that the sand and silt may settle down in between the inner and outer rings of the channel, thus avoiding the drawing of the sand into the pumps. This remedy will not be effective for the reasons given below.

(a) Due to the limited space between the inner and outer rings, dredging for removing the rapid silting would be a difficult task.

(b) The construction of an inner ring will also present some difficulties on account of the accumulation of silt in the well and removing the same below R.L. 868.

(c) As the well has more or less spent its useful life, any additional expenditure in its modification would not be justified.

(iii) A better and more practical solution to overcome the present difficulties would therefore be to construct a new Intake Well."

As regards the location of the new Intake Well, Mr. Nadirshah recommended that the following aspects would have to be considered.

"(a) The variations in the cross-section of the dry water channel from time to time.

(b) Formation of any sand banks which might divide the dry weather channel upstream of the Intake Well.

(c) Construction of small training bunds at the upstream end of the sand bank to induce scouring in the nearer channel."

Final Recommendations

6. In April 1951, the Manager, Kanhan Water Works, referred the case to this Research Station and sought advice as how to remove the silt trouble. The history of the well shows that it was originally designed to get the percolated water through the 1,200 holes in the steining, and from there to lift the same by means of pumps. The designers had provided a reservoir in the form of a well where the percolated water could be collected in 30 ft. depth (R.L. 845 to 880) through the percolation holes. This depth of water provided in the well was sufficient for the pumps' capacity to work and also to leave sufficient margin for the silt to settle at the bottom provided there was enough head of water around the river to supply the percolated supplies through holes only. The full capacity of the pumps is 3,00,000 gallons per hour or 13.3 cusecs. The estimated yield at that time was 1,20,000 gallons per hour or 5.3 cusecs only. The surface area of 1,200 holes works out $1,200 \times \frac{2}{12} \times \frac{2}{12} \times \frac{\pi}{4}$ or about 26 sq. ft. So in order to give a yield of 5.3 cusecs, the percolation velocity required was only $\frac{5.3}{26}$ or about 0.2 ft. per second.

7. The supply of water from the Kanhan Water Works commenced in December 1941, when the water level in the river was below R.L. 887. In January 1942, the Executive Engineer in charge reported "the yield in the Intake Well was less than one quarter of the full capacity of even one small pump". In February 1942, however, the Superintending Engineer in charge reported "it appeared that the holes in the steining were getting clear of silt and that the yield was much better". The Intake Well might have yielded enough supply, but unfortunately, its efficiency

could not be tested even for one full season, due to the fact that during the monsoon of 1942 there was an abnormal flood in the Kanhan river, and the water rose to R.L. 931.88. The Intake Well was completely submerged under water, resulting in deposits of silt and sand to a considerable depth. Thus from the very first year of its working, where there should have been a reservoir of 30 ft. depth of water in the well, there was a solid column of silt and sand to a height of 23 feet from the bottom and there was only 7 ft. to 10 ft. depth of water in the well for the pumps to lift. This deposit of sand and silt can be removed by manual labour to R.L. 868.0 only.

8. Under the existing conditions during the dry season, there is not much flow through the holes by means of percolation. Hardly about 7/30 of the total number of holes are above the silted bed in the well, and assuming all of them to be open (which is not at all certain) their surface area comes to $7/30 \times 26$ or about 6 sq. ft. The bed level of the river is at R.L. 882 and lowest water level in the river is 885.0. The present rate of pumping is 2,00,000 gallons per hour or about 9 cusecs. This means that if all the holes above the silted bed were open, the water would be sucked through them with a velocity of 1.5 ft. per second; and, if half of them are choked, the sucking velocity would increase to 3 ft. per second. All these holes being below bed level of the river, it is no wonder that along with water, actually fine sand is sucked through these holes, which has been estimated as much as 6 cft. per hour. In order to augment the supplies, the penstocks are opened. The lowest penstock is at R.L. 880, i.e., below the bed level, and hence is closed. The next above is at R.L. 885, which when opened draws in the sand laden water from the bed of the river, because there is hardly 3 ft. of water in the river and the area of the penstock is only 6 sq. ft.

As the well stands at present, with hardly 3.0 ft. depth of water round it, it can never give the required yield of even 1,20,000 gallons per hour (which quantity is pumped at present) without sucking an enormous amount of bed silt and sand from the river. In fact there is not enough depth of water in the river itself to give a silt-free supply into the well or to increase the yield of the well.

Photos 709
to 712 & 715.

9. During the monsoon season, the trouble is further aggravated due to the confluence of the Kolar river with the Kanhan river just above the Intake Well. The Kolar river passes through black cotton soil area and carries a very heavy suspended load of clay. Besides, the flow at the confluence must be highly disturbed so as to throw the bed material into suspension. Though the surface water only is admitted into the Intake Well, by means of penstocks, a considerable amount of bed material in addition to suspended material must also be entering the well due to disturbed flow. In other words, the position of the present Intake Well is most unfavourable. Had it been further downstream and away from the confluence, say near the Mahadeo Ghat, the bed material thrown in suspension at the confluence would have time enough to re-deposit and flow along the bed of the river. On enquiry it was found that large quantities of sand and silt are drawn by the Intake Well when the *Kolar river is in floods*. The Kolar river is also responsible for unsteadiness of the river flow near the well. It pushes the main Kanhan flow towards the left bank opposite Intake Well. Had it not been for the Kolar river, the course of the river near the Intake Well would have been favourable for admitting water into the well. During monsoon period large sums have to be spent in removing silt and sand from the well and in using large quantity of alum for getting rid of silt and suspended material. Secondly, the after effects of this pushing of Kanhan flow to the left bank are, that during the dry season the bed of the river on right side, where the well is situated, is left high and dry and the river channel has to be diverted towards the well by means of bunds, etc. There is thus a very shallow depth of water round the well during winter.

Photos 716
& 717.

ITEM NO. 26.
A. R. T. 1951

Kanhan Water Works

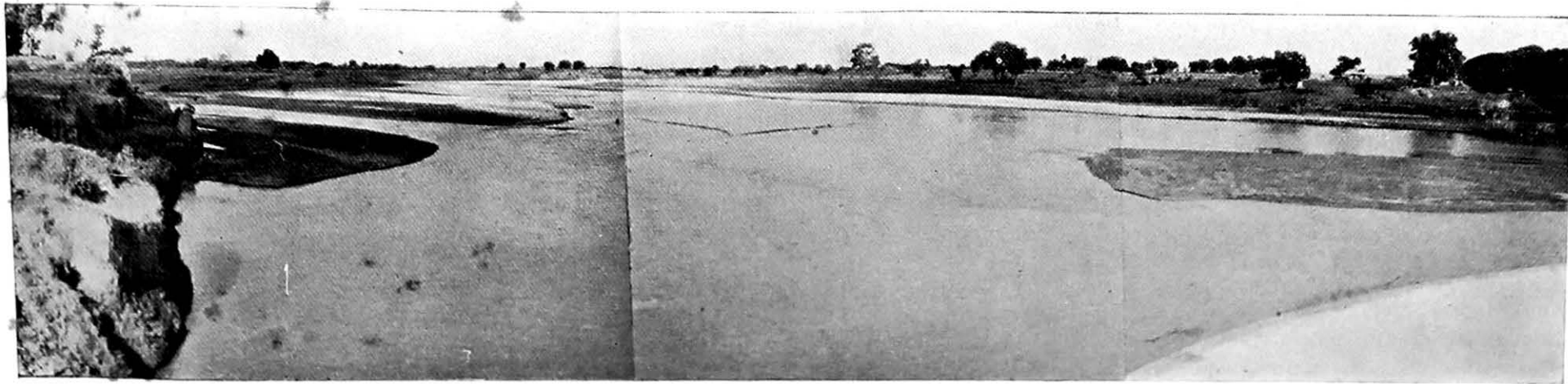


Photo No. 709-710-711

Composite photo from water intake tower looking upstream towards the junction of the Kolar River with the Kanhan.

KANHAN WATER WORKS.



Photo No. 712
From right bank at junction of Kolar River looking
upstream.



Photo No. 715
From right bank at junction looking downstream
towards the Shiv Temple and well on the left bank
opposite intake tower.

KANHAN WATER WORKS



Photo No 716
Mahadeo Ghat on right concave bank. Looking
upstream

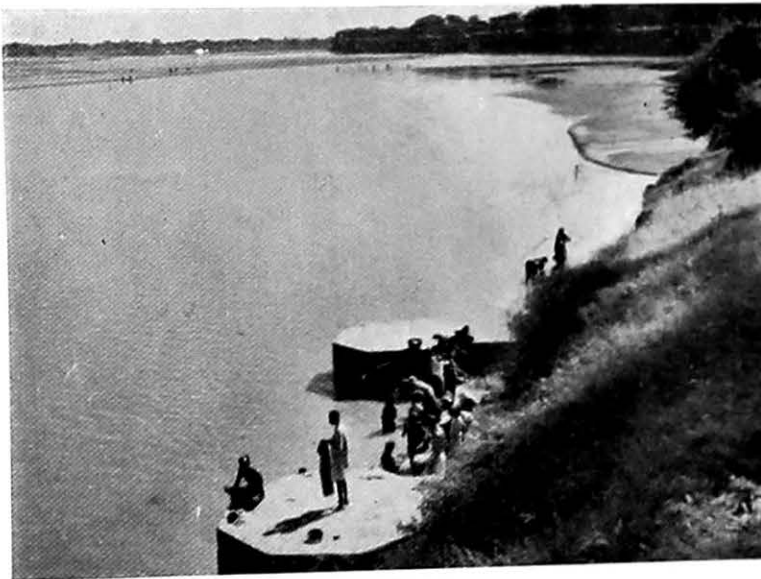


Photo No. 717
Mahadeo Ghat on right concave bank-Looking
downstream.

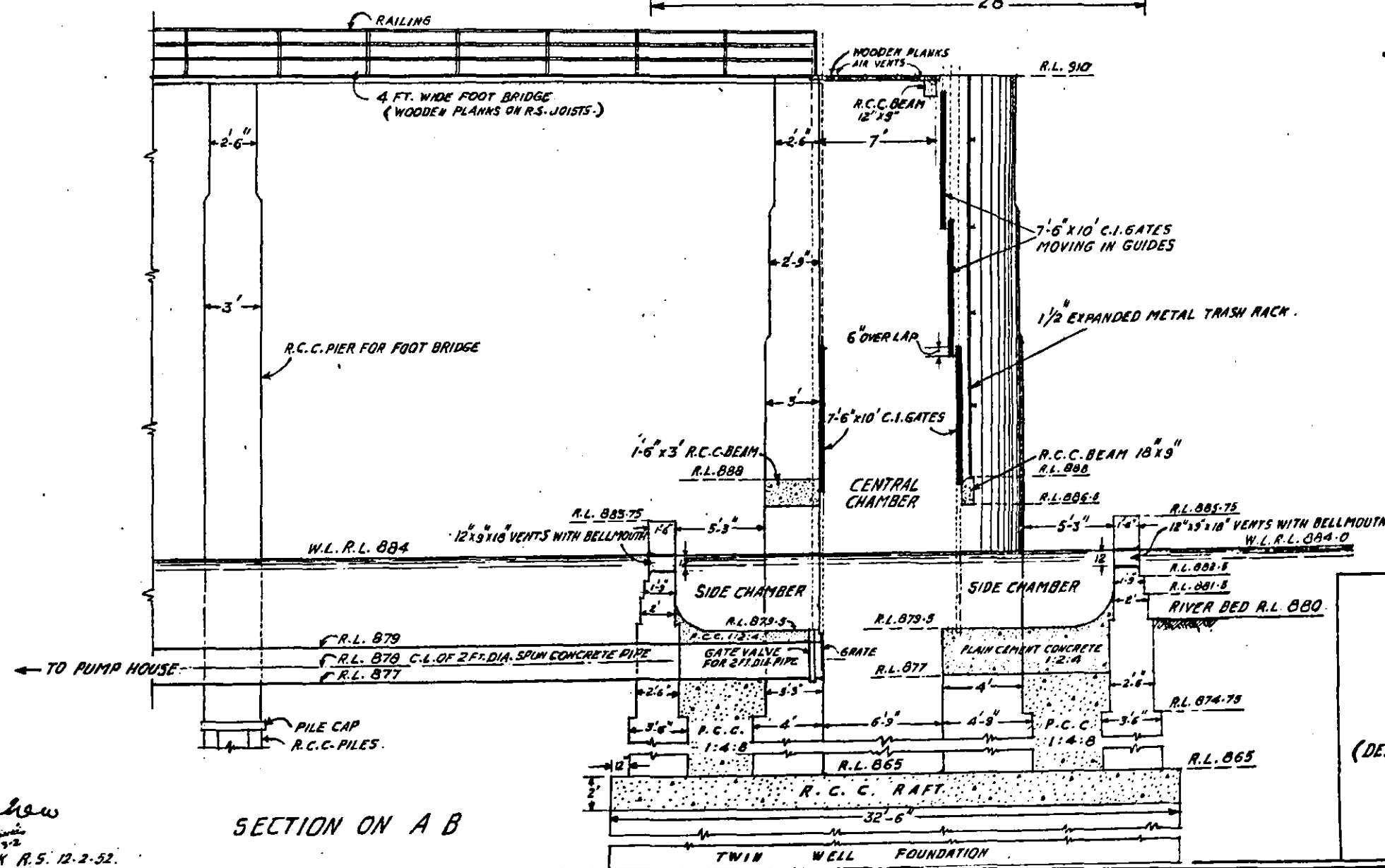
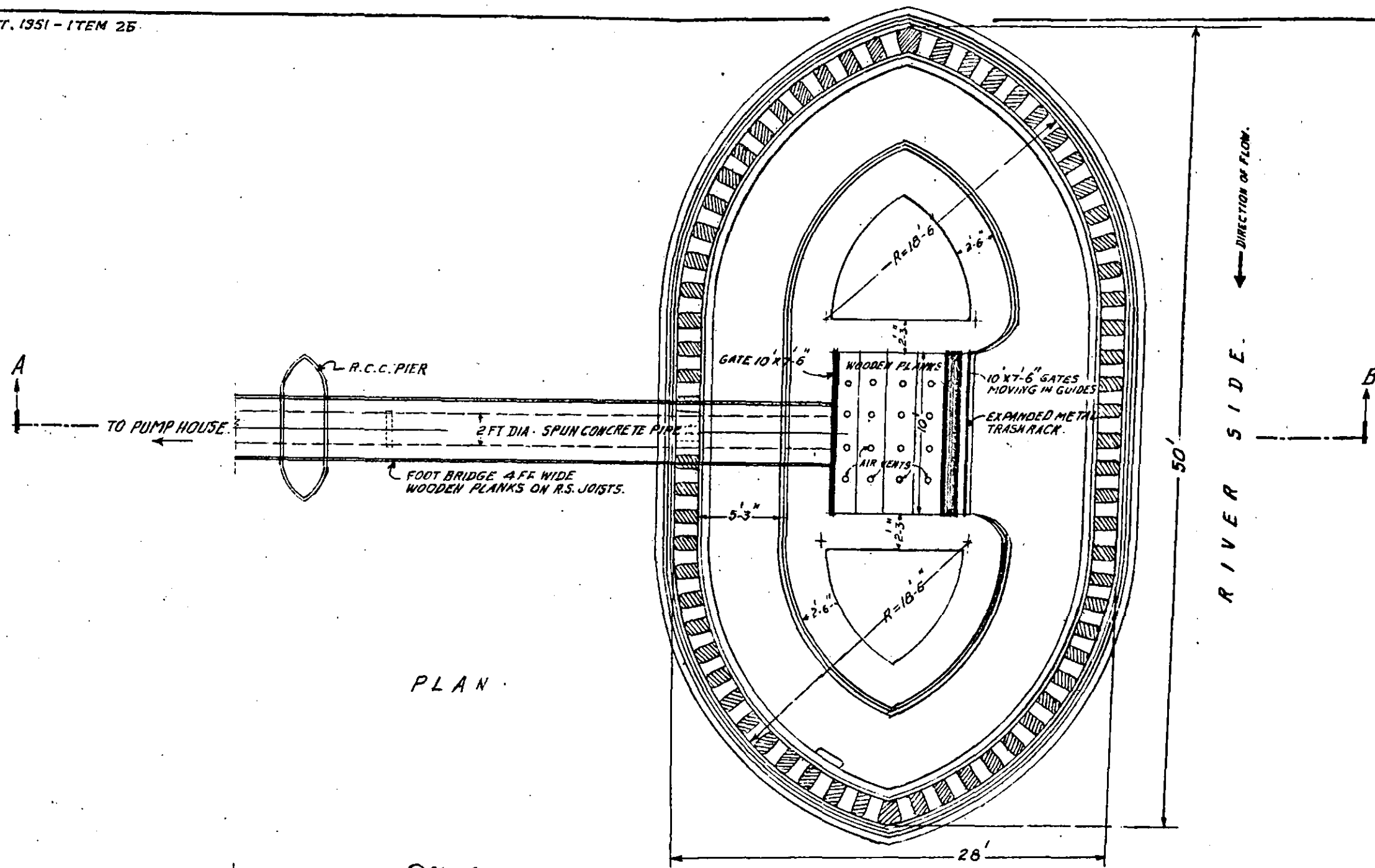


FIG. 4
INTAKE TOWER
FOR
KANHAN WATER WORKS
(DESIGN PROPOSED BY C.W.&P. RES. STN. POONA.)
SCALE, 8 FT. TO AN INCH.

Handwritten signature
COMP. BY
TRACED BY: NAIK R.S. 12-2-52.

10. The above analysis clearly shows the desirability of diverting the Kolar river into the Kanhan further upstream, so that the region of disturbed flow would also be shifted far upstream of the Intake Well. This proposal on inspection, was found to be very expensive and hence was not considered. The other alternative is to build the new Intake Well near the Mahadeo Ghat, which is situated on the concave side of the river bend and where the water is much deeper than at the present side of well, and hence the flow conditions both during the monsoon and winter seasons would be favourable. The water will have to be led from the new Intake Well to the Water Works through a pipe line.

11. At present, the yield of the well is not enough to cope with pumping and to meet the water supply needs of the town. Besides, the main trouble is that the Intake Well actually sucks large quantities of sand and silt both during the monsoon and dry seasons. The latter not only requires heavy maintenance whole the year round to remove the sand from the well, but the sand damages the propellers of the pump. During monsoon months when the water is very turbid, alum worth as much as Rs. 700 per day is required for coagulation, and during the dry period the river channel has to be diverted by means of bunds to the Intake Well, in order to feed the same.

12. The most unfortunate part is that the pumping has to be continued from this well throughout the period in order to supply water to the town. On no account can pumping be stopped and whatever dredging of sand is done by manual labour, it has to be done while the pumps are working. The result is that the sand is removed only to small depth, and it fills up again.

13. There are also second and third stages of the project when about 10 and 15 million gallons per day (*i.e.*, about 18 and 27 cusecs) of the supply are to be pumped from the river. The existing well can never give such increased supplies. For increased supplies it is necessary to construct another well. This will also be used as stand-by to the existing well so that sand deposits can be cleared from either of the wells without interrupting the pumping. As stated in para. 8, the only suitable place for this second well is in the vicinity of Mahadeo Ghat.

14. The immediate remedial measure, which can be suggested to reduce the sand entry into the present Intake Well, is that the velocity of flow into the well should be reduced to such an extent that there is no sucking action of sand. The capacity of the pumps is 3,00,000 gallons per hour or about 9 cusecs. There exists one penstock of 6 sq. ft. area at R.L. 885. When all the supply is drawn through this penstock, the velocity of inflow will be of the order of 1.5 ft. per second which is high enough to suck the bed sand along with water. If the number of penstocks at that level is increased to say 3 or 4, the velocity would be reduced to about 0.4 to 0.5 ft. per second which is low enough.

15. As regards 23 ft. depth of solid silt with which the bottom of the well is at present filled up, this silt can be dredged only after the second Intake Well is built, unless arrangements can be made for air lift pump and water jet under high pressure; in the latter case the effect of percolation supply through 1,200 holes 2" diameter can be tested to see if they can draw required amount of discharge as originally designed.

16. The design of the Intake Well has been prepared on the lines of one for supply of water to Jamshedpur Iron and Steel Works from the Subarnarekha river (Figure 4). This design allows only surface water to be drawn into the Intake Well during monsoon; and during the summer months, admits water into the Intake Well with as little velocity as possible, so as not to draw sand along with water. As the tower recommended to be sited near Mahadeo Ghat will be in deep pool of water, there will be no difficulty in getting adequate supplies even during dry season.

Fig. 4.

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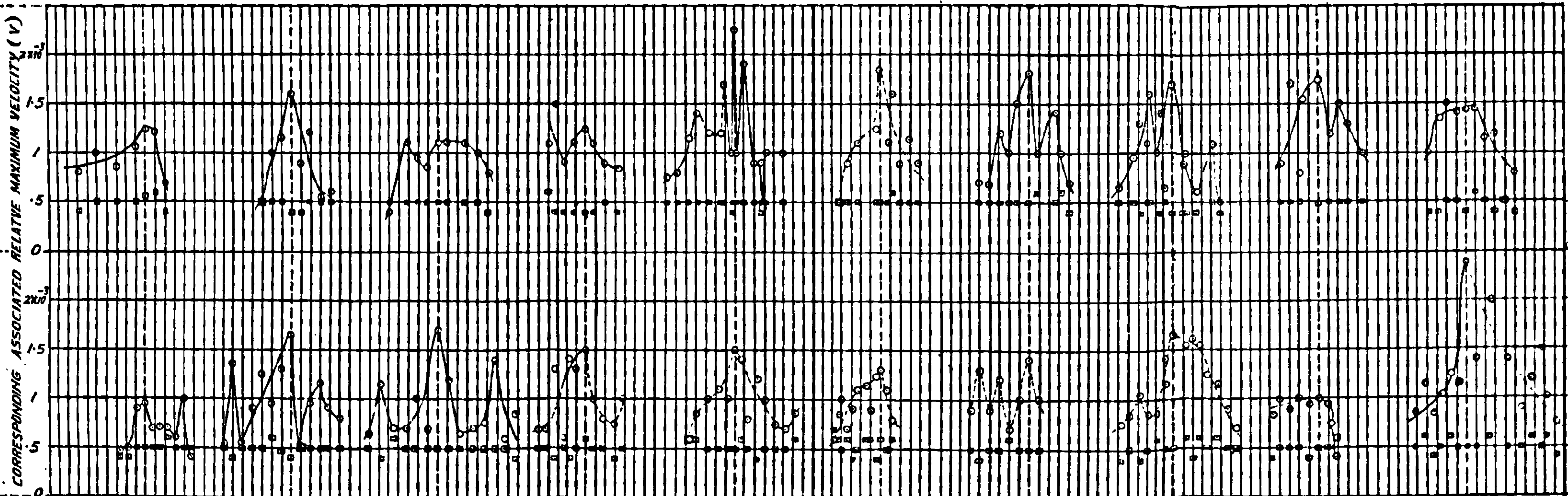
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RECORD WITH GEOPHONES ON DAM
STATION ⑤ AND DOME OF SIPHON NO. 7

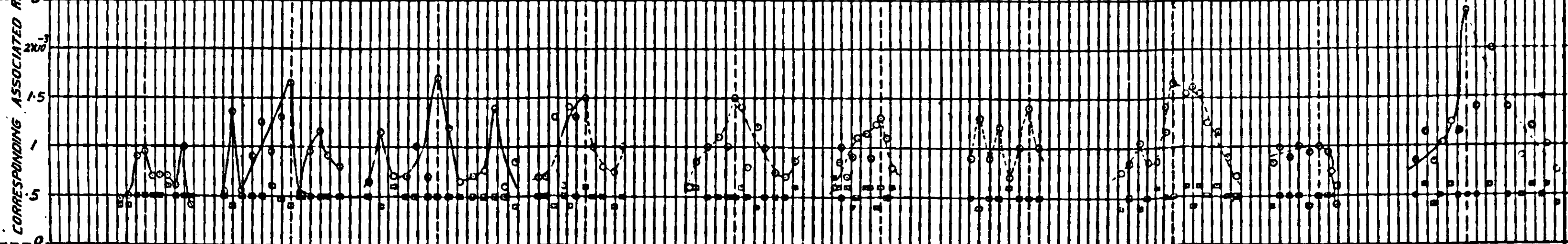
STATION OF
OBSERVATION

ON THE DAM -
STATION OF OBSERVATION ⑤

ON THE DOME OF SIPHON NO. 7 -
STATION OF OBSERVATION ⑥



PERIOD OF INDIVIDUAL WAVE
IN WAVE-GROUP (T)
IN SECOND



PERIOD OF INDIVIDUAL WAVE
IN WAVE-GROUP (T)
IN SECOND

FIG. 19

HIREBHASGAR DAM - 5, D G

ANALYSIS OF STRUCTURE OF TYPICAL WAVE GROUPS
DURING PRIMED STAGE ON THE DOME ⑥ OF SIPHON NO. 7
AND ON THE DAM STATION ⑤

REFERENCES -
PERIOD OF INDIVIDUAL WAVE IN WAVE GROUP (T) □
CORRESPONDING ASSOCIATED RELATIVE MAXIMUM VELOCITY (V) ○

EACH SMALL DIVISION REPRESENTS AN INTERVAL OF FIVE WAVES WITHIN THE GROUP

EACH SMALL DIVISION REPRESENTS AN INTERVAL OF TEN WAVES WITHIN THE GROUP

COMPONENT OF
VIBRATION

VERTICAL

PERPENDICULAR TO THE AXIS OF DAM

PARALLEL TO THE AXIS OF DAM

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