



*Note.*—Page Nos. have not been printed on plates bearing figs. 2 to 75.

*Errata*

Against "Guide Bank, bund and wall" on page 3 read 135 in place of 231 under 1949.

Against "Pier(s)" on page 5 read 135 in place of 133 under 1949.

(See fig. 81A  
on page 251)

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MINISTRY OF WORKS, MINES AND POWER

(GOVERNMENT OF INDIA)

**CENTRAL WATERPOWER IRRIGATION  
AND  
NAVIGATION COMMISSION  
CWIN RESEARCH STATION, POONA**

*Director :* D. V. JOGLEKAR, B. E.

Research Publication No. 15

**ANNUAL REPORT (Technical) 1949**



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Research Assistant	...	R. Rangarajan, M.A.

*Physics Section*

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Assistant Research Officer	...	B. Pant, M.Sc., A. Inst. P.
"	...	S. K. Guha, M.Sc.
Research Assistant	...	V. Venkateswarlu, M.Sc.

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

The expansion of the Research Station has progressed steadily during the year. The entire area of 109 acres of land required for this expansion has since been acquired.

The main water supply arrangements for the new area at Khadakwasla are almost complete. The construction of link lines and channels to the various experiments in the area will be taken in hand, the first being the supply channel to the Hooghly Model (Port of Calcutta). Arrangements for the supply of electric power to the field Station at Khadakwasla have also been receiving active attention and are likely to be finalised in the near future.

Work on levelling of the area for the rating tank, workshop and other buildings, field hydraulic laboratory and equipments for the various laboratories of the station has been taken in hand.

The staff of the Soil Mechanics and Chemistry Section has been receiving productive training on the Hirakud and Kosi Projects. At Poona work on stress analysis by Photoelastic method has been initiated.

The station has been busy during the year with the solution of a large variety of problems pertaining to the utilisation of water resources in India which have been referred to it by the Central and State Governments and Corporations. Problems connected with the Hooghly river and the Cochin and Madras Harbours are among the more important ones. Other problems under investigation are the training of the Ganga river near Mokameh, erosion around the pier of the Malaviya bridge at Benaras, Dimna Nala and other Spillways.

A wave recording apparatus for the Madras Harbour Model and an automatic electronically controlled tide reproducing mechanism for the Hooghly model have been evolved at this Station and are giving excellent service. This is creditable work.

During the year, the station has been visited by some distinguished foreign visitors, namely, Mr. Peter Koinange from East Africa, Dr. Shen Yi, Chairman, Bureau of Flood Control, ECAFE, Mr. J. B. Justin, Construction Civil Engineer, U. S. A., Mr. E. B. Burwell, Geologist, Washington, D. C. etc.

The Director of the Station attended the last meeting of the International Association of Hydraulic Structures Research held at Grenoble, and visited various laboratories and research institutions in Europe and the United Kingdom. He has submitted a report on his visits. This visit of the Director will be of great value to the Station and will help in better exchange of research knowledge and facilities between this and other hydraulic research stations of the world.

The Director, officers and staff at Poona and Khadakwasla have continued to do good work during the year.

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## INTRODUCTION

As reported last year, staff for the various sections of the expanded Station was selected and majority of them joined in the year under review. Some posts of C.R.O.s, R.O.s, A.R.O.s are still lying vacant in the various sections of the Station. Due to financial stringency the Government of India have stopped any new recruitment, but in the interest of work steps are being taken to request the Government to lift the ban and permit the recruitment of at least some staff against the posts still vacant.

Estimates for equipment for the various sections were prepared and submitted to the Government for sanction. Steps have been taken to procure the same.

The various sections of the Station have been busy in carrying out useful investigations on problems connected with Irrigation and Navigation Engineering. Advisory and suggestive reports were prepared and sent to the concerned institutions. The report of these investigations is given in the following pages.

The River and Canal Hydraulics Section, the oldest in the Station, has made some interesting investigations, on training of rivers. Work has been done in connection with spillways, canal falls and protective works for the safety of Railway Bridges and for arresting erosion of river banks. An advancement has also been made in the existing methods of dissipation of energy for spillways and a device named as Bucket-type-splitter was developed during the course of model experiments on Vaitarna Dam Spillway.

Madras Harbour Authorities reported that the harbour presents a serious danger to ships stationed in it during a cyclonic storm and they have, therefore, to ask the shipmasters to remove their ships from the harbour to a relatively safer place in the sea. They accordingly approached the Station for suggesting measures to reduce the 'range' in the harbour. The work in this connection has been in progress during the year.

Calcutta Port Trust Authorities have to spend a considerable amount of money in keeping the Hooghly River Estuary in a navigable condition. The conditions which control navigation change from year to year. The discharges and water levels are changing continuously due to tides and the estuary is subject to heavy siltation and a few lakhs of rupees are required for dredging it every year. Model experiments have been in progress during the year to suggest measures for its improvement.

Experiments were undertaken during the year to improve the Cochin Harbour. It is subject to heavy siltation and to keep it navigable, dredging has to be done at a cost of Rs. 10,00,000 every year.

Experiments on the model of the Ganga river at Garhmukteswar on the Delhi-Muradabad railway section, where the river was on the point of outflanking the railway bridge, were completed during the year. Protective measures were suggested which have since been carried out and the results so far reported are satisfactory.

The Statistics Section has been busy in analysing the data in connection with the specific problems tackled by the different sections of the Station. Some of the important investigations completed by the Section are resolution of tides in the Hooghly River Model, forecasting

of southern limit tides from Saugor data and distribution of the reciprocal of a normal variable. These have been given in the report.

The staff of the Soil Mechanics Section is being trained at Sambalpur by Dr. Hoon, Deputy Director. The Chief Research Officer of the Section has been busy in procuring equipment for the Section and setting up the laboratory. The staff of the section will be called back to the Station as soon as the laboratory of the section is fully equipped.

Physics Section started work on Stress Analysis of structures by photoelastic method. A study of gelatine models was undertaken with a view to investigating the extent to which the results thus obtained could be applied to stress problems in soil mechanics and civil engineering. A few stress patterns have been photographed and are being analysed. Comparison is being made with results obtained on theoretical considerations.

Work has also been undertaken on problems connected with porosity and permeability of soils, sands, cement, concrete blocks and rocks and friction of wet sands.

It has, however, to be mentioned that the work of the Section was handicapped due to lack of equipment. Estimate of equipment of the Section has been submitted for sanction and attempts are being made to procure the same.

It has as yet not been possible to recruit the staff of the Navigation Section and Concrete and Material of Construction Section.

The Director of the Station, D. V. Joglekar, had been on tour to Europe in connection with the Hydraulic Research Conference at Grenoble (France). During his tour he visited some of the dams in France and Research Institutions, Engineering Concerns and Factories connected with manufacture of apparatus (Scientific and Engineering) in France, Switzerland, Holland, Sweden, England and Scotland. He has brought out a good deal of useful information, embodied in a note on the subject for the use of Government Research Institutions in India.

Fig. 1

The Director visited a number of places in India shown in fig. 1. The figure also shows places where advice was given without model experiments and also those for which suggestions were made after carrying out model experiments.

C. V. Gole, Research Officer, Navigation Section, proceeded on study leave to Sweden, U. K. and U. S. A. to receive training in research work on navigation, ship building and other allied subjects.

The Station took part in the Engineering Exhibition held at the time of the Centenary of the Thomason College of Engineering, Roorkee. Five running hydraulic models and a number of other rigid models were exhibited.

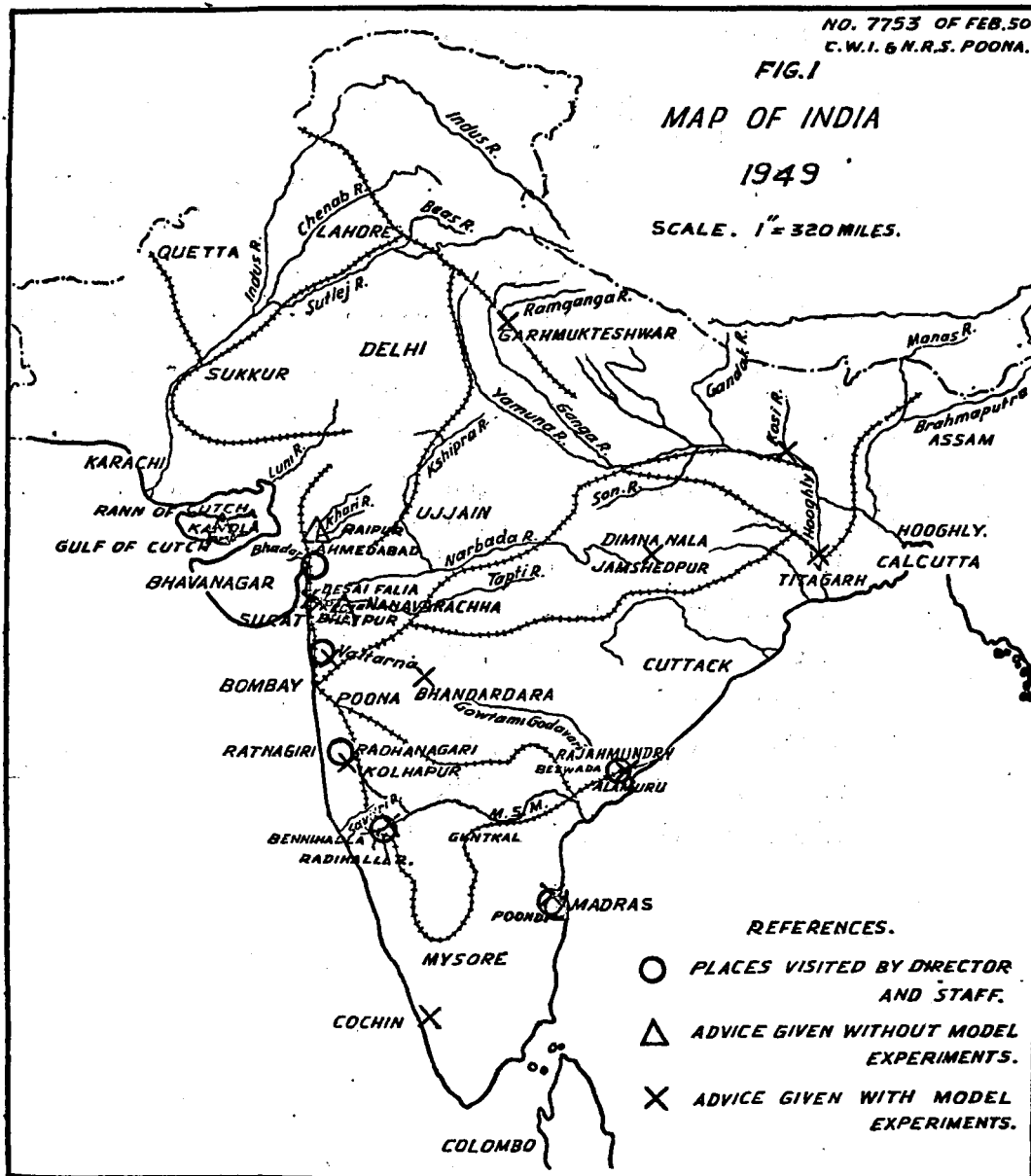
As usual the Station was visited by a number of visitors including Engineers and high officials of the Central and Provincial Governments.

The Honourable Shri N. V. Gadgil, Minister for Works, Mines and Power, the Honourable Shri J. N. Mehta, Minister for P. W. D., Bombay. The Honourable Shri M. P. Patil, Minister for Agriculture, Bombay and Sir M. Visvesvaraya were the distinguished visitors to the Station during the year. Other prominent visitors were :—

1. Shri V. Srinivasan, Administrative Officer, Cochin Harbour
2. " S. K. Banerji, Director General of Observatories
3. " M. L. Champhekar, S. E., D. I. C., Poona
4. " A. R. Venkatachari, Chief Engineer, Irrigation, Madras



INTRODUCTION



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5. Shri M. P. Mathrani, C. E. (Irrigation), Bihar
6. " R. C. Shenoy, S. E., Madras
7. " N. M. Thadani, Deputy Chief Engineer, G. I. P. Rly., Bombay
8. " B. N. Chopra, Deputy Chief Engineer, E. I. Rly.
9. " A. Saldhana, C. E., G. I. P. Rly., Bombay
10. Dr. S. K. Muranjan, Principal, Sydenham College, Bombay
11. Shri A. Nageswara Aiyer, Planning Officer (Roads), Government of India
12. " S. Nanjundiah, Administrative Officer, Vizagapatam, Port
13. Commander Pawsey, Deputy Conservator, Port of Calcutta
14. Shri J. N. Mitra, Director, U. P. Irrigation Research
15. Mr. G. McKelvie, C. E. (Roads), Govt. of India
16. Shri Pandit L. Maitra, Chairman, Food Grains Investigation Committee
17. Dr. Shen Yi, Chairman, Bureau of Flood Control, Ecafe, U. N.
18. Shri A. N. Chopra, Additional Chief Engineer, Central P.W.D.

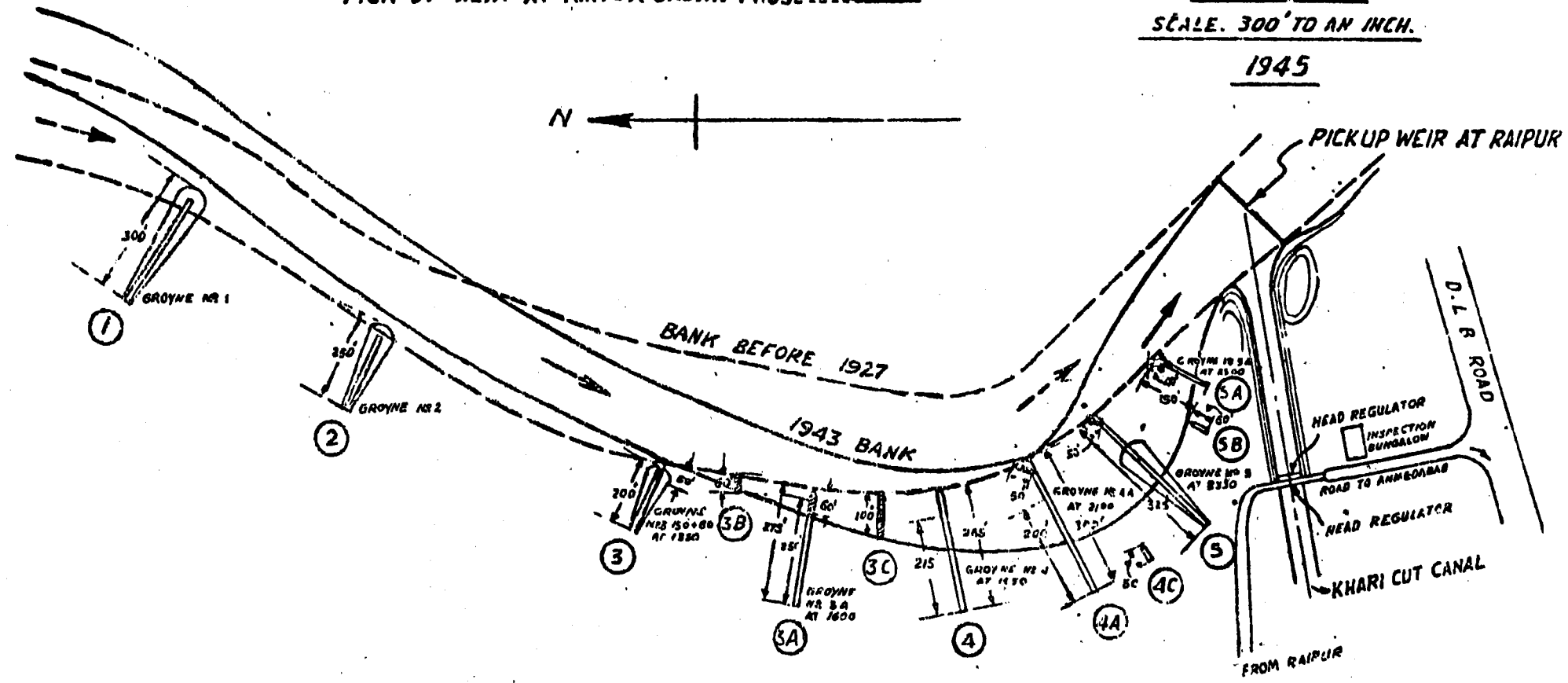
Students and Professors from the following colleges and institutions also visited the Station during the year :—

1. School of Engineering, Bangalore
2. St. Mary's School, Poona
3. College of Engineering, Poona
4. College of Engineering, Bangalore
5. College of Agriculture, Dharwar
6. Benares Hindu University, Benares
7. Ahilyadevi High School for Girls, Poona

**NOTE:-**  
 1927 COURSE SHOWN THUS .....  
 1943 " " " " .....  
 WORK DONE BEFORE FLOODS OF 1944 SHOWN THUS .....  
 REPAIRING THE DAMAGED GROYNES AT RAIPUR - .....  
 INCLUDING STRENGTHENING GROYNES 1944-45 AS SHOWN THUS .....  
 REPAIRING GROYNES NO 4A, 5 & 5 A UPSTREAM OF THE  
 PICK-UP WEIR AT RAIPUR SHOWN THUS.....

**FIG. 2**  
**PLAN SHOWING TRAINING WORKS**  
**IN THE KHARI RIVER ON THE**  
**UPSTREAM OF THE PICK-UP WEIR**  
**AT RAIPUR**  
**SCALE. 300' TO AN INCH.**

**1945**



TRACED  
 BY L. B. B. NISHPANDER 8-2-50

CHECKED BY M. SINGH

## SECTION A—THEORY AND PRACTICE OF MODEL EXPERIMENTS

### 2. PRACTICE

#### (iv) Limitations of model experiments and comparison of models and prototype results

##### 1. TRAINING THE KHARI RIVER AT RAIPUR (DISTRICT AHMEDABAD)

###### 1. *Introduction*

The Khari river at Raipur was tending to outflank the pick-up weir near the village Raipur and this Station was approached for advice regarding the alignment and length of groynes proposed to prevent this tendency. The advice given by the Station was acted on and the results to-date are very satisfactory as will be seen from the note written by Shri N. C. Balsara, Superintending Engineer, North Gujarat Circle, and reproduced below.

###### 2. *Note by Shri N. C. Balsara*

The pick-up weir was constructed across the river Khari at Raipur in 1884 to feed the Khari cut canals. At the time of construction the river was straight for about 100 ft. upstream of the weir and the weir was at right angle to the general direction of the river flow. However, after the catastrophic floods of 1927 in Gujarat, the villagers of Raipur constructed a groyne (groyne no. 3) for the purpose of affording protection to the village which is situated just on the right bank of the river, and thereafter the current of the Khari showed a tendency to change its course. It gradually swung to the right bank with the bend upstream of the weir. The effect of this progressive scour nearly reached the weir site. It was feared that if the tendency of the river in shifting towards the right bank was not checked the weir might be outflanked. Fig. 2.

In 1933, the work of training the river was undertaken with the object of deflecting its course to its original position in 1884. Only one sand groyne 200 ft. long was constructed with ends protected by wooden bullies (round teak rafters) at a cost of Rs. 2,000 (groyne no. 1 in Fig. 2). It was only 200 feet long but was later extended to 300 ft. as mentioned below. Babul plants were also raised in the river bed near the bank with a view to stabilising the bank. These measures were successful for some time but in 1940 some protective works were considered necessary and accordingly one more sand groyne (groyne no. 2 in Fig. 2) was constructed and the first one was extended at a total cost of Rs. 3,300. However, all these protective works were washed away in the floods of 1941. Even the approach channel to the Khari cut canal and the head regulator were in danger of being scoured.

The protective works were started fresh in 1942, the sand groynes (groynes nos. 4 and 5 in Fig. 2) at right angles to the original course were constructed with their noses protected with wooden stakes. The original course of the river was also excavated to a width of 100 ft. to serve as a training channel. The cost of work was Rs. 4,000. The floods of the same year washed away the groynes partly. Before the next monsoon (1943) the five groynes (nos. 1, 2, 3, 4 and 5) were reconditioned and strengthened with a cover of sand bags. The river bed was also re-excavated along its original course. The cost of these works was Rs. 10,000.

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It was observed during the monsoon of 1943 that the works were still susceptible to damage and could not be depended upon. It was then considered that a series of 5 groynes as shown at nos. 3, 3A, 4, 4A and 5 in Fig. 2 should be constructed. Groynes nos. 3, 3A, 4 and 4A were to be timber groynes 6 ft. high above bed level and the groyne no. 5 was proposed to be pitched with bricks on the slopes and protected by a rubble apron at the toe. An additional groyne no. 6 on the left bank was proposed above the old groyne no. 1 (425 ft. u/s).

These proposals were referred to the Director, Hydrodynamic Research Station, Poona. He suggested that the groyne no. 5 should not be made Pacca and that the whole of the groyne should be built in timber as porous groynes. The construction of groyne no. 6 was not approved by him. Later on the Director suggested an additional timber groyne no. 5A. Ultimately the following proposals were adopted:—

Long porous groynes nos. 3, 3A, 4, 4A, 5 and 5A (6 nos.). Short porous groynes nos. 4B and 5B (2 nos.). Two short groynes nos. 3B and 3C constructed after monsoon of 1945.

Accordingly six porous groynes along with 4 short groynes were constructed with a frame work of round wooden rafters as shown in Fig. 3. The panels of the wooden framework were closed with intertwined brushwood as shown in Fig. 4 keeping the whole groyne porous. The work was completed before the monsoon of 1944.

The noses of three of the lowest groynes were washed away during the first year of floods and there was initial silting between most of the groynes. These noses were subsequently rebuilt giving them an up-stream slant as advised by the Director (*vide* fig. 2). The space between successive groynes is silted up progressively. The groynes are being carefully attended to and any defects observed are being quickly remedied.

The success of these groynes is due mainly to the porous nature of their construction and the natural process of slow deposition and consolidation which it induces. The land reclaimed is being stabilised by Babul plantation. Thus the scouring action of the river is effectively checked. A strip, 200 ft. wide, in the portion, where the river has extended the most towards the right bank, has now been well stabilised. New strips of land reclaimed are being similarly stabilised and protected.

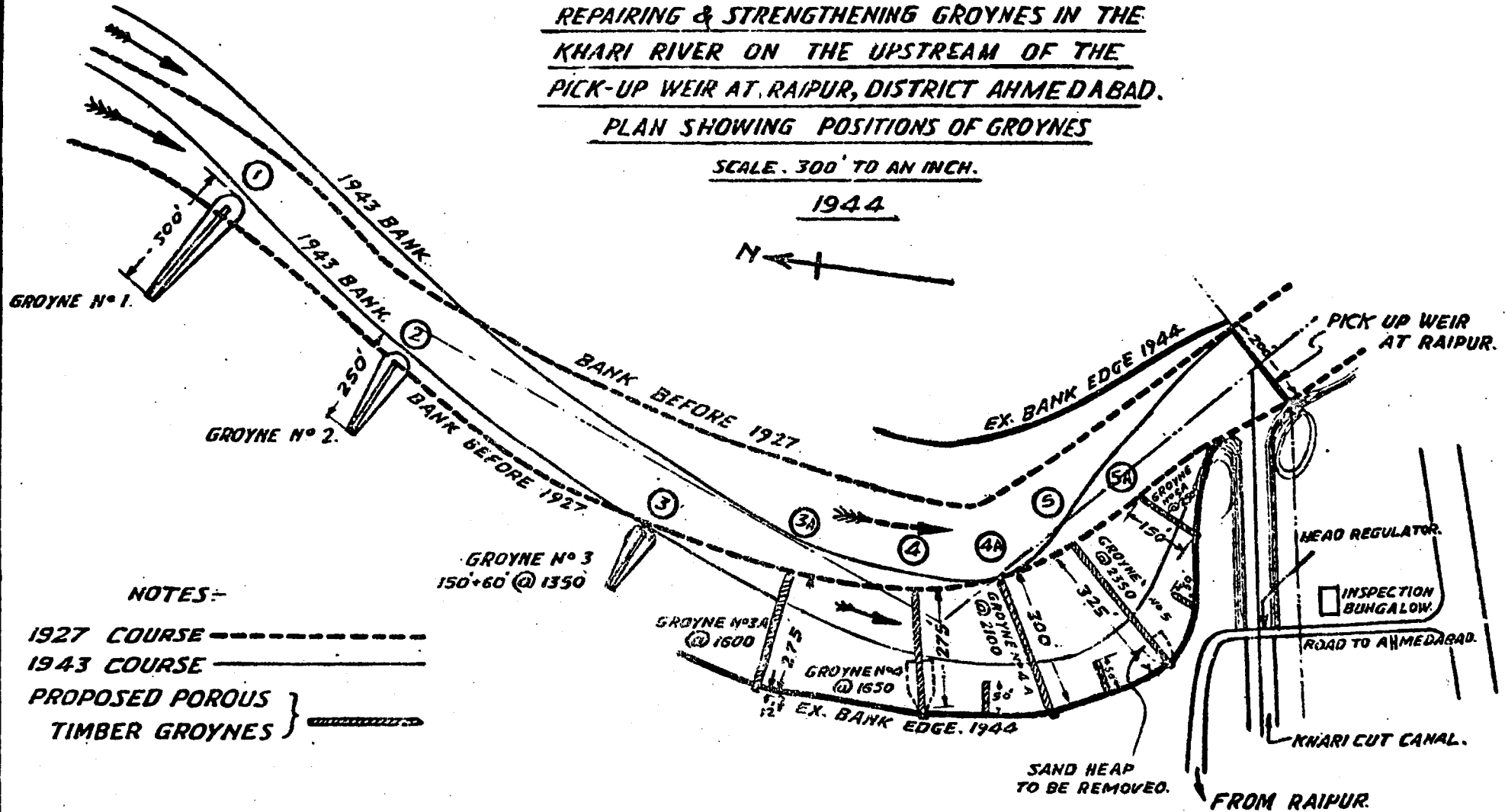
Photos- 5802  
to 5806.

Some photographs of the site are appended which show clearly how the porous groynes helped to reclaim the lost land in stages and in training the river.

**FIG. 3**  
**REPAIRING & STRENGTHENING GROYNES IN THE**  
**KHARI RIVER ON THE UPSTREAM OF THE**  
**PICK-UP WEIR AT RAIPUR, DISTRICT AHMEDABAD.**  
**PLAN SHOWING POSITIONS OF GROYNES**

SCALE - 300' TO AN INCH.

1944



NOTES:-

1927 COURSE -----

1943 COURSE \_\_\_\_\_

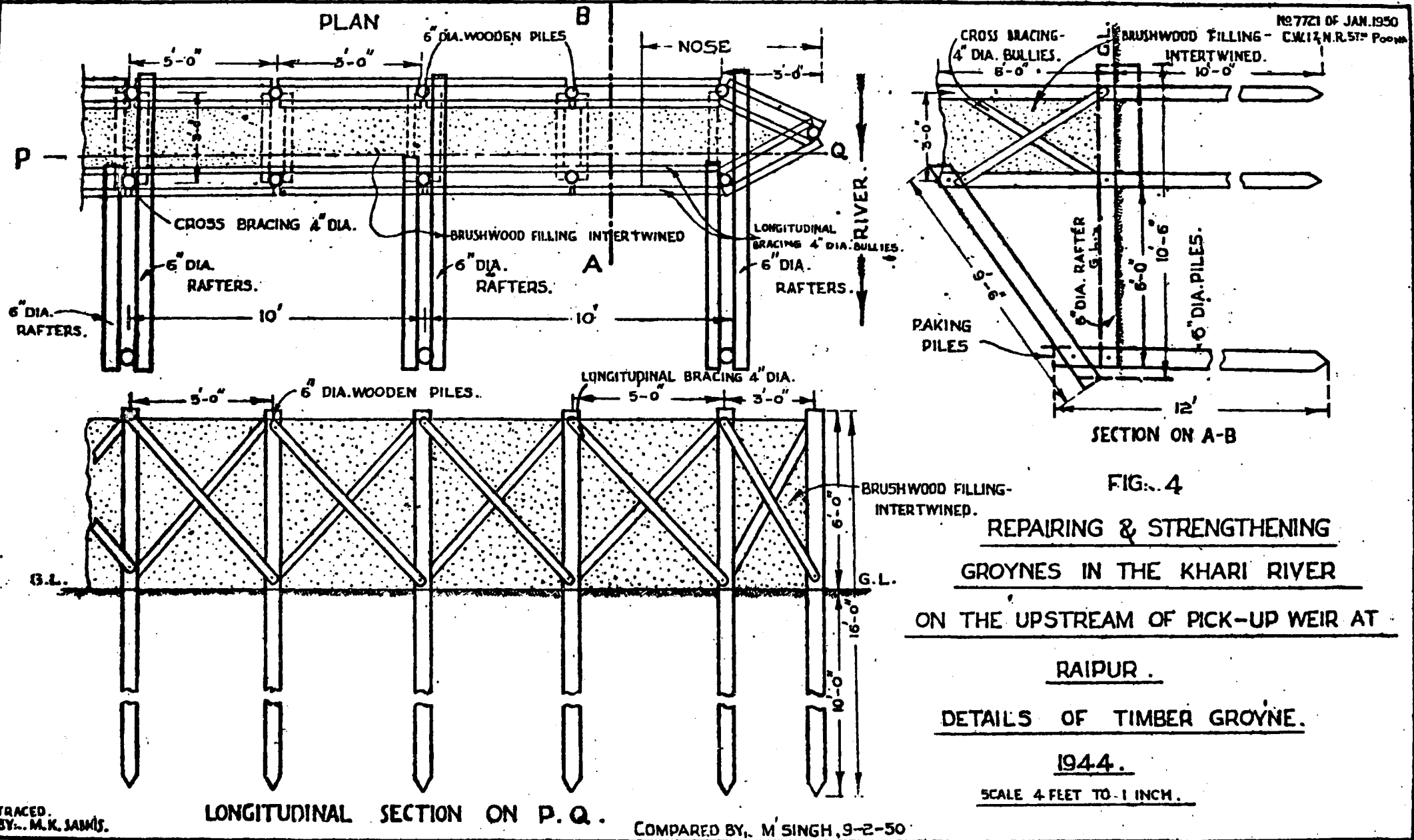
PROPOSED POROUS  
TIMBER GROYNES } 

BY SAPKAL

CHECKED BY M. SING.

SAND HEAP  
TO BE REMOVED.

FROM RAIPUR.



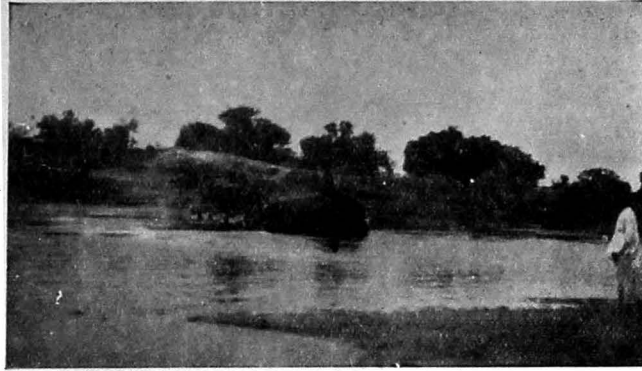


Photo No. 5805—Taken from the opposite bank showing the position of Groyne No. 3 after the flood had subsided.



Photo No. 5806—Showing the growth of Babul plantation on the reclaimed land.

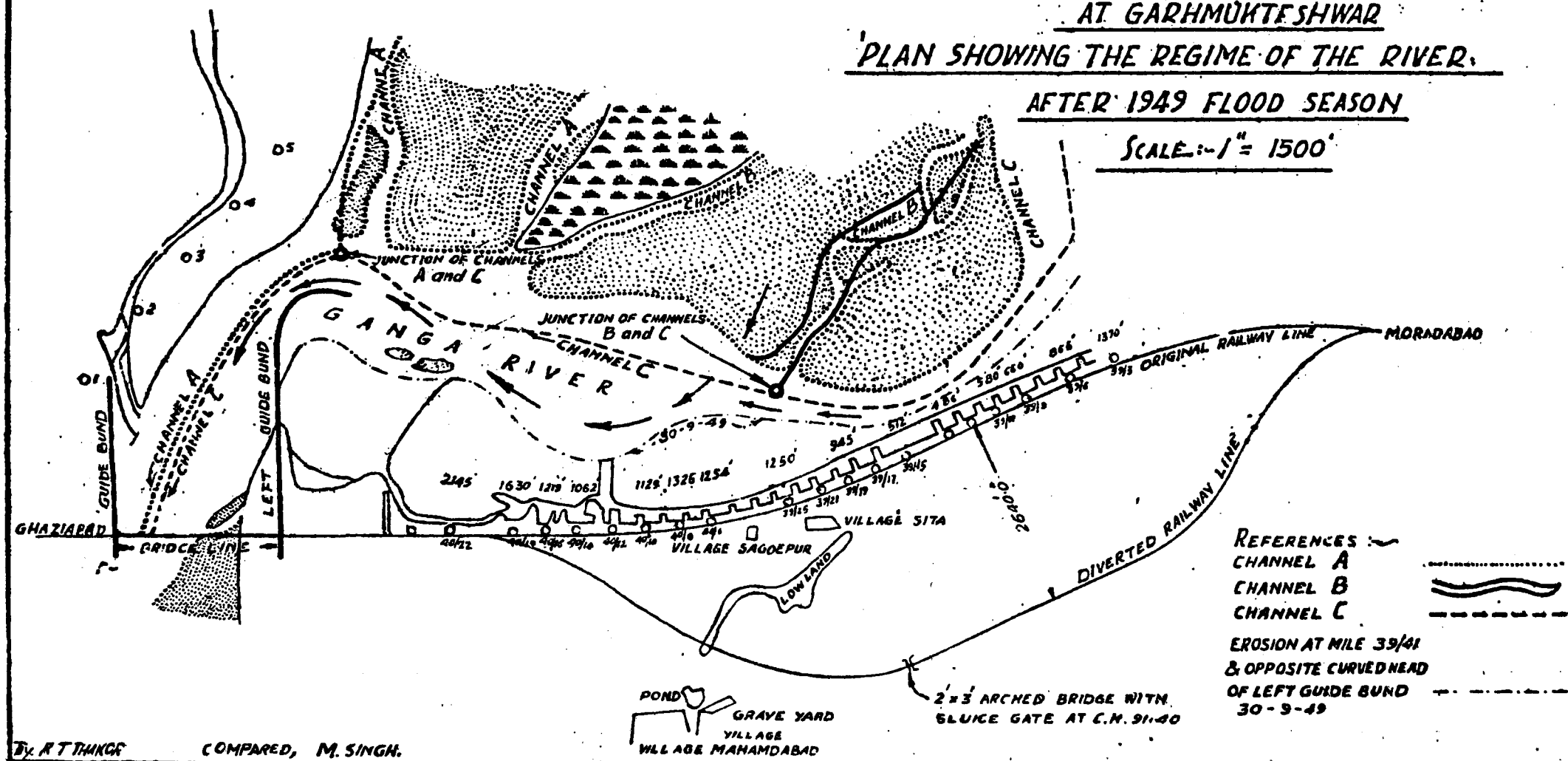


FIG. 5

E.I. RAILWAY DELHI BRANCH.  
BRIDGE NO. 52 ACROSS THE GANGA RIVER  
AT GARHMUKTESHWAR

PLAN SHOWING THE REGIME OF THE RIVER,  
AFTER 1949 FLOOD SEASON

SCALE: 1" = 1500'



REFERENCES:

- CHANNEL A
- CHANNEL B
- CHANNEL C

EROSION AT MILE 39/41  
& OPPOSITE CURVED HEAD  
OF LEFT GUIDE BUND

30-9-49

By R. T. THAKUR COMPARED, M. SINGH.

## 2. TRAINING OF THE GANGA RIVER AT GARHMUKTESWAR :

### FIELD RESULTS OF MEASURES ADOPTED AFTER MODEL EXPERIMENTS

The Ganga river had approached very dangerously the railway track near Garhmukteswar railway bridge (E. I. Railway) during the 1948 floods and there was every danger of bridge being outflanked. To prevent this catastrophe it was recommended by the Research Station and confirmed later by model experiments that the left guide bund should be extended. This advice was acted upon and the guide bund was extended before the 1949 floods. This training measure proved to be very effective as will be seen by the following description of the behaviour of the river during 1949 floods. The model experiments are fully described in chapter 6.

The highest flood level during the monsoons of 1949 rose to R. L. 653.25 against the maximum flood R. L. 658.0 at the Bridge and the discharge then was 1.43 lakh cusecs with a maximum velocity of 7.41 ft./sec.

It was observed that there was no scour in the left bank channel, nearest the railway embankment. Heavy silting took place in this channel and in the area between this channel and the embankment. This channel became a mere rivulet after 1949 floods.

Channel 'B' further to the west and away from the railway embankment which was discharging a negligible quantity of water in 1948, developed into a main stream after the floods of 1949. During the latter part of the floods, this channel bifurcated and gradually the right side bifurcation developed. During earlier floods when channel nearest the railway embankment silted and channel 'B' developed, there was some erosion on the left bank of the river opposite mile 40/12 but this almost stopped when the channel bifurcated, in the latter part of the floods.

Fig. 5 and  
Photos 5800  
and 5801.

Opposite the curved head the right bank scoured by widening the constricted channel by as much as 400 feet.

The extended portion of the guide bund stood well and the apron on the downstream face between chains 38-39 and 34-35 launched.

No erosion took place at the head of the right guide bund where a sufficient stock of boulders had been kept in reserve.

It was indeed fortunate that the river did not rise to R. L. 658.0 in the 1949 flood season and thus the training measures were not subject to a severe strain. With the conditions of the river as they are now it seems that the training works will now be successful for even the maximum flood.

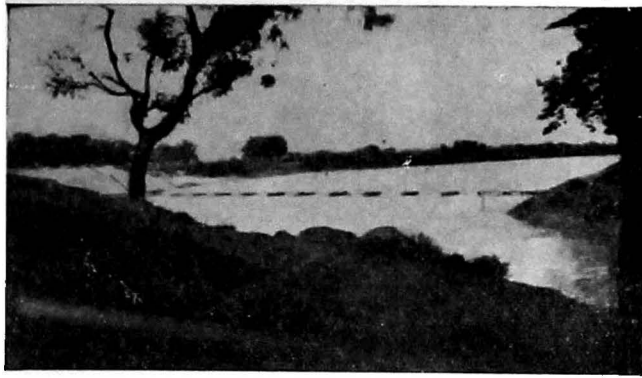


Photo No. 5802—Showing the pickup weir looking downstream and in partial floods.

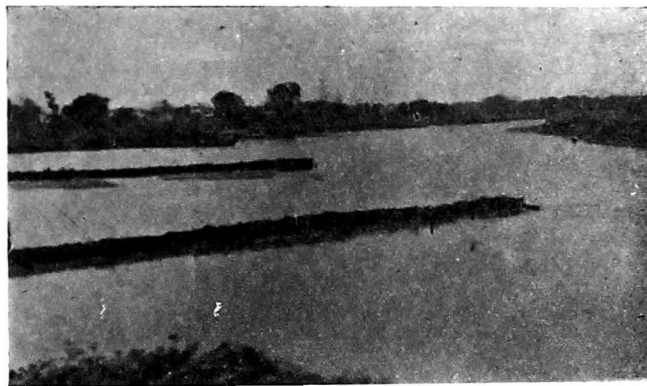


Photo No. 5803—Showing Groyne Nos. 4-A and 5, during floods.

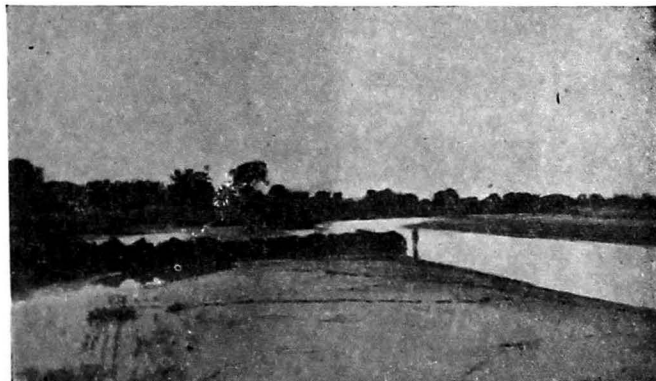


Photo No. 5804—Showing silt deposit between Groyne Nos. 4-A and 5 in the foreground after floods had subsided.

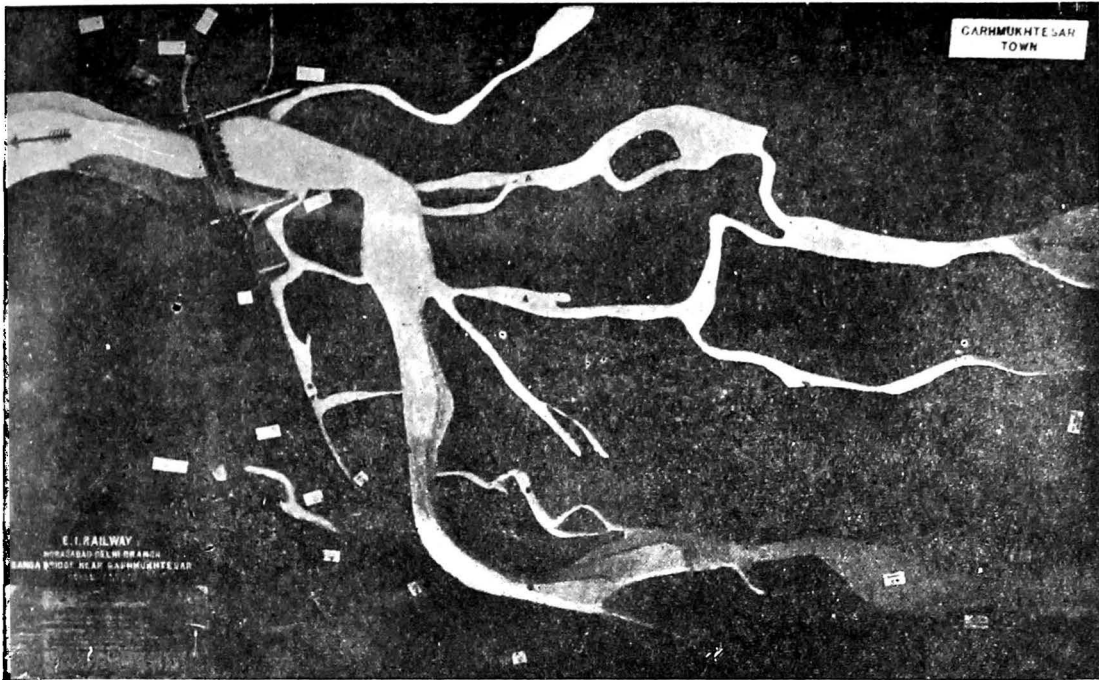


Photo No. 5800 - River GANGA at Garhmukteswar Bridge showing conditions before protective works were carried out - River approached dangerously the railway track; conditions before 1949 flood season.



Photo No. 5801 - The River Ganga Garhmukteswar Bridge (E. I. Rly.) showing the extended left guide bank and conditions after 1949 flood season.

## SECTION B—HYDRAULIC WORKS

### 1. RIVER CONTROL

#### (i) Flood Protection Works including scour and erosion

### 3. PREVENTION OF DAMAGE TO SURAT WATERWORKS ON TAPTI AT NANA VARACHHA

#### 1. Introduction

Surat Waterworks are situated on the left bank of the Tapti river near Nana Varachha village about 10½ miles upstream of Surat and consist of percolation wells in the river bed connected by an inter-connecting pipe system to a jack well on the bank from which water is pumped for further supply to the city. Continuous, though gradual, erosion of the left bank in the vicinity of the pump house has been occurring for a considerable period, but since 1944, owing to heavier floods, it is progressing at an alarming rate. The winter channel has also recently been found to hug the pump house bank and the collecting pipe system is thus endangered as at places it is protected by sand not more than a couple of feet deep. The Public Health Engineer to the Government of Bombay consulted the Station for devising suitable protective measures for the Waterworks which, being of vital importance, have to be immediately and adequately safeguarded. All available data were thereupon studied and an officer inspected the site to get additional first hand information and to discuss with the local officers several points needing further elucidation and clarification.

#### 2. Inspection of the Tapti river near Surat Waterworks

The river has a gentle bend at the Waterworks. From the available plans it is apparent that the river has not appreciably deviated from this general configuration since 1917. On visual inspection it was clear that the bend, though not so pronounced, is still sufficiently developed to pin down in high floods the deep channel along the left, i.e., Nana Varachha bank. The plan of 1917 shows the water channel along the right bank opposite the waterworks. It is to be remembered, however, that this is not an indication of the course of the main flood channel being also along the same alignment since the curvature would not permit the main flow in floods to leave the outer left bank in the bend. Development of a different winter channel apparently was due to the sand bars and islands thrown across the flood channel in the falling floods.

Observations on the site suggested that in designing a suitable training measure the following features would have to be borne in mind:—

(i) There does not exist any natural control point upstream or downstream of the pump house in the neighbouring river reach, in the shape of rock or kunkar outcrop or hard soil formation which can be used for anchoring the training works.

(ii) At the pump house, for arresting erosion, nothing can be done except providing pitching over the bank. Pitching has already been provided at the time of construction of the pump house in 1826 and a provision of Rs. 8,000 is made for new pitching to be provided to protect the extension of the waterworks now under construction. The pitching cannot obviously guarantee that the river will not cut into the bank above or below the pitching and if this happens the scour at the toe of the pitching around the protrusion thus formed would be accentuated and maintaining and reinforcing it each year will become prohibitive in cost.

(iii) The bed sand being fine, deep and extensive scour will be caused by the protective measures. They would have, therefore to be located sufficiently upstream or downstream, with an adequate, if not liberal, margin of safety, from the filtration wells and the collecting pipe system; lest the latter be endangered.

### 3. Model Experiments

Several alternative training measures were contemplated. To make a choice from these and to test the efficiency and finalise the details of the one ultimately selected, model experiments were resorted to.

Experiments were accordingly conducted in a vertically exaggerated model of the Tapti river originally constructed in connection with the protection of Rander and Surat towns and the lands of Bhatpur village lower down. The scales of the model were:—

Horizontal scale = 1/330

Vertical scale = 1/72

Vertical exaggeration = 4.6

Discharge scale = 1/20,200

Slope scale = 2.5.

To start with, a submerged vane pointing downstream was tested on the right bank half a mile upstream of the waterworks. This was expected to guide and deflect the bed sand towards the left bank at the waterworks which would protect the pipe system and at the same time prevent the left bank erosion. The vane did act as expected but owing to curvature effect, the accretion could not occur near the waterworks where it was especially desired.

A repelling spur was next tried on the left bank. Prima-facie position of the spur below the waterworks was unsuitable (which was later confirmed by the model) since to provide protection to the bank by virtue of the resulting afflux the spur would have to be too near the waterworks. This was not permissible as the scour at the nose would damage the wells and the pipe system. Further downstream position would, on the other hand, necessitate a very long spur to provide equal protection to the bank and to prevent the bed scour. This would not only be prohibitive in cost but would also cause excessive constriction of the river. Different positions above the waterworks were, therefore, tried. Finally a 200 ft. long spur normal to the bank with top at half flood level and located half a mile upstream of the pump house was found to provide sufficient protection at a reasonably low cost. Results of the experiments with this spur are given below:—

#### Model Observations

Fig. 6  
(a & b).

Fig. 6 (a & b) show the flow line with and without the spur. With the spur, the throw-off of the eroding flood flow was sufficiently away from the waterworks bank.

Fig. 7  
(a & b).

Fig. 7 (a & b) show velocity distribution at the cross-sections of the river in the vicinity and upstream of the pump house. These indicate that, above the spur, velocities near the left bank will be slightly increased due to attraction but at the pump house below they will be considerably reduced, say by about 15 per cent. Opposite the pump house, near the right bank, velocities were correspondingly higher and the river bed was also scoured to a depth of about 4 ft. Near the pump house the model showed a tendency for accretion, the actual rise of bed levels here being equivalent to 6 ft.

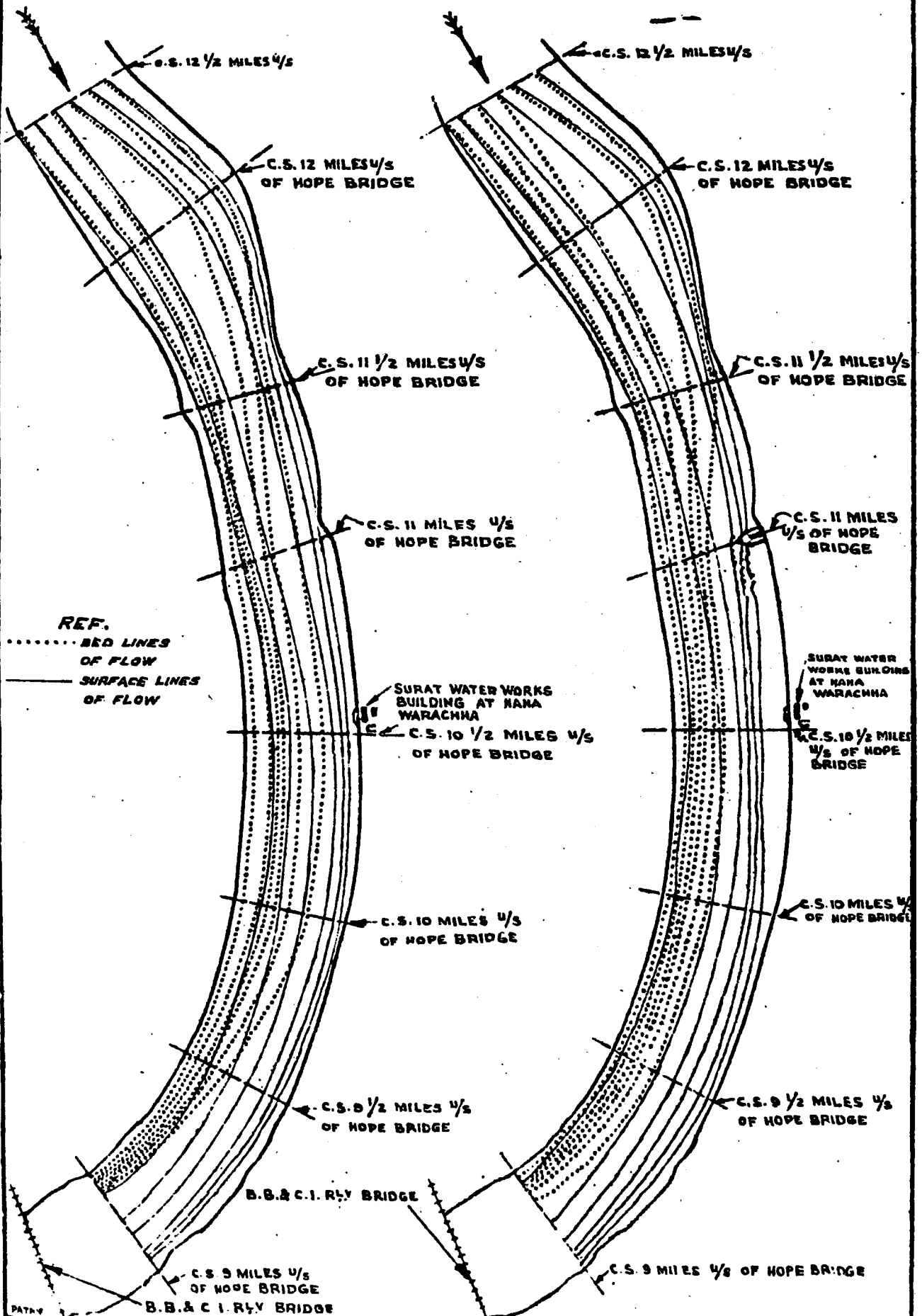
#### Damage to Surat Works in October 1949

Proposals for protective works were already finalised by model experiments explained above; but before these could be forwarded to

**FIG. 6 (a) & (b)**  
**PART PLAN OF TAPTI RIVER**  
**NEAR SURAT WATER WORKS AT NANA VARACHHA**  
SCALE 1" = 6' ± 1980

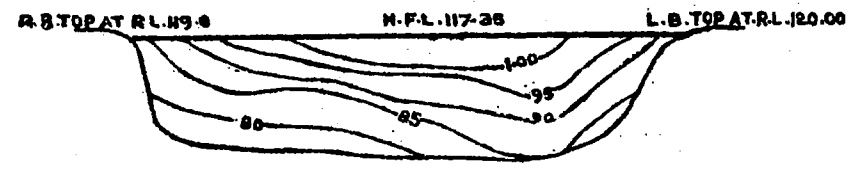
**(a)**  
FLOW LINES UNDER  
EXISTING CONDITIONS

**(b)**  
FLOW LINES WITH 200 FT. LONG HALF  
SUBMERGED GROUPE ON THE LEFT BANK  
1/2 MILE UPSTREAM OF THE WATER WORKS

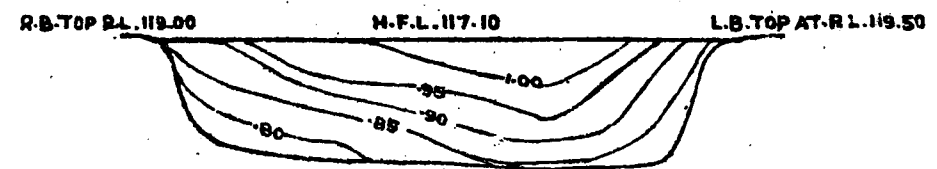


**FIG: 7 (a)**  
**SHOWING ISOVELS IN CROSS SECTIONS**  
**AT AND ABOVE THE SURAT WATER WORKS**  
**NEAR NANA VARACHHA UNDER THE EXISTING CONDITIONS**

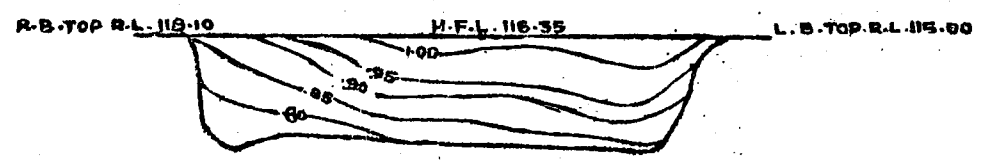
SCALE: HOR: 1" = 2' = 660'  
VER: 1" = 1' = 72'



C.S. BETWEEN 1/2 MILE AND 1 MILE  
UPSTREAM OF THE SURAT WATERWORKS



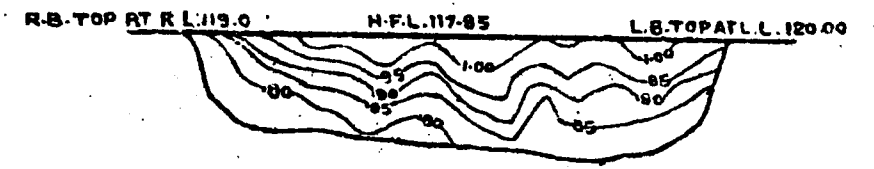
C.S. BETWEEN SURAT WATERWORKS  
& 1/2 MILE U/S OF IT (1590 FT. U/S OF W.WORKS)



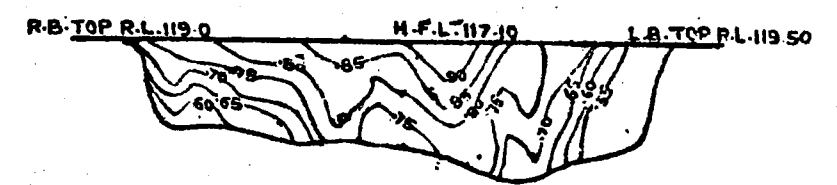
C.S. 10 1/2 MILES U/S OF HOPE BRIDGE  
AT THE SURAT WATERWORKS.

**FIG-7(b)**  
**SHOWING ISOVELS IN CROSS SECTIONS**  
**AT AND ABOVE THE SURAT WATER WORKS**  
**NEAR NANA VARACHHA WITH THE 200 FT LONG**  
**1/2 MILE UP STREAM OF THE WATER WORKS**

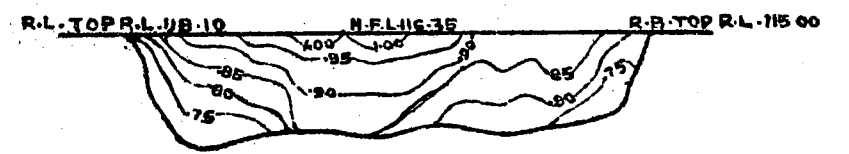
SCALE: HOR: 1" = 2' = 660'  
VER: 1" = 1' = 72'



C.S. BETWEEN 1/2 MILE AND 1 MILE UPSTREAM OF  
THE SURAT WATERWORKS



C.S. BETWEEN SURAT WATERWORKS  
AND U/S OF IT (1590 FT. U/S OF W.WORKS)



C.S. 10 1/2 MILES U/S OF HOPE BRIDGE  
AT THE SURAT WATERWORKS



## DAMAGE TO SURAT WATERWORKS ON TAPTI

the Government of Bombay, heavy floods were reported to have caused some damage to the Waterworks necessitating drastic curtailment in the water supply to the Surat town. Accurate soundings were not available, but on inspection it was found that the river bed, near the left bank, in the vicinity of the wells and collecting pipes appeared to be getting scoured progressively year after year and it was felt that some emergent measures before the submerged groyne could be constructed were essential to check this tendency and restore the bed levels to provide adequate cover for the pipe system. Sixteen feet long steel sheet piles were available at the site and it was suggested on the basis of model observations that these could be used for this purpose. It must be mentioned that though the pile line can cheaply and immediately prevent scour at the pipe system, it cannot arrest bank erosion and additional measure in the shape of the submerged groyne will be necessary to achieve this end.

### *The pile line*

The pile line should be aligned below the pipe system as shown in Fig. 8. Piles should be driven into the river bed at least two-thirds of their length and should be protected by stone or concrete block pitching, not lighter than about 120 lbs. each, as shown in Fig. 9.

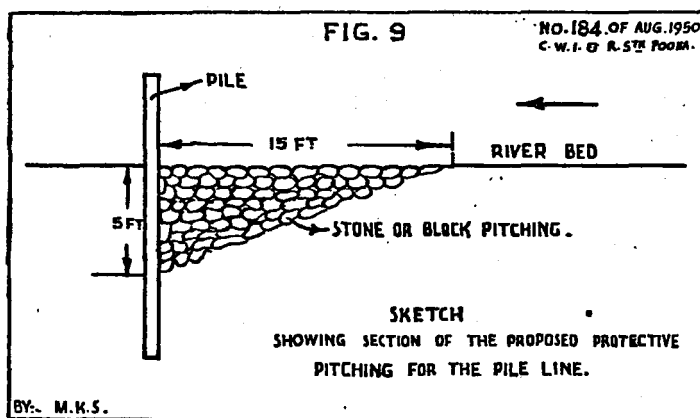


Fig. 9

### *Recommendations*

1. For arresting further bank erosion and to prevent scour of the river bed in the vicinity of the waterworks, a submerged spur, 200 ft. long, half a mile upstream of the pump-house and on the same bank will be suitable. Details of the spur are given in Appendix I. Appx. I.
2. For immediate and local prevention of bed scour at the collecting pipe system, the pile line may be constructed as described above.
3. No observations of bed levels in the flood have so far been made. It is very desirable to make these observations at least for one year of high flood. The observations should preferably be made at about two-thirds of the falling flood stage when the scour is generally the maximum.

## APPENDIX I

### *Details of the proposed 200 ft. long spur*

1. Length of the spur = 200 ft.
2. Top level at R. L. 92.00 (H. F. L. = 116.35)
3. Top width = 10 ft.
4. Side slopes 2 to 1.
5. Nose slope 3 to 1.

Figs. 10, 11  
and 12.

Fig. 10.

6. Position and Orientation—Half a mile above the pump house, on the left bank as shown in Fig. 10.

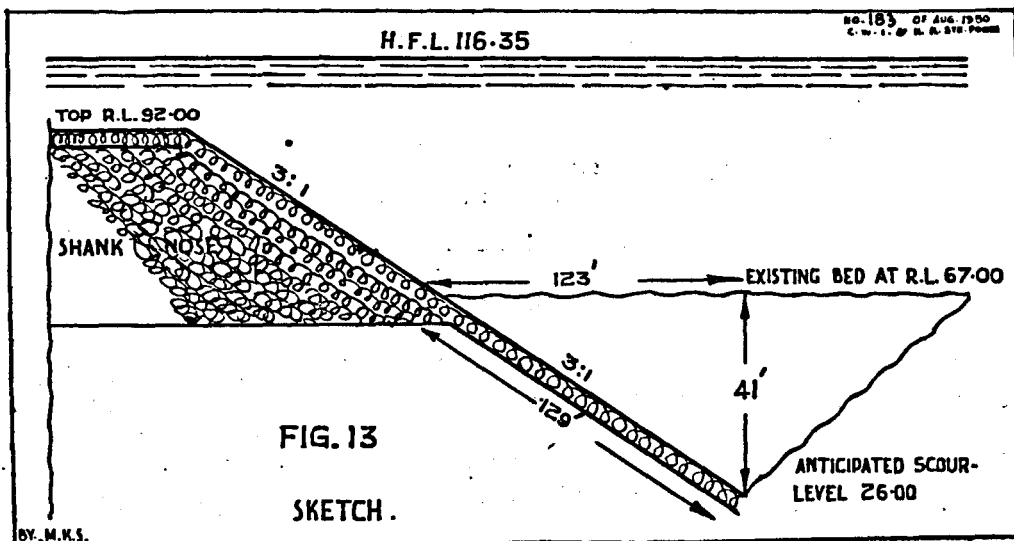
7. Material:—Shank may be built of sand in the core while the top and sides should be finished by 3 ft. thick layer of hand packed one man (about 1 cft.) or heavier stones. The nose should be wholly in heavy stones as it will be open to heavier stresses.

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

In the model, the maximum scour obtained was 1/15 feet = 83 feet below H.F.L.

The analysis and study of the available data about scour depths at the noses of 'full height' groynes show that the maximum depth below H.F.L. to be anticipated is from 2 to 2.75 R where  $R = 473 \left(\frac{Q}{f}\right)^{\frac{1}{3}}$ , Q being maximum discharge in cusecs and 'f' the sand factor =  $1.76\sqrt{m}$ , where 'm' is the mean diameter of the bed material in mm.<sup>(1)</sup> According to this, maximum scour depth with Q = 750,000 cusecs and m = 0.25 mm. works out to 90 feet to 123 feet below H.F.L. As the scour depth = 2R is nearly equal to that observed in the model the apron design is made for 90 feet depth. (Fig. 13).

Fig. 13.



Allowing 33% extra thickness for apron for absence of soling and it being not hand pitched.

$$129 \times 4 = 516 \text{ cft.},$$

$$\text{say } = 567 \text{ cft./ft.}, \text{ allowing for loss in dispersion.}$$

The maximum width may be 100 feet tapering down to 20 feet at the shank.

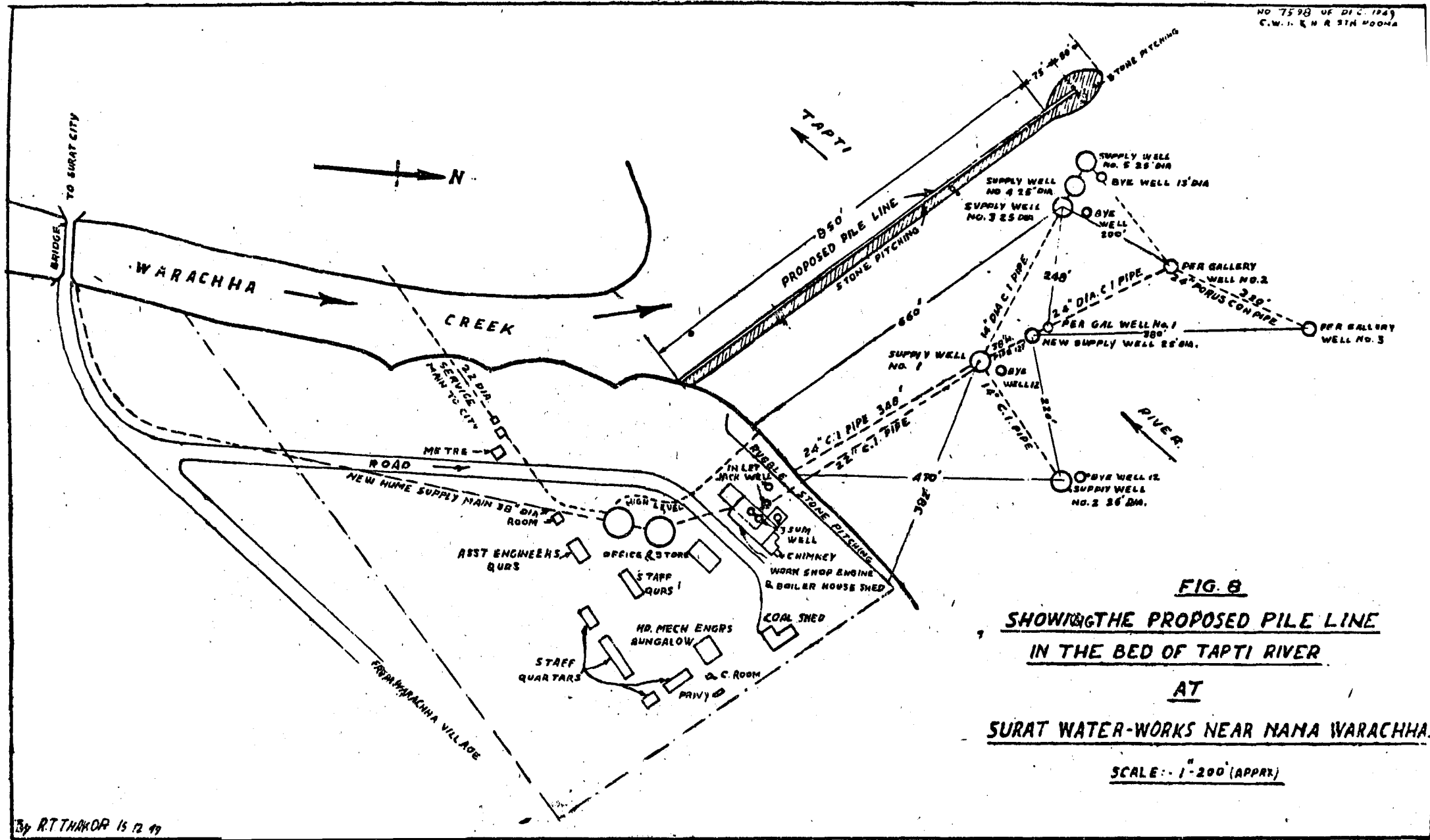
$$\text{Therefore, the average thickness} = \frac{567}{100} = 5.67 \text{ say } 5' - 9''.$$

The apron thickness at the toe of the slope should be 5 ft. and at the outer end 6' 6". The shape of the apron in plan should be as shown in Fig. 11 which generally confirms the probable scour-hole indicated by the model, See photo No. 5514.

Fig. 11.

Photo 5514.

(1) Annual Report (Technical), 1944, CWNBS, p. 78 and C. B. I. Publication No. 12, p. 155.

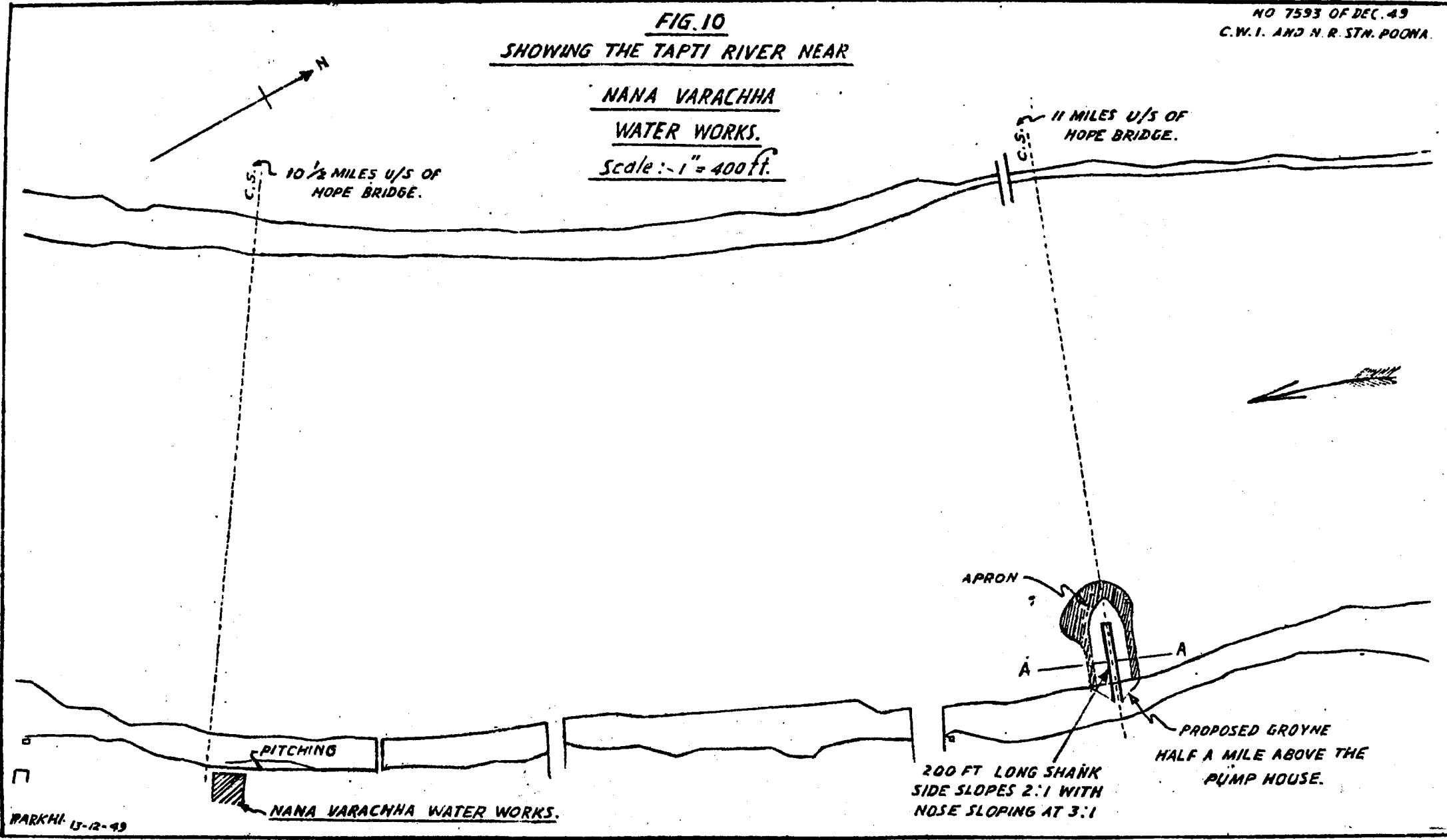


**FIG. 8**  
**SHOWING THE PROPOSED PILE LINE**  
**IN THE BED OF TAPTI RIVER**  
**AT**  
**SURAT WATER-WORKS NEAR NAMA WARACHHA.**  
**SCALE: 1" = 200' (APPRX)**

FIG. 10  
SHOWING THE TAPTI RIVER NEAR

NANA VARACHHA  
WATER WORKS.

Scale :- 1" = 400 ft.



PARKHI 13-12-49

NANA VARACHHA WATER WORKS.

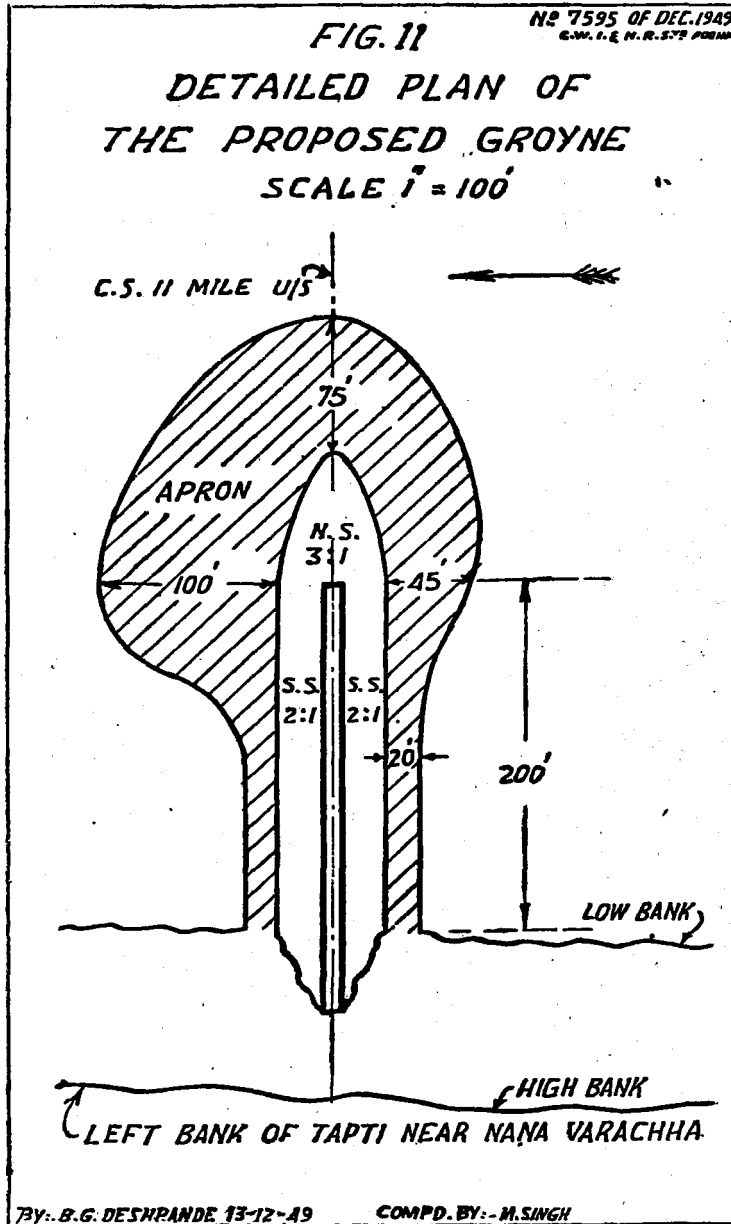
200 FT LONG SHANK  
SIDE SLOPES 2:1 WITH  
NOSE SLOPING AT 3:1

PROPOSED GROUYNE  
HALF A MILE ABOVE THE  
PUMP HOUSE.

10 1/2 MILES U/S OF  
HOPE BRIDGE.

11 MILES U/S OF  
HOPE BRIDGE.

DAMAGE TO SURAT WATERWORKS ON TAPTI

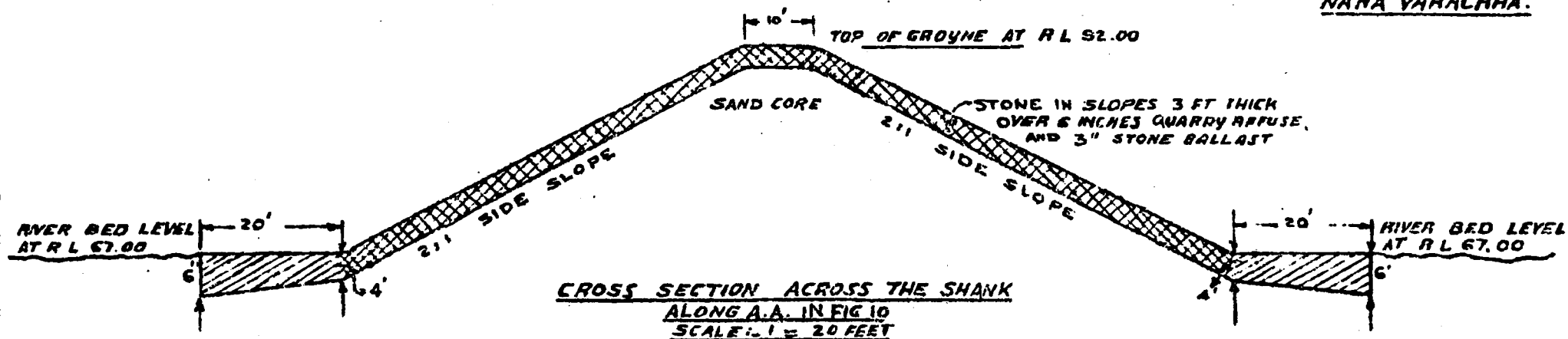


DESIGN CALCULATIONS

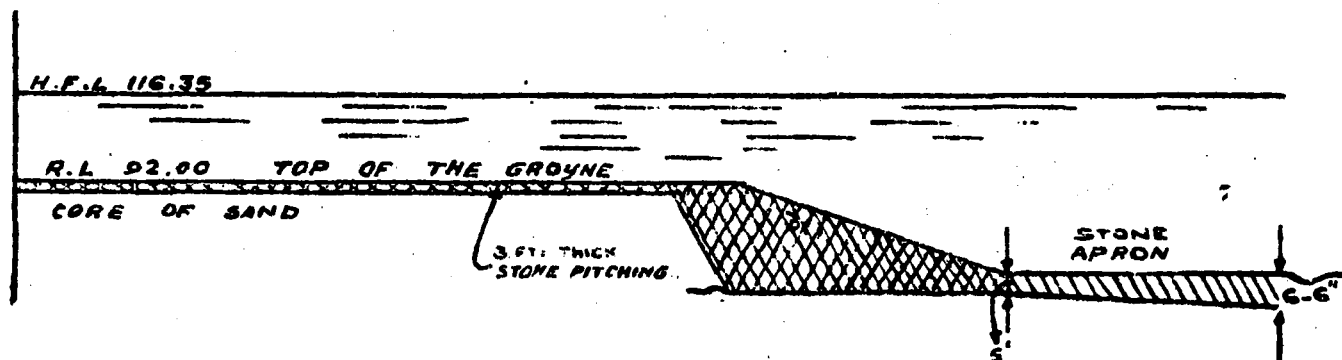
H.F.L. 116.35  
 SCOUR DEPTH 1.3 D = 59 FT. BELOW H.F.L.  
 APRON AT BED LEVEL I.E. R.L. 67.00  
 ∴ SCOUR DEPTH BELOW APRON 10 FT.  
 ∴ LENGTH ALONG SLOPE =  $10 \times 2.24$   
 SAY = 22 FT.  
 CFT. OF APRON STONE  $5 \times 22 \times 4 = 440$   
 + 10% SAY 100 CFT. / FT.  
 WITH WIDTH OF 20' AVERAGE THICKNESS = 5 FT.  
 KEEPING 4 FT. AT THE TOE, THICKNESS AT THE END = 6 FT.

DRG. NO. 796 OF DATE 1925  
 C.W.T. & N.R. STATION, PUNNA.

**FIG. 12**  
**SHOWING DETAILS OF THE**  
**PROPOSED GROUYNE IN THE**  
**TAPTI RIVER NEAR**  
**NANA VARACHHA.**



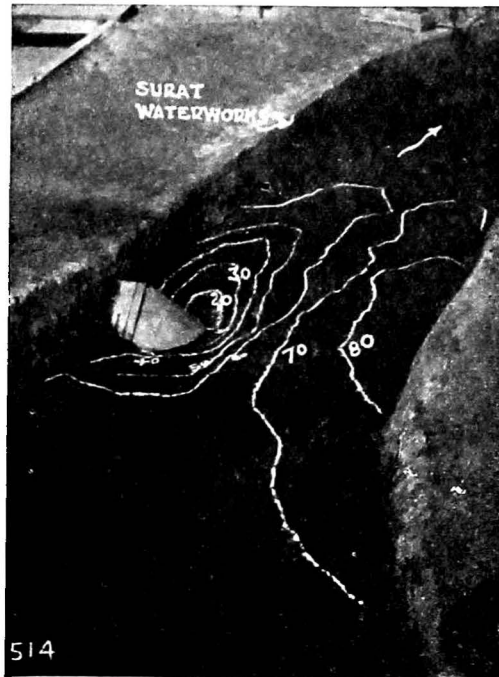
LEFT BANK.



By Manchundia

Photo No. 5514

Proposed Groyne



Showing the bed scour caused in the model by the Groyne proposed for the protection of river bank at Surat Water Works near Nana Varachha, looking downstream.

Photo No. 5597



- Upper curved vane.
- Deep channel below the first vane.
- Lower vane.
- Deep channel below the lower vane.

Showing the deep channel caused by the submerged vanes on left bank near Umra village, looking upstream.



## (ii) River Training Works

### 4. PROTECTIVE WORKS FOR ARRESTING EROSION ON THE RIGHT BANK NEAR BHATPUR ON THE TAPTI

#### 1. Introduction

Very valuable fertile land, about one square mile in area, and costing about Rs. 3,000 per acre, has been eroded by the Tapti river in the embayment near Bhatpur as can be seen from Fig. 14. Main cause of this erosion is strong prolonged action in floods, though tidal action is partially responsible in accentuating the damage. With a view to stop further inroads into the bank different types of training works were tested in the model.

Fig. 14.

#### 2. Model tests and their results

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

Reducing the discharge in the Bhatpur channel can be achieved in several ways. The left arm can be induced to widen and deepen so as to increase its capacity; otherwise a new channel will have to be opened through the central sand island to relieve the Bhatpur channel. Both these remedies indicate attracting type of training works which have a tendency to attract and pin down the current by virtue of the scour holes formed by them.

The second alternative, viz. the shifting of the main current inside the river thus making it leave the bank throughout the Bhatpur embayment was found to require a long and heavily armoured groyne which would have been more expensive than the groynes suggested for protection of banks at Rander and Surat, or else a system of comparatively shorter groynes, each properly designed and suitably positioned, aggregate cost of which would appreciably have been higher than a single large groyne considered above and would have been more costly to maintain.

The other alternatives aimed at reducing the discharge of the Bhatpur channel were, therefore, tested in the  $\frac{1}{300} : \frac{1}{2}$  scale Tapti model.

Various types of attracting works were tried in the left bank channel at various positions, but the natural curvature of the river being unfavourable for development of the left channel, its discharge could not appreciably be increased.

Attempts were made to develop a new channel through the dividing sand 'Bela' by means of an artificial attracting island. A single island was found to be totally inadequate while a series of such islands—each costing not less than about 6 to 8 lakh rupees—were out of question on economic grounds. A pilot cut in conjunction with other training measures was also ruled out for the same reason.

Submerged vanes, single and in pair were then tested on the left bank as shown in Fig. 14. Of all the different measures contemplated for reducing Bhatpur channel discharge, two curved submerged vanes one below the other gave most promising results.

Fig. 14.

As will be seen from Photo No. 5597, a deep channel was formed along the left bank below the first upstream vane which was maintained

Photo  
No. 5597.

and developed by the next lower one. This deep channel was the result of the surface water being attracted and bed material being deflected by the curved submerged vane. The model indications were that the concentrated high velocity flow caused by the vanes would help the gradual erosion of the Bela in the river or else would develop a cut across the sand island thus short circuiting the loop. This behaviour could not, however, be reproduced in the model as the actual model velocities were too low to erode the island.

Increased action along the left bank below the first vane attendant with the high velocities and discharge concentration near this bank were however, definite. With the vanes, some bank erosion here is expected and suitable protection against this will have to be provided in the shape of stone pitching, if there are any important structures.

Appx. I.

The details of the vanes are given in Appendix I.

The proposed vanes though quite efficacious in considerably ameliorating the present conditions at Bhatpur may not provide complete relief and protection. The basic remedy for general bank erosion, lies in controlling floods by flood storage dams. Multi-purpose dams proposed in the Tapti Project when materialised would reduce the maximum flood discharge of Tapti to about 5 lakh cusecs, that is about half the present discharge in high floods and in the post-project period, erosion at Bhatpur is expected to be negligible.

Final decision as to whether it is worth the cost to provide for Bhatpur bank protection, for the present, can only be taken after carefully considering the stakes involved and examining the detailed estimates.

### 3. Recommendations

If it is decided to protect the bank in the Bhatpur embayment, submerged vanes as detailed in Appendix I should be constructed. In the first instance only one vane—the upstream one—may be built and its behaviour closely watched in the next flood season.

In view of the large expenditure incommensurate with the protection which will be afforded, it was decided not to proceed with the proposed work for the present.

### APPENDIX I

#### *Details of the proposed curved submerged vanes near Umra on the left bank in the Tapti*

Fig. 15.

(1) Positions, lengths and alignments of the vanes should be as in Fig. 15.

Fig. 16.

(2) Height of vanes above river bed should be one-third depth of flow at bankful stage of the river. From the sections available, the top of the upstream groyne should be at about R. L. 68.00 while that of the lower one should be at R. L. 72.00. The nose ends of the vanes for length of 100 ft. should be made sloping as shown in Fig. 16 in order to minimise the tendency for scour at the noses.

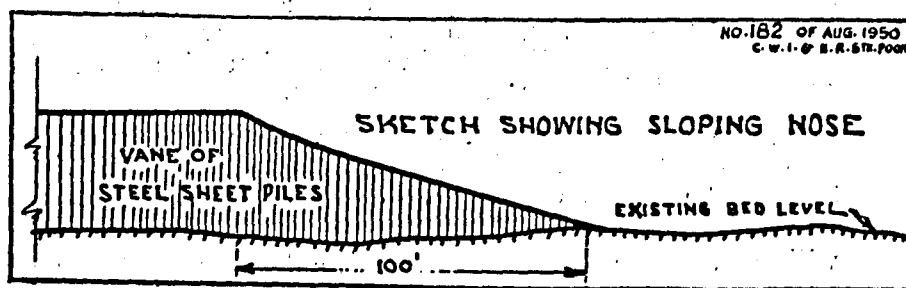
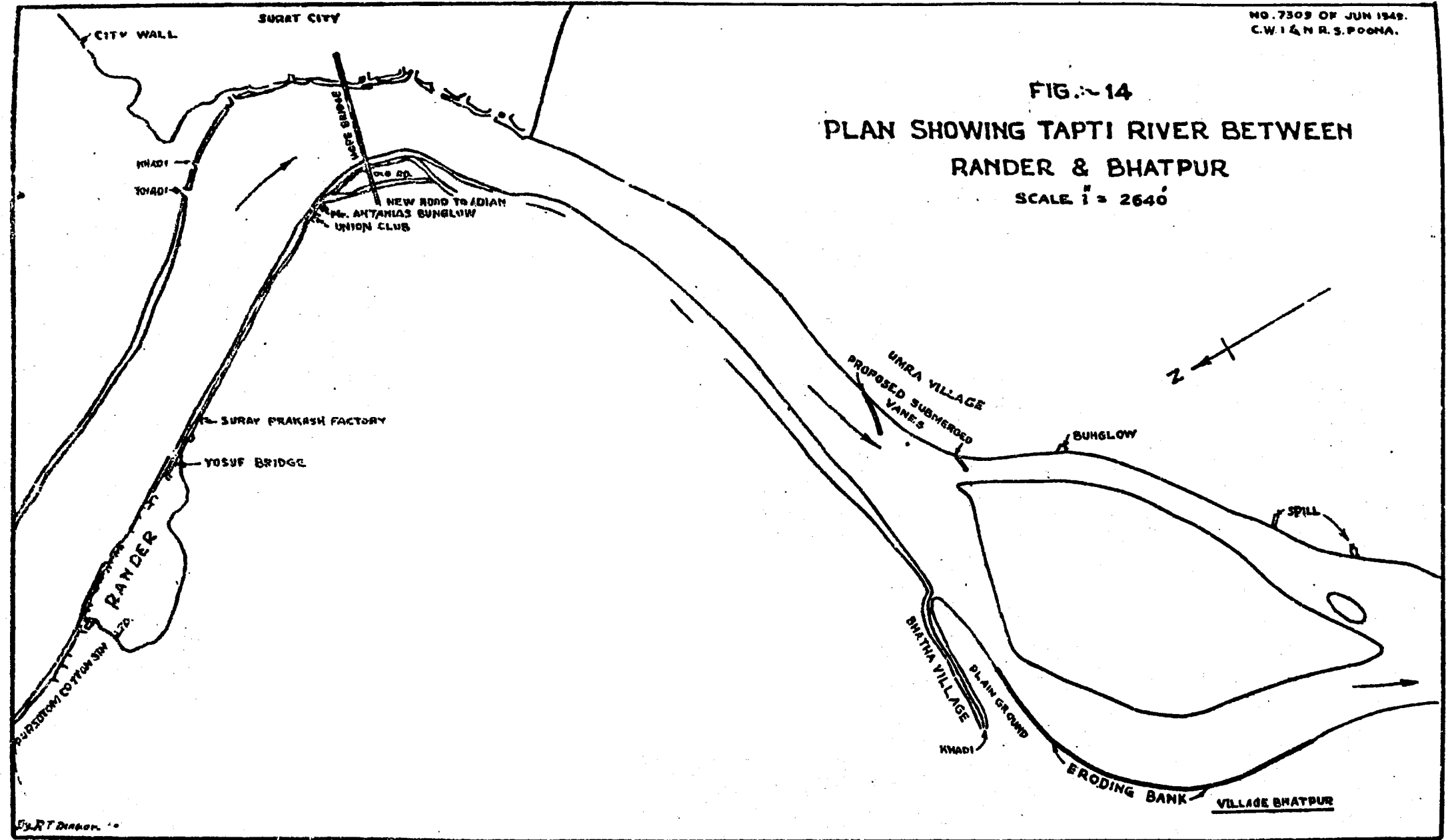


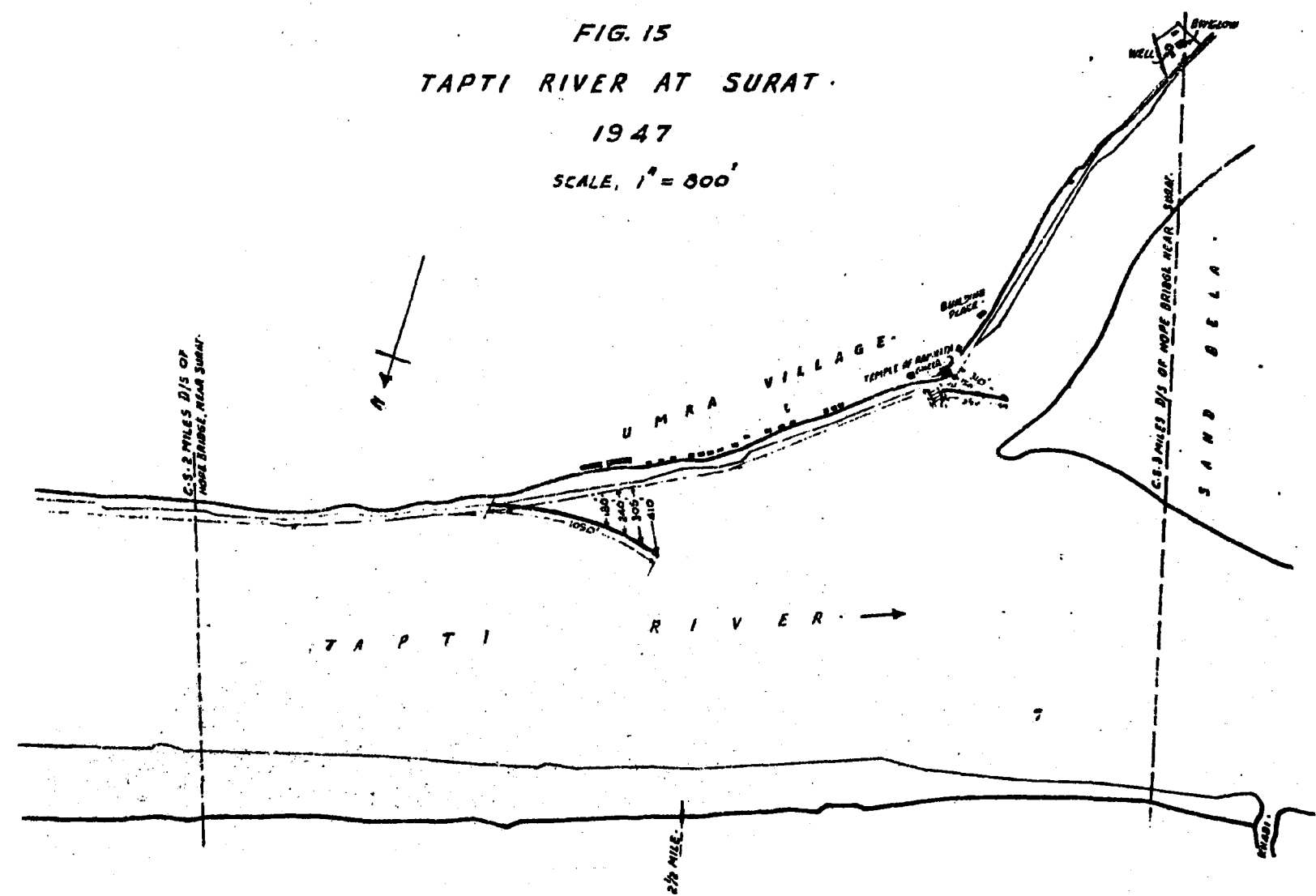
FIG. 16

FIG. 14  
PLAN SHOWING TAPTI RIVER BETWEEN  
RANDER & BHATPUR  
SCALE 1" = 2640'



By J.T. DARGA

FIG. 15  
TAPTI RIVER AT SURAT.  
1947  
SCALE, 1" = 800'



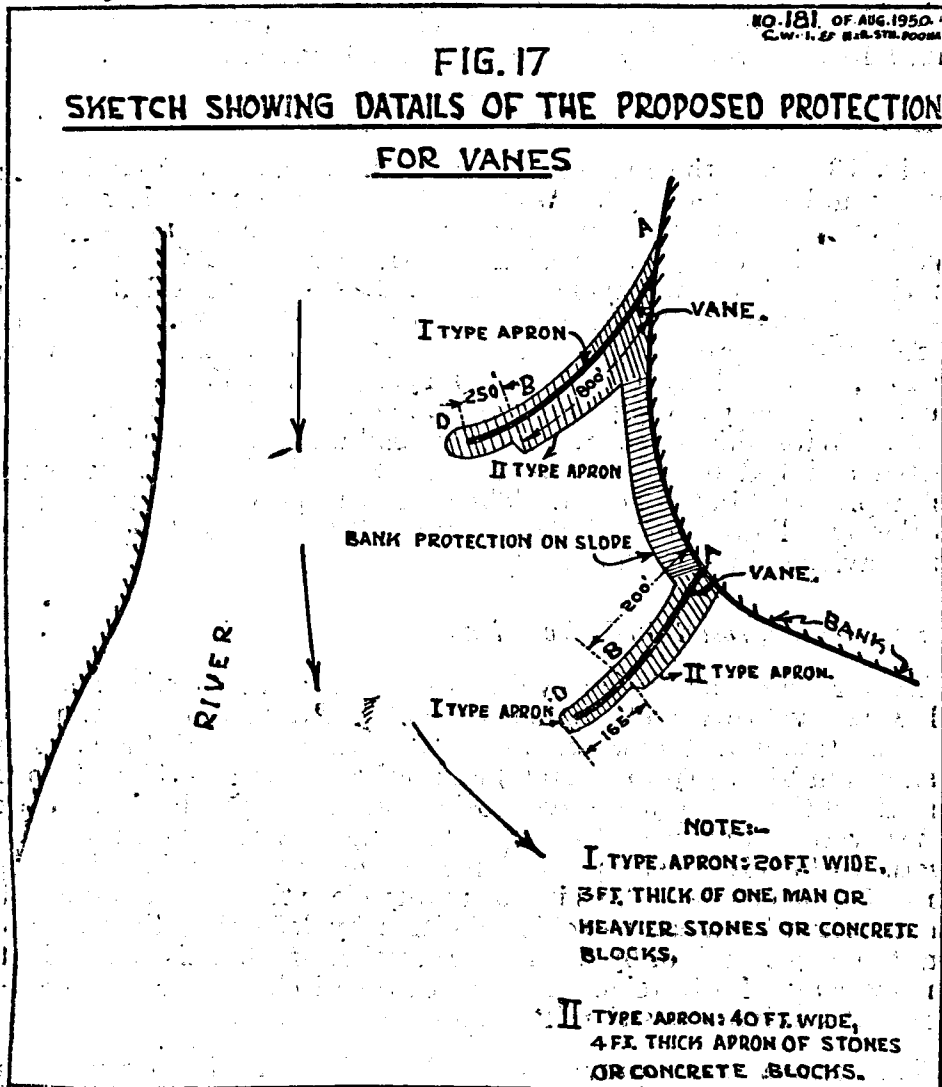
PROTECTIVE WORKS ON TAPTI

(3) Vanes may be constructed of interlocking steel sheet piling in single row along the alignment shown (Fig. 15). The piles for length AB, (Fig. 17) should be 50 feet long while for length BD shorter piles say 30 feet long will suffice.

Fig. 15.

(4) Apron protection along the upstream side at river bed should be provided as shown in Fig. 17 for the prevention of damage and subsequent failure of the vane due to scour.

Fig. 17.



## 5. MEASURES NECESSARY TO PROTECT THE RIGHT BANK OF THE RIVER PURNA NEAR THE VILLAGE DESAI FALIA SURAT DIST., BOMBAY STATE.

### 1. Introduction.

As permanent measures were found necessary for the adequate protection of the right bank of the river Purna, near the village Desai Falia, 1/150:1/25 vertically exaggerated model was constructed.

### 2. Model

Fig. 18.

Fig. 18 shows the reach incorporated in the model. Model was laid in 0.50 mm. Koregaon sand. As the slope of the river is very steep, viz. 1 ft. per 1,000 ft. the slope exaggeration given to the model was 2:0 only, based on the past experience of the Station. After running the first rising and falling cycle from 26,000 cusecs to 1,20,000 cusecs excessive bed movement was noticed in the model. Also, the model did not faithfully reproduce the conditions in the river. The material was, therefore, replaced by coarse sand varying from 1.75 mm. to 2 mm. in diameter. The slope exaggeration given to the model was same as before, viz. 2:0. With this change, excessive bed movement was immediately arrested and bed movement as desired was obtained throughout the reach reproduced in the model. The surface slope observed in the model also closely followed the bed slope given to it thus indicating the consistent behaviour of the model.

### 3. Experiments, results and discussion

Figs. 19 (a)  
and 20 (a).

Two rising and falling cycles were run in the model and lines of flow and velocity distribution were observed at cross sections 0, 1 and 2 upstream and downstream of the village Desai Falia. These are shown in Figs. 19 (a) and 20 (a) respectively. It is evident from the figure 19 (a) that the progressive erosion on the right bank is mainly due to severe and persistent eddy and diving flow and sudden change of curvature which is too sharp even for commonly occurring discharges such as 50,000 cusecs. Thus it was amply clear that a substantial relief along the right bank can be achieved either by deflecting the flow away from the right bank or by a partial reduction of the discharge in the river by way of diversion of some of the discharge through a cut-off channel at a suitable position if possible.

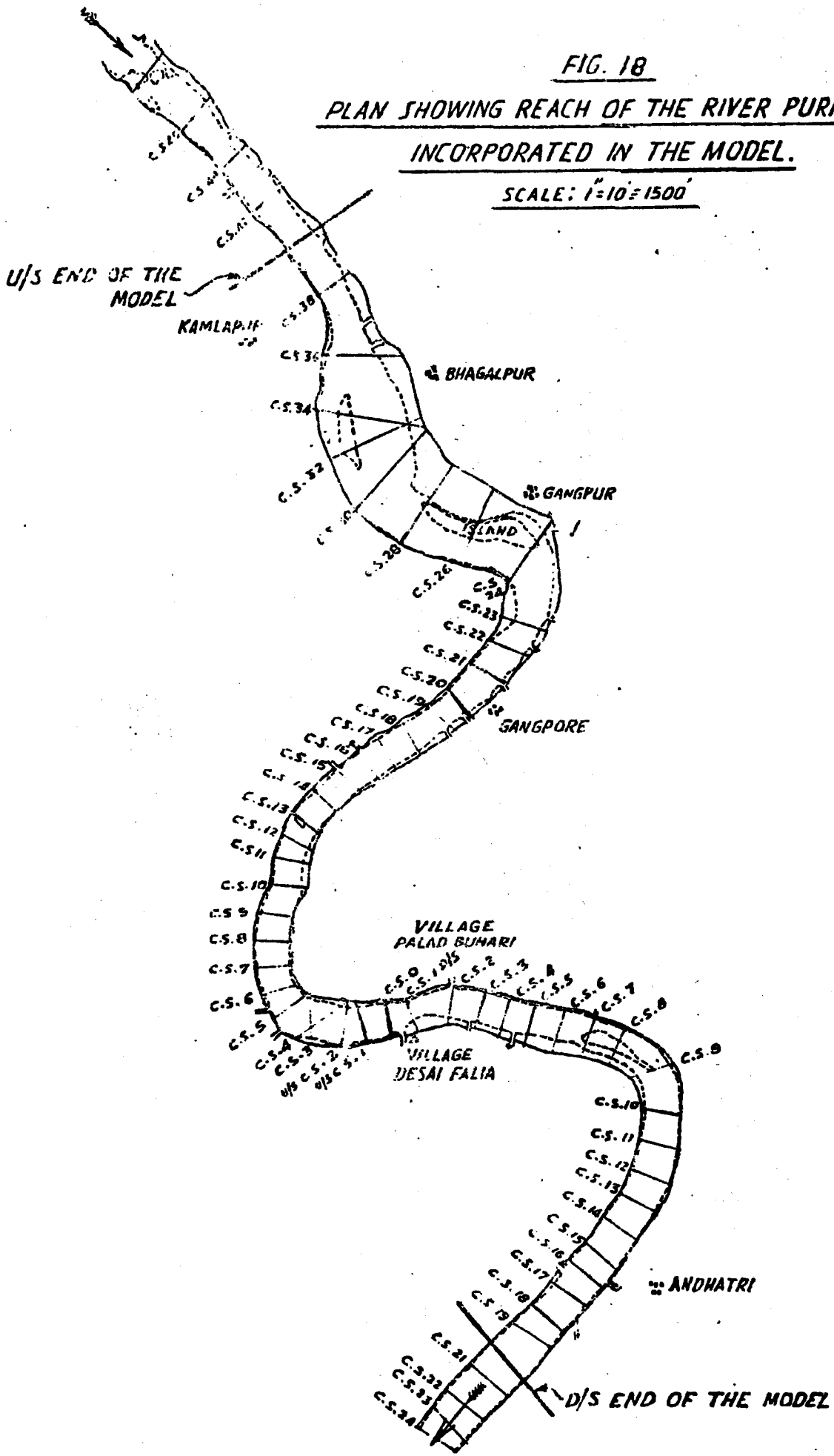
The permeable tree groyne would have been a cheap remedy had it not been for the deficiency of the sand and silt in suspension, which would have been trapped by it and thus increased its efficacy. Tests were, therefore, made with various lengths of impervious groynes. Groynes in length from 100 ft. to 250 ft., tested between C. S. 10 upstream to C. S. 4 upstream, did not show any promising position as it was observed that the flow downstream, instead of being deflected away quickly, returned to the right bank and squarely attacked the very reach which needed protection. Even with the intermediate position at cross sections 3 to 1 upstream, the groyne was ineffective. The position of the groyne was therefore shifted downstream between C. S. 0 to C. S. 1 downstream; the groyne faced upstream at an angle of 70° to the right bank. With this position the flow pattern near the right bank, just in the vicinity of the village, improved.

Figs. 19 (b)  
and 20 (b).

Figs. 19 (b) and 20 (b) show the lines of flow and velocity distribution at C. S. 0, 1 and 2 upstream and C. S. 1 downstream of the river. Comparison between Figs. 19 (a) and 19 (b) will show that the extent of

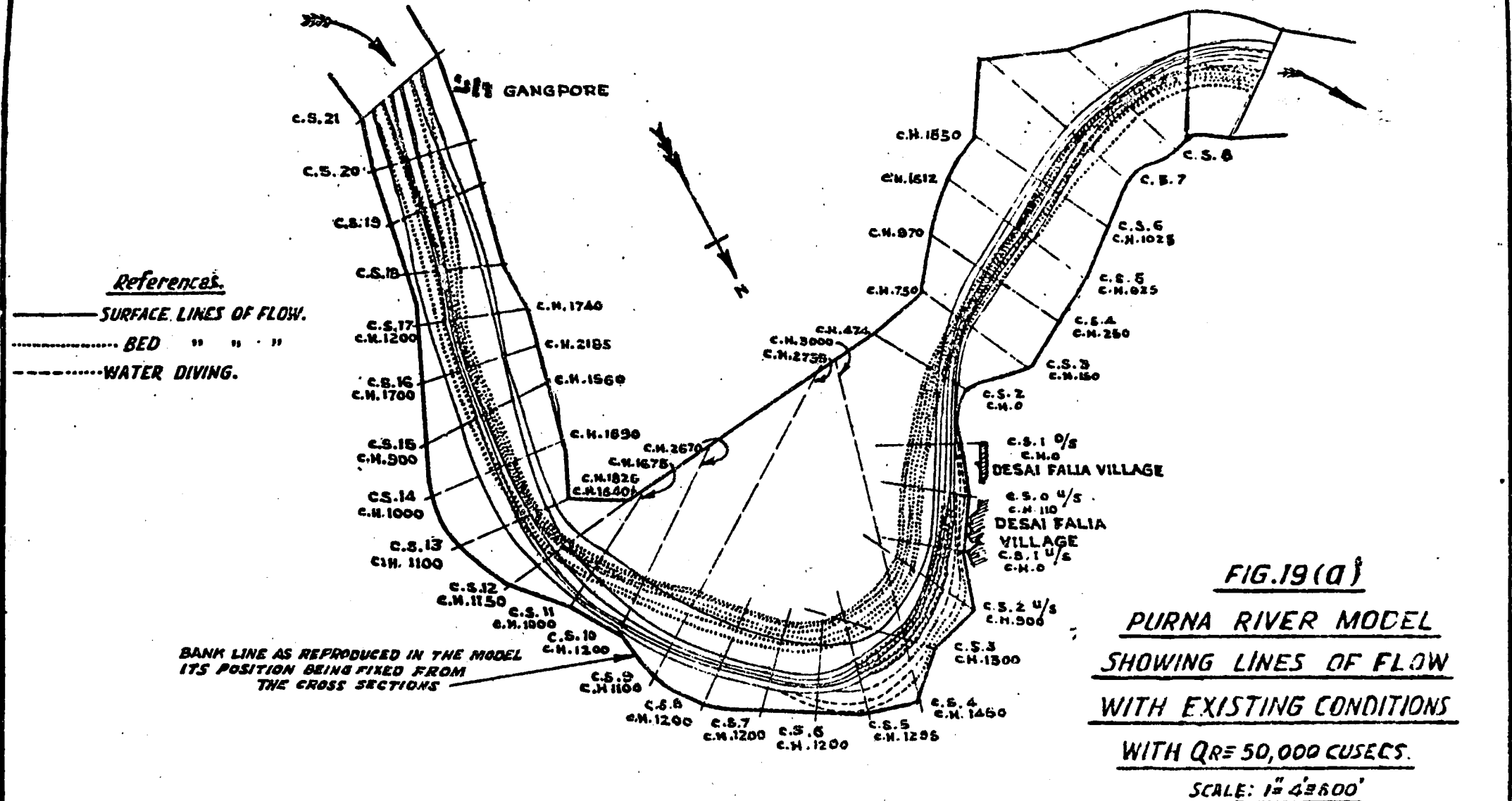
**FIG. 18**  
**PLAN SHOWING REACH OF THE RIVER PURNA**  
**INCORPORATED IN THE MODEL.**

**SCALE: 1" = 10' = 1500'**

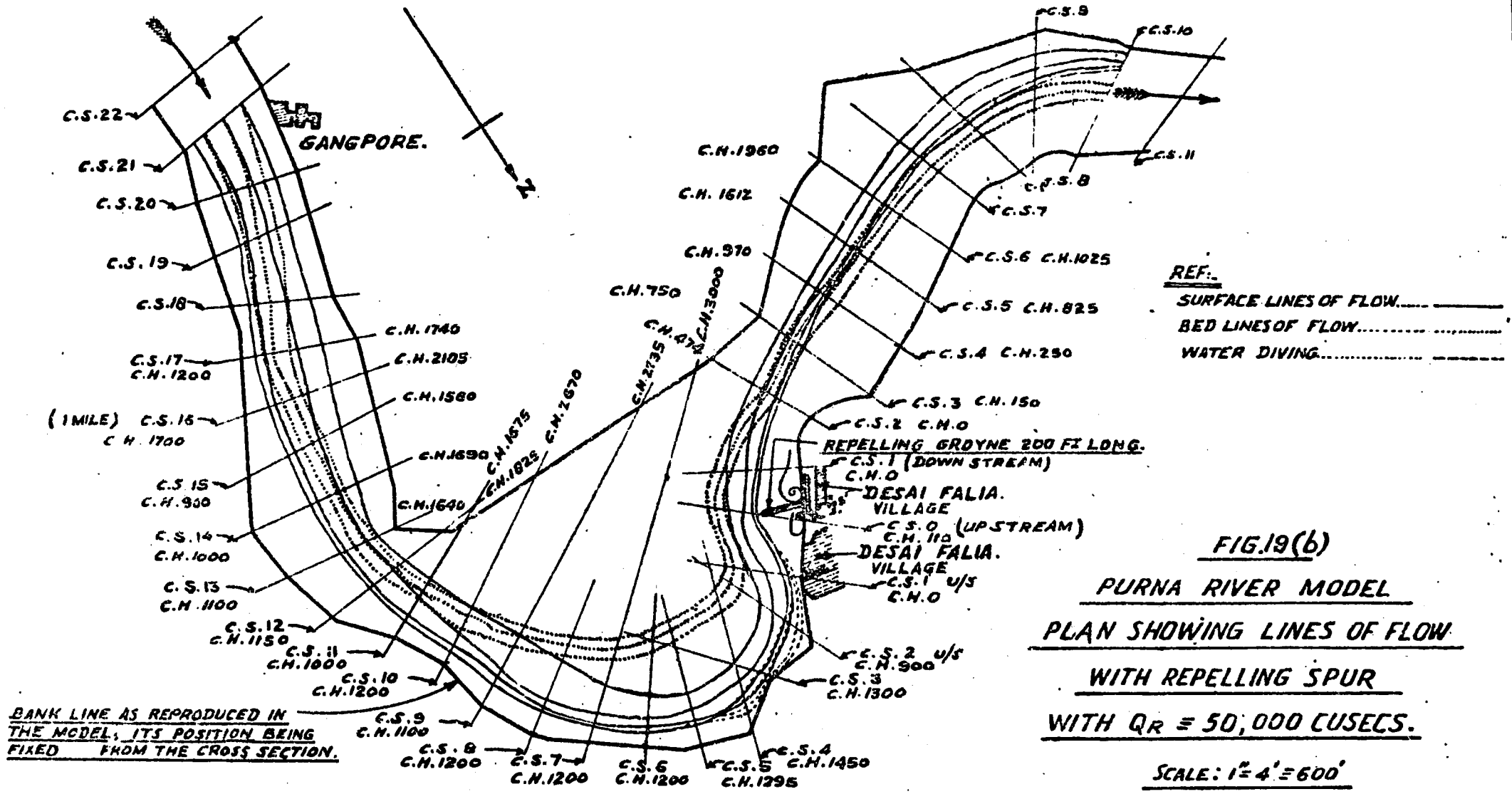


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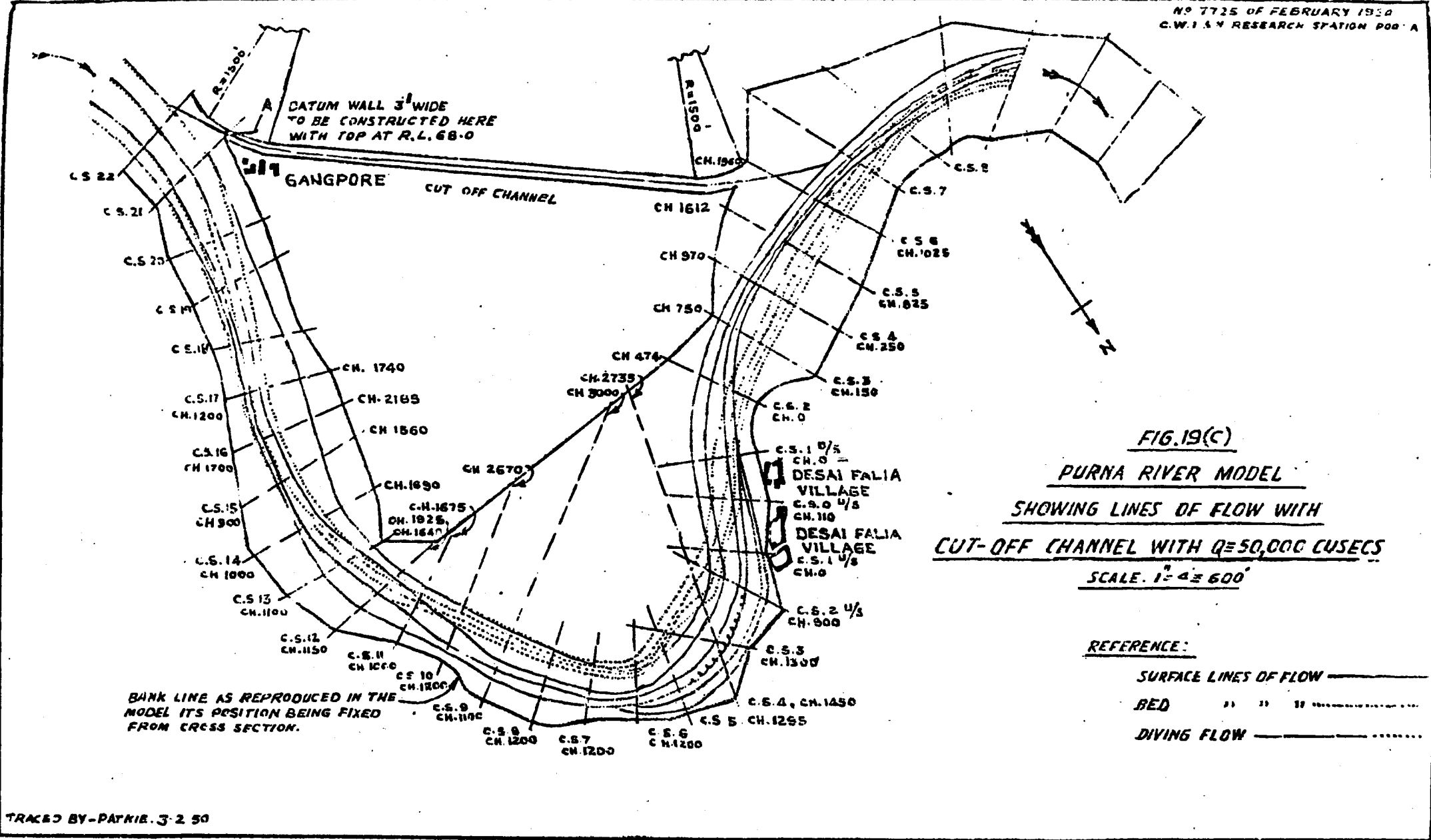
REF.:  
 SURFACE LINES OF FLOW.....  
 BED LINES OF FLOW.....  
 WATER DIVING.....

**FIG. 19(b)**  
**PURNA RIVER MODEL**  
**PLAN SHOWING LINES OF FLOW**  
**WITH REPELLING SPUR**  
**WITH  $Q_R \approx 50,000$  CU SECS.**

SCALE: 1" = 4' = 600'

BANK LINE AS REPRODUCED IN THE MODEL, ITS POSITION BEING FIXED FROM THE CROSS SECTION.

T. By: S. A. Manchundia



**FIG. 19(C)**  
**PURNA RIVER MODEL**  
 SHOWING LINES OF FLOW WITH  
**CUT-OFF CHANNEL WITH Q=50,000 CUSECS**  
 SCALE. 1" = 400'

**REFERENCE:**  
 SURFACE LINES OF FLOW —————  
 BED " " " " - - - - -  
 DIVING FLOW - - - - -

FIG.20(a)

PURNA RIVER MODEL.

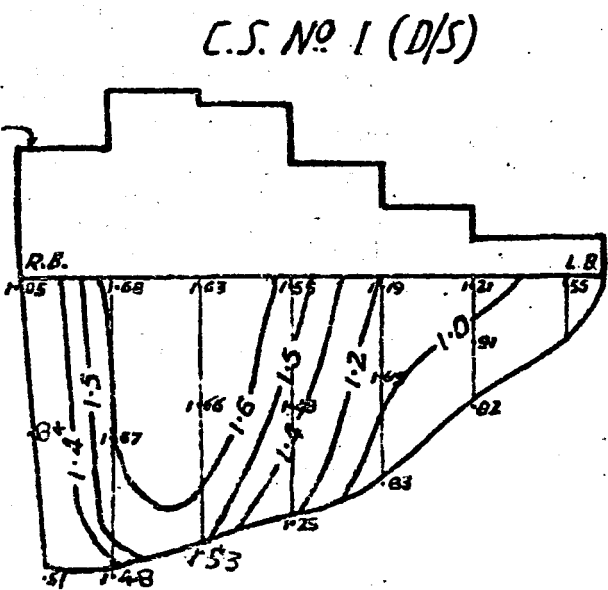
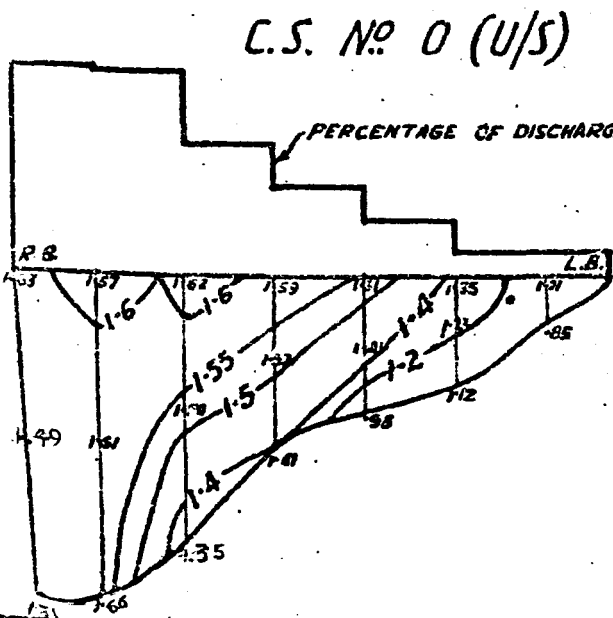
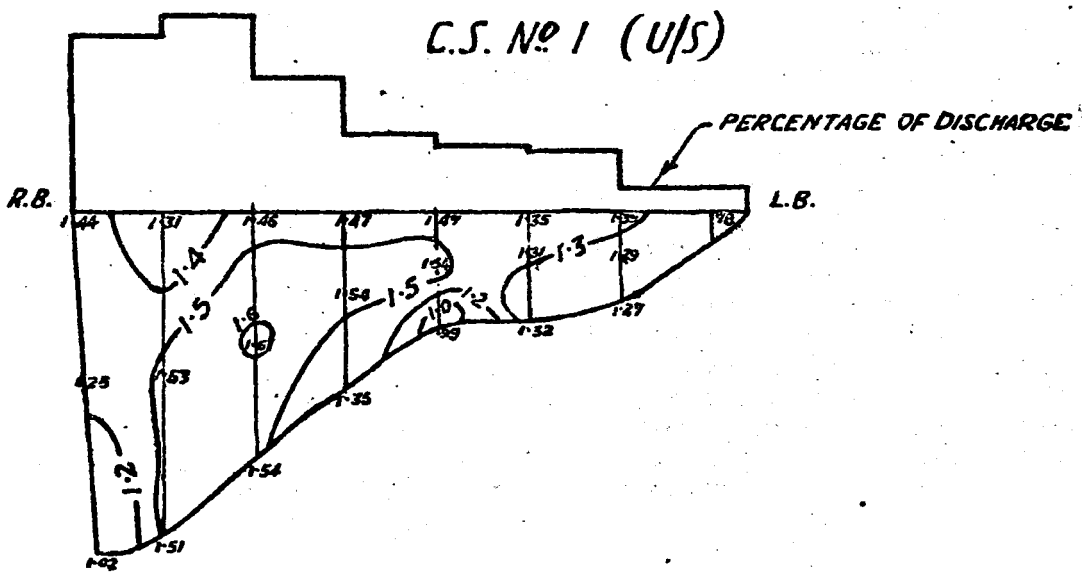
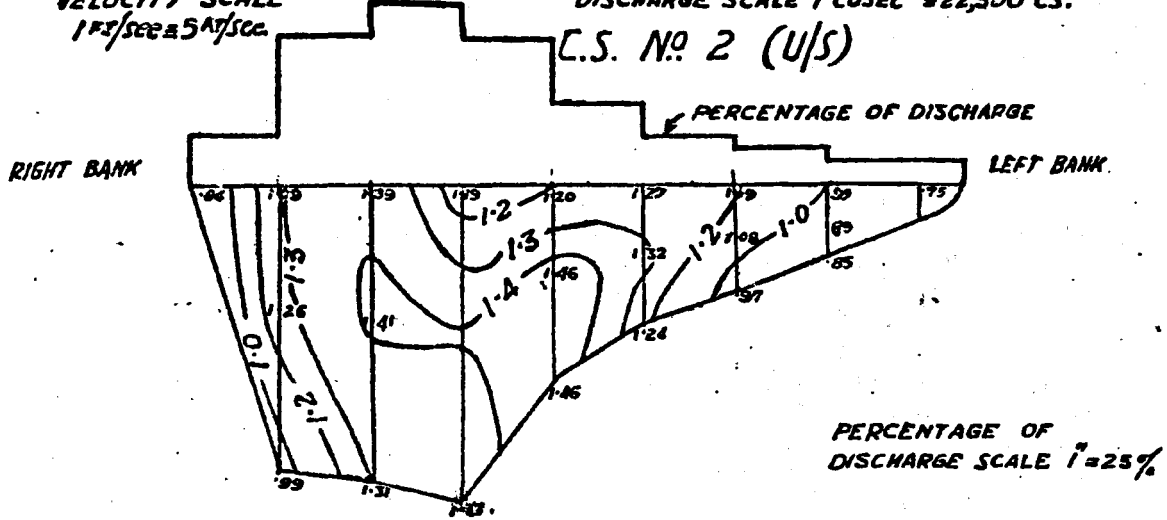
SHOWING ISOVELS & PERCENTAGE DISCHARGE DISTRIBUTION WITH EXISTING CONDITIONS.

FOR Q = 50,000 CUSECS AT CROSS SECTIONS 0, 1 & 2 U/S AND C.S. 1 D/S

HOR. 1" = 1' = 150'  
SCALES - VER. 1" = 1/2' = 12.5'

VELOCITY SCALE  
1 FT/SEC = 5 CM/SEC

DISCHARGE SCALE 1 CUSEC = 22,500 CS.



## PURNA RIVER NEAR DESAI FALIA

eddy and diving flow near the village is reduced with the repelling groyne. The surface flow which used to strike against the right bank near the village was also deflected sufficiently away from the bank. This will also be evident from the velocity distribution and percentage of discharge with and without the groyne [figs. 20 (a) and 20 (b)]. The comparison with cross sections in 20 (a) and 20 (b) also shows that deep channel has shifted from the extreme right bank to the central portion of the river. Thus the groyne was found to ameliorate the trouble at the village. Though some action was noticed along the left bank opposite, it was not considered dangerous or to cause any serious harm to the village on that bank.

The final length to the centre of the nose of the groyne which gave optimum results was 200 ft. The details of the groyne such as the length, nose, launching apron etc., are shown in Fig. 21. The nose of the spur and first 10 ft. length of the shank should be entirely of stone, the latter with a view to provide a good junction of the nose with the shank. The shank will have also to be protected with apron on the downstream as well as on the upstream side. The nose of the spur should slope at 1 in 3 (i.e. 1 vertical to 3 horizontal). The launching apron is provided according to the depth and extent of scour obtained in the model with due allowance for vertical exaggeration. It was however noted from the velocity distribution that the effect of the groyne would not reach sufficiently upstream (hardly up to C. S. 2) and this can be attributed to the very steep slope of the river. The velocities are also not reduced to the extent desired to completely neutralise the sharp curvature upstream. Owing to deficiency of sand in the river there would also be very little material depositing upstream and downstream of the groyne.

Fig. 21.

A cut-off channel was therefore next tested in the model. The alignment of the channel is shown in Fig. 19 (c). The length of the channel would be 2,700 ft. Two rising and falling cycles as run previously were run in the model. It was observed that with the channel the conditions substantially improved near the village, in that the eddy flow completely disappeared and water could smoothly negotiate the existing curve. This will be evident from the lines of flow shown in Fig. 19 (c). The velocity distribution was also markedly improved as can be seen from Fig. 20(c). The channel was found to carry about 9,000 cusecs when the river discharge was 50,000 cusecs. The optimum position of the head of the channel was found out by trial and error such that it did not draw sand load disproportionate to the discharge.

Fig. 19 (c).

Fig. 19 (c)

Fig. 20 (c).

Two sections of cut-off channel having bed widths of 150 ft. and 75 ft. and 1:1 side slopes were tested in the model. The latter section was found just enough to relieve the conditions at the village.

The head of the channel had also to be designed carefully such that it drew water as smoothly as possible without any disturbance so as to minimise the entry of heavy sand entering the channel. Accordingly, different radii of curvature were tested in the model and the one which gave the best result is shown in Fig. 19 (c). The downstream end of the channel was also smoothly joined with the parent river to maintain the proper hydraulic conditions at the exit. A masonry datum wall 3 ft. wide with top at R. L. 68.0 should be constructed at the head of the channel; the river bank about 150 ft. upstream of it should be pitched with one man stone pitching, if the bank is erodible, so as to preclude the possibility of the channel drawing an excessive discharge. The design of the channel is shown in Appendix I.

Fig. 19 (c).

Appendix I.

4. Conclusion

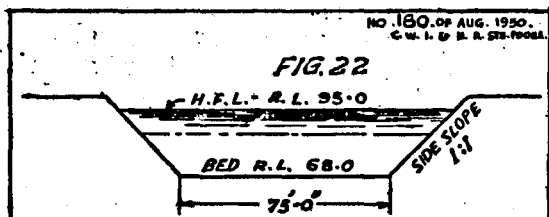
A cut-off channel is an ideal solution for eliminating the trouble at the village; whether it will be practical or not will much depend on the cost of land acquisition and the characteristics of the bed along its alignment.

As the consideration of cost will prevail over those of an all time efficiency, the groyne 200 ft. long may be constructed immediately.

APPENDIX I

Design of a cut-off channel

H. F. L. at C. S. 21 upstream	= R. L. 95.0.
H. F. L. at C. S. 6 downstream	= R. L. 87.27.
Therefore Difference in levels	= 7.73 feet.
Length of the cut-off channel	= 2700 feet.
Therefore Slope of the channel per 1000'	= 2.85 feet.



W. L. at C. S. 21 upstream for  $Q=50,000$  cusecs will be R. L. 80.0.

Discharge passing through the cut-off channel at this stage can be worked out according to the Manning's formula

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

$$\begin{aligned} \text{Area of cross section} &= \frac{99 + 75}{2} \times 12 \\ &= 87 \times 12 = 1044 \text{ sq. ft.} \end{aligned}$$

$$R = \frac{A}{P} = \frac{1044}{103} = 10.1 \text{ ft.}$$

Assuming  $N = .035$

$$\begin{aligned} V &= \frac{1.486}{.035} \times (10.1)^{\frac{2}{3}} \times \left(\frac{2.85}{1000}\right)^{\frac{1}{2}} \\ &= \frac{1.486}{.035} \times 4.68 \times \frac{1.69}{31.6} \\ &= \frac{118}{11.1} = 10.5 \text{ ft./sec.} \end{aligned}$$

$$\therefore Q = 10.5 \times 1044 = 10962 \text{ cusecs.}$$

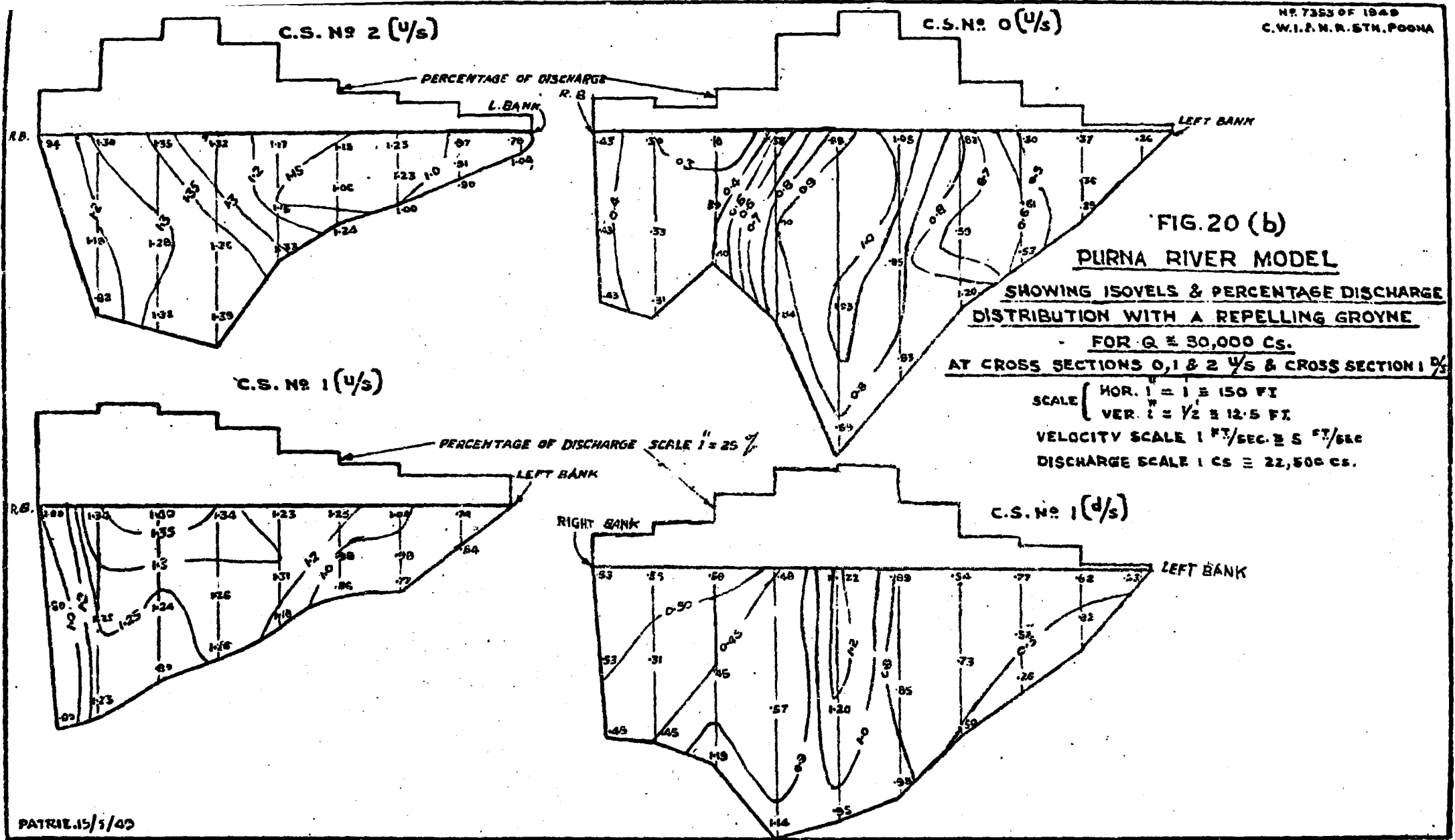


FIG. 20 (b)  
PURNA RIVER MODEL

SHOWING ISOVELS & PERCENTAGE DISCHARGE DISTRIBUTION WITH A REPELLING GROUPE  
FOR  $Q \approx 30,000$  CS.  
AT CROSS SECTIONS 0, 1 & 2 U/S & CROSS SECTION 1 D/S

SCALE ( HOR. 1" = 1"  $\approx$  150 FT  
VER. 1" = 1/2"  $\approx$  12.5 FT  
VELOCITY SCALE 1 FT/SEC.  $\approx$  5 FT/SEC  
DISCHARGE SCALE 1 CS  $\approx$  22,500 CS.

**PURNA RIVER MODEL**  
**SHOWING ISOVELS & DISCHARGE DISTRIBUTION**  
**WITH A CUT-OFF CHANNEL FOR Q=50000 C.S.**

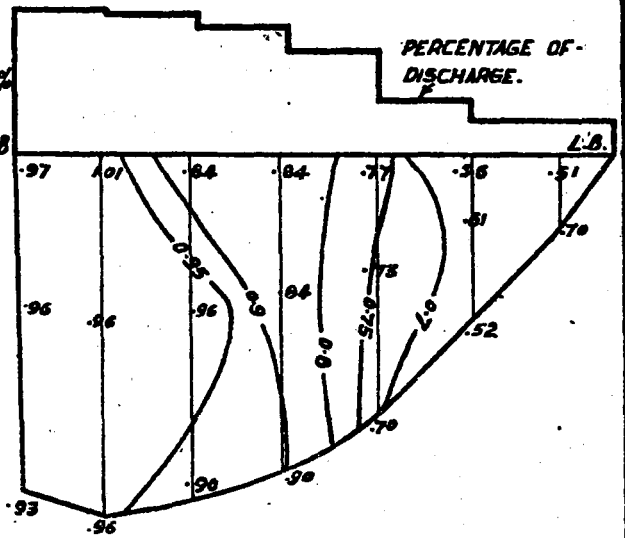
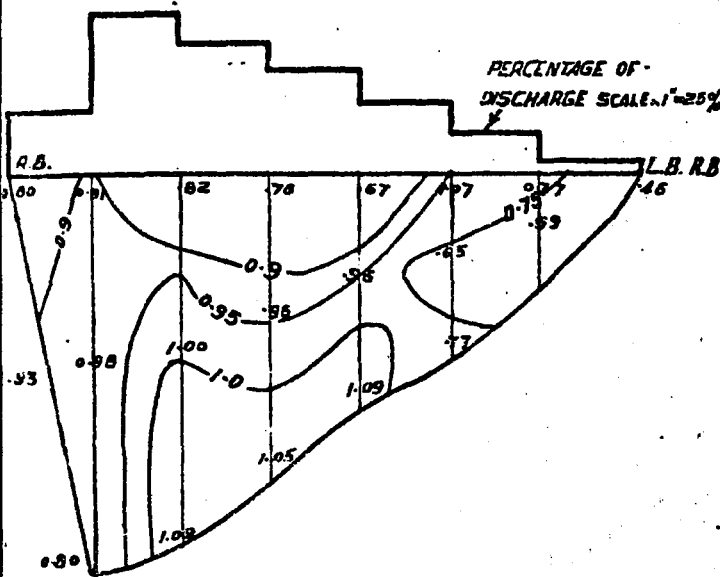
**FIG: 20 (C)**

AT CROSS SECTIONS, 0, 1, 2, 4s & C. SECTION 1 0/s .

SCALE: HOR. 1" = 1' = 150 FT.  
VER. 1" = 1/2' = 12.5 FT.  
VELOCITY SCALE: 1 FT./SEC. = 5 FT./SEC.  
DISCHARGE SCALE: 1 CUSECS = 22500 CUSECS.  
DISCHARGE " 1" = 25%

**C.S. NO. 2 (4/s)**

**C.S. NO. 0 (4/s)**



**C.S. NO. 1 (4/s)**

**C.S. NO. 1 (0/s)**

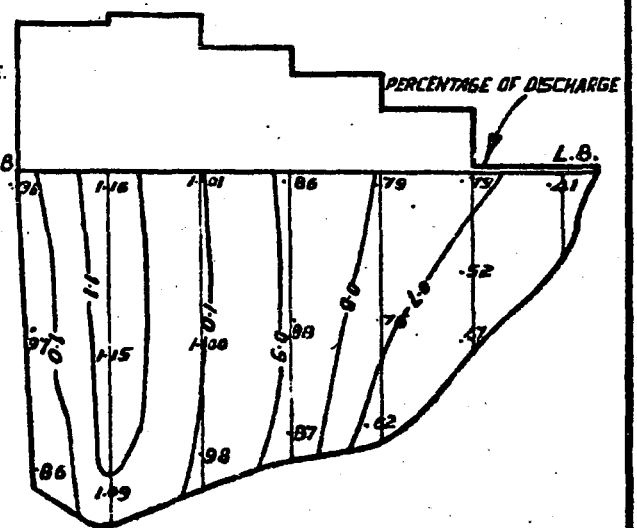
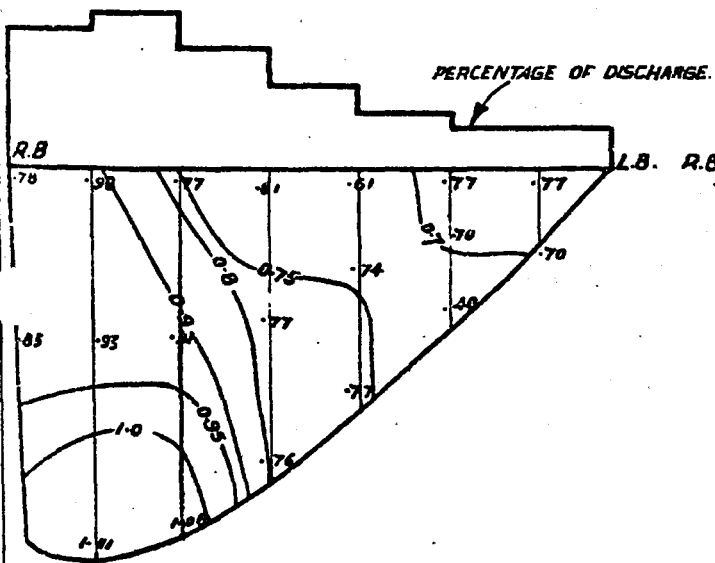
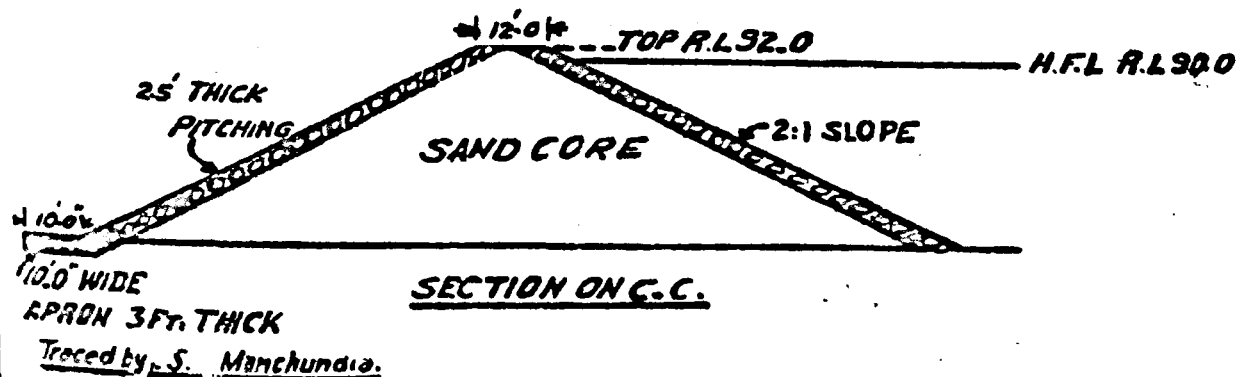
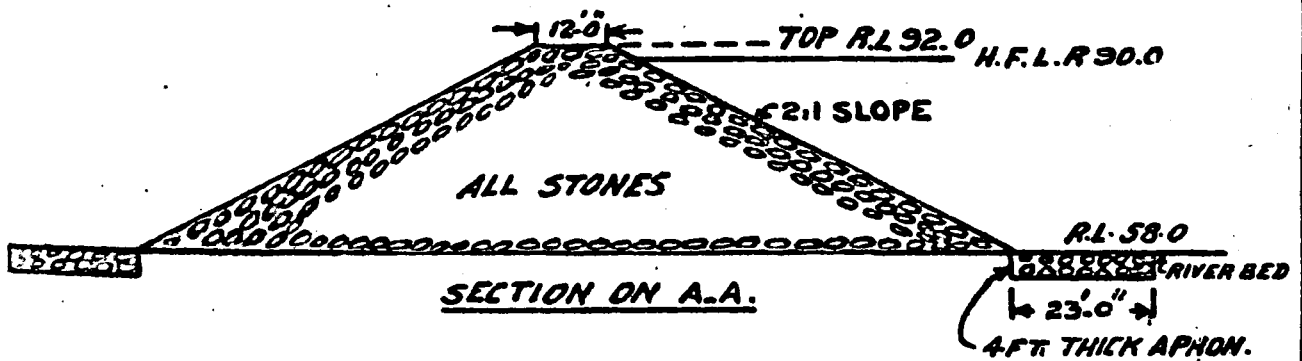
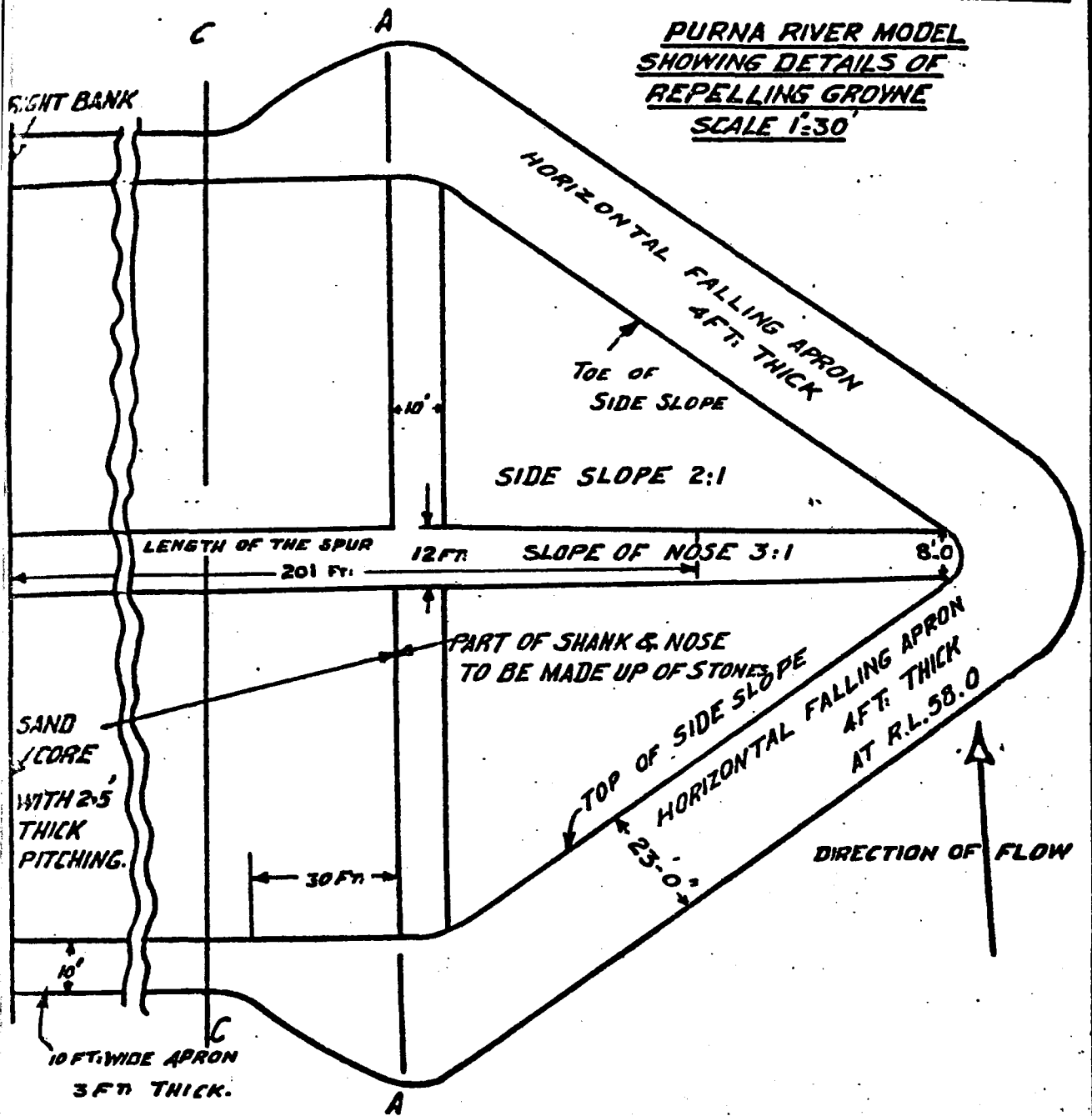


FIG. 21

DRG. NO 7358 OF JULY 1949  
C.W.T & N.R. STN. POONA. I

**PURNA RIVER MODEL  
SHOWING DETAILS OF  
REPELLING GROUYNE  
SCALE 1:30**





## 6. TRAINING THE GANGA RIVER NEAR THE GARHMUKTESWAR RAILWAY BRIDGE, (E. I. RLY.), U. P.

### *Introduction*

In continuation of the preliminary note given in the Annual Report (Technical), 1948, the present note briefly explains the results of model tests.

On 13th September 1948, Member Engineering and Director, Civil Engineering, Railway Board, inspected the flood affected area at Garhmukteswar bridge by air. The flood had then subsided somewhat, and the deep channel was clearly visible.

It was considered that the river had come dangerously close to the track (Fig. 23) and immediate remedial measures were necessary to maintain lines of communication. It was also proposed by the Railway Board that the Left Guide Bund should be extended upstream by about 2,000 ft. and a curved mole head provided, the extension covering the deep water channel.

Fig. 23

With the extension of the Left Guide Bund the threat to the Railway Line would have been considerably lessened, but as the river had advanced quite close to the line, the possibility of a further advance and cutting of the line, during the following flood season could not be altogether discounted. It was also suggested for consideration if it might be necessary to divert the line to a safe distance from its present location. Later it would also be necessary to extend the Right Guide Bund in conformity with the Left Guide Bund and provide a mole head.

Fig. 24

The Director, GWINR Station, who happened to be in Delhi at that time was also consulted in this matter. Subject to the final recommendations, based on the results of model experiments, he agreed with the suggestions.

When on inspection tour to Calcutta, the Director was also contacted by the Chief Engineer, Shri Gupta, who discussed the Garhmukteswar bridges case along with other cases and suggested a series of four T headed spurs at miles 39/5, 39/11, 39/16 and 39/21 to immediately arrest the tendency of the river to erode the left bank above the bridge, (Fig. 24). The Director did not agree with the proposal because these spurs when in action might neutralise the beneficial effect of the extended Left Guide Bund to a certain extent by attracting the flow towards these heads and might perpetuate the river course along the railway track.

In November 1949, the Director inspected the site of the damage with Mr. B. N. Chopra, Dy. Chief Engineer, E. I. Rly. from the bridge to the village Lathera on left bank about 3 miles upstream of the bridge. The proposal of opening channel 'A' below Lathera, (Fig. 23), by putting a bund across the channel B and providing a leading cut in 'A' was discussed. This proposal though it might have worked with very low floods was considered to be unsatisfactory with higher floods. The channel 'A' being on the inside of a curve, it was expected to draw sand, thus obliterating any channel which might have been developed by low floods.

Fig. 23

As regards extending the Left Guide Bund, from practical considerations and constructional difficulties it was not possible to extend it up to Ch. 34 and then give it a curved head. It would not have been possible to complete the whole work and if the extension work were left to the next year it would have been almost impossible to carry out the work due to deep channel which would form after the 1949 floods and hence, the Director agreed that the bund may be extended as far up to Ch. 30 or beyond if possible and give it a curved head of 120°.

The Director also agreed to send a design of protection to the extended guide bund based on expected scour factors.

Fig. 25

Considering the gravity of the situation and the very short time available for construction, pending model verification it was suggested to divert the track line  $\frac{1}{4}$  mile away from and downstream of the existing one (Fig. 25). It was also decided to take up model experiments under top priority.

#### *The Model*

The model extended 8.5 miles upstream of the bridge to 2 miles downstream of it. The scale ratios adopted were:—

Length scale ratio	=	1/200.
Depth " "	=	1/60.
V. E. " "	=	3.33.
Bed slope exaggeration	=	4.56.

#### *Prototype data*

The data for operating the model consisted of:—

- (i) A Survey Plan of the river after 1948 floods and cross-sections
- (ii) Graphs showing water level at the bridge against dates for the years 1943 to 1948
- (iii) Samples of bed sand
- (iv) Topo Sheet showing the survey of the year 1928
- (v) Aerial photographs showing conditions after 1946 and 1947 floods.

(i) The cross-sections had H. F. L. marked on them. As there are no gauges above the bridge, in the reach under consideration, the water levels were noted down on local enquiry. The flood slope according to this data worked out to 0.21 ft. per 1000 against 0.285 ft. mentioned by Sir Francis Spring.

#### *Maximum flood discharge*

(ii) The maximum flood level at the bridge is stated to be 658 but this never reached during the period 1943-48. The most essential data for the model operation is the discharge gauge graphs; but unfortunately no observations of flood discharges were available. The maximum discharge was stated to be of the order of 1.0 lakh cusecs.

This is a braided river with not very well defined meanders. The representative meander length appeared to be of the order of 3 miles which would give a maximum discharge of 2.8 lakh cusecs according to the formula

$$M_1 = 30 \sqrt{Q_{\max}}$$

and a dominant discharge of 1.58 lakh cusecs according to

$$M_1 = 40 \sqrt{Q_{\text{dom}}}$$

According to the Mississippi formula

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

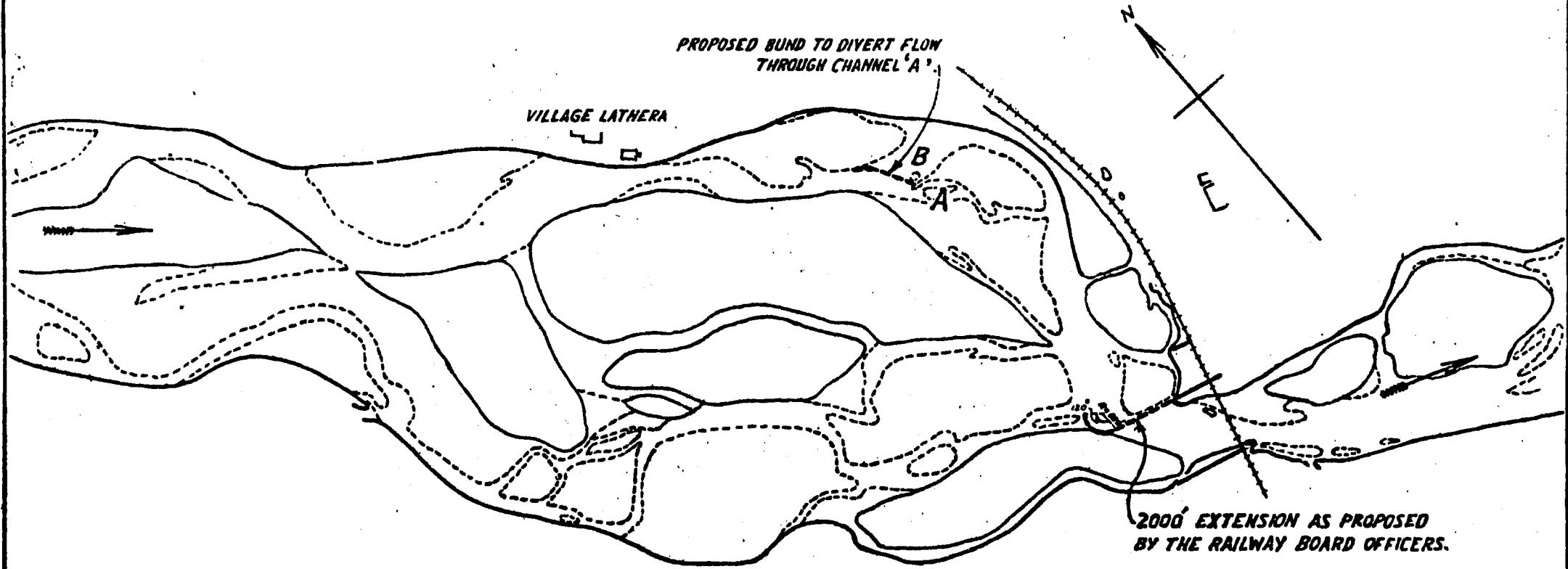
$\bar{V}$  worked out to 6.12 ft./sec. and making a 10 per cent allowance for scour during floods the maximum discharge through the bridge worked out to

$$\begin{aligned} Q &= AV \\ &= 1.10 \times 46000 \times 6.12 \\ &= 3.10 \text{ lakhs cusecs} \end{aligned}$$

It will be noticed that the discharge of the river could not be accurately assessed, but it was apparent that the maximum flood

FIG. 23  
SHOWING GANGA RIVER AT GARHMUKTESHWAR &  
THE EXTENSION OF THE 2,000 FT LEFT GUIDE  
BUND AS FIRST PROPOSED BY THE  
RAILWAY BOARD.

SCALE : 1" = 4,000 FT

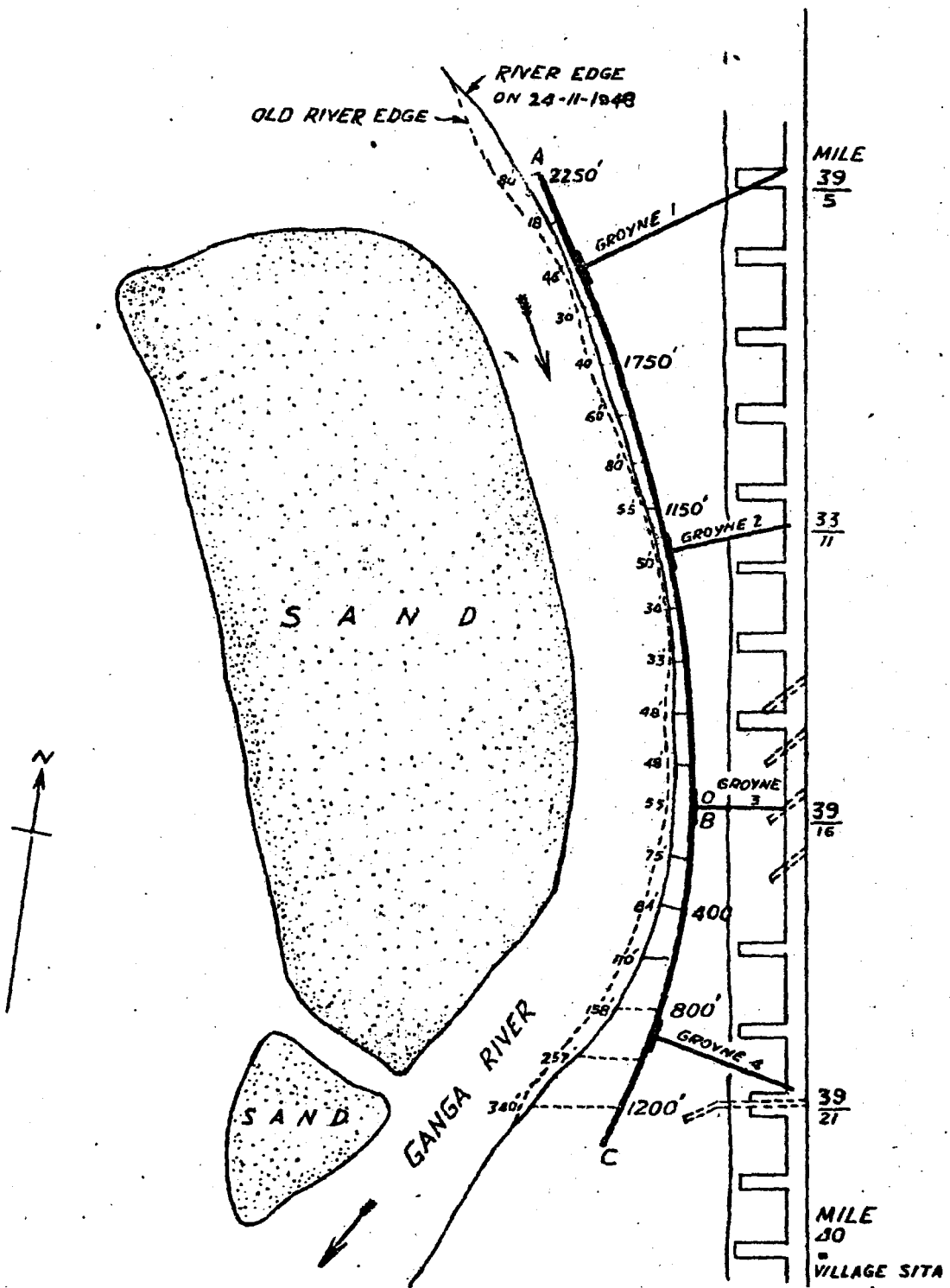


2000' EXTENSION AS PROPOSED  
BY THE RAILWAY BOARD OFFICERS.

TRACED  
By - D. S. BETHWANE.

CHECKED BY - *M. S. J.*  
1/1-2-30.

FIG. 24  
PLAN SHOWING THE RIVER GANGA  
AND PROPOSED TRAINING WORKS  
AT MILE 39-40  
(E. I. R. DELHI BRANCH)  
SCALE: 1" = 600'



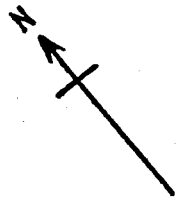
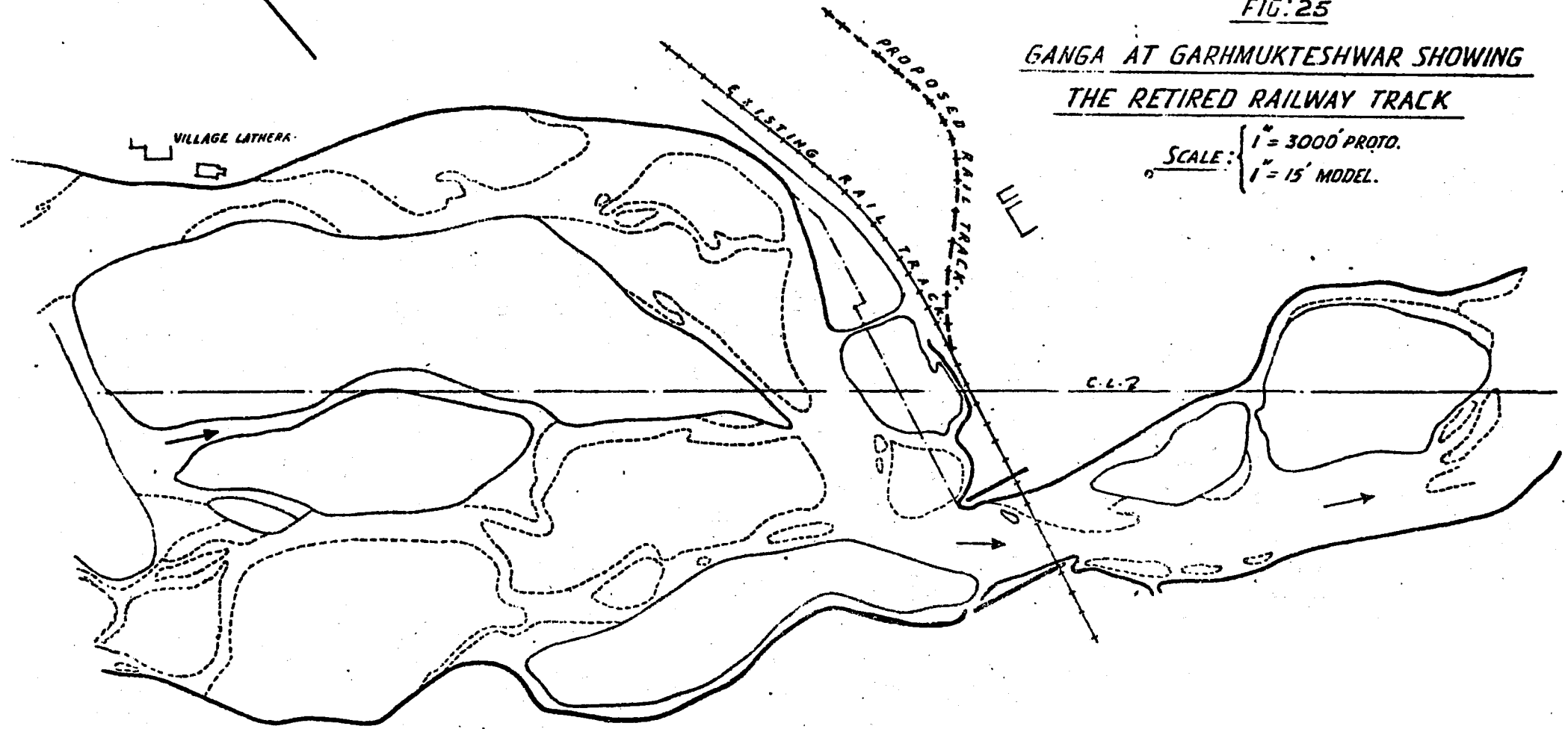


FIG: 25

GANGA AT GARHMUKTESHWAR SHOWING  
THE RETIRED RAILWAY TRACK

SCALE:  $\left\{ \begin{array}{l} 1'' = 3000' \text{ PROTO.} \\ 1'' = 15' \text{ MODEL.} \end{array} \right.$



TRACED BY: R. S. NAIK. 1-2-50. Checked by: *Mej* 17-2-50

## TRAINING GANGA RIVER AT GARHMUKTESWAR

discharge with water level 658 at the bridge was certainly more than a lakh of cusecs, and later, model results also gave a maximum discharge equivalent to 2.1 lakh cusecs.

### *Design of Apron round the extended Left Guide Bund*

Maximum depth of scour:—In his paper "River training and Control", Spring suggested that the maximum depth of scour of the Ganga at Garhmukteswar might be taken as 40 ft. below low water. With a flood rise of 15 ft. this means a maximum depth of scour  $d_s = 40 + 15 = 55$  ft. below H. F. L. The scour round the head of the Guide Bank will, of course, be deeper as shown below:—

According to the analysis of scour round Guide Banks, bridge piers, etc., the scour for large radius curves of guide banks may be taken as  $2.75 D_{Lacey}$  and  $1.75 D_{Lacey}$  in straight shanks.

$$D_{Lacey} = 0.47 (Q/f)^{\frac{1}{3}}$$

where

$Q$  = Discharge in cusecs

$f$  = Lacey's silt factor

$$= 1.76 \sqrt{m}$$

$m$  being the mean diameter of sand = 0.186 in the present case.

So	$f = 0.76$	
and	$D_{Lacey} = 33.6$ feet	if $Q = 2.8$ lakh cusecs.
	= 28 "	if $Q = 1.58$ " "
	= 35.2 "	if $Q = 3.09$ " "

Table 1 gives the computations for working out the size of the apron having a 2:1 slope of the scour. Table 1.

Though the actual maximum flood discharge has not so far been observed, it is considered that approximately, 2.8 lakh cusecs may be considered as a possible value and the apron designed on this discharge may, therefore, be adopted.

Figs. 26 and 27 show the design as initially suggested by this Station subject to final modification after model tests. Figs. 26 & 27.

### *Experiments*

*Experiment 1.*—With existing conditions.

*Experiment 2.*—With extended guide bank; the extended shank ending at Ch. 30 with a 600 ft. radius, 120° curved head beyond.

*Experiment 3.*—With extended left guide bund and four T spurs at miles 39/5, 39/11, 39/15, 39/21 as proposed by the Railway Officers.

*Experiment 4.*—With a cut 100' × 10' through the island, about 2 miles upstream of Lathera, to divert main flow away from the Lathera channel.

*Experiment 5.*—With a repelling spur above Lathera to see if the village can be protected.

*Experiment 6.*—With Lathera bank cut back and a hypothetical meander laid to see the effect on the flow at mile No. 39.

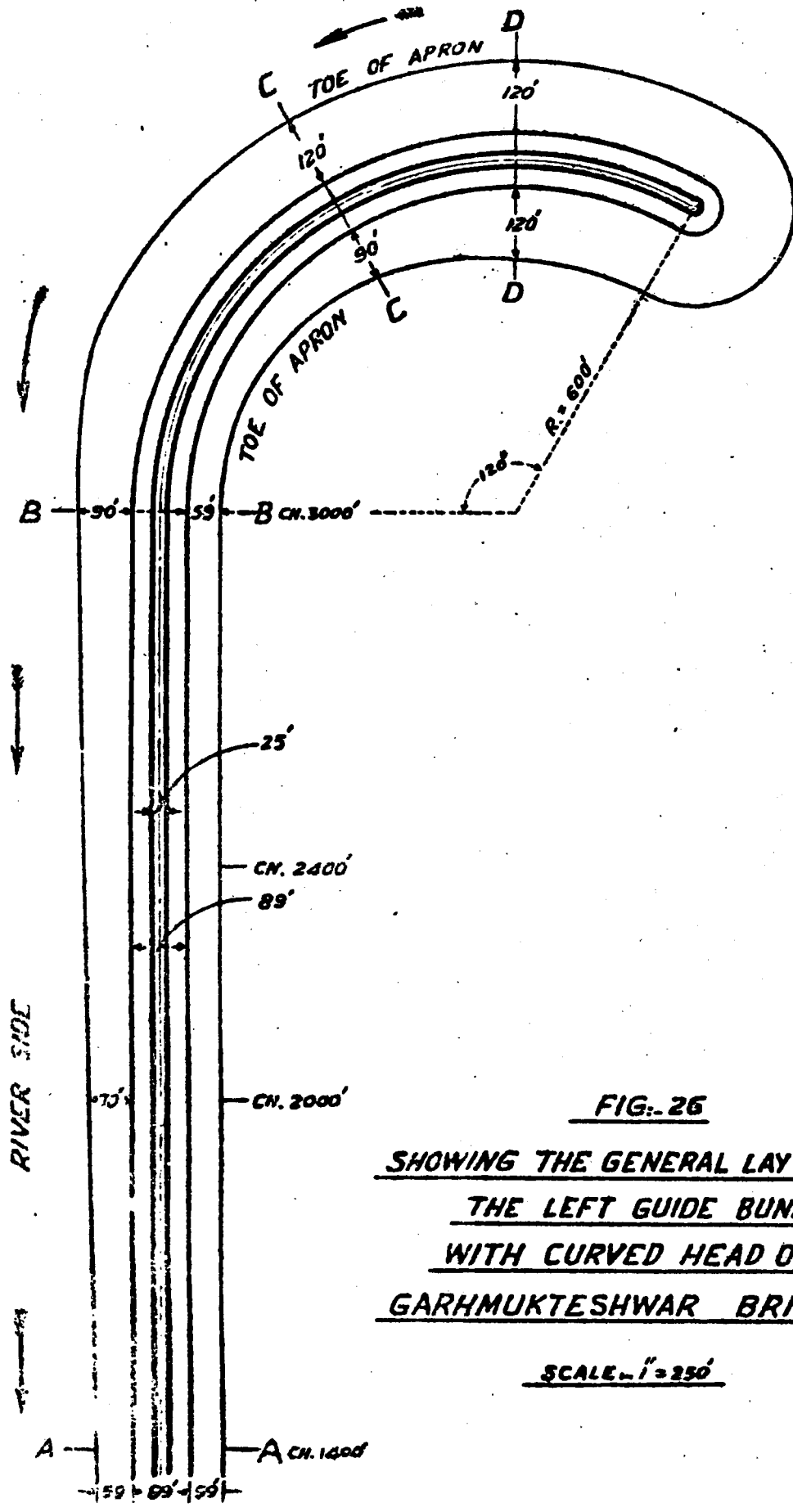
### *Procedure*

In each experiment the model was laid to the 1948 survey and a cycle of rising and falling floods repeated with bed sand change till model had stabilised. The discharges run were according to the 1947 flood level

TABLE 1

Showing computations of design of the apron.

1	2	3	4	5	6	7	Apron Design for curved head with depth of scour=2.75 D.			Apron design for straight shank with depth of scour=1.75D.		
							8	9	10	11	12	13
Formula used for calculating discharge.	Discharge in lakh cusecs.	D Lacey in ft.	1.75D in ft.	2.75D in ft.	Av. bed level R. L.	R. L. of scoured bed.	Quantity of stone required in cft. per foot length of apron.	Length and size of apron.	R. L. of scoured bed.	Quantity of stone required in cft. per foot length of apron.		
$M_1 = 30\sqrt{Q_{max}}$	2.8	33.6	59.0	92.5	646.0	565.5	795		599	480		
$M_1 = 40\sqrt{Q_{dom}}$	1.58	28.0	49.0	0	646.0	575	640		603	440		
$V = [(3.0966 R\sqrt{S})^{\frac{1}{2}} - 0.388]^2$	3.09	35.2	61.5	97.0	646.0	561	840		596.5	505		



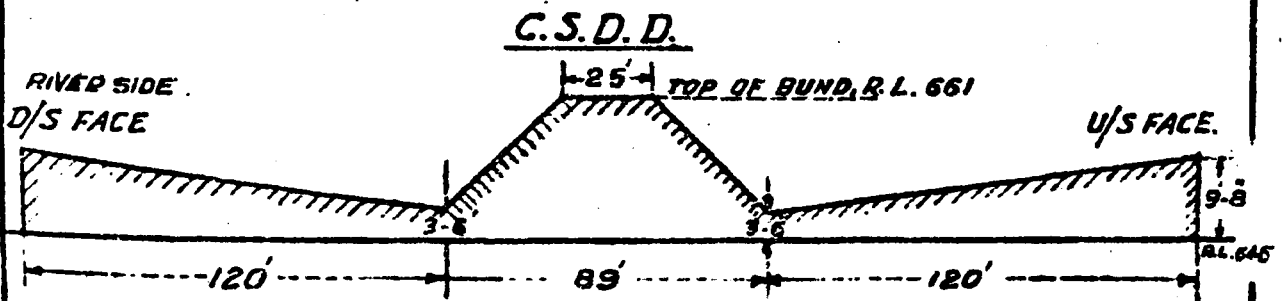
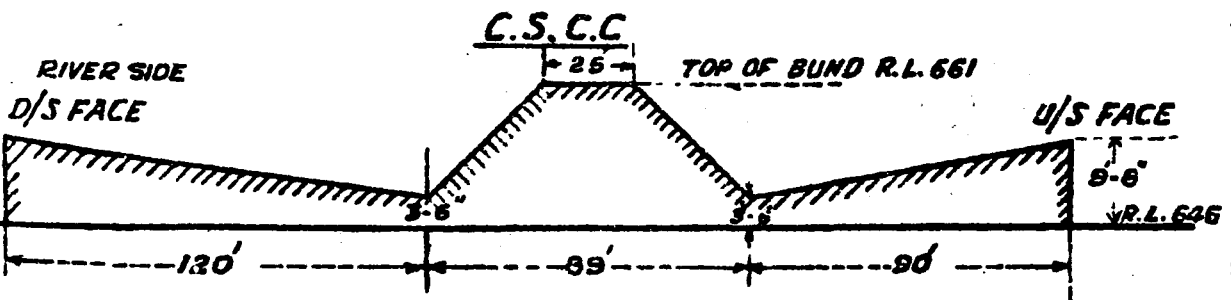
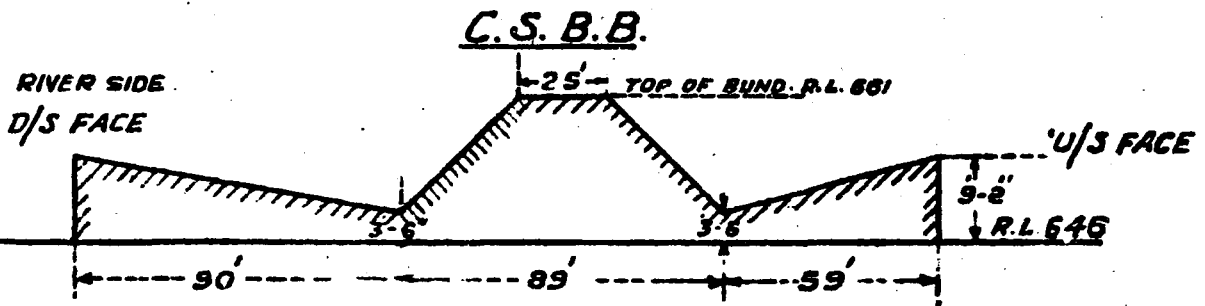
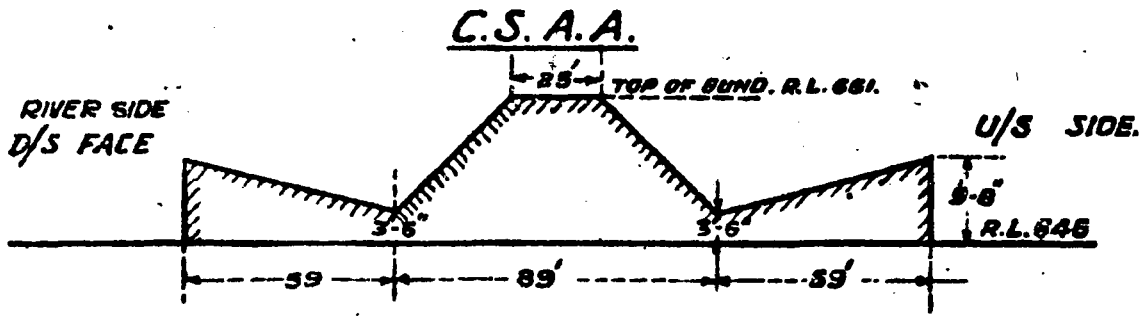
**FIG.-26**  
**SHOWING THE GENERAL LAY OUT OF**  
**THE LEFT GUIDE BUND**  
**WITH CURVED HEAD OF**  
**GARHMUKTESHWAR BRIDGE.**

**SCALE - 1" = 250'**



**FIG. 27**  
**SHOWING THE CROSS SECTIONS OF**  
**THE LEFT GUIDE BUND**  
**OF THE GARHMUKTESHWAR BRIDGE.**

**SCALE** -  $\frac{\text{HOR. } 1'' = 50'}{\text{VER. } 1'' = 20'}$



## TRAINING GANGA RIVER AT GARHMUKTESWAR .

chart (Fig. 28) with the hypothetical peak level of R. L. 658 added. This had to be done because during the years 1943-48 for which the data was supplied, the river had never attained the maximum R. L. 658 at the bridge, the observed maximum water level being R. L. 654.5 in 1947 i.e. 3.5 feet short of the maximum-ever flood level. Fig. 28.

### Results

As usual with models of wide rivers with interlacing channels, considerable difficulty was encountered in proving the model; because with high floods when water spilled over the shoals and island, due to exaggerated slope and absence of brush wood and vegetation, relatively greater percentage of discharge flowed over the islands thus reducing bed movement in the main channel.

With the help of the aerial photograph, after floods of 1947, these reaches of the islands which were seen to be above water level were raised so that water did not spill over them when the water level at the bridge was R. L. 654.5—the maximum level in the prototype during floods of the year when the aerial photo was taken.

Considerable modifications had to be done at the entry to simulate correct curvature of flow as indicated by the survey. The entry conditions as adopted are shown in Fig. 29. Fig. 29.

With correct entry conditions and corrected shoals, optimum flow conditions were obtained. Mr. Chopra, Deputy Chief Engineer, confirmed this during his visit to the Station.

In experiment 1 with existing conditions it was noticed that the erosion at the bank was severe from the bankful stage down to a flood of 50,000 cusecs.

With optimum flow conditions reproduced in the model, the relation between the discharge and the W. L. at bridge in the model was worked out. It was seen that with B. S. E. (Fig. 30) S. E. = 4.65, the maximum flood discharge observed was equivalent to 2.12 lakhs cusecs. Fig. 30.

Figs. 31 and 32 show lines of flow with a discharge of 70,000 cusecs in experiment 1, with existing conditions and in experiment 2 with extended left guide bund, the length of shank excluding curved head being = 3,000 feet. Fig. 31 & 32.

It will be seen that at the embayment near mile No. 39, in both the experiments, the top silt free water followed the left bank whereas the bed flow was deflected away from it. This was as expected, because, the intrinsic effect of curvature would never vanish unless, the curvature of flow itself vanished and the extended guide bund would not materially change the curvature at mile No. 39, unless reversal of curvature takes place further upstream.

Fig. 33 shows the final scoured bank line in the model in experiments 1 and 2 from which it will be seen that with the extended guide bund, bank erosion was less than under existing conditions. It was thus, however, clear that in spite of the lengthening of the guide bund, the bank erosion might continue and that the railway should be prepared for this. This also showed the necessity of diverting railway line to a convenient distance away from the existing one, the existing bank carrying the track being used as a first line of defence. Fig. 33.

Having completed observations of Experiment 2, a leading cut was made in the shoal opposite the embayment (Fig. 34) to see if it develops and thus reduces attack on the left bank. As anticipated, however, it did not develop because it was on the silting side of curved flow. This was just in conformity with the discussions which the Director had with the Deputy Chief Engineer as stated earlier. Fig. 34.

In experiment 3, the Railway proposal to construct four T headed spurs was tested. It was observed that of the four T spurs, the lower two came into action during the first floods. Erosion of the bank continued and embayments formed just upstream of these spurs. With higher floods, when water spilled over the bank, eddies formed at the upstream nose of the T of these spurs which tended to deepen the scour-hole. The maximum depth of scour at the nose of the third spur was = 52.2 ft. as against the normal depth of about 25 ft. without the spur.

Fig. 35.

Fig. 35 shows the lines of flow observed with a discharge of 70,000 cusecs in Experiment 3. It will be noticed that the curvature effect began to show at a point midway between spurs 2 and 3 which also added to the deepening tendency at the nose of spurs 2 and 3.

After running a couple of cycles this experiment was abandoned as results were not satisfactory.

In all the proposals so far tested it was evident that the bank erosion near mile 39 would continue unless the curvature itself reduced. With this object in view a test was carried out with a cut above the Lathera village to divert a substantial portion of the discharge into the far right bank channel. If successful the same cut would also be useful in reducing the bank erosion at the Lathera village.

At this juncture Shri Chopra, Dy. Chief Engineer, E. I. Railway paid a visit to the Station and to demonstrate to him the effect of a fully developed cut, before actually going into the details of an initially small cut to be later developed into a full one, a trial run was made with a cut equivalent to 600 ft. wide and 15' deep, shown in Fig. 36 (*vide* No. 1 line). The results of this run were satisfactory in that, there was no bank erosion near mile No. 39.

Fig 36.

Having ascertained the effect of a developed cut, a number of alignments were tested with an initially small cut to see if it develops. On detailed analysis of the data, however, it was seen that there was no chance of a small cut developing as sufficient fall was not available.

Basing calculations on 70% of the flood flowing through the Lathera channel as observed in the model, with a discharge of 1.1 lakhs cusecs, the value of Manning's N worked out to 0.026 and with this value the cut required a slope of 1.71 ft. per mile to pass a discharge of 5,000 cusecs initially, as against 1.5 ft. per mile which was the slope of the main river as noted by Sir F. Spring.

An initially large cut may have worked but was prohibitive in cost as even a 100 ft. wide cut was estimated to cost Rs. 15 lakhs.

At the time of visit Mr. Chopra handed over the plans of the extended guide bund from which it was seen that the length of the shank of the extended guide bund as actually constructed was 2,700 ft. as against 3,000 ft. tested in the model.

Experiment 2 was, therefore, repeated with the guide bund as extended on site. Fig. 37 shows the scour round the nose of the extended guide bund. As model results were now available, the quantity of stone in apron was reduced—see table 2.

Fig. 37.

Table 2.

TABLE 2

	Apron along the Shank.		Apron along the nose..	
	River side.	u/s side.	River side.	u/s side.
Station proposal without experiments	480	480	795	795
As adopted by the Railway	356.25 from Ch. 14 to 27.	237.5 from Ch. 14 to 27.	475 from Ch. 27 to end.	475 from Ch. 27 to end.
As based on model scour	31.5	31.5	584	91

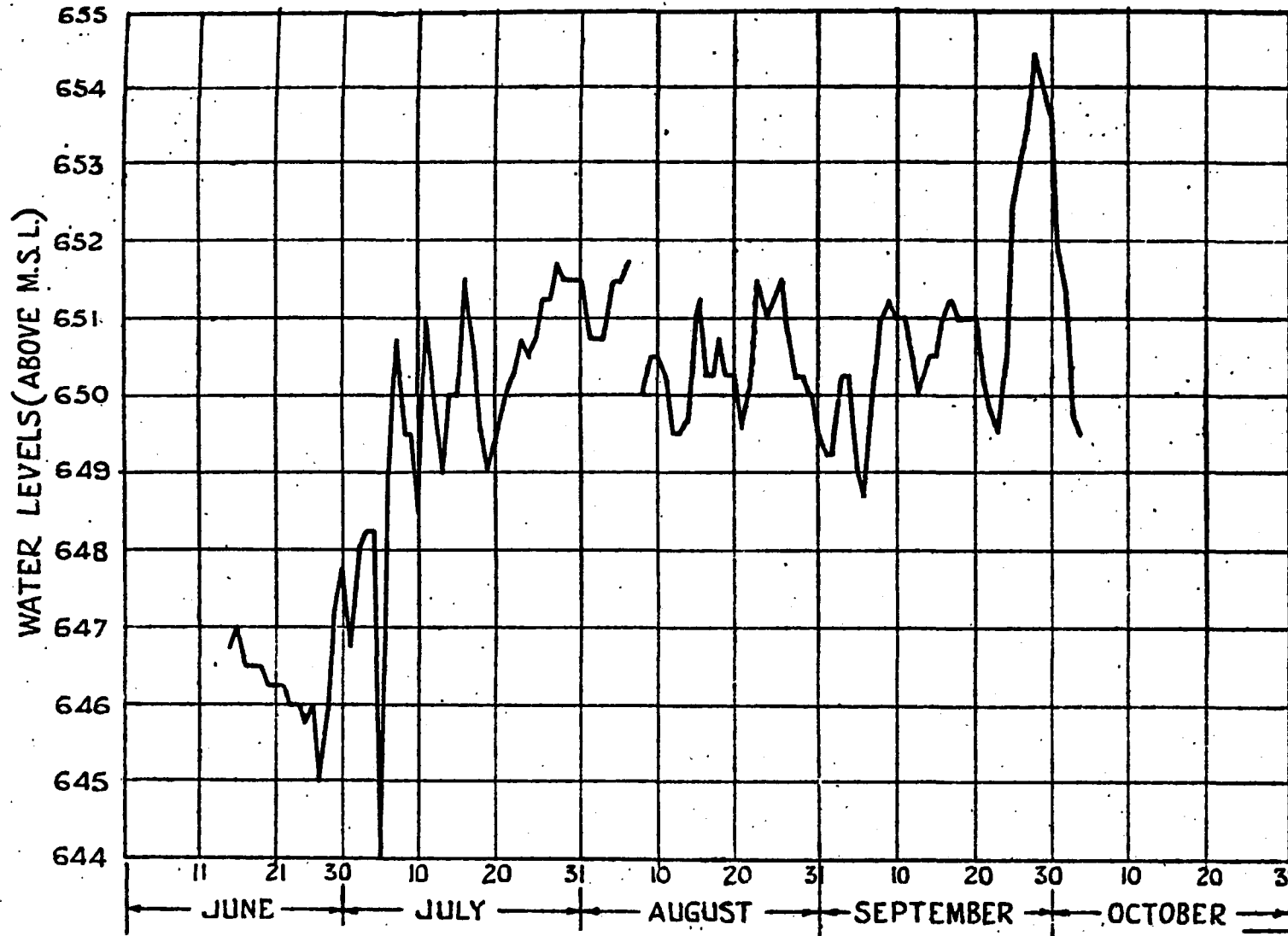
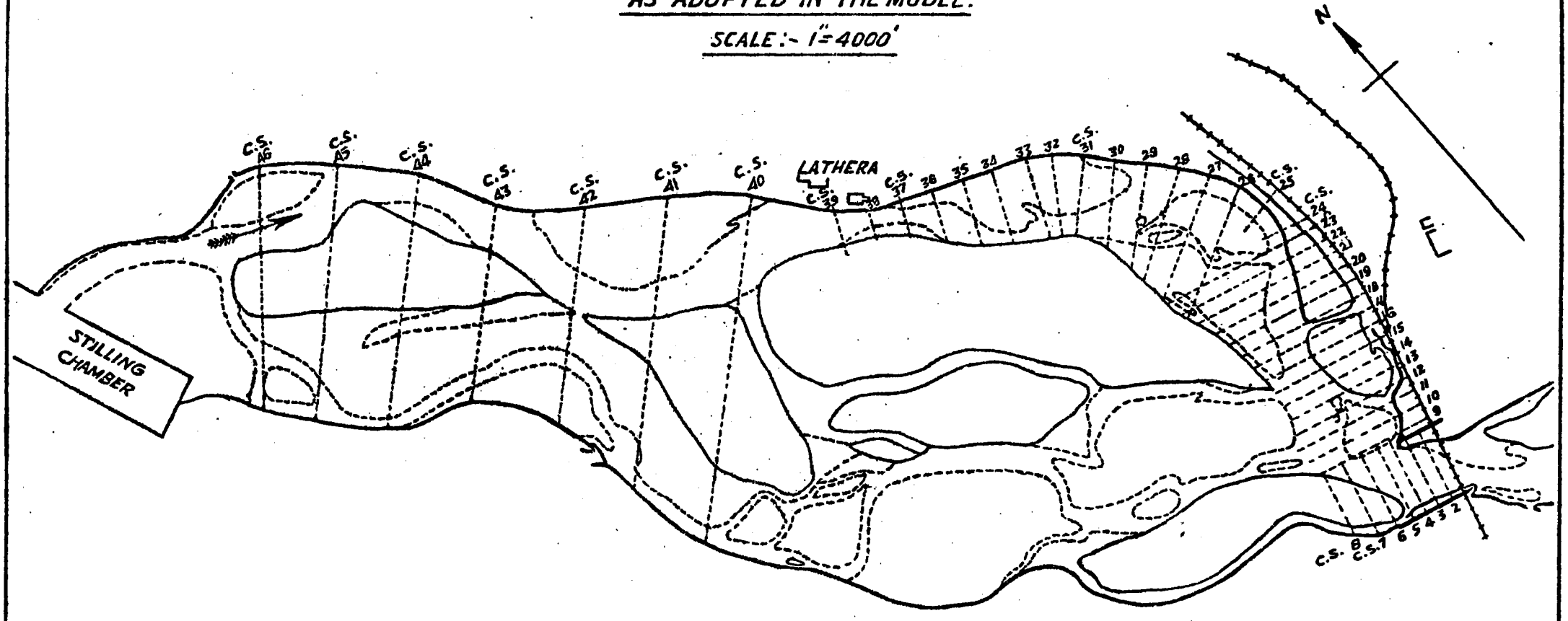


FIG.28  
SHOWING FLOOD LEVELS  
AT GARHMUKTESHAWAR.  
BRIDGE (E. I. RAILWAY)  
IN 1947 FLOODS.

DATUM 643.00

FIG:29  
GANGA RIVER NEAR GARHMUKTESHWAR  
SHOWING ENTRY CONDITIONS  
AS ADOPTED IN THE MODEL.  
SCALE:- 1"=4000'

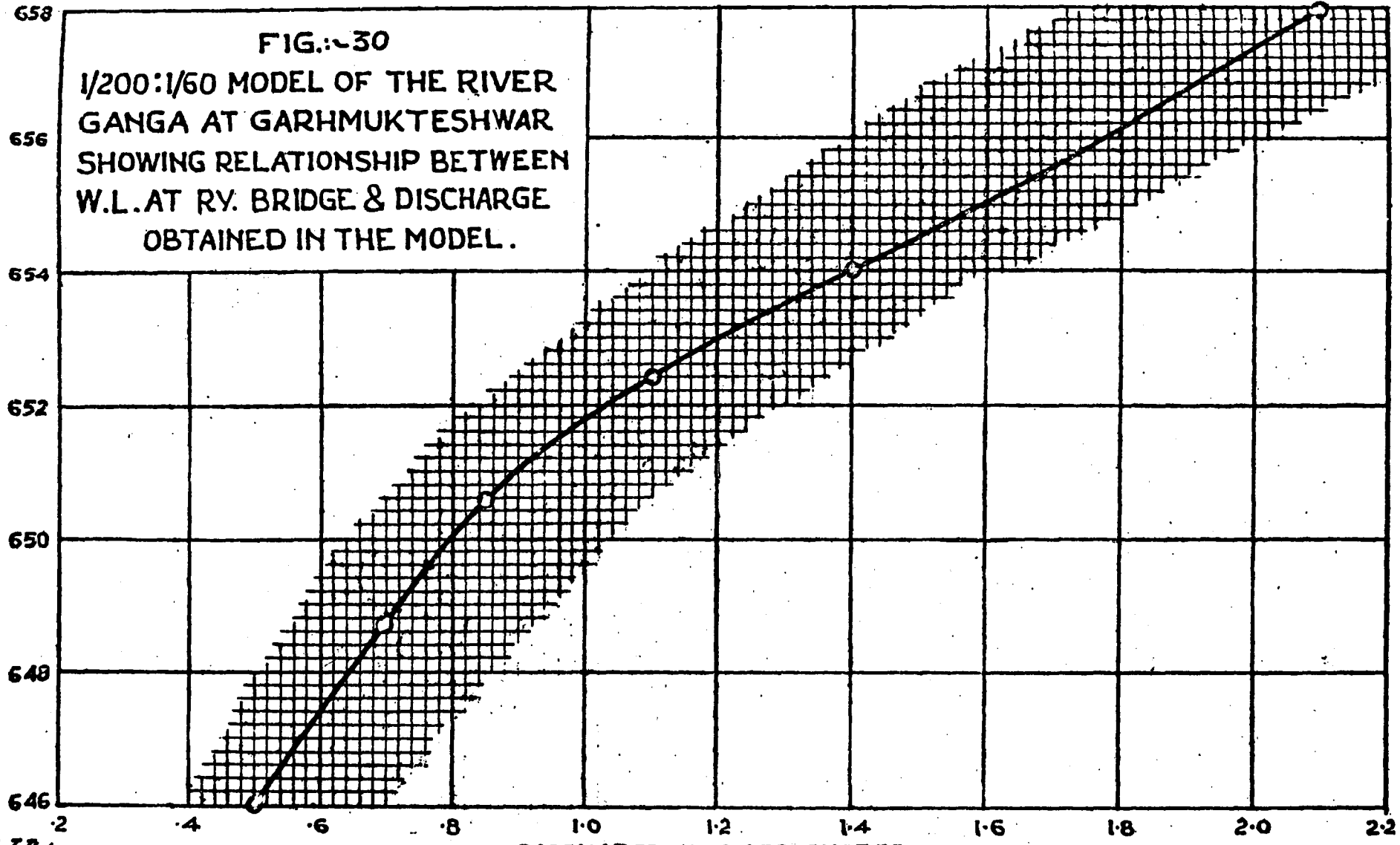


TRACED  
BY:- B.G. DESHPANDE

CHECKED BY:- *M. Singh* 1950

FIG.: 30  
1/200:1/60 MODEL OF THE RIVER  
GANGA AT GARHMUKTESHWAR  
SHOWING RELATIONSHIP BETWEEN  
W.L. AT RY. BRIDGE & DISCHARGE  
OBTAINED IN THE MODEL.

W.L. AT THE RAILWAY BRIDGE.



R. T. Baker

DISCHARGE IN LACS CUSECS.

FIG. 31

CW 1 NASTH POSHA

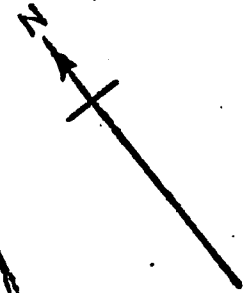
GANGA RIVER NEAR GARHMUKTESHWAR, SHOWING-  
LINES OF FLOW WITH  $Q=70000$  C.S. IN-  
EXPT. I WITH EXISTING CONDITIONS.

SCALE. 1"=2000' PROTO.

1"=20' MODEL.

1/2 MILE 39

LATHERA  
VILLAGE



- REFERENCE.
- SURFACE LINE FLOW.
  - - - - - BED LINE FLOW.
  - WATER EDGE.

FR. M.K. SABNIS.

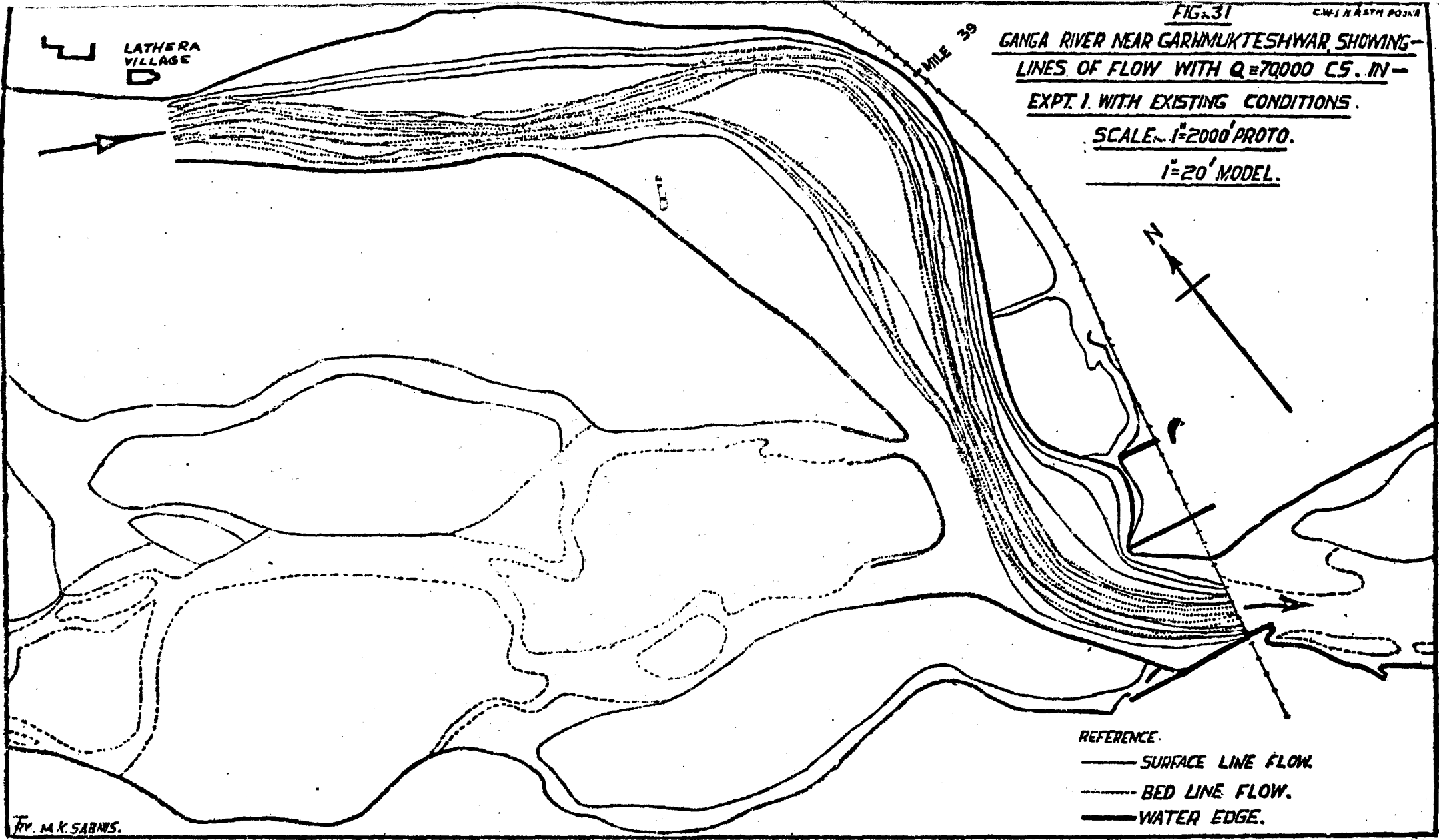
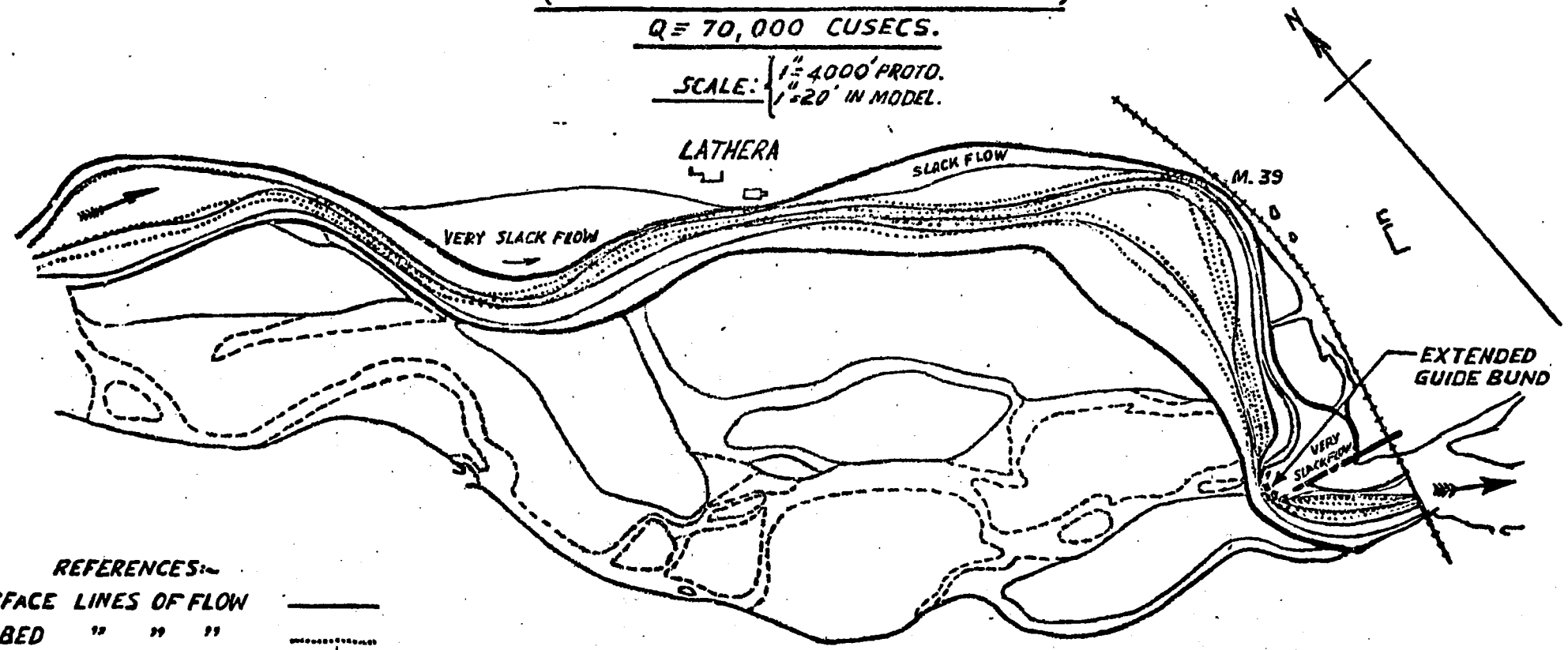


FIG:32 (EXPT. 2)  
GANGA RIVER AT GARHMUKTESHWAR  
1/200:1/60 MODEL OF THE GANGA  
SHOWING BED AND SURFACE FLOW LINES.  
(WITH THE EXTENDED GUIDE BUND.)  
Q = 70,000 CUSECS.

SCALE:  $\left\{ \begin{array}{l} 1'' = 4000' \text{ PROTO.} \\ 1'' = 20' \text{ IN MODEL.} \end{array} \right.$

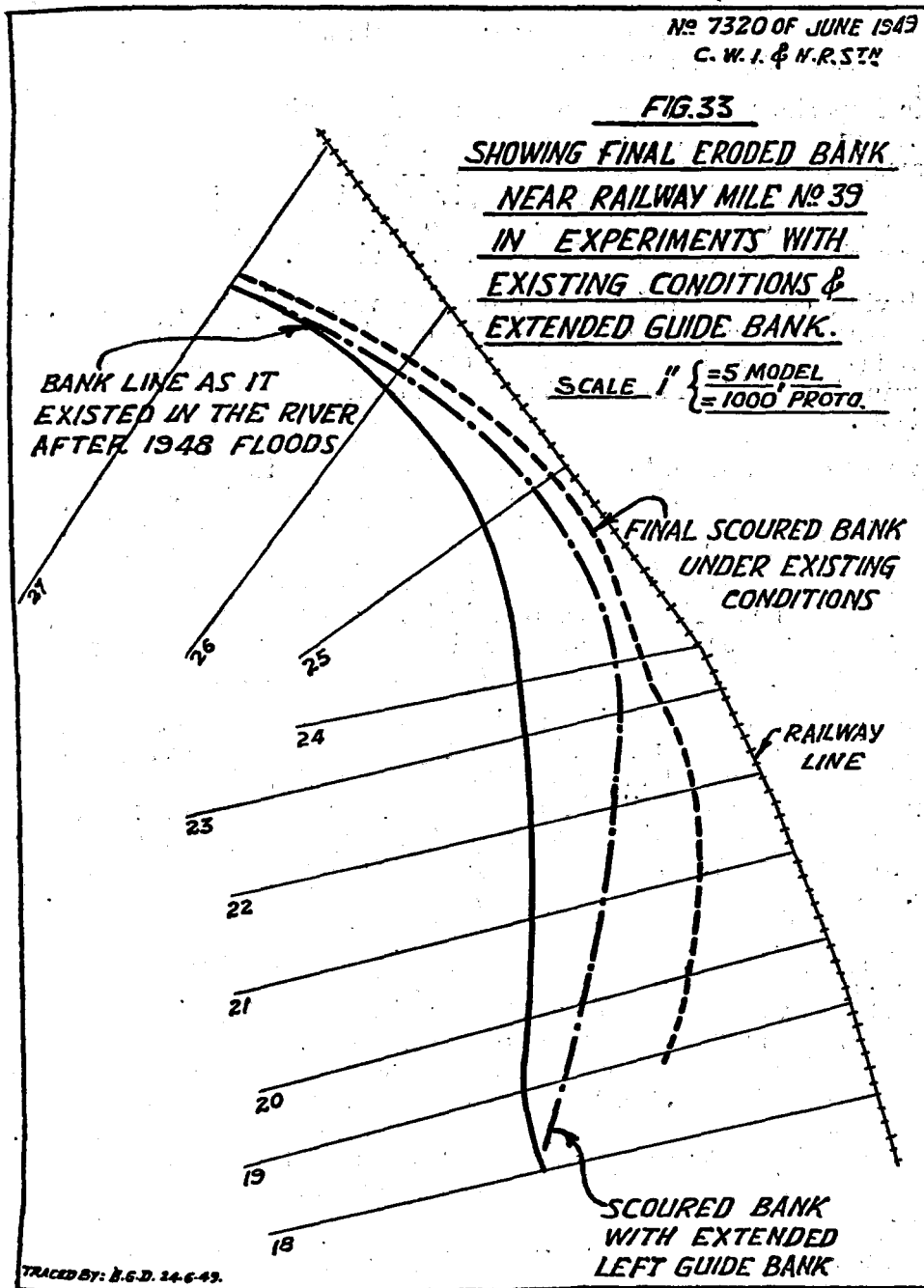


REFERENCES:-  
SURFACE LINES OF FLOW —————  
BED " " " " .....  
WATER EDGE. —————

TRACED BY:- B.G. BISHNAPANDE. -CHECKED BY:- *M. S. ...* 1-7-50



TRAINING GANGA RIVER AT GARHMUKTESWAR



In view of the data of table 2 it was considered that the pitching in the apron as provided on the upstream side of the shank up to Ch. 32 was not necessary and the safety of the same will be ensured if the pitching on the slopes only and on the apron round the curved head beyond Ch. 32 is completed. It was also suggested that the thickness of the apron round the curved head starting from Ch. 32 on the upstream side, round the curved head and down to Ch. 27 on the river side should be strengthened by 9" in depth. It was also recommended that the protection of the shank on river side should not be reduced in view of the river changing its course in future.

In Experiment No. 5, a test was carried out with existing configuration of banks and bed at Lathera and a spur to divert the high velocity flow away from Lathera village bank. A repelling spur upstream of Lathera was visually observed to give local protection but had an adverse effect on the scour near mile no. 39. As the conditions did not appear satisfactory this experiment was abandoned.

Having collected information on the various points required, an experiment was carried out with the constriction at Lathera village removed (Fig. 38) as it was considered that the stiff clay bank at this place constrained free river movement and thus tended to aggravate conditions at mile 30.

It was observed that during the first stages of the experiment the bank near mile No. 39 scoured but the rate of erosion was less. As experiment progressed, the bank erosion stopped and a channel tended to scour on the shoal opposite the point at mile No. 39. This channel formation is clearly seen in photo 5639. The deep water channel obtained in this experiment is shown in (Fig. 38) from which it will be noticed that the scoured bank at mile 39 was still nearly 400 ft. away from the railway line as against 250 feet in experiment 2.

Considering also that in the river, scouring continues till the river falls very low whereas in the model the scour ceased at a relatively high discharge equivalent to 30,000 cusecs and failed to reproduce accretion of suspended silt in slack water areas—one of the model limitations—it was felt that had the stiff clay bank at Lathera not resisted the river attack, conditions would have been less severe at the bridge.

### *Conclusion*

Fig. 39 shows the probable final channels, with and without extended guide bund. This was worked out on the worst observed 2° curve. It will be seen that if river conditions further upstream do not change with the extended bund, the tendency of the river would be to assume a limiting course (see broken line in fig. 39). The nearest bank point, being 2,000 ft. away from the retired line; and that without the extension of the left guide bund, outflanking of the bridge was obvious.

In other words, retiring the line and extending the left guide bund appear essential and either of these cannot be avoided as no feasible economic methods could be worked out for stopping the bank erosion.

Yet another point worth noting was that the bank at mile 39 consists of clay and stands vertical.

It is thus clear that in spite of the extended guide bund, erosion of the bank is likely to continue and that the Railway should be prepared to hold the existing Railway line, as this will serve as the first line of defence.

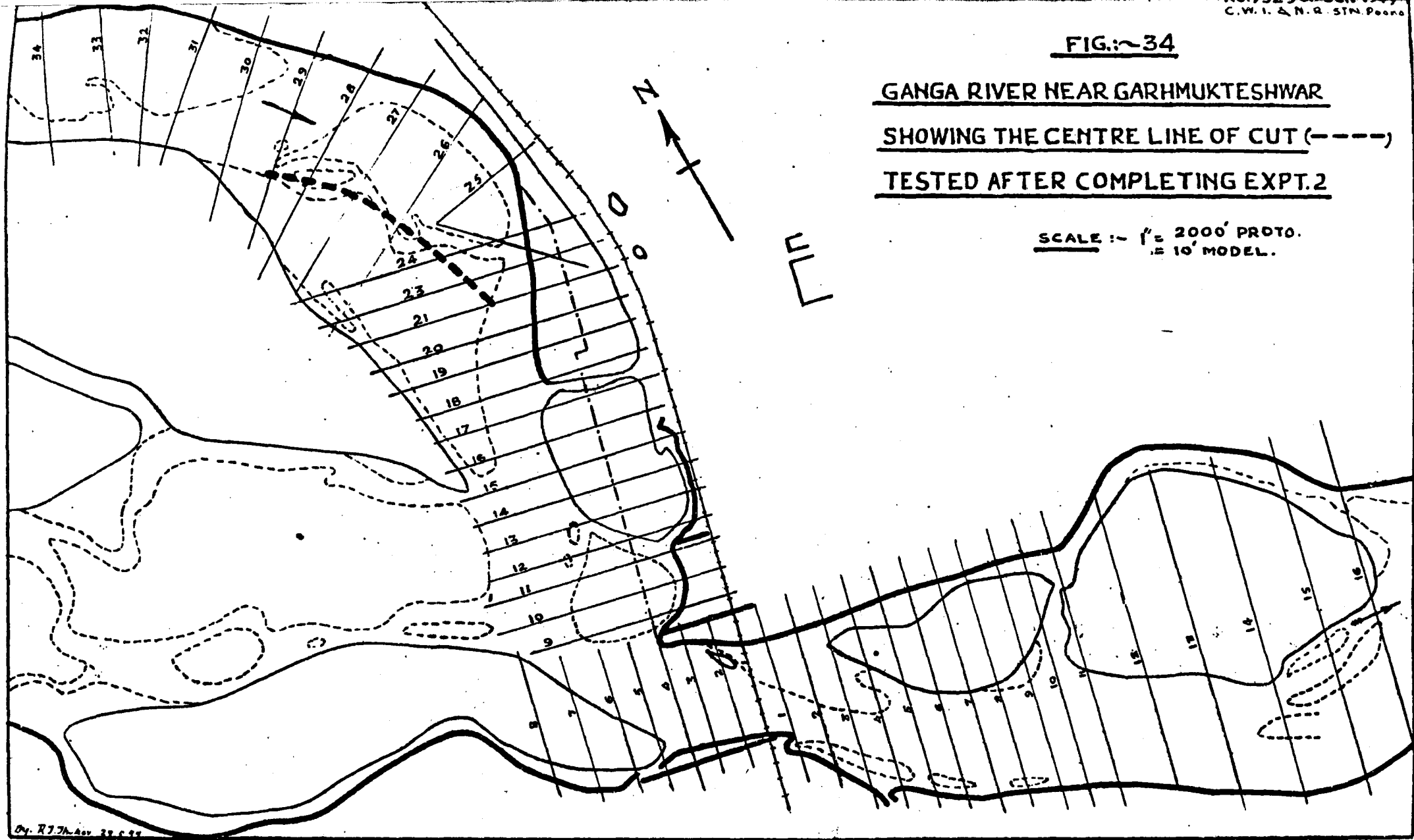


FIG. 34

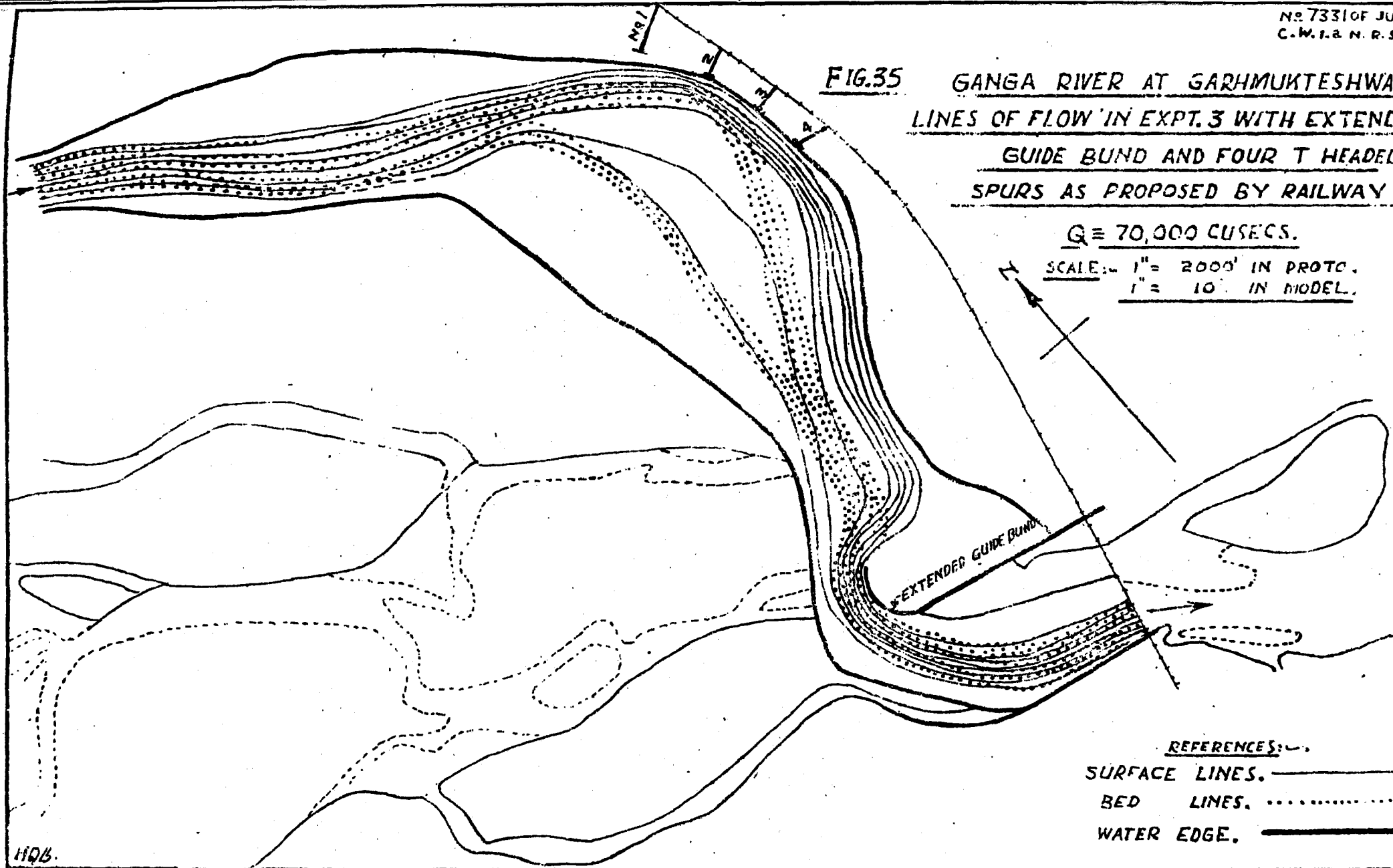
GANGA RIVER NEAR GARHMUKTESHWAR  
SHOWING THE CENTRE LINE OF CUT (-----)  
TESTED AFTER COMPLETING EXPT. 2

SCALE :- 1" = 2000' PRD TO.  
1" = 10' MODEL.

**FIG.35** GANGA RIVER AT GARHMUKTESHWAR  
LINES OF FLOW IN EXPT. 3 WITH EXTENDED  
GUIDE BUND AND FOUR T HEADED  
SPURS AS PROPOSED BY RAILWAY

$Q \approx 70,000$  CUSECS.

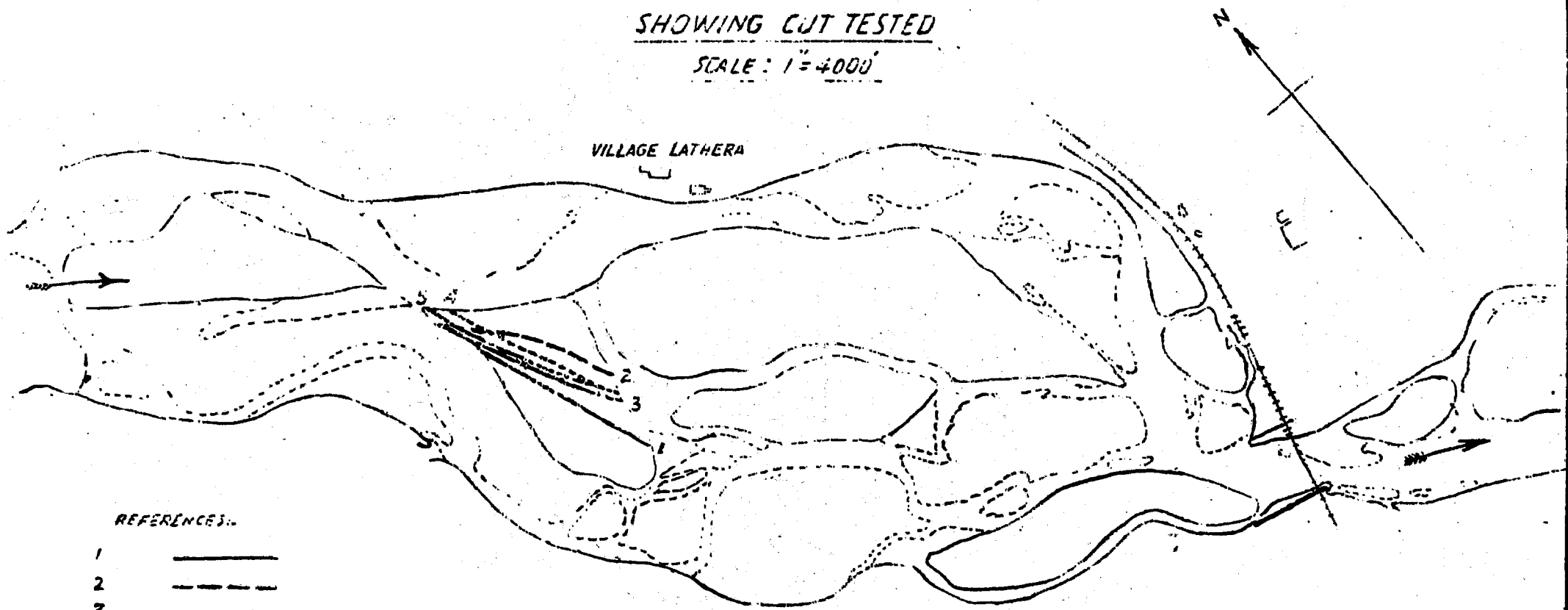
SCALE: - 1" = 2000' IN PROTC.  
1" = 10' IN MODEL.



REFERENCES:  
SURFACE LINES. —————  
BED LINES.           .....  
WATER EDGE.        —————

HQB.

FIG. 36  
GANGA RIVER  
NEAR GARHMUKTESHWAR  
SHOWING CUT TESTED  
SCALE: 1"=4000'



- REFERENCES:-
- 1 —————
  - 2 - - - - -
  - 3 ————/———
  - 4 ————x———
  - 5 ————|———

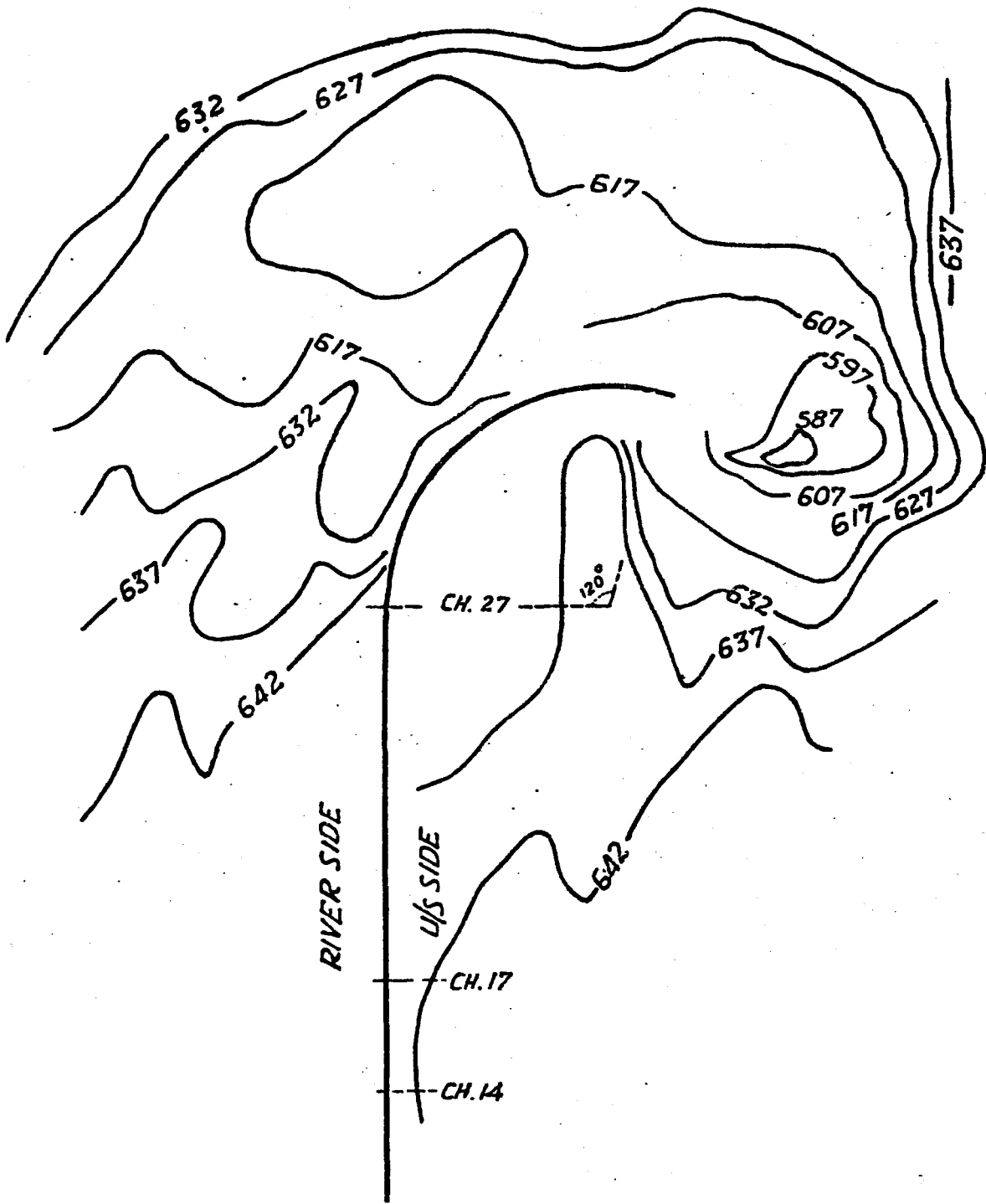
DRAWN BY: B. G. DESHPANDE

CHECKED BY: *M. S. Patil* 12-50

FIG. 37

SHOWING THE CONTOURS OF SCOUR  
OBSERVED ROUND THE CURVED HEAD  
OF THE LEFT GUIDE BANK.

SCALE: 1" = 400'



## Railway Bridge

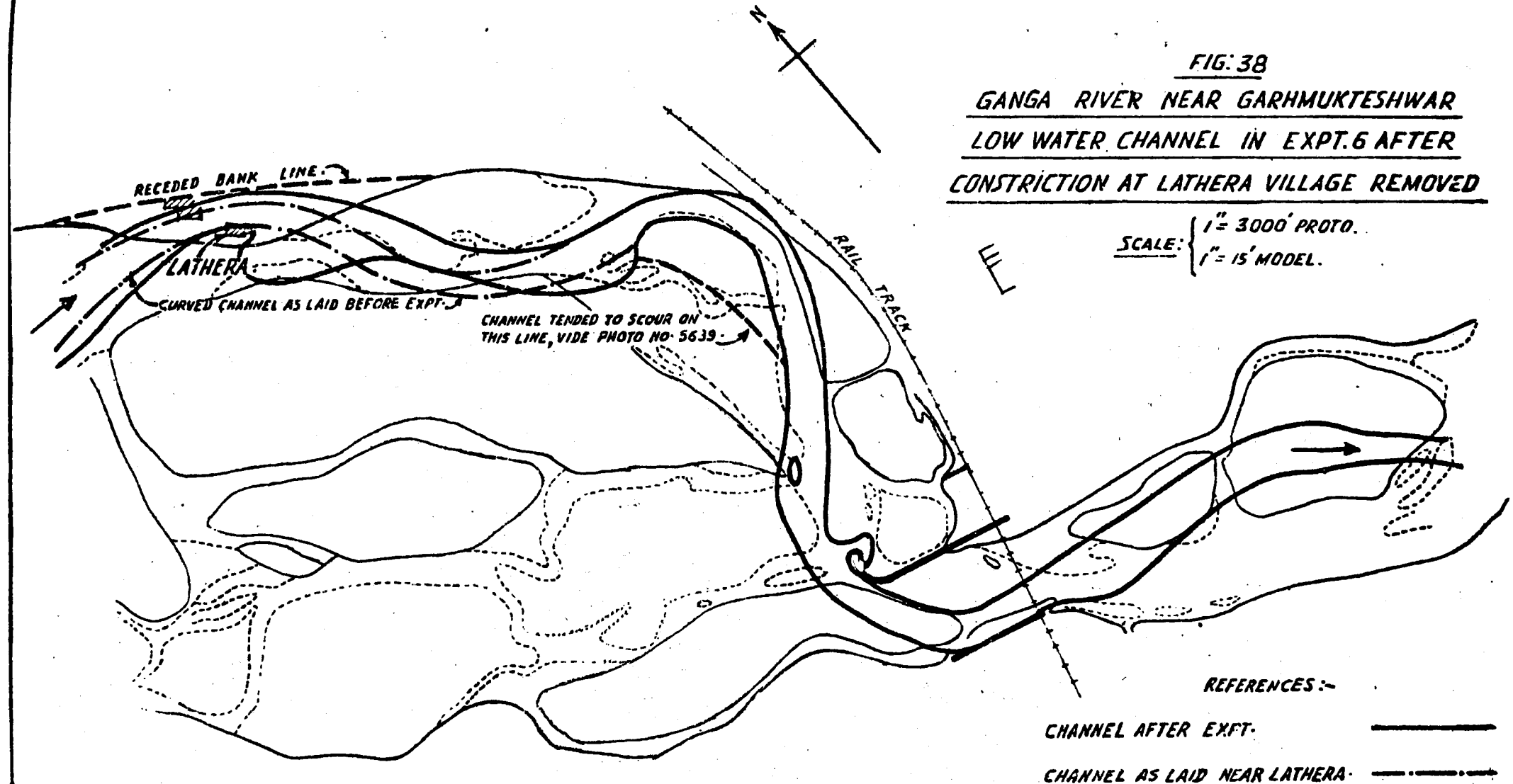


Photo No. 5639—Photo looking downstream shows tendency of cutting a channel through the shoal in experiment 6; with Lathera bank cut back. It may be noted that in experiment 2, an artificial cut on the same shoal did not develop but was obliterated.

1. Railway Bridge:
2. Extended Left Guide Bank
3. Railway Track.

**FIG. 38**  
**GANGA RIVER NEAR GARHMUKTESHWAR**  
**LOW WATER CHANNEL IN EXPT. 6 AFTER**  
**CONSTRICTION AT LATHERA VILLAGE REMOVED**

SCALE:  $\left\{ \begin{array}{l} 1'' = 3000' \text{ PROTO.} \\ 1'' = 15' \text{ MODEL.} \end{array} \right.$



REFERENCES :-

- CHANNEL AFTER EXPT. —————
- CHANNEL AS LAID NEAR LATHERA. - - - - -



FIG. 39.

GANGA RIVER NEAR GARHMUKTESHWAR.

SHOWING MAXIMUM SWING OF THE RIVER

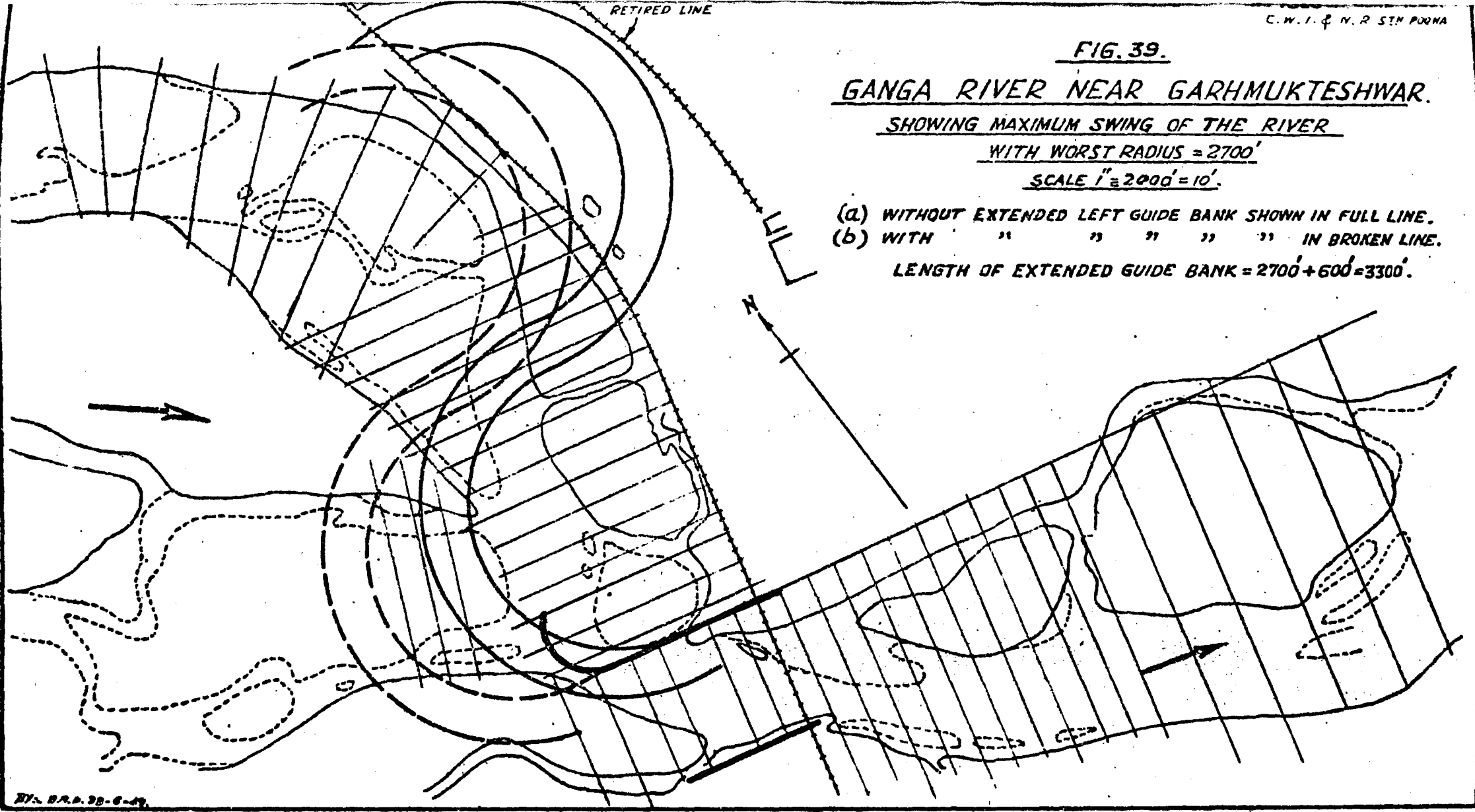
WITH WORST RADIUS = 2700'

SCALE 1" = 2000' = 10'.

- (a) WITHOUT EXTENDED LEFT GUIDE BANK SHOWN IN FULL LINE.
- (b) WITH " " " " " " IN BROKEN LINE.

LENGTH OF EXTENDED GUIDE BANK =  $2700' + 600' = 3300'$ .

RETIRED LINE



#### TRAINING GANGA RIVER AT GARHMUKTESHWAR

The erosion at mile 39 is mainly due to the resistant clay bank at Lathera village. The river has an unnatural constricted section at this place and even without the extension of the left guide bund the Lathera village is in danger. Extending the guide bank has no tendency to increase attack on Lathera village as indicated by the model.

Due to the extended guide bund, the flow swings round more further upstream than at present with the result that the right guide bund will have to be extended by 175 ft. The mole head can be added later on as found necessary.

The extended guide bund will cause afflux upstream till the channel in front of the curved head develops sufficiently. This would have been dangerous if the river had been flashy so that due to insufficient time for scour to occur, the afflux would have been serious. As, however, it rises gradually it is anticipated that the channel will scour out and that the free board provided will be sufficient.

Anything done to protect the Lathera village under existing conditions, will be harmful from the point of view of the Railway unless, further upstream, the meanders change causing reversal of curvature near mile 39.

## 7. ROAD BRIDGE ACROSS THE GAUTAMI-GODAVARI RIVER AT ALAMURU BELOW RAJAHMUNDRY—MADRAS STATE

### 1. General

National Highway No. 5:—Madras-Calcutta Road runs along the East Coast of India and crosses the river Godavari at Rajahmundry, 360 miles to the north of Madras, where there is only a railway bridge. Under the Post-War Road Planning recommended by the Nagpur Report the Government of India have assumed responsibility for the maintenance and development of this important National Highway. Construction of a bridge across the river Gautami has been provided for in the first five-year plan of the Post-War Road Development Scheme.

### 2. River Godavari

2-1. The Godavari River rises in the Western Ghats and after draining an area of 1,15,570 sq. miles in its course of about 900 miles falls into the Bay of Bengal, south of Kakinada. During the greater portion of its course it runs through the high table lands of the Deccan and Hyderabad and thence finds its way through the Eastern Ghats where it narrows in places to about 600 ft. wide and 150 to 200 ft. deep during floods.

Fig. 40.  
Photo 5582.

2-2. After leaving the hills the river enters an alluvial plain which widens out over 60 miles over the coast forming a large delta. From the hills to Dowleshwaram it is contained in a single channel called Akhanda Godavari which has a width varying from a mile to two.

Photos 5569  
and 5573.

2-3. At Dowleshwaram, a weir extending over 19,000 feet was constructed in the year 1852. From the hills where the river emerges into a plain, there are marginal embankments to protect the low lying country on either side. From this anicut the river splits into two main channels called the Gautami-Godavari and Vasista-Godavari. A large and very fertile delta is situated between the two arms of the river and the sea. There are marginal embankments running from the anicut to the sea to protect the low lying country on either side of the Gautami and Vasista from floods. Wherever the embankment is under attack from the flood it is protected by stone pitching and/or 'Nanal' growth. Close to these flood-banks and running parallel to them are irrigation and navigation canals which have limited the possibility of retiring the marginal embankments under attack. Under such conditions very costly protective works involving dumping of large quantities of boulders are done to protect the embankments from the attack of the floods.

### 3. Existing Means of Communications across the River Godavari

3-1. Across the river Godavari in its course of 200 miles in Madras State there is no road bridge. The river is perennial and does not admit of any traffic wading through the river during any part of the year. The Northern Trunk Road crosses the river at Kovvur on the southern right bank and Rajahmundry on northern bank, a ferry service bridging the gap.

3-2. *Railway Bridge*:—There is a Railway Bridge at Rajahmundry (M. S. M. Rly.) built in the year 1900, carrying a single line of broad gauge traffic. It consists of 56 spans of 150 ft. and one span of 40 ft., the total length being 9,000 ft. It was designed to pass a maximum flood of 15,00,000 cusecs.

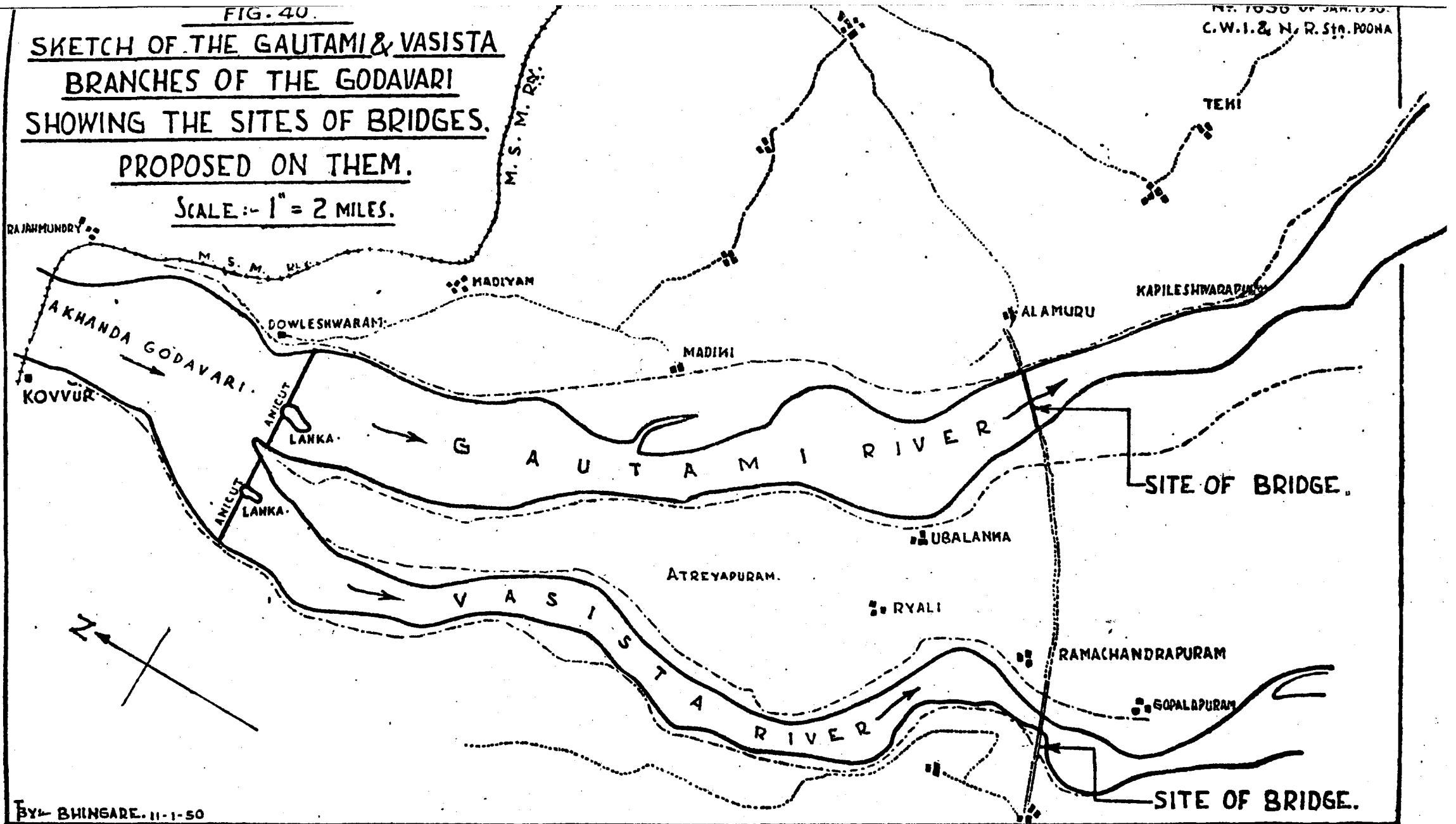
Photo 5582.

3-3. During the dry months of April and May limited traffic is allowed over the Anicut. But in any case this is no public thoroughfare. There are also ferry services running at many places but are very inconvenient and troublesome causing inordinate delays. There is thus an urgent necessity for a Road Bridge on this important National Highway.

FIG. 40.

SKETCH OF THE GAUTAMI & VASISTA  
BRANCHES OF THE GODAVARI  
SHOWING THE SITES OF BRIDGES.  
PROPOSED ON THEM.

SCALE :- 1" = 2 MILES.



ROAD BRIDGE ACROSS GAUTAMI-GODAVARI

4. *Dowleshwaram Anicut*

4.1. The anicut, built in the year 1852, consists of 4 sections with islands in between. The total length of anicut including the islands is 19,214 feet.

The length of Dowleshwaram Anicut	4,839 ft.	}	Making up the
"    "    " Ralli	2,859 "	}	Gautami River.
"    "    " Maddur	1,550 "	}	Making up the
"    "    " Vizzeshwaram	2,601 "	}	Vasista River.

5. *Bridge across Gautami at Alamuru*

5.1. Two alternative sites have been very critically considered by the Project Authorities; they are the one at Alamuru and the other at Rajahmundry. On considering all the pros and cons of the two alternatives they have finally accepted to locate the bridge across the Gautami, at Alamuru. The Research Station, under the terms of reference, is given freedom to finalise this site within a furlong upstream or downstream of the proposed site and as such had no scope to offer its comments on the relative merits of the two sites.

5.2. Under the terms of reference the following points indicate the scope of the Research Station in solving the problem with the aid of model experiments:—

(a) to finalise the site within a furlong upstream or downstream of the proposed site 12 miles 6 furlongs and 295 ft. on the left bank of the Gautami (12/6 + 295 GLB) and 15 miles 4 furlongs 495 feet on the right bank of Gautami (15/4 + 495 GRB) at Alamuru;

(b) to determine the exact orientation of the bridge;

(c) to determine the waterway through the bridge; and

(d) to design suitable protection works for the bridge.

5.3. Detailed investigations and surveys have been made for the bridge at Alamuru. The river has a width of 8,600 feet between the marginal bunds; the width during a normal flood, however, is 6,700 feet. A study of the river at this site indicates that the deep course has been shifting from bank to bank. The river carries a maximum flood discharge of 10,55,600 cusecs as observed at the Dowleshwaram Anicut during the floods of 1942. With the experience gained at the bridging of the rivers in Northern India one school of thought considers it economical and desirable to constrict the river and provide for the bridging of the minimum section of the river capable of passing, safely a maximum flood discharge with adequate protective works, thus ensuring complete control. Another school of thought however is of opinion that any constriction practised in the North Indian Rivers which have their own peculiar features of terrain and flow may not be practicable and desirable in the case of South Indian Rivers. Further, there is a feeling that such a constriction—though may or may not be safe for the bridge—may have disastrous effects from the point of view of the safety of the river, existing river protection works, irrigation works of the delta and the delta itself.

5.4. As there can be no unanimity of opinion the issues involved being of a very complex and important nature, model experiments alone would have thrown some light on the solution of the problem and as such the matter was referred to the Research Station for advice.

6. *Hydraulic Data*

6.1. It must be recognised that the Madras Project Authorities have collected large data of the hydraulic particulars of the problem. However there is rather a paucity in the precise data necessary for model experiments as will be explained below.

6.2. *Estimates of maximum flood discharge*

Fig. 41. 6.2.1. From the anicut to the sea there are marginal bunds along the Gautami and Vasista and as such they do not receive additional drainage in this region to what passes down in the two branches from over the anicut at Dowleshwaram. It is, therefore, quite permissible to measure the discharge over the anicut itself and for that purpose gauges have been installed upstream and downstream of each of the four sections of the anicut. The discharges are then computed from these gauge readings and such record exists for well over a hundred years.

Fig. 41. 6.2.2. It must be noted that neither the formula used nor the tables employed to compute the discharge, were supplied to the Station. It was stated that the flood discharge computations from the gauge data were fairly reliable and might be adopted for design purposes. However the correctness or otherwise of this data depends exclusively on the location of the downstream gauges which ought to record the height after the jump. The position of this jump or of the undulation at high submergence ratio is changing and as such the downstream gauge records a doubtful reading. For these reasons the basis on which these computations have been made would have been very valuable for verification.

6.2.3. In 1942, the river received the highest flood viz. 10,55,600 cusecs which has a frequency of 1 in 108 years. This frequency is suitable for adoption in designing a high level bridge. The following computation shows the estimate of this very flood as 6,87,437 cusecs as obtained from a formula usually employed:—

$$\begin{aligned}
 Q &= Q_1 + Q_2 \\
 Q_1 &= \frac{2}{3} C_1 l \sqrt{2g} [(h + h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}}]; & C_1 &= 0.571 \\
 Q_2 &= C_2 l d \sqrt{2g} (h + h_a); & C_2 &= 0.80. \\
 Q &= l \left\{ 3.1 [(h + h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}}] + 6.4d (h + h_a)^{\frac{1}{2}} \right\}
 \end{aligned}$$

where  $d$  = depth of tail water above crest  
 $h$  = actual head  
 $h_a$  = velocity head

For Dowleshwaram Anicut:—

$$d = 16.75 \text{ feet, } h = 0.35 \text{ feet, } h_a = 0.38 \text{ feet, } l = 4847.25 \text{ feet.}$$

For Ralli Anicut:—

$$d = 16.75 \text{ feet, } h = 0.25 \text{ feet, } h_a = 0.34 \text{ feet, } l = 2859.0 \text{ feet}$$

$$Q = 4,49,767 + 2,37,670 \text{ cusecs}$$

= 6,87,437 cusecs against the figure of 10,55,600 cusecs stated above—which is a very large discrepancy.

6.2.4. *Distribution of Flood Discharge in the Gautami and Vasista*

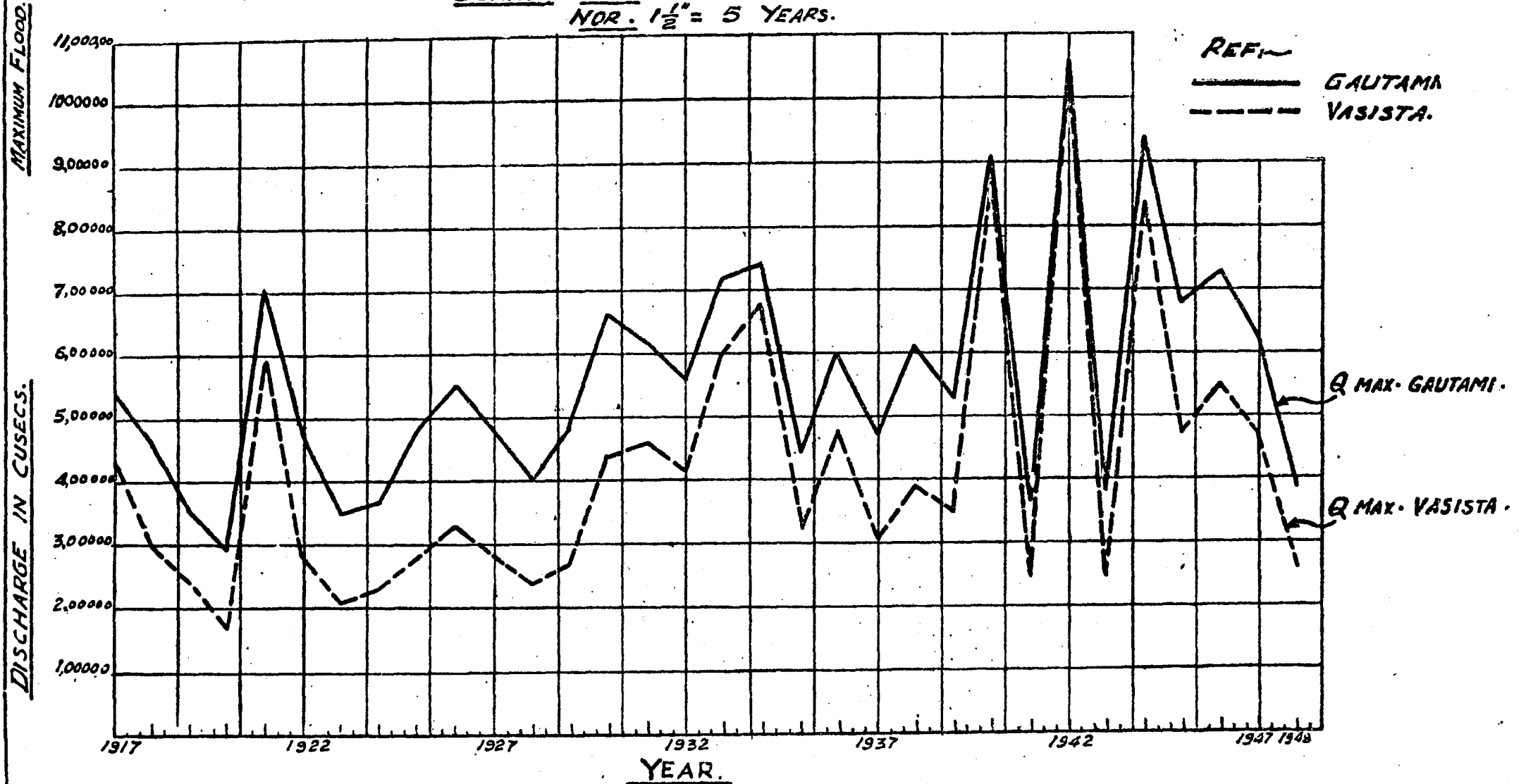
6.2.4.1. From the gauge data it was seen that the distribution of floods between the two arms of the River, viz. Gautami and Vasista, is changing from year to year. The ratio  $Q_{\max} \text{ Gautami} / Q_{\max} \text{ Vasista}$  is above unity for low maximum floods and tends to unity with high maximum floods. It may be remembered that the ratio of lengths of anicuts of Vasista and Gautami is 1:2 (approximate). If the gauge data and subsequent computations of maximum floods are presumed correct, the probable reasons for the change in the ratio of distribution appear to be differential silting up of the bed upstream and the presence of shifting shoals. The anicut has been in existence for the last 100 years and the bed upstream must have silted up to its full capacity. If the river upstream of the anicut has, therefore, attained a certain amount of stability the variation in the ratio of distribution will depend entirely on the correctness of the gauge data and the discharge computations.

ROAD BRIDGE ACROSS GAUTAMI RIVER  
DISTRIBUTION OF MAX. Q IN GAUTAMI  
AND VASISTA IN VARIOUS YEARS

NO. 1704.  
 C.W.I. S. N. R. STN. POONA.

FIG. 41.

SCALE:- Ver. 1" = 2.0 LACS. CUSECS.  
 HOR. 1 1/2" = 5 YEARS.



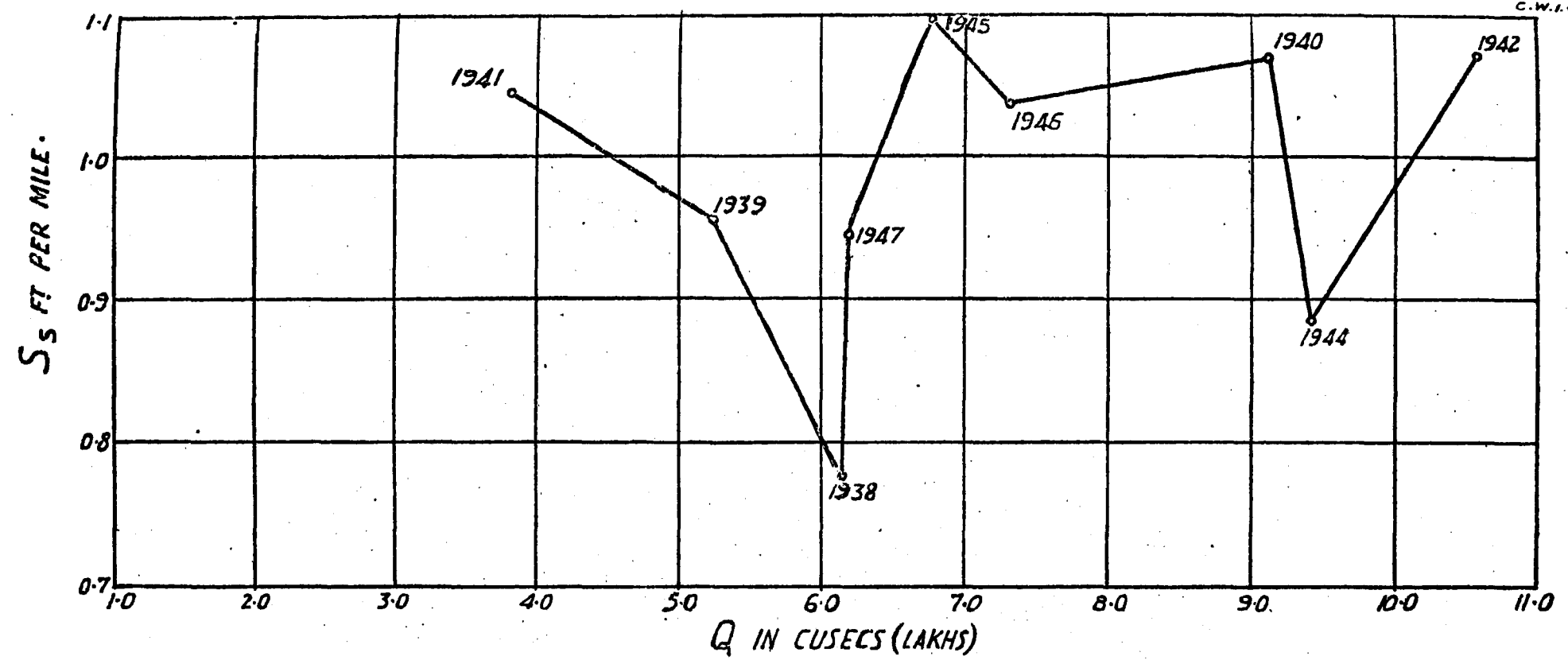


FIG. 42  
ROAD BRIDGE ACROSS GAUTAMI RIVER.  
AVERAGE SURFACE SLOPE : Q<sub>MAX</sub> OF THE PARTICULAR YEAR.  
S<sub>s</sub> : Q<sub>MAX</sub>.  
(WITHIN 0% TO 10% MILES OF GAUTAMI)



## ROAD BRIDGE ACROSS GAUTAMI-GODAVARI

6.2.4.2. There is again a proposal to construct a reservoir—Ramapadasagar—across the river Godavari at Polavaram about 25 miles upstream of the anicuts. It is considered that this reservoir when constructed will have a certain moderating effect on the floods. The Ramapadasagar Project provides for a maximum flood of 3.5 million cusecs. The Bridge Officers, on the basis of various aspects of the probability of such a flood discharge ever occurring, considered that there was no case for providing for 3.5 million cusecs discharge in the design of the bridges across Gautami and Vasista and, therefore, the Gautami Bridge has been designed for a maximum flood discharge of 10,55,600 cusecs and the Vasista Bridge for 10,33,000 cusecs.

Fig. 41.

### 6.3. River Slope Data

6.3.1. Along the Gautami banks there are no gauges installed for observing flood levels. From mile 0/0 to 10/0 on the right and left banks, pegs were driven at the highest water edge reached during the year and after a considerable lapse of time flying levels were taken for the pegs by the survey parties. A graph of average slope (average of right bank and left bank surface slopes) to max. discharge (Q) shows a very large scatter. Further, the data is confined to the region upstream of the bridge site and is not liable to extrapolation to the bridge site.

Fig. 41.

Fig. 42.

6.3.2. In the absence of any correct slope data the surface slope 1.1 ft. per mile observed for the 1942 floods was taken as the basis and slopes for other discharges were allowed to be developed in the model.

### 6.4. Bed Material and Boring Samples

6.4.1. A good number of samples from the exposed bed material and the boring taken in March 1949 in the alignment 12/6+460 GLB to 15/4+330 GRB at 6,826 ft. from GLB were analysed by the Puri Silto-meter and indicated the presence of soils as follows—(All River Levels refer to the mean sea level datum)

- R. L. + 3.0 to -31.0 coarse to medium sand,
- 31.0 to -61.0 black moist clay,
- 61.0 to -67.0 coarse sand with small pebbles,
- 67.0 to -70.0 stiff moist clay mixed with coarse sand,
- 70.0 to -97.0 coarse sand mixed with small pebbles.

6.4.2. The following is a statement of 'm' in mm. and the Lacey  $f_a$  values ( $f_a = 1.76\sqrt{m}$ ) of the samples from the boring in the Gautami at a distance of 6,826 ft. from the left bank along the alignment:—

	R.L.	m	$f_a$
(1) Depth 10' below bed level ...	7.0	0.570	1.33
(2) " 20' " " " ...	17.0	1.920	2.43
(3) " 30' " " " ...	27.0	0.582	1.34
(4) " 40' " " " ...	37.0	0.0092	0.169
(5) " 50' " " " ...	47.0	0.286	0.94
(6) " 60' " " " ...	57.0	0.329	1.01
(7) " 64' " " " ...	61.0	1.66	2.26
(8) " 70' " " " ...	67.0	0.654	1.43
(9) " 73' " " " ...	70.0	4.160	3.59
(10) " 80' " " " ...	77.0	3.875	3.56
(11) " 90' " " " ...	87.0	3.766	3.41
(12) " 100' " " " ...	97.0	6.441	4.46
(13) Pit No. 1 1000' from GLB ...		0.2667	.91
(14) " " 2 2000' " " ...		0.258	.895
(15) " " 3 3000' " " ...		0.407	1.12
(16) " " 4 4000' " " ...		0.492	1.23
(17) " " 5 5000' " " ...		...	...

## 7. Experimental Procedure

### 7.1. Model scales are shown in Appendix 1.

Experiments were made for the following conditions of the river.

Existing conditions of the river were compared with result obtained with

(i) a bridge of total length 7,070 ft. (This is in accordance with the proposal of having a bridge from the right marginal bank to the left flood margin.)

(ii) a bridge of total length 5,050 ft. (This is the proposal made by the Government of India Roads Organisation and on the basis of this proposal tentative designs and estimates have been made).

(iii) a bridge of total length 4,040 feet. (This is a proposal which admits of the idea of maximum permissible constriction of the waterway).

7.2. In all these experiments several rising and falling discharge cycles were run in the model until the model was stabilised. The hydrograph data consisted of gauge readings over the anicut during the flood season (June to October) for ten years. The maximum discharge for each year is also available. From the data of maximum discharge and peak gauge readings, the relation between the two has been investigated. Assuming the estimated relation to be true for the intermediate gauge readings the discharge figures were obtained, and a study of frequency of discharges for all these different years gave the discharge distribution and its characteristics. The discharge hydrograph based on these averaged frequencies as shown in Fig. 43 (C) was used for model experiments. It was found that the worst conditions in respect of bank attack and high velocities occurred in the model at the bankful stage of discharge which worked out as 7'00 cusecs in the model ( $\approx 7,70,000$  cusecs). Consequently the hydraulic particulars of the lines of flow, velocities and contours were taken for the dominant discharge only. The range for observing the lines of flow was from cross section M. 9/0 GLB-11/0 GRB to cross section M. 17/0 GLB-22/0 GRB. The velocities were observed at the following cross sections:—

(i) 9/0 GLB-11/4 GRB.

(ii) 660 feet upstream of the bridge site. (Figs. 45A to 45D).

(iii) 660 feet downstream of the bridge site. (Figs. 46A to 46D).

(iv) 15/0 GLB to 17/4 GRB.

(v) 16/0 GLB to 20/0 GRB (K. Puram section) (Figs. 47A to 47D).

Gauges to observe water levels were installed at places marked X in the index plan. In all experiments with the bridge the right abutment was located at 1,440 ft. along the bridge alignment from the right marginal bund.

## 8. Experiments

8.1. *Orientation of the bridge.*—The new orientation is 12/6+295 GLB to 15/4+495 GRB as against the proposed 12/6+460 GLB to 15/4+330 GRB. This orientation gives normal flow conditions through the bridge and obviates skew condition for piers.

### 8.2. Location of the Right Abutment

It was recommended that the right marginal bank should be heavily protected with pitching and a falling apron; the right abutment should be located 100 ft. inside the present cutting edge of the abnk on the cross-section mentioned in para. 8. 1, as a margin of safety.

FIG: 43

HISTOGRAMS SHOWING THE FREQUENCIES

OF DISCHARGES IN -

DIFFERENT YEARS.

HOR. = DISCHARGE IN 10000 Cs.

VER. = FREQUENCY OF OCCURENCE.

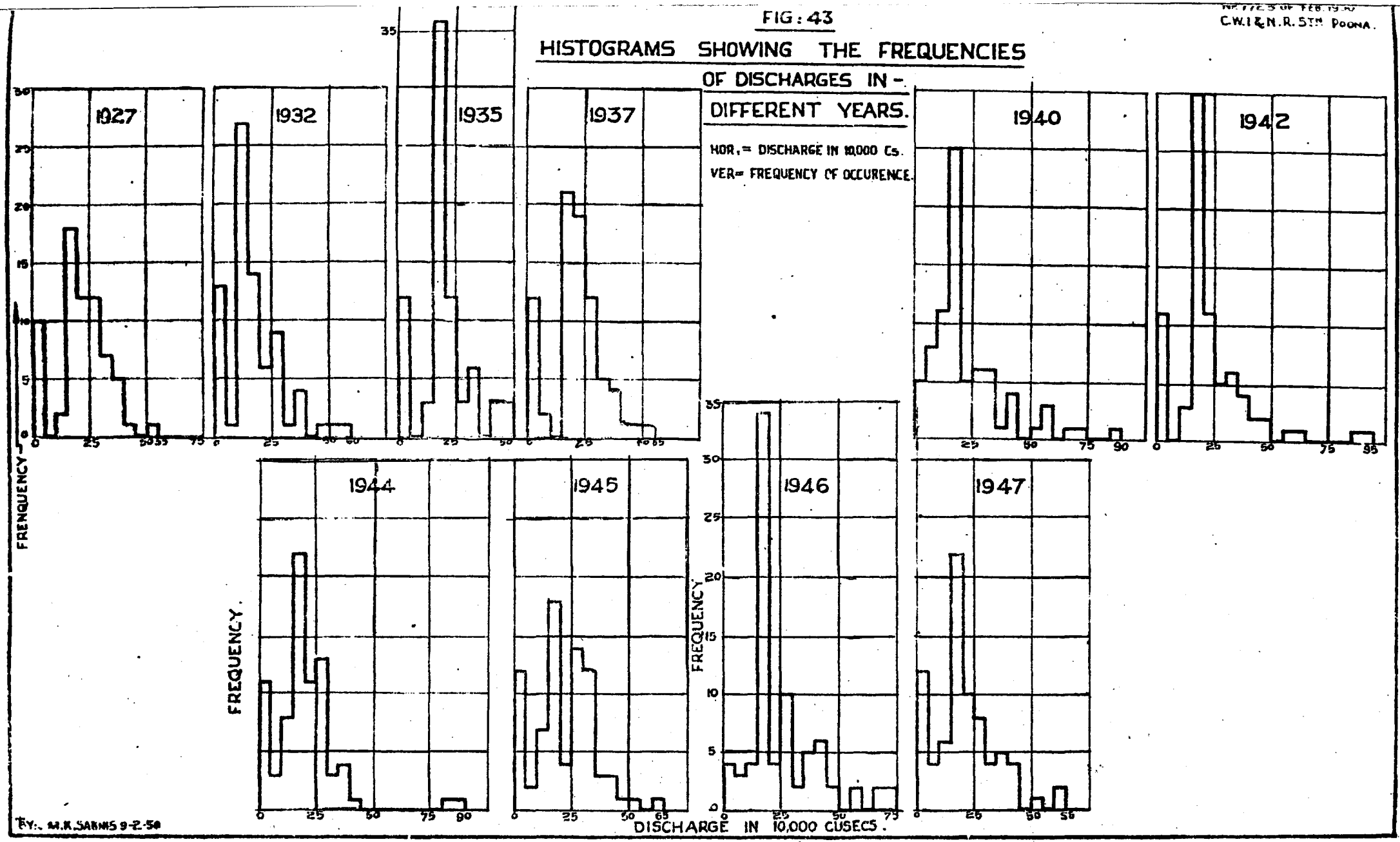
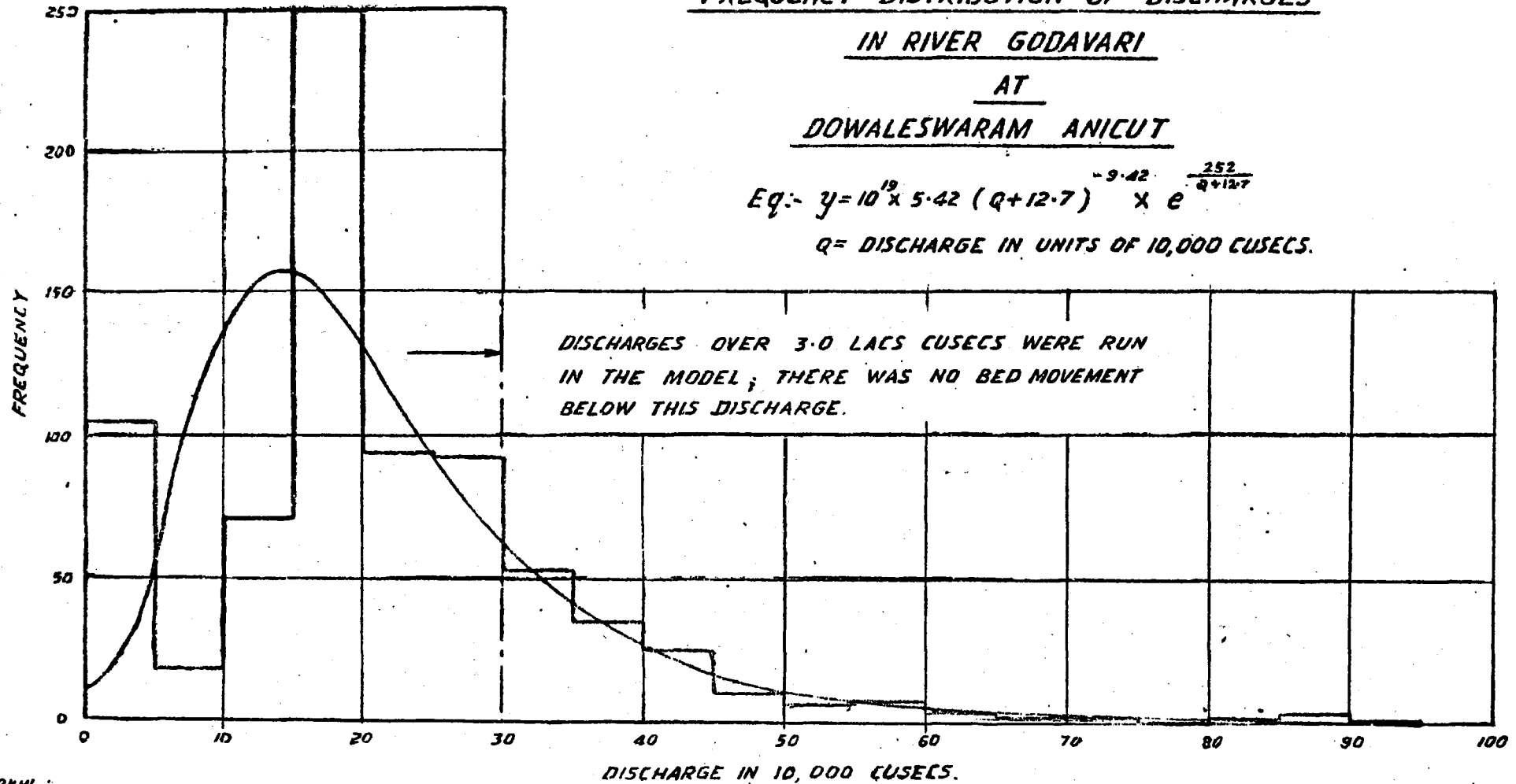


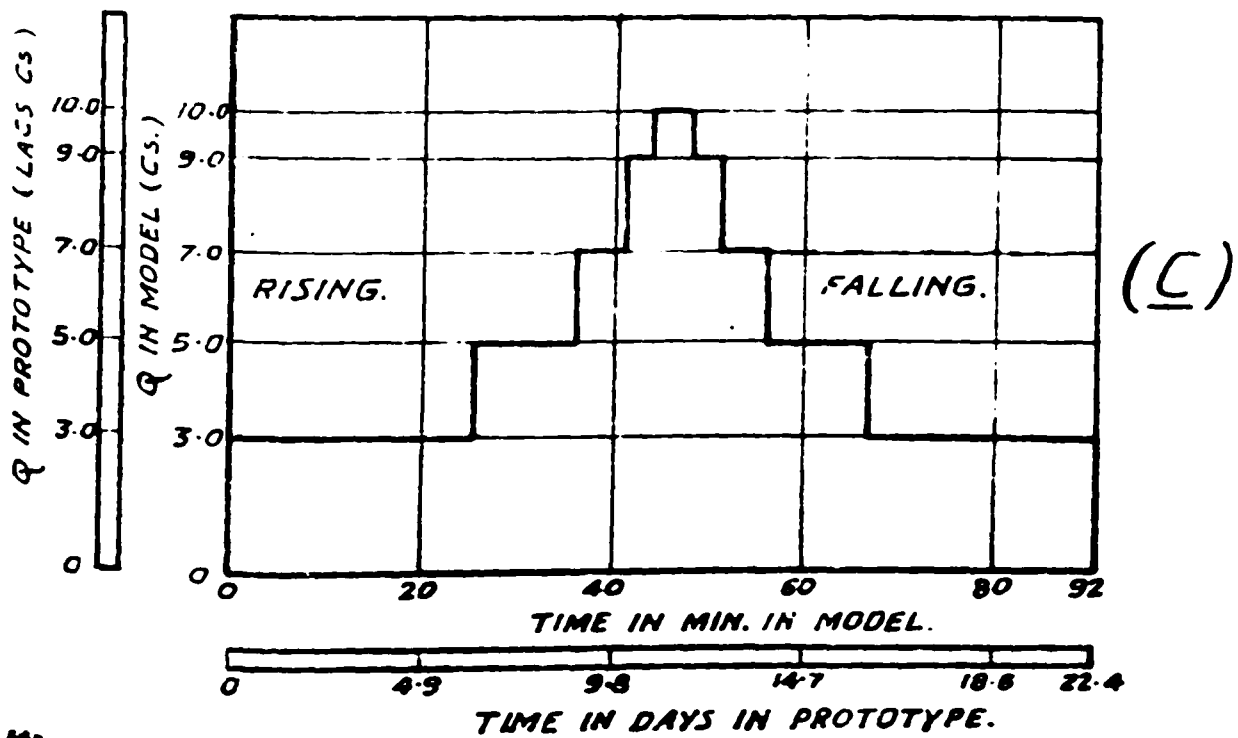
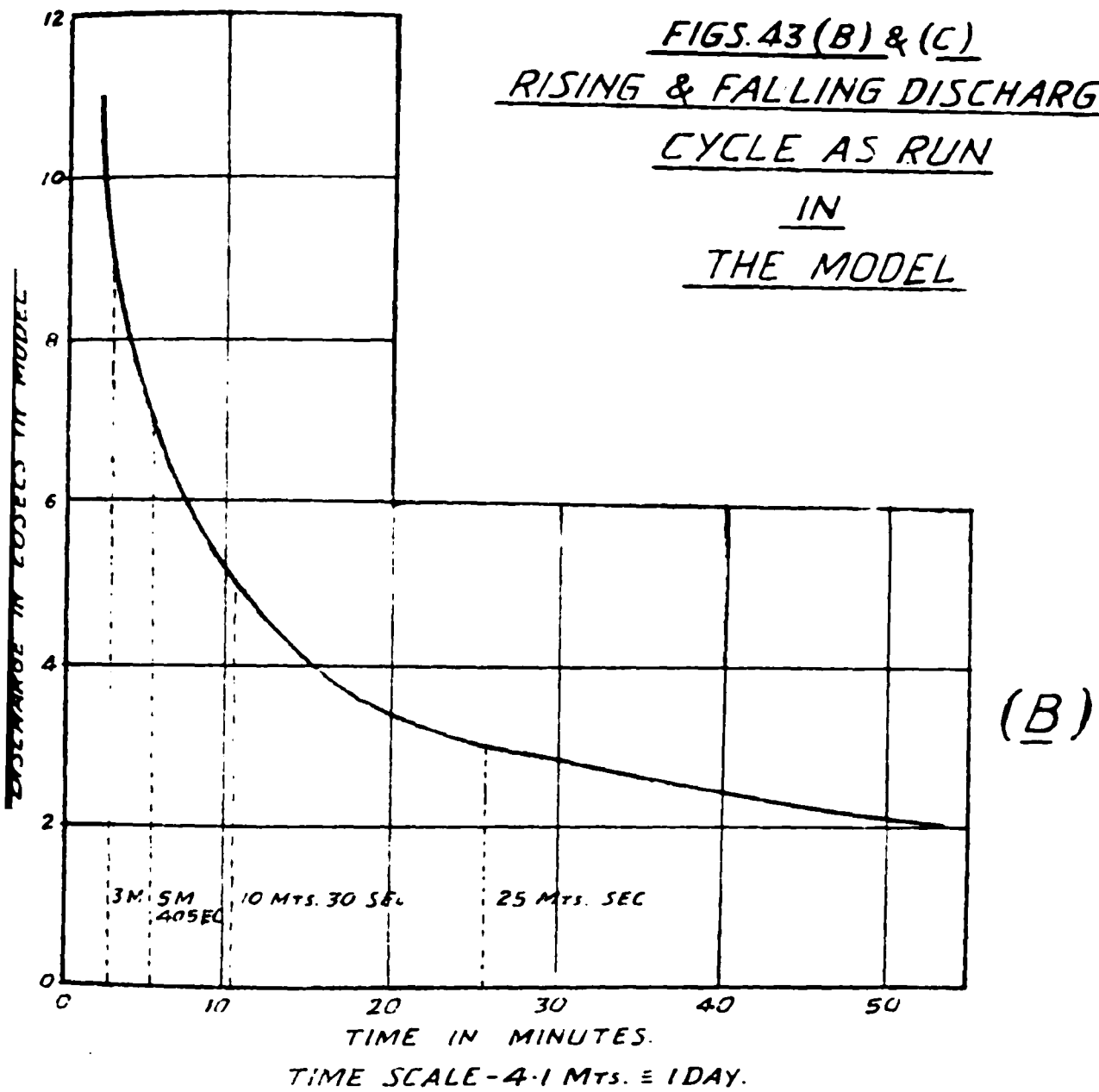
FIG. 43(A)  
FREQUENCY DISTRIBUTION OF DISCHARGES  
IN RIVER GODAVARI  
AT  
DOWALESWARAM ANICUT

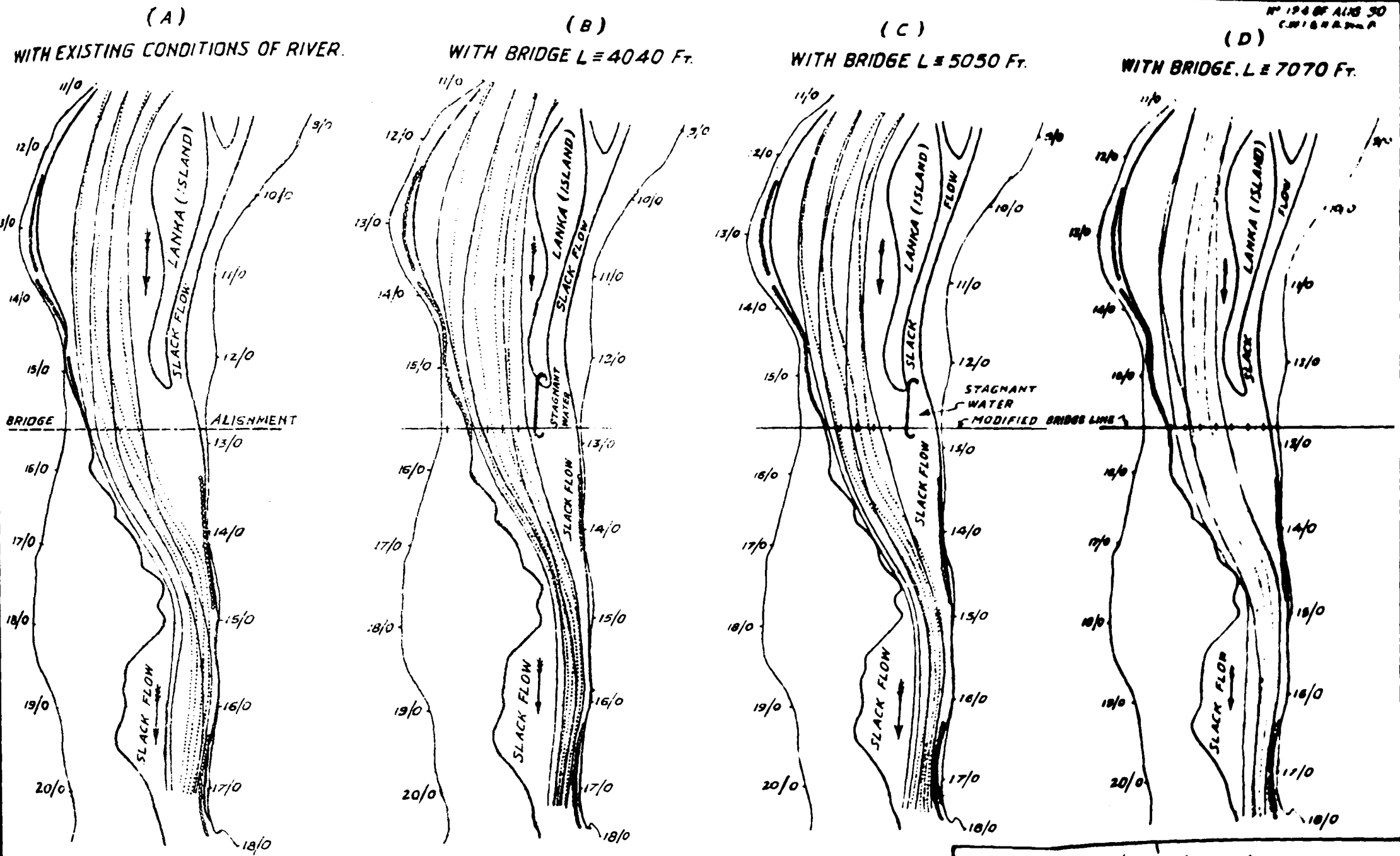
$$\text{Eq.:- } y = 10^{19} \times 5.42 (Q + 12.7)^{-9.42} \times e^{\frac{252}{Q + 12.7}}$$

*Q = DISCHARGE IN UNITS OF 10,000 CUSECS.*



FIGS. 43 (B) & (C)  
RISING & FALLING DISCHARGE  
CYCLE AS RUN  
IN  
THE MODEL





REFERENCE

WATER EDGE. ————

RIVER BANK. ————

PITCHING. ————

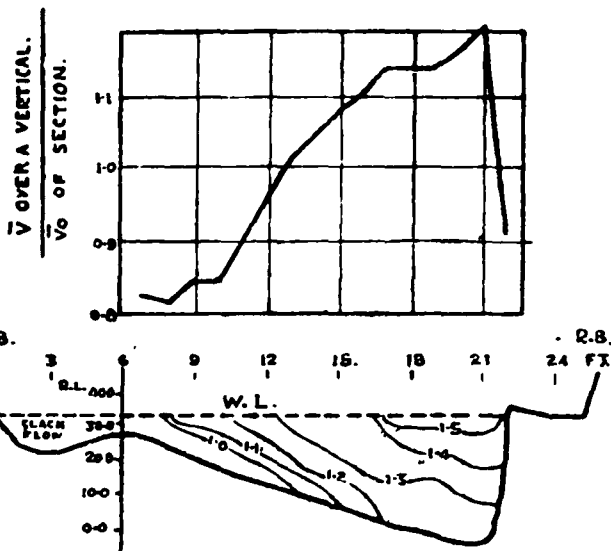
SURFACE LINES OF FLOW. ————

BED LINES OF FLOW. ————

FIG 44 (A TO D)  
ROAD BRIDGE ACROSS GAUTAMI RIVER  
1/330:1/50 MODEL OF GAUTAMI GODAVARI RIVER  
AT ALAMURU  
LINES OF FLOW  
Q = 7,70,000 CU SECS  
SCALE 1" = 1 MILE

(A)

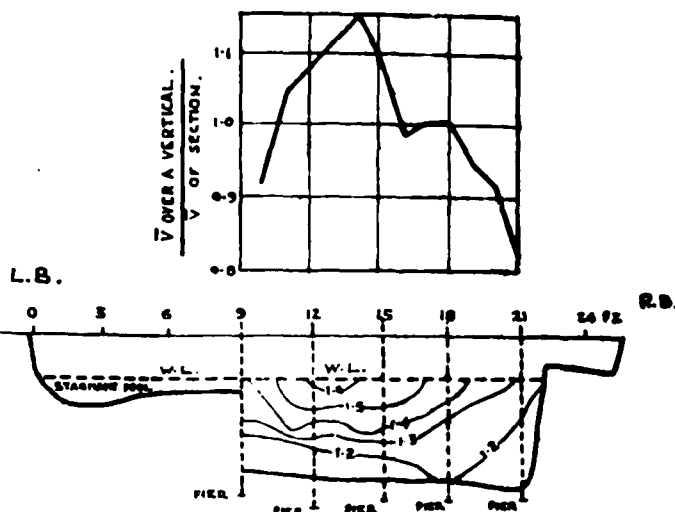
WITH EXISTING CONDITIONS OF RIVER



CROSS SECTION 660 FT. UPSTREAM OF BRIDGE.

(B)

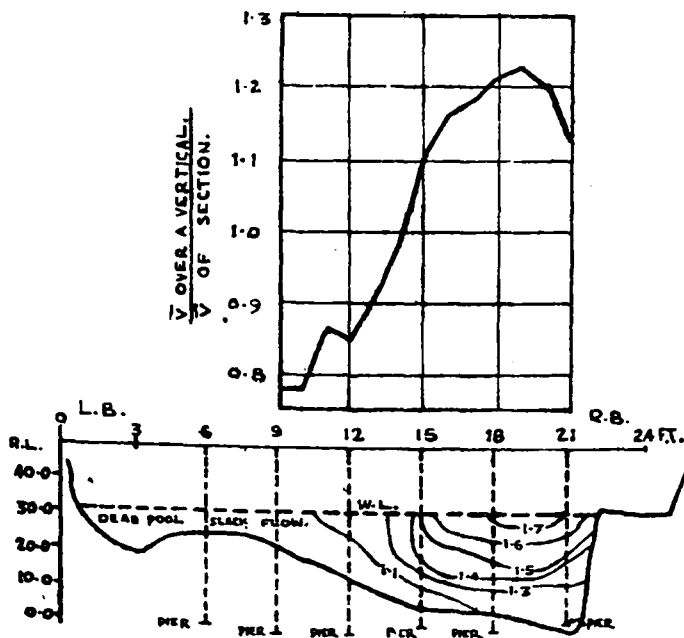
WITH BRIDGE = 4040 FEET.



CROSS SECTION 660 FT. UPSTREAM OF BRIDGE.

(C)

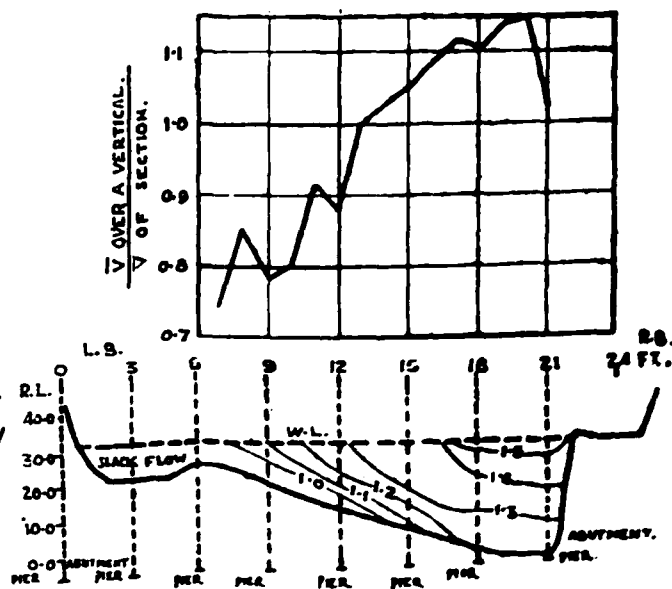
WITH BRIDGE = 5050 FEET.



CROSS SECTION 660 FT. U/S OF BRIDGE.

(D)

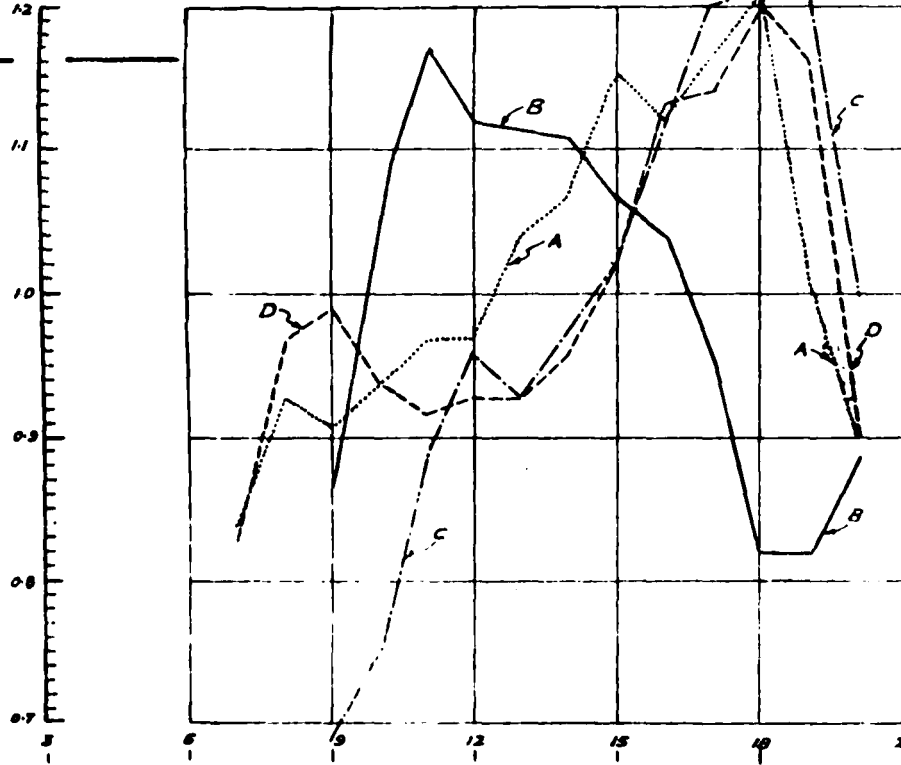
WITH BRIDGE = 7070 FEET.



CROSS SECTION 660 FT. U/S OF BRIDGE.

FIG-45 (A TO D)  
ROAD BRIDGE ACROSS GAUTAMI RIVER  
VELOCITY DISTRIBUTION CURVES & ISOVELS.  
 $Q = 7,79,000$  CS.

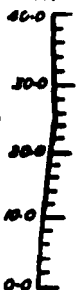
$\frac{\bar{V}}{V}$  OVER A VERTICAL  
OF SECTION



REFERENCES

- A. WITH EXISTING CONDITION OF RIVER ..... (dotted line)
- B. WITH BRIDGE = 4040 FEET ..... (solid line)
- C. WITH BRIDGE = 5050 FEET ..... (dashed line)
- D. WITH BRIDGE = 7070 FEET ..... (dash-dot line)

LEFT BANK  
R.L.



RIGHT BANK

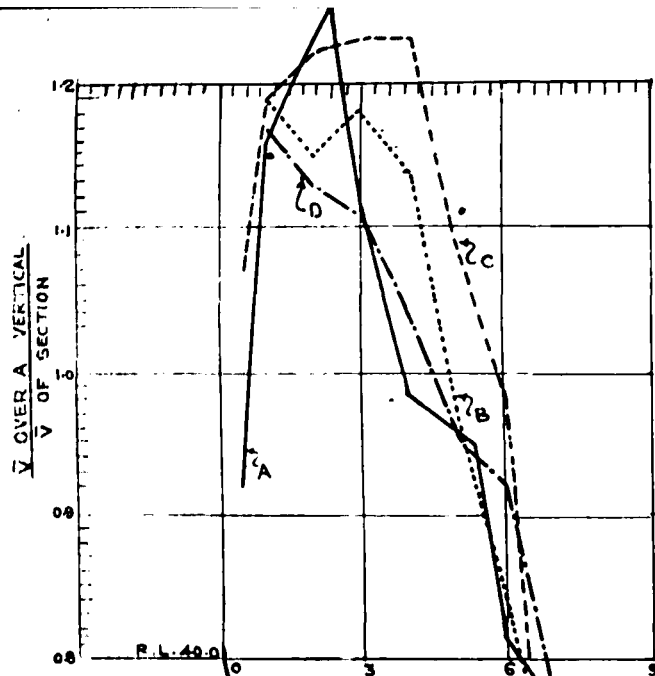
24 FT

CROSS SECTION 660 FT D/S OF BRIDGE

FIG A6 (A TO D)  
ROAD BRIDGE ACROSS GAUTAMI RIVER  
VELOCITY DISTRIBUTION CURVES

$Q \cong 7,70,000$  CUSECS.





- A WITH EXISTING CONDITION ———
- B. WITH BRIDGE = 4040 FEET ..... (dotted)
- C. WITH BRIDGE = 5050 FEET - - - - (dashed)
- D. WITH BRIDGE = 7070 FEET - . - . - (dash-dot)

CROSS SECTION  
 17% GAUTAMI LEFT BANK 20% GAUTAMI RIGHT BANK  
 (KAPILESHWARAPURAM SECTION)

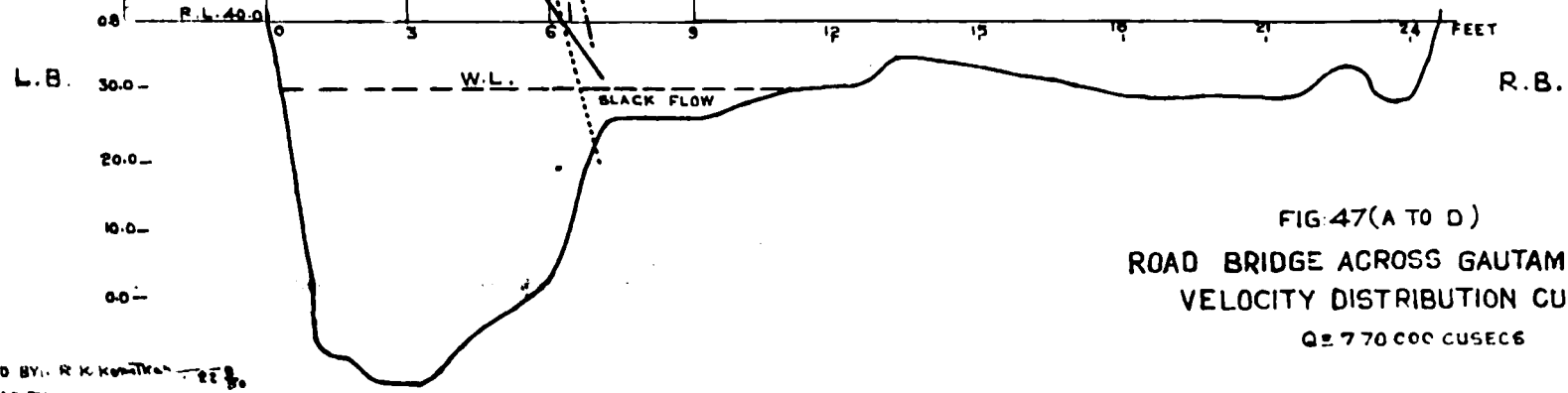
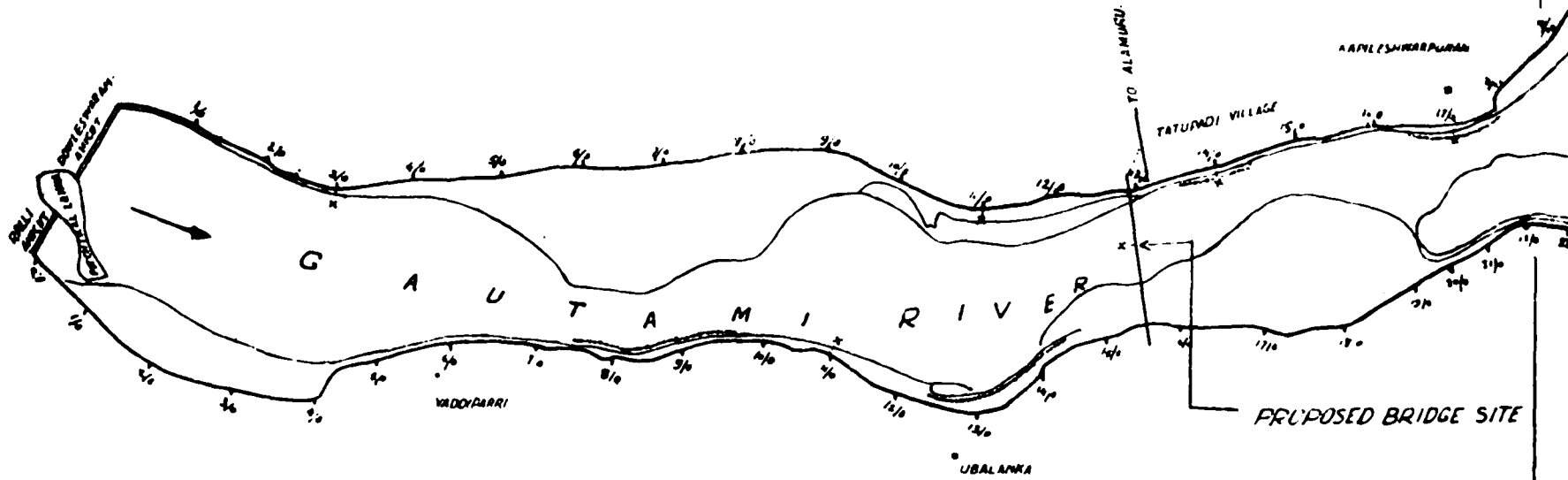


FIG:47(A TO D)  
 ROAD BRIDGE ACROSS GAUTAMI RIVER  
 VELOCITY DISTRIBUTION CURVES  
 Q = 770 C00 CUSECS

TRACED BY: R. K. K...  
 CHECKED BY: ...

FIG. 48  
SURVEY OF GAUTAMI RIVER (1945).  
SHOWING THE MODEL REACH.

SCALE 1/2" = 3 MILES.  
x - GAUGES IN MODEL.



8.3. *Linear Waterway through the Bridge*

8.3.1. With a 5,050 feet long bridge the left guide bund did not come into action at the dominant stage of the discharge. It had some effect in higher than dominant floods but as the duration of these floods is very small and frequency of occurrence also very low it would take a very long time for this bund to come into action. The flow conditions near the right bank became worse on account of the concentration of the whole river flow through the restricted waterway. The afflux recorded was  $\approx 2\frac{1}{2}$  inches and surface slope  $\approx 1.28$  feet/mile for dominant discharge. Fig. 44 (C).  
Fig. 46 (C).

8.3.2. With a 4,040 ft. long bridge the guide bund was effective in drawing the flow away from the right bank and the velocity distribution was more even through the bridge and the river was canalised. However due to the more restricted waterway as compared to that of 5,050 feet long bridge the conditions of attack on the left bank from mile 14/4 right down to Kapileshwarapuram mile 17/0 became slightly worse as the velocity near the bank increased by 8 per cent. The levels at the bridge site evened out and the depth of scour at the pier No. 2 from right—equivalent to pier 12 in the prototype—reached a maximum value of  $\approx$  R.L. 14.0 feet. The afflux recorded was  $\approx 6$  inch and surface slope  $\approx 1.34$  feet per mile for dominant discharge. Fig. 44 (B).  
Fig. 46 (C).

8.3.3. With a 7,070 ft. long bridge the guide bund was automatically dispensed with but the heavy attack on the right bank still persisted. All conditions were the same as exist in the river at present except that the waterway was restricted to the extent of the width of the piers. There was no afflux and surface slope was  $\approx 1.26$  feet per mile for dominant discharge.

8.3.4. *Lacey Co-efficient of Waterway*

For various bridge lengths and the river width the co-efficient 'C' in the Lacey formula  $P = C\sqrt{Q}$  works out to:—

(i) 4,040 feet Bridge,	$C = 3.86$
(ii) 5,050 feet Bridge,	$C = 4.81$
(iii) 7,070 feet Bridge,	$C = 6.73$
(iv) 8,600 feet Bridge,	$C = 8.18$

The length of waterway as per Lacey's formula works out to 2,700 ft. with  $C = 2.67$ . The value of  $C = 3.86$  is quite adequate and is corroborated by the experience in Orissa where for the bridges on Katjuri and Kuakhai rivers the value of 'C' is 3.1 and 4.12 respectively.

8.4. *Depth of Scour*

According to Lacey—  $D = 0.473 \left(\frac{Q}{f}\right)^{\frac{1}{3}}$   
where D = depth of scour below H. F. L.

$$\therefore D = 0.473 \left[\frac{11,00,000}{2}\right]^{\frac{1}{3}}$$

$$= 38.8 \text{ feet below H. F. L.}$$

$$\begin{aligned} \text{Maximum scour at nose of pier} &= 2D \\ &= 2 \times 38.8 \\ &= 77.6 \text{ feet.} \end{aligned}$$

According to the practice followed by the Railway Board as enunciated in Railway Board Tech. Bulletin No. 78, "Well Foundations for Bridges", it has been recommended that the foundations should be taken to a depth  $\frac{1}{3}$  greater than the calculated depth of scour below H.F.L., to allow for a sufficient grip.

$$\begin{aligned} \therefore \text{Depth of foundations} &= \frac{2}{3} \times 77.6 \\ &= 116 \text{ feet.} \end{aligned}$$

It is proposed to take the foundations 110 ft. below H. F. L., i.e. up to R. L. - 67'00.

To avoid the danger of settlement of pier columns founded on clay, as has occurred at the Railway Bridge at Rajahmundry, the foundations must be taken below R. L. - 60'0 and hence the uniform depth of foundations up to R. L. - 67'0 is considered adequate and safe.

#### 9. Discussion of Experiments and Experimental Results.

A study of the Gautami River course and history of the 'Sets' indicate that the river has been moving about, considerably within its marginal banks and there are a series of sets, dead or active, on either of its banks. At the bridge site the deep channel of the river was entirely along the left bank but within a period of 12 years has shifted completely to the right bank and has eroded the high marginal right bank to the extent of 1,000 ft. towards the outer flood banks. In its present conditions the river seems to be developing a new set or rather the Ubalanka set has travelled down to mileage 13/0-15/6 of the right bank. The model reproduced these conditions and the river bore heavily on its right bank within this reach. The most active set at Kapileshwarapuram was also reproduced in the model. Banks of alternate layers of clay and sand have always proved difficult of protection though there is no alternative but to protect them from slipping unless of course, the site is changed which under the terms of reference was not permissible.

[The protection of the right marginal bank is going to be rather expensive and troublesome. It is our considered opinion that there exists better sites such as the one upstream, 10/5 GLB to 13/0 GRB where there can be two well defined guide bunds of Bells' type and the entry conditions are more normal as against curved entry at the proposed site].

However to make the best under given conditions it is absolutely essential to protect the right marginal bank by stone pitching and an apron. The attack on the right bank will be decreased if the flow is pulled away from it and this can be achieved by means of a Bell bund on the left bank. Thus the right marginal bank, the left guide bund and the constricted waterway thus formed are all complementary units.

The main problem, therefore, is to determine to what extent the waterway can be safely constricted. This idea of constricting the waterway has arisen from the experience gained in bridging the North Indian Rivers. The model has shown that it is possible to constrict the Gautami Bridge on the same lines. In all tests with the constricted waterway the conditions of attack on the left bank downstream of the bridge worsened a little. The maximum worsening occurred at Kapileshwarapuram Section downstream when the constricted waterway was 4,040 feet. On the other hand during the test of full span bridge (7,070 feet) the severe conditions of attack on the right bank upstream of the bridge persisted.

Therefore, to conclude, the bridge with maximum permissible constriction (4,040 feet) as seen in the model requires protection measures for the left bank downstream, in particular for the Kapileshwarapuram set, which can be effectively done by projecting a repelling spur from the left bank upstream of Kapileshwarapuram set. On the other hand the bridge with full span-length (7,070 feet) requires heavy protection to the right bank upstream which again may be achieved by constructing a repelling spur upstream of the bridge on the right bank, but it should be noted that the design has no intrinsic merit of river control, which the 4,040 feet long bridge has.

Experiments for the design, position, length etc. of these spurs are in progress.

Appendix II gives the details of the design of the left guide bund and the revetment of the right marginal bank.

Fig. 48

Figs. 44  
(B, C, D)

Fig. 44 (D)

Fig. 49



75. *Bridge Dimensions in the Model*

= no. of piers  $\therefore n+1$  = no. of spans

$$\therefore \frac{w_p}{d_p} = \frac{w_m}{d_m} \text{ for each span}$$

$$\therefore \frac{w_p}{d_p (n_p + 1)} = \frac{w_m}{d_m (n_m + 1)}$$

$$\therefore n_m = \frac{d_p}{d_m} \cdot \frac{w_m}{w_p} \cdot (n_p + 1) - 1 = \frac{n_p + 1}{V. E.} - 1.$$

For a bridge of total length 5,050 feet, the spans proposed are 32 of 150'-0" clear each.

$$\therefore 31 + 1 = 6.6 (n_m + 1)$$

$$\therefore n_m = 4 \text{ (piers)}$$

= 5 spans.

Diameter of foundation well = 20 feet

" " pier column = 7 feet.

*Clear span in model*

(in prototype)

$$32 \times 150 \equiv \frac{32 \times 150}{330 \times 5} \text{ (in model)}$$

$$= 2'-11''$$

$$31 \times 7 \equiv \frac{31 \times 7}{4 \times 330} = 2'' \text{ diameter of column.}$$

$$31 \times 20 \equiv \frac{31 \times 20}{4 \times 330} = 5.64'' \text{ diameter of foundation well.}$$

Clear width between wells = 7'

$$\equiv \frac{7}{50}$$

= 1.68" (in model).

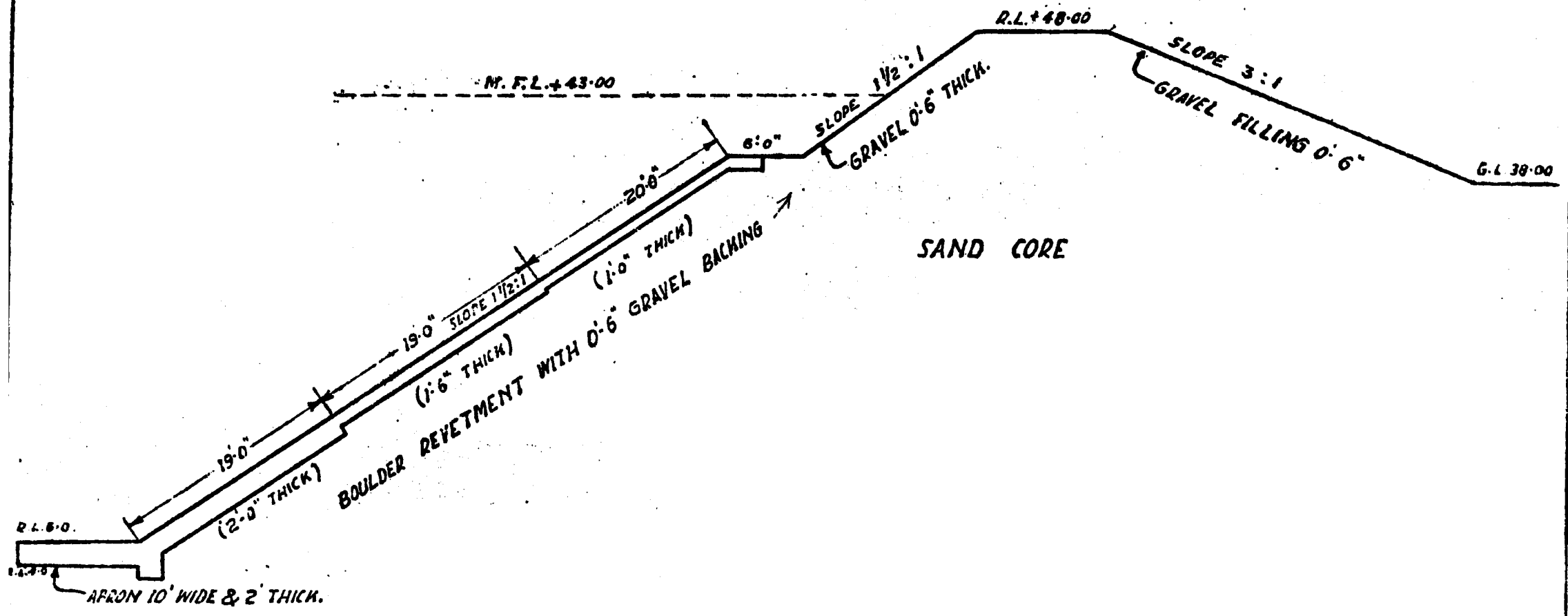
For simplicity these dimensions were adopted for all the three bridge lengths viz. 4,040, 5,050, 7,070 feet tested in the model.

Height of pier column =  $43/50 = 10.32''$  (in model).

Height of well =  $67/50 = 1'-4''$  ( " ).

**FIG. 49**  
**ROAD BRIDGE ACROSS GAUTAMI RIVER**  
**TYPICAL SECTION OF RIGHT MARGINAL GUIDE BANK REVETMENT**

**SCALE: 1" = 10'**



BY: BHINGARE . 7.2.50.

## APPENDIX II

## 1. River Protection Works

1.1. The river protection works as reproduced in the model consisted of the following:—

- (i) Marginal revetment on the right marginal bank;
- (ii) Cross bund at 1,000 ft. upstream of the right approach to prevent parallel currents along the right approach; and
- (iii) Bell's type guide bund on the left flank.

## 1.2. Marginal Revetment on the right margin:—

A typical section of the revetment is shown in fig. 49. The margin is made up of alternate layers of sand and clay and is standing more or less vertical or to a slope of  $\frac{1}{2}:1$  at the bridge site. It is known that such banks are difficult of protection. However a design was worked out based on the experience of protecting the existing river 'Sets'. The existing revetment on the right margin ends at chainage 4700' upstream of the right abutment. It is therefore necessary to join up the new revetment with the existing one along the margin curve, and should extend 500 ft. downstream of the right abutment.

Fig. 49.

1.3. *Cross Bund*:—The depth of water over the marginal spill lands is 6 to 8 feet in the highest floods. It is necessary to protect the right approach from the parallel currents which may erode the approach embankment. For this purpose a small cross bund of length 800 feet and 10 feet high and situated at chainage 1,000 feet from the right abutment was tested in the model and gave better results than the proposed long guide bund situated just above the marginal revetment and running parallel to it, shown dotted in fig. 50.

Fig. 50.

1.4. *Left Guide Bund*:—The plan of the left guide bund and its typical cross-section are shown in figs. 51 and 52. The design was found suitable for a 4,040 feet long bridge and is based on the principles enunciated by Sir Francis Spring.<sup>(1)</sup>

Figs. 51  
and 52.

(1) Technical Paper 153 of the Railway Board "River Training and control, the guide bank system" by Sir Francis Spring.



ANNUAL REPORT (TECHNICAL) 1949

TABLE I

*Frequencies of occurrence in flood season (June-October)*

Cusecs.	Year										All 10 years
	1927	1932	1935	1937	1940	1942	1944	1945	1946	1947	
0 to 5	10	13	13	13	5	11	11	12	4	12	102
5 to 10	0	1	0	2	8	0	3	2	3	4	23
10 to 15	2	27	3	0	11	3	8	7	4	6	71
15 to 20	28	14	36	21	15	30	22	18	34	22	250
20 to 25	12	6	12	19	5	11	11	4	4	10	94
25 to 30	12	9	3	12	6	5	13	14	10	8	92
30 to 35	7	1	6	5	6	6	3	12	2	4	53
35 to 40	5	4	0	4	1	4	4	3	5	5	35
40 to 45	1	0	3	1	4	2	1	3	6	4	25
45 to 50	0	1	3	1	0	2		1	2	0	10
50 to 55	1	1		1	1	0		1	0	1	6
55 to 60		1			3	1		0	2	0	7
60 to 65					0	1		1	0	2	4
65 to 70					1				1		2
70 to 75					1				1		2
75 to 80					0						0
80 to 85					0		1				1
85 to 90					1	1	1				3
90 to 95						1					1
95 to 100											
Total	78	78	73	78	78	78	78	78	78	78	780

*Godavari River* :—Table showing the frequency discharge distribution over different years.

*Note* :—Discharges on every alternate day were considered in working out the number of occurrence

FIG. 50  
ROAD BRIDGE ACROSS GAUTAMI RIVER  
1/330: 1/50 MODEL OF GAUTAMI GODAVARI RIVER AT ALAMURU  
SHOWING NEW BRIDGE ORIENTATION & PROTECTION WORKS  
ON THE TWO BANKS.  
1 SPAN IN MODEL  $\equiv$  6 1/2 SPANS IN PROTOTYPE.  
SCALE: 2" = 1 MILE.

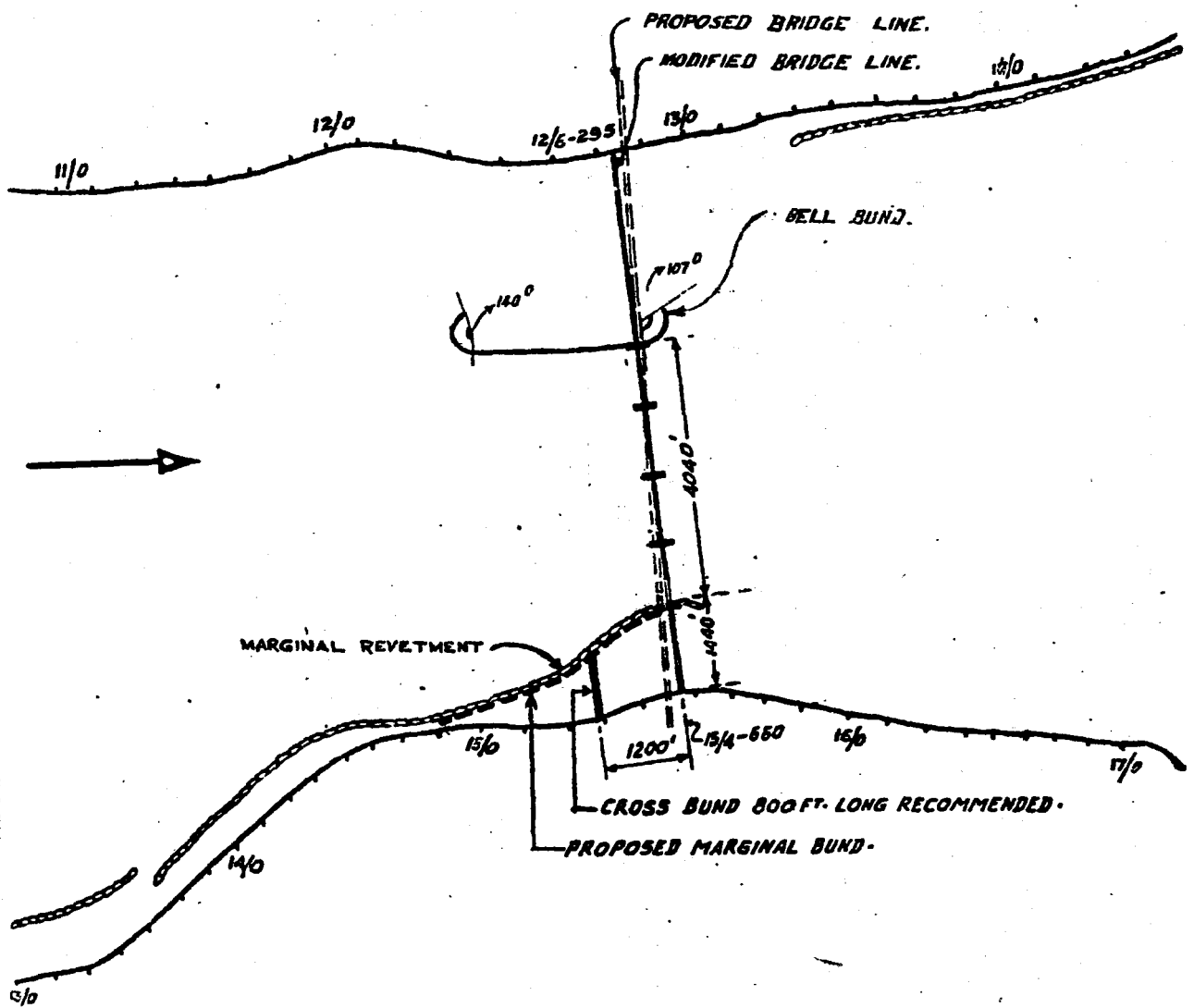


FIG. 51

ROAD BRIDGE ACROSS GAUTAMI RIVER

PLAN OF LEFT GUIDE BANK.

SCALE - 1:400

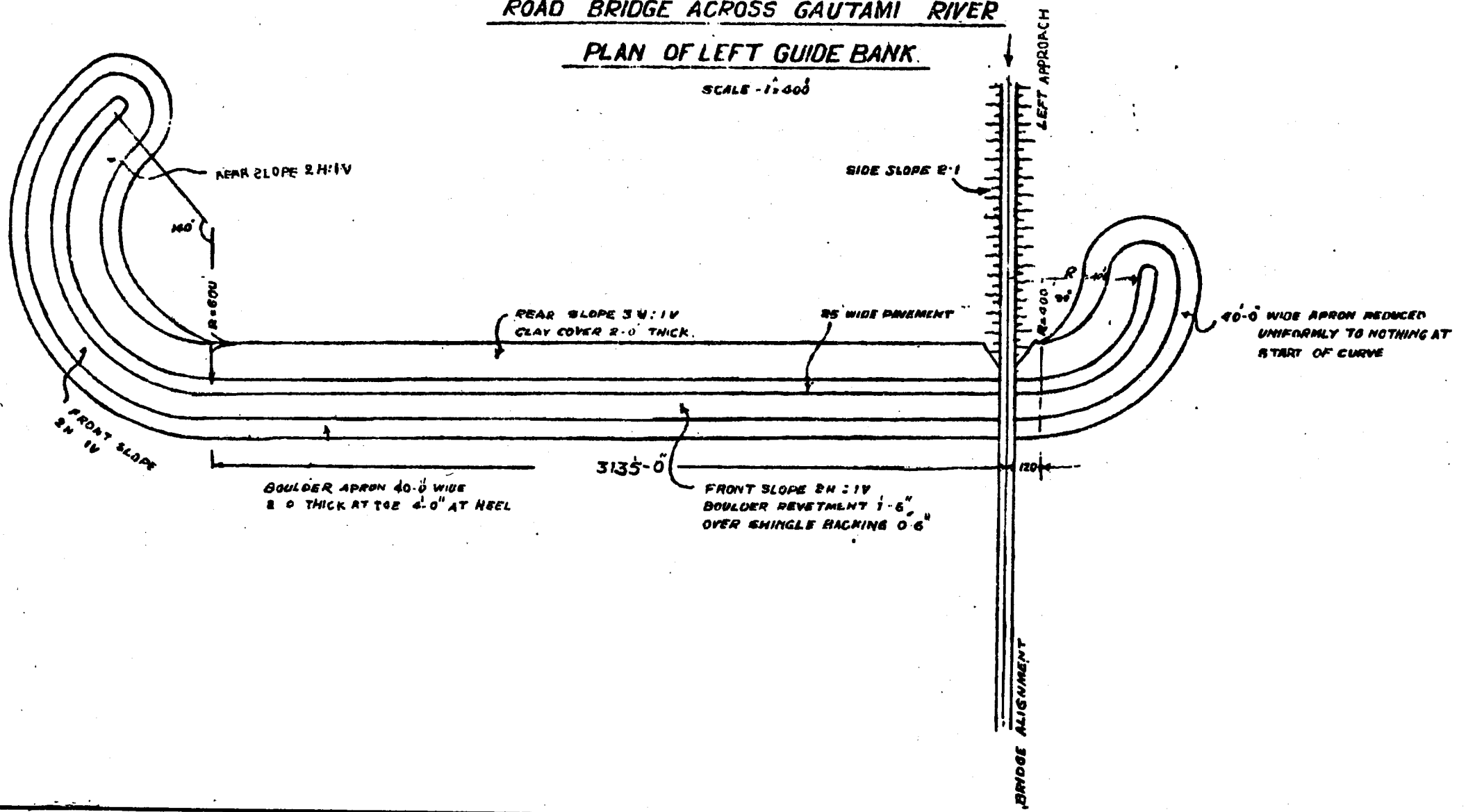
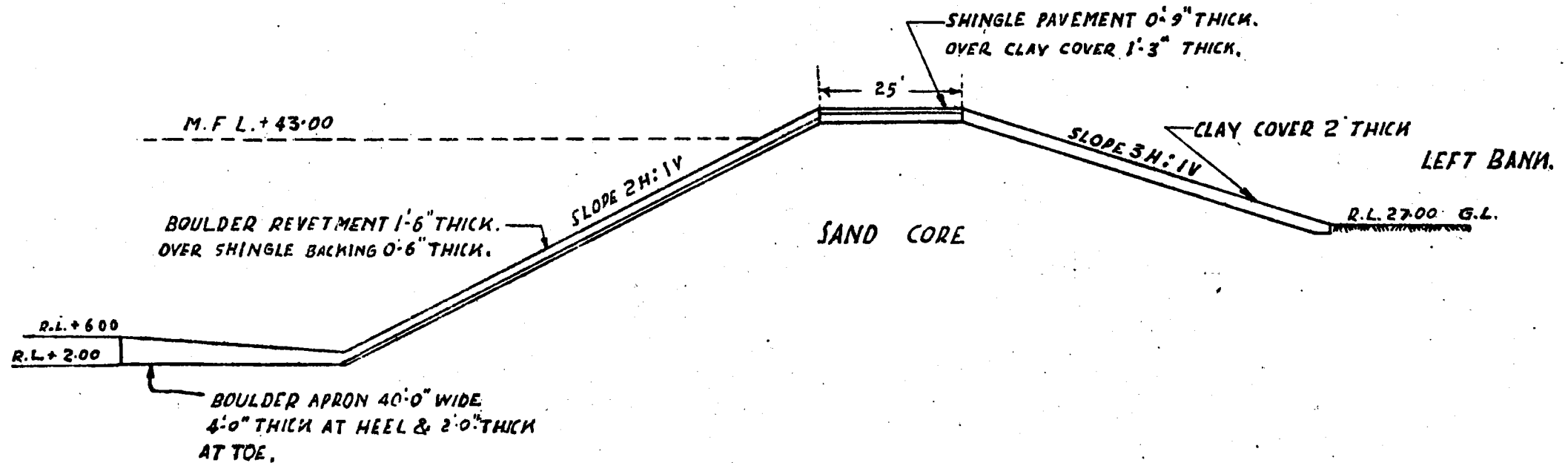


FIG. 52

ROAD BRIDGE ACROSS GAUTAMI RIVER  
TYPICAL SECTION OF GUIDE BUND ON LEFT ABUTMENT

SCALE:- 1" = 24'



BY:- THAKOR. 7-2-50.

1/330, 1/50 Scale Model of Gautami at Alamuru.

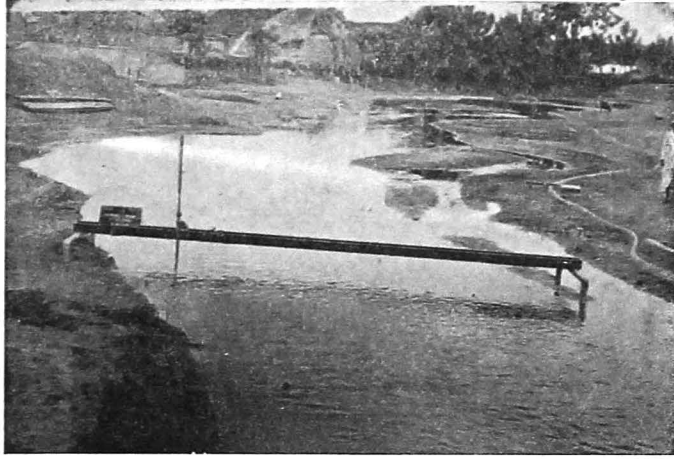


Photo No. 5659—Showing a general view of the model looking upstream. The girder indicates the bridge site and the discharge stage is slightly higher than the dominant.

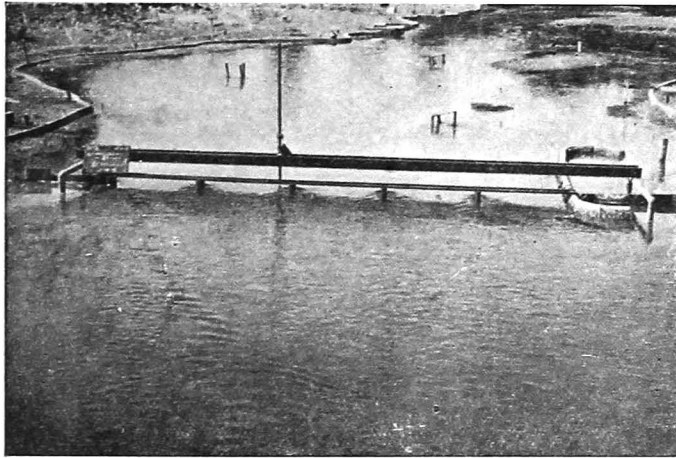


Photo No. 5676—(Looking upstream) Showing full floods passing through a bridge 33 spans and length 5.050 ft. Note the guide bund on the right.

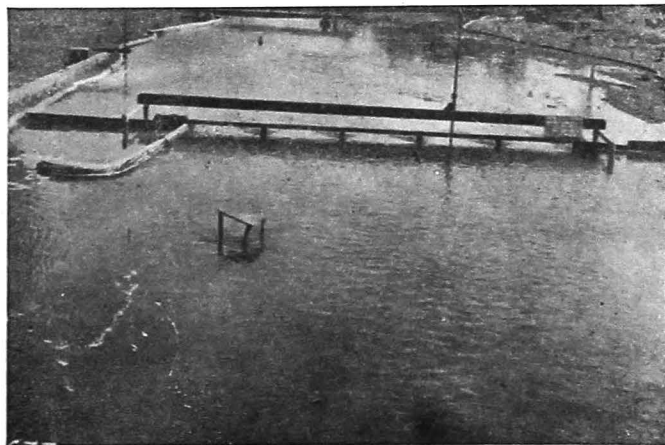


Photo No. 5677—The same as Photo No. 5676 looking downstream.

ROAD BRIDGE ACROSS GAUTAMI-GODAVARI RIVER  
(MADRAS STATE)

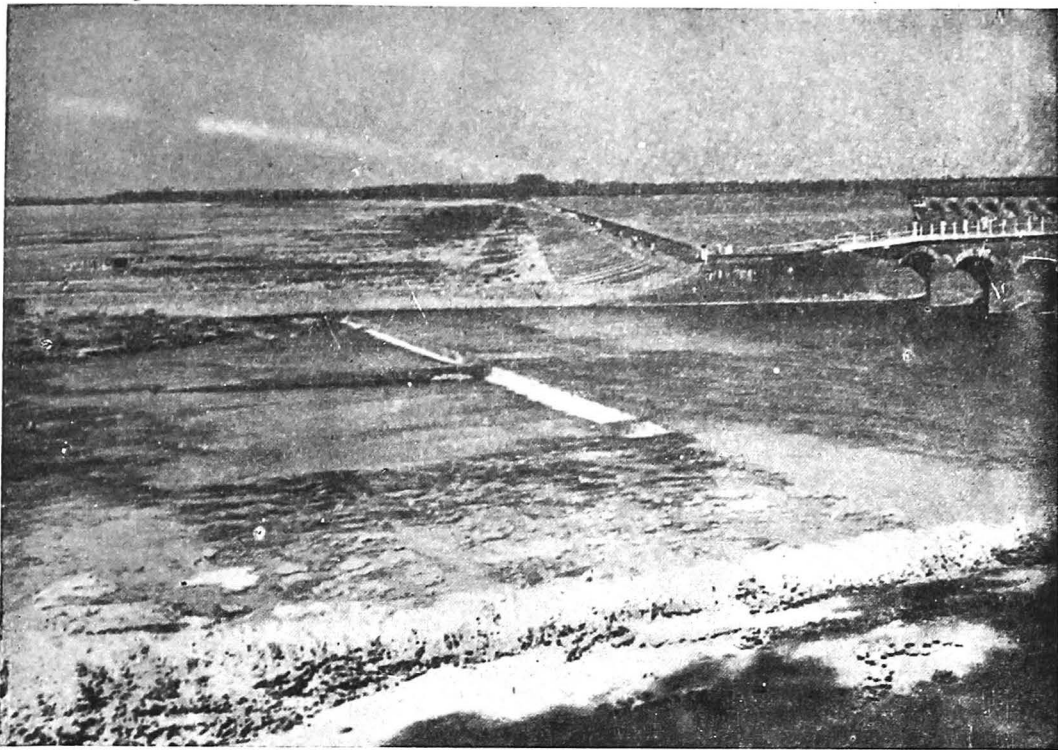


Photo No 5582—Anicut across Godavari at Dowleshwaram from left bank looking towards scouring sluices.

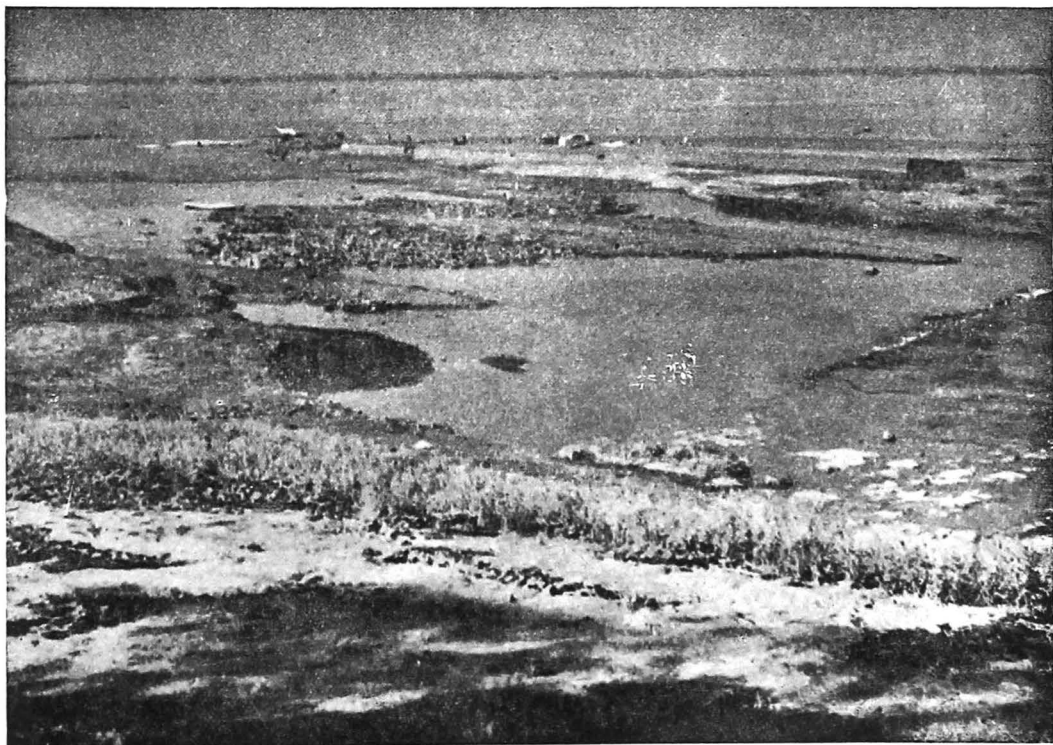


Photo No. 5581—Dowleshwaram Anicut from left bank showing bed scour downstream of scouring sluices.



Photo No. 5577—Gautami river 200' Ft. above right abutment showing steep scoured bank looking downstream.



Photo No.5580—Proposed site of Gautami Bridge (right abutment) flaggcd pole on centre line on Bridge.



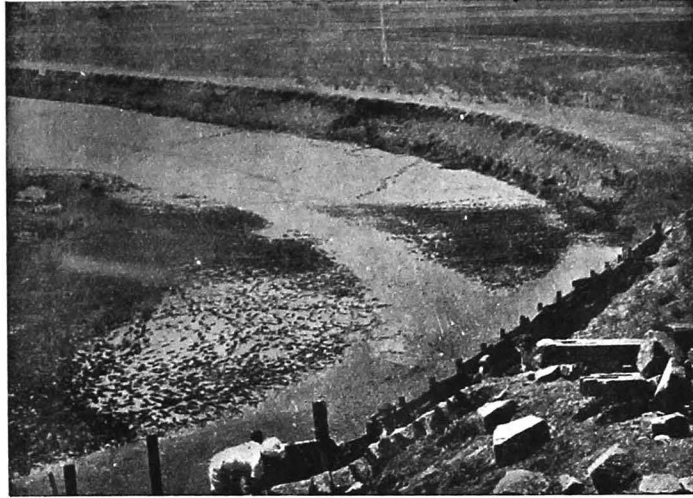


Photo No. 5578—Gautami River 200 feet above right abutment showing steep scoured bank looking upstream.

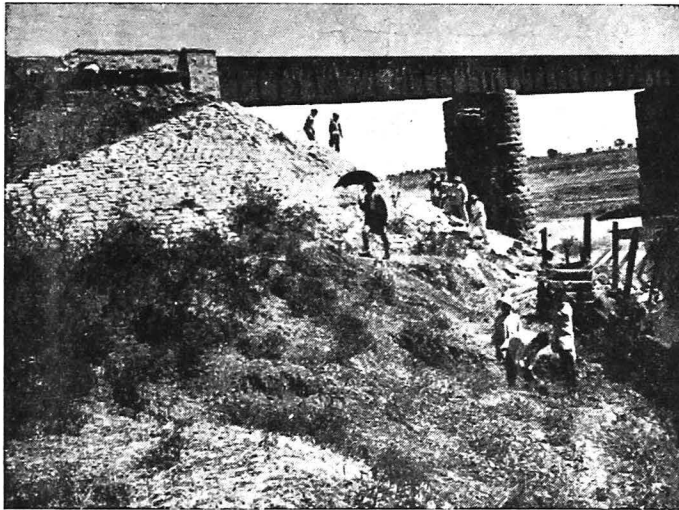


Photo No. 5574—Gautami River right bank at Ubalanka mile 14/5 looking downstream.

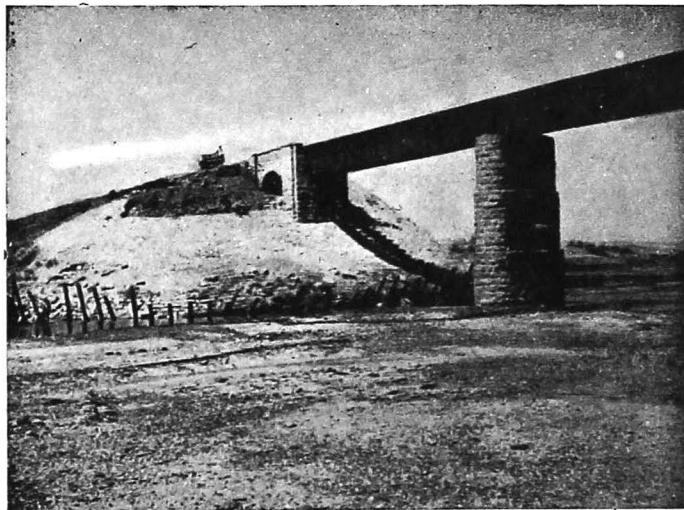




5526—Showing Bennihalla left bank upstream of the bridge. Bank erosion and embayment can be clearly seen.



5527—Abutment apron under repairs.



5531—A distant view of apron round left abutment with steel sheet piling in foreground. Apron is under repairs.

## 8. BRIDGE No. 10 ACROSS RADIHALLA, BENNIHALLA RIVERS ON HUBLI-GADAG LINE, M. S. M. RAILWAY

### 1. Introduction

The Chief Engineer, M. & S. M. Railway referred this problem to this Research Station in October, 1945. The bridge, 9 spans of 60 feet each, carrying single line of metre-gauge traffic is situated exactly over the confluence of the two rivers Radihalla and Bennihalla at mile 87/22 on Hubli-Gadag Line. As can be seen (from Fig. 53) the bridge is located on a curve where the left bank has eroded very badly and has formed a large embayment just upstream of the left abutment. The bank erosion has resulted in developing an attack on the left abutment itself which rests on rather shallow foundations. The problem was, therefore, to prevent erosion of the left bank and outflanking of the bridge behind the left abutment. Photo 5534.  
Fig. 53 and  
Photo 5526.

### 2. The Bridge

The bridge consists of 9 spans of 60 feet plate girders resting on solid masonry piers. The masonry piers rest on deep foundations, the exact levels of which are not known. The nature of pier foundations is not known although it is conjectured that the piers may be resting on well foundations. To ascertain the depth, the founds were probed along the face sufficiently deep without touching the footings. The bridge is situated exactly over the confluence of the two rivers, Radihalla and Bennihalla, on a sharp bend—a bad location. Photo 5528.  
Fig. 54.  
Photo 5531.

### 3. West Abutment (Left bank, Hubli end)

The abutment has been badly damaged by the attack of the two rivers. It carries a land arch through it and is protected upstream and downstream by an apron of boulder pitching. After the 1948 floods the pitching had slipped and the sheet piles driven at its base had bent out. The river Bennihalla has formed a big embayment upstream of this abutment by eroding severely. Photo 5581.  
Fig. 55.  
Photo 5526.

### 4. River conditions and Hydraulic Data

4.1. The embayed portion of the bank consists of layers of sand lying underneath 4 to 5 feet of black cotton soil, which is a feature causing caving action. The bed of Bennihalla consists of black cotton soil which forms a treacherous mass when wet. The bed of Radihalla consists of nodules of kunker mixed with fine sand.

4.2. About 1,500 ft. upstream of the Bridge a small cut-off channel has naturally formed leading from Bennihalla to Radihalla. It has an average width of 80 ft. and carried about 4 per cent of Bennihalla discharge into Radihalla. In fact this natural phenomenon has shown a partial solution of the problem of reducing the discharge intensity of Bennihalla. Figure 60 also shows a cut made previously further downstream but this has now silted up as it was located on the wrong side—silted side—of a curve. Photo 5530.

4.3. *River Discharge*:—As in the majority of cases the hydraulic data available was very meagre. In the absence of any hydrograph data the maximum river discharge was worked out by the following methods:—

4.3.1. Ryves' formula—  $Q = C M^{\frac{3}{4}}$

$$\therefore Q = 560 \times 130^{\frac{3}{4}} \quad (C=560) \\ M=130 \text{ sq. miles.} \\ = 22,200 \text{ cusecs for Bennihalla.}$$

$$Q = 560 \times 124^{\frac{3}{4}} \quad M=124 \text{ sq. miles.} \\ = 21,000 \text{ cusecs for Radihalla.}$$

4.3.2. Meander formula—

$$M_1 = 30\sqrt{Q} \quad M_1 = \text{Meander length.}$$

$$M_1 = 3,600 \text{ feet Bennihalla.}$$

$$M_1 = 3,060 \text{ feet Radihalla.}$$

$$\therefore Q = 14,400 \text{ cusecs, Bennihalla.}$$

$$Q = 10,400 \text{ cusecs, Radihalla.}$$

In the absence of any maximum flood observations, it was considered safer to adopt the higher discharges worked out by Ryves' formula. Thus the total discharge to be passed through the bridge worked out to 43,200 cusecs and the Lacey width for this discharge worked out to

$$W = 2.67\sqrt{Q}.$$

$$= 2.67\sqrt{43,200}.$$

= 560 feet as against the actual width of 540 feet through the bridge.

4.4. *River Slope*:—The river surface slope of 1.2 feet per mile as ascertained during the maximum floods of 1938 which attained the H. F. L., so far recorded, of R. L. 1919-20 was used for the maximum discharge and slopes for various discharges were allowed to develop in the model.

4.5. *River Bed Material*:—Sand samples from the two rivers, on analysis, gave the following 'm' in mm. and Lacey  $f_d$  ( $f_d = 1.76\sqrt{m}$ ) values. Size distribution curves are given in figs. 56 (A) and 56(B).

S. No.	Particulars of location.	m in mm.	$f_d$ .	Specific gravity.	Soluble salts percentage.
1	Bed sample of Bennihalla u/s of the confluence.	0.0226	0.265	2.565	10.45
2	Bed sample of Radihalla u/s of the confluence ...	0.037	0.398	2.59	10.876
3	Bed sample of common stream at bridge site ...	0.058	0.423	2.720	4.064
4	Bed sample 2 ft. below bed surface at pier No. 2.	0.195	0.704	2.62	6.87
5	Sample 6" below sample 4	1.98	2.52	2.59	8.46
6	Sample 6" below sample 5	1.36	2.04	2.72	7.62

These samples contained very high percentage of soluble salts. In the model the 'm' values for bed material used in Bennihalla and Radihalla were 0.18 mm. and 0.34 mm. respectively.

5. *Experimental Procedure*

5.1. Originally it was proposed to protect the bridge by constructing a left guide bank; but this was found to be totally unsuitable on inspection of the site, as the clay banks and the sticky material on the bed would never have allowed the apron to function satisfactorily. Three other solutions were tried in the model and the results obtained were compared with those of the existing conditions of the rivers.

Photo 5850. (i) with a 300 ft. wide cut along the alignment of the existing natural cut 1,500 ft. upstream of the bridge.

Photo 5841. (ii) with an attracting spur 200 ft. long pointing downstream at an angle of 30° with the projecting bank and located 400 ft. downstream of the bridge, on the right bank.

(iii) with a combination of the above-mentioned 300 ft. wide cut and the attracting spur.

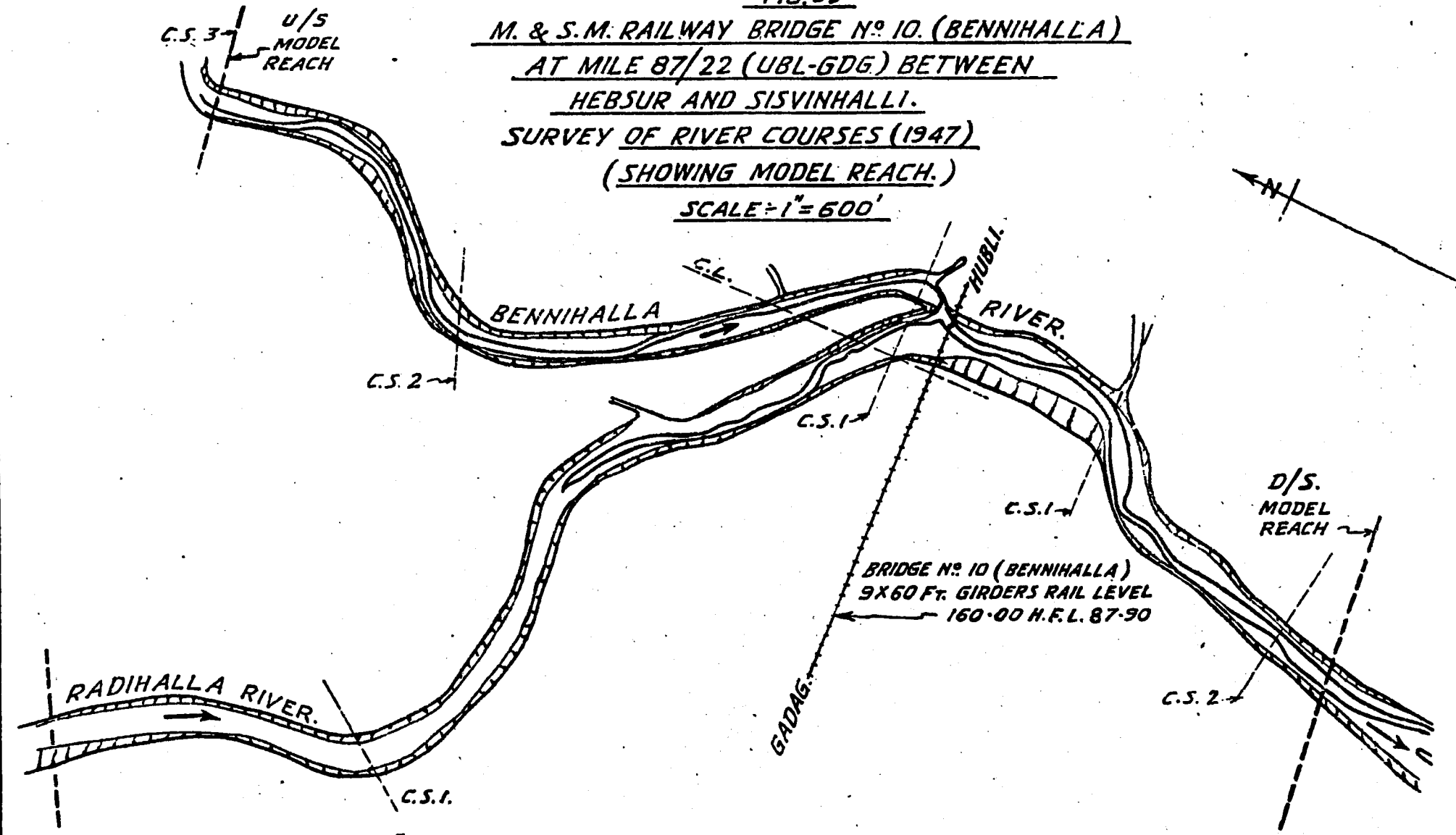
Appx. I. Model scales are shown in Appendix I.

FIG. 53

M. & S.M. RAILWAY BRIDGE NO. 10. (BENNIHALLA)  
AT MILE 87/22 (UBL-GDG.) BETWEEN  
HEBSUR AND SISVINHALLI.

SURVEY OF RIVER COURSES (1947)  
(SHOWING MODEL REACH.)

SCALE = 1" = 600'



TRACED BY: SAPKAL. CHECKED BY: [Signature]



Photo No. 5573—Gautami right bank showing steep clay bank (pitched with stone.) mile 15/2.

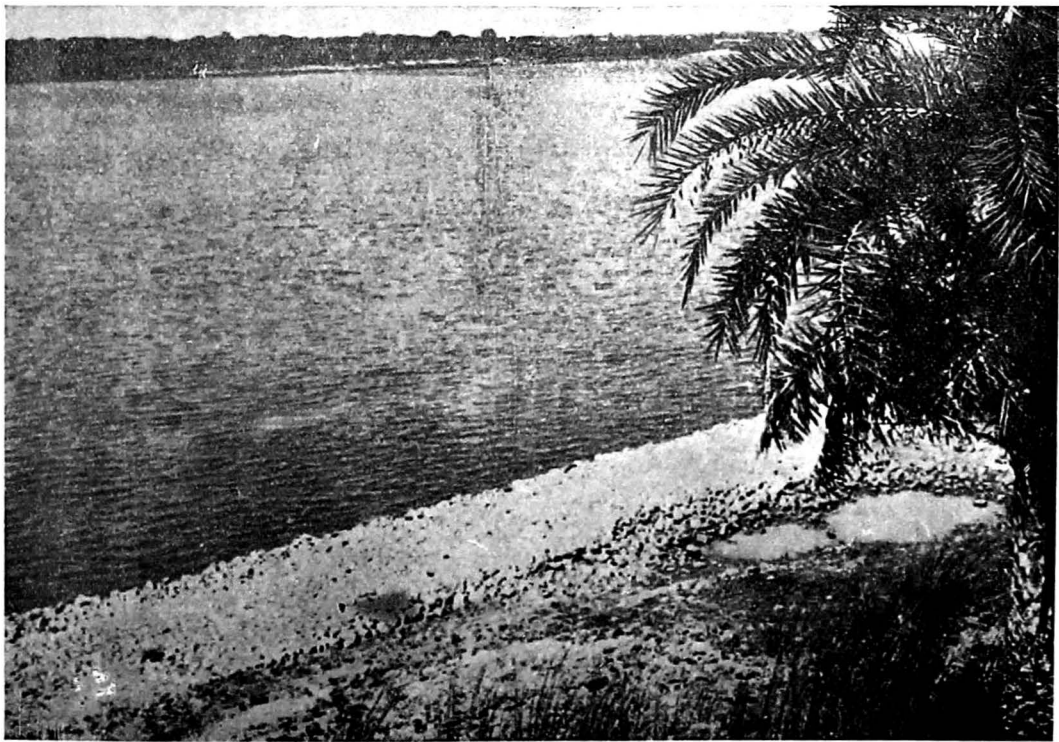


Photo No. 5569—Gautami at Kapileshwarapuram showing river attack on the left bank.

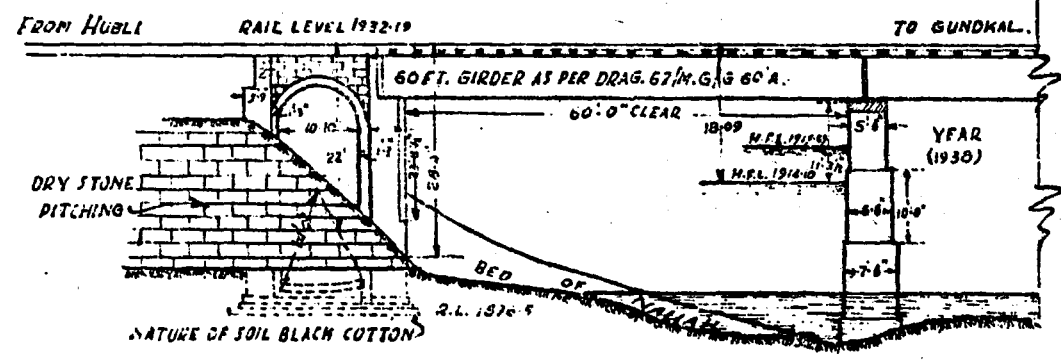
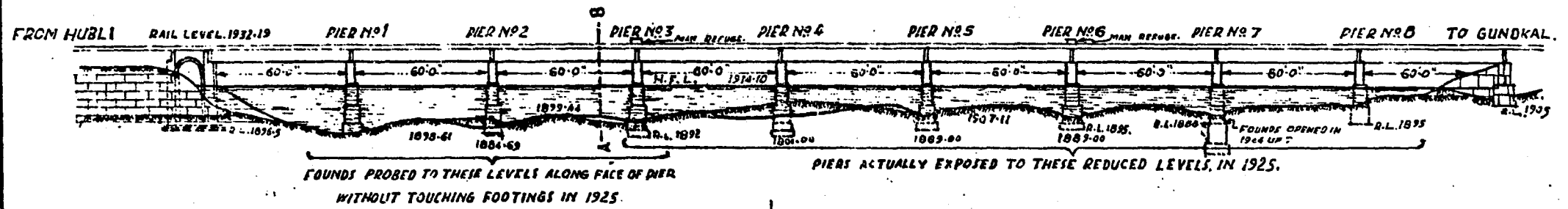
N<sup>o</sup> 7771 OF FEB. 1950.  
 C. W. I. & N. R. STATION.  
 POONA.

M. & S. M. RLY.  
BRIDGE N<sup>o</sup> 10 AT MILE 87/22 HUBLI GADAG  
(BENNI HALLA)

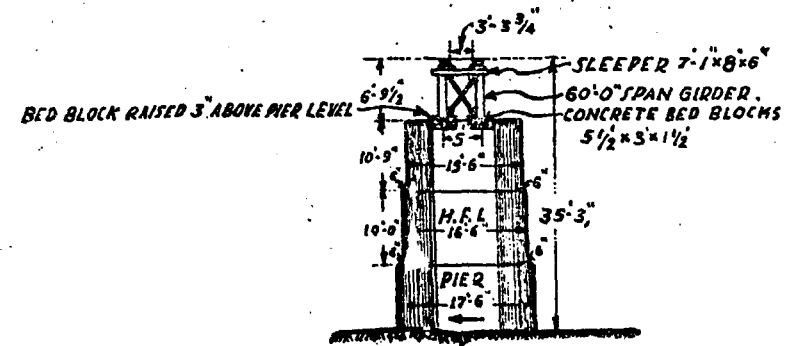
FIG:54

COMPLETION DRAWING AS REGISTERED IN 1923

SCALE:- 60' = 1"



ELEVATION  
SCALE:- 1" = 24'



SECTION ON A B.  
SCALE:- 1" = 24'  
PIER N<sup>o</sup> 3

BY BHINGARE. 21230.

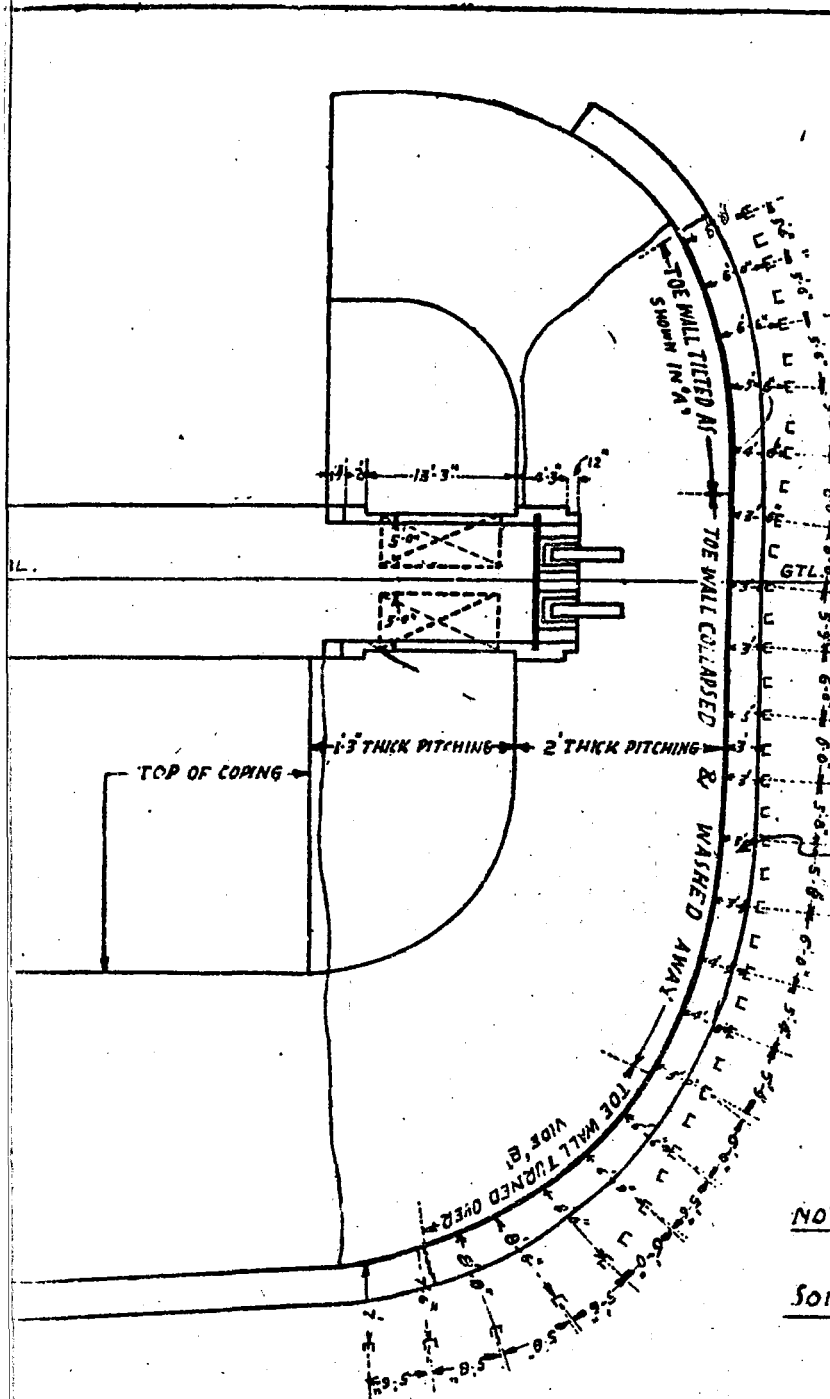
COMPARED BY: [Signature]

FIG. 55

M. S. M. PLY.

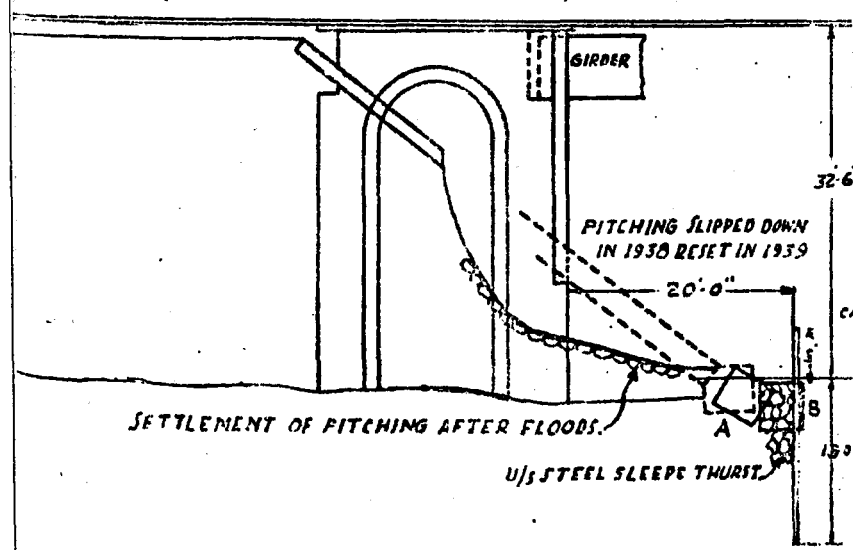
PLAN OF WEST ABUTMENT  
OF BRIDGE NO. 10  
SHOWING THE IMPROVEMENTS  
DONE FOR PITCHING  
DURING 1939 AFTER FLOODS  
IN SEPTEMBER 1938

SCALE: - 1" = 16'



NOTE: TOE OF EXISTING PITCHING  
MARKED IN THICK LINE.  
SOIL: - BLACK COTTON SOIL.

SECTION



SETTLEMENT OF PITCHING AFTER FLOODS.

U/S STEEL SLEEPS THURST

NATURE OF SIDE IS BLACKCOTTON





## BRIDGE NO. 10 ACROSS RADIHALLA-BENNIHALLA

5.2. In all these experiments no definite discharge cycle could be run as no hydraulic data was available. The dominant discharges for the two rivers which worked out to 16,000 cusecs in Bennihalla and 12,000 cusecs in Radihalla were run in the model until it attained stability. It was found that the bank erosion and the attack on the left abutment were worst at  $\frac{2}{3}$  the maximum flood discharges on falling floods. Therefore, the experiments were conducted for these dominant discharges only. Observations included lines of flow, velocities at cross-sections 200' upstream and 200 ft. downstream of the Bridge and bed contours after each experiment.

### 6. Experiments

6.1. *With existing conditions of the river:*—The lines of flow indicated heavy attack on the Bennihalla left bank upstream of the Bridge and the left abutment. A whirlpool was formed on the spill margins just upstream of the left approach embankment. There was very negligible flow through the 3 spans on the right and almost all the discharge was concentrated in the remaining 6 spans, the heaviest concentration being through the first two spans near the left abutment as shown by the velocity distribution diagrams. The existing natural cut carried 640 cusecs, i. e. only 4 per cent of the Bennihalla dominant discharge. Fig. 57 (A).  
Figs. 58 (A)  
and 59 (A).

6.2. *With a 300 ft. wide cut along the alignment of the existing natural cut:*—The lines of flow indicated the opening of the 3 right spans and reducing the flow concentration on the left side at the bridge site. The flow was centralised through the bridge and the velocity distribution curve showed moderation of peak velocities by 18 per cent. The cut carried 15 per cent of Bennihalla dominant discharge and the intensity of the whirlpool upstream of the left approach was not to be seen at all. Nose of the existing island between the two was protected with stone pitching. Fig. 57 (B).  
Figs. 58 (B)  
and 59 (B).

6.3. *With an attracting spur pointing downstream, 200' long at an angle of 30° with the projected bank situated 400' downstream of the bridge on the right bank:*—In this case the spur came into action by pulling the flow towards the right bank and easing the concentration on the left side. There was scour around the nose of the spur, the toe of which will have to be provided with a falling apron. The plan and typical section are shown in fig. 61. The velocity distribution curve at the cross-section 200 ft. upstream of the bridge however showed that the flow was not centralised through the bridge, the concentration remaining equal on the left and right of the central spans. The moderation of peak velocity was 10 per cent. Fig. 57 (C).  
Figs. 58 (C.)  
and 59 (C).

6.4. *With a 300' wide cut and the attracting spur downstream as stated in paras. 6.2 and 6.3:*—The solution gave the best results and had essentially acted as a combination of the merits of the two devices. The lines of flow indicated the elimination of whirlpool action upstream of the left abutment and the flow was evenly distributed in all spans as seen from the velocity distribution curves. Fig. 57 (D).  
Figs. 58 (D)  
and 59 (D).

### 7. Discussion of Experiments and Experimental Results

7.1. Frequently nature shows the way of solving a problem created by itself. In this case it did so during the floods of 1947 by making a breach leading from Bennihalla to Radihalla and that too at a location slightly downstream on the outside of a concave bend, a situation very favourable for its prolonged existence. Actually an artificial cut was made some time back but it silted up on account of its faulty location. It was seen from the results of experiments that this cut if widened to about 300 feet would have a carrying capacity of 15 per cent of the dominant discharge of Bennihalla. The by-passing of floods from Bennihalla, which is at present pressing heavily on the left bank, would Fig. 60.

result in pulling the flow away from the left bank and thus reducing the intensity of attack on the left abutment. Nearly the same results were obtained by constructing an attracting spur downstream of the bridge on the right bank. Therefore to accelerate the favourable results of the 300 ft. wide cut it is considered that the same can be done by combining the cut and the spur

7.2. It may be mentioned that the solution of a guide bank of Bell's type located on the left bank was ruled out from the beginning. It was noticed during inspection of the site in February 1949 that the banks consist of alternate layers of sand, clay and such materials are known to be very difficult of protection, much more so along a bend. The slipping of the pitching at the site indicated that the guide bund would not function well and the falling apron would not launch.

Therefore, to conclude, a cut 300 feet wide along the alignment of the existing natural cut and an attracting spur on the right bank downstream of the Bridge are recommended.

#### APPENDIX I

##### 1. The model and model scales

Fig. 53. 1.1. The model was reproduced from C. S. 3 of Bennihalla upstream and C.S. 2 of Radihalla upstream to C.S. 2 of Bennihalla downstream of the bridge. The model reach is shown in fig. 53.

1.2. *Length scale*:—On considerations of suitable span and availability of space the length scale was fixed as 1:100.

1.3. *Discharge scale*

$$\begin{aligned} Q &= L^2 \\ &= 100 \times 100 \\ &= 10,000. \end{aligned}$$

1.4. *Depth scale*:—Based on the experience with models of incised rivers such as the Kshipra River at Ujjain which had similar characteristics of the two Rivers Radihalla and Bennihalla, the vertical exaggeration was fixed as 1:4. Therefore, the depth scale worked out 1:25.

1.5. *Slope scale ratio*:—Slope exaggeration and slope scale ratio were fixed as S.E. = V.E. = 4.0.

##### 1.6. Bridge Dimensions in model

For dynamic similarity—

$$\frac{w_p}{d_p} = \frac{w_m}{d_m} \text{ for each span.}$$

If  $n_p$  = No. of piers in prototype = 8

$n_p + 1$  = No. of spans in prototype.

$$w_p = \frac{w_p}{n_p + 1} w_m = \frac{w_m}{n_m + 1}$$

$$\therefore \frac{w_p}{d_p(n_p + 1)} = \frac{w_m}{d_m(n_m + 1)}$$

$$\therefore n_m = \frac{w_m d_p}{w_p d_m} (n_p + 1) - 1$$

$$= \frac{n_p + 1}{V.E.} - 1$$

$$= \frac{8 + 1}{4} - 1$$

$$= 1.25 \text{ or say, 2 nos. of piers in the model.}$$

FIG.57(A).

BRIDGE No 10, RADIHALLA-BENNIHALLA RIVERS

$\frac{1}{100}, \frac{1}{25}$  SCALE MODEL

LINES OF FLOW WITH EXISTING CONDITIONS

WITH EXISTING NATURAL CUT

$Q \approx 16,000$  CUSECS (B)

$\approx 12,000$  CUSECS (R)

SCALE  $\bar{1} = 600$  FT.

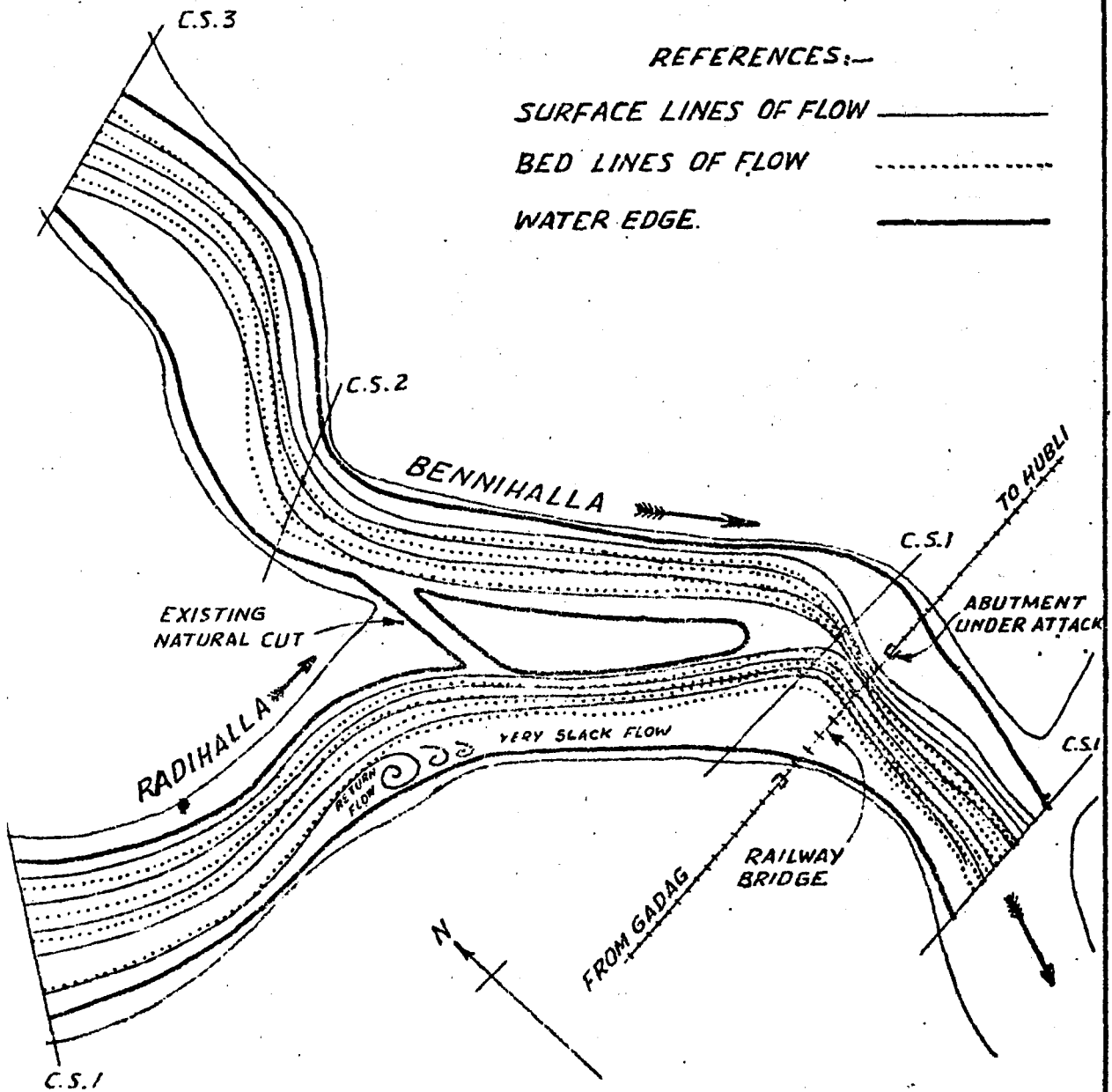


FIG. 57(B)

BRIDGE Nº 10, RADIHALLA-BENNIHALLA RIVERS

$\frac{1}{100}, \frac{1}{25}$  SCALE MODEL

LINES OF FLOW

WITH 300 FT. WIDE CUT

$Q \approx 16,000$  CUSECS (B)

$\approx 12,000$  CUSECS (R)

SCALE:  $\bar{1} = 600'$

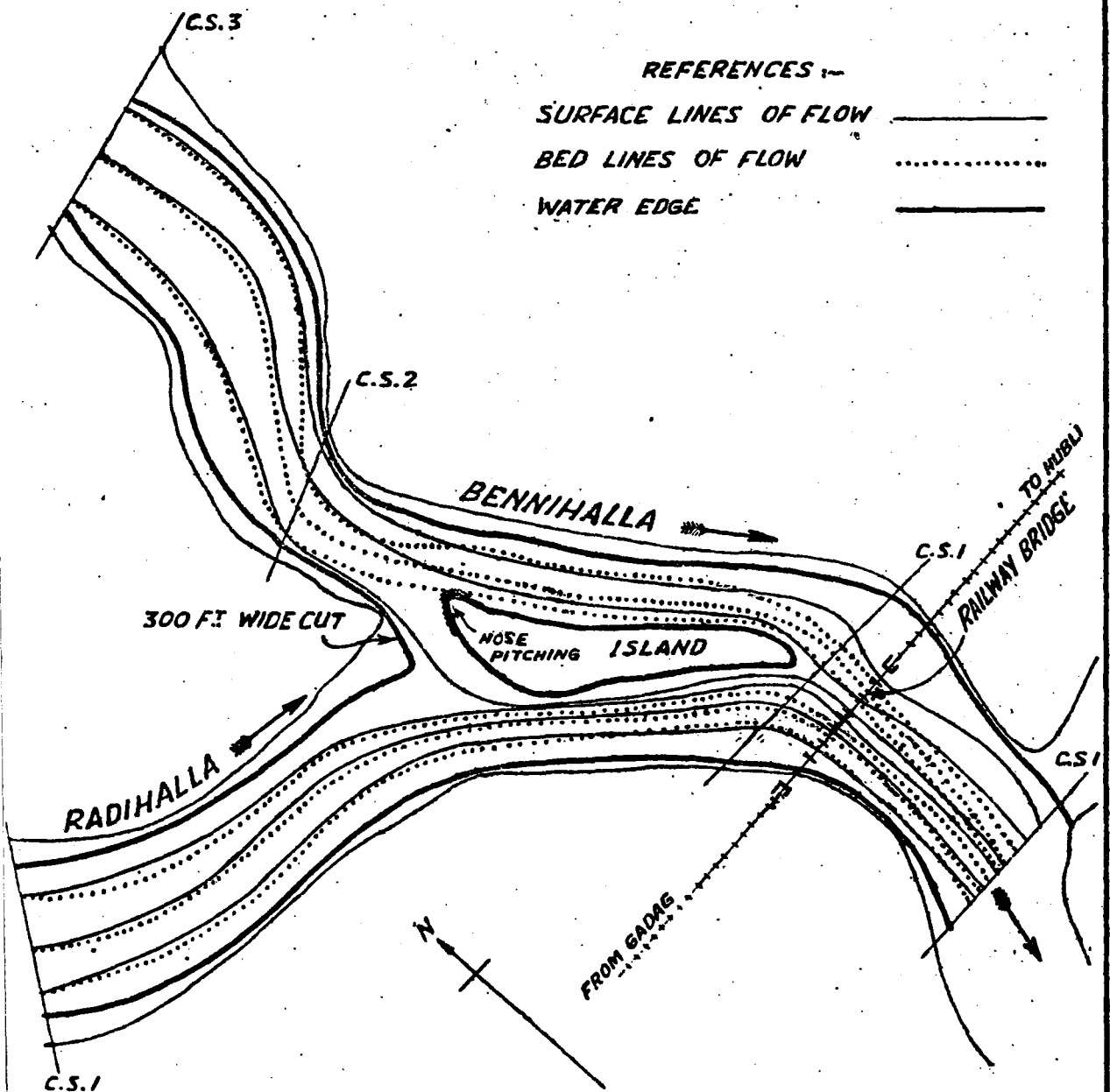


FIG.57(C)

BRIDGE No 10, RADIHALLA-BENNIHALLA RIVERS

$\frac{1}{100}, \frac{1}{25}$  SCALE MODEL

LINES OF FLOW

WITH ATTRACTING SPUR D/S

$Q = 16,000$  CUSECS (B)

$= 12,000$  CUSECS (R)

SCALE:  $1'' = 600'$

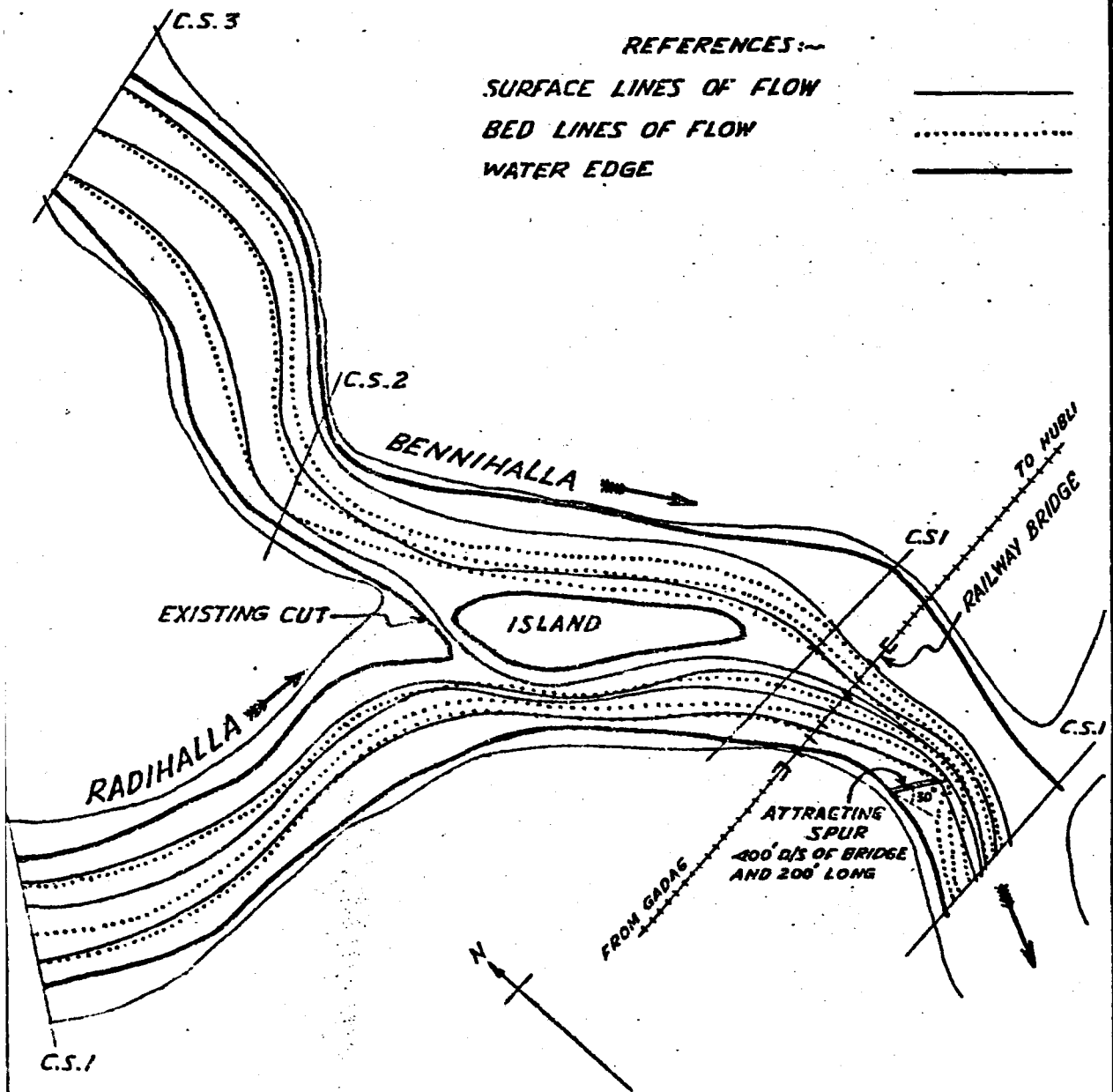


FIG. 57(D)

BRIDGE NO. 10, RADIHALLA-BENNIHALLA RIVERS

$\frac{1}{100}, \frac{1}{25}$  SCALE MODEL

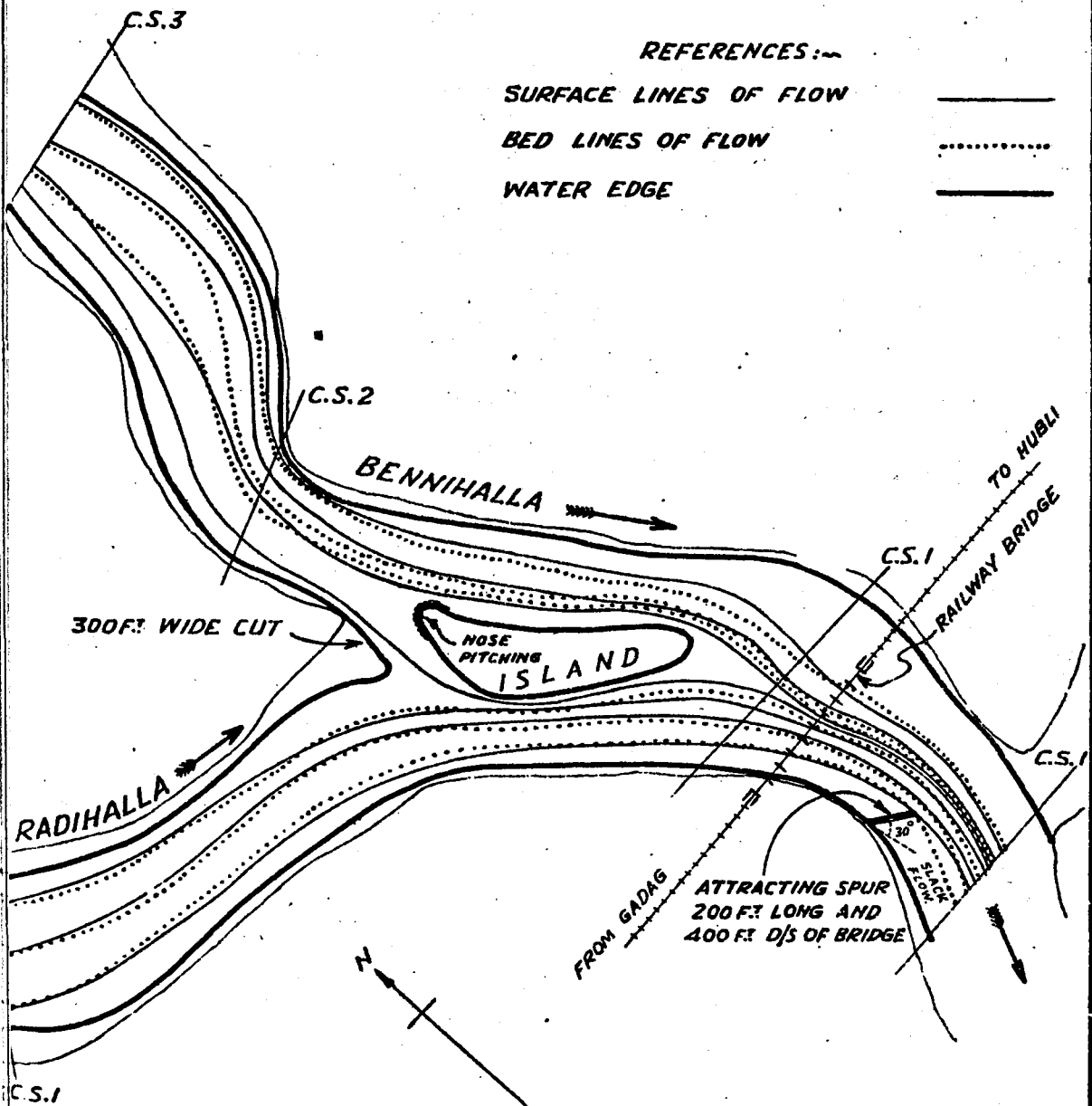
LINES OF FLOW WITH ATTRACTING SPUR

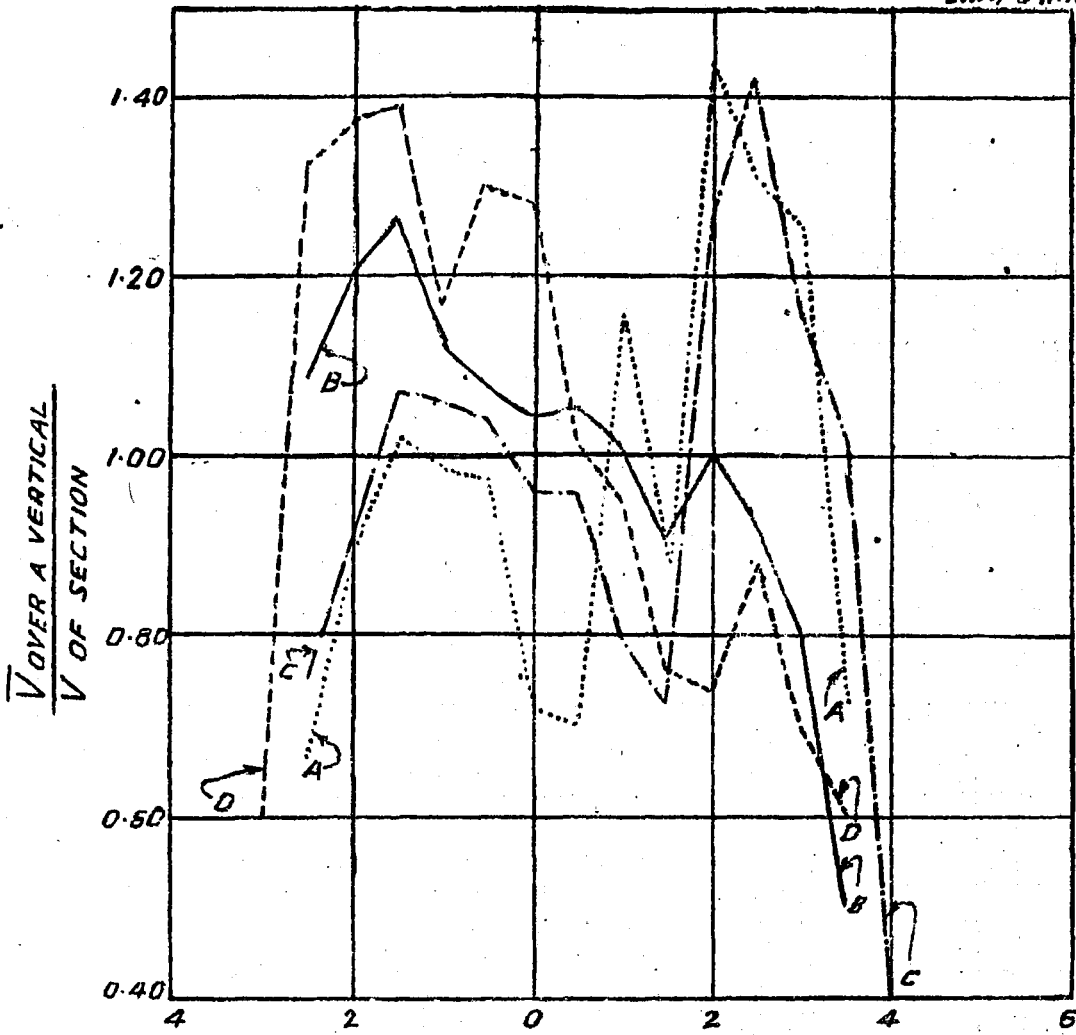
D/S OF BRIDGE AND 300 FT. WIDE CUT.

$Q \equiv \underline{16,000 \text{ CUSECS (B)}}$

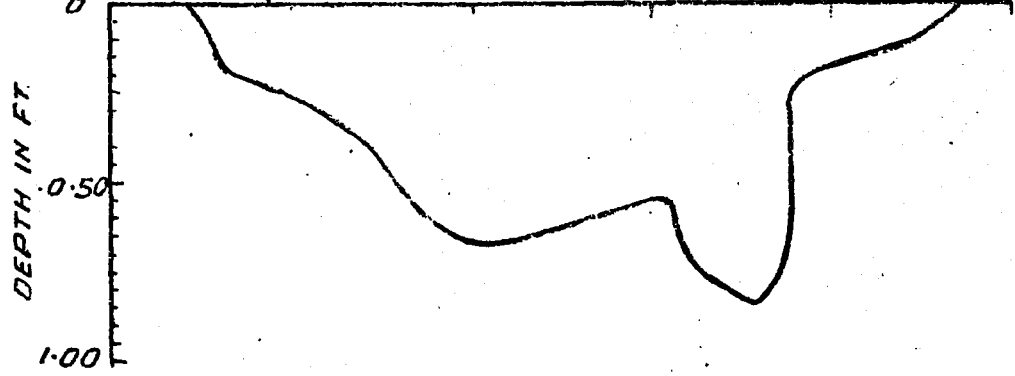
$\equiv \underline{12,000 \text{ CUSECS (R)}}$

SCALE  $\bar{1}'' = 600'$





RIGHT BANK R.L. 1912.00 2 0 2 4 6 FT. LEFT BANK



- REFERENCES.
- A---EXPERIMENT WITH EXISTING CONDITIONS WITH THE NATURAL CUT.....
  - B---EXPERIMENT WITH 300 WIDE CUT-----
  - C---EXPERIMENT WITH ATTRACTING SPUR D/S-----
  - D---EXPERIMENT WITH 300 WIDE CUT AND ATTRACTING SPUR D/S.-----

FIG. 58(A TO D)  
BRIDGE NO. 10, RADHALLA-BENMIHALLA RIVERS  
HUBLI-GADAG LINE, M.&S.M. RAILWAY  
VELOCITY DISTRIBUTION CURVE  
CROSS SECTION 200 FT. U/S OF BRIDGE  
Q = 16,000 CU SECS. (B)  
= 12,000 CU SECS (R)

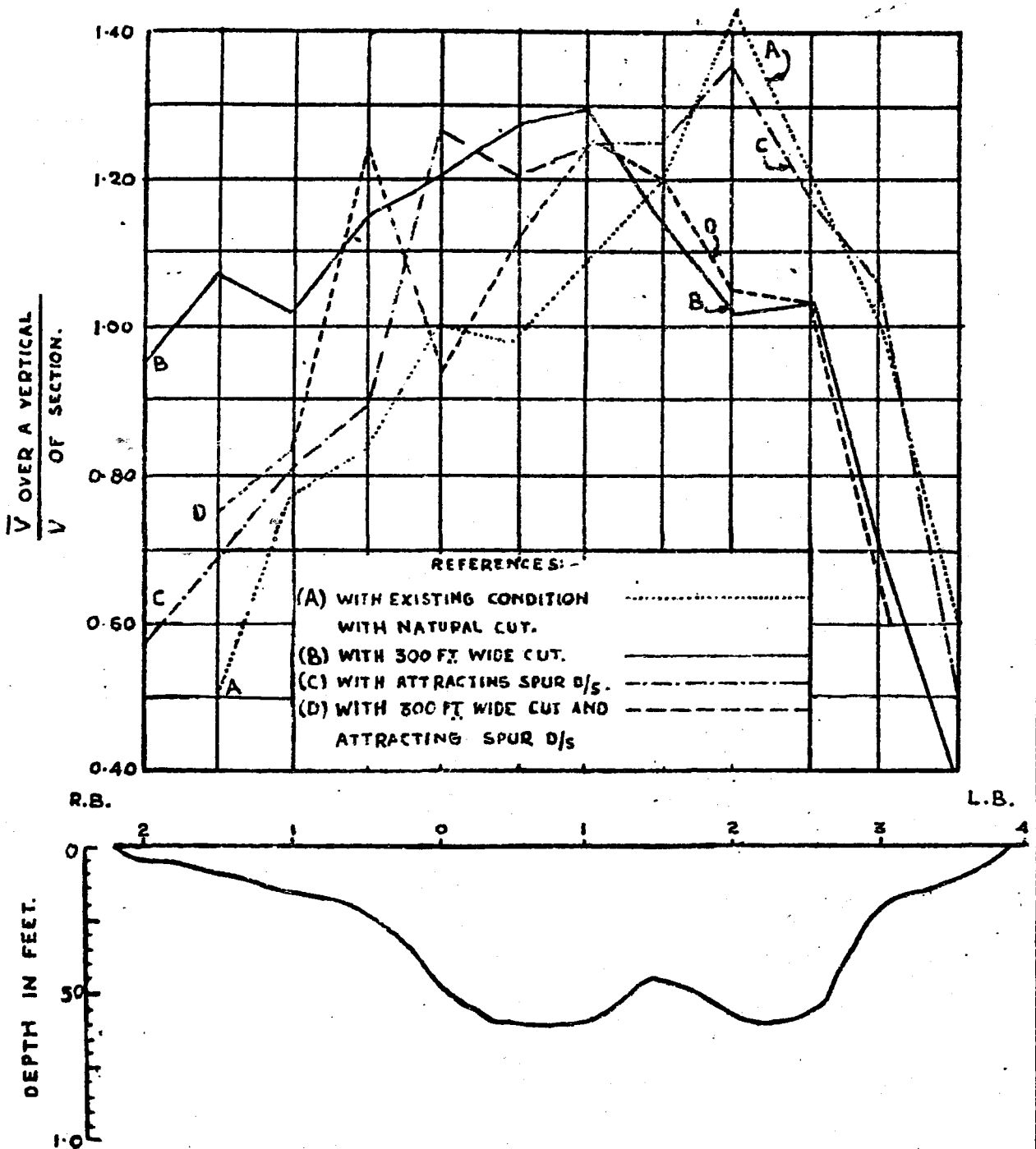
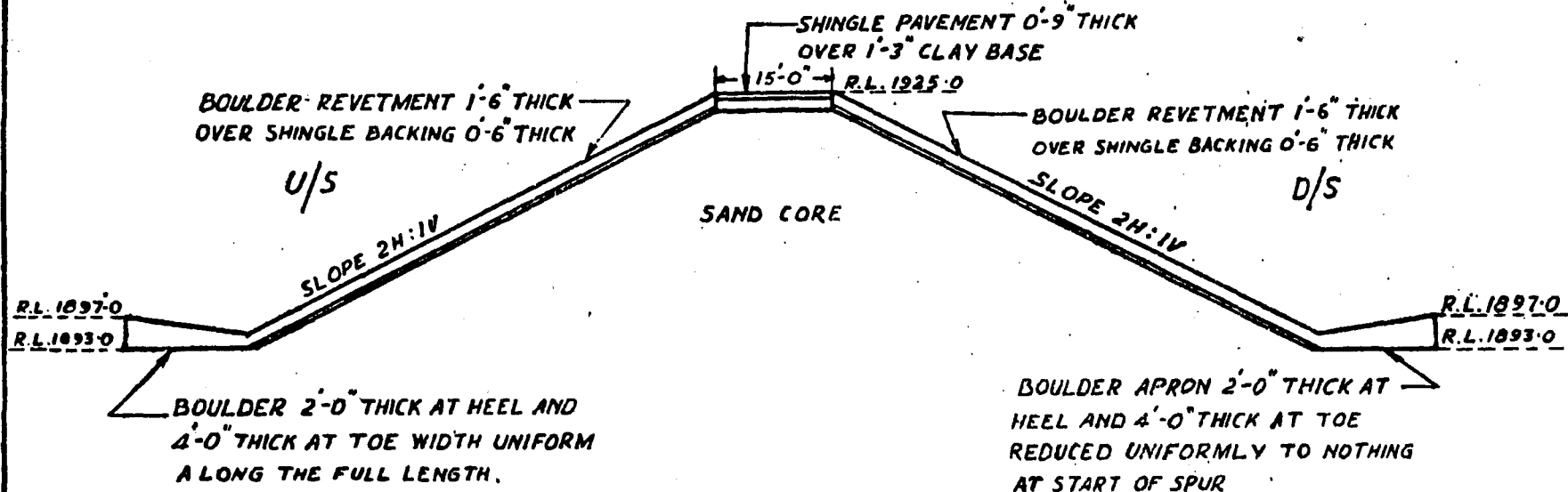


FIG. 59 (A TO D)  
BRIDGE NO. 10, RADIHALLA-BENNIHALLA RIVERS  
HUBLI-GADAG LINE M. S. M. RLY  
VELOCITY DISTRIBUTION CURVE  
CROSS SECTION 200' D/S OF BRIDGE.  
Q = 16,000 CUSECS. (B)  
= 12,000 CUSECS (R)

COMPARED BY:-  
BY-BNINARE







BOULDER APRON 2'-0" THICK AT HEEL AND 4'-0" THICK AT TOE REDUCED UNIFORMLY TO NOTHING AT START OF SPUR

SKETCH PLAN OF THE SPUR.  
(NOT TO SCALE)

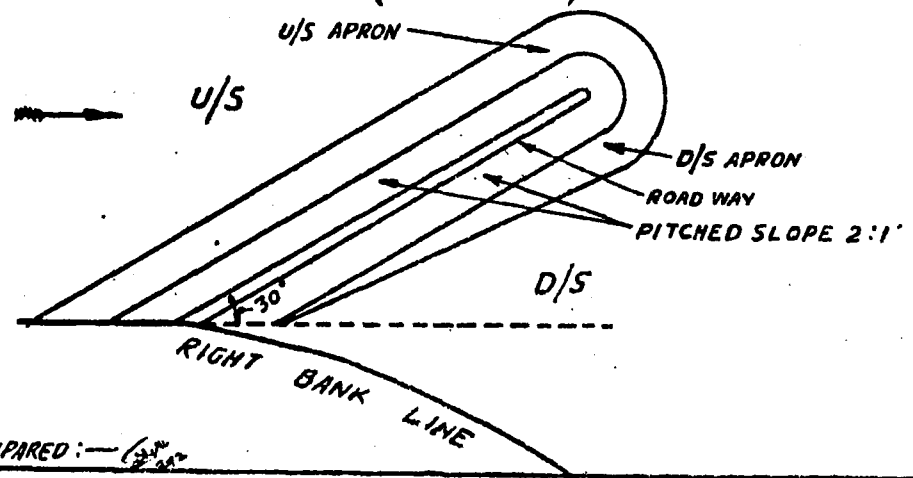


FIG. 61

M. S. M. DLY

BRIDGE NO. 10, RADIHALLA-BENNIHALLA RIVERS  
TYPICAL SECTION OF ATTRACTING SPUR

SCALE - 1" = 20'

BY: THAKOR. 24.2.50. COMPARED: — (2/5)

Bridge No. 10 Badihalla Rennihalla Rivers  
M. S. M. Railway.



Photo No. 5525-Showing the Radihalla approach  
to the Railway Bridge.

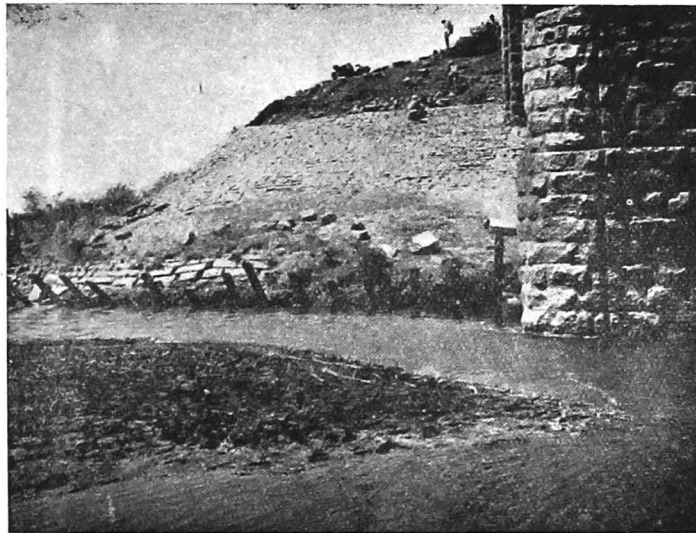


Photo No. 5532-Showing more details of abutment  
apron and damaged sheet piles.



Photo No. 5530- Showing the developed cut from  
Bennihalla to Radihalla.

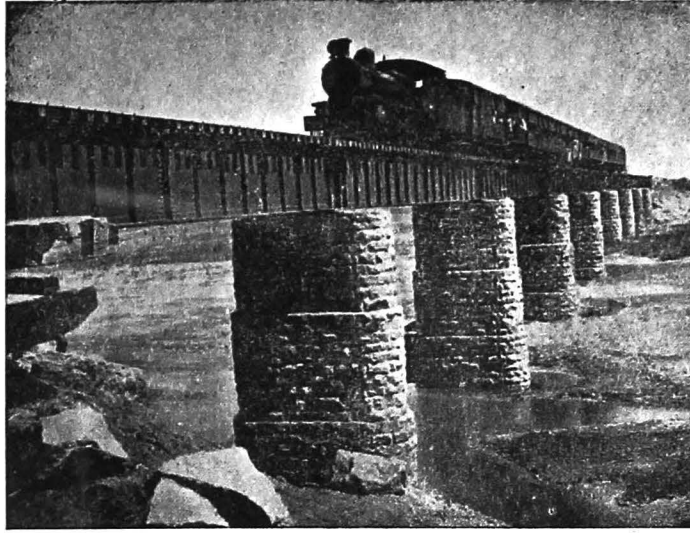


Photo No. 5534—Showing deep channel near left abutment

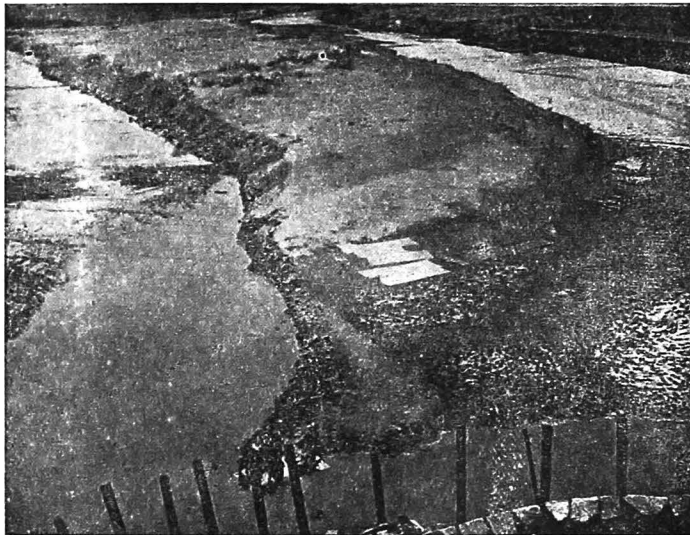


Photo No. 5533 - View looking upstream of the bridge, showing the two rivers.



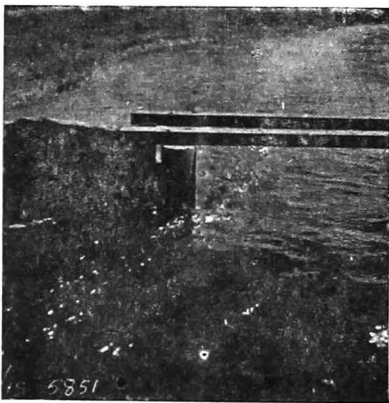
Photo No 5529—A close-up of the standing sheet piling; some piles were pushed out of vertical position by the slipping of apron

1/100, 1/25 Scale Model of Radihalla-Bennihalla Rivers.  
 Photo No. 5758



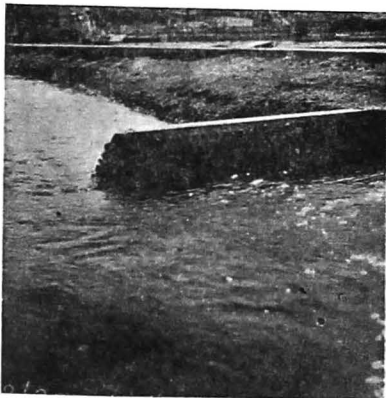
Showing the 300 ft. wide cut from Bennihalla to Radihalla with the Bridge in background.

Photo No. 5851



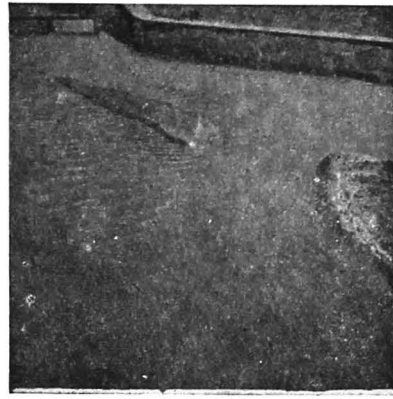
Showing the turbulence around left abutment with dominant discharge in the two rivers.

Photo No. 5849



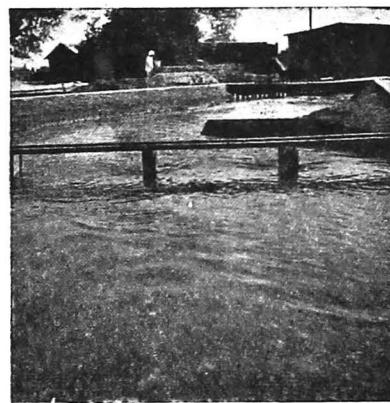
The attracting spur in action at the dominant discharge,

Photo No. 5850



Close up of the 300 ft. wide cut from Bennihalla to Radihalla.

Photo No. 5847



The Bridge in model with the attracting spur pointing downstream on the right bank in background.

Photo No. 5857



Shows contours of scour hole around the attracting spur.

BRIDGE NO. 10 ACROSS RADIHALLA-BENNIHALLA

The Bridge in model consisted of 3 spans equivalent to 9 spans in the prototype and 2 piers and 2 abutment piers equivalent to 8 piers and 2 abutment piers in the prototype. The land arch through the left abutment on account of its negligible dimensions was not reproduced in the model.

Thickness of each pier in prototype = 2'-6" and total thickness of 8 piers = 20 feet.

$$\text{Total thickness of 2 piers in model} = \frac{20}{100} = 0.2 \text{ feet.}$$

$$\therefore \text{ pier length in prototype} = 17'-6''$$

$$\therefore \text{ pier length in model} = 8.4 \text{ inches.}$$

APPENDIX II

2. *Design of Attracting Spur*

2.1. *Location of the Spur*:—The attracting spur 200 feet long pointing downstream was located on the right bank 400' downstream of the bridge at an angle of 30° with the projecting bank as shown in fig. 58.

2.2. *Depth of Scour*

$$\begin{aligned} D_{\text{Lacey}} &= 0.47 (Q/f_a)^{1/2} \\ &= 0.47 (43,200/1.76)^{1/2} \quad f_a = 1.76 \sqrt{m} \\ &= 15.2 \text{ feet.} \end{aligned}$$

Fig. 58.

For scour around nose of the spur, with a factor of safety of 2, depth or scour = 2  $D_{\text{Lacey}}$

$$= 30.4 \text{ feet (below H.F.L.)}$$

Photo 5857

Therefore the apron was designed so as to launch up to R.L. 1889.00. The top level of the spur was maintained at R.L. 1925.0 which allowed for a free board of 5.3 feet above H.F.L.

2.3. *Design of the Spur*:—A typical section of the spur is shown in fig. 61. The spur should consist of sand core with side slopes 2 Horz.=1 Vert. which should be pitched by boulder pitching 1'-6" thick over a layer of 0'-6" thick shingle backing. The 15'-0" road width on the top should have shingle pavement 0'-9" thick over 1'-3" clay base. The apron should consist of 15'-0" width of boulders 4'-0" thick at toe reduced to 2'-0" thick at heel. The apron width should be full around the nose and upstream; on the downstream side however, it may be reduced gradually to nothing at the starting point of the spur on the right bank.

Fig. 61.

### 3. DAMS AND WEIRS

#### (ii) Design

#### 9. APPLICATION OF PHOTOELASTICITY TO SOIL MECHANICS.

1. *Introduction*:—The utility of the photoelastic studies in Engineering Design problems is now too well established to need any introduction. Although a vast amount of literature has accumulated on the applications of photoelastic methods to mechanical engineering design, the literature on the applications of this method to Civil Engineering problems, in general, and Soil Mechanics in particular, is still comparatively meagre. Not only is it due to the intrinsic complexity of the problems met with in Civil Engineering and Soil Mechanics but also due to the non-availability of a photoelastic material suitable for the purpose. However, gelatine, though by no means an ideal photoelastic material, has been successfully used in photoelastic studies of the stress distribution in Dams, <sup>(1) (2) (3)</sup> foundations <sup>(3) (4)</sup> and ground tunnels<sup>(5)</sup>. The Research Station is naturally interested in problems of Civil Engineering and Soil Mechanics. It was, therefore, proposed to make, with whatever meagre equipment and material that could be collected, a systematic study of gelatine as a photoelastic material and to determine the validity and limitations of results obtained by using it.

2. *Scope and limitations of the Method*:—The applications of the photoelastic method implicitly assumes that Hooke's law holds for the material of the structure and, therefore, the results cannot be directly applied to any soil problem.

This above fact, however, does not detract from the utility of this method of stress analysis in soil problems. None of the various methods of calculating stresses in soils can give the correct results, as they are based on theories for idealised materials. In fact, for computing shearing stress elastic formulas alone are available. The best practice, advocated in actual design is to compute the stresses by the various methods, including the one based on the assumption of perfect elasticity so as to obtain the limiting values.<sup>(6)</sup> Photoelastic method, therefore, retains its importance in the solution of stress problems in Soil Mechanics and affords a rapid and convenient method for the solution of mathematically tedious, and sometimes insoluble, problems with complicated boundary conditions.

The choice of gelatine for the preparation of photoelastic models of structures dealt with in Soil Mechanics, is based mainly on the following considerations:—

(a) It has the advantage of being optically highly sensitive, thus permitting the loadings to the same scale as the dimensional scale of the model (ref. 2).

(b) It can be easily cast in various forms.

(c) Its sensitivity is sufficient to show appreciable relative retardation due to its own weight and can thus be used for the study of problems involving body forces.

As against these desirable properties (from the point of view of Soil Mechanics problems) there are the following, more or less, serious draw-backs in the use of gelatine.

(a) A high degree of deformability (small rigidity or a low value of Young's Modulus.)

- (b) A large variation of fringe constant with concentration.
- (c) Appreciable variations in fringe constant with temperature added salts etc.
- (d) Appreciable creep and a tendency to form "frozen stresses" if load exceeds a certain limit.
- (e) Marked "Time Edge" effect.

In fact, the photoelastic behaviour of gelatine is still far from being well investigated and it offers a fruitful field for further research. [For earlier work see ref. 1, 5, 7]. It will be obvious from above, that, to obtain reliable results with gelatine, a high degree of skill in the preparation of the models and caution in the interpretation of data is essential.

In the present series of investigations, which are still in progress, it will be convenient to discuss the work, so far conducted, under the following heads:—

- (a) Preparation of gelatine models.
- (b) Study of stress strain and stress-fringe order, relations.
- (c) Determination of the fringe constants.
- (d) Study of stress patterns.

### 3. *Preparation of Gelatine Model*

3.1. *Technique*—Fine shavings of gelatine were mixed thoroughly with glycerine and the requisite amount of hot water was gradually added. In order to avoid lumps the mix was well stirred. The mix was heated till all the gelatine was dissolved, care being taken that the temperature did not go beyond about 70°C. It was then cooled to about 40°C and passed through a copper mesh into a specially constructed glass cell with detachable glass sides. The mix was allowed to set for about 48 hours and the cast block was removed by gently warming the sides of the cell.

3.2. *Results*—After studying the blocks prepared with varying concentration of the three constituents it was found that blocks with near about the following composition are the most satisfactory as regards both rigidity and transparency.

Gelatine	15%	}	(by weight)
Glycerine	30%		
Water	55%		

Addition of glycerine appears to contribute to the transparency and rigidity of the block within certain limits. It also reduces the loss of water from the block on evaporation.

### 4. *Study of Stress-Strain and Stress-Fringe Order Relation*

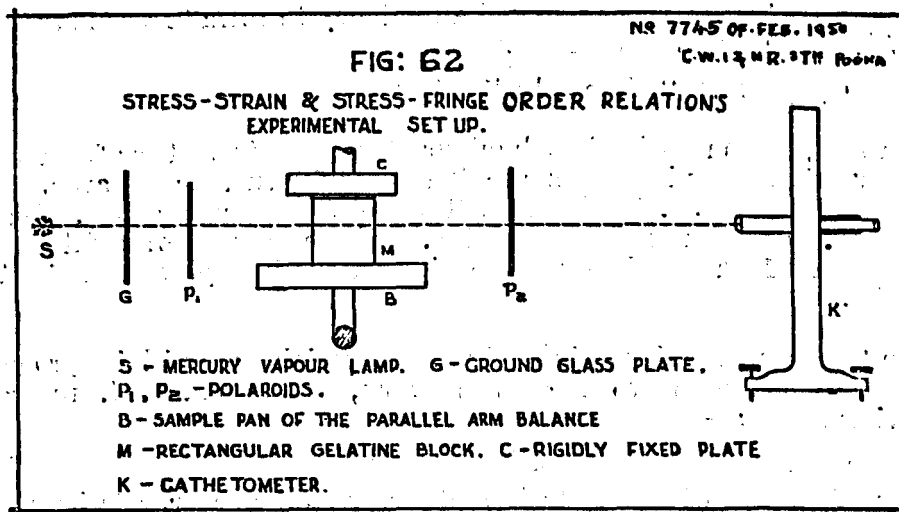
The purpose of these investigations was to determine the limit within which Hooke's law and the photoelastic law hold for gelatine mix. The composition of the mix used was.

Gelatine	16%
Glycerine	26%
Water	58%

4.1. *Apparatus*:—A diagrammatic sketch of the experimental set up is given in fig. 62 which includes a brief description of the parts used.



The load on the block was gradually increased and both the load and the strain were noted, as a dark broad fringe covering practically whole of the block appeared.



4.2. *Results*:—Observations and the calculations have been tabulated in Table I, and the relations have been plotted in Figs. 63 and 64.

TABLE I

Area of the block,  $A = 1.3 \times 2.3$  sq. cm. = 0.4635 sq. in.  
(1,000 gms. = 2½ lbs.; 1 sq. cm. = 0.155015 sq. in.).

Fringe order n	Wt in gms.	Wt. in lbs.	Cathetometer Reading (cms)		Height of block (b - a)	Contraction (C)	Strain $\frac{C}{1.42}$	Stress 'p' $\frac{W \text{ in lb.}}{A \text{ in sq. in.}}$
			(a)	(b)				
1	2	3	4	5	6	7	8	9
0	0	0	5.656	7.028	1.442	...	...	...
1	50	.11	5.696	6.850	1.154	.288	.200	.257
2	120	.26	5.866	6.862	.996	.446	.309	.570
3	200	.44	6.008	6.880	.872	.570	.395	.949
4	300	.63	6.150	6.900	.750	.692	.480	1.421
5	440	.97	6.362	6.976	.614	.828	.574	2.089

Both the relations are parabolic. It is interesting to note that after removal of the load the block regained practically its initial size within a short time. These results, however, are not of much significance in photoelastic work as too high stresses and strains are involved in the observations. The limitation arose out of the fact that with the present arrangement it was not possible to obtain a sufficiently uniform stress distribution within a larger block. With the small block, used, the stress involved, even for the first order fringe, was too high for practical purposes. It is now proposed to develop a stressing arrangement which will enable the use of larger blocks and to use a Babinet Compensator for a close study of the interesting region near the origin.

FIG. -63

NO. 1721-17 JANUARY 1950  
U.S. RESEARCH STATION

STRESS-STRAIN CURVE FOR GELATINE

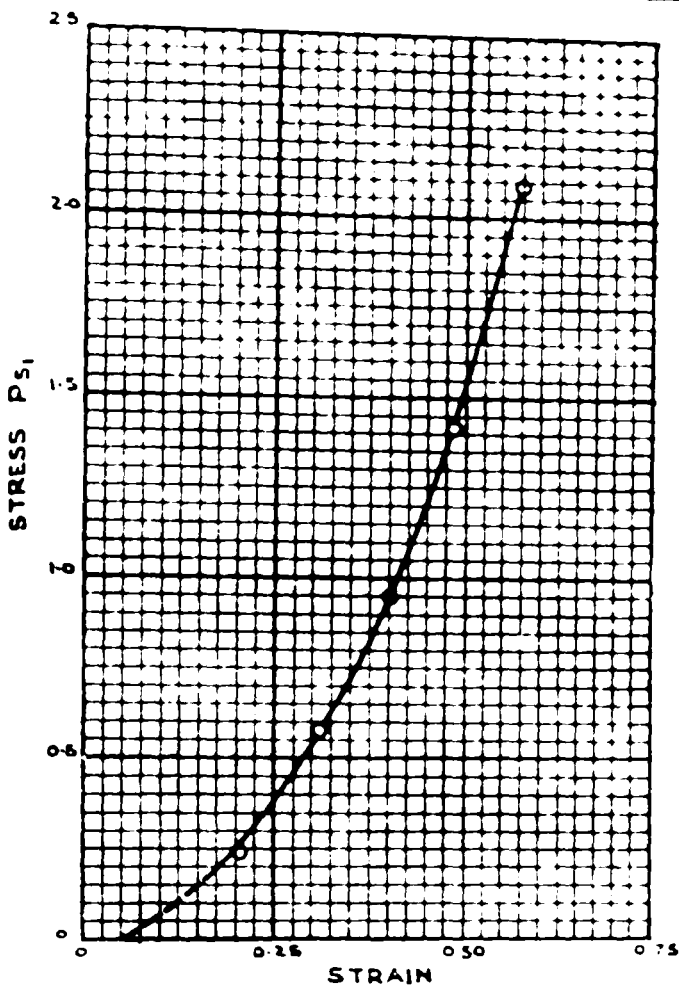
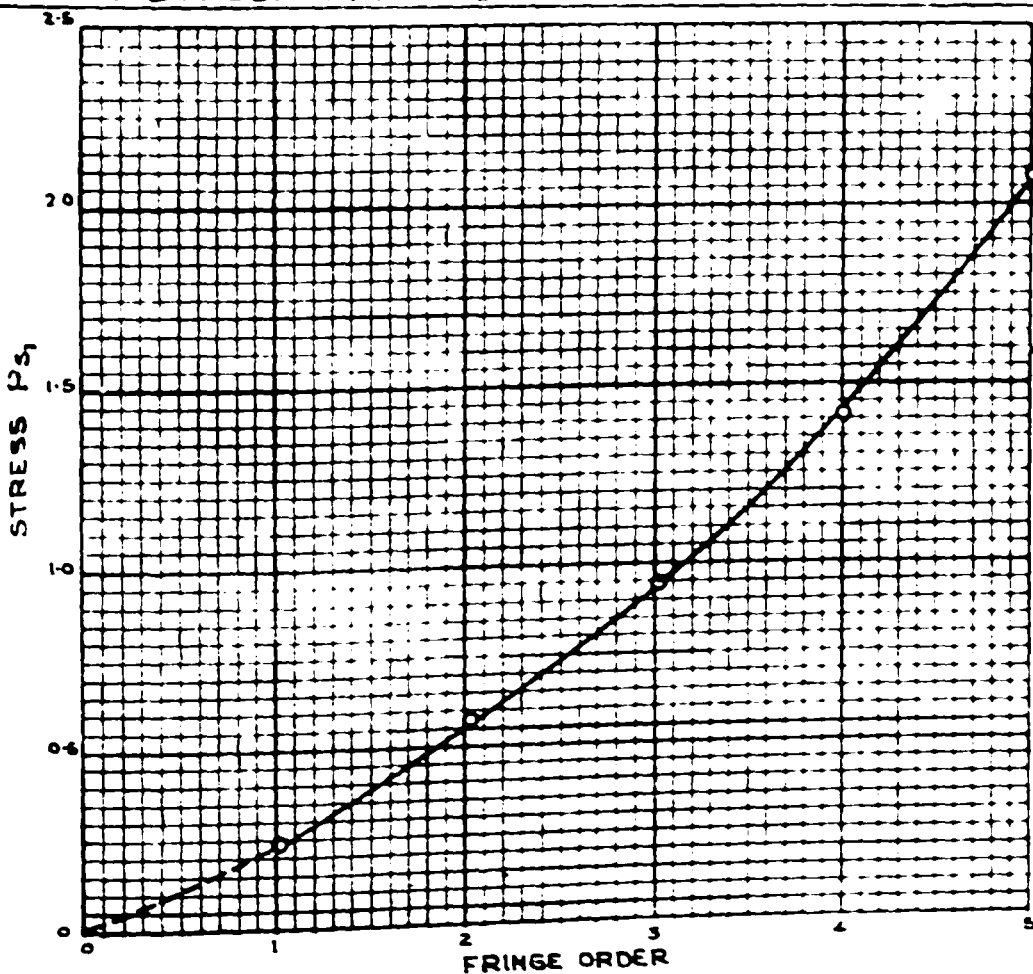


FIG. 64-STRESS-FRINGE ORDER CURVE FOR GELATINE.



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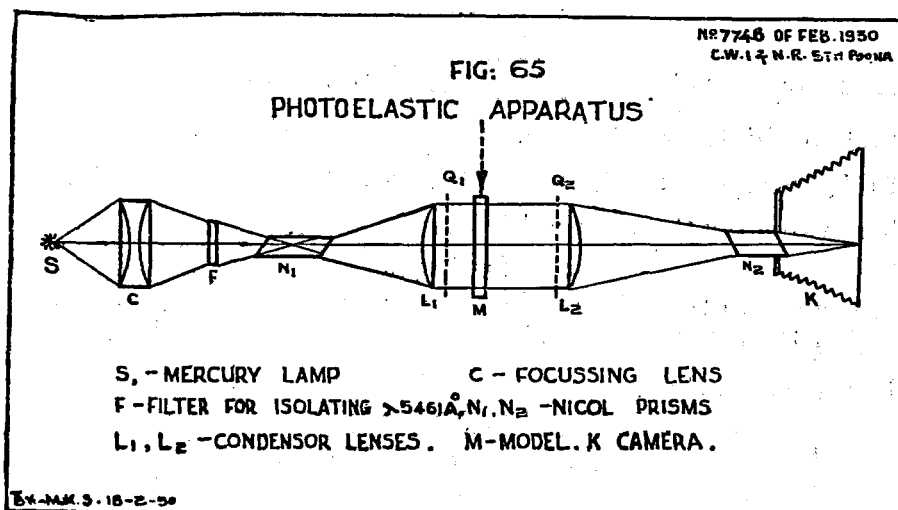
It must be noted that the values of stresses given in Table I and plotted in figs. 63 and 64 are not the actual stress values but the nominal ones, calculated on the initial area. The changes in the horizontal sections with increase in load, however, are considerable and in future work, it is proposed to measure these changes and to compute the actual stresses, as well. Incidentally, this procedure will give us the data for calculation of the Poisson's Ratio also.

From the graph the approximate value of Young's Modulus for low stresses is equal to 3 lbs/sq. inch.; which is less than that found by F. B. Farquharson and R. G. Hennes for a mix of roughly the same composition.

5. *Determination of Fringe Constant*:—As already stated, our arrangement for direct compression test was not satisfactory and it was therefore proposed to determine the material fringe value by the method of F. B. Farquharson and R. G. Hennes. However, in our case, as no confining glass cell was used, a state of plain stress rather than plane strain exists. The complete theory of the method has been discussed by M. M. Frocht in his "Photoelasticity Vol. II" (Ref. 8) and it is directly applicable to the case of plane stress. Composition of gelatine mix used in the case was

Gelatine 17%  
Glycerine 35%  
Water 48%

5.1. *Apparatus and technique*—A small demonstration polariscope was adopted for the work by suitably changing the optics and the final arrangement giving details of the various parts is given in fig. 65. The useful field was about 1 inch in diameter and no quarter wave



plates were used as the small plates available would have further reduced the field. The plane of vibrations was set at 45° to the axis of the load so as to have the least interference in the isochromatic pattern due to the superimposition of the isoclinics. Mercury line 5461 Å was used for photographing the isochromatics. The load was applied by means of a small rectangular steel block pressed by the weight of a rod passing through a hole.

5.2. *Results*.—Photograph No. P1 shows the stress pattern thus obtained which shows four isochromatics. Details of observations and calculation of the fringe constant are given in table 2.

Photo  
No. P1.

TABLE 2

Calculations made on an enlarged copy of Photo P-1  
width of the block,  $d=1.6$  cms.

(Refer to Fig. 66)

No.	Fringe order (n)	Y as measured in photo	$\tan \frac{\alpha}{2} = \frac{d}{2y}$	$\frac{\alpha}{2}$	$\alpha$	$\sin \alpha$	$\frac{\sin \alpha}{n}$	Average	Y actual in inches.	$F \times n$
1	2	3	4	5	6	7	8	9	10	11
1	1	9.9	.081	4°33'	9°16'	.161	.161	.162	.753	.123
2	2	5.7	.140	7°58'	15°56'	.275	.138		.434	.246
3	3	3.2	.250	14° 2'	28° 4'	.471	.157		.260	.369
	4	1.7	.471	25°18'	50° 26'	.771	.193		.173	.492

$$\text{Total load} = 146.8 + 2.8 = 149.6 \text{ gms.}$$

$$= \frac{149.6}{453} \text{ lbs.} = 0.33 \text{ lbs.}$$

$$\text{Area of Block} = .13 \times 1.06 = .1378 \text{ sq. inches.}$$

$$P = \frac{.33}{.1378} = 2.395 \text{ lb./sq. inch.}$$

$$\tau_{\max} = \frac{P \sin \alpha}{\pi}$$

$$\tau_{\max} = F n$$

$$\therefore F = \frac{P \sin \alpha}{\pi n}$$

$$\frac{P}{\pi} = 0.762.$$

$$\text{Model fringe value } F = \frac{P}{\pi} \left\{ \frac{\sin \alpha}{n} \right\}_{\text{av}} = .762 \times .162 = 1.23 \text{ lbs./sq. in.}$$

$$t = 1.06 \text{ (thickness of plate).}$$

$$f \text{ (Material fringe value)} = Ft. = 1.23 \times 1.06 = 0.130 \text{ lbs./sq. in.}$$

The value for the material fringe constant,

$$f = .13 \text{ psi}$$

which is close to that obtained by Farquharson and Hennes<sup>(5)</sup> for a block of roughly the same composition.

In their case the model fringe value has been stated as

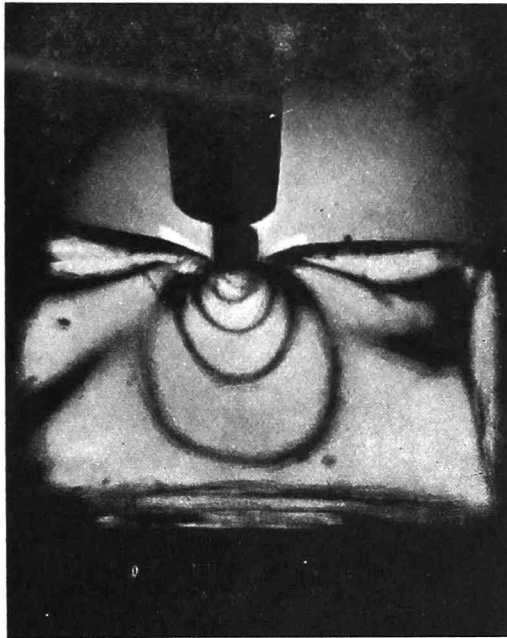
$$F = 0.071 \text{ psi}$$

$$\text{Now } f = Ft. = 0.071 \times 1.5$$

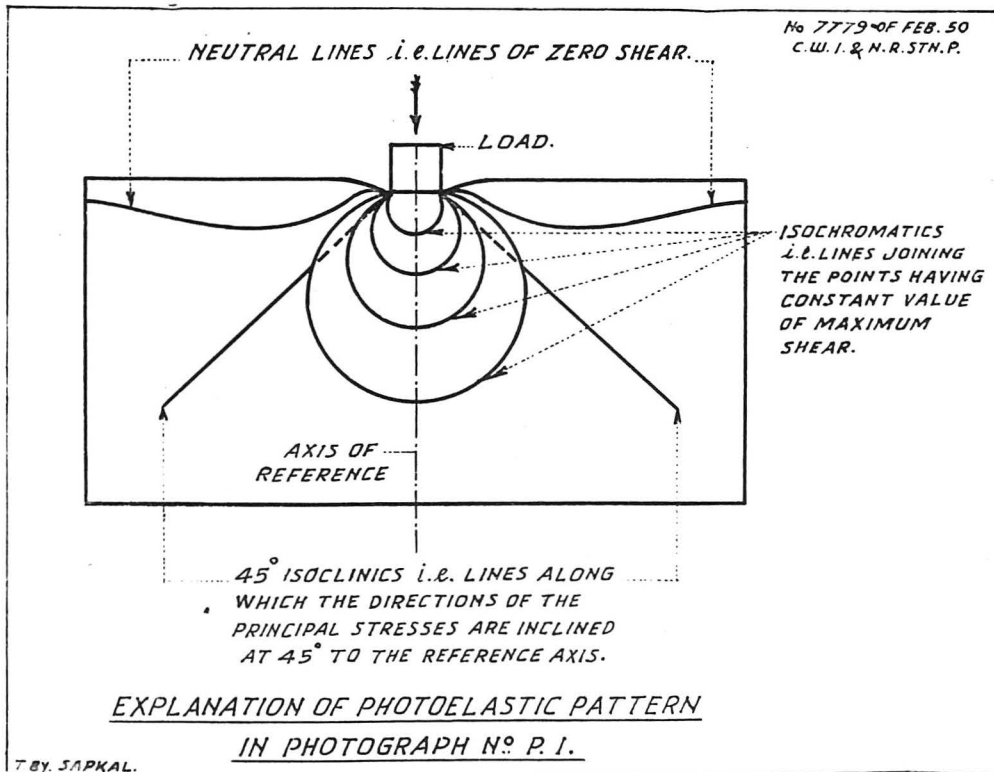
$$= 0.107 \text{ psi}$$

where  $f$  is the thickness of the block.

It must be pointed out that the material fringe value found above for the particular sample is tentative, showing only the order of magnitude. The deviation from mean  $\left( \frac{\sin \alpha}{n} \right)$  in table I is rather large for any exact determination of the constant. This is partly due to large distortion below the load and the limited size of the gelatine blocks and partly due to other spurious effects so that mathematical theory cannot be applied exactly. It is intended to use larger and thicker blocks in future so as to attain a closer approach to the ideal conditions assumed in the theory.

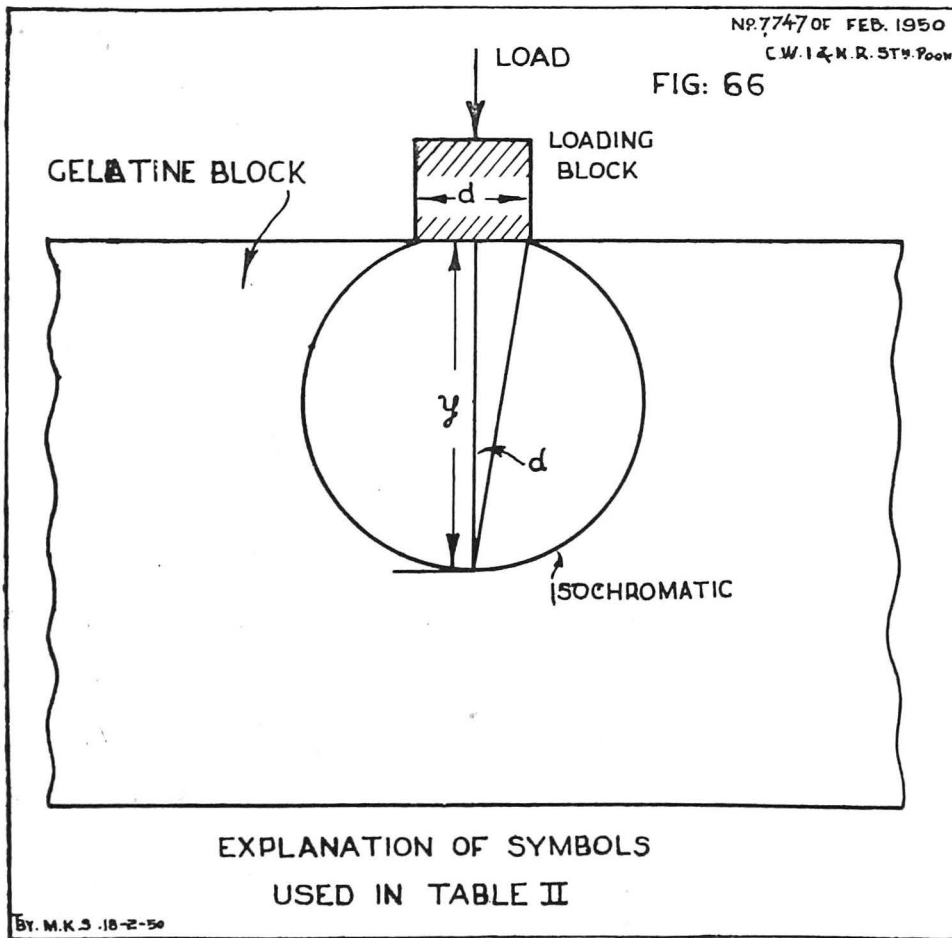


Photograph No. P. 1.





Photograph No. P. 2.



A qualitative idea of the shear stress distribution is given by plotting the fringe order against depth. This has been done in fig. 67. A theoretical curve obtained by the application of the Boussinesq formula has also been plotted for comparison. The general form of the curve agrees very well. Fig. 67

5.3. A notable feature of Photograph P-1, is the appearance of the dark neutral line which indicates that the region below it is in compression while that above it is in tension. This, of course, is due to the high degree of deformation just below the load and has not been taken account of in the theory. Although it is still premature for us to attempt a definite application of this observation to Soil Mechanics, it will be of interest to note that a region under tension has been assumed in foundation of Piers (see E. S. Barbar's discussion on stresses under piers).<sup>(7)</sup>

6. *Study of Stress Patterns*:—This detailed study was undertaken with a view to determine how far the patterns obtained with gelatine models were a faithful reproductions of those obtained from elastic theory. It was, therefore, necessary to select a load distribution which would give a pattern sufficiently complex but at the same time mathematically soluble in a straightforward manner without requiring a prolonged period for computation. It was finally decided to take up the study of the case of two distributed loads separated by a certain distance. It may have a practical significance as well, for the model stimulates the action of two piers on an elastic foundation such as rock. The work has not yet been completed and only a brief outline of what has been done so far is given below. The gelatine mix used for this experiment had the following composition:—

Gelatine	16%
Glycerine	26%
Water	58%

6.1. *Apparatus*:—In order to eliminate the isoclinics a circular polariscope was used. The apparatus was the same as indicated in fig. 65 except for the inclusion of two mica quarter wave plates  $Q_1$  and  $Q_2$  in the positions indicated by the dotted lines. These were set in the usual crossed position with their axes at  $45^\circ$  to the incident vibration. Fig. 65

6.2. *Technique*:—The general procedure used for photographing the patterns was the same as described by Frocht in his "Photoelasticity Vol I". A calibration photograph with single load was taken in the beginning.

(a) *Isochromatics* were photographed by using the mercury green line  $5461 \text{ \AA}$ . Loading of the gelatine model was accomplished by two similar rectangular steel blocks pressed by means of a glass plate which in turn was loaded by a heavy rod passing through a hole. To obtain equal loads in both the blocks the loads were adjusted till a symmetrical pattern was obtained.

(b) *Isoclinics*:—The quarter wave plates  $Q_2$  and  $Q_1$  were removed and a pointolite lamp was used as the light source. It was not found possible to obtain clear isoclinics with a gelatine model and a glass model had to be used for the purpose. For the convenience in drawing isoclinics a special reflecting table constructed in station workshop was used.

6.3. *Results*:—Photo P-2, gives the isochromatic pattern which corresponds to curves having same value of maximum shear stress,  $\tau_{\max}$ . The general agreement of these curves with those obtained mathematically (see para. 6.4 below) appears to be fairly good, though the detailed comparison and quantitative evaluation still remains to be done. The main interesting features of the pattern are the central isotropic point with zero shear stress and the pockets of maximum shear stress near the edges of the block further away from the centre.

Fig. 68. The isoclinics obtained with glass model are given in fig. 68 mainly for checking the existence of the isotropic point. Efforts are now being made to obtain reliable sets of isoclinics from gelatine models themselves.

6.4. *Mathematical Computations*:—The Boussinesq formula for a single distributed load has been used and at each point the effect of both the loads has been superimposed. A brief outline of the theory and details of computation are included, as Appendix I. Figure 68 of Appendix I shows  $\tau_{\max}$  curves thus obtained.

7. *Discussion of Results*:—Consistent results were obtained with gelatine as regards the photoelastic constant. However, care is necessary in handling, calibrations, photographing of isochromatics and drawing of isoclinics. Fairly clear isochromatics were obtained but it was not possible to draw reliable isoclinics. As the models were only one inch thick, the strains corresponding to even a few fringes were very high. It is proposed to increase the thickness of models in the further work from 2" to 3" so as to reduce the strain without affecting the sensitivity. In the investigations, so far made, body forces are negligible and for such purposes, it is realised, that gelatine is not a suitable material and plastics would give more accurate results. Though ultimately, it is intended to study cases involving body forces, it is not possible at present because of the small field of the polariscope. The purpose of the present investigations, however, was not so much to analyse a particular stress problem, as to study the behaviour of gelatine in photoelastic work and to develop suitable technique of handling it.

8. *Summary*:—A study of gelatine models for photoelastic analysis of stress problem in Soil Mechanics was undertaken to determine the extent of application of this material. The advantages and drawbacks in the use of this material in the above field have been cited. Gelatine blocks of various composition were prepared and the elastic and photoelastic constant, obtained for some samples, were compared with those of the previous workers. Stress pattern for the particular case of two loads at certain distance apart resting on a large block is photographed and comparison of the results with those obtained mathematically is being made. The work is still in progress.

9. *Acknowledgements*:—Thanks are due to the Principal and Professors in charge of the Mechanical Engineering and Physics Departments, Engineering College, Poona, for giving on loan the polariscope mentioned above and also for providing the facilities of their laboratory and library.

The mathematical computational work was done in Statistics Section of the Research Station.

Our thanks are also due to the Central Board of Irrigation for providing most of the necessary literature.

#### References:—

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- (2) Knappen and Phillipe "Soil Mechanics at Muckingum III" Engineering News Record, April 23, 1936.
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- (6) Report of Committee on Stress Distribution in Soils "Study of Stress Distribution in Soils from 1920 to 1940"—Proc. of the 21st Annual Meeting of the Highway Research Board 1941.
- (7) E. C. Coker and L. N. G. Filon—"Treatise on Photoelasticity" Cambridge Press, 1931.
- (8) M. M. Frocht "Photoelasticity", Vols. I and II—John Wiley and Sons.



FIG. 67  
MAXIMUM SHEAR  
STRESS DISTRIBUTION.  
THEORETICAL & EXPERIMENTAL.

----- EXPERIMENTAL CURVE.

-----  $T_m: 0.832 \times \frac{p}{\pi} \times \frac{d \cdot y}{\left(\frac{d^3}{4} + y^2\right)}$

—————  $T_m: \frac{p}{\pi} \times \frac{d \cdot y}{\left(\frac{d^2}{4} + y^2\right)}$

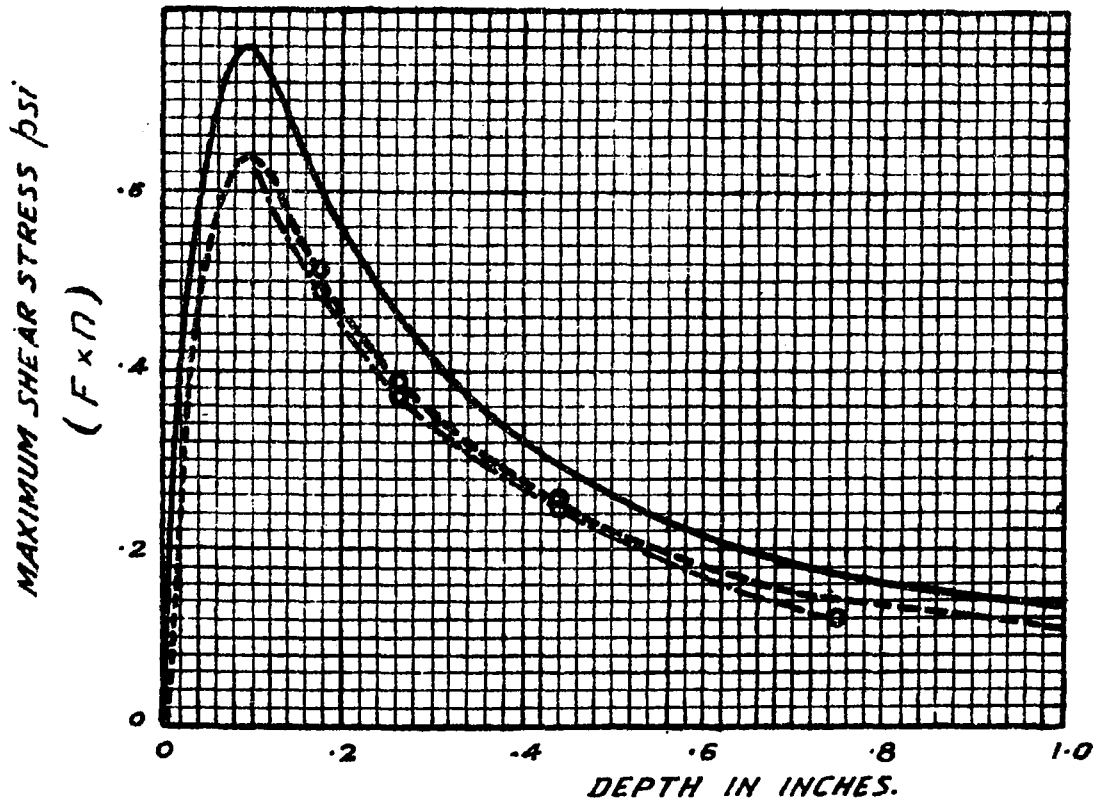
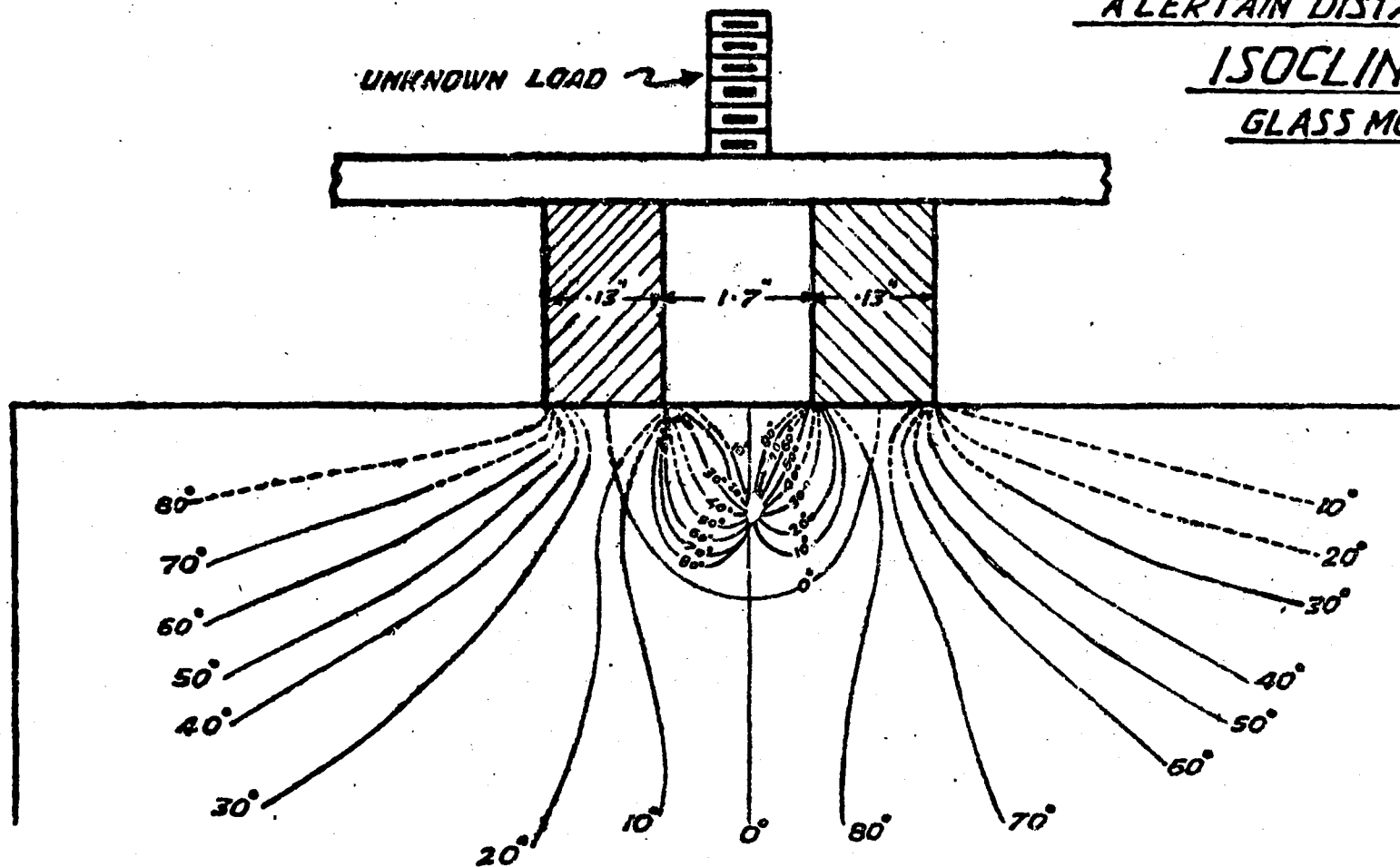


FIG. 68  
STRESS DISTRIBUTION  
BELOW TWO DISTRIBUTED LOADS  
A CERTAIN DISTANCE APART.  
ISOCLINICS  
GLASS MODEL.



APPENDIX I

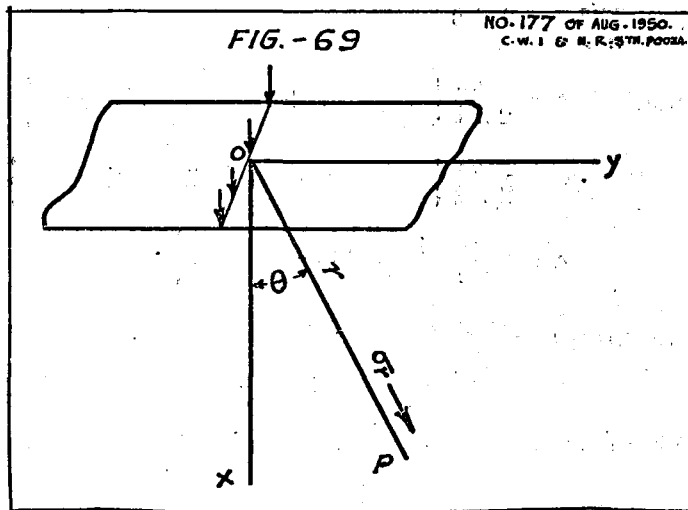
*Determination of Stress Distributions*

This note mainly deals with the evaluation of stress distribution curves inside a semi-infinite plate subjected to two normal rectangular loads, separated by a known distance, and acting on its straight edge.

In the case of a single normal load  $P$  acting on a semi-infinite plate the stress distribution is found to be purely radial. Let the origin of the co-ordinate system be taken at the points of application of the load (point load being considered) and the positive direction of the  $x$ -axis parallel to the load going into the body of the plate. The angle  $\theta$  is measured positively in the counterclockwise direction from the positive branch of the  $x$ -axis to the radius vector connecting the point inside the plate. Then the expression for the stresses which may be a compression or tension for any point  $(r, \theta)$  in the plate is given in polar co-ordinates as

$$\sigma_r = \frac{-2 P \cos \theta}{\pi t r}, \quad \sigma_\theta = 0 \text{ and } \tau_{r\theta} = 0$$

where  $\sigma_r$  and  $\sigma_\theta$  are the radial and tangential stresses and is the  $\tau$  shear and  $t$  is the thickness of the plate. Fig. 69.



*Convention of Signs:*—The sign of the concentrated load is taken positive if it acts in the same direction as the positive branch of the  $x$ -axis, so that a normal force which produces compression will be called positive. As usual, compressive stresses are taken as negative and tensions as positive.

Then the rectangular stress components due to the load  $P$  can be derived as

$$\sigma_x = \sigma_r \cos^2 \theta = \frac{-2 P \cos^3 \theta}{\pi t r}$$

$$\sigma_y = \sigma_r \sin^2 \theta = \frac{-2 P \sin^2 \theta \cos \theta}{\pi t r}$$

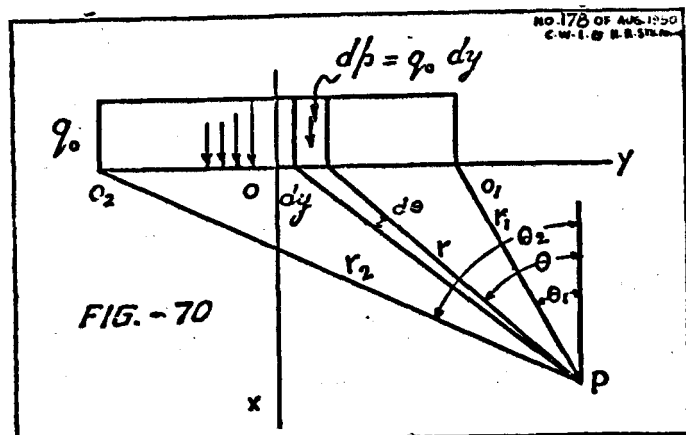
$$\tau_{xy} = \sigma_r \sin \theta \cos \theta = \frac{-2 P \sin \theta \cos^2 \theta}{\pi t r}$$

If  $d_p$  denotes a load on an infinitesimal length  $d_y$  on the edge of the plate, then we have

$$d_p = q_0 d_y; \quad d_y = \frac{r d \theta}{\cos \theta} \quad \text{and} \quad d_p = q_0 \frac{r d \theta}{\cos \theta}$$

Fig. 70.

The rectangular stress components due to elementary load are obtained from Fig. 70 as



$$d \sigma_x = \frac{-2 q_0}{\pi t} \cos^2 \theta d \theta; \quad d \sigma_y = \frac{-2 q_0 \sin^2 \theta d \theta}{\pi t}$$

$$d \tau_{xy} = \frac{-q_0}{\pi t} \sin^2 \theta d \theta$$

$$\text{whence } \sigma_x = \frac{-q_0}{2 \pi t} \left\{ 2(\theta_2 - \theta_1) + (\sin 2\theta_2 - \sin 2\theta_1) \right\}$$

$$\sigma_y = \frac{-q_0}{2 \pi t} \left\{ 2(\theta_2 - \theta_1) - (\sin 2\theta_2 - \sin 2\theta_1) \right\}$$

$$\tau_{xy} = \frac{-q_0}{2 \pi t} \left\{ \cos 2\theta_1 - \cos 2\theta_2 \right\}.$$

And the maximum shear is given by

$$\tau_{\max} = \frac{1}{2} \sqrt{(\sigma_x - \sigma_y)^2 + 4 \tau_{xy}^2}$$

In the case of a single load, the above simplifies to

$$\frac{q_0}{\pi t} \sin \alpha$$

where  $\alpha$  is the angle which the two extremities of the load subtends at the stressed point.

The same formula can be extended for two or more rectangular normal loads and the stress components are written as Fig. 71

$$\sigma_x = \frac{-q_0}{2 \pi t} \left\{ 2(\theta_2 - \theta_1) + (\sin 2\theta_2 - \sin 2\theta_1) + 2(\theta_2 - \theta_1) + (\sin 2\theta_2 - \sin 2\theta_1) \right\}$$

and similarly for  $\sigma_y$  and  $\tau_{xy}$

Then  $\tau_{\max}$  can be easily calculated from these stress components.

In the case of a single normal load, whether the final formula for  $\tau_{\max}$  simplifies to a great extent and reduces to a single parametric relation and the equi  $\tau_{\max}$  curves are clearly the different circles having the line of contact of the plate with the load as common chord. But when we consider the case of two normal loads, the evaluation of equi-curves is not so simple. Firstly, the distribution will be dependent on the distance between the loads. Even for a known distance the value of  $\tau_{\max}$  depends upon two parameters and the relation is too much complex.



PHOTOELASTICITY AND SOIL MECHANICS

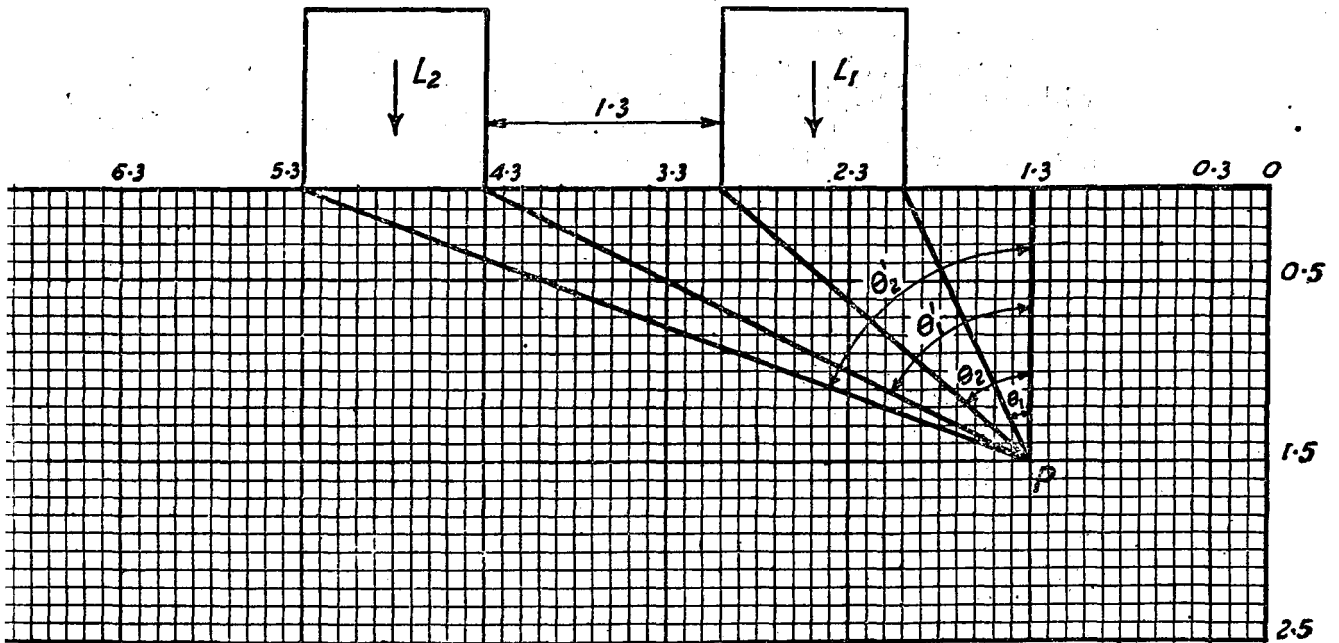
That is to say  $\tau_{\max} = F(\theta_1, \theta_2)$  for a known distance between the loads.

The direct solutions for  $\theta_1, \theta_2$  in the above case for constant values of  $\tau_{\max}$  seems to be impossible. So alternatively for a fixed distance between the loads, we propose to divide the interested region to a lattice design. The network should be sufficiently close to get a fair representation of the entire field under consideration. The stresses at all the different intersections of the network can then be calculated from the formulae given above. These stresses will then enable the evaluation of  $\tau_{\max}$  at the corresponding points.

For a particular case where the distance between the normal loads is 1.3 times the base width of either load in contact, the various stresses and shears along with their equi-stress lines have been obtained.

FIG. 71.

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C. W. I. & H. R. STY. ROOM.



The Fig. 71 shows that the whole region has been divided into 73 x 25 sq. by means of 73 vertical and 25 horizontal equally spaced lines. It is evident from the formulae that the stresses at the corners of the square are symmetrical with respect to the mean vertical lines (except for signs in some cases), and the calculations are done for only one-half of the plate, the remaining being a simple replica.

Distribution curves attached are sufficient to indicate the characteristics of stress distributions inside the semi-infinite plate.

## 10. A NOTE ON CERTAIN PRECAUTIONS TO BE OBSERVED IN PHOTOELASTIC WORK

The use of Photoelastic method for stress analysis and engineering design problems has been made very widely in countries other than India. For some time back a few institutions in this country also have been trying to use this method.

Work on the subject was reported by the Punjab Irrigation Research Institute, Lahore, in their Annual Report for the year 1945. A close study of the work brought out the following points:—

### 1. *Stress Optic Law*

The stress optic law has been stated as:—

“The photo-elastic method of stress analysis is based on the fact that in a model of uniform thickness the fringe order is proportional to the stress, i.e.  $p = nf$ , where  $n$  is the fringe number,  $f$  is the fringe constant of the material and the light source and  $p$  is the principle stress.”

In general, at any point in a two-dimensional stress system, there will be two principle stresses, namely, the major (generally denoted by  $p$ ) and minor (generally denoted by  $q$ ). In stating the law as above it was not stated which of the stresses was meant. The more exact statement would be

$$n\lambda = ct(p - q) \quad \dots(1)$$

where  $n$  is the fringe order (dark fringes),  $\lambda$  is the interfering wave length and  $c$  is the stress optic co-efficient of the material. The relation will hold on either plain or circular crossed polariscope,  $t$  = thickness of the model.

Since  $\frac{p-q}{2} \tau_{\max}$  the maximum shear, we can put equation

$$(1) \text{ in the form } \tau_{\max} = \frac{nf}{t} \quad \dots(2)$$

where  $f = \lambda/2c$  a constant only if light of a certain wave-length is used every time,  $f$  being a constant for the material is called material fringe value. In the same model, as thickness,  $t$  is also constant,

$$\tau_{\max} = nF \quad \dots(3)$$

where  $F$  is called the model fringe value.

Assuming  $q = 0$  if we compare equations (2) and (3) with that given in the report under consideration it appears that  $f$  corresponds to the model fringe value and not to the material fringe value as the value so found will vary with the width of the model used.

### 2. *Calibration*

For the calibration procedure it is stated that:—

“In order to determine the fringe constant of the mixture, a portion of the model is loaded with a brass rod; 5.5 mm. in diameter = width of the model. This showed tri-coloured bands, green, red and yellow, of which green was the most pronounced and was, therefore, selected for observation. The pressure on the brass rod was increased by equal increments and the number of green fringes which appeared on the screen was counted in each case.”

In any calibration the mathematically determined  $\tau_{\max}$  at any particular point is to be divided by the observed fringe order at the point to get the value of model fringe constant and multiply it by the thickness of the model to obtain the material fringe value. It appears that no particular point is chosen for the purpose of calibration and the fringes

## PRECAUTIONS IN PHOTOELASTIC WORK

are counted over the entire surface. Roughly the procedure may correspond to calibration at a point immediately below the load and this also will perhaps give only an approximate value. This is because it is difficult to detect the fringe immediately below the load and from the very nature of the experiment, errors of the order of one fringe are likely to occur.

The appearance of the green band has been attributed to the monochromatic radiation  $\lambda=4200 \text{ \AA}$ . However, the green thus observed is far from being monochromatic as it is produced by the destructive interference of the far red ( $\lambda 7000 \text{ \AA}$ ) in white light leaving a visual impression of green. An account should also have been taken of the fact that the green of different orders do not correspond to the same wave-length and use of formulae like  $p=nf$  is not valid [Equation (1) would have given better results].

### 3. Results obtained

In Fig. 87 of the work under consideration, the graph on the right hand side indicates that the stress  $p$  within a very small distance, immediately below the load, is zero. If  $p$  is to be taken as a principal stress as stated in the work, such a graph will mean that there is no reaction at the point. The use of a rod of round section for simulating a study of plain strain is in itself perhaps not a sound procedure.

The statement "Study of stress distribution in an earth mass due to a concentrated load (heading of fig. 86)" appears to be rather misleading. Krynine<sup>(1)</sup> has pointed out that materials like gelatine are far from being soil and they always do not duplicate the stress conditions in an earth mass and he has further suggested a large number of limitations if gelatine models are to be used for investigation on problems of earth works.

In simulating stresses in an earth mass gelatine models are in no way better than those of any other photo-elastic material. The only advantage of using gelatine comes from the fact that because of its high sensitivity it can be used to study the effects of body forces in models of practicable sizes. This could also be studied in plastic models; but, the size of the model would be much bigger and so the use of plastic models for the study of effects produced by body forces is not practicable. In the work under consideration the body forces could not be taken into account. From the data given for the densities it can be presumed that the density of the mixture would be of the order of 1.1 gms. per cubic cm. and maximum pressure at a depth of 3.5 cm. will be  $3.5 \times 1.1 = 3.85$  gms. weight per sq. cm. and using calibration data it corresponds to a fringe order of  $\frac{3.5 \times 1.1}{4.97} = \frac{1}{126}$ , which cannot make any perceptible difference in stress distribution. Neither body forces nor other characteristic properties of soils appear to have been taken into account and, therefore, what has been represented is the stress distribution in an elastic medium below a load distributed over a small area.

(1) "Soil Mechanics" by Krynine, page 276.



#### 4. HYDRAULIC STRUCTURES ON PERMEABLE FOUNDATIONS

##### 11. COMPARISON OF SHRI. KHOSLA'S AND DR. RAO'S METHOD FOR THE DETERMINATION OF THE DISTRIBUTION OF UPLIFT PRESSURES UNDER STRUCTURES ON PERMEABLE FOUNDATIONS

Though there is a scope for logical criticism and alterations Dr. Rao's paper contains a new application of basic principles to the solution of uplift pressure distribution. However, it has itself created a new approach which given more attention to, may be important to solve the problem.

(1) It is noted from the paper that Dr. Rao is not satisfied with the results obtained by the Electrical Analogy method at critical points. It may, however, be said that Electrical Analogy gives very accurate values even at these points as can be seen from the work of Vaidhinathan and Gurdas Ram. Reference can be made to their original papers on the subject and to that of Salem in Journal Am. Soc. of Civil Engineers, Vol. 71, No. 10. It will be seen that Electrical Analogy can be applied to determine the distribution at any and every point of the dam or weir including the critical points referred to in the paper. It can also be used to determine the effect to stratification on the distribution of uplift pressures.

(2) About Khosla's method of independent variables, the paper mentions:—

(a) that it becomes empirical when analysing the actual structures:

(b) that it does not give pressures at all points of the floor; but at a few selected points,

(c) that it requires the availability of charts;

(d) that the method is applicable only for homogeneous soil conditions, and cannot take note of stratified layers.

*Regarding (a):* It is to be noted that the method has been based on vigorous mathematical theory and accurate electrical analogy and hydraulic model experiments. Though the correction was suggested as such from purely empirical ideas, they have since been given theoretical basis by Malhotra and Taylor and the method is still subject to modification and verification by a more rigid theoretical proof.

*Regarding (b):* Though Khosla mentions about pressures at few selected points only, pressures at other points can be obtained by reference to the theoretical and electrical analogy work referred to above on which the method has been based. According to his method a complex weir is to be broken up into several simple cases and distribution of uplift pressures in these simple cases can be obtained from the charts. So, the method gives pressure values at all points.

*Regarding (c):* It may be noted that design of important structures like weirs and dams are generally made in the office where charts will be available. Of course, if charts are not available, Dr. Rao's method will be very useful.

*In connection with (d):* Shri Khosla has not taken into account the effect of stratified layers in his method and Dr. Rao's method is better in this respect. It may, however, be noted that the pressures in the case of stratified foundations can be found by determining the relative permeability of the different layers of the foundations.

#### UPLIFT PRESSURES ON PERMEABLE FOUNDATIONS

(3) Figure (6) in Dr. Rao's paper gives a new graphic representation of variation of pressure with distance, and is easily readable.

(4) It has been stated in Dr. Rao's paper that his method starts with an approximate method and then making use of the values obtained by this method, an accurate method is applied to determine the pressures without the help of any charts. It has been stated on page 7 of the paper that—

“...the distance along the horizontal floor to which the decreased rate is felt can be approximately taken as  $=\frac{1}{2}$  the pile depth and the rate is also assumed as  $\frac{1}{3}$  of the rate obtained at the point when the whole floor is considered as horizontal.”

It is not clear on what data this assumption is based. However, the validity of the assumption can only be judged from the usefulness and its application and results.

(5) Dr. Rao's exposition in explaining the Southwell's relaxation has been most explanatory and useful for grasping the whole situation. It will, however, be seen that to get useful floor data from the residual equation.

$R = P_1 + P_2 + P_3 + P_4 - 4P_0$  will involve laborious calculations.

(6) It is realised that in case of sheet piles this new method with an arbitrary assumption merges to convergent chain of approximations in the method of relaxation suggested by Dr. Rao unlike Shri Khosla's method of independent variables, which starts with a strict and fundamental idea and merges into an approximation but later on substantiated both by experiment and theory. It would be better if pressures are calculated under all the actually existing weirs and the amount of statistical mean deviation from actual case is determined.

(7) Dr. Rao's extension in case of stratified non-homogeneous strata is an advance over Shri Khosla's method but, it may be noted that Shri Khosla's method can be extended to the case of stratified layers by use of electrical analogy method as is already done by Salem.

(8) In his Fig. (9), the data obtained by Rao's method for exit gradients differs much from that obtained by Shri Khosla, and, an experimental verification is suggested to decide the correctness of one method or other.

## 12. MEASUREMENT OF FRICTIONAL FORCES IN WET SAND

Many Engineering structures are embeded in sand or built on sand foundations. Uplift, slipping or any other kind of motion relative to sand foundation will give rise to frictional forces in wet sands. Such a force will also be encountered in sinking of wells. Such frictional forces in wet sands are of great importance to the design engineer.

Frictional forces in dry sands have been more or less standardized but this is not so with those in wet sands. It is, therefore, considered desirable to examine on the one hand the physical factors contributing those forces and, on the other hand, its variation, specially with respect to the particle size. In order to avoid the effect of the crystalline structure and angular characteristics it is intended to experiment at first, as far as possible, with sands free from foreign matter. Hooghly river sand, which was readily available was used in the experiment.

Frictional forces of a sand depend on its composition and geology and age of the rocks from which the sand was formed. However, before, embarking on such a detailed analysis of this important physical property, it is desirable to have an overall idea of the magnitude and characteristics of this property in a general way.

It is important to note that the present analysis deals with the nature of the frictional forces of completely saturated sands.

In order to give practical shape to the problem it is intended, ultimately, to investigate frictional forces between wet sand and masonry surfaces; but, in the present investigation a glass surface has been used.

The method of experimenting was to utilise the hydrostatic uplift on a body embeded in sand due to a head of water. The motion of the body in contact with sand was then subject to a frictional resistance proportional to its area of contact with the wet sand. Knowledge of all the forces involved makes it possible to determine the frictional force per unit area.

Fig. 72.

The apparatus used is shown in fig. 72. It consists of a glass jar 'J' of relatively bigger diameter compared with that of the glass beaker 'B' against the surface of which the frictional force of the sand is required to be determined. The jar was packed with sand in water upto a height of 5 cm. and water stored to a height of 5 cm. above the surface of sand. The total height of the beaker under experiment was divided into four parts by marks, each 3 cm. apart.

Microscope 'M' of long focal length was used to detect the tendency of the beaker to move.

The beaker was filled with water and was at first placed on the surface of sand in the jar embeded in water only. The water in the beaker was siphoned out till it showed a tendency to move. The weight of water 'W<sub>1</sub>' in the beaker at this moment was determined.

The beaker was then again filled with water and placed in the jar, but this time it was embeded in sand upto a height 'h'. It was again subjected to the same hydrostatic pressure as in the previous experiment and the water siphoned out again till it tended to be uplifted. The weight of water 'W<sub>2</sub>' in the beaker at this moment was also determined.

In the second case the motion of the beaker was subjected to a frictional force F per unit area due to wet sand.

If "a" is the outer radius of the beaker, it can be easily shown that

$$F = \frac{W_1 - W_2}{2\pi ah}$$

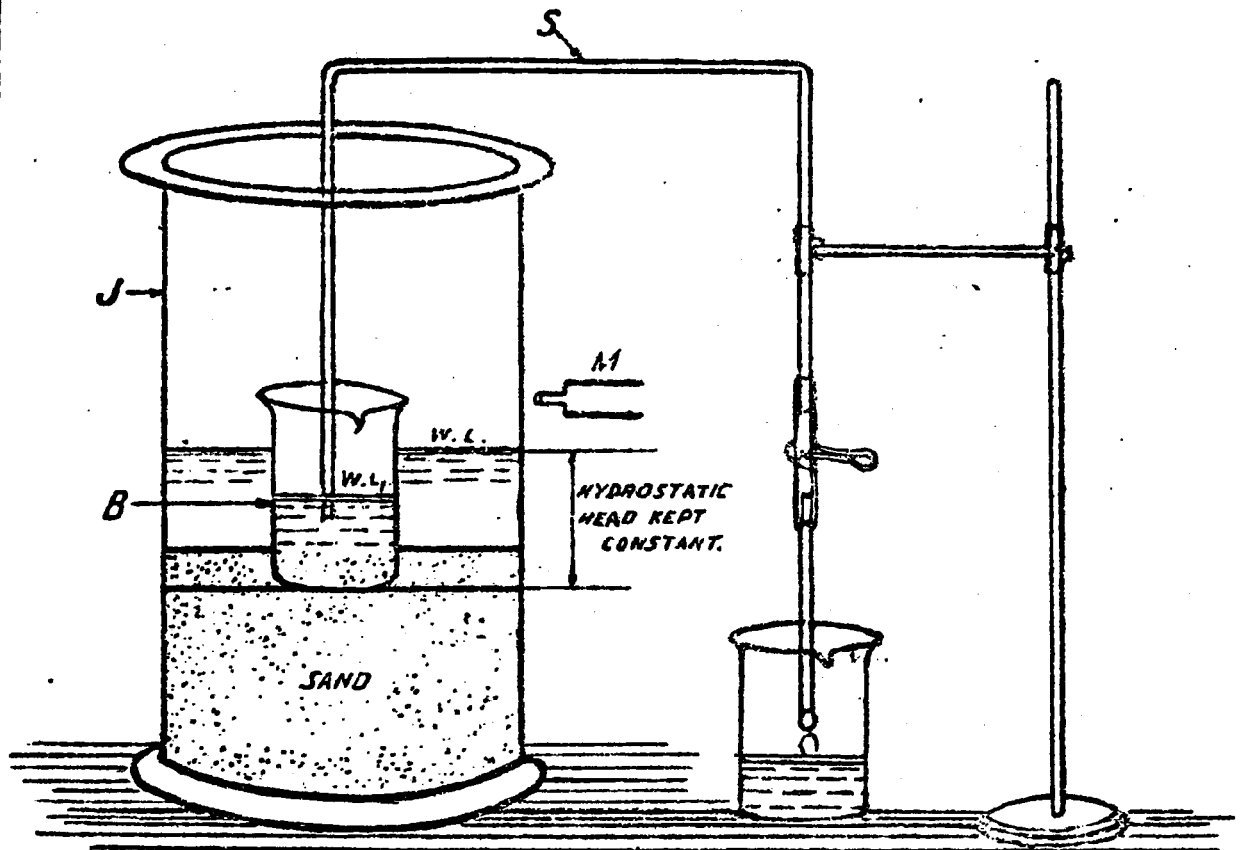


FIG. 72

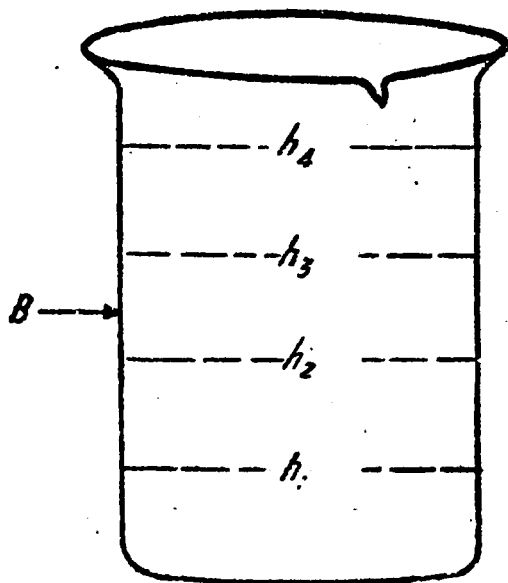


FIG. 73

REFERENCES.

- J.....JAR.
- B.....BEAKER.
- W.L.....WATER LEVEL IN JAR
- W.L<sub>1</sub>.....WATER LEVEL IN BEAKER.
- S.....SIPHON.
- M.....MICROSCOPE.

FRICIONAL FORCES IN WET SAND

In order to avoid uncertainty of accurately determining area of the beaker near its bottom, weight of water in the beaker at the time of uplift when it was embeded upto a height of 3 cms. was determined. This was treated as correction in all the subsequent observations.

The beaker was embeded in sand upto various heights as shown in fig. 73; but subjected to some hydrostatic uplift measured from water surface in the jar to the bottom of the beaker. The observations have been given in Table 1. Here  $W_2$  is the weight of water in the beaker just at the time of uplift when it is embeded upto a height ( $h_1$ ) 3 cms.  $W_3$  when it is embeded upto ( $h_2$ ) 6 cms. and  $W_4$  when embeded upto ( $h_3$ ) 9 cms. in the sand.

Fig. 73

TABLE 1

W IN GRAMS

Sand particles between meshes	$W_2$	$W_3$	$W_4$	$W_2 - W_3$	$W_3 - W_4$	$W_2 - W_4$
20 - 40.	387	341	284	46	57	103
40 - 60	387	351	293	36	58	94
60 - 80	395	357	306	41	51	92

The relation

$$F = \frac{W_1 - W_2}{2\pi ah} \text{ can be put as } F = A(W_1 - W_2), \text{ when } A = (2\pi ah)^{-1}.$$

It shows that  $F \propto (W_1 - W_2)$ .

It was chosen that  $h_2 - h_1 = h_3 - h_2 = 3$  cms. but though  $h_2 - h_1 = h_3 - h_2$ , it is seen from the table that frictional resistance in these two belts is not equal. The present data is not sufficient to conclusively explain this variation. It is hoped that future work on an extensive scale will perhaps make it susceptible to explanation.

On examination of column  $W_2 - W_4$  it will be seen that the frictional force shows a tendency to decrease with size. This also requires extensive work for arriving at conclusive results.

Though it is possible to calculate absolute value of frictional resistance per unit area from the formula  $F = (W_1 - W_2) A$ , it has not been done as it is intended first to substantiate by a large number of observations that no extraneous forces exert any influence. Effect of sizes of the inner beaker has also to be investigated.

*Summary*

A simple method, depending on hydrostatic uplift, is adopted to measure the frictional resistance in wet sand.

The formula

$$F = \frac{W_1 - W_2}{2\pi ah} = \text{gms. wt. per sq. cm.}$$

was used to calculate this frictional force. This investigation shows some interesting results regarding variation of this force with respect to sizes of particles.

## 6. SPILLWAYS

### 13. EXPERIMENTS FOR DISSIPATION OF ENERGY BELOW THE PROPOSED VAITARNA DAM SPILLWAY (BOMBAY)

#### Introduction

1.1. In connection with the Vaitarna Dam proposed to be constructed by the Bombay Corporation for the augmentation of water supply to the city of Bombay, experiments were carried out with a 1/50 scale G.S. Model of the central spillway for studying the hydraulic features of the design with various discharges in continuation of the tests performed<sup>(1)</sup> on the 1/30 G. S. part-width model.

1.2. The following experiments were carried out:—

(a) To investigate the pressures on the weir profile for all conditions of flow with discharges varying from 20,000 to 2,00,000 cusecs.

(b) To determine the co-efficient of discharge for various discharges ranging from 20,000 to 2,00,000 cusecs.

(c) To find out the depth, extent and position of scour with 5.0 ft. bucket for different discharges and corresponding tail water levels.

(d) To locate the position where the upper face of the jet issuing from the bucket impinges on downstream water level.

1.3. A part-width geometrically-similar model of the central spillway to the scale of 1/50 and representing 140 feet length of the weir with two piers of 14.0 feet each was constructed in a 3.5 feet wide flume. A bucket of 5.0 feet was provided at the toe and one side of the flume was made of glass to study the action of the bucket. Photo 5612 shows the model, looking upstream, with the bucket at the toe in the foreground. A Rehbock Weir was fixed at the tail for the exact measurement of discharge and the downstream water levels were regulated by means of an adjustable gate.

Photo 5612.

(a) *Pressure observations*:—Experiments for pressure observations were carried out by fixing 20 piezometric tubes on the entire profile of spillway with due consideration to every change of curvature or slope. Figure 74 shows the location of piezometric points on the profile. The following tests were made:—

Fig. 74.

Test I with Q=27,380 cusecs.	Test V with Q= 95,380 cusecs.
Test II with Q=47,960 cusecs.	Test VI with Q=115,300 cusecs.
Test III with Q=65,960 cusecs.	Test VII with Q=135,260 cusecs.
Test IV with Q=86,540 cusecs.	Test VIII with Q=169,450 cusecs.

Fig. 75,  
Sheet I.

The pressures in the above eight tests are plotted in Fig. 75 and their values are given in Sheet I in tabular form.

*Results of Pressure Observations*:—From tables in Sheet I it will be that the pressures are positive upto a discharge of 1,00,000 cusecs and only slight negative pressures develop near the crest with discharges higher than 1,10,000 cusecs; thus the weir is quite safe from the danger of cavitation as the maximum expected discharge is 92,000 cusecs.

(b) *Co-efficient of Discharge*:—Fifteen tests for the determination of the co-efficient of discharge were carried out by running various discharge from 20,000 to 200,000 cusecs measured accurately by means of a Rehbock Weir fixed at the tail end of the model and observing the actual depth over the crest for corresponding values of discharges. Curves between "Discharge" and "Measured head over crest" and "Discharge" and "Co-efficient of discharge" are plotted in Figs. 76 and 77. The value of 'C' in the formula  $Q = CLH^{3/2}$  for a maximum flood discharge of 92,000 cusecs was found out to be 3.99.

Figs. 76, 77.

(1) Annual Report (Tech.) 1948.

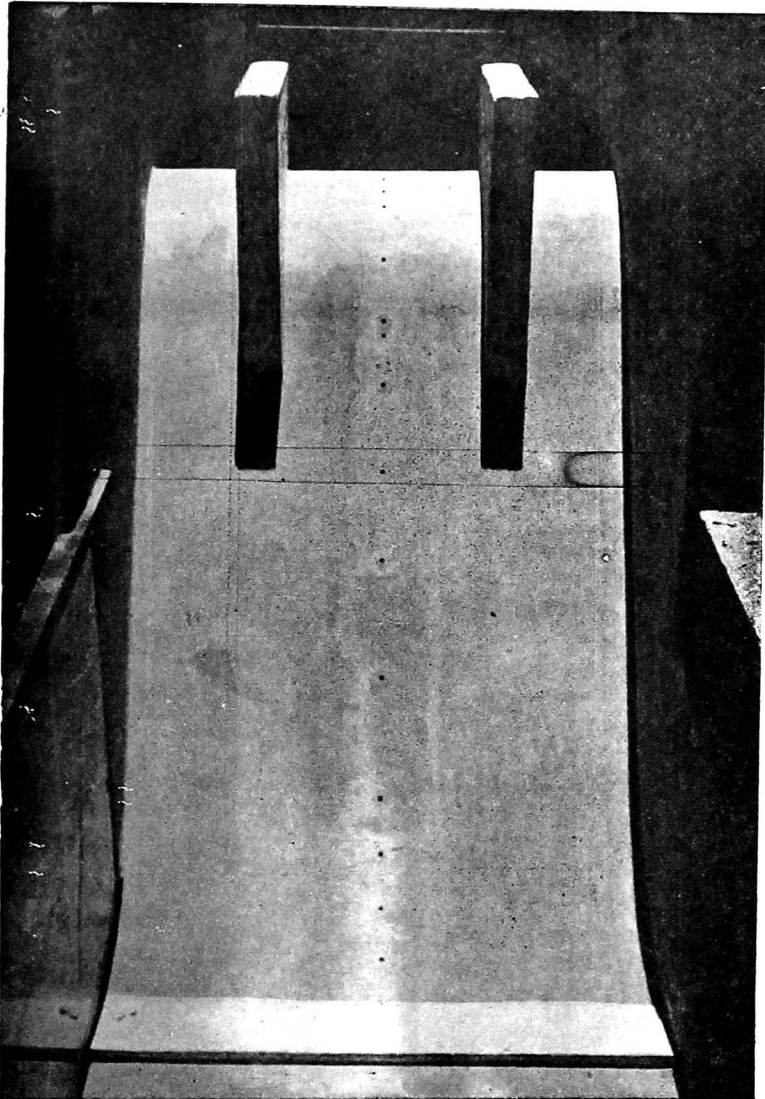
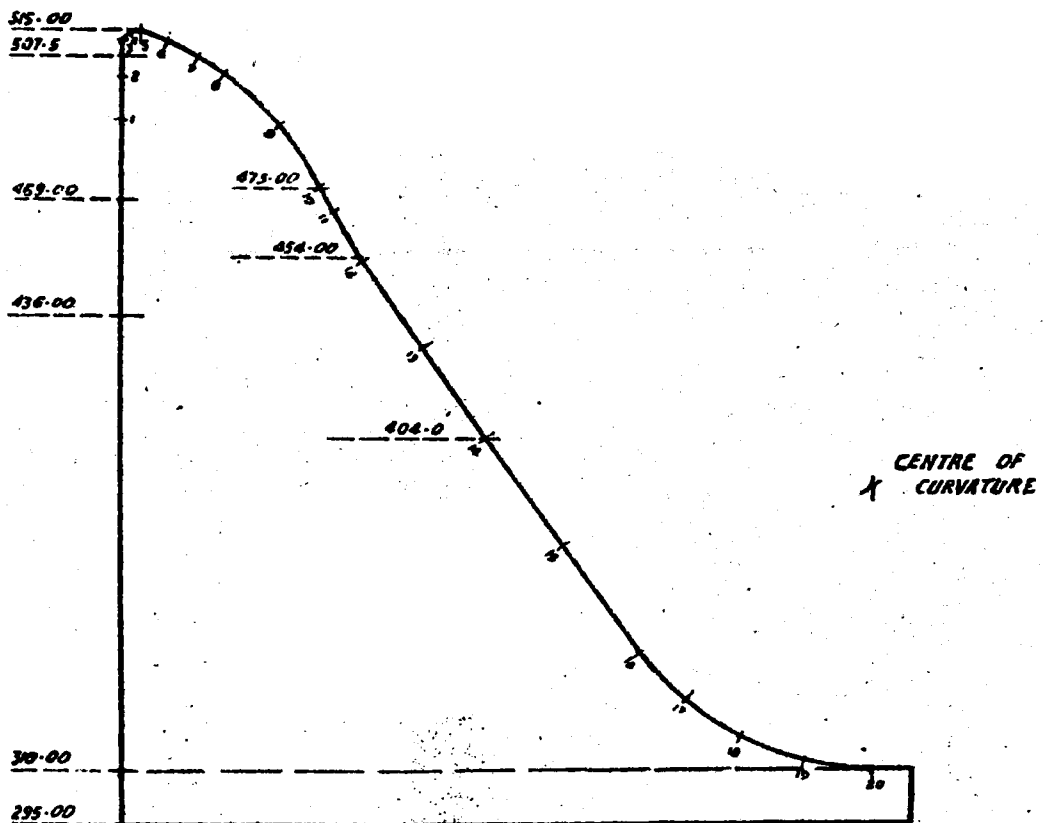


Photo No. 5612—showing the record piezometric points and the 5'0 feet level at the toe, looking upstream.

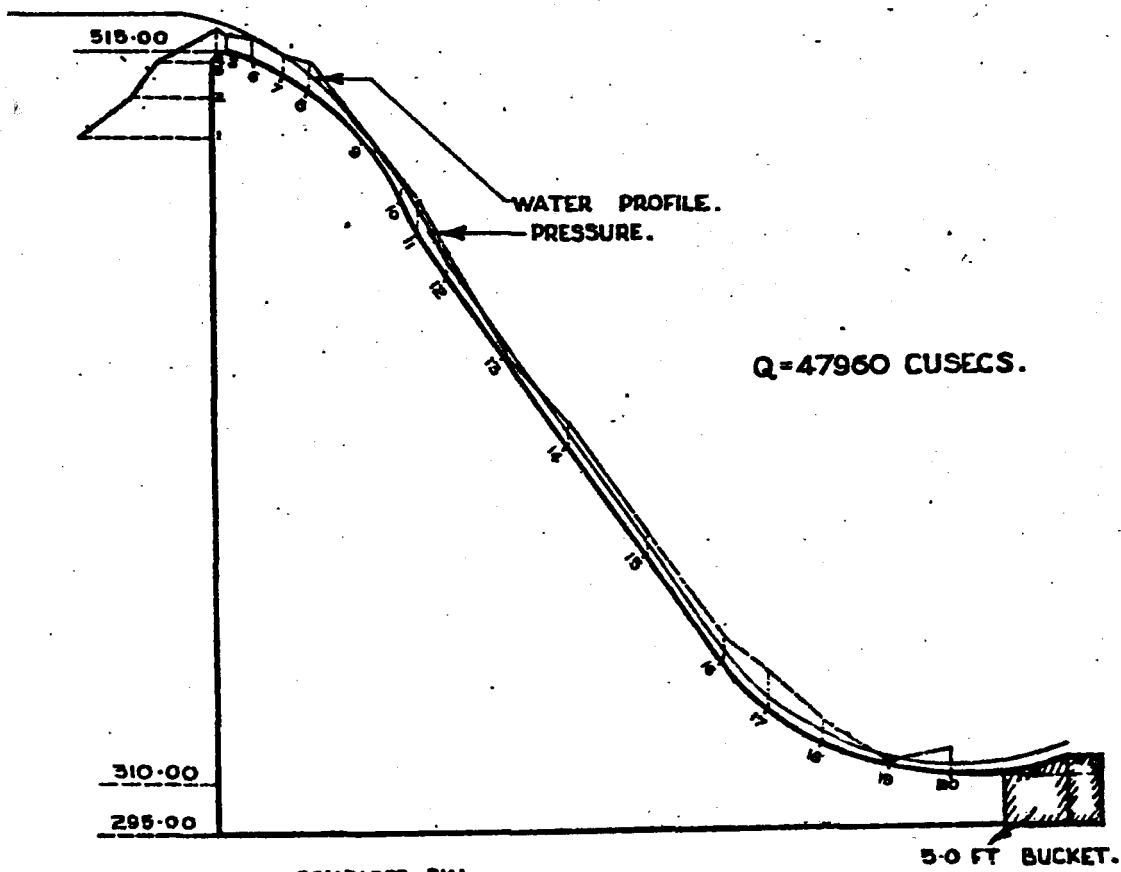
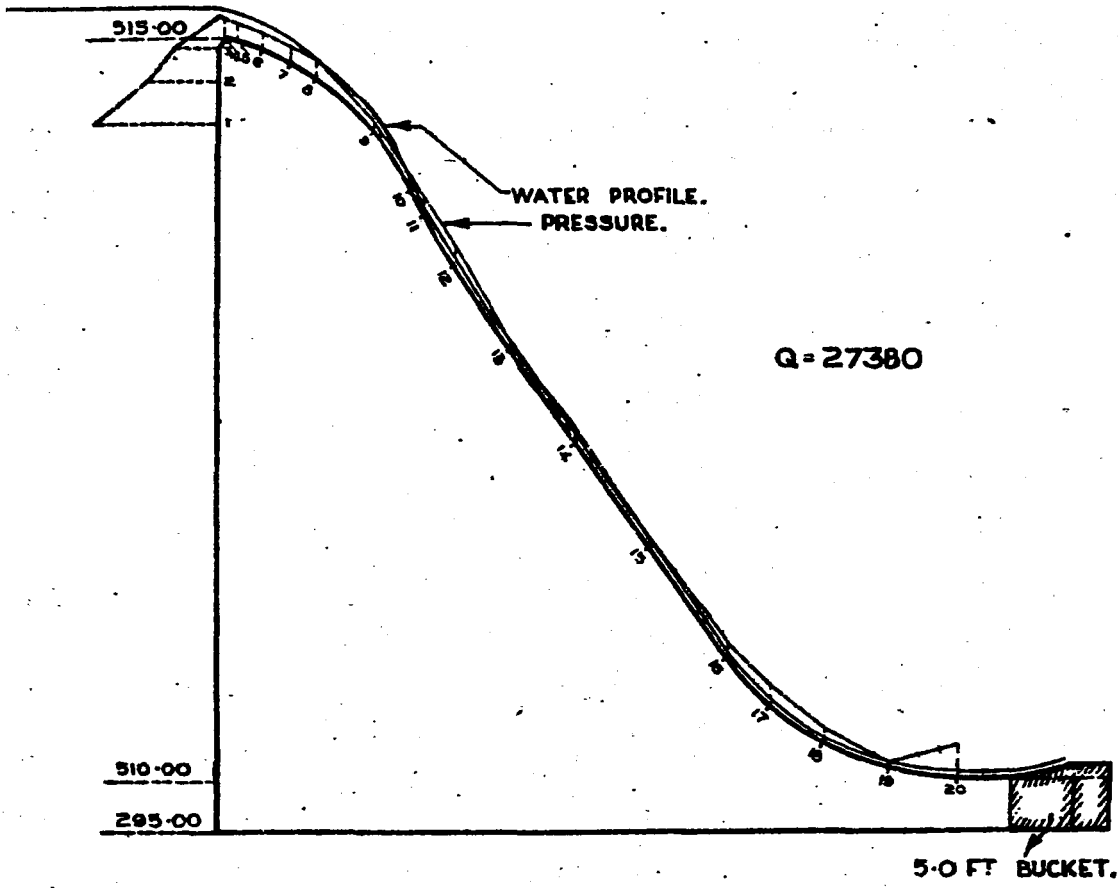
FIG. 74  
SHOWING THE LOCATION OF  
PIEZOMETRIC POINTS ON THE PROFILE OF  
VAITARNA SPILLWAY MODEL.

SCALE. 1" = 50'  
( MODEL: 1/50 )





**FIG: 75**  
**PRESSURE OBSERVATIONS**  
**ON VAITARNA SPILLWAY.**  
**MODEL SCALE: 1/50.**  
**SCALE OF PLOTTING 1" = 50'.**



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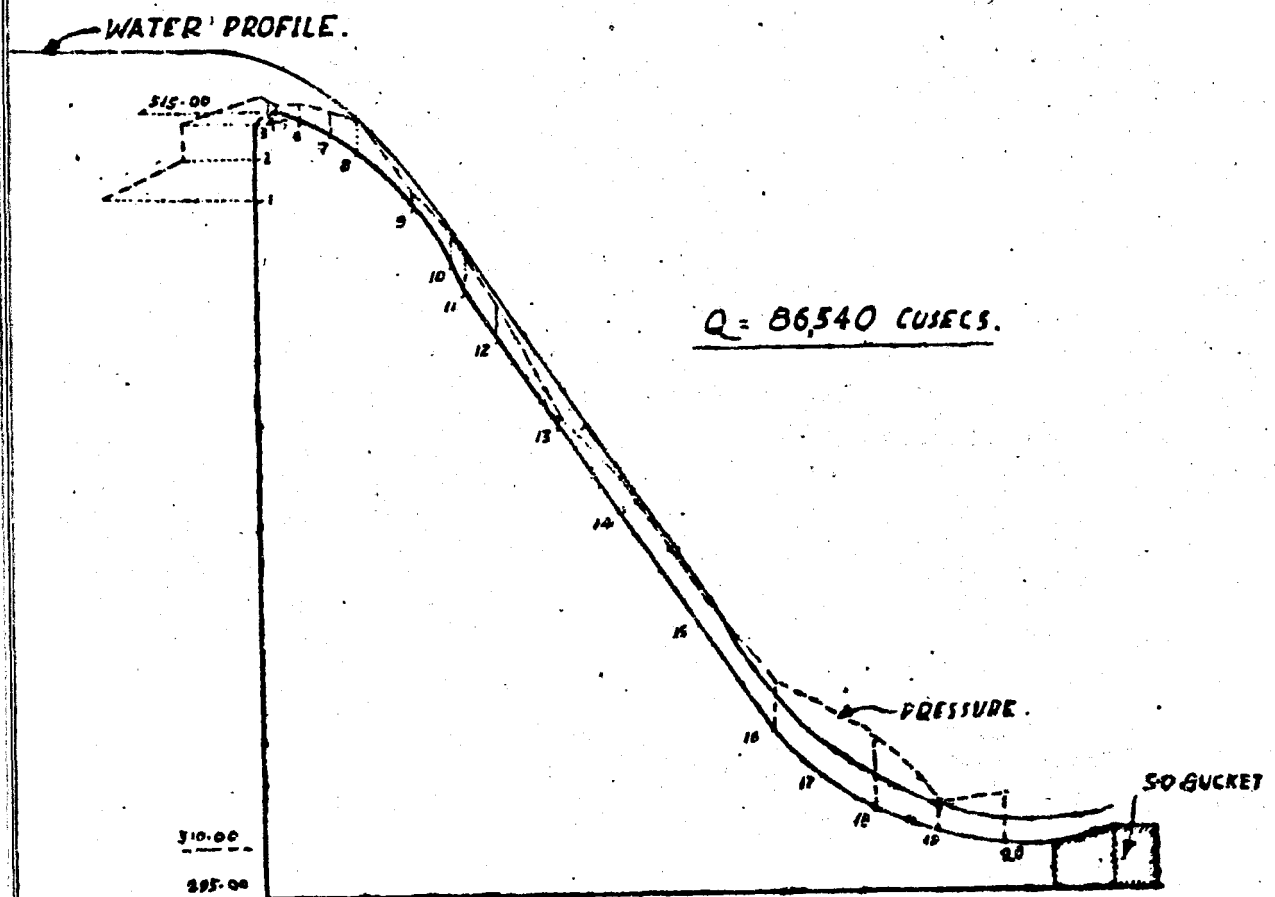
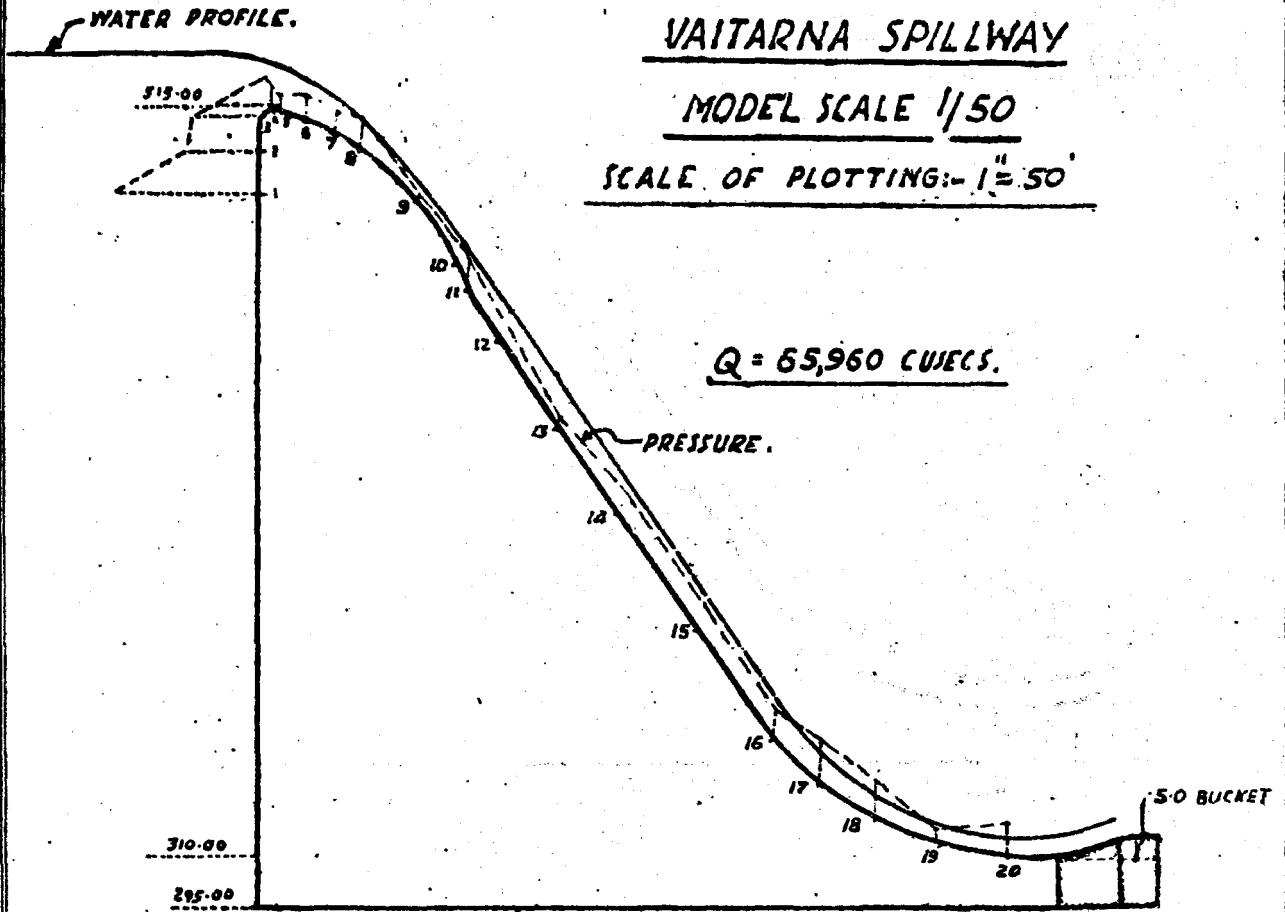
FIG. 75

PRESSURE OBSERVATIONS ON

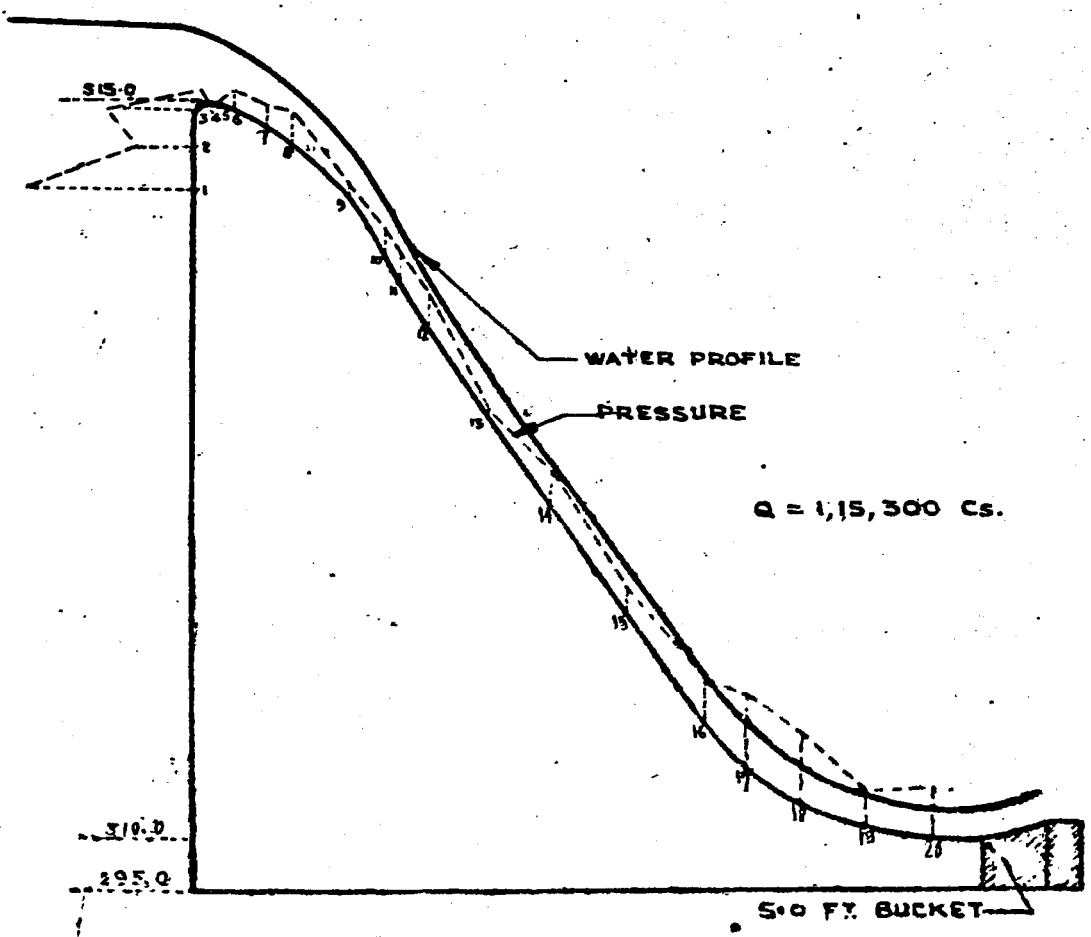
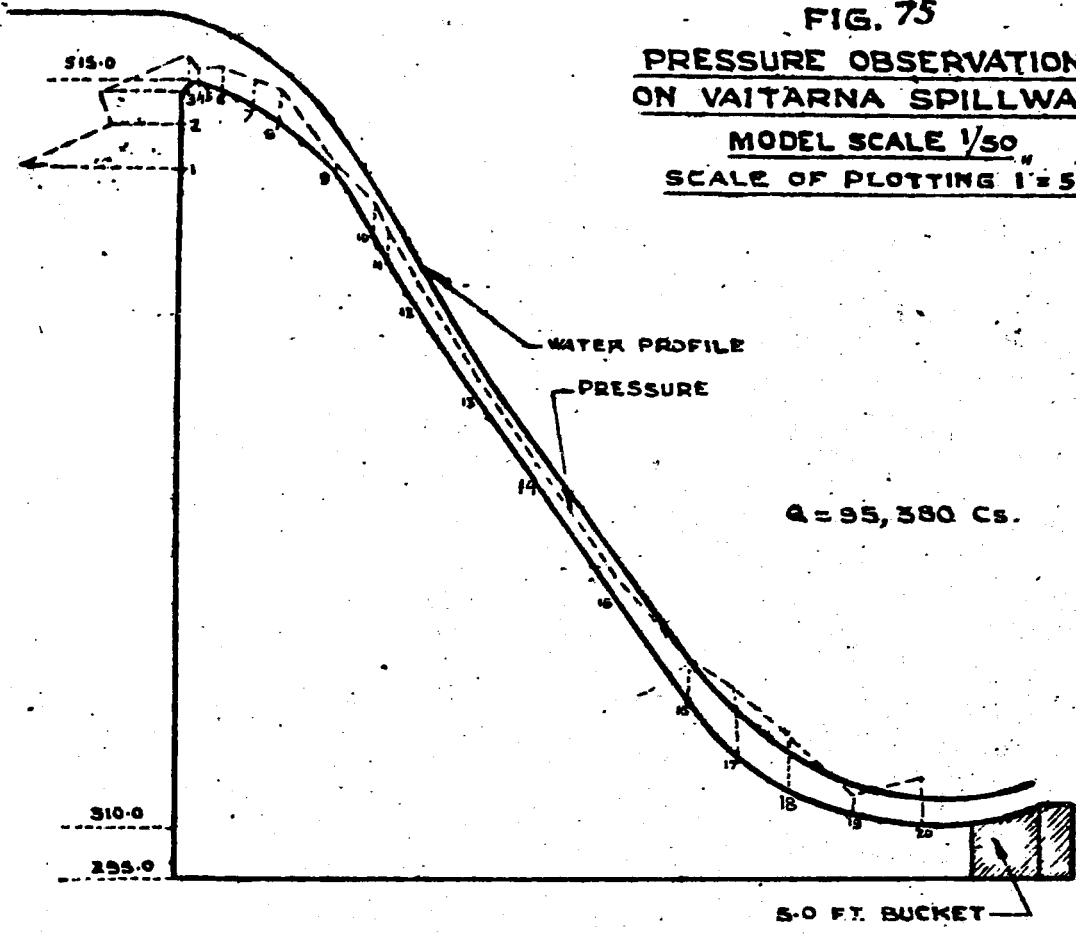
VAITARNA SPILLWAY

MODEL SCALE 1/50

SCALE OF PLOTTING: - 1" = 50'



**FIG. 75**  
**PRESSURE OBSERVATIONS**  
**ON VAITARNA SPILLWAY**  
**MODEL SCALE 1/50**  
**SCALE OF PLOTTING 1" = 50'**



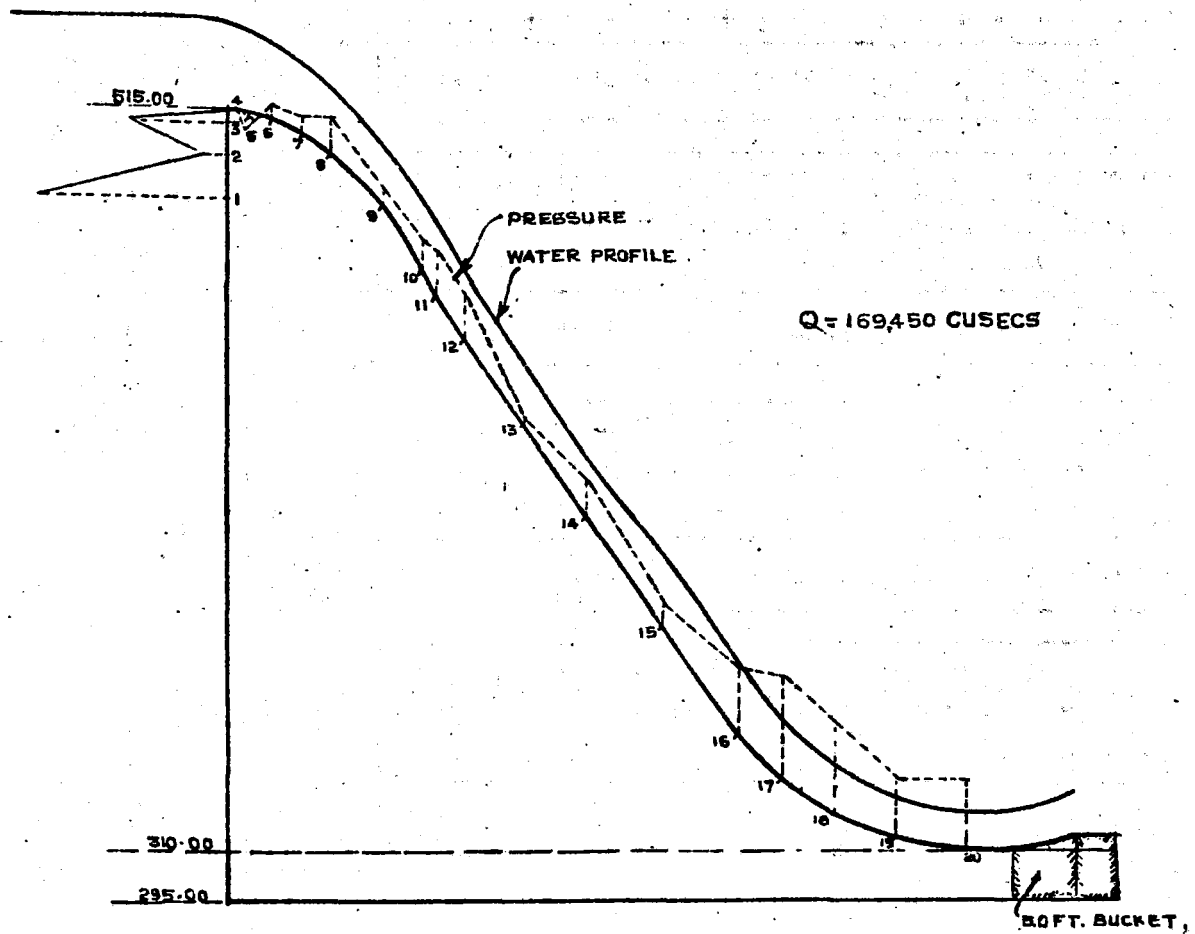
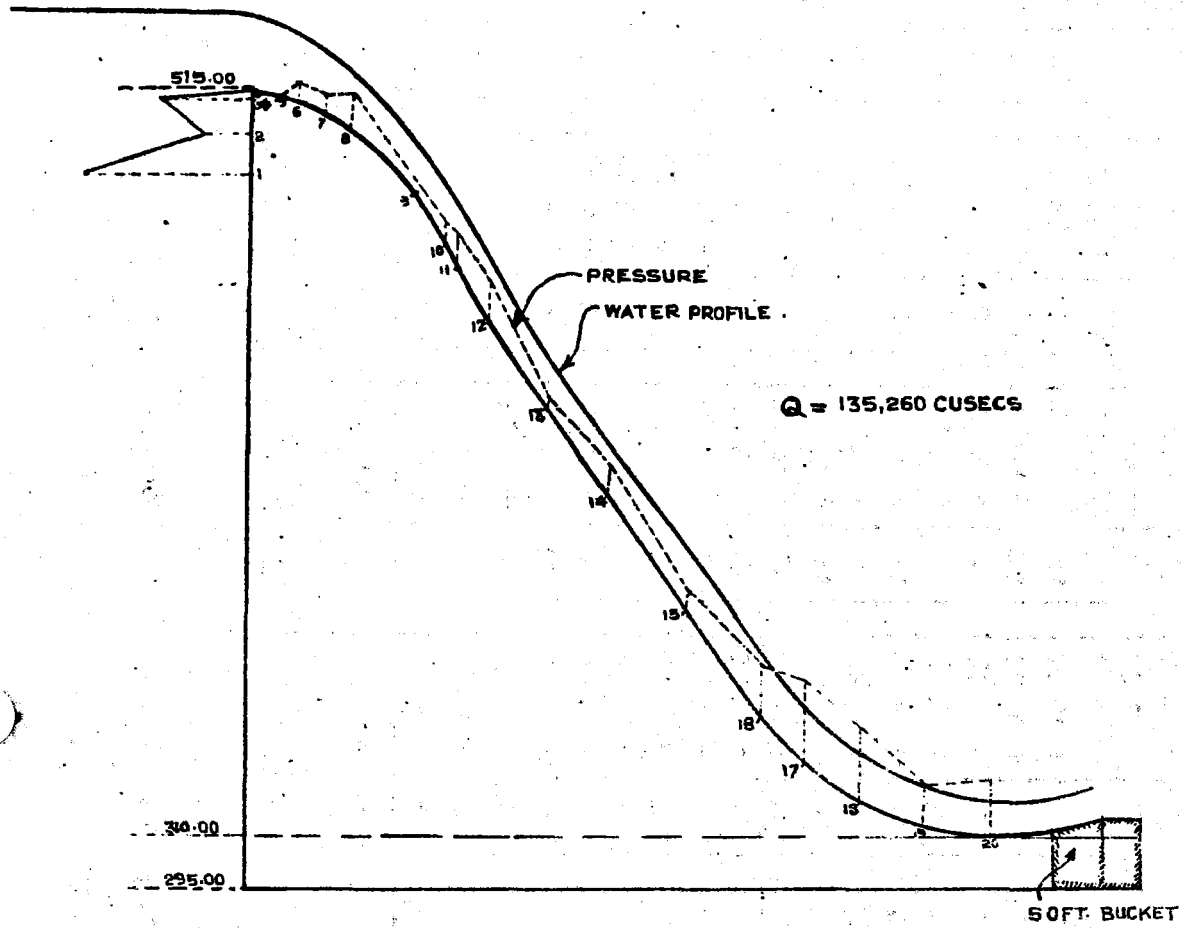
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5.8

FIG. 75

PRESSURE OBSERVATIONS ON VAITARNA SPILLWAY.

MODEL SCALE = 1/50  
SCALE OF PLOTTING = 1" = 50'



TRACED BY: R.K. Kankar  
COMPARED BY: \_\_\_\_\_

# SHEET I

## PRESSURE OBSERVATIONS ON THE PROFILE OF VAITARNA SPILLWAY.

MODEL SCALE 1/50

NO 7806 OF MAR 1950.  
C. W. I. & R. R. 5TH POONA

**TABLE NO 1**  
Q = 27,380  
CUSECS.

PIEZO METER POINTS	PRESSURE IN FEET
1	+35.25
2	+20.25
3	+12.75
4	+8.10
5	+4.95
6	+5.10
7	+3.75
8	+5.50
9	+2.90
10	+5.35
11	+5.90
12	+9.70
13	+0.90
14	+5.80
15	+4.25
16	+3.75
17	+6.05
18	+3.25
19	+1.80
20	+10.80

**TABLE NO 2**  
Q = 47,960  
CUSECS.

PIEZO METER POINTS	PRESSURE IN FEET
1	+38.30
2	+21.30
3	+16.15
4	+8.75
5	+4.45
6	+5.80
7	+4.60
8	+6.90
9	+2.60
10	+5.25
11	+7.15
12	+10.95
13	+7.00
14	+6.95
15	+5.85
16	+5.30
17	+9.95
18	+5.65
19	+1.80
20	+8.00

**TABLE NO 3**  
Q = 65,960  
CUSECS.

PIEZO METER POINTS	PRESSURE IN FEET
1	+40.80
2	+21.05
3	+18.75
4	+8.55
5	+3.60
6	+5.95
7	+4.80
8	+7.15
9	+2.30
10	+5.95
11	+6.80
12	+11.10
13	+1.35
14	+7.50
15	+5.55
16	+7.50
17	+12.70
18	+9.60
19	+2.25
20	+9.70

**TABLE NO 4**  
Q = 86,540  
CUSECS.

PIEZO METER POINTS	PRESSURE IN FEET
1	+44.45
2	+20.25
3	+21.80
4	+7.35
5	+1.80
6	+5.55
7	+9.65
8	+7.40
9	+1.60
10	+6.20
11	+7.55
12	+12.15
13	+1.70
14	+8.45
15	+6.95
16	+9.50
17	+15.45
18	+15.25
19	+6.90
20	+13.85

**TABLE NO 5**  
Q = 95,380  
CUSECS.

PIEZO METER POINTS	PRESSURE IN FEET
1	+45.85
2	+19.30
3	+23.05
4	+9.85
5	+1.60
6	+5.65
7	+5.60
8	+7.95
9	+1.90
10	+7.80
11	+8.00
12	+13.55
13	+4.50
14	+8.25
15	+6.55
16	+11.00
17	+18.05
18	+16.95
19	+6.95
20	+14.60

**TABLE NO 6**  
Q = 115,300  
CUSECS.

PIEZO METER POINTS	PRESSURE IN FEET
1	+47.45
2	+15.75
3	+24.65
4	+4.65
5	-0.20
6	+4.80
7	+4.85
8	+8.10
9	+2.30
10	+6.85
11	+8.10
12	+14.05
13	+1.90
14	+8.65
15	+6.15
16	+11.50
17	+20.24
18	+19.25
19	+8.70
20	+14.30

**TABLE NO 7**  
Q = 135,280  
CUSECS.

PIEZO METER POINTS	PRESSURE IN FEET
1	+49.20
2	+12.85
3	+26.60
4	+1.35
5	+4.05
6	+4.10
7	+4.50
8	+8.85
9	+2.60
10	+7.05
11	+9.95
12	+15.55
13	+2.75
14	+9.15
15	+7.15
16	+13.55
17	+23.10
18	+21.75
19	+10.70
20	+15.75

**TABLE NO 8**  
Q = 169,450  
CUSECS.

PIEZO METER POINTS	PRESSURE IN FEET.
1	+53.10
2	+6.50
3	+29.50
4	-1.20
5	-5.65
6	+3.80
7	+4.20
8	+8.15
9	+2.70
10	+7.60
11	+9.25
12	+17.35
13	+3.25
14	+11.05
15	+7.75
16	+18.60
17	+24.40
18	+27.20
19	+16.70
20	+19.40

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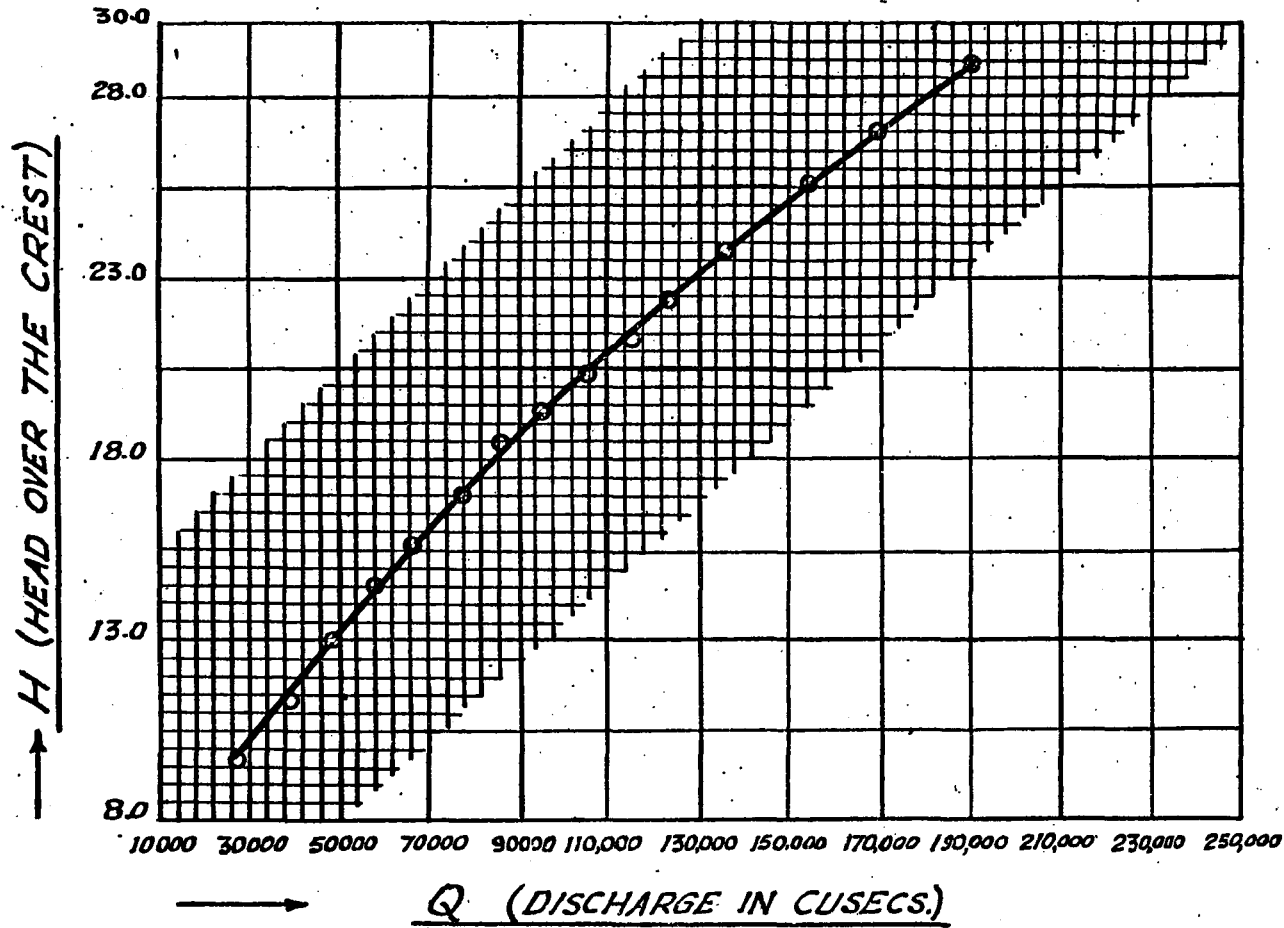
# VAITARNA SPILLWAY MODEL EXPERIMENTS

SCALE = 1:50

PLOTTING OF "Q" (DISCHARGE IN CUSECS) & "H" (HEAD OVER THE CREST)

NO. 7312 OF APRIL 1949  
C. W. I. & N. R. STN. POONA.

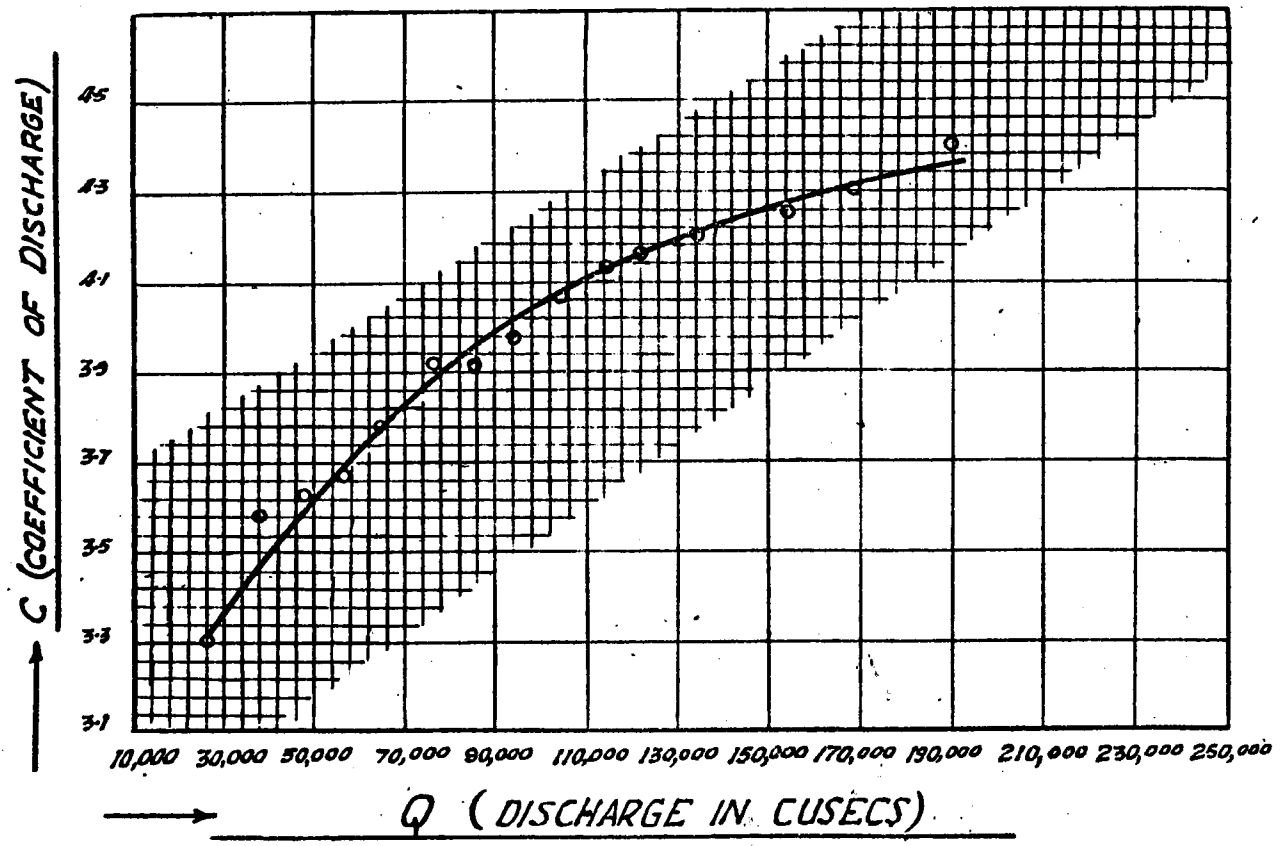
FIG. 76



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FIG. 77  
SPILLWAY MODEL OF PROPOSED VAITARNA DAM  
SCALE: 1/50

PLOTTING BETWEEN COEFFICIENT OF DISCHARGE & DISCHARGE ON PROTOTYPE



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DISSIPATION OF ENERGY BELOW VAITARNA DAM SPILLWAY

(c) *Results of Scour Observations*:—With a 5.0 feet high bucket at the toe five tests with discharge 95,380, 86,540, 76,980, 65,960 and 38,580 cusecs with downstream water-levels corresponding to the value of the respective discharges were carried out to make a qualitative study of the extent of scour downstream. Pebbles of  $\frac{1}{4}$ " to  $\frac{1}{8}$ " diameter were used in the bed which was originally moulded at R.L. 310.00 and scour observations were recorded for each test when the scour stabilised. Observations of water profile and scour profile downstream are plotted in fig. 78 and the results are tabulated below:—

Fig. 78.

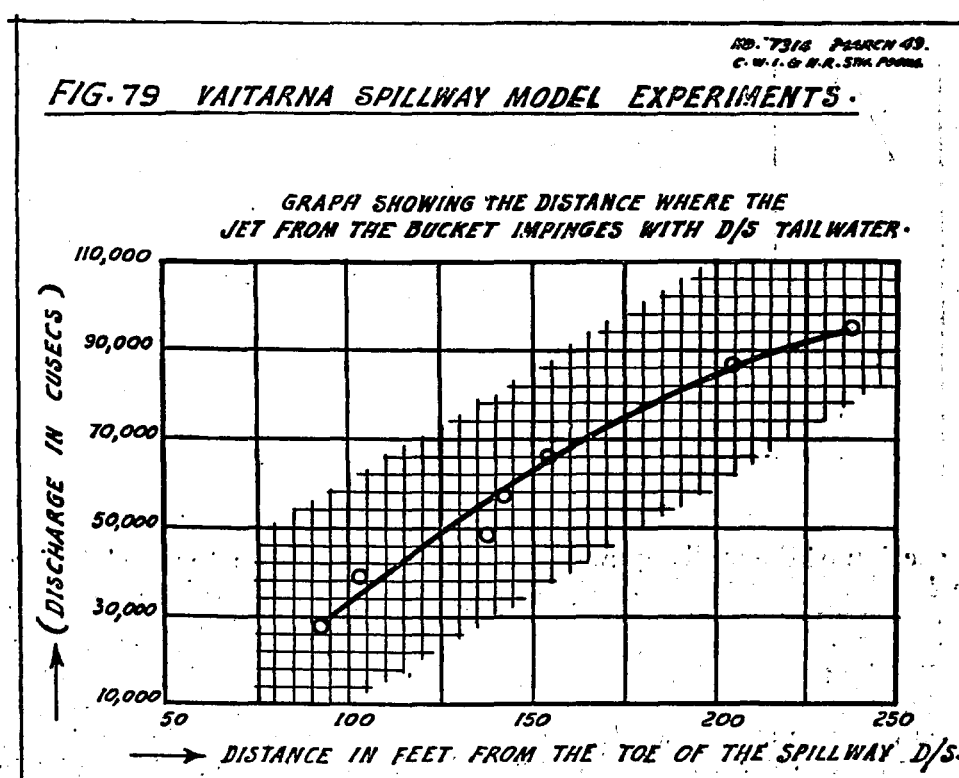
TABLE 1

Test No.	Discharge in cusecs.	Distance in feet from toe of bucket where maximum scour occurred.	Maximum depth of scour in feet below river bed R. L. 310.
I	95,380	320.0	40.0
II	86,540	310.0	38.0
III	76,980	270.0	35.5
IV	65,960	240.0	27.5
V	38,580	150.0	14.0

The downstream water level in each test was kept corresponding to the respective value of discharge run. The table shows the effectiveness of the bucket under various discharges and the position of scour-hole of maximum depth.

(d) *Position where the Upper Face of the Jet impinges on Water*:—As automatic gates may be installed at a later stage, varying discharges will flow over the weir till that time, so the position where the shooting jet first comes into contact with the tail-water level was noted for discharges varying from 20,000 to 1,00,000 cusecs and its distance from the toe of the bucket was recorded. A curve was drawn between Q and the horizontal distance from the toe where the jet impinged (Fig. 79). From this figure it will be seen that for a discharge of 92,000 cusecs the upper surface of the jet impinges on the tail water at a distance of 225 ft. from the toe of the bucket.

Fig. 79.





1.4. From the results of (c) and (d) it was noticed that the bucket seemed to be an effective and economical measure for securing bed protection downstream of the weir. But as the river turns sharply to the left side downstream of the proposed bridge which is at a distance of 600 ft. from the vertical axis of the dam, it was considered essential to test the effectiveness of the bucket on a full width model with particular reference to the downstream flow conditions; these tests are described in Part II.

2. PART II

2.1. In Part II a study was made on  $\frac{1}{60}$  scale model of the entire spillway for testing the efficiency of the 5.0 feet bucket. A method of effective dissipation of energy by means of splitters was also developed.

2.2 *Introduction.*—It was considered essential to test the effectiveness of the bucket in a full-width model representing the river bed contours both upstream and downstream. G. S. Model of the entire spillway section having a length of 322 feet including three piers of 14.0 feet each was, therefore, constructed in 8.0 feet wide channel to a scale of  $\frac{1}{60}$  representing bed contours of the river to the extent of 600 feet and 1,600 feet towards upstream and downstream respectively. Figure 80 gives

Fig. 80.  
Photo No.  
5606.

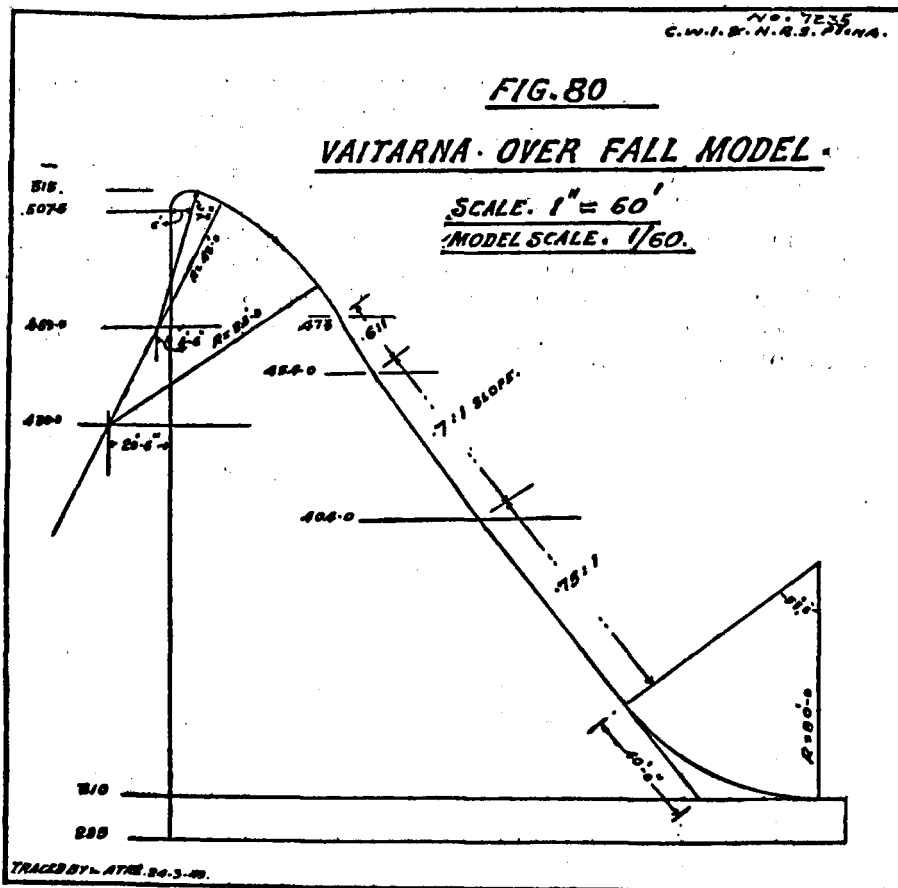


Fig. 81.

the section of the spillway and Photo No. 5606 shows the model with a five feet bucket at the toe. For accommodating the flow into the narrow bed of the river the downstream guide walls which were considered essential for the purpose were moulded according to the original design supplied by the Special Engineer, Bombay Corporation and are shown in Fig. 81 in dotted line. A maximum discharge of 3.3 cusecs = 92,000 cusecs was run and downstream flow conditions were observed. The water profile over the spillway and bucket was taken and the plotting

FIG. 78

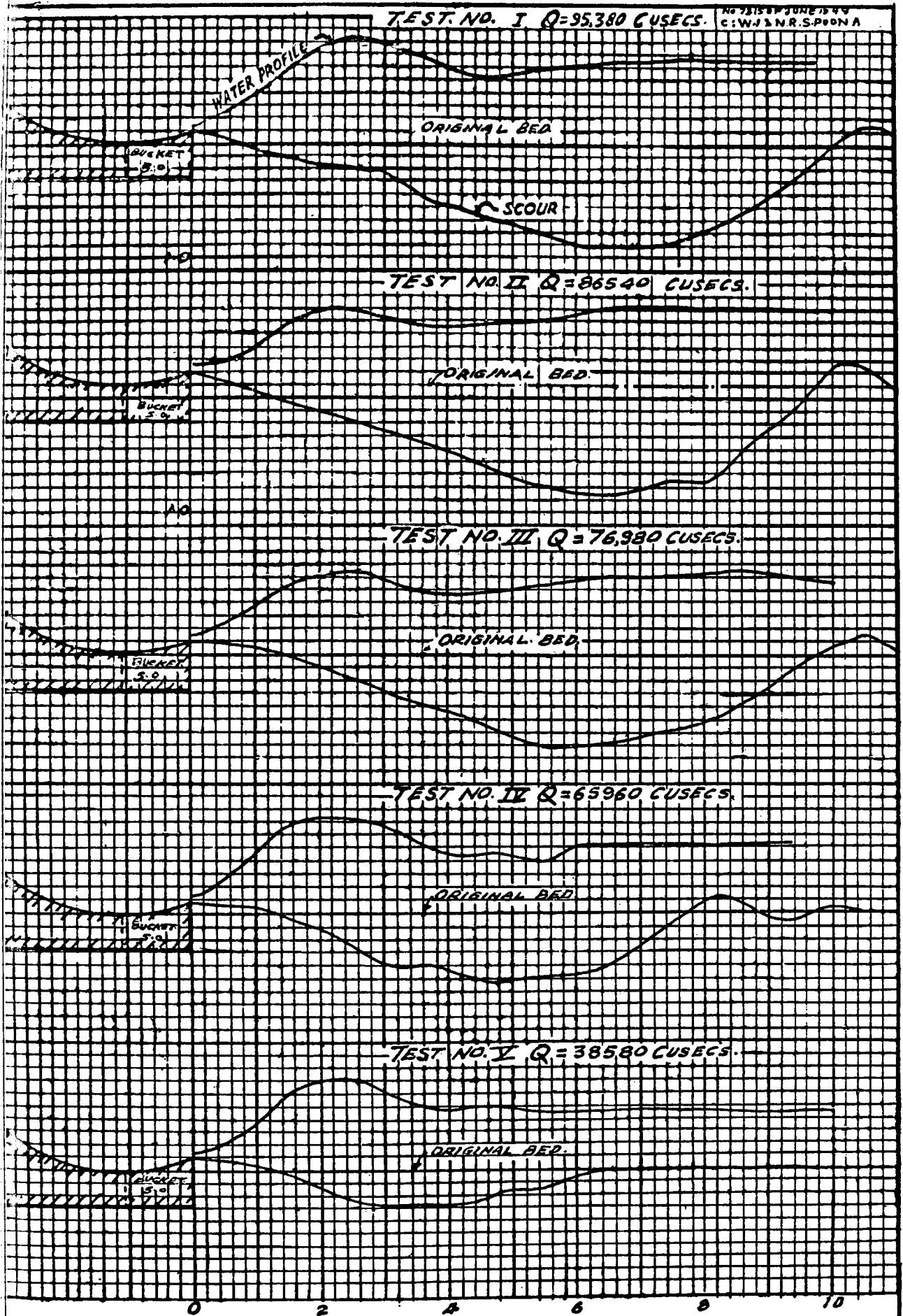


FIG. 78 VAITARNA SPILLWAY MODEL EXPTS. SCALE  $\frac{1}{50}$   
SCOUR & WATER PROFILES D/S

SCALE - HOR.  $1'' = 2' \approx 100'$   
VER.  $1'' = 1' \approx 50'$

B.K. ATRE.

## DISSIPATION OF ENERGY BELOW VAITARNA DAM SPILLWAY

is shown in fig. 82. Due to the curved course of the river downstream the central shooting flow from the bucket attacked the right bank and great turbulence was observed as is shown in Photo 5619. The bucket was considered to be unsuitable due to the turbulent flow near the site of the proposed bridge downstream, possibility of erosion on the right bank, deep scour on sides resulting from the back rollers caused by guidewalls and other geologic conditions of the dam site. The bucket was removed and the following alternative device was adopted.

Fig. 82.

Photo No.  
5619.

2.3. *Use of Splitters and their Action:*—Experiments were carried out with horizontal blocks known as “Splitters” fixed on the downstream face of the spillway with equal spaces between them and having a continuous ledge below them. These splitters break the flowing sheet of water into a number of small streams and the section of the ledge below them is to create two “boil streams” from such “space stream” and bring about their mutual interference resulting in an air-entrained mass which sprays down as rain drops. This reduced the scour to a great extent in the vicinity of the dam but to secure maximum protection for the bed and also to shift the scour further away from the toe of the dam a new design of splitters henceforth called “Bucket-type-splitters” was evolved by experimentation. They served a two-fold purpose i.e. breaking the flowing sheet of water into a number of small streams of reduced intensity and also as buckets to throw these streams sufficiently away from the toe of the spillway.

2.4. *Experiments for Dissipation of Energy:*—Various designs of splitters were tested at different levels on the face of the spillway. The space between the splitters was varied and the position of the ledge was also tested at different distances below them to get at the optimum distances for ensuring maximum aeration.

Horizontal splitters were first tested at R. L. 450 (top) (Crest R. L. 515) when the sheet of water entrained by air fell at the toe of the dam at R. L. 310. This naturally caused maximum scour near the toe. The position was later shifted downwards to R. L. 380 and the maximum scour took place at 60.0 feet downstream of the toe. If the splitters were still moved below R. L. 380 there would not have been sufficient space below the nappe to aerate the mass of water falling in the form of spray. So, bucket-type-splitters were tried. When they were tested at the former level i.e. R. L. 450 it was found that the maximum scour was still near the toe; the splitters were shifted to R. L. 380 and the maximum scour observed was about 200 ft. away from the toe and was considered safe enough having regard to the geological conditions at the site.

It was found that bucket-type-splitters of length 10.0 ft. and radius 20.0 feet fixed at R. L. 380 with the ledge of same design at R. L. 370 as shown in fig. 83 and Photo No. 5647 gave satisfactory results in respect of their greater throw off. Photo 5640 shows the running conditions of the model.

Fig. 83.  
Photo No.  
5647.  
Photo No.  
5640.

With a view to localise the flow within the narrow gorge of the river downstream, tests were performed with sidewarping. The position and extent of the sidewarps on the profile of the spillway are shown in fig. 81, after paying due consideration to their effect on aeration of splitters and the increase of flow per foot run in the side spans which would create great scour below, near the sides of the river bed. Twenty bucket-type-splitters were fixed at R. L. 380, the width of each splitter being 6.0 ft. and the free space between two was kept equal to 8.0 feet Photo No. 5647 shows the position of the splitters with continuous ledge below them and the sidewarping.

Fig. 81.

Photo No.  
5647.

2.5. *Tests with Side Warping:*—By warping the sides it was observed that due to the decrease in width of the two side spans the flow concentration was increased therein which would result in more scouring action near the guide walls downstream than in the centre and in order to overcome this two side-buckets, the design and dimensions of which are given

Fig. 83. in fig. 83, were fixed on each side on the warped walls at the position shown in Photo 5669. Photo 5642 shows the action of the side-buckets fixed on left side warped wall. The technique adopted in designing them and locating them at effective positions is such that the lower side-bucket comes into action at low discharges and converges the side flow of increased depth inwardly as well as throws it clear of the spillway in the form of a parabolic thin sheet of water falling over the spraying mass while the upper side bucket comes into a similar action at high discharges and also brings about more aeration around the splitters. The function of these four side-buckets is to bring about a uniform distribution of flow by converging towards the centre the flow from the two side spans. The scour pattern obtained downstream was nearly uniform.

Fig. 81.

2.6. *Downstream Guidewalls*:—For stabilising the flow from spillway length of 322.0 feet to the narrow width of 100-150 feet of the river downstream four different designs of guidewalls were tested in addition to the original design supplied by the Bombay Corporation which was found unsuitable. Figure 81, shows the different designs of curves adopted for experimentation. From a study of the flow conditions and scour pattern downstream obtained in the bed of the river which was originally moulded at R. L. 310 with pebbles of diameter  $\frac{1}{4}$ " to  $\frac{1}{8}$ ", the fourth design of guidewalls which are straight up to 125 feet—which length was necessary to ensure proper aeration from below—followed by hyperbolic contraction curve determined by the relation

$$B_x = \frac{B_o B_s L}{L B_s - (B_s - B_o)x}$$

where L = Length of contraction,

B<sub>o</sub> = Overall-throat width,

B<sub>s</sub> = Bed width of channel,

B<sub>x</sub> = Width of contraction at any distance x from the end of contraction,

gave better results comparatively because the side rollers were completely eliminated and the flow conditions downstream were normal. As the bed of the river is composed of natural rock the area within the range of these straight and hyperbolic guide walls as indicated in fig. 81A will have to be excavated to R. L. 310.

Fig. 81 A.

see fig. 81A  
page 251)

2.7. *Pressures around a Bucket Type Splitter*:—It was considered essential to study the distribution of pressure around a Bucket-type-splitter and also to observe scour and downstream flow conditions with the maximum expected discharge of 140,000 cusecs. This maximum discharge has been recalculated after making allowance for unexpected high floods which may occur due to abnormal rainfall in a cyclonic storm as recently, in the case of adjoining Tansa Catchment, unexpected high flood occurred due to a cyclonic storm on 22nd November 1948.

2.8 *Pressures*:—For finding out the pressures around a bucket-type splitter experiments were carried out in 1/50 G. S. Model on which the splitters were fixed at R. L. 380.0 with the ledge at R. L. 370.0, the crest of spillway being at R. L. 515. Piezometric observations of pressures were recorded by means of a U-tube manometer in each of the four tests run with the following discharges—

Test 1	Q = 50,000 cusecs.
Test 2	Q = 92,000 "
Test 3	Q = 120,000 "
and Test 4	Q = 140,000 "

Table 2.

Table 2 shows the results of pressure observations. In Test 4 with Q=140,000 cusecs it was found that negative pressures of the order of 7.20 ft. occurred on the right and left sides of the splitter and these could be recorded by the U-tube method.

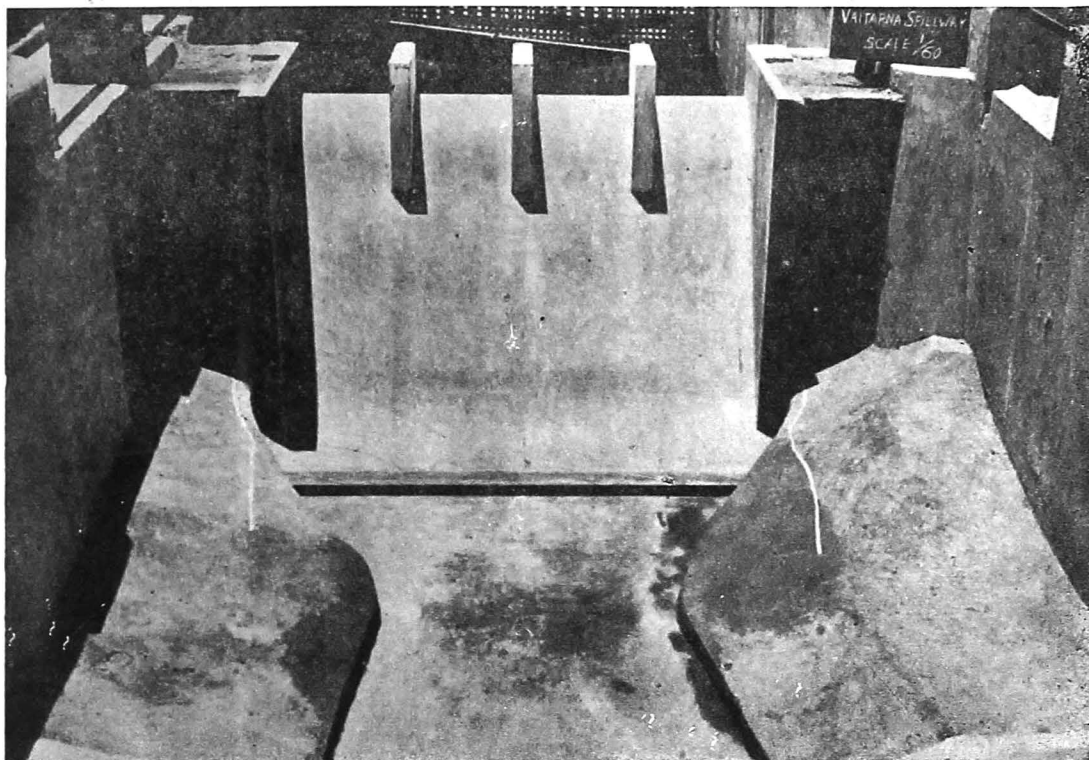
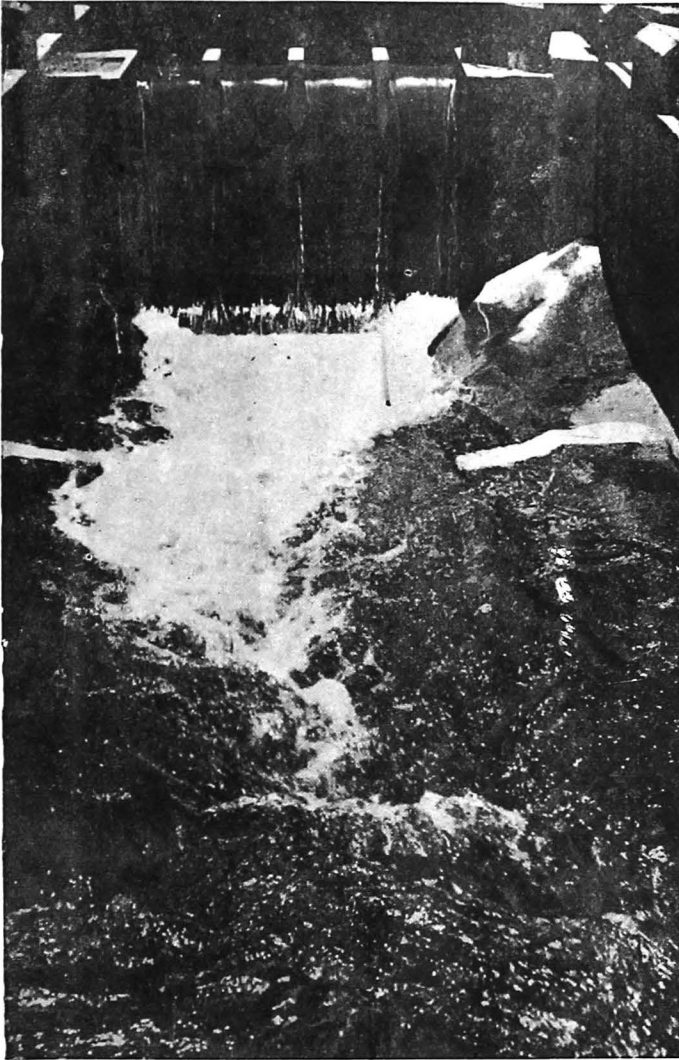
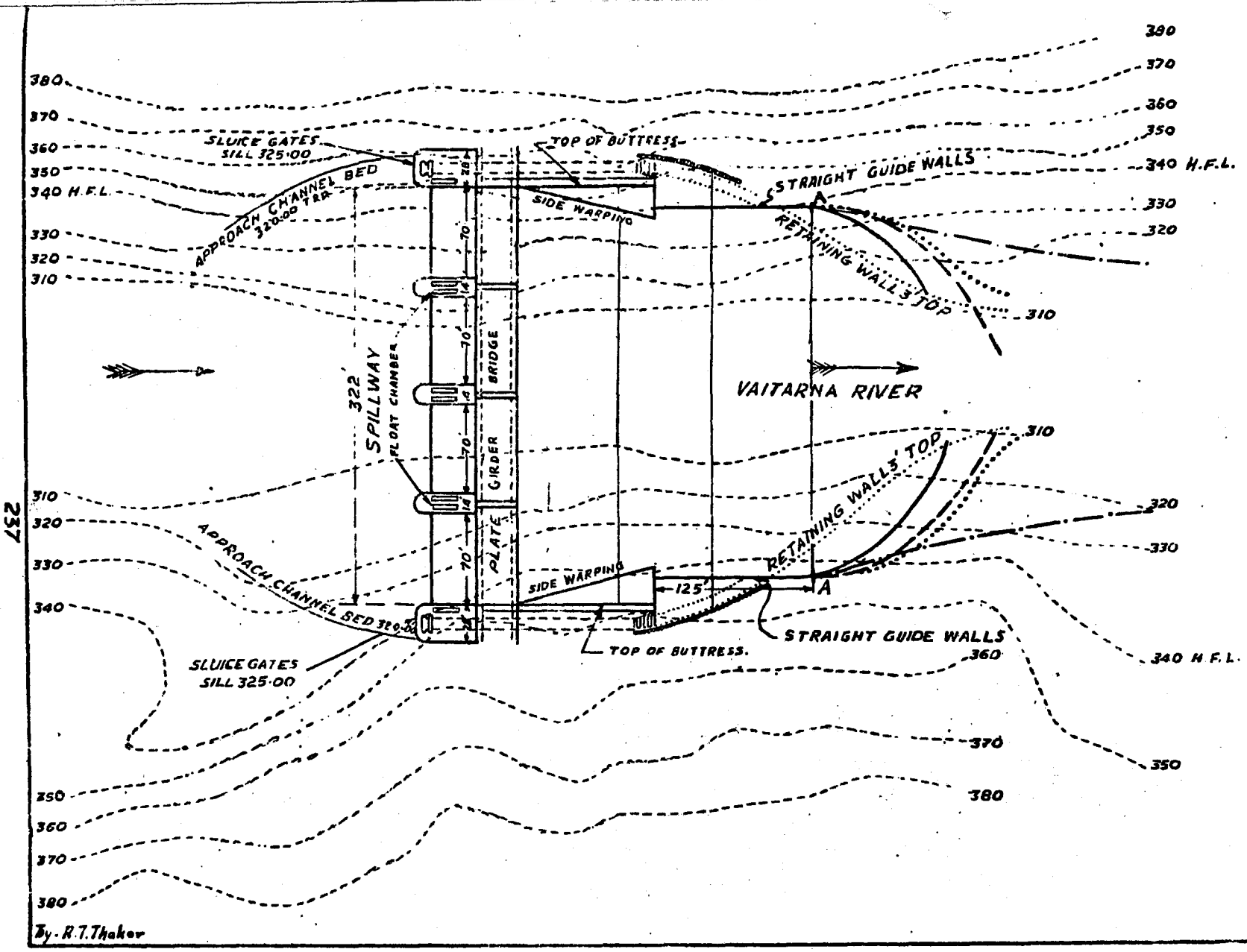


Photo No. 5606 - Shows the model with a 5 feet Bucket at the toe.



Proposed bridge site.

Photo No. 5619—Showing the action of 5'0 feet bucket and the attack of the shooting flow on the right bank downstream.  $Q \cong 92,000$  cusecs, looking upstream.



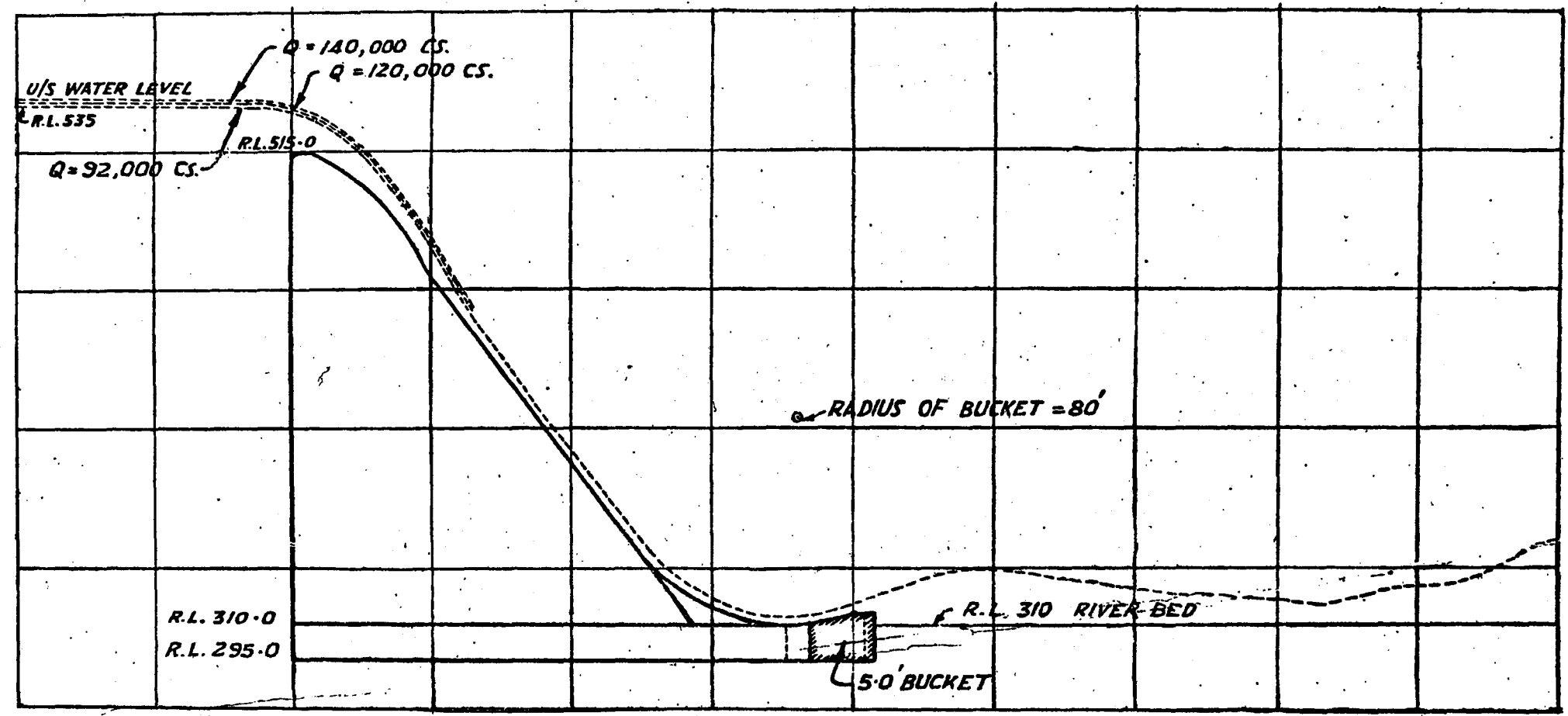
**FIG. 81**  
**SHOWING SIDE WARPING**  
**AND**  
**DESIGNS OF DOWNSTREAM**  
**GUIDE WALLS.**  
**PLAN OF PROPOSED DAM AT VAITARNA.**  
**SCALE - 1"=100'**

- Ref.:-**
- ORIGINAL DESIGN OF GUIDE WALLS AS PROPOSED BY THE SPECIAL ENGINEER BOMBAY MUNICIPALITY. -----
  - OTHER DESIGNS TESTED IN THE MODEL.
  - 1 ST DESIGN. -----
  - 2 DESIGN. -----
  - 3 DESIGN. -----
  - 4 DESIGN WITH HYPERBOLIC CONTRACTION CURVE. -----

TRACED FROM BOMBAY MUNICIPALITY  
VAITARNA CUM TANSA SCHEME DRG. NO. V.T./144

By R.T. Thakur

FIG. 82  
VAITARNA SPILLWAY MODEL SCALE 1/60  
SCALE OF PLOTTING 1" = 1' = 60'  
PLOTTING SHOWING WATER PROFILE WITH DISCHARGES OF = 92 000 CS.  
= 120,000 CS.  
= 140,000 CS.

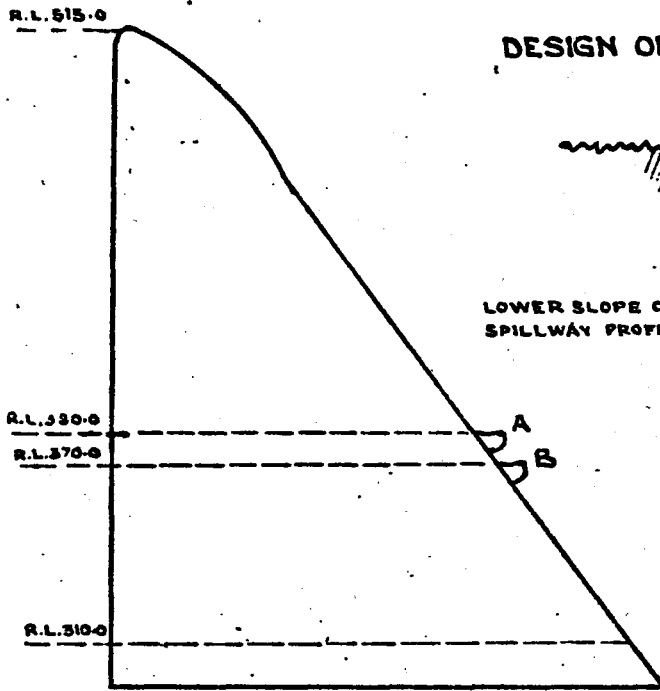


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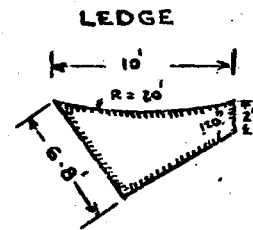
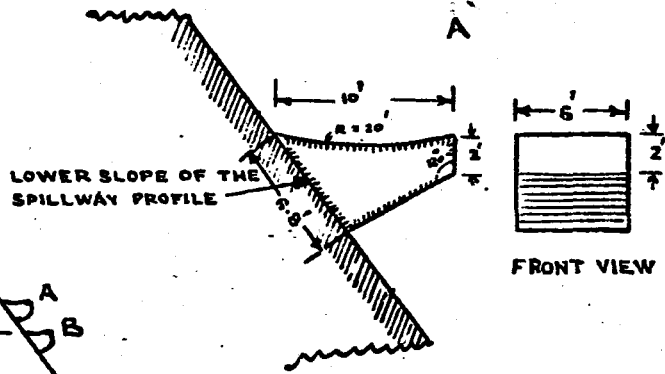


**FIG. 83**

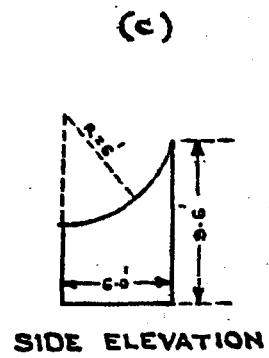
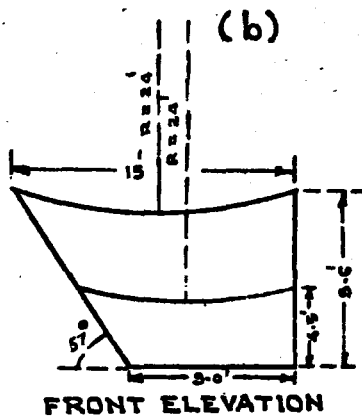
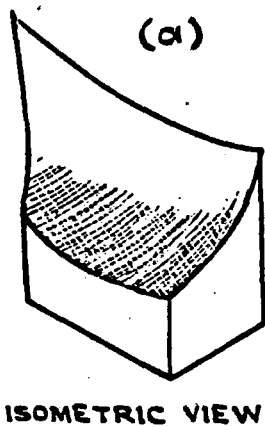
**VAITARNA SPILLWAY**  
MODEL SCALE  $\frac{1}{60}$   
SCALE 1" = 60'



**DESIGN OF A BUCKET-TYPE-SPLITTER**  
SCALE 1" = 10'



**DESIGN OF A SIDE BUCKET**  
SCALE 1" = 10'



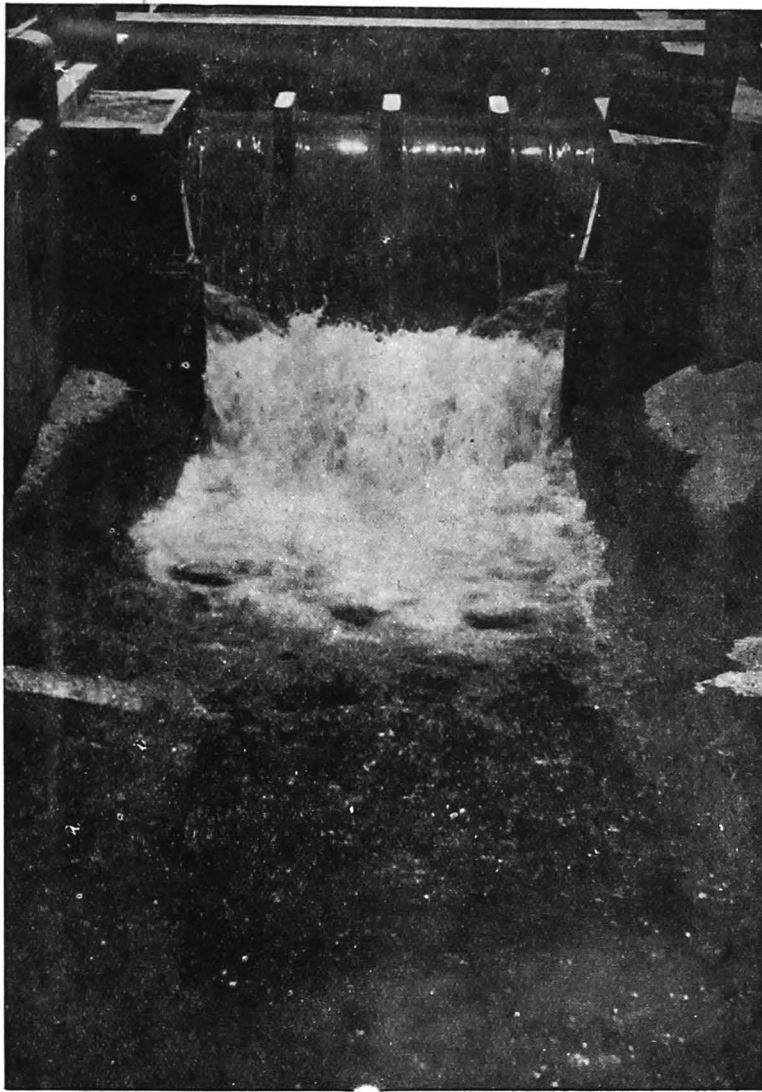


Photo No. 5640—Showing the downstream conditions, looking upstream with  $Q \cong 92,000$  cusecs.

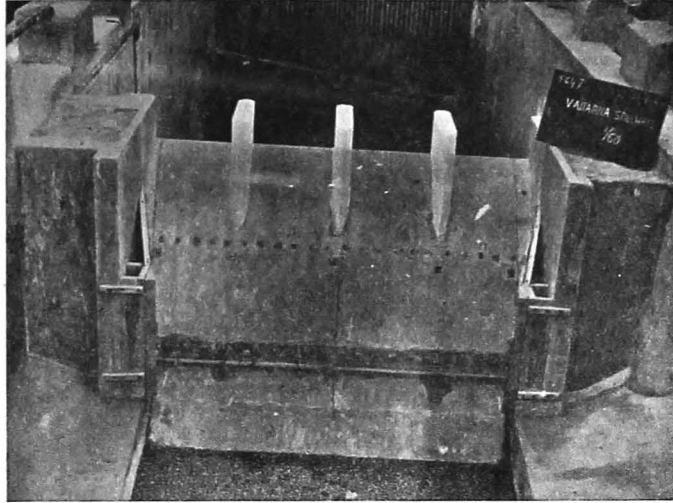


Photo No. 5647—Shows the position of splitters with cotinuous ledge below and side warping.

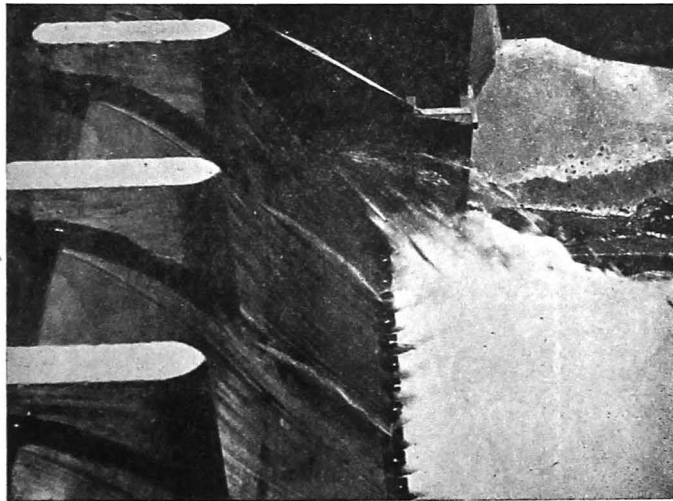


Photo No. 5642—(Side view) Showing action of a side bucket and the Bucket-type-splitters.

29. *Scour Downstream of Spillway*:—Six tests with depths equal to 8, 10, 12, 16, 20 and 24 ft. over the spillway crest were conducted and the model was run till scour stabilised. Photo 5671 shows the model in operation and the downstream flow conditions when a maximum discharge of 140,000 cusecs was run. The flow downstream was quite normal after the energy had been dissipated. Table 3 gives the maximum depth of scour which occurred in the centre and the horizontal distance in feet from the toe of the spillway to the spot where the maximum scour occurred, while Fig. 84 shows the plotting of central scour profile for each of the six tests. The river bed in the model was represented by sand to study the relative scour for various tests though there is rock on the bed on the prototype. The model showed the maximum depth of central scour = 23 feet at a distance of 200 feet away from the toe of the dam which is well away. A detailed scour pattern downstream in case of test 6, i.e. with  $Q=140,000$  cusecs is shown in Fig. 85.

Photo 5671.

Table 3.

Fig. 84.

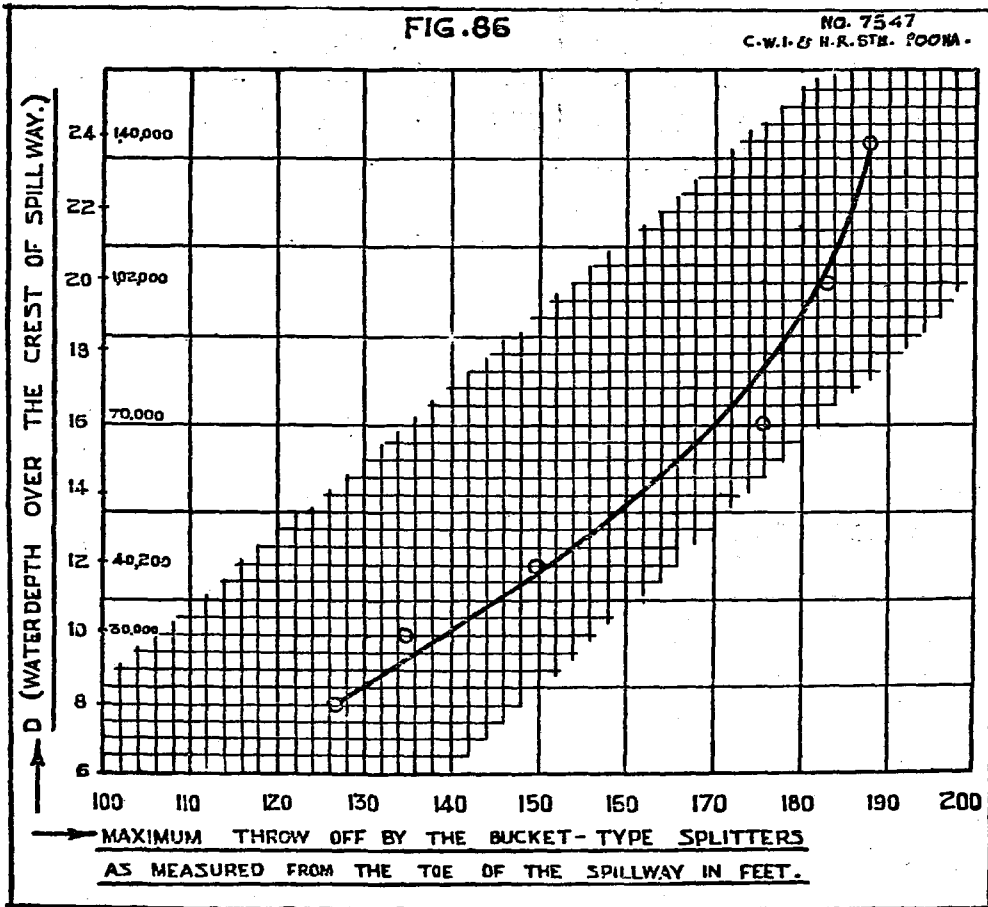
Fig. 85.

*Maximum Throw-off by the Bucket-Type-Splitters*

In order to see the efficiency of the bucket-type-splitters under various upstream flow conditions the model was run with  $D=8, 10, 12, 16, 20$  and 24 feet over the crest and the position where the upper profile of the jet comes into contact with downstream water was recorded for each test. The horizontal distance in feet showing the throw-off by the splitters against values of  $D$  is plotted in fig. 86.

Fig. 86.

The design of bucket-type-splitter will now be tested against the danger of cavitation in a vacuum tank by investigating the pressure distribution under varied conditions of flow.



**TABLE 2 & 3**

**RESULTS OF PRESSURE OBSERVATIONS  
AROUND A BUCKET TYPE SPLITTER.**

**TABLE 2.**

S. NR	TEST NR	DISCHARGE IN CUSECS	PRESSURE IN FEET AROUND A SPLITTER + ABOVE ATMOSPHERE - BELOW ATMOSPHERE				
			UPPER SURFACE	FRONT SURFACE	LEFT SIDE	RIGHT SIDE	LOWER SURFACE
1	I	50,000	+ 55.60	+ 5.41	+ 2.90	+ 2.90	+ 1.25
2	II	92,000	+ 63.45	+ 6.60	+ 3.30	+ 3.30	+ 1.60
3	III	120,000	+ 88.10	+ 6.60	+ 4.16	+ 4.16	+ 3.54
4	IV	140,000	+ 98.55	+ 6.60	+ 7.75 - 7.20	+ 7.75 - 7.20	+ 4.58

**TABLE 3.**

S. NR	TEST NR	HEAD OVER CREST IN FT.	DISTANCE IN FEET FROM TOE OF SPILLWAY WHERE MAX. SCOUR OCCURRED	MAX. DEPTH OF SCOUR IN FEET.
1	I	8.0	113.0	15.0
2	II	10.0	141.0	20.4
3	III	12.0	172.8	21.6
4	IV	16.0	187.0	21.9
5	V	20.0	193.0	22.2
6	VI	24.0	200.0	23.0

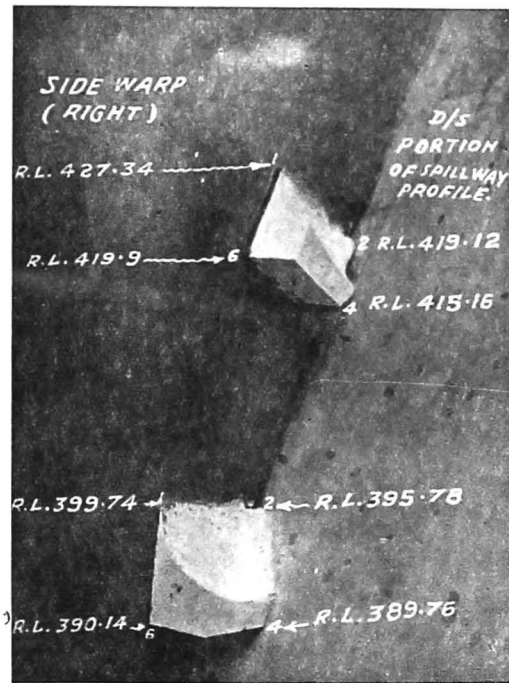


Photo No. 5669—Showing the position of the two side-buckets on the right looking sideways along the glacis.

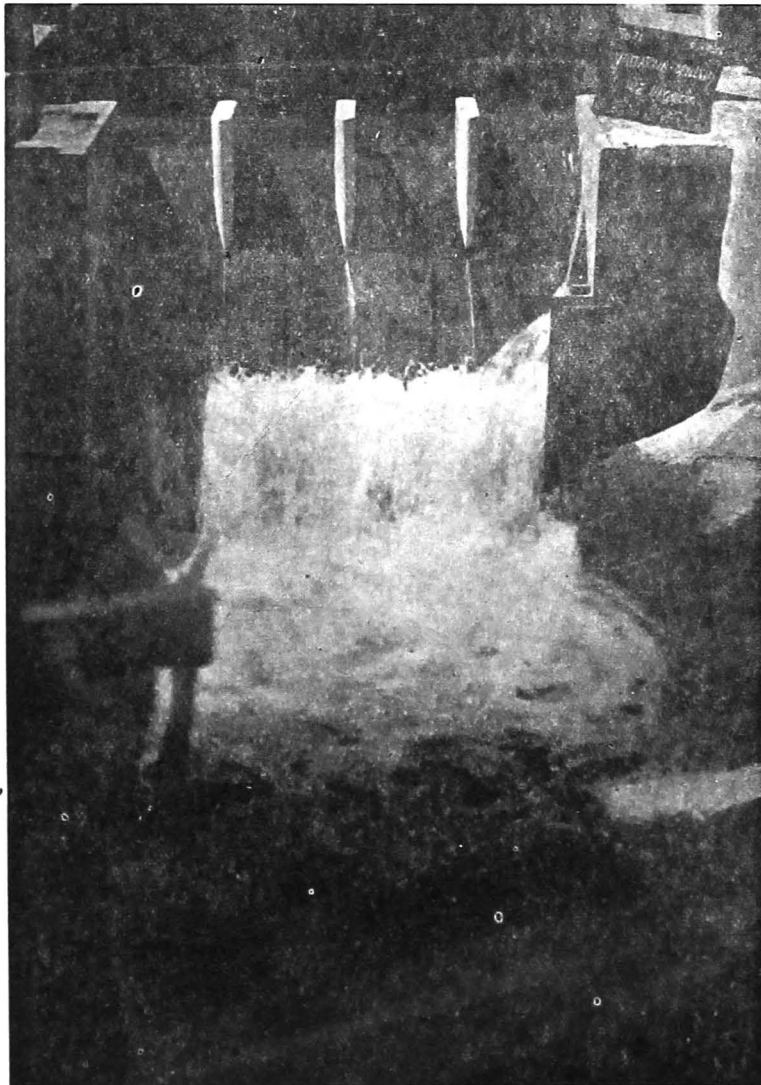


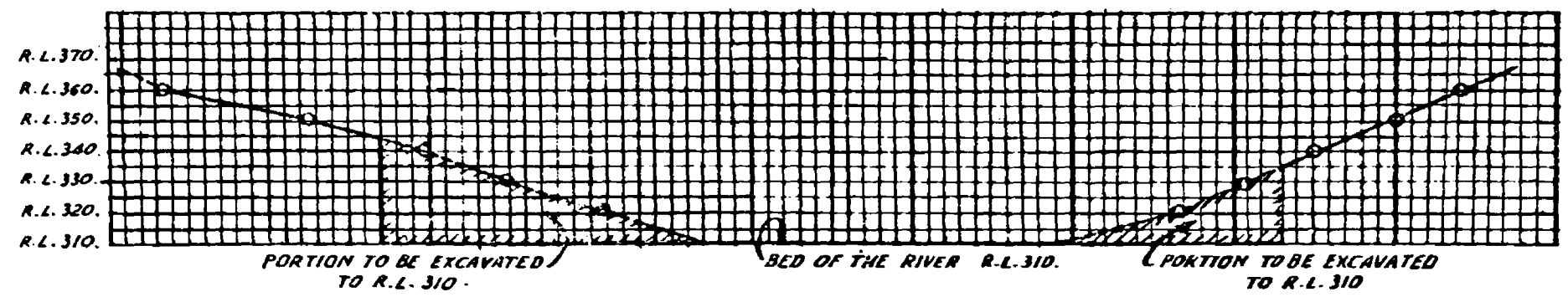
Photo No. 5671—Showing flow condition downstream of spillway,  $Q = 140,000$  cusecs, looking upstream.

FIG. 81 (A)

SHOWING THE PORTION TO BE EXCAVATED TO R.L. 310.  
AT A LONGITUDINAL SECTION AT 'A A' OF GUIDE WALLS. (DESIGN NO. IV.)

SCALE. ONE INCH TO 50 FT.

LONGITUDINAL SECTION -

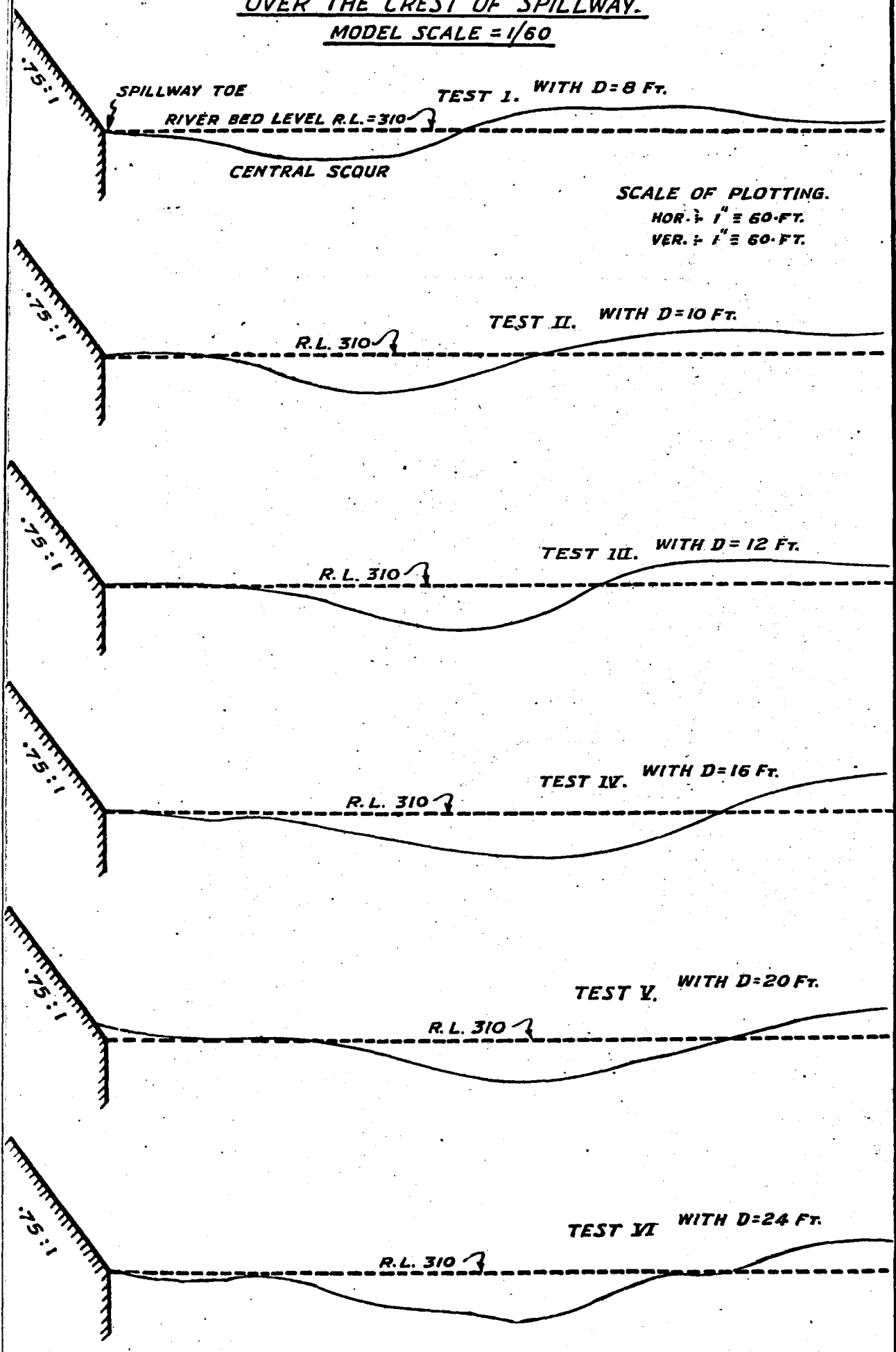


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FIG. 84

SHOWING THE PLOTTING OF CENTRAL SCOUR  $D/S$   
UNDER DIFFERENT DEPTHS OF FLOW  
OVER THE CREST OF SPILLWAY.

MODEL SCALE = 1/60



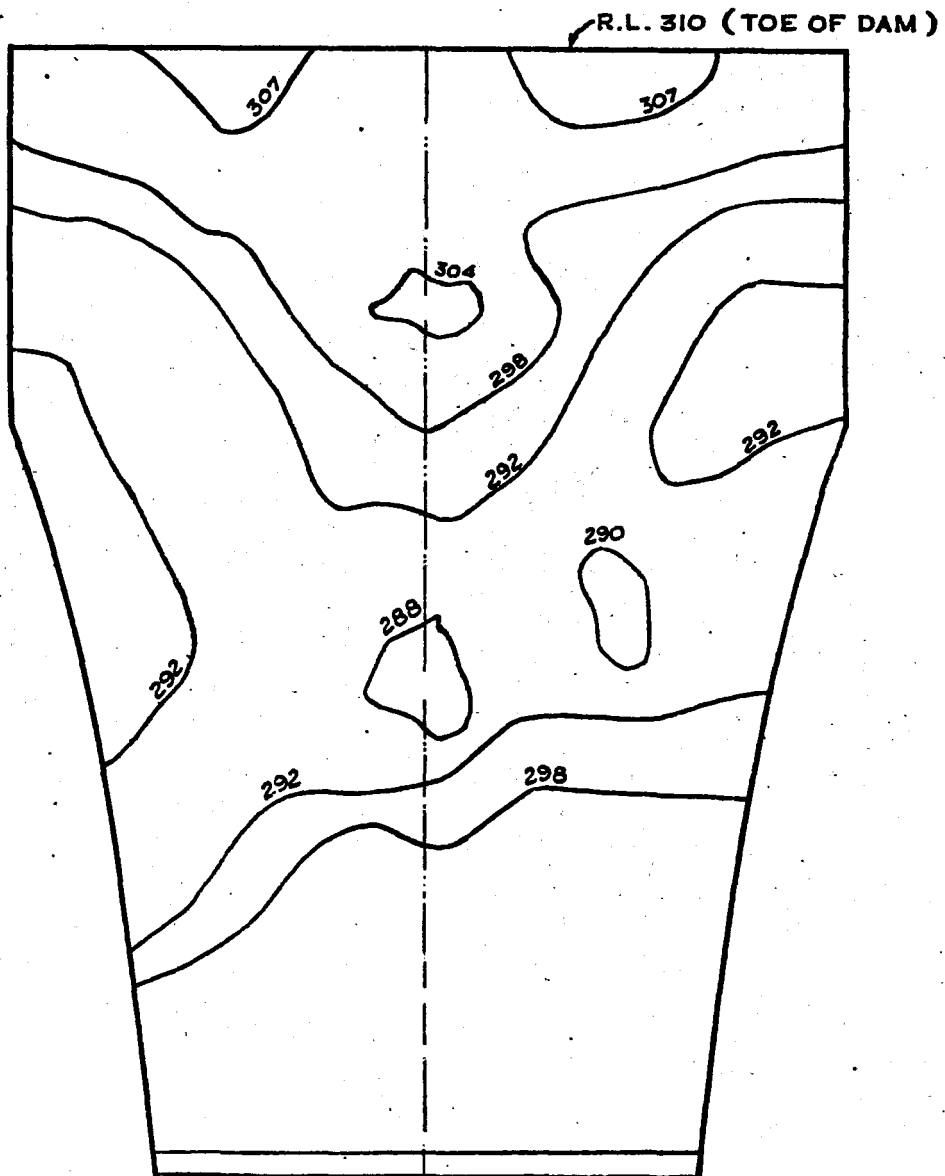
SAPKAL.



FIG.85  
VAITARNA SPILLWAY 1/60 MODEL  
SHOWING THE D/S SCOUR CONTOURS

Q = 140,000 CS.

SCALE. 1" = 60'



## 14. INTERIM REPORT ON MODEL EXPERIMENTS WITH KOSI DAM CENTRAL SPILLWAY (KOSI PROJECT, NEPAL)

### *Introduction*

The design adopted for study by means of hydraulic models was as outlined in "Study No. 15". The details and drawings of it were supplied by the Project Officer, Kosi Project, CWINC, New Delhi.

It was tentatively decided to test the following features of the design:—

(1) Co-efficient of discharge of the crest profile and pressures developed on the face of the spillway with various depths of flow over the crest upto a maximum depth of 40 ft. (M. F. L.):

(2) Action of the ski-jump, with various discharges over the spillway, also in combination with discharges from the sluice outlet.

(3) The scour pattern and the maximum depth of scour developed in the river bed downstream as a result of impingement of the jet from the ski-jump on the river bed.

(4) Similar action of ski-jump in alternative position.

The present report embodies results obtained so far with a geometrically similar, 1/125 scale model of the central spillway.

### 2. *Model and scales of model*

Fig. 87 shows the L-section and upstream elevation of the dam. The height of the spillway crest above the normal river bed (R. L. 350.0) is 610 ft. and the maximum W. L. in the reservoir is 40 feet above the crest level. With this water level the discharge passing over the central spillway only, is expected to be 4,07,780 cusecs. The discharge intensity per foot run is 926.8 cusecs. The scale of the model chosen was naturally limited by the availability of head and corresponding discharge required for the model. Taking these into consideration and the information that was sought from the model, the first model was constructed to a G. S. scale of 1:125. Photo No. 5812 shows that experimental flume and the spillway. The channel 70 feet long, 6.5 feet deep and 3.52 feet wide was prepared for these experiments. The discharge passing over the spillway was measured over a Rehbock weir having L=2.255 feet and crest 2.0 feet above the upstream channel bed level. The length of the flume upstream of the Rehbock was 25 feet and the gauge chamber was 10 ft. upstream of the Rehbock weir. The upstream and downstream gauges for measuring head over the spillway and W. L. in the river downstream were located 6 feet upstream and 20 feet downstream of the spillway crest respectively.

Fig. 87.

Photo 5812.

### 3. *Weir shape tested*

The model spillway was constructed in cement by preparing accurate template of the profile, the details of which are shown in Fig. 88. The weir surface was then polished to make it as smooth as possible. The ski-jump tested in this series was at R. L. 420.0. Details of sluices and sluice outlets were not available initially and later, their dimensions were considerably modified. It was, therefore, decided to complete experiments with the overflow spillway only in the first instance, the experiments with sluice outlets being deferred to a later series.

Fig. 88.

A second model of the spillway to 1/72 scale was also constructed. Due to its larger size this model allowed detailed observations on flow through the sluices and also evaluation of scale effect on co-efficient of discharge. These experiments are in progress.

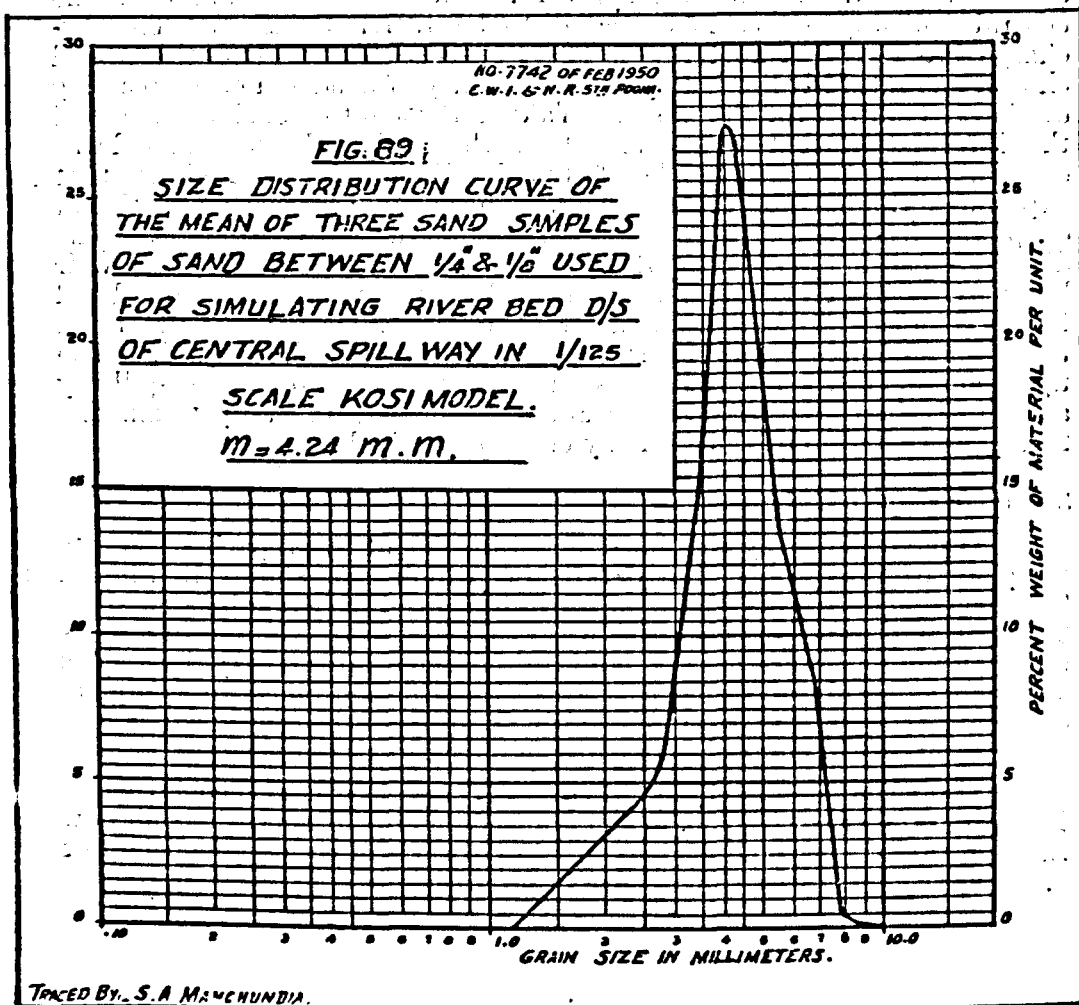
4. Test procedure

Photos 5798,  
5818, 5655,  
5656.

In the first series of experiments various depths of flow were maintained over the crest ranging from 10 feet to 45 feet and the discharge passing over the spillway was measured over the Rehbock weir. From results of these observations, a depth discharge relation for the crest profile was established. These experiments were again repeated in the second series when the two piers on the central spillway were constructed in the model. This gave basis for finding the contraction co-efficient of the piers.

In the third series the action of the ski-jump and the maximum scour developed in the river bed with various depths of flow over spillway and with water level in the river portion corresponding to the discharge (as obtained from the stage : discharge curve obtained in the river) was studied. It is an established fact that the results obtained for scoured depth under laboratory conditions where the river bed downstream is generally simulated by incohesive sand are at best of qualitative value. No attempt was, therefore, made to reproduce the contours of the river bed in the channel downstream of the spillway. The flume bed which had the same width as the central spillway was laid in sand of 4.24 mm. mean diameter to a depth 2 feet below the R. L. of 350 which is the normal bed level in the river. Fig. 89 gives distribution curve of the bed material.

Fig. 89



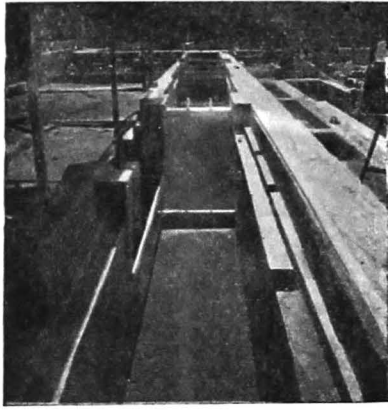


Photo No. 5812—Showing  
Experiment Flume.  
Looking upstream.

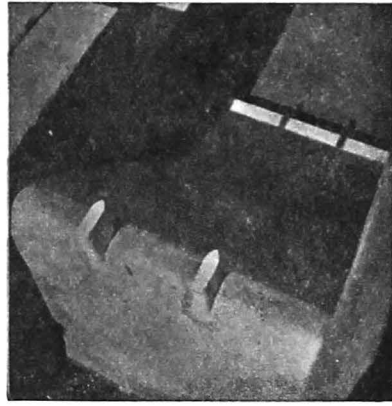


Photo No. 5813—Showing  
spillway from above.  
Looking downstream.

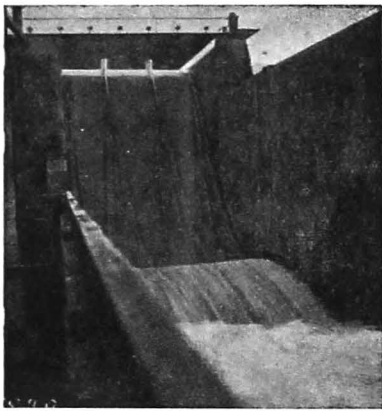


Photo No. 5828—Shows con-  
dition at surface at impact  
point.

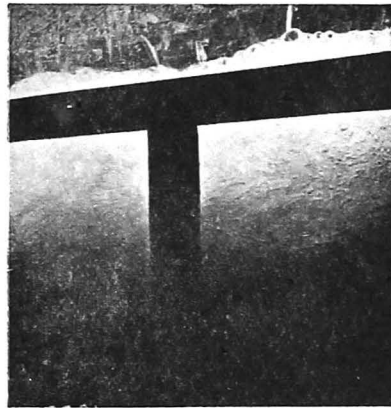


Photo No. 5821—Showing re-  
turn eddy near bed.

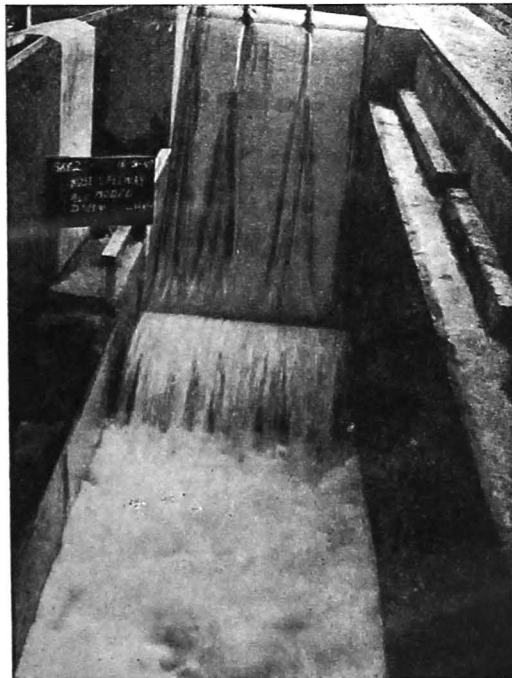


Photo No. 5662—Showing action of  
the bucket and downstream flow  
conditions.

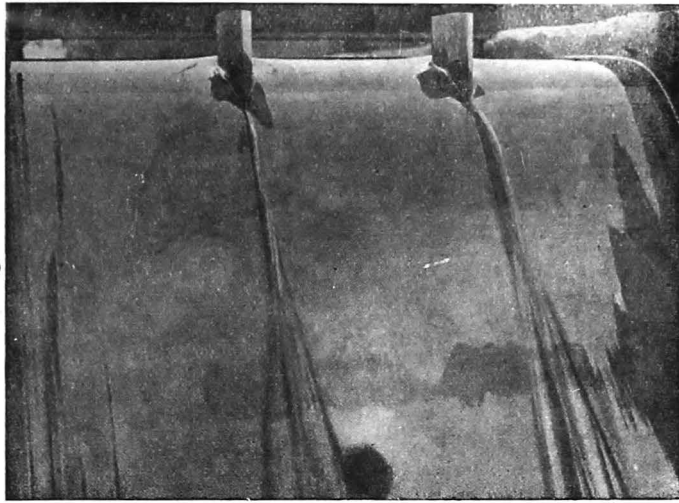


Photo No.5655—A close-up view of model in operation.

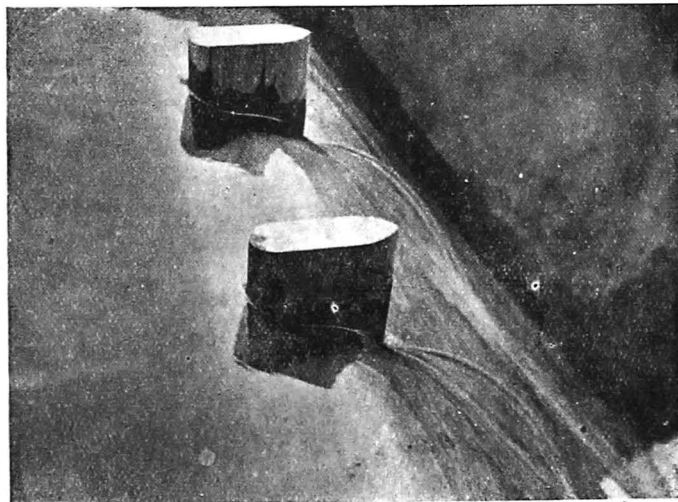
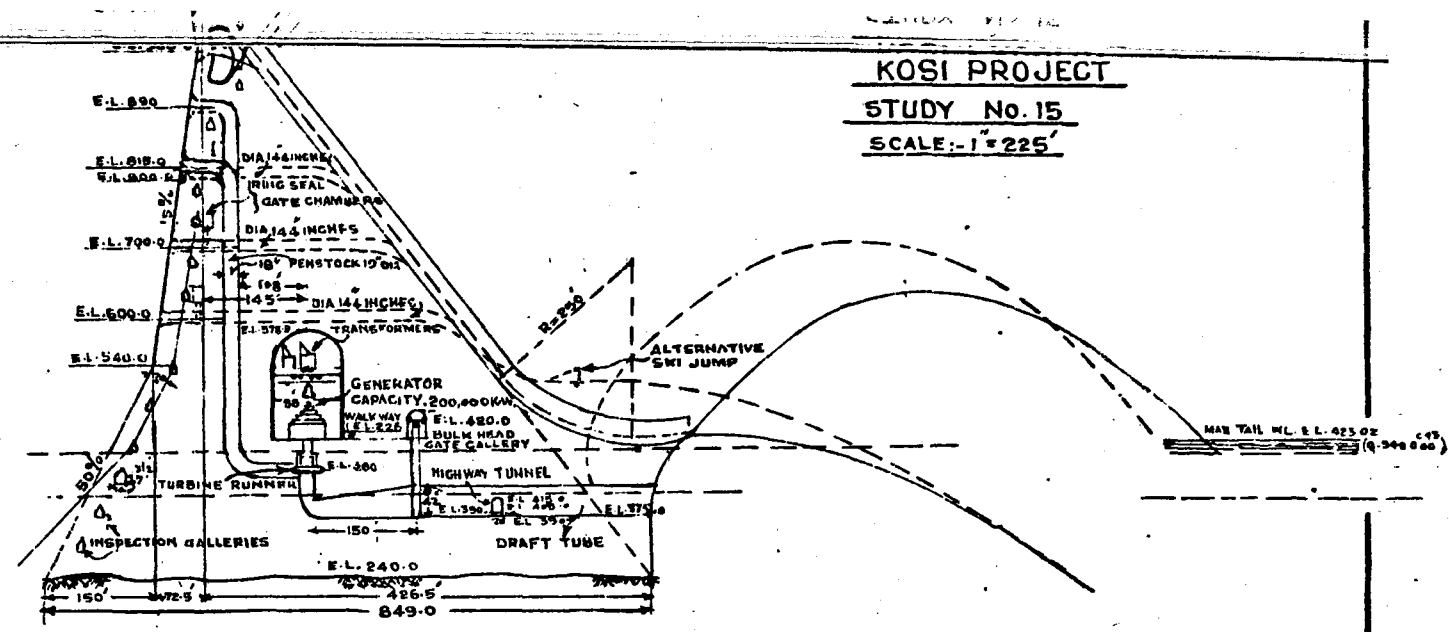


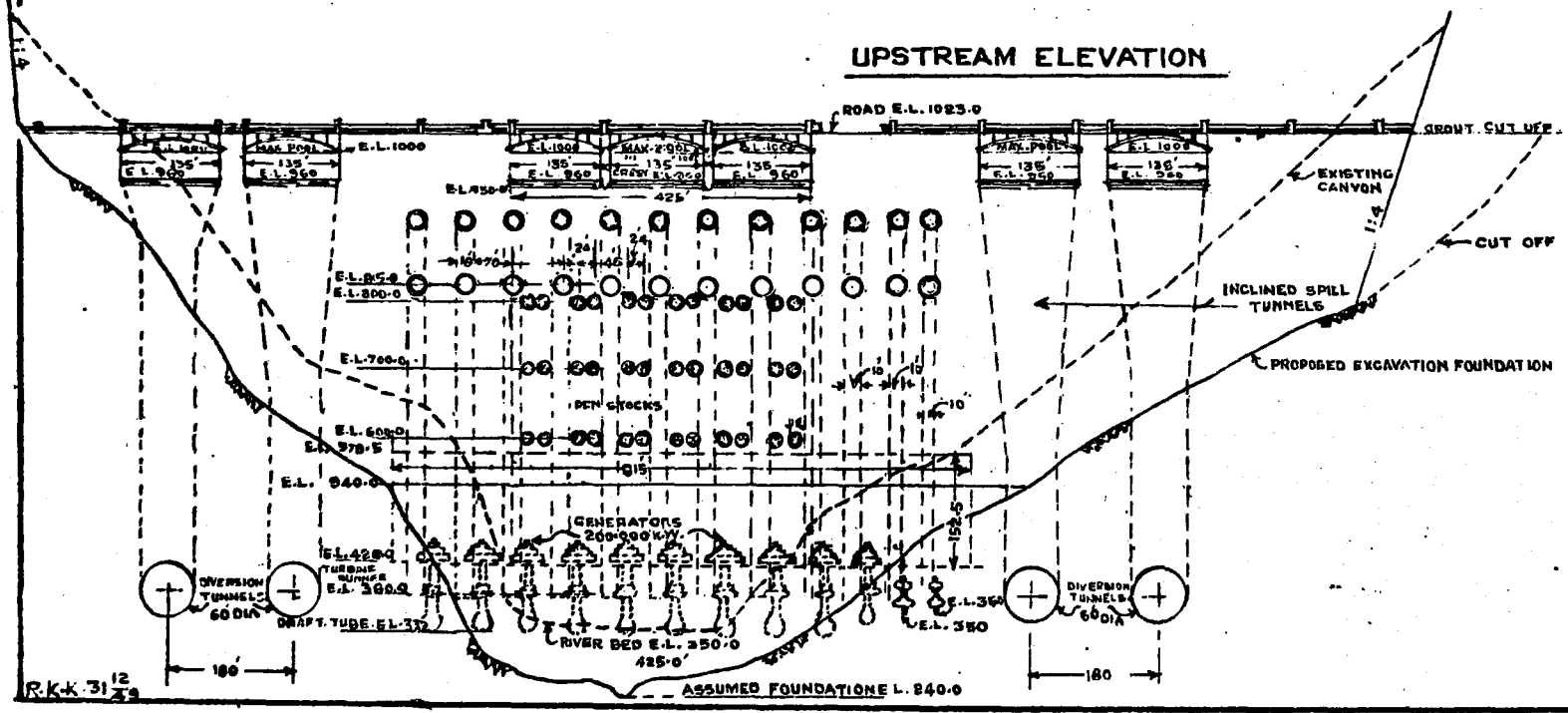
Photo No. 5656—Side view showing the design of piers and position of Piezometric tubes. (Centre bottom),

**KOSI PROJECT**  
**STUDY No. 15**  
**SCALE: - 1" = 225'**



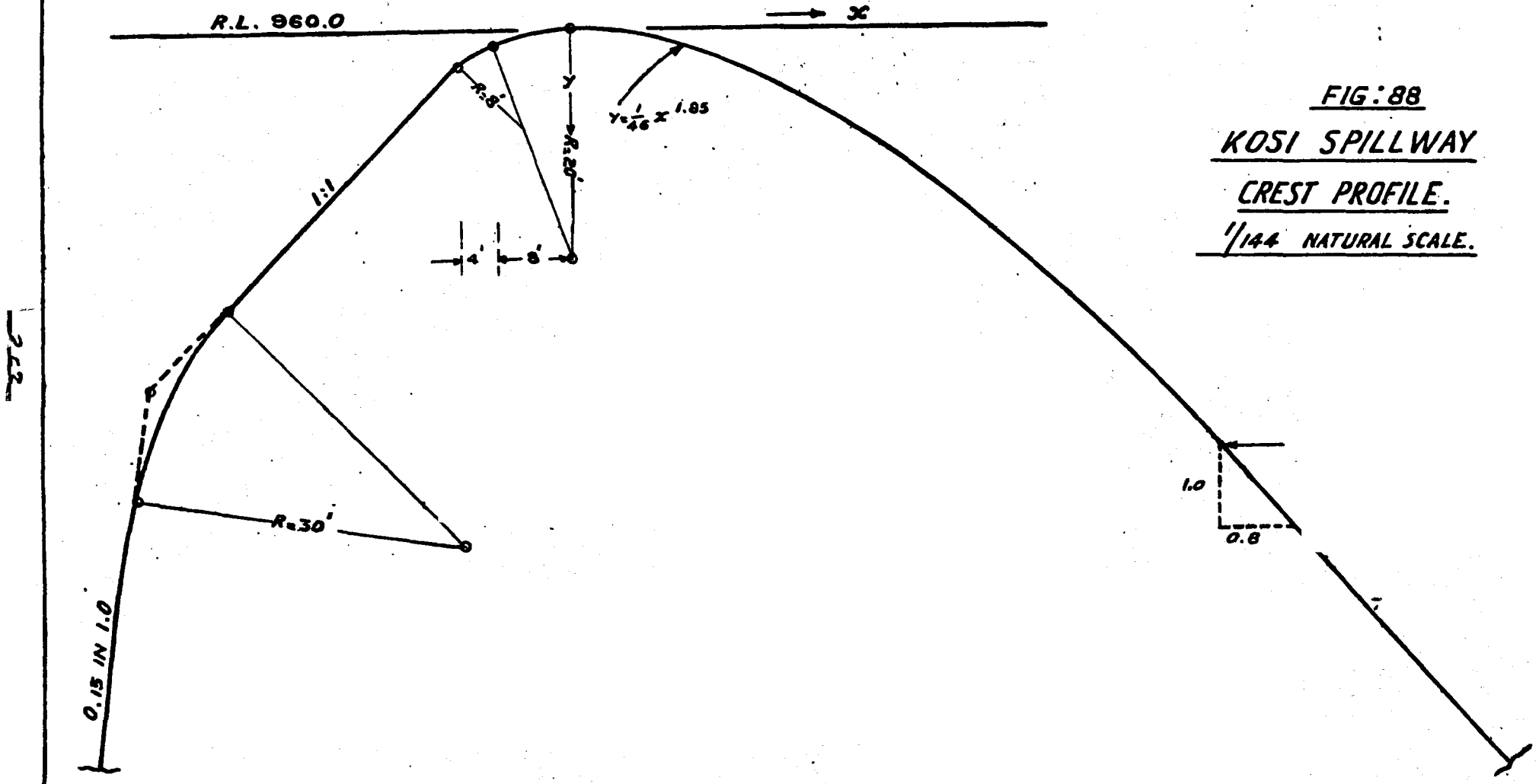
**UPSTREAM ELEVATION**

261



R.K.K. 31<sup>12</sup>  
29

FIG: 88  
KOSI SPILLWAY  
CREST PROFILE.  
1/144 NATURAL SCALE.



Execd S. A. Manchundia.

5. River Water Level Simulated in the Model

The scour pattern and the maximum depth of scour in the river would very much depend on the depth of flow and W. L. in the river. Due to the nature of outflow control arrangements, the stage in the river has not enough time to build up and keep pace with the rising discharge which would flow over the spillway and is imposed on the river during the flood. Hence it will not be incorrect to assume that the W. L. in the river would be slightly less than the normal stage: discharge curve which is obtained in pre-reservoir period. This occurs only for a short period, the extent of which cannot be exactly determined.

But what is more important is the retrogression in the bed levels, and/or water levels which are very likely to develop in the river reach downstream in the course of time. In U. S. A. in almost all such cases, retrogression has been observed, the extent depending on various factors in individual cases such as discharge intensity, slope, deficiency of sand load in the outflow due to heavier material depositing in the lake etc. In Kosi spillway the provision of deep seated sluices to allow part of the density currents carrying greater percentage of sand load and construction of barrage about 4 to 5 miles downstream with pond level at R. L. 375 would counteract retrograding forces to a certain extent. Even then as these effects cannot be evaluated correctly, it is desirable to provide for this eventuality both in the design of ski-jump and probably also in the Chattra Weir design.

From the above consideration it was decided to test the spillway ski-jump with 20 to 25% retrogression in the river W. L. downstream of the spillway instead of according to the stage: discharge curve obtained in the river. In each experiment with a given depth of flow over spillway the experiment was continued till it was observed that no further change took place in the scour pattern and maximum depth of scour. Pressures were obtained by locating piezometers on the crest and on the spillway face. Water surface profiles were also observed simultaneously.

Photo 5555.

6. Results (Series I and II):—Results obtained in series I and II are plotted in Fig. 90 which shows depth: discharge and depth: co-efficient of discharge relationships for the spillway crest profile, both without piers and with piers. The co-efficient of discharge rises from 3.74 at depth of flow = 10 feet to 3.96 at depth of flow = 40 feet for spillway without piers while with the two piers, the co-efficients of discharge for these depths are 3.69 and 3.93 respectively.

Fig. 90.

The apparent reduction in discharge co-efficient is due mainly to two factors:

- (1) reduction of effective width of spillway due to boundary layer development at the pier face.
- (2) energy losses due to impact and subsequent eddy losses at the pier.

The first can be observed in the model provided the model is sufficiently large, but the reduction due to the second cause increases with the size of the model and is not appreciable in the smaller model.

The co-efficient of contraction as evaluated in U. S. A. varies from  $K=0.12$  to  $0.2$  and higher co-efficient up to  $0.4$  has been observed. Even with the higher value of the co-efficient in the formula:

in which

$$Q = C (L - NKH) H^{3/2}$$

Q = discharge in cusecs,

C = co-efficient of spillway without piers,

L = clear length of spillway (feet),

N = number of contractions, 2 for each pier,

K = contraction co-efficient,

H = depth of overflow over spillway (ft.).

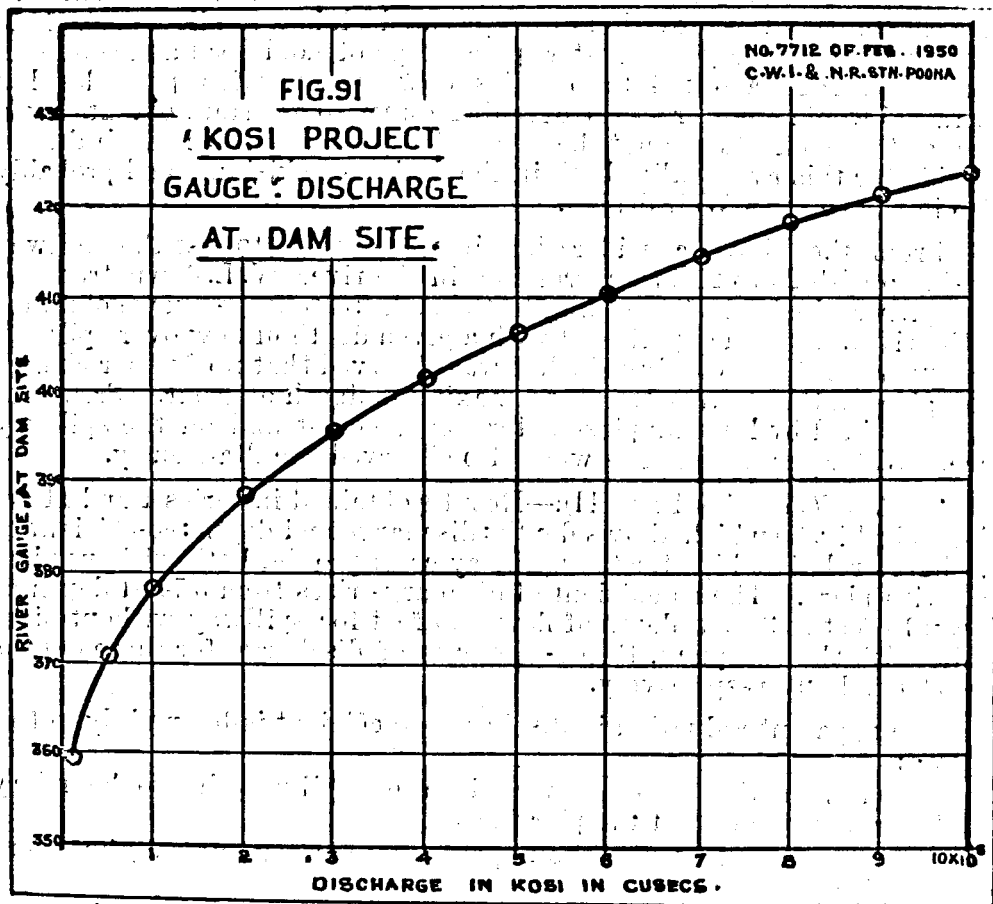


The reduction in co-efficient in the Kosi Spillway with a depth of overflow of 40 feet due to contraction would be less than 1.6%. In the 1/125 scale model the contraction co-efficient worked out to .02 and the reduction in co-efficient about 1%. Considering that the systematic errors in the model due to various reasons are nearly of the same order or even higher, it is a moot point whether the contraction co-efficient thus evaluated would be reliable and of any real value.

*Series III*

In series III with various depths of flow over the spillway corresponding W. L. in the river downstream (with 25% retrogression for reasons as explained earlier) were maintained. The gauge : discharge curve under existing conditions is shown in Fig. 91. The period of

Fig. 91.



Figs. 92 to 95

Figs. 94 and 95

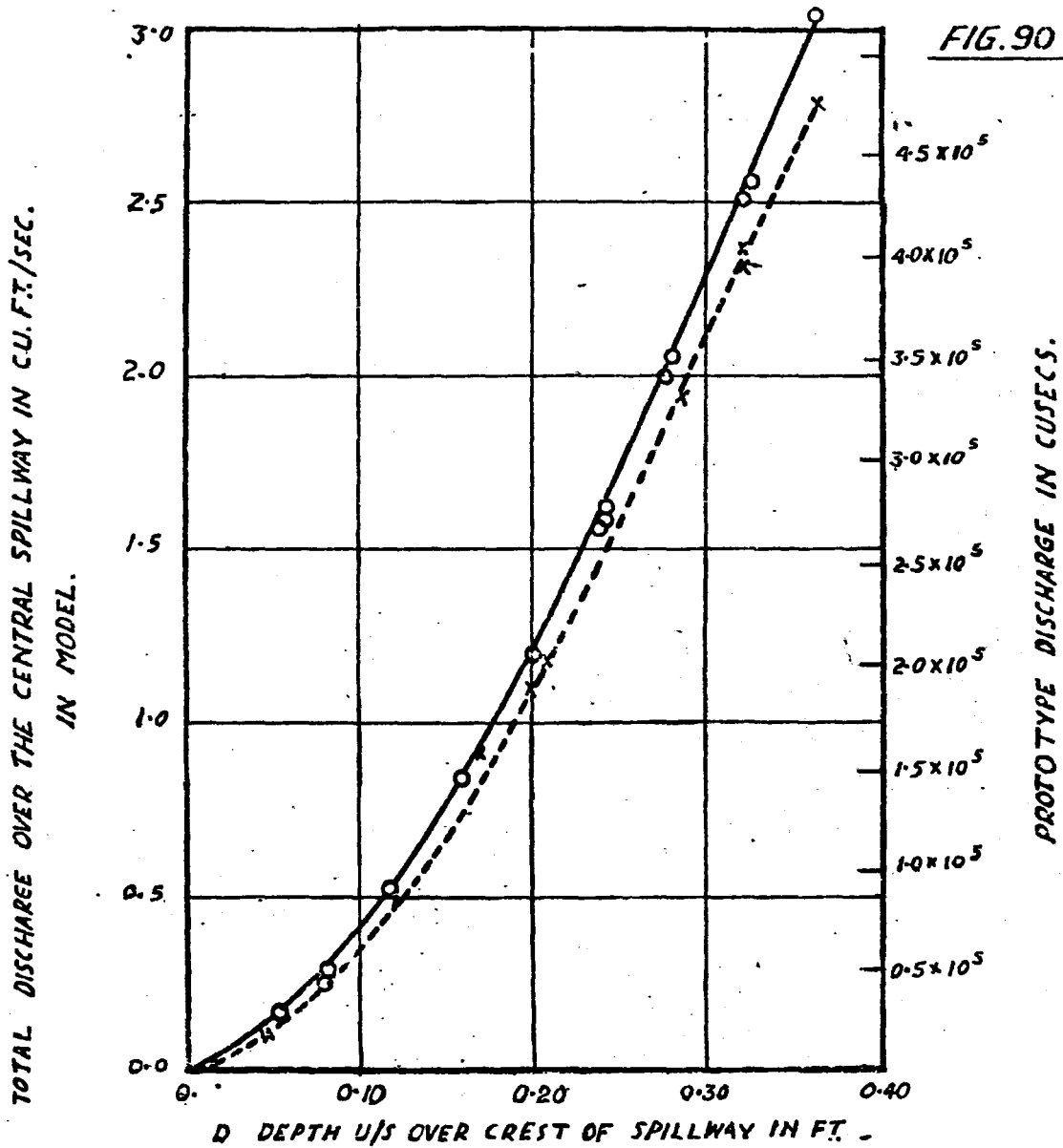
the test in each case was 6 hours. The scoured depth remained constant beyond this period. Figures 92 to 95 show the L-profile of spillway and water profile. They also show the profile of scoured river bed. Though the experiments to find out the maximum depths of scour in the river bed downstream were qualitative, they indicated that for depths of overflow greater than 20 feet, accretion would occur near the toe of the dam as shown in figures 94 and 95; this material was from the scour hole and swept upstream by the return roller and deposited where the force of the return eddy had diminished in strength. With lower depths of overflow, however, the return roller was not so strong and hence no material could deposit. It was also noted that just below the ski-jump, there was an air pocket and below this, a region of still water near the toe, thus indicating that the designed ski-jump would throw the jet sufficiently far away from the toe.

**KOSI PROJECT**  
**CENTRAL SPILLWAY**  
**MODEL SCALE 1/125**

No 7751 OF FEB. 1950.  
 C. W. I. & N. R. ST. POONA.

**DEPTH OF FLOW OVER CREST: DISCHARGE**

○ WITHOUT PIERS.  
 x WITH PIERS.



**DEPTH OF FLOW OVER CREST: COEFFICIENT OF DISCHARGE**

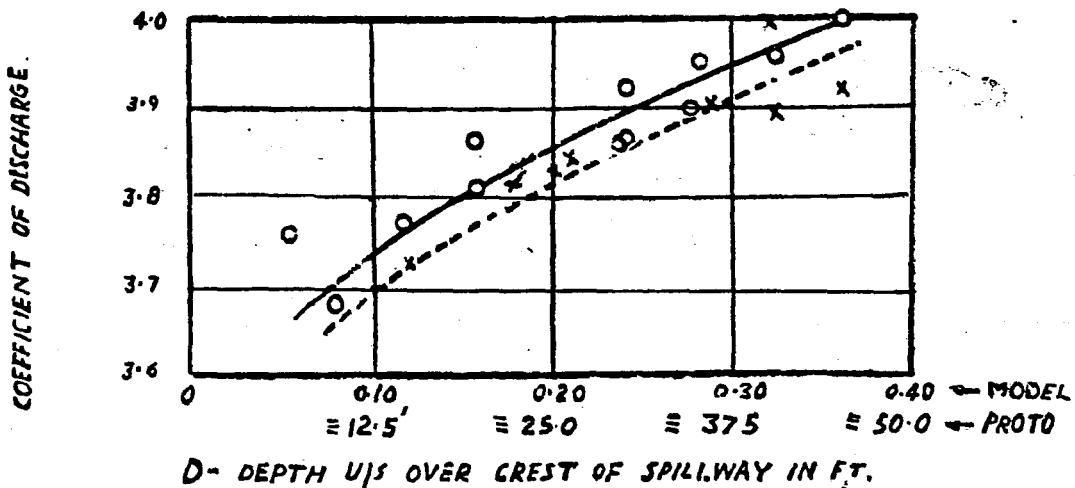


FIG-92 & 93.

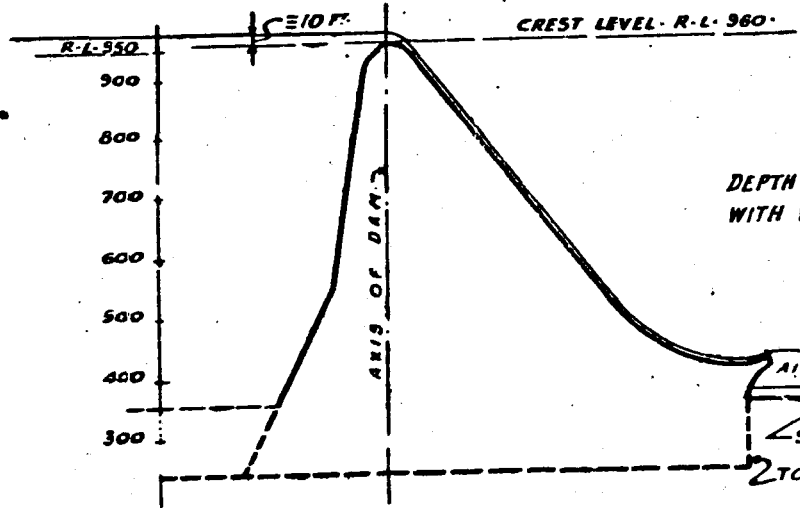
KOSI PROJECT

KOSI CENTRAL SPILLWAY ACTION OF SKI-JUMP.

MODEL SCALE 1/125.

SHOWING WATER PROFILE & SCOURED BED PROFILE.

SCALE: 1" = 300'



DEPTH OF FLOW OVER SPILLWAY  $\approx$  10 FT.  
WITH W.L. IN RIVER D/S  $\approx$  R.L. 372.50.

AIR

D/S W.L.  $\approx$  R.L. 372.50 (WITH 25% RETROGRESSION)

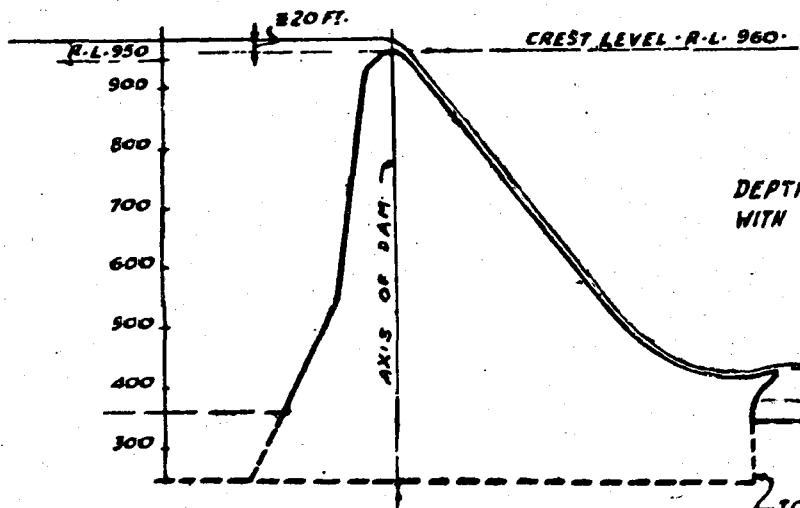
ORIGINAL BED LEVEL

SCOUR BED

MAX. SCOUR  $\approx$  21 FT. (R.L. 329.0)  
AT CH.  $\approx$  213 FT. FROM TOE.

TOE

21.5'



DEPTH OF FLOW OVER SPILLWAY  $\approx$  20 FT.  
WITH W.L. IN RIVER D/S  $\approx$  R.L. 385.00.

D/S W.L. R.L. 385.0 AT CH.  $\approx$  1800 FT. FROM TOE.

ORIGINAL BED LEVEL

SCOUR BED

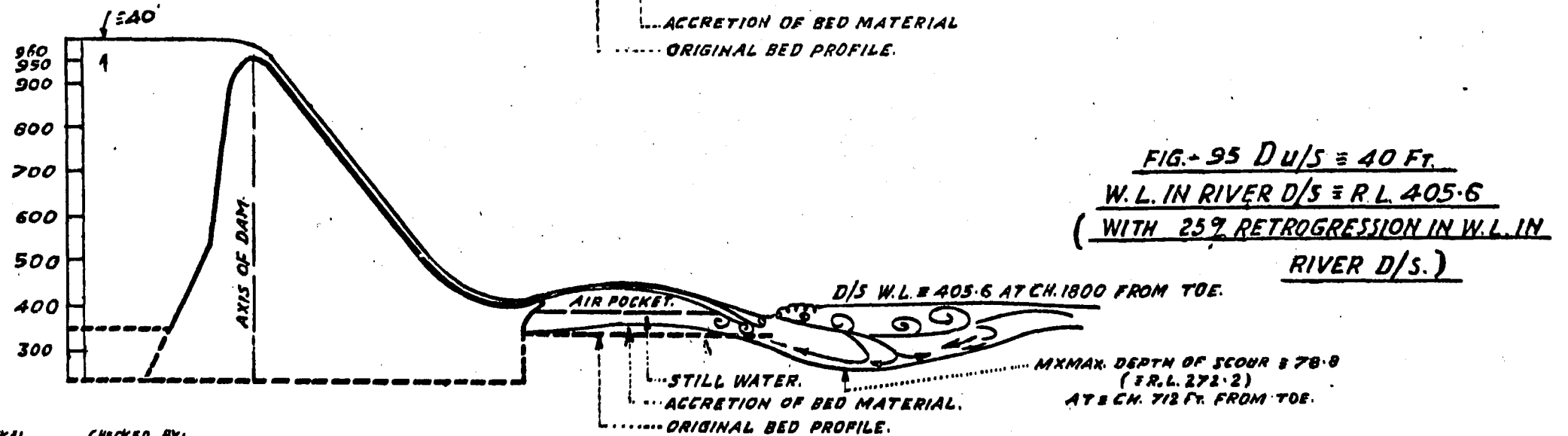
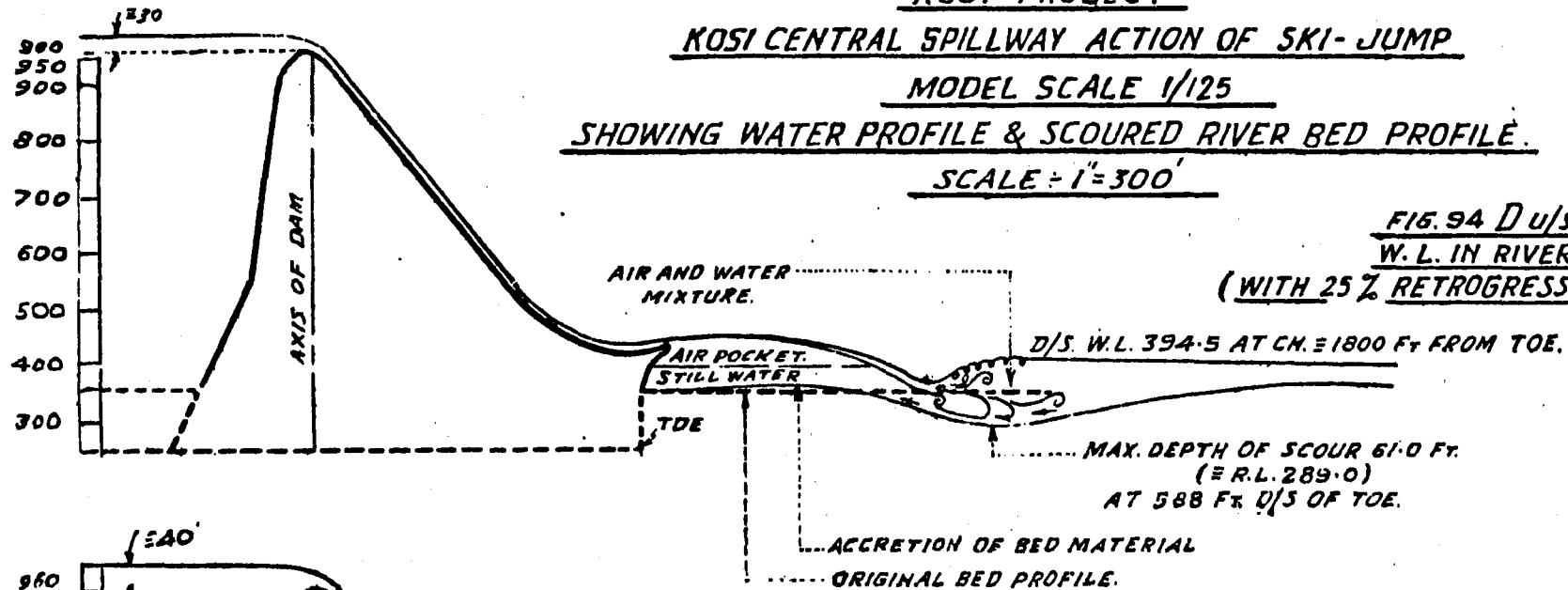
MAX. DEPTH  $\approx$  52 FT. (R.L. 298.0)  
AT CH.  $\approx$  400 FT. D/S FROM TOE

TOE

35.0'

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**FIG. 94 & 95**  
**KOSI PROJECT**  
**KOSI CENTRAL SPILLWAY ACTION OF SKI-JUMP**  
**MODEL SCALE 1/125**  
**SHOWING WATER PROFILE & SCOURED RIVER BED PROFILE.**  
**SCALE = 1" = 300'**



BY SAPKAL. CHECKED BY:

## KOSI DAM CENTRAL SPILLWAY

The sand particles on the downstream slope of the scour-hole rolled down due to gravity and were swept upstream by the return eddy. With  $D = 10$  feet, scour developed nearer the toe, but still away from the toe. Photo 5821.

Table in Fig. 96 shows the maximum depth observed with depths of overflow = 10, 20, 30 and 40 feet and their distances from the toe. The contours of the scoured bed are shown in Figs. 97 to 100. The increased scour developed at the channel sides due to vertical eddies was disregarded. Table below shows the depth of water cushion in each case. Fig. 96  
Figs. 97 to 100

TABLE NO. 1

Depth of flow over crest	Depth of water maintained above river bed R. L. 350.	Maximum depth of scour below river bed R. L. 350.	Depth of water cushion.
10	22.5'	21.0'	43.5'
20'	35.0'	52.0'	87.0'
30'	44.5'	61.0'	105.5'
40'	55.6'	78.8'	184.4'

These are plotted in Fig. 101 and show a good correlation. Other factors such as spillway height above ski-jump, the ski-jump radius and angle of projection will have to be taken into consideration when comparing scour depths with other position of ski-jump. The effect of actual rock level on the scour pattern will also be observed in later series. Fig. 101

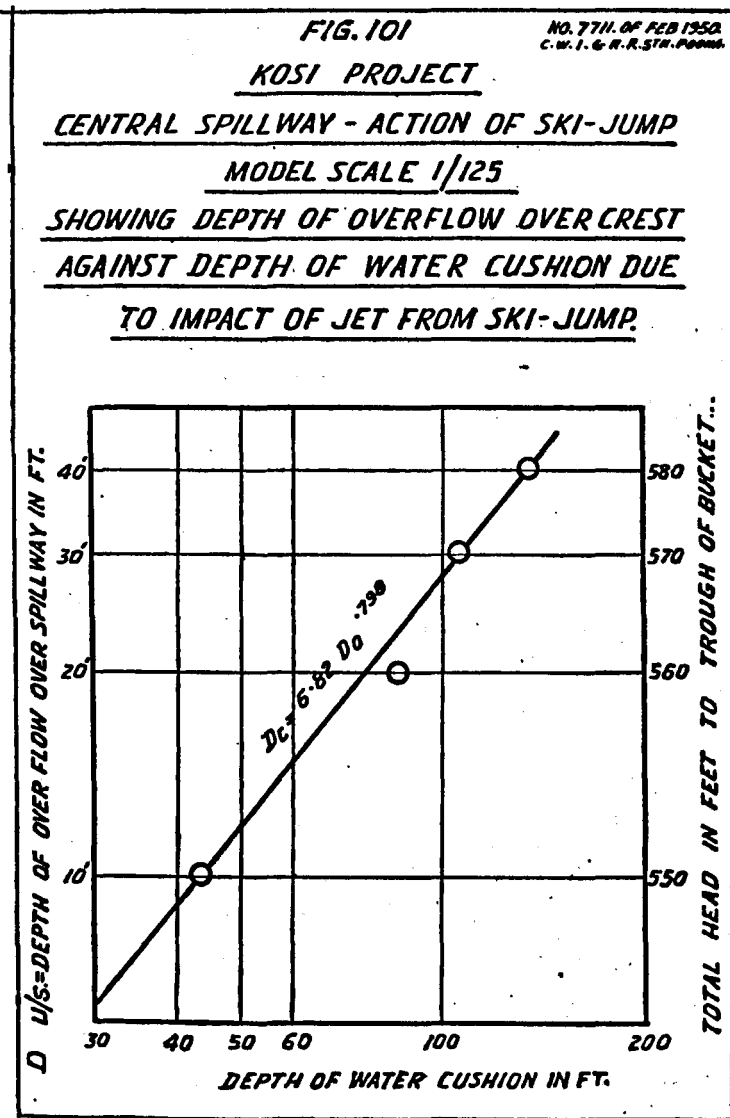
### *Pressures on the Spillway*

The pressures observed on the crest of the profile with various depths of overflow are shown in Table No. 2 and are plotted in Fig. 102. It will be seen that the maximum negative pressure = 2.9 feet of water only is obtained with depth of flow = 40 feet at P. This may be considered as satisfactory. Table 2,  
Fig. 102

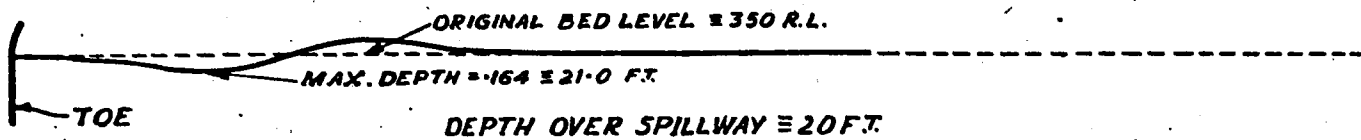
### *Analysis of Results*

Results from 1/125 scale model for the central spillway indicate that the maximum depth of scour under 30 feet and 40 feet of overflow is likely to be 700 feet to 830 feet away from the toe of the dam. The modifying effects of a part of the flood being allowed through the deep seated sluices and over the side spillways will have to be observed in the composite model. The only effect which cannot be observed in the model is that of air entrainment in the jet which will occur in the prototype as jet flows with a high velocity down the face of the spillway. This point could be estimated roughly from actual observation on steep chutes. The exact effect of this air entrainment on the jet and scour development is uncertain though it is expected that with 40 per cent to 50 per cent entrainment the effective density of water will be reduced and help in reducing the depth of scour.

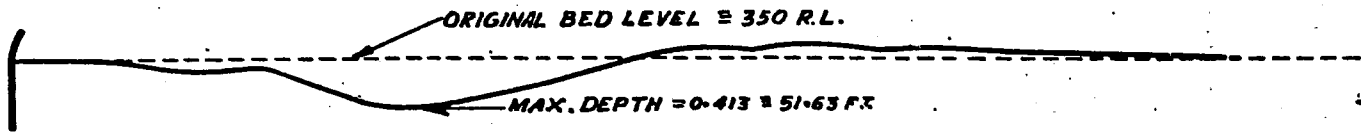
The co-efficient of discharge of the spillway as obtained in the model for  $D = 40$  feet is 3.96. The prototype value will be judged from results of 1/72 scale model which are in progress.



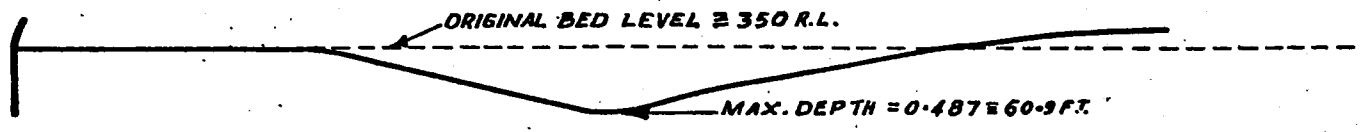
DEPTH OVER SPILLWAY  $\approx$  10 FT.



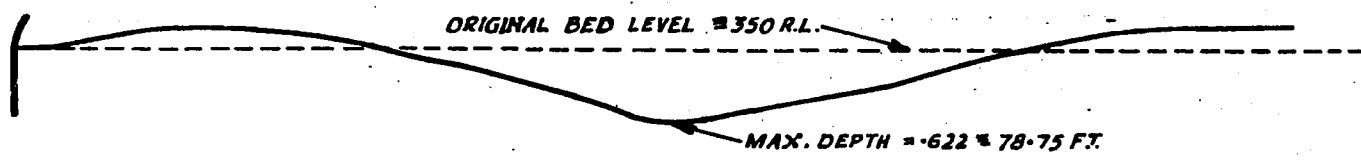
DEPTH OVER SPILLWAY  $\approx$  20 FT.



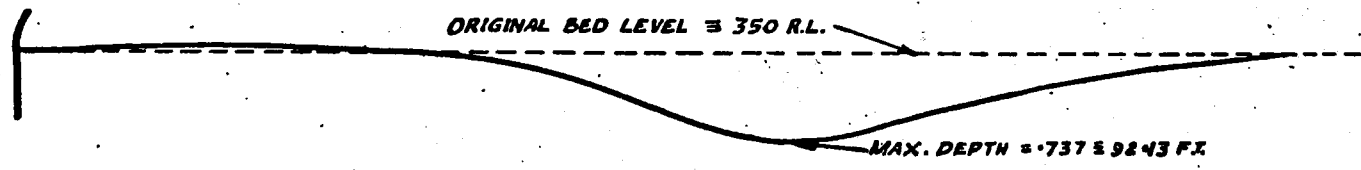
DEPTH OVER SPILLWAY  $\approx$  30 FT.



DEPTH OVER SPILLWAY  $\approx$  40 FT.



DEPTH OVER SPILLWAY  $\approx$  45 FT.



DISTANCE FROM TOE OF DAM IN FT.

By. P. T. THAKOR.

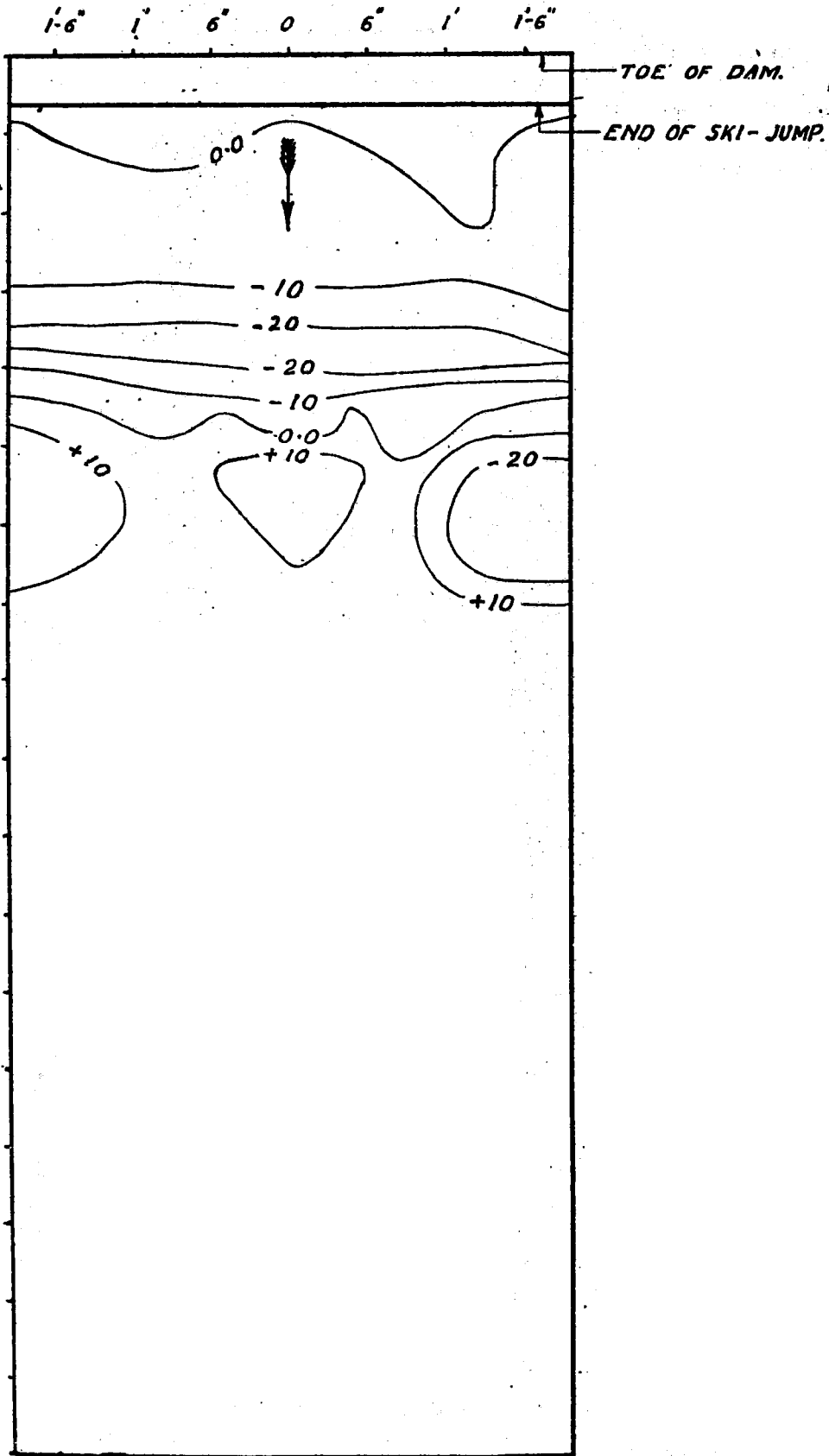
**FIG. 96**  
**KOSI PROJECT**  
**EXPERIMENTS WITH CENTRAL SPILLWAY**  
**1/125 SCALE MODEL**  
**FINAL L PROFILE OF SCOUR DEVELOPED IN**  
**THE RIVER D/S WITH VARIOUS DEPTHS OF**  
**FLOW OVER CREST.**

**TABLE**  
**SHOWING RESULTS OF SCOUR OBSERVATION**

SL. No.	DEPTH OVER CREST OF SPILLWAY "D"	MAX. SCOUR IN FEET. BELOW BED LEVEL OF RIVER D/S R.L. 350	DISTANCE IN FT. FROM TOE OF BUCKET WHERE MAX. SCOUR OCCURED
1	10.0	21.0	212.5
2	20.0	51.6	400
3	30.0	60.9	587.5
4	40.0	78.8	712.5
5	45.0	92.1	828.3

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**FIG. 97**  
**KOSI CENTRAL SPILLWAY**  
**MODEL SCALE 1/125**  
**SHOWING CONTOURS OF SCOURED BED DOWNSTREAM**  
**WITH DEPTH OF FLOW OVER CREST = 10 FT.**  
**W.L. IN RIVER D/S = 372.5**  
**SCALE. 1" = 1' = 125'**



NOTE. RIVER BED INITIALLY LAID AT R.L. 350

V. R. T. Thakot.



FIG. 98

NR. 7625 OF FEB. 1950  
C. W. I. & N. R. S. D. POONA

KOSI CENTRAL SPILLWAY  
MODEL SCALE 1/125

SHOWING CONTOURS OF SCOURED BED DOWNSTREAM  
WITH DEPTH OF FLOW OVER CREST = 20 FT  
W. L. IN RIVER D/S = 385.0  
SCALE. 1" = 125'

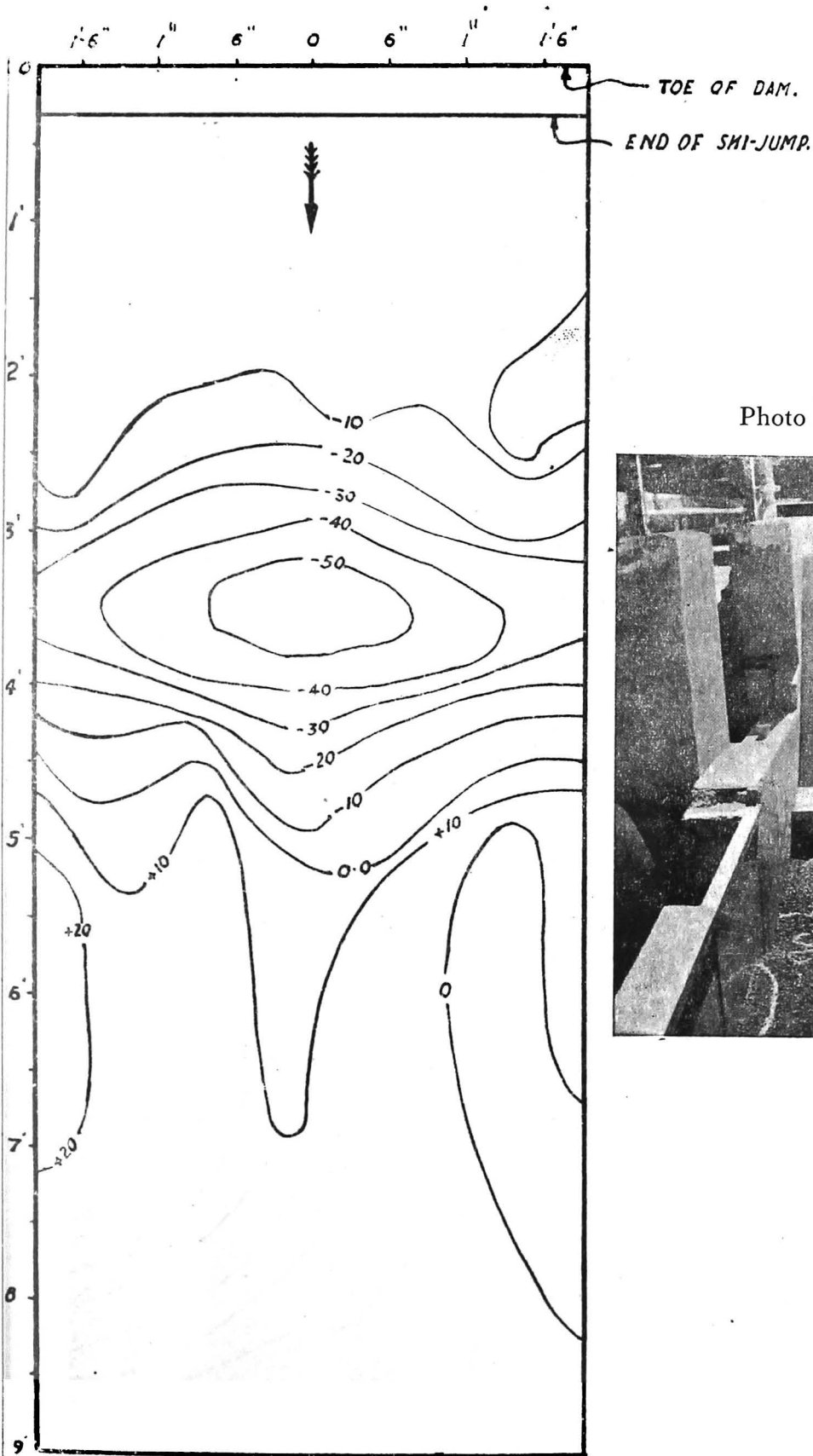
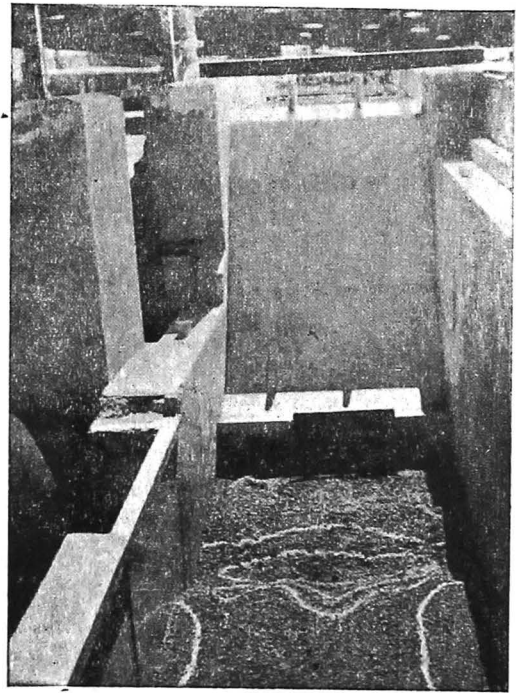


Photo No. 5739



NOTE:-

RIVER BED INITIALLY LAID AT R.L. 350.

BY: BHINGARE. 9-2-50.

FIG. 99

NR 7625(B) OF FEB. 1950  
C. W. I. & N. R. S'p. POONA.

KOSI CENTRAL SPILLWAY

MODEL SCALE 1/125

SHOWING CONTOURS OF SCOURED BED DOWNSTREAM

WITH DEPTH OF FLOW OVER CREST  $\approx 30$  FT.

W. L. IN RIVER D/S  $\approx 394.5$

SCALE:- 1" = 1' MODEL  $\approx 125$

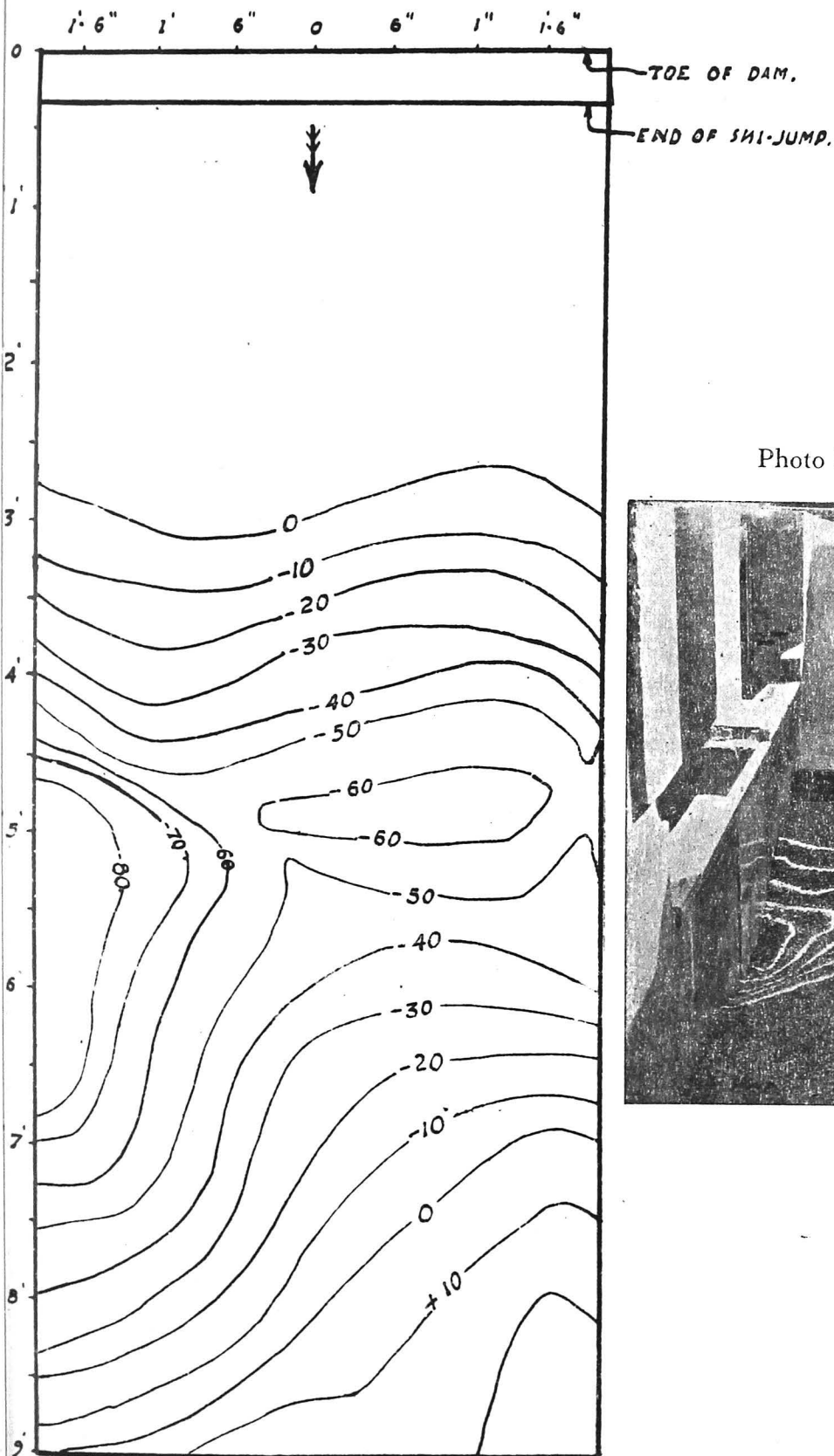
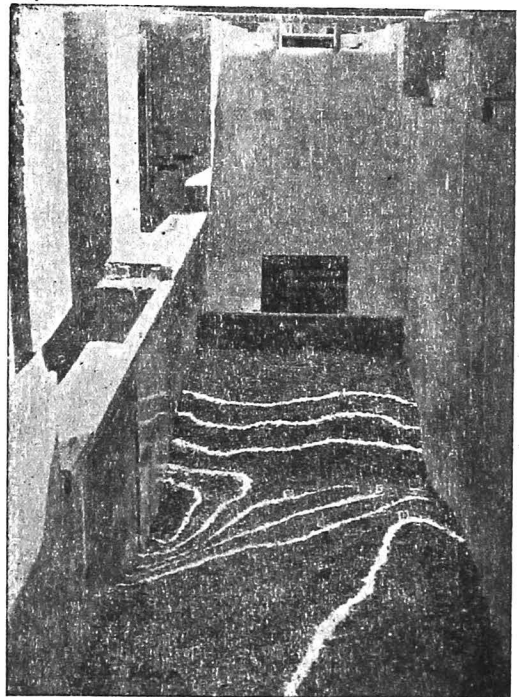


Photo No. 5670



NOTE:- RIVER BED INITIALLY LAID AT R.L. 350.

BY:- BHINGARE. 9-7-50.

**FIG. 100**  
**KOSI CENTRAL SPILLWAY**  
**MODEL SCALE 1/125**  
**SHOWING CONTOURS OF SCoured BED DOWNSTREAM**  
**WITH DEPTH OF FLOW OVER CREST = 40 FT**  
**W. L. IN RIVER  $D/S \approx 405.6$**   
**SCALE. 1" = 125 FEET.**

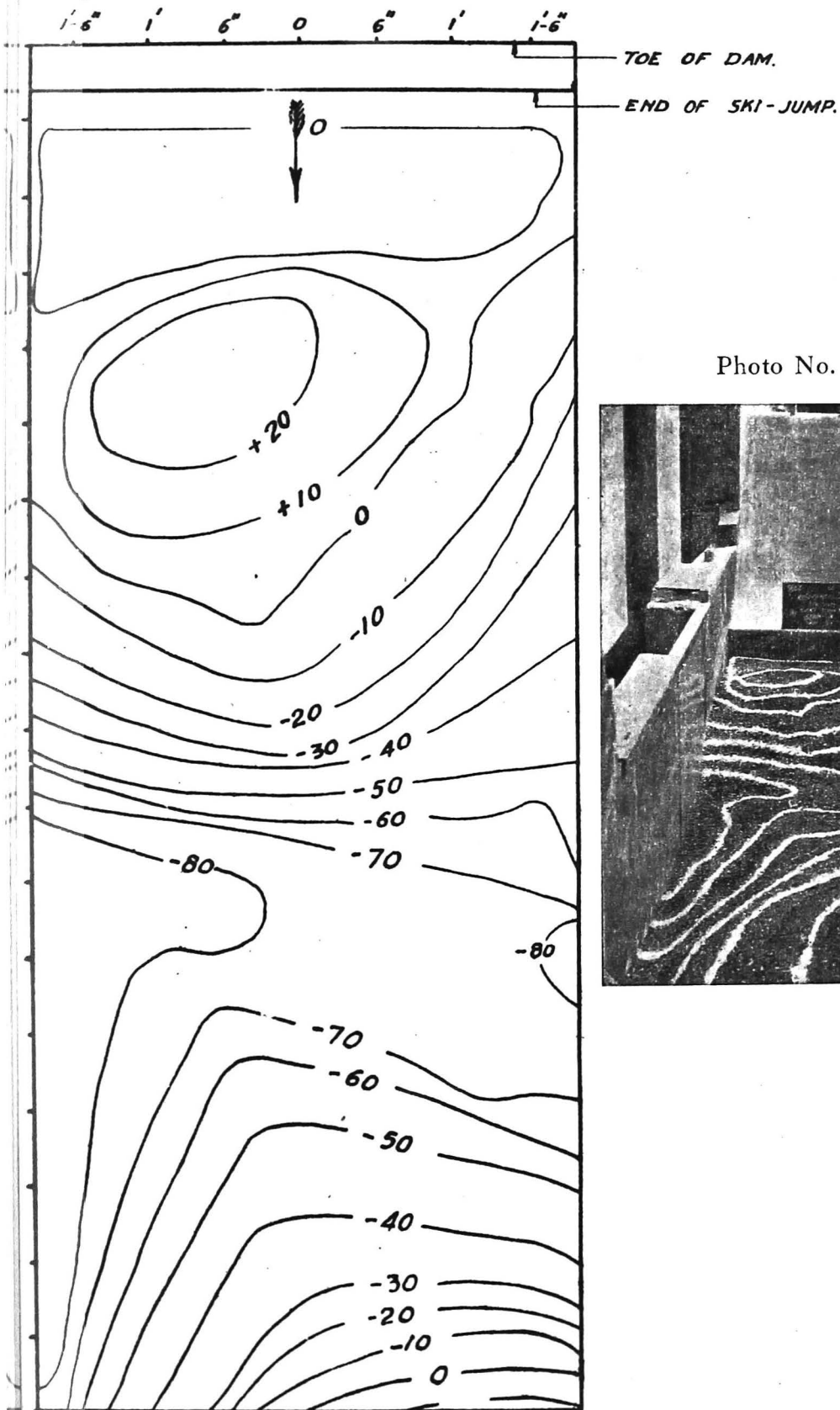


Photo No. 5667



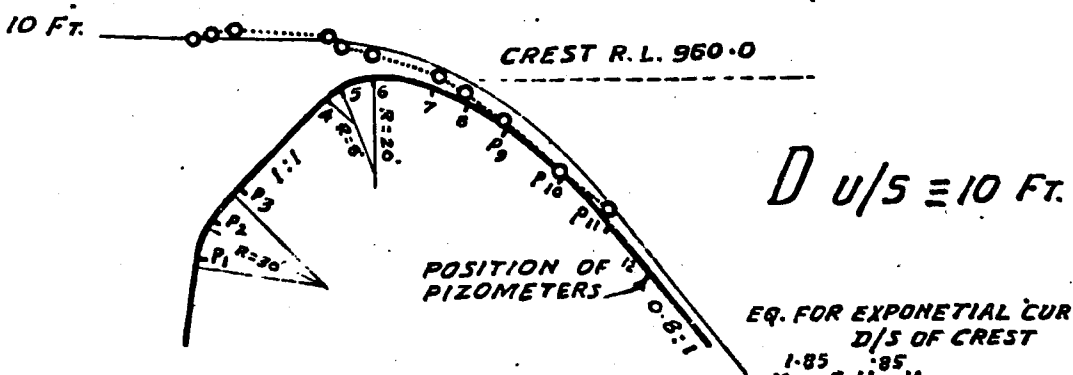
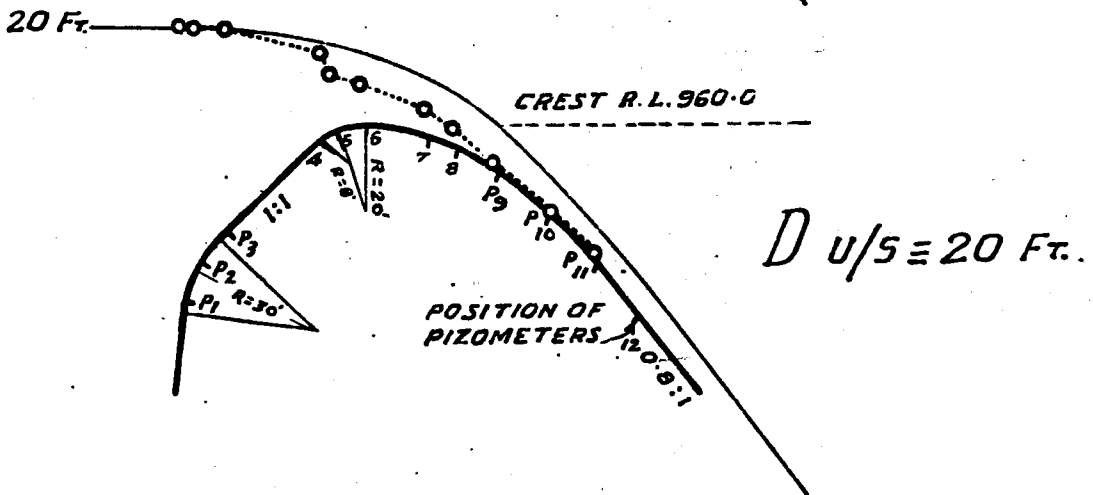
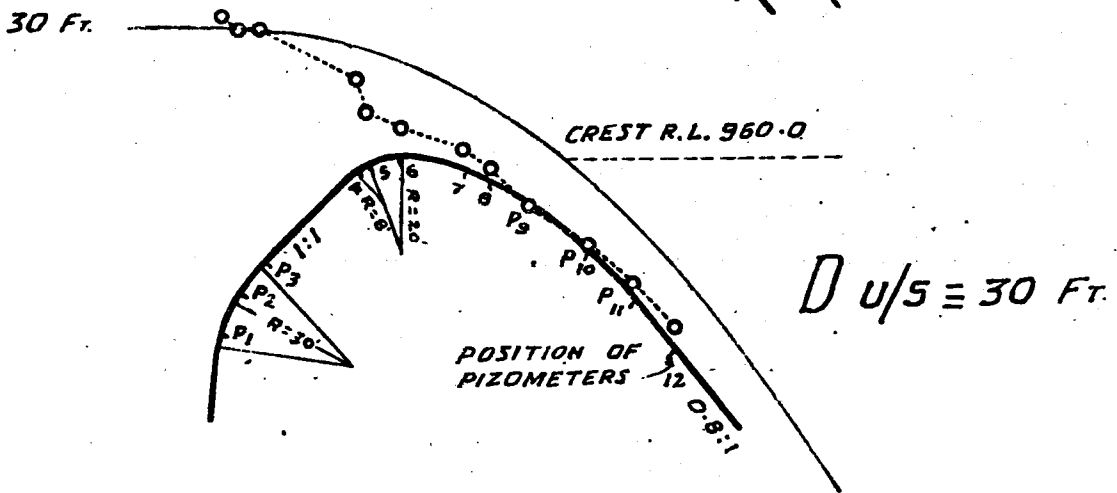
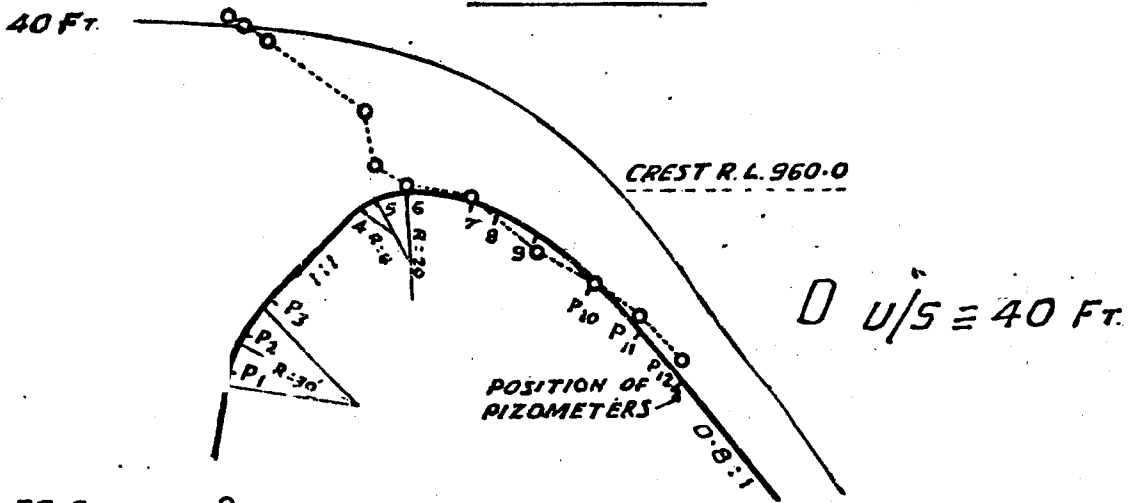
NOTE, RIVER BED INITIALLY LAID AT R. L. 350  
S. R. T. THAKOR.

FIG.102

No 7702 OF FEB. 56.  
C.W.I. & N.R.STN. POONA.

KOSI PROJECT CENTRAL SPILLWAY MODEL 1/125 SCALE  
WATER PROFILE & PRESSURES ON CREST  
WITH VARIOUS DEPTHS OF FLOW.

SCALE: 1"=40'



EQ. FOR EXPONENTIAL CURVE  
D/S OF CREST  
 $X = 2 H_c Y$   
 $H_c = 40 \text{ Ft.}$

V SAPPAL.

KOSI DAM CENTRAL SPILLWAY

TABLE No. 2

Showing pressures on crest with different depth of overflow (1/125 scale model of central spillway)

	Piezometer No.	Level below crest in ft.	Pressures with atmospheric pressure as datum, in ft. of water.				Remarks.
			D u/s $\cong$ 10'	D u/s $\cong$ 20'	D u/s $\cong$ 30'	D u/s $\cong$ 40'	
<div style="display: flex; flex-direction: column; align-items: center; justify-content: center;"> <div style="margin-bottom: 10px;">U/S of crest</div> <div style="margin-bottom: 10px;"> </div> <div style="margin-bottom: 10px;">CREST</div> <div style="margin-bottom: 10px;"> </div> <div>D/S of crest</div> </div>	P <sub>1</sub>	39.9	49.8	61.8	71.3	79.3	
	P <sub>2</sub>	30.75	41.0	52.0	58.9	68.0	
	P <sub>3</sub>	24.00	34.9	44.5	52.9	59.4	
	P <sub>4</sub>	3.00	12.8	18.4	20.3	20.9	
	P <sub>5</sub>	1.13	8.6	11.5	11.4	7.6	
	P <sub>6</sub>	0.0	6.0	8.8	6.3	1.6	
	P <sub>7</sub>	2.5	8.8	5.5	4.4	1.8	
	P <sub>8</sub>	5.25	2.5	3.6	3.0	-0.4	
	P <sub>9</sub>	10.33	0.9	1.8	-0.4	-2.9	
	P <sub>10</sub>	0.90	0.6	1.0	1.4	-1.1	
	P <sub>11</sub>	31.25	1.8	2.8	3.8	3.5	
	P <sub>12</sub>	42.13	1.1	2.2	4.6	5.0	

## 15. REMODELLING OF THE DIMNA NALA OVERFLOW WEIR, MAIN SPILLWAY

### *Synopsis*

The Dimna Nala Reservoir, constructed for supplying water to Jamshedpur, has two spillways but their surplusing capacity is insufficient to pass the maximum possible flood discharge. Model experiments were, therefore, undertaken to modify the weirs and thus increase the discharging capacity to meet a catastrophic flood. This note describes alteration necessary in the sill of the waste weir to pass that discharge.

### *Problem*

Fig. 103

The Dimna Nala Reservoir, formed by an earthen dam, supplies water to Jamshedpur. The reservoir has a catchment area of 36 sq. miles and lies in a belt where severe storms accompanied by heavy rainfall of short duration, are not uncommon. In the scheme, as completed, two spillways are provided. The main spillway leads waste water to a nala whose bed level is 80 ft. below the high flood level 533.0. The fall is constructed in two steps. The weir has a 15 feet level crest at R.L. 524.00 and is 167 feet wide. The emergency spillway consists of an open cut, in a saddle, having a width of 600 feet and 350 feet length in the direction of flow. The average crest level is R. L. 530.00.

The combined capacity<sup>(1)</sup> of the two spillways is inadequate to meet a catastrophic flood which according to the Station fan catchment formula is estimated at 40,000 cs. The following proposals were, therefore, made to meet this emergency:—

(i) to remodel the main spillway so as to pass 21,500 cs. with M. F. L. 533.00.

(ii) to lower the emergency spillway level to R. L. 529.00 so as to pass 10,000 cs. with M. F. L. 533.0.

The town authorities also desired to raise the retention level to R. L. 529.00 by installing gates to provide for the increase in demand in future. The above modifications adequately provide for a 40,000 cs. flood of four hours duration with initial reservoir level at R. L. 529.00 taking into consideration flood absorption up to R. L. 533.00.

Fig. 103

Model experiments were, therefore, carried out,

(a) to modify the main spillway to pass a discharge of 21,500 cs. with M. F. L. at R. L. 533.00;

(b) to improve hydraulic conditions below the spillway for efficient dissipation of energy.

### *Model Experiments for main Spillway*

A model of the main spillway including the swimming pool and tail channel was constructed to 1/40 G. S. scale. This was satisfactory as the depth over sill in the model was  $\frac{533.0 - 524.0}{40} = 0.225$  feet. The effect of surface tension on discharge co-efficient is negligible for this depth. The results of calibrated S.W. F. confirm this as the formula  $Q = CBD^{1.5}$  is applicable down to 0.1 feet depth with only +1.3% correction.

Final proposals for modification of the spillway will be checked for discharge co-efficient with a 1/15 scale model.

### EXPERIMENTS

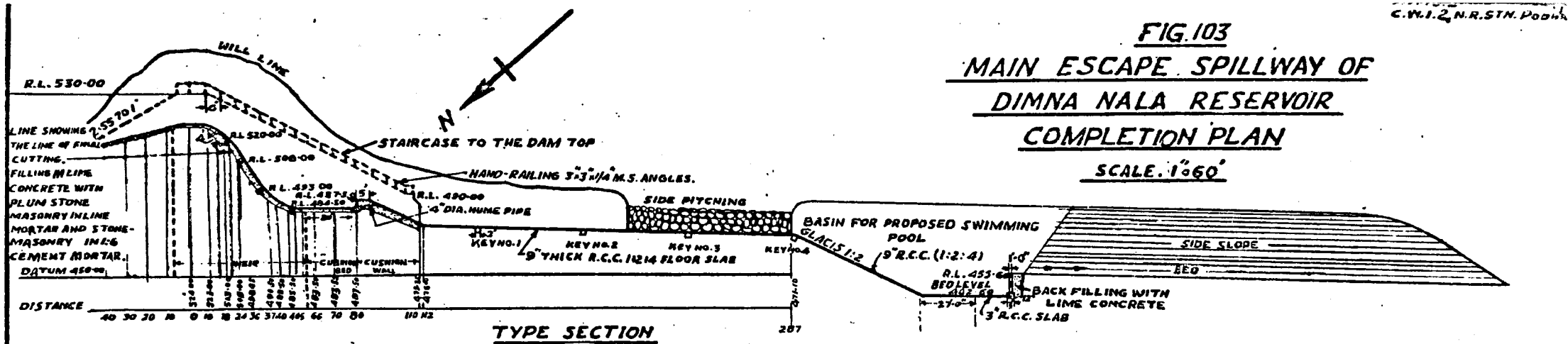
#### *Series 1*

The model was constructed according to completion plan of the spillway. During construction, rock near the left end of the spillway

(1) Annual Report (Tech.) 1948. "Remodelling of the Dimna Nala over-flow Spillway, Jamshedpur water supply scheme".

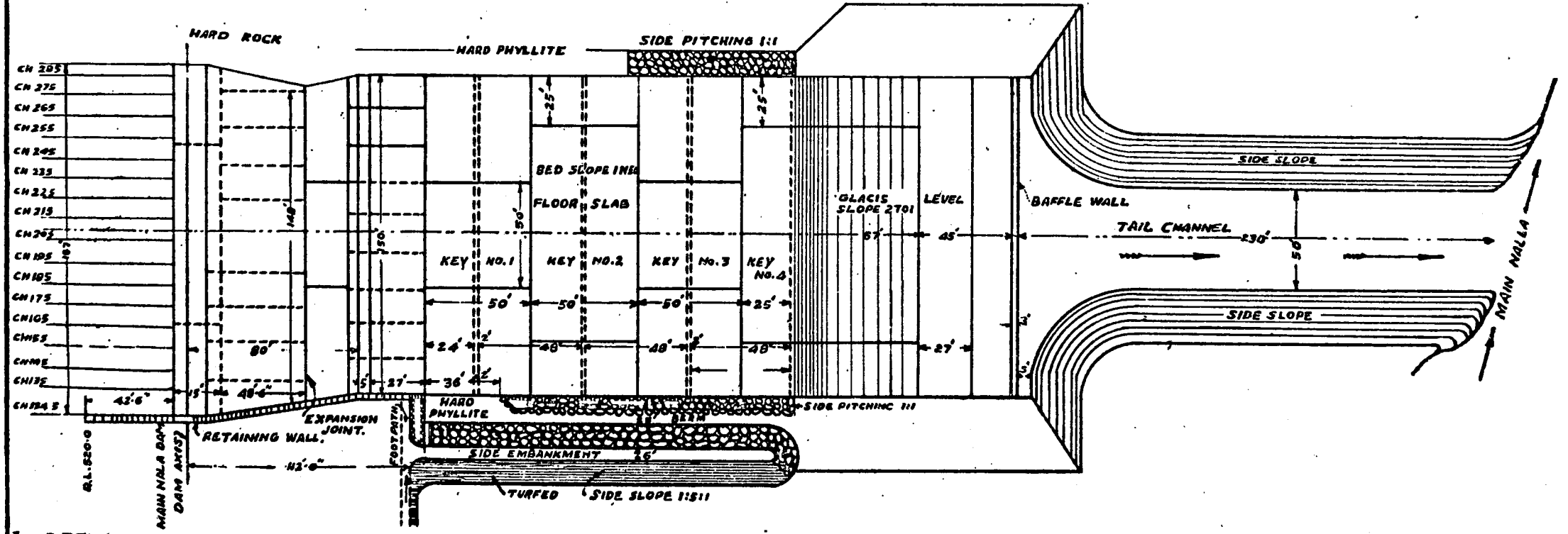
**FIG.103**  
**MAIN ESCAPE SPILLWAY OF**  
**DIMNA NALA RESERVOIR**  
**COMPLETION PLAN**

SCALE: 1"=60'



**TYPE SECTION**

289



By R.T.Thakor, 22.12.43.

## DIMNA NALA SPILLWAY

was not removed: it blocked 49 feet of waterway out of the total 167 feet. As detailed spot levels of this rock projection were not available, its top was kept at R. L. 530.00.

Discharge over the spillway for reservoir levels varying from R.L. 530.00 to R.L. 533.00 were measured in a two feet S.W.F. Reservoir level was observed in the model with a point gauge at 4 feet  $\approx$  160 feet upstream of centre line of spillway. Pressures were observed with piezometric tubes provided along the centre line of spillway.

### Results

The weir with a horizontal sill 15 ft. long in the line of flow and 9 ft. depth of water is expected to act as a broad crested weir approximately and, therefore, should have a co-efficient near about 3.09.

Discharges and corresponding co-efficients C for different upstream water levels are given in Table 1. The co-efficient is assessed from the formula  $Q = CBD^{1.5}$  for  $B = 118$  feet. For reservoir levels above R. L. 530.0 water over-flowed the rock projection and consequently a slightly higher co-efficient was registered. The discharge for M. F. L. 533.00 was only 10,291 cs.

TABLE 1

S. No.	Modification.	Fig. No.	Reservoir level.	Q = Discharge in cs.	Co-efficient C.	Remarks.
	2	3	4	5	6	7
1	Main Spillway as completed with sill at R.L. 524.00 Rock projection top at R. L. 530.0.	104 (A)	533.00	10,291	2.26	Effective $B = 118$ feet.
			531.00	6,250	2.02	
2	Main spillway as completed. Rock projection removed.	104 (B)	533.00	13,220	2.94	Effective $B = 167.0$ feet.
			531.00	9,079	2.94	

### Pressures

Negative pressures occurred only on the curved portion downstream of the horizontal sill, the minimum being 2 ft. Pressures were positive on the rest of the glacis. There was a definite impact on the baffle, the pressure at the centre being +18.6 ft. with no pressure to counteract on the other side.

### Action of baffle

The action of baffle was far from desirable for discharges with reservoir levels above R. L. 528.00. No stilling action took place and a sheet of water was shot up into the air as high as R. L. 517.00 and dropped on the level floor with considerable impact. This, if allowed, in the prototype would destroy the concrete slab.

Insufficient height of baffle (Appendix 1) is mainly responsible for this undesirable behaviour.

In further series of experiments modifications were made to get a higher discharge co-efficient only. After this is finalised in 1/15 scale model, experiments will be done in 1/40 G. S. model to improve flow conditions downstream of glacis. No further reference to the downstream flow conditions will, therefore, be made.

### Series 2.

#### Modifications

Rock projection on the sill of the weir was removed and the width was made equivalent to 167.00 feet. The discharge with reservoir level at R. L. 533.00 was 13,220 cusecs and the co-efficient was 2.94.

Photos  
5672 and  
5673



*Conclusions*

The spillway capacity is much below the desired value of 21,500 cusecs. This can be increased by either of the following two methods:—

- (i) Adopting a high co-efficient weir profile,
- (ii) Lowering the sill level sufficiently.

In the first alternative, the coefficient will have to be increased to 4.76. This will require considerable alterations in the present weir. In the second alternative the sill will have to be lowered to R. L. 520.5 assuming a reasonable co-efficient of 3.0.

To raise the retention level either radial or hinged leaf gates will have to be used which will form an important item in the modification of spillway and it will, therefore, be essential to keep the height of gates small to make the scheme as economical as possible.

*Series 3, 4 and 5* (length of sill = 15 feet).

Fig. 104  
(c, d, e)

In these experiments the sill was lowered to R. L. 522.00. Hump and upstream slope were tested. Modifications are shown in Fig. 104 (c, d, e) and the results are given in Table 2.

Discharge co-efficient, C, is assessed from  $Q = CBD^{1.5}$  where  $B = 167.0$  feet. Velocity of approach being negligible was not taken into account. Discharge observations were made with two reservoir levels 533.0 and 531.0 as a check.

## RESULTS

*Series 3, 4 and 5* (Sill length 15.0 feet)

With a vertical upstream face, the discharge coefficient C was reduced to 2.58. This was due to return roller formed on the upstream bed with consequent loss of head. With a hump, C increased to 2.8. One in four slope which probably acted as a hump with a large radius gave a still higher co-efficient, equal to 2.88. Discharge with upstream slope was 17,050 cusecs with reservoir level 533.00 in series 6.

*Series 6 and 7.* (Sill length = 10 feet)

Fig. 104  
(f and g)

With reduction in the length of sill to 10 feet, the discharge with upstream hump was 17,600 cusecs and 17,860 cusecs with 1 in 4 slope.

*Series 8 and 9.* (Sill length = 5 feet).

Fig. 104  
(h and i)

With sill length cut to 5.0 feet and an upstream hump, the discharge was 18,775 cusecs and C was equal to 3.07. A sloping upstream glacis, however, gave a lower co-efficient, as apparently this prevented full development of curved flow; thus it appeared that upstream characteristics of the weir were closely linked with the length of the sill.

*Series 10 and 11*

Fig. 104  
(j and k)

With the crest at R. L. 522.0 and a hump upstream, the co-efficient of discharge increased to 3.34 with M. F. L. 533.00 and the discharge was 20,391 cusecs. A sloping upstream glacis reduced the discharge to 19,400 cusecs.

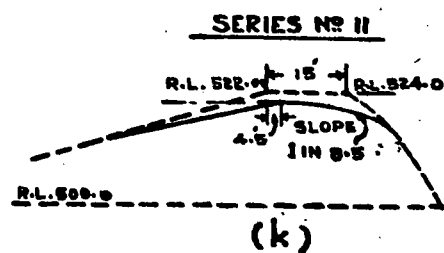
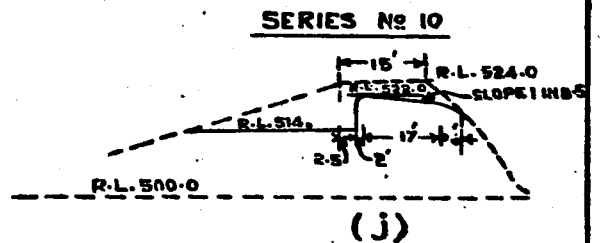
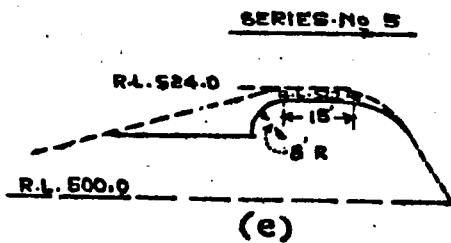
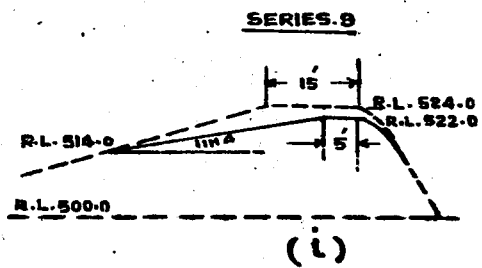
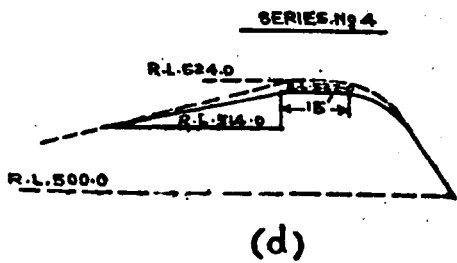
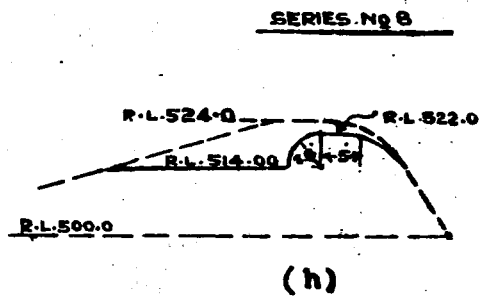
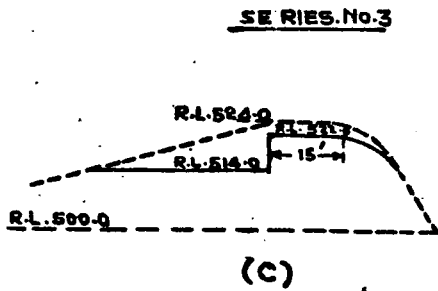
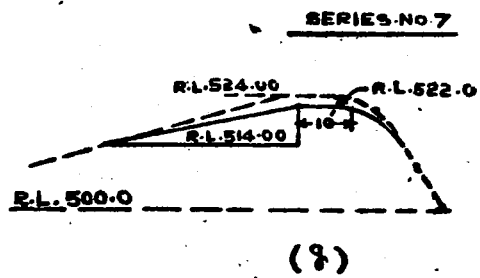
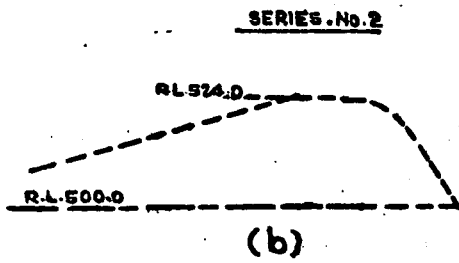
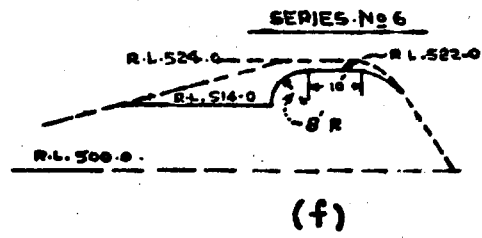
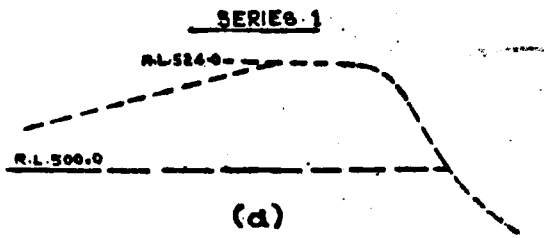
*Conclusions*

Reduction in the horizontal length of sill caused a gradual increase in C. This is as one would expect, due to development of curved flow. Increase in discharge was marked for 5 feet long sill. As the length of sill is made shorter a draw-down effect is produced due to curvature of flow.

In these experiments a sloping upstream glacis was found to be beneficial as compared to hump for longer lengths of horizontal sill i.e. when the weir was a broad crested one. For short length, i.e. when weir became narrow-crested; it was contrary to this, as this prevented the development of curvature.

**FIG. 104(a To k)**  
**MAIN ESCAPE WEIR PROFILE**  
**OF DIMNA NALA RESERVOIR**  
**WITH MODIFICATIONS**

SCALE: 1" = 40'  
Ref: ORIGINAL PROFILE - - - - -  
MODIFIED PROFILE - - - - -



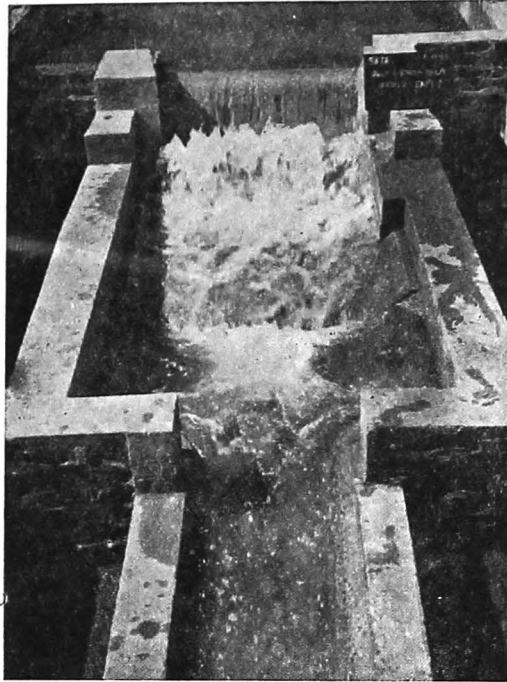


Photo No. 5672—Looking upstream.  
Showing general flow conditions;  
Series 2. Experiment 1. Reservoir  
Level 533.0.

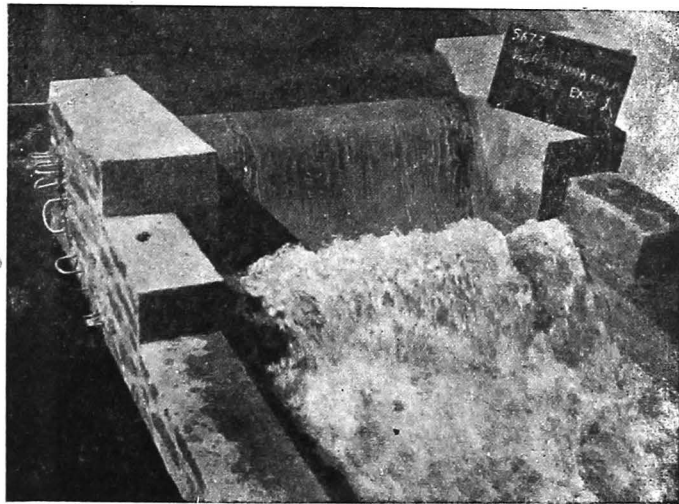


Photo No. 5673—Showing sheet of water thrown  
up by baffle dropping on level floor down-  
stream, Series 2, Experiment 1.

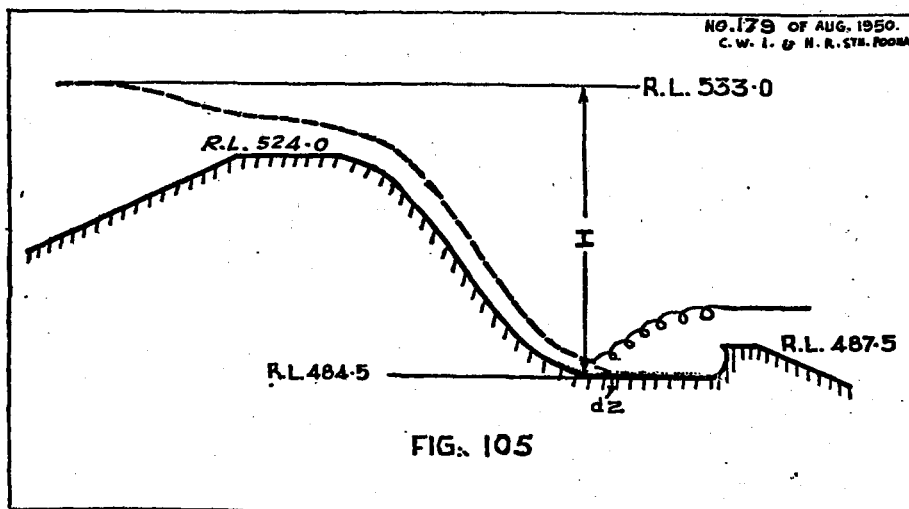
DIMNA NALA SPILLWAY

TABLE 2

S. No.	Modifications.	Fig. No.	Reservoir level.	Discharge Q in cs	Co-efficient C.	Remarks.
3	Sill lowered to R. L. 522.00, length 15', vertical face upstream down to R. L. 514.00.	104 (c)	533.000	15,750	2.58	
			531.00	11,604	2.58	
4	Sill at R. L. 523.00 Length 15 ft., 1 in 4 slope upstream	104 (d)	533.00	17,800	2.85	
			531.00	12,200	2.71	
5	Sill at R. L. 522.00, Length 15 ft., Hump with radius 8 ft. u/s.	104 (e)	533.00	17,050	2.8	Not observed.
			531.00	...	...	
6	Sill R. L. 522.00, length 10 ft., Hump with radius 8 ft. upstream.	104 (f)	533.00	17,600	2.88	
			531.00	12,311	2.73	
7	Sill R. L. 522.00 length 10 ft., 1 in 4 slope upstream	104 (g)	533.00	17,866	2.98	Not observed.
			531.00	...	...	
8	Sill R. L. 522.00, length 5 ft., Hump with 8 ft. radius upstream.	104 (h)	533.00	18,775	3.07	
			531.00	13,523	3.00	
9	Sill R. L. 522.00, length 5 ft., 1 in 4 slope upstream.	104 (i)	533.00	17,169	2.82	
			531.00	13,119	2.90	
10	Sloping sill with crest at R. L. 522.00.	104 (j)	533.00	20,391	3.34	
			531.00	14,634	3.24	
11	Sloping sill with crest at R. L. 522.00, 1 in 4 slope upstream.	104 (k)	533.00	19,400	3.18	
			531.00	14,331	3.17	

APPENDIX 1

Height of baffle and length of cistern.



$$H = 533.0 - 487.5 = 45.5 \text{ feet.}$$

$$V_2 = C\sqrt{2gH}$$

where  $V_2$  = velocity on level floor in ft./sec.

$C$  = Co-efficient depending on Manning's  $N$ , found to be 0.96 from model experiments.

$H$  = Head available, in feet.

$$\therefore V_2 = 0.96\sqrt{2 \times 32.2 \times 45.5} = 53.4 \text{ ft./sec.}$$

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Now, Q, Total discharge = 21,500 cs.

Width of spillway allowing for pier thickness, after gates are installed = 150 feet.

$$\therefore q = \frac{21,500}{150} = 143 \text{ cs./feet.}$$

$$\therefore d_s = \frac{143}{53.4} = 2.68 \text{ feet.}$$

$$\begin{aligned} \text{Now } d_o &= \sqrt[3]{\frac{q^2}{g}} \\ &= \sqrt[3]{\frac{143^2}{32.2}} = \sqrt[3]{635} \\ &= 8.6 \text{ feet.} \end{aligned}$$

Height of baffle should be =  $d_o - d_s$  (2).

$$= 8.6 - 2.68.$$

$$= 5.92 \text{ feet.}$$

The present height of baffle is only = 3.0 feet.

Distance of baffle from toe of glacis  
should be  $5 \times$  height of baffle =  $5 \times 5.92$ .

$$= 29.60 \text{ feet.}$$

Length provided = 25 feet.

## 10. CANAL FALLS

### 16. BASIC STUDY ON A FLUMED VERTICAL FALL

#### 1. Introduction

In continuation of the experiments reported in Annual Report (Tech.) 1947, further experiments were conducted with a 1/6 scale G. S. model of a flumed vertical fall with the object of improving its performance. The data of the design was:

$Q=100$  cs,  $H=8$  feet,  $d_1=d_3=3.58$  feet,  
 $B_1=B_3=16.2$  feet, upstream bed R. L. = 100,  
downstream bed R.L. = 92, Crest R.L. = 101,  
Cistern R.L. = 89.12, Fluming ratio = 50%  
 $m=0.34$  m.m. for local Koregaon sand.

In the experiments reported in Annual Report (Tech.) 1947, with a 2:1 side expansion below parallel throat and a cistern of length 25 feet and width 21.34 ft., there was no bed scour; but, there was side erosion to a certain extent below the cistern. The performance of the flumed vertical fall was, therefore, observed under 25% retrogression with a view to improve upon this aspect.

#### 2. Experiments

The experiments were carried out as under:—

- (1) With a central pier in the parallel throat.
- (2) After raising, lowering or bowing the sill below parallel throat.
- (3) With (a) 2:1 straight side expansion, (b) hyperbolic side expansion, in a 2:1 divergence and (c) parabolic sides with a 2:1 divergence below parallel throat.
- (4) With a standing wave formed under full supply conditions at the downstream edge of the sill.
- (5) With the portion downstream of the parallel throat removed so that the jet fell directly into the cistern from the end of parallel throat.
- (6) With (a) longitudinal vanes (b) staggered blocks, fixed inside the cistern.

The modifications introduced in the design under the present series of experiments are shown in Table 1.

Table 1.

#### 3. Procedure

The bed and banks of the downstream channel in the model were moulded with sand of 0.34 mm. diameter. The model was run for a period of 48 hours and in every experiment observations were made to study.

- (i) the distribution of discharge intensity over the sill below parallel throat,
- (ii) flow conditions, velocities, etc., in the cistern and in the downstream channel, and
- (iii) bed and bank erosion.

#### 4. Results

Results of these experiments are exhaustively given in Table 1.

#### 5. Comparison of Results

(a) Velocity distribution at a cross-section 30 feet downstream of the downstream end of the cistern.

With a raised sill below parallel throat and 2:1 side expansion in experiment No. 1 (vide Table 1) the velocity distribution was unsatisfactory

because of high velocity near sides and non-uniform distribution of velocity contours as shown in Fig. 106(a). The intensity of the two side jets in the downstream channel was, however, found to be reduced when compared with that in the experiment with flat crest and the velocity distribution under which is shown in fig. 106(b). A central pier in the parallel throat also failed to bring about satisfactory flow conditions in the downstream channel because distinct side jets persisted near the banks as shown by velocity distribution in Fig. 106(c). The velocity distribution was found to be normal and satisfactory with submerged longitudinal vanes of height  $\frac{d_s}{4}$  in the cistern as shown in Figs. 107(a), 107(b) and 107(c), where  $d_s$  is depth of water in the cistern. There were no side jets and the distribution of isotachs was uniform. The best velocity distribution, was, however, found to be in Experiment No. 5 (a) vide Table 1, with 2:1 hyperbolic side expansion below parallel throat and three submerged longitudinal vanes of height  $\frac{d_s}{4}$  and spacing  $\frac{B}{4}$  inside the cistern as can be seen from Figs. 107(b) and 108, where B = bed width of the channel.

(b) *Side erosion*:—Cross sections at 0.6 and 12 feet downstream of the cistern (fig. 110) showed a decrease of side erosion with a central pier in the parallel throat as compared with that in experiments with flat or raised sill below the parallel throat. The extent of bed and side erosion with and without a central pier is shown in fig. 111. With three submerged longitudinal vanes inside the cistern, a minimum bed and bank scour was obtained with 2:1 hyperbolic side expansion as shown in Fig. 114.

## 6. Conclusions.

The investigation so far conducted has helped to arrive at the following conclusions:—

(1) Expansion of the hypercritical jet above a flumed vertical fall by 2:1 side expansion below parallel throat resulted in negligible bed erosion but a considerable side erosion.

(2) Raising, lowering or bowing of the sill below parallel throat was not of much help in controlling the performance of the flumed vertical fall.

(3) The side erosion was minimised with the introduction of a central pier in the parallel throat but the central pier also failed to bring about by itself quite satisfactory conditions of flow in the downstream channel below the fall, besides affecting the co-efficient of discharge.

(4) The side erosion and bed erosion were completely eliminated by forming a standing wave at the downstream edge of the sill under full supply conditions but the flumed fall could not work as meter under low supply conditions as it caused afflux.

(5) Submerged longitudinal vanes, suitably spaced inside the cistern, were found to be quite effective in minimising the side erosion as well as enforcing a satisfactory velocity distribution downstream of the cistern by preventing cross-flow in it.

(6) With submerged longitudinal vanes fixed inside the cistern, a 2:1 hyperbolic side expansion below parallel throat produced better results as compared with a straight or parabolic side expansion of splay 2:1. This was the final design.

## 7. Further experiments.

In order to standardize the design of flumed vertical fall, H will be first varied and the discharge kept at 100 cusecs. The designs for H = 2', 4' and 6' will be tested. After these experiments, the discharge will be varied and experiments will be carried out to evolve a standard design.

(a)

RAISED SILL BELOW PARALLEL THROAT.

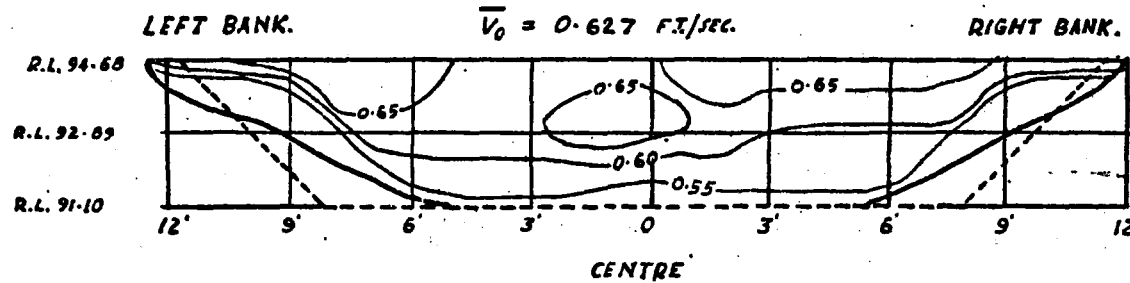


FIG. 106

FLUMED VERTICAL FALL.

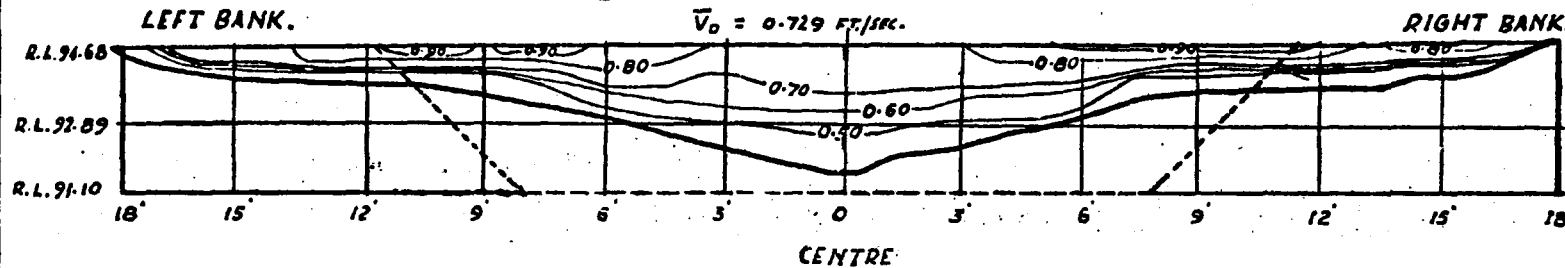
VELOCITY DISTRIBUTION AT CROSS SECTION 30 FT.

0/5 OF 0/5 END OF CISTERN.

SCALE: 1" = .75' (MODEL)  
 1" = 4.5' (PROTO)

(b)

LEVEL SILL BELOW PARALLEL THROAT.

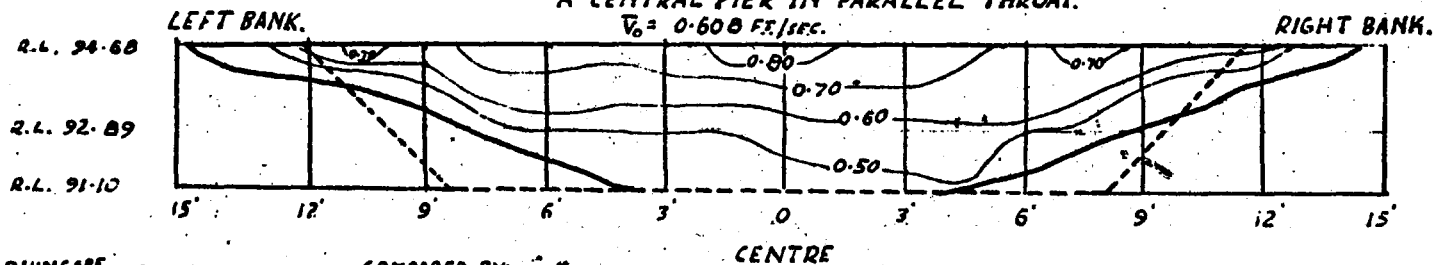


REFERENCES:-

ORIGINAL BED: - - - - -  
 ERODED BED: —————

(c)

A CENTRAL PIER IN PARALLEL THROAT.

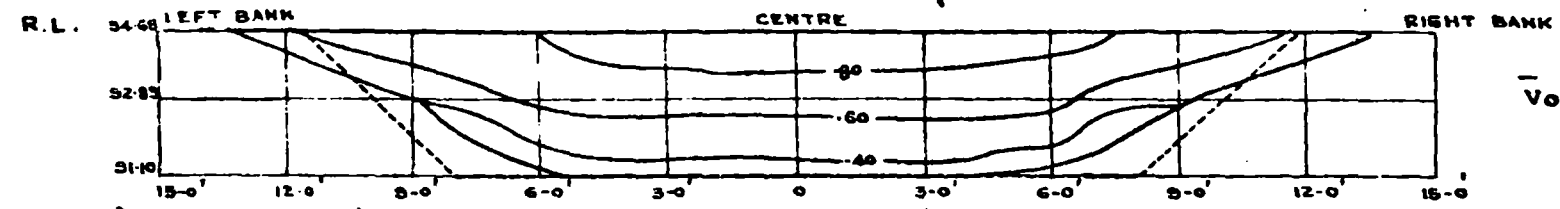


303



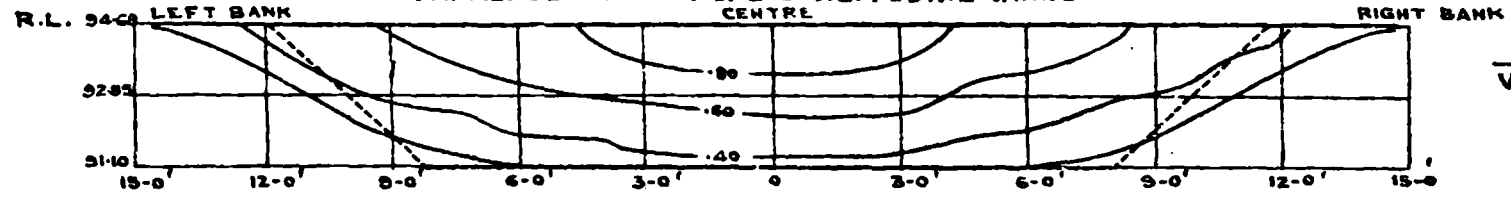
FIG. 107 a TO C  
 FLUMED VERTICAL FALL  
 VELOCITY DISTRIBUTION AT CROSS SECTION 30 FT. D/S  
 OF D/S END OF CISTERN  
 SCALE 1" = .75' (MODEL)  
 = 4.5' (PROTO)

FIG (a) 2:1 STRAIGHT SIDE EXPANSION BELOW  
 PARALLEL THROAT & 3 LONGITUDINAL VANES



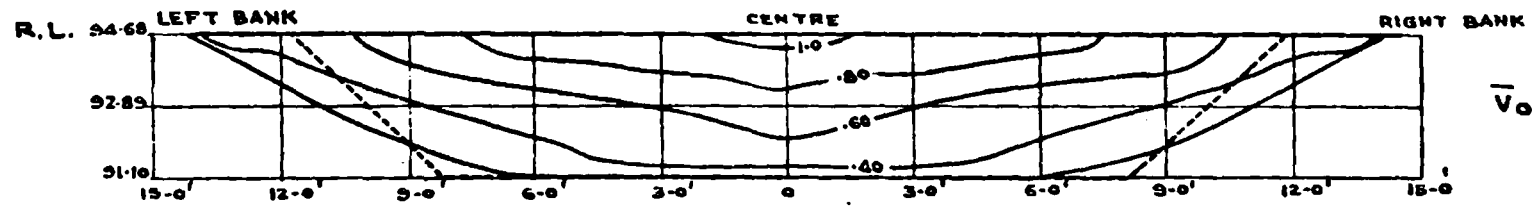
$\bar{V}_0 = 0.629$  FT/SEC.

FIG. (b) 2:1 HYPERBOLIC SIDE EXPANSION BELOW  
 PARALLEL THROAT & 3 LONGITUDINAL VANES



$\bar{V}_0 = 0.569$  FT/SEC.

FIG. (c) 2:1 PARABOLIC SIDE EXPANSION BELOW  
 PARALLEL THROAT & 3 LONGITUDINAL VANES



$\bar{V}_0 = 0.589$  FT/SEC.

REF. --- ORIGINAL BED  
 ——— ERODED BED

COMPARED BY- *[Signature]*  
 TRACED BY- PATKIS. 27-2-50

305

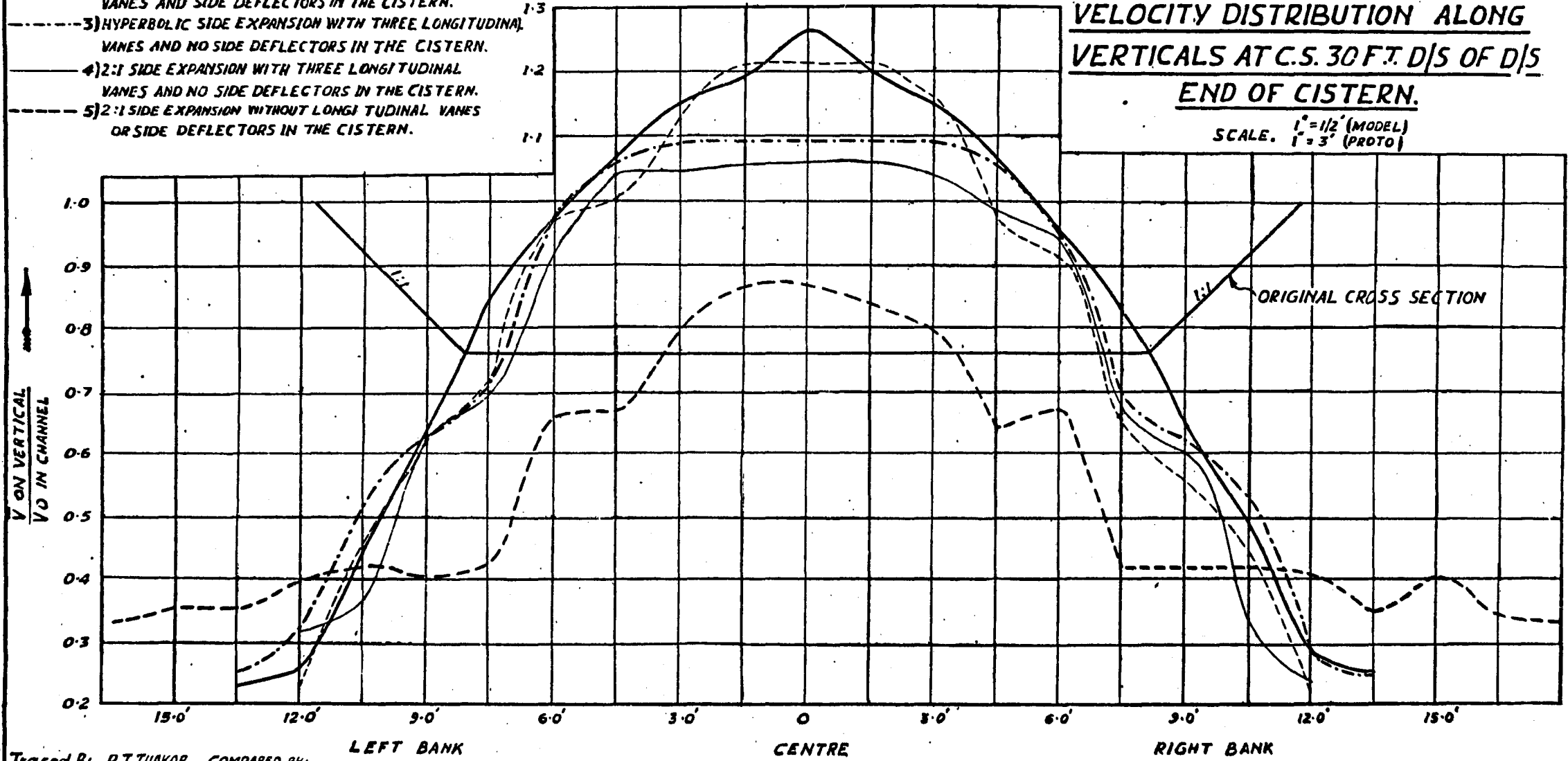
Ref. ~

FIG. 108

FLUMED VERTICAL FALL  
VELOCITY DISTRIBUTION ALONG  
VERTICALS AT C.S. 30 FT. D/S OF D/S  
END OF CISTERN.

SCALE.  $\frac{1''}{1'} = \frac{1}{2}$  (MODEL)  
 $\frac{1''}{3'} = \frac{1}{3}$  (PROTO)

- 1) PARABOLIC SIDE EXPANSION WITH THREE LONGITUDINAL VANES AND SIDE DEFLECTORS IN THE CISTERN.  
 2) HYPERBOLIC SIDE EXPANSION WITH THREE LONGITUDINAL VANES AND SIDE DEFLECTORS IN THE CISTERN.  
 3) HYPERBOLIC SIDE EXPANSION WITH THREE LONGITUDINAL VANES AND NO SIDE DEFLECTORS IN THE CISTERN.  
 4) 2:1 SIDE EXPANSION WITH THREE LONGITUDINAL VANES AND NO SIDE DEFLECTORS IN THE CISTERN.  
 5) 2:1 SIDE EXPANSION WITHOUT LONGITUDINAL VANES OR SIDE DEFLECTORS IN THE CISTERN.



Traced By: R.T. THAKOR. COMPARED BY:

*[Handwritten signature]*

PLAN

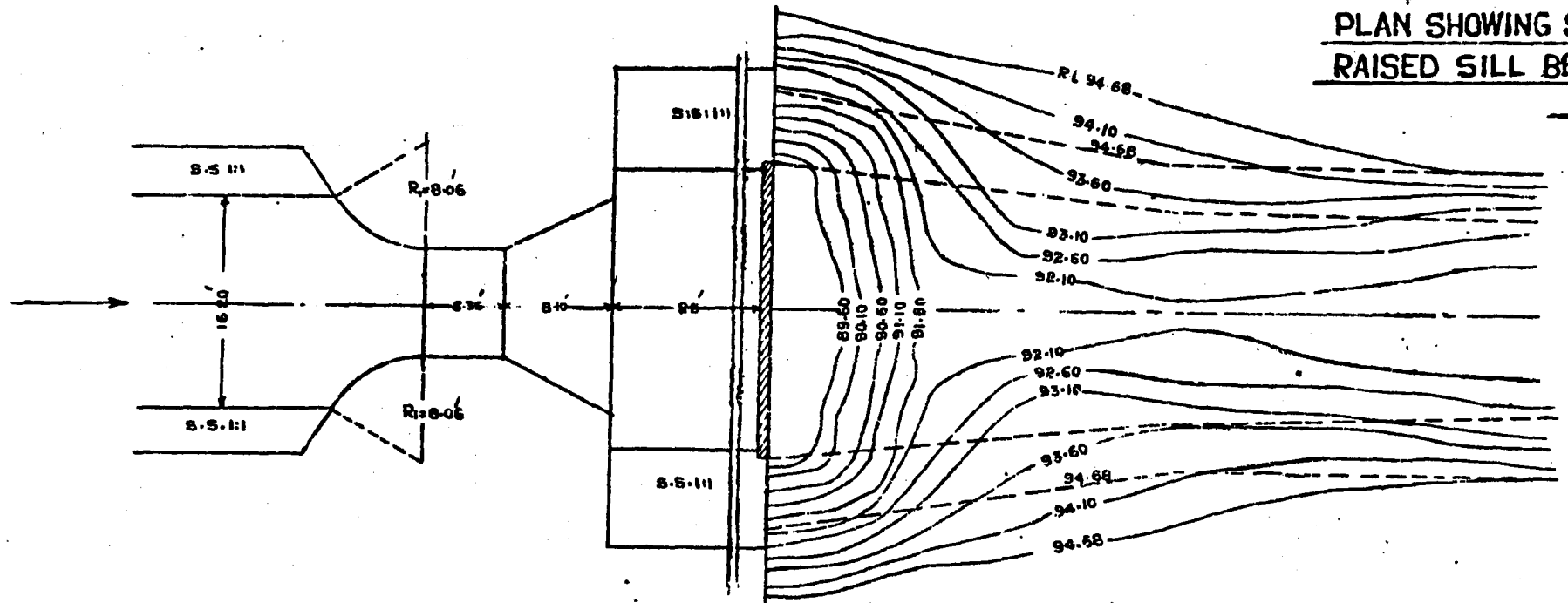
FIG. 109

**FLUMED VERTICAL FALL  
PLAN SHOWING SCOUR PATTERN WITH  
RAISED SILL BELOW PARALLEL  
THROAT**

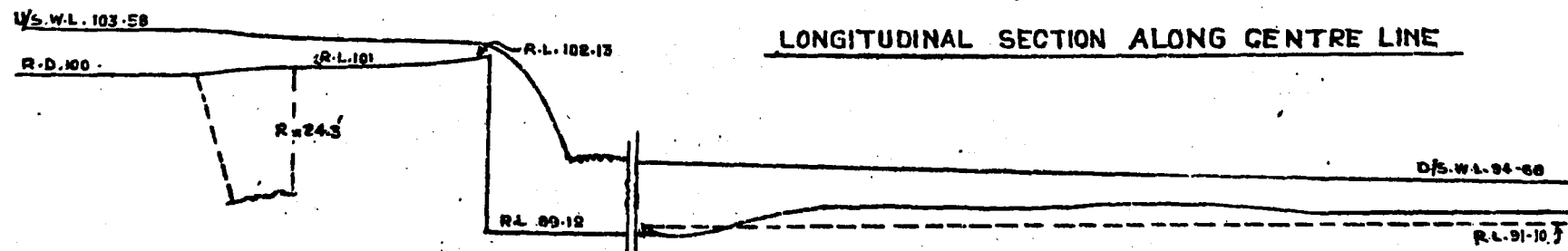
THROAT

SCALE: 1" =  $\frac{2}{12}$  (MODEL)  
 $\frac{12}{12}$  (PROTO)

REF  
ORIGINAL BED - - - - -  
ERODED BED - - - - -



**LONGITUDINAL SECTION ALONG CENTRE LINE**



309

Drawn by R.K. Handia

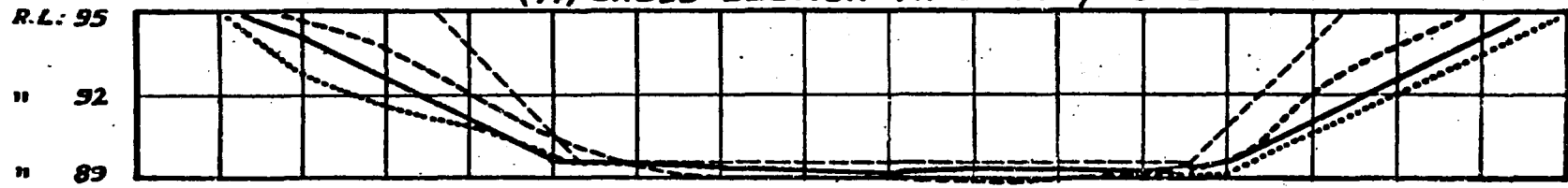
C.BY *[Signature]*  
785

**FIG. 110**  
**FLUMED VERTICAL FALL**  
**CROSS-SECTIONS SHOWING SIDE EROSION**  
**SCALE: 1"=1' (MODEL.); 1"=6' (PROTO.)**

**LEFT BANK.**

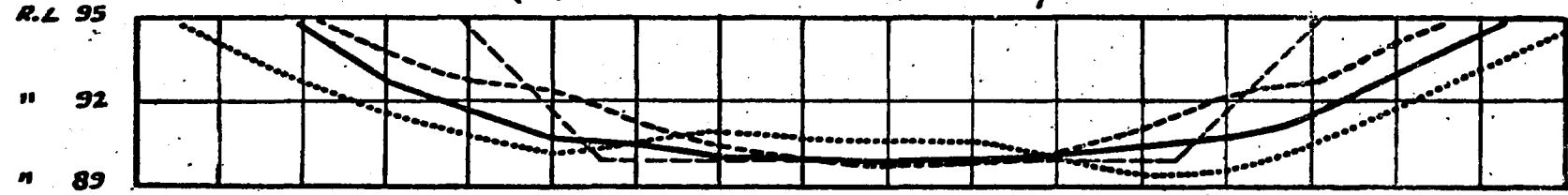
**RIGHT BANK.**

**(A) CROSS-SECTION AT 0-0 FT. D/S OF CISTERN.**

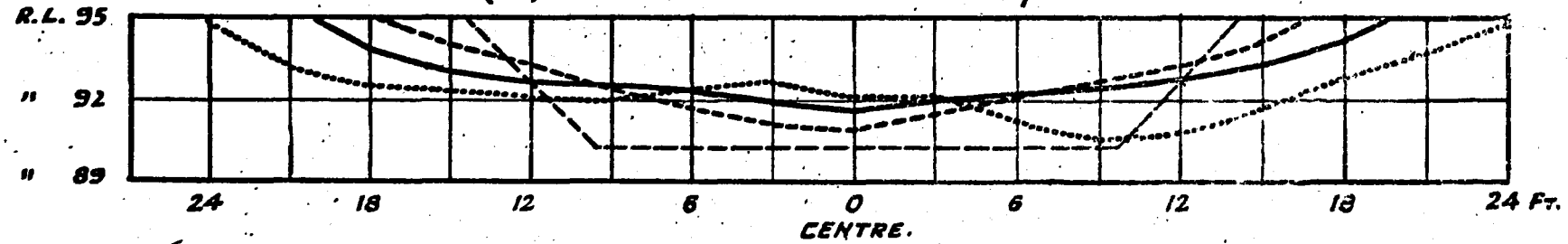


- REFERENCES.**
- ORIGINAL BED. ....
  - ERODED BED - WITH FLAT SILL. - - - -
  - ERODED BED - WITH RAISED SILL. \_\_\_\_\_
  - ERODED BED - WITH 0.6 FT. WIDE PIER. - . - . - .

**(B) CROSS-SECTION AT 6-0 FT. D/S OF CISTERN.**



**(C) CROSS-SECTION AT 12-0 FT. D/S OF CISTERN.**



CENTRE.

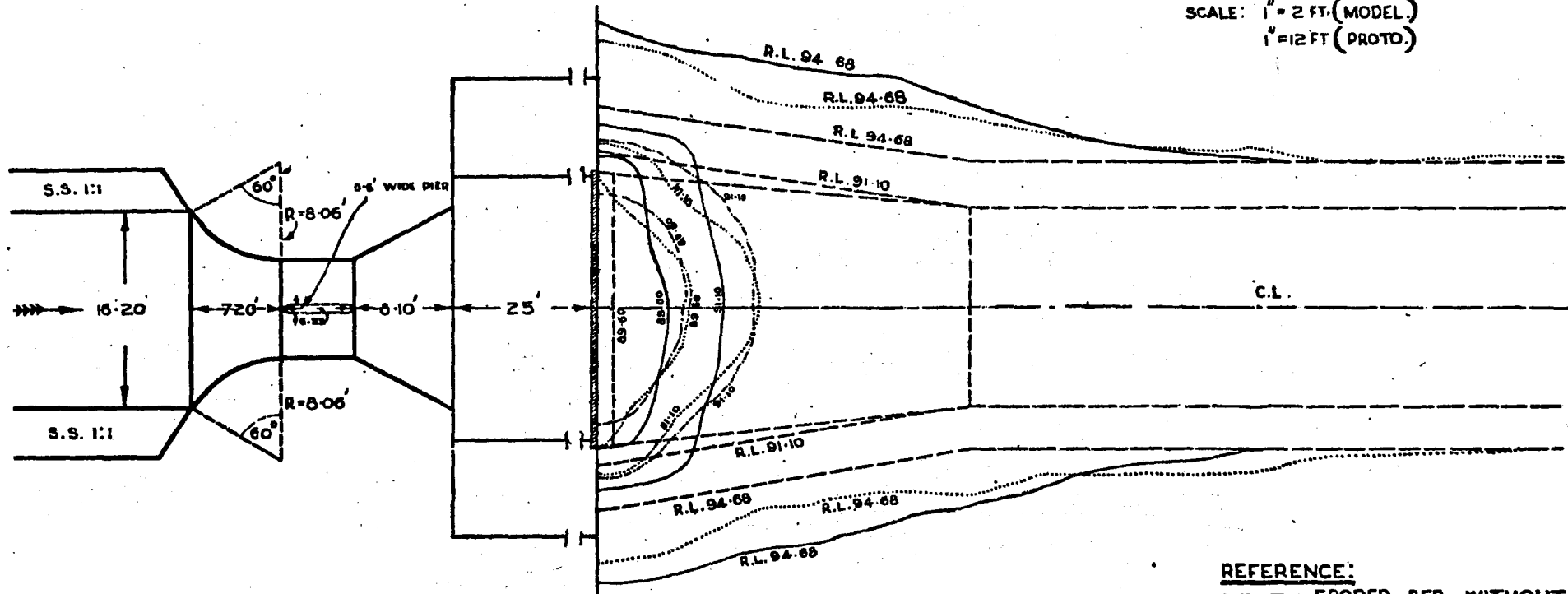
311

*[Handwritten signature]*  
78

**FIG. III**  
**FLUMED VERTICAL FALL**  
**PLAN SHOWING BED AND SIDE EROSION**

SCALE: 1" = 2 FT. (MODEL)  
 1" = 12 FT. (PROTO)

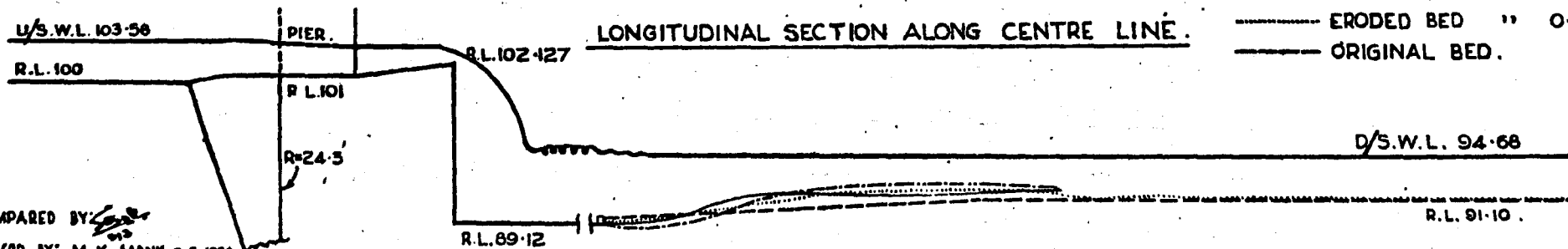
PLAN



**REFERENCE:**

- ERODED BED WITHOUT PIER.
- ERODED BED WITH 0.25 FT WIDE PIER
- · - · - ERODED BED " 0.6 FT " "
- ORIGINAL BED.

**LONGITUDINAL SECTION ALONG CENTRE LINE.**

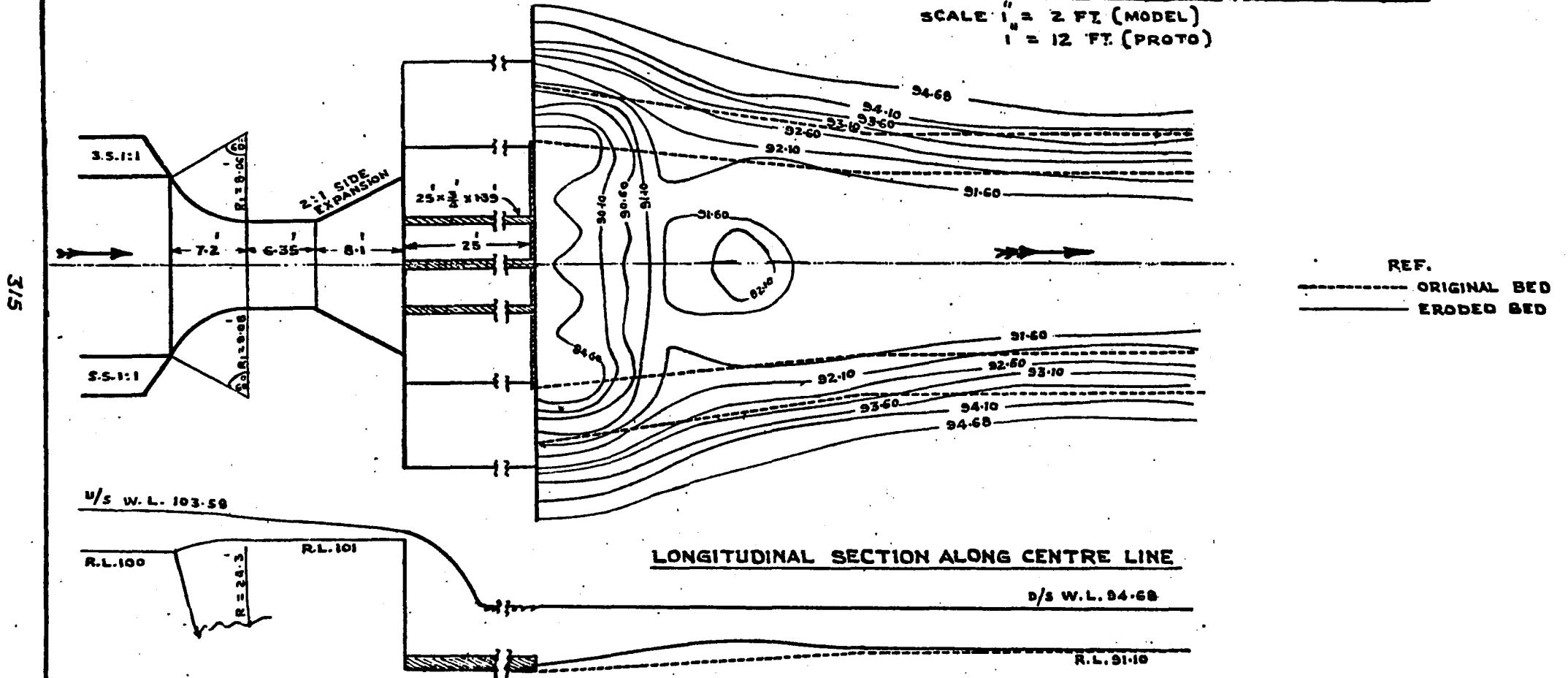


COMPARED BY: *[Signature]*  
 TRACED BY: M.K. JANNIS, 2 3-1950

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FIG.112  
FLUMED VERTICAL FALL  
PLAN SHOWING SCOUR PATTERN  
WITH 2:1 SIDE EXPANSION BELOW PARALLEL THROAT  
& THREE LONGITUDINAL VANES OF HEIGHT  $\frac{D}{4}$  IN THE CISTERN

SCALE  $\frac{1}{4}'' = 2$  FT. (MODEL)  
 $1'' = 12$  FT. (PROTO)

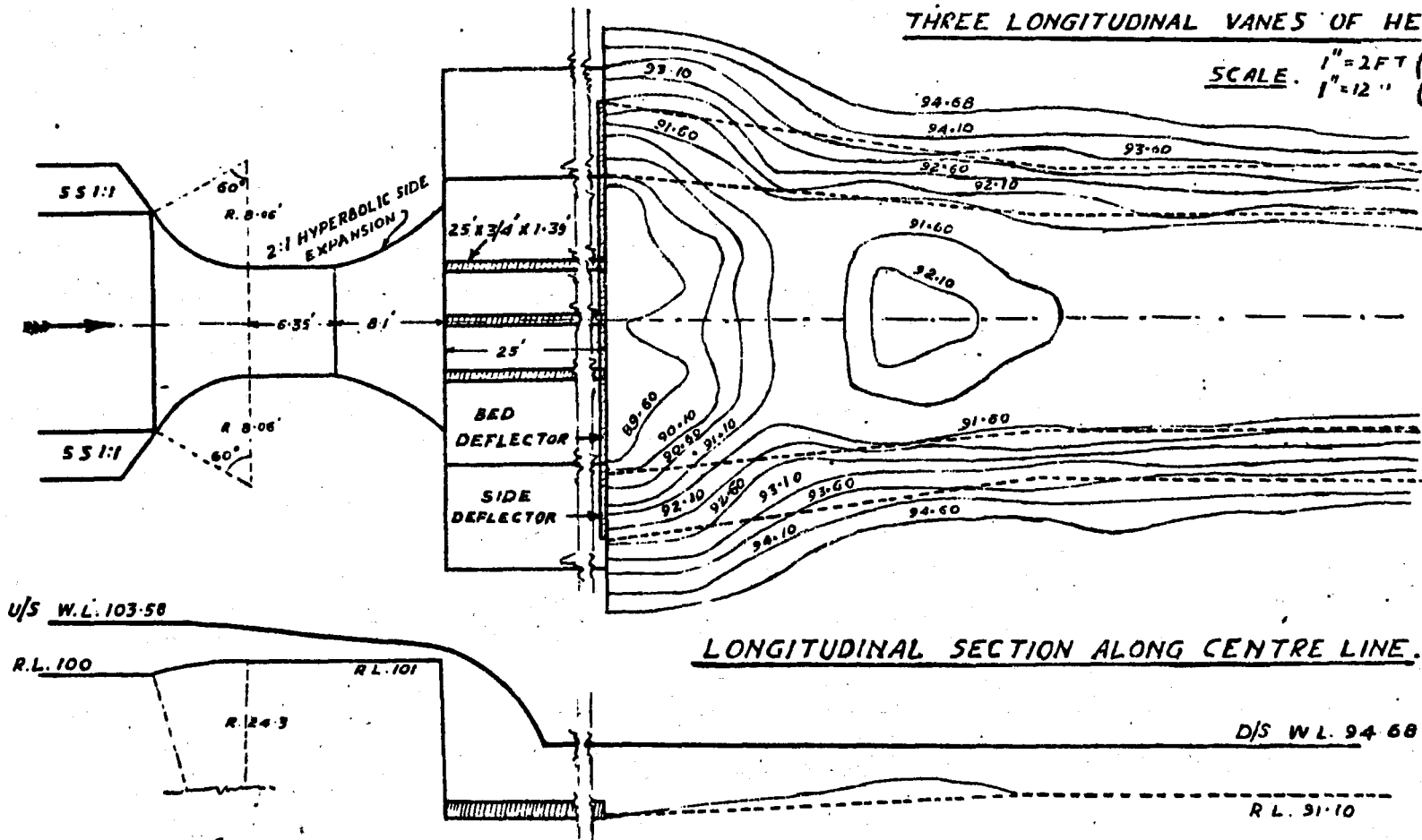


COMPARED BY - *[Signature]*  
TRACED BY - PATRIE. 1-3-50

**FIG. 113**  
**FLUMED VERTICAL FALL**  
**PLAN SHOWING SCOUR PATTERN WITH**  
**2:1 HYPERBOLIC TRANSITION BELOW PARALLEL THROAT**  
**AND**  
**THREE LONGITUDINAL VANES OF HEIGHT  $D/4$  IN THE CISTERN.**

SCALE. 1" = 2 FT (MODEL)  
1" = 12" (PROTO)

REF.  
ORIGINAL BED - - - - -  
ERODED BED - - - - -

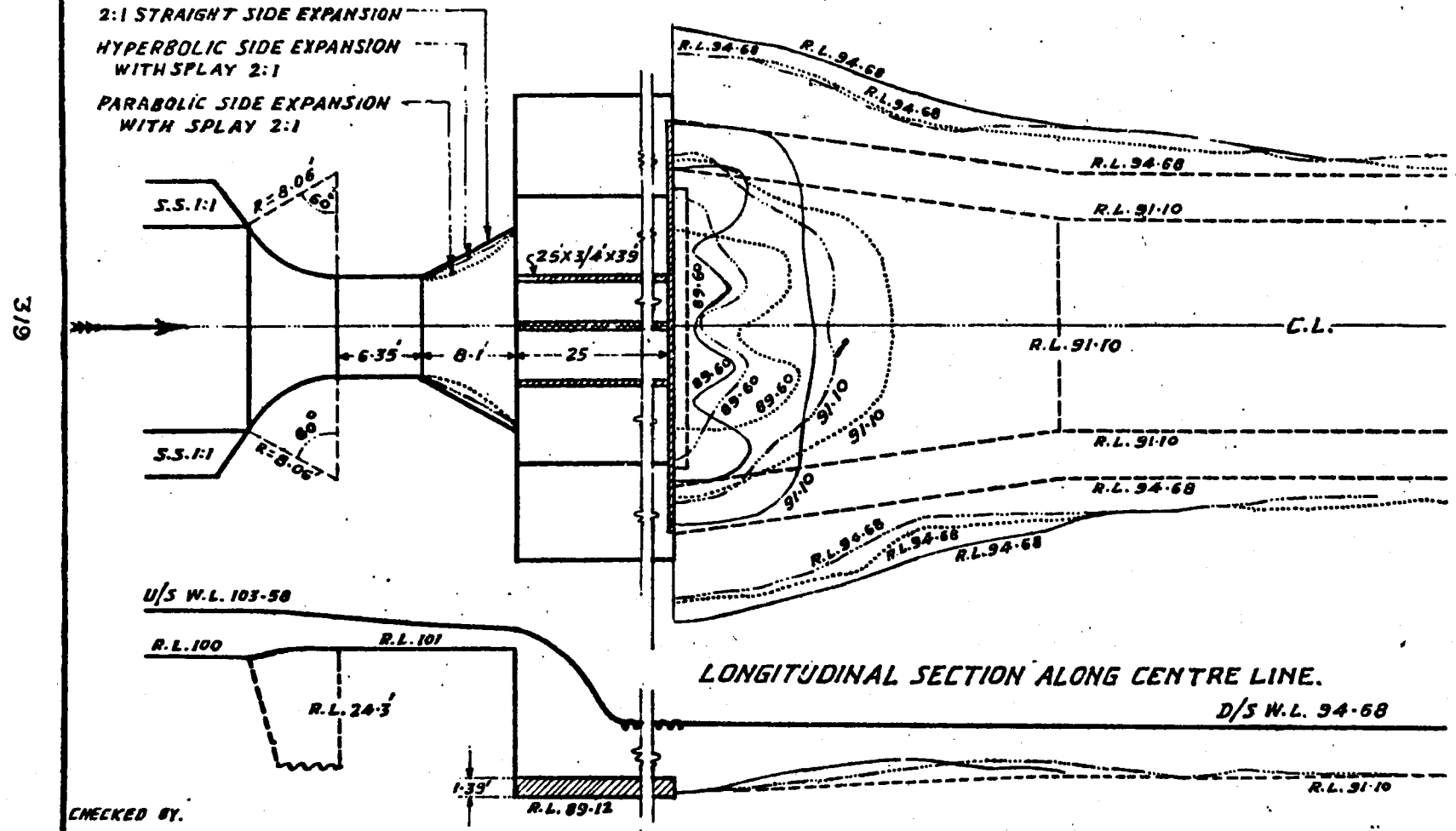


317

Desg: P.T. Tikur Compared by: *[Signature]*

**FIG. 114**  
**FLUMED VERTICAL FALL**  
**PLAN SHOWING BED AND SIDE EROSION.**

SCALE:  $1'' = 2'$  (MODEL)  
 $1'' = 12'$  (PROTO)



- REFERENCES.**
- ORIGINAL BED. -----
  - ERODED BED WITH 2:1 STRAIGHT SIDE EXPANSION. } -----
  - ERODED BED WITH HYPERBOLIC SIDE EXPANSION } -----
  - ERODED BED WITH PARABOLIC SIDE EXPANSION. } -----

CHECKED BY.  
TRACED BY- SAPKAL.



## FULMED VERTICAL FALL

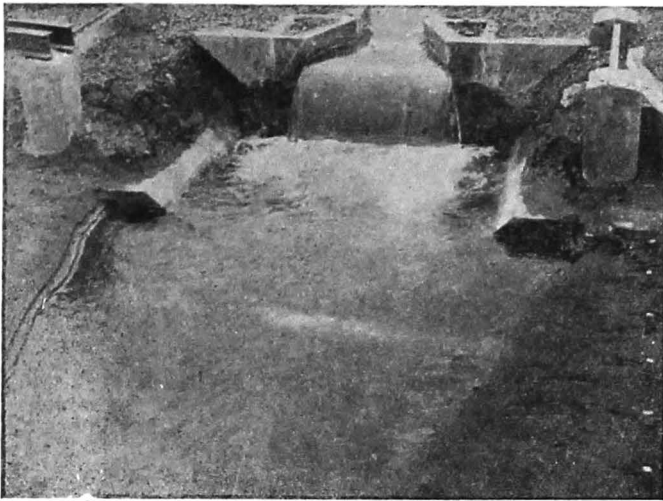


Photo No. 5241—Looking upstream, shows flow conditions with a raised sill below parallel throat and 2 : 1 side expansion.

Photo No. 5287—Looking downstream, shows the flow conditions after introducing a 0.15 feet central pier in the parallel throat.

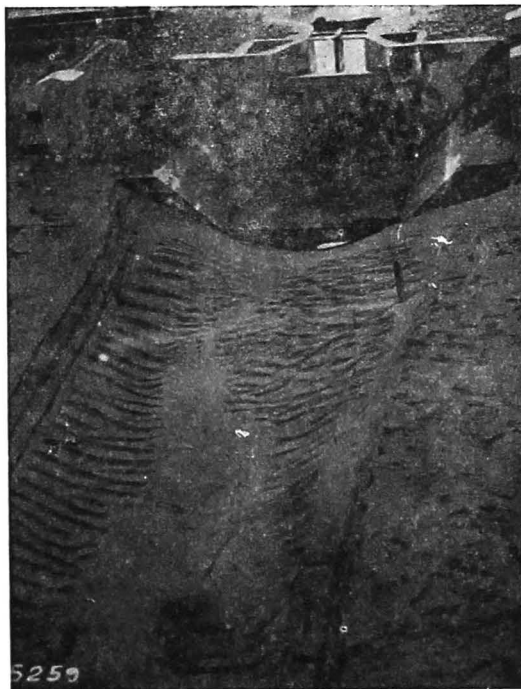
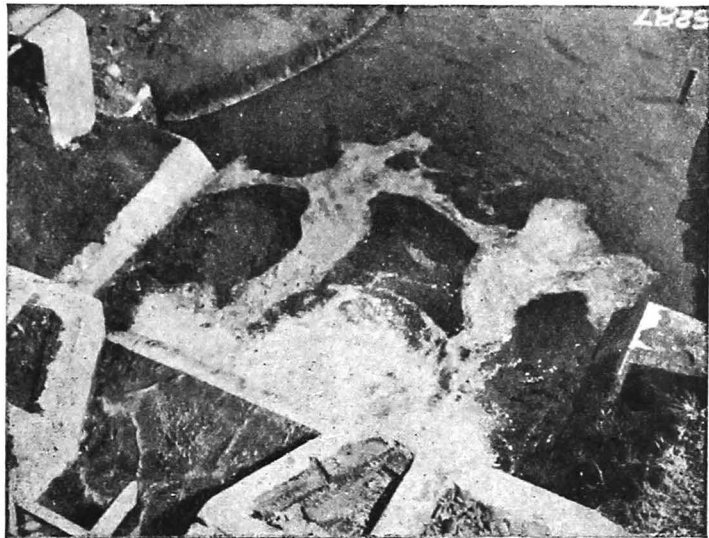


Photo No. 5259—Looking upstream, shows the scour pattern after a run of 28 hours with a 0.6 feet wide central pier in the parallel throat.

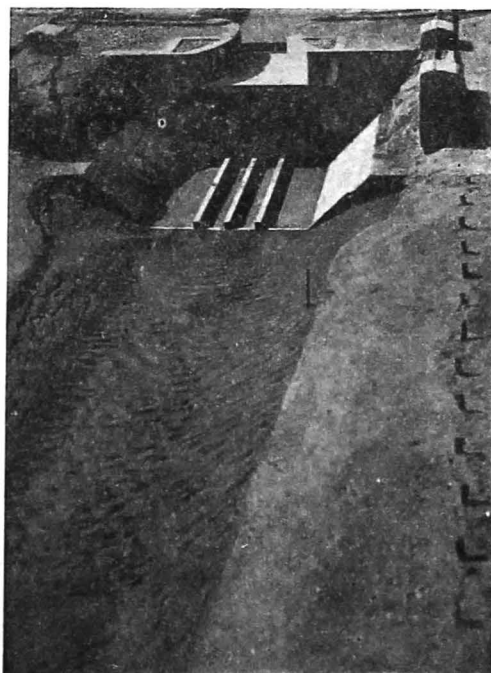
## FLUMED VERTICAL FALL

Photo No. 5330—Looking downstream, shows the two side jets of forward flow with a return flow in the centre and near banks after bowing the horizontal sill below parallel throat.



Photo No 5513—Looking downstream, shows the flow pattern with hyperbolic side expansion below parallel throat and three longitudinal vanes inside the cistern.

Photo No. 5512—Looking upstream, shows the scour pattern under conditions of flow of Photo No. 5513 above.



## FLUME VERTICAL FALL



Photo No. 5539—Looking downstream, shows central jet of very high velocity accompanied with two, low velocity, side jets in the cistern fitted with three longitudinal vanes after introducing a parabolic side expansion below parallel throat.

Photo No. 5544—Looking downstream, shows the flow pattern with a parabolic transition below parallel throat after removing the three longitudinal vanes from inside the cistern



Photo No. 5543—Looking upstream, shows the scour pattern after a run of seven hours under conditions of flow depicted in Photo No. 5544. Notice the heavy deposit of eroded material formed by the movement of sand from the eroded sides towards the central portion.

## FLUME VERTICAL FALL

Photo No. 5632 - Looking upstream, shows negligible scour of bed and sides with a standing wave formed at the downstream edge of sill



Photo No. 5658 - Looking downstream, shows the flow pattern with a standing wave formed at the downstream edge of sill under full supply conditions.

Photo No. 5746 - Looking downstream, shows a concentrated forward flow in the centre with back rollers near the sides with the jet falling into the cistern at the downstream end of parallel throat.



## 6. SPILLWAYS

### 17 WASTE-WEIR AT BHANDARDARA (BOMBAY) SYPHON SPILLWAY

#### *Problem*

The maximum-runoff in the Bhandardara Catchment according to the fan catchment formula ( $Q=7000A/\sqrt{A+4}$ ) is 51,000 cusecs. A syphon spillway is to be provided to pass this discharge without exceeding M.F.L. 269.5; the dam top being at R.L. 270.0. The spillway is, therefore, to be designed to pass about 85.0 cs./foot with 269.5 as reservoir level. It is also necessary to have as small priming depth as possible so that the retention level can be raised to the limit consistent with present top level of the dam. R.L. 270.00.

#### *Previous Experiments*

A syphon design with 4.0 ft. barrel depth was tested. Undercutting on the downstream side of syphon was taken to R.L. 248.00, present waste-weir level being about 260.00. Maximum discharge passed with this design was 72.0 cs./ft. It would be uneconomical to get the desired discharge by lowering the downstream leg by further cutting of rock. The design was, therefore, modified, the following being the pertinent dimensions:—

Fig. 115.

Depth of barrel	...	4.5 feet.
Crest level of weir	...	269.00 feet.
Lip level of outlet	...	252.50 feet.

#### *Series 1*

Two syphons were constructed each 1.0 ft. wide with a partition wall of 0.1 ft. thick to a scale of  $\frac{1}{8}$ . A priming weir was provided downstream of the syphon with 1:3 and 1:1 upstream and downstream slopes respectively. Bed, upstream of the syphon to a length of 40'≅320' was made rough, which represented a typical section in the prototype weir.

Fig. 116

Priming depth and discharge per ft. were observed with modifications as given in Table 1. Priming depth was always observed at 40 ft.≅320 ft. upstream of the syphon crest i.e. in the reservoir itself. Similarly gauge at 40 ft.≅320 ft. upstream of syphon line was maintained at R.L. 269.5 for all discharge observations.

For assessing co-efficient of discharge of syphon, upstream water level in the lake is assumed as R. L. 269.5. This introduces a slight error as loss of head in 320 feet length is not deducted from available head. As the variation in water level near the syphon, while running full, was 0.08 ft.≅0.64 ft., estimation of frictional losses became unreliable and hence were not considered.

#### *Results*

The design was satisfactory as regards discharge, being 91.6 cs./ft. against 72 cs./ft. in the previous design. Priming depth, however, was much in excess of the maximum allowable viz. 0.75 feet.

#### *Series 2*

To improve priming characteristics of the syphon, the downstream leg was gradually contracted from 4.5 feet at the crest to 4.0 feet at the outlet. Results with different sill levels of priming weir and bed levels are given in Table 2.

#### *Results*

Contraction of the downstream leg caused a reduction of priming depth to 0.99 ft. as well as that of discharge to 86.2 cs./ft. Priming depth

attained with this design was still above the desired value. Further contraction of downstream leg was not adopted as it would have reduced discharge below the permissible value.

#### Series 3

Fig. 115. A triangular ledge, its upstream face making an angle of  $150^\circ$  with leg of syphon was tested. Advantage of this shape lies in the fact that an angle iron embedded in concrete can easily be provided in the prototype.

Two sizes were tested, one with 0.48 feet projection and the other 0.4 feet projection. The outer shell was removed. The throw-off with these, fixed at R.Ls. 262.0 and 260.0 was observed in  $\frac{1}{4}$  G.S. model for 0.56, 0.96 and 1.36 feet upstream depths of water above syphon crest.

#### Results

Ledge with 0.4 feet projection at R.L. 260.00 was satisfactory. The water jet met the outer leg just above outlet level. The projected area was also less.

#### Series 4

Fig. 117. Ledge with 0.4 ft. projection was fitted at R.L. 260 in  $\frac{1}{4}$  G. S. model. In second experiment the syphon outlet was given an expansion to recover head and thus reduce losses. Results are given in Table 3.

#### Results

The performance of the syphon improved considerably by addition of a ledge. Priming depth reduced to 0.3 ft. as against 0.75 ft. required. A discharge of 89.2 cs./ft. compares quite favourably with 91.8 cs./ft. in Series 1 (Table 1, Experiment 1).

Expansion of the outlet gave a slightly higher discharge, 90.0 cs./ft. while increased the priming depth to 0.38 feet. This was due to ineffective sealing of the outlet due to expansion.

Fig. 115. Pressures were observed with piezometric tubes fitted in the model at points shown in Fig. 115. These are given in Table 4.

Maximum negative pressure of the order of 19.52 ft. occurred at the crown in Experiment 2 with expanded outlet. Pressures in the vicinity of ledge are of particular interest. These being positive on the upstream face; below the ledge, though the pressure was negative, it was safely below cavitation limit.

#### Further tests

The syphon design with 4.5 feet barrel depth and a ledge is satisfactory as it complies with the discharge and priming depth requirements. It can be improved still further in the following respects.

Comparison of coefficients of discharge of syphon with and without expansion will show that the latter is much higher, though apparently the performance is nearly the same. This is due to the presence of priming weir which causes submergence of outlet. If priming weir is removed, syphon can pass a much higher discharge with expanded outlet.

In the case of Bhandardara waste-weir it is of no use to design a syphon for discharge higher than 85 cs./ft. this being limited by approach length of the Waste-Weir.

If it is possible to attain a small priming depth without priming weir, the outlet of syphon can be placed at higher level (present level of centre of outlet being 252.25). This will reduce cost of syphon as well as that of rock cutting.

The following experiments will be conducted:—

(i) To see priming depth and discharge of present syphon with expanded outlet and ledge fixed with priming weir removed.

(ii) To test a syphon design with expanded outlet, centre of outlet being at R.L. 255.0.









**FIG. 117**  
**BHANDARDARA SYPHON**

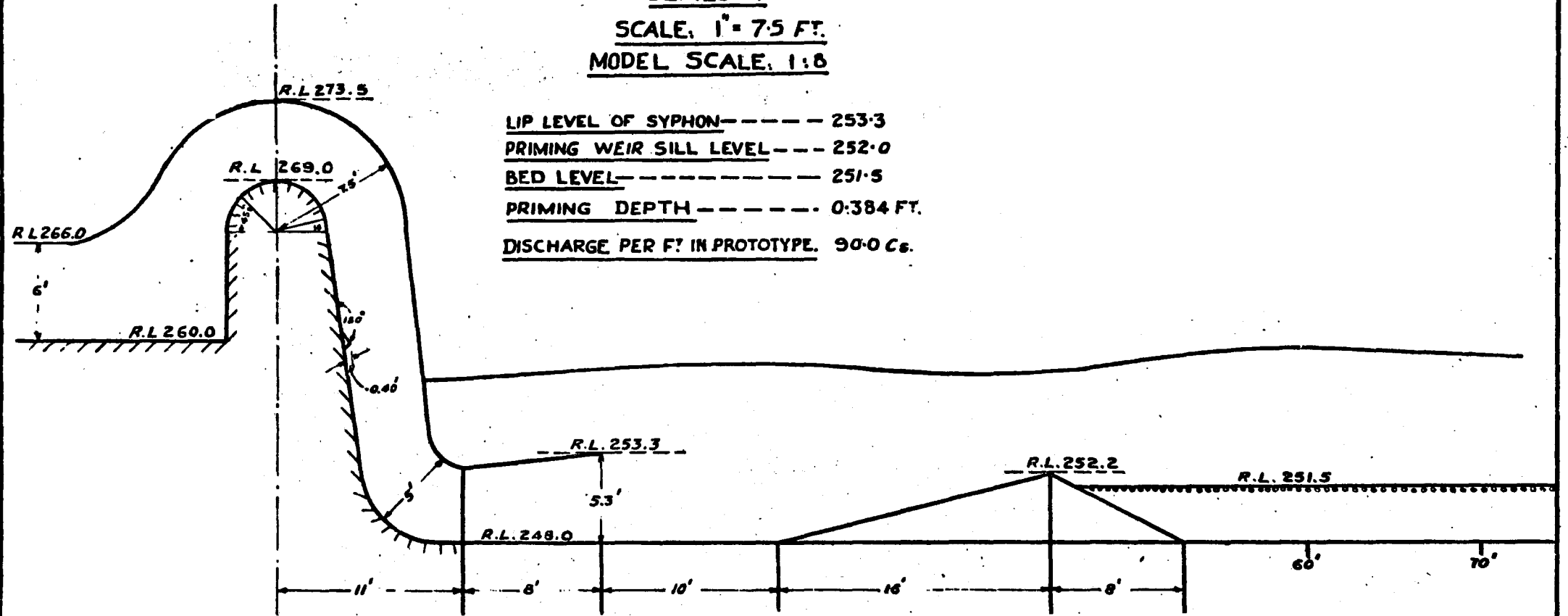
**SERIES 4**

**SCALE, 1" = 7.5 FT.**

**MODEL SCALE, 1:8**

- LIP LEVEL OF SYPHON** ----- 253.3
- PRIMING WEIR SILL LEVEL** ---- 252.0
- BED LEVEL** ----- 251.5
- PRIMING DEPTH** ----- 0.384 FT.
- DISCHARGE PER FT. IN PROTOTYPE.** 90.0 C<sub>s</sub>.

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By S.A. Manchundia. Comp by..

WASTE-WEIR AT BHANDARDARA

TABLE 1 (Series 1)

S.No.	Modifications.	Priming W. L. at 320' u/s.	Priming depth. ft.	W. L. at 320' u/s of syphon.	Discharge per ft. es. Q	D/S W.L. when outlet submerged.	Level of centre of outlet.	Effective head H	Co-efficient of discharge $C = \sqrt{\frac{Q}{2gH \times 4.5}}$	Remarks.
1	Priming weir sill at R. L. 252.00.	270.96	1.96	269.5	91.8	Free flow	250.25	19.25	0.58	
2	Priming weir sill raised to R. L. 252.83.	270.23	1.23	269.5	91.5	"	250.25	19.25	...	
3	Priming weir sill raised to R. L. 253.17.	270.22	1.22	...	...	...	...	...	...	
4	Priming weir sill raised to R. L. 253.86.	270.22	1.22	...	...	Not observed.	...	...	...	

TABLE 2 (Series 2)

S.No.	Modifications.	Priming W. L. at 320' u/s.	Priming depth. ft.	W. L. at 320' u/s of syphon.	Discharge per ft. es. Q	D/S W.L. when outlet submerged.	Level of centre of outlet.	Effective head H	Co-efficient of discharge $C = \sqrt{\frac{Q}{2gH \times 4.5}}$	Remarks.
1	Sill of priming weir 252.17 bed level 252.00.	269.99	0.99	269.5	85.2	...	250.0	19.5	0.535	
2	Sill of priming weir 252.17 bed level 251.00.	269.99	0.99	269.5	86.2	...	250.0	19.5	0.54	
3	Sill of priming weir 251.67 bed level 251.00.	270.01	1.01	269.5	84.2	...	250.0	19.5	0.53	
4	Sill of priming weir 251.67 bed level 251.5.	270.01	1.01	269.5	85.5	...	250.0	19.5	0.535	

TABLE 3 (Series 4)

S.No.	Modifications.	Priming W. L. at 320' u/s.	Priming depth. ft.	W.L. at 320' u/s of syphon.	Discharge per ft. es. Q	D/S W.L. when outlet submerged.	Level of centre of outlet.	Effective head H	Co-efficient of discharge $C = \sqrt{\frac{Q}{2gH \times 4.5}}$	Remarks.
1	Ledge 0.4 ft projection at R. L. 260.00 priming weir sill R. L. 252.0 D/S bed level 251.5.	269.3	0.3	269.5	88.8	...	250.25	19	0.56	*
2	Deflector 0.4 ft. projection R. L. 260.00; Priming weir sill 26 ft. D/S of syphon; Outlet expanded to 5.3 ft., Upper leg given an expansion of 1 : 10.	269.38	0.38	269.5	90.0	257.00 (submerged)	...	12.5	0.707	*

\* Co-efficient not comparable with above co-efficients, as the outlet was submerged.

ANNUAL REPORT (TECHNICAL) 1949

TABLE 4

*Pressure Observations*

Tube No.	Position.	Pressure in ft. of water above or below atmosphere.				Remarks.
		Series 4 3	Expt. 1 4	Series 4 5	Expt. 2 6	
1	2					7
1	15° upstream of crest ...	-17.56	-19.53	-13.49	-15.83	
2	0° on crest ...	-16.06	-19.56	-13.75	-14.55	
3	15° downstream of crest ...	...	-18.36	-12.77	-14.05	
4	30° " " ...	-14.06	-17.26	-11.67	-14.23	
5	45° " " ...	-12.36	-16.84	-9.49	-13.17	
6	60° " " ...	-8.96	-12.00	-14.17	-15.61	} Series 4 Expt. 2 Observation unreliable.
7	75° " " ...	-12.26	-13.46	-15.81	-16.51	
8	90° " " ...	-11.16	-11.96	-10.61	-11.65	
9	Just above ledge ...	-4.36	+2.83	+1.81	+2.91	
10	Upstream sloping side of ledge ...	+0.24	+1.44	...	...	
11	Downstream sloping side of ledge ...	...	-14.72	-10.0	-11.0	

*Note.*—Tubes provided along the centre of syphon weir.

## 12. BRIDGES AND CULVERTS

### 18. WATERWAY THROUGH BRIDGES

An inquiry was made by the Executive Engineer, Survey, B. B. and C. I. Railway Ajmer, regarding suitable size of bridge openings on the Khandwa-Hingoli Section and the information supplied to him being of a general nature regarding which frequent references are made to the station, is reproduced below :—

A narrow river flowing in a deep channel, with defined banks either in rock or in alluvium may be crossed over from high bank to high bank. The narrowing of the river is, however, economically feasible and advisable in the case of an alluvial river having a wide Khadir, only a part of which is occupied by a cold weather live channel but which in monsoon comes into spate filling the full section though fairly shallow in most of the part excepting the main channel.

The limit of the narrowing, on economic considerations is guided by the comparative costs of bridging the whole of the river and the combined cost of a shorter bridge and its protective works.

Spring's method of determining the proper waterway for a bridge is based on arbitrary scour depths advocated by him<sup>(1)</sup> as appropriate for bed sands of different sizes; while Gale's method primarily consists in making use of the river section at a bend, in flood, to assess the extent to which the section of a bridge can advantageously be allowed to develop.<sup>(2)</sup> Natural width of waterway has been however correlated by Mr. Lacey with the discharge and the size of the bed sand and a general equation according to him is  $P=2.667 Q^{1/2}$  where  $P$ =surface width and  $Q$ =discharge. Lacey formula though supported by numerous data, is frequently found to be not in concurrence with the actual flood widths of rivers, the divergence being due to several factors unaccounted for in the  $P:Q$  relation. Nevertheless it is a good guide for determining the optimum waterway and though not directly applicable, is useful in assessing the divergence to be allowed for in fixing the bridge water height. Experience alone will enable us to make the necessary allowance for divergence from Lacey width.

Lacey formula requires reasonably correct estimate of the maximum discharge of the river to be passed through a bridge. The experience of this Station is that the data of actual discharge observations are rarely available and hence maximum discharges have to be roughly estimated by various empirical and theoretical formulae such as Dicken's, Cousin's, Ryve's, Inglis', Mississippi, Station meander formula, Lacey  $V$  and  $R$  formulae and the like.

In the Annual Report (Technical) of this Station for 1943, a review of the limitations of the Dicken's empirical formulae and their general applicability has been made. Appendix 1

The catastrophic or maximum-ever discharge computed and corroborated by judicial application of different formulae does not, however, solely determine the necessary openings for the bridge. Excepting in the case of major, non-submersible, high-level bridges, the extent to which the river should be narrowed down depends on various hydrological as well as extraneous considerations such as constructional difficulties, nature of bed strata, depth of foundations in the case of existing bridges, flashy or non-flashy nature of the river, rising tendency of the river bed, importance of the uninterrupted continuous communication etc.

Thus design of waterway through a bridge cannot, for practical reasons, be generalised and each case has, therefore, to be decided in the light of the conditions applicable.

(1) Page 16, Chapter VI of River Training and Control on the Guide Bank System by Sir F. Spring.

(2) Page 26, The Principles of River Training for Railway Bridges, etc., I. C. E. Paper No. 5167 by Sir Robert Gale.

## APPENDIX 1

Three empirical "maximum run-off" formulae and Whiting's curve (Bombay) have been used to a large extent in India *see* fig. 64 of 1943 Annual Report.

$$\text{Col. Dickens' formula (Bengal) } Q_{\max} = 825 A^{3/4} \quad \dots \quad (1)$$

$$\text{Ryve's formula (Madras) } Q_{\max} = C A^{2/3} \quad \dots \quad (2)$$

$$\text{Inglis' fan-catchment formula } Q_{\max} = \frac{7000A}{\sqrt{A+4}} \quad \dots \quad (3)$$

where  $Q_{\max}$  = maximum flood discharge,

A = area of catchment in square miles,

C = a co-efficient which Ryves put at 450 within 15 miles of the sea; 562.5 from 15 to 100 miles and 675 for limited areas near hills;

*Note.*—Dickens' formula was based on only 4 points. He assumed:

- (1) A maximum rainfall on 8 acres of 4" an hour
- (2) Sir P. Cautley's rule of half inch an hour for small rivers (50 sq. miles)
- (3) Flood discharge equal to 1/8 inch per hour from the 7,000 sq. miles of the Damodar catchment and
- (4) A flood discharge observed on the Son equal to 1/10 inch per hour over the whole area of 27,000 square miles.

Two points in these formulæ are worth noting: (i) the exponent has progressively decreased from 3/4 in the earliest formula to 1/2 in the latest; and (ii) the estimated run-off has progressively increased—except that Ryves' figures were lower than Dickens; but Ryves' figures were meant to apply to designing rather than predicting of maximum possible floods and consequently were lower.

Subsequently, various Engineers have adopted higher co-efficients both for Ryves' and Dickens' formulas up to 1600 being used.

When writing "A critical study of run-offs and floods of catchments of the Bombay Presidency,"<sup>(1)</sup> the Director found that nearly all the data for small catchments in Bombay Presidency were for floods measured at masonry works — canal culverts, super-passages, weirs, etc., — and as a consequence, flood absorption and deflection of flow into adjoining Nalas, greatly reduced the discharges. This explains the marked curvature of Whiting's curve. There is, however, also a natural hump in maximum run-off curves. This is connected with the area that is effectively covered by a maximum storm. Another important factor is 'time of concentration' and hence, the shape of the catchment. Hence, any general formula must be modified where the catchment is not of the standard shape. Such formula should, therefore, be applied with one consideration and caution.

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(1) Technical Paper No. 30, Bombay, P. W. D., 1920 by C. C. Inglis and A. J. Descoteaux.



Photo No. 5666—Showing end of the approach channel with grids for equalisation of velocities and ensuring smooth flow.

Photo No. 5668—Model sluice from upstream.

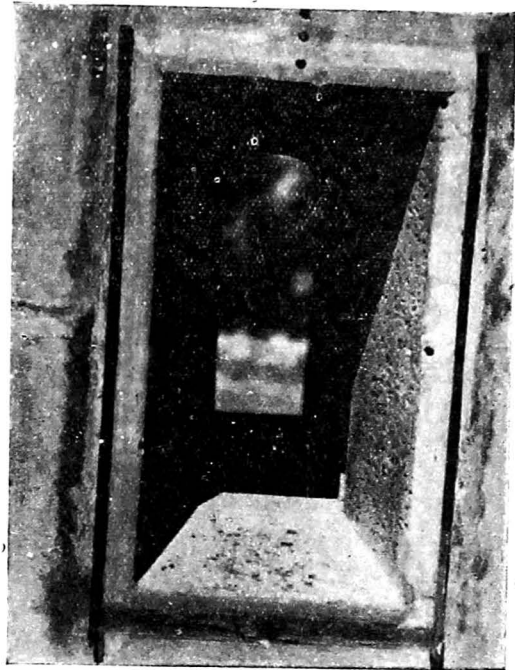


Photo No. 5687—Showing downstream view of the sluice model—Radhanagari Dam Works.

## 14. OUTLETS OR SLUICES

### 19. RADHANAGARI DAM—DETERMINATION OF SLUICE DISCHARGE CAPACITY

#### 1. Introduction

Excess monsoon flow in the river Bhogavati at the Radhanagari Dam is proposed to be escaped mainly through a battery of five sluices shown in Fig. 118. Each of these sluices is 8 feet wide and 18' 9" deep and are provided with a converging entrance. These have to pass an aggregate discharge of 28,000 cusecs at H. F. L. and the balance of 12,000 cusecs of the estimated maximum flood discharge is to be passed over an emergency spillway (fig. 119). Experiments were undertaken to determine the discharging capacity of sluice and the results thereof are given below.

#### 2. Model set up

A geometrically similar, 1/20 scale, sluice model was separately moulded in cement and then fixed in a deep masonry tank. Water was fed to this tank through a flume in which a sharp crested weir was installed for measurement of discharge. Proper approach and complete aeration for the sharp-crested weir, satisfactory stilling in the tank representing the lake, accurate water level recording and other similar details were attended to before starting the experiments.

#### 3. Experiments

Water supply to the tank was controlled to obtain a predetermined water level with the sluice discharging freely without gate obstruction. Model was run for a sufficiently long time to make sure that head over the sluice was constant and then the discharge was measured over the sharp-crested weir. Discharge through the sluices with smooth sides for different lake water levels were determined after averaging the reading of three repetitions. Experiments were also done to find the reduction in discharging capacity owing to side roughness equivalent to that of rough dressed stone masonry.

#### 4. Results of experiments

They are tabulated below and their graphic record is shown in Fig. 120.

TABLE 1

W. L. in the lake.	Single sluice discharge in cusecs with smooth sides of the sluice entrance.	% reduction in discharge with rough sides.
299.00	2880	3.84
303.00	3810	3.82
308.00	4095	3.87
313.00	4754	3.00
318.06	5331	3.50
323.00	5791	2.70
328.00	6368	3.00
333.00	6344	2.90
338.00	7263	2.00
343.00	7656	2.12
348.00	8134	2.34

Fig. 120.

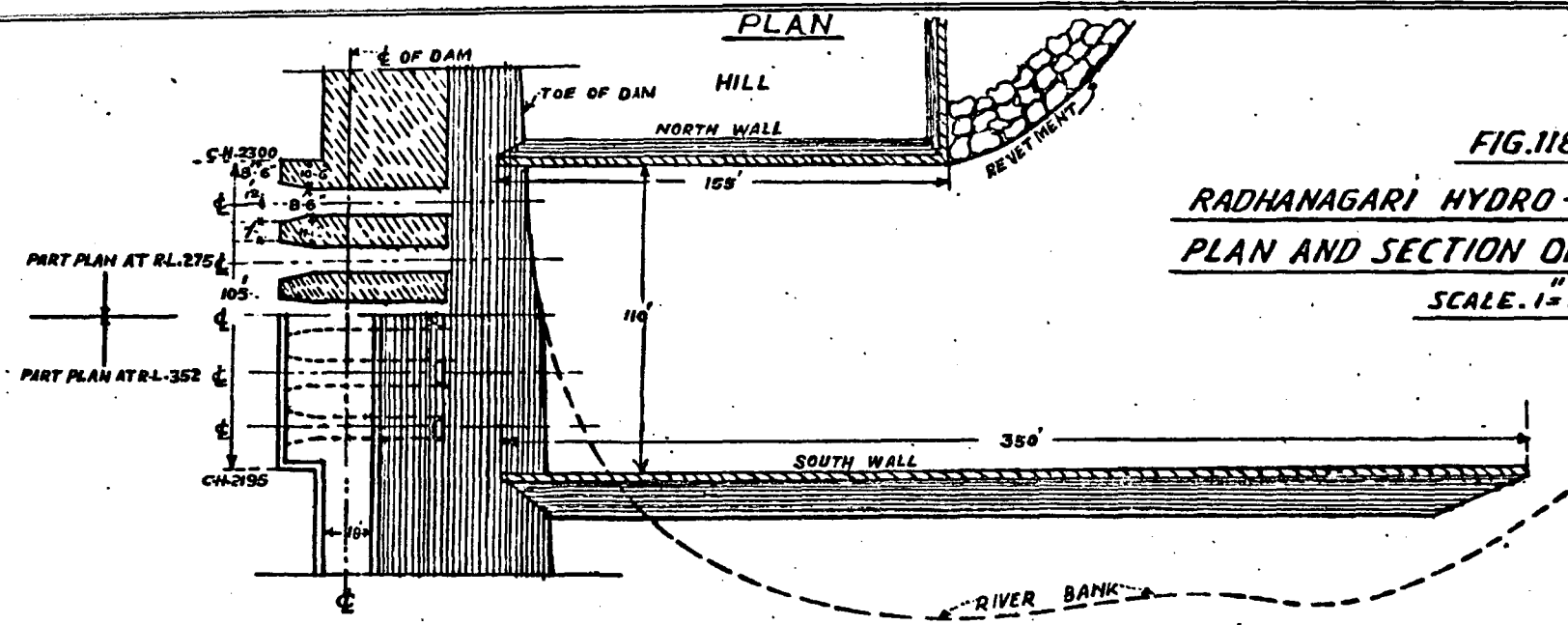
ANNUAL REPORT (TECHNICAL) 1949

5. *Conclusion*

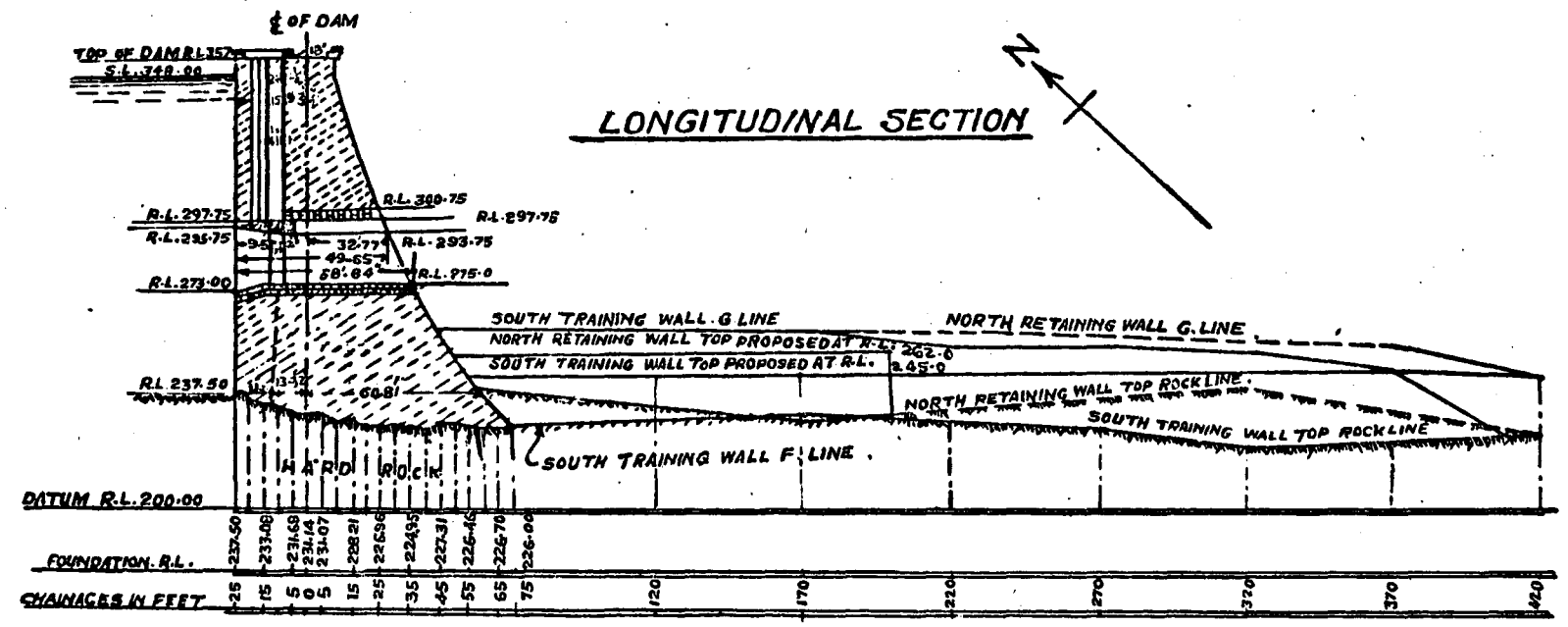
(1) The model results indicate that the sluice capacity at lake water level of 348.00 will be about 7,940 cusecs. Thus the aggregate capacity of all the sluices with the tail race discharge of 600 will not be less than the estimated probable maximum discharge of 40,000 cusecs.

(2) Some reduction in discharge may result due to interference as the sluices are placed side by side. This is not, however, expected to be appreciable as the separating walls are as thick as 11 feet. Experiments will, however, be done to determine the effect of interference when the metal sluices since ordered are received.





**FIG.118**  
**RADHANAGARI HYDRO-ELECTRIC WORKS**  
**PLAN AND SECTION OF UNDERSLICES.**  
**SCALE. 1"=60'**



T. BY. R.K. KAMDEKAR  
C. BY.

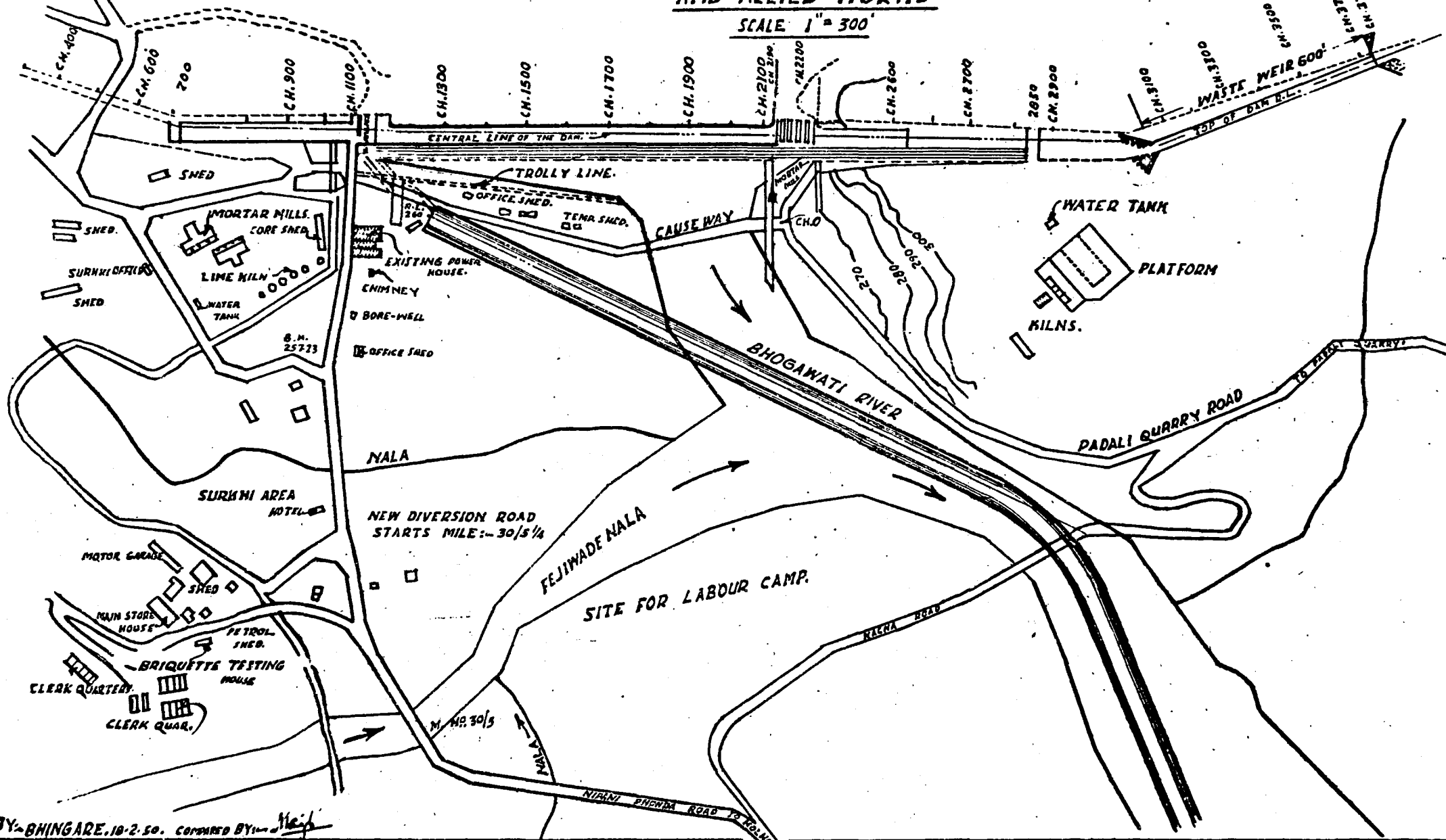
171

S. M. L. TANK.

# SITE PLAN SHOWING RAINWATER DAM AND ALLIED WORKS

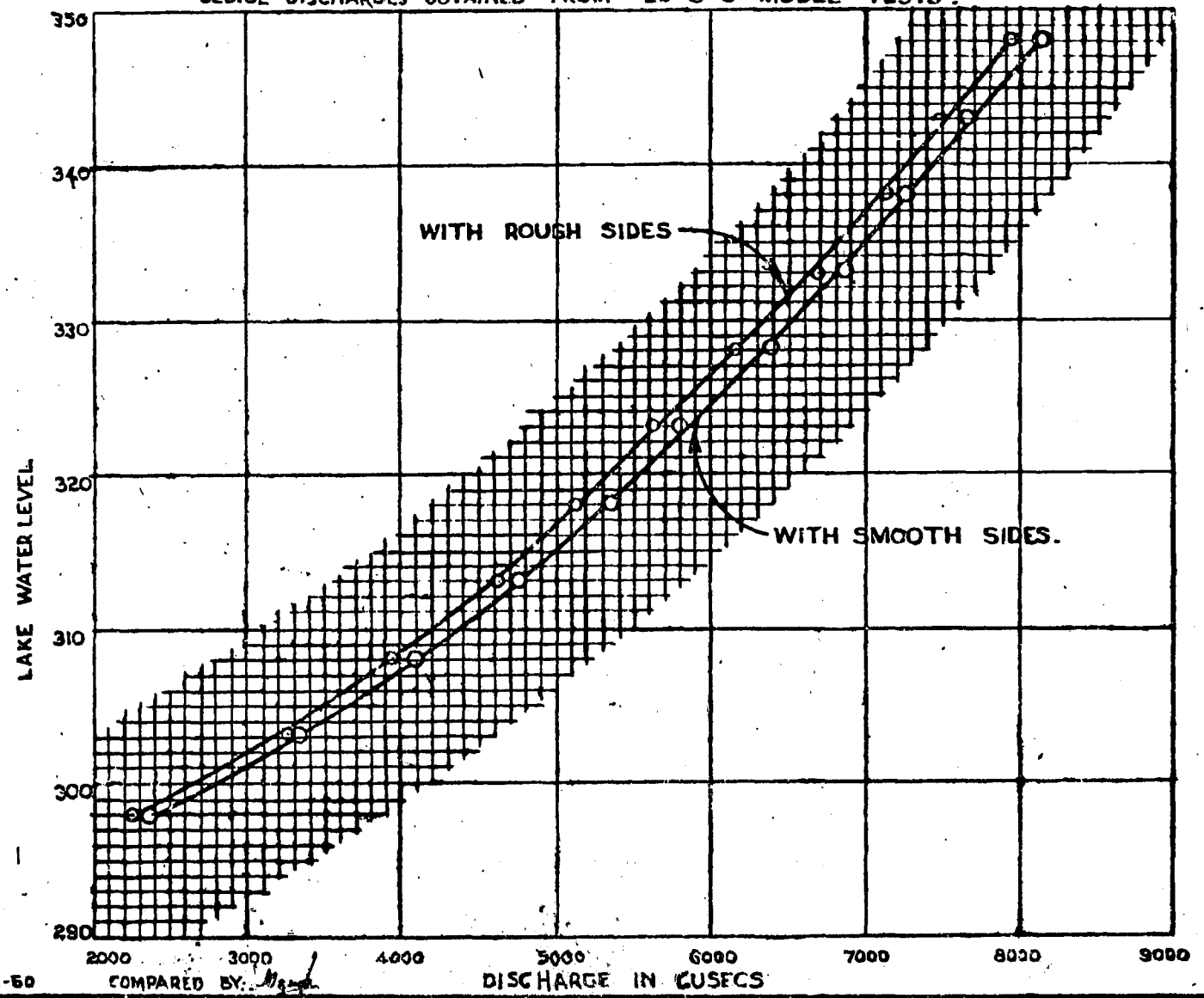
SCALE 1" = 300'

343



P.Y. BHINGARE. 10-2-50. CORRECTED BY: [Signature]

FIG. 120  
RADHANAGARI DAM SLUICE EXPERIMENTS SHOWING EQUIVALENT  
SLUICE DISCHARGES OBTAINED FROM 1/20 G S MODEL TESTS.



NOTE:  
BOTH, THE EMERGENCY AND  
MAIN GATES WERE FULLY  
RAISED.

345

TRACED BY: M. K. SABNIS. 16-2-50

COMPARED BY: *[Signature]*

DISCHARGE IN CUSECS

FIG. J21

MAP

OF

JODHPUR & AJMER MERWARA STATES

SCALE 1" = 16 MILES

OF 5000  
W. P. S. P. 1000

TRACED FROM 1900 SURVEY NO. 40

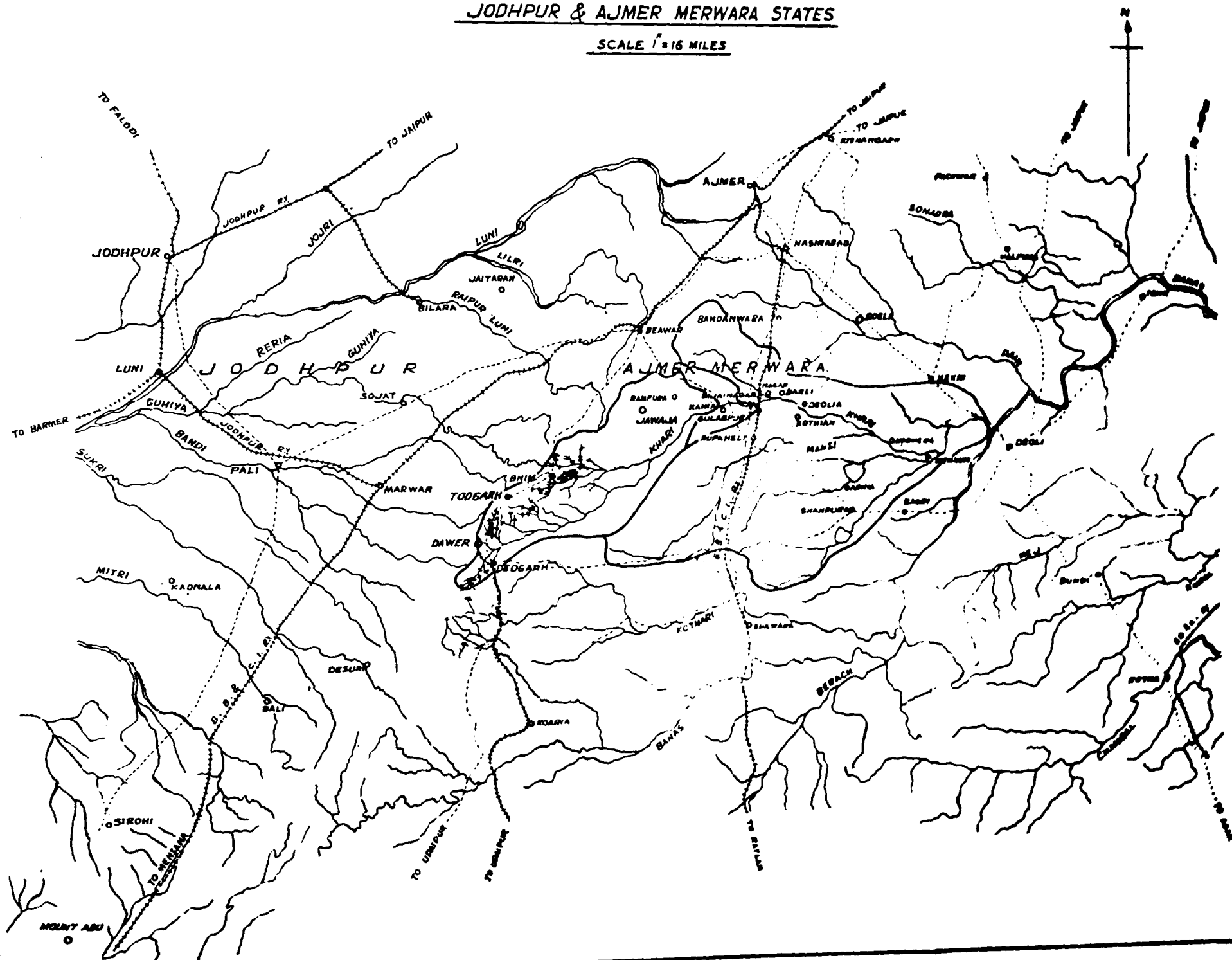


FIG. 122 & 123  
KHARI RIVER CATCHMENT  
DATA OF THE RAINGAUGING STATIONS  
1891 - 1945

FIG. 123

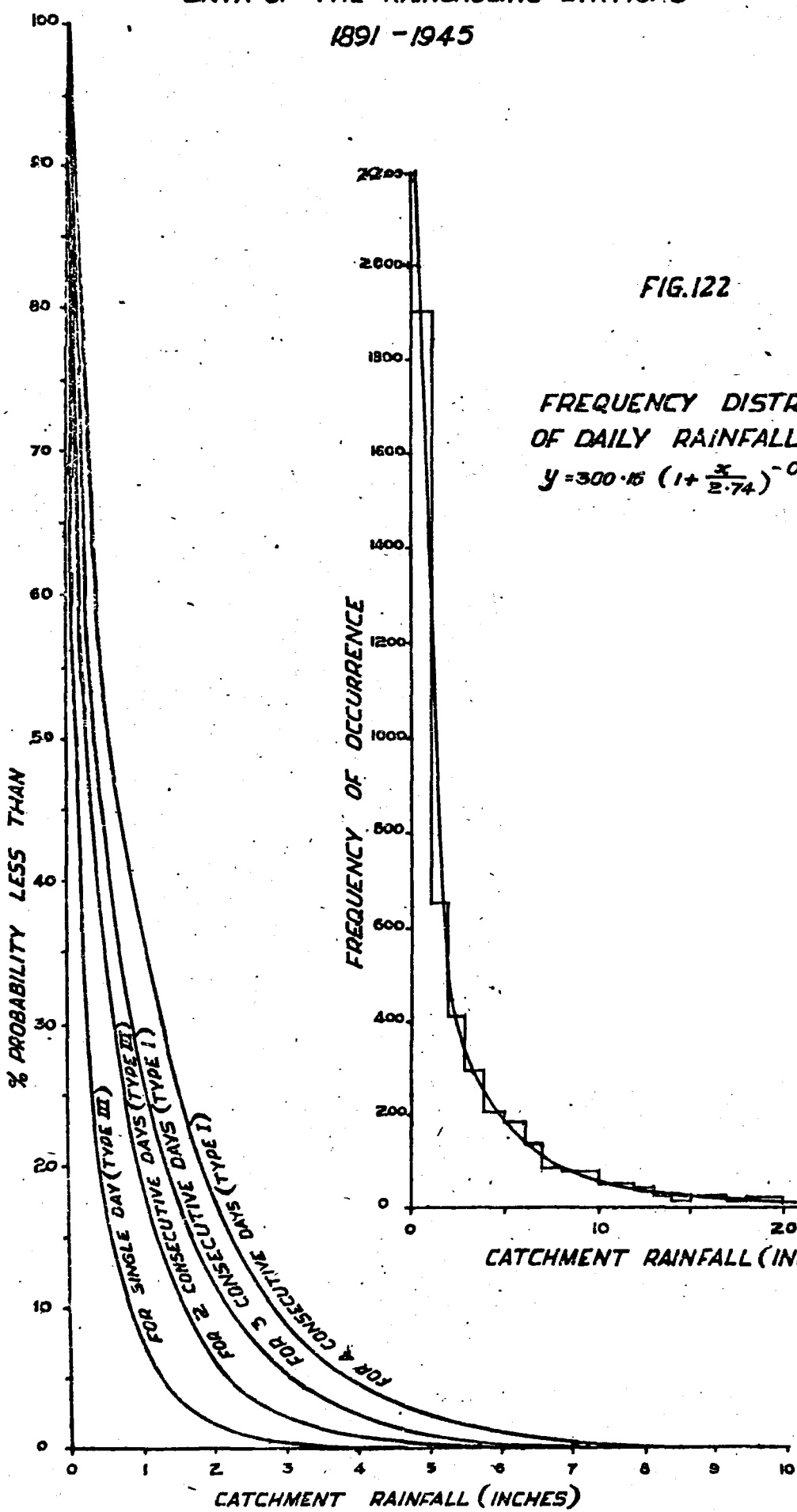
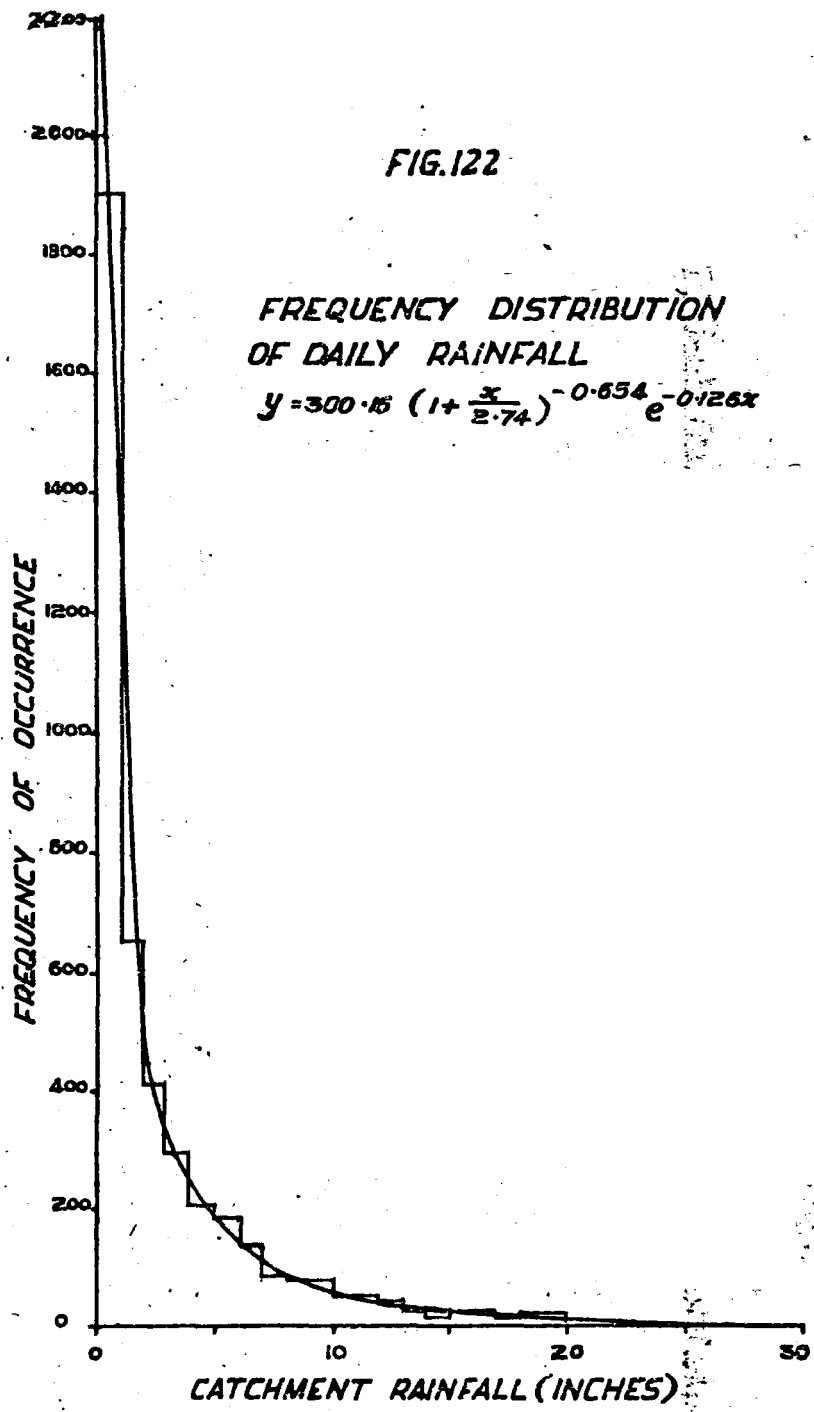


FIG. 122

FREQUENCY DISTRIBUTION  
OF DAILY RAINFALL

$$y = 300 \cdot 15 \left(1 + \frac{x}{2.74}\right)^{-0.654} e^{-0.126x}$$



SECTION D—HYDROLOGY

1. RAINFALL AND RUNOFF

(i) Measurement of Rainfall

20. ESTIMATION OF HIGH RAINFALL INTENSITIES IN KHARI RIVER CATCHMENT: (1891-1945)

An examination of the average intensity of rainfall recorded in the Khari Catchment for 55 years (1891-1945) for which data are available with the India Meteorological Department, was undertaken to investigate if the intensity recorded during 1943 could furnish any clue to the unprecedented flood which caused several breaches in the lakes lying along the course of the Khari river in Ajmer-Merwara.

The ten raingauging stations in the catchment (Fig. 121) include Mt. Abu, Ajmer, Dawar, Deoli, Desuri Geola, Jawala Kakri, Shahpura and Todgarh, the pooled average daily intensity at which has been regarded as the mean intensity of the rainfall over the catchment. The variation of the average intensity during the 4329 wet days during June to September months of the years 1891-1945 covers the range up to 5.71 inches. A frequency table was constructed for the data and the appropriate type of distribution obtained as

$$y = 300.16 \left[ 1 + \frac{x}{2.74} \right]^{-654} e^{-1.126x}$$

from which the auxiliary curve (Fig. 123) has been derived for the percentage probability of different specified intensities being exceeded on any one day. A few percentage probability measures for expected high rainfall intensities read from the curve are listed below to present the extreme low probability measures of the high intensities being attained on any one day. Thus recurrences of high rainfall intensities like 5.71 inches.

Daily rainfall intensity exceeding (inches)	% probability (less than)
4.0	0.08
4.5	0.04
5.0	0.02
5.5	0.01
6.0	0.0044

recorded on the night of 29-30 July 1943, which caused the unprecedented flood, may be expected to be as rare as only 9 in 100,000 or less.

The intensity of the rainfall recorded on the preceeding night i.e. 28-29 July 1943 was 3.27 inches the probability of which occurring on a single day is read from the curve as 0.26% and cannot, therefore, be termed a very rare phenomenon. But considering the aggregate intensity on two consecutive days the record measures 8.98 inches, or an average of 4.49 inches per day. It was, therefore, thought interesting to estimate the probability of the record being exceeded for two consecutive days. The original data for 4329 rainy days was, therefore, recast for the intensities recorded on two consecutive days and a similar frequency study undertaken. The derived percentage probability curve for the distribution is shown in Fig. 123, from which the percentage probability of the observed intensity 8.98" occurring on any two consecutive days is read as 5 in 1,00,000. Thus even this occurrence cannot be regarded to be a common phenomenon.

It is concluded, therefore, that the breaches on the lakes along the course of the Khari River are directly attributable to the abnormal precipitation that fell in the catchment during two successive nights of 28-29 and 29-30 July 1943. As a matter of theoretical interest, and also as an aid to the Irrigation Engineer, similar probability curves for rainfall intensities recorded on 3 and 4 consecutive days respectively are appended (Fig. 123).

## SECTION E—SOIL SCIENCE AND SOIL MECHANICS

### 4. LAND RECLAMATION

#### (1) Saline land

#### 21. RECLAMATION OF RANN OF CUTCH

##### 1. *Introduction*

The Railway Board asked for advice in connection with the proposals for reclaiming the Rann of Catch by way of constructing Rail embankments with one-way sluices and to ascertain whether the construction of such works would, in any way, adversely affect the silting of the deep water approaches to the existing Kandla port which has now been selected as the site of the future major port. If silting was likely to take place the opinion of the Station was required to suggest an alternative form of construction, or alignment to avoid silting.

Fig. 124.

The Railway Board also wanted to know the effect of such construction on the Brine Springs of the Kharaghoda Salt Works and the measures to be adopted to secure continuation of the salt pans. The proposed Railway alignment crosses the Little Rann of Cutch over a strip of about 6½ miles. The possibility of reclaiming the Rann of Cutch for agricultural purposes has been put up by the Indian Agricultural Reserach Institute, New Delhi. One of the proposals for accomplishing this is to provide one-way sluices or a weir-cum-bridge at the crossing of the Railway over the Little Rann. The effect of this will be,

(i) To shut off the entry of the sea water on to the Little Rann situated to the east of the Railway crossing; and

(ii) to permit the flood waters from the various rivulets which first drain into the Little Rann of Cutch and finally into the sea to gradually wash off the salt contents from the soil of the Rann.

These two operations are expected, in due course, to render the Little Rann fit for agriculture.

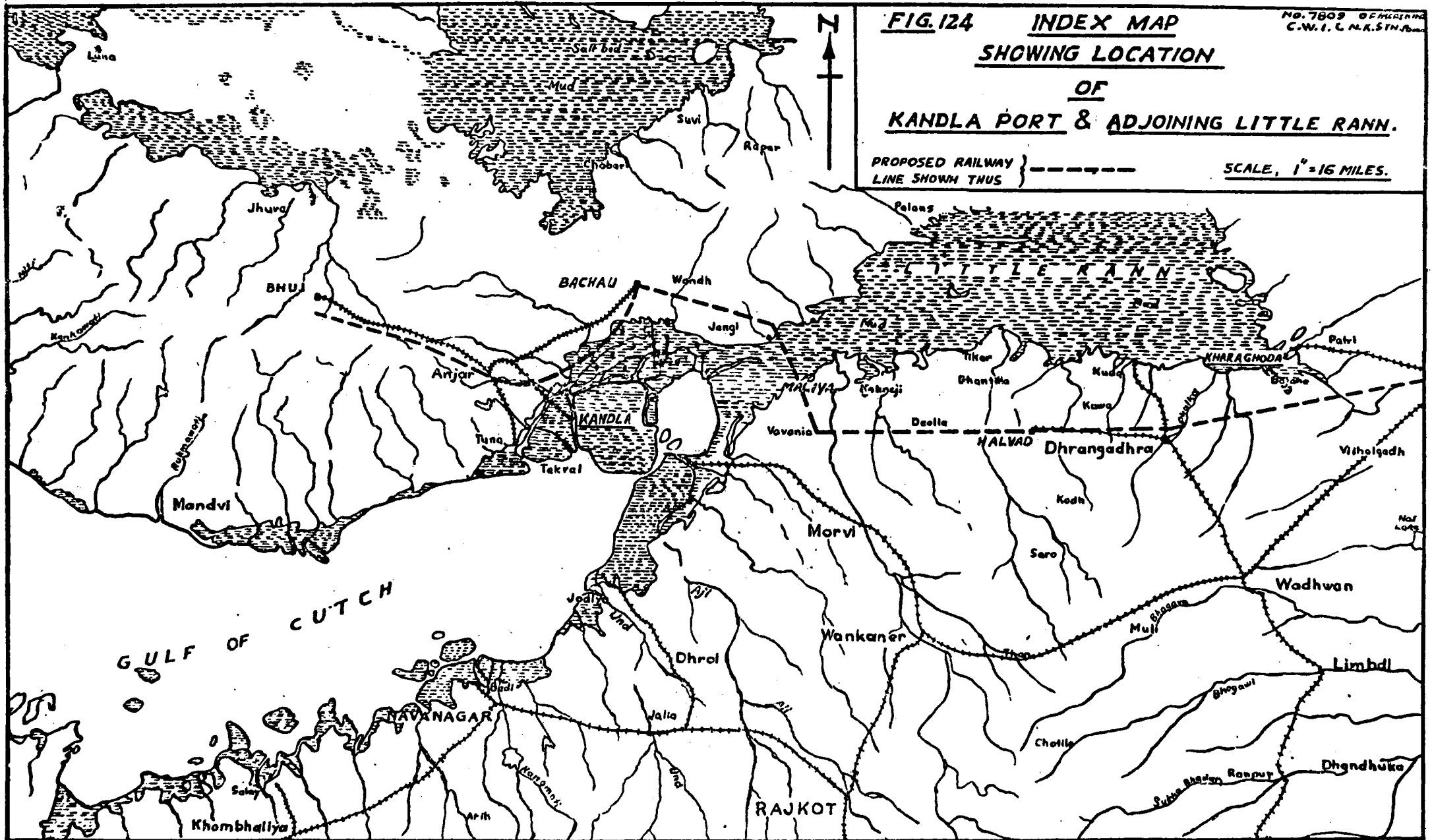
The Government of India has recently decided to develop the Kandla Port as the major deep sea-port and it is therefore desirable that the channel leading to the Kandla Port should remain free from silting. It was, therefore, required to be investigated as to how far the construction of a Railway embankment together with one-way sluices or weir-cum-bridge crossing the Little Rann was likely to cause silting of the deep water approaches to the Kandla Port.

The Salt Works at Kharaghoda are situated at the eastern extremity of the Little Rann. The question of the effect of the proposed railway line on the Brine Wells of Kharaghoda was referred to the Geological Department, Government of India, and in the opinion of Dr. B. C. Roy of that Department, the origin of the Brine in the Brine Wells at Kharaghoda is connected with the sea water and with the constant replenishment during the annual inundation of the Rann by sea water. In Dr. Roy's opinion blocking of the sea water at the Railway crossing would adversely affect the Brine Wells and the salt production.

The following detailed Note was sent to the Deputy Chief Engineer, Survey, B. B. and C. I. Railway, from whom this inquiry was received.

##### 2.0. *Terms of Reference*

As stated in the Deputy Chief Engineer's letter No. E/9/645/19-613 of 6th December 1948 in amplification of No. E-9-645/19-C13 of 20th October 1948 and as a result of subsequent personal discussion with the





## RECLAMATION OF RANN OF CUTCH

Deputy Chief Engineer which helped to visualise the local conditions of the problem more clearly, the following are assumed to be the final terms of reference:—

I (a) What in your opinion should be the proportion of the maximum aggregate discharge for which the Railway bridge across the Little Rann between Malia and Surajbari should be designed?

(b) Verification of the total of the individual maximum discharges from the various rivers, drainage areas etc. which drain into the Little Rann (above the point of the railway crossing) before finally discharging into the sea.

II (a) The Railway Board do not consider a girder bridge necessary and have suggested the flood openings with automatic one-way flap valves at their sea faces so as to shut off the entry of sea water from West to East with a view to reclaim the vast areas of the Rann for agricultural purposes. In view of the above do you consider the proposal of providing 6 ft. diameter Hume Pipes with flap valves at their ends, distributed over the entire length of the Rann crossing, a feasible engineering undertaking? What will be the number of Hume pipes that will be required to be provided to take the maximum discharge and maximum velocity of flow through the openings?

(b) If in your view the proposal of providing Hume pipes is unsound what can be the tentative design of the one-way bridge such that the sea water is shut off?

III. Having regard to paucity of data available to substantiate the likelihood of silting or otherwise of the deep approaches to the Kandla Port as a result of the construction of one-way bridge with a view to reclaim the vast areas of Rann as envisaged by the Director, Indian Agricultural Research Institute, do you justify the construction of one-way bridge at this stage?

IV. If your reply to question III above is that the justification of the construction of one-way bridge is ruled out at this stage, what in your view should be the design of the bridge?

V. Have you any other general criticism or suggestions to offer in respect of the design of the bridge etc?

VI. In view of the construction of one-way-bridge, what measures should be adopted to maintain the salt pans at Kharaghoda?

### Query I

3.1. Before arriving at the maximum discharge for which the bridge should be designed query I (b) should be answered.

Dr. Uppal has worked out (*vide* page 499 of the Railway Report<sup>(1)</sup>) the discharges of the various rivers by applying certain surface flow formulae and other empirical formulae correlating discharge with catchment areas. However, the post-flood hydraulic data cannot be applied with exactitude to the various surface flow formulae owing to the appreciable divergent hydraulic conditions during the floods nor is it possible to estimate the discharge on a rational basis by the catchment formula of the form  $Q = CA_n$  which are limited in their accuracy and application by the variations in the constants assumed, which in turn depend on the nature and quality of, and the fit to, the data from which the formulae are derived. The Deputy Chief Engineer's method of estimating the maximum discharges in this case on the basis of the maximum discharges accepted at the railway crossing in view of the observed discharges at some of the existing crossings of the metre and narrow gauge lines appears sound and rational (*vide* page 229 of the Railway Report). The method, however, needs further elaboration. Since

(1) B. B. and C. I. Railway Engineering Report, Part I-A on "Final Location Survey of Bombay-Sind connection Railway (Cutch Route)".

there is sufficient bulk of data to substantiate that runoff is proportional to  $\sqrt{A}$  and not  $A^{2/3}$ ,  $A^{3/4}$  or  $A$  it is pertinent to point out that the addition to the discharge accepted at the railway crossing (presuming this to be correct for the reasons explained above) should be proportional to the square-root of the respective areas. The discharges worked out accordingly are shown in Appendix I.

3.2 On the same page (229) the discharge from the Little Rann area of 1,414 sq. miles is stated to be 2,29,000 cusecs as against 2,62,000 cusecs worked out according to fan-catchment formula of the station.

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

This formula was used to check up the run-off of various catchment areas and in no case the run-offs exceeded the values worked out from this formula. As the storm core is not expected to cover the whole of the extensive and arid area of 1,414 sq. miles of the Little Rann the allowance of only 1,51,000 cusecs, made according to the normal flood

formula of the Station  $Q = \frac{4,000 A}{\sqrt{A+4}}$  appears reasonable.

3.3. The Station has also found that the shape of the catchment is a very important factor in the correct estimation of discharge. The formula as developed is

$$Q = K^{1/3} 7,000 \frac{A}{\sqrt{A+4}}$$

where  $A$  is the area of catchment in sq. miles.

$$K \text{ is a shape factor } \frac{2}{\sqrt{\pi}} \frac{\sqrt{A}}{L}$$

where  $Q$  = discharge in cusecs, and where  $L$  is the storm length of the catchment in miles.

Though the formula is applicable to Deccan catchment areas, it will serve as a check against the discharges worked out for areas greater than 100 square miles and can be a rough guide to see that no undue excessive allowance is made in the estimation of discharges given in the railway report.

3.4. Regarding query I (a) viz. the percentage of discharge for which the bridge should be designed 33 per cent of the total of the individual maximum discharges given in the Railway Report (Appendix V) appears to be on the safe side for the following reasons:—

(a) Owing to the appreciable percolation losses in the arid tracts through which these rivers pass, the peak discharges as assumed in the design are likely to be lower.

(b) The Sukkur Barrage was originally designed for 15,00,000 cusecs which is only 43 per cent of the maximum total aggregate discharge (34,75,000 cusecs) and not 19,00,000 cusecs as stated by Dr. Uppal. The maximum observed discharge at Sukkur to date has not exceeded 9,00,000 cusecs which is only 26 per cent of the maximum total aggregate. The total of all the catchment areas of the Rann is 8,957 square miles. As the rain storms will rarely cover the whole of this area and that there will be substantial reduction due to flood absorption in catchments allowance more than 26 per cent is not justified; 33 per cent however may be taken as a reasonable figure in view of the paucity of data to substantiate the maximum estimated discharges.

3.5. It is, therefore, recommended that the bridge may be designed for passing the discharge 3,23,000 cusecs.

1. See page 89 of Annual Report (Tech.) for 1942-43.

RECLAMATION OF RANN OF CUTCH

4. *Query II (a)* Regarding the proposal of providing hume pipes in the embankment.

4.1. According to Deputy Chief Engineer's Letter No. E/9/645/19-613 the following assumptions were made:—

(i) The maximum safe height to which the flood water may be taken to head up on the east face upstream of the embankment to be R. L. 17.0.

(ii) The invert level of the hume pipes to be taken as R. L. 7.0.

(iii) The sea water may be taken to head up to R. L. 11.0 against the flap valves on the downstream side.

4.2. The number of openings required in the embankment were first worked on the basis of 6 ft. of head (i.e. difference between upstream water level R.L. 17.0 and downstream water level R. L. 11.0).

There was, however, some incipient misunderstanding about the downstream water level R. L. 11.0 which was later cleared in discussion. It was made clear that as the openings are to be designed for passing the anticipated maximum discharge of 3,79,000 cusecs<sup>(1)</sup> the downstream water level will be the same as the maximum water level observed so far, which is R. L. 17.0. The number of openings are accordingly worked out after assuming 1 ft. of afflux, i. e. water level east of embankment at R. L. 18.0.

The discharge through one 6 ft. diameter hume pipe opening, assuming co-efficient=0.6, will be.

$$Q = CA\sqrt{2gh}$$

$$= 0.6 \times \pi \frac{D^2}{4} \times 8 \times 1 = 0.6 \times 8 \times 9 \times \pi = 136 \text{ cusecs.}$$

The number of openings required to pass the anticipated maximum discharge of 3,79,000 cusecs would be

$$= \frac{3,79,000}{136} = 2,790 \text{ openings or say } 2,800 \text{ openings.}$$

As each opening will require the length of the pipe of not less than 100 ft. across the embankment, the total length of the pipe will be = 2,800 × 100 = 2,80,000 feet.

The cost of laying this length of pipe at Rs. 80 per foot which includes the cost of supplying, laying and other incidental charges will be roughly = 2,80,000 × 80 = Rs. 2.24 crores.

This cost is, however, exclusive of the cost of pitching upstream and downstream faces and flap valves proposed to be fitted at the downstream end of the pipe opening.

On the grounds of cost alone, apart from the odds against the possibility of synchronous breaches at the several openings owing to improper jointing between the hume pipes and the earth above, the suggestion of hume pipes should be rejected and completely ruled out of any serious considerations.

4.3 Even if the water level to the east of the bridge is allowed to rise above R. L. 18.0, which will present further complications such as raising the formation level of embankment etc. the proposal will not outweigh any likely gains by further increase in head and subsequent discharge through pipes.

5. Queries II (b) and III regarding the tentative design of one-way bridge and the justification of the construction of such a bridge at this stage will have to be answered simultaneously.

(1) This figure may be changed to 3,23,000 cusecs, if Para. 3.5 is accepted.

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5-1. With a view to reclaim the vast areas of the waste saline tracts of the Rann the Railway Board intend to construct one-way-bridge by flap valves to shut-off the entry of sea water in the Little Rann. Director, however, feels that before arriving at a conclusion of open bridge (i. e. a girder bridge) or one-way-bridge, it is vital to analyse the possibilities of reclaiming vast areas of 9,000 sq. miles, having regard to the paucity of data to substantiate the likelihood of silting or otherwise of deep approaches to Kandla Port. A continuous leeching of salts year after year may in years to come reclaim the vast areas of the Rann as envisaged by the Director, Indian Agricultural Research Institute. In the opinion of the Station, however, this scheme appears to be too ambitious for reasons furnished below :—

5-2 (a) First of all, it is very difficult to predict how far the leached areas would remain in that condition as it is likely that part of the salt below the unharmed layers of leached soil, may rise by capillary action and impregnate the soil each year and thus this vicious circle once started will never end till inexhaustible quantity of salt is removed and that the non-existence of subsoil connection between the sea and Rann is substantiated by positive proof.

(b) It has been pointed out by Dr. Roy on page 357 of the railway report that "in spite of closing the entrance of the sea, the subsoil water will always remain partly saline, not suitable for irrigation, necessitating canal irrigation over the whole of the area of the Rann". If this is so, the problem of bringing water to the Rann thus reclaimed, by constructing suitable dams at various sites as proposed and contemplated by the Director, Indian Agricultural Research Institute, will be a major problem to the authorities concerned to deal with the subsequent problem of irrigation economically, as worked out below :—

(i) For irrigating 9,000 sq. miles of area i.e.  $9,000 \times 640 = 5.76 \times 10^6$  acres.

Assuming a duty of 100 acres, the approximate quantity of water would be  $\frac{5.76 \times 10^6}{100} = 57,600$  cusecs. Even if the irrigation be extended during either Kharif or Rabi season only the storage required would roughly be  $= \frac{57,600 \times 120 \times 24 \times 3,600}{43,560} = 13.7 \times 10^6$  acre feet.

This figure is exclusive of either the evaporation losses in the lake or the seepage losses through the canals and storage reservoirs.

(ii) The cost of the scheme for providing such a huge storage would roughly be Rs. 137 crores (the rate conservatively assumed on the basis of storage figures of some of the recent major irrigation projects). The above estimate is reasonably optimistic as it neither takes into account the dead storage due to silting which is expected to be very rapid ; as the rivers are flashy in nature and carry a very heavy charge of bed material nor does it include the cost of reclaiming the lands by leeching.

The financial aspects on this basis appear to be very gloomy.

Area to be irrigated in acres	...	$5.76 \times 10^6$ acres
Area that can be irrigated allowing 25% remissions	...	$4.32 \times 10^6$ "
Maximum assessment at Rs. 8 per acre	...	Rs. $33.56 \times 10^6$
Less working expenses at Rs. 2 per acre	...	" $864 \times 10^6$
∴ Maximum annual revenue	...	" $2.492 \times 10^6$
(i.e. Rs. 24.92 crores or say Rs. 2.5 crore)		
Revenue expected at 3% interest charges $\frac{1}{100} \times 3 = 3\%$		= Rs. 4.11 crores.

## RECLAMATION OF RANN OF CUTOCH

Hence, even on full development and on the most favourable hypothesis the scheme will be financially impracticable, the ultimate revenue under the most optimistic forecast would substantially fall short of the interest charges on the capital locked in the scheme. Under any 'Pro forma' or 'ways and means' of system of accounting such a project would be completely barred from the serious consideration.

(c) It is immensely doubtful whether such a huge quantity of storage water required for irrigation can be economically and dependably tapped from the flashy rivers like the Luni, the Banas etc. It is stated in the Railway Report (page 355) that water can also be brought from the Indus. This would have been an impracticable proposition, as already pointed by this Station in connection with another enquiry<sup>(1)</sup> besides no riparian rights can be established on the waters of the river Indus which unhappily now falls in another dominion.

(d) The construction of high banked canals in this marshy area, for commanding all the contemplated area would be difficult and costly as they will have to be lined to save seepage and prevent consequent water-logging of the area recovered by leeching at prohibitive cost.

5-3. All this analysis drives home the point that the reclamation scheme will be prohibitive in cost and very serious consideration should be given before the Project is taken in hand.

5-4. Even though the prohibitive cost involved in such a hazardous scheme is not considered to be the overwhelming objection by Government in view of the possibility of reclaiming areas for irrigation purposes serious and harmful possibility of deterioration of the deep approaches to the Kandla creek as a result of shutting of penetration of sea water by one-way-bridge will have to be seriously considered before embarking on its construction as contemplated by the Railway Board in view of the decision of the Government of India to develop the Port as one of the major western ports of India.

5-5. It may be reiterated and re-emphasised here that the paucity of minimum essential data such as

(i) distribution of sand on a vertical at high and low flood tides in the Hansthal and Kandla Creeks ;

(ii) a few tidal curves at the bridge site ;

(iii) sand charge at the various stages of the flood tide ;

(iv) diameter of bed and suspended sand and silt in Hansthal and Kandla Creeks ;

(v) plans showing courses of the Hansthal Creek and the Kandla Creek for a few years ;

(vi) details regarding any significant rise of bed levels of the Little Rann and Hansthal Creek ; and

(vii) salinity of water in the creeks makes one diffident to definitely say at this stage what exactly the repercussions of shutting of the entry of sea water would be.

The Director, therefore, feels that in view of the possible deterioration of the Creek the decision of one-way-bridge should be postponed till the data asked for is available for maturer examination and scrutiny.

5-6. The tentative design of one-way-bridge is, however, given below for information.

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(1) See 1947 Annual Report (Technical), page 219.

Fig. 125.

Fig. 125 shows section of the weir with automatic shutters on its top. As the existing bed at the proposed bridge site is R. L. 7.0 the sill of the weir may be at the same R. L. The top of the gate may be kept at R. L. 11.25. The gate<sup>(1)</sup> will be designed in such a way that it falls on the downstream glacis of the weir as soon as the water level east of the bridge starts going above R. L. 11.25, during the floods. When the floods recede and the water levels starts going below R. L. 11.25 the gate will automatically come up and thus prevent the entry of sea water to the east of the bridge (the sea water being assumed to head up to a maximum of R. L. 11.0 against the gate). The moment of the water pressure on the shutter about the pivot can be counter-balanced either by over-counterweight or by floats which can be suitably housed in the piers of the bridge.

Similar counter-weight types of gates were only recently installed, after carrying out extensive and intensive model experiments at the Station, on the Tansa Dam (Bombay Province) with a view to increase the present storage capacity. The cost of the gates (28 gates each 50 feet long) was roughly Rs. 11.50 lakhs. On the same basis the cost of the gates would roughly be Rs. 35.4 lakhs as the length of the weir will be 5,900 ft. (vide para 5.8 below).

It may, however, be mentioned here that the gates and their accessories will have to be made in cast iron or other metal immune to the action of brackish water as the span of life of steel in brackish water would be very short. It may be mentioned that the flap gates of steel used by the Bombay Corporation were eaten away by salt water within two to three years.

5.7. The question therefore is of balancing the advantages of open bridge against the additional cost and advantages obtainable with one-way bridge.

If the weir, as shown in Fig. 124, is now constructed, the valuable waterway which is otherwise available in an open bridge by virtue of scour below R. L. 7.0 up to R. L. —8.0 as assumed in the project design, will not be available and substantial increase in the waterway of one-way-bridge will have to be made commensurate with the discharge to be passed through the bridge with the maximum upstream water level at R. L. 17.50 as shown below :—

The waterway proposed for the open bridge by the Deputy Chief Engineer 54 spans of 60' each = 3,240 feet.

The waterway required with the proposed weir will be

$$= \frac{3,79,000}{1.9 \times D^{1.5}} = \frac{3,79,000}{1.9 \times 1.05^{1.5}} = 5,900 \text{ feet}$$

i.e. about twice as much as the waterway required for the open bridge.

[ Note :—The waterway was worked out by the formula

$$Q = C \times B \times D^{1.5}$$

where B is the length of the weir in ft

C is the co-efficient of discharge.

D is the depth of water in ft. over sill of the weir.

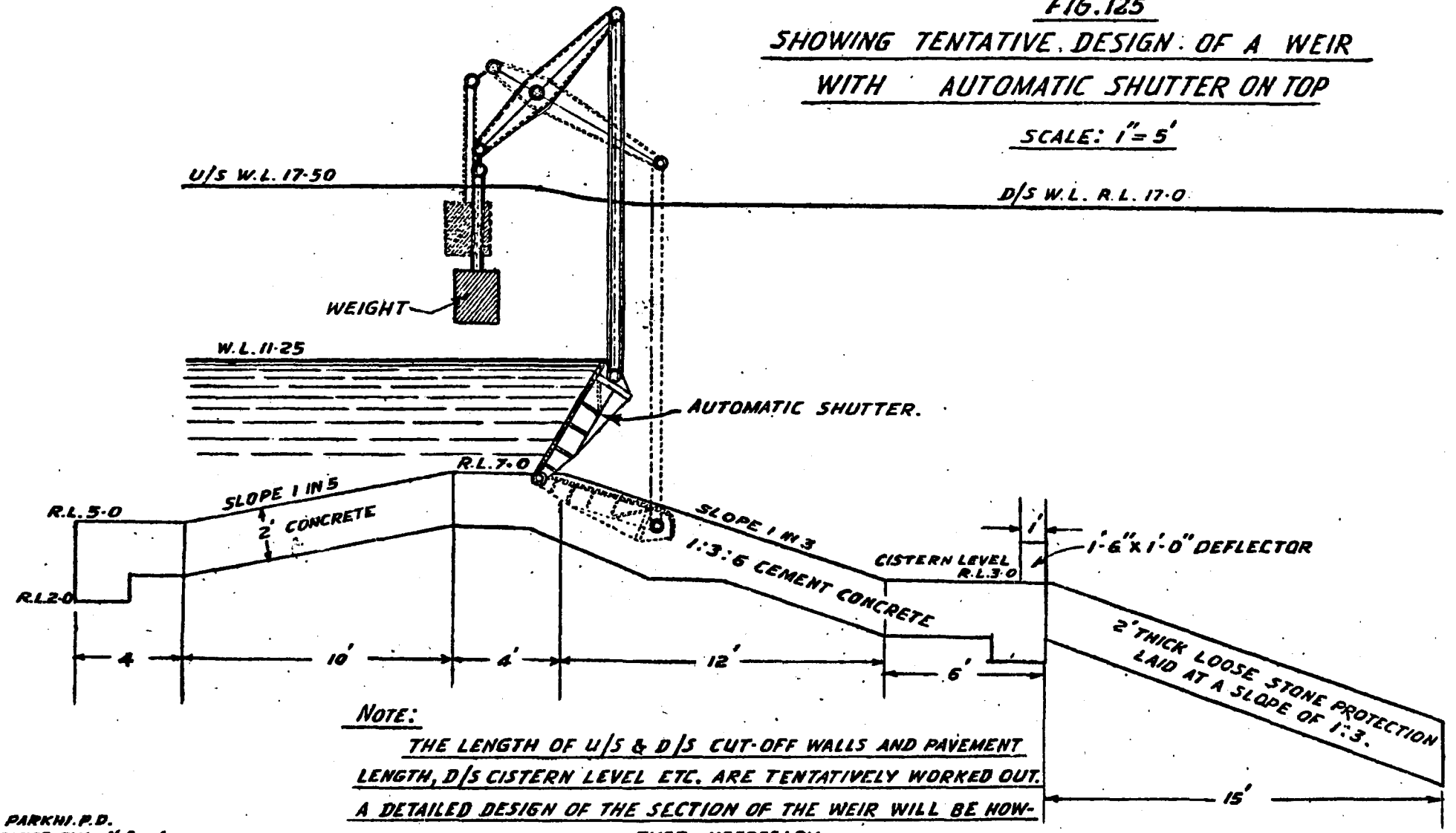
Q is the discharge in cusecs.

The drowning ratio being  $\frac{10}{10.5} = 96\%$ , the co-efficient of 1.90 is only expected on the basis of experiments carried out at the Station and the waterway is estimated accordingly.]

(1) Hydraulic Structures Vol. II. A. Schoklitch, p. 169.

**FIG. 125**  
**SHOWING TENTATIVE DESIGN OF A WEIR**  
**WITH AUTOMATIC SHUTTER ON TOP**

**SCALE: 1" = 5'**



**NOTE:**  
**THE LENGTH OF U/S & D/S CUT-OFF WALLS AND PAVEMENT**  
**LENGTH, D/S CISTERN LEVEL ETC. ARE TENTATIVELY WORKED OUT.**  
**A DETAILED DESIGN OF THE SECTION OF THE WEIR WILL BE NOW-**  
**-EVER NECESSARY.**

TRACED BY: PARKH. P.D.  
COMPARED BY: M. SINGH

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It is, however, possible to increase the velocity through the bridge and waterway reduced by inordinately increasing the water level upstream up to R. L. 20.5 and possibly kept the same as proposed for the open bridge but this would necessitate raising of the formation level and require strong and heavy downstream protection to dissipate excess energy by holding the standing wave on the glacis so as to prevent undermining of the structure:—

The approximate cost of such one-way-bridge would roughly be Rs. 118.6 lakhs as shown below:—

(a) The cost of the bridge over the weir 5,900' long (on the basis of the cost of open bridge of Rs. 36 lakhs for a length of 3,240', verbally stated by Deputy Chief Engineer during discussion at Khadakwasla).	Rs. 65.50 lakhs
(b) Cost of the automatic shutters ...	Rs. 35.40 lakhs
(c) Cost of the weir @ Rs. 300 per foot...	Rs. 17.70 lakhs
	Rs. 118.60 lakhs

It is, therefore, for the Railway Board to consider whether the additional burden of expenditure involved in the construction of one-way bridge is justified at this stage in view of the huge expenditure entailed in reclaiming the saline tracts of the Rann.

5.8. It is suggested that research, in the first instance may be carried out in areas of great Rann on a miniature scale if the area is otherwise suitable to justify the scheme of reclamation in a reasonable period with attractive returns on the capital and thus bring the area under cultivation.

Besides, the area of little Rann forms only 16 per cent of the whole area contemplated to be reclaimed. The research can first be carried out in the areas of great Rann, which are not at present penetrated by the sea water. The scheme can then be expanded, if found to be suitable to embrace the areas of the great Rann which, prior to the construction of railway embankment, were penetrated but would in years to come leech the salts gradually after the construction of railway embankments.

The efficient working of this scheme alone, which forms a substantial percentage of the scheme as a whole as now contemplated, would require sufficiently long period before one may look to the Little Rann for reclamation.

Even in the Little Rann area, which is not now penetrated, it is possible to tap the mineral resources. In the balance, it may be said that the programme of reclamation of the great Rann may precede that of Little Rann.

Another marked advantage of the open bridge at this stage, which should not be lost sight of, would be to dispense with the necessity of shifting Khargoda and Khuda Salt Works which otherwise would seriously deplete according to the report of Dr. Roy or else have to be supplied with a channel taking off from extreme end spans of one way bridge at prohibitive cost as made amply clear in para. 8 below.

### 6.1. *Query IV Regarding the Design of the Open Bridge*

While arriving at a waterway of the girder bridge, the average velocity assumed is 5.5'/sec. *vide* page 222 of Railway Report. With this velocity it is assumed the scour will develop down to R. L.—8.0 i.e. the level at which red murum is likely to be met. It is, however, doubtful whether the scour would reach the expected R. L.—8.0 owing to the presence of sticky blue clay in the first few feet below the existing bed in the Little Rann.



6.2. The limiting transporting force for moving the clayey soil is 0.25 lb./sq. ft. according to the actual experiments carried out by Government engineers in Nurenberg<sup>(1)</sup>.

$$\text{Transporting force} = \rho R. S. = 0.25$$

where  $\rho$  is the density of water

$R$  is the hydraulic mean depth

$S$  is the slope.

$$\text{By substituting for } R. S. \text{ from } V = C\sqrt{RS}$$

assuming  $C=100$

$$\text{Transporting force} = \rho \times \frac{V^3}{C^3}$$

$$V^3 = 100 \times 100 \times \frac{0.25}{62.4}$$

$$\therefore V = 6.5 \text{ ft./sec.},$$

Velocity assumed in the design is 5.5 ft./sec., which in other words means that it is possible to reduce the waterway of the bridge by 14 per cent or the total waterway required would be  $\frac{3,79,000}{6.5} = 58,300$  sq. ft.

6.3. Assuming  $1\frac{1}{2}$  spans are taken by pitched slopes and that maximum scour reaches down to R. L. -3.0 and -8.0 respectively in half the number of spans, 47 spans of 60' will be sufficient.

In fact, the waterway can still be further reduced as the velocity required for transporting the sticky clay will be still higher than that worked out above. It, however, appears unsafe to construct the waterway in absence of data for correctly estimating the maximum discharge for which the bridge is to be designed.

#### 7.1. Query V Regarding any Suggestion

If after scrutiny of data when made available it is revealed that shutting of entry of sea water will not adversely affect the creek it is possible to close this gap altogether by diverting the water of the Little Rann into the great Rann. On page 224 of the railway report it is stated.

“The volume of flood waters of great Rann (from the various catchment areas during the monsoon months) that will be required to be impounded by the bank, will be considerably less than the holding capacity of the great Rann. It can safely be assumed that the highest water level to which this flood water may stand against the railway embankment will be about R. L. 5.0 and will in no case exceed R. L. 6.0”.

If this is so and if the highest level in the Little Rann area is R. L. 17.0 there should not be any insurmountable difficulty in establishing flow from the Little to Great Rann.

#### 8. Query VI in view of Construction of one-way Bridge what Measures should be adopted to maintain the salt Pans at Kharaghoda

8.1. Reply—The problems of reclamations of the Little Rann and at the same time maintaining Kharaghoda pans are diametrically opposite. This has been also substantiated by Dr. B. C. Roy in his note on the possible effects of the proposed bridge on the brine wells at Khuda and Kharagoda, (Appendix XII of the Report) in which he says “It is thus clear that the origin of brine in wells at Khuda and Kharaghoda is linked to the sea water and that the brine is being repeatedly replenished during the annual high sea inundation in the monsoon”.

(1) Hydraulic Structures, Vol. I by A Schoklitch.

## RECLAMATION OF RANN OF CUTCH

Dr. Roy in his report further stated "A large scale underground percolation of the brine in the heart of the Rann up to Kharagoda is not possible."

Under these circumstances the only possible way of maintaining Kharaghoda pans alive is to provide a feeder channel, off-taking through one or two extreme end openings of the bridge depending on the requirements of Kharaghoda pans. This feeder channel will be run up to Kharaghoda works and designed for the discharge which will meet the requirements of brine at Kharaghoda. The feasibility and salient features of such a channel such as its alignment, full supply level,  $\frac{\text{width}}{\text{depth}}$  ratio, etc. will depend on a number of factors, such as natural configuration of the country, sand charge of brine water, nature and range of tidal curve at the proposed bridge site etc.

As the minimum distance of Kharaghoda from the proposed bridge site is 40 miles, the feeder will have necessarily to be 40 miles in length and will have to be isolated from the Little Rann by high banks (top of banks will have to be kept higher than the H. F. L. of R. L. 17.0 east of the bridge) all along its length to prevent the free entry of fresh water in this channel at high flood stages.

Thus such a feeder channel in very heavy banks (in reaches 15' to 20' above ground) would be expensive to construct, liable to breach and thus difficult and costly to maintain. The transit losses will also be heavier than expected. Probably sufficient provision of cross drainage works will also be necessary in the Little Rann area.

This proposal will, however, be prohibitive in cost and it is for the Railway authorities to decide whether the stakes are so important as to maintain the Kharagoda pans at such a prohibitive cost.

8.2. The proposal of feeder channel becomes, however, more feasible on the whole if the Kharaghoda pans are shifted just near about the proposed alignment of the bridge where, now that the new railway line will be run up to Kandla Port, it is possible to provide along the railway line an area equivalent to 50 square miles which is the existing area of Kharaghoda Pans. And now that the brine sources at Kharaghoda are fast depleting and will seriously deplete after the construction of the bridge as explained in Dr. Roy's report makes this proposal all the more worthy of consideration. Such a feeder channel will be shorter in length and thus be economical, cheaper and quicker in construction, will have very few cross-drainage works, and less transit losses. Moreover, such a channel will have potentialities substantially higher than those with brine wells at Kharagoda as stated below:—

(a) Such a gravity channel can preclude the necessity of lifting water otherwise necessary in brine wells at Kharagoda where water has to be lifted from a depth of 20' to 40'.

(b) The factors such as inconsistency of the brine in the wells, dependability of sources on the depth of the brine table and the subsequent quality and quantity are all avoided by providing from such a channel brine water of uniform density and ample quantity.

(c) It will also leave sufficient scope for the future extension and development of the works if and when necessary as it is independent of the underground resources.

According to Dr. Roy's report the maximum output of salt at Kharagoda during the years 1941 to 1946 was 1,39,951 tons. In order to obtain

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this quantity of salt the discharge of such a feeder channel works out to 127 cusecs as shown below:—

Table below gives the annual yield of salt between 1941 to 1946.

Year.	Total yield (tons)
1941-42	... 1,24,871
1942-43	... 1,39,951
1943-44	... 1,20,544
1944-45	... 96,506
1945-46	... 1,17,896

Assuming the salinity of 1 per cent, quantity of brine required per day for the maximum yield of 1,39,951 tons per year

$$= \frac{1,39,951 \times 2,240 \times 100}{63.5 \times 360} \text{ (Sp. gravity of brine assumed = 1.02)}$$

$$= 13,70,000 \text{ c. feet.}$$

As this quantity of brine water is to be obtained from the flood tide, the effective duration of which in one day is assumed to be 6 hrs. then the discharge

$$= \frac{13,70,000}{6 \times 3,600} = 63.5 \text{ cusecs.}$$

Assuming even 100 per cent seepage losses, the maximum discharge of the channel will be 127 cusecs.

### 9. Recommendations

*Regarding Query I.*—The bridge may be designed passing the discharge of 3,23,000 cusecs as shown in Appendix I.

*Regarding Query II and III.*—The open bridge should be constructed now, having regard to the paucity of data to prove the likelihood of silting or otherwise of Kandla Creek and uncertainty about the reclamation of the areas of Rann. If the reclamation of Rann is, however, desired and pressed for, the reclamation of Great Rann which forms about 85 per cent of the area contemplated to be reclaimed, should precede that of Little Rann till sufficient data is available to substantiate that shutting off sea water will not have any adverse effect on the deep approaches to the Kandla Port.

*Regarding Query IV.*—As the velocity to scour the sticky clay met below the existing bed R.L. 7.0 will be higher than 5.5 ft./sec. assumed in the project design the number of spans of open bridge, i.e. Girder bridge can be reduced from 54 to 47 of 60' each.

*Regarding Query V.*—If the data substantiates that the shutting of entry of sea water will not have adverse effect on the creek, the bridge can be dispensed with by diverting the waters of the Little Rann to the Great Rann as there is enough difference between the maximum water levels in the two Ranns with an adequate holding capacity of the Great Rann to absorb the discharges from the Little Rann.

RECLAMATION OF RANN OF CUTCH

APPENDIX I

Item No.	Names of Rivers, Nalas, etc.	Catchment area above the alignment.	Catchment area up to Rann.	Accepted Maximum discharge at the point of crossing with Rly. Alignment.	Total Maximum discharge as worked out by the Station.	Total Maximum discharge, accepted by the Railway.	Remarks.
1	2	3	4	5	6	7	8
		Cusecs.	Sq. Miles.	Cusecs.	Cusecs.	Cusecs.	
1	Godadhroi ...	155	208	25,000	28,000	34,820	
2	Gangasar ...	14	78	6,000	14,150	20,700	
3	Shabhan ...	340	423	1,00,000	1,12,000	1,22,600	
4	Dokamarda ...	10	83	10,000	28,600	30,000	
5	Khari or Kankavati.	80	126	45,000	56,500	58,500	
6	Goadh ...	7	83	7,650	26,400	28,250	
7	Nala between Goadh & Phalku.	...	58	...	20,900	20,900	Not crossed by the alignment.
8	Phalku ...	110	170	34,000	42,000	51,250	
9	Chandrabhaga ...	50	137	12,000	20,000	26,350	
10	Umali ...	70	77	16,000	16,800	18,200	
11	Nalas near Bajana.	...	197	...	20,300	26,300	"
12	Nalas near Khara ghoda.	...	58	...	10,600	10,600	"
13	Rupan ...	1,020	1,114	30,000	31,300	42,100*	*Crossed by the Northern Route alignment.
14	Saraswati ...	880	1,088	40,000	44,500	61,900*	
15	Banas ...	2,620	2,991	2,40,000	2,57,000	2,73,800*	
16	Nalas west of Banas.	...	112	...	13,700	13,700	Not crossed by the alignment.
17	Nalas from Cutch main-land.	...	540	...	67,220	67,220	"
18	Discharge from Rann at rate of run-off of $\frac{1}{2}$ " per hour over the drainage area of the Rann viz, 1,414 sq. miles.	...	1,414	...	1,51,000	2,29,000	The discharge worked out according to normal flood formula of the Station $Q = \frac{4,000A}{\sqrt{A+4}}$
	Total ...	...	8,957	...	9,66,970	11,36,090	

## SECTION H—NAVIGATION PRACTICE

### 3. TRAINING AND CONTROL OF TIDAL RIVERS

#### 22. TIDAL MODEL OF THE HOOGLHY ESTUARY

1. *Resume of Work Done in 1948*:—The nature of the problem, the influence of the tidal behaviour of the Hooghly river on its regime and the terms of reference with regard to the investigation in this Research Station were mentioned in the Annual Report for 1948.

The investigation was proposed to be conducted with the help of three models:—

(1) a 1/2000:1/160 model representing the area from 25 miles south of Saugor Island up to Bansbaria, a reach of about 150 miles which would indicate the nature of changes occurring in the reaches below Diamond Harbour.

(2) a 1/800:1/80 model covering the reach between Balari and Bansbaria for the behaviour of the most important reach in the Hooghly, viz., between Diamond Harbour and Calcutta.

(3) a 1/300:1/45 scale model of the reach between Buj Buj and Konnagar to help study in detail the various problems confronting the Port of Calcutta in its vicinity.

The first model has been laid, the prominent features being (see Fig. 126)

(a) it can be run either by gravity flow or by recirculation according to requirements;

(b) a constant discharge corresponding to a little higher than the highest of springs will be allowed into the model through five-foot flumes;

(c) a pair of sliding weirs on either section of the tide chamber will be operated to reproduce the tide cycles in the model;

(d) the river discharges required for the Hooghly, Rupnarain etc., will be supplied through underground pipes drawing their supply from a supply chamber; and

(e) a surveying girder moving on trolley tracks erected for use in laying the model and making current and tidal observations in the sea portion, the span being 132 feet.

The following analysis of the various factors of the problem will give an insight into the complexities involved.

2. *Bed Materials and Samples*:—Bed samples of various bars in the Hooghly have been analysed. It can be seen from Table 1 that the sand everywhere is very fine, varying from 0.078 mm. at Ninan to 0.151 mm. at Moyapur. For the model under investigation, sand of 0.09 mm. (from Ninan reach) is chosen.

The quantity of silt carried down each year by the Hooghly is enormous<sup>(1)</sup> and even if a small part of it were to deposit regularly each year, it would be aided by the silt brought up by the flood tides, have deteriorated the river long ago. Actually, however, only local bars form and there is evidence to show that there is no significant deterioration of the whole river as such.

The lighter sediment will have no opportunity of settling anywhere in the higher reaches and such deposition is only markedly noticeable in the far south, e. g., in the Eastern and the Western channels, about 25 miles south of Saugor. It can be generally stated that the deposition of river silt does not begin until after the Balari reach towards the sea.

1. See "Hooghly river and its head waters" by Reeks, p. 185.



HOOGHLY RIVER-TIDAL MODEL

TABLE 1

Laboratory Nos.	Hooghly river sample No.	Description.	m in mm.
L-5568	S 2	Pipe Stn. Moyapur, on II track V outer 49° - 15' Ball BN.	... -141
L-5539	S 3	Pipe Stn. Moyapur, on W. F. W. V outer 62° - 30' Ball BN.	... -151
L-5572	S 5	Pipe Stn. Royapur, III track sank 59° - 40' VII Back col.	... -070
L-5573	S 6	Pipe Stn. Royapur, VI track "O" outer 99° - 20' VII Back Col.	... - 12
L-5574	S 7	Pipe Stn. Balari Bagnapara Mark, 39° - 10' Beacord 86° - 00' sample	... -129
L-5582	S11	Pipe Stn. Eastern Gut on dredging transit Lower Nurpur, Mark F. W. outer	... -081
L-5584	S12	Pipe Stn. Ninan IV Anchoring Track ship col. f Cauriapal Mark	... -090
L-5585	S13	Pipe Stn. Fulta Bungalow 60° - 35' Hyma col. 50° - 35' Dhaja tower	... -101
L-5587	S15	Pipe Stn. Ninan IV Anchoring track and No. 1 western gut	... -085
L-5588	S16	Pipe Stn. Ninan IV Anchoring track and III Ninan	... -087
L-5591	S18	Pipe Stn. Eastern Gut III Western Gut track and No. 1 Dredging Transit	... -078
L-5593	S19	Pipe Stn. Eastern Gut II Eastern Gut track and No. 1 Dredging Transit...	... - 09
L-5595	S20	Pipe Stn. Poojali Gogalbai Chy 96° - 40' Orient Chy	... -113
L-5597	S21	Pipe Stn. Poojali on "O" Track Lawrence Chy 59° - 40' Orient Chy	... -110
L-5598	S21	Nozzle Stn. Poojali on "O" Track Lawrence Chy 59° - 40' Orient Chy	... - 08
L-5599	S22	Pipe Stn. Pir Sarang, IV Track, Outer Funnel, 110° - 00' F. W. INNER Col.	... -145
L-5600	S23	Pipe Stn. Pir Sarang, "O" Track, Bata Funnel, 68° - 00' Tide Gauge	... -123
L-5601	S24	Pipe Stn. Sankral, Hangman's point 72° - 35' Painted Tree 38° - 50' Semaph.	... -113
L-5604	S26	Pipe Stn. Panchpara, Union South Funnel, 52° - 35' Badhir BN 62° - 35' IV outer.	... -196
L-5605	S27	Pipe Stn. Panchpara, F. W. Inner 99° - 20', Boundary pillar 41° - 40', Kilm funnel.	... -107

The flood waters of the Damodar river contain red silt and carry a proportion of extremely fine sediment which is difficult to remove even by filtration. It is interesting to note that though the Damodar silt is red, the Ninan bar, which is immediately below the junction of the Damodar with the Hooghly, consists of grey sand. This indicates that Damodar silt has no effect on the Hooghly bars.

It may also be stated that the Hooghly silt hardly offers any impediment to navigation, but only the bed sand forming the various bars.

3. *Dredging in the Hooghly*:—In order to negotiate the various bars in the Hooghly, the Port Commissioners have maintained a system of dredging costing about Rs. 13 lakhs a year. The James and Mary Sands and those at Fulta, Royapur and Moyapur are always kept dredged and occasionally dredging has been done near the Eden channel and the Balari bar also.

Though dredging is not a very successful remedy for keeping the river navigable, it has, it appears, helped a good deal in a temporary way.

Though dredging is done to the extent of 60 to 70 million cubic yards of sand per year from the various bars, it is clear that an insignificant portion of it can reach the sea. It is redeposited in other parts and probably a fraction finds its way back to the same place owing to tidal action.

*Tidal Data*.—The tidal period in the Hooghly varies from about 12 hours 12 minutes to 13 hours 10 minutes, and the average period may

Fig. 127

be taken as 12 hours 20 minutes. The ranges vary from place to place (see Fig. 127) as also the periods of the flood and the ebb tides as indicated in table 2.

TABLE 2

*Tide ranges and periods of the flood and the ebb*

Place.	Maximum range.	Minimum range.	Flood period average.		Ebb period average.	
	ft.	ft.	hrs.	mins.	hrs.	mins.
Southern limit	12	2½	6	10	6	10
Saugor	17	8½	5	40	6	40
Hooghly point	18	6	4	50	7	30
Moyapur	17	6	4	15	8	5
Garden reach	15½	4½	3	20	9	20

It is evident that the variation of the range from the neaps to the spring is considerable and the simulation of such variation in a model is very important, especially so during the freshets, because the changes in the bed are a function of the relative strengths of the tides and the freshet discharge. This fact necessitates the choice of representative semilunation instead of single averaged tides. In consultation with the Liaison Officer of the Post Commissioners, the following deductions were made with regard to the choice of representative semilunations.

All the bars and crossings are at their best at the tail end of the freshets, i. e. in October and November. Thereafter, with the cessation of head water supply and gradual increase in the strength of the flood current, a general deterioration takes place in all the bars from November-December to the end of January. (The eastern Gut bar at Hooghly point which deepens out to 24 ft. in September, reappears with a bar of less than 18 ft. by the middle of December. This deteriorates further in January springs.)

With the increase in the strength of the flood currents, all bars deteriorate considerably during the period February-May the extent of shoaling depending on Springs or neaps.

The bars all appear to be steady after this period till the advent of the freshets towards mid July after which there is general improvement and in October the river is generally at its best.

Thus the year can be divided into four seasons and in accordance with the factors involved, the representative semilunations chosen in each season.

(i) *The flood tide season: from February to middle of May:—*As the greatest bed movement is during the equinoxial perigee spring tides, the semilunation with the greater neap range should be chosen as representative of the period.

(ii) *The prefreshet period: middle of May to middle of July:—*Bars are generally steady after the deterioration during season (i). A few cycles of lower springs need be chosen.

(iii) *The freshet period: middle of July to October end:—*A semilunation of the minimum neap range should be selected, so as to fully show the holding back the flood current by the freshet discharge.

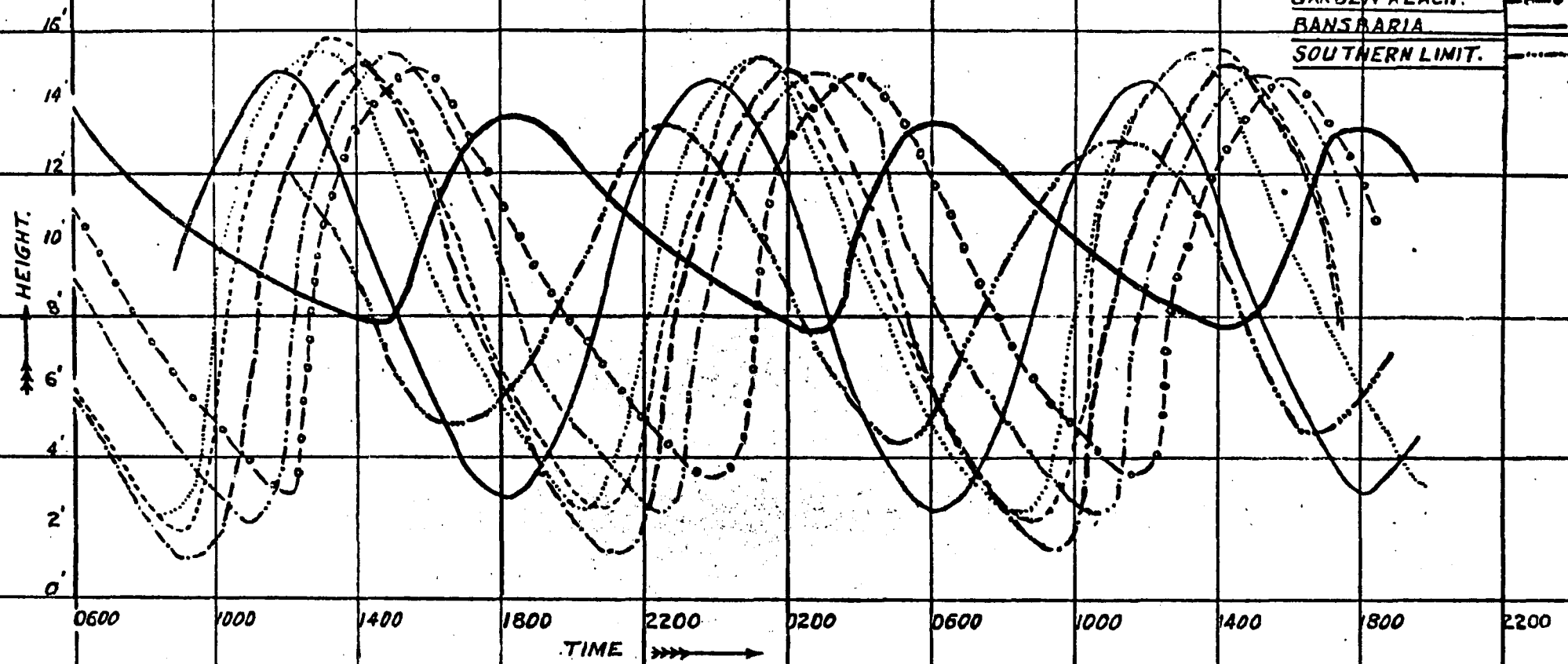


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FIG. 127  
HOOGHLY ESTUARY  
COMPARATIVE TIDAL CURVES FOR 3rd MARCH  
1949.

REFERENCES:-

SAUGOR.	————
BALARI	.....
DIAMOND HARBOUR.	-----
HOOGHLY POINT.	-----
MOYAPUR.	.....
GARDEN REACH.	—●—
BANSBARIA	————
SOUTHERN LIMIT.	.....



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Traced By: S.A. Manchundia.

Comp. By: [Signature]  
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HOOGHLY RIVER—TIDAL MODEL

(iv) *Post-freshet period: November to January*:—Here again a slight deterioration of bars sets in. A few springs representing higher ranges should be run.

In accordance with this analysis, the following representative semilunations of the Saugor Tides were chosen for the year 1939 for the flood and freshet seasons; the other two being not very important in creating changes were not considered to start with. [See table 3, figs. 128 (a) and 128 (b)].

TABLE 3.

Season.	Representative period.	Maximum range.		Minimum range.	
		Feet	inches	Feet	inches
Flood	5th February to 20th February 1939	15	10	4	8
Freshet	13th September to 27th September 1939	17	3	4	2

4. *Variation of Mean tide level*:—The Hooghly river is subjected, as many others are, to the variation of the mean tide level from season to season. The rise of mean tide level results in an increase in velocity for the same range of tide. It may be observed from fig. 129 that in August and September, the mean water level will be at its maximum, the increase in level being less and less as one moves down the river.

5. *Influx discharge of the Hooghly at the southern limit of the model reach*:—One of the essential requirements of designing a river model is to know the maximum discharge. In the present case of the Hooghly, with the help of the simultaneous half hour gauge readings at various points within the reach, Mr. Kilford, (the then River Surveyor, Port of Calcutta, worked out the maximum influx discharge of the Hooghly at the southern limit to be 50 million cusecs for the spring tide of March 1940. The method of his calculations, in brief, was as follows:—

(1) From the simultaneous tidal curves—fig. 131—it was noted that when it was high water at Saugor, *vide* index plan (fig. 130), it was low water at Akra. Therefore, this section was used to calculate water passing Saugor. Similar other reaches are enumerated below:—

High water at Lat. 27-15N (Southern limit) corresponded with low water at Moyapur.

High water at Saugor corresponded with low water at Akra.

High water at Gangra corresponded with low water at K. G. D.

High water at Balai corresponded with low water at Konna-gar.

High water at Hooghly Point corresponded with low water at Chinsurah.

High water at Garden Reach corresponded with low water at entrance to Churni.

(2) From the data recorded at various points, half hourly rise in the river level was noted.

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(3) Width of the river at high and low waters at various stations was noted from the charts and with the rise of tide and the width of the river the sectional area was worked at each station for every half hour after the low water upto high water.

From the sectional areas thus calculated and with the distance between one section and the next, the quantity of tidal waters in that portion of the river was obtained at half hourly intervals, the increase or decrease in successive values giving the influx.

(5) The influx into Rasulpur, Haldia and Damodar river was not worked out for want of data; but, their tidal capacities being very small it could not affect the calculations seriously.

(6) The tidal capacity of the Rupnarain for a tide similar to that of 25th March 1940 was approximately 11,420,000,000 cft. This would increase the inflow at Saugor by 2 or 3 per cent. At Gangra and Balari both the mean and maximum inflow may be increased but only slightly.

In the third step of the calculations mentioned above, if more accuracy is desired, one may compute the area in plan of the reach between the stations for various half hourly levels which, with the corresponding half hourly rise will give the quantity of water that passed the lower station during the interval.

This is what has been done in the method explained by Pillsbury.<sup>(1)</sup>

The inflow discharge figure for Lat. 21-15 N. as worked out according to this method worked out to 49,772,558 cusecs as against 49,189,689 cusecs by the former. Maximum influx at some other stations are given in table 4.

Figs. 132 and 133 show the  $\frac{1}{2}$  hourly influx discharge worked out by both methods for the southern limit and the Garden Reach from which it will be noted that the difference in the computations is inappreciable in the case where the section is large but is considerable in the reach where the section is small.

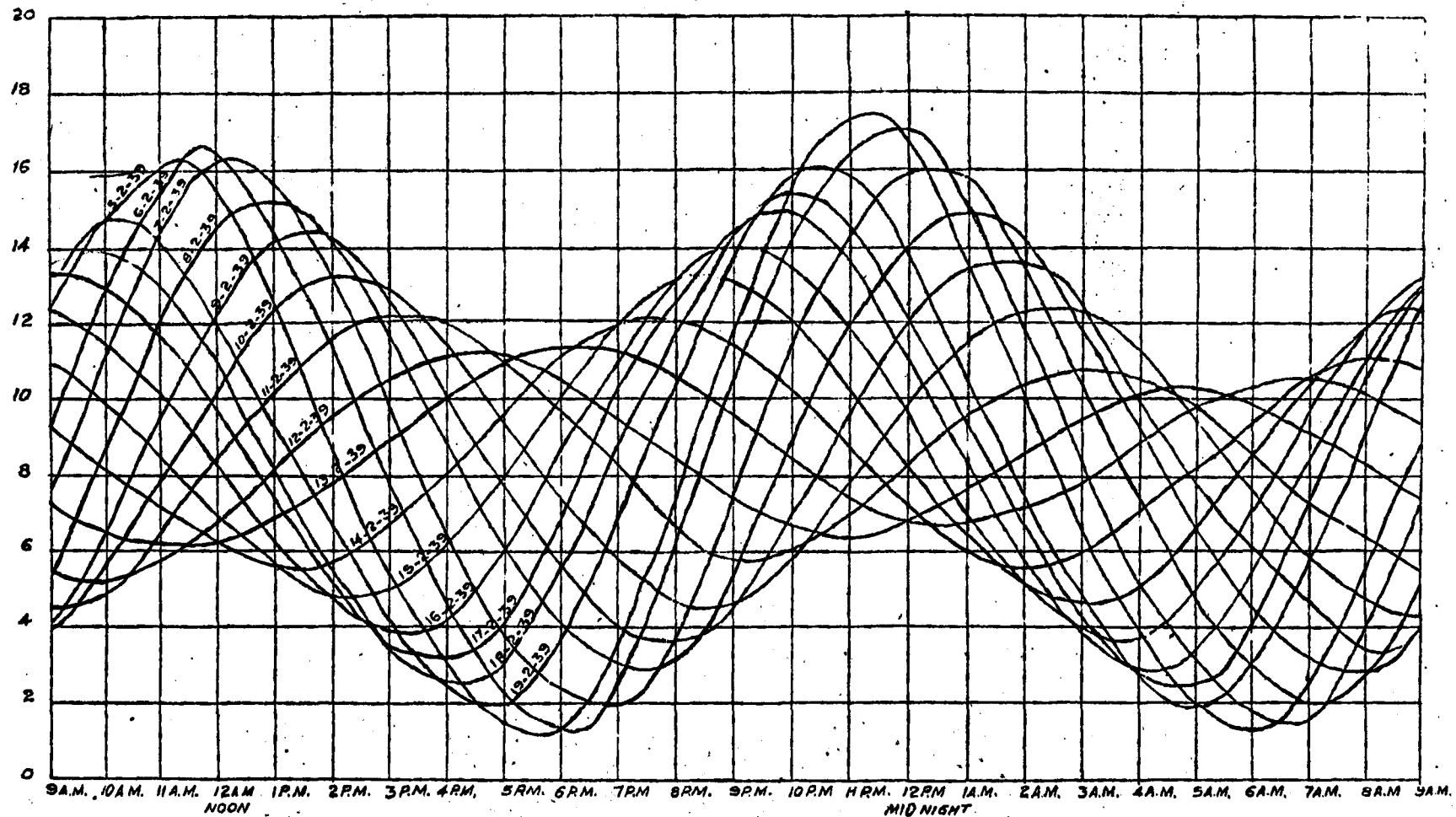
TABLE 4.

S. No.	Place.	Maximum discharge in mill. cs.	Maximum velocity ft./sec.
1	Southern limit	50	...
2	Saugor	10	5.7
3	Balari	6	5.7
4	Hooghly Point	1.5	5.8
5	Calcutta	0.7	6.4

6. *Currents and their directions*:—One peculiar behaviour of the Hooghly estuary is the rotary currents. One would normally expect that the flood and the ebb south of Saugor Island would be northward and southward respectively, or at any rate the ebb would be in the opposite direction to that of the flood.

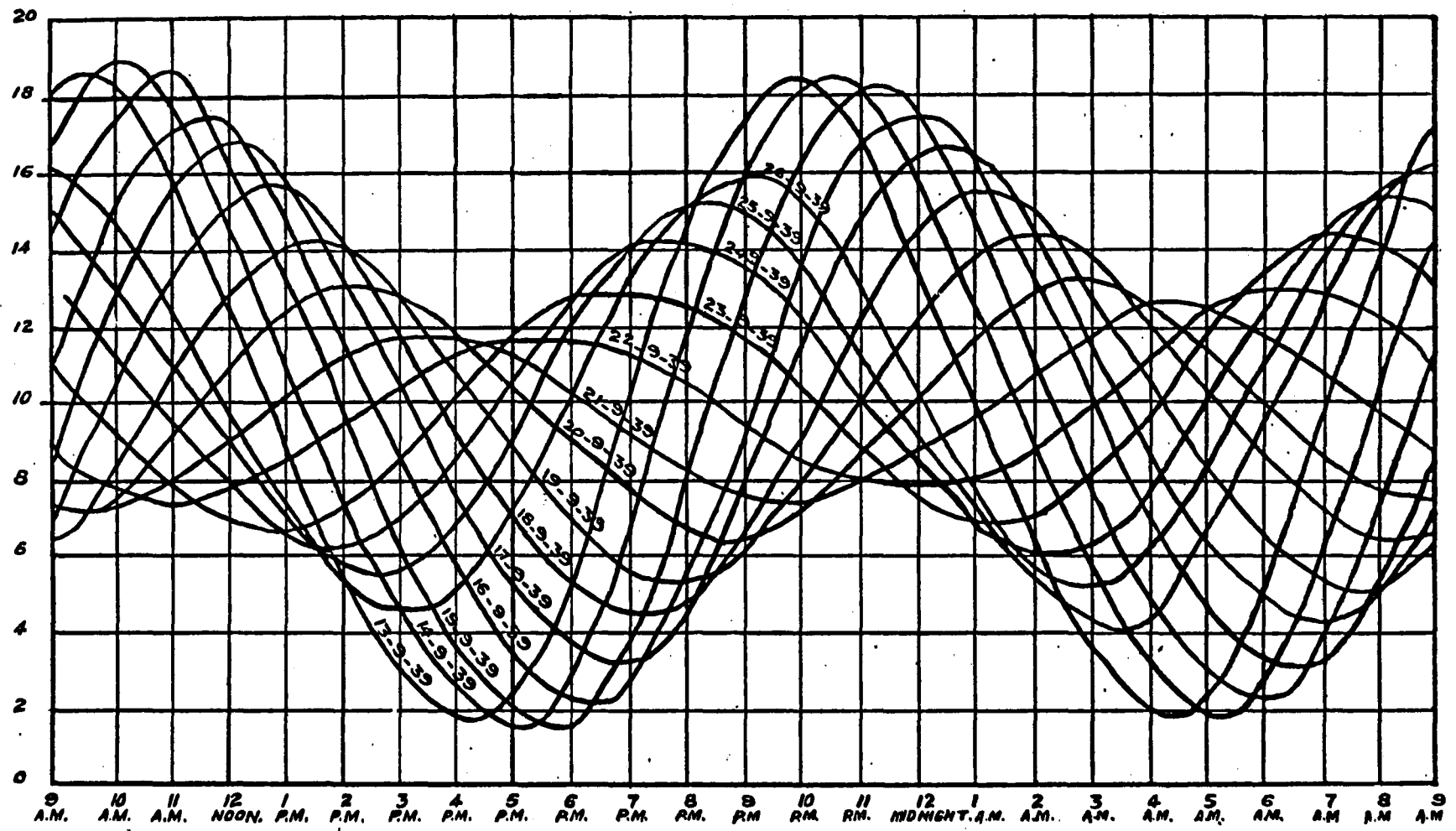
(1) 'Tidal Hydraulics' by Brig. Pillsbury, p. 153.

FIG. 128 (C)  
CALCUTTA PORT COMMISSIONERS  
SAUGOR SEMAPHORE  
DIAGRAM SHOWING LEVEL OF WATER IN FEET  
5<sup>TH</sup> FEBRUARY TO 19<sup>TH</sup> FEB. 39



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FIG. 128(b)  
CALCUTTA PORT COMMISSIONERS  
SAUGOR SEMAPHORE  
DIAGRAM SHOWING LEVEL OF WATER IN FEET  
13<sup>TH</sup> SEPT. TO 26<sup>TH</sup> SEPT. 1939

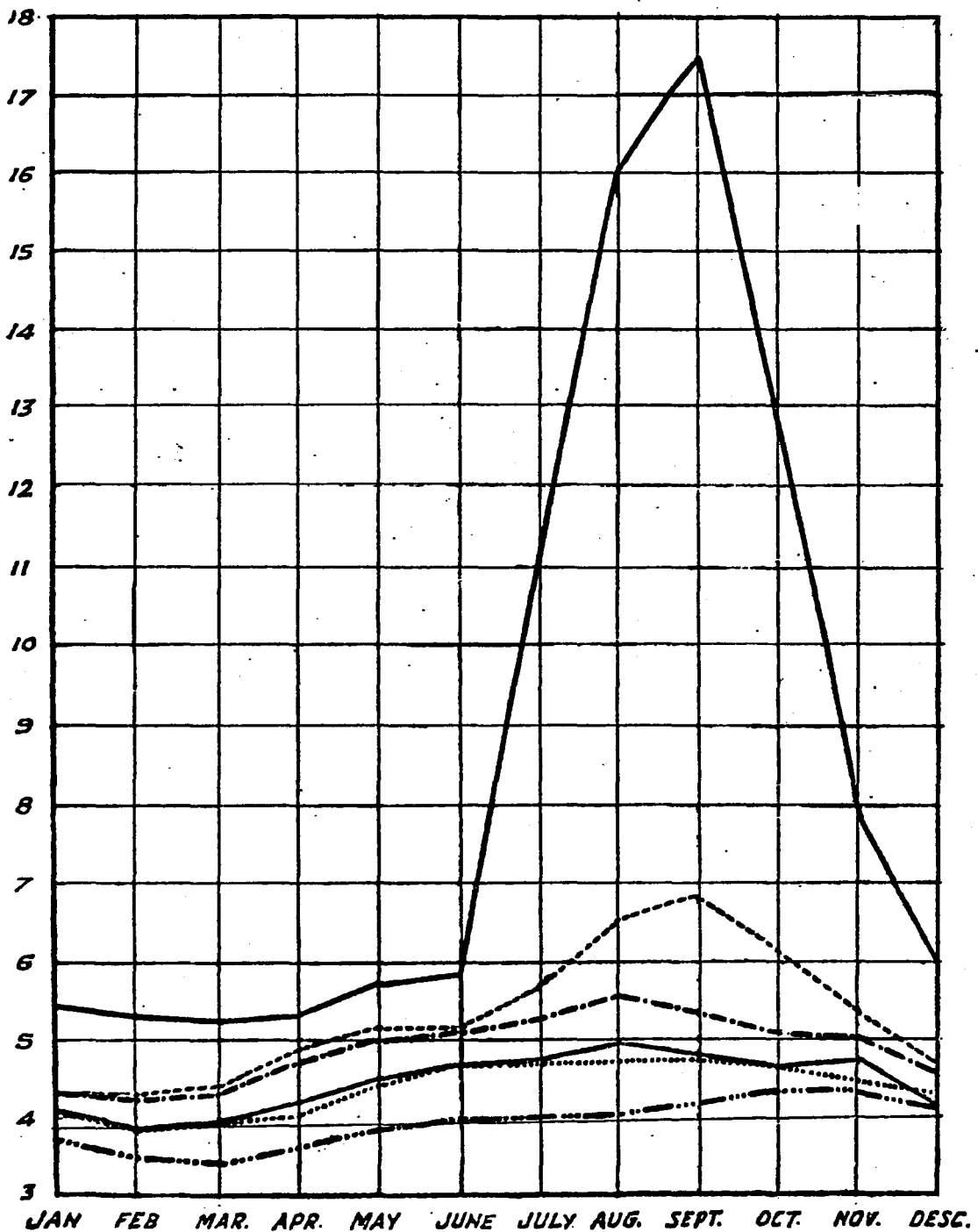


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FIG. 129  
MEAN TIDE LEVEL ABOVE K.O.D.S.  
(KIDDERPORE OLD DOCK SILL)  
1948.

REFERENCES.

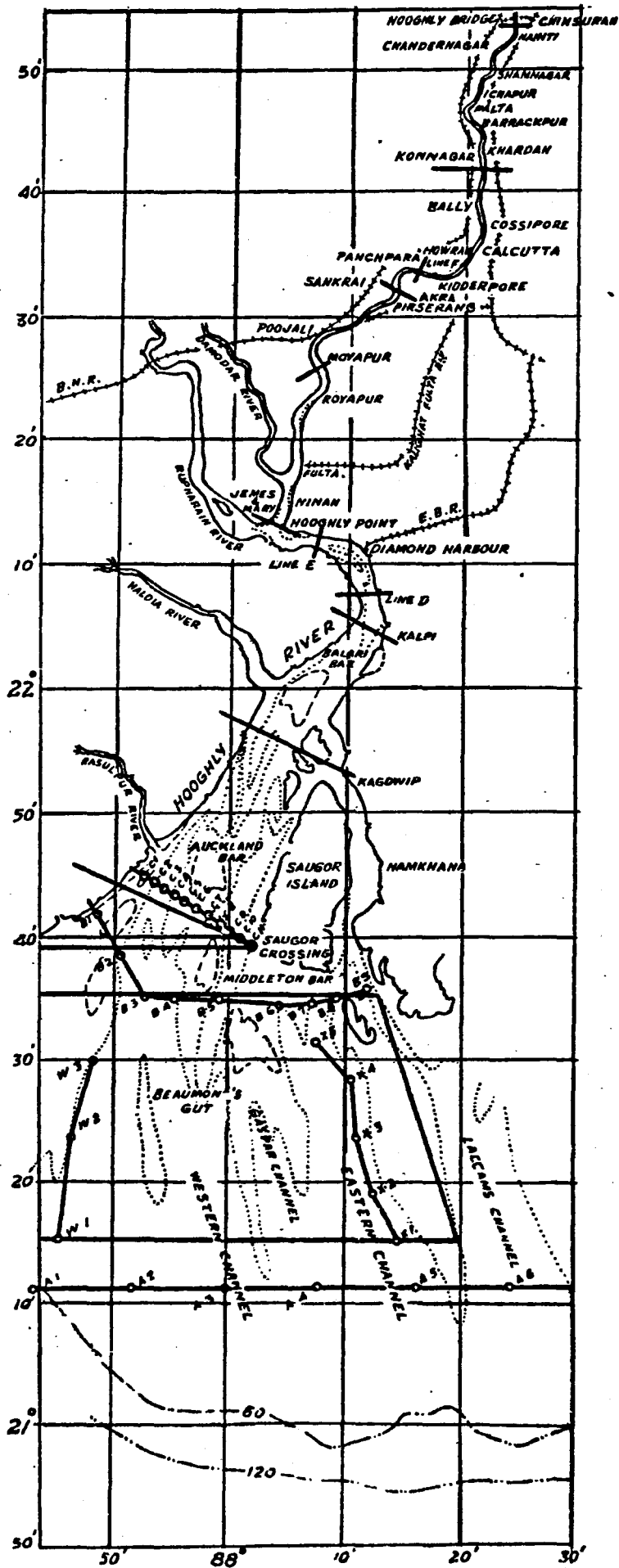
- MEAN WATER LEVEL SWARUPGANJ. —————
- GARDEN REACH... - - - - -
- DIAMOND HARBOUR. ....
- SAUGOR. ....
- MONTHLY AVERAGE FOR SAUGOR. ....
- 1937 TO 1943 (FROM SURVEY OF INDIA.)
- VIZAGAPATAM. ....



1948.

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FIG. 130  
SKELETON MAP OF RIVER HOOGHLY  
SCALE. 1" = 12 N. MILES.



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B. C. DESHPANDE  
CHECKED BY,





FIG:132

NO: 7784 OF FEB: 1950.  
C.W. I & M. R. STN. POONA.

INFLUX AND DISCHARGE AT LAT. 21-15N.

25TH MARCH 1940.

—— METHOD OF CONTOURS.(PILLSBURY)  
- - - METHOD OF CROSS SECTION.(KILFORD)

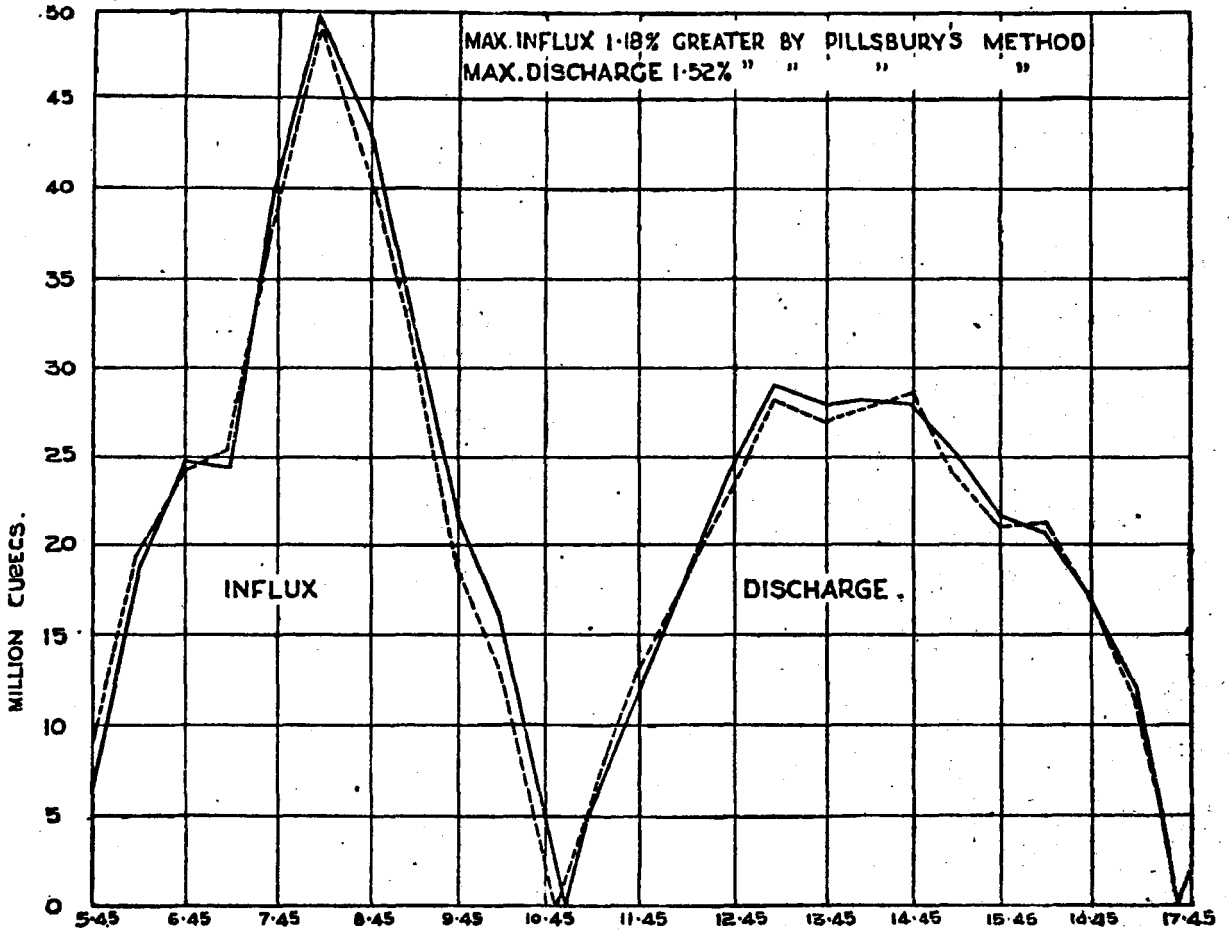
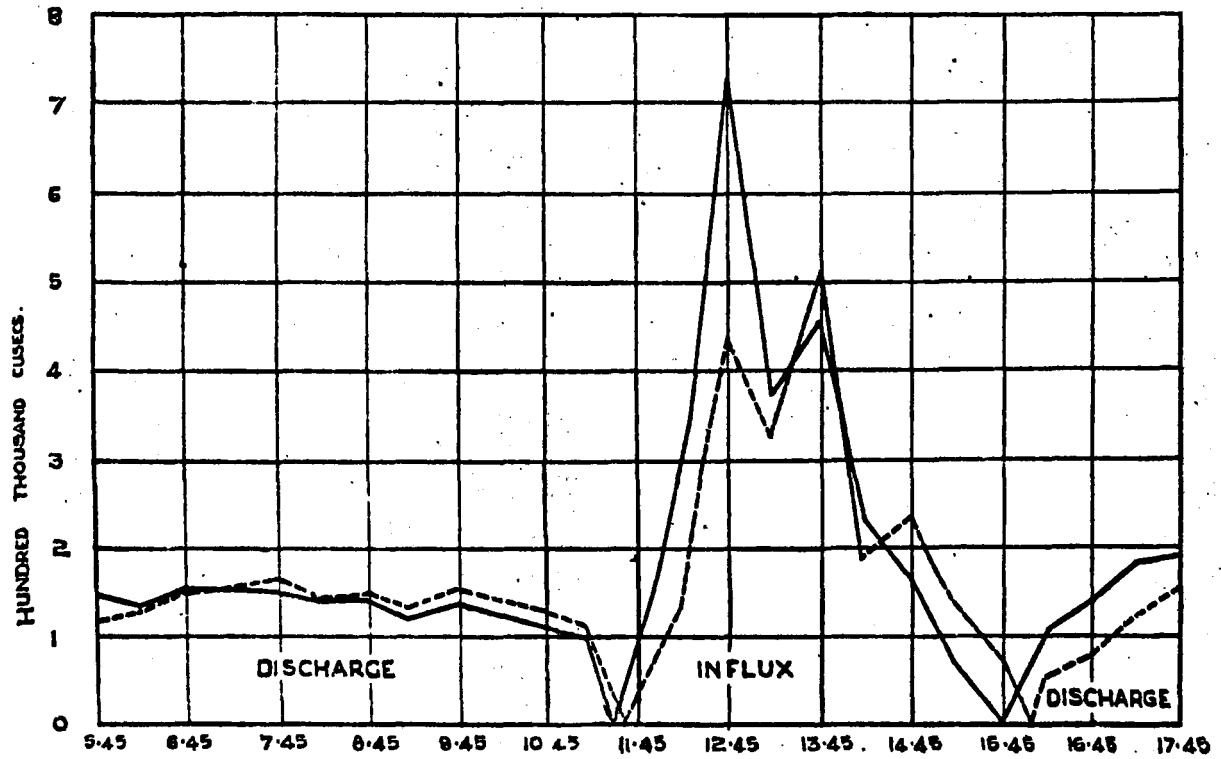


FIG:133

DISCHARGE AND INFLUX AT GARDEN REACH.

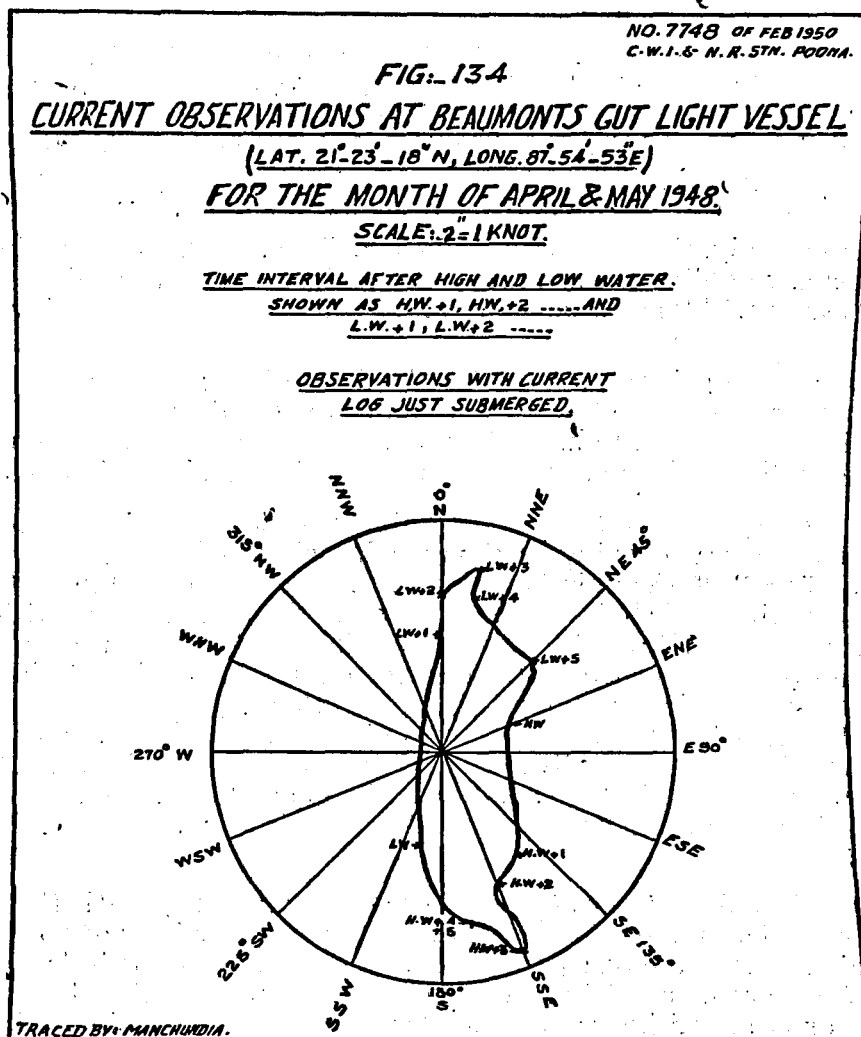


TRACED BY: M.K. SABNIS. COMPARED BY: *[Signature]*

HOOGHLY RIVER—TIDAL MODEL

A perusal of the diagram in fig. 134 shows that it is not so. It is, of course, true that the flood and the ebb have a greater velocity in the northern and southern directions, but the actual direction of the current varies during the course of a cycle as shown in the diagram. No attempt is made here to discuss the possible causes of such 'rotary currents' but suffice it to say that if the flow conditions in the Saugor and Haldia reaches are seriously affected by the existence of such currents in the south, attempts will have to be made to simulate them in the model.

Fig. 134.



The maximum velocities obtaining in the river at various places have been also computed. (See table 4.)

There are data also about surface velocities below old Howrah Bridge for all the days in the year 1938 at intervals of one hour, and some float observations conducted between Akra and Moyapur in 1922 in connection with sewage disposal.

7. *Water characteristics*:—As was mentioned before, for about 8 months in a year there will be hardly any freshet discharge in the Hooghly river and its tributaries (see fig. 135) and the whole reach under investigation will behave like a saline tidal river. But, during the freshet period, the neap tides are overpowered by the fresh water flowing downstream and at Calcutta, the tides are almost held back from progressing upstream. The interaction of the tidal influx and the freshet discharge may result in what is known as a 'salt-water wedge', the denser saline water moving upstream on the bed and the lighter fresh water

Fig. 135.

moving downstream on the surface. This is obviously a disadvantageous factor but as its effect is not felt below Calcutta, it has not been seriously considered.

Fig. 136 Samples of water from the various places in the Hooghly have been analysed for salinity and the findings are in general given in Fig. 136 at Saugor the salinity factor affects the rate of sedimentation and above Calcutta salt water wedge affects the upstream bed movement also.

For the simulation of sedimentation, addition of alum in suitable proportion is most economical.

8. *Head water supply*:—While the importance of head water supply for the Hooghly river in keeping the river alive need not be stressed, any attempt to augment it at the head reaches may increase a charge of coarser silt and result in deterioration of the Hooghly. More offtakes from the Ganges at suitable places may be a good remedy, the importance being laid on getting more silt-free head water supply.

Accordingly, the proposed Damodar, Mor and the Ganges barrage schemes have a direct bearing on the regime of the Hooghly and the model study will take these factors into consideration during investigation.

It has been pointed out by Sir Claude Inglis that the effect of the Ganges barrage may be to raise the bed at the offtakes of the Nadia and hence increase the silt charge there. Accordingly data of the various schemes will be collected in the later stage of model investigation.

9. *During the year 1949*:—Some difficulty was experienced to start with in an accurate laying of the model, owing to the large size of the model. The Port Commissioners have helped a good deal in that direction by sending their liaison officers now and then and helping with their experience in the river. In order to minimise all errors, a network of grid lines is made permanently and with reference to this all the accurately surveyed triangulation stations and other objects are permanently marked and with reference to these, in turn, the bank lines and cross-section positions are carefully marked. In the sea portion, however, before the surveying girder is erected, the model is moulded with reference to level pads spread at intervals; but later, the girder will be used for the purpose.

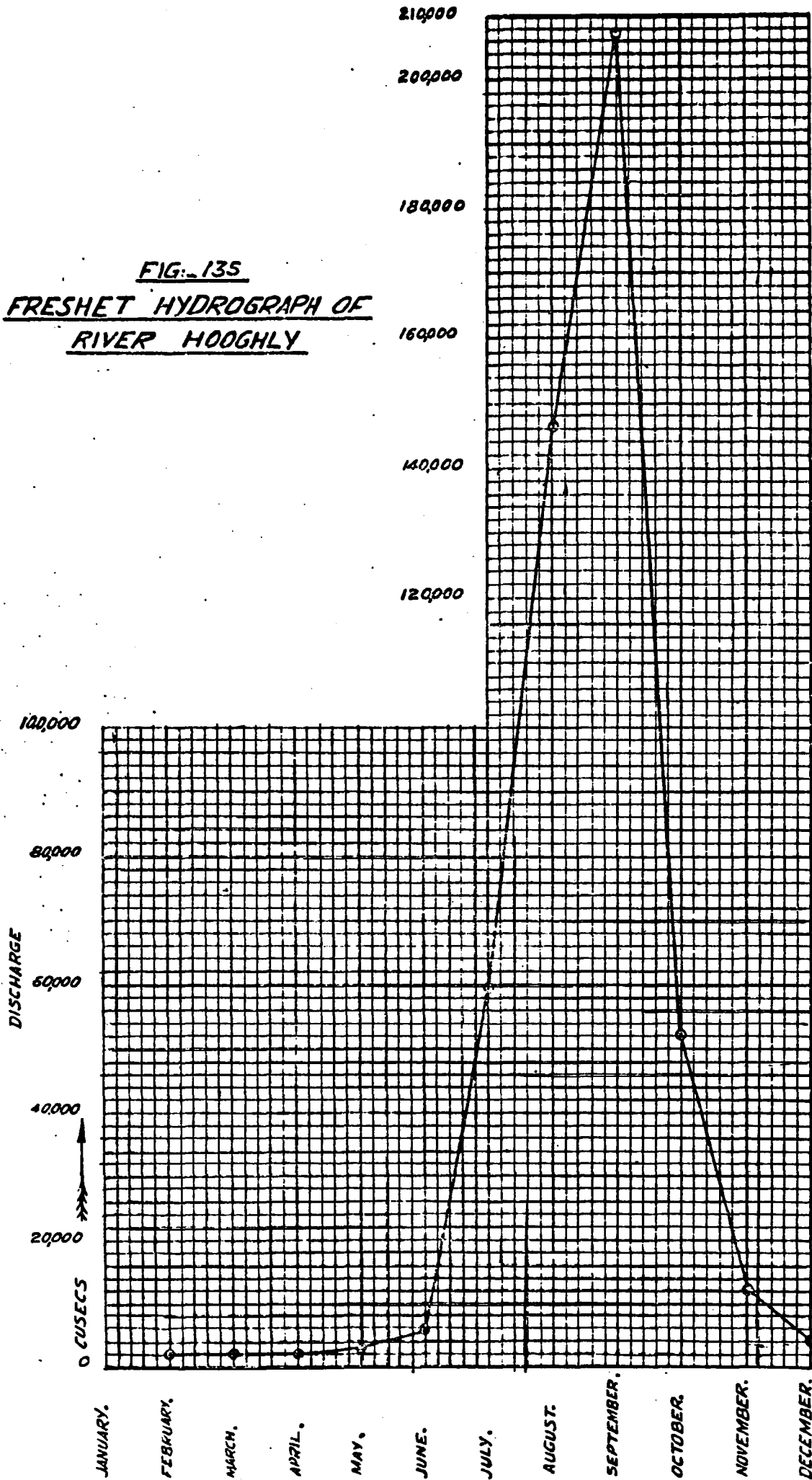
Photo 5648 10. *Scum-formation on the Bed*:—The Hooghly sand used in the model, being very rich in food for the growth of micro-organisms of both plant and animal nature, develops a thin crust of scum on the bed; this prevents free movement of bed material. Under water this film appears spongy exposing millions of air bubbles, (see photograph), thus indicating the aerobic nature of the organisms involved. Microscopic examination indicated that slimy diatoms together with the growing algae formed a tenacious conglomeration of the sand particles and made the bed flow-resistant. This condition was highly undesirable and experiments were undertaken in the Public Health Laboratories, Poona, to evolve economical measures for the suppression of their growth. The results were briefly as follows:—

(a) *Copper sulphate treatment*:—The minimum dose required to kill the algae was 1000 parts per million of sand. This was not practicable. Hence, this treatment was given up although it was very successful as no copper was left in solution and there was no growth even after four weeks.

(b) *Heat treatment*:—The sand was heated until the colour changed to brown. No growth appeared after standing in contact with water and light for more than a fortnight.

(c) *Potassium Permanganate Treatment*:—This treatment appeared to be economical but it gives a very dirty appearance due to the

FIG. 135  
FRESHET HYDROGRAPH OF  
RIVER HOUGHLY



Traced by: S.A Manchundia

Comp. by: [Signature] 391

293

200

160

120

80

40

0

SALINITY GRAINS PER GALLON.

JAN. FEB. MAR. APRIL MAY JUNE JULY AUG. SEP. OCT. NOV. DEC.

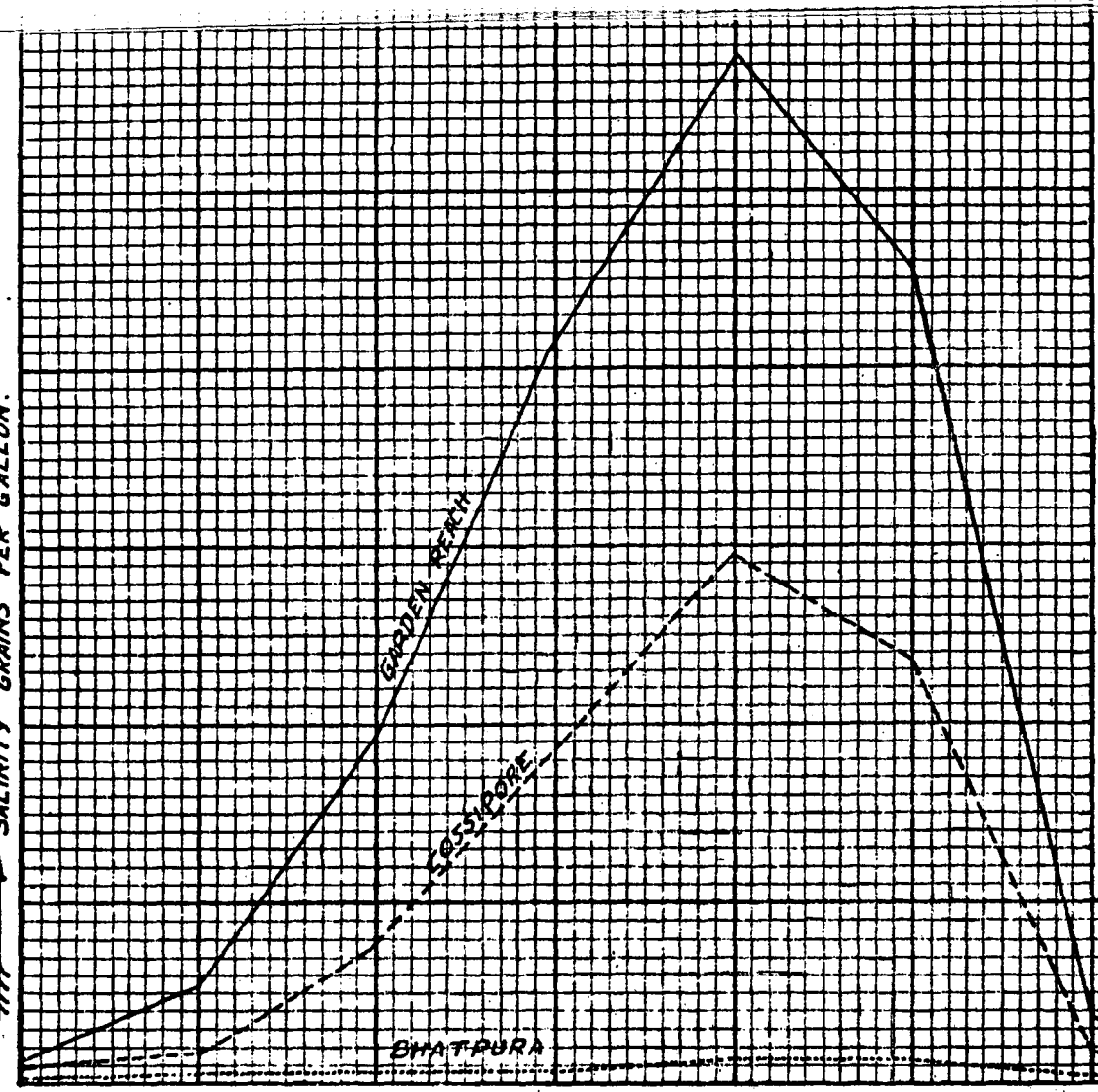


FIG. 136

SALINITY VARIATION MONTH TO MONTH  
(AT BHATPURA, COSSIPORE & GARDEN REACH)

1940 TO 1946.

TRACED BY: S. A. MANCHUNDIA

COMP BY: SPAN

## MODEL OF THE HOOGLY ESTUARY

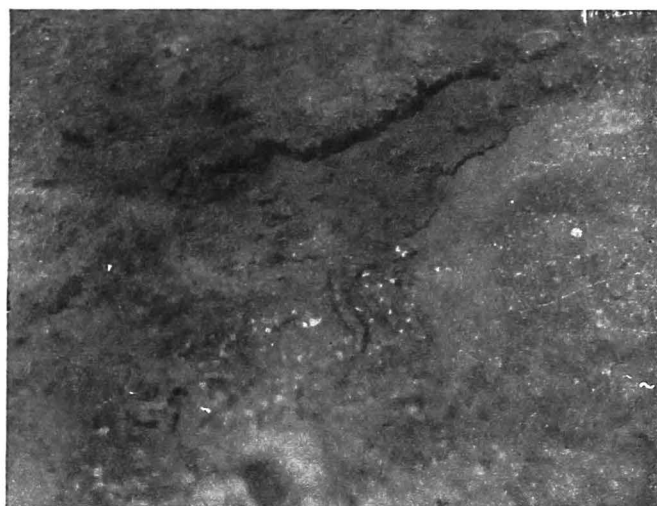


Photo No. 5648-Development of scum on bed of model.

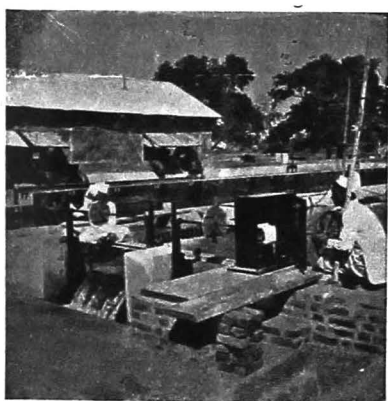


Photo No. 5764-Operation of the sliding weirs for the reproduction of tides.

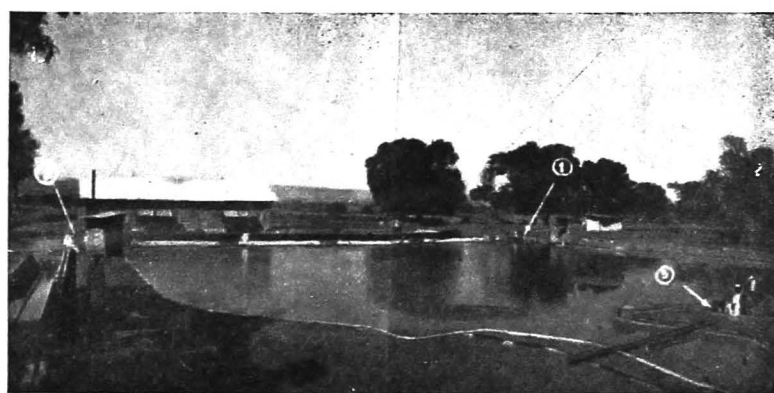


Photo No. 5767—The sea end of the Hooghly model showing the positions of water level recorders and the sliding weirs.  
(1) and (2) Sliding Weirs.  
(3) Recorder

colour of the precipitates of the manganese oxides and hydroxides. The dose required was 10 grains per pound of sand.

(d) *Chlorine treatment*:—Chlorine dose up to 50 p. p. m. was tried and it was found that the treatment was to be given almost every week to check the growth of scum.

Sodium Chlorate and Borax are being tried.

One aspect in the treatment of the sand is to loosen the surface before treatment in order to make it effective.

Some preliminary runs were conducted after treating the bed with 20 p. p. m. of chlorine and subsequently the process has been repeated and it may be stated that chlorine gives good relief to the bed, though it is necessary to treat it every now and then.

11. *Tide reproduction in the model*:—The model being laid as per plans of December 1938, a number of preliminary runs were run with the representative semilunation for the flood period of 1939. The procurement of the several automatic recorders delayed the progress in the early part of the year and attention was paid to the accurate laying of the model, prevention of scum-formation and the analysis of the various data.

Fig. 128

The semilunation simulated is given in Fig. 128. The bore tides were automatically reproduced in a remarkable manner. The surface currents at Balari and Rangafalla, the time lag between the tides in the channel creek and the Saugor Roads, the bed currents in the Ninan reach and the flood and the ebb periods at various places were observed in the model and good simulation was indicated.

Photos 5767 and 5764 indicate the position of the various recorders in the seaward part of the model and the method of manual operation of the sliding weirs for the reproduction of the tides.

Photos 5767  
and 5764

12. *Proposals for the tide generating apparatus*:—The tide period in the model being approximately 4 min. 20 sec., manual operation of the weirs is rather difficult. In the earlier stages of projecting the investigations, it was proposed to run the tides by an automatic cam-electric relay mechanism as was used in the Lake of Maracaibo experiments at the U. S. Waterways Experiment Station, U. S. A.

To describe very briefly, the apparatus consisting of a cam, representing the tide cycles to be reproduced, is kept run at constant speed by a synchronous motor; the water level and the cam are always kept synchronised by means of a reversible motor operating the gates.

An attempt was made to procure a similar apparatus from the U. S. A. through the India Supply Mission; it was however found that each tide control relay system would cost about Rs. 45,000. For the Hooghly model requiring two such sets, it would have cost Rs. 90,000. This price was considered to be rather exorbitant.

Besides, it was found very difficult to design a cam of the U. S. A. apparatus to reproduce one semilunation consisting of about 28 tides, as the size of the cam drum would have been 8 to 9 feet in diameter; or a spiral cam drum should have been designed. There were in addition many other mechanical requirements, viz. synchronous motors for the cam drum and a good method of switching on from one drum to another smoothly, etc.

In the early stages, for lack of an alternative design, it was proposed to split the tides into two or more components for mechanical simplicity and easy design of the cams. The actual process of resolution has been explained in note No. 23 *ibid.*

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In order to make the operation of the weirs easy, it was necessary that there be little distance between the weirs and the gauge station whose tides were to be represented on the cam. It was thus necessary to study the tidal behaviour at the southern limit rather than going on with a knowledge of the Saugor tides alone.

With the limited data that were made available for two days at a great expense, a tentative estimation formula for the southern limit gauge in terms of the observed saugar gauge was derived see note No. 24 *ibid.*

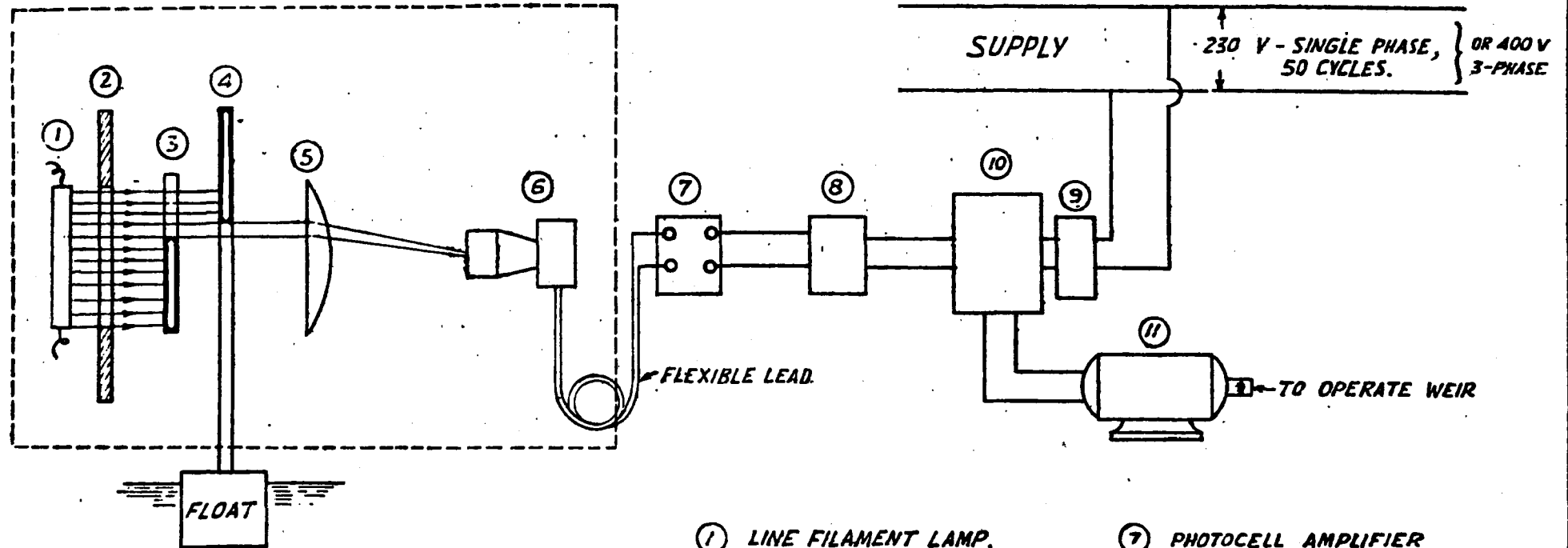
It is now proposed to adopt a design of a new mechanism which would eliminate cams altogether.

The complete arrangement as envisaged is given in the diagrammatic sketch (fig. 137).

Fig. 137





DIAGRAM 1.

ITEMS 1 TO 6 TO BE ENCLOSED IN DARK BOX

- |   |                                    |
|---|------------------------------------|
| ① LINE FILAMENT LAMP.                         | ⑦ PHOTOCELL AMPLIFIER              |
| ② THIN METALLIC SLIT.                         | ⑧ CONTROL PANEL FOR MOTOR, ITEM 11 |
| ③ TIDE CHART MOVED BY CLOCK MECHANISM         | ⑨ TRANSFORMER                      |
| ④ FLEXIGLASS STEM CONNECTED TO FLOAT OR WEIR. | ⑩ RECTIFIER                        |
| ⑤ CYLINDRICAL LENS OF SMALL FOCAL LENGTH.     | ⑪ D.C. MOTOR 3/4 HP.               |
| ⑥ PHOTOCELL                                   |                                    |

## 23. RESOLUTION OF HOOGHLY TIDES AT SAUGOR INTO TWO PERIODIC COMPONENTS

### *Introduction*

It was originally intended to design a cam representing the tidal cycles to be reproduced based on the tidal data at the southern limit, where the automatic tide generating mechanism of the model is proposed to be installed.

As mentioned earlier in the Report, for better reproduction, it was considered desirable to choose a complete semilunation consisting of 28 tides, varying from neap to the spring and to the neap, instead of assuming a single tide to represent various seasons. If the periphery of the cam was shaped according to these tides, it is evident that the cam would be unmanageably large. So it was decided that a number of smaller size cams should be designed in such a way that when they are run together, the resultant tides are reproduced.

Obviously, the larger the number of cams, the better is the accuracy of the result; on the other hand the mechanical equipment necessary for the synchronised operation of a large number of cams with different periods of rotation would have become extremely complex and difficult and hence it was decided to resolve the tides only into two components which would enable the design of two cams of manageable sizes.

At the time of this investigation no tidal data at the southern limit were available. So, to start with, a semilunation was selected from the Saugor data records for the resolution of the tides; using the same method, the southern limit data, when available, may be similarly treated.

2. The theory for the resolution of tide gauge variations observed at Saugor into two periodic components is briefly outlined below. The data has been considered for gauges recorded at half-hourly intervals for an entire semilunation commencing from 00:30 hours on 1st May 1938 to 14.00 hours on 15th May 1938 (i. e. for 700 observations).

3. Let the periodic series  $S_p$  contain  $p$  equally spaced readings

$$Z_1, Z_2, \dots, Z_p \quad (1)$$

To find if  $S_p$  can be generated within the desired degree of accuracy by means of two component periodic series

$$S_{r_1} \equiv x_1, x_2, \dots, x_{r_1} \quad (2)$$

$$\text{and } S_{r_2} \equiv y_1, y_2, \dots, y_{r_2} \quad (3)$$

where  $r_1$  and  $r_2$  are any two positive integers (less than  $p$ ), having  $p$  as their least common multiple; also let  $r_1 > r_2$ .

Suppose, the two series,  $S_{r_1}$  and  $S_{r_2}$  actually possess the desired characteristics so that by compounding the  $t^{\text{th}}$  member of  $S_{r_1}$  with the  $t^{\text{th}}$  member of  $S_{r_2}$  an estimate of  $t^{\text{th}}$  member of the parent series  $S_p$  is obtained. Denote the estimated series by

$$S_p' \equiv Z_1', Z_2', \dots, Z_p' \quad (4)$$

The  $p$  equations of expectation are written down as

$$\left. \begin{aligned} E(Z_1) &= x_1 + y_1 \\ &\dots \\ E(Z_{r_2}) &= x_{r_2} + y_{r_2} \\ E(Z_{r_2+1}) &= x_{r_2+1} + y_1 \\ &\dots \\ E(Z_p) &= x_{r_1} + y_{r_2} \end{aligned} \right\} \quad (5)$$

The best estimates of the  $r_1 + r_2$  values of the parameters

$$x_1, \dots, x_{r_1} \text{ and } y_1, \dots, y_{r_2}$$

are then determined from the  $(r_1 + r_2)$  normal equations obtained by minimising the sum of squares

$$U \equiv \sum (Z_t - x_t - y_t)^2 \quad (6)$$

in accordance with the principle of least squares, the appropriate periodic suffixes repeating for  $x_t$  and  $y_t$  after  $t = r_1$  and  $t = r_2$  respectively.

If now  $k$  is the highest common factor of  $r_1$  and  $r_2$  so that

$$r_1 = mk \text{ and } r_2 = nk \quad (7)$$

$m$  and  $n$  being two integers prime to each other, the normal equations (5) will group themselves into  $k$  independent sets containing  $m + n$  equations in each set. A peculiar feature of the  $(m + n)$  normal equations in any set, however is that they do not all appear as  $(m + n)$  independent equations but have one inter-connected relation between themselves as can be easily verified. Thus it will not be possible to obtain unique solutions for the  $x$ 's and  $y$ 's while a combination like  $(x_i + y_i)$  of two elements building up an estimate of a  $Z$  remains uniquely soluble. In other words while an initial set of  $k$  values (corresponding to the  $k$  independent sets of normal equations) among the series (2) and (3) can be fixed in an infinite number of ways the generated series for (4) will remain unique.

4. The abstract discussion will be better understood when the normal equations are actually written down for the series  $S_p$  of 700 half-hourly gauge readings mentioned in para 2, above. From various practical considerations the values of  $r_1$  and  $r_2$  have been fixed respectively at 175 and 140 so that

$$m = 5, n = 4 \text{ and } k = 35 \quad (8)$$

With these values of  $r_1$  and  $r_2$  a single run of the cams would cover respectively one-fourth and one-fifth of a semilunation. The  $(m + n)$  normal equations for the  $s^{\text{th}}$  set are obtained by differentiating  $U$  with respect to the  $(m + n)$  unknown parameters.

$$\left. \begin{array}{l} x_s, x_{s+35}, x_{s+70}, x_{s+105}, x_{s+140} \\ y_s, y_{s+35}, y_{s+70}, y_{s+105} \end{array} \right\} \quad (9)$$

They are respectively

$$\begin{aligned} 4x_s + y_s + y_{s+35} + y_{s+70} + y_{s+105} \\ = Z_s + Z_{s+175} + Z_{s+350} + Z_{s+525} \end{aligned} \quad (i)$$

$$\begin{aligned} 4x_{s+35} + y_s + y_{s+35} + y_{s+70} + y_{s+105} \\ = Z_{s+35} + Z_{s+210} + Z_{s+353} + Z_{s+560} \end{aligned} \quad (ii)$$

$$\begin{aligned} 4x_{s+70} + y_s + y_{s+35} + y_{s+70} + y_{s+105} \\ = Z_{s+70} + Z_{s+245} + Z_{s+420} + Z_{s+595} \end{aligned} \quad (iii)$$

$$\begin{aligned} 4x_{s+105} + y_s + y_{s+35} + y_{s+70} + y_{s+105} \\ = Z_{s+105} + Z_{s+280} + Z_{s+455} + Z_{s+630} \end{aligned} \quad (iv)$$

$$\begin{aligned} 4x_{s+140} + y_s + y_{s+35} + y_{s+70} + y_{s+105} \\ = Z_{s+140} + Z_{s+315} + Z_{s+490} + Z_{s+665} \end{aligned} \quad (v)$$

$$\begin{aligned} 5y_s + x_s + x_{s+35} + x_{s+70} + x_{s+105} + x_{s+140} \\ = Z_s + Z_{s+140} + Z_{s+280} + Z_{s+420} + Z_{s+560} \end{aligned} \quad (vi)$$

$$\begin{aligned} 5y_{s+35} + x_s + x_{s+35} + x_{s+70} + x_{s+105} + x_{s+140} \\ = Z_{s+35} + Z_{s+175} + Z_{s+315} + Z_{s+455} + Z_{s+595} \end{aligned} \quad (vii)$$

RESOLUTION OF HOOGHLY TIDE

$$5y_{s+70} + x_s + x_{s+35} + x_{s+70} + x_{s+105} + x_{s+140} = Z_{s+70} + Z_{s+210} + Z_{s+350} + Z_{s+490} + Z_{s+630} \quad \text{(viii)}$$

$$5y_{s+105} + x_s + x_{s+35} + x_{s+70} + x_{s+105} + x_{s+140} = Z_{s+105} + Z_{s+245} + Z_{s+385} + Z_{s+525} + Z_{s+665} \quad \text{(ix)}$$

...(10)

By adding both the sides of equations (i) to (v) and again of equations (vi) to (ix) the same identical relation holding between the five x's and four y's will be separately produced twice from the two separate operations, and thus provide evidence that the 9 normal equations are not all entirely independent as already mentioned in the previous paragraph as a peculiar feature of the sets of the normal equations. By simple algebraic process the normal equations will yield the following neat solution:

$$Z'_{s+l} = (\text{Mean of the column containing } Z_{s+l}) + (\text{Mean of the row containing } Z_{s+l}) - (\text{Mean of all } Z_s) \quad \text{...(11)}$$

In the block below:—

$Z_s$	$Z_{s+560}$	$Z_{s+420}$	$Z_{s+280}$	$Z_{s+140}$
$Z_{s+175}$	$Z_{s+35}$	$Z_{s+505}$	$Z_{s+455}$	$Z_{s+315}$
$Z_{s+350}$	$Z_{s+210}$	$Z_{s+70}$	$Z_{s+630}$	$Z_{s+490}$
$Z_{s+525}$	$Z_{s+385}$	$Z_{s+215}$	$Z_{s+105}$	$Z_{s+665}$

...(12)

As an illustrative example the numerical solutions of the s=5th set of normal equation are obtained as under:—

					Total	Mean
$Z_s=51$	136	141	71	185	584	116.8
58	117	140	65	164	544	108.8
86	155	143	67	176	627	125.4
74	108	104	46	176	508	101.6
Total	269	516	528	249	701	2263
Mean	67.25	129.0	132.0	62.25	175.25	113.15

$$Z'_s = 67.25 + 116.8 - 113.15 = 70.90$$

$$Z'_{40} = 129.0 + 108.8 - 113.15 = 124.65$$

$$Z'_{75} = 144.25 \quad Z'_{285} = 65.90 \quad Z'_{495} = 187.5$$

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$Z'_{110} = 50.20$	$Z'_{320} = 170.90$	$Z'_{530} = 55.70$
$Z'_{145} = 178.90$	$Z'_{355} = 79.50$	$Z'_{565} = 132.65$
$Z'_{180} = 62.90$	$Z'_{390} = 117.45$	$Z'_{600} = 127.65$
$Z'_{215} = 141.25$	$Z'_{425} = 135.65$	$Z'_{635} = 74.50$
$Z'_{250} = 120.45$	$Z'_{460} = 57.90$	$Z'_{670} = 163.70$

5. The numerical solutions were obtained by the above method in respect of all the 35 sets of normal equations by giving special weightages to the high and low gauges in order to obtain better agreement between the observed and the estimated extreme values of tide gauges. The choice of the weights which is subjective enhances the accuracy between the range values at the cost of intermediate ones and vice versa. The 700  $Z'$  values are plotted against the corresponding observed values in Fig. 138(a). If time lag is discernible in certain reaches of the fit, and the high and low values are not always attained, it is only pointed out that in the process of the resolution of the data, the discrepancies may be attributable to the limitations imposed on

- (i) the number of cams, and
- (ii) the period of rotation of the cams.

6. Thus after obtaining unique solutions for the  $Z'$ 's, the problem of the resolution into the respective  $x$  and  $y$  components was tackled. Availing the 35 degrees of freedom for the initial fixation of 35 values of  $x$ 's or  $y$ 's the first 35 values of the  $x$ 's have been arbitrarily fixed in the present instance. The remaining values of  $x$ 's and  $y$ 's which follow therefrom, are all shown concisely in tables 1 and 2.

Fig. 138 (b). For the construction of the two cams the series of values of  $x$ 's and  $y$ 's have been actually plotted in Fig. 138(b) and the two curves drawn. The ordinate for the curve for  $Z'$  at any point can now be easily verified to be equal to the sum of ordinates for the curves for  $x$  and  $y$ , which thus provide the desired resolved components of the parent series.

7. It may be noted in conclusion that the process of resolution has at no time put any specifications on the nature or form of the component series, thus leaving a wide choice for the actual mode of graduation.

HALF-HOURLY TIDE GAUGE DATA AT SAUGOR  
FOR SEMILUNATION 1ST TO 15<sup>TH</sup> MAY 1938

REG. NO. 1808 OF MARCH 1937  
 S. D. 1202 27th APR 1938

RESOLUTION IN TWO PERIODIC COMPONENTS

FIG.138(a) OBSERVED GAUGES AT SAUGOR (—)  
SUPERIMPOSED BY ESTIMATED GAUGES (---)

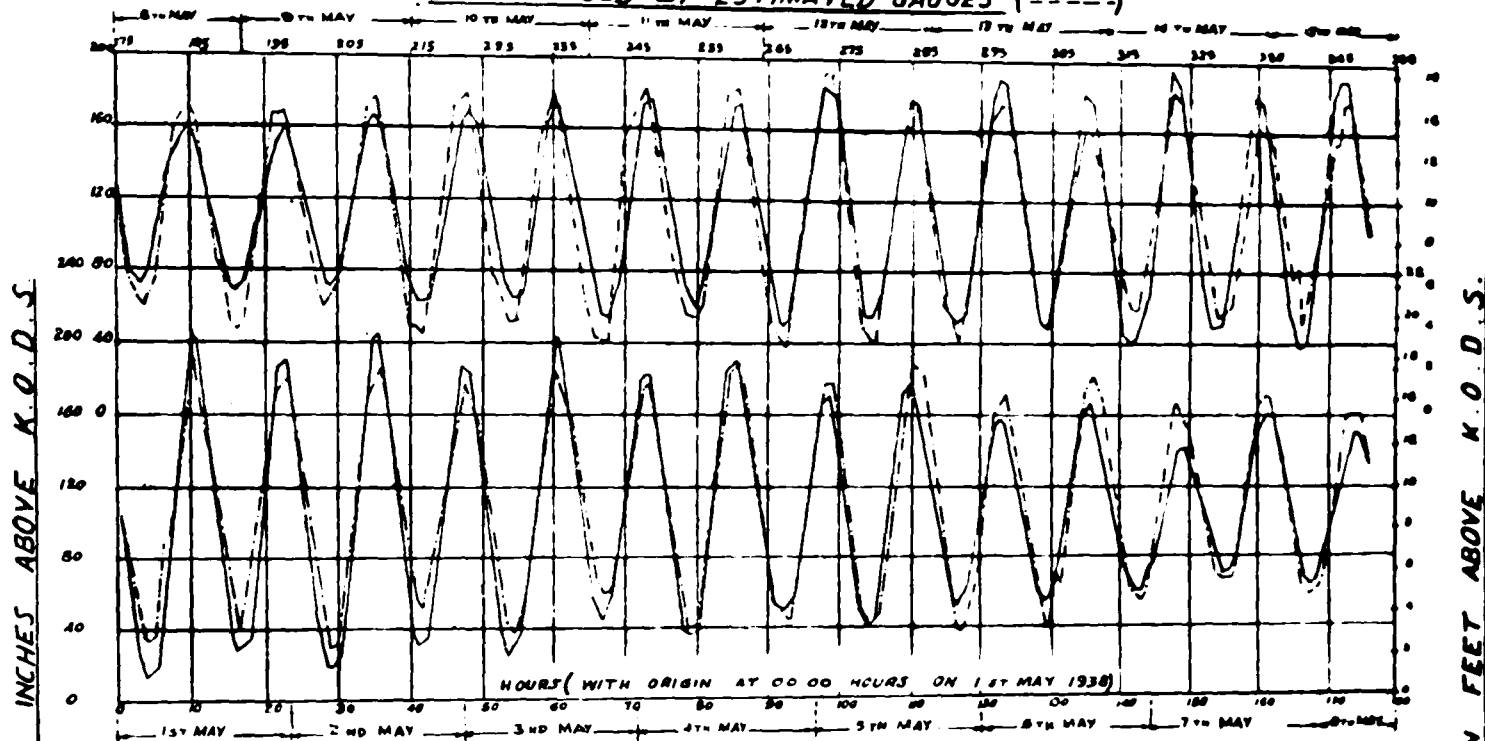


FIG.138(b) RESOLVED COMPONENT WITH PERIOD 140 (HALF HOURS)

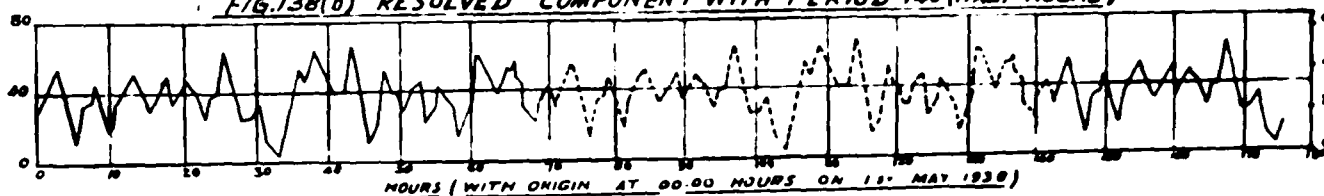
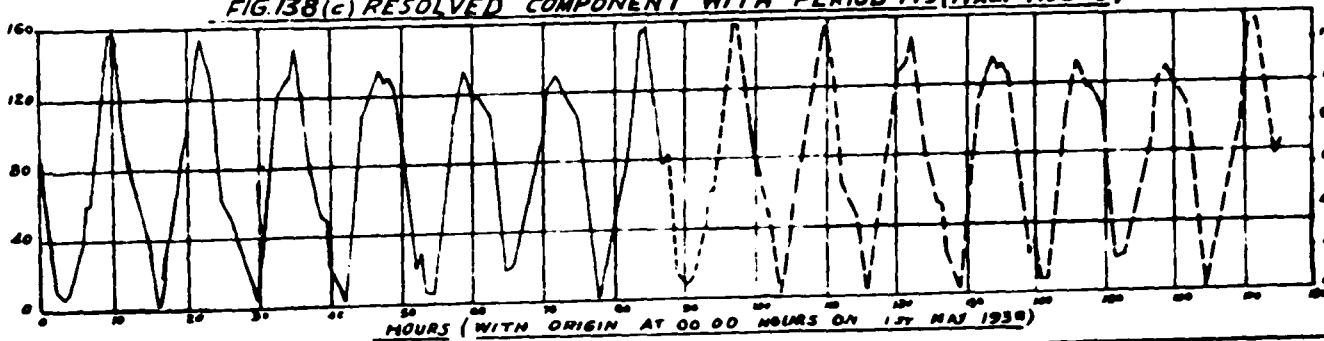


FIG.138(c) RESOLVED COMPONENT WITH PERIOD 175 (HALF HOURS)



RESOLUTION OF HOOGLHY TIDES

TABLE 1

Values of the resolved component series with period 175  
(in inches measured above K. O. D. S.)

Read horizontally for  $x_1$  and vertically for  $x_m$  for cell value  $x_{im}$ .

	$x_0$	$x_1$	$x_2$	$x_3$	$x_4$	$x_5$	$x_6$	$x_7$	$x_8$	$x_9$
$x_0$		87	69	51	33	16	12	8	11	13
$x_1$	26	32	40	58	61	69	83	107	127	144
$x_2$	160	138	143	130	110	88	82	72	62	57
$x_3$	48	34	23	9	2	5	37	45	56	67
$x_4$	78	95	114	133	146	155	152	140	130	94
$x_5$	72	62	59	53	48	47	30	16	9	1
$x_6$	9	27	44	69	88	107	118	126	131	133
$x_7$	134	150	133	120	101	81	72	62	54	53
$x_8$	47	26	18	14	10	4	18	43	66	88
$x_9$	118	118	127	132	136	129	133	130	128	119
$x_{10}$	107	87	69	49	33	22	31	19	10	9
$x_{11}$	11	26	41	64	84	110	121	124	134	132
$x_{12}$	120	119	121	118	112	108	85	69	51	37
$x_{13}$	23	23	24	33	37	40	54	62	69	79
$x_{14}$	89	121	127	131	130	124	123	118	115	109
$x_{15}$	103	84	62	48	26	3	1	10	21	32
$x_{16}$	47	54	65	75	82	94	114	136	154	158
$x_{17}$	162	153	132	113	95	77	...	...	...	...

TABLE 2

Values of the resolved component series with period 140  
(in inches measured above K. O. D. S.)

Read horizontally for  $y_1$  and vertically for  $y_m$  for all value  $y_{im}$ .

	$y_0$	$y_1$	$y_2$	$y_3$	$y_4$	$y_5$	$y_6$	$y_7$	$y_8$	$y_9$
$y_0$	...	29	39	45	50	55	49	36	26	20
$y_1$	10	22	30	33	34	43	40	30	23	19
$y_2$	15	32	35	34	41	50	48	44	36	33
$y_3$	29	33	33	40	41	48	30	37	40	41
$y_4$	47	41	40	36	32	23	33	36	37	47
$y_5$	64	60	49	40	31	21	22	23	24	23
$y_6$	33	28	9	10	4	1	13	18	34	45
$y_7$	53	41	42	48	51	64	60	57	50	46
$y_8$	35	35	36	37	55	65	62	46	34	23
$y_9$	9	13	25	31	43	50	36	35	27	26
$y_{10}$	26	35	39	40	44	45	18	26	27	40
$y_{11}$	40	37	35	28	23	12	19	25	23	62
$y_{12}$	62	54	50	49	42	35	39	46	51	50
$y_{13}$	66	44	41	27	24	21	18	26	32	40
$y_{14}$	43	...	...	...	...	...	...	...	...	...

## 24. ESTIMATION OF SOUTHERN LIMIT GAUGE FROM SAUGOR GAUGE RECORDS

Tide gauges are regularly observed at Saugor in the Hooghly Estuary. Consequent on the problem for improving the navigability of the estuary being referred for model investigation, the installation of a tide generating apparatus at a point in Latitude 21-11-00N, Longitude 88-16-15E (in this paper called the southern limit) 25 miles downstream of Saugor was considered a necessity though unfortunately the gauge records for this point were not observed. The Port Commissioners of Calcutta accordingly made special arrangements on 3rd and 4th March 1949 for the simultaneous observations of gauges at Saugor and at southern limit with the object of investigating the possibility of a relation for estimating the southern limit gauge from the past records maintained for the Saugor gauge.

The continuous gauge data of two stations for 31 hours from 12-00 hrs. of 3rd March 1949 to 19-00 hrs. of 4th March 1949 obtained from the Port Commissioners has been reproduced in Fig. 139(a). Scrutiny of the two gauge curves reveals that the variation of the gauges follows the law of simple harmonic motion, with

- (i) different amplitudes,
- (ii) a phase difference and
- (iii) almost the same period.

Following the premises of the simple harmonic law, the gauge data for southern limit were plotted against the corresponding gauge data for Saugor (in Fig. 140), when the points were found to lie along the circumference of an ellipse. The major axis of this ellipse, as is only natural is inclined to the X-axis and the centre does not coincide with the origin as the gauges are measured from the Kidderpore Old Dock Sill as datum (and not from mean water level).

Before attempting to formulate an expression for the ellipse, it may be useful to remember that it will obviously depend on the amplitudes of the two harmonic motions. The process of estimation however remains rather laborious and as the amplitudes too do not remain constant for any considerable length of time, it was preferred to have recourse to an alternative method.

As the periods of the two gauges and the phase lag between them were observed to be nearly constant in the limited data, the best alternative method was to fit a straight line to the two simple harmonic motions. A primary condition, of course, remains the elimination of time lag, which has been graphically estimated at nearly one hour. This has been further confirmed by the highest value (.99) obtained for the coefficient of correlation for one hour time lag, when separate correlation studies were carried out between the two gauges separated by intervals of 30, 45, 60, 75 and 90 minutes.

The data of the Saugor gauge have therefore been plotted against the data of southern limit observed an hour earlier in Fig. 141, and the points conform to a linear distribution. The equation derived for this line by the method of least squares is

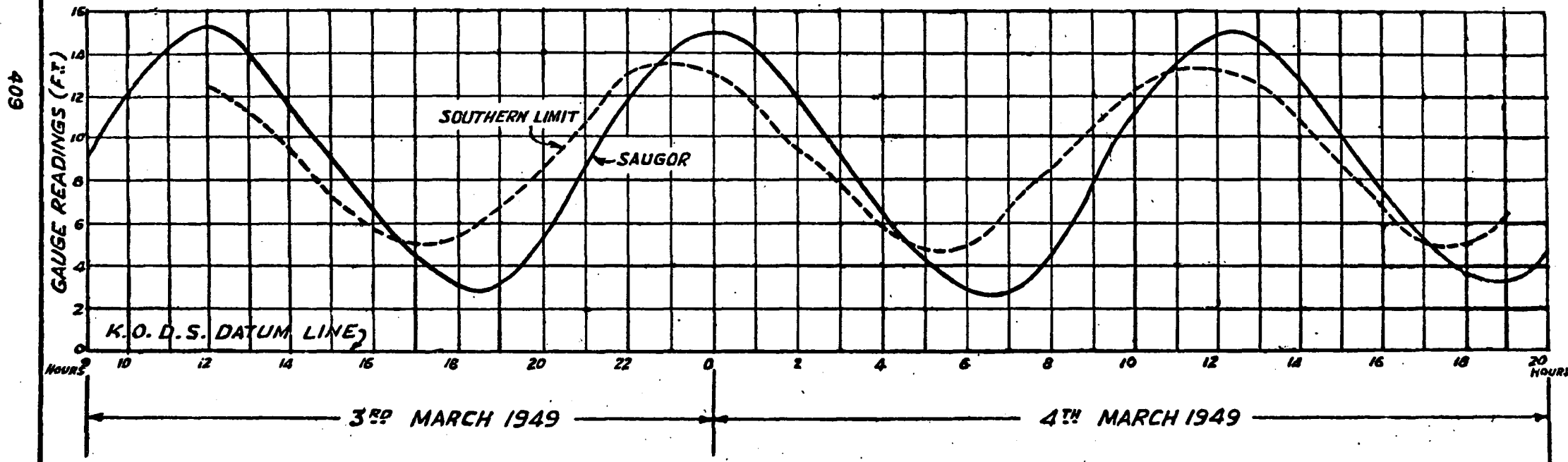
$$y_{n-1} = 0.71x_n + 2.83 \quad \dots (1)$$

where  $x_n$  = the height of Saugor gauge in feet measured above K. O. D. S. at  $n^{\text{th}}$  hour.

$y_{n-1}$  = height of southern limit gauge in feet above K. O. D. S. measured at  $(n-1)^{\text{th}}$  hour.



FIG. 139  
COMPARATIVE TIDAL CURVES  
AT SAUGOR AND LOWER SAUGOR SAND BUOY (SOUTHERN LIMIT)  
A5 (LAT. 21-11-00 N. LONG. 88-16-15 E.)  
ON 3<sup>RD</sup> AND 4<sup>TH</sup> MARCH 1949 (APOGEE SPRING TIDE)



TRACED BY B.G. DESHPANDE 25-2-50 CHECKED BY..

FIG. 140 & 141  
DATA OF GAUGE  
AT SOUTHERN LIMIT PLOTTED AGAINST GAUGE AT SAUGOR  
HALF HOURLY READINGS ON 3RD AND 4TH MARCH 1949

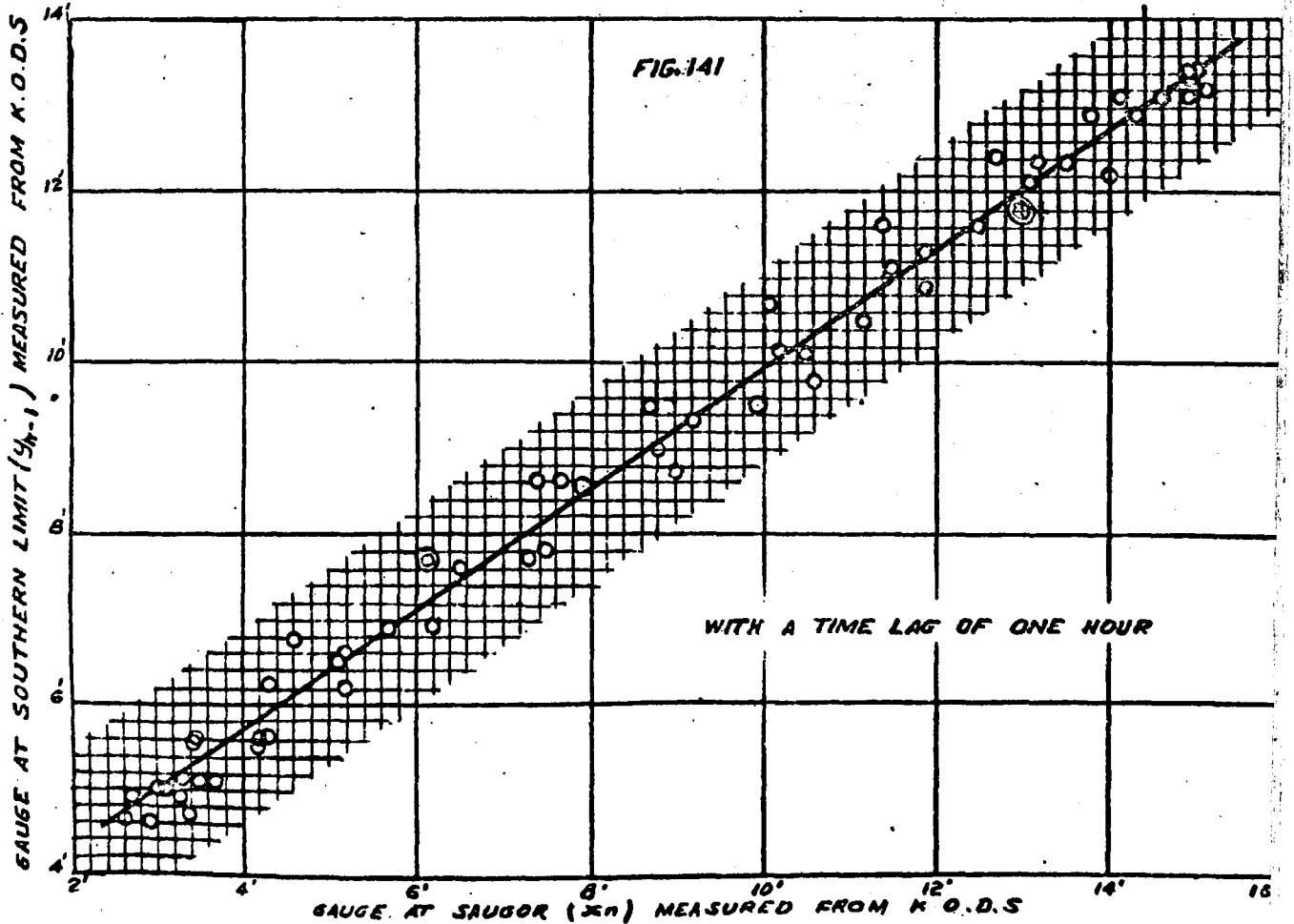
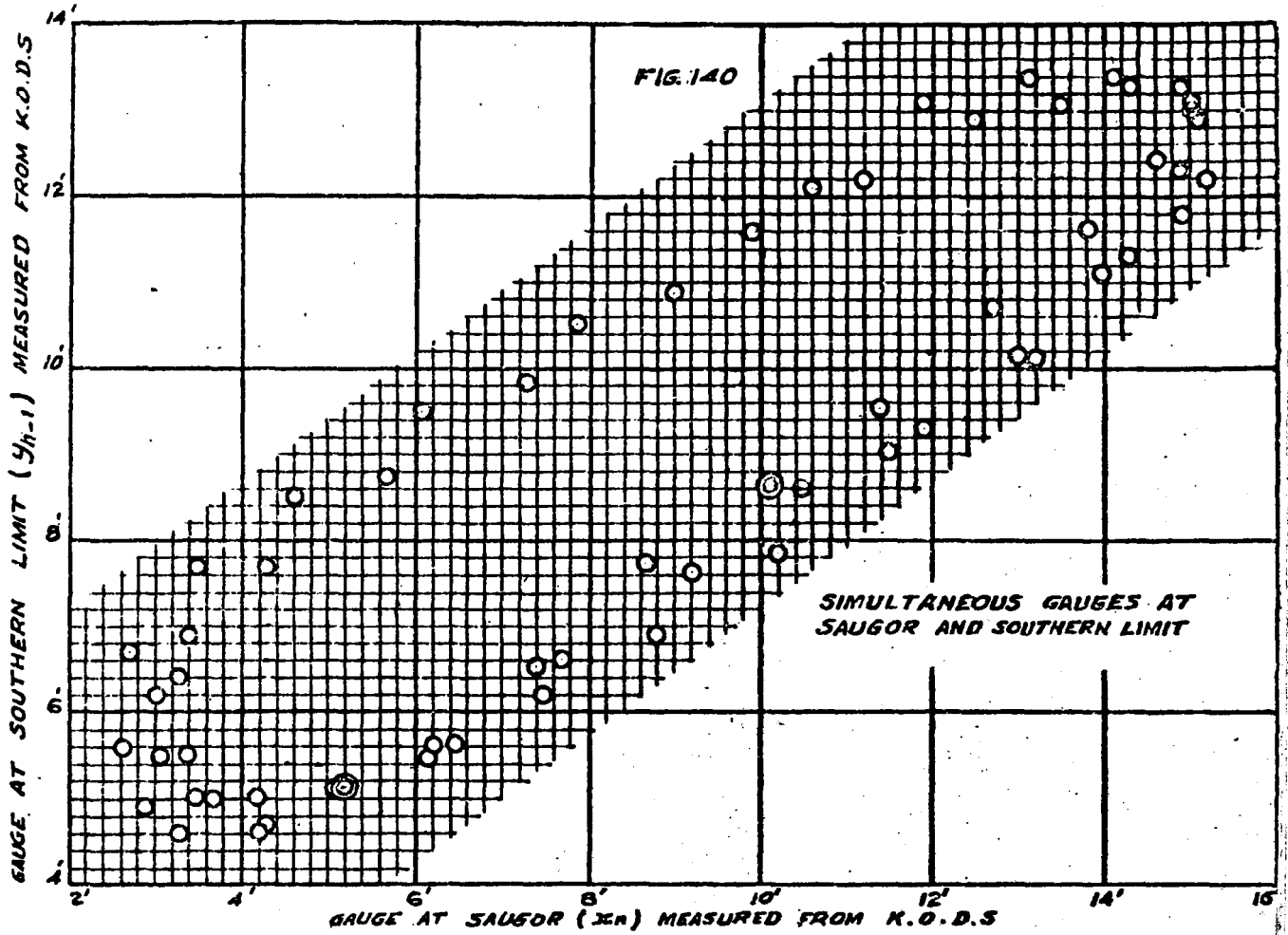
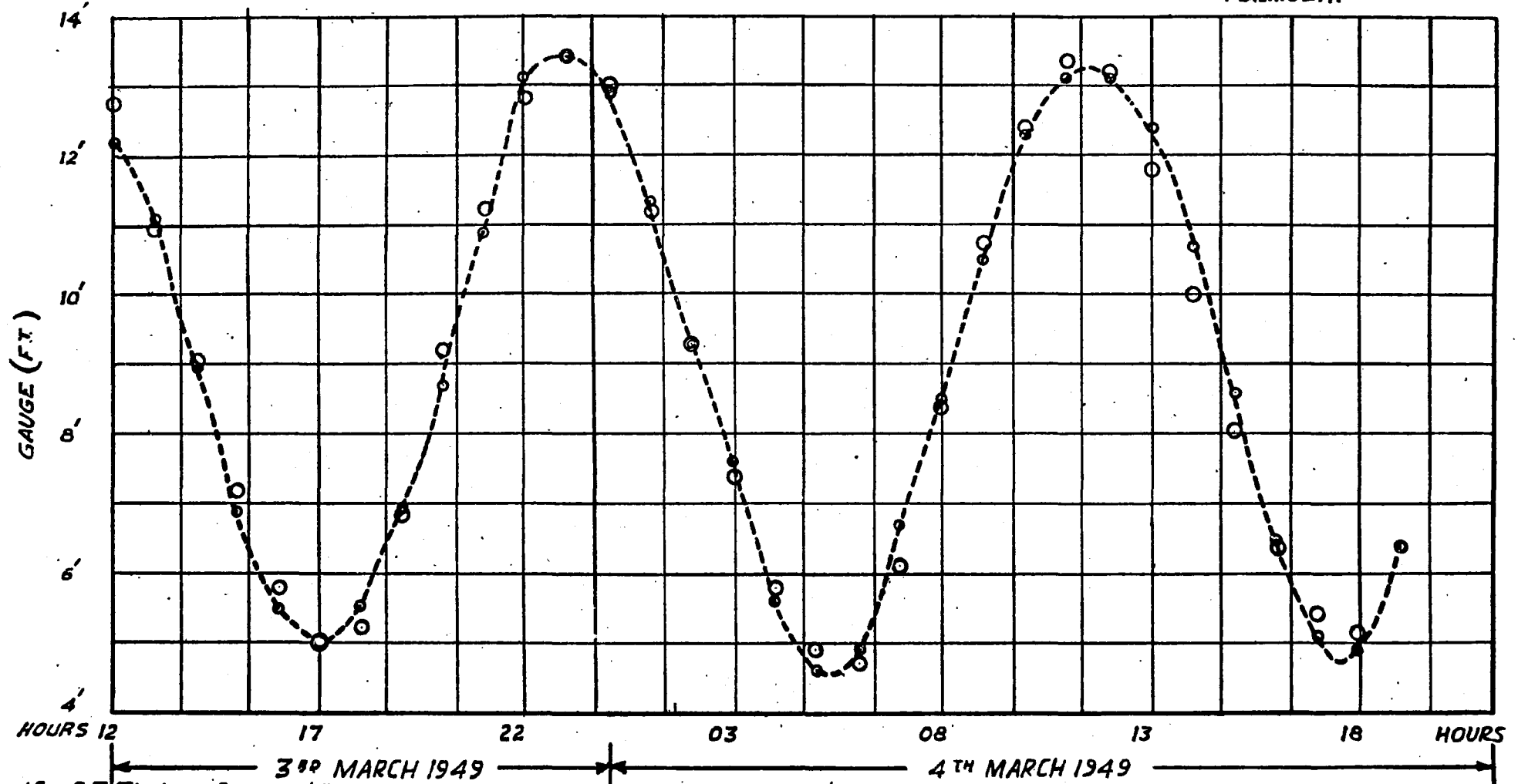


FIG. 142

ESTIMATED GAUGES AT SOUTHERN LIMIT  
SUPERIMPOSED ON OBSERVED GAUGES.

○ OBSERVED TIDAL READINGS.  
○ ESTIMATE BASED ON THE ST. LINE  
FORMULA.



114

Traced By, R.T. Thakor. Compared By.

#### ESTIMATION OF SOUTHERN LIMIT GAUGE FROM SAUGOR GAUGE RECORDS

The standard error of the fit equals 0.32 feet which expressed as a co-efficient of variation  $= \frac{0.32}{9.3} \times 100 = 3.64$  per cent. The small value of the co-efficient of variation shows the remarkable goodness of fit accomplished by the simple linear relation. This is also clear (Fig. 142) from the close agreement of the superimposed values of the estimated gauges at southern limit, with their observed values. Fig. 142

Considering the physical concept of the co-efficient of  $x_1$  in (1), it is easy to see that the slope (71) measures the ratio of the amplitudes of the two simple harmonic motions. The validity of the equation thus remains subject to the constancy of the ratio of the two amplitudes in any length of time.

Equation (1) has been utilised for the purpose of estimating the gauges at southern limit from the gauges observed at Saugor for the semilunations commencing from 5th February 1949 to 19th February 1949. It will be most illuminating to verify the strength of the equation with further observed data.

## 25. MODEL EXPERIMENTS FOR REDUCTION OF 'RANGE' IN THE MADRAS HARBOUR

### *Preliminary Note*

*Synopsis:*—Madras Harbour is an important port on the east coast of India and the only one which can admit vessels up to 26 ft. draught. At the time of cyclonic storms which originate in the Bay of Bengal, generally in the months of October and November, disturbance is experienced in the harbour and ships cannot berth along the quays comfortably. In storms of severe intensity there is a very great danger of ships breaking their shackles and going astray. Model experiments are, therefore, undertaken on behalf of the Madras Port Trust to devise measures to minimise this trouble.

*Old Harbour:*—The harbour, according to original plan, was completed in the year 1895. It consisted of a pair of breakwaters, 3,000 ft. apart stretching out seaward parallel to one another and then coming round towards each other until they left an opening of 515 ft. between their pier heads. They thus enclosed an area of about 200 acres, of which three-fourths were available for vessels clearing between 13 and 15 feet.

This entrance became untenable in two ways:—

(i) *Wave action:*—Waves entered freely into the harbour through the 515 feet entrance. There was neither lateral expansion of the waves nor was there any spending beach to dissipate this wave energy. The swell, entering the harbour with full force, nearly all the year round, made the handling of the vessels slow, difficult and dangerous.

(ii) *Sand accretion and silting of the harbour:*—The after-effect of construction of the harbour i. e. an artificial projection into the sea was an accretion of sand to the south of the harbour and erosion on the north.

By 1911 the shore line had advanced by 2,500 ft. since 1877 and the base of this triangular patch reached as far as 5 miles south of the harbour. There was a corresponding erosion of the shore for a length of about 3 miles to the north of the harbour. This was however checked by revetting the shore with loose stone pitching for a distance of about  $1\frac{3}{4}$  miles.

Two theories were advanced as the probable causes of the accretion:—

(i) *Littoral Drift:*—It was believed that littoral drift along the coast was responsible for this accretion. Any artificial projection caused slackening of velocities in its neighbourhood with consequent piling of material on one side of the projection. Actual observation of velocities of littoral drift conclusively showed this had comparatively little to do with the formation of the accretion especially as the latter is composed of sand too coarse to be carried in suspension by velocities which are very slack close to the shore.

(ii) *Wave Action the Cause of Wave Travel:*—“There is, now, ample evidence to show that the accretion is due, chiefly, to the net effect in the year, of two kinds of oblique wave action. On this coast there exist, two classes of waves, one during the prevalence of the south-west monsoon (March to August), rolling in, on the land with a somewhat northerly trend, at the most perhaps 30° out of parallelism with the general line of the shore and the other during

## REDUCTION OF RANGE IN MADRAS HARBOUR

the north-east monsoon (middle of October to middle of January) rolling in, on the land with somewhat a southerly trend and also, at the most, 30° out of parallelism with the general line of shore”.

The waves rolling in from the deeper sea begin to affect the bottom appreciably at about 4 fathom line and throw the bed material in suspension. Due to the oblique approach of waves to the shore, water acquires a long-shore component. This gains more and more momentum as depth of water decreases and finally when the wave dissipates on the shore, the net effect is transportation of bed material along the shore. As the duration of south-west monsoon is longer, the resulting tendency is that of accretion to the south and that of erosion to the mouth of the harbour.

The whole phenomenon was a source of anxiety to the port authorities for the reason that the finer variety of sand was finding entrance into the harbour and was silting up at the rate of 1 foot per year. It was soon realised that extension of shore line up to the end of south break-water was a matter of time only and the sand would find an easy way into the harbour at an alarming rate, making it impossible to maintain proper depths without having recourse to heavy dredging.

Both these points led the authorities to take up remodelling of the harbour seriously. The following alterations were made after 1905:—

(i) The old eastern entrance was closed and a new entrance 400 feet wide was opened through the north break-water.

(ii) A shelter arm, 36 feet wide and projecting about 1,200 feet was provided. It was constructed on sloping slice principle and blocks weighing 25 to 30 tons were used.

(iii) A boat basin 1,050 feet long and 420 feet wide was constructed for the port dredger. Eventually it provides protection to small crafts and lighters which were previously exposed to severe swell resulting in many wrecks every year.

These alterations immensely improved the operating conditions in the port. Disturbance in the harbour caused by the swell admitted through the eastern entrance was stopped and the north shelter arm afforded sufficient protection from the north-east monsoon. Ships can now berth quickly alongside the quays for most part of the year; this was impossible before.

### *Present trouble*

Though the conditions in the harbour are satisfactory for the most part of the year, it becomes insecure for ships during cyclonic storms.

It is these storms which generally occur in the months of October and November affect the harbour at present. Direction of the swell caused by high velocity winds, is sometimes 30° out of parallelism with the base line of the harbour. Even the 1,200 feet long shelter arm fails to offer protection from outside wave action. Thus the swell produces ‘range’ in the Madras Harbour.

It will not be out of place if the nature of this phenomenon termed ‘range’ is considered in detail. The tremendous energy of the swell which gathers momentum as it approaches the shallow shore is transmitted to the enclosed mass of water in the harbour through the entrance which produces slow oscillating movement in the harbour. The period of this surging in the harbour is considerably large and

depends upon the period and length of waves generated in the deep sea. The effect of such heaving, on ships, is to produce a similar periodic, bow and sternwise movement which goes on increasing to dangerous limits when moorings are stretched beyond their capacity.

'Range' should not be confused with surface waves in the harbour created by local wind. These have little energy as compared to that of 'range'.

#### *'Range' in the Madras Harbour*

'Range' is the chief disability from which this port suffers. Some years ago, 'range' readings were recorded and the highest observations which did not take into account 'wind lop' (i. e. superimposed, short waves due to wind) were as follows:—

- (i) Across the entrance 2' 9".
- (ii) At south quay wall 3' 6".
- (iii) At north quay wall 2' 9".

'Range' of this magnitude is sufficient to render every berth in the harbour untenable. Usually at sea-ports exposed to wave action the highest readings occur at the entrance and gradually become less owing to:—

- (i) expansion or lengthening of waves once they pass the entrance, and
- (ii) reaction of the still water inside the harbour.

At Madras, however, the highest readings occur at the South Quay the harbour entrance being at the North Quay. The reason for this is that the harbour is bounded by four vertical walls which help to build up wave action. The South Quay berths are, therefore, the first to go out of action (even before the East Quay which is nearer the entrance) and if the storm is severe, the West Quay and North Quay follow owing to reflected wave action from the South Quay.

#### *Model experiments*

Fig. 148.

The following are the terms of reference to be decided by model experiments:—

- (i) To find out the most effective measures to reduce the 'range' in the harbour without any detriment to the navigability of the port.
- (ii) To test the effect of these measures on the proposed wet dock extension schemes.

#### *Model Scales*

It is a general practice to make the models, involving study of wave phenomenon, geometrically-similar. This is due to the fact that the important wave characteristics, such as, length, frequency, amplitude and energy are governed by Froude's criterion.

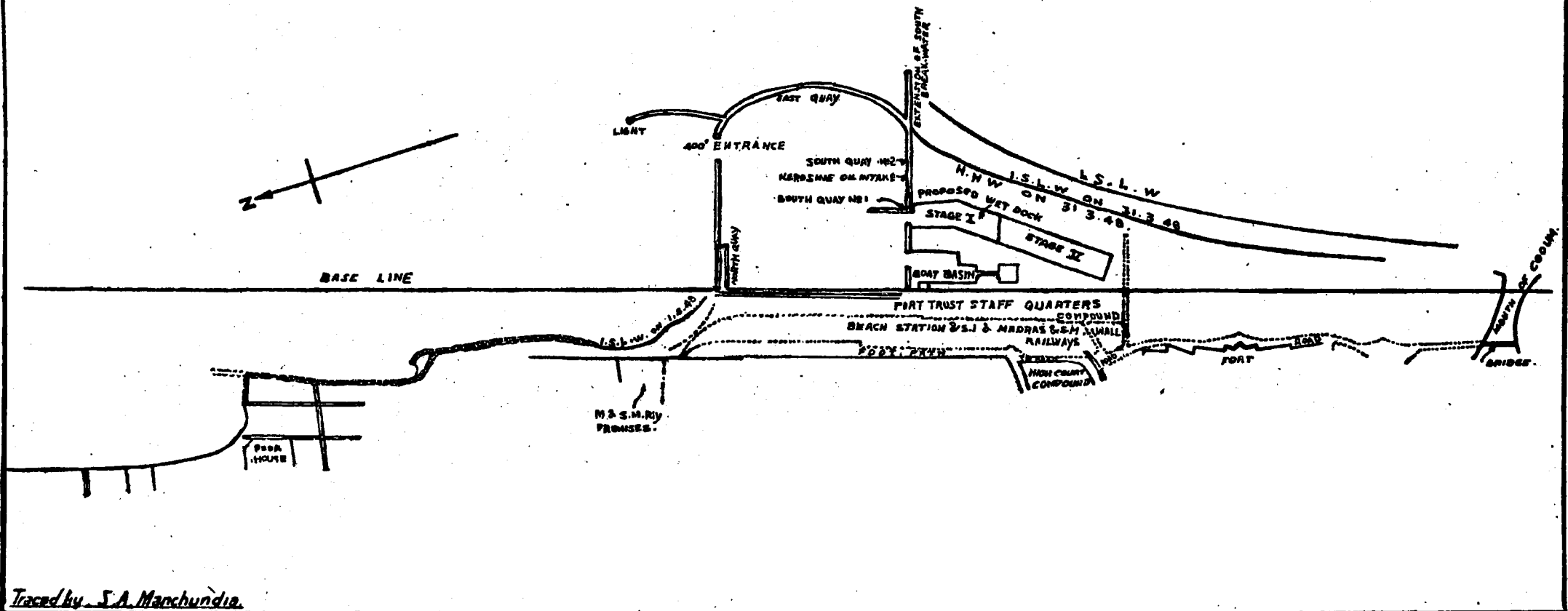
Models with vertical exaggeration have also been tried previously. One such model was constructed in Germany<sup>(1)</sup> for the study of location of breakwater for the Harbour of Heligo-land and was made to a scale 1/300:1/200. Another model was that of Tabla Bay Harbour,<sup>(2)</sup>

(1) Hydraulic Laboratory Practice by Freeman, The Hydraulic Experiment Station at the Navy Yard at Wilhelmshaven, page 387.

(2) 'Range Action in Tabla Bay Harbour', The Dock and Harbour Authority November 1946.

FIG. 143  
MADRAS HARBOUR  
IN 1948  
SCALE: 1" = 2000 FT.

419.



Traced by S.A. Manchundia.



## REDUCTION OF RANGE IN MADRAS HARBOUR

the problem being that of reduction of 'range'. This was to a scale of 1"=100' longitudinal and 1"=12' vertical, the vertical exaggeration being 8.33.

It was not considered practicable to construct a geometrically similar model of Madras Harbour, at this stage, for the following reasons:—

(a) The model would be too big if a large scale is adopted which would be desirable to have an appreciable value of 'range' in the model.

(b) Adoption of small scale to limit the size of model would make range observation in the harbour not only difficult but also unreliable, this being more so, as the model was to be constructed in the open ground.

The following explanation will make the above points more clear:—

The waves generated in the open sea, at the time of cyclonic storms attain some times a length of 300 feet. Waves in the open sea are purely oscillatory and are affected by the bottom when the depth becomes less than half the wave-length. In the case of Madras Harbour, this would be at about 150 ft. contour-line. The model in the ideal case should include a contour line of 150 ft. depth of sea. Purely oscillatory waves would be produced, by a triangular plunger from outside the 150 ft. line. The distance from the shore to the 150 feet contour is about 30,000 feet. To include this portion would necessitate a very small model scale.

The second deciding factor is 'range'. This is the phenomenon which is to be finally observed in the model. It is necessary to adopt such a scale that the height in model equivalent to 1.5 ft. range in the prototype has a significant value and which can be easily and accurately observed.

The limiting factor in this case is the effect of surface tension of water. Sea waves are gravity waves and in order to maintain similitude, the force of surface tension in the model should not be appreciable as compared to that of gravity. These considerations demand as large a model scale as possible.

### *Model of Madras Harbour*

To achieve a compromise between these conflicting factors, a model with vertical exaggeration was adopted.

The model is constructed to a scale of 1/250:1/50. The eastern limit includes 60 feet bed contour. A shore line about 10,000 feet to the north of North Quay and 5,000 feet to the south of South Way is reproduced. All details of the harbour such as breakwaters, wharfs, boat basin are reproduced correctly to scale.

Photo Nos.  
5757  
5702  
5751  
5750

### *Effect of vertical exaggeration.*

Similitude cannot be obtained in every respect due to vertical distortion of the model; but it is expected that by keeping the ratio of

Energy of waves at the entrance to the Harbour

Mass of water inside the Harbour

the same for model and prototype, a satisfactory reproduction of 'range' in the model will be possible.

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It is also expected that experiments in this model will result in useful indications on the following points:—

- (i) The minimum reach of sea which must be included in the model for satisfactory simulation of conditions in the harbour;
- (ii) tentative measures for reducing 'range' in the harbour.

Vertical exaggeration is then proposed to be reduced by stages, by reducing the depth scale and the results compared.

This procedure, it is hoped, will provide very valuable information as regards the utility of vertically distorted models for the study of wave problems.

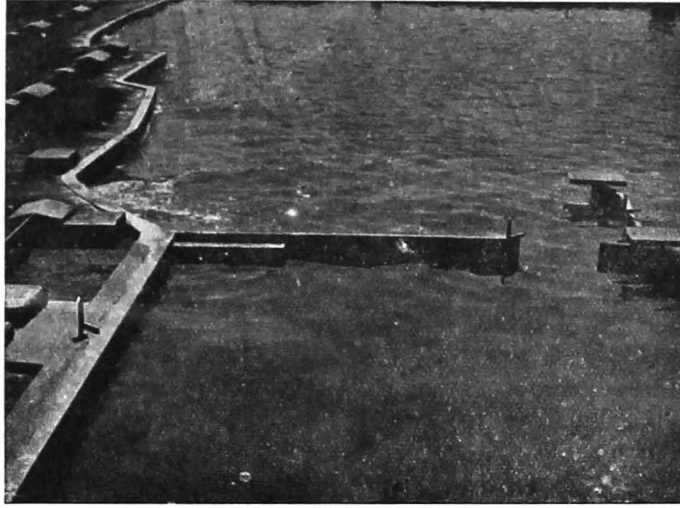


Photo No. 5750—Showing disturbance of the entrance of the harbour due to waves generated in the sea portion.

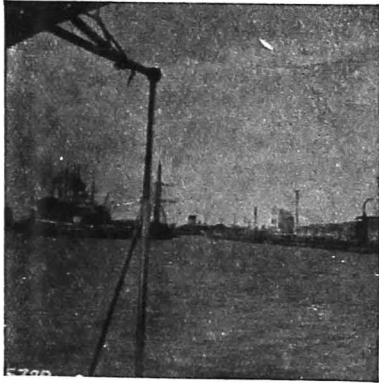


Photo No. 5328—Showing small boat basin near the North Quay.

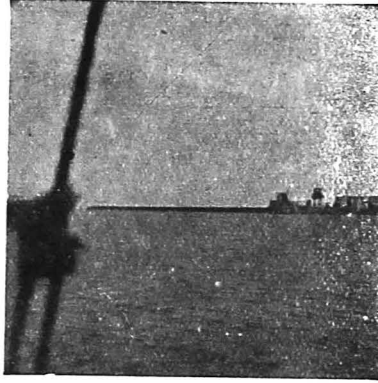


Photo No. 5322—Madras Harbour Entrance sheltering Arm.



Photo No. 5329—Showing accretion between east Quay and projecting arm of South Quay. Concrete blocks as wave breakers in the foreground.

#### 4. DOCKS AND HARBOURS

##### 26. 1/1000:1/20 COCHIN HARBOUR MODEL

###### *Preliminary Experiments*

The sea-bed and the bed of the backwater area were initially laid with sea sand sieved through 30 mesh ( $m=0.19$  mm.) and the tidal curve observed in the prototype on 3rd August 1948 and 4th August 1948 was reproduced and repeated several times to enable the observations of current at various places. When the bed was laid with sea sand it was observed that there was practically no bed movement in the sea portion and very little movement in the North back-waters. The bed in the sea portion was then laid as per observation in the year 1938 with (i) clay, (ii) mixture of clay and sand and (iii) sand. The Mattancherry and Ernakulam channels were laid as per sections observed in the year 1944. The tidal curve observed in the prototype on 3rd August 1948 and 4th August 1948 was then reproduced, mainly to study how the bed movement took place under these conditions as maximum silting occurred during the monsoon period and at the same time it was possible to see how the tidal curve in the monsoons was reproduced. In the above experiments the discharges of the Labyrinths reproduced in the model were as follows:—

South Labyrinth.	Middle Labyrinth.	North Labyrinth i.e. Periyar river Labyrinth.
1,08,008 cs.	12,003 cs.	48,000 cs.

The recorders were placed at Station Nos. 8, 9 and 10 (i.e. old Station No. 9 in the prototype which is at the southern limit of the present model). As no water level data at Station No. 10 in the month of August was available, as the gauge is open to rough sea action, the approximate tidal curve was obtained by using the known water level relation between Station Nos. 9 and 10 in the month of April and May 1948.

Fig. 144.

The reproduction of this curve at Station No. 10 did not give a satisfactory reproduction at Station No. 9 and it was thought that the same was due to the approximate curve obtained for Station No. 10 by correlating the water levels at Station Nos. 9 and 10 in the month of April and May, whereas other conditions in model were for the month of August.

The probable tidal curve at Station No. 10 was then obtained from the correlation of water levels actually observed at Station Nos. 9 and 10 in the model during the flood and ebb tide. The reproduction of this curve at Station No. 10 by the usual operation of sliding weirs gave, to a more or less extent satisfactory reproduction of range at Station No. 9.

The recorders were then removed from Station Nos. 8 and 9 and were placed at Station Nos. 1 and 3. The tidal curve obtained from the model relation, as stated above, was reproduced at Station No. 10. It was observed that water levels at Station Nos. 1 and 3 were generally higher than in the prototype and the flood tide in the Mattancherry channel would go only up to C. S. 3 with no discharge in the southern labyrinth. In the prototype under similar conditions the flood tide goes far beyond the southern limit of the channel. It was thought, initially,

Fig. 145.

that this was due to the incorrect discharge admitted in the southern labyrinth and so the current observations in the Mattancherry channel were done with different discharges. The various discharges tried were 1,08,000 cs., 54,000 cs., 27,000 cs., and no discharge. With the discharge of 1,08,000 cs. it was found that the direction of surface and bed flow during flood tide was northwards (which is in the direction of ebb currents usually). The strength of this northward current during flood tide reduced as the discharge in the southern labyrinth was reduced and when the discharge was nil, there was a feeble flood current which reached as far south as cs. 3 in the Mattancherry channel. It may be noted that exact discharges in the southern backwaters were not observed and hence various discharges had to be tried in the model to see which discharge gave approximately correct tidal conditions.

The above experiments showed that it was necessary to select a tidal curve in fair weather season only when the river backwater discharges are nil and thereby eliminate the unknown factor of discharge. A tidal curve observed in the prototype on 9th April 1948 was, therefore, selected having the additional advantage that the simultaneous water level readings at Station No. 10 were available on this date. The tidal curve on 9th April 1948 was reproduced at Station No. 10 and the curves at Station Nos. 3 and 4 were observed. Curves obtained at both the Stations were higher than required. The recorders were then placed at Station Nos. 1 and 4 and the curve at Station No. 10 was reproduced again. It was found that the curves reproduced at these Stations (Nos. 1 and 4) were higher than the required curves. It was, therefore, decided to find out the curve at Station No. 10 to reproduce the curve at Station No. 1 only, and then see what curve is obtained at Station No. 4. After some trials it was possible to get the tidal curve for Station No. 10 which gave a nearer reproduction of the curve at Station No. 1. These experiments showed that the model curve at the southern end (i.e. Station No. 10) was different from the prototype due to this reach being within the entry reach of the model and hence affected by draw-down effect and varying head of pumping etc. The curve obtained at Station No. 4 was, however, higher than actually required. This showed that the absorption capacity of the southern labyrinth in the model was not enough and thus needed extension.

The labyrinth was, therefore, increased in area by stages viz. from 220 sq. ft. to 484 sq. ft., 792 sq. ft. and 1,042 sq. ft. Satisfactory reproduction of tidal curve was obtained at Station No. 4 when the labyrinth was 1,042 sq. ft.

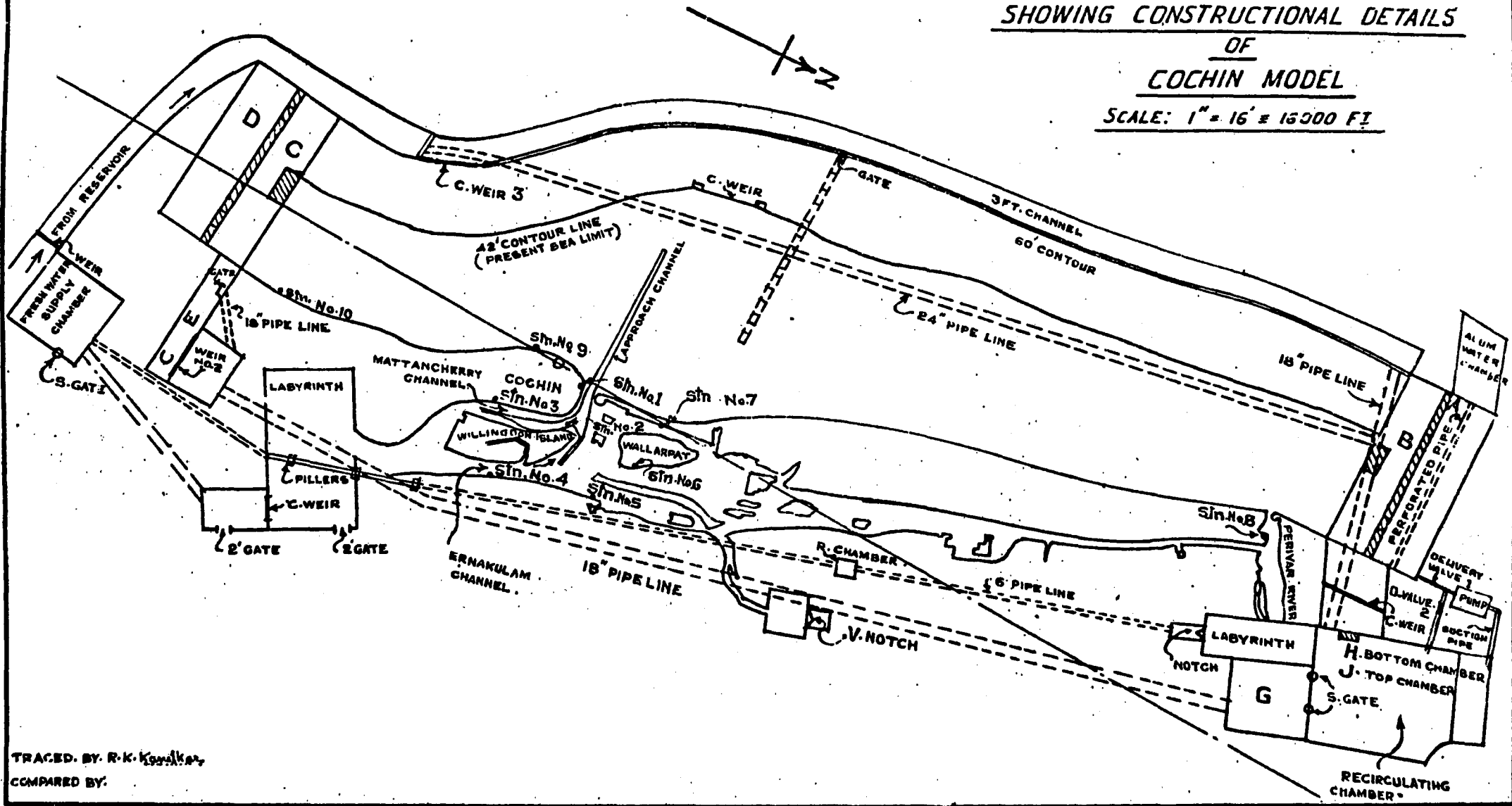
In some experiments injection of the dried Cochin mud, wet Cochin mud and then the local red silt was done to see where the deposition of silt occurs. This injection was done, just north of the approach channel, within the mud bank fringe. Alum water was injected slightly north of the mud injection as circulation of alum water was found inconvenient due to considerable leakage discharge. The effect of alum water was limited and the deposition of the injected material occurred mostly in the North backwaters.

*Experiment No. 1:*—After modifying the southern labyrinth to get the correct tidal curves at Station Nos. 1 and 4, lines of flow and velocity observations were carried out in the Mattancherry and Ernakulum channels during flood tide and ebb tide. The tidal curve of 9th April 1948 was reproduced.

Fig 145

Figure 145 shows the surface lines of flow during ebb in both the channels. The region north of the Willingdon Island and north of

**FIG. 14A**  
**SHOWING CONSTRUCTIONAL DETAILS**  
**OF**  
**COCHIN MODEL**  
**SCALE: 1" = 16' = 16000 FT**



427

TRACED BY: R. K. KONDKAR  
COMPARED BY:

MODEL OF MADRAS HARBOUR

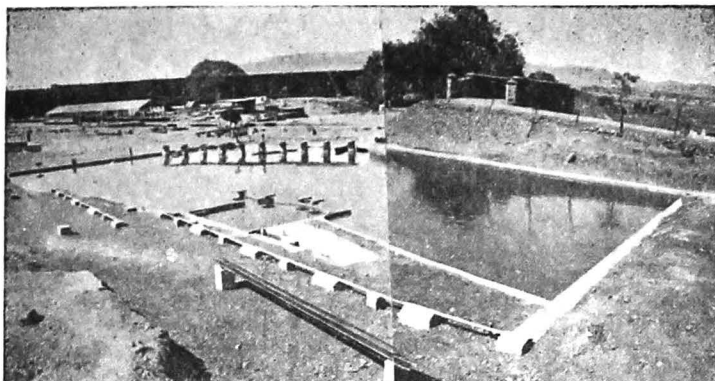
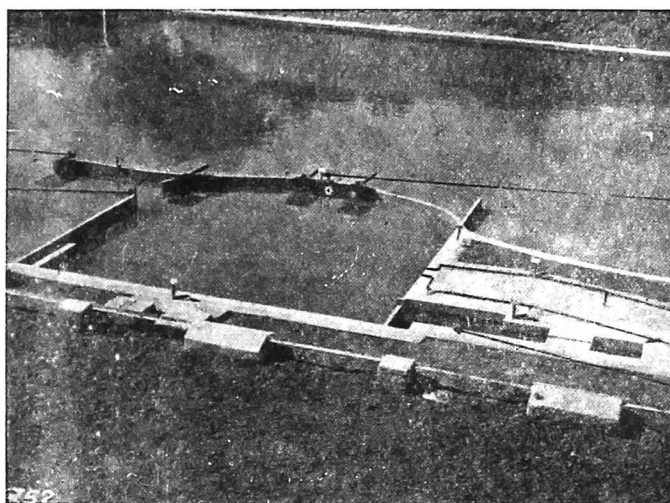


Photo No. 5757 - A general view of the Madras Harbour Model Scale 1 250:1,50.



North Shelter  
arm.  
New entrance

Old. eastern  
entrance.

Proposed ex-  
tension of  
wet dock.

Boat basin.

Photo No. 5752 - Showing details of the harbour and proposed wet dock extension.

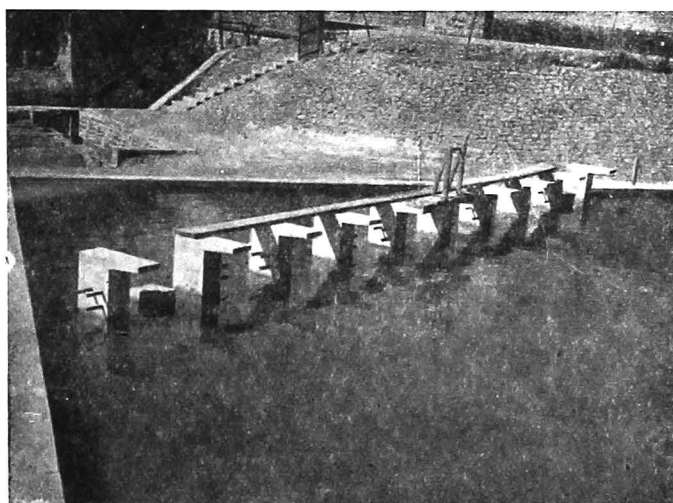
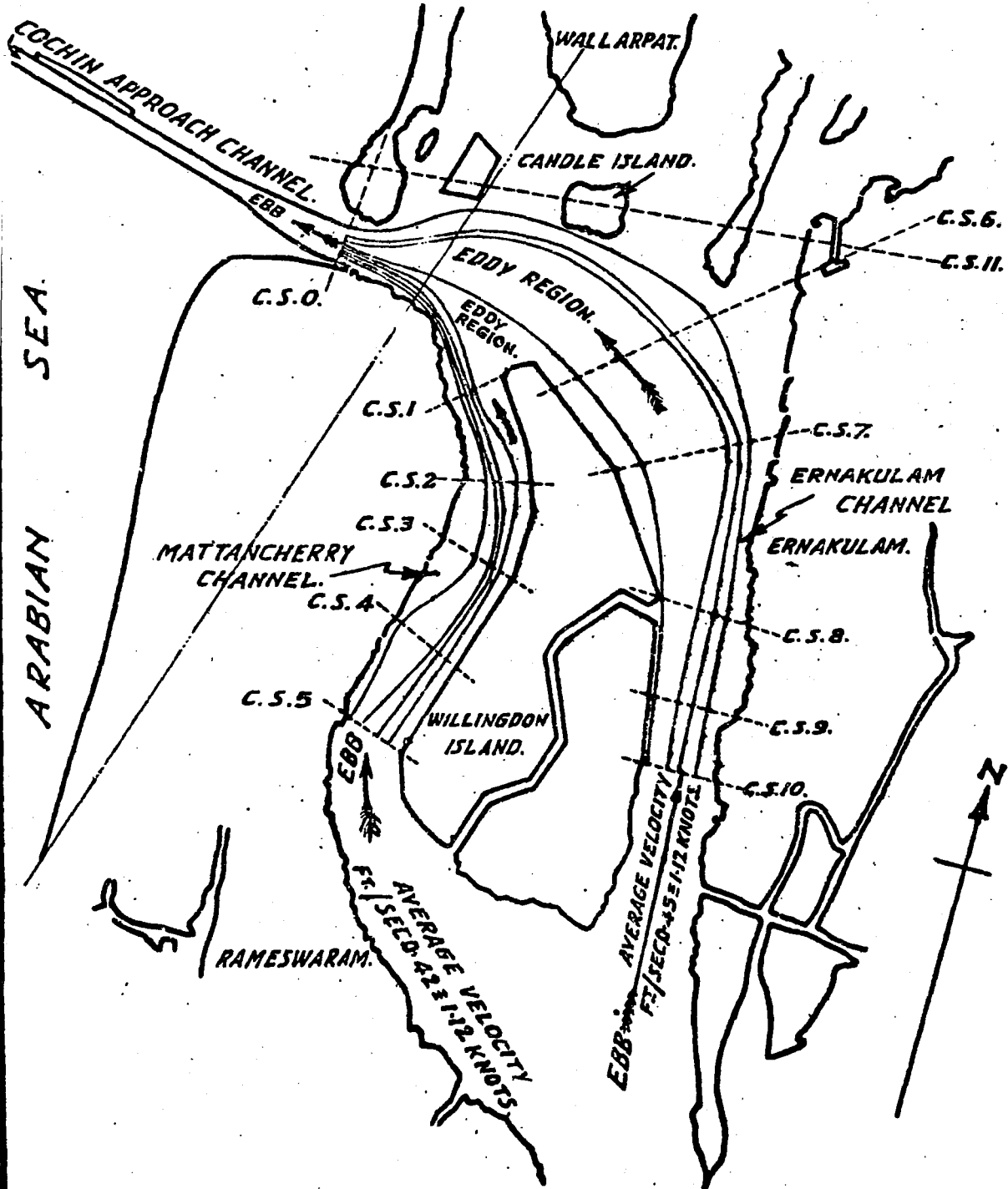


Photo No. 5751—Showing concrete foundation blocks for wave generating machine.

FIG. 145  
COCHIN MODEL  
EXPT. No. 1  
TIDAL CURVE OF 9-4-48  
SHOWING LINES OF FLOW  
IN MATTANCHERRY & ERNAKULAM CHANNELS  
DURING EBB TIDE.  
(FROM BEGINNING OF EBB TO MIDDLE OF EBB)  
SCALE: 1" = 4000 FT

REFERENCE:—  
SURFACE CURRENTS. —————



T. BY. - SAPHAL.



COCHIN HARBOUR MODEL

C. S. 7 was eddy region. The average surface velocity in the Mattancherry channel during ebb tide was equivalent to 1.1 knots in prototype, the floats being released just at the beginning of the ebb tide. The average surface velocity in the Ernakulam channel was equivalent to 1.2 knots.

The bed velocities were observed with plastic balls (specific gravity 1.05). In the Mattancherry channel the bed velocity was equivalent to 0.5 knots and in the Ernakulam channel it was equivalent to 0.6 knots during ebb tide.

Figure 146 shows lines of flow during flood tide. Surface velocity in Mattancherry channel was equivalent to 1.2 knots and in the Ernakulam channel it was equivalent to 1.4 knots. The bed velocity, with the plastic balls, was found to be equivalent to 0.66 knots in the Mattancherry channel and 1.1 knots in the Ernakulam channel.

*Experiment No. 2.*—In this Experiment the monsoon conditions were reproduced. The tidal curve observed in the prototype on 3rd and 4th of August 1948 was reproduced. The discharge of the Periyar river was equivalent to 40,000 cs., the middle labyrinth discharge was equivalent to 12,000 cs. and the southern labyrinth was equivalent to 70,000 cs. The tide was repeated 20 times; red-silt ( $m=0.088$  mm.) was injected at 1.75 cft./hr. The rate of injection was based on the observations on 19th August 1940 in the prototype during flood tide (see Appendix I) and the silt was injected during flood tide only. As no alum was used in this experiment, the size of particles of silt injected was so chosen as to be approximately 3.32 times<sup>(1)</sup> the size of particle in prototype (0.025 mm. on the western edge of the Mattancherry channel).

Fig. 146

Appx. I.

The relation (3.32) of the size of particle injected and the size of particle in the prototype was obtained as follows:—

Let  $S$  and  $s$  be the velocities of fall of mud particles in nature and in the model respectively. Then,

$$s = 11.1 S^{(2)}$$

But  $s \propto r^2$  as shown by L. Bonnet and J. Lamoen<sup>(1)</sup>

$$\therefore r^2 = 11.1 R^2$$

$\therefore r = 3.32 R$  showing that the linear dimension of model mud particles should be 3.32 times that of natural particles.

After the completion of 20 tides the experiment was stopped and the silt deposited in the Mattancherry channel was collected, without allowing the bed to be disturbed, and the silt so collected was dried and weighed. The silt collected weighed 12 lbs. i.e.  $\frac{1}{2}$  cft. (weight of 1 cft. of dry red silt being 71 lbs.).

Therefore, silting in model in 20 tides (=10 days)

$$= \frac{12}{71} \times \frac{1000 \times 1000 \times 20}{27} \text{ cub. yds.}$$

$$= 1,25,000 \text{ cub. yds.}$$

$\therefore$  silting in 1 month = 3,75,000 cub. yds.

After some trials the rate of injection of 1.4 cft./hr. was decided which gave  $\frac{1}{4}$  lbs. of silting (weight of dry silt) equivalent to

Appx. II

(1) Model experiments on the Belgian Ports of the North sea by L. Bonnet and J. Lamoen.

(2) Annual Report 1947 p. 109.

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3,18,000 cub. yds. in the Mattancherry channel. This compares very well with the silting in August 1935 and August 1936 of 3,10,100 cub. yds. and 2,60,600 cub. yds. respectively.

The rate of 1.4 cft./hr. was, therefore, adopted in further experiments under monsoon conditions.

Fig. 147

Figure 147 shows the surface lines of flow in the Mattancherry and Ernakulam channels during ebb. The average velocity of the surface current in the Mattancherry channel was equivalent to 1.36 knots and that in the Ernakulam channel was 2.62 knots.

The bed velocity in the Mattancherry channel was equivalent to 0.46 knots and in the Ernakulam channel it was equivalent to 1.06 knots during ebb tide.

Fig. 148

Figure 148 shows the surface lines of flow on the flood tide. The velocity of current in the Mattancherry channel was equivalent to 0.96 knots and it was equivalent to 1.41 knots in the Ernakulam channel.

The velocity of the bed current was equivalent to 0.8 knots and 0.44 knots in the Mattancherry and Ernakulam channel respectively.

Table 1

Table 1 attached shows at a glance the velocities in both the channels under different conditions. The bed velocity in the Mattancherry channel during flood tide, under monsoon conditions, is more than that of the Ernakulam channel, whereas the ebb velocity is considerably less in the Mattancherry channel than the Ernakulam channel.

Experiments to test the various proposals are in progress.

TABLE I

*Showing the equivalent Velocities in Mattancherry and Ernakulam channels.*

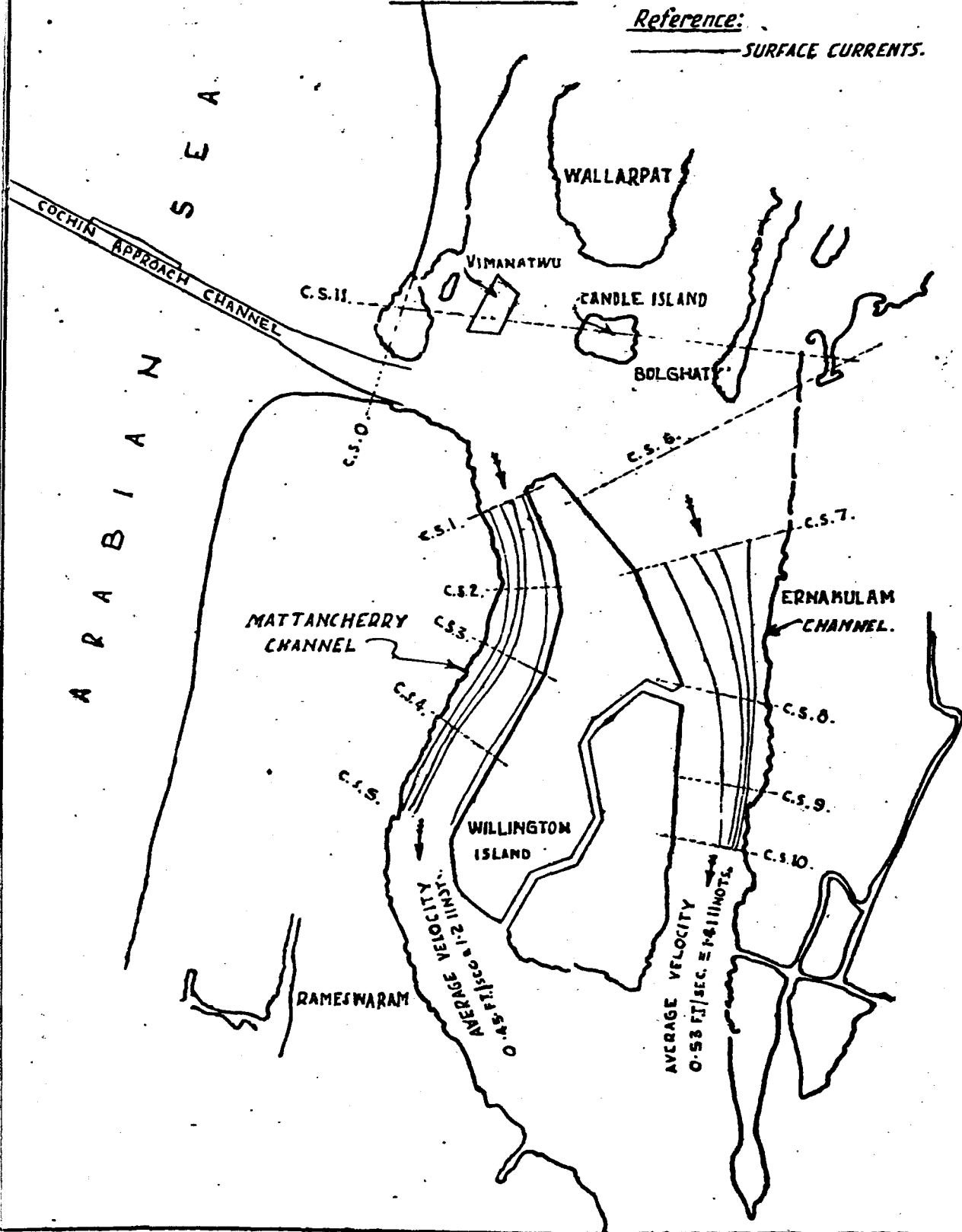
Channel.	Expt. No. 1 (tidal curve 9/4/48).				Expt. No. 2 (tidal curve 3rd and 4th August '48.)			
	Surface velocity.		Bed velocity.		Surface velocity.		Bed velocity.	
	Flood (knots).	Ebb (knots).	Flood (knots).	Ebb (knots).	Flood (knots).	Ebb (knots).	Flood (knots).	Ebb (knots).
Mattancherry channel ...	1.2	1.1	0.66	0.5	0.96	1.36	0.8	0.46
Ernakulam channel ...	1.4	1.2	1.1	0.6	1.4	2.62	0.44	1.06

FIG. 146  
COCHIN MODEL  
EXPERIMENT I.  
TIDAL CURVE OF 9-4-1948  
SHOWING LINES OF FLOW IN MATTANCHERRY  
AND  
ERNAKULAM CHANNELS DURING FLOOD TIDE  
(FROM BEGINNING TO MIDDLE OF FLOOD TIDE.)

SCALE: 1" = 4000'

Reference:

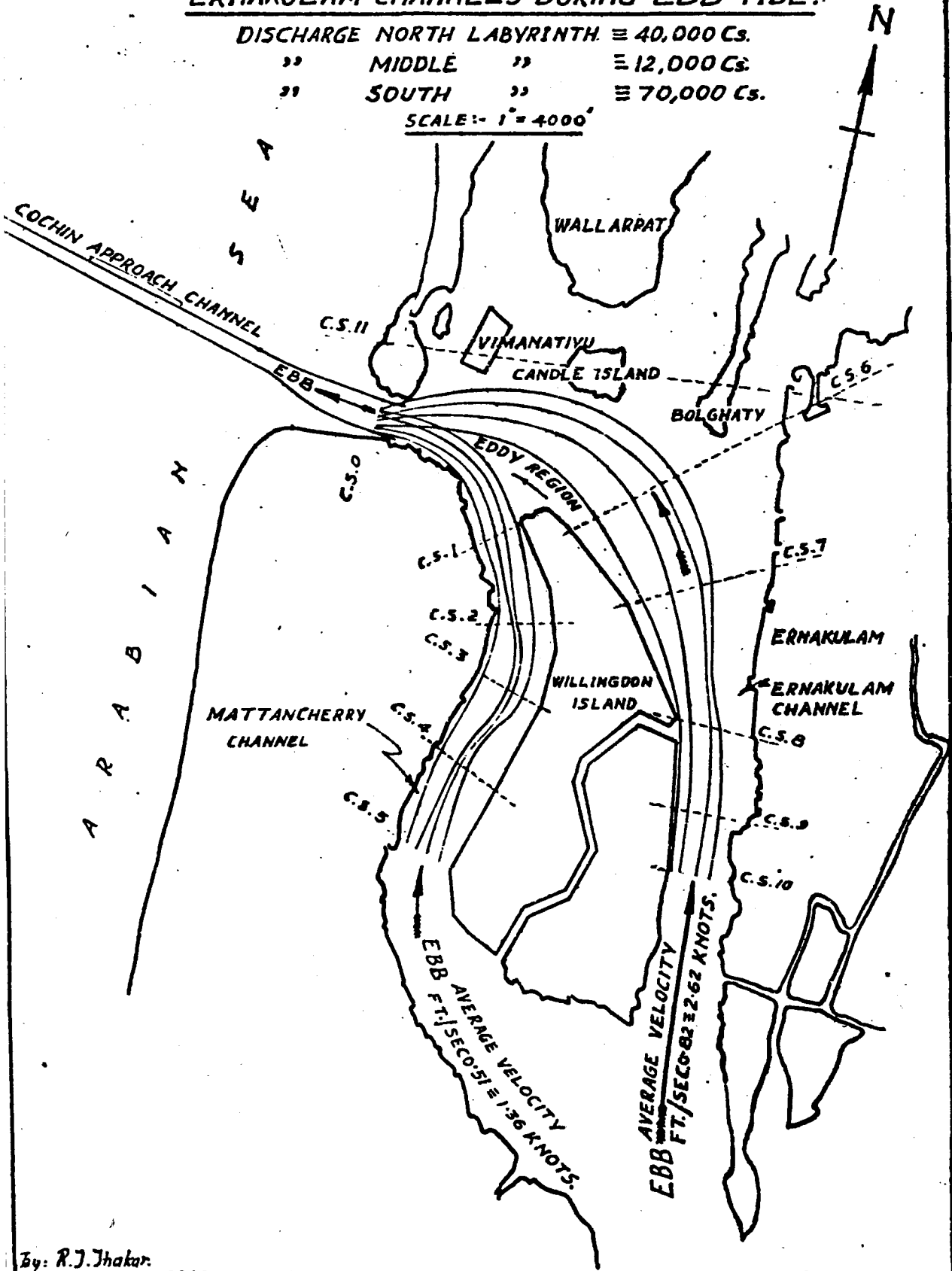
— SURFACE CURRENTS.



**FIG. 147**  
**COCHIN MODEL**  
**EXPT. 2**  
**TIDAL CURVE OF 3 & 4-8-48**  
**SHOWING LINES OF FLOW IN MATTANCHERRY**  
**AND**  
**ERNAKULAM CHANNELS DURING EBB TIDE.**

DISCHARGE NORTH LABYRINTH  $\cong$  40,000 Cs.  
" MIDDLE "  $\cong$  12,000 Cs.  
" SOUTH "  $\cong$  70,000 Cs.

SCALE: 1" = 4000'



By: R.J. Thakar.

FIG. 148

COCHIN MODEL

EXPT. 2

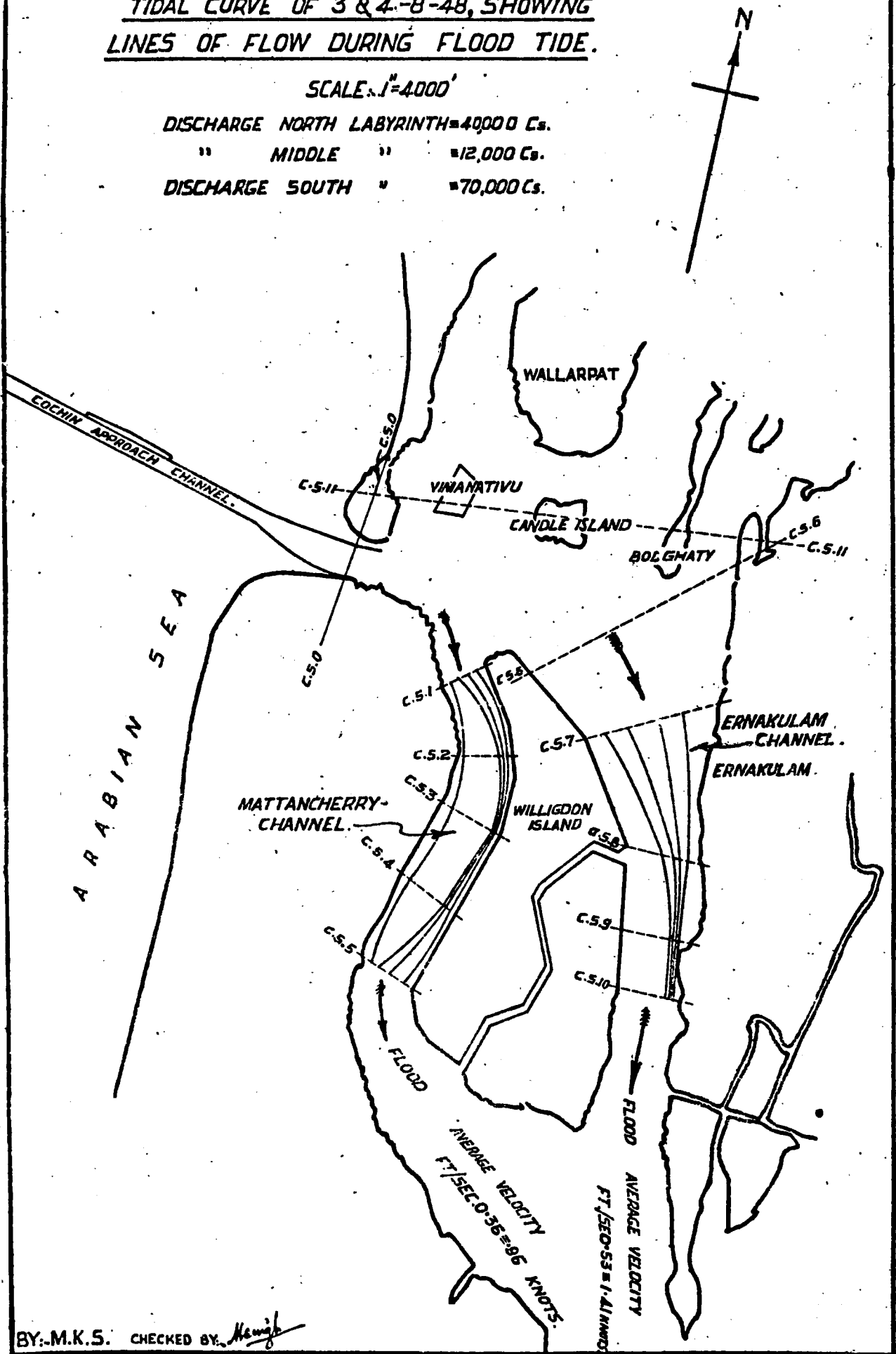
TIDAL CURVE OF 3 & 4-8-48, SHOWING  
LINES OF FLOW DURING FLOOD TIDE.

SCALE: 1"=4000'

DISCHARGE NORTH LABYRINTH=40,000 Cs.

" MIDDLE " =12,000 Cs.

DISCHARGE SOUTH " =70,000 Cs.



BY: M.K.S. CHECKED BY: *Heugh*

COCHIN HARBOUR MODEL

APPENDIX I.

Actual tidal and silt observations at Cochin on 19/4/48 show that during flood tide 18,000 cub. yds. entered from the sea into the gut when the total discharge was 2,627 m. cft.

Therefore, the silt charge was

$$\frac{486000 \times 75.5 \times 1000}{2627 \times 106 \times 62.4}$$

$$=0.224 \text{ parts per 1000}$$

Maximum discharge entering the model in the gut (i.e. East of Station No. 1) is 5.2 cs. and the average is 2.6 cs.

Assuming the same silt charge in model as in the prototype the rate of silt injection will be

$$\frac{2.6 \times 3600 \times 62.4 \times 0.224}{1000 \times 75.5}$$

$$=1.73 \text{ cft/hr.}$$

$$=1.75 \text{ (say).}$$

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APPENDIX II.

(PROTOTYPE DATA OF SILTING IN THE MATTANCHERRY CHANNEL IN AUGUST 1935 AND 1936)

Year.	Berth.	Average depth of silting (ft.)	Total quantity of silting.
1935	4	0.7	$\frac{1100 \times 1000 \times 0.7}{27} = 28,600$ cub. yds.
	5	0.6	$\frac{1100 \times 1000 \times 0.6}{27} = 24,500$ ..
	6	1.4	$\frac{1100 \times 1000 \times 1.4}{27} = 57,000$ ..
	7	1.2	$\frac{800 \times 1000 \times 1.2}{27} = 35,600$ ..
	8	1.2	$\frac{1000 \times 1200 \times 1.2}{27} = 53,400$ ..
	9	2.5	$\frac{1000 \times 1200 \times 2.5}{27} = 1,11,009$ ..
			Total ... 3,10,100 ..
1936	4	0.9	$\frac{1100 \times 1000 \times 0.9}{27} = 36,800$ ..
	5	0.9	$\frac{1100 \times 1000 \times 0.9}{27} = 36,800$ ..
	6	1.5	$\frac{1100 \times 1000 \times 1.5}{27} = 61,000$ ..
	7	1.7	$\frac{800 \times 1000 \times 1.7}{27} = 50,500$ ..
	8	1.0	$\frac{1000 \times 1200 \times 1.0}{27} = 44,500$ ..
	9	0.7	$\frac{1000 \times 1200 \times 0.7}{27} = 31,000$ ..
			Total ... 2,60,600 ..

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*Errata*

Against "Guide Bank; bund and wall" on page 3 *read* 135 in place of 231 under 1949.

Against "Pier(s)" on page 5 *read* 135 in place of 133 under 1949.

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