DESIGN OF STABLE CHANNELS IN ALLUVIAL SOILS

By

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C O N T E N T S

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SYMBOLS

No.	Symbol	Definition	Dimensions
1.	A	Area	L^2
2.	B	Bed width	L
3.	C, c	Coefficients	-
4.	D	Depth of the channel	L
5.	d	Diameter of particle	L
6.	F	Force	F
7.	f	Silt factor	-
8.	K	A numerical constant	-
9.	L	Length	L
10.	m	An exponent	-
11.	N	Coefficient of roughness	-
12.	P	Wetted perimeter	L
13.	Q	Discharge in cfs	L^3/T
14.	q	Discharge per unit width	L^3/T
15.	q_c	Critical discharge at which sediment movement starts.	L^3/T
16.	R	Hydraulic mean depth	L
17.	S	Slope or energy gradient	-
18.	S_s	Specific gravity of sediment	-
19.	V	Velocity of flow	L/T
20.	V_o	Critical velocity	L/T
21.	v	Velocity of fall of particles	L/T
22.	W	Weight of a certain volume of material	F
23.	w	Unit weight of water in lbs/cft	F/L^3

24.	$\alpha_1, \alpha_2, \alpha_3$	Constants	-
25.	α	Side slope	-
26.	μ	Viscosity	FT/L^2
27.	ν	Kinematic viscosity	L^2/T
28.	τ	Tractive force per unit area	F/L^2
29.	τ_c	Critical tractive force	F/L^2
30.	ρ	Density of fluid	FT^2/L^2
31.	β	Constant	-
32.	ϕ	Angle of repose in degrees	-
33.	ϕ_1, ϕ_2	Constants	-

Note:-

In addition to the above list, a few symbols, not presented above, have been explained whereever they occur.

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INTRODUCTION

Irrigation is an age old art. When the intensity and distribution of rainfall is such that it is not possible to bring the crops to maturity, water has to be supplied by artificial channels drawn from some source such as a well or a perinial river or a lake. Channels are excavated to the required cross section and bed slope so as to carry the necessary quantity of water for crops.

It is rather difficult to give the exact date from which irrigation started. Civilization developed largely in fertile plains and disintegrated after some time, due to various reasons, one of them being scarcity of water, till people found some method of artificial supply of water by channels from some river nearby. In India, irrigation is being practised since ancient times. The Cauvery Delta system was built by the Tanjore kings in South India during 200 A.D., and it is remarkable that this system remained functioning upto early nineteenth centuary when re-modelling was done. During the Moghul period the waters of Yamuna were harnessed to supplement the crop needs in the fertile Punjab lands. Similarly in other countries like Egypt, Mesopotamia, Spain and Rome irrigation canals existed in olden days. All this shows that the ancients had some knowledge of the laws of flow of water on which they based the design and alignment of channels.

Since the last centuary,considerable advances have been made in the design of channels as this subject has been engaging the attention of hydraulicians and irrigation engineers. From the hydraulic stand-point,the problem of channel design involves two basic aspects; one the discharging capacity of the channel and the other is the prevention of deposition or erosin of the bed and banks of channel. A stable channel can be defined as one which has the required discharging capacity to irrigate the estimated area and which neither scours nor silts during its routine flow. In other words the cross-sectional area of a stabbe channel should be sufficient enough to carry the required quantity of water and its longitudinal slope should be such that it gives the water a velocity at which neither deposition of silt occurs nor the erosin of bed material takes place.

There are two different approaches to the problem of stable design of channels,one is the theoritical and the other is ~~emx~~ emperical. In the former approach research has been done and is being done by mathematicians and hydraulicians to find out the physical forces which cause the solid particles to rise from bed and remain in suspension while the water flows. In the emperical approach attempts have been made by irrigation engineers of India,Egypt and other countries to find out some emperical formulae for the design of channels. The formulae evolved by these engineers are the result of their long experience of canal maintenance and construction and these formulae are being extensively used in practice.

It is interesting to note that though the rational approach aims at a complete and exhaustive solution to the problem it has not yet been able to provide a definate design procedure. Although several solutions have been given by the research workers in this field based on laboratory experiments and some field observations,none of them has achieved a universal recognition and practical application. On the other hand the emperical approach has provided a certain procedure of designing canals to the practical engineer,but these emperical methods lack a mathematical and rational proof. These emperical formulae are very widely used in India,Egypt and some other countries,but the American engineers believe that they are applicable only under certain conditions and hence cannot be universally accepted unless a rational proof is given.

Hence the design of stable channels has to be studied in the light of both these approaches because both deserve equal attention.

Objective:-

The major objective of the studies reported in this thesis is to make an exhaustive and critical study of the literature available on the subject and to present it in a more simplified manner so as to clarify the sound elements of progress and make them more easily understandable to engineers engaged in irrigation works. Attempt is also made to suggest further line of action for doing experimental research on the subject.

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GENERAL REVIEW OF THE LITERATURE

Since the earliest times engineers have experienced difficulties due to sediment carried by the streams and channels. Deposition of sediment raises the stream bed, thereby inreasing the flood levels and inundations, it piles up sediment in huge quantities behind dams, thereby reducing reservoir capacities and their function; it causes the rivers to meander and often leave their original course of flow, thus devastating vast areas of excellent land and it creates a variety of other problems for the irrigation engineer. A regular and scientific study of this phenomanon has started very recently, but it will be interesting to review the work done in the past on the subject.

The Chinese were probably the first to study the silt problems in relation to floods in Yellow River. Pan C.H., a river expert in the Ming Dynasty studied the river problems for about thirty years. He observed that a proper way to remove silt deposited in the bed of a river was to concentrate the flow of water in the silted portion.

In the seventeenth centuary, extensive investigations were carried out in Italy. Guglielmini and Frizi were the two pioneers in this field. They stated their rules of sediment transportation, in terms of discharge and slope. They observe that as the streams approached the sea their slopes became flatter and that the size of sediment became smaller. Guglielmini concluded that the sediment became smaller in the course of flow because the larger stones and boulders in the mountain torrents were fractured and worn down during their travel to the sea until they became sand which existed at the river mouth. He gave two principles; the greater the quantity of water a river carries the less will be its flow.

In ~~xxx~~ 1775 Chezy, a French engineer derived a formula for flow of water from the consideration of the resistance of channels to flow which in a simplified form is expressed as

$$V = C\sqrt{RS} \quad \text{---------} \quad 1.$$

from the limited data available at that time Chezy believed C to be a constant. Later investigations, however, showed that C was not a constant, but varies with the characterstics of the channel. This was a very important advance in the field of channel design and even today the Chezy's formula is extensively used.

$$V = C\sqrt{RS} \quad \cdots\cdots\cdots \quad 1.$$

Many engineers started working to find out the value of C in the Chezy's formula. Darcey and Bazin in France, Humphrey and Abbot in America and many others made valuable research for finding a correct formula to ascertain a correct value of C. The formula presented by Ganguillet and Kutter, the two Swiss engineers deserves ~~xxxx~~ consideration as it can be applied under varying conditions of flow. They evolved the following formula in 1870.

$$C = \frac{41{\cdot}66 + \frac{0{\cdot}00281}{S} + \frac{1{\cdot}811}{N}}{1 + \left(41{\cdot}66 + \frac{0{\cdot}00281}{S}\right) N/\sqrt{R}} \quad . . . . 2$$

In this formula N is the roughness factor, generally known as the rugosity coefficient which can be given various values for various grades of roughness of channels. This formula is slightly complicated to evaluate but the author's claim to have derived it after collecting all the correct data of velocities, discharges hydraulic mean depth and surface slopes etc of the natural and artificial channels available in those days.

In 1890 Mannings gave his formula for C in the Chezy's formula as

$$C = \frac{1{\cdot}4858}{N} R^{1/6} \quad . . . . 3$$

where N has the same values as that in Kutter's formula. From this the velocity formula can be conveniently written as

$$V = \frac{1{\cdot}4858}{N} R^{2/3} S^{1/2} \quad . . . . . 4.$$

The Mannings formula avoids all laborious calculations of the Kutter's formula.

Bazin published his formula for the constant C in 1897 with the assumption that C depends on R and is independent of slope.

$$C = \frac{157{\cdot}6}{1 + m/\sqrt{R}} \quad . . . . 5$$

where m is again a coefficient of roughness in the values are given in table 2.

These formulae have been extensively used in the design of channels but they are deficient as they ane not having any reference to silt grade or silt charge which the water carries in earth-

Table 1

Values of N in the Kutters and Manning Formulae

Nature of Surface	Range in N	Commonly used.
Canals		
Earth, straight and uniform	0.017 – 0.025	0.0225
Rock cuts, smooth and uniform	0.025 – 0.035	0.033
Rock cuts, jagged and irregular	0.035 – 0.045	
Dredged in earth	0.025 – 0.033	0.0275
Earth bottom rubble sides	0.028 – 0.035	0.032
Natural Streams		
Clean straight and uniform	0.025 – 0.033	0.02
" " with weeds	0.03 to 0.04	
Winding with pools and shoals	0.033 – 0.045	

Table 2.

Values of m in the Bazin's formula

For very smooth cement surfaces	0.109
For well laid brick or concrete	0.290
Rubble masonry or poor brickwork	0.833
For earthen bed in perfect condition	1.54
For earth bed in ordinary condition	2.35
For earth bed in very bad condition	3.17

ern irrigation channels. The result was that the channels designed on these formulae silted very badly and large amounts of money had to be spent every year for silt clearance.

Towards the theortical design the earliest quantitative studies were made by Dubaut in 1786 to determine the velocities of water which would cause the grannular material in open channels to move. He observed that a velocity of one foot per second was enough to produce sand waves in a bottom where grains were large enough to be easily visible.

Dupuit stated that the transportation of sediment in suspension was due to excess of velocity on the upper side of the particle as compared with that of the lower side. His idea was that an object floating in a stream travels faster than water sorrounding it and therefore it would tend to go towards the faster moving water. Since the velocities in a stream ordinarily decrease from the surface towards the bottom, the particle in moving towards the region of higher velocities would tend to move upwards. He also observed that the concentration of sediment at the bottom was greater than that near the surface.

In 1879 DuBoys presented his drag theory which has been widely accepted as the basis of bed load movement. The advantage of DuBoys drag principle was that it furnished an elementary approach to the problem of bed load movement. In brief his observations can be stated as that there is a critical tractive force for each kind of material at which the movement just starts. The magnitude of critical tractive force depends upon the size of material, this being larger for larger sizes and smaller for smaller sizes. For the tractive force less than the critical no movement takes place and the amount of material moved would be proportional to the excess of the tractive force acting over the critical value.

In India, the studies of stable channel design developed because of the difficulties encountered in the large irrigation canals as a result of sediment deposits. The large rivers from the Himalayas spread over the entire northern India and provide facilities of irrigation. Since the second half of the last centuary extensive irrigation development started under the guidance of British engineers. The waters coming from mountains are heavily charged with sediment and in some of the projects great expenditure is involved because of the silting of resevoirs and canals.

In 1895 the first study of non-silting canals sections was made by Kennedy, then Executive Engineer on the Upper Bari Doab canals, Punjab. He selected over thirty reaches of the channels in this canal system which were stable and were niether silting nor scouring. He stated that for every discharge, there exists a velocity in the channel at which there is niether silting nor scouring which he termed as critical velocity. He gave a formula relating velocity and depth of the channel. He recognised that the grade of sand played an important part in the relationship and regarded the silt of Upper Bari Doab canals as the standard. For other places he introduced a factor known as the critical velocity ratio which is the ratio of the actual velocity and the Kennedy's critical velocity. Kennedy's formula was a significant advance in the design of channels and all irrigation channels were designed with the critical velocity in view. He also printed hydraulic diagrams which were extensively used in northern parts of India. Although Kennedy's work is very useful and provided some basis of design it suffers from many draw-backs, which we shall take up later in the following chapters.

Kennedy should be considered as a pioneer with regard to the emperical channel design as he provided with a definate method of approach and we see that during the same period many irrigation engineers in India started thinking in terms of checking the Kennedy's equation for their own regions. We have the work of Garrett, Woods, Lindley followed by Lacey. In 1913 Garrett published a set of hydraulic diagrams for the design of channels which were widely used for designing channels.

In 1917 Woods found that by Kennedy's diagrams many alternative designs were possible for one critical velocity which put the practical engineer in a fix as to use which combination of bed width and depth. He analysed the data of many existing canals and suggested a table of ratios of bed width and depth. Later in 1927 he presented a few equations giving relations in the different elements of the channel as bed width, depth, velocity and slope.

In 1919 Lindley found that Kennedy's equation was not universally acceptable because both the constants varied from place to place. He also observed that only one equation was not enough to determine a regime channel section and hence suggested his equations giving relations in bed width and depth, velocity and depth and velocity and bed width. Lindley's work can be considered as an advance over Kennedy but it also suffers with some of the draw-backs of the Kennedy's work. But the data collected by Lindley was of great statistical importance.

In 1930 Garald Lacey presented his theory of regime channels which is a great advance in the present studies of stable channel design. According to Lacey regime flow in channels connotes physical stability, a balance between silting and scouring and a dynamic equillibrium of forces generating and maintaining the channel cross section and gredient. He stated that regime conditions will be established when a channel flows in an unlimited incoherent alluvian and when the discharge and silt grade are constant and the channel is absolutely free to move in any lateral direction. The Lacey's concept of regime is infact applicable only when the channel is neglected like a river in plains, but some of the conditions of initial regime can be obtained in the channels excavated in alluvial soils. He has given a number of equations relating the different elements of channels which require an exhaustive study. A critical study of Lacey's work has been presented in chapter IV.

The above mentioned work towards finding an emperical solution of stable channels was done in India while engineers and hydraulicians were busy in finding the rational formula for channel desigh in other countries like France, Italy and United States.

The year 1920 brought a very rapid change in the sediment transportation studies. The progress was due to the use of hydraulic models in the investigation of hydraulic structures. In hydraulic models it is easy to relate the laws of the model to that of the prototype but no such laws exist for movable bed models. A desire to obtain these laws brought about numerous laboratory experiments; among these are the studies of Schaffernack, McDougal, Kramer and Mavis. Studies on large scale models were also made by Mayor Peter, Favre and Einstein at the Swiss Federal Institute of Technology. In 1933, O'Brien applied the theories developed in studying the mixing of atmosphere to the problem of sediment transportation in suspension and points out some relations. O'Brien concluded that none of the equations of critical tractive force were sufficiently reliable for use in channel design.

About the same time Kramer conducted a series of experiments and concluded that for moderate slope the critical tractive force for a given material is nearly constant, and the movability of the material is affected by the grain size and denseness of the mixture; that the tractive force varies directly with the slope and that well graded course mixtures reduce the tendancy of excessive ripples.

Schaffernack observed that the individual grains of material showed their first sign of motion by vibrating or oscillating without actually being carried down by the water. The individual grains roll out of their positions when water acquires a bottom velocity approximately twice as that at which the vibration or oscillation becomes apparent. Grains of sand and tossed into the flowing water up-stream from the sand bed were kept in motion across the bed at a velocity less than that required to cause individual grains, initially at rest, to be moved from the sand bed. These particles tossed into the flowing water were kept in motion at a velocity approximately 30% greater than the velocity required to cause vibration. His experiments indicated that after the critical bottom velocity has been acceded the capacity is directly proportional to the square of the bottom velocity.

Extensive studies of sediment transportation in streams has been done by Dr. Straub of United States. He proposed a theory of sediment transportation based on the hypothesis that the amount of material moved is the function of the transporting force and that it is definable by the hydraulic characterstics of the stream.

This is a brief resume of the worked done on the subject both emperical and theortical upto the year 1930. In the last two decades considerable advances have been made all over. The work of Lane.E.W., and others in U.S.A., work of White C.M. in U.K. and the work of Inglis C.C., Malhotra and Bose in India have all been progressing. It is curious to note that still it has not been possible to arrive at some definate procedure of designing stable channels which could be universally accepted. The gap between the theortical and emperical work still exists which looks to be difficult to bridge.

SEDIMENT TRANSPORTATION IN OPEN CHANNELS

Sediments moved by the flowing water is classified according to the physical process by which it is moved as Contact load, Suspended load and Saltation load. Particles of sediment which are carried by water donot move in straight lines but move in various directions. The stream flow is always turbulent and the degree of turbulance depends upon the roughness of banks and bed of the channel and the velocity of flow.

Contact load is the material rolled or slid along the bed in contineous contact with the bed. Due to the turbulance of water, the velocity causing the sediment to move is not constant and therefore whenever the magnitude of this velocity is sufficient enough to exert a force on the particles to overcome the inertia motion is produced. In an experimental flume the motion of contact load can be seen very clearly.

Suspended load is the material which is moving in suspension and which is maintained in suspension by the component of upward current of turbulant water or by colloidal suspension. Strong upward currents may act on a small particle placing and maintaining it in suspension until the currents lessen and the force of gravity attracts the particle to the stream bed. The smaller particles of colloidal dimensions are moved upwards and in the general direction of flow by the impulse force applied by the molecules of water on the particles. Due to the extremely small settling rate of smaller particles, they may be struck by a number of constantly upward moving particiles of water to sustain in contineous suspension.

Saltation load is the material boucingg along the bed or moved directly or indirectly by the impact of the bouncing particles. A small amount of material in the form of particles moving near the stream bed may intermittently strike the bed and bounce upward or as a result of the impact, the striking particles may force other particles upward into the flowing water for a temporary period.

Except in very high velocities or very turbulent water, material of gravel and larger sizes move almost entirely as contact load. Under many conditions most of the sand sizes move both as contact lead and as suspended load. Each of the type of movement follows a different law but in any stream all types may be in progress at the same time. Materials moving as contact load, the saltation load and coarse part of the suspended load moving near the bottom are ordinarily named as 'bed load'.

The physical laws which govern the transportation of sediment in water have not been fully developed and therefore only a partial analysis of this phenomena is possible. The laws of bed load and suspended load movement have been evolved to a certain extent but very little has been done to quantitatively evaluate the movement of saltation load in water. When measuring sediment load carried by a stream it is not possible to separate suspended load and saltation load as they are intimately mixed together.

Movement of Bed Load

A study of the movement of non-cohesive material along the bed of a straight uniform flume subjected to the action of flowing water will show that with a certain depth of flow, when the velocity of flow increases to a certain value, some particles of sand presumably those protruding out of the bed, commence intermittent movement by rolling and sliding along the bed with further increase in the velocity of flow particles rolling and sliding along the bed increase in number. Depending upon the physical characterstics of the bed material such as size composition shape etc and with the increase in the rate of bed material movement, the original smooth bed gradually assumes an undulating form consisting of ripples with a gentle slope on the windward side and a steep slope on the leeward side.

At this point two questions enter the discussion of bed load movement. One is the law governing the commencement of the bed material movement and the other is the law relating to the rate of bed load transportation. Starting with the bed material at rest and with the increase of forces of currents acting on it the movement of sediment takes places rather gradually. The critical stage of movement is therefore defined differently by different authors as initial movement or general movement etc. A more reasonable way would be to plot the rate of transportation against tractive force and to extrapolate the rate of transportation to zero. The corresponding tractive force will give the critical stage of movement.

To evaluate the relations of bed load movement, DuBoy's theory of tractive force is very useful. For uniform flow the force of running water exerted at the bed of a channel is known as the tractive force. The tractive force is equal to the product of the specific weight of water, hydraulic mean depth and the slope.

$$\tau = \omega R S \quad \text{-------- } 6.$$

Krey observed that the resistance of sediment to motion is proportional to the diameter of the sediment particles d and the specific weight of sediment in water $\omega(S_s - 1)$. For critical tractive force Krey suggested the following relation.

$$\tau_c = C \omega (S_s - 1) d \quad \text{-------- } 7$$

Where C is a dimensionless constant. The equation could be put in this way also

$$C = \frac{\tau_c}{\omega (S_s - 1) d}$$

Shields conducted several experiments and based on his observations stated that C as suggested by Krey is not a constant but a variable. He instead suggested the following equation.

$$\frac{\tau}{\omega (S_s - 1) d} = \phi_1 \left[\alpha_1, \alpha_2, \alpha_3 \frac{d\sqrt{\tau g/\omega}}{\nu} \right] \quad \text{- } 8$$

To derive this he first assumed that the force exerted by the flow of water upon the sediment particles could be expressed in terms of drag as

$$F = C A \omega \frac{V^2}{2g}$$

where C is the drag coefficient which is usually a function of Rynold's number, shape of the particles, the velocity of flow and the diameter of particles. He also assumed that the resistance of particles to motion dependd upon the roughness of the bed and the immersed weight of particles. Shield's equation takes a simple form in case of a level bed comprising of particles of uniform size.

$$\frac{\tau}{\omega(S_s-1)d} = \phi_1\left[\frac{d\sqrt{\tau g/\omega}}{\nu}\right] \quad \ldots\ldots 9.$$

Fig (1) shows the form of the function as determined by Shield's for a considerable range of each of the different variables. It is necessary to note here that Shield's experiments were limited to particles of uniform size only and if the material were non-uniform then the curve takes an altogether different form.

The range within which the Shield's formula is applicable is important in the application of experimental results. For practical purposes function may be assumed to be constant and hence with the specific gravity of gravel and sandy silt taken as ~~xxx~~ 2.65, the critical tractive force would then vary directly with the first power of the diameter d of sediment particles.

For practical use Lane has suggested a set of $\tau - d$ relations which are shown in fig (2). It will be noted that values tentatively recommended by Lane for clear water in fine, non-cohesive material are very much higher than those used in a laboratory but usually have a little binding material which greatly increases the resistance to motion.

The tractive force expression $\tau = \omega R S$ gives only the mean tractive force over the perimeter of a cross section without defining the distribution along the perimeter. For sections with large ratio of width to depth, the hydraulic mean depth R approacches the mean depth and therefore $\tau = \omega D_m S$. This again gives the mean tractive force of the section and cannot be interpreted to mean that the distribution of tractive force is proportional to the depth within the cross section.

The actual distribution of tractive force depends upon the distribution of velocity gradient within the cross section as given by

$$\tau = \mu \frac{dV}{dD} \qquad \text{- - - - - -} \qquad 10.$$

So long as the sheer force distribution in a channel along the perimeter and along the depth is not definately defined it is rather difficult to arrive at any conclusion. Much work has been done on the flow of water in rough and smooth pipes by Karman Parandtl and Nikuradse but a final formula giving the velocity in rough pipes with turbulent flow has not yet been arrived at. These are the discrependies which come in our way in the theoretical evaluation of bed load formula.

This is the situation of flow in straight pipes but the channels which are changing directions the theoretical approach becomes rather difficult and so we are forced to look to the emperical approach.

The problem of practical importance in channels is the effect of meandering on the critical tractive force. In a winding channel the normal distribution of velocity is distorted and the point of maximum velocity shifts towatds the concave bank. The development of cross currents greatly increase the tractibe force in a stream. Lane has tentatively suggested a reduction of critical tractive force for different degrees of sinuosity of channel which are given in the table 3.

When the tractive force of a stream exceeds the critical value certain quantity of sediment is set in motion. The first equation of sediment transportation was given by DuBoy's in which he assumed the rate of sediment transportation was proportional to the excess of the prevailing tractive force over the critical value required to initiate the movement.

$$q_s = C_s \tau (\tau - \tau_c) \quad - - - - - - \quad 11.$$

where q_s is the rate of transportation in volume of sediment per unit width, Cs is the coefficient depending upon the character of sediment. Straub summerized the results of various investigators and gave the value of Cs and τ_c for various sizes of sediment having a specific gravity of 2.65, which are given in table 3a

Investigators in different countries have done work on the bed load transportation and have evolved the following formulae:-

Chang Y.L.	$q_s = C n \tau (\tau - \tau_c)$	(Uniform sand)
Schoklitsch	$q_s = A \frac{S^{3/2}}{\sqrt{d}} (q - q_c)$	(")
Meyer Peter	$q_s = (C_1 S q^{2/3} - C_2 d)^{3/2}$	(Uniform sand large size)
McDougall	$q_s = C S^m (q - q_c)$	(Sand mixture)
O'Brien	$q_s = C (V/R^{1/3})^m$	(")
U.S. Waterways Exp. Station	$q_s = \frac{c}{n} (\tau - \tau_c)^m$	(")

As there are so many relations for sediment transportation

Table No 3.

Critical Tractive Force on Sinuous Canals.

(Lane E. W.)

Degree of Sinuosity	Percentage T_c reqd to initiate bed load movement as compared with straight canals.	Corresponding percentage of mean velocity
1. Straight Canals	100	100
2. Slightly sinuous Canals	90	95
3. Moderately sinuous Canals	75	81
4. Very sinuous Canals	60	78

Table 3a

q_s	Size of sediment mm	1/8	1/4	1/2	1	2	[illegible]
ft^3/s [illegible]	lbs/sft	[illegible]·016	[illegible]·017	0·022	0·032	[illegible]	[illegible]
$m^3/s/$[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
$ft^3/s/$[illegible]	[illegible]	[illegible]	[illegible]·48	0·24	[illegible]·17	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

it is rather confusing as to which should be taken as practically useful. Johnson compared the formulae listed above by plotting the same data according to different formulae. By means of statistical analysis of various plotted graphs, he found that all the formulae were correct to the same extent and so he concluded that the choice of equation could be made on the basis of convenience in measuring the variables involved in them. The maximum q_c convered by the various authors was of the order of 0.03 m^3/sec per meter width of channel or 0.3 cft per sec per foot width of channel.

Shields has given one dimensionly homogenous equation for sediment of uniform size as follows:-

$$\frac{q_s S_s}{q S} = 10 \frac{(\tau - \tau_c)}{\omega (S_s - 1) d} \quad \cdots \quad 18$$

Einstein has developed a new function for bed load. In his method he has assumed that any one particle that would begin to move in a given unit time could be expressed in terms of rate of transport, the size and the relative weight of the particle and a time factor equal to the ratio of the particle diameter to its velocity of fall. The same probability was expressed again in terms of the ratio of forces exerted by the flow to the resistance of the particle to motion. The forms of probability relationships were then equated to yield a general function

$$\phi = f(\psi) \quad \cdots \quad 19$$

in which,

$$\phi = \frac{q_s S_s}{\sqrt{g(S_s-1)}\, F d^{3/2}} \quad \text{and} \quad \psi = \frac{\omega (S_s-1) d}{\tau}$$

$$F = \sqrt{\frac{2}{3} + \frac{36 \nu^2}{g d^3 (S_s-1)}} - \sqrt{\frac{36 \nu^2}{g d^3 (S_s-1)}}$$

Einstein investigated the form of the indicated function by plotting the experimental measurements against ψ as ordinate and ϕ as abssisa. The plot is presented in fig (3). For sediment of uniform grain size the fact that results from the various workers follows a curve is remarkable since the material used in these experiments varied from 28.6 to 0.35 m.m.in diameter and the depth of flow varied from 0.6 to 3.6 ft.

Hunter Rouse plotted the experimental data of s ϕ against $\frac{1}{\psi}$ and tried to reduce it to a single linear function. The function as per Rouse becomes

$$\phi = 40\left(\frac{1}{\psi}\right)^3 \quad - - - \quad 20.$$

It is seen by the above study that the formulae are based on experiments in the laboratory. How far these can be applicable to the natural streams is again a problem as highly mathematical expresions are seldom used by practical engineers.

Out of the many formulae derived, Einstein's formula is a new approach. He claims that his laboratory results agree to a great extent to the field measurements. Further research on the lines of Einstein will surely result in some better formula which can have a wide application.

Movement of Suspended Load

Movement of suspended load is an advanced stage of the bed load movement when particles in saltation are caught by the upward component of the turbulent velocity and are kept in suspension. Therefore sediment transportation in suspension is always accompanied by bed load movement and if the suspended load is derived from the material of the river bed, it is very difficult to differntiate between the two in the region near the bed where the sediment particles are also in saltation.

The problem of calculating the suspended load consists of firstly to clarify the load in a stream and secondly to co-relate it with the bed load to calculate the total load. Uptil now only the first part has been achieved to some extent but progress in calculating the total load has not reached a satisfactory stage.

The approach to the measurement of suspended load is based on the mechanics of turbulent flow of fluids. If the concentration of sediment at some height above the bed is denoted by C, expressed in weight of sediment per unit volume of fluid and the settling velocity v, the rate at which the materials settle through a unit area at a height will be balanced by the rate of upward movement due to turbulent mixing and therefore

$$v c = - E \frac{dc}{dy} \qquad 21$$

where E is known as the exchange coefficient, having the dimensions of velocity times length. The value of E is not constant throughout the section and can be determined from movement transfer and velocity distribution by the formula

$$\tau = \rho E \, {dV}/{dy} \qquad 22$$

where V is the velocity at a distance Y from the bottom. So C_a is the concentration at a distance a from the bed then the equation

$$\frac{C}{C_a} = \left(\frac{D-y}{y} \cdot \frac{a}{D-a} \right)^Z \text{ where } Z = \frac{v}{K\sqrt{gDS}} \qquad 23$$

gives the relative concentration. K is the constant for turbulent flow which is about 0.4 for clear fluids. As the above equation gives only a relative concentration, it is not possible to calculate the total transport of sediment. To make this possible the value of Ca at some elevation a should be known.

It can be seen that as Z becomes small indicating sediment with small velocity of fall or the flow with greater tractive force, the concentration and distribution tends to be more uniform over the entire depth.

Vanoni V.A. carried out experiments to check up the above equation. For $d = 0.16$ m.m. and $Z = 1.03$ experimental results compared well with the theory. A detailed report of his experiments can be seen in Paper No. 2267 of Transactions of ASCE. Vanoni has observed that the distribution of the relative concentration of the suspended load has the form of the above equation but does not agree quantitatively with it i.e. the value of exponent Z given by theory does not agree with Z1, the exponent that fits the experimental results. When the suspended sediment is fine Z1 is less than Z which means that the relative concentration is more uniformly distributed and has a greater average value. The disagreement between the calculated and measured distribution should be due to the action of turbulent fluctuations in suspending sediment and the slip between the fluid and the sediment as the sediment gets accelerated. For fine materials the coefficient of sediment transfer tends to exceed the coefficient of movementum transfer and vice versa. Suspended load reduces the resistance to flow thus causing sediment laden water to flow faster than the clear water. In the laboratory and average suspended load of 1.2 grams per liter reduced the friction as much as 20% and correspondingly the Manings roughness coefficient was reduced as much as 10%.

The above conclusion can be related to three effects that occurs in the flow in the presence of sediment. One is that the sediment appears to damp out the turbulence in such a way that the momentum transfer is reduced. Second one is that the turbulence which is not a factor in the transfer of momentum, contributes to the transfer of sediment and the third one is the slip between the fluid and the sediment tends to make the sediment transfer coefficient less than momentum transfer coefficient.

The increase in velocity due to addition of sediment to clear water does not give rise to an increased sediment transporting capacity since it is accompanied by a reduction in turbulence. Suspended load tends to cause the flow to become unevenly distributed. These are the observations of Venoni which can serve as a guidance to the research workers.

Lane and Kalinske presented an equation giving relation between sediment concentration above the bed in a stream in terms of the composition of bed material as follows:-

$$\frac{v}{\sqrt{gDS}} = f\left(\frac{C}{C_b}\right) \quad - - - - \quad 24$$

wheres C is the concentration of sediment just above the stream bottom and Cb is the percentage of particles of the total stream bed material having a fall velocity v. Measurements taken on the Mississippi, the Missouri and the Colorado Rivers in United States indicate a definate relation between the two functions within the range of values of the abstract number $\frac{v}{\sqrt{gDS}}$ from 0.02 to 1.0. This has an important meaning in this respect that rapid change of concentration of sediment of larger sizes takes place near the bottom.

Thus we see that the formula gives only the distribution of sediment and it cannot be used to calculate the total sediment load in suspension. In this part much work needs be done to ~~mx~~ evaluate a rational formula for suspended load which has not been done uptil now. There is also difficulty in separating the bed load and the suspended load which makes the problem more complicated to measure the two loads. However, it is practicable to measure the total load passing at a certain section of a stream by the different types of sediment samplers. Samplers of different patterns have been deviced by different people but as sampling of sediment forms a subject by itself it will not be proper to dialate from the present discussion to sediment sampling.

Calculation Of Total Load.

Einstien developed a method for calculating total load transported in a stream. His approach to the problem rests on two principles namely the restriction of bed load to the bed layer and corelating of suspended load concentration at the surface of the bed layer to the concentration or the rate of transport of bed load. The corelation of suspended load to bed load implies that the former is, as a result of mixing, derived from the later and consequently from the stream bed itself. Thus fine particles of any sediment load which donot appear in the composition of river bed material will not be ~~xxxx~~ included in the computation. For practical purposes, Einstien suggested that one may exclude the finest 10% by weight of the river bed material since these donot usually represent a structural part of the bed but only loosely fill the pores between the large particles.

For any given bed material and under certain prevailing tractive force of a stream the rate of bed load transportation can be ascertained by means of any suitable bed load formula. Then the concentration of the suspended load of flow at a distance just above the bed layer should be corelated with the rate of bed load transportation. With this known value of concentration at a distance equal to the thickness of the bed layer from the river the distribution of the suspended load can be calculated along the entire depth. The total suspended load can be obtained by integrating the product of two curves along the depth namely the sediment distribution and the velocity distribution curve. The total suspended load when added to the bed load will be the total sediment transported by the stream under prevailing conditions of flow and composition of bed material.

The method can be explained like this. The distance of travel of small masses of fluid is commonly known as mixing length. The mixing length becomes smaller as the bed or any wall is approached. The flow at the bed layer in which the mixing length is so small that suspension becomes impossible has been found by Einstein to be above 2 grain diameters thick. This is designated as the bed layer. Assuming that the bed load material moves with the average velocity of Ub within the bed layer having a thickness of 2d, the average concentration of the fraction of bed load within certain interval of the total bed load expressed in weight of sediment per unit volume of mixture is $\frac{i_B q_B}{U_B 2d}$ where q_B is the total bed load transportation expressed in weight per unit time and unit width of the channel and i_B the fraction of bed load of certain size.

The concentration of suspended load corresponding to the size interval also expressed in weight per unit volume mixture is related to the average concentration of bed load by the simple relation

$$C_a = A_1 \frac{i_B q_B}{2d\, U_B} \qquad (25)$$

where A1 is assumed to be constant. The velocity U_B is not known. Assuming U_B to be directly proportional to the sheer velocity at the bed the above expression can be written as

$$C_a = A_2 \frac{i_B q_B}{2d\, u_*} = A_2 \frac{i_B q_B}{2d\sqrt{gRS}} \quad - - - - - 26$$

The dimensionless constant A_2 must be determined experimently. According to Einstein, the average value of A_2 based on special set of 26 experiments using different sand mixtures can be taken as $A_2 = 1/11.6$. With this known value the total suspended load over the entire depth for various size intervals at the bed can be calculated.

Einstein's approach of corelating bed load with suspended load is a significant advance in sediment transportation studies. Further work on Einstein's lines looks to be promising.

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EMPERICAL METHODS OF CHANNEL DESIGN

An earthern channel excavated in coherent alluvium is said to be stable when there is practically no scouring of its bed and banks and there is no deposition of sediment in flow, when considered over a long period. The stability of a channel depends upon two factors; one is the resistance of the material composing the channel against the erosive power of flow and the other is the capacity of flow to carry a certain quantity of sediment load without significant deposition.

In general the unstable channels can be classified into three catagories; (i) those where only scour occurs (ii) those where only deposition takes place and (iii) those in which both scour and deposition occur. The first class of channels usually carry water with little or no sediment. The second class of channels are ordinarily found where heavy sediment loads are transported with their banks and bed of a material which is highly resistant to scour. The third class of channels are those where both scour and deposition occurs; they carry heavy sediment loads and are excavated in easily erodible material.

Kennedy's Work On Stable Channels

The first pioneer work done towards the design of non-silting channels was by Kennedy R.G., based on his observations on varimus channels in the Upper Bari Doab canals, Punjab. He selected over thirty reaches on this canal system which were stable from maintenance point of view. These canals did not have either silting or scour trouble for the past thirty years. He made observations of the velocity and depth on these reaches and tried to corelate under certain conditions.

He observed that sediment in the flowing canal is kept in suspension solely by the vertical component of the constant eddies which can be always observed over the full width in any stream, boiling up gently towards the surface. These eddies rise on account of the roughness of the bed and work up against the depth of the channels. From the sides sights also some eddies occur but they are horizontal for a greater part and hence of no silt transporting power. So the silt transporting power in a stream is proportional to the width of the stream and not the total perimeter. A regime canal, accoriing to Kennedy, is that where neither silting nor scouring occurs.

He also stated that for every discharge there is a certain critical velocity at which the channel is non-silting and non-scouring. If the velocity of the channel is less than the critical then silting takes place and if it is more than the cititical, scour of bed and banks will result. The depth at which the critical velocity is obtained was termed by Kennedy as the critical depth. He plotted the various data of his observations and gave a general law of critical velocity as

$$V_o = C\,D^m \qquad 27.$$

where Vo is the critical velocity at depth D and C and m are the constants. For the conditions in which Kennedy evolved this equation, the values of C and m were found to be 0.84 and 0.64.

He also recognised that the grade of silt played an important part in this relationship and regarded the silt of the Upper Bari Doab canal as standard. His formula can be written as

$$V_o = 0.84\, D^{.64}$$

It was soon found ~~by~~ Kennedy that the grade of silt varied to a great extent in different regions where the canals were constructed. He then introduced the Critical Velocity Ratio which is a ratio between the actual mean velocity in a channel to the critical velocity calculated by the above formula. C.V.R represents a factor which is a measure of variations in the silt condition from the standard silt of Upper Bari Doab canal. The C.V.R. i.e. V/Vo was kept near about unity in all designs of canals. Kennedy considered that a velocity of ~~is~~ 3.5'/sec was just sufficient and safe and a slight increase in this would endanger the stability of side slopes. It meant that there was a limit to depth as well.

$$D_{max} = \left(\frac{3.5}{0.84}\right)^{1/.64} = 9.3'$$

So according to Kennedy a channel cannot be stable at a depth of more than 9.3' and a velocity of 3.5'/sec.

He tried to work out the silt transporting capacity of channels on the assumption of the sides of channel being vertical. This assumption is not completely wrong as in the case of very wide channels the sides can be considered vertical.

Let p be the percentage of silt carried by water.

The area of cross section = B.D

Discharge at critical velocity = Vo.B.D

Amount of silt carried $= p.Vo.B.D.$

Assuming the silt carried to be proportional to the nth power of V

Amount of silt carried $= KBVo^{n} \; p = pVoBD$

$$Vo = \left(\frac{pD}{K}\right)^{\frac{1}{n-1}}$$

For Punjab the value of the exponent m was 0.64 and so

$$\frac{1}{n-1} = 0.64 \quad \text{So } n = 2.5 \text{ (approx)}$$

~~That is~~ Thus Kennedy concluded that the silt transportive power depends on $Vo^{5/2}$.

From this 'x' the amount of silt carried at velocity V other than Vo can be calculated like this.

$$x = kV^{5/2} \quad \text{and} \quad p = kV_0^{5/2}$$

$$\therefore \quad x = p\,(V/V_0)^{5/2}$$

When the mean velocity of a channel exceeds the critical velocity the silt carrying capacity will be more ~~and~~ than the bed can resist and scour will occur. The amount of scour will be

$$x - p = p\left[(V/V_0)^{5/2} - 1\right]$$

and if V is less than Vo deposit will take place.

$$p - x = p\left[1 - (V/V_0)^{5/2}\right]$$

~~Theee~~ These formulae for the silt carrying capacity were just derived but were not checked in practice .

Kennedy carried out no investigations to find out the correct ~~slow~~ slope formula which is so necessary in the design of irrigation channels. He took the Kutter's formula and gave the value of N equal to 0.0225, as the average value for Punjab conditions. He designed a set of hydraulic diagrams known as the Kennedy's diagrams for designing channels with different slopes showing discharges, depths and velocities. It was soon found that the value of N was not constant and it varied greatly even with the size and discharge of channels. That is why, he himself later suggested the value of 0.02 for large canals and a value of 0.025 for small irrigation channels.

Kennedy's work can be considered as a pioneer work in the field of the design of stable channels. But as all basic works have some draw-backs it also has got a few points of defect which came to light later and people critisised his work. The author considers his work to be of basic value even though it has a few weak points. The data which he collected is very valueable which Lacey later took to derive different equations.

The first defect of Kennedy's work is that he depended upon the Kutter's formula for slope calculations. In Kutter's formula, the fixing of the value of N is arbitrary and on the discretion of the user. For a particular type of material one might give a certain value to N and another person might assign some different value for the same material. There is nothing definate about it and hence Kennedy's assumption of Kutter's formula was basically wrong; consequently all his derivations based on the above assumption are not quite correct. This discrepencey was pointed out to him and then he said that a value of N = 0.02 should be taken for large channels and 0.025 for small channels. His statement that the critical velocity ratio varied according to the silt conditions was too vague.

Secondly in the derivation of the critical velocity formula Kennedy assumed that the eddies from sides have no part to play in sediment transportation and so he related the velocity to depth. But it was not quite correct.

Thirdly he did not notice the importance of the bed width and depth ratio. These are, infact, so many variables and by one critical velocity equation a complete design cannot be sought unless some relations in different variables are know. Similarly he forgot to recognise that slope plays a great role in channels. For the same section the critical velocity will not remain non-silting and non-scouring if the bed slope is changed.

Lastly he did not take into account the silt grade and silt charge in his observations. The quantity and type of silt plays a very important part in channels.

Garret's Diagrams.

In 1913 a set of hydraulic diagram was prepared by Garret for the design of channels. These diagrams are available for any discharge from one to 12000 cusecs, for any bed slope from 1/100 to 1/10000, and for any rugosity coefficient from 0.018 to 0.03. Though Garret diagram donot offer any silt theory they serve as a good tool for design with provisions for following the Kennedy's formula.

Lindley's Work On Stable Channels.

Next to Kennedy an extensive survey of stable channels was carried out by Lindley E. S. He stated that when an artificial channel is used to convey silty water, both banks and bed scour and fill changing depth gradient and width until a certain state of balance is attained at which the channel is said to be in regime. These regime dimensions depend upon discharge, quantity and nature of silt and rugosity coefficient of the silted section. He believed that rugosity is affected by velocity also which determines the size of wavelets into which the silted portion is thrown. He concluded that the different relations of bed width, depth and longitudinal slope of a channel were all fixed by nature to carry a certain supply of silt load.

Lindley's observations covered over 2700 miles of channels on an entire canal system. He did not select the so called regime sections but instead selected straight and regular reaches. He derived the following regime equations:-

$$V = 0.95\, D^{.57} \tag{28}$$

$$V = 0.57\, B^{.355} \tag{29}$$

$$B = 3.80\, D^{1.61} \tag{30}$$

Unlike Kennedy Lindley gave three different equations relating velocity to depth, velocity to bed width and bed width to depth. This was a definate improvemant over Kennedy.

The data on which these formulae were based included bed widths, depths and gradient of channels. Lindley did not observe the velocities and discharges but employed Kutter's formula and Chezy's formula for calculating velocities assuming N to be constant having a value of 0.0225. It was the same mistake which Kennedy did in employing Kutter's formula which allows guess work in determining the value of N. Lindley did not co-relate rugosity and silt grade nor did he give any relation in width and depth with the discharge.

Lindley's work cannot be considered of any great importance but the data collected by him was of great statistical importance which was later used by Lacey to derive his equations in 1930. During the same period engineers in different places started taking observations on their respective channel sections to check the validity of Kennedy's critical velocity formula. It was found that neither C nor m in the Kennedy's formula were constant but varied greatly in different parts.

In Egypt Ghaleb K.O. evolved his equation for critical velocity as

$$\left.\begin{aligned} V_o &= 0{\cdot}39\, D^{\cdot 73} \quad \text{for lower Egypt} \\ V_o &= 0.47\, D^{\cdot 73} \quad \text{" upper "} \end{aligned}\right\} \quad 31$$

The work of Molesworth, Yenidunia and Buckley is also important. The former suggested a slope formula as

$$D = (9060\, S + 0{\cdot}725)\sqrt{B} \qquad 32$$

and Buckley suggested

$$D = \frac{0{\cdot}0025(10000\, S + 8)^2}{1{\cdot}62}\, B \qquad 33$$

In table No. 4 the different values of C and m of the velocity formula observed at different places are given which might be of some interest.

Wood's Formula.

In 1927 Woods proposed general formulae covering velocity,

Table 4.

The values of C and m for different Systems in Kennedy's Formula

No.	System	C	m	Remarks.
1.	Upper Bari Doab	0.84	0.64	By Kennedy.
2.	Lower Chenab Canal	0.95	0.57	By Lindley
3.	Sind Canals	0.64	0.63	
4.	Egyptian Canals	0.39 0.47	0.73 0.73	For lower Egypt " Upper " } By Ghaleb.
5.	Shwebo Canal Burma	0.97	0.57	By Stand.
6.	Godavari Western Delta	0.64	0.55	
7.	Kistna Western Delta	0.93	0.52	

depth and mean width and slope as under:-

$$D = B^{\cdot 434} \qquad 34$$

$$V = 1{\cdot}34 \operatorname{Log} B \qquad 35$$

$$S = \frac{1}{2 \operatorname{Log} Q \times 1000} \qquad 36$$

These equations cover not only width and depth of channel but also cover discharge and slope. According to the slope formula of Woods there can be one and only one slope for every given discharge at which the channel remained silt stable.

During the same time Bottomley W.I. presented his concept saying that the canal will remain free from silting or scouring if its slope was of the same order of magnitude as that of the parent river. Comments on Bottomley's concept of slope are not available anywhere. It looks, people did not consider it of much importance. The author feels it was just a silly idea because for the same slope in the river sometimes heavy scoure occurs and sometimes deposition takes place if the discharge is more or less. There does not seem to be any meaning in saying that the stability of a channel depends only on slope and all other elements just donot affect.

Lacey's Work On Stable Channels.

Lacey's work on stable channels is supposed to be very authoritative and systematic even though there is lot of criticism on it these days from the American engineers. His paper in the Institution of Civil Engineers, London, in 1935 and the Central Board of Irrigation publication No. 22 are very important in the study of stable channels in Alluvium.

According to Lacey regime flow in silt transporting channels excavated in alluvium connots a physical stability or balance between silting and scouring and a dynamic equilibrium in the forces generating and maintaining the channel cross section and gradient. For regime conditions to be established the fundamental requirements are that the discharge should be constant, the channel flowing uniformly in unlimited incoherent alluvium of the same character as transported and the silt grade and silt charge are a constant. In-coherent alluvium, as defined by Lacey is that loose granular material which can be scoured as readily as it can be deposited. For true regime the channel should flow in an unlimited alluvial plain of the same grade as the material transported and there should be complete freedom for lateral movement. Sandy rivers in alluvial plains achieve to some extent this freedom and by meandering adjust their length and slope. A constant discharge transporting silt of a given grade and flowing in a self transported alluvial plain of the same grade tends to assume a gradient which is determined by the discharge, the silt grade, the mean velocity, hydraulic mean depth and wetted perimeter will tend towards unique determination. Such a constant discharge will also tend to transport a fixed regime silt charge.

Channels excavated in the first instance with defective slope and somewhat narrow dimensions are free by immediately throwing down incoherent silt on bed to increase their slope and velocity and try to attain an initial regime. Channels of this type attain a working stability. Final regime represents the condition set up in theory when all variables are free to vary. The first adjustment which a channel, heavily charged with silt, will make ~~xxxx~~ is that between the mean velocity and depth. The ultimate adjustment will depend upon the extent to which the wetted perimeter, the slope and the channel length are free to vary. In all artificial channels the length is usually restricted and hence an ideal final regime cannot be established.

Like Kennedy Lacey also believed that the silt is suspended by the vertical components of eddies but ~~xxxxx~~ says that the channel section is generated at all points by forces normal to the wetted perimeter and therefore Lacey adopts the hydraulic mean depth rather than the wetted perimeter as the variable.

The first thing which Lacey did was to plot Kennedy's and Lindley's data and got an equation of form

$$V_o = \alpha_1 R^{\beta}$$

For Punjab data he got $V = 1.138\, R^{.4995}$ and for Madras data he got $V = .79 R^{.508}$. He termed the coefficient as the 'silt factor' f which was an index of the silt grade. Just to make the relation appear simple he introduced a constant K and put f and R under the root sign because his plot showed that the power of R should be near about half. Thus his equation took the form

$$V = K\sqrt{fR}$$

The value of K which suited his plotting of datas was given by him as 1.17 and hence his first formula derived from the Kennedy's data was

$$V = 1.17\sqrt{fR}$$

The second formula which was derived by Lacey on plotting f^2A verses V was $$A f^2 = 3.8 V^5 \qquad 38$$

By these two equations he got several other equations as a corrollary. The important formula thus obtained was a relation in wetted perimeter and discharge as

$$P = 2.667\sqrt{Q} \qquad 39$$

By this formula it shall be seen that for a given discharge the silt stable perimeter is independent of the type of dediment and directly varies as the square root of discharge.

The third important formula evolved by Lacey by plotting the entire data was his general regime equation

$$V = 16 R^{2/3} S^{1/3} \qquad 40$$

and the slope formula as

$$S = 0.000387 f^{3/2} \div Q^{1/9} \qquad 41$$

Lacey says that the regime equation of flow should be of great practical utility in estimating the maximum discharge in rivers flowing in incoherent material. The same equation can be put in the form of Chezy('s equation

$$V = 64 (R/V)^{1/2} \sqrt{RS}$$

where constant C will have a value of $C = 64 (R/V)^{1/2}$

He said that there is some confusion about the use of rugosity coefficient N and lot of guess work has to be done to assign a certain value to N. He introduced Na as an absolute rugosity coefficient which could be determined solely by the average size and density of the incoherent bed material of the channel. The standard grade of silt is that which connots a rugosity coefficient of 0.0225 at a hydraulic mean depth of one meter. He gave a relation in the absolute rugosity coefficient and silt factor as

$$N_a = 0.225 f^{1/4} \qquad 42$$

He suggested a flow equation for channels which depart from regime as

$$V = \frac{1.3458}{N_a} R^{3/4} S^{1/2} \qquad 43$$

From the previous basic equations he derived many other relations as under:-

$$S = \frac{0.000549}{Q^{1/6}} f^{5/3} \qquad 44$$

$$R = 0.47247 \frac{Q^{1/3}}{f^{1/3}} \qquad 45$$

$$V = 0.7937\, Q^{1/6} f^{1/3} \qquad 46$$

In this way he tried to corelate all the variables of channels.

All these equations were published in his paper of 1930 which naturally got lot of criticism and then he tried to modify the equations whereever he could not answer his critics.

Later on he introduced another factor 'Shock' in channels due to bends and irregularities and due to the channel conditions. With the shock consideration his flow equation for non-regime channels took the shape as

$$V = \frac{1.3458}{N_a} R^{3/4} (S-s)^{1/2} \qquad 47$$

for s he suggested the values given in table (5).

We see that in all these Lacey's formulae silt factor

4 ✓

Table No 5.

Values of Shock, s, in different conditions

Channel condition	N	Shock S	Description of Channel
Perfect	0·025	0·000S	Natural Stream Channels straight banks.
Good	0·0275	0·174S	
Fair	0·0300	0·306S	
Bad	0·033	0·426S	
Very good	0·0225	·000S	Earthen channels under ordinary conditions.
Good	0·0250	0·190S	
Indifferent	0·0275	0·331S	
Bad	0·0300	0·437S	
			S indicates Slope

plays a very important part and hence its determination is very important. For an existing channel it could be determined by the equations in which it occurs if we know other elements of channel. But if the channel is in the regime, the value of f got by different regime formulae should be same. Any variation will indicate the extent to which it is out of regime. By averaging the value of f, a fairly correct value can be obtained.

In case the data is not available then the value of f can be got by a formula which relates f to mean diameter of particles.

$$f = 1 \cdot 76 \sqrt{d_m} \qquad 48$$

So this fact is worth appreciating in Lacey's work that he has tried to eliminate the guess work to a minimum and has given a definate formula which gives the correct value. This is the great advance by Lacey over Kennedy and Lindley's work. The selection of the value of rugosity coefficient is also given definately by the formula $N_a = 0 \cdot 0225 f^{1/4}$. Unlike Kennedy and Lindley he gave a regime slope formula which can serve as a guide in designing channels.

The use of Lacey's equations involves an appreciable amount of calculation ~~which~~ with chances of error. To eliminate such error and to expedite the work, Lacey has provided two diagrams. In both these diagrams the coordinates are devided lograthmically to secure a suitable reduction of scale at higher values. The first is entitled Regime Dimension Diagram and is provided in two parts one from 4 to 100 cusecs and the other from

100 to 20000 cusecs. From this diagram for any known value of discharge Q and silt factor f the values of bed width and depth can be obtained on the horizontal and vertical coordinates respectively. These diagrams give values for a channel section which is trapizoidal with side slopes as 1/2 to 1. The second diagram which is known as the Regime Slope Diagram gives slopes for different discharges and silt factor. From these two diagrams it is possible to determine the slope and dimensions of any channel if the two factors discharge and the silt factor are known. Lacey's diagrams for the design of channels are very extensively used in Punjab and Uttar Pradesh.

In the above paragraphs we have seen the Lacey's concept of regime channels and his method of designing stable channels. Lacey's work is referred to as a regime theory in India. These days there is a criticism over the use of word theory for Lacey's work. Leliavasky in his recent book 'Fluvial Hydraulics' says that though Lacey's papers on channels in alluvium is one of the very important works on the subject, it cannot be considered as a theory. He says that there is a difference between an emperical concept and a scientific theory. Lacey's work cannot be called his theory because it is just an emperical method of design of channels and it is not possible to give a rational proof for his work.

The circumstances which led Lacey to evolve the so called theory are rather strange. At the end of thired decade of this centuary, there were so many alternative methods (So called theories) that the irrigation engineer was at a loss to know which to choose for his design. There was lot of data on channels collected by Kennedy, Lindley and Madras engineers. To get over this, Lacey was asked by the U.P. Government to put some order into the mass available data and produce some standard design procedure. It should be noted here that Lacey did not produce any un-published information on the subject based on his own observations or research. At the Central Board of Irrigation, Lacey worked for quite a long period to do his assignment. He tried to put Kennedy's and Lindley's data on the same plot as both represented stable channels and attempted to fit in some formula which satisfied both the field observations. So in brief Lacey's work can be considered as a clever manipulation and interpretation of Kennedy's and Lindley's observations. Thus Kennedy's and Lindley's deserves the credit which has gone to Lacey as it was the real basis for all Lacey's formulae.

This is the reason why the present workers on the subject refuse to consider Lacey's work and his emperical formulae as any new theory. The fact cannot be denied that the word theory is used to describe certain system of logical thinking which reduces a certain observed phenomennon to rational mechanics. In this sense Lacey's concept cannot be called a theory as neither Lacey himself nor anybody else has been able to provide a rational proof for his formulae. The author too feels that the use of 'theory' for Lacey's work is not justified and so he has been careful enough to call these concepts and emperical relations as certain tools of design or certain methods of design.

Lacey's paper which is an outstanding achievement in providing certain definate design procedure to practical engineers but it has not recbived the recognition it deserves. It has many pecularities which we will discuss now.

We know that Lacey evolved his formulae by plotting the data of Kennedy and Lindley and tried to fit in some expression in it. It should be possible therefore to fit in some other expressions also on the same plot in a more accurate way than Lacey did. For instance in his plot of regime formula, Malhotra of Punjab tried to fit in the equation

$$V = 18{\cdot}178\, R^{\cdot 6321} \cdot S^{\cdot 3426} \qquad 49$$

which looks to be more suitable for the plot than Lacey's regime equation $V = 16 R^{2/3} S^{1/3}$. In the same way his plots could be interpreted by some other formulae which might be more accurate than Lacey's equations. Secondly the observation of Lindley that Lacey arrived at these formulae with exponents in round numbers which had atleast the appearance of a rational basis, seems to be correct.

Another instance of this is his new theory of shock which Lacey presented without any new observational material. He just looked at his previous plots and found that he can produce a new theory of shock ~~which replaced~~ replacing those equations which he presented in this previous paper. The original and the modified equations are listed below:-

Original	Modified
$S = 0.000387\, f^{3/2}/Q^{1/9}$	$S = 0.00055\, f^{5/3}/Q^{1/6}$
$V = 1.17\sqrt{fR}$	$V = \frac{1.3458}{N_a} R^{3/4} S^{1/2}$
$N = 0.022\, f^{1/5}$	$N_a = 0.0225\, f^{1/4}$

Dr.Hurst has also critisised Lacey's set of formulae. He says that it is not sufficient to devise emperical formulae from the examination of stable channels alone but in order to prove their application, it is necessary to show that they did not apply to non-regime channels. Another objection raised by Hurst that in dealing with the observational material it is always advisable to operate upon directly recorded figures and not on derived values. Kennedy had collected the data of depths and velocities but not slopes, on the other hand Lindley collected slopes, hydraulic mean depths but not velocities and discharges. Lacey calculated slopes from Kennedy's data and calculated velocities from Lindley's data and tried to make his plot. This might or might not give correct results. When ~~only~~ observed data is the basis of work, deductions from observed data only should be made and not from the derivations of these observed material. Thirdly he got only three equations by plotting Kennedy's and Lindley's data and from these three equations he derived so many other equations which is another mistake. That is the reason whey a particular dimension of channel calculated from a certain formula is different from the one calculated by some other formula for the same conditions.

Now a word about his regime concept. Regime conditions will be established when the discharge is constant, silt grade is constant and the channel is free in its lateral movement. An ideal regime can only be found in rivers flowing in plains which are quite free to adpot their own course of flow. In an excavated channel this condition is absolutely impossible to reach. Only the so called initial regime may be attained by channels after a number of years alapse. In that case all channels excavated in alluvium have got to attain regime conditions even if they have defective slopes and sections because as they grow old they will adjust themselves towards the regime condition That way it is not necessary to maintain irrigation canals at all because according to Lacey they will ultimately become stable one day or other. But in practice we see that the case is different.

The great difference in a river and a channel is that a river flows whereever it likes depending upon its slope but a channel has to go along a definate part assigned by the engineer So the author feels that to apply regime conditions of rivers to channels is not wholely correct. Canals have got to be taken by a certain definate route depending upon the area they have to irrigate and on ~~its~~ their way so many structures are to be constructed and so on.

Now let us examine his formulae. It has now been widely accepted that the constant 2.667 in the formula relating discharge and perimeter $P = 2.667\sqrt{Q}$ is not true and its value varies from 2.12 to 3.200 This shows, Lacey's contention that in the formula silt has not to play any part, is absolutely wrong. This wide variation in the constant should be due to the different types of materials found at different places. Secondly the values of the constant cannot be true as it has been derived from the two formulae and not directly observed by anybody.

In the above paragraphs it was stated that the formulae give conflicting results. If a channel of constant cross sectional area with its depth and width changing will give varying hydraulic mean depths. The velocity calculated by $V = 1.17\sqrt{fR}$ increases with R and consequently there is an increase in discharge. But if R increases in the formula $P = 2.67\sqrt{Q}$ discharge Q decreases. Since the area has been assumed constant, a decrease in discharge means a decrease in velocity. Thus while the former equation gives increased velocity with a rise in R, the latter gives decreased velocity with the rise in R. This is the conflict in results for the same data. Not only that. From the formula $Af^2 = 3.8\,V^5$ the velocity should be constant for a constant sectional area. By this reasoning if Lacey's formulae are to be employed for calculating the dimensions of a channel each dimension should be calculated separately from different formulae. Under no case should a value of any dimension be assumed and then the remaining calculated with it.

Lacey's formulae are applicable only in a certain range and any attempt to design channels below that range will give confusing combinations. So for very small discharges design by Lacey's formulae is not possible.

Lacey assumed a fixed section for the particular discharge and silt grade. If it is accepted and if a channel be constructed with a uniform cross section but with different slopes then it will scour heavily or silt badly. So a basic relation in cross section and discharge without any reference to slope is absolutely wrong. Lastly the Lacey's formulae are not dimensionly correct. In m, l, t, system of dimensions, the dimensions of f in the formula $V = 1.17\sqrt{fR}$ will be L/T^2 but from the formula $Af^2 = 3.8V^5$ the dimensions of f work out as $L^{3/2}/T^{5/2}$ which means that the value of f cannot be same in both the equations. If we assume the value of f to be constant then the constants 1.17 and 3.8 must be having some variable factor. In $P = 2.67\sqrt{Q}$ the dimensions on one side are L and on the other side the dimensions are $L^{3/2}/T^{1/2}$ which does not seem logical.

One most important thing left out in his observations and formulae is the silt charge which plays a very significant role in the stable channel shape. It has been the case even with Kennedy, Lindley and others that a quantity of solids in motion has not received the attention that its importance warrants.

Lacey, later in his paper "A general theory of flow in alluvium" in 1946, restated the conclusions which he had drawn in his previous papers, with a physical background. The most important advance was the replacement of his original silt factor

$$f_{vR} = .75\frac{V^2}{R}$$

by

$$f_{sv} = 48\sqrt{SV}$$ 50

adopting Lane's suggestion and postulating that particles of silt of the same grade and density fall in the restricted depth of a model at the same rate as in the greater depths of prototype. He produced a relation

$$S \cdot V = K v$$

where v is the terminal velocity of particles falling in still water. This was based on his new definition of a regime channel as 'a stable channel transporting the minimum bed load consistent with a fully active bed'.

In this paper he produced an equation

$$S \cdot V = K (R^{1/2} S)^{n}$$

one of which was

$$S \cdot V = 16{\cdot}00 (R^{1/2} \cdot S)^{4/3} \qquad 51.$$

It can be seen that this was done by Lacey by looking at the same old plottings in the light of criticism which he received on his previous papers. There is nothing original in this paper.

<u>White's Paper on the influence of transported solids upon rivers.</u>

White C.M., of the Imperial College of Science, London has presented a formula on the basis of dimensional analysis.

$$\frac{V}{(gR)^{1/2}} = \frac{0{\cdot}7\, v^{1/4}}{g^{1/10} \cdot Q^{1/20}} \qquad 52$$

where v is the terminal velocity of fall of particles in water. This can be compared with the Lacey's silt factor formula as under:-

$$f = {\cdot}75 \frac{V^2}{R} = \frac{0{\cdot}3675\, g^{4/5}}{Q^{1/10}} v^{1/2}$$

Work in the Punjab Research Institute.

Bose, Malhotra after several years of painstaking collection and statistical analysis of data derived the following formulae using f.p.s.units axcept for d

$$P = 2.68\,Q^{1/2} \qquad 53$$

$$S = 0.00209\, d^{0.86}/Q^{0.21} \qquad 54$$

$$R/P = S^{1/4}/6.25d \qquad 55$$

where d is the weighted mean diameter of dediment in m.m.

Both the silt factor f of Lacey and the weighted mean diameter d of Bose define the size of the sediment transported in a stable channel, but not sediment charge or rate at which the sediment is transported. It can be anticipated, therefore, that these fomulae are applicable to canals that carry sediment at approximately the same concentration as the canals in India from which these formulae have been derived.

It is curious to note that the Research Institute of Punjab has not produced any rational formula based on experimental work but tried to check up the Lacey's formulae on the statistical data of Kennedy and Lindley. This clearly shows how Lacey's work has impressed all the Indian research workers to think only in one line of emperical approach. It is high time that atleast Research Institutes take up some basic research on the problem of channel design and try to give a rational solution as the Research Institutes of other countries.

Work of Inglis, C.C.

Experiments initiated by Inglis at Poona, when he was the Director of the Indian Waterways Experimental Statioh, has shown that when the grade of material was kept constant the charge ~~may~~ ~~be~~ varied, the exponents approximated to the Lacey's formulae. To take case of the sediment charge Inglis introduced a set of formulae. The constants involved have not yet been determined. Inglis explained further that sediment charge has a small effect on the area of a channel, relatively great effect on slope and shape and considerable effect on channel width. The formulae together with the Lacey's formulae are listed below:-

Lacey's Formulae	Inglis' Formulae	
$P = 2.67\, Q^{1/2}$	$B = C_1 \dfrac{Q^{1/2}}{g^{1/3}\, \nu^{1/2}} \left(\dfrac{cv}{d}\right)$	56
$A = 1.26\, \dfrac{Q^{5/6}}{f^{1/3}}$	$A = C_2 \dfrac{\nu^{1/36} \cdot Q^{5/6}}{g^{7/18}\, (dcv)^{1/12}}$	57
$V = 0.7937\, Q^{1/6} f^{1/3}$	$V = C_3 \dfrac{g^{7/18}}{\nu^{1/36}}\, Q^{1/16}\, (dcv)^{1/12}$	58
$R = 0.4725 \left(\dfrac{Q}{f}\right)^{1/3}$	$D = C_4 \dfrac{\nu^{1/9}\, Q^{1/3}\, d^{1/6}}{g^{1/18}\, (cv)^{1/3}}$	59
$S = 0.000547\, \dfrac{f^{5/3}}{Q^{1/6}}$	$S = C_4 \dfrac{(dcv)^{5/12}}{\nu^{5/36}\, g^{1/8}\, Q^{1/6}}$	60
$R/P = 5.65\, Q^{1/6} f^{1/3}$	$B/D = C_5 \dfrac{Q^{1/6} \cdot (cv)^{7/12}}{g^{5/18}\, \nu^{7/36}\, d^{5/12}}$	61

In the foregoing equations $C = \frac{Q_s (s_s - 1)}{Q}$
where Q_s is the rate of sediment transport expressed in volume per second.

The set of formulae presented by Inglis is the only one developed in India which has taken into consideration the sediment charge. Unfortunately the constants involved have not yet been determined and the validity of the formulae have not yet been verified by taking actual sediment discharge measurements in canals which are considered stable. It is regretful to note that the subject of stable channels which was developed almost exclusively in India has been completely neglected in the recent years. Designs of new canals on the formulae of Kennedy and Lacey are extensively done in India though they dispute among the engineers on the selection of the values of constants involved still exists. Considerable practical experience is always required to choose suitable coefficients for particular formulae to an engineer who has to use them. For engineers who are not familiar with the conditions on the basis of which the formulae are applied, they are completely at a loss to make proper use of them.

As regards the above mentioned Inglis' formulae the author feels that they are too complicated to be used by practical irrigation engineers and hence it is doubtful whether people would take notice of them.

<u>Diagrams of Isher Das</u>

Ishar Das in his recent paper published in March, 1950 issue of the Central Board Of Irrigation Journal says that the silt factor f of Lacey is very confusing. He says that V varies as ~~R½~~ $R^{1/2}$ and P varies as $f^{1/2}$. He has introduced two coefficients C_1 and C_2 and has put the above two formulae of Lacey as under:-

$$V = C_1 \sqrt{R} \qquad 62$$

$$P = C_2 \sqrt{Q} \qquad 63$$

He has reduced from these the following relations:-

$$R = C_3 Q^{1/3} \qquad 64$$

where $C_3 = \left(\frac{1}{C_1 C_2}\right)^{2/3}$

also $C_1 C_2 = \frac{Q^{1/2}}{R^{3/2}} \qquad 65$

$$S = \frac{0.157\,(C_1 C_2)^{5/6}}{Q^{1/9}} \qquad 66$$

Explaining the physical significance of C_1, C_2 and C_1C_2, he says that the factor C_1 represents the grade and quantity of silt carried by the stream. In other words C_1 is a function of the detritus in flowing water but unlike f it is a measureable element. C_2 is a function of the nature of the periphery of the stream i.e. it represents the nature of the soils surface forming the margin of the stream which is another measureable entity. The product C_1C_2 has been defined by him as silt soil factor. This product defines the hydraulic mean depth in terms of the discharge and statistical analysis of some reliable data shows that ~~kk~~ slope is a function of Q and R. It follows that the silt soil factor is important in not only determining the shape R but also in determination of slope.

Ishar Das has presented three sets of curves for his three equations to save labour involved in the calculations.

No comments on Ishar Das' formulae are available as these have not yet been checked by any practical engineer, neither his formulae have been checked in any laboratory for their validity.

It is seen from the above study that the formulae mentioned have not yet reached any final stage with regard to their use on stable channels. Formulae evolved by Lacey are not universally adopted and not even in various states of India. The conditions in Southern India are entirely different from those of Northern India and hence Lacey's formulae are not adopted in the Southern part of this Sub-Continent. Much work regarding the collection of fresh data from field as well as from experiments in the research laboratories is required for evolving universally applicable formulae. India has done appreciable work on the subject as seen in this chapter and provides necessary facilities for further studies in view of the large irrigation system in different parts of the country wherefrom a necessary data can be collected.

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LANE'S METHOD OF DESIGNING STABLE CHANNELS

In the previous chapter, the emperical methods of designing stable channels evolved in India by Kennedy, Lacey and others have been discussed in full detail. Although ~~fax~~ these formulae provide workable relations for the conditions for which they were developed they cannot have a universal application as they were developed emperically from a very limited range of conditions. In United States while the design work on the All American Canals on the Lower Colorado river was in progress, it was found that the Lacey's relations did not give satisfactory solutions when applied to the fine sediment of that region. After a thorough study of the available literature on the subject, a statement of the general principles of stable channels was prepared by Lane E.W. at the U.S. Bureau of Reclamation.

Factors affecting the Stable Channel Design

In order to develop a rational design, Lane studied the fundamentals of channel design and systematically analysed the factors controlling the shape of a channel in erodible material. The factors which enter into the stable channel design are as follows:-

1) Hydraulic factors such as slope, roughness, hydraulic mean depth, mean velocity, velocity distribution temparature etc.
2) Channel shape factors as the bed width, depth and side slopes.

3) Nature of materials transported, depending upon size, shape, specific gravity, dispersion, quantity and the materials of bank and bed.
4) Miscellaneious such as alignment, non-uniformity of flow and aging.

To obtain a rational solution it is absolutely necessary to consider all the above factors and to determine accurately the factors of major importance and neglect those having very minor effects.

Of the hydraulic factors the slope roughness, hydraulic mean depth and mean velocity are inter dependent and their relation is known from the usual velocity formulae. Roughness of banks and bottom, certainly has effect upon the movement of material in suspension and by traction, which has not been fully known, yet to some extent they can be mathematically analysed. The laws of movement of bed load and suspended load, too, are not fully known The progress made upto date has been shown in chapter No.3 which is certainly not of direct importance on the design of channel. Velocity distribution and the mean velocity in a channel section, are very important facters, of which mean velocity can be calculated but the velocity distribution has not yet been fully understood for trapizoidal channel sections. The presence of the boundary layer too cannot be forgotten, which ~~marks~~ makes the problem of velocity distribution still more complicated. Temparature has an effect on viscosity of water and consequently the rate at which the solid particles settle. The effect of temparature plays an insignificant role in stable channel shape and hence it would not be much harmful if it is neglected. Moreover, in all the works done uptil now temparature data has not been collected.

The shape factors are the depth, width, side slopes and the wetted perimeter. It will be necessary to find a section which gives minimum perimeter for a given area and discharge.

The nature of materials transported includes the sediment particles as shape, specific gravity, dispersion quantity and sub - grade material. The size of the particle is of importance though shape may be neglected for practical purposes, as the laboratory experiments show that the rounded particles move with a little less velocity than the angular ~~areas~~ ones. The specific gravity of sediment may be taken as uniform because even if it slightly varies it is of less importance. The study of actual dispersion of particles of various sizes in water has not been made uptil now and this is open to further studies by physicists and chemists. The quantity of solids moving is of prime importance and though some studies have been made to determine the suspended load and the bed load movement, much remains to be done to evolve a universal formula. The bank and the sub grade material has also its share in the studies of channel shape and the stable side slopes depend upon the angle of repose etc. If the materials composing the bed and banks are resistant to scour, higher velocities can be used than if the material is easily erodible.

In the miscellaneous group, we may include canal alignment, non-uniformity of flow and aging. Alighment is important because bank scour is more likely to occur on curves. The age of canal is also important because after water has run for sometime in a channel, the particles composing the bed arrange themselves in such a manner that they are more difficult to move than when the water is first conveyed through the canal. If the water is silty this material forms a kind of weak cement which binds the bed material together and makes it more resistant.

Canals for conveying water for irrigation or power are usually designed to meet three sets of conditions. The first is when it is desired to have a lowest practical velocity in order that the slope be a minimum. This is done in irrigation canals to command more area for a given length and in power canal this is done to obtain greatest feasible head. In the second type it is required to reduce the size of the canal to a minimum in order to make the cost as small as possible without making slope steeper than necessary. In the third condition it is sometimes desired to carry the canal on an alignment that has a slope as steep as possible in order to reduce the cost of drops and falls on the way.

General Aspects Of Stable Channels

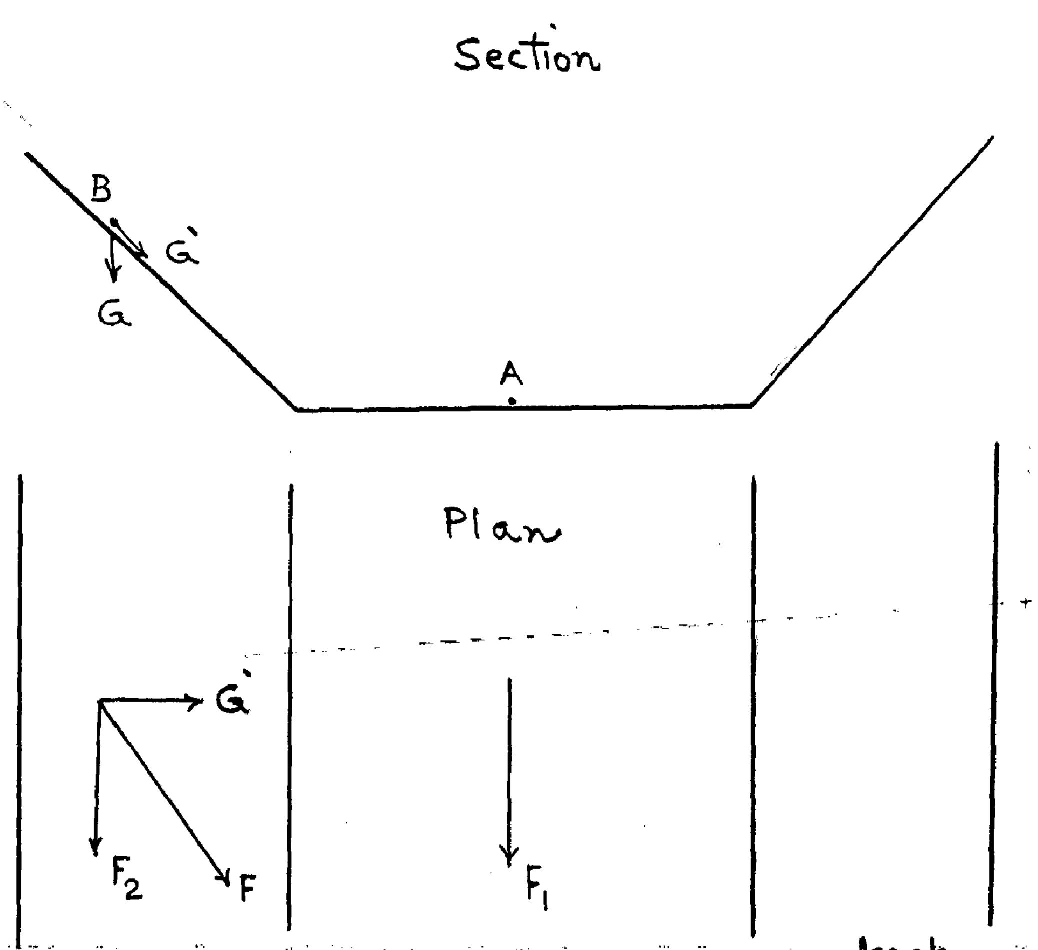
The defination of stable channels as per Lane is " A stable channel is an unlined earthern channel for carrying water, the banks and bed of which are not scoured by moving water and in which objectionable deposits of sediment donot occur". Sediment has been defined as fragmental material transported by suspended in, or deposited by water or accummulated in beds by any other natural agents.

There can be in general three classes of ~~mak~~ unstable channels - (i) Channels where banks and bed are scoured without deposits. This will occur when the water is either clear or the sediment carried is very small in quantity. (ii) Channels where deposits occur without much scour. This will occur from the sediment brought into the canal with flowing water or scoured from the banks bed upstream. (iii) When channels have objectionable scour and deposits, both present at the same time. For prevention of unstability of the first class an analysis of scouring action is necessary. For the second case it is necessary to ensure that the sediment brought in the canal is carried upto the downstream end. The third case is the combination of both.

To be stable from hydraulic stand point a canal in erodible material must not scour on the sides and bottom and objectionable deposits of sediment should not take place. In order that the bottom and sides may not scour, it is necessary that the conditions of flow at all points on the wetted perimeter ~~composing~~ ~~the banks~~ of the canal be such that the particles composing the banks and the bed are not displaced and to prevent sediment deposi~~ting~~tion, the flow should be such that sediment is not allowed to deposit but it flows till the end of canal. One important point in design of channels carrying large sediment is to investigate the hydraulic factors of canal which will have a transporting capacity sufficient to carry the material introduced at the upper end.

We shall now try to see as to what forces cause scour on canal banks and bed. Scour on the banks and bed takes place when particles composing the side and bottom are acted upon by forces sufficient to cause them to move. When a particle is resting on a level bed of canal, the force acting to cause motion is that due to the motion of water past the particles. If scour in bed is to be prevented the motion of water must not be rapid enough to produce forces on the particle sufficiently large to cause it to move. If a particle is on a sloping side of a canal, it is acted upon not only by the velocity of water but also by the force of gravity which tends to make it roll or slide down the slope. The force tending to cause the downward motion is the component in the direction of side slope, of the force of gravity acting on the particle. If the resultant of the force due to the motion of water and component of the force of gravity acting on the particle is large enough than the resisting power of particle, motion of particle takes place.

This can be illustrated like this. Consider a particle on the perimeter of the cross section of ~~xxx~~ canal is level, the motion



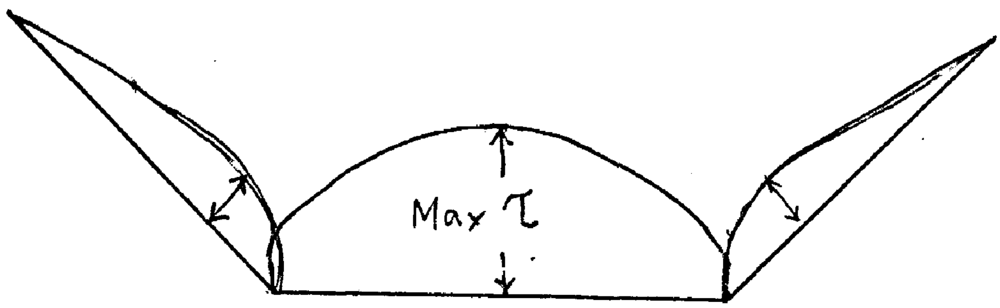
of particles at A will occur when water moves ~~fast~~ past the particles with sufficient velocity to produce a force F, large enaough to cause it to role or slide longitudinally down the canal. At B, on sides, the force of water F_2 will act in the longitudinal direction and the force of gravity G will have a component G' in the direction of side slope. Motion at B will occur when the resultant F of the longitudinal force F_2 and the gravity component G is large enough to cause motion.

The movement of material on the banks and bed of canal depends upon the steepness of side slopes and the velocity distribution near the banks. The forces due to the slope of sides are easy to analyse but the velocity near the banks is very difficult to determine. Secondly the presence of boundry layer cannot be ignored which makes the problem still complicated.

Tractive force as explained by DuBoys, is the force which the water exerts on the perimeter of the canal due to its motion and in the direction of motion. It is a force exerted on a certain area and not on any single particle. This tractive force is equal and opposite to the force exerted by the bed material on the flowing water. These two forces being equal and opposite there is no acceleration to water. If the bed material were not to exert any force on water, it would flow as a frictionless ball on an inclined plane. In a channel of constant slope where the water is flowing steadily, there is no accelatation because the force causing motion is equal to the force preventing motion.

Tractive force is the component of weight of water in the direction of flow. In a channel of infinate width and length with uniform slope, the tractive force exerted by the water on one square foot of area is the component of weight ~~of water~~ in the direction of flow. The weight of water is wD and its component in the direction of flow will be wDS.

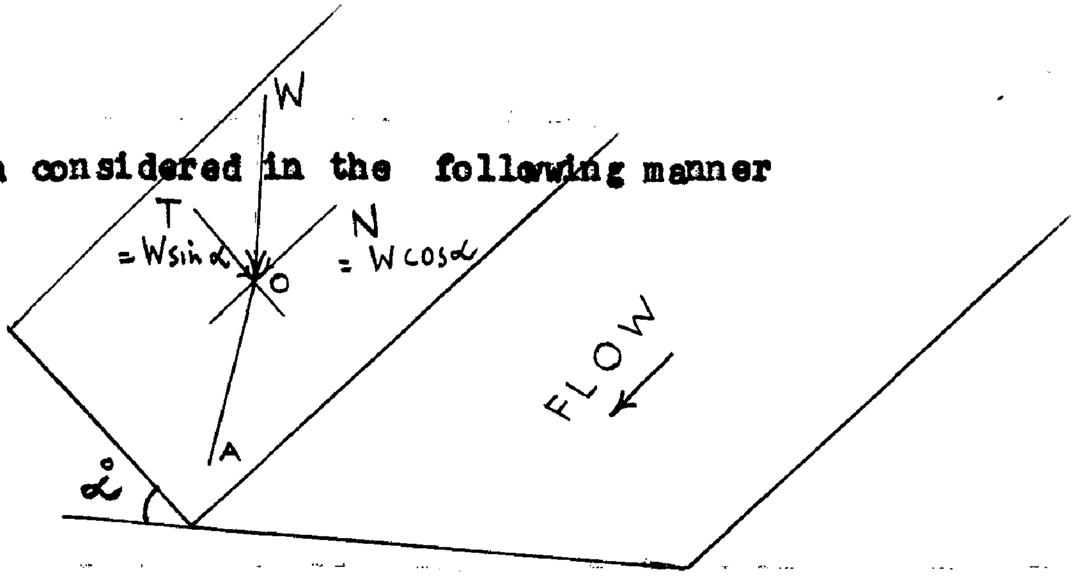
In trapizoidal canals the tractive force is not uniformly distributed over the bed and banks but approximate to that shown as below



In the present studies, Lane has assumed that the tractive force distribution will be similar in all canals having the same B to D ratio and the same side slope. So if the tractive force distribution for one section is known then it can be assumed that the distribution will be similar in similar cross sections. After an exhaustive study of all available data of velocity distribution and the sheer ~~shear~~ force distribution, Lane arrived at certain limiting values of tractive force on sides and bottom and plotted figures (4). His results indicate that for trapizoidal channels of the shape ordinarily used in canals the maximum sheer force on the bottom would be close to the value of wDS and on the sides the maximum tractive force will be 0.76 wDS.

To arrive at the limiting values of tractive forces Lane not only did laboratory experiments but he also observed the canals, located, where the Rio Grande leaves the mountains and flows out on an alluvial cone. The materials in this section decreased in size from the appex outwards and provided canals in the materials of a wide range of size. These canals were straight, stable and regular in cross sections and were steep enough to give high velocities and tractive forces. They gave him a complete opportunity of observation on prototype canals built in graded material. Fifteen different reaches were observed by Lane having discharge from 17 cusecs to 1500 cusecs and having longitudinal slopes of 4.2' to 51' per mile. The results of these observations are plotted in fig (4).

The effect of side slopes on the limiting tractive force has been considered in the following manner



The particles on the side slopes are subjected to two forces, one is the force of water tending to move the particle down the canal in the direction of flow and the other is the force of gravity having its component in the direction of side slope tending to move the particles down the sloping side. By combining the two actions the effect of the slope of sides on the critical tractive force necessary to cause motion can be calculated. In the above fig. let W be the weight of a certain volume of material located on the surface of a side bank the slope of which is α. The force W can be resolved into its normal and tangential components $N = W\cos\alpha$ and $T = W\sin\alpha$. These two equations would have supplied a solution in still water but the canal is carrying certain discharge and so in addition to T and N we have to consider the effect of tractive force τ . It now follows that in this case, if there is back of equilibrium due to tractive force the trajectory of a moving soil particle will not be the line of maximum slope falling in the planes perpendicular to the canal axis but on inclined route as O A . It is obivous now that for computing the stability, we must equate the friction force $N\tan\phi$ to the resultant $\sqrt{\tau^2+T^2}$ of the two forces τ and T. Since the tractive force is proportional to the depth D and slope S and since the slope S is constant for a certain range we will have $\tau = \omega DS$ where ωS is constant.

$$\sqrt{\tau^2 + T^2} = N \tan\phi$$

$$\therefore \quad \sqrt{\tau^2 + W^2 \sin^2\alpha} = W \cos\alpha \tan\phi$$

where ϕ is the angle of repose of the materials of bed and bank and α is the side slope of bank.

Now substitute τ_c instead of τ giving the critical tractive force and we have

$$\tau_c^2 + W^2 \sin^2\alpha = W^2 \cos^2\alpha \cdot \tan^2\phi$$

$$\tau_c^2 = W^2 (\cos^2\alpha \tan^2\phi - \sin^2\alpha)$$

$$\therefore \tau_c = W\sqrt{\cos^2\alpha \cdot \tan^2\phi - \sin^2\alpha}.$$

This equation holds good for all values of α the side slope. If $\alpha = 0$ then

$$\tau_c/W = \tan\phi$$

which means that the grannular material forming the bed of canal will remain stable so long as the resultant of the tractive force and the weight will remain within limits defined by the angle of repose.

For practical application Lane tried to determine quantitatively the ratio between the critiaal tractive force for side slopes and critical tractive force on the bed. Combining we have

$$\frac{\tau_c \text{ (for slope)}}{\tau_c \text{ (for bed)}} = K = \frac{\sqrt{\cos^2\alpha \cdot \tan^2\phi - \sin^2\alpha}}{\tan\phi}$$

Or
$$K = \cos\alpha\sqrt{1 - \frac{\tan^2\alpha}{\tan^2\phi}}$$

For convenience in design K has been worked out as the ratio between the critical tractive force on sides to that at the bottom. Lane has prepared a diagram giving a graphical representation of this equation. See fig. (5). For example in a material whose angle of repose if 35°, the critical tractive force which would move the material on the side of a canal with $1\frac{1}{2}:1$ side slope will be 0.28 times that which would cause motion on a level bed.

Lane has given another diagram (7) which gives the angle of repose ϕ as a function of the shape and size of particles. By means of these two diagrams the problem of tractive force on side slopes, so far as scour is concerned, is completely solved.

Now the problem is to find out the tractive force on the bed. The — basic parameter for immovability of the grains, ~~as~~ opposed to the critical tractive force is the size of the grains d. For the ~~material in the canal~~ purpose of design, Lane has devided all the materials in three types ~~(2)~~ which require different methods of analysis (1) Coarse and noncohesive material (2) fine noncohesive material (3) Cohesive material. For the design of canals in coarse noncohesive material, the action of particles rolling down on sides should also be considered in addition to limiting tractive force as the bottom. Where the canal is constructed in cohesive material the particles are prevented from rolling due to cohesion. Canals in fine noncohesive material are intermediate between the two. In this case the effect of small amounts of cohesive sediment in the water or in the material through which the canal flows, becomes important.

A plot fig. (6) has been prepared showing the limiting value of tractive force for design of channels in coarse and noncohesive material based upon the data of San Luis Valley Canals. The line A on the figure represents relation; tractive force in pounds per square foot equals 1/2 the diameter of particles in inches such that 25 per cent of the material is coarser. Though the line A represents nearly the true values of tractive force in canals, there is not enough factor of safety for use in design and ~~hemme~~ hence line B is drawn tentatively such that the limiting tractive force in pounds per square foot equals to 0.4 the diameter in inches for which 25 per cent of the material is larger, and line B is recommended for design purpose. One thing should be noted that all these relations are meant for only straight canals and ~~xxxx~~ so these should not be applied to canals with bends and curves. Second thing is that the materials analysed for fig.(6) has a specific gravity of ~~xxxx~~ 2.56. If it is used for materials having different specific gravity, the tractive force for a given size as indicated by the diagram should be multiplied by the ratio of the unit weight of the material submerged to the unit weight of material having specific gravity of

2.56 when submerged. In the cases of coarse material the ~~nit~~ net ~~width~~ weight with voids filled with water should be used.

An example will clasify the matters. Suppose a canal is to be built in slightly angular material 25 per cent of which is one inch or over in diameter and that the canal water section has a 10 feet bed width and 5 feet depth with 2.1 side slope.

$$B = 10'$$

$$D = 5'$$

$$\text{Side Slope} = 1:2$$

$$\therefore \frac{B}{D} = \frac{10}{5} = 2$$

The maximum tractive force on the bottom of canal with B/D = 2 is shown by fig.(4) to be 0.89 wDS.

No motion will occur on the bottom if this 0.89 wDS does not exceed the limiting value for the material.

The limiting value of tractive force for material of which 25 per cent is over 1 inch in diameter is shown by fig. (6) as 0.4 pounds per square foot.

The limiting longitudinal slope of canal is therefore

$$S = \frac{0.4}{0.89\, w \cdot D}$$

$$= \frac{0.4}{0.89 \times 62.4 \times 5} = 0.00144$$

The safe angle of repose of slightly angular material of 1" dia is shown by fig.(7) to be 36°.

For side slopes 2.1 and angle of repose 36" the safe tractive force on side slope is 0.64 of that on the bottom, or that is $0.64 \times 0.4 = 0.26$ lbs/sq

The maximum tractive force on the sides of a trapizoidal channel with $B/D = 2$ and side slope 2:1 is from the fig (4)u 0.76 wDS.

$$S = \frac{0.26}{0.76 \times \omega \times D}$$

$$\therefore \quad = 0.00108$$

Since the limiting slope for sides is smaller than that for the bottom, the former would control the design and hence a slope of 0.00108 should be adopted in the above case.

The above method of channel design evelved by Lane is an entirely new line of attack to the problem of stable channel design. In this solution empericism goes hand ink hand with theoretical analysis. The emperical part is confined to the assumptinn that the design of canal is a problem of tractive force, but the calculation of that force and its permissible limits is a matter of clear cut theory based on laboratory experiments. This method has been very recently evolved and it has not been in practice. Once a few canals are constructed on this basis many more things will be explored.

Lane's approach to the problem of channel design is, indeed, a great advance in stable channels. First of all he has studied the entire litarature available on the subject, both emperical and theoretical and prepared a combined diagram showing relations of bed width and depth for the important emperical formulae (~~fig~~). Secondly he is probably the first person who has tried to analyse and account for all the fundamental factors which control the shape of stable channels, and discuss as to what extent each of the factors be given importance.

One factor seems to have remained unnoticed in the above list and that is the effect of seepage into or out of the canal. Seepage may have an appreciable effect on the resistance of canal banks to scour by flowing water. The seepage out of canals produces forces on the particles which tend to hold them down on the bed or on the side slope and thus make them less subject to movement by the force of water. Similarly seepage into the canal has an opposite effect since it tends to lift the particles and thus make them move more readily. So seepage effect, the author feels, cannot be ignored in all channels in noncohesive material. There is no doubt that in the first few days after starting a new canal, the finer sediment flowing along with the water might fill up the void spaces of slightly bigger size of material and thus form a sort of an impervious layer on the perimeter. But it has been observed that in all canals in alluvial soils the seepage losses are heavy and so seepage cannot be neglected from our discussion.

While analysing the tractive force on the bed and bottom Lane says that velocity distribution plays an important part in the distribution of tractive force but as its determination is rather complicated, ~~and so~~ he has entirely left it out. There is a boundry layer present which also plays an important part in velocity distribution which has also been completely omitted.

Again there are eddy currents present in a channel section, the vertical components of which try to keep the flowing material in suspension. In their papers both Kennedy and Lacey have mentioned that sediment in a flowing canal is kept in suspension soley by the vertical components of the constant eddies which can always be observed in any channel boiling up gently to the surface. The eddies rise due to the roughness of the bed and sides and work up against the depth of channel. So these eddy currents too must be playing a significant part in channel design which seems to have slipped Lane's attention.

The basis of calculating the tractive force on bed and side slope needs some changes. The particles on the side slopes are not only acted upon by two forces, one, the flow of water and the other, the component of weight in the direction of slope which tries to slide the particles along the side slope, but there is a third force of seepage which will try to keep the particles in their position in case of seepage outflow and which will try to expediate the motion in sloping direction in case of seepage in flow. Hence it is necessary to calculate the pattern of seepage in the bed and slope and properly incorporate it in their derivation of tractive force formulae.

The calculation of shear force has been done on the assumption that the pattern of tractive force will be similar in similar cross sections of channels. Under this assumption all canal sections having the same B to D ratio and the same side slope will have the same tractive force. This assumption does not seem quite correct to the author. The tractive force or the shear force directly depends upon the velocity distribution in a channel cross section. When we admit this fact it is evident that unless the velocity pattern is similar in two similar cross sections, the tractive force distribution cannot be similar. For velocity to be similar the longitudinal or bed slope too should be similar. Secondly it is something like a model and a prototype. Studies on hydraulic models specially the movable bed models have shown that the experiments on models give just qualitative results and not quantitative. Sometimes the flow pattern becomes entirely different, for instance on a small model a particular flow if reduced in proportion changes the type of flow altogether i.e. a turbulent flow becomes a laminer flow which is governed by the viscos forces. Hence the author feels that though the assumption is good to start with, it has got to be seen in the light of the above discussion and suitable coefficients will have to be suggested for tractive force in similar cross section depending upon the discharge and bed slope.

Lane has prepared a diagram for the angle of repose of different materials. It is not mentioned whether the values have been observed in submerged conditions or not because submergence will have its effect on the fine non-cohesive material though its effect might be less preominent in materials of bigger size. Secondly the angle of repose for only a certain size and diameter are given but in nature, most of soils are found to have particles of various sizes and in that case the angle of repose diagram may not hold good. This fact needs an urgent attention otherwise Lane's methods of design may not be applicable in any practical case. Thirdly, consideration whether a material is very angular, moderately angular or slightly angular and very rounded, moderately rounded and slightly rounded is entirely left to the entire discretion of the user which brings in all possible mistakes. A material which has been designated by one as slightly angular can be called moderately angular by another and thus there is no basis of correct judgement.

One more thing about the side slope of a channel. Lane considers that the stable channel will have a side slope as per the angle of repose of material of which the bank is composed. Experience in India, of many eminent channel experts like Kennedy, Lindley and Lacey shows that a stable channel assumes a semi elliptical shape. The channels observed by Kennedy were reported to be having practically vertical sides and horrizontal bottoms. Lacey has stated that natural silt transporting channels have a tendency to assume a semi elliptical shape with major axis horizontal and with the ratio of major axis and semi minor axis depends upon the nature of silt carried, being greater for coarser material and vice versa. At present the pprocedure of design in a new canal system is that though the excavation or embankment is done as per the slope to which the soil can stand yet the calculation of discharge are done with the assumption of 1/2 to 1 side slope. It has been observed that the banks ultimately assume appeoximately side slope of 1/2 to 1, after they become stable or attain what is known as the initial regime. This matter should be considered by Lane.

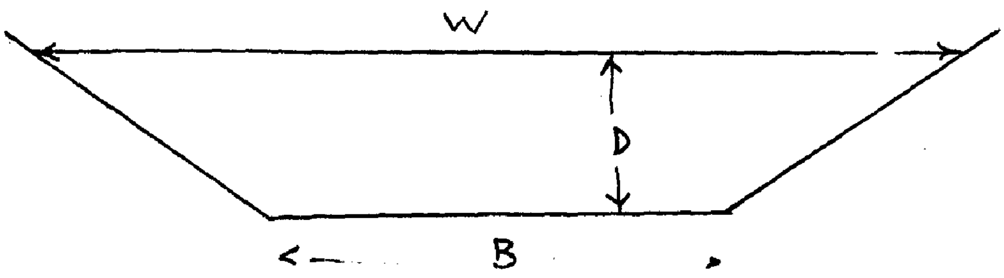
In the concluding paragraphs of his paper Lane has stated that the effect of bend is under study. No one has yet taken notice of bedds in studying channel shape. Kennedy has selected a few straight reaches and so also Lindley. In practice it has been observed that there is a great scour on the outer side of a curve and silting on the inner side. Scour can be reduced by lowering the velocity of flow on curves which can be accomplished by using larger canal areas but this would result in an increased cost. It will often be economical to allow scour to start and then to stop it by protecting the banks at points of scour. The author feels that some special sections should be provided at bends with the consideration of a likely scour. Some method should be found to estimate the scour at bends depending upon the curvature discharge and the material in which the canal is to be constructed. The section should be such as to take care of the proper velocity distribution on bends. Between the section at bend and the section on the straight portion some smooth transition should be provided so as to keep the scour to a minimum. This should be tried first on small scale and seen its possibility of application in practice.

Lastely we have to consider the case of heavy sediment flowing in a canal. This becomes a theoretical problem of sediment transportation. Much work has been done as we have seen in Chapter III but more remains to be done which would give some basis of design. We have to find out the ~~xkwx~~ laws which will give conditions of flow such that the sediment introduced in the canal at the upper end should be carried till the end of the canal without any deposition whatsoever at any place. At this point we are able to appreciate the importance of the theoretical studies of sediment transportation which might be needed to be intwoduced in a rational design of stable channel.

SHAPE OF STABLE CHANNEL IN ALLUVIUM

The only investigator who has attempted ~~to~~ a closer definition of the shape of the stable channels is Lacey. His statement ~~He says~~ that natural silt transporting channels have a tendency to assume a semi-ellipitical section, is confirmed by an inspection of a large number of channels in final regime and by an examination of the cross section of ~~discharge silts of~~ rivers having to well defined straight reaches of known stability.

In an article in the Central Board of Irrigation Journal Lacey has tried to give some reasoning on his insistance of elliptical shape. He says that for a given water surface and a given area the section of least work would be a semi ellipse. A number of observations were made by him on actual canals which had attained certain stability. He called the ratio of mean depth to depth as the shape factor. Mean depth was calculated by him by dividing the area by the bed width. This he did because he considered the side slope to be of 1/2 to 1 inclination and hence the channel approximated to a rectangular section. The shape factor observed by him varied from 0.7 to 0.9. The reason for variation was due to the silt distribution and size of sediment particles. In the figure



If W is the width at water surface, D is the depth and B the bed width then Shape factor Z

$Z = Dm/D$ where $Dm = A/B$

$$Z = \left(\frac{W+B}{2}\right).D.1/B.D$$

$$= \left(\frac{W+B}{2B}\right)$$

Let $Y = W/B$

then $Z = 1/2\,Y + 1/2$

or $Y = 2Z - 1$

If Y = 0 then Z = 0.5 the shape is triangular

" Y = 1 " Z = 1 the shape is rectangular

But for Z = 0.7 to 0.9 the shape works out to semi-elliptical approximately. Thus we see that the values of shape factors obtained by Lacey on examining the regime canals give the shape of semi-ellipse. If the bed material is coarser the elliptical section flattens and if the bed material is finer the section becomes steeper. So Lacey concluded that stable channels would be semi elliptical with the major axis horizontal and the ratio of major axis and the semi minor axis is greater or smaller depending upon the nature of silt carried. Lacey says that it should be possible to keep a horizontal bed and yet adopting a proper value of Y by varying side slopes to obtain a desired value of Z. There is no need of adhereing to a particular side slope as it may change with the changed value of Z. If such a canal is constructed there will be some scour on side but no scour on the bottom. After a couple of seasons the section should become stable.

Lane in his paper on "Stable Channels in Erodible Material" says that his investigations donot support Lacey's conclusions. A convenient way of comparing channel cross sections is by means of ratio which may be called the "form factor" between the area of the channel section upto the water surface and the area of the enclosing rectangle. For an ellipse this ratio would be = 0.79, for a parabola 0.67, for a triangle 0.5 and for a rectangle it would be 1. A study of a large number of cross sections of channels has disclosed ratios from 0.56 to 0.92. The stable channels observed by Kennedy are reported to have practically vertical sides whereas those seen by Lacey were semi alliptical.

Pettis C.R. in the discussion on Lane's paper mentioned in the above paragraph says that he had investigated the shape of natural stable channel which would best comply with the known hydraulic conditions. The evidence indicated that the cross section should be of parabolic shape where D varied as B^k. The value of k was not less than 2 and not more than 3 and the value best suited was 2.5. A semi elliptical shape was not an ideal shape for channels. He derived many formulae which have a striking resemblance to those of Lacey's formulae. He termed the ratio of R/D as the form factor which for a parabolic section was 0.715. Consequently R = 0.715 d.

Pettis says that ideal stable river cross section is a curve of a type in which D varies as $B^{2.5}$. It is characterstic of rivers that they are subjected to very variable flows. No given cross section can be theoretically perfect for more than one value of Q. Since there is certain range of stability such a channel will be safe for a certain amount of overflow. If the overflows are frequent or high the original shape may be modified slightly to meet this condition. When the water falls below full bank stage, such a channel will remain within the limits of stability, for flows at medium stages. At low stages there will be a tendency to silt along the sides, which may cause irregularity of flow and which may be followed by some scouring. The ideal stable channel is one which would remain stable for all ordinary conditions.

Just what shapes are produced for stable channels by all the variety of considerations have not yet been determined. The author is of opinion that the shape of stable channel cannot be correctly ~~done~~ studied in a laboratory because similar conditions cannot be reproduced on a model. For the study of the shape of alluvial channels, all the present stable channels should be inspected and correct cross sections be found out at various places. Other data of climatic conditions and geological conditions too should be recorded. Cross sections on the curves should be asscertained more carefully and a series of ~~emptions~~ sections should be taken at smaller interval from the straight reach upto the curved section and then from the curved section upto the straight section. This will indicate the type and pattern of changes, the section undergoes due to curvature. The author holds a strong view that the cross sections on the curves and bends should neither be rectangular nor be trapizoidal but should have a section which takes into account the likely scour on the curve. From the straight reach to the curve a certain smooth transitional sections be provided if scour is to be avoided. He feels that an approximately semi-elliptical section will be suitable in straight reaches and a parabolic section should be best suited on bends.

The above discussion takes into account the section after a certain regime or stability has been attained but when a new excavation is done this sor-t of ideal section cannot be adopted because of many other difficulties. A loose grannular material would neither stand on semi elliptical section nor on parabolic section. It will have to be constructed with the angle of repose in view. Due to original compactness of earth, the sides can be given a steeper slope but in embankment the slopes will never stand steeper than the angle of repose. This brings in a rather passimistic view that a definate shape cannot be obtained.

The problem of channel shape should be attempted with the new background of soil mechanics. Most stable slopes to which an earth composing of different sizes of material can stand, be analysed in a soils laboratory. Then the porosity of such a mixture of soil and permeability can be known and with all the above soil consierations the shape problem should be attacked. As far as the water is concerned the author feels that its surface tension, capillarity and adhesion might also be playing a part in the determination of the section. So the problem should be studied separately in the laboratory on the soil mechanics consideration and in the field by observing the existing stable channels both the straight and curved reaches.

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COMPARATIVE STUDY OF THE DIFFERENT METHODS OF CHANNEL DESIGN

In the previous chapters we have seen the different methods of channel designs and have discussed the merits and demerits of each method. Here it is proposed to take up one numerical example which will be solved by different methods so that it will be easy to compare one with the other.

Suppose we have to construct a channel for a discharge of 900 cusecs. The section has to be trapizoidal with side slopes 1/2 : 1 as usually assumed and let the grade of material be such as to have a regosity coefficient of N = 0.0225 and a silt factor f = 1.

Kennedy's Method.

In Kennedy's method, we have to assume certain dimensions of channel and then see whether it can pass the required discharge with CVR near about unity. In this case let us assume a bed width of 73' and depth of 5' with a longitudinal slope of 1/5684.

$$A = (B + \cdot 5 D) D = 377 \cdot 5 \text{ sft}$$

$$P = B + 2 \cdot 24 D = 84 \cdot 18 \text{ ft}$$

$$R = \frac{A}{P} = \frac{377 \cdot 5}{84 \cdot 18} = 4 \cdot 48 \text{ ft}$$

P.T.O.

From the Kutter's formula the value of C works out to 85.66 with the value of N = 0.0225, R = 4.48 and S = 1/5684.

$$V = C\sqrt{RS}$$
$$= 85.66\sqrt{4.48 \times 1/5684}$$
$$= 2.4'/sec$$
$$Q = A \cdot V = 906 \text{ cusecs}$$

So the section having a bed width of 73' and a water depth of 5' will be able to pass the required discharge. For this depth the Kennedy's pritical velocity will be

$$V_o = 0.84\, D^{.64}$$
$$= 2.35'/sec$$
$$\text{C.V.R} \quad V/V_o = 2.4/2.35 = 1.02.$$

This shows that the C.V.R.is near about unity and hence the section could be safely adopted.

Hence the values of the channel dimensions as per Kennedy's method are

$$B = 73'$$
$$D = 5'$$
$$V = 2.4'/sec$$

<u>Lacey's Method.</u>

In Lacey's method it is just sufficient to know the discharge and the silt factor by which all other elements can be calculated.

$$P = 2.67\sqrt{Q} = 80'$$
$$R = 0.4752\,(Q/f)^{1/3}$$
$$= 4.56'$$

By calculation we get $D = 5.13'$ and $B = 68.5'$

$$S = \frac{f^{5/3}}{1830\ Q^{1/6}} = 0.000176 \text{ or } 1/5680$$

$$V = 0.7937\, Q^{1/6} f^{1/3} = 2.47'/\text{sec}$$

otherwise also $V = Q/A = 2.47'/\text{sec}$

Hence by Lacey's method for the same data we get the following values

$$B = 68.5'$$
$$D = 5.13'$$
$$V = 2.47'/\text{sec}$$

This example can be very easily and quickly solved by Lacey's diagrams which are very commonly used for the designing of channels.

Lane's Method.

For solving the above problem by critical tractive force, one diagram which gives a critical tractive force for a particular diameter of particles, will have to be used, The other diagrams of Lane mentioned in chapter V cannot be used because the values of tractive force for the particles size and side slopes of the present problem are not shown therein.

Now let us assume a slope of 1/5684 and a silt factor equal to 1 which will give the particle diameter as 0.323 m.m,

$$f = 1.76\sqrt{d_m}$$

$$d_m = \frac{1}{(1.76)^2} = 0.323 \text{ mm}$$

From the diagram (fig 2) the critical tractive force for a particle size of 0.323 m.m. is given as 0.055 lbs/sq' . As the tractive force is equal to wRS, we have

$$R = \frac{0.055 \times 5684}{62.4 \times 1} = 5.01'$$

Velocity by the Manning's formula is

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

$$= 2.56'/\text{sec}$$

$$A = Q/V = 352 \text{ sft}$$

$$P = A/R = 70.2'$$

$$\text{Then} \quad \left.\begin{aligned} (B + .5D)D &= 352 \\ B + 2.24D &= 70.2 \end{aligned}\right\}$$

Solving the above equations we have
$D = 5.75$ and $B = 58'$.

These values give an area of 354 sft and with a velocity of 2.56'/sec a discharge of 900 cusecs can be easily passed through the section.

Hence the values of channel elements as per critical tractive force method are

$$B = 58'$$

$$D = 5.75'$$

$$V = 2.56'/\text{sec}$$

upto P 88.

Einstein's Method.

In this method there are many elements to be determined or assumed to be used in the expression

$$\frac{q_s S_s}{\sqrt{g(S_s-1)}\, F d^{1.5}} = 40\left[\frac{\tau}{\omega(S_s-1)d}\right]^3$$

where

$$F = \sqrt{\frac{2}{3} + \frac{36\nu^2}{g d^3 (S_s-1)}} - \sqrt{\frac{36\nu^2}{g d^3 (S_s-1)}}$$

For Ss equal to 2.64 and kinematic viscosity of water as 0.01 $cm^2/sec.$, the values of F work out to 0.81 which can be safely used for ~~[illegible]~~ particle diameter of ~~12~~ 1 to 10 m.m. From the previous equation we have

$$q_s = \frac{40\, F \sqrt{g}}{\omega^3\, S_s\, (S_s - 1)^{2.5}} \, \frac{\tau^3}{d^{1.5}}$$

for $F = 0.81$, $\omega = 62.4$ lb/cft, $g = 32.2'/\text{sec}/\text{sec}$ and $S_s = 2.65$
and $\tau = \omega R S = \omega D S$ for wide shallow streams
we get

$$q_s = 106{,}000 \, \frac{D^3 S^3}{d^{1.5}} \quad \text{(in f.p.s. units)}$$

$$= 349000 \, D^3 S^3 / d^{1.5} \quad \text{(in c.g.s. units)}$$

And Discharge $q = 1/N \cdot D^{1.67} S^{0.5}$ (in c.g.s units)

$$q = \frac{1.486}{N} D^{1.67} S^{0.5} \quad \text{(in f.p.s. units).}$$

and

$$q_s/q = 1070 \, \frac{D^{1.33} S^{2.5}}{d^{1.33}} \quad \text{(in f.p.s units)}$$

which expresses the rate of sediment transported in terms of depth, slope and mean diameter of sediment. For practical application of the formula in design of channels, the surface width is first chosen arbritrarily. Then with known values of q and q_s and d the two unknowns D and S can be determined from equations A and B. Different sets of D and S will suit different values of B from which the most appropriate one can be chosen with a view of topographic limitations of slope, minimum excavation etc.

Let us assume a width of 60' and a bed load volume of 0.025%.

$$q = \frac{900}{60} = 15 \text{ cfs.}$$

$$q_s = \frac{15 \times 0.025}{100} = 0.375 \times 10^{-3} \text{ cfs.}$$

and $$q = \frac{1.486}{N} D^{1.67} S^{0.5}$$

$$\therefore D = \left[\frac{qN}{1.486\, S^{.5}}\right]^{1/1.67}$$

Substituting the values and simplyfying the expression

we get $D = 5.5'/\text{sec}$

Now $$V = \frac{1.486}{N} D^{2/3} S^{1/2}$$

we get $\therefore V = 2.70'/\text{sec}$

$$A = (B + 0.5D)D$$

$$\therefore \quad \doteq 344 \text{ sft}$$

$$Q = 344 \times 2.7$$

$$\therefore \quad = 918 \text{ cfs.}$$

As the values assumed for ~~the~~ B give a satisfactory discharge this can be safely assumed for the channel. Hence by Einstein's method the different dimensions of channel are as under:-

$$B = 60'$$
$$D = 5.5'$$
$$V = 2.7'/\text{sec}$$

The above example worked out by different methods gives a comparative picture of all these methods.

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RECOMMENDATIONS AND CONCLUSIONS

The science of stable channel design has been developing over a long period. The most extensive and pioneer work was done in India, where extensive irrigation systems are located. Since the past two decades the work on stable channel studies has remained rather standstill in India, though other countries like United States are going ahead. The reason of this stagnation seems to be that the people of India are slightly tradition-minded and the methods of Kennedy and Lacey are being used for irrigation systems though in some places these methods are showing absured results. The other reason of stagnation in this direction seems to be the introduction of lined channels which are extensively being built on almost all modern irrigation systems. The lined channels are neither silting nor scouring because higher velocities can be allowed to see that silting does not take place and deposition never occurs. Scouring is out of question on a good lining material. But it should not be forgotten that lining of channels is prohibitively costely, which necessitates the construction of unlined channels. India has to build very large irrigation systems in the coming years and hence the irrigation engineers, both in the field and in the research laboratories should take up the problem of designing stable channels in all seriousness.

The work of stable channels should start in two different directions simultaneously. In the research laboratory basic research should be taken up to find out the laws of sediment transportation. Uptil now no work in this direction has been done in our country. So keeping in mind the work done in other countries, research engineers should begin their experiments on the soil samples available in our country. Work has been done on material of uniform grade and size to find out the laws of bed load movement. The author is of the opinion that the samples should be such as are available in nature so that they will give a correct picture of actual movement.

On the other hand, all the present channels which have assumed a certain stability due to age, should be surveyed and seen whether they could give any clue to this problem. Data of all the essentials should be collected afresh unlike Lacey to find the channel behaviour. The data should include the bed width, depth, slope, discharge, cross sections on straight and curved reaches, sediment characterstics such as grade and charge, geological conditions and temparature variations etc. Un-disturbed samples of soil should be taken from the soils where these stable channels are existing to see the permeability, compaction and other soil properties.

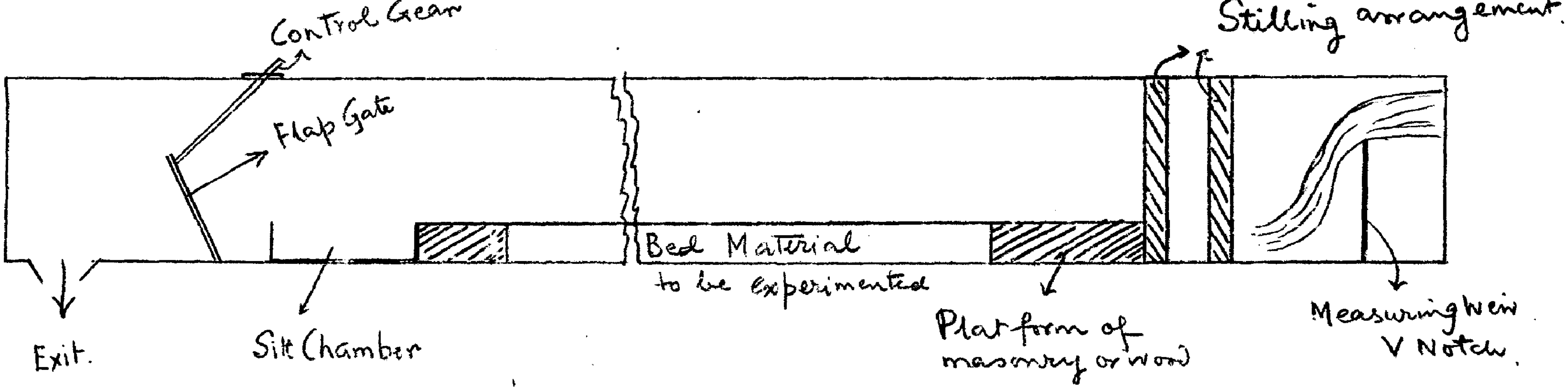
The author wishes to suggest the following laboratory experiments to be carried out which will add to the knowledge on the subject to a considerable degree; He feels that experiments carried out under controlled conditions in the laboratories are bound to give more useful information.

Experiment for Critical Tractive Force.

A series of experiments are required to be carried out for the determination of critical tractive force or the velocity at which the movement of bed material starts. Bed material should be sieved and properly graded from very fine silt upto as large a size of sand as can possibly be experimented upon in the laboratory flume. Each grade of material should be tested for its critical tractive force. The arrangement in the testing flume should be as shown on page 94. Measured quantity of water should be let into the flume which should pass through stilling arrangement and over the platform before it meets the bed material that is being experimented. The level of water can be controlled by means of a flap gate towards the end of the flume. Beforet the water goes out of the flume through the exist it will drop all the material carried by it in the sediment champer. The sediment chamber should be such that it could be taken out whenever wanted for measuring the quantity of sediment. The velocity of water can be controlled by the movement of the flap gate or by increasing or decreasing the discharge which is measured on the notch before hand. Two sets of readings of velocity, discharge and sediment loads be taken for each grade of material. Firstly when the ~~water~~ movement of bed material just starts with the velocity increasing and secondly when the movement of

EXPERIMENT TO STUDY CRITICAL TRACTIVE FORCE.

Not to Scale.



bed material just stops with the velocity decreasing.

While the experiment is going on a few additional observations should also be taken which will be of great help. They are the measurement of the quantity of bed load and sediment load movement. Just at the bed all the material which is in motion, is moving as bed load. At points above the bed the quantity in suspension goes on increasing as the depth of the point of observation increases. But beyond a certain depth the quantity in suspension again falls. It would be worth-while investigating the quantity of material carried in suspension at various depths starting from as near the surface of water and going down as near the bottom as possible. There are various methods of measuring the suspended load. One simple method is to syphon out samples at various depths and to measure the percentage of solids present in each sample. Another method is by passing a beam of light through different depths of moving water carrying suspended load and corelating the intensity of light of the beam at different depths with that through the plain water not carrying any sediment. The intensity of light can be determined by a photo-meter or by a photo-electric cell. The differenciation of bed load and suspended load can be done in much better way by this method. The third method could be of using a sediment sampler.

Determination Of Angle Of Repose.

Determination of the angle of repose for different soils is another important thing in stable channel studies. The angle of repose for soils which are completely submerged in water should be found for different sizes of soils and different mixtures of soils that are likely to occur in nature. Lane has carried out certain experiments on the angle of repose but they donnot cover all the required sizes and combinations of size. In a laboratory, the measurement of angle of repose should not be very difficult.

Work On Lane's Method.

The author feels that the work done by Lane at the U.S.Bureau of Reclamation on stable channels is of great signifimance although it has a few draw-backs which have been dealt fully while discussing the Lane's method of channel design. The diagrams given by Lane donot cover the range of Indian conditions and hence work in that direction will really be promising. The studies should include:-

1. The critical tractive force for different materials and different mixtures of material available in nature.
2. The angle of repose under water, for different materials and different mixtures of material.
3. Velocity and sheear force distribution in the channels in the straight as well as in the curved reaches. The effect of curvatures in channels on velocity distribution is very important and has not been studied uptil now.
4. The effect of boundry layer on the bed load movement.

5. Study of scour resistance of clay soils and its relation to the properties of clay involved in structural stability.
6. The effect of seepage on tractive force both on slide slopes and bed.
7. The shape of channel in different types of soils.

The author feels that if study on the above points is made some universal solution of stable channels can be found out.

The auther has tried to compile an up-to-date information on the studies of stable channel design and has tried to suggest future line of action in obtaining a perfect solution to the problem. He is confident that this thesis should serve to guide further research.

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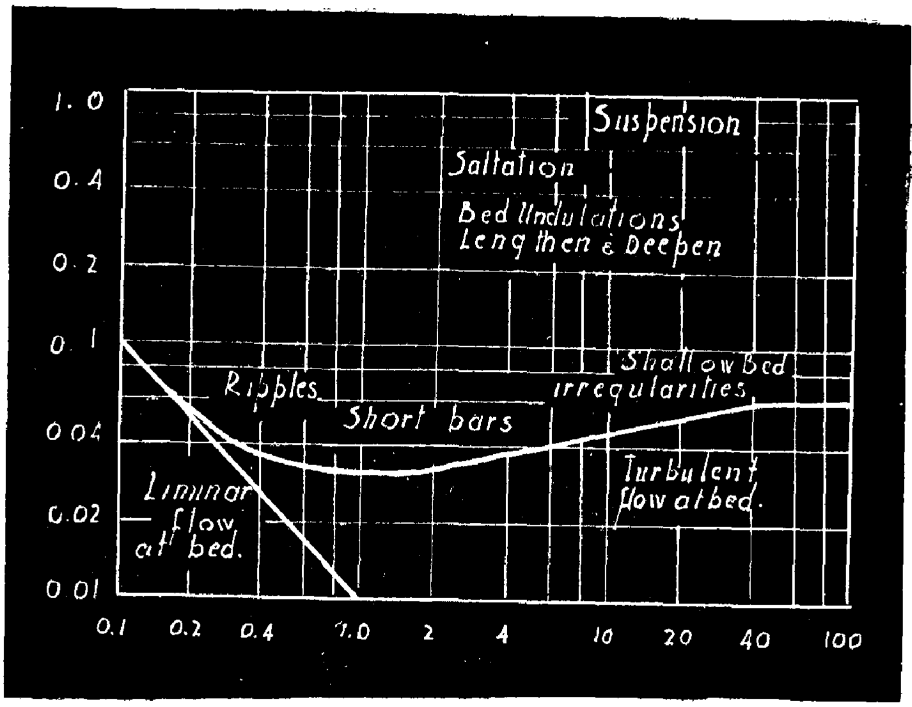
Fig 1

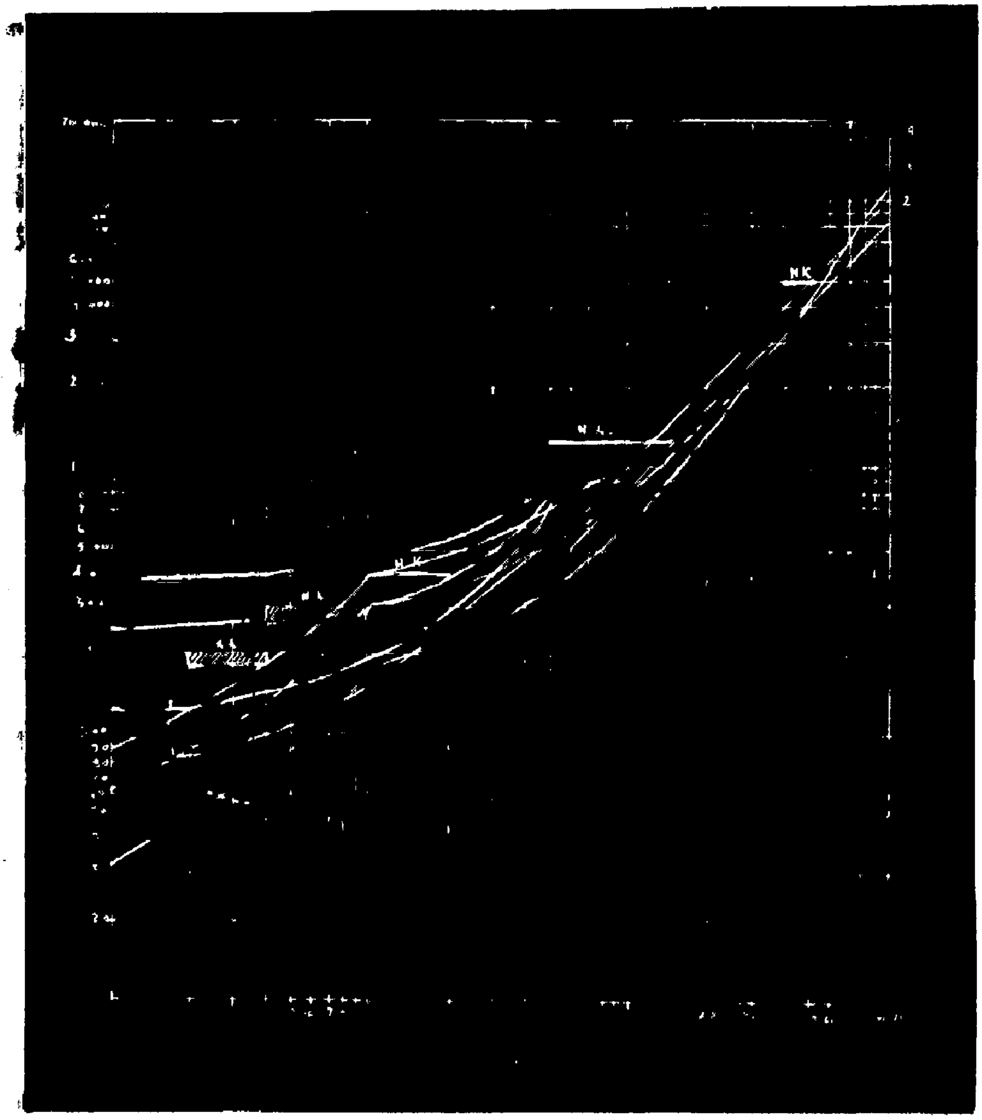


Values of $\frac{\tau}{\omega (S_s - 1) d}$

Values of $\frac{d\sqrt{\tau g/\omega}}{\nu}$ w?

Tractive force plotted against Rynolds Number

FIG 2



τ_c in gms/m^2

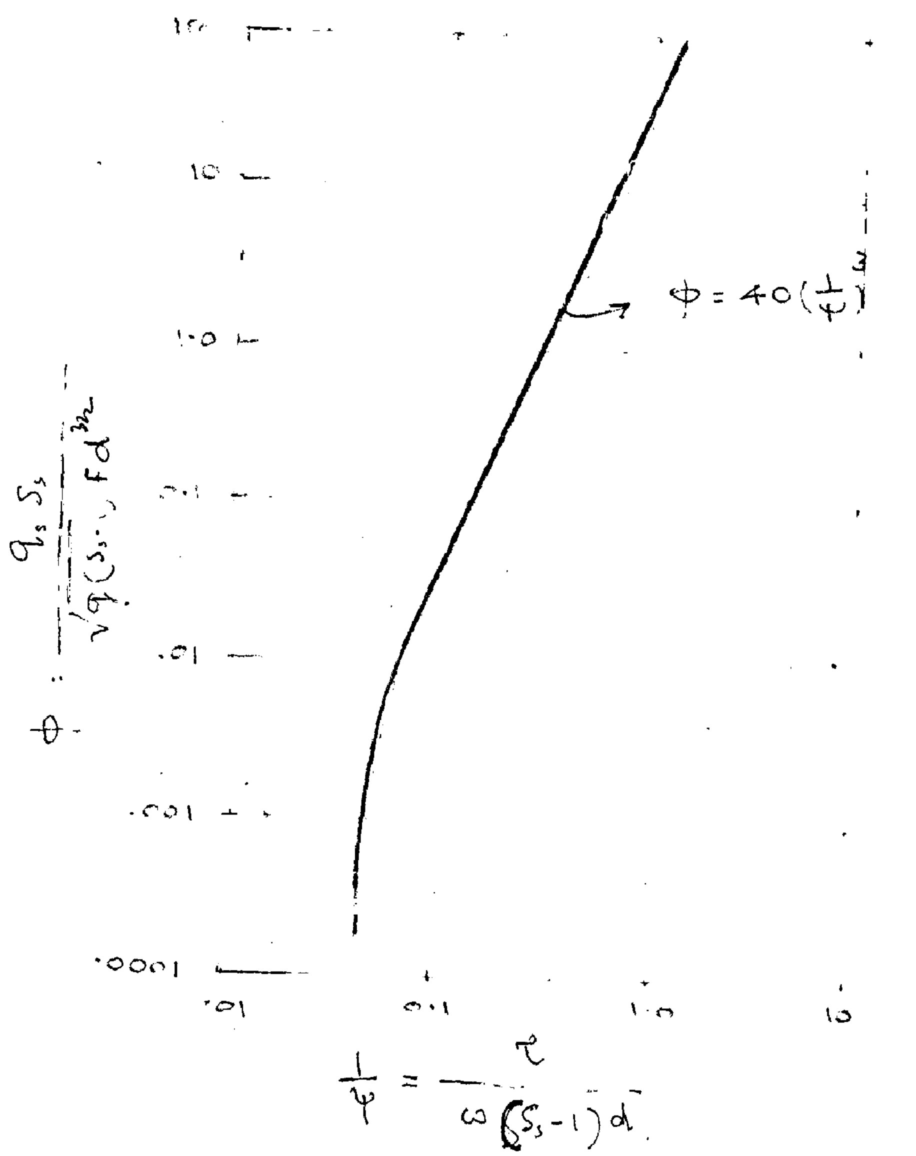
τ_c in lbs/ft^2

Mean Diameter in m.m

(1). For canals with high sed content
(2) " " " low " "
(3) " " " clear water

Limiting Tractive force Recommended for Canals

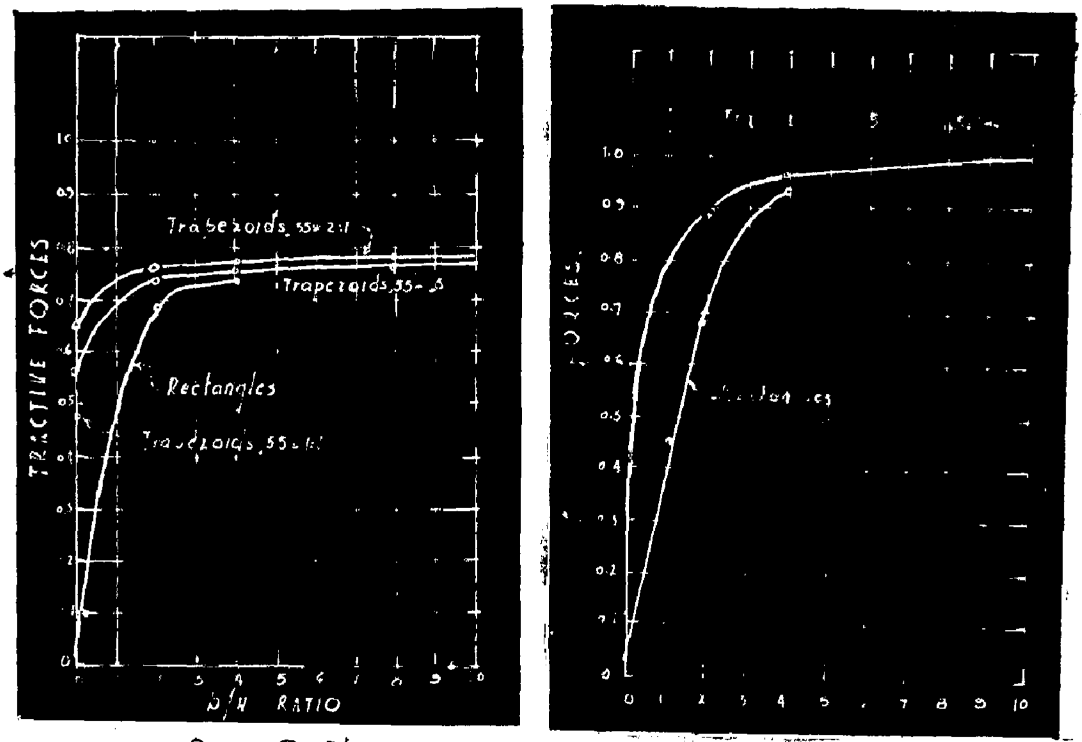
FIG 3



Plot of Einstein's Bed Load Function.

FIG 4.

0.76



B/D Ratio

B/D Ratio

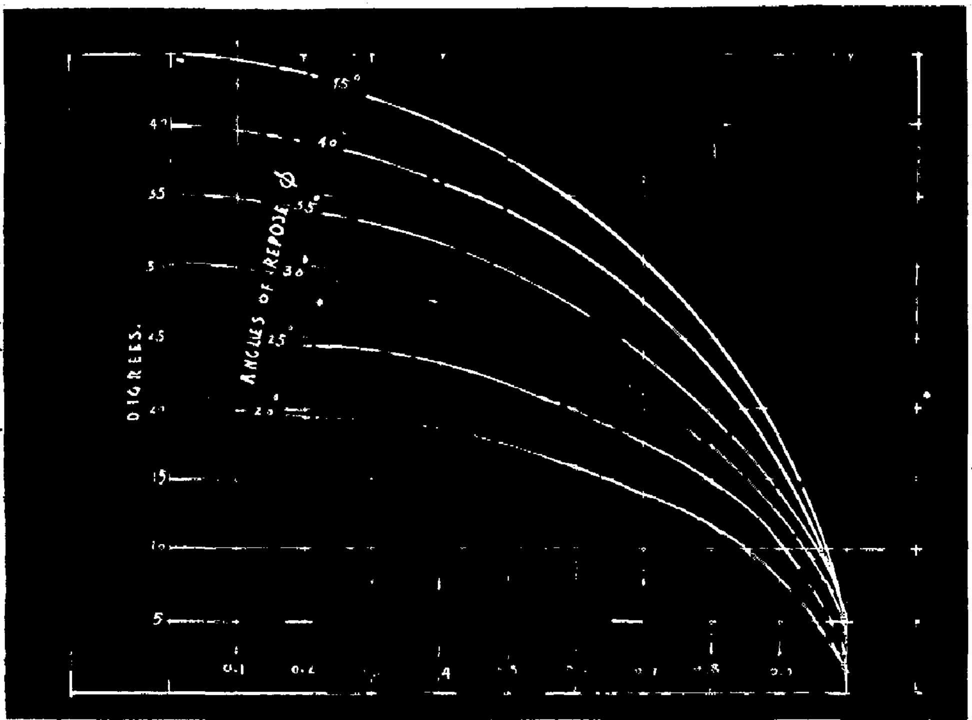
Tc on Sides

Tc on Bed.

Maximum Tractive Force on Sides and Bed.

FIG 5

Side Slopes.
1¼:1
1½:1
1¾:1
2:1
3:1



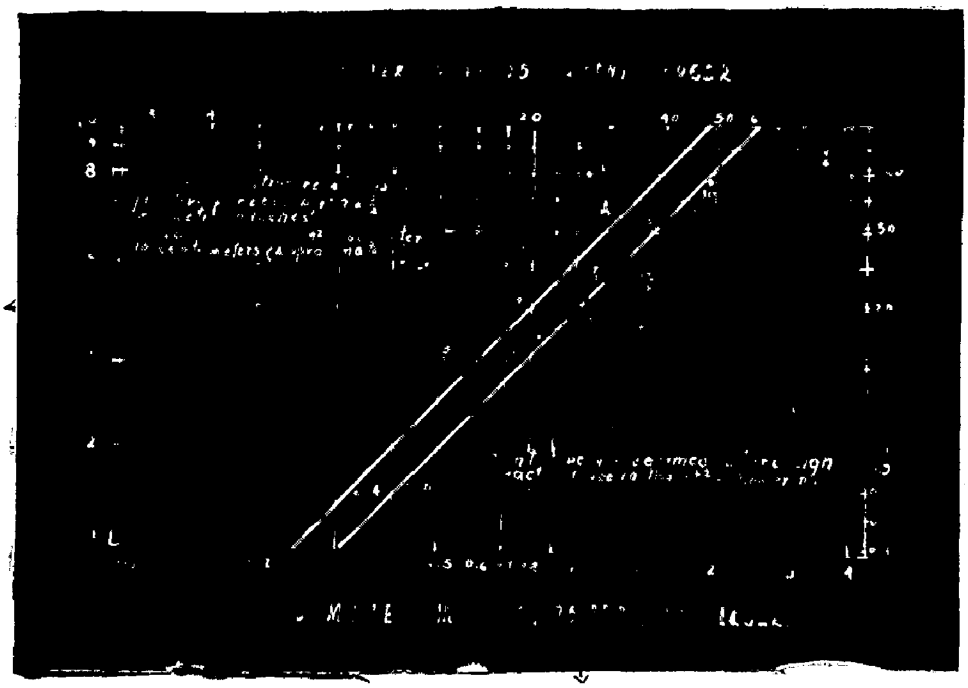
T_c on sides in fraction of the value of bed
0.64

Tractive Force Required to Initiate Movement.

FIG 6.

τ_c in lbs/sq"

0.4



1"

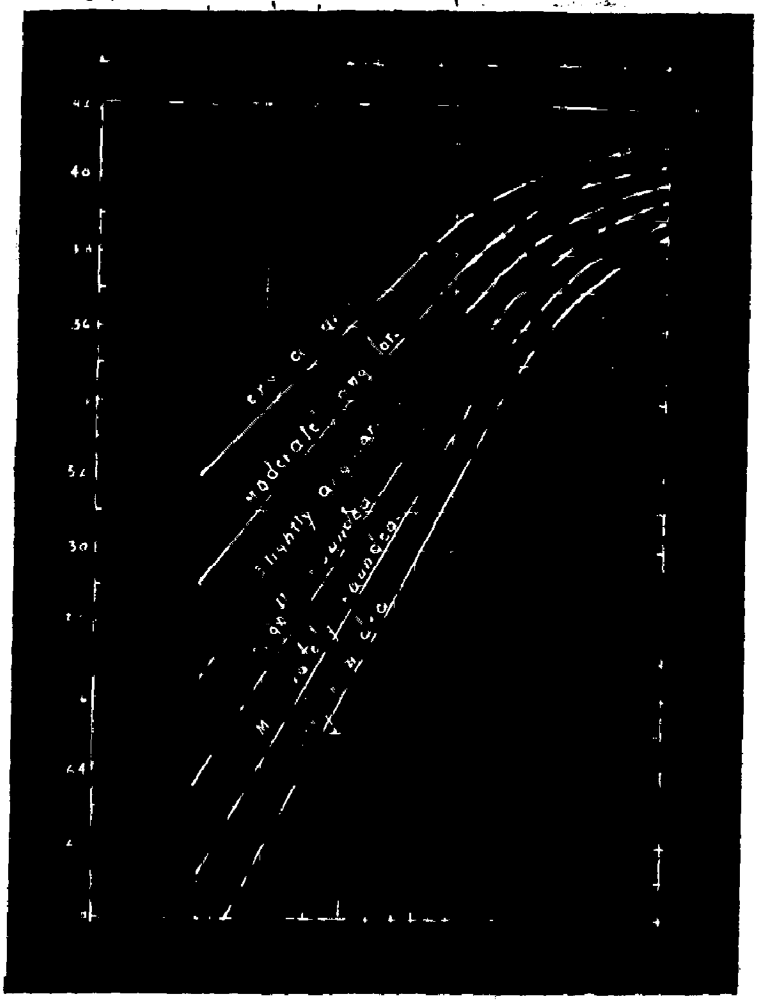
τ_c plot against Mean Diameter

FIG 7.

Particle Sizes in inches.

0·1 0·2 0·3 0·4 1"

Angle of Repose



Angle of Repose for different Sizes of particles