

Government of India

CENTRAL IRRIGATION AND HYDRODYNAMIC RESEARCH STATION, POONA

Director .

C. C. INGLIS, C.I.E., B.A., B.A.I., M.Inst.C.E., I.S.E.

RESEARCH PUBLICATION No. 1
ANNUAL REPORT OF WORK DONE
DURING 1937-38

BY

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A. R. THOMAS, B.Sc. (Eng.), I.S.E.,
Executive Engineer,

Rao Sahib D. V. JOGLEKAR,
Assistant Engineer,

AND

STAFF AT KHADAKVASLA



DELHI: MANAGER OF PUBLICATIONS
1939

ERRATA.

- Page 4, line 34 : “ problems ” *for* “ probloms ”.
Page 6, line 20 : *omit* comma after Now.
Page 6, 8th line from bottom : *add* comma after 5.
Page 24, 4th line from bottom : “ close ” *not* “ closest ”.
Page 55 : *omit* “ thus ” from beginning of 13th line from bottom.
Page 63, 5 lines from bottom : *omit* “ which ”.

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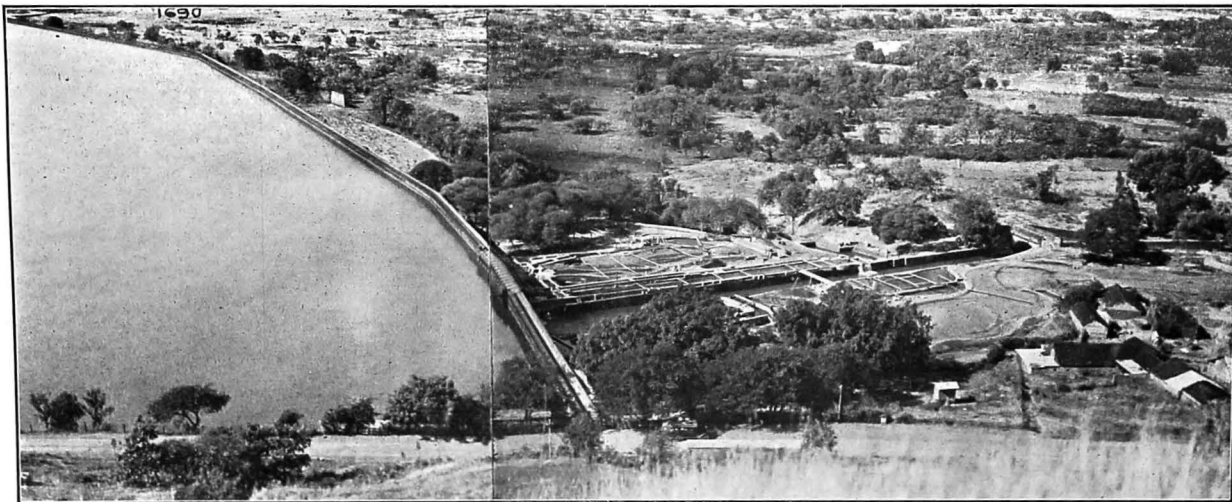
* Agents for publications on Aviation only.

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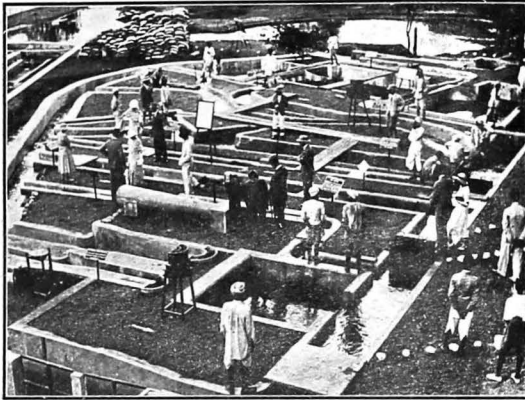
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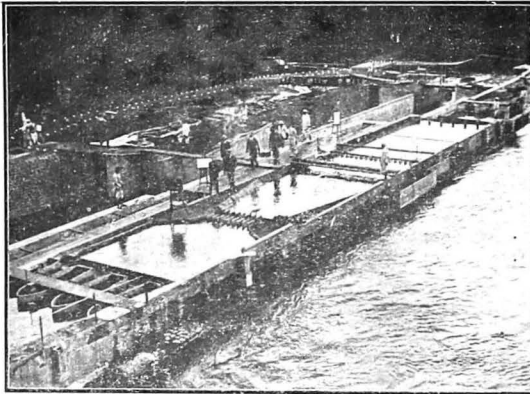
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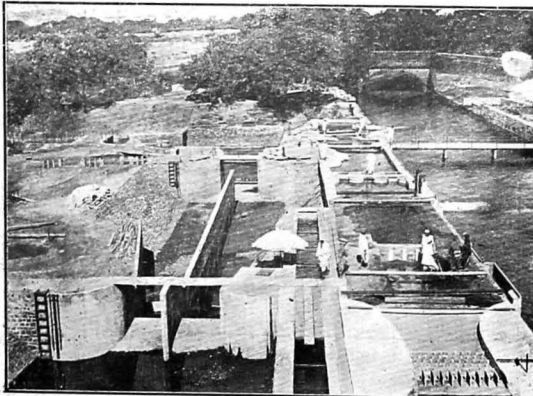
1690. A composite photo taken from the top of the hill above Khadakvasla Dam and shows the Lake Fife Research Station just downstream.



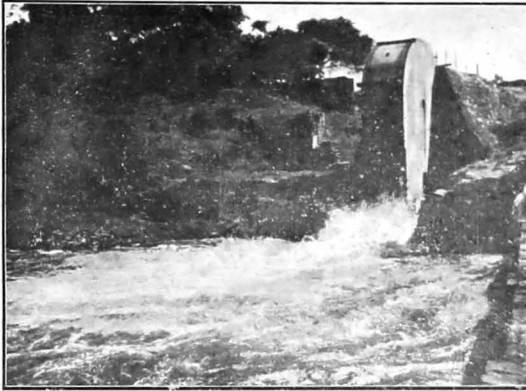
1203. Showing the demonstration models on the Right Bank of the Mutha Right Bank Canal.



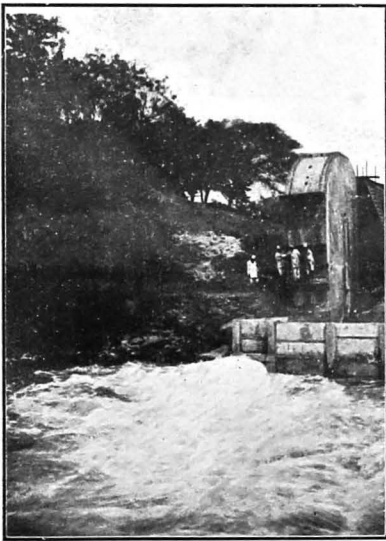
1205. Showing a canal fall model in the 18 ft. Left Bank channel.



1232. A small scale fall experiment in the 18 ft. Left Bank channel and a large scale twin fall model discharging 240 cs. in an adjoining channel.

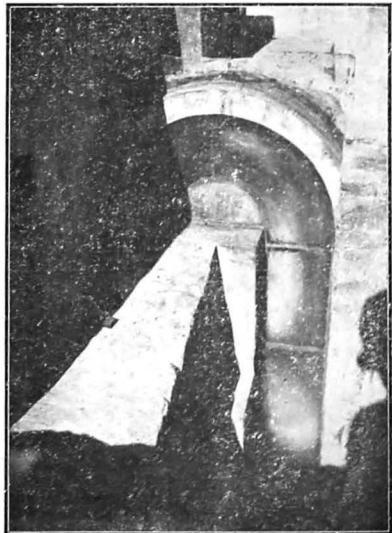


328. Full scale siphon spillway model discharging 520 cs.

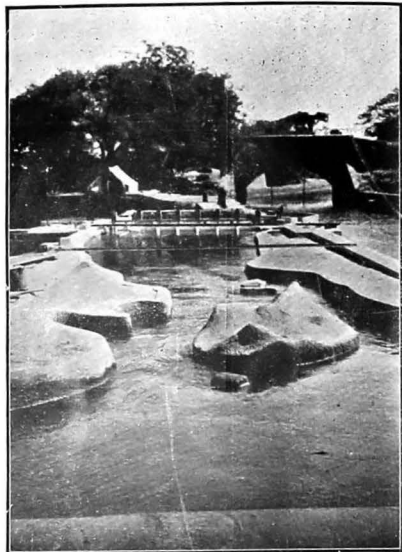


334. The same siphon in photo 328 running with the outlet flared at 1 ft 4 in. downstream.

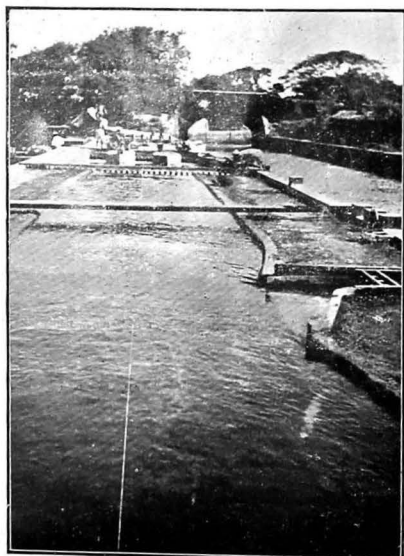
850. Shows $\frac{1}{4}$ scale replica of the 520 cusecs siphon with glass sides.



842. Showing the Sukkur Barrage model looking downstream with Bukkur island standing out in the middle foreground.



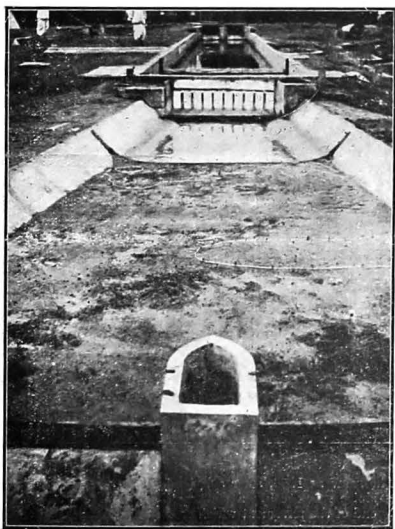
521. Showing a 'part' width model of the Sukkur Barrage showing deep scour at the vertical nose of the Divide wall.



841. A model of the Jsmrao Canal at mile 17 with warped, converging, silt-excluding head for Dim and Khadro minors.



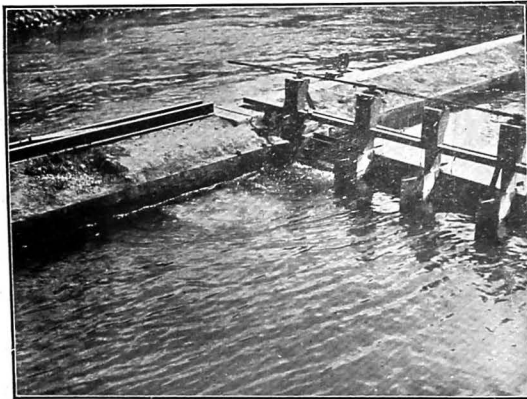
662. Showing heavy scour which would have occurred downstream of the Tando Mastikhan Fall (Rohri Canal, Sind) with 1 in 10 divergences without remodelling.



826. The same design in photo 662 after baffle and deflector had been added.



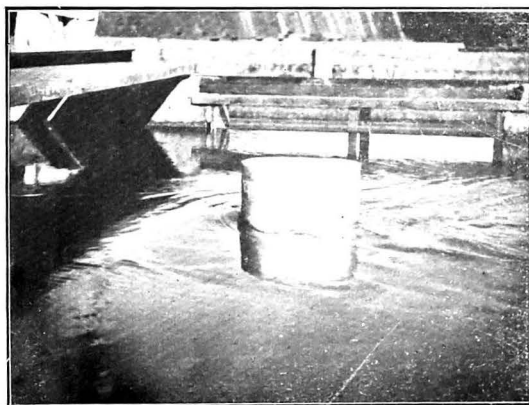
1256. A part-width model of Sukkur Barrage before downstream protection was added.



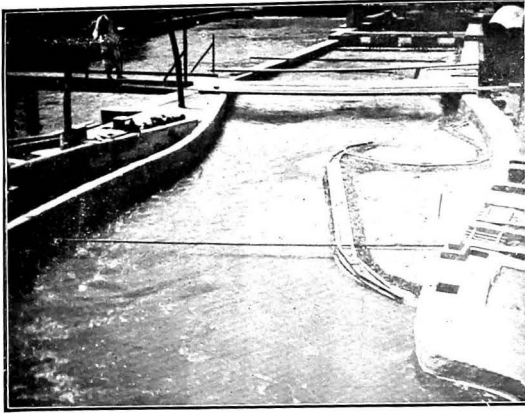
1263. The same as 1256, after a baffle had been added downstream of the sluice gate causing a standing wave to form and a deflector at the end of the pavement which created a bed roller.



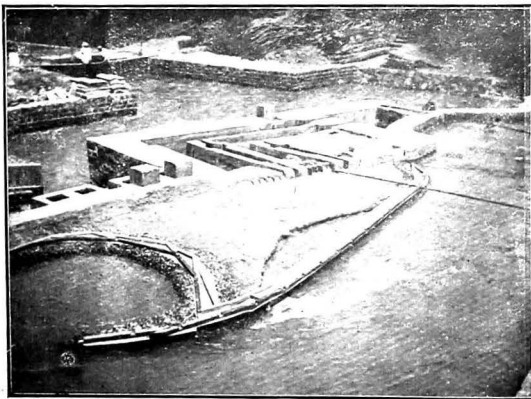
1167. Showing a falling apron experiment using different coloured pebbles to trace how "launching" takes place.



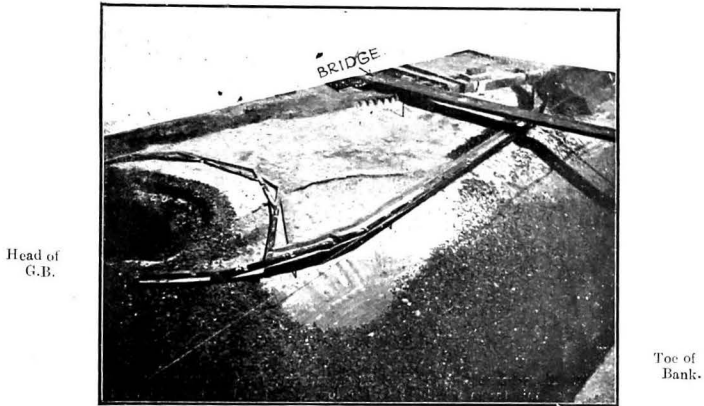
1012. Showing profile of flow in a pier experiment.



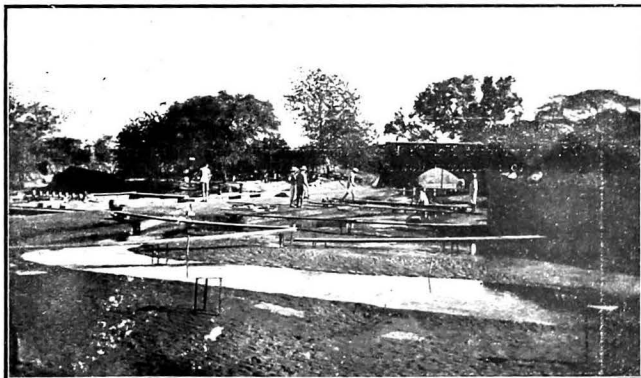
1183. A large scale model of the Right Guide Bank of the Hardinge Bridge, looking upstream, showing condition of flow before slip of 26th September 1933 occurred.



1187. Same as 1183, showing how the slip occurred.



1192. Experiments with Falling Aprons. Model of Guide Bank at Hardinge Bridge showing formation of breach (looking downstream towards bridge).



943. A portion of the 550 ft. long model of the Ganges River above the Hardinge Bridge.

DISTRIBUTARY 5 MODEL OF THE EASTERN NARA RIVER AT MAKHI.

Makhi Regulator

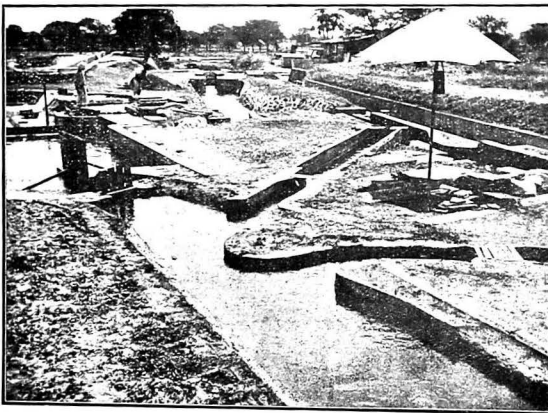


Khipro Canal

Mithrao Canal

838. Experiment No. 2: (Looking downstream.) A spur projecting to midstream was built at the downstream end of the Mithrao to create curved flow to exclude bed silt.

Makhi Regulator

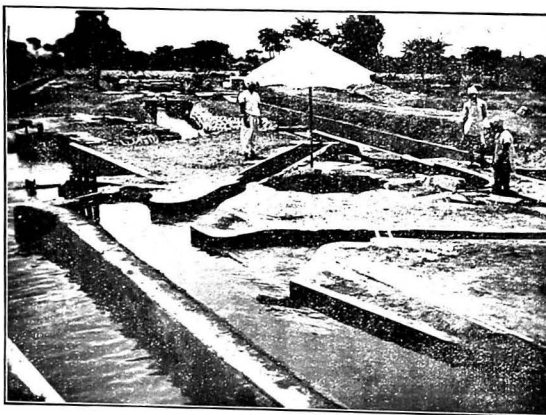


Khipro Canal

Mithrao Canal

855. Experiment No. 3: 1 in 5 convergence upstream and a spur projecting to midstream downstream of the Mithrao. (Looking downstream.)

Makhi Regulator



Khipro Canal

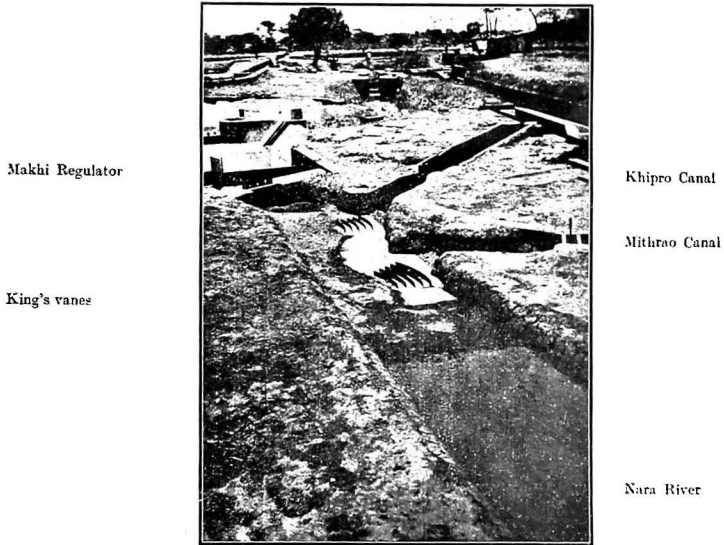
Mithrao Canal

Long raised sill with 1 in 5 convergence upstream.

865. Experiment 4: 1 in 5 convergence upstream of the Mithrao and a long raised sill in front of the Mithrao and Khipro Canals. (Looking downstream.)

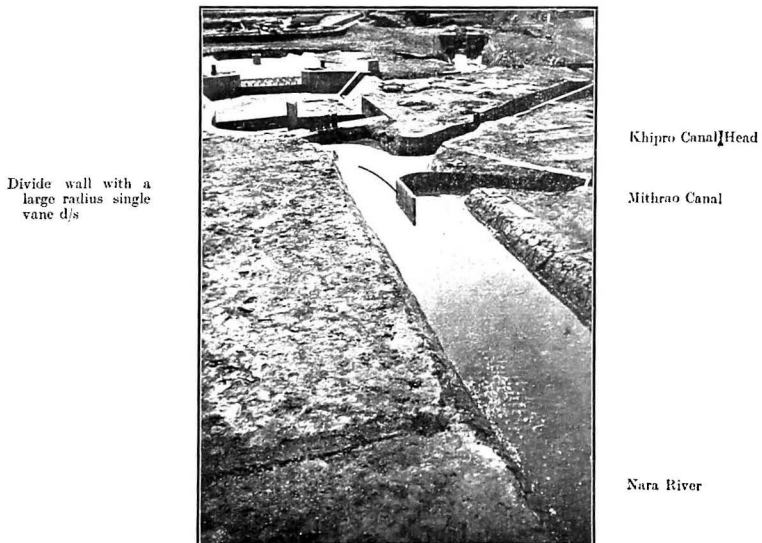
MODEL OF THE EASTERN NARA AT MAKHI.

EXPERIMENT 7 (a) : KING'S VANES OPPOSITE MITHRAO AND KHIPRO CANALS.



890. Looking downstream.

EXPERIMENT 8(a) : DIVIDE WALL OPPOSITE MITHRAO AND A LARGE RADIUS SINGLE VANE IN CONTINUATION OF THE DIVIDE WALL AT THE DOWNSTREAM END TO EXCLUDE BED SILT FROM KHIPRO CANAL.



888. Looking down-stream.



1409. Showing the Eastern Nara on the left and the approach channel to the Mithrao on the right, shortly after construction. (Looking downstream.)

MODEL OF MULA RIVER AND CAUSEWAY AT RAHURI (DISTRICT AHMEDNAGAR).



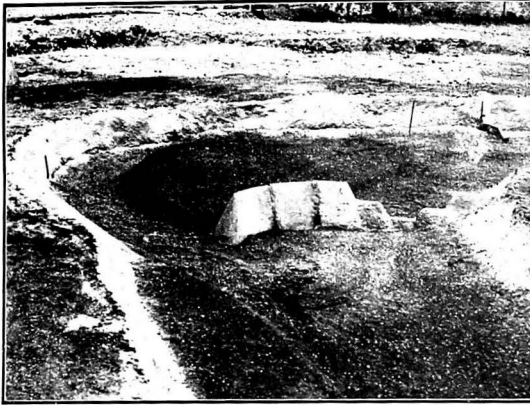
1534. Looking downstream (1936 conditions) with maximum flood of the 2nd season, showing the high velocity jet impinging on left bank (marked I in photo) upstream of landing stage.



← Ferry
Line.

1539. Experiment 17 (1936 conditions). Looking downstream. Showing heavy scour of left bank upstream of ferry after 3 flood seasons.

MODEL OF MULA RIVER AND CAUSEWAY AT RAHURI (DIST. AHMEDNAGAR) HORIZONTAL SCALE RATIO = $\frac{1}{80}$;
 VERTICAL SCALE RATIO = $\frac{1}{13.35}$.

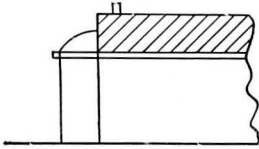


1538. Looking downstream, showing the proposed Burma type spur on the convex bank with two trapezoidal notches in the stem of the spur.

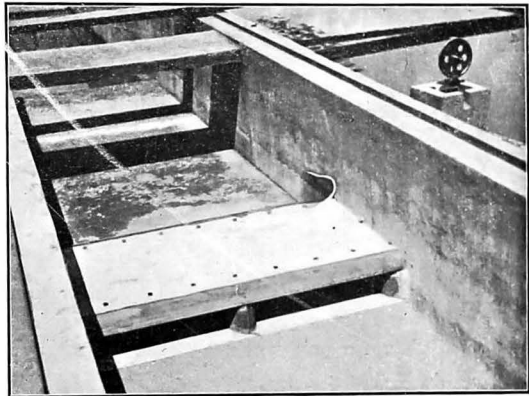
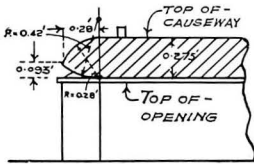


1517. Looking downstream. The spur in action: Discharge in model=6.28 cusecs (\cong 32,500 cs., with 4 ft. depth over causeway).

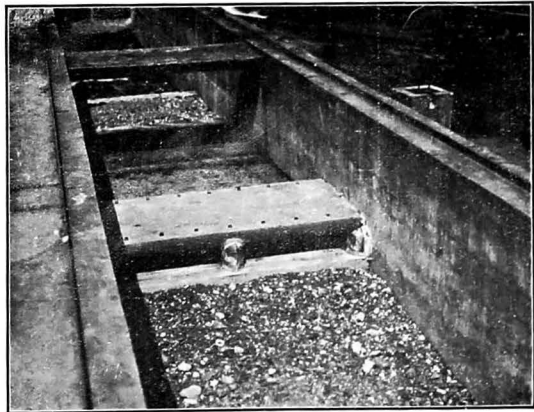
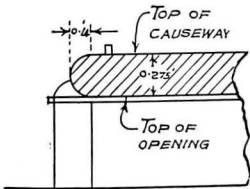
1/10 SCALE GEOMETRICALLY SIMILAR PART-WIDTH MODEL OF MULA RIVER CAUSEWAY.



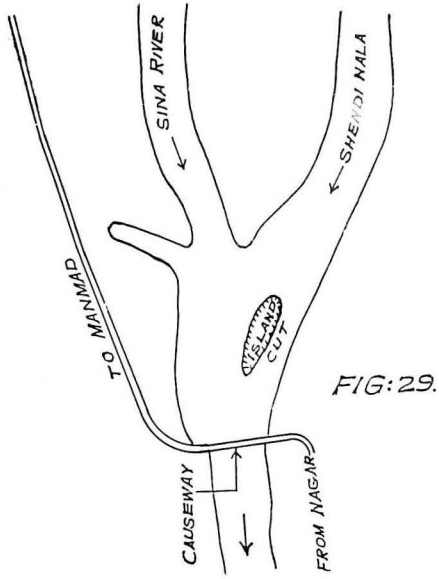
1390. Present design with faces of slabs at right angles to flow. Looking upstream.



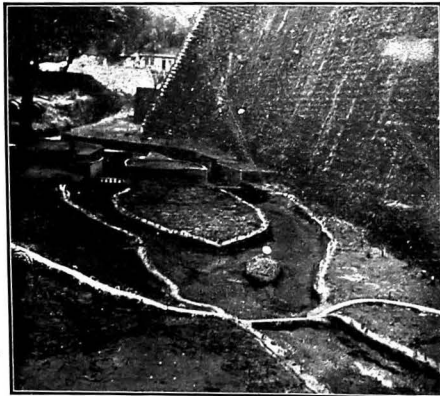
1389. With curved bellmouth added to slabs upstream and downstream. Radius of bottom arc = 1.5 x depth of slab and of top arc = depth of slab. Looking upstream.



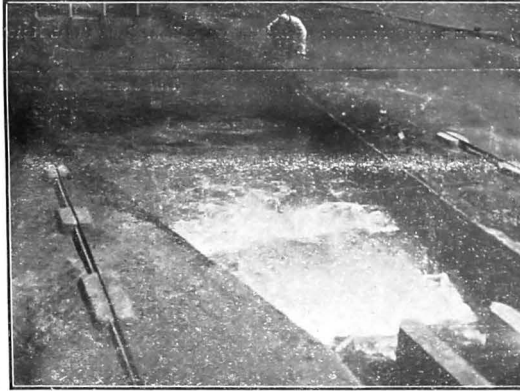
1394. With semi-circular bellmouths upstream and downstream. Looking upstream.



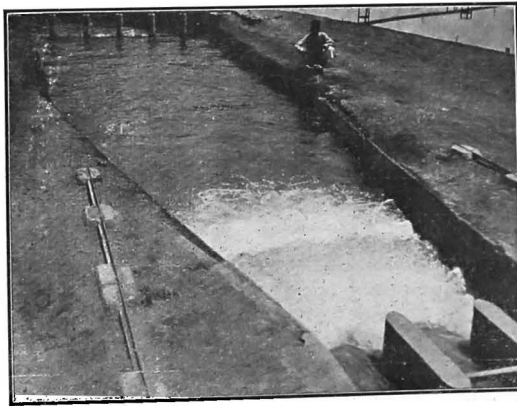
1524.



1/5TH SCALE MODEL OF 700 CUSECS (6.63 FT.) PROPORTIONAL STANDING WAVE FLUME METER FALL WITH BAFFLE
CISTERN AND DEFLECTORS.



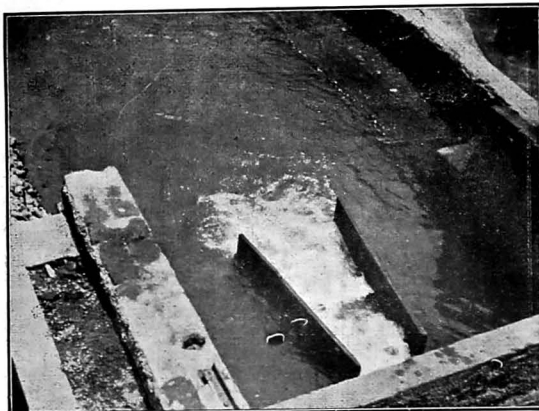
1685. Looking downstream, showing the condition of flow with F. S. $Q_1=12.5$ cusecs ($\cong 700$ cusecs), $H=1.33'$ ($\cong 6.63'$) and $d_1=1.02$ ft. ($\cong 5.1$ ft.); with baffle $\cong 1.96$ ft. high without sloping ends. The height of waves near banks= 0.10 ft. ($\cong 0.50$ ft.)—Experiment 2.



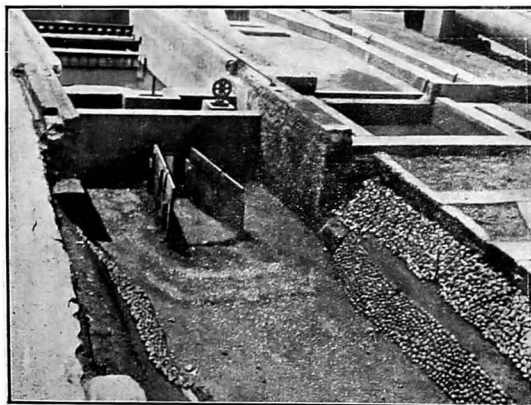
1686. Same as 1685, except that baffle height increased to $\cong 2.6$ ft. Standing Wave downstream of the baffle was clear and height of waves near banks= 0.07 ft. ($\cong 0.35$ ft.)

GEOMETRICALLY SIMILAR MODEL.

(1/9TH SCALE) OF THE BELLMOUTH SLUICE OF THE GODAVARI RIGHT BANK CANAL HEAD REGULATOR.



1548. Looking downstream. $Q \cong 384$ cusecs. Upstream W. L. $\cong 530.10$ and downstream W. L. $\cong 528.45$. Conditions of flow with baffle, cistern and 1 in 10 divergence.



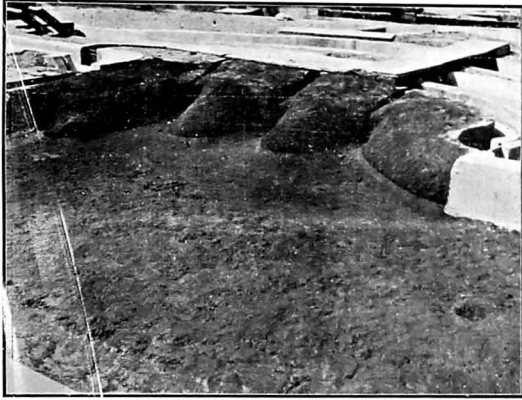
Side slope
pitching of
stones.

Side slope
pitching of
stones.

1550. Looking upstream: Sides diverge at 1 in 10; a baffle $\cong 13\frac{1}{2}$ " high at end of sluice with a cistern $\cong 3.3$ ft. deep and $\cong 11$ ft. long downstream. Showing bed scour after 12 hours' flow with $Q \cong 384$ cusecs. Upstream W. L. $\cong 530.1$ and downstream W. L. $\cong 528.45$. Scour was negligible and there was no scour near banks.

A MODEL OF THE GANGES AND THREE OF ITS TRIBUTARIES.

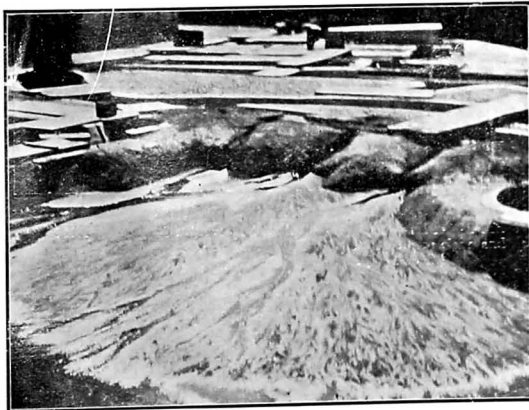
(SCALE RATIO=1 FOOT \equiv 10 MILES.)



1588. Model as laid.

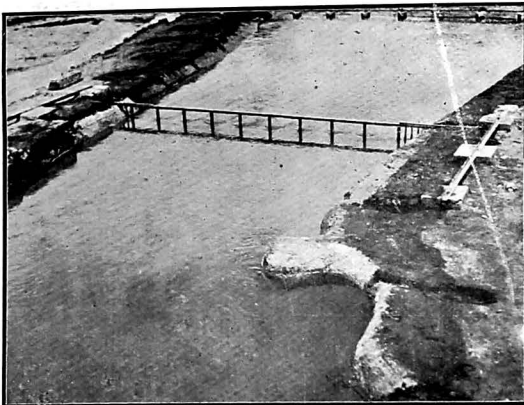


1592. Showing conditions after 31 hours' flow (\equiv 15 years).

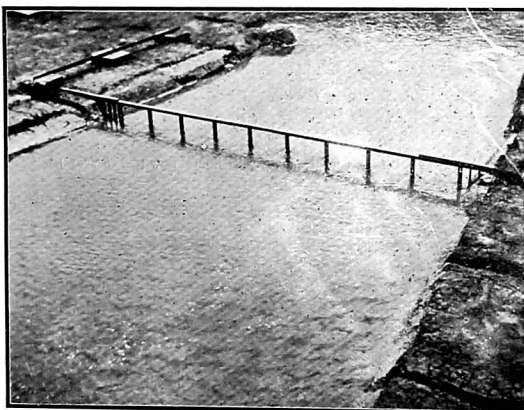


1595. Showing conditions after 47 hours' flow (\equiv 23 years).

MODEL OF THE RIVER BRAHMAPUTRA AND THE PROPOSED EASTERN BENGAL RAILWAY BRIDGE
AT AMINGAON.



1680. Looking downstream ; with piers and guide banks. Conditions of flow with $Q \cong 935,000$ cusecs.



1681. Same, looking upstream.

PART-WIDTH GEOMETRICALLY SIMILAR MODEL OF THE WASTE WEIR OF SPIN-KAREZ RESERVOIR.

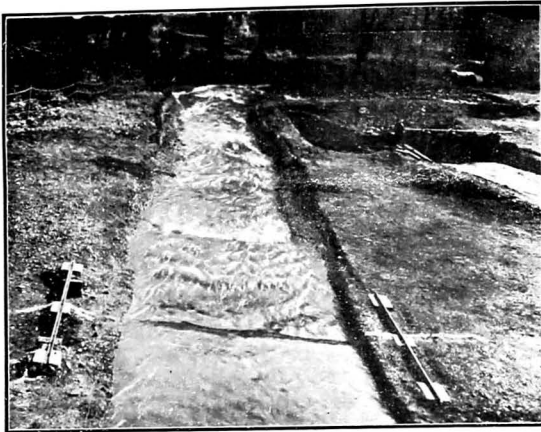
LONGITUDINAL AND VERTICAL SCALE RATIO=1/20. LATITUDINAL SCALE RATIO=1.30.



1575. Condition of flow in the waste weir tail channel—as designed, looking downstream, with $Q \cong 12,500$ cusecs.



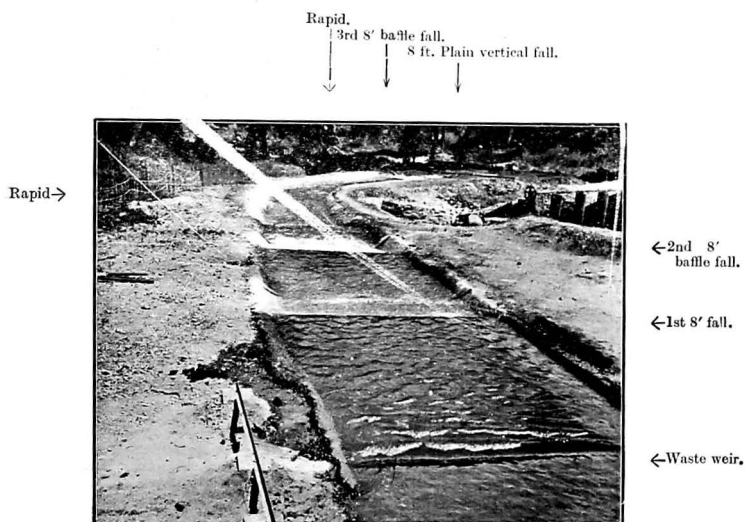
1576. Showing scour after 2 hours' flow ($\cong 9.5$ hours in prototype). Looking downstream.



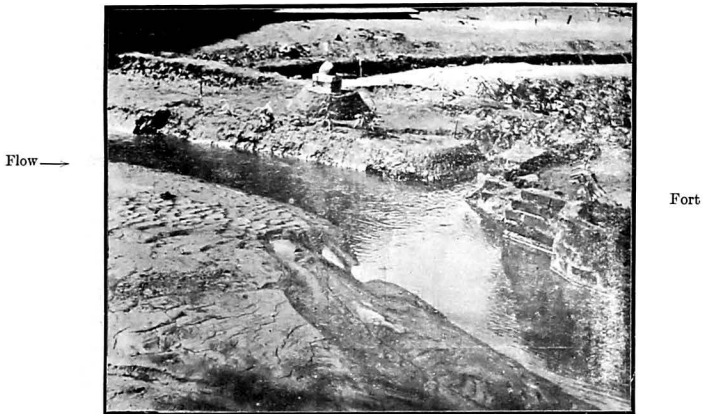
1579. Showing condition of flow in the tail channel, with bed protected by a series of wire crates \cong 3-33' high spaced \cong 250' apart in experiment 2. $Q \cong$ 12,500 cusecs. Looking downstream.



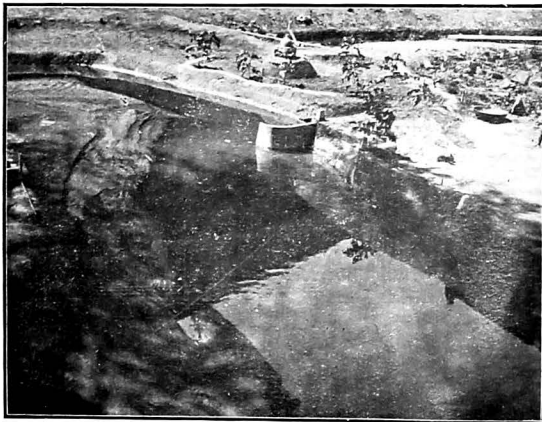
1580. Showing scour after 2 hours' (\cong 9-5 hours) flow of $Q \cong$ 12,500 cusecs in experiment 2 above.



1587. Showing conditions of flow with $Q \cong 12,500$ cusecs; with two $\cong 8'$ baffle falls spaced at $\cong 250'$ intervals downstream of waste weir; followed by a continuous rapid having superlevation and a $\cong 8'$ baffle fall and a plain vertical fall still further downstream.



1628. Model of Watrak River at Kaira (E_v =Vertical exaggeration=5). Experiment 1 showing conditions after floods of 1937. The corner of the Fort which subsided into the river may be seen on the right.



1634. Vertically Exaggerated Model. Experiment 4 with Burma type spur, showing siltation induced along Fort wall.

Fort



1669. Part-width, geometrically similar, model. Experiment 4. Sloping spur with mobile protection as laid before flood. Looking downstream.



1670. Part-width model. Experiment 6. Burma type spur, showing very deep scour at nose. Looking downstream.

Annual Report of work done during the year 1937-38.

PART I.

1. INTRODUCTORY.

In June 1916, the Special Irrigation Division was created to carry out research into problems of land drainage, reclamation, and hydrodynamics. In 1920, a Hydrodynamic Research Station was opened at Distributary 5 of the Mutha Right Bank Canal with a maximum discharge of 8 cusecs. Early experiments carried out there, have been dealt with in Technical Paper No. 28 of the Bombay P. W. D. : "Research work done by the Special Irrigation Division since its formation in June 1916".

In 1925, it was decided to construct a full scale model of a syphon spillway (5' x 4' and 29' high) for comparison with a $\frac{1}{4}$ th scale model. This was constructed at Khadakvasla, downstream of the Dam of Lake Fife. Experiments were continued for about 3 years with discharges up to 520 cusecs. About the same time it was found that the supply at Distributary 5 was insufficient, and Khadakvasla Hydrodynamic Research Station was started. During the past 9 years this Station has been gradually expanding, until now there are 13 experimental channels and the aggregate length of channels and models exceeds 2,400 ft.

Figures 1 and 2 show a plan of the area and figs. 3 and 4 show cross sections running from the Right Bank to the river bed on the left.

The maximum discharge of experiments in progress at one time exceeded 110 cusecs, and in general up to 6 model experiments have been in progress simultaneously. The water, after passing through the models, is discharged into the Mutha Right Bank Canal during the fair season ; but during the flood season a large part of the water, being in excess, is returned to the river.

Some 20 photos of work done at Khadakvasla in previous year have been selected from over 3,000 and are shown on Photo Sheets 1 to 9.

Photo 1690 is a composite photo taken from the top of the hill above Khadakvasla Dam and shows the Lake Fife Research Station just downstream.

Photo 1203 shows 22 demonstration models on the Right Bank.

Photo 1205 shows a Canal Fall model in the 18 ft. Left Bank Channel. This channel is now used for silt experiments and has been increased in length to 200 ft.

Photo 1232 shows a small scale fall experiment in the 18 ft. Left Bank Channel and a large scale twin—fall model, discharging 240 cusecs in an adjoining channel.

Photo 328 shows the full scale syphon spillway model discharging 520 cusecs ; and

Photo 884 the same syphon running with the outlet flared at 1 in 5 downstream.

Photo 850 shows a $\frac{1}{4}$ th replica with glass sides. Pressures and velocities at all points in the stream were observed and compared and it was found that the model gave closely similar results except as regards priming depths.

Photo 842 shows the Sukkur Barrage Model, looking downstream, with Bukkur island standing out in the middle foreground.

Photo 521 shows a part width model of the Sukkur Barrage, showing deep scour at the vertical nose of the Divide Wall.

Photo 841 shows a model of the Jamrao Canal at Mile 17 with warped, converging, silt-excluding, head for Dim and Khadro Minors.

Photo 662 shows heavy scour which would have occurred downstream of the pavement of Tando Mastikhan fall with 1 in 10 divergences, without remodelling, and

Photo 826 the same design after baffle and deflector had been added.

Photo 1256 shows a part width model of Sukkur Barrage before downstream protection was added.

Photo 1263 the same, after a baffle had been added d/s of the sluice gate (causing a standing wave to form) and a deflector at the end of the pavement, which created a bed roller.

Photo 1167 shows a 'falling apron' experiment, using different coloured pebbles to trace how 'launching' takes place. It will be seen that stones launch almost at right angles to the axis of flow.

Photo 1012 shows profile of flow in a pier experiment at the pier.

Photos 1183 and 1187 show large scale model of the Right Guide Bank of the Hardinge Bridge, showing how the slip of 26th September 1933 occurred.

Photo 1192 Experiments with Falling Aprons. Model of Guide Bank at Hardinge Bridge showing formation of breach (looking downstream towards bridge).

Photo 943 shows a portion of the 600 ft. long model of the Ganges River above the Hardinge Bridge.

Khadakvasla has several exceptional natural facilities :—

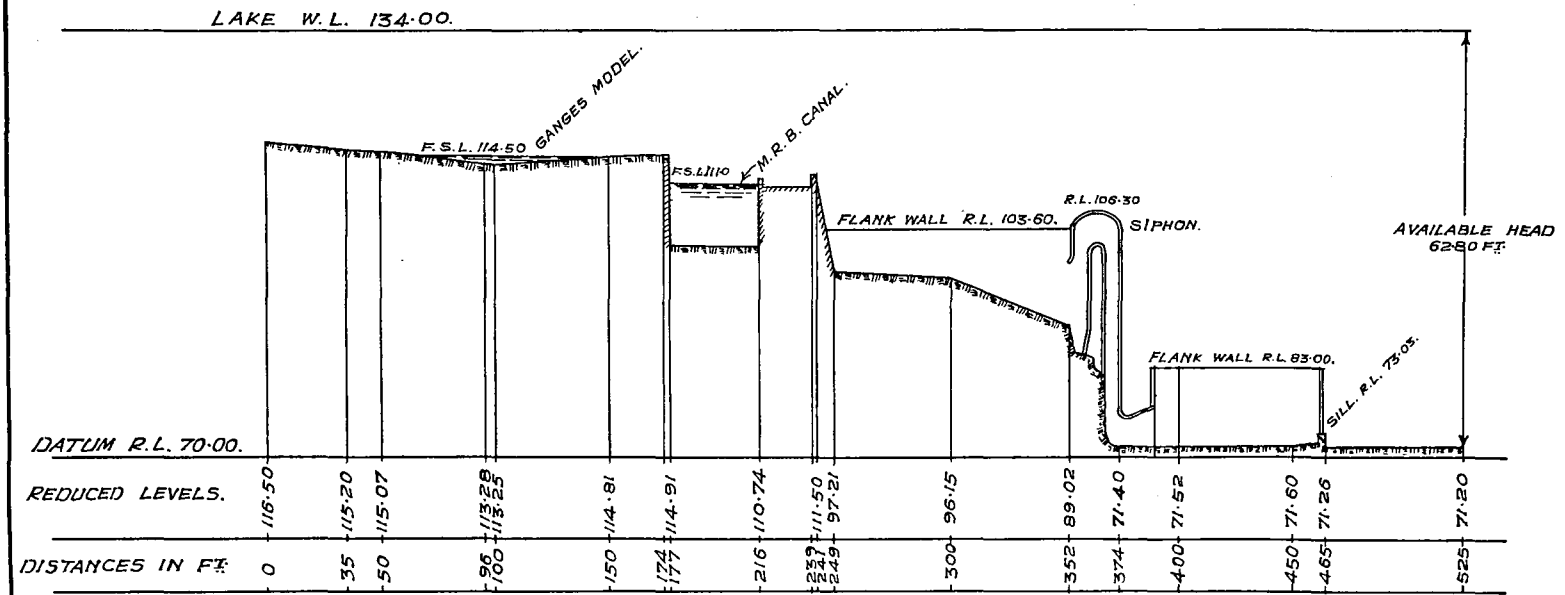
- (1) Water is crystal clear for 7 months of the year; almost clear (visibility 2 ft.) for 3 months of the year and contains only very fine colloidal silt in the remaining 2 months.
- (2) Owing to water being drawn from a deep lake, the temperature of the water varies little throughout the year and scarcely at all during the day and from day to day.
- (3) The climate is equable.
- (4) The Station is only 11 miles from Poona.
- (5) No pumping is required.
- (6) No charge is made for water as it is discharged without waste into the Mutha Right Bank Canal.
- (7) The slope of the country is steep and the 'head' available up to 60 ft. which makes the Station eminently suitable for full-scale model experiments with sluices, etc.
- (8) Ample head being available, conditions are suitable for deep 'measuring chambers', small in area, which give high accuracy and quick stilling.
- (9) 8 low-level sluices with sills at Canal bed level draw water from Lake Fife. Each of these is regulated independently and as water is drawn off under considerable head from a large body of still water, the discharge of each sluice is almost absolutely constant, the variation being but a small fraction of what experience in Sind and the Deccan shows occurs when water is drawn from a Canal. In some experiments, small fluctuations in discharge are unimportant; but where the object is the determination of discharge formulæ or small variations from straight line formulæ, it is essential to have a supply in which fluctuations are so extremely small as to be negligible.

The Station at present comprises 6 acres; but further extension will be necessary if important large scale work, which the Station has recently been asked to take in hand, is accepted.

CROSS SECTION THROUGH THE SIPHON SPILLWAY

AT KHADAKWASLA.

SCALE. $\left\{ \begin{array}{l} \text{HOR: } 1'' = 60' \\ \text{VER: } 1'' = 20' \end{array} \right.$



CROSS SECTION ON A. A.

FIG. 3.

CROSS SECTION ON LINE B. B.

AT KHADAKWASLA.

SCALE: $\left\{ \begin{array}{l} \text{HOR: } 1" = 60' \\ \text{VER: } 1" = 20' \end{array} \right.$

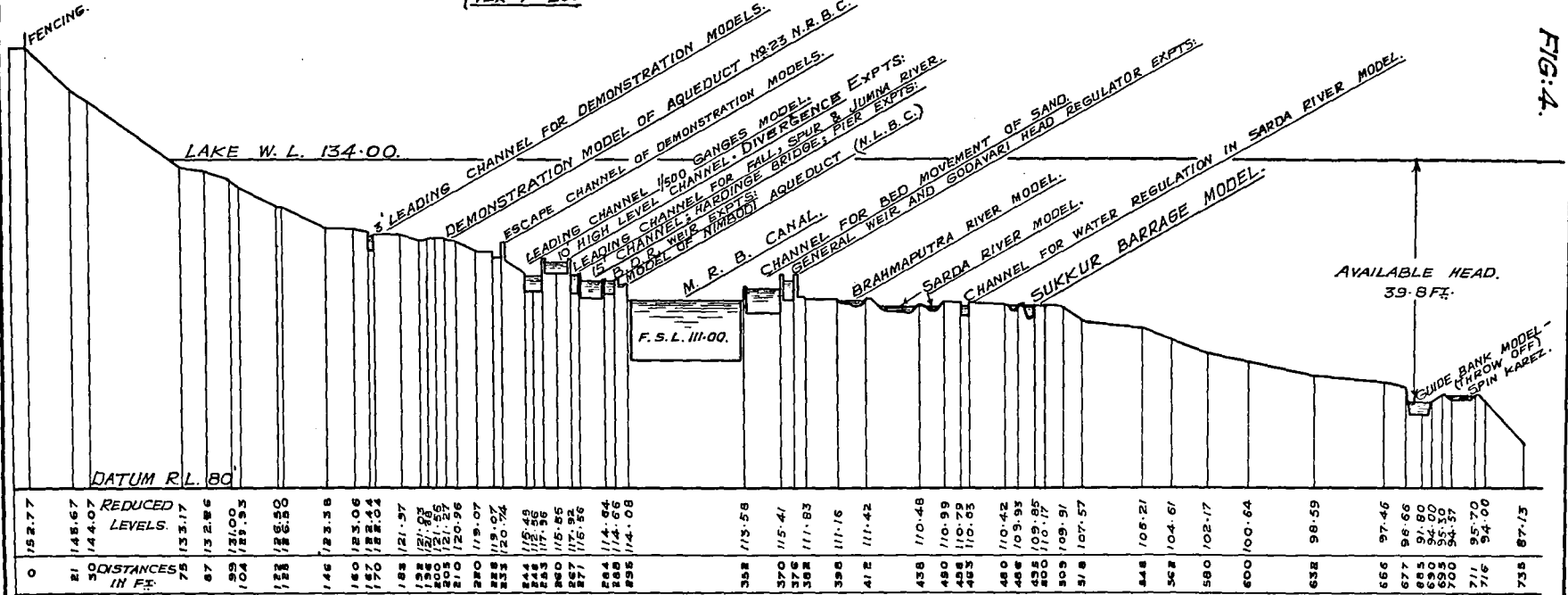


FIG. 4.

PART II.

2. TECHNICAL PAPERS.

Three Technical Papers have been issued during the year :

Bombay P. W. D. Technical Paper.

- No. 56 "Land Drainage in the Deccan Canal Areas" by W. A. Evershed, B.Sc., M. Inst. C. E., Ex. Eng., with C. C. Inglis C. I. E., B.A., B. A. I., M. Inst., C.E.
- No. 57 "Disposal of Poona Sewage Effluent" Part I, Note on effluent distribution by C. C. Inglis, C.I.E., B.A., B.A.I. (Dub), M. Inst. C.E., I.S.E., Superintending Engineer, and D. V. Joglekar, B.E., B.S.E., Assistant Engineer.
- No. 58 "Summary of the Annual Note on Experiments carried out by the Irrigation Research Division during 1936-37 at Lake Fife Hydrodynamic Research Station near Poona."

and three Papers have been written :

- (1) Paper on "The use of models for elucidating flow problems" written for the Jubilee meeting of the Indian Science Congress, by C. C. Inglis, C.I.E., B.A., B.A.I., M. Inst. C.E.
- (2) Paper on "The exclusion of silt from heads of Canals and Distributaries" by C. C. Inglis, C.I.E., B.A., B.A.I., M. Inst. C.E., and Rao Shaheb D. V. Joglekar.
- (3) Paper on "The eradication of water weeds from Deccan Canals" by C. C. Inglis, C.I.E., B.A., B.A.I., M. Inst. C.E., and V. K. Gokhale, M. Ag.

3. EXTRACTS OF PAPERS WRITTEN DURING THE YEAR.

The use of models for elucidating flow problems.

Of recent years, so many claims have been made regarding the successful application of model research to the solution of hydraulic problems that the public are now under the impression that any engineering problem can be simply solved by the use of models. A corrective to this view was provided when Herbert D. Vogel, Director of the U. S. Waterways Experiment Station at Vicksburg, in replying to the discussion on his paper entitled "Hydraulic Laboratory results and their application in nature" published in the December 1935 issue of the proceedings of the American Society of Civil Engineers, wrote :—

"In an effort to find proof—or disproof—of the reliability of open-channel experimentation, this paper was tossed out as a challenge. The results, while disappointing, were nevertheless illuminating in their paucity, and the question could now be asked, 'Are current studies being undertaken : (1) Because of the proved validity of the method ; (2) because models are a cheap means of approximation ; or (3) simply because it is fashionable to carry on 'Scientific' research ?' To run down the answer to this query, circular letters were sent out from the U. S. Waterways Experiment Station during August, 1934, to more than fifty laboratories or individuals engaged in hydraulic research. It was requested in each case that information be furnished regarding the performance of any model tests, the results of which had been substantiated subsequently by results obtained in the field. Twenty of these letters were directed to addresses within the United States and the remainder were sent to foreign countries.

"Replies were received from twenty-eight of those addressed, twelve of these being from laboratories within the United States. With a few notable exceptions, the replies contributed scarcely any information directly pertinent to the subject."

Nearly all the cases cited were of models with rigid boundaries, several of which gave only approximate correlation, and "little or no information was received in justification of movable bed models."

The object of this Paper is to show :—

- (i) that there are eight markedly different types of models, only one of which presents little difficulty and gives results suitable for immediate application; whereas other types, especially those relating to flow in rivers, present great practical difficulties;
- (ii) that much larger models than have generally been used in hydraulic laboratories in the past are required to give accurate results, especially in the case of erodible models;
- (iii) that most flow problems—even investigations into the movement of silt—which have hitherto generally been investigated in laboratories in parallel sided flumes, as though they were 2-dimensional questions, are essentially 3-dimensional problems; and
- (iv) that model experiments are a very valuable aid in the solution of practical engineering problems, but that success with them depends mainly on engineering skill which they cannot replace.

Summary and conclusions.

The Author's original intention was to group over 100 successful model investigations carried out at Poona; touching on the salient features of each group. Enumeration of the experiments had, however, to be omitted to cut down the length of the Paper.

8 main types of models are described with examples showing why some types present little difficulty and give results suitable for immediate application; whereas other types, especially those relating to alluvial rivers, present very great practical difficulties.

Geometrically similar models in which parallel flow or free vortex flow are established give geometrically similar results, provided they are not too small to make it possible to reproduce the boundary conditions of the prototype. In other cases full similarity is not attained and the divergence may be very great as shown in paragraph 8.

It is only in some models of types I, II and V that parallel sided flumes give reliable results; all other types should be treated as 3-dimensional problems.

Channels may be divided into two classes :—

- (a) Channels flowing within rigid boundaries.
- (b) Channels flowing in incoherent alluvium.

In the former, flow is under flumed conditions and a heavy super-charge of silt can be carried provided the slope is sufficiently steep. In the other type, flow is 'natural,' adjustments to changing conditions being brought about by scouring and silting. Lindley, in 1919, put forward the original theory that 'the dimensions, width, depth and gradient of a channel to carry a given supply loaded with a given silt charge were all fixed by nature' and some ten years later Gerald Lacey produced a series of formulæ which fixed gradient and shape of regime channels.

According to Lacey, $P=2.67 \sqrt{Q}$, from which it follows that $P/R=7.12 V$, where P =wetted perimeter; Q =discharge; R =hydraulic mean depth and V =velocity. This is the Lacey 'shape formula'. According to Lacey's 'initial regime formula' $V=1.17 \sqrt{fR}$, where f is the silt factor, which= $.73 V^2/R$, and hence is proportional to V^2/gR , the Froude number. So on the assumption that a regime channel approximates to a semi-ellipse, with the semi-circle as the limiting shape, the limiting discharge in cubic ft. per second= $Q_{lim}=1.82/f^2$. Actually, however, for reasons explained in the paper, $Q=3.2/f^2$ is the minimum discharge with which it is safe to work.

From the Lacey formula and from Nikuradze's work on roughened pipes it follows that the finer the silt, the larger the discharge required in the model.

There is a natural regime silt charge, which varies with the discharge and silt grade, and there is a minimum velocity for each silt, below which the regime silt charge cannot be carried.

For the same grade of silt, the lower the discharge the more sensitive is the model to a super-charge of silt.

The regime conception must form the basis of all accurate model work dealing with movement of silt, and its division between offtakes.

Experiments with *combined erodible channels and rigid models* present difficulties; because the flow formula is different in the 2 parts, and hence a model which is identical in plan in model and prototype cannot give similar results. In many cases this can be overcome by 'fitting'. Part of the difficulty is due to the fact that though vertical exaggeration has been adopted as common practice in model work, lateral exaggeration, which is also natural, has, so far as the author can determine, been overlooked.

One of the best known types of models is the 'tidal model', which falls under the semi-rigid model type. In this, conditions are imposed, the sides, and in many cases a considerable part of the bed, being held against scour. Such models give valuable results provided they are correctly designed; but as conditions are imposed, either complete data must be available—which is practically never the case in river work—or else results depend to a marked extent on the field experience of the experimenter. Rigid models are used in America for determining the effect of 'cut-offs' on upstream water levels and they give valuable information as to the immediate effect on water levels upstream; but that is all.

Models with mobile protection, *e.g.*, falling aprons, pier protection, etc., give geometrically similar results of great value.

In almost all experiments except Type I, scale effect is important; and in practice, where conditions are normally complex, this can best be determined by constructing 3 or more models of different sizes and analysing the data, which, if found suitable, can be extrapolated to prototype scale.

Combined rigid, mobile and meandering river models present many difficulties, because the laws governing each of the 3 types, here combined, are different. To solve such problems, experiments to determine the scale effect of various factors have to be carried out in sets of separate models and the results of these combined and applied in the large, full length model, changes being made by hand when conditions arise which the other experiments indicate would lead to changes occurring. Though accurate results can be obtained in a model for one particular discharge, where discharges vary widely, the degree of accuracy varies at different discharges and with low supplies, difficulties arise in reproducing scour in the model. This can only be partially overcome by selecting a suitable silt and having a varying discharge scale; but where the determination of scour with small discharges is important, separate large scale part models are required.

The main fact which follows from this Paper is the importance of large 3-dimensional models; and at Poona the discharges now preferred range between 6 cusecs and 20 cusecs in each model, but may be as much as 30 cs. in a single model, and up to 520 cusecs have been used in a model investigation of type II.

Although in general the larger the model the better the results, it must be remembered that doubling the size of a model, quadruples the area, the discharge and the labour involved; and increases the time factor to approximately 1.6 times.

The general conclusion is that in competent hands, a very wide range of experiments with large models gives results of high qualitative accuracy and may also give quantitative accuracy; but, in the case of river models, the data available is generally meagre, and though the gaps in data can be filled to a large extent by an officer with wide field experience, so that a model can be made to reproduce what has previously occurred in the prototype under known conditions of discharge and silt charge, the problems which generally have to be tackled are of immediate urgency, the discharge data being inadequate and the silt charge data *nil* and we are asked 'what will happen if nothing is done?' 'or what should be done to prevent further damage?'

Satisfactory answers depend to a marked extent on an intimate knowledge of the engineering side of the problems under consideration combined with a flair for diagnosis.

2. The exclusion of silt from the heads of Canals and Distributaries.

The experiments described were carried out during the years 1932-34, but were not written up until last year. The scheme recommended was adopted and has proved entirely successful. The following are extracts from that Paper :

"An earlier Paper of this series—Technical Paper 52 'Factors affecting exclusion of bed silt from Canals' may be read, with advantage, as an introduction.

In this Paper, the control of silt entering the Mithrao Canal is the particular question dealt with ; but the results are of wide application.

The Mithrao Canal derives its supply from the Eastern Nara above the Makhi Regulator ; and the Eastern Nara—which also supplies water to the Jamrao Canal, some miles upstream, and the Hiral and other Canals downstream—was previously a river which drew its supplies from the River Indus above Sukkur ; but it is now fed from the Eastern Nara Canal, the most upsteraam of the 4 Canals taking off from the left bank of the River Indus at the Sukkur Barrage.

Coarser silt than previously drawn began to enter the Mithrao in 1932 ; and, as the slope required to pass silt varies as (diameter)^{5/6} (1) the previous drop into the Canal of 2 ft. 6 inches at Full Supply discharge gradually disappeared, *i.e.*, *command was lost*. Now, this is a very serious matter ; because the slope required (for the same silt) increases inversely as the $\frac{1}{6}$ th power of the discharge : $S \propto \frac{1}{Q^{1/6}}$; so once command is lost, deterioration is rapid.

Several different methods were tested :—

	See Photos.
(i) Spur downstream of Mithrao Canal Head	Sheet (10), 838
(ii) Same, with 1 in 5 converging approach upstream of Mithrao	Sheet (10), 855
(iii) 1 in 5 converging approach combined with high sill	Sheet (10), 865
(iv) Same, but allowing for scour of opposite bank
(v) Single, large-radius, vane upstream of Mithrao Canal
(vi) King's vanes	Sheet (11), 890
(vii) Divide wall	Sheet (11), 888
(viii) Old (River) conditions before the Nara was canalised
(ix) Approach channel from concave bend upstream	Sheet (12), 1409 and Fig. 5.

In the first 7 methods, curvature of flow was induced in order to sweep away bed silt from the face of the Canal Regulator ; in (viii) some natural curvature existed, and in (ix) the old river bed was partly re-opened—to carry water to the Makhi Regulator and Khipro Canal—the existing straight channel, after reduction in width, being used as an approach channel to the Mithrao Canal. This lay-out, figure 5, was such that the approach channel took off where the beneficial effect of curvature was a maximum."

The original conditions are shown in figure 6, the bed lines of flow explaining why bed silt was drawn by the Mithrao. Photos 838, 855, 865, 890 and 888 (sheets 10 and 11) show five of the different methods of silt exclusion tried ; and the final solution adopted is shown in figure 5 and Photo 1409 (sheet 12) shows the remodelling as actually carried out in the prototype. Before remodelling, the ratio of silt recovered per cusec in the Mithrao canal compared to that recovered from the Nara river (according to the model result) was 2.15, whereas after remodelling the ratio fell to 0.1 ; and when the design was adopted, 'command' was quickly re-established, with a drop into the Mithrao Canal of $1\frac{1}{2}$ '. The reason for the success of this solution is clear from figure 7, which shows the effect of curvature of flow at the bend.

(1) Gerald Lacey, Paper No. 4736, Instt. of Civil Engrs. on "Stable channels in alluvium" (1930) and No. 4893 on "Uniform flow in alluvial rivers and Canals" (1934).

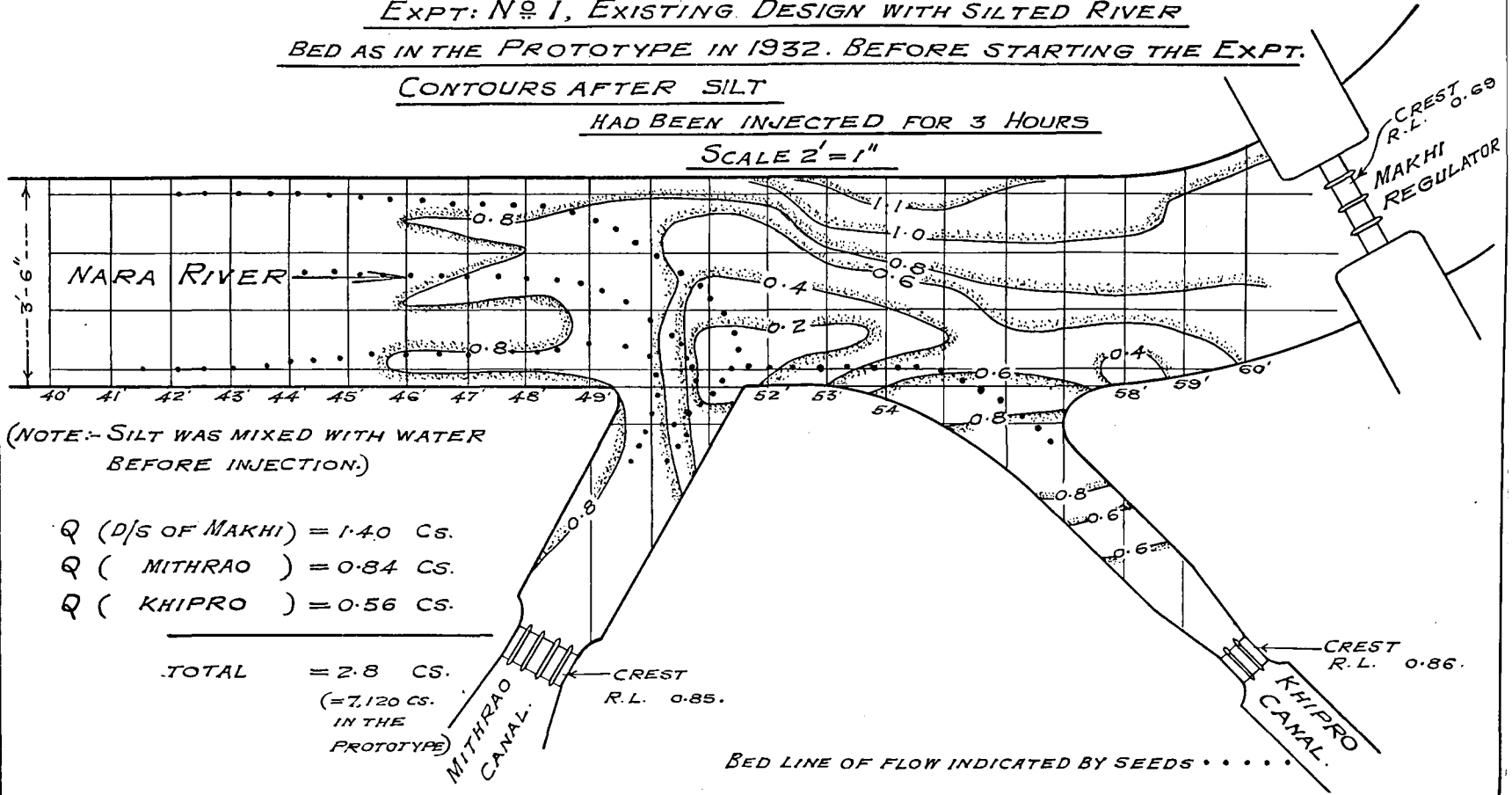
FIG: 6. MODEL OF THE EASTERN NARA AT MAKHI (SIND)

Nº 482
FIG: 6

EXPT: Nº 1, EXISTING DESIGN WITH SILTED RIVER
BED AS IN THE PROTOTYPE IN 1932. BEFORE STARTING THE EXPT.
CONTOURS AFTER SILT

HAD BEEN INJECTED FOR 3 HOURS

SCALE 2' = 1"



(NOTE:- SILT WAS MIXED WITH WATER
BEFORE INJECTION.)

Q (D/S OF MAKHI) = 1.40 CS.

Q (MITHRAO) = 0.84 CS.

Q (KHIPRO) = 0.56 CS.

TOTAL = 2.8 CS.

(= 7,120 CS.
IN THE
PROTOTYPE)

CREST
R.L. 0.85.

CREST
R.L. 0.86.

BED LINE OF FLOW INDICATED BY SEEDS

(3) *Water weeds and their eradication from Canals.*

Although this Paper deals with conditions in the Bombay Deccan, similar conditions are found over a large part of India and in other countries.

The rapid growth of water weeds in the Canals has long been recognised as a serious obstacle to efficient management.

In 1922 the Senior Author stated in a Paper on this subject, entitled "Some important factors affecting the design and working of Deccan Canals" (Paper LXVIII, Bombay Engineering Congress):—

"In the case of the Pravara Left, the canal when first opened in 1919 was capable of giving a discharge of 500 cusecs with only 6 ft. of water, whereas by 1922 only 272 cusecs could be obtained with that depth, and only 73 cusecs or 15 per cent. of the 1919 figure after the weeds had grown, which occurred within 2½ months of a full weeds clearance."

The conclusion drawn in that Paper, after sifting a large amount of data, was that silt in suspension was the dominant factor in the control of weed growth.

No further action would have been taken had not Mr. D. V. Narayanayya written a Paper in 1928 on "Aquatic weeds in Deccan Irrigation Canals" (in Volume XVI, No. 1, Feb. of the Journal of Ecology).

In his summary he stated:—

"The presence or absence of weeds is determined by the deposition of silt. The effect of silt in suspension is not proved. The three canals differ in amount of suspended silt, but the comparative poverty of aquatic flora in canals with more suspended silt can be better correlated with greater velocity and less deposition of silt."

In order to solve this question finally, further research was carried out to ascertain the relative importance of various factors and to work out methods for eradicating water weeds.

During the monsoon, Deccan Canals obtain most of their supply from river flow; but for the rest of the year they are dependent on supplies from large artificial lakes, formed by the construction of masonry dams across river valleys in the foot-hills on the eastern side of the Western Ghats. These dams are about a mile long, and vary in height from 100 to 270 ft. The Canals, except the Mutha Canals, take off from above Pick-up Weirs constructed across the Rivers some 20 to 53 miles downstream of the storages. These Weirs vary from 10 ft. to 40 ft. in height; and from 890 ft. in length in the case of the Pravara Pick-up weir at Ojhar, to 3,618 ft. in the case of the Godavari Pick-up weir at Nandur Madhmesh-var.

The Mutha Canals, however, take off direct from Lake Fife. Like all Deccan storages, Lake Fife is free from water weeds—due to the wide fluctuations in water level at different times of year inhibiting growth.

Weed growth is not uniform throughout the year, and except in the case of the Mutha Right Bank Canal gives little trouble during the monsoon.

Weed growth generally starts in October, and by December is heavy; and under the old method of running canals, another clearance was required early in March and another about the end of April.

Closures for weeds clearances lead to delayed waterings, which tend to reduce the yield of cane; and, as closures dislocate normal distribution of water, they put a heavy strain on staff and lead to unavoidable mistakes and friction between irrigators and staff—not merely immediately after a closure but for some weeks, until normal conditions can be re-established. It is very desirable, therefore, to eliminate them, and with that object in view several types of saws have been tried, but without success, hand weeding during closures having been found to be the most successful method of removing weeds.

The following water weeds are found in the Deccan:—

- (1) *Vallisneria spiralis*,
- (2) *Potamogeton pectinatus*,
- (3) *Potamogeton perfoliatus*,

- (4) *Potamogeton indicus*,
- (5) *Ceratophyllum demersum*,
- (6) *Hydrilla verticillata*,

but for simplicity, water weeds may be divided into only 2 groups :—

- (1) *Vallisneria spiralis*—a grass-like weed, up to 14" in length, which grows in tufts ;

and

- (2) *Potamogeton* varieties and associates—the worst of which is *Potamogeton perfoliatus* which grows to a length of 12 ft. or even more.

(1) is the first weed to appear, and is comparatively unimportant except in small channels. (2) indicates a much more serious condition of weed growth.

NOTE :—In the Paper, of which this is merely a Summary, the vegetative characteristics of each type of weed are described.

Characteristics of water weeds.

They are perennial in nature and where channels go dry they survive by developing special organs. Growth is profuse between November and May, but is markedly less in the rest of the year. The roots of water weeds are firmly rooted in the soil ; but it appears that they also absorb soluble salts and carbon dioxide. Aquatic plants, like land plants, require oxygen for respiration and tissue building. The amount of O₂ and CO₂ required varies in different weeds ; those which require large quantities can persist where the velocity is fairly high, e.g., *Vallisneria spiralis* ; but other varieties, like *Potamogeton Indicus*, can only live where the velocity of water is low, and to avoid asphyxiation, this weed has developed large spongy leaves. Type of weed is therefore an indication of canal conditions—*Vallisneria spiralis* being dominant where velocity is high and the *Potamogeton* group growing only where the velocity is sufficiently low for it to become established. *All the varieties overlap to a certain extent and once a weed has become established under favourable conditions, it tends to persist in unfavourable conditions.*

Factors which may be expected to affect the growth of water weeds :—

- (i) infection.
- (ii) temperature of water.
- (iii) chemical composition of water.
- (iv) velocity—effect on weeds due to silting velocity and mechanical, or drag, effect.
- (v) depth of water.
- (vi) deposition of silt.
- (vii) reduction of light due to turbidity and depth.

Several inherent difficulties make the proof of relative effects very difficult. Thus :—

- (a) it has been found impracticable to reproduce natural conditions on a large enough scale under controlled conditions ;
- (b) in canals, conditions vary throughout the year, so that although conditions may be unfavourable for the greater part of the year, yet they may be favourable for long enough to allow weeds to become established ; when they will tend to persist ;
- (c) under canal conditions, several factors are linked ; thus a reduction of discharge leads to a reduction of velocity and depth ; which, in turn, tends to cause deposition of silt and a reduction of silt in suspension.

(i) *Infection.*—Under Deccan conditions, water weeds grow profusely in all talas and ponds—*i.e.*, infection is universal—and this enquiry showed there is no relation between weed growth in Canals and intensity of infection.

(ii) *Temperature of water.*—In the Deccan water temperature varies between 20° and 30° C, and hence is highly favourable for weed growth.

(iii) *Chemical composition of water.*—There is no traceable relation between intensity of growth in various canals and composition of water; but alkalinity, by causing dispersion of clay, has a secondary effect; because it increases turbidity and reduces the amount of light at the bed.

(iv) *Velocity*, quā velocity, has little effect except where it exceeds 2 ft. per second; but it has an indirect effect, in that when the velocity is in excess of regime velocity, deposition of seeds and cuttings does not occur.

(v) *Depth of water.*—Water weeds can grow in clear water at depths down to 18 ft. and there is no evidence that depth, quā depth, has any inhibiting effect on weed growth.

(vi) *Deposition of silt* is undoubtedly favourable to weed growth; but deposition only occurs where the velocity is considerably below regime velocity and such low velocities do not occur in properly designed channels. When Canals are newly opened, they are generally run with low discharges and water has to be headed up at regulators. This reduces the velocity and causes deposition of silt, especially along the banks, rendering the water clear; so that all conditions are then favourable for weed growth. Thereafter the weeds tend to persist.

(vii) *Reduction of light due to turbidity.*—There is an *a priori* reason to anticipate that exclusion of light will reduce weed growth, in that light is required for carbon assimilation and tissue building (photo-synthesis) and there are no cases in which weeds grow where the water is turbid throughout the year, whereas weeds grow profusely where water is clear. Under the latter condition silt is normally depositing. There are two important questions, therefore, for consideration:—

- (A) Are there any cases in which silt is depositing, yet weeds do not grow?
- (B) Are there any cases in which light exclusion appears to be the only possible explanation?

As regards (A):

There are such cases:—

- (a) When the Nandur Madhmeshwar pick-up weir basin at the head of the Godavari Canals first filled with water, weeds grew profusely even where the depth of water was 18ft. deep. At that time the water was clear from the end of the monsoon until the following monsoon. In 1935 although silt was still depositing, water weeds were limited to the zone between R. L. 529 to 531, the upper limit being due to water being kept below that level in the fair season, and the lower limit being, so it appears, a depth limit at 4 ft. below the top of the weir at R. L. 533.
- (b) Silt is depositing at places in all Canals yet water weeds do not grow in Canals like the Nira Left.
- (c) The discharge of the Nira Right Bank Canal was, until recently, less than $\frac{1}{5}$ th the designed full supply discharge and the silt was depositing, especially above 'stop-gates', where water was being headed up; but whereas there was a heavy growth of weeds elsewhere, there were no weeds for about 4 miles above each 'stop-gate' though the velocity was a minimum and silt deposition a maximum.
- (d) Dim minor was weed-infested so long as it was fed with clear water from the Makhi Dhand; but when it was transferred to Jamrao Canal, which carried silty water, weeds died out, in spite of the fact that silt deposition was occurring.

The conclusion to be drawn from the above cases is that though water weeds generally grow luxuriantly where silt is depositing, several factors then being favourable, the above cases show that weeds do not grow where silt is depositing unless the water is also relatively clear.

As regards (B):

Cases in which exclusion of light is the only explanation.

Cases (a), (c) and (d) of the previous paragraph (A) are 3 examples. The following are other examples:—

(c) When silty water arrives at the beginning of the monsoon, the discharge of channels for the same gauge reading does not at first increase, but after some time, weed growth decreases and the discharge increases. Similarly the discharge of channels begins to decrease about 20 days after turbidity decreases.

(f) When the Godavari Canal was opened in 1912, water was at first clear, the disc reading being about 4 ft. At that time there was weed growth all down the canal. By 16th April 1924 weeds had died out, except in places, down to mile 17—even where all conditions, except turbidity were highly favourable to weed growth. But by 1934 *Potamogeton Perfoliatus* had died, down to mile 28, with only a moderate growth of *Vallisneria spiralis* from mile 23-28. Further down the Canal there was still a heavy growth of all kinds of weeds including *Ceratophyllum demersum*. This gradual dying of weeds was steadily extending down the canal as silty water penetrated the weeds.

Mile	Depth of water.	Turbidity (disc reading).
1	6' 0" to 5' 2"	} 1.5'
26	5' 2" to 4' 6"	
39	4' 3" to 3' 9"	3.5'

(g) A raft was moored in Vir Basin and another in a pond connected to Vir Basin by a narrow channel. Rafts were laid across the rafts and tubs suspended from the ends of the rails—clear of the shadow of the rafts—at depth of 6", 1', 2' and 3'. The chemical composition of the water of the pond and Vir Basin was similar, *viz.* :

	Vir Basin	Pond.
pH	8.2	8.2
Conductivity	115	130

In this experiment, velocity was negligible; and conditions except light exclusion, were practically uniform. The experiment was started on 20th March 1928.

The weed growth was examined 6 weeks later: It was found that in the silty water of Vir Basin—(disc reading=0.8')—*Vallisneria* decreased with depth down to 2½ ft. (and had almost disappeared at 3'); and *Potamogeton perfoliatus* was stunted, even at 6", and was dying off where the depth was 1' or more; whereas in the pond, (disc reading=2.8'), both weeds were growing strongly at all depths down to 3 ft.

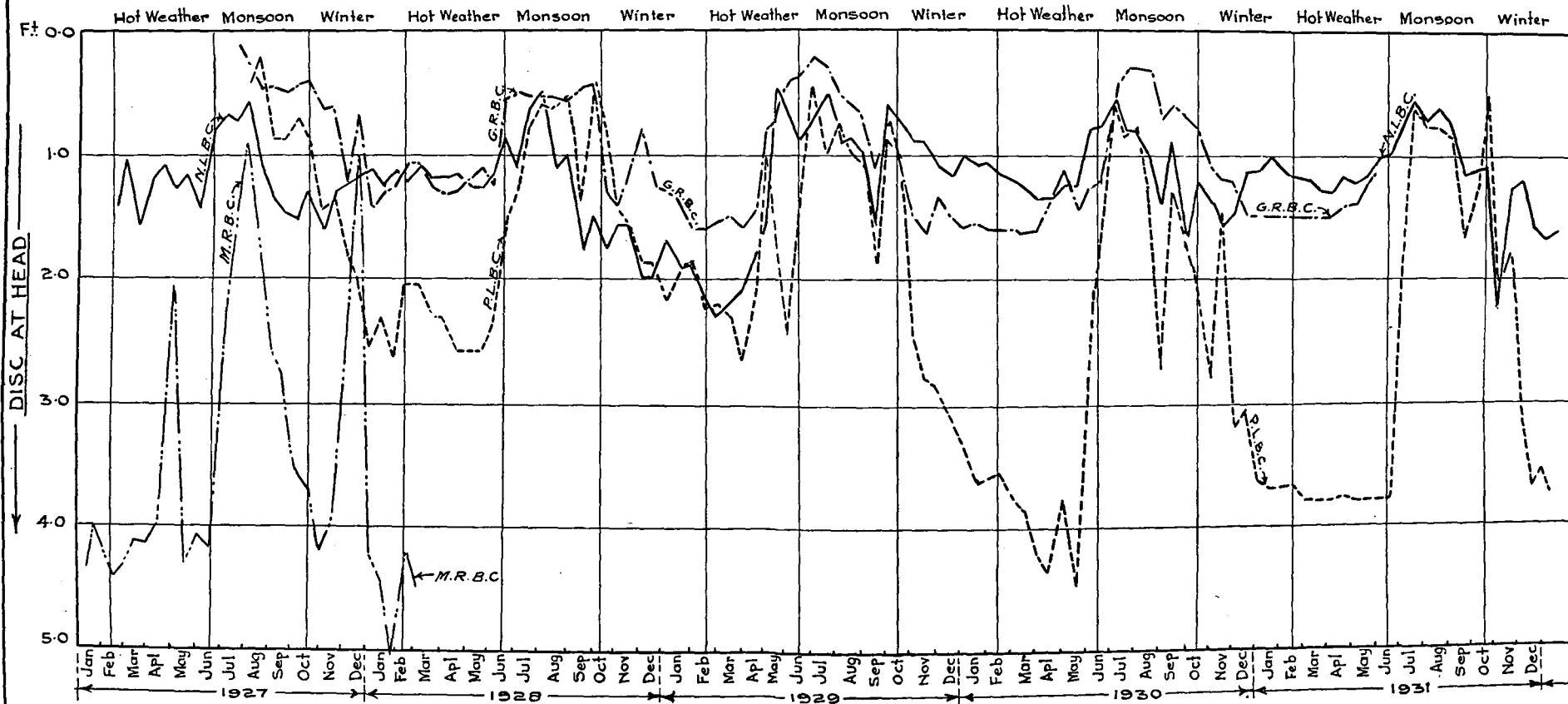
The conclusion that exclusion of light is the dominant factor controlling weed growth was also borne out by the data in figure 8 which shows turbidities at the heads of the larger canals between 1927-31, when white disc readings were observed daily.

It will be seen that turbidity was greatest at the head of the Nira Left Bank Canal (except in the fair season of 1928-29), when there was a marked drop—(due to the discharge let out from Bhatghar being more constant than in other years). This Canal has always been more free from weeds than any other. The Godavari Right Bank was the only other Canal in which weeds were, at this time, decreasing; and the disc reading was also small. In 1930 there was a marked increase of disc reading in the Pravara Left Bank Canal, and thereafter weed growth became serious. The worst Canal of all for weed growth was the Mutha Right Bank Canal, in which the disc reading generally exceeded 4 ft.

Conclusion.—

- (i) In every case examined, weed growth and turbidity bore a close inverse relationship;
- (ii) No other single factor inhibited weed growth under normal canal conditions;

GRAPH SHOWING TURBIDITY OF CANAL WATER NEAR
HEAD WORKS AS OBSERVED WITH THE WHITE DISC



REFERENCES

NIRA LEFT BANK CANAL —————

MUTHA RIGHT BANK CANAL

PRAVARA LEFT BANK CANAL - - - - -

GODAVARI RIGHT BANK CANAL - . - . -

- (iii) Where $d > 3D$, no weeds grew ; Where D =disc reading and d =depth.
- (iv) Where $d > D^2/2$, only Vallisneria grew ;
- (v) Turbidity, which leads to exclusion of light was the dominant factor controlling weed growth in Deccan Canals.

Application of conclusions.

The questions for consideration at this stage of the inquiry were :—

- (1) How to create and maintain turbidity in pick-up-weirs.
- (2) How to create or increase turbidity in Canals.

(1) Where a pick-up-weir is very large, *e.g.* Lake Fife, water issuing therefrom is clear and hence conditions are unsuitable for preventing weed growth. Where the pick-up-weir is very small, as at Ojhar, water is clear almost throughout the fair season. In between these two conditions there is an optimum condition (as in Vir Basin at the head of the Nira Left Bank Canal). At one time the turbidity in Vir Basin was attributed to intermittent flow causing a rush of water, which stirred up deposited silt on its way down the River. We now know that the increase was mainly due to fluctuating the water level in Vir Basin, leaving fresh levels of the side slopes exposed to wavewash (caused by the wind, which springs up each evening) which attacked the newly exposed silt face. This relationship is shown in figure 9.

In 1934 hot weather, the wind velocity and turbidity were observed every 3 hours and it was found (see figure 10) that turbidity in Vir Basin increased rapidly with wind for depth less than 10.5 ft. ; but for higher gauge readings the effect of increased wind velocity was small. The explanation was that there was a dense growth of polygonum, which grows 'between wind and water' along the banks of the Basin ; so that when water rises above 10 ft., the waves have their energy dissipated by the weed growth, which is partially submerged at that level. When the water is fluctuated between 8 ft. and 9.5 ft. turbidity is considerable ; because a large face of silt is then exposed ; and the proportion of silty bank-water, to clear, midstream, water ; and the velocities of flow down the Basin, are then maxima.

Conclusion.—

A pick-up-weir, in which about 3 days supply can be stored, with a range of fluctuation of about 2 to 5 ft., makes it possible to maintain sufficient turbidity to eradicate weeds.

(2) *How to maintain or increase turbidity in Canals :* The Statement, below, shows the weight of silt per 100 c.c., of various sizes, found in suspension, determined at miles 1, 9, 25 and 42 of the Nira Left Bank Canal :

Site with mileages.	Disc depth in ft.	Silt 0.02 to 0.008 mm.	Finest silt 0.008 to 0.003 mm.	Clay 0.003 and less mm.	Total parts by weight in grams per 100 c.c.
Average results grams in 100 c.c.					
Vir Head works	1.56	0.015	0.004	0.012	0.031
Pimpri Mile No. 9	1.46	0.011	0.003	0.014	0.026
Vadgaon Mile No. 25	1.35	0.008	0.003	0.016	0.027
Baramati Mile No. 42	1.25	0.018	0.004	0.015	0.029
Difference Head and Mile 42	0.31	—0.005		—0.003	0.002

This showed that though the total weight of silt per 100 c.c.s. decreased by about 6 per cent. between the Head and Mile 42, yet the clay increased by about 25 per cent. ; and the white disc disappeared at 1.25 ft., compared to 1.50 ft. at the head. This increase in dispersion, and hence in light absorption, tended to vary directly as the increase in alkalinity—though other factors, which could not be eliminated, affected the relationship.

Conclusion.—

There appears to be no practical way of increasing turbidity in a Canal.

Eradication of weeds by the 'rush rotation' system of irrigation.

This system was first introduced to facilitate remodelling the Nira Left Bank Canal, to increase the discharge of the canal by 50 per cent.

When first introduced, this system of closing the canal for 5 days in ten gave rise to strong opposition, which it would have been difficult to withstand, had it not been essential for remodelling. That the system had to be introduced was most fortunate ; because it has proved to have several important merits :—

- (i) it generally, though not invariably, eradicates weed growth ;
- (ii) it makes it possible to run channels with full supply—thereby preventing waste and increasing efficiency of distribution ;
- (iii) it leads to a canal being run under the conditions for which designed and with maximum velocity, and hence optimum conditions for rapid control of flow ;
- (iv) it gives ample time and facilities for maintenance and improvements at the Headworks and in the canal.

The effect of the 'rush rotation' system on weed growth can be plainly seen by comparing the upper and lower charts (before and after introduction) of figures 11 and 12.

The reasons why this system has been so remarkably successful in eradicating weeds are that it brings into operation a variety of factors, all of which tend to check weed growth.

- (1) The depth is greater and so more light is excluded.
- (2) The sides are completely dry during closures, and, where central drains have been excavated, the bed dries, and the weeds are exposed to the Sun's rays.
- (3) Although weeds can withstand a single long dry period, by producing special organs ; alternate wet and dry periods—by alternately encouraging growth and then drying off the growth—has a markedly adverse effect ; as shown by the fact that closures once in 20 days killed off weeds even more rapidly than closures at intervals of 10 days : From which it would appear that the effect is greater when the closures takes place just when new growth is beginning.

It is generally asserted that the 'rush rotation' system reduces percolation. This, however, is only true as a secondary result of eradication of weeds, which leads to the same discharge being passed with a smaller depth, whereas losses from Deccan Canals \propto (depth)^{1.7} to (depth)². Where only partial closures are possible, weed growth generally increases and higher depths are then required for the same discharge, which leads to greater percolation.

Conclusions.—

The growth of water weeds in Deccan Canals has hitherto interfered to a serious extent with canal administration, due to the fact that closures for weeds clearance were necessary in the hot weather at intervals of 40 to 50 days. These closures, which took place when the discharge had fallen to minimum supply, necessitated an increase in the period between waterings and considerable dislocation of normal supplies, with the danger of mistakes and the certainty of water being taken out of turn by some irrigators. The consequent dislocation of normal conditions did not end when the Canal was reopened, but persisted for about 2 rotations after each closure. These closures inevitably led to serious loss to irrigators, a severe strain on staff, and a considerable amount of unavoidable friction.

Temporary labour had to be collected—often a difficult matter—and work had to be carried out against time : because no mechanical method of weeds clearance has been found which is as satisfactory as doing the work by hand,

GRAPH SHOWING TURBIDITY (DISC READINGS) AT HEAD (VIR BASIN) AND TAIL (MALEGAON COLONY), ANEMOMETER & CONDUCTIVITY IN THE NIRA L. B. CANAL

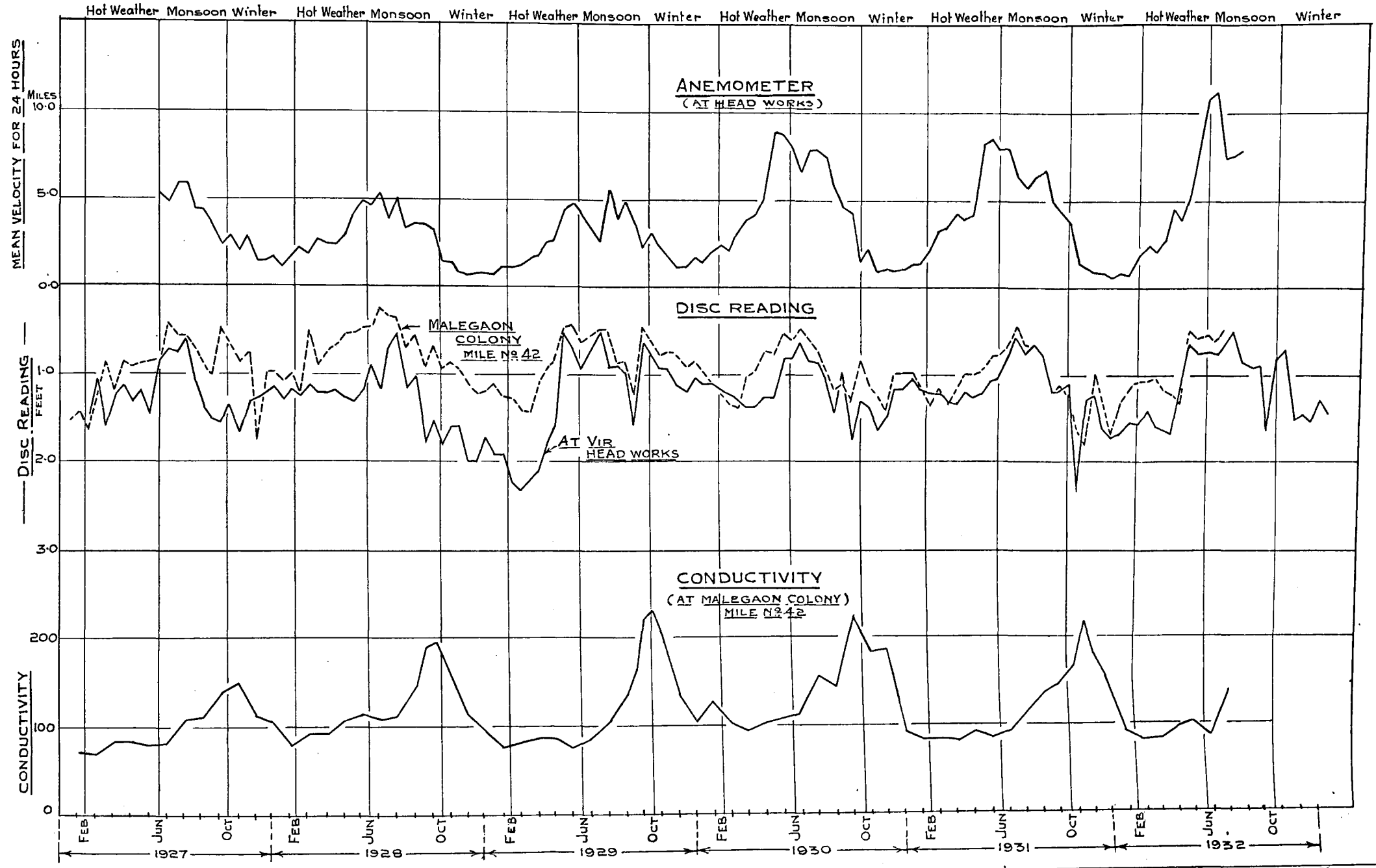


FIG:10

GRAPH SHOWING INCREASE IN TURBIDITY
DUE TO WIND WITH WATER AT DIFFERENT
GAUGE LEVELS
IN VIR BASIN NIRA CANALS

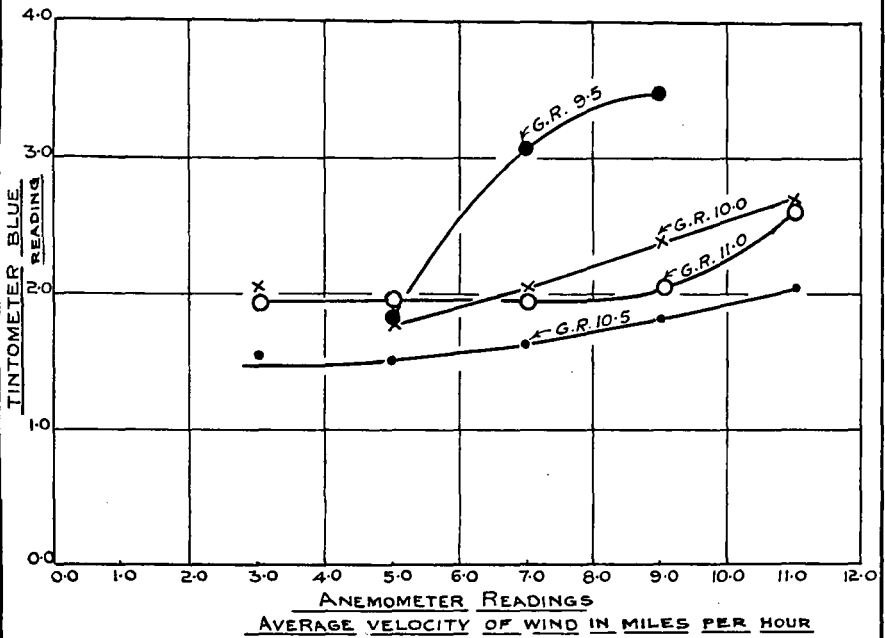


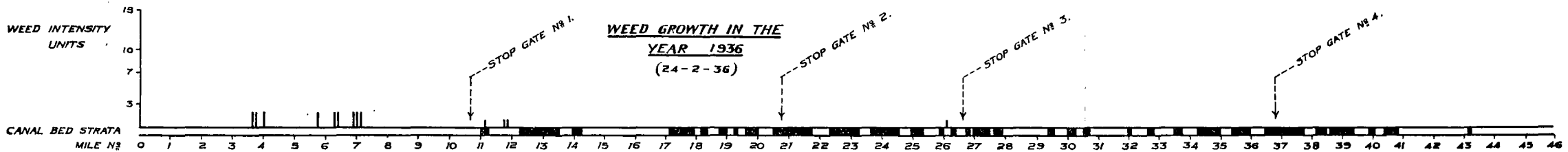
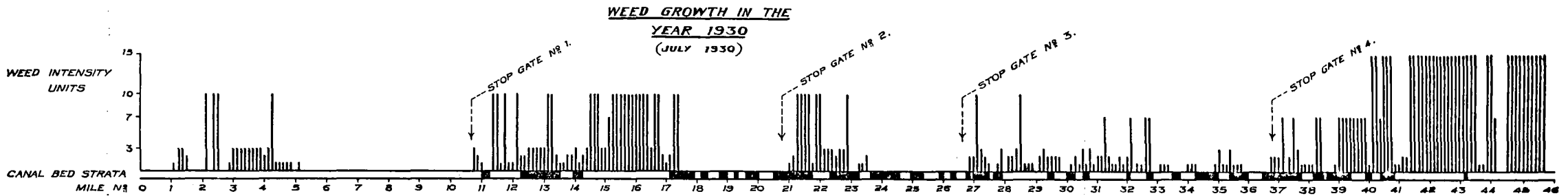
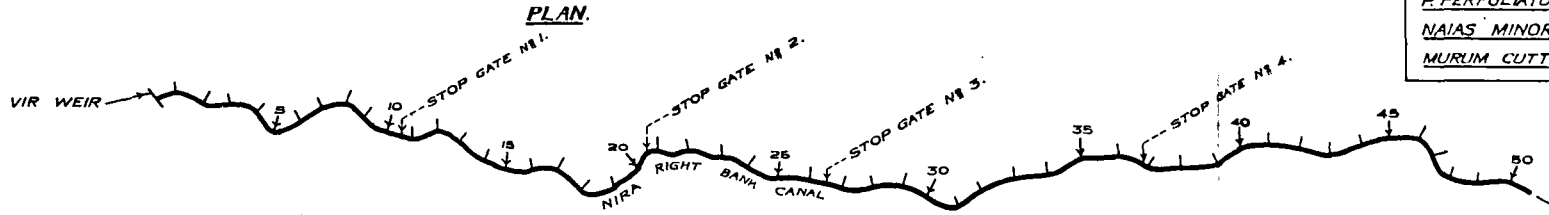
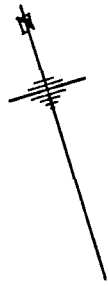
CHART SHOWING WEED GROWTH

IN NIRA R. B. CANAL

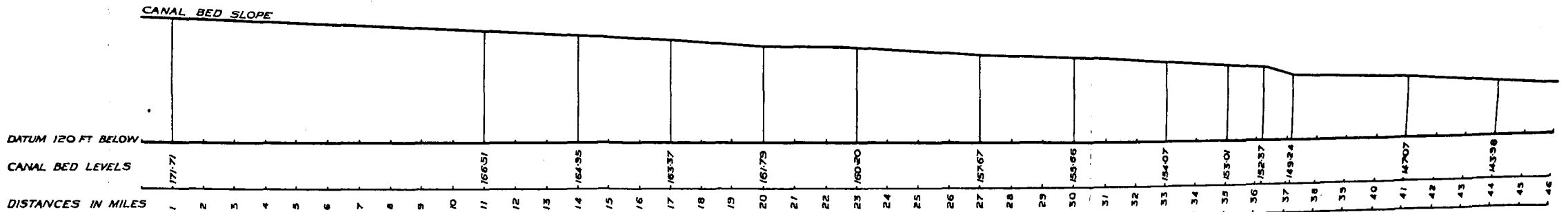
FIG. 11 AZ.(N.R.B.C.)

REFERENCES

NO GROWTH OR ISOLATED GROWTH.....	0
VALLISNERIA SPIRALIS (THIN GROWTH).....	1 I
DO (MODERATE GROWTH).....	2 I
DO (THICK GROWTH).....	3 I
NITELLA E. P. PECTINATUS.....	7 I
P. PERFOLIATUS.....	10 I
NAIAS MINOR.....	15 I
MURUM CUTTING.....	█



BED SLOPE
SCALE - VER. 1" = 40 FT
HOR. 1" = 3 MILES.



The 'rush rotation' system has been so successful (on most of the Deccan Canals) that there is a danger of it being looked on as a panacea. Where, however, the slope of a canal is very small, it may be very difficult to drain off the stagnant water completely, and, where this occurs, the weeds will not die off and may even increase. There is also a danger that some of the weeds will accommodate themselves to changed closure conditions, and persist; because they have a marked capacity for adaptation, and are descended from land plants.

In certain cases it may not be desirable or even possible, to adopt the 'rush rotation' system. Weeds will, however, be inhibited where there is adequate turbidity.

Under Deccan conditions no weeds grow where the depth exceeds three times the depth at which a 3" white disc just disappears from view; and where the depth exceeds $D^2/2$ (D =disc reading) the Potamogeton group does not persist.

Where natural turbidity in the parent River is inadequate, turbidity can be created by providing a pick-up-weir designed to hold about three days supply, for a range of 2 to 5 feet in water level.

Water can then be let out from the storage intermittently, flow being run for say 1 day in 3. Wavewash will then erode the exposed face of silt, deposited during the flood season, and will keep this silt in suspension until it reaches the head of the Canal. Where, however, the pick-up-weir basin is too large (Lake Fife), or silted berms with fairly steep faces have not had time to form (Nandur Madhmeshwar); or where the pick-up-weir basin is too small (Ojhar weir at head of Pravara Canals) turbidity will be in defect. In such cases, weed growth can be reduced by running velocities considerably in excess of regime velocities throughout the year.

PART III.

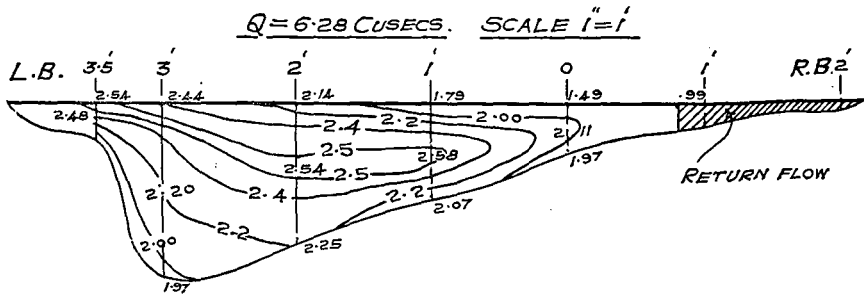
4. TRAINING THE RIVER MULA ABOVE THE RAHURI CAUSEWAY—DIST. AHMEDNAGAR—SUPPLEMENTARY EXPERIMENTS.

(1) Introduction.

In 1922 a causeway was built on the Ahmednagar-Manmad Road on the line of the then existing Ferry; and the Ferry crossing was moved upstream towards the River bend. At that time, the Mula River followed the concave bank round the bend; but when the ratio of 'length of river course at midstream to chord length' exceeded 1.5, the river began to short-cut across the convex, projecting, right bank nose, impinging on the left bank at the Ferry landing stage—rendering it unsafe—, while a back-water formed along the right bank on the ferry line, so that the ferry would not work. The unsatisfactory conditions in 1936 can be seen in photo 1534 (sheet No. 13) and figure 13, downstream of a 100° curve on the Mula River, near Rahuri.

A model with erodible banks was run for the equivalent of 2 years, and scour took place, as shown in photo 1539—sheet No. 13. Figure 14 shows the lines of flow at the surface and bed during the maximum flood of the first season, from which it will be seen that there was a large area of return flow near the concave bank and return flow near the right bank at the ferry. The high velocity flow impinged against the left bank upstream of the ferry and the left bank was eroded by the equivalent of 140 feet at cross section 4. At cross section 2, on the line of the ferry, the bed was scoured near the left bank and was silted near the right bank, i.e. the conditions became increasingly bad. Figure 15 shows isotachs at the cross section, on the ferry line.

1936
FIG. 15, MULA RIVER EXPT. AS AT PRESENT, WITHOUT
ANY TRAINING DEVICE EXCEPTING THE EXISTING-
GROYNE. ISOTACHS AT C.S. 2 (FERRY) IN 2ND. SEASON



(2) Object.

The object of the experiments was to determine how to make the surface flow normal to the ferry crossing and how to prevent jetting and return flow, maintaining comparatively natural uniform velocities all across the section.

(3) Experiments.

In continuation of experiments carried out last year—described in 1936-37 Annual Report, and Summary—8 series of experiments have been carried out this year; but only 2 will be mentioned.

The scale ratios were as before viz.:—

Horizontal scale ratio = 1/80

Vertical scale ratio = 1/13.75

Discharge scale ratio = $8.8/45500 = 1/5170$

- (i) Last year's experiment, with perforated groynes pointing 30° upstream of cross sections 6 to 10, which had been run for only one flood season, was continued for the equivalent of 4 seasons; but as it was found that conditions deteriorated, it was abandoned.
- (ii) A design was then tried consisting of a Burma type spur, shaped like a hockey stick pointing upstream—see photo 1538—sheet No. 14—and red line of figure 13. The detailed design is shown in figure 16. In this design, the nose of the spur was above H. F. L., whereas the stem of the spur consisted of a compound trapezoidal notch, the higher part having its sill at R. L. 104 and the lower part at R. L. 100.0, the object being to deflect the whole of the cold weather flow round the spur and along the concave bank; but when $Q > 3400$ cusecs, the low level notch would begin to overflow and when $Q > 11300$ cs., the high level part of the notch would begin to overflow as well.

The existing groyne upstream of the landing stage of the ferry could, with advantage, be raised from R. L. 100.65 to 105.86, to dissipate energy downstream of the groyne in a standing wave, so as to reduce velocity at the landing stage.

This design was tested for the equivalent of 9 seasons with high floods. Each season was represented by a rising flood and a falling flood in the model:—

Rising flood	{	$Q = 3.66$ cs. ($\equiv 18900$ cs. with 2' depth over causeway) for 1 hour ($\equiv 15$ hours)
		$= 4.90$ cs. ($\equiv 25400$ cs. with 3' depth over causeway) for 1 hour ($\equiv 15$ hours)
Falling flood	{	$= 6.28$ cs. ($\equiv 32500$ cs. with 4' depth over causeway) for 1 hour ($\equiv 15$ hours)
		$= 4.90$ cs. ($\equiv 25400$ cs. with 3' depth over causeway) for 1 hour ($\equiv 15$ hours)
		$= 3.66$ cs. ($\equiv 18900$ cs. with 2' depth over causeway) for 1 hour ($\equiv 15$ hours)

During the first season, the sand bank downstream of the nose of the spur tended to broaden, but when the flood rose to 4 feet over the causeway, the top of the island began to scour and the tendency was for the island to move downstream. Approximately stable conditions were attained after the 6th year—see figure 17. With this design, conditions steadily improved, so there is every reason to believe that if properly maintained, this design would give satisfactory results for a long period of years. Photo 1517—sheet No. 14—and figure 17 show conditions of flow with 6.28 cusecs (*i.e.* the maximum flood of the first season).

With the highest flood tested the surface water was deflected efficiently by the curved nose; and the main surface flow, after swinging round the nose, passed towards the left bank between C. S. 6 and 7, and then followed the left bank until it was deflected towards midstream by the groyne; and there was satisfactory normal forward flow across the full width of the River at all stages.

It was also found that surface velocities observed on the ferry cross section at the end of the 9th season—shown in figure 19—were better than during the first season—see figure 18.

(4) The proposed foundation levels of the spur are based on scour after 6 years of heavy floods—*vide* longitudinal section along the spur shown in figure 20. It is essential that the foundation of the spur must cut into the lime kankar ridge to a minimum depth of 3 feet and the trench round the spur should be filled with concrete; and if the lime kankar ridge should, in the course of time, become scoured, the toe of the spur may have to be protected.

If this design be adopted, scour at the nose must be watched and after some years the nose may have to be under-pinned. This will present no difficulties.

(5) An alternative to the Burma spur design would be to raise the existing causeway to such a height that it would only be topped in maximum floods, which occur at long intervals and only continue for a short time. The ferry could then be dispensed with altogether.

FIG:18, MULA RIVER ExPT:16, WITH A BURMA TYPE - SPUR. ISOTACHS AT C.S. 2 (FERRY) 1ST YEAR.

Q = 6.28 CUSECS. SCALE 1" = 1'

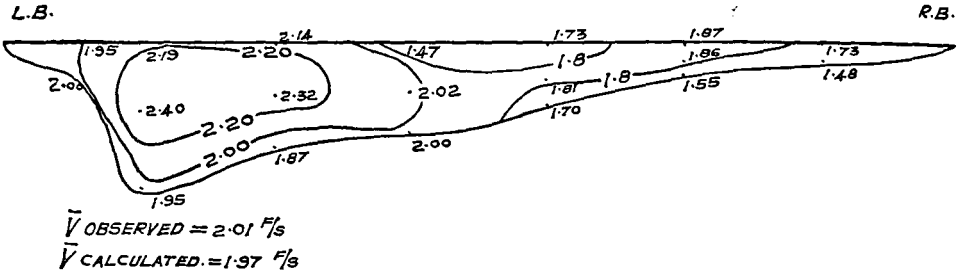


FIG:19, ExPT:16, VELOCITIES WITH MAX: FLOOD OF THE 9TH YEAR.

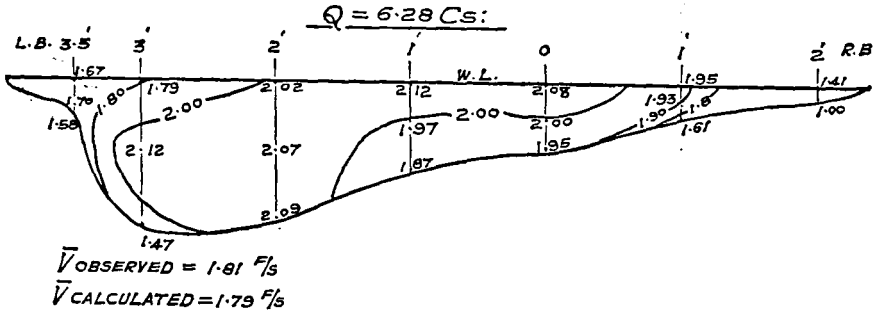
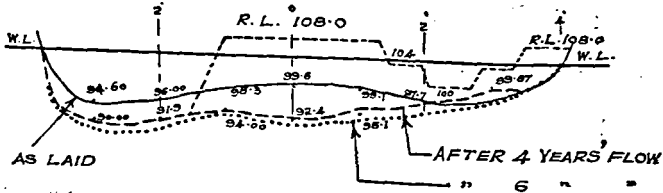
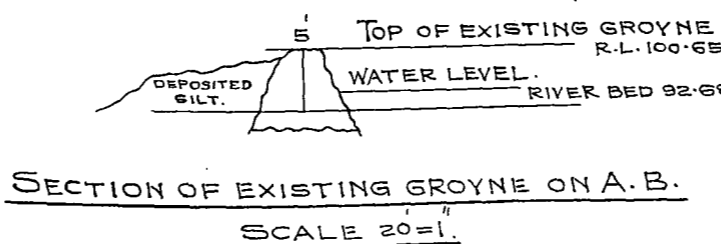
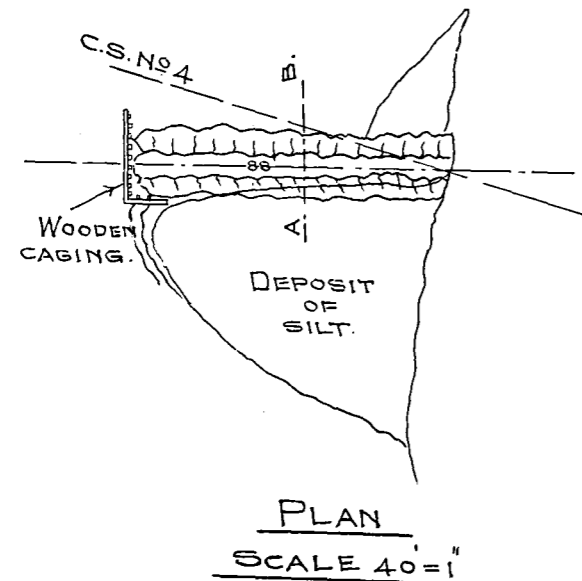
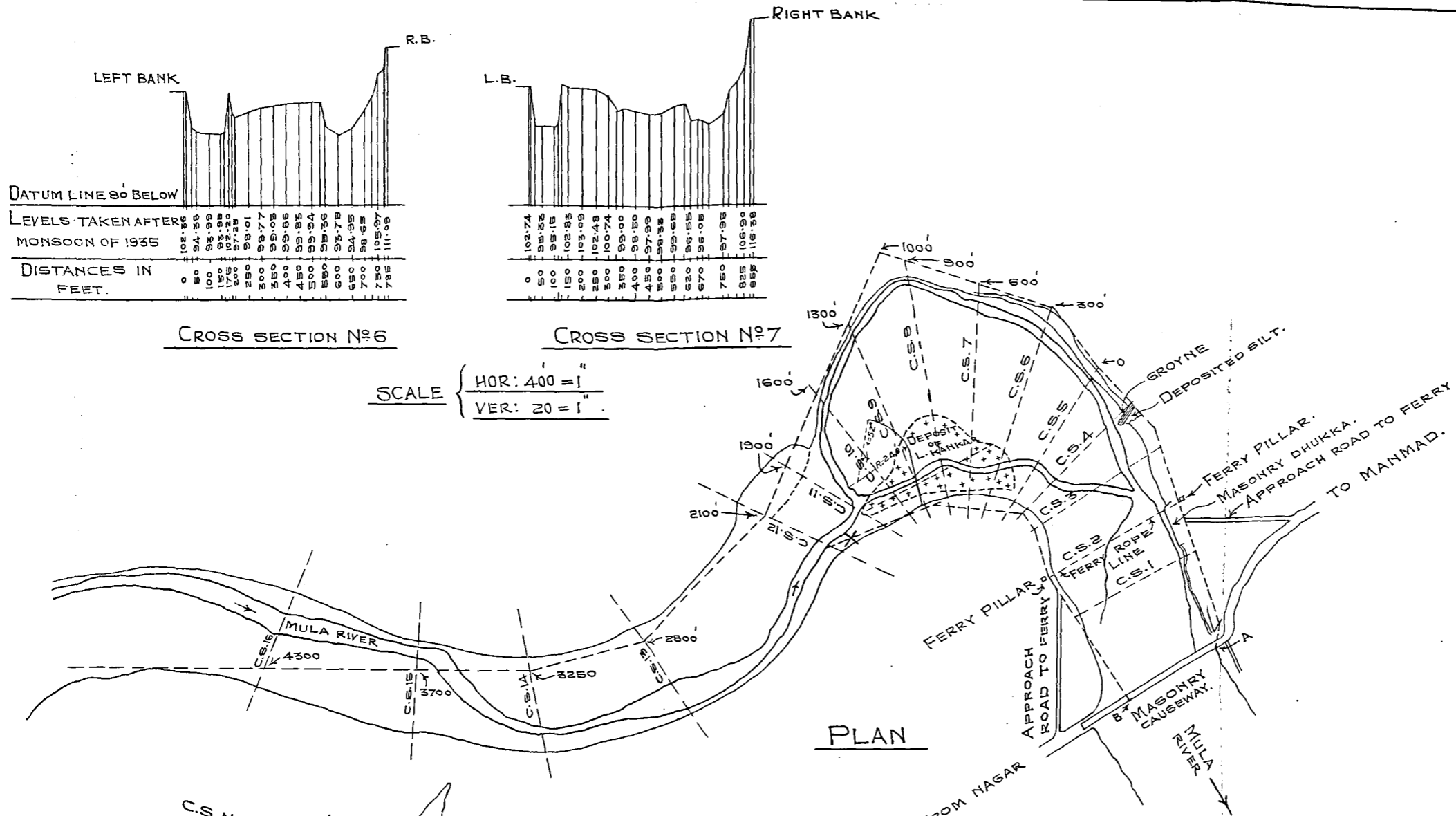


FIG: 20, CROSS SECTION. 10

SCALE 2" = 1'





PLAN AND SECTION OF EXISTING GROUYNE NEAR C.S. No 4.

FIG. 13.

IRRIGATION RESEARCH DIVISION
POONA
MULA RIVER AT RAHURI
PLAN SHOWING RIVER - (1936)
UPSTREAM OF CAUSEWAY.

SCALE: 400' = 1"

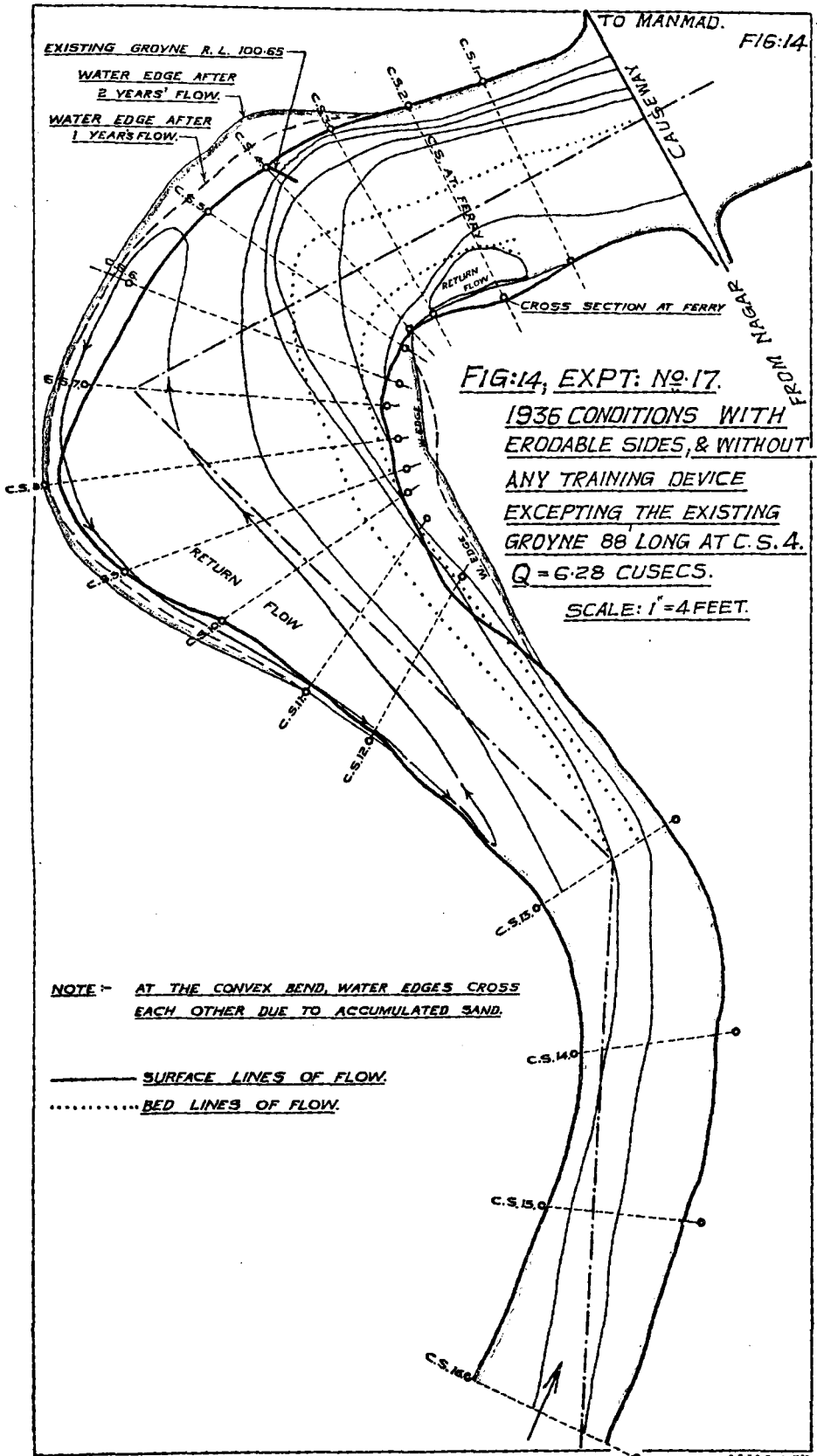
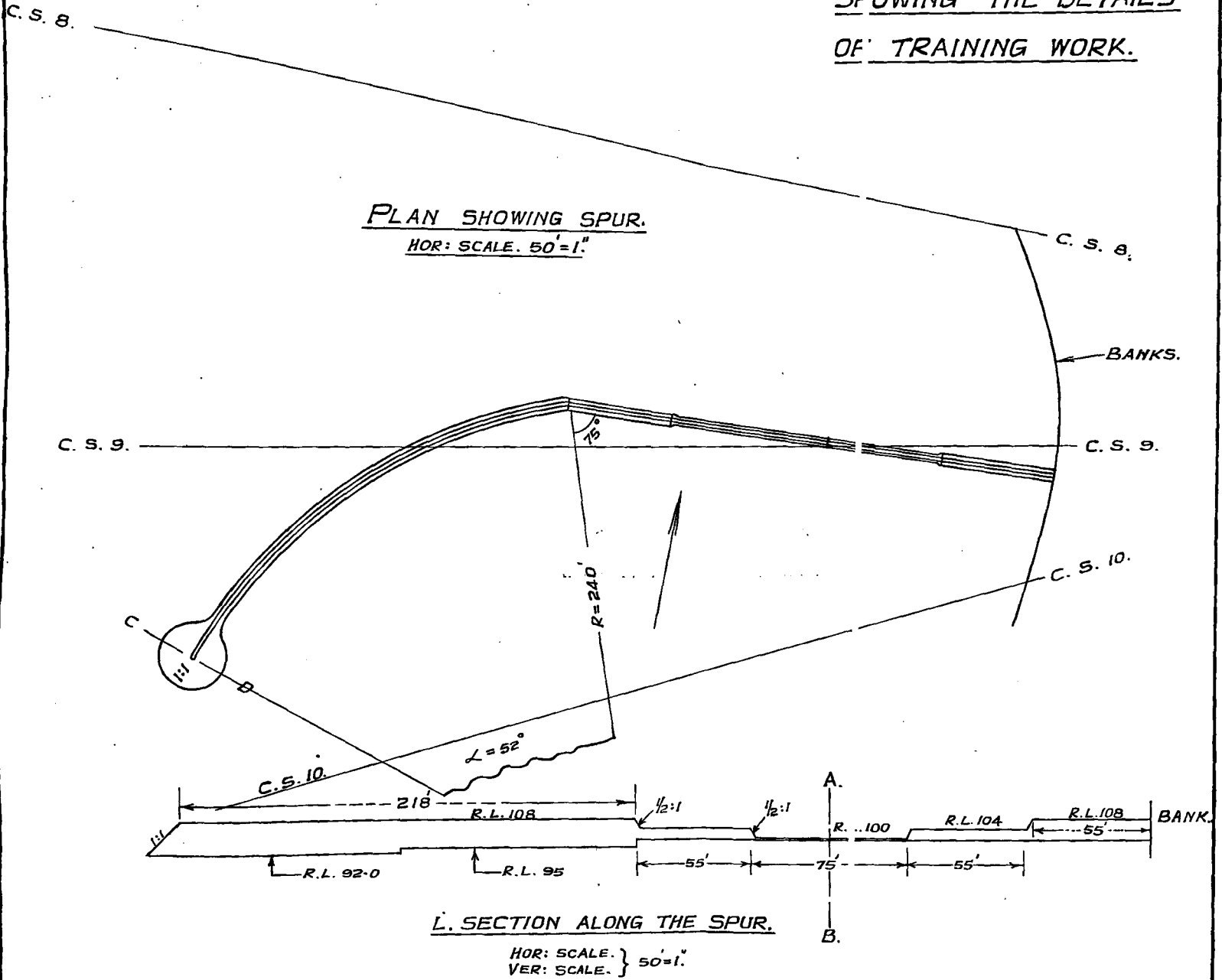
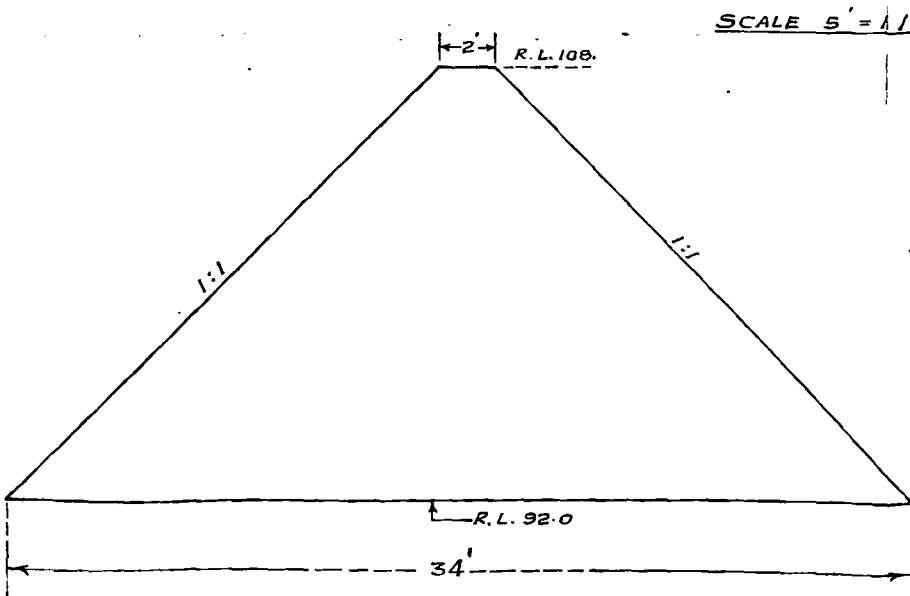


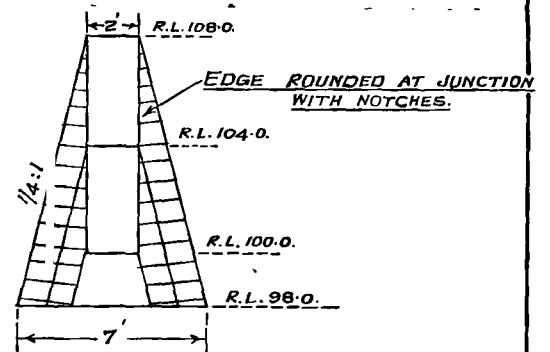
FIG:- 16,
M'ILA RIVER TRAINING
SHOWING THE DETAILS
OF TRAINING WORK.



C. S. C.D. AT NOSE.



C. S. A. B.



NOTE:-
WALL SHOULD BE PROVIDED 4 FEET BELOW
SURFACE OF LIME KANKAR AND TRENCH
EXCAVATED ROUND IT FILLED IN WITH
LARGE STONES OR LIME CONCRETE.

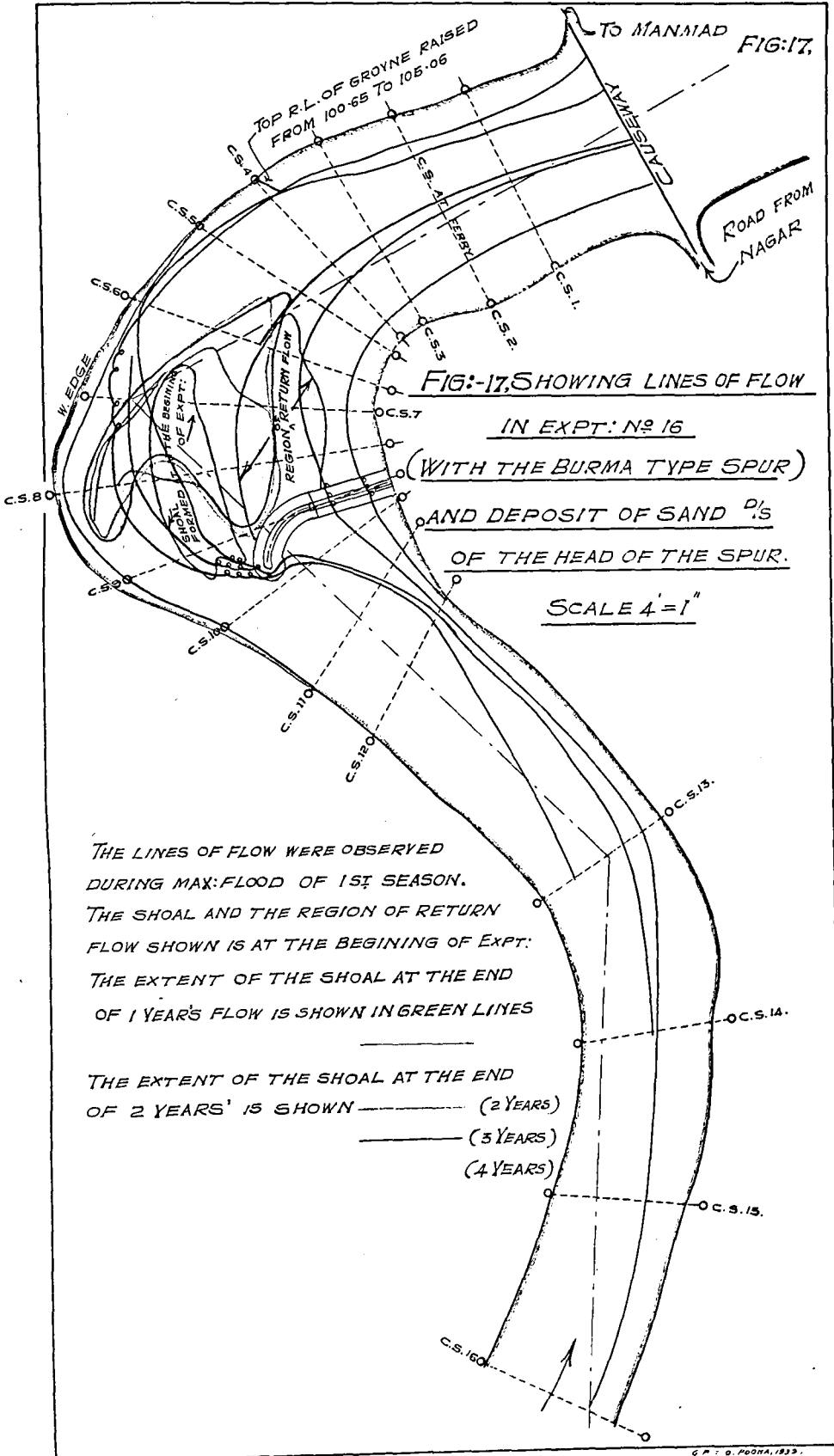


FIG:17,

FIG:-17, SHOWING LINES OF FLOW

IN EXPT.: N° 16

(WITH THE BURMA TYPE SPUR)

AND DEPOSIT OF SAND OF THE HEAD OF THE SPUR.

SCALE 4" = 1'

THE LINES OF FLOW WERE OBSERVED DURING MAX. FLOOD OF 1ST SEASON. THE SHOAL AND THE REGION OF RETURN FLOW SHOWN IS AT THE BEGINNING OF EXPT. THE EXTENT OF THE SHOAL AT THE END OF 1 YEAR'S FLOW IS SHOWN IN GREEN LINES

THE EXTENT OF THE SHOAL AT THE END OF 2 YEARS' IS SHOWN (2 YEARS)
 (3 YEARS)
 (4 YEARS)

(6) *Conclusions.*

There are two alternatives :—

- (i) A Burma spur projecting from the convex bank to deflect the main flow round the bend, so as to create satisfactory flow conditions at the Ferry.
- or (ii) Building a slab bridge on the causeway which would be open for traffic during normal floods; but would be overtopped for a few hours during very severe floods.

5. AFFLUX EXPERIMENTS TO DETERMINE THE EFFECT OF BELLMOUTHS ADDED UPSTREAM AND DOWNSTREAM AT THE TOP OF THE OPENINGS OF THE MULA RIVER CAUSEWAY AT RAHURI—DISTRICT AHMEDNAGAR.

(1) *Model.*

A part width model of the Mula River causeway was constructed, with two openings, a central pier and half piers at sides. The model was made geometrically similar to a scale ratio of $\frac{1}{10}$. The piers had cut-waters with equilateral arcs of circles and semi-circular ease-waters. The upstream and downstream faces of the top slab of the openings were vertical.

Each opening in the prototype was \equiv 20 feet wide and 3.5 feet high, with the top slab 2.75 feet thick. Figure 21 shows the plan and section of the model—see also photo 1390 (sheet No. 15).

(2) *Experiments.*

Upstream water levels were observed for various discharges :

- (i) with the present design,—see figure 21.
- (ii) with bellmouths (consisting of arcs of circles) added upstream and downstream.

The dividing line of the nose was at $\frac{2}{3}$ rd the depth of the slab from top, the radius of bottom arc being equal to 1.5 depth of slab and that of top arc = depth of slab—see photo 1389 (sheet No. 15) and sketch opposite.

- (iii) with semi-circular bellmouths added upstream and downstream—see photo 1394 and sketch opposite.

As the downstream water level is nearly always below modular limit for low and medium discharges and the relationships between discharges and the downstream water levels in the prototype were not known, the downstream water levels were maintained in each case at the modular limit.

Table I shows the upstream and downstream water levels for various discharges and Figure 22 shows the relationship between discharge per foot run and upstream depth over sill of causeway.

TABLE I: showing D_1 (=upstream depth over top of causeway) in model.
 d_1 (=upstream depth over sill of opening) in model.
 D_3 (=downstream depth over top of causeway at modular limit.)

	S. No.	q = discharge per ft. run in cusecs.	$\frac{D_1}{d_1}$			D_1 in ft.			Afflux (ft.)		D_3 at M. L.			$D_1 - D_3 =$ Loss of head at M. L.		
			Design No. 1.	Design No. 2.	Design No. 3.	Design No. 1.	Design No. 2.	Design No. 3.	Difference of		Design No. 1.	Design No. 2.	Design No. 3.	Design No. 1.	Design No. 2.	Design No. 3.
									1 & 2.	1 & 3.						
<i>Design No. 1 :-</i> Present design with square faces.	1	0.85	0	0.42	0.225	0	0.185	0.115	0.185	0.115	0.200	0.282	0.233	0.200	0.097	0.118
<i>Design No. 2 :-</i> With curved bellmouth U/S & D/S (radius of top arc = depth of slab & 1.5 depth of slab for bottom arc).	2	1.45	0.265	0.144	0.194	0.225	0.105	0.150	0.120	0.075	0.03	0.187	0.083	0.255	0.202	0.233
<i>Design No. 3 :-</i> With semi-circular bellmouth U/S and D/S.	3	2.46	0.444	0.421	0.416	0.500	0.457	0.447	0.043	0.053	0.130	0.143	0.220	0.370	0.314	0.227
	4	3.16	0.520	0.502	0.500	0.675	0.629	0.625	0.046	0.050	0.305	0.315	0.312	0.370	0.314	0.313
	5	4.04	0.596	0.572	0.568	0.925	0.838	0.825	0.087	0.100	0.605	0.505	0.506	0.320	0.333	0.310

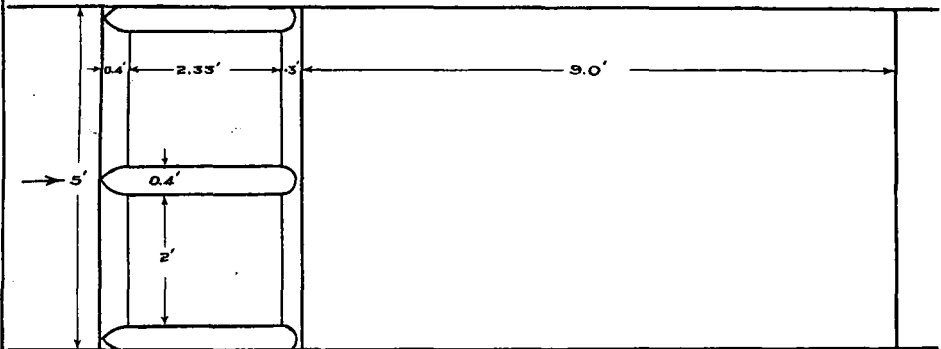
Model was geometrically similar to a scale ratio of 1/10.
 Height of opening = 0.35 ft. (\cong 3.5 ft.)
 Top of slab from sill of opening = 0.625 ft. (\cong 6.25 ft.)

Fig:21.- Part width model of Mula River Causeway.

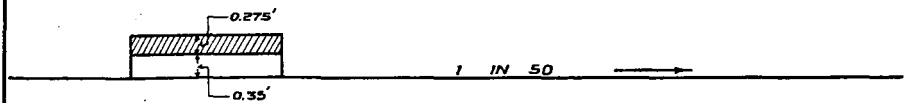
Present design with faces of slabs at right angles to flow.

Scale ratio of model = 1/10.

Scale :- 2 ft. = 1 inch.



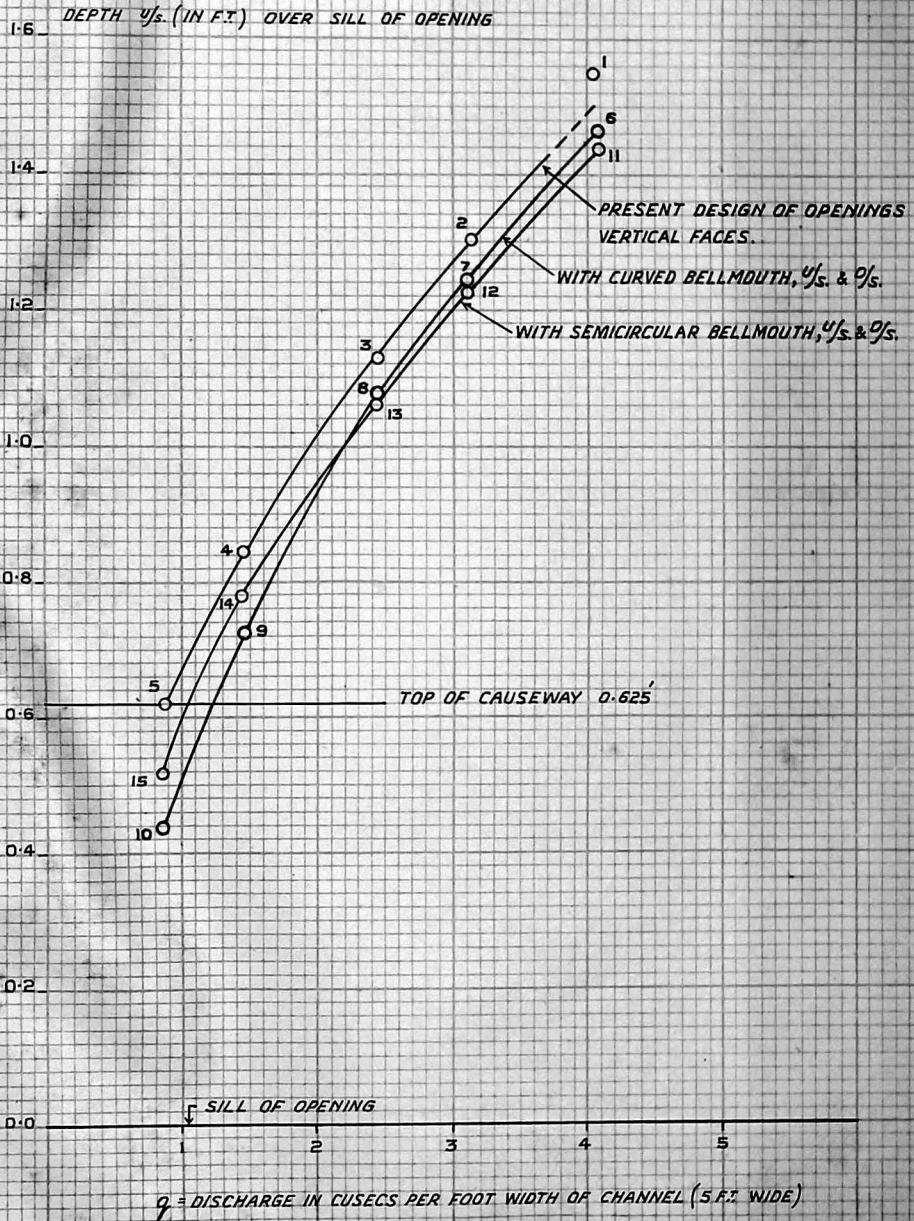
Plan



Section

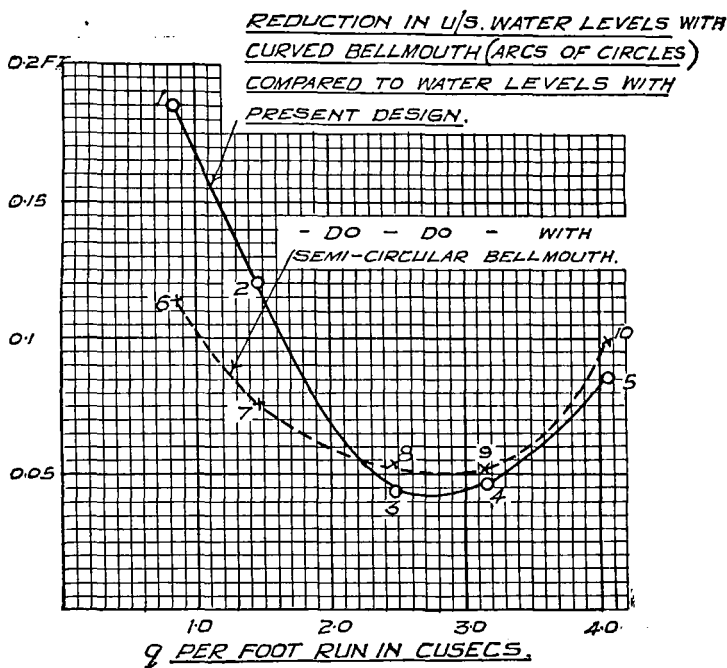
FIG. 22,

GRAPH SHOWING RELATIONSHIP
BETWEEN DISCHARGE PER FOOT
RUN AND y/s DEPTH OVER
SILL OF CAUSEWAY.



Of the two designs, the curved bellmouth was best for low discharges; but the semi-circular bellmouth gave better results for high discharges and hence is slightly better—see figure 23.

FIGURE - 23



6. SUPPLEMENTARY EXPERIMENTS WITH A MODEL OF THE SINA RIVER AND CAUSEWAY AT MILE 5 OF THE AHMEDNAGAR-MANMAD ROAD.

Experiments 1 to 9 have been described in the 1936-37 Annual Note and the Summary; and the conclusions and recommendations were:

The experiments have clearly shown that the erosive action on the right bank is mainly due to the Shendi Nala flood flowing across with high velocities to the right bank upstream of the causeway. The two masonry groynes—which were meant to deflect the high velocity flow towards the middle of the causeway—were not effective. The intensity of attack on the right bank of the causeway was very effectively checked by drawing the high velocity top water from the outside of the left concave bank of the Shendi Nala by a cut leading towards the left bank of the causeway. The entrance to the cut should be widened so as to form a convergence, mainly on the right (island) side. The cut would be 755 feet long, 70 feet wide at bottom and 7.5 feet deep.

It is not necessary to remove the island, but the whole of the land will have to be acquired as the island will eventually be scoured away and experience shows that it is wise to acquire the whole area in the first instance. The convex protrusion on the left bank downstream of the cut should be removed. The masonry groynes in the Shendi Nala should also be removed because they tend to draw the current across to the right.

No lengthening of the causeway is necessary; but if desired, when the cut has been made, the wing wall on the right bank near the causeway may be joined up to the right bank at its upstream end, by a fence of stakes and brushwood to induce silting behind.

The experiments were continued this year to see the effect of:

- (i) extending the flank wall upstream to the right bank ;
- or (ii) removing the flank wall.

The scale ratios of the model, based on Lacey's formulae, were as before :—

Horizontal scale ratio=1/70

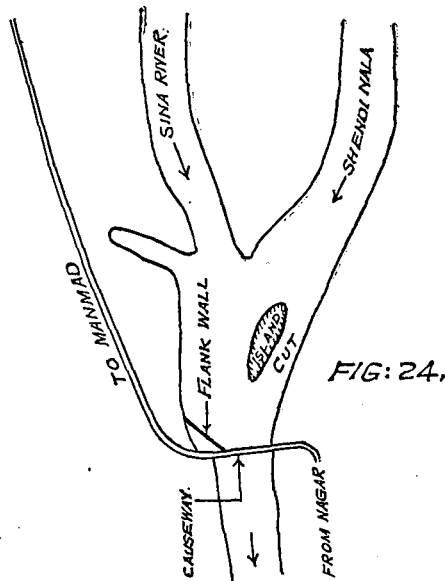
Vertical scale ratio=1/12.6

Discharge scale ratio=1/3896

Vertical scale exaggeration=5.55

Discharges through the Sina river and the Shendi Nala were equivalent to 6,350 cusecs in each, with water level at causeway \cong 105.6.

In experiment 10, the flank wall was extended as shown in figure 24 in plan.



The high velocity flow was deflected from the right bank above the causeway towards the left bank below the causeway and the isotachs were distorted as shown in figure 25 on a cross section \cong 35 feet upstream of the causeway and further out-flanking was probable.

In experiment 11, the flank wall was removed and the velocities in the wider section were correspondingly reduced, resulting in less scour and better distribution of velocities as compared to those in experiment 10—see figure 26.

In experiment 12, the roadway on the right bank was raised above flood level and a bank constructed along the right bank of the River downstream of the causeway, with top above H. F. L., so that the land downstream of the road was not submerged—*vide* photo 1524 and figure 29 on sheet 16. Figures 27 and 28 show isotachs and lines of flow respectively.

The distribution of velocities was much more normal than in experiment 11.

The surface flow through the cut, after passing over the causeway, flowed towards the right bank ; but the bed flow was even and normal to the causeway.

Recommendations.

In addition to the construction of the cut and removal of the groynes in the Shendi Nala, the flank wall should be removed and the roadway on the right bank should be raised above H. F. L. Though the cut in the model had an equivalent bed width of 70 feet, in actual practice, the cut may be say 30 feet wide, but the River should, from time to time, be assisted to widen the cut.

FIGURES: 25 TO 27 SHOWING ISOTACHS FOR A
C.S. 6" $\frac{4}{5}$ OF THE CAUSEWAY (≈ 35 FEET.)

DISCHARGE THROUGH SHENDI NALA = 1.63 Cs. (≈ 6350 Cs.)

DISCHARGE THROUGH SINA RIVER = 1.63 Cs. (≈ 6350 Cs.)

FIG: 25, EXPT. 10. WITH CUT ON LEFT BANK & FLANK WALL RAISED ABOVE
H.F.L. & EXTENDED TO MEET RIGHT BANK

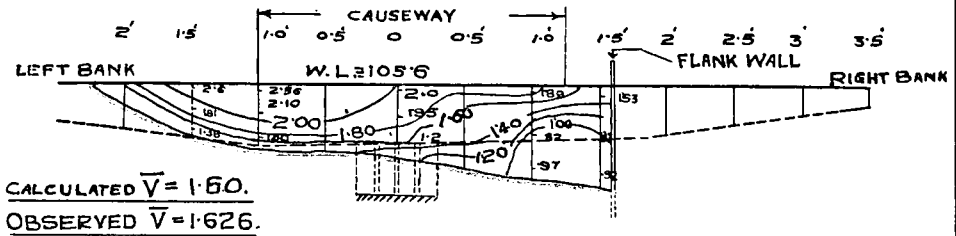


FIG: 26. EXPT. 11. (WITH CUT ON LEFT BANK AND
FLANK WALL REMOVED.)

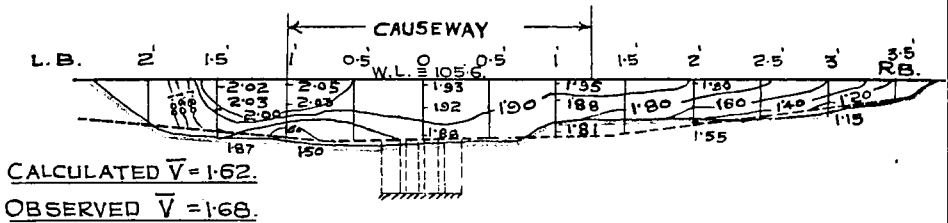
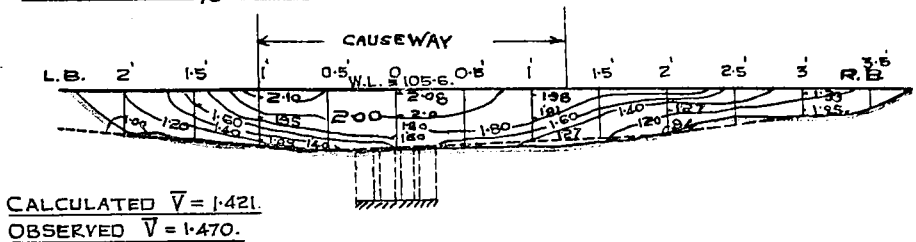


FIG: 27, EXPT. 12. WITH CUT ON LEFT BANK. FLANK WALL
REMOVED. ROADWAY ON RIGHT BANK AND RIGHT BANK OF
THE RIVER $\frac{D}{5}$ OF CAUSEWAY RAISED ABOVE H.F.L.



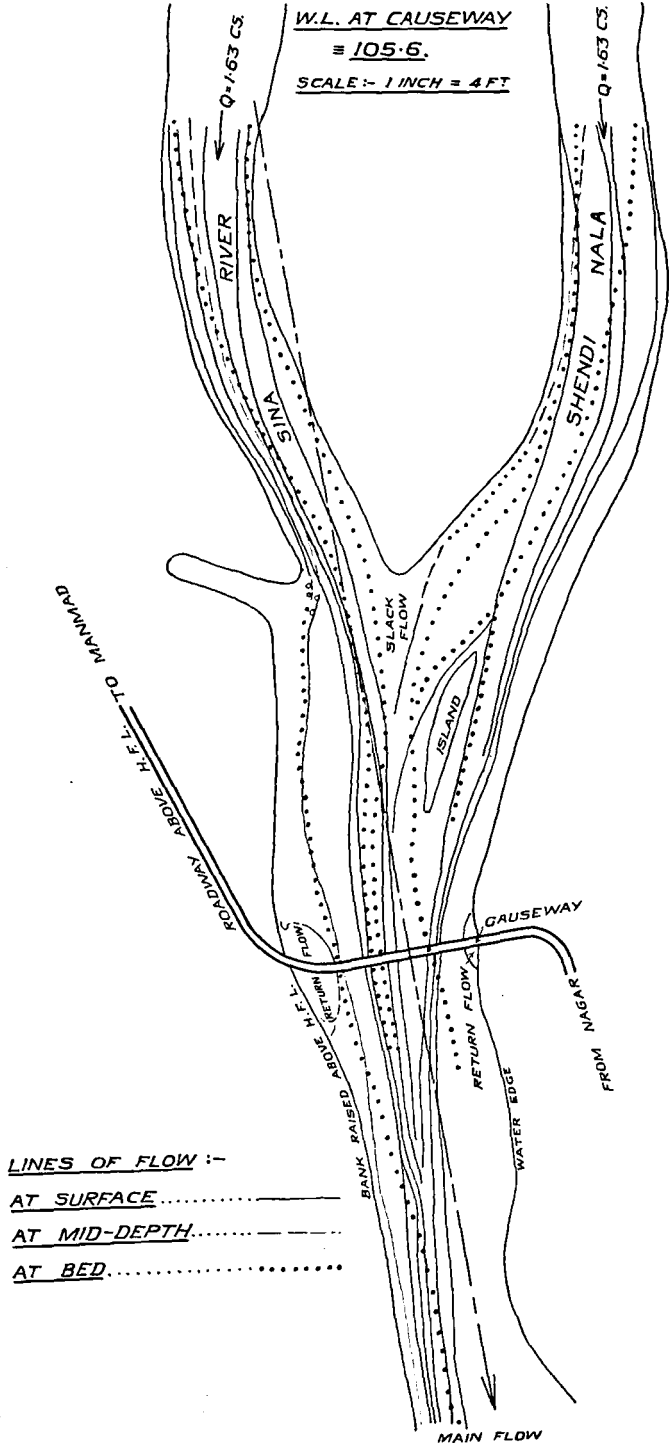
SCALE: 1" = 2 FEET.

FIG: 28, SHOWING LINES OF FLOW IN EXPT N^o 12.

WITH CUT ON LEFT BANK $\frac{1}{3}$ OF CAUSEWAY, FLANK WALL REMOVED;
ROADWAY ON RIGHT BANK & RIGHT BANK OF THE RIVER $\frac{2}{3}$ OF
CAUSEWAY RAISED ABOVE H. F. L.

DISCHARGE THROUGH SHENDI NALA = 1.63 CS. (= 6350 CS.)

" " SINA RIVER = 1.63 CS. (= 6350 CS.)



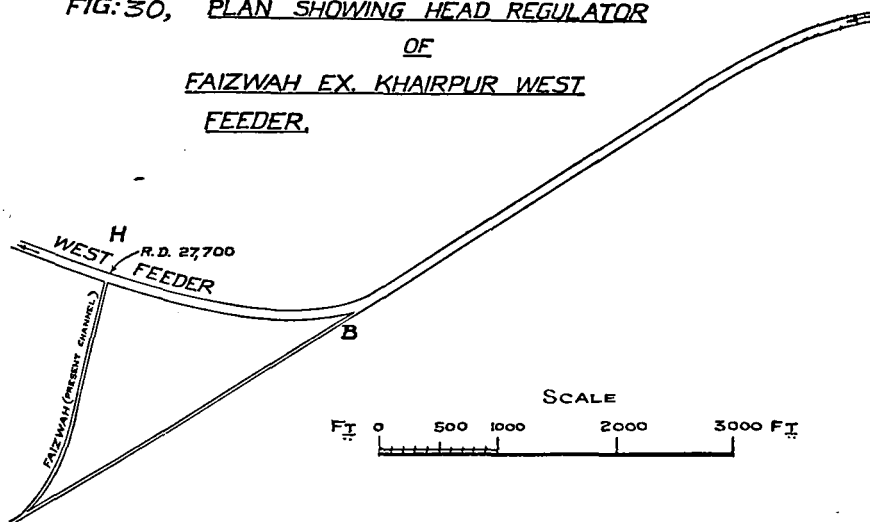
LINES OF FLOW :-
AT SURFACE.....
AT MID-DEPTH.....
AT BED.....

7. SILT EXCLUSION EXPERIMENTS WITH A MODEL OF FAIZWAH HEAD REGULATOR—EX WEST FEEDER (KHAIRPUR STATE, SIND).

Introduction.

The designed discharge of the Faizwah, which takes off from the Khairpur Feeder West at R. D. 27,700 (see figure 30) is 310 cusecs; and the discharge of the parent channel is 1836 cusecs.

**FIG. 30, PLAN SHOWING HEAD REGULATOR
OF
FAIZWAH EX. KHAIRPUR WEST
FEEDER.**



The head regulator of the Faizwah was constructed at H, taking off at right angles from downstream of a curve, and consequently the head reach silted rapidly.

On 22nd August 1936 a new head was opened at B at R. D. 26,200, using an old abandoned channel on the Left Bank. This was closed at the end of the season and silt clearance done in the silted Faizwah channel below its Head Regulator.

In 1936 the discharge of the Khairpur Feeder West varied between 600 cusecs and 1700 cusecs.

In May and June 1937 the discharge was about 800 cusecs rising to 1450 cusecs in July. From 20th August it ran with full discharge.

The silting of the Faizwah Head Regulator started again in 1937 and the new mouth "B" had to be re-opened on 22nd July 1937. Early in August the new head also began to silt slightly, due to a supercharge of silt in the parent channel, and water had to be headed up at Rameja Regulator, 2 miles downstream, by 0.4 to 0.5 feet, but the silting only amounted to a couple of inches and ceased in the 3rd week of August, after which the discharge slightly improved.

Table 2 shows the designed and Lacey dimensions for $Q=1836$ cusecs and actual and Lacey dimensions for various discharges, and table 3 shows similar information for Faizwah.

Discharges up to 1980 cusecs were observed in 1937 and for the period from 21st August to 2nd September, the average observed discharge was about 1875 cusecs. There is evidence, however, to show that, except as a result of purely temporary fluctuations, the actual discharge did not exceed the designed discharge, and that, as a result of running low discharges, the natural 'discharge' of the channel was little in excess of 1450 cusecs, the hydraulic depth (R) being greater than Lacey and width (P) less, due to fluctuating discharges.

Three alternatives were suggested by the State Engineer to set matters right :—

- (1) A new head at R. D. 34,200, downstream of H. This may be ruled out at once.
- (2) A new Head Regulator at B—1500 feet upstream of the original Head Regulator, which would be effective, since it would take off just downstream of a concave bend. This would cost Rs. 4,000.
- or (3) To remodel the present head regulator which originally cost Rs. 3,642.

TABLE 2.

Showing designed dimensions, Lacey dimensions and actual dimensions with various discharges on Khairpur West Feeder.

	Actual.	Lacey.	Actual.	Lacey.	Actual.	Lacey.	Actual.	Lacey.	Designed.	Lacey f=0.69.	Remarks.
	Q=1,350 cs.	Q=1,350 cs.	Q=1,400 cs.	Q=1,400 cs.	Q=1,450 cs.	Q=1,450 cs.	Q=1,500 cs.	Q=1,500 cs.			
f	0.69	..	0.69	..	0.69	..	0.69	..	0.69	
Q (cusecs)	1,350 cs.	1,350 cs.	1,400	1,400	1,450	1,450	1,500	1,500	1,836	1,836	
P (ft.)	95	98	95	100	95	101.8	95	103	102	114	
A (Sq. ft.)	595	574	595	594	595	608	595	627	678	734	
R (ft.)	6.25	5.85	6.25	5.9	6.25	5.98	6.25	6.05	6.66	6.45	
S	$\frac{1}{10260}$	$\frac{1}{11040}$	$\frac{1}{10260}$	$\frac{1}{10800}$	$\frac{1}{10260}$	$\frac{1}{11170}$	$\frac{1}{10260}$	$\frac{1}{11230}$	$\frac{1}{10000}$	$\frac{1}{11600}$	
V (f/s)	2.27	2.35	2.35	2.36	2.44	2.38	2.52	2.39	2.71	2.48	
N _s	0.0232	0.0207	0.0224	0.0207	0.0216	0.0207	0.0209	0.0207	0.0206	0.0205	
d (depth) ft.	7.15	..	7.15	..	7.15	..	7.15	..	7.9	..	
b (bed width in ft.)	78	..	78	..	78	..	78	..	78	..	
of a semi-elliptical channel. $\left\{ \begin{array}{l} B_s = \text{surface width} \\ D_s = \text{maximum depth} \end{array} \right.$	96	..	96.8	..	98	..	100	..	111.0	
	..	7.85	..	7.87	..	7.88	..	8.04	..	8.4	
f ₁ (from $V=1.17 \sqrt{f_1 R}$)	0.60	0.69	0.646	0.69	0.695	0.69	0.744	0.69	0.81	0.69	
f ₂ (from $S = \frac{10^{-5} \times 54 f_2^{6/3}}{Q^{1/6}}$)	0.72	0.69	0.74	0.69	0.74	0.69	0.75	0.69	0.77	0.69	

TABLE 3.

Showing particulars of Faizwah—designed and actual.

	As designed.	Lacey dimensions with $f=0.60$.	Actual on 1st August 1937 at R. D. 3000' downstream of 'B'.	Actual on 3rd October 1937 at R. D. 3000' downstream of 'B'.
F. S. L.	190.50
Q (cusecs)	310	310	264	324
P (ft.)	47.2	46.9	48	50
A (sq. ft.)	155	168.1	150.5	181.75
R (ft.)	3.28	3.58	3.14	3.04
S	$\frac{1}{9000}$	$\frac{1}{8630}$	$\frac{1}{7129}$	$\frac{1}{8571}$
V (f/s)	2.0	1.84	1.75	1.78
d (depth in ft.)	5.0	..	4.1	4.6
b (bed width in ft.)	26	..	33	35
$f_1 (=0.7305 V^2/R)$	0.72	0.654
$f_2 (f_1 = 10^6 R^{-1/2} S/370.68)$	0.765	0.712
$f_3 (f_3 = \frac{1/4 N_a}{0.0225})$	0.818	0.833
$f_{SQ} (f_{SQ} = \frac{5/3}{1780 S Q^{1/6}})$	0.763	0.696
$N_s (= \frac{1.3458}{V} R^{3/4} S^{1/2})$	0.0214	0.0215

It was pointed out that if the present Regulator could be so modified as to exclude silt, alternative (3) would be cheaper than spending Rs. 4,000 on a new regulator at B—though the latter would be a safer solution.

The object of these experiments, therefore, was to determine how to modify the present Head Regulator at H to exclude silt effectively.

Three conditions were considered:

- (a) normal conditions
- (b) when there was an excess silt charge in the West Feeder
- and (c) when the proportion of the total discharge in the West Feeder decreased and water levels were maintained by heading up the water level at Rameja Regulator.

Design of model.

3.8 cusecs in the model corresponded to 1836 cusecs in the prototype and the horizontal and vertical scale ratios worked out to 1/25.4 and 1/8.00, as shown in table 4.

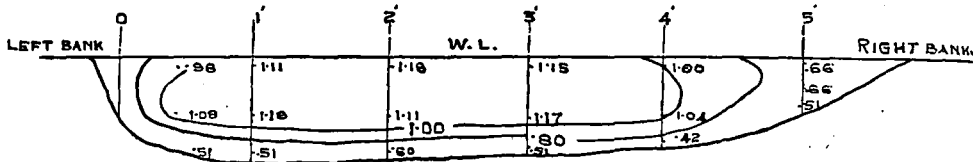
TABLE 4.

Channel dimensions.	K. W. F. as designed (Lacey).	K. W. F. model (Lacey).	Remarks.
Q	1,836 cs.	3.8 cs.	Q in K. W. F. u/s of Faizwah head = 3.8 cusecs. Q in Faizwah = 0.64 cs.
f	0.60	1.25	
P	114.0 ft.	5.20 ft.	
V	2.48	1.076 f/s	
R	6.45	0.678 ft.	
A	734 s. ft.	3.53 s. ft.	
B _s	111 ft.	4.397 ft.	
D _s	8.4 ft.	1.05	
Reynold's number	..	73,000	
H. scale	..	1/25.4	
V. scale	..	1/8.00	The model was built finally to a scale of 1/25 horizontal and 1/8 vertical.

Model allowed to modify itself under natural flow.

In the model built to the scales shown in table 4, a natural section was allowed to establish itself for a discharge of 3.80 by running the model for 40 hours. Figure 31 shows this section and the velocity-distribution in it at a distance 2 ft. upstream of the Faizwah head (\cong 50 ft. in the prototype).

**FIGURE. No. 31,
CROSS SECTION AT 2' U/S OF FAIZWAH.**



$$\frac{V_m}{V} \text{ (CALCULATED)} = 1 \frac{F}{S}$$

$$\frac{V_m}{V} \text{ (OBSERVED)} = 1 \frac{F}{S}$$

**NORMAL SECTION OF K.W.F.
CHANNEL IN THE MODEL
SCALE: 1" TO 1'**

TABLE 5.

	Model (Lacey).	Model actual.	Difference.
	$f=1.25$		Per cent.
Q	3.8	3.8	..
P	5.2	6.5	+25
R	0.68	0.585	-14
V	1.076	1.000	-8
A	3.53	3.8	+7
B _c	4.37	5.24	+20
S	0.000649	0.000671	+3½
q73	0.585	-20
'C ₁ ' in $V=C_1 \sqrt{R^{1/3}}$	16.0	16.31	+2
'C ₂ ' in $P=C_2 \sqrt{Q}$	2.67	3.33	+25
'C ₃ ' in $P/R=C_3 V$	7.12	11.11	+56
$f_{SQ} (f_{SQ}^{5/3} = 1,780 S Q^{1/6})$		1.275	
$f_4 (=0.4 \sqrt{d})$		0.917	
$f_1 = 0.7305 \frac{V^2}{R}$		1.249	
$f_2 (f_2 = 10^6 R^{1/2} S/370.68)$		1.244	
$f_{Na} (f_{Na} = \frac{N_a}{0.225})$	1.252	1.20	
$N_a (= \frac{1.3458}{V} R^{3/4} S^{1/2})$	0.238	0.0233	
$f_m (= \sqrt{f_1 f_2})$		1.246	
$C_L (= \frac{V}{16.02 R^{2/3} S^{1/3}})$		1.002	

The P and R values diverged widely from Lacey figures indicating that either these formulae did not hold for such small channels or else the divergence was due to natural berms not having had time to form. What was very marked was the closest agreement with the regime test formula $V=16 \sqrt[3]{R^2 S}$, showing that the channel had adjusted its slope; and f_{Na} , f_{SQ} , f_1 and f_2 all agreed closely; but $f_4 (=0.4 \sqrt{d})$ gave a value 26 per cent lower than f_1 and f_2 .

Experiment with the existing design.

Experiment 1.—The first experiment was carried out with the head regulator of the Faizwah as originally constructed, a discharge of 3.80 cusecs, corresponding to 1836 cusecs in the prototype, and the water level maintained at R. L. 190.96 (F. S. L.) opposite Faizwah head in the Khairpur Feeder. Silt was injected. Lines of flow and velocity observations were taken after 25 hours flow, and the amount of silt collected from the Khairpur Feeder and Faizwah measured.

Figure 32 shows lines of flow.

The silt ratio of Faizwah to Khairpur West Feeder was 10 to 1.

This explains the heavy silting of the Faizwah, due to faulty location of the Regulator below a bend.

Velocity observations at chainage 0 ; 2 ft. and 4 ft. upstream of the Regulator and 2 ft. downstream showed high velocities at the surface and low velocities at the bed of Khairpur Feeder, which is highly favourable to silting in the oftaking Faizwah channel.

In *experiment No. 2* the design was modified, as shown in figure 33, the left bank being warped to vertical just upstream of the Faizwah—from chainage 8.5 in the model,—the channel at F. S. L. being made to converge at 1 in 5. The downstream diverged at 1 in 10 at the surface and was warped back to natural side slope.

R. L. of raised sill is kept at R. L. 187.90, so that depth of water over this is nearly 60 per cent of that at F. S. L. in the Khairpur Feeder, and the glacis slope of the raised sill in the Faizwah is 1 in 5. Discharges in K. F. W. and Faizwah and the silt charge were as in experiment 1.

Figure 33 shows lines of flow at surface and bed.

Figure 34 shows the contour plan of the bed after 36 hours flow. The normal B. L.=R. L. 0.70. The deepest portion was just upstream of the upstream side of Regulator, which showed that curved flow had become established, as at the outside of a curve. This prevented excess silt entering the Faizwah.

Velocities near the sill were high. Silt at an off-take has a tendency to pile up at the entrance, but with this design silt was washed past the face of the regulator, due to high velocity.

Silt ratio of $\frac{\text{Faizwah}}{\text{Khairpur Feeder}}=0.7$

Experiment No. 3.—To see the effect of a supercharge of silt, silt was injected at 1.35 times the previous natural silt charge. At the end of a 42 hours run, lines of flow, velocities, and bed contours were observed.

This experiment showed that the percentage of bed water entering the Faizwah was slightly more than with a normal silt charge, due to a tendency for silt to form a slope upstream to sill level ; but downstream of this, there was a deep portion, so that immediately the excess silt charge ceased, the silt ramp was washed away—*vide* figure 35. Table 6 shows the divides at surface, mid-depth and bed in experiments 1, 2 and 3.

TABLE 6.

Divides from left bank at C. S. 12 ft. u/s of Faizwah head.	Experiment 1.	Experiment 2.	Experiment 3.
	Present design.	Modified design with 1 in 5 convergence u/s and raised sill.	Same as experiment 2 but with silt charge =1.35 natural silt charge.
	Natural silt charge		
Surface	Nil	0.75'	0.75'
Mid-depth	0.9'	1.0'	1.0'
bed	1.8'	0.7'	0.75'

Silt ratio of $\frac{\text{Faizwah}}{\text{Khairpur Feeder}}=1.02$

This experiment showed that with a supercharge of silt, the silt ratio of silt charge in the Faizwah Regulator increased relative to the West Feeder from 0.7 to 1.02. Bearing in mind that the ratio was 10 in 1937, the marked silt excluding properties of the design are obvious. The channel bed in this experiment consisted of a series of waves, which is a typical indication of an excess silt charge.

Experiment to determine silt ratio when Faizwah was drawing F. S. Q.; but Khairpur Feeder was running with $\frac{2}{3}$ F. S. Q.=2.38 cusecs \equiv 1,147 cs.

For F. S. L. at Rameja, W. L. at Faizwah head is=R. L. 190.52. This was maintained in the model opposite Faizwah entrance. A normal silt charge was maintained.

The bed contours were much more uniform, the effect of upstream convergence being relatively small.

Silt ratio of $\frac{\text{Faizwah}}{\text{Khairpur}}=1.132$

As was to be expected, owing to the increased proportion of discharge entering the Faizwah channel—viz., 27 per cent against 17 per cent—a higher proportion of silt entered the Faizwah than in experiment 2; but as the silt charge in the Feeder would be low when the parent channel would be carrying only $\frac{2}{3}$ this discharge, this would not cause silting in the Faizwah.

Table 7 shows the silt charge ratios of all the experiments.

TABLE 7.

Design.	Silt charge ratio $\frac{\text{Faizwah}}{\text{K. F. W.}}$
(1) Existing design	10
(2) New design with normal silt charge	0.705
(3) New design with super silt charge (=1.35 normal silt charge).	1.02
(4) New design with $\frac{2}{3}$ F. S. Q. in K. F. W. and water headed up at Rameja to F. S. L. and W. L. opposite Faizwah=190.52.	*1.132

* [NOTE—As silt charge in West Feeder would be subnormal under such conditions, Faizwah could carry the silt charge.]

Conclusion.

Although this design of silt excluding head would be successful provided conditions in the West Feeder did not deteriorate; building a new regulator at B, 9,900 ft. upstream of Rameja regulator, would exclude more silt. As to whether this is desirable or not depends on local conditions.

8. EXPERIMENTS WITH A MODEL OF THE INDUS AND THE LLOYD BARRAGE AT SUKKUR.

As a result of model experiments carried out 8 years ago to ascertain whether "still pond" or "open flow" regulation would be preferable for excluding silt from the Sukkur Barrage Canals⁽²⁾, it was found that either method was successful for excluding coarse silt from the Left Bank Canals; but that 'still pond' regulation gave markedly better silt exclusion in the case of the Right Bank Canals. This was fully confirmed when the Canals were opened in 1931.

It was also predicted that a silt bank would form along the Right Bank; and, to prevent this, it was proposed that a long, submerged, curved vane—height = $\frac{1}{3}$ rd depth of water—should be constructed upstream of the Canals on the right bank starting from the outfall section. This, alone, would have tended to pull the main flow of the River across to the right bank, so a similar vane was proposed for the left bank, and model experiments showed that with these, channels would persist along both banks, bed silt being deflected towards mid-stream. This recommendation was not, however, acted on. In the floods of 1936, the River carried an exceptionally heavy charge of bed silt, and the predicted sand bank formed along the right bank from Bunder Station across to the nose of the Divide

(²) Bombay P. W. D., Technical Paper 45 "Note of May 1931 on Silt exclusion from Canals 'still pond' versus 'Open Flow'" by C. C. Inglis and Technical Paper 46 "Note on experiments with the full width model of the Sukkur Barrage" by C. C. Inglis and D. V. Joghtkar.

REFERENCES

- SURFACE LINES OF FLOW.
- - - MID-DEPTH.
- BED LINES.

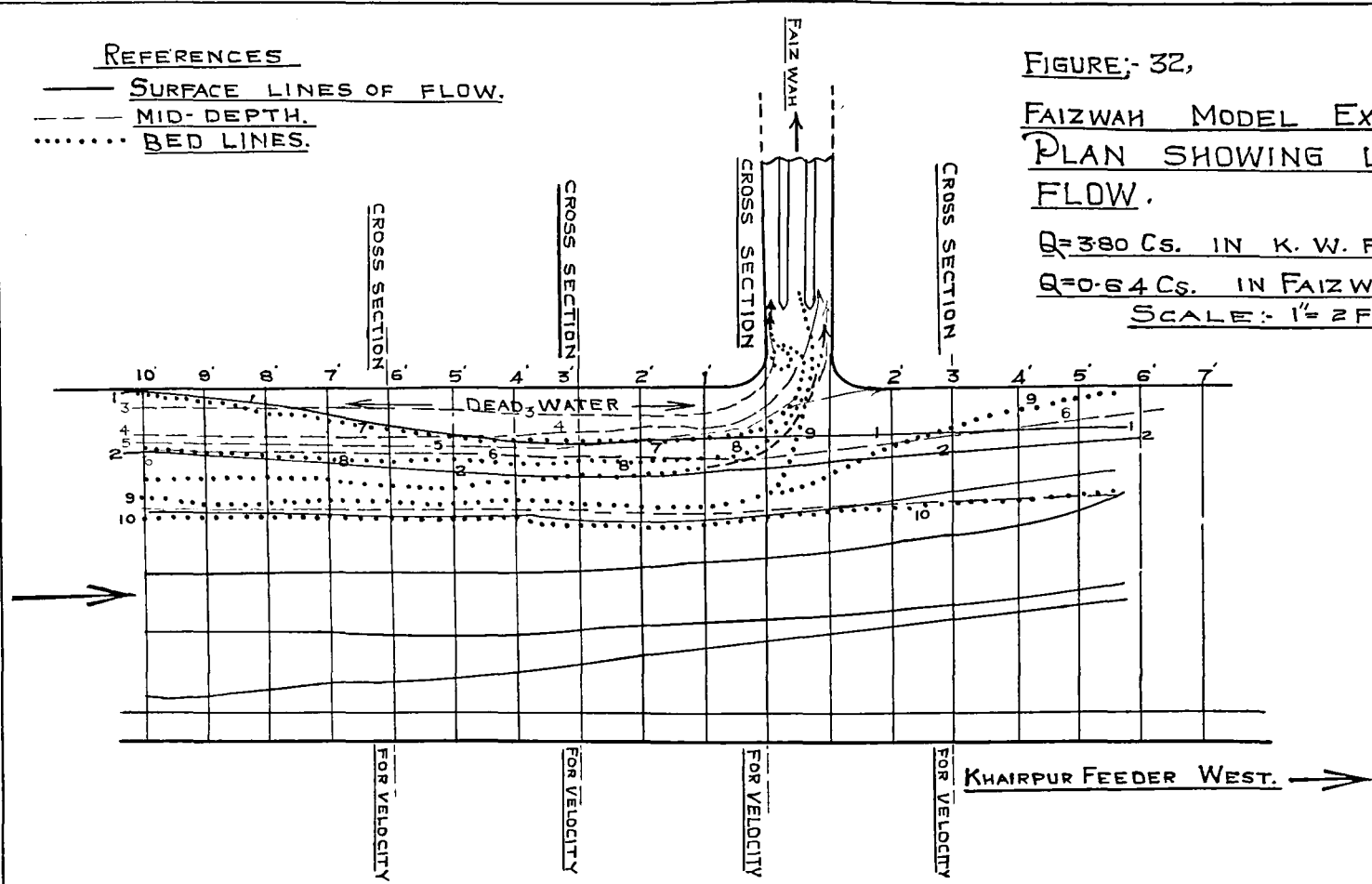
FIGURE:- 32,

FAIZWAH MODEL EXPERIMENTS.
PLAN SHOWING LINES OF FLOW.

$Q = 380$ Cs. IN K. W. F.

$Q = 0.64$ Cs. IN FAIZWAH.

SCALE:- 1" = 2 FEET.



Khairpur Feeder West. →

№ 971

REFERENCES

- SURFACE LINES OF FLOW.
- MID-DEPTH.
- BED LINES.
- BED LINES RISING TO MID-DEPTH.
- BED SEED.

SURFACE DIVIDE 0.75 FT AT 12 FT $\frac{4}{5}$.
MID-DEPTH DIVIDE 1.0 FT AT 12 FT $\frac{4}{5}$.
BED DIVIDE 0.7 FT AT 12 FT $\frac{4}{5}$.

CROSS SECTION FOR

FAIZWAH

VELOCITY

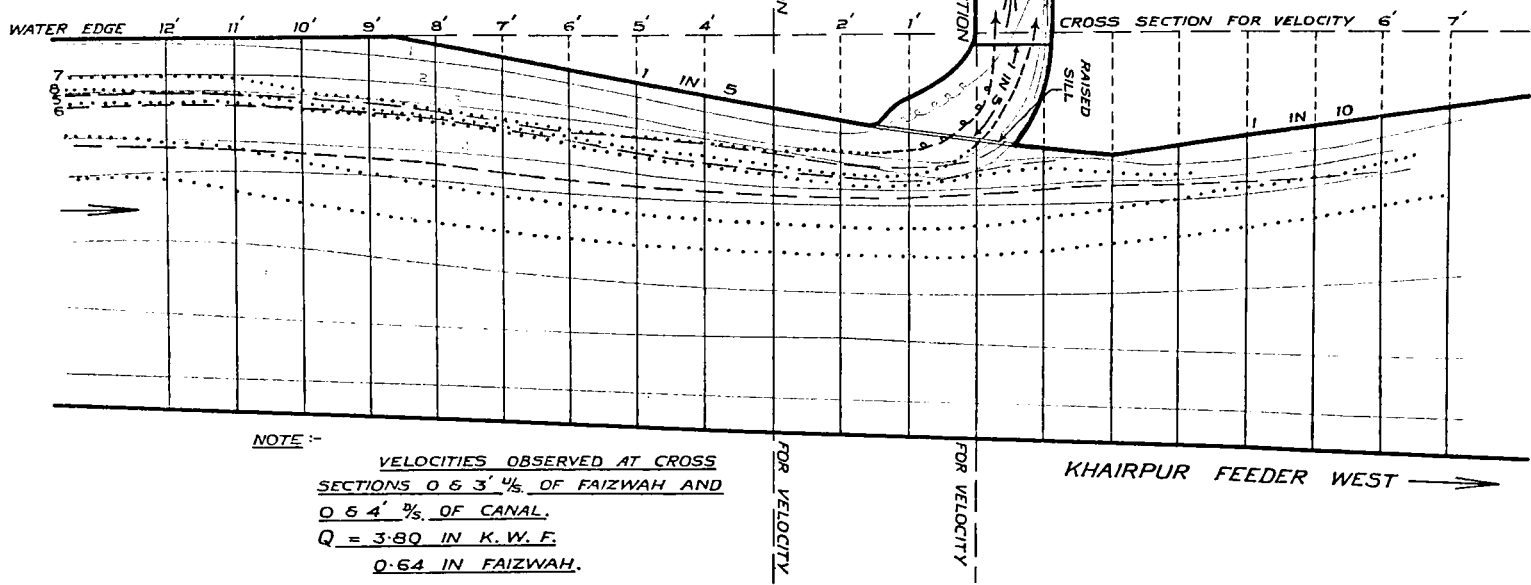
FIGURE: 33,

FAIZWAH MODEL EXPERIMENTS.
LINES OF FLOW OBSERVATIONS.
REMODELLED DESIGN.

SCALE :- 1 INCH = 2 FEET.

R. L. OF SILL = 187.90.

GLACIS SLOPE $\frac{1}{5}$ OF SILL IN
FAIZWAH = 1 IN 5.

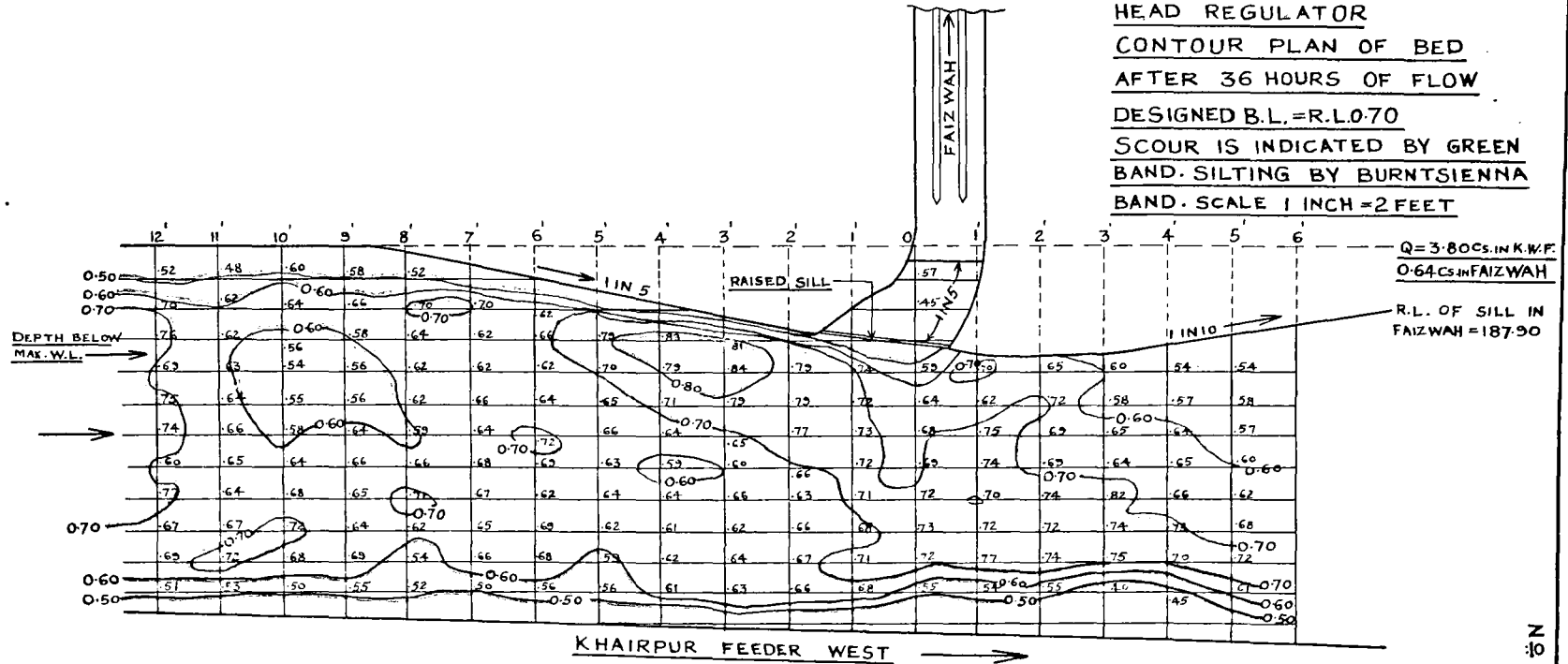


NOTE :-

VELOCITIES OBSERVED AT CROSS
SECTIONS 0 & 3' $\frac{4}{5}$ OF FAIZWAH AND
0 & 4' $\frac{3}{5}$ OF CANAL.
Q = 3.80 IN K.W.F.
0.64 IN FAIZWAH.

N: 863.

FIGURE:34,
EXPERIMENTS ON FAIZWAH
HEAD REGULATOR
CONTOUR PLAN OF BED
AFTER 36 HOURS OF FLOW
DESIGNED B.L.=R.L.0:70
SCOUR IS INDICATED BY GREEN
BAND. SILTING BY BURNTSIENNA
BAND. SCALE 1 INCH =2 FEET



N. 864

FIG:35,

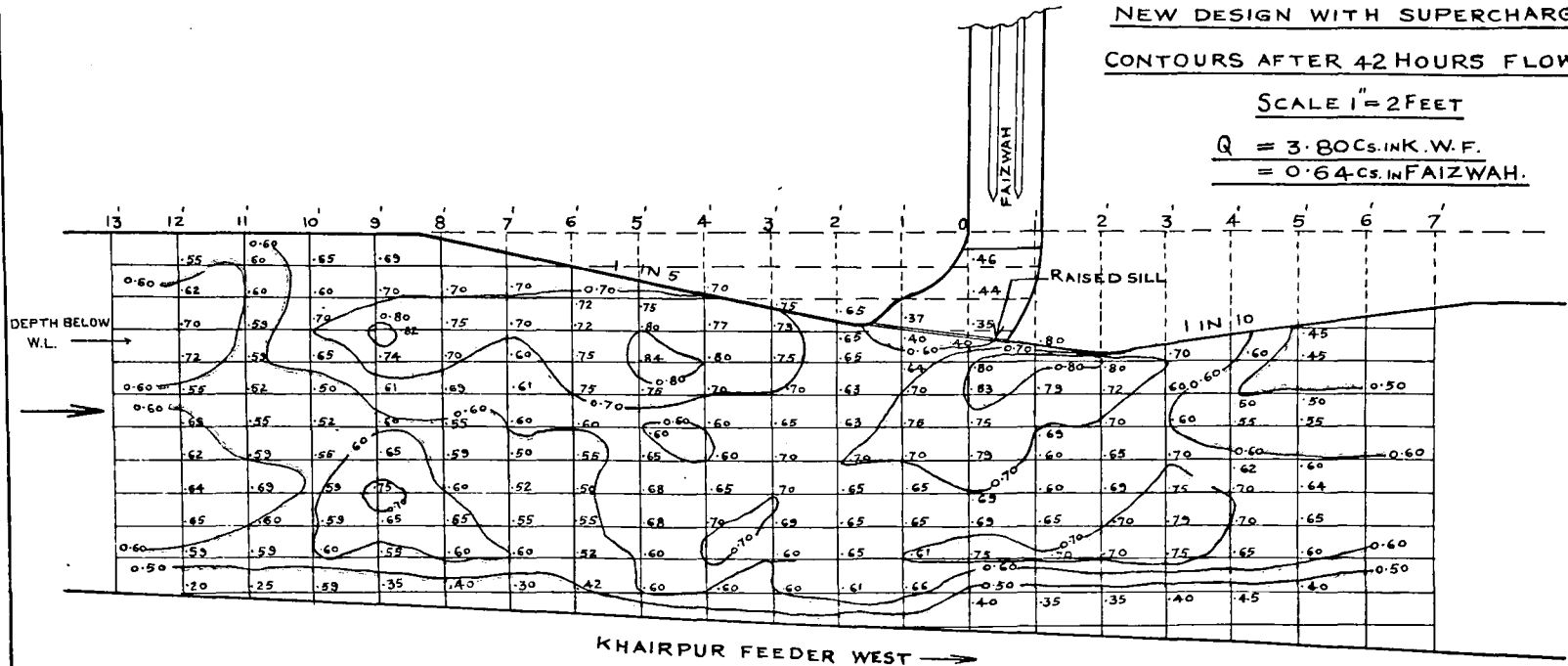
FAIZWAH MODEL EXPERIMENTS.
 NEW DESIGN WITH SUPERCHARGE OF SILT.

CONTOURS AFTER 42 HOURS FLOW.

SCALE 1" = 2 FEET

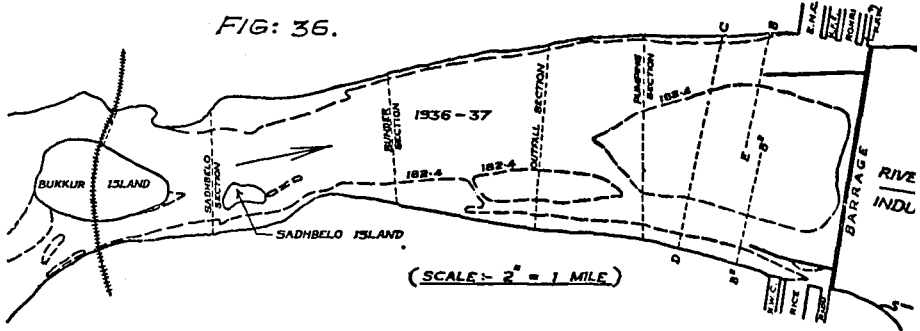
$$Q = 3.80 \text{ C.S. IN K. W.F.}$$

$$= 0.64 \text{ C.S. IN FAIZWAH.}$$



N:865

Wall of the Left Bank Canals—see figure 36; and silting occurred in the North West Perennial Canal. The original solution—a long vane—being no longer possible, experiments to find a new solution have been started.



The model is 170 ft. long representing 7 miles above and 3 miles below the Sukkur Barrage. Scale ratios, based on Lacey's formulæ,⁽¹⁾ are :—

Longitudinal and Latitudinal scale ratio = $\frac{1}{300}$

Depth scale ratio = $\frac{1}{36}$

Figure 37 shows 1930 and 1932 plans of the portion of the River Indus comprised in the model. These show the wide swing of the river above the gorge. In the gorge portion, between the towns of Sukkur and Rhoori rock is exposed during floods, and rigid, flume, conditions exist. Figure 38 shows rock contours in the gorge. Between the gorge and the Barrage, both banks of the River have been pitched with stone; and silt has since deposited along the banks.

This model presents the following difficulties :—

- (1) The swing of the River above the gorge affects results below the gorge.
- (2) In this model, conditions change from mobile conditions to rigid; and back again to mobile. For the mobile portions vertical exaggeration is natural; for the rigid portion different laws hold; but the greatest difficulty is due to the junctions between rigid and non-rigid portions.

Preliminary experiments are being done to see how far the model laid according to 1936 cold weather bed, reproduces 1937 conditions. It will later be decided what modifications can be made to improve results.

9. EXPERIMENTS WITH A MODEL OF PROPORTIONAL STANDING WAVE FLUME METER FALL AT R. D. 40280 MONTGOMERY—PAKPATTAN LINK (PUNJAB).

Notation.

- B_1 and B_2 = Bed width of channel upstream and downstream.
- B_c = Bed width of flume at contracted section (excluding piers).
- B_o = Bed width of flume at contracted section (including piers).
- d_1 and d_2 = depth of water in channel upstream and downstream of fall.
- $d_c = \sqrt[3]{\frac{q^2}{g}}$ = critical depth.
- d_z = depth of water at toe of fall (assuming that no standing wave has formed).
- d_z = depth of water in the cistern.
- H = Fall in water level *i.e.*, difference of water levels upstream and downstream of fall (below standing wave).
- h_b = Height of baffle.

Q	=Total discharge in cusecs.
q	=Discharge per foot width.
q _o	=Discharge per foot run of overall width of flume in the throat.
\bar{V}	=Mean velocity.
F. S. Q.	=Full supply discharge.

Design.

In accordance with the resolution passed in the Research Officers' meeting of the Central Board of Irrigation held at Lahore in February 1938, different designs of falls are to be tested on the Montgomery—Pakpattan Link.

A design for a proportional, Standing Wave, Flume, Meter (P. S. W. F. M.) Fall with baffle, cistern and deflector was supplied to the Director, Central Designs, Lahore—*vide* figure 39.

Data.

F. S. Q.	=700 cusecs.
H	=6.63 ft.
d ₁ and d ₃	=5.1 ft.
B ₁ and B ₃	=60 ft.
Side slope of channel	= $\frac{1}{2}$: 1
\bar{V}_1 and \bar{V}_3	=2.19 f/s.
Full supply level upstream	=527.69
„ downstream	=521.06
Bed level upstream	=522.59
„ downstream	=521.06

As the ratio of B₃/d₃=11.76, the standard side divergence⁽³⁾=1 in 9.2; but side divergence adopted was 1 in 8.

Details of the design of the baffle platform, baffle, cistern and deflectors.

Throat width=24 ft.

Two piers each 2.5 ft. thick are provided; so overall width at end of throat =29 ft.

$$q_o = \frac{700}{29} = 24.14 \text{ cusecs}$$

From graphs⁽³⁾, the R. L. of the baffle platform was 516.19.

h_b = height of the baffle

$$\begin{aligned} &= C (d_o - d_2) \\ &= C \left[\sqrt[3]{\frac{q_o^2}{g}} - d_2 \right] \\ &= C (2.63 - 1) \end{aligned}$$

Various values of C were tried and the value of 1.6 was adopted.

So h_b=2.6 ft.

Distance of the baffle from toe=5.25 × height.

Design of the cistern.

The pavement extended up to 0.9 times the length of expansion. Energy of flow was calculated at section E F (which was 0.64 times the length of expansion from the end of throat) and section I J—the end of expansion—see figure 39. Bed width at section E F=48.8 ft. and q=14.35 cs. The depth of water on this section was calculated from

$$V = q/d_1 = 0.84 \text{ c } d_1^{0.64}$$

As f₁ (Lacey silt factor)=0.81

$$c = 0.9 \text{ and } d_1 = 6.02 \text{ ft.}$$

(3) Appendix V of the Central Board of Irrigation Publication No. 10 "Design of Canal Falls".

FIG. 37. INDUS RIVER COURSES

SCALE:- 1" = 1 MILE.

COURSE FOR 1930-31

COURSE FOR 1932-33

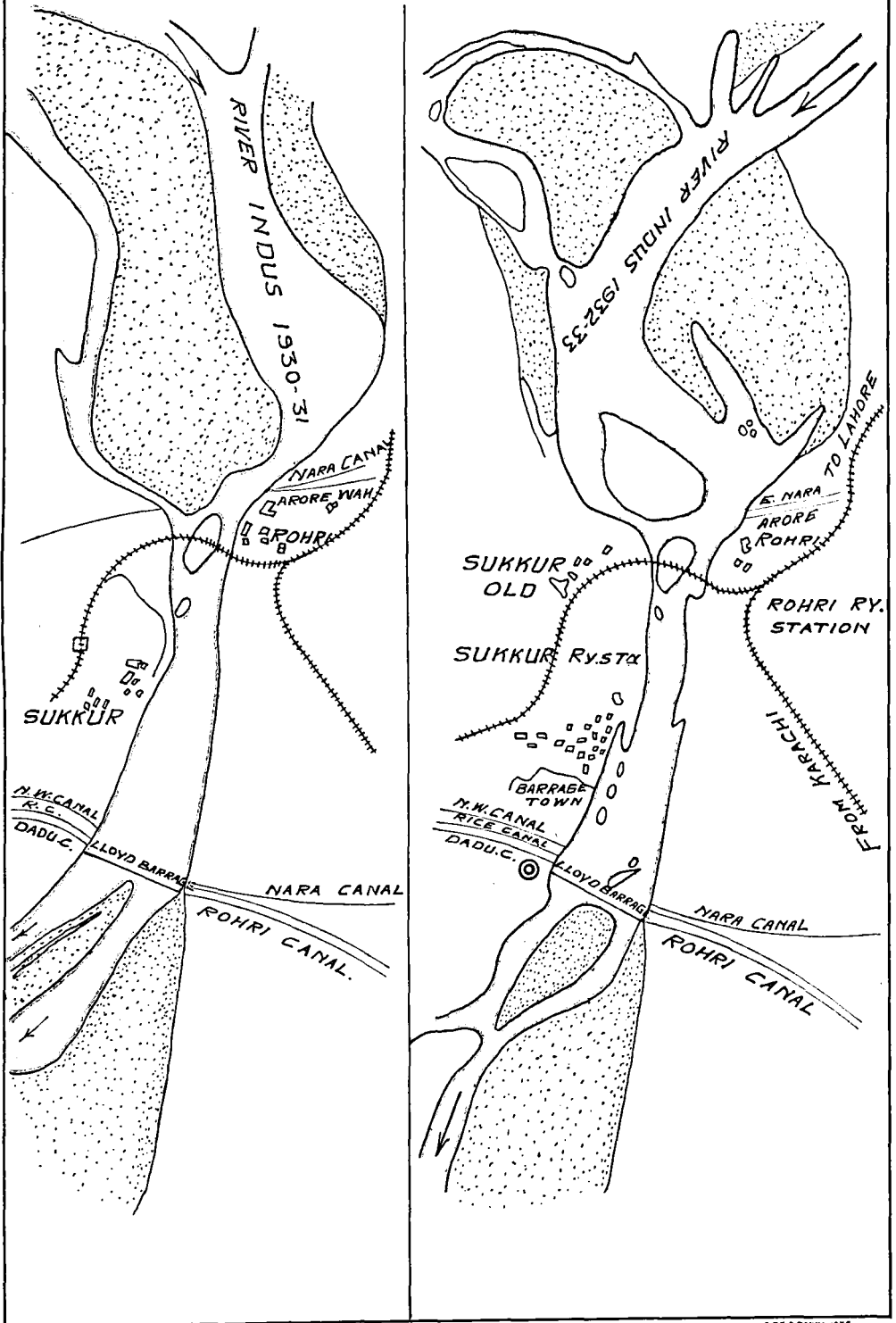
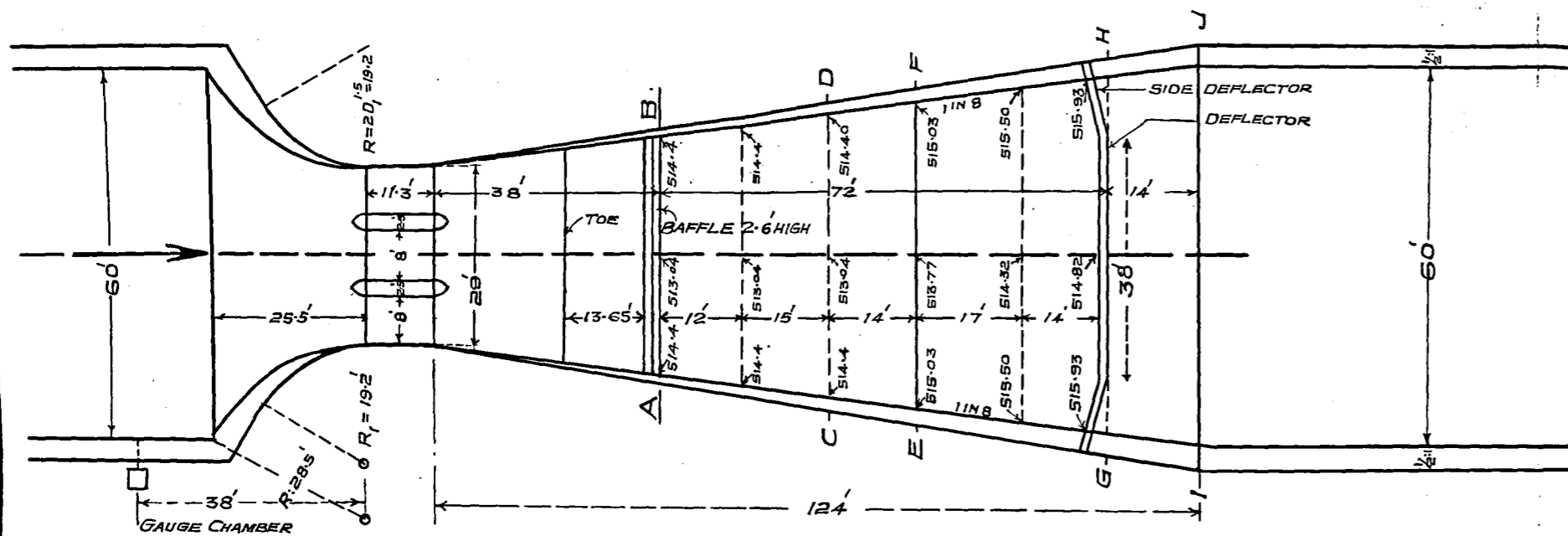
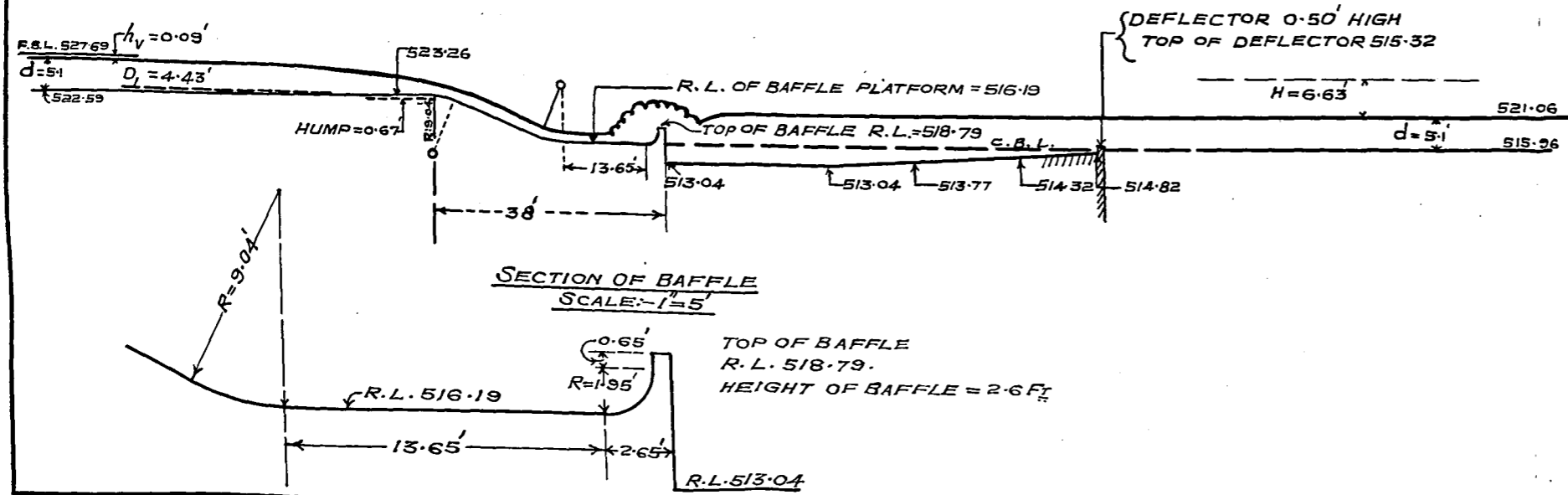


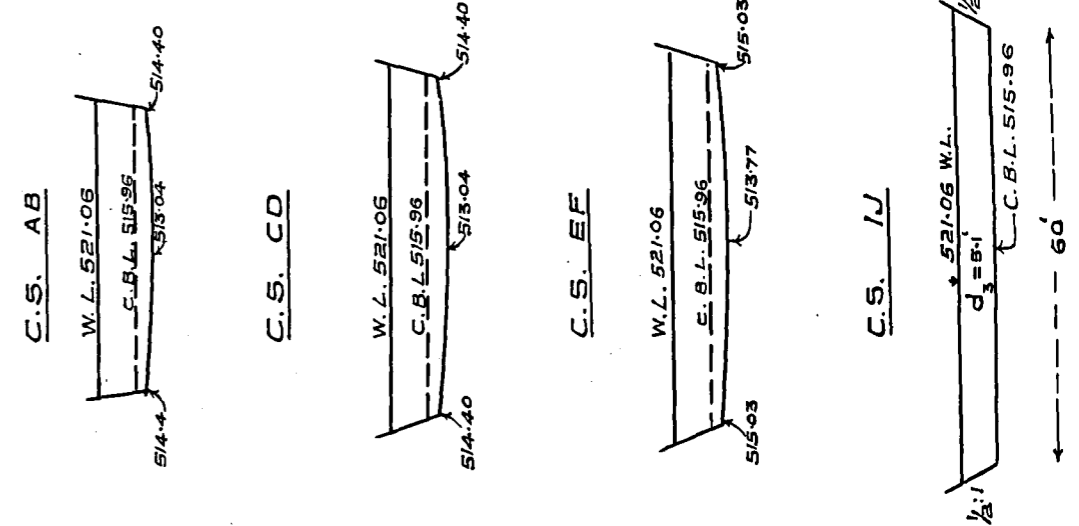
FIG: 39



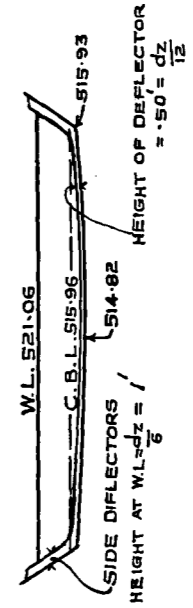
LONGITUDINAL SECTION



SECTION OF BAFFLE
SCALE: 1/5



C.S. GH



SCALE FOR PLAN & SECTION 1" = 20
DESIGN OF A 6.63 FEET PROPORTIONAL S.W. FLUME METER FALL COMBINED WITH V.R. BRIDGE AT R.D. 40280 MONTGOMERY PAK - PATTAN LINK, PUNJAB.

- F.S.Q. = 700 CS.
- BED WIDTH = 60 FT WITH 1/2:1 SIDE SLOPES
- F.S.D. = 5.1 FT
- V = 2.19 F/5.
- F.S.L. = 527.69 / 521.06
- BED LEVEL = 522.59 / 515.96

EXPERIMENTS WITH A 700 CUSECS PROPORTIONAL STANDING WAVE FLUME METER FALL MODEL.

TABLE 8.

Showing height of baffle, deflector, wavewash, etc., in various experiments with Model, scale ratio = 1/5; Q=12.5 cusecs (≡ 700 cusecs).

Experiment No.	d_1 (ft.)	d_2 (ft.)	h_b = height of baffle (ft.)	Height of bed deflector (ft.)	Height of side deflectors (ft.)	H (ft.)	d_3 ft.	H/ d_3 .	Maximum depth of scour below pavement level at end of cistern in mid-stream (ft.)	Height of waves near banks. (ft.)	Length of waves. (ft.)	Choppiness $\frac{\text{height of waves}}{\text{length of wave}}$.	Silt slope d/s of deflector in midstream.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
1	$\frac{2.63}{5}$	$\frac{1.0}{5}$	$\frac{1.96}{5}$ [= 1.2 ($d_1 - d_2$)] Baffle with ends sloping down at 1 in 25.	$\frac{0.50}{5}$ [= $\frac{d_1}{12}$]	$\frac{1.0}{5}$ [= $\frac{d_1}{6}$]	$\frac{6.63}{5}$	$\frac{5.1}{5}$	1.3	$\frac{0.55}{5}$	$\frac{0.50}{5}$	0.80 × 5	0.12	1 vertical to 4 horizontal.
2	$\frac{2.63}{5}$	$\frac{1.0}{5}$	Baffle of uniform height = $\frac{1.96}{5}$	[= $\frac{d_1}{12}$]	[= $\frac{d_1}{6}$]	$\frac{6.63}{5}$	$\frac{5.1}{5}$	1.3	$\frac{0.65}{5}$	$\frac{0.50}{5}$	0.75 × 5	0.13	1 in 3
3	$\frac{2.63}{5}$	$\frac{1.0}{5}$	$\frac{2.6}{5}$ [= 1.6 ($d_1 - d_2$)]	[= $\frac{d_1}{12}$]	[= $\frac{d_1}{6}$]	$\frac{6.63}{5}$	$\frac{5.1}{5}$	1.3	$\frac{0.60}{5}$	$\frac{0.35}{5}$	0.80 × 5	0.09	1 in 3
4	$\frac{2.63}{5}$	$\frac{1.0}{5}$	$\frac{2.6}{5}$ [= 1.6 ($d_1 - d_2$)]	[= $\frac{d_1}{12}$]	[= $\frac{d_1}{6}$]	25% retrogression in d's W. L.			$\frac{1.55}{5}$	$\frac{0.50}{5}$	0.83 × 5	0.12	1 in 4.6
						$\frac{7.95}{5}$	$\frac{5.1}{5}$	1.56					

The longitudinal bed profile was then worked out according to Mr. Montagu's method⁽⁴⁾ downstream of C D ; but was flat upstream up to the baffle platform.

The cross section of the bed of the cistern was bowed by a catenary curve, the amount of bowing in midstream being equal to 25 per cent. of R, R being the hydraulic mean depth at each section.

The amount of bowing at sections A B and C D = 1.36 ft.
 " " E F = 1.26 ft.
 " " G H = 1.11 ft.

Though the amount of bowing was different at various sections in this particular design, in standard designs the amount of bowing may be $\frac{d_2}{5}$ ($\cong 1.20$ ft.) at all sections. A bed deflector of height = $\frac{d_2}{12} = 6"$ was fixed at the end of the cistern and the side deflector = $\frac{d_2}{6}$ or 1 ft. high.

The central straight portion of the bed deflector at right angle to the flow = $\frac{3}{4}$ th the bed width ; the bed deflector near the banks diverged at 14°.

Experiments.

Experiments were carried out with a geometrically similar model with a scale ratio of $\frac{1}{5}$.

For determining bed scour, sand of 0.0096 inch mean diameter was used. At the beginning of each experiment, silt was relaid on the bed and contours were observed at the end of 30 hours flow after bed scour had completely ceased. Velocities were observed 6 ft. ($\cong 30$ ft.) downstream of the end of divergence, where the canal section was normal.

4 experiments were done, the results of which are shown in table 8.

In *experiment 1*, the height of baffle $\cong 1.96$ ft. = $1.2(d_1 - d_2)$ with ends sloping down at 1 in 25, which gave good results in a previous experiment with 1 in 6 divergences.

The standing wave downstream of the baffle was not clear and the flow was slightly disturbed. When the baffle was made of uniform height ($\cong 1.96$ ft.)—*experiment 2*—, the wavewash and the choppiness ratio = $\frac{\text{wavewash}}{\text{length of wave}}$ were the same as in experiment 1, viz., 0.10 ft. ($\cong 6"$) and 0.13 respectively—see photo 1,685 (sheet 17) showing conditions of flow downstream of the baffle. But isotachs and bed contours were more normal.

In *experiment 3*, the height of baffle was increased to the equivalent of 2.6 ft. [$\cong 1.6(d_1 - d_2)$]. The standing wave downstream of the baffle was then clear and the flow was sweeter than in experiment 2—see photo 1,686 (sheet 17). The flow in the channel downstream was also more steady, as indicated by less fluctuations of the float of the Bentzel velocity meter, observed 6 ft. downstream of end of divergence—see table 9 below:—

TABLE 9.

	Range of fluctuations of the float in Bentzel tube.		
	$\frac{1}{4}"$ below surface.	Mid-depth.	$\frac{1}{4}"$ above bed.
<i>Experiment 2.</i>			
Baffle $\cong 1.96'$ high	0.19	0.14	0.10
<i>Experiment 3.</i>			
Baffle $\cong 2.6'$ high	0.04	0.05	0.02

(4) Central Board of Irrigation Publication No. 5 "Fluming".

The range of fluctuations near the banks was very nearly the same in experiments 2 and 3, *viz.*, 0.05 near surface and near the bed and 0.03 at mid-depth. Wave-wash and choppiness factor were less with a higher baffle *viz.*, 0.07' (\cong 0.35 ft.) and 0.09' respectively in experiment 3, against 0.10' (\cong 0.50') and 0.13' in experiment 2.

The bed scour was more normal than in experiments 1 and 2—*vide* figures 40 and 41. The maximum depth of scour below the pavement level in midstream = 0.60 ft.; the isotachs were normal—see figure 42.

Experiment 4 was carried out with \cong 1.3 ft. (25 per cent) retrogression in downstream water level, d_3 being the same as before (\cong 5.1 ft.). It was thus increased from \cong 6.63 ft. to \cong 7.93 ft. The flow became slightly unstable. The wavewash slightly increased to 0.10' (\cong 0.50 ft.), and though the bed scour naturally increased, the increase was appreciably less than the retrogression and the silt slope downstream of the deflector was actually flatter, *viz.*, 1 in 4.6 against 1 in 3 in experiments 2 and 3; the isotachs were still satisfactory—see figures 43, 44, and 45. If retrogression occurs in actual practice, or is anticipated, 'Tarungars' or crates of stones should be laid along the toe of the banks for 25 ft.

Conclusions.

(1) With the standard divergence, the height of baffle for a 700 cusecs fall should be 1.6 ($d_3 - d_2$).

(2) Flow became slightly unstable with 25 per cent. retrogression downstream.

Experiments are being continued to improve the design further.

10. EXPERIMENTS TO FIND THE OPTIMUM POSITION AND HEIGHT OF A BAFFLE BELOW THE TOE OF A FALL IN A PARALLEL SIDED CHANNEL.

Object.

Energy is dissipated in a Standing Wave Flume fall in four ways, *viz.*—

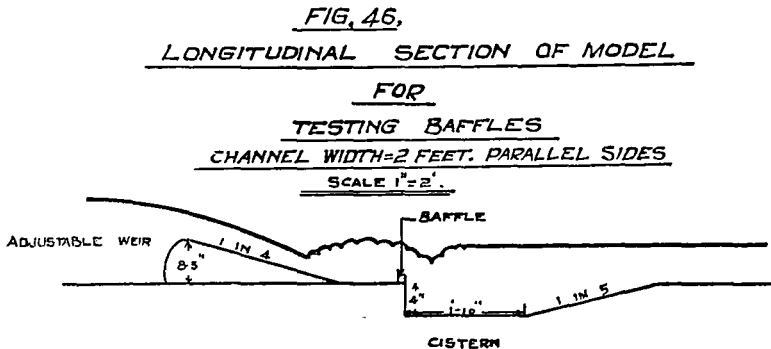
1. Boundary resistance on glacis slope.
2. Standing wave (above baffle).
3. Secondary wave (below baffle).
4. Boundary resistance on platform and in cistern.

In low falls there is not sufficient head to form both the primary and secondary waves. In high falls, both a standing wave and a secondary wave are possible and desirable. A baffle is used to create a standing wave upstream and a secondary wave downstream of it.

The object of these experiments was to find, for 2-dimensional flow, the optimum height and position of the baffle for a given discharge and height of fall, to produce smooth flow downstream.

Description of Channel and weir.

Figure 46 shows the section of the fall which was constructed in a 2 ft. parallel sided channel.



The height of fall was 8·3" and upstream depth of water over the weir=·453'. The cistern was 4" deep and 1' 10" long with 1 in 5 rising bed. Baffles of heights 1½", 2", 2½" and 3" were used, each being tested separately at 10", 12" and 15" from toe of the fall.

A discharge of 1 cusec per ft. width was run and the downstream depth varied by means of heading up gates.

Experiments.

For every set of conditions, fluctuations of the float of a Bentzel tube were observed downstream as a measure of the fluctuations in the turbulence of the water. Conditions were taken to be optimum when the fluctuations were minimum. These fluctuations were observed at ·1' above bed, mid-depth and ·1' below surface at a section 12' downstream of the baffle.

Figure 47 shows the fluctuations of the Bentzel meter float (mean of bed, mid-depth and surface readings) for the various baffles, positions of baffle and downstream depths.

It will be seen that minimum fluctuations of the float were obtained when the 2½" baffle was used. The formula adopted in 1934 for calculating the height of a baffle based on complete (3-dimensional flow) fall models—*vide* Annual Note for 1934-35—is $C(d_c-d_2)$

where d_c =critical depth of flow

d_2 =actual depth of flow (without baffle)

and C is a coefficient depending on the divergence of the sides and the discharge. The calculated critical depth for a discharge $q=1$ cusec is ·315 ft. and the observed depth $d_c=·13$ ft. so that the coefficient for the optimum height of baffle obtained was 1·13.

It will also be seen that, considering all the results, downstream velocity fluctuations were least when the baffle was placed at a distance of $5 \times$ height of baffle from the toe of the glacis, which corroborated the correctness of this figure, first adopted in 1928—*vide* Bombay P. W. D. Technical Paper 44.⁽⁴⁾

11. EXPERIMENTS WITH GODAVARI RIGHT BANK AND LEFT BANK HEAD REGULATORS.

Introduction.

At the request of the Superintending Engineer, Deccan Irrigation Circle, experiments were carried out with geometrically similar models of the Head regulators of the Godavari Right and Left Bank Canals, respectively, to determine what steps should be taken to restrict scour downstream. When first constructed, each Head Regulator consisted of three 10 ft. spans, regulated by means of horizontal baulks. Baulk regulation was, however, unsatisfactory; because when the Godavari River was in flood, the upstream water level rose and fell so rapidly that accurate regulation was impossible. A single bellmouth sluice was, therefore, constructed in the central span, 5 ft. high and 6 ft. wide in the case of the Godavari Right Bank Canal; and 4 ft. high and 5 ft. wide in the case of the Godavari Left Bank Canal.

The R. L. of the top of the wall on the weir is 533 and the maximum flood level up to which Canals are run is R. L. 539.

Godavari Right Bank Canal.

Figure 48 shows a plan of the head portion of the Godavari Right Bank Canal and figure 49 shows details of the bellmouth sluice in the central span.

R. L. of sluice bed=522·0.

F. S. L. in Godavari R. B. C. downstream of regulator=528·45 for F. S. Q.=384 cs.

When the river level exceeds R. L. 534·0, the canal discharge is reduced to 280 cusecs and the Canal is closed when the River level exceeds R. L. 539·0.

(4) Bombay P. W. D. Technical Paper 44 "Dissipation of energy below falls" by C. C. Inglis and D. V. Joglokhar.

1/5TH. SCALE MODEL OF 700 CUSECS (6.63 FT)

Nº 957.
C.I.&H.R.STM

P. S. W. F. METER FALL.

EXPT. 3 - HEIGHT OF BAFFLE $\cong 2.6$ FT $\{= 1.6 (d_c - d_2)\}$
WITH CISTERN, BED DEFLECTOR ($\frac{d_2}{6}$ HIGH) AND SIDE DEFLECTOR
($\frac{d_2}{12}$ HIGH). $d_1 = d_3 \cong 5.1$ FT
SCALE 1 INCH = 3 FEET.

NOTE :- CONTOURS ARE WITH RESPECT
TO PAVEMENT LEVEL AT END
OF CISTERN IN MID-STREAM.

FIG: 4D PLAN.

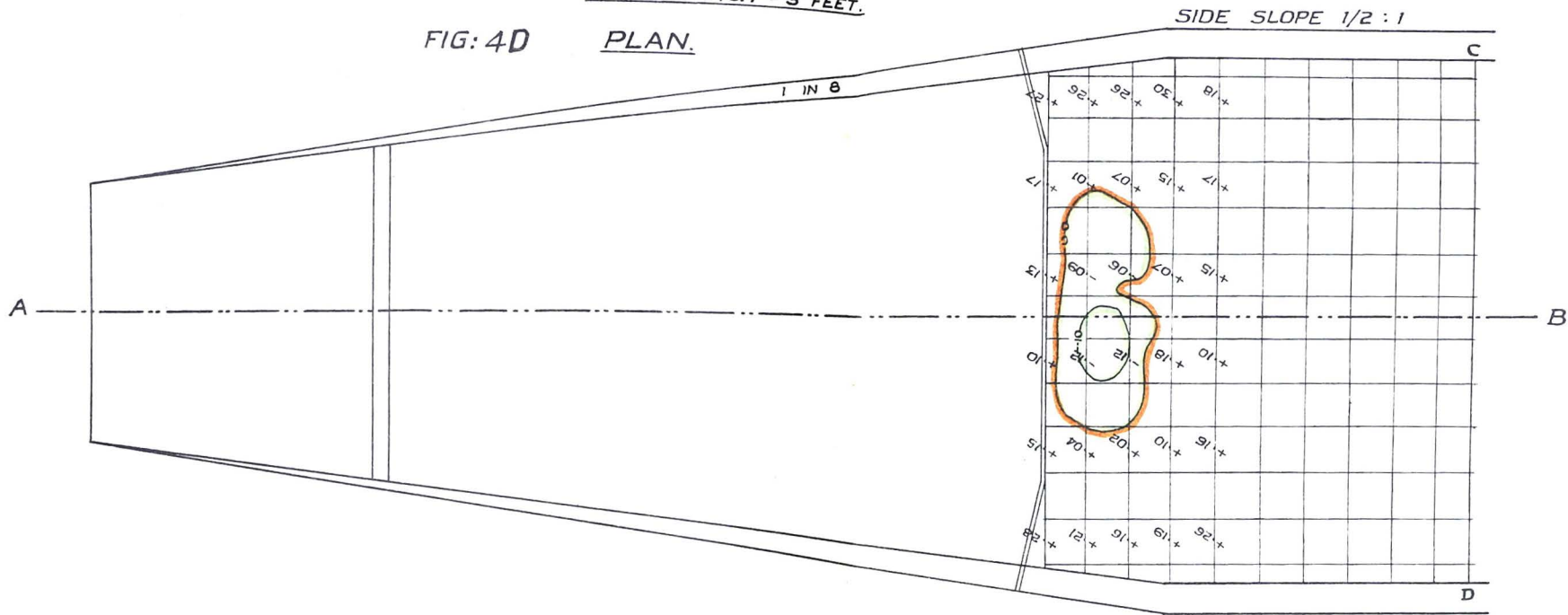


FIG: 41 LONGITUDINAL SECTION ON A B.

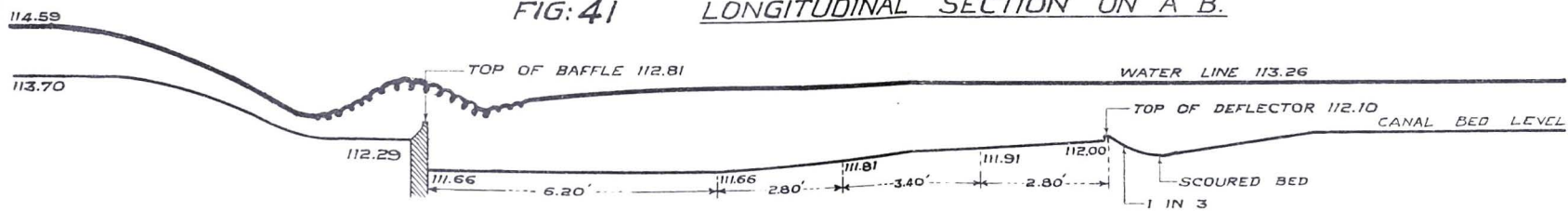
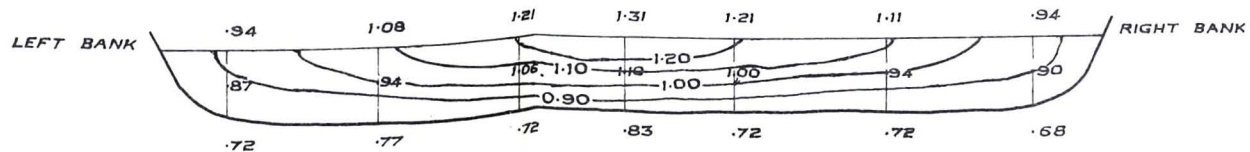


FIG: 42 ISOTACHS ON C. S. C.D. 6.8 ($\frac{34}{100}$)^{2/5} OF THE END OF EXPANSION.



$V_{OBS.} = 0.96$ f/s.
 $V_{CAL.} = 0.97$ f/s.

FIG. 43 TO 45 1/25TH SCALE MODEL OF 700 CU SECS

(6.63 FT) P.S.W.F.M. FALL

EXPT. 5 WITH 25% OF RETROGRESSION IN D/S

WATER LEVELS. HIGHT OF BAFFLE = 2.6 FT

CISTERN, BED AND SIDE DEFLECTORS.

($d_2/6$ AND $d_2/12$ HIGH RESPECTIVELY)

$d_1 = d_3 = 5.1 FT, H = 7.95 FT$

SCALE 1" = 3'

FIG. 43 PLAN

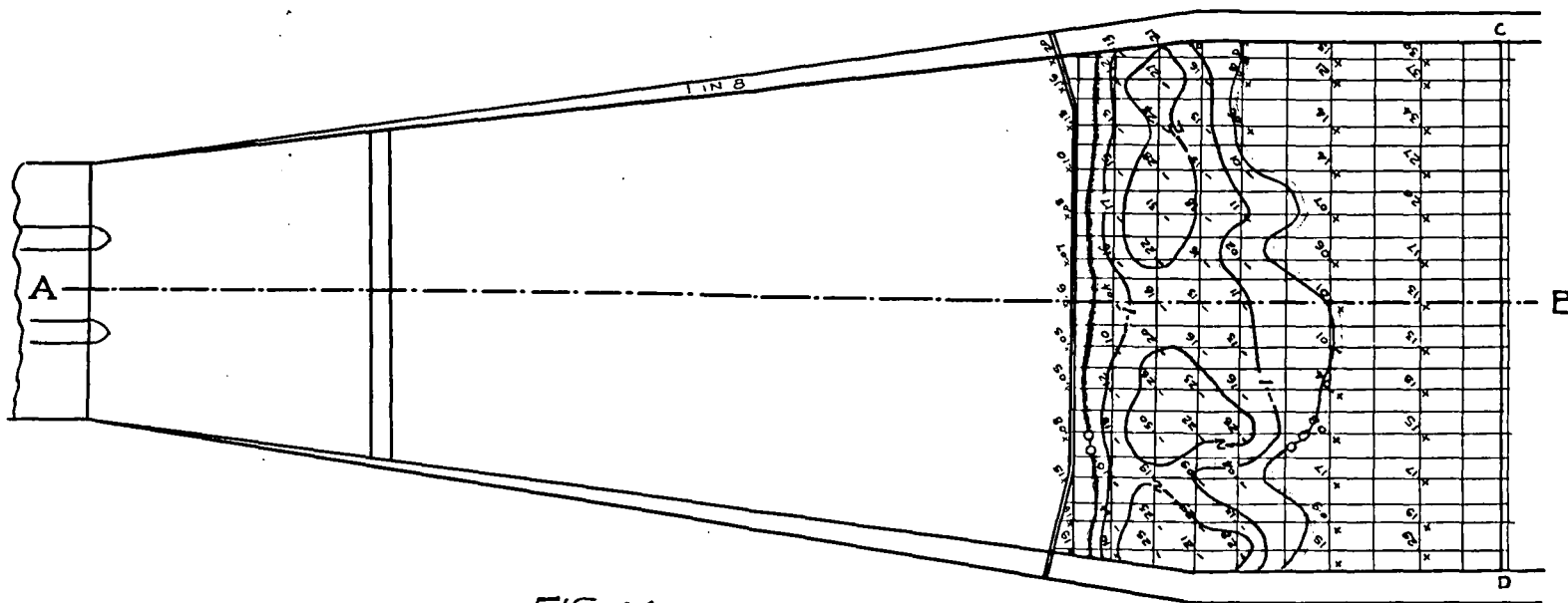


FIG. 44 LONGITUDINAL SECTION ON AB.

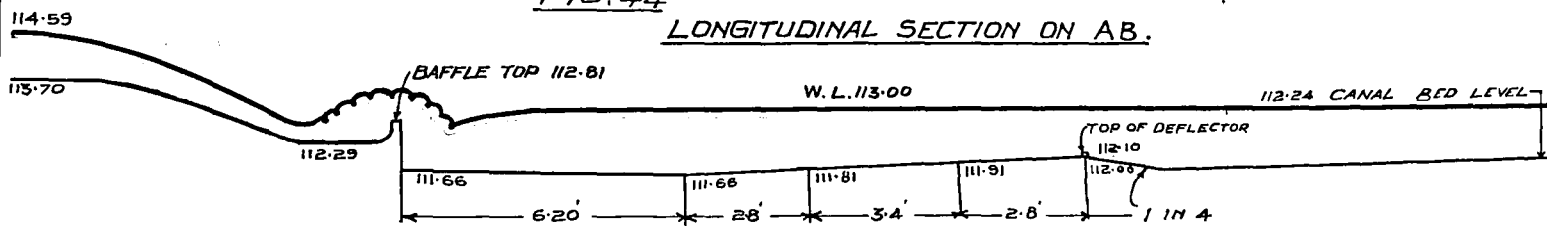
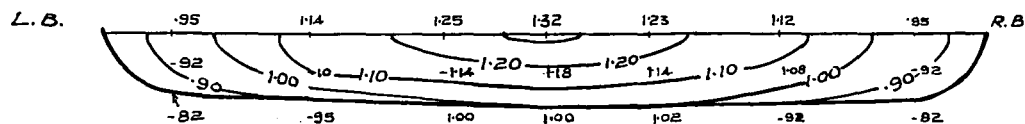


FIG. 45 ISOTACHS ON C.S. ON CD 6.8 (=34') D/S. OF EXPANSION. SCALE 1" = 2'



$$\frac{\bar{V}}{\bar{V}} \text{ (OBSERVED) } = 1.05 F_1$$

$$\frac{\bar{V}}{\bar{V}} \text{ (CALCULATED) } = 1.00 F_1$$

NOTES.

(1) CONTOURS ARE WITH RESPECT TO PAVEMENT LEVEL AT END OF CISTERN IN MIDSTREAM

(2) IF RETROGRESSION OCCURS OR IS ANTICIPATED, 'TARUNGARS' OR CRATES OF STONES SHOULD BE LAID AT THE TOE OF BANKS FOR 25 FEET.

FIG: 47,

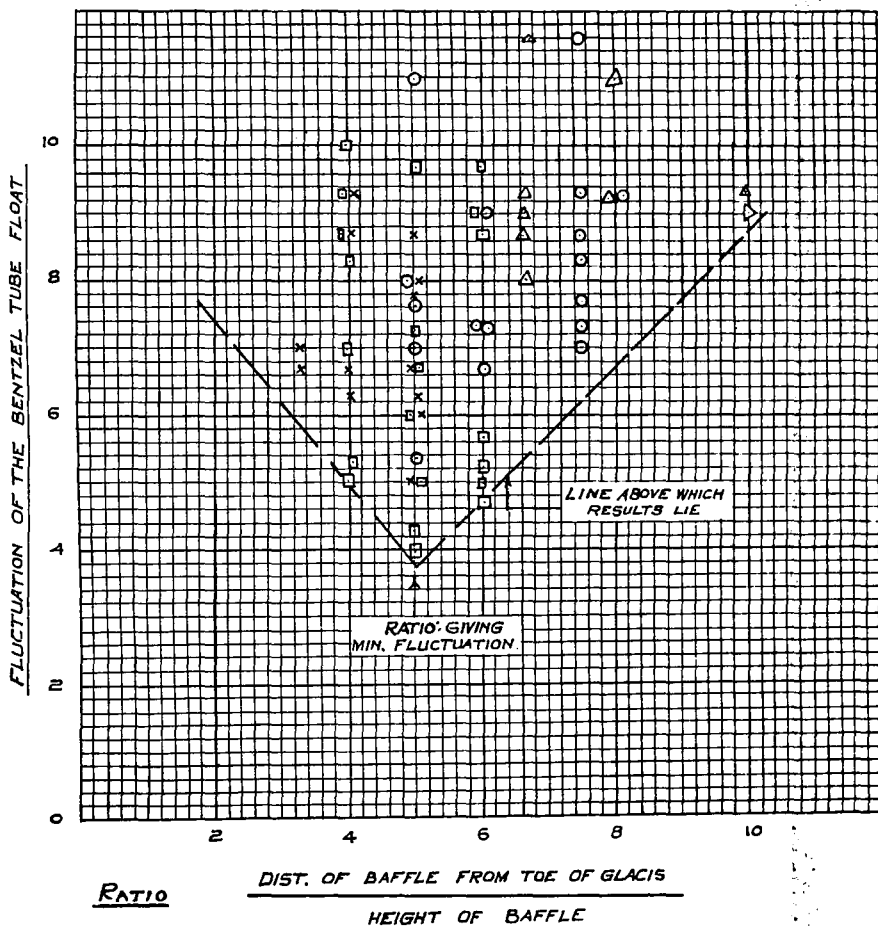
NO 948
C.I.B.H.R.STN

RESULTS OF TURBULENCE TESTS OF BAFFLES IN A
PARALLEL SIDED CHANNEL.

THE AMOUNT OF TURBULENCE WAS MEASURED BY
THE FLUCTUATIONS IN A BENTZEL VELOCITY METER
(MEAN OF READINGS OF SURFACE, MID-DEPTH AND BED)

HEIGHT OF FALL 8.3 INCHES
DEPTH OF CISTERN 4 INCHES
DISCHARGE 1 CU SEC PER FT
WIDTH.

BAFFLE 1/2 INCH --- Δ
HEIGHT 2 INCHES --- ○
2 1/2 " --- □
3 " --- x



PLAN OF HEAD REGULATOR OF THE
GODAVARI RIGHT BANK CANAL.

SHOWING BELLMOUTH SLUICE IN CENTRAL SPAN AND
ORDINARY SLUICE IN RIGHT SPAN.

SCALE: 40 FEET = 1 INCH.

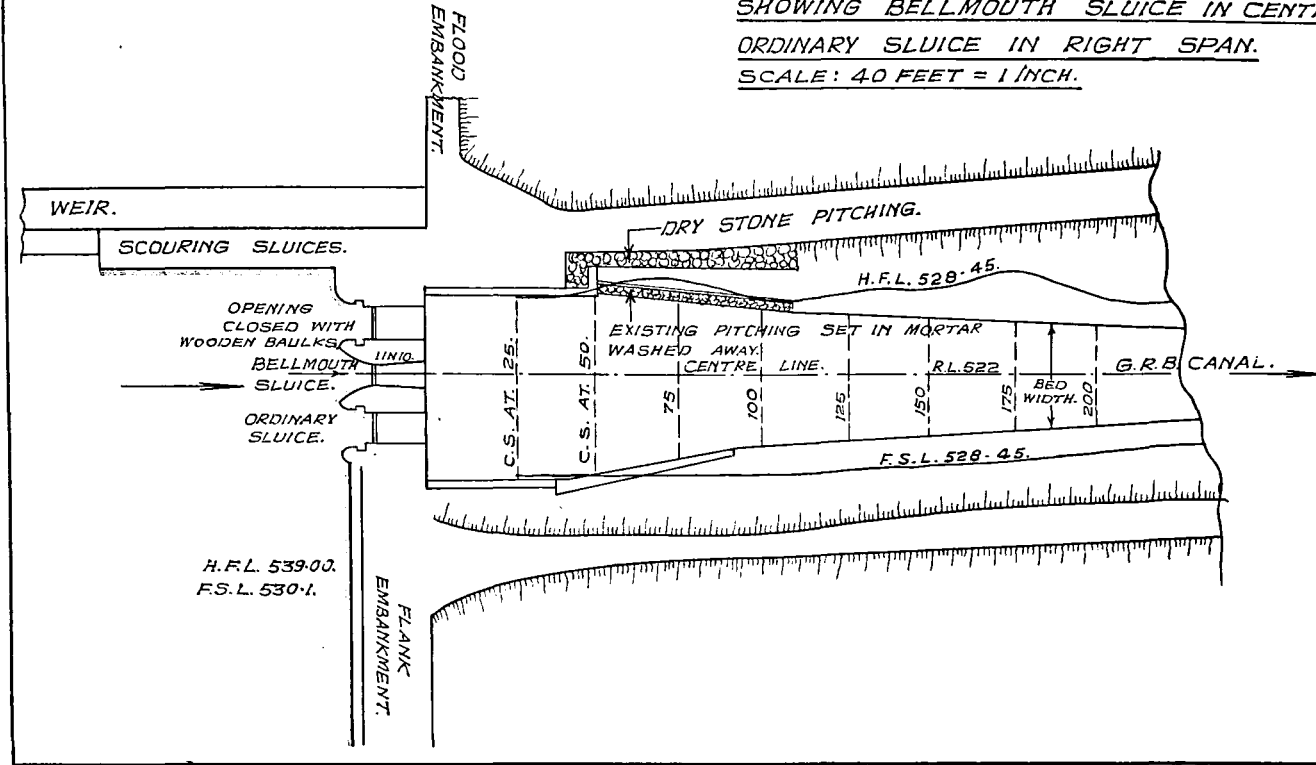
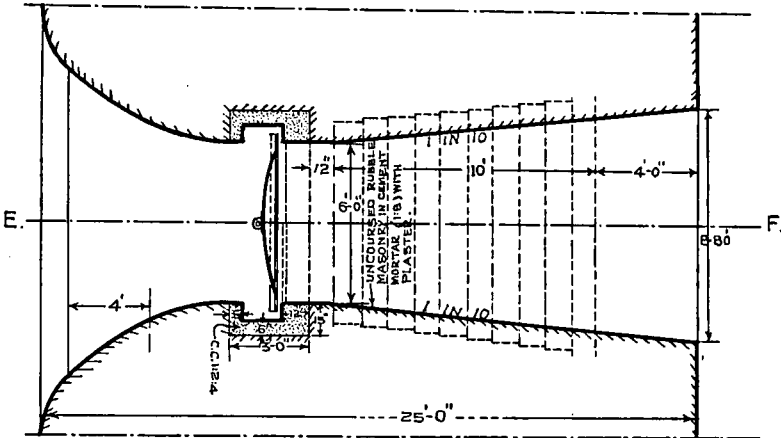
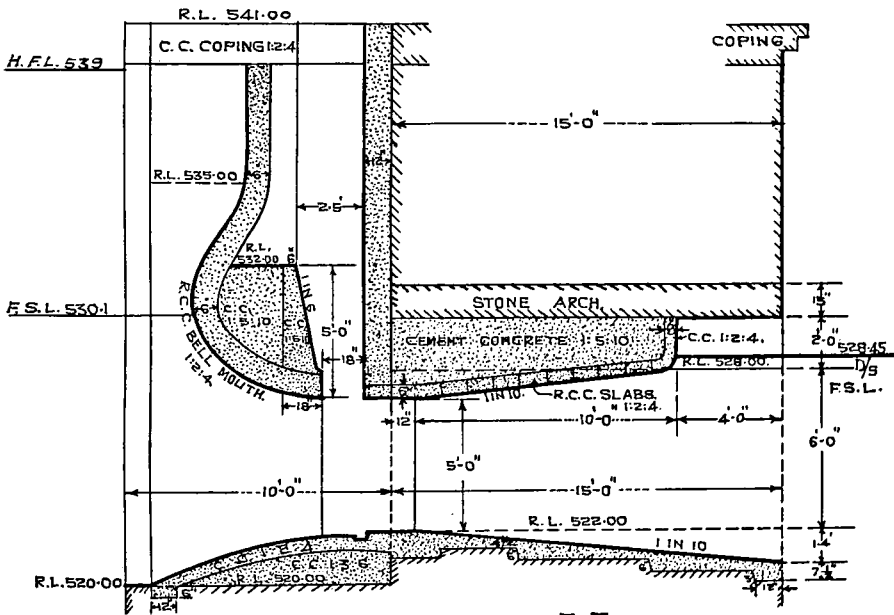


FIG:48

FIG: 49,
SHOWING PLAN & SECTION OF
GODAVARI HEAD REGULATOR
WITH BELL-MOUTH SLUICE
ON THE RIGHT BANK.
SCALE: 5' = 1 INCH.



PLAN



SECTION ON E.F.

With F. S. Q. of 384 cusecs, the downstream water level in the canal is 528.45, and the upstream water level with gate fully open is R. L. 530.1 thus giving a loss of head=1.65 ft. This loss of head consists of

- 58 ft. loss at entrance
 - 03 ft. friction in barrel
 - 31 ft. loss at exit.
-
- 92 ft.

and the remaining energy (1.65 ft.—.92 ft.)=.73 ft. is lost in a standing wave downstream. *This loss in the wave would not, of course, occur, were the downstream water level in the canal .73 ft. higher. Alternatively the sluice would pass F. S. Q. with an upstream W. L. .73 ft. lower if the level of the sluice were lowered by that amount.*

When the upstream water level is higher than R. L. 530.1, the additional head has also to be dissipated. This extra head is partly absorbed by shock losses, due to the gate being partly closed, to reduce discharge; but a large part of the excess energy, due to the high velocity through the gate opening, remains; causing scour in the Canal downstream. This trouble will always arise where there is only a single gate, unless it is built at a low level; but where there are three or more bellmouth sluices,—as at the head of the Nira Left Bank Canal, where this method of regulation was first introduced—there is little difficulty in regulating in such a way that excess energy can be dissipated satisfactorily.

Experiments.

Scale ratio of geometrically similar model=1/9. Several different designs were tried to determine the best way to dissipate this excess energy.

The best design was found to be :—

- (a) to construct side walls in extension of the side walls of the existing sluice, diverging at 1 in 10 for a length of 35 ft.
- (b) to construct a baffle 13½" high at the end of the sluice and
- (c) to excavate a cistern 3.3 ft. deep and 11 ft. long—as shown in figure 50.

With this design satisfactory results were obtained—*vide* photos 1548 and 1550 (sheet 18)—, when the level in the Lake was between R. L. 530.1 and R. L. 532 and also when the W. L. exceeded R. L. 535; but less satisfactory results were obtained between these two levels.

While the experiments were in progress, it was decided to construct an ordinary gate in the right span of the regulator, of the same size as the bellmouth gate, namely 5 ft. high by 6 ft. wide. Figure 51 shows all possible gate openings of the 2 gates, the combined discharge of which=F. S. Q. of 384 cusecs for various depths upstream; and the full black line shows the optimum values.

The method of regulating gates should be such that it should not be necessary to open one gate and close another simultaneously, which would be difficult to control when river levels were fluctuating rapidly.

Figure 52 shows a suitable method of regulation, the only objection being that wavewash would be considerable when the water level was between R. L. 533 and R. L. 534. It is probable, however, that this amount of wavewash does not matter in practice, and the canal will rarely, if ever, be run with full supply at this level.

Godavari Left Bank Canal.

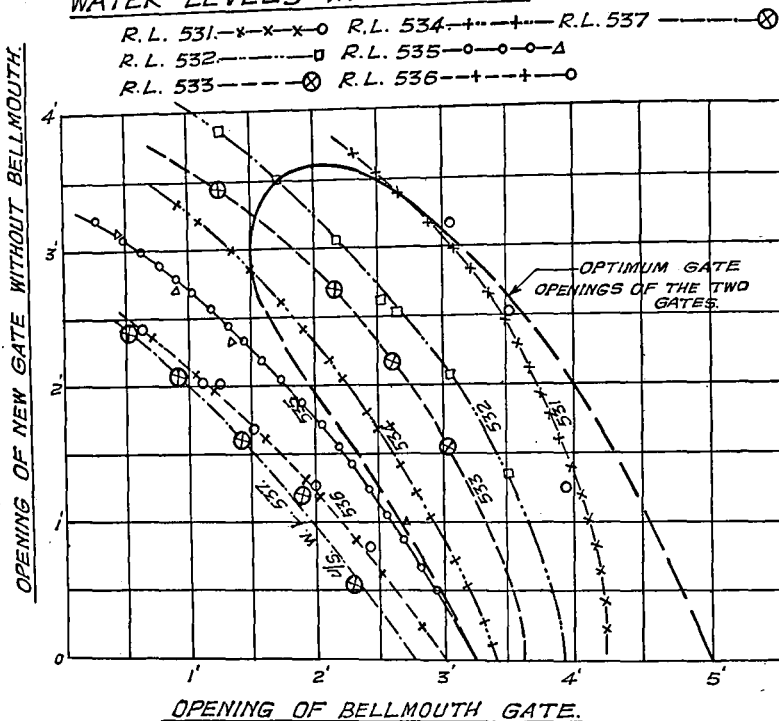
R. L. of sluice bed=523.0.

F. S. L. in Godavari L. B. C. d/s of regulator=529.0 for F. S. Q.=245 cusecs.

When the River level exceeds R. L. 534.0, the Canal discharge is reduced to 191 cusecs. The canal is closed when the River level exceeds R. L. 539.0.

Experiments carried out with a geometrically similar 1/9th scale model gave little scour and maximum wavewash was equivalent to only 7.5"; so conditions

FIG. 5], CURVES SHOWING OPENINGS OF TWO GATES WHICH GIVE F.S. Q = 384 Cs. FOR VARIOUS $U/5$ WATER LEVELS IN THE WEIR.



do not appear to be sufficiently bad to necessitate any change in the design. However if it is still considered necessary to improve conditions, this can be done by extending the expansion of the sluice by 25 ft. and constructing a cistern 2.5 ft. deep and 10 ft. long.

12. TRAINING THE RIVER SARDA ABOVE THE BARRAGE AT BANNBASSA—UNITED PROVINCES.

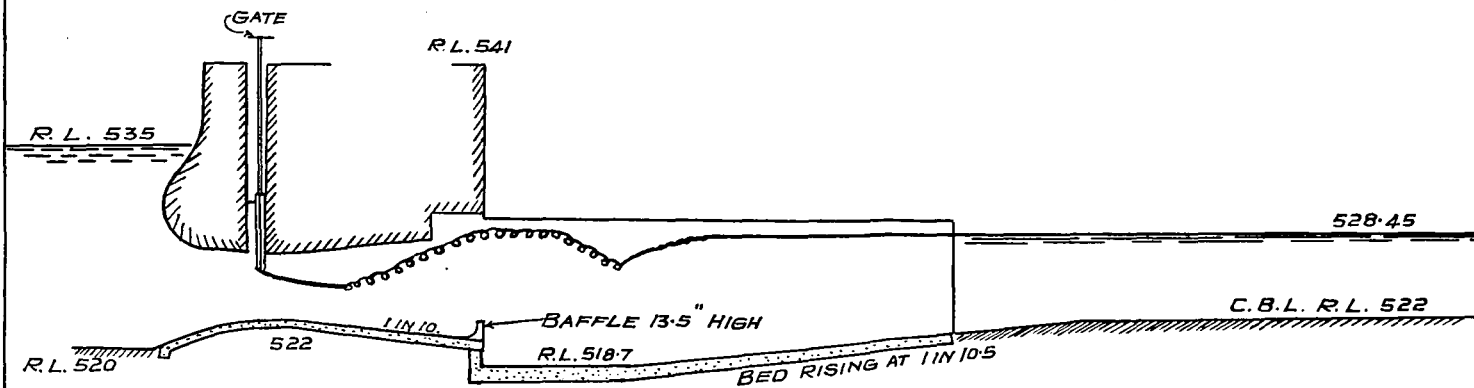
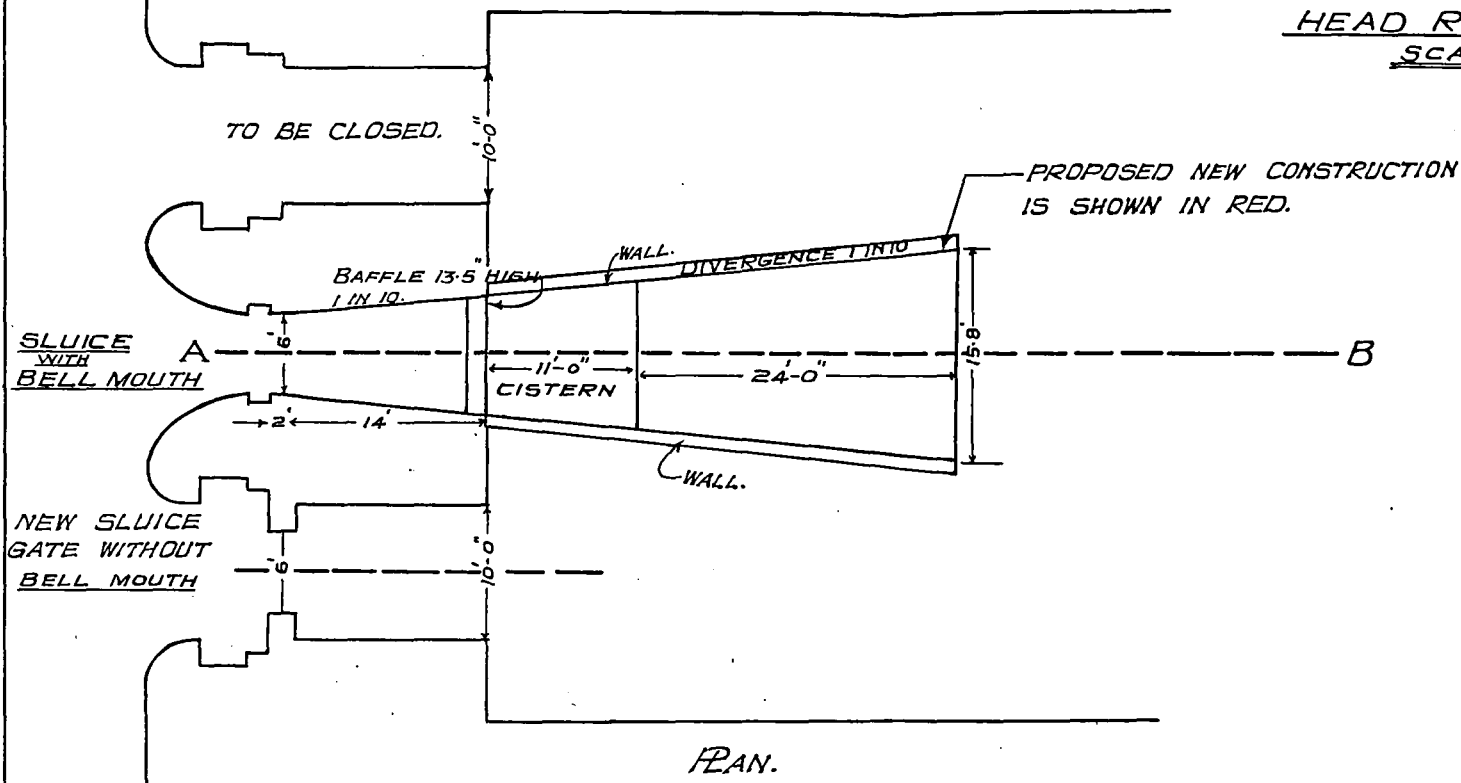
Introduction.

Prior to the construction of the Sarda Barrage in 1923-25, the main channel flowed in what is known as the "Solani Got" channel, about three quarters of a mile to the west of the present channel—see figure 53; but when the canal Head-works were under construction in 1924-25, a crate bund was built 3 miles upstream of the Barrage site in order to reduce the flow of the Solani Got channel which was interfering with the progress of work. This caused a shoal to form, which deflected the main River from the western, right, flank to the eastern, left, flank and the River began to attack the eastern, Nepal, bank.

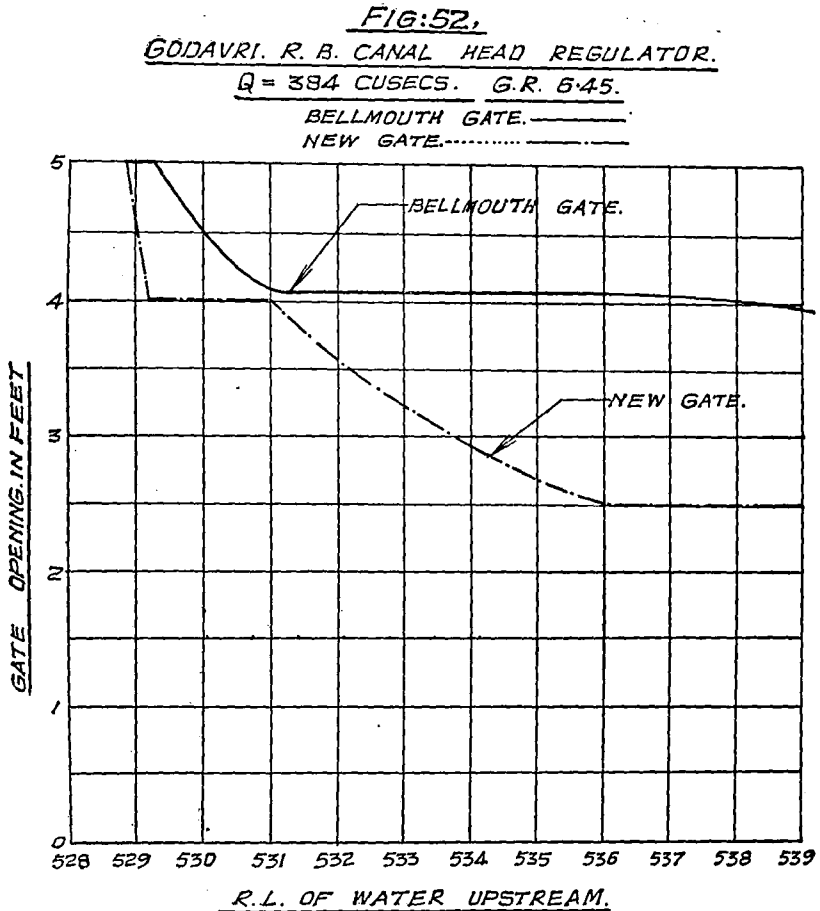
To reduce the attack on the left flank, the Nepal spur was constructed in 1928 one mile upstream of the Barrage; and in 1929, the Burma spur was constructed half way between the Nepal spur and the Barrage.

FIG: No. 50,

PLAN OF GODAVARI.
R. B. CANAL
HEAD REGULATOR
SCALE 1"=10'



Though the Nepal spur has successfully checked the attack on the Left Afflux Bund, the flow still follows the outside of the bend from the nose of the Nepal spur, passing outside the Burma spur, to the nose of the Left Upstream Bell's Bund, whence it swings across towards the Sarda Canal head, flowing along the face of the Barrage. The "Solani Got" channel has, however, carried little water, except during floods, for several years.



1936-37 experiments.

Experiments described in the 1936-37 Annual Report and Summary, were carried out to test various alternative suggestions, which, it was considered, would reduce flow across the face of the Barrage during floods and yet maintain an open channel, throughout the fair season, from the main channel at the left flank to the canal at the right bank—preferably through the western channel—see figure 54—which takes off from the eastern, left flank, channel opposite the Nepal spur and flows along the east and south margin of the Kala Jala island towards the head of the canal.

It was considered that extending the Nepal spur would not give satisfactory results owing to the restricted channel between it and the Kala Jala island and that the main stream could be diverted more easily towards midstream by extending the Burma spur.

The scale ratios of the model were :—

Longitudinal and latitudinal scale ratio	1/300
Depth scale ratio	1/27.5
Vertical exaggeration	10.9

Figures 54 and 55 show several alternative methods of training the river by extending the Burma spur. These were tested with a constant discharge of 2.4 cusecs, maintaining gauge levels corresponding to 240,000 cusecs at the Barrage.

Conclusions of 1936-37 experiments.

With 2.4 cusecs, the worst results were obtained with the extended spur facing 10° upstream of the present alignment of the Burma spur and the best results when the spur was aligned 20° downstream. When the discharge was increased to 4 cusecs, particularly when the extension was long, the advantage of a 20° downstream alignment was still greater; because in the other designs, high velocity flow scoured a straight channel along the east edge of the Kala Jala island and this high velocity jet tended to persist towards the L. U. S. Bund. When this jetting occurred, flow through the western channel was slack and there was a tendency for it and the other channels between the Kala Jala island and the Barrage to silt, and for shoals D, G and H to extend—which would accentuate cross flow above the Barrage towards the canal—see figures 54 and 55.

The best results were obtained with the Burma spur extended 1,020 ft. at an angle of 20° downstream of the present alignment (experiment 4).

The next best result was in experiment 5, with a straight spur 1,000 ft. (20° d/s) and a low level extension 600 ft. long on the same alignment; but the experiments indicated that it was doubtful whether the western channel could be revived satisfactorily; and under unsatisfactory river conditions it might tend to silt—even though the Burma Spur were extended. In 1937 a model 160 ft. long was built to a larger scale :—

Latitudinal and longitudinal scale ratios	1/150
Depth scale ratio	1/50
Vertical exaggeration	3.0

The discharge scale ratio was worked out from the formulæ :—

$$\frac{Q_p}{Q_m} = \frac{N_{a_m}}{N_{a_p}} \left(\frac{L_p}{L_m} \right)^{\frac{1}{2}} \left(\frac{d_p}{d_m} \right)^{7/4} \left(\frac{h_p}{h_m} \right)^{\frac{1}{2}}$$

$$\text{and } \frac{N_{a_m}}{N_{a_p}} = \left(\frac{f_{d_m}}{f_{d_p}} \right)^{\frac{1}{4}} \dots \dots \dots \text{(Lacey).}$$

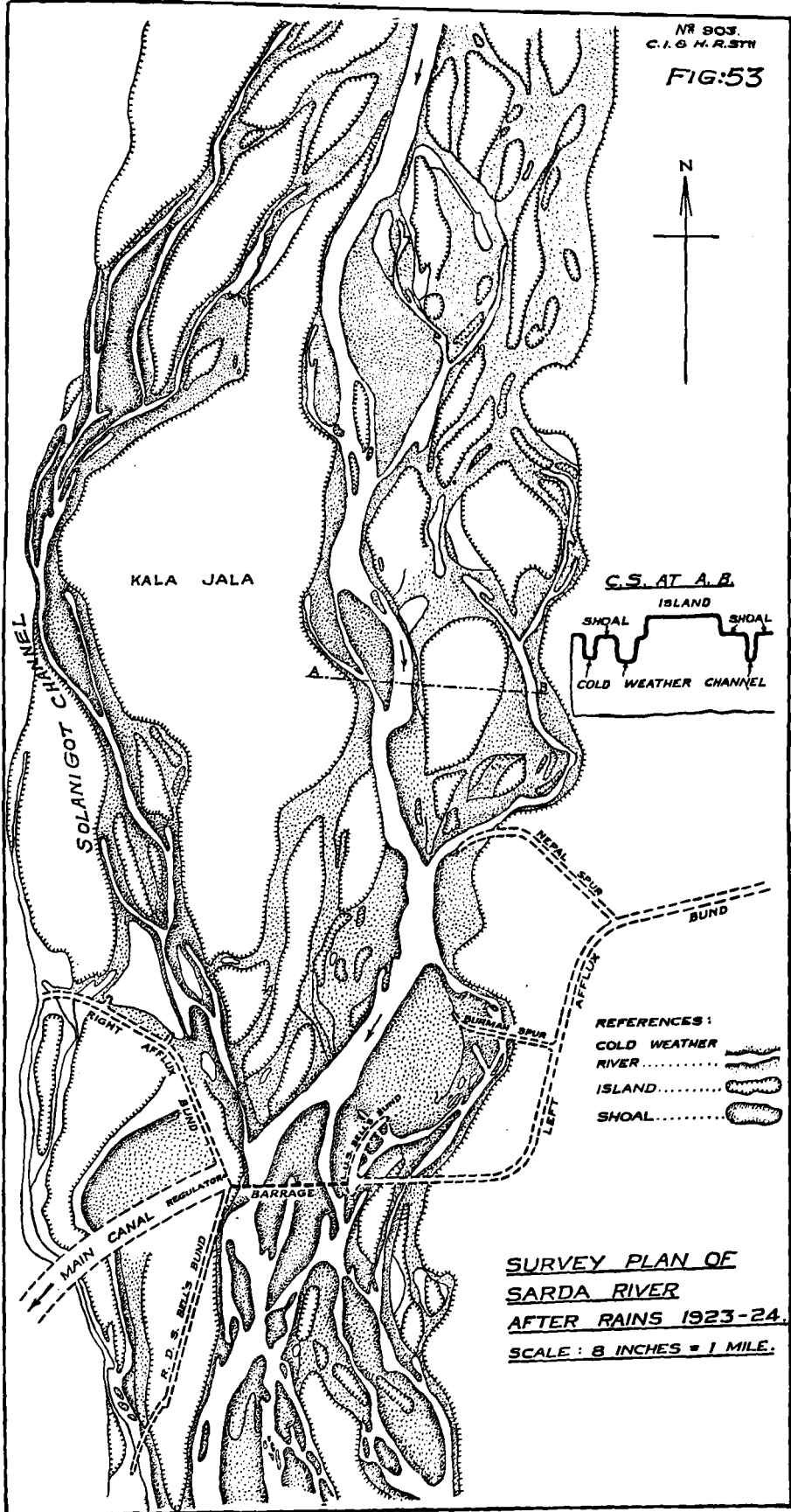
In the prototype, the bed consists of boulders varying in size from 6" to 18" ; which were represented in the model by sand grains varying from $\frac{1}{8}$ " to $\frac{1}{2}$ ". The model comprised a $3\frac{1}{2}$ mile stretch of river above the Barrage—shown in figures 56 and 57.

The first experiments were run with a view to reproduce 1937 river conditions (figure 57) starting from 1936-37 river survey (figure 56).

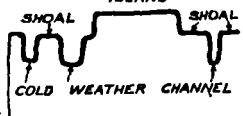
As there was no detailed survey of the distribution of boulders of various sizes, this had to be worked out in the model by considering river movements during the past 15 years; by judgment and by observation; but the most important point demonstrated was that *the results obtained depended primarily on entry conditions*; and it was found that in this, as in other experiments, correct entry conditions could only be achieved by modifying the plan at entry—in other words, *success depends on the experience and practical skill of those carrying out the experiments.*

NR 903.
C.I.G.H.R.STN

FIG:53



C.S. AT A.B.



- REFERENCES:
- COLD WEATHER RIVER
 - ISLAND
 - SHOAL

SURVEY PLAN OF
SARDA RIVER
AFTER RAINS 1923-24.
SCALE : 8 INCHES = 1 MILE.

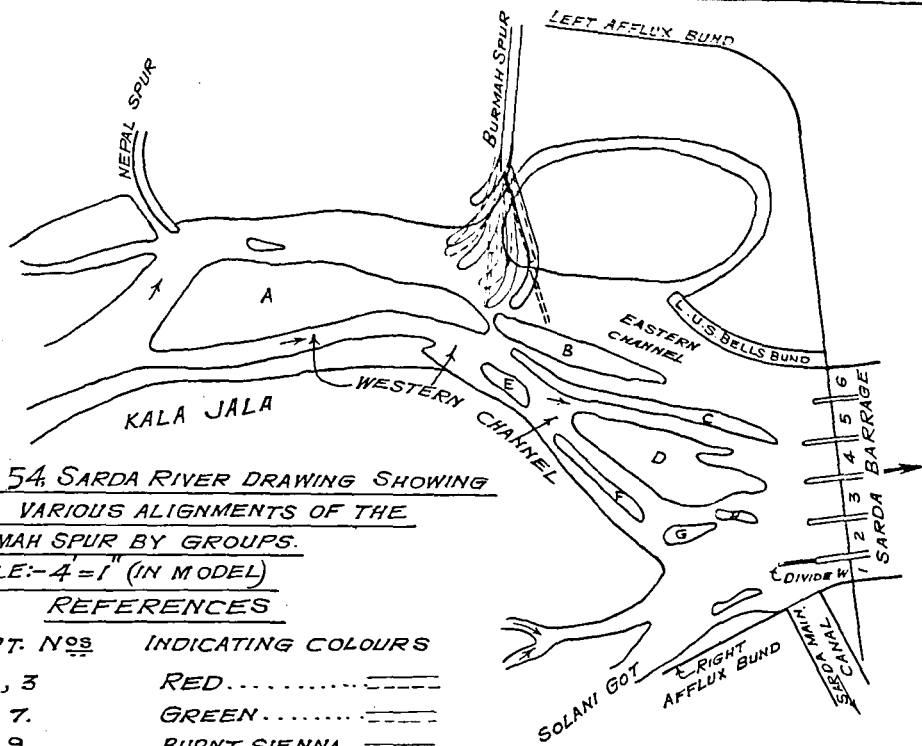


FIG 54, SARDA RIVER DRAWING SHOWING THE VARIOUS ALIGNMENTS OF THE BURMAH SPUR BY GROUPS.

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

REFERENCES

EXPT. NOS	INDICATING COLOURS
2, 3	RED.....
6, 7.	GREEN.....
8, 9	BURNT SIENNA.....
4, 5	BLUE.....

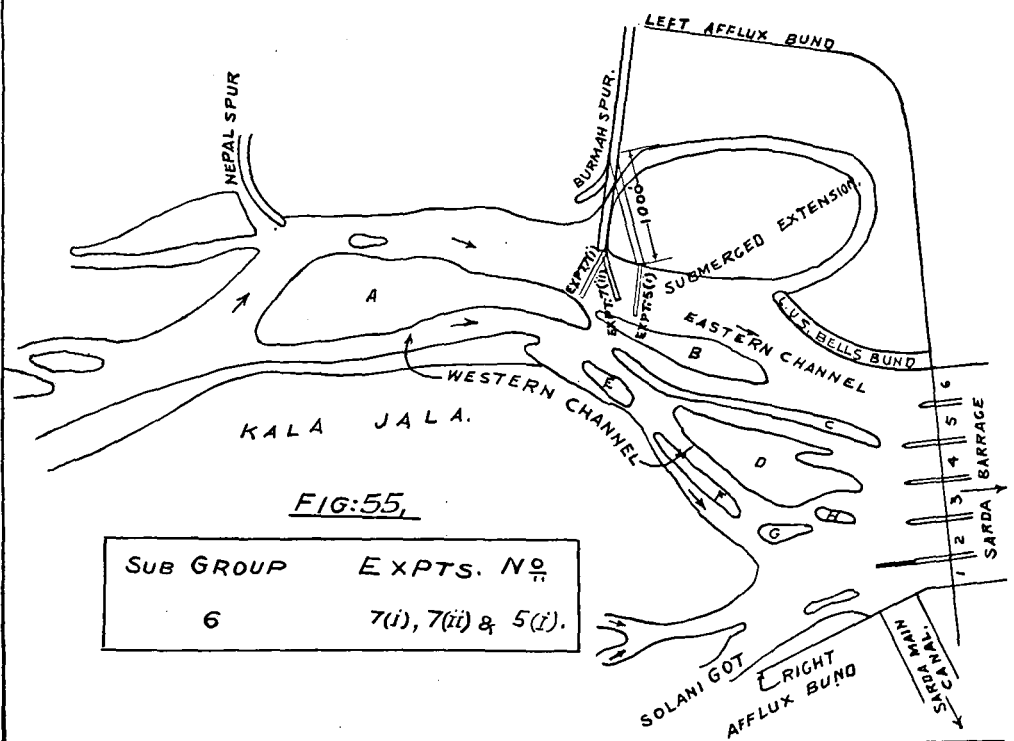





FIG:55,

SUB GROUP	EXPTS. No.
6	7(i), 7(ii) & 5(i).

FIG: 56.
SURVEY PLAN OF
SARDA RIVER
AFTER RAINS 1936.
SCALE:- 8" = 3 MILES.

REFERENCES.
COLD WEATHER RIVER. 
ISLAND..... 
SHOAL..... 

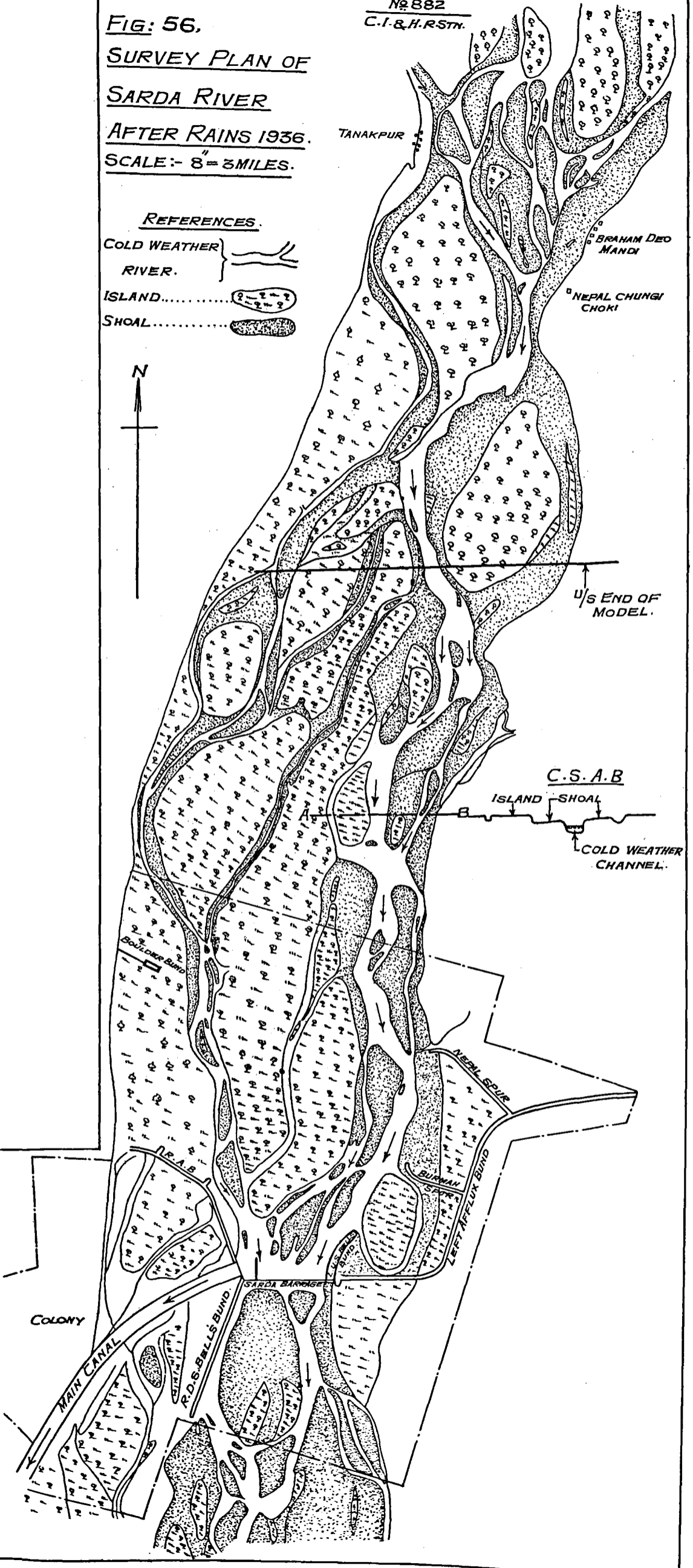
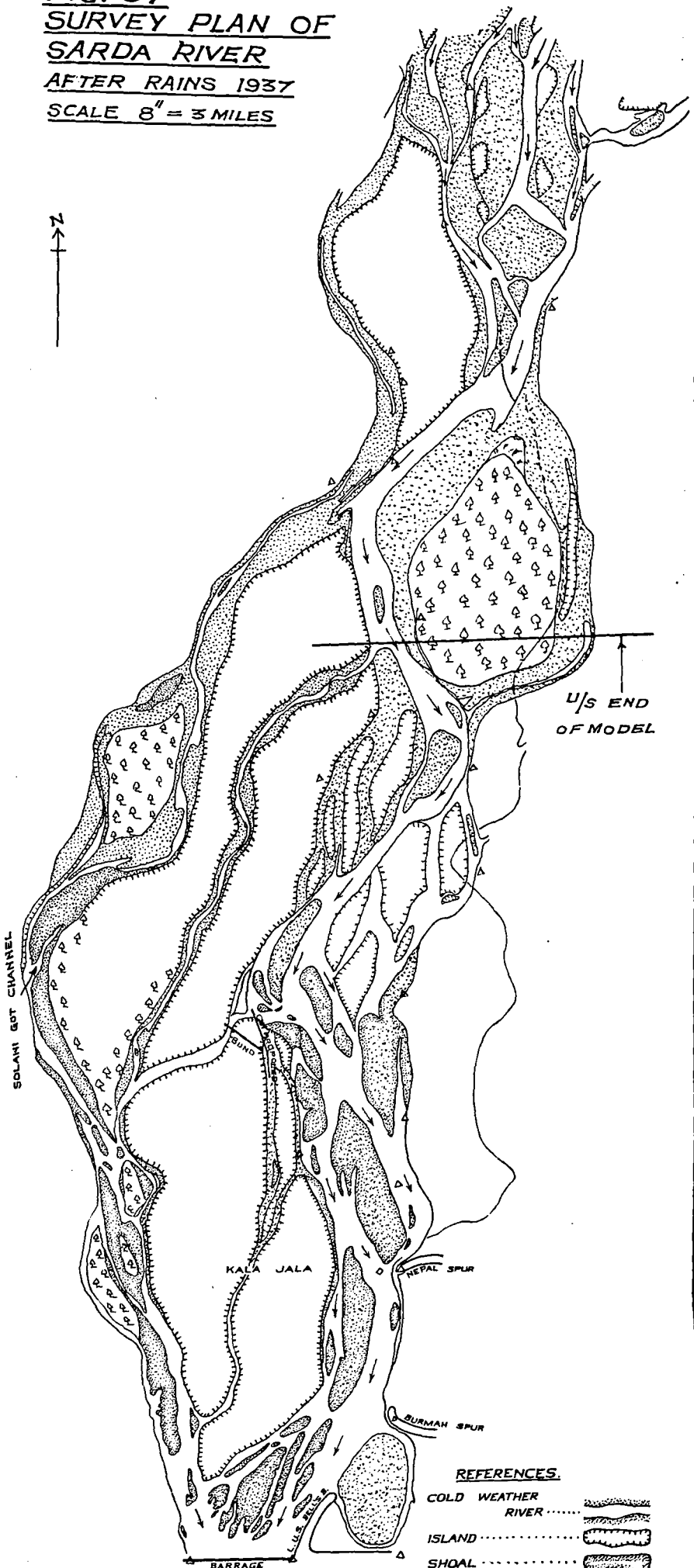


FIG. 57
SURVEY PLAN OF
SARDA RIVER
AFTER RAINS 1937
SCALE 8" = 3 MILES



- REFERENCES.**
- COLD WEATHER RIVER [stippled pattern]
 - ISLAND [dotted pattern]
 - SHOAL [wavy line pattern]

Conclusions drawn from the enquiry up to date.

It has been stated in Part II of the History of the Headworks of the Sarla main Canal from 1928 to June 1935 that "the Sarla River provides a striking example of the vagaries of Indian Rivers" and during the 1937 flood season, the sinuosity of the river above the Nepal spur increased to a marked extent.

As a result, a new channel started to cut across the Kala Jala island into the Solani Got channel above the Nepal spur—compare figure 53 showing 1923-24 survey of the River and figures 56 and 57 showing 1936-37 and 1937-38 conditions—.

These rapid changes show that it is necessary to consider the question not as 'a local problem for the next few years'; but as a question which calls for a comprehensive solution, taking all possibilities into account.

The main factors which affect this question are:—

- (i) A River always tends to hug one bank or the other, and is unstable in any intermediate position; and though rivers often tend to flow in two or more channels, one channel is always tending to enlarge while the other channels are tending to silt. This continues, until a limit is reached, when one of the silted channels begins to scour and the main channel to silt.
- (ii) In general, a channel taking off from the upper end of the outside of a concave curve will scour, while a channel taking off from the inside of a curve will silt.
- (iii) When a river is controlled, as at Headworks, the river adheres very tenaciously to one or other bank. This is because a channel is strongly attracted by, and tends to flow along, the face of a masonry work or pitched bank; and a deep channel forms along its face. If the masonry work juts out into the stream, a deep hole is scoured outside its nose, and the stream is permanently held in this position.

In the case of the Sarla River, the Nepal spur and the L. U. S. Bund have held the river along the left bank; and during 1937—see figure 57—flow was held more rigidly to the left flank than ever before and so it is very doubtful whether the proposal to reopen the Western channel by extending the Burma Spur would be successful unless a considerable part of the east and south-east bank of the Kala Jala island could be eroded by extending the head of the Nepal spur from year to year. Indeed there is reason to believe that there is a considerable danger of the Western channel dying.

A second alternative would be to induce the existing small channel through the Kala Jala island, which takes off the main river a little above the Nepal spur, to open out to become the main stream.

A third alternative, which may occur naturally, is for the river to cut across the Kala Jala island into the Solani Got channel upstream of the Nepal Spur—as it started to do last flood season.

Only some of these alternatives are capable of demonstration by model experiments; the rest depend on judgment, based on practical experience. These will be dealt with in a later report.

13. PRELIMINARY EXPERIMENTS WITH A MODEL OF THE GANGES AND THREE OF ITS TRIBUTARIES TO DEMONSTRATE THE MEANDERING TENDENCY OF RIVERS CARRYING SILT IN AN ALLUVIAL PLAIN.

A very small model of three tributary rivers was made, as shown in the plan figure 58, to see whether conditions such as occur in Bihar could be reproduced qualitatively, to determine whether the wide variations in the positions of rivers—over a width of as much as 60 miles—can be mainly ascribed to the construction of flood embankments or is natural.

The model was to a scale of approximately 1 foot equal to 10 miles. Photo 1588 (sheet 19) shows the model as laid.

Discharges and silt charges in the Tributaries and in the Ganges were according to table 10. The cycle of operations was for 2 hours, corresponding to one flood season in the prototype. The low water stage—for the remaining 8 months—was omitted, because no scour would occur in such a small model.

TABLE 10 FOR WORKING THE GANGES TRIBUTARY MODEL.

Period 2 hours \equiv 4 months in prototype.

	Time.	Ganges.	Tributary. A	Tributary. B	Tributary. C
Silt charge	15 Minutes	Low Flood	Low Flood	High Flood	Low Flood
	$\frac{1}{2}$ Cft.	$\frac{1}{2}$ Cft.	$\frac{1}{2}$ Cft.
Silt charge	15 Minutes	Rising Flood	High Flood	Low Flood	Low Flood
	$\frac{1.5}{8}$ Cft.	$\frac{1}{2}$ Cft.	$\frac{1}{2}$ Cft.
Silt charge	15 Minutes	High Flood	Low Flood	High Flood	High Flood
	$\frac{1}{2}$ Cft.	$\frac{1}{2}$ Cft.	$\frac{1.5}{8}$ Cft.
Silt charge	15 Minutes	Falling Flood	Low Flood	High Flood	High Flood
	$\frac{1}{2}$ Cft.	$\frac{1}{2}$ Cft.	$\frac{1.5}{8}$ Cft.
Silt charge	15 Minutes	Low Flood	High Flood	Low Flood	Low Flood
	$\frac{1}{2}$ Cft.	$\frac{1}{2}$ Cft.	1 Cft.
Silt charge	15 Minutes	Rising Flood	High Flood	Low Flood	Low Flood
	$\frac{1}{2}$ Cft.	$\frac{1}{2}$ Cft.	$\frac{1}{2}$ Cft.
Silt charge	15 Minutes	High Flood	High Flood	High Flood	Low Flood
	$\frac{1}{2}$ Cft.	$\frac{1}{2}$ Cft.	$\frac{1.5}{8}$ Cft.
Silt charge	15 Minutes	Falling Flood	Low Flood	High Flood	Low Flood
	$\frac{1}{2}$ Cft.	$\frac{1}{2}$ Cft.	$\frac{1}{2}$ Cft.

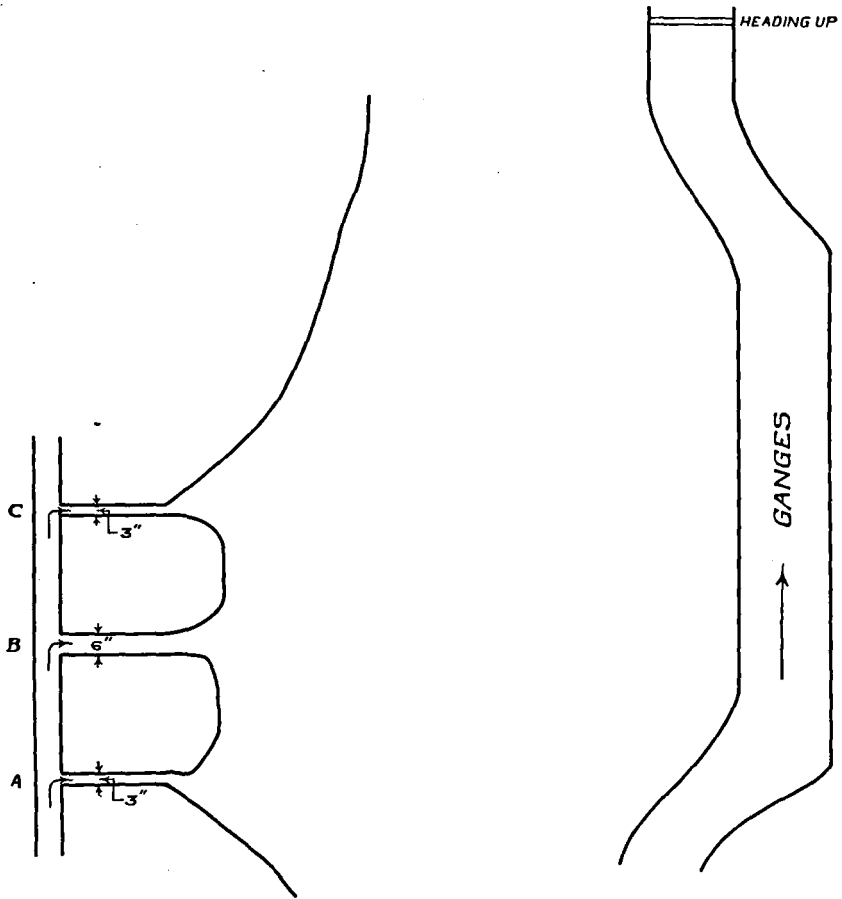
The ground across which the rivers were to flow was made level before the experiment was started. After 47 hours flow, with silt injection, it had attained a slope of 1 in 25.

It will be seen, from photo 1592, that for the first 30 hours each river was flowing through one channel—either its own or that of its neighbour—; but as the beds became higher, each river divided into a number of branches—*vide* photo 1595.

The central river (B) carried more water and more silt. It, therefore, raised its banks and bed quicker than the other two; and when its bed had risen higher than that of either A or C, it tended to flow into one or other of them.

This model was merely run to indicate general conditions, and being very small, its results were only very roughly qualitative. There is reason to believe, however, that the wide movement of Bihar rivers is natural; and though embankments must intensify the severity of changes, these would occur in any case, though more gradually and continuously.

FIG. 58,
PLAN OF 3-RIVER MODEL



14. RAISING THE WASTE WEIR AT LAKE ARTHUR HILL, BHANDARDARA (BOMBAY PRESIDENCY).

Introduction.

Lake Arthur Hill, in the Western Ghats, stores water for the Pravara Canals. When the dam was completed in 1926, the capacity of the lake was in excess of the demand for water, so automatic waste weir gates were not constructed. The waste weir consisted simply of a cut, 750 ft. wide, through the rocky saddle of the hill which forms the right flank of the gorge in which the dam is constructed; and so water was stored only up to the sill level of the cut, which is 10 ft. below the maximum safe level for the dam; the intention being to construct automatic gates later, when justified by the demand for water.

Two years ago, owing to the demand for water having overtaken the supply, this Station suggested that, instead of constructing automatic gates or their modern counterpart — automatic siphon spillways — a wall be constructed across the waste weir cut, the sectional profile being so shaped as to give a high coefficient of discharge, *i.e.*, induce a high discharge for a small head over the crest. The higher the coefficient, the higher the wall could be built for the same measure of safety, and the more water stored. With the coefficient actually obtained, this wall could store 6 feet more water compared with 9 feet by siphon spillways, while the cost of construction would be only a small fraction of siphons.

This solution was suitable not only on account of cheapness but also because this Lake does not fill in famine years; and so, if greater storage is desired, it would be better to provide it near Sangamner, as already accepted by Government.

The investigation can be conveniently divided into four parts, *viz.* :—

- I. Calibration of the existing weir cut and estimation of the maximum flood since the dam was built.
- II. Design of a high coefficient weir profile.
- III. Determination of the best alignment for the new weir wall, and calibration.
- IV. Calibration of a temporary weir wall constructed, pending the final design of the permanent one.

I.—Calibration of the existing weir, etc.

The calibration of the existing weir, by means of models, was described in last year's Annual Note on the work done by the Irrigation Research Division^(*) during 1936-37 in which it was shown that the discharge formula was

$$\left. \begin{array}{l} q=0.77 D^{1.95} \\ \text{or } Q=608 D^{1.95} \end{array} \right\} \dots \dots \dots (1)$$

where q =discharge in cusecs per foot width

Q =total discharge in cusecs.

D =Depth of water in lake above weir sill level.

It was also mentioned in the note that the maximum flood discharge of the catchment of the lake, which is 47 square miles in area, would, according to the Inglis multiple fan catchment formula, be 44,400 cusecs, but that pending further investigations the figure given by Beale's curve, which contains some allowance for flood absorption, was adopted, *viz.*, 28,000 cusecs. Using formula (1), it was found that the greatest discharge over the waste weir in the 10 years since the dam was completed occurred on 24th July 1937 with a gauge reading of 4.25' and amounted to 10,270 cusecs.

It was later proposed that, taking into account the great flood absorptive capacity of the lake, the figure for design purposes should be 20,000 cusecs, which allows a large margin of safety. As will be seen from IV, a flood of very exceptional size was experienced in 1937, when the discharge over the waste weir, with temporary wall constructed, was estimated to be 16,400 cusecs.

(*) Bombay Technical Paper No. 58.

II.—Design of High Coefficient Weir Profile.

The shape of the weir profile is shown in figure 59. The principle is that owing to the sharp curvature of flow, a region of low pressure is induced along the lower nappe which creates a steep pressure gradient in the filaments in the direction of flow, thus producing a high discharge for a given head of flow.

As the discharge is gradually raised, the pressure on the upper part of the curved face of the weir, which with low head is positive, reduces, particularly at the crest, where the curvature is greatest. The higher the discharge the lower these pressures, until a stage is reached when the boundary layer separates from the face of the weir and a back roller forms. From this point, the coefficient of discharge C , in the formula

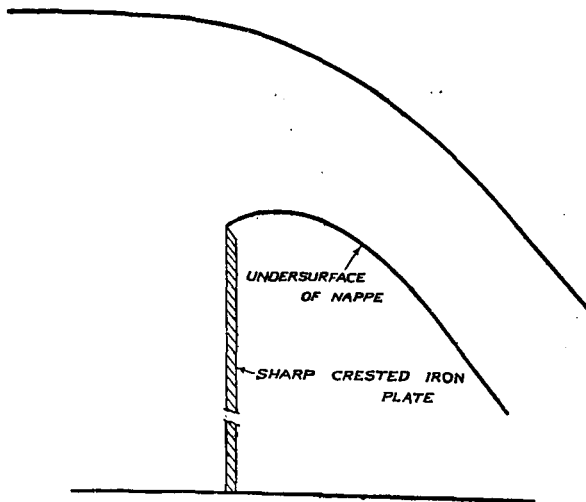
$$q = C D^{3/2} \quad . \quad . \quad . \quad . \quad . \quad (2)$$

(where D is the effective upstream depth over the crest, allowing for velocity of approach) decreases.

The object, in design, is to so shape the profile that the stage of separation with increasing discharge is delayed as far as possible.

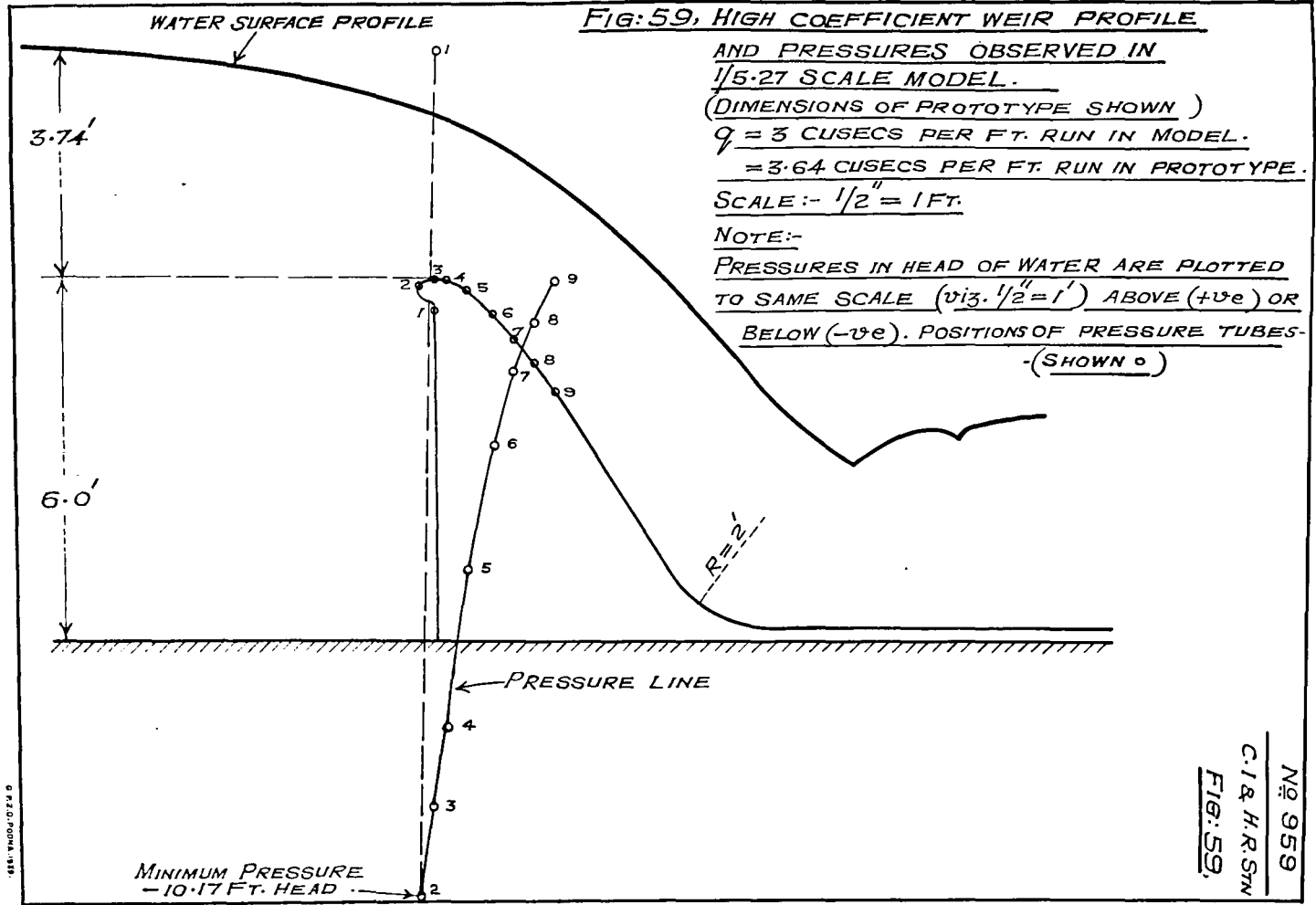
As the design of the best form of profile on a purely theoretical basis is not feasible, it was developed by model experiments carried out in parallel sided channels. As a starting point, a curve was obtained by tracing the profile of the undersurface of a nappe springing clear from a sharp crested weir, see figure 60, and

FIG. 60.



shaping the weir to that. Better results were later obtained by tracing the profile of a nappe springing clear from a sharp-crested weir with a lip on the upstream face,—see figure 61—the lip being then incorporated in the weir as shown in figure 61. When the discharge over the solid weir, so shaped, is equal to the discharge over the sharp weir for which the profile was traced, the pressure along the face is very nearly atmospheric. As the discharge is raised further, the pressure drops and the coefficient increases, for the reasons stated above.

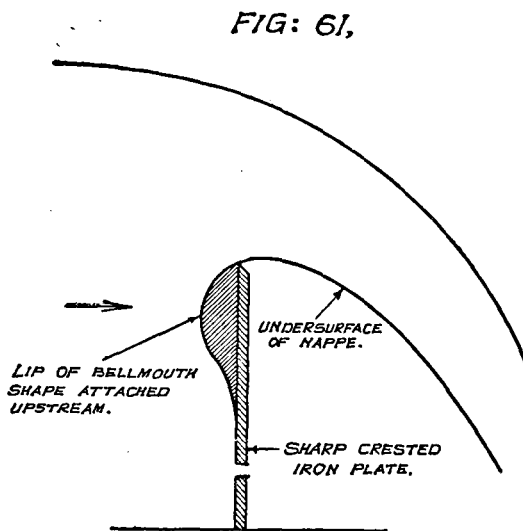
This is a case in which models do not reproduce the flow in the prototype, with dynamical similarity as gravity is a dominant factor in producing the flow against a resistance due appreciably to both inertia and viscosity, the effect of the latter lying in the development of a boundary layer and in the transmission



G.P.O. FORM 1515

No 959
 C. I. & H. R. STN
 FIG: 59

of momentum from the region of high velocity near the face of the weir to that of lower velocity at the upper surface, the flow being streamline. The coefficient which would be developed in the prototype had, therefore, to be estimated by extrapolation from the results of models of increasing scale ratios. Figure 62



shows the rise of the coefficient to a maximum and subsequent fall with increasing discharge in models of different scale ratios, and figure 63 shows the maximum coefficients plotted against scale ratios and extrapolated to prototype scale. It will be noticed that the 1/4-5 scale curve does not conform to the others. This is due to a recently introduced improved method of constructing and calibrating the model which eliminates the effects of the sides of the channel. When a final design has been obtained models to other scales will be calibrated in the same way, and fresh values for the prototype extrapolated.

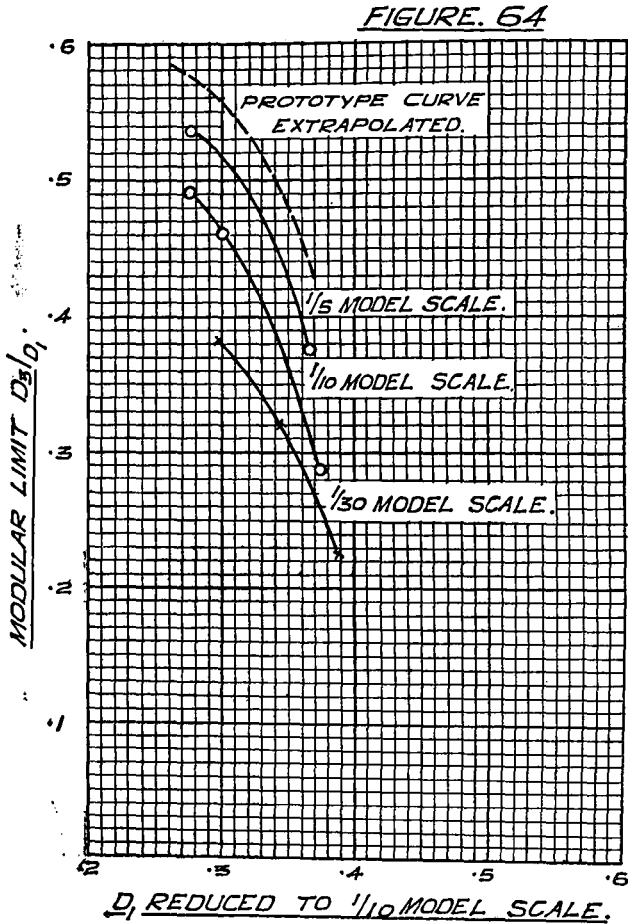
Figure 59 shows the pressures on the face of the weir as estimated for the prototype; the weir has to be designed structurally to resist these. It is also necessary to verify in such cases that cavitation will not occur owing to the reduced pressure. In practice this has been found to occur when the pressure reaches about 7 ft. of water (absolute) or, say 25 ft. below atmospheric pressure. It will be seen that the lowest pressure in the present case is equal to 10.2 feet of water below atmospheric pressure which is well on the safe side, but had the weir been more than twice the size there would have been danger of cavitation occurring and it would have been necessary to ease the curvature.

Another point which must be watched is whether there is a tendency for the flowing sheet of water to spring clear of the face of the weir, should air be given access; but the design shown in fig. 59 is found not to be subject to this occurrence owing to the rapid removal of air by the flowing sheet of water.

It is believed that the coefficient of about 5.0 anticipated for the prototype is considerably higher than that of any weir hitherto constructed in practice.

The modular limit, *i.e.*, the point at which a rising downstream water level begins to affect the discharge, is a factor of importance in this weir; because parts of the weir were some distance from the downstream edge of the cut so that the depth of water downstream of the weir was considerable. Figure 64 shows the modular limits obtained in the scale models with a range of upstream depths,

With the plan-alignment eventually selected (see III) there was free flow over the whole length of weir.



III.—*Determination of the best alignment and calibration.*

The waste weir, shown in figure 65, is irregular in plan and has a very rough surface, the rock protrusions varying from 1 to 2 ft. on the left (North) side to $\frac{1}{2}$ to 1 ft. on the right side. Apart from the roughness, the bed is about level.

The object being to pass a given maximum discharge with minimum upstream depth over the weir crest, curves are introduced to increase the length; but if the alignment is too serpentine, the overall coefficient is lowered, as the concentration of discharge at certain points, if upstream of the raised weir causes loss of head and may cause partial drowning of the weir, if downstream. The alignment which will give a maximum discharge lies between the two extremes.

10 different alignments were tested in a 1/30 scale model of the whole waste weir. Owing to the variety of roughness and the irregular shape, the flow was far from being parallel, and due to the small scale, which was unavoidable as the whole weir had to be included in the model, the weir wall differed from the prototype in respect of both coefficient and modular limit. The experiments

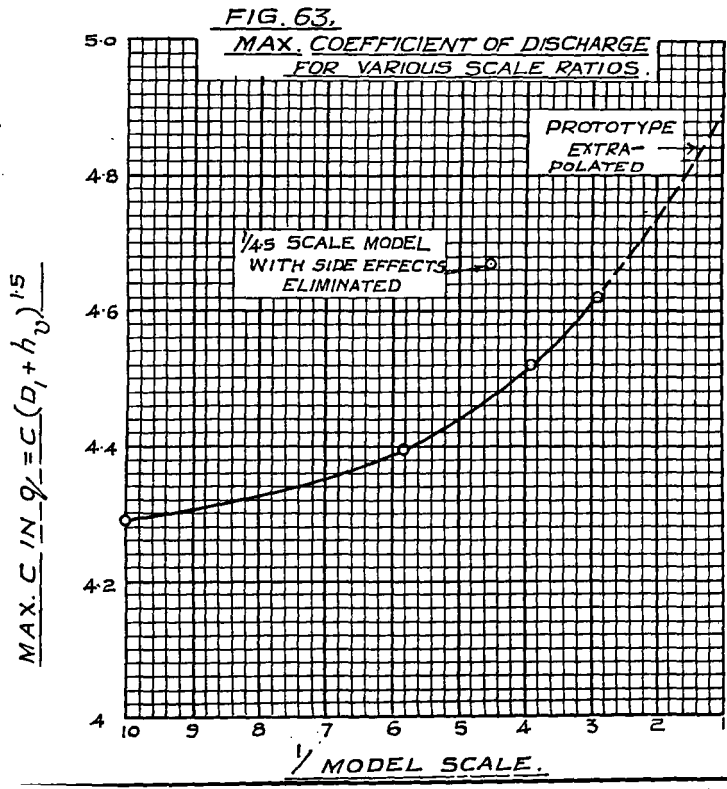
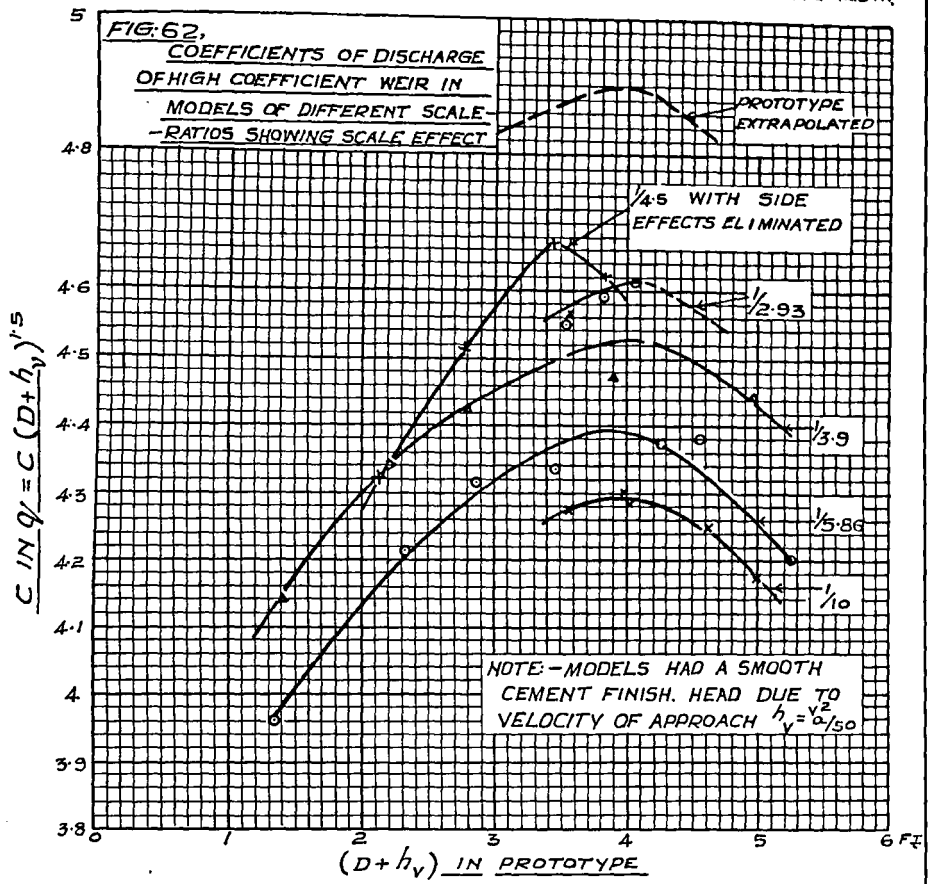
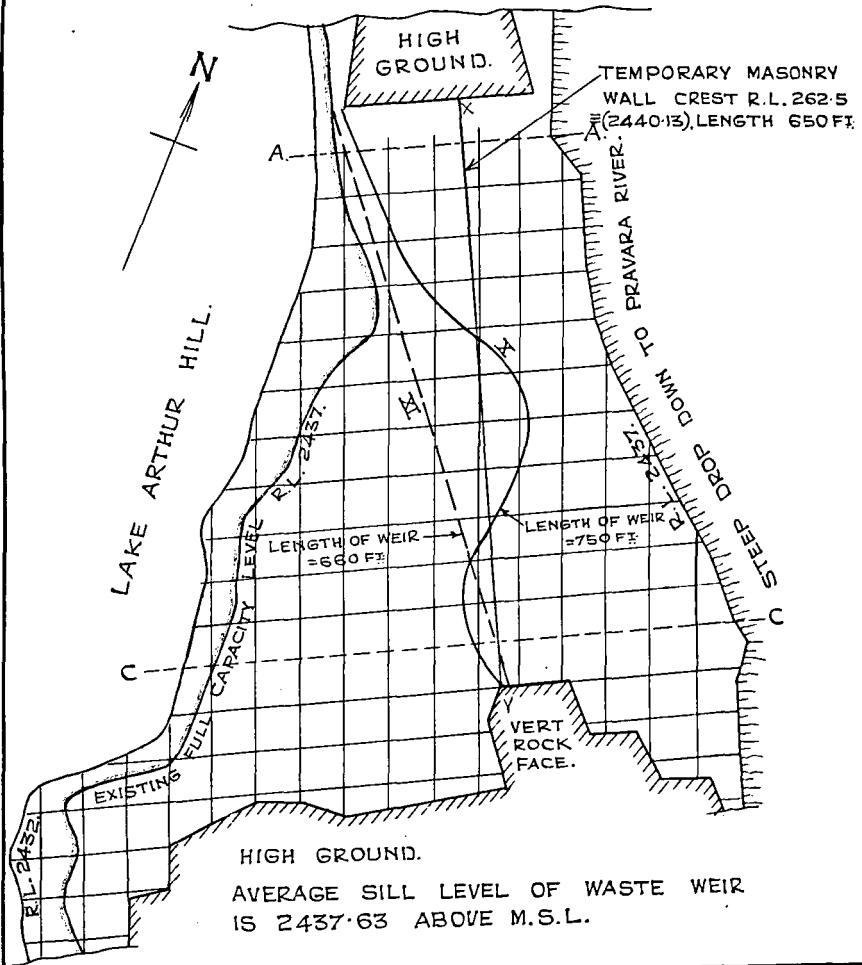


FIG: 65.
PLAN OF WASTE WEIR
OF
LAKE ARTHUR HILL
AT BHANDARDARA
AHAMEDNAGAR DIST.
SCALE:-150' = 1 INCH.



were therefore only qualitative, but in comparison with each other provided sufficient data to indicate the best alignment. The quality of the rock on the alignment, and particularly at the toe of the wall, also had to be considered; because in some places the rock in the bed of the cut was fissured and showed a tendency to disintegrate.

Alignment X was chosen as the most suitable. This has free flow over its whole length, which is 750 ft., compared with 660 ft. of the straight alignment IX (see figure 65). The following are the approximate comparative figures, based on designs for a flood discharge of 20,000 cusecs, which show that the additional annual return in years of full demand would cover the cost of construction 6 times over, and that the extra cost of the serpentine weir would be reimbursed in one year.

TABLE 11.

Alignment.	IX straight.	X Serpentine.	Increase of X over IX.	
Length Ft.	660	750	+90	
Maximum depth of water in lake over crest Ft.	3.55	3.34	-0.21	
Height of weir Ft.	0.45	0.66	+0.21	
Storage	M. Cft.	1,256	1,299	+43
		Cusecs for 8 months	60.5	62.6
Additional annual revenue at Rs. 1,500 per cusec	Rs. 90,750	93,900	+3,150	
Approximate cost of construction	„ 12,500	15,500	+3,000	

The waste weir with wall of curved high coefficient profile has not yet been accurately calibrated, pending the adoption of a final design of profile.

IV.—Calibration of Waste Weir with temporary masonry wall.

Pending the construction of a high coefficient weir with curved profile, a temporary masonry wall 2' 6" high, 650 ft. long, and of section shown in figure 66 and alignment in figure 65, was built straight across the waste weir cut in 1937 to provide immediate increase in storage, amounting to 471 million cubic feet.

On the 24th July 1937 the water level in the lake rose to 266.5, *i.e.*, 4 ft. above the crest of the temporary weir.

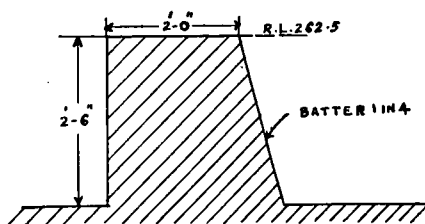


FIG. 66.

Model experiments have been carried out to calibrate the temporary weir and from the results a calibration curve for the waste weir has been computed.

The model experiments were done in 3 different scale ratios, *viz.*, $\frac{1}{4}$, $\frac{1}{5}$ and $\frac{1}{8}$, so that the prototype results could be extrapolated. The models were geometrically similar to the prototype and were tested in a 2' 6" channel. The results are shown in figure 67 where the coefficient C in the formula $Q = C B D^{1.6}$ is plotted against D, the upstream head including velocity of approach, $V^2/50$.

Experiments were also done with a rough bed upstream and downstream of the weir to reproduce more correctly the rocky bed of the cut in the prototype, but no appreciable effect on the weir coefficient was observed.

The value of C obtained for $D=4$ ft. by extrapolation from the model results, *viz.*, 3.375 compares with 3.03 obtained by interpolation from Cornell's experimental results for rectangular flat crested weirs with fully aerated nappes.⁽⁷⁾ In the experiments now described, the space behind the nappe was filled with a back roller which remained unchanged when admission of air was allowed, *i.e.*, the nappe was not depressed through non-admission of air.

The discharge over the whole waste weir for a given water level in the lake is dependent not only on the coefficient of the masonry weir but also on the roughness and length of the cut upstream. For calculation purposes the cut was considered in two halves, the left being 175 ft. long with rugosity coefficient

$N_a=0.40$, in the Lacey formula $V=\frac{1.346}{N_a} R^2 S^3$, and the right 250 ft. long with

$N_a=0.35$. Allowing for the drop in surface level at the upstream end of the cut, the energy loss in the flow through the cut, the velocity of approach to the weir and the variable coefficient of discharge of the weir, the calibration curve of the whole weir was drawn, shown in figure 68. It will be observed that the maximum discharge recorded on the 24th July, was estimated to be 16,400 cusecs. The catchment area is 47 square miles and the area of water surface at crest level 184 million square feet.

15.—EXPERIMENTS WITH A MODEL OF THE BRAHMAPUTRA AND THE PROPOSED EASTERN BENGAL RAILWAY BRIDGE AT AMINGAON.

Figure 69 is a plan of the Brahmaputra River at Amingaon for 6.2 miles above the proposed bridge site to 6.7 miles downstream. Upstream of the bridge, the river passes through a narrow section with rock exposed at places on both banks — shown in blue wash —.

The river is about half a mile wide, 2,400 ft. upstream of the bridge site — D D —. Below the gorge, the river banks diverge at 1 in 10 down to about 2½ miles downstream of the bridge site, where it splits into two channels.

The Railway company originally selected an alignment 1,000 ft. upstream of the site D D, now proposed; but the steamer companies stated that they required more room "to enable steamers with two or three flats or barges in tow to manœuvre and turn, at the height of the flood season, without being swept down by the current to the piers".

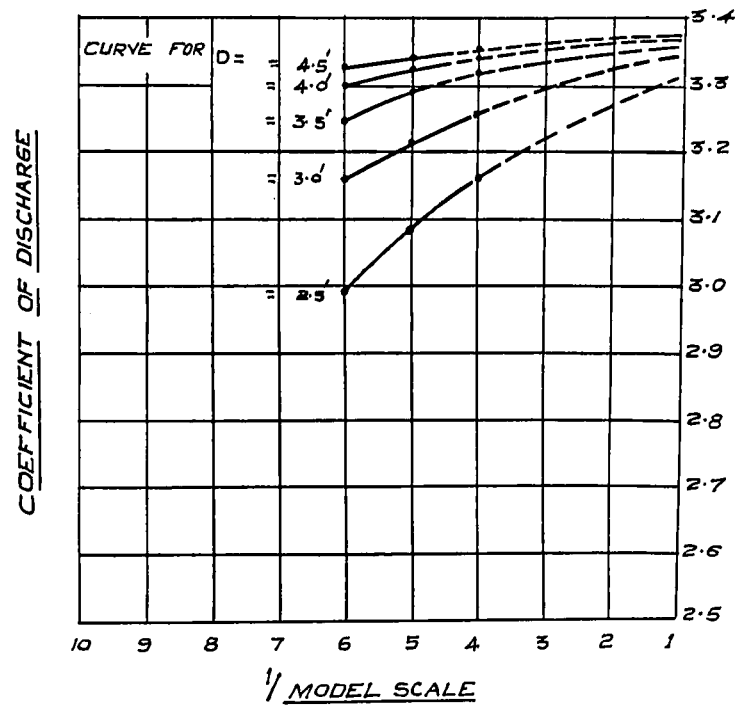
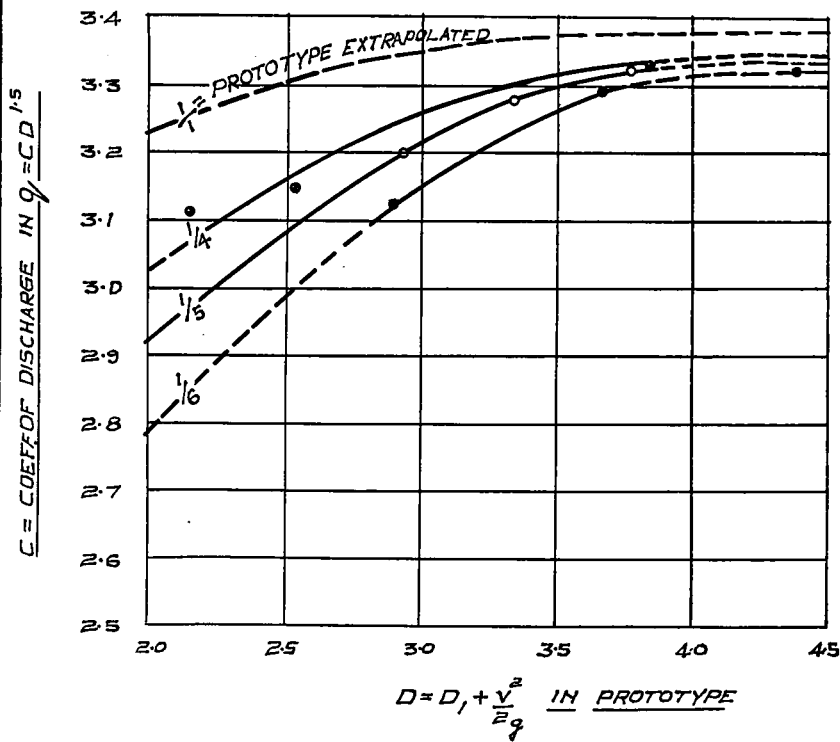
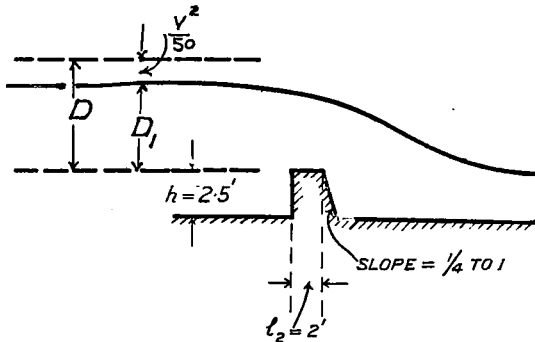
Table 12 shows Lacey silt factors based on sand samples collected at different depths below the bed.

TABLE 12.

Depth below bed where sample collected.	Mean diameter in mm. (including all sizes of particles).	Mean diameter in mm. excluding particles less than 0.08 mm.	$f_a=9.4\sqrt{d}$ (Lacey) d=mean diameter in inches, excluding particles < 0.08 mm. (determined by Puri siltometer).
(1)	(2)	(3)	(4)
0 to 65 ft.	0.385	0.390	1.17
65 to 75 ft.	0.397	0.406	1.19
75 to 80 ft.	0.545	0.545	1.37
80 to 90 ft.	0.438	0.438	1.24
90 to 95 ft.	0.640	0.640	1.49
95 to 115 ft.	0.623	0.623	1.47

(7) Gibson's Hydraulics, 3rd Edition, page 172.

FIG. 67.
COEFFICIENTS OF DISCHARGE OF
MODEL OF TEMPORARY MASONRY
WASTE WEIR AT
LAKE ARTHUR HILL.



No 942

C. I. & H. R. STN.

FIG. 68.

LAKE ARTHUR HILL WASTE WEIR

CALIBRATION. CURVE FOR

WASTE WEIR

WITH TEMPORARY MASONRY WALL

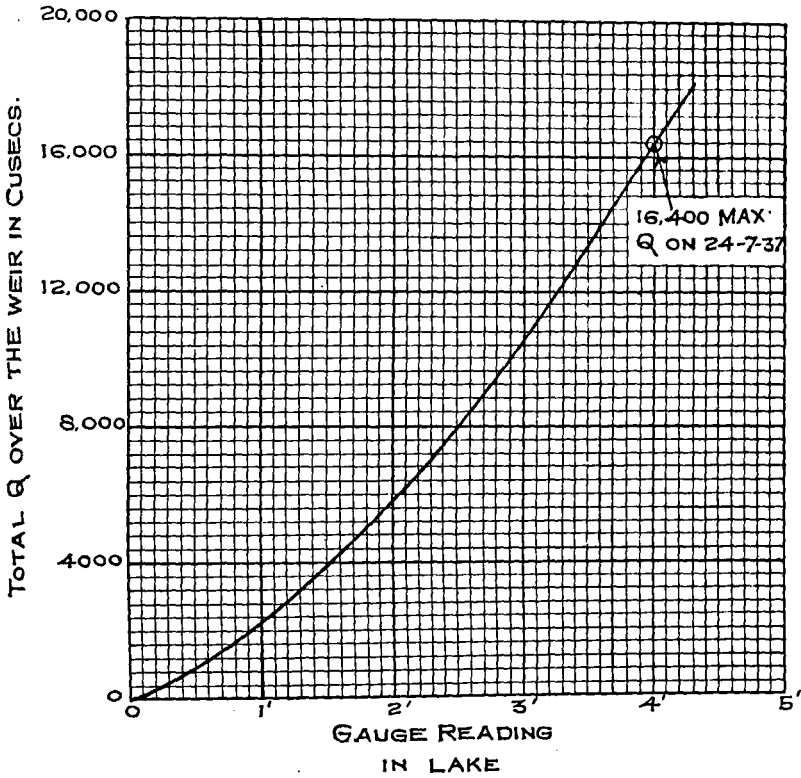
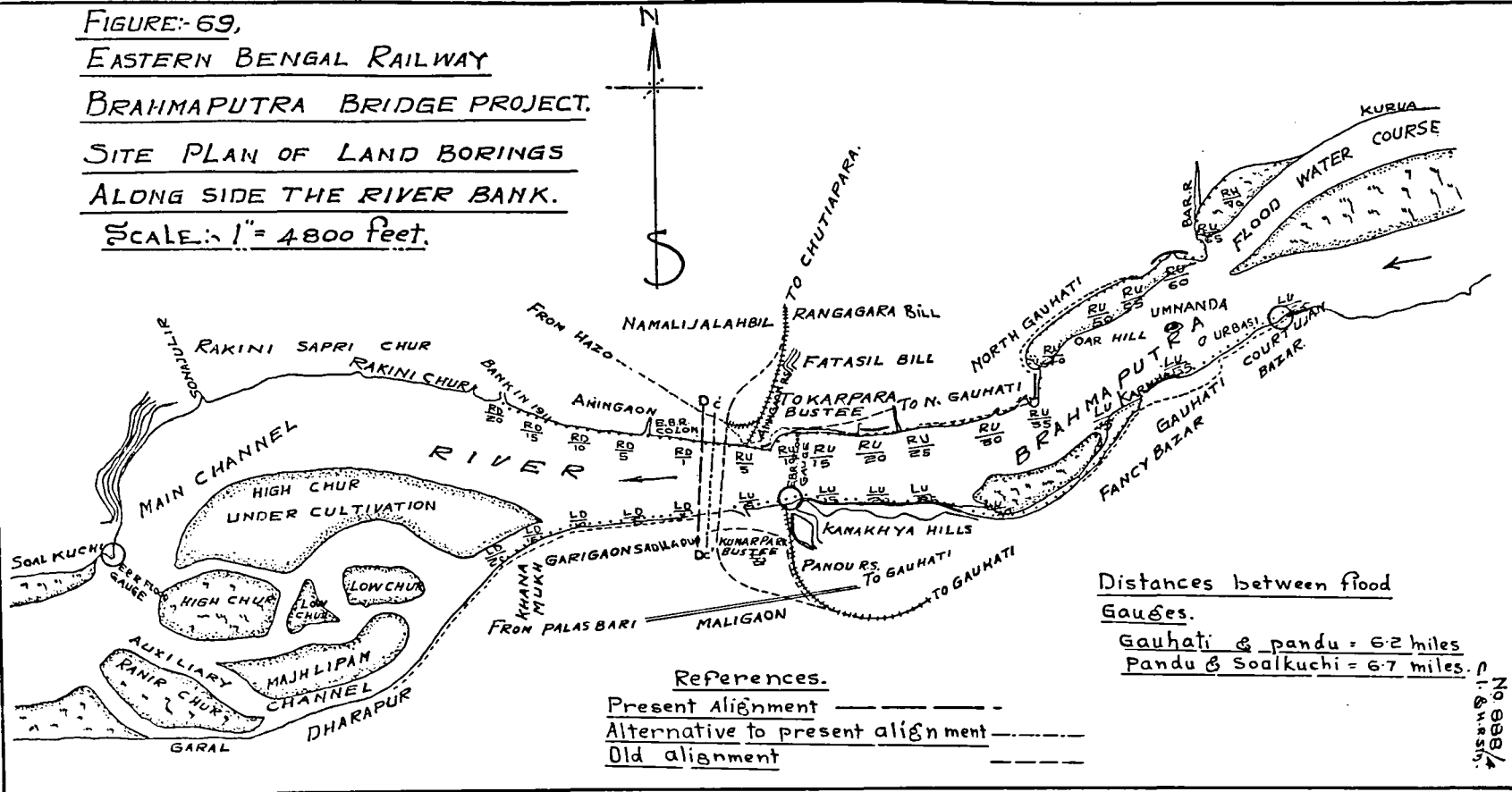


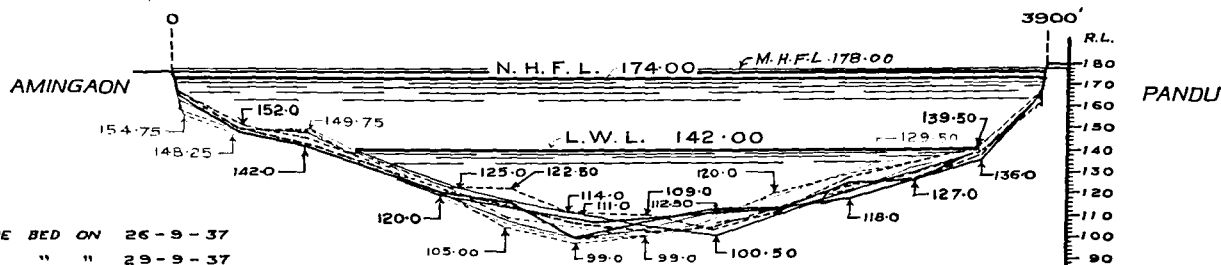
FIGURE-69,
EASTERN BENGAL RAILWAY
BRAHMAPUTRA BRIDGE PROJECT.
SITE PLAN OF LAND BORINGS
ALONG SIDE THE RIVER BANK.
SCALE: 1" = 4800 feet.



No. 998/4
 C. I. & R. S. No.

BRAHMAPUTRA BRIDGE PROJECT
CROSS SECTIONS ON ALIGNMENT LINE D.D.
BETWEEN 26-9-37 TO 5-11-37.

SCALE { HOR. 600 FT = 1 INCH
 VER. 60 FT = 1 INCH



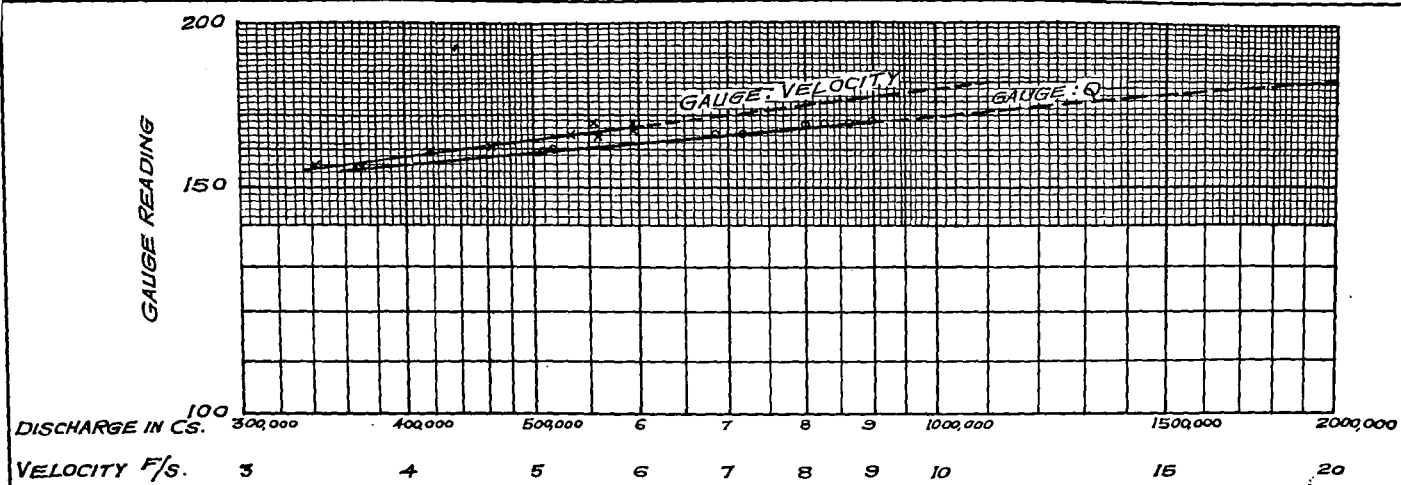
REFERENCES

FULL BLACK LINE	BED ON	26-9-37
DOTTED "	" " "	29-9-37
FULL RED "	" " "	5-10-37
DOTTED "	" " "	8-10-37
FULL BLUE "	" " "	14-10-37
DOTTED "	" " "	18-10-37
FULL BURNT SIENNA "	" " "	22-10-37
DOTTED "	" " "	26-10-37
FULL GREEN LINE "	" " "	29-10-37
DOTTED "	" " "	5-11-37

FIG. 70.

C. I. G. H. R. STN.

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BRAHMAPUTRA RIVER EXPT.

SITE DD

CURVES OF $G \propto Q$.

" $G \propto V$

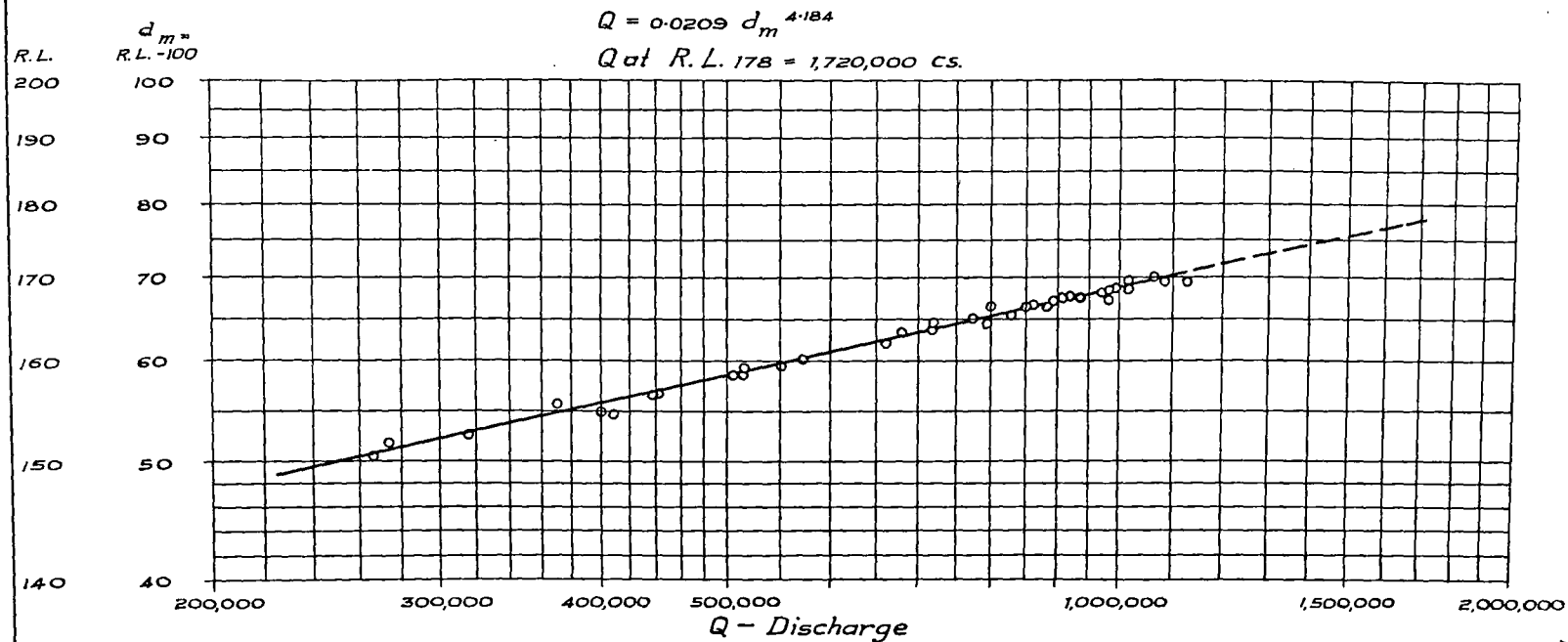
$Q = 1800,000$ FOR G.R.L. 178

$V = 10.3$ f/s " " "

FIG: 71.

NS 920
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Brahmaputra River Experiment.



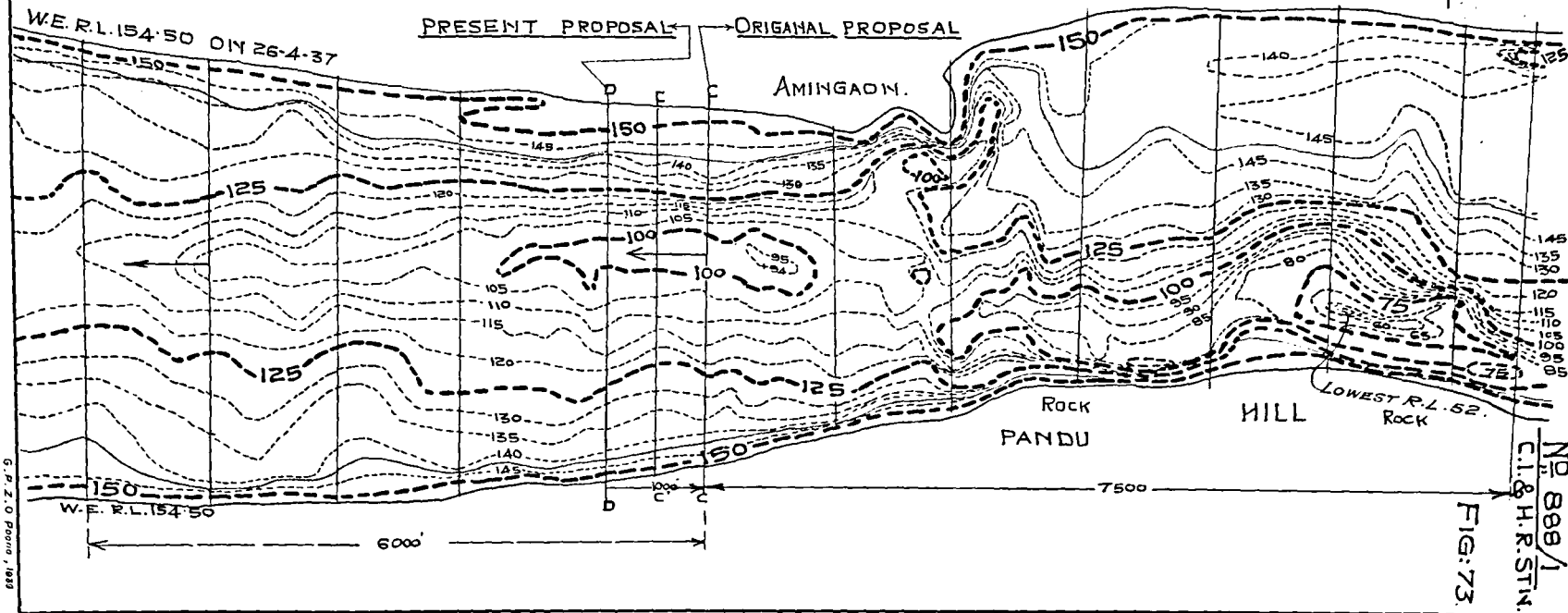
R.L. 100 has been assumed as R.L. at which $Q = 0$ and depths are calculated as above R.L. 100.

CONTOUR PLAN OF THE RIVER BED AT THE PROPOSED BRIDGE-
ACROSS BRAHMAPUTRA AT AMINGAON ON 26-4-37

SCALE :- 1 INCH = 1200 FEET

Q = 397.809 CUCECS

W.L. 154.5.

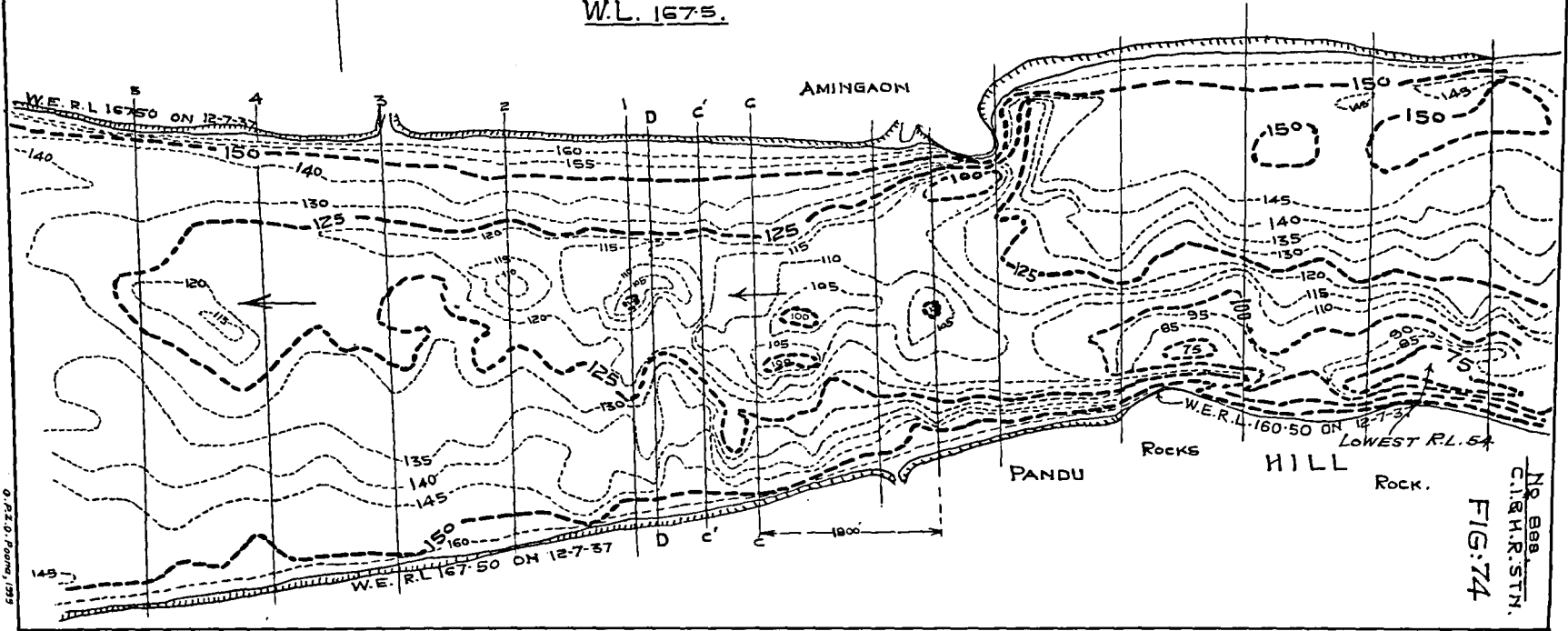


CONTOUR PLAN OF THE RIVER BED
AT THE PROPOSED BRIDGE ACROSS
BRAHMAPUTRA AT AMINGAON ON 12-7-37

SCALE: 1 INCH = 1200 FEET

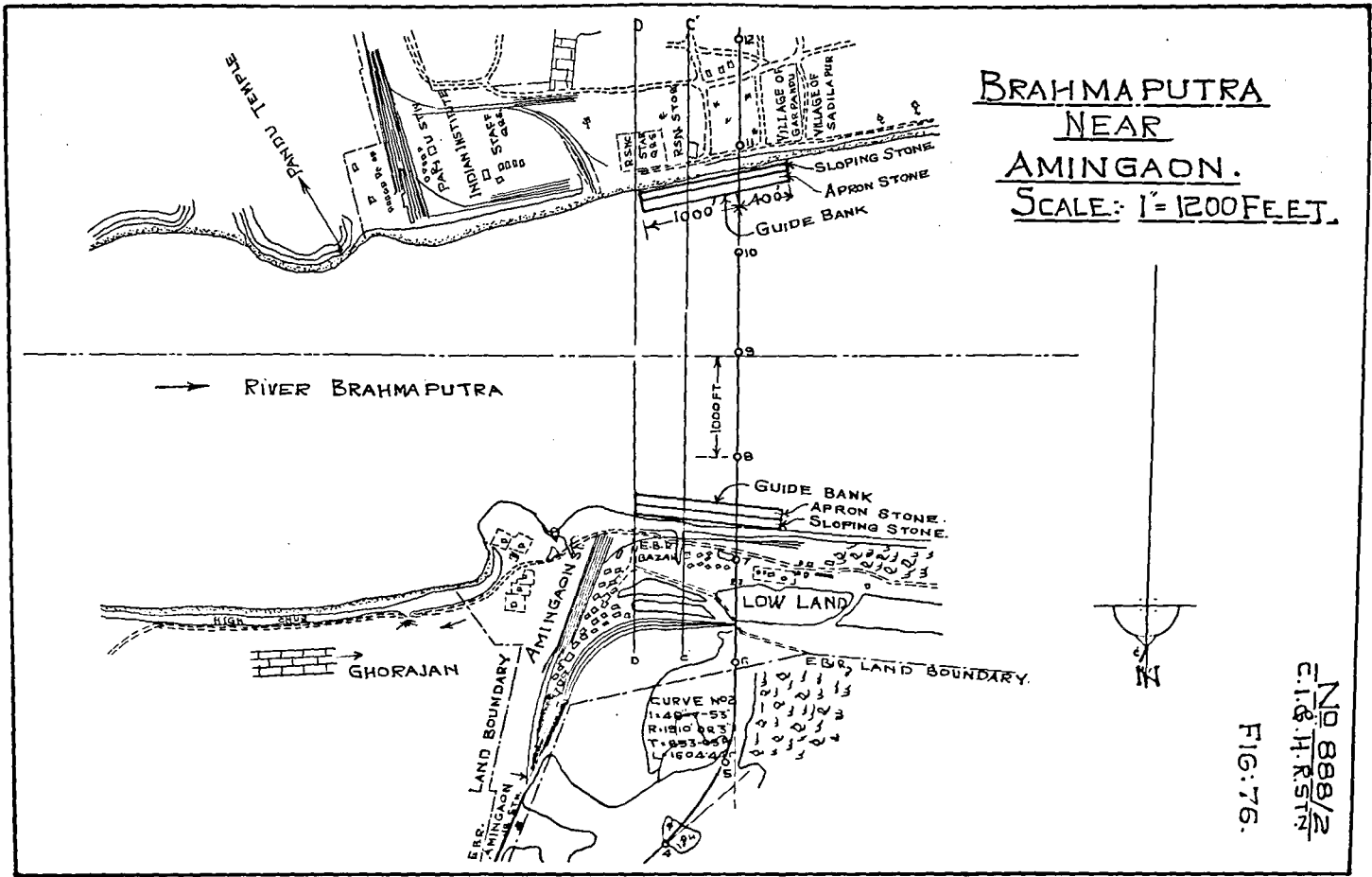
Q = 935237 CUCECS

W.L. 1675.



0.12.10. P. 125

No. 888
C. I. R. R. STN.
FIG: 74



BRAHMAPUTRA MODEL EXPTS.

SCALE HORIZ. 300=1" (=1' IN MODEL)
 VERT. 50=1" (=1' IN MODEL)

C.S. AT BRIDGE SITE WITH AND WITHOUT PIERS:

FIG: 77
 BRIDGE SITE DD
 C.S. WITHOUT PIERS & GUIDE BANKS

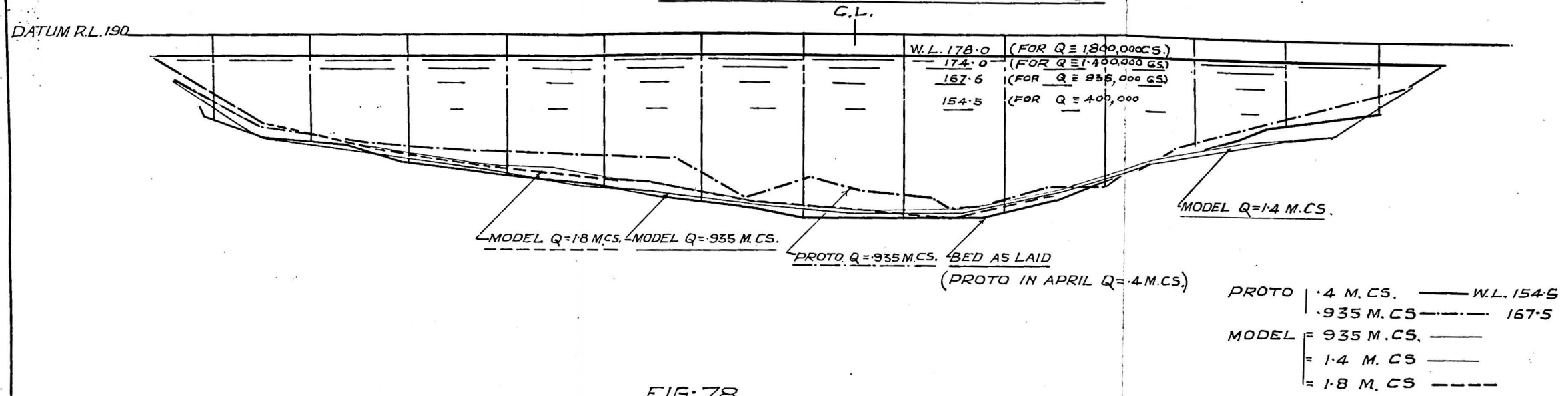
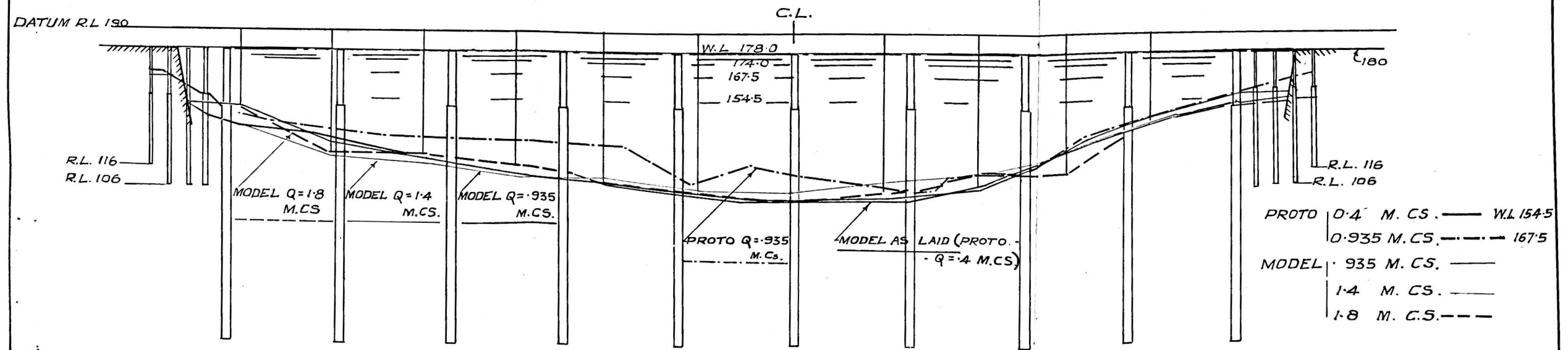


FIG: 78.
 BRIDGE SITE DD
 C.S. WITH PIERS & GUIDE BANKS



Sand at lower depths was coarser than at river bed. For working out scale ratios, the lowest value of $f_0=1.17$ has been adopted; because fine silt gives maximum scour.

Figure 70 shows cross sections observed at the Bridge site DD on various dates. From this it will be seen that the lowest bed R.L. only varied between R. L. 99 and 102.5, *i.e.*, scarcely at all. From the same data, C in the formula

$$A=C B_s d_m$$

—where B_s =surface width

d_m =mean depth

& A=sectional area,

shows no trend, and the average value of C works out to 0.575, as against 0.79, had the channel been semi-elliptical.

Figure 71 shows Q and V plotted against gauge R. L. From this curve, extrapolated, Q_{max} with gauge R. L. 178 — maximum W. L. — works out to 1,800,000 cusecs

$$Q=C B_s d_m V$$

$$Q=1,800,000 \text{ cs.}; C=0.575; B_s=3,880 \text{ ft.}$$

$$\text{and } V=10.3 \text{ f/s.}$$

$\therefore d_m=78.3 \text{ ft.}$, which indicates that R. L. of bed at maximum flood

$$=178 - 78.3=99.7 \text{ or say R. L. } 100.0.$$

Figure 72 shows Q against (G—100) for the data at Pandu; and the formula works out to

$$Q=0.0209 d_m^{4.184} \text{ and the discharge for G. R. L. } 178$$

=1,720,000 cusecs, or slightly less than the figure calculated above.

Figures 73 and 74 show contour plans of the bed from 7,800 ft. upstream of the bridge alignment (D D) to 5,600 ft. downstream for

(i) W. L.=R. L. 154.50 and $Q=397,800$ cusecs (=22 per cent. of maximum Q), and

(ii) W. L.=167.50 and $Q=935,237$ (=52 per cent. of maximum Q).

The plan shows deepest scour 7,800 ft. upstream of the bridge site at R. L. 52; and there is no danger of scour moving downstream; because scour is local and is due to concave curvature of the river and exposed rock — *see* figure 69.

Figure 75 shows the proposed bridge on alignment DD with 9 river spans of 374 ft. each, and 4 land spans on each bank.

Piers.

It is proposed to construct piers 30 ft. wide from bed to L. W. L. (R. L. 142), and 24 ft. wide from L. W. L. to H. F. Level, and to sink the piers 120 ft. below L. W. L., allowing for 20 ft. possible scour due to piers and guide banks. This means that

$$\text{L. W. L.}=R. L. 142; \text{ so bottom of piers}=142-120=R. L. 22$$

$$\therefore \text{Depth of pier bottom below H. F. L. R. L. } 178-22=156 \text{ ft.}$$

Present lowest bed level is at R. L. 99 and 20 feet scour allowed for, or say R. L. 79; then depth of water at maximum flood=178—79=say 100 ft. and depth of pier below B. L.=79—22=57 or more than half the depth of water, and hence considered to be safe.

Figure 76 shows the proposed guide banks extending 1,000 ft. upstream and 400 ft. downstream of the bridge. It has been recommended that the pitching should conform as nearly as practicable with the existing banks, so as to interfere as little as possible with present conditions of flow; and to this end it has been proposed that instead of stone being laid as an apron and allowed to launch; silt should be removed and the stone laid, so far as practicable, where finally desired.

Experiments.

The aim of the model experiments was—

- (i) to reproduce similar conditions to those in the prototype when the same discharge was flowing,
- (ii) to determine scour caused by a maximum flood,
- (iii) to determine the effect of adding piers and guide banks on scour.

Brahmaputra sand was used in the model.

Scale ratios were worked out from Lacey formulæ:—

$$\text{Discharge scale ratio} = \frac{13.8}{1,000,000} = \frac{1}{72464}$$

$$\text{Velocity scale ratio} = \frac{1}{6.32}$$

$$\text{Longitudinal and latitudinal scale ratio} = \frac{1}{300}$$

$$\text{Depth scale ratio} = \frac{1}{40}$$

$$\text{Slope scale ratio} = 6.32$$

Preliminary experiments showed that the vertical exaggeration was less than estimated and that the depth scale ratio should be 1/50 instead of 1/40. The discharge scale ratio was altered to suit the new depth scale, keeping velocity scale ratio the same.

$$\begin{aligned} Q_m/Q_p &= \frac{L_m}{L_p} \cdot \frac{d}{d_p} \cdot \frac{V_m}{V_p} \\ &= \frac{1}{300} \times \frac{1}{50} \times \frac{1}{6.32} \\ &= \frac{1}{94800} \end{aligned}$$

The following discharges were run successively:—

- Q=4.22 cusecs (≅400,000 cusecs)
- =9.88 cusecs (≅935,000 cusecs)
- =14.78 cs. (≅1,400,000 cs.)
- =19.0 cs. (≅1,800,000 cs.)

To start with, sand was laid on the bed according to April 1937 contours. Experiments were done (i) without and (ii) with piers and guide banks. Cross sections upstream and downstream of the bridge site DD were observed at the end of each experiment. Photos 1680 and 1681 (sheet 20) show conditions of flow at the bridge with a discharge ≅ 935,000 cusecs.

Figure 77 shows the cross section at the bridge before piers were added, and figure 78 after piers had been added.

Table 13 shows the equivalent bed R. Ls. in the model with and without piers, and 'actual bed R. Ls. without piers' in the prototype.

These experiments showed that the lowest bed R. L. at the bridge site in the model was unaffected not merely up to 935,000 cusecs, as in the prototype, but also up to a maximum discharge of 1,800,000 cusecs.

Experiments to determine scour round piers—geometrically similar — are in progress.

16.—NOTE ON EXPERIMENTS WITH A MODEL OF THE WASTE WEIR CHANNEL OF THE SPIN-KAREZ RESERVOIR—BALUCHISTAN.

(1) *Introduction.*

There were hitherto two sources of water supply for Quetta—

- (i) at Urak, and
- (ii) Nar Nullah dam.

It is now proposed to supplement the existing supply by constructing an earthen dam across the Spin Karez River, 7 miles upstream of Quetta, the catchment area of which is 28 square miles.

TABLE 13.—*Bed R. Ls. at Bridge site DD.*

	Left side.							C. L. 0	Right side.						
	2,100	1,800	1,500	1,200	900	600	300		300	600	900	1,200	1,500	1,800	2,100
C. S. AT BRIDGE SITE DD.															
<i>Model—without piers.</i>															
Q = 0.4 million cs.	141	132	126	119	112	107	102	102	112	120	134	142
= 0.935 " "	139	132	126	117	112	106	102	102	112	127	135	141
= 1.4 " "	144	134	127	124	117	107	104	104	112	120	137	142
= 1.8 " "	147	134	128	122	117	108	104	100	112	120	135	142	164	..
<i>Model—with piers.</i>															
= 0.4 " "	151	132	123	113	108	103	101	103	114	126	141	152
= 0.935 " "	150	130	123	114	110	104	101	102	107	128	145	157
= 1.4 " "	160	139	131	119	112	112	91	104	111	114	120	142	152
= 1.8 " "	152	128	127	121	112	107	102	107	117	117	142	155
<i>Prototype—without piers.</i>															
Q = 0.4 " "	140	137	124	118	111	105	100	99	108	127	140	151
= 0.935 " "	145	137	133	132	130	110	115	105	115	128	144	157

PART III.

The dam is to be 2,375 ft. long with a maximum height of 83 ft., to store water up to R. L. 6510.0, with a maximum flood level of R. L. 6516.5. Contents of the Lake = 162 million Cft. and the equivalent inch-depth over the catchment = 2.5 inches.

The waste weir has been designed for a discharge of 12,500 cusecs. This discharge has been based on the North West Frontier Province formula $Q=1220A^{2/3}$, where A is the catchment area in square miles.

$$\begin{aligned} \text{So } Q &= 1220 A^{2/3} \\ &= 12,146 \text{ cusecs.} \end{aligned}$$

The questions to be investigated were :—

- (1) the discharging capacity of the Waste Weir channel as designed,
- (2) the extent to which scour would occur in the waste weir channel upstream of the weir and whether measures were required to prevent such scour, and
- (3) how best to prevent scour in the long escape channel downstream of the Waste Weir crest.

I

(2) Experiments with the approach channel of the Waste Weir.

(a) Approach channel to Waste Weir.

Figure 79 shows the plan of the Waste Weir channel which was intended to pass 12,500 cusecs ; and figure 80, the longitudinal section of the channel as designed. It will be noted that the bed of the Waste Weir channel was designed sloping up from the Reservoir for 1,300 ft. at 1 in 1,000 ; and that the curvature in the bend had an inner radius of 265 ft. and an outer radius of 515 ft.

The first point to decide was the rugosity of the channel : The Spin Karez River has a slope of about 100 ft. per mile and according to the Lacey formula $S=0.00053 f^{5/3}/Q^{1/6}$, with a flood discharge of 12,500 cusecs — a figure roughly borne out by the width of the River — $f=22.0$; which, according to the Lacey regime formula would indicate a channel bed consisting of pebbles of a mean effective diameter approximating to $5\frac{1}{2}$ " ; but under non-regime conditions, it probably indicates a very heavy supercharge of silt, consisting largely of finer material. Nearly the whole of this material will be trapped by the Reservoir and only fine silt in suspension will pass through the waste weir channel.

Assuming a Lacey ' f ' value of 1.0, which would be approximately correct for clear water before scour exposes coarse materials ; and comparing these figures with the proposed dimensions :—

TABLE 14.

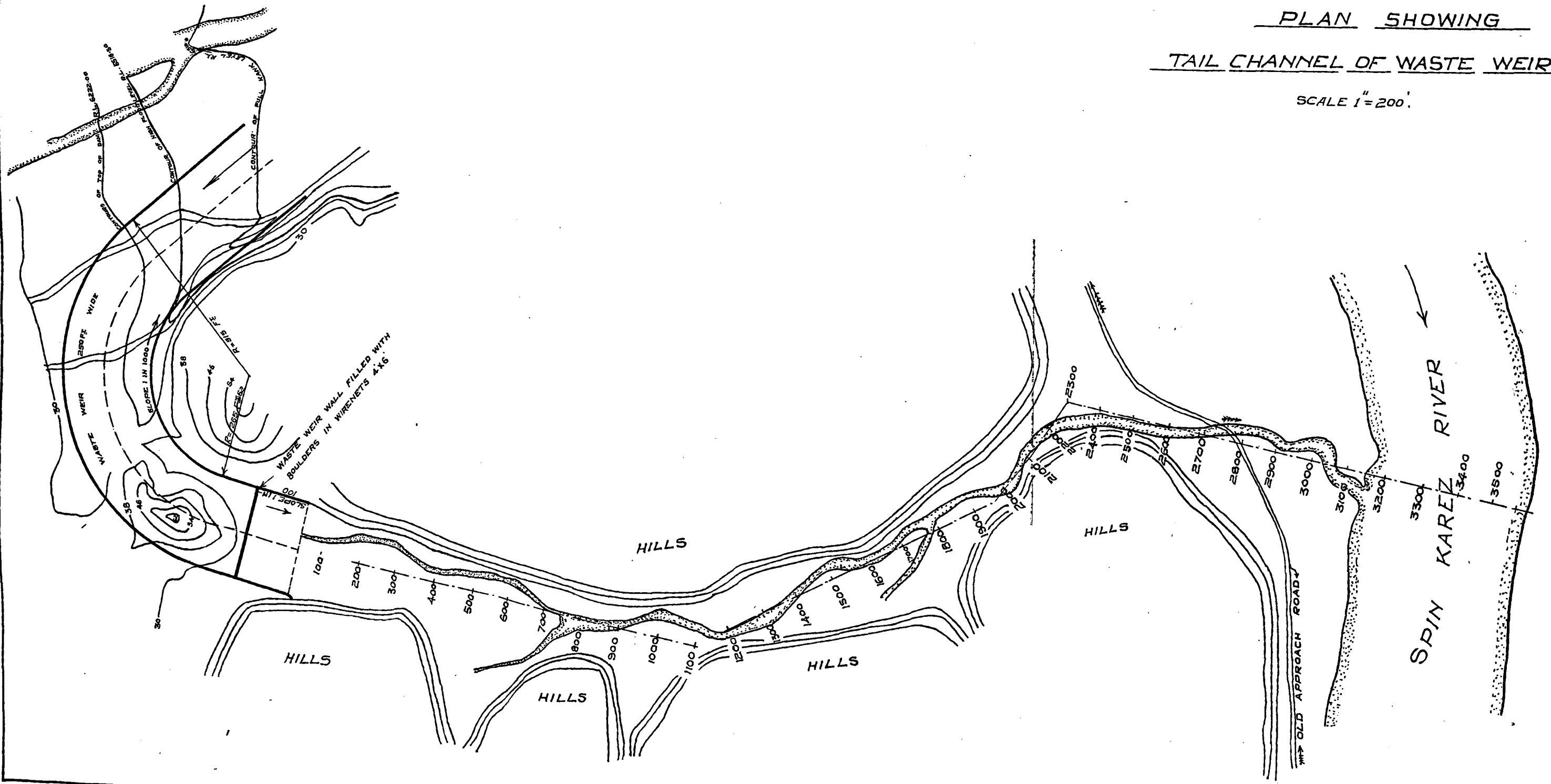
	Lacey f=1.	As designed	
		at entrance.	at Weir crest.
P (ft.)	298	265.6	283
R (ft.)	10.9	7.55	6.18
A (sq. ft.)	3,248	1,950	1,625
V (f/s)	3.85	6.41	7.7
Bed width (ft.)	270	250	250

It will be seen that the designed depth is quite unnatural and the velocity would be excessive. This was borne out by the model experiments, in which the outside of the curve and the channel bed scoured rapidly, with the result that pebbles embedded in the soil became exposed and the rugosity increased rapidly as the scour progressed. In the original design, it was assumed that the discharge formula, based on F. S. level in the Lake (= R. L. 6516.5), was $3.02 D^{1.5}$, where D=depth of sill below flood level in the Lake. This is the standard broad-crested weir formula : in other words, the design assumed no loss of energy between the Lake and the weir crest.

FIG: 79.

SPIN KAREZ RESERVOIR
PLAN SHOWING
TAIL CHANNEL OF WASTE WEIR

SCALE 1" = 200'



With $f=1.0$; and a regime channel 11 ft. deep and 270 ft. wide, the loss of energy in the length of 1,300 ft. would be $2''$; but with the designed channel scoured until lined with pebbles, the loss of energy head in the model was found to be equivalent to 1.14 ft. in the prototype — see figure 81 (L. S.); or alternatively, a flood level of 6,516.5 would be reached with a flood of approximately 10,000 cusecs.

Conclusions of I.

(i) Owing to the very sharp curvature in the Waste Weir channel, the outer bank must be lined with large stones from where the curve begins down to the weir crest. This outer pitched bank will scour at its toe in a flood of 12,500 cusecs to a total depth of 16 ft. or more — according to the material left as a residue.

(ii) The rising bed design (of Strange) was intended for waste weirs in rock or inerodible material and has no merits for Baluchistan conditions.

(iii) Either the channel must be designed large enough to take the F. S. Q. without scour or else the head lost in the Waste Weir channel will be considerable, necessitating either—

(a) an increase of High Flood Level in the reservoir from R. L. 6,516.5 to R. L. 6,517.5,

(b) lowering of sill level from R. L. 6,510 to R. L. 6,508.9, thus reducing maximum storage by approximately 7.5 million c.ft., or

(c) continuous removal of boulders exposed by scour from the bed (when the rugosity can perhaps be kept down to $N_s=0.025$), combined with a waste weir with $C=3.5$;

(c) is the best solution.

II

(3) *Escape channel downstream of Waste Weir crest.*

A model of the whole Waste Weir channel down to its junction with the Spin Karez River was constructed with a longitudinal scale of 1/20 and a latitudinal scale of 1/30.

It will be seen (from the L. S. in figure 80) that the ground slopes steeply downstream of the Weir at—

(i) 1 in 14 for 200' downstream of weir crest,

(ii) 1 in 28 from 200 ft. to 1,000 of weir crest, and

(iii) 1 in 33 from 1,000' to the junction with the River 3,300 ft. downstream.

The channel in its first 500 ft. length below the weir consists of shingly material (clayey silt, sand and boulders); and the remainder consists of black argillaceous shale, very hard when dry; but very soft when wet. There are no boulders in this material. The material of the first 500 feet has one virtue — that the boulders and shingle will tend to pitch the channel as scour occurs and will also provide boulders for filling up scour holes where they occur; but even this material cannot stand up to a slope of 1 in 28; while the material in the remaining 2,800 ft. of channel, down to the junction with the Spin Karez, has no merits at all, and will scour rapidly.

(4) *Experiments.*

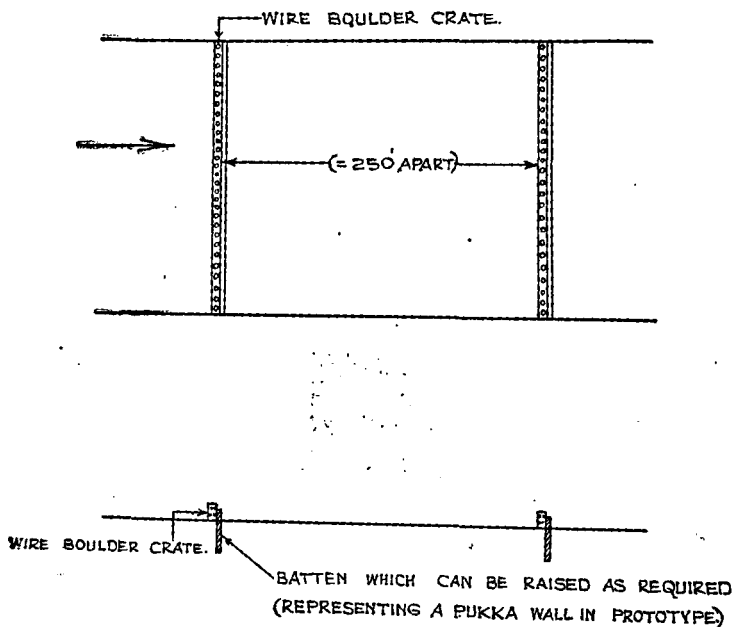
Experiment 1 without any devices to prevent scour:—Photo 1,575 (sheet 21) shows conditions of flow after 2 hours flow ($\cong 9.5$ hours in the prototype); photo 1,576 and figures 83 and 84 show the scoured bed. As was to be expected — seeing that the fall of the nala was about 100 times the natural regime slope — scour was extremely rapid, starting at the lower end and working upstream rapidly, showing how essential it is to provide adequate protection.

The original proposal was to give a bed slope of 1 in 100 for the first 100 ft. downstream of the Waste Weir crest and to pitch the bed and side slopes with stones — see figure 80 —; and in the remaining length — where the slope is 1 in 28 to 1 in 33, to construct crated boulder curtain walls at 500 ft. intervals, the number of these walls to be increased if found necessary.

This was tried in experiment 2.

Figure 85 shows the crates in experiment 2 supported by batters downstream—representing a pukka wall in the prototype. These were spaced 12.5 ft. apart (=250 ft.)

FIG: 85
PLAN & L. SECTION OF WIRE CRATES
SUPPORTED BY WALLS D/_S
SCALE: 1"=5.



L. sections and cross sections of this experiment are shown in figures 83 and 84. It was found that deep scour holes developed—*vide* hole marked 'A' in figure 83—which showed a tendency to work upstream—see photos 1579 and 1580 (sheet 22). The experiment was therefore discontinued, as results were definitely unsatisfactory.

Experiment 3.

Alternative methods of preventing scour were then considered:—

- (i) A series of baffle falls with cisterns and deflectors—
 - (a) without a glacis.
 - (b) with a glacis added downstream of the weir.
- (ii) A continuous rapid.
- (iii) A plain vertical fall.

These were all constructed in the Waste Weir channel and tested simultaneously.

Two baffle falls at R. D. 250 ft. and 500 ft. were constructed at the upper end—see figures 86, 87 and 88. The depth downstream of the fall was according to Kennedy's formula $V=0.84 d^{0.66}$. The falls were designed for $H=8$ ft. The height of the baffle= d_1-d_2 where $d_1=\sqrt[3]{q^2/g}$ and d_2 =free flow depth at toe. A deflector was constructed downstream of the baffle, with height= $\frac{1}{3}$ depth of water. Downstream of these two falls, a continuous rapid was constructed

FIG: 81.

SPIN KAREZ RESERVOIR EXPTS.

GRAPH SHOWING THE ENERGY OF FLOW
(i.e. $d + \frac{v^2}{2g}$) AT VARIOUS POINTS IN THE
APPROACH CHANNEL WITH $Q = 0.555$ CS.
(≈ 50 CS. PER FOOT RUN IN PROTOTYPE) AND

$Q = 4.63$ (≈ 12500 CU SECS)

VERTICAL SCALE 10:1

HORIZONTAL SCALE 100:1

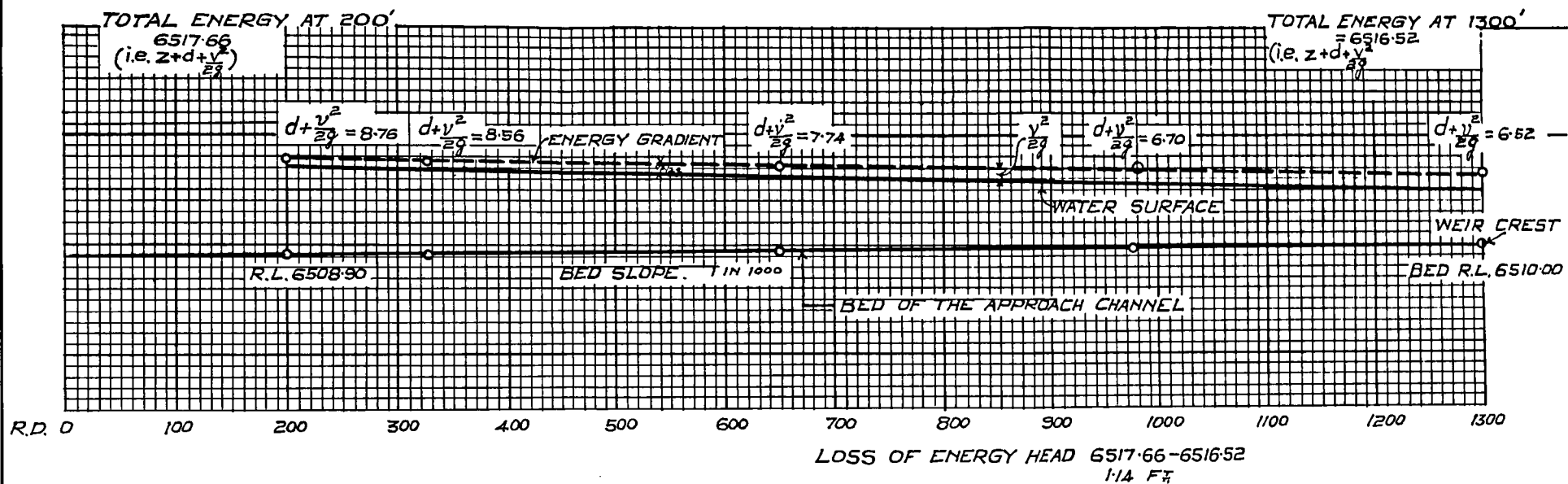
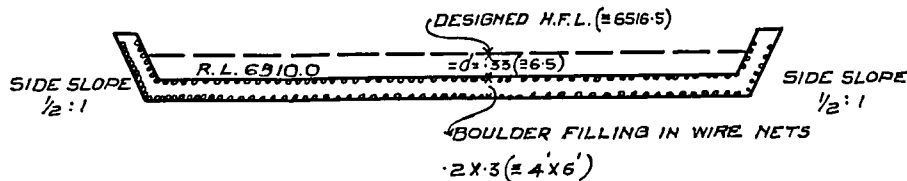


FIG: 82 C.S. (DESIGNED) OF WEIR AT R.D. 1300.



Spin karez expts: fig. Showing L. section
along the middle of Nala.

SCALE:- HOR:- 1 inch = 20 feet, (model scale = 1/20)
VER:- 1 inch = 2 feet.

L. section showing Nala bed after 2 hours (= 9.5 hours.)
flow with f.s.a.

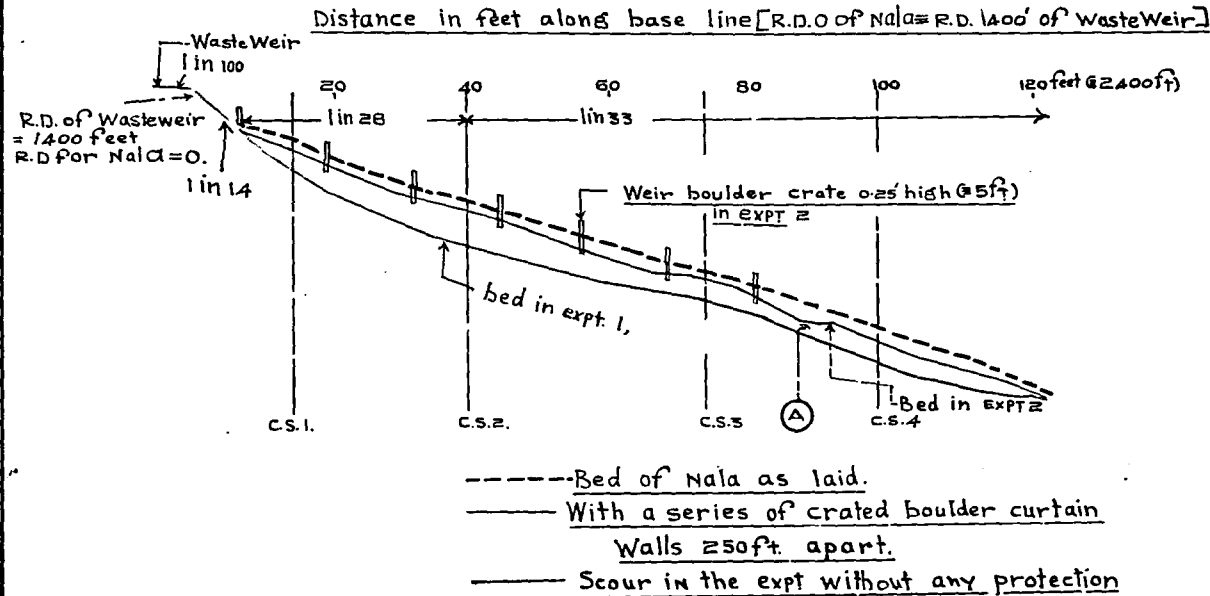
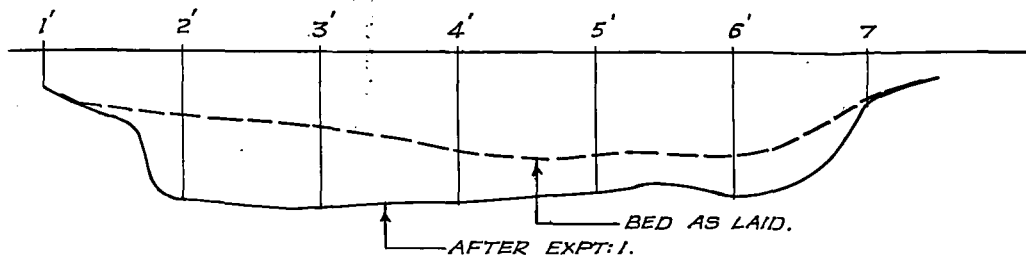


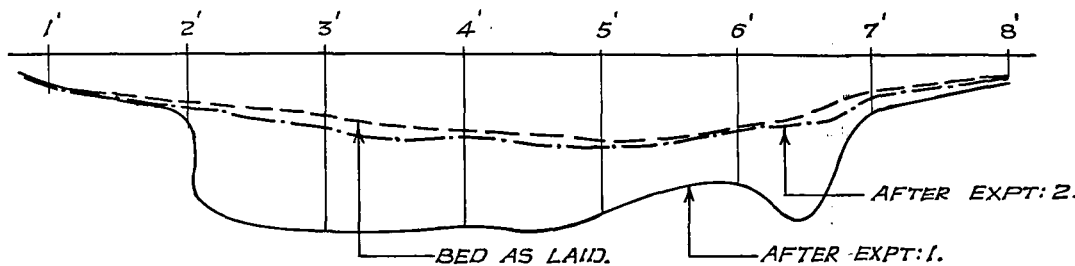
Figure:- 83.

N_o 547.
 C.I. & H.R.Shy.

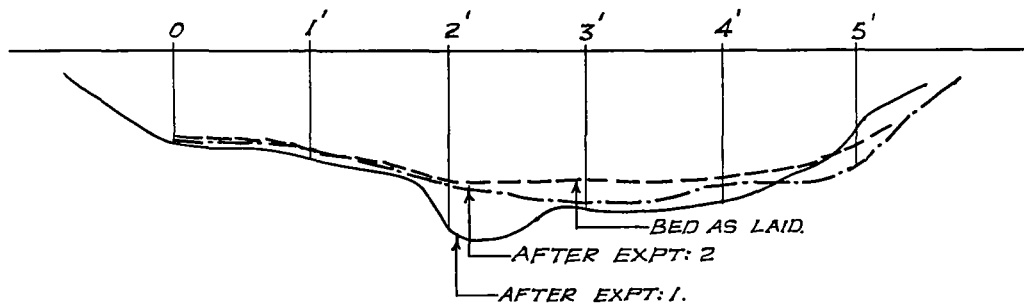
C. S. 1. AT R.D. 15 (≅300)



C. S. 2. AT R.D. 40 (≅800)



C. S. 3. AT R.D. 75 (≅1500)



C. S. 4. AT R.D. 100 (≅2000)

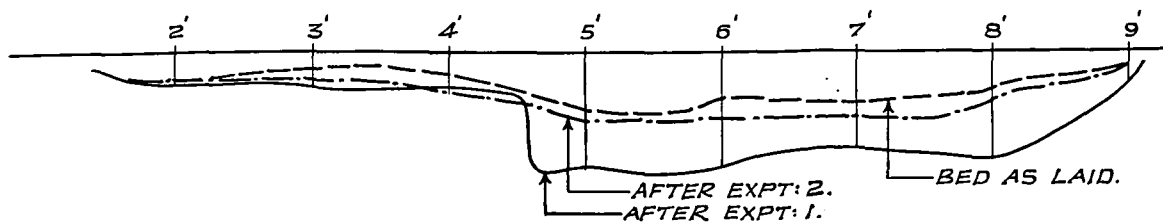


FIG: 84.

CROSS SECTIONS SHOWING THE SCOUR IN THE BED OF THE NALA D/S OF WASTE WEIR AFTER 2 HOURS (≅ 9.50 HOURS IN PROTO) FLOW OF F. S. Q. I. e. 4.46 Cs. (≅ 12500 Cs. IN PROTO) HDR: SCALE: 1" = 1 FOOT. VER: SCALE: 1" = 1 FOOT.

- CROSS SECTION AS LAID.
- CROSS SECTION AFTER 2 HOURS FLOW IN THE EXPT: 1. I. e. WITHOUT ANY PROTECTION.
- · - · - · - CROSS SECTION AFTER 2 HOURS FLOW IN THE EXPT: 2 I. e. WITH A SERIES OF CRATED BOULDERS 250' APART WITH F. S. Q.

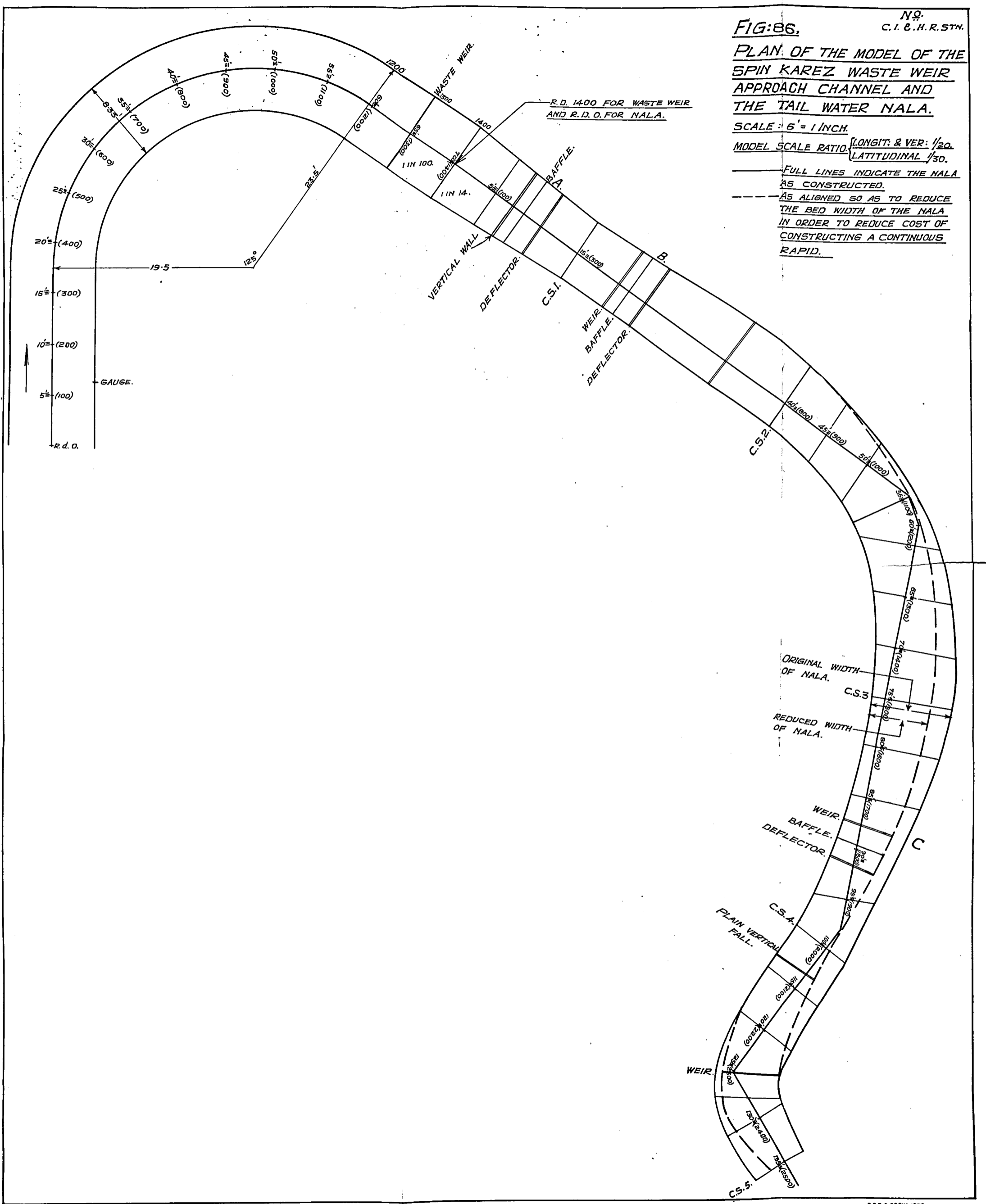
NOTE:-
SCOUR WAS IN PROGRESS IN EXPTS: 1 AND 2.
EXPT: 2 WAS DISCONTINUED AFTER 2 HOURS FLOW,
AS CRATES WERE BEING UNDERMINED RAPIDLY.

FIG:86.

PLAN OF THE MODEL OF THE SPIN KAREZ WASTE WEIR APPROACH CHANNEL AND THE TAIL WATER NALA.

SCALE: 6' = 1 INCH.
 MODEL SCALE RATIO: (LONGIT. & VER: 1/20. LATITUDINAL 1/30.

— FULL LINES INDICATE THE NALA AS CONSTRUCTED.
 - - - AS ALIGNED SO AS TO REDUCE THE BED WIDTH OF THE NALA IN ORDER TO REDUCE COST OF CONSTRUCTING A CONTINUOUS RAPID.



in the curved length of the channel, and in this length the width of the channel was gradually reduced from 8 ft. to 6 ft.—as shown in dotted lines in figure 86—to reduce cost—and the 'rapid' was given super-elevation. The 'rapid' was followed by another 8 ft. baffle fall—see figure 89—; but in this baffle fall, a glacis slope of 2 horizontal : 1 vertical was provided downstream of the weir ; below this, a plain 8 ft. vertical fall was provided—see figure 90.

(a) The baffle fall with a sloping glacis gave better results than the baffle fall without a glacis, in that dissipation of energy was more effective ; but all the baffle falls were effective in preventing scour—*vide* photo 1587 (sheet 23) showing conditions of flow.

(b) The lined rapid was very satisfactory ; but (c) heavy scour occurred downstream of the vertical falls.

(5) *Conclusions of II.*

The solution recommended to pass 12,500 cusecs is to construct—

- (1) a 3 ft. baffle fall with glacis, baffle, cistern and deflector to serve as weir crest—to maintain the F. S. D. upstream, without danger of scour downstream ;
- (2) an 8 ft. baffle fall at R. D. 200 of the Nala (or 300 ft. downstream of the weir crest) ;
- (3) an 8 ft. baffle fall at R. D. 500 of the Nala ;
- (4) a rapid from R. D. 700 to R. D. 2,600 ;
- (5) an 8 ft. baffle fall at R. D. 2,900 ; and
- (6) a baffle fall at the junction of the Nala with the River to prevent retrogression.

(6) *Recommendations.*

It is extremely difficult to put up definite recommendations in connection with this scheme ; because if the waste weir had to stand up to annual floods of 5,000 cs. or more, it would be necessary to protect the channel throughout its length as described in para. (5).

If such floods actually occurred annually, the reservoir would become silted up to waste weir level in a short time, and judged by the standards of Peninsular India this scheme would be condemned. It is obvious, however (both from letters received from local officers and from the fact that during the period April 1921 to December 1934, the maximum rainfall in any one month was 5.03" (in February 1931) ; that rain exceeded 3" in only 5 months in that period and exceeded 2" in only 20 months), that the framers of this scheme have assumed that the waste weir will seldom flow with more than a few hundred cusecs, and are 'banking on' large floods being of such rare occurrence, and short duration, that the damage done can be repaired before another large flood occurs. This would be cheaper than providing for the maximum flood at the start ; but it is for very serious consideration whether the risk is justified.

17. EXPERIMENTS WITH SILT ABRADER.

Despite the fact that silt becomes progressively finer from the hills to the sea, there has been a widespread opinion that this phenomenon could not be explained by abrasion (*i.e.*, by particles colliding with each other and thus becoming finer by mechanical action) ; because the time taken for silt, carried by water, to reach the sea was too short to explain the facts.

Experiments were therefore carried out with a silt abradar—starting in November 1932—to determine the rate of abrasion.

The silt abradar consists of a circular tank of 2 ft. radius, 4 ft. high, with a central shaft to which three tiers of blades are fixed, each tier being set at 40° to the other two. The shaft was revolved 30 times per minute, so the velocity of the blades at the outer edge was 6.9 f/s. Sukkur sand was placed in the tank to a depth of 6" and water maintained at a constant level, 3'-6" over the bed of the tank. The abradar has worked for 930 hours (=39 days) since last year's report, the total working hours to date being 7,150 hours (=298 days).

At the end of 7,150 hours, the sand on the bottom was thoroughly mixed and 6 random samples were analysed. The method of analyses was the same as for samples of the preceding years, viz., particles < 0.2 mm. were measured by the sedimentation method and coarser particles by sieves. Table 15 shows the results.

TABLE 15.—Showing various grades and percentage fraction of sand to total sand of 6 samples in the silt abrader after 7,150 hours working.

	Diameter of particles in mm.	Sample 1.	Sample 2.	Sample 3.	Sample 4.	Sample 5.	Sample 6.	Mean of 6 samples.
(1) Coarse sand 1.0 to 0.2 mm.	1.0 —0.5	0.072	0.042	0.122	0.078	0.065	0.079	0.076
	0.5 —0.33	1.202	0.502	0.777	0.637	0.760	0.827	0.784
	0.33—0.25	0.930	0.200	0.352	0.409	0.911	0.494	0.549
	0.25—0.20	43.187	33.510	35.820	43.515	45.330	37.506	39.811
		45.391	34.254	37.071	44.639	47.066	38.906	41.220
(2) Fine sand 0.2 to 0.02 mm.	0.2 —0.1	45.375	43.759	50.164	44.482	46.667	46.433	46.143
	0.1 —0.075	4.952	7.262	4.823	5.581	3.622	5.679	5.319
	0.075—0.04	2.484	9.520	4.757	3.564	1.760	5.587	4.612
	0.04—0.02	0.900	2.470	2.041	0.824	0.494	1.804	1.422
		53.711	63.011	61.785	54.451	52.543	59.503	57.496
(3) Very fine sand 0.02 and less.	0.02 and 0.02 and less.	0.305	0.306	0.165	0.043	0.062	0.259	0.190
	TOTAL . . .	99.407	97.571	99.021	99.133	99.671	98.668	98.906
Mean diameter in mm.	. . .	0.1806	0.1604	0.1702	0.1777	0.1830	0.170	0.1738
σ —standard deviation	. . .	0.0586	0.0628	0.0561	0.0562	0.0532	0.0612	0.05802

Table 16 below shows the weighted mean size in mm. of each of the six sample and the standard deviation ' σ '.

TABLE 16.

(1)	Weighted mean size in mm.	' σ ' (or standard deviation).
(1)	(2)	(3)
Sample 1	0.1806	0.0586
2	0.1604	0.0628
3	0.1702	0.0561
4	0.1777	0.0582
5	0.1803	0.0532
6	0.1700	0.0612
	Mean 0.1738 \pm 0.0073	0.05802 \pm 0.00325

The weighted mean size is 0.1738 mm. and the variation of ± 0.0073 is small and the value of P (Probability) is far below 0.01 against the limiting value of 0.05 laid down by Fisher.⁽⁴⁾

Standard deviation or σ .⁽⁵⁾

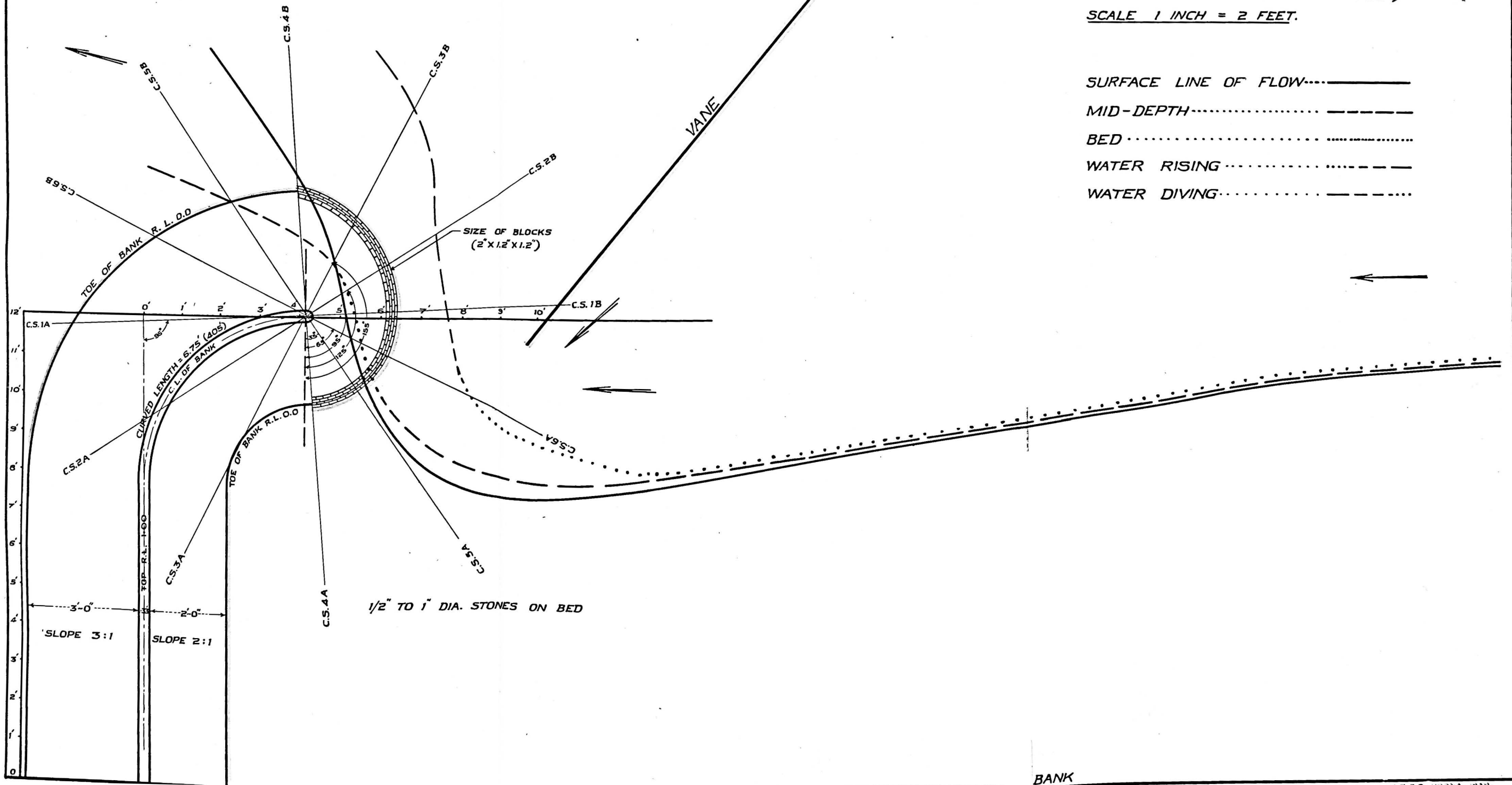
As it is possible that samples with the same mean size may differ, one having a large percentage of particles with diameters near the mean size and the other with diameters varying more widely, the standard deviation (σ)= ' the root mean square of the squares of divergences ' is calculated as a measure of the dispersion.

In this case the mean standard deviation=0.05802 and the variation= ± 0.00325 , which is very small, so the method of sampling is reliable. Table 17 shows the grades and the percentage fraction of grades of sand to total sand by weight—the average of 6 samples—in the original sand and sand after 5,382, 6,220 and 7,150 hours.

(4) " Statistical methods for Research workers " by Fisher—Table of P in Chapter 5.

(5) Punjab Irrigation Research Institute Publication, Vol. II, No. 15 " Punjab Practice in Silt Observations ".

FIG:100 - PLAN OF SPUR AS LAID BEFORE EXPERIMENTS. (GEOMETRICALLY SIMILAR MODEL OF THE BURMAH SPUR TO A SCALE RATIO OF 1/60).
SCALE 1 INCH = 2 FEET.



BANK

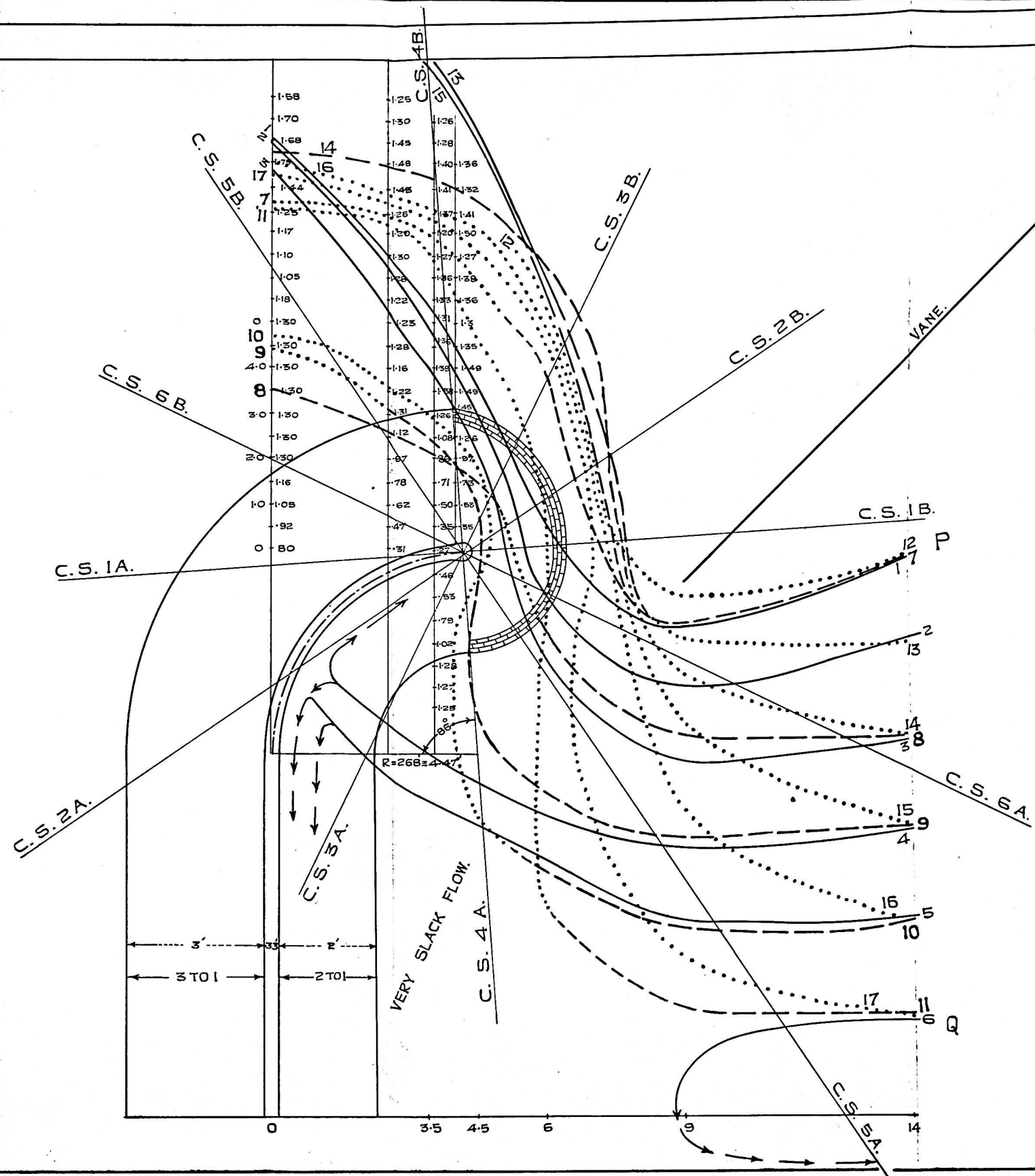
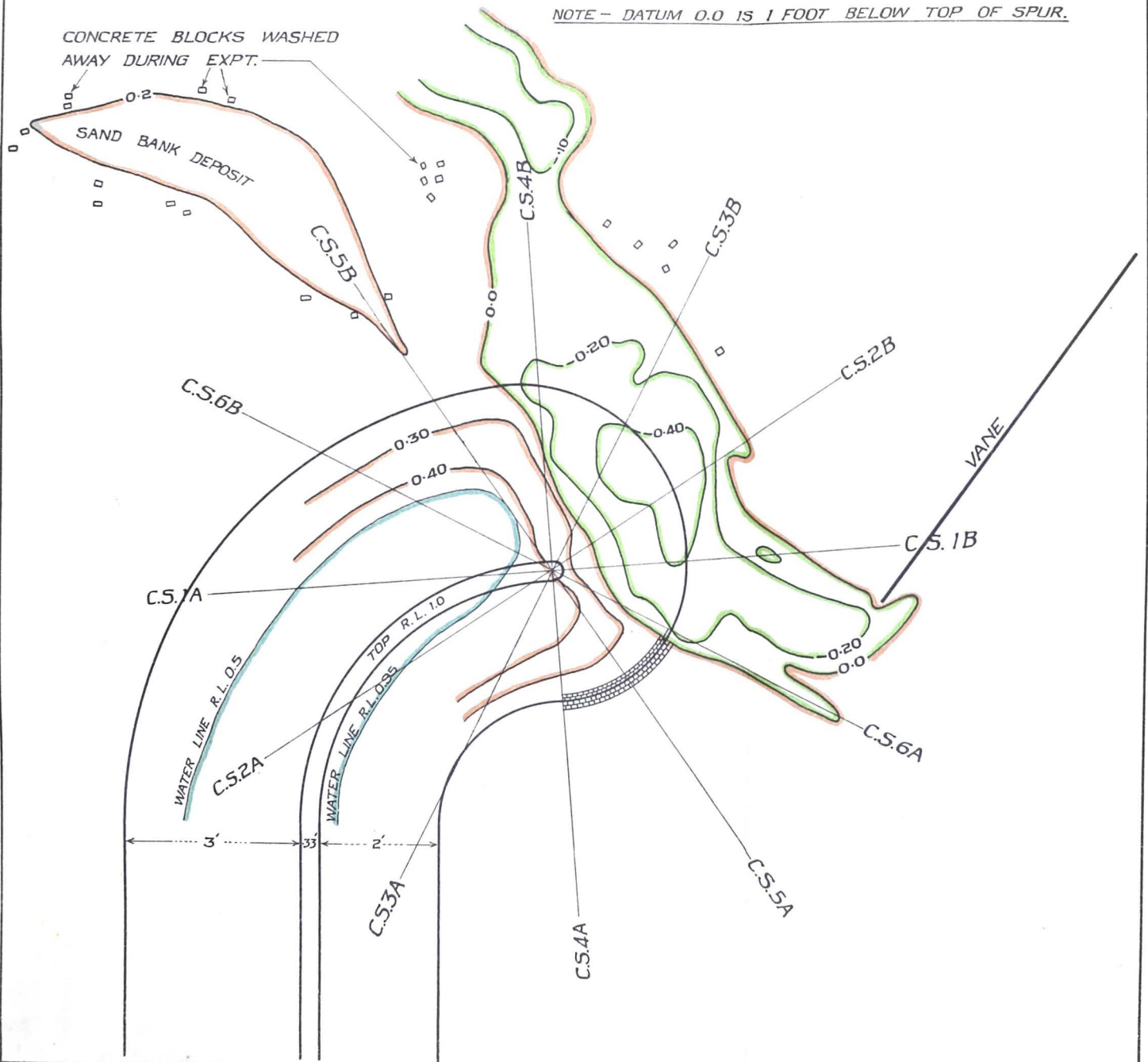


FIG. 101, LINES OF FLOW.
 WITH Q = 12 CUSECS.
 SURFACE LINE OF FLOW. ———
 MID-DEPTH. - - - - -
 BED. ·····
 BED WATER RISING. - · - · -
 MID-DEPTH WATER DIVING. - - - - -
 SCALE OF PLAN. 1" = 2 FEET.

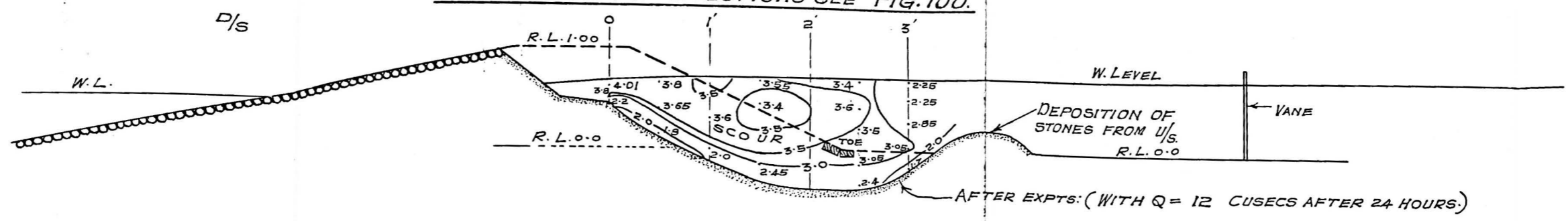
FIG. 102 - CONTOUR PLAN
AFTER EXPTS., SHOWING SCOUR
AFTER 24 HRS. FLOW WITH Q = 12 CFS.

SCALE 1 INCH = 2 FEET.

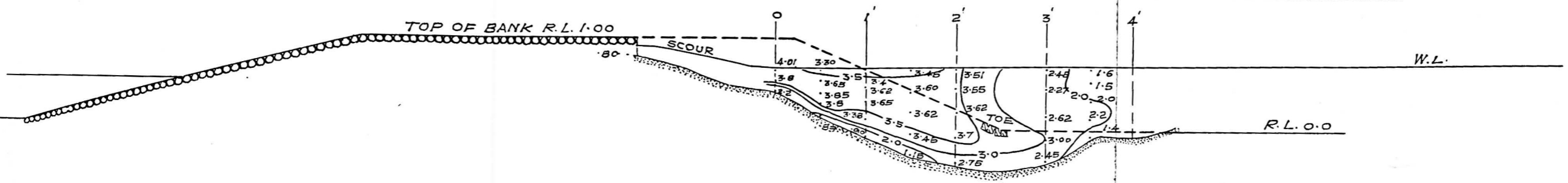
NOTE - DATUM 0.0 IS 1 FOOT BELOW TOP OF SPUR.



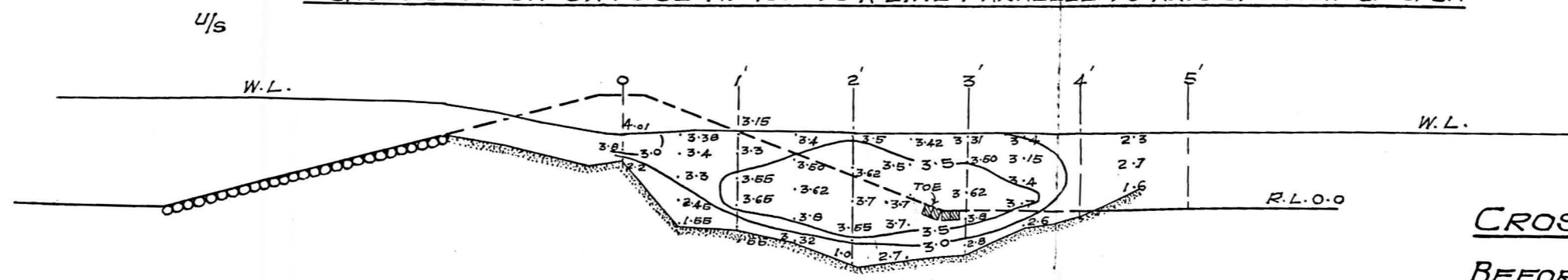
CROSS SECTION 1A TO 1B (90°) TANGENTIAL TO CENTRE LINE OF NOSE OF SPUR AND 95° TO AXIS OF STEM
OF SPUR
 FOR POSITION OF C. SECTIONS SEE FIG. 100.



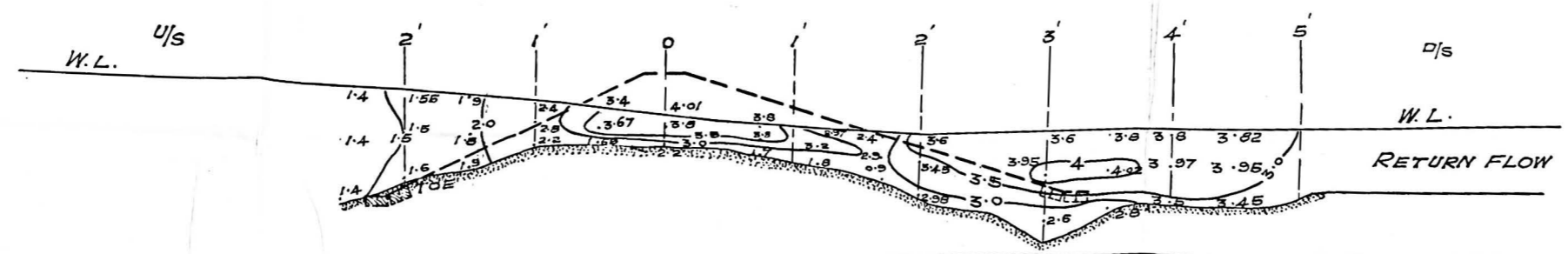
CROSS SECTION 2A TO 2B AT 125° TO A LINE PARALLEL TO AXIS OF STEM OF SPUR



CROSS SECTION 3A TO 3B AT 155° TO A LINE PARALLEL TO AXIS OF STEM OF SPUR



CROSS SECTION 4A TO 4B AT ANGLE OF 5° TO A LINE PARALLEL TO AXIS OF STEM OF SPUR -
ON LINE THROUGH CENTRE OF SEMI CIRCULAR NOSE OF SPUR



CROSS SECTIONS
BEFORE AND AFTER EXPERIMENT
WITH ISOTACHS AFTER EXPTS.
SCALE: - 1" = 1 INCH.

TABLE 17.—Showing various grades and percentage fractions of sand to the total sand in the silt abrader at different intervals of time (mechanical analyses by sedimentation and sieve method).

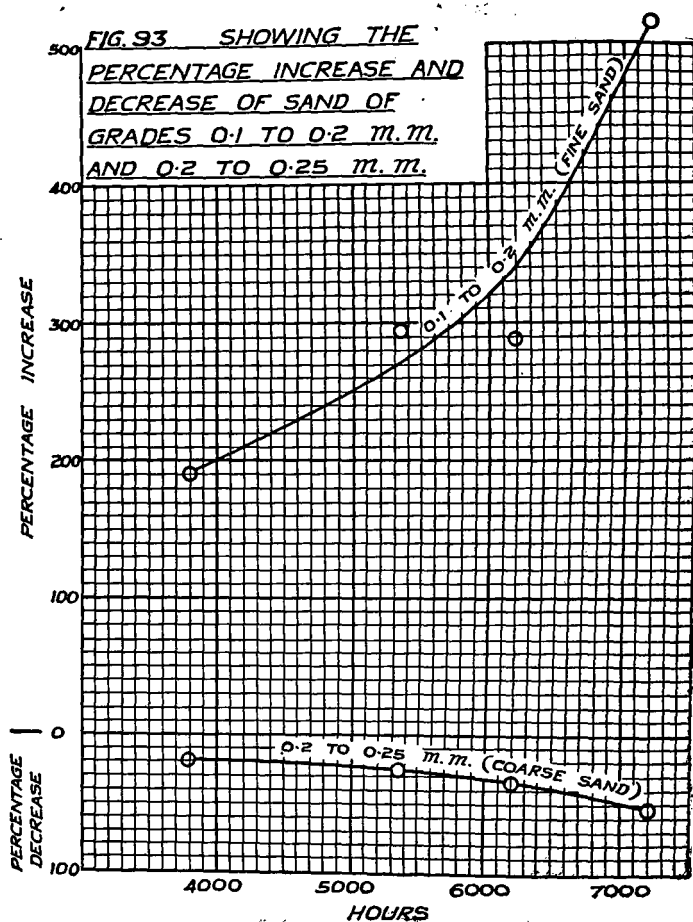
(1)	Diameter of particles in mm.	Original silt (average of 6 samples).	Sample after 5,382 hours (average of 6 samples).	Difference in columns 3 and 4.	Col. 5 Col. 3 × 100.	Sample after 6,220 hours (average of 6 samples).	Difference in columns 3 and 7.	Col. 8 Col. 3 × 100.	Sample after 7,150 hours (average of 6 samples).	Difference in columns 3 and 10.	Col. 11 Col. 3 × 100.
	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
(1) Coarse sand 1.0 to 0.2 mm.	1.0—0.5	0.18	0.16	—0.02	—11	0.14	—04	—22	.08	—0.10	—56
	0.5—0.33	4.19	1.30	—2.89	—69	0.94	—3.25	—78	0.78	—3.41	—81
	0.33—0.25	1.49	0.50	—0.99	—68	0.52	—0.97	—65	0.55	—0.94	—63
	0.25—0.20	84.04	62.17	—21.87	—26	54.14	—29.90	—36	39.81	—44.23	—53
(2) Fine sand 0.2 to 0.02 mm.	89.90	64.13	—25.77	—29	55.74	—34.17	—38	41.22	—48.68	—54	
	0.2—0.1	7.42	29.56	+22.14	+298	29.22	+21.80	+290	46.14	+38.72	+520
	0.1—0.075	0.38	2.59	+2.21	+582	7.55	+7.17	+1900	5.32	+4.94	+1300
	0.075—0.04	0.43	2.22	+1.79	+416	4.32	+3.89	+900	4.61	+4.18	+970
(3) Very fine sand 0.02 and less	0.04—0.02	0.48	0.41	—0.07	—15	1.08	+0.60	+125	1.42	+0.94	+296
	0.02 and less	8.71	34.78	+26.07	+300	42.17	+33.46	+380	57.49	+48.78	+727
	1.76	0.50	—1.26	1.72	0.19
TOTAL	..	100.37	99.41	99.63	99.91
(4) Mean dia. (in mm.) excluding particles <0.08 mm.	..	0.2289	0.2021	0.1932	0.1814
(5) f_s = Lacey silt factor of particles >0.075 mm. = $9.4/\sqrt{d}$, d being dia. in inches.	..	0.894	0.834	1.810	0.796

Figure 91 shows the size distribution curves of original silt, and silt obtained after 6,220 and 7,150 hours, based on analyses of particles <0.2 mm. by sedimentation and >0.2 mm. by sieves.

Figure 92 shows the size distribution curves of silt after 6,220 and 7,150 hours, based on analyses by Puri's siltometer.

[The original sample could not be analysed by Puri's siltometer, as this meter was evolved only lately. The future samples will be tested by the new method.]

Figure 93 shows the percentage of 0.1 to 0.2 mm. sand increased with time (shown as abscissa) and the percentage of 0.2 to 0.25 mm. sand decreased.



Consideration of results.

(1) The percentage fraction of coarse sand (0.2 to 1.0 mm.) obtained has decreased from 90 per cent. at the commencement of the experiment to 41 per cent. after 7,150 hours and the percentage of fine sand (0.2 to 0.02 mm.) has increased from 9 per cent. to 57 per cent.

(2) The rate of per centage decrease of coarse sand per hour (>0.2 mm.) and of per centage increase of fine sand per hour (<0.2 mm.) increases as the material

FIG: 91.

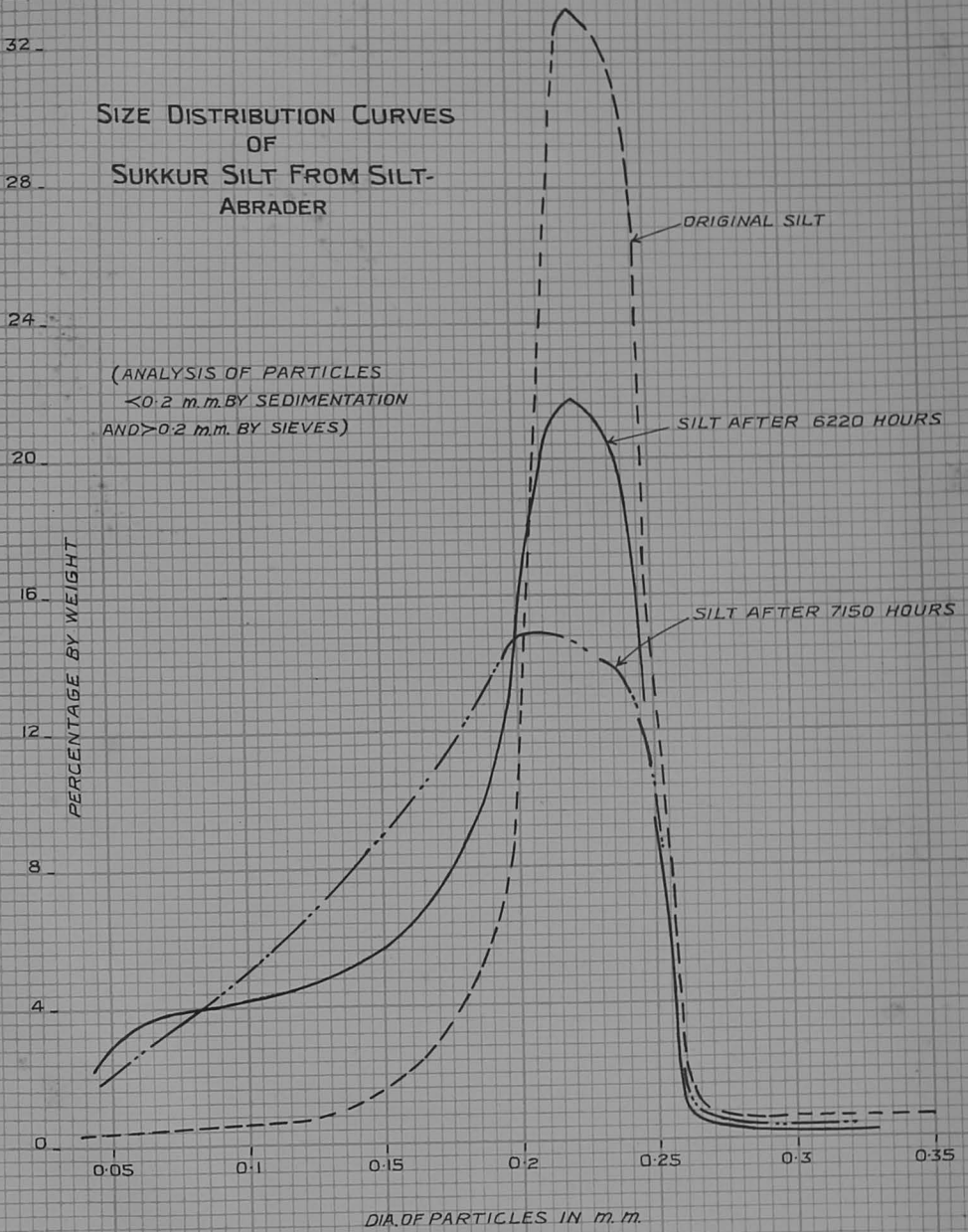
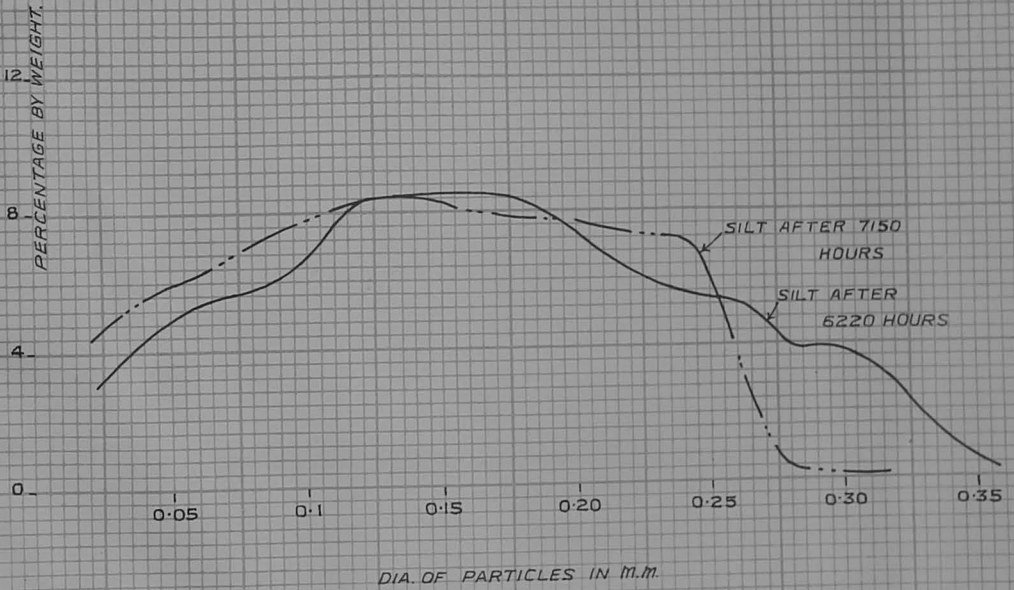


FIG: 92.

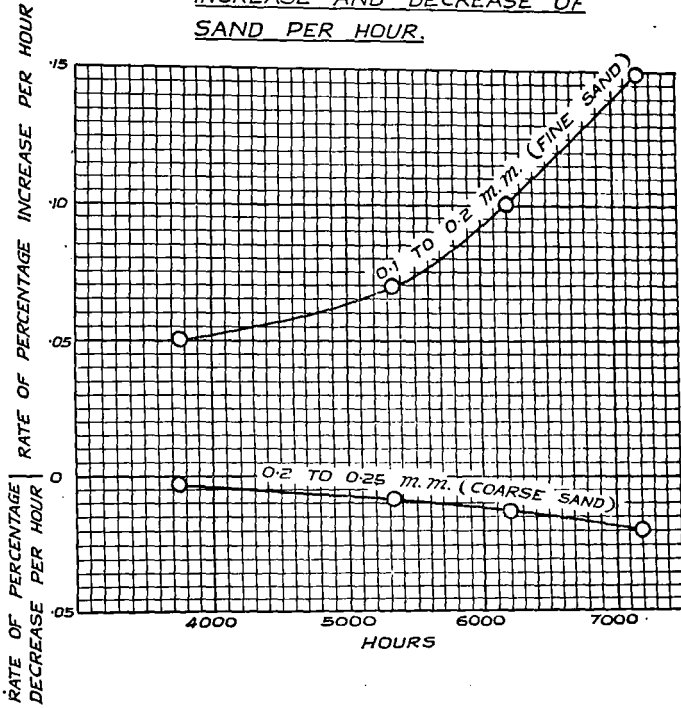
SIZE DISTRIBUTION CURVES
OF
SUKKUR SILT FROM SILT
ABRADER

ANALYSIS DONE BY PURI'S SILTOMETER.



becomes finer, until the silt reaches a certain fineness below which abrasion decreases rapidly due to the sand then being in suspension—see figure 94.

FIG. 94—SHOWING THE RATE OF PERCENTAGE INCREASE AND DECREASE OF SAND PER HOUR.



[Analysis of sand in suspension in the abrader after it worked for 7,150 hours gave the following results. The sample was taken at mid-depth at a point $\frac{1}{3}$ rd radius from the centre at the moment the abrader stopped working.

Diameter in mm.	% fraction to total by weight.
0.1 to 0.2	15.48
0.07 to 0.1	12.56
0.04 to 0.07	14.64
0.02 to 0.04	21.74
below 0.02	35.58
	100.0

Thus the maximum diameter of the silt in suspension was 0.2 mm.]

(3) The Lacey silt factor ($f_d = 9.4\sqrt{d}$) based on particles greater than 0.075 mm. in diameter changed from—
0.894 (original sand) to

- 0.871 after 3,775 hours (or 157 days),
- 0.834 after 5,382 hours (or 224 days),
- 0.819 after 6,220 hours (or 259 days), and
- 0.796 after 7,150 hours (or 298 days).

It is clear that abrasion in the boulder and pebble reaches of rivers occurs mainly while the stones are at rest—due to finer material passing over the stones. This is also true to a large extent in the case of coarse silt. With finer silt, abrasion takes place as the sand waves move along the bed forming ripples, which progress downstream.

18. DIVERGENCE EXPERIMENTS.

It is frequently desirable and economical to construct masonry works which are narrower than the downstream channels: This necessitates the construction of expansions, which, in the past, have been a frequent cause of trouble due to flow not 'fanning out' to fill the expansion, with the result that a jet of high velocity has flowed towards one bank with return flow along the other. Very little appears to be known about the theory underlying the flow in a divergence beyond Prandtl's proposition that separation at the boundary, followed by return flow, occurs when the layer of fluid adjacent to the boundary loses its kinetic energy and comes to rest, owing to the rate of energy dissipation at the boundary exceeding the rate of replenishment from the interior of the fluid.

The factors affecting this question are:—

- (i) the degree of divergence,
- (ii) velocity,
- (iii) width,
- (iv) $\frac{\text{width}}{\text{depth}}$ ratio, and
- (v) degree of turbulence.

Little experimental work appears to have been done on this important problem.

Experiments.

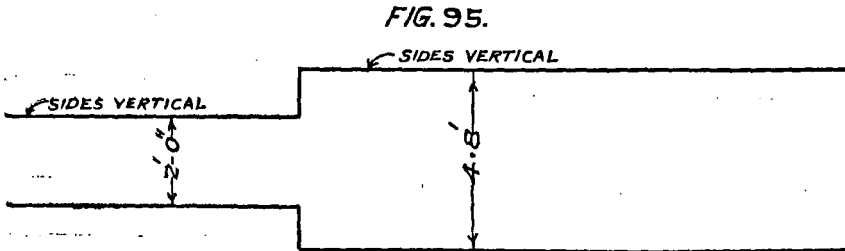
Experiments were therefore commenced to enable the distribution of flow in a divergence to be examined; the loss of energy to be measured; and, in the light of this information, to test the effect of various practical devices intended to make the flow 'fan out' satisfactorily.

Series I.

In the first series, various discharges were passed from a channel 2 ft. wide to one 4.8 ft. wide through an expansion, the sides of which diverged at from 1 in 3 to 1 in 10 in different experiments, the depth of flow also being varied. The intention was to correlate the loss of energy due to expansion with other factors, but the loss of energy was so small and the experimental error in measuring energy was relatively so large—owing to the distribution of velocities downstream being very uneven—that useful results were not obtained.

Series II.—Sudden divergence.

In this series the enlargement from 2 ft. channel to 4.8 ft. channel was sudden, see figure 95. A constant discharge of 8 cusecs was run and the loss of energy was observed for various depths of flow.

**SUDDEN DIVERGENCE.**

If there was no conversion from velocity energy to pressure energy the loss would be $\frac{V_0^2 - V_3^2}{2g}$, or taking account of the velocity distribution say, $\frac{V_0^2 - V_3^2}{50}$

where V_0 = mean velocity in 2 ft. channel

V_3 = mean velocity in 4.8 ft. channel.

From Newton's momentum law, however, as applied to pipes,⁽⁷⁾ the loss would be $\left(\frac{V_0 - V_3}{50}\right)^2$, while another formula⁽¹¹⁾ given is $V_3 \left(\frac{V_0 - V_3}{g}\right)$.

(7) Hydraulics and its applications by Gibson, 3rd edition, 1925, page 89.

(11) Hydraulics with working tables by E. S. Bellasis, 4th edition, 1924, page 32.

The losses observed in these experiments, after deducting the approximate loss due to ordinary frictional resistance by Manning's formula

$$[\text{viz., } h_f = \frac{L V^2}{C^2 R^{1.49}}$$

where h_f =loss of head due to friction,

L =length,

V =mean velocity,

$$C = \frac{1.486}{N} \text{ and } N = 0.012,$$

R =hydraulic mean depth]

are plotted in figure 96.

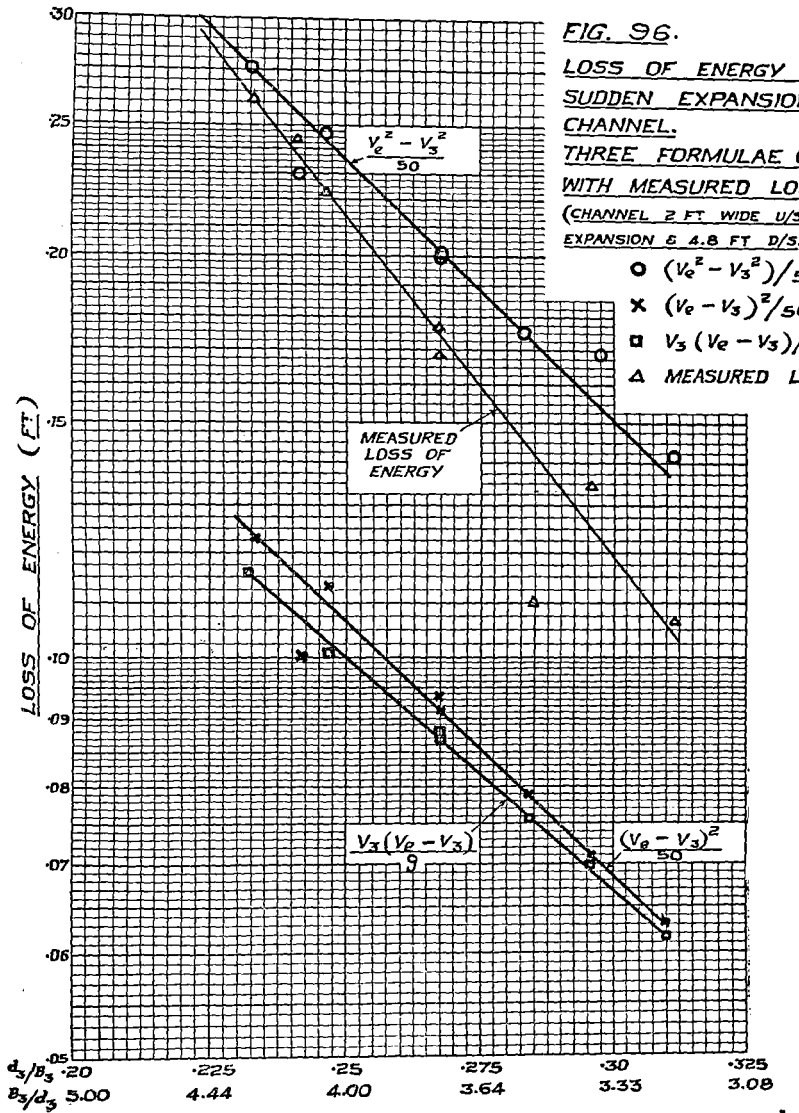


FIG. 96.
LOSS OF ENERGY AT
SUDDEN EXPANSION IN
CHANNEL.
THREE FORMULAE COMPARED
WITH MEASURED LOSSES.
(CHANNEL 2 FT. WIDE U/S. OF
EXPANSION & 4.8 FT. D/S.)

- $(V_e^2 - V_3^2)/50$.
- × $(V_e - V_3)^2/50$.
- $V_3(V_e - V_3)/g$.
- △ MEASURED LOSS.

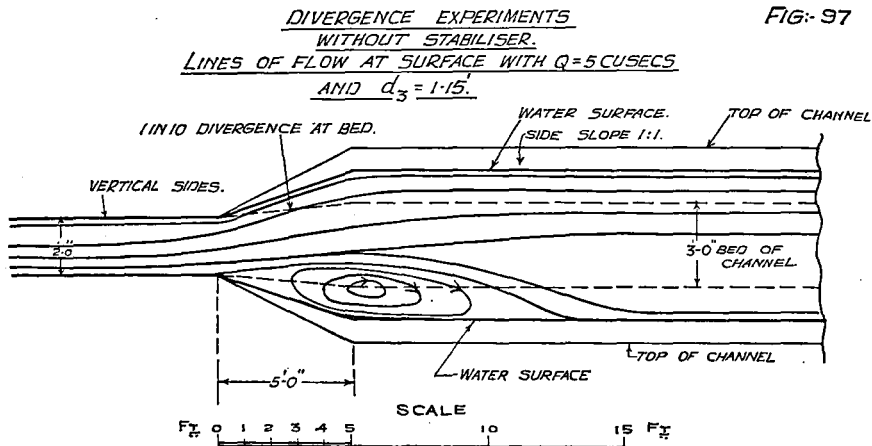
It will be seen that $\frac{V_0^2 - V_3^2}{50}$ gave results which were nearly correct where the bed width was more than 4 times the depth and though other factors enter, it appears that in normal channels this formula gives approximately correct results.

There was in this case, apparently, little recovery of velocity head at the expansion.

It is intended to do further experiments with other expansion ratios.

Series III.—Tests of practical devices.

The channel used was one of transition from a section 2 ft. wide with vertical sides to one having a bed 3 ft. wide with 1 : 1 sloping sides, as shown in figure 97.



The side divergence at the bed was 1 in 10 and at the surface considerably sharper and the bed was level. This is the type of divergence which is frequently adopted at the downstream ends of aqueducts and falls in the Deccan.

A constant discharge of 5 cusecs was run and a downstream depth of about 1 ft. was maintained. Where there was no stabiliser, the flow followed one side only, a large eddy forming on the other side, as shown in figure 97 (The lines of flow shown are for surface only whereas the flow was 3 dimensional).

Various devices were then tested, these being of two main types (a) "baffle type stabilisers" which force flow from the bed upwards thus making the water spread out to follow the expansion, see figure 98.

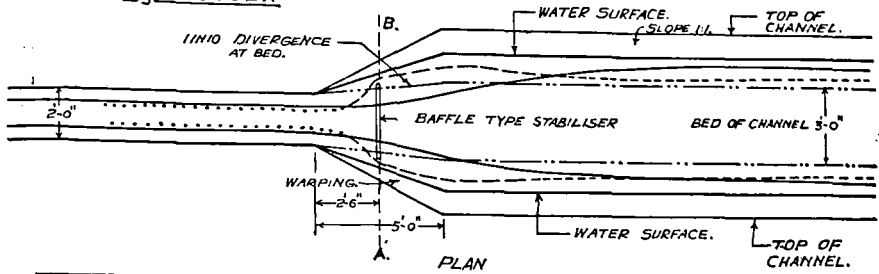
The other type (b) consists of stabilisers designed to create turbulence so facilitating the transference of energy and equalising the velocity across the channel, see figure 99.

The most satisfactory design of type (a) consisted of a 0.3125 ft. high 'baffle-stabiliser' 2½ ft. downstream in the divergence, *i.e.*, midway along the divergence. The ends of the stabiliser at the sides of the channel sloped up at 1 in 5 to a maximum of 2", as shown in figure 98.

In type (b) a grid of 3 verticals; the central one ½ inch thick and the other two 1½ inches thick, placed 6½ inches on either side of the central one, was found to be most satisfactory. This grid was placed 1½ ft. downstream along the divergence, as shown in figure 99.

Under laboratory conditions the latter was very satisfactory, but would be liable—like all grids—to catch brushwood, and would then be liable to cause considerable damage. The former has given satisfactory results in Sind and will be successful where the silt charge is normal; but it remains to be determined whether a supercharge of silt may not render it less effective. If this is found to be the case, modification may be necessary.

FIG-98 DIVERGENCE EXPERIMENTS. FIGURE SHOWING LINES OF FLOW WITH BAFFLE TYPE STABILISER. $Q=5$ CUSECS. HEIGHT OF STABILISER 0.3125 FT. $d_3 = 1.33$ FEET.



C.S.A.B THROUGH STABILISER.

- REFERENCES.
- WATER AT SURFACE. _____
 - WATER MID-DEPTH. - - - - -
 - $\frac{3}{4}$ DEPTH. - - - - -
 - BED. ······
 - WATER RISING. ······
 - WATER DIVING. ······

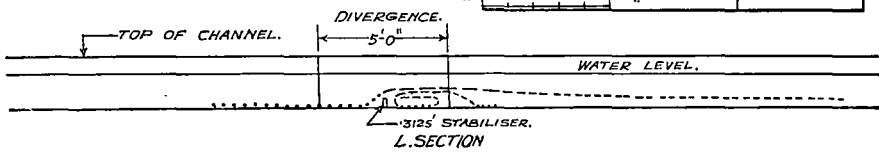
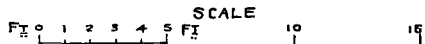
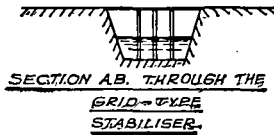
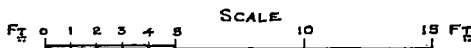
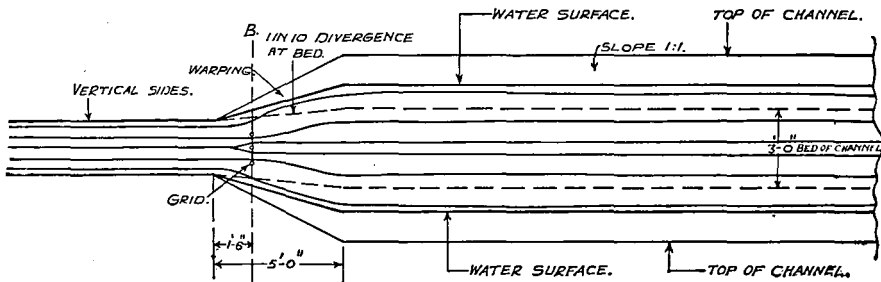


FIG:99 DIVERGENCE EXPERIMENTS. FIGURE SHOWING LINES OF FLOW AT SURFACE WITH A GRID TYPE STABILISER $1'-6''$ $d_3 = 0.9$ FT. $Q = 5$ CUSECS.



SECTION A.B. THROUGH THE GRID TYPE STABILISER.

19. EXPERIMENTS WITH GEOMETRICALLY SIMILAR MODELS OF VARIOUS DESIGNS OF SPURS.

Object.—This series of experiments has been planned to test the relative merits of various types of spurs by means of geometrically similar models.

Model.—In this experiment, a model of a Burma type spur was tested. This spur has a nose shaped like a hockey stick, pointing upstream (see figure 100). In shape, the model spur was a $\frac{1}{60}$ th scale replica of the Burma spur constructed half a mile upstream of the Sarda Barrage at Banbassa. The spur was composed of 1" diameter stones and the bed of the channel was laid with stones of $\frac{1}{2}$ " to 1" diameter. The spur was 1 ft. high, having 2 to 1 and 3 to 1 slopes upstream and downstream respectively. The radius of the curved head was 4.8 ft., equivalent to 268 ft. and the curved length was 6.75 ft. (≈ 405 ft.). Concrete blocks $2'' \times 1.2'' \times 1.2''$ ($\approx 10' \times 6' \times 6'$) were laid round the toe of the nose—see figure 100.

Experiments.

In practice, spurs are only attacked when they project into, and sharply divert, the main flow of a river.

In order to reproduce the resultant severe attack, conditions had to be imposed in the model by means of a vane which directed flow towards the nose of the spur—see figure 101. This caused curved, diving flow.

The water level downstream of the spur was kept 6" below the top of spur, and the discharge was gradually increased to 12 cusecs. Movement first took place in the 1" stones above and below the concrete blocks around the nose of the spur at C. S. 2-B; after which the blocks, thus exposed, were washed downstream. Scour then spread, and eventually extended from C. S. 6A (upstream) to C. S. 4B (downstream) (see figure 102).

Figure 101 shows lines of flow, and figure 103 isotachs. Surface water flowed with high velocity, across the nose of the spur; while the bed water twisted sharply round the vane, well away from the nose, with mid-depth water diving towards the bed, across the toe of the spur at its nose. Scour took place where the direction and velocity of flow between mid-depth and bed water varied abruptly from point to point with periodic fluctuations due to instability of flows.

Concrete blocks between cross sections 4A and 6A were undisturbed; but the remaining blocks, between 6A and 4B, were washed away and deposited downstream. Reynold's number $\frac{VR}{\nu}$ at cross sections 1B, 2B and 3B was about 200,000.

The experiment was repeated after numbering each block and painting stones in the various zones in different colours, the object being to locate where stones and blocks from different parts of the spur came to rest (figure 104).

It will be noted that where the mid-depth water dived and turned sharply in plan, near the nose, blocks and stones were carried a longer distance towards the left bank than where the stones were carried by the bed water flowing across the nose.

Conclusion.

The main object of this investigation is to compare various designs of spurs in other words to obtain qualitative results.

The Burma type of spur, dealt with here, has the marked advantage that the attack, though severe, is confined to the nose; and, provided the nose can be made strong enough to withstand attack, the spur will remain intact.

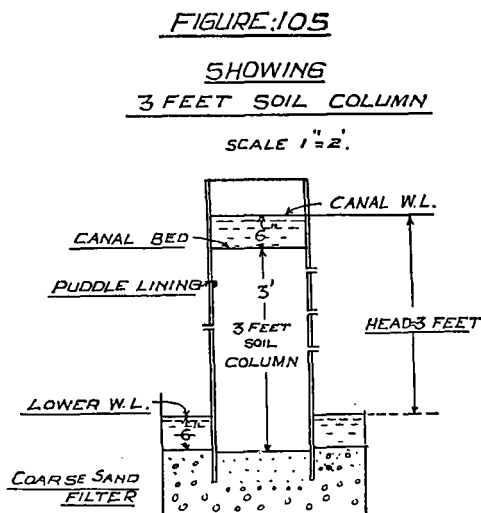
20. EXPERIMENTS IN CONNECTION WITH SEEPAGE FROM CANALS.

Mr. Crump, of the Punjab, concluded, on theoretical grounds, that losses from Canals vary according to the level of the subsoil water table, increasing as the water table falls below the water level in the canal until a maximum is reached

when the subsoil water table is a little below the canal bed level; below which, the loss rapidly decreases as saturated flow changes to unsaturated flow, after which resistance to flow is balanced by gravity and a further drop in subsoil water level has no effect.

At the suggestion of Mr. Crump a series of experiments was started to verify the accuracy of this theory, by measuring, under controlled laboratory conditions, the seepage through columns of silt when water was maintained on top of the column at a uniform depth, representing a canal: Each column was also kept submerged in water at the bottom, to represent the water table, which could be varied in different experiments. Experiments with the same object, but on a field scale, have been carried out in the Punjab, the results of which have been described elsewhere.⁽¹¹⁾

In the experiments at Poona, silt was packed uniformly in steel cylinders $1\frac{1}{2}$ ft. in diameter, and of various lengths, fixed vertically as shown in figure 105. The water in the "canal" at the top was kept constant by an automatic feed and that in the chamber at the bottom by an overflow pipe. After passing through this,



the seepage water was measured. The silt used in the earlier experiments was Khadakvasla red silt, a slightly cohesive silt having an average particle diameter of 0.31 mm.; but experiments are also in progress using silt from the river Ganges, which is a typical incoherent sand of 0.27 mm. mean diameter.

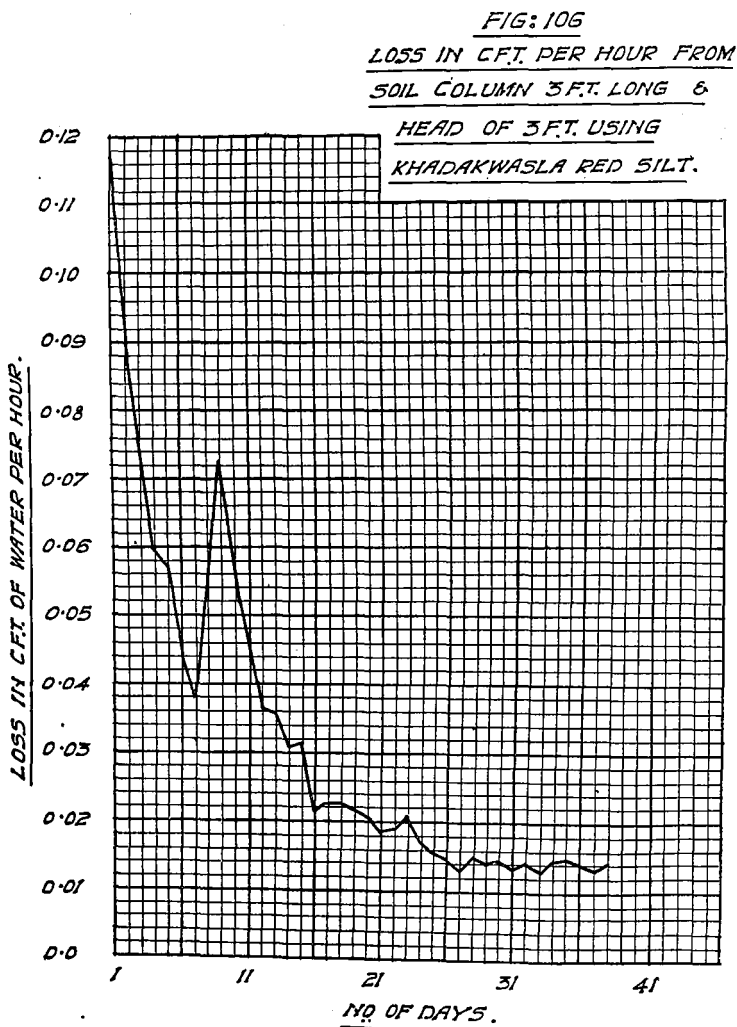
These experiments do not exactly reproduce the conditions of seepage from canals, where the stream lines may be curved, the flow having a horizontal component; but are intended to provide data on the basic laws of downward flow above a water table.

In each experiment the depth of water table was kept constant and the seepage was allowed to continue until stability had been reached. The rate of seepage was initially several times as great as the final stable rate, the decrease at first being very rapid; and a considerable period, months in some cases, was

(11) Gopal and Sharma: Punjab Engineering Congress, Paper No. 209, 1938.

required before stability was reached. Figure 106 shows a typical curve of seepage: time. This reduction in seepage rate is thought to be due to the gradual rearrangement of the air contained in the silt, a view supported by the fact that if, when stability was reached, a layer from the bed of the canal was removed or the bed slightly disturbed with the fingers, bubbles of air arose from the bed and the rate of seepage jumped considerably, subsequently continuing and stabilising at a higher figure.

NOTE.—Air was contained initially in the silt which was filled into the cylinders in a dry state; but a certain amount of the air was presumably expelled through the pipes fixed at 1 ft. intervals in the sides of the cylinders when the water from the "canal" permeated the silt column, yet it appears that even then a considerable amount of air was trapped; and in the shorter cylinders the whole of the air was trapped, because the side tubes had to be kept closed to prevent egress of water.



The seepage rate was very sensitive to disturbance: for example, on one occasion seepage jumped to a very high figure due to the disturbance caused by a rat which had fallen into the lower chamber.

The final, stable, results, corrected for temperature (viscosity) to 25°C, and for evaporation, are shown in figure 107. It will be seen that where the water table was more than 3.0 ft. below the canal bed the rate of seepage was practically constant, because the resistance to flow through the unsaturated soil and the pull of gravity had attained a balance.

When, however, the water table lay within 2.5 ft. of the canal bed, continuous streams of water existed in the silt and the rate of seepage was dependent on the head of water. This rate was therefore a maximum when the water table was just above the position at which the condition of flow changed.

In cases where the height of silt column was constant, the rate of flow decreased as the water table rose. These results corroborated those obtained in the Punjab by Messrs. Gopal and Sharma.

Considering the results of columns of various heights with various heads, within the saturated range, the following are the values obtained for the transmission constant λ in the equation

$$Q = \frac{\lambda A g \rho h}{\mu l}$$

where Q = rate of seepage.

A = cross sectional area of column.

g = gravity acceleration.

ρ = density of water.

μ = viscosity of water.

h = difference in head.

l = height of soil column.

Height of column.	Head ft.	Seepage rate C.ft. per hour.	$\lambda \times 10^{11}$
1.0	1.0	.0171	.90
1.5	1.5	.0357	1.87
1.5	1.0	.0285	2.24
2.0	2.0	.0310	1.62
3.0	2.0	.0251	1.97
3.0	2.5	.0297	1.86
3.0	3.0	.0364	1.90

No explanation has yet been found for the low value of λ in the first case.

The differences between the others may be due to differences in packing and air content.

Pressures were observed by inserting manometer tubes into the centre of the soil columns through apertures in the walls of the pipes. In the case of unsaturated flow the inside pressure was shown to be atmospheric. Water in a horizontal tube 0.18" in diameter inserted into the soil 1 ft. below the top of the column was sucked in at a constant rate of 0.28" per day, due to capillary action.

In the case of a 3 ft. soil column with saturated flow, instead of a proportionate loss of pressure head from canal to lower chamber, 4 tubes at different levels all registered the canal water level, showing direct communication of pressure from the canal—due presumably to the arrangement of the air contained in the soil allowing pressure to be transmitted.

On the other hand a similar experiment, carried out in a 2" diameter glass tube packed with Sukkur sand, mean diameter 0.22 mm., in which the silt was carefully and completely saturated, which showed proportionate loss of head *i.e.*,

$$\frac{\text{loss of head}}{h} = \frac{\text{depth from canal bed}}{l}$$

A 6" column in a 1½ ft. cylinder, similar to the original ones except that it contained Ganges sand, has recently been prepared by laying the silt *in water to*

ensure that no free air can be entrapped. In this case, proportionate loss of head was observed initially, but this was followed by a gradual tendency to equalisation of pressures, which this time were not equal to that of the canal above but to the lower water level, accompanied by a drop in the rate of seepage. This appears to have been due to the release of air from solution in the water. In this experiment the silt was incoherent and apparently free from colloidal matter.

Some subsidiary experiments were done in the same connection: One series was carried out using a glass container, 14 cm. in diameter, as shown in figure 108 filled with Sukkur sand (mean diameter of particle 0.22 mm.), and provided with manometer tubes.

FIG: 108.

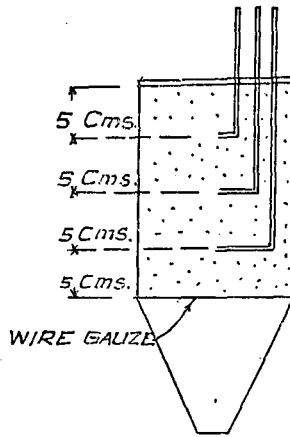


Figure 109 shows graphically the pressures observed for various rates of seepage, when the apparent water table was kept just visible at the top of the column. The real water table was of course lower by an amount equal to the head required to produce the flow. The points lie fairly well on straight lines; which, however, seem to meet the zero discharge ordinate at a negative pressure loss, showing that there was a positive pressure at the top surface of about 2 cms.

There is considerably more loss of pressure between the surface and the top tube than between the other tubes, the losses being in the ratios 16 : 7 : 6 approximately. This feature has been noticed in other cases, and is probably due to the collection of a greater proportion of air near the top than below.

The transmission constant λ computed from the lowest tube readings is about 11×10^{-8} , compared with 8.3×10^{-8} obtained in the experiment with the 2" tube previously mentioned, using the same sand.

Another series of experiments was carried out with a model of a canal in section in a glass-sided tank, 45" long \times 18" high, filled with Sukkur sand and provided with numerous pressure tubes. The position and slope of the water table was controlled by the water levels in two tanks, one at each end, and the object of the experiments was to observe the shape of water table under various conditions with seepage from a channel.

Owing to the small size of the tank the whole of the sand was saturated, so that only qualitative results with saturated flow were obtainable, but the shape of the water table is indicated in figure 110 which shows the results obtained with one set of conditions.

FIG. 107

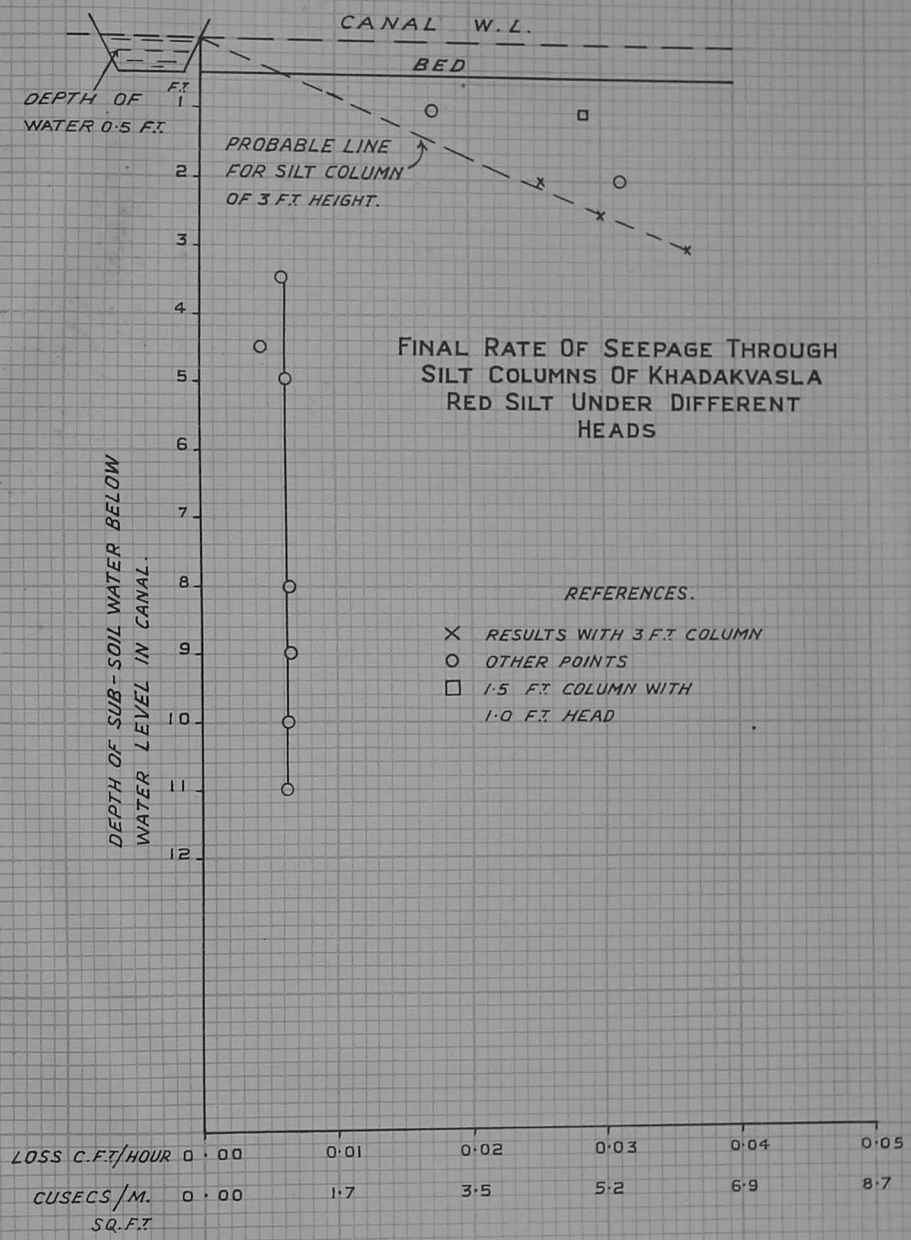


FIG. 109.

SHOWING LOSS IN HEAD FOR DIFFERENT DISCHARGES. WATER TABLE AT THE SURFACE OF SILT.

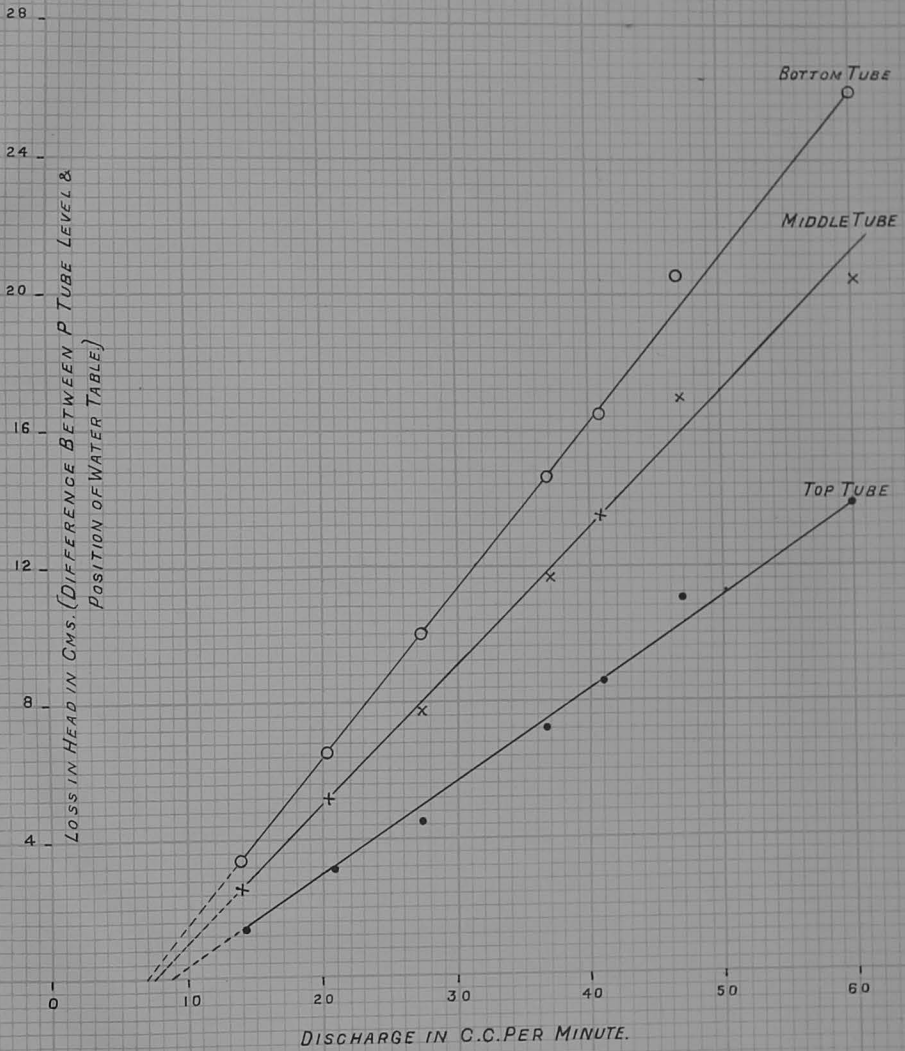


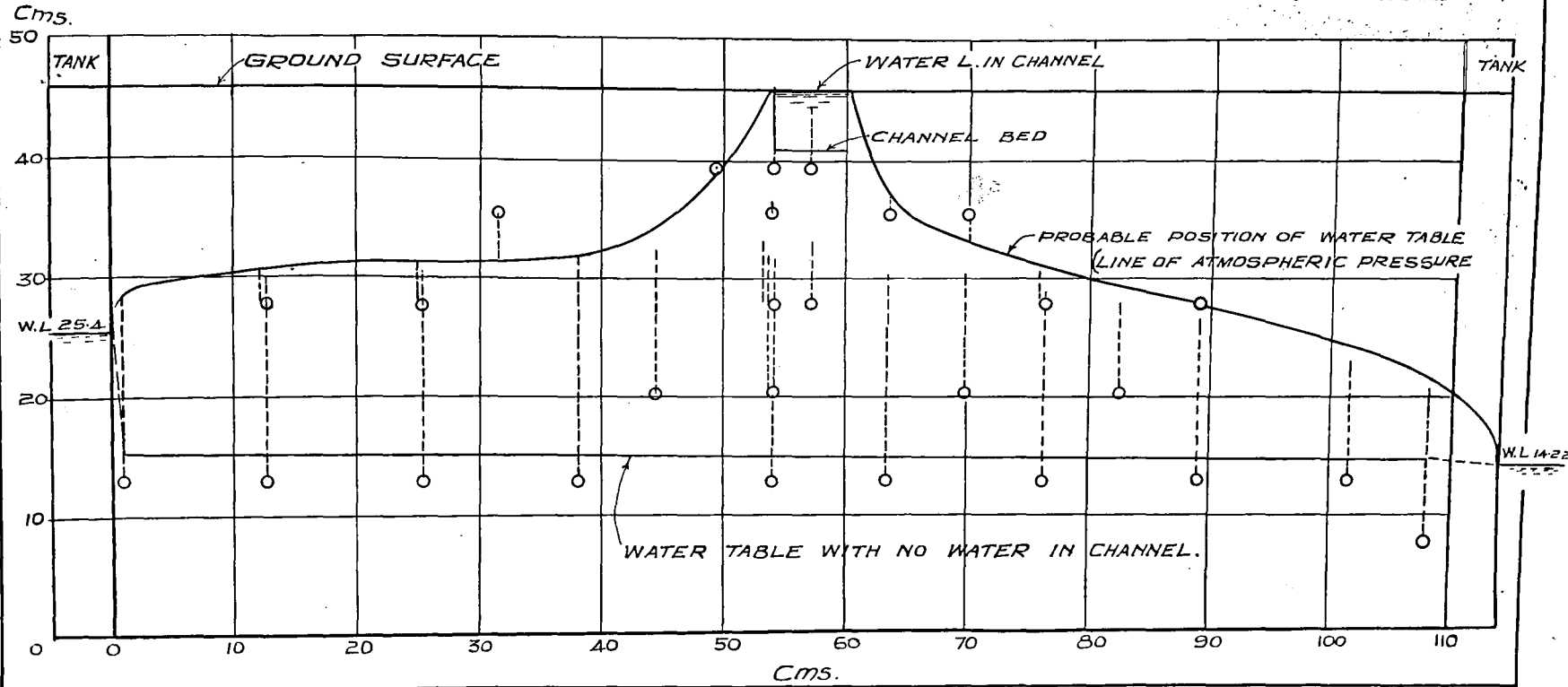
FIG. 110.

No 964
C.I. & H.R. STN.

CIRCLES O REPRESENT PRESSURE TUBES
AND LINES FROM THEM PRESSURE RECORDED
IN Cms. OF WATER TO SCALE (1"=10 Cms)

MODEL SHOWING SEEPAGE
FROM CHANNEL

SCALE $\left. \begin{matrix} \text{HORI} \\ \text{VERT} \end{matrix} \right\} 1" = 10 \text{ Cms.}$



The subsoil water level in the silt was about 10 cms. lower than the water level in the left chamber when there was no percolation from the channel. This 10 cms. drop was independent of the rate of flow and has been observed in other experiments.

These experiments are being continued.

21. THE TRAINING OF THE RIVER JUMNA AT NEW DELHI.

Introduction.

Prior to 1913, there was only one pumping station for Delhi—on the right bank of the River, at Chandrawal, two miles upstream of the Railway Bridge—. In 1913, a new Pumping station was constructed at Wazirabad $3\frac{3}{4}$ miles upstream of the Bridge. In 1922, the Delhi Gate Pumping Station was built 2 miles downstream of the Railway Bridge on the right bank. The River was then flowing along the right bank near the Pumping Station. Figure 111 shows a plan of the River from the Railway Bridge to Okhla Weir in 1924; figure 112 shows conditions at the end of 1936 (from 4 miles upstream of the Bridge to Okhla Weir) and figure 113 conditions at end of 1937 below the Bridge.

Spurs 1, 2 and 3 were 3 of a series constructed in 1870 along the left flank embankment down to Okhla Weir. Spur No. 5 was constructed before the floods of 1935 and spurs 4 and 6 were extended after the 1935 floods—with the intention of pushing the River towards the Pumping Station on the right bank.

The reason why the River has been progressively moving across towards the left bank from the Pumping Station is because as a result of the right bank upstream of the Railway Bridge having been pitched with stone in 1915-16, the River now hugs that bank and is held against it, so that the water now flows through the Bridge at an acute angle from the right bank towards the left bank, and the distance between the Bridge and the Pumping Station is insufficient to allow the River to complete a whole meander under natural conditions. This could be changed by building a 'Bell bund' guide bank projecting upstream from the Delhi end of the Railway Bridge; but this would change flow condition so radically that there would be a danger of the River shifting its course abruptly within a few years; which would be liable to cause serious trouble; because though changes in a River course due to natural causes are accepted in a philosophical spirit, any change brought about by Government would be resented by all those who suffered thereby.

The first alternative considered was :—

To remove the greater part of spurs 4 and 6, which have prevented embayment above spur 5; which, in turn, would tend to throw the River across towards the Pumping Station; but, it is anticipated that the translation due to spur 5 as at present constructed would be insufficient and that in order to bring deep water near the Pumping Station, the spur would have to be extended.

Description of model.

The flood discharge in most years is less than 100,000 cusecs. Scale ratios of the model—worked out from the Lacey formula—were as follows :—

Latitudinal & longitudinal scale ratio	1/200
Depth scale ratio	1/27
Vertical exaggeration	7.41
Time scale	1/40

Max. Q in model=3.6 cusecs (\equiv 100,000 cusecs). The length of the model was equivalent to 1.4 miles upstream of the Railway Bridge and 2.7 miles downstream. The right bank upstream of the Railway Bridge was built in concrete—to represent pitched banks and pukka 'ghats' in the prototype.

Experiments.

Two series of experiments were done with this model: Series I with 1937 conditions and Series II after removing spurs 4 and 6. At the beginning of each series, the bed was laid according to the survey done after 1936 floods; and experiments reproduced a rising and falling flood as shown below:—

Series I (1937 conditions). Experiment 1.

(≡ 3 flood seasons).

1st year—

- (i) Discharge rising from 10,000 to 50,000 cs. in 1 week.
- (ii) Discharge 50,000 cusecs for 1 day.
- (iii) Discharge falling from 50,000 to 10,000 cusecs in 1 week.

2nd year—

- (iv) Q rising from 10,000 to 100,000 cs. in 1 week.
- (v) Q 100,000 cusecs for 1 day.
- (vi) Q falling from 100,000 to 10,000 cs. in one week.

3rd year—

- (vii) Q rising from 10,000 to 50,000 cs. in 1 week.
- (viii) Q 50,000 cusecs for 1 day.
- (ix) Q falling from 50,000 to 10,000 cs. in 1 week.

As Series I showed that the major changes took place in the 2nd year; in Series II, only the 2nd was run.

Series II. (Spurs 4 and 6 removed).

- (i) Q rising from 10,000 to 100,000 cs. in 1 week.
- (ii) Q 100,000 cusecs for one day.
- (iii) Q falling from 100,000 to 10,000 cs. in 1 week.

Figure 114 shows the lines of flow at surface in Series I with $Q \equiv 100,000$ cusecs and figure 115 shows the lines of flow in Series II with $Q \equiv 100,000$ cusecs.

In Series I—1937 conditions—the main flow was along the left bank between the Railway Bridge and the nose of spur No. 4; where flow concentrated and action was severe. Downstream of this spur, the flow spread out, brushing past the noses of spurs 5 and 6 and flow was far away from the Delhi Gate Pumping Station.

In Series II—with spurs 4 and 6 removed—the throw off caused by spur No. 5 was markedly increased but flow was not deflected towards the Pumping Station to any marked extent downstream of the spur, but instead spread out into a wide channel with a flat bed.

This result showed that merely removing spurs 4 and 6 would not induce the main stream to flow close to the Pumping Station and that spur No. 5 would have to be increased in length.

At this stage it was decided to extend the model downstream by the equivalent of 3.3 miles; because it was realised that a better solution would probably result if, in addition to the extension of spur 5 on the left bank, some short spurs were constructed on the right bank below the pumping station—to “pull” the river across to that bank. These later experiments are yielding satisfactory results. Work is still in progress.

22. EXPERIMENTS WITH A LARGE SCALE MODEL OF THE GANGES ABOVE THE HARDINGE BRIDGE (EASTERN BENGAL RAILWAY).

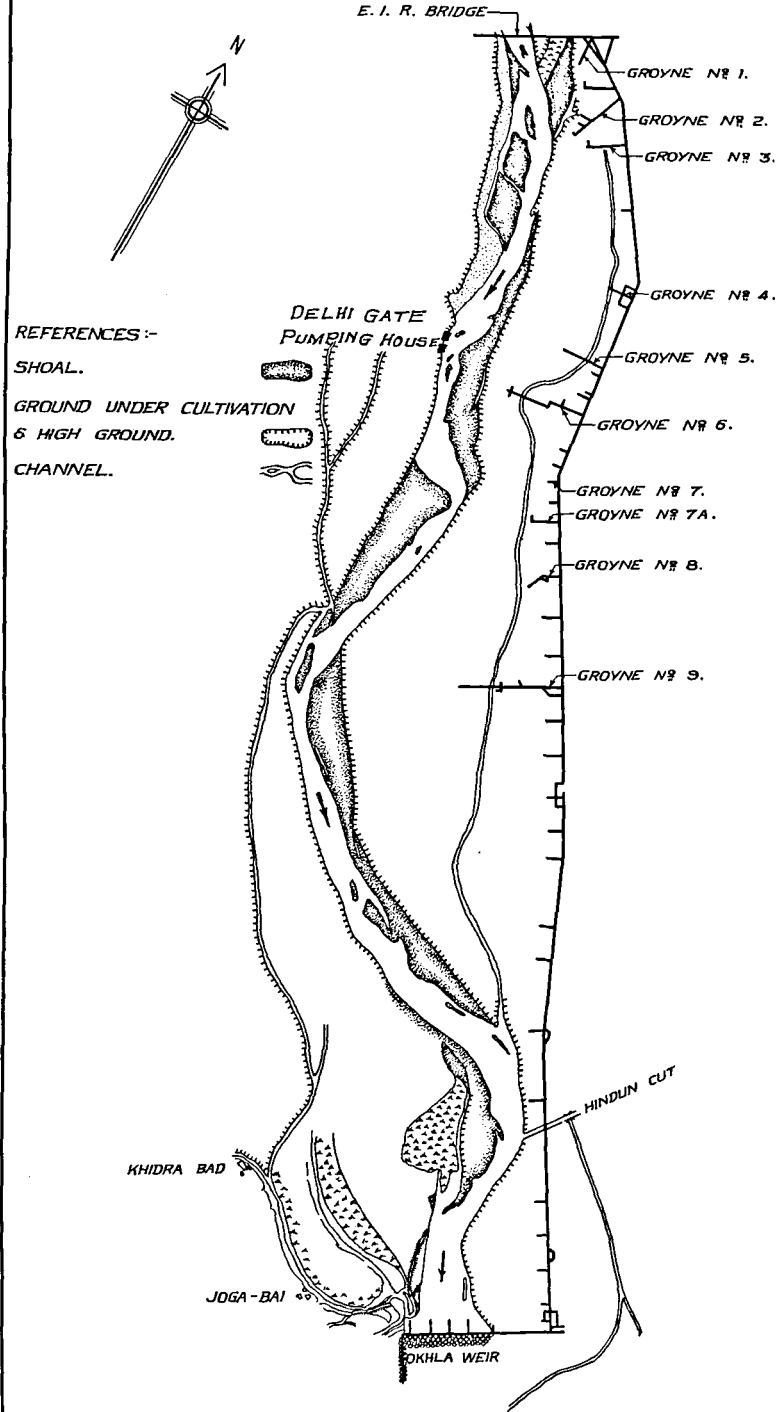
The experiments carried out previously in connection with the protection of the Hardinge Bridge (Series I to XIII)⁽¹²⁾ were concerned with flow conditions between Raita and the Bridge; and the model extended only 19 miles upstream and 8 miles downstream of the Bridge.

Subsequently it was considered desirable to ascertain what changes were likely to occur in the river course during the next few years; and in order to

(12) Described in Bombay Technical Paper No. 55 and Annual Reports for the years 1935-36 and 1936-37.

SURVEY PLAN OF
JUMNA RIVER
AFTER FLOODS OF 1924
SCALE 8 INCHES = 6 MILES

Nº 967.
C. I. & H. R. ST.
FIG: III.

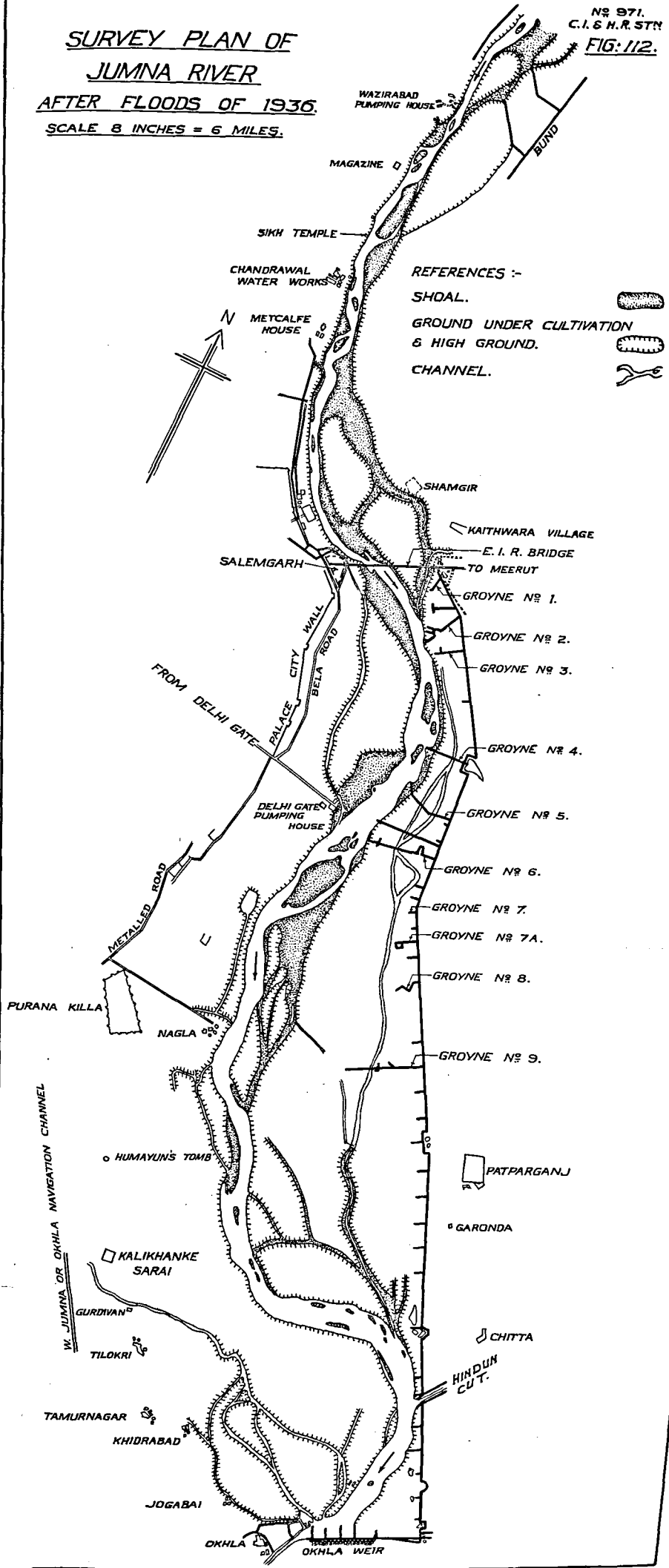


SURVEY PLAN OF JUMNA RIVER

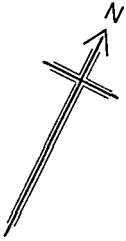
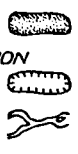
AFTER FLOODS OF 1936.

SCALE 8 INCHES = 6 MILES.

NO 971.
C.I. & H.R. STR
FIG: 112.



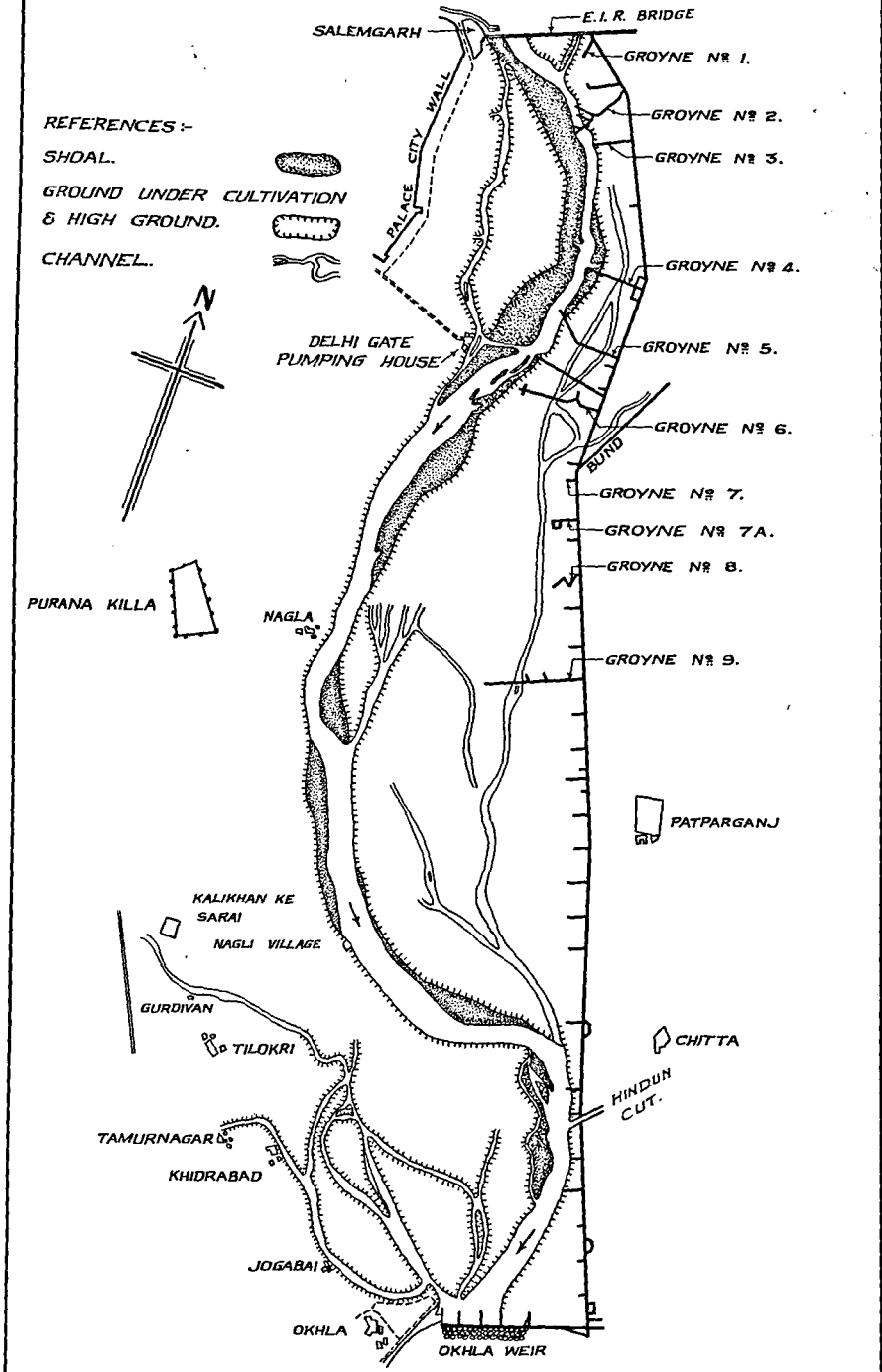
REFERENCES :-
SHOAL.
GROUND UNDER CULTIVATION
& HIGH GROUND.
CHANNEL.

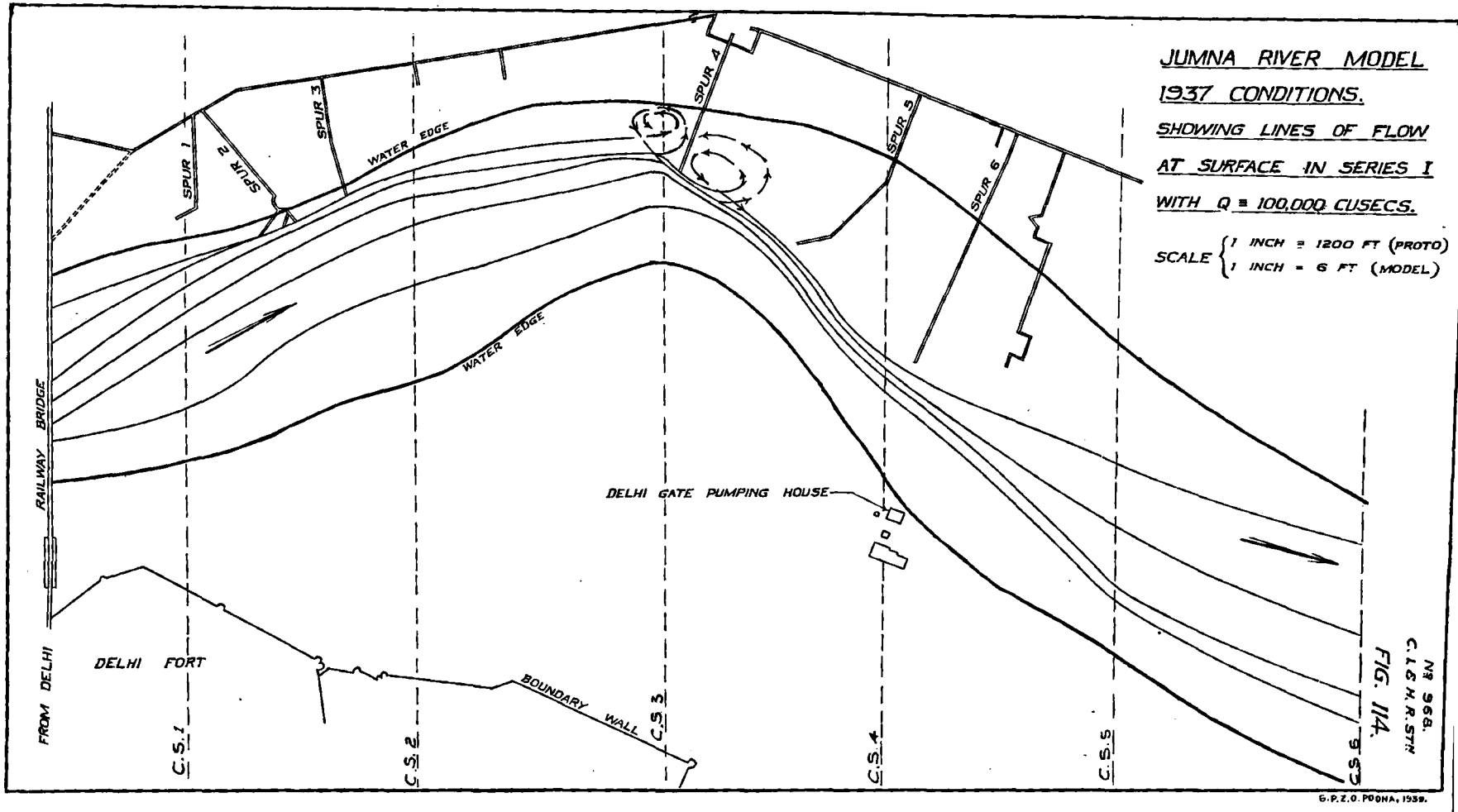


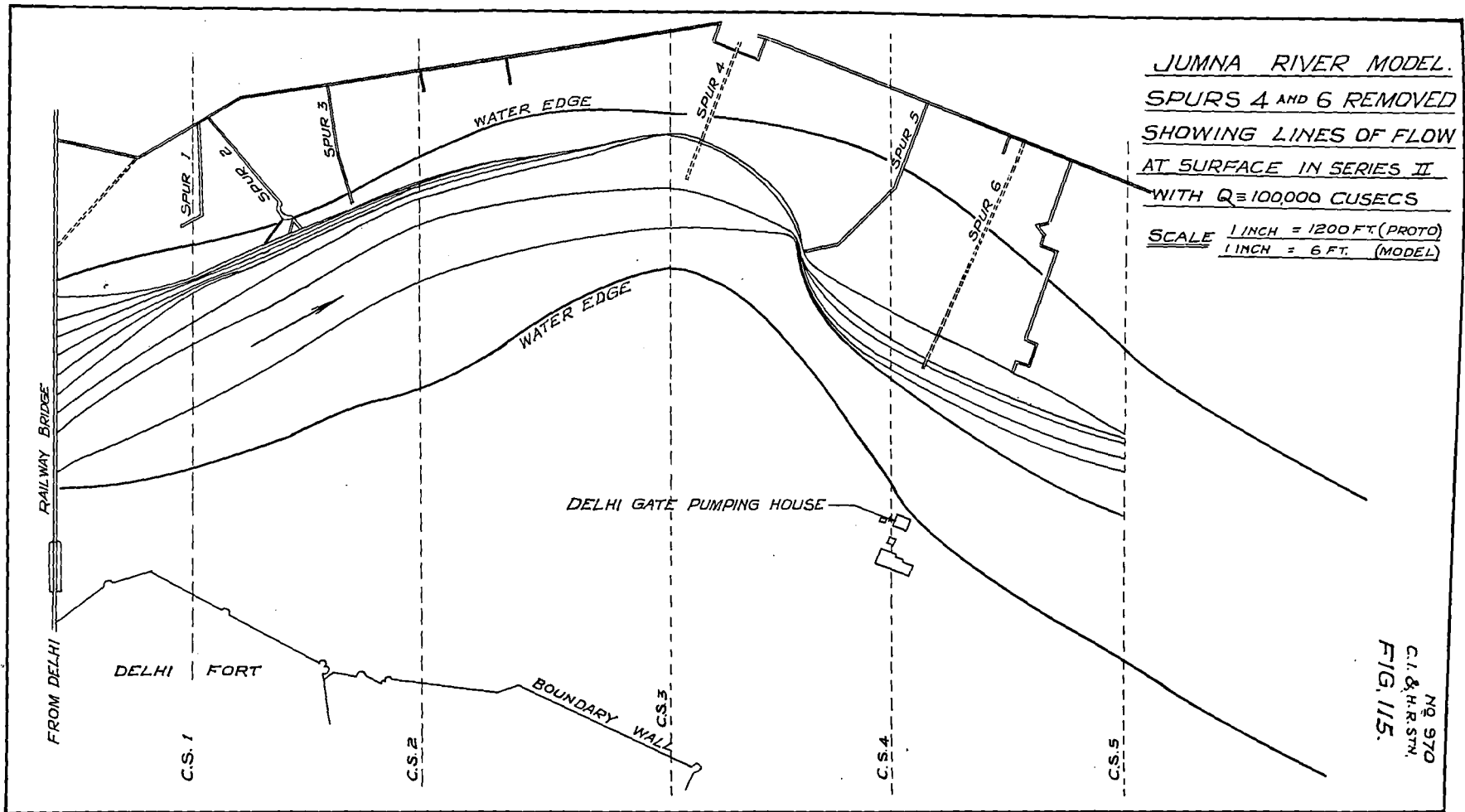
SURVEY PLAN OF
JUMNA RIVER
AFTER FLOODS OF 1937.

N^o 972.
C. I. & H. R. STN
FIG. 113.

SCALE 8 INCHES = 6 MILES.







JUMNA RIVER MODEL.
SPURS 4 AND 6 REMOVED
SHOWING LINES OF FLOW
AT SURFACE IN SERIES II.
WITH $Q \approx 100,000$ CUSECS

SCALE 1 INCH = 1200 FT. (PROTO)
 1 INCH = 6 FT. (MODEL)

NO. 970
 C. I. & H. R. STN.
 FIG. 115.

determine whether this was possible, it was decided to carry out experiments to see if the changes which had occurred in the river during the past few years could be reproduced in a model.

It was pointed out, from the first, that the difficulties to be overcome were enormous and that no one had hitherto grappled with, much less solved, a model problem of such difficulty and complexity, which raised not merely the ordinary questions of scales, vertical exaggeration, and slope exaggeration, as affected by silt conditions; but also modified vertical exaggeration of rigid portions; the length and swing of meanders; the effect of silt charge on slopes, channel changes and meanders; and the effect of all these on the time scale.

References to other Stations and to the work of others, indicated that most of these problems were untouched, and even where some work had been done at them, it had not been reduced to any theoretical basis, which could be applied. Had it not been for the background supplied by the Lacey theory and formulæ, little progress could have been made in the experiments now described.

Model.—The first essential in a model is correct entry conditions. It was assumed that this would be available where there was a length of channel which had remained stable in position for a long period of years. Such conditions existed at Sardah, 38 miles upstream of the Bridge, where the river had not changed its position for over 100 years; so the new, extended, model was constructed starting at the top of this reach and extended down to the mouth of the Gorai River, an effluent channel, 16 miles downstream of the Bridge (see figure 116). The total length of the model was 550 ft. (\approx 40 miles upstream and 12 miles downstream of the Bridge). The horizontal and vertical scale ratios were the same as those of the previous shorter model, viz., 1/500 and 1/65 respectively, (based on the Lacey regime formulæ for 2,000,000 cusecs), and the material of the model was Khadakvasla red silt, as before. This silt, being slightly coherent, was found to be suitable for the banks and edges formed by the river where it was tending to scour its banks, but was rather sticky in its original state to reproduce scour in the bed. It was therefore replaced in the latter places by "washed" silt, collected from the downstream silt pits, which was relatively free from the fine grades of silt.

During the experiments it has been found that despite the Sardah reach being stable in plan, flow conditions are not invariable—that is to say, conditions upstream of this reach have an effect on curvature of flow in the Sardah reach—and hence it is not sufficient merely to construct the Sardah reach correctly in plan, but entry conditions must be so imposed that the distribution of isotachs at the lower end of the Sardah reach is correct. This was done by constructing the stable length, forming the lower portion of the bend, in rigid material (cement mortar) and of a shape which imposed correct curvature conditions.

Above this, at the entrance to the model, a device was constructed to facilitate control of silt injection and silt in suspension. At the place where the silt was dropped into the water the latter was passed through a grid of fine wire mesh and a shallow rapid in order to throw the silt into suspension. Below the rapid, was a chamber in which the depth of water was controlled by rising gates, the purpose of which was to adjust the velocity of flow and hence the grade of silt entering the model. By this means, although silt of the same grade was injected throughout the flood cycle, only the finer grades were allowed into the model at the lower stages—so as to reproduce what occurs in Nature. The rate of silt injection was, of course, adjusted according to the discharge.

The increased length of the model, made slope an important factor. As it was necessary that the depths of water over the whole length should be according to the prototype, the water level at the Bridge was maintained correct while the discharge was gradually increased, until the water level at Markoti, near the upper end of the model, was also correct.

In the early experiments it was assumed that the slope exaggeration should equal the vertical scale exaggeration, which according to Lacey holds when the silt in the model is the same as that in the prototype. With this slope, however, the flow appeared sluggish and as there was little scouring action, except locally, and most of the silt injected at the upper end was deposited before it had been carried far, leaving the water lower down nearly clear, changes in the river course were very slow. To observe the effect of greater discharges and velocities, the

slope was then increased by 50 per cent., keeping the vertical scale as before; when it was found that not only was more silt carried in suspension and scouring action greater, but the discharges which gave the correct water levels with this increased slope were proportionately higher at the lower stages. This was an additional advantage; because scour in the model, which was particularly in defect with small discharges, was thus made to approximate more closely to the prototype.

This investigation, though started with a specific object, soon took on the character of basic research for determining the effect of specific factors and their relative importance. The horizontal and vertical scale ratios were the same in all the experiments, but the slope (and therefore discharge) scales, rate of silt injection, grade of silt admitted into the model at various stages, and time scales were varied in different experiments. In each experiment, the flood cycles of 1935 and 1936 in the Ganges were run through in the model and the results compared with the changes in the prototype (as shown by the surveys and the weekly reports). Figure 116 shows the change in low water course in the prototype in 1934-35 and figure 117 the low water course in the model (in Series XX, experiment 11) compared with that of the prototype after the 1936 flood. The course is not identical in model and prototype but the general conditions of flow were reproduced correctly. These experiments are still in their early stages.

23. EXPERIMENTS WITH FALLING APRONS.

Introduction.

A falling apron is a covering of stones laid on a river embankment to protect it from the erosive action of the current. Its design and form of construction are described in detail in Sir Francis Spring's "River Training and Control on the Guide Bank System"⁽¹²⁾ Chapter IX, from which figure 118 is an extract.

Briefly, the protective covering consists of stones, usually "one man" stone weighing 60-120 lbs., laid during low water on the slope of the bank and in a horizontal "apron" on sand or silt deposited by the river, see figure 118. The stone on the slope is intended to remain in position but the "apron" is intended to "launch" or subside, when attacked by the river current, to form a continuation, below water level, of the upper slope and so provide a complete protection against erosive action from top to toe.

The experiments to be described were carried out at the request of the Railway Board, in connection with the protection of the Hardinge Bridge. The experiments were concerned firstly with the launching of an apron; secondly the cause of failure of apron protection; thirdly with the best design of aprons laid on sand and on alternating strata of sand and clay; and fourthly with the best method of reinforcing aprons already launched.

Launching of an Apron.

A study of the action of an apron in launching, though simple, is an essential to the understanding of the behaviour of an apron protection under attack. The following is a description of the launching observed in a model consisting of an apron of $\frac{1}{8}$ "- $\frac{1}{4}$ " stones laid horizontally on the berm of a channel formed of Ganges sand. The launching took place gradually, simultaneously with the gradual raising of discharge in the channel and consequent enlargement of the latter. Figure 119 shows the apron as laid.



FIG. 119

(12) Technical Paper No. 153, Railway Board, India.

FIG:-116, SURVEY PLAN OF GANGES RIVER

LOW WATER COURSE WITH W.L. VARYING

FROM: R.L. 219 TO 224 AT HARDINGE BRIDGE.



YEARS:- 1934 & 35

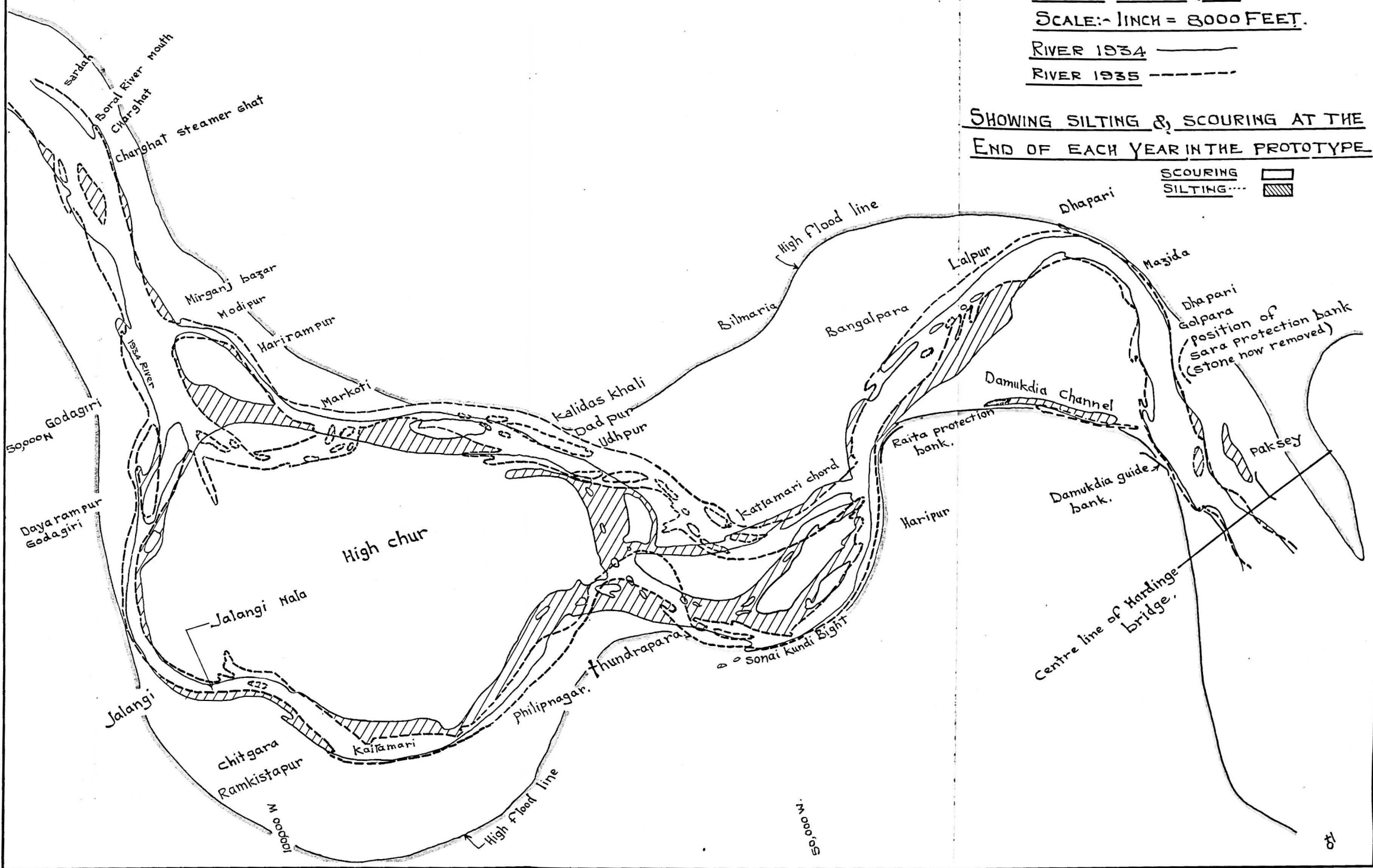
SCALE:- 1 INCH = 8000 FEET.

RIVER 1934 ———

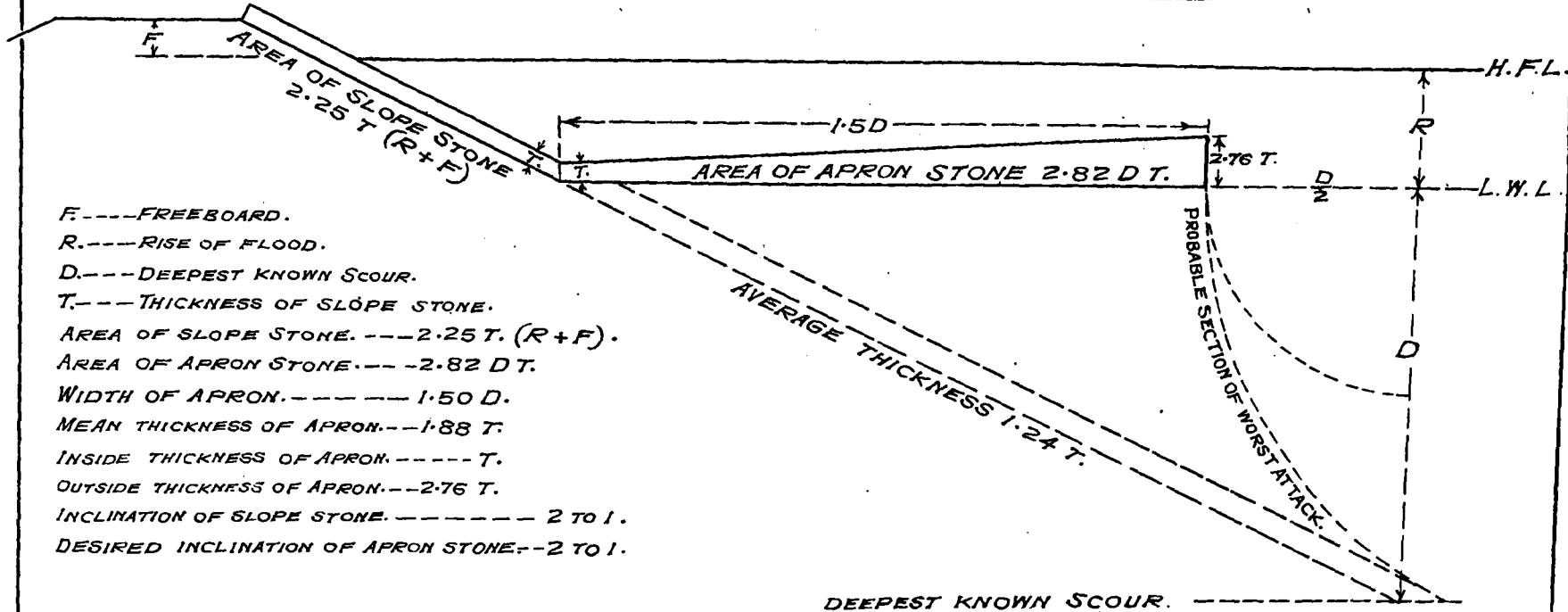
RIVER 1935 - - - - -

SHOWING SILTING & SCOURING AT THE
END OF EACH YEAR IN THE PROTOTYPE

SCOURING 
SILTING..... 



THE DIMENSIONS OF GUIDE BANK APRONS.



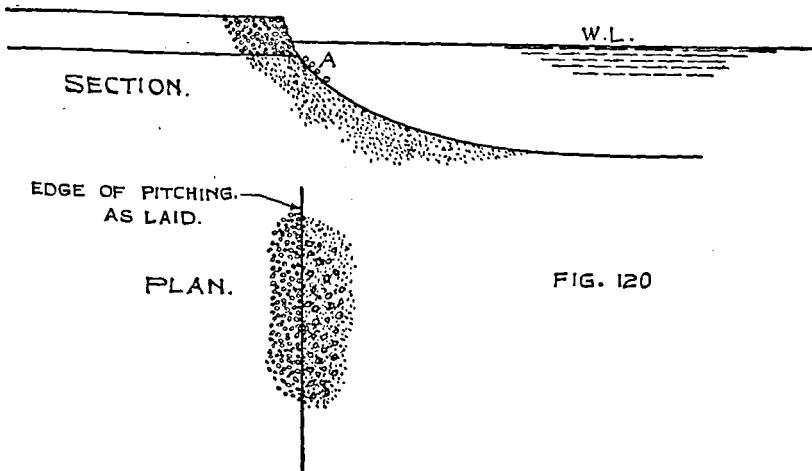
- F. --- FREEBOARD.
- R. --- RISE OF FLOOD.
- D. --- DEEPEST KNOWN SCOUR.
- T. --- THICKNESS OF SLOPE STONE.
- AREA OF SLOPE STONE. --- $2.25 T (R+F)$.
- AREA OF APRON STONE. --- $2.82 D T$.
- WIDTH OF APRON. --- $1.50 D$.
- MEAN THICKNESS OF APRON. --- $1.88 T$.
- INSIDE THICKNESS OF APRON. --- T .
- OUTSIDE THICKNESS OF APRON. --- $2.76 T$.
- INCLINATION OF SLOPE STONE. --- 2 TO 1.
- DESIRED INCLINATION OF APRON STONE. --- 2 TO 1.

PLATE I

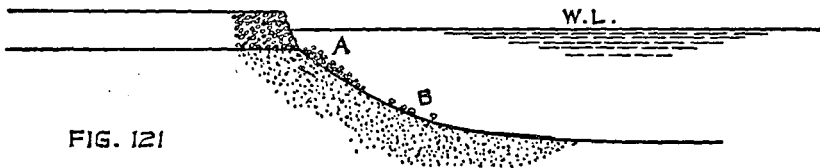
(REPRODUCED FROM RIVER TRAINING AND CONTROL
ON THE GUIDE BANK SYSTEM.
BY SIR FRANCIS J. E. SPRING, C.I.E.)

FIG. 118

As the river rises, the channel begins to deepen and widen. The layer of stones is undermined by scour at A—*vide* figure 120

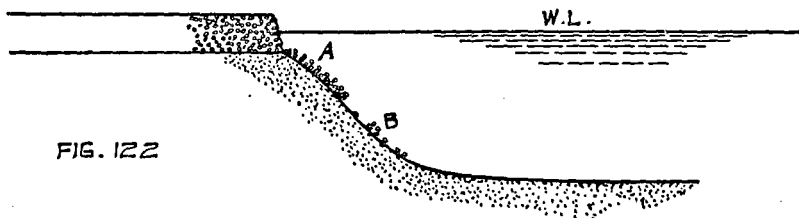


and the undermined stones settle on the slopes. These are not sufficient, however, to prevent scour of the sand at A; so further scour takes place at A causing the horizontal layer of stone to be again undermined and more stones are set in motion, some remaining with the previously fallen stones at A and others rolling down to B,—see figure 121. Scour now occurs at B

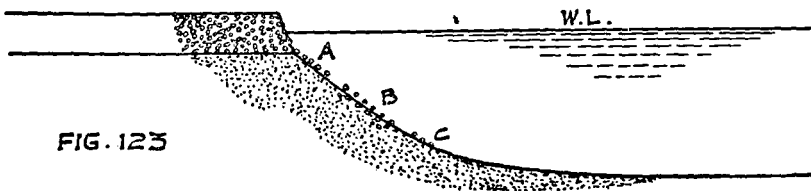


but not at A, as this is fully covered with stones.

The scour at B has undermined the stones at A, some of which will now fall and roll to B,—figure 122



and a few even beyond, to C, as shown in figure 123.



This causes the sand to be again exposed at A, which therefore scours, undermining the horizontal apron until more stones fall, and again the sand at A is covered and scour prevented. Meanwhile scour occurs at C, until stones roll down from B, uncovering sand there, and so allowing scour, which undermines A. Thus the process goes on, the sloping bank, covered with pitching stones, extending deeper and deeper as scour occurs, the pitched slope simultaneously receding towards the bank. It is clear that the fallen apron will slope at the angle at which rolling or sliding stops, since if the slope were steeper the stones would roll down further and if it were flatter no stones would have rolled down as far. If the flow is axial the velocity will, in the early stages, be almost the same over the whole slope. The inclination is therefore approximately constant. When more apron has fallen and the bank has deepened, the velocity at the bed is likely to be higher than the velocity near the berm, so there is a tendency for the apron to be at a flatter slope near the bed because the angle of repose is less where the velocity is greater. The apron finally settles as shown in figure 124 with its toe resting on the bed in its position of final scour.

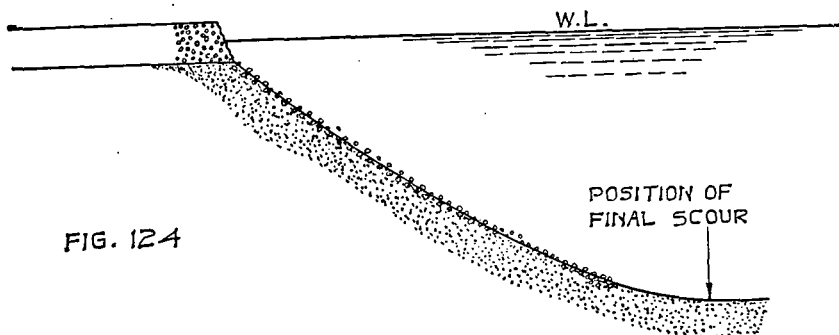


FIG. 124

Photo 1167 (Sheet 7 of Part I) shows the apron in process of launching. It will be noticed that settlement of the stones was uniform and straight under steady axial flow conditions. Some of the stones were coloured in bands so that it could be seen where they finished up. The results (as may be seen in the photo) showed that the stones subside down the slope without any appreciable motion along the channel, except a few stones at the toe of the slope which were drawn along in small "gulleys" formed in the sand bed of the channel.

Characteristics of falling apron protection.

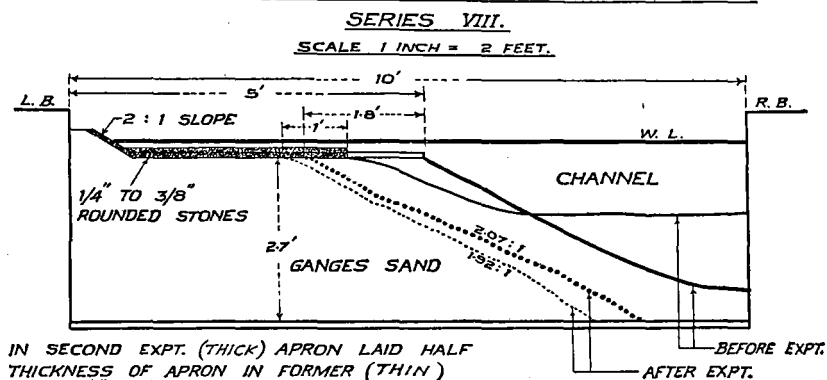
The outstanding feature of this form of protection is its flexibility in adjusting itself to resist attack, both in position and thickness of stone. In practice, attack starts at the toe, of the slope; partly because the slope is protected, whereas sand is exposed at the toe, and partly because with a sloping bank the flow conditions, tending to cause scour, are almost always most severe between mid-depth and the bed. But wherever attack takes effect it invariably causes the protected bank to recede, which usually spreads the attack and reduces its intensity, and causes more stone to slide down automatically from above on account of the recession, the layer of stone becoming more compact or greater in thickness until no more sand is scoured from beneath it.

This action is, of course, dependent on there being a sufficient reserve of apron stone above to replace what subsides or is carried away, and on the material of the bank having a flatter angle of repose than that of the stones, as otherwise the stones would not remain on the slope, as occurs in the case of clay. With this proviso, the inclination of the slope depends only on the stones, not on the sand, though the depth of slope and its lateral position will depend on the depth of channel and thus on the sand; because the finer the sand the greater the depth of scour. The inclination of the slope is the natural angle of repose of the stones under the flow conditions, *i.e.*, velocity, turbulence, etc., obtaining.

Effect of disposition of stone.

The lateral position assumed by a slope pitched with launched stone depends on where the stone is laid and on the quantity of stone laid per foot width of horizontal apron. If there is a thick apron, recession is relatively small; because a large quantity of stone falls per foot width of scour; but if only a shallow apron is laid, recession will be relatively much greater. In theory the slope recedes until sufficient stone is brought down for protection against the river current then existing, and no more; so that the quantity of stone required would be the same whether a narrow deep apron or a shallow wide apron were laid.

This point was tested in Series VIII of the experiments, in which the launching of an apron in one experiment was compared with that of an apron twice as wide and half as thick in another experiment; see figure 125. The discharge

FIG:-125.- EXPTS WITH FALLING APRON

in both experiments was raised by similar stages to the same maximum. The quantity of stone which launched should have been the same in both cases but was actually a little less with the wide apron, and the slope was less steep (see fig. 125), which difference was at least partly due to the velocity being higher in the wide shallow apron (1.86 : 1.72 ft./sec.) on account of the restricted waterway during launching of the wide apron experiment, which effect would, of course, not be so marked in an actual river. Taking this into account, the proposition stated may be said to have received approximate confirmation.

Inclination of slope.

A study of the action of an apron in launching, as already described, showed that the angle of slope assumed by the fallen apron depended on the shape of the stones and was in fact likely to be the steepest slope at which stones rolling or sliding on sand would come to rest.

For comparison with the actual angles of slope obtained in the experiment, a series (IX) was done with a "tilting board," which could be set at any angle, and on which stones were laid, to find the natural angles of repose of the stones in water with various velocities of flow. The stones were rolled down from the top and as it was found that at a relatively flat slope most of the stones came to rest but a few rolled to the bottom, while at a greater slope a few came to rest and the majority rolled to the bottom, an arbitrary criterion for the angle of repose was adopted, viz., the angle at which 50 per cent. of the stones came to rest and 50 per cent. rolled to the bottom.

The angle for stationary stone was also observed, *i.e.*, the angle of slope at which stones lying stationary on the slope would begin to slide down. The following results (Table 18) were obtained with still water :—

TABLE 18.

	Angle for stationary stones.	Angle for rolling stones.
Rounded stones laid on similar stones	38° ≡ 1.3 to 1	31° ≡ 1.7 to 1
Angular stones laid on similar stones	40° ≡ 1.2 to 1	34° ≡ 1.5 to 1
Rounded stones laid on Ganges sand	31° ≡ 1.7 to 1	25° ≡ 2.1 to 1

With flowing water the angles were less. Figure 126 shows the curve obtained by plotting the sine of the angle against the velocity $\frac{1}{2}$ " above the stones and, for comparison, the points plotted from data of aprons actually launched. These show considerable scatter, but in general they lie about the curve.

It will be seen that the angle of repose is dependent to a small extent on the velocity; the slopes of launched aprons tending to be flatter where the velocity of flow is high and attack severe. If the stones are flat and angular, the slope is steeper.

Conditions under which Apron Protection may fail.

Owing to the flexibility of an apron protection in adjusting itself to resist an attack, the failure of an apron can occur only in the following ways :—

- (1) the carrying away of stones by the current,
- (2) launching beyond the capacity of stone reserve, *i.e.*, the utilisation of all available stone in the horizontally laid apron and in the original slope above, leaving the bank material exposed to the current and wave action,
- (3) slips and "blow outs" in the bank, due to a steep subsoil water gradient,
- (4) irregular disposition of stone on the slope due to the presence of a coherent material such as clay.

In connection with (1), several experiments (Series VII) were done to correlate the local conditions under which stones are carried away by the current (*viz.*, velocity, slope of apron), with the properties of the stone (*viz.*, size and shape).

Sufficient data was not obtained to indicate this correlation but it was found that when the point velocity at which stones were carried away was plotted against size, the results lay on curves representing velocity to a power between 1.3 and 1.5, thus suggesting that with the small sizes of stones used (up to $\frac{1}{2}$ " grade) viscosity had appreciable effect, which precluded extrapolation to larger grades with any degree of accuracy.

The effect of slope (*i.e.*, side slope) was interesting. It was found that stones on a slope were carried away at a slightly lower velocity than stones in a horizontal layer, but the results for various angles of slope were practically the same.

Reproduction of breach in Guide Bank, Hardinge Bridge (Series X).

The conditions leading to the breach in the Right Guide Bank of the Hardinge Bridge in 1933 had been reproduced in the 1/500 scale model of the Bridge and River,⁽¹²⁾ but it was desired to reproduce the actual breach and examine the process in detail.

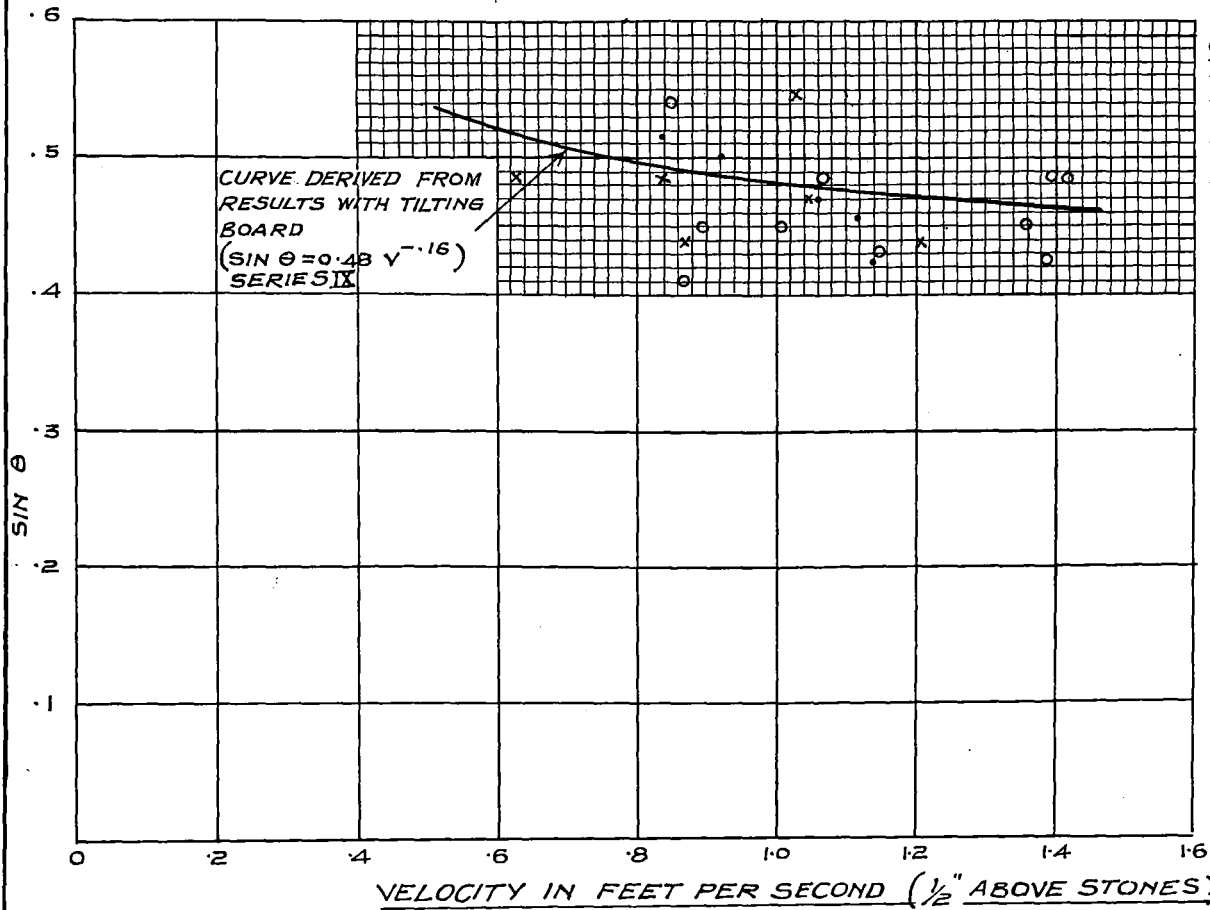
Figure 127 shows, in plan, the Bridge and guide banks and the place where the breach occurred. A large scale model was necessary to reproduce the detail and permit geometrical similarity, which was necessary for the correct behaviour

⁽¹²⁾ Bombay P. W. D. Technical Paper No. 55 : "Report on experiments carried with models in connection with the protection of the Hardinge Bridge, on the Eastern Bengal Railway," and Annual Report, 1935-36.

FIG:-126.

COMPARISON OF RESULTS
 WITH TILTING BOARD (ANGLE
 OF REPOSE FOR ROLLING
 STONES) (SERIES IX)
 WITH THOSE OF LAUNCHED
 APRONS
 (ANGULAR STONES $\frac{1}{8}$ " - $\frac{1}{4}$ ")

SERIES ---- IV -----•
 SERIES ---- V -----x
 SERIES ---- VIII -----o
 θ = ANGLE OF INCLINATION OF SLOPE.

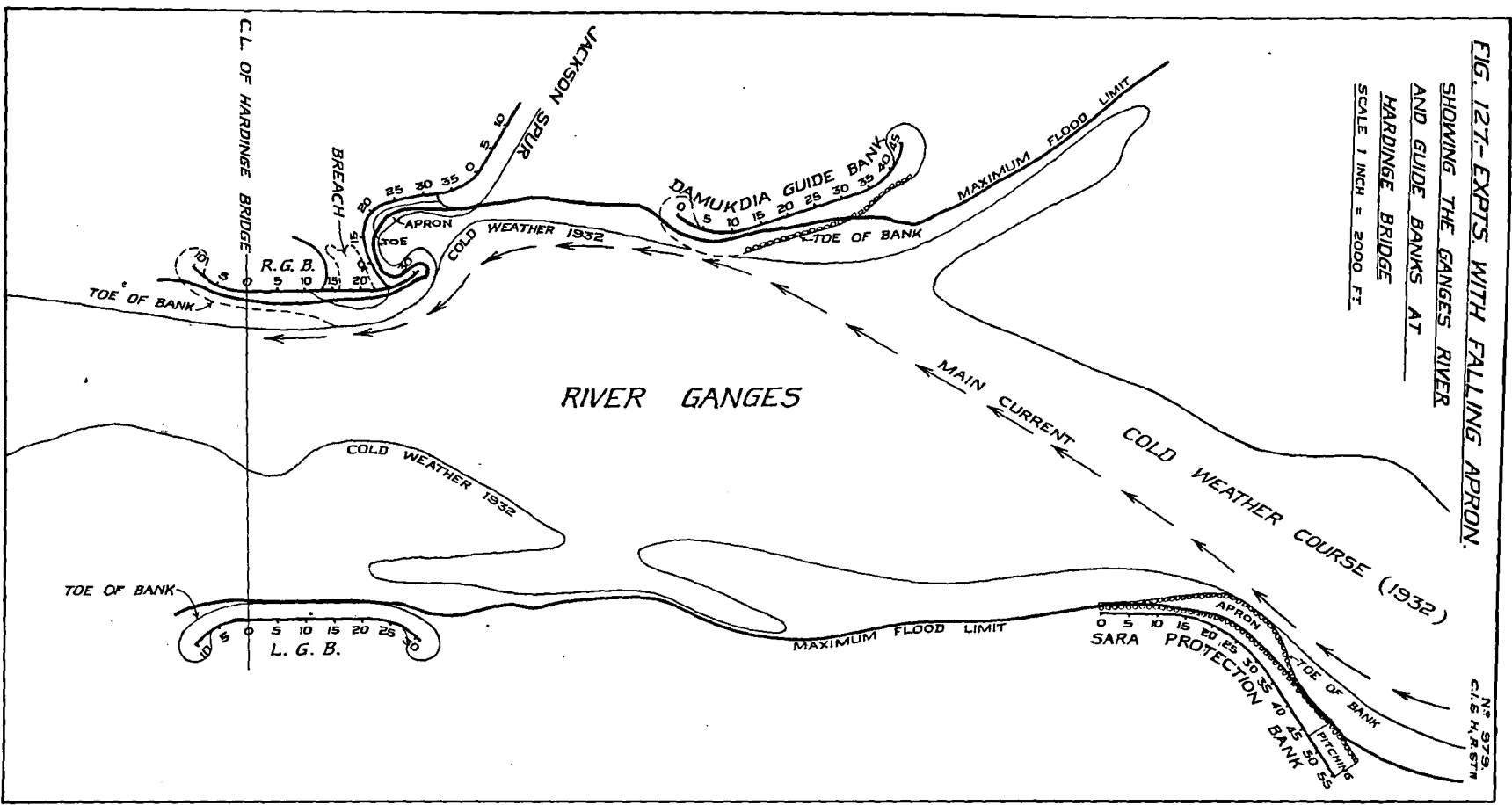


NO
 C.I. & H. STN.

FIG. 127-EXPTS. WITH FALLING APRON.
SHOWING THE GANGES RIVER
AND GUIDE BANKS AT
HARDINGE BRIDGE

HARDINGE BRIDGE
SCALE 1 INCH = 2000 FT.

N^o 979
C.I.S. H.R. 511



© P. T. O. P. 20014, 1935.

of the stones. The scale chosen was 1/200, and in consequence only a part width of the river could be included. Moreover, to fit the channel, the model had to be reversed, the guide bank used being a left guide bank. At first $\frac{1}{8}$ "- $\frac{1}{8}$ " grade stone was used for the apron, this being approximately "one-man" stone to scale, but this was found to be too small to give correct results and was replaced with $\frac{3}{8}$ " stone. A high discharge was required, which deepened the channel considerably beyond the depth corresponding to the prototype depth to scale, indicating either that the stone protection laid in the horizontal apron was relatively thicker than it was in the prototype, or that vertical distortion is necessary in silt models of this type.

Photo 1183 (Sheet 8 of Part I) shows the experiment in progress and photo 1187 (Sheet 8 of Part I) after the slip had occurred. The wall on the left in the photo was so aligned as to reproduce the lines of flow observed in the 1/500 model and was perforated by holes below the surface to draw off bed water sideways, so as to create curved flow, which set up the conditions which caused the breach.

The following extracts from the reports of the Hardinge Bridge staff at the time of the breach describe briefly the events as observed by them:—

"Early in September a definite change in the direction of the main current was observed after it had left the Damukdia Guide Bank and . . . the current impinged on the head of the Right Guide Bank at about chainage 27."

"Dangerous eddies were observed in front of ch. 14 to 17 and on the 26th September at about 4-00 hours this guide bank was breached between ch. 19 and 22 on a rapidly rising river. During the course of the day the breach extended from ch. 14 to ch. 25 by erosion from inside the embayment."

The breach extended rapidly and by 12-30 hours was 1,200 ft. long and the embayment behind was 500 ft. across. The river continued to rise and on the 2nd October eroded the narrow neck connecting the head of the guide bank with the main land, thus isolating the head and allowing a clear flow from the embayment behind the head into the new embayment behind the breach and out again into the river. Reserve stone was rushed up and a retired bund was formed to prevent further extension of the breach and the river later subsided.

An observer might have thought that the breach, which started at a high level was due to severe eddy action and strong currents near the surface. The model however showed that the surface eddies were practically harmless, while the severe attack was at the toe of the bank, whence stone was carried away and swept downstream. Stone from above then slid down to replace it, only to be in turn carried away, and this process continued until a patch of bank was denuded of stone and the sand core exposed just below the water surface. This exposed patch was then eroded very rapidly under the combined action of wavewash and breathing in the river. This breathing first filled the breach with water which subsequently discharged back into the river with high velocity, sucking out sand.

The breach in the model can be clearly seen in photo 1192 (Sheet 9 of Part I) taken after the top of the bank had been denuded and scoured, but before appreciable flow had occurred from the embayment on the left. The wooden frame shows the original line of the top of the bank. Figure 128 shows the isotachs observed at ch. 10 and 18 of the guide bank.

In order to observe more closely what was happening at the toe to cause the stones to be carried away, the portion of the bank where the breach occurred was replaced by a glass window through which movements of the bed material and stones could be seen. This was Series XIII. The glass panel was made flush with the bank at each end and extended from surface to bed. The discharge was then raised to the value at which the breach had occurred, keeping other conditions the same. Cross wires and battens were fixed on the glass to hold one or two rows of stones in place when desired.

An excellent picture was obtained of what was happening. Near the bank, at the surface, there was slow return flow. At the toe was vigorous forward flow. In between was slight fluctuating motion, evidently forming the base of the large slow eddy visible from above. The fact that sand deposited at mid-depth showed that the action here was not severe; in fact it was obviously severe

only near the toe, where sand and occasional stones could be seen rushing past at high velocity.

It was not however a steady velocity but came in surges (about 2 to 5 per second) with a surge of greater velocity every few seconds. The mean flow appeared to be horizontal, not diving, but the direction fluctuated up and down with the surges, perhaps within $\pm 10^\circ$. The stones on the battens were not carried away by the small surges but were sometimes rocked and sometimes carried away by the bigger surges.

It was noted that sand would remain on the glass under a group of stones, being sheltered by them.

It was clear that the severity of attack was due not only to high velocity but to surges of high velocity accompanied by a fluctuating direction of flow which removed stones and carried them away.

In order to obtain a measure of the severity of attack, cement cubes from $\frac{1}{2}$ " up, were lowered on a string to the toe and it was noted what was the smallest cube which would stay in position, resisting the current. Pitot tube readings were also taken and recordings made with the hot wire instrument, all for various conditions of flow, *viz.*, with and without cross-holes open, flow directed by vanes at the head of the guide bank, at the bank just upstream of the breach and into the embayment and straight axial flow down the channel. All these conditions were hypothetical except the first mentioned; which actually caused the breach.

The size of cube remaining in position was 2" in the case of the flow directed into the embayment, $3\frac{1}{2}$ " when the current was directed at the bank (*i.e.*, worst case) and $2\frac{1}{4}$ "- $2\frac{1}{2}$ " in other cases. The velocities recorded gave the same indications. The maximum velocity recorded at the toe was about 3 ft./sec., which was when the flow was directed at the bank.

The hot wire instrument showed the most violent fluctuation (measured 3" away from the toe) when the flow was directed at the head of the guide bank, and nearly as much under the conditions which caused the breach; for other conditions they were relatively small.

Observation of action on larger scale.

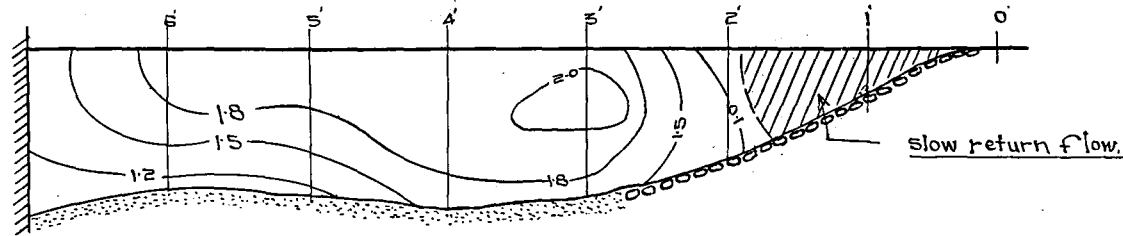
All the previous experiments had been done using small stones. To compare the action of larger stones, 3" stones were laid on Ganges sand in a bank at 1:6 to 1 slope and subjected to an increasing current. On the curve at the head, where velocity was higher than in the straight, sand was sucked out from between the stones, which then closed up, giving more protection to the sand below. The sand sucked out was deposited on the apron lower down. As the discharge was increased the bed scoured and the stones at the toe moved down followed by those above. There was no rolling as had happened when the aprons of smaller rounded stones launched, the movement was sliding and so slow as to be almost imperceptible.

In order to be able to see the stones carried away by the current, the Ganges sand was replaced by red silt, which would not be sucked out so easily. The discharge was raised until the attack at the toe just downstream of the curve at ch. 22 caused the stones there to fall into the scoured bed. The discharge was raised to the maximum (22 cusecs) and the cross holes opened, but no stones were carried away although they slipped down generally and silt was scoured from the area left bare at the top.

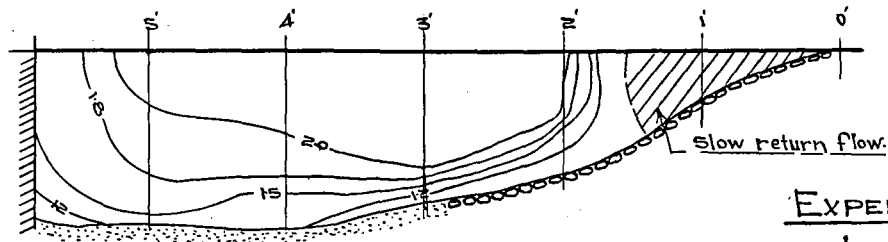
The bank was then relaid with $1\frac{1}{2}$ " stones and the experiment repeated but only with discharges up to 12 cusecs. General slipping or launching of the apron occurred as before.

9" stones were gently pushed over the berm, to represent large blocks put in for extra protection. These did not reach the toe but remained half way and the turbulence generated by these stones displaced some of the $1\frac{1}{2}$ " stones.

As a further experiment a model was constructed having a mixture of $1\frac{1}{2}$ ", 3" and 5" stone laid in a horizontal apron, the idea being that the larger stone would form the main protection while the smaller would fill the interstices to protect the bank from turbulence caused by the larger stones. The apron launched in the usual way. When the discharge was raised, silt was sucked out from between



Cross Section at chainage 10 of Right Guide Bank.



Cross section at chainage 18 of Right Guide Bank.

FIG:- 128.

EXPERIMENTS WITH FALLING APRON,
ISOTACHS OBSERVED IN
MODEL BEFORE BREACH
OCCURRED,

SCALE:- 1" = 1'

NO 978
 CIVIL
 ENGRS

the stones as in other experiments but later the bigger stones appeared to lie more on top of the smaller stones, which then protected the silt below.

Where large boulders or blocks are used, it appears advisable to have also stones of several smaller grades, each grade protecting the next smaller grade from the turbulence caused by the larger stones.

Conditions under which a failure due to slips in sand or to "blow-outs" would occur (Series XV).

It was considered at the Hardinge Bridge that slips sometimes occurred due to the water level behind the bank being some feet higher than in the river; so a model of the Right Guide Bank was constructed with Ganges sand and a gradually increasing hydraulic gradient imposed. There was no failure until the mean hydraulic gradient became 3.6 to 1, when water oozed from the toe, and by removing sand, caused a slip from above. This gradient is far steeper than any which could occur in the prototype; and the explanation of slips (imputed to high water level behind the bank) was that these slips occurred on a rapidly falling river due to saturation of the bank causing a steep gradient just at the face of the bank and these slips were only local.

It was also considered whether a slip could occur due to heavy superimposed loading on the berm, such as that due to stacking stones, with or without a hydraulic gradient. Though loading up to 700 lbs. per square foot was imposed near the edge of the bank no slip took place.

Behaviour of an Apron laid on alternate layers of sand and clay.

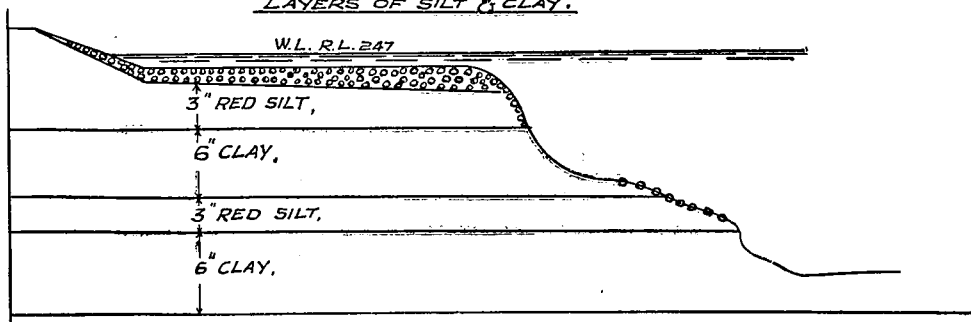
Borings taken at the Sara Guide Bank showed that that bank consisted of alternate layers of sand and clay.

In order to test the behaviour of an apron laid on such a bank, a 1/40 scale model was constructed (Series XVI) consisting of alternating layers of red silt and clay, 3" and 6" thick respectively. The top layer was of silt, on which an apron of $\frac{1}{4}$ "- $\frac{3}{8}$ " rounded stones was laid.

No scour took place with axial flow, so a concentrated discharge was directed at the bank with the aid of a vane, the mean velocity past the bank being equivalent to 17 ft./sec. in prototype.

In the case of the sand layers, the stones slid down as scour occurred, but the clay layers scoured unevenly, the clay scouring into steep cliffs, so that the apron could not rest on it uniformly but fell to the bottom. (See figure 129).

FIGURE 129.
EXPERIMENTS WITH FALLING APRON
SHOWING APRON LAUNCHED ON ALTERNATING
LAYERS OF SILT & CLAY.



It may be said that an apron does not launch satisfactorily unless—

- (i) the angle of repose of the underlying material is flatter than that of the stones
and
- (ii) the material scours easily and evenly.

Summary of Conclusions.

1. An apron of loose stones on incoherent sand launches to a slope which is the angle of repose of the stones rolling or sliding on sand under the flow conditions prevailing.

2. The concentration of stone on the slope is such that adequate protection is given to the sand beneath. If the scouring action becomes more severe, stone falls and the slope recedes, bringing down more stone from above, thus strengthening the protection and automatically adjusting itself to the attack.

3. The lateral position of the slope is determined by the initial disposition, the quantity of stone in the horizontal layer per ft. width, and the depth of scour; and the slope recedes until sufficient stone is brought down to protect the toe and face of the bank.

4. Normally the reinforcement of an apron is automatic and all that is required is to lay sufficient stone in the horizontal layer, but in order to avoid recession of slope or to provide protection consisting of larger stones at the toe, it may be advantageous to drop stone from barges.

5. An apron does not launch satisfactorily unless—

(i) the angle of repose of the underlying material is flatter than that of the stones,

and

(ii) the material scours easily and evenly.

A loose apron protection is not suitable for a bank of alternating layers of sand and clay.

6. The danger of slips due to a steep hydraulic gradient or to a heavy superimposed loading is negligible with Ganges sand; but when the river is falling quickly a slip may occur due to the bank being saturated.

7. An apron protection over incoherent sand may fail on account of—

(a) insufficient stone being provided for the protection of the slope,

or

(b) the stone at the toe being carried away by the current resulting in stone sliding down until all the available stone is launched, when a further slip exposes part of the bank face; after which though attack is most severe near the toe of the bank, the failure occurs through the exposed upper portion of the slope being scoured by the current; wave action and river breathing.

(a) can be prevented by providing sufficient stone and (b) by providing stone of sufficiently large size. A graded mixture is more resistant than stones of a single grade and when large stones are used it is advisable to provide smaller stones to protect the sand beneath. Turbulence is a major cause of the removal of stone by the current and this was intensified in the case of the Hardinge Bridge by the manner of approach of the main current, *i.e.*, directed at the head of the guide bank.

8. In general it appears that stone of the size normally used would not be washed away, when laid on incoherent sand, unless—

(i) the rise of the river, or increase of velocity, were rapid, as otherwise the apron would have time to adjust itself by launching more stone,

or

(ii) the turbulence is present to a high degree, as this results in rapid fluctuations in velocity and direction of flow,

or

(iii) the river were opposed by restriction in such a way that it could not develop a natural channel so that velocities were unnaturally high,

or

(iv) the guide bank were incorrectly designed, as for example a head designed too short or with too sharp a curve.

Apart from these, the falling apron would appear to be impregnable, if supplied with sufficient stone, and self-adjusting to resist all possible attack.

24. TRAINING THE WATRAK RIVER TO PROTECT KAIRA TOWN, GUJERAT (BOMBAY PRESIDENCY).

Experiments carried out with models.(1) *Introduction.*

In 1937 exceptionally high floods in the river Watrak caused erosion of the left bank of the river above Kaira town, where a portion of provincial road was washed away, and the N. W. corner of the town wall was undermined and collapsed together with a few houses near the edge. Figure 130 shows the river course and the area eroded by the flood (in red).

This river flows in an alluvial bed of fine sand (mean diameter=about 0.25 mm.). It is remarkable for two sharp right angle bends: one (shown on Figure 130) a quarter of a mile and the other, one and quarter miles above the town and for the fact that until 1927, when in a high flood the erosion shown in the figure began, there was apparently little change in the river course from year to year.

After the 1927 flood four groynes (made of teak piles filled between with sand bags and brushwood) were constructed on the concave bank above the town. Two were washed away in 1930 and the remaining two in 1937. Two similar groynes which had been constructed in the year 1923-24 along the city wall have remained in place.

A high level causeway was proposed at site C (figure 130) and later a bridge at site B. Neither has, however, yet been constructed.

No flood discharges have been observed; but there are indications that the maximum flood discharge at R. L. 87.00 is about 100,000 cusecs with mean velocity of 6 ft. per second, while the normal annual flood discharge at R. L. 78.00 is about 40,000 cusecs. The normal cold weather water level is at R. L. 64.00.

(2) *Experiments.*

After the 1937 flood it was clear that unless protective measures were taken, further erosion would be caused below the bend by large floods and the town would be gradually washed away. Experiments were started at Khadakvasla, to enable the existing condition of flow to be studied and to test remedies.

Two models were used, one of horizontal scale 1/125 and vertical scale 1/25 to reproduce the flow of the whole river from above the second bend to below the town and to test the effect on the river course of spurs of various designs, and the other of scale 1/30, geometrically similar to the prototype to show the flow and local scour around certain spurs of selected design. The latter was a part width model. Both models were constructed of Khadakvasla red silt.

(3) *1/125 Scale model. Details of Model.*

The following scale ratios were adopted for this model:—

Horizontal	1/125
Vertical	1/25
Vertical exaggeration	5
Max. Discharge	1/16,650
Time	1/30
Reynolds No. (at 100,000 cusecs)	91,000.

The water levels in the prototype were known for two estimated discharges, viz., 40,000 and 100,000 cusecs. Level for other discharges were inter- or extrapolated from these figures.

As practically no data was available regarding the floods and flood periods, it was assumed that the normal monsoon discharge was 10,000 cusecs and that when a flood of 100,000 cusecs occurred the rise occupied 2 days, the duration of maximum flood being 1 day and the fall to 10,000 cusecs occupied 2 days.

Silt was injected at the upper end at rates varying from 144 grains per cubic ft. of water for \equiv 10,000 cusecs to 315 grains per cubic ft. for \equiv 100,000 cusecs.

The model was laid according to the survey done after the 1937 floods and similarly relayed before each subsequent experiment.

(4) *Experiments with vertically exaggerated model.*

In experiment 1, the river in its present condition was modelled and run with the flood described in paragraph 3, in order to observe the flow and to provide a basis for comparison for results obtained in other experiments. The corner of the

fort wall, which was constructed of clay, was severely attacked by the current and subsided into the river, as occurred in the prototype. Downstream of the corner the main current hugged the left bank as shown in figure 131, causing scour along the left bank and town wall. Photo 1628 (sheet No. 24) shows this experiment.

The subsequent experiments dealt with alternative measures of protection. It was necessary both to protect the corner of the fort wall and to throw the river away from the wall further downstream.

In experiment 2 a rigid, Burma (hockey stick) type of spur, (see photos 1634 and 1670 sheets Nos. 24 and 25) 187 feet long was tested, with curved head. It was found, however, that this did not throw the current sufficiently away from the wall and in experiment 3 a longer Burma type spur was tried. The throw-off was then more satisfactory but there was still some attack at the fort wall upstream of the spur, and to avoid this, the radius of curvature of the head of the spur was increased from 60' to 102.5' (experiment 4)—see figure 132, which also shows the lines of flow obtained with this design. A satisfactory feature was the deposition of silt near the fort wall just downstream of the spur (see photo 1634 in sheet No. 24).

A Burma spur, similar to the previous one, except that the nose was extended by 40 feet in the upstream direction, was tested in experiment 5. Flow was very similar to that in experiment 4 but was slightly more spread out downstream. The design tested in experiment 4 was considered to be the best of this type. The depth of scour at the nose was less, and the cost of construction less, than the design tested in experiment 5 and the results were as good. Figure 133 showing limits of main surface flow indicates how the current was diverted from the town wall in experiments 2 and 4 compared with 1. Figure 134, cross section 7 (downstream of the spur) in various experiments shows how scour along the town wall in experiment 1 was prevented by construction of the spur, the river being diverted away from the fort wall, this effect being most marked in experiment 4.

(5) *1/30 Scale, part width model. Details of model.*

It was then decided to test a spur sloping down into the bed at 1 in 5. As the vertical exaggeration of the 1/125 model was 5, this spur would have been at 1 to 1 in this model and hence the throw-off of the current would be different to that in the prototype. A geometrically similar, large scale, part width, model was therefore constructed. This model did not include the full width of the river (see figure 135) and so movements of the river as a whole could not be exactly reproduced but local flow and scour around the spur—which was the important factor—could be studied in greater detail than in the vertically exaggerated model and the spurs used in the latter could be tested here also and compared with the sloping spur.

(6) The linear scale ratio of the model was 1/30. The discharge scale was fixed by running up the discharge until the velocities on a line upstream of cross section 6 were similar to those observed in the first model for 100,000 cusecs (taking into account difference in vertical scale). This occurred with 20 cusecs = 1/5000 the prototype discharge, which was practically the same as given by $1/(\text{linear scale})^{5/2}$. The time scale, given by $1/(\text{linear scale})^{1/2}$ would have been 1/5.5, but 1/4 was adopted to allow for the silt used being slightly more coherent than the bed material in the prototype. Actually it was found that the scour reached finality in the flood period allowed, *viz.*, 2 days rising from 10,000 to 100,000, 2 days at 100,000 and 2 days falling again to 10,000, and the results were, probably, well on the safe side as regards depth of scour, because floods of such magnitude are so infrequent that the material of the river would become consolidated to a degree far in excess of that in the model.

The rate of consolidation is, in fact, the same in the prototype as in the model and as we are dealing with annual floods in the prototype we cannot reproduce the aging effect of time in the model, nor have we been able, as yet, to prepare any model material which simulates the toughness of naturally consolidated silt.

(7) *Experiments with geometrically similar, part-width model.*

In experiment 1, with this model, a sloping spur extending into the river in line with the fort wall facing the Vehela was tested, but the lines of flow showed

FIG. 130.
WATRAK RIVER.

No 969.
C.I. B. H. RSTN.

NEAR KAIRA TOWN
SCALE:- 8 INCHES A MILE.



NOTE:- ERODED PORTION SHOWN IN RED.

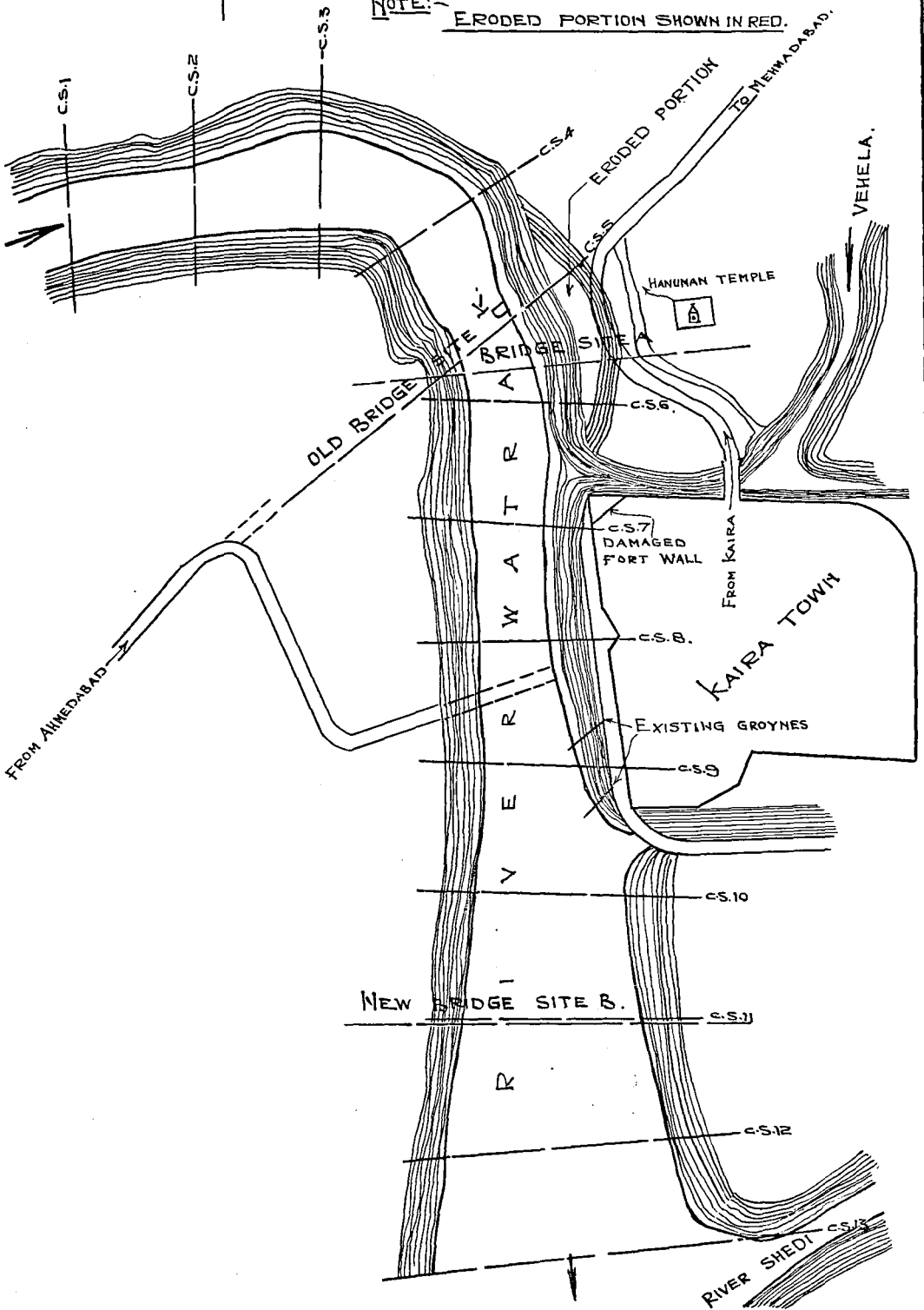


FIG: 132, SERIES I

NO 373
C.I. & H.R. 51

EXPT: 4

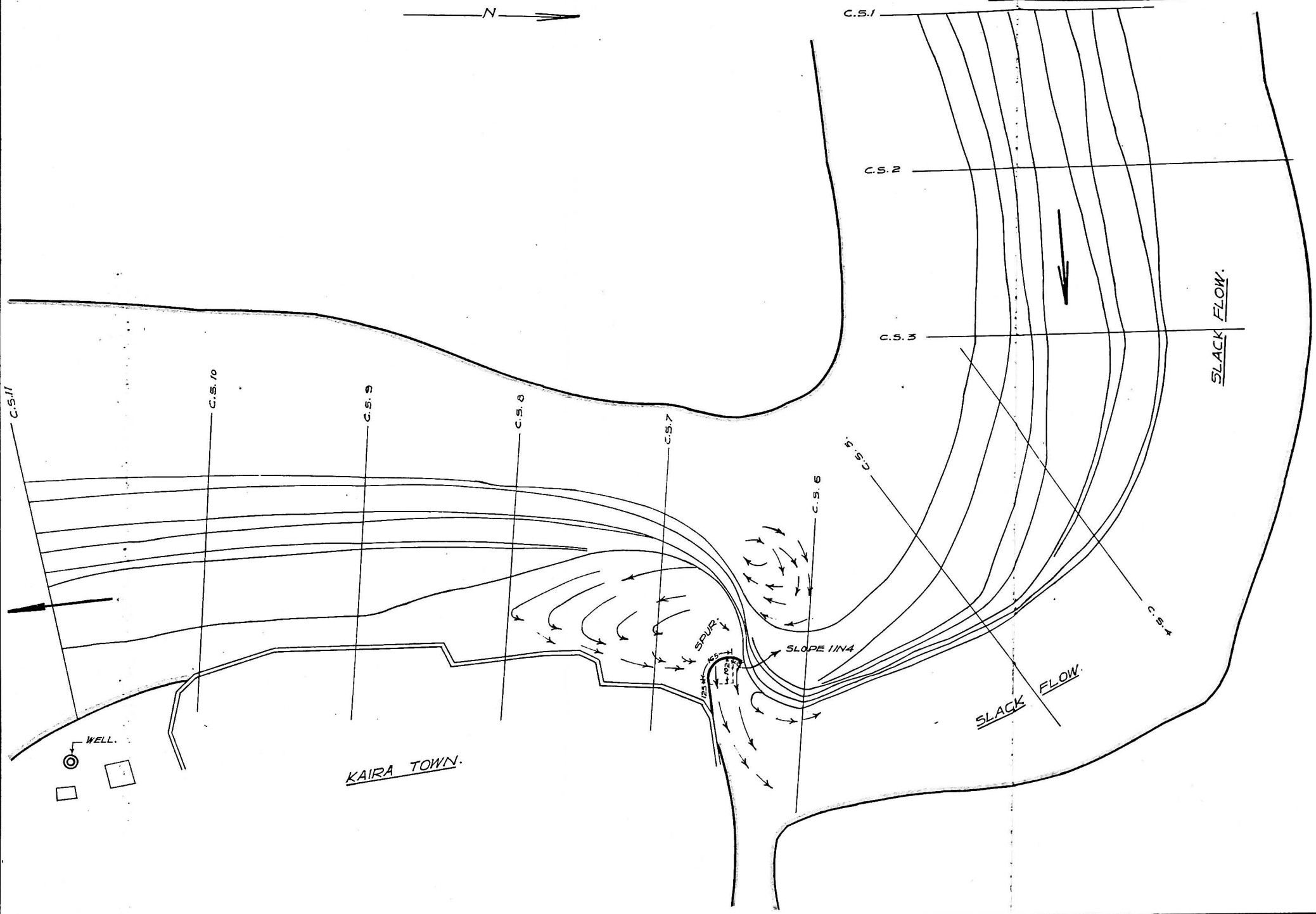
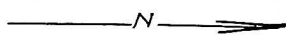
WATRAK RIVER MODEL

LINES OF SURFACE FLOW.

SPUR LENGTH = 286'

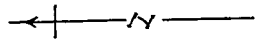
MAX: SCOUR TO = R.L. 23

SCALE - 1" = 3' IN MODEL = 375' IN PROT.



REFERENCES

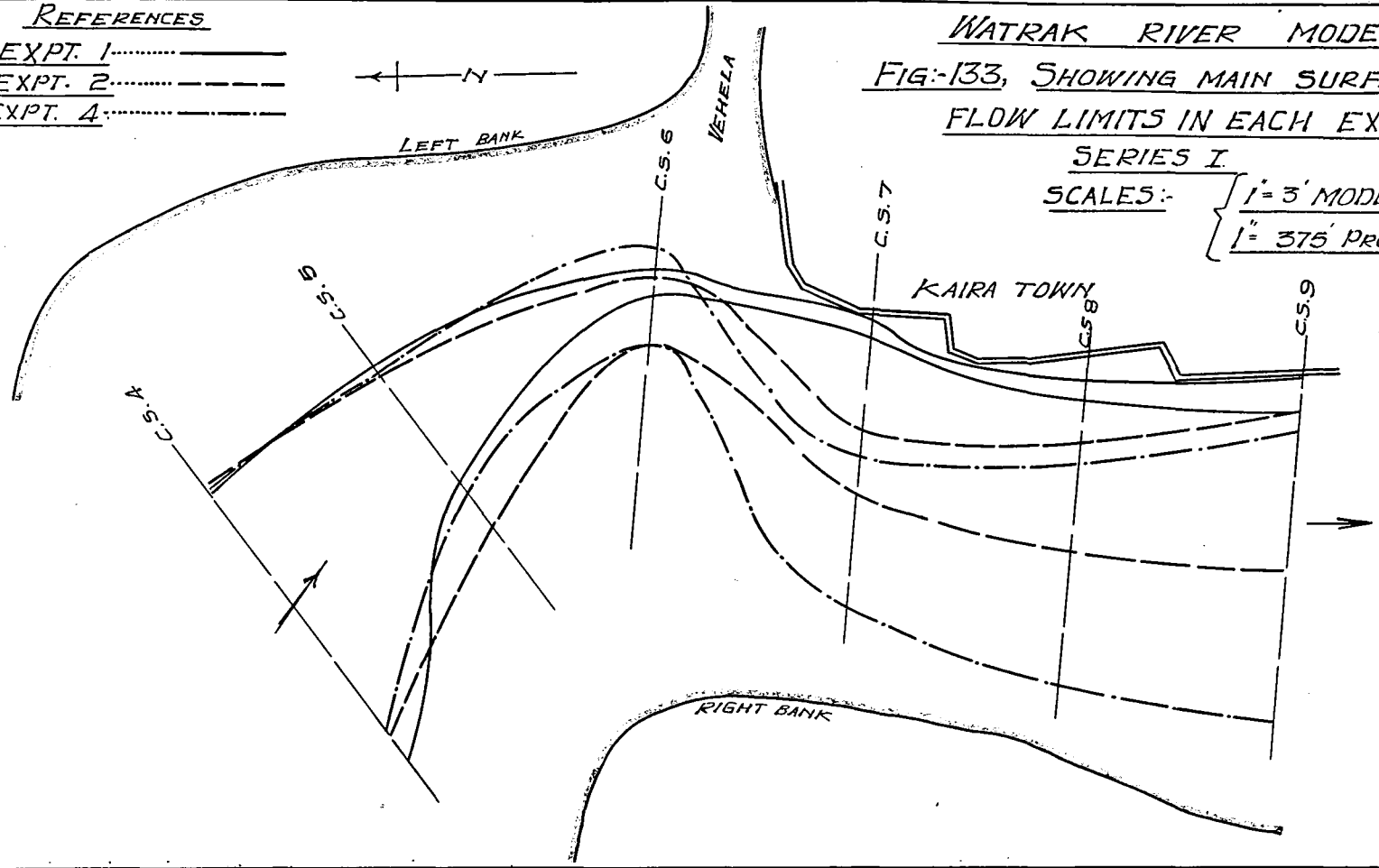
- EXPT. 1.....
- EXPT. 2.....
- EXPT. 4.....



WATRAK RIVER MODEL
FIG-133, SHOWING MAIN SURFACE
FLOW LIMITS IN EACH EXPT.

SERIES I

SCALES:- $\left\{ \begin{array}{l} 1" = 3' \text{ MODEL} \\ 1" = 375' \text{ PROTO} \end{array} \right.$

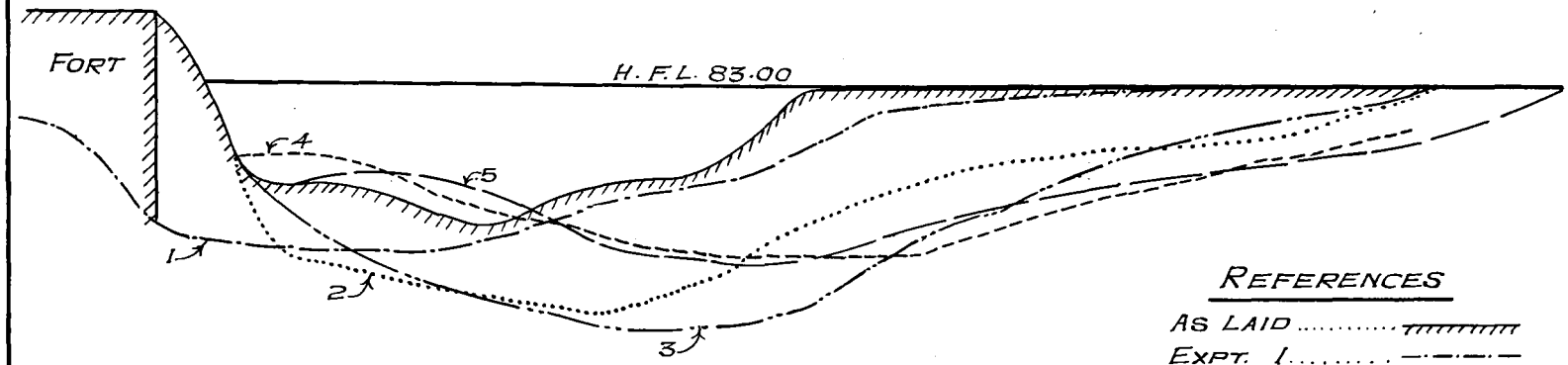


NO 1036
C.I. & H.R.S.

SERIES I. FIG. 134.
WATRAK RIVER MODEL.
SHOWING SCOUR IN VARIOUS EXPTS.

SCALES { HOR: = 1" = 1' MODEL = 125' PROTO.
 VER: = 1" = 1' MODEL = 25' PROTO.

C.S. 7.



REFERENCES

- AS LAID
- EXPT. 1
- EXPT. 2
- EXPT. 3
- EXPT. 4
- EXPT. 5

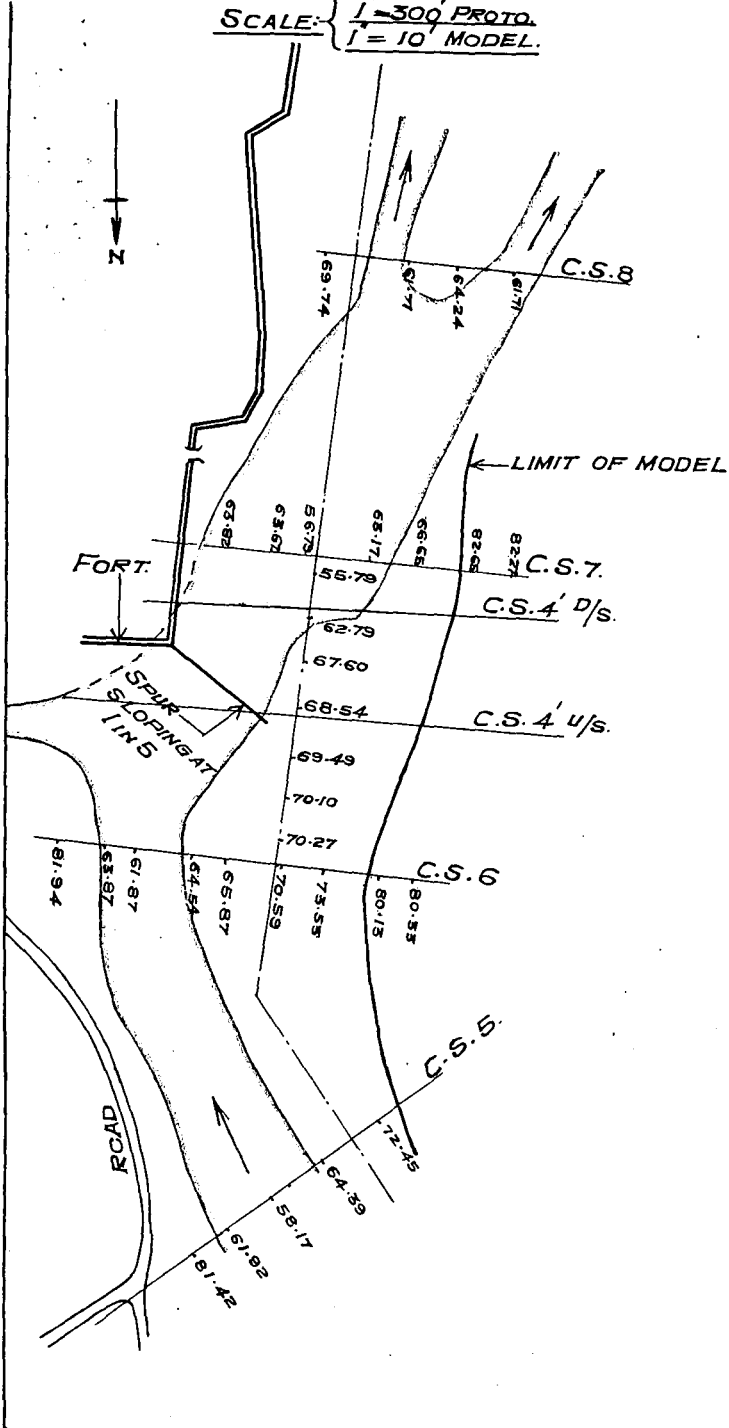
NO
 C. I. & H. R. S. M.

FIG:135, $\frac{1}{30}$ SCALE MODEL

No 1000
C.I. & H.R. STN.

OF WATRAK RIVER (SERIES II)
AT KAIRA. (PART WIDTH MODEL)

SCALE: $\left\{ \begin{array}{l} \frac{1}{4} = 300' \text{ PROTO.} \\ 1 = 10' \text{ MODEL.} \end{array} \right.$



that the main current approached too close to the fort wall before swinging around the spur, and there was deep scour at this point, so the experiment was stopped and the spur refixed pointing upstream at an angle of 35° .

The size and position of this spur are shown in figure 136 which also shows the lines of flow. With this design the main current was away from the wall upstream, the bed water was thrown across more than the surface flow, but crossed back downstream which was a satisfactory feature, as thereby silt was deposited near the wall and the main current kept away. The deepest scour occurred 210 feet from the corner of the wall and was 75 feet below water level, *i.e.*, at R. L. 8 compared to the water level at R. L. 83.

As, however, the spur would have to be constructed on wells sunk into the bed, there were practical difficulties in the way of continuing the slope beyond where it would meet present bed level. In experiment 3, the spur was therefore cut short with a vertical nose at 180 feet from the wall, where the slope met the present bed.

With this spur the lines of flow were satisfactory but the maximum scour, which was at the vertical nose, was greater than with the full length spur, being 85 feet below water level as compared to 75 feet.

With the idea of combining the good points of both designs, the spur used in the last experiment was extended by a level portion at R. L. 60, 60 feet long, in experiment 4. Due however to greater restriction of waterway, the maximum depth of scour with this was slightly greater being 89 feet below water level.

In experiment 5, mobile protection was laid using the design of spur of experiment 2 and laying loose brick blocks upstream and downstream as shown in figure 136 and photo 1669 on sheet 25. Brick was used as this could be made locally at low cost, whereas stone was expensive. The blocks corresponded in size to $12" \times 12" \times 8"$ in the prototype and were laid at the present bed level one block thick, 45 feet wide, along the upstream face of the spur and over a small area downstream where deep scour had occurred.

The discharge was gradually raised from 60,000 cusecs to 100,000 in one day. The block protection remained in place until 90,000 cusecs had been reached when scour began and the blocks gradually sank. At the end of the experiment, it was found that the point of maximum scour was 150 feet from the corner of the fort, compared with 210 feet in experiment 2, and that the scour hole was approximately of the same depth as before.

It thus appeared that mobile protection of this size and weight was ineffective in reducing scour.

Comparison was required with the Burma type spur which had proved most successful in the vertically exaggerated model, so this was tested under identical conditions in the geometrically similar model in experiment 6. This spur had vertical sides and was given a sloping nose, as shown in photo 1670—sheet 25—which also shows the scour pit formed by the maximum flood. This spur was effective in its purpose of protecting the Fort wall, but the depth of scour at its nose was greater than occurred with any of the straight sloping designs, *viz.*, 109 ft. below water level. A spur of the same design but with a vertical nose below present bed level (for ease of construction) was tested in experiment 7. The flow with this was similar to the other (experiment 6) but rather less satisfactory.

(8) *Discussion of Results of geometrically similar, part-with, model.*

The object of the spur was to hold the main current away from the corner of the Fort and to keep it away from the Fort wall downstream. All the designs, except the sloping spur at right angles to the wall (experiment 1), served this purpose. The short sloping spur tested in experiment 3 was however the best in this respect as was shown by cross sections 7 and 8 taken after each test (figure 137). Although the full length sloping spur (experiment 2) gave the least depth of scour the short spur would have to be only 10 feet deeper, would avoid practical difficulties and would cost less to build than any of the others.

Even the short spur of experiment 3 would, however, be very costly to build owing to the great depth of scour. The possibility of combining the river training work with a high level causeway or bridge, in order to save on the combined cost, is now being explored by means of further model experiments.

[Table No. 18 shows brief description of various experiments.]

TABLE NO. 18.

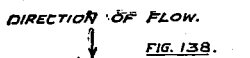

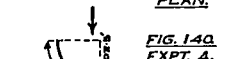
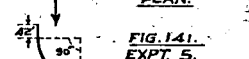
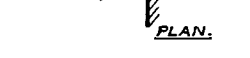
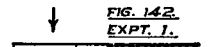
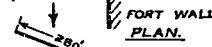

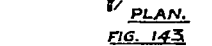
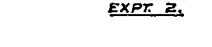

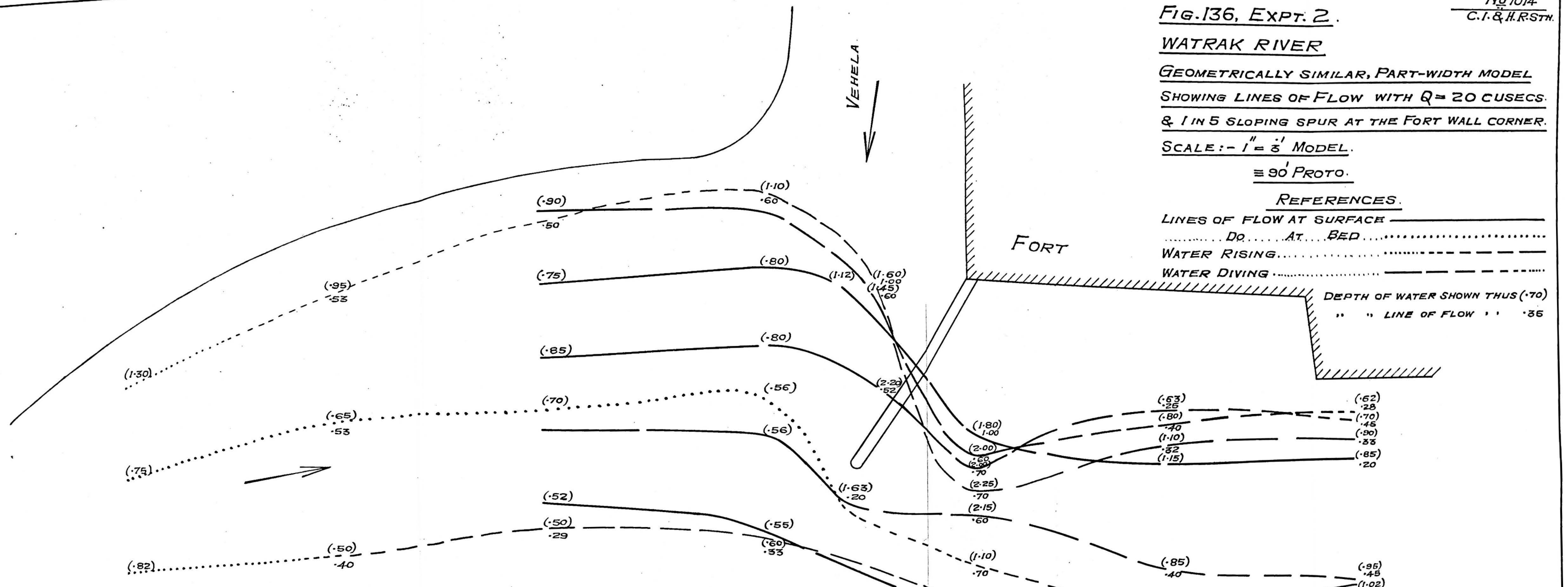
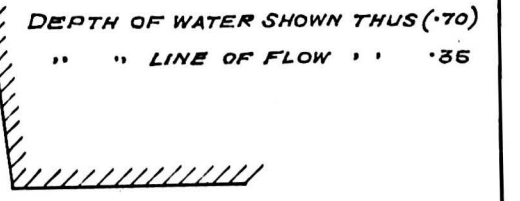
Model.	Experiment No.	Description of the experiment.	Figure.
<p><i>Series I.</i></p> <p>Vertically exaggerated model.</p> <p>Horizontal scale 1/125.</p> <p>Vertical scale 1/25.</p>	1	Present conditions.	 <p>DIRECTION OF FLOW.</p> <p>FIG. 138. EXPT. 2.</p> <p>R = 48'</p> <p>125'</p> <p>FORT WALL</p> <p>PLAN.</p>
	2	Burma spur at the fort corner, 187' long, radius of curvature of head 48'.	 <p>FIG. 139. EXPT. 3.</p> <p>125'</p> <p>FORT WALL</p> <p>PLAN.</p>
	3	Longer Burma Spur, radius of curvature 48'.	 <p>FIG. 140. EXPT. 4.</p> <p>125'</p> <p>FORT WALL</p> <p>PLAN.</p>
	4	Burma spur, radius of curvature 102.5 ft.	 <p>FIG. 141. EXPT. 5.</p> <p>125'</p> <p>FORT WALL</p> <p>PLAN.</p>
	5	Same as 4 but the nose was extended by 42'.	 <p>SPUR</p> <p>125'</p> <p>FORT WALL</p> <p>PLAN.</p>
<p><i>Series II.</i></p> <p>Geometrically similar model.</p>	1	Sloping spur in the line of the fort wall.	 <p>FIG. 142. EXPT. 1.</p> <p>280'</p> <p>FORT WALL</p> <p>PLAN.</p>
	2	Sloping spur pointing upstream at an angle of 35°.	 <p>280'</p> <p>35°</p> <p>FORT WALL</p> <p>PLAN.</p>
	3	Sloping spur cut short with vertical nose at 180' from the wall.	 <p>180'</p> <p>FORT WALL</p> <p>PLAN.</p>
	4	Spur in experiment 3 was extended by a level portion, 60 feet long at present bed level.	 <p>60'</p> <p>FORT WALL</p> <p>PLAN.</p>
	5	Spur as in experiment 2 but with mobile protection of loose brick blocks on upstream and downstream of the spur.	 <p>PLAN.</p>
	6	Burma spur—same as in experiment 4 in vertically exaggerated model, but with sloping nose.	 <p>FIG. 143. EXPT. 2.</p>
	7	Burma spur as in experiment 6, but with a vertical nose below present bed level.	<p>SCALE: 1" = 3' IN MODEL. = 375' IN PROTO.</p>

FIG. 136, EXPT. 2.

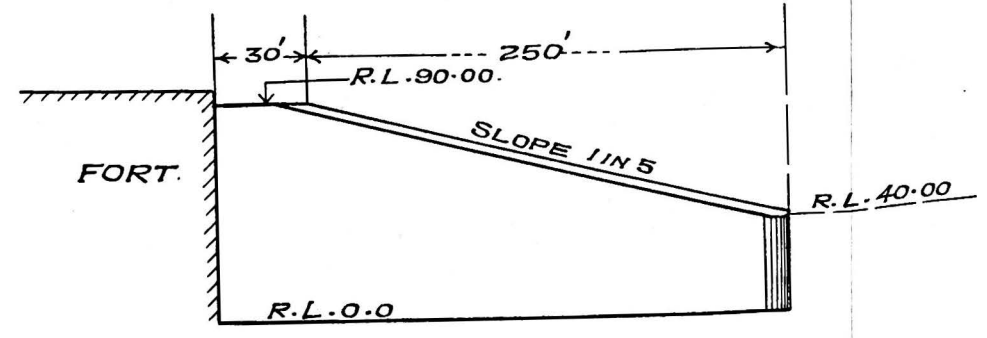
WATRAK RIVER

GEOMETRICALLY SIMILAR, PART-WIDTH MODEL
SHOWING LINES OF FLOW WITH Q = 20 CUSECS.
& 1 IN 5 SLOPING SPUR AT THE FORT WALL CORNER.
SCALE: - 1" = 3' MODEL.
= 90' PROTO.

REFERENCES.
LINES OF FLOW AT SURFACE —————
..... DO..... AT..... BED.....
WATER RISING.....
WATER DIVING.....

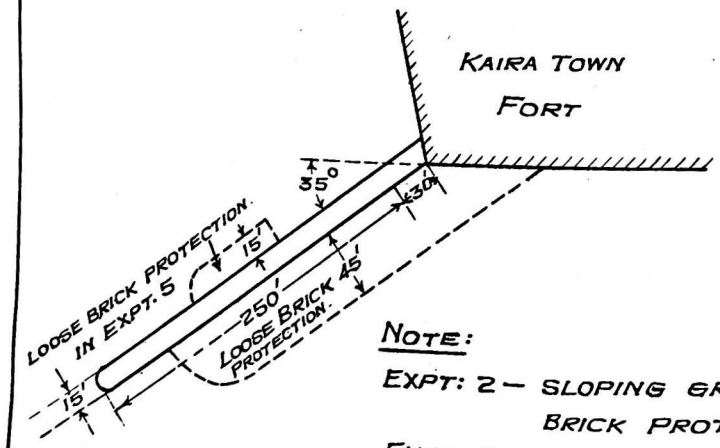


ELEVATION OF SPUR
(LOOKING DOWNSTREAM)



PLAN.

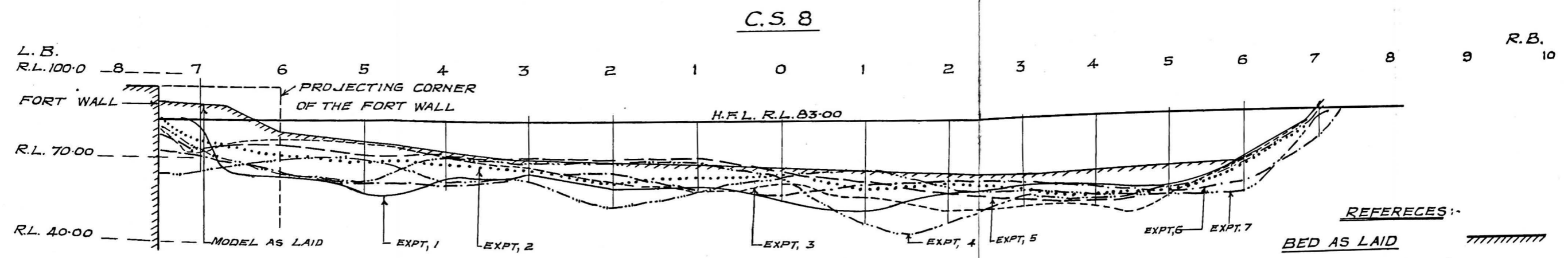
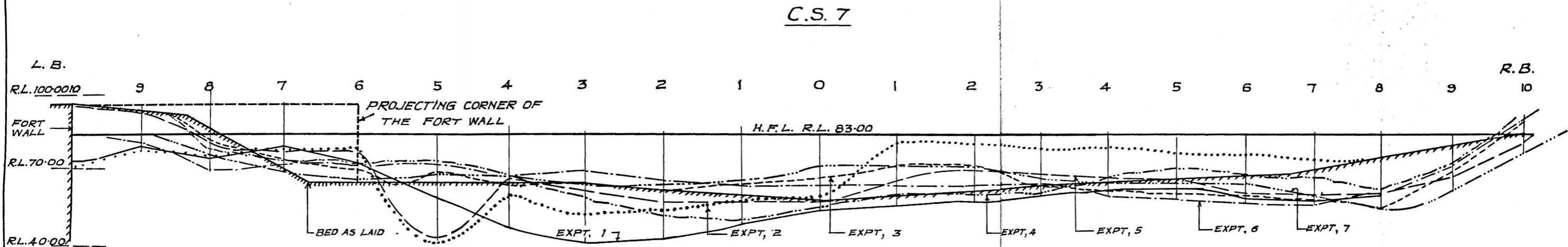
KAIRA TOWN
FORT



NOTE:

EXPT: 2 - SLOPING GROUYNE WITHOUT
BRICK PROTECTION.
EXPT. 5 - SLOPING GROUYNE WITH LOOSE BRICK
PROTECTION AS SHOWN IN DOTTED LINES.

FIG: 137,
WATRAK RIVER EXPT.,
GEOMETRICALLY SIMILAR, PART WIDTH,
MODEL $\frac{1}{30}$ SCALE. CROSS SECTIONS 7 & 8
TAKEN AFTER EACH EXPT.,
SCALE 1" = 1' (MODEL) = 30' (PROTO)



REFERENCES:-

BED AS LAID	TTTTTTTTT
BED AFTER EXPT., 1	—————
" " " 2
" " " 3	-----
" " " 4	-----
" " " 5	-----
" " " 6	-----
" " " 7	-----

PART IV.

Advisory—without experiments.

25. INTRODUCTION.

During the year, several problems were submitted for advice: either because they were urgent or because earlier model experiments had indicated their solution.

Some of these cases were disposed of at once; some are under correspondence; but in most cases we were able to recommend immediate remedies; and it was suggested that results of these should be observed before further action was taken.

In a couple of cases, model experiments were proposed to supplement previous experience.

Requests for advice are increasing and one of the chief functions of a Central Research Station should be to advise promptly on urgent problems—whether this be followed by experiments or not.

26. DESIGN FOR STANDING WAVE FLUMES FOR THE JODHPUR WATER SUPPLY CHANNEL.

At the request of Mr. Edgar, Public Works Minister, Government of Jodhpur two standing wave flumes were designed: one for 30 cusecs, at the head of the channel, and one for 20 cusecs, at the tail of the channel. Figure 144 shows the design supplied for the 20 cusecs standing wave flume.

$$Q = B_2 D_2 V_2 = B_2 C(D_1 + V_1^2/50)^n \quad (1)$$

where B_2 = throat width (= 2.75' for 20 cs. S. W. F. and 3.0' for 30 cs. S. W. F.)

C = a coefficient

D_1 = upstream depth over sill of throat

V_1 = mean velocity of approach

and n = an exponent (approximating 1.5).

The formulæ worked out to

$$Q = 8.25 D_1^{1.49} \text{ for 20 cs. S. W. F.} \quad (2)$$

$$\text{and } Q = 9.0 D_1^{1.49} \text{ for 30 cs. S. W. F.} \quad (3)$$

Where D = effective depth, taking velocity of approach into account = $(D_1 + V_1^2/50)$

When stated in terms of D_1 —the actual depth over sill—

$$D = 1.02 D_1^{1.003} \quad (4)$$

$$\text{or } D_1 = 0.98 D^{0.97}$$

from which

$$Q = 8.498 D_1^{1.491} \text{ for 20 cs. S. W. F.} \quad (5)$$

$$\text{and } = 9.270 D_1^{1.491} \text{ for 30 cs. S. W. F.} \quad (6)$$

If the exponent is assumed = 1.5, which is convenient,

$$Q = 8.47 D_1^{1.5} \text{ for 20 cs. S. W. F.} \quad (7)$$

$$\text{and } = 9.24 D_1^{1.5} \text{ for 30 cs. S. W. F.} \quad (8)$$

with an error of ± 1 per cent. between F. S. Q. and $2/3$ F. S. Q., which is negligible; and hence these formulæ may be adopted.

27. TRAINING THE NALA PASSING THROUGH AQUEDUCT 14 OF THE NIRA RIGHT BANK CANAL.

The flow of the Nala upstream of Aqueduct 14 is sharply oblique, and as a result, during the 1937 flood, the flood level rose 2 feet above the designed H. F. L.; and the protective wall on the east end of the aqueduct was damaged. Neither a straight cut, nor training by groynes is feasible, so it has been proposed that a Bell-bund extending 160 feet upstream (equal to the length of the aqueduct),

finishing off with a 120° curved head of radius equal to 35 feet, should be constructed to prevent scour of the canal bank and to turn the nala water into the aqueduct. With this design, the scour just inside the entrance of the approach may be as much as 21 feet if the bed material is sand—assuming the maximum discharge to be 20,000 cusecs, as estimated.

28. EXCLUSION OF EXCESS SILT FROM KATTALAI HIGH LEVEL CHANNEL—*ex-SOUTH BANK CANAL (MADRAS PRESIDENCY)*.

The Kattalai High Level channel takes off from the right bank of the South Bank Canal just above the Regulators at 0/5 (see figure 145). The sill level of the Regulator is at R. L. 313.75.

The Kattalai channel was opened on 19th July 1934. Its Head Regulator with sill level at R. L. 315.25 was set back 270 feet from the South Bank Canal.

As was to be expected—bearing in mind that 35 per cent. of the discharge was drawn off at an angle of 110° (see Bombay P. W. D. Tech. Paper 52⁽¹⁴⁾), curved flow was generated and bed silt concentrated in the Kattalai channel.

With the idea of preventing this, curved vanes were constructed in the South Bank Channel in 1935 to divert bed silt down the main South Bank Channel and a sill with R. L. at 315.25 was built across the entrance to the Kattalai channel (see figure 145). These had little effect and the F. S. Q. decreased from 411 cs. to 280 cs. with F. S. D.=4 feet and a silt bank formed which was 1.8 feet high at the head and 6" high at 1 mile downstream.

Remedies suggested.—Vanes being quite unsuitable for such conditions, a divide wall projecting upstream from the S. B. C. regulators to a distance a little more than the still pond pocket width above the upper end of the entrance to the H. L. channel has been recommended (see red line in figure 145).

F. S. Q. of S. B. C. upstream of offtake=1,200 cusecs.

F. S. Q. of H. L. C.=411 cusecs.

So the F. S. Q. ratio=800 : 410.

In order that the velocity factor in the approach channel of the Kattalai channel should equal that in S. B. C., the divide wall should divide the channel into $\frac{4}{7}$ ths and $\frac{3}{7}$ ths and as there are 7 spans, 3 spans will be for the Kattalai channel and 4 spans for the main channel.

The 3 spans of the approach channel will normally remain closed and the arrangement will act as a "still-pond silt excluder"; but if it is found desirable to scour silt from the approach channel, the three spans may be opened occasionally. While this is being done, the Kattalai regulator must be kept closed. During scouring, the water level on the two sides of the divide wall will be different, and provision should be made for a difference of 2½ feet when considering stability.

If the division of silt is not exactly right, the proportion of silt can be subsequently modified by extending the divide wall at 1 in 5 upstream, so as to increase the entrance width of the channel in which it is desirable to reduce silt:—quite a short extension will have a big effect.

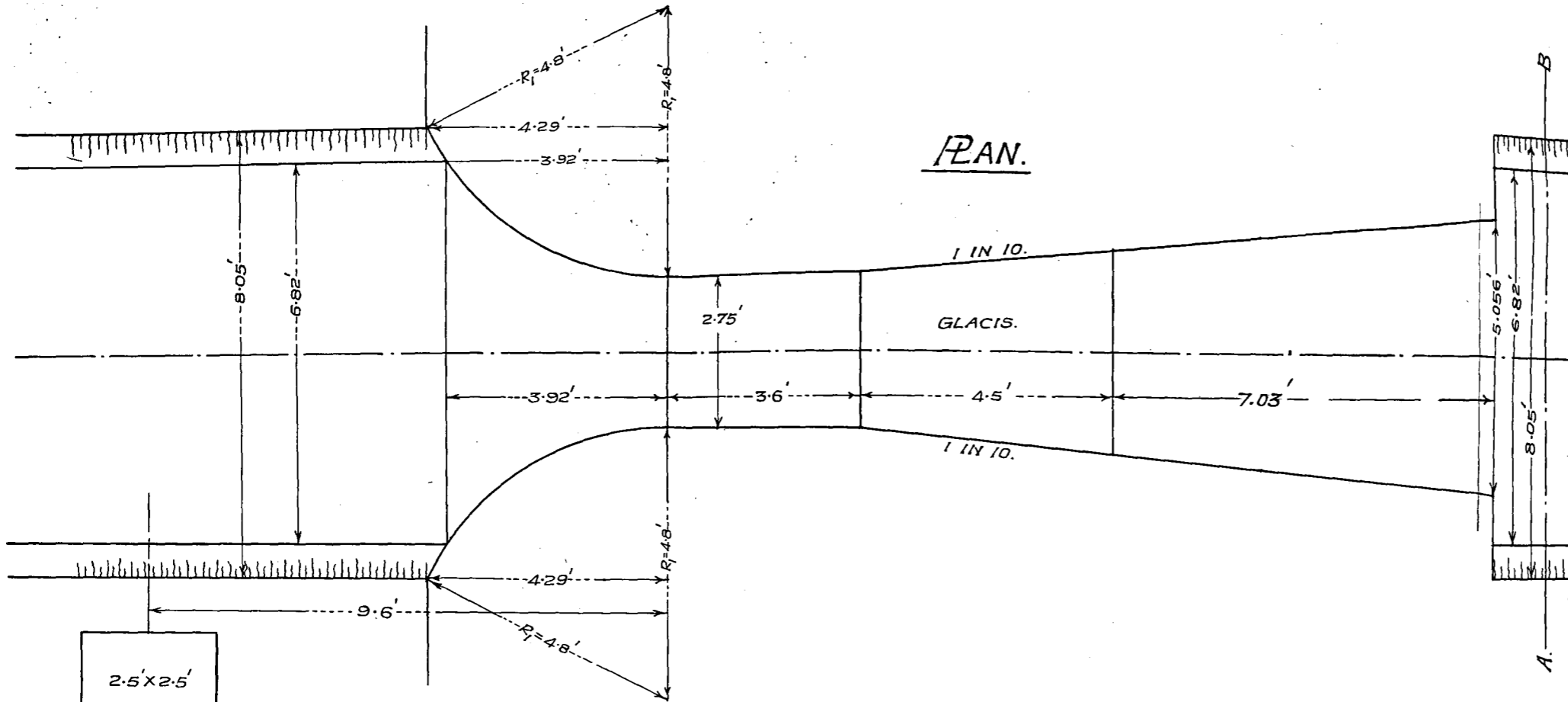
29. MEASURES TO PREVENT SILT ENTERING THE VADAVAR CHANNEL OF THE LOWER COLEROON ANICUT SYSTEM (MADRAS PRESIDENCY).

The Vadavar Canal takes off from the River Coleroon at the Lower Anicut, in Tanjore District—*vide* figure 146. The Coleroon branches off from the River Cauvery at the upper Anicut about 9 miles upstream of Trichonopoly. The Lower Anicut is about 70 miles downstream of the upper Anicut and consists of two regulators. The north regulator has 30 vents and the south regulator 40.

At the left end of the North Branch, two Canals, Vadavar and the North Rajan Vaikal take off. There is no silt trouble in the North Rajan; but the Vadavar has silted heavily. The Vadavar was widened in 1927 from 100 feet to 120 feet to discharge 2,200 cusecs with 8.5 feet on the rear gauge of the head sluice. The discharge observed with this gauge was 2,030 cusecs at the end of 1927. The discharge gradually reduced to 1,810 cusecs in 1931 and to 1,768 cusecs in 1936. The depth of silt on the Canal bed at the head of the Canal in 1937 was about 4 feet. Figure 147 shows the site survey of the Canals.

(14) Bombay P. W. D. Technical Paper 52 :

"Factors affecting exclusion of bed silt from Canals taking off from alluvial Rivers" by C. C. Inglis and D. V. Joglekar.



PLAN.

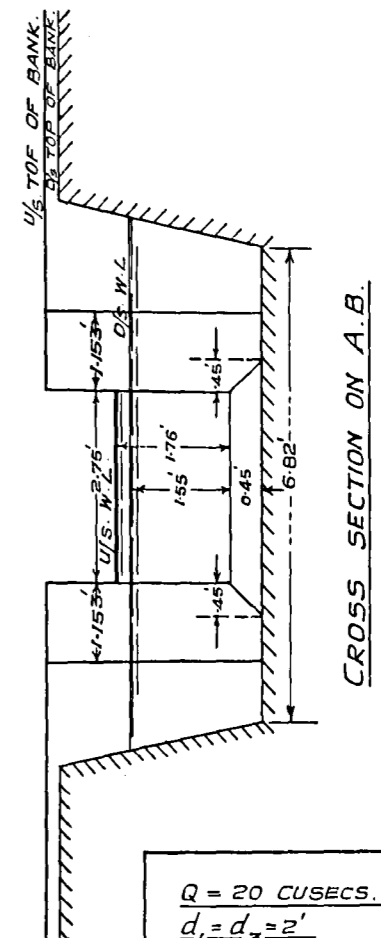
1 IN 10.

GLACIS.

1 IN 10.

2.5' x 2.5'
GAUGE
CHAMBER.

LONGITUDINAL SECTION.



CROSS SECTION ON A.B.

$Q = 20$ CUSECS.
 $d_1 = d_3 = 2'$
 HEIGHT OF HUMP ABOVE U/S BED = $0.24'$
 HEIGHT OF HUMP ABOVE D/S BED = $0.45'$
 AFFLUX = $0.21'$
 $B_2 = 2.75'$
 $D_1 = 1.76'$, $D_3 = 1.55'$ & $D = 1.00'$
 LENGTH OF THROAT = $3.6'$

DESIGN PREPARED BY C.I.&H.R.S.

OF

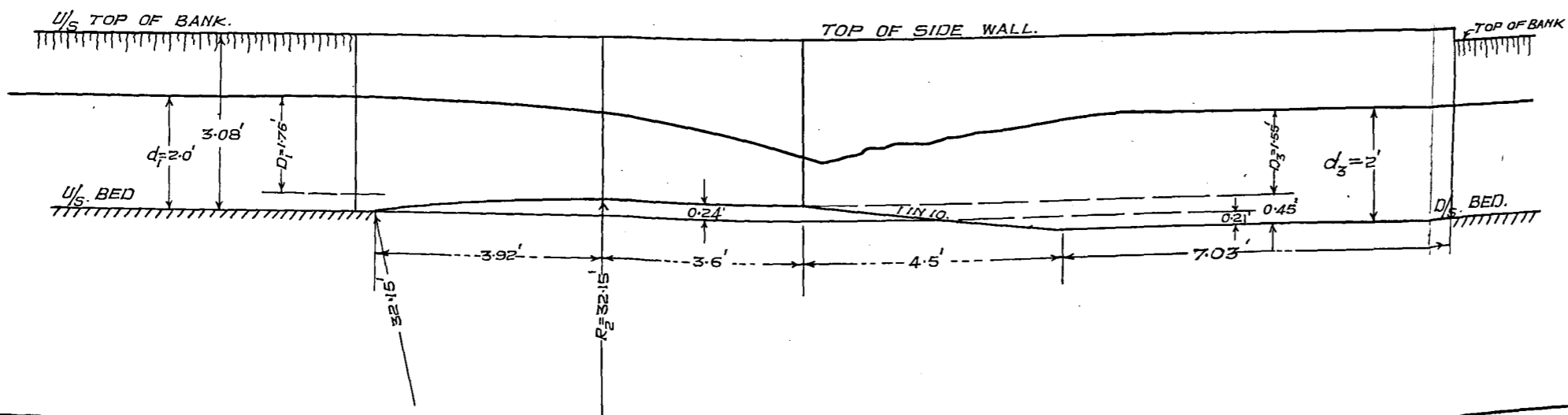
S. W. FLUME

FOR

JODHPUR CHANNEL.

$Q = 20$ CUSECS

SCALE: $1'' = 2'$



S. P. Z. O. P. O. N. A. 1. 1353

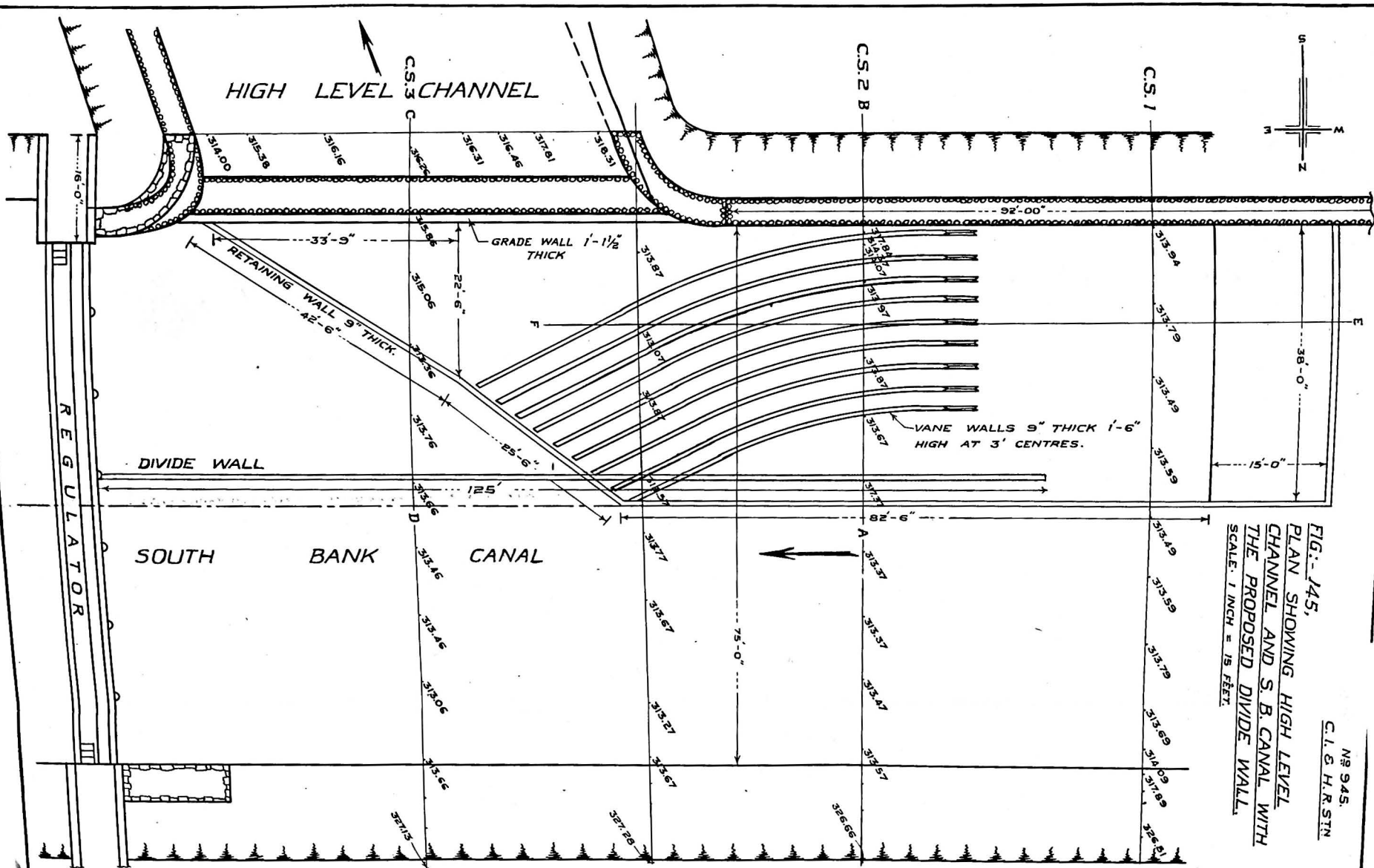


FIG. 145,
 PLAN SHOWING HIGH LEVEL
 CHANNEL AND S. B. CANAL WITH
 THE PROPOSED DIVIDE WALL.
 SCALE: 1 INCH = 15 FEET.

NO 945
 C. I. & H. R. STIN

FIG: 146,
PLAN OF COLEROON RIVER
NEAR THE LOWER ANICUT.

SCALE 32 FEET = 1 MILE.

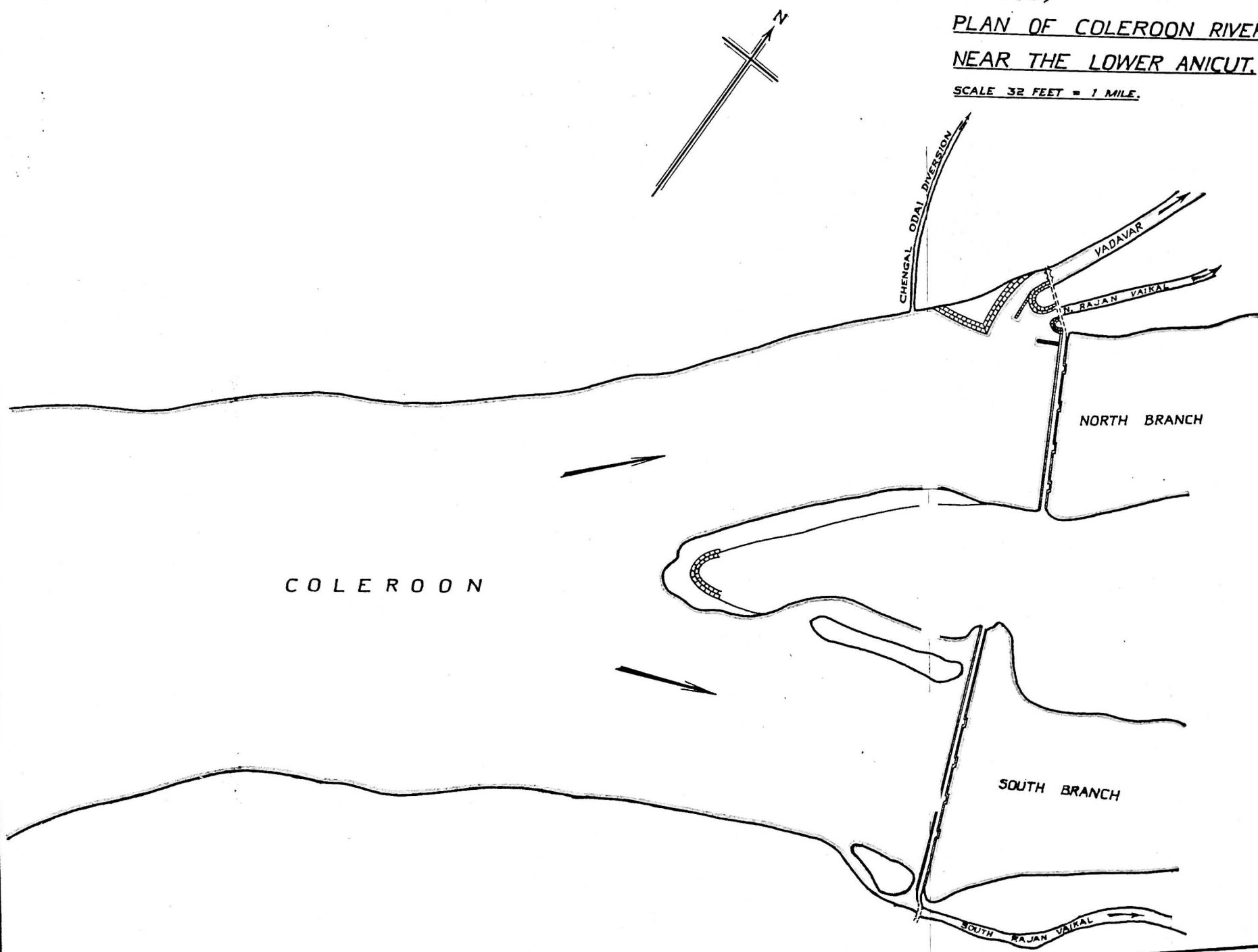
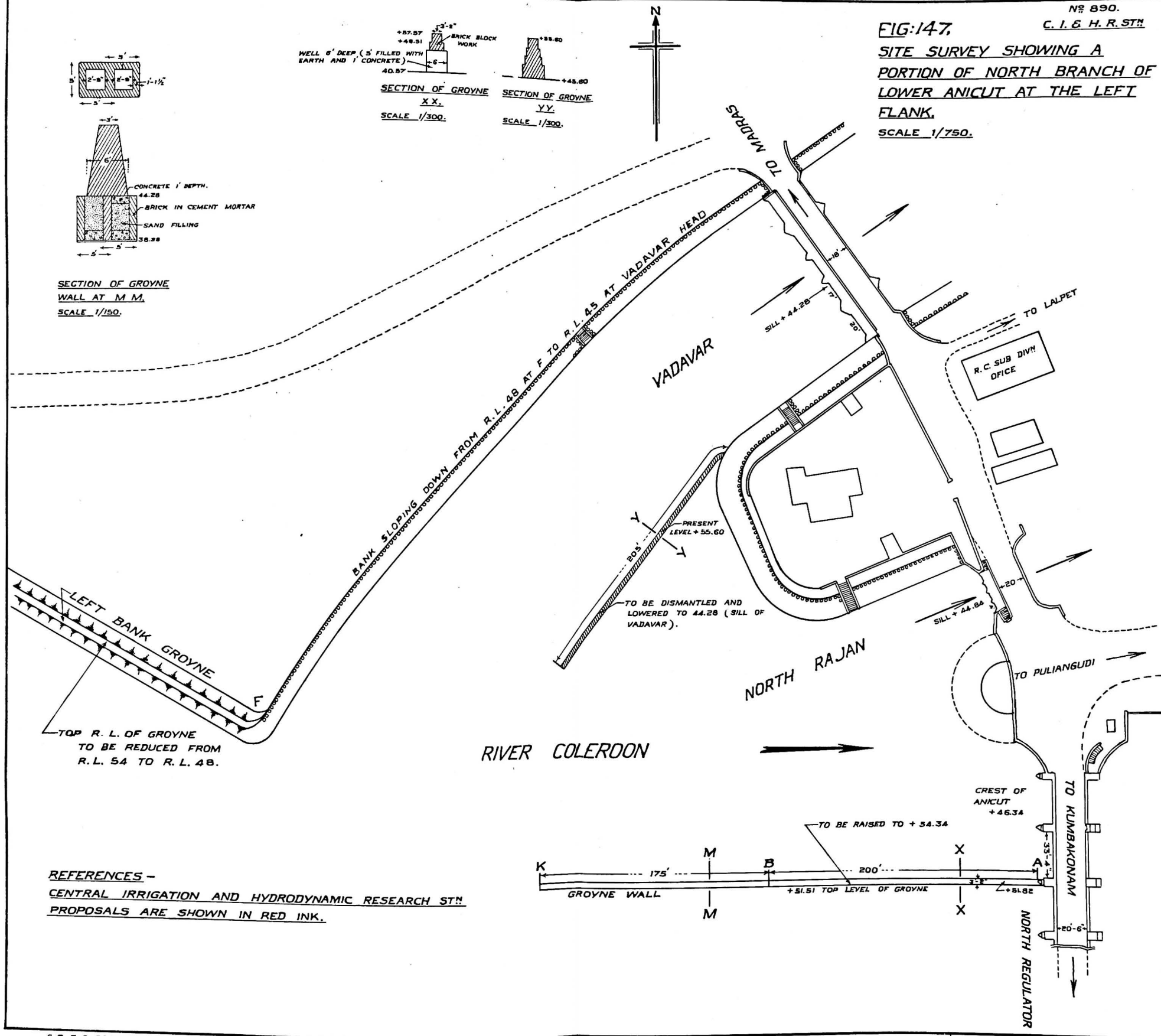


FIG:147.
SITE SURVEY SHOWING A
PORTION OF NORTH BRANCH OF
LOWER ANICUT AT THE LEFT
FLANK.
SCALE 1/750.



The cause of the trouble, so far as the Vadavar is concerned, is the main groyne running out from the left bank ; because it drives the top water outwards which causes the bed water to be deflected inwards. This is because top water, flowing with a high velocity, tends to flow straight on past the entrance of the Canal ; and, as some water has to pass into the Canal, it is the slow moving, easily deflected, bed water which does so—*vide* Bombay P. W. D. Technical Paper 52⁽¹⁴⁾.

The recommendation outlined, subject to modification in the light of further data, was to remove the left upstream groyne down to R. L. 48, and the bank from F to the Head regulator of the Vadavar down to R. L. 45, all the intervening soil being removed to give a slope from 48 down to 45. The top water would then flow over this into the Vadavar. The effect of this would be to deflect silt from the Vadavar Canal, which would tend to pass into the North Rajan Canal. To counteract this, the first two spans of the North regulator should normally be kept closed—between A and the North Rajan Channel—; and the adjoining spans of the North regulator kept fully or partially open, according to the supply available. A divide wall extending upstream to about K, on the line AB extended, would further improve matters. The top of this wall or groyne should be above the water level at such times as it is desired to exclude bed silt. The groyne should be designed to withstand at least 6 feet difference of water level.

Assuming these recommendations are acted on, it would have to be decided whether the silt already accumulated in the Vadavar should be removed or allowed to scour, to be redeposited downstream. It is better to remove the silt

(i) to get full supply at once,

and (ii) to prevent the silt now accumulated at the head raising the bed level further down the Canal, which might necessitate raising the banks. If, however, the banks downstream are so high that silting would not matter, only a small amount of silt need be removed. Further data is awaited. A model experiment is desirable, though not essential in this case.

30. CONTROLLING EROSION OF THE RIVER "BUR GANDAK" AT SIKANDARPUR IN MUZAFFARPUR DIVISION (BIHAR).

The Chief Engineer, P. W. D., Patna (Bihar) sent papers and a plan in connection with erosion of the River Bur Gandak, for remarks on the general hydrodynamic problem. Figure 148 shows the survey of the River from Daudpur to Chanwora Muzaffarpur (1921-27). In this case, scour has hitherto been resisted by protective works at the points of attack. It is a physical fact that spurs, pitched banks, and walls have a very strong attraction for water and cause an intensification of attack ; so that unless such works are enormously strong, it is generally only a question of time till they are undermined and washed away. Spurs and other protections are therefore liable, in the course of time, to do more harm than good. In this case the immediate trouble could be overcome by making cuts as shown in red lines in figure 148. It appears from the plan that cut-offs have already been contemplated both here and across the peninsula projecting towards Chandawar ; but in both cases the point of off-take shown would be unsuitable, and locating an off-take properly is of supreme importance. Such cuts would be expensive and might lead to damage upstream, due to increased slopes ; but there does not appear to be any simple way of protecting the banks where they are now being attacked (at the outside of sharp curves) which would not, considered over a long period of years, be prohibitively costly. It has been thought that wave action was partly responsible for the erosion. This is not the case. Erosion always starts at the toe of pitching, which then slides or collapses into the hole thus formed, and wave wash has a negligible effect. What actually happens is that water eroding a bank—especially if it is impinging at right angles—dives down and creates curved, unstable, surging, flow at the bed. This creates pulsating vortices, which wash away even large stones from the toe of the slope. As regards the statement :

“ the river bed is full of old trees fallen in past years and these are to some extent responsible for the swinging of the channel ”.

This is undoubtedly an important factor, and some years ago a writer in the American News Record stated that after stumps of trees had been removed from the Missouri River, it decreased in length by 100 miles over a length of about 400 miles.

31. EXCLUSION OF BED SILT FROM THE JAMRAO CANAL (SIND).

The bed silt entering the Jamrao Canal has increased and a model is to be constructed at the Karachi Testing Station to ascertain the changes required to exclude excess silt. Suggestions were made as regards design of the model and experiments, based on experience obtained in Poona.

32. DISSIPATING ENERGY BELOW THE MASONRY WASTE WEIR OF THE TIMBA TANK AT HARSOL, TALUKA PARANTIJ, DIST. AHMEDABAD.

The Timba Tank is an old tank, which was improved in 1923. The waste weir is 300 feet long with crest at R. L. 107.0 and a discharging capacity of 5,300 cusecs. In 1923, a water cushion, 4 feet wide, was provided downstream of the weir, with bed R. L. = 102.3. Scour was first noticed after the great flood of 1927, when scour downstream of the water cushion was filled with soil, which was covered with a layer of rubble. Slight scour occurred in 1931 and 1933; but in 1937, the bed downstream of the cushion chamber scoured to a depth of 6 to 7 feet over a width of 75 feet.

The upper 6" to 2' layer of the bed consists of black soil; below this there is 2' to 6' of soft murum and below that hard murum mixed with lime kankar. The original bed of the waste weir channel was excavated down to R. L. 104.0 with a slope of 1 in 200 for about 600 feet, to where it met the natural ground level. At its 18th chain it tails into an adjoining nala which in turn falls into the Luni River. The distance between the waste weir and the Luni River is 4,700 feet, with a steep fall of 1 in 250. This steep fall creates a high velocity which has led to about 2 feet depth of scour in the murum and kankar bed of the channel.

Remedies suggested.—Figure 149 shows the action suggested by local officers and figure 150 shows our recommendations.

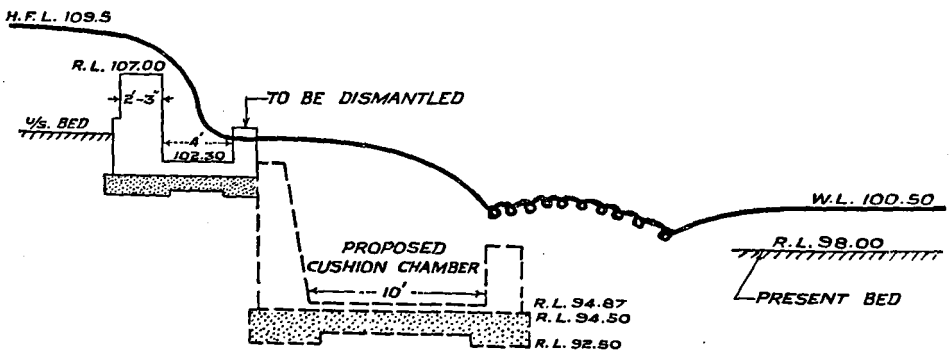
FIG. 149. SHOWING NEW CUSHION CHAMBER

PROPOSED BY LOCAL OFFICERS

BELOW WASTE WEIR OF TIMBA TANK

AT HARSOL.

SCALE 1 INCH = 8 FT



PROPOSED WORK SHOWN IN DOTTED.

EXISTING WORK SHOWN IN FULL.

The natural slope of a channel depends on the discharge and material of the channel bed. In regime channels, Lacey has shown that

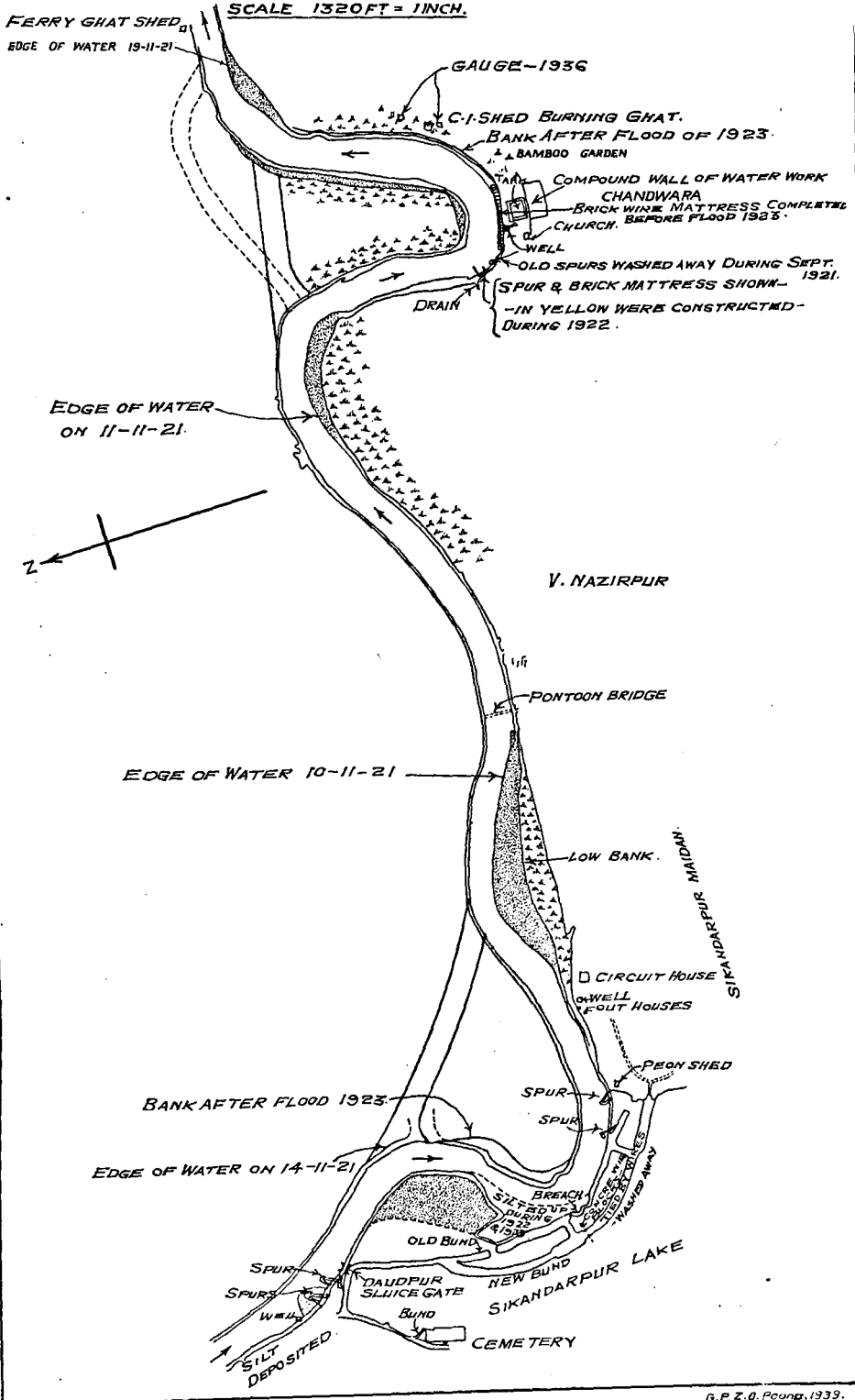
$$\text{Slope} = \frac{54 \times f^{5/3}}{10^6 \times Q^{1/6}}$$

and though the slope is somewhat steeper with fluctuating discharges, $S \propto \frac{f^{5/3}}{Q^{1/6}}$

SURVEY MAP OF RIVER BUR
GANDAK.
FROM DAUDPUR TO CHANDWARA
1921 - 1927.

Figure-148.

SCALE 1320 FT = 1 INCH.



holds. When a dam is built, the silt is wholly trapped and so scour increases downstream. The problem to be solved, therefore, is how best to counteract progressive downstream scour, due to clear water,—which is an entirely different problem to dissipating energy below the weir. There are two alternatives:—

- (i) A continuous rapid or lined flume,
- or (ii) a series of falls; only the first fall required at present.
- (i) would be unnecessarily expensive in first cost;
- (ii) has the advantage that it can be built by stages, as required, and the work may be stopped, when scour stops.

As a first step, it has been proposed that a single baffle fall—see figure 150—should be constructed. When scour below this baffle increases to the stage at which undermining is threatened, a simple weir fall may be added downstream, etc. The only essential at this stage is to see that the drop wall under the baffle is designed as the main wall of a 4 feet weir fall to be constructed later when required. This is provided for in the design in figure 150.

NOTE.—Figure 150 is a hydraulic diagram and does not show details of construction of glacis—which will depend on bed materials below the dam.

FIG. 150, SHOWING BAFFLE FALL WITH A GLACIS
D/S. OF WASTE WEIR OF TIMBA TANK
AT HARSOL.
SCALE 1 INCH = 8 FT.

