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Public Works Department

Technical Paper No. 58

Summary of the Annual Note on Experiments  
carried out by the Irrigation Research Division  
during 1936-37 at Lake Fife Hydrodynamic  
Research Station near Poona

BY

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Summary of the Annual Note on experiments carried out during 1936-37  
at Lake Fife Hydrodynamic Research Station at Khadakvasla near Poona.

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In this Note—

$Q$  = total discharge.

$q$  = discharge per foot run.

$\equiv$  = equivalent to.

Thus  $Q = 6$  cusecs  $\equiv 480$  cusecs means discharge in model was 6 cusecs equivalent to 480 cusecs in the prototype.

*Sina River and Causeway at mile 5 of Nagar-Manmad Road.*



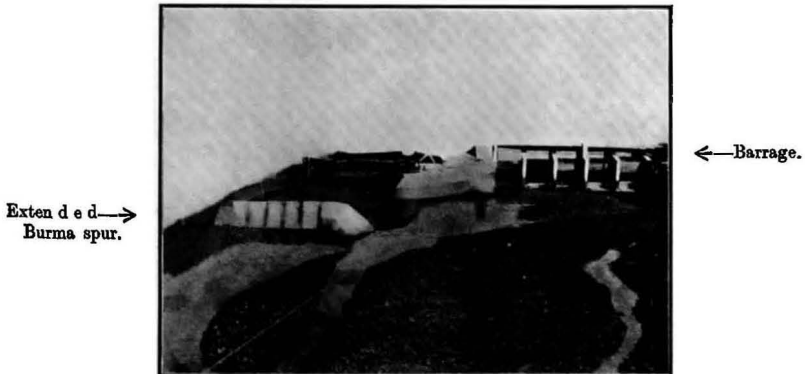
1399.—*Model of junction of the Sina River and Shendi Nala in experiments 8 and 9. Looking upstream.*

*Mula River at Rahuri, district Ahmednagar.*



1401.—*Experiment 9 with old groyne lengthened on left bank and 5 groynes round the convex right bank. Looking downstream.*

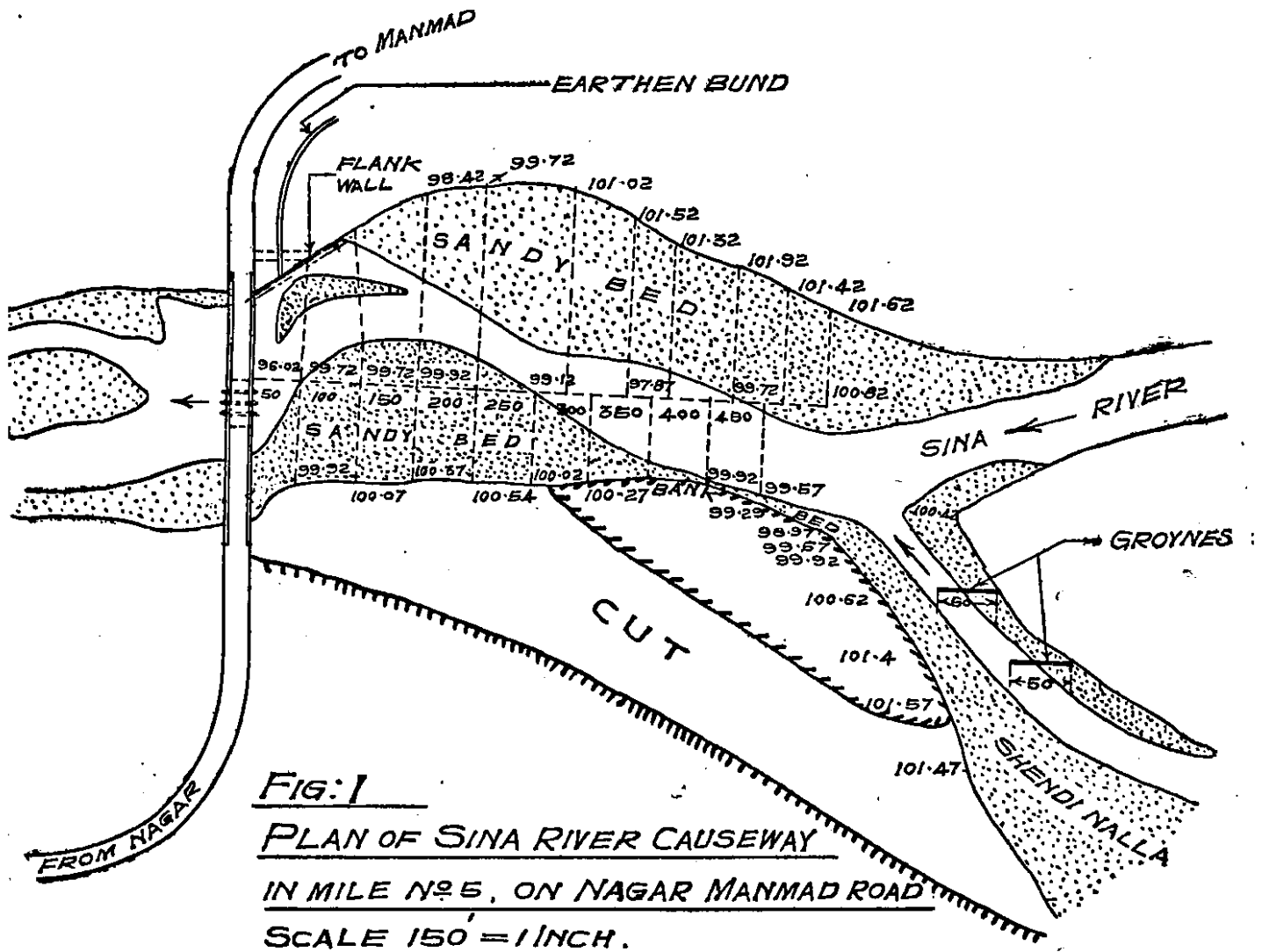
*Model of the Sarda River and Barrage (U. P.).*  
(1/300 Horizontal scale).



1436.—*Experiment 4. Burma spur extended by equivalent of 1020 feet at an angle of  $20^\circ$  d/s of the present alignment. Looking downstream.*

1. EXPERIMENTS WITH A MODEL OF THE SINA RIVER AND CAUSEWAY AT MILE 5 OF THE AHMEDNAGAR-MANMAD ROAD (DISTRICT AHMEDNAGAR, BOMBAY PRESIDENCY).

A causeway was built in 1920 downstream of the junction of the Sina River and Shendi Nala, 5 miles from Ahmednagar on the Manmad Road. Since then, scour has been occurring on the right bank above the causeway which, if not checked, will lead to the causeway being outflanked. In 1929, two masonry groynes were constructed at the right side of the Shendi Nala above its junction with the Sina River, with the idea that this would deflect the current towards the left bank above the causeway, and a flank wall was also constructed upstream of the causeway on the right bank—*vide* Photo 1399 and Fig. 1.



The former increased scour, and erosion took place behind the flank wall. The Shendi Nala is wide and shallow and has a steeper bed-fall than has the Sina River, which is narrow and deep, and the two meet at an angle of about 70°.

Experiments were carried out with a model, the scale ratios being worked out by the Lacey formulæ for a flood discharge of 13,000 cusecs. The scale ratios were:—

Horizontal scale ratio	=	1/70.
Vertical scale ratio	=	1/12.6.
Vertical exaggeration	=	5.55.
Discharge scale ratio	=	1/3896.

The experiments showed that the erosive action on the right bank upstream of the causeway was mainly due to the Shendi Nala flood flowing across to the



right bank with a high velocity and that the two masonry groynes constructed in the Shendi Nala, which were meant to deflect the high-velocity flow towards the middle of the causeway, only deflected the low velocity bed water and so accentuated the erosive action of the high-velocity top water. Experiments showed that erosive action can be checked by making a cut through the projecting nose on the left bank above the causeway as shown in thick black hatched lines in Fig. 1, the island being assisted to scour naturally—the design being similar to the cut through the Sara nose proposed to ameliorate conditions above the Hardinge Bridge over the Ganges on the Eastern Bengal Railway<sup>(1)</sup>.

**2. PART I:—RIVER TRAINING EXPERIMENTS WITH A MODEL OF THE MULA RIVER AT RAHURI (DISTRICT AHMEDNAGAR, BOMBAY PRESIDENCY) TO DETERMINE HOW BEST TO ERADICATE HIGH VELOCITY FLOW ABOVE THE CAUSEWAY WHICH INTERFERES WITH THE WORKING OF THE FERRY.**

In 1922, when the new high level causeway was built, the ferry across the Mula River at Rahuri, which had previously been located on the line of the causeway, was moved 550 feet upstream—*vide* Fig. 2. At low discharges, the River flow follows the left bank around the bend; but in high floods the flow shortcuts across the lime kankar projection on the right bank and the high velocity surface flow passes across to impinge on the left bank at the ferry landing stage, which tends to swamp the pontoons and makes landing unsafe. On the right bank, on the other hand, a reverse current exists—as a result of which the ferry boat swings round and is not propelled towards the bank by the current. A groyne constructed in 1929 on the left bank upstream of the ferry, with the idea of diverting the high velocity flow away from the left bank ferry landing, did not materially improve conditions.

The object of the model experiments was to test the relative merits of various alternative methods of training works designed to obtain smooth flow for all working conditions of the ferry.

Experiments were carried out with a model having an exaggerated vertical scale, derived from Lacey's formulæ. The horizontal and vertical scale ratios were 1/80 and 1/13·35 respectively; the discharge scale ratio was 1/5170.

The model reproduced existing conditions accurately. Experiments showed that the present groyne—which was built in 1929—even when extended by 50 per cent., did not protect the ferry. A second groyne, built between the old groyne and the landing stage, was also not effective. When, however, the high-velocity flow on the right bank near the convex bank was checked by constructing 5 groynes pointing 30° upstream to the lines of cross sections 6 to 10—shown in strong black lines in Photo 1401 and Fig. 2—the left bank groynes came into action and the velocity distribution was markedly improved, becoming almost normal.

**2. PART II:—EXPERIMENTS WITH A DEFLECTOR ON THE DOWNSTREAM GLACIS OF THE MULA RIVER CAUSEWAY TO PREVENT THE PAVEMENT BEING UNDERMINED.**

A pakka pavement, 90 feet long, sloping down at 1 in 50, has been constructed downstream of the causeway, and considerable scour takes place just downstream of the pavement. Hitherto stones have been thrown into the scour hole every year as protection; but these are washed downstream during floods. To prevent this scour, a standard deflector 1·5 feet high (= 1/10th the depth of water during floods) was constructed at the downstream end of the pavement and an experiment was carried out with a geometrically similar model (1/10th scale ratio), sand being laid horizontally downstream of the pavement. Without a deflector there was a definite tendency to undermining; but with a deflector added, sand actually piled up against the downstream face of the deflector. This result [see Bombay P. W. D. Technical Paper No. 44<sup>(2)</sup>] is due to the deflector stabilising flow and causing a bed roller with horizontal axis to form just downstream of the deflector.

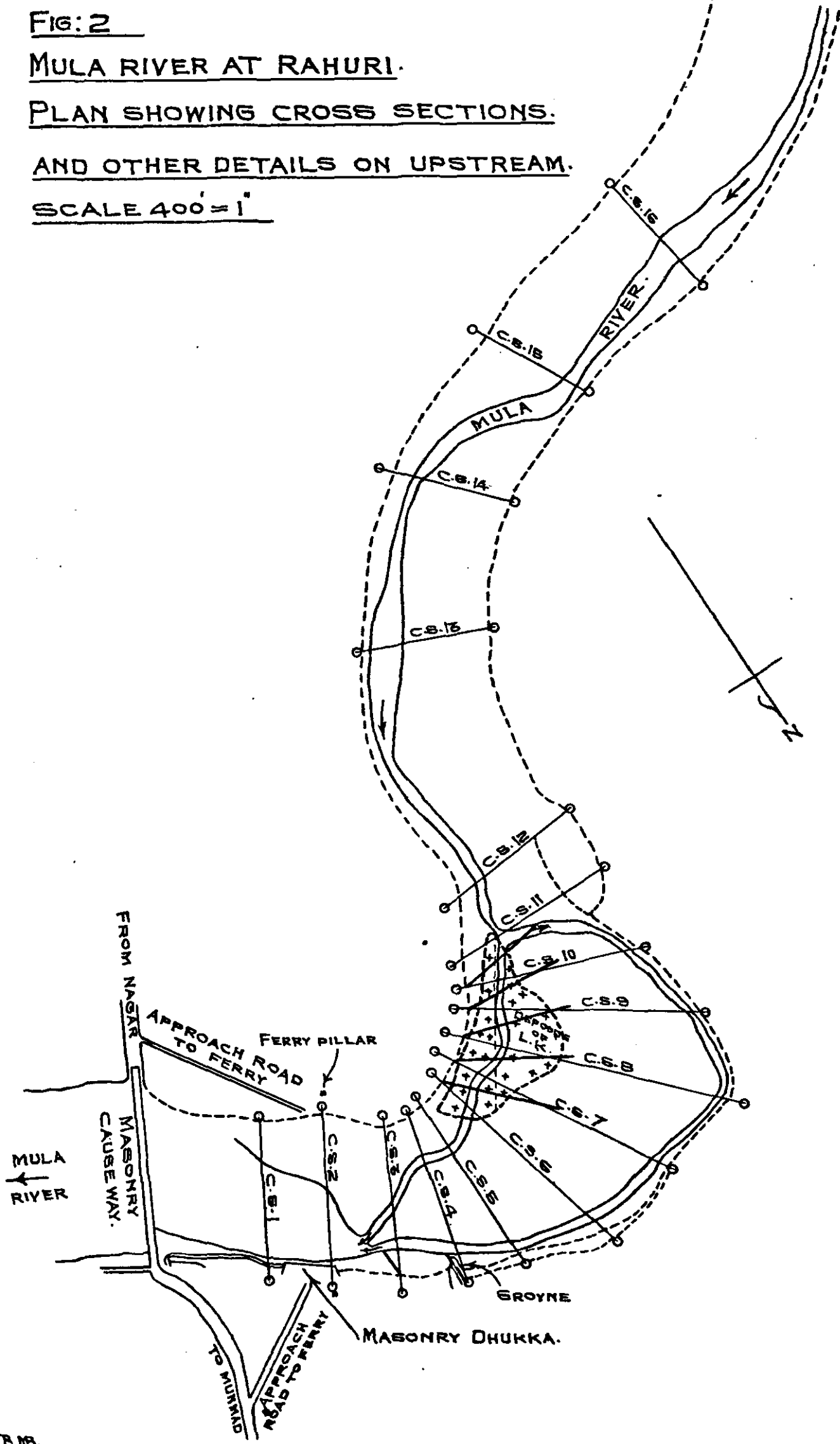
FIG: 2

MULA RIVER AT RAHURI.

PLAN SHOWING CROSS SECTIONS.

AND OTHER DETAILS ON UPSTREAM.

SCALE 400' = 1"



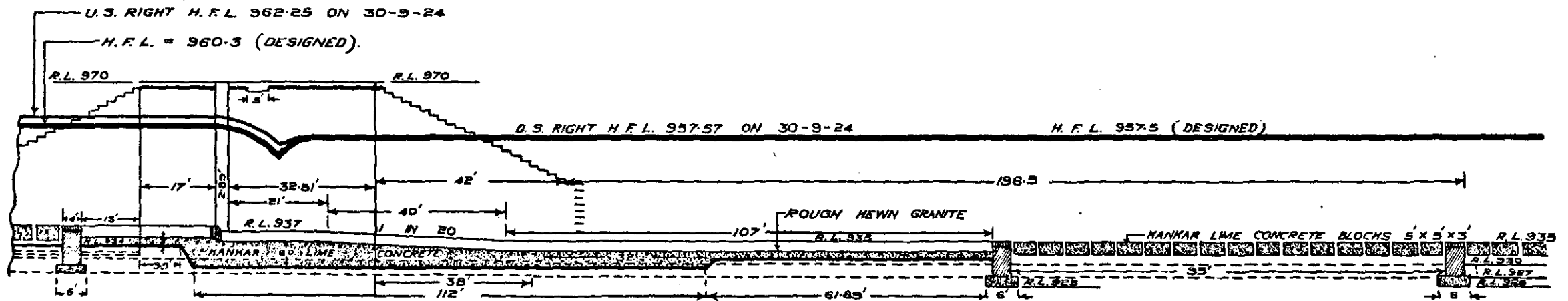


FIG: 3

GANGES CANAL

BHIMGODA HEADWORKS SECTION

OF UNDER SLUICES

DISCHARGE OF SLUICES = 140000 CS.

ON 30-9-1934

OR PER FOOT RUN 465 CUSECS.

SCALE 1 INCH = 30 FEET.

3. EXPERIMENTS WITH A MODEL OF THE SLICES OF THE BHIMGODA WEIR CONSTRUCTED ACROSS THE GANGES AT HARDWAR (UNITED PROVINCES) TO ASCERTAIN HOW BEST TO PREVENT JETTING AND ABRASION OF THE FLOOR DOWNSTREAM.

The Bhimgoda Weir and River sluices at Hardwar were completed in 1920. The Upper Ganges Canal takes off from the Right Bank. The weir consists of 6 bays of sluices, each 50 feet long on the right bank, and 6 bays of open weir, each 300 feet long on the left bank. The maximum recorded flood was 620,000 cusecs and occurred on 30th September 1934, when the discharge per foot run through the sluices was 465 cusecs, the upstream water level being 962.25 and the downstream 957.57. This flood was in excess of the estimated maximum flood of 510,000 cusecs with an assumed upstream depth of 960.3.

The river bed is composed of boulders of various sizes up to 1' 9" diameter. The problem under investigation was how best to protect the weir downstream of the under-sluices. The pavement is surfaced with granite blocks and boulder sets and is flat for a length of 21 feet. It then slopes down at 1 in 20 for 40 feet and is again flat for 107 feet, the total length of pavement being 168 feet. This is followed by a horizontal talus which was originally composed of loose concrete blocks, protected at its downstream end by a masonry curtain wall. As, however, these blocks had worn away very rapidly, the spaces between the blocks were grouted with cement concrete in 1932-33 and the whole area of the talus brought to the original level—see Fig. 3. During the past two flood seasons, the concrete has been abraded by as much as 2 feet depth in places and a decision was necessary as to further action. One remedy suggested was to lay a granite floor in place of, or on top of, the concrete blocks. This would withstand abrasion for several years but would cost some Rs. 50,000.

Experiments were carried out to see if a relatively cheap alternative, based on successful results hitherto obtained with a baffle and deflector<sup>(2)</sup>, would meet the case.

The floor of the under-sluices was reproduced in a geometrically similar model to a scale ratio of 1/36, representing a width of 65 feet. A mixture of  $\frac{5}{8}$ " pebbles and  $\frac{1}{8}$ " sand was laid on the bed downstream of the pavement, representing boulders and shingle respectively.

Two series of experiments were carried out. In Series I, the gate opening was equivalent to 6 feet with  $q \equiv 86$  cusecs and in Series II, the gate was fully open with  $q \equiv 465$  cusecs.

In both series, the baffle used was 2 feet high, constructed 10.5 feet downstream of the toe of the 1 in 20 sloping glacis and the deflector sloping up at 1 in 5 was also 2 feet high at its downstream end, at the end of the granite block pavement; the height of the deflector was 1/11th the depth of water during maximum flood. In Series I, without a baffle or a deflector, there was a strong tendency to cross flow, and the scour which took place immediately downstream of the pavement was uneven, being greatest at one side.

When a deflector was added, there was less tendency to cross flow and sand piled up against the downstream face of the deflector. With a baffle and deflector, the flow was more stable and cross flow and scour were less.

In Series II, sand and pebbles were injected upstream of the gates.

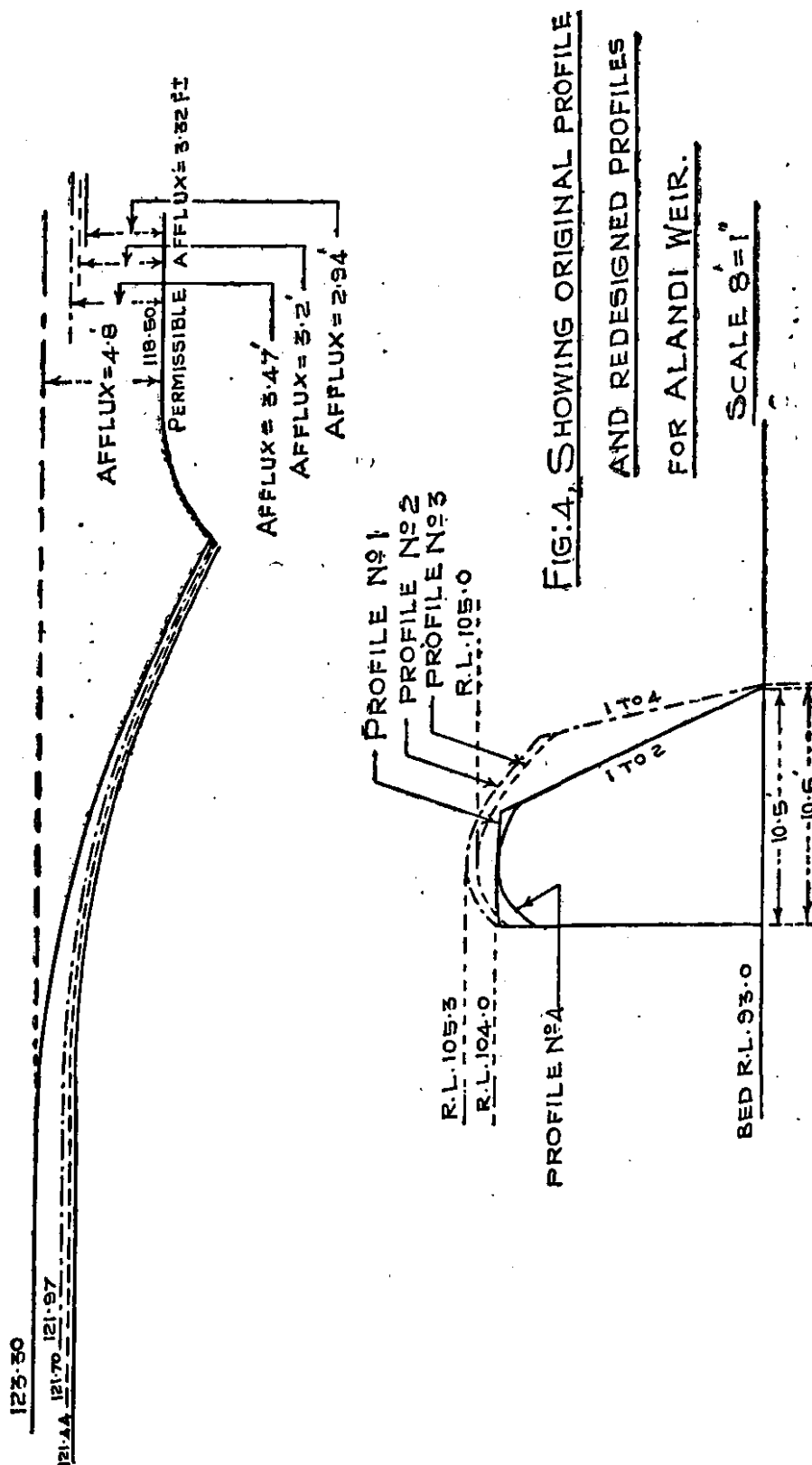
Without a baffle or deflector, heavy scour occurred downstream of the pavement; with a deflector only, the pebbles passed over the deflector and piled up against the downstream face. When a baffle was added, pebbles and sand passed over the baffle and moved downstream with a lower velocity than when there was no baffle and banked up to a slope of 1 in 3.8 downstream of the deflector. The results were in general agreement with those mentioned in Bombay P. W. D. Technical Paper No. 51<sup>(3)</sup> and those obtained by Messrs. Butcher and Atkinson<sup>(4)</sup>. It was also found that at corresponding velocities, boulders of sizes up to 6 feet diameter, weighing 40 times as much as those at Bhimgoda, were carried over the baffle. There was thus no danger whatever of boulders lodging upstream of the baffle.

4. EXPERIMENTS WITH A MODEL OF A WEIR DESIGNED TO STORE WATER FOR THE ALANDI WATER SUPPLY PROJECT (DISTRICT POONA) TO ASCERTAIN AFFLUX CAUSED THEREBY AND THE BEST DESIGN TO REDUCE AFFLUX.

In the Alandi Water Supply Project, a weir is to be built across the River Indrayani to pass a flood of 85,870 cusecs, with overfall for flood discharge over the whole length of the weir, equivalent to 256 cusecs per foot run.

The bed level of the River is	=	93.0
and high flood level	=	118.5.
Stipulated afflux	=	3.3 feet.
Maximum permissible level above weir =		121.8.

Experiments were carried out in a part width, geometrically similar model with the weir  $\cong$  11 feet high and a discharge  $\cong$  256 cusecs per foot run. With the weir as originally designed, afflux was 4.8 feet, which exceeded the permissible afflux by 1.5 feet—*vide* Fig. 4.



A high coefficient weir profile was then designed by shaping the top surface of the profile according to the undersurface of an aerated nappe of a sharp crested weir with free flow downstream.

The afflux with this weir profile was 3.2 feet, with a weir  $\cong$  12 feet, and 3.47 feet when the weir was 12.3 feet high compared with the permissible afflux of 3.3 feet. A weir was also tested having the same height as the original, viz., 11 feet and the same top-width  $\cong$  5 feet; but with crest shaped according to the improved design. The afflux was found to be 2.94 feet compared with 4.8 feet of the original design.

It was thus found possible to raise the crest to 12 feet height without causing excess over the permissible afflux, thus increasing the storage capacity of the reservoir by 25 per cent., or alternatively, to modify the original profile to reduce the afflux by 39 per cent.

##### 5. DESIGN AND CALIBRATION OF A TRAPEZOIDAL STANDING WAVE FLUME FOR THE POONA DRAINAGE OUTFALL SEWER AT BAHIROBA PUMPING STATION.

The Poona Drainage Outfall Sewer is egg-shaped in section—5'  $\times$  3' 4". It discharges into the grit chambers of the Bahiroba Pumping Station and then flows into the balancing tanks. A Lea Recorder had been fixed in the sewer about 170 feet upstream of the grit chambers, which was designed to measure discharges calculated according to Kutter's formula under uniform flow conditions. But due to fluctuations of water level in the balancing tanks and the grit chambers, the levels in the sewer were affected by any drawdown or afflux, so that uniform flow conditions were not obtained and the recorder showed wrong discharges.

It was, therefore, proposed to construct a standing wave flume of special design. Conditions were unsuitable for a standard, vertical-sided, standing wave flume (5), because

- (i) no hump was permissible as it would cause deposition of heavy detritus carried in the channel;
- (ii) a high downstream water level, without drowning, was necessary for a wide range of discharges (viz., 5 to 40 cusecs) to prevent temporary high levels in the balancing tanks affecting the readings;
- (iii) at high discharges the modular limit had to be as high as practicable to prevent undue heading up upstream, which would submerge the sewer.

The design found suitable for the particular conditions is shown in Fig. 5, and consists of a flume with compound trapezoidal throat section (5).

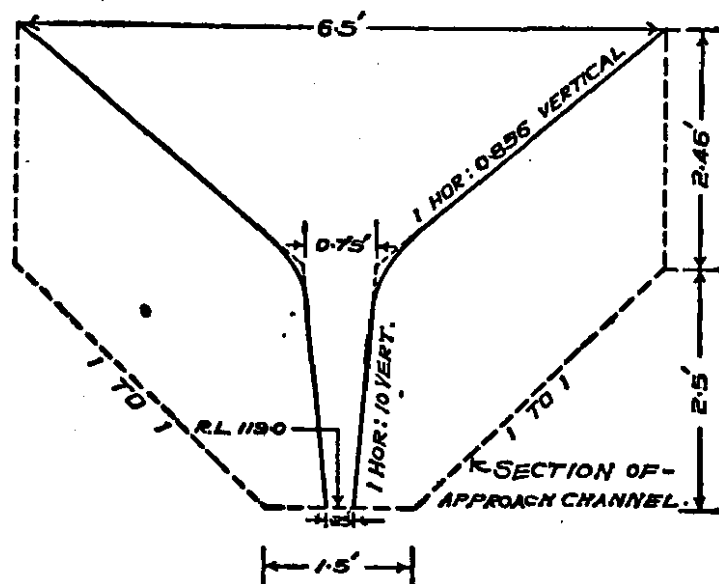


FIG:- 5 (SCALE 1" = 2')

Experiments were carried out in a geometrically similar model (Scale ratio =  $\frac{1}{2}$ ) and water levels were observed upstream and downstream for various discharges. It was found that the modular limit—that is the maximum water level downstream which just does not affect the upstream water level at any particular discharge—, was slightly lower on a falling gauge than on a rising gauge; so the true modular limit must be based on a falling gauge.

It was found that a higher downstream water level was permissible in the trapezoidal flume than in a parallel sided 1 foot flume for all discharges up to the equivalent of 28 cusecs; which is desirable, because at low supplies downstream levels are liable to cause drowning. For discharges greater than 29 cusecs, the danger of drowning is small; but it then becomes important to pass as large a discharge as possible with the limiting depth of 4.75 feet upstream; because any further rise would cause the sewer, which is 5 feet high, to flow full which would cause considerable afflux. The discharge of the trapezoidal flume with the equivalent of 4.75 feet upstream depth was 34 cusecs as against 30.8 cusecs in the parallel sided 1 foot flume and for higher discharges, the heading up in the sewer would be increasingly greater with the parallel sided flume than with the trapezoidal design. (A high upstream level above the flume would not throw a flume out of action, because there is to be an approach channel 50 feet long, with open top, and the S.W.F. discharges freely into the grit chambers; but it is nevertheless undesirable to cause flooding in the sewer upstream.)

The discharge formulæ were worked out for the prototype, after converting the model discharges and upstream depths ( $d_1$ ) to those of the prototype, by multiplying discharges by  $2^{2.5}$  ( $= 5.657$ ) and upstream depths by 2. This is permissible in this case because the Reynold's number in the model was greater than 50,000. The formulæ for discharges less/greater than 9.25 cusecs are:—

$$Q = 0.588 d_1^{2.258} \quad \text{and} \quad Q = 0.074 d_1^{3.956}$$

The data fit the curves within 1 per cent. The upper and lower portions of the discharge curve meet at  $Q = 9.25$  cusecs and  $d_1 = 3.39$  feet so the discharge curve, like the flume slope, is rounded off between  $Q = 8.7$  and 10 cusecs.

6. INVESTIGATIONS TO DETERMINE THE WEIR SECTION WHICH, WHILE DESIGNED TO PASS A MAXIMUM FLOOD, WILL STORE THE MAXIMUM POSSIBLE SUPPLY AT LAKE ARTHUR HILL (WILSON DAM) AT BHANDARDARA (BOMBAY PRESIDENCY).

The waste weir consists of a cut 750 feet wide through a rock saddle (see figure 5). The length of the cut from the upstream to downstream is 250 feet on the left bank and 650 feet on the right bank. The rocky floor of the cut is very rough and uneven, especially the left half. The upstream edge of the waste weir slopes steeply and there is an abrupt fall into the deep valley at the downstream edge, see Fig. 6.

The catchment area is 47 square miles, the length of the catchment being 9.5 miles. The annual rainfall varies from 260 inches near the upper end of the catchment to 110 inches near the dam.

The maximum flood discharge according to Inglis' fan catchment formula<sup>(6)</sup> is—

$$Q = \frac{7000A}{\sqrt{A+4}} = 44,400 \text{ cusecs}$$

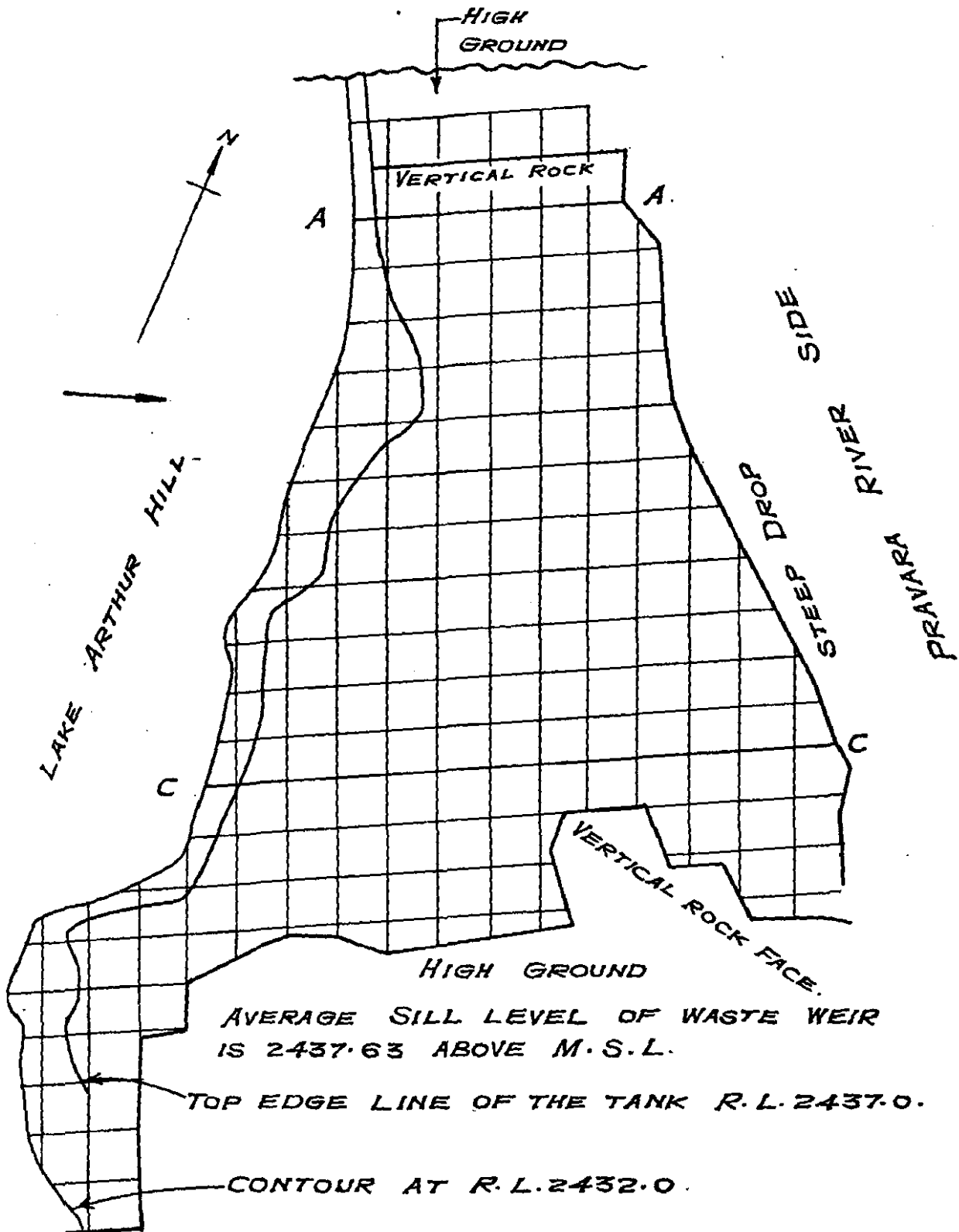
where A is the catchment area in square miles.

According to Beale's curve, the maximum flood run-off is 28,000 cusecs. The big difference between these two figures is mainly due to Beale's curve having been based on discharges of Canal culverts in the case of small catchments up to 150 square miles, so that flood absorption must have markedly reduced peak floods in his data. Pending further investigation, Beale's figure has been adopted for design, though, on account of the very great flood absorptive capacity of this Lake at full supply, the maximum possible discharge will be appreciably less. The maximum flood since the dam was completed in 1926 occurred on 28th July 1927,

FIG. 6

PLAN OF  
WASTE WEIR

SCALE = 150' = 1"





when the average depth over the waste weir was 4.25 feet. The discharge formula used hitherto for this weir was the broad crested weir formula :

$$Q = 3.1 B D^{1.5}$$

where  $Q$  = discharge in cusecs

$B$  = width of weir

$D$  = upstream depth over sill of weir

$B$  = being taken as 750 feet.

According to this formula the maximum flood discharge for  $D = 4.25$  feet on 28th July 1927 was 20,390 cusecs. Inspection showed, however, that this formula was not applicable, because the bed was rough and the slope less than that required to neutralise friction losses ; so, as a preliminary experiment, observations were carried out to ascertain the correct discharge formula.

Owing to the irregular shape of the waste weir, there was irregular, cross flow, so that a model of the whole weir was necessary. This model (I), though built to a small scale, (1/30) was 25 feet long. With this small scale model, the depth of flow was very small, compared with the plan dimensions, and so errors in friction losses would be unavoidable. Two other models were therefore constructed :

*Model II.*—A part width model (Scale ratio = 1/10) representing a width of 50 feet in the prototype.

*Model III.*—An exact replica of model II to a scale ratio = 1/30 i.e. this model was a  $\frac{1}{3}$ rd scale model of Model II.

Thus, Model I was a complete model—to take three-dimensional flow into account—. Model II was sufficiently large to reproduce conditions in the prototype correctly, Reynold's No. being 90,000, and Model III was a link between the other two. The discharge formula in model II for section AA—shown in figure 6—which was rough, but short, was found to be :

$$q = 2.23 D^{1.03} \quad \dots \quad (1)$$

and for section CC—which was smoother but longer

$$q = 2.11 D^2 \quad \dots \quad (2)$$

where  $q$  = discharge per foot in cusecs.

Combining the above two

$$q = 2.16 D^{1.95} \quad \dots \quad (3)$$

The formula for the prototype—after allowing for scale ratio was found to be

$$q = 0.77 D^{1.95} \quad \dots \quad (4)$$

Allowance had, however, to be made for excess friction in the smaller model (III) ; which was appreciable. This was done by making the bed of the smaller model smoother than according to geometrical similarity, by trial and error, until the water profiles obtained in model II were reproduced in model III ; and subsequently the bed roughness of model I was made to reproduce the roughness of model III. A discharge of 5.68 cusecs ( $\equiv 28,000$  cusecs in the prototype) was then run, for which the upstream depth over the sill of model I was found to be 0.28 feet ( $\equiv 7.14$  feet in the prototype).

$q$ —the discharge per foot run—in model II, for the corresponding depth of 7.14 feet in the prototype ( $\equiv 0.714$  feet) was 1.12 cusecs from the formula (3)

$$q = 2.16 D^{1.95}$$

This was equivalent to  $\frac{1.12}{3\sqrt{3}} = 0.216$  in model III.

The total discharge in model I was 5.18 cusecs ( $\equiv 28,000$  cusecs in the prototype). The effective width =  $5.68/0.216 = 26.3$  feet which is equivalent to 789 feet or say 790 feet in the prototype, as against a minimum width of 750 feet but this is downstream of the control section. The discharge formula at full supply for the prototype is therefore

$$Q = 0.77 \times 790 D^{1.95} = 608 D^{1.95}$$

and the discharge for the maximum observed depth of 4.25 feet, worked out to 10,270 cusecs, as against 20,390 cusecs by the original, incorrect, formula, or approximately half. Experiments to determine the best design of waste weir profile will now be taken in hand.

#### 7. EXPERIMENTS WITH A MODEL OF THE HEAD SLUICES OF THE NIRA LEFT BANK CANAL.

Experiments were carried out with a geometrically similar model (Scale ratio = 1/3) of the head sluices of the Nira Left Bank Canal (7) and it was found that the coefficient of discharge  $C_H$  in the equation

$$C_H = \frac{Q}{A\sqrt{2gH}}$$

was 0.81 with the gate fully open for the original design but 1.93 when a bellmouth had been added upstream of the sluices and an expansion constructed downstream with 1 in 10 divergences on the bed, sides and top. Since the publication of that Technical Paper (54) cases have occurred in which the above high value of  $C_H$  has been assumed for other cases, in which the mean velocity through the barrel, length of barrel, relative roughness of barrel and expanding flume, were different, and also in cases in which the downstream water level was below the modular limit. In order to clarify this question and make it easy to calculate the correct value of  $C_H$  for any conditions, experiments were carried out with the original model of the Nira Left Bank Canal Head Regulator and with another model in which the barrel was made 4 times as long—*vide* Fig. 7.

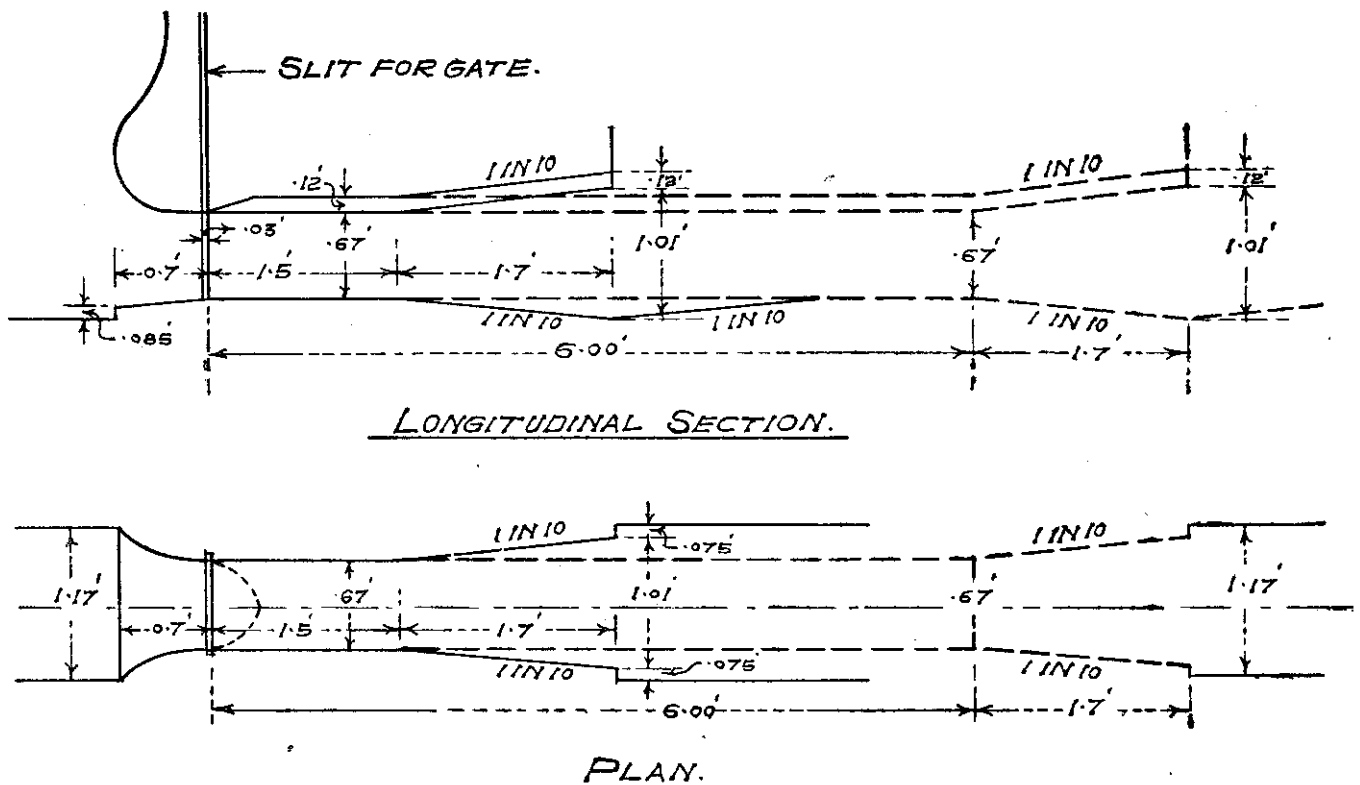


FIG: 7, SHOWING PLAN AND L. SECTION OF THE -  
MODEL OF THE NIRA HEAD REGULATOR.  
(BOTH SHORT AND LONG.)

SCALE 6"=1 FEET.

NOTE: EXTENSION OF THE MODEL IS SHOWN IN RED.

It was assumed that the total loss of head consisted of :—

$$H = h_g + h_{f_1} + h_{f_2} + h_e \quad \dots \quad \dots \quad \dots \quad (A)$$

where  $h_g$  = loss of head at entrance to bellmouth to beyond the gate slot with gate fully open.

$$= C_g \frac{\bar{V}_g^2}{2g} \text{ where } \bar{V}_g = \text{mean velocity through gate opening.}$$

$$h_{f_1} = \text{loss of head in the barrel.}$$

$$h_{f_2} = \text{loss of head in the expanding flume.}$$

$$h_e = \text{loss of head at exit, downstream of expanding flume.}$$

*Loss of head at expansion* was found to be approximately equal to

$$\frac{(\bar{V}_e - \bar{V}_3)^2}{2g}$$

where  $\bar{V}_e$  = mean velocity at the end of expanding flume—with the expanding flume submerged and natural expanding flow and no jet flow.

$\bar{V}_3$  = mean velocity in the downstream channel at a section where full recovery of head has been effected.

*Loss at bellmouth, in barrel and expansion :*

$h_{f_1}$  and  $h_{f_2}$  were calculated both by Manning's formula

$$\bar{V} = \frac{1.486}{N}$$

$$R^{2/3} (h_f/L)^{1/2}$$

and by Lacey's formula

$$\bar{V} = \frac{1.346}{N_a} \cdot R^{1/2} (h_f/L)^{1/2}$$

Where R and L = hydraulic mean depth and length of barrel and expanding flume.

Having worked out  $h_e$  — the loss at exit—

$$h_g + h_{f_1} + h_{f_2} = H - h_e$$

$$C_g \text{ in } h_g = C_g \frac{\bar{V}_g^2}{2g}$$

was then calculated, assuming various values of N and the value of N which gave the most constant value of  $C_g$  was considered to be correct and the corresponding values of the rugosity coefficient of the model were

$$N \text{ (Manning)} = 0.015$$

The bed and sides of the sluices in the models were "smooth cement" plastered and  $C_g$  was found to be 0.085 and the loss at entrance was then

$$h_g = 0.085 \frac{\bar{V}_g^2}{2g}$$

It may be noted that the method outlined above is not exact but a close approximation. The general formula thus became, using Manning :—

$$H = h_g + h_{f_1} + h_{f_2} + h_e \quad \dots \quad \dots \quad \dots \quad (A)$$

$$= 0.085 \frac{\bar{V}_g^2}{2g} + \frac{N^2}{2.21} \left( \frac{\bar{V}_1^2 L_1}{R_1^{4/3}} + \frac{\bar{V}_2^2 L_2}{R_2^{4/3}} \right) + \frac{(\bar{V}_e - \bar{V}_3)^2}{2g} \quad \dots \quad \dots \quad \dots \quad (B)$$

This formula assumes natural expanding flow in the expanding flume and a downstream water level not lower than the modular limit. If the downstream water level is below the modular limit, which implies standing wave conditions, the expansion no longer recovers head and energy is dissipated in the standing wave:

Two cases will now be cited to exemplify the above points:—

*Case I.*—The Head Regulator of the Godavari Right Bank Canal has a barrel which is only 2 feet long and the 1 in 10 expanding flume is 10 feet long. For  $Q=347$  cusecs, the calculated loss of head according to formula (B) was 0.60 feet; but as the downstream water level was 1.07 feet lower than the modular limit, the calculated loss plus 1.07 feet loss in wave, worked out to 1.67 feet, against 1.70 feet observed, and if the downstream water level were raised by 1.07 feet or the whole structure lowered by 1.07 feet,  $C_H$  would become 1.82.

*Case II.*—The Head Regulator of Distributary 6, of Pravara Left Bank Canal, had a barrel 23.5 feet long and a 1 in 10 expanding flume, with horizontal top 10 feet long.  $N$  in Manning's formula was determined directly from observations with different discharges and was found to be 0.0207—a high value, mainly due to the barrel of this regulator consisting of rough masonry. The mean value of  $C_H$  worked out to 0.75, which was increased to 0.90 when the top of exit was just submerged. About 86 per cent. of the head lost in this sluice was due to high friction loss in the long, rough, barrel; and this could have been reduced to half by cement plastering the barrel which would have increased  $C_H$  from 0.90 to 1.20 and the discharge by 33 per cent.

The statement below shows the comparative values of  $C_H$  in the three cases:—

Head Regulator of	Discharge (Prototype) in cusecs.	$C_H$ free flow downstream.	$C_H$ (top slab at exit submerged).
Nira Left Bank Canal	720	.....	1.93
Godavari Right Bank Canal	347	1.11	1.82
Distributary 6 of Pravara Left Bank Canal	32	0.75	0.90

From this it will be seen that there is a very great discrepancy in values. This question is so tricky that it is always advisable to refer the matter for opinion, or where necessary, further experiments.

Further experiments are in progress.

#### 8. TRAINING THE RIVER SARDA ABOVE THE BARRAGE AT BANBASSA, UNITED PROVINCES.

Opposite the Burma Spur the main channel bifurcates, the smaller branch, which carries about 25 per cent. of the discharge, flows along the south margin of the Kala Jala Island towards the Head of the Sarda Canal and is known as the Western channel.

Since 1929 the whole of the attack has been borne by the Nepal Spur, the Burma Spur being out of action.

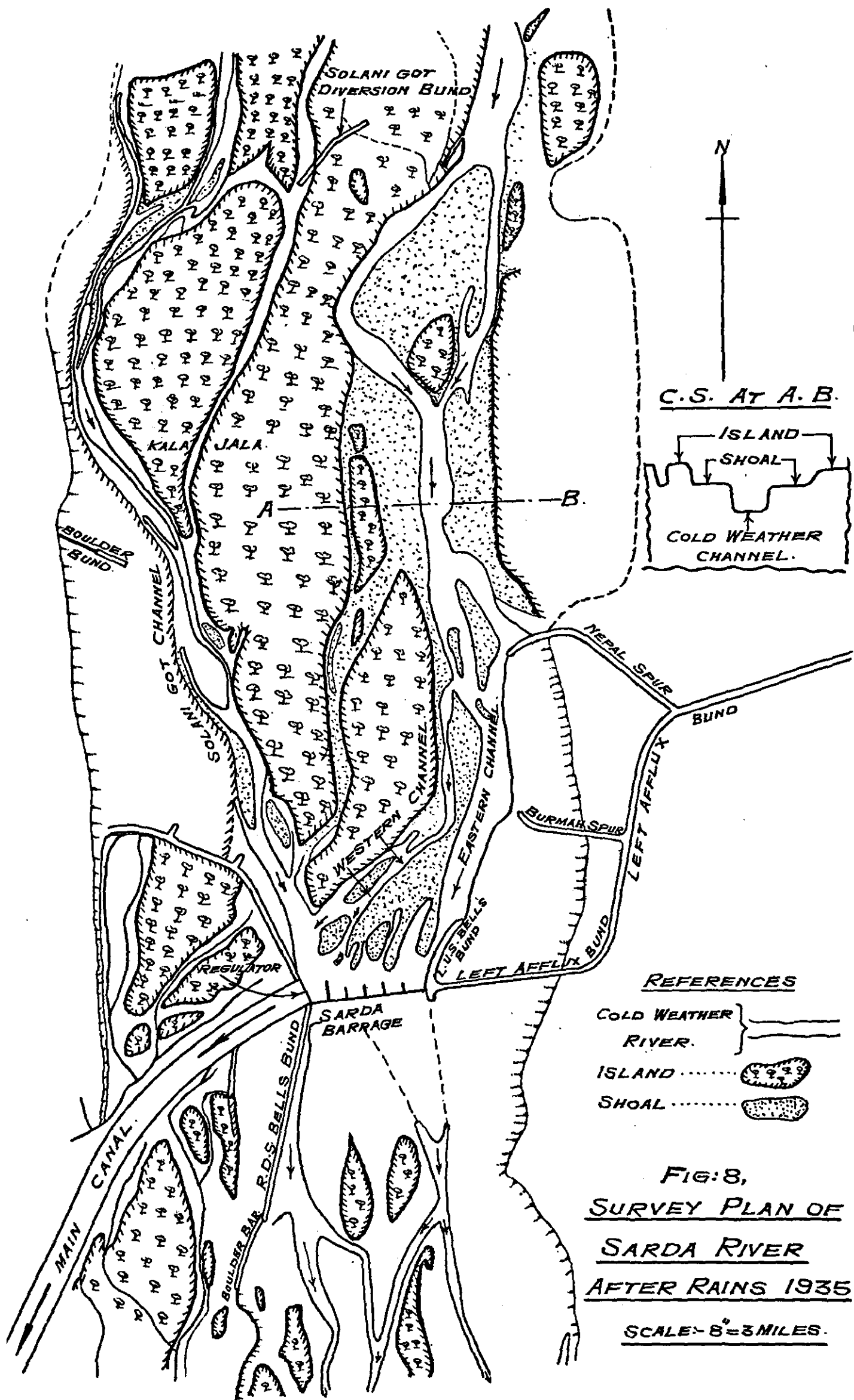
Though the Nepal Spur has successfully checked the attack on the Left Afflux Bund, the main stream has continued to follow the outside of the bend—from the nose of the Nepal Spur, past the Burma Spur to the nose of the L. U. S. Bell's Bund—whence it swings sharply towards the Head of the Sarda Canal, across the face of the Barrage, creating dangerous conditions.

Figure 8 shows conditions in the Sarda River in 1935. Model experiments were started to test various remedies which had been suggested to reduce cross flow during floods and also to provide an unfailing direct supply from the Western channel to the Sarda Canal. Experiments were carried out in a vertically exaggerated model.

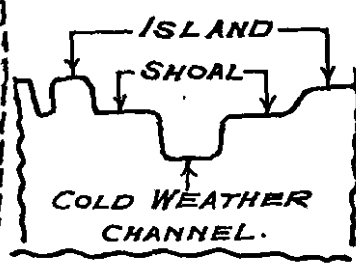
In experiment 1, the flow conditions of 1936, described above, were reproduced in the model.

In experiments 2-4, 6 and 8, the Burma Spur was extended in various ways, with the object of deflecting water into the Western channel.

The Burma Spur—in use in Burma—consists of a curved head, shaped like the head of a hockey stick, pointing upstream, the idea being to create pond conditions



C.S. AT A.B.



REFERENCES

- COLD WEATHER RIVER.
- ISLAND .....
- SHOAL .....

**FIG: 8,**  
SURVEY PLAN OF  
SARDA RIVER  
AFTER RAINS 1935  
SCALE: 8" = 3 MILES.

upstream, so that attack is concentrated at its nose. As a consequence the stem of the spur can be constructed relatively cheaply. It may be said that experiments now in progress "to investigate the relative merits of various types of spurs" show that this design has great merits.

In experiments 2 and 3, the Burma Spur was extended 450 feet and 600 feet respectively, at an angle  $10^\circ$  upstream of the present alignment.

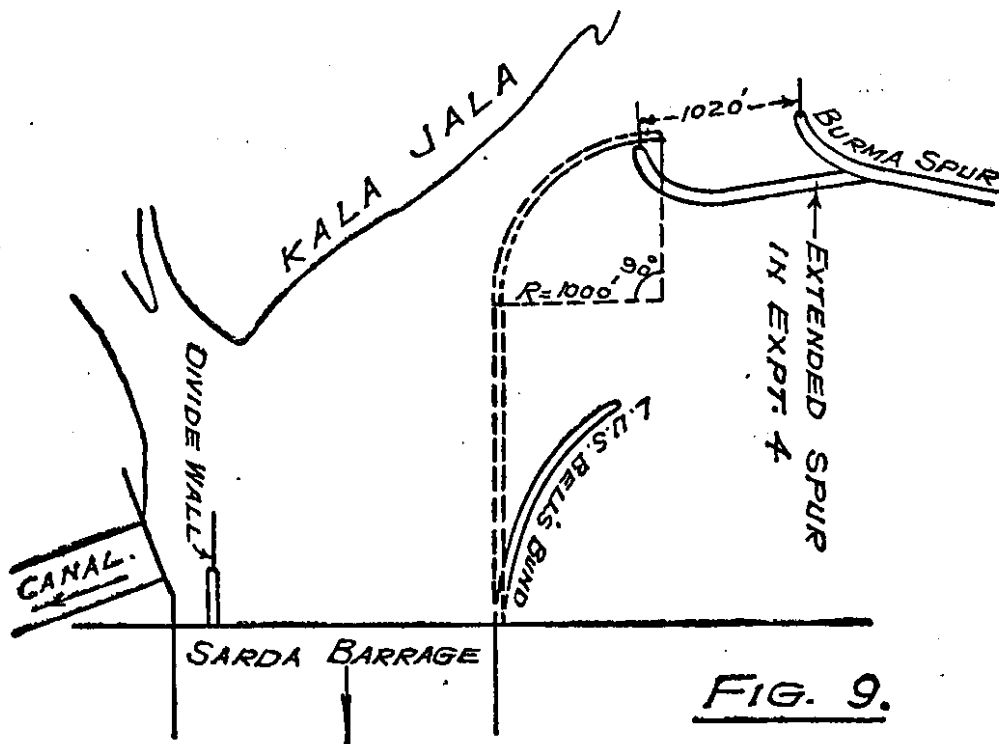
In experiment 6, the Burma Spur was extended 600 feet along the present alignment.

In experiment 8, the spur was extended 930 feet on a line  $10^\circ$  downstream of the present alignment.

Results in experiments 2, 3, 6 and 8 were unsatisfactory, because heavy scour occurred at the nose of the extended spur in each case, and the high velocity surface flow was directed towards the L. U. S. Bund, while the bed flow upstream of the Burma Spur swung across to midstream and flowed over the top of shoals upstream of the Barrage, and the flow through the Western channel was slack—as at present—and tended to decrease.

Experiment 7 was the same as experiment 6 except that there was no curved head but instead, a straight, submerged, bund 450 feet long was built out in continuation of the high level bund, 750 feet long. A standing wave formed downstream and severe action occurred around the outer end of the submerged bund. Sand was then injected, which caused a shoal to form in the main channel, upstream of the submerged bund, which caused the bed flow to swing over to the L. U. S. Bund and the high velocity top water flowed over the top of the shoals near the left bank upstream of the Barrage. The flow along the edge of the Kala Jala and over the top of shoals near the Divide wall was, however, slack. With higher discharges, the high velocity top water tended to move back to the L. U. S. Bund. It was found that in the event of the submerged bund being breached, present flow conditions would be quickly re-established.

*Experiment 4—vide photo 1436.*—Spur stem extended 1,020 feet at an angle pointing  $20^\circ$  downstream of the present alignment ending in a Burma head—see figure 9. This design gave satisfactory results, because it induced the desired curvature of flow and deflected the high velocity, surface flow towards midstream; the bed water, carrying boulders, flowing towards the left flank. This alignment gave the best results, because it created conditions similar to those which would follow were the Left Bell Bund of the standard design (i.e. at right angles to the Barrage for a length equal to length of Barrage = 1,964 feet finished off by a curved head of 1,000 feet radius—*vide* Fig. 9).



The conditions of experiment 4 were further tested with discharges gradually rising from 2.4 cusecs—with which the experiments described above were carried

out—to 4·5 cusecs, and decreasing to 0·5 cusecs ( $\equiv$  40,000 cusecs). As the flood rose, the main flow was over the shoal opposite the Nepal Spur. Downstream of the Burma Spur, the main surface flow was away from the L. U. S. Bund and a shoal tended to form between the extended spur and L. U. S. Bund which, on a falling flood, increased considerably, so that a suitably curved channel began to form on the alignment of the Western Channel.

This experiment showed that conditions on a falling flood are highly critical.

Further experiments with a larger model will shortly be started.

9. EXPERIMENTS WITH LARGE SCALE MODEL OF THE GANGES AT THE HARDINGE BRIDGE (HORIZONTAL SCALE RATIO= $1/500$ ) ON EFFECT OF LENGTH OF GUIDE BANKS ON FLOW (FOR THE RAILWAY BOARD).

The Hardinge Bridge Committee at their meeting in November 1935 decided that the Guide Banks of the Hardinge Bridge, which are now 2,850 feet long= $0\cdot53$  times the length of the Bridge, should ultimately be extended to 5,385 feet—the length of the Bridge.

Extension of the Left Guide Bank to this extent would, however, involve the demolition of certain bungalows in Paksey Colony upstream of the Bridge—*vide* figure 10.

We were, therefore, asked to carry out experiments with the Left Guide Bank 5,385 ; 5,885 and 6,385 feet long, to ascertain, whether, in the event of an embayment forming above the Left Guide Bank head, the main current might swing across the River and impinge on the Right Guide Bank upstream of the Bridge and so cause dangerous scour near the Bridge.

The Experiments were done with both Right and Left Guide Banks extended equally, Damukdia Guide Bank being removed (which the Committee decided, should be done as soon as silting occurs between the Damukdia Guide Bank and the head of the Right Guide Bank).

In Series XIIA, the length of the Guide Banks upstream of the Bridge .. .. = length of Bridge  
 $\equiv$  5,385 feet.

In Series XIIB, the length of the Guide Banks upstream of the Bridge .. ..  $\equiv$  6,385 feet.

There were two experiments in each series:—

**EXPERIMENT 1**,—Conditions of flow as in 1934, in which the main flow was along the Right Guide Bank at the Bridge—see figure 10. In order to get an idea of the limits of erosion of the Sara Protection Bank under present conditions, the model was laid according to the results obtained in Series X, in which the effect of removing the stone pitching from the bank was tested—as described in last year's Annual Report ; see also Bombay Public Works Department Technical Paper 55<sup>(1)</sup>. In order to admit rapid results, the whole of the left bank from the Sara Protection Bank to the head of the extended Left Guide Bank—which in the prototype, is composed of alternating layers of silt and clay—, was constructed of silt.

**EXPERIMENT 2** was carried out with the River flowing as in 1918, i.e., the main current was along the old Damukdia channel and along the Left Guide Bank at the Bridge—*vide* figure 10. Little difference in conditions existed whether the Guide Banks were 5,385 or 6,385 feet long.

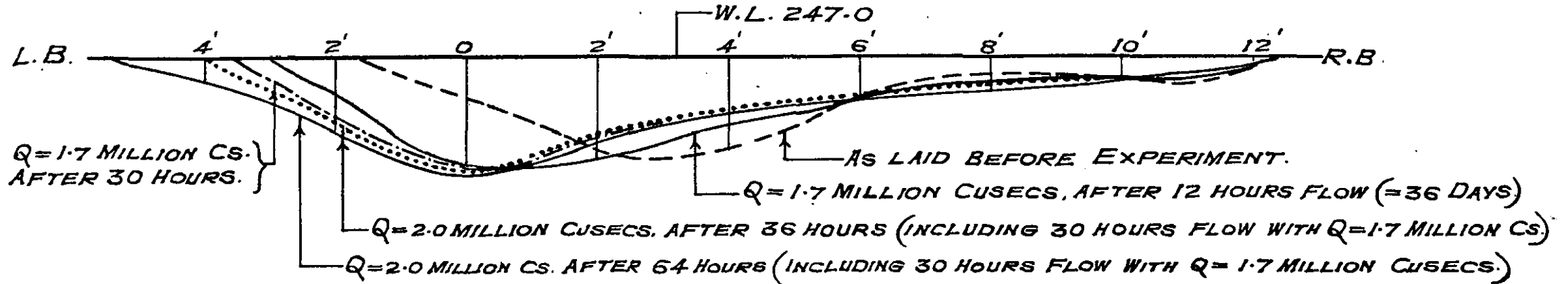
Experiment 1 was carried out with discharge = 10·1 cusecs ( $\equiv$  2 million cusecs in the prototype) for 48 hours ( $\equiv$  144 days). While the Sara Protection Bank was scouring, the main surface flow shifted towards midstream, passing through the middle of the Bridge and only a slight embayment occurred behind the head of the Left Guide Bank, indicating that when the River follows this course, action is not severe.





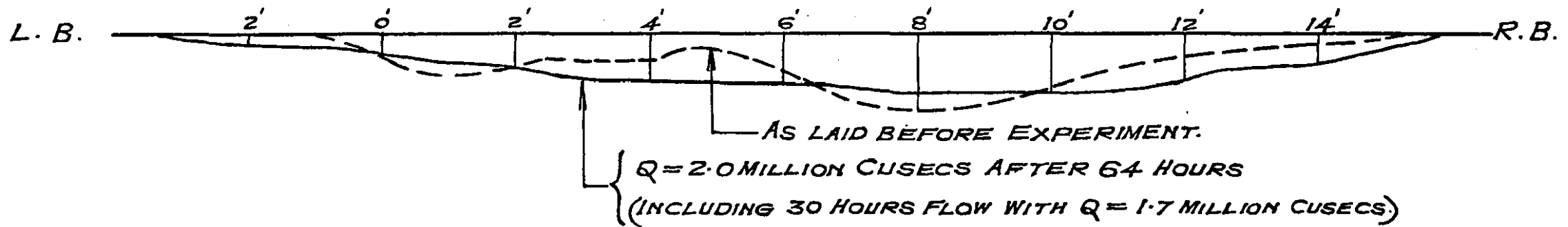


FIG: II,



C.S. 4C - 1200 FT. U/S OF HEAD OF LEFT GUIDE BANK.

SCALE:- 2' = 1"



C.S. 5A - 6800 FT. U/S OF HEAD OF LEFT GUIDE BANK.

*Experiment 2 with 1918 course of the River* was carried out with a discharge of 8.59 cusecs

( $\equiv$  1.7 million cusecs) for 30 hours

( $\equiv$  90 days) and thereafter with 10.1 cusecs

( $\equiv$  2.0 million cusecs) for 34 hours ( $\equiv$  102 days).

Upstream of the head of the Left Guide Bank, there was return flow with eddies. The main surface flow was between the middle of the Bridge and the Left Guide Bank, and the flow along the Right Guide Bank was slack. The embayment made by the River is shown in Fig. 10.

Figure 11 shows cross sections 4-C—1,200 feet upstream of the head of the Left Guide Bank, showing progressive scour of the embayment and 5-A—6,800 feet upstream of the head of the Left Guide Bank showing scour at the end of the experiment. It will be seen from C.S. 4-C that the point of maximum depth of scour did not change appreciably after 12 hours flow ( $\equiv$  36 days) with  $Q \equiv$  1.7 million cusecs, though the embayment at water level increased. Scour near the top of the bank was mainly due to wave-wash and heaving of the water surface in the embayment; this would be greatly reduced in the prototype by bamboos and vegetation. It was therefore assumed that the limit of scour at bed level had been approximately reached after 64 hours ( $\equiv$  192 days) with  $Q \equiv$  2.0 million cusecs (including 30 hours flow with  $Q \equiv$  1.7 million cusecs), and the experiment was discontinued.

By the end of the experiment, the left bank had scoured from Sara down to the equivalent of 850 feet downstream of the nose of the extended Left Guide Bank and to a width of 1,600 feet behind the nose. The equivalent depth of scour at the nose was 117 feet (*i.e.* to R.L. 130).

*Design of the head of the Guide Banks.*—In the above experiments, the design of the head of the guide banks was the same as in the prototype *viz.*, the radius of curvature was equivalent to 950 feet with a  $60^\circ$  arc. Series XIII was then carried out, in which the effect of extending the head to  $120^\circ$ —as recommended by Sir Francis Spring<sup>(6)</sup>—was tested.

[*Note.*—In this series, an artificial embayment equivalent to 1.1 miles wide was made upstream of the Left Guide Bank; but as the River did not follow this artificial embayment, a deep channel was artificially cut along its face and the channel of the previous experiment was filled in with silt.

Such conditions are entirely unnatural and could not occur unless free embayment were interfered with by constructing an artificial guide bank in an unsuitable position. Such extreme conditions were imposed in the model in order to accentuate damage and hence exaggerate the difference in protective value of alternative designs of head.]

Experiments were carried out with the curved head subtending angles of  $60^\circ$  and  $120^\circ$  with a discharge equivalent to 2 million cusecs for 18 hours ( $\equiv$  54 days).

The results showed that the  $120^\circ$  curved head was superior to the  $60^\circ$ , because the “throw-off” of the current towards the Right Guide Bank was less violent; the velocity gradients near the head were less steep; and scour in the embayment, opposite the head, was less severe.

The conclusion is that Sir Francis Spring’s design should be adopted.

#### 10. EXPERIMENTS WITH (a) GUIDE BANKS AND (b) FALLING APRONS (FOR THE RAILWAY BOARD).

(a) *Guide Banks.*—In last year’s Annual Note, it was stated at the end of paragraph 3 on the experiments with falling aprons:—

“The slip which occurred on 26th September 1933 in the Right Guide Bank of the Hardinge Bridge was reproduced in a model to a horizontal scale

ratio of 1/200. This experiment, which was only qualitative, showed how slips occur due to attack at the toe of an embankment. Experiments are in progress to show the behaviour of aprons under severe complex conditions."

The following factors were investigated under axial flow and cross flow conditions :—

- (1) Inclination of slope.
- (2) Point velocity, magnitude and direction.
- (3) Curvature of flow.
- (4) Velocity gradient normal to bank.
- (5) Turbulence.
- (6) Size, shape and density of stones.

Investigations (2), (4) and (6) are still incomplete. It was found that even with curved flow a slip did not occur if the face of the slope were flatter than 1·6 horizontal to 1 vertical; and when an apron, laid horizontally, launched, the slope taken up by the stones was normally 2 to 1 in the case of axial flow and about 2·6 to 1 in curved flow. These slopes are slightly steeper than the prototype, which takes up slopes of 2½ to 3 to 1 according to whether flow is axial or curved.

A glass panel was fixed at the place where the slip occurred; and the movement of silt and stones was observed; and it was seen that there was return flow near the surface and forward flow near the toe of the bank. Near mid-depth, flow was unstable and silt sometimes remained stationary on the face of the glass; while at others it was driven about in eddies—just like dust on a road when thrown up in whirls by gusts of wind. The fact that sand settled on the glass showed that action was not severe down to ¾th depth. In marked contrast with this action was very severe near the toe, and the stream lines were contorted and swung in all directions; while the velocity was changing abruptly. These observations showed that attack occurred only near the toe of the bank, where stones were carried away in surges of rapidly changing energy, with steep velocity gradient. Each surge was followed by a relatively still period; the frequency of surges in the model being about 1 surge every two seconds, and the still period between successive surges varying between 2 and 5 seconds.

The severity of attack was tested by means of cement cubes of uniform specific gravity. The size of the smallest cube which remained at the toe was taken as a measure of the severity of attack.

The following River conditions were tested—with and without cross flow :—

- (1) Current directed at the head of the Right Guide Bank.
- (2) Flow directed towards the head of the guide bank in a jet.
- (3) Straight parallel flow down the channel.
- (4) Flow directed towards the part of the Right Guide Bank where the slip occurred on the night of 26th September 1933 (which later developed into a breach).
- (5) Flow directed into the embayment behind the head.

The attack was always found to be more severe with curved and diving flow than with axial flow. The most severe attack was found in case (4), the next worst being in No. 2.

(b) *Falling Aprons.*—(1) It was found that better protection was afforded to the bank by a mixture of stones of different sizes than by stones of uniform size.

(2) The effect of hydraulic gradient through the guide bank, due to the water level behind the bank being higher than River level in front, was tested. It was found that the bank did not collapse until the gradient exceeded 3·33 horizontal to 1 vertical; and this slope was independent of the thickness of pitching on the

face of the bank. This result tallies with canal experience. From this, it is clear that the slips which occur when the water level behind the guide bank is higher than in the River are due to saturation of the bank causing slips when the River level falls. This point will now be investigated.

Further experiments are in progress with various sizes of stones to correlate the stone moving velocity and the velocity gradient with

- (1) the angle of the apron slope ; and
- (2) the angle which the apron makes with the stream.

#### 11. EXPERIMENTS TO ASCERTAIN THE BEST DESIGNS FOR DISSIPATING ENERGY DOWNSTREAM OF FALLS.

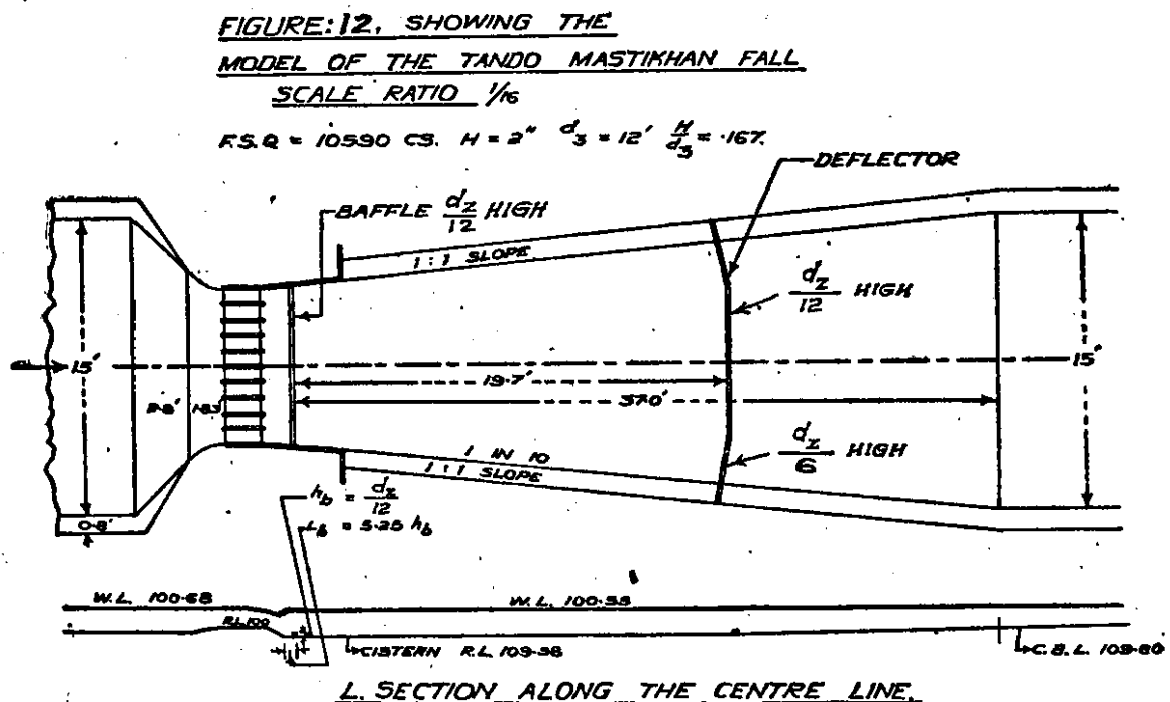
These experiments were commenced in 1934 in accordance with resolution 27 passed at the 4th Annual Meeting of the Central Board of Irrigation in November 1933.

It had been hoped to submit a final report on the engineering aspects of this problem in time for the meeting of the Research Officers on 21st-24th June 1937 ; but as this is not possible, the present report will merely deal with the results of last year's experiments and such general conclusions as are indisputable.

*Experiments carried out during 1936-37 with (1) Proportional Standing Wave Flume Meter Fall with 1 in 10 side divergences and  $H/d_3 = 2'/12' = 0.17$  and  $D_3/D_1 = 0.80$ . The fall was practically drowned, preventing an efficient dissipation of energy by means of a standing wave. Under such conditions, the baffle ceases to act in its role of dissipating excess energy ; but is still required in its secondary role of a deflector—to dissipate the high velocity at the bed and redistribute the velocity isotachs to approximate to natural flow in the channel downstream. Experiments were, therefore, carried out to compare downstream conditions with and without baffle.*

*Experiment 1.*—Standard cistern, bed deflector =  $d_x/12$  high and side slope deflectors  $d_x/6$  high, but *without baffle*,  $d_x$  being the depth in cistern.

*Experiment 2.*—Same as (1) but with *Baffle*  $d_x/12$  high—*vide* Fig. 12.



Though water followed the expansion in experiment 1—without a baffle—the velocities near the banks, especially near the right bank, were considerably lower than normal, and the flow was slightly towards one side, with slightly uneven scour.

In experiment 2, with a baffle,  $d_s/12$  high, the distribution of velocities agreed closely with the normal distribution.

*Experiment 2 (a).*—Same as 2, except that there was 12 per cent. retrogression in water levels, but without lowering the downstream bed—*i.e.* the downstream depth was reduced from 12 feet to 10·56 feet,  $H = 3·44$  feet and  $d_s = 10·56$  feet and  $H/d_s = 0·32$ .

Such extreme conditions could not occur unless the downstream material were inerodible—when it would not matter. The test was therefore unduly severe.

*Experiment 3.*—Same as 2 (a), but with vanes spaced from sides to midstream 1·26 R; 1·09 R; 0·97 R; 0·88 R and 0·8 R (*i.e.* the alignment found best with 1 in 5 divergences and  $H = 8·72$  feet and 4·5 feet); baffle  $d_s/12$  high; bed and side slope deflectors.

*Experiment 4.*—Same as 2 (a), but with three rows of arrows and two rows of control blocks, both  $d_s/5$  high; and bed and side slope deflectors added to improve results.

With a baffle and deflectors but without vanes in experiment 2 (a), the isotachs were distorted and the flow was one sided, resulting in slightly uneven scour. When vanes were added in continuation of piers, results improved markedly, *viz.* the isotachs closely approximated to normal, the flow was sweet, and scour more natural.

In experiment 4, with arrows and control blocks, there was a jet of high velocity in midstream and low velocities near banks, velocities near the left bank being lower than those near the right bank.

It may, however, be said that—except in the case of notched falls—scour was not serious below any 2 feet fall.

#### *Types of Falls tested up to date.*

Experiments have been carried out with several well-known types of falls:—

(1) Proportional standing wave flume meter fall with sides downstream of the fall diverging at—

(a) 1 in 10—with glacis, baffle, cistern with bed of bowed cross section—as in Mr. Montagu's design—, and bed and side deflectors;

(b) 1 in 5 with vanes, glacis, baffle, bowed cistern bed and deflector—as at Tando Mastikhan Fall, Rohri Canal, Sind.

(2) Proportional standing wave flume meter fall with curved divergences downstream; friction blocks, and bed of cistern of bowed cross section—design prepared by Mr. A. M. R. Montagu.

(3) Weir falls—free overfall type—.

(4) Weir falls with glacis, cistern of arrows and control blocks.

(5) Weir falls with glacis, baffle, bowed cistern and deflector.

(6) Notched falls.

#### *General Conclusions. Relative Merits.*

(6) Notched falls (Punjab Type) were more costly than any other design and also gave inferior results, so may be finally discarded. (2) The flume fall with friction blocks, according to a design prepared by Mr. Montagu, was about 7 per cent. more expensive than the standard flume fall design, gave inferior results under

all conditions and was definitely unsuitable where retrogression existed. The cost of the remaining four types worked out as under :—

Table showing cost of falls (in rupees).

Height of fall in feet.	(3) Weir falls—free overfall type.	(4) Weir falls with arrows and control blocks.	(5) Weir falls with baffles and deflectors.	1 (a) Standard proportional S.W.F.M. Fall.
<i>Falls combined with Regulators—</i>				
8·7 .. .. .	2,25,000	2,37,000	2,31,000	1,67,000
4·5 .. .. .	2,03,000	2,11,000	2,07,000	1,58,000
2·0 .. .. .	1,88,000	1,91,000	1,88,000	1,48,000
<i>Falls combined with Bridge—</i>				
8·7 .. .. .	1,20,000	1,32,000	1,26,000	1,17,000
4·5 .. .. .	98,000	1,08,000	1,02,000	1,06,000
2·0 .. .. .	81,000	86,000	83,000	98,000
<i>Plain Falls—</i>				
8·7 .. .. .	99,000	1,11,000	1,05,000	1,10,000
4·5 .. .. .	82,000	90,000	86,000	99,000
2·0 .. .. .	67,000	72,000	69,000	92,000

The following facts are definite :—

(i) Where a fall is combined with a regulator the cost of a standard flume meter fall is much less than

- (a) a weir or
- (b) a weir combined with arrows or
- (c) a weir combined with baffle and deflectors.

Other advantages are :—

Where several distributaries take off, it is pre-eminently suitable, giving a neat layout, and preventing excessive silt draw by off-taking channels<sup>(9)</sup>.

Regulation is by screw gates, which makes it possible to reduce regulator staff to a minimum.

(ii) The weir design is definitely unsuitable for large discharges ( $q$  greater than 30 cusecs) where  $H/d_s$  exceeds about  $1/3$ , so the standard flume fall, which gives the best conditions downstream, is best even for falls without regulators or bridges where  $H/d_s$  is greater than  $1/3$ .

(iii) For falls where  $H/d_s$  is less than  $1/3$ , the question is not so much whether a particular design is good, as to whether it is "good enough". Where it is not considered important to have proportionality or a highly accurate measuring device, and where considerable wave-wash is not considered to be objectionable, the extra cost of the standard flume fall may not be justified. The choice would then lie between a plain weir or a weir with downstream glacis slope and

- (a) arrows and control blocks or
- (b) baffle and deflectors.

These experiments have shown that the latter are cheaper and better. The only point, therefore, for decision is whether a plain weir or one with glacis, baffle and deflector, costing an additional 3 per cent. to 5 per cent. according to whether  $H/d_s$  is 0·17 or 0·38, is better.

The addition of a baffle and deflectors reduces scour downstream under all conditions; but is particularly valuable should retrogression occur; because the baffle causes a standing wave to form at the baffle under all conditions, and so the cost of maintenance is considerably lower in this baffle design than in a plain weir fall.

## 12. EXPERIMENTS WITH SILT ABRADER.

Despite the fact that silt becomes progressively finer from the hills to the sea, there has been a widespread opinion that this phenomenon could not be explained by abrasion, because the time taken for silt, carried by water, to reach the sea was too short to explain the facts.

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

The silt abrader consists of a circular tank of 2 feet radius, 4 feet high, with a central shaft, to which three tiers of blades are fixed, each tier being set at 40° to the other two—*vide* Fig. 13—. The shaft was revolved 30 times per minute, so the velocity of the blades at their outer edge was 6·9 f/s.

Sukkur sand was placed in the tank to a depth of 6" and water maintained at a constant level, 3' 6" over the base of the tank. The abrader has worked for 838 hours (=35 days) since last year's report, the total working hours to date being 6,220 hours (=259 days). Fig. 13 shows the size distribution curve of original sand and sand after 6,220 hours, particles less than 0·2 mm. being measured by sedimentation and coarser particles by sieves.

(1) The percentage fraction of coarse sand (0·2 to 1·0 mm.) obtained has decreased from 90 per cent. at the commencement of the experiment to 56 per cent. after 6,220 hours and the percentage of fine sand (0·2 to 0·02 mm.) has increased from 9 per cent. to 42 per cent.

(2) The rate of decrease of sand of "0·33 to 0·5 mm." was greater than that of sand varying from "0·2 to 0·33 mm."

(3) The rate of increase of sand of "0·075 to 0·1 mm." was greatest, the next in order being sand of "0·04 to 0·075 mm."

(4) The amount of sand "0·02 to 0·04 mm." did not increase appreciably, indicating that fine sand in suspension becomes finer at a decreased rate.

(5) The Lacey silt factor—based on particles greater than 0·075 mm. in diameter—has changed from 0·894 (original sand) to

0·871 after 3,775 hours or 157 days,

0·834 after 5,382 hours or 224 days,

0·819 after 6,220 hours or 259 days.

( $f=9·4 \sqrt{d}$ , where  $d$  is the diameter in inches. The constant "9·4" is more approximately correct for Indian data than "8·0" which was based to a large extent on European data and low temperatures.)

*Note.*—There is no indication as to the relative rate of abrasion of various sizes; because this is marked by increase of finer grades from all coarser grades—not from a particular grade. This question will be tested later by abrading a single size of sand at a time.

The experiment is being continued.

## 13. FLOOD RUN-OFF FROM CATCHMENTS.

In 1929 Technical Paper 30, Bombay Public Works' Department, entitled "A critical study of run-off and floods of catchments of the Bombay Presidency with a short note on losses from lakes by evaporation" was published.

In that Paper two formulæ were derived for

(a) fan catchments (of general application),

(b) valley catchments (for particular conditions frequently found in the Bombay Presidency).

$$\text{Discharge} = 7,000\sqrt{A} \text{ or } q = 7,000/\sqrt{A}$$

where  $A$ =area of fan catchment in square miles,

$Q$ =maximum flood run-off from catchment,

and  $q$ =maximum flood run-off per square mile of catchment.

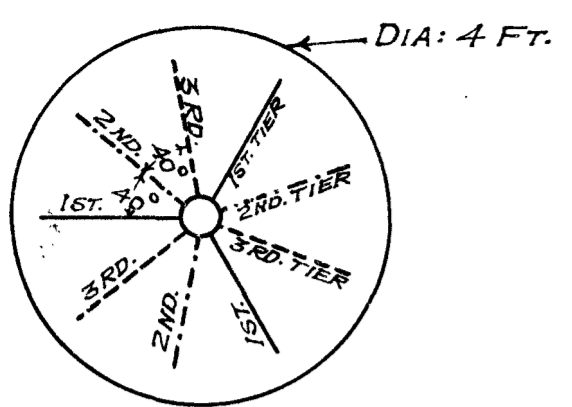
# IRRIGN: DEVPT: & RESEARCH CIRCLE.

## SILT ABRADER

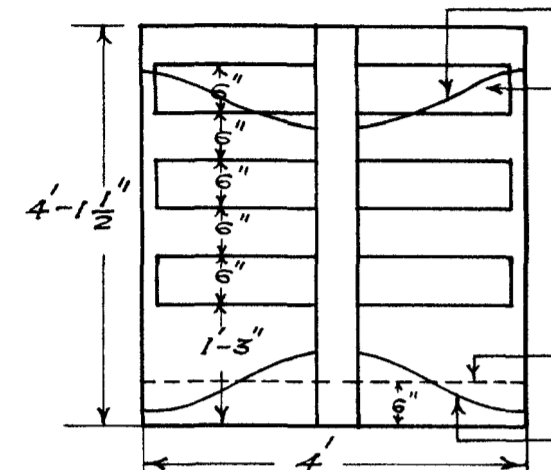
(4 in diameter and 4-1 1/2" high)

Details of paddles consisting of 3 tiers of 3 blades each.

Each tier is set 40° different to the other two.



PLAN OF ABRADER



WATER SURFACE WHEN-  
ABRADER WAS WORKING.

BLADE.

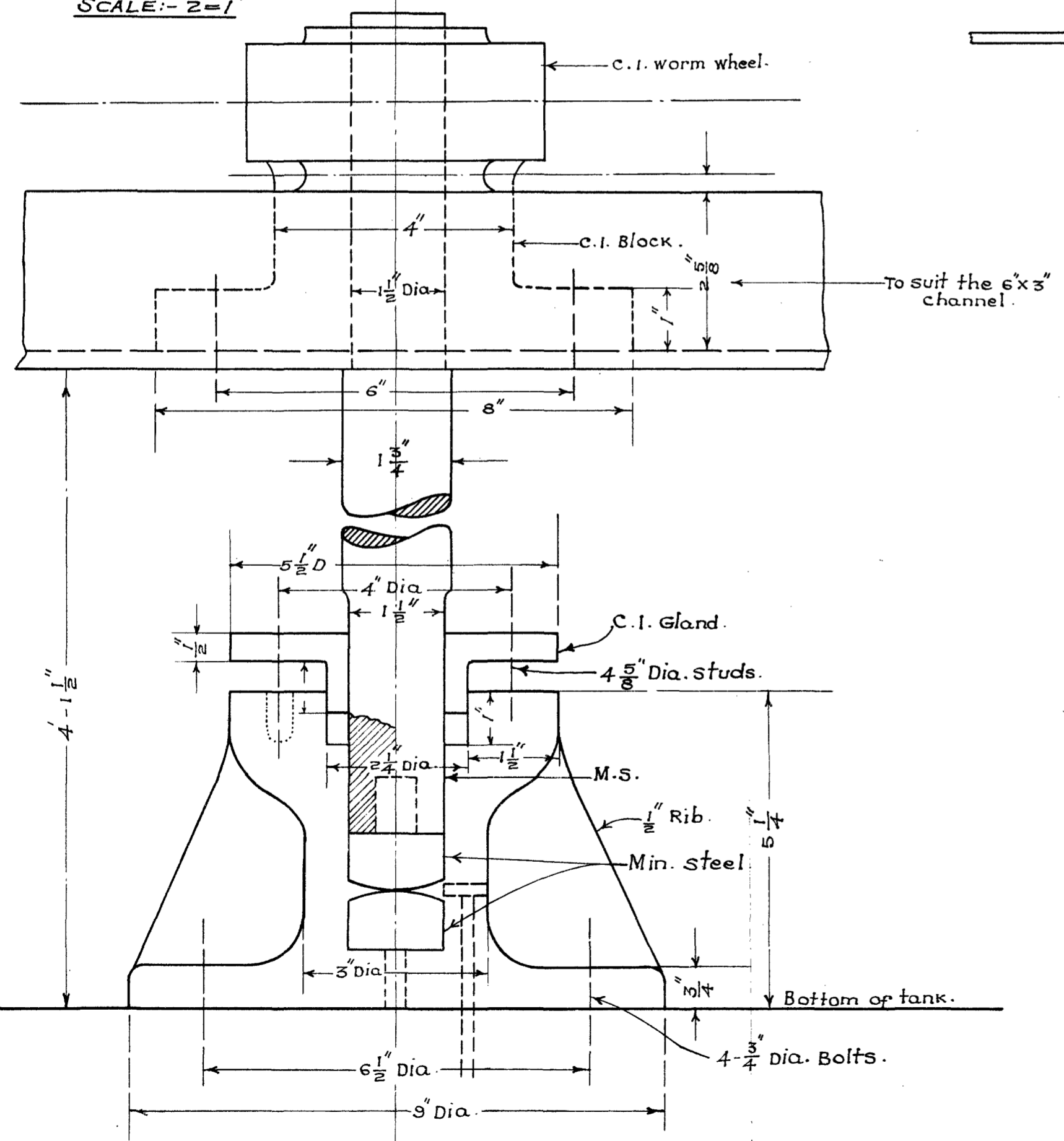
1ST. TIER OF THREE BLADES IN PLAN SHOWN THUS \_\_\_\_\_

2ND. TIER OF THREE BLADES IN " " " - - - - -

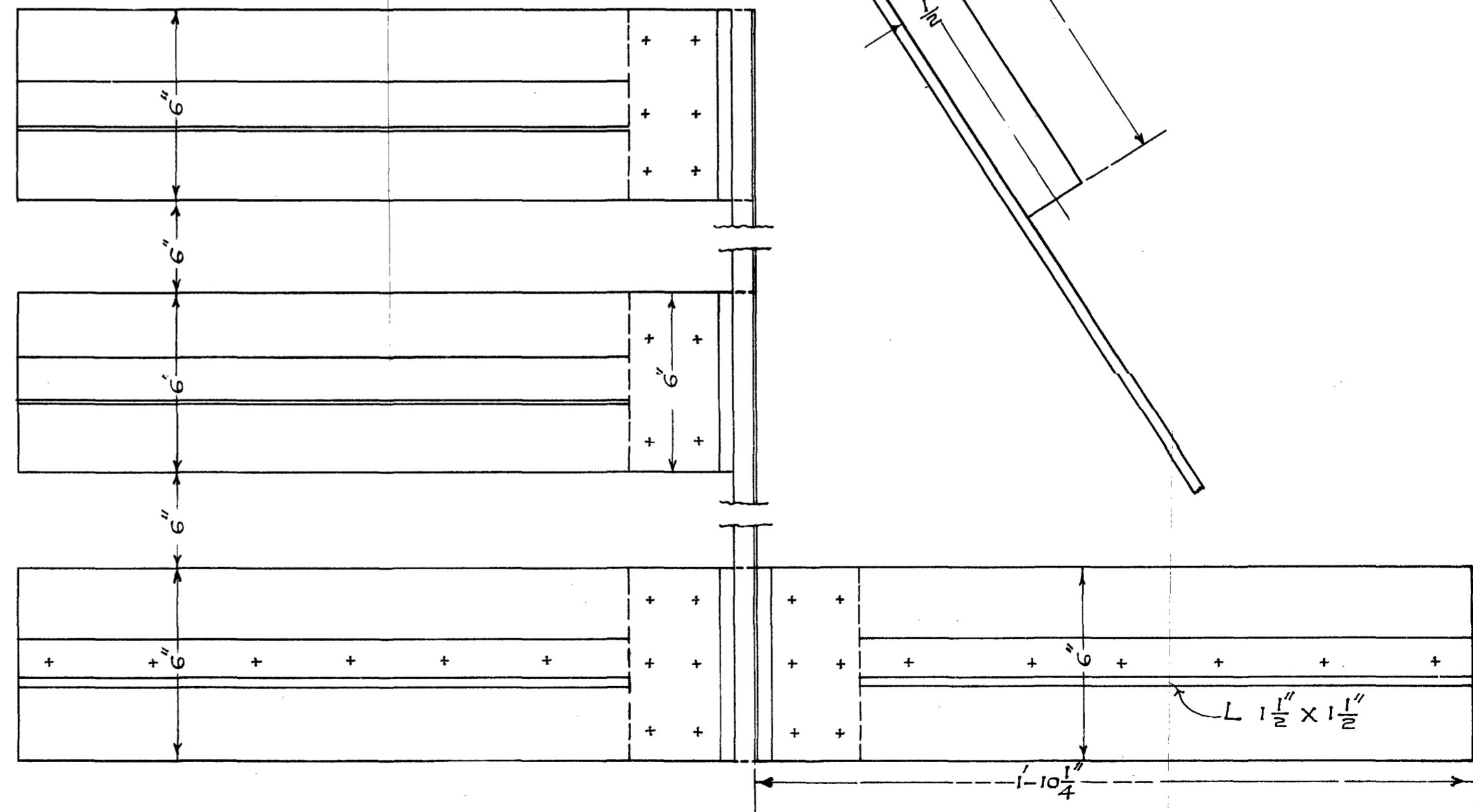
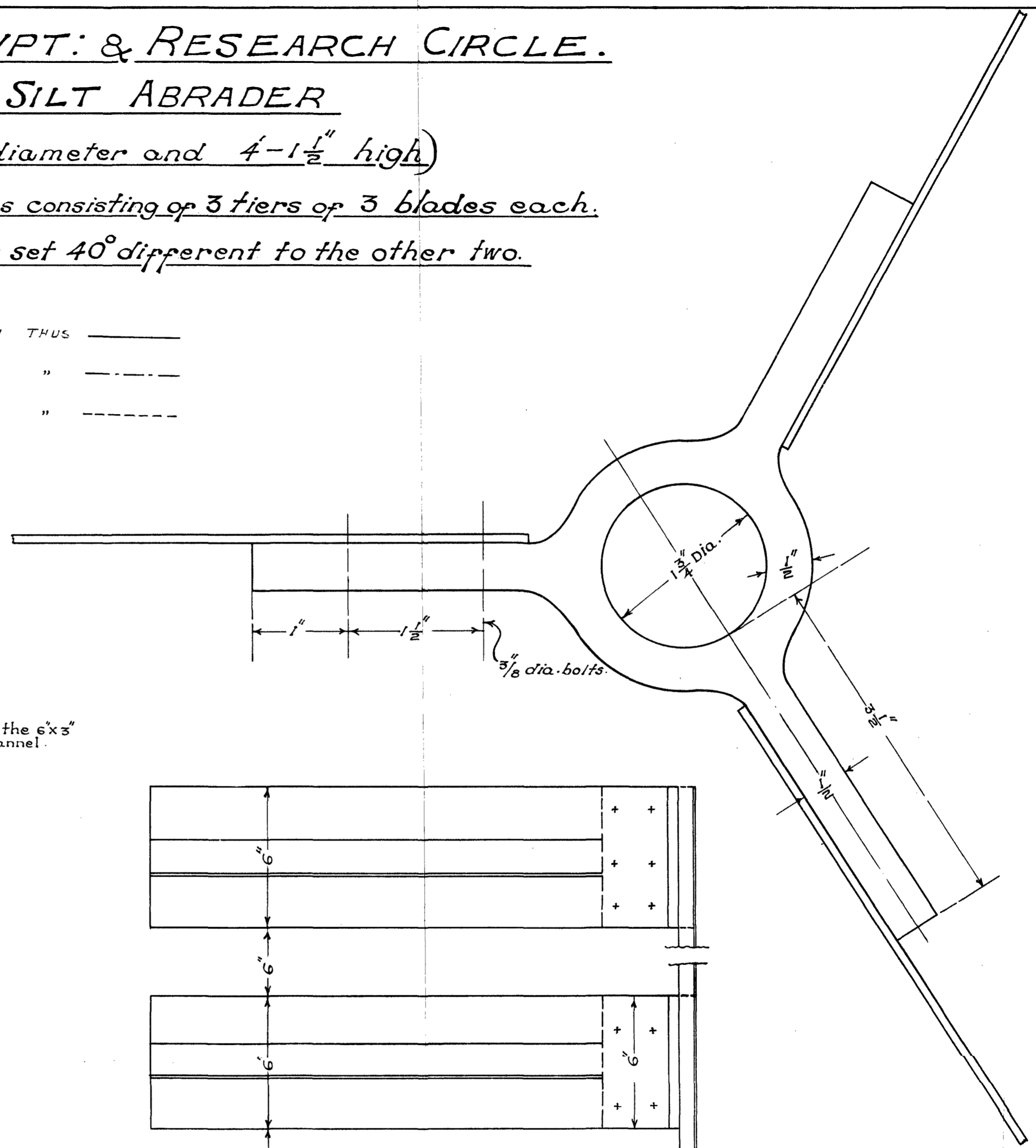
3RD. TIER OF THREE BLADES IN " " " - - - - -  
SILT BEFORE EXPT:

SILT WHEN ABRADER WAS WORKING.

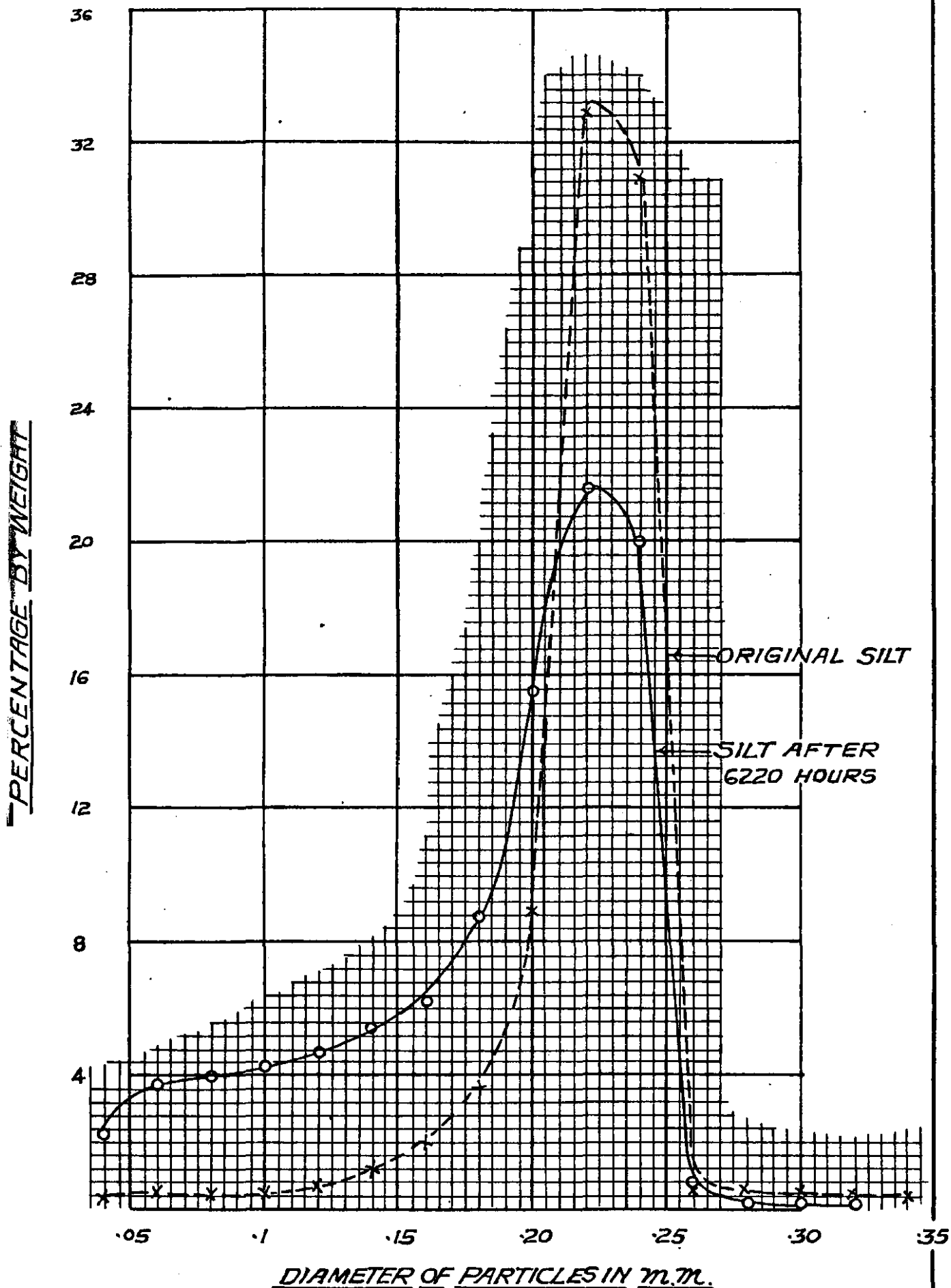
SECTION  
SCALE:- 2=1"



c.i. Foot step Bearing.





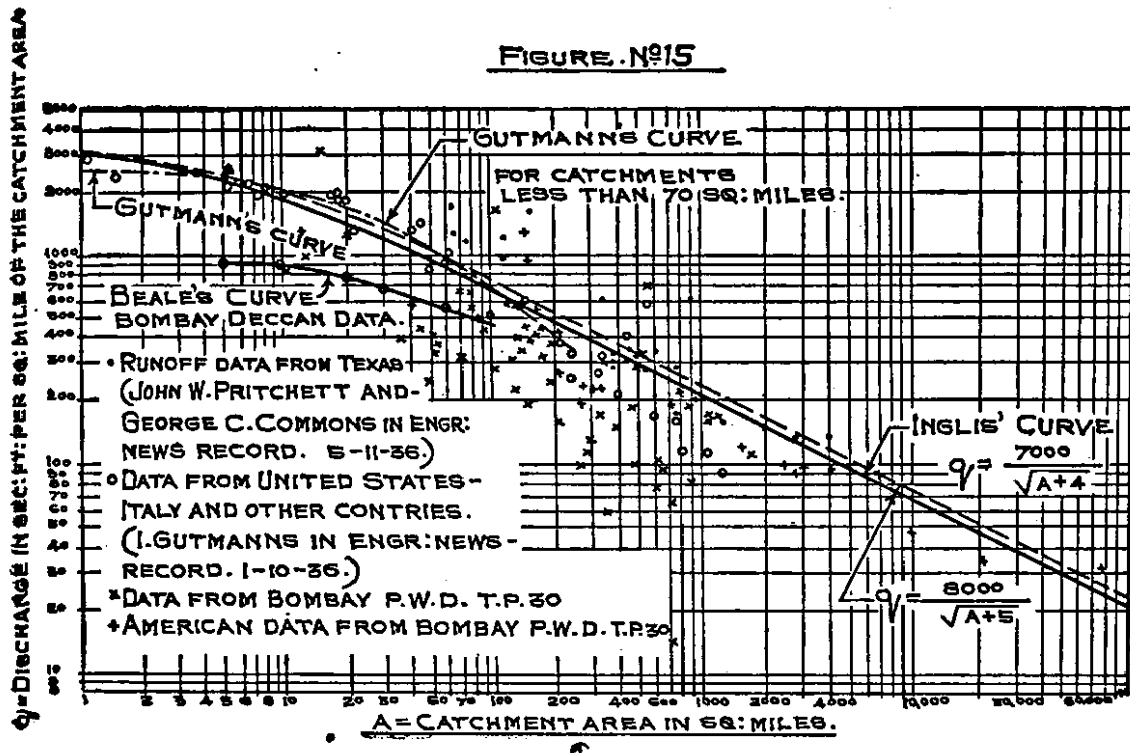


RESULTS BASED ON MECHANICAL ANALYSIS BY SEDIMENTATION & SIEVE METHOD

FIG 14  
SIZE DISTRIBUTION CURVES  
OF SUKKUR SILT FROM SILT  
ABRADER

o—o SILT AFTER 6220 HOURS  
x---x ORIGINAL SILT.

In Engineering News Record of October 1, 1936, I. Gutmann had proposed a formula for small catchments up to 70 square miles (see Fig. 15).



and in bringing this to Mr. Inglis' notice pointed out that though the formula  $q = 7,000/\sqrt{A}$  was suitable for large catchments (excluding the extraordinary Texas data), it was not suitable for small catchments.

The original formula has, therefore, been modified to  $q = 7,000/\sqrt{A+4}$ , so as to be applicable to small as well as large catchments. This gives a discharge at the rate of 3,500 cusecs per square mile for an infinitely small catchment—equivalent to  $5\frac{1}{2}$ " maximum run-off per hour.

The data shown on the plan includes the American data shown by Gutmann, Pritchett and Commons, and Creagar and Justin, and the Indian data of Technical Paper 30. Though a curve  $q = 8,000/\sqrt{A+5}$  fits the data better than  $q = 7,000/\sqrt{A+4}$ , experience shows that maximum floods are nearly always over-estimated by more than 10 per cent. and hence the lower figure is more correct.

The only points appreciably above the curve are:—

- 1, 2, 3, 4, 5, 6, 7, 11, 19 and 22 of the Texas data ;
- 28 of Mr. Gutmann's data, which is for San Caterina Monterey, Mexico, and
- 3, 22 and 52 of Indian data (Technical Paper 30) two of which are definitely valueless and the third untrustworthy, and
- 9 Devil's N. Mouth, Iowa, and 10 Salado Creek, Texas, from Creagar and Justin.

The Texas flood run-off figures are up to 4 times as great as those for other parts of the world and may therefore be left out of consideration. The formula  $q = 7,000/\sqrt{A+4}$  fits the remaining data well.

Prior to the publication of Technical Paper 30, Beale's curve, shown on Fig. 15, was in general use in the Deccan, and in most cases fulfilled requirements admirably.

The explanation of the difference between it and the later curve is that Beale based his curve on floods which had passed without damaging existing canal culverts—in other words, his curve took the flood absorptive capacity of the culverts into account.

Flood absorption is considerable in the case of Deccan canal culverts ; because when water is headed up, the upstream canal bank impounds the flood and the flood absorptive capacity increases inversely as the size of catchment.

So long as Beale's curve is applied only to cases in which the flood absorptive capacity of the structure to be built is of the same order as it was in his data, no harm will result; but in some cases, little or no afflux is desirable. In that case, or where true run-off is required, Inglis' curve should be used. In yet other cases, the flood absorptive capacity may be much greater than allowed for by Beale's curve—e.g. Bhandardara Waste Weir at Lake Arthur Hill.

In general it may be said that the safest method to follow is to work out the maximum discharge—based on whether it is a fan catchment, valley catchment, or intermediate type—; and then allow for the flood absorptive capacity of the structure to be built.

The question of flood absorptive capacity is being investigated.

#### 14. BED SILT OF CANALS.

In the discussion on Dr. Bose's Paper No. 192 on "Silt movement and design of Canals"—page 75m, P.E.C. 1936—Mr. Inglis pointed out that in the data of two canals contained in Dr. Bose's Paper, the mean diameter of silt particles in different channels of each canal varied approximately as  $Q^{1/10}$ . This has since been found to hold also for the Jamrao Canal in Sind and it is clearly not due to a abrasion—i.e. distance from canal head.

Some time ago Mr. C. G. Hawes pointed out that silt found in berms and in suspension in the Jamrao Canal was almost all finer than 0.08 mm., and was practically uniform throughout the canal system. Subsequently Mr. D. V. Joglekar, of this office, found that excluding silt finer than 0.10 mm., Jamrao bed silt was practically uniform (over the whole system) giving a value of  $f$  in the formula  $f = 9.4\sqrt{d}$ , =  $0.74 \pm 0.008$  for the 7 sites in regime and  $0.74 \pm 0.015$  for the 24 sites (excluding only site No. A at the head of the canal, where coarse silt is blown into the Canal from adjoining sand hills).

From this it appears that the inverse variation in mean size of sand particles—which roughly varies as  $Q^{1/10}$ —is mainly due to a larger proportion of fine silt being required in the bed silt of small channels than in large channels, to maintain silt equilibrium. This follows because (i) berms (of silt less than 0.08 mm.) form a larger proportion of the wetted perimeter and (ii) "the depth: width ratio" is greater in small than large channels, and so the ratio of fine silt to coarse silt is greater at all corresponding points.

## List of references.

- (1) BOMBAY PUBLIC WORKS DEPARTMENT TECHNICAL PAPER 55 :  
 " Investigations carried out by means of model at the Khadakvasla Hydrodynamic Research Station in connection with the protection of the Hardinge Bridge which spans the River Ganges near Paksey— Eastern Bengal Railway ", by C. C. Inglis and D. V. Joglekar.
- (2) BOMBAY PUBLIC WORKS DEPARTMENT TECHNICAL PAPER 44 :  
 " The dissipation of energy below falls ", by C. C. Inglis and D. V. Joglekar.
- (3) BOMBAY PUBLIC WORKS DEPARTMENT TECHNICAL PAPER 51 :  
 " Experiments with a short-crested weir model ", by C. C. Inglis and D. V. Joglekar.
- (4) " The causes and prevention of bed erosion, with special reference to the protection of structures controlling Rivers and Canals ", by Butcher and Atkinson— Inst. C. E. Paper 4865 (Vol. 235).
- (5) BOMBAY PUBLIC WORKS DEPARTMENT TECHNICAL PAPER 15 :  
 " Note on Standing Wave Flumes and Flume Meter Falls ", by C. C. Inglis.
- (6) BOMBAY PUBLIC WORKS DEPARTMENT TECHNICAL PAPER 30 :  
 " A critical study of run-off and floods of catchments of the Bombay Presidency ", by C. C. Inglis.
- (7) BOMBAY PUBLIC WORKS DEPARTMENT TECHNICAL PAPER 54 :  
 " Note on experiments with a model of the Head Sluices of the Nira Left Bank Canal ", by C. C. Inglis.
- (8) RAILWAY BOARD'S TECHNICAL PAPER NO. 153 :  
 " River Training and Control on the Guide Bank System ", by Sir Francis Spring.
- (9) BOMBAY PUBLIC WORKS DEPARTMENT TECHNICAL PAPER 26 :  
 " Report on tests made on a model of the group of regulators at Oderolal at mile 169 approximately of the Rohri Main Canal, Sind ", by C. G. Hawes and H. S. Kahai.