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EXPERIMENTS WITH A SHORT-CRESTED WEIR MODEL

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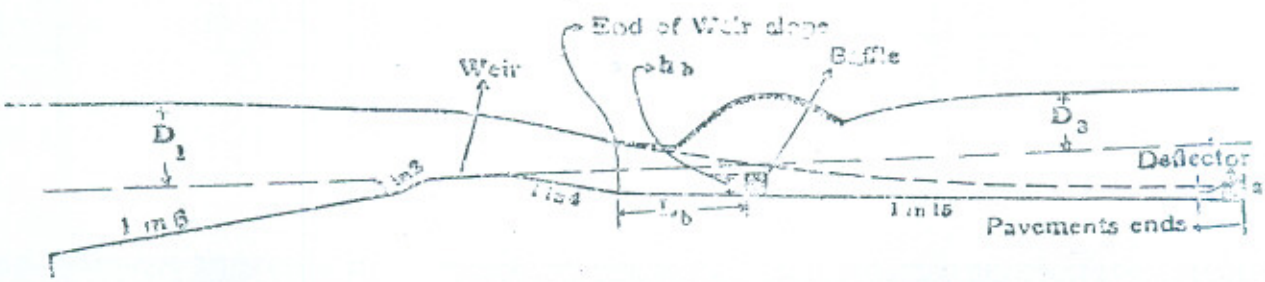
Introduction.

When in the Punjab recently I was asked by the officer in charge as to whether in a case in which a Standing wave formed far down on the pitching of a weir, it could be made to form near the weir by constructing a baffle of the same type as recommended for falls in Bombay P. W. D. Technical Paper No. 44 "The Dissipation of Energy Below Falls." The answer given was that it was certainly possible and that the best design would be determined by experiments. These were carried out in the 3 ft. experimental channel at the Hydrodynamic Testing Station at Distributary No. 5 of the Mutha Right Bank Canal near Poona with a model of a weir which, though long-crested at low supplies, became short-crested for large discharges.

The investigation necessitated only 4 days' experiments with a simple model of the weir. The case is one of general interest, not merely as regards dissipation of energy, but also as regards the discharge formula suitable for this common type of weir.

Notation and definitions.

FIG. 1



Q=Total discharge in cusecs.

q=Discharge per foot run of the weir crest in cusecs.

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

D_1=Depth of water upstream above the sill of the weir.

D_3=Depth of water downstream above the sill of the weir.

d_2=Depth of water at the end of the pavement of the weir (assuming free flow).

d_3=Depth of water in the river downstream of the weir.

h_b=Height of baffle.

L_b=Distance of baffle from the toe of the downstream slope of the weir.

R=Hydraulic mean depth.

h_1=Loss of head in friction.

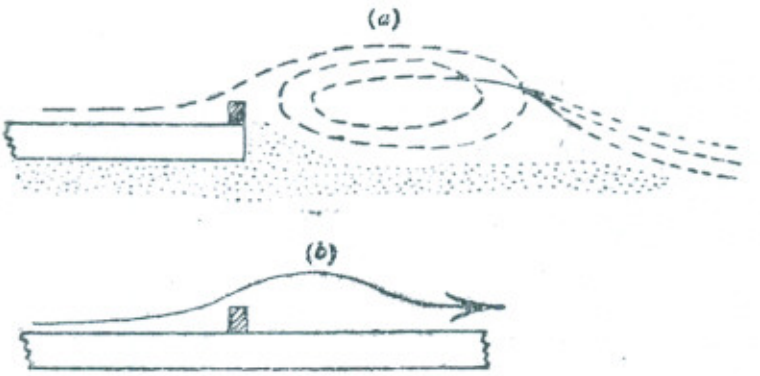
S =Surface slope.

N_a=(Lacey's) Rugosity coefficient.

A baffle (1) is a wall built downstream of the weir slope to dissipate kinetic energy.

A deflector (2) is similar in design to a baffle ; but its function is different. A baffle is primarily required to dissipate energy by generating small eddies, whereas a deflector deflects the high velocity stream towards the surface and causes a horizontal bed roller to form. Where protection is provided downstream of a deflector it should slope down at 1 in 5 so that a bed roller can form as shown in Fig. 2 (*a*). If the bed is flat—as in Fig. 2 (*b*)—no roller forms and the high velocity bed filament flows straight on.

FIG. 2



Description of the model.

The model was constructed in cement concrete. The longitudinal and vertical scale ratios of the model were 1/24, *i.e.*, the model was geometrically similar to the prototype.

The discharge constant=$\left(\frac{1}{24}\right)^{2\cdot8} = \frac{1}{822}$

(provided allowance is made for excess friction of model).

(1) and (2). For detailed description, *see* Bombay P. W. D. Technical Paper No. 44 " The Dissipation of Energy Below Falls," by C. C. Inglis and D, V, Joglekar.

Gauges were fixed upstream and downstream of the model so as to correspond with those in the prototype, *viz.*, 50′ upstream from the shutter line and 500 ft. downstream.

The discharge was measured by means of a standard 2 ft. standing wave flume at the head of the channel.

Experiments. Two series of experiments were carried out :—

Series I to find the discharge formula of the model, and

Series II to determine the best design of baffle and deflector.

The discharge formula.

In the model, the length of the weir crest in line of flow was 5,″ equivalent to 10 ft. (Fig. 3).

FIG. 3

Section of Weir Crest.

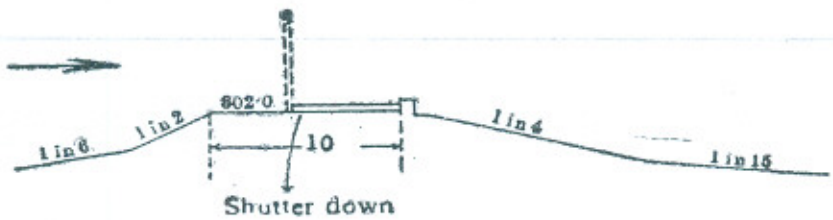


Table I shows the discharge observations of the model.

TABLE I.

Experiment No.	Q Cusecs.	q Cusecs.	D_1 ft.
1	4·86	1·620	0·587
2	4·38	1·460	0·551
3	3·40	1·133	0·481
4	2·41	0·803	0·400
5	2·01	0·670	0·362
6	1·55	0·517	0·309
7	1·00	0·333	0·241
8	0·802	0·267	0·208
9	0·732	0·246	0·196
10	0·659	0·220	0·186
11	0·525	0·175	0·161
12	0·382	0·127	0·134

The discharge formula for the model worked out to :

$$q = 4{\cdot}087\ D_1^{1{\cdot}764}$$

and $$q = 3{\cdot}628\ D^{1{\cdot}664}$$

Before a formula for the prototype can be worked out from the model formula a correction should be made for the excess friction in the model.

The friction in the model will be in excess for 2 reasons :

(*i*) The model was only 3 ft. wide =72 ft. in the prototype, whereas the spans of the prototype were some 500 ft.

(*ii*) Loss by friction varies inversely as $R^{1{\cdot}5}$ and hence the friction is relatively less in the prototype than in a geometrically similar model.

The way to calculate excess friction losses is explained in Appendix I.

When friction is allowed the formulæ becomme

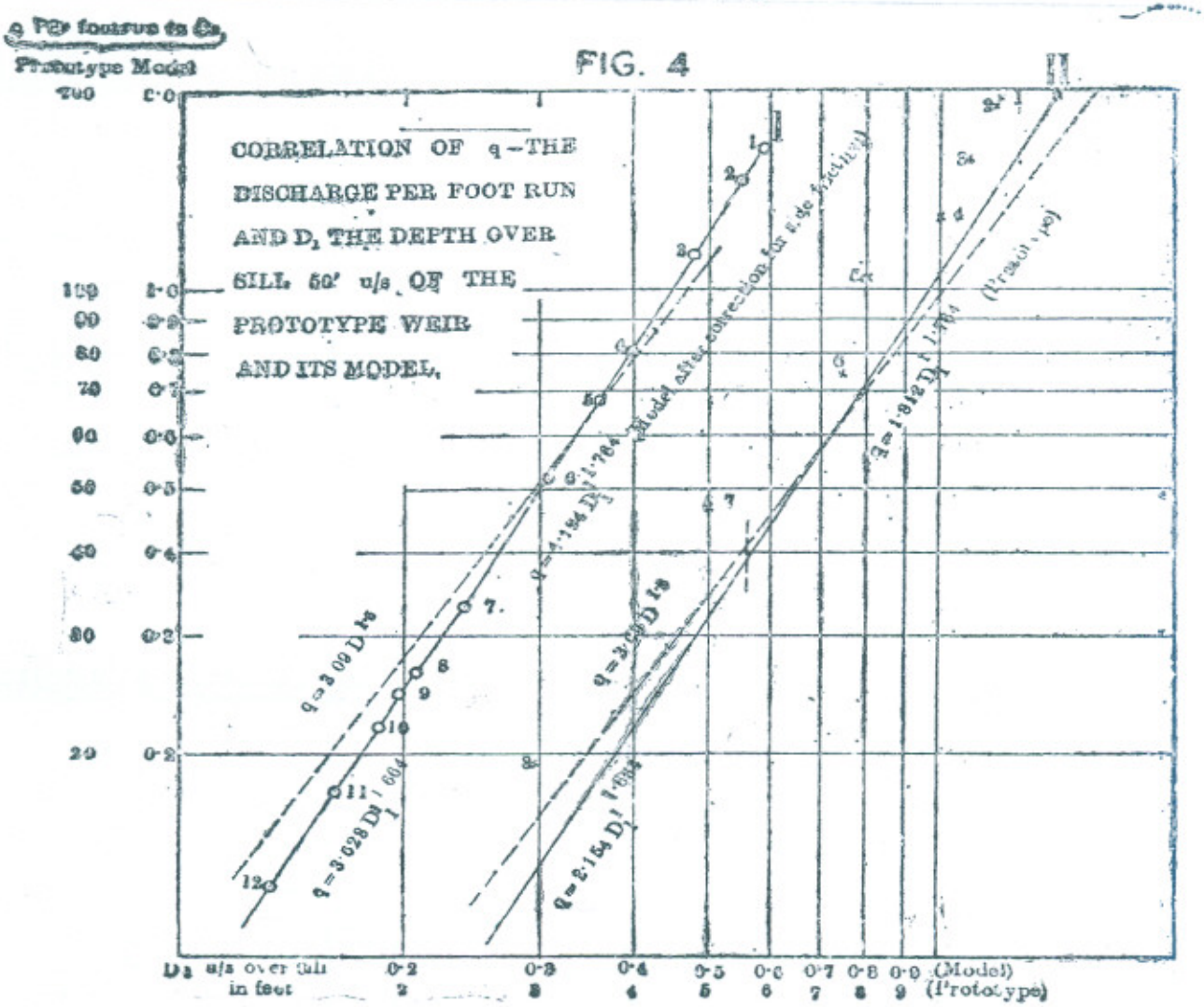
$$q = 4{\cdot}194\ D_1^{1{\cdot}764}$$

and $$q = 3{\cdot}628\ D_1^{1{\cdot}664}$$

and the point of intersection of the curves occurs at $D_1 = 0{\cdot}235' = 5{\cdot}64$ ft. in the prototype.

These are the formulæ of Curve I in Fig. 4.

FIG. 4



We can now work out the corresponding formulæ for the prototype on the assumption that flow in the model and prototype are dynamically similar.

We know that in the case of long-crested weirs, the formula is very approximately

$$q = 3{\cdot}09\ D1{\cdot}5$$

irrespective of size—when allowance is made for friction losses—, but in this case the upstream slope was sharp (1 in 2) and the shutter supports projected above the sill, consequently the exponent was more than 1·5, while the discharge for depths upstream of 5 ft. (=L/2) and less, were less than $3{\cdot}09\ D_1^{1{\cdot}5}$.

Using Q ; q and D_1 for the prototype and Q′ ; q' and D_1' as the corresponding figures for the model ; and S as the scale ratio :

Then assuming dynamic similarity the discharge for any depth in the model can be written $q' = KD'^{1{\cdot}5}$ and the corresponding discharge in the prototype as $q = KD^{1{\cdot}5}$ and $D^1 = D/S$ and $q' = q/S^{1{\cdot}5}$ but $q' = C'\ D'_1 x$.

So substituting

$$\frac{q}{S^{1{\cdot}5}} = C^1\ (D_1/S)^x \text{ in which } x \text{ may be greater than } 1{\cdot}5.$$

So $q = C'\ (S^{1{\cdot}5-x})\ D_1 x$ but $q = CD_1 x$ therefore $C = C'\ (S^{1{\cdot}5-x})$ .....(1)

Or as $x > 1{\cdot}5$, $C = \frac{C'}{S\ (x-1{\cdot}5)}$.

Thus the coefficients are the same for the model and the prototype when the exponents are 1·5 ; but when the exponents are $>^{1{\cdot}5}$ the coefficient in the prototype is less than in the model.

Applying this to our particular case we find for

Case (*i*) $D_1 > 0{\cdot}235'$ (>5·64 ft. in the prototype).

$q' = 4{\cdot}194\ D_1'^{\,1{\cdot}764}$ in the model deducting excess friction losses.

$$\text{So } \frac{q}{(24)^{1{\cdot}5}} = 4{\cdot}194\ (D_1/24)^{1{\cdot}764}$$

$$\text{or } q = \frac{4{\cdot}491 \times (24)^{1{\cdot}5}}{(2{\cdot}4)^{1{\cdot}764}} \times D_1^{\,1{\cdot}764}$$

$$= 1{\cdot}812\ D^{1{\cdot}764}$$

and in Case (*ii*) $D_1 < 0{\cdot}235'$

$$q' = 3{\cdot}628\ D_1'^{\,1{\cdot}664}$$

$$\text{or } q = \frac{3{\cdot}628 \times (24)^{1{\cdot}5}}{(24)^{1{\cdot}664}}\ D_1^{\,1{\cdot}664} = 2{\cdot}154\ D_1^{1{\cdot}664}$$

Vide Curve II in Fig. 4.

[It may be noted that the scales for q and D_1 of the prototype in curve II are not relatively similar to those of the model in curve I, therefore $D_1 = 5{\cdot}64$ ft., in the prototype and the corresponding $D_1^1 = 0{\cdot}235$ ft. in the model do not cut the horizontal at the same place. The scales used were adopted for simplicity in plotting and reading.]

It will be seen that when the discharge for particular approximate-gauge-readings upstream—supplied to us, *vide* Table II below are compared with curve II—*vide* Fig. 4—, they do not tally and give discharges 30 °/₀ to 50 °/₀ more than would be possible if the values of the upstream depths were for similar conditions to those in the model.

These differences might be due to a high velocity of approach in the prototype or to incorrect discharge observations or to the gauge readings where observed not representing the true effective depths above the weir.

Whatever the explanation of the discrepancy may have been, the position of the standing wave below the weir depends only on the discharge and so the experiments were carried out on a discharge basis.

TABLE II.

Observation No.	Q Cusecs.	q Cusecs.	d_1 ft.	D_1 ft.	D_1 ft.	D_3 ft.	D_3/D_1
					*	*	*
1	718,000	191·0	22·0	12·10	16·52	8·35	0·69
2	700,000	186·0	21·9	12·0	16·67	8·5	0·71
3	600,000	156·0	20·9	11·0	15·67	7·5	0·68
4	500,000	129·0	19·9	10·0	14·57	6·4	0·64
5	400,000	102·0	17·9	8·0	13·47	5·3	0·66
6	300,000	74·0	17·4	7·5	12·37	4·2	0·56
7	200,000	47·0	14·9	5·0	11·17	3·0	0·60
8	100,000	19·5	12·9	3·0	9·37	1·2	0·40

* Approximate figure.

Position of standing waves.

The maximum discharge recorded in the prototype weir was 718,000 cusecs. The equivalent model discharge was

$$\frac{718{,}000}{3{,}759} \times \frac{3 \times 24}{2822} = 4{\cdot}86 \text{ Cusecs ;}$$

or $q^1 = 1{\cdot}62$ cusecs ; and the experiments were carried out with this discharge.

The water level for the discharge calculated to be 718,000 cusecs 50′ upstream was 814·10 in the prototype and the downstream water level was approximately 810·35.

The sill of the weir =802·0.

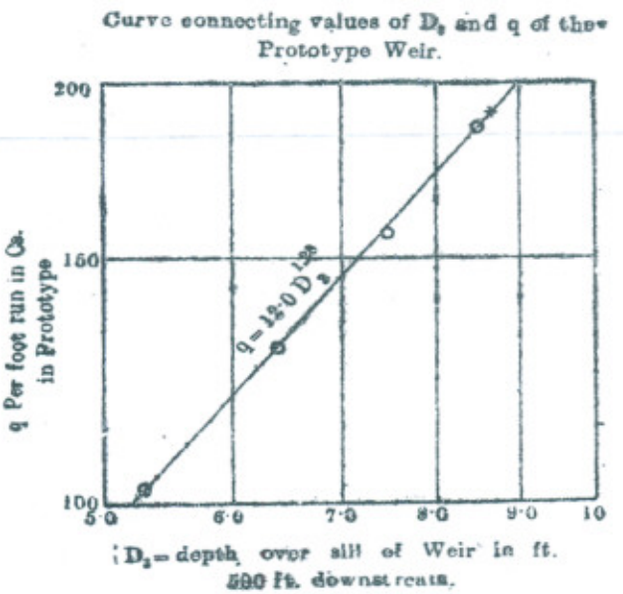
Figure 5 shows the correlation between q and D_3 of the prototype, D_3 being measured 500′ downstream of the weir.

By extrapolating the curve shown in Fig. 5, the value of D_3 for q =191·0 cusecs or Q=718,000 cusecs was found to be 8·68 ft. as against the approximate figure of 8·35.′

The equivalent water level in the model was 100·36, with the sill at R. L. 100·00.

With this water level downstream and Q=4·86 cusecs the upstream water level was 100·587 or D^1=0·587 (=12·10 in prototype approx.).

FIG. 5



The following experiments were carried out: Experiment 1 without a baffle or a deflector D_3=·36′(=8·68′) (*See* Figs. 1 and 2 of Plate 1).

Experiment 2 with no baffle, but a deflector at the end of the pavement (Figure 1 of Plate II).

Experiment 3 with a baffle and deflector (Fig. 2 of Plate II).

Local silt was spread on the bed downstream of the pavement level; and silt contours were observed at the end of one hour (equivalent to five hours in the prototype, the time-scale ratio being $\sqrt{24}$).

Experiment 1 was continued for 6 hours (=29½ hours) to see if the pavement would be undermined and the extent of scour.

It will be noted that the depth of scour which occurred in 1 hour was a maximum when the pavement was not protected either by a baffle or a deflector as shown in Table III below :—

TABLE III.

	Maximum depth of scour below pavement after model run for 1 hour equivalent to 5 hours flow in the prototype.
	Model.
Without a deflector or a baffle	0·85′
With a deflector	0·71′
With a baffle and a deflector ..	0·60′

Fig. 1 of Plate I and Figs. 1 and 2 of Plate II show the results in detail. The height of the deflector was 0·07′ (=1/10th the depth of water in the river downstream=1·68′ in prototype) and the height of the baffle was 2″ (4′ in the prototype)=0·56d_2 where d_2=·295 is the free flow depth at the baffle position (without baffle fixed). The baffle was constructed 15″ away from the toe of weir of the model=30′ from the weir in the prototype (=4·2d_2)

The optimum height of baffle=d_2 and the optimum distance from the toe=5·25d_2 ; but the position of the baffle was fixed by the line of wells and even with the top of the baffle, at R. L. 99·96—*i.e.*, 0·04′ lower than the sill—the upstream water level with Q equivalent to 718,000 cusecs was affected ; so a higher baffle could not be used.

When the model was run for 6 hours without a baffle or a deflector, the scour went on increasing and the concrete bed of the channel was exposed.

The pavement was not, however, undermined but this would occur if there were any set of the current towards either bank—as is usual at weirs.

The slope of silt just downstream of the deflector in Experiments 2 and 3 was approximately 1 in 5, which has been found to be the approximate natural protective slope below a deflector in all our experiments.

Positions of standing waves were observed with various water levels downstream in the three experiments, the results are shown in Table IV.

The position of the primary wave over the baffle and of the secondary wave downstream were observed and are shown in Plate II for one condition.

TABLE IV.

Q=4·86 cs. (=718,000 cs.) Sill of prototype weir=802·0. Upstream water level=100·587. Sill of model=100·00. Length of pavement from toe=40″ (=80′).

Water level downstream.		Without a baffle or a deflector		With a deflector.		With a baffle and a deflector.			
		Position of S. W. downstream of toe.		*Position of S. W. downstream of toe.*		*Position of primary wave d/s of toe.*		*Position of secondary wave d/s of toe.*	
Model.	Prototype.	Model.	Prototype.	Model.	Prototype.	Model.	Prototype.	Model.	Prototype.
100·42	812·08	15″	30′	8″	16′	1½″	3′	22″	44′
100·36	810·64	24″	48′	17″	34′	1½″	3′	24″	48′
100·32	809·68	31″	62′	24″	48′	..	..	..	..
100·30	809·20	34″	68′	26″	52′	1½″	3′	30″	60′
100·28	808·72	40″	80′	30″	60′	..	..	..	..
100·25	808·00	78″	156′	66″	132′	1½″	3′	33″	66′
100·22	870·28	..	...	...	...	1½″	3′	35″	70′

Note.—The secondary wave is a surface wave which kills energy and is not associated with bed scour.

It will be seen that the baffle held the wave within 1½″ (=3 ft. in the prototype) from the toe of the downstream slope of the weir over the whole range tested—even for a retrogression of water levels of 3.36 ft.—whereas, without a baffle or a deflector, the standing wave would form dowstream of the pavement, with a retrogression of 1·92′.

7. Design of baffle deflector below weirs.

Although the baffle and deflector as shown in Figure 1 was successful, yet if a new weir were being constructed, the design should be slightly different as shown in Fig. 6.

This design differs from that shown in Fig. 1 in that—

(1) the 1 in 4 glacis is taken right down to the pavement level below the weir, instead of the original design of 1 in 4 and 1 in 15 glacis slopes.

FIG. 6



With this level, a standing wave would form at the toe downstream of the weir under maximum flood conditions—which is the optimum pavement level at which the baffle can be fixed. (For calculations, *see* Appendix II.)

The advantages of steep glacis slope and a flat pavement are described in Bombay P. W. D. Technical Paper [3] No. 29.

The way to prepare the best design is explained in detail in Technical Paper No. 44 ; the following points should be noted :—

(1) The baffle should be designed for full supply conditions.

(2) Should any retrogression of water levels occur downstream of a weir, the baffle will still hold the standing wave near the downstream toe of the weir glacis and thus protect the pavement, but if retrogression is anticipated, it would be wise to lower the pavement downstream of the baffle by the equivalent amount—*vide* dashed line in Fig. 6.

(3) The baffle is most effective when the pavement is built at such a level that a standing wave would form naturally at the toe of the glacis without a baffle.

(4) If a deflector is constructed at the end of the pavement it causes a horizontal bed roller to form as a result of which the silt banks up against the downstream of the deflector at a 1 in 5 slope—*vide* Fig. 6. If protection is provided it should not be laid flat, but should slope down at 1 in 5 downstream of the deflector.

(5) Baffles and deflectors should be provided below all weirs.

Continuation.

Subsequently to writing the previous Note, it was suggested that it was desirable to test the effect of omitting the baffle and constructing a deflector at the end of the concrete blocks and it was also suggested that it was desirable to determine the effect of the baffle on the pressures

[3] Bombay P.W.D. Technical Paper No. 29 " Note on experiments carried out with various designs of Piers and Sills in connection with the Lloyd Barrage at Sukkur " by C. C. Inglis and C. G. Hawes.

acting on the pavement. Experiments were, therefore, carried out in continuation of Experiments (1), (2) and (3) of the original Note.

Experiment 1 of the original Note : Without any baffle or deflector—Fig. 9.

Experiment 2 of the original Note : No baffle and deflector on 'C' line of wells—Fig. 10.

Experiment 3 of the original Note : Baffle 2" high on 'B' line of wells and deflector on 'C' line—Fig. 11.

[In Experiments 1 to 3, concrete blocks below 'C' line were omitted to ascertain limit of scour.]

Experiment 4 :— No baffle, and deflector at the end of horizontal concrete blocks—Fig. 12.

Experiment 5 :— Baffle 2" high on 'C' line and deflector at the end of blocks—Fig. 13.

Experiment 6 :— Baffle 2" high on 'B' line and deflector at the end of concrete blocks—Fig. 14.

[Figs. 9 to 14 show scour after 1 hour's flow, equivalent to 5 hours in the prototype with discharge per foot run=1·624 cusecs, equivalent to 191 cusecs per foot run in the prototype of a total discharge of 718,000 cusecs. Statement I shows the depth of scour below the pavement level along the centre line of the channel.]

Experiment 4—as would be expected—gave results which were practically identical with those obtained in *Experiment* 2 in which the deflector was fixed further upstream over the 'C' line of wells.

Experiment 5—adding a baffle 2" high on 'C' line—did not reduce scour, but moved the trough of the standing wave upstream and this increased the depth of water and hence the pressure on the pavement. The optimum position of the baffle from the downstream toe of the weir is 5·25 d_2 against 12 d_2 in this experiment—d_2 being the free flow depth downstream—; hence the baffle in this experiment was not so effective in dissipating energy as in *Experiment* 3.

Experiment 6 was therefore, carried out with a baffle 2" high on the 'B' line of wells—the optimum position—, and with a deflector at the end of the concrete blocks. This experiment gave the best result obtained.

Comparing the scour of *Experiments* 2 and 4 (with a deflector only) with that of *Experiments* 3 and 6 (with a baffle and deflector), it will be seen that the baffle only dissipates energy and brings the standing wave near the downstream end of the weir, thus increasing the depth of water on the pavement, but it also stabilises the flow and checks any tendency to cross flow. This is important, as cross flow is always liable to start severe local scour.

Table V shows the slope at which sand (just downstream of the pavement) became stable in Experiments 1 to 6.

TABLE V.

Experiment No.		Natural slope of sand just downstream of deflector.
1	Without a baffle or a deflector ...	0·21 in 0·5 or 1 in 2·4
2	Deflector on 'C' line ...	0·15 in 0·5 or 1 in 3·3
3	Baffle on 'B' line and deflector on 'C' line.	0·12 in 0·5 or 1 in 4·2
4	Deflector at end of blocks ...	0·12 in 0·5 or 1 in 4·2
5	Baffle on 'C' line and deflector at end of blocks.	0·16 in 0·5 or 1 in 3·1
6	Baffle on 'B' line and deflector at end of blocks.	0·16 in 0·5 or 1 in 3·1

It will be noted that wherever the deflector is fixed the material downstream will tend to scour to a slope of about 1 in 3 to 1 in 4 and for the deflector to be fully effective this scour should be allowed to occur and pitching should be laid to the natural slope.

The results are in general agreement with those obtained by Messrs. Butcher and Atkinson.[4]

Pressure Observations.

TABLE VI.

	Expt. (1) *Without a baffle or a deflector.*	Expt. (4) *With a deflector* (0·07′ high) at the end of block.	Expt. (6) With a 2″ high baffle on 'B' line of wells and *a deflector* (0·07′) at the end of block.	
	Depth of water ft.	Depth of water ft.	Depth of water ft.	Pressures observed.
At toe ..	0·390	0·390	0·490 (u/s of S. W.)	0·53
3″ d/s of toe ..	0·344	0·344	0·580	
5″ ,, ,, ..	0·347	0·320	0·600	
7″ ,, ,, ..	0·311	0·320	0·640	
9″ ,, ,, ..	0·306	0·310	0·680	
11″ ,, ,, ..	0·299	0·320	0·680	
13″ ,, ,, ..	0·296	0·320	0·650	
15″ ,, ,, 'B' line of wells.	0·295	0·340	0·640 0·630	0·70 just u/s of baffle. 0·42 just d/s of baffle.
18″ ,, ,, ..	0·291	0·360	0·600	
20″ ,, ..	0·285	0·380	0·520	
24″ ,, ,, ..	0·285 (u/s face of S. W.)	0·400 (u/s of S.W.)	0·480	0·56
27″ ,, ,, ..		0·450	0·420 (u/s face of 2nd wave d/s of baffle).	
30″ ,, ,, ..	0·450	0·520	0·580	
38″ ,, ,, 'C' line of wells.	0·605	0·620	0·670	0·70

[4] "The causes and prevention of bed erosion, with special reference to the protection of structures controlling rivers and canals" by Butcher and Atkinson—Paper No. 4865. Minutes of Proceedings of the Institute of Civil Engineers, Vol. 235, Session 1932-33, Part I.

Table VI shows the pressures observed near the bed along the middle of the channel and the depths of water in *Experiments* 1, 4 and 6.

The pressures observed in *Experiment* 6—with a 2″ high baffle on 'B' line of wells and 0·07′ deflector at the end of concrete blocks—are shown in Statement II and Plate III.

In Fig. 2 of Plate III—

'*a*' shows the water curve and '*b*' shows the pressure curve along the centre line of the channel ;

'*c*' shows the water curve in *Experiment* 1 without a baffle or a deflector and '*d*' shows the water curve in *Experiment* 4 with a deflector at the end of concrete blocks for comparison with curve '*a*' of *Experiment* 6.

Consideration of results.

(1) Without a baffle or a deflector a standing wave forms downstream of the 'B' line of wells and the depth of water (and hence the pressure on the pavement) is much lower than with a baffle and a deflector—*vide* L. Section along the centre of the channel in Plate III.

(2) With a deflector at the end of the blocks the depth of water is more than without a deflector ; but is considerably less than with a baffle and a deflector.

(3) Where the flow is parallel and in straight lines, the pressure head is equal to the depth of water—this was approximately the case in cross-sections I, IV and V of Plate III. Immediately upstream and downstream of the baffle—cross-sections II and III—the flow was curved and the pressure head at the bed was in excess of the depth of water upstream of the baffle, whereas downstream, it was in defect. (Pressure curve assuming parallel flow is shown in full line and the pressure curve actually observed is shown in dotted line in Fig. 3 of Plate III.)

The flow conditions produced by a correctly designed baffle are shown in Fig. 15.[5]

[5] *See* Bombay P.W.D. Technical Paper No. 44 "The Dissipation of Energy below falls" by C.C. Inglis and D.V. Joglekar.

Final conclusions.

Quite apart from the danger of a deflector constructed at the end of the blocks being washed out, the desirability of a baffle constructed on the 'B' line of wells is undoubted—not merely to bring the wave to the toe, and so increase the pressure on the pavement, but even more to stabilise the flow.

The best design is a combination of a baffle 2″ high (=4 ft.) on the 'B' line of wells with a deflector 0·07′ high (=1·68 ft.) either at the end of the blocks if this can be relied on ; or, if it cannot be trusted, then on the 'C' line of wells. In either case the protection immediately downstream should slope down at 1 : 3.

A deflector alone does not keep the wave at the toe—and hence pressures on the pavement are low, nor does it kill the kinetic energy at the earliest possible moment.

The baffle, besides killing the kinetic energy and stopping the high velocity jet along the bed—which is the main cause of serious damage when it occurs—also checks cross flow and return flow.

APPENDIX I.

Loss of head in friction in the model and prototype.

FIG. 7



Q=4·86 cusecs ; and q=1·620 cusecs.

We need only consider the loss of head in friction from point C (position of upstream gauge) to point A (*i.e.*, the downstream end of the crest of the weir) ; because friction downstream of A does not affect the discharge.

Model.

Section at A—the downstream end of weir crest.

Depth =0·442 ft.

Width B= 3 ft.

Area= 1·33 sq. ft. ; P=3·88 ft. and

R=A/P=0·34

v=3·66 f/s.

Section at B—the upstream end of the weir crest.

Depth=0·522 ft., B=3 ft.

Area =1·57 sq. ft. ; P=4·04′ ; R=0·39′

v =3·10 f/s

$$\therefore \quad \bar{V} = \frac{V_A + V_B}{2} = 3{\cdot}38 \text{ f/s (approx.)}$$

$$\bar{R} = \frac{R_A + R_B}{2} = 0{\cdot}365 \text{ ft. (approx.)}$$

Let Na—Lacey's Rugosity coefficient =0·012.

∴in length A B which is 0·42 ft. the loss of head in friction.

$$hf=\frac{V^2\ Na^2}{(1{\cdot}3458)^2}\times\frac{L}{R^{1\cdot5}}$$

$$\frac{11{\cdot}4\times{\cdot}000144\times0{\cdot}42}{1{\cdot}811\times{\cdot}3985}={\cdot}0017$$

Section at C—

d=0·84 ; A=2·52 sq. ft. ;

P=4·68′ ; R=0·54′

V=1·93 f/s

$$\therefore\ V=\frac{V_B+V_C}{2}=2{\cdot}52\ \text{(approx.)}$$

$$\text{and}\ \overline{R}=\frac{R_B+R_C}{2}0{\cdot}47'\ \text{(approx.)}$$

and L=1·80′

∴hf in length BC=·003 (approx.)

∴total loss of head in A C=0·0017+0·003=0·0047

As hf varies as $\frac{V^2\ Na^2\ L}{1{\cdot}811\ R^{1\cdot5}}\propto\frac{S\times l\times S}{l\times S^{1\cdot5}}\propto\sqrt{S}$

the friction losses in the prototype instead of being geometrically similar and proportional to S will vary as $\sqrt{S}$; as the excess friction losses in the model=

$$hf\left(1-\frac{1}{\sqrt{S}}\right)={\cdot}0047\left(1-\frac{1}{4{\cdot}9}\right)={\cdot}0038'$$

for discharge equivalent to 718,000 cusecs. But D_1' observed=0·587′

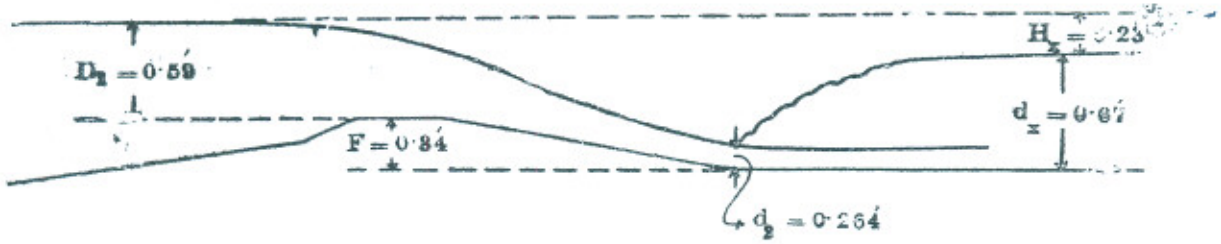
∴ D'_1 allowing for friction is 0·587—·004 =0·583

For smaller discharges, the excess friction is proportionately very approximately the same and hence the exponent in the formula $q=CD^x$ is unchanged and only the coefficient altered.

APPENDIX II.

CALCULATIONS FOR OPTIMUM LEVEL OF BAFFLE PLATFORM.

FIG. 8



Q =4·86 cusecs (=718,000 cusecs in the prototype)

and q=1·62 cusecs.

D_1 =0·58′ (=12·10′ approx.)

and D_3=0·36′ (=8·68′).

Let the Standing Wave form at toe when F=0·34′ (*i. e.*, let the baffle platform be at the pavement level).

d_2 is found to be 0·264′ by trial and error (*vide* Technical Paper No. 44).

and $v_2 = \dfrac{1{\cdot}62}{0{\cdot}26} = 6{\cdot}15$ *f/s.*

$$\therefore \quad d_x = -\frac{d_2}{2} + \sqrt{\frac{2v_2^2}{g} \times d_2 + \frac{d_2^2}{4}}$$

$$= 0{\cdot}67'$$

$\therefore$ $H_x + d_x = 0{\cdot}23 + 0{\cdot}67 = 0\ 90'$ and nearly tallies with $D_1 + F$ $= 0{\cdot}58 + 0{\cdot}34$.

$\therefore$ the baffle platform should be at the same level as the down-stream end of the pavement.

SHORT-CRESTED WEIR MODEL. *Statement I.*

Statement showing the depth of scour below pavement level along the centre line of channel in Experiments 1 to 6 after 1 hour's flow. $q = 1 \cdot 624$ *Cs.* (=191 *Cs. or* $Q = 718{,}000$ *Cs. in the Prototype*). *W. L. upstream* = 100·587 ; *W. L. downstream* = 100·360 ; *crest level* 100·00.

Distance in ft. from the end of pavement.	Expt. 1 (without a baffle or a deflector).	Expt. 2 (Deflector ·07′ high on 'C' line of wells.	Expt. 3 (Baffle 2″ high on 'B' line of wells and deflector on 'C' line).	Expt. 4 (Deflector at end of concrete blocks).	Expt. 5. Baffle 2″ high on 'C' line and deflector at end of concrete blocks.	Expt. 6. Baffle 2″ high on 'B' line and deflector at end of concrete blocks.
	ft.	ft.	ft.	ft.	ft.	ft.
0·5	0·04	..	0·12	0·12	0·16	0·16
1	0·25	0·25	0·42	0·29	0·33	0·28
2	0·50	0·42	0·54	0·39	0·46	0·41
3	0·77	0·50	0·33	0·47	0·58	0·50
4	0·83	0·44	0·18	0·41	0·52	0·46
5	0·74	0·29	0·12	0·35	0·50	0·37
6	0·58	0·26	0·08	0·29	0·35	0·29
7	..	0·19	0·08	0·22	0·29	0·25
8	0·27	0·16	..	0·20	0·27	0·16
9	..	..	0·04	..	..	0·12
10	0·15	0·12	0·0	0·14	0·16	0·08
11	..	..	..	..	..	..
12	0·0	0·0	..	0·07	0·09	0·0
13	..	..	..	..	..	..
14	..	..	..	0·0	0·0	..

SHORT-CRESTED WEIR MODEL.

Statement II.
(See Plate III).

Statement showing pressures observed in Experiment 6—Baffle 2″ high on 'B line of wells and deflector (0·07′) at the end of concrete blocks.

q=1·624 *cs.* (=191 *cs. or Q*=718,000 *cs. in the prototype).* *w. l. upstream* =100·587;
w. l. downstream=100·36;
crest level=100·00

Section along centre line of channel.

Pressure Pipe No. ..	1	6	11	13	18	20	25
R. L. of pressure point ..	99·86	99·80	99·87	99·87	99·78	99·74	99·66
Pressure R. L. ..	100·39	100·50	100·48	100·20	100·20	100·30	100·36
Water level ..	100·35	100·44	100·44	100·43	100·41	100·22	100·33
Depth of water in ft. ..	0·49	0·64	0·57	0·56	0·63	0·48	0·67
Pressure head in ft. ..	0·53	0·70	0·61	0·33	0·42	0·56	0·70

Vertical Sections along Right Bank.

Section on I (Water level 100·35) Section on II (w. l. 100·44). Section on III (w. l.=100·41).

Pressure Pipe No. ..	2	3	4	5	7	12	8	9	10	16	14	17	18	19
R. L. of pressure point ..	99·86	99·98	100·11	100·23	99·80	99·87	99·92	100·05	100·17	99·78	99·87	99·99	100·03	100·15
Pressure R. L. ..	100·37	100·29	100·31	100·29	100·49	100·47	100·46	100·31	100·23	100·17	100·20	100·21	100·29	100·36
Depth of water in ft. ..	0·49	0·37	0·24	0·12	0·64	0·57	0·52	0·39	0·27	0·63	0·54	0·51	0·38	0·26
Pressure head in ft. ..	0·51	0·31	0·20	0·06	0·69	0·60	0·54	0·26	0·06	0·39	0·33	0·31	0·26	0·21

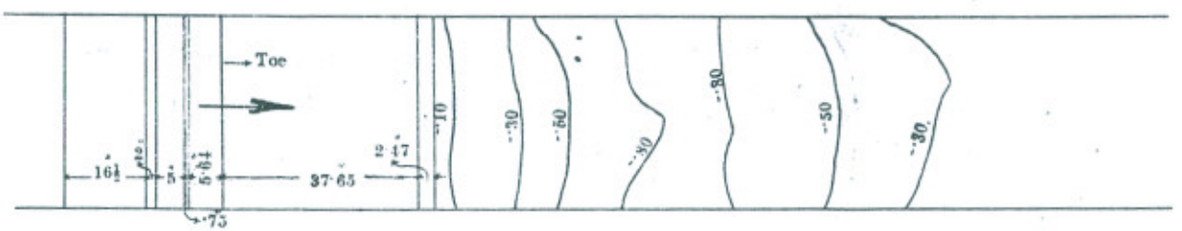
Section on IV (w. l.=100·22). Section on V (w. l.=100·33).

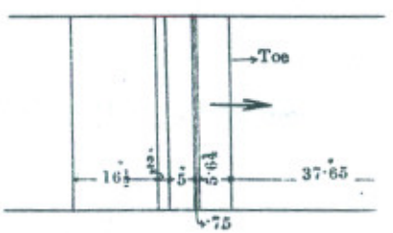
Pressure Pipe No. ..	21	22	23	24	26	27	28	29
R. L. of Pressure point ..	99·74	99·86	99·99	100·11	99·66	99·78	99·91	100·03
Pressure R. L. ..	100·32	100·30	100·25	100·19	100·36	100·36	100·35	100·35
Depth of water in ft. ..	0·48	0·36	0·23	0·11	0·67	0·55	0·42	0·30
Pressure head in ft. ..	0·58	0·44	0·26	0·08	0·70	0·58	0·44	0·32

FIG. 1, CONTOURS OBSERVED AFTER 1 HOURS FLOW

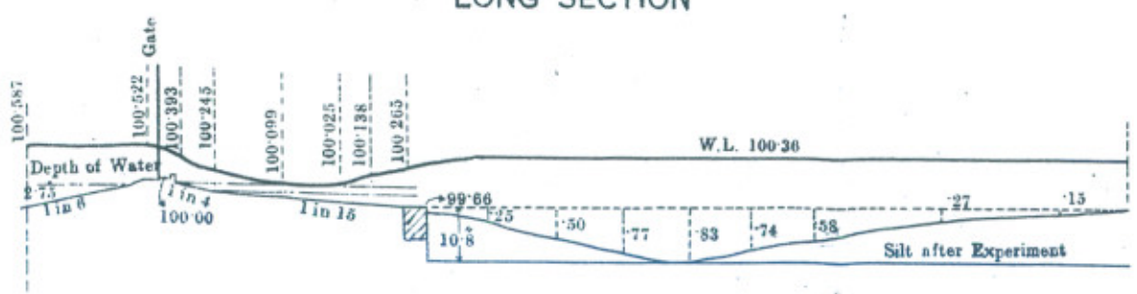
FIG. II

PLAN





LONG SECTION



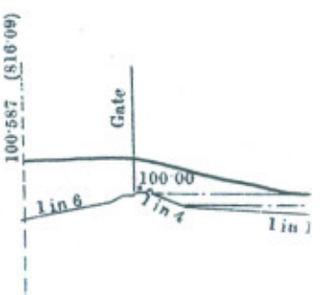
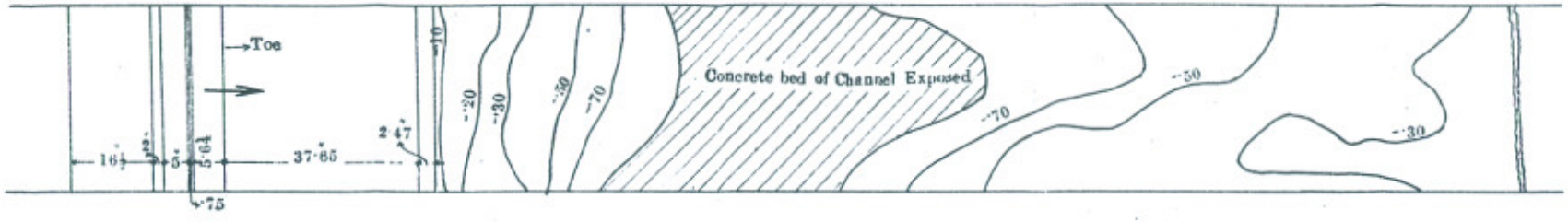


FIG. II SCOUR AFTER 6 HOURS FLOW NO DEFLECTOR & NO BAFFLE

PLAN



LONG SECTION

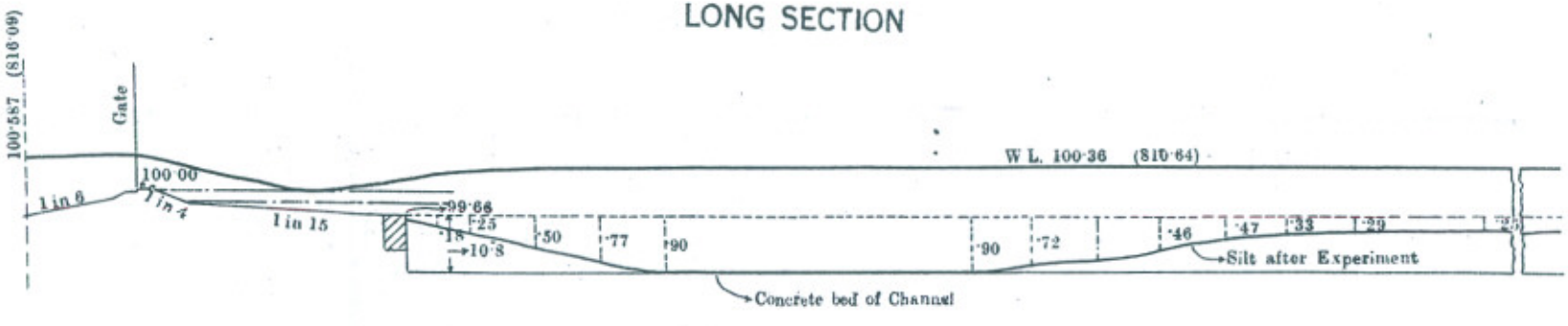
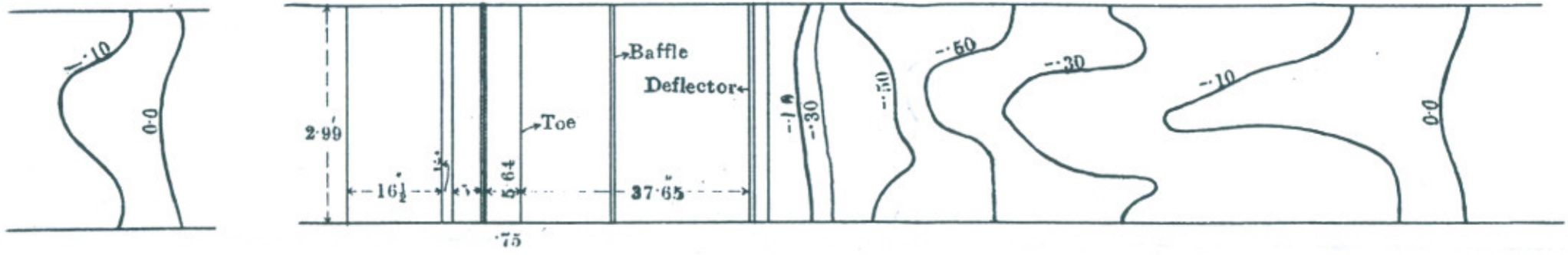


FIG. II

BAFFLE 2" HIGH & 15" FROM TOE & DEFLECTOR ·07 HIGH AT END OF PAVEMENT SCOUR AFTER 1 HOUR'S FLOW

PLAN



LONG. SECTION

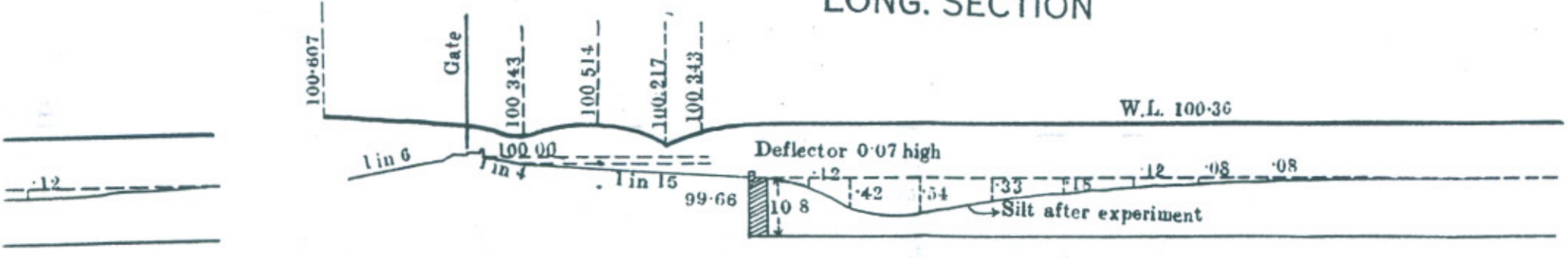


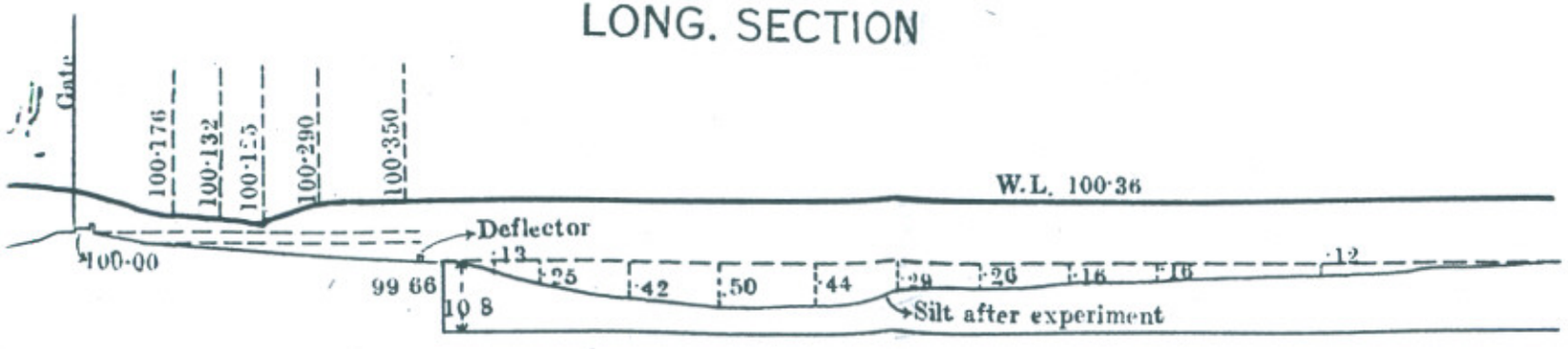
FIG. 1

CONTOURS OBSERVED AFTER 1 HOUR'S FLOWWITH DEFLECTOR ONLY ·07 HIGH AT END OF PAVEMENT

PLAN

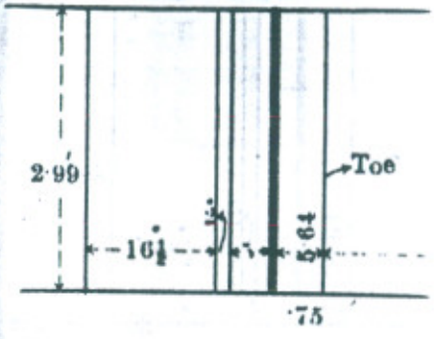


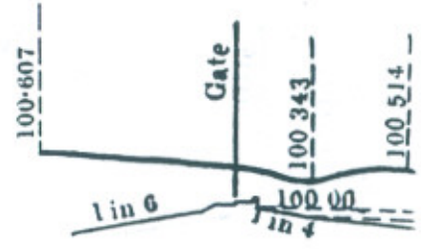
LONG. SECTION



BAFFLE 2 HIGH &

PAVEM

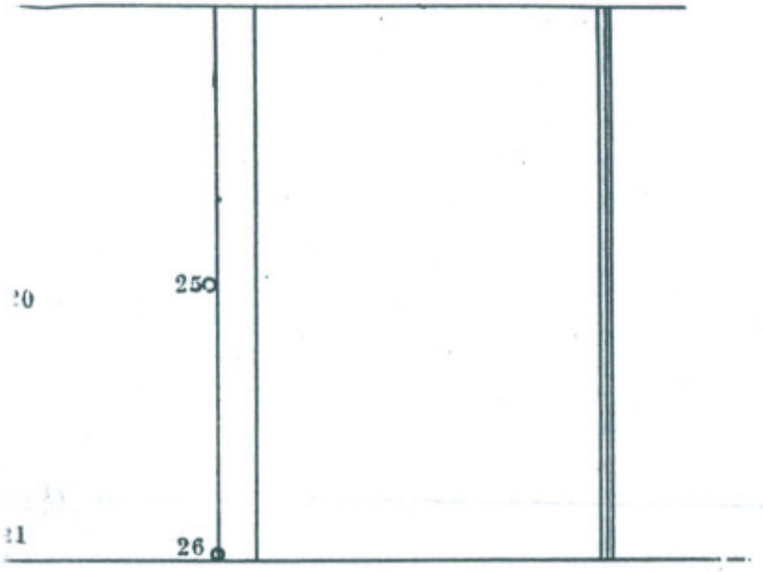




PRESSURE OBSERVATIONS

PLATE III

I PLAN



tion along Centre Line

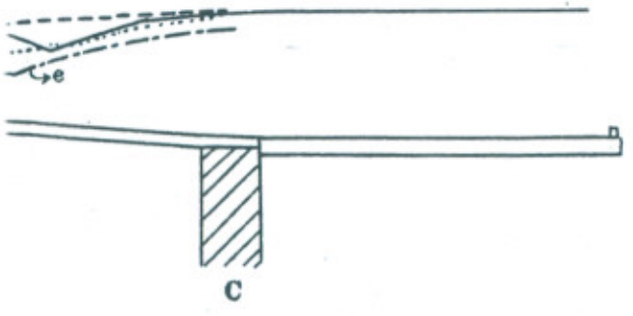
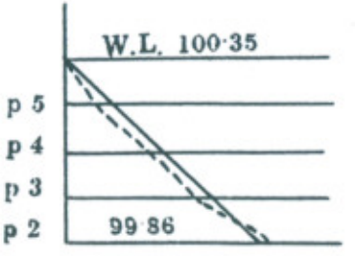


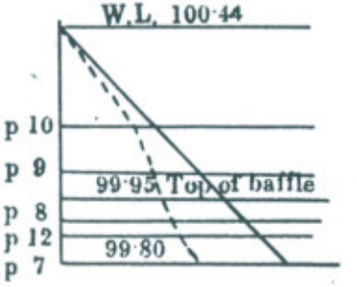
FIG. III VERTICAL SECTIONS ALONG RIGHT BANK

Scale 0·5 = 1'

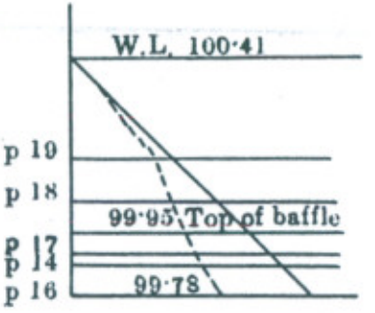
I



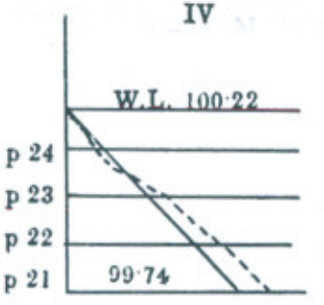
II



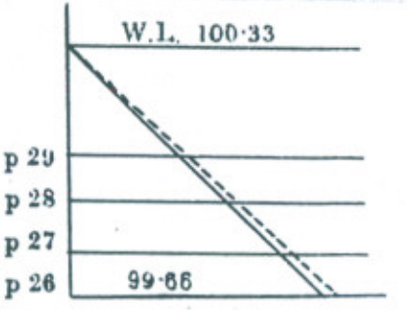
III



IV



V



PRESSURE OBSERVATIONS

FIG. I PLAN

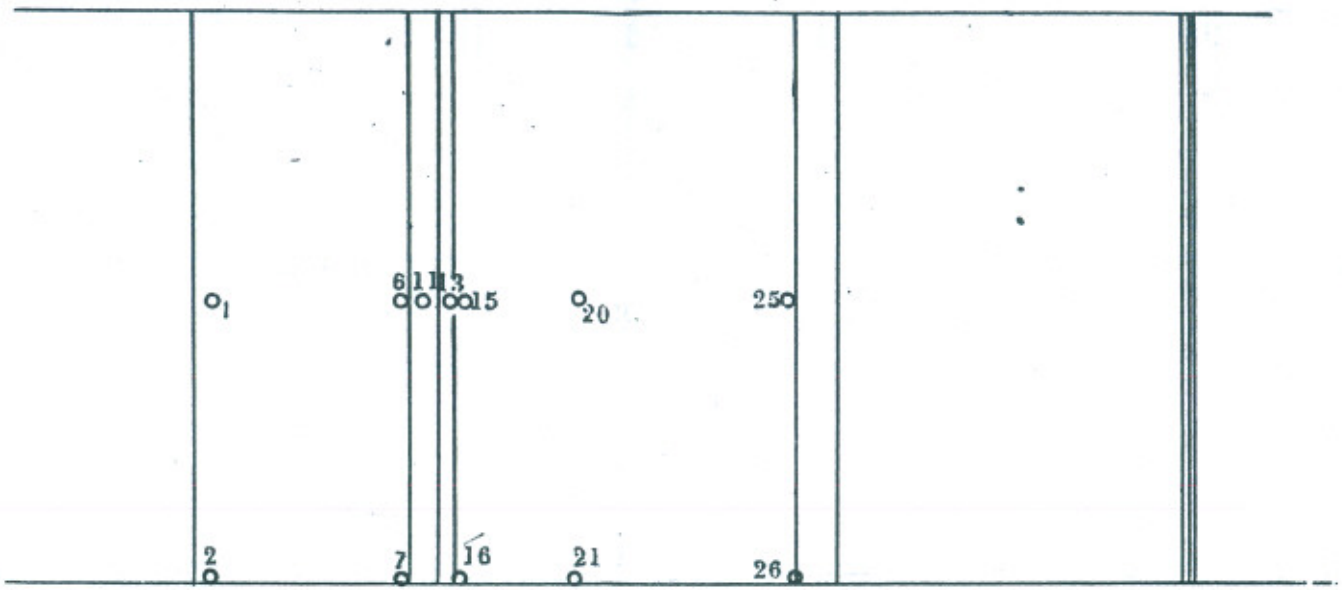


FIG. II Section along Centre Line

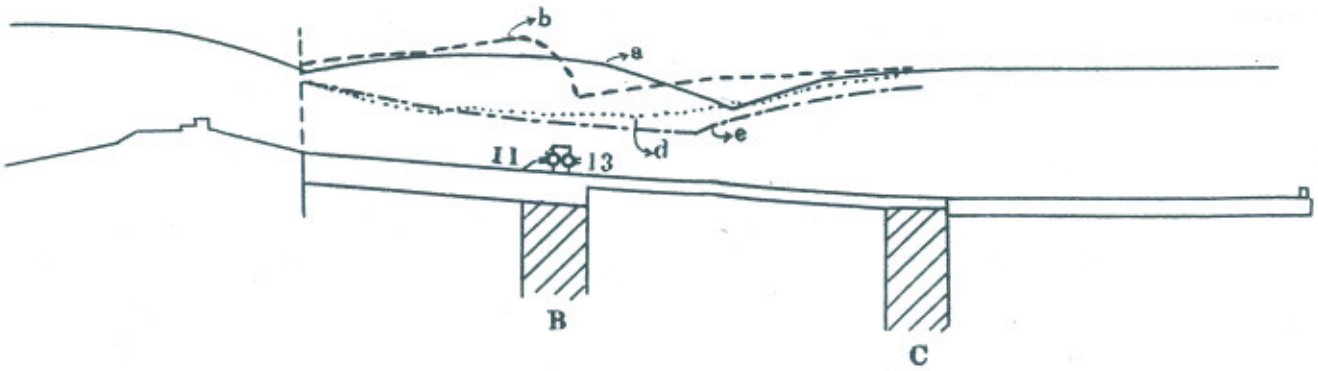
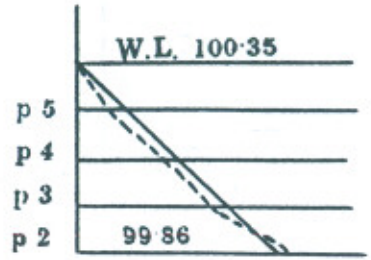


FIG. III VERTICAL SE
Sc

I



III

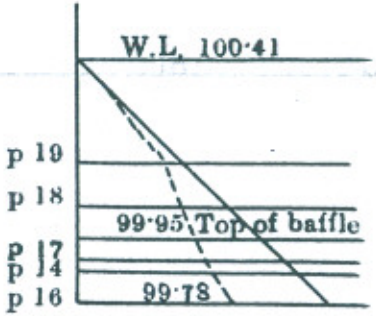


FIG 15

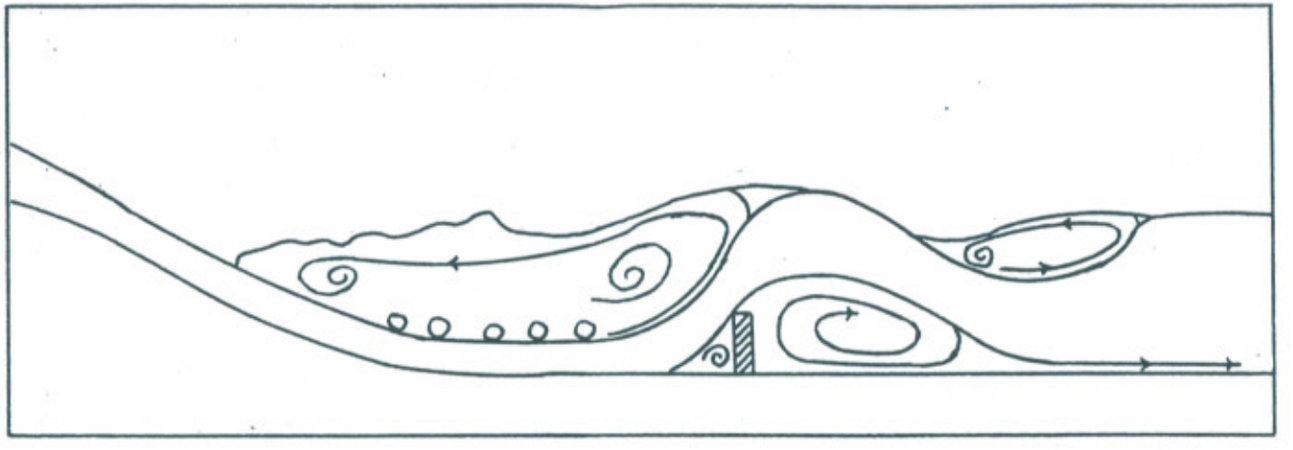
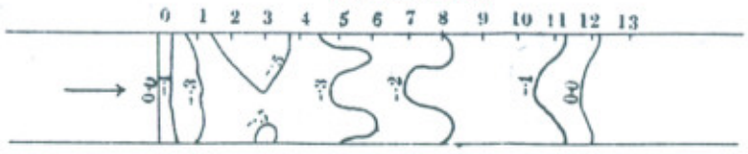


FIG. X PLAN



L. Section along centre line

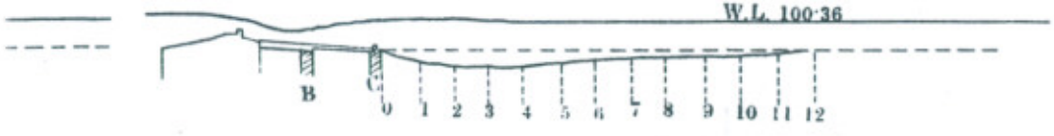
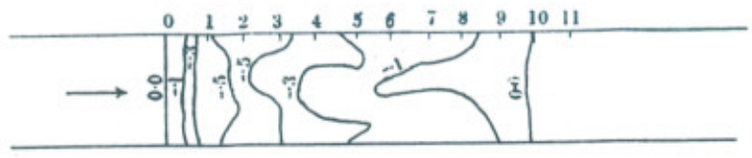


FIG. XI PLAN



L. Section along centre line

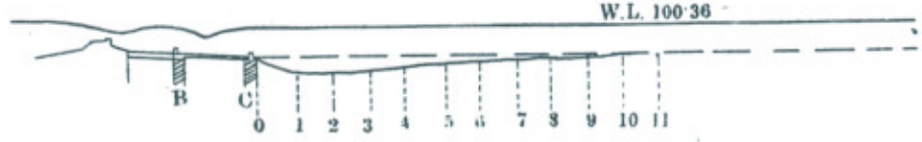
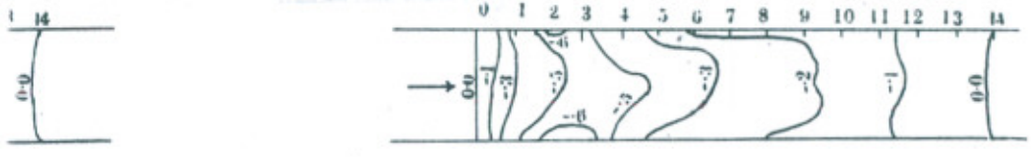


FIG. XIII PLAN



L. Section along centre line

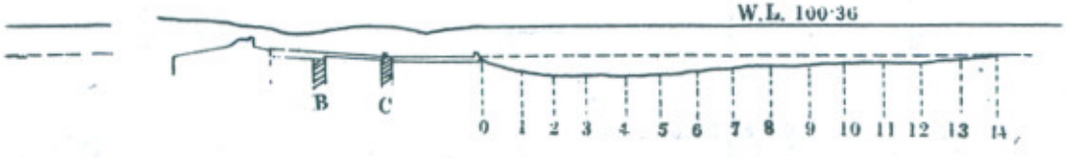
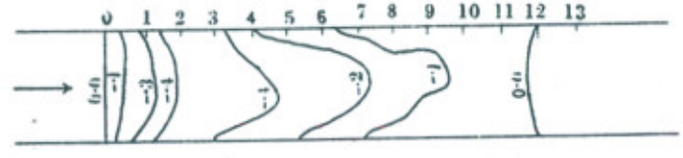


FIG. XIV PLAN



L. Section along centre line

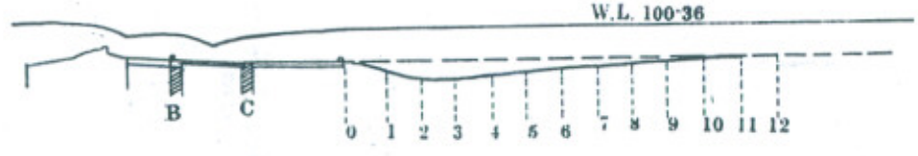
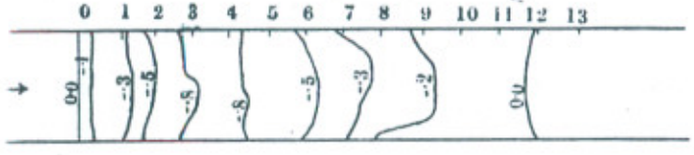


FIG. IX PLAN



L. Section along centre line

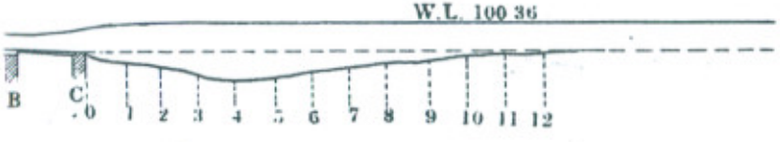
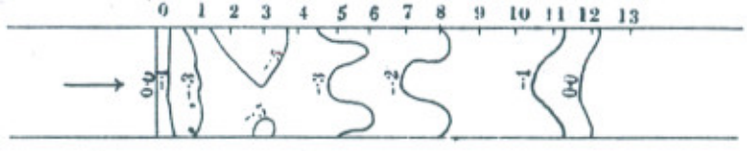
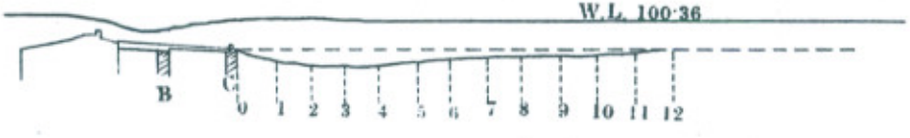
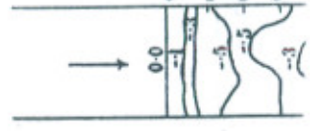


FIG. X PLAN



L. Section along centre line





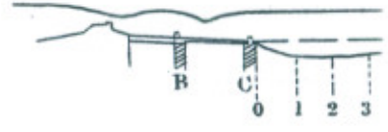


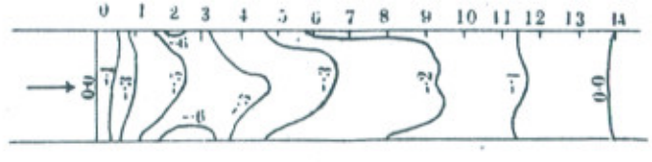
FIG. XII PLAN



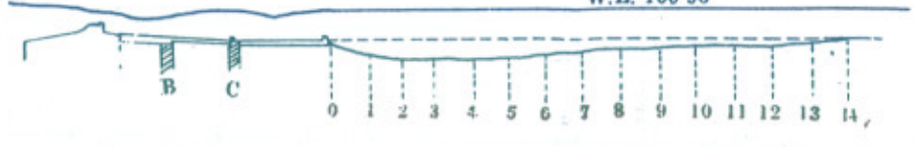
L. Section along centre line



FIG. XIII PLAN



L. Section along centre line





DISCUSSION.

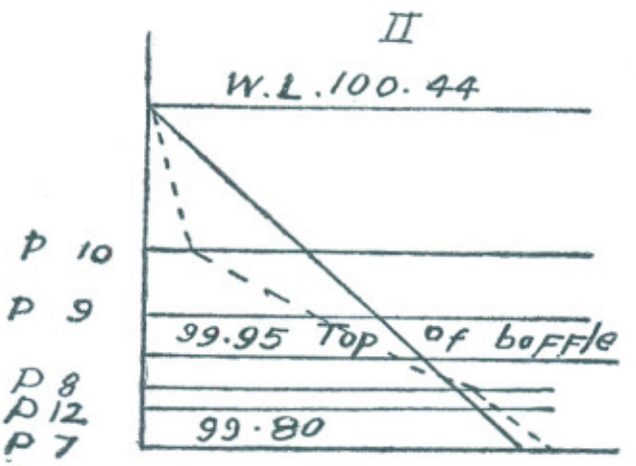
Mr. Inglis in introducing his paper said that the paper described the last of a long series of experiments. The earlier experiment had been described in Bombay Technical Paper No. 44 on " The Dissipation of Energy below Falls."

There are a few mistakes in the Paper, mainly due to the second part having been sent in late.

Figs. IX, X and XI on the last sheet of drawings were intended to replace Fig. I of Plate II and Figs. 1 and 2 of Plate II, but both have been printed. Figs. IX, X and XI are the more correct.

In Fig. 2 of Plate II the baffle has been omitted and also the 'B' line of wells. *See* Fig. XI.

In Fig. 1 of Plate 2 and Fig. X, a scour hole on the left bank to a depth of 0·71" has been omitted in re-drawing. This omission is important because it shows that there was a tendency for a vortex to form when only a deflector was used. The dotted line of Fig. 2 of Plate III is incorrect, it should be :—



There is a marked difference in the functions of a baffle and a deflector. The function of a deflector is to deflect the high velocity bed stream towards the surface so as to redistribute the velocities more suitably and to cause a horizontal bed roller to form, which prevents the work being undermined. The function of the baffle, on the other hand, is to dissipate energy by generating small eddies and by throwing up the water which is highly aerated so that when it falls it forms a secondary wave, by means of which the remaining energy is dissipated.

A deflector should be constructed at the end of the pavement of every masonry work. It is cheap to construct and highly effective. A baffle is ineffective unless the head available is in excess of $\frac{1}{3}$ the depth of water over the sill. It becomes increasingly efficient as the available head increases. The Author had recently inspected the Tando Masti Khan fall, which has been constructed some 25 miles from the Head of

the Rohri Canal. The discharge is approximately 10,000 cusecs and the fall 8 ft. In this fall the divergences are 1 in 5 ; but the water has been fanned out satisfactorily by constructing radiating vanes in continuation of the piers. *See* photograph on page X of Technical Paper No. 44 of the Bombay P. W. D. The excess energy has been destroyed quite close below the baffle by means of highly aerated primary and secondary waves.

Mr. Colyer speaking on the previous paper referred to the " biff-wall " design. A paper recently read before the American Society of Civil Engineers showed that where flow is parallel a wall built at the end of the standing wave roller gives excellent results. Under such conditions a " biff-wall " would be satisfactory; but in the experiments carried out it was found that where there was an expansion or cross, or uneven flow, a " biff-wall " was not at all successful as it did not prevent corkscrew eddies and return flow.

Mr. Dundon remarked that Mr. Inglis had put in tremendous amount of work on the paper which would be very helpful in future in this Province. He considered it rather a pity that this paper had come at the end of the day because in some ways the occasion was unique. It was the first paper that had been presented by a member of the Congress coming from another Province. There was another way in which the paper was unique as being the first paper dealing with the design of a work based on model experiments. This was the first paper presented to the Congress on model experiments.

The speaker had no criticism to offer on the paper. He considered the paper showed that remarkable results could be obtained with properly designed baffle and deflector walls.

Referring to the Marala Weir, he stated that during the last three years as much as six lacs cft. of stone had been used in the weir apron, *i.e.*, to restore the damage done to the weir, and he referred to the suggestion in the paper which he was convinced would solve the problem which faced the Department very definitely in the Punjab, that is, the damage which results from the destructive action of the standing wave below a weir.

Proceeding, Mr. Dundon wanted to inquire from Mr. Inglis whether the Lloyd Barrage was designed with similar baffle and deflect walls, and if not, why not, and whether a baffle would throw a jet right to the surface which in falling would result in heavy dynamic action on the floor ? He thought that the results of these experiments would give a useful lead to similar work in the Research Laboratory.

The speaker referred to the model work being done in Egypt and drew attention to a paper entitled " The causes and prevention of bed erosion with special reference to the protection of structures controlling rivers and canals " by A. D. Butcher and John Dekeys Atkinson of the Egyptian Irrigation Department, the subject of a recent article in the *Indian Engineering*. He read the following extracts from that article : " The authors during a period of several years were in charge of the

Hydraulic Research Station at the Delta Barrage. During the course of their general investigations they were struck with the lack of precise information, as regards the design of protective aprons, and began experiments in order to provide an answer to certain specific questions which had been referred to them. The first results obtained were sufficiently promising to warrant a more general inquiry, the results of which are set forth in the paper.

"In general, scale models provided with a friable bed material were used to produce scouring action, and a series of experiments and comparisons with existing structures demonstrated the reliability of these models both as regards the magnitude and direction of the currents resulting from various methods of regulation and the extent and character of the erosive action produced. The models varied in size from 1/100 to 6--7/100 of the full scale.

"The authors strongly advocate the use of models for every major structure during the process of design, so that the effect of the structure in combination with the channel it serves, on the current-distribution and scouring action may be determined, and protective measures adopted to the special circumstances of the case incorporated before the design is complete."

The speaker strongly advised that Punjab Engineers should not lag behind those in other provinces of India or in Egypt in advancing their knowledge by model work. Our designs would then be based on a scientific and sure foundation, and our mistakes and theories would be fewer.

Concluding he believed that this paper was one of the most important and interesting papers that had been read at the Congress for many years. He also believed that Mr. Inglis had come to spread the light of a new gospel, the gospel of model work and assured him that he could count on the speaker as one of his disciples.

The author, in replying to the discussion said that two points were raised by Mr. Dundon, *viz.*, whether a baffle and deflector had been constructed downstream of the Lloyd Barrage and whether a baffle would throw up a jet to the surface which would be likely to cause dynamical action on the floor.

With regard to the first point : the author was not called on to advise about the design of the Barrage until after the undersluices portions had been completed. In the original design the sill of the Barrage and undersluices was at R. L. 176·0 and the downstream glacis sloped at 1 in 70 down to R. L. 172·0. The model experiments showed that with this design the wave under certain conditions, would form downstream of the protection, more than 310 ft. from the Barrage. Under such conditions the action in the experiments was very severe and the floor was rapidly washed away. As a result of these experiments the sill in the river portion of the Barrage was raised by 1 ft. and was reduced in length from 80 ft. to 4 ft. in the line of flow. The upstream slope of the weir was made 1 in 5 down to R. L. 171·0 and the downstream slope 1 in 10

down to R. L. 171·0. By these modifications the standing wave was brought up to the Barrage for all stages of flow during the flood season, even should retrogression occur, and though the sill has been raised 1 ft. yet the new design will give the same discharge as originally allowed for, namely 1½ million cusecs; but another foot of command is thus obtained, which is important because in the original design the working head for the Eastern Nara Canal was only 0·3′. This work has been described fully in Bombay P. W. D. Technical Paper No 29.

The question of dissipating energy otherwise than by a plain standing wave was not taken up until later—in connection with canal falls. Baffles and deflectors are now being constructed below all falls and regulators and it is highly probable that even at this stage the deflectors will be constructed below the Barrage.

A baffle throws up a highly aerated jet, especially where the fall is considerable, and when this jet falls, forming a secondary wave, it resembles spray so that dynamical action on the floor is negligible.

In conclusion the author said that it was exceedingly kind of Mr. Dundon to be so appreciative in his remarks and he could only say in return that it has been a great privilege to address the Punjab Engineering Congress the work of which he had admired for many years.

The President thanked Mr. Inglis on behalf of the Punjab Engineering Congress and hoped that after this no one would say that Bombay had not co-operated with the Punjab. He also believed that co-operation would continue to be offered.