

Government of India

CENTRAL IRRIGATION AND HYDRODYNAMIC RESEARCH STATION, POONA

Director :
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RESEARCH PUBLICATION No. 2
ANNUAL REPORT OF WORK DONE
DURING 1938-39

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PUBLISHED BY MANAGER OF PUBLICATIONS, DELHI
PRINTED BY MANAGER, GOVERNMENT OF INDIA PRESS, CALCUTTA
1940

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Annual Report of work done during the year 1938-39.

INTRODUCTION.

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(from Oct. 1938).

Mr. V. K. Rajadhyaksha, B.Sc., B.E.
(from Oct. 1938).

Mr. C. C. Inglis proceeded on leave on the 24th February 1939, from which date Mr. A. R. Thomas became officiating Director.

Publications.

The following was published during the year, as Bombay Public Works Department Technical Paper No. 59 :—

“Silt control at Heads of Canals and Distributaries” By C. C. Inglis, C.I.E., and Rao Sahib D. V. Joglekar.

39 Notes on specific experiments and problems were issued and 26 Notes on basic subjects (mainly for the use of the staff).

Model experiments and Enquiries.

During the year, 19 investigations were in progress with the aid of model experiments, 9 being of specific character and 10 of basic character. In addition, 9 enquiries were dealt with without experiments.

The following experiments were in progress during the year, but owing to urgency of other experiments insufficient work was done for inclusion in the present report. They are being continued in 1939-40 :—

- (i) Standardisation of design of high coefficient weirs.
- (ii) Experiments with Head Regulators to ascertain factors causing loss of head.
- (iii) Scale effects of rigid structures in geometrically similar and vertically exaggerated models.

Figure 1 shows, on a map of India, the places in connection with which experiments were done and enquiries dealt with.

Programme of experimental work for the year 1939-40.

The following is a provisional list of experiments with models to be carried out :—

I. SPECIFIC EXPERIMENTS.

Experiments on behalf of the Railway Board.

1. With 1/500 scale model of the Ganges and the Hardinge Bridge for investigating the movements of the River (continued).

2. With models of the piers of the Hardinge Bridge to investigate scour and the behaviour of stone protection (continued).

Experiments for the Central Government.

3. Training of the Jumna River near Delhi Gate Pumping Station (continued).

4. Protection of the proposed Road between the Delhi Gate Pumping Station and the Okhla weir by means of spurs.

Experiments for the United Provinces Government.

5. Training the Sarda River at Banbassa and protecting of the Right Afflux Bund (continued).

6. Methods of preventing scour and protecting the floor downstream of the Sarda Barrage.

Experiments for the Government of Sind.

7. With 1/300 scale model of the Indus River above the Sukkur Barrage to find the effect of river curvature and proportion of discharge in two gorges on silt charge, and experiments with the proposed new right bank approach channel designed to exclude silt from the Right Bank Canals (continued).

8. With 1/150 scale model of the Indus River above the Sukkur Barrage for obtaining additional data on the above and observing the effect of model scales on results (continued).

9. With 1/40 scale, geometrically similar, part-width model of the Indus River above the Sukkur Barrage and the proposed new Right Bank Approach Channel to compare results of the geometrically similar model and vertically exaggerated models in (7) and (8) and to test alternative designs of banks and raised sill and observe scour.

Experiments for the Government of Bengal.

10. Scour round piers of the proposed bridge at Burdwan across the Damodar River and stone protection round piers.

Experiments for the Central Provinces Government.

11. Factors affecting stability of the decking slabs of submersible bridges (continued).

Experiments for the Orissa Government.

12. With a model of the Mahanadi River on flood control by embankments.

13. With a model of the Brahmani River on flood control by embankments.

Experiments for the Government of Bombay.

14. To prevent jetting downstream of Aqueduct No. 3 of the Girna Canal.

II.—BASIC EXPERIMENTS.

15. Investigating bed movement with various types of sand, and silt, under controlled conditions (continued).

16. Laboratory experiments to determine factors affecting seepage from Canals (continued).

17. Dissipation of energy below falls (continued).

18. Standardisation of the design of S. W. Flumes.

19. Rate of abrasion of silt (continued).

20. Scale effects of rigid structures in geometrically similar and vertically exaggerated models (continued).

21. With various types of spurs to see the effect on bed scour and throw-off.

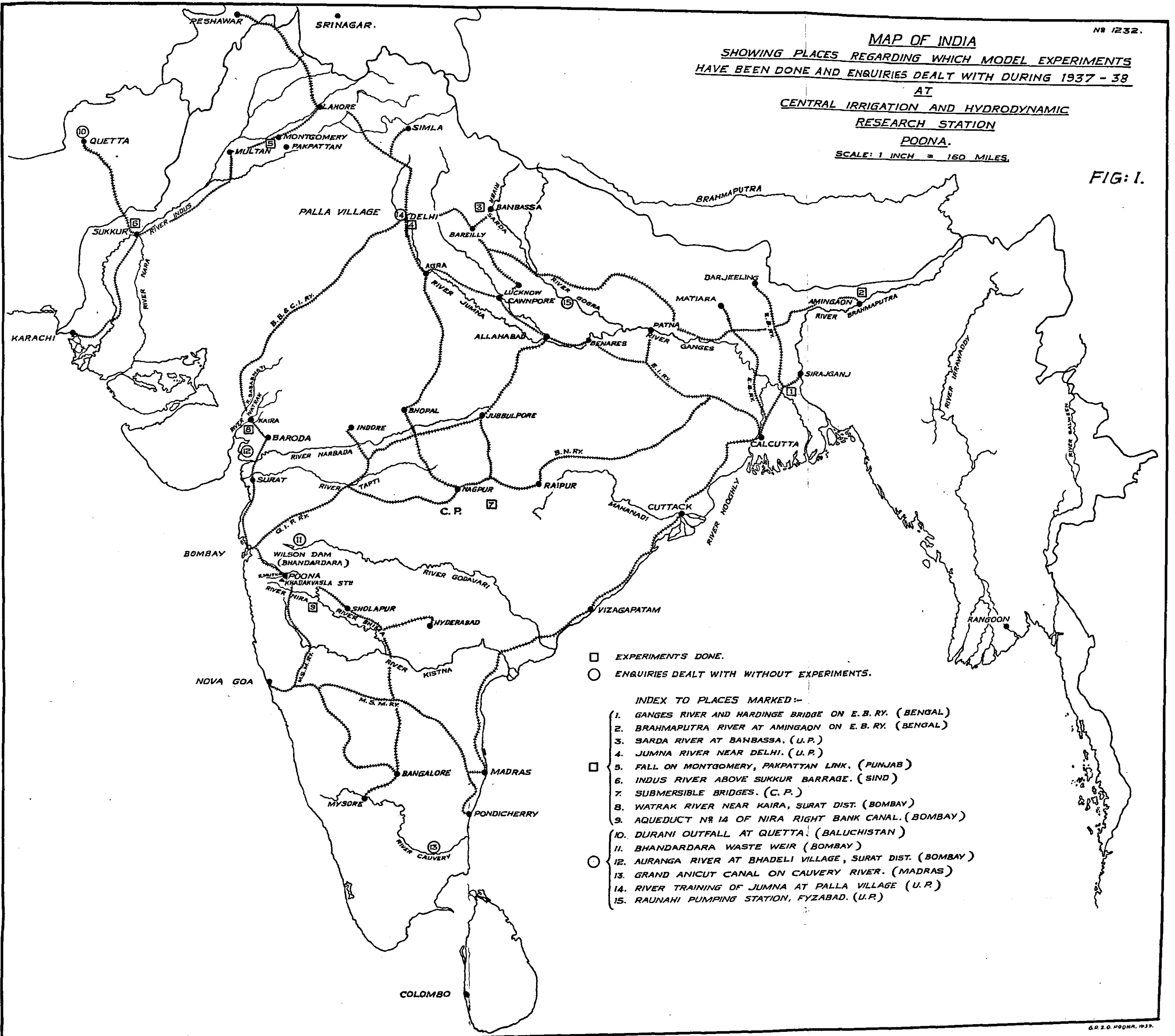
22. Standardisation of design of high coefficient weirs (continued).

MAP OF INDIA
 SHOWING PLACES REGARDING WHICH MODEL EXPERIMENTS
 HAVE BEEN DONE AND ENQUIRIES DEALT WITH DURING 1937 - 38

AT
**CENTRAL IRRIGATION AND HYDRODYNAMIC
 RESEARCH STATION**
POONA.

SCALE: 1 INCH = 160 MILES.

FIG: 1.



- EXPERIMENTS DONE.
- ENQUIRIES DEALT WITH WITHOUT EXPERIMENTS.

INDEX TO PLACES MARKED:-

1. GANGES RIVER AND HARDINGE BRIDGE ON E. B. RY. (BENGAL)
2. BRAHMAPUTRA RIVER AT AMINGAON ON E. B. RY. (BENGAL)
3. SARDA RIVER AT BANBASSA. (U.P.)
4. JUMNA RIVER NEAR DELHI. (U.P.)
- 5. FALL ON MONTGOMERY, PAKPATTAN LINK. (PUNJAB)
6. INDUS RIVER ABOVE SUKKUR BARRAGE. (SIND)
7. SUBMERSIBLE BRIDGES. (C. P.)
8. WATRAX RIVER NEAR KAIRA, SURAT DIST. (BOMBAY)
9. AQUEDUCT NR 14 OF NIRA RIGHT BANK CANAL. (BOMBAY)
- 10. DURANI OUTFALL AT QUETTA. (BALUCHISTAN)
11. BHANDARDARA WASTE WEIR (BOMBAY)
12. AURANGA RIVER AT BHADELI VILLAGE, SURAT DIST. (BOMBAY)
13. GRAND ANICUT CANAL ON CAUVERY RIVER. (MADRAS)
14. RIVER TRAINING OF JUMNA AT PALLA VILLAGE (U.P.)
15. RAUNAHI PUMPING STATION, FYZABAD. (U.P.)

23. Experiments with head regulators to ascertain factors causing loss of head (continued).

24. Divergence experiments for determining optimum conditions ensuring stable flow (continued).

25. Venturi flume experiments—to produce type designs and calibrate them (continued).

26. Scour round piers and the behaviour of stone protection (continued).

27. Factors affecting meanders of rivers.

LIST OF SYMBOLS.

A=Cross sectional area (sq. ft.).

B=Width of channel (ft.).

B_s=Surface width.

C=Coefficient, (usually coefficient of discharge=Q/BD^{1.5}).

D₁=Depth of water upstream over sill of a flume (ft.).

D=D₁+h_v.

d=Depth of water in channel (ft.).

d_s=Depth of scour below W. L. (ft.).

f=Lacey Silt Factor.

g=Gravitational constant.

h_v=Head due to velocity.

m=Mean diameter of particle in m. ms.

N=Coefficient of rugosity.

N_a=Absolute coefficient of rugosity (Lacey).

P=Wetted perimeter (ft.).

Q=Total discharge (cusecs).

q=Discharge per unit width (cusecs/ft.).

R=Hydraulic mean depth (ft.).

S=Hydraulic gradient.

V=Velocity (ft. per second).

V_c=Critical velocity= \sqrt{gd} .

w=Weight of water (lbs. per C. ft.).

ρ=Density of water.

μ=Viscosity of water.

ν=Kinematic viscosity= $\frac{\mu}{\rho}$

F.N.=Froude number.

$$= \frac{V^2}{gd} = \frac{q^3}{g^{2/3}d^2}$$

R.N.=Reynold's number= $\frac{Vd}{\nu}$

F.S.Q.=Full supply discharge (cusecs).

W.L.=Water level.

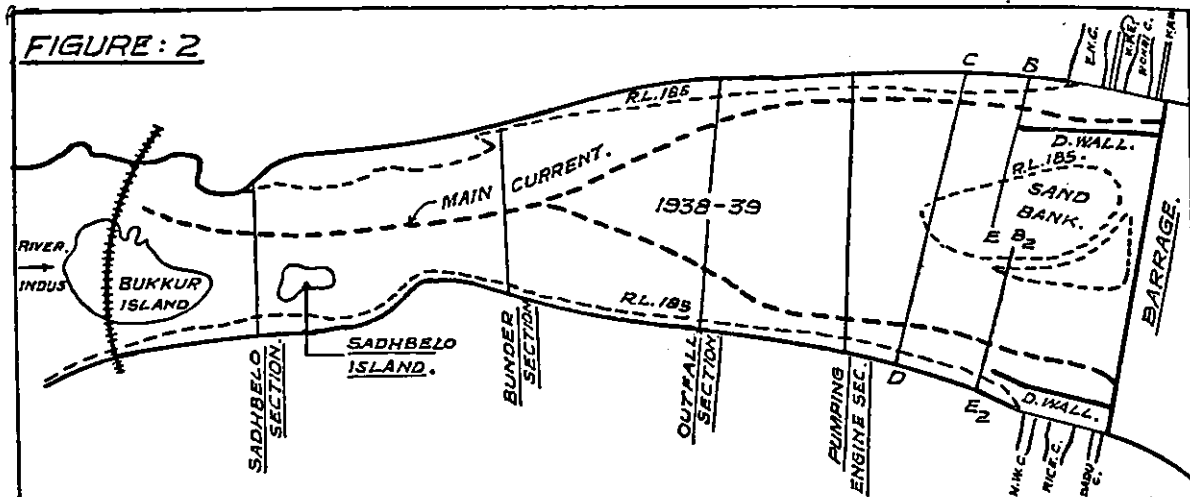
≡ =Equivalent to.

Thus Q=10 cusecs≡100,000 cusecs means discharge in model was 10 cusecs equivalent to 100,000 cusecs in the prototype.

PART I—SPECIFIC EXPERIMENTS.

1. EXPERIMENTS WITH MODELS OF THE INDUS AND THE LLOYD BARRAGE AT SUKKUR.

For an introductory Note, see Annual Report for 1937-38. Figure 2 shows the river upstream of the Barrage.



As a result of the slight bend, the left bank Canals draw little silt but the right bank Canals, in particular the North Western Canal, draw excessive bed silt. The object of these experiments was to test various alternatives designed to reduce silting in the right bank canals.

Two models were used, one being of 1/300 horizontal scale, embracing 7 miles of the river above the Barrage, and the other 1/150 scale embracing the length of river shown in Figure 2.

Verification of scale ratios in 1/300 scale model.

Preliminary experiments were carried out to find the discharge, silt, and time scale ratios which would reproduce cross sections similar to those of the prototype—allowing for relative vertical scales.

Three discharge scale ratios, calculated by different methods, were,

(a) $Q_m/Q_p=1/51,500$, scales being worked out from the rectangular cross sections of the river between the Divide Wall at the nose with
 Latitudinal scale=Longitudinal scale=1/300
 Depth scale for river and gorge=1/36.

(b) $Q_m/Q_p=1/68,500$, the scales being based on elliptic dimensions of a regime channel according to the Lacey formulæ, allowing for the same divergence from regime at Outfall Section as in the prototype.

The latitudinal scale=the longitudinal scale=1/300

'P' scale=1/262

Depth scale=1/40 for the river
 and =1/50 for the gorge.

(c) $Q_m/Q_p=1/84,600$, method being similar to that in (b), except that
 'P' scale=1/300 and depth scale=1/47 for the river and=1/50 for the gorge.

The following difficulties were met with in this model:—

(1) The river portion of the models had vertical exaggerations of 8.3; 7.5 and 6.4 in (a), (b) and (c) respectively. In the gorge portion, for similarity of flow conditions, the longitudinal scale was required to be the same as the vertical scale; but this would make the gorge portion very long and would lead to other difficulties, i.e., the expansion and curvature of flow would be affected to a marked extent.

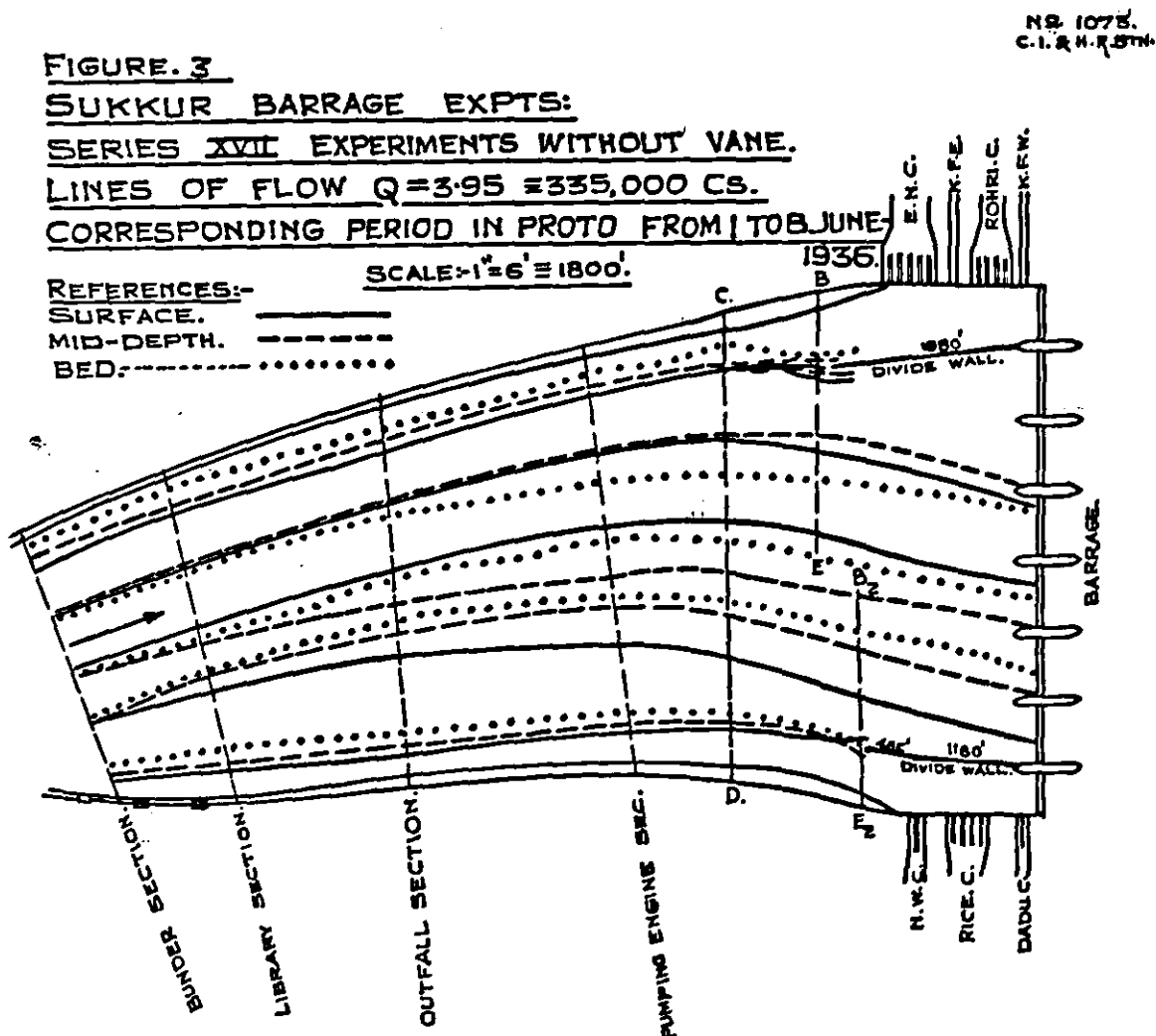
- (2) Owing to the small length of the model, it was not possible to get silt into natural suspension, and most of the moving silt—which is always relatively much less in the model than in the prototype—was bed silt and relatively coarser. For these reasons, it was impossible to reproduce silting in the model similar to prototype. Silt movement was negligible in the model except during the flood season, so silt experiments were limited to that period.

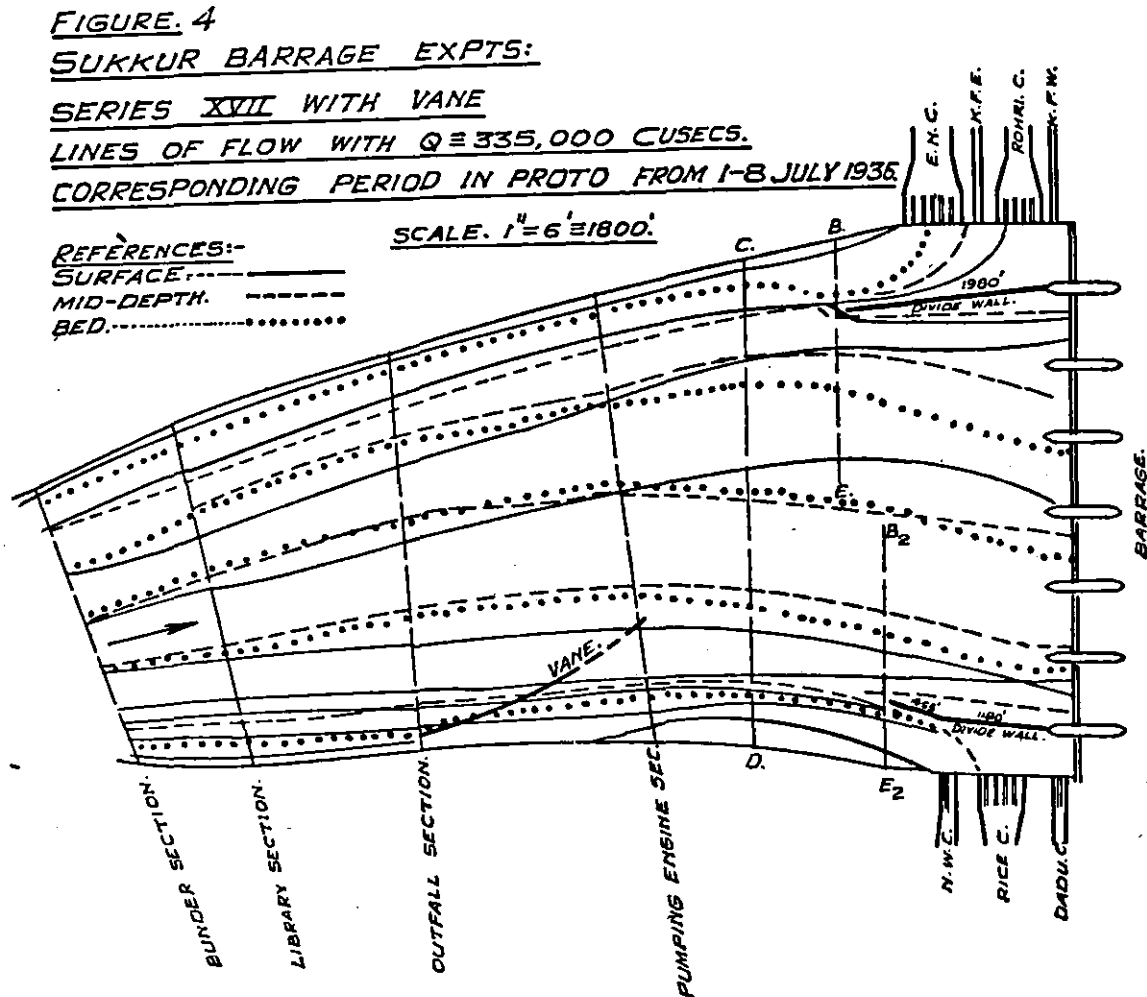
The suitability of the scale ratios was tested by comparing the bed levels in the prototype observed at fortnightly intervals at various sections upstream of the Barrage in the prototype with those obtained in the model. Experiments were done for a flood period equivalent to that of 1936—in which the maximum discharge of the river was 422,000 cusecs. Best results were obtained with the discharge scale= $1/68,500$ (i.e., according to method (b) above). The time scale ratio= $1/46$ and silt charge= $1/2$ natural silt charge for the model velocity (allowing for prototype divergence).

EXPERIMENTS WITH VARIOUS ALTERNATIVES DESIGNED TO REDUCE SILTING IN THE RIGHT BANK CANALS AND IN THE NORTH WEST CANAL IN PARTICULAR (1/300 MODEL).

A. Single large radius vane starting from Outfall Section.

Series XVI (with 1935 flood conditions approximately, maximum Q being $\approx 572,000$ cusecs) and Series XVII (with 1936 flood conditions) were carried out without and with "4,000 ft. radius vane". The top of the vane in XVI, was at \approx R. L. 184.0 and in XVII, was at \approx R. L. 190.0 at Outfall section, sloping down to \approx R. L. 185.0 at the end (see fig. 4). The upstream face of the vane was vertical but the downstream face was protected with 1 in 5 sloping pitching. Figures 3 and 4 show the lines of flow in XVII without and with vane.





With a discharge of 3.95 cusecs ($\approx 335,000$ cusecs) on a rising flood, the following were the divides measured on Bunder Section.

TABLE 1.

| Series XVII. | Width of water drawn by Right Pocket, measured at Bunder Section from R. B. | |
|------------------------|---|---|
| | Surface water. | Bed water. |
| Without vane | 0.55 ft. | 1.0 ft. |
| With vane | 1.35 ft. | Bed silt near right bank was just not excluded. |

It will be seen that with the vane, more surface water and less bed water was drawn by the Right Pocket. Figure 3 also shows a marked general trend of bed water towards the right bank and away from the left bank due to the right bank being on the inside of the curve. By inducing spiral flow by a vane, the bed silt was almost excluded from the right pocket.

When the discharge was increased to $\approx 474,000$ cusecs scour upstream of the vane began, and with $Q \approx 572,000$ cs., the discharge along the right bank increased, while the channel on the left bank began to silt. On a falling flood, however, the upstream face of the vane began to silt.

Good silt exclusion was obtained with high discharges, which is in agreement with the results of experiments carried out in 1932-34; and it appears that the vane would be effective in good years—as in 1935; but with a low river as in 1936, it would not have been helpful and with an even less favourable river it might make conditions worse.

B. Series XIX.—Closure of right gorge between Bukkur and Sukkur.

It was considered that the proportion of discharges passing through the Bukkur-Sukkur gorge and through the Bukkur-Rohri gorge was a very important

factor in influencing the curvature of flow downstream. In 1936, the main stream of the river was along the right bank upstream of Bukkur island and the discharge then drawn by the right (Bukkur-Sukkur) gorge was about 40 per cent of the total river discharge. In 1938, the main stream shifted to the left bank upstream of Bukkur island. The right gorge then drew only 20 per cent. of the total river discharge; and by the end of 1938 Abkalani, conditions at Sukkur improved considerably, because instead of one deep channel along the left bank, two channels, one on each bank appeared which will be seen from the prototype contours of the river bed in figures 5(a) and (b).

It seemed possible, therefore, that improved conditions might be maintained by controlling the proportion of discharge passing through the right gorge between Bukkur and Sukkur. This was tested in Series XIX by closing the gorge and the effect on flow conditions, velocity distribution at Outfall Section, and silting and scouring between Bunder Station and Barrage was observed for the full flood period of 1936.

Figure 6 shows the lines of flow in Series XIX, which may be compared with those in figure 3 in XVII (with 40 per cent. of the discharge flowing through the Sukkur branch of the gorge).

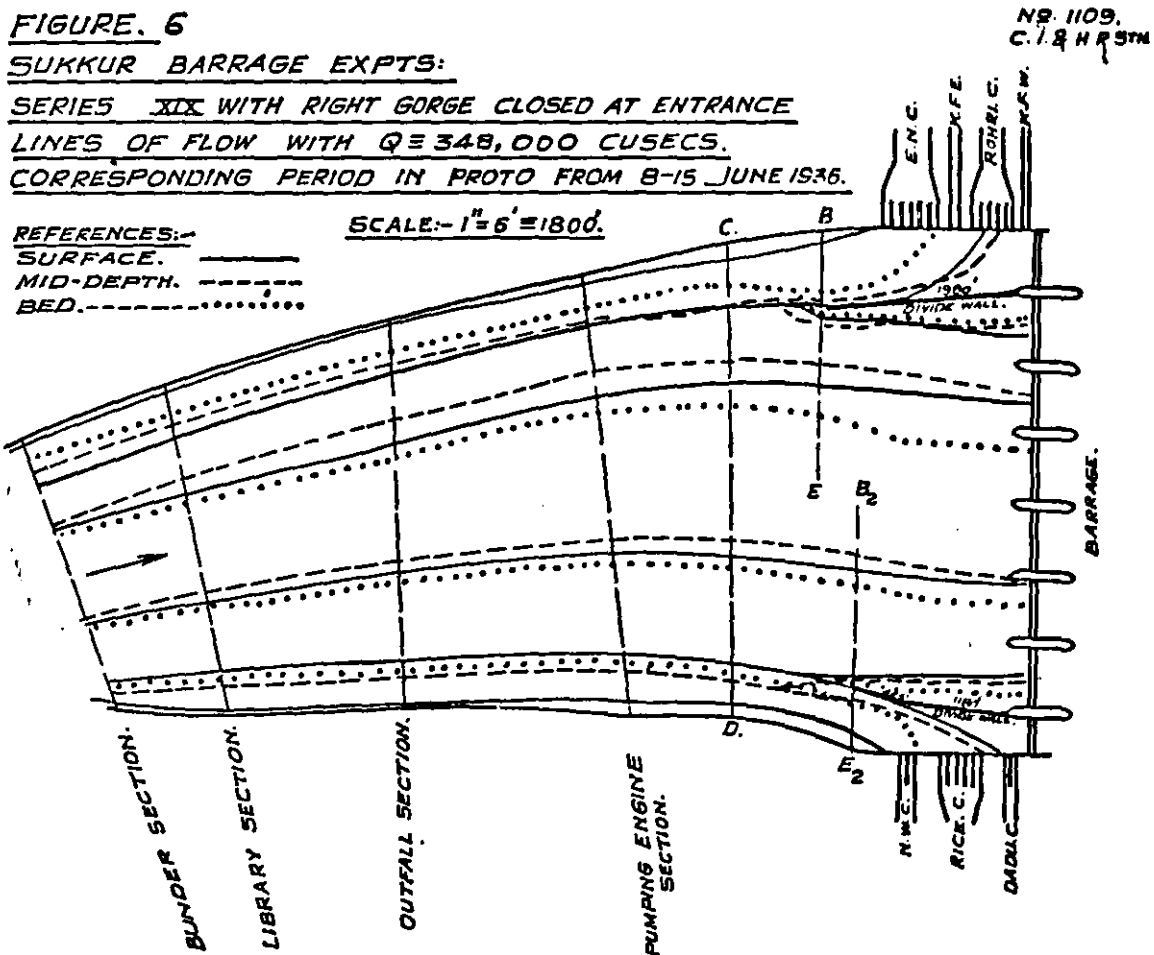


Table 2 shows 'divides' measured at Bunder Section, which indicated that more top water and less bed water was drawn by the right pocket in XIX than in XVII.

TABLE 2.

| Series. | Width of water in ft. drawn by Right Pocket, measured at Bunder Station from right bank. | | |
|--------------------------------------|--|----------|-----------------------|
| | Surface. | Bed. | Ratio of Bed/Surface. |
| Series XVII (with both gorges open) | 0.55 ft. | 1.00 ft. | 1.82 |
| Series XIX (with right gorge closed) | 0.90 ft. | 0.78 ft. | 0.87 |

Velocities were observed at Outfall Section, and were found to be more uniformly distributed than in XVII.

Cross sections, observed at intervals of \approx one month, showed greater deposition of silt near left bank and more scour near right bank in XIX than in XVII. The scour on the right bank in XIX indicated that a channel had formed close to it and was maintained throughout the flood period—*vide* figure 7 showing bed contours.

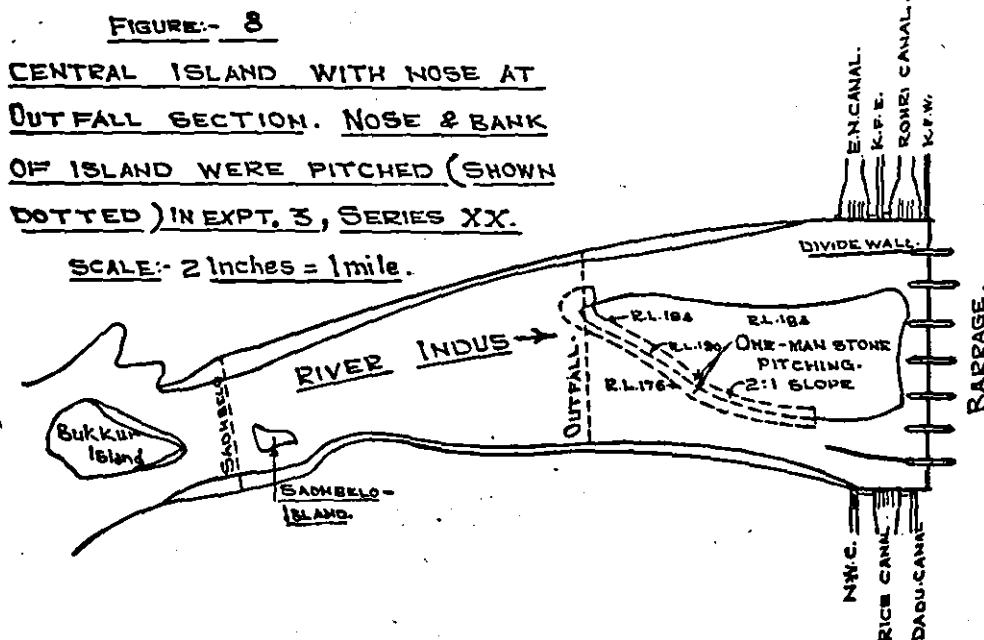
These experiments showed that the less the proportion of discharge passing through the right gorge, the less was the quantity of bed silt moving along the right bank, resulting in conditions more favourable for the Right Bank Canals.

C. Central sand bank upstream of the Barrage (experiments 1 to 5 of Series XX).

Owing to the ponding of water upstream of the Barrage since 1931, the central sand-bank had increased steadily in area and height up to the end of the cold weather of 1936-37. Every Abkalani flood season, the top of the sand-bank is slightly lowered by scouring action, while from October to May, silting takes place more or less uniformly due to the deposition of fine silt from suspension as a result of very low velocity of flow.

The object of this series of experiments was :—

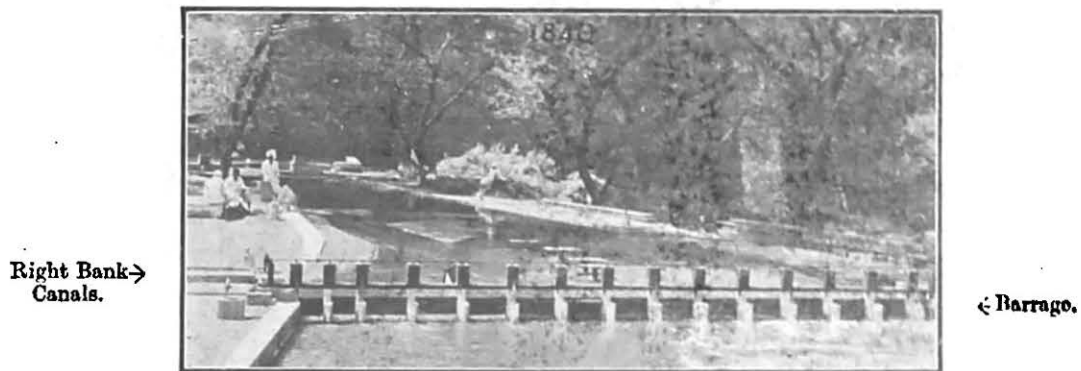
- to find out whether the central sand-bank could be maintained permanently at a desired R. L. ;
- to see whether, with a permanent sand-bank, a deep channel would persist near the right bank throughout the year as on the left bank, due to curved flow being established ; and
- to investigate the possibility of establishing adequate curved flow to carry bed silt outside the Right Divide Wall, by pitching the nose and side of the sand-bank on a suitable, curved, alignment,—see figure 8.



Experiments carried out with 1936 flood conditions, showed that the central, high level, sand-bank was not a permanency; but scoured during high floods. Even with $Q \approx 350,000$ cusecs, the "one-man-stone" protection laid in the model along the margin of the sand-bank scoured down to \approx R. L. 184.0; and with $Q \approx 750,000$ cusecs, it would probably scour down to R. L. 177.0.

If, therefore, it were desired to maintain the margin of the sand-bank at R. L. 184.0, blocks weighing at least 700 lbs. would be required and scour would be very severe along the left bank channel, due to restriction of waterway.

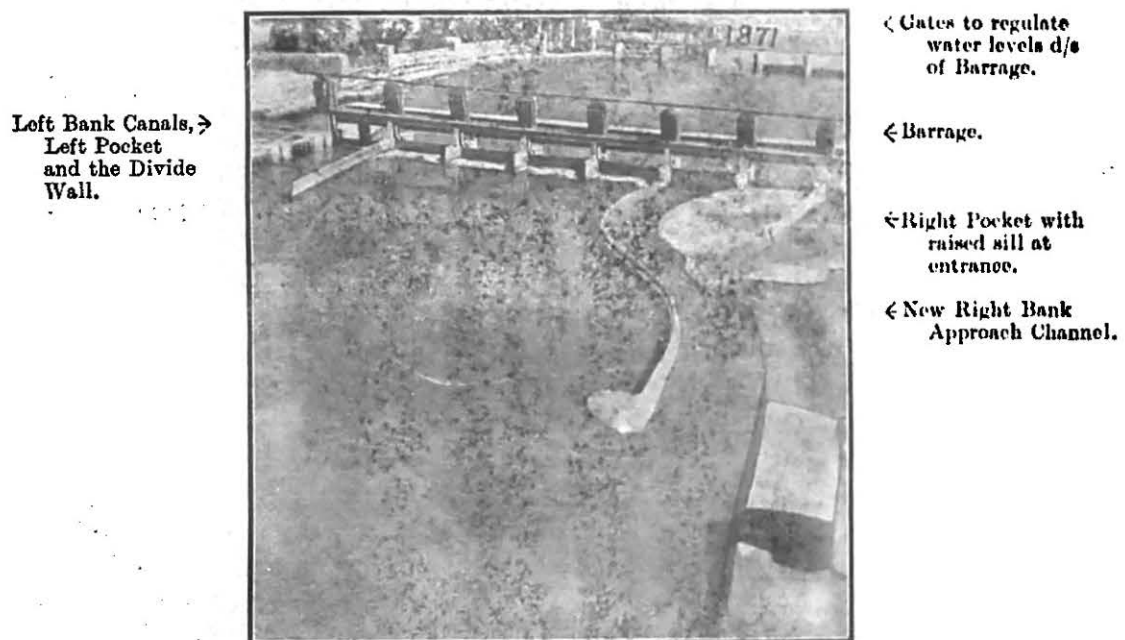
EXPERIMENTS WITH MODELS OF THE RIVER INDUS ABOVE THE SUKKUR BARRAGE.



Right Bank →
Canals.

← Barrage.

1840. General view of 1/150 scale Sukkur Barrage model, looking upstream. (Depth scale ratio=1/30 and discharge scale ratio=1/20,700).



Left Bank Canals, →
Left Pocket
and the Divide
Wall.

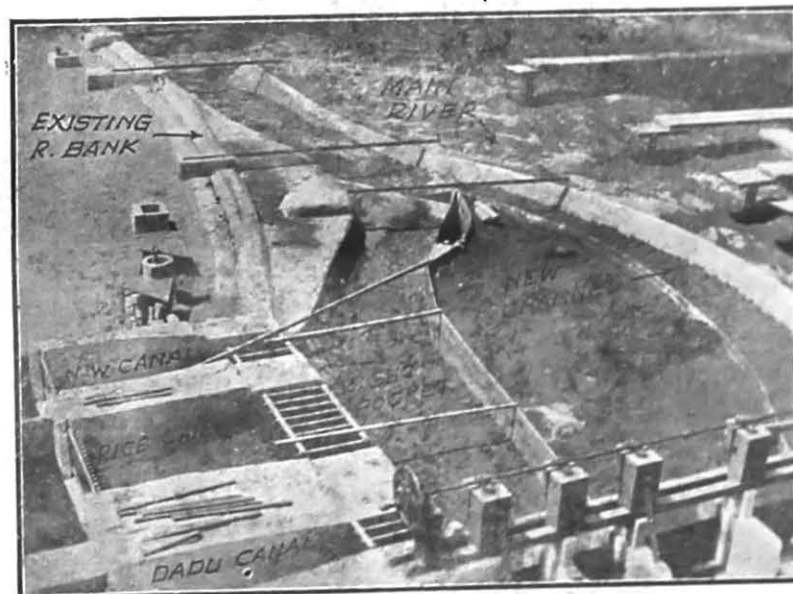
← Gates to regulate
water levels d/s
of Barrage.

← Barrage.

← Right Pocket with
raised sill at
entrance.

← New Right Bank
Approach Channel.

1871. 1/300 scale Sukkur Barrage model (Depth scale=1/40 and discharge scale=1/68,500) with the new Right Bank Approach Channel, showing flow conditions with Q River=400,000 cusecs and of Right Bank Approach Channel=40,000 cusecs, looking downstream,



1967. 1/150 scale Sukkur Barrage Model, showing the new Right Bank Approach Channel, Right Pocket with raised sill and Right Bank Canals, looking upstream.

FIGURE. 7

SUKKUR BARRAGE EXPTS:

SERIES XIX WITH RIGHT GORGE
CLOSED AT ENTRANCE.

BED CONTOURS AFTER Q=4.2 C_s.

≅ 348,000 C_s. RUN FOR 14.4 HOURS

≅ 30 DAYS. I. E. APPROX: DISCHARGE

CONDITIONS BETWEEN

16 JULY TO 15 AUGUST 1936.

SCALE :- 1" = 4' ≅ 1200'

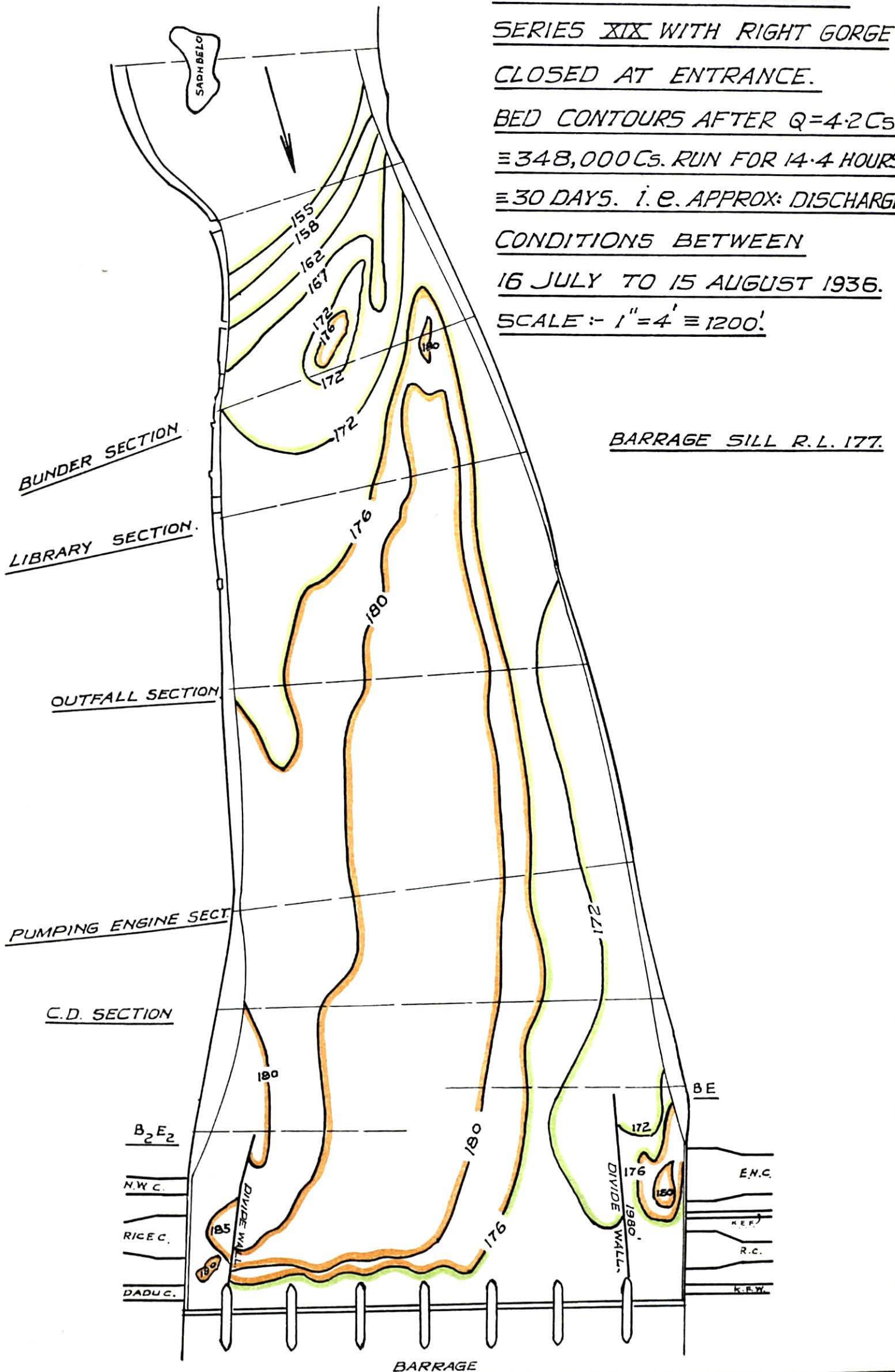
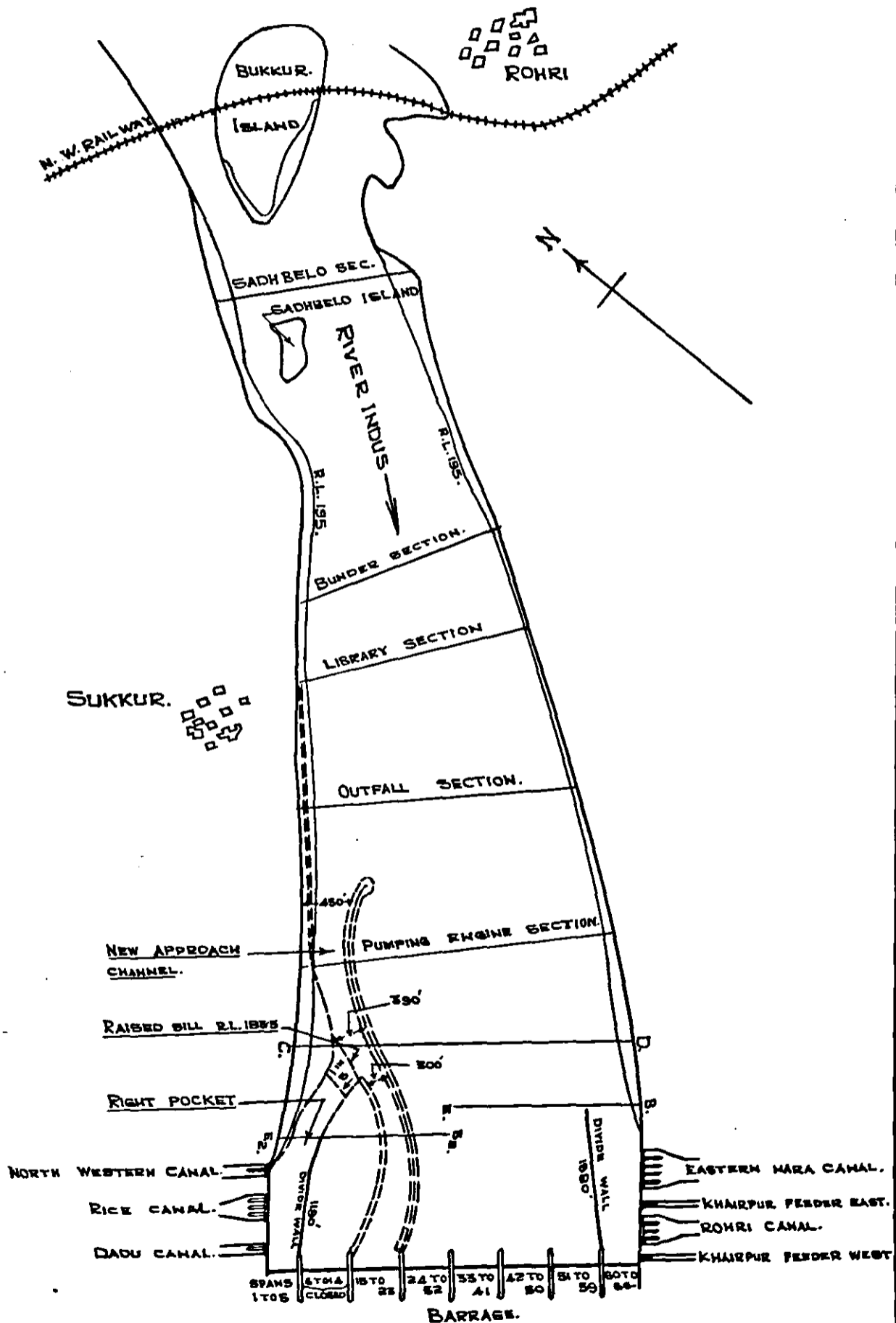


FIGURE 9.
SUKKUR BARRAGE EXPERIMENTS.
SERIES XX (EXPT:6)

PLAN SHOWING DESIGN OF NEW APPROACH CHANNEL
ALONG RIGHT BANK TO EXCLUDE SILT FROM
RIGHT BANK CANALS AT SUKKUR (SHOWN DOTTED)
SCALE - 1" = 6' = 1800'



D. Design of new Right Bank Approach Channel to exclude silt from the Right Bank Canals.

The requisites for silt exclusion from the Right Pocket appear to be :—

- (i) concave river curvature along the right bank between Outfall section and the Right Pocket, in order to deflect bed silt away from the pocket ;
- (ii) an adequate discharge along the right bank throughout the year to prevent alternate silting and scouring ;
- (iii) a favourable velocity ratio at the nose of the Right Divide Wall for the period when bed silt is moving ; and
- (iv) conditions favourable to scouring the undersluice pocket.

The two essentials are curvature of channel flow and control of discharge.

All these desirable features would be insured and maintained by adopting the design of new approach channel shown in figure 9. This design was tested in the 1/300 and also in a new 1/150 scale models, constructed to test the scale effect, and was found to be satisfactory—*vide* photos 1840, 1871 and 1967. The maximum discharge run in the 1/300 model was 13.15 cusecs (\cong 900,000 cusecs) and in the large model, 29 cusecs (\cong 600,000 cusecs).

The design consists of a curved approach channel—5,000 ft. long from the nose to the Barrage and 2,250 ft. long from the nose to the off-take of the Pocket—this design creates favourable curvature of channel flow and so exclude bed silt from the Right Pocket. To improve silt exclusion still further, a raised sill, with top at R. L. 183.5 is provided at the entrance to the Pocket.

Discharges in the approach and Right Pocket channels would be controlled by the Barrage gates and the most favourable combination would be found by actual test in the prototype. Either 'semi-open flow' or 'still pond' method of running might be followed. This design is based on the same principle as the silt excluding approach channel to the Mithrao Canal—Ex Nara at Makhi—, which has proved so successful⁽¹⁾; and the model has confirmed that it would exclude practically all bed silt.

At river discharges exceeding 300,000 cusecs, the entrance to the approach channel would itself act as a silt excluding head⁽²⁾, during the time when heavy bed silt was in motion.

The approach channel as designed would be \cong 450 ft. wide at entrance, reducing down to 390 ft. at the upstream end of raised sill, and to 300 ft. at the downstream end of raised sill. Further downstream, the channel would flow through spans 15 to 23 of the Barrage ; spans 6 to 14 being completely closed. The approach channel would normally carry a discharge ranging between 26,000 and 35,000 cusecs.

When the model was run with a discharge \cong 600,000 cusecs with all the river gates fully open, including the gates of spans 15 to 23, the discharge through the approach channel was found to be \cong 90,000 cusecs. This showed that even when all the gates of the river portion of the Barrage were open—when the head available for forcing water through the approach channel would be minimum—, the approach channel discharge would have to be throttled down by gate regulation. This is a very important factor ; because it means that it would be possible to maintain a scouring velocity in the approach channel under all conditions except in the cold weather months, when the discharge passing through the approach channel undersluices would be very small, as nearly the whole of the supply is then required by the Canals. This would not, however, cause trouble, because under such conditions there is very little bed movement and any small deposition of fine silt could be scoured away.

It may be thought that a reduction in length of the Barrage would be an undesirable feature of this design. The Barrage was, however, designed to pass an estimated maximum flood discharge of 1,500,000 cusecs ($q=379$ cusecs per foot

¹ Bombay Public Works Department, Technical Paper No. 59 : " Silt control at the Heads of Canals and Tributaries " by C. C. Inglis and Rao Sahib D. V. Joglekar.

² Bombay Public Works Department, Technical Papers Nos. 45 and 46 " Note on Silt exclusion from Canals " Parts I and II, by C. C. Inglis.

run), whereas the maximum discharge recorded during the last 25 years has been about 800,000 cusecs. The design of the approach channel is based on a maximum flood discharge of 900,000 cusecs (or $q=348$ cusecs per ft. width which is less than the designed q).

It has been suggested that the approach channel would deflect the river course towards the left bank downstream of the Barrage. Any such tendency would, however, be counterbalanced by the larger discharge passing through the right bank undersluices and the approach channel undersluices in the cold weather causing a right bank channel to form. Moreover as the river course below the Barrage is largely determined by conditions above the gorge and downstream of the Barrage, it is doubtful if the effect would be considerable.

The design may require modification of detail, taking practical requirements into account, and experiments with both models will be continued during the next year.

2. TRAINING SARDA RIVER ABOVE THE BARRAGE AT BANBASSA—UNITED PROVINCES.

Introduction.

The experiments conducted during the year had two objects:—

One of the main problems in connection with the Sarda Barrage, as described in the 1937-38 Annual Report, is the tendency for the flow from the Eastern Channel to cling to the Left Upstream Bell Bund, from where it has to be drawn across the noses of the Barrage piers to feed the Right Pocket (see Figure 10(a)). Attempts to open up the Western Channel by gate operation failed; but model experiments, previously described, showed that an extension of the Burma Spur would be beneficial.

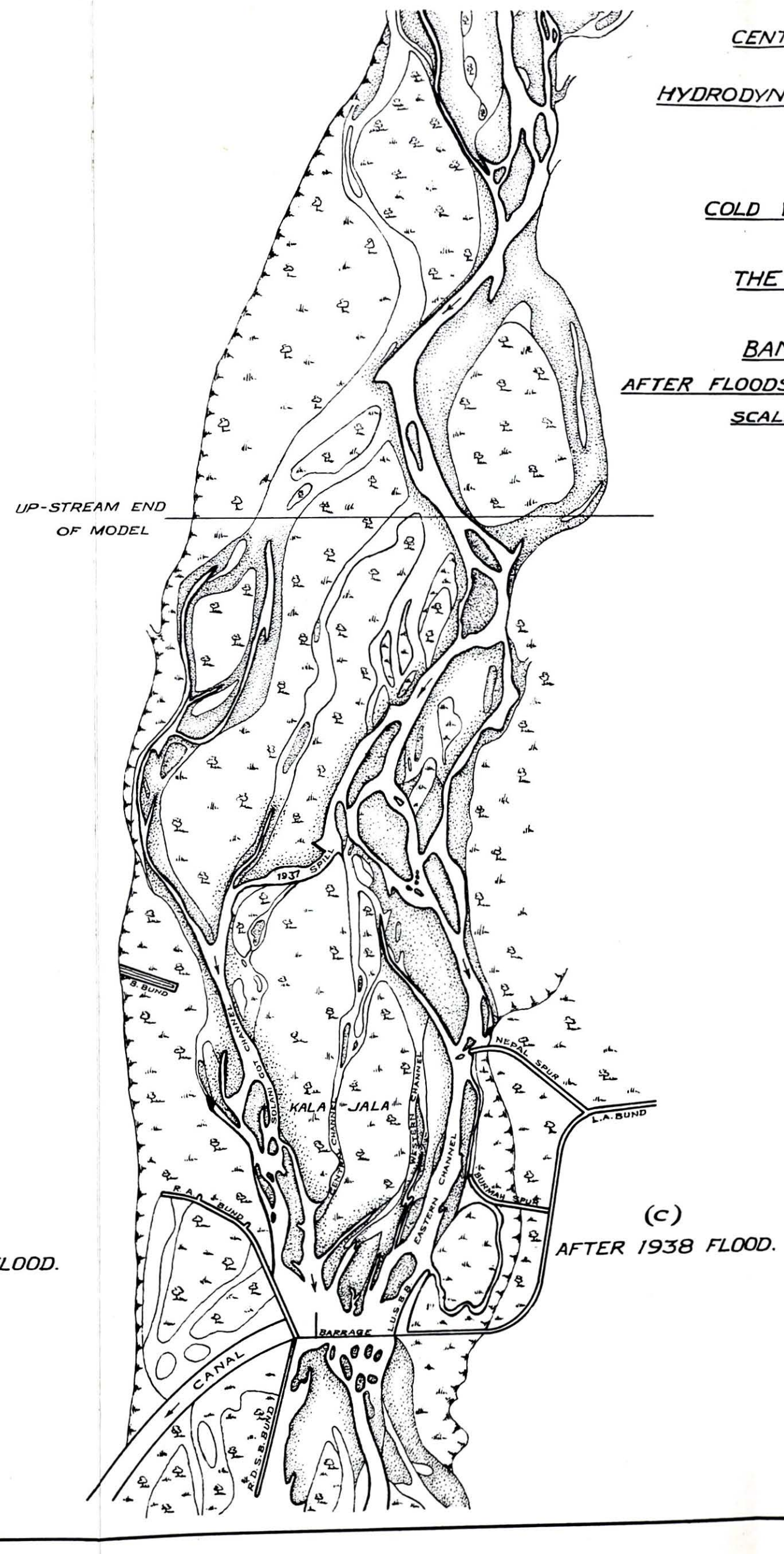
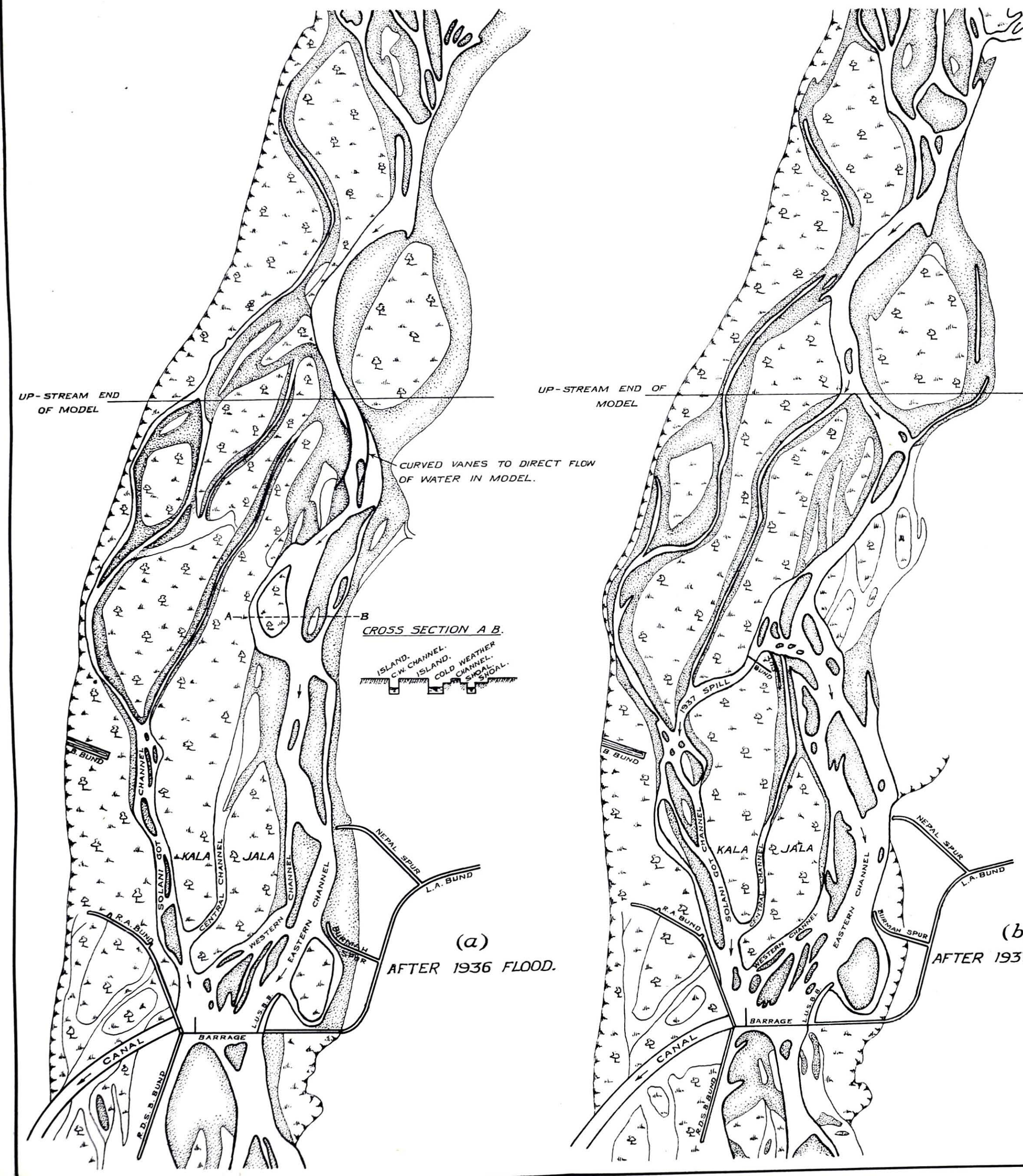
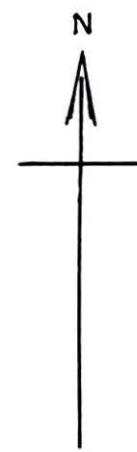
As stated in the 1937-38 Annual Report, however, it is doubtful whether the proposal to reopen the Western Channel by extending the Burma Spur would be successful, unless a considerable part of the east and south-east bank of the Kala Jala island could be eroded by extending the head of the Nepal Spur. Experiments were, therefore, carried out to see the effect of extending the Nepal spur on the Western Channel.

The other problem concerns the protection of the Right upstream Afflux Bund. Figures 10(a), (b) and (c) show the cold weather survey of the Sarda River at Banbassa after floods of 1936, 1937 and 1938 respectively. Until the beginning of the 1937 flood season, the main stream was through the Eastern Channel, the Solani Got Channel taking about 25 per cent. of the discharge during floods. During the 1937 floods, a complete meander upstream of the Nepal Spur was established and a new channel—shown as 1937 spill in Figure 10(b)—developed from the main channel to the Solani Got Channel. This was considered to be undesirable by the local officers, because in the event of the Solani Got Channel becoming the main stream, the Right Afflux Bund would be endangered. So a boulder bund was constructed across the mouth of the 1937 spill to prevent its further development, and a cut was made from the main river in front of the 1937 spill into the old spill of the Kala Jala island, called the Central Channel—see figure 12. It was predicted by the Director of this Station, and subsequently shown by the model, that as conditions in 1937 had been very critical, the boulder bund would be washed away and the 1937 spill would enlarge. This actually happened during the 1938 floods, the maximum discharge during the year being 380,000 cusecs—which was the second largest flood passed through the Barrage (the largest being 522,000 cusecs in 1934). The central channel through the Kala Jala did not open—which had also been shown by the model. During the 1938 floods, the Solani Got Channel carried 50 per cent. of the discharge, of which 30 per cent. passed through the 1937 spill, and the remaining 50 per cent. through the Eastern Channel.

The flow conditions at the Barrage in 1938 were better than hitherto, in that nearly half the discharge flowed almost normal to the Barrage through the Solani Got channel; and the flow from the Eastern Channel spread towards midstream, instead of being concentrated along the left upstream Bell Bund.

CENTRAL IRRIGATION
&
HYDRODYNAMIC RESEARCH STATION
POONA.

FIG. 10
COLD WEATHER SURVEY
OF
THE SARDA RIVER
AT
BANBASSA (U.P.)
AFTER FLOODS OF 1936, 1937 & 1938.
SCALE 1 INCH = 1/2 MILE.



Since 1937, the river has been in a very sensitive condition. The changes in the river course show the desirability of constructing groynes along both banks, so that the river will be controlled within a width not more (and preferably less) than the length of the Barrage. Experiments were carried out to show the effect of building a short spur at the corner of the Right Afflux Bund to protect the latter from attack, due to increased discharge in the Solani Got channel.

Experiments.

The scale ratios of the model were :—

| | | |
|------------------------|-----------|----------|
| Horizontal scale ratio | | 1/150 |
| Depth scale ratio | | 1/50 |
| Discharge scale ratio | | 1/70,000 |
| Time scale ratio | | 1/300 |

The first essential was to reproduce the prototype conditions. Series I to IV were done with this object in view, the criterion being that the model laid according to 1936 conditions should reproduce the development of 1937 conditions. The bed, islands and banks in prototype consist of widely varying materials, from fine sand to large boulders (maximum diameter=18"). In the model, this was simulated by fine sand to $\frac{1}{2}$ " dia. pebbles. The tops of islands and banks, which are covered with vegetation, were laid in cohesive silt.

The upper limit of the model is shown in figure 10(a). At this point, the river in the model tended to flow straight, instead of following a meandering course as in the prototype. It was, therefore, found necessary to construct curved vanes to direct the flow. The lower part of the model depends on conditions upstream, which are highly unstable. If the entry to a model can be located in a stable reach, entry conditions present little difficulty; but when, as in the Sarda River, the whole river (except at the Barrage) is unstable, entry conditions are all-important; and success was obtained only after experiments had shown how to impose correct entry conditions.

Series V with the extended Nepal Spur.—The Nepal spur was extended by the equivalent of 720 ft.—shown dotted in Fig. 11—so as almost to block the rapid. (See p. 12).

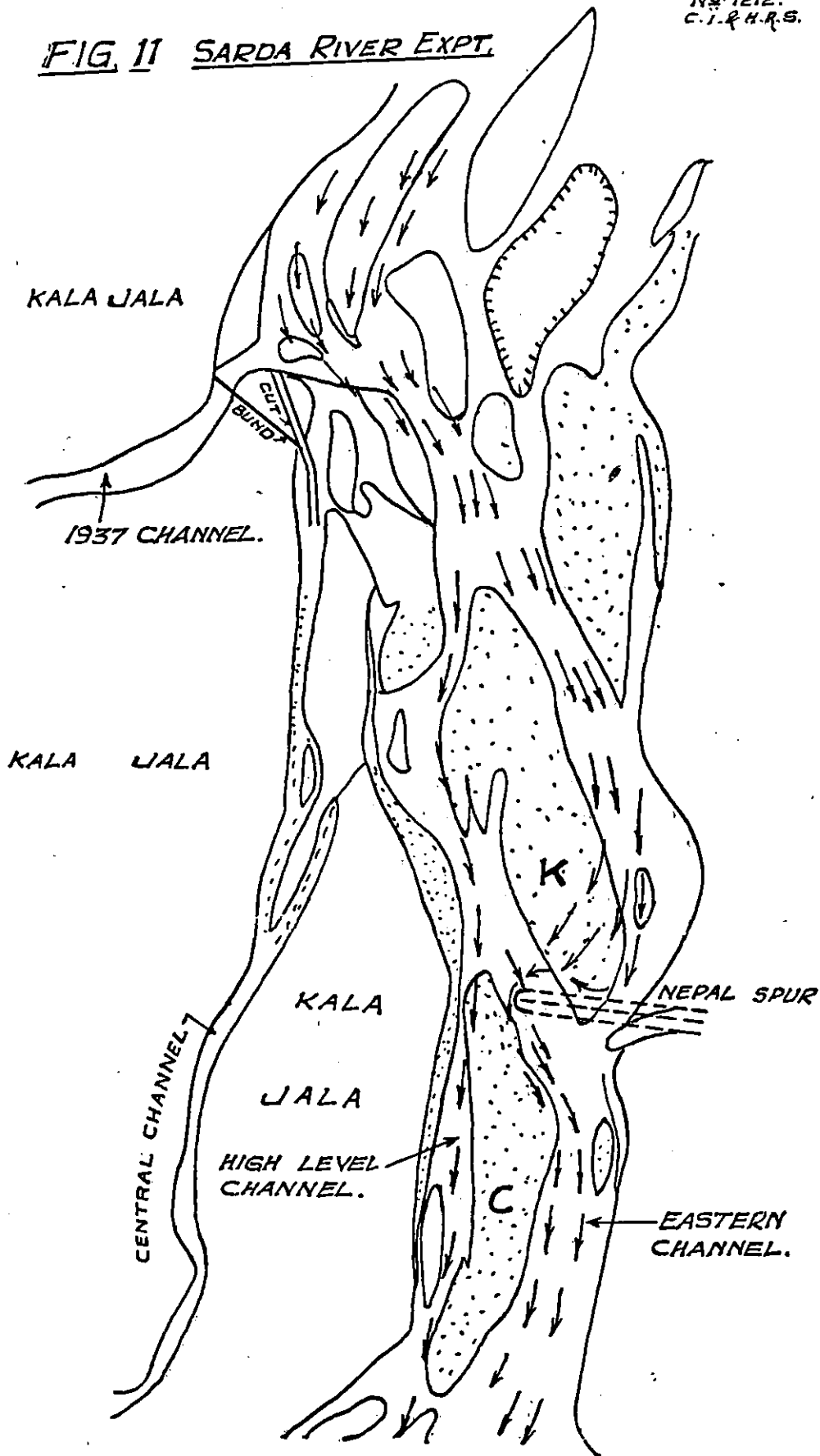
The model was run for the equivalent of 5 years, discharges in the first 4 years being as in 1936 with maximum $Q \cong 240,000$ cusecs; but in the 5th year, the maximum Q run was $\cong 522,000$ cusecs. Upstream of the extended Nepal Spur, the main stream cut through the shoal 'K' and flowed round the nose into the Eastern channel—see figure 11. Eventually, deposition of silt scoured from upstream of the Nepal spur took place in the Eastern Channel, and a channel began to form through the island 'C', thus increasing the discharge running along the edge of the Kala Jala island. The flow conditions downstream of the Burma Spur were, however, little changed *i.e.*, water did not flow through the Western Channel; but flowed straight towards the left bank spans (19 to 34) of the Barrage. Thus, extending the Nepal spur failed to cause any considerable curvature of flow, which is essential for major effects in the prototype. In this series, the proportion of discharges carried by the 1937 spill, the Eastern channel and the Central channel was recorded and the results agreed closely with those of the prototype. Also, the bund constructed across the mouth of the 1937 spill was scoured away and the channel enlarged during the experiment, as actually happened in the prototype.

Series VI.—In the prototype, the central channel through the Kala Jala island does not show any signs of opening up; and the Western Channel silted in 1938 floods. In the model, two bunds, one to divert water into the Central Channel and the other into the Western Channel—shown in dotted lines in figure 12—were constructed, in addition to the extended Nepal spur of Series V, with the object of increasing the flow through these channels. The mouth of the Central Channel is situated on the outside of a curve. At the beginning, part of the flow was diverted into the Central channel; but this carried scoured bed material along with it, which deposited at the mouth of the channel, thus preventing further opening. Neither was the Bund, which projected into the main stream opposite the Burma Spur effective in diverting part of the flow into the Western channel, because the bed material scouring from the upstream side entered the Western channel and blocked it still further.

Series VII.—The object of this series was to verify the effect of proposals designed to protect the Right Afflux Bund from possible scour, due to increased

NO. 1212.
C. I. & H. R. S.

FIG. 11 SARDA RIVER EXPT.



discharge through the Solani Got channel; and also to see whether short groynes along the eastern face of the Kala Jala island would pull the main stream towards the Kala Jala island.

Experiment 1—figure 13—

- (i) with a short spur at the corner of the Right Afflux Bund. Length of straight stem \cong 300 ft. and 90° curved head of radius \cong 300 ft. and
- (ii) with short groynes pointing 45° downstream, also constructed at three points along the eastern face of the Kala Jala island to see their effect on pulling the main stream across to the Kala Jala island.

FIG: 12 SARDA RIVER AT BANBASSA

THE BUNDS TESTED IN EXPTS.

OF SERIES 6 ARE

SHOWN IN DOTTED LINE.

SCALE OF PLAN 1" = 1200 FT.

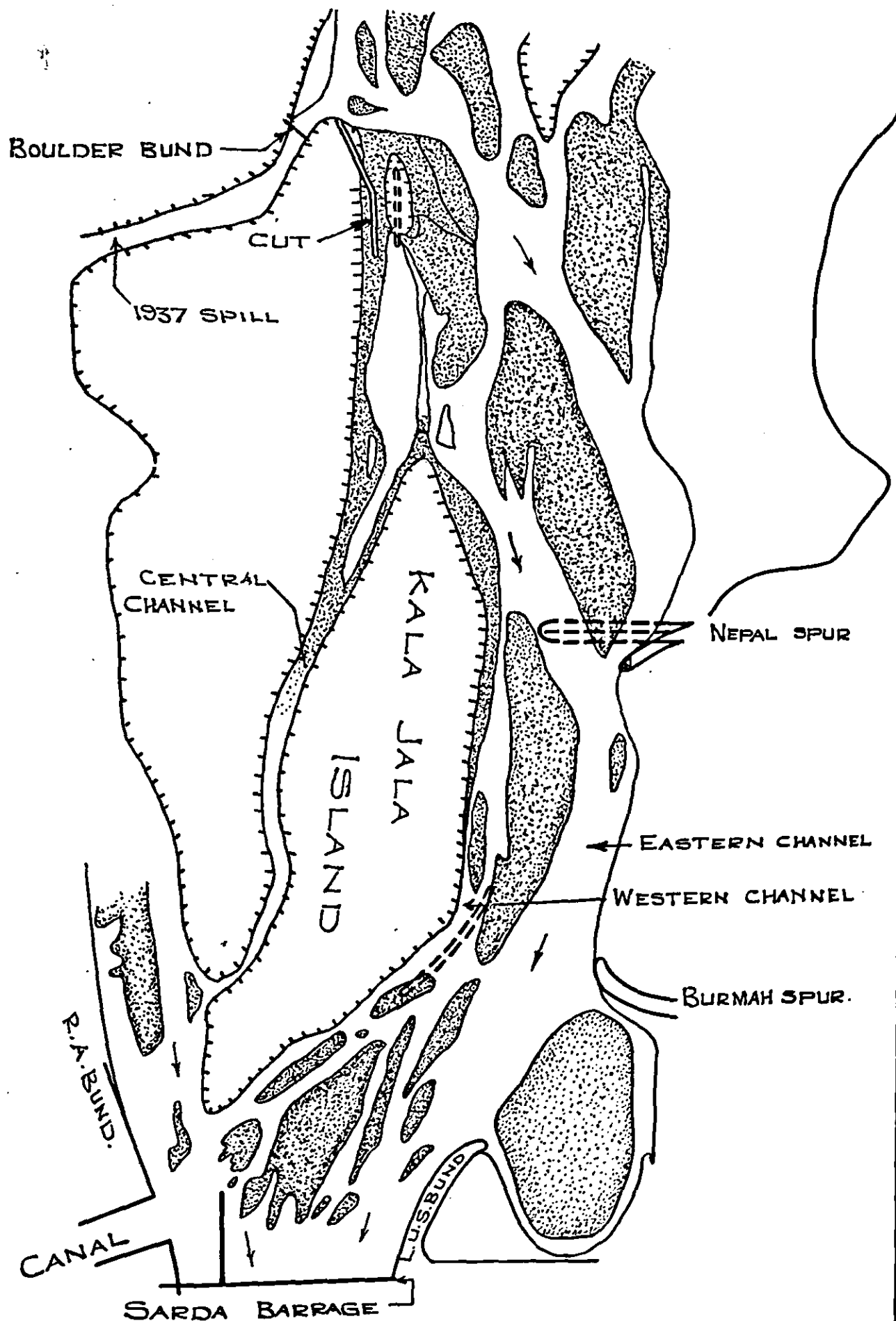


FIG:13.
SARDA RIVER EXPTS.
SERIES VII EXPTS. 1 AND 2.
LINES OF FLOW AFTER FLOW \approx 5 YEARS.
W. L. AT NEPAL REGULATOR U/S OF BARRAGE \approx 730.6.

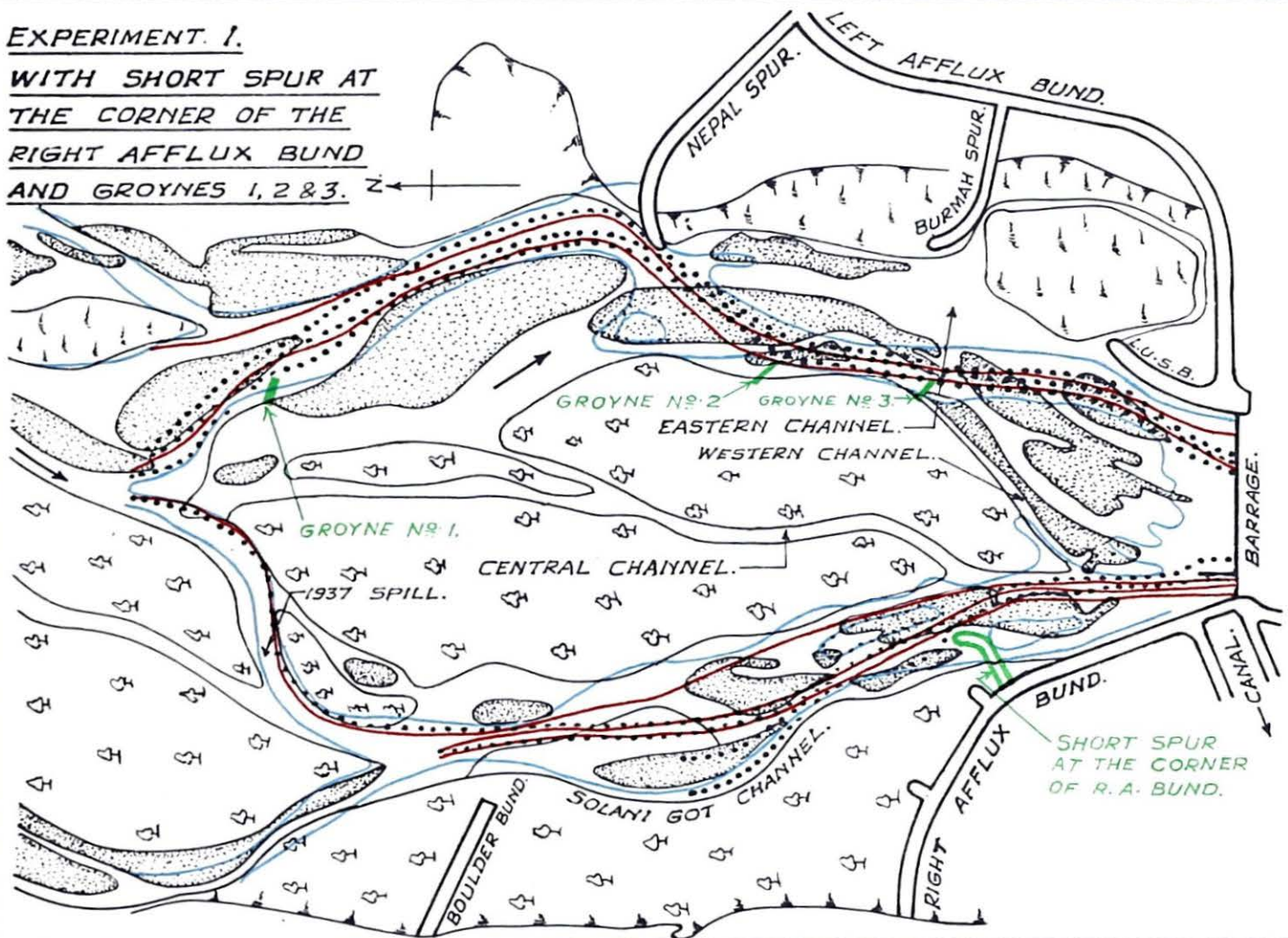
SCALE: 8" = 3 MILES.

REFERENCES.

| | |
|------------------------|-------|
| SURFACE LINES OF FLOW. | — |
| BED | |
| WATER EDGE. | — |

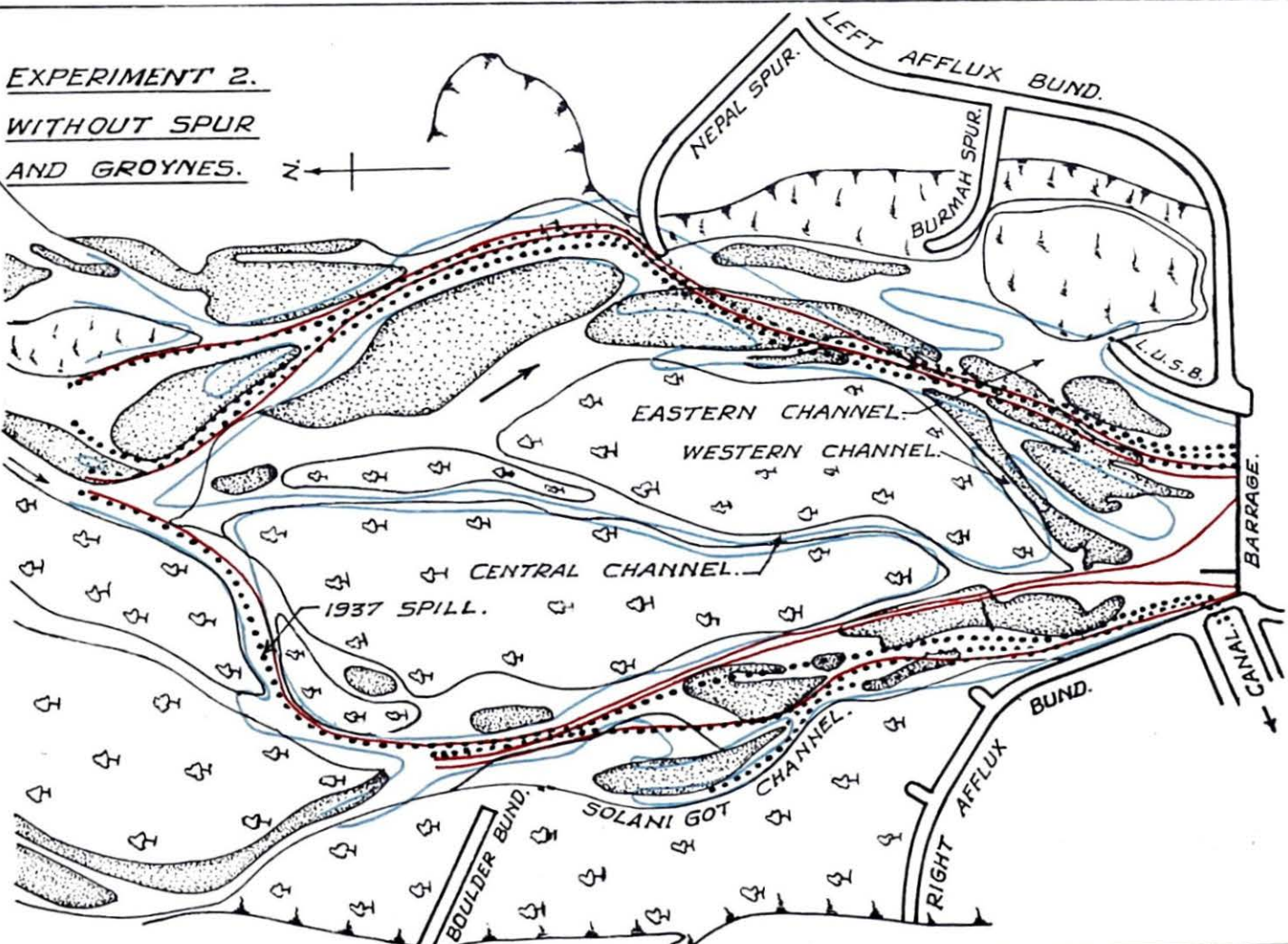
EXPERIMENT 1.

WITH SHORT SPUR AT THE CORNER OF THE RIGHT AFFLUX BUND AND GROYNES 1, 2 & 3.



EXPERIMENT 2.

WITHOUT SPUR AND GROYNES.



MODEL OF THE SARDA RIVER AT BANBASSA.

Discharge scale ratio=1/70,000.

Horizontal scale ratio=1/150.

Depth scale ratio=1/50.



←Barrage.

←Right Afflux Bund.

←Old low level boulder Bund.

1869. Series VII, Experiment 2, without spur on Right Afflux Bund (*i.e.*, 1938 conditions), looking downstream.



← Approved short spur.

1891. Series VII, Experiment 3, with approved short spur at corner of the Right Afflux Bund, looking downstream.

Experiment 2 without spurs—*vide* photo 1869 and fig. 13. The model was laid according to the 1938-39 survey plan and discharges were run as in the previous series.

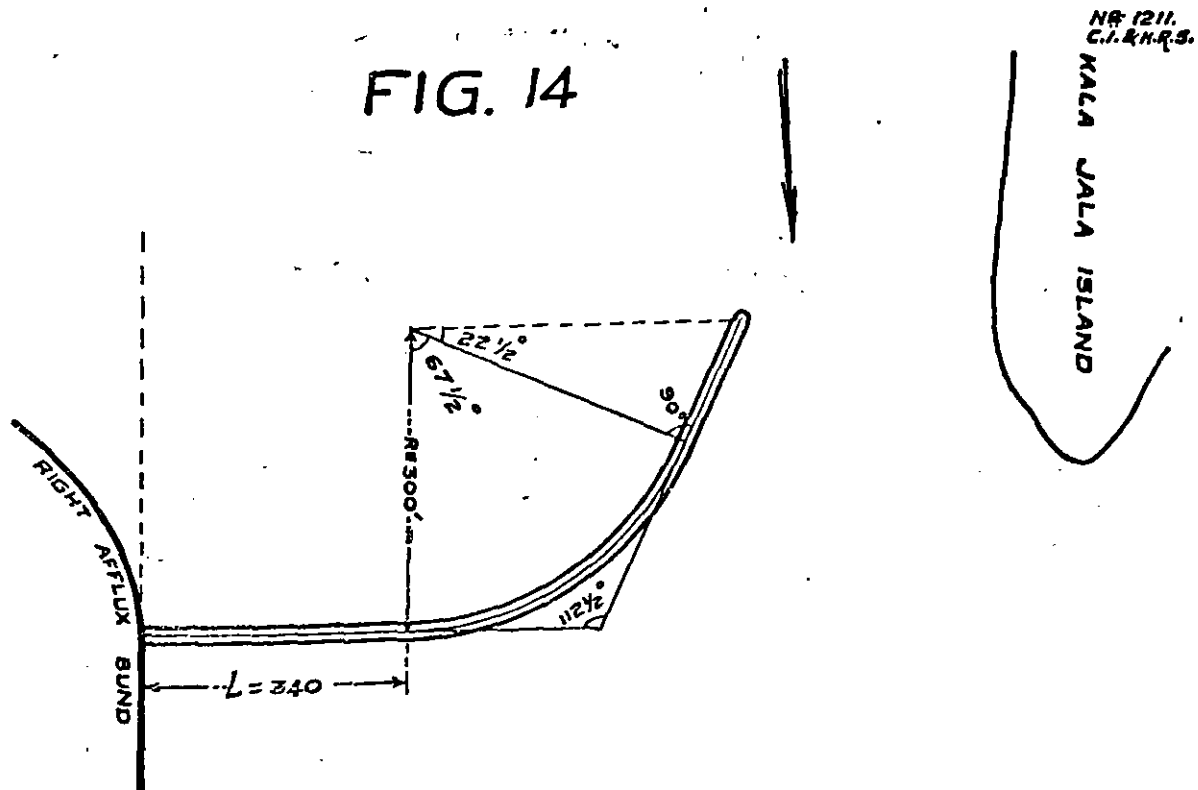
Eastern Channel.—Groyne No. 1 in experiment 1 led to local scour and concentrated flow along its nose, which caused an increase of flow in the Eastern Channel; but as this design was very sensitive and its effect hard to predict, it was not recommended. In experiment 2—without groynes—, there was a natural tendency for the main stream to flow across from the left flank at the Nepal Spur to the Kala Jala island, due to a meander swing of the river occurring at the Nepal Spur. This tendency was accelerated by adding groyne No. 2 in experiment 1. Groyne No. 2 was, however, not recommended, because even without it the natural tendency of the river under 1938-39 conditions is to swing over. Groyne No. 3 was not effective in pulling the stream towards the Western Channel on account of scoured sand from upstream depositing in the channel.

Solani Got Channel.—With the spur at the corner of the Right Afflux Bund, the bed flow was deflected away from the Canal head; whereas, without the spur, the bed water flowed along the Right Afflux Bund into the Canal. The flow upstream and downstream of the spur was slack, only the nose being subject to severe action—*vide* figure 13. Velocities along the Right Afflux Bund were reduced with the spur. With the spur, both quantity and grade of sand entering the Canal were reduced. The deep channel which, without the spur, formed along the Right Afflux Bund upstream of the pocket, was away from the right bank with the spur added. Thus the short spur

- (i) protected the Right Afflux Bund by diverting the high velocity flow away from it;
- (ii) reduced silt entering the canal; and
- (iii) improved flow conditions by diverting the main channel outside the divide wall.

Although experiment 1 of Series VII did not indicate any appreciable tendency for the spur to cause embayment upstream, it had been foreseen that this might happen and could be neutralised. Further experiments are in progress.

Series VII (Experiment 3).—In this experiment, the design of the short spur on the Right Afflux Bund was modified to suit the construction proposed at Banbassa—*see* figure 14 and photo 1891. The stem \cong 340 ft. long



(as against \cong 300 ft.) and curved head $67\frac{1}{2}^\circ$ plus $22\frac{1}{2}^\circ$ straight (against 90° curved head of experiment 1). The spur was moved \cong 100 ft. upstream of the

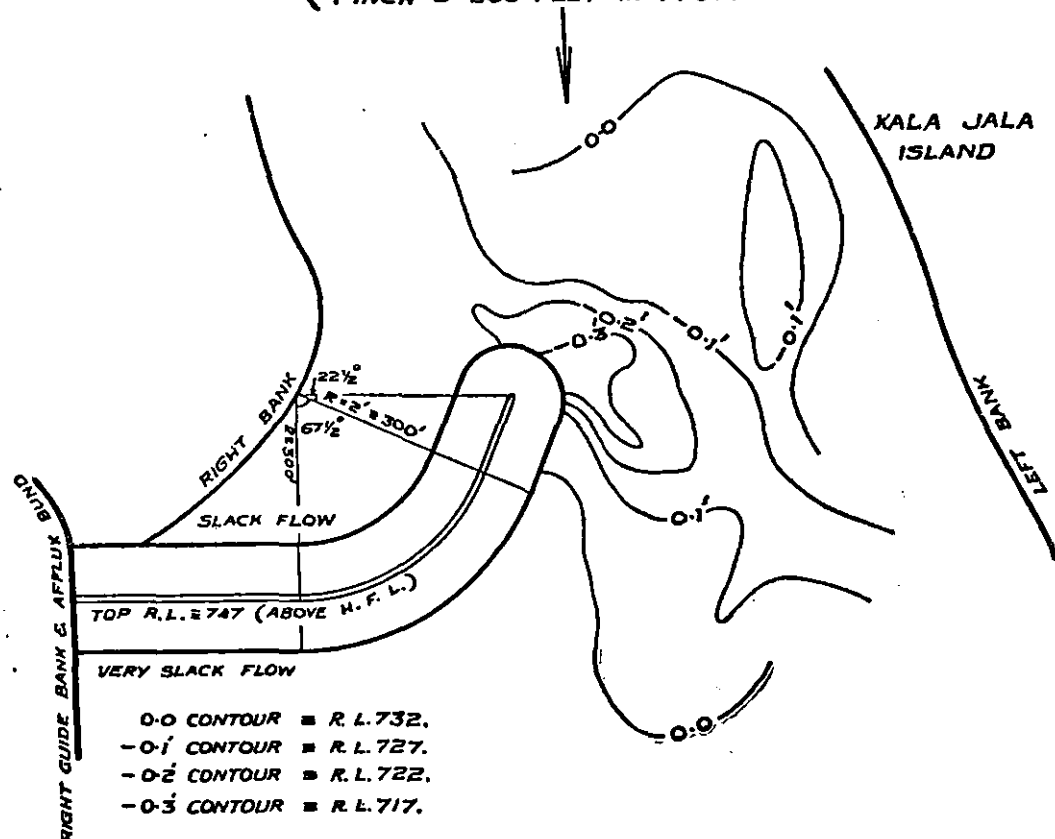
position of the spur of experiment 1. Although the effects of these changes were expected to be small, it was considered desirable to compare results with the earlier design.

The experiment was run for a period ≈ 3 years, the 5th flood of experiments 1 and 2, in which maximum $Q \approx 522,000$ cusecs, being run in the 3rd year.

The Right Afflux Bund was not attacked and the flow was very slack along its face. The canal drew surface water and the amount of sand trapped was slightly less than in experiment 1. The results were very similar to those of experiment 1.

In order to determine the amount and nature of scour round the nose, it was left without protection. Figure 15 shows scour after the high flood was run in the 3rd year.

**FIGURE 15 SHOWING SCOUR ROUND THE NOSE OF THE SPUR
(AS PROPOSED TO BE CONSTRUCTED)
AFTER FLOW ≈ 3 YEARS IN EXPT. 3 (SERIES VII)**
SCALE { 1 INCH = 2 FEET IN MODEL.
1 INCH = 300 FEET IN PROTO.



It will be seen that it was only the nose of the spur that suffered heavy attack. This will have to be strongly protected. The velocity on the upstream face of the spur was very low and no scour took place in the model.

Further experiments are in progress.

3. TRAINING THE JUMNA RIVER AT NEW DELHI.

This problem was described in the Report for 1937-38, to which reference may be made for a detailed history.

In brief, the Delhi Gate Pumping Station was constructed on the right bank of the river in 1922 when the river course was along that bank. The river subsequently moved across to the left bank below the railway bridge, crossing to the right again downstream of the Pumping Station, so that, in order to obtain water in the low-water season, cuts have to be made each year across the sandy bed of the river.

The 1937 course is shown in figure 16. The river is held firmly on the right bank upstream of the bridge by masonry works along the bank; and on the left bank downstream of the bridge by groynes Nos. 4 and 5, extended to form spurs with armoured heads.

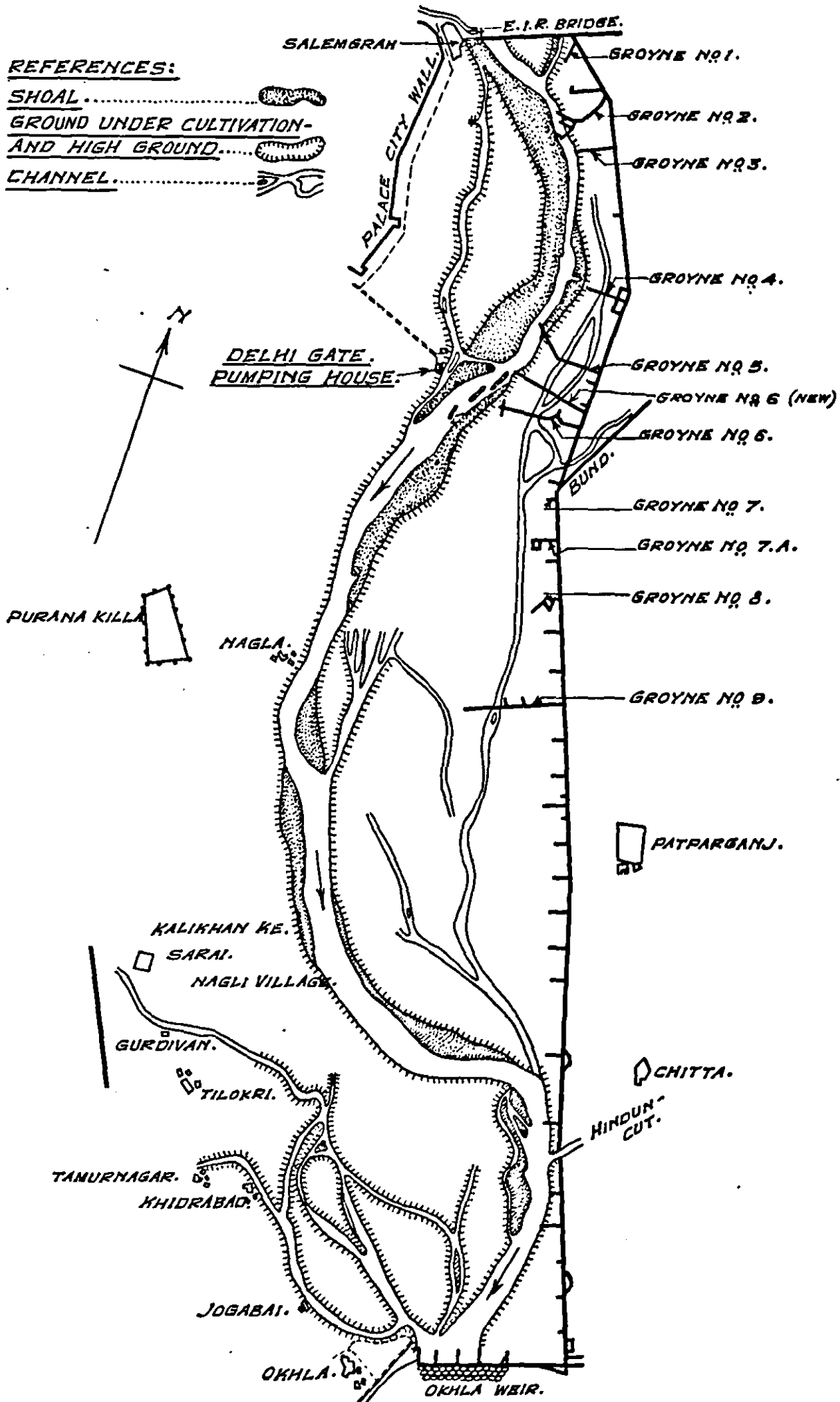
SURVEY PLAN OF -
JUMNA RIVER.
AFTER FLOODS OF 1937.
SCALE. 8 INCHES = 6 MILES.

NO. 1227.
C.I. & H.R. STN.
FIG: 15

OR
1 INCH = 3960 FT.

REFERENCES:

- SHOAL.....
- GROUND UNDER CULTIVATION- AND HIGH GROUND.....
- CHANNEL.....



To induce the river to cross to the right bank upstream of the Pumping Station, it was considered that the removal of groyne 4 would be necessary, to enable the river to form an embayment upstream of groyne 5 and swing across to the Pumping Station; also that groyne 6 should be removed as it tended to hold the river on the left bank.

The effect of this was tested in Series I of the experiments, with a model of 1/200 horizontal scale and 1/27 vertical scale, as described in the Report for 1937-38. It was shown that the removal of groynes 4 and 6 was insufficient to induce the river to cross upstream, and an extension of groyne 5 was thought necessary in addition.

At this stage the model was lengthened to include an additional 3 miles downstream and the experiments were repeated under the improved conditions. Instead of lengthening groyne 5, however, an alternative was tested, comprising the construction of a short spur on the right bank near the Pumping Station, the object being to 'pull' the river across to that bank.

As before, the model was laid according to the 1936 survey, and 3 floods were run in succession, representing 50,000, 100,000 and 50,000 cusecs, followed by a steady discharge, equivalent to 1,000 cusecs, to form the cold-weather low-water channel. The discharge scale was 1/28,000 and time scale 1/40, based on the Lacey formulæ, but the discharge scale was increased for lower discharges. Silt was injected as usual.

Figure 17 shows the lines of flow

- (a) with the present arrangement of groynes,
- (b) with groynes 4 and 6 removed, and
- (c) like (b) but with the addition of a short spur on the right bank near the Pumping Station.

It will be seen that, in spite of some embayment upstream of groyne 5, the model did not indicate any appreciable improvement as a result of removing groynes 4 and 6. This was because the embayment which formed was insufficient to create adequate curvature of surface flow. In (c), however, the surface flow was drawn to the right bank by the action at the short spur.

The low-water channels after 3 floods are shown in figure 18. As the experiments were essentially comparative, the results with (a) groynes 4 and 6 removed and (b) with the addition of the short spur, are shown superimposed on the result obtained in experiment 3, with the existing training works. Whereas in (a) two channels formed downstream of groyne 5, one near each bank, in (b) there was only one channel on the right bank brushing the Pumping Station.

It is likely that the reason why the removal of groynes 4 and 6 alone was insufficient to cause an earlier translation of the river was the already short meander, from the bend above the bridge, as a result of which the embayment above groyne 5 was insufficient to throw the surface flow across to the right bank. Instead, the bed flow was thrown across. Only with the assistance of the short spur near the Pumping Station was the surface flow induced to cross.

It is to be noted, however, that the greater the degree of vertical exaggeration, the greater is the influence of groynes and curvature of flow on a river course, owing to the region of scour at the noses of the groynes being far more extensive in plan than it would be in the prototype, to the horizontal scale.

Due to this, not only was the low-water channel downstream of the groyne wider than would be found in the prototype, the results therefore appearing better, but the small spur near the Pumping Station exerted more influence on the river.

To study this effect the model was re-constructed with a vertical exaggeration of only half what it was in the first 3 series, the vertical scale being reduced from 1/27 to 1/54 while the horizontal scale remained unchanged at 1/200. The discharge scale was also reduced, being 1/80,000 for high floods, against 1/28,000, and the timescale, being dependent on the velocity scale, became 1/27, against 1/40.

Figure 19 shows the low-water channel resulting with the existing training works after reduction of exaggeration (Series IV), superimposed on that obtained

in the previous experiments (Series III). Both flood channel and low-water channel downstream of the spurs were narrower and in some ways bore greater resemblance to the channels of the prototype.

Further experiments with this model are in progress.

4. TRAINING THE WATRAK RIVER TO PROTECT KAIRA TOWN, GUJARAT (BOMBAY PRESIDENCY).

The problem and the two first series of these experiments were described in the 1937-38 Annual Report.

In brief, erosion has been taking place at a sharp bend of the river just upstream of Kaira Town, see figure 20, the N. W. corner of the town wall was undermined and collapsed into the river together with some houses, and there was every indication that the erosion would extend. Groynes constructed on the concave bank had been washed away.

The primary object was to protect the corner of the town, but it was also important to deflect attack away from the west wall of the town for its whole length, and, if practicable, to reduce further erosion of the river upstream.

Independently of the above, a high-level causeway had been proposed at site C (fig. 20) and later a bridge at site B, but neither had been constructed.

Series I and II.—The experiments described in the 1937-38 Report were carried out with two models: Series I in a vertically exaggerated model, scale 1/125 horizontally and 1/25 vertically, in which the effects on the river course of spurs of various designs, located at the N. W. corner of the town, were tested; and Series II in a geometrically similar model, of scale 1/30, embracing a shorter length and part-width of river, to show the flow and local scour around certain spurs, particularly sloping spurs which could not be tested satisfactorily in the vertically exaggerated model.

Figure 21 shows the main surface flow under present conditions (experiment 1) compared with the flow after constructing a Burma type spur at the corner of the town (experiment 4), which deflected the high velocity current away from the town wall.

The design of spur selected as most suitable was that of Series II, experiment 3, a sloping spur pointing 35° upstream, shown in figure 22. This was as effective as the Burma spur, but was sweeter in action and would be cheaper to construct.

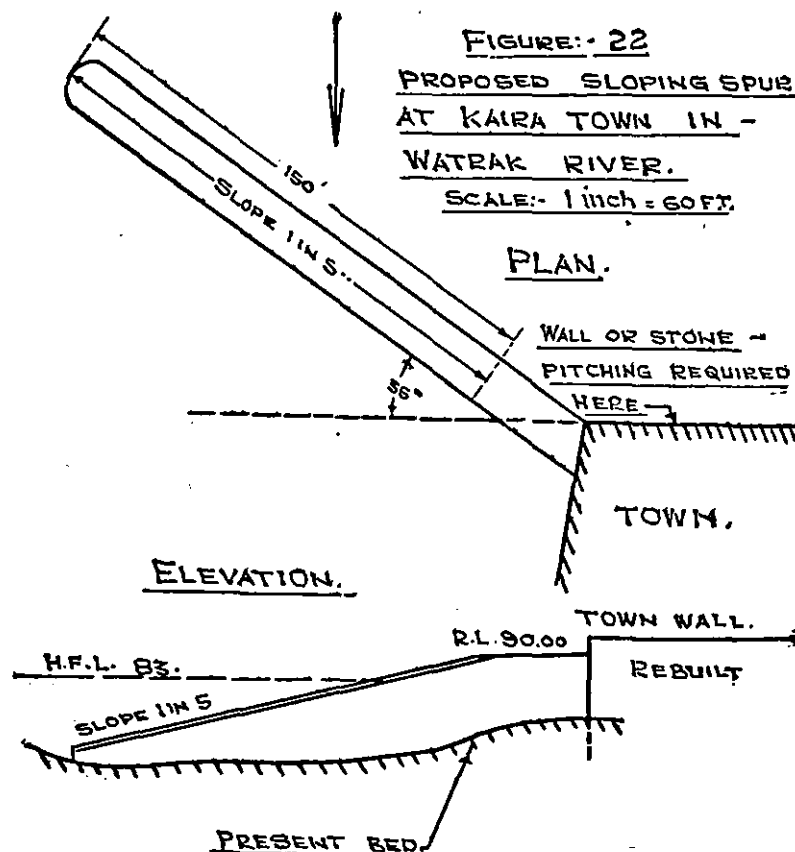


FIG. 17 - MODEL OF JUMNA RIVER AT DELHI. SERIES III. DIRECTION OF SURFACE & BED FLOW. Q = 3.6 CU SECS = 100,000 CU SECS. SCALE OF MODEL: 1/200 HOR. 1/27 VERT. SCALE OF DRAWING 1 INCH = 6 FT IN MODEL. 1 INCH = 1200 FT IN PROTO.

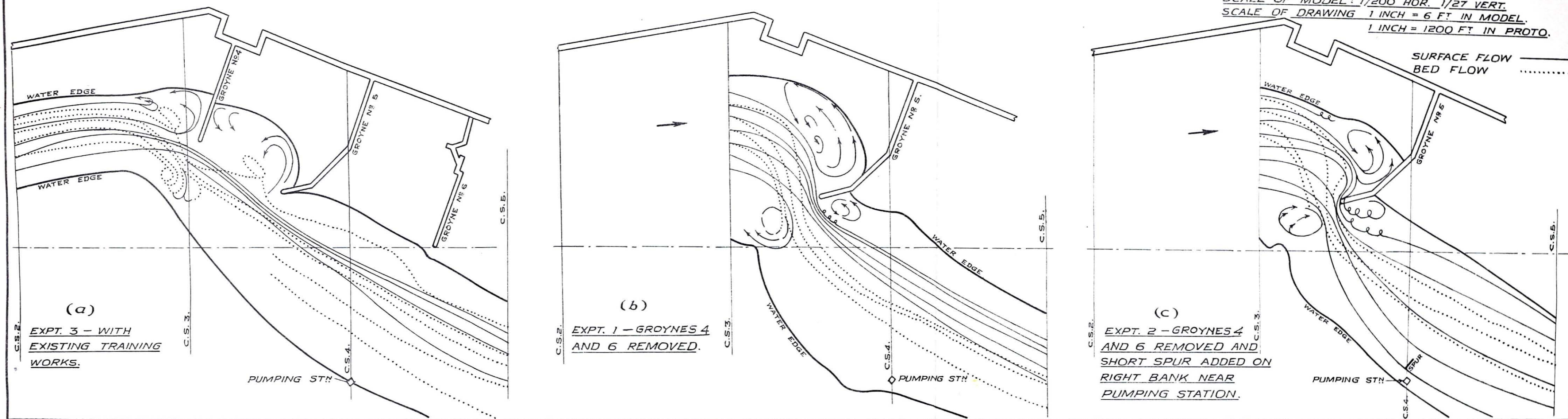


FIG. 18 - MODEL OF JUMNA RIVER AT DELHI. SERIES III. LOW-WATER CHANNELS AFTER 3 FLOODS. SCALE OF MODEL: 1/200 HOR. 1/27 VERT. SCALE OF DRAWING 1 INCH = 6 FT IN MODEL. 1 INCH = 1200 IN PROTO. CHANNELS ARE SHOWN SUPERIMPOSED ON CHANNELS OBTAINED WITH EXISTING TRAINING WORKS.

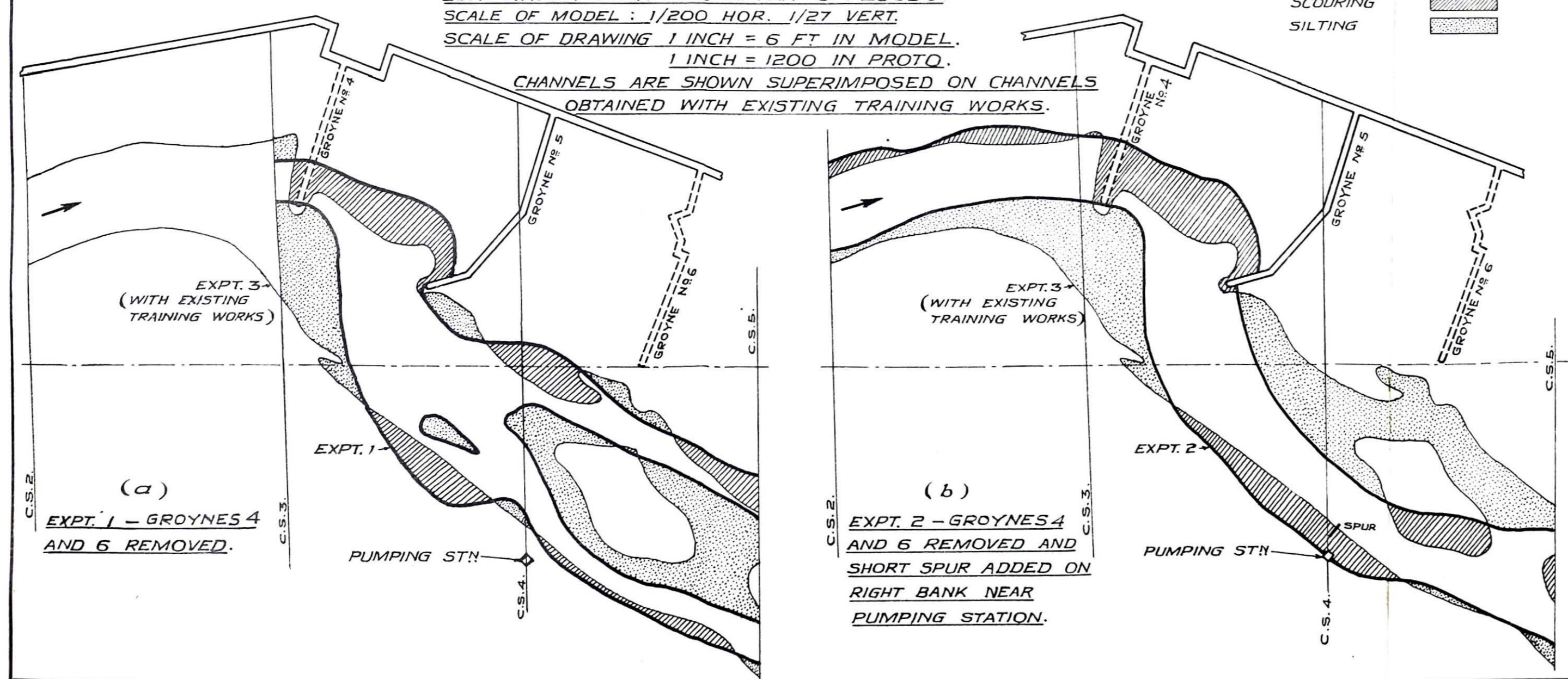


FIG. 19 - MODEL OF JUMNA RIVER AT DELHI. LOW-WATER CHANNELS AFTER 3 FLOODS, COMPARING SERIES IV, EXPT. 1 WITH REDUCED VERTICAL EXAGGERATION WITH SERIES III, EXPT. 3 (BOTH WITH PRESENT TRAINING WORKS.)

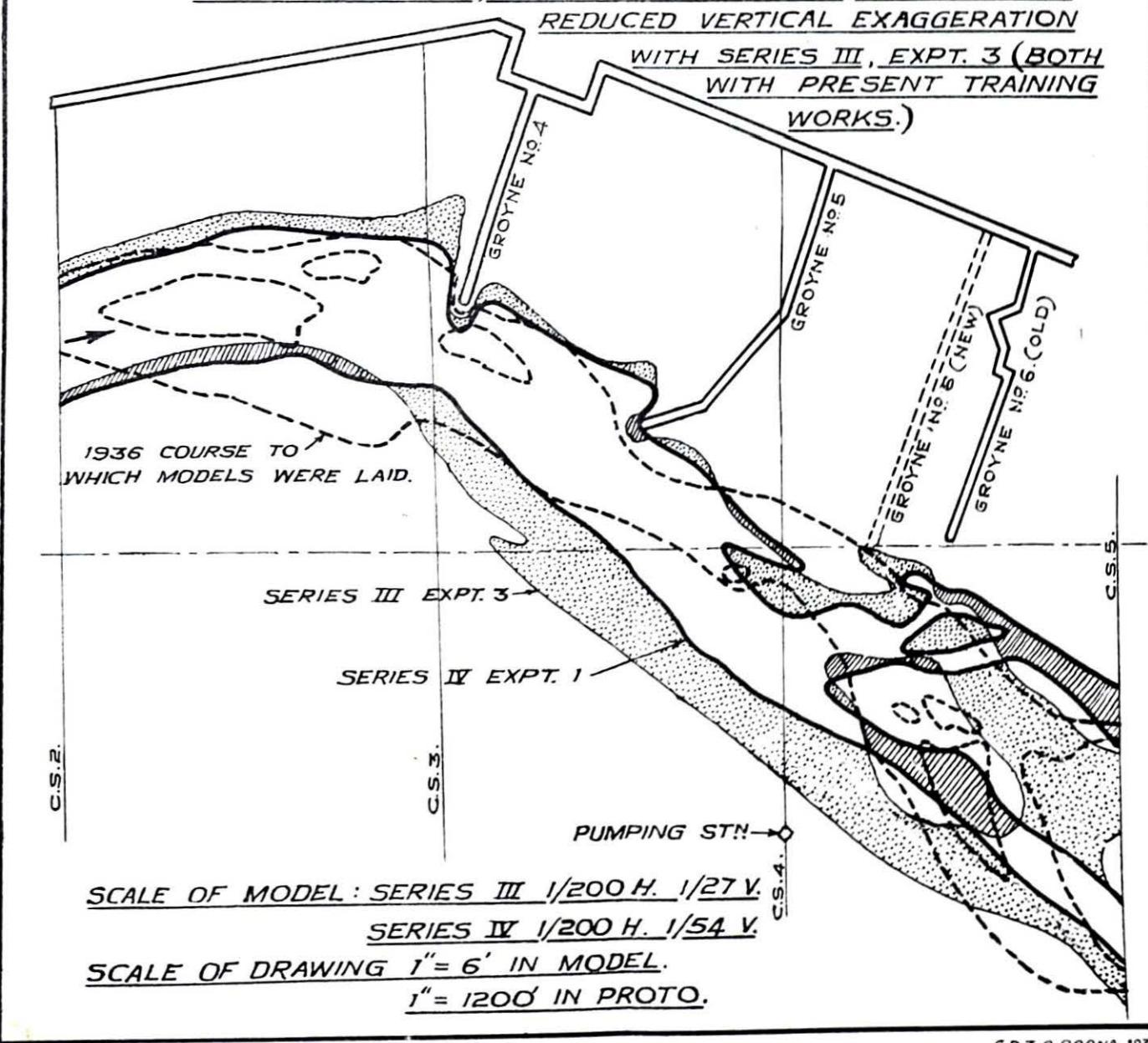
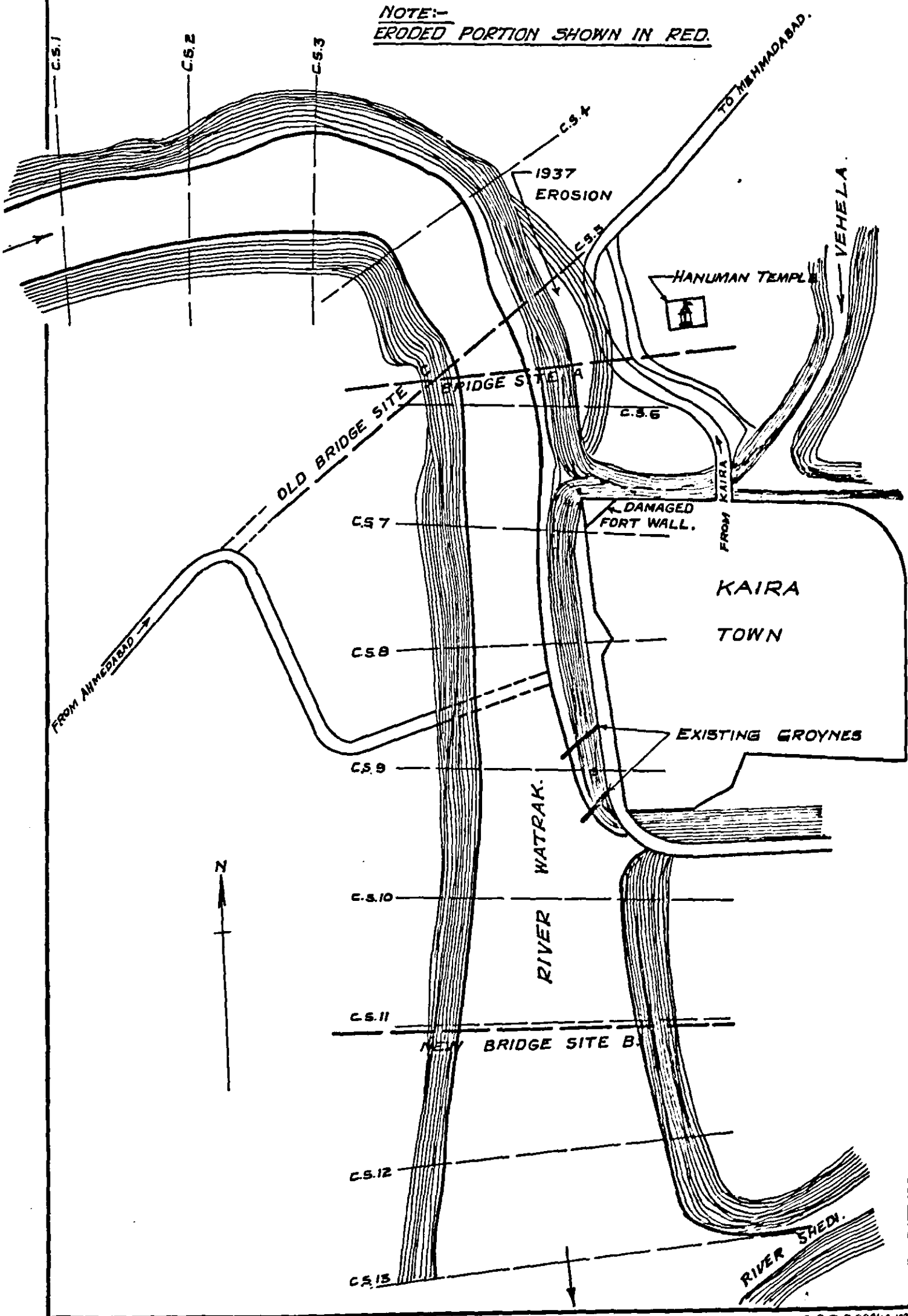


FIGURE.20
WATRAK RIVER.
NEAR KAIRA TOWN.
SCALE: 8" = 1 MILE.

NOTE:-
ERODED PORTION SHOWN IN RED.



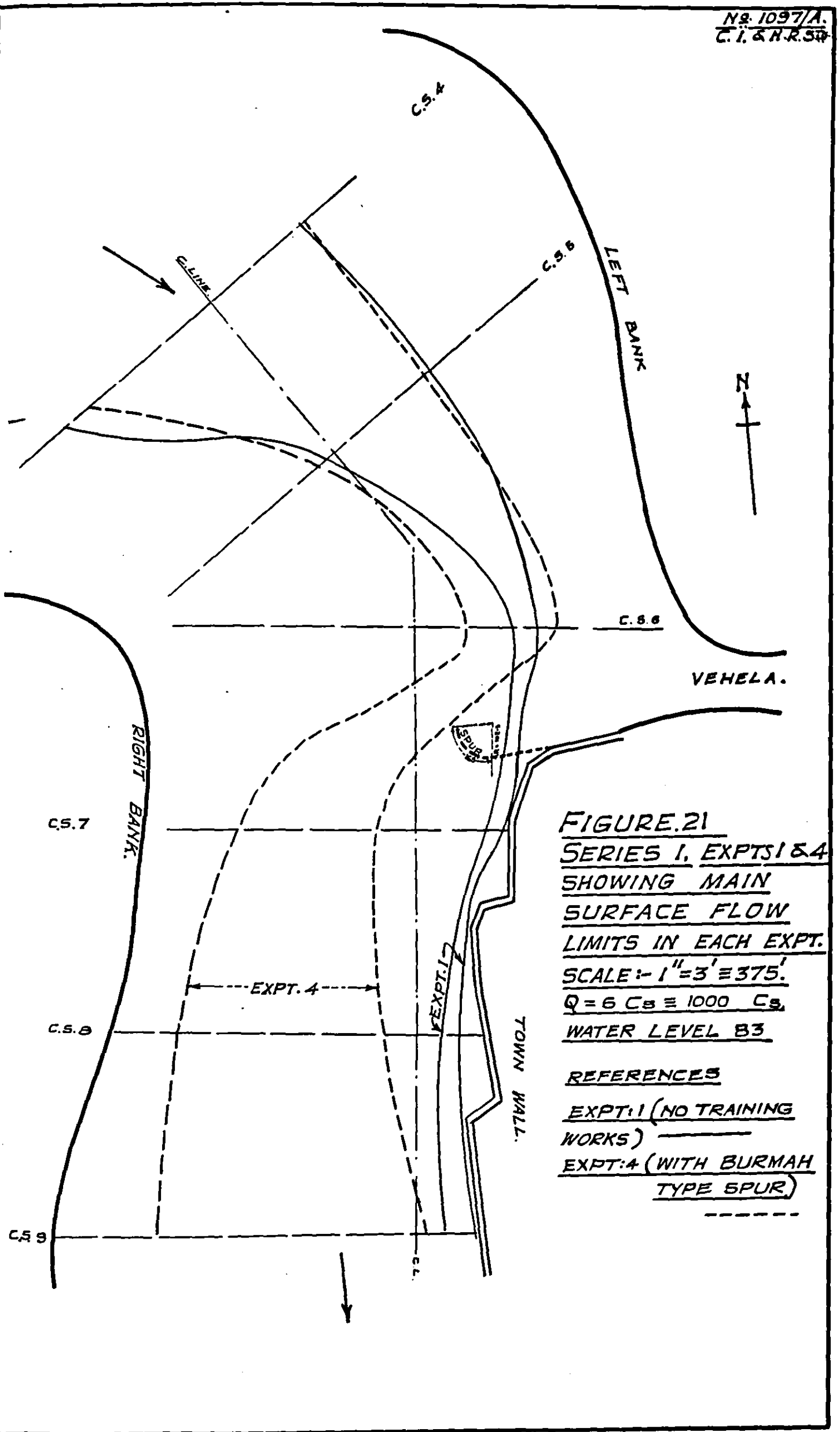


FIGURE. 21
SERIES I, EXPTS 1 & 4
SHOWING MAIN
SURFACE FLOW
LIMITS IN EACH EXPT.
SCALE:- 1" = 3' = 375'
Q = 6 C_B = 1000 C_s
WATER LEVEL B3

REFERENCES
EXPT. 1 (NO TRAINING
WORKS) ———
EXPT. 4 (WITH BURMAH
TYPE SPUR) - - - - -

Experiments now described.—In view of the intention of bridging the river, however, it was desirable to see in what way a causeway or bridge could be designed to train the river in the required manner.

Experiments with this object were carried out—Series III to V in the vertically exaggerated model and VI in the geometrically similar model. Series VII was carried out in the former to show the probable future erosion upstream, if the N. W. corner of the town were rebuilt and strengthened to resist erosion but nothing were done to train the river. This series will be described first.

Series VII: Results without river training.—In this experiment, carried out in the vertically exaggerated model, the corner of the town wall was rebuilt and the corner and north wall were protected against deep scour. The concave bank upstream of the town was formed of loose silt to allow rapid free scour.

A series of similar flood cycles were run, each reaching a peak of 6 cusecs \equiv 100,000 cusecs, and continued until bank erosion ceased, which occurred after 8 floods.

Figure 23 shows the extent of erosion, the present bank line being drawn for comparison, and the lines of flow at maximum flood.

At lower discharges the flow followed the left bank in the embayment, and, swinging around the town wall corner, was thrown away from the wall downstream. At the peak flood, however, the main flow cut across the bend and closely followed the west wall, as shown in the figure. There was severe action at the corner, to resist which, deep foundations would be required.

Moreover, it will be seen that the river cut more than 1,100 ft. into the bank. Erosion to this extent in the prototype would not only absorb a large area of land, containing a temple and a length of provincial road, but would be liable to cause encirclement of the town by an avulsion to the low ground and valley to the east.

Series III with high level causeway.—The effect of a causeway, 6 ft. above cold weather water level, was tested, locating it at various positions in different experiments. The 1 in 4 sloping ramp on the left bank (\equiv 1 in 20 in prototype) acted in a similar way to the sloping spur of Series II. Photo 1863 shows experiment 6, with causeway upstream of the N. W. corner.

Experiments 4 and 5 with the causeway at the corner gave the best results. These are not described in detail as similar experiments were done later with a submersible bridge at a high level, which was preferable from the aspect of communications.

Series IV to determine the effect of a small u/s groyne.—A groyne, 134 ft. long, had already been constructed on the left bank just upstream of the bend early in 1938, with the object of deflecting the main flow across the bend away from the town corner.

The river flow was observed in the model under existing conditions, and with high-level causeway in two positions upstream of the town, in each case with and without the groyne. Figure 24 compares the main surface flow with causeway, with and without groyne.

The experiments indicated that the effect of the groyne was undesirable but small.

Series V with submersible bridge.—This bridge was 10 ft. higher than the causeway tested in Series III and IV and would be submerged only in the highest floods. The effect of the bridge was tested both upstream of the town and at the corner, and in the former case the effect of a training wall running 250 ft. upstream from the left abutment (see figure 25) was seen.

This wall resulted in less scour upstream on the left bank and straight flow through the bridge, but the advantage was not sufficient to warrant the extra cost of construction.

The best position for the bridge, like the causeway, was found to be at the town wall corner, but pointing 30° upstream and with openings about 500 ft. from the corner. Figure 26 shows the surface and bed lines of flow at maximum flood. This may be compared with figures 21 and 23. Photo 1862 shows the river at low-water after a high flood.

With the bridge in this position some erosion would be likely on the left bank of the bend, but only a small fraction of that shown in figure 23 (with no training works), and the main river would be held away from the town wall and corner.

As bores taken in the meantime in the river showed the existence of a hard stratum underlying the silt 34 ft. below low-water level (50 ft. below bridge level), a hard bottom was placed in the model, at the corresponding level, in the later experiments of this series. This restricted the depth of scour at the bridge, which otherwise would be very considerable, and in consequence the flow downstream of the scour hole was less fanned out—more concentrated in midstream.

Series VI: Submersible Bridge in geometrically-similar model.—The best position of the bridge having been chosen by results obtained in the vertically exaggerated model, the flow was examined in detail in a 1/40 scale geometrically-similar model as a final test, and incidentally to obtain useful basic data on the comparative results of vertically exaggerated and geometrically-similar models.

The latter type, if to a sufficiently large scale, gives more correct lines of flow and scour in detail over a short length. In a long model the channel tends to assume its natural section, which would be narrower and deeper, leading to incorrect results. Moreover, the size of model having sufficient discharge per unit width (Reynold's No.) for adequate bed movement is usually so big that only a short length, and usually part width, of river can be accommodated. A vertically exaggerated model, on the other hand, is necessary where changes of river course are concerned, because the much smaller discharge of the model than of the prototype requires a much greater depth-width ratio, for similarity of behaviour, just as in Nature smaller channels are deeper relative to their width³. Also, owing to its horizontal scale being smaller than its vertical scale, a greater length of river can be accommodated without excessive reduction of Reynold's No. For many problems, such as this one, a model of each type is required.

The model of Series VI included almost the full width of the river from C. S. 4 on the bend to C. S. 10, 2,250 ft. downstream of the corner of the town (see figure 27 and photo 1873).

Experiment 1 was carried out with the submersible bridge in the position found most suitable in Series V. The maximum discharge was 9.9 cusecs, calculated to give equal Froude No., V^2/gd , to that given by the maximum discharge of 6 cusecs in the vertically exaggerated model. The discharge scale for the lower discharges was higher to induce sufficient bed-movement. In experiment 2 the maximum discharge was increased to 12.3 cusecs, the lower discharges being increased in proportion, as this was found to induce general bed-movement and yield similar depths of channel to those obtained in the exaggerated model.

Figure 27 shows the lines of flow observed at maximum discharge in this experiment. It may be compared with figure 26, the lines of flow in the corresponding experiment in the vertically exaggerated model. Figure 28 shows comparative cross-sections on the upstream and downstream faces of the bridge.

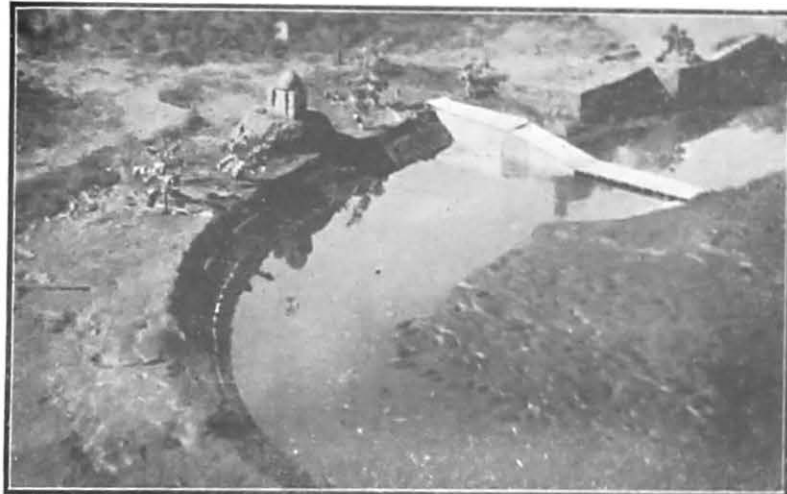
It will be seen that the lines of flow (figs. 26 & 27) though similar were not identical. The eddy upstream of the right approach was more extensive in the vertically exaggerated model and pushed the main current towards the left. The flow through the bridge was therefore skew and the main current downstream was thrown more over to the right bank.

The vertically exaggerated model was more correct in the lines of approaching flow, correct entry conditions not being obtained in the geometrically-similar model due to the shortness of the model, but the eddy in the former was too large. The cross-sections are approximately the same in both (fig. 28).

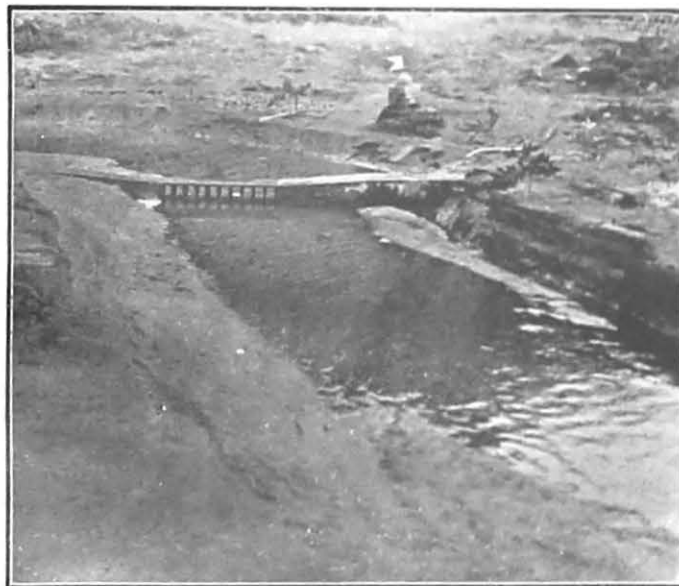
Experiments 3 and 4 were carried out with a high level bridge, clear of all floods, placed in the same position. There was, of course, no difference in results except at very high floods, when owing to the concentration of surface flow through the openings, the flow through the bridge was slightly skew towards the town, and the main river downstream therefore nearer the left bank.

³ "G. Lacey": "Stable Channels in Alluvium." Proceedings, Institute of Civil Engineers, 1930.

EXPERIMENTS WITH A MODEL OF WATRAK RIVER NEAR KAIRA (BOMBAY).

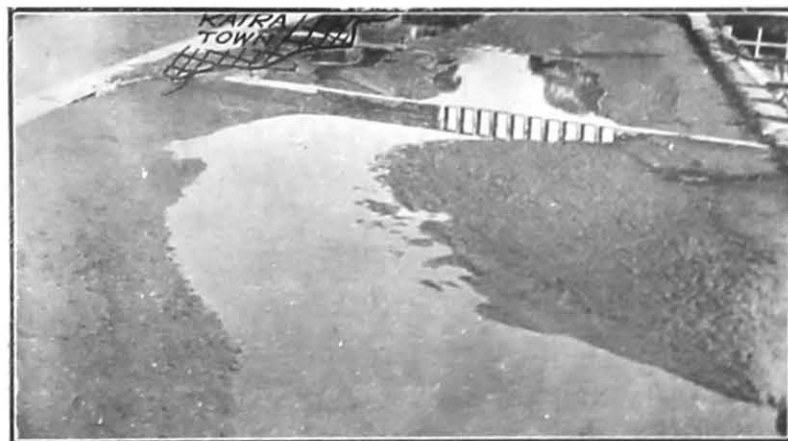


1863. Showing vertically exaggerated model with causeway upstream of mouth of Vehela. Silting downstream may be seen (Series III, Expt. 6) looking downstream.



Scale ratios:
1/125 horizontal,
1/25 vertical.

1862. Showing vertically exaggerated model with submersible bridge at town wall corner directed 30° upstream—recommended position—silt bank may be seen along the town wall (Series V, Expt. 9) looking upstream.



1873. Showing the 1/40 scale geometrically similar model with high level bridge at corner of town. (Series VI, Expt. 4) looking downstream.

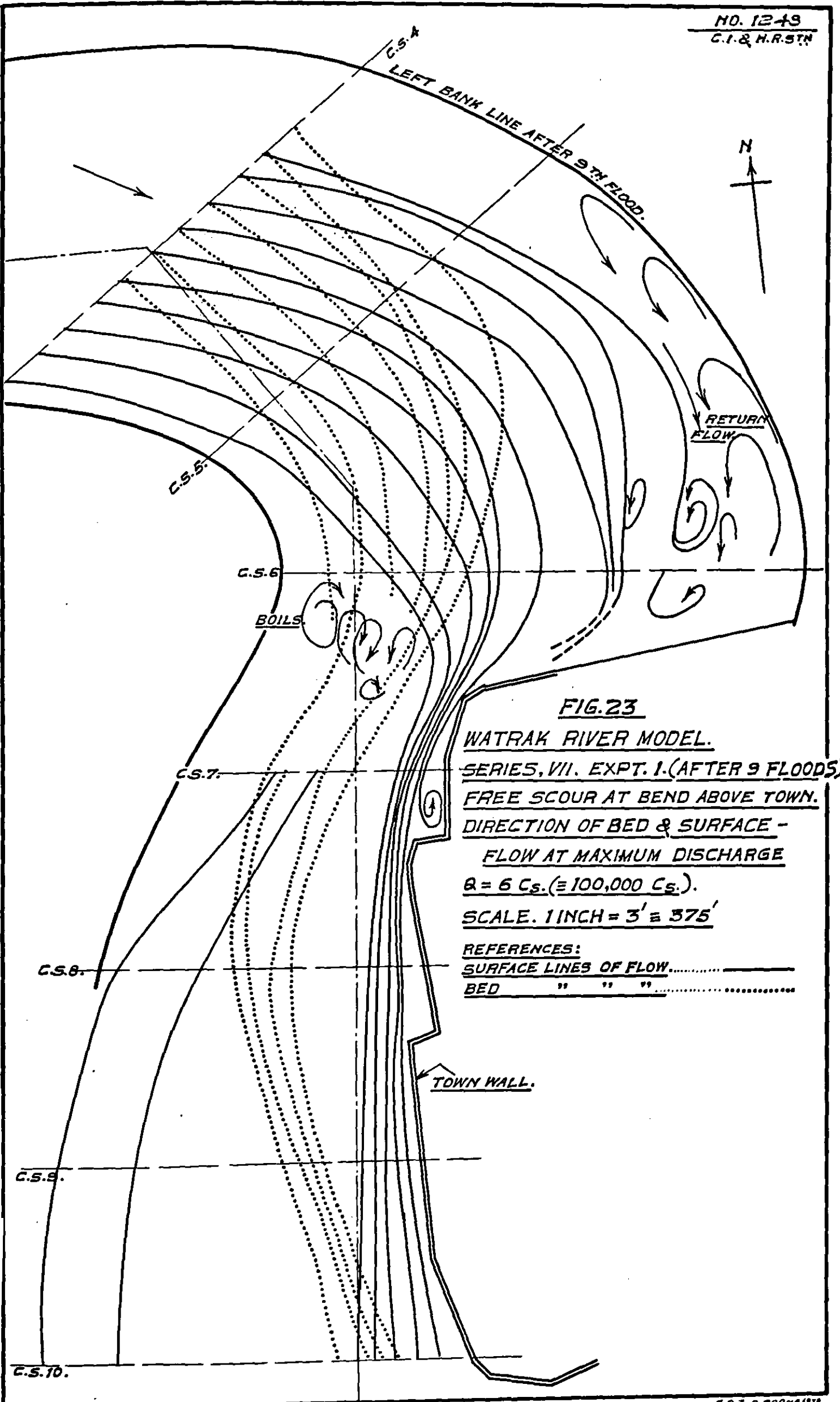


FIG. 23
WATRAK RIVER MODEL.
SERIES, VII. EXPT. 1. (AFTER 9 FLOODS)
FREE SCOUR AT BEND ABOVE TOWN.
DIRECTION OF BED & SURFACE -
FLOW AT MAXIMUM DISCHARGE
 $Q = 6 C_s. (\approx 100,000 C_s.)$
SCALE. 1 INCH = 3' \approx 375'
REFERENCES:
SURFACE LINES OF FLOW.....
BED " " ".....

FIGURE.24

NR 1100/E.
C.T. & H.R. 5/57

WATRAK RIVER MODEL.

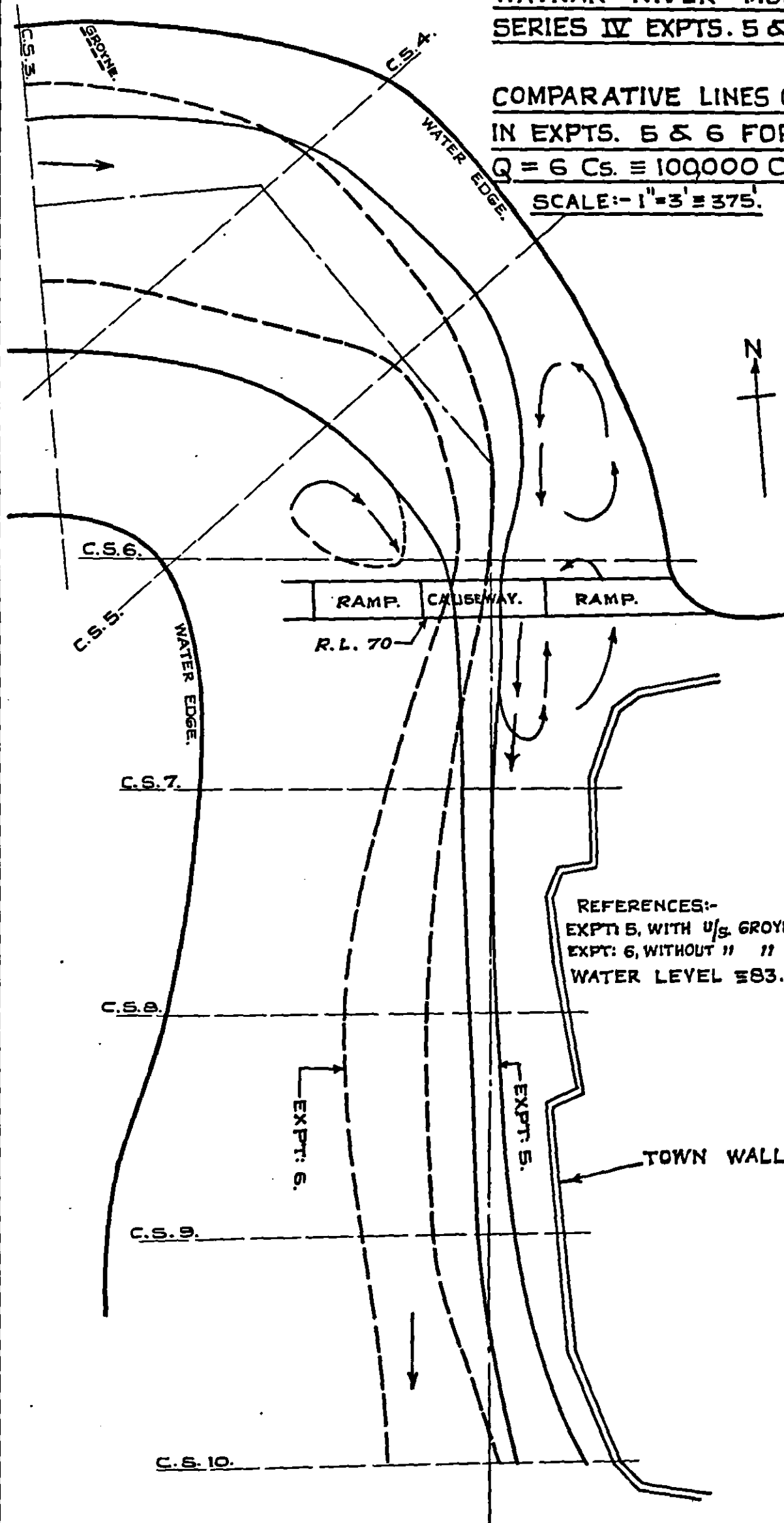
SERIES IV EXPTS. 5 & 6

COMPARATIVE LINES OF FLOW

IN EXPTS. 5 & 6 FOR

$Q = 6 \text{ Cs.} \approx 100,000 \text{ Cs.}$

SCALE:- 1" = 3' = 375'



REFERENCES:-
 EXPT. 5, WITH U/S GROUYNE. ———
 EXPT. 6, WITHOUT " " ———
 WATER LEVEL ≈ 83 .

EXPT. 5.
 EXPT. 6.
 TOWN WALL.

FIGURE 25.
WATRAK RIVER MODEL
SERIES V, EXPTS: 1 & 3.
COMPARATIVE LINES OF FLOW IN EXPTS: 1 & 3.
FOR $Q = 6 Cs. \approx 100,000 Cs.$
SCALE: $-1" = 3' \approx 375'$
WATER LEVEL $\approx 80.$

REFERENCES:-
EXPT: 1, WITHOUT TRAINING WALL. ———
EXPT: 3, WITH TRAINING WALL. - - -

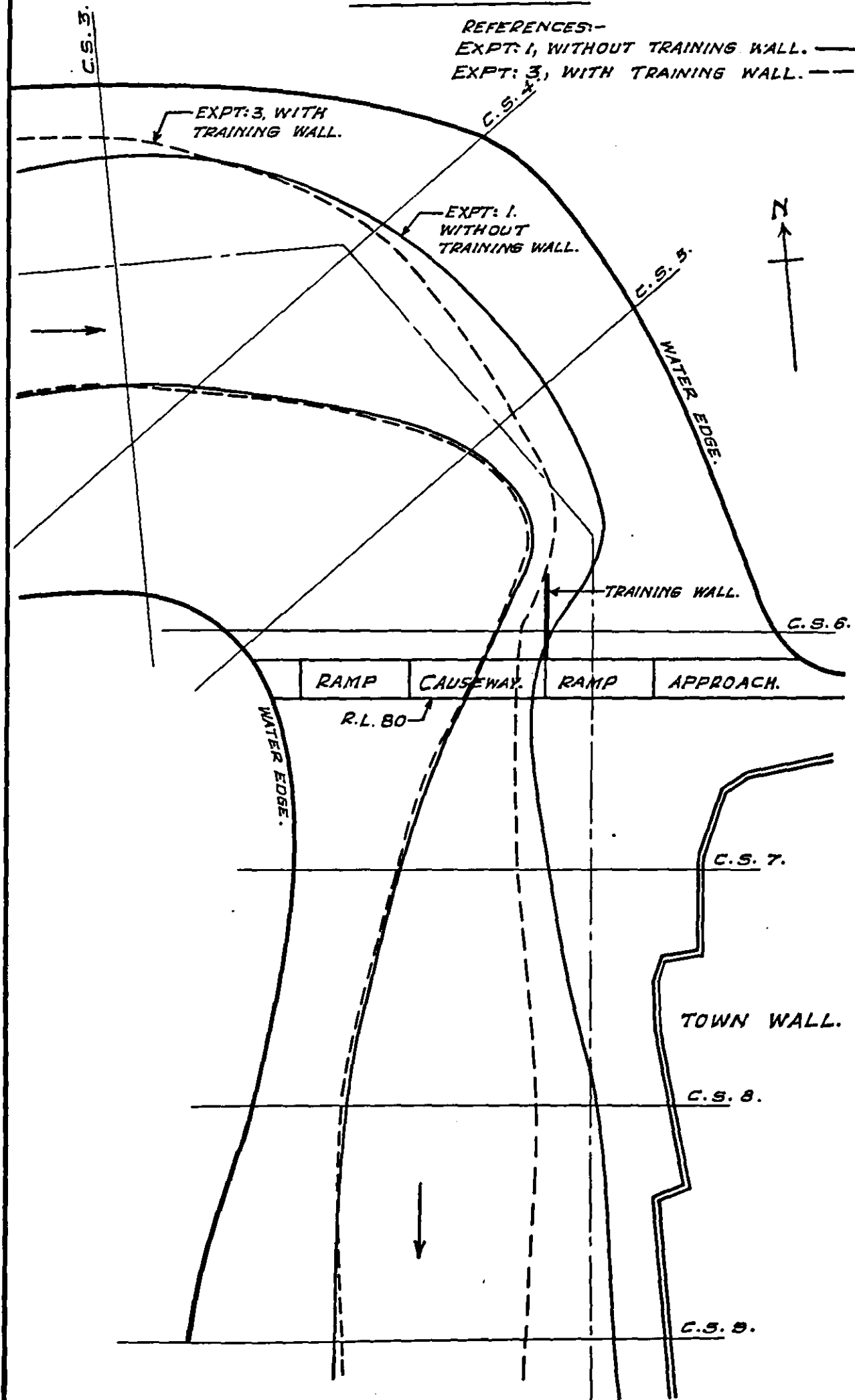


FIGURE. 26.
WATRAK RIVER MODEL
SERIES V, EXPT: 8.
WITH SUBMERSIBLE BRIDGE AT N.W. CORNER
SHOWING DIRECTION OF BED & SURFACE FLOW
WITH $Q = 6$ Cs. $\approx 100,000$ CUSECS.
SCALE:- $1'' = 3' \approx 375'$

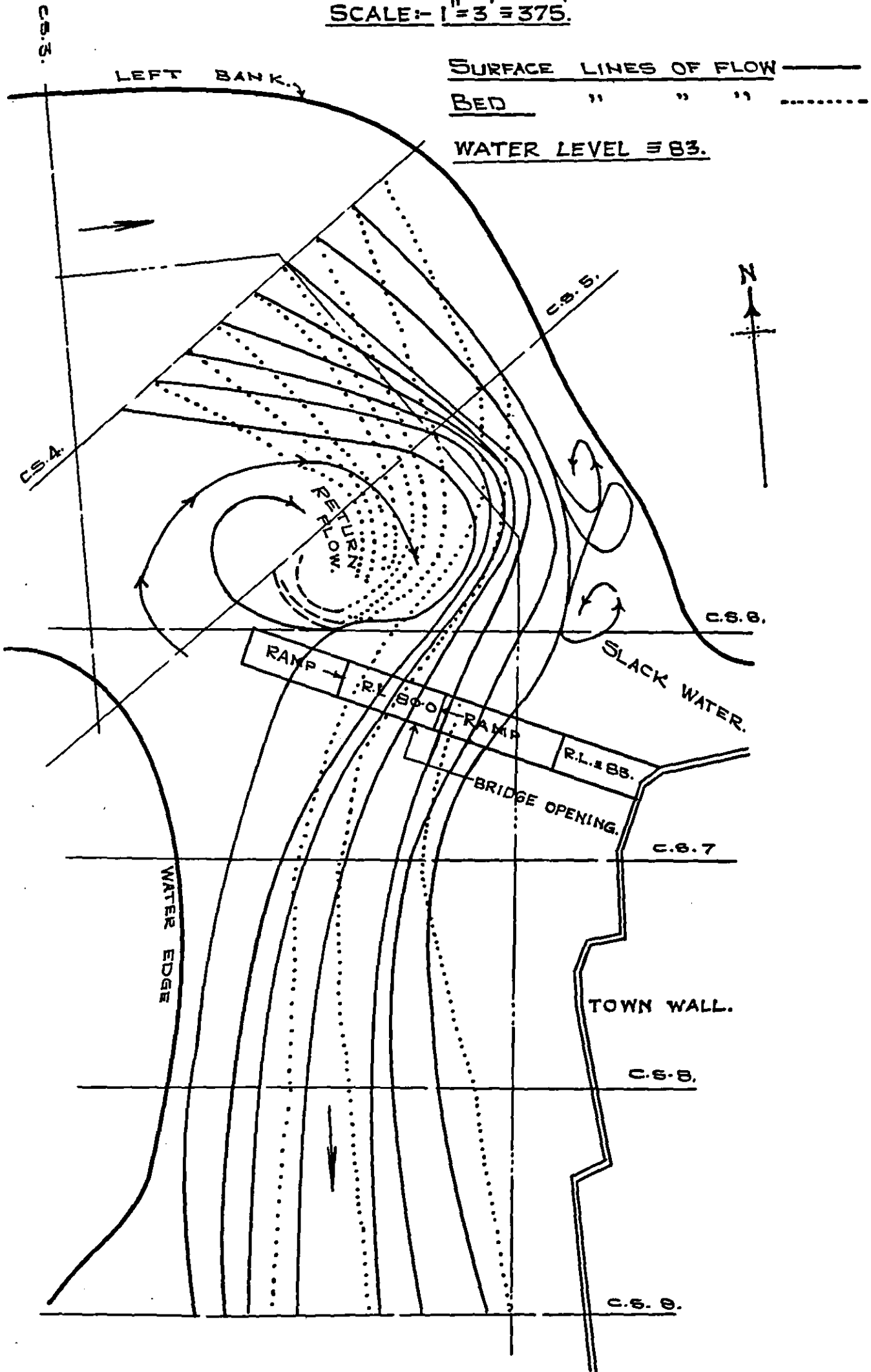


FIGURE 27.
1/40 WATRAK RIVER MODEL (GEOM. SIMILAR)
SERIES VI EXPT: 2.
WITH SUBMERSIBLE BRIDGE
AT TOWN WALL CORNER FACING 30°^U/s.
SHOWING DIRECTION OF BED & SURFACE FLOW
WITH Q = 12.3 Cs. ≅ 100,000 Cs.
SCALE: -1" = 9.3' ≅ 375'.

SURFACE LINES OF FLOW. ————
BED " " "
WATER LEVEL. ≅ 83.

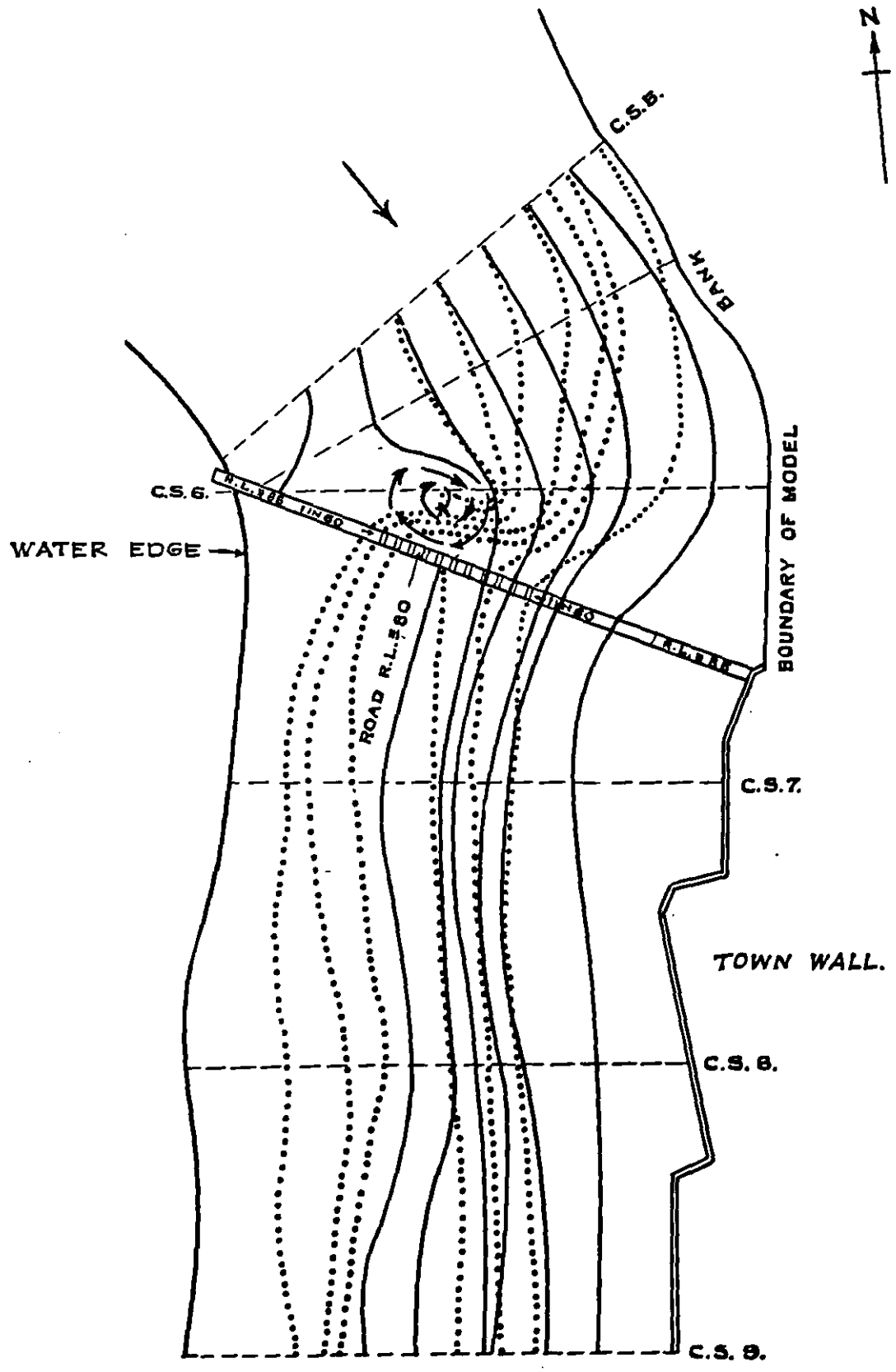


FIG. 28.

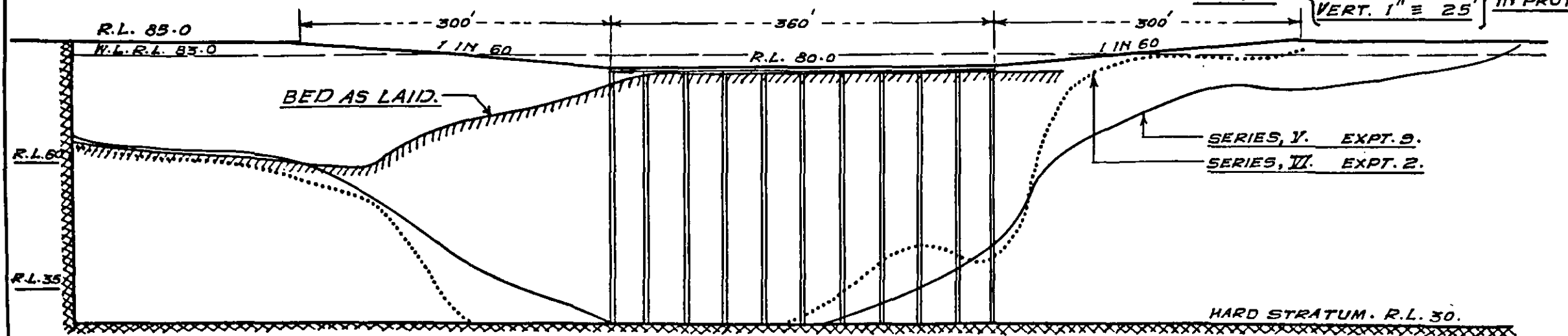
WATRAK RIVER MODELS.

SHOWING COMPARATIVE CROSS SECTIONS.

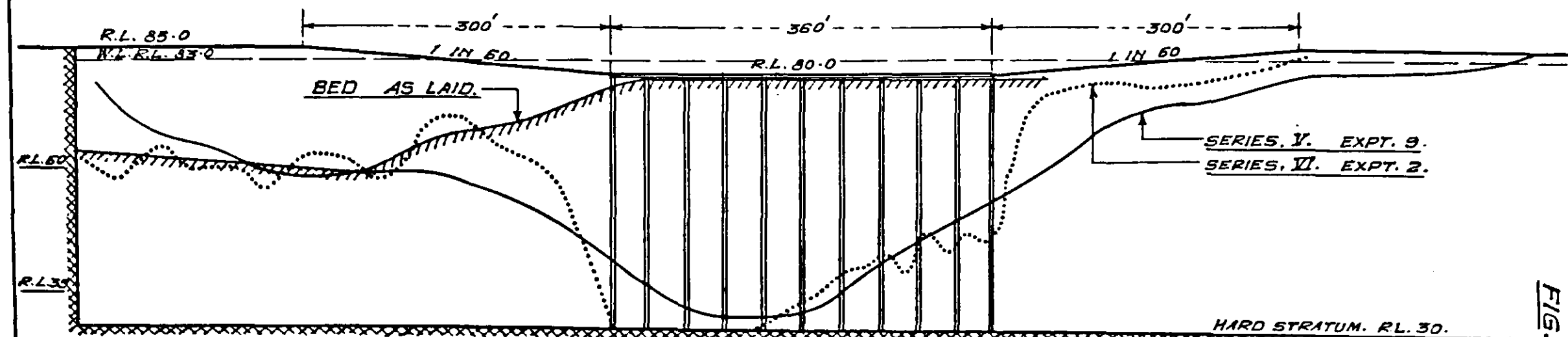
Q. = 100,000 C.S.

- REFERENCES:-
 SERIES, V. EXPT. 9.....
 SERIES, VII. EXPT. 2.....
 AS LAID (1937.).....

SCALE. { HOR. 1" = 125' }
 { VERT. 1" = 25' } IN PROTO.



C.S. ON U/S FACE OF BRIDGE.



C.S. ON D/S FACE OF BRIDGE.

NO. 1241.
 C.I. & H.R. SYN.
 FIG. 28.

Conclusions and Recommendations—

There were three alternatives :—

- (i) to construct the bridge in the best position for protecting the town and training the river *i.e.*, from the N. W. corner of the town (fig. 26),
- (ii) to construct the bridge below the town
 - (a) protecting the town by a spur, 180 ft. long, sloping down at 1 in 5 (fig. 22) or
 - (b) deepening and strengthening the corner of the town wall.

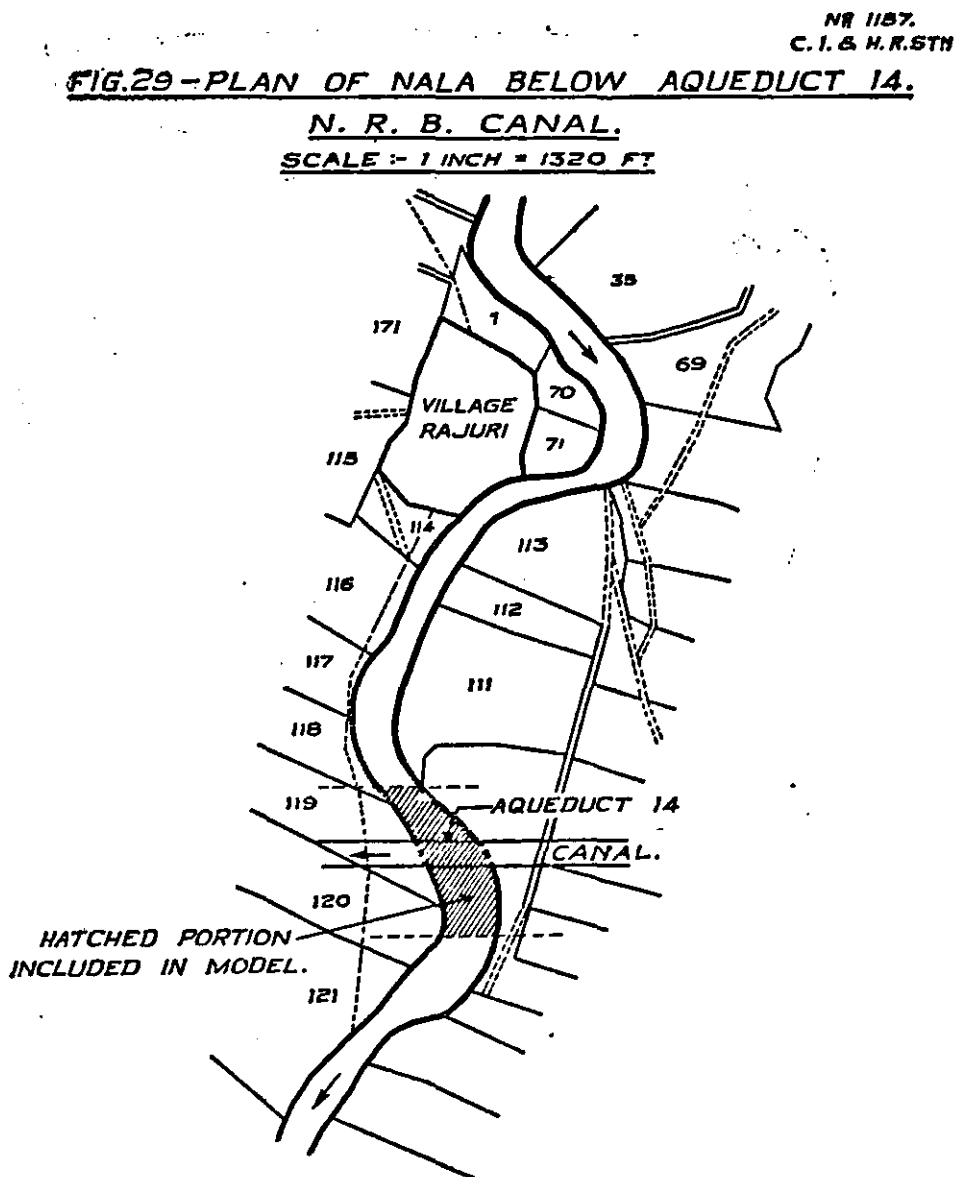
It was pointed out that from the protection point of view (i) was much the best, but much more costly than (ii) (a), and would locate the bridge at a less satisfactory point in respect of communications than in (ii) (a) and (b). (ii) (b) while costing least in outlay had obvious objections as described earlier.

The Government of Bombay have decided to adopt (ii) (a), the sloping spur.

5. EXPERIMENTS WITH A MODEL OF THE RAJURI NALA AND AQUEDUCT 14—NIRA RIGHT BANK CANAL (BOMBAY PRESIDENCY).

Introduction.

The Rajuri Nala flows through Aqueduct 14 from the right flank at an angle of about 45° —*vide* figure 29.



The right upstream wing wall was originally as shown in figure 30 in full line, but was extended as shown in chain-dotted line to protect the toe of the canal bank, the extension consisting of a dry stone wall with pointed joints.

The toe of this extended wall which was exposed to the main channel—as may be seen in photo 1826—collapsed during the severe floods of 1937 and the original object of the experiments was to determine the cause of the failure and what protective measures were necessary. It was presumed that the failure was due to undermining by the current, but when the model was run, it was obvious what was the real cause: During the flood, the wing wall was overtopped by $2\frac{1}{2}$ ft. and the backing had evidently burst through due to the back pressure of the entrapped water.

The remedy for this is, of course, to raise the wing wall above H. F. L., also making provision for the escape of seepage water.

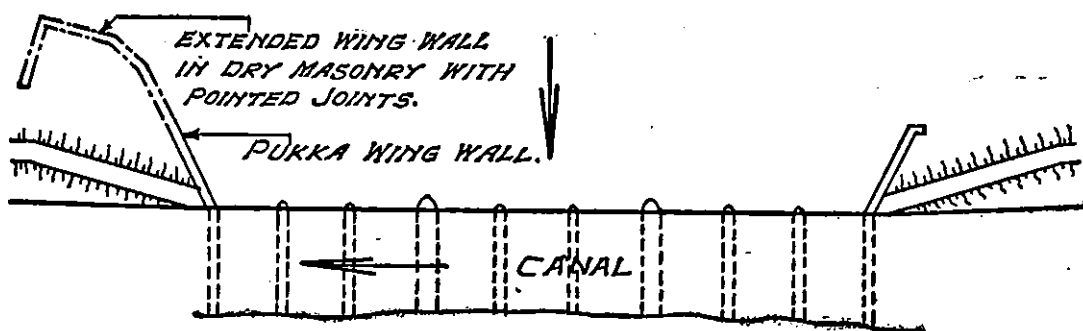
It was found from the model that, though scour was negligible, afflux was considerable, due to the skew flow of the Nala towards the aqueduct. Experiments were, therefore, done to see how far the afflux could be reduced by straightening the flow approaching the aqueduct.

FIG. 30

NO. 1165.
C. J. & H. R. STY.

PLAN SHOWING THE PUKKA WING WALL & THE
SUBSEQUENT EXTENSION IN DRY MASONRY.

SCALE:— $\left\{ \begin{array}{l} \text{FOR PROTO } 100' = 1'' \\ \text{FOR MODEL } 4' = 1'' \end{array} \right.$



Description of the model.

The model was geometrically-similar, the scale ratio being $1/25$. The discharge constant was therefore $=25^{2.5} = 3120$. Rock in the prototype was represented by concrete with a coat of cement plaster in the model, and massive kankar—locally called 'mân'—by clay. Massive kankar consists of sharp particles of fine quartz sand cemented with lime. It contains many vesicles and is porous. The bed material used in the model was local sand, of mean diameter of 0.24 mm. ($f_d = 1.76 \text{ m}^3 = 0.86$ for régime conditions).

Experiments.

The portion of the Nala represented in the model is shown in hatched lines in figure 29. There is a bend in the Nala upstream of the aqueduct and, to impose natural flow conditions at entry to the model, various alternative entries had to be tried before correct flow conditions were obtained. Figure 31 shows the flow in the model with the existing wing wall (experiment 1). It will be seen that 2 out of 9 spans were out of action due to return flow and sand banks.

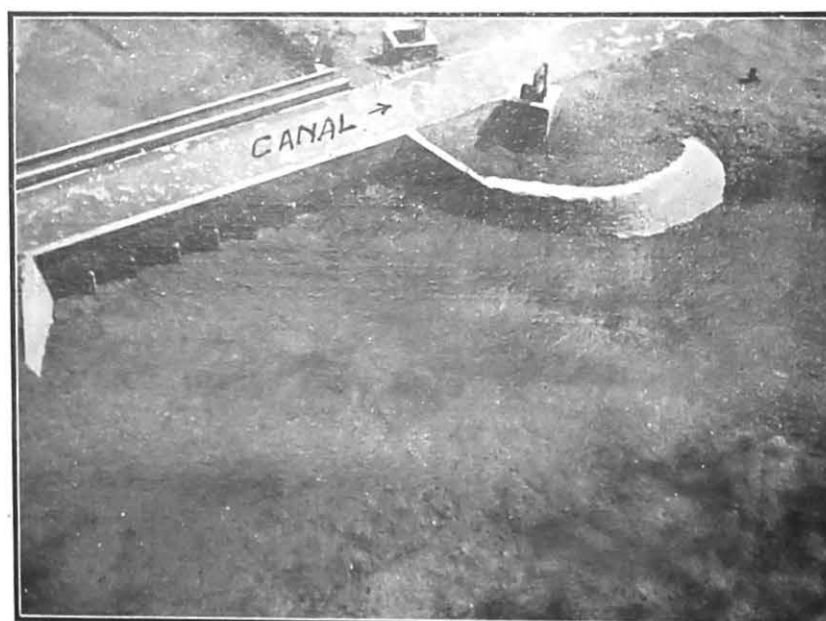
Various designs of right upstream wing wall with top above H. F. L. were then tried, the object being to create sweet flow under the aqueduct and to ensure forward flow in all spans—see figure 32.

MODEL OF THE RAJURI NALA AND AQUEDUCT 14 (BOMBAY).

Scale ratios : 1/25 Horizontal; 1/25 Vertical.



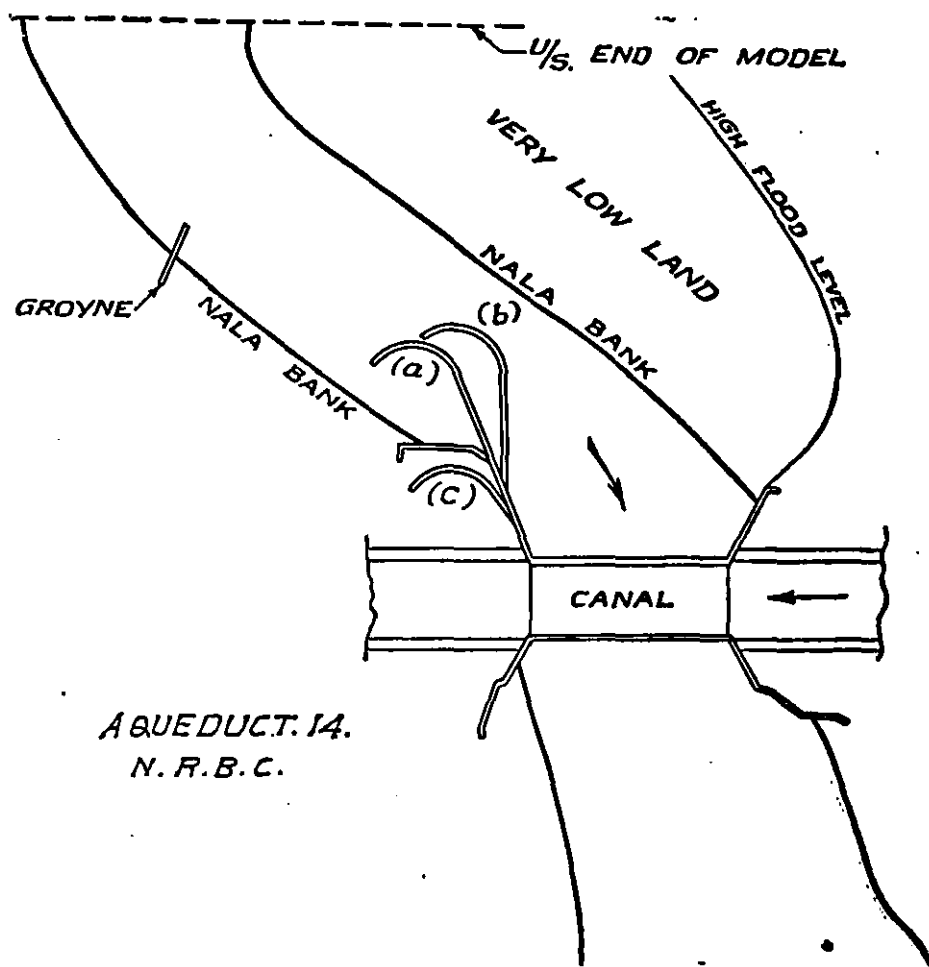
1787. Showing the model in experiment 1 (present conditions with repaired wing wall) looking downstream.



1831. Showing warped curved head used in experiment 3.

FIGURE 32
SCALE = 1 INCH = 160 FT

NO. 1155.
C.I. & H.R. STN.



With the wing wall as shown in (a), the improvement over flow conditions with the old design of wing wall was slight, and when the extension was at right angles to the aqueduct—see (b) in figure 32—the conditions were worse, there being severe return flow. With a wall as shown in (c) conditions were slightly better than those with the existing wall; and flow was, later, greatly improved by adding a groyne on the right bank, 200 ft. upstream of the wing wall.

More detailed experiments were carried out, comparing the two most promising designs with existing, viz:—

Experiment 1.—With existing wing wall—photo 1787.

Experiment 2.—With curved head and upstream groyne.

Experiment 3.—With a warped curved head—photo 1831.

In each experiment, lines of flow, velocities at various sections, water profiles etc., were observed under maximum flood conditions, viz., $Q \cong 10,640$ cusecs. The water level downstream of the aqueduct was maintained at R. L. $\cong 140.50$.

In experiment 1 there was undesirable diving flow at the nose of the sharp-angled wing wall, which was markedly less in experiments 2 and 3.

In experiments 1 and 3, the bed flow at the aqueduct was concentrated in the central spans, whereas in experiment 2, it fanned out and distributed in the different spans—vide figures 31, 33 and 34.

In experiments 2 and 3 there was forward flow in all spans.

Velocities upstream of the nose of the wing wall were slacker in experiment 2 than in experiment 1 or 3. This was to be expected, because flow was directed away from the right bank by the groyne.

Experiment 3 showed markedly better velocities along the right side than in experiment 1, and slightly better than in experiment 2. In experiment 1, the high velocity was entirely on the left bank upstream and downstream of the aqueduct, while on the right flank, there was return flow.

Water levels \cong 40 ft. upstream from the upstream face of the aqueduct and \cong 50 ft. downstream from the downstream face of aqueduct were observed and Table 3 shows the results.

TABLE 3.

| Experiment No. | Equivalent upstream W. L. | Equivalent downstream W. L. | Equivalent afflux (ft.) |
|---|---------------------------|-----------------------------|-------------------------|
| 1. With existing wing wall | 141.60 | 140.72 | 0.88 |
| 2. With curved head and upstream groyne | 141.25 | 140.67 | 0.58 |
| 3. With a warped curved head | 141.27 | 140.62 | 0.65 |

It will be seen that the afflux was minimum in experiment 2, and that in experiment 3 it was slightly more than in experiment 2 but less than in experiment 1, indicating that the designs of experiments 2 and 3 gave a higher co-efficient of discharge than in experiment 1.

Conclusion—

The results with a curved head and the groyne (experiment 2) were slightly better than those of experiment 3; but the cost of construction and maintenance would be greater. The warped wing wall head without a groyne is also preferable because the groyne might, in the course of time, be neglected or even removed by villagers; whereas, the warped wing-wall being an obvious necessity and part of the structure of the aqueduct, would certainly be maintained. This design was therefore recommended.

6. 1/500 MODEL OF THE GANGES ABOVE THE HARDINGE BRIDGE (E. B. RAILWAY). EXPERIMENTS TO REPRODUCE THE CHANGES IN THE RIVER COURSE IN 1935-37, AS IN FIG. 35.

These experiments, comprising Series XV with the model of the Ganges (scale ratios 1/500 horizontal : 1/65 vertical) were more of basic than specific character in that their object was the development of the technique of reproducing, in a model, the changes in the course of a large alluvial river.

In the Annual Report for 1937-38, the difficulties encountered and the methods used to overcome them were described. The greatest difficulty was to reproduce silt deposition, which plays a large part in the changes of river course. Small channels carry a much smaller proportion of silt in suspension than large channels, so that when a model carries a natural silt charge in relation to its discharge, the silt available for deposition is insufficient for similar deposition to that in the prototype. When more silt was injected in this model, it deposited near the entry, despite devices used to throw the silt into suspension.

Experiments had shown that the bifurcation into the Dadpur and Jalangi channels was too near the upstream end of the model for correct conditions to develop naturally. Although the upstream end of the model was located at Sardah, where the reach of river in the prototype was very stable, it became clear that changes in the course upstream of Sardah affected the flow characteristics in that stable reach, thus influencing the flow at the bifurcation.

A greater length of "controlled" river course was, therefore, required in the model to impose correct entry conditions, and this was provided in experiment 13, by treating the model from the entry to below Markoti and Jalangi (C. S. 19, see figure 36) as a "control length". Little could be done at high flood, but on a falling flood the course was corrected in this length to reproduce the flow of the prototype.

The results of the 1935 flood in the model were similar to those in the prototype, the flow below Raita being nearly correct; except, as in all previous experiments, for the tendency to cut across the high chur opposite Lalpur Bight and thus to straighten the course.

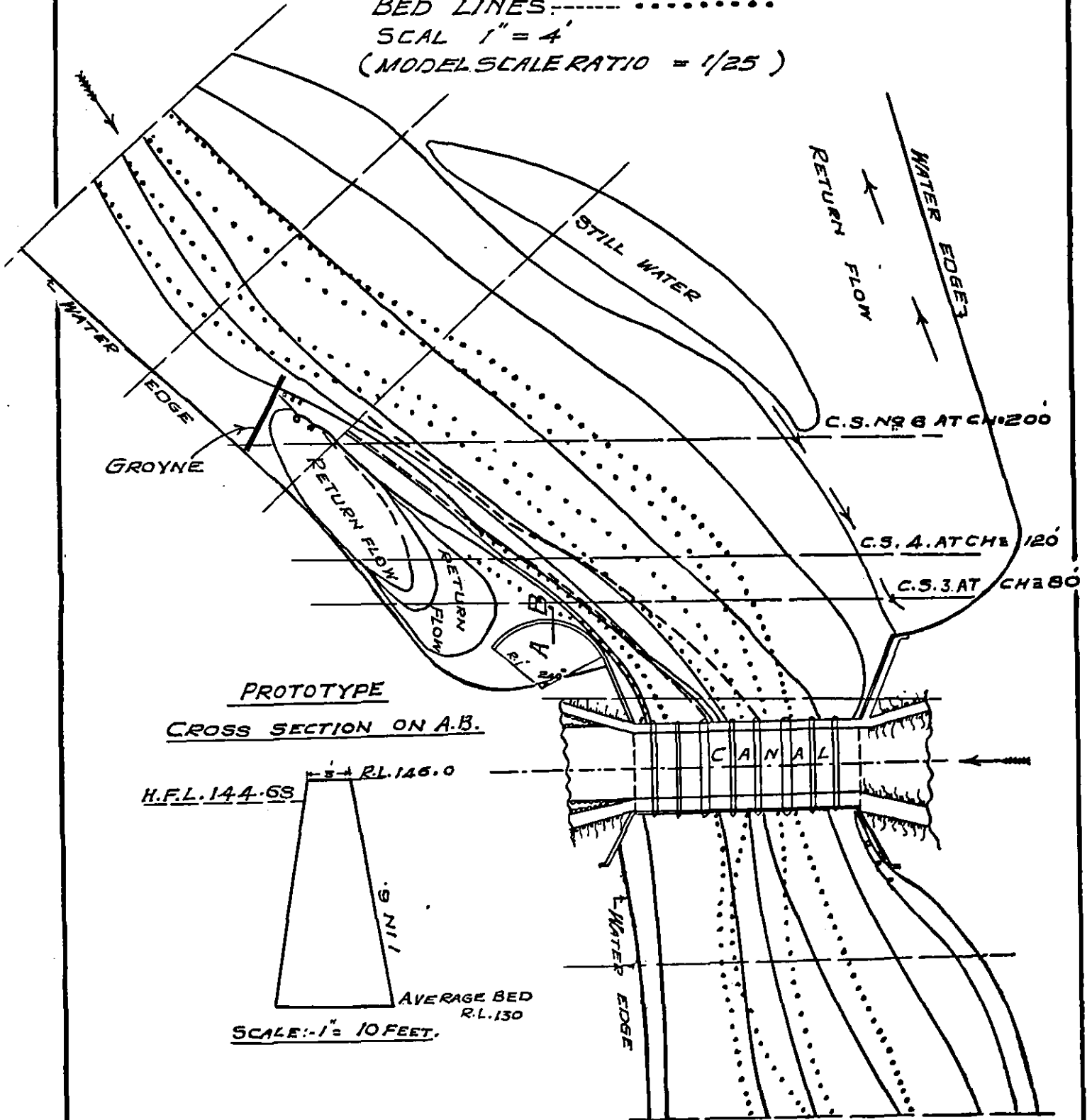
In 1936, flow through the Sonaikundi Bight was excessive in the model, and the Bight was maintained as a cold weather channel instead of silting up, due to the slow rate of silting in the model. In 1937, however, it silted up and the course above Raita was similar with that of the prototype—*vide* figure 36. Below Raita, the same tendencies as before, of cutting across the chur and central flow through the Bridge, were observed.

FIG:- 33. AQUEDUCT. 14 .N.R.B.CANAL.

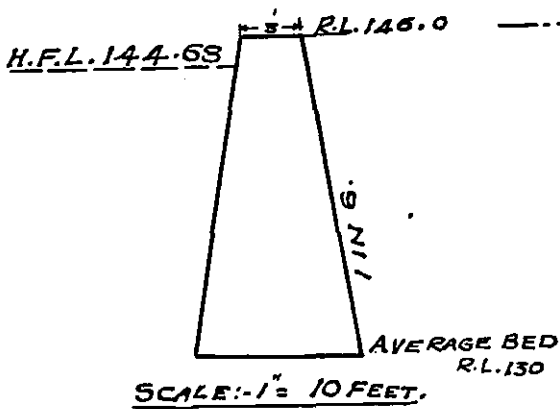
No. 1074/A.
C.I. G.H.R. 5TH

EXPT. 2, LINES OF FLOW WITH CURVED HEAD (HAVING SIDE SLOPE 1 IN 6) AT THE END OF WING WALL, AND A 2:1 SLOPING (DOWN) GROUYNE, WITH $Q=10640$ Cs. (=3.41 Cs.)
D/S. W.L. WAS MAINTAINED AT ≈ 140.50 .

SURFACE LINES. —————
MID-DEPTH LINES. - - - - -
BED LINES. ······
SCALE 1" = 4'
(MODEL SCALE RATIO = 1/25)



PROTOTYPE
CROSS SECTION ON A.B.



ELEVATION OF THE GROUYNE

SCALE: 1" = 2' FOR MODEL.
1" = 50' FOR PROTO

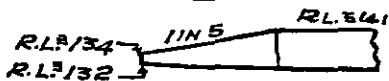


FIG. 34. AQUEDUCT 14, N.R.B. CANAL.

EXPERIMENT 3, LINES OF FLOW
WITH WING WALL (WARPED); SLOPE
DECREASING FROM 1 TO 6 AT THE END
OF PUKKA WALL TO 1/2 TO 1 AT THE NOSE.
Q = 10640 Cs. W.L. D/S. OF AQUEDUCT = 140'5.

SURFACE LINES. —————
MID-DEPTH LINES. - - - - -
BED LINES. ·······
SCALE 1" = 4'

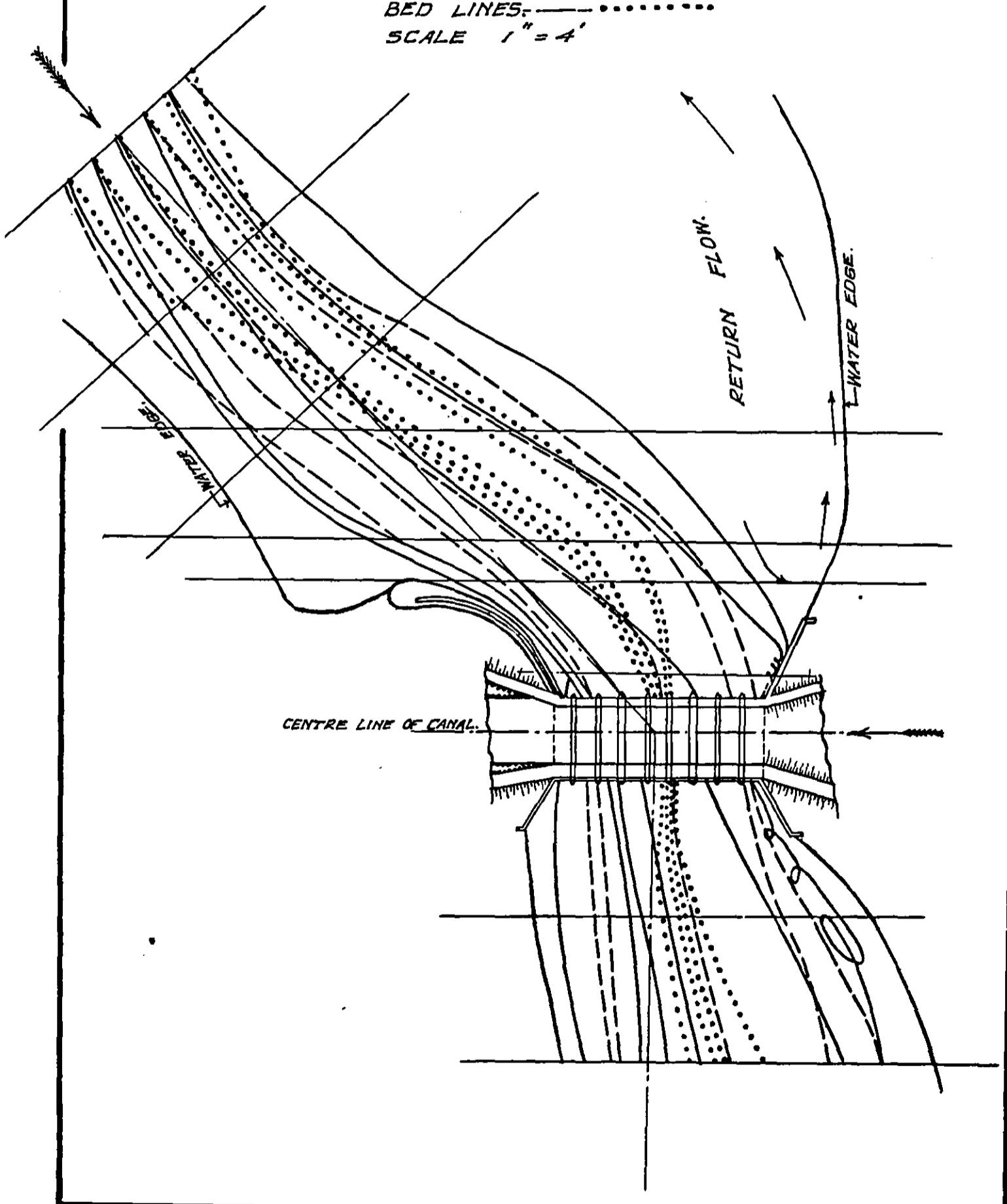






FIG. 35
PLAN OF GANGES ABOVE
HARDINGE BRIDGE.
COMPARING LOW-WATER COURSES
AFTER 1934 & 1937 FLOODS.

SCALE 1 INCH = 8,000 FEET.

REFERENCES.

- SCOUR..... 
- SILTING..... 
- WATER EDGE 1934..... 
- WATER EDGE 1937..... 

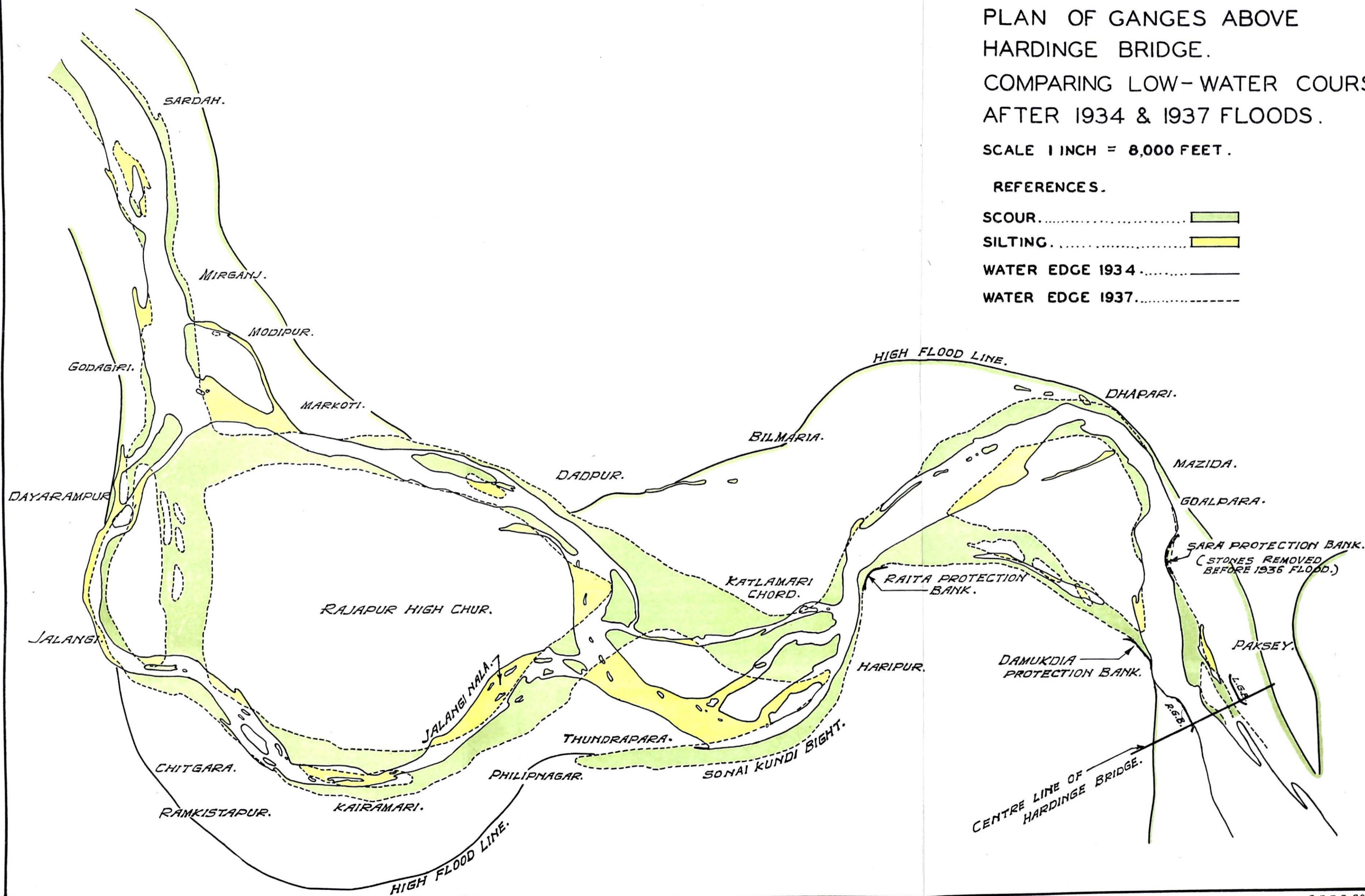
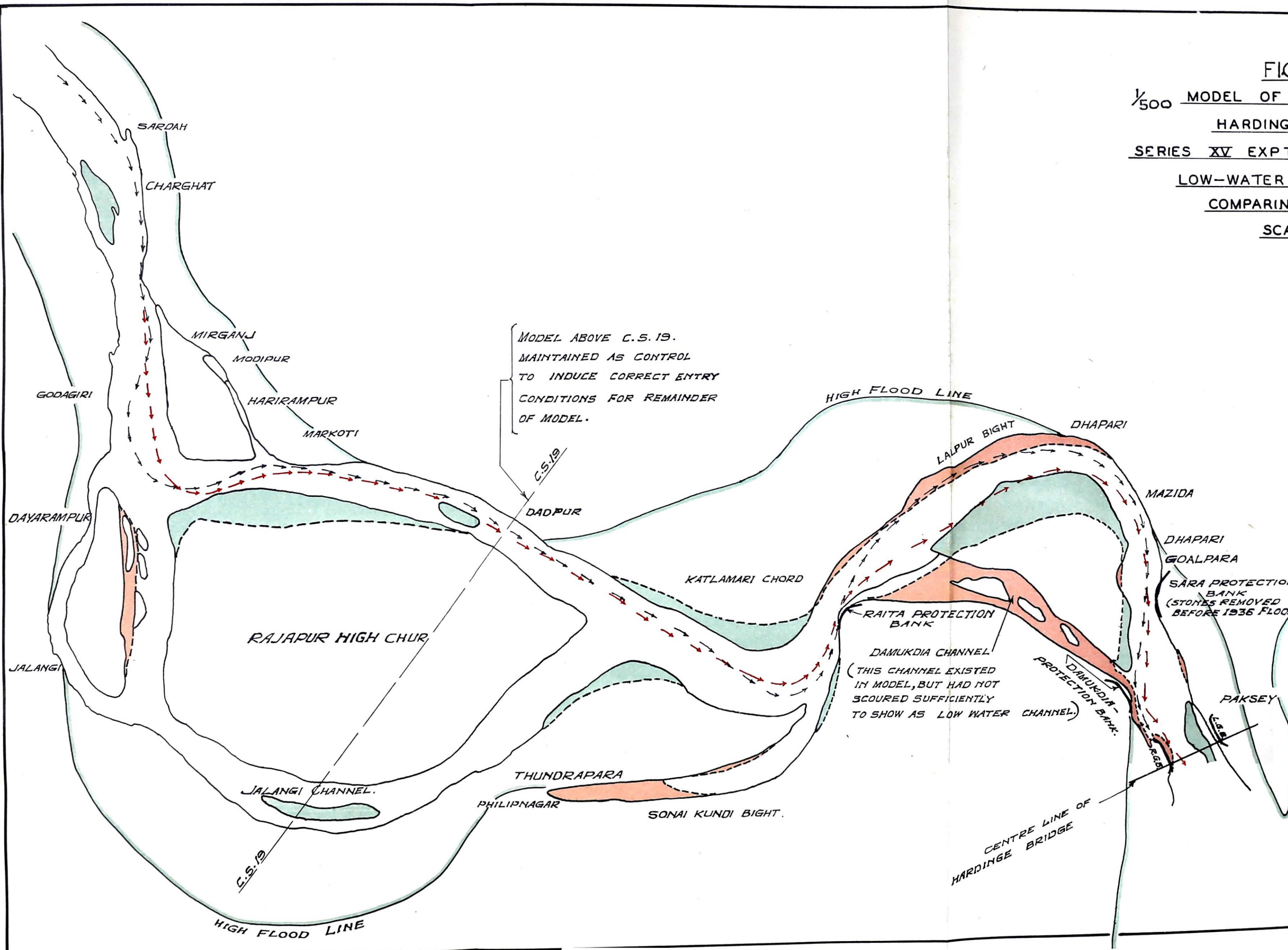


FIG: 36
 $\frac{1}{500}$ MODEL OF GANGES ABOVE
 HARDINGE BRIDGE
 SERIES XV EXPT. NO. 13 DATE. 24-8-38.
 LOW-WATER COURSE AFTER 1937 FLOOD.
 COMPARING PROTOTYPE & MODEL.
 SCALE 1" = 8000 FT.

DATA OF MODEL.
 MAX. Q = 11.3 CUSECS \equiv 1.42 M.C.S.
 SILT CHARGE = 8 X REGIME CHARGE.
 TIME SCALE = $\frac{1}{28}$
 SLOPE EXAGGERATION = 50 %

REFERENCES.
 CHANNELS IN PROTOTYPE NOT REPRODUCED IN MODEL.
 CHANNELS IN MODEL NOT EXISTING IN PROTOTYPE.
 MAIN FLOW (PROTO).
 MAIN FLOW (MODEL).
 PROTO. WATER EDGE.
 MODEL. " "

NOTE:-
 CHANGES IN RIVER COURSES
 IN 1934-37 ARE SHOWN
 IN FIGURE 35



The Damukdia channel took its correct proportion of discharge at high flood (20 per cent.), but the bed level of this channel was rather too high so that no channel existed in the cold weather survey as in prototype.

Discussion of Results.

The measures adopted for increasing the silt charge of the model, and thus its capacity for depositing silt, were partially successful as the amount of bed silt in movement was greater—for example a silt bank was thrown up after the 1935 flood opposite Thandrapara, as occurred in the prototype. The amount of silt in suspension, however, was still deficient. Instead of aiming at similar deposition, which, it was clear, could not be expected, the injection of light-weight material (as is being used in the model of the Indus and Sukkur Barrage), or quantities of very fine silt, might indicate areas where deposition is likely to occur, which could then be artificially raised by laying silt by hand. If this were done, the necessity for a very high silt charge would disappear.

Experiment 13 showed the value of a "control length" to impose and control entry conditions. This is regarded as an essential feature of river models at this Station.

The sloping guide and protection banks at Raita, Sara and the Bridge, which had been given the full exaggeration in vertical scale in the earlier experiment of Series XV, were reverted to a flatter slope as a compromise between full exaggeration and geometrical similarity, as had been done in Series I-X, to obtain more correct throw-off or deflection of the current. The results, however, were still not quite satisfactory and research in this matter is being conducted on other models in the Station.

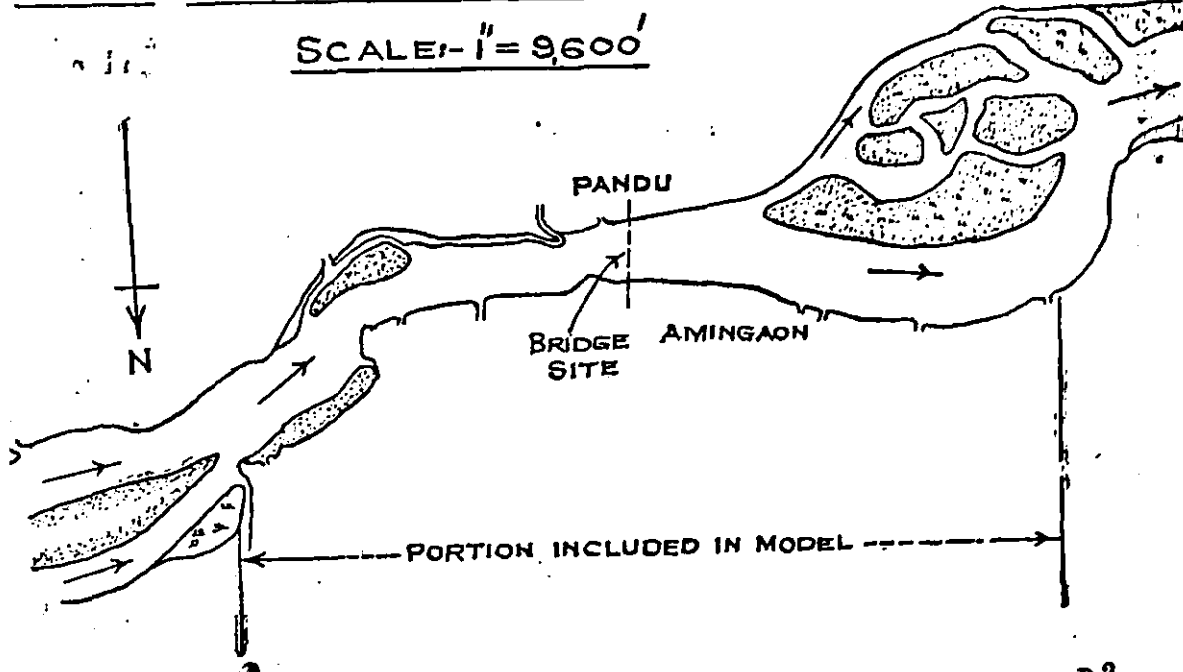
In general, it could be said that means had not yet been found of reproducing river changes in the Ganges for several years in succession. This was precluded by the inability of the model correctly to reproduce deposition of silt, and any forecast of river movements derived from the results of models in incapable hands might be highly inaccurate. It was however, possible to forecast *tendencies* to change in river course, and the accuracy with which this could be done would increase with further improvement in the technique of river model research.

Further experiments are in progress.

7. EXPERIMENTS WITH MODELS OF THE PROPOSED RAILWAY BRIDGE (EASTERN BENGAL RAILWAY) AT AMINGAON ACROSS THE BRAHMAPUTRA RIVER.

The experiments with this model, carried out on the basis of the 1937 flood data, were described in the 1937-38 Annual Report. The model is shown in photo 1647 and in Figure 37.

FIG. 37. PLAN OF BRAHMAPUTRA RIVER.



The aim of the experiments was :—

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

The results showed that

- (a) water followed the expanding sides below the gorge and the flow at the bridge site was sweet ;
- (b) upstream of the gorge, which is 2,400 ft. above the proposed bridge site, scour is deep but there is no danger of this scour moving downstream, because it is local, due to exposed rock and concave curvature of the river ;
- (c) when piers were added, the general conditions of flow at the bridge site were unaffected ;
- (d) the stone pitching of the guide banks of the bridge should conform as nearly as practicable to the existing banks so as to interfere as little as possible with present conditions of flow ; so, instead of stone being laid as an apron and allowed to launch, sand should, when practicable, be removed and the stone laid where finally desired.

Comparison of 1937 and 1938 flood data.

The 1938 flood data was found to be markedly different from the 1937 flood data.

Figure 38 shows the curves of G. R. : Q for 1937 in black and 1938 in red.

Figures 39 and 40 show discharges, velocities, areas, water levels at Pandu and C. S. areas below R. L. 170.0 (termed " datum areas ") for 1937 and 1938.

The G. R. : Q curve when extrapolated from the 1937 data to the estimated H. F. L. (R. L. 178.0) gave a maximum flood discharge of 1,720,000 cusecs. In 1938, points lay on the 1937 curve up to 900,000 cusecs ; but above that, the relationship between gauge height and discharge was markedly different.

From 16th June, 1938, to 12th July, 1938, discharges were much higher than according to the 1937 extrapolated curve. From 12th July to 31st July, 1938, discharges were not observed—the launch being out of order, which was unfortunate because the maximum gauge R. L. 173.85 was observed on 23rd July, 1938 at Pandu ; and thereafter discharges were less for the same water level than between 16th June to 12th July, 1938, these points lying between the June 1938 and extrapolated 1937 curves—see figure 38.

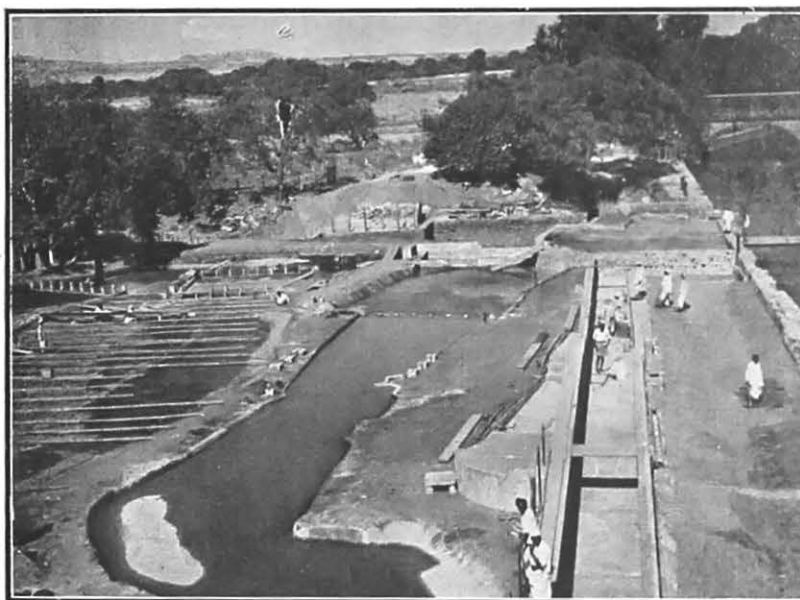
It will also be seen that if the June 1938 data were extrapolated, the curve gave a discharge of 3 million cusecs at R. L. 174.0 and 6 million cusecs at R. L. 178—indicating that either conditions would change with high flood levels or else that under the conditions of 1938, the flood could never rise even to R. L. 174.0.

From figures 39 and 40, it will be seen that the datum area (*i.e.*, cross-sectional area below R. L. 170) was continuously less than 170,000 sq. ft., until October, 1938, when, after heavy scour from mid June (when the datum area was 145,000 sq. ft.), it reached 190,000 sq. ft. From discharge observations on 4th July, 1938, 7th July, 1938 and 11th July, 1938, it will be noted that the area increased despite a decrease in water level and discharge, leading to a rapid decrease in velocity. This indicates the importance of the time factor on results.

The lowest bed R. L. at the bridge site varied in 1937 between 99.0 and 102.5 *i.e.*, scarcely at all ; whereas in 1938, the variation was large and the scour was maximum on a falling flood, minimum bed level being R. L. 79.0 on 22nd August, 1938, when $Q=1,304,328$ cusecs and $W. L.=171.05$ —see figure 41.

MODEL OF THE PROPOSED RAILWAY BRIDGE AT AMINGAON (E. B. RAILWAY) ACROSS THE BRAHMAPUTRA RIVER.

| | | |
|--|-----------|---------|
| Longitudinal and latitudinal scale ratio | | 1/300 |
| Depth scale ratio | | 1/50 |
| Discharge scale ratio | | 1/04800 |



Model of \rightarrow
the Sarda
River (U. P.).

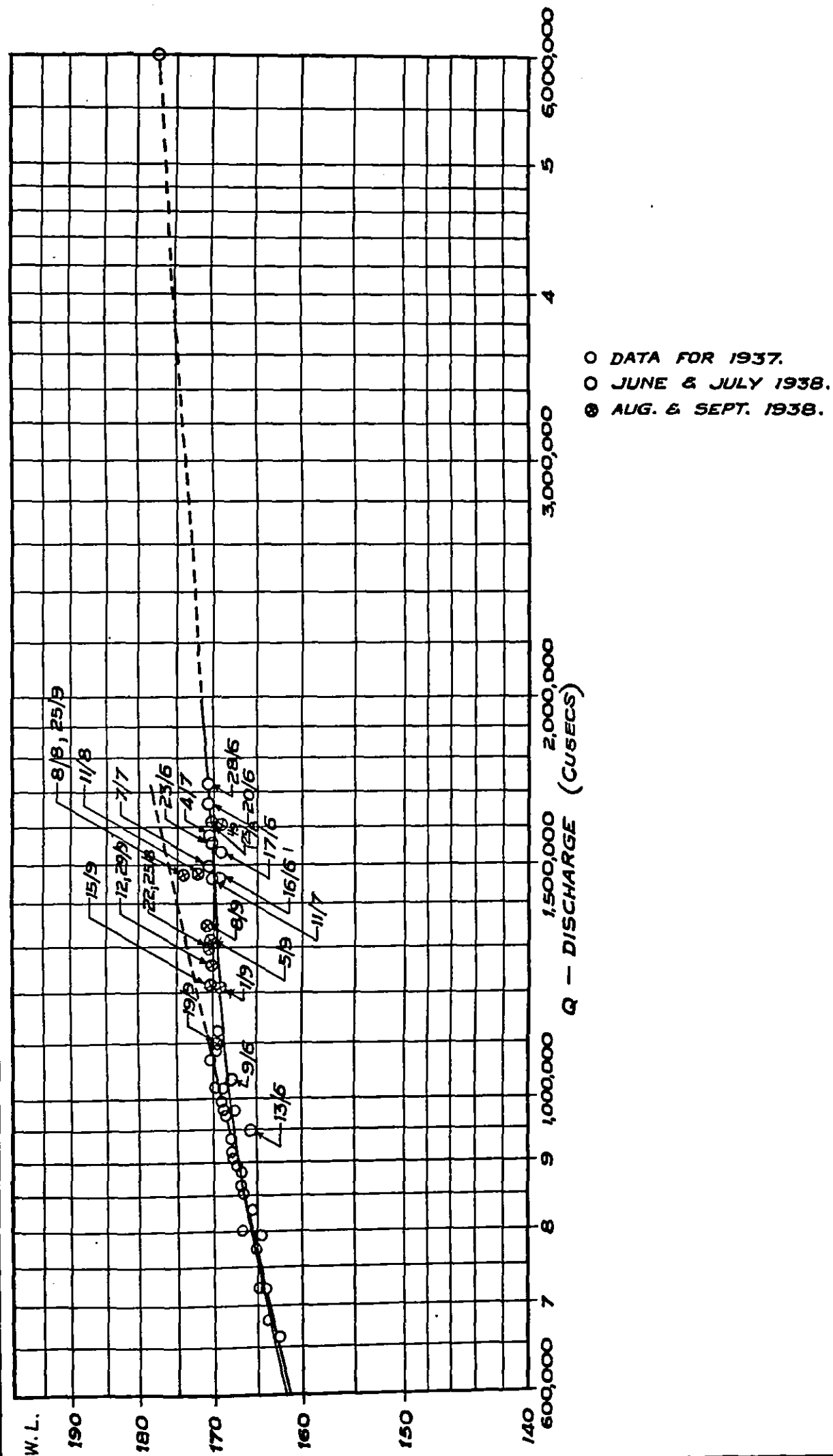
\leftarrow Silt channel.

\uparrow
Brahmaputra River model.

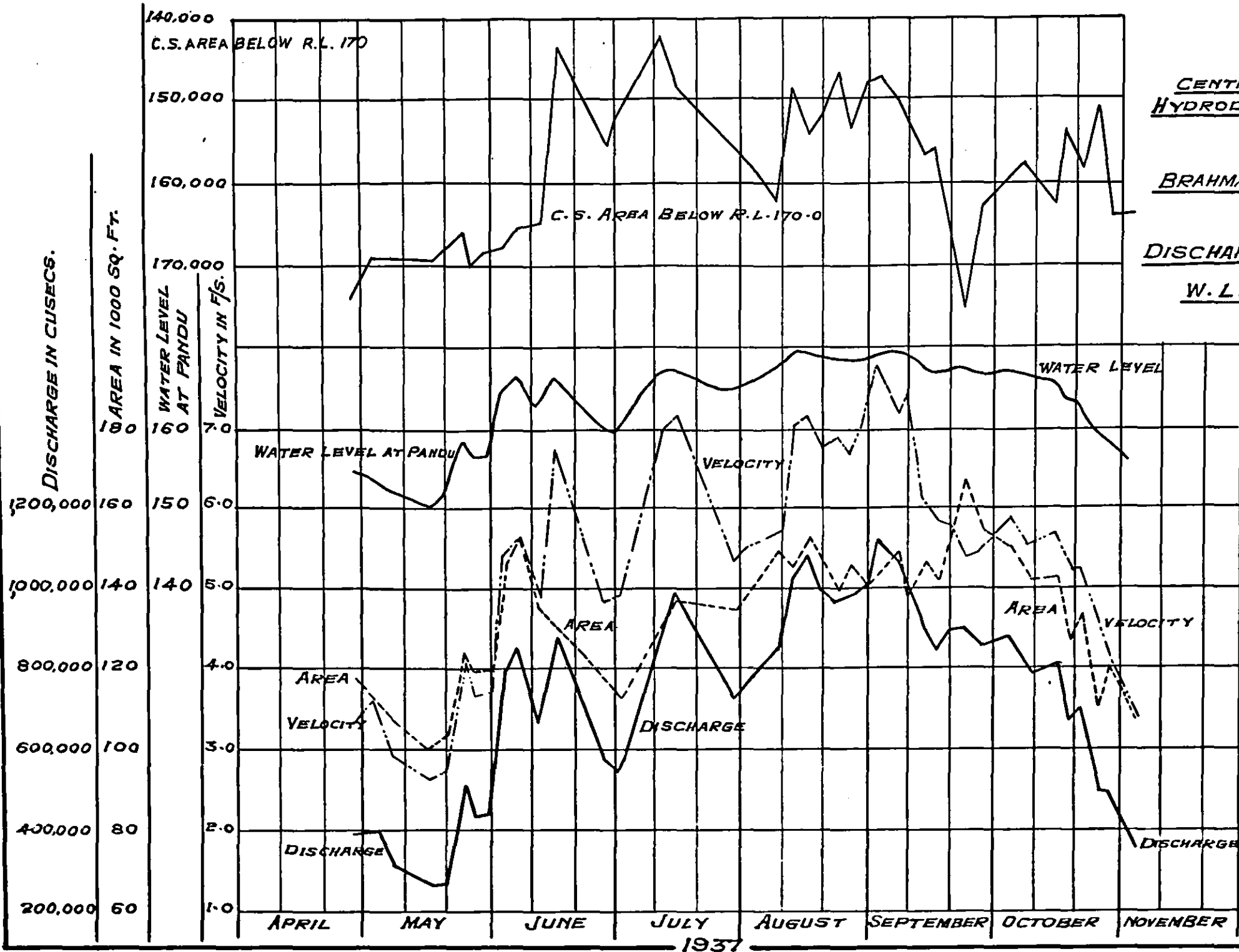
1647. Model of the Brahmaputra River, looking downstream.

FIG. 38

BRAHMAPUTRA RIVER
GRAPH SHOWING RELATION BETWEEN
WATER - LEVEL & DISCHARGE



No 1162
 CENTRAL IRRIGATION AND
 HYDRODYNAMIC RESEARCH STATION
 POONA
 FIG-39
 BRAHMAPUTRA RIVER AT
 AMINGAON.
 DISCHARGE, AREA, VELOCITY &
 W. L. AT PANDU IN 1937.



No 1162
 FIG-39

CENTRAL IRRIGATION
&
HYDRODYNAMIC RESEARCH STATION
POONA.

FIGURE.40
BRAHMAPUTRA RIVER
AT AMINGAON.

DISCHARGE, AREA, VELOCITY
& W.L. AT PANDU IN 1938.

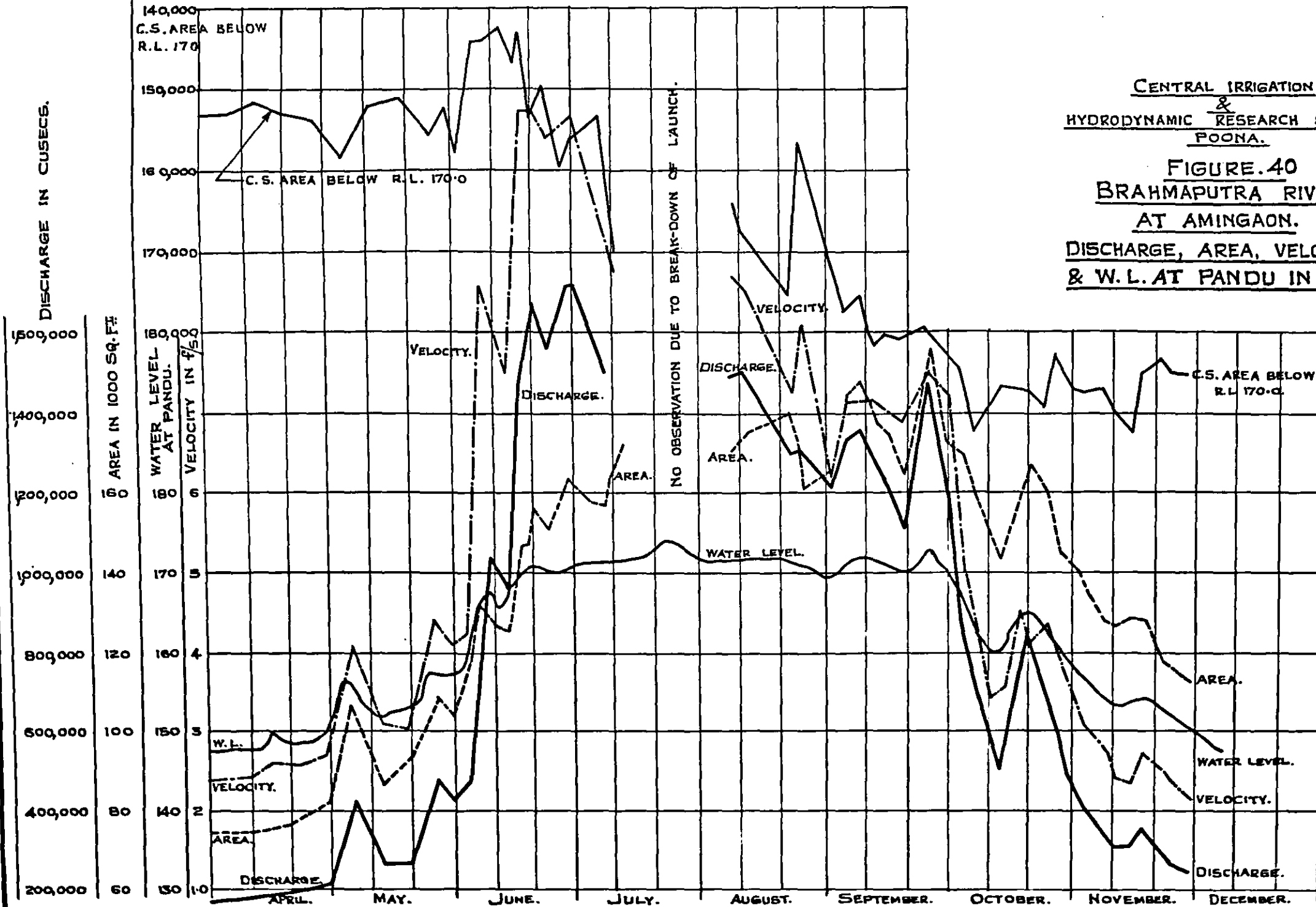
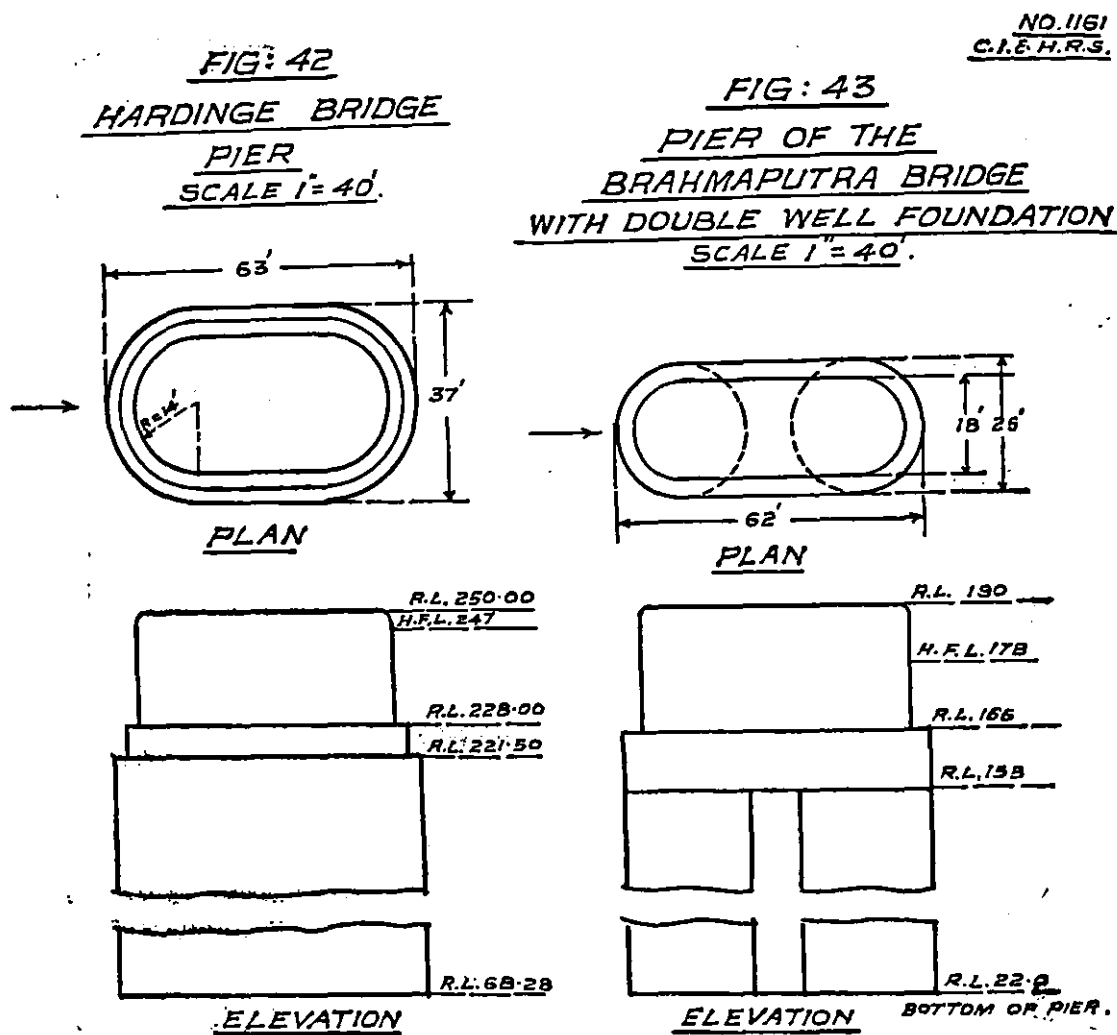


FIG:40
N.S. 1162/A.
C.I. R. R. 579.

The width of the Hardinge Bridge pier, which is solid, is 37 ft. The Brahmaputra Bridge pier is not solid for its whole depth ; but consists of two wells as shown.



The thickness of the pier is 25 ft. below R. L. 155.0. It is considered to be dangerous for the depth of pier embedded permanently in sand below maximum scour to be less than half the maximum depth of water.

The bottom of the proposed Brahmaputra Bridge piers is at R. L. 22.0, and H. F. L. is 178. On the above assumption, the minimum safe level of the top of stone protection is R. L. 74.0.

The level at which the pitching would stabilise around piers of the Brahmaputra Bridge was worked out to R. L. 80.0 by method (A) and to R. L. 83.0 by method (B), compared with which the assumed safe level of scour is R. L. 74.0. The way in which these results were obtained is shown below :—

A. Direct application to the Brahmaputra Bridge of known scour round the Hardinge Bridge piers.

The maximum depth of scour round the Hardinge Bridge piers is known and it is possible to estimate the depth of scour under the same conditions, had there been no piers. At the Brahmaputra Bridge, the maximum depth of scour in 1938 without piers is known ; so the depth of scour, which would have occurred round the Brahmaputra Bridge piers under 1938 conditions, was approximately estimated from the actual results at Hardinge Bridge by applying the same ratio of depth of scour with and without piers. The Brahmaputra piers, like the Hardinge Bridge piers, would be protected by loose stone pitching.

Figure 44 shows scour at piers 1 to 9 of Hardinge Bridge in November 1934 ; on 24th August 1938 (W. L.=246.7) and on 22nd September 1939 (W. L.=244.9). Cross sections of what the river bed would have been on 24th August and 22nd September, 1938, had there been no piers, are shown by dotted lines.

**FIG. 44. SHOWING SCOUR AT PIERS 1 TO 9
OF HARDINGE BRIDGE**

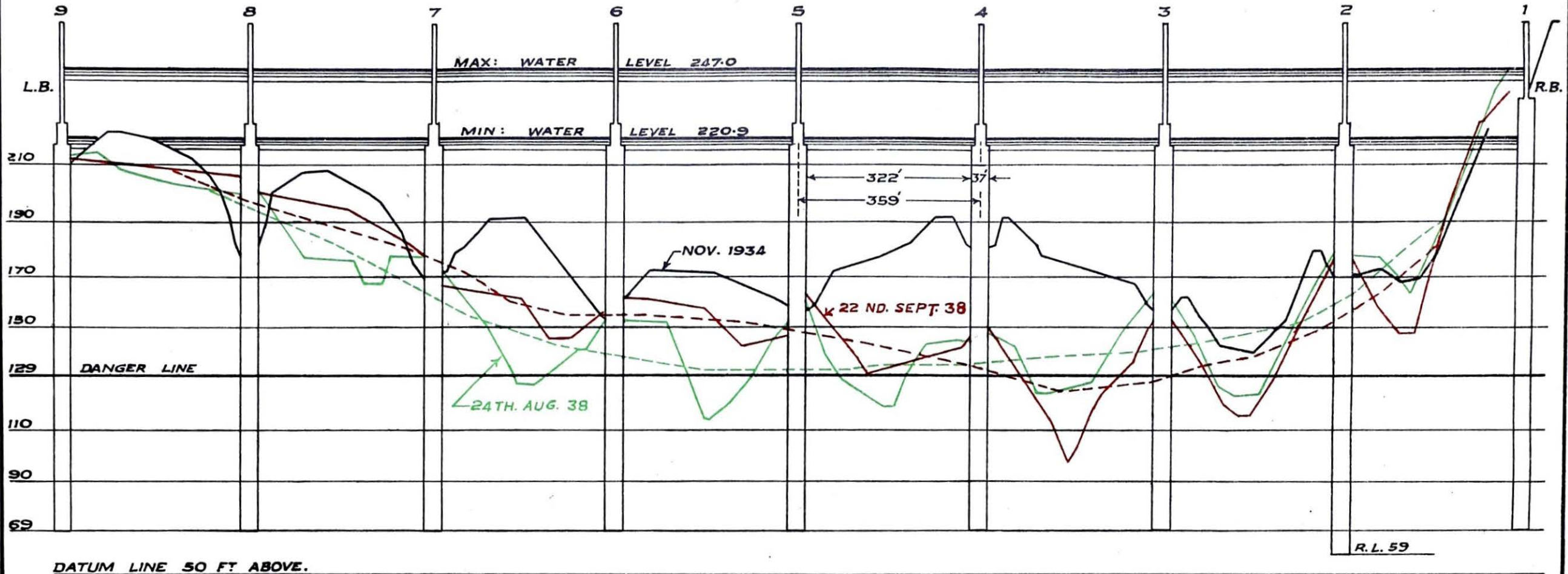
N^o 1101
C.I.&H.R.STM

SCALE :- HOR. 1 INCH = 240 FT
VER. 1 INCH = 48 FT

REFERENCES :-

- NOVEMBER 1934.
- 24 TH. AUGUST 1938. WATER LEVEL 246.70. Q = 1,620,000 CU SECS.
- 22 ND. SEPTEMBER 1938. WATER LEVEL 244.90
- - - ESTIMATED CROSS SECTION WITHOUT PIERS FOR 24 TH. AUGUST 1938 FLOOD.
- - - ESTIMATED CROSS SECTION WITHOUT PIERS FOR 22 ND. SEPTEMBER 1938 FLOOD.

PIERS 10 TO 16 NOT SHOWN.



stable, proposed to be used at the Brahmaputra Bridge) was laid at a depth of 1.18 ft. (\cong 74 ft.) below water level, and it sank (due to scouring action under steady axial flow with $q=1.8$ cusecs/ft. \cong 900 cusecs/ft. if the linear scale ratio were 1/63) to 1.4 ft. (\cong 88 ft.) below water level. Under these conditions, the stone protection at the Brahmaputra Bridge pier would stabilise at R. L. 171—88 = R. L. 83.0, which is 9 ft. higher than the minimum safe level. In the experiment the equivalent pier width was 36 ft. : so that less scour would be expected with a pier 25 ft. wide.

Summary of conclusions.

(1) When the discharge exceeds about 1 million cusecs, scour at the bridge site begins, and reaches a maximum on a falling flood.

(2) During a very rapidly rising flood, the river does not scour and may actually silt and there is a very rapid increase in velocity ; whereas during a steady flood, scour takes place and the velocity and gauge readings drop. For this reason, the discharge : gauge graph is not reliable, the scatter for different conditions being wide ; and it may be said that if a gauge of 178 has ever been attained, it must have occurred on a rapidly rising flood with a rising bed. It is considered unlikely that the discharge has ever exceeded $2\frac{1}{4}$ million cusecs.

(3) Wherever practicable, stone pitching round piers should be laid at the level at which it is finally desired. This would probably be cheaper in the long run as the stone would be less liable to be carried away by scour and would be more satisfactory, giving no further trouble.

(4) Based on a maximum flood of $2\frac{1}{4}$ million cusecs, the top of stone protection round the Brahmaputra piers is likely to stabilise at about R. L. 80 and is not likely to sink below the safe limit of R. L. 74.0.

8. EXPERIMENTS WITH MODELS OF PROPORTIONAL AND NON-PROPORTIONAL STANDING WAVE FLUME METER FALL AT R. D. 40280, MONTGOMERY-PAKPATTAN LINK (PUNJAB).

Notation.

F. S. Q. = Full supply discharge.

Q = Total discharge in cusecs.

q = Discharge per foot width in cusecs.

\bar{V} = Mean velocity f/s.

d_1 and d_3 = Depth of water in channel u/s and d/s of fall.

d_2 = Depth of water at toe of fall (assuming that no standing wave has formed).

$d_c = \sqrt[3]{q^2/g}$ = Critical depth.

d_1 = 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.

L_b = Distance of baffle from toe of fall.

\cong = Equivalent to.

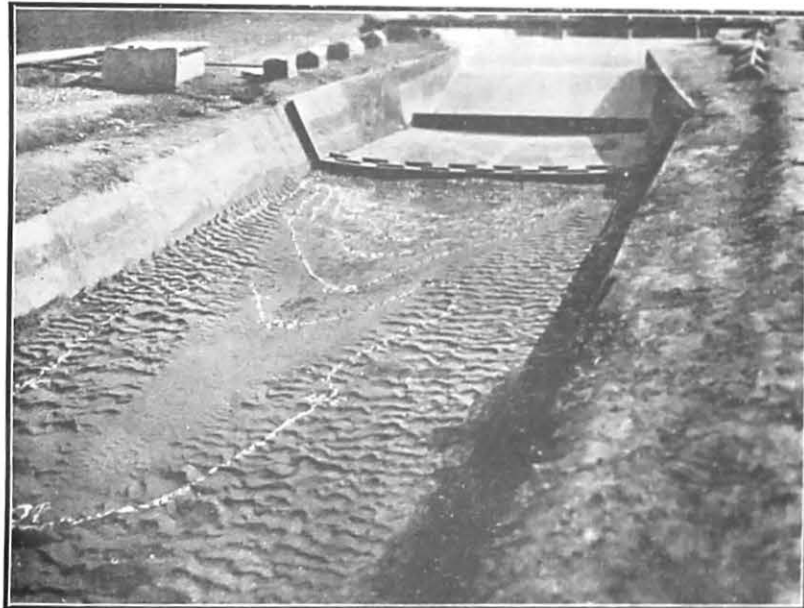
(i) *Experiments with a proportional S. W. F. Meter Fall.*

Experiments with a 1/5th scale model of a 700 cusecs proportional S. W. F. Meter Fall were described in the Annual Report of this Station for 1937-38. The design consisted of a sloping glacis ; a baffle platform ; a baffle 1.6 ($d_c - d_2$) high ; a cistern of bowed cross section with a rising pavement ; bed and side slope

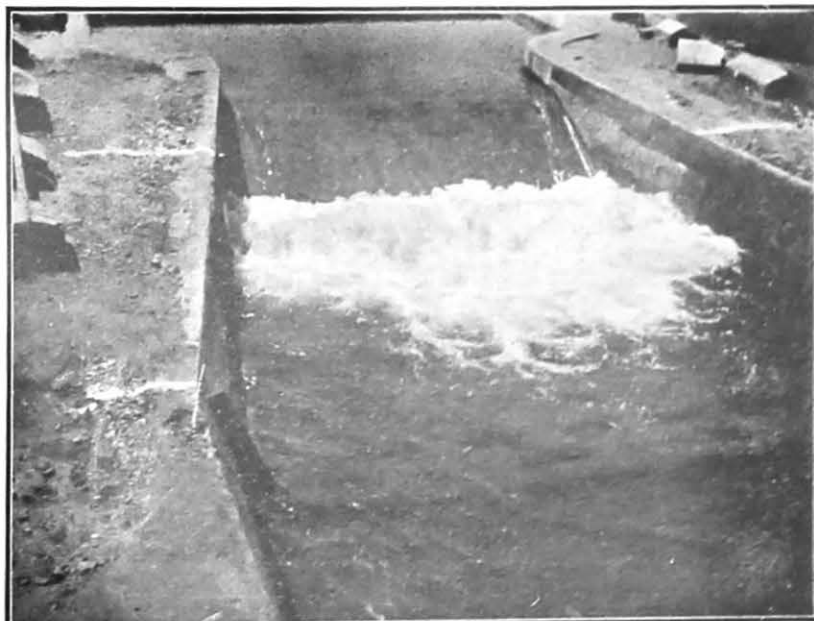
1/5 SCALE MODEL OF 700 CS. FALL AT R. D. 40, 280, MONTGOMERY-PAKPATTAN LINK (PUNJAB).



1848. *Proportional Standing Wave Flume Meter Fall, Looking u/s, with baffle, deflectors and control blocks, showing scour d/s of the fall with 25 per cent. retrogression of d/s W. L. Looking upstream.*



1960. *Non-proportional S. W. F. M. Fall with a high hump $0.4 d_1$ high. Experiment 4 with $h_b = 1.1 (d_1 - d_2)$; deflectors and control blocks. Looking upstream.*



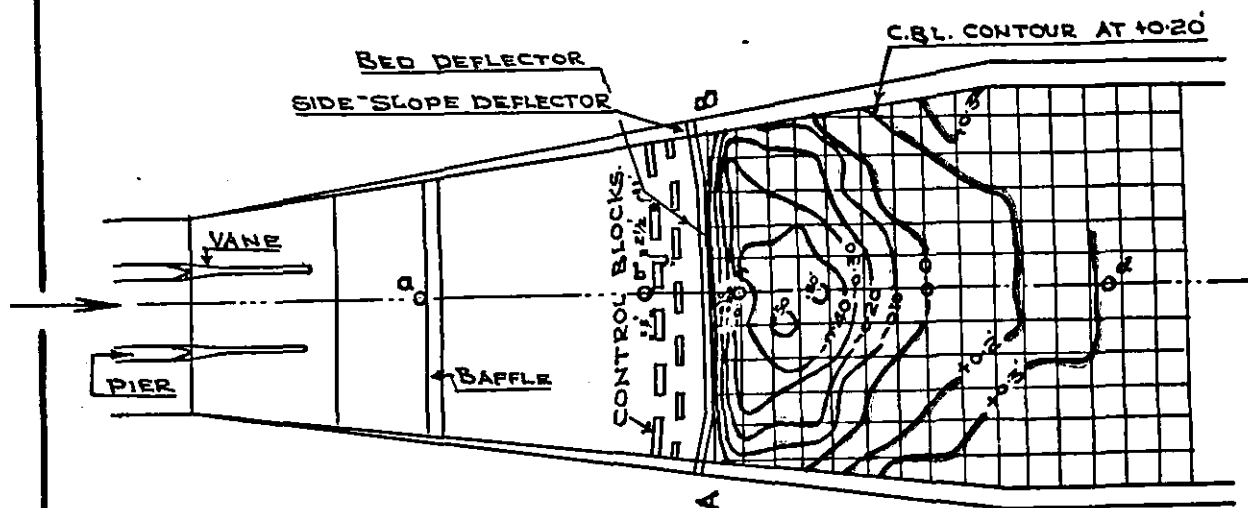
1962. *Same as 1960 looking upstream, showing the effective working of the baffle, with primary and secondary waves clear.*

FIGURE:- 46 EXPT I.

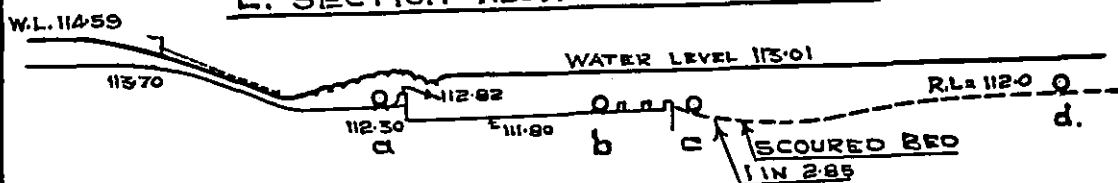
1/5 SCALE GEOMETRICALLY SIMILAR FALL MODEL
FOR 700 CUSECS.

SCOUR AFTER 26 HOURS FLOW WITH TWO
ROWS OF PUNJAB CONTROL BLOCKS AND BED
AND SIDE SLOPE DEFLECTORS AT THE END OF
PAVEMENT. HEIGHT OF BLOCKS = $\frac{d}{8}$ (= 0.18 FEET)
= 0.90 FT CLEAR SPACE BETWEEN BLOCKS IN
ONE ROW = 0.60' (= 3 FEET) CLEAR SPACE BETWEEN
TWO ROWS = 0.50' (= 2.5 FEET) WITH $Q = 12.5 C_s$ (= 700 C_s)
& WITH 25% RETROGRESSION OF $D/5$ W.L. WITH
VANES IN EXTENSION OF PIERS AS FAR DOWN
AS THE $U/5$ FACE OF THE STANDING WAVE.
MAX. DEPTH OF SCOUR = 0.53' (= 2.65 FEET)

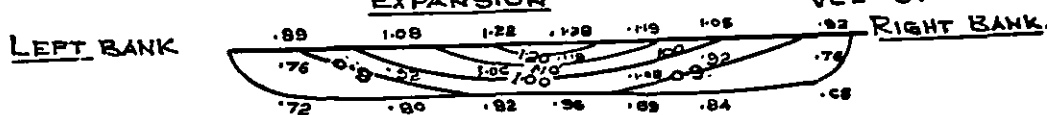
SCALE: 1" = 6 FEET.



L. SECTION ALONG THE CENTRE.

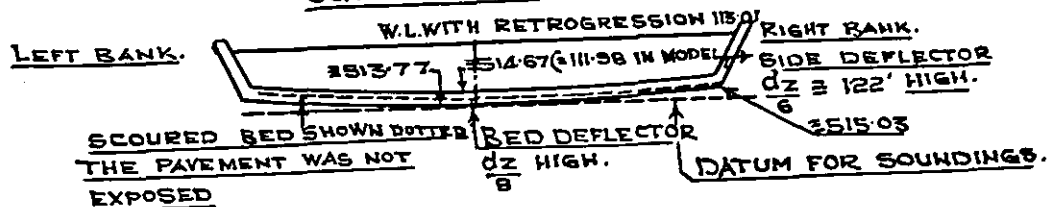


CROSS SECTION = 34' $D/5$ OF
EXPANSION
 $V_0 = .86$
 $V_c = .97$



CROSS SECTION ON AB.

SCALE: 1 INCH = 4'

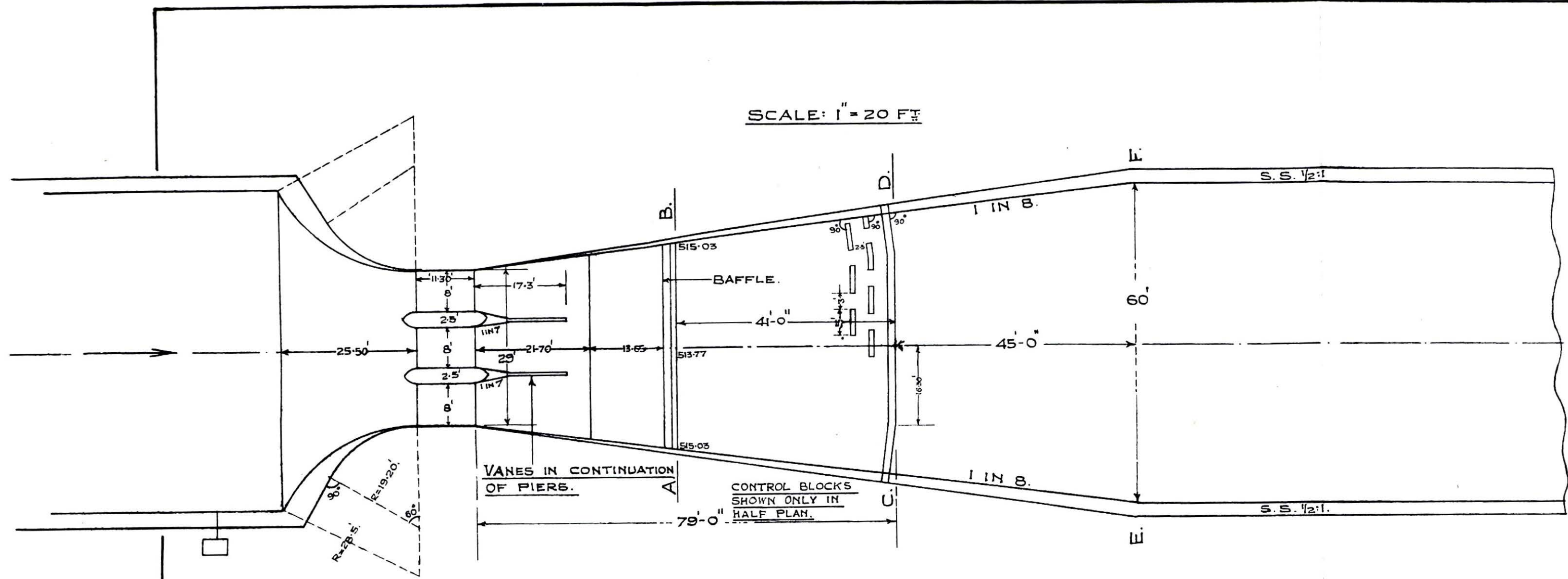


NOTE:-

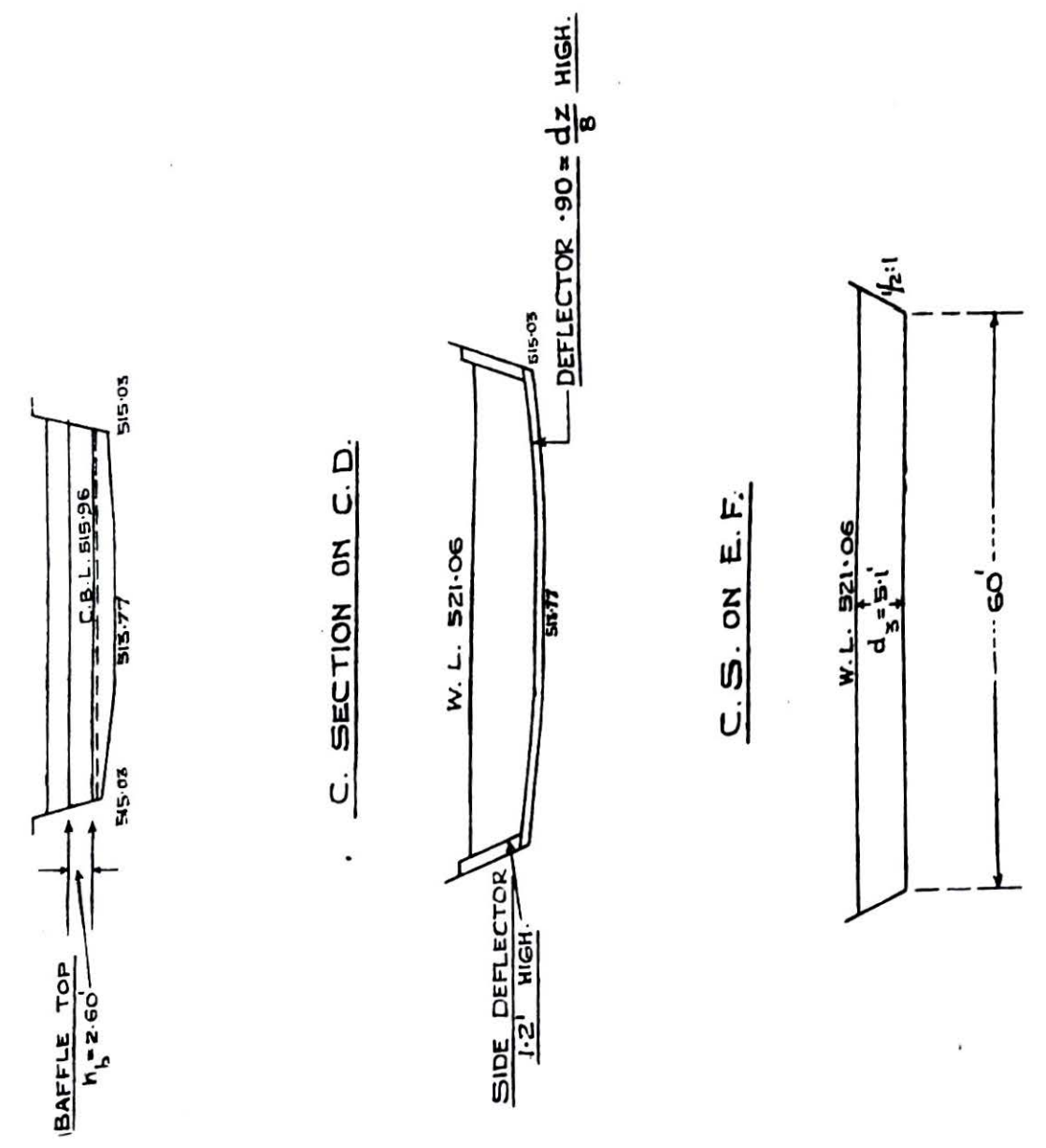
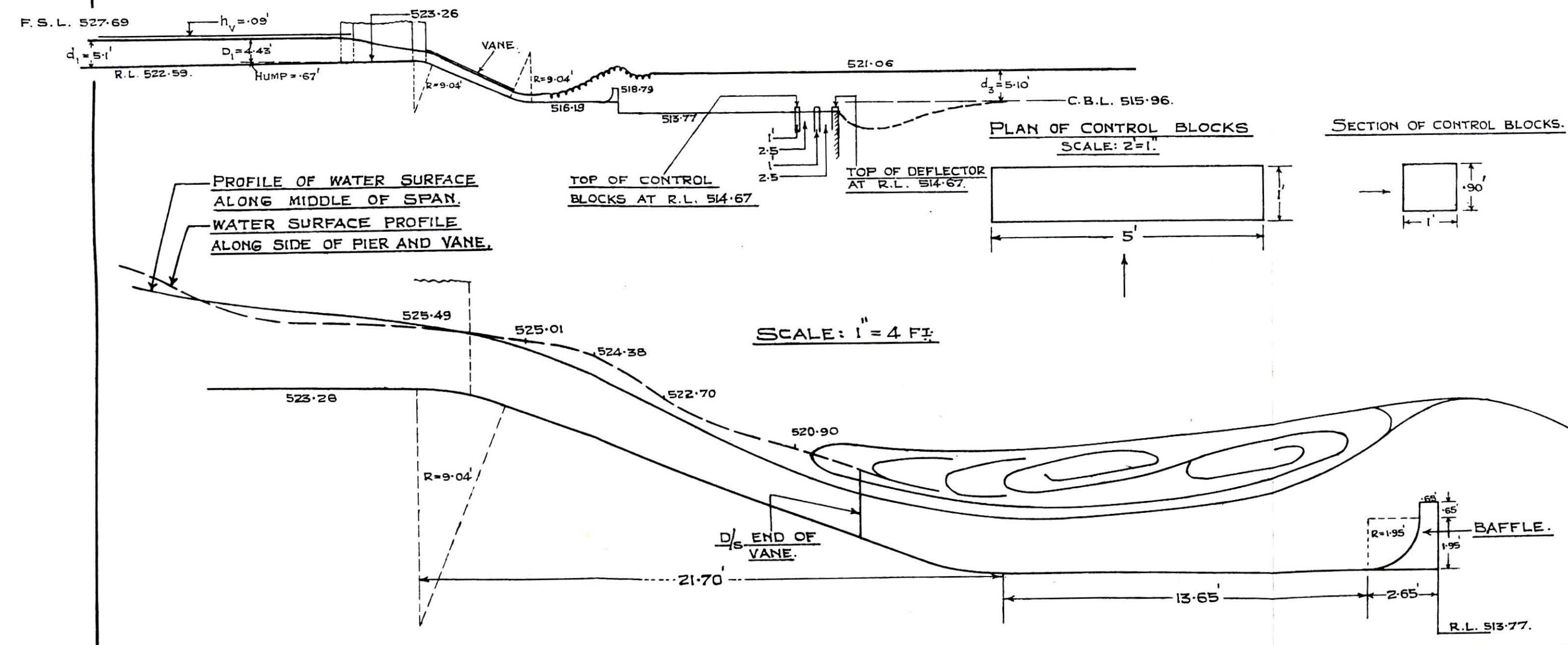
- (1) PROPORTIONAL DESIGN RECOMMENDED FOR CONSTRUCTION
ON THE MONTGOMERY PAK-PATTAN LINK (PUNJAB)
- (2) a, b, c & d SHOW POINTS WHERE HOT WIRE OBSERVATIONS
WERE TAKEN-SEE PARA 10 & FIGURE 55.

FIG. 47

MODIFIED DESIGN FOR A FALL
COMBINED WITH V. R. BRIDGE AT R.D. 40280
MONTGOMERY PAK-PATTAN LINK PUNJAB. F.S.Q=700 Cfs.
STANDARD DESIGN OF A 700 CUSECS. 6.63 FEET.
P. S. W. F. M. FALL WITH VANES IN CONTINUATION OF
PIERS, BAFFLE, TWO STAGGERED ROWS OF
CONTROL BLOCKS, BED AND SIDE DEFLECTORS
ON CISTERN BED. SPACING OF CONTROL BLOCKS:-
CLEAR SPACE BETWEEN CONSECUTIVE BLOCKS
IN THE SAME ROW=3 FT. CLEAR SPACE BETWEEN
TWO ROWS=2.5 FEET.
BED WIDTH = 60 FT. WITH 1/2:1 SIDE SLOPES.
F. S. D. 5.1 FT. $\bar{V}=2.19$ FT. PER. SEC.
F. S. L. U/S. 527.69. D/S. 521.06.
BED LEVEL U/S. 522.59. D/S. 515.96.



SCALE: 20' = 1 INCH.



deflectors. This design gave good results with normal water level downstream; but the results were not so good when there was retrogression in downstream water level, when the flow became slightly unstable and uneven bed scour took place downstream of the fall. The experiments were, therefore, continued to improve the design.

The instability of flow was overcome by building vanes in continuation of piers down to the upstream face of the standing wave. In order to reduce the cost, the length of pavement was reduced from 72 ft. to 41 ft. and the rising floor of the cistern was replaced by a flat pavement. This design worked satisfactorily; but the maximum depth scour (with 25 per cent. retrogression downstream) increased from ≈ 1.5 ft. to 3.35 ft. Though this scour was ≈ 10.5 ft. downstream of the pavement, and therefore not dangerous, it was desired to reduce it, for which purpose control blocks (Punjab design) were added and tested in the following experiments:—

Experiment 1.—Figure 46 and photo 1848.

With two staggered rows of ≈ 5 ft. long control blocks $d_s/8$ high, spaced $\approx 2\frac{1}{2}$ ft. apart; the spacing of blocks in a row ≈ 3 ft.; bed deflector $d_s/8$ high with side slope deflectors $d_s/6$ high at end of pavement.

In *Experiment 2*, the bed deflector was made discontinuous by making gaps ≈ 0.75 ft. at intervals of ≈ 5 ft. and in *Experiment 3*, the deflector was replaced by one row of control blocks. Table 4 shows the results.

TABLE 4.—(F. S. $Q_s=12.5$ cusecs ≈ 700 cusecs and 25 per cent. retrogression in downstream W. L.).

| Expt. No. | Maximum depth of scour (ft.). | WAVE WASH NEAR BANKS. | | | Distribution of velocities in channel d/s of fall. |
|-----------|-------------------------------|-----------------------|------------------|-----------------------------|--|
| | | Height of waves. | Length of waves. | Choppiness height = length. | |
| 1 | 0.53 (≈ 2.65) | 0.104 | 0.63 | 0.166 | Symmetrical. |
| 2 | 0.43 (≈ 2.15) | 0.11 | 0.65 | 0.17 | Slightly right sided. |
| 3 | 0.48 (≈ 2.40) | 0.12 | 0.70 | 0.17 | Ditto. |

It will be seen that there was little difference in wavewash. In experiment 1, the depth of scour was more, yet the scour was more central than in experiments 2 and 3; besides, the maximum scour of ≈ 2.65 ft. being in midstream and ≈ 10 ft. downstream of pavement is not objectionable. With no retrogression downstream the results would be markedly better, so were not tested.

Conclusion.—The standard design of bed and side slope deflectors, with two rows of control blocks added, was considered suitable and was finally adopted. Fig. 47 shows the design for the prototype.

(ii) *Experiments with a non-proportional S. W. F. M. Fall.*

Flumed Falls designed to be "proportional"—i.e. no afflux or draw-down throughout the range of discharge—have a lower hump and narrower throat than would otherwise be necessary. This adds to the length of expansion and thus increases cost.

Proportionality is frequently not required: for example, the general practice in Northern India is to run canals only at full supply, and many engineers consider that even where the discharge varies considerably, afflux at low supplies is permissible. An alternative design to that described in (i) above was therefore tested, having a high non-proportional hump and wider throat. This is shown in

fig. 48. Table 5 shows the relative dimensions of the proportional and non-proportional S. W. F. Meter Falls :—

TABLE 5.

| Type | Recommended design proportional hump (fig. 47). | Design with a non-proportional hump (fig. 48). |
|--|---|--|
| | 48% | 60% |
| Fluming ratio | | |
| (1) Overall throat width | 29 ft. | 41.5 ft |
| (2) Height of hump | 0.13 d_1 | 0.4 d_1 |
| (3) Length of expansion | 124 ft. | 74 ft. |
| (4) Length of pavement downstream of baffle | 41 ft. | 47 ft. |
| (5) Height of baffle | 2.6 ft. | 1.53 ft. |
| (6) Depth of oistern below C. B. L. in midstream | 2.19 ft. | 0.15 ft. |
| (7) Ditto near banks | 1.26 ft. | Nil. |
| (8) Cost | Rs. 6,100 | Rs. 5,600 |

There were two rows of control blocks, bed and side slope deflectors, as in the recommended design.

Experiments.—The scale ratio of the model was 1/5 as before and all tests were carried out with 25 per cent. retrogression of downstream W. L.

As a preliminary, baffles of different heights ranging from 0.8 to 1.6 ($d_c - d_2$) were tried to observe which worked satisfactorily by flow characteristics. Scour was then observed with three heights of baffles, viz. :—

Experiment 4 with $h_b = 1.6$ ($d_c - d_2$).

Experiment 5 with $h_b = 1.1$ ($d_c - d_2$)—fig. 49 and photo 1960 and 1962.

Experiment 3 with $h_b = 1.0$ ($d_c - d_2$).

When the baffles were working properly, the Bentzel tube fluctuations were symmetrical. Results were therefore judged by scour and distribution of velocities in the channel downstream. Table 6 shows the results :—

TABLE 6.—Results of the 1/5th scale model of 700 cusecs fall.

| $\frac{h_b}{(d_c - d_2)}$ | WAVES. | | Choppiness = $\frac{\text{height}}{\text{length}}$ | Maximum depth of scour in midstream (below pavement level) (ft.). | Nature of scour. |
|--|---------------------------|----------------------------|---|---|---|
| | Height. (ft.). | Length. (ft.). | | | |
| <i>Non-proportional S. W. F. Meter Fall.</i> | | | | | |
| 1.60 | 0.096 (≈ 48) | 0.03 (≈ 4.15) | 0.12 | 0.56 (≈ 2.80) | One sided. |
| 1.10 | 0.08 (≈ 40) | 0.80 (≈ 4.00) | 0.10 | 0.50 (≈ 2.50) | Central and symmetrical. |
| 1.00 | 0.08 (≈ 40) | 0.80 (≈ 4.00) | 0.10 | 0.47 (≈ 2.35) | Central but slightly greater on right side. |
| <i>Proportional S. W. F. Meter Fall.</i> | | | | | |
| 1.6 | 0.104 (≈ 50) | 0.63 (≈ 3.15) | 0.17 | 0.53 (≈ 2.65) | Central and symmetrical. |

Whereas, in the proportional design no baffles less than 1.6 ($d_c - d_2$) worked, it will be seen that in this, best results were obtained with $h_b = 1.1$ ($d_c - d_2$). Photo 1962 shows that the primary and secondary waves were clear. Bed conditions were better than in the proportional flume (figure 46). There was, for example, silt deposit at sides just downstream of pavement.

FIG. 48.

NO 1142
C.I.B.H.R.S.

700 CS. FALL DESIGN.
WITHOUT BRIDGE & WITH A NONPROPORTIONAL HUMP

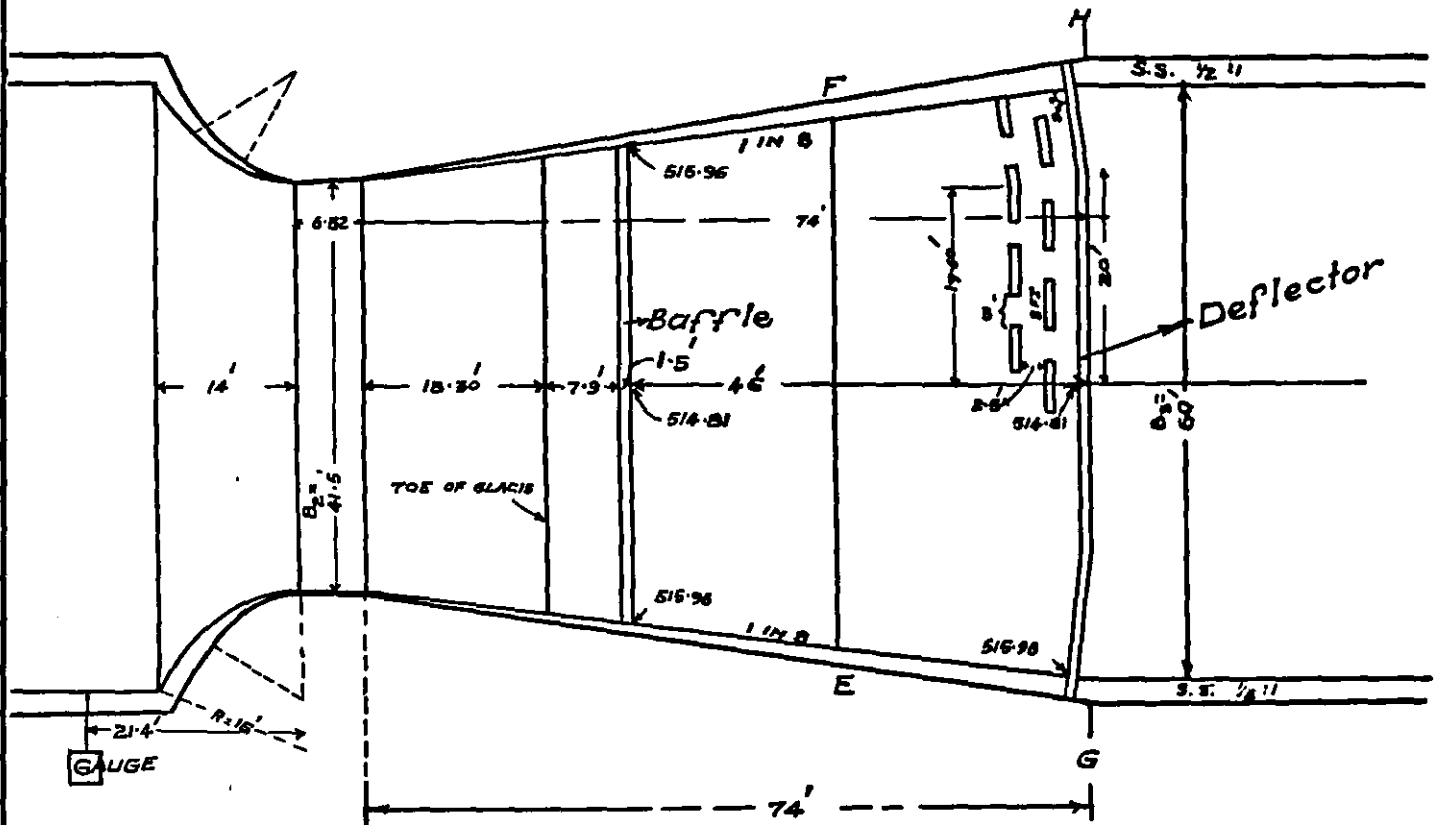
$(HEIGHT = 0.4d_1)$

$Q = 700 \text{ CS}; B_2 = 41.5 \text{ FEET};$

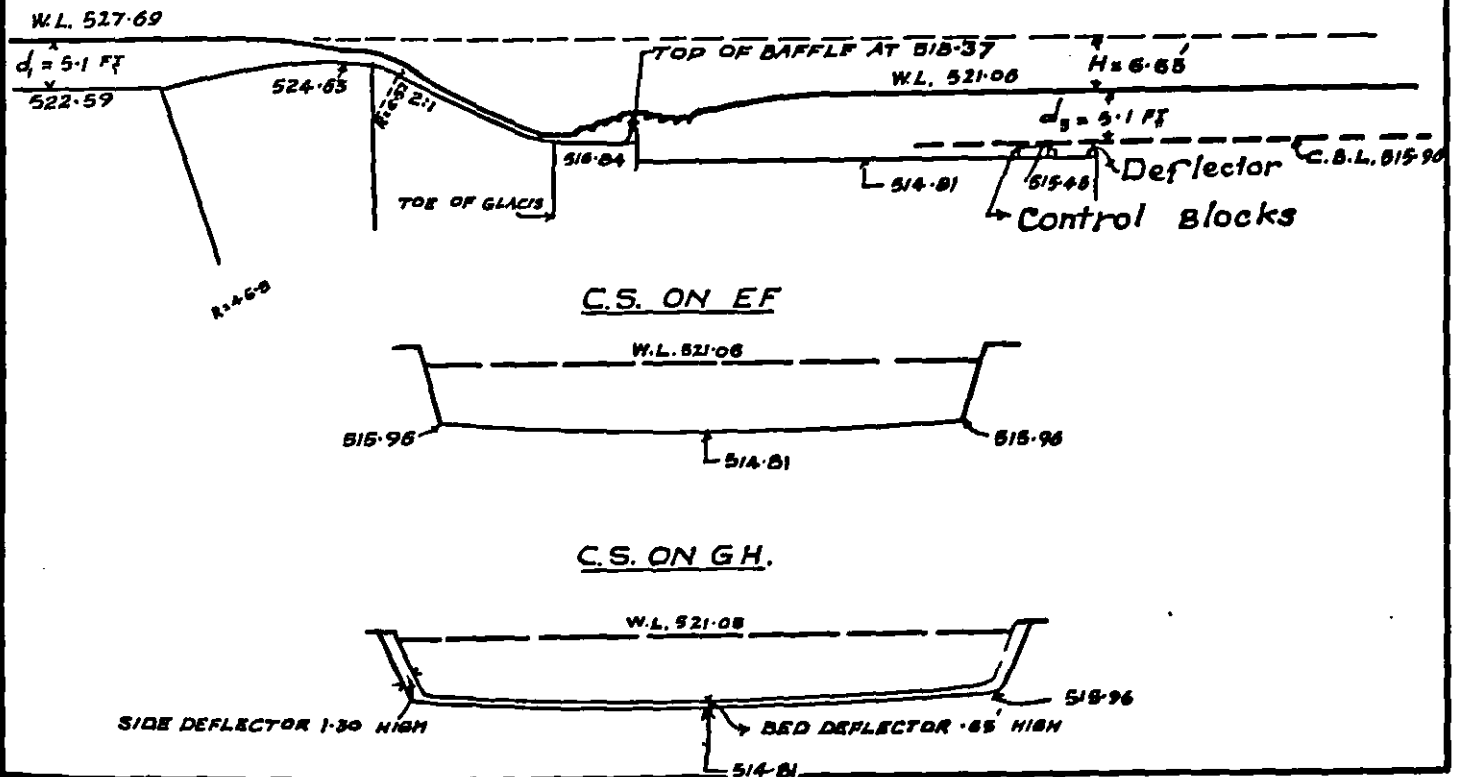
$B_3 = 60 \text{ FT}; H = 6.63 \text{ FT};$

$d_1 = d_3 = 5.1 \text{ FT}.$

SCALE 1" = 20'



L. SECTION ALONG CENTRE.



CENTRAL IRRIGATION
 Δ
 HYDRODYNAMIC RESEARCH
 STATION POONA.

No 1199 A.
 FIG. 49

FIGURE 49

EXPT. WITH $\frac{1}{8}$ TH MODEL OF 700 CUSECS FALL
 WITH HEIGHT OF HUMP = $0.4 d_1$
 SHOWING SCOUR AFTER 26 HOURS FLOW
 HEIGHT OF BAFFLE = $1.1 (d_1 - d_2) = 0.30115'$

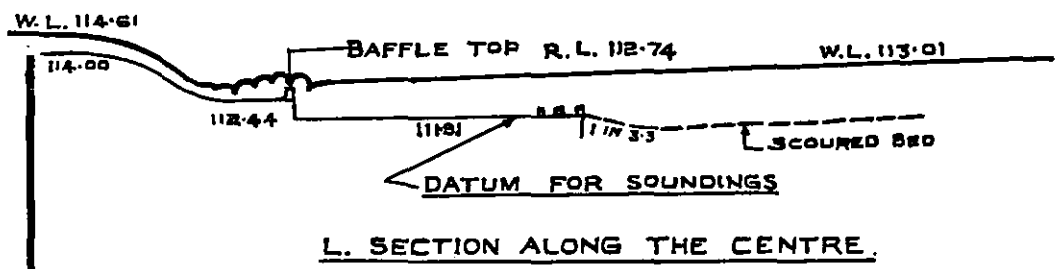
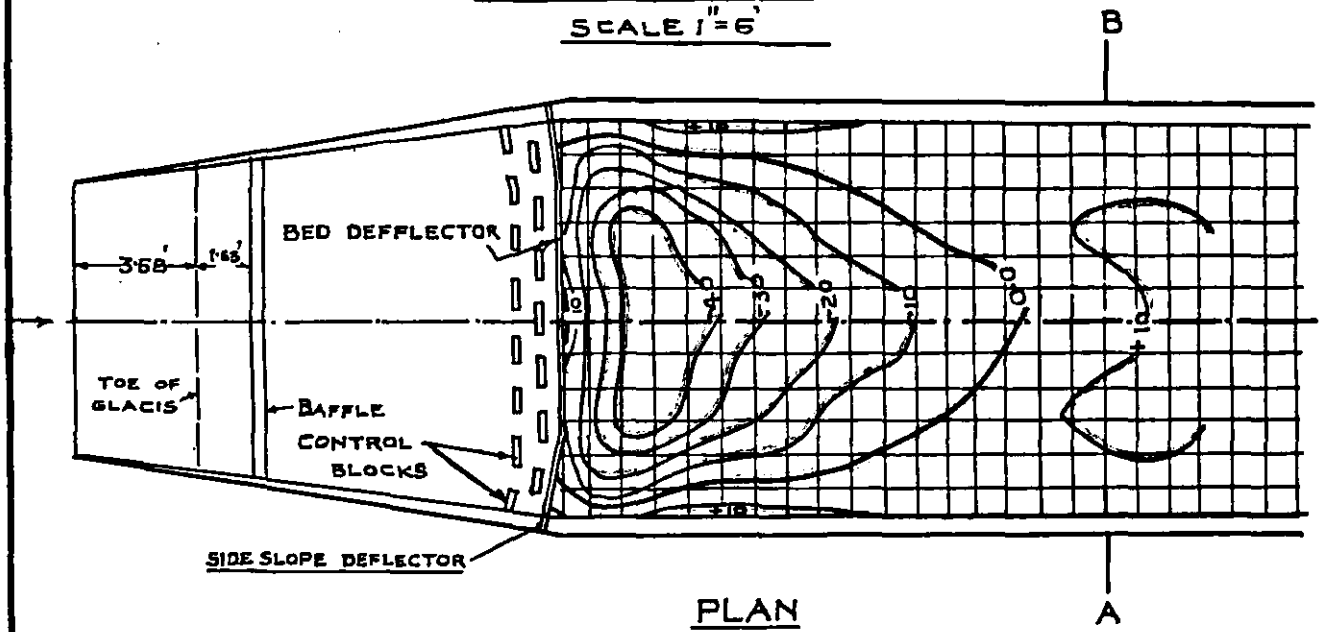
DISCHARGE = 12.5 C.S. = 700 C.S.

$d_1 = d_3 = 1.02 \text{ FT} = 5.1 \text{ FEET.}$

$H = \text{DROP IN W.L.} = 1.59 = 7.95 \text{ FT}$

RETROGRESSION 25 %

SCALE 1" = 6'



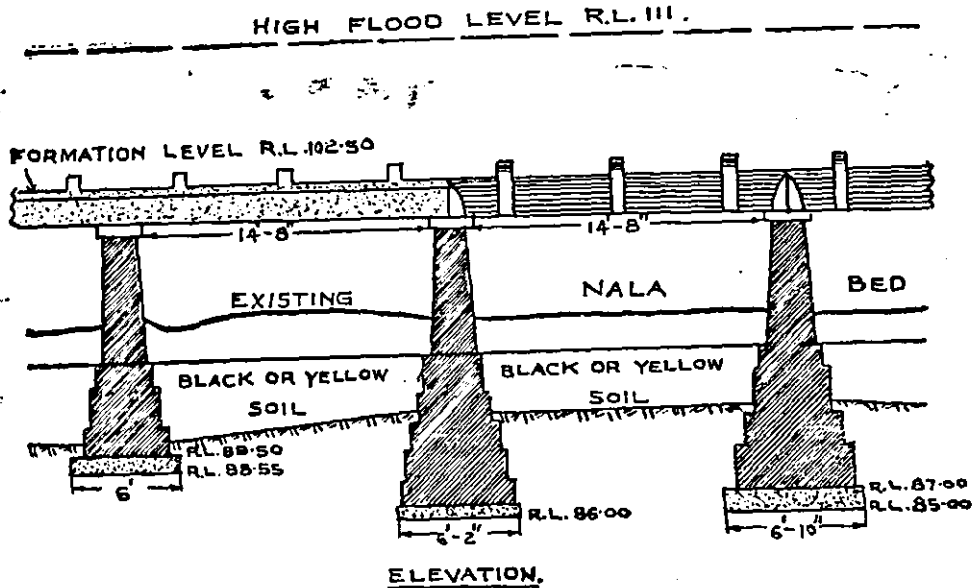
C.S. ON AB SHOWING ISOTACHS.

Experiments were carried out with models ranging from 1/16 to 1/7.4 scales, flood levels ranging from slab level to double the height of bridge, and Froude

C.I. & H.R.S.
No. 1238

SUBMERSIBLE BRIDGE
OVER
KALAMBA NALA (C.P.)
SCALE 1"=10'

FIG: 51

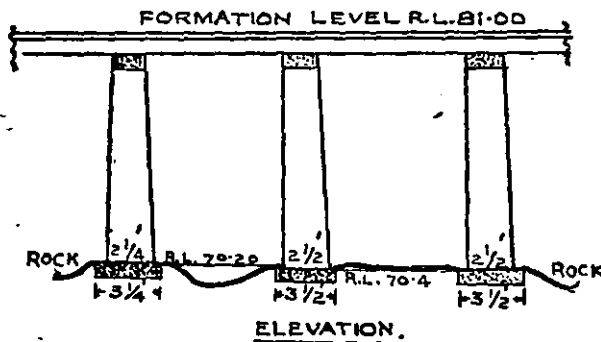


SUBMERSIBLE BRIDGE
OVER
KHARMER RIVER
SCALE 1"=10'

1936 FLOOD WHICH CAUSED DAMAGE.

R.L. 87.75

PREVIOUS HIGH FLOOD LEVEL R.L. = 84.0



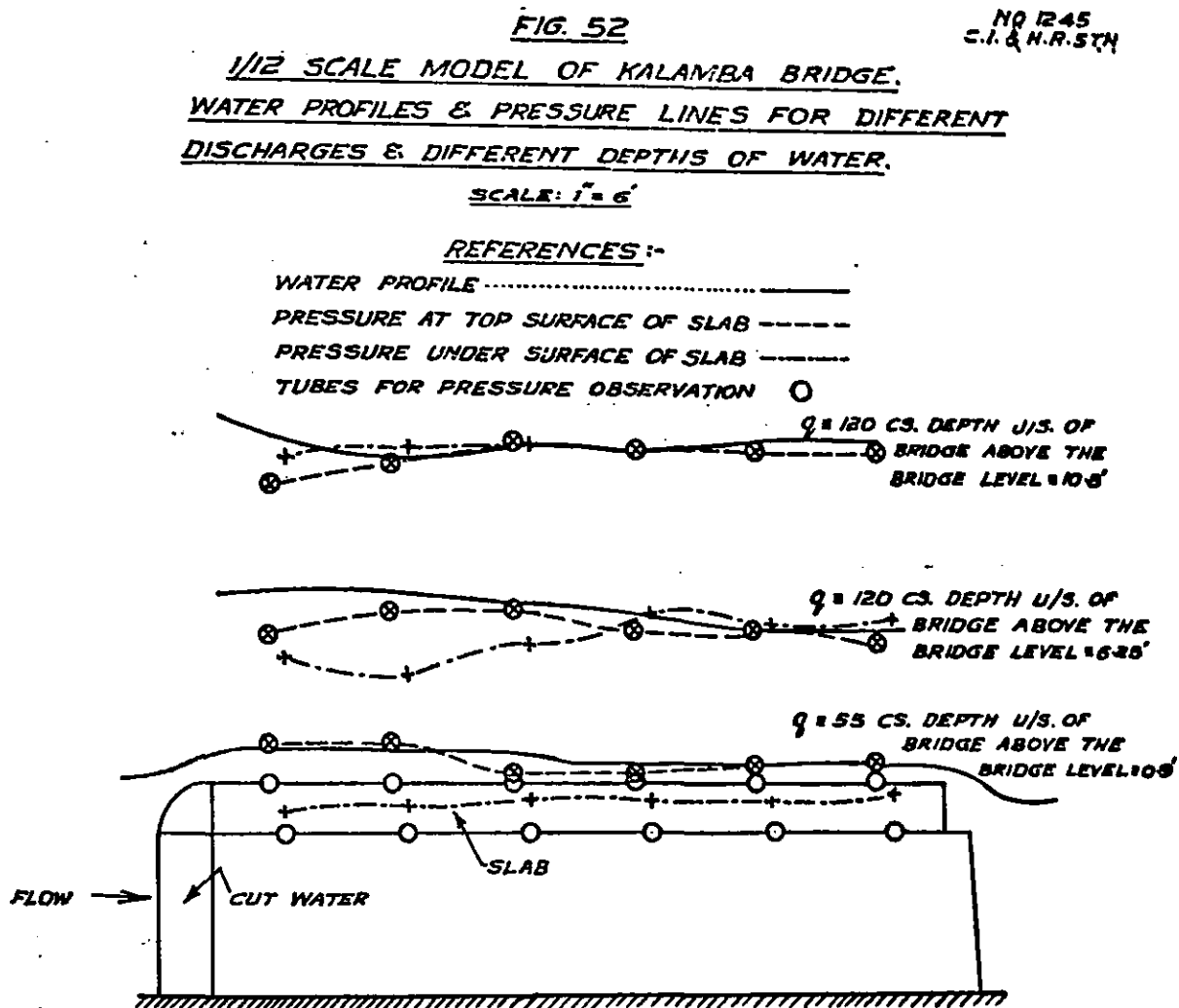
No. from 0.03 to 0.9 i.e. nearly to hypercritical flow. The slab, however, did not move or show any tendency to be lifted with steady normal flow.

In order to carry it away, the slab was then subjected to various conditions of flow unlikely to occur in normal circumstances. A standing wave was made to form beneath the bridge, touching the under surface of the slab. A wave was forced up the channel, submerging the bridge when it reached it, and sudden floods were let down from above. It was only when the flow was dammed close upstream and then suddenly released in the form of a high advancing wave enveloping the bridge with considerable shock that the slab was eventually carried away. There does not seem to be any possibility of such conditions having occurred, and this could not be the explanation of the repeated failures in the Central Provinces.

Pressure observation.

Pressures on the upper and lower surfaces of the slabs were observed under steady flow conditions by means of a fixed slab with pressure tubes inset flush

with the surfaces and connected to manometers. Some of the results are shown in figure 52.



By comparison with the surface profiles, which are also shown, it will be seen that the pressures recorded were very little different from the static pressures due to depth of water. The net uplift pressure is the difference between the pressures on the lower and upper surfaces of the slab. In no case was there appreciable uplift—the maximum at any point was 3.5 lbs. per sq. ft. while the average uplift from upstream to downstream did not exceed 1.4 lbs. per sq. ft. Compared with these the weight of concrete slab in water was about 10 lbs. per sq. ft. As the model was 1/12 scale, the above pressures were equivalent to 42 and 17 lbs. per sq. ft. respectively in the prototype, and the weight of slab in water 120 lbs. per sq. ft.

Pressures near the upstream edge, both on top and under surfaces tended to be lower than static due to the curved flow around the edge, the resultant being downwards, holding the upstream end of the slab down on its seating, particularly at higher values of F. N.

When the upstream water level was at the top of the slab there was a small total downward pressure, tending to hold the slab on its supports.

Uplift due to instability or turbulence.

As it was clear that uplift pressures due to steady flow were small and as attempts to cause a concrete slab to be carried away under normal conditions had not been successful, a light, hollow slab was substituted, with an arrangement for changing the weight of the slab by inserting thin sheets of metal. It was thus possible to reduce the weight sufficiently for the slab to be carried away, the frictional resistance at the supports due to its weight being exceeded by the combined horizontal impact force of the current on the upstream face and frictional drag of the flow on the upper and lower surfaces.

Under such conditions, the light-weight slab was found to slide downstream on its supports, slowly and almost imperceptibly, until suddenly it would tip up

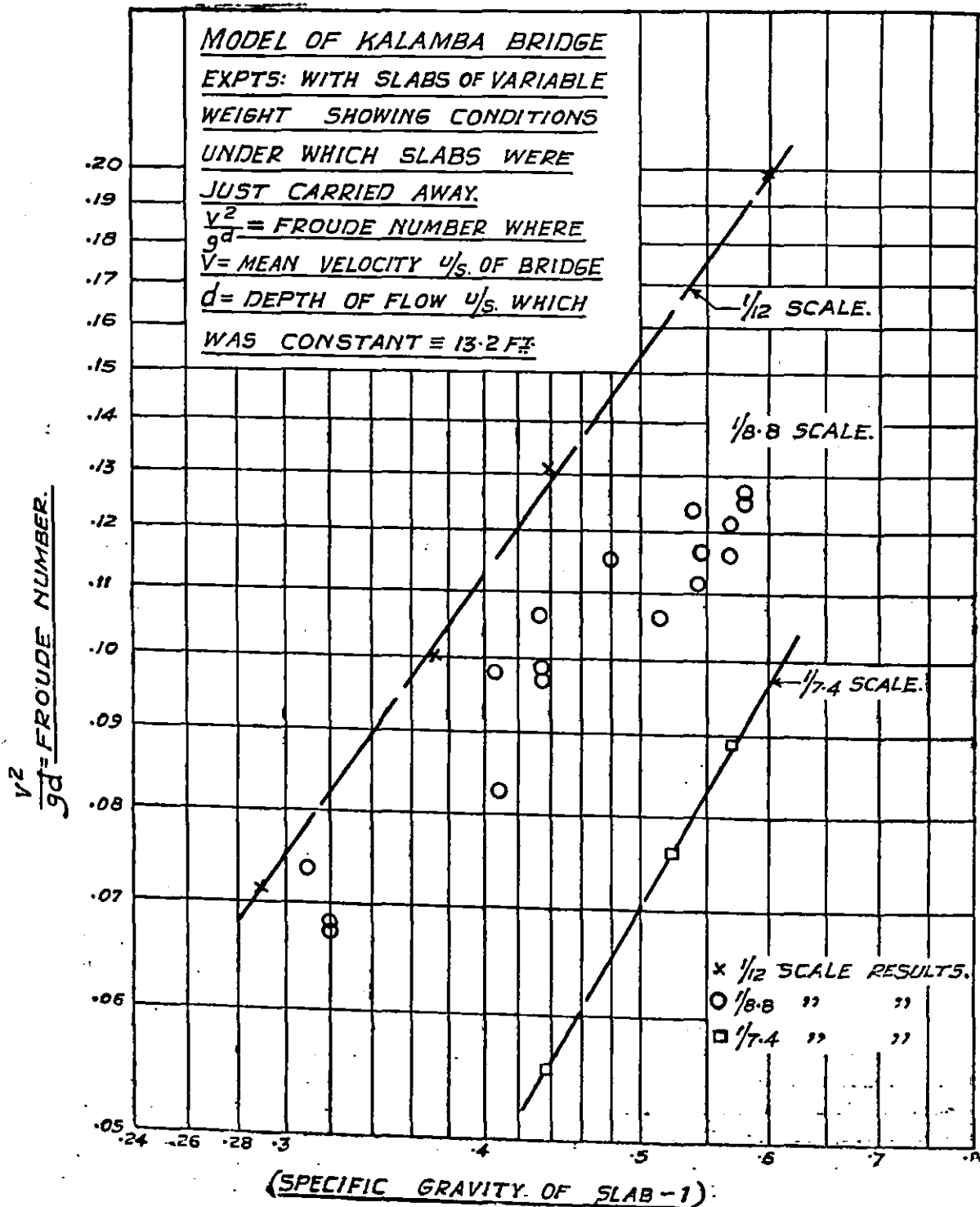
and, pivoting on the downstream end of the supports, be caught by the current and deposited downstream on the bed of the channel.

This process, when observed, immediately suggested what might be taking place in the prototype; owing to the turbulence or instability of flow, the slab must be subjected to rapidly fluctuating pressures upwards and downwards, the former of which would momentarily decrease the effective weight of the slab and allow the horizontal force of the current to push the slab along on its supports in short jerks with result similar to what was observed with the light-weight slab. This result, depending on turbulence, and therefore essentially on the R. N., would increase with increase in scale; so that if the river in the prototype were sufficiently large, the effect would be sufficient to allow even concrete slabs to be carried away. This scale effect was tested by constructing models of Kalamba Bridge to 3 different scales—1/12, 1/8.8 and 1/7.4.

The conditions were observed under which slabs of various weights were just moved on their supports, maintaining similar depth of flow, to scale, in each case. The results are shown in figure 53.

FIGURE 53.

No 124
C.I.&H.I



While the scatter was too great to allow a definite correlation to be derived or results to be extrapolated to prototype scale, there is clear indication that the specific gravity of slab, which will just be carried away by the current, increases with scale, for any given value of F. N., and therefore that unanchored concrete slabs may be expected to be carried away in this manner in large rivers.

Confirmation was obtained by the observation of the horizontal force necessary to apply in order to move the light-weight slab at various velocities, the upstream depth of flow being maintained constant. The results obtained are shown in Table 7 below, where

P_1 = Calculated horizontal force due to impact and surface friction
 $= 0.2 V^2$ lbs.

P_2 = Applied force, lbs.

$P = P_1 + P_2$ = total horizontal force necessary to start movement of slab.

d = Upstream depth of flow = 1.1 ft.

V = Mean velocity upstream.

W = Total weight of slab in water = 11.8 lbs.

TABLE 7.

| V ft./sec. | F. N. $V^2/gd.$ | R. N. Vd/v | P_1 lbs. | P_2 lbs. | P lbs. | P/W |
|---------------|--------------------|--------------------|---------------|---------------|-----------|-------|
| 0 | 0 | 0 | 0 | 3.05 | 3.05 | 0.259 |
| 1.33 | 0.05 | 1.46×10^5 | 0.35 | 2.75 | 3.10 | 0.263 |
| 1.70 | 0.08 | 1.87×10^5 | 0.58 | 2.40 | 2.98 | 0.253 |
| 2.08 | 0.12 | 2.29×10^5 | 0.87 | 1.50 | 2.37 | 0.201 |
| 2.46 | 0.17 | 2.71×10^5 | 1.21 | 0.25 | 1.46 | 0.124 |

It will be seen that, as the velocity of flow increased, progressively less total horizontal force P, sufficed to start the movement of the slab. As the uplift pressure due to steady flow was insufficient to reduce the effective weight of the slab to the extent to produce such results, the explanation must be the effect of fluctuating pressures due to turbulence.

The ratio P/W given in the last column with $V=0$, viz. 0.239, represents the coefficient of friction between slab and supports in water. Owing to the smooth metal face of the hollow slab this was very much less than with concrete slabs, when it was about 0.70.

A record of the turbulence of flow above the slab was obtained in the 1/8.8 scale model ($d=1.5$, F. N. = 0.125 and R. N. = 3.7×10^5) using the hot-wire apparatus described elsewhere (pages 36-37) in this Report and is shown in figure 56 of Part II. The turbulence shown in (f), (g) and (h) is local in close proximity to the surface of the slab. The inherent turbulence of the stream (e) would also be responsible for the fluctuations in pressure on the slab.

Rounded upstream Edge.—The effect of rounding the upstream edge of the slab was tested by observing the horizontal applied force necessary to move the slab under high flood conditions. It was found that a slab with rounded top edge was more stable, and with rounded lower edge less stable than with square edge. The difference, however, was small. Rounding the upstream edge, of course, reduces afflux and impact force.

Conclusions.

(1) Steady uplift pressures on submersible bridges are small at all stages of floods and are unlikely to be the direct cause of the carrying away of slabs.

(2) An unanchored slab may gradually slide downstream as a result of turbulence of flow, and then tip up and be carried away.

(3) Accumulation of floating trees against the upstream edge of the slab would increase the horizontal force and hence the tendency of the slab to be carried away.

(4) Rounding the upper upstream edge of the slab increases stability to a small degree but rounding the lower edge reduces it.

(5) In general, heavy slabs are more stable than light ones and any means of preventing the slabs from sliding downstream on their supports would reduce danger of damage by floods.

PART II.—BASIC EXPERIMENTS:

10. RECORD OF TURBULENCE BY HOT WIRE AND OSCILLOGRAPH.

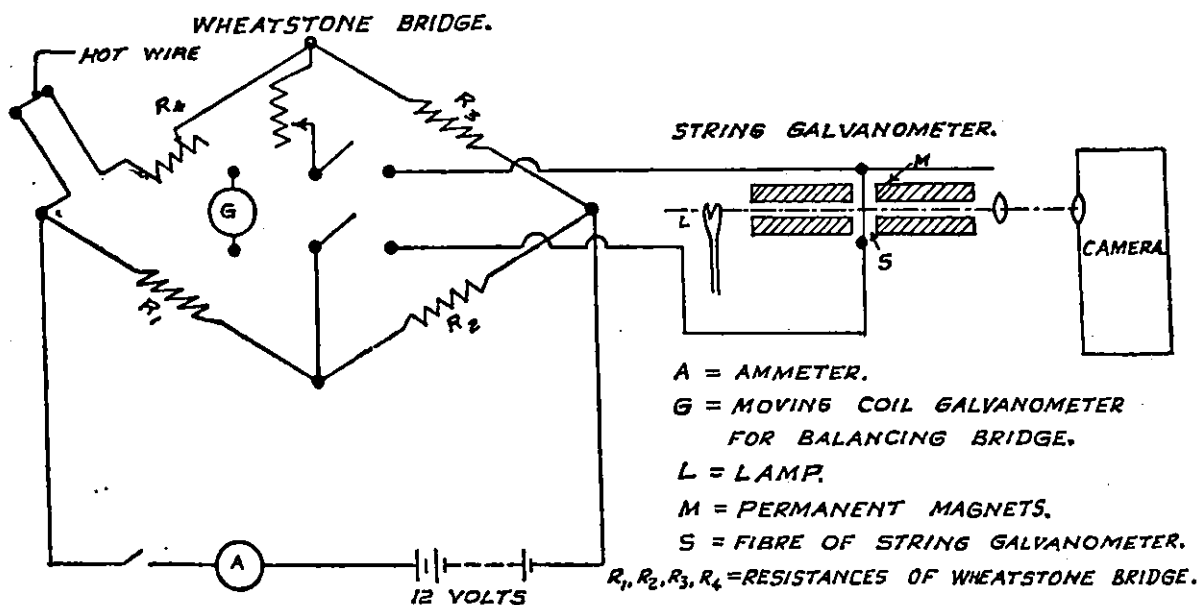
The Hot Wire Anemometer has been used for some years to detect and record turbulence in air streams, being of particular value in exploring the boundary layer.

A similar apparatus was obtained by this Station in 1931 for recording the turbulence of flowing water, and has been developed with the assistance of the National Physical Laboratory, London. Special difficulties were, however, encountered in its application to the flow of water in the portable form required for use with models, and it was not until the present year, after the substitution of a very sensitive string galvanometer not previously available, that satisfactory results were obtained.

Principle of action.

The circuit diagram of the apparatus is shown in figure 54.

FIG. 54 CIRCUIT DIAGRAM
OF
HOT WIRE APPARATUS



In brief, the hot wire consists of a short platinum wire (in this case 1 cm. long and 0.1 mm. dia.), through which an electric current is passed, sufficient to heat it. The wire when placed in flowing water is cooled to a degree which depends on the speed of flow. The cooling reduces the resistance and so induces a greater flow of current through the wire, thus the higher the speed the greater the current.

If the flow of water is laminar, the speed and therefore also the electric current are constant. In turbulent flow, however, the hot wire is subjected to a fluctuating velocity, the amplitude and pattern of the fluctuations depending on the degree and pattern of the turbulence. The flow of electric current therefore fluctuates in a corresponding manner, and when the fluctuations, rendered visible by the vibrating string of the galvanometer, are projected on to a moving-paper camera, a recorded picture of the turbulence is obtained.

The hot-wire circuit contains a Wheatstone Bridge, which is balanced beforehand by means of a moving coil galvanometer (see figure 54), so that only the plus and minus fluctuations are passed through the sensitive string galvanometer.

FIG:-55 RECORD OF TURBULENCE
BY HOT WIRE AND OSCILLOGRAPH
IN 1/10 SCALE MODEL OF 700CS. S.W.F. FALL.

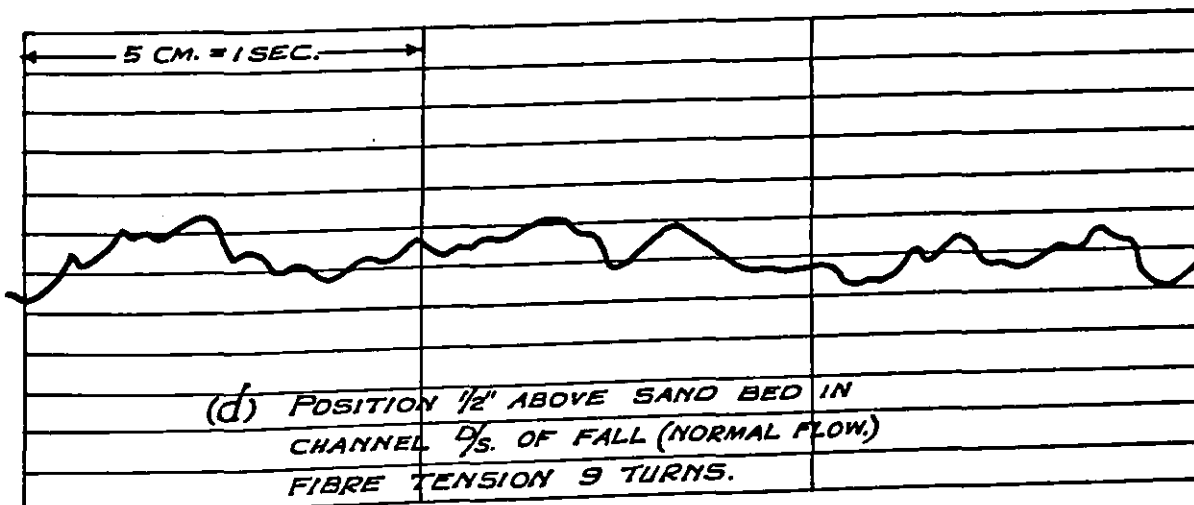
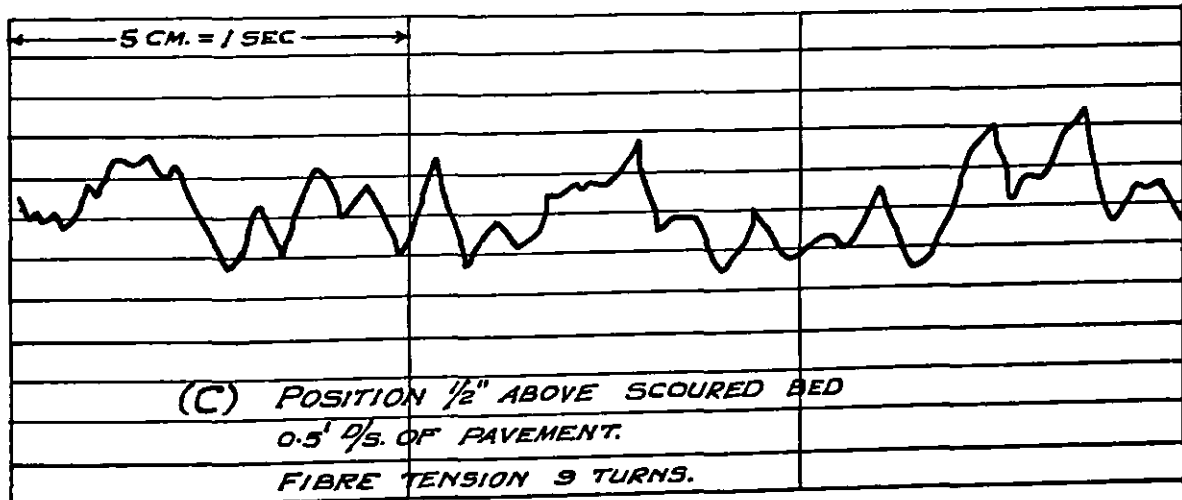
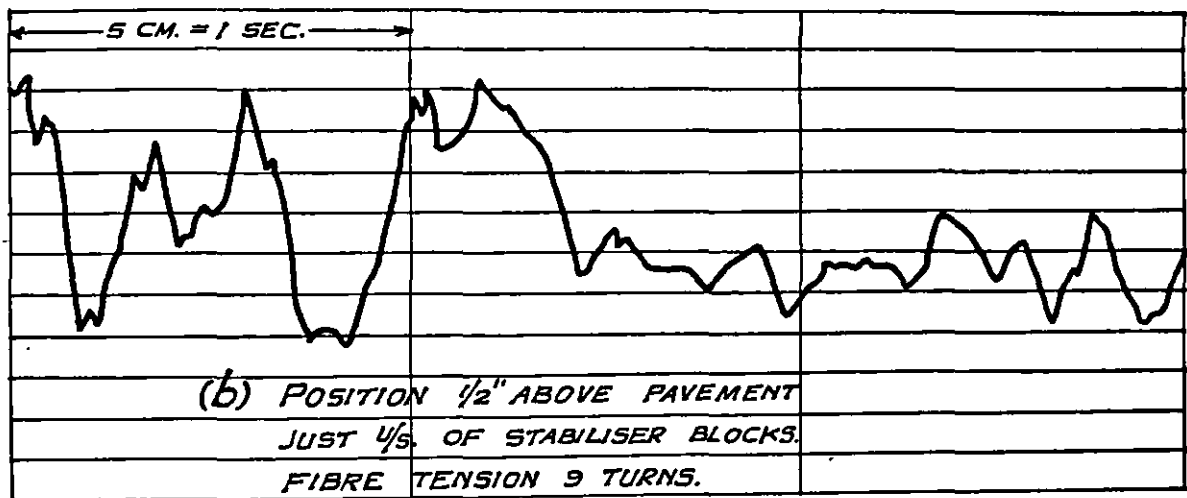
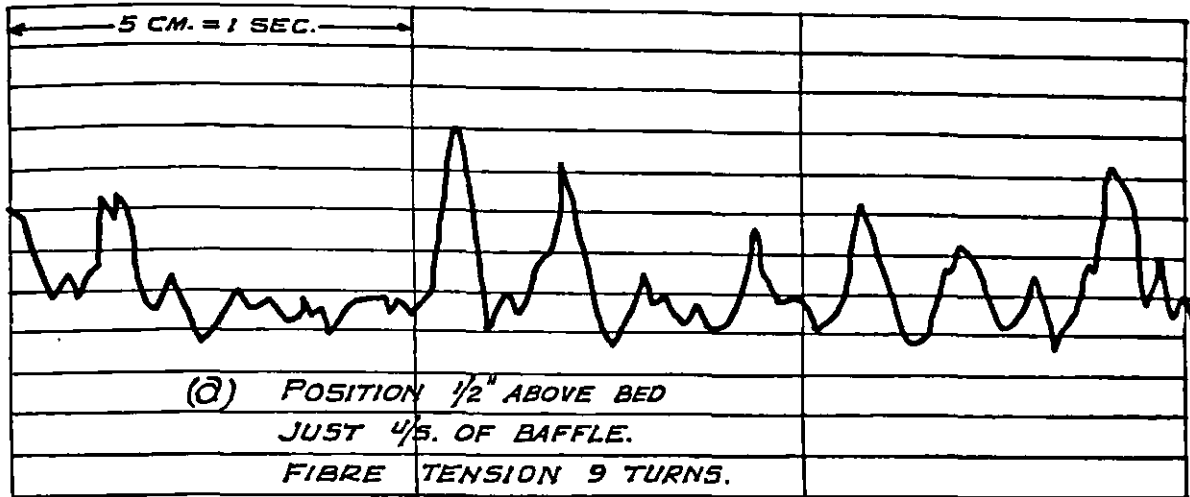
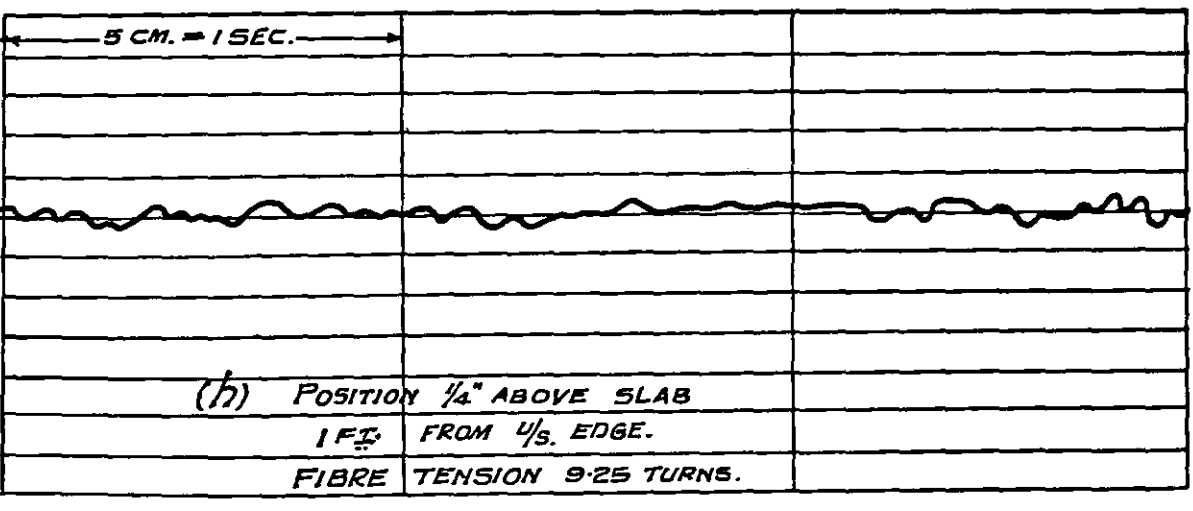
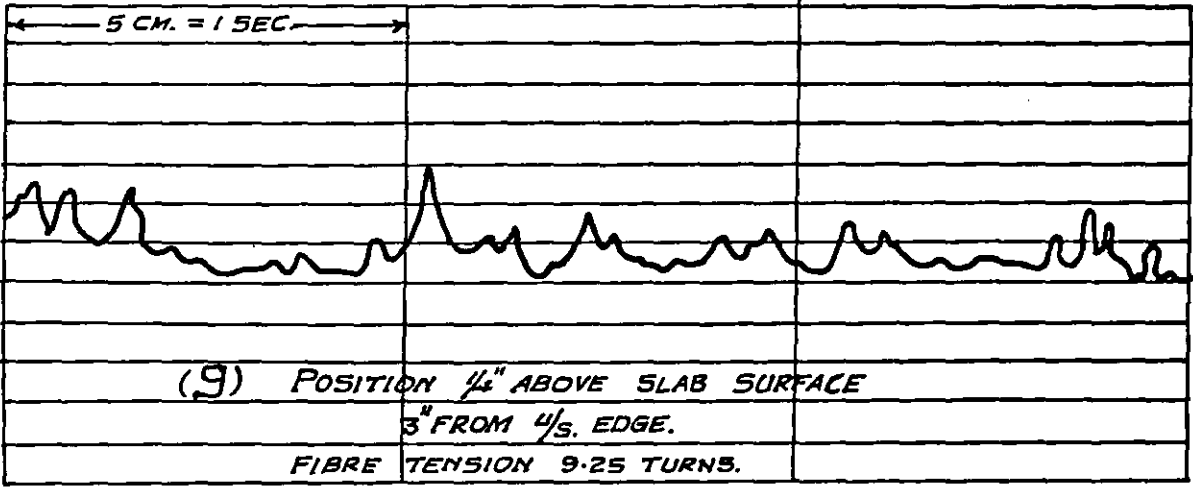
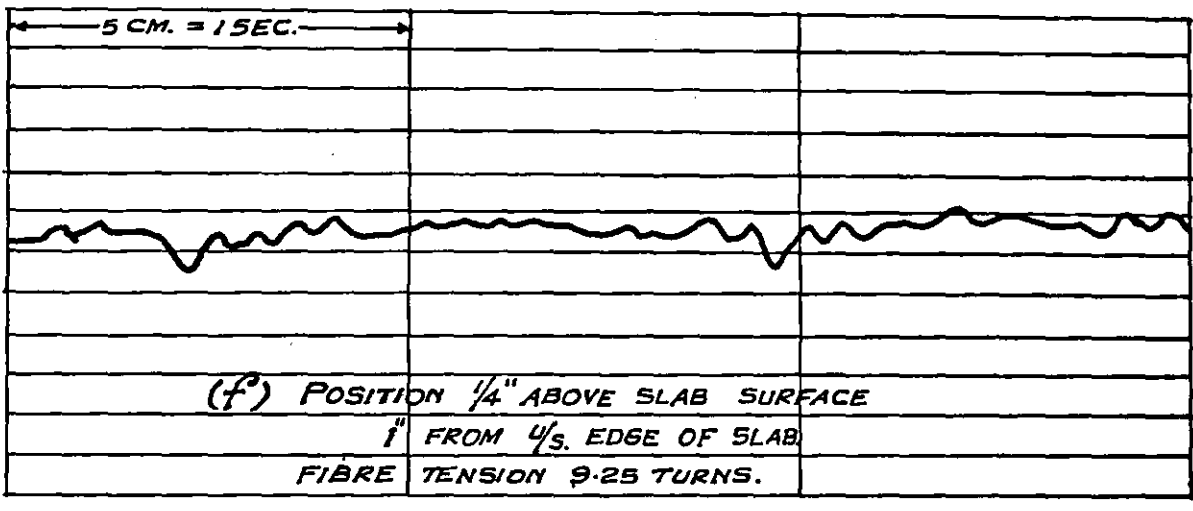
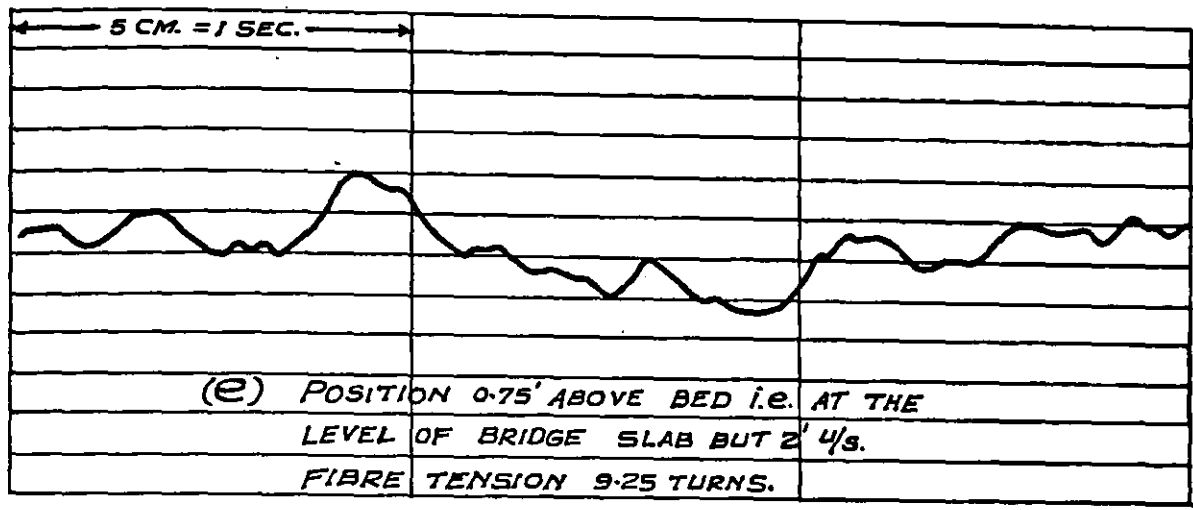


FIG:-56 RECORD OF TURBULENCE
BY HOT WIRE AND OSCILLOGRAPH
IN 1/88 SCALE MODEL OF
SUBMERSIBLE BRIDGE.



Results.

Some results obtained in models of a S. W. F. Fall and a submersible bridge are shown in figures 55 and 56.

The positions in the former, in which the hot wire was placed, may be found in figure 46 of Part I with the aid of the descriptions given below each of the diagrams in figure 55. In Figure 55: (a) gives a picture of the turbulence beneath the standing wave, and (b) downstream of the secondary wave. The violent fluctuations of the latter are reduced in (c), above the scoured bed, and considerably damped in (d), some distance downstream, where the amplitude is reduced and the peaks are rounded.

Figure 56 shows the turbulence in the 1/8.8 scale submersible bridge model described on pages 31 to 35. The turbulence here is of a much less violent nature, the amplitude being less, although the fibre tension of the galvanometer was slightly reduced to increase sensitivity. The transition from the inherent low frequency, rounded, turbulence of the upstream channel (e) to the more rapid fluctuations induced by the flow around the sharp upstream edge of the slab, first increasing in amplitude (f) and (g) and later becoming damped by the proximity to the upper surface (h), is clearly visible.

11. EXPERIMENTS WITH FALLING APRONS: EFFECT OF DISPOSITION OF STONE. SERIES VIII.

Introduction.

One of the features of the launching of loose stone aprons for the protection of banks, brought out by the experiments described in the Annual Report of this Station for 1937-38, was the fact that the original disposition of the stone—the width and thickness of apron—has little influence on the quantity of stone in the launched slope.

As the general impression, it is believed, is that the quantity of stone launched depends directly on the thickness of apron laid, and as this is a matter of considerable practical importance, further experiments were carried out in Series VIII with falling apron models to test the conclusions arrived at.

Experiments.

The channel was 15 ft. wide, with a bed composed of Ganges sand, which is fine, micaceous and incoherent, having mean diameter=0.29 mm. The stones used were of two kinds; rounded pebbles, collected from a river bed, corresponding in shape to the "one-man" stone used in aprons, but with corners rounded, and angular broken stone. The specific gravity of both these was 2.9.

Experiments 1 and 2 were carried out under identical parallel flow conditions, the only difference being the thickness of original apron. In experiment 1, the horizontal apron when laid was 31 ft. long, 2½ ft. wide and 1" thick, and was composed of ¾"—½" rounded stone. It was launched with a discharge which was increased gradually to 18 cusecs and continued until launching had ceased. Figure 57(a) shows a typical cross section, before and after launching.

The stones which had launched in experiment 1 were gathered and relaid in experiment 2 in a layer double the thickness, *i.e.* 2", and the bed and berm also relaid, as shown in figure 57(b). The width of apron which had not launched was left untouched. The same discharge was then run until launching ceased.

A comparison of figure 57(a) and (b) will show that the same quantity of stone launched in each experiment; and the quantity of stone per sq. ft. of launched apron was found to be 46 c. ins. in each case (*see* table 8). Had the quantity of stone launched been proportional to the thickness of apron, double this quantity would have been launched in experiment 2.

Launching under severe conditions.

Experiments 3 and 4 were similar to 1 and 2 except that the apron was laid on a bend exposed to the force of the current, and the discharge was raised immediately to 15 cusecs, so that the conditions during launching would be more severe. In consequence, the initial launching was very rapid and less regular than before, and about 20 per cent. of the stones launched were carried away by the current. It is to be noted, however, that stones were carried away not when launching from the horizontal apron but from the toe of the bank, due to high velocity flow over sand waves in the bed. This could be reduced, and the apron protection therefore considerably strengthened against severe attack, if stones of a larger size were laid at the outer edge of the apron.

Photo No. 1978 shows the bank after launching. Comparative cross sections are shown in figure 57(c) and (d). The quantity of stone per square ft. of launched apron was 42 and 48 c. ins. respectively (table 8), which difference is small enough to lie within experimental error. Part of the apron was laid in bands of different colours. The two coloured bands on the launched apron (which unfortunately cannot be differentiated in the photo) were inter-mixed only to a small extent, showing that the launching took place by sliding rather than rolling.

In order to observe launching under extremely severe conditions, the apron was relaid in experiment 5 as at the start of experiment 4, and a high velocity current (about 3 ft./sec.) was directed at the bank by means of a curved vane, shown in photo No. 1983, which also shows the result after 20 minutes flow. A cross-section of the bank is given in figure 57(e). As can be seen from the photo, there were large bed waves, and a considerable quantity of stone was swept away from the toe of the bank, due to the rapid increase in velocity before the bed had time to scour to sufficient depth. Under such conditions, the regular line of the upper part of the slope is noteworthy, and the experiment demonstrated the great resistance to attack of this type of protection, provided sufficient stone is available for launching.

Effect of shape and size of stone.

Experiment 6 was exactly similar to experiment 3 except that angular stones of the same grade ($\frac{3}{8}$ "— $\frac{1}{2}$ ") were substituted for the rounded stones. The results (figure 57(f) and table 8) showed that there was no marked difference due to the angularity of the stones, but as would be expected, a smaller quantity was carried away in the launching—a negligible quantity, in fact, in this experiment.

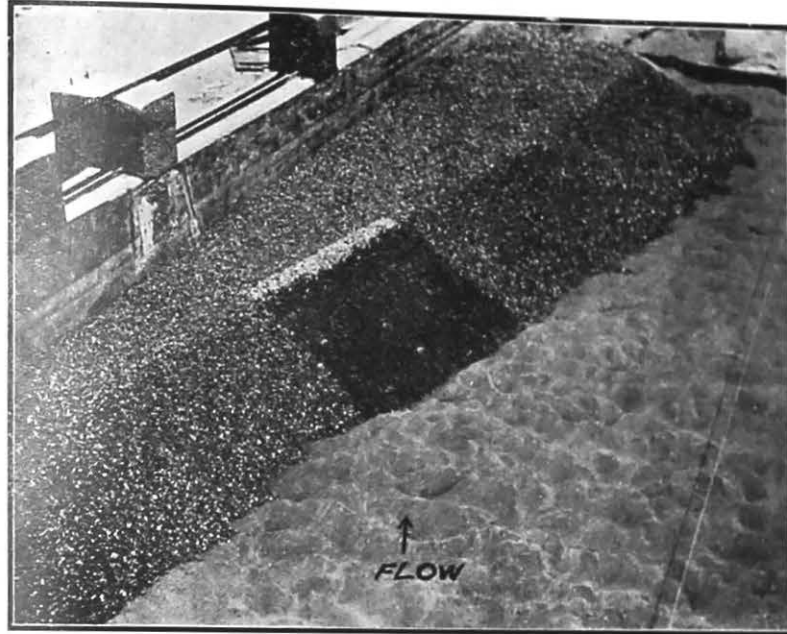
In experiments 7, 8 and 9, the apron was composed of rounded stones of $\frac{3}{8}$ "— $\frac{1}{2}$ ", $\frac{1}{4}$ "— $\frac{3}{8}$ " and $\frac{1}{2}$ "— $\frac{3}{4}$ " grades respectively, 1" thick and laid straight as in experiment 1. Cross sections are shown in figure 57(g), (h) and (i). The results showed little difference in quantity of stone per square foot of launched apron. In all the experiments, the launched apron was only one-stone thick; the larger stones must therefore have been spaced further apart; but it is clear that in general, the larger the stone (giving better protection) the greater the quantity required.

Slope of Bank.

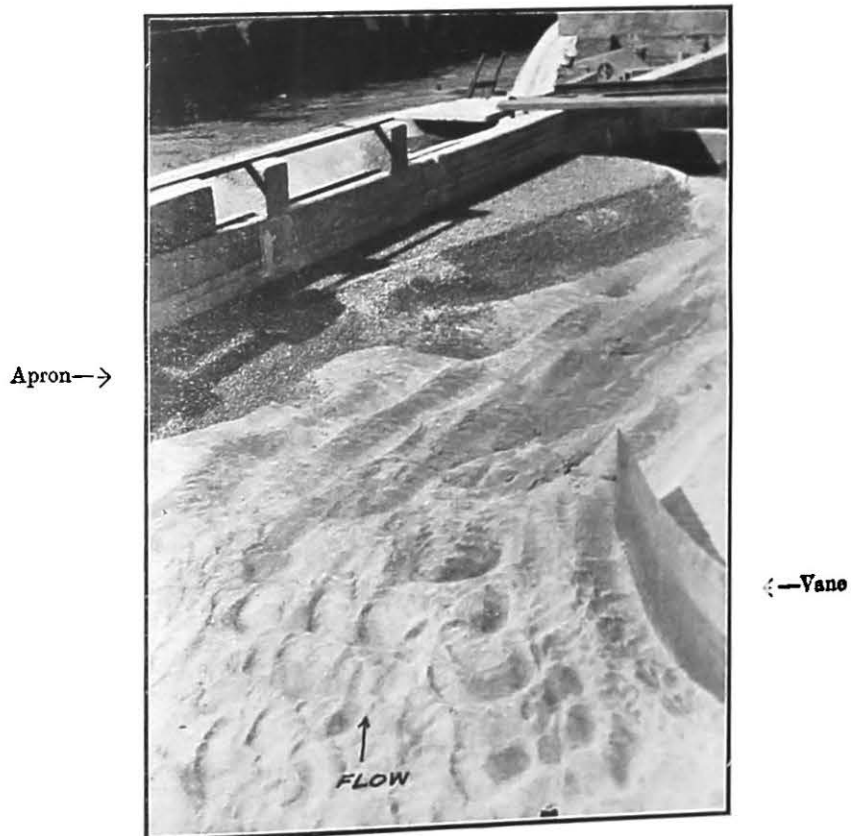
The slope of launched apron (average of 5 cross sections) varied from 1.75 : 1 to 2.35 : 1 (see table 8), rather steeper than the slopes obtained in practice with one-man stone, which is usually about 2.5 : 1. As deduced from observation of the process of launching (see Annual Report for 1937-38) there was a definite tendency for the slope to be flatter under conditions of higher velocity, but the range of slope was small. The slopes of experiments 1, 2 and 7 under parallel flow conditions may be compared with those of experiments 3 and 4 with the apron on a bend, the local velocities of the latter being higher, and with those of experiment 5 under even more severe conditions.

EXPERIMENTS WITH FALLING APRONS.

Effect of disposition of stone.



1978. Experiment 4. Apron after launching under conditions of severe attack.



1983. Experiment 5. Apron after launching under very severe attack, high velocity current being directed at the bank by means of the vane seen at the right bottom corner. Stone carried away may be seen on the bed.

FALLING APRON EXPTS:
SERIES VIII.
CROSS SECTIONS
THROUGH BANK.
SCALE: 1"=1'.

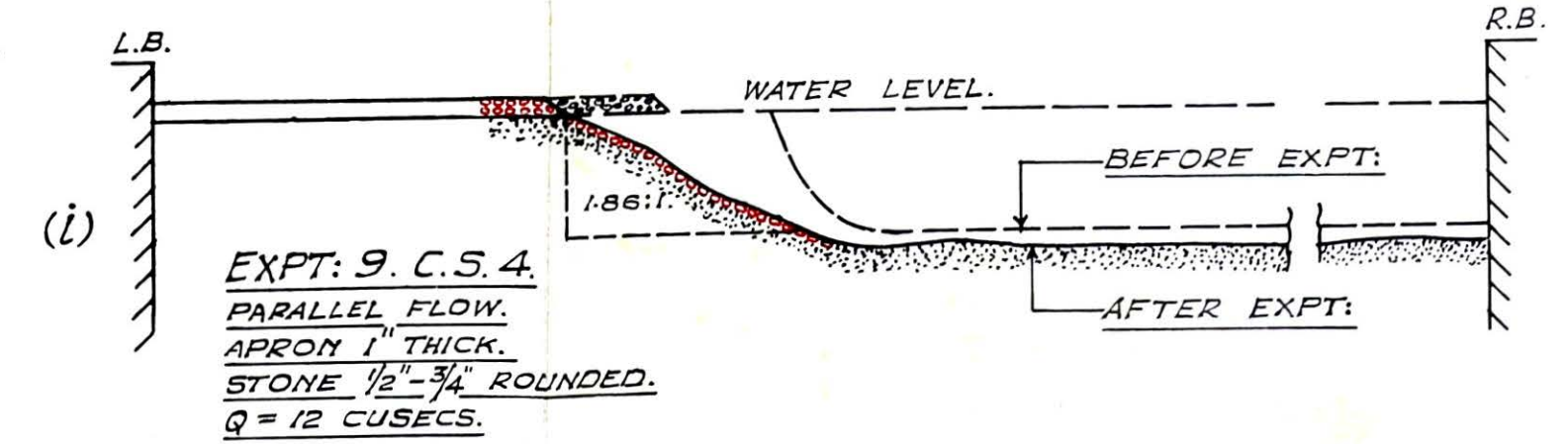
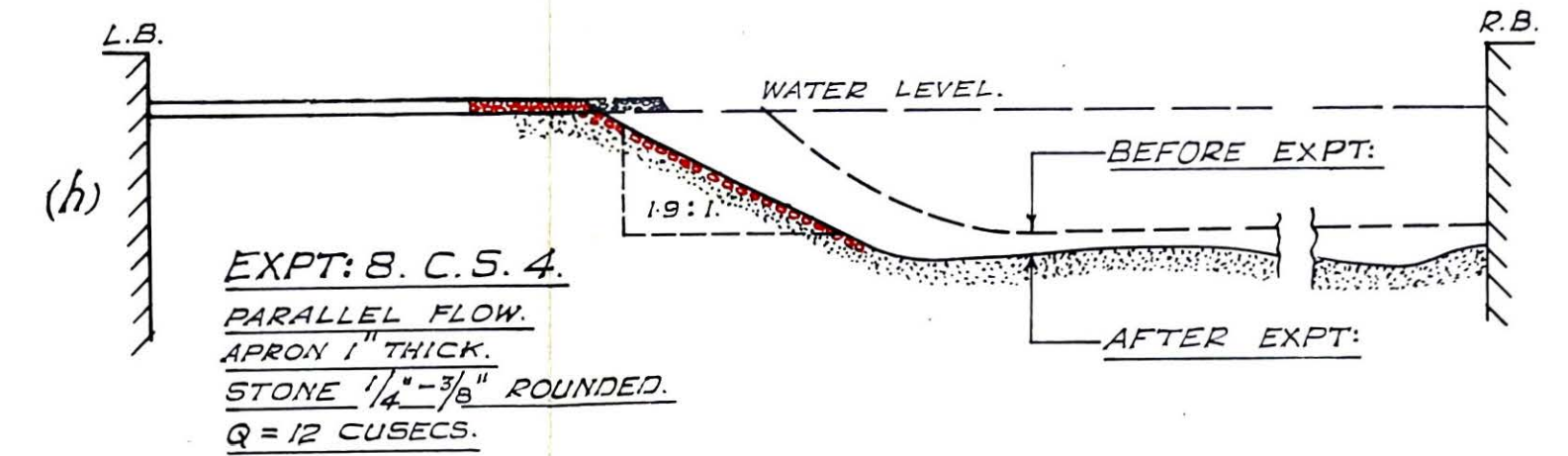
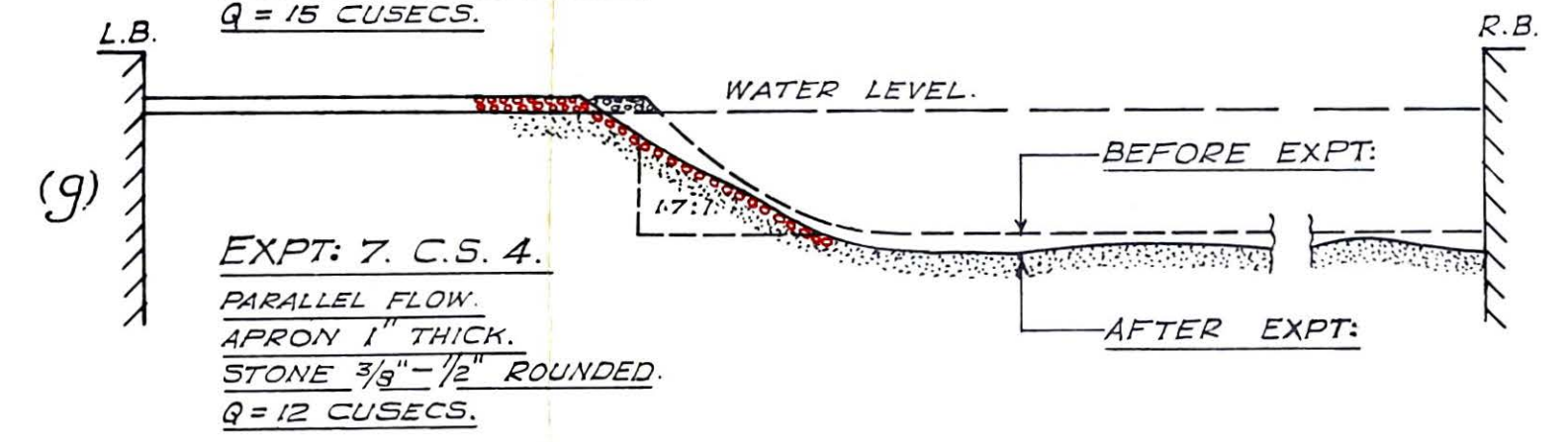
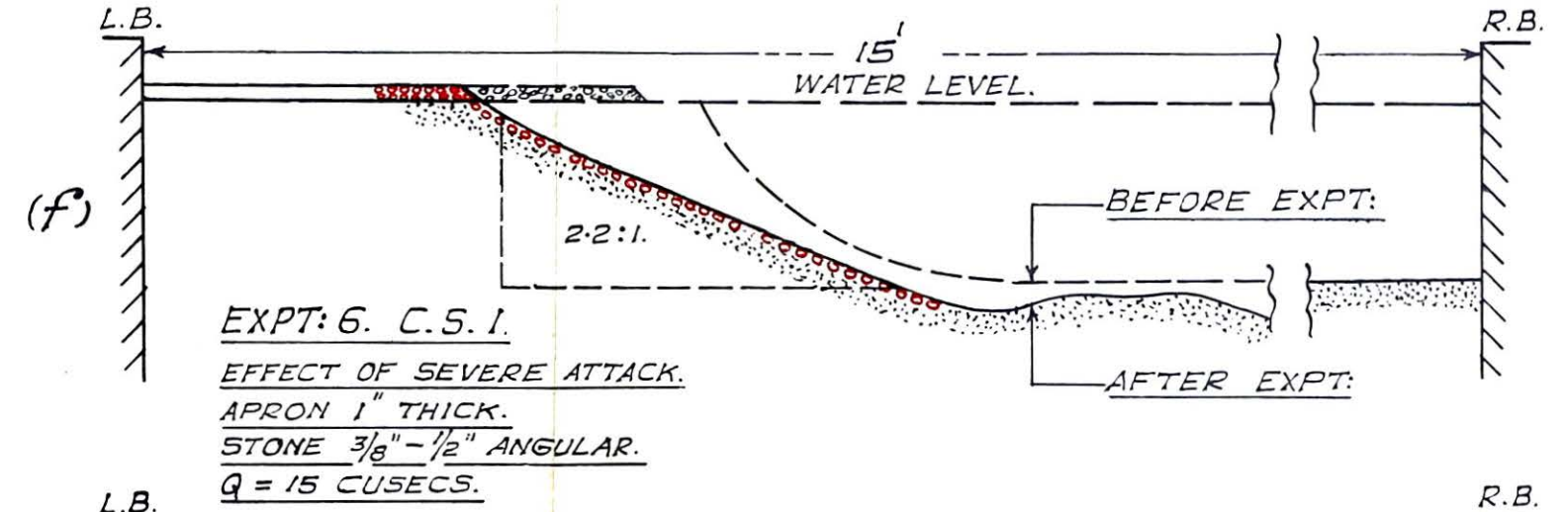
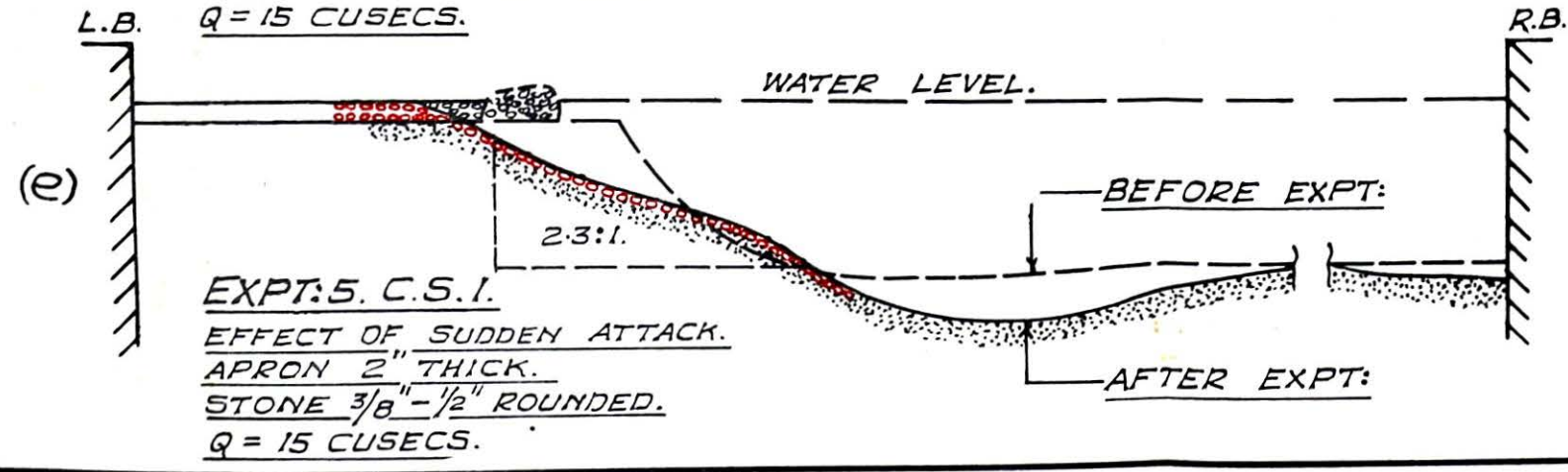
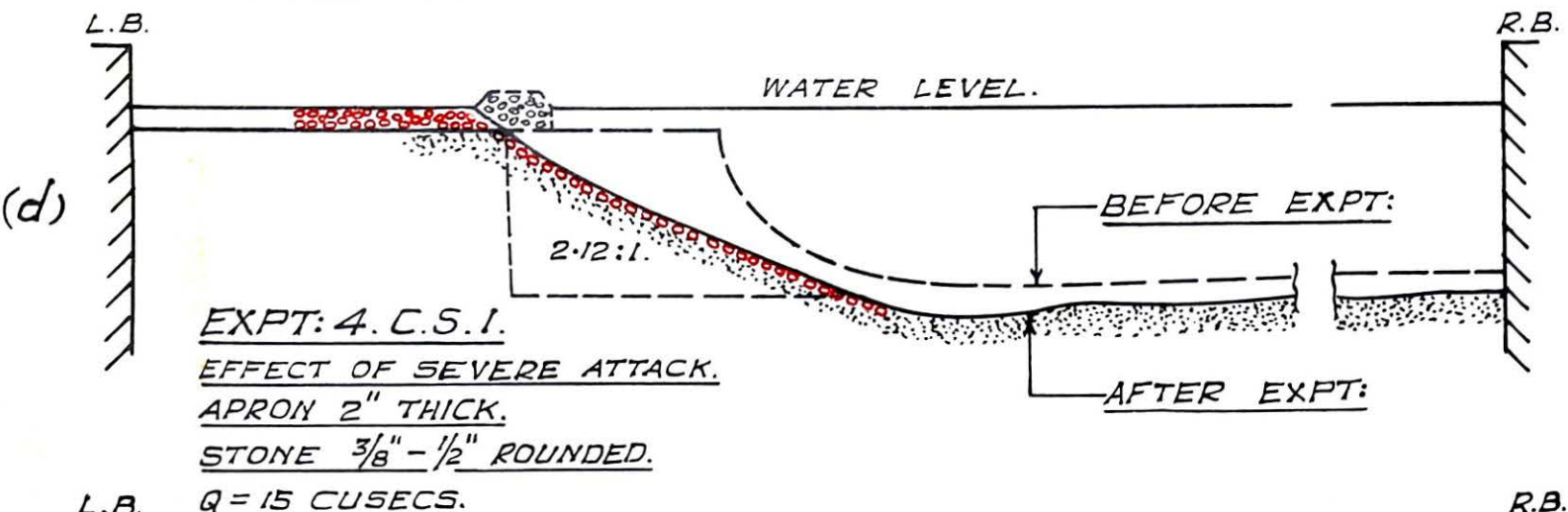
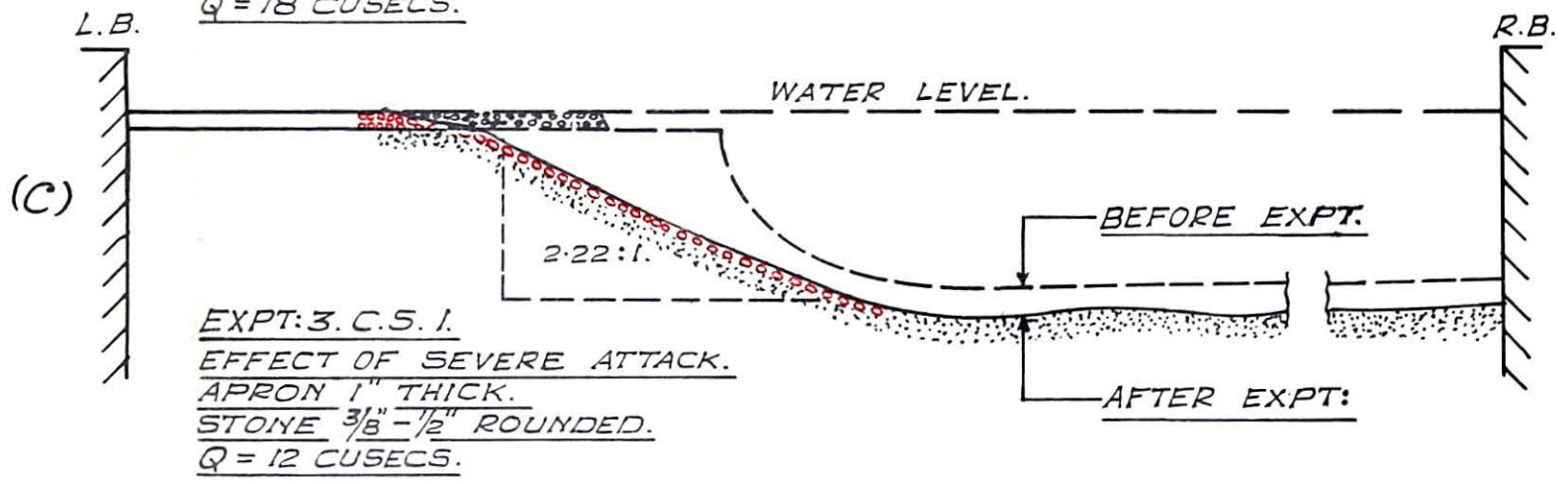
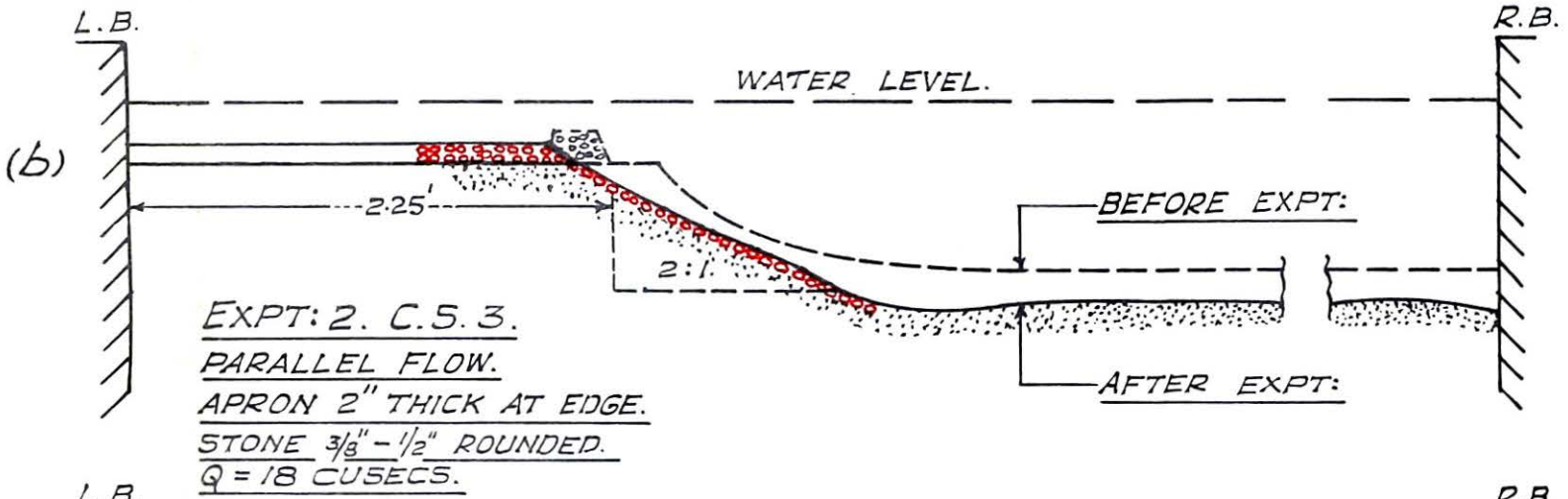
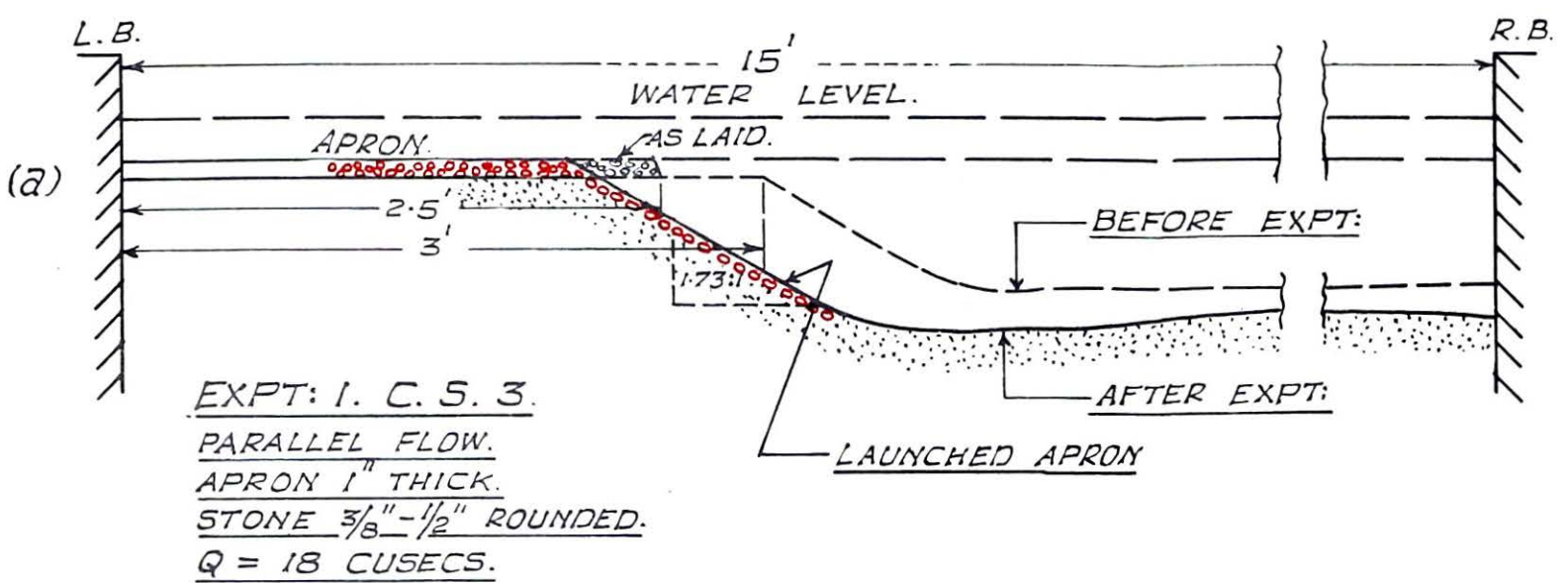


TABLE 8.
Falling Apron Experiments—Series VIII.

| Experiment No. | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 |
|--|-----------------------------|----------|----------------|----------|----------------|-------------------------------|----------------|----------|----------|
| Conditions. | Parallel flow. | | Apron on bend. | | Severe attack. | Name as 3 but angular stones. | Parallel flow. | | |
| Grade of stones . . . | 1"-1" | 1"-1" | 1"-1" | 1"-1" | 1"-1" | 1"-1" | 1"-1" | 1"-1" | 1"-1" |
| Mean weight per stone (lbs.). | .0032 | .0032 | .0032 | .0032 | .0032 | .0038 | .0032 | .0021 | .012 |
| Thickness of Apron at edge | 1" | 2" | 1" | 2" | 2" | 1" | 1" | 1" | 1" |
| Discharge (cusecs) . . | 18 | 18 | 15 | 15 | 15 | 15 | 12 | 12 | 12 |
| Water level | ← 0.2' above apron level. → | | | | | | | | |
| Approximate width of apron launched. | 4.5" | 2.5" | 10" | 4.5" | 6" | 8.5" | 4" | 3.5" | 3.75" |
| Quantity of stone in launched apron (c. ins. per sq. ft.). | 46 | 46 | 42 | 48 | 42 | 46 | 51 | 42 | 46 |
| Average slope of launched apron. | 1.80 : 1 | 2.08 : 1 | 2.25 : 1 | 2.22 : 1 | 2.35 : 1 | 2.11 : 1 | 1.75 : 1 | 2.08 : 1 | 2.08 : 1 |
| Final velocity on slope 1" above stones (ft./sec.). | 0.92 | 1.10 | 1.20 | 1.43 | .. | 1.10 | 0.95 | 1.00 | 0.81 |
| Final velocity on slope 3" above stones (ft./sec.). | 1.15 | 1.36 | 1.46 | 1.60 | .. | 1.46 | 1.20 | 1.32 | 1.14 |

REMARKS.—1. The stones were all rounded pebbles except in experiment 6 when angular broken stone was used.
2. All experiments were continued until launching had ceased except experiment 5 which was continued for 20 minutes only.

Summary of Conclusions.

(i) Regarding the main object of the experiments, the thickness of the stone apron as laid was shown to have no appreciable effect on the quantity of stone in the launched apron. Launching continued until there was sufficient stone on the slope to protect the sand beneath.

(ii) When launching took place under severe conditions, the slope of bank was flatter, but the range of slope under various conditions was not great. The quantity of stone per sq. ft. of launched apron, however, was constant and showed no correlation with the severity of attack.

(iii) Under severe attack, stone is likely to be carried away from the toe, resulting in recession of the slope and launching of more apron. This could be reduced and the protection considerably strengthened by laying larger stone at the outer edge of the apron before launching.

(iv) The results with angular stone were very similar to those with rounded stone, but a smaller proportion of angular stone was lost during launching. From this it may be inferred that angular stone is more economical and offers better protection than rounded stone.

12. THE PROTECTION OF BRIDGE PIERS AGAINST SCOUR.

Introduction.

At the request of the Railway Board, experiments are being carried out to determine the best means of protecting bridge piers against scour, with particular reference to the piers of the Hardinge Bridge (E. B. Railway) across the River Ganges. The object is to find what action takes place, during periods of high flood, to pitching placed round piers, the best methods of protection against straight and oblique flow and whether it is preferable to delay pitching until scour has taken place, when the stones could be dropped in from above.

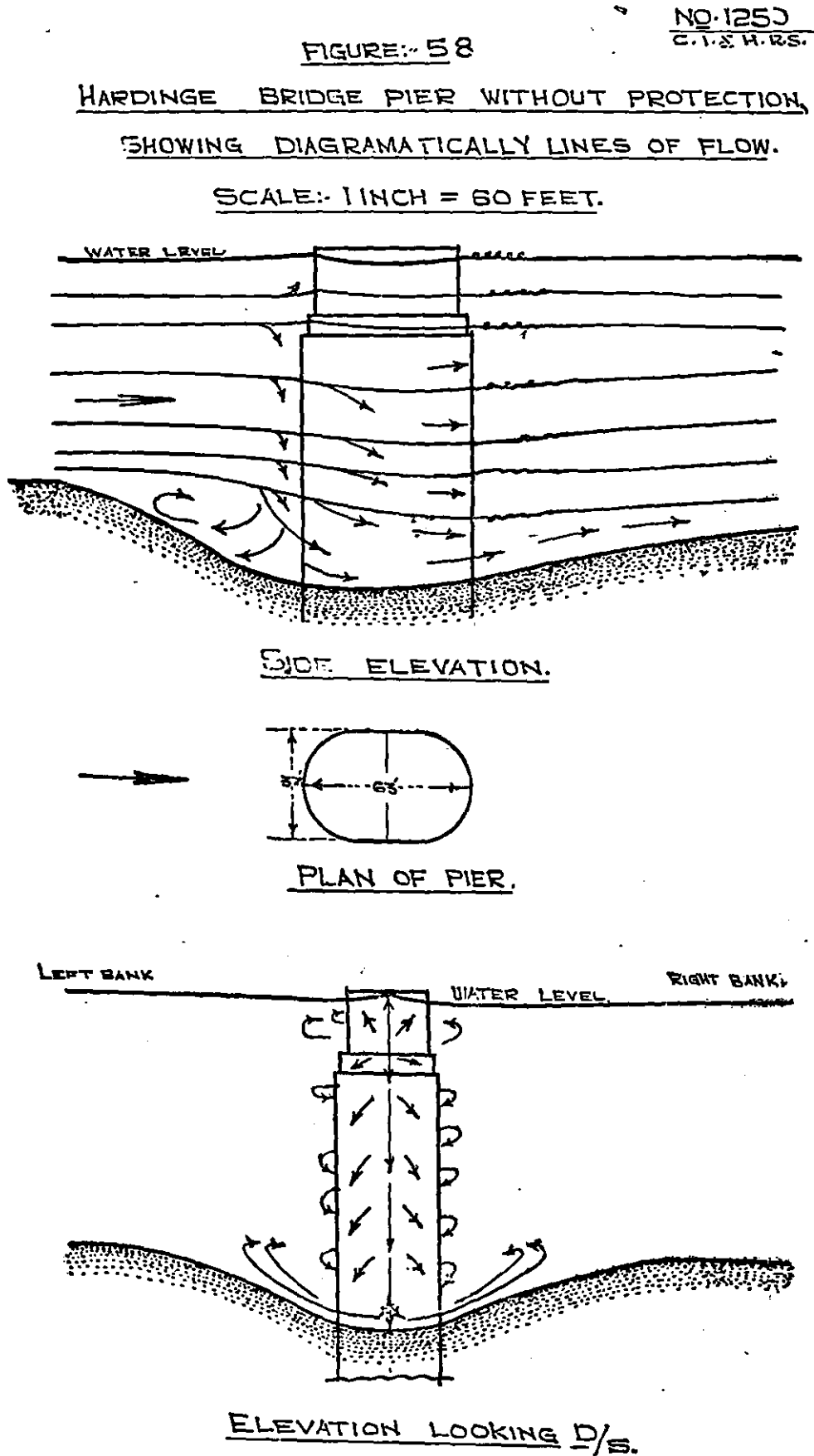
The Hardinge Bridge piers were sunk to a depth of 160 ft.—180 ft. below maximum water level, which may be compared with the Lacey regime depth of 90 ft. and with the maximum depth of scour which has occurred in the river in this vicinity, viz. 210 ft. below maximum water level, at the Raita Guide Bank and 206 ft. at the Sara Guide Bank, 12 miles and 3.2 miles upstream of the Bridge respectively, caused by curved flow round the noses of protection banks. The greatest depth so far observed at the piers,—which have been protected by loose stone,—is 104 ft. below maximum W. L.

Models.

The piers used in the experiments were geometrically similar replicas of those of the Hardinge Bridge. A single pier was placed in the centre of a parallel sided channel, embedded in the sand forming the bed. This was Ganges sand, of mean diameter 0.29 mm., brought from the site of the Bridge.

Scour around unprotected piers.

Figure 58 shows diagrammatically the flow around a pier



Owing to the obstruction to the flow, water spreads at the nose, upwards near the surface, causing a small surface wave, but mainly downwards and to each side, forming the familiar cup-shaped scour pit (see photo 1410).

PROTECTION OF BRIDGE PIERS AGAINST SCOUR.



$\frac{1}{16}$ scale pier.

1410. Showing cup-shaped scour round pier with no stone protection (looking upstream).



$\frac{1}{16}$ scale pier.

1085. Pier with loose stone protection after scour (looking upstream). Pier was fixed with axis at 15° to direction of flow.

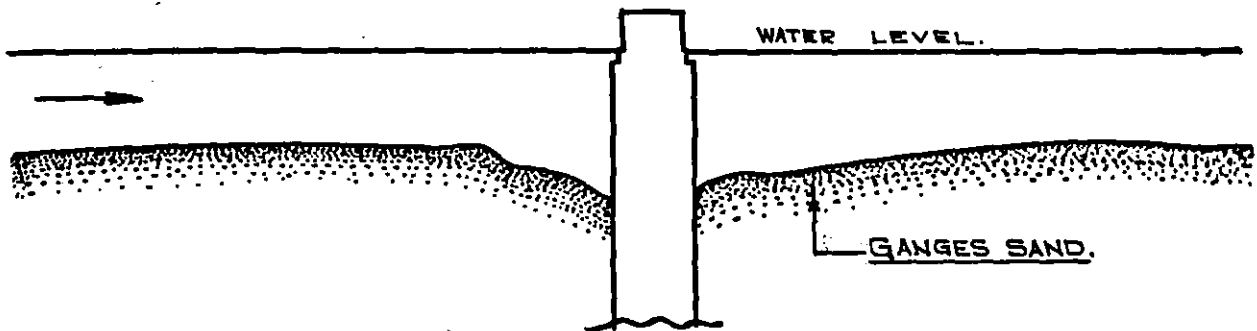
FIGURE:- '59

1/105 SCALE MODEL OF HARDINGE
BRIDGE PIER, IN GANGES SAND
SCOUR WITHOUT PROTECTION.

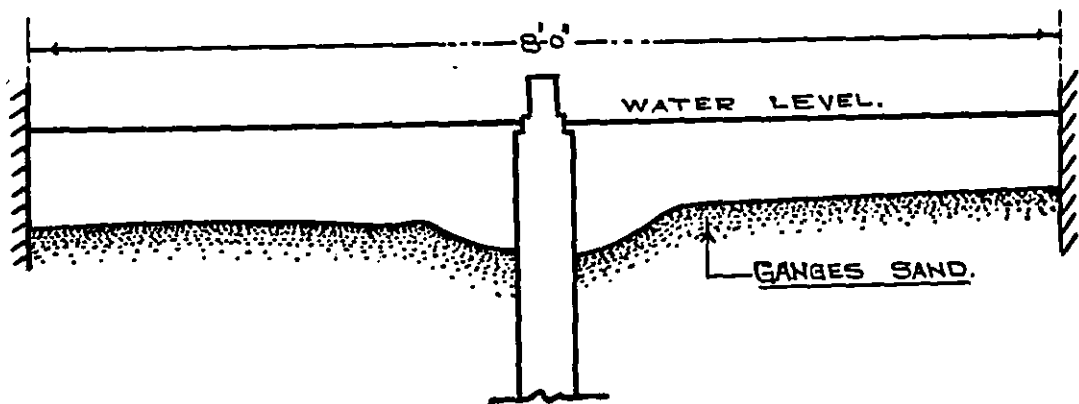
CHANNEL WIDTH = 8 FEET.

Q = 6 CS. q = 0.75 CS./FT

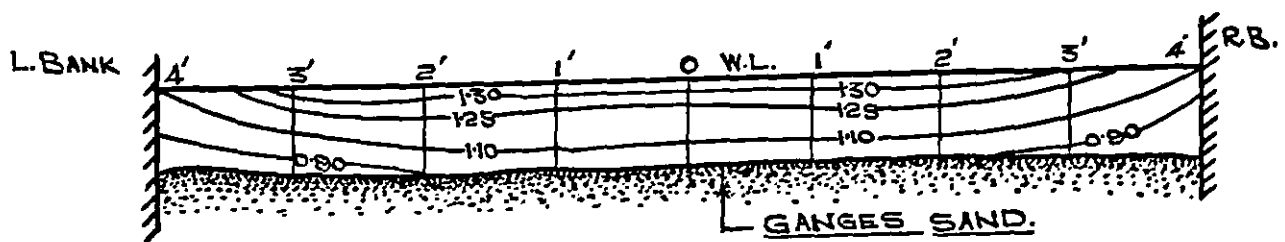
SCALE:- 1" = 1.5 FT.



L. SECTION.



CROSS SECTION.



CROSS SECTION WITH
ISOTACHS 2' U/S OF PIER.

AREA = 5.52 SQ. FEET.

√CAL = 1.09 F/S.

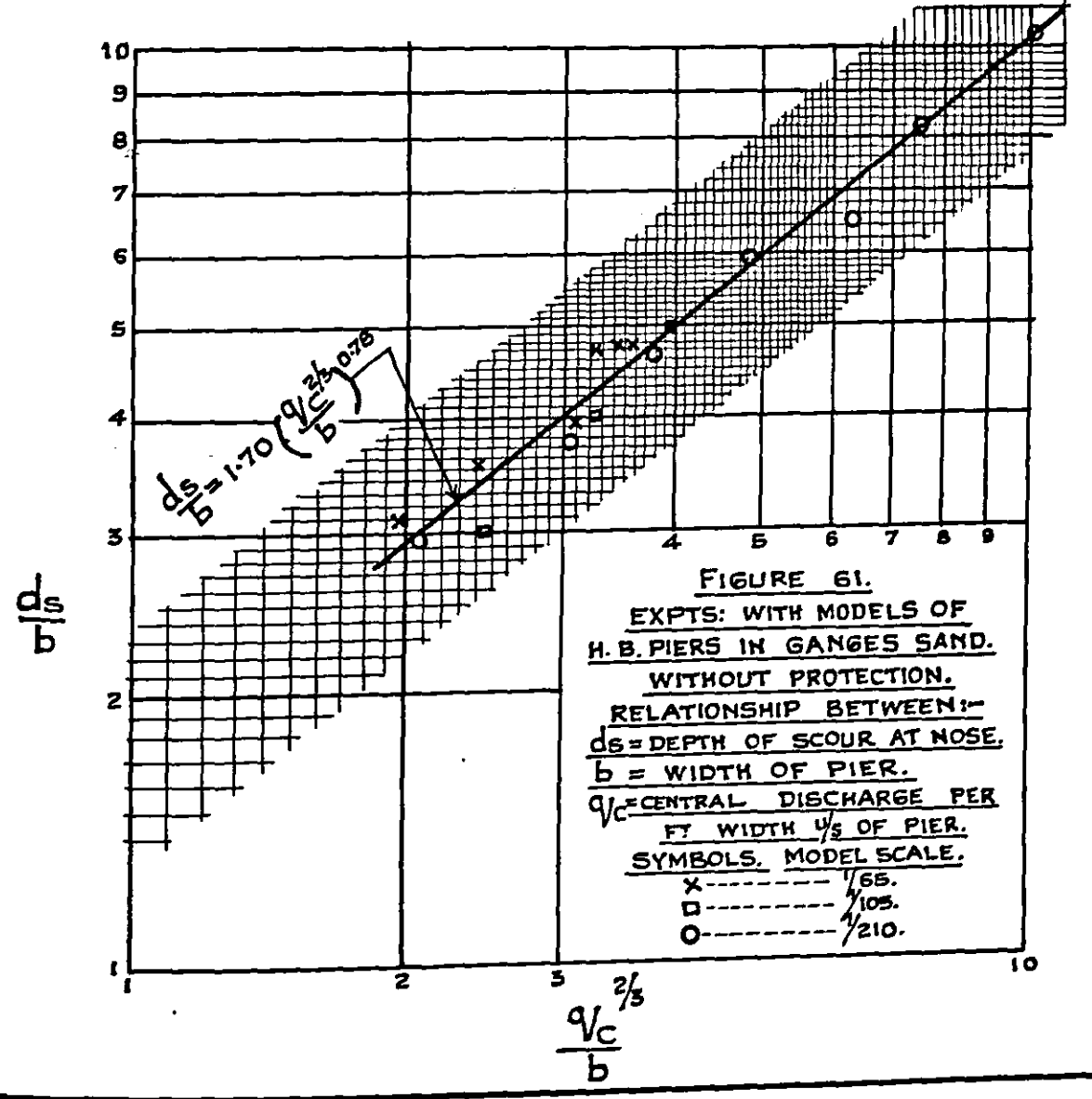
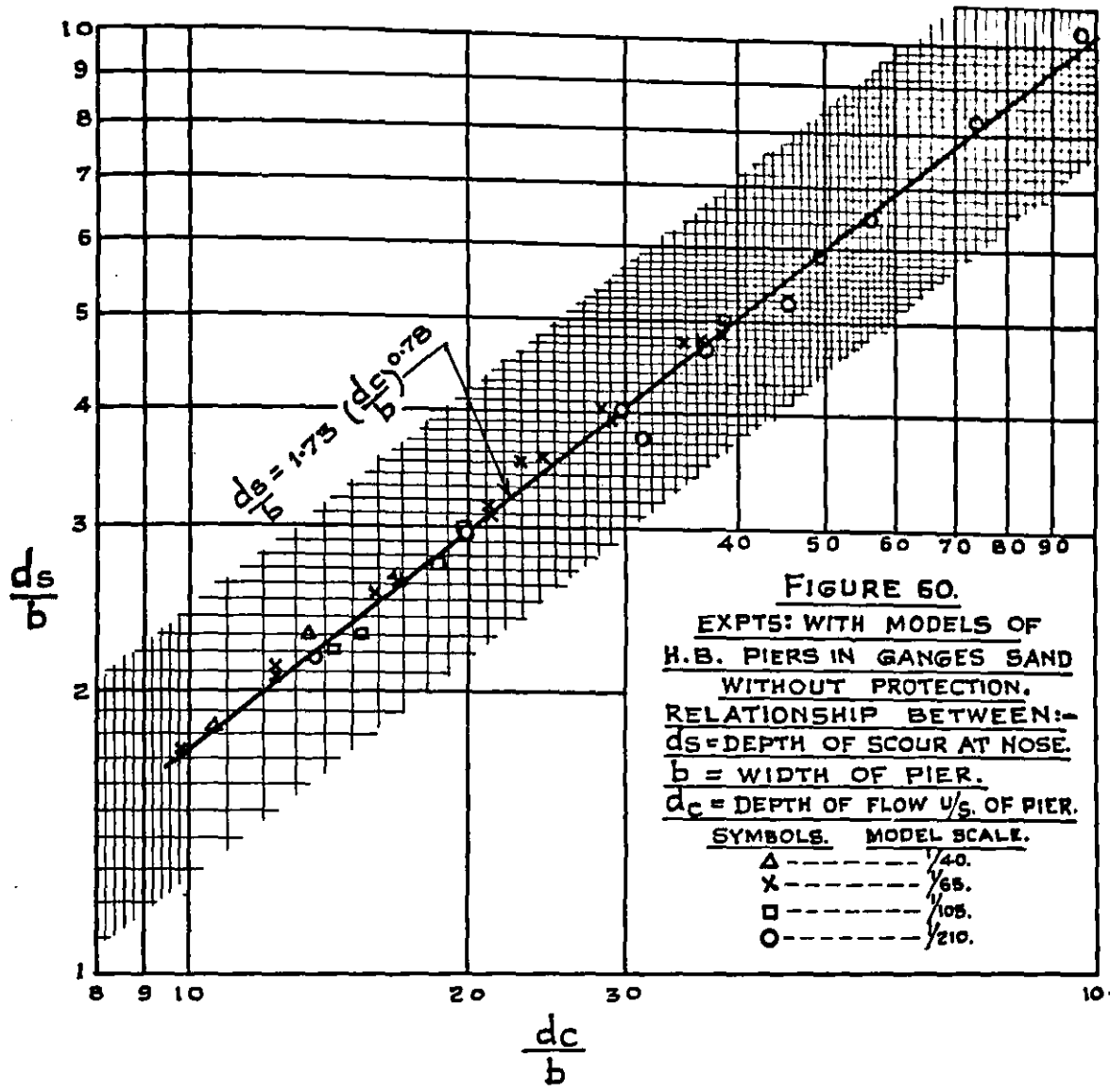


FIGURE 62

C.I. & H.R.S.
No. 1252

EXPTS. WITH 1/65 SCALE MODEL OF
HARDINGE BRIDGE PIER WITH STONE PROTECTION.

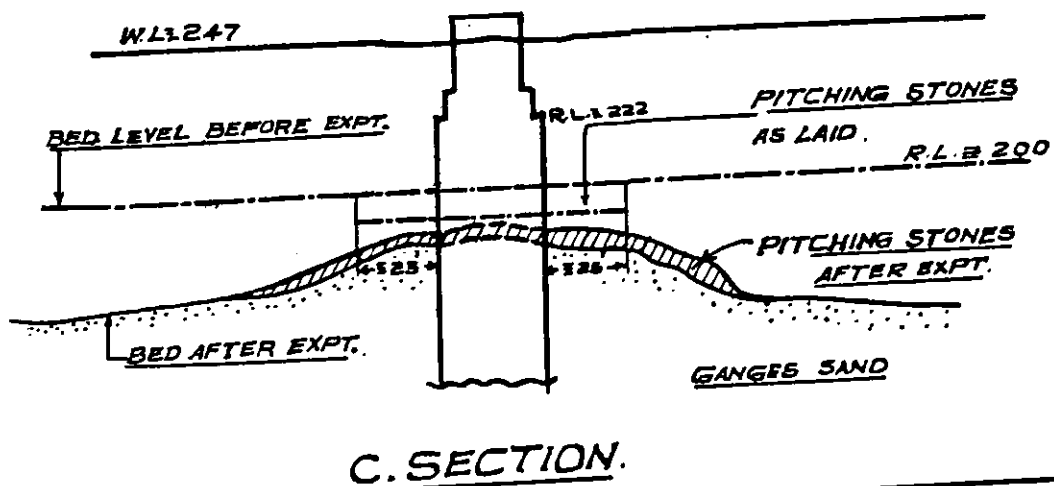
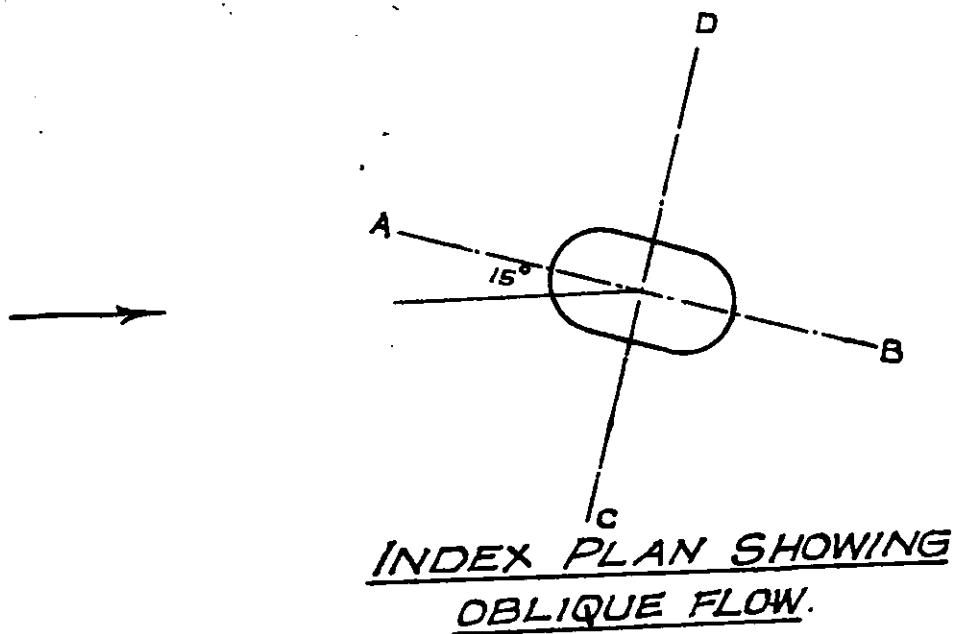
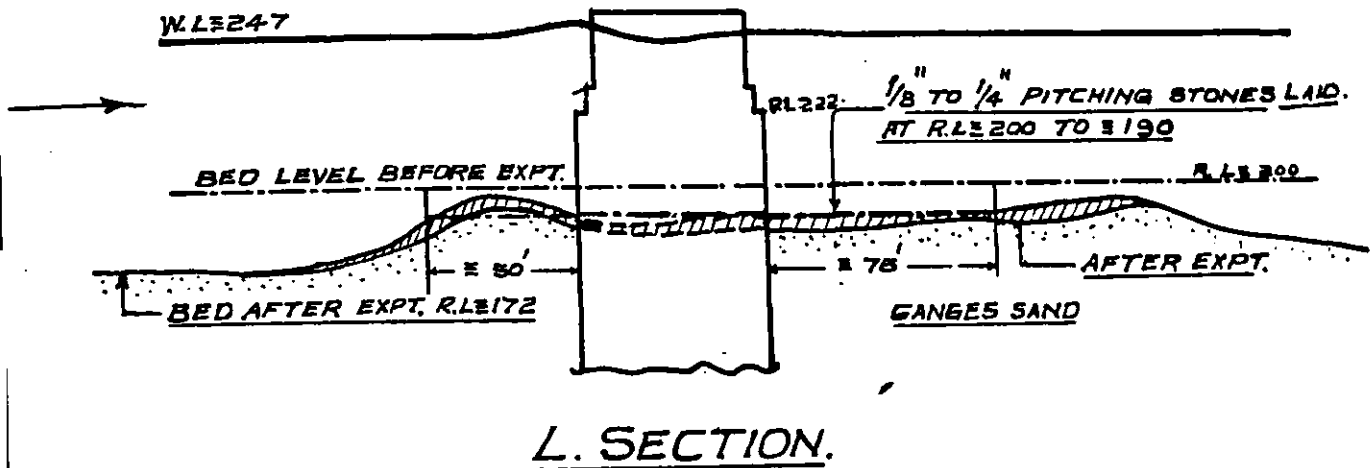
SHOWING STONE BEFORE & AFTER EXPT.

STONE : 1/8" - 1/4" \approx 8" - 15", LAID \approx 10 FT THICK.

FLOW OBLIQUE. $Q = 1.65 C_s. \approx 865 C_s. PER FT WIDTH.$

SCALE 1" = 1' IN MODEL.

≈ 65 IN PROTD.



Before experimenting with stone protection, it was desired to determine the relationship between depth of scour and discharge and dimensions of pier, and also to see whether scour was reproduced to scale in a geometrically similar model. For this purpose scour was observed with models of the Hardinge Bridge piers of four different scales: 1/40, 1/65, 1/105 and 1/210, with various discharges.

As a general procedure with each size of pier, the sand bed was laid flat at a high level, so that it would scour to reach its natural depth; a constant discharge was then run, with constant water level, until all scour in the channel and around the pier had ceased, when observations were taken. This was followed by a higher discharge, without relaying the bed, and so on, until the highest discharge in the range had been completed. No silt was injected.

Figure 59 shows a typical case of scour around an unprotected pier. The scour was always deepest at the nose.

Figures 60 and 61 are plotting of the maximum depth of scour.

As a test of geometrical similarity the results were plotted in the form shown. It will be seen that the points lie about the lines—

$$\frac{d_s}{b} = 1.73 (d_c/b)^{0.78} \text{ or approximately } d_s = 1.7 b^{1/5} d_c^{4/5} \dots (1)$$

$$\text{and } \frac{d_s}{b} = 1.70 \left(\frac{q_c^{2/3}}{b}\right)^{0.78} \text{ or approximately } d_s = 1.7 b^{1/5} q_c^{1/3} \dots (2)$$

where d_s = maximum depth of scour at pier (below W. L.).

d_c = depth of channel upstream of pier.

q_c = discharge per unit width upstream of pier.

b = width of pier.

As the points from experiments of all four scale ratios lie about the same lines, geometrical similarity in respect of depth, with equal F. N., is shown, and the two equations may be used to give an approximate indication of the maximum depth of scour around a pier with cylindrical nose in Ganges sand, if the flow is straight and normal, within the range of d_c/b and $q_c^{2/3}/b$ shown.

Piers with loose stone protection.

In order to protect bridge piers by reducing the depth of scour around them, it is the practice to lay an apron of loose stone around the pier, the stone usually being of "one-man" size, i.e. 100 to 120 lbs. weight. Instead of a cup shaped scour hole then forming, the maximum bed scour takes place midway between the piers, and the apron subsides at its outer edge. This may be seen in photo 1085 and, more clearly, in the cross section at the Hardinge Bridge in figure 44, facing page 26.

The stone protection may be laid at the level of the bed in the low-water season or at a lower level after excavation or, if conditions render it practicable, may be thrown in during floods after scour has taken place. The higher the level at which it is laid, the less will be the depth of pier exposed after scour, provided the stone is not carried away by the force of the current. The time scale of the model also enters here, because the more rapid the rise of flood in relation to the rate of scour of the bed, the more severe is the attack on the stones. Generally stone protection is laid much too high and so is washed away in every flood.

Figure 62 shows the result obtained in a preliminary experiment with oblique flow. The pier in this case was a 1/65 scale model of a Hardinge Bridge pier, protected by an apron of $\frac{1}{8}$ "— $\frac{1}{4}$ " (\cong 8" to 15") stone \cong 10 ft. thick, laid with its top at cold-weather river bed level. The discharge was gradually raised to 1.65 \cong 865 cusecs per ft. width which was maintained until scour had ceased.

It will be seen that the stone spread considerably but remained finally at a higher level than the surrounding bed, and considerably higher than the bottom of the cupshaped pit which would have scoured had there been no protection.

Further experiments are in progress.

13. DIVERGENCE EXPERIMENTS—(SERIES III).

Experiments were continued to find practical methods of causing flow to follow a divergence or expansion, in an open channel, without the development of return flow or one sided or unstable flow downstream.

In the 1937-38 Annual Report it was stated that the devices tested could be divided into two main types (a) baffle type stabilisers, which force flow from

the bed upwards, thus making it spread out and follow the expansion; and (b) stabilisers, designed to create turbulence, so facilitating the transference of energy and equalising the velocity across the channel.

Both these types have practical disadvantages: where there are waves of bed silt the baffles of type (a) are liable to be rendered less effective by the deposition of silt or sand upstream, while the grids of type (b) are liable to catch floating debris. A third type was therefore investigated, *viz.*: (c) stabiliser blocks fixed to the bed. Though more costly than (a), this type has neither of the disadvantages mentioned above, because, owing to their discontinuity, they do not cause sand to deposit upstream and the upstream faces of the blocks can be sloped to avoid catching debris.

Model.

The channel in which the tests were made was 2 ft. wide with vertical sides, expanding to a section having bed-width of 3 ft. with 1 : 1 side slopes. The bed divergence was 1 in 10 each side and the side walls were warped. The bed was level. Figure 63(a) shows this channel, and also the one-sided flow with return flow, which developed when no stabiliser was used. The merits of the stabilisers were judged on (i) the elimination of return flow and (ii) the velocities on a cross section, 10 ft. downstream of the beginning of the divergence. The tests were made with a discharge of 5 cusecs, depth of flow being 1 ft.

Curved Stabilisers, type (a).

The previous experiments with type (a) had been done with straight baffles. The effect of curving the baffle in plan, see figure 63(b) and (c), was seen. A 3" space was left below the baffle to allow the passage of bed material. The baffle which was convex upstream (c) gave better results than either the concave and straight designs, return flow was eliminated and the bed velocities near the banks, downstream, though higher than normal, were lower than with the latter designs.

Stabiliser Blocks type (c).

It had been found that the straight stabiliser baffle was insufficiently effective unless its height exceeded $d/5$, where d =depth of flow, and better results were obtained with height $d/4$. On the other hand when it was much higher than $d/4$ a surface wave tended to form, causing afflux. The same conclusions were reached with stabiliser blocks. $d/4$ was the optimum height.

The cubical shape of block, figure 64(a, i) was found to be best, but the results were nearly as good when the upstream face was sloped at 45° , figure 64(a, ii) and as the latter shape was less likely to catch debris, it was adopted for further experiments. V-shaped blocks, figure 64(a, iii) pointing upstream, were not so good.

A large number of arrangements of blocks were tested, in one or more rows, and placed in different positions in the expansion. The best results were obtained with the arrangement shown in figure 64(b), comprising two rows as shown, the first consisting of blocks 3" high ($=d/4$), 2" wide with 2" spaces, and the second row of blocks 2" high ($=d/6$), 2" wide and 1" spaces. There was no return flow and the velocities were nearer normal than with the curved baffle of figure 63(c).

The best arrangement using only one row of blocks was as shown in figure 64(c) comprising blocks 3" high ($=d/4$), 3" wide with 1" spaces, placed just downstream of the beginning of the divergence. There was no return flow, but the results were not so good as with the design of figure 64(b) as the main flow moved slightly from side to side and the bed velocities at the sides were higher.

1 in 5 Divergence.

The design of divergence was changed to that shown in figure 64(d), having vertical sides diverging at 1 in 5, and sharp returns to meet the 1 : 1 banks downstream. There was, of course, always return flow downstream of the returns, but the arrangement of blocks shown in figure 64(d), consisting of a single row of blocks 3" high, 3" wide and 1" spaces, placed near the downstream end of the divergence, reduced it to a minimum. The velocities downstream were good but less normal than with the warped divergence with blocks described above.

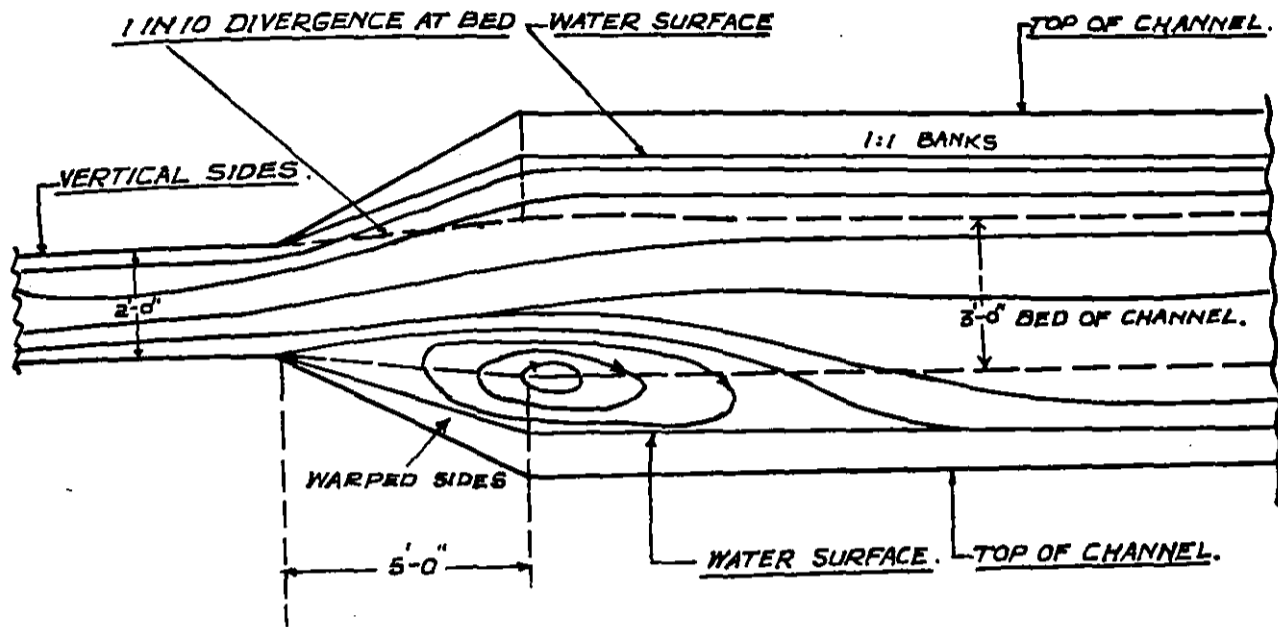
The experiments will be continued with other shapes of divergence.

DIVERGENCE EXPTS. (SERIES III)

SCALE 1" = 4'

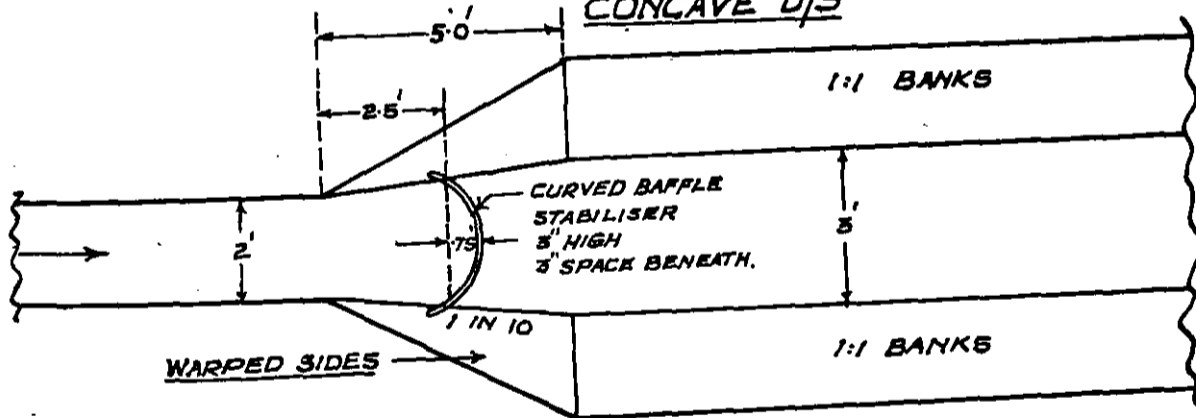
(a)

WITHOUT STABILISER
LINES OF FLOW AT SURFACE WITH Q=5 CFS.
AND $d_3 = 1.15$ FT



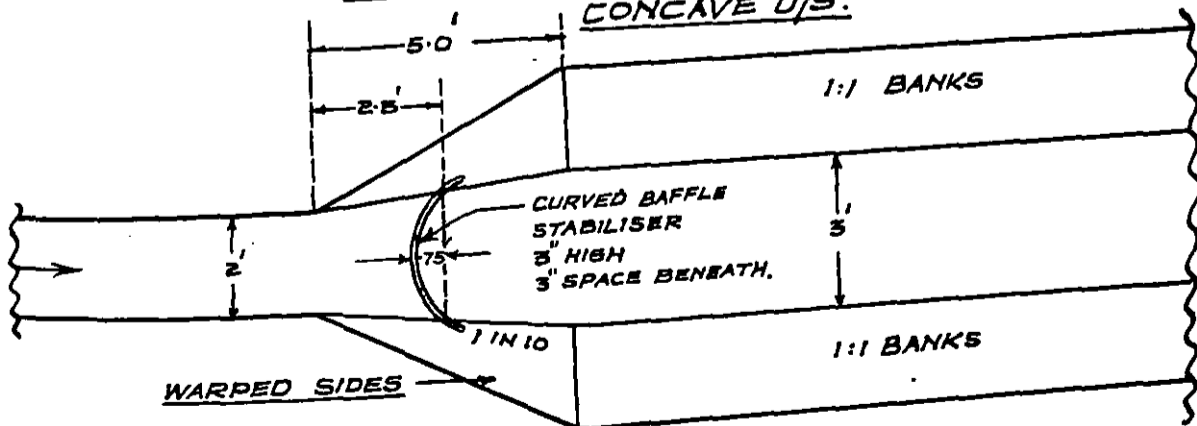
(b)

CURVED BAFFLE STABILISER
CONCAVE U/S



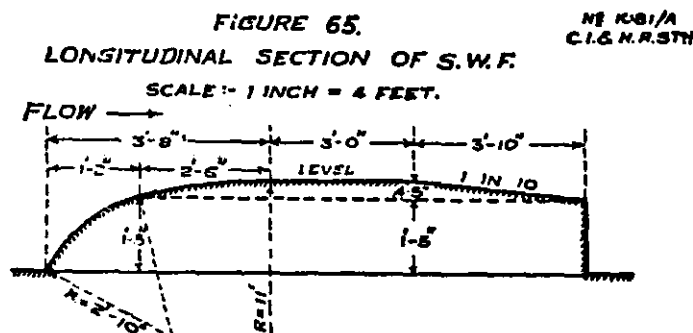
(c)

CURVED BAFFLE STABILISER
CONCAVE U/S.



14. CALIBRATION OF A STANDING WAVE FLUME WITHOUT SIDE CONTRACTIONS IN A 5-FT. CHANNEL.

This type of standing wave flume is suitable for measuring discharges where only a small loss of head is permissible. The flume calibrated consisted of a hump 1.79 ft. high and 10.5 ft. long, as shown in figure 65, in a rectangular channel

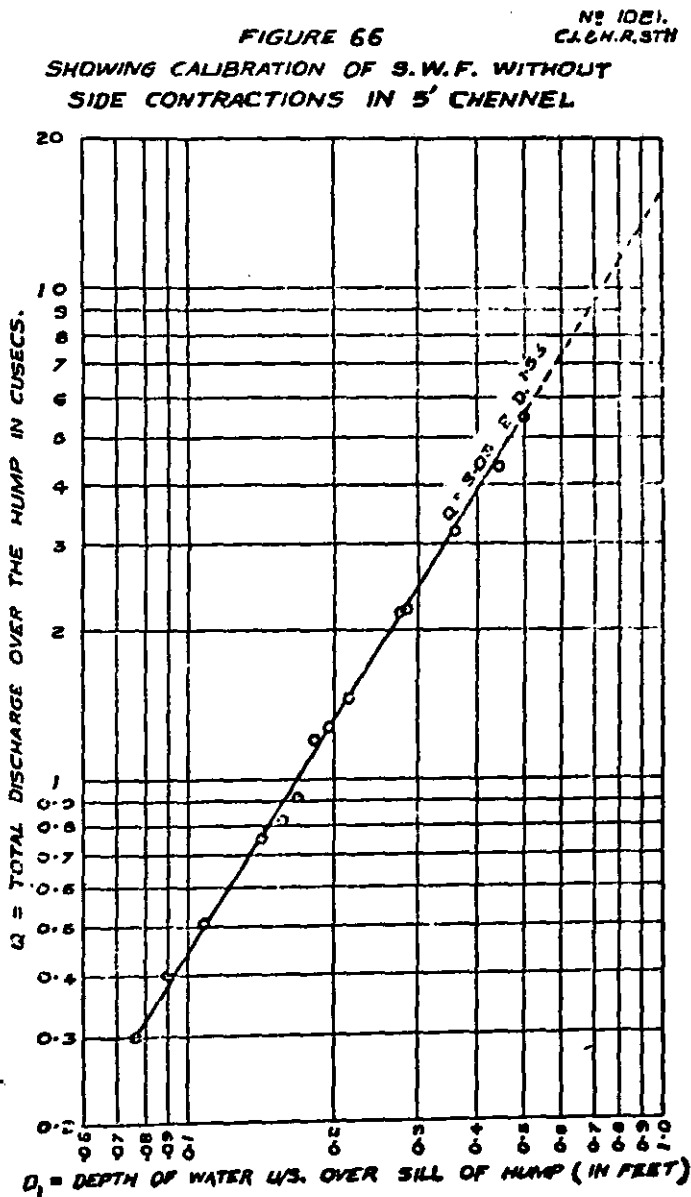


5 ft. wide, extending the whole width of channel, *i.e.*, there were no side contractions. The curved upstream face was provided for smooth entry, the level throat sill was designed to be of sufficient length to provide long-crested weir conditions and the downstream glaciis, sloping at 1 in 10, to recover velocity head with high downstream water level.

The flume was calibrated in the range $q=0.06$ to 1.08 cusecs per ft. width, using a standard 2 foot Standing Wave Flume located downstream.

Discharge Formula.

The results are plotted in figure 66.



For discharges above 0.4 cusecs/ft., in which range the scatter was very small, the discharge formula derived statistically is

$$Q = 3.03 B D_1^{1.53}$$

the difference from the standard formula $Q = 3.0 B D^{1.5}$ being due largely to the effect of velocity of approach.

The more simple formula

$$Q = 3.0 B D_1^{1.5}$$

is correct within ± 1 per cent for $q = 1$ to 3 cusecs/ft. ($D_1 = 0.5$ to 1.0) as shown in table 9.

TABLE 9.

| D_1 (ft.) | $Q = 3.03 B D_1^{1.53}$ (Cusecs.) | $q = Q/B$ (Cusecs/ft.) | $q = 3 D^{1.5}$ (Cusecs/ft.) | Per cent error. |
|-------------|--------------------------------------|---------------------------|---------------------------------|-----------------|
| 1.20 | 20.01 | 4.00 | 3.94 | -1.5 |
| 1.00 | 15.160 | 3.030 | 3.000 | -1.0 |
| .75 | 9.767 | 1.951 | 1.949 | -0.1 |
| .50 | 5.247 | 1.049 | 1.061 | +1.1 |
| .2 | 1.291 | .258 | .268 | +3.9 |

Range.

The hump was not calibrated for discharges greater than 5.3 cusecs ($q = 1.06$ cusecs/ft.) owing to the small size of standard flume available for calibration, but it may be assumed that the above formula is applicable with small error with depths up to $0.4 \times$ throat length *i.e.*, $D_1 = 1.2'$ and $q = 4.0$ cusecs/ft. approximately.

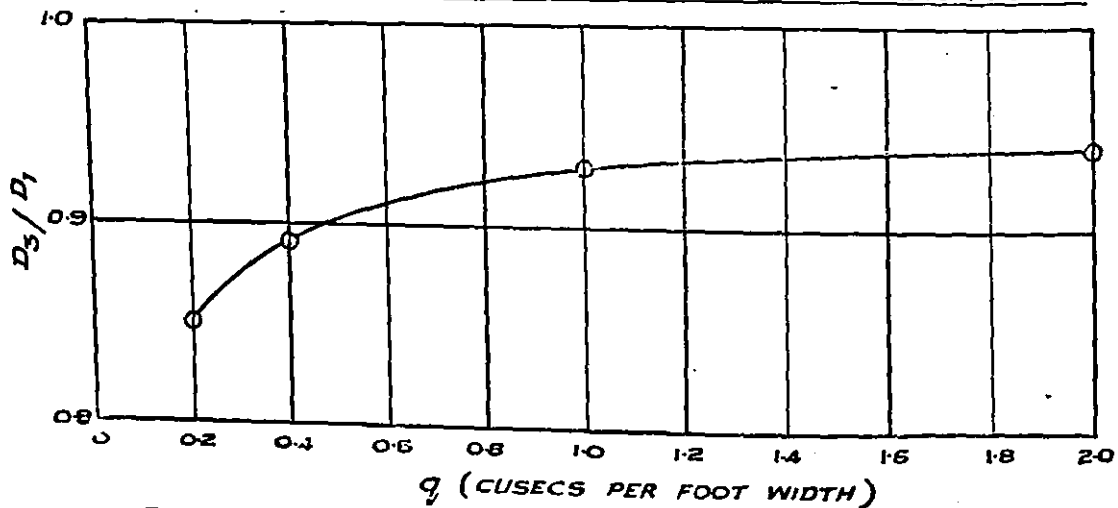
There was scatter with discharges less than 0.2 cusecs/ft. width, due probably to the effect of slight roughness and unevenness with the very small depth over the hump. The useful range of this flume may therefore be taken as 0.2 to 4.0 cusecs/ft.

Modular Limit.

The modular limit of the flume, indicating the downstream water level above which the wave becomes "drowned" and the calibration formula ceases to apply, is shown in figure 67. It will be noted that the modular ratio D_3/D_1 at the limit exceeds 0.9 for discharges greater than 0.6 cusecs/ft. The recovery of head is therefore satisfactory.

NR 1153.
C.I.E.H.R. STS

FIG. 67. MODULAR LIMIT OF S. W. F. IN 5 FT CHANNEL WITHOUT SIDE CONTRACTIONS.



D_1 = DEPTH U/S. ABOVE HUMP LEVEL.

D_3 = DEPTH D/S. ABOVE HUMP LEVEL.

q = MEAN DISCHARGE PER FOOT WIDTH.

15. EXPERIMENTS WITH SMALL MODEL CHANNELS TO FIND THEIR DIVERGENCE FROM LACEY REGIME.

It is found that the small channels of limited length, flowing in silt or sand without side restraint, as used for experimental purposes at this Station, tend to assume stable dimensions which depart from the Lacey formulas for regime channels. In general the sections are broader and shallower.

This difference is to be expected as the length and time are insufficient for the channels to generate a natural distribution of silt and form berms of fine material. The object of the experiments described below was to find the stable dimensions of channels under such conditions.

(1) *Khairpur Feeder West Experiments.*

Experiments were carried out with a model of Khairpur Feeder West Channel (Sind) with a constant discharge of 3.8 cusecs. The model was built in local red silt according to Lacey regime dimensions, with $f_1=1.25$ as shown in column 2 of table 10 below. This silt is of slightly cohesive character, with mean diameter = 0.24 mm. A natural silt charge of 2.6 c. ft. per hour was injected and a natural section was allowed to establish itself by running the model for 40 hours. Table 10 compares the actual and regime dimensions and their ratios.

TABLE 10.

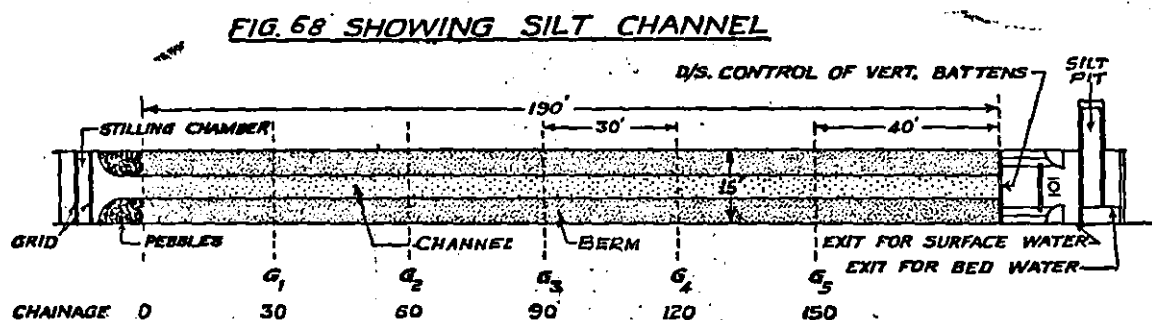
| 1 | Lacey Regime dimensions $f_1 = 1.25.$ | Actual dimensions. | Ratio of actual dimensions to regime dimensions. |
|---|--|--------------------|--|
| | 2 | 3 | 4 |
| Q (cusecs) | 3.8 | 3.8 | 1.0 |
| P = 2.67 $Q^{1/2}$ (ft.) | 5.2 | 6.6 | 1.25 |
| R = 0.47 $\left(\frac{Q}{f_1}\right)^{1/3}$ (ft.) | 0.68 | 0.585 | 0.86 |
| V = 1.1512 $\sqrt{f_1 R}$ (f/s) | 1.076 | 1.0 | 0.93 |
| A = PR (sq. ft.) | 3.53 | 3.8 | 1.075 |
| B ₀ (ft.) | 4.37 | 5.24 | 1.20 |
| S = $\frac{f_{sq}}{1859 Q^{1/6}}$ | 0.00063 | 0.00067 | 1.064 |
| q (cusecs) discharge per ft. run | 0.73 | 0.585 | 0.80 |
| f _d = 0.4 $\sqrt{d^2}$ | .. | 0.917 | .. |
| f ₁ = $\frac{3}{4} \frac{V^2}{R}$ | 1.25 | 1.249 | 1.0 |
| f _{RS} = 193 R ^{1/3} S ^{2/3} | 1.25 | 1.244 | 1.0 |
| f _{SV} = 48.46 $\sqrt{S V}$ | 1.25 | 1.25 | 1.0 |
| f _{sq} = 91.54 S ^{3/8} Q ^{1/16} | 1.25 | 1.275 | 1.02 |
| f _{vq} = 2.014 V ³ /Q ^{1/2} | 1.25 | 1.035 | 0.83 |
| f _{Na} = 3,901, 688 N _a ⁴ | 1.25 | 1.20 | 0.96 |
| f _n = $\frac{3106 R S}{V}$ | 1.25 | 1.216 | 0.98 |
| N _a = $\frac{1.3458}{V} R^{3/4} S^{1/4}$ | 0.0238 | 0.0233 | 1.0 |
| C ₁ = $\frac{V}{R^{3/8} S^{1/8}}$ | 16.00 | 16.31 | 1.02 |
| C ₂ = $\frac{P}{Q^{1/2}}$ | 2.67 | 3.33 | 1.25 |
| C ₃ = $\frac{P}{R V}$ | 7.12 | 11.11 | 1.56 |
| C ₄ = P/R = 5.64 Q ^{1/6} f ^{1/2} | 7.65 | 11.10 | 1.45 |

There was a very close agreement with the regime test formula $V=16 R^{2/3} S^{1/3}$, showing that the channel had adjusted its slope. V was 7 per cent too low and slope 6 per cent too high and R , 14 per cent too low, f_1 ; f_{RS} ; f_{sv} ; f_{sq} ; f_E and f_{Na} all ≈ 1.25 approximately; while $f_d (=9.4\sqrt{d''})$ gave a value of 0.92 which was 26 per cent lower. $f_{vq} = 1.04$ or 17 per cent lower.

(2) *Silt channel experiments without silt injection.*

Subsequently, experiments were carried out with channels in incoherent sand, designed and laid according to the Lacey formulas for discharges 1; 1.5; 2; 4; 6; 9; 12 and 18 cusecs. The channel, 190 ft. long, may be seen in photo 1896.

The mean diameter of the sand was 1.4 mm. and the silt factor $f_d = 2.185$. Bed and berm were constructed of the same material. Turbulent flow was created at entrance by building the approach channel of graded pebbles from $\frac{1}{2}$ " at upstream end to $\frac{1}{8}$ " lower down—figure 68 shows plan of channels. The section



at the downstream end of the channel was controlled by adjustable vertical wooden battens fixed across the section. Sand was not injected and the experiments were continued till bed movement ceased. In order to maintain a straight channel, banks tending to slip at one place were immediately repaired and made uniform with the shape above and below. In some of the repeated experiments, instead of laying the channel according to regime section, it was made slightly narrower and deeper than the final section previously obtained. For finding the mean dimensions, P , R , V , A & B_s , cross sections were observed at intervals of 10 ft. between chainage 60 ft. and 120 ft., this length of channel being found to be least affected by the conditions at entry and exit.

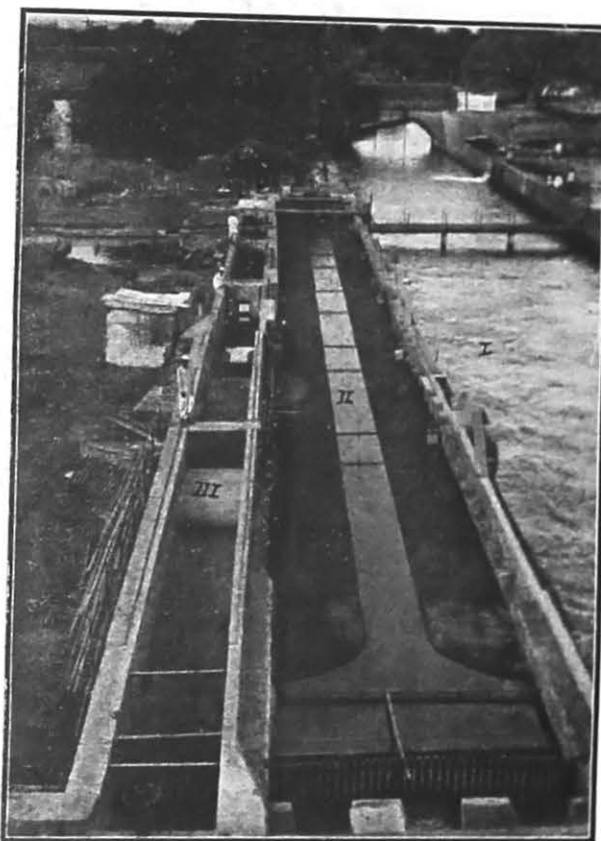
Slope observations.

Slopes were observed by different methods as shown below:—

- (a) Water levels were observed in midstream by point gauges at chainage 30, 60, 90, 120 and 150 ft. and velocities on verticals in midstream were measured with a pitot tube, from which energy gradients were worked out, allowing for the velocity head.
- (b) In order to eliminate errors, water levels were observed with point gauges 2' and 1' upstream and 1' and 2' downstream of a section and across the channel at various points, and the mean of all the readings was taken to represent the water level at that section.
- (c) Water levels were observed by fixing impact tubes at about half depth in midstream at chainages 30, 60, 90, 120 and 150 ft. connected with gauge wells fixed near the right bank of the channel. These observations included the effect of the head due to velocity.
- (d) A cathetometer was used to measure water level (i) in midstream and (ii) in the gauge wells, by reading the top of a point gauge—the pointer of which touched the water surface.

SILT CHANNEL EXPERIMENTS.

Length of channel=190 ft.



- I. Mutha Right Bank Canal.
- II. Silt channel.
- III. 5 ft. experimental channel.

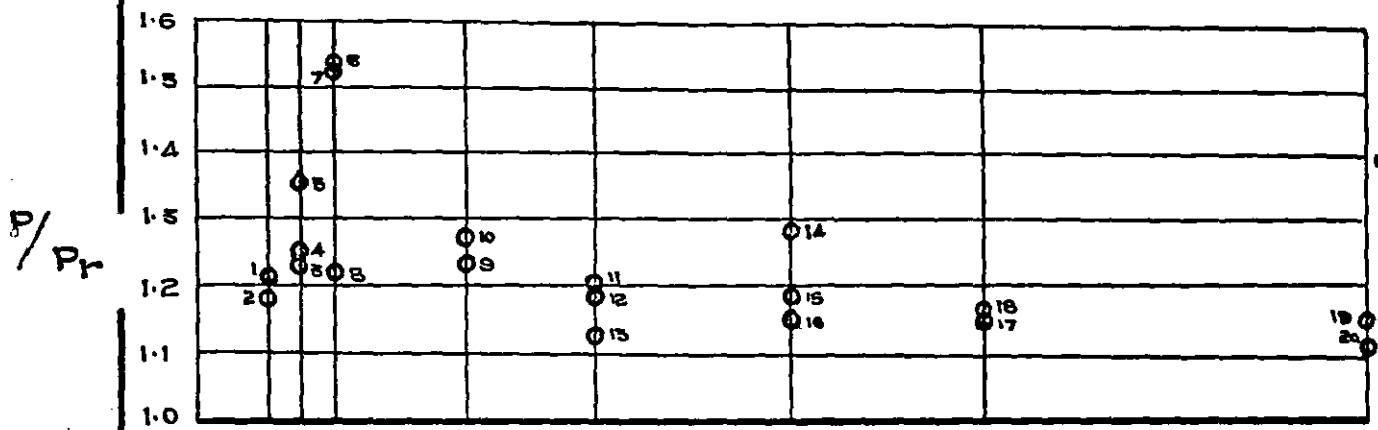
1896. Silt experiments with $Q=1.5$ cusecs after 135 hours flow with silt injection. Channel was initially laid according to regime dimensions. Mean diameter of sand=1.4 mm. and $f_s=2.185$.

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HYDRODYNAMIC RESEARCH STATION
POONA.

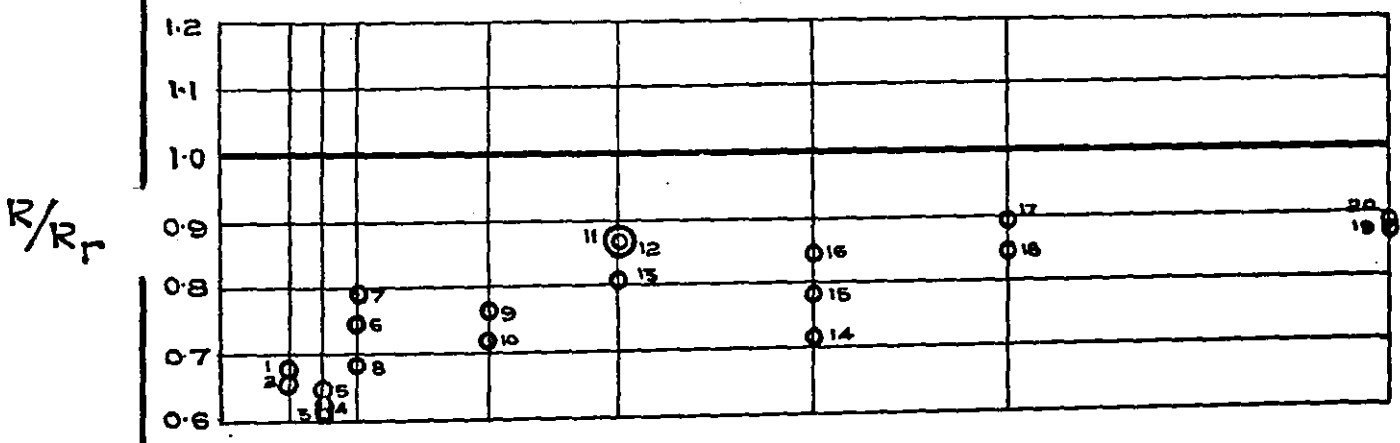
SILT CHANNEL EXPERIMENTS.

MEAN DIA. OF SAND = 1.4 m.m.

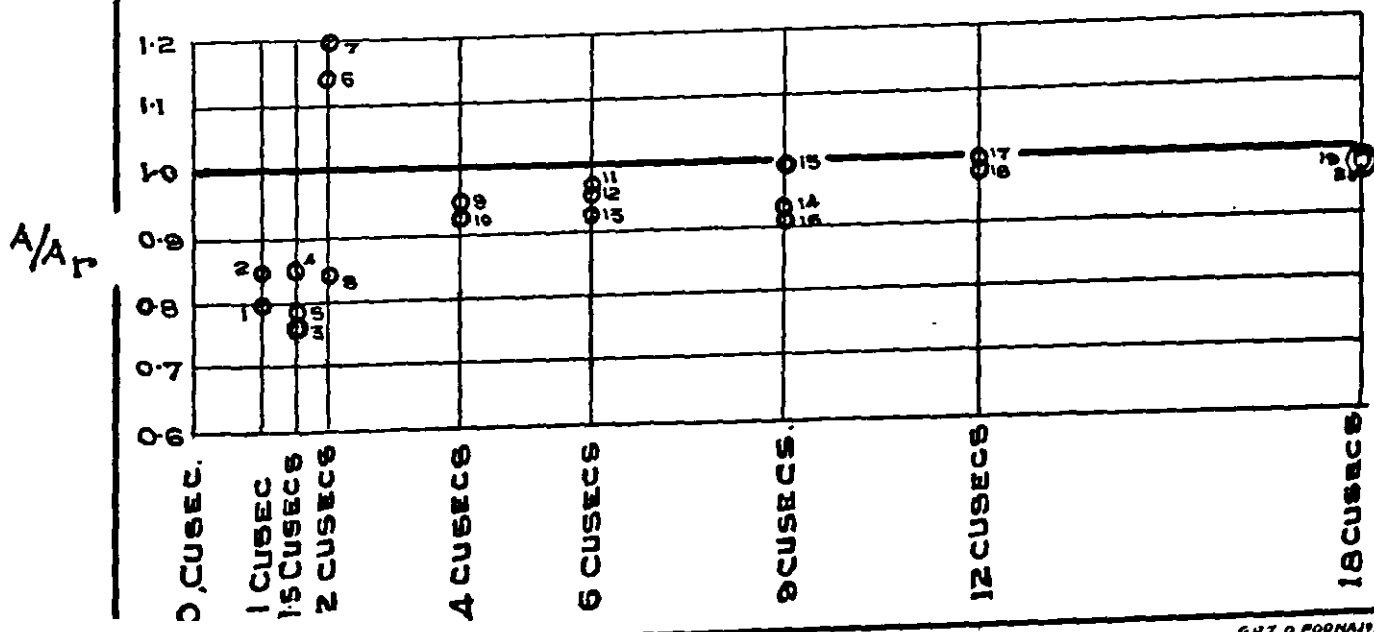
(a) $\frac{P}{P_r} \left(\equiv \frac{P}{2.67\sqrt{Q}} \right) : Q$



(b) $\frac{R}{R_r} \left(\equiv \frac{RF^{1/3}}{0.473 Q^{1/3}} \right) : Q$

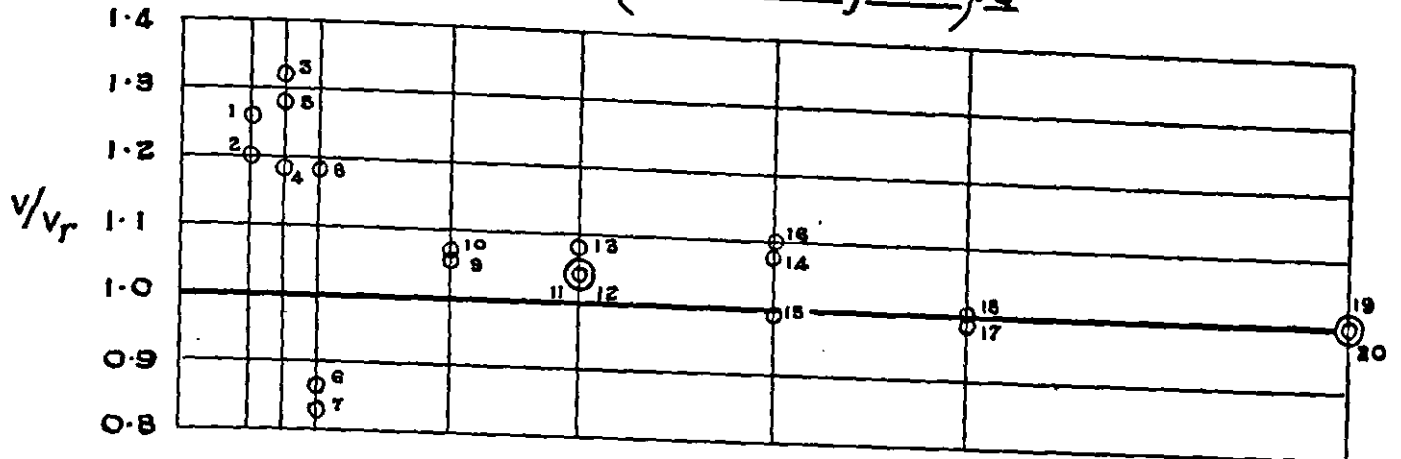


(c) $\frac{A}{A_r} : Q$

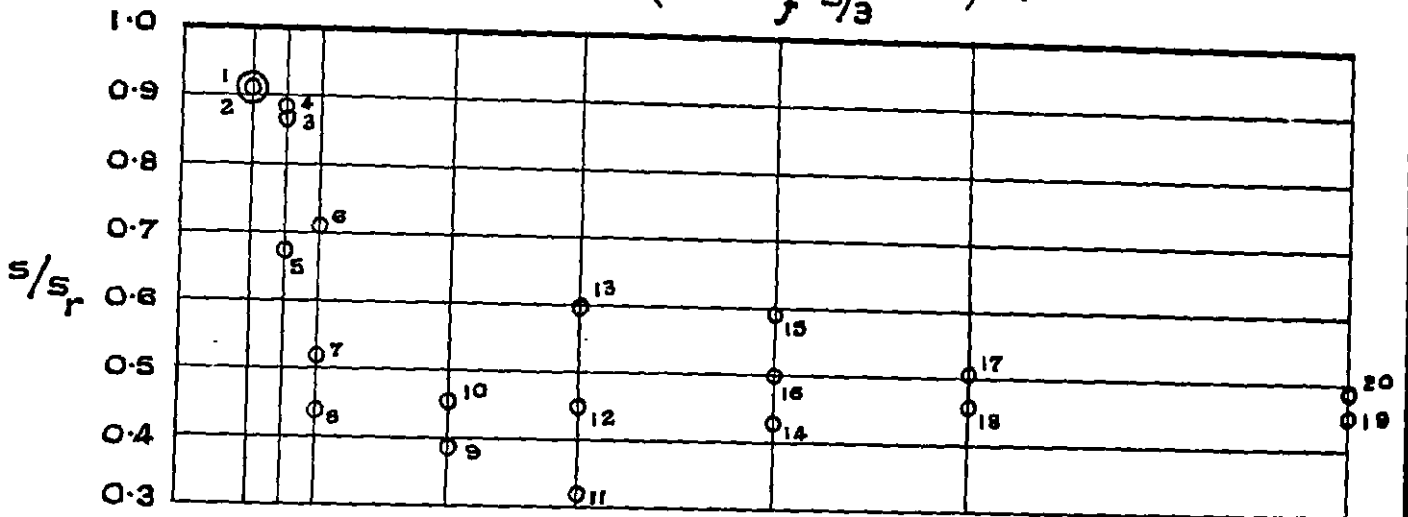


SILT CHANNEL EXPTS.
MEAN DIAMETER OF SAND = 1.4 MM.

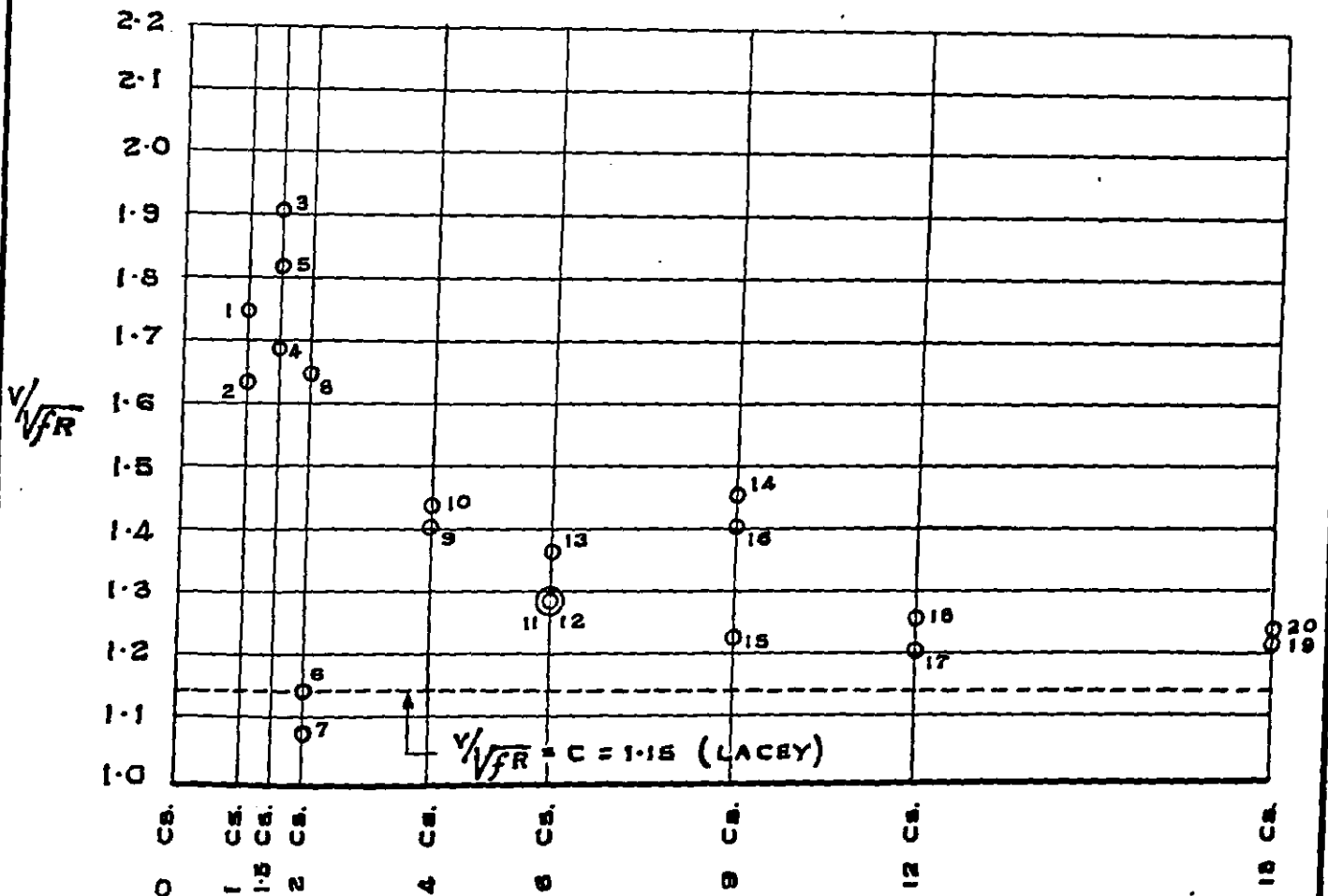
(a) $\frac{v}{v_r} \left(\frac{v}{0.79 Q^{1/6} f^{1/3}} \right) : Q$



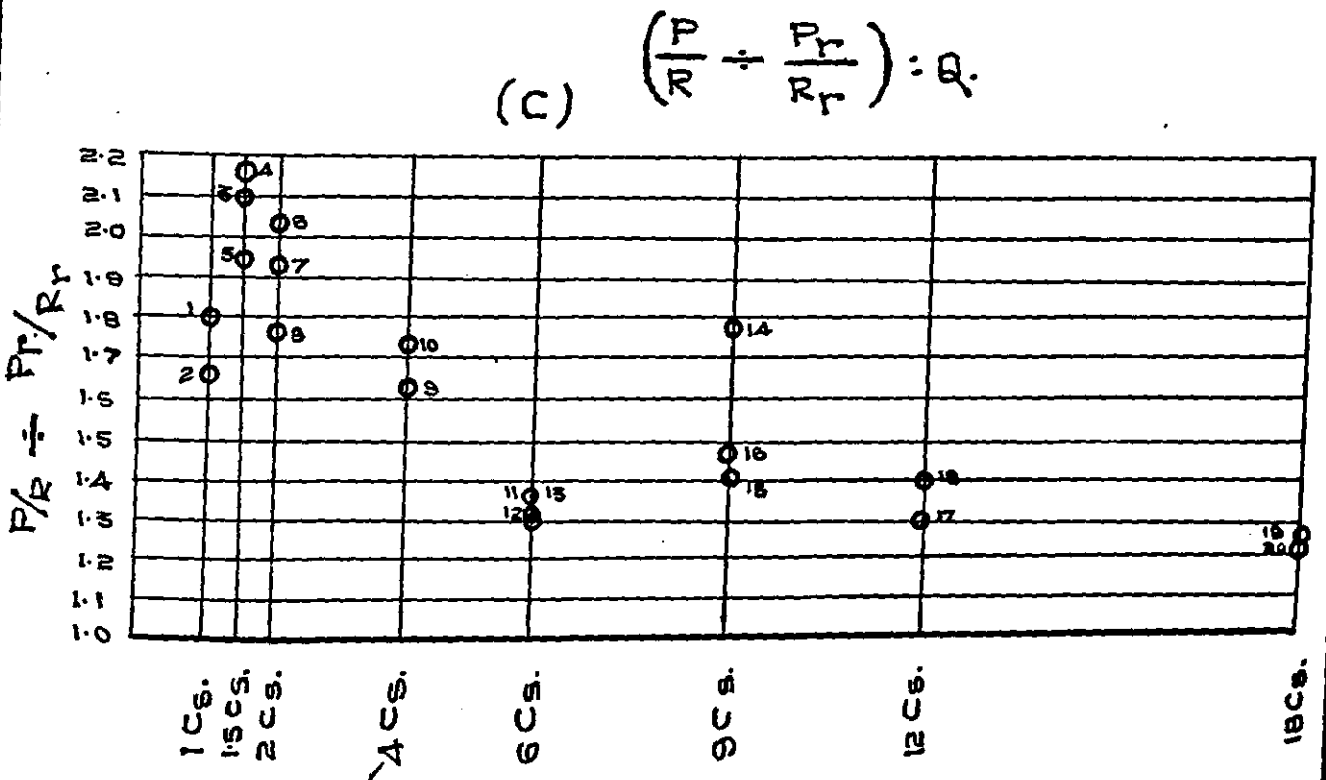
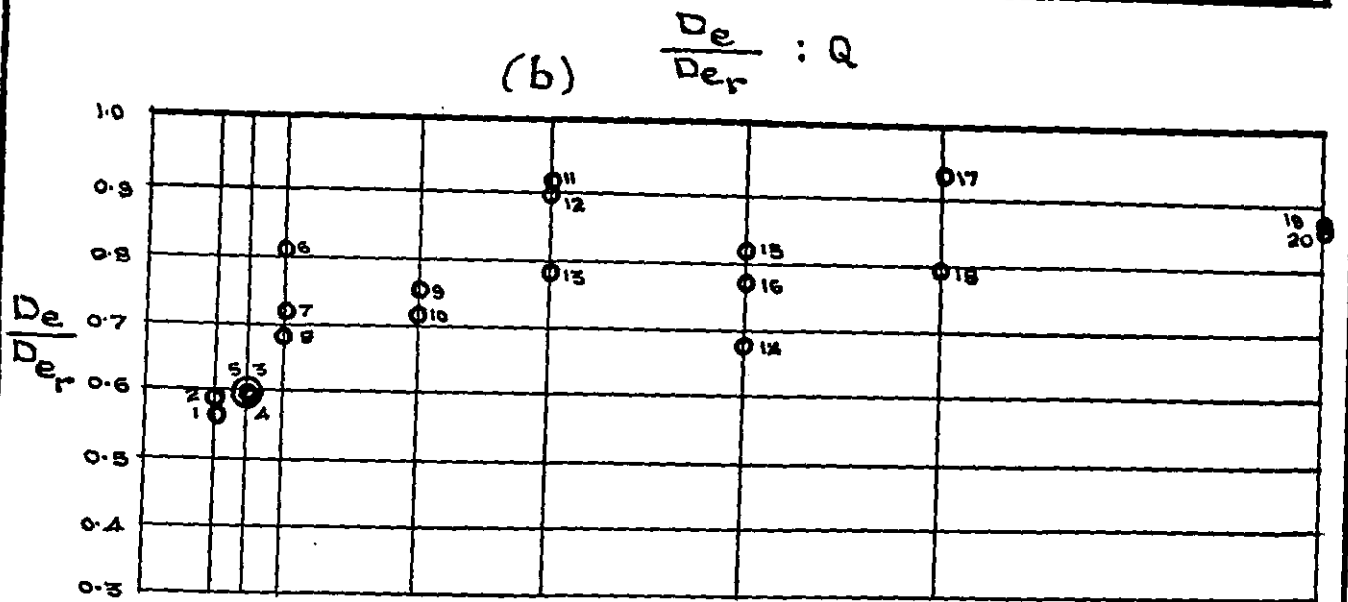
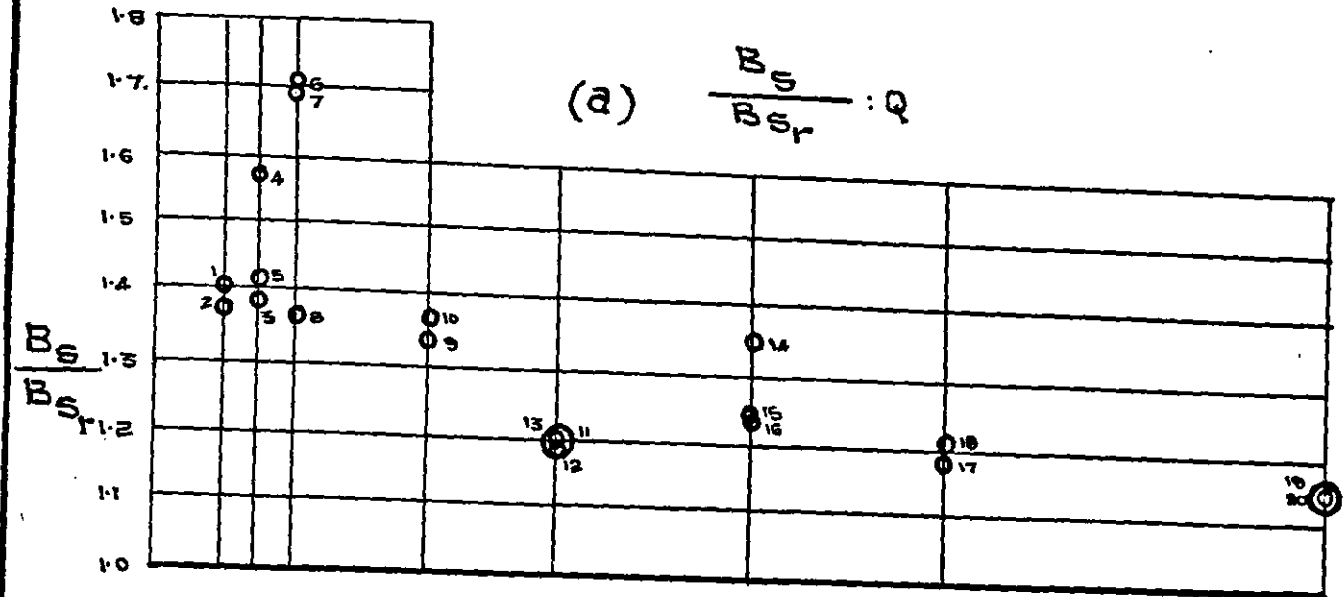
(b) $\frac{s}{s_r} \left(= \frac{1859 Q^{1/6} s}{f^{5/3}} \right) : Q$



(c) $\frac{v}{\sqrt{fR}} (= C) : Q$



SILT CHANNEL EXPERIMENTS
MEAN DIA. OF SAND = 1.4 mm



As the surface slope observations were much affected by wind, these were done as far as possible when there was no wind, and under such conditions fairly concordant results were obtained especially by method (d) (ii) in which water levels were observed in gauge wells by a cathetometer, but method (d) (i), i.e., observing water levels in midstream by a cathetometer was also satisfactory. The cathetometer results were better than those by method (c) because of the elimination of errors in zero readings of gauges, the difference in water levels being read directly.

Figures 69 to 71 show respectively ratios of P/P_r ; R/R_r ; A/A_r ; V/V_r ; S/S_r ; $\frac{V}{\sqrt{fR}}$; B_s/B_{sr} ; D_s/D_{sr} ; $\frac{P}{R} \div P_r/R_r$ plotted against discharges. The

subscript 'r' refers to regime dimensions for the same discharge, according to the Lacey formulæ. The large scatter was considered to be due largely to differences in the time allowed initially to run up to the designed discharge. It was found that the more gradually the discharge was raised at the beginning, the better were the results; owing to the tendency with rapid increase for the sides to fall in, forming large bed-silt-waves. Omitting these observations, the trend of the points is to lie approximately on curves asymptotic to a line parallel to regime; and for practical purposes, for $Q > 5$ cusecs, curves may be assumed to be horizontal with

$$\begin{array}{ll} P/P_r=1.17 & S/S_r=0.46 \\ R/R_r=0.85 & B_s/B_{sr}=1.25 \\ A/A_r=0.98 & \text{and} \\ V/V_r=1.02 & \bar{d}/\bar{d}_r=0.80 \end{array}$$

These differ considerably from regime excepting V (and hence A).

As regards the various 'f' values f_{vQ} appeared to agree with f_d better than other f values do for discharges of 4 to 18 cusecs, the formula being $f=2.014 V^3/Q^{1/2}$ or say $2 V^3/Q^{1/2}$.

It will be noted that P/R ratio (i.e., $P/R \div P_r/R_r$) is about 1.4, whereas $V/V_r=1.0$; the practical value of this is that in models with *moveable beds and sides*, the vertical exaggeration will be reduced in the ratio of 1:1.4, whereas the velocity will remain the same. Also, where there is no silt movement, the slope ratio should be only 1/2 the regime ratio.

These conclusions are tentative and experiments are to be continued with different grades of sand.

16. EXPERIMENTS WITH SILT ABRADER.

This experiment, which was started in November 1932 to determine the rate of abrasion of sand, was continued during the year.

The object is to show whether abrasion can be an explanation of the feature of large rivers that the bed sand becomes progressively finer from the hills to the sea. As described in the Annual Report for 1937-38, the silt abrader 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.

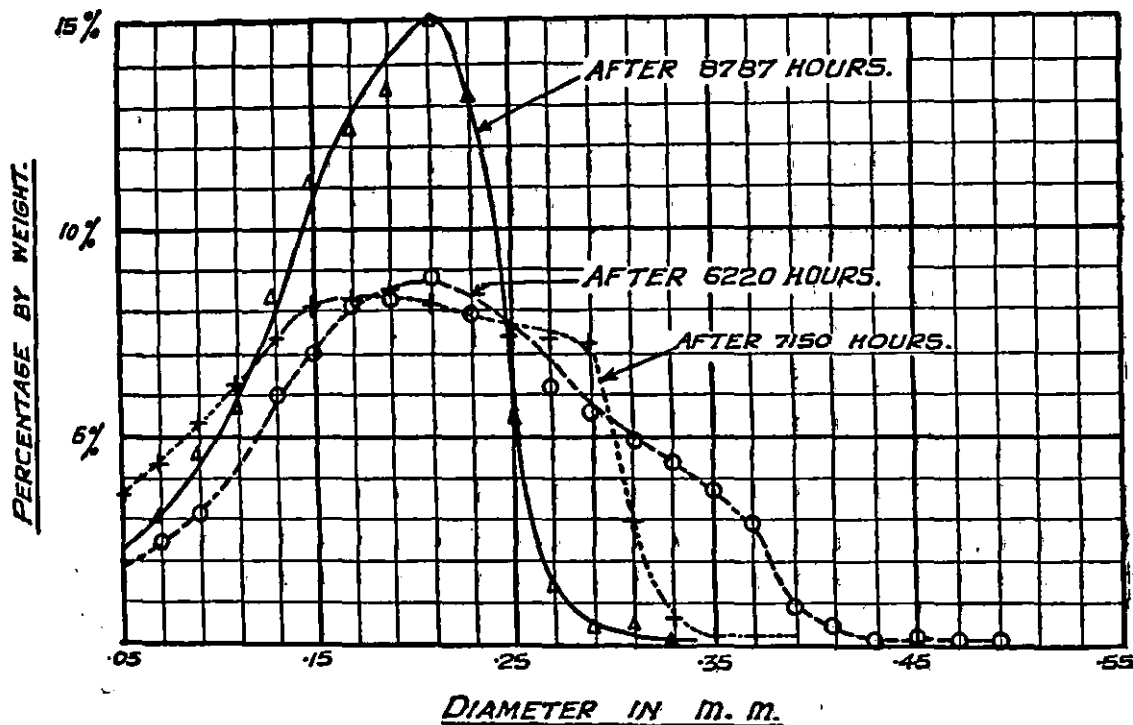
Sukkur silt was placed in the tank to a depth of 6" and water maintained at a constant level, 3'-6" over the bottom of the tank.

The abrader has worked for 1637 hours (=68 days) since last year's report, the total working hours to date being 8787 hours (=366 days). At the end of this time, the silt on the bottom was thoroughly mixed and analysed by the Puri siltometer. Figure 72 shows the distribution curves of silt obtained after 6220, 7150 and 8787 hours.

It will be seen that, between 7150 and 8787 hours, abrasion of silt particles of 0.25 to 0.35 mm. grade took place, resulting in the increase of per cent silt

No. 1249.
C.I. & H.R.S.

FIG:-72, SHOWING DISTRIBUTION CURVES OF SUKKUR SILT FROM SILT ABRADER.
(EXPT. DISCONTINUED AFTER 8787 HOURS.)
ANALYSES BY PURI SILTOMETER.



NOTE:-
VALUES OF THE DISTRIBUTION CURVES WERE OBTAINED
FROM SUMMATION CURVES AT INCREMENTS OF 0.02 m. m. DIA.

[The distribution curve of the original silt is not shown as it was not analysed by the Puri siltometer.]

between 0.11 to 0.25 mm. diameter. There was very little percentage increase of silt < 0.08 mm., which shows the limit of abrasion; because below that limit, the silt goes in suspension. Table 11 shows the progressive reduction in the grade of silt at various periods during the experiment.

TABLE 11.

| Total hours of working. | \bar{d}_{mm} . | $f_d = 1.76\sqrt{\bar{d}_{mm}}$, based on particles > 0.075 mm. diameter. |
|-----------------------------------|------------------|--|
| Original sample | 0.229 | 0.84 |
| 3775 hours (= 157 days) | 0.216 | 0.82 |
| 5382 hours (= 224 days) | 0.196 | 0.78 |
| 6220 hours (= 259 days) | 0.193 | 0.77 |
| 7150 hours (= 298 days) | 0.181 | 0.75 |
| 8787 hours (= 366 days) | 0.169 | 0.72 |

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 36 per cent after 8787 hours, and the percentage of fine sand (0.2 to 0.02 mm.) has increased from 9 per cent to 63 per cent. This result clearly shows the effect of abrasion.

After 8787 hours, the experiment with Sukkur silt was discontinued, because there was no indication as to the relative rate of abrasion of various sizes, this being masked by increase of the finer grades (<0.20 mm.) from all coarse grades (>0.2 mm.).

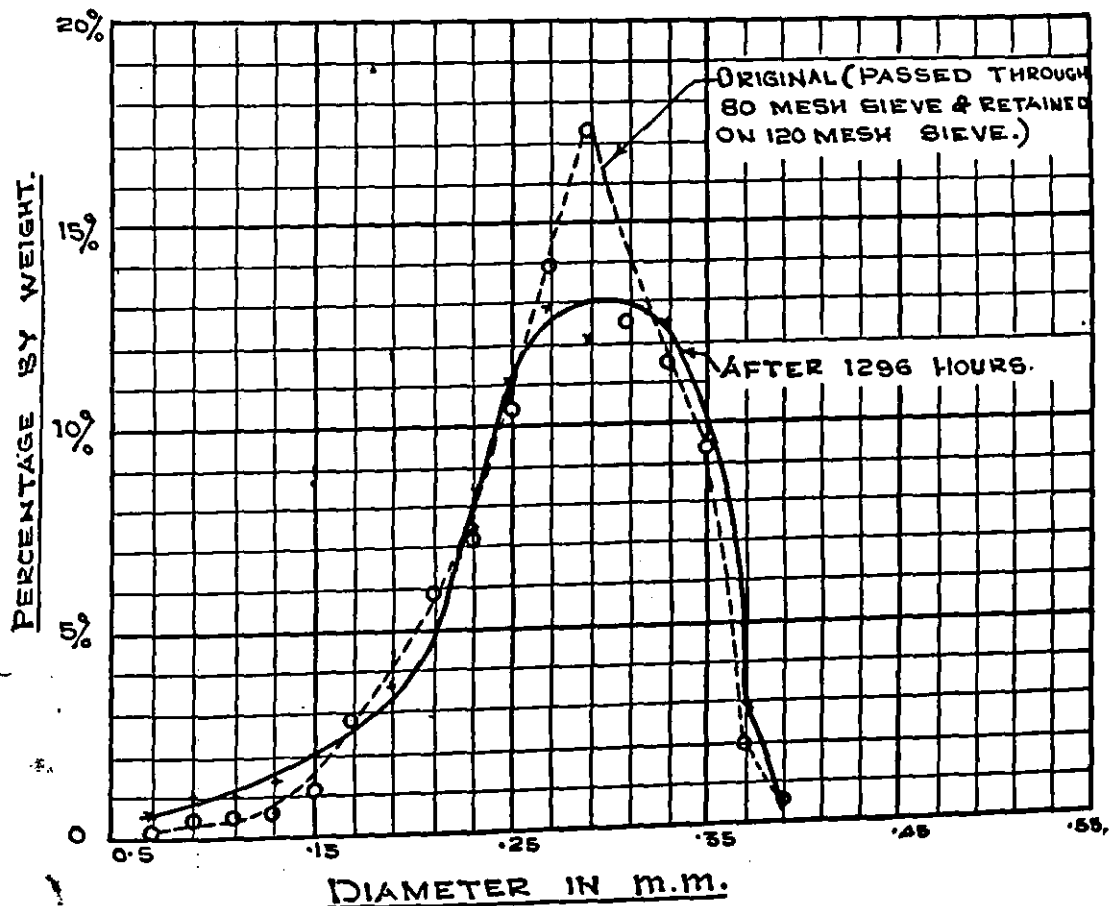
Therefore, a new experiment with silt obtained from the bed of the Brahmaputra at Amingaon was started on 15th September 1938, the silt being passed through 80 mesh sieve, but retained on 120 mesh sieve. The mean d_{mm} after sieving was 0.276 ± 0.056 mm. The silt thus obtained was placed in the circular tank to a depth of 6" and water was maintained at a constant depth of 3'-6" over the bottom of the tank as in the previous experiment. The abrader was stopped working after 1296 hours (=54 days) and the silt was analysed by the Puri siltometer. The temperature of water in the abrader, observed daily at 8 A.M. and 12 P.M., was found to vary between 17°C and 28°C.

Figure 73 shows the distribution curves of silt at the commencement of the experiment and after 1296 hours.

FIGURE:- 73

№. 1249/A.
C. I. & H. R. S.

SHOWING DISTRIBUTION CURVES OF BRAHMAPUTRA
SILT (AT AMINGAON) FROM SILT ABRADER.
ANALYSIS BY PURI SILTOMETER.



NOTE:-
VALUES OF THE DISTRIBUTION CURVES WERE OBTAINED FROM
SUMMATION CURVES AT INCREMENTS OF 0.02 M.M. DIA.

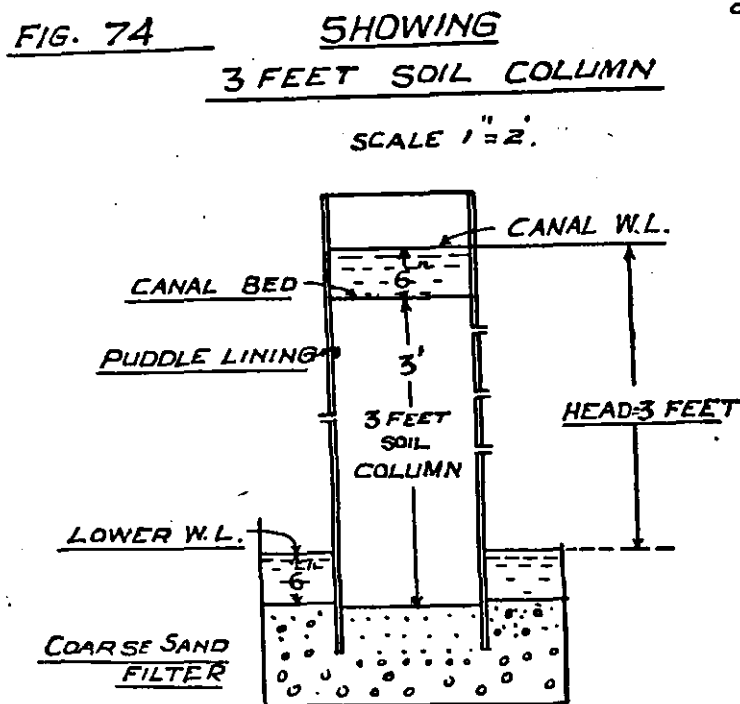
It will be seen that the rate of abrasion is very slow, as was the case at the beginning of the last experiment with Sukkur silt. The percentage of coarse sand (0.2 to 0.36 mm.) decreased from 91 per cent to 87 per cent and the percentage of fine sand (<0.2 mm.) increased from 9 per cent to 13 per cent. The

mean diameter decreased from 0.276 to 0.271 mm. and the Lacey silt factor decreased from 0.98 to 0.97.

The experiment is being continued.

17. EXPERIMENTS IN CONNECTION WITH SEEPAGE FROM CANALS.

The object of these experiments, which were started over two years ago at the suggestion of Mr. Crump (Punjab), is to correlate seepage with depth of water table, under controlled laboratory conditions. For this purpose the rate of seepage is observed through columns of silt or sand, from water maintained at a constant depth on the top of the column, representing a canal, to a chamber below, representing the subsoil water table, in which water is maintained at various levels in different experiments. Figure 74 shows a typical column.



As explained in the 1937-38 Annual Report, to which reference may be made for further details, the rate of seepage through each column reduces very considerably from the start of the experiment, and does not become steady until many days, and in some cases months, have elapsed.

This has been found to be due mainly to staunching caused by the deposition of very fine particles of silt in the top surface layer of the column, *i.e.*, the canal bed. Table 12 shows the analysis of samples taken from the top layers of one of the columns of Ganges sand:—

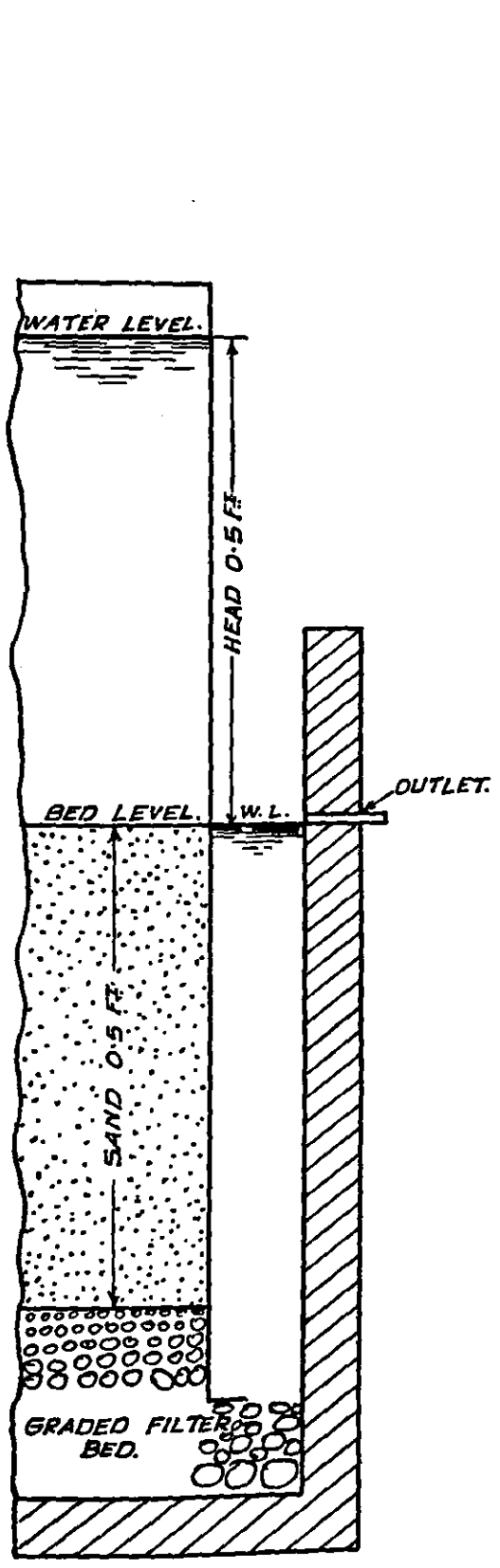
TABLE 12.

| | Top layer 2 mm. thick. | 2nd layer 6 mm. thick. | 3rd layer 6 mm. thick. | Original Ganges sand. |
|---------------------------------|---------------------------|---------------------------|---------------------------|--------------------------|
| Clay <0.002 mm. | 5.8 | 0 | 0 | 0 |
| Silt 0.002—0.020 mm. | 5.7 | 8.5 | 6.2 | 0.1 |
| Fine sand 0.02—0.06 | 11.1 | | | |
| Coarse sand 0.06—0.60 | 77.4 | 91.5 | 93.8 | 99.9 |
| TOTAL | 100.0 | 100.0 | 100.0 | 100.0 |

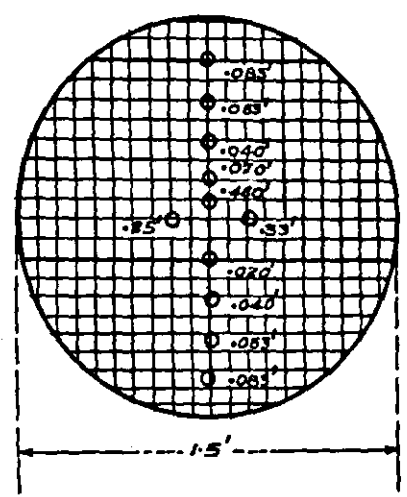
It will be seen that the top layer, 2 mm. thick, contained clay, and that the two lower layers each 6 mm. thick, while containing no clay had a greater proportion of finer particles than had the original sand.

Chemical analysis of samples of the top layers showed that no sodiumisation had taken place. The staunching thus appears to be entirely a physical phenomenon, the particles of clay in the top layer causing an additional obstruction to the flow.

FIGURE: 75
PRESSURES OBSERVED IN 0.5 FE COLUMN OF
GANGES SAND
DIFFERENCE OF HEAD 0.5 FE



SECTIONAL ELEVATION.



INDEX PLAN SHOWING POSITIONS OF
PRESSURE TUBES
FIGURES SHOW DEPTH BELOW BED LEVEL.

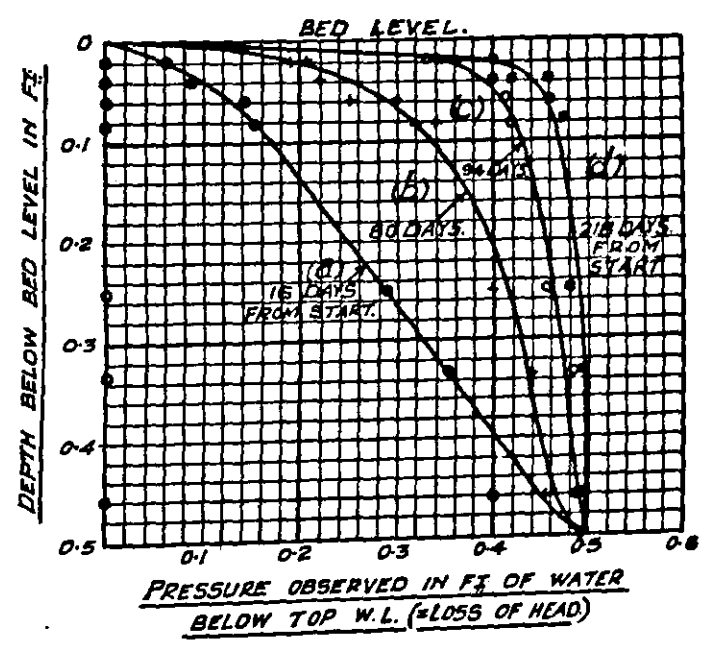
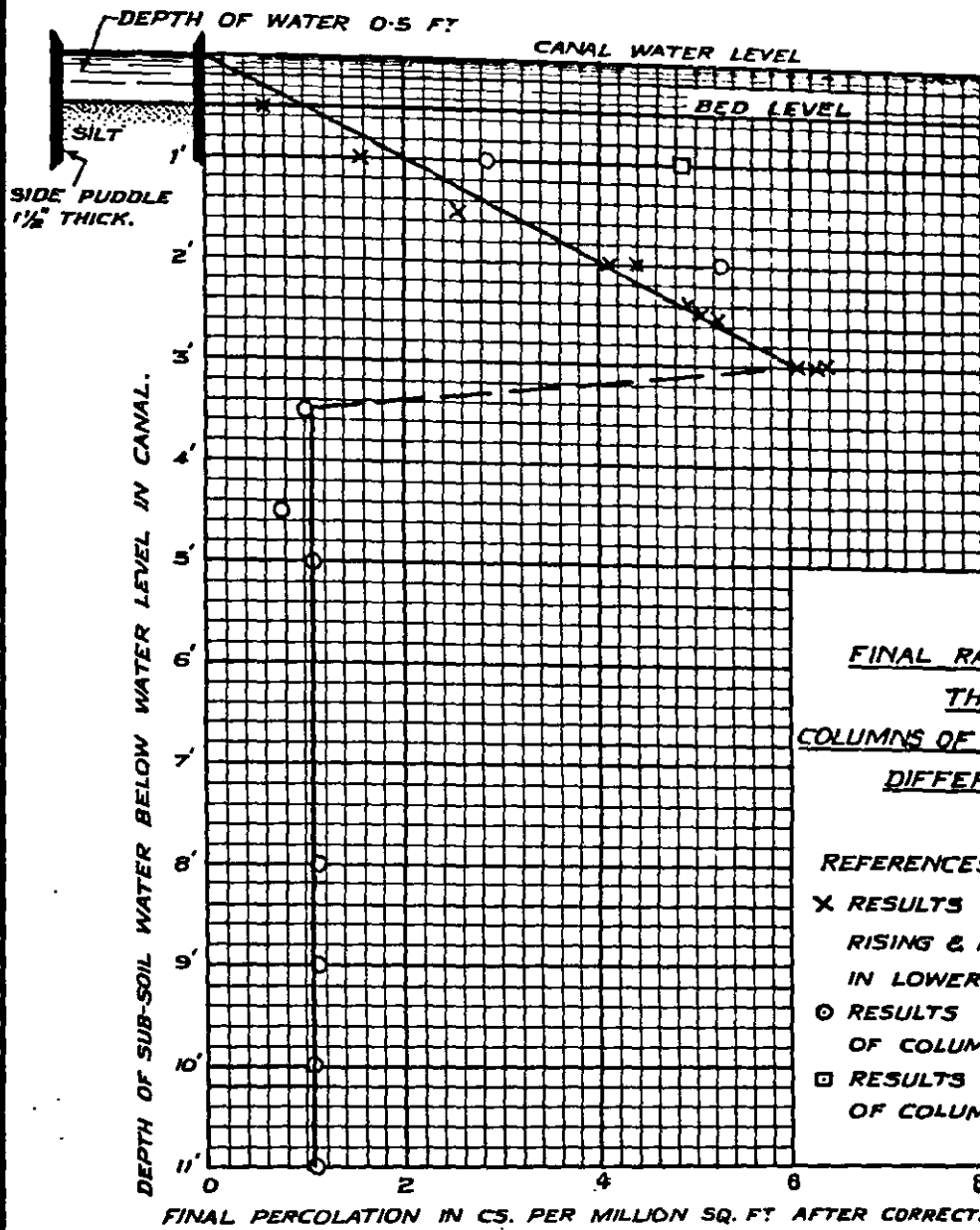


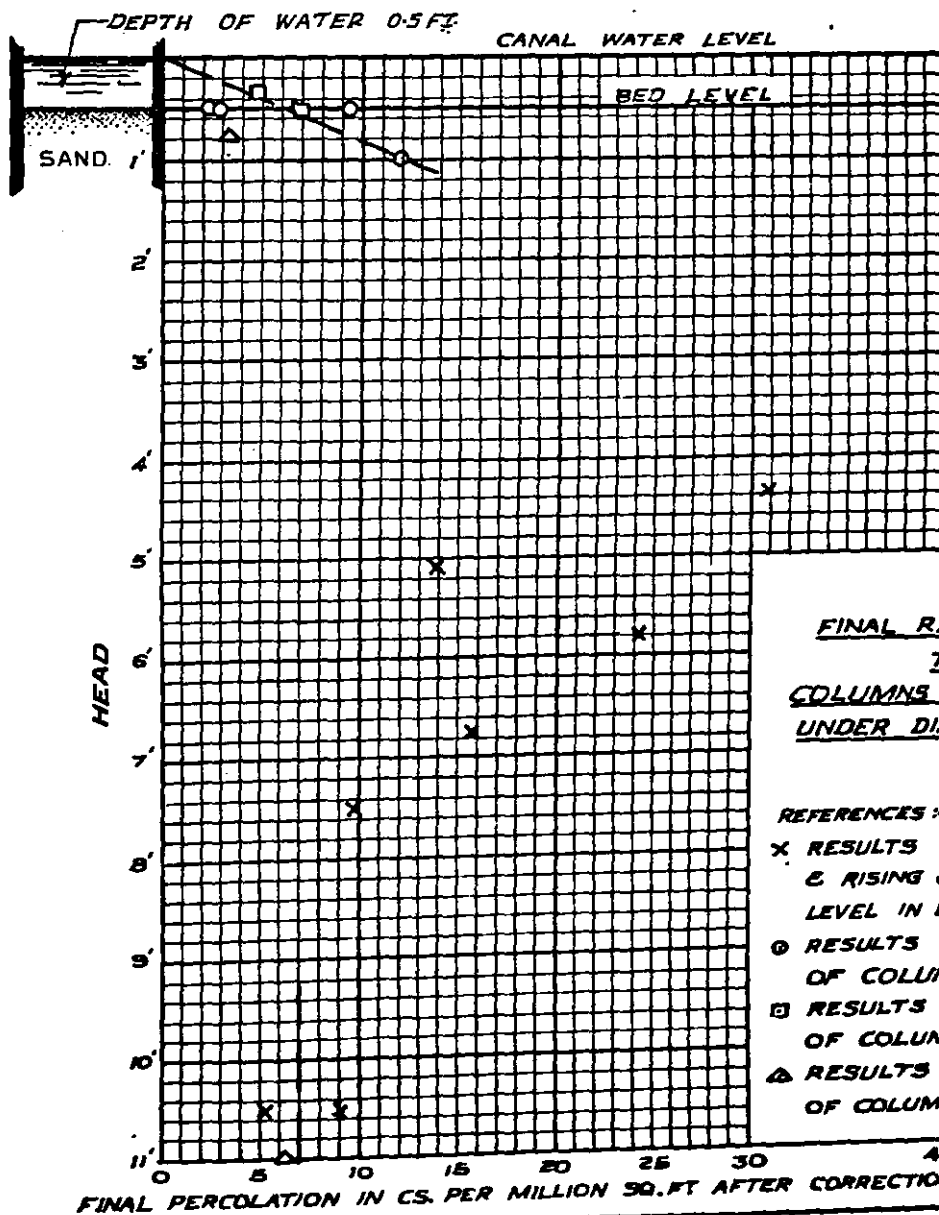
FIG. 76.



(a)
FINAL RATE OF SEEPAGE
THROUGH
COLUMNS OF KDK. RED SILT UNDER
DIFFERENT HEADS.

REFERENCES:-

- X RESULTS WITH 3FT COLUMN & RISING & FALLING WATER LEVEL IN LOWER CHAMBER.
- RESULTS WITH HEAD = LENGTH OF COLUMN.
- RESULTS WITH HEAD < LENGTH OF COLUMN.



(b)
FINAL RATE OF SEEPAGE
THROUGH
COLUMNS OF GANGES SAND
UNDER DIFFERENT HEADS

REFERENCES:-

- X RESULTS WITH 7.5 FT. COLUMN & RISING & FALLING WATER LEVEL IN LOWER CHAMBER.
- RESULTS WITH HEAD = LENGTH OF COLUMN.
- RESULTS WITH HEAD < LENGTH OF COLUMN.
- △ RESULTS WITH HEAD > LENGTH OF COLUMN.

As a result of this staunching, almost the entire head of water was lost in the top layer, as may be seen from figure 75, a diagram of pressure observations in one of the saturated columns, obtained by means of vertical glass tubes inserted to various depths from the top. The readings shown were taken—

- (a) 16 days after the start of the experiment, when the rate of seepage was high, and the loss of head was about proportional to depth below the top,
- (b) & (c) after 80 and 94 days, when the top layer was becoming staunched, and
- (d) finally, when the rate of seepage had reduced to a very low but steady value.

In all cases the seepage time curve flattened out and the rate of seepage ultimately became steady, though very sensitive to any disturbance of the water or sand. There was, therefore, a limit to the degree of staunching which could occur under the particular conditions.

Staunching took place whether the water was visibly silty or clear, and was found to occur even in a small experiment using filtered town water supply. In all the experiments, however, the top (canal) water was still. Experiments are now being started to observe seepage with the top water in continuous motion, as would occur in a canal.

Figure 76(a) shows the final rates of seepage through columns of Khadakvasla red silt, of various lengths with various depths of subsoil water, corrected for difference in viscosity due to temperature differences.

In one series with a 3 ft. column, the water level in the lower chamber was raised and lowered in stages, allowing the seepage to reach stability at each stage. In the other cases the columns were separately prepared in each experiment.

It will be seen that the curve obtained with depths of 3 ft. and less was quite different from that with subsoil water below 3 ft. from the canal bed. In the former case the rate of seepage was proportional to the head, while in the latter it was practically constant, independent of the head, at a value about 1/6th the maximum, which was obtained with 3 ft. head.

These experiments thus provided a striking confirmation of Mr. Crump's conclusions and of the field experiments later carried out by Gopal and Sharma.⁵

Figure 76(b) shows the results obtained up to the present with columns of Ganges sand, which is of more uniform grade than Khadakvasla red silt and contains no clay, as may be seen from figures of analysis given in Table 13.

TABLE 13.

| — | Khadakvasla red silt. | Ganges sand. |
|------------------------------------|-----------------------|--------------|
| | 0.32 mm. | 0.27 mm. |
| mean diameter | | |
| Clay < 0.002 mm. | 1.0 | 0 |
| Silt 0.002 to 0.02 mm. | 4.5 | 0.1 |
| Fine sand 0.02 to 0.06 mm. | 18.9 | |
| Coarse sand— | | |
| 0.06 to 0.60 mm. | 67.0 | 99.9 |
| 0.6 to 2.5 mm. | 8.6 | 0 |
| TOTAL | 100.0 | 100.0 |

The maximum seepage observed with Ganges sand was \cong 31 cusecs per million sq. ft. with a 7.5 ft. column under a head of 4.35 ft., compared with which the maximum observed with red silt was 6.3 cusecs per million sq. ft., with a 3 ft. column under 3 ft. head. Observations are still to be made, however, for Ganges sand, with columns and heads between 1 ft. and 4 ft.

It appears likely that the seepage curve for Ganges sand will be found to have the same characteristics as the curve for Khadakvasla red silt, though the rate of seepage is very much greater due to the small proportion of fine particles.

The experiments are being continued.

PART III.—ENQUIRIES DEALT WITH WITHOUT EXPERIMENTS.

18. TRAINING THE JUMNA NEAR PALLA VILLAGE (DELHI PROVINCE).

The right bank of the Jumna River at Palla village in Delhi Province is being eroded. The problem was referred to the Station to suggest remedies to prevent further erosion.

Two alternatives were suggested, based on the survey plan,

- (i) to reduce the discharge in the main river by improving the cut-off nala,
- or (ii) it might be possible to move the attack upstream of the village by constructing a short groyne upstream and another short groyne downstream of the village on the opposite bank.

If the cut-off nala was already increasing naturally, erosion at the village might soon decrease and no remedial action need then be taken. Further data regarding discharges were requested and it was suggested that a model experiment would be desirable, after a field inspection.

19. TRAINING THE AURANGA RIVER TO PROTECT BHADELI VILLAGE, DISTRICT SURAT (BOMBAY).

Bhadeli village is situated 3 miles from the sea on the right bank of the River Auranga—*vide* figure 78. The maximum flood discharge recorded in 1933 was 45,000 cs. The river is tidal for 5 miles upstream of the village and the force of tides at the latter is considerable.

Erosion along the right bank at the village has taken place steadily during the past 3 years and, as can be seen from the figure, a considerable area of village land has been lost. The present bye channel shown in dotted red, formed part of the old water course and though moderately wide and well defined is being silted up, while the present channel around the bend tends to scour. To stop further erosion, the local authorities proposed to construct the four groynes shown in red. The Government of Bombay referred the question to this Station for opinion.

The left bank at Bulsar, a Taluka town upstream of Bhadeli, had also been eroded by the combined action of the tides and high floods and, about the year 1895, the safety of the Bunder there, which served a very good traffic of country craft, appeared to be in danger. 4 groynes were therefore constructed on the left bank, shown in figure 78, and later in 1915 three more groynes added. These groynes not only had the desired effect of arresting progressive scour on the left bank, but some of the scoured land of the Bunder was reclaimed.

The two full lines and two inner dotted lines in black represent the two banks and the fair weather course of the river in 1895 before the construction of groynes. The red outer lines on Bhadeli side represent the present bank and the two inner dotted lines show the present low water course.

It was pointed out that the seven groynes on the left bank upstream of the village had no effect on throwing the river to the other bank; and might, in fact, have been expected to hold the flow to the left bank. As the river was so unstable, the cost of river training was likely to exceed its benefits,—especially as the present bye-channel was tending to silt. It was suggested that as the river had probably nearly reached its maximum swing, all that was required was to protect the village by an embankment with a stone face.

20. REMODELLING THE INTAKE FOR THE RAUNAH PUMPING STATION OF FYZABAD PUMPING AND GOGRA ELECTRIC SUPPLY SCHEME (U. P.).

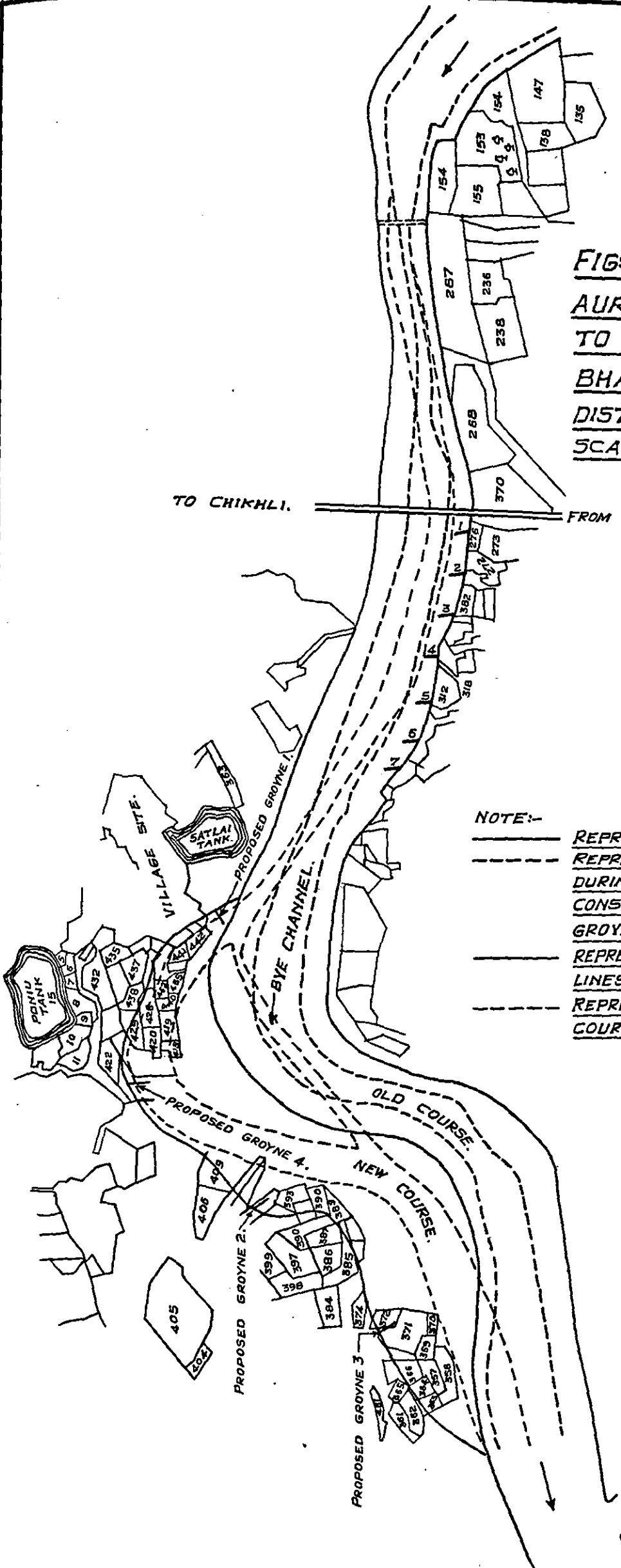
The Raunahi Pumping Station, which pumps a discharge of 180 cusecs for irrigation from the Gogra River into the Fyzabad Canal system, by three 60 cusecs electrically operated 35 ft. lift vertical spindle pumps.

FIG:-78 TRAINING OF
AURANGA RIVER
TO PROTECT
BHADELI VILLAGE
DIST. SURAT (BOMBAY.)
SCALE:- 1 INCH = 1230 FT.

TO CHIKHLI. FROM BULSAR

NOTE:-

- REPRESENTS BANK LINES.
- - - REPRESENTS WATER COURSE LINES DURING FAIR SEASON BEFORE CONSTRUCTION OF 1 TO 7 GROYNES ON LEFT BANK.
- REPRESENTS PRESENT BANK LINES NEAR BHADELI VILLAGE.
- - - REPRESENTS PRESENT WATER COURSE LINES DURING FAIR SEASON.



Since this Pumping Station was opened in February 1937, constant trouble has been experienced with excessive wear on the bottom bearings of the pumps. The main stream of the river passes at the foot of the cliff in front of the Station; which as will be seen from figure 79 is set back about 25 ft., and has no wing walls. A bar of silt forms across the mouth of the intake, as will be seen from cross section No. 5 shown in figure 79, and silt is drawn into the pumps, causing the excessive wear.

The Chief Engineer suggested the following alternatives for improving the intake :—

- (1) Cutting away the cliff on both sides of the Station along the lines PQ and RS—see figure 79—to eliminate the set-back and bring the river past the entrances to the pump chambers.
- (2) Providing a curved approach and a regulator to shut the station off during floods, when the pumps are not working.
- (3) Leaving the entry as it is and providing a dredger to keep it clear of silt deposits.

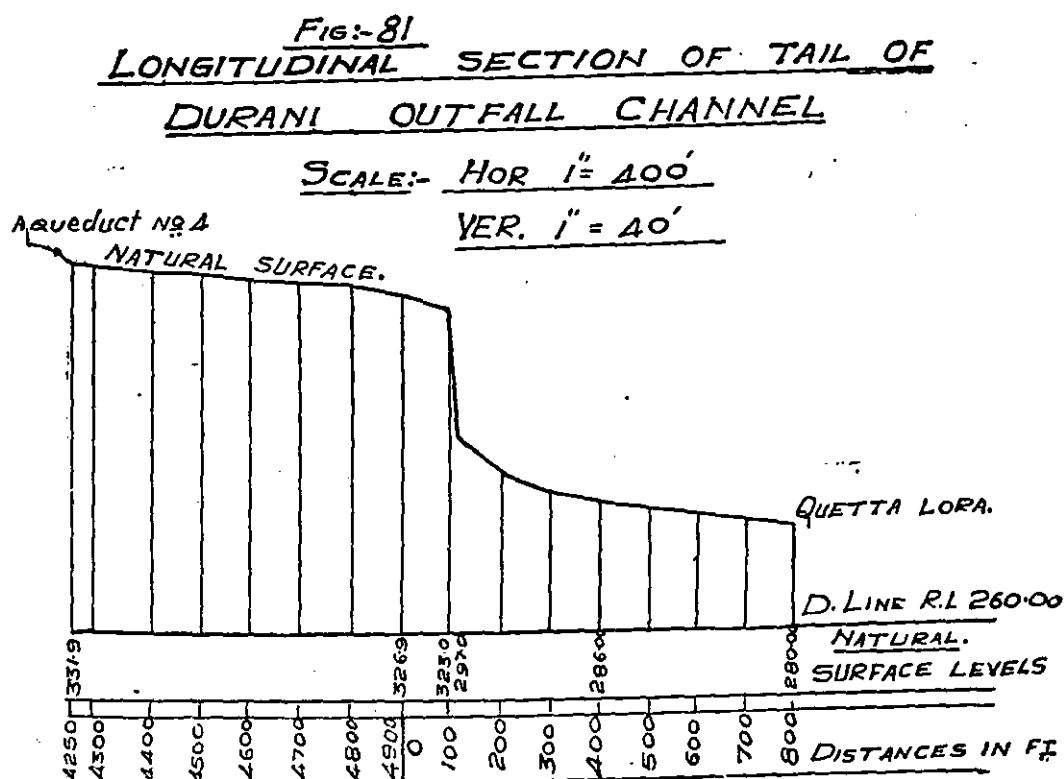
The following recommendations were made :—

It would not be possible to improve entry conditions to any considerable extent, except by projecting the downstream abutment straight out into the river and curving its end upstream, but allowing water to flow under this or providing undersluices—which would, however, be expensive. Cutting back the bank upstream may have a slight effect, but silt would again deposit. Any projection downstream—even a short one—would be beneficial.

The action created by the suction, supposing no changes were made, would be to draw bed water and bed silt into the entrance; the top, clear, water tending to flow straight on. This being so, a dredger would not be a solution; because the inflow of silt would be continuous so long as bed silt is in movement.

21. DURANI OUTFALL CHANNEL, QUETTA, (BALUCHISTAN).

The flood run-off from Quetta town and Cantonment is passed through the two nalas—Habib and Durani—shown in figure 80. Below the junction 'J', the discharge is taken into what is known as the "Durani Outfall", with estimated maximum discharge of 6,500 cusecs, leading into the Quetta Lora. The construction of the Outfall was completed in 1937, since when the Outfall below the last masonry structure has developed a longitudinal section as shown in figure 81.



It will be seen that a fall of 30 ft. has developed, which is scouring back very rapidly.

The local authorities proposed to construct a pucca tail channel with a fall at D (shown at AB as "first proposal" in figure 82) and referred the problem to this Station for opinion on the design. It was pointed out that had this question been considered when the fall was near the Lora, it would have been comparatively simple, but as the fall had scoured back almost to the sharp curve, the effect of velocity of approach would not be uniform and the water would pile up on the right bank. The mean velocity of approach with the proposed 30 ft. wide channel, 9.2 ft. deep, would be 21 f/s, equivalent to 8.2 ft. velocity head. This would be serious even were the channel straight, but would preclude the possibility of satisfactory results on a curved channel; besides which, any masonry work would be liable to be outflanked or undermined owing to the high fall and the treacherous nature of the soil, which when wet is semi-liquid.

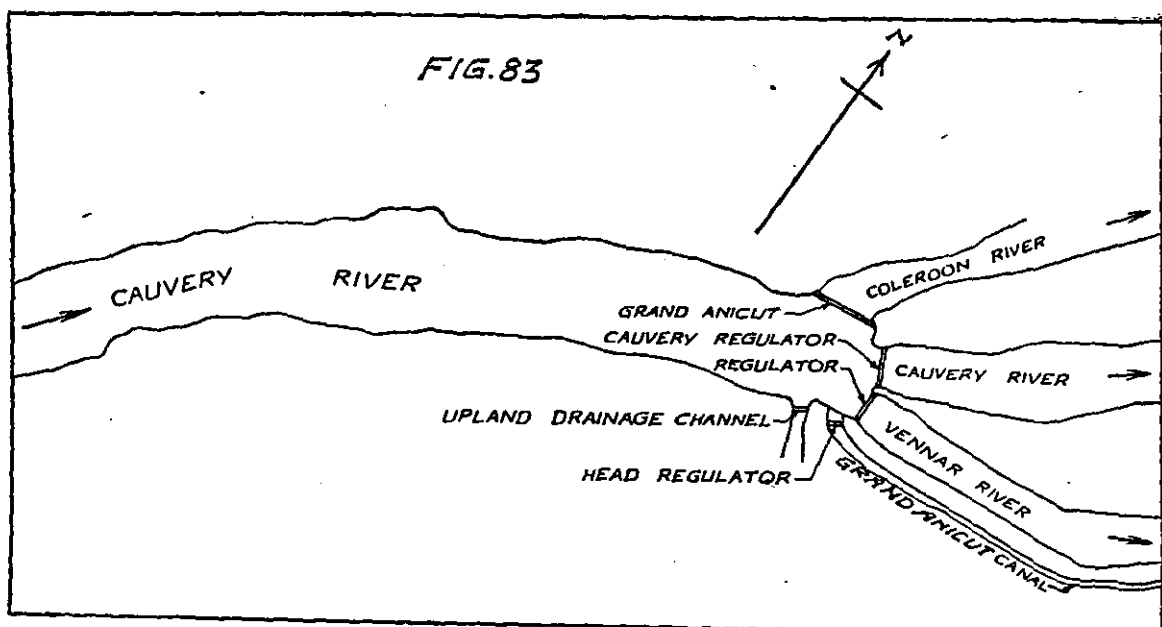
Suitable solutions were :—

- (1) to construct a channel with a velocity of 2.6 f/s, width = 160 ft. and depth = 15 ft., with baffle falls to dissipate energy, or
- (2) to construct a rapid, similar to the one recommended in the case of the tail channel of Spin Karez waste weir—*vide* experiments with a model of the Waste Weir channel of the Spin Karez Reservoir (Baluchistan) described in Part III of 1937-38 Annual Report.

The local authorities then suggested abandoning the existing outfall channel and constructing a rapid along the alignment AC—*vide* figure 82. The bed slope of the proposed rapid = 1 in 25. It was pointed out that a slope of 1 in 25 would develop a velocity of about 28 f/s, if the flume were of smooth cement concrete, and the main danger in a steep flume constructed in treacherous soil was that, in the course of time, the soil might be scoured from under the flume, causing subsidence. If, however, this was not considered to be a likely trouble, then the flume would obviously be cheaper.

22. SILTING OF THE GRAND ANICUT CANAL, CAUVERY RIVER (MADRAS).

The Cauvery river is split into 4 channels at the Grand Anicut—see figure 83.

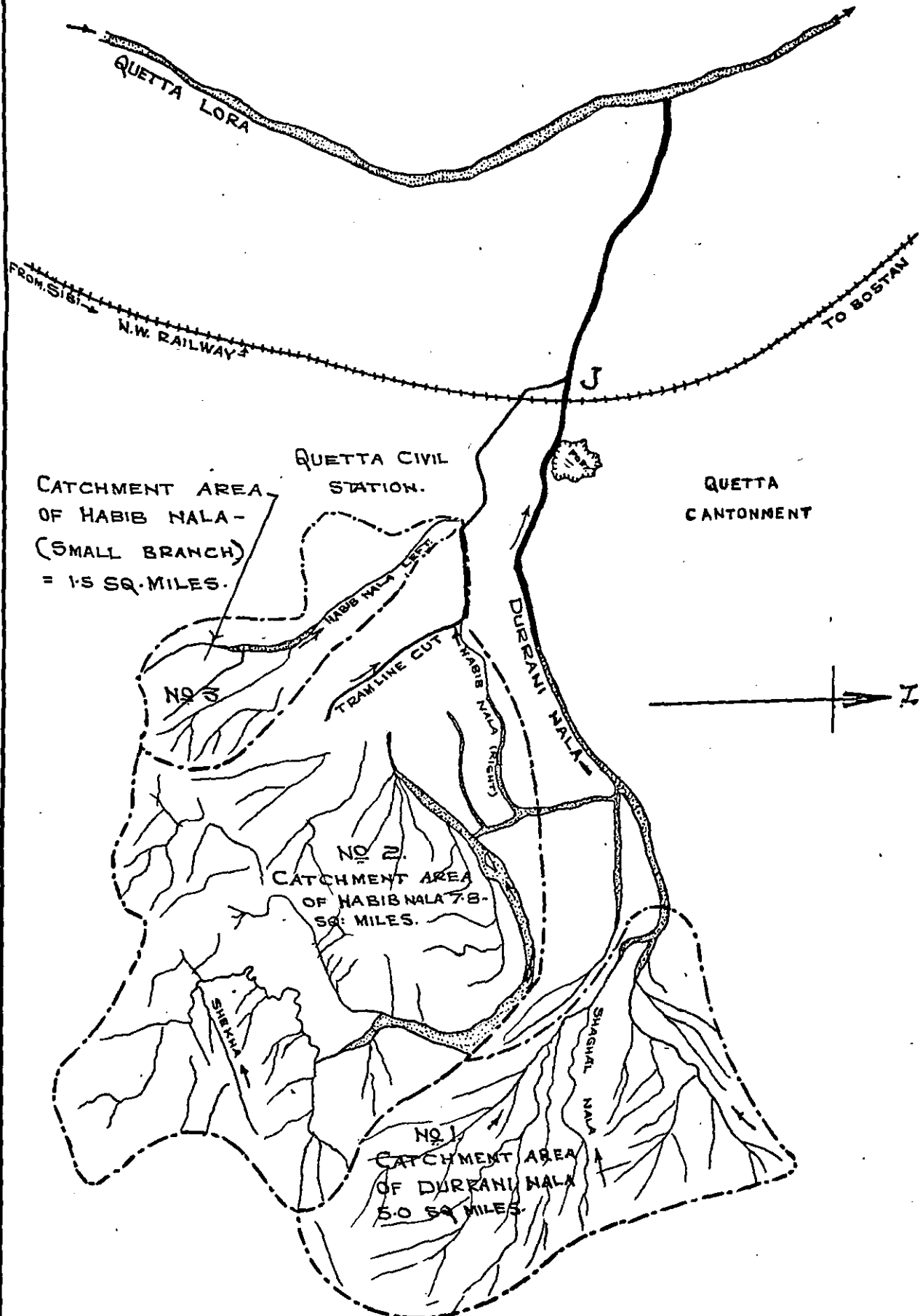


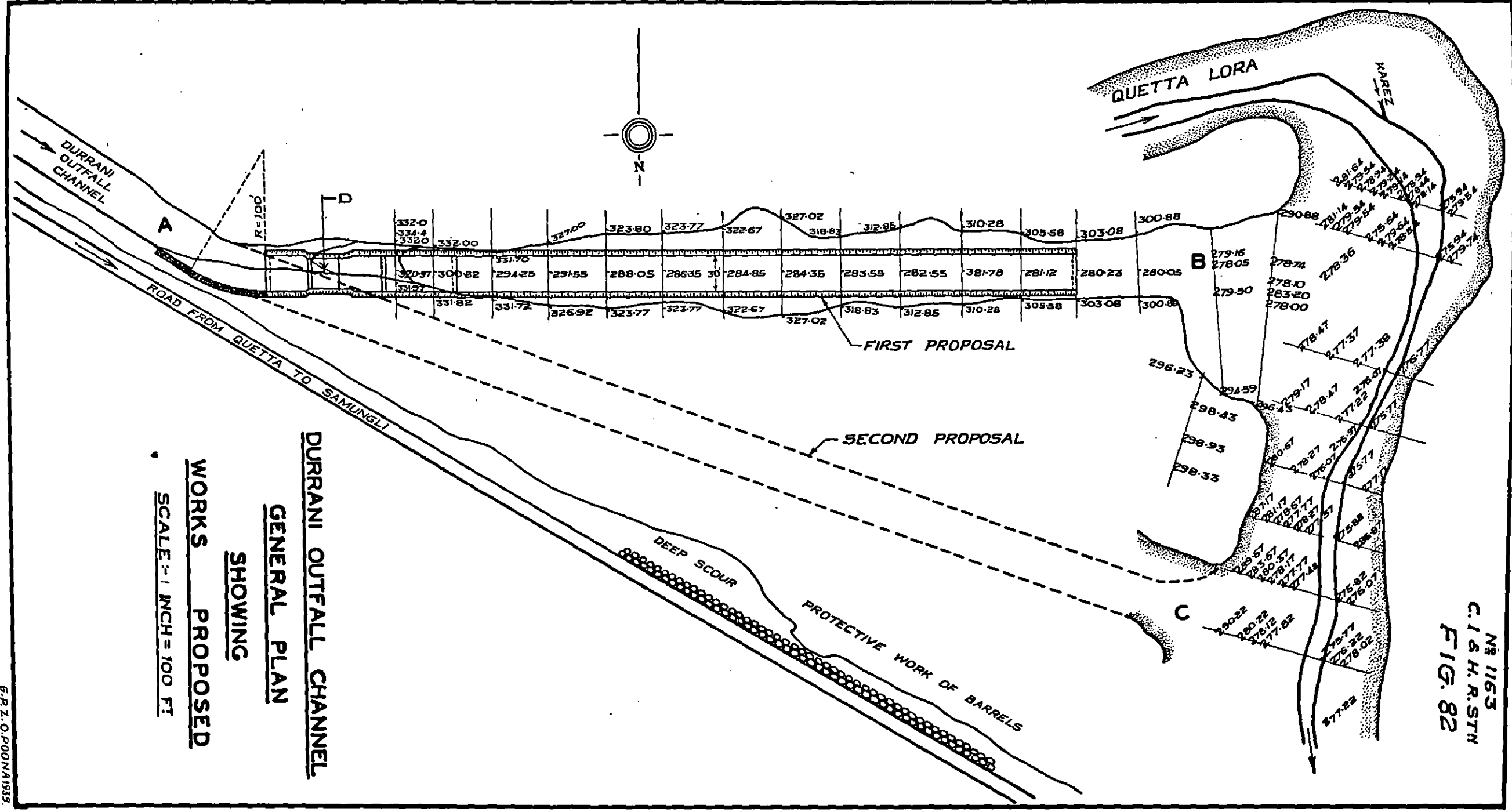
Water spilling over the Grand Anicut on the left bank, flows into the Coleroon river; and two regulators in midstream pass water into the Cauvery and Vennar rivers.

FIG: 80

PLAN SHOWING
DURRANI & HABIB NALAS
WITH THEIR CATCHMENT AREAS
NEAR QUETTA.

SCALE 1"=1 MILE





DURRANI OUTFALL CHANNEL
GENERAL PLAN
SHOWING
WORKS PROPOSED
 SCALE - 1 INCH = 100 FT

No 1163
 C. I. & H. R. STN
 FIG. 82

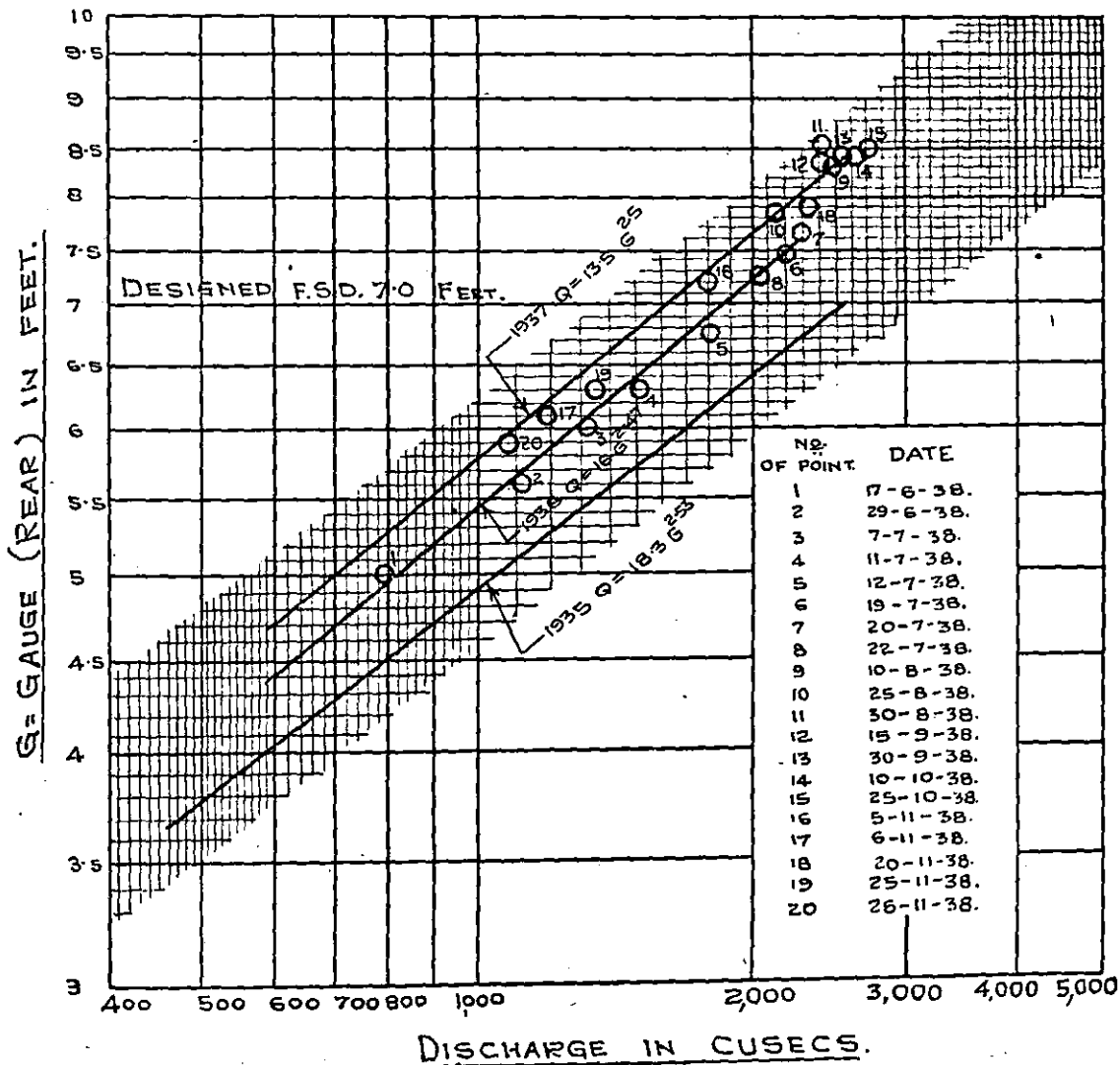
The Grand Anicut Canal, which was opened in 1933, takes off from the Cauvery at right angles from the right flank between the Vennar and Upland Drainage channel, and flows only from June to January.

Figure 84 shows discharge : gauge curves of the Canals for the years 1935, 1936 and 1937 from which the deterioration of the canal during that period is obvious.

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**FIG: 84 SHOWING DETERIORATION
IN DISCHARGE: GAUGE (REAR) IN GRAND ANICUT CANAL
IN 1935, 1936, 1937, AND 1938.
DUE TO SILTING OF G. A. CANAL.
DESIGNED DISCHARGE = 4350 Cs. WITH F.S.D. OF 7 FT.**

NOTE:- 1938 POINTS SHOWN THUS. ○



The problem of excluding silt from the Canal was referred to this Station.

The Canal design was first considered.

Table 14 below shows the ratio of various dimensions of the Canal, as designed compared with the corresponding regime dimensions according to the Lacey formulæ for a silt factor $f_d=1.6$, derived from the analysis of bed silt of the canal downstream of the Head Regulator (mean diameter = 0.74 mm.)

TABLE 14.

| Serial No. | — | Regime dimensions with $f_d=1.80$ F. S. Q.=4,360 cusecs. | Designed dimensions. | Ratio of $\frac{\text{designed dimensions}}{\text{regime dimensions}}$ Col. 4/Col. 3. | Lacey formulae used in Col. 3. |
|------------|-----------------|--|----------------------|--|--------------------------------|
| 1 | 2 | 3 | 4 | 5 | 6 |
| 1 | P (ft.) . . . | 176 | 200 | 1.14 | $P = 2.67 \sqrt{Q}$ |
| 2 | R (ft.) . . . | 6.6 | 6.55 | 0.99 | $R = 0.473 (Q/f)^{1/3}$ |
| 3 | A (sq. ft.) . . | 1160 | 1309 | 1.13 | $A = PR$ |
| 4 | V (f/s) . . . | 3.76 | 3.33 | 0.89 | $V = 1.1512 \sqrt{fR}$ |
| 5 | Slope . . . | 0.000293 =1/3413 | 0.00015 =1/6667 | 0.51 | $S = \frac{Vf}{3106 R}$ |

It will be seen that the designed slope was inadequate, being only half that required for silt of the grade found at the head of the Canal.

In 1938, the water in the river upstream of the canal regulator was raised to reduce the quantity of silt drawn by the canal. By referring to fig. 84 (in which circles and statement in right margin show conditions in 1938), it will be seen that though up to about 22nd July 1938, the 1938 points were an improvement on the 1937 conditions and lay on the 1936 curve, thereafter the canal began to deteriorate and the remaining points 9 to 20 (*i.e.*, from 10th August 1938 to 26th November 1938) lay close below the 1937 curve. From this it appeared that though the new method of regulation (*i.e.*, increasing the gauge reading above the Anicut) caused a temporary respite, remedial measures were likely to be required later. The possibility of such measures is under investigation.

23. APPROXIMATE FORMULA FOR VENTURI FLUMES.

An enquiry was received for a formula which would give the approximate discharge passing through a Venturi Flume, where the cost of accurate calibration by means of a model was not justified.

The direct application of the Bernoulli energy equation

$$\frac{V_1^2}{2g} + D_1 = \frac{V_2^2}{2g} + D_2$$

is often very inaccurate, due mainly to the velocity head exceeding $v^2/2g$ because of non-uniform velocity distribution and the gauge readings in the throat being affected by curved flow.

As both these errors are closely proportional to V^2 , both can be reduced by the use of a coefficient in the expression for velocity head, V_1^2/K_1 being taken as the velocity head at the upstream gauge and V_2^2/K_2 that in the throat.

The discharge equation, derived from the modified Bernoulli formula, is then

$$Q = B_1 B_2 d_1 D_2 \sqrt{K_1 K_2} \sqrt{\frac{d_1 - D_2 - X - h_f}{K_1 B_1^2 d_1^2 - K_2 B_2^2 D_2^2}}$$

Where Q = discharge

B_1 = mean width at upstream gauge

B_2 = mean width at throat gauge

d_1 = depth at upstream gauge

D_2 = depth at throat gauge

X = height of hump

h_f = frictional loss of head between gauges.

The above formula, when applied to Jeur Aqueduct flume (Nira Left Bank Canal—Bombay), which had been calibrated by means of a model, showed that the error was least when $K_1=K_2=57$, the formula then becoming,

$$Q=7.55 B_1 B_2 d_1 D_2 \sqrt{\frac{D_1 - D_2 - h_f}{B_1^2 d_1^2 - B_2^2 D_2^2}}$$

as against: the equation derived from Bernoulli

$$Q=8.02 B_1 B_2 d_1 D_2 \sqrt{\frac{D_1 - D_2 - h_f}{B_1^2 d_1^2 - B_2^2 D_2^2}}$$

The errors of both formulæ are shown below for 4 examples covering the range of discharge of this flume :—

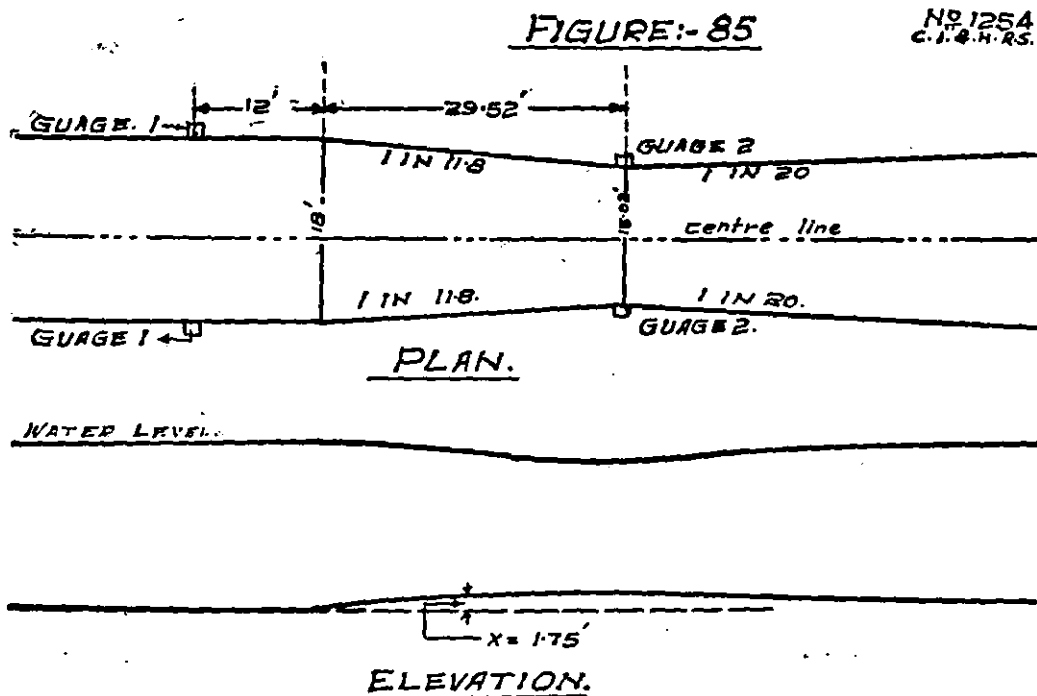
TABLE 15.

| | | | | |
|--|------|-------|------|-------|
| 1. D_1 in ft. | 3.8 | 5.0 | 5.38 | 7.2 |
| 2. D_2 in ft. | 2.5 | 4.0 | 4.0 | 6.0 |
| 3. Q cusecs (actual) | 306 | 426 | 505 | 704 |
| 4. Q cs. (calculated by new formula) | 294 | 431 | 501 | 733 |
| 5. Error per cent (of 4) by new formula | -3.9 | +1.2 | -0.7 | +4.0 |
| 6. Q (calculated by Bernoulli directly applied) cusecs | 312 | 456 | 532 | 779 |
| 7. Error per cent (of 6) by Bernoulli formula | +2 | +7.05 | +1.4 | +10.6 |

(D_1 = depth at upstream gauge, above hump level.)

The error of the new formula lies within -4 and +1 per cent compared with +1 to +11 per cent with direct application of the Bernoulli equation.

The dimensions of the Jeur Aqueduct flume are shown in fig. 85. In other flumes of similar design, the same values of K_1 and K_2 are likely to give moderately accurate results.



24. MEASURING DEVICE AT THE HEAD OF THE MUTHA RIGHT BANK CANAL (POONA).

The Mutha Right Bank Canal draws its supply from Lake Fife direct from sluices in the Dam. A measuring device was required to replace the present method of computing discharge from gate openings and differences of head with the aid of tables. A standard Standing Wave Flume would have been suitable were it not for two particular requirements—firstly that it must operate with a large variation in water level downstream due to the periodic growth of weeds, there being some 2 ft. difference in canal gauge before and after weed-clearance with the same discharge (see figure 86), and secondly that afflux must be negligible when

the water in the lake was low, as this would restrict the supply by reducing the head on the sluices.

A Standing Wave Flume with hump sufficiently high to satisfy the first requirement would not satisfy the second. A Venturi Flume with a low hump and operating with subcritical velocity would be more suitable, but even this would present difficulties because, if it operated with sub-critical velocity when the canal gauge was near the lower curve in figure 86, the Venturi head when the gauge was near the upper curve would be so small that the accuracy of the measurement would be inadequate, particularly with small discharges.

The difficulty was overcome by recommending a flume which would operate with either critical or sub-critical flow, that is, as a Standing Wave Flume in part of its range and as a Venturi flume with subcritical velocity in the remainder.

The characteristics of two alternative designs are shown in Figure 86 by the curves representing the modular limit, that is the stage at which the flow is just critical. Points above the modular limit curve represent Venturi or subcritical flow, when the discharge would be calculated from calibration tables using gauge readings observed upstream of the flume and in the throat or alternatively by applying a correction coefficient for partial drowning. Points below the curve represent critical flow in which case a standing wave would occur and the discharge would be calculated by means of the ordinary Standing Wave Flume formula, using only the upstream gauge reading. By this means afflux would be avoided under those conditions when it would be undesirable; and the Venturi head would always be sufficient to give the required accuracy of measurement.

The disadvantage of this combination of Venturi and standing wave methods is the slight instability of flow, and therefore of gauge readings, which would occur within the small range when the flow was just below critical. In the case of the Mutha Canal, however, it would be possible slightly to raise or lower the discharge without inconvenience, in order to avoid this region of instability.

The design recommended, which is shown in figure 87, had a throat width 12 ft., hump height 4.5 ft., circular side contractions and a 1 in 10 expansion. The total length of this flume was 60 ft. which was incidentally no longer than required for a S. W. F.

25. THE EFFECT OF CANAL CLOSURES ON WEED GROWTH IN THE MUTHA RIGHT BANK CANAL, POONA, DURING THE COLD WEATHER SEASON.

Introduction.

The Mutha Right Bank Canal is heavily infested with weeds and the canal is generally closed 4 times a year in October, January, April and June or July for removal of weeds, each closure lasting 3 days and the intervals between two closures being about 3 months. The following analysis was done with the object of seeing the effect of the January closures on weed growth.

Owing to silting in the Canal, little water flows below 3.5 ft. and the formula

$$Q=C (G. R.-3.5)^{1.8}$$

gives a useful rough correlation for $Q > 100$ cusecs where G. R.=gauge reading in ft. at the head of the Canal.

Fig. 88 shows C in the formula $Q=C (G. R.-3.5)^{1.8}$, plotted against days after closure during October-November for 1934-35, 1937-38 and 1938-39. The decrease in C was due to weed growth.

Two different methods of regulating the Canal discharge have been followed in the recent past. In 1934-35, the Canal was run with an almost constant discharge throughout the rotation; whereas in 1937-38 and 1938-39, the Canal was run with high discharges during the first 5 or 6 days of each rotation and then with very low discharges (20 to 40 cusecs) for the remaining period *i.e.*, just enough discharge for the water supply of the City and Cantonment. It is understood that the change was made with the object of reducing weed growth, but it has had little effect on the rate of weed growth after the October closure—as can be seen by comparing graphs for 1934-35 and 1937-38 (figure 88).

It will be seen from figure 88 that the coefficient begins to be affected about 20 to 30 days after the closure, the depth of flow required to pass a given discharge then being progressively greater. As losses increase rapidly with depth

FIG:-86 SHOWING RELATIONSHIP BETWEEN GAUGE READINGS & DISCHARGE BEFORE & AFTER CLOSURE FOR WEEDS CLEARANCE. VALUES OF d_3 LYING ABOVE THE MODULAR LIMIT LINES—SHOWN CHAIN DOTTED GIVE VENTURI CONDITIONS & BELOW THESE LINES GIVE STANDING WAVE CONDITIONS.

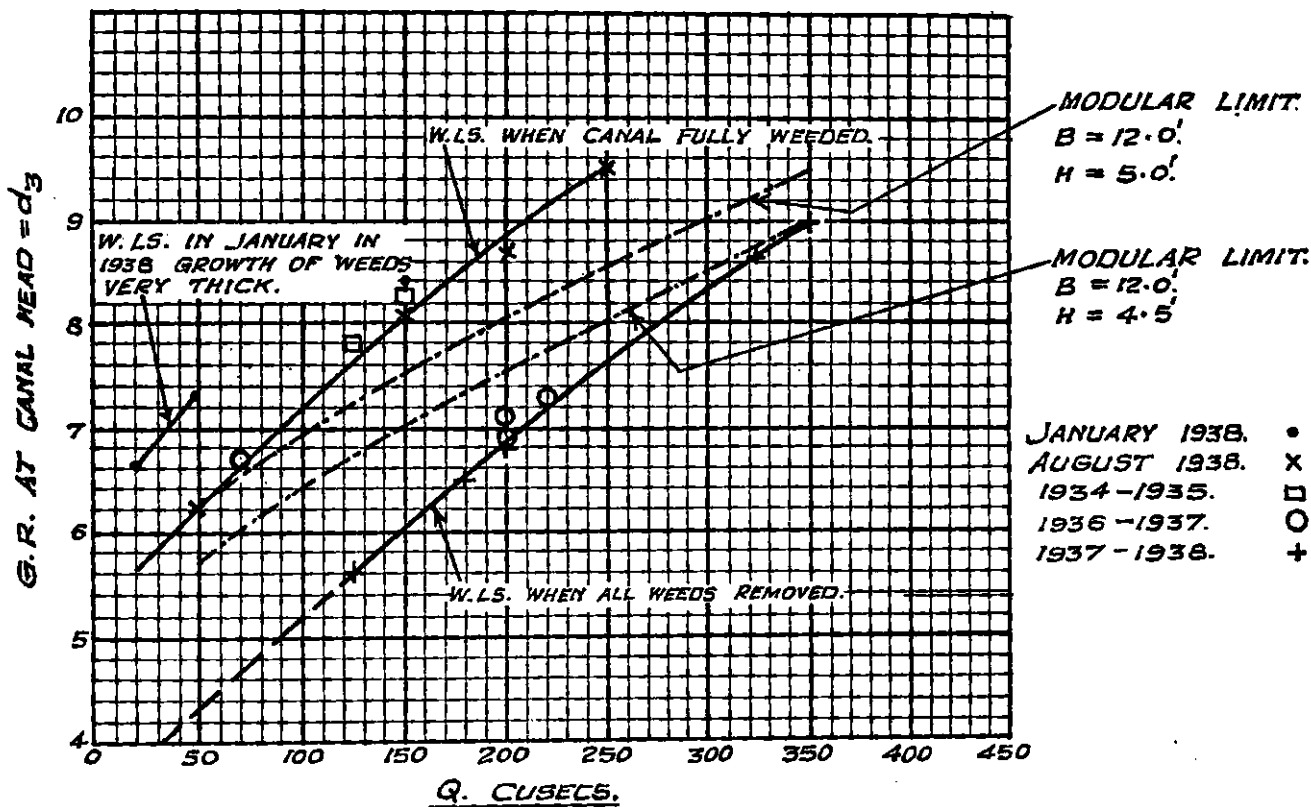


FIG-87, VENTURI FLUME FOR M. R. B. CANAL.

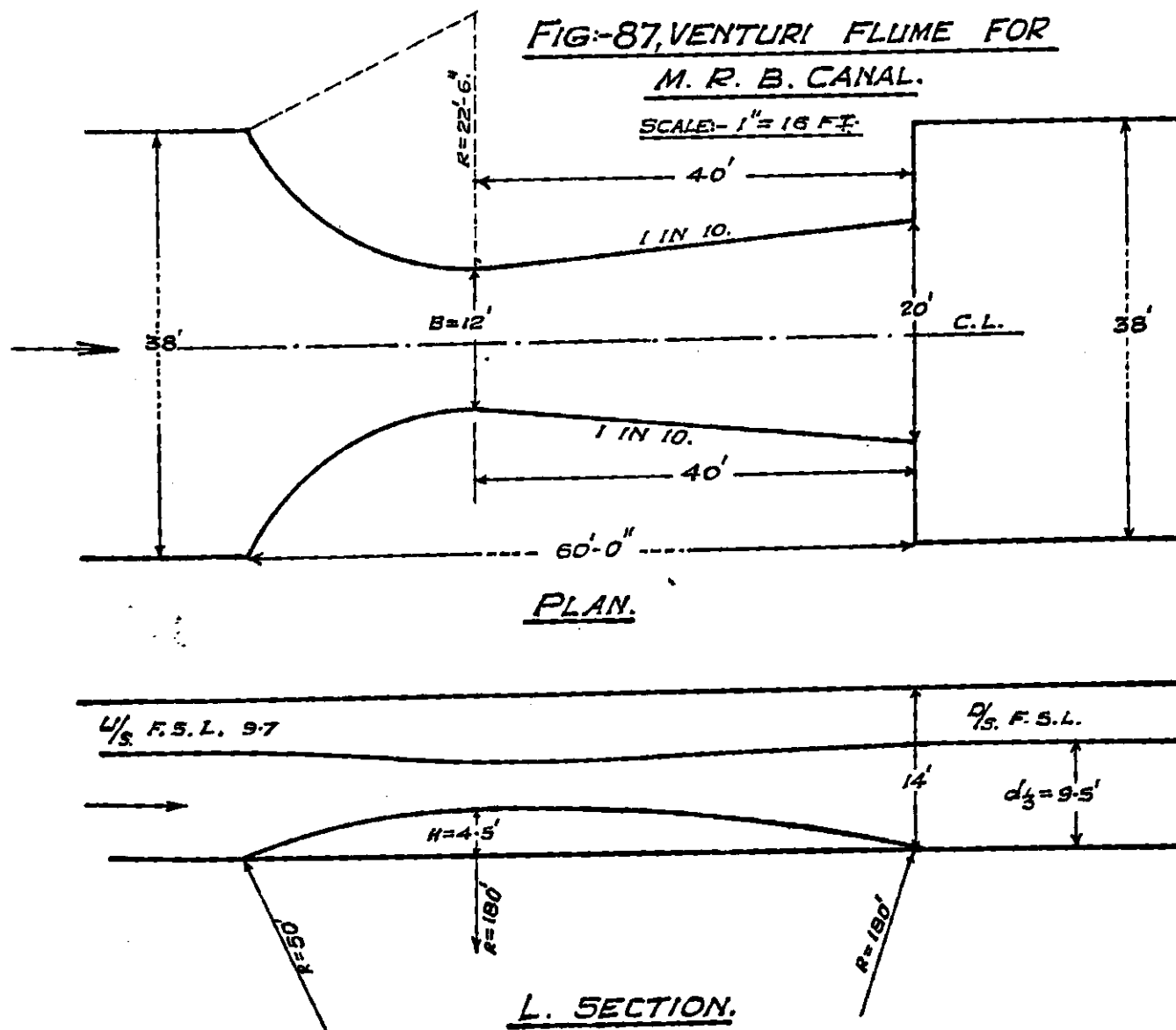
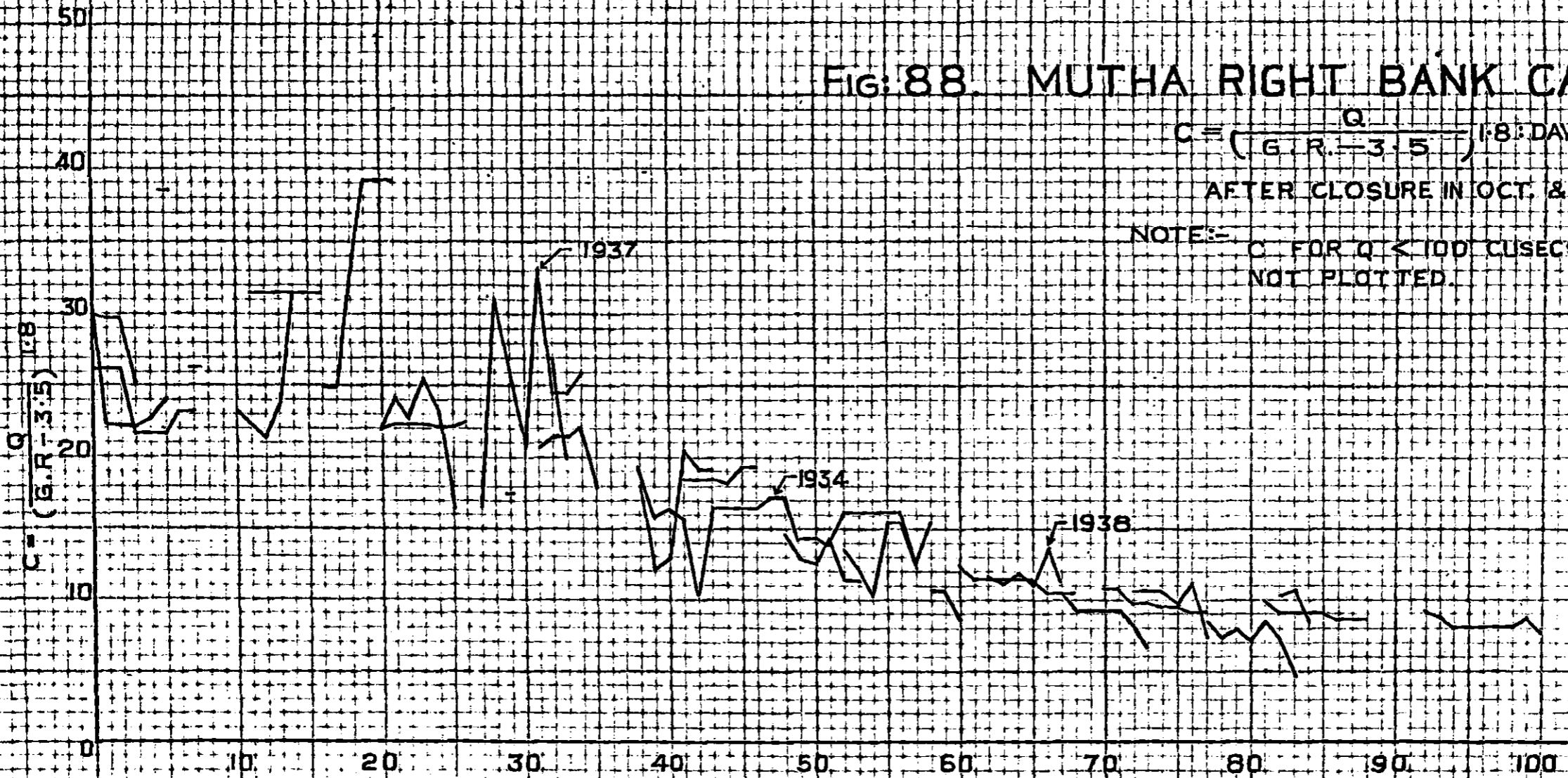


Fig: 88. MUTHA RIGHT BANK CANAL

$$C = \frac{Q}{(G.R. - 3.5)} \cdot 1.8 \text{ DAYS}$$

AFTER CLOSURE IN OCT. & NOV.

NOTE:- C FOR Q < 100 CUSECS NOT PLOTTED.



NUMBER OF DAYS AFTER CLOSURE ON: 16-10-34 _____
 19-10-37 _____
 13-11-38 _____

Fig: 88

it was suggested that it is advisable to have the January closure within 2 months of the previous closure.

The conclusions drawn were that :—

- (1) Weed growth in the Mutha Right Bank Canal being a maximum after the October closure is little affected by the method of regulation.
- (2) As conditions in the cold weather are highly favourable to weed growth, it is desirable in order to reduce losses, which increase rapidly with depth, to have the January closure within 2 months of the previous closure.

26. UNSUITABILITY OF PROPOSAL FOR SIPHON SPILLWAY OVER THE WASTE WEIR OF THE LAKE ARTHUR HILL AT BHANDARDARA (BOMBAY).

In connection with the proposal to increase the storage capacity of Lake Arthur Hill, the Superintending Engineer, Deccan Irrigation Circle, requested opinion on the suitability of a siphon spillway and asked for a rough design for comparison with other proposals, such as waste weir with automatic gates.

It was pointed out that the conditions are not such as would make an automatic siphon spillway advantageous. The normal advantages of a siphon spillway are :—

- (i) greater discharge (about 3 times) per foot run,
- (ii) no waste of water,
- (iii) automatic action,
- (iv) freedom from maintenance cost.

Of these (i) and (ii) would not hold at Bhandardara ; because regarding (i) the waste weir consists of a cut in the saddle of a hill, at a level only 10 ft. below H. F. L. and having upstream to downstream length varying from 250 ft. to 600 ft. In order to give suction head, the siphon spillway would have to be situated near the downstream edge, where its discharge capacity would be limited by the shallow approach, thus necessitating an increase in the width of spillway. This would be very costly. Regarding (ii), leakage through gates would not lead to waste owing to the existence of a pick-up weir lower down the valley.

The only advantage therefore would be the freedom from maintenance cost and the extra safety of an automatic device ; as a set-off against which there would be 1 ft. less storage than with gates.