PAPER No. 96.

PRINCIPLES OF DESIGN FOR PROPORTIONATE FLUMES AT HEADS OF CHANNELS.

By Prabh Singh.

I. Practically on all the canals in the Punjab the proportionate flume distributor, sometimes called the "waisted and crested" distributor, is coming into vogue for Head Regulators of Minors. The chief object is generally the automatic distribution of supply under fairly wide limits of working. The great utility of such flumes for silt exclusion has not yet been sufficiently recognised but practical working has shown that there are great potentialities in this respect.

II. The success in working depends on proper design, mainly in the height at which the crest is kept. A crest kept too low by as small an amount as ·25′ has resulted in failure and no sooner it was raised by that little amount the working markedly improved and the flume became successful. From the practice followed so far it is seen that no definite procedure is adopted to fix upon the height of crest; some people fix upon the length of the crest as an arbitrary proportion of the normal bed-width of the channel and arrive at the height of the crest from this while there are others who fix upon the crest level as an arbitrary figure and calculate the length from this. Such methods are nothing but shots in the dark and while some result in success, most go wrong.

III. If definite mathematical equations are followed in the calculation of these flumes, simplicity in design and uniformity in practice will be the natural result. Such equations exist and it is the object of this note to collate them.

IV. It is important to mention that before it is proposed to remodel the head regulator of any channel it is necessary to take a complete hydraulic survey of that channel. The data to be collected in this survey is given in Appendix A of this note. In silting channels such a hydraulic survey might reveal that the "designed longitudinal section" is defective and conditions of command are such as make it impossible for the regime to conform to the "designed longitudinal section." If the head regulator of a minor is not defective it will have a silting tendency if—

- (*a*) its critical velocity ratio in the head reach is less than the critical velocity ratio of its parent channel;
- (*b*) it has poor command into the outlets;
- (*c*) it has drowned bridges or tight falls or notches which obstruct flow;
- (*d*) it has poor command from the parent channel.

These defects must be overcome by revising the longitudinal section suitably and raising the bridges if necessary. In several cases however, it will be observed that none of these defects exist and yet the channel silts up badly and manual clearance has no permanent results. It is in such cases that the "waisted and crested flume" comes in most useful.

V. Assuming that the defects mentioned in para. IV have been overcome, two conditions might be met with in practice :—

(*a*) The command from the parent channel into the minor is good.

(*b*) The command from the parent channel into minor is poor.

Of course " good " and " poor " are only relative terms depending on the depth in the parent channel. Perhaps the following relations will be found to work well in practice :—

Command of anything less than ·3′ to be considered " poor " when the depth in the parent channel exceeds 2·0′.

Command of anything less than ·6′ to be considered " poor " when the depth in the parent channel exceeds 3·0′ and so on.

These are only rough approximations and may have to be suitably varied according to local circumstances. In case " (*a*) " the calculations to arrive at the length of the crest are naturally simpler than in case " (*b*) " as detailed below.

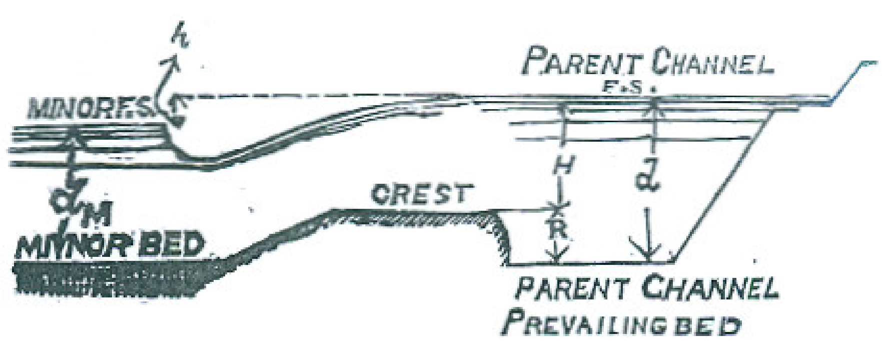
VI. Suppose it is found after suitable changes in the longitudinal section to meet the conditions in para. IV. that the command is " good " and no tilting up of the supply is called for in the parent channel. For proper silt control and proportionate distribution of supply it is essential that—

(*a*) the flumes should be constructed in a pair, *i.e.*, a flume in the minor and the other in the parent channel just below the offtake of the minor ;

(*b*) crests of both the flumes should be at the same level.

The equations to determine the height of the crest are :—

1. Drowning should not be more than 2/3rd or 66 per cent., *i. e.*, H-h should not be greater than 2/3 H or H should not exceed 3 h. (With proper splays the drowning might be allowed as 80 per cent. or even 90 per cent. but it is erring



on the side of safety to limit it to 66 per cent.) For a mathematical explanation of this condition see para. XIV at the end of this note. It is necessary to limit the drowning to figure in order to make the discharge through the flume independent of the downstream supply level.

2. $R + H = d$ which is a known quantity.
3. $I_F = I_M$ where I_F means the silt-index of the flume and I_M means the silt-index of the minor.

$$\text{Now } I_F = \frac{1{\cdot}5 \times 3\sqrt{H}}{R + H/2} \text{ and}$$

$$I_M = \frac{\text{Vel. in the minor}}{{\cdot}3 \times \text{depth in minor}}$$

$$= V_M / {\cdot}3d_M$$

$$\text{therefore } \frac{1{\cdot}5 \times 3\sqrt{H}}{R + H/2} = V_M / {\cdot}3d_M$$

Find the values of R and H from the two equations 2 and 3; if the value of H thus obtained satisfies condition 1 then adopt this value. If not, it is obvious that H should be reduced in value to make it equal to 3h. This reduced value of H will give a higher value of R and consequently a lower value of I_F. This will mean that the silt-index of the flume will be less than the silt-index of the flume minor which is an advantage to a certain extent. Briefly summed up therefore, the value of H and R should be determined from equations 2 and 3 given in this paragraph and if this value does not satisfy condition 1, then simply use the equations 1 and 2 which are—

$$H = 3\,h.$$

$$R + H = d.$$

Having determined H we use the formula for a free fall to determine L which is the width of the gullet; this formula is :—

$$Q = \text{Discharge} = 3\,L\,H^{3/2}$$ (*vide* para. XIV).

VII. In cases where, after remodelling the longitudinal section suitably, it is observed that the command from the parent channel into the minor is "poor" or negative, it is obviously necessary that the Full Supply in the parent channel be tilted up to feed the offtaking channel. The minimum amount of this tilting as well as the level at which the crest ought to be kept can be determined from the following three equations :—

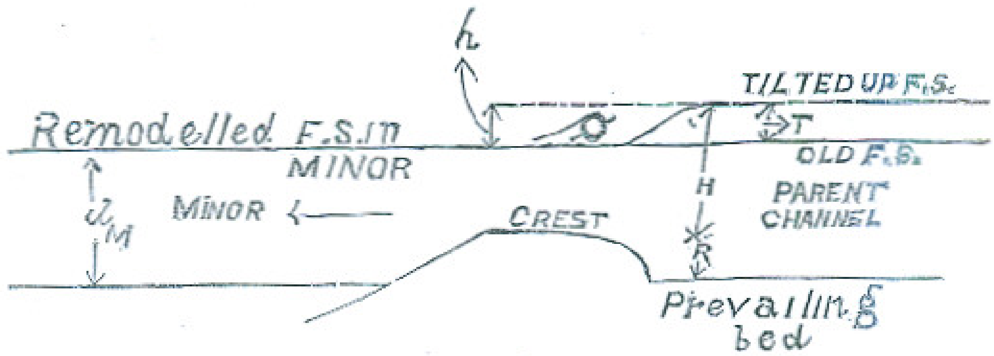
1. Drowning should be 66 per cent. or,

$$H = 3h.$$

2. $I_F = I_M$, or

$$\frac{1{\cdot}5 \times 3\sqrt{H}}{R + H/2} = V_M / {\cdot}3d_M$$

3. R + H = Remodelled F. S. level in the minor + h − prevailing bed level in distributary.



The three unknown quantities are H, h and R and to determine these we have the above three equations. Having determined H, the width of the gullet is found from the equation Q = Discharge = 3 L. $H^{3/2}$ where L is the width of the gullet (*vide* para. XIV).

VIII. In connection with the design of flume heads on the above lines the following points deserve mention :—

(*a*) Tilting up of the supply line in the distributaries to improve command.

(*b*) Effect of raised crests on the bed regime of the parent channel upstream.

(*c*) Silt-index theory.

Each of these points is dealt with briefly in paras. IX, X, XI and XII.

IX. Tilting the supply in the distributaries to feed offtaking channels :—Not a few officers are met with who strongly object to the water surface being tilted up in distributaries. The very idea is hateful to them, but the only objections that are ever urged are as follows :—

(*a*) Regime of channels is disturbed by doing so.

This is a most futile objection, for what is regime ? It is not something God sent, and obviously because the offtaking minor silts up and does not work properly the conclusion is that the regime is unsuitable and requires change. Why not change it then ?

It is certainly poor consolation to bring down water in a canal hundreds of miles from the head and then to find that it cannot irrigate by flow. The primary object of canals is irrigation by flow and for this purpose command must be regained even if the supply has to be tilted up and the regime has to be changed. To know that change is the remedy and then not to introduce this change is poor engineering.

(*b*) Raising the supply increases the absorption. Perhaps it does, but the increase is so small that it is practically a negligible quantity because in almost all cases a tilting up of 1·0 to 1·5′ is all that is required. Even this increase in depth is maximum at the site of the flume and disappears a short distance above where the back-water curve disappears.

(*c*) The channel upstream of the flume is likely to silt up. This matter is dealt with in detail in para. X below.

(*d*) It is expensive as banks have to be strengthened.

Only inexperienced men can be deterred by this objection. Take a channel of 1/3636 slope and suppose the supply is tilted up by 1·0′. It is well within the limits of safety to take the back water curve as extending to a distance of 2 × (tilting) × 3636 = 7272 feet.

If the bank width is 5·0′ the quantity of earth-work required will be only about 40,000 cubic feet costing less than Rs. 200. This is certainly not much considering the improvement anticipated.

X. Effect of raised crests on the bed upstream—

Two conditions have to be examined in this connection :—

(*a*) When the F. S. level is not tilted up, *i. e.*, the surface slope u/s of the crest is not flattened in Full Supply conditions.

The velocity and flow as well as the critical velocity ratio and silt regime in a channel depend on the surface slope and it is therefore obvious that during the F. S. days the bed regime will continue as before the introduction of the crest. Supplies less than Full Supply will affect the surface slope thus—

Let Q = F. S.

H = Depth on crest for F. S.

d = Depth in channel, well u/s of the crest in Full Supply.

R = Height of raised crest above the bed.

Let Q′, H′, d′ denote the same data for a supply which is less than F. S.

Then $Q = kd^{5/3}$ and also $= cH^{3/2}$

and $Q' = kd'^{5/3}$ and also $= cH'^{3/2}$

therefore,

$Qd'^{5/3} = Q'd^{5/3}$, and

$QH'^{3/2} = Q'H^{3/2}$

or,

$d' = d\,(Q'/Q)^{3/5}$ and

$H' = H\,(Q'/Q)^{2/3}$

Fall in water surface at the crest is

$$H - H' = H - H\,(Q'/Q)^{2/3}$$
$$= H\,[1 - (Q'/Q)^{2/3}]$$

Fall in water surface in the channel well u/s of the crest, beyond the afflux effect, will be

$$d - d' = d\,[1 - (Q'/Q)^{3/5}]$$

If we take $Q' = \frac{1}{2}\,Q$, then

$$H - H' = H\,[1 - (0{\cdot}5)^{2/3}]$$
$$= H\,(1 - {\cdot}63)$$
$$= {\cdot}37\ H$$

and $d - d' = d\,[1 - ({\cdot}5)^{3/5}]$

$$= d\,(1 - {\cdot}66)$$
$$= {\cdot}34d$$

If, therefore, half supply conditions were continued for *a very long* period the maximum amount of silt deposit "S" in the channel will be

$$S = (d - d') - (H - H')$$

$= {\cdot}34\,d - {\cdot}37\,H$, because as soon as this amount of silt has deposited the new surface slope will have become parallel to the original surface slope.

It is interesting to examine the actual quantity of this deposit for channels of different depths and this is done in the statement below :—

Channels having values of d and H as under.

Maximum amount of Silt = S = ·34d—·37H	d = 2·0 H = 1·5 R = ·5	d = 2·0 H = 1·25 R = ·75	d = 2·0 H = 1·0 R = 1·0	d = 3·0 H = 2·5 R = ·5	d = 3·0 H = 2·25 R = ·75	d = 3·0 H = 2·0 R = 1·0	d = 4·0 H = 3·0 R = 1·0	d = 4·0 H = 2·5 R = 1·5
	·12	·22	·31	·09	·19	·28	·35	·43

It is very unlikely that half supply conditions will prevail sufficiently long to induce this maximum amount of deposit but should such an extreme state of affairs ever come into existence the maximum quantity of silt is nothing serious.

A comparison of the values of "S" with the values of "R" in the above statement will show that as a rough approximation, if half supply conditions are prolonged sufficiently the quantity of silt that will deposit may be taken as equal to 1/3 to 1/4 of the height of the crest above the bed.

As soon however, as F. S. is again put into the channel, conditions will be reversed and the rise at the crest will be less than the rise in the channel up-stream with the result that the surface slope will be steeper than it was in the half supply conditions and the freshly deposited silt will get automatically picked up and carried down.

It is therefore clear that for channels in which no tilting up has been done at the flume the question of silt deposit up-stream need not be viewed with any anxiety or alarm.

(*b*) When the surface is tilted up at the crest to secure better conditions of command, it means that the water surface slope is flattened and silt may be expected as a natural consequence. The maximum amount of silt can theoretically be equal to the amount of tilting. In practice it will be found that the maximum amount is always less than the amount of tilting of the F. S. but at its worst it can only be equal and never more.

When this amount of silt has deposited and the new surface slope has become parallel to the original surface slope, conditions will come into existence as described above and any *permanent* deposit ceases to take place. It must be mentioned here that this silt deposit is an indispensable necessity and its absence was a draw-back as shown by the very fact that the supply in the channel was not sufficiently high and had to be tilted up. The absence of this silt which was causing the off-taking minor to suffer due to lack of command, was a defect and that this should be removed is a welcome improvement and not something to be frightened about.

XI. One thing is important to mention in this connection when the supply is tilted up and it should not be lost sight of :—

The silt up-stream of the flume which will deposit so as to steepen the slope to the same value as before tilting, will also affect the silt index of the flume. The value of R in the equation

$$I_F = \frac{1{\cdot}5 \times 3\sqrt{H}}{R + H/2}$$ will decrease but H will remain the same

and this will obviously mean an increased value of I_F. The consequence will be that it will exceed I_M and as a result the minor will begin to silt up so that its I_M becomes equal to the new increased value of I_F. When this begins, one of the two remedies mentioned below must be employed and the result will be a permanent cure :—

(*a*) If the silting up of the minor to the extent that its I_M becomes equal to new I_F is permitted by local circumstances, this silting may be allowed to remain as a permanent feature and the longitudinal section may simply be revised. The local circumstances to be considered are :—

(1) Effect on the supply reaching the tail of the minor due to the over-draw by the outlets in the silted reach.

(2) Effect on the standing wave in the minor flume. If this standing wave continues crisp and well-defined, in spite of the silt, the minor may be assumed to be taking its due share of discharge.

(3) Clearance in bridges on the minor.

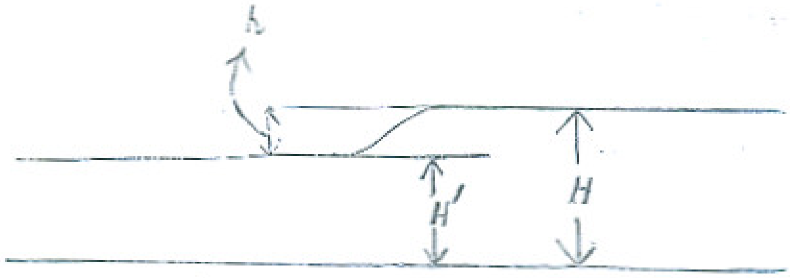
(*b*) If local circumstances do not permit that this silt should remain in the minor, the original design of the flumes must be modified so as to reduce the new value of I_F to its original value which had been kept equal to I_M. This would simply mean raising the crest and widening the gullet and reducing H. No tilting up is now required and the values of R and H are therefore easily determinable from the simple rules given in para. VI above. When this has been done no further silting need be feared as explained in para. X above and a permanent regime is the result.

XII. It is tragical indeed that the "waisted and crested distributor" is sometimes seriously condemned on the score of its tendency to induce silting u s, when the real defect lies in expecting it to give permanent regime in the channel *before* it has been given a final suitable design itself. This final design can easily be given if either of the two remedies mentioned in para. XI are adopted. Unless this simple remodelling is done, the flume cannot continue to work satisfactorily.

XIII. In determining the design of the flume, use has been made of Mr. Wood's silt index theory. It is known that some people do not accept this theory but in the present case its use gives us a simple guide in arriving at a certain workable height of the crest and obviates the necessity of being guided by rough guesswork approximations which could only depend on the idiosyncrasies of each individual designer. Uniformity of design is the result and officers who have doubts of this theory can work out some designs as indicated in this note and find out for themselves what the results are.

What is the silt index theory after all? Briefly summed up is it not this? "The silt depositing capacity of a channel or a masonry work depends on the height of the velocity in that channel or through that work above the bed." To utilise this simple principle to determine the value of R does not appear to be objectionable in any way especially if the designer does not find it defective in any other respect.

XIV. One of the conditions laid down for the design is that H'



is not to be greater than 2/3 H or that drowning is not to exceed 66 per cent. This condition implies that so long as this relation is fulfilled, the discharge through the flume can be taken as a discharge through a free fall or, that discharge for it is given by the formula : $Q=CLH^{3/2}$ where :—

Q=Discharge.
C=A co-efficient which gives good results if taken as 3·0
L=Width of gullet of flume.
H=Height of water on the crest, as in sketch.

The mathematical proof of this well-recognised fact is given as under :—

According to D' Aubisson's theory the discharge through a contracted section is given by the formula

$$Q = CL\,(H - h)\sqrt{2gh}, \text{ see sketch above.}$$

$$= CLH'\sqrt{2g\,(H - H')}$$

Now Q will reach its maximum limit when its first differential $\frac{dQ}{dH'}$ is zero, provided its second differential $\frac{d^2Q}{dH'^2}$ is negative.

$$Q = CL\sqrt{2g}\,(H - H')^{\cdot} H'$$

$$\frac{dQ}{dH} = CL\sqrt{2g}\left[(H - H')^{\frac{1}{2}} - \frac{H'}{2\,(H - H')^{\frac{1}{2}}}\right] = 0.$$

$$\therefore\ (H - H')^{\frac{1}{2}} = \frac{H'}{2(H - H')^{\frac{1}{2}}}$$

$$\text{or, } 2\,(H - H') = H',\ 2H = 3H'.$$

$$H' = \frac{2H}{3}, \text{ which means } 66\% \text{ drowning.}$$

$$\frac{d^2Q}{dH'^2} = \sqrt{2g}\ CL\left[-\frac{1}{2(H-H')^{1/2}} - \frac{2(H-H')^{1/2} + \dfrac{H'}{(H-H')^{1/2}}}{4(H-H')}\right]$$

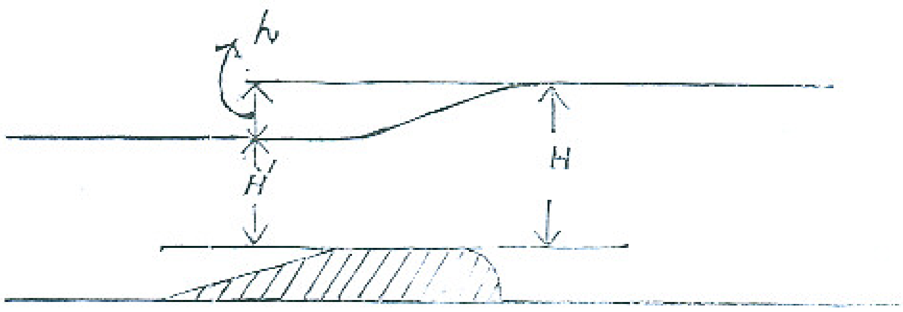
$$= \sqrt{2g}\ CL\left[-\frac{1}{2(H-H')^{1/2}} - \frac{2(H-H') + H'}{4(H-H')^{3/2}}\right]$$

$$= \sqrt{2g}\ CL\left[-\frac{1}{2(H-H')^{1/2}} - \frac{2H - H'}{4(H-H')^{3/2}}\right]$$

which is obviously a negative quantity when $H' = {}^2/{}_3\ H$.

Therefore Q reaches its maximum when $H' = \frac{2H}{3}$, and this value is a constant $\times L \times H^{3/2}$.

This matter can be proved in another way also. The discharge



through a flume is independent of the F. Supply down-stream so long as a standing wave is formed. The moment this standing wave begins to disappear the discharge becomes dependent on "drowning" and is no longer the same as for a free fall.

Imagine a condition when the standing wave is on the point of disappearance ; at this critical moment the discharge is passing from the discharge of a free fall to the discharge of a partially drowned weir and this discharge is given by the ordinary formula,

$$Q = C'\ 2/3\ Lh\sqrt{2gh} + C''\ L\ H'\sqrt{2gh}$$

Now $C' = .577$ and $C'' = \cdot 8$ (*vide* Love's Hydraulics, Art. 42)

Therefore,

$$Q = L\sqrt{2gh}\ (2/3 \times {\cdot}577h + {\cdot}8H')$$
$$= L\sqrt{h}\ (6{\cdot}4\ H' + 3{\cdot}07\ h)$$

Mean velocity through depth H' is

$$V = \frac{Q}{L\ H'}$$
$$= \frac{L\sqrt{h}\ (6{\cdot}4\ H' + 3{\cdot}07h)}{L\ H'}$$
$$= \frac{(6{\cdot}4\ H' + 3{\cdot}07h)\ \sqrt{h}}{H'}$$

Because the standing wave also exists, though it is a critical condition, the relation $V^2 = gH'$ is also satisfied (*vide* Love's Hydraulics, Art. 95). At this juncture, therefore, we have

$$V = \sqrt{gH'} \text{ and also,}$$
$$V = \frac{(6{\cdot}4\ H' + 3{\cdot}07h)\sqrt{h}}{H'}$$

$$\text{or,}\quad gH' = \frac{(6{\cdot}4\ H' + 3{\cdot}07\ h)^2\ h}{H'^2}$$

$$32\ H'^3 = 9{\cdot}42\ (h^2 + 4{\cdot}16\ H'\ h + 4{\cdot}32\ h'^2)\ h$$
$$\text{or } H' = 1{\cdot}98\ h$$
$$\text{or } H = 2{\cdot}98\ h$$
$$\text{or } H'/H = 1{\cdot}98/2{\cdot}98 = {\cdot}665,\ \textit{i.e.},\ 2/3 \text{ or } 66\ \%.$$

In other words when h becomes 1/3 H or drowning becomes 2/3rd, the weir will just cease to act as a free fall.

APPENDIX A.

Data required for Hydraulic Survey.

1. Longitudinal section showing the following:—

(1) Existing F. S. levels at every 500′

(2) ,, Bed ,, ,, ,,

(3) ,, Berm ,, ,, ,,

(4) ,, Bank ,, ,, ,,

(5) Natural surface levels ,, ,, (These should not be taken in borrow pits).

(6) Bed and F. S. levels above and below each Bridge, Fall, and Head Regulator of all offtaking channels.

(7) Bed and F. S. levels in the distributary opposite each outlet.

(8) Bed and F. S. levels in each water-course about 10′ below the tail end of the outlet.

(9) Reduced level of the bottom of each outlet orifice and R. L. of crest of any type of flume outlet.

(10) Size and type of each outlet (to be measured in inches).

(11) Line diagram of all masonry works showing dimensions and reduced levels.

(12) Width of water surface at F. S. at every 1,000′.

2. All reduced levels must be based on some well-defined bench-mark of which a sketch and full description to be given in the field-book as well as on the longitudinal section sheet. Preferably this B.M. should be on the head regulator of the channel itself.

DISCUSSION.

The Author introduced his paper and mentioned that the method of design indicated had been employed successfully in several cases. He showed three instances in which scour had been markedly brought to notice after the lapse of a whole silting season from March to October 1924. The three channels were the Manochal Branch, the Khemkaran Branch and the Dhodher Minor.

Mr. B. P. Varma said he was glad that such an interesting subject had been brought forward for discussion. He considered that the co-efficient of 3·0 appeared somewhat low, that 3·1 was nearer the mark and that on small flumes the coefficient varied. The velocity of approach was a great factor.

Mr. Bostock pointed out that on page 71 it was shown that the Silt

$$\text{Index } I_M = \frac{V_M}{0{\cdot}3\, d_M} \qquad (1)$$

But was not Kennedy's C V R a Silt Index?

$$\text{Kennedy had shown that } C\,V\,R = \frac{V_m}{V_o} = \frac{V_m}{0{\cdot}84\, d_m{}^{{\cdot}64}} \qquad (2)$$

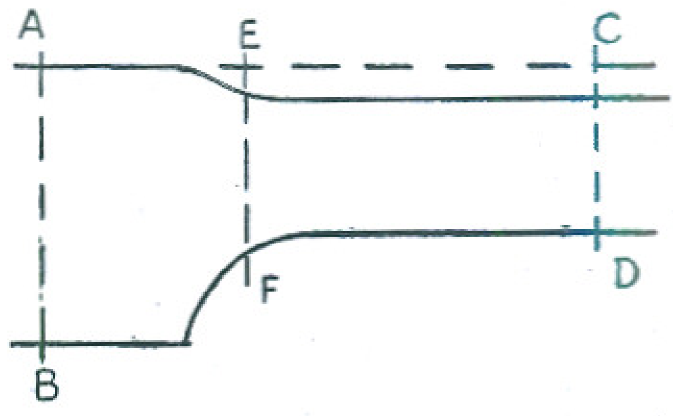
How could the equations (1) and (2) both be true? He had read the paper with interest until he found that Wood's Silt Index had been introduced and this he could not accept.

Mr. F. H. Burkitt was afraid he had not read the paper. The Silt Index question came in and he did not consider it was reasonable to hold to this principle. He failed to see what relation could exist between level of the crest of a broad crested weir and the quantity of silt going into the minor.

Mr. Iqbal Hussain asked the author to explain the meaning of the words "height of the velocity" in the last line of page 76.

Mr. F. F. Haigh said Mr. Burkitt had condemned the Wood's Silt Index theory in general.

While holding no brief for the Wood's Theory, he would like to point out that if it was properly applied to the case of the broad crested weir, it indicated that no silting could result from the introduction thereof.



The theory assumed that the silt carrying capacity of a stream at any point was proportional to the ratio of the velocity to the mean depth.

In applying the index to the broad crested weir, the Author had taken the velocity over the plane C. D. of sketch and the depth on plane A. B. Surely it would have been more correct to take the two at the same point and if this had been done, it would be obvious from consideration of any plane such as E. F. that the silt capacity must progressively increase in the curved approach and be very much higher on the crest than in the bed of the channel.

MR. W. P. THOMPSON said the Author had not made clear what he meant by the failure of the channels referred to in the paper, and that he found difficulty on following the longitudinal sections exhibited on the board, and also in understanding how such a small alteration as mentioned, *viz*., 0·25 feet in the position of the floor of the flume could make the difference between success and failure. The Author alleged that there was a lack of method in the fixation of the essential dimensions of flumes, but the speaker did not agree that such needed to be the case. Most of the channels that had to be dealt with were so conditioned that the difference between water level in the parent channel and in the offtaking channel was a small quantity. For example, if the difference of level was 5 inches, the floor of the flume had to be placed at a depth 3 × 5 inches=15 inches below water level in the parent channel. " h " was 15 inches and " l " was derivable at once from the formula in use :

If the difference in level was 2 inches, and it was found the height 3 inches×2 inches=6 inches was insufficient in that the gullet was too wide or that the " proportionality " of the distribution was affected, it became necessary to adopt the method noted by the Author and to raise the supply level in the parent channel.

If there was considerable difference of level between the water surfaces in the two channels it then became necessary to be guided entirely by the depth in the parent channel ; taking " h " as the depth or something less than the depth, if preferred, and determining " l " accordingly.

It did not seem possible to make a mistake. The coefficient of discharge applicable to the formula had been mentioned by several speakers. He had consistently found 3·1 to be the coefficient for weirs with straight approaches and 3·33 for flumes with converging approaches. This increase of nearly ten per cent. in the coefficient was also seen in the table of coefficients given on page 104 of Unwin's " Hydraulics."

MR. C. A. COLYER asked the Author whether he held to what had been stated at the foot of page 70, *viz*., that the ratio of drowning should not be more than 66 per cent. in order to make the discharge independent of the downstream supply level.

Mr. Crump and others had proved that it was possible to work up to 72 per cent. or even 80 per cent. without making any extraordinary provisions in the designs. The speaker personally designed up to 4½ times the working head and did not consider it was quite correct to state that it was essential to limit the " drowning " to 66 per cent.

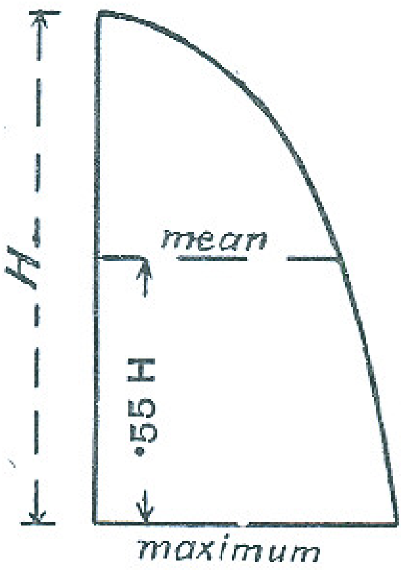
The Author had used the expression "tilting up" on page 72. The speaker thought that "tilting" meant "canting" and if a channel was "tilted" it meant the water surface, slope and all the conditions of the channel were altered. By "lifting" a channel, on the other hand, presumably the surface slope of the channel was lifted but kept parallel to the original surface slope. He asked the Author to say clearly whether he meant "tilting" or "lifting."

PANDIT JAGAN NATH remarked that the Author in the last 2 lines of page 76 mentioned that the silt depositing capacity of a channel on a masonry work depends on the height of the velocity in that channel, or through that work above the bed." Presumably, following this enunciation the Author gave the silt indices on page 71 as follows :—

(1) $$I_F = \frac{1{\cdot}5 \times 3\sqrt{H}}{R + \frac{H}{2}}$$, and

(2) $$I_M = \frac{\text{Velocity in the minor}}{{\cdot}3 \times \text{depth in the minor}}$$

From (1) it was seen that $3\sqrt{H}$ was the mean velocity through the flume, though more correctly it should have been $3{\cdot}08\sqrt{H}$. Since the velocity in the free fall was given by the formula $V = C\sqrt{2gH}$, it followed that the



profile of the velocity curve was a semi-parabola whose maximum ordinate was at the bottom and which was $1\frac{1}{2}$ times the mean ordinate, hence 1·5 appeared in the numerator to find the maximum velocity from the mean velocity. The height of this maximum velocity was only R above the bed of the parent channel, and it was not understood why $\frac{H}{2}$ had been inserted in the denominator.

If, on the other hand, the silt index of the flume was to be $\frac{\text{average velocity through flume}}{\text{height of that velocity above the bed}}$, then I_F must be $\frac{3{\cdot}08\sqrt{H}}{R + {\cdot}55\,H}$, because the mean ordinate of the parabola was at a height of ·55 H above the bottom. There was much difference between the two, and the results to achieve which, the Author laid so much stress on page 69 were vitiated at the outset.

Next, taking I_M, it was seen that ·3 appeared in the denominator. Evidently it was the height of the mean velocity filament above the bed. Velocity observations of important canals were invariably made with velocity rods, which floated with ·94 of their depth in water and ·06 above it, thus giving the mean velocity of the vertical, past which they floated. Now the static pressure on the rod was represented by a triangle whose centre of gravity was at a height of $\frac{\cdot 94d}{3} = \cdot 313d$ above the bottom of the rod and this was the point which was also the centre of pressure, hence the height of the mean velocity above the bed was ·313 d + ·06 d = ·373d above the bed and I_M should be equal to $\frac{\text{velocity in the minor}}{\cdot 37 \times \text{depth in the minor}}$.

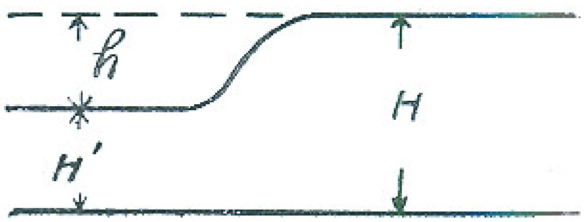
Another point to which attention was drawn was that on page 77, para. XIV the Author said that the "drowning" should not exceed 66 per cent. and this condition implied that so long as this relation was fulfilled, the discharge through the flume could be taken as a discharge through a free fall as given by the formula $Q = C\,L\,H^{\frac{3}{2}}$

This was not so. The chief criterion was that there had to be a standing wave formed downstream of the flume. As long as the wave existed, no matter what the "drowning," the head of water H on the crest was constant and the discharge through the flume entirely independent of the downstream level. On the other hand, if there was no standing wave, very little submersion would affect the discharge through the flume and either H would be changed to pass a given discharge (see Table No. 1 following) or the discharge adjusted itself if H was constant (Table 2).

The fluctuations to which the flume would be subjected, were apparent from the tables and either the parent channel would suffer or the offtake, and the formula $Q = \frac{2}{3} C L H^{\frac{3}{2}} \sqrt{2g}$ would no longer hold good but that of a partially submerged notch should be used, *viz.*—

$$Q_1 = \tfrac{2}{3} C_1 \, L\sqrt{2g}\, h^{\frac{3}{2}}$$
(free portion)

$$Q_2 = C_2 \, L\sqrt{2g}(H-h)h^{\frac{1}{2}}$$
(submerged)



$$\text{or, } Q = L\sqrt{2g}\left\{ \cdot 385\, h^{\frac{3}{2}} + \cdot 8\,(H-h)\,h^{\frac{1}{2}} \right\}$$
$$= L\left\{ 3{\cdot}08\ h^{\frac{3}{2}} + 6{\cdot}4\,(H-h)\,h^{\frac{1}{2}} \right\}$$

and without the standing wave, the maximum discharge took place, when the submersion was $\frac{1}{3}$.

Hence the standing wave was the only criterion.

When the approaches to the flume were properly shaped the wave had been seen to exist at 88 per cent. submersion, as was the case of the Sulki flume experiments in the Shahpur Division, Lower Jhelum Canal, which were witnessed by the Chief Engineer, Mr. W. P. Sangster and several engineers of the Lower Jhelum Canal.

Table No. 1.—Discharge constant, H variable (concrete example).

(See Figure on page 80 d.)

$\frac{h}{H}$	Submergence.	H. feet.	h feet.	H^1 feet.	Q. cusecs per foot length.	REMARKS.
1·00	0	*6·0	6·0	0	48·51	Free fall.
·90	·10	6·08	5·46	·62	,,	
·85	·15	*6·0	5·10	·90	,,	
·80	·20	5·95	4·76	1·19	,,	
·70	·30	5·87	4·11	1·76	,,	
·67	·33	5·86	3·91	1·95	,,	Minimum H.
·60	·40	5·87	3·52	2·34	,,	
·50	·50	5·94	2·97	2·97	,,	
·45	·55	*6·01	2·70	3·31	,,	
·40	·60	6·11	2·44	3·67	,,	
·33	·67	6·33	2·11	4·22	,,	
·30	·70	6·45	1·94	4·51	,,	
·20	·80	7·10	1·42	5·68	,,	
·10	·90	8·62	·86	7·76	,,	

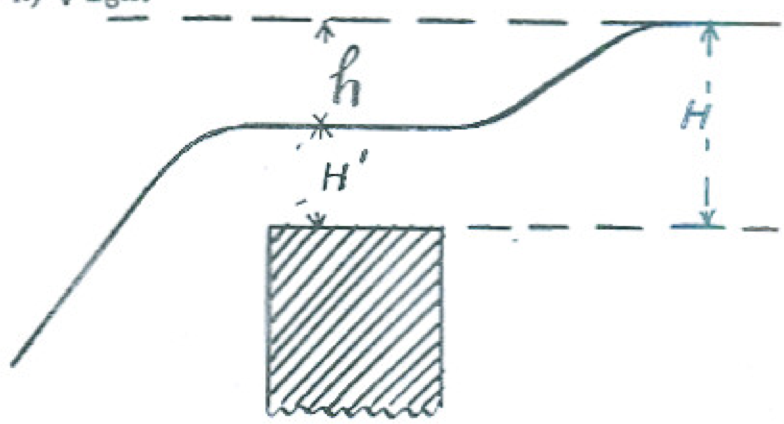
*Denotes submergence having total head H=that of the free fall and passing the same discharge.

Table No. 2.—H constant, Discharge variable (concrete example).

h.	H^1.	*Q. per foot length of weir in cusecs.*			REMARKS.
		Freefall.	Submerged portion.	Total.	
1	5	3·30	32·00	35·30	5/6 submersion.
2	4	9·33	36·20	45·53	$\frac{2}{3}$,,
3	3	17·15	33·26	50·41	$\frac{1}{2}$,,
$3\frac{1}{2}$	$2\frac{1}{2}$	21·61	29·93	51·54	5/12 ,,
4	2	26·40	25·60	52·00	$\frac{1}{3}$,, (maximum).
$4\frac{1}{2}$	$1\frac{1}{2}$	31·50	20·36	51·86	$\frac{1}{4}$ submersion.
5	1	36·89	14·31	51·20	1/6 ,,
6	0	48·51	0	48·51	No submersion.

Again D'Aubuisson's formula quoted by the Author on page 77 was out of place. It was given that—

$$Q=CL\,(H-h)\sqrt{2gh}.$$



The discharge passing through depth H^1 was equal to the discharge passing through depth H. Now to eliminate the drawing down effect of water over the weir, the head H was always measured well upstream of the weir and then the discharge was computed by the formula, $Q=\frac{2}{3}C\,L\sqrt{2g}\,H^{\frac{3}{2}}$ (1)

H was contracted down to H^1 and D'Aubuisson's formula as quoted above gave the discharge at H^1 as $Q=C\,L\,(H-h)\sqrt{2gh}$ (2) as if it were a submerged orifice under head "h."

(1) and (2) were equal, therefore,

$$\tfrac{2}{3}\times \cdot 577\ L\ \sqrt{2g}\ .\ H^{\frac{3}{2}} = C\ L\ \sqrt{2g}\ (H-h)\ h^{\frac{1}{2}}$$

$$\text{or } \cdot 385\ H^{\frac{3}{2}} = C\ (H-h)\ h^{\frac{1}{2}}.$$

Putting $h=\frac{H}{3}$, or taking the "drowning" as $\frac{2}{3}$rds, as the Author had taken,

$$\cdot 385\ H^{\frac{3}{2}} = C \times \frac{2\ H^{\frac{3}{2}}}{3\sqrt{3}}$$

Therefore $C = \frac{\cdot 385 \times 3\sqrt{3}}{2} = \cdot 998$ or practically 1.

Did this not mean that the velocity of every filament of water at the section H^1 had been assumed to be the same throughout, from top to bottom and the depth H^1 represented the ideal "vena contracta" of H? It followed, therefore, that H^1 was not the "drowning" of the weir, but on the other hand "contraction," or "squeezing" of depth H to H^1 due to the drawing down effect of the water over the weir. The formula of D'Aubuisson was simply another form of formula (1) above with $C=1$, and contracted depth of water H^1, and in no way represented the "drowning" of the weir, but clearly shewed that the greatest depth the "vena contracta" could have was $\frac{2}{3}$ H.

The Author had made use of this formula to arrive at the maximum "drowning." The Q over the weir was a constant quantity for efficient regulation under full supply conditions and could have no maximum or mininum. The only thing which varied was the "vena contracta" H^1 according to the value of C and drawing down effect of water, hence Q could not be differentiated with respect to H^1 but the value of H^1 could be arrived at by the method shown above.

The second alternative which the Author had adopted on page 78 to find the "drowning" at the critical moment of the standing wave was better than the attempt on page 77.

This, however, could have been curtailed and made simpler by the following reasoning:—

As long as there was a standing wave, the discharge over the weir was that over a free fall and $Q=\frac{2}{3}CL\ \sqrt{2g}\ H^{\frac{3}{2}}$.

When the standing wave was just disappearing the velocity through $H^1 = \frac{2\times CL\times\sqrt{2g}\ H^{\frac{3}{2}}}{3\times L\times H^1}$, and for critical moment of standing wave, this must equal $\sqrt{gH^1}$ therefore,

$$\cdot 385\ \sqrt{2g}\ H^{\frac{3}{2}} = \sqrt{g}\ H^{1\frac{3}{2}}, \text{ and}$$

$$\frac{H^1}{H} = (\cdot 385\sqrt{2})^{\frac{2}{3}} = \cdot 66\dot{6}$$

or, exactly $\frac{2}{3}$rds.

This equation was easier than the cubic quoted on the paper which could only be solved by trial and error.

This submergence, however, assumed that the bed was horizontal, and if the bed was sloping, which was practically always the case, the "drowning" would be much more. It was the speaker's opinion that the greater the velocity at the cill the greater would be the amount of silt sucked by the offtake from the parent channel.

Mr. M. D. Mithal criticised Pundit Jagan Nath's remarks shewing that the centre of mean velocity did not lie at ·3d but at ·373d above the bed and thought that he had confounded centre of static pressure with height of mean velocity. These terms, however, were not synonymous by any means because, whereas the static pressure in the liquid in a channel increased with the depth and therefore the centre of pressure was at two-thirds the depth, the velocity of water decreased with the depth generally.

Mr. Natha Singh asked whether the Author had ever cured the silting tendency of a channel without "tilting" up the supply. He drew attention to paragraphs VI and XI (*b*) which seemed contrary to one another, in one case the Author advocating keeping the crests at the same level while on the other case he advocated raising the crest of the minor.

Mr. Balkishan Kapur agreed with the Author that before the construction of a flume at the head of a channel, a hydraulic survey of the channel should be made and carefully examined and the channel remodelled so as to give the flume a fair chance.

Regarding para. VI (*a*) where it was stated that the flumes should be constructed in a pair, one in the minor and the other in the parent channel just below the offtake of the minor, he considered that on account of the disturbance in the water due to the flume, the heavy silt would rise up and be drawn into the minor, and therefore suggested it would be better to construct the flume in the offtaking channel at least 50 feet upstream of the parent channel flume. If this were not possible, the flume in the minor should be placed about 50 feet below the point of offtake, the latter being left open. In para. X (*b*), it was stated that the maximum amount of silt could theoretically be equal to the amount of "tilting," and that when this amount of silt had deposited and the new surface slope became parallel to the original, no more permanent silting could take place. This was true, but when the new surface slope ultimately became parallel to the original, it should be called the "lifting" and not the "tilting" of the supply. When the supply was "tilted" the backwater effect was felt only for a mile or two depending on the water surface slope and the amount of "tilting," but gradually the bed silted up to a new slope and the water surface became parallel to this new slope which generally differed very slightly if at all from the original surface slope, and in fact was most likely to be parallel to it.

Thus in the example quoted by the Author in para. IX (*d*), though the immediate effect of the "tilting" only extended up to a distance of 7,272 feet, the ultimate effect would be felt right to the fall next above. The quantity of earthwork as calculated by the Author seemed incorrect as the earth to be put on the slopes of the channel appeared to have been neglected and even assuming that the effect only extended up to 7,272 feet, the cost would not have been less than Rs. 500. The ultimate cost however might be much more, and if the channel had a patrol road and was in high embankment as was generally the case when the command was poor, the cost might be anything up to Rs. 10,000 or more.

It would not be out of place to mention that the supply at R. D. 37,700 of the Kotbhai Distributary of the Bhatinda Branch, Sirhind Canal, was tilted up about a foot some two years previously and the banks strengthened for 7,700 feet upstream of the fall, as suggested by the Author, but due to the exceptionally heavy floods of the summer of 1924, the channel, instead of silting up gradually, silted up at once and in October 1924, when demand for canal water became keen, it was found that the distributary was not strong enough to carry its full supply. About Rs. 3,000 had already been spent on strengthening the banks of the channel and at least Rs. 3,000 more would have to be spent, although about 2 miles of the channel was in cutting. The speaker thought that in places where command was poor, it was better, however, to raise the supply even at heavy cost, rather than to leave the channel alone and have recourse to repeated silt clearances and "tilting" of outlets and minors for feeding the tails of the system.

The Author had made use of Mr. Wood's Silt Index Theory, in determining the design of the flumes. The speaker was of opinion that if a flume was designed to give H equal to about 3h, after carefully remodelling the channel, it would work without trouble and if it still gave trouble, the fault lay in the parent channel, or in the relative positions of the flumes and no silt theory could correct it.

In para. XIV it was shewn that the discharge through a submerged weir was maximum when the "drowning" was 66 per cent. and the discharge with this submersion was the same as through a free fall.

This had been proved by the help of D'Aubuisson's theory which shewed that the discharge through a contracted section was

$$Q=CL\sqrt{2gh}\,(H-h) \qquad (1)$$

For a free fall, h=H, and from this Q=O, which evidently was not true.

If the ordinary formula for a submerged weir was considered —

For the free fall : $Q = \frac{2}{3}CL\sqrt{2gh} \times h$

For the submerged portion : $Q = CL\sqrt{2gh}\,(H-h)$

Total discharge $Q = CL\sqrt{2gh}\,(\frac{2}{3}h+H-h)$

$$= CL\sqrt{2gh}\,(H-\frac{h}{3}) \qquad (2)$$

Differentiating with respect to h and equating to zero—

$$\frac{H-\frac{h}{3}}{2\sqrt{h}}-\frac{\sqrt{h}}{3}=0$$

$$\text{or, } H-\frac{h}{3}=\tfrac{2}{3}h, \text{ or } h=H,$$

i.e., Q was maximum when H=h, or in other words when the fall was free.

In the table given below, the values of discharges had been worked out for H=1·0 and L=1·0, from the formulæ (1) and (2) above, taking C=1·0 and $\frac{2}{3}$ respectively.

h =	0	0·1	0·2	0·3	0·4	0·5	0·6	0·7	0·8	0·9
Submersion =	0	10%	20%	30%	40%	50%	60%	70%	80%	90%
Q from (1) =	..	..	2·03	2·06	2·48	2·83	3·04	3·07	2·86	2·28
Q from (2) =	3·56	3·55	3·50	3·44	3·30	3·14	2·92	2·63	2·23	1·63

It was evident that there was a fallacy in the use of the formula (1) and this appeared to be a wrong assumption that C was a constant.

The following observations made by the speaker on a 3 inch wide open flume with H=2·6, constructed according to the Type Design given in Appendix D Irrigation paper No. 26 also confirmed this.

Submersion =	0	50%	60%	66%	75%	80%	85%	90%	91%
Discharge =	Unity	96%	94%	92%	90%	88%	85%	75%	67%

It was inconceivable that in a broad crested weir the discharge with a free fall should not be more than with 66 per cent. submersion as stated in the paper.

MR. COLYER did not agree with Pundit Jagan Nath's statement that the discharge over a broad crested weir was constant so long as a standing wave was formed. He had thought so himself at one time but had discovered later that this was wrong and that it was possible to get a slight reduction of discharge while still a definite standing wave was formed.

MR. BURKITT stated that the coefficient of a weir depended entirely on its shape and that where the glacis was steeper than 1 : 5 the standing wave did not work as a meter unless the crest of the weir was sufficiently wide.

THE AUTHOR replied to the discussion and agreed with Mr. Varma that on small flume outlets, the coefficient varied to a certain extent but so long as the flumes were used as proportionate distributors, the exact value of the coefficient did not matter. He considered that the question of velocity of approach was of so little importance in the cases of distributaries and minors, that it could well be ignored.

Mr. Bostock had questioned the value adopted for I_M, the Silt Index of the minor, and had expressed it as a function of the critical velocity ratio and depth thus :—

$$C\ V\ R = \frac{V_M}{V_o}, \text{ also } V_o = 0{\cdot}84\ d_M^{\cdot 64}$$

$$\text{Now } I_M = \frac{V_M}{0{\cdot}3 d_M} = \frac{C\ V\ R \times V_o}{0{\cdot}3 d_M}$$

$$= \frac{C\ V\ R \times 0{\cdot}84 d_M}{0{\cdot}3 d_M} = \frac{C\ V\ R \times 2{\cdot}8}{d_M^{\cdot 36}}$$

where the channel was in perfect regime, C V R=1.

Therefore for channels in regime, which were neither silting nor scouring,

$$I_M = \frac{2{\cdot}8}{d_M^{\cdot 36}}, \text{ but for other channels, } I_M = \frac{C\ V\ R \times 2{\cdot}8}{d_M^{\cdot 36}}$$

This was only a different manner of expressing the Silt Index and in no way affected the usefulness of the theory. Mr. Burkitt had condemned the Silt Index Theory. The Author suggested that he should design a few flume heads on the lines indicated in the paper and test the results for himself. He apologised for the omission of the words "of the line of mean," after the word "height" and before the word "velocity" in the last line of page 76 to which attention had been drawn by Mr. Iqbal Hussain. Mr. Haigh had supported the Author's own contention that the Silt Index was not absolute but comparative.

In reply to Mr. Thompson, the Author mentioned the history of the Head Regulator of the Khemkaran Branch, Upper Bari Doab Canal. This channel was an offtake from the Khemkaran Distributary and had been giving silt trouble for many years ; its authorized discharge was 84 cusecs, but it could not carry more than 60. In 1921 the local canal officers referred the matter to Mr. Crump who designed a pair of proportionate flumes for the fall below the offtake in the parent channel and for the head of the branch. There was no improvement however and the matter was again referred to Mr. Crump who advised that a divide groyne should be put in with its intake narrower than the outlet. This was done but still the trouble was not cured. The matter was again considered and the changing of the alignment of the channel, shifting the head up so as to give a skew offtake, was seriously contemplated. This would have meant heavy expense and as a trial, the crests in both of the flumes were raised three inches with the result that silt got washed out automatically and the channel worked properly.

Mr. Colyer had raised questions as to the extent of "drowning" permissible and on the meaning of "tilting." These objections had already been dealt with in the paper itself.

The Author invited Pundit Jagan Nath to enter into correspondence with him on the mathematical points raised. It was obvious that "the centre of static mean pressure" and "line of mean velocity in a moving liquid" which Pundit Jagan Nath had assumed to be the same, were not synonymous terms. He referred Mr. Natha Singh to the Dudher minor, the longitudinal section of which had been exhibited during the discussion, as a case where silting tendencies had been cured, the "tilting" being only about one inch. He referred Mr. Balkishan Kapur to Paper No. 26 in the Proceedings of the American Society of Engineers for a complete discussion on the application of D'Aubuisson's formula for discharge through a free fall.

Subsequently Communicated.

Mr. Iqbal Hussain wrote that the Author had recommended "tilting" up of the water surface to gain head in a minor of poor command, without apprehending any danger of silting, while on the other hand in para. IV (*c*) it had been stated that with drowned bridges, tight falls, etc., silting was expected to result. The two statements were contrary.

With regard to para. IX (*a*), he did not consider it to be the primary object of a canal to irrigate by flow, as suggested by the Author. It was not sound engineering to head up water excessively simply in order to command a small area of land.

He did not agree with the theory expressed in para. X (*b*). What actually took place when the water surface was "tilted" was that silt continued to accumulate and the backwater effect, in due course disappeared.

The Author replied that Mr. Iqbal Hussain was not correct in stating that he (the Author) had laid down that silting was not apprehended when the water surface of a channel was tilted to give better command, and referred to paras. X (*a*) and (*b*) of the paper.

It was certainly not economical engineering practice to provide lift irrigation, earning half the flow irrigation rates, so long as it was practicable to provide flow irrigation for an area, and there were many cases where the cost of "tilting" up the water surface in channels had been repaid several times over in one year, by the increased revenue earned.