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HEADLESS CANAL METERS.

By F. H. Burkitt.

Few of our large channels in the Punjab have a fall near the head where the discharge can be obtained, by the broad crested weir method, and resort is made to current meters. It is the object of the present article to show that the discharge can be obtained, without any appreciable fall, from one or more pairs of gauge readings at a suitably designed masonry work with an accuracy nearly, if not quite, as great as from a free fall broad crested weir.

The theory is extremely simple and scarcely needs mention. If at two sections in a stream close together, where there are no sudden changes to upset a stream line flow, the breadths are b_o and b_1, and the depths y_o and y_1, the depression in water surface between the two sections being h, then the discharge Q is given by the equation :—

$$Q = 8{\cdot}025\ b_o\, y_o\, y_1 \sqrt{\frac{h}{\frac{b_o^2}{b_1^2} \cdot y_o^2 - y_1^2}}$$

For :—

$$V_1^2 - V_o^2 = 2\ gh$$

$$\therefore \frac{Q^2}{A_1^2} - \frac{Q^2}{A_o^2} = 2\ gh$$

$$\because Q^2 = \frac{2\ gh\ A_o^2\, A_1^2}{A_o^2 - A_1^2}$$

$$\therefore Q = 8{\cdot}025\ b_o\, y_o\, b_1\, y_1 \sqrt{\frac{h}{b_o^2\, y_o^2 - b_1^2\, y_1^2}}$$

$$\therefore\ = 8{\cdot}025\ b_o\, y_o\, y_1 \sqrt{\frac{h}{\frac{b_o^2}{b_1^2} \cdot y_o^2 - y_1^2}}$$

The possibility of using this method for obtaining discharges was brought prominently before the writer's notice in 1921 when Mr. Murray and he examined some observations on smooth waves at a masonry work on the Upper Jhelum Canal. It was found that the discharge could be computed with great accuracy from the depths at the crest of the first wave and at the trough between the first two waves. Accordingly they made a design for a meter on these lines on the Upper Jhelum Canal below the Level Crossings, but Mr. Murray left the Canal on promotion to Chief Engineer and his successor dropped the matter.

It was perhaps just as well that it was dropped as sufficient data did not exist at that time for making a very satisfactory design.

Mr. Fane's most valuable experiments, the results of which were read before this Congress in 1927, gave a large series of weir discharges with high drowning ratios which enabled this type of design to be attempted seriously.

The first meter of this type to be constructed was that near the head of the Dipalpur Canal, where the opportunity was taken of combining the meter with a road bridge.

In this design the following requirements had to be satisfied :—

(1) Reasonable accuracy, *i.e.*, a reasonably large depression in water surface between the upstream and downstream gauges, was required for discharges ranging from 2,357 cusecs to 7,071 cusecs.

(2) The loss of head through the meter was limited to 0·2 feet in all conditions.

(3) The velocity at the upstream gauge had to be well above th assumed critical velocity ratio to ensure that the area he would always be constant.

(4) The two sections where the gauge readings are taken were not to be so far apart that the effect of friction would appreciably affect the accuracy.

(5) The contraction in plan was not to be so great as to make the convergence and divergence, upstream and downstream, too expensive.

(6) On closing the canal it would be necessary to unwater the canal upstream.

(7) The spans were not to be too great for a reinforced concrete bridge.

(8) The intensity of discharge upstream and downstream of any span was not to be so much greater than that of its neighbours as to be likely to cause excessive eddies.

In all designs requirements similar to (1), (2), (3) and (4) have to be met. On a new theory of the writer's (explained later) the convergences and divergences can be much cheapened so that (5) is not so important as he thought at the time. (8) also is not very important. The velocities at entrance and exit of the flumes are so comparatively low that eddies here would not cause any appreciable loss of head.

(2) above seems to conflict with the adjective "Headless" used in the title of this paper. The design was based on Mr. Fane's coefficients which were obtained from experiments on comparatively small works. In large well designed works the coefficients with high drowning ratios are considerably greater than those given by Mr. Fane. This Dipalpur Meter was designed for a loss of head of 0·2 feet. In practice it has been found that the loss of head through the meter is actually less than the head required for a similar length of the earthen channel of the canal which has a slope of 1/10,000.

In order to satisfy the eight requirements mentioned, the design adopted consisted of five spans of 35 feet, the four side spans having a raised crest, 2·5 feet above bed, without any side contractions, while the central span had its floor at bed level, its contraction in area being entirely in plan. This contraction in plan involved two very wide piers with long tapering downstream noses. The latter however merely consisted of outer skins of reinforced brickwork, tied together, with sand filling between, so that they were not very expensive.

In the side spans the length of the crest was made 15 feet, and the downstream gauge holes were located 5 feet upstream of the downstream edge of the crest to avoid any interference from bending of the stream lines at the latter point. The length of level crest upstream of these gauge holes was thus 1·54 H. The floor in the region of the upstream auge holes was placed 0·5 feet above bed level to ensure a scouring velo-y. These gauge holes were placed 5 feet upstream of the bottom of e upstream glacis and 10 feet downstream of the end of the curves of ie wing walls and pier noses.

In the central span it was found that an upstream width of 58 feet could be used with a certainty of keeping the section clear of silt. The contraction from 58 to 35 feet was made by a pair of two-centred curves of radii 25 feet and 50 feet (2·8 H and 5·6 H). The upstream gauge holes were located 6 feet above the start of this curve, and the downstream holes 10 feet below the end of it and 5 feet above the start of the downstream splays of 1 in 10. Above the upstream holes was a straight length of pier of 10 feet.

As judged by results all these dimensions have proved satisfactory. As judged by eye, however, the last dimension, or the shape of the noses of the central piers, is wrong. In very low and in very high supplies there is an eddy at the site of the upstream gauge holes. These central pier noses are now being lengthened and the cut-waters made symmetrical on both sides.

In this design the upstream gauge wells were made to extend to a position abreast of the downstream wells in case it were desired to measure h with greater accuracy by means of some mechanical or electrical device. This is a complication which is perhaps scarcely necessary.

Gauges consisting of plain lengths of wood with brass tops were fixed in the gauge wells, and water surface levels have been obtained by measuring down from the brass tops with a boxwood scale. In due course hook gauges will be installed. In practice it took about 5 minutes to read the 5 pairs of gauges, and about half an hour, with a slide rule, to work out the discharges of the five spans. It is feasible to construct a diagram connecting upstream gauge, h, and discharge, so that the latter may be determined at a glance.

This meter was designed for a depression in water surface of 0·39 and 0·24 feet in the side spans with 2357 and 7071 cusecs respectively, and of 0·33 and 0·57 feet in the central span. With these depressions it was calculated that the percentage error, caused by reading one gauge 0·005 too much and the other the same amount too little, would be as follows :—

Discharge.	Side Spans.	Central Span.	Whole Meter.
2,357 ..	·92%	1·3%	1·05%
7,071 ..	1·82%	·8%	1·49%

In practice, even without a hook gauge, it is found possible to read to a much greater accuracy. The designed depression heads have not yet been obtained, as the canal, according to the policy on the Sutlej Valley Project for non-perennial channels, has not yet been dug to its full widt (276 feet), and we have been running greater depths than those for co responding discharges in the final design.

The first readings taken gave depression heads round about 0·(feet only. Here the calculated discharge gave 1,732 cusecs as against th current meter discharge of 1,740 cusecs, a difference of less than half a per cent.

The only method of arriving at the accuracy, or otherwise, of this meter was by comparing its discharges with current meter observations. Appendix I compares the results of the two methods in the case of 28 discharges taken from April to October. The greatest divergence obtained was 6·4% on 28th June 1928 and the average was 1·18%, the Discharge Bridge giving this percentage greater than current meter observations.

With a channel in constant regime evidence could be obtained as to the respective accuracies by plotting discharge against gauge and drawing smooth curves through the observed points. In the case of the Dipalpur Canal during the last season, however, such evidence was not easy to obtain as the channel, on the whole, was scouring heavily, with short temporary intervals of slight silting.

However considerable information can be obtained from plotting, to a large scale, the observations from 23rd June 1928 to 23rd July 1928 and from 23rd July 1928 to 2nd September 1928. These are shown in Diagrams 1 and 2. In the first diagram the weather conditions are noted on six occasions. On the two calm days the divergences were 1·13% and 0·13%, while on the windy days the current meter observations were from 2·58% to 6·4% less than those of the Discharge Bridge. By taking current meter observations in a vertical plane, Mr. Blench found that, when there was an upstream wind, the current meter observations would give discharges of the order of 5% too little if the meter was placed at 0·6 of the depth. Accordingly on 23rd July 1928 he took current meter observations both in vertical and horizontal directions when the current meter discharge worked out to 5,699 cusecs as against the Discharge Bridge result of 5,702 cusecs, a difference of 0·05%.

The evidence given by diagram No. 2 is also strongly in favour of the Discharge Bridge Observations.

In the opinion of the writer, and of the local officers who have been making the observations, the Discharge Bridge appears to give results which are correct within about one per cent, while carefully taken current meter discharges may be as much as 5% in error. If the latter are not very carefully taken the error may be very much greater. There are five possible sources of error with the current meter :—The length of the wire, the soundings, the depth of the current meter, the calibration of the current meter, and the state of the weather. In the discharge flume there is only one variable source of error, the gauge reading.

The design of this meter was enormously complicated by depressing the floor in the central span to bed level, which was done in order to unwater the canal, when closed, upstream. The design becomes quite simple when a weir of the same height right across is employed. Where it is necessary to unwater the canal upstream it would be better to provide a separate bypass for the purpose, as shown in the design for the meter on the Eastern Canal Main Line.

The next meter to be constructed was that at the head of the Upper Sohag Branch. This consists of a single span, 15 feet wide at the site of the downstream gauge with crest raised one foot, and 25 feet wide at the upstream gauge where the floor is at bed level. Experience gained from the Dipalpur Meter showed that the loss of head would be much less than would be calculated from Mr. Fane's co-efficients, and, as a slight fall here would not be objectionable, the downstream divergences were not given splays of 1 in 10, but were made in the form of curves of 100 feet radius.

The Upper Sohag Branch is an old channel which is too wide and deep for its new requirements. It will be silted up and finally there will be little or no fall at the site of this meter, but during the past summer the meter has been acting as a free fall and the discharge can, therefore, be obtained from the broad crested Weir Formula as well as by the method described in this paper. The results of observations showed that the latter gave discharges which averaged 2·4% less than former, the greatest divergence observed being 3% and the smallest 1·8%.

The reason for this error is that the downstream gauge hole is too close, in present conditions, to the downstream edge of the crest.

At present water surface here draws down to a depth of less than two-thirds of H. When the depression head eventually becomes less, this source of error will probably disappear. However, to be on the safe side, it would be better to extend the crest a couple of feet.

Although satisfactory results had been obtained with the comparatively small depression heads at the Dipalpur Meter, it was thought that possibly large depression heads, while theoretically giving greater accuracy, might prove in practice to be less accurate, due perhaps to greater bending of the stream lines. The results obtained from the Upper Sohag Meter, where the depression head has been as much as 1·657 feet, tend to dispel this fear.

In the meter at the head of the Eastern Canal foundations for two piers are being put in, but the piers will not be built unless found necessary. If it is found to be accurate with a single bay, piers and a bridge will be saved, and its discharge will be computed the quicker. The writer is inclinded to think, however, that the intensity of discharge will not be sufficiently uniform, across such a wide span, and that the latter will have to be divided up.

In this meter the crest is all at one level, and a bypass is provided for unwatering the canal when necessary.

Here, and in the case of the meter at the head of the Jallalabad Branch, a new idea is being introduced which may prove useful for flumes in general. A divergence of 1 in 15 will, for the velocities with which we deal, recover all the head which it is possible to recover, and a divergence of 1 in 10 is nearly as good and is usually adopted in our throated bridges.

In the case of a great contraction in plan, this splay of 1 in 10 involves considerable expense.

Now it appears obvious that a change in the direction of flow along the side walls is caused by water pressure at right angles to the latter. It should, therefore, be our aim to keep this pressure constant, and as the velocity of the water drops, the radius of curvature of the side wall should decrease. As shown in Diagram V, this means that RA^2 should remain constant, where R is the radius of curvature of the side walls and A is the area of waterway.

If the floor downstream followed a constant slope, a differential equation could be obtained connecting distances downstream of the crest with offsetts to the side walls, but, in practice, this is not possible as questions of critical velocity ratios upset the uniformity of the floor gradient.

Diagram V shows how these walls may be laid out for any initial radius.

In the designs of the Eastern and Jallalabad Branch meters, the initial radius has been made 200 feet. This type of divergence is very much cheaper than a splay of 1 in 10.

For explaining the method of designing this type of meter, the calculations for the meter at the head of the Jallalabad Branch are given in Appendix II. In this case the factors ruling the design are :—

(*a*) There must be no appreciable loss of head with half share supply.

(*b*) There may, if otherwise desired, be considerable loss of head with maximum supply.

(*c*) Such great accuracy is not required with maximum as with half share (one-third of maximum) supply.

Experience derived from the Dipalpur Design shows that if we take Fane's co-efficients, and a loss of head as shown by him of 0·2 feet, the result will be a design requiring no loss of head. Accordingly start off by assuming a loss of head of 0·2 ft. with minimum supply.

Fane in his experiments has neglected velocities of approach and recess, which is all right in the case of small works. In the design of these meters, however, it much simplifies calculations to add these, so that H_1 becomes still pond level upstream.

Try the crest raised a certain amount above bed. From this H_2 min. is obtained, and H_1 min. is 0·2 greater than H_2 min. The drowning ratio follows, Fane's curve gives C, whence $q_{min.}$ is obtained. The latter gives the width of the crest. From this width at crest $q_{max.}$ is known. From the assumed height of crest H_2 max. is known, and H_1 max. is found by trial and error ; various values of H_1 are taken, with the corresponding values of the drowning ratio and C, till the correct $q_{max.}$ is obtained. The value of H_1 max. thus found gives the loss of head through the meter with maximum supply.

Next find the width at the upstream gauge which will certainly remain silt clear in all conditions. This width gives $q_{max.}$ and $q_{min.}$ at the site of the upstream gauge.

The final step is to find the depression in water surface, both with maximum and minimum supplies, between the upstream and downstream gauges. The writer found this very laborious in the case of the Dipalpur design, and he is indebted to Mr. Brown for a very quick method of obtaining it.

The annexed curve, called "Brown's Curve", gives, for any value of $\frac{q}{H_1}$, the ratio, p, of the depth at the point where the discharge is q, to the height, H_1, of still pond level above that point.

In the case of the Jallalabad Branch Meter it is found that with crest raised 0·75 feet above bed and with 0·2 loss of head (according to Fane) with minimum supply, the width at crest is 24·1 feet, the loss of head with maximum supply is 0·28 feet, the width at upstream gauge should be less than 51 feet (it is taken as 45 feet), with maximum supply the depression in watersurface between the two gauges is 0·675 feet, and with minimum it is 0·488 feet.

With the crest raised 1·0 feet above bed, the width at crest becomes 28·87 feet, the loss of head with maximum supply is 0·21, upstream gauge width is the same as before, and the depressions in water surface become 0·466 and 0·453 with maximum and minimum supplies respectively.

With the crest raised, 1·25 feet above bed, the width at crest becomes 35·72 feet, the loss of head with maximum supply is 0·145, and the depression in water surface 0·291 with maximum and 0·418 with minimum supply.

In this particular case any of these three alternatives gives a satisfactory design, but the second is the cheapest, and has been adopted.

APPENDIX I.

Comparison of Dipalpur meter readings with those given by Current meter.

Date.	Gauge.	Discharge given by Dipalpur Meter.	Discharge given by current Meter.	State of Weather.	Percentage Difference.
..	5·457	1,732	1,740	..	—0·46
..	5·626	1,844	1,851	..	—0·38
25-4-28	7·010	2,884	2,954	..	—2·43
9-5-28	7·483	3,804	3,835	..	—0·81
17-5-28	6·880	3,307	3,130	..	+5·36
23-6-28	7·502	4,260	4,212	Calm	+1·13
28-6-28	7·624	4,637	4,340	U.S. Wind	+6·40
5-7-28	7·728	4,857	4,570	U.S.Wind	+5·91
12-7-28	8·007	5,183	5,176	Calm	+0·13
16-7-28	8·272	5,388	5,160	U. S. Wind	+4·23
17-7-28	8·258	5,348	5,210	Slight ditto	+2·58
18-7-28	8·407	5,663	54.64	..	+3·50
23-7-28	8·374	5,702	5,699	Current meter observations taken vertically as well as Horizontally.	+0·05
26-7-28	8·014	5,076	4,941	..	+2·66
28-7-28	7·855	4,746	4,736	..	+0·21
30-7-28	8·069	5,119	5,017	..	+1·99
31-7-28	8·085	5,163	5,036	..	+2·45
1-8-28	8·074	5,153	5,044	..	+2·11
3-8-28	8·239	5,476	5,204	..	+4·97
13-8-28	8·186	5,359	5,202	..	+2·93
23-8-28	8·236	5,447	5,580	..	—2·44
2-9-28	8·210	5,458	5,224	..	+4·29
12-9-28	5·465	2,321	2,435	..	—4·91
21-9-28	7·844	5,097	5,132	..	—0·65
28-9-28	7·791	4,698	4,768	..	—1·49
6-10-28	6·708	3,549	3,694	..	—1·27
10-10-28	6·009	2,999	2,921	..	+2·60
12-10-28	5·810	2,649	2,790	..	—5·32

APPENDIX II.

Discharge Meter at Head of Jallalabad Branch.

$Q_{max.} = 654$ Cusecs.

$Q_{min.} = \frac{654}{3} = 218$ Cusecs.

Bed Width = 58 Feet.

Slope = 1/6666

$D_{max.} = 5'{\cdot}0$

$D_{min.} = 2'{\cdot}6$

$V_{max.} = \frac{654}{5 \times 60{\cdot}5} = 2{\cdot}16$

$\therefore$ head producing velocity of recess $= \frac{2{\cdot}16^2}{2g} = {\cdot}07$ with maximum supply.

$V_{min.} = \frac{218}{2{\cdot}6 \times 59{\cdot}3} = 1{\cdot}41$

$\therefore$ head producing velocity of recess $= \frac{1{\cdot}41^2}{2g} = {\cdot}03$ with minimum supply.

$D_{max.} + \frac{V^2_{max.}}{2g} = 5{\cdot}07$

$D_{min.} + \frac{V^2_{min.}}{2g} = 2{\cdot}63$

Fall permissible when minimum supply is flowing = $0{\cdot}2'$;

When maximum supply is flowing, the fall may, if desired, be greater. Greatet accuracy is required with small supplies.

FIRST TRIAL.

(1) Try crest raised $0{\cdot}75$ above bed.

$\therefore H_{2\,min.} = 2{\cdot}63 - 0{\cdot}75 = 1{\cdot}88$

$\therefore$ Since we are allowed $0{\cdot}2$ loss of head, $H_{1 min.} = 2{\cdot}08$

$\therefore$ drowning ratio $= \frac{1{\cdot}88}{2{\cdot}08} = {\cdot}904$

From Fane's curve, $C = 3{\cdot}016$

$\therefore q_{min.} = 3{\cdot}016 \times 2{\cdot}08^{3/2} = 9{\cdot}05$

$\therefore$ Width at crest $= \frac{218}{9{\cdot}05} = 24{\cdot}1'$

$q_{max.} = 3\,q_{min.} = 27{\cdot}15$

(2) Find by trial and error, using Fane's curve, H_1 max. knowing that

H_2 max $= 5{\cdot}07 - {\cdot}75 = 4{\cdot}32$

H max	4·59	4·60
Drowning Ratio.	·941	·939
C	2·725	2·756
q max	26·80	27·19

$\therefore H_1$ max $= 4{\cdot}60$

$\therefore$ Loss of head$=4{\cdot}60-4{\cdot}32=0.28$

(3) Next find width at upstream gauge which will certainly remains clear of silt.

$$q = {\cdot}75\, D^{5/3}$$

Take $D = H_1 + {\cdot}75$ (actually it is slightly less, due to velocity head)

$$q \text{ max} = {\cdot}75 \times 5{\cdot}35^{5/3} = 12{\cdot}27$$

$$q \text{ min} = {\cdot}75 \times 2{\cdot}83^{5/3} = 4{\cdot}25$$

Maximum width permissible, at upstream gauge, for maximum $= 53{\cdot}3$

Maximum width permissible, at upstream gauge, for minimum $= 51{\cdot}3$

make the width 45′

so that q max at upstream gauge $= \dfrac{654}{45} = 14{\cdot}53$

so that q min at upstream gauge $= \dfrac{218}{45} = 4{\cdot}84$

(4) Find draw down to upstream gauge

(a) With max $\dfrac{q}{H_1{}^{3/2}} = \dfrac{14{\cdot}53}{5{\cdot}35^{3/2}} = \dfrac{14{\cdot}53}{12{\cdot}375} = 1{\cdot}174$

(From Brown's curve, $p = {\cdot}978$, $1-p = {\cdot}022$

$\therefore$ draw down $= 5{\cdot}35 \times {\cdot}022 = 0{\cdot}118$

(b) With min. $\dfrac{q}{H_1{}^{3/2}} = \dfrac{4{\cdot}84}{2{\cdot}83^{3/2}} = \dfrac{4{\cdot}84}{4{\cdot}758} = 1{\cdot}017$

From Brown's curve, $p = {\cdot}983$, $1-p = {\cdot}017$

$\therefore$ Draw down $= 2{\cdot}83 \times {\cdot}017 = {\cdot}048$

(5) Find draw down to downstream gauge.

(a) With max $\dfrac{q}{H^{3/2}} = C = 2{\cdot}756$

From Brown's curve, p = ·8275, 1—p = ·1725

∴ draw down = 4·60 × ·1725 = ·793

(b) With minimum $\frac{q}{H_1^{3/2}} = C = 3·016$

From Brown's curve, p = ·742, 1—p = ·258

∴ draw down = 2.08 × ·258 = ·536

(6) Depression in water surface between upstream and downstream gauges.

Maximum = ·793 — ·118 = ·675

Minimum = ·536 — ·048 = ·488

Analysis of design :—

This design is fairly suitable. It requires a loss of head of 0·2 with minimum discharge, and 0·28 with maximum. The difference in gauge readings is perhaps unnecessarily great, especially with maximum supply.

The contraction at the crest is however rather great, and involves expense in convergence and divergence.

Therefore try a wider design, with a higher crest.

SECOND TRIAL.

(1) Try crest raised 1·0 above bed.

∴ H_2 min = 2·63 — 1·0 = 1·63

∴ H_1 min = 1·83

∴ Drowning ratio = $\frac{1·63}{1·83}$ = ·8907

From Fane's curve, C = 3·05

∴ Q min = 3·05 × $1·83^{3/2}$ = 7·55

∴ Width at crest = $\frac{218}{7·55}$ = 28·87

q max = 3 q min = 22·65.

(2) Find, by trial and error, from Fane's curve, H_1 max ; H_2 max being 5·07 — 1·0 = 4·07

H_1 max	4·26	4·27	4·28	
Drowning ratio	·955	·953	·951	∴ H_1 max = 4·28
C	2·485	2·526	2·5615	∴ loss of head = 0·21
q max	21·85	22·29	22·68	

(3) Depths at upstream gauge are practically the same as in the first trial, so that the same upstream width of 45′ will be retained.

q max at up-stream gauge = 14·53.

Q min at up-stream gauge = 4·84.

(4) Find draw down to upstream gauge.

(a) With maximum $\frac{q}{H_1{}^{3/2}} = \frac{14{\cdot}53}{(4{\cdot}28+1)^{3/2}} = \frac{14{\cdot}53}{5{\cdot}28^{3/2}} = 1{\cdot}198$

From Brown's curve, $p = {\cdot}977$, $1\text{-}p = {\cdot}023$

$\therefore$ Draw down $= 5{\cdot}28 \times {\cdot}023 = {\cdot}122$

(b) with minimum $\frac{q}{H_1{}^{3/2}} = \frac{44{\cdot}84}{(1{\cdot}83+1)^{3/2}}$ = as before $= 1{\cdot}017$

From Brown's curve, $p = {\cdot}983$ $\quad 1-p = {\cdot}017$

$\therefore$ Draw down $= 2{\cdot}83 \times {\cdot}017 = {\cdot}048$

(5) Find draw down to downstream gauge

(a) with maximum, $\frac{q}{H_1{}^{3/2}} = C$ (already found) $= 2{\cdot}5615.$

From Brown's curve, $p = {\cdot}8625$, $1-p = {\cdot}1375$

$\therefore$ Draw down $= 4{\cdot}28 \times {\cdot}1375 = {\cdot}588$

(b) with minimum $\frac{q}{H_1{}^{3/2}} = C = 3{\cdot}05$,

From Brown's curve, $p = {\cdot}726$, $\quad 1-p = {\cdot}274$

$\therefore$ Draw down $= 1{\cdot}83 \times {\cdot}274 = {\cdot}501$

(6) Depression in water surface between up stream and down stream gauges.

With maximum supply $= {\cdot}588 - {\cdot}122 = {\cdot}466$
With minimum supply $= {\cdot}501 - {\cdot}048 = {\cdot}453$
This design is suitable in every way.

THIRD TRIAL.

(1) Try crest raised $1{\cdot}25'$ above bed.

$\therefore H_2$ min $= 2{\cdot}63 - 1{\cdot}25 = 1{\cdot}38$

$\therefore H_1$ min $= 1{\cdot}38 + {\cdot}2 = 1{\cdot}58$

$\therefore$ Drowning ratio $= \frac{1{\cdot}38}{1{\cdot}58} = {\cdot}8734.$

From Fane's curve, $C = 3{\cdot}073$

$\therefore$ q min $= 3{\cdot}073 \times 1{\cdot}58^{3/2} = 6{\cdot}10$

$\therefore$ Width at crest $= \frac{218}{6{\cdot}1} = 35{\cdot}72.$

$\therefore$ q maximum $= 3$ q minimum $= 18{\cdot}30$

(2) Find H_1 max by trial and error from Fane's curve.

H_2 max ; $= 5{\cdot}07 - 1{\cdot}25 = 3{\cdot}82$

H_1 max	3·98	3·96	3·965
Drowning Ratio	·960	·965	·963
C	2·398	2·297	2·323
q max	19·04	18·10	18·34

$\therefore H_1 \max = 3{\cdot}965$

$\therefore$ Loss of head $= 3{\cdot}965 - 3{\cdot}82 = {\cdot}145$

(3) Depths at up-stream gauge are substantially the same as in first trial, so that the same up-stream width of 45′ will be retained.

q maximum at up-stream gauge = 14·53

and q minimum at up-stream gauge = 4·84

(4) Find draw down to up-stream gauge.

(a) With maximum $\dfrac{q}{H_1{}^{3/2}} = \dfrac{14{\cdot}53}{(3{\cdot}965+1{\cdot}25)^{3/2}} = \dfrac{14{\cdot}53}{5{\cdot}215^{3/2}} = 1{\cdot}220$

From Brown's curve, p = ·976, 1-p = ·024

$\therefore$ Draw Down = 5·215 x ·024 = ·125

(b) With maximum, every thing is the same as before, and draw down = ·048

(5) Find Draw down to down-stream gauge

(a) With maximum $\dfrac{q}{H_1{}^{3/2}} = C = 2{\cdot}323$

From Brown's curve, p = ·895. 1-p = ·105

Draw down = 3·965 x ·105 = ·416

(b) With minimum, $\dfrac{q}{H_1{}^{3/2}} = C = 3{\cdot}073$

From Brown's curve, p = ·705, 1-p = ·295

$\therefore$ Draw down = 1.58 x.295 = .466

Depression in water surface between up-stream and down-stream :—

with maximum supply = .416—·125 = ·291

With minimum supply = .466—·048 = ·418

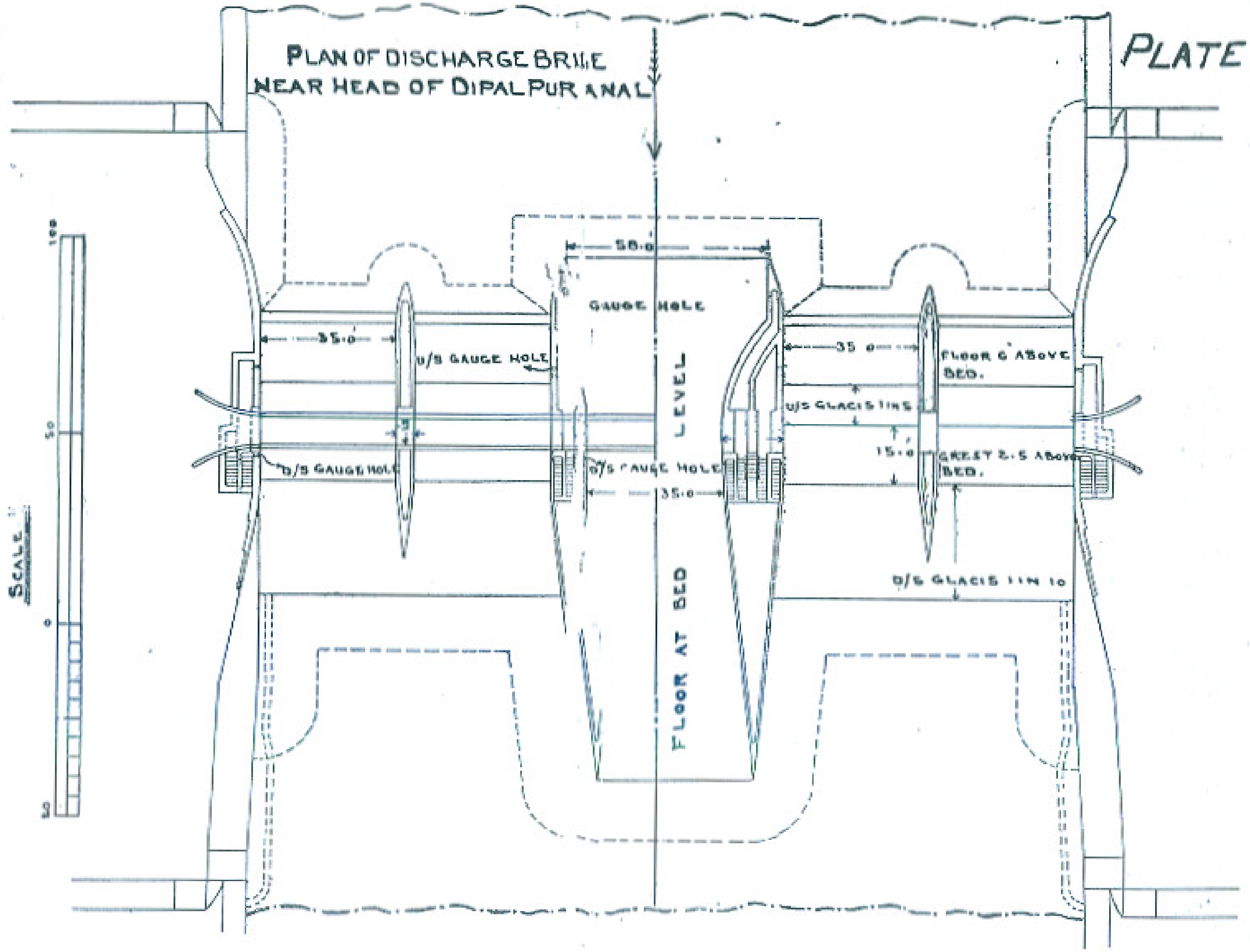
This design is suitable, but it is a little more expensive than (2)

So (2) should be adopted.

Width at crest 29′

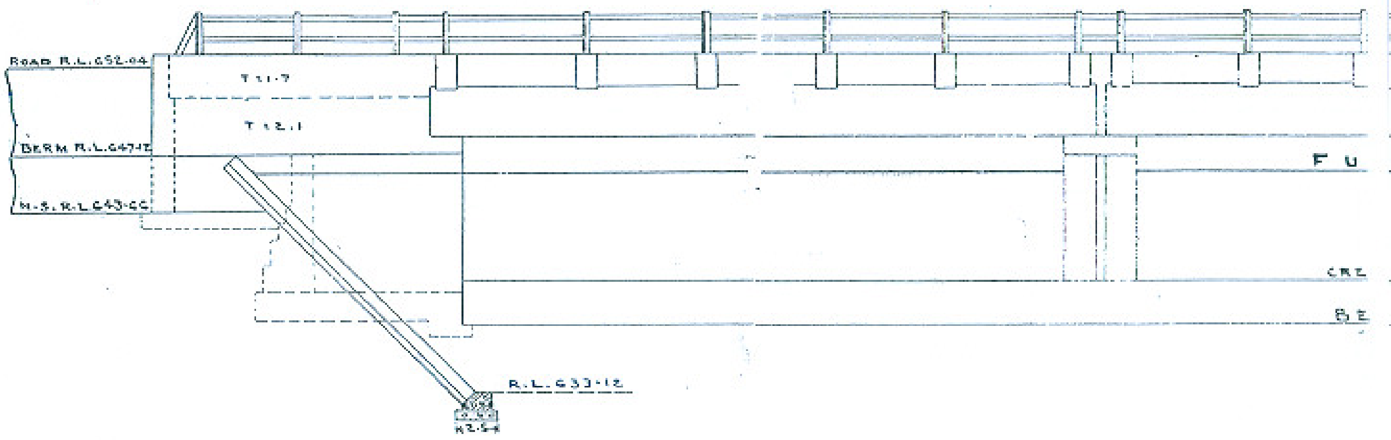
Crest raised 1′.0

PLATE I


PLAN OF DISCHARGE BRIɩɛE NEAR HEAD OF DIPALPUR ANAL
SCALE
100
50
0
50
58.0
GAUGE HOLE
35.0
U/S GAUGE HOLE
D/S GAUGE HOLE
D/S GAUGE HOLE
35.0
LEVEL
FLOOR AT BED
35.0
FLOOR 6' ABOVE BED.
U/S GLACIS 1 IN 5
15.0
CREST 2.5 ABOVE BED.
D/S GLACIS 1 IN 10

METER AT HEAD OF DIPALPUR CANAL.

HALF D/S ELEVATION



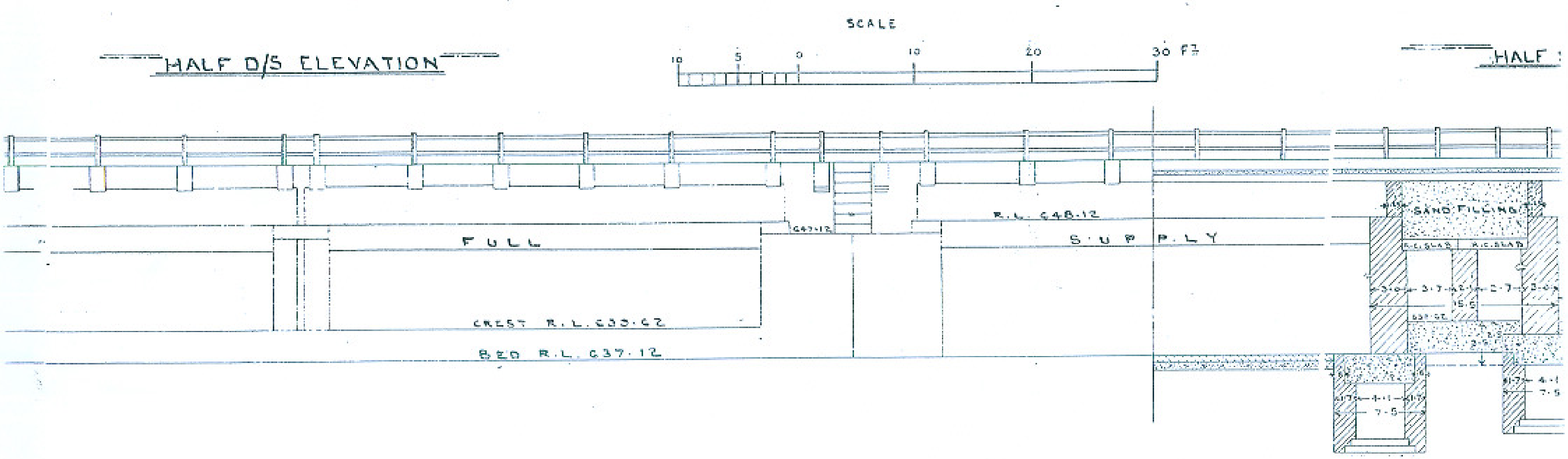
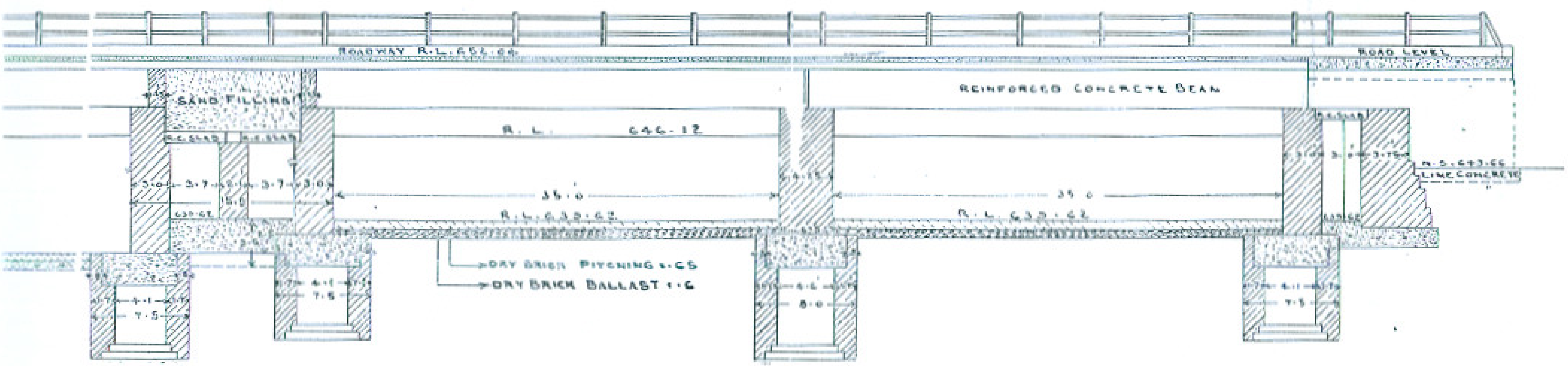
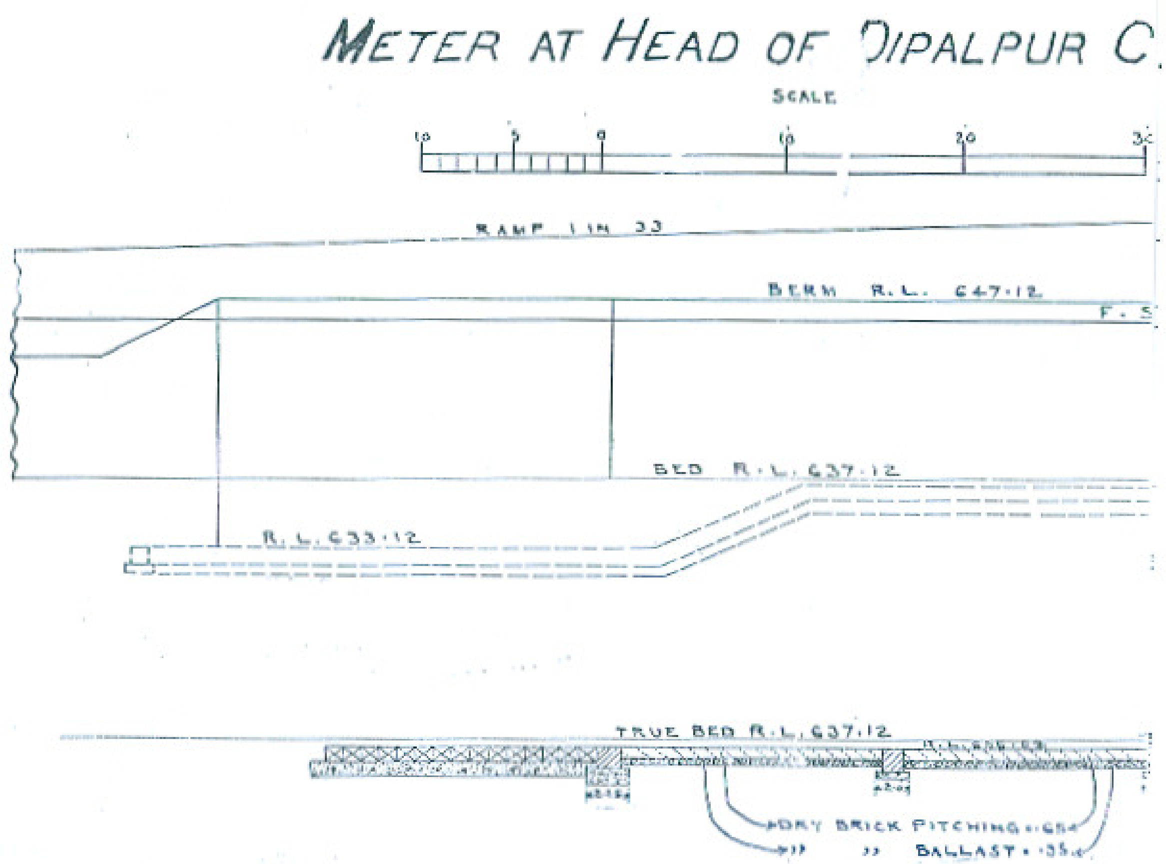

HALF D/S ELEVATION
SCALE
10
5
0
10
20
30 FT
HALF
R.L. 648.12
FULL SUPPLY
647.12
CREST R.L. 633.62
BED R.L. 637.12
SAND FILLING
R.C. SLAB
R.C. SLAB
15.5
7.5
4.1

PLATE II

HALF SECTION THROUGH CENTRE OF BRIDGE




METER AT HEAD OF DIPALPUR C.
SCALE
10
5
0
10
20
30
RAMP 1 IN 33
BERM R.L. 647.12
F. S
BED R.L. 637.12
R.L. 633.12
TRUE BED R.L. 637.12
DRY BRICK PITCHING
BALLAST

METER AT HEAD OF DIPALPUR CANAL

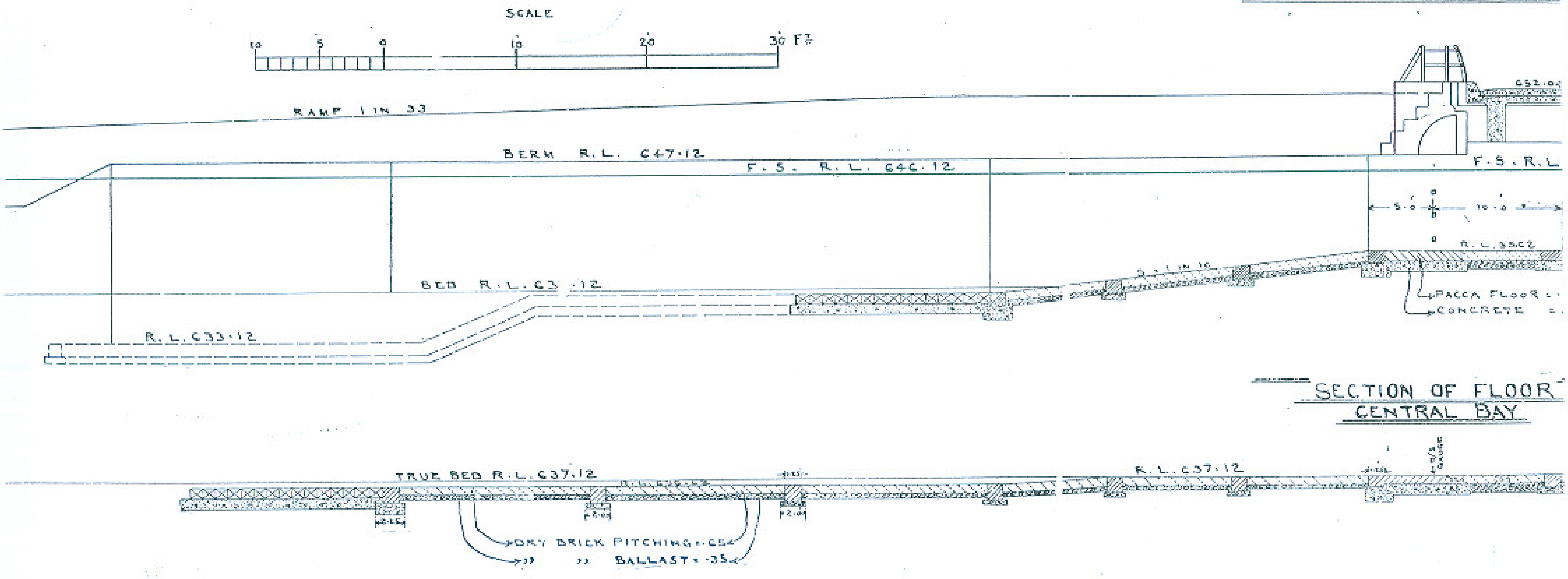
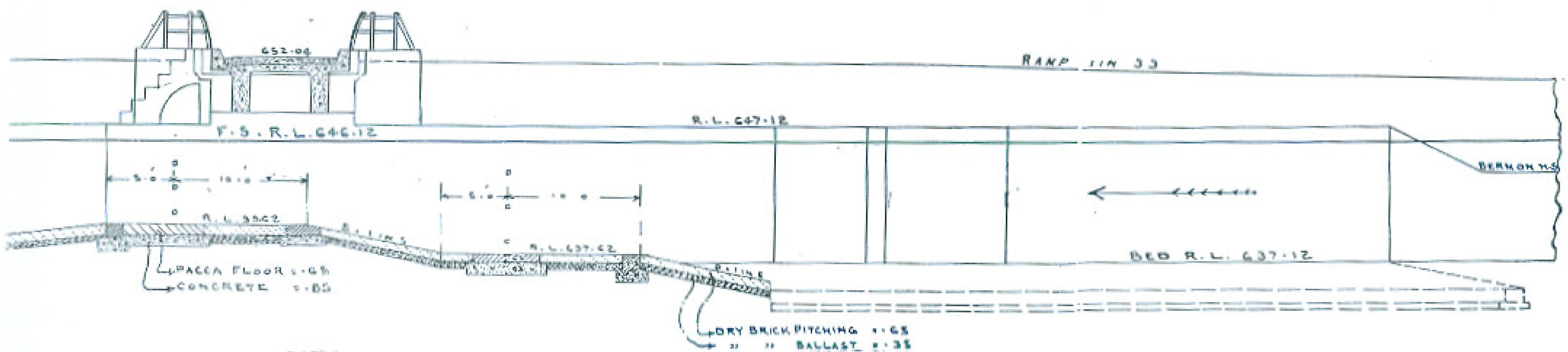


PLATE III

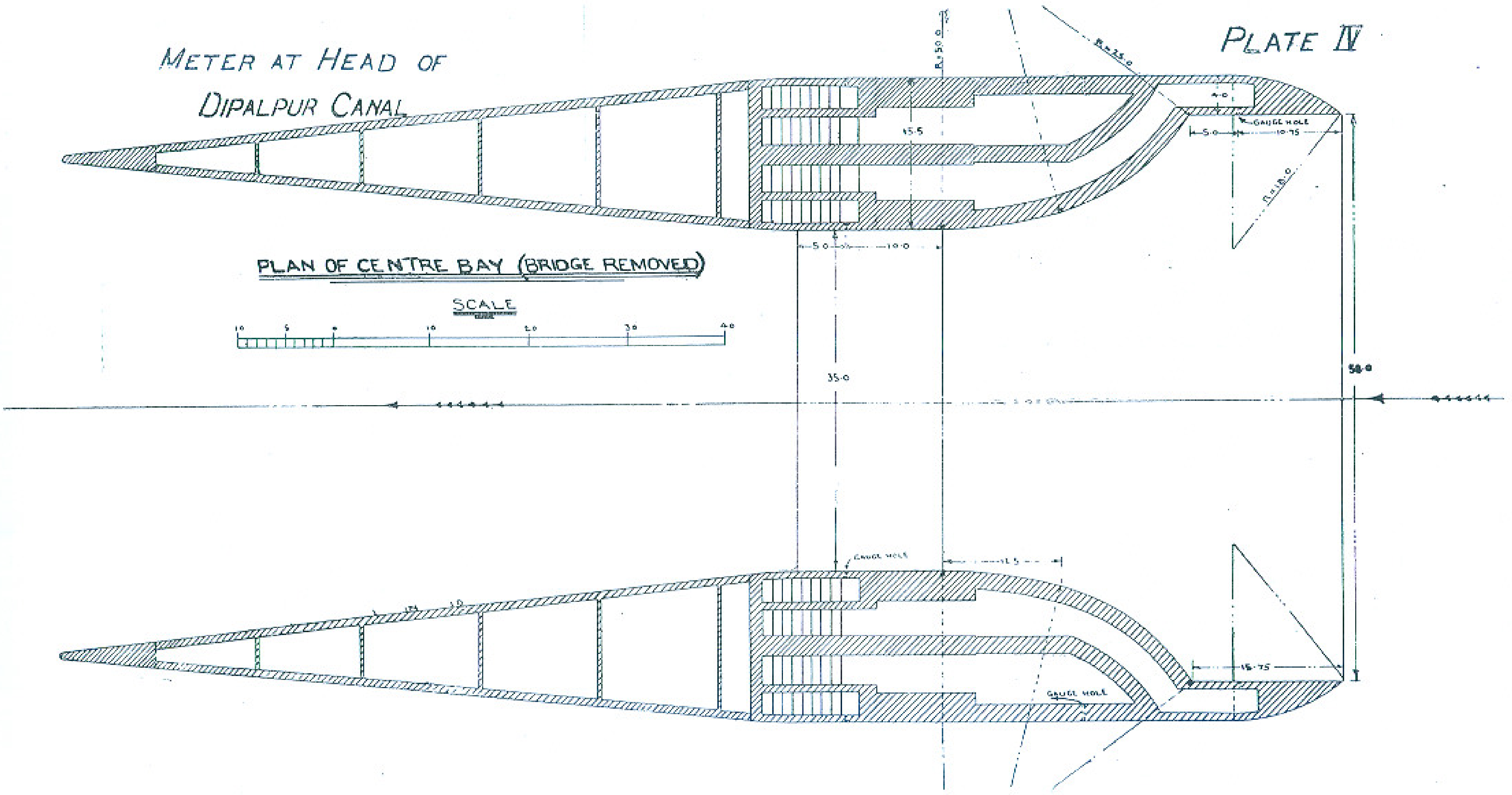
LONGITUDINAL SECTION THROUGH SIDE BAYS



SECTION OF FLOOR CENTRAL BAY

PLATE IV

METER AT HEAD OF DIPALPUR CANAL



PLAN OF CENTRE BAY (BRIDGE REMOVED)

SCALE

LONG

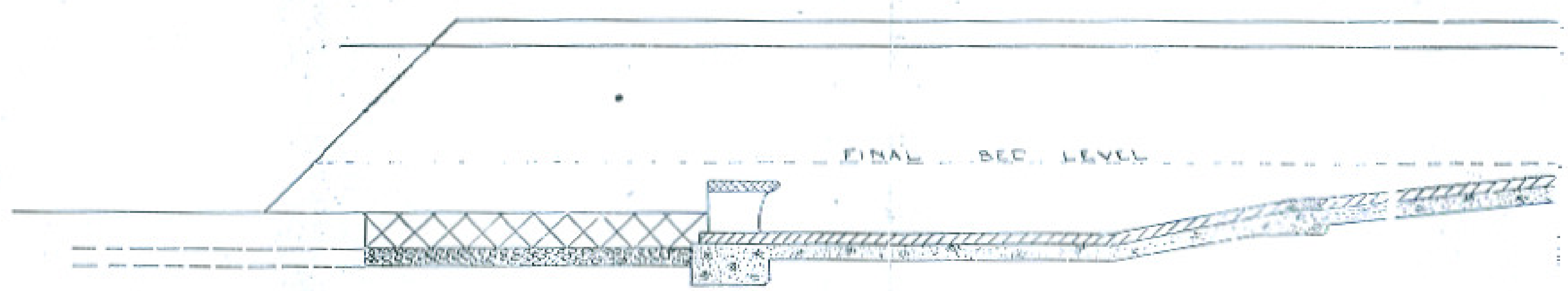
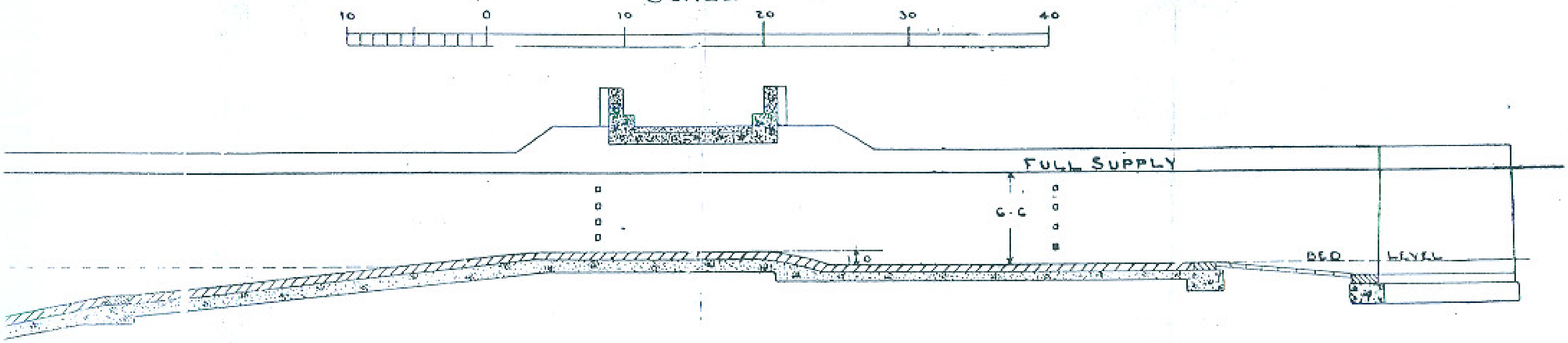
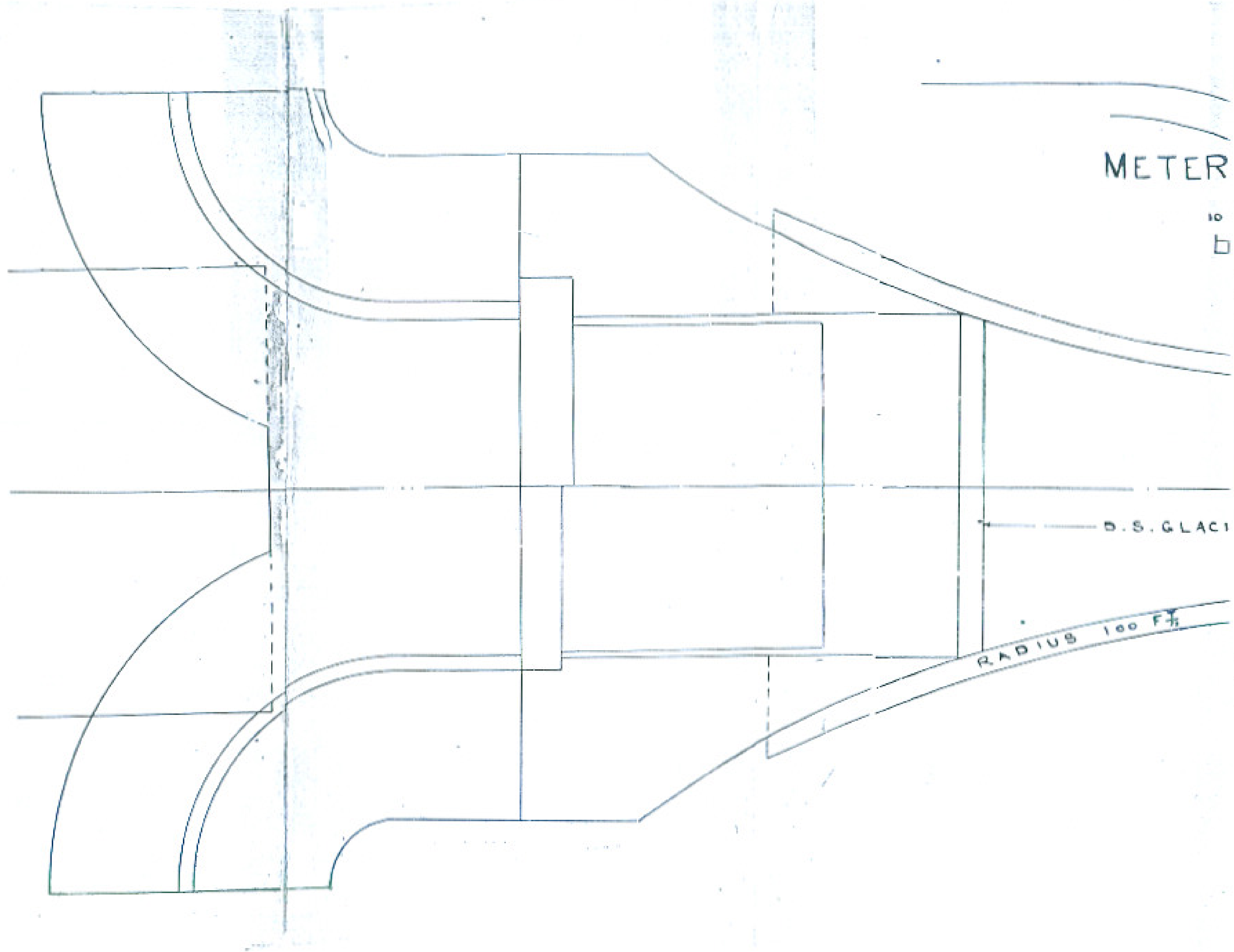


PLATE VII

LONGITUDINAL SECTION OF METER AT HEAD OF UPPER SOHAG BRANCH

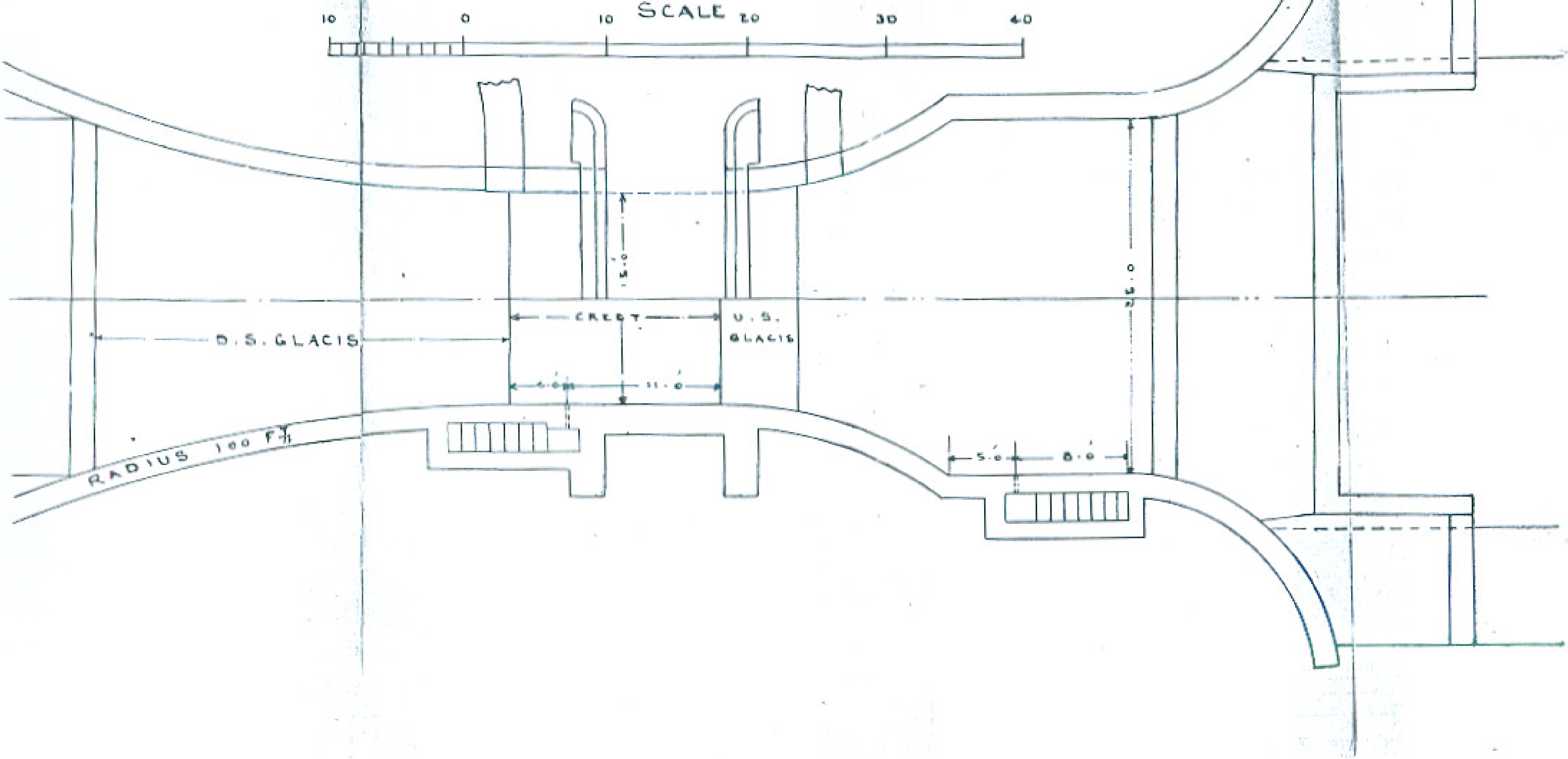




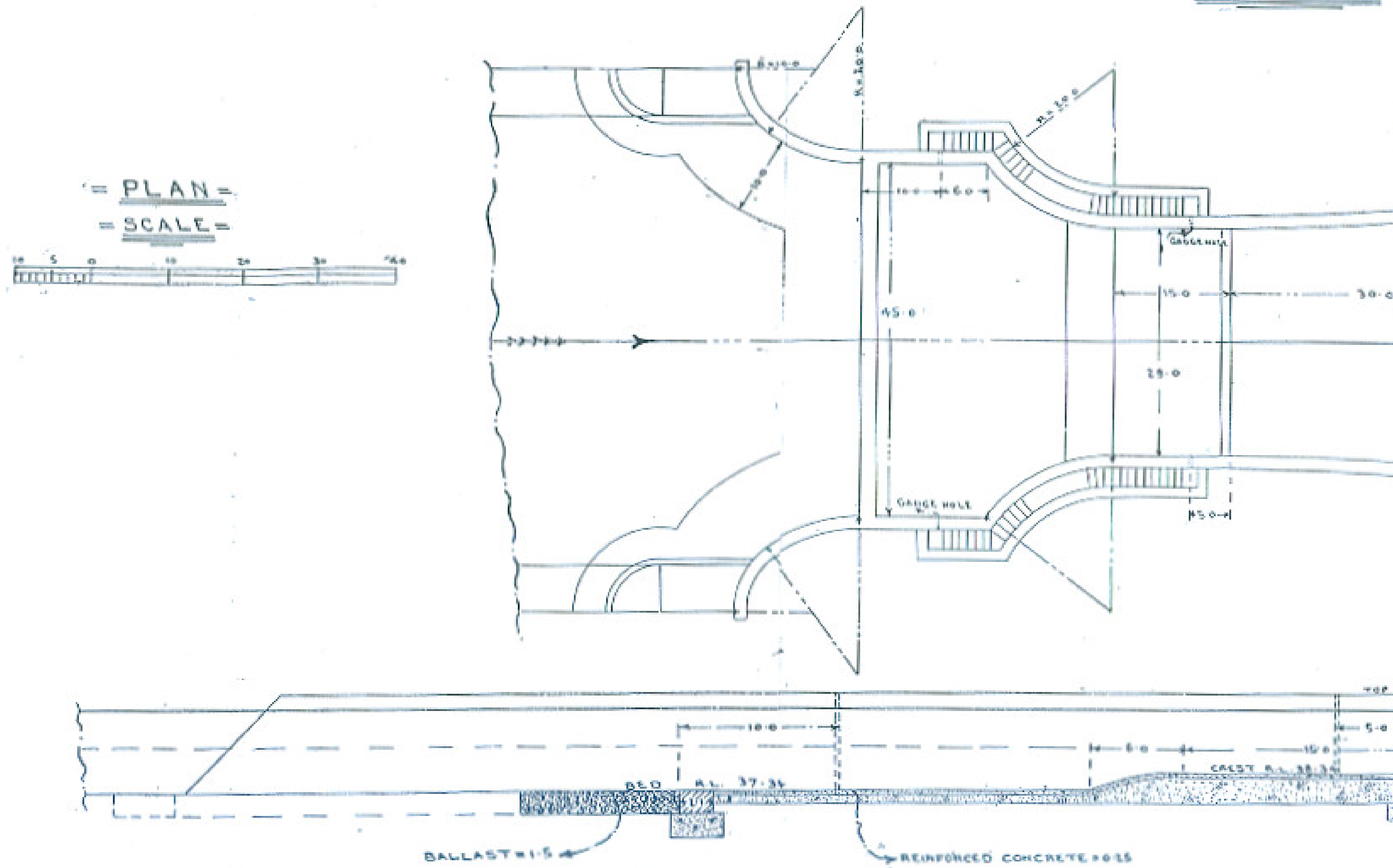
METER
D.S. GLACI
RADIUS 100 FT

PLATE V

METER AT HEAD OF UPPER SOHAG BRANCH



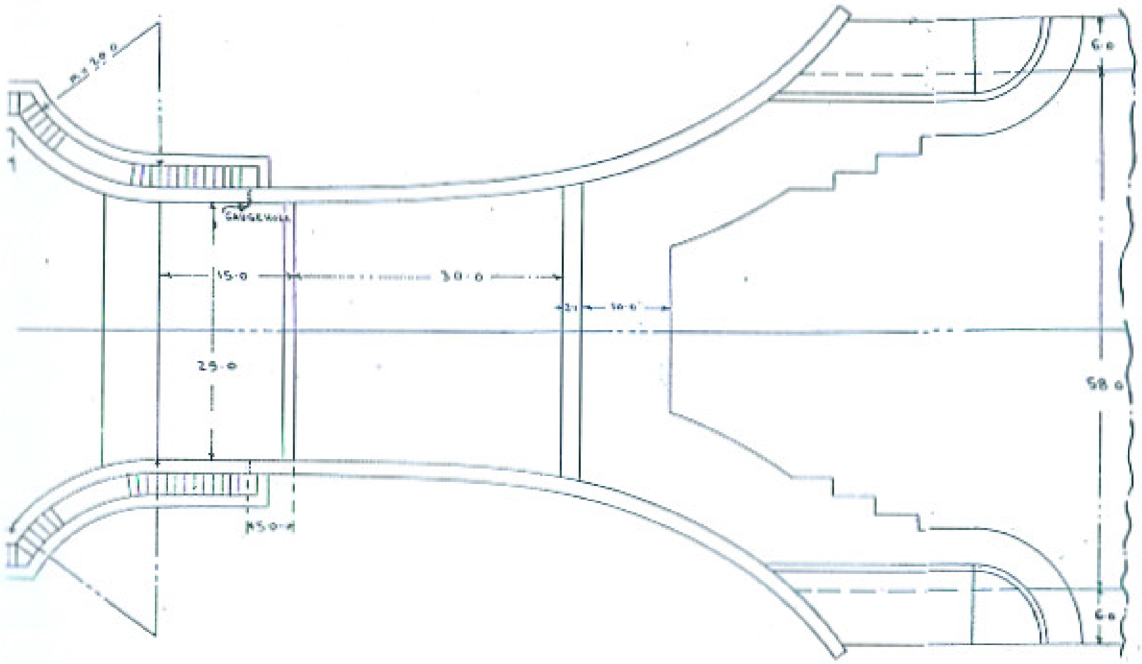
= PLATE IX =



= PLAN =

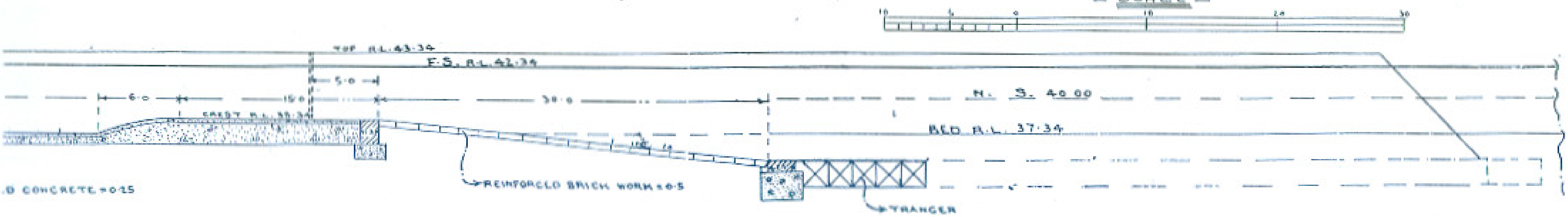
= SCALE =

= PLATE IX =



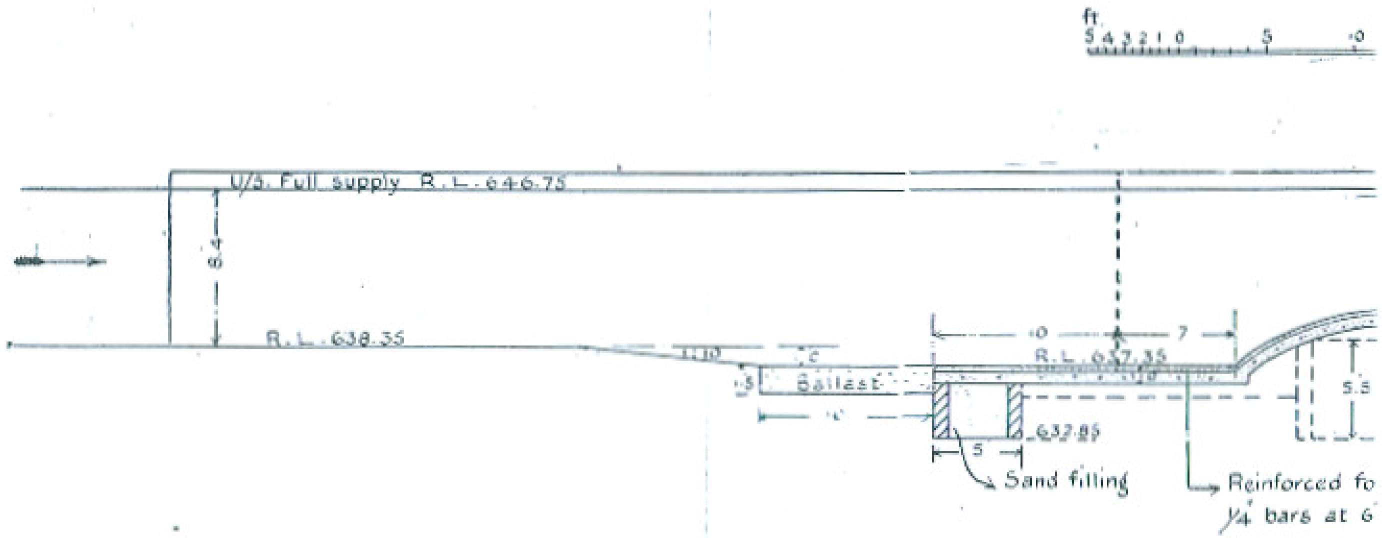
= METER AT HEAD =
= OF JALALABAD BRANCH =

= LONGITUDINAL SECTION =
= SCALE =



Longitudinal Section
at Head of E

Scal



Longitudinal Section of Discharge Meter at Head of Eastern Canal

PLATE VIII

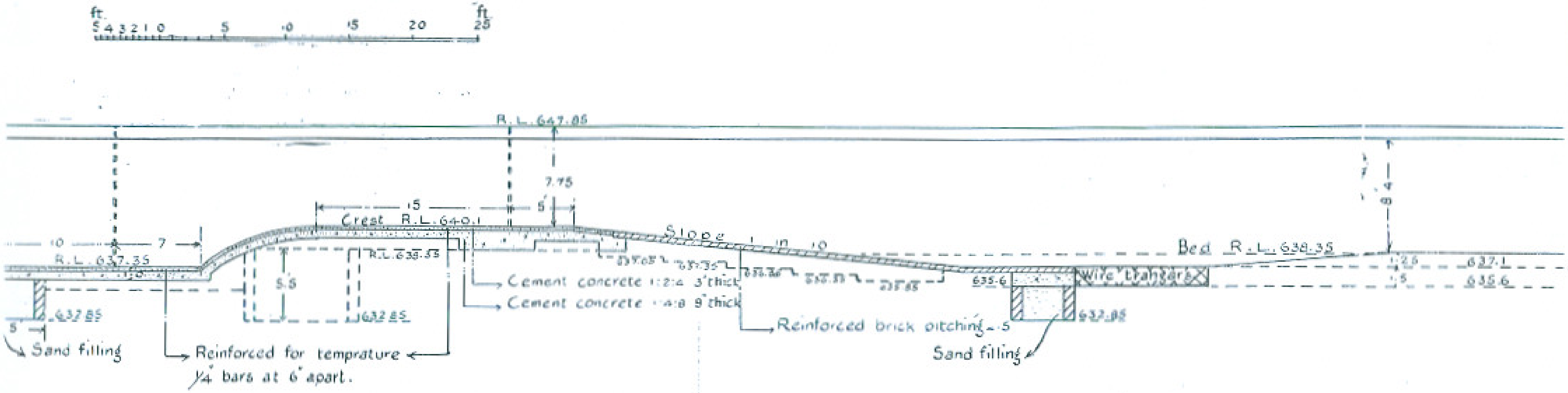


PLATE VIII

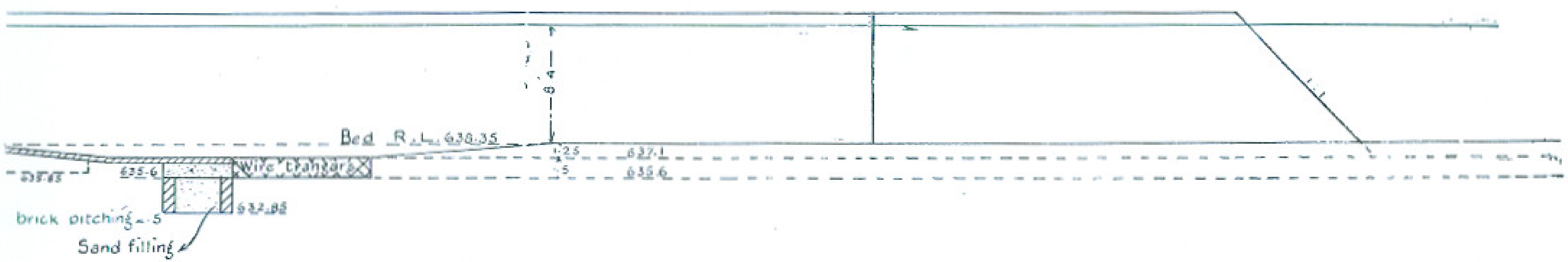

Bed R. L. 638.35
8.4
1.25
637.1
.5
635.6
1:1
635.65
635.6
Wire crates
632.85
brick pitching .5
Sand filling

DIAGRAM III

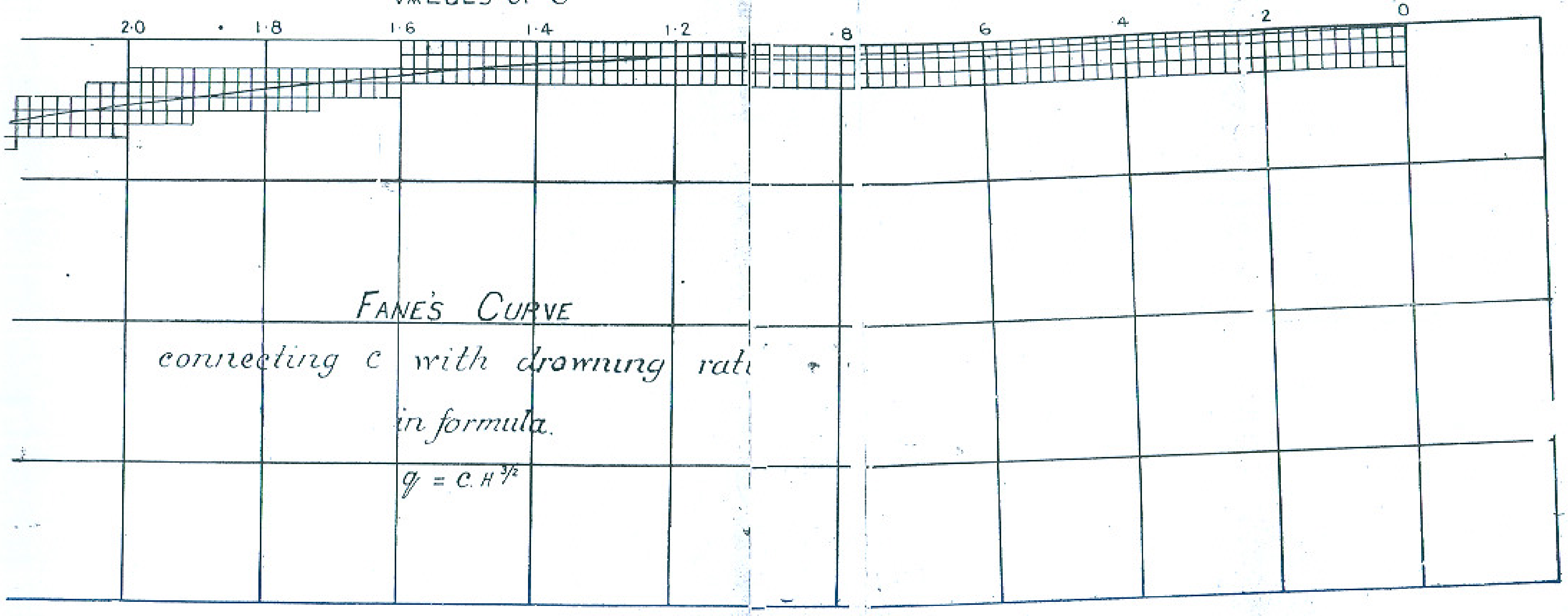
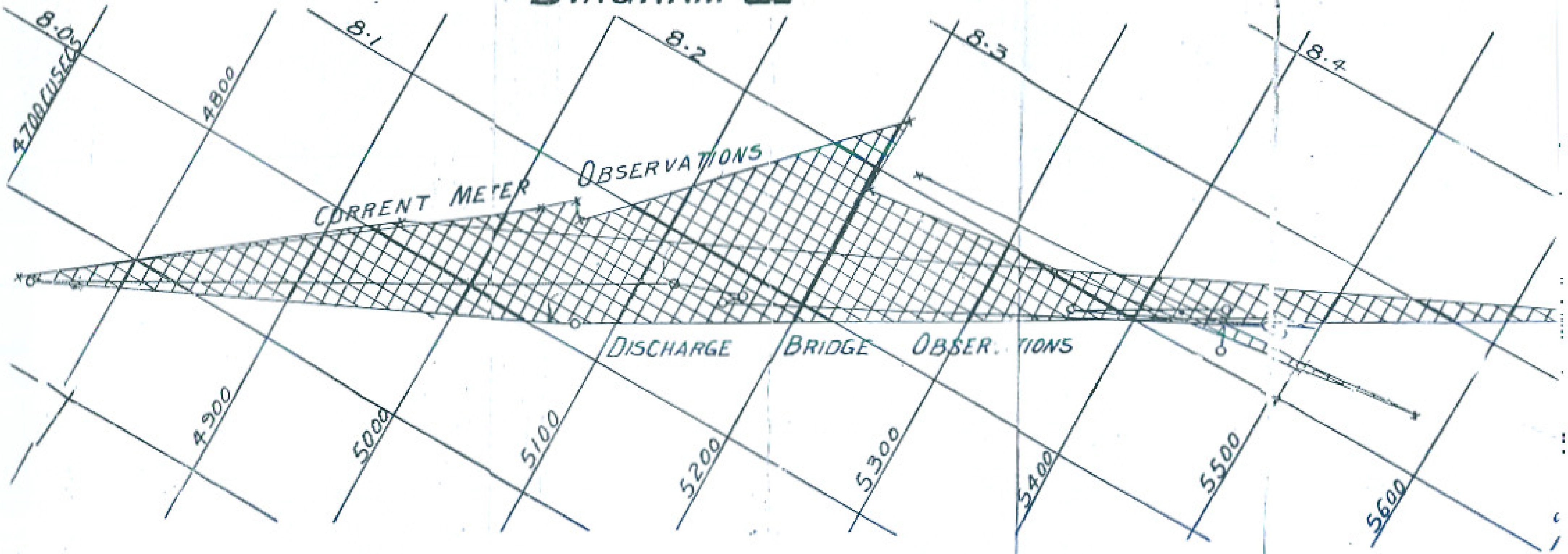
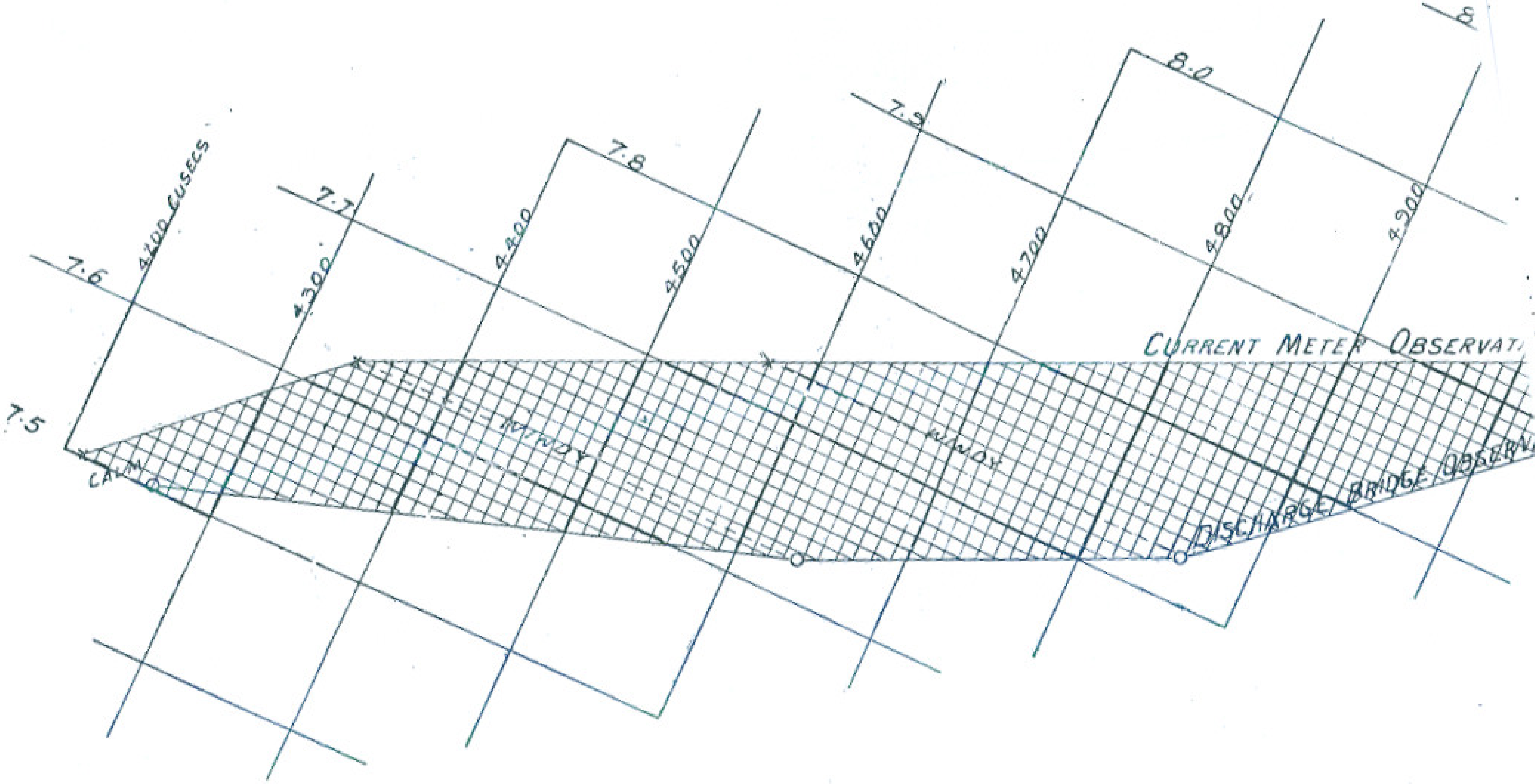

VALUES OF C
2·0
1·8
1·6
1·4
1·2
·8
6
·4
·2
0
FANE'S CURVE
connecting c with drowning rati
in formula.
q = C.H 3/2

DIAGRAM II



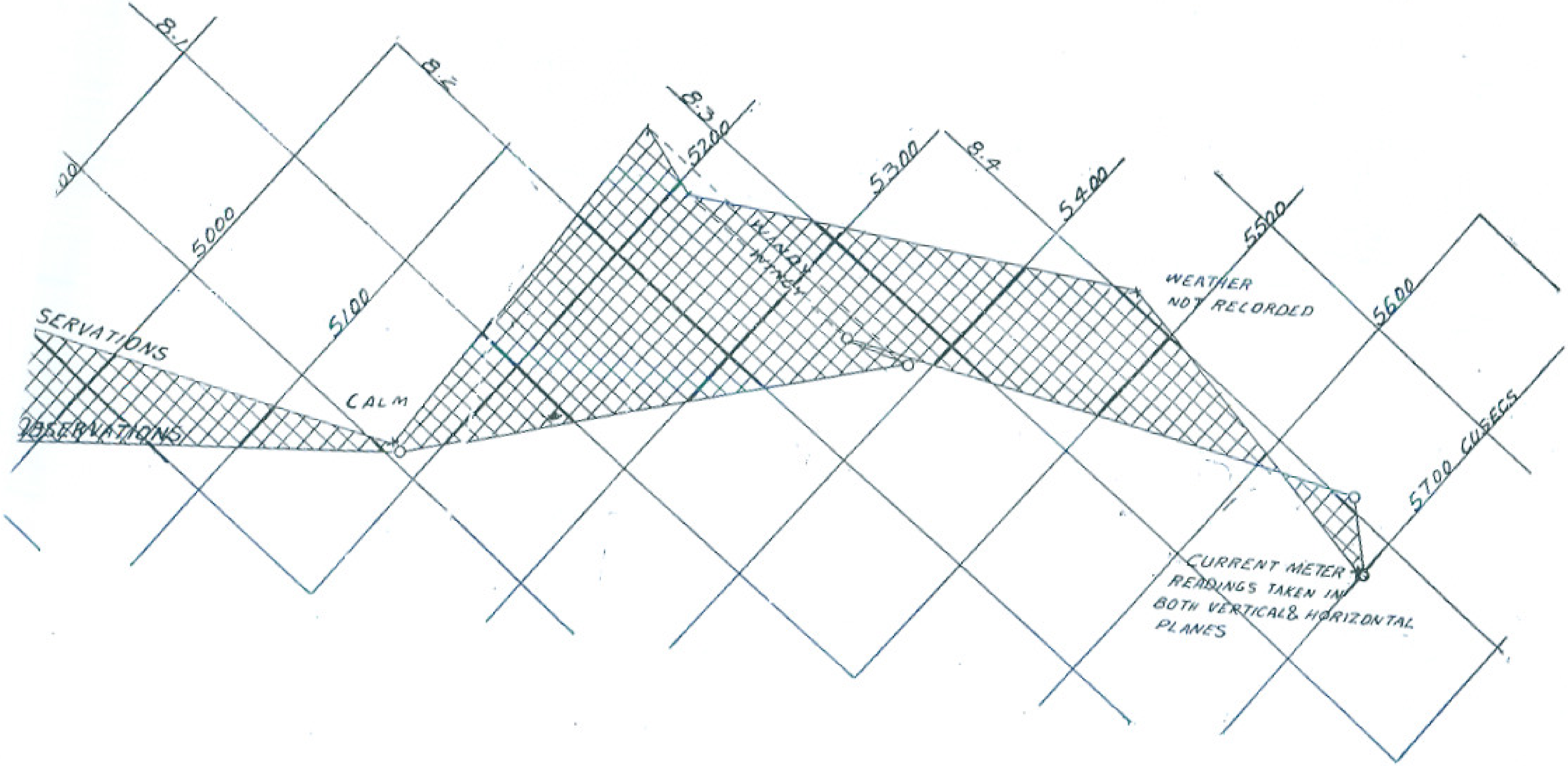
8·0
8·1
8·2
8·3
8·4
4700 CUSECS
4800
4900
5000
5100
5200
5300
5400
5500
5600
CURRENT METER OBSERVATIONS
DISCHARGE BRIDGE OBSER. TIONS

DIAGF

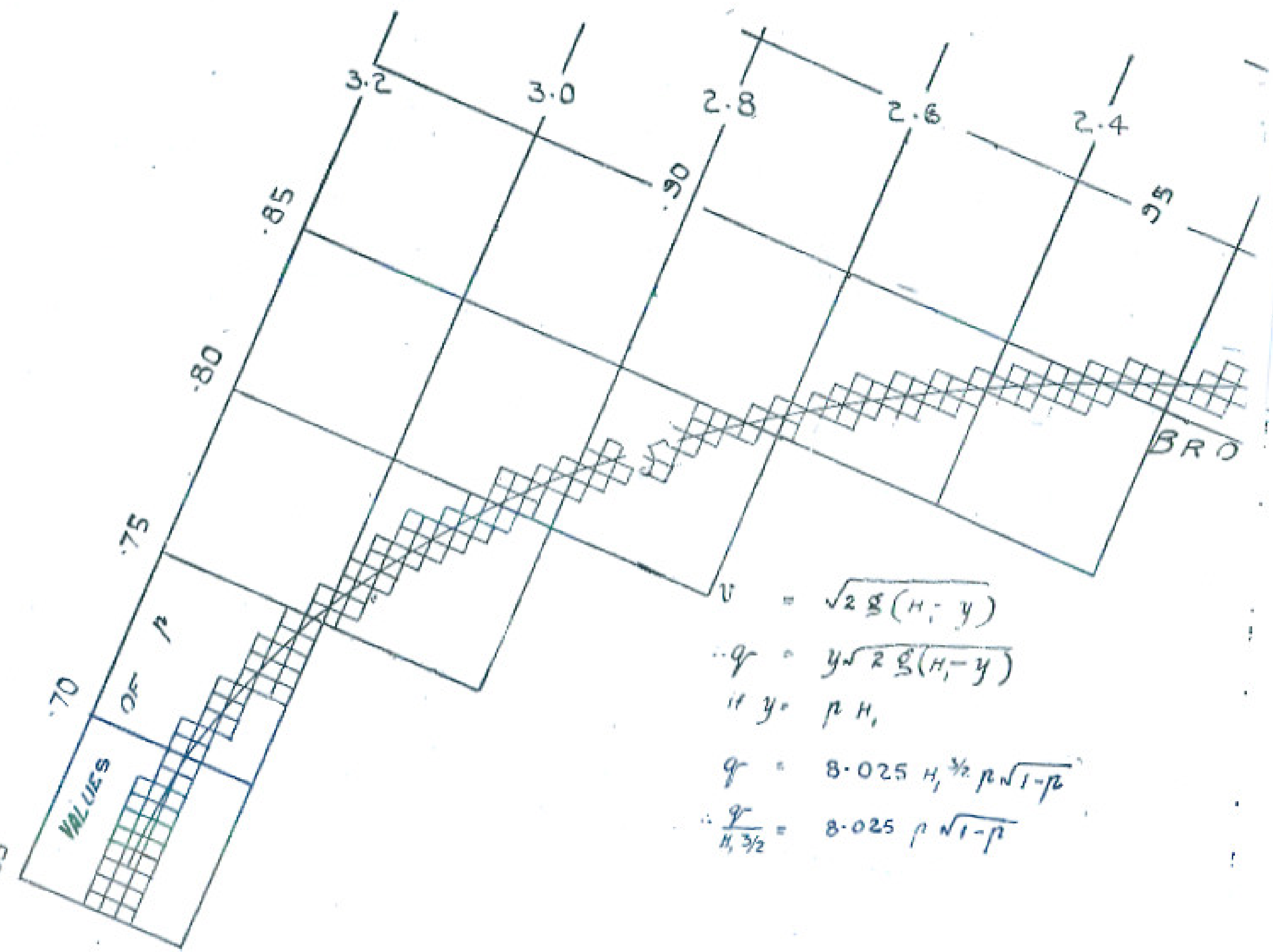


7.5
7.6
7.7
7.8
7.9
8.0
8
4200 CUSECS
4300
4400
4500
4600
4700
4800
4900
CURRENT METER OBSERVATI
DISCHARGE BRIDGE OBSERV
CALM
WINDY
WINDY

AGRAM I



8.1
8.2
8.3
8.4
5000
5100
5200
5300
5400
5500
5600
5700 CUSECS
SERVATIONS
OBSERVATIONS
CALM
WEATHER
NOT RECORDED
CURRENT METER
READINGS TAKEN IN
BOTH VERTICAL & HORIZONTAL
PLANES



$v = \sqrt{2g(H_1 - y)}$

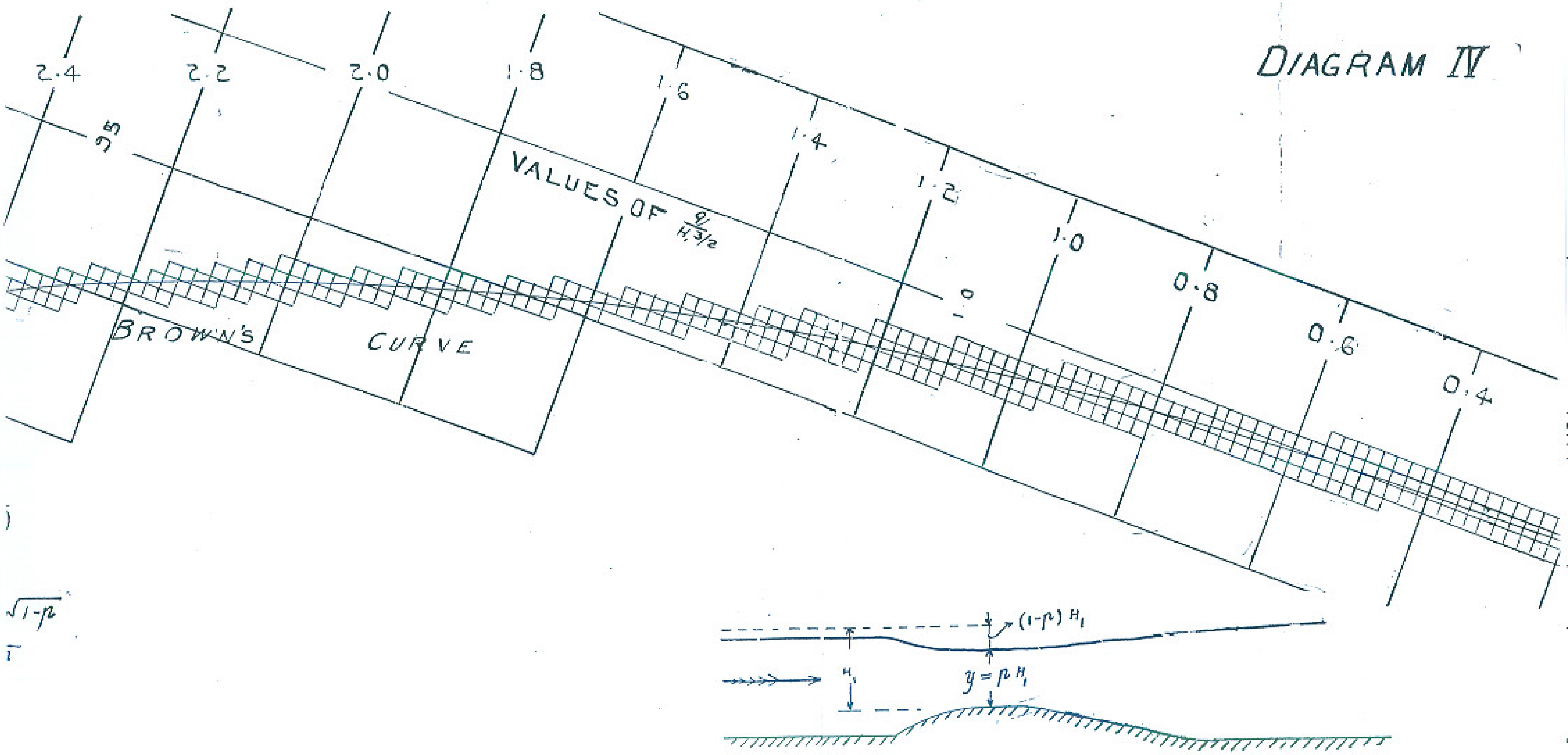
$\therefore q = y\sqrt{2g(H_1 - y)}$

if $y = pH_1$

$q = 8.025\, H_1^{3/2}\, p\sqrt{1-p}$

$\therefore \frac{q}{H_1^{3/2}} = 8.025\, p\sqrt{1-p}$

DIAGRAM IV



2.4
2.2
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
VALUES OF $\frac{q}{H_1^{3/2}}$
BROWN'S CURVE
$(1-p)H_1$
H_1
$y = pH_1$
$\sqrt{1-p}$

DIAGRAM V

$$\sin \phi_1 = \frac{X}{R_1}$$

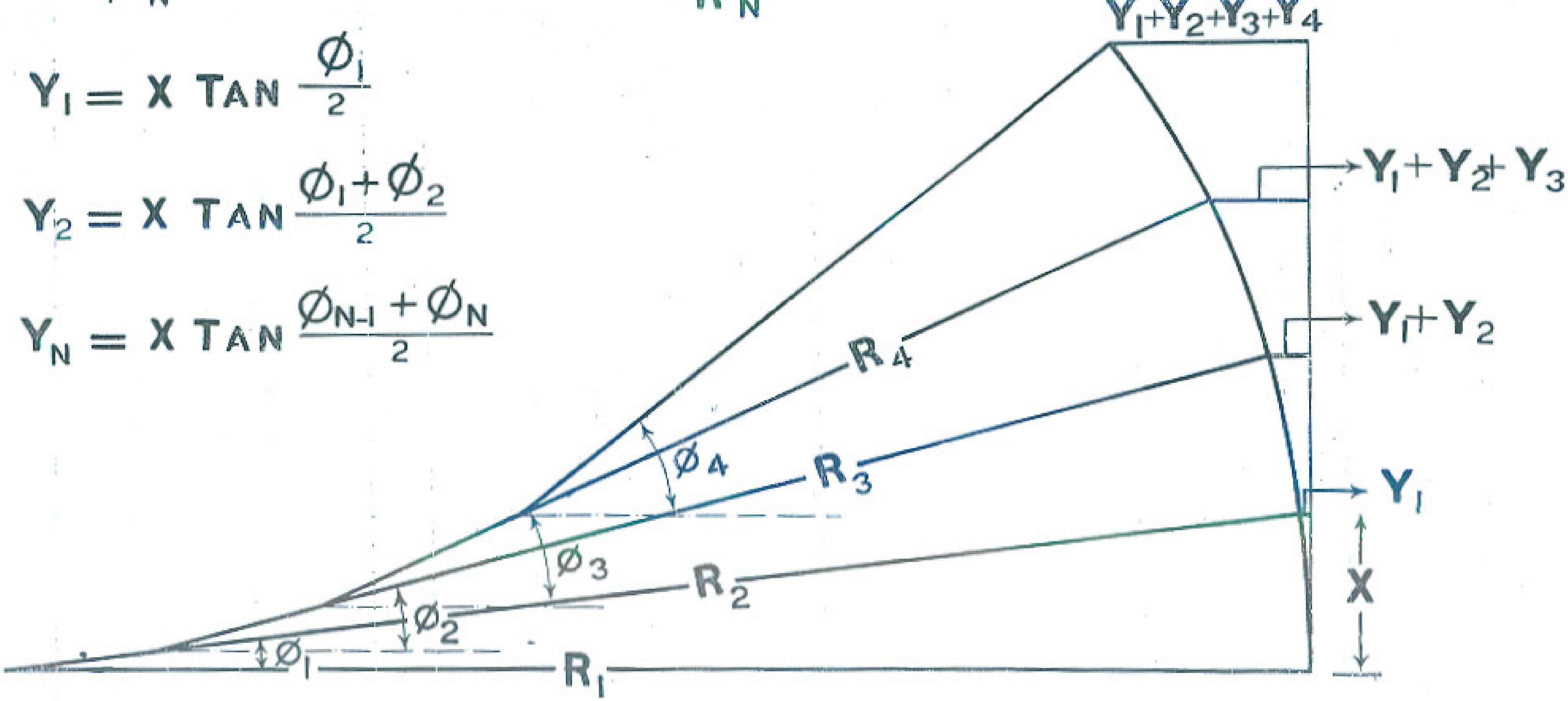
$$\sin \phi_2 = \frac{{}_2X - (R_1 - R_2) \sin \phi_1}{R_2}$$

$$\sin \phi_N = \frac{{}_NX - (R_1 - R_2) \sin \phi_1 - (R_2 - R_3) \sin \phi_2 - \cdots - (R_{N-1} - R_N) \sin \phi_N}{R_N}$$

$$Y_1 = X \tan \frac{\phi_1}{2}$$

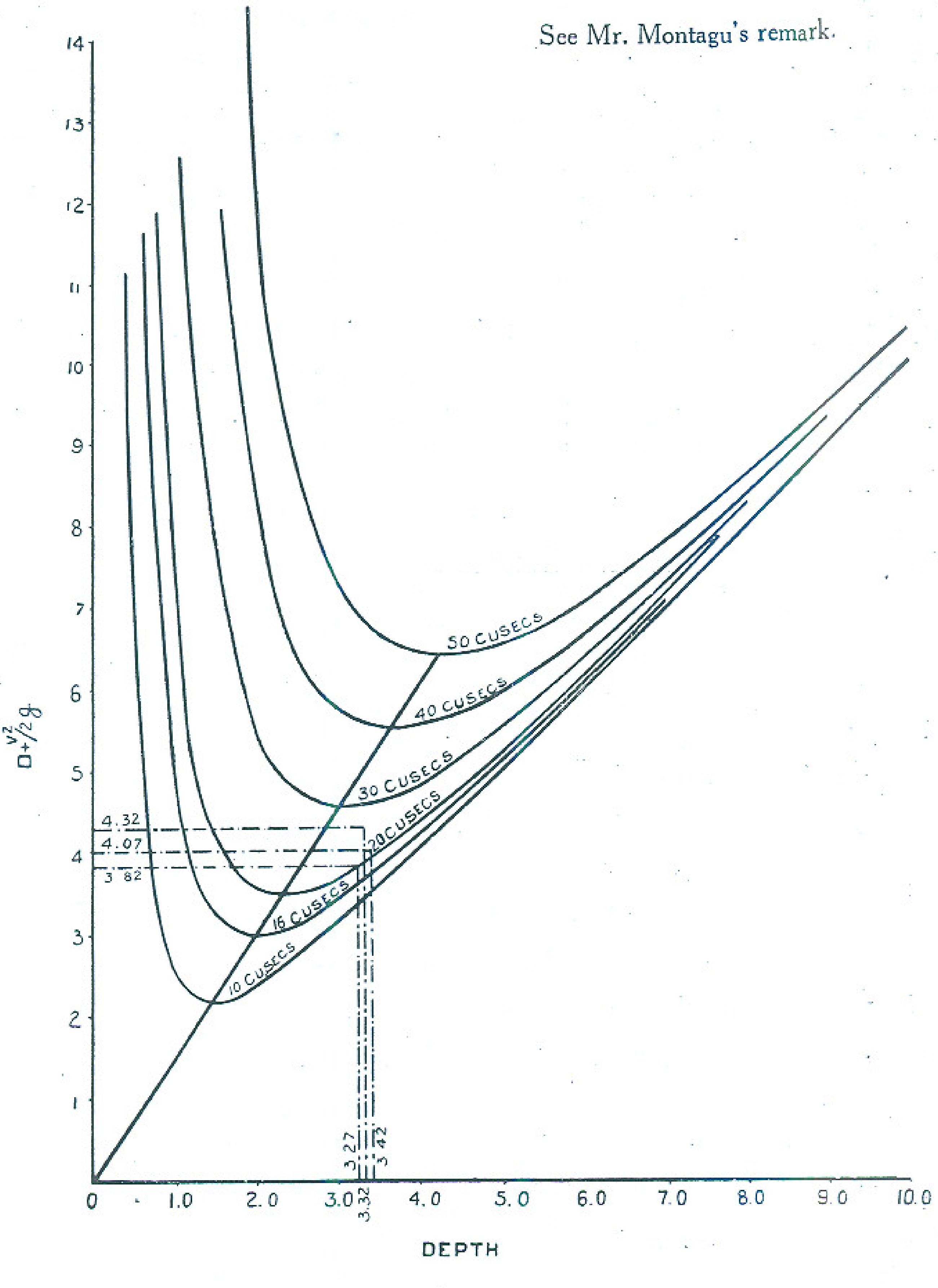
$$Y_2 = X \tan \frac{\phi_1 + \phi_2}{2}$$

$$Y_N = X \tan \frac{\phi_{N-1} + \phi_N}{2}$$



ENERGY OF FLOW CURVES

See Mr. Montagu's remark.



DISCUSSION.

MR. MONTAGU prefaced his remarks by saying that, adequately to discuss Mr. Burkittt's paper would occupy most of the remaining time of Congress. As this was not permissible his remarks must be so concise as to leave much to be understood.

2. Before discussing the details of Mr. Burkitt's design, he had to clear some ground.

Losses over any masonry work fell into four classes.

i. Energy lost in friction of the boundary.

ii. Energy lost in the standing wave, if any, or other localised turmoil.

iii. Energy lost in curvature even when the curvature of the boundary is theoretically accurate.

iv. Energy lost in the attempt of the stream lines to build up their own transition curves in cases where the curvature (or change of curvature) of the boundary were positively wrong. For purposes of this discussion he designated this as " loss due to shock ".

The sum of these energies is the difference in still pond level upstream and downstream.

3. In the formula

$$q = C H^{3/2}$$

used for such cases in which critical velocity and depth is reached, the constant has a true theoretical value.

Any variation of discharge from the value so derived, is due to an inaccuracy in determining the true value of H.

4. It follows that, when the downstream level is headed up, or (from any other cause) a distinct departure from the above value of the constant becomes apparent, then the condition precedent to the application of this formula (*viz.* attainment of critical depth and velocity) no longer exists.

Under such circumstances, to apply the formula and vary the coefficient is purely artificial and is radically incorrect.

5. In Mr. Fane's Paper No. 110 read before this Congress in 1927 the so called " free fall" formula was applied to "drowned conditions" and the constant so obtained was recorded. Mr. Fane went further and plotted a quantity which by nature is constant, against the " Drowning ratio ".

The speaker submitted that the process was illogical. He went further and held that the variable constants (a contradiction in terms), found, hold in each case only for the particular Weir, the particular discharges and the particular curvatures upon which the experiment were carried out, and are simply the result of applying the $H^{2/3}$ formula to a set of conditions to which it is not strictly applicable.

6. It was perfectly clear from Mr. Burkitt's paper that this was the case and that the author utilised Mr. Fane's results merely as a guide to the order of the total loss which be might expect.

The speaker was perfectly well aware of the reasons which led to the adoption by Mr. Fane of the method just described, but as Mr. Burkitt's results were clearly inconsistent with Mr. Fane's it seemed only reasonable to place the explanation on record.

7. On page 5 of his paper, Mr. Burkitt pointed out that the design was enormously complicated by depressing the central span of the Dipalpur Flume, with a view to unwatering the channel upstream. In the case of the Upper Sohag and Jalalabad branches a level crest had been designed, and a separate unwatering duct provided.

8. There are three distinct types of venturi flume design open to designers.

i. The obstruction may be in the bed, and its height may be designed to give a drowned weir, at any desired stage of discharge.

ii. The obstruction may be in the form of side contraction which will drown at any discharge, as per No. i.

iii. A combination of *i* and *ii.* In this case the floor may be depressed if desired and "free fall" conditions either entirely eliminated or permanently enforced as seems most desirable, by properly proportioning the bed.

9. The advantages of a wide crest are :—

i. Reduction of splay below the work.

ii. Low velocities.

iii. Simplicity and cheapness of construction.

iv. Low consumption of head.

The principal disadvantage is shallow depression and fineness of gauging.

The principal advantage of a narrow flume is the considerable depression available for gauging.

The disadvantages are the converse of the advantages of *i.* above.

The combination work, in conjunction with the depressed f admits of the velocity being anything desired and also the surface dep sion within wide limits.

10. Where a bridge has to be constructed the tendency will be wards a narrow flume and where no bridge is required, the broader flu appears indicated.

11. Having cleared the ground so far, the speaker desired to ask Mr. Burkitt for a further explanation in regard to one detail of his method of design. Why did he start off with the maximum available loss of head at minimum supply ?

It would appear more logical to deal with the loss of head through the work when maximum discharge is passing. As the discharge falls, loss due to friction would generally fall with the velocity, together with curvature and shock losses although, of course, the mean hydraulic radius will also change.

12. As the discharge continues to fall, the free fall condition is approached. There will be a phase in the discharge during which it will be uncertain as to whether free fall or drowned conditions prevail. Not only will the discharge be uncertain, but the precise position of this phase will vary with the silted conditions of the bed upstream and downstream.

Moveover if the discharge in the channel remains for any length of time at the precise point where the change of condition occurs, then fluctuation will be set up, more or less serious.

13. The most satisfactory state of affairs would appear to exist at low discharges when true free fall conditions obtain, and $q = 3{\cdot}09\ h^{3/2}$.

It was surely an important advantage to have only one gauge to read, a straight forward table, and no doubtful phase.

Mr. Lindley, expressed his views and those of Mr. Clemens Hershell, together with American engineers, more or less to the same effect, during the discussion on Mr. Fane's paper in 1927.

14. "Free fall" weirs can be constructed now with very little loss of head at full supply discharge, and the loss is of no importance at lesser discharges.

15. The speaker also enquired if Mr. Burkitt had had any observations taken to indicate if any difference existed in between the water surface levels in the gauge wells and the mean water surface levels in the stream opposite the gauge holes. Such differences, due to curvature might be expected, but in a work of this type where the velocities are low, would be small. It appeared from the *tabulated* results that if existing, they were negligible so far practical purposes were concerned and the question was one chiefly of academic interest.

16. Mr. Burkitt's calculations appeared to the speaker a little laborious, apart from the fact that they were based on the trial and error system. (He suggested two days as being a suitable period to allow for design).

He used several guesses at co-efficients derived from Mr. Fane's curves and avoided several more by using Mr. Brown's ingenious device. The speaker wondered this morning what would be the effect of using Mr. Montagu's curves for discharges per foot run.

He worked out the result over breakfast that morning and his results differed somewhat from Mr. Burkitt's. They are as follows :—

Data $q_o = \dfrac{654}{\text{mean bed}}$

mean bed $= 60{\cdot}5'$

Velocity $= 2{\cdot}16$ f p s

$\therefore$ y $= 5{\cdot}07''$

q_ε at entry $= \dfrac{654}{45} = 14{\cdot}5$ cusecs

D_ε ,, ,, $= 4{\cdot}93$ y $= 3{\cdot}07'$

Width at throat.	25′	30′	35′
q at throat ..	26·1	21·8	18·7
elevation of bed ..	0·75′	1·0′	1·25″
$\therefore y_t$ at throat ..	4·32′	4·07′	3·82″
$\therefore D_t$ (from diagram) ..	3·32′	3·42	3·27
$\therefore V_t$,, ,, ..	7·86	6·37	5·72
velocity head at entry ..	0·14	0·14	0·14
$\therefore y_t - D_t$ ..	1·00	0·65	·55
Depression ..	·86	·51	·41

17. He would point out that the speaker's curves were not influenced by co-efficients from experimental observations, so were free from that particular source of error. On the other hand they neglected curvature as did Mr. Burkitt.

He also neglected friction : this could be allowed for, by giving the "total energy" line, a slope based on the velocities and hydraulic mean depths calculated for the purposes, if meticulous accuracy was required.

But the designer only required the "order" of the depression and the *proportions* of the flume—of which there were an infinite number.

Mr. Burkitt's calculations gave no indication of proximity to the critical depth, whereas the speaker's method kept one fully informed.

18. Mr. Montagu went on to say that he was extremely interested to find that Mr. Burkitt had tackled the curvature problem (top of p. 3 of the paper). On p. 6 (last para but one) he had received a further hint, which did not enlighten him. The wording was not too clear.

He ventured to suggest instead, that "a change in the transverse velocity is caused by differences of pressure over the cross section." The normal reaction on the walls has two components (*i*) Longitudinal—which forms part of the sum of pressures along the line of flow and causes changes of momentum and all that such connotes.

(*ii*) Transversely,—which is only one item in the series of pressures that causes adherence to the wing wall.

19. The last para of p. 6 appeared to indicate that Mr. Burkitt's new theory is confined to cases of divergence of flat floors. In this case it will have but little general application because most hydraulic works combine sloping if not a curved floor, with the divergence.

20. Therefore, he turned to Diagram V. for further enlightenment. He felt that the case was nothing like so simple as Mr. Burkitt postulated.

The formula for angular acceleration may apply (he did not say it did) to a body of water as a whole, or even a pencil of water whose velocity and cross sectional area remains constant, but it certainly did not apply to a filament in which velocity and direction are constantly changing.

21. Moreover, on a flat floor and in a divergence, the slope of the water surface is not horizontal and the area of the surface of contract is constantly changing, hence the intensity of pressure.

Again, the pressure from top to bottom of any vertical line of contact is not constant nor even of uniform variation and owing to the curvature of the surface the pressure on the bottom is not represented by the depth.

22. Passing by the above considerations, important as they are, the velocity being considered is that tangential to the wall but Mr. Burkitt equated this to $\frac{Q}{A}$. The two bear no known relation.

23. There appeared to be no reason whatever why "f" should be constant. In fact the direct contrary appears to be the case, if they examined the velocities at the beginning and end of the divergence.

24. The pressure within the throat is easily calculated. The radius of curvature at this point is infinite.

Clearly Mr. Burkitt's formula did not hold within the throat.

If now the water emerges from the throat, it must take up a definite value of radius of curvature. The two are mutually incompatible.

25. He had studied Mr. Burkitt's Diagram V, and on the information given therein he had to confess the disappointment, and his conviction that the divergence problem remained yet to be solved.

There were a few points about which there could be no dispute in regard to divergence curvature, and he submitted them to Mr. Burkitt's consideration.

26. The radius of curvature, for all velocities within the throat, begins at infinity.

The radius of curvature decreases and the rate of decrease is dependent on the following factors.

i. Initial momentum.

ii. Value of friction.

iii. Curvature and slope of the floor.

iv. The desired amount of logitudinal component of pressure for change of momentum.

27. The above would be made clearer by a consideration of the limits.

(*a*) When the rate of change of the curvature is zero *i.e.*, when it remains a straight line, no expansion can take place.

(*b*) When the rate of change of curvature is a maximum, subject to the limits permitted by *i*, *ii* and *iii* above. In this case the pressure on the wall is, theoretically, zero although contract is established.

In such a case, a jet is on the point of forming.

28. Between these two cases any curve whose curvature varies regularly, will be suitable, so far as the expansion goes. But there will be one curve such that the longitudinal component of pressure is a maximum.

And this is the optimum wing wall, because the maximum longitudinal pressure ensures maximum rate of change of momentum. Therefore the desired total change of momentum is attained in the shortest distance.

The above remarks suitably modified, apply to converging approach walls also.

29. The speaker, was, however, deeply grateful to Mr. Burkitt for his demonstration that curved wing walls were the proper thing. He had said very much the same thing in the discussion on Mr. Fane's paper in 1927, but at that time he was obsessed by the energy equation.

Subsequent work on this subject has shewn that the curvature and the momentum were of first importance and the energy might be left to take care of itself.

30. In conclusion two isolated points deserve mention.

Calculation p. 11 para 4 (*a*).

Should not "H" be written "D/max" and in para 4 (*b*) should not "H" be written "D/min".

Mr. Bedford said that there were three points on which he would like to comment.

(1) *Accuracy.*—On page 4 of Mr. Burkitt's paper it is said that the accuracy of the headless meter was tested by comparing it with the results of current meter observations. On the next page the headless meter is stated to give five time the accuracy of the current meter. Now to compare a device by a standard, and then to say the device is five times as accurate as the standard is not convincing; nor is the fact of the discharge curve being regular in itself a proof of accuracy. Speaker would not raise the question of the feasibility of measuring to 1/1000 of foot.

(2) *Cost.*—It would be interesting to know the cost of a headless meter, so that it may be judged if the expenditure involved is justified.

(3) *Silting*:—Headless meters appear to have been evolved to provide an accurate gauging site for channels in which superfluous head does not exist. Suppose such a meter were to be put in a channel at mile 4 and this channel has just sufficient slope to insure regime and also the minimum working head from the parent channel or river. The meter is designed with a raised crest of 2·0′.

If this channel is run with a depth of 2·0′ at head of channel, before water can pass over the raised crest it is headed up and from the premises, the water is flowing with a silting velocity and it will silt. The silting may not merely be local but in the course of time may occur throughout the whole length of four miles and eventually become so serious as to make it impossible for the channel to take the full supply discharge. Channels with raised crests involving heading up have before now reached such a pass and it is a point that requires careful consideration when the question of such a headless meter arises.

L. BHIM SEN WADEHRA pointed out that Mr. Burkitt's general principles could also be applied to estimating the depression of water surface through a flumed bridge. If he put forward calculations showing that a depression of 1 foot could be obtained in a channel of 3·65 F. S. D. working on Fane's coefficient and asked Mr. Burkitt whether in his opinion this depression would be found in practice.

MR. TATE said that two meter flumes were constructed in the Bikaner Canal during the closure of last January, with the object not only of ascertaining the supplies entering the canal but also to determine the absorption losses in the concrete-lined channel.

These were designed by Mr. Burkitt at my request.

A large number of discharge observations have been made. One feature which has revealed itself is the necessity for reading the gauges on each side of the flume and arriving at the discharge from the mean of the two observations, as it is found that the results of each side seldom agree absolutely (The greatest variation recorded is 3% though most are well within 1%). These discrepancies are not due to inaccuracy of flume construction or in observation but appear to be due to inequalities of flow in the channel and a slight set of the current to one side or the other. It is to be expected that this feature would be more apparent in a comparitively wide and shallow flume, as is the case with the meters under review, which are 50 feet wide but where "H" seldom exceeds 6′. (I believe the same phenomenon has been noticed in the Dipalpur Canal meter, though to a lesser degree).

MR. Burkitt has suggested dividing the flume into two portions by means of a pier in the centre (or, better still, into three). But this would still necessitate two (or three) sets of readings and unless it can be shown that the present method of observing the discharge on each side of the flume and taking the mean does not give the true result, there seems nothing to be gained by making any alteration.

As regards loss of head through the flumes the few observations so far made show losses varying from 0·002′ to 0·046′. These were observed on one side only and, as in the case of discharges, it seems necessary to observe on both sides if the true loss of head is to be ascertained. (It will be possible to do this when gauge wells have been constructed on both sides).

Results to date are remarkably consistent, independent observers arriving at almost precisely the same figures of discharges. Moreover when supplies are sufficiently low for the flumes to work as free falls and, it is possible to check the discharge obtained by the application of the formula for flat-crested weirs, no appreciable discrepancy is apparent, the greatest variation in results to date being only about 0·03%.

Concerning Mr. Tate's remarks regarding the probable desirability of providing two flumes, he said that both Mr. Harvey and himself had found that twin-flumes caused transverse pulsation. He was not sure if the cause had been definitely established, but thought that the stream-lining of the downstream divide-wall would probably check this tendency.

MR. HADOW said that it had been argued that Mr. Burkitt had cramped his designs of the meters for the Dipalpur Canal on account of financial consideration. As far as the speaker remembered it was estimated that the combined bridge and flume would cost about Rs. 10,000 more than if an ordinary discharge flume and a bridge had been built separately. There was consequently no justification for this assumption, as it was considered that the experiment was worth the extra expense. After waiting for six months to see the results, he considered that the experiment was a success and several more meters had been put in other canals on the S. V. P.

In any case even an ordinary discharge flume could not be put in a large channel for a few hundred rupees, and the difference between a meter of this kind and an ordinary flume was not very much and was worth the money.

MR. B. N. SINGH, referring to the various plans attached to the paper said that in the case of the Eastern Canal the breadth of the crest at the Upper gauge is the same as that at the Lower gauge, while in the case of the Jalalabad Branch it was wider at the Upper gauge than at the Lower gauge, and asked Mr. Burkitt to explain what superiority the one had over the other and why a uniform system had not been adopted.

He also asked Mr. Burkitt to state the formula adopted by him fc the length of the crest because he found that it did not bear any defini relation to the depth of water above crest or to any other depth.

MR. COLYER said that this form of meter had been known for som time past as a probability, but that he thanked Mr. Burkitt for puttin it into a form accessible to the 'average engineer' thus rendering a grea service to the profession and to the Department.

Ten years ago in a paper read before the Congress Mr. Harvey had drawn attention to the need of metering canal supplies. This had been followed by the gradual introduction of broadcrested weirs into canals which possessed the necessary working head.

Mr. Burkitt's device had rendered practicable the establishment of meters in channels in which very little working head was available.

A crucial stage had been reached when the efficient distribution of the supplies from our already denuded rivers rendered absolutely essential the universal metering of all canal channels whether by broad-crested weirs or by other devices, and any reasonable amount of money spent on a meter was bound to be repaid by increased efficiency of distribution.

With regard to Mr. Montagu's remark that in the case of a broad-crested weir the wider crest gave the less action downstream, he disagreed, as he had himself been surprised to find that a 20 cusec weir, with no vertical contraction, *i.e.*, with only side-contractions, with a gentle glacis and the sides both flared and warped, had left a deposit of 6 inches of silt just downstream of the shallow cistern (which of course was of the 'Biff' type).

CORRESPONDENCE.

Mr. G. Lacey. The Sarda Canal had now been completed but the problem of the precise metering of its supplies remained yet for solution. Mr. Burkitt's paper therefore was to be welcomed as a valuable contribution to the subject. He had set out to demonstrate that a meter of considerable accuracy could be constructed in such a manner that the loss of head was negligible, and the works that he had actually constructed proved his contention. He was to be congratulated in putting theory successfully into practice. The methods of calculation which he adopted were admittedly laborious but discussion as to whether more simple methods might not prove just as effective (an elaboration of Julian Hind's very ingenious diagrams for example) should not detract from the utility of the device. With an extended use of such meters improved technique in design must of necessity follow.

From a preliminary examination of the meters it would appear that there was a certain loss of head due to shock between gauges and this vould be caused by re-entrant angles instead of smooth curves. The neter might be designated a "difference head meter" as this indicated he principle on which it was designed.

It was clear that the gauges and the differences in gauges need not e measured with the same degree of accuracy. For example an upstream uge of 5·05′ could be read very easily from an ordinary gauge pit. difference of say ·536′ was however a matter of difficulty. The iter would suggest that the upstream gauge be measured from a gauge and the differences in gauge measured by taking pipes from the up and nstream gauge pits and reading the differences by means of two vertical glass pipes placed side by side. They would be joined together at the top and the water surface brought up to any desired level by applying a small vacuum and then closing the union cock. The upper gauge less the difference in gauge and the height of crest would give the lower gauge and the discharge could then be calculated. Such a device was commonly employed when differences in pressures were required. The gauge could be read in comfort and oscillations in the water surface allowed for. Two or more gauge pits could be connected up and a correct mean value produced in one glass gauge. The piping should present no great difficulty. The writer intended checking a venturi meter recently erected on one of the distributaries on the Upper Gauges Canal by this method.

On the Sarda Canal it will be necessary to calibrate some of the falls by current meters and the writer was therefore somewhat alarmed to hear that current meter readings carefully taken may be as much as 5% in error. In Physical Department, Paper No. 24 of the Ministry of Public Works, Egypt published in 1928 the final conclusion recorded after checking twenty years of tank observations against current meters is " the proof that current meters are reliable instruments." The Report of the Mission to Lake Tana 1920—1921 published by the same authority gives a very interesting description of accurate current meter methods. It also refers *inter alia* to " a new and improved method of current meter rating adopted in 1921 by the Physical Dept., Egypt." The method will be found in Physical Department, Paper No. 14 of 1924, Egyptian Ministry of Public Works. The error in readings should be of the order of one per cent. instead of five.

From the nature of the aberrations in the current meter readings quoted by Mr. Burkitt there was some possible justification for the conclusion that the meter was more reliable than current meter observations as conducted at the site of the experiments, but the writer feels in view of the publicity given to the Congress Proceedings that the accuracy of current meters might be impugned by a casual reference to Mr. Burkitt's paper. Meters slung from wires or cables are subject to error as has been pointed out in Hogan's " River Gauging " (Dept. of Scientific and Industrial Research, England).

One great advantage of the " headless " meter was that it eliminated the velocity of approach difficulty which made the design of large broad crested weirs complicated. Mr. Inglis however (Bombay Technical Paper No. 15) had been very successful with standing wave flumes and had reduced the loss of head to a practically negligible amount.

Mr. Burkitt in reply to the discussion on his paper said that h wanted to retract the statement made in his paper that the central depres sed span in the Dipalpur Meter was an unnecessary complication. Usual for small supplies greater accuracy is obtained by a long raised we while for high supplies a contraction in plan gives the greater accuracy.

In a design where several spans are required, however, it is awkward to make a contraction in plan in each span. The alternative is either to have one such span contracted in plan, with a floor at bed level, or else to have the floor, at the site of the upstream gauge, below bed level. He, personally, had no objection to the latter alternative so long as the velocity was such as to be certainly a scouring velocity.

In reply to Mr. Bedford's contention that the accuracy of the Dipalpur Meter could only be gauged by comparison with current meter discharges and that therefore it was not possible to say which was the more accurate. Mr. Burkitt drew attention to the diagrams attached to his paper and also to his remarks on the subject in the paper itself.

As regards Mr. Bedford's fear that this meter, by flattening the slope upstream, might cause silting, he pointed out that, as the meter loses no head between the limits for which it is designed, no such flattening of the slope would occur between these limits. If the meter is designed with a raised crest then there will be a flattening for a few minutes during filling. If Mr. Bedford was still afraid of this condition then one span could be built with its floor at bed level as in the design of the Dipalpur Meter.

In reply to Mr. Montagu's statement that Mr. Fane's coefficients would not hold, generally, for high drowning ratios, he said that, upto date, there was nothing better than Mr. Fane's coefficients (which were obtained with small works) to go on. In the case of the Dipalpur Canal he could give no figures for "C" for high drowning ratios, as there the drowning ratio was generally measured to be greater than unity, any loss due to eddies being more than made up for by the deceased coefficient of rugosity.

Since writing his paper, however, he had been able to obtain some figures from the meters on the Bikaner Canal which is a concrete lined channel and where therefore there is no change in the rugosity coefficient.

So far he had obtained the following results from which members could plot a new curve connecting "C" with drowning ratio, and which would be a better one to use than Mr. Fane's for works of this nature :—

Up to 0·875 drowning ratio, c = 3,089. Drowning ratio 0·986, c = 2·63. Drowning ratio 0.993, c = 2·295. Drowning ratio 0·997, c = 2·23.

He considered that the labour involved in Mr. Montagu's proposed method of calculating the shape of the downstream wing walls would not be commensurate with any advantage gained. The experiments on the Dipalpur Meter showed, generally, a minute gain in head, and any further efinements could therefore accomplish little.

In reply to Mr. Montagu's question as to why (in the case of the alalabad Meter) he started off with the loss of head at minimum supply, e said the reason was purely a local one, there was no head to spare with nall supplies while there was nearly a foot of head available with large pplies.

He said there might be one condition, with a supply lower than the minumum designed for, when the discharge in any meter would not be accurately obtained, the condition being somewhere in the region of 0·9 drowning ratio.

Mr. Montagu advocated the use of a free fall in preference to a headless meter. With this opinion he was in entire agreement and wherever a free fall was obtainable it should of course be adopted. Mr. Montagu's statement that loss of head with supplies smaller than full supply is of no importance is certainly not correct in the case of the canals in the Sutlej Valley Project where there is always plenty of head with large supplies and none with small supplies.

In reply to Mr. B. N. Singh he said that the computation of the discharge with a slide rule was quicker if the breadths at the upstream and downstream gauges were the same, but, by adopting the same breadth at both places sufficient depression heads might not be obtained with large supplies, hence, in many cases, it is necessary to widen out at the upstream gauge.

He had not yet any formula for the length of crest, but suggested a minimum of $1\frac{3}{4}$H from the upstream edge to the gauge, and of half this amount from the gauge to the downstream edge.

Mr. Burkitt also observed that single large span would not, as a rule, be so accurate as a number of spans as the discharge might not be uniform across a large span (the same remark, of course, applying to a broad crested fall). And he said this type of meter should not be used for very small discharges till further experiments had been carried out as the effects of friction might vitiate the results. Moreover with very small discharges the depression head would have to be small to avoid approach to the critical depth.

In reply to Mr. Bhim Sen he expressed the opinion that one foot free board under the girders would not be obtained in practice owing to the fact that Mr. Fane's coefficient for high drowning ratios are two low. Taking the coefficients obtained into the case of the Bikaner flumes, the free board obtainable will only be eight inches or less.