# ERRATA TO PAPER No. 249

*Page* 251, line 2, *instead of* $V=1\cdot17\ \sqrt{fR}$ *read* $V=1\cdot17\ \sqrt{fR}$.

*Page* 253, line 13, *instead of* $31974\ R^{10/3}$ *read* $31974\ R^{10/3}.S$.

*Page* 253, line 14, *instead of* $S=31974\ R^{10/3}$ *read* $S=\frac{Q}{31974\ R^{10/3}}$.

*Page* 256, line 8, *instead of* $R=72\cdot2$ *read* $B=72\cdot2$.

*Page* 256, case No. 1, *after* $S=1/5714$ *add* $\sqrt{f}=0\cdot9704$.

*Page* 258, in statement I, in heading of col. 10 *instead of* $\sqrt{1}$ *read* $\sqrt{f}$.

*Page* 260, in statement in the bottom col. 6, *instead of* $C_3=\frac{1}{Q^{\frac{1}{3}}}$

*read* $C_3=\frac{R}{Q^{\frac{1}{3}}}$.

*Page* 261, line 17, *instead of* $V=1\cdot4\ \sqrt{R}$ *read* $V=1\cdot14\ \sqrt{R}$.

*Page* 263, col. 4, under $P=2\cdot8\ Q^{\frac{1}{2}}$ *instead of* $PR=0\cdot47\ Q^{\frac{1}{3}}$,

$V=1\cdot1\sqrt{R}$ *read* $R=0\cdot47\ Q^{\frac{1}{3}}$, $V=1.12\sqrt{R}$ $Q=PRV$ *read* $PRV=Q$.

*Page* 263, col. 5, under $P=2\cdot67\ Q^{\frac{1}{2}}$.

*instead of* $P.R=\cdot48\ Q^{\frac{1}{3}}$ *read* $R=\cdot48\ Q^{\frac{1}{3}}$.

*Page* 263, col. 6, *instead of* $P.\ R=0\cdot50\ Q^{\frac{1}{3}}$, $V=1\cdot14\ R^{\frac{1}{2}}$ under *P*$=2\cdot5Q$ which should be inserted.

*read* $R=\cdot50\ Q^{\frac{1}{3}}$ $V=1\cdot14\ R^{\frac{1}{2}}$.

*Page* 264, line below the formula of C, instead of NV *read* N.

*Page* 269, line 1st and 2nd, *instead of* $j\sqrt{fR}$ *read* $\sqrt{fR}$.

*Page* 274, paragraph 3, line 2nd, *instead of* $\sqrt{\quad}=$ *read* $\sqrt{f}=$.

*Page* 286, line VI, *instead of* $S\ 10^3=\frac{Q^{3/2}}{80\ R^5}=\frac{\frac{(R)^{9/2}}{(\cdot47)}}{80R^5}=\frac{\frac{R^{9/3}}{(\cdot47)^{9/2}}}{80R^5}$

$=\frac{1}{2\cdot67\ R^{\frac{1}{2}}}$ *read* $S\ 10^3=\frac{Q^{3/2}}{80R^5}=\frac{\left(\frac{R}{\cdot47}\right)^{9/2}}{80R^5}=\frac{\frac{R^{9/2}}{(\cdot47)^{9/2}}}{80R^5}=\frac{1}{2\cdot67\ R^{\frac{1}{2}}}$.

*Page* 288 line 3rd, *instead of* $N=0\cdot0225\ S10^3=\frac{\cdot0333Q}{R^{10/3}}=\frac{Q}{R^{10/3}}$

$=\frac{1}{2\cdot42Q^{1/9}}$ *read* $\frac{\cdot0333Q}{R^{10/3}}=\frac{Q}{30R^{10/3}}=\frac{1}{2\cdot42\ Q^{1/9}}$

## HYDRAULICS OF IRRIGATION CHANNELS

By Ishwar Das, B.Sc. (Bris.), P.S.E.

## PAPER NO. 249

## INTRODUCTION

*Necessity of the Paper.*—Of late different ideas regarding the theory of the flow of silt-laden water in canals have been advanced.

For purposes of design, some engineers are still sticking to Kennedy's Hydraulic Diagrams, for instance, Examinees in the last Departmental Professional Examination were asked to bring in their own Kennedy's Diagrams for answering questions set in the examination paper, whereas others have entirely discarded Kennedy's in favour of Lacey's Diagrams. Yet there are engineers who will neither accept the one nor the other.

Mr. R. K. Khanna has advanced his own theory while the Irrigation Research Institute has evolved new formulæ.

It is in face of these diverse opinions that it has been considered desirable to discuss in some detail the hydraulics of the irrigation channels with particular reference to Kennedy's and Lacey's Theories.

The conclusion of the writer is that it is not safe to take Kennedy's "C. V. R." or Lacey's "f" as a basis of design.

Kennedy's Diagrams have been improved by superimposing thereon the curves for Lacey's Perimeter Formula, so that without reference to any values of "C. V. R." or "f" the dimensional design for any discharge for a given slope could be picked up at a glance.

For determining slopes new formulæ have been evolved and tables made for picking up values of the same for any given discharge.

Also a Discharge Slide Rule has been made for solving problems in design quickly.

It is hoped that Improved Kennedy's Hydraulic Diagrams and the Discharge Slide Rule will solve the difficulties met with in designing channels at present.

## CHAPTER I

## SILT AND ITS TRANSPORTATION

The sources of our canals are rivers. Rivers are formed from water of many streams and hill torrents pouring their contents therein. Over high hills when it rains or when the snow melts, water comes down with a tremendous velocity from all directions and thus hill torrents are formed with very high velocities, taking down with them large and small stones, boulders and pebbles. When these torrents meet erodable soil, water gets muddy with the contents of the eroded soil. In the onward course of a stream or river, some of this stuff—stones, boulders, pebbles and sand and earth—is left and other picked up.

Generally we see that when the water reaches the plains, the heavy boulders and stones are left behind, but sand and silt or at the most small pebbles only are there. All these are found in the water which we take off into our canals.

If we examine the contents of water and silt deposits at about the head of a canal and at other different sites on its way to its tail we find that the contents are different. It is the behaviour of these contents, which we call silt, that we are very much concerned with.

In the case of hill torrents we noticed that the contents were big and small boulders with rounded corners, tiny pebbles and coarse and fine sand. As the velocity of the stream lessened due to the flattening of the slopes or other causes the biggest boulders were left out first, then the small boulders and then the pebbles and then the *bajri*. This fact could be seen when travelling up in a *nalla* from the foot of the hills over high mountains.

The fine Pathankot *bajri* is collected from the bed of the torrents at the foot of the hills which in their upper reaches have big and small boulders.

As in the case of hill torrents, so in the case of our canals, when the velocity of the canals is high enough it carries coarse silt but when the velocity is reduced the water becomes incapable of carrying heavy grades of silt which then are deposited in the bed of upper reaches of canals. As we go down the canal the proportion of heavy grades of silts becomes less and less.

In case of torrents in the hills where very high velocities are obtained, the weak stuff, such as sandstones, gets disintegrated and broken. By impact through the process of rolling some of the big boulders break, others are rounded and the *bajri* and the coarse and fine sands come out of this process. But in the case of our canals the velocities of the flowing water are comparatively so small that no conversion of one grade of silt to another through the above processes is possible except in an imperceptible degree which could conveniently be ignored.

What happens in case of our canals may briefly be stated as below :

Different grades of small pebbles, coarse sand and fine sand are such that for their transportation they are governed by particular volume, slope and velocity of water. As the volume and the velocity of a stream decrease it becomes incapable of rolling down or carrying in suspension heavy grades of silt which are thus left to deposit in its bed. The process is continued and reduction in discharges and velocities results in the corresponding deposit of silt and so on till the very fine silt is left at the tail.

As distinct from coarse and fine sands, there are other soluble clays and soils of very fine consistency which appear in our canal water during the periods of rains and floods. Such tenaciously fine substances are relatively incomparable with sand and silt. The increase of these contents in our flowing canals does not affect very much the silt-carrying capacity of our waters. If at all, it has a slightly lubricating effect on the filaments of flowing water, such that for the same gauge at a particular site the muddy water may have slightly more velocity than the clear water. But this effect is so insignificantly small that no particular notice is taken of it.

In the case of canals, where water is clear and free from coarse sands or where silts are of very fine order, no high velocities are required for the transportation of silt. Very flat slopes and low velocities are required for carrying silted water of fine grades of silt. And these velocities should be provided for in the design of the channels. Generally speaking, the same quantities and the same grades of the silt will not always be found in all the channels. In silting (summer) season there may be more silt while during winter there will be clear water. Excessive quantities of coarse grades of silt will deposit during the summer and that would result in a steeper gradient in the water surface level. But during the winter months some of the silt will be washed down further below and the slope may flatten.

The natural corollary to this assumption would be that all coarse sands incapable of transportation should be excluded from entering into the canal otherwise any such coarse sand once entered into a canal will have to be removed. Entry into the distributaries of light heavy grades of silt which roll down on the bed and cannot be taken down by the water in suspension should be a prohibited thing. Only the fine grades of silt should be allowed to go into the distributaries which could be equitably taken by the outlets to serve as fertilizers.

Geologically the formation of soils and stones in the hilly tracts at the source and along the course of our rivers is different and therefore the grades of silt entering into the different canals of the Punjab are different indeed, such as in the U. B. D. Canal very heavy grades of silt enter into the canal, the like of which is not found in the Lower Bari Doab Canal.

Similarly, the grades of silt entering the Upper Jhelum Canal are much heavier than the grades of silt found in the Lower Jhelum or Lower Chenab Canals.

According to this theory the installation of Silt Ejectors on the U. B. D. C. and U. J. Canal are steps in the right direction in spite of what Mr. R. K. Khanna may say in his publications.

The theory at once explains the very flat slopes obtaining on the Nile in Egypt and the Krishna and the Godawari in India as well as in some other deltaic rivers in different countries.

In the very nature of things, high grades of silt deposits should be found only in the head reaches of the main lines or in the branches only if the silt has been allowed to be taken down into them instead of having been removed or excluded by means of Silt Ejectors.

Keeping in view the velocities as run in the lower reaches of a canal system in the Punjab, the quantity and quality of silt are not so very different and one can safely with discretion frame out rules which should govern the design of our channels.

In the preceding paragraphs notice is taken only of silt and its grades. But there are other factors which govern the flow of water in a stream, such as the hydraulic mean depth of the section, the rugosity of its wetted perimeter, and the layout of a stream, or the straightness or crookedness of its edges. The slope of the water surface has also its effects. The nature of the soil may have an influencing factor in determining suitable velocities. Hard soils may stand while inferior soils may cause scour of bed and sides. Canals may have to be dug or may have to be made artificially in filling. These and some other cases of local nature have to be considered in the design.

## CHAPTER II

## MEANDERINGS OF STREAMS

The study of the meanderings of rivers and streams suggests another important factor which governs the flow of water.

Meandering is most where the bed width of a channel is unnecessarily large and is uncontrolled by berms and banks. In an unnecessarily large bed the stream unloads its silt charge in one place and finds its course easily in another.

Abrupt curves in its course are formed with attending centrifugal action resulting in an unstable flow with varying velocities and different surface slopes which bring in their train further silting and scouring. This vicious circle does not end till a well-defined section with a good berm and banks is formed wherein the stream begins to flow in a stable regime without any silting and scouring.

As in natural streams so in our artificial canals, it is such a regime section that is required for a given discharge, where there should be neither silting nor scouring.

To determine such non-silting non-scouring sections for our canals, efforts have been made in the past. It is the object of this paper to survey in brief some of those efforts and place before the learned reader a few suggestions which should, in the opinion of the writer, govern the design of regime section for our canals and guide their maintenance.

## CHAPTER III

## DIFFERENCE OF OPINIONS

It appears that early Hydraulicians—Chezy, Bazin, Ganguilet, Kutter, Manning, Barnes, Francis and others did not take any cognizance of the presence of silt in flowing water in determining velocity formulæ nor was any notice taken of it in the design of channels.

Early in the nineties of the last century, silt troubles arose rather in an acute form on the Sirhind Canal. Some glimpses of this trouble could be had by going through the Gazetteer of the Sirhind Canal by Mr. Woods. The head reach of this canal silted up almost solidly for miles and lakhs and lakhs of rupees were spent on silt clearance.

It was in the face of these actual difficulties that Kennedy evolved his silt theory and produced the well-known formula $V_o = .84\, d^{.64}$.

It is true that the silt trouble was due as much to the faulty design of the head regulator, if not more, as to the design of the channel itself. However, it cannot be denied that the formula was very useful and saved large amounts to the Government.

As in other sciences so in the science of hydraulics, the wheel of progress must go on.

Kennedy's Diagrams give a wide range of selection and even the cleverest are liable to err. For the formula neglects the width altogether. For the guidance of Irrigation Officers "Wood's Normal data" were useful for the time as these provided a certain ratio of depth to bed width.

The subject was more thoroughly dealt with by Lindley who in his paper before the Punjab Engineering Congress in 1919, produced actual data of channels on the L. C. Canal and suggested that Nature ordained only certain bed widths (neither more nor less) for particular discharges in silt-laden water. This brought a sort of relationship between the discharge and the dimensions of the channels.

Lacey, however, took exception to Kennedy's geometric conception of depth in hydraulics and substituted the old hydraulic conception "Hydraulic Mean Depth" for the geometrical depths and the wetted perimeter for the geometrical bed width.

Lacey's fundamental formulæ are :

$$(i)\quad V = 1.17\sqrt{fR}$$
$$(ii)\quad Af^2 = 3.8\,V^5$$

from which he derived $P = 2.668\sqrt{Q}$. The new term "f" introduced by Lacey in these formulæ was termed by him the "Silt Factor."

This "Silt Factor" has not explicitly been defined but has loosely been said as an equivalent of Kennedy's Vo or CVR = 1.0.

Lacey's wetted perimeter formula $P = 2.668\sqrt{Q}$ has been found to fit in with recorded data remarkably well. The previous formulæ were empirical ones but this one, though derived empirically, is scientifically sound according to the energy theory and has found general support all over the world.

## CHAPTER IV

## ENERGY THEORY OF FLOW AND LACEY'S $P_W = 2.67\sqrt{Q}$

In a flowing stream with a continuous fall in its surface, some energy is expended throughout its length at every step of its way in the performance of some kind of work to overcome the resistance offered by the bed and sides of the stream or, in other words, its wetted perimeter.

It is known that the resistance to flow is directly proportional to the superficial areas of the surface of contact, *i.e.*, to the wetted perimeter (the friction being independent of the fluid pressure) and that it is also proportional to the square of the velocity and inversely proportional to the cross-sectional area of the stream.

Putting this in plain language the energy of the flowing water is balanced by the resistance offered by the wetted perimeter at any section of the stream and, therefore, steady streams flowing with constant discharge and slope must have related wetted perimeter and sectional area. Change in one will naturally involve change in the other.

As the canals are expected to flow practically with constant discharge and constant slope, these canals must have constant wetted perimeter and sectional area.

Any formula which connects discharge Q with wetted perimeter $P_W$ and also with slope S and Hydraulic mean depth R will be useful.

$$\text{Lacey's } P_W = 2.67\sqrt{Q}$$

Mr. G. Lacey, whose contributions to Hydraulic Engineering are so valuable and unique, gives some formulæ based on a vast number of observations.

Plotting various data in his possession, he found that:

$$V = 1.17\sqrt{fR} \quad (1)$$

$$Af^2 = 3.8\, V^5 \quad (2)$$

In these formulæ a new term "f" is introduced by Mr. Lacey which he calls "Silt Factor" and puts it as an equivalent to standard unit C. V. R. of Mr. Kennedy, *i.e.*, $\sqrt{f}$ = I. C. V. R.

By substituting (1) in (2) and putting $A/P_W$ for R and Q for AV, he deduced his third formula:

$$P_W = 2.67\sqrt{Q} \qquad (3)$$

Other important and useful formulæ can be deduced by connecting Lacey's and Manning's formulæ

Lacey's $V = 1.17 \sqrt{fR}$ and

Manning's $V = 67 R^{\frac{2}{3}} S^{\frac{1}{2}}$ (when N = .0222)

Equating we get $\sqrt{f} = 57.265 R^{\frac{1}{6}} S^{\frac{1}{2}}$........(4)

Again Lacey's $V = 1.17 \sqrt{fR}$ and

$$Af^2 = 3.8 V^5$$

$$\text{Or} \quad Qf^2 = 3.8 V^6$$

Therefore, $Qf^2 = 3.8 \ 1.17^6 f^3 R^3$

Or $Q = 9.75 fR^3$..........(5)

Now substituting values of "f" from (4) in (5) we get:

$$Q = 9.75 \times (57.265)^2 \ R^{\frac{1}{3}} S R^3$$

$$= 31974 \ R^{10/3}$$

Or $S = 31974 \ R^{10/3}$................(6)

(Or say S equals to $Q/32000 \ R^{10/3}$)

This slope formula is an important one, as it connects S with R and Q. This relationship was originally derived at by the writer's daughter, Miss Shanta, B.A.

Again

$$Q = AV$$

Therefore, $Q = RP_W \ 67 \ R^{\frac{2}{3}} Q^{\frac{1}{2}}$

But $A = RP_W$

$V = 67 R^{\frac{2}{3}} S^{\frac{1}{2}}$

and $S = Q/31974 \ R^{10/3}$

or $S^{\frac{1}{2}} = \dfrac{Q^{\frac{1}{2}}}{\sqrt{31974} \ R^{5/3}}$

$$\text{Therefore,} \quad Q = \frac{RP_W \ 67 \ R^{\frac{2}{3}} \ Q^{\frac{1}{2}}}{\sqrt{31974} \ R^{5/2}}$$

$$\sqrt{Q} = \frac{67 \ P_W}{\sqrt{31974}}$$

Or $P_W = 2.67 \ \sqrt{Q}$ that is, the same as Lacey's (3).

This equation is of an intrinsic value and has found general support. This establishes that there is a definite wetted perimeter for any given discharge flowing in a channel of rugosity to which the formulæ used for derivation of perimeter equation:

$P_W = 2.67 \ \sqrt{Q}$ apply.

Looking at Lacey's own data from which he deduced his formulæ, it will be seen that the constants of his formulæ are averages and not absolute and are thus liable to modifications.

By adopting slightly different values of the constants in firs two formulæ the constant of his perimeter equation is changed. W can easily get:

$$P_W = 2.5\sqrt{Q} \text{ or } P_W = 2.8\sqrt{Q}$$

Similarly, constants in Manning's formula for values of N othe than 0.0222 would be different and with no difficulty we can get the equation $= P_W = 2.5\sqrt{Q}$

$$\text{Or } P_W = 2.8\sqrt{Q}$$

It is not suggested that 2.5 and 2.8 are the only constants which can be substituted for 2.67 in the equation $P_W = 2.67\sqrt{Q}$.

These could vary from say 2.2 to 3.2 depending on the rugosity of channels and the material of the silt load. The constants 2.5 and 2.8 are selected for reasons of their being generally the common limits.

In smoother surfaces the equation $P_W = 2.5\sqrt{Q}$ and for coarser surfaces the equation $P_W = 2.8\sqrt{Q}$ would fit in better than the equation $P_W = 2.67\sqrt{Q}$. However, the equation $P_W = 2.67\sqrt{Q}$ is quite a good average.

NOTE.—These three equations are plotted on the Kennedy's Hydraulic Diagrams for limiting a zone of selection.

Inter-relationship of these equations can be explained diagramatically as under:


Say originally the channel dug is A B C D with F. S. L. as A. B. This was designed on the equation $P_W = 2.67\sqrt{Q}$.

In course of time, due to falling of berms, section becomes a b c d with W. S. L. at a b. The equation applicable in this case will be $P = 2.8\sqrt{Q}$.

If due to scour and silting of berms the section becomes $a_1 b_1 c_1 d_1$ with F. S. L. $a_1$ $b_1$ equation applicable in that case will be $P = 2.5\sqrt{Q}$

In case of running channels generally we find that when silt is coarse, it deposits itself below falls or control points and water surface levels rise. Above falls there is no rise in water surface levels. Thus the water surface slope becomes steeper. The result of the steepening of the slope is that the channel becomes shallow by falling of berms and in this case perimeter equation tends to have higher coefficient than 2.67, say, $P = 2.8\ Q^{\frac{1}{2}}$ or near about. This is true because in case of a steeper slope there is excessive energy available which results in higher velocity and for higher velocities a greater loss of energy is needed requiring a greater wetted perimeter which the falling of berms affords.

On the other hand if, due to scour, the slope becomes flatter, the berms of the channel will have a tendency to grow and the channel becomes narrower and deeper. The perimeter equation in this case will have a lesser coefficient than 2.67 and the perimeter equation may be $P_w = 2.5\ Q^{\frac{1}{2}}$.

## CHAPTER V

## SOME CHANGING PHASES OF OUR CANALS

Let us consider a canal which for a discharge of one thousand cusecs is designed and built in digging and filling with following sections :

Q = 1,000 cusecs.
S = 1/5714 ,,
R = 72.2 ,,
D = 5.37 ,,
A = 402.13 ,,
V = 2.5 ,,
R = 4.77 ,,
P = 84.23 ,,
$\sqrt{f}$ = 0.989 ,,


Consider the changing phases of the site in digging only. After some time the shape of the channel is changed and these changes could be represented as under :

*Case No.* 1.—When water surface level and slope remains the same such as channel gets wider and shallower. The dimensions actually become as under :

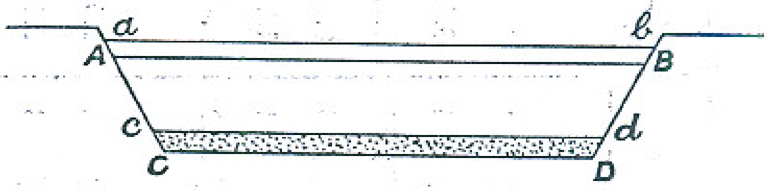
Q = 1,000 cusecs.
B = 80.0 ,,
D = 5.0 ,,
A = 412.5 ,,
R = 4.42 ,,
P = 91.2 ,,
S = 1/5714 ,,


*Case No.* 2.—When the water surface levels and slope still remain the same, but the channel, instead of getting wider, becomes narrower and deeper, such as :

Q = 1,000 cusecs.
B = 66 ,,
D = 5.7 ,,
A = 395.45 ,,
R = 4.98 ,,
P = 78.77 ,,
S = 1/5714 ,,
$\sqrt{f}$ = 0.993 ,,

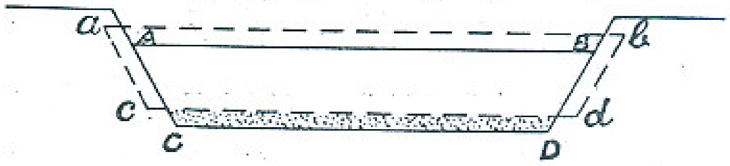
*Case No.* 3.—When the water surface level rises throughout. There is some silt in bed but width does not change appreciably:

Q = 1,000 cusecs.
B = 72 ,,
D = 5.6 ,,
A = 418.9 ,,
V = 2.39 ,,
R = 4.96 ,,
P = 84.55 ,,
S = 1/6666 ,,
/f = 0.923 ,,



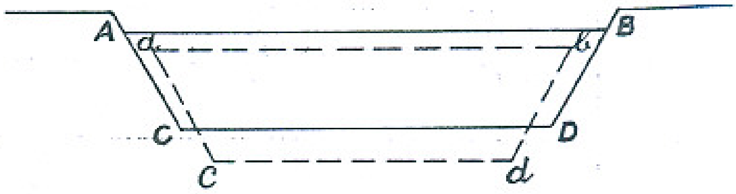

*Case No.* 4.—When due to silt the section and the slope both change, such as:

Q = 1,000 cusecs.
B = 78 ,,
D = 4.75 ,,
A = 381.78 ,,
V = 2.62 ,,
R = 4.31 ,,
P = 88.64 ,,
S = 1/4444 ,,
/f = 0.848 ,,




*Case No.* 5.—When, due to the scour in bed, both the section and slope change:

Q = 1,000 cusecs.
B = 65 ,,
D = 6.3 ,,
A = 429.35 ,,
V = 2.33 ,,
R = 5.43 ,,
P = 79.11 ,,
S = 1/8000 ,,
/f = 0.848 ,,




These examples, instead of actual measurement at site, have been taken out from Improved Kennedy's Hydraulic Diagrams. Such changes as above often come to notice when the channels change their shape which may be due to the wetted surface of the channel getting smoother or rougher or it may be due to change in the size and quantity of the materials of the silt.

The dimensional data of the canal under different conditions of flow as described above have been tabulated in Statement No. 1. It would be noticed that the values of V, R and P are different in all the different phases of the canal. This point is particularly to be remembered when reading Chapter VI on the Irrigation Research Institute Formulæ.

## STATEMENT No. 1

| | Q | B | D | A | V | R | P | S | $\sqrt{I}$ | Coefficient in $P=C\sqrt{Q}$ |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | *By Kennedy's Diagrams* | | | | | | |
| Original design .. | Cusecs 1,000 | 72.2 | 5.37 | 402.13 | 2.5 | 4.77 | 84.23 | 1/5,714 | 0.989 | 2.67 |
| Case I .. | 1,000 | 80.0 | 5.0 | 412.5 | 2.42 | 4.52 | 91.2 | 1/5,714 | 0.978 | more than 2.8 |
| Case II .. | 1,000 | 66.0 | 5.7 | 392.45 | 2.55 | 4.98 | 78.77 | 1/5,714 | 0.993 | 2.5 |
| Case III .. | 1,000 | 72.0 | 5.6 | 418.9 | 2.39 | 4.96 | 84.55 | 1/6,666 | 0.923 | 2.67 |
| Case IV .. | 1,000 | 78.0 | 4.75 | 381.78 | 2.62 | 4.31 | 88.64 | 1/4,444 | 1.097 | 2.8 |
| Case V .. | 1,000 | 65 | 6.3 | 429.35 | 2.33 | 5.43 | 79.11 | 1/8,000 | .848 | 2.5 |
| | | | | *By Lacey's Diagrams* | | | | | | |
| Original design .. | 1,000 | 72.5 | 5.3 | 398.3 | 2.51 | 4.66 | 84.37 | 1/5,714 | 1.0 | 2.67 |
| Case III .. | 1,000 | 72.0 | 5.5 | 411.13 | 2.43 | 4.88 | 84.32 | 1/6,666 | 0.91 | 2.67 |
| Case IV .. | 1,000 | 73 | 5.0 | 377.5 | 2.65 | 4.48 | 84.20 | 1/4,444 | 1.16 | 2.66 |
| Case V .. | 1,000 | 72 | 5.7 | 426.65 | 2.34 | 5.03 | 84.77 | 1/8,000 | 0.82 | 2.68 |

*Very important* {
For the same one slope Lacey's Diagrams give only one dimensional design. Different conditions of rugosity need different sections even with the same slope.
Kennedy's Diagrams give wide range of dimensional designs for a given slope now restricted by perimeter curves to suit different rugosities. In this lies the superiority of Kennedy's Diagrams to Lacey's.

# CHAPTER VI

## IRRIGATION RESEARCH INSTITUTE FORMULAE

The Irrigation Research Institute have produced the following relationships:

$$V = 1.12 \sqrt{R} \qquad (1)$$
$$P = 2.80 \sqrt{Q} \qquad (2)$$
$$R = .47\, Q^{\frac{1}{3}} \qquad (3)$$
$$S = \frac{2.09\, m^{0.86}}{1{,}000\, Q^{.21}}$$

Implication of these relationships could be explained as under:

$$\frac{A}{P} = R$$
$$A = PR$$
$$AV = PRV$$
$$Q = PR\, 1.12 \sqrt{R} = 1.12\, PR^{3/2}$$
$$Q = 1.12\, P\, (.47\, Q^{\frac{1}{3}})^{3/2}$$
$$Q = 1.12 \times .47^{3/2}\, Q^{\frac{1}{2}}\, P$$
$$\sqrt{Q} = 1.12 \times .47^{3/2}\, P$$

Or $$P = \frac{1}{1.12 \times .47^{3/2}} \times Q^{\frac{1}{2}} \qquad \text{(A)}$$

$$\underline{P = 2.8\, Q^{\frac{1}{2}}}$$

Suppose $V = 1.13 \sqrt{R}$

$R = .48\, Q^{\frac{1}{3}}$

The equation will read:

$$P = \frac{1}{1.13 \times .48^{3/2}} \sqrt{Q}$$
$$= \frac{1}{.375}\, Q^{\frac{1}{2}}$$
$$P = \frac{8}{3}\, Q^{\frac{1}{2}} \text{ Exactly Lacey's}$$

Again suppose $V = 1.14 \sqrt{R}$

and $R = .50\, R$

Then the equation will read:

$$P = \frac{1}{1.14 \times .50^{3/2}}\, Q^{\frac{1}{2}}$$
$$P = \frac{1}{.40}\, Q^{\frac{1}{2}} = 2.5\, Q^{\frac{1}{2}}$$

*Another Implication.*—If we accept the constants of the first three Punjab formulæ as rigid and absolute, then it will come to this that in designs for any given discharge in the Punjab channels V, P and R are fixed and therefore A is fixed because $\frac{V}{Q} = A$ and also $P \times R = A$.

As a corollary to this, slope is also fixed and in that case there is no necessity of the 4th formula $S = \frac{2.09m^{0.86}}{1,000\ Q^{.21}}$. On the other hand, if the slope be first fixed with this formula it is likely the other three formulæ may not exactly apply. Also, what about actual changes in the water surface slopes and water widths resulting in change of P and R as well? All this happens under conditions of the same discharge.

Change in slope due to bigger grade of silt means change in velocity and consequently change in its waterway, i.e, A which is equal to $P \times R$.

It has been established that the constant of the equation $P = C_2\ Q^{\frac{1}{2}}$ does vary from 2.2 to 3.2 although limits of 2.5 to 2.8 have been advocated.

The range of change in R under variation of S and P is not precisely established but it is manifest.

Accepting the form of the formula $R = .47\ Q^{\frac{1}{3}}$ as $R = C_3\ Q^{\frac{1}{3}}$ an effort is made to determine different values of the constant $C_3$ in this formula under various phases of the canal represented by the perimeter equations $P = C_2\ Q^{\frac{1}{2}}$.

Taking Kennedy's Data of the Upper Bari Doab Canal appearing in Appendix A of Publication No. 20 of the Central Board of Irrigation, the values for constants are worked out as given in the following table:

| Q | P | R | V | $C_2 = \frac{P}{\sqrt{Q}}$ | $C_3 = \frac{P}{Q^{\frac{1}{3}}}$ | $C_1 = \frac{V}{R^{\frac{1}{2}}}$ | Remarks |
|---|---|---|---|---|---|---|---|
| 1700.6 | 98.63 | 6.14 | 2.82 | 2.4 | .514 | 1.06 | |
| 1498.4 | 90.01 | 5.98 | 2.79 | 2.32 | .528 | 1.14 | |
| 1254.1 | 86.74 | 5.55 | 2.02 | 2.45 | .515 | 1.11 | |
| 941.1 | 80.42 | 4.79 | 2.46 | 2.62 | .49 | 1.12 | |
| 673.6 | 63.60 | 4.49 | 2.37 | 2.45 | .51 | 1.11 | |
| 302.9 | 37.17 | 3.62 | 2.21 | 2.14 | .539 | 1.16 | |
| 129.6 | 23.55 | 2.75 | 2.00 | 2.07 | .54 | 1.10 | |
| 78.3 | 20.36 | 2.24 | 1.71 | 2.30 | .52 | 1.15 | |
| 65.0 | 18.69 | 2.07 | 1.68 | 2.30 | .51 | 1.16 | |
| 29.1 | 13.48 | 1.58 | 1.36 | 2.69 | .51 | 1.08 | |

From the above the undermentioned values of the coefficient $C_3$ in the formula $R = C_3 Q^{\frac{1}{3}}$ are tentatively suggested for different coefficients in the perimeter equations:

When $P = 2.3 Q^{\frac{1}{2}}$ then $R = .52 Q^{\frac{1}{3}}$

,, $P = 2.4 Q^{\frac{1}{2}}$ ,, $R = .51 Q^{\frac{1}{3}}$

,, $P = 2.5 Q^{\frac{1}{2}}$ ,, $R = .50 Q^{\frac{1}{2}}$

,, $P = 2.6 Q^{\frac{1}{2}}$ ,, $R = .49 Q^{\frac{1}{3}}$

,, $P = 2.7 Q^{\frac{1}{2}}$ ,, $R = .48 Q^{\frac{1}{3}}$

,, $P = 2.8 Q^{\frac{1}{2}}$ ,, $R = .47 Q^{\frac{1}{3}}$

,, $P = 2.9 Q^{\frac{1}{2}}$ ,, $R = .46 Q^{\frac{1}{3}}$

,, $P = 3.0 Q^{\frac{1}{2}}$ ,, $R = .45 Q^{\frac{1}{3}}$

A test

We have $P \times R$ is equal to A and

$$V = \frac{A}{Q}$$

Adopting $V = 1.12 \sqrt{R}$ for equation $P = 2.8^{\frac{1}{2}}$

,, $V = 1.13 \sqrt{R}$ ,, ,, $P = 2.67^{\frac{1}{2}}$

,, $V = 1.4 \sqrt{R}$ ,, ,, $P = 2.5 Q^{\frac{1}{2}}$

The following statement called "Table A" is worked out for different discharges from 10 to 20,000 cusecs.

A study of Table A (page 263) shows that the results are all within 1 per cent.

It will be seen that the coefficient .47 in the formula $R = .47 Q^{\frac{1}{3}}$ is correct for equation $P = 2.8Q^{\frac{1}{2}}$ only. It cannot be correct for other coefficients of the perimeter equation $P = C_2 Q^{\frac{1}{2}}$ because for a constant discharge $(Q = P \times R \times V)$ when P gets less than the product of $R \times V$ should be more and *vice versa*.

The Equation $R = .48 Q^{\frac{1}{3}}$ for Equation $P = 2.67 Q^{\frac{1}{2}}$ and the Equation $R = .50 Q^{\frac{1}{3}}$ for Equation $P = 2.5 Q^{\frac{1}{2}}$ fit in as nearly enough as

Equation $R = .47 Q^{\frac{1}{3}}$ for Equation $P = 2.8 Q^{\frac{1}{2}}$

Generally speaking, for any particular discharge the relationship will hold $P \times R \times V = P_1 \times R_1 \times V_1 = P_2 \times R_2 \times V_2$, etc. etc.

As in permutations and combinations, change in one results in change in others to keep up the balance, so in hydraulics of a channel, balance is to be maintained between P and R to pass a certain constant discharge.

In this connection it will be interesting to refer to statement No. 5 with Chapter XIII and see the corresponding variations in values of P and R (and also of V because $V = C\sqrt{R}$ ).

Take, for instance, data of design for a particular discharge say 500′ cusecs for a particular slope, say, 1/5,000.

First mark variations in P, R and V under different perimeter equations. Then notice the corresponding changes when the slope flattens to 1/5,714 or 1/6,666 or 1/8,000. Similarly, changes may be seen when the slope steepens to 1/4,444 or 1/4,000 or 1/3,333.

Such a study will be found very suggestive and instructive as well. To maintain a certain slope in any channel of a constant discharge for obtaining desired water surface levels, it will be necessary to maintan a certain relationship between P and R.

Various corresponding values of constants as suggested above will be found good enough for practical purposes. These should not be taken as absolute and rigid.

## TABLE A

| Q | $Q^{\frac{1}{2}}$ | $Q^{\frac{1}{3}}$ | $P = 2.8\ Q^{\frac{1}{2}}$ P | $R = .47\ Q^{\frac{1}{3}}$ R | $V = 1.1\sqrt{R}$ V | $Q = P \times R \times V$ | $P = 2.67\ Q^{\frac{1}{2}}$ P | $R = .48\ Q^{\frac{1}{3}}$ R | $V = 1.13\sqrt{R}$ V | $P \times R \times V = Q$ | P | $R = .50\ Q^{\frac{1}{3}}$ R | $V = 1.14\ R^{\frac{1}{2}}$ V | $P \times R \times V = Q$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 10 | 3.16 | 2.154 | 8.85 | 1.01 | 1.13 | 10.10 | 8.44 | 1.03 | 1.15 | 10.00 | 7.9 | 1.07 | 1.18 | 9.97 |
| 50 | 7.07 | 3.68 | 19.80 | 1.73 | 1.47 | 50.35 | 18.88 | 1.77 | 1.50 | 50.13 | 17.67 | 1.84 | 1.55 | 50.39 |
| 100 | 10.00 | 4.642 | 28.00 | 2.18 | 1.65 | 100.7 | 26.7 | 2.23 | 1.69 | 100.62 | 25.0 | 2.32 | 1.74 | 100.9 |
| 150 | 12.242 | 5.313 | 34.29 | 2.50 | 1.77 | 151.74 | 32.7 | 2.55 | 1.80 | 150.09 | 30.62 | 2.65 | 1.86 | 150.9 |
| 250 | 15.81 | 6.299 | 44.27 | 2.96 | 1.93 | 252.9 | 42.21 | 3.02 | 1.96 | 249.8 | 39.53 | 3.15 | 2.02 | 251.5 |
| 800 | 22.36 | 7.937 | 62.61 | 3.73 | 2.16 | 504.45 | 59.7 | 3.81 | 2.81 | 502.7 | 55.9 | 3.97 | 2.27 | 503.8 |
| 1,000 | 31.6 | 10.0 | 88.48 | 4.70 | 2.43 | 1,010.5 | 84.4 | 4.8 | 2.48 | 1,004.7 | 79.0 | 5.0 | 2.55 | 1,007.3 |
| 2,000 | 44.72 | 12.60 | 125.12 | 5.92 | 2.72 | 2,016.34 | 119.04 | 6.05 | 2.78 | 2,008.0 | 111.8 | 6.30 | 2.86 | 2,014.4 |
| 5,000 | 70.71 | 17.10 | 189.0 | 8.04 | 3.18 | 5,062.05 | 188.52 | 8.20 | 3.24 | 5,008 | 176.8 | 8.55 | 3.33 | 5,033.8 |
| 10,000 | 100.0 | 21.54 | 280.0 | 10.13 | 3,56 | 10,097.58 | 267.0 | 10.34 | 3.63 | 10,022 | 250.0 | 10.77 | 3.74 | 10,070 |
| 20,000 | 141.42 | 27.14 | 395.97 | 12.74 | 4.00 | 20,210 | 377.6 | 13.03 | 9.08 | 20,074 | 353.6 | 13.57 | 4.20 | 20,153 |

## CHAPTER VII

## VELOCITY FORMULA

Mr. Ivan E. Houk of the Miami Conservancy Project in his book "Calculation of Flow in Open Channels," published by the Miami Conservancy Directorate, has, after a very thorough and exhaustive examination of the formulæ known for finding the velocity, declared that the Ganguillet and Kutter's expression for C in the Chezy's formula $V = C\sqrt{Rs}$ gives more reliable and constant results than any other formula yet known. It is not proposed to repeat the text. The reader is referred to pages 125 to 261 of the book.

Kutter's expression for C in Chezy's formula is:

$$C = \frac{41{\cdot}6 + \frac{1{\cdot}811}{N} + \frac{0{\cdot}00281}{S}}{1 + \left(41{\cdot}6 + \frac{{\cdot}00281}{S}\right)\frac{N}{\sqrt{R}}}$$

In this it will be seen that value of C depends on R S and N V.

For any particular section and slope R and S are constants and it is N which changes the coefficient. For open channels in alluvium values of N have been determined and adopted.

In the Punjab the practice is to take the value of N as ·0225 and this is the one that generally obtains and is safe.

In this connection the writer collected the data of some channels on the L. J. C. and also took advantage of the most valuable data of discharge observations published by Mr. Lacey in his Paper No. 4893 before the Institution of Civil Engineers.

The data of about 210 sites (out of which 139 have been taken from Lacey's paper) have been collected and arranged in ascending order of R as well as of S; the value of Kutter's N has been calculated from these authentic data of artificial channels. These values are plotted and a line of N = .0225 is drawn. It has been seen that this line is a fairly good average line and supports the Punjab practice to a considerable extent.

Mr. Kennedy very wisely chose his basic stand on the universally accepted Chezy's formula with the value of C given by Kutter's expression. Kutter's N has been kept as .0225.

The Kennedy's Hydraulic Diagrams based on such an accepted formula should be very accurate indeed. This led the writer to a closer study of these diagrams.

# CHAPTER VIII

## KENNEDY'S HYDRAULIC DIAGRAMS—A STUDY

For the design of earthen channels no other book has, during the past 30 years, been so widely and authoritatively used as the Kennedy's Hydraulic Diagrams. In spite of the fact that so much of late has been written on the shortcomings of Kennedy's Rule and its fallacies, these diagrams still hold the ground and have not officially been replaced by any other publication. What is the bright side of Kennedy's Diagrams? What is their dark side? What is the cause of their popularity? What is the cause of complaint about them? Is it possible to retain the good points and remedy the flaws?

It is in reply to the above questions and some more that the following lines are written:

### Utility of the diagram

For design work calculations are considered tedious and the average designer prefers to use diagrams if these diagrams give accurate results. Kennedy's Hydraulic Diagrams represent the universally accepted Kutter's Formula with the value of $N = .0225$ and, therefore, their correctness becomes acceptable.

For their accuracy and directness they have received respectful recognition. For a given slope they give at a glance the breadth, depth and velocity for any discharge. This utility of the diagrams is commendable.

### Kennedy's silt theory

But associated with these diagrams is Mr. Kennedy's Vo Silt theory. Mr. Kennedy deduced from observations made on channels of the Upper Bari Doab that for a non-silting, non-scouring channel in regime there is always one velocity and that velocity is a function of its depth. Mr. Kennedy calls this the critical or non-silting non-scouring mean velocity and expresses it as Vo, where $V_o = 0.84\ D^{.64}$. In the introduction of his Hydraulic Diagrams, Mr. Kennedy says that velocity greater than Vo for a particular depth will scour and velocity less than Vo will deposit silt.

This standard Vo velocity applies to the silt grade prevailing on the Upper Bari Doab System. But when the grade of silt is different so as to require a different velocity (V) that would cause neither silt nor scour, then the ratio between these two velocities, *viz.*, V/Vo, is defined as Kennedy's Critical Velocity Ratio. Kennedy has left to the designer on other channels to determine for himself the C. V. R. which conforms to Vo conditions on the U. B. D. C.

## Hydraulic Diagrams

These hydraulic diagrams have been drawn with the value of Kutter's N as .0225. But for a channel, where the value of Kutter's N is more or less than .0225, some correction factor is necessary. Calculations of formula with Kutter's N having lower values than .0225 give higher velocities and with values of N greater than .0225 give lower velocities. Mr. Kennedy, in the introduction to his diagrams, says that for values of Kutter's N = .02 the discharge should be taken as about 12½ per cent. more than that given in the diagrams or, in other words, the velocity is 12½ per cent. higher than what is given for that particular depth to which the velocity corresponds. This would, incidentally, raise the C. V. R. for the depth on account of increased velocities; therefore, the curves for C. V. R. are altogether vitiated by considering a Kutter's N as different from .0225.

The object of the present note being to examine the hydraulic diagrams as published, comments regarding C. V. R. (and velocity Vo) are applicable to the published diagrams having Kutter's N = .0225.

Mr. Kennedy has drawn curves for Vo and multiples of Vo on his hydraulic diagrams. There are ten sets of diagrams. The last being only for Vo, *i.e.*, for the standard velocity at any particular depth for all slopes and for any discharge. The previous nine sets are for slopes 1/10,000 to 1/2,000. Besides curves for velocity and discharge, curves in dotted lines are given for critical velocity ratios and the table below gives the approximate range of C. V. R. in different diagrams :

BED WIDTHS

| 2′ to 32′ to 200′ | | | | |
|---|---|---|---|---|
| *Slopes* | | | | |
| 1/10,000 | .65 | .72 | .86 | |
| 1/8,000 | .65 | .80 | .92 | |
| 1/6,666 | .70 | .88 | .Vo | |
| 1/5,714 | .70 | .92 | 1.05 | |
| 1/5,000 | .70 | .Vo | 1.18 | |
| 1/4,444 | .70 | 1.08 | 1.23 | |
| 1/4,000 | .8 | 1.15 | 1.35 | |
| 1/3,636 | .8 | 1.2 | 1.35 | 70 ft. of bed-width to 40 ft. bed-width to 39 ft. |
| 1/3,333 | .85 | 1.30 | 1.35 | bed-width. |
| 1/2,857 | .85 | 1.35 | | |
| 1/2,500 | .90 | 1.50 | | |

## Hydraulic Diagrams

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| 2′ to 32′ to 200′ | | | | |
|---|---|---|---|---|
| *Slopes* | | | | |
| 1/10,000 | .65 | .72 | .86 | |
| 1/8,000 | .65 | .80 | .92 | |
| 1/6,666 | .70 | .88 | .Vo | |
| 1/5,714 | .70 | .92 | 1.05 | |
| 1/5,000 | .70 | .Vo | 1.18 | |
| 1/4,444 | .70 | 1.08 | 1.23 | |
| 1/4,000 | .8 | 1.15 | 1.35 | |
| 1/3,636 | .8 | 1.2 | 1.35 | 70 ft. of bed-width to 40 ft. bed-width to 39 ft. bed-width. |
| 1/3,333 | .85 | 1.30 | 1.35 | |
| 1/2,857 | .85 | 1.35 | | |
| 1/2,500 | .90 | 1.50 | | |

From the perusal of the above table it appears that it will not be possible to attain Vo Velocity for any discharge with slopes 1/10,000 and 1/8,000 for channels of any dimensions whatsoever, and with slopes 1/6,666 and 1/5,714 for channels of bed-widths less than 32 feet, unless values of N are lower than .0225. In other words, such channels will ever remain non-scouring and silting for localities which, according to Mr. Kennedy, should have velocity Vo for non-scouring conditions. Similarly, channels with steep slopes and bed-widths requiring velocities more than Vo would ever remain non-silting and will always be scouring. It is clear, therefore, that Vo velocity cannot be obtainable for all slopes even on a system requiring Vo velocities such as the Upper Bari Doab Canal.

Often large areas are found where nothing but flat slopes are available, or where steep slopes are unavoidable. Many irrigation engineers must have noticed channels of all dimensions working with flat slopes without showing signs of silting and many channels with steep slopes which show no signs of scour.

Not only are there difficulties of the sort mentioned above but there are no hard and fast rules by which to determine the values of C. V. R. for the design of channels. Much is, therefore, left to the experience (or the lack of it) and to the personal likes and dislikes of the designer who may, moreover, be handicapped by the slopes obtainable. It is for lack of such guides that Kennedy's Hydraulic Diagrams offer a wide range of dimensional design of channels for any given discharge for a particular given slope. Even Mr. Kennedy's own Vo theory does not help much. This is the most vulnerable point in Kennedy's theory.

## Neglecting even Kennedy's Vo Theory, his Diagrams Still Useful

But, leaving aside Kennedy's Silt Theory of Vo and neglecting curves of C. V. R., the Hydraulic Diagrams still represent Chezy's Formula with Kutter's Coefficient. So long as the correctness of this formula is accepted, these hydraulic diagrams should serve our purpose, if we can only have means of spotting at once the correct section out of the wide range offered by them. This is dealt with in another chapter.

In the discussion that follows, the writer has tried to put up facts as he has found them before the reader and has suggested some additions in Kennedy's Hydraulic Diagrams which will not only keep up the identity of their great author intact but will also meet the critics to a great extent.

Thus Improved Kennedy's Hydraulic Diagrams retain all their previous utility and contain matter which will bring them in conformity with the present-day knowledge and requirements.

## CHAPTER IX

## IMPROVED KENNEDY'S HYDRAULIC DIAGRAMS

For design work, calculations are considered tedious and the average designer prefers to use diagrams. It is for this psychology of the human mind that Kennedy's Diagrams are still so popular in spite of the fact that many engineers have lost faith in Kennedy's Vo Theory, yet they use their individual discretion in picking up a design (out of the wide range offered by the diagrams) by fixing some C. V. R. arbitrarily.

Experience has shown that Kutter's Formula, though cumbersome, gives the best results. It has been established that the general value of N found from the observed data of channels in the Punjab is .0225. Therefore, Kennedy's Diagrams based on Kutter's Formula with N=.0225 are considered to fit in with actual data very well indeed. Lacey's Perimeter Curves plotted on these diagrams bring them abreast of the present-day knowledge and enhance the utility of these diagrams.

The perimeter curves plotted on Kennedy's Diagrams take away all the indefiniteness of these diagrams and make them exact for practical purposes. Thus improved, the diagrams, which may be called "Improved Kennedy's Hydraulic Diagrams," will help the designer to put his finger on the more or less exact required section without his bothering about the C. V. R. or "f."

For this purpose it will not be necessary to take a new edition of these diagrams. All that is needed is that perimeter curves be drawn on the existing copies by the Head Draftsmen of various offices.

Blue-print copies of the complete set of Improved Kennedy's Hydraulic Diagrams will be shown during the session. However, as a specimen one diagram only is printed with the paper.

### Diagrams Explained

The three perimeter equations have been superimposed on Kennedy's Hydraulic Diagrams. Intersection of discharge curves with these lines spots the dimensions of the channels for any given discharge. As pointed out, the line for the equation $P_w = 2.67\sqrt{Q}$ is quite a good average and the other lines for equation $P_w = 2.5\sqrt{Q}$ and $P_w = 2.8\sqrt{Q}$ may be treated as limits which define a useful zone for selecting dimensions keeping in view the rugosity of the section.

Another information plotted on these diagrams is the value of $\sqrt{R}$. Curves of $\sqrt{R}$ are shown in dotted lines. It is significant to note that their layout is practically parallel to the velocity curves in all the diagrams.

Considering curves of $\sqrt{f}$ R parallel to the curves of V where values of $\sqrt{R}$ and V are known and by taking Lacey's formula $V=1.17\sqrt{fR}$ into consideration, then on any curve of $\sqrt{R}$ parallel to curve of velocity the value of $\sqrt{f}$ is the same throughout.

Therefore, on the curves of $\sqrt{R}$ the values of $\sqrt{f}$ have been given and they serve a useful purpose. We can now say that these curves of $\sqrt{R}$ serve as curves of $\sqrt{f}$ as well.

With the plotting of perimeter curves we get out of the difficulty of the wide range of dimensional design given by original Kennedy's Diagrams. These curves satisfy the wetted perimeter equations and spot B, D, V, $\sqrt{R}$ and $\sqrt{f}$ for any discharge for a given slope.

### Yet Another Difficulty

On each diagram for different slopes the data of design for any discharge are spotted, but for a design of any channel one has to provide for the highest to the lowest discharges with different slopes and it is desired to have a diagram by which one can know the slopes one may adopt for these varying discharges which give the desirable value of $\sqrt{f}$ if one is so inclined as to take $\sqrt{f}$ as a guide.

### Difficulty Solved

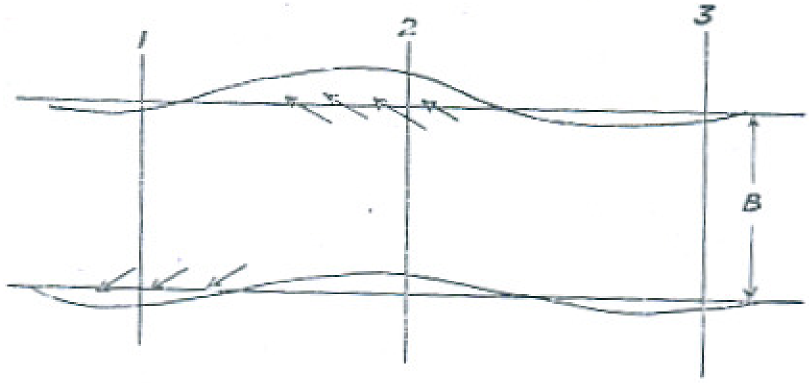
A separate diagram has been prepared to meet the requirements. In fact this is an abstract of the nine diagrams. It is also on the same scale. On a blank form of Kennedy's Diagrams are transferred perimeter curves for $P_W=2.67\sqrt{Q}$ for all slopes from 1/10,000 to 1/2,000. Points of intersection of the perimeter curves with curves of Q, V, F and $\sqrt{R}$ are joined which thus give the curves for discharges, velocities $\sqrt{f}$ and $\sqrt{R}$ on the new diagram.

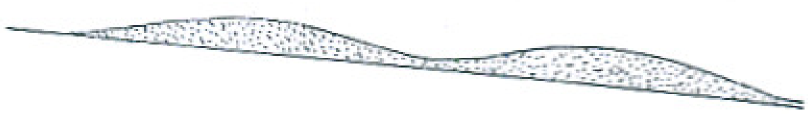
This diagram is unique and is very useful. On it one can design the channels from the highest 10,000 cusecs to the lowest discharge adopting values of $\sqrt{f}$ he desires. This diagram gives also the alternative dimensional design of a channel for the same discharge with different slopes.

This diagram gives an interesting study ; B. D. V. $\sqrt{R}$ and $\sqrt{f}$ are given for any discharge on any slope. Curves satisfying the equation $P_W = 2{\cdot}8\sqrt{Q}$ and $P_W = 2.5\sqrt{Q}$ have not been drawn but the reader is strongly recommended to refer to detailed diagrams to know their effect on the design. It is useful to note that for a particular discharge the difference in bed-width is not much. For steeper slopes bed-widths slightly increase and depths diminish, and for flatter slopes bed-widths decrease and depths increase. Taking an example of a channel running with 1,000 cusecs the bed-width will vary from 70 feet (for slope 1/10,000 to 74 feet for slope 1/4,000). What a small change in width. Why at all should channels be allowed to get wider and wider ? More light is thrown on this aspect in the next chapter.

# CHAPTER X

A plan and long section of an actual reach of a channel is given below :



At the three sites 1, 2 and 3

$$A_1 V_1 = A_2 V_2 = A_3 V_3 = Q$$

$$\text{Or} \quad B_1 D_1 V_1 = B_2 D_2 V_2 = B_3 D_3 V_3 = Q$$

There is no appreciable difference in velocities ; the abnormal increase in widths is due to falling of berms ; therefore, to keep the required waterway D must diminish and the silting of bed is the result.

When such a state occurs in a channel there is a direct thrust of current against the berms and they give way and fall off. If unprevented, this vicious circle of change in the direction of current goes on. The falling of the berms and the silting of the whole channel gradually become too evident. For the proper upkeep of a channel, straight and parallel berms are essential and must be so maintained even artificially when necessary to achieve some sort of a regime in an artificial channel.

In this connection a word about watching the regime is suggested. In olden days bed blocks were fixed to watch the bed level and locate the centre line. In certain places even profiles were made but this is too expensive. The suggestion is to fix two pieces of rails equidistant from the centre line, say, at every 1,000 feet, and also on curves. With these, one can know how the channel is behaving.

## Effect of silt and slope on widths

Lacey has said that for a given discharge and given silt factor the cross-sectional area and wetted perimeter are uniquely determined. With no reference to any silt factors Lindley said in his Congress Paper of 1919 almost the same thing to which reference has already been made in Chapter III.

Lacey's $P = 2.67\ Q^{\frac{1}{2}}$ signifies that for any discharge there is a definite P. Punjab Research Institute's formula $R = .47Q^{\frac{1}{3}}$ establishes that for any discharge there is a definite R, the Hydraulic mean depth. So Nature has fixed a definite section of the channel with definite P and R and so a definite area of any particular discharge.

For three particular discharges 50 cusecs, 500 and 5,000 cusecs channels, dimensions and other data have been taken from Lacey's Curve $P=2.67\sqrt{Q}$ on Improved Kennedy's Hydraulic Diagrams for 12 different slopes and tabulated in statement No. 1A. Taking Q=50 cusecs for the slope of 1/5,000, B=13.0 and D=2.5, if due to fine silt and smoother surface the slope flattens by scour up to 1/10,000 the channel width will become 11 feet wide depth 3.4; on the other hand, if silt is coarse and surface rough, it may attain a steeper slope by the falling of berms and the silting of bed, say, up to 1/2,000, the section will be in that case B=15 and D=1.75. These are the limits.

It must be admitted that the range of change in widths is not very marked. If one designs a channel with perimeter curves from regime slope and maintains the berms straight and parallel the channel would attain some sort of regime and in course of time will adjust to its right section, not very much different to one designed for it. The difference in width will be very small indeed.

It is desirable not to allow channels to get wider and wider. Timely action should always be taken to prevent side erosion in odd places and to keep berms always straight and parallel.

To make channels wider than designed by unnecessary berm-cutting or trimming should be avoided.

## STATEMENT NO. 1A

| Slope | Q=50 | | | | | Q=500 | | | | | Q=5000 | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | B | D | V | /f | R | B | D | V | /f | R | B | D | V | /f | R |
| 1/10,000 | 11 | 3.4 | 1.16 | .65 | 2.32 | 47.5 | 5.42 | 1.84 | .74 | 4.56 | 116.5 | 9.9 | 2.95 | .83 | 9.12 |
| 1/8,000 | 12 | 3.0 | 1.21 | .69 | 2.19 | 48.5 | 4.95 | 1.98 | .82 | 4.27 | 168.5 | 9.2 | 3.11 | .92 | 8.46 |
| 1/6,000 | 12.5 | 2.85 | 1.29 | .76 | 2.10 | 49 | 4.70 | 2.07 | .88 | 4.06 | 169 | 8.70 | 3.31 | .99 | 8.00 |
| 1/5,714 | 13.5 | 2.60 | 1.33 | .83 | 1.98 | 50 | 4.40 | 2.22 | .94 | 3.88 | 170 | 8.35 | 3.44 | 1.06 | 7.70 |
| 1/5,000 | 13.0 | 2.5 | 1.40 | .86 | 1.90 | 50 | 4.2 | 2.25 | 1.00 | 3.72 | 171 | 7.95 | 3.58 | 1.13 | 7.36 |
| 1/4,444 | 13.8 | 2.35 | 1.43 | .90 | 1.84 | 50.5 | 4.05 | 2.34 | 1.05 | 3.57 | 172 | 7.70 | 3.70 | 1.18 | 7.16 |
| 1/4,000 | 14.0 | 2.2 | 1.47 | .94 | 1.78 | 50.5 | 3.90 | 2.40 | 1.09 | 3.49 | 172 | 7.45 | 3.81 | 1.22 | 6.94 |
| 1/3,636 | 14.0 | 2.15 | 1.53 | 1.0 | 1.73 | 57.0 | 3.85 | 2.67 | 1.13 | 3.42 | .. | .. | .. | .. | .. |
| 1/3,333 | 14.5 | 2.10 | 1.57 | 1.01 | 1.70 | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. |
| 1/2,857 | 14.5 | 1.95 | 1.63 | 1.09 | 1.62 | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. |
| 1/2,500 | 14.5 | 1.95 | 1.7 | 1.16 | 1.56 | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. |
| 1/2,000 | 15.0 | 1.75 | 1.79 | 1.26 | 1.47 | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. |

# CHAPTER XI

## STUDY OF THE IMPROVED KENNEDY'S HYDRAULIC DIAGRAMS

C. V. R. and $\sqrt{f}$ compared.

Comparing the layout of the curves of $\sqrt{f}$ with the curves of C.V.R. it is to be noticed that their layout is practically at right angles to each other. Although they do not belong to the same family, it is significant to note that these curves of the same value of $\sqrt{f}$ and C. V. R. intersect almost within the zone of the perimeter curves.

### Variations in the Values of C. V. R. and $\sqrt{f}$

For Lacey's Equations $P_w = 2.67\sqrt{Q}$ values of C. V. R. and $\sqrt{f}$ have been calculated for different discharges from 10 cusecs to 10,000 cusecs for different slopes. These values of C. V. R. and $\sqrt{f}$ are abstracted in a single Statement No. 2 appended.

In this statement under each slope and opposite given discharges are shown values of C. V. R. as numerator and values of $\sqrt{f}$ as denominator.

A perusal of this statement gives an interesting study. The relationship between C. V. R. and $\sqrt{f}$ is vividly noticeable. The statement also shows what should be the C. V. R. or $\sqrt{f}$ for different discharges under different slopes, where the perimeter equation $P_w = 2.67\sqrt{Q}$ holds good, accepting, of course, that the equation $P_w = 2.67\sqrt{Q}$ is to be satisfied to form a regime channel. It would be clear that for non-silting non-scouring regime channel values of C. V. R. or $\sqrt{f}$ will be different for different discharges and for different slopes.

There cannot practically be the same C. V. R. or the same silt factor for the whole system of a canal ranging from head with say 10,000 cusecs to tail say 10 cusecs, although theoretically it may be possible to select slopes arbitrarily to admit of the same $\sqrt{f}$.

### Changing Values of $\sqrt{f}$ and C. V. R.

A closer examination of this statement indicates that with the increase of the discharge, the value of $\sqrt{f}$ increases as also with the steepening of the slope.

The above conclusion from the study of the Improved Kennedy's Hydraulic Diagrams is very important and it finds support in Lacey's writings.

In the Technical Paper No. 1 of the United Provinces, "Regime Diagrams for the Design of Canals and Distributaries" by Lacey, there is a regime slope diagram (Plate II). The study of this diagram confirms 100 per cent. the conclusion drawn above. Taking for instance any particular discharge and reading against different slopes the silt factor on vertical lines, it would be noticed that with the steepening of slopes the silt factor increases. Again, reading against any particular slope on a horizontal line, it will be noticed that with an increase in discharge the value of "f" also increases.

To make the meaning clear, a tabular statement (No. 3) of the values of "f" for discharges 10 to 10,000 cusecs for different slopes is prepared from Lacey's Regime Slope Diagram Plate II in U. P. Technical Paper No. 1. It will be noticed that the value of "f" increases with the increase in the discharge and it increases also with the steepening of the slope.

In chapter IV, the value of $\sqrt{f}$ was worked out as $\sqrt{f} = 57.265 R^{\frac{1}{6}} S^{\frac{1}{2}}$ which also shows that the value of $\sqrt{f}$ is governed by the section and slope of a channel.

Taking Chezy's formula $V = C\sqrt{RS}$ and equating it to $V = 1.17\sqrt{f R}$ we get $\sqrt{f} = 0.854\, C\sqrt{S}$. Therefore, Lacey's $\sqrt{f}$ is subject to all the variations of the coefficient C which Chezy's formula is subject to. Although C and $\sqrt{S}$, when multplied, give a figure below or above unity, that figure is not an exclusive silt index but one connoting jointly C and $\sqrt{S}$.

Table No. 4 of values of $C\sqrt{S}$ in the formula $V = C\sqrt{RS}$ when C is Kutter's expression is attached for different values of R and slopes. It would be noticed that the range of variations is somewhat from 0.5 to 3.0, which is not all due to the silt load.

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## STATEMENT NO. 2

VALUES OF C.V.R. AND /f FOR VARIOUS DISCHARGES LACE
KENNEDY'S DIAGR

| Discharge | 1/10,000 | 1/8,000 | 1/6,666 | 1/5,714 | 1/5,000 | 1/4,444 |
|---|---|---|---|---|---|---|
| 10 C.V.R. / $\sqrt{f}$ | .435 / .53 | .556 / .63 | .59 / .66 | .70 / .73 | .76 / .75 | .79 / .76 |
| 20 | .552 / .65 | .63 / .66 | .68 / .69 | .74 / .73 | .82 / .78 | .92 / .92 |
| 30 | .583 / .61 | .662 / .69 | .722 / .71 | .82 / .87 | .89 / .84 | .96 / .89 |
| 40 | .59 / .61 | .682 / .68 | .755 / .73 | .83 / .79 | .92 / .86 | 1.00 / .92 |
| 50 | .63 / .65 | .703 / .69 | .781 / .76 | .87 / .84 | .91 / .86 | 1.00 / .90 |
| 70 | .62 / .64 | .720 / .67 | .802 / .78 | .87 / .83 | .87 / .83 | 1.01 / .90 |
| 80 | .636 / .65 | .720 / .71 | .803 / .76 | .87 / .83 | .960 / .89 | 1.03 / .94 |
| 100 C.V.R. / $\sqrt{f}$ | .673 / .67 | .747 / .73 | .816 / .78 | .89 / .83 | .987 / .91 | 1.07 / .98 |
| 200 | .70 / .69 | .781 / .76 | .858 / .04 | .92 / .88 | 1.01 / .94 | 1.073 / .98 |
| 300 | .734 / .75 | .81 / .79 | .885 / .86 | .94 / .89 | 1.04 / .97 | 1.09 / 1.01 |
| 400 | .75 / .74 | .827 / .80 | .894 / .86 | .97 / .92 | 1.053 / .99 | 1.13 / 1.05 |
| 500 | .74 / .74 | .838 / .82 | .91 / .88 | .. / 1.00 | 1.06 / 1.00 | 1.13 / 1.05 |
| 600 | .756 / .75 | .842 / .82 | .935 / .90 | .. | 1.08 / 1.02 | .. |
| 700 | .751 / .75 | .854 / .83 | .937 / .90 | .98 / 1.06 | 1.085 / 1.02 | 1.14 / 1.07 |
| 900 | .791 / .79 | .864 / .87 | .955 / .93 | 1.04 / 1.02 | 1.097 / 1.06 | 1.16 / 1.11 |
| 2,000 C.V.R. / $\sqrt{f}$ | .79 / .79 | .89 / .88 | .961 / .94 | 1.04 / 1.02 | 1.10 / 1.07 | 1.16 / 1.22 |
| 3,000 | .81 / .83 | .89 / .95 | .983 / .98 | 1.05 / 1.04 | 1.11 / 1.09 | 1.18 / 1.15 |
| 4,000 | .80 / .83 | .89 | .98 / .98 | 1.04 / 1.04 | 1.12 / 1.11 | 1.19 / 1.16 |
| 5,000 | .81 / .83 | .896 / .83 | .985 / .92 | 1.05 / 1.06 | 1.13 / 1.13 | 1.20 / 1.18 |
| 6,000 | .819 / .85 | .905 / .93 | .98 / 1.00 | 1.06 / 1.07 | 1.13 / 1.13 | [illegible] / [illegible] |
| 7,000 | .819 / .85 | .91 / .95 | 1.00 / 1.01 | 1.06 / 1.07 | 1.13 / 1.14 | [illegible] / [illegible] |
| 10,000 | .818 | .90 | .994 | 1.054 | 1.147 | [illegible] |

## STATEMENT NO. 3
PREPARED FROM PLATE II OF THE U. P. TECHNICAL PAPER NO. 1

**SLOPE**

| Discharge | 1/10,000 | 1/9,000 | 1/6,666 | 1/5,714 | 1/5,000 | 1/4,444 | 1/4,000 | 1/3,636 | 1/3,333 | 1/2,857 |
|---|---|---|---|---|---|---|---|---|---|---|
| | f | f | f | f | f | | | f | | f |
| 10 | .67 | .71 | .75 | .82 | .88 | .. | .. | .92 | .. | 1.01 |
| 20 | .69 | .73 | .79 | .85 | .91 | .. | .. | .96 | .. | 1.05 |
| 30 | .711 | .75 | .80 | .87 | .93 | .. | .. | .98 | .. | 1.07 |
| 40 | .72 | .75 | .81 | .88 | .94 | .. | .. | 1.0 | .. | 1.09 |
| 50 | .73 | .77 | .82 | .89 | .95 | .. | .. | 1.01 | .. | 1.10 |
| 60 | .73 | .87 | .82 | .91 | .96 | .. | .. | 1.02 | .. | 1.11 |
| 80 | .75 | .89 | .04 | .92 | .97 | .. | .. | 1.03 | .. | 1.13 |
| 100 | .75 | .80 | .85 | .93 | .98 | .. | .. | 1.04 | .. | 1.14 |
| 200 | .78 | .83 | .88 | .96 | 1.02 | .. | .. | 1.08 | .. | 1.15 |
| 300 | .80 | .85 | .90 | .98 | 1.04 | .. | .. | 1.10 | .. | 1.20 |
| | | | | | 1.06 | | | | | |
| 400 | .81 | .86 | .91 | .99 | 1.07 | .. | .. | 1.12 | .. | 1.22 |
| 500 | .81 | .87 | .92 | 1.01 | 1.08 | .. | .. | 1.13 | .. | 1.23 |
| 600 | .82 | .88 | .93 | 1.01 | 1.09 | .. | .. | 1.14 | .. | 1.24 |
| 700 | .83 | .88 | .93 | 1.02 | 1.11 | .. | .. | 1.15 | .. | 1.25 |
| 1,000 | .85 | .91 | .95 | 1.04 | 1.13 | .. | .. | 1.17 | .. | .. |
| 1,500 | .86 | .92 | .97 | 1.06 | 1.15 | .. | .. | 1.20 | .. | .. |
| 2,000 | .87 | .99 | .99 | 1.05 | 1.17 | .. | .. | 1.22 | .. | .. |
| 3,000 | .89 | .. | 1.01 | 1.00 | 1.19 | .. | .. | 1.24 | .. | .. |
| 4,000 | .91 | .. | 1.02 | 1.11 | 1.20 | .. | .. | 1.26 | .. | .. |
| 5,000 | .92 | .. | 1.03 | 1.12 | 1.21 | .. | .. | 1.27 | .. | .. |
| 6,000 | .93 | .. | 1.04 | 1.13 | 1.22 | .. | .. | .. | .. | .. |
| 7,000 | .93 | .. | 1.05 | 1.14 | 1.23 | .. | .. | .. | .. | .. |
| 8,000 | .94 | .. | 1.06 | 1.15 | 1.24 | .. | .. | .. | .. | .. |
| 9,000 | .94 | .. | 1.06 | 1.15 | 1.24 | .. | .. | .. | .. | .. |
| 10,000 | .95 | .. | 1.07 | 1.16 | .. | .. | .. | .. | .. | .. |

## STATEMENT NO. 4

### TABLE OF VALUE OF $C\sqrt{S}$ IN CHEZY'S FORMULA $V=C\sqrt{RS}$ WHEN C IS KUTTER'S EXPRESSION

| R | $\sqrt{R}$ | 1/10,000 | 1/8,000 | 1/6,666 | 1/5,714 | 1/5,000 | 1/4,444 | 1/4,000 | 1/3,636 | 1/3333 | 1/2,857 | 1/2,500 | 1/2,000 | 1/1,000 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| .5 | .7 | .463 | .529 | .583 | .594 | .688 | .735 | .775 | .816 | .861 | .936 | 1.010 | 1.98 | 1.624 |
| 1 | 1.00 | .584 | .663 | .728 | .738 | .853 | .909 | .954 | 1.006 | 1.057 | 1.150 | 1.236 | 1.461 | 1.975 |
| 2 | 1.41 | .715 | .800 | .877 | .883 | 1.01 | 1.083 | 1.135 | 1.192 | 1.252 | 1.357 | 1.456 | 1.710 | 2.319 |
| 3 | 1.73 | .800 | .863 | .965 | .969 | 1.11 | 1.183 | 1.238 | 1.303 | 1.364 | 1.479 | 1.584 | 1.869 | 2.477 |
| 4 | 2.00 | .847 | .940 | 1.01 | 1.02 | 1.18 | 1.252 | 1.310 | 1.376 | 1.442 | 1.563 | 1.672 | 1.968 | 2.638 |
| 5 | 2.24 | .895 | .984 | 1.06 | 1.07 | 1.23 | 1.306 | 1.365 | 1.433 | 1.467 | 1.625 | 1.736 | 2.044 | 2.093 |
| 6 | 2.45 | .925 | 1.02 | 1.10 | 1.11 | 1.26 | 1.35 | 1.403 | 1.476 | 1.546 | 1.668 | 1.788 | 2.103 | 2.033 |
| 7 | 2.65 | .955 | 1.04 | 1.13 | 1.14 | 1.30 | 1.395 | 1.444 | 1.514 | 1.586 | 1.712 | 1.830 | 2.052 | 2.859 |
| 8 | 2.83 | .970 | 1.07 | 1.16 | 1.16 | 1.33 | 1.410 | 1.472 | 1.544 | 1.617 | 1.744 | 1.866 | 2.092 | 2.913 |
| 9 | 3.00 | .985 | 1.09 | 1.18 | 1.18 | 1.35 | 1.434 | 1.499 | 1.570 | 1.643 | 1.774 | 1.894 | 2.225 | 2.954 |
| 10 | 3.16 | 1.000 | 1.11 | 1.20 | 1.20 | 1.37 | 1.455 | 1.519 | 1.592 | 1.667 | 1.798 | 1.920 | 2.256 | 3.235 |
| 11 | 3.32 | 1.022 | 1.12 | 1.21 | 1.22 | 1.39 | 1.476 | 1.540 | 1.613 | 1.688 | 1.823 | 1.944 | 2.284 | 3.039 |
| 12 | 3.464 | 1.041 | 1.14 | 1.23 | 1.23 | 1.41 | 1.494 | 1.557 | 1.631 | 1.707 | 1.841 | 1.962 | 2.306 | 3.063 |

## CHAPTER XII

## THIS C. V. R. AND $\sqrt{f}$ !

Looking on the beds of some dry canals, main and branches a even some distributaries, the signs of rolling silts are vividly visil By designing channels with higher values of C. V. R. or $\sqrt{f}$ to be fi arbitrarily in their upper reaches under the advice of Kennedy a Lacey, it has been the practice to wash down silt further bel until it becomes impossible to do so any more and, therefore, it m be cleared out.

Huge sums of money spent annually all over the canals on t item, although staggering, are not the only consequences of this har ful practice, but most of our troubles due to the falling of berms a turning once regime channels out of regime again with the coming rolling silt and thereby causing their berms to fall are the very resı of that practice. Channels get wide and shallow and their wh hydraulics are changed.

Many engineers would have seen the effects of rolling silt in acti on the Lower Chenab Canal. Due to heavy movements of this si water surface levels rose in places not by inches but by feet and some places caused breaches. In places the water surface becan higher than even the bank levels. Consequently thousands a thousands of rupees were spent on raising the banks to a safer level.

On a lower scale the baneful effects of this practice can be see by any one on the Lahore Branch Upstream of Lahore Rest Hous The lovely shrubs with pretty flowers once centrally situated on t beautiful grass berms of this canal are now in course of being unde mined and their very existence is threatened.

But why should this occur ? It is simply because large quantiti of coarse silt, which are made to be washed down from above, hav come into it and unless the silt is removed this process of falling bern will continue.

Any coarse sand or silt which cannot serve as a fertilizer fo agricultural lands can be compared to a snake.

"Don't allow it to come near you. Kill it as you see it otherwis this snake is likely to do mischief."

Silt-ejectors should be encouraged and silting tanks made more commonly. The strengthening and raising of banks should all possibly be done with silt from bed or by digging below the bed as this soon gets filled up.

The only reason generally advanced for the continuance of this practice in spite of the ravages it causes, with which most of the irrigation engineers are familiar, is that it is necessary to do so for keeping the channels in working order to avoid their head reaches getting solidly silted up. But, if one is permitted to say frankly, it is simply to throw one's own burdens on another man's shoulders.

If the silts are not to be washed down, it is true that there will be more silt deposits in the head reaches. The answer to this is that wherever the silt is found, tackle it *there and then*—why let it go down to do mischief?

Head reaches can be provided with silt-ejectors and even bye-passes.

It will be found that, in the long run, money spent in the head reaches will only be a fraction of what is now spent all over on the silt clearance of channels and forced raising and strengthening of banks.

# CHAPTER XIII

## LACEY'S AND KENNEDY'S DIAGRAMS COMPARED

*Some Examples.*—The taste of the pudding lies in its eating. Therefore, reverting from the domain of academic discussion to the practical side, it is proposed to place before the reader some examples of the design worked out from Lacey's Diagrams published in U. P. Technical Paper No. 1 and also from Improved Kennedy's Hydraulic Diagrams for discharges of 10, 50, 150, 500, 1,000, 2,000, 5,000 and 10,000 cusecs for different slopes (see Statement No. 5).

In case of design from Lacey's Diagrams, the method adopted is that for a given slope and a given discharge the value of "f" has been determined from his Regime Slope Diagrams Plate II. With this determined value of "f" the dimensions—bed-width and depth taken from his Plates Ia and Ib, assuming side slopes to be $\frac{1}{2}$ to 1 worked out and velocities have been calculated as $\frac{Q}{A}$.

It may be remarked that Lacey's method of design is different. Lacey and his followers fix up the values of "f" arbitrarily and then proceed to design the channel. The channel in that case can be designed from Plates Ia and Ib but it must be remembered that for a particular discharge and a particular value of "f" there is always one slope and only one which is given by his Regime Slope Diagrams Plate II. For other slopes there are different values of "f." As it is practically impossible to assign only one particular slope and no other for a given discharge in all localities, the idea of keeping one "f" for the whole system of a canal is untenable and impossible to achieve in design and in practice.

Examples from Lacey's Diagrams have been worked out for different values of slopes for different discharges and values of "f" taken. It will be seen that the values of $\sqrt{f}$ in the statement for Lacey's designs rise with the steepening of the slope and also with the increase of discharge.

For design from Improved Kennedy's Hydraulic Diagrams for any particular slope, one has only to glance where the line for discharge intersects Lacey's Perimeter Curve $P = 2.67\sqrt{Q}$. Opposite point of intersection read B and D and see the value of V, C.V.R., $\sqrt{f}$ and $\sqrt{R}$.

On the diagrams are plotted also two other curves $P = 2.8\sqrt{Q}$ and $P = 2.5\sqrt{Q}$. This is the restricting zone of selection for varying rugosities

It will be noticed that designs from Lacey's and Kennedy's curves for $P=2.67\sqrt{Q}$ have almost the same B but there is little difference in D. Practically they are almost the same. In Kennedy's there is a zone of selection based on our conception of rugosity—a discretion which could not be much amiss.

According to the writer, "f" is a variable and could not and should not form a basis of design as its value is different for any conceivable different discharge and different slopes. It is unsound to assign arbitrary values for "f" for practical design work. Lacey's Diagrams are no substitute for Improved Kennedy's Diagrams.

Kennedy's Hydraulic Diagrams could be improved still further to cover a higher range of discharges and embody a more flexible range of slope. But even in doing so, it would be honest to keep intact the identity of their great author who gave us these lines in print.

For the same discharges, designs have also been worked from the Punjab Research Institute Formula $V=1.12\sqrt{R}$, $P=2.8\sqrt{Q}$ and $R=.47\ Q^{\frac{1}{3}}$. These formulæ give the design for one particular slope and it would be for the Punjab Research Officers to state for what slope their designs should be taken for, keeping in view their slope formula $S=\dfrac{2.09\ m^{0.86}}{1{,}000\ Q^{\cdot 21}}$ because the dimensional design is complete without even referring to any consideration of slope. Is there any claim that these dimensions will fit in with any slope that may actually be met with or be arrived at with their slope formula $S=\dfrac{2.09\ m^{0.86}}{1{,}000\ Q^{\cdot 21}}$?

Suppose for several localities the value of m is actually found different for a fixed discharge, naturally the formula will give different values of S. Then for those different values of S should there be the same R, P and V which are to be got from the Institute's formulæ?

## CHAPTER XIV

## SLOPE FOR CHANNELS

For the design of an irrigation channel for a given discharge, there are no hard and fast rules by which the slope to be given can be precisely determined. Hitherto the practice has been to fix the slope arbitrarily or through the exigencies of the levels available and then work out the dimensions of the required channel for the given discharge under that slope from some diagrams—may be Kennedy's or Garret's.

Experience showed that slopes fixed so arbitrarily did not obtain actually in some channels while in others they were found quite near the mark.

Observations tended to point out that by the entry of coarse grades of silt into channels, the slope steepened and, where fine grades of silt remained intact, the slope also remained unaffected. But no direct relationship between the two got established.

It devolved on Lacey to put forward the relationship in some tangible form in the shape of his slope formula $S = \frac{f^{5/3}}{1,788\ Q^{\frac{1}{6}}}$. The idea is commendable but the difficulty is that the value of his "f" is different for any different discharge and for different slopes. Therefore, for practical determination of slope, it does not help much.

Lacey has given a rough rule that "f" $= Cd^n$ where d is the diameter of silt particle.

This is very difficult to measure accurately the diameter of the average particle of silt. For practical purposes to do so is difficult.

To give this idea a precise form, the Punjab Irrigation Research Institute has produced slope formula in which S has been connected with Q and m, where m is the mean diameter of the silt particle. The formula is $S = \frac{2.09\ m^{\cdot 86}}{1,000\ Q^{\cdot 21}}$.

When means are available to measure the diameter of silt particles and weigh them accurately, this may be found useful, such as for remodelling of existing channels.

But when new channels are to be constructed with or without silt selective head regulators, no one could say with precision which grade of silt will be actually flowing at a particular site. So, for the determination of slope, something more handy is needed.

Some time ago, the writer's daughter, Miss Shanta, B.A., arrived at the relationship $S_{10}{}^{3} = \frac{Q}{30\ R^{10/3}}$ (nomogram attached) when she was asked by him to work out algebraic equations of Manning's velocity formula with Lacey's. This formula was brought to the notice of the Punjab Engineering Congress in its 1940 Session. Since its general form is worked out $S_{10}{}^{3} = \frac{65.838\ N^2Q}{R^{10/3}}$

Then came to his notice the Punjab Irrigation Research Institute formula $R = .47\ Q^{\frac{1}{3}}$. With this S could be directly connected with R or Q. This has been done in next chapter in which some more relationships have been arrived at purely from Lacey's formulæ or otherwise.

With the resulting formulæ actual values of slope per thousand feet have been worked out and tabulated as per Statements 6 and 7, which could be a good guide for actual design of channels, new as well as old.

It will be easy to remember these thumb-rule formulæ:

(1) $S_{10}{}^{3} = \frac{66\ N^{2}\ Q}{R^{10/3}}$

(2) $S_{10}{}^{3} = \frac{80\ N_a{}^{2}\ Q}{R^{7/2}}$

(3) $S_{10}{}^{3} = \frac{1}{1.8\ Q^{1/6}}$

(4) $S_{10}{}^{3} = \frac{1}{2.6\ R^{\frac{1}{2}}}$

In presenting these relationships, a very high standard of precision is not claimed. What is claimed is the simplicity and directness of their forms.

The first two only give sufficient flexibility in the way of different values of N or $N_a$ while the last two are quite good averages which are commonly to be found all over. All of these could safely be adopted for practical design work.

It will be useful to consult Statements 6 and 7 (referred to above) of the calculated values of $S_{10}{}^{3}$ for different discharges. These will offer an instructive study for adopting a suitable slope.

By plotting regime data of the channels published by the Punjab Irrigation Research Institute as well as by Lacey it is found that the relation between S and R or between S and Q is not very rigid. The exponents of R as well as of Q have a tendency to change to the lower values when the R or Q increase in value. Therefore the flexibility of the constant in the first two formulæ afford accommodation.

Later it may be possible to establish the relation of S and R or of S and Q as the minor and major axis of an ellipse or a parabola but this idea should not stand in our way in accepting the above stated formulæ as good thumb-rule guides for fixing slopes in practical design work.

## CHAPTER XV

Lacey's main slope formula is :

$$S = \frac{f^{5/3}}{1844.3\ Q^{1/6}}$$

His other formulæ in which S—slope appears are :

$$V = 16.05\ R^{\frac{2}{3}}\ S^{\frac{1}{3}}$$

$$V = \frac{1.3458}{N_a} R^{\frac{3}{4}}\ S^{\frac{1}{2}}$$

Using Manning's formula—

$$V = \frac{1.4858}{N} R^{\frac{2}{3}}\ S^{\frac{1}{2}}$$

and some other formulæ from Lacey, the following six relationships are derived (calculations appended) in which S is connected with Q, R, Kutter's N and Lacey's $N_a$ and then finally related directly to Q as well as to R.

### I. Shanta's Slope Formula :

$$S_{10}{}^{3} = \frac{65.838\ N^2}{R^{10/3}}\ Q \text{ (Manning-}\textit{cum}\text{-Lacey).}$$

This general formula for different values of Kutter's N works out as under :

(*i*) $N = \cdot 018 = S_{10}{}^{3} = \dfrac{Q}{47\ R^{10/3}} = \dfrac{1}{3\cdot 786\ Q^{1/9}}$

(*ii*) $N = 0\cdot 020$ ,, $= \dfrac{Q}{38\ R^{10/3}} = \dfrac{1}{3\cdot 06\ Q^{1/9}}$

(*iii*) $N = \cdot 0225$ ,, $= \dfrac{Q}{30\ R^{10/3}} = \dfrac{1}{2\cdot 42\ Q^{1/9}}$

(*iv*) $N = 0\cdot 025$ ,, $= \dfrac{Q}{24\ R^{10/3}} = \dfrac{1}{1\cdot 93\ Q^{1/9}}$

(*v*) $N = 0\cdot 030$ ,, $= \dfrac{Q}{17\ R^{10/3}} = \dfrac{1}{1\cdot 37\ Q^{1/9}}$

II.—$S_{10}3 = \dfrac{80\ N_a{}^2\ Q}{R^{7/2}}$ (*Purely from Lacey's*)

This general formula for different values of Lacey's $Na^2$ works as under :

$N_a = \cdot 018 \quad S_{10}{}^{3} = \dfrac{Q}{38\cdot 5\ R^{7/2}} = \dfrac{1}{2\cdot 74 Q^{1/6}}$

$$N_a = .020 \quad S_{10}{}^3 = \frac{Q}{31{\cdot}25\,R^{7/2}} = \frac{1}{2{\cdot}23\,Q^{1/6}}$$

$$N_a = {\cdot}0225 \quad ,, \quad = \frac{Q}{24{\cdot}7\,R^{7/2}} = \frac{1}{1.75\,Q^{1/6}}$$

$$N_a = .025 \quad ,, \quad = \frac{Q}{20\,R^{7/2}} = \frac{1}{1.42\,Q^{1/6}}$$

$$N_a = .030 \quad ,, \quad = \frac{Q}{11\,R^{7/2}} = \frac{1}{1.0\,Q^{1/6}}$$

Following are from Lacey where N or $N_a$ does not come in :

$$\text{III.}—S_{10}{}^3 = \frac{Q^{3/2}}{80\,R^5} = \frac{1}{1.83\,Q^{1/6}}$$

$$\text{IV.}—S_{10}{}^3 = \frac{Q^{3/2}}{75\,R^5} = \frac{1}{1.72\,Q^{1/6}}$$

$$\text{V.}—S_{10}{}^3 = \frac{Q^{3/2}}{71.21\,R^5} = \frac{1}{1.63\,Q^{1/6}}$$

Relationships at the end column are arrived at by substituting for R from $R = .47\,Q^{\frac{1}{3}}$

$$\text{VI.}—S_{10}{}^3 = \frac{1}{2.000\,Q^{1/6}} = \frac{1}{2.0\,Q^{1/6}}$$

(Bose-*cum*-Lacey.)

Q in the same six relationships has been converted into R and direct relation of S and R established.

$$S_{10}{}^3 = \frac{65{\cdot}838\,N^2\,Q}{R^{10/3}}$$

$$\text{When } N = {\cdot}018 \; S_{10}{}^3 = \frac{Q}{47R^{10/3}} = \frac{\dfrac{R^3}{{\cdot}47^3}}{47R^{10/3}} = \frac{1}{4{\cdot}89\,R^{\frac{1}{3}}}$$

$$,, \quad N = {\cdot}020 \; S_{10}{}^3 = \frac{Q}{38R^{10/3}} = \frac{Q}{30R^{10/3}} = \frac{1}{3{\cdot}11\,R^{\frac{1}{3}}} = \frac{1}{3{\cdot}95\,R^{\frac{1}{3}}}$$

$$,, \quad N = {\cdot}025 \; S_{10}{}^3 = \frac{Q}{24\,R^{10/3}} = \frac{1}{2{\cdot}49\,R^{\frac{1}{3}}}$$

$$,, \quad N = {\cdot}030 \; S_{10}{}^3 = \frac{Q}{17\,R^{10/3}} = \frac{1}{1{\cdot}77\,R^{\frac{1}{3}}}$$

$$\text{II.}—S_{10} = \frac{80\,N_a^2 Q}{R^{7/2}}$$