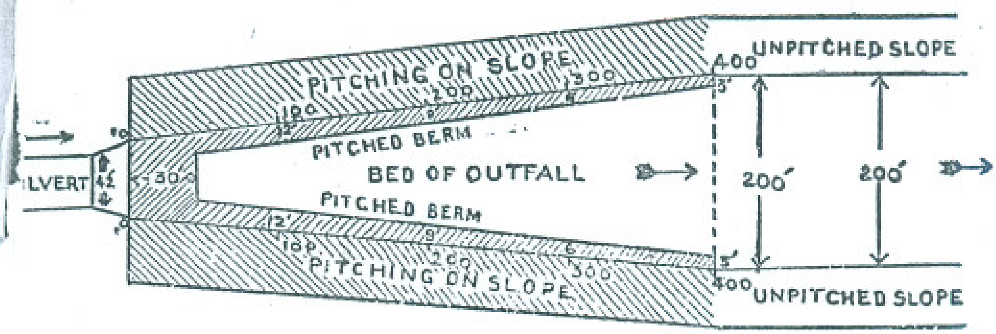
The plan of the outfall bed may now be drawn and would ... as follows :



It will be obvious that the pitching in the berm will decrease in width from fifteen feet at zero to say three feet at R. D. 400.

FLUMES.

Masonry Flumes, Upper Swat River Canal.—

In order to determine the value of Kutter's "N" in brick flumes, some observations were made by Mr. S. H. Bigsby, ... ssistant Engineer, and Mr. Fakhar-ud-din, Assistant Engineer, ... masonry flumes irrigating Peshawar, and the following ... ere obtained :—

... scharge in cusecs.	Bed slope.	Velocity.	H. M. D.	Value of Kutter's "N".
a...	1/118·04	3·12 f. s.	not given	·01496
... ...	1/35·5	9·67 „	·238	·0117
... ...	1/158·75	5·88 „	·299	·01185
...8	1/81·9	6·25 „	·286	·013895
1·605	1/75·75	4·28 „	·1875	·014253

So that the average value of N was ·01333 for these brick flumes.

Many of the distributaries on the Upper Swat Canal pass through country which falls rapidly. This is especially the case on the Maira and Indus Branches where distributaries irrigate the slopes descending from a table-land. Where the slope was great, flumes, lined with bricks or with concrete, were found more economical than earthen channels with falls at frequent intervals. The velocity in these flumes was limited to six feet per second, and an example of the method of designing them is given below—

Discharge 5·85 cusecs. Velocity 6 f. s.

Take N = ·013. Try bed 1·25 feet wide, and F. S. 0·65 feet. Using a channel of the best discharging section,* by Manning's formula.†

$$\text{Velocity} = \frac{1{\cdot}4858}{{\cdot}013} R^{\frac{2}{3}} S^{\frac{1}{2}}$$

$$\text{therefore } 6 = 114{\cdot}3 \times R^{\frac{2}{3}} \frac{1}{L^{\frac{1}{2}}}$$

$$\text{Or } L^{\frac{1}{2}} = 19{\cdot}05\, R^{\frac{2}{3}}$$

$$\text{Sectional area} = {\cdot}81.\ \text{W P} = 2{\cdot}55 \ \therefore\ R^{\frac{2}{3}} = \left(\frac{{\cdot}81}{2{\cdot}55}\right)^{\frac{2}{3}}$$

$$\log {\cdot}81 = \bar{1}{\cdot}90363$$

$$\log 2{\cdot}55 = {\cdot}40654$$

$$\therefore \log R = \bar{1}{\cdot}49709$$

$$\therefore \log R^{\frac{1}{3}} = \bar{1}{\cdot}83236 \ddagger$$

$$\text{And } \log R^{\frac{2}{3}} = \bar{1}{\cdot}66472$$

$$\text{But } L^{\frac{1}{2}} = 19{\cdot}05\, R^{\frac{2}{3}}$$

$$\text{And } \log 19{\cdot}05 = 1{\cdot}27989$$

$$\log R^{\frac{2}{3}} = \bar{1}{\cdot}66472 \text{ as found above}$$

$$\therefore \log L^{\frac{1}{2}} = 0{\cdot}94461$$

$$\text{Or } \log L = 1{\cdot}88922$$

$\therefore L = 77\frac{1}{2}$ which gives the proper slope for the given discharge and velocity.

* (Pages 300-301, Gibson's Hydraulics).

† Page 118, Buckley's Irrigation Pocket Book.

‡ Page 721, Molesworth, 27th ed.

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GUIDE BANKS OR TRAINING BANKS.

Very useful information on the su ect of guide bunds will be found in Spring's "River Trainin and Control" from which the following has been abstracted.

The length of guide banks upstream should be equal to the length of the bridge, while the downstream length may be 1/10th to 1/5th the length of bridge. The ends of guide banks should be curved well back, 120° to 140°.

Where the length of a bridge is much less than the width of the sandy bed of the river, impregnable heads are required at the extremities of the upstream guide banks. To design these, first ascertain the maximum depth of scour, by means of soundings taken in the deepest pools (or by the method based on Kennedy's formula) and make the width of the stone apron twice the greatest scour depth.* The top width of the impregnable head should not be less than that depth (which is usually twenty to thirty feet), and the base width five times the scour depth. There should be four to five feet of free board.

Rivers having a bed slope up to one and a half feet per mile, with sandy beds, may safely be contracted by guide banks to a width, which will cause an all-over mean scour of from eight to sixteen feet between abutments.

Side slopes of guide banks should not be less than 2 to 1 ; and where the material is sand, one to three feet of clay covering should be given to resist wave-lap, and vegetation encouraged. Twenty per cent. should be allowed for settlement.

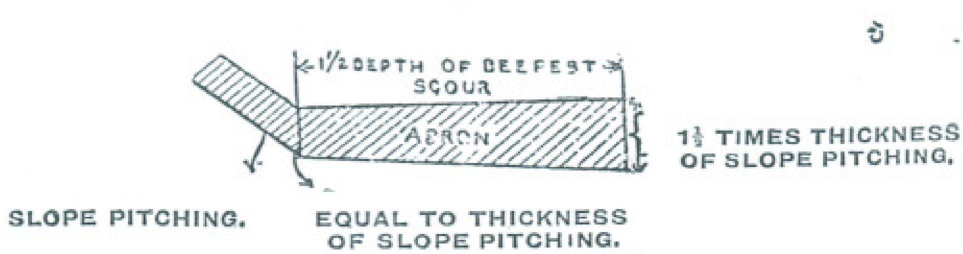
Stone pitching of large rough stones, or of block kankar, is preferable to boulder pitching (which easily rolls away) on the water slope, and underneath this six inches of shingle or ballast, or two layers of bricks, is necessary. The thickness of the stone pitching, excluding the shingle, should be in accordance

* In rapid torrents near the hills the width of the apron is made $2\frac{1}{2}$ times the depth of the scour.

with the slope of the river bed, and the classification of its sand.

Kind of sand.	SLOPE OF RIVER BED.				
	1/21,120.	1/7,040.	1/5,280.	1/3,520.	1/2,640.
Very coarse sand, mesh 10,16, 30	10″	12″	15″	18″	21″
Coarse sand, mesh 30 to 40	15″	18″	21″	24″	27″
Medium sand, mesh 40 to 75	21″	24″	27″	30″	33″
Fine sand, mesh 100 ..	24″	27″	30″	33″	36″
Very fine sand, less than mesh 100 ...	30″	33″	36″	42″	45″
Probable velocity along guide bank ...	6·7 f.s.	8·8 f.s.	9·4 f.s.	10·5 f.s.	

With regard to the apron of the guide bank (excluding the impregnable head for which the width of apron has already been specified) make its width one and a half times the depth of the deepest scour, and its mean thickness one and a quarter times the thickness of the slope pitching (tabulated above) as sketched below.



Further details regarding the design of the impregnable heads are : the heads must be turned round to face the current from the embayments. The thickness of pitching on the slopes of the impregnable heads, should be twenty-five per cent. more than on the guide bank, while the width of the apron should be twice the depth of the maximum scour. The following table gives the radii proposed for the curves at the impregnable heads :—

Kind of sand.	Probable maximum scour.	SLOPE OF RIVER BED.				
		1/21,120.	1/10,560.	1/7,040.	1/5,280.	1/[illegible]
		Radii for impregnable heads in feet.				
Very coarse sand ...	Under 20′ ...	200	250	300	350	400
Very coarse sand ...	Over 20′ ...	250	310	375	440	500
Coarse sand ...	Over 30′ ...	350	430	510	590	670
Medium sand ...	Over 40′ ...	450	550	650	750	850
Fine sand ...	Over 50′ ...	600	725	825	925	1,020
Very fine sand ...	Over 60′ ...	800	900	1,000	1,100	1,200

Note.—The radii of the curves for the downstream ends of the guide banks should be half these dimensions.

Sir Francis Spring states that a *thick* armour of rough stone pitching will withstand a velocity of eighteen feet per second.

The training banks of the Lower Chenab Canal Headworks, where the river has a slope of about 1/3100 are of sand armoured with one foot of clay, six inches of ballast, and eighteen inches of hand-packed stone.

At the headworks of the Lower Jhelum Canal, the left training bank was armoured with thirteen to fifteen inches of hand-packed stone over a layer of ballast while the apron is 30 feet × 4 feet. The maximum depth of scour is sixteen to twenty feet, at which depth harder material is reached. The right training bank is armoured with eighteen inches of hand-packed stone (one flat and one on edge) on six inches of ballast, $1\frac{1}{2}''$ gauge. This apron is 60 feet × 4 feet.

At the larger Suketar level crossing on the Upper Jhelum Canal, the training banks have aprons fifty feet wide, and four feet average thickness at points subjected to strong attack ; and forty-five feet wide, and three feet average thickness where the attack is not so severe. The top width is thirty feet. These were designed by using Kennedy's formula. An extension of this formula to ascertain the amount of silt carried by scouring velocities is given in Engineering News, dated 8th June 1911, and in Kennedy's Diagrams.

$$\text{Amount of silt carried by given scouring velocity} = \text{Amount of silt carried by equilibrium velocity} \times \left(\frac{\text{scour velocity}}{\text{equilm. velocity}}\right)^{\frac{5}{2}}$$

Similarly the amount of silt deposited by a deposition velocity is given by the following formula :—

Silt carried by equilm. vely.— Silt carried by deposition vely.

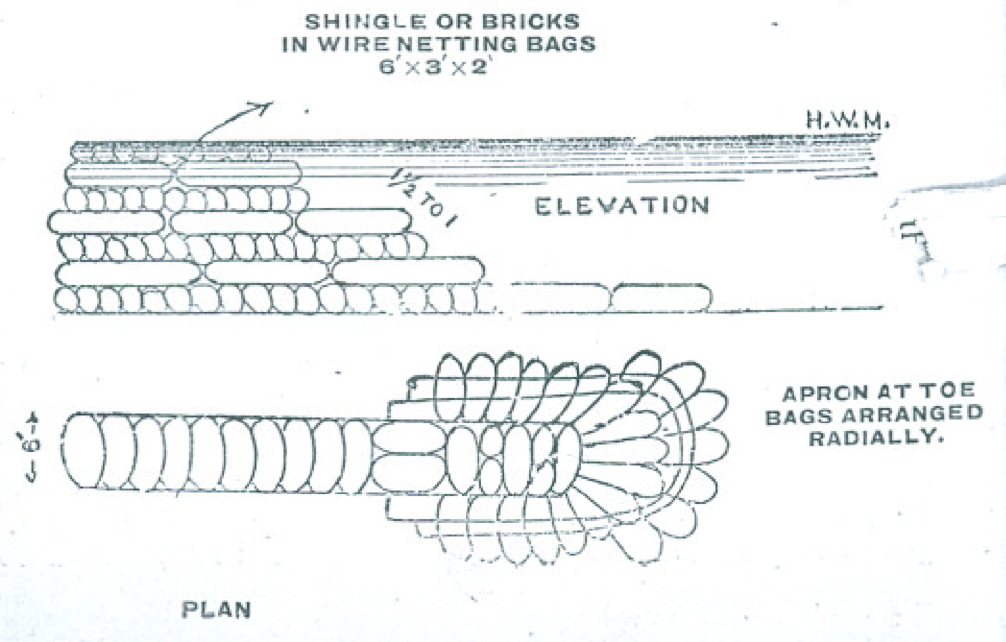
i. e mount of deposition

$$= \text{Silt carried by equilm. vely.} \times \left\{ 1 - \left(\frac{\text{deposition vely.}}{\text{equilm. vely.}}\right)^{\frac{5}{2}} \right\}$$

The rate of scour or deposition can thus be calculated, and is useful in estimating the time required for the clearance of siphon barrels, or the time required to silt up a silting or warping reach.

A table of 5th powers is given at page 747 of Molesworth (27th edition), and of $\frac{5}{2}$ powers at page 47 of Buckley's Irrigation Pocket Book. Kennedy takes the transporting power of water as approximately $V^{2\cdot5}$, hence the $\frac{5}{2}$ powers shown in the formulæ.

The use of galvanized wire (formed into a network) has been found to materially assist both the slope pitching and the apron at points of very strong attack. Ordinary rabbit netting, made into bags 6 ft. × 3 ft. × 2 ft. and filled with shingle or old brick bats, affords good protection in spurs of moderate height and short length. The bags can be laid on top of each



other to form a dike or wall sloping down to the toe, the toe being protected as usual. Clay and vegetation soon becomes arrested by the wire and the spur becomes bound together by them, before the wire rus away.

In considering methods and effects of scour, it is necessary to keep in view the fact that scour at *obstructions* such as piers, is not due to frictional flow, but to impingement of the lower curve of the jet, which acts like falling water, and may cause an abrupt scour twenty to forty feet deep where the impingement of the jet is direct and not oblique. The heaviest protection is therefore required up against the obstruction, to check the down sweep and hollowing action of the jet. The receiving angle, the width of obstruction, the velocity, depth, fineness or looseness of bed material, and the pre-existing silt-charge, all affect the scour.

Dr. J. P. Stratton in "Analysis of movements and scour action of water at obstructions" advises that the width of the protection at an obstruction should be equal to the utmost breadth of the down-sweep produced by the maximum flood. Heavy material is necessary at the top, light material can only be used at some depth down, since the greater the depth the less the power of the jet to eject the material.

Scour is less intense at the downstream ends of piers. Also a sloping obstruction is less liable to scour than a vertical one, because the slope deflects the down sweep in an upward direction. The pressure of water in motion is $1{\cdot}8\ V^2$.

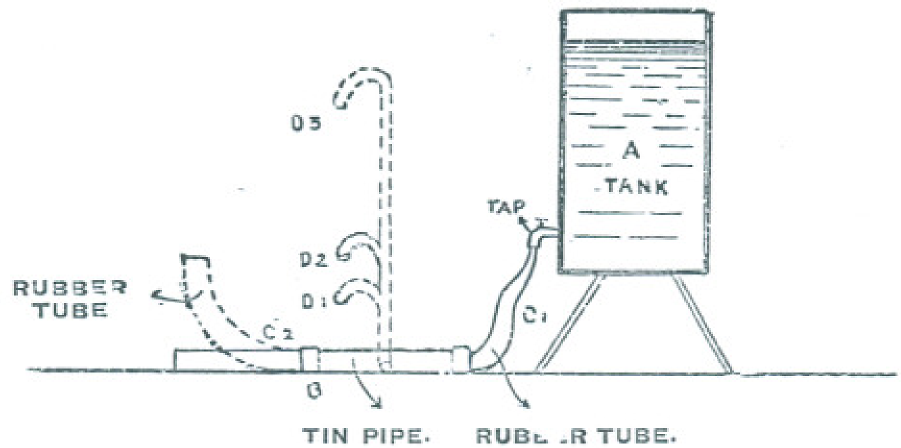
DRAINAGE SIPHONS AND CULVERTS.

The intensity of hydrostatic pressure against the soffits of siphons is sometimes miscalculated. Where afflux has been induced upstream in order to increase the velocity through the barrels, the additional head obtained to generate the increase does not exert any hydrostatic pressure on the barrels, and in no case is it correct to join the water surfaces upstream and downstream by a line and call it the hydraulic gradient.

When the water surface downstream of the work does not rise above the soffit, the hydraulic gradient is very low, being merely that due to the head necessary to overcome friction in the barrel.

When the downstream end of the siphon is turned up, but the water surface does not rise above the downstream soffit, the hydraulic pressure on the barrels is that due to the amount of turning up, plus that due to the friction head. When the surface of the water downstream rises above the soffit, and the end of the barrel is turned up, the pressure is that due to the extent, of the rise above the soffit, plus the amount of the turn-up, plus the friction head.

Conditions of pressure alter radically if there is a complete stoppage of the flow because the whole head upstream immediately comes into operation. This might occur in very small culverts if they became completely blocked. The following experiment, which can easily be carried out by any one, will show what is meant :—



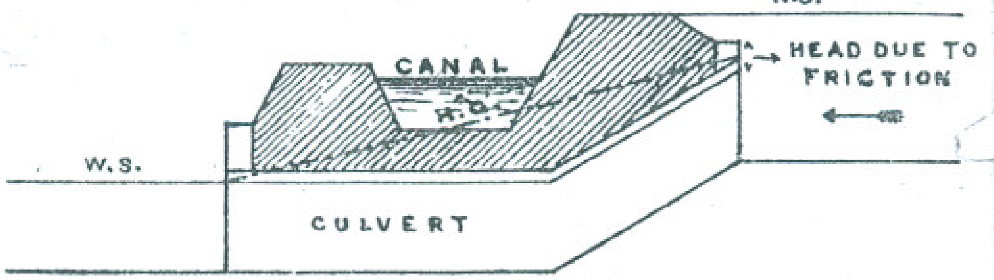
A is a tank containing water, B is a length of pipe or tin tube with a small orifice in its top (at the position shown by the jets, D_1, D_2, D_3.) C_1 and C_2 are lengths of rubber tubing, the former being attached to the tap which controls the supply from the tank, and the latter to the extremity of the downstream end of the pipe.

Case I.—First lay C_2 down flat and turn on the tap. It will be found that the jet, issuing from the orifice, rises to a small height D_1 which is independent of the great head in the tank, and is merely due to the resistance or friction offered in the length of pipe and rubber tube situated *downstream* of the orifice.

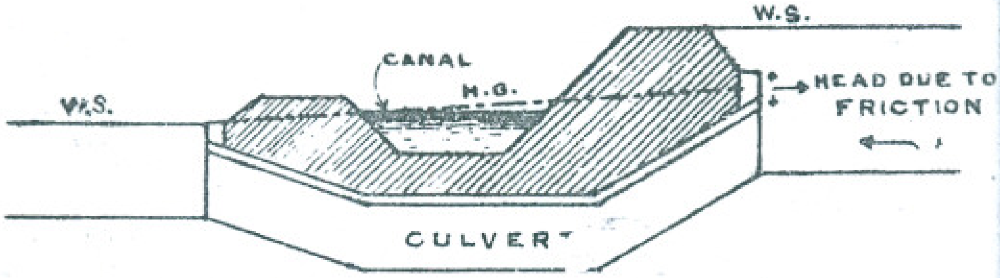
Case II.—Now raise the end of the rubber tube C_2 to the position shown by the dotted lines, and it will be found that the jet will raise to D_2 showing that the internal pressure in the pipe, in the vicinity of the orifice, has been increased by the height to which C_2 has been raised ; that is to say the height of the jet is due to the resistance or friction downstream of C_2 including additional friction introduced by bending C_2, plus the height to which the water surface of C_2 has been raised.

Case III.—Next lay C_2 down flat again, but close it by pressing on the rubber so that the flow is *completely*

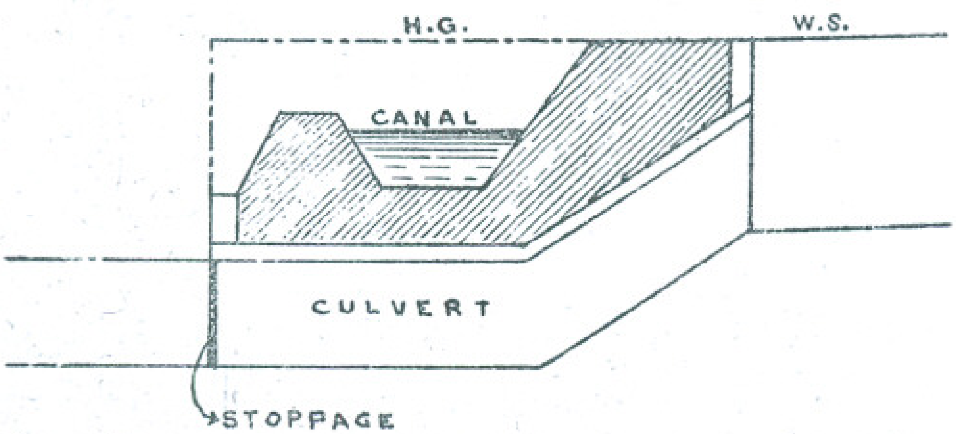
CASE I



CASE II



CASE III



pted. There is then a complete stoppage of the flow, e jet will rise to D_3, because the internal pressure is no independent of the full head in the tank A. The height jet will represent the full head less frictional losses. water supply pipes which are liable to complete stoppage have to be designed for this third case : but culverts, are not liable to interruption of flow, fall under Case I or

The above experi t will also illustrate the great advan- scored by provid weep holes in wing walls, in floors of siphons or depressed works, and in dams, *i. e.*, by obtaining *motion*, the full head of pressure is not brought into action, and the blowing up pressure falls under Case I or II instead of under Case III.

The attached plate shows the hydraulic gradient in the three cases.

In the first case, although the water level may be headed up on the upstream side, in order to generate a good velocity through the culvert, the hydraulic pressure within the barrels is merely influenced by the friction.

In the second case the barrel pressure is again independent of the heading up on the upstream side, but is influenced by the height to which the water surface has been raised above the soffit, of the middle of the barrel, on the downstream side, as well as by the friction within the barrel.

The third case, in which the full head would exert hydraulic pressure on the barrels, could occur in practice only if the culvert became absolutely choked, and the flow was completely interrupted.

It is a good plan, when designing the thickness of soffits and the superincumbent load, to allow a reserve of one-third over and above the weight necessary to counterbalance the fluid pressure.

The principles stated above are Bernouilli's theorem,* *viz.*, Static head = velocity head + pressure head (when friction is neglected and the flow is steady). The theorem may also be stated thus : Gravity head + pressure head + velocity head = a constant quantity at any point of the same pipe or siphon (neglecting friction and impact, and assuming that the flow is steady).

* $H = h + \frac{v^2}{2g}$

Design of Masonry Siphons.—

T. Claxton Fidler's "Calculations in Hydraulic Engine Part II" affords great assistance in designing masonry si Page 74 gives valuable co-efficients for masonry barrels, al rivetted steel tubes and pipes, while page 161 gives a fo for the total head, based on these co-efficients. This form as follows :—

$$\text{Total Head} = \frac{V^2}{2g}\left(1 + \text{hd lost at entry}\right) \cdot \text{..gth of } \ldots \text{ (less two diame} \ldots$$

$$\times \text{ co-efficient of friction} \times \frac{(\text{Barrel velocity})^{1\cdot8}}{(\text{H. M. R.})^{1\cdot2}}$$

the co-efficients 1·8 and 1·2 being those suited to rough bric work.

In calculating the discharge through the barrels, the squar root of velocity head is taken for use in the formula,

$$\text{Discharge} = \text{Sectional area} \times \sqrt{2\,gh}$$

In designing the arch and its load, the upward pressure i taken as that due to the friction in the barrel, plus the amount o turn-up downstream, plus the height of the downstream wate level above the soffit at exit. The velocity head, and the hea lost at entry do not affect the upward pressure, so long as the siphon is in flow.

CHANNELS.

When designing channels the values of "C" (the co-efficient of velocity) calculated by the use of *Bazin's* co-efficients will be found in Appendix I of Love's *Hydraulics*, or the value may be calculated from the formula.

$$C = \sqrt{\frac{2g}{\text{co-efficient of friction}}}$$

Where the co-efficient of friction $= \alpha\left(1 + \frac{\beta}{R}\right)$

R = the hydraulic mean depth, and the values of α & β are as follows:—

		α	β
I.	Cement and planed planks ...	·003 ...	·1
II.	Ashlar and brickwork	·004 ...	·2
III.	Rubble masonry and stone pitching	·005 ...	·8
IV.	Earth	·00592 ...	4·1

The values of "C" given in Love's *Hydraulics* are calculat-
or earth.

If it is desired to use Kutter's values for C, the tables given ppendix II of Love's *Hydraulics*, or pages 566 to 570 of utwine (1913 issue) should be employed.

Flynn and Hering's simplification of Kutter's formula for s—

$$C = \frac{K}{1 + \frac{x}{\sqrt{R}}}$$

Values of K and x for various values of N are :—

N	K	x
·010	225·6	·4447
·011	209·13	·4891
012	195·41	·5336
·013	183 85	·5781
·014	173·85	·6225
·015	165·23	·6670
·016	157·68	·7115
·017	151·0	·7560

"C" is practically constant for all values of slopes steeper then 1/1000.

Higham's Tables.

Higham's Tables may be briefly summarised as follows :—
Since velocity $= C\sqrt{R.S}$.

Discharge = Sectional area × velocity = $C. A. \sqrt{R} \times \sqrt{S}$

Table I gives $C\sqrt{S}$ (Bazin's co-efficients) also the sine of the slope *viz.* $\frac{1}{l}$ per lin. ft.

Table II gives $\sqrt{R}$, and $A\sqrt{R}$ and C for side slopes $\frac{1}{2}$ to 1 (see page 11)

so that a combined use of Tables I and II enables us to find the velocity, when C, $\sqrt{R}$, and $\sqrt{S}$ are known, and can be taken from the tables.

Discharge. When C, $A.\sqrt{R}$ and $\sqrt{S}$ are known, a table of differences in Table II permits $A\sqrt{R}$ to be ascertained for any bed width, F. S. D., and slope.

If the increase in bed width is nine feet, take the figures ten feet and subtract the difference for one foot.

Table III gives Bazin's "C" for values of R (not $\sqrt{\ }$ so that it is useful for channels *other* than earthen, su as plastered flumes, cut stone work, or brick-work flumes, rubl masonry flumes, and earthen channels. Values of 'C' in exc of those given in Tables I and II may be found by looking half values and doubling the results. Table III also giv Bazin's ratios of mean velocity to maximum velocity, based o the formula

$$\frac{\text{mean velocity}}{\text{maximum velocity}} = \frac{C}{25{\cdot}3 + C}$$

Table IV is an extension of Table III, and gives more values of R and C for earthen channels alone; also the velocity ratios, as in Table III.

Table V. Kutter's co-efficients are here used for the first time in the book, and are given for values of N ranging from ·016 to ·030, and for several slopes from 1/1000 to 1/10,000. A column gives the differences to be added to C for each ·01 increment of $\sqrt{R}$.

By using Table II, $\sqrt{R}$ and $A\sqrt{R}$ are found: then taking the value of N, and C, from Table V. (S being known, and $C\sqrt{S}$ taken from Table I) the velocity and discharge can be found.

Table VI enables masonry or concrete flumes to be designed. If the slope, bed width, and depth are known, then R and $\sqrt{R}$ can be calculated, C can be taken from Table VI for the desired value of N—the required difference being added to C, if necessary, for each increment of ·01 in $\sqrt{R}$. Half values of C can be found in Table I under the proper slope, and $C\sqrt{S}$ found and doubled. The discharge and velocity are then ascertainable.

Table VII is supplementary to Table V and enables C and $\sqrt{R}$ to be found, for cases not given in Table V.

Table VIII gives the combined length of side slopes from $\frac{1}{2}$ to 1, to 3 to 1, for calculating wetted perimeters; it is a useful table, but is often lost sight of.

Table IX.—The information given in Tables I and II, etc., relates to side slopes of $\frac{1}{2}$ to 1. Multipliers are given in Table IX which will alter the results to suit other slopes.

nedy's Diagrams—

Kennedy's Diagrams need not be explained in detail, as they ery simple to use Kutter's co-efficients have been adopted the formula for the non-silting mean velocity $\cdot 84\,d^{\cdot 64}$ pulated for depths up to a hundred feet on sandy beds. nedy says scour may be prevented by widening the bed reducing the depth, or finally by reducing the bed slope ell. In ordinary soil, $3\frac{1}{2}$ feet is the maximum safe velocity, 9 or $9\frac{1}{2}$ feet the maximum safe depth for non-silting channels.

An important development of Kennedy's principle is the lue of the mean non-scouring velocity in very stiff clays (containg kankur) and in gravel and shingle. A value of $1\cdot3$ Vo has een found safe in coarse sand, and the writer has heard 5 Vo nentioned as safe for shingle up to a supply of three feet, but nore information is needed to confirm this.

Diagram No. 10 shows by means of the blue curves, the *slopes* necessary to give discharges, bed widths, and depths which result in a non-silting velocity. By means of this table non-silting channels of any desired discharge, bed width, and depth, may be designed. If the discharge only is compulsory, the designer has a wide range of choice amongst channels of various dimensions and bed slopes, all giving a non-silting velocity. The channel of maximum transporting power may also be found from the diagram.

Buckley's Pocket Book—

A Table on page 105 of Buckley's *Pocket Book* gives the slopes for Vo in channels for various depths and values of 'N,' and the table on page 107 gives the length of the two side slopes for various depths, and is useful for calculating the wetted perimeter. Dimensions for the best discharging channel are given at page 139.

Flood marks and the Discharge of a torrent—

Frequently there is no guide as to the discharge of a *nala* except a flood mark. This will, however, enable the surface slope to be observed (and compared with the bed slope). Also it will admit of cross-sections being taken from which the sectional area, wetted perimeter, and H. M. D. may be calculated.

Kutter's "N" is usually taken as $\cdot0275$ (or $\cdot03$ for a very rough channel) for sandy torrents. The velocity can then be calculated by Manning's formula.

$$V = \frac{1\cdot4858}{N} \times R^{\frac{2}{3}} \times S^{\frac{1}{2}}$$

and Discharge = sectional area × velocity.

Similarly, if the discharge has been calculated from catchment area, and there is no maximum flood mark, approximate position may be ascertained by trial as follows :-

The discharge is known (from catchment area.) Find bed slope, and assume a surface slope for H. F. M. slightly excess of this bed slope. Take three cross-sections and calcul sectional areas and W. P.

Then $\frac{\text{sectional area}}{\text{W. P.}} = R$

Mean area $= \frac{A_1 + 2A_2 + A_3}{4}$

and mean $R = \frac{R_1 + 2R_2 + R_3}{4}$

Then by Manning's formula

$$V = \frac{1 \cdot 4858}{N} \times R^{\frac{2}{3}} \times S^{\frac{1}{2}}$$

Assume N = ·0275 or ·03 according to circumstances

Discharge = A × V

If the result is very near to the discharge obtained by the catchment area calculations, then the H. F. M. assumed is correct. If not, try again.

Relation between mean and bottom Velocities.—

Merriman (*Hydraulics*) recommends Darcy's formula :

Mean velocity = bottom velocity + $11 \sqrt{R.S.}$ and states that the velocity of submerged objects in water is usually half that of the current.

A rough experiment made recently by Mr. C. W. Johnson, Executive Engineer, bears this out. The results are given below.

Weight of boulder in water.	Size of boulder.	Velocity of water by which transported.	Velocity of boulder.	REMARKS.
(1) 154·28 lbs .	1·55 cft,	11·5 f. s.	7·2 f. s.	Boulders passed over floor of undersluices at headworks, Upper Swat Canal.
(2) 238·63 lbs ...	2·38 ,,	13·3 ,,	6·7 ,,	
(3) 160·45 lbs ...	1 6 ,,	10·9 ,,	5·5 ,,	Boulders passed over natural sand and shingle bed of Dargai [illegible], Upper Swat [illegible].

ux—

Depth on crest and back water.—The usual problem o find the height to which a stream or river will be uxed by the obstruction of a weir, or by the contraction of bed due to guide bunds, or by the spans and piers of a lge, aqueduct, etc.

Love takes the co-efficient of contraction for a bridge as 5 and gives as an approximate formula for afflux caused by a *idge.*

$$\text{Approximate afflux} = \frac{(\text{original velocity})^2}{2g} \times \left\{ \frac{1{\cdot}1\ (\text{mean width of stream})^2}{(\text{waterway of bridge}^2} - 1 \right\}$$

nd if further refinement is required, the value of the afflux thus btained can be substituted in the following equation :—

$$\text{Accurate afflux} = \frac{(\text{original velocity})^2}{2g} \times \left\{ \frac{(\text{mean width of stream})^2}{{\cdot}95^2 \times (\text{waterway of bridge})^2} - \frac{(\text{mean original depth})^2}{(\text{mean original depth} + \text{approximate afflux})^2} \right\}$$

Similar calculations would answer for the afflux caused by a guide bund to be constructed in a river bed without the presence of a weir.

In the case of a submerged weir, the afflux is found by trial from the formula for a submerged weir,—thus for a known discharge of say 200,000 cusecs, and afflux assumed as three feet, we would get $200{,}000 = (\text{length of weir})\ [3{\cdot}1\ \{ (3 + \text{approach velocity head})^{\frac{3}{2}} - (\text{approach velocity head})^{\frac{3}{2}} \} + 6{\cdot}4$ depth on crest $(3 + \text{approach velocity head})^{\frac{1}{2}}]$

If the result does not approximate to 200,000 cusecs, other values for the afflux must be tried until an approximation is secured. Instead of the whole discharge, the volume discharged per lineal foot of weir may be taken, in which case one foot should be substituted for *length of weir* in the first expression of the formula.

If the weir is not submerged, the afflux may be found by trial : as before, assuming afflux equals three feet, then $200{,}000 = 3{\cdot}1 \times \text{length of weir}\ \{ (3 + \text{approach velocity head})^{\frac{3}{2}} - (\text{approach velocity head})^{\frac{3}{2}} \}$, or calculate the discharge per lineal foot of weir, and then omit the length of the weir in the formula. Values of $\frac{3}{2}$ powers are given at page 47, Buckley's *Irrigation Pocket Book.*

If it is desired to speed up water through an aqueduct fr three feet per second to ten feet per second, and the entrance the aqueduct is suitably designed to offer very little friction resistance at entry, then the afflux, or additional head requires will be $\left(\frac{10^2-3^2}{2g}\right)$ which is equivalent to saying that the head due to ten feet per second minus the head due to three feet per second is the additional head required. To this must be added the head required to give the necessary surface slope in the flume, or the flume itself must be suitably graded. If the entrance needs the application of a co-efficient "C" the expression for additional head becomes $\left\{\frac{\frac{10^2}{C^2}-3^2.}{2g}\right\}$

If it be desired to know the height to which a weir must be built to give a *desired afflux*, the normal depth of the river and the flood discharge being known, then as the afflux has been fixed, solve for the total depth of the river above the weir after construction.

Buckley considers three cases of *afflux* caused by a weir, *viz.*—

(I) When the weir is level with the original water surface in the river.

(II) When the weir projects above the original surface in the river.

(III) When the weir is lower than the original surface in the river. He gives the following formulae, in which the velocity of approach is neglected.

For (I)

$$\text{Afflux} = \sqrt[3]{\frac{\text{discharge}^2}{(\frac{2}{3}\times\text{co-efficient}\times\text{length of weir})^2\times 2g}}$$

For (II)

$$\text{Afflux} = \sqrt[3]{\frac{\text{discharge}^2}{(\frac{2}{3}\times\text{co-efficient}\times\text{length of weir})^2\times 2g}}$$

+ height of projection of weir above original water surface.

For (III)

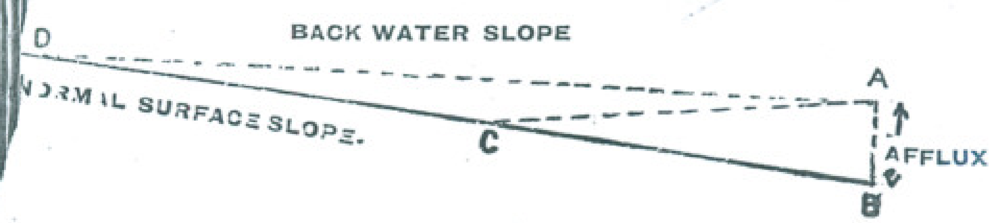
$$\text{Afflux}+\text{depth to weir crest} = \frac{\text{Afflux}+\text{depth to crest}}{3}$$

$$+\frac{\text{discharge}}{\text{co-efficient}\times\text{length of weir}\times\sqrt{2g\,(\text{afflux}+\text{depth to crest})}}$$

From which the afflux may be found by trial. In all three ...s Buckley takes a co-efficient of 0·7.

...ngth of backwater or heading back of afflux.

Length to which afflux heads back $= 1{\cdot}5 \operatorname{cosec} \alpha$, where α ... the slope of the river bed.



A graphic method is sketched above. If A B be the afflux draw A C horizontally to meet the normal surface slope line at C. Measure off C. D. = C. A. Then join D A.

Ruehlmann's formula makes the effect of afflux extend further back (Buckley's *Pocket Book, page* 66). Backwater

$$\text{distance} = \frac{\text{depth before affluxing}}{\text{original bed slope}} \times \text{f.} \times \frac{\text{afflux}}{\text{depth before affluxing}}$$

The values for f are given on page 67 of Buckley's *Pocket Book*. Merriman's formula is given on page 69.

Flow off from Catchment Areas.

Dicken's formula.

$$\text{Discharge} = \text{co-efficient} \times (\text{square miles of catchment})^{\frac{3}{4}}$$

A coefficient of 825 was taken, but in some cases the co-efficient actually obtained is more than double that figure.

Ryves' formula.

$$\text{Discharge} = \text{co-efficient} \times (\text{square miles of catchment})^{\frac{2}{3}}$$

The co-efficient taken varied from 450 to 675, but has proved inadequate for general application.

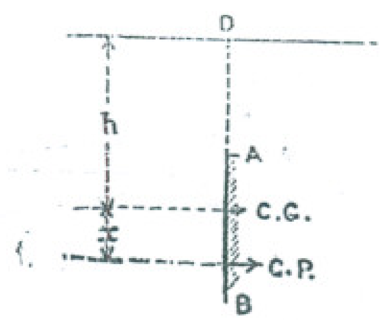
No formula is as good as direct observations made at the actual localities concerned. On the Upper Swat Canal, a flow off of 2,400 cusecs per square mile of catchment has been adopted for small catchments in the hills, and corresponds to discharges actually observed.

MISCELLANEOUS.

Centre of Pressure.

The formula below is only applicable to rectangular surface, but it is often necessary to determine the centre of pressure for ear and water pressure against dams, abutments, retaining walls, et

LOAD LINE OR WATER SURFACE



A B is the plane upon which the centre of pressure is de- ired. The centre of gravity is at the middle of the plane and e centre of pressure is always below it. The *distance between* *points* is given by the formula $x = \frac{d^2}{12h}$ where d is the he. of plane and x the required distance. The total distance of the ntre of pressure below the load line or water surface is—

$$h + \frac{d^2}{12h}$$

This is a simpler formula than the uual one

$$\text{Depth to C. P.} = \tfrac{2}{3} \times \frac{DB^3 - DA^3}{DB^2 - DA^2}$$

Centrifugal force.

Centrifugal force enters into problems connected with wheels and other parts of machinery, also into problems connected with curves, whether on roads, railways, or canals.

$$\text{Centrifugal force} = \text{Mass} \times \frac{\text{Velocity}^2}{\text{radius}}$$

$$\text{But Mass} = \frac{\text{Weight}}{g}$$

$$\text{And Momentum} = \text{Mass} \times \text{velocity}$$

$$\therefore \text{Centrifugal force} = \frac{\text{Weight}}{\quad} \times \frac{\text{velocity}^2}{\text{radius}} = \text{momentu}\ \frac{\text{elocity}}{\text{radius}}$$

In the case of rotary bodies the *angular velocity* is taken in this formula instead of the linear velocity. This equals the number of revol ons per minute $\times \frac{\pi}{30}$. Angular velocity is the an describe one second, or, in other words : the velocity of point (on the rim or elsewhere) per second, divided by the radius to that point.

Moment of Inertia.—

The mass of each particle multiplied by the square of its perpendicular distance to the axis about which the moment is desired, is its moment of inertia, and the sum of the moments of the particles equals the moment of inertia of the whole body.

The axis usually selected is the neutral axis or the axis passing through the centre of gravity. The moments of inertia for various shapes of cross-section about the neutral axis are given in many text books. *

If it be desired to use the moment of inertia with respect to an axis which is *not* the neutral axis, but is parallel to it, it will be

Usual moment of inertia $+ \frac{W}{g} \times$ square of perpendicular distance to desired axis. †

The moment of inertia is the resistance which a section offers to movement about the neutral axis. This resistance increases the further the outer edge of a beam is distant from the neutral axis, hence in the formula for beams, the stress being zero at the neutral axis and a maximum at the outer edge, the moment of resistance

$$= \frac{\text{moment of inertia} \times \text{modulus of rupture of the material}}{\text{distance of extreme fibres from neutral axis.}}$$

By equating this with the bending moment the transverse strength of beams and girders is calculated, but a subject of girders is not a very important one in canal wo it need not be further considered here.

Radius of Gyration—

Since the moment of inertia $=$ mass $\times \left(\text{perpr : distance to neutral axis,}\right)^2$

it follows that there is a value of this perpendicular distance at which the mass may be supposed collected so that the moment of inertia remains unchanged. The perpendicular distance which gives this result resembles a radius, and is called the *radius of gyration.*

*Wood's "Strength and Elasticity of Structural Members," pages 14 to 150 and 156, Molesworth, page 138, etc.

†$I_2 = I + My^2$ *Vide* Molesworth, 27th Edition, p. 370.

Hence moment of inertia = mass × (radius of gyration)^2

or moment of inertia $= \frac{\text{Weight}}{g} \times$ (radius of gyration)^2 there-

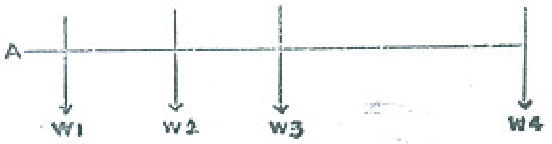
[illegible]re radius of gyration $= \sqrt{\frac{\text{moment of inertia} \times g}{\text{weight.}}}$

Mo[illegible] of Section.

The [illegible]nt of inertia divided by the distance from the neutral axis to the extreme fibres (or outer edge of cross section) is called the *modulus of section.* It is the equivalent area strained with a stress equal to that of the outer fibres.

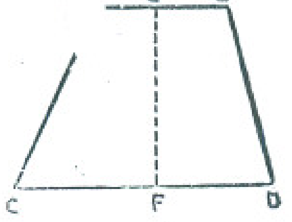
Centre of Gravity—

Where there are several forces in action their *centre of gravity* may be found by taking moments about a point and equating thus



Let x = distance of C. G. from A. Taking moments about A, $x \times (W_1 + W_2 + W_3 + W_4)$ = mt. of W_1 + mt. of W_2 + mt. of W_3 + mt. of W_4.

The C. G. of a trapezoidal abutment may be found as follows :—



Bisect A. B. and C. D. Joint the bisections E. and F. Divide E. F. in the proportions 2 A B + C D : 2 C D + A B, and the division will give the C. G.

Useful Memoranda,

For 75° arches, radius = ·821 × span

rise = ·170 × span

For 90° arches, radius = ·707 × span

rise = ·207 × span

Weight of a cubic foot of water $= \frac{1}{36}$th of a ton

A miner's inch = ·02 to ·025 cusec, according to locality

A foot pound per second = ·001818 H. P.

One foot in depth of water over an area-43,560 c. ft. er acre.

Head in feet = 2·307 × pressure in lbs. per sq. inch
or ·016 × pressure in lbs. per sq. ft.

Pressure of still water in lbs. per square inch = ·4335 × ead in feet.

Water pressure in lbs. on a vertical plane per sq. ft. = 1·2125 × height^2

One ton per sq. ft. = 15·55 lbs. per sq. inch

One inch of rain on land = 3,630 c. ft. per acre
or 2,323,200 c. ft. per sq. mile

Head = ·01555 V^2

Water heavily charged with silt may weigh as much as 75 lbs. per c. ft. (page 16, Buckley's Pocket Book).

This Paper is mainly a compilation, the object being to condense the information given by well-known authors or experimentists, and to present it in a concentrated form. Occasionally this information has been supplemented by local practice or experiment. It has not been possible to deal exhaustively with each subject, nor even with *all* the mathematical problems which occur in canal practice. Many have had to be deferred for future consideration.

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DISCUSSION.

MR. WADLEY, in introducing his paper, said that it had een his intention to bring together the methods of various mathe-aticians and to discuss and contrast them, but, unfortunately, ant of time had prevented this, except in the case of the first two ems, namely soils and the allowable pressures on foundations, nd the strength of materials. This would explain why no eference had been made in the paper to certain well-known methods, such, for instance, as the elastic arch, or Scheffler's theory regarding the voussoir arch. A very good method of finding the equilibrium curve was given at page 192 of Molesworth (27th edition). For the same reason graphic solutions, by the theory of wedges, etc., had not been referred to, while piles, struts, columns, girders and steelwork generally had not been treated at all.

In that part of the paper which dealt with hydraulics, many items had purposely been omitted, such as the depth of films of water on rapids and glacis, the formation of standing waves, the value of Kutter's 'N' in rock tunnels, the use of baffle blocks and stepped aprons, the impact of water on obstructions, the use of siphon sluices, re-inforced concrete tubes, and so forth. These subjects could more suitably be considered later, and, notwithstanding its incompleteness, he ventured to think the paper would be useful. He had heard men say that, when they joined the department, they knew little regarding the practical design of hydraulic works, and no one had the time to enlighten them.

The percolation gradient was an important factor in the design of canal works. Bligh had studied the designs of a great many works, and set himself to discover the causes of failure of two of these, *viz.*, the Narora weir and the Khanki weir. The conclusion arrived at was, that the percolation gradient had not been fully taken into account. Starting with this as a basis, and applying the idea to the designs of other works, he had evolved the principle, that the length of the percolation gradient, necessary to the stability of such works, was mainly dependant upon the class of sand found in the river bed. The idea of a percolation gradient was very old. D'Arcy who was water works engineer of Paris in 1857, introduced a formula for it, *viz* :

$$\text{sub-soil velocity} = \text{co-efficient} \times \frac{\text{head of percolation}}{\text{length of soil column}}.$$

The co-efficient depending on the character and porosity of t soil. In California, for instance, the co-efficient was found vary from ·0001 to ·0015*.

If irrigation engineers were not prepared to acce Bligh's percolation gradients, they should begin to observe su soil velocities and gradients themselves, and such velocities cou probably be observed by lowering a Pitot tube into the sub soil. Fluctuations of spring level, drainage conditions, and so evaporation were all stated to influence the percolation gradient.

MR. SCHÖNEMANN said that Mr Wadley had presente them with a vast array of formulæ, chiefly culled from works c reference, but he could only find time to comment on one or tw of these. As regard flood marks in a hill torrent, and the indication of the magnitude of the flood discharge, Mr. Wadle had said that Kutter's 'N' was usually assumed to be 0·027 (or 0·03 for a very rough channel), but he had not stated where or by whom this practice was regarded as usual ; nor had he give any reasons in support of the practice. This was disappointin It was not enough to say that a certain practice was usua What engineers would want to know was whether it was sound If Mr. Wadley had said that it was customary to regard the moo as being square in shape, his hearers would want to know why As a matter of fact, the value of Kutter's co-efficient had bee found, by a very precise hydraulic survey, to average 0·027 o the main branch of the Upper Bari Doab, and 0·031 on th main line of the Western Jumna Canal. These were canals i equable flow, with mean velocities not exceeding four feet pe second, and certainly were not comparable, in point of rugosity with hill torrents. On hill torrents the co-efficient might well b as high as 0·05 or 0·06—increasing with increase of velocity That being so, the method described at page 162, of calcula ting the maximum flood level from calculated discharge, wa unreliable.

Mr. Wadley had also said that the run-off his catchment amounted to 2,400 cusecs per square mile, but, if such intensitie of flow-off were to be expected, they would have been recorde as having occurred in other countries, whereas there was no re liable record any where in the world of a run off exceedin 1,000 cusecs per square mile. If 2,400 cusecs per squar mile had been discharged from Indian catchments, the date o observation and calculation ought to be published in detail, fo the benefit of the profession throughout the world.

* Engineering News, October 30th, 1913.

Bligh had been quoted on the question of the hydraulic lients of percolation flow through soil. He was under the ression, however, that Beresford, Clibborn and others had n the chief demonstrators in this subject, while Bligh was r a shrewd compiler, who had collected the results of obser- on, experiment, and inference by other original investigators. is writer had stated that the hydraulic gradient of flow through sand, "like that of the Himalayan rivers" was 1 in 15, as though the sand of the Ganges did not vary, in point of rseness or fineness, from the shingle and boulder strewn bed Hardwar, to the quicksands of the Hooghly at Calcutta!

MR. NICHOLSON suggested the paper being put in its irety before the research section advocated by Mr. Colyer, as re was sufficient matter to keep the section employed for some rs. The author had stated that he had made experiments with bles, but the nature of these experiments was not clear. Peb- s of different colours had been filed down to accurate spheres small diameter and then weighed, but, by ascertaining the specific vity of the pebbles, the weights of the spheres could have been culated much more easily, and the tedious filing avoided. It did appear that these actual spheres had been used in running ter to verify the calculations made by the author regarding the nsporting the velocity. Such results would have been of great erest. He objected to Love's Hydraulics (price Rs. 1-8) ng quoted as an authority. Too much reliance was placed on ything printed being applicable to all conditions, whereas it s very necessary, when using a formula or co-efficient, to ow the conditions under which it had been experimentally termined, in order to judge whether the circumstances in which was being applied justified its use.

Mr. DUTHY pointed out that at present the method ed for determining the discharge of a nullah in flood was tremely inaccurate. It had been shown on the Upper Jhelum anal that there was a very heavy scour in a nullah bed ring a flood. This scour could be determined by sinking s of surki or charcoal in a nullah bed, and, after the flood d subsided, seeing how much of them had been washed away, t what could not be determined was, the level of the flood the time of maximum scour, and what the scour had been at e time of maximum level. He did not believe that the aximum flood level occurred at the same time as the maximum our, and consequently, if scour were neglected, the result obtained would be much too low, whereas. if it was taken into consideration, and it was assumed that maximum flood level and maximum scour occurred simultaneously, the result obtained would be much too high.

RAI BAHADUR GANGA RAM said the paper was interesting, containing, as it did, many formulae in a condensed form. With reference to pressure on soils Mr. Wadley had not mentioned how to deal with soil consisting of made earth when such had to be used for a foundation. He had had to deal with such soil in Lahore for the past twenty years, and a typical instance was the case of the foundations of the Roman Catholic Church on Empress Road, where he sunk a pit, twenty feet deep, and yet came across brickbats.

Another point he would like to touch upon was the thickness of abutments. Many years ago a friend had shown him a graphic method of finding out this, which was as follows—taking for example a bridge of one hundred feet span, and twenty-five feet high, all one needed to do was to continue the line of extrados, and this line continued would give the correct thickness of the abutment at the base.

Another point was the question of retaining walls. About the year 1875 the Lahore water-works reservoir failed, and a committee was appointed to investigate the cause of failure, but no specific reason could be assigned. Soon afterwards, being fresh from college, he had been asked to try and discover by mathematical calculations the reason for the deplorable occurrence and he found that the reason of failure was due to the outer toe of the retaining wall not having been carried sufficiently far out to bring the maximum intensity of pressure on the soil within safe limits. Precautions had been taken for the safety and stability of the wall, and this was satisfactory, but the outer toe had not been properly calculated for the safe intensity of pressure on foundations which consisted of made earth.

MR. WADLEY, in reply, said that Mr. Schönemann had objected to values of ·0275 to ·03 having been taken for Kutter's "N." They had, however, been deduced from observed discharges, which was the only safe method of using this co-efficient. This speaker had further mentioned that values of ·031 and ·027 had been obtained on the Western Jumna Canal and the Upper Bari Doab Canal. This was not surprising. For the first eleven miles of its course, the former canal resembled a mountain torrent, with velocities at high canal varying from ı feet per second at the head to four feet per second near ıdapur, while the bed varied from boulders to gravel. For ɜ next twenty-one miles the velocity varied from reach to ıch; the silt carried was of a heavy grade, and the regime ıs unstable; while at some points there used to be islands the canal. The Upper Bari Doab Canal was also known have had such erosive velocities in the top reaches that the ıannel had to be pitched. Regarding both canals, in the aches referred to above as resembling torrents, the values of N' given by Mr. Schönemann confirmed his figures for the tor-ɛnts of the Upper Swat Canal.

The following were some values of "N" derived from xperiments made with a current meter in the Braden Canal *

(*i*) 801·7 cusecs passing over masonry side walls of *rounded* stones having three inch projections beyond joints, bed smooth ... ... N = ·023

(*ii*) 452·4 cusecs passing over loose rock, points projecting three to six inches ... N = ·025

(*iii*) 230·7 cusecs passing over seamy rock section, with points projecting three to twelve inches, and with holes in the sides twelve to fifteen inches deep ... ... N = ·029

Mr. F. H. Burkitt, executive engineer, had just sent him the results of some observations made on a rough stone rapid on the Lower Swat Canal where the value of Kutter's N was found to be ... ... ·030

Molitor (1908) the designing engineer of the Panama Canal commission, quoted the following values for Kutter's N.

For canals in very fine gravel ... ... ·020

For canals and rivers free from stones and weeds ... ... ·025

For canals and rivers with some stones and weeds ... ... ·030

For canals and rivers in bad order ... ... ·035

Merriman (1912) gave the same values in the same words, except that he has used "firm" instead of "fine" in defining the first value ·020.

Few modern authorities appeared to give higher values than ·035. Love quoted a value ·040, and personally he was

* Engineering News, dated 22nd May 1913.

convinced from the evidence available that ·0275 and ·03 were reliable values of N for the torrents crossed by the Upper Swat Canal.

Mr. Schönemann also said that he thought the run-off of 2,400 cusecs per square mile allowed was excessive. On page 165 it was stated that a flow-off of 2,400 cusecs per square mile of catchment had been adopted for small catchments *in the hills* and corresponded to discharges actually observed. This was correct, but, to avoid giving the impression that a run-off of 2,400 cusecs was indiscriminately used, the full table for run-offs was given below :—

	Cusecs per sq. mile.
For catchments in the hills 0 to $2\frac{1}{2}$ square miles in area	2,400
For catchments in the hills $2\frac{1}{2}$ to 5 sq. miles in area	2,000
For catchments in the hills 5 to $7\frac{1}{2}$ sq. miles in area	1,750
For catchments in the hills $7\frac{1}{2}$ to 10 sq. miles in area	1,500
For catchments in the hills 10 to 15 sq. miles in area	1,250
All above fifteen square miles up to fifty square miles	1,000
Above fifty square miles	800
For catchments *not* in the hills, but situated within a zone of 5 to 10 miles from them	2/3rds of above values.
For catchments *not* in the hills, but situated within a zone of 10 to 15 miles from them	[illegible] above values.

The table had been arrived at by actual discharges, made either on the Upper Swat or the Lower Swat Canal, and a margin had been added for safety.

Mr. Schönemann's had also referred to the percolation gradients adopted by Bligh. The credit of discovering the percolation gradient appeared to belong to D'Arcy, who first formulated the law referred to in the author's introductory remarks, though the investigations made by Mr. Beresford and Col. Clibborn threw more light on the subject. Like Kutter's N, however, it was not absolutely safe to use a percolation gradient without having first determined its value for the class of material in the river concerned.

Mr. Nicholson had suggested that the paper should be put before the scientific research section, and, while welcoming the suggestion, he feared the matter in the paper would be quite out of date by the time the research section came into existence, unless the need for it was more keenly felt by the Department generally. This speaker had criticised the pebbles having been filed down into spheres, but it was necessary to do this, because Merriman's deductions, with regard to transporting power, were based on the diameter of the body transported. The results of the experiment were given on pages 136 and 137, and showed that shingle, 1·1 inch in diameter, had a critical velocity ratio of 6·1 V_o, and that sand passed by mesh 8 and rejected by mesh 12, had a C. V. R. of 6·1 V_o, while sand passed by mesh 12, but rejected by mesh 20 had a C. V. R. of 1·3 V_o. With regard to Love's hydraulics, the 1914 edition (11th edition) was up to date, and had been compiled from the works of the best hydraulicians.

MR. DUTHY considered that the present methods of determining the discharge of a nala in flood were extremely inaccurate, as, owing to scour effects, the level of maximum flood constantly fluctuated. He did not think maximum scour synchronised with maximum flood level, but, if scour effects were not taken into account, the figures for discharges would be too low ; while, if it were assumed that maximum scour synchronised with maximum flood, then the discharge figures would be too high. The measurements of discharges on the Upper Swat Canal were based, as usual in such cases, upon measurements of cross-sectional areas and of the mean velocities through them.

In desiging irrigation works, full consideration had to be given to the fact that years of drought were succeeded by wet years, and that the works must be able to cope with the run-off of the wettest year. It was the extreme flood which had to be legislated for, and, although the transporting power of water decreased as the depth increased, common experience and common knowledge showed that it was the extreme flood which swept away shoals and obstructions, and that therefore the cross-sections taken after such a flood were less liable to error than those taken after smaller floods. The discharge sites on the Upper Swat Canal were selected to give as uniform a flow as possible, and the flood marks were pegged out, and checked by the *débris* arrested by shrubs.

Bellasis on pages 36—37 of his Hydraulics had pointed out that the "drift" or volume of silt in motion along the bed was unaffected by the depth, and, though a stream already fully charged with silt, would not pick up any more, the quantity of drift rolled would probably be unaflected. In the absence of precise information as to the depth of drift in motion during maximum floods, it appeared to be unwise to attempt to include it in the discharge. He believed that the surkhi and dust-coal bed-pits, which had been tried on the Upper Jhelum Canal, afforded no reliable information as to the depth of drift in motion, and, when it was recollected that the velocities secured were not *extremely accurate mean surface velocities*, the exclusion of the drift from the volume of discharge was of little consequence, and obviously tended to avoid excessive computation of the volume. In a tract where the fluctuations of rainfall were great, too nice a refinement in this matter was out of place. What was needed was an absolutely safe figure, which would stand the strain of torrential rain. At one of the rain gauge stations on the Upper Swat Canal, eleven inches of rain had been recorded in five hours by Mr. Oram, assistant engineer, during 1914.

He felt it was more important to study *these* fluctuations, and to make allowance for them, than to aim at intense accuracy in the method of discharge observations. If there were great fluctuation in the rainfall, it would be unsafe not to add, to all results obtained, a suitable margin for security. Actual discharges, observed subsequent to 1909, had shewn that the run-offs adopted in that year for catchments on the Upper Swat Canal were pitched none too high.

In reply to Rai Bahadur Ganga Ram, the usual method of founding on weak soils (assuming that the site could not be avoided) was to reduce the pressure intensity by means of spreading the foundations to that which the soil would safely bear; the bearing power having first been determined by actual experiment. The depth of such foundations would have to be calculated by Rankine's formula given on page 115 of the paper. Re-inforced work was sometimes used as a monolithic raft under a structure in bad cases, or deep wells were sunk which would support the work by skin friction. In order to secure a firm working area in sloppy soil, a layer of shingle would be found useful, as affording a base upon which to lay the foundations, assuming that the moisture could not be drained off. Rankine gave a formula for calculating the least necessary thickness of a firm layer over weak soil which would permit it to be entrusted with the weight of a given structure.

MR. SCHÖNEMANN interposed a correction on a point of fact. The value of 0·031, which had been obtained for Kutter's N on the Western Jumna Canal, had been obtained in the thirty mile reach between Dadupur and Indri, where the soil was a soft sandy alluvium, and where the mean velocity did not exceed three feet per second. Mr. Wadley was mistaken in supposing that the soil was of boulders and gravel, or that the mean velocity ranged from four to ten feet per second.

In the case of the Upper Bari Doab Canal the portion referred to as having an average value of 0·027 for Kutter's N was the twenty-five mile reach between Tibri and Aliwal, where the bed was of coarse sand in perfectly good regime, though the sides of the channel were eroded.

It was absurd to compare these portions of these canals, (with their mean velocities of three or four feet per second, and with gradients flatter than 1 in 4000,) in point of irregularity and value of Kutter's N, with the mountain torrents dealt with by Mr. Wadley, whose mean velocities were alleged by him to be upwards of ten feet per second, and whose declivities ranged from about 1 in 500 to about 1 in 50.

Mr. Wadley in his rejoinder had quoted fresh figures for Kutter's N on certain canals, which ranged from 0·020 to 0·035; but these referred to cases in no way comparable to those of the mountain torrents which he had credited with run-offs ranging up to 2,400 cusecs per square mile of catchment area. In Kutter's original treatise many cases had been cited of channels whose value for N ranged from 0·040 to 0·060; and in the article on Hydraulics in the Encyclopædia Brittanica 0·050 was given as the value for "torrential streams encumbered with detritus." Mr. Wadley's hill torrents came under this description, and he had given no good reason why he had rejected this higher value for Kutter's N, and selected in preference values so low as 0·0275 and 0·030.

Mr. Wadley had said that his values of 0·0275 and 0·030 had been "deduced from observed discharges, which was the only safe method of using this co-efficient;" but Mr. Schönemann's point was that the co-efficient had not been deduced from the discharges, but the discharges from the co-efficient! Certain flood marks had been observed, and certain channel sections measured up to the flood marks; and then discharges had been inferred from these data through the medium of unwarrantable assumptions as to the value of Kutter's N.

Before the engineering profession could be asked to believe in these extraordinary allegations of run-off per square mile of catchment, the data of the alleged observations should be displayed and submitted to criticism.

Mr. WADLEY in reply to Mr. Schönemann's further objection with regard to the value of Kutter's N, which had been suggested as suitable for sandy torrents, felt the subject was so complex that it would be unsafe to take a very high value of N for general use, even if in one or two isolated instances a high value had been obtained by actual observation. Actual observation of discharges was the only safe course. A perusal of Parker's "Control of Water," pages 471 to 478, was invited.

Where low velocities were concerned, the value of N was more influenced by weeds, slime, and jungle, than by the nature of the surface forming the channel. Thus surfaces of brick, stone, cement, and iron were apt to give the same value for N if there was a growth of slime. In channels in earth, and in all channels where the velocity was such as permitted silt to line the bed and slopes, the silt had the effect of reducing Kutter's N, provided the velocity had not been sufficient to produce silt waves in the bed.

Where high velocities were concerned, the value of N was influenced in the long run by the susceptibility of the containing surface to erosion. If the surface did not remain jagged, but was worn down to smoothness, the value of N would not be nearly so high as if a hard, rugged surface persisted, defying erosion. In the Central Provinces some of the rocks wore down to a smoothness resembling polishing.

If the velocity was sufficient to cause waves of silt along the bed, the smoothening effect of silt was lost, and it became an obstruction, which instead of reducing the value of N, tended to increase it; while these silt waves, if large, also caused waves in the surface of the water, thus testifying to their obstructing effect. Parker* mentions 0·020 as the highest value likely to be obtained in a regular channel having its bed and sides completely covered with a smooth lining of silt, and states that N may rise to 0·027 if the deposit of silt is copious enough to form into silt waves.

*Control of Water, page 478.

Mr. Schönemann had obtained ·031 as the value of N. on the main line of the Western Jumna Canal, and had now made it clear that he was referring to the reach from Dadupur to Indri. A reference to his own remarks, regarding velocities of four to ten feet a second with a bed varying from boulders to gravel, would show that he was writing of the first eleven miles of the canal, *viz.*, from Tajawala to Jaidhri. He could not agree with Mr. Schönemann that the reach of the canal upon which he experimented was in equable flow. Its sectional area varied considerably, and it received several streams of natural drainage during the rains. The silt brought down by these, and the volumes of silt passed into the canal from the Somb torrent at Dadupur, or brought down from the Jumna, must produce results which were quite out of keeping with the usual conditions of equable flow, and must bring things more into line with the conditions prevailing in sandy torrents. Silt waves or terraces two feet deep could be seen in places in the canal bed at certain times of the year during closures and, as stated by Parker, these would increase the value of N. The extremely *local* character of the value of N obtained from observations in a channel of variable flow had also to be taken into account.

Parker gives the following values of Kutter's N :—

	Value of 'N'.	Authority.
Channels in order, below the average ...	·0275	Jackson.
Channels in bad order ...	·03	Kutter.
Channels in very bad order ...	·035	Kutter.
Channels of worst possible character with turbulent flow and large obstructions ...	·04	Jackson.

Gibson at page 294 of "Hydraulics and its Applications" remarks "when badly choked with weeds the value of N in Kutter's formula might become much greater than ·035." Consequently it was evident that slime, weeds, grasses, silt waves and rugged surfaces produced more effect in increasing

N than moderate quantities of sand or silt alone.

The torrents on the Upper Swat demonstrated that the were able to cause many of the culverts, which had been designed for them, to run full bore, and that was the best test of th correctness of the fixed values of run-off taken.

In the Central Provinces, the run-off used for catchment up to one square mile, varied from $1\frac{1}{2}$ to 3 cusecs per acre according to the slope of the catchment, and the extent to which it was covered with jungle. These values gave a run-off varying from 960 to 1,920 cusecs per square mile. For larger catchments, the run-off was taken as $1,400\ M^{\frac{3}{4}}$. Thus for a catchment of eight square miles, the run-off would be 6,66 cusecs, while on the Upper Swat Canal it would have bee $\frac{2}{3} \times 8 \times 1,500 = 8,000$ cusecs*. There was thus no great disparity between the results.

In fixing values for the run-off from catchments, it was necessary to allow a good margin for safety, if the values were to be applied to a series of works (as was generally the case), and to take into account the records of torrential bursts of rain. It was mentioned that eleven inches of rain had fallen at Hamzakote in five hours. During 1915, a burst of fourteen inches of rain in twenty-four hours was registered at Jubbulpore in the Central Provinces, and the records show even heavier showers at other towns during the last forty years. In the face of downpours of this kind it would be rash not to allow a good margin for safety. In 1910 a rainfall of fifteen inches was recorded in twenty-four hours at the Kurud tank in the Central Provinces (the maximum ever recorded being 16·62 inches). There was an automatic gauge on the waste weir, and from the diagram of the gauge a maximum discharge of 5,520 cusecs was obtained.† The catchment area was 5·7 square miles, the slopes of the catchment were moderate, and bare. A steeper catchment would have given a higher flood. He had seen the catchment, and it corresponded in flatness, and distance from the hills, to those for which half values were taken on the Upper Swat Canal. A maximum discharge of 4,038 cusecs would have been allowed for on the Upper Swat Canal. The discharge, by Dickens' formula, with a co-efficient of 1,400 as used in the Central Provinces, would have been 5,161 cusecs, which was very close to the Upper Swat Canal result. Although

* *Vide* page 170.

† *Vide* Appendix II, pages 32 and 33 of "A General Theory of the Storage Capacity and Flood Regulation of Reservoirs" by Captain Garrett, R. E.

the total annual rainfall in the Central Provinces was greater than that in the North-West Frontier, the intensity of showers was not in excess of those experienced on the North-West Frontier, and it is upon the maximum rainfall in twenty-four hours, or upon the intensity of maximum showers (whichever was the greater) that designs for masonry works are based. The total annual rainfall did not help.

As stated in his paper, the high values of 2,400 and 2,000, &c. cusecs per square mile were merely applied to very steep and small catchments in the hills. The discharges observed, after torrential rain in the "Cusecs Nala" and the Butkhela Nala first gave warning that allowance must be made for phenominal run-offs where such catchments were concerned. The results were subsequently confirmed by the discharges observed in other nalas, and a margin of about twenty-five per cent. was wisely added by the Chief Engineer for safety. The hills are very steep and very bare. The corresponding value in use in the Central Provinces where the hills are wooded is 1,920 cusecs per square mile. For catchments not in the hills. two-thirds and half the higher values were taken, according to the distance from the hills at which the catchments were situated.

It was recognised that in addition to the intensity and distribution of showers, the width and length of a catchment, its slope, the extent to which it was covered with foliage, the nature of the materials composing its surface, and the extent to which these were saturated at the time, were factors affecting the run-off. For accurate designing, a series of co-efficients for each of these factors would need to be scientifically determined, and applied with proper discrimination; but as there is no research section in the Public Works Department, a table of observed run-off values. which would approximately embrace all conditions and be reasonably safe, had to be found and applied to all cases. Some of these it fitted closely, in others there was a margin. The conditions, which affect the run-off, varied so greatly, however, that there was no other course.

On page 202 of his "Irrigation Pocket Book" Buckley quotes a flow off of 1,290 cusecs per square mile from a catchment of 4·4 square miles on the Koregaon tank. This is in excess of that arrived at by the formula 1,400 $M^{\frac{3}{4}}$ which gives 4,249 cusecs, or 966 cusecs per square mile. Amongst the examples given by Buckley, this was the smallest catchment mentioned, and the result points in the same direction as those obtained the Upper Swat Canal, *viz.*, that as the catchment decreas the flow-off per square mile increased and needed spec treatment.

As a matter of fact Kutter's N was very little used the Upper Swat Canal. The values for the run-offs fro catchments were fixed by actual observation plus a margin f safety. But such values of N as ·0275 or ·03 approximate closely, in most cases, to the discharges fixed, where the beds nalas were not rocky, but composed of sand.

The following interesting information on the subject Kutter's N and the run-off from hilly catchments is abstracte from Engineering News, dated 10th February 1916, page 272 to 275 :—

	Nature of channel.	Value of 'N' obtained.
San Gabriel River.	Banks composed of granite boulders 4" to 24" diameter. Bed of sand and gravel with rocks up to 3" diameter, small eddies in surface of water.	·028, approximat value ·03.
	West bank of boulders 2" to 12" diameter, average diameter 4". East bank of boulders 3" to 36" diameter, average 12". Bed of boulders 9" to 24" diameter. Waves in surface of water.	·0354, approximat value ·035.
	Both banks of granite boulders 4" to 24" diameter, numerous large rocks in water. Bed of boulders 6" to 24" diameter. Surface of water full of eddies and waves.	·0416, approximat ·04.

Nature of channel.	Width in feet.	W. P.	Surface slope.	C.	Discharge in cusecs.	Cross sectional area in square feet.	H. M. [illegible]	Kutter's 'N'.	Velocity in feet per second.	Catchment area in square miles.	Run-off in cusecs per square mile.	Location of catchment.
Los Angeles River. Sandy with piles of debris in stream.	267	275	·0035	59	24,420	2,380	8·65	·0375	10·27	334·8	73·2	242 sq. miles in mountains, 92·8 miles in valley.
Arroyo Seco. River. Bed rough, with coarse sand and boulders.	[illegible]3	169	·009	45	7,610	810	4·84	·045	9·4	42·7	178	30 sq. miles in mountains, 12·7 miles in valley.
Big Tejunga River. Channel rough, strewn with boulders; one bank rock.	240	246·5	·0124	52·5	13,600	1,068	4·74	·0375	12·73	118	115	All in mountains.
Santa Anita Wash.— Banks low and lined with brush. Bed rough with large boulders.	78·5	79·5	·0142	42·0	3,168	312·2	4·002	·045	10·02	18·17	174	All in mountains.
San Gabriel River. Bed rough, with gravel and boulders, 12″ to 18″ diameter. Banks of light soil.	383	386	·0079	58	*26,680	2,178·2	5·64	·035	12 25	228·68	117	All in mountains.

*47,000 in 1884 from 220 square miles.

The average rainfall at Los Angeles for thirty-eight [y]ears is 15·81". Most of it falls in December to March, *viz.*, 3·16 [in] January, and 3·17" in February; but the rainfall goes up to 38·[...] and 34·84". In 1914, a rainfall of 23" produced great floods, w[ith] 10·35" of rain in January and 7·04" in February. The [Los] Angeles rain gauge, however, gives no indication of rainfall in [the] hills. The smallest streams showed run-offs of 300 to over 7[00] cusecs per square mile. If twenty-five per cent. be added [to] the run-offs for safety, we get 375 to over 875 cusecs per squ[are] mile, and a reference to the catchment areas shown in the statem[ent] indicates that the engineers were not dealing with such sm[all] catchments as 0 to $2\frac{1}{2}$ square miles or even 0 to 5 square mil[es]. The smallest catchment mentioned is one of 18·17 square mil[es] and none of the figures for run-off, etc., relate to the maxim[um] floods, when the engineers were not able to get near the rivers.

Applying a run-off value of 800 cusecs per square mile [to] a catchment of ten square miles we get 8,000 cusecs; applyi[ng] Dickens' formula $1{,}400 \times 10^{\frac{3}{4}}$ we get 7,874 cusecs; applyi[ng] the $\frac{2}{3}$rds values for a catchment of 10 to 15 square miles, as giv[en] in the Upper Swat Canal table, we get 8,340 cusecs.

The engineers at work on the San Gabriel and Los Ange[les] rivers thus appear to be arriving at approximately the same valu[es] for Kutter's N, and for the run-offs from steep catchments, as th[ose] obtained in the Punjab and Central Provinces under simil[ar] conditions. They had not yet given run-off values for very sm[all] catchments however, nor Kutter's 'N' for sandy torrents.

Erratum.

Page 165, lines 12 and 13 from bottom. *Read* :—

Dickens' formula.

Discharge = co-efficient × (square miles of catchment).$^{\frac{3}{4}}$