[Ex/66]

UNIVERSITY OF ROORKEE
ROORKEE (U.P.)

Certified that the attached Thesis/Dissertation on A Review on Designs of Stable Channel by Empirical Formulae

was submitted by

Sri Abdul Ghani Safi

and accepted for the award of Degree of ~~Doctor of Philosophy~~/Master of Engineering in W. R. D.

vide Notification No. Ex/ 424 P-65 (Degree)

dated April 17, 1975

Dated......

Assistant Registrar (Exam.)

PSUP (R) 197 Ch /376—1974—20 Pads.

A REVIEW ON DESIGNS OF STABLE CHANNEL BY EMPIRICAL FORMULAE

A Dissertation
submitted in partial fulfilment of the
requirements for the award of the Degree
of
MASTER OF ENGINEERING
in
WATER RESOURCES DEVELOPMENT

By
ABDUL GHANI SAFI
(AFGHANISTAN)

WATER RESOURCES DEVELOPMENT TRAINING CENTRE
UNIVERSITY OF ROORKEE
ROORKEE U.P. (INDIA)
April, 1975

C.P.SINHA
B.Sc.(Engg.)Hons, M.E.(WRD),Hons,
M.I.E.(India),
Reader Planning
W.R.T.D.T.C. at University of Roorkee
Roorkee, (U.P),INDIA

Certified that the dissertation entitled " A REVIEW ON DESIGNS OF STABLE CHANNEL BY EMPIRICAL FORMULAE" which is being submitted by Shri ABDUL GHANI SAFI in partial fulfilment of requirements for the award of the Degree of Master of Engineering in Water Resources Development of the University of Roorkee, is a record of the candidate's own work carried out by him under my supervision and guidance. The matter embodied in this dissertation has not been submitted for the award of any other Degree or Diploma.

This is further to certify that he has worked for more than six months from 1st October 1974 to 12th April, 1975.

Roorkee
Dated April 14, 1975.

(C.P.SINHA)
INTERNAL GUIDE

A C K N O W L E D G E M E N T S

The author expresses his deep sense of gratitude to Sri C.P.SINHA, B.Sc.(Engg.)Hons., M.E.(W.R.D), Hons., M.I.E.(India), Reader Planning in WRDTC, University of Roorkee, Roorkee (India) for his able guidance given to the author at every stage of work in preparing this dissertation.

The author is also thankful to Prof. Prahlad Das, Dr. Bharat Singh and other staff members of W.R.D. T.C., for their constant encouragement and valuable suggestions given to the author in preparing this dissertation.

A. G. SAFI

CONTENTS

--

SYNOPSIS

Design of sediment-laden channels has baffled irrigation engineers from time immemorial. Generally, when a sediment-bearing canal in alluvium is run steadily with a definite discharge, it tends to adjust its width, depth and slope to equilibrium values irrespective of their originally constructed values. The subject was studied by different irrigation engineers and research workers in India from particularly about 1890, and onwards, notably by Kennedy, Lindley, Lacey, Inglis and Blench.

Some attempts have been made by Lane, Ning Chien, Simons and Albertson, Leopold and Maddock etc in other countries.

In this dissertation an effort has been made to review different methods given by various investigators. All available existing regime theories as mentioned above are described and a general review of the rational approach is also given.

An attempt has been made to analyse the various regime theories in light of available knowledge of field and laboratory data as well as rational approach. At the end some conclusions have been mentioned and a suitable design method is recommended.

NOMENCLATURE

The following symbols have been used all through this dissertation. Others have been explained in the place where they are used.

Symbol	Descriptions	Dimensions
A	Area of cross section	L^2
B	Bed width	L
C	Chezy's coefficient	$L^{1/2}T^{-1}$
C	Coefficient in resistance equation $V=CR^XR^Y$	$L^{1/2}T^{-1}$
D	Depth of water	L
D_m	Mean depth	L
d	Grain diameter	L
E	Lacey's shock factor	-
E	Complete elliptic integral of second kind	-
e	Lacey's correction factor	-
F_b	Bed Factor	LT^{-2}
F_r	Froude number	LT^{-2}
F_s	Side Factor	L^2T^{-2}
f	Lacey's factor	LT^{-2}
f_m	Mean silt factor	"
f_{RS}	Silt factor	"
f_{VS}	Silt Factor	"
G_s	Rate of sediment Transport	MLT^{-3}
g	Acceleration due to gravity	LT^{-2}
g_s	Rate of sediment transport per unit width	MT^{-3}
I	Inglis number	-

H	Height of ripple	L
K_s	Equivalent sand roughness	L
N	Sediment concentration	—
N_a	Absolute coefficient of rugosity	$L^{-1/4}T$
n	Actual working value of Manning Resistance coefficient for the channel as a whole	$L^{-1/3}T$
n'	Manning's roughness coefficient with respec-t to grain size of bed material (Strichler's equation)	"
n_w	Manning's coefficient for sides wall	"
P	Wetted perimeter of channel cross section	L
Q	Water discharge	L^3T^{-1}
Q_b	Bank-full discharge	"
q	Water discharge per unit width	L^2T^{-1}
q_s	Sediment discharge/unit width (Bed load)	
q_T	Sediment discharge/unit width (total sediment)	
R	Hydraulic mean depth or hydraulic radius	L
R'	Hydraulic mean depth of grain resistance	L
R_b	Hydraulic mean depth of ass-ociated with bed	L
R_w	Hydraulic mean depth associated with sides	L
R_o^o	Particle Renold Number $\sqrt{Zd/\rho\nu}$	
Ro	Renodl Number	
S	Longitudinal slope of channel	
S_s	Specific gravity of solid particles	
S°	Slope per thousand	
V, V_m	Mean velocity	LT^{-1}
$V_{s,v}$	Fall folocity	"

V_*, U_*	Shear velocity (gRS)	LT^{-1}
W_s	Water surface width	L
W^*	Average width	L
γ_s	Unit weight of solid	$ML^{-2}T^{-2}$
γ_w	Unit weight of water	— do —
γ_f	Unit weight of fluid	— do —
τ	Shear stress at the bed	$ML^{-1}T^{-2}$
τ_c	Critical shear stress (at threshold motion)	— do —
ρ	Mass density	ML^{-3}
ν	Fluid kinematic viscosity	L^2T^{-1}

If Units are not mentioned it should be considered in fps units.

CHAPTER - I

INTRODUCTION

1.1 GENERAL

Irrigation is one of the oldest arts of civilization. Historically, civilization has followed the development of irrigation. Civilizations have risen on irrigated lands. They have also disintegrated in irrigated regions.

Even though the origin of irrigation is lost in the haze of unrecorded history of antiquity, it is well documented throughout the written history of mankind, that first use of irrigation as an aid to agriculture was probably made about the same time as man changed his nomadic existence and fashioned himself to a social way of life with settlement in the lush river valley [1] .

The Nile River water has been used in irrigation of lands in Northern Egypt for 5,000 years, by the First Pharoh of the First Dynasty, King Menes (Circa) to enrich his Kingdom with its rich cultivatable soil . The first world's oldest dam, 108 m long and 12 m high was built in Egypt [2] .

In China, where reclamation was begun more than 4000 years ago, the success of their early king s was measured by their wisdom and progress in water- control activities. The famous Tu-Kiang Dam, still a successful dam today, was built by a man named Mr. Li and his son in the Chin-Dynasty (200 B.C.) and provides irrigation water for about 2,00,000 hectares of rice field.

The chart of the Amu Darya (a river between Afghanistan and USSR) delta soil irrigated in the 6th and 4th centuries B.C. prepared under the supervision of Tolstov , showed that even 2,500 years ago a very large area was covered by irrigation system in this region.

India claims to be the craddle of irrigation practice and development. Numerous references to dams, canals , wells and tanks exist in the Vedas (very old scripture), the earliest sacred books of the Aryans. There are reservoirs in the South of India (Srilanka) more than 2000 years old. Writings of 300 BC indicate that whole country was under irrigation and very prosperous because of the double crops which the people were able to reap each year.

The Spaniards on their entrance into Mexico and Peru found elaborate provisions for storing and conveying water supplies which had been used for many generations. Extensive irrigation works also existed at that time in the Southwestern United States. The early Spanish missionaries brought knowledge of irrigation from their Mediterranean homes.

It is unfortunate that in most cases the founder fathers had not recorded their experiences or that if they did, the ravage of time have destroyed all the records. However, history is replete with references to the paractice of irrigation from wells, tanks and canals.

1.2 IRRIGATION AS A SCIENCE

The present history of science of irrigation does not trace itself to more than two or three centuries back. Vestiges, records and ruins of irrigation works existing in different parts of the world, bear testimony to the fact that irrigation, both as a science and art, has been practised in several countries. This indicates that our ancestors were instinctively conversent with atleast the empirical laws of hydraulics.

In arid and semi-arid regions of the world, irrigationwas an age old art, but later, it became a science. The science now covers not only the scientific application of water artificially to the soil to make up the deficiency of moisture under conditions necessary for plant growth, but also the investigations, planning (single-purpose, multi-purpose, basinwise or regional), design, construction , maintenance and operation of structures and channels for conveyance of water from the source to the point of application and all related problems (including engineering, agronomical, economical, financial and social) and researches appurtaining thereto, to ensure permanent agriculture.

Optimum economical and efficient designs of canals and hydraulic structures have been developed through a continuous process, first through individual intuitive 'hunches' and efforts of great irrigation engineers and later by organised systematic research.

Early canals were excavated in earth when little was known about the theory of movement of sediment. There are numerous instances, illustrating how shortly after their opening, the canals were silted up badly (e.g., the canals in Egypt and India) which were built in 19th century and necessitated regarding or provision of silt-exclusion devices at the head works.

The evaluation of canal design from the Kennedy Theory (1895) to the relative modern concepts of empirical regime theory (Lacey, Inglis Blench, Bose, Ming Chien and Simons etc.) or the more rational Tractive Force Theory (Lane and others) provides engineers with practical tools for the rational design of channels.

1.3 DEVELOPMENT OF REGIME THEORY

Since irrigation in India has been practised from pre-historic times and appears to have been an open field laboratory for analysing the data which the various pioneers of regime theory were collecting. These pioneers are : Kennedy, Lindley, Lacey, Inglis, Bose and Blench etc [3].

These authors have compiled the data of canals in the indo-Gangetic plain for the purpose from all available sources. This is because , the largest unlined canal systems in alluvium in the world were located in India and Pakistan and almost all pioneering works in design of stable channels were carried out on these systems.

1.4 DESIGN OF STABLE CHANNELS

The design of stable channel requires the application of entire knowledge about river, canal, hydraulics, sediment transport etc. The problem appears even more complex, due to large number of variables which effect the situation and sediment load that such channels generally carry.

The problem is not so easy to be solved by a single equation of Chezy of Manning type, as originally was used. The present knowledge of hydraulics can provide only two equations, namely continuity equation and flow or resistance equation :

$$Q = AV \quad \text{and} \quad V = f\,(N, R, S).$$

As is usual, Q and N may be known. Thus determination of

A and R leaves V and S as still to be decided. Hence two more conditions must be imposed or two more equations written down before the channel dimensions can be calculated out for the given shape of cross-section.

The alluvium in which the channels generally flow is more or less erodible and the channel section may thus be considered flexible rather than rigid . The aim of designer in respect of such channels is to obtain - physical stability, a balance between silting and scouring and dynamical equilibrium in the forces generating and maintaining channel crosssection and gradient. Therefore it is necessary to study an approach which must be based on known laws of mathematics and principles of hydraulics by which an analysis can be made, of the interrelation of the forces acting on the s-olid particles during the flow of water.

We have two approaches of design for stable channels:

(1) Empirical Approach and (2) Rational Approach. Since the rational approach is based on some theories which are still incomplete and cannot be universally accepted, some assistance from empiricism is necessary. This requires the design of stable channels to be studied in the light of both the approaches.

CHAPTER II

EMPIRICAL APPROACH

2.1 GENERAL

The solution of the problem of stable channel design with sediment laden and an erodible section is very difficult theoretically. Irrigation engineers noticed that while some of the channels silted badly, others on the same condition or system attained stability. They rightly concluded that a particular cross-section and slope provide stability. Attempts have been made by irrigation engineers in India, Egypt and some other countries to determine empirical correlations for such channels. These relationships have based upon the results of long experience of irrigation engineers or the experiments of research workers.

The derivation of these formulae are not based upon a rational or mathematical analysis still they are used extensively in stable channel design.

2.2 REGIME THEORY (INDIAN)

According to Lacey, regime flow in silt transporting channels excavated in alluvial means physical stability, a balance between silting and scouring, and a dynamic equilibrium in the forces generating and maintaining channel cross-section and gradient [4]. The theory is largely empirical but recognizes that width, depth and slope of the channels are vairables. The fundamental basis of this concept is that : a channel constructed in alluvial soils with erodible boundaries, if allowed to flow unrestricted will forms its own dimensions to carry a given discharge and sediment load at state of final equilibrium. The theory was first developed in India and Pakistan where vast network of canals provided. The opportunity for formulation of regime equations. The canals draw water directly from large rivers and in flood season carry sediment consisting of silt and fine and median sand, the load generally not exceeding 2,000 ppm. Unless the dimensions and slope of a channel are constructed to regime requirements, the geometry will suffer gradual change and performance will deteriorate. Inadequate slope leads to siltation and loss of discharge capacity and excessive slope to scour endangering structures. Inadequate width leads to bank scour, excessive to meandering.

After regime theory was derived from data of Indian canals, many measurements of canal and river dimensions have been made in different parts of the world by various authors. Among these are Pettis (Florida, U.S.A), Leopold, Moddock (Western U.S.A.) Simons and Henderson (U.S.A.) and Morshal Nixon (U.K). Most of these found that there is a significant corelation between each channel dimension and equilibrium discharge, and index of the discharge is very nearly the same as given by the formulae derived from Indian canals. The beginning of the regime theory was made by R.G. Kennedy in 1855.

2.3 KENNEDY's THEORY

R.G. Kennedy [5] published his classic empirical equation on the concept that the silt was maintained in suspension by vertical eddies of water rising against the bed. His general idea was that the mean speed of flow, when a channel had settled down to what it had chosen for itself, was a function of depth. Thus large depth would require strong eddies, and hence faster velocity. Kennedy's formula is most conveniently be written as,

$$V_o = CD^y \qquad \ldots(2.1)$$

where V_o = Non silting and non scouring velocity

D = Vertical depth measured on the approximately horizontal silted bed for a channel very closely in shape as trapezoidal section

C and y were thought to be constant originally and were assigned values of 0.84 (in fps units) and 0.64 respectively.

To test his theory Kennedy plotted the data from channels of the Upper Bari Doab canal System which had not required any silt clearance for last 30 years and were thus considered stable. He noticed that for every discharge, there existed a velocity in canal at which there is neither silting nor scouring, which he termed 'critical velocity'. Kennedy recognised that grade of silt played a part in his relationship, and introduced another factor in equation, called by him "The critical velocity ratio" or CVR. and given the symbol 'm'.

$$CVR = m = \frac{\text{Critical velocity for canal of different silt grade}}{\text{Critical velocity for Bari Doab canals}}$$

He regarded the sandy silt of the upper bari Doab canals standard, the coefficient (m) for that cana being unity. The equation then was written as

$$V = 0.84\ m\ D^{0.64} \qquad \ldots(2.2)$$

where, $m = \frac{V}{V_o} = C.V.R.$

In metric units,

$$V = 0.55\ mD^{0.64} \qquad \ldots(2.2m)$$

For silt of Bari Doab canal system m = 1 .

For finer silt $m < 1$ (m = 0.80 - 0.90)

For coarser silt $m > 1$ (m = 1.1 - 1.20)

Kennedy made no correlation between the water surface slope of regime channels and the mean velocity, or the vertical depth. He assigned slopes to his regime channel by employing the Kutter equation.

The difficulty in this formula is assigning correct value of rugosity coefficient (n). When Kennedy published his equation there was a general opinion that a channel of moderate size transporting typical Northern Indian sand, if in regime, or very good condition worked to a value of 'n', approximately 0.0225. There was however, the difficulty that, without regard to the material in which the channel was excavated or the type of silt transported, there appeared to be a real and measurable variation in the value of 'n' with the channel. Thus Kennedy himself [6] suggested in 1904 that the value of 0.0225 for 'n' better suited to small canals and a value of 0.0200 was appropriate for large canals. In Egypt [6] also where the silt is finer, a value of 0.0200 was recommended for ordinary use, and 0.0170 for large canals. Garrett in his diagrams assigned a value of 0.0225 and 0.0200 for moderate and large canals respectively.

The condition of earthen channels was also very important in calculation of slope and assignment of Kutter's 'n'. Thus Buckley [6] gave this recommendation

Table 2.1

Conditions of Channel	n
Above average (very good)	0.0225
Tolerably good (good)	0.0250
Below average (Indifferent)	0.0275
Bad (poor)	0.030

Therefore it should be realized that the value of 'n' varies both with scale and channel condition. Kennedy made no attempt to relate the rugosity coefficient 'n' with critical velocity ratio 'm' as a measure of silt grade.

Kennedy also concluded that the eddies rise due to friction force against flowing water from bed and work up against the depth of the channel. The sediment is kept in suspension solely by vertical components of the eddies. From the sides also, some such eddies may start, but most of such eddies are horizontal, have no power for supporting silt in suspension. Therefore kennedy considered that sediment supporting power of a stream depends upon the width of the stream and not on the total wetted perimeter.

Now Kennedy's V-D relationship provides an additionalcondition for designing the regime channels. But the main defect of Kennedy's equation is that, we can design different channels by varying slope satisfying both Kutter's and Kennedy's equations. Kennedy equation provides no clues as to which of those channels would be best. Certainly some hints are necessary to choose the most suitable one. The necessity of doing so was recognised by Kennedy himself and has recommended some rough working rules for the ratio of B/D (bed-width to depth ratio) at later date in 1904. The bed width-depth ratios were fixed in different states by experience. In Punjab the ratio was subsequently extended by Mr. Woods [7]. The main point to be considered in this rule is that B/D ratio increases with discharge . As depth approaches to 3.9 ft instead of increasing in an orderly manner increases without limit. Mr. Kennedy cons-idered that a velocity in excess of 3.50 fps would scour the banks of canal irrespective of discharge. Hence

$$D_{lim} = \left[\frac{3.5}{0.64}\right]^{-1/0.64} = 9.30 \text{ ft.}$$

In U.P.(India) the following empirical rules were generally followed [8].

For channel upto 14 cumecs (in metric units)

$D = 0.55 \sqrt{B}$ for slope of 1/9,400

$D = 0.52 \sqrt{B}$ for a slope of 1/8,250

$D = 0.47 \sqrt{B}$ for a slope of 1/6,600 to 1/4,400

For the channels of 14 cumecs and over the following depths were considered satisfactory.

Q (cumecs)	P (meters)
14	1.68
28	1.83
35	2.14
70	2.28
140	2.59
280	2.90 to 3.05

For very small channels upto 1.4 cumecs(50 cusecs) this equation can be used $D = 2.8 \sqrt{Q}$

where D is in metres and Q is in cumecs. Kennedy tried to work out the silt transporting capacity of a channel on the assumption that the side are vertical and total quantity of silt transported depends directly on B and some power of critical velocity V_o. Hence the quantity of silt transported, Q_t is proportional to BV_o^n, where n is the unknown index. Let P be the percentage of silt in water and Q the discharge of the channel. Then the amount of silt carried

$Q_t = p.B\ D.\ V_o^n$ (approximate) , A = B.D.

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

Then $KBV_o^n = p.B.D.V_o$

$$V_o = (P/K)^{\frac{1}{n-1}} \ (D)^{\frac{1}{n-1}}$$

But for the Punjab canal $V_o = CD^y = CD^{0.64}$

Hence, $(P/K)^{\frac{1}{n-1}} = C = 0.8$, and $D^{1/n-1} = D^{0.64}$

$$\therefore \quad \frac{1}{n-1} = 0.64$$

or n = 2.56 or about 2.5 , or 5/2.

Hence the quantity of silt transported is

$$Q_t = KBV_o^{5/2} \qquad \ldots(2.3)$$

K = constant of proportionality the value of which was not further investigated by Kennedy. From this equation X the amount of silt carried at velocity V other than V_o can be calculated as $X = K\,V^{5/2}$ while $P = KV_o^{5/2}$

$$\therefore \quad x = P\left(\frac{V}{V_o}\right)^{5/2} \qquad \ldots(2.3a)$$

When the mean velocity of a channel exceeds the critical velocity ($V > V_o$) , the silt carrying capacity will be more than the bed can resist and scour will occur. The amount of scoured will be X - P or

$$X - P = P\left[(V/V_o)^{5/2} - 1\right] \qquad \ldots(2.3b)$$

But if V is less than V_o deposit will occur . Amount of deposit = P-X = P $\left[1 - (\frac{V}{V_o})^{5/2} \right]$

After Kennedy published his equation, many regional formulae were derived. Some of these are given below (for fps units)

Values of C	Values of 'y'	Descriptions.
0.84	0.64	Kennedy, India, Upper Bari Doab Canal, sand silt.
0.91	0.57	Stawell, Burma, Shwebo Canal for 'fine sand'
0.67	0.55	Madras, India, Godavari Delta (Western), 'fine s ilt'.
0.93	0.52	Madras, Krishna Western Delta
0.95	0.57	Lindley, India, Lower Chenab Canal 'Standard sand"
0.39	0.73	Ghaleb, Egypt, Selected data, Egyptian canals" Fine Silt"
0.39	0.67	Molesworth and Yerdunia,Egypt (Upper)
0.475	0.67	-do- (lower)
0.94	0.59	Griffith and Bottomley.

We can conclude that if we want to make use of Kennedy type equation in different canal systems both the constant and exponent should be changed.

In 1913, Capt. A. F.F.Garrett prepared a diagram which provides a graphical solution of Kennedy's and Kutter's equation, which is a very handy tool for design purpose [9].

2.4 LINDLEY'S EQUATION

It was Lindley who in 1919 introduced the width of a channel as a regime variable, and thus made a valuable advance in regime theory. By advancement engineering opinion had arrived at bas-ic fact that unknown variables of regime channels required four equations for complete design of ultimate canal, velocity, breadth, depth and slope.

Lindley stated " when an artificial channel is used to convey silty water, both bed and banks scour or fill, changing depth, gradient, and width until a state of balance is attained at which the channel is said to be in regime. These regime dimensions depend on discharge, quantity and nature of bed and berm-silt, and rugosity of silted section, rugosity is also affected by velocity, which determines the size of wavelet into which the silted bed is thrown" [10].

The statement clearly recognises the influence of the quantity together with the nature of silt and ripples. Unfortunately those two factors were not given their proper weight for considerable time afterwards.

Lindley collected data from 786 observations, covering over 4350 Km of canals. The sections selected by him were only regular and straight reaches. He did not attempt to select the stable reaches. Because the purpose of his investigation in his own words was analysed "to see to what the average would lead. The collection of data was thus of considerable statistical value, covering a large system which as a whole might be considered as oscillating between silting and scouring. No discharge were observed but widths, depths and slope were measured.

Lindley employed the Kutter formula to compute velocities and obtained the following relations in (fps units)

$V = 0.95 \ D^{0.57}$ ..(2.4)

$V = 0.59 \ B^{0.355}$ ..(2.5)

By combining them it can be written

$B = 3.8 \ D^{1.61}$ ..(2.6)

The corresponding formulae in metric units are :

$V = 0.567 \ D^{0.57}$ ..(2.4m)

$V = 0.274 \ B^{0.355}$ ..(2.5m)

$B = 7.8 \ D^{1.61}$ ..(2.6m)

In which B is average bed width so that A = BD. Two regime equations of Lindley, in conjunction with those of continuity and flow give four equations for the determination of four unknowns, B,D,V and S. Since A and R both are a function of B and D, can be easily expressed in terms of B and D.

Lindley assumed a formal cross section consisting of a horizontal bed and $\frac{1}{2}$ to 1 sides slopes, and it is practice to apply his equation to natural channels.

Lindley relied on Kutter's equation for determining velocity. Therefore his analysis was incomplete. He made no attempt to correlate rugosity and silt grade, but assumed n = 0.0225.

Lindley's equations are valid for $B < 32$ m (150ft), $D < 2.7$ m (9 ft) and $V < 1.0$ m/sec (3.3 ft/sec) . From Lindley's data, Lacey [4] derived the relationship below by assuming that in large channels B approaches to P as a limit and D approaches R.

Now $Q = AV = P.R.V.$ or $R = \frac{Q}{PV}$

Then from Eqn(2.6) we can write (fps units)

$$P = 3.8\, R^{1.61} = 3.8 \frac{Q^{1.61}}{P^{1.61}\; V^{1.61}}$$

$$\text{or } P^{3.18} = \frac{3.8}{(0.59)^{1.61}}\, Q^{1.61}$$

$$P = 1.984\, Q^{0.506} \qquad \ldots(2.7)$$

Later Inglis [11] obtained the following equations from the same data (fps units)

$$W_s = 2.3\, Q^{1/2} \qquad \ldots(2.8)$$

$$D = 0.57\, Q^{1/3} \qquad \ldots(2.9)$$

In 1920, Khanna [12] gave the following relations

$$V = 0.0276\, RS \qquad \ldots(2.10)$$

In 1921, Bolodia and Malakal gave two other relations

$$V = 0.028\, RS \quad \text{(Bolodia)} \qquad \ldots(2.11)$$

$$V = 0.046\, RS \quad \text{(Malakal)} \qquad \ldots(2.12)$$

The above relations based on Egyptian data, later influenced Lacey's thinking on the subject.

2.5 LACEY'S REGIME EQUATIONS

The statement of original Lacey theory appeared first in Western Publication in 1929 [13] and 1933-34 [14]. In 1939 in Technical paper No.20 [4] he revised his equations from earlier publications. The treatment given here is based on the revised version presented in 1939. Official design diagrams were issued as early as 1932 [15]. A further Western Publication of 1946 [16] adds some speculation to the original material. In the paper published in 1958 Lacey discussed about the objections for the use of P and R as parameter [17]. In his paper in 1966, he has discarded his silt factor completely and instead considered the sediment concentration, settling velocity and median diameters [18]. Gerald Lacey retired as a chief Engineer, Irrigation Department U.P.(India) in 1945. He carried out a long detailed studies of the problem of design of stable channels in alluvium and contributed significantly towards the advance of knowledge in this subject.

Lacey's outlook was essentially that of the physist. He appreciate that, behind the relation shown by empirical plotting, there must be relatively a simple law connecting dynamical quantities such as forces and energies. Further he made use of what conventional fluid Mechanics would call similarity considerations, that is, he visualized small channels as modelling larger ones, and he paid attention to the way in which different dimension was 'scaled up'.

2.5.1 Regime as a Physical Concept

According to Lindley postulate a delicate and precise balance in all the dimensions of an open alluvial channel can very seldom be perfectly achieved, for there are too many variables, which in practice not all are equally free to vary.

For regime to be stabilished fundamental requirements are that :

1. The channel is flowing in unlimited incoherent alluvium of the same character as that transported.
2. The silt grade and the silt charge are constant, and
3. The discharge is constant.

If there is a complete freedom for lateral movement , it is called "true regime". Lacey also defined regime channel as stable channel transporting a regime silt charge or it can be defined "A channel will be in regime when it flows uniformly in incoherent unlimited alluvium of the same character as that transported and silt and silt charge are all constant" [4] .

In 1946, Lacey defined a regime channel with respect to sediment charge as " a stable channel transporting the minimum bed load consistent with a fully active bed" [16]

Incoherent Alluvium

Soil composed of loose granular material which can be scoured as readily as it can be deposited. The material is a common place in the laboratory, but seldom reproduced perfectly in the field. The material may range from very fine sand encountered in rivers with small slopes, to gravel pebbles and boulders met with in torrents. In this kind(truely incoherent) of material the regime balance is very delicate, since any small fluctuation in velocity or turbulent , from whatever cause, must set up in-stability. In nature the entire wetted surface of the channel is seldom incoherent. When the silt transported is very fine and mixed with clay such silt once deposited and compacted is tenaceous, and may prove for more difficult to scour than coarser incoherent material.

Fluctuatión in discharge provide 'rest period' for fine silt which aggravate this "cementing" tendency in the bed of channel.

Regime Silt Charge

It is the minimum transported load consistent with fully active bed.

Regime silt Grade

By a silt grade Lacey meant not the average mean diameter but the gradation between the small and big particles. This may probably give symmetrical c-iltograms or size distribution curve.

Regime In River

In sandy and boulder rivers the only quasi-regime or temporary regime may exist. Although the sand in a river bed may perfectly incoherent it is at"full stage", or high flood that river is 'alive' at other time the bed may be inert. In gravel and boulder river and torrents the effect is still more marked. It is only during the floods of great magnitude that river is fully active for a short period between the scour of rising and deposit of falling flood stage. As the flood subside the boulders, gravel and finally even the sand becomes inerts. Hence the river in low discharge may be considered as rigid boundary s o far as the sediment movement is concerned. In Sandy rivers in alluvial plain the freedom for lateral movement is achieved to some extent and by meandering adjust their length and slope. In all boulder rivers, and many foot hill sandy torrents, the slope is a dominating feature imposed by the terrain.

Regime in Artificial Channels

An artificial channel will have cohesive banks and as such when it comes into the service first, it will have a given discharge, width, silt grade and silt concentration. Channel excavated in first instance with defective slopes, and somewhat narrow dimensions, are free by immediately throwing incoherent silt on the bed to increase their slopes, and by generation of increased to achieve a non-silting initial regime. Such channels if the berms are grassed and the banks are tenneous , will be subjected to considerable lateral restraint and if the bank soil is of clay the sides may resist erosion almost indefinitely. Channels of this type achieve a working stability but are not in 'final' regime. Thus the initial regime is the state of channel that has formed its section but had not attained the final slope. Different initial regimes shapes may progressively occur at one site as the slope is in the process of attaining the final regime, which may be either scour or silting. 'Final' regime represents the condition, when all elements of channel cross section as well as slope are equally free to vary. In the case considered the wetted perimeter far from being a variable is appreciably constant. When the channels are protected on the bed and sides with some kind of material, the channel section cannot be scoured up and so there is no possibility of slope to change, this type of channel is said to be in permanent regime. Regime theory is not applicable to it.

2.5.2 Regime Equations

Lacey adopted Lindley's earlier concept and with respect to unique section he stated " A constant discharge transporting silt of regime grade, and flowing in self transporting alluvial plain, tends eventually to assume a gradient solely determined by the discharge as silt grade, the mean velocity, hydraulic mean depth and wetted perimeter will also tend to unique determination" [4] .

Such a constant discharge will tend also to transport a fixed 'regime' silt charge. According to this statement in all regime channels in incoherent material the primary fundamental variables are mean velocity, the hydraulic mean depth, the water surface, and the bed silt grade and there are only two independent variables, the discharge and the silt factor. Lacey's idea was that the cause of suspension of silt is the vertical components of eddies generated in all points by the forces normal to the wetted perimeter and for this reason he adopts a hydraulic mean depth instead of vertical depth as the variable. Lacey plotted V against R, by employing the Kennedy, Lindley and Madras data and obtained first relationship as a general form

$$V = CR^{1/2} \qquad \ldots(2.13)$$

The square root of R fitted data in all cases and unlike Kennedy's equation, the exponential power had not altered to suit different conditions. He introduced a silt factor f as a measure of silt grade. He defined (f) such that the concept of Kennedy's critical velocity ratio (m) is preserved.

$$f = \left(\frac{V}{V_o}\right)^2 = m^2 \qquad \ldots(2.14)$$

Then the general form of the equation (2.13) is

$$V = CR^{1/2} = KmR^{1/2} = K\sqrt{fR} \qquad \ldots(2.15)$$

Thus in regime channel of the same mean velocity the hydraulic mean depth varies inversely as the silt factor. Before m or f can be computed the value of K must be determined. The numerical value of K will be determined when the standard silt with a critical velocity of unity has been more rigidly defined. From Eq. (2.15) we can write,

$$C = Km = K f^{1/2} \qquad \text{..(2.15a)}$$

The constant C shows the measure of turbulence in regime channel. It would appear to be a useful general criterion of the grade of alluvial material. Regime observations of channel in gravel and boulders are lacking but observations for channel in active full stage, corresponding to quasi-regime are recorded. The following table gives observations over an extra ordinary range.

Table 2.2

Value of C	Description of Silt grade channel [4]
0.732	Ismailia Canal, Egypt , Very fine silt
1.056	Upper Ganges canal distributary,India Fine sand silt.
1.460	Missoury River, U.S.A., Coarse river sand,
1.620	Miami River, Hamilton, U.S.A. Gravel sand and Clay.
2.430	Rhine at Basle. Gravel and Shingle.
2.919	Isar River Bavaria, Coarse gravel and detritus.
4.28	Kander River, Canton Berne, Very coarse detritus.

The table shows that the ratio $(V/R^{1/2})$=F(grade material transported).

2.5.3 Lacey's Flow Equations

The form of general equation $V = CR^{1/2}$ shows the possibility of comparison with classic Chezy equation

$V = C\sqrt{RS}$. Unfortunately the slope S enters the Chezy equation as a third variable which make any true comparison impossible. Modern practice favours, replacing the Chezy equation with an exponential of the Manning type:

$$V = C' R^n S^{1/2} \qquad ..(2.16)$$

In metric unit it is in form of

$$V = \frac{1}{N} R^n S^{1/2} \qquad ...(2.16a)$$

The slope's power is uniformly assigned a value of 1/2, but for R there are different values according to different authors. The following table gives the values of exponents of some modern flow equations.

Table 2.3

n	Authority	Remarks.
0.667	Manning, Gauckler	This equation, be very closely replaces the Kutter formula when rational slopes are employed.
0.700	Barnes, Forchheimer	Barne's "approximate formula" for earthen canals and rivers . Also known as the Forchheimer.
0.75	Lea	Lea's approximate equation for "earth channels in ordinary condition".

All of the above formulae are empirical, it can be concluded that one from the regime view point may prove fundamental.

From Eq. (2.15) and (2.16) we can write

$$C' R^n S^{1/2} = C R^{1/2} \quad \text{and}$$

$$R^{(n-\frac{1}{2})} S^{1/2} = C/C' \qquad ...(2.17)$$

From the plotting of Lindley's data (in FPS units) by the method of least square, the most probable exponential form is as follows -

$$R^{0.490} S = 0.000347$$

or having regard to the probability of simple power $R^{1/2}S = 0.000347$. The correlation of hydraulic mean depth and the slope can be written in general form

$$R^{1/2} S = C'' \qquad ...(2.18)$$

From Eq. (2.17) and Eq. (2.18).

$$2n-1 = \frac{1}{2} \text{ and that } n = \frac{3}{4}$$, this is similar power as in Lea's equation.

Eqn(2.18) may be written in another form,

$$\left[\frac{(R^{1/2} S)}{C''}\right]^n = 1$$

From Eq.(2.13), the general is obtained as

$$V = C R^{1/2} (C'')^{-n} (R^{1/2}S)^n \qquad ...(2.19)$$

From the form of Eq. (2.15) and (2.18) it is evident that it is possible to correlate C and C" and thus obtained a general regime equation for alluvial channels in terms of V,R and S, the rugos-ity being implicit in the valuesof R and S adopted by the channel. To do so, it is therefore possible to express one silt grade coefficient as a function of the other. If we consider Eq. $V = CR^{1/2}$, the coefficient C is a constant when the channel is in perfect regime and silt grade is a constant. Similarly in the equation $R^{1/2}S=C''$, the coefficient C" is a constant for constant silt grade and perfect regime conditions. Therefore we can write effectively

$C = F(C^n)$

or $V/R^{1/2} = F(R^{1/2} S)$

From plotting a large mass of data and statistical analysis Lacey found the best fit relationship

$$C = 16.0\ (C^n)^{1/3} \qquad ..(2.20)$$

or putting the value of $V^2/R^{1/2}$ and $R^{1/2}S$ from C and C^n respectively

$$V = 16.0\ R^{2/3}\ S^{1/3} \qquad ...(2.21)$$

In metric units $V = 10.8\ R^{2/3}\ S^{1/3}$ ...(2.21m).

The equation (2.21) is very important in computation of high flood discharge. This equation is s-uppos d to apply to rivers, only for short period at the stage, when they h ve just ceased to scoured and have not yet begun to deposit sediment. At that time a river is in momentary regime. To this tempor ry condition in rivers and torrents Lacey applies the term Quasi-regime . In this condition the flow equ tion of Manning or Kutter cannot be used, because of high flood when a river is in a temporary regime, the above quation can be used to find velocity with precise accuracy. An illumin ting equation can be obtained by cubing equation (2.21) and (2.21m)

$$\left.\begin{aligned} V^3 &= 4096\ (R^2S) \\ V^2 &= 4096\ (R/V)RS \\ V &= 64.0\ (R/V)^{1/2} \sqrt{RS} \end{aligned}\right\} \qquad ...(2.22)$$

Corresponding equations to metric units are :

$$\left.\begin{aligned} V^3 &= 1260\ R^2 S \\ V^2 &= 1260\ (R/V) S \\ V &= 35.5\ (R/V)^{1/2} \sqrt{RS} \end{aligned}\right] \quad \ldots(2.22m)$$

Then Chezy coefficient = $64.0(R/V)^{1/2}$ (in fps units) and in metric units it is equal to $35.5\ (R/V)^{1/2}$. Now putting $V = K\sqrt{fR}$ (from Eq. 2.15) , then Chezy

$$\text{coefficient } C = \frac{35.5\ R^{1/4}}{K^{1/2}\ f^{1/4}} \quad \text{(in metric units)}$$

or

$$C = 64.0\ R^{1/2} / K^{1/2}\ f^{1/4}\ R^{1/4}$$

$$= 64\ R^{1/4} / K^{1/2}\ f^{1/4} \quad \text{(in fps units)}.$$

The general regime equation can therefore be converted into a flow equation of Manning types in which the Chezy coefficient is represented by $R^{1/4} / n$ in metric units and by $1.3458\ R^{1/4}/n$ in fps units. Again putting $35.5/(K^{1/2} f^{1/4})$ $= K_1/N_a$ where N_a is being Lacey absolute rugosity coefficient. That is a coefficient determined solely by the average size and density of incoherent bed material of the channel.

Then Eq. (2.22) can be written as

$$\left.\begin{aligned} V &= \left(\frac{K_1}{N_a}\right) R^{1/4} \sqrt{RS} \quad \text{(in metric units)} \\ \text{or } V &= \frac{K_1}{N_a} R^{3/4}\ S^{1/2} \\ V &= \frac{1.3458\ K_1}{N_a}\ R^{3/4}\ S^{1/2} \quad \text{(in fps units)} \end{aligned}\right] \quad \ldots(2.23)$$

The equation (2.23) is the same form as Eq. 2.21. but the indices of R and S are different. The latter applies to perfect regime condition only and former can be applied to regime and non regime channels alike, to channel with rigid boundary and non rigid boundaries.

2.5.4 Na-F Relation and Determination of Numerical Coefficients

As previously mentioned the value of absolute rugorisity of coefficient Na of Lacey depends only on the grade and density of boundary material and are independent of channel conditions. Kennedy treated as standard silt the upper Bari Doab canal silt with $f = 1$ and the value of $m = 1$. Lacey treats as standard the sandy silt in a regime channel, of hydraulic mean depth equal to one metre and operating with a value of Kutter's n of 0.0225 This definition is rigid and applies whether the Kuttor, Manning, Forchheimer or any other equation of Manning type is employed. Hence lacey eq.(2.23) is comparable to Mannin flow equation $\left[V = \frac{1}{n} R^{2/3} S^{1/2}\right]$ and would give the same discharge when R is equal to one metre.

From the above description the following relationship can be deduced. Comparing Eq. 2.23 and Manning formula, we have,

$$V = \frac{1}{n} R^{2/3} S^{1/2} = \frac{K_1}{N_a} R^{3/4} S^{1/2}$$

Putting $R = 1$, we get,

$$n = N_a \text{ and } K_1 = 1$$

Thus the Lacey's flow equation (2.23) will be

$$V = \frac{1}{Na} R^{3/4} S^{1/2} \text{ (in metric units)} \quad ...(2.23)$$

$$V = \frac{1.3458}{N_a} R^{3/4} S^{1/2} \text{ (in fps units)} \quad ...(2.23a)$$

For standard grade silt value of $f = 1.0$ and $m = 1.0$ and $N_a = 0.0225$. As already derived $K_1/N_a = \frac{35.5}{K^{1/2} f^{1/4}}$.

This means that, $N_a \propto f^{1/4}$, since both K_1 and K are numerical coefficients not dependent on the grade of silt and since $N_a = 0.0225$ where $f = 1.0$ then $N_a = 0.0225 f^{1/4}$

$$N_a = 0.0225 m^{1/2} \quad ...(2.24)$$

The value of N_a are given in Table 2.4.

From the relation already derived Cheze's coefficient

$C = 64.0 (R/V)^{1/2} = 64.0 R^{1/4}/K^{1/2} m^{1/2}$ and Eq. (2.23a)

$$C = 64.0 (R/V)^{1/2} = 64.0 R^{1/4} / K^{1/2} m^{1/2} = 1.3458 / N_a$$

Substituting the value of N_a from Eq. (2.24)

$$64/K^{1/2} m^{1/2} = 1.3458 / 0.0225 m^{1/2}$$

or $K = 1.1547$ or $K^2 = \frac{4}{3}$ in fps units.

Similarly

$$K = \sqrt{\frac{2}{5}} \text{ (in metric units)}$$

Now equation (2.15) can be written as

(fps units)		metric units	
$V = \frac{2}{\sqrt{3}} \sqrt{fR}$	..(2.15)	$V = \sqrt{\frac{2}{5}} \sqrt{fR}$	...(2.15m)
$R = \frac{3}{4} \frac{V^2}{f}$	..(2.15a)	$R = \frac{5}{2} \frac{V^2}{f}$	...(2.15am)
$f = \frac{3}{4} \frac{V^2}{R}$	..(2.15b)	$f = \frac{5}{2} \frac{V^2}{R}$	...(2.15bm)

To evaluate the value of C' in Eq.(2.18) we can write from Eq. (2.20) that

$$C'' = C^3/(16)^3 = C^3 / 4096$$

The value of $C = K\,m = \frac{2}{\sqrt{3}}\,m$

Then in Eq.(2. 18) substituting this value and simplifying we get,

$$R^{1/2}S = C'' = C^3/4096 = \left(\frac{2}{\sqrt{3}}\right)^3 m^3 / 4096$$

or $R^{1/2}S = 0.0003727\ m^3 (f^2 = m)$

$$S = 0.0003727 R^{-\frac{1}{2}}\ f^{3/2} \qquad \text{...(2.18a)}$$

$$f = 193.10\ (R^{1/2}S\ )^{2/3} \qquad \text{...(2.18b)}$$

In metric units,

$$S = 0.000204\ R^{-1/2} f^{2/3} \qquad \text{...(2.18am)}$$

$$f = 290\ (R^{1/2}S)^{2/3} \qquad \text{...(2.18bm)}$$

Eq. (2.18b) is of great practical value in computing a working silt factor for irrigation channels. If the channel is in perfect regime it will conform with Eq (2.21) and it will immaterial if the silt factor is computed from Eq.(2.15) or from Eq.(2.18b). The two results will be identical. If the channel is not in perfect regime but represents the optimum standard of local maintenance, and also neither silts nor scours Eq.(2.18b) is evidently a better criterion.

2.5.5 Silt Factor, Grain Size Relation (f - d relation)

Lacey correlated the (f) silt factor to the size of bed material (d) on the basis of that the silt factor f will be dependent on the average size of boundary (d) material in the channel and its density. The specific gravity of transported material in Indian rivers is very nearly that of quartz, i.e. 2.65 and is practically constant. Therefore, in the case of regime channel, in which the transported material and boundary material have the same characteristics, the silt factor (f) can be directly related to the average grain size(d). This attempt was done empirically by Lacey. The data with size of material ranging from boulders in 'Song River' 63.5 cm in diameter and to fine sand in lower Missisipi with f=0.357, were used as criterion of relationship. The relationship has been found as below-

$$d = \frac{f^2}{64} \quad \text{or} \quad f = 8\sqrt{d} \qquad \ldots(2.25)$$

where d is taken in inches. If d is in mm, then

$$f = 1.76\sqrt{d} \qquad \ldots(2.25a)$$

The values of (d) and (f) are given in Table 2.5.

TABLE 2.4

Material of Channels	Values of N_n
Cement Plaster	0.010
Achlar and good brick work	0.013
Rough brick works or good stone work	0.015
Stone work in Poor conditions	0.018
Earthen chan el in Excellent order	0.020
Earthen channel in Moderate order	0.0225
Earthen channel in Poor order	0.0250
Earthen channel in Bad order	0.0275
Earthen channel in Very bad order	0.0300

Table 2.5 [4]

f	d	Na	Type of channel to which applicable
00.400	0.052	0.0179	Very fine silt. Ismailia Canal, Egypt
00.500	00.081	0.0189	Fine silt Madras Godaveri Western Delta
00.600	00.120	0.0198	Fine silt. Jamrao Canals
00.700	00.158	0.0206	Fine s ilt Krishna Western Delta type
00.850	00.233	0.0216	Medium s ilt-Ganges canal Distributory
01.00	00.323	0.0225	Standard silt . Punjab delta
01.250	00.505	0.0238	Medium sand Griffith
01.500	00.725	0.0250	Coarse sand, Kennedy, Buckley.
01.750	00.988	0.0259	Fine Bajri and sand N.W.F. Province
02.000	01.290	0.0268	Heavy sand Griffith
02.750	02.420	0.0290	Coarse Bajri and sand N.W.F. Province
04.750	07.280	0.0333	Coarse gravel
09.750	26.100	0.0390	Gravel and Bajri N.W.F.Province
12.500	50.400	0.0424	Boulders and Gravels "
15.000	72.500	0.0442	Boulders and Shingle Jamuna River
24.150	188.80	0.0500	Large boulders and shingle Song River, Sowaliks United Province

2.5.6 Derived Relationship

Lacey obtained from Lindley observation , Lindley had stated that he saw no reason why the law governing velocities and widths should not be of the same character as that governing velocity and depth [4] . This idea has implicit concept of channel section generated by forces resolved into lateral and vertical components. From this Lacey argued that

$$V = F\ (fP)$$

or $fP = f'P' = f''P''$, etc ..= $F'(V)$.

From the equation (2.15), in all channels of the same mean velocity, it follows that

$$fR = f'R' = f''R''\ ,\ \text{etc.} \ldots = F''(V)$$

Should also hold good. Multiplying the series together

$Af^2 = A'f'^2 = A''f''^2$, ect. ..... $= F_1(V)$

or $Af^2 = F_1(V)$

From available data (Kennedy, Madras and Lindley) plotted by Lacey, it has been found

$$Af^2 = 4.0\ V^5 \quad \text{(infps units)} \quad \ldots(2.26)$$

and $$Af^2 = 140\ V^5 \quad \text{(in metric units)} \quad \ldots(2.26m)$$

by dividing the first series by second we will get,

$$P/R = 7.12\ V \quad \text{(in fps units)} \quad \ldots(2.27)$$

Now we have deduced three basic equations by which all other equations can be deriven as below. Basic equations are-

fps Units		Metric units.	
$V = \frac{2}{\sqrt{3}} \sqrt{fR}$	..(2.15)	$V = \sqrt{\frac{2}{5} fR}$	..(2.15m)
$Af^2 = 4.0\ V^5$	..(2.26)	$Af^2 = 140\ V^5$	.. (2.26m)
$V = 16.0\ R^{2/3} S^{1/3}$	..(2.21)	$V = 10.8\ R^{2/3} S^{1/3}$	..(2.21m)

(1) P and Q relationship

Raising both sides of Eq. 2.15m to fourth power

$$V^4 = \frac{4}{25} f^2 R^2$$

Eliminating f^2 from this equ tion and Eq. 2.26 we get,

$A\left(\frac{25}{4R^2}\right) = 140.0\ V^5$ or $A\left(\frac{25}{4R^2}\right) = 140.0\ V.$

Multiplying both sides, by A, we get,

$\frac{25A^2}{4R^2} = 140\ AV = 140\ Q$ and $P^2 = \frac{A^2}{R^2}$

Then $P^2 = \frac{4 \times 140\ Q}{25} = \frac{56\ Q}{25}$

$\therefore \quad P = 4.75\ \sqrt{Q}$ ...(2.28m)

In fps units $P = \frac{8}{3}\sqrt{Q}$ ...(2.28)

2. V-Q-f Relationship

From Eq. (2.26), by multiplying both sides by V we get,

$$Af^2V = 140.0V^6$$

$$Qf^2 = 140.0\ V^6$$

$$V = \left[\frac{Qf^2}{140.0}\right]^{1/6} \quad \text{...(2.29m)}$$

$$V = \left[\frac{Qf^2}{4}\right]^{1/6} \quad \text{...(2.29)}$$

3. S-Q-f Relationship

Cubing both sides of Eq. 2.21m

$$V^3 = 1260\ R^2S \quad \text{or} \quad S = \left(\frac{V^2}{R}\right)^{5/3} \frac{1}{1260\ (RV)^{1/3}}$$

...(2.21mA)

From Eq. 2.15m $V^2/R = \frac{2}{5} f$ and putting $R = A/P$ and $P = 4.75\ Q^{1/2}$ We will get,

$$S = (\tfrac{2}{5} f)^{5/3} \frac{1}{1260(A_P V)^{1/3}} = \frac{(\frac{2}{5})^{5/3} f^{5/3}}{1260(\frac{Q}{P})^{1/3}}$$

$$= \frac{(\frac{2}{5})^{5/3} f^{5/3}}{1260(\frac{Q}{4.75Q^{1/2}})^{1/3}}$$

$$S = 0.00030 \frac{f^{5/3}}{Q^{1/6}} \qquad \ldots(2.30)$$

In metric units $S = 0.000542\ (f^{5/3}/Q^{1/6})$ ...(2.30m)

4. R-S-f Relations-hips

Cubing both sides of Eq. 2.18m and Eq. 2.14m and eliminating V^3 between both equations we get,

$$V^3 = 1260\ R^2S \quad \text{and } V^3 = (\tfrac{2}{5})^{3/2}\ f^{3/2}\ R^{3/2}$$

$$\text{or } S = \frac{f^{3/2}}{4980\ R^{1/2}} = 0.000204 \frac{f^{3/2}}{R^{1/2}} \qquad \ldots(2.31m)$$

$$\text{In fps Units } S = 0.000373 \frac{f^{3/2}}{R^{1/2}} \qquad \ldots(2.31)$$

5. S-f-q relationship

If we put q = RV and make some simplification in Eq. (2.21mA) we get,

$$S = (\tfrac{2}{5})^{5/3} \frac{f^{5/3}}{1260\ q^{1/3}} = 0.000178 \frac{f^{5/3}}{q^{1/3}} \qquad \ldots(2.32m)$$

$$\text{(In fps) } S = \frac{(3/4\ f)^{5/3}}{4131.1} q^{-1/3} = 0.000391 \frac{f^{5/3}}{q^{1/3}} \quad ..(2.32)$$

6. Regime Scour Depth Relationship

From Eq.2.15am as already derived,

$$R^2 = \frac{25}{4} \frac{V^4}{f^2}$$

$$\text{or } R = \frac{5}{2} \frac{V^2}{f}$$

And from Eq. 2.15m as alreadyderived,

$$V^2 = \left(\frac{Qf^2}{140}\right)^{1/3}$$

Hence,

$$R = \frac{5}{2} \left(\frac{Qf^2}{140}\right)^{1/3} \frac{1}{f} = 0.47\ (Q/f)^{1/3} \quad ...(2.33m)$$

$$\text{(In fps units) } R = 0.47\ (Q/f)^{1/3} \quad ...(2.33)$$

If we put $q = RV$, and $R = \frac{5}{2} \frac{V^2}{f}$, then

$$q = \frac{5}{2} \frac{V^3}{f} \quad ..(2.33a)$$

Substituting for V from Eq.(2.29m) we get

$$q = \frac{5}{2} \cdot \frac{1}{f} \left(\frac{Qf^2}{140}\right)^{3/6} = 0.21\ Q^{1/2} \quad ..(2.34)m$$

$$\text{in fps units } q = 0.375\ Q^{1/2} \quad ..(2.34)$$

By substituting for $Q = (q/0.21)^2$ in Eq. 2.33 it will be

$$R = 0.47 \frac{1}{f^{1/3}} \left(\frac{q}{0.21}\right)^{2/3} = 1.35\left(\frac{q^2}{f}\right)^{1/3} \quad ...(2.35m)$$

$$\text{In fps units, } R = 0.9\ (q^2/f)^{1/3} \quad ..(2.35)$$

The Idea of Shock

Lacey proposed the value of absolute rugosity coefficient should be constant and independent of channel conditions and dependent only on the grade of boundary. Therefore he divided the slope into two parts - one to overcome the friction and other to meet the losses due to irregularities or mounds, bends and channels conditions. This new idea introduced the concept of s-hock losses.

The value of shock losses due to irregularities roughly coincides with form resistance. But the variation of rugosity coefficient was observed by canal engineers. This variation probably is due to:

1. The actual relationship between mean velocity, channel slope, depth and rugosity coefficient is logarithm

   $$V/V_* = 6.25 + 5.75 \log \frac{R}{K_s}$$

   But it is approximated over a wide range by one sixth power exponential equation as $V/V_* = 8.12(R/K_s)^{1/6}$. But Lacey's equation is in the form $V/V_* \propto (R/K_s)^{1/4}$, and this is valid only in a narrow range of low R/K_s values.
2. In case of movable bed channels, ripple formation takes place on the bed. When the flow takes place over these ripples, the pressure on the front (the side facing the flow) of these ripples are more than the pressure on their rear. This difference in pressure adds to a considerable proportion of the total force balancing the tangential component of the weight of water and only the remainder is provided by friction resistance. This kind

of resistance to flow caused by the form of channel bed and bank is called form resistance. The form resistance is a function of size, shape and spacing of the forms, but these are themselves dependent on the flow condition. Therefore, there is no reliable method, at present to calculate the form resistance. Thus the form 'resistance' and friction 'resistance' are different in nature and hence the total resistance cannot be related to grain size alone.

3. Lacey realized that true regime channels are free from external resistance and 'shock" . But in sediment transporting channel, which normally have bed ripple formation, this statement is not correct.

If S' denotes the slope required to withstand shock losses. Then Eq.(2.23) can be written as below-

$$V = \frac{1}{N_a} R^{3/4} (S-S')^{1/2} \quad \text{(in metric units)} \qquad ...(2.36m)$$

$$V = \frac{1.346\, R^{3/4}}{N_a} (S-S')^{1/2} \quad \text{(in fps units)} \qquad 2.36$$

For S' , he suggested values corresponding to n in kutter's formula as given in Table 2.6.

Table 2.6

Lacey's shock factor 'S'' for different channel conditions as given by Buckley.

Channel condition.	n	S'
Very good	0.0225	0.0050 S
Good	0.0250	0.1905 S
Indifferent	0.0275	0.3315 S
Bad	0.0300	0.4375 S

Now if we consider a channel in good condition, then from Table 2.4 for good condition N_a=0.025 and for very good condition N_a=0.0225. Then,

$$\frac{R^{3/4}(S-S')^{1/2}}{0.0225} = \frac{R^{3/4}S^{1/2}}{0.025} \text{ or } (S-S')^{1/2} = 0.9S^{1/2}$$

or S' = 0.19 S.

This means that in good condition, the channel adjusts by 19% of gross slope to over come 'shock.' In practice the value of Na cannot be constant and different values of N_a must be used for different channel conditions as given in Table 2.4 or Table 2.6.

2.5.7 Lacey's Works in 1946-1966 [16,17,18,19]

Lacey has modified his equations in subsequent publications. In 1946, he also sought support from model analysis theory and presented physical basis for his equations [16]. He demonstrated that his equations could be made dimensionally homogeneous. In 1958 he discussed the important parameters that govern the various flow formulae [17]. He argued that for the cross-section, the width and depth are the correct parameters to use rather than P and R. But in all equations involving gradient the hydraulic mean depth 'R', enters as a variable. Therefore in Eq.(2.15) D_m should be used in place of R and in Eq. 2.28, W_s in place of P, in slope formula 'R' remains there. In 1964 [19] Lacey further modified his equation by introducing two new factors c and E which was defined as

$$E = P/W_s = \frac{D_m}{R} \qquad \ldots(2.37)$$

$$e = \frac{0.375}{Q^{1/2}} W_s$$, e is a correction factor for using W_s instead of perimeter P in formula $P = 2.67\, Q^{1/2}$

Thus his new equations are - (fps)

$$W_s = 2.67\ Q^{1/2} \qquad \ldots(2.38)$$

$$D_m = 0.47 \left(\frac{Q}{e^2 f} \right)^{1/3} \qquad \ldots(2.39)$$

$$S_* = 1.73 \ \frac{ED_m}{W_s} \qquad \ldots(2.40)$$

$$A = 1.26\ (e/f)^{1/3}\ Q^{5/6} \qquad \ldots(2.41)$$

Where D_m = mean depth, W_s = Water surface] So that $W_s D_m = PR = A$.

Lacey used [17] the data which were collected by Buckley at Baleida discharge site on the Nile in Egypt in 1921. He plotted the square root of Froude Number ($\frac{V^2}{gR}$ x 10^4) against sediment concentration, N. He demonstrated that for a given sediment grade the parameter V/RS is constant and is independent of sediment concentration. He obtained the equation

$$V \propto \frac{g^{1/2}}{d^{1/2}} \ R.S \qquad \ldots(2.42)$$

In his latter paper [18] Lacey discarded his silt factor completely and instead assumed, the sediment concentration N in ppm . Settling velocity V_s and median diameter of bed material d.

$$V = \text{constant } q^{1/3} \ (NV_s)^{1/6} \quad .. (2.43)$$

$$D_m = \text{Const } q^{2/3} \ (NV_s)^{1/6} \quad ...(2.44)$$

$$S/E = \text{Const. } (NV_s)^{1/3} \frac{d^{1/2}}{q^{1/3}} \quad ...(2.45)$$

$$q = D_m . V = 0.375 \ Q^{1/2} \quad ...(2.46)$$

While using D_m and W_s are often more convenient to use, the formulae utilizing the revised not tion have not so far been substantiated by data . However, except in small channels, the differences are not very significant, and either, system may be used. Using D_m instead of R in Eq. 2.15 , and rearranging,

$$f^{1/2} = \frac{4.9 \ V}{(gd_m)^{1/2}} = 4.9 \ F \quad ...(2.47)$$

where F is Froude number and g is gravitational accelera- tion . This equation is very useful in connection with model.

2.6 PUNJAB IRRIGATION RES ARCH INSTITUTE EQUATIONS

Lacey's paper in 1929-30 impressed all research workers as well as field engineers to think only in one line of empirical approach. The aim of thes investigations was to check up the Lacey's formula by means of statistic theory. The observations showed, that sediment charge plays an important role in design of regime channel. But they have not t ken into account silt charge.

N.K.Bose [20] in 1936, after several years of painstaking data collection and statistical analysis from Punjab's regime channels, obtained the following equation.

Corresponding Lacey's Equations.

$P = 2.68\ Q$	..(2.48)	$P = 2.678\ Q^{1/2}$	..(2.28)
$V = 1.12\ R^{1/2}$	..(2.49)	$V = 1.155\ f^{1/2} R^{1/2}$	..(2.15)
$S = 0.00209\ \dfrac{d^{0.86}}{Q^{0.21}}$	..(2.50)	$S = 0.0014\ \dfrac{d^{0.833}}{Q^{0.166}}$	
$\dfrac{R}{P} = \dfrac{S^{1/4}}{6.25\ d}$	..(2.51)	$(f = 1.76\ \sqrt{d})$	..(2.30)

where d is the particle size in mm.

In 1939-40 , J.K. Malhotra has found [21] out

$$V = 18.18\ R^{0.632}\ S^{0.343} \qquad ...(2.52)$$

Comparing the above equations with Lacey's equations, it is seen that Q-P and V-R relationships are fairly close in the two cases, except the slope equation. In the above equation the power of grain size closely conform but the power of Q is close to 1/5 rather than 1/6.

2.7 INGLIS LACEY EQUATIONS [22,23,24]

In 1941, Inglis statistically analysed the mass of the data collected from Lower Chenab canal and had tried to overcome the shortcomings of Lacey's work in 1929. Inglish objected mainly to Lacey's equations that they have no correlation between sediment charge and canal variables.

In 1936 in a discussion on Dr. Bose Paper [20] he mentioned that Lacey's silt factor (f) cannot be constant in a particular channel, since the value of f depends on bed material size and the bed material size does not remain constant from head to tail.

So Inglis used these notations F_{VR}, F_{RS} and F_m rather than (f) in Lacey's equation as follows (infps units).

$$F_{RS} = 193.1 \; (R^{1/2} \; S)^{2/3}$$

$$F_{VR} = 0.75 \; \frac{V^2}{R}$$

$$F_m = \sqrt{F_{VR} \quad F_{RS}}$$

$$S = \frac{0.000547 \; f_m^{1/3} \quad F_{RS}^{4/3}}{Q^{1/6}} \qquad \ldots(2.53)$$

$$N_a = 0.0225 \; \frac{F_{RS}^{3/4}}{F_{VR}^{1/2}} \qquad \ldots(2.54)$$

$$V = 16 \; R^{2/3} \; S^{1/3} \; \sqrt{\frac{F_{VR}}{F_{RS}}} \qquad \ldots(2.55)$$

The range of the data used by Inglis is as below :

Range	Standard Deviation
P values from 0,82 to 1.45 $\bar{P}$	0.178
V values from 0.89 to 1.21 $\bar{V}$	0.095
S values from 0.69 to 1.31 $\bar{S}$	0.177

where $\bar{P}$, $\bar{V}$, $\bar{S}$ represented mean values obtained by Lacey's equations. Inglis pointed out that these deviations, were not due to chance or coherence, but they resulted because of variation in sand charge entering different channels.

Inglis argued that N the sediment charge and V_s, the settling velocity of particles in still water, are the parameters which fully describe the characteristic of sediment in relation to regime flow. In 1940-41, he concluded by model study in Hydraulic Research station, Poona, that as long as $N.V_s$ is held constant, the amount of material depositing in water was constant and the channel would be in regime, irrespective of whether the load was constant and grade varied or grade was constant and load varied. Following the lead of Prof. White's dimensionless expression he used the parameter (NV_s) in White's equation and gave a new set of equations for quartz sand in water to make a comparison with Lacey's equivalents. The indices of Q in different equations were kept the same as those in Lacey's Three main equations together with corresponding ones from Lacey's are given as below:

$$W_s \propto \frac{I^{1/4}}{g^{1/4}\, d^{1/4}}\, Q^{1/2} \quad \ldots(2.56) \qquad P = \frac{8}{3} Q^{1/2} \quad ..(2.28)$$

$$A \propto \frac{I^{1/12}}{g^{5/12} d^{1/12}}\, Q^{5/6} \quad \ldots(2.57) \qquad A = 1.26 Q^{5/6}/f^{1/3} \quad ..(2.29)$$

$$S \propto \frac{I^{5/12}\, g^{1/2}\, d^{5/12}}{Q^{1/6}} \quad ..(2.58) \qquad S = \frac{f^{5/3}}{1844\, Q^{1/6}} \quad ..(2.30)$$

The others can be derived from the basic ones,

$V \propto I^{1/12} g^{5/12} d^{1/12} Q^{1/6}$ ..(2.59) $V = 0.794 Q^{1/6} f^{1/3}$ ..(2.29)

$D_m \propto \frac{v^{1/9}}{g^{1/18}} \frac{d^{1/6}}{(NV_s)^{1/3}} Q^{1/3}$ ..(2.60)

$R = 0.473 (Q/f)^{1/3}$ ..(2.33)

Where I = Inglis number = $\frac{NV_s}{(vg)^{1/3}}$

In 1957 [23] Inglis evaluated the constants for mobile sandy beds for water with kinematic viscosity $v = 1.25 \times 10^{-5} ft^2/sec$ (at 15 °C). Observation was made on rivers and the formulae were restated by him as below:

$$W_s = 17.8 \left(\frac{NV_s}{d}\right)^{1/4} Q^{1/2} \quad \text{(fps systems)} \quad ...(2.56a)$$

$$D_m = 0.012 \frac{d^{1/6}}{NV_s^{1/3}} Q^{1/3} \quad ...(2.60a)$$

$$V = 4.67 (NV_s d)^{1/12} Q^{1/6} \quad ...(2.59a)$$

$$S = 3.86 (NV_s d)^{5/12} Q^{-1/6} \quad ...(2.58a)$$

$$\text{Froude No. } Fr = \frac{V}{\sqrt{gD}} = 7.51 (NV_s)^{1/4}$$

where

$$N = \text{charge} = \frac{\text{Solid volume of sediment transported/sec.}}{\text{Discharge/sec.}}$$

D_m = mean depth A/W_s in ft, and

d = Weighted mean diameter of sediment in ft.

Inglis stated that the values of constants are only approximate. Therefore the formulae will hold good for canals for a dominant charge during the year and for rivers for dominant discharge, since the charge is related to discharge.

In 1960 in his discussion on the paper 'Uniform Water conveyance channel in alluvial material' by Simons and Albertson, he [24] restated the equations, using the values of d, g, V_s and v as ratios to the standard values at 20°C for quartz grains and Lacey's coefficients as constant, he obtained the following equations in f.p.s. units.

$$P = 2.668 \frac{Q^{1/2}}{(gd)^{1/4}} \left[\frac{NV_s}{(vg)^{1/3}} \right]^{1/4} \quad ...(2.61)$$

$$R = 0.473\, Q^{1/3} \left(\frac{d}{g} \right)^{1/6} \left[\frac{NV_s}{(vg)^{1/3}} \right]^{-1/3} \quad ...(2.62)$$

$$A = 1.26 \frac{Q^{5/6}}{g^{5/12}\; d^{1/12}} \left[\frac{NV_s}{(vg)^{1/3}} \right]^{-1/12} \quad ...(2.63)$$

$$V = 0.794\, Q^{1/6}\, g^{5/12}\; d^{1/12} \left[\frac{N\sqrt{3}}{(gv)^{1/3}} \right]^{\frac{1}{12}} \quad ...(2.64)$$

$$S = 0.000547\, Q^{-1/6}\, g^{1/12}\, d^{5/12} \left[\frac{NV_s}{(vg)^{1/3}} \right]^{\frac{5}{12}} \quad ...(2.65)$$

$$S.V = 0.00043\, (gd)^{1/2} \left[\frac{NV_s}{(vg)^{1/3}} \right]^{1/2} \quad ...(2.66)$$

$$\frac{V}{(gR)^{1/2}} = 1.155 \left[\frac{NV_s}{(vg)^{1/3}} \right]^{1/4} \quad ...(2.67)$$

The Inglis formula will yield zero width, velocity and slope and infinity area for zero silt charge, i.e., for clear water.

The equations have not proved consistent, when they were checked by field and laboratory observations [25].

From the equations it can be concluded that sediment charge N has small effect on the area of the channels, comparatively more effect on width and large effect on slope. The variations in the experimental work is due to channels bed in laboratory, lower discharge, steeper slopes and greater sediment concentrations.

2.8 BLENCH's EQUATIONS

From 1939 onwards, in a number of publications [26-32] T. Blench has been another notable contributor to the regime theory. The existence of different values of silt factor (f) from Lacey's equations impressed him to investigate the reason for it and modify Lacey's equations.

In order to separate out the effects of sides and beds from 'f' and introduce the effect of sides in the slope equation, he used the channel breadth 'W' and depth 'D', instead of Lacey's P and R where W = Average width of channel and D = depth of the channel, so that A = W.D.

Blench has shown that:

$$P = 2.67\,Q^{1/2} \qquad \ldots(2.28)$$

$$\frac{P^2}{Q} = \frac{P^2}{VRP} = \frac{P}{VR} = 2.67^2 = 7.12$$

The left hand side can be written

$V^2/R \div V^3/P = 7.12$, If we replace R by D and P by W, it becomes $V^2/D \div V^3/W = 7.12.$ 108220

From the above expression Blench expected that there must be some hidden dynamical meaning, the constant would depend on bed nature, side nature or both. But the factor V^2/D is proportional to Froude number in terms of depth which must have dynamical meaning, therefore V^3/W must have dynamical meaning, and, presumably, is related to erosive action on sides, in somewhat the same way as V^2/D is related to erosive action on bed.

Blench assumed the side condition smooth and borrowed from rigid boundary hydraulics that the square of mean tractive force intensity on hydraulically smooth sides is $\frac{\rho\mu V^3}{W}$ or $\frac{\rho^2 \nu V^3}{W}$, where ρ is the mass density of water. So for the Renolds numbers, sectional shapes and sides material covered by Lacey's analysis, he concluded that the normal side flow is smooth turbulent. Analysis of W/D, in terms of Renold's numbers and other nondimensional variables leads King to discover that a double logarithmic plot of gDS/V^2 against VW/ν gave a straight line of slope $(-1/4)$. Now, if the friction factor$\left(\frac{2gDS}{V^2}\right)$ for smooth circular pipes of diameter D is plotted against VD/ν, the Blasius straight line of rigid - boundary hydraulics is obtained with slope minus 1/4, so the discovery was that the regime boundary is just a generalised smooth one, Thus, for channels with cohesive sides, sand dunes on bed, and very small bed-load charge, the original Lacey equation became generalised to

$$F_b = V^2/D \qquad \ldots(2.68)$$

defining bed-sediment factor or 'bed factor'

$$F_s = V^3/W \qquad \ldots(2.69)$$

defining 'side-factor'

The later factor multiplied by $\rho^2 v$, is believed to be a factor in, but not a proportion to, the square of the mean tractive force intensity on the sides, provided, as appears to be the case in moderately well maintained channels, the sides are hydraulically smooth, and

$$\frac{V^2}{gDS} = 3.63 \left(\frac{VW}{v} \right)^{0.25} \qquad \ldots(2.70)$$

which is a generalized form of the Blasius equation for smooth rigid circular pipes. The practical design form of equations are

$$W \doteq \sqrt{\frac{F_b Q}{F_s}} \qquad \ldots(2.70a)$$

$$D = \sqrt[3]{\frac{F_s Q}{F_b^2}} \qquad \ldots(2.71b)$$

$$S = \frac{F_b^{5/6} F_s^{1/12}}{\left[\frac{3.63g}{v^{1/4}} \right]} Q^{-1/6} \qquad \ldots(2.72a)$$

According to Eq.(2.70) the value of Chezy Coefficient without silt charge C_o is given by Blench as below

$$\frac{C_o^2}{g} = 3.63 \left(\frac{VW}{v} \right)^{1/4} \qquad \ldots(2.70b)$$

For large bed-load charges, dunes on the bed the constant 3.63 should be replaced by 3.63(1+ac) or,

$$S = \frac{F_b^{5/6} F_s^{1/12} v^{1/4}}{3.63(1+ac) g Q^{1/6}} \qquad ...(2.72b)$$

In which the value of a is a constant very poorly known but recommended-tentatively as 1/233 for natural sands and gravels and 1/400 for uniform sand such as used in the classic Gilbert flume experiments, C is a bed-load charge in hundred-thousandths by weight. But the whole term(1+ac) is unlikely to differ appreciably from 1.0 except in canal with abnormal loads.

According to Blench the shape of the channel cros-ssection depends on F_b/F_s ratio. He proposed the shape of the channel as trapezoidal section with side slope about two upon one.

The recommended values for side factor in design are as follows:

F_s = 0.1 for bank materials of slight cohesiveness

F_s = 0.2 for bank materials of medium cohesiveness

F_s = 0.3 for bank materials of high cohesiveness

F_s = $F_{bs}^2/8$ for rounded gravel, imbedded in fines with slight cohesiveness where F_{bs} means the bed factor that the side material would have if it were bed material of small charge. But this is not accurate.

For the cohesive materials, practical experience with relatively warm water may be transferred to colder water, if change of temperature is assumed not to affect cohesiveness Thus if F_{s1} is a side factor found suitable for a given material of kinematic viscosity v_1, the same material would work to a factor F_{s2} in water of kinematic viscosity

v_2 where $v_1F_{s1} = v_2F_{s2}$

Bed Factor F_b

A – for Small charge (1) $F_{bo} = 1.9 \sqrt{d_m}$, in which suffix zero means bed load is very small, and d_m is median diameter of bed sample in millimeters in sand range – Table 2.7.

For Gravels $F_{bo} \propto d_m^{1/3}$

in which the proportionality constant should be determined from predominant gravel size. Gravels allowed to canals are rare.

B– for Appreciable Bed-Load Charge

(i) For dune in natural bed material upto and including fine gravels–

$$F_b = F_{bo}(1+0.12C) \qquad \ldots(2.73)$$

For coarser natural material, the same formula may be used with mental reservation.

(ii) For sheet flow (when the dunes are vanished) which will be assumed to start suddenly of critical velocity $(gD)^{1/2}$ although there is probably a transition stage, with uniform material upto and including gravel.

$$F_b = 32.2 + 0.06\,(C-C_c) \qquad \ldots(2.74)$$

where C_c is the critical charge occurring when F_b=32.2 ft/sec^2 = g or it can be found from the Eq. (2.74) by inserting F_b= g = 32.2 ft/sec^2. For coarser material it should be used with mental reservation.

(iii) For antidunes nothing is known definitely.

Limit of Small Charge

1- If the bed factor had been determined by a method that did not require C to be known the value should be C ≤ 10.

2- Practical value with sand canal C ≤ 2.0

Material Classification by size [31]

Material of bed has been classified by Blench as below.

Table 2.7

Top limits (mm)	.002	.006	.02	.06	.2	.6	2	6	20	60	> 60
Grade	-	Fine	Mod.	Coarse	Fine	Mod.	Coarse	Fine	Mod.	Coarse	
Class	Clay	Silt			Sand			Gravel			°

°Boulders.

2.9 OTHER REGIME FORMULAE

Parallel with development of regime methods in India many measurements of canals and river dimensions have been made in different parts of the world since Lacey put forward his correlations. Among these are those given by Ghaleb (Egypt), Mathews (Sudan), Marshall Nixon (U.K.), Pettis (Florida, U.S.A) Leopold and Maddock (Western U.S.A) and so on.

Most of these research workers through out the world find that there is a significant correlation between each channel dimension and equilibrium discharge. The relations given by them have nearly the same index of discharge as given by Lacey, but the numerical constants, however, show considerable variation.

Even so, one cannot but feel impressed by remarkable uniformity of natural laws-the data from so many different sources yielding similar equations.

Some of the main works of these investigators are briefly discussed in subsequent sections.

2.9.1 Egyptian Equations

Ghaleb in 1929-30 published a formula for nonsilting and non-scouring conditions as below:

$V_o = 0.39\ D^{0.73}$ (in fps units) ...(2.75)

This looks like Kennedy's equation, but the exponent of D is higher. Subsequently this exponent has been changed to 2/3.

In upper Egypt and Lower Egypt Molesworth and Yordania, after a careful examination of large number of stable canals recommended the following relations:

$V_o = 0.39\ D^{2/3}$ for upper Egypt ...(2.76)

$V_o = 0.475\ D^{2/3}$ for lower Egypt ...(2.77)

$D = (90608 + 0.725) \sqrt{B}$ ...(2.78)

2.9.2 Mathew's Work [33] (Sudan)

Mr. Mathews studied 39 reaches of the main canals in Sudan (Gizira) which have been operating perfectly without silt clearance for 15 years.

The data which he had investigated were divided in two groups, i.e., one was for canals stable for 15 years and of 10 reaches with discharge range 0.85-10 m^3/sec, and the other consisted of 29 reaches with discharge ranging from 19 to 120 m^3/sec. and have been concerning canals stable for 3-6 years.

The canals were constructed in soils whose properties are given as below:

Sand and cement	greater than 2 mm dia.	2.6%
Coarse sand	0.2mm to 2 mm	8.6%
Fine sand	0.02 to 0.2 mm	23.3%
Silt	0.002mm to 0.02mm	9.5%
Clay	Less than 0.002mm	56 %

The sample of suspended sediment has the following properties

0.2 mm	Nil
0.04mm -0.2 mm	28.3%
0.04 mm	71.7%

The volume of suspended material has not been ascertained. He has tried to verify the Lacey's equations as below:

$$P = K_p \, Q^{1/2} \qquad \ldots(2.79)$$

$$A = K_a \, Q^{5/6} \qquad \ldots(2.80)$$

$$S = K_s \, Q^{-1/6} \qquad \ldots(2.81)$$

where S is measured in cm/km and K_p, K_a and K_s are constants. The average values of constants, the arithmatic means and scatter of individual values from these means are as follows:

(table on next page)

It can be seen that there is a close agreement between K_p and Lacey's value 4.84 , but there is wide variation in the value of K_s. Though there is considerable scatter in the values of constants for each reach.

Constants (Average) in metric System	Lacey's values	No.of reaches	% of observations within the following range of values			
			5%	10%	15%	25%
K_p = 4.91	4.84	26	46	73	96	100
K_a = 2.64	3.39	39	23	49	74	100
K_s = 14.05	30.0	39	5	25	38	69

He also determined the value of F_{VR} and F_{RS},

F_{VR} = 0.629 and F_{RS} = 0.623

Comments

The type of soil in which the flow took place was clayey which has resistance to scour and is not the same as Lacey's proposed, i.e. incoherent alluvium.

The value of K_p corresponds very well to Lacey's, but there is small variation in K_a and the value of K_s varies much from Lacey's and also has too much scatter.

2.9.3 White's Formulae (U.K.) [34]

Dr. C.M.White of the Imperial College of Science, London tackled the problems from a dimensional approach in 1939. His work consists of correlating various variables which he considered as dependent and other independent variables. He mentioned the main independent variables in rivers, the discharge Q, grain size d, sediment charge NQ, fluid density, fluid viscosity ν, density of the grain, shape of grain, fall velocity of grains as independent variables, and channel geometry, slope and velocity as dependent variables.

White used fall velocity V_s, of grain in still fluid, which can describe the properties of sediment such as grade shape and specific gravity and fluid density and visicosity ν.

He deduced two primarily non-dimensional ratios for independent variable, i.e. $\frac{g^{5/2}\, Q^{1/5}}{V_s}$ and N.

For the dependent variables, he used cross-sectional area as the dimensionless ratio, $\frac{Ag^{2/5}}{Q^{4/5}}$, He thought that this factor can be measured very easily. The functional relation is therefore,

$$\frac{Ag^{2/5}}{Q^{4/5}} = \phi \left[\frac{(g^{2/5}\, Q^{1/5})}{V_s} , N \right]$$ where N=sediment concentration.

By using available data from river , with a discharge range 10^8, the function was found to be nearly independent of N and obtained as

$$\frac{Ag^{2/5}}{Q^{4/5}} = 2.4 \left(\frac{g^{2/5}\, Q^{1/5}}{V_s} \right)^{0.22} \quad ...(2.82)$$

The other two variables R and S were considered to be strongly dependent on N . Within the range of (estimated as 200 - 1000 ppm) of the available data he obtained approximately

$$\frac{V}{\sqrt{gR}} = 0.7 \left(\frac{V_s}{g^{2/5}\, Q^{1/5}} \right)^{1/4} \pm 20\% \text{ (varies with N)} \quad ..(2.83)$$

$$S \doteq 0.012 \left[\frac{V_s}{g^{2/5}\, Q^{1/5}} \right]^{0.9} \pm 50\% \text{ (varies with N)} \quad ..(2.84)$$

These equations can be compared to Lacey's equations (in fps units)

Lacey's

$$A = \frac{2.4\, Q^{0.844}}{g^{0.312}\, V_s^{0.22}} \qquad\qquad A = \frac{1.26\, Q^{0.833}}{f^{1/3}}$$

$$S = \frac{0.012\, V_s^{0.9}}{g^{0.36}\, Q^{0.18}} \qquad\qquad S = \frac{1}{1844}\ \frac{f^{5/3}}{Q^{0.167}}$$

$$\frac{V}{(gR)^{1/2}} = \frac{0.7\, V_s^{1/4}}{g^{10}\, Q^{1/20}} \qquad\qquad f = 0.75\ \frac{V^2}{R} = \frac{0.3875\, g^{4/5} V^{1/2}}{Q^{1/10}}$$

The following results can be drawn from the above equations-

1. Similarity of the index of significant variables with different data and premises of sets support each other quite well.
2. Due to non-availability of sediment data the equations 2.83 and 2.84 could not be correlated to N.
3. The channel geometry, meanders, shoaling, nature of the banks etc are decided by values of two fundamental parameters N and $\dfrac{g^{2/5}\, Q^{1/5}}{V_s}$.

Dr. Bharat Singh [35] made a little algebraic manipulation in Eq.(2.82,2.83) of White and derived this equation

$$P = \frac{4.04}{g^{0.136}}\ \frac{Q^{0.432}}{V_s^{0.16}} \qquad \ldots(2.85)$$

The index of this equation is 14% less than Lacey's equation. He concluded that the differences were not too much in moderate discharges and material size in regime range when he tested in different discharges and different material sizes [35]. But for coarser material gave narrow channels .

2.9.4 Marshall Nixon (U.K) [36]

Marshall Nixon studied the behaviour of rivers in England and Wales collecting data from 11 rivers. Since in a river the discharge is fluctuating severelly and and it is difficult to decide which discharge should be related to variables. Inglis mentioned that there is a dominant discharge which has the properties as below:

1. Equilibrium between various parameters is achieved,
2. The tendency to be changed is least.
3. The condition is the integrated effect of all varying condition over a long period of time, and
4. The value of this discharge may be 60% of the maximum discharge.

Leopold - Maddock suggested that regime conditions are attained in rivers for discharge having same frequency. Mr. Nixon, therefore, assumed that regime conditions have presumed to have been attained in bankful discharge conditions if they give equal frequency. He has obtained the frequencies of the order of 0.6% which are independent of discharge, even though the ratio of peak discharges to bankful discharges varied widely for different rivers. This means that the flow was equal or exceeded for the rivers studied, 0.63 of time. Then he has found these relations

$$W = 1.65 \ Q_b^{1/2} \qquad ..(2.86)$$

$$D = 0.545 \ Q_b^{1/3} \qquad ...(2.87)$$

$$V = 1.112 \ Q_b^{1/6} \qquad ...(2.88)$$

$$A = 0.9 \ Q_b^{5/6} \qquad ...(2.89)$$

where,

W = width of channel at bankful stage in ft,

D = Depth of channel at bankful stage in ft.

V = Mean velocity at bankful discharge in ft/sec,

A = Area of cross-section at bankful stage (ft^2), and

Q_b = Bankful discharge.

The scatter of the points relating to width is much less than, those relating to velocity and depth, the greatest being those relating to velocities. Then he concluded these results :

1. The rivers adjust themselves in width more readily than they do in depth and that adjustment of velocity and hence slope is a much slower process.
2. Frequency of bankful discharge should be same for all regime rivers.
3. Value of bankful discharge is independent of the size of catchment area and of maximum discharge.
4. If the above equations are satisfied, a self-formed river channel will remain stable or in regime.

5. The above formulae applied only to rivers in England and Wales, but may be used for other rivers with similar sediment load (The suspended sediment in those rivers were 27.2 ppm by weight). It is further noticed that the above formulae give very small width than even given by Lacey for artificial channels.

2.9.5 Pettis Equations (U.S.A) [37]

He investigated the behaviour of the Miami River system in U.S.A. and derived the following equations :

$$V = 0.8\, Q^{0.2} \qquad \ldots(2.90)$$

$$A = 1.25\, Q^{0.8} \qquad \ldots(2.91)$$

$$R = 0.511\, Q^{0.3} \qquad \ldots(2.92)$$

$$P \simeq 2.45\, Q^{0.5}$$

It can be seen that his equations for river system supported regime concept.

2.9.6 Leopold - Maddock's Equations (U.S.A.) [38]

They collected many available data from stream in U.S.A. to investigate the applicability of regime theory to American rivers. The data consisted of depth, width, velocity, discharge, and suspended loads. After analysing those data they concluded that suspended loads and channel dimensions are direct functions of discharge and can be expressed as follows :

$$W = a\, Q^{b} \qquad \ldots(2.93)$$

$$D = c\,Q^{f} \quad \ldots(2.94)$$

$$V = K\,Q^{m} \quad \ldots(2.95)$$

$$L = P\,Q^{j} \quad \ldots(2.96)$$

where,

L = sediment transported in units of wt/unit time.

They made their studies for the following two conditions-

1. Determination of channel geometry and sediment load at a station for discharges of varying frequency.
2. Determination of channel geometry and sediment load at different stations going downstream, for discharges of equal frequency.

By plotting the data expressed in the above equations they found the values of exponents as below:

	At a station (AV)	Downstream direction(AV)
b	0.26	0.50
f	0.40	0.40
m	0.34	0.10

The downstream direction is comparable to regime equations.

Effect of Suspended Load

They have concluded that suspended load concentration increases with discharge. They have also derived empirical quantitative relation among various measurements of width, velocity, discharge, and suspended load . They have concluded that depth, width, as well as velocity are functions of the load transported in the channel.

They plotted j against m/f and have drawn lines of equal values of b. For downstream condition for b=0.5

and $\frac{m}{f} = \frac{0.1}{0.4}$, the value of j was found to be 0.80. Their indices also corroborate Lacey indices reasonably well.

2.9.6 Nedeco Equations for Rivers (North Holand)

Nedeco made a comprehensive study on rivers Niger and Benue for their improvement in 1959. He derived an equation for the channel width 'w', with the following simplifications [39] .

(i) The channel was defined by 'W' and D.

(ii) Characteristics of material were condensed in one figure, the grain size 'd'.

(iii) Roughness of bed and banks was put in one figure for which equivalent sand roughness of Nikuradse 'K_r' was used.

He employed two following equations.

(i) $Q = C.\ W.D^{3/2}\ .S^{1/2}$ (Chezy Equation) ...(2.97)

(ii) $Q_s = 6.5\ W.d^{3/2}\ \Delta^{1/2} g^{1/2} \left(\frac{\mu\ DS}{\Delta d} - 0.047 \right)$ ...(2.98)

(Meyer-Peter and Mueller formula)

where C = Chezy constant,

$$\Delta = \frac{\rho_s - \rho_f}{\rho_f} = 1.68$$

$$\mu = \text{ripple factor} = \left(\frac{K_s}{K_r} \right)^{1/4}$$

K_s = sand roughness of grain in metres.

K_r = equivalent sand roughness of the bed in metres

Q = Water discharge in m^3/sec.

Q_s = Transporting material in m^3/sec.

D = Depth of flow in metres.

W = Water surface width in metres

S = slope of bed

d = sediment grain size, d_{50} in metres.

By combining the two equations we get,

$$\frac{C}{6.5d^{3/2}\Delta^{1/2}g^{1/2}} \cdot \frac{Q_s}{Q} = \frac{\left(\frac{\mu D.S}{\Delta d} - 0.047\right)^{3/2}}{S^{1/2} D^{3/2}} \quad ..(2.98a)$$

If the value of Q_s/Q is constant, equation above may be a function of 'D','S', when other factors are taken constant.

Differentiating equation (2.98a), it can be written,

$$S_{min} = \frac{C Q_s}{8 g^{1/2} \mu^{3/2} Q} \quad ...(2.99)$$

This equation is independent of sand grain size 'd'. Thus there will be three equations for finding the cross-section of the channel.

1. $Q = CWD^{3/2}S^{1/2}$ (Chezey Equation) ...(2.97)

2. $Q_s = Wd^{3/2} S^{1/2} \Delta^{1/2} g^{1/2} f\left(\frac{\Delta d}{\mu DS}\right)$ ..(2.98)

3. $S_{min} = \frac{CQ_s}{8 g^{1/2} \mu^{3/2} Q}$ ..(2.99)

(Minimum slope equation)

The idea of minimum slope is somewhat consistent with Dr. Bharat Singh's contention that a channel first adjusts its section so as to be able to have a maximum capacity of movement of bed load and then adopts the necessary slope[36].

The drawback for this method is that the selection of Chezy coefficient is difficult to be certain.

2.9.7 Simons and Albertson's Regime Theory [40,41]

The main objectives of Simons and Albertson's reseach were as below:

1. To find methods of designing uniform alluvial channels.
2. To modify regime theory of India.
3. A modification of tractive force theory and relating it to regime theory in so far as possible.

For the above mentioned purposes he studied and analysed the following field data:

1. Indian canals (Punjab and Sind)
2. United States
   i. U.S.B.R. data (San Luis Valley of Colorado)
   ii. Canals, in Wyoming, Colorado and Nebraska (Author's data)
   iii. Imperial Valley Canal data.

The range and field conditions of data are summarized in Table 2.8. The imperial valley canals are characterised by their heavy load of suspended sediment.

SALIENT FEATURE OF DATA

Sl. No.	Name of Canal	Type of Soil in which canal is dug.	Discharge cusecs.	Slope	Av.dia. of bed material (mm)	Mean silt intensity ppm	Water Temp. °F	Source
1.	U.S.B.R. San Luis Valley (15 reaches)	Coarse non-cohesive	17-1500	0.79×10^{-3} to 0.97×10^{-3}	Varies considerably	-	65 *	Lane.
2.	Punjab Canals	Alluvium	5-9000	0.12×10^{-3} to 0.34×10^{-3}	0.43	238	70 *	Punjab Irrigatic Res.Inst.reporte the year ending April,1941
3.	Sind canal (13 canals and 28 reaches)	Alluvium	311-9057	0.0592×10^{-3} to 0.0995×10^{-3}	0.0346 to 0.1642	3590 to 156	70 *	C.B.I. Annual Report No.29,194 (Technical)
4.	Imperial Valley canal data	Similar to punjab canals	-	-	-	2500 to 8000 most of this is wash load	-	Fortier, Samuel and Blanny "Silt colorado river a its relation to Irrigation"U.S.I of Agr.Tech.Bull 67:1-94,1928 and B.Chandrasekhara Raja Master Repo Colorado A and M College-1955
5.	Simmons and Bender	Alluvial and Gravel	43- 1039	0.058×10^{-3} to 0.088×10^{-3}	0.096 to 0.805	Varies widely	61.5 to 82	Collected by Sim and Bender.

* Assumed from Climatological data.

2.9.7.2 Regime Flow and Forms of Bed Roughness

In alluvial channel the forms of bed roughness are a function of the bed material, the sediment in transport, and the flow. Resistance to flow in alluvial channels varies between wide limits and is extremely complex.

According to study based on laboratory experiments of alluvial channels and the field study as previously mentioned they inferred that the following forms of bed roughness occurred for different regimes of flow (for well graded material of 0.2 mm median dia.).

Tranquil flow regime $F_r < 1$ [42]	Sediment Load ppm
1. Plane bed no movement	Nil
2. Ripple bed 3. Ripple superposed on dunes	0 to 90
4. Dunes	90 to 1000
5. Transition from dunes to rapid flow	1000 to 3000
6. Plane with movement of bed material	2000 to 5000
Rapid Flow $F_r > 1$	
7. Standing water waves and sand waves 8. Antidunes	In excess of 4000

They argued that the magnitude of the total sediment varies with form of the bed roughness and the relationship between configuration and total sediment load depends on the characteristics of bed material, channel geometry, shear on the bed and proportion of suspended load. If the very fine sediment (wash load) exist in channel as suspended load, it reduces the resistance to flow and influences channel geometry.

2.9.7.3 Channel Stability as Related to Froude Number $\left[Fr = V/\sqrt{gD}\right]$

According to field study of stable channels which was conducted by them and subsequent discussion of channel stability with others, they concluded that for channel to be stable (no appreciable bed and/or bank scour or accretion occuring with time) F_r must in most cases be less than 0.3 for alluvial material in the sand size range and finer When $F_r > 0.3$, the bank scour in the straight reaches of channels and the bends must be stabilized to confine them to their rights-of way. This limitation ($F_r < 0.3$) implies that design problem is confined to tranquil flow regime i.e. the channel should be designed only with ripple or dune form of bed roughness depending on channel slope limitation dictated by the terrain and magnitude of sand load, which the channel must transport.

Therefore the magnitude of sand silt sediment load which can be transported, likewise is severely limited to usually less than 500 ppm. For carrying more load than 500 ppm bank stabilizing is necessary.

They also tried to establish a relation between sediment load, Froude number and channel stability. They made a considerable improvement in the Blench-King Regime slope Formula regarding bed and side factors and variations in the degree of bank cohesion. The regime slope equations recommended by Blench for design formula is

$$S = \frac{F_b^{5/6} \; F_s^{1/12}}{2080 \; r \; Q^{1/6}} \qquad \ldots(2.72a)$$

where,

$$r = \left(\frac{10^{-5}}{\nu} \right)^{1/4}$$ and the basic slope formula

$$\frac{C^2}{g} = \frac{V^2}{gDS} = 3.63 \left(\frac{VW}{\nu} \right)^{1/4} \qquad \ldots(2.70b)$$

They examined this relationship by plotting V/gDS versus VW/ν (Fig. 2.8) and found that the relationship is not perfect. The value of ν corresponding to 70°F, has been assumed for Indian canals.

The Punjab canal data yield points that close to straight line on log-log paper between limits of $10^5 < \frac{VW}{\nu} < 10^7$. Beyond this upper limit, the value of V^2/gDS terms are nearly constant and the slope of the line flattens until it lies approximately parallel to horizontal axis. The sind data lie more or less of an extention of the straight line portion of Punjab data and the Simons and Benders data inter-mingle with Indian canal data.

This shows that the Indian canal data is close to straight line in range of $10^5 < \frac{VW}{\nu} < 10^7$. Beyond $VW/\nu > 10^7$ most points fall below straight line rather than above it. However, this may be a function of canal sampled.

2.9.7.4 Analysis of Data

The tabulated basic data and parameters computed there from were analysed by them to find out all the parameters involved in Lacey's theory and Blench theory. The data of Punjab canals have no average depth on the bed and average width. They gave only P, R and W_s. Therefore the average bed depth was correlated with hydraulic radii in Fig.(2.1) and P was correlated with average width from the data of Simon and Bender and Sind Canals, are plotted inFig.(2.2). The relation between average width, W, and Top width W_T, to be used in circumstances, if it may be advantageous to convert average width, W, to top width

W_T is given in Fig. (2.4).

The imperial valley canal data have been plotted in all figures to show the effect of heavy charge to parameters of flow variables.

They then plotted data as shown in Figs.2.1 to 2.9 with the data groupwise to obtain the relations. The canal data were class ified into following five distinct groups:

A - Canalas with sandy beds and banks or sandy material

B - Canals with sand bed and cohesive banks as in Punjab and Sind.

C - Canals with cohesive bed and banks

D - Canals with coarse-noncohesive as studied by US BR

E - Imperial Valley canal data grouped as B, but with heavy sediment loads (2000-8000 ppm).

The relations which they got between P, R, V and A against discharge are summarized as below-

$$P = K_1 Q^{0.512} \quad \text{Fig.2.3} \quad ..(2.100)$$

$$R = K_2 Q^{0.361} \quad \text{Fig. 2.5} \quad ..(2.101)$$

$$V = K_3 (R^2 S)^d \quad \text{Fig. 2.7} \quad ..(2.102)$$

$$A = K_5 Q^{0.873} \quad \text{Fig.2.6} \quad ..(2.103)$$

$$\frac{C^2}{g} = \frac{V^2}{gDS} = K_4 \left(\frac{VW}{v}\right)^{0.37} \quad \text{Fig.2.8)} \quad ...(2.104)$$

The values of K_1, K_2, K_3, K_4 and d are given below-

	A	B	C	D	E
K_1(P-Q relations)	3.33	2.51	2.12	1.67	1.67
K_2(R-Q relations)	0.52	0.43	0.37	0.247	0.34
K_3($V-R^2S$ relations)	13.86	16.0	-	17.9	15.8
K_4($\frac{C^2}{g}$ etc relation)	0.33	0.54	0.87	-	-
K_5(A-Q relations)	1.07	1.076	-	0.45	-
d (Index of R^2S)	0.33	0.33	-	0.286	0.29

2.9.7.5 Correlation of Tractive Force and Mean Diameter of Bed Material

They also plotted tractive force YRS versus d mean diameter of bed material. This correlation provides a very useful mean of establishing the design slope of channels in coarse non-cohesive materials, provided size of bed and banks material can be estimated with reasonable accuracy. The Fig. (2.9) shows this relations.

From this Figure the following facts are apparent-

1. A general line extending through all of the data can be drawn. There is , however, much scatter about this line.
2. For different classes of canals, secondary lines crossing the major trend line have been drawn.
3. Roughness of channel bed seems to increase travelling from the bottom secondary line to the fourth (top) secondary line associated with sand bed and banks.

4. Next consider the five secondary lines. Moving along these lines in direction of increasing shear it is found that canal capacity increases. The point of extreme right end shows the maximum and that of extreme left of these same lines corresponds to minimum Q_{min}.

5. The Imperial Valley canal data have not been plotted, because of uncertainty regarding the mean size of bed material.

For combining tractive force method and regime theory, now it is required to determine first mean size of bed material, hydraulic radius, type of bank conditions and Q from the foregoing Figures, giving the regime data. Then from Fig. 2.9 the slope S can be found. Since there is much scatter in value for the same material, this figure does not have much utility.

2.9.7.6 <u>Summary and Conclusion</u>

1. Two basically different theories are currently introduced because of superiority over other available existing methods -

(i) The regime theory of India as developed by Kennedy, Lindley, Lacey, Bose, Blench and others, and

(ii) The limiting tractive force theory as proposed by Lane and others.

2. The regime theory has been emphasized by this paper.

3. Indian regime theory is valid only for the limited range of conditions upon which they are based as follows:

   (i) Channels having sand beds, and slightly cohesive to cohesive banks, the banks of which are usually formed by the berming action of suspended sediment.

   (ii) Channels that are not required to carry a heavy charge of sediment for sustained period of time. The upper limit of sediment load (sand and silt size) that can be transported without appreciable bank erosion is 500 ppm.

4. The range of the data for the first four classes of the canal given by the authors are as below-

   Q varies between 5 - 9,000 cfs

   S varies between 0.000058 to 0.000388

   Average width (W) varies between 2ft -264 ft.

   D varies between 2.8 ft to 10.5 ft.

   Sediment concentration varies from 50 -500 ppm. (excluding four canals of class E (2,500-8,000 ppm larger part of which must be wash load)).

5. Within the range of the canal data presented, it is possible to determine area, average width, top width bed depth, average depth, hydraulic radius, wetted perimeter and average velocity with ease and practical

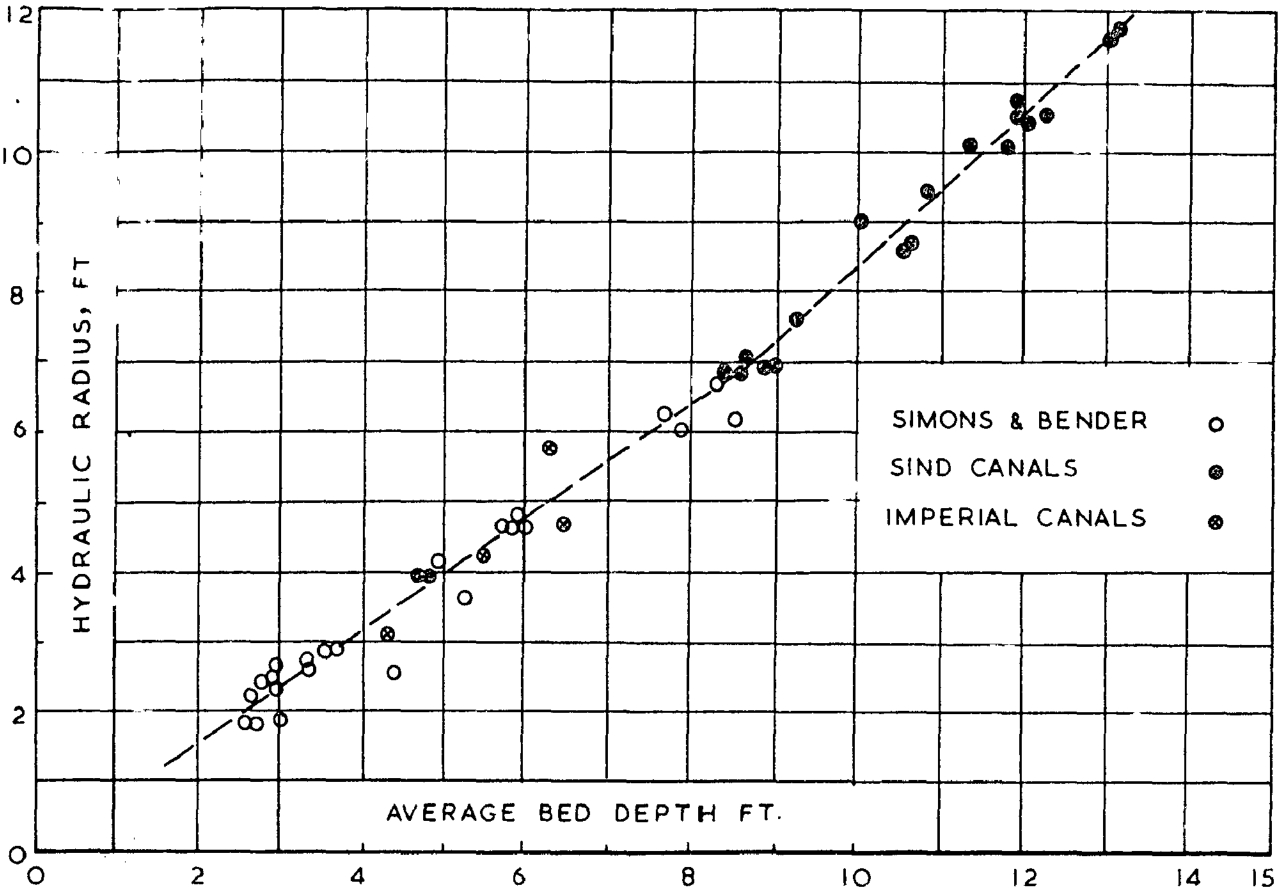


FIG.2.1 VARIATION OF HYDRAULIC RADIUS R WITH DEPTH D

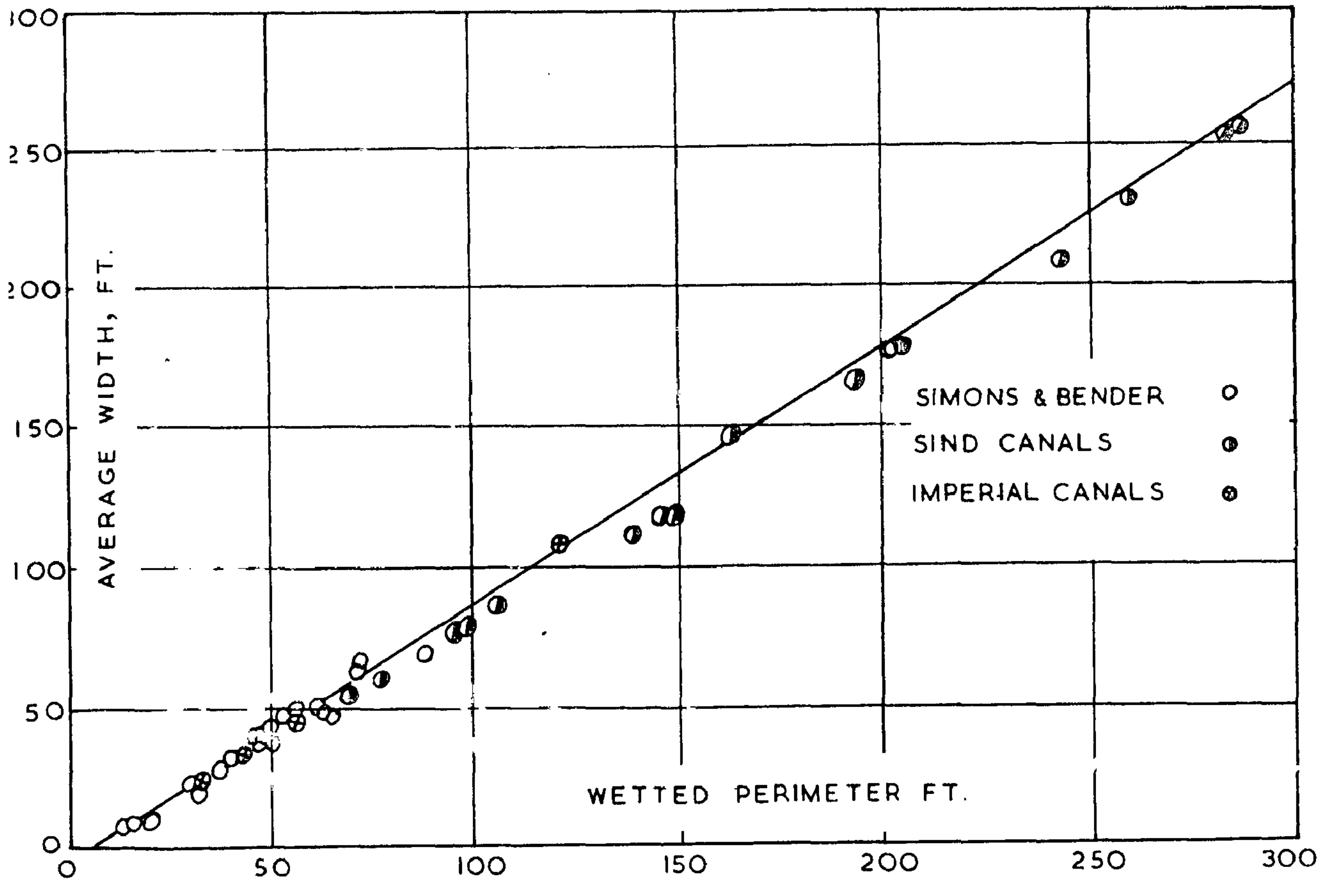


FIG.2.2 VARIATION OF AVERAGE WIDTH W WETTED PERIMETER P

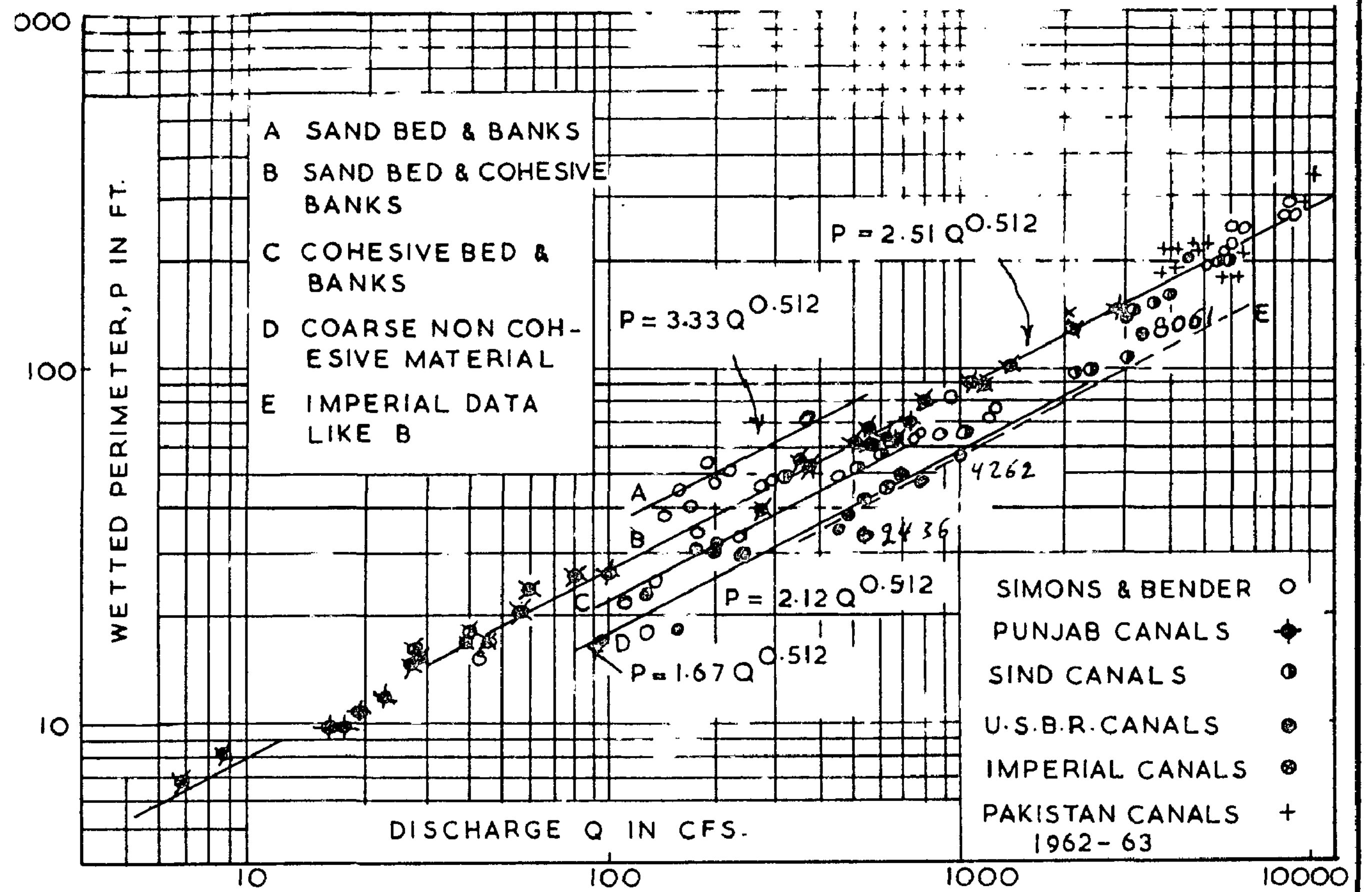


FIG.2.3 VARIATION OF WETTED PERIMETER P WITH DISCHARGE Q AND TYPE OF CHANNEL

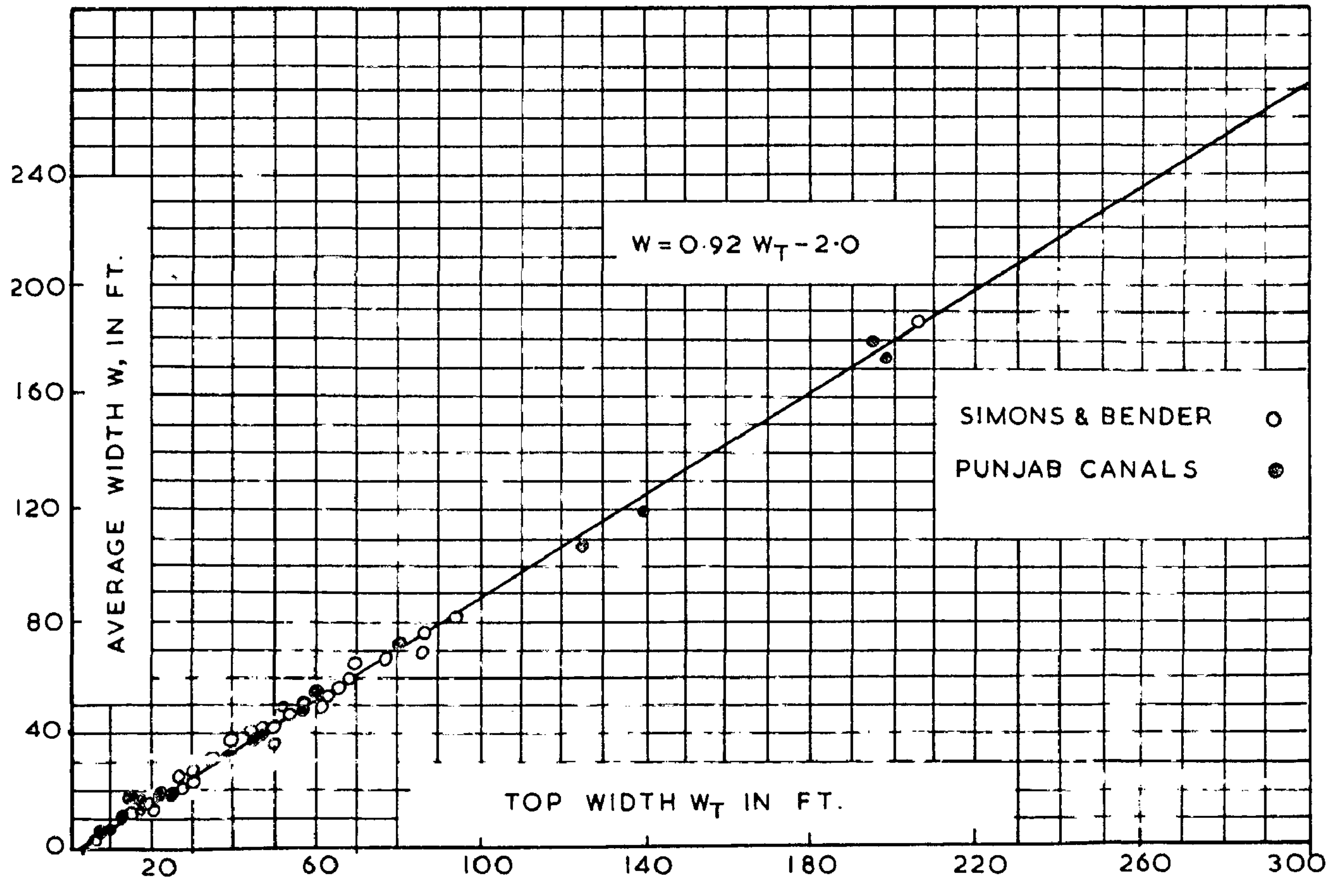


FIG.2.4 VARIATION OF AVERAGE WIDTH W WITH TOP WIDTH W_T

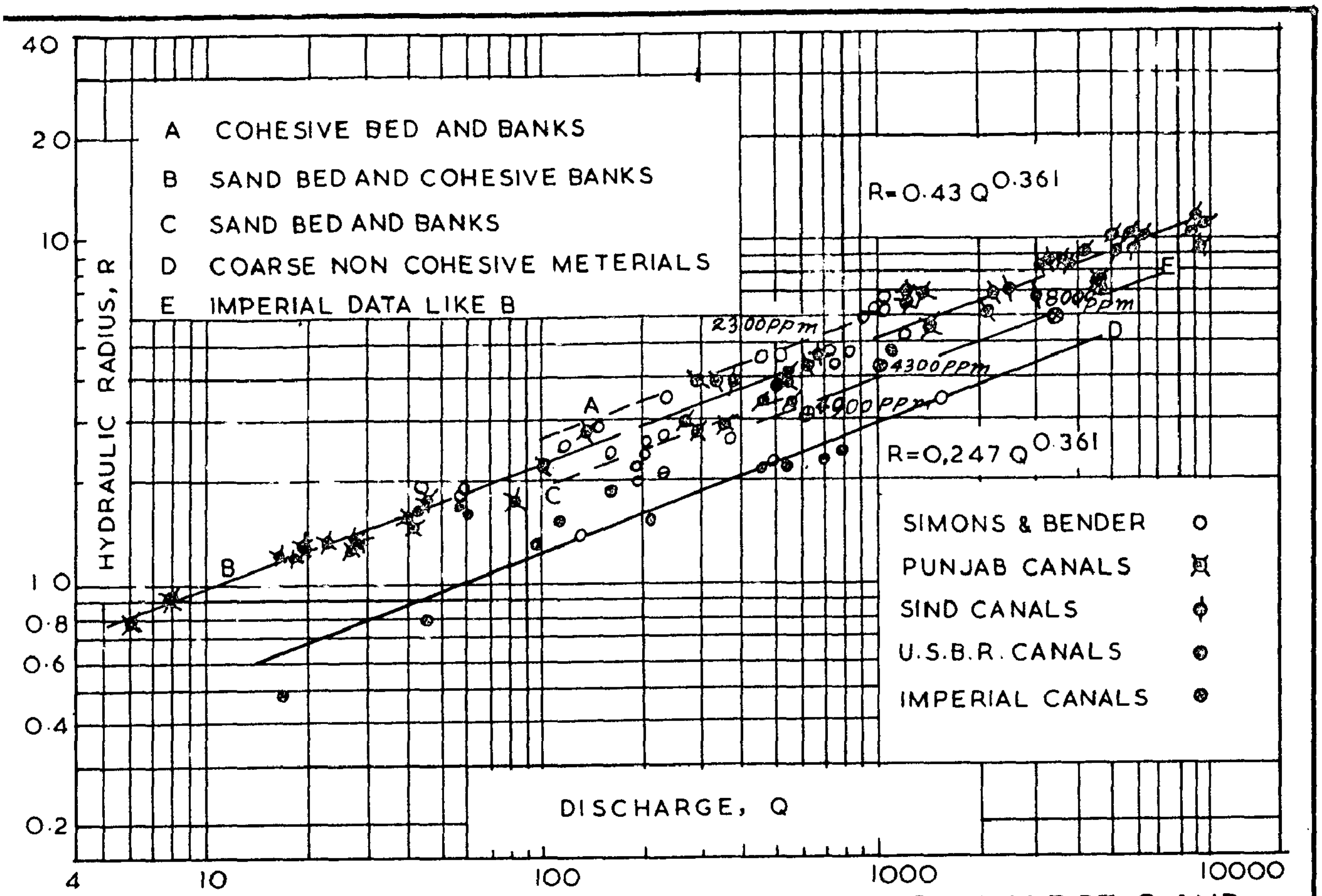


FIG.2.5 VARIATION OF HYDRAULIC RADIUS R WITH DISCHARGE Q AND TYP OF CHANNEL ALL DATA

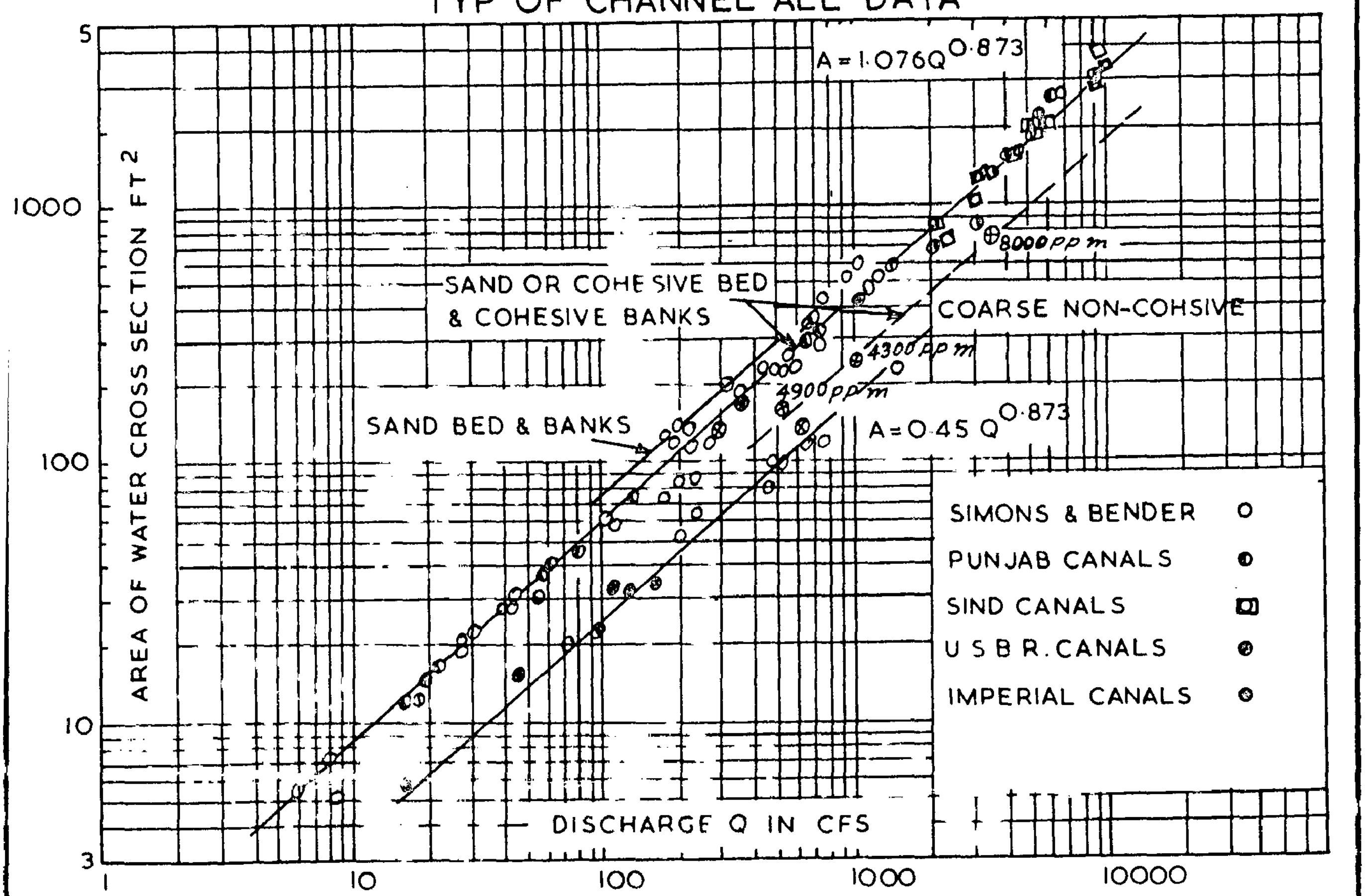


FIG.2.6 VARIATIO OF AREA OF WATER CROSS SECTION A WITH DISCHARGE Q AND TYPE OF CHANNELS

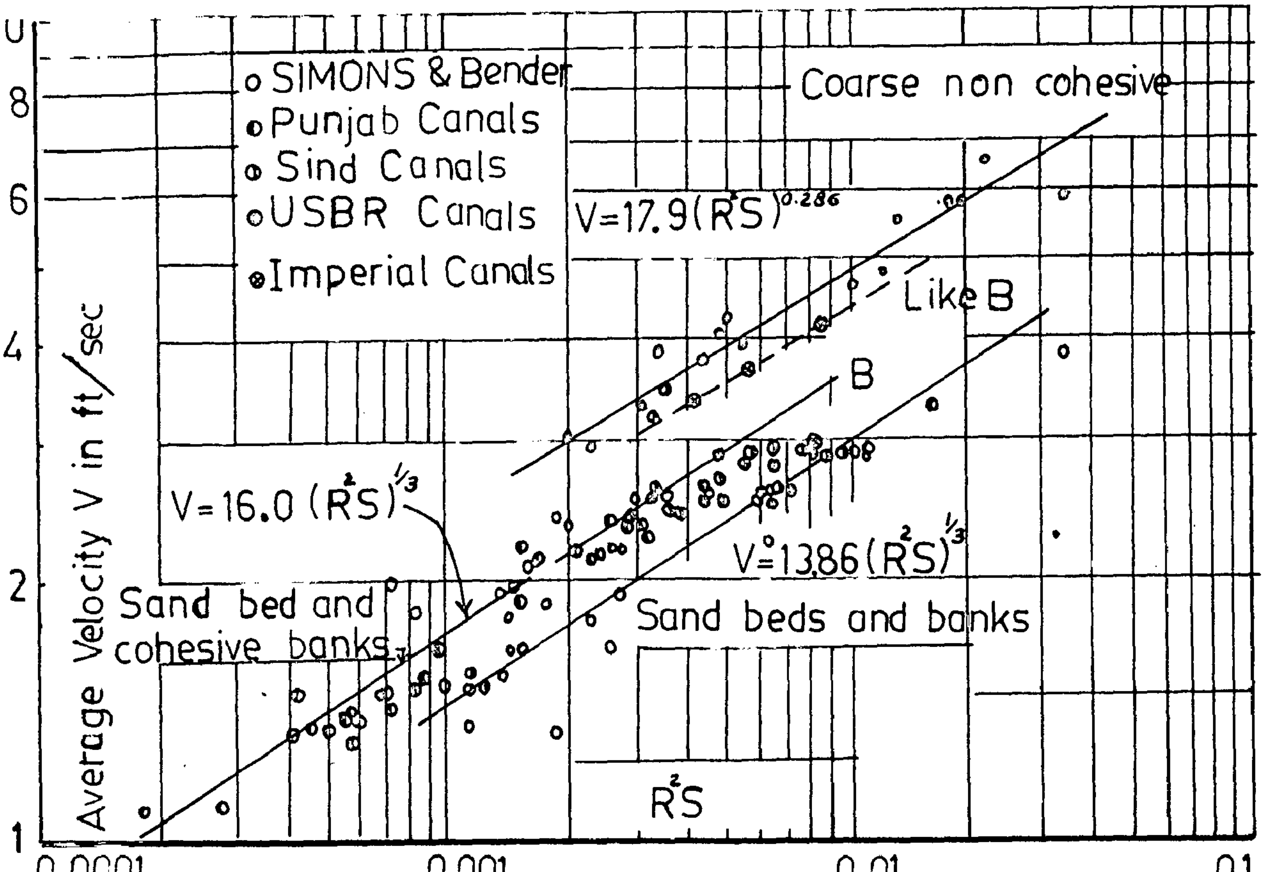


Fig. 2.7 Variation of Average velocity V with R^2S and type of channel all data

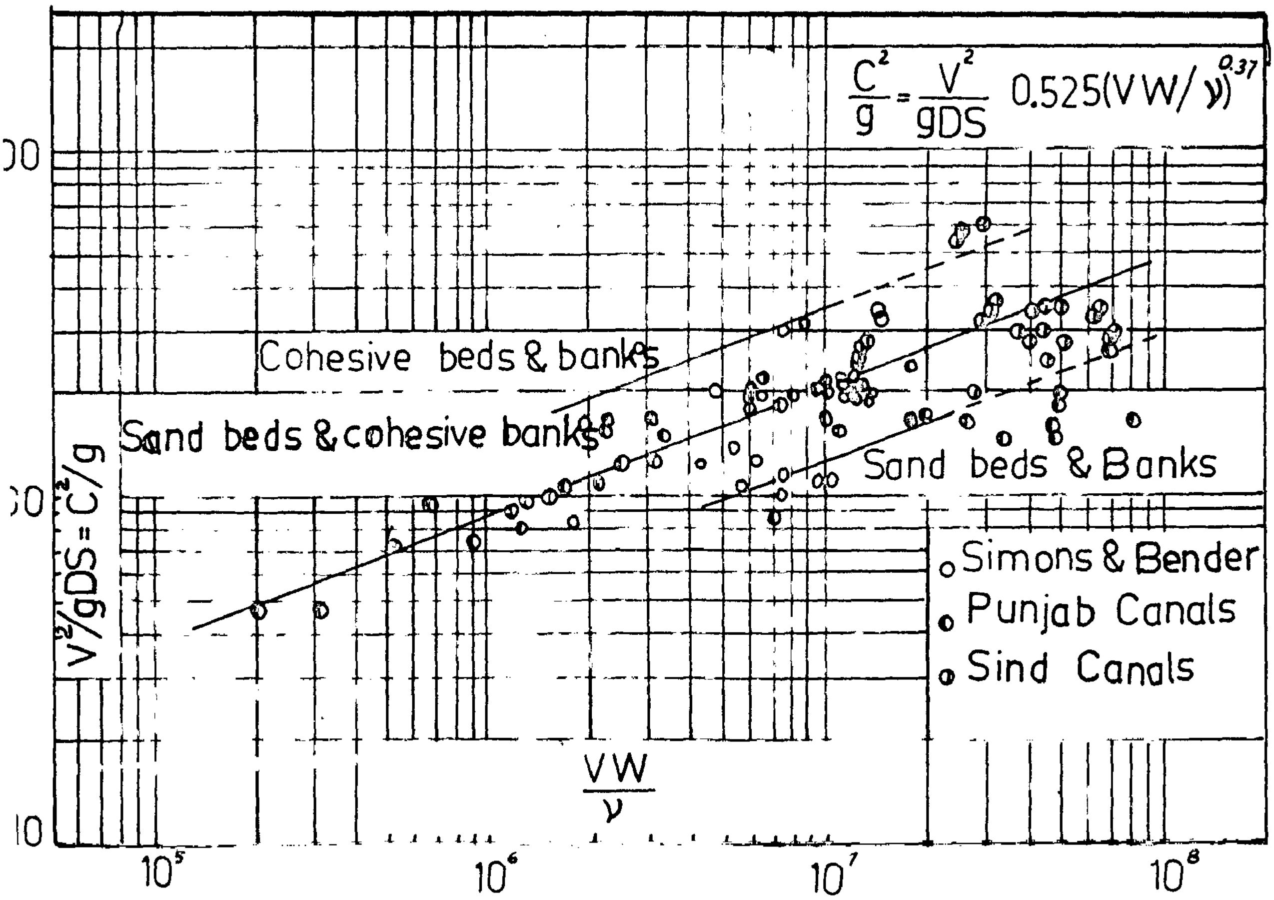


Fig. 2.8 Variation of V^2/gDS with VW/ν and type of channel

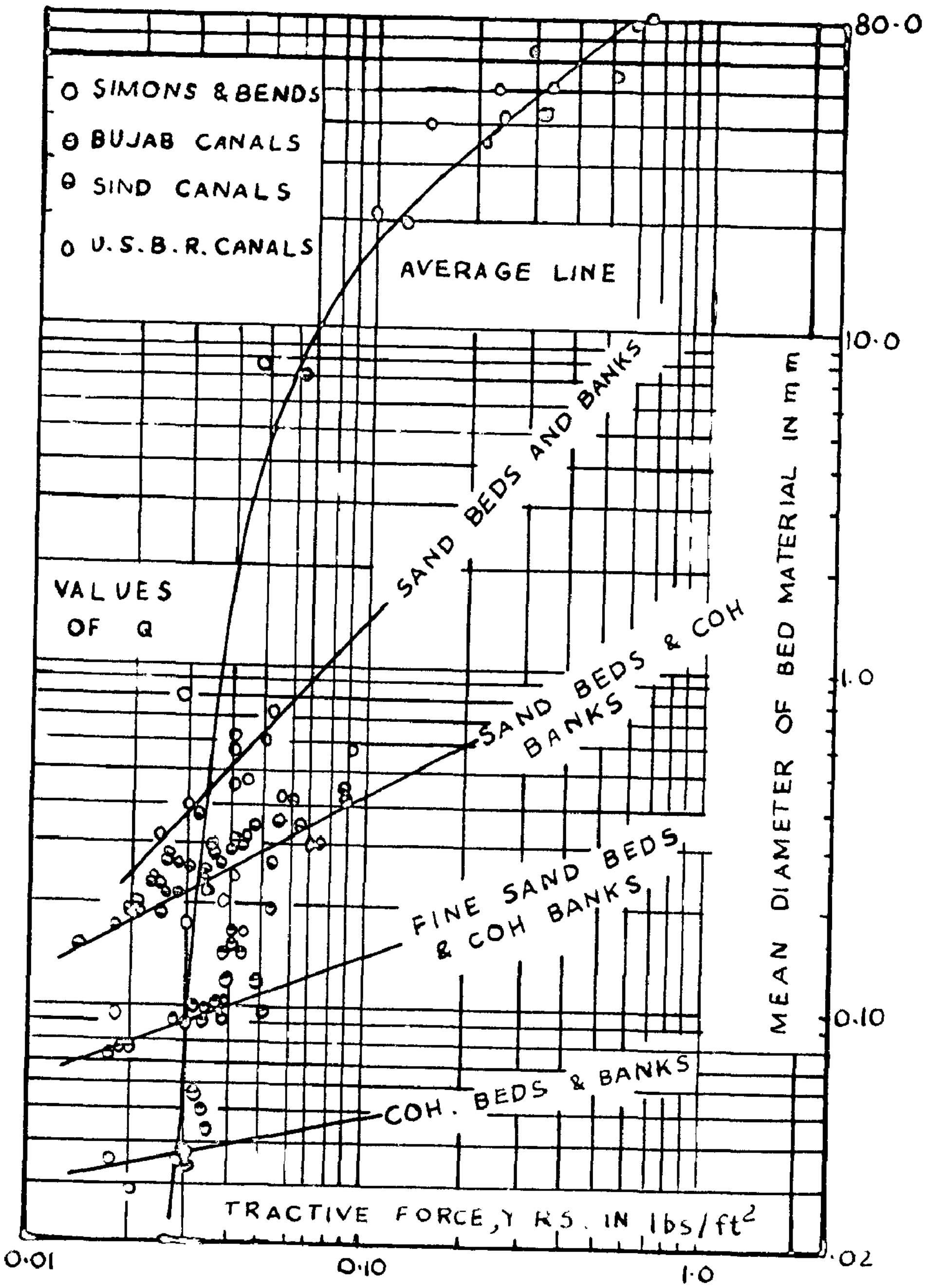


FIG. 2·9 – VARIATION OF TRACTIVE FORCE WITH BED MATERIAL d TYPE OF CHANNEL AND DISCHARGE Q degree of accuracy with the help of Figs. 2.1 to 2.9 and equations.

2.9.8 R.M.HAUXE and D.B. SIMONS [43]

They analysed massive data from laboratory flumes, canals and rivers, throughout the world and proposed a design procedure which does not involve the preselection of roughness factor which always brings out uncertainty in mobile boundary channels. Thus they gave a procedure for design of sandy bed alluvial channel, by means of certain family of curves by which the designer does not have to introduce an additional source of error by selection of a roughness factor with probable accuracy.

By analysis of river, canal and laboratory flume data they plotted a set of curves, which show the relation between V/V_*, the shear Renolds number V_*D/ν and $\Delta D/D$, ΔV and R, and discharge Q and R,

where,

V = Average velocity, ν = kinematic viscosity

V_* = Shear velocity, D = Average depth of flow and

ΔD = Depth correction.

This can be computed as follows:

$D = D' + \Delta D$, in which D = depth of flow from the field data and D' = depth such that $V'D' = q$. V' is the velocity for smooth boundary condition and q is the discharge per unit width.

If the discharge Q, the median fall diameter of bed material d_{50} and the water temperature is given, it is possible to design a sand bed alluvial canal, by the design procedure suggested by the author as below.

1. With Q known, select a tentative value of the hydraulic radius, R, from the plot of 'R' versus 'Q' considering the anticipated characteristics of the bank material Fig. 2.10.
2. Using this value of 'R', select a value of depth 'D' from Fig. 2.11.
3. Select an initial trial slope 'S', based on anticipated bed material discharge, the slope of the surrounding terrain, the slope of the existing canal which are operating successfully at the selected 'R' and other guiding factors.
4. Using the selected values of R and S, read the value of ΔV from the correlation of ΔV versus 'R' Fig. 2.13.
5. Compute shear velocity $V^{o} = \sqrt{gRS}$
6. Compute $\Delta V / V^{o}$.
7. Compute the shear Renolds number $Ro = \frac{V^{o}D}{v}$. Enter the plot of V/V^{o} versus $\log (V^{o}D/v)$ Fig. (2.12) and from the curve of smooth boundary, select the corresponding value of V'/V^{o}.
8. Compute $\frac{V}{V^{o}} = \frac{V'}{V^{o}} - \frac{\Delta V}{V^{o}}$, and then determine the average velocity V, to be expected in the channel.

9. Compute the stream power, $\tau V = \gamma VDS$.

10. Using the stream power, τV and the median fall diameter d_{50}, of the bed material, refer to the plot of τV versus d_{50} (Fig. 2.14) and determine whether the channel designed will be in regime in which dunes exist. If within this regime, proceed to next step, if not return to step 2 and select a new R and S or both and repeat the design procedure

11. Using the value of D, S and V obtained, compute the width of channel giving the necessary side slopes designed to suit the existing conditions.

2.9.9 U.S.B.R. Studies 44 (Torrel and Borland)

According to P.W.Torrell and W.M. Borland canals designed in alluvial soil by U.S.B.R. are class-ified into two categories:

I - Those which carry almost clear water, and

II - Those which transport appreciable sediment load.

The first category was analysed by Lane and others which will be discussed in detail in the next Chapter.

The second category of channels are designed by U.S.B.R, with help of Einstein's or some other sediment transport method. But it is noticed that the existing sediment transport functions are not very accurate . Thus U.S.B.R. [39] prefers another formula as below

$$\frac{W}{D} = \left[\frac{S^{1/2}/n}{225(NV_s)^{0.395}}\right] Q^{0.555} \qquad ...(2.105)$$

Where n = roughness coefficient, N = Sediment concentration in ppm by weight, V_0 = mean fall velocity ft/sec., Q = dominant discharge (Annual Average discharge), W = width in ft, D = Depth in ft., and S = slope.

When N = 0 , i.e., for clear water, W/D → ∞
This brings another drawback in the use of this formula. The selection of accurate 'n', is another limitation for this method. However, U.S.B.R. make use of different available design approaches and employ them in combination. Therefore the precisionof design method depends only on judgement, experience and skill of the designer.

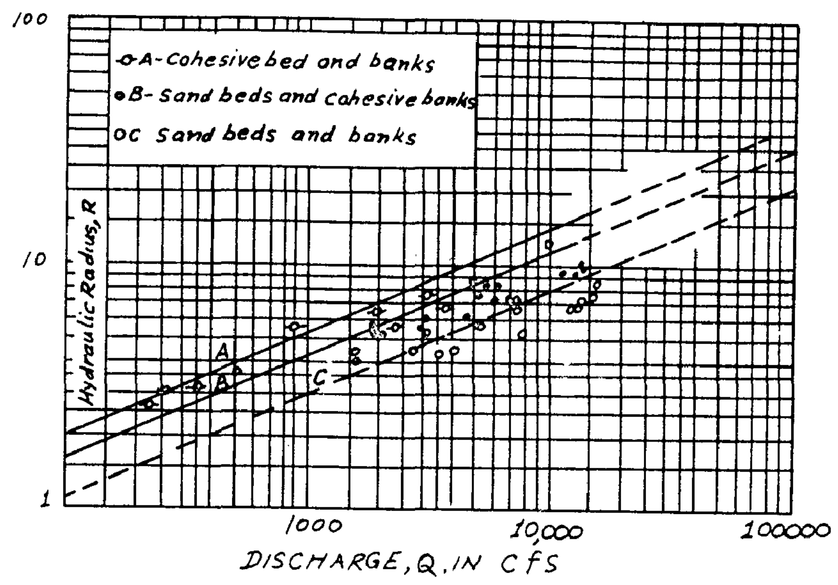


FIG. 2·10 VARIATION OF HYDRAULIC RADIUS R WITH DISCHARGE Q AND TYP OF CHANNEL AFTER SIMONS & ALBERTSON (1963)

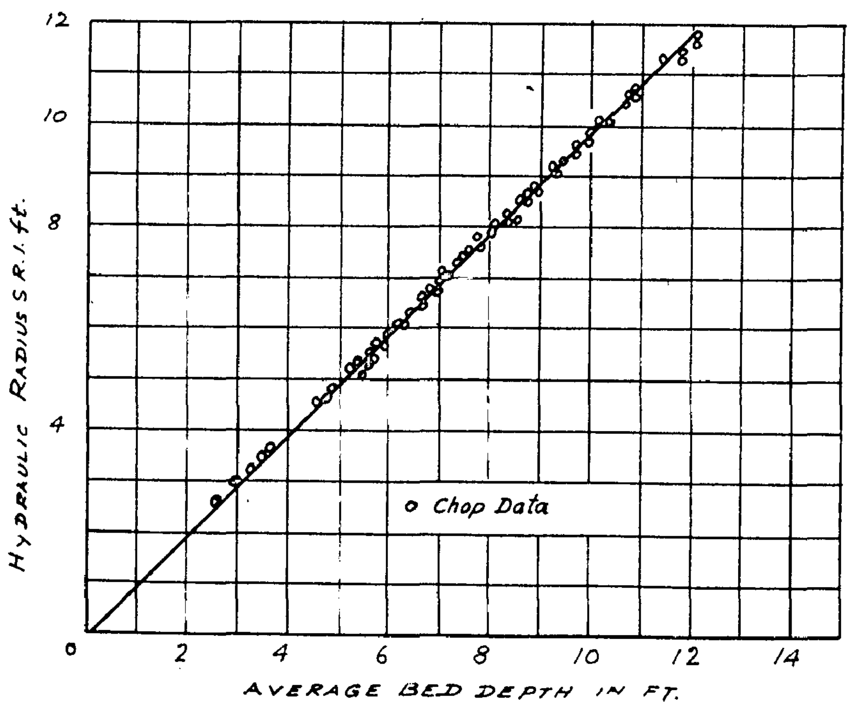


FIG. 2·11 VARIATION OF HYDRAULIC RADIUS R WITH DEPTH AFTER SIMONS AND ALBERTSON (1963)

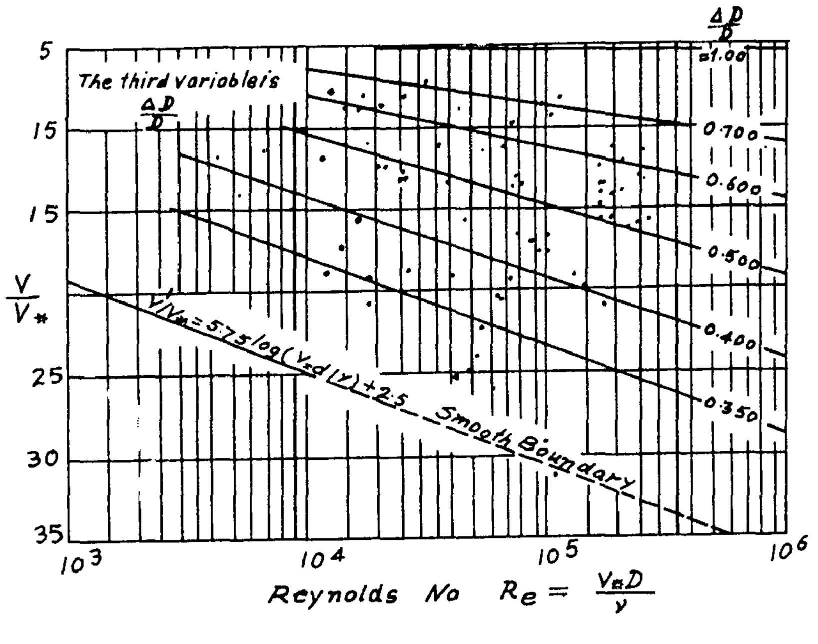


FIG. 2·12 FLOW RESISTANCE DIAGRAM FOR CHANNELS.

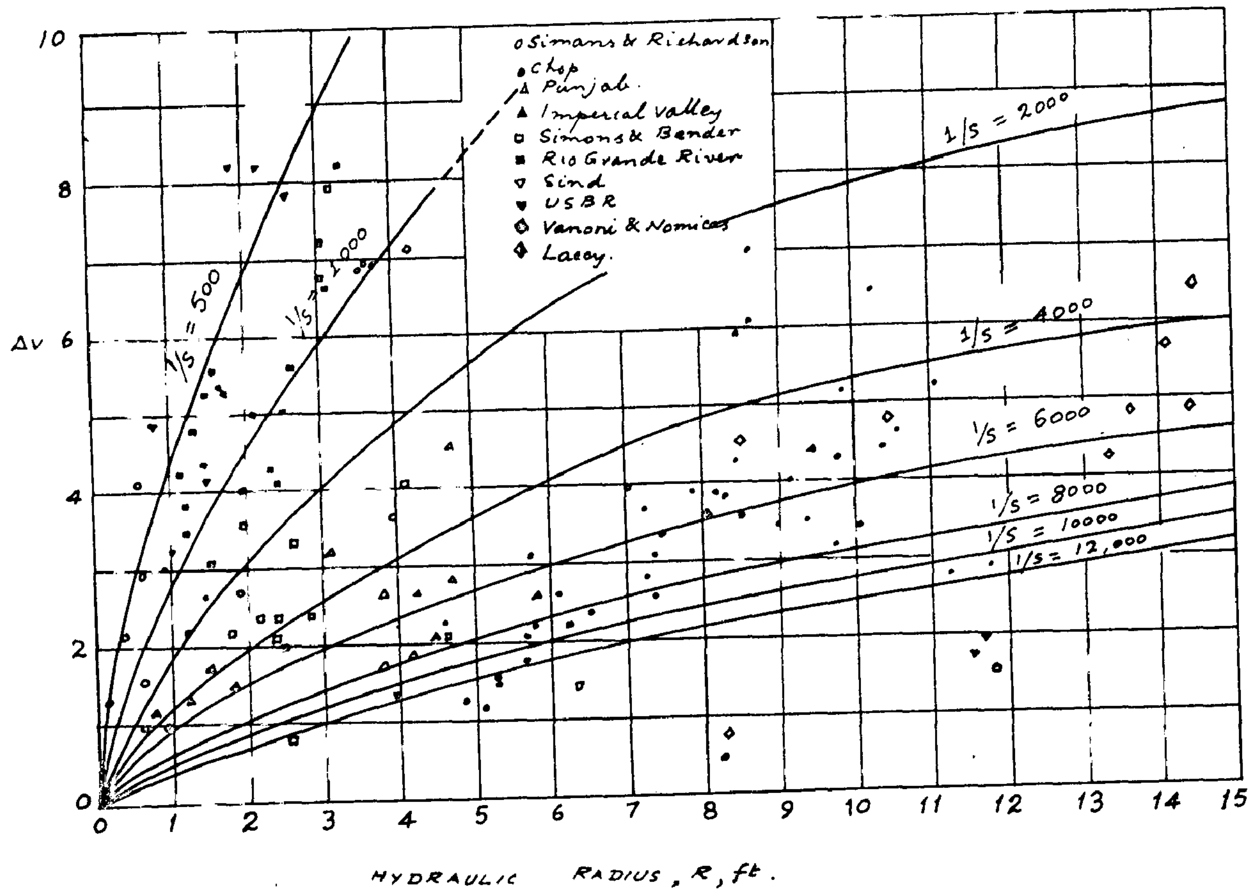


FIG. 2·13 HYDRAULIC RADIUS, R, FT.

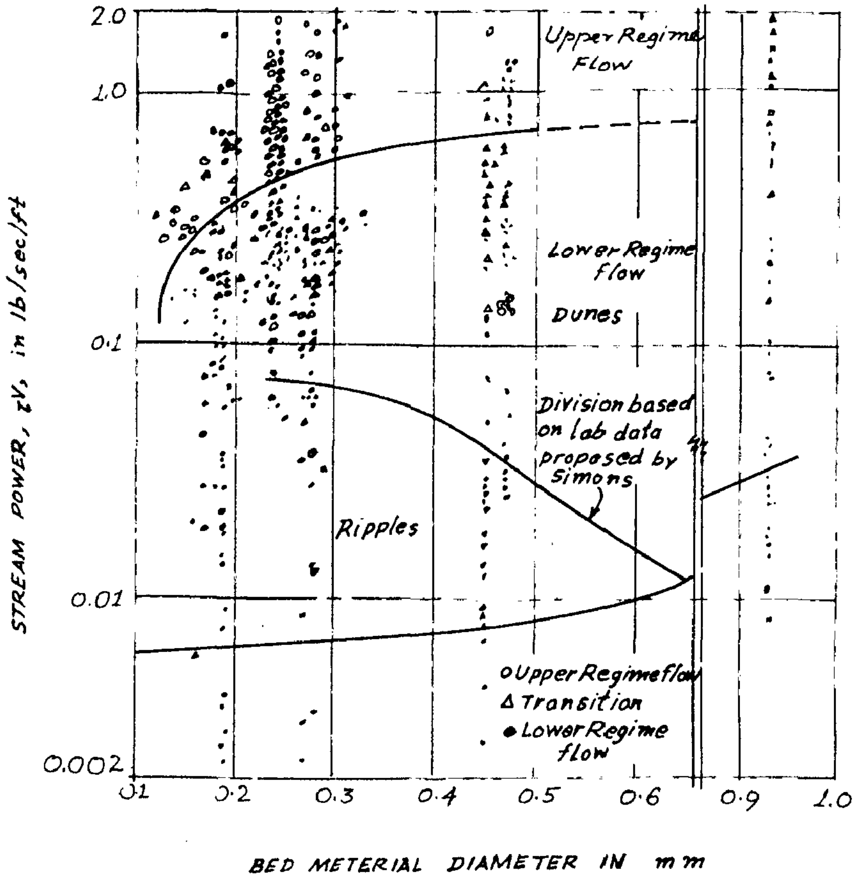


FIG. 2·14 REGIMES OF FLOW RELATED TO STREAM POWER AND BED MATERIAL DIAMETER AFTER SIMONS AND RICHARDSON (1966)

C H A P T E R - III

RATIONAL APPROACH

3.1 General

One of the outstanding theoretical approaches is the theory of tractive force. The theory developed by USBR is commonly referred to as the "tractive force" theory, although a better name would perhaps be the "threshold" theory, because it postulates a granular bed on threshold of motion. The use of the tractive force concept is not confined to the USBR theory, it is also implicit in Ning Chien's use of the Einstein bed load function.

The tractive force design is formulated on the basis of stability of bank and bed to resist erosion resulting from drag force exerted on the moving water.

This concept has been widely applied to the theory of sediment transport and fluid mechanics both in the United states and in other countries, but only to a limited extent in connection with design of earthen-canals. Use of this method for design has been suggested by William [49] and Armin Schoklitch [46]. The latter suggested that it was not possible to give fixed and definite values for maximum tractive force for different soils but the following values, could serve as a basis:

Soil	Permissible tractive force in lb/ft^2
Loam	0.062
Sand	0.102
Strong and Loamy soil	0.082
Course gravel	0.205
Very compacted soil	0.256

The foregoing tractive-force concept was developed further under the leadership of Lane [47], while with the United States Bureau of Reclamation (USBR). The USBR procedure is based on the hypothesis that the practical canal design is a tractive-force problem-beyond that, the approach is new and theoretical. The corresponding canal cross-section has been developed at the USBR by the use of Lane's method which goes by the name of 'Lane's Theory'. The design of channel is done by assuming that the channel is in an equilibrium stage i.e., a balance between tractive force of running water and the resistance of material forming the boundary of channel. The simplest case of design of a canal and which can be explicitly solved is one when $q_s = 0$, i.e., for clear water with sediment on threshold of motion at every point of the cross-section.

Lane [48] studied the fundamentals of channel design to find out a mathematical solution for the design of stable channels. He therefore systematically analysed the factors controlling the shape of channel in erodible material as below:

1. Hydraulic factors such as slopes, roughness, hydraulic mean depth, mean velocity, velocity distribution and temperature etc.
2. Channel shape, as bedwidth , depth and side slopes
3. Nature of material transported, depending upon the size, shape, specific gravity, dispersion, quantity and material of bank and bed.
4. Miscellaneous factors such as alignment, nonuniformity of flow and aging etc.

In derivation of rational approach the major effects must be taken into consideration but the minors may be neglected.

3.2 DEFINITION OF STABLE CHANNEL

Lane defined a stable channel as follows-

A stable channel is an unlined earthen channel(a) which carries water , (b) the banks and bed of which are not scoured objectionably by the moving water, and (c) in which objectionable deposit of sediment does not occur.

Three distinct classes of instability have been defined by Lane as follows:

1. Channels subjec ted to scour that donot silt,
2. Channels in which objec-tional deposition occurs but do not scour, and
3. Channels in which objec tional scour and silting both occur.

Case I - Class 1 occurs when the sediment free water is present in a channel or the sediment carried is very small in quantity or so fine that there is little chance of its getting deposited. This class of instability is the simplest of the three proposed and fortunately , it is also the one of primary importance, since most of the present and future canal problems are and will be clear-water problems.

Case II- The second class of instability (deposit without scour) can only be caused by the sediment brought into the canal with flowing water or that scoured from the banks and bed of an upstream channel. An example of this case is a lined canal- or a canal cut through a scour-resistant material into which the large quantity of coarse material enters with inflowing water.

Case III - The third class of instability (scour and deposit) usually occurs when water containing large quantities of coarse sediment enters a canal, the bank and bed of which are composed of material which has little resistance to scour.

3.3 PREVENTION OF INSTABILITY

For prevention of instability in the first class only analysis of scouring action is necessary.

For prevention of second class of instability, it is necessary to be sure that sediment brought into the canal at the upstream end is carried out at downstream end. For this problem the basic analysis of sediment transportation is required. The prevention of instability in the third class involves an analysis of the combination of scour and transportation problems.

3.4 THE MAIN FACTORS TO BE CONSIDERED IN DESIGN PROCEDURE

a. Distribution of tractive force over the channel periphery for different side slopes with special emphasis on the magnitude of shear exerted on the sides as compared to the bed.

b. Relative stability of soil particles on the bed and on sloping sides of the channel, and

c. Magnitude of safe tractive force for different mean size and gradations of non-cohesive materials.

3.5 FORCES CAUSING SCOUR

When water flows in a channel, a force is developed that acts in the direction of flow on the channel's periphery. This force, which is simply the pull of water on wetted area, is known as the tractive force or shear drag or drag force.

The first step in analysing the problem of scour in canals appears to be a consideration of the forces causing such a scour. Scour on the bed and banks of canals occurs when particles composing the sides and bottom are acted on by forces sufficient to cause them to move.

As mentioned [49] when a particle is resting on the level bottom of canal, the force, acting to cause motion is that force caused by the motion of water past the particle. If scour is to be prevented, this motion must not be rapid enough to produce forces on the particles that sufficiently large to cause it to move.

If the particle is on sloping side of a canal, it is acted on, not only by water, but also by force of gravity which tends to make it roll or slide down this slope. The force causing the downward motion is the component in the direction of the slope, of the force of gravity acting on particle. If the resultant of the force caused by the motion of the water and the component of the force of gravity acting on the particle is large enough, the particle will move. For the particle to be moved when cohesion is present, the forces ac-ting must be sufficient to overcome this cohesion also.

3.6 TRACTIVE FORCE DISTRIBUTION

The concept of tractive force is generally believed to have been introduced into hydraulics literature by du Boys in 1879 [50] . However, the principle of balancing this force with the channel resistance in a uniform flow was stated by Brahms early in 1754.

The shear or tractive force is equal to, and in the opposite direction from the force which the bed exerts on flowing water. If no force were exerted by the banks or bed on the water, the water would continue to accelerate as would a frictionless ball rolling down an inclined plane. In a uniform channel (having constant slope) in which the water is moving at steady uniform rate, the water is not accelerating because, the force tending to prevent motion is equal to force causing motion. The tractive force under these conditions is therefore equal to the force tending to cause the water to move. This force is equal to the effective component of gravity force acting on the body of water, parallel to the channel bottom and equal to $\gamma_w A\ LS$. Where S = sine of the angle which the bed makes with horizontal. γ_w is the unit weight of water, A is wetted area and L is the length of the channel reach. Thus the average value of the tractive force per unit wetted area, or so called unit tractive force τ_o, is equal to $\gamma_w ALS/PL = \gamma_w RS$, where P is equal to wetted perimeter and R is the hydraulic radius, that is,

$$\tau_o = \gamma_w RS \qquad \ldots(3.1)$$

In a wide open channel, the hydraulic radius is equal to the depth of the flow, y, hence,

$$\tau_o = \gamma_w y\ S \qquad \ldots(3.2)$$

It should be noted that the unit tractive force in channels except for wide open channels, is not uniformly distributed along the wetted perimeter. Many attempts have been made to determine the distribution of tractive force in a channel. Leighly [51] attempted to determine this distribution in many trapezoidal, rectangular and triangular channels from published data on the velocity distribution in the channels. Unfortunately, owing to deficiency of data, the results of his study were not very conclusive. In the USBR Olsen and Florey [52] and other engineers have used membrane analogy, method of finite difference for trapezoidal section, and mathematical for rectangular and triangular channels. A typical distribution of tractive force in trapezoidal channel, resulting from membrane-analogy study is shown in Fig.(3.1). The patter of distribution varies with shape of the section but is practically unaffected by the size of the section. It means all canals having the same ratio of bed width, b, to depth 'y' and the same side slopes , the tractive force distribution would be similar, that is, the tractive force at any point in one cross-section will be similar to that at any other point in corresponding position in similar section. Thus if the tractive force distribution in any channel can be determined, the distribution will be the same in any other channel of similar cross-section and roughness distribution The similarity of the distribution is not only true in trapezoidal channels, it can be applied also to other channel shapes. Based on such studies curves (Fig. 3.2) showing the maximum unit tractive forces on the sides and bottom of various channel sections have been prepared for use in canal design. Generally, speaking, for the trapezoidal channels of the shapes ordinarily used in canals, the maximum tractive force on the bottom is close to the value (γyS), and on the side close to $0.76\ \gamma yS$.

3.7 TRACTIVE FORCE RATIO

Consider a grain resting on a slope at angle θ to the horizontal. Fig. (3.3). There are two forces acting : the tractive force $a\tau_s$ and the gravity force-component $W_s \sin\theta$ which tends to cause the particle to roll down the side slope [53, 54, 49, 55] . Where a = effective area of the particle, τ_s = unit tractive force on the side of channel, W_s is submerged weight of the particle, and θ = angle of side slope. The resultant of these two forces, which are at right angles to each other, is

$$P = \sqrt{W_s^2 \sin^2\theta + a^2\tau_s^2}$$

When this force is large enough, the particle will move. By the principle of frictional motion in mechanics, it may be assumed that, when motion is impending, the resistance to the motion of the particle is equal to the force tending to cause the motion. The resistance to motion of the particle is normal force $W_s \cos\theta$ multiplied by coefficient of friction, or $\tan\phi$ where ϕ is the angle of repose. Then,

$$W_s \cos\theta \tan\phi = \sqrt{W_s^2 \sin^2\theta + a^2\tau_s^2} \qquad \ldots(3.3)$$

or

$$\tau_s = \frac{W_s}{a} \cos\theta \tan\phi \sqrt{1 - \frac{\tan^2\theta}{\tan^2\phi}} \qquad \ldots(3.4)$$

τ_s is unit tractive force causing impending motion on sloping surface.

From the motion of particle in level bed in impending stage by tractive force $a\tau_L$, we obtain from Eq.(3.3) when θ = 0,

$$W_s \tan\phi = a\,\tau_L$$

$$\text{or} \quad \tau_L = \frac{W_s}{a} \tan\phi \qquad \ldots(3.5)$$

τ_L = unit tractive force causing impedning motion on level bed surface.

The ratio of τ_s and τ_L is called the tractive force ratio. This factor is very important for design purpose.

$$K = \frac{v_B}{v_L} = \cos\theta \sqrt{1 - \frac{\tan^2\theta}{\tan^2\phi}} \qquad \ldots(3.6)$$

$$\text{or } K = \sqrt{1 - \frac{\sin^2\theta}{\sin^2\phi}} \qquad \ldots(3.7)$$

This ratio involves only the angle of side slope and the angle of repose of material(ϕ) . For convenience in finding the value of K Fig. 3.6 is prepared.

3.8 ANGLE OF REPOSE OF NONCOHESIVE MATERIAL

A study was made of this subject, beginning with thorough review of available pertinent literature. This review was followed by a limited number of laboratory investigation on a large number of stock piles of various size of material and at gravel washing-plant.

The results showed that, for large sizes the angle were not materially different for the various conditions of stocking in air or water, but for the sand sizes these condition had more effect. The average results of all observations of the angle of repose of stock piles at gravel washing plants are shown in Fig. 3.5. Although the data shows that the angle of repose increases with both size and angularity. The curves show values of angle of repose for noncohesive material above 0.2 in in diameter for various degree of roughness.

The diameter referred to is the diameter of a particles than which 25% (by weight) of material is larger.

For cohesive and fine noncohesive materials, the cohesive forces, even with comparatively clear water, become So great in proportion to the gravity force component causing the particle to roll down that the gravity force can safely be neglected. Therefore, the angle of repose need be considered only for coarse noncohesive materials.

3.9 PERMISSIBLE TRACTIVE FORCE

The permissible tractive force or limiting tractive force is the maximum unit tractive force that will not cause serious erosion of the material forming the channel on a level surface. This unit tractive force can be determined by laboratory experiments, and the value of thus obtained is known as critical tractive force. A very thorough study was carried out to determine the limiting tractive stress in canals.

The recommended values depends on the type of material in which the canal is flowing. However, experience has shown that the actual canal in coarse non-cohesive material can stand substantially higher values than critical tractive forces measured in laboratory. This is probably because the water and soil in actual canals contain slight amounts of colloidal and organic matter which provide a binding power and also because movement of soil particles can be tolerated in practical designs without endangering channel stability, since the permissible value may be taken less than the critical value.

The types of the material in which canals, are constructed are divided as follows-

1. Coarse noncohesive material,
2. Fine noncohesive material, and
3. Cohesive material.

In the first class of material the method outlined above is directly applicable. The USBR has made a comprehensive study of the problem, using data for coarse non-cohesive material obtained from experiments in fifteen reaches of the San Luis valley canals, having discharges ranging from 17 cu.ft per sec. to 1,500 cfs, and slope from 4.2 ft per mile to 51 ft per mile [48]. The results of measurements are summarized in Fig. 3.4 . The size used for comparison was that size of which 25% (by weight) is larger. This is because it was expected that the water with high tractive forces, in flowing through these canals, would remove all the material below a certain size. The limiting stress in lbs/sft on level bed is determined from the relationship 0.4 times d_{25} in inches, which is practically the critical stress.

For the design of canals in coarse non-cohesive material, one must consider not only the limiting tractive force on the bottom but also the action of the particles in rolling down the sloping sides of the canal. This involves the angle of repose and value of K. The tractive force distribution and hydraulic roughness is also very important.

Figure (3.4) shows the data obtained by USBR on the canals of the San Luis Valley. It was developed for the material having specific gravity of 2.56. If used for material with appreciably different specific gravities the tractive force for a given size must be multiplied by the ratio of the submerged unit weight of the other material to the submerged unit weight of the material having a specific gravity of 2.56. In the case of porous material the saturated unit weight should be used. Since most observation points fall either above or very close to line A, therefore, it is true limiting value. For sufficient factor of safety, tentatively line B was recommended for design purposes.

Canals in fine, non-cohesive material are intermediate between other two classes. In this class the effect of small amounts of cohesive sediment in the water or in the material through which canals flow becomes important.

Based on a consideration of all available data the best recommendations which can be made for canals constructed in fine non-cohesive material are those given in Table 3.1. The comparison of these recommendations with most of the data available as of 1952 is shown in Fig.3.7 . By high content of fine sediment is meant a load of 2% or more of silt and clay size on an average of two or three times a year, with a low constant of sand. Unfortunately, sufficient information is not available (as for 1952) to set limit for the allowable sand content. Where much sand is carried, this method of analysis is not applicable. A low content of fine sediment is defined as a content of silt and a clay size reaching 0.2% concentration of the average of two or three times a year. The sand content should be very low.

The median size, or size of which 50% (by weight) is larger specified for the fine non-cohesive material is chosen. But for coarse non-cohesive the size of 25% larger size is chosen. In the vicinity of 5 mm size, which was somewhat arbitrarily selected as division between those two classes, whichever classification give the lower tractive force is chosen.

Where the canal is constructed in cohesive material, the particles are prevented from rolling down by cohesion. Hence, the part of analysis is not applicable. The design, therefore, involves only the distribution of the tractive force for material in which the canal is constructed. In these canals the hydraulic roughness is not a function of the size of the particle but of the surface irregularities on the banks and usually of the ripple formation on the bed. For the design of canal in this cases the only data safe on tractive force available (as of 1952) are those obtained by converting limiting velocity. They are given in Tables 3.2, 3.3 and 3.4

3.10 LIMITING TRACTIVE FORCE FROM LIMITING VELOCITY

The data for critical tractive forces obtained from laboratory are applicable to the case of coarse non-cohesive material. But the results of these experiments show that the limiting tractive force are more than that shown in laboratory.

No laboratory data are available for limiting tractive force in cohesive material. For both of these cases the only method of determining limiting-tractive force is the limiting velocity which these canals sustain . Since the velocity is not a completely rational parameter in determination of scouring, therefore, these velocity data are not entirely satisfactory. There are three sources of limiting-velocity data:

1. B.A. Etchcverry data [56] , Table 3.2
2. Messrs. Fortier and Scobey [57] , Table 3.4
3. USSR Data [58] , Tables 3.3, 3.5

The data in Table 3.2 is not related to the size of canal. The data obtained by Messrs Fortier and Scobey (Table.3.4) are for a 'depth 3 ft or less'. They suggest that, for depth greater than 3 ft a mean velocity greater by 0.5 fps may be allowed. They also state that the values are applicable to canals with long tangents predominating through their length and that for canals in sinuous of alignment a reduction of about 25% is recommended. The values are for canals which have been 'aged'.

Table 3.3 shows Russian data giving the permissible value for granular material of various diameters, for a mean depth of 1m, and for water carrying less than 0.1% of sediment of less than 0.005mm size. For other mean depths, the values in Table 3.3 can be multiplied by the factor shown in Table 3.3(a). It is also stated [58] that the permissible velocity in Table 3.3 can be increased by the following percentages for flow containing from 0.1% to 2.5% of sediment less than 0.005mm in diameter, from 25% to 65% for sand, from 10% to 45% for gravel, and from 0% to 25% for pebbles. For cohesive material the values are given for 1m mean depth in Table 3.5 and correction in Table 3.5a.

To convert the values of limiting velocity in the literature into exactly equivalent values of tractive force, it is necessary to know the size, shape shear distribution and energy gradient of the channels to which these values apply. Because these data are not given, it is necessary to make certain assumption regarding them. A canal with a 3 ft depth, bottom width of 10 ft and side slope of $1\frac{1}{2}$ to 1 had been used and with the help of Manning formula the energy slope was determined by Mr. Lane.

3.11 CANAL CARRYING HEAVY SEDIMENT LOAD

The principles for designinf of canals with heavy sediment loads are as follows: If appreciable qunatities of sediment are introduced in the canal with the water, the stability of canal will depend on the fact that this sediment is carried through the canal without deposit.

That is the canal must be able to transport the sediment which is introduced into it. Usually the greatest difficulty is experienced in transporting the sand coarse sizes of material.

Because most of this material travels near the bed, a stable channel must have sufficient shear acting on the bed to transport this load. At the same time, however, the shear on the sides must not be great enough to scour the sides. Also the shear on the bottom must not be so much greater than that required to transport the sediment that scour of the original material of the bed results.

If large quantities of fine sediment are carried, the shear through the section must be sufficient to keep this material in suspension from deposition . The shear, however, must not be of such a magnitude that scouring of the original material will occur. If both fine and coarse materials are carried by channel in appreciable quantity, there must be enough shear on the bed to cause the transportation of the coarse material, and enough shear on the banks to prevent the deposition of the fine material, but not enough to cause the scour of material forming the banks.

Thus the laws of sediment transportation, are very important in stable channel problem, but these are not complete and still remain to be accomplished.

3.12 EFFECT OF BENDS

Canals of USBR have been designed to limit the radius of bends to six times the water-surface width or fifteen times the water depth.

A number of other suggestions have been made [56] The scour on bends can be reduced by lowering velocity of flow . This can be done by increas ing canal area and hence larger cost. It will be more economical to allow the scour to begin and then stop it by protecting the banks at the points where the scour occurs, rather than to use larger cross-section necessary to insure that no scour will take place.

The values of limiting tractive force are given for straight canals. The corrections for bends are given in Table 3.6. In order to define the meaning of these various degrees of sinuosity, the following may be useful-

1. Straight canals have straight or slightly curved alignments , as is typical of canals in flat plains.
2. Slightly sinuous canals have a degree of curvature which is typical of slightly undulating topography.
3. Moderately sinuous canals have a degree of sinuosity which is typical of moderately rolling topography.
4. Very sinuous canals have a condition of curvature which is typical of canals in foot hills or mountainous topography.

The values are based on judgement rather than observed data.

3.13 NON SCOURING CANALS OF MINIMUM EXCAVATION AND WIDTH

In designing stable channel, the trapezoidal section was found the tractive force made equal to the permissible value over only a part of the perimeter of the section. Forces of less magnitude than these acted on most of the perimeter. In developing a stable hydraulic section for maximum efficiency, it is necessary to satisfy the condition that impending motion shall prevail everywhere on the channel perimeter. For material of with a given angle of repose and for a given discharge, this optimum section will provide not only the channel of minimum water area, but also the channel of minimum top width and maximum mean velocity, and minimum excavation . The shape of the channel is dictated by the following assumptions-

1. At and above the water surface, the side slope is at the angle of repose of the material.
2. At points between the centre and edge of the channel, the particles are in state of incipient motion, under the action of the resultant of the gravity component of particles, submerged weight acting down theside slope and tractive force of the flowing water.
3. At the centre of the channel the side slope is zero, and the tractive force alone is sufficient to cause incipient motion.
4. The particle is held against the bed by the component of the submerged weight of the particle, acting normal to the bed.
5. The tractive force on any area is equal to the component of the weight of the water above the area in the direction of flow. Under the first three assumptions the particles on the entire perimeter of the canal cross-section are in a state of impending motion. If assumption 5 is to hold there can be no lateral transfer of

tractive force between adjacent currents moving at a different velocities in the section - a situation, however, that never actually occurs . But studies showed by mathematical analysis in Bureau that the actual transfer of tractive force has little effect which can safely be ignored. The detailed discussion of this is found in [59]

The critical shear stress, τ, on the side slope of angle θ to the horizontal, is related to the critical shear stress τ_c on a bed width no side slopes by an equation derived as already,

$$\frac{\tau}{\tau_c} = \cos\theta \sqrt{1 - \frac{\tan^2\theta}{\tan^2\phi}} \qquad \ldots(3.6)$$

If it is assumed that the local value of shear stress τ is directly proportional to local depth y Fig. 3.8. Equation (3.6) yields a y-θ relation that can be integrated to give the following x-y equation

$$\frac{y}{y_o} = \cos \frac{X \tan\phi}{y_o} \qquad \ldots(3.7)$$

so that the required profile is a simple cosine curve. A further integration shows that the area, A, of the cross section is equal to

$$A \doteq \frac{2y_o^2}{\tan\phi} \qquad \ldots(3.8)$$

A slight extension of the theory gives an expression for wetted perimeter P. It is equal to -

$$2\int_0^{X_o} \sqrt{1 + (dy/dx)^2}\, dx = 2\int_0^{x_o} \sqrt{1+\tan^2\phi\sin^2 \frac{X\tan\phi}{y_o}}\, dx$$

...(3.9)

The substitution $\alpha = \frac{x \tan \phi}{y_o}$, and some further manipulation leads to the integral

$$\frac{2y_o}{\sin\phi} \int_0^{\pi/2} \sqrt{1 - \sin^2\phi \cos^2\alpha}\, d\alpha \qquad ...(3.10)$$

Because the limits of integration are $\pi/2$ and 0, $\cos^2\alpha$ may be replaced by $\sin^2\alpha$ without changing the value of integral,

$$P = \frac{2y_o}{\sin\phi} \int_0^{\pi/2} \sqrt{1 - \sin^2\phi \sin^2\alpha}\, d\alpha$$

The integral is the standard complete eliptic integral of second kind, usually denoted by E. For any given value ϕ its value is readily obtained from tables. Now

$$P = \frac{2\, y_o E}{\sin\phi} \qquad ...(3.11)$$

Hence the hydraulic radius, R equal to

$$R = \frac{A}{P} = \frac{2y_o^2}{\tan\theta} \cdot \frac{\sin\phi}{2\, y_o\, E} = \frac{y_o \cos\phi}{E} \qquad ...(3.12)$$

The key values for $\theta = 35^o$ may be computed,

$$\text{Surface width } W_o = 2X_o = \frac{\pi Y_o}{\tan\phi} = 4.49\, y_o \qquad ...(3.13)$$

$$\text{Area, } A = \frac{2y_o^2}{\tan\phi} = \frac{2y_o^2}{0.698} = 2.86\, y_o^2 \qquad ...(3.14)$$

$$\text{Perimeter, } P = \frac{2y_o E}{\sin\phi} = \frac{2y_o \times 1.432}{0.574} = 4.99\, y_o \quad \ldots(3.15)$$

$$\text{Hydraulic Radius } R = \frac{y_o \cos\phi}{E} = \frac{y_o\, 0.818}{1.432} = 0.572\, y_o \quad \ldots(3.16)$$

The y-θ relationship which leads to Eq. (3.7) does not uniquely define a complete channel section. The y-θ equation would be equally satisfied by insertion of a section of constant depth between the two curved banks (Type A) or by removing a section from the centre (Type C) in Fig. 3.9. Type B of the above curve consists of the complete curved bank with $\theta = 0$ at $y_o = y$, but without a central insert.

TABLE 3.1

TENTATIVELY RECOMMENDED LIMITING VALUES OF TRACTIVE FORCE FOR CANALS IN FINE NONCOHESIVE MATERIAL

Median Size of material in mm	Limiting Tractive Force lb/sq.ft		
	Clear water	Light load of fine sediment	Heavy load of fine sediment
0.1	0.025	0.050	0.075
0.2	0.026	0.052	0.078
0.5	0.030	0.055	0.083
1.0	0.040	0.060	0.090
2.0	0.060	0.080	0.110
5.0	0.140	0.165	0.185

TABLE 3.2

COMPARISON OF ETCHEVERRY'S MAXIMUM ALLOWABLE VELOCITIES WITH TRACTIVE FORCE VALUES

Material	Values of Manning(n)	Velocity fps	Tractive force lb /sq.ft
Very light pure sand of quick sand character	0.020	0.75-1.00	0.006 -0.011
Very light loose sand	0.020	1.00-1.50	0.011-0.025
Coarse sand or light sandy soil	0.020	1.50-2.00	0.025-0.045
Average sand soil	0.020	2.00-2.50	0.045-0.070
Sandy loam	0.020	2.50-2.75	0.070-0.084
Average loam, alluvial soil, volcanic ash soil	0.020	2.75-3.00	0.084-0.100
Firm loam, clay loam	0.020	3.00-3.75	0.100-0.157
Stiffclay soil, ordinary gravel soil	0.025	4.00-5.00	0.278-0.434
Coarse Gravel, Cobble and shingles	0.030	5.00-6.00	0.627-0.903
Conglomerate, cement gravel soft state, tough hardon, soft sedimentary rock	0.025	6.00-8.00	0.627-1.114

TABLE 3.3

USSR LIMITING VELOCITIES AND TRACTIVE FORCES IN COHESIVE MATERIAL (SAND CONTENT LESS THAN 50%)

Compactness of bed								
Descriptive term	Loose		Fairly compact		compact		Very compact	
Void Ratio	2.0-1.2		1.2-0.6		0.6-0.3		0.3-0.2	
Principal cohesive Material of bed	Limiting mean velocity ft/sec.and Limiting tractive force lb/sec.							
	ft/sec	lb/sft	ft/sec	lb/sft	ft/sec	lb/sft	lb/sec	lb/sft
Sand clay(sand content < 50%)	1.48	0.040	2.95	.157	4.26	.327	5.90	0.630
Heavy clagey soils	1.31	0.031	2.79	0.141	4.10	.305	5.58	0.563
Clay	1.15	0.024	2.62	.124	3.94	.28	5.41	.530
Lean clayey soil	1.05	0.020	2.30	.096	3.44	.214	4.43	.354

TABLE 3.3(a)

USSR CORRECTIONS OF PERMISSIBLE VELOCITY FOR DEPTH OF COHESIVE MATERIALS

	AVERAGE DEPTH							
meters	0.3	0.5	0.75	1.00	1.50	-2.0	2.5	-3.0
Foot	0.98	1.64	2.46	3.28	4.91	-6.25	8.20	-9.84
Correction Factor	0.8	0.9	0.95	1.0	-1.1	-1.1	1.2	-1.2

TABLE 3.4

COMPARISON OF FORTIER AND SCOBEY'S LIMITING VELOCITIES WITH TRACTIVE FORCE VALUES - STRAIGHT CHANNELS AFTER AGING

Material	n	For clear Water		Water transporting colloidal silts	
		Velocity ft/sec	Tractive force lb/sft	Velocity ft/sec	Tractive force lb/sft
Fine sand colloidal	0.02	1.50	0.027	2.5	0.075
Sandy loam no colloidal	0.02	1.75	0.037	2.50	0.075
Silt loam noncolloidal	0.020	2.00	0.048	3.00	0.11
Alluvial silts noncolloidal	0.020	2.00	0.048	3.50	0.15
Ordinary firm loam	0.020	2.50	0.075	3.50	0.15
Volcanic ash	0.020	2.50	0.075	3.50	0.15
Stiff clay very colloidal	0.025	3.75	0.26	5.00	0.46
Alluvial silts colloidal	0.025	3.75	0.26	5.00	0.46
Shales and hardpans	0.025	6.00	0.67	6.00	0.67
Fine gravel	0.020	2.50	0.075	5.00	0.32
Graded loam to cobbles when non colloidal	0.030	3.75	0.38	5.00	0.66
Graded silts to cobbles when colloidal	0.030	4.00	0.43	5.50	0.80
Coarse gravel noncolloidal	0.025	4.00	0.3	6.00	0.67
Cobbles and shingles	0.035	5.00	0.91	5.50	1.10

TABLE 3.5

USSR DATA ON PERMISSIBLE VELOCITIES FOR NON COHESIVE SOILS

(D=1m . Sediment less than 0.1% of size< 0.005mm)

Material	Particle diameter mm	Mean velocity ft/sec.
Silt	0.005	0.49
Fine sand	0.05	0.66
Median sand	0.25	0.98
Coarse sand	1.00	1.80
Fine gravel	2.50	2.13
Median gravel	5.00	2.62
Coarse gravel	10.00	3.28
Fine Pebbles	15.00	3.94
Medium Pebbles	25.00	4.59
Coarse Pebbles	40.0	5.91
Large Pebbles	75.00	7.87
Large Pebbles	100.0	8.86
Large pebbles	150.0	10.83
Large Pebbles	200.0	12.00

TABLE 3.5(a)

USSR CORRECTIONS OF PERMISSIBLE VELOCITY FOR DEPTH NON COHESIVE MATERIAL

	Average depth						
Meters	0.3	0.60	1.00	1.50	2.00	2.50	3.00
Foot	0.98	1.97	3.28	4.92	6.25	8.20	9.84
Correction factor	0.8	0.9	1.00	1.10	1.15	1.20	1.25

TABLE 3.6

COMPARISON OF PERMISSIBLE TRACTIVE FORCES IN SINUOUS CANALS WITH PERMISSIBLE VALUES IN STRAIGHT CANALS

Degree of Sinuosity	Relative limiting tractive force	Corresponding relative velocity
Straight canal	1.00	1.00
Slightly sinuous canals	0.90	0.95
Moderately sinuous canals	0.75	0.87
Very sinuous canals	0.60	0.78

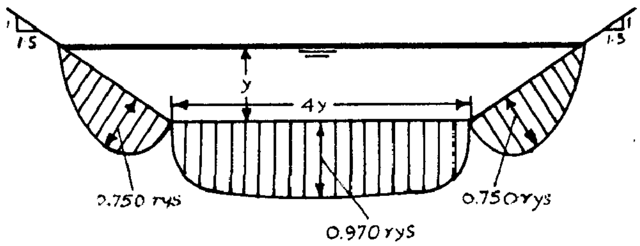
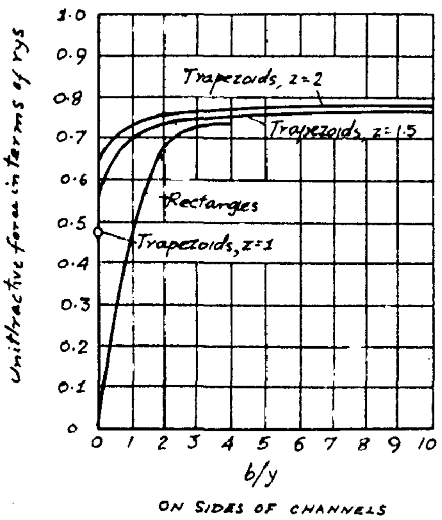


FIG. 3·1 DISTRIBUTION OF TRACTIVE FORCE IN A TRAPEZOIDAL CHANNEL SECTION



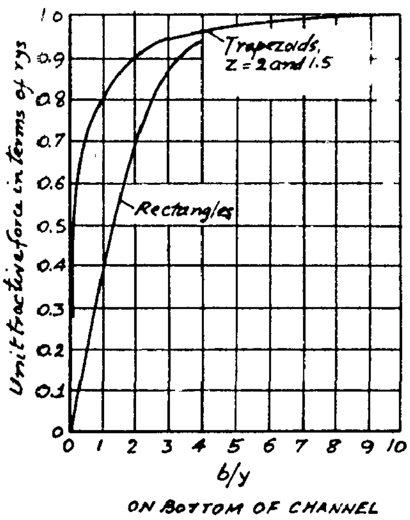


FIG. 3·2 MAXIMUM UNIT TRACTIVE FORCES IN TERMS OF γYS.

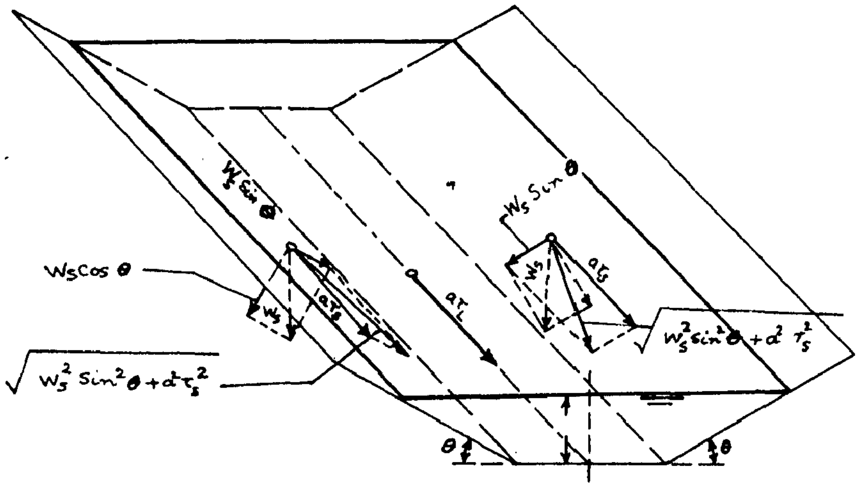


FIG. 3·3 ANALYSIS OF FORCES ACTING ON A PARTICLE RESTING ON THE SURFACE OF A CHANNEL BED.

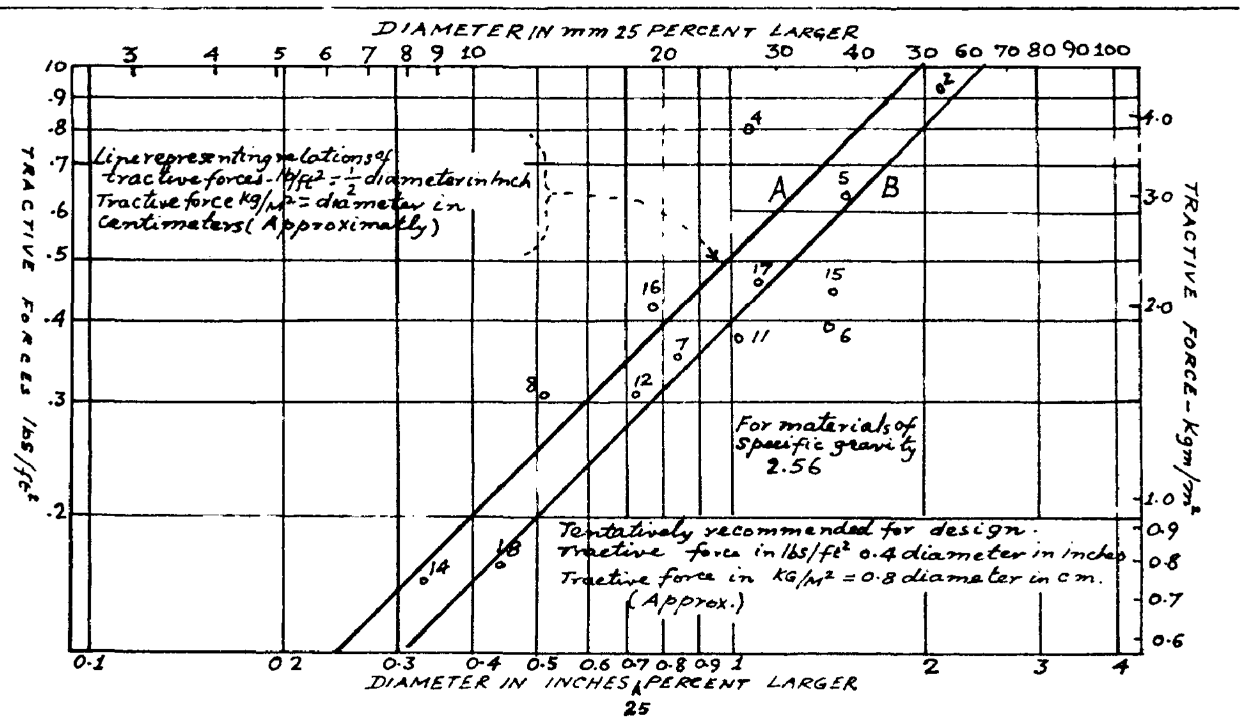


FIG. 3·4 RESULTS OF STUDIES ON SAN LUIS VALLEY CANALS

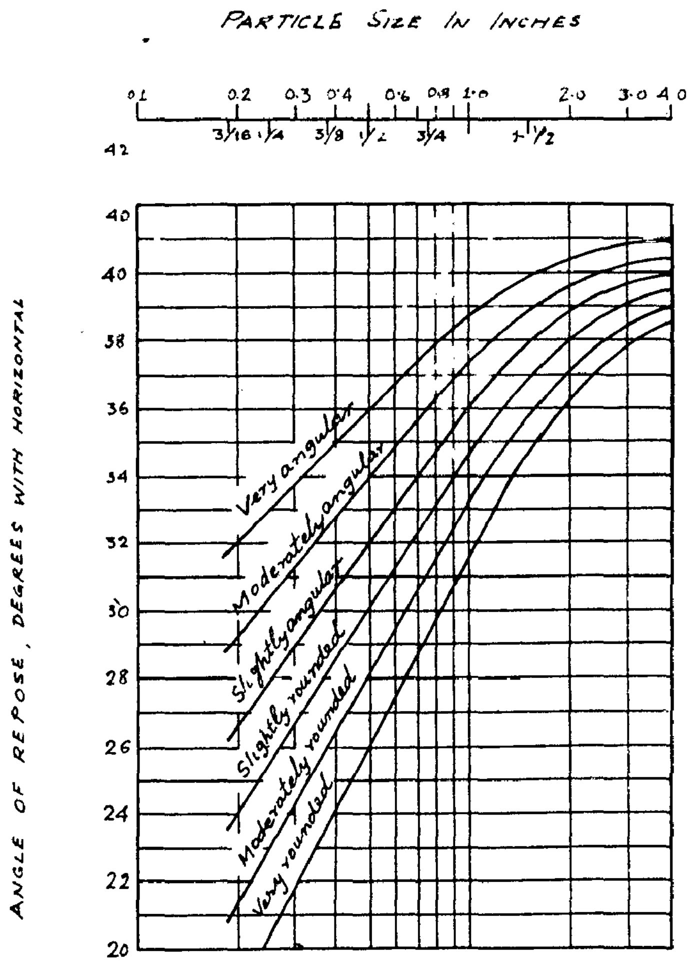


FIG. 3·5 ANGLES OF REPOSE OF NONCOHESIVE MATERIAL

(U S. BUREAU OF RECLAMATION.)

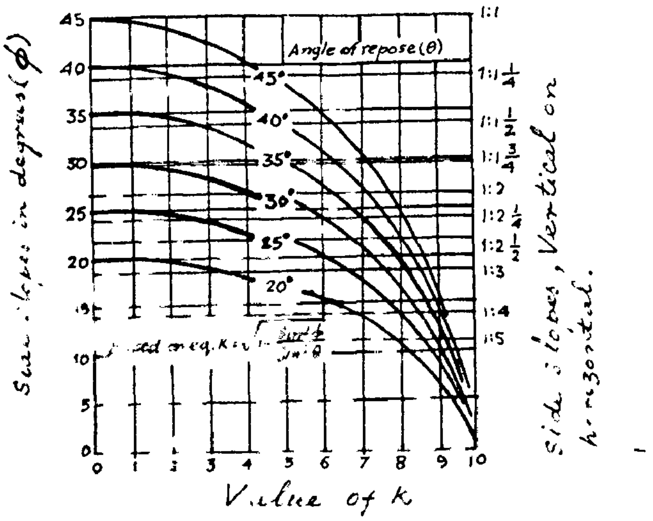


FIG. 3·5 CRITICAL SHEAR STRESS ON INCLINED SLOPES.

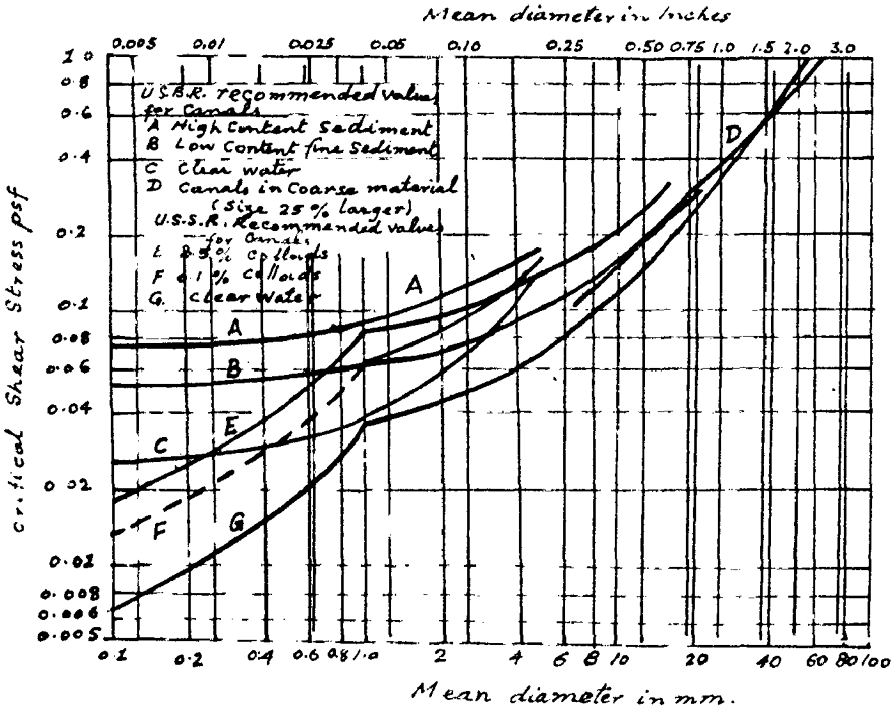


FIG. 3·7 RECOMMENDED VALUES OF LIMITING SHEAR STRESS

(FROM AMARICAN SOCIETY OF CIVIL ENGINEERS)

3.14 FRANCIS M. HENDERSON'S THEORY [60]

M. Henderson by making slight extention of W.L. Lanes theory of stable channel design, and by combining it with Strichler's formula deduced these formulae which are similar to G. Lacey's "regime" equations.

He also proved that Lacey's width-discharge relation of $P \propto Q^{1/2}$ is true for narrow channel developed by Lane's theory. He also pointed out that the similarity is not useful because the regime condition implies that the bed is live, whereas the tractive force criterion assumes that the bed is only on the thresh-hold of motion . For the latter criterion, the shear and hence RS , is required to be constant at all points along the channel. But for the former it is required $R^{1/2}$ S is constant along the channel. This criterion has been proved by him by using Einstein bed load function. He mentioned also that only two equations for stable channel can be obtained from consideration of bed conditions and flow resistance, and that third equation, such as Lacey's $P \propto \sqrt{Q}$ can be true only if there is a certain slope-discharge relationship, that is, a longitudinal profile.

The author wanted to introduce the size of bed material d. Therefore, he used shield's entrainment function.

The experimental work by Shield and others has shown that for values of the particle Reynolds number ($R_e^o = \sqrt{\frac{\tau}{\rho}}\frac{d}{\nu}$) above approximately 400, the following relation holds-

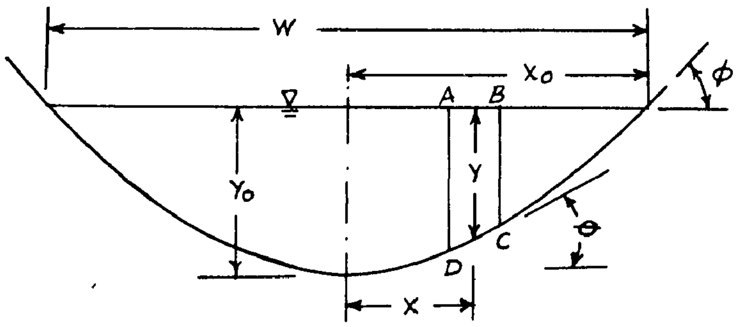


FIG. 3·8 DEFINITION SKETCH FOR STABLE CHANNEL ANALYSIS AFTER E.W. LANE

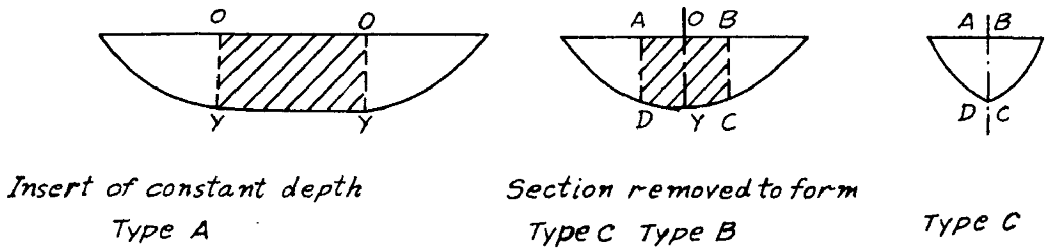


FIG. 3·9 ALTERNATIVE SOLUTIONS IN LANE'S STABLE CHANNEL THEORY

3.14 FRANCIS M. HENDERSON'S THEORY [60]

M. Henderson by making slight extention of W.L. Lanes theory of stable channel design, and by combining it with Strickler's formula deduced these formulae which are similar to G. Lacey's "regime" equations.

He also proved that Lacey's width-discharge relation of $P \propto Q^{1/2}$ is true for narrow channel developed by Lane's theory. He also pointed out that the similarity is not useful because the regime condition implies that the bed is live, whereas the tractive force criterion assumes that the bed is only on the thresh-hold of motion . For the latter criterion, the shear and hence RS , is required to be constant at all points along the channel. But for the former it is required $R^{1/2}$ S is constant along the channel. This criterion has been proved by him by using Einstein bed load function. Hementioned also that only two equations for stable channel can be obtained from consideration of bed conditions and flow resistance, and that third equation, such as Lacey's $P \propto \sqrt{Q}$ can be true only if there is a certain slope-discharge relationship, that is, a longitudinal profile.

The author wanted to introduce the size of bed material d. Therefore, he used shield's entrainment function.

The experimental work by Shield and others has shown that for values of the particle Reynolds number ($Re^* = \sqrt{\frac{\tau d}{\rho \nu}}$) above approximately 400, the following relation holds-

$$\frac{\tau_o}{\gamma_w (S_s - 1) d} = 0.056 \text{ (Approx.) (in fps units)} \qquad ...(3.17)$$

S_s = the ratio of solid to fluid density, d is the sediment size in inches.

The expression on the left hand side is known as the entrainment function. For a section wide enough for τ to be constant over entire bed, τ/γ_w=RS, in which the value of S_s =2.6.Eq.(3.17) shows that the condition

$$d = 11RS \quad \text{(wide channel)} \qquad ...(3.18)$$

holds at threshold movement.

For narrow channels the maximum shear stress is $\gamma_w Y_o S$ in which Y_o is the maximum depth. Then

$$d = 11\, Y_o S \qquad ...(3.19)$$

In particular, for limiting type B Fig. 3.9 cross section developed by Lane's theory, R = 0.572 Y_o (From Eq. 3.16)

$$d = \frac{11RS}{0.572} = 19\ RS \qquad ...(3.20)$$

Combining with Strickler's formula relating the bed material size d to the Manning n , $n = 0.034\ d^{1/6}$ ...(3.21)

from which

$$V = \frac{1.49\ R^{2/3}\ S^{1/2}}{0.034\ d^{1/6}} \qquad ...(3.22)$$

As per Eq. (3.18), d = 11RS , for wide channels

$$V = 29\ R^{1/2}\ S^{1/3} \qquad ...(3.23)$$

And for type B channel derived in Lane's theory

$$d = 19RS \text{ (as per Eq. (3.20))}$$

$$V = 27\ R^{1/2}\ S^{1/3} \qquad ...(3.24)$$

Evidently equations 3.23 and 3.24 are both very similar in form to the Lacey's equation

$$V = 16\ R^{2/3}\ S^{1/3} \qquad ...(2.21)$$

If we want to describe type B channel cross-section completely. Then by Eq. (3.24 with 10% increase in constant)

$$Q = VRP \approx 30\, R^{3/2}\, P\, S^{1/3} \quad ..(i)$$
$$R = d/19S \text{ (by Eq.3.20)} \quad ..(ii)$$
$$R = 0.572Y_o \text{ and } P = 4.99\, y_o \text{ or } P = \frac{4.99}{0.572} R$$
$$P = 8.75\, R \quad ..(iii)$$
$$\Bigg\} \quad ..(3.25)$$

Elimination of P and R from these equations yields

$$S = 0.44\, d^{1.15}\, Q^{-0.46} \quad ...(3.26)$$

An equation that gives the limiting values of slope at which the limiting, or Type B, channel shape derived in Lane's Theory would just be stable. In greater slope than that value, the wide Type A channel of less scouring capacity is required. If the slope is less than that given by Eq.(3.26), then type C channel is apparently required.

Meanwhile an equation will be given to the surface width W_s. From Eq.(3.13) and (3.19)

$$W_s = 4.49\, Y_o = \frac{4.49d}{11S} = \frac{0.41d}{S} \quad ...(3.27)$$

From Eq.(3.26) we get $W_s = 0.93\, d^{-0.15} Q^{0.46}$ ...(3.27a)

Similarly for wetted perimeter (P (by eq. 3.15, 3.19, and 3.26))

$$P = 1.03\, d^{-0.15}\, Q^{0.46} \quad ...(3.27b)$$

These equations are remarkably similar in form to the Lacey Eq. $P = 2.67\, Q^{1/2}$ ...(2.28)

Although they are more general in that they are taking account of the bed material size, equations 3.27b and 2.28 give the same result with d = 0.02 in approximately, which is well within the range of silt sizes of the Indian canals on which Lacy's equations are based. For verifying Lacey relationship as

$$V = 1.55 \sqrt{fR} \qquad ..(2.15)$$

The author used Eqs. (3.20 and 3.24) and eliminated S between the above equations. The resulting equation is with 10% increase in constant,

$$V = \frac{30\ R^{1/2} d^{1/3}}{(19R)^{1/3}} = 11\ d^{1/3}\ R^{1/6} \qquad ...(3.28)$$

This equation is substantially different in form from the corresponding Lacey equation.

In summary from Lane's tractive force theory the following equations, which are applicable in limiting or Type B channel cross-section, are derived. The specific gravity equal to 2.6 and angle of repose 35°.

$$V = 11\ d^{1/3}\ R^{1/6} \qquad ...(3.28)$$

$$V = 30\ R^{1/2}\ S^{1/3} \text{(10\% increase in constant)} \qquad ..(3.24)$$

$$W_s = 0.93\ d^{-0.15}\ Q^{0.46} \qquad ...(3.27a)$$

For the other proporties of bed material only the coefficients will be changed. The two latters equations are similar in form to corresponding Lacey's equations but the first one is different. Although the three equations are derived for the Type B channel section, the first two will be true for all width-depth ratios, with only slight changes in constants. But the third one is applied only to the Type B section. For simplicity of comparing the particular values of $\emptyset$ and S_s, were assumed. The general forms of the equations are as follows with number of particular forms, of equations on left of each equation, that is, that for which $\phi = 35^{\circ}$ and $r = 1.6$

$$(3.18, 3.19, 3.20) \quad d = \frac{18RS}{r} \quad \text{or} \quad \frac{18Y_oS}{r} \qquad \ldots(3.29)$$

$$(3.23) \quad V = 30.6\, r^{1/6} R^{1/2} S^{1/3} \qquad \ldots(3.30)$$

$$(3.24) \quad V = 30.6\, R^{1/2} S^{1/3} \left(\frac{r \cos\phi}{E} \right)^{1/6} \qquad \ldots(3.31)$$

$$(3.26) \quad S = 0.24 r^{1.15} \left[\text{Cot}\,\phi \sqrt{\frac{\cos\phi}{E}} \right]^{0.46} d^{1.15} Q^{-0.46} \ldots(3.32)$$

$$(3.27a) \quad W_s = 0.73 \; \frac{\cos\phi \cot\phi}{E} \left(\tan\phi \sqrt{\frac{E}{\cos\phi}} \right)^{0.46} r^{-0.15} d^{-0.15} Q^{0.46} \qquad \ldots(3.33)$$

$$(3.27b) \quad P = 0.47 \cot\phi \left(\tan\phi \sqrt{\frac{E}{\cos\phi}} \right)^{0.46} r^{-0.15} d^{-0.15} Q^{0.46} \ldots(3.34)$$

$$(3.28) \quad V = 11.7\, d^{1/3} R^{1/6} \left(\frac{r \cos\phi}{E} \right)^{1/2} \qquad \ldots(3.35)$$

$$(r = S_s - 1)$$

As mentioned before that the above formulae are applied when the particle Reynolds number is above approximately 400. Now if we eliminate τ_c between Eq. (3.17) and the Eq.$Re^* = 400$, with Kinematic viscosity ν assumed to be 1.2×10^{-5} ft^2/sec., the lowest value of d to which the equations are applied, will be found approximately 0.24 in or the sediment size is larger than 1/4 in. For further generalization to be applied in the sediment size less than 1/4 in, it is necessary to deal with the dip in Shields curve of entrainment function versus particle Reynolds number ($Re^* = \sqrt{\frac{\tau\, d}{\rho \nu}}$). When $Re^* < 400$ and the entrainment function , $1/\varphi$ is accordingly less than 0.056, say 0.056 K. Then the coefficient 11 in Eq.(3.18) will be 11/K and the coefficient of other equations altered accordingly. The effects may be summed as below.

Equation number	3.18, 19 20, 29	3.23, 24 30, 31	3.26 3.32	3.27a 3.33	3.27b 3.34	3.28 3.35
Multiply right hand side by	K^{-1}	$K^{1/6}$	$K^{1.23}$	$K^{-0.23}$	$K^{-0.23}$	$K^{1/2}$

In the second step the author proved that: in "Regime" equation $R^{1/2}S$ = constant, and in threshold theory RS= constant. For this purpose he used Einstein's bed load function ,

$$\phi = \frac{q_s}{Fd\sqrt{g(S_s-1)d}} \qquad \ldots(3.36)$$

in which q_s is the sediment discharge per unit width of the channel. Einstein showed that ϕ was related to the entrainment function

$$\frac{1}{\varphi} = \frac{\tau}{(S_s-1)\gamma d} \qquad \ldots(3.37)$$

in this way $\phi \rightarrow 0$ as $1/\varphi$ 0.056, and for $1/\varphi > 0.1$ approximately

$$\phi = 40\ (1/\varphi)^3 \qquad \ldots(3.38)$$

On the straight line portion of the $\phi - \frac{1}{\varphi}$ curve as mentioned. Because $\tau = \gamma_w RS$, Eq(3.38) can be written as

$$\frac{q_s}{Fd\sqrt{gd}} = \frac{40}{(S_s-1)^{5/2}}\left(\frac{RS}{d}\right)^3 \qquad \ldots(3.39)$$

in which F is a function of grain size and fluid property ($F = \frac{2}{3} + \sqrt{\frac{36\nu^2}{gd^3(S_s-1)}} - \sqrt{\frac{36\nu^2}{gd^3(S_s-1)}}$). Therefore, for any a canal whose bed material is the same along its whole length

$$q_s \propto R^3 S^3 \qquad \ldots(3.40)$$

Further, along any onee table channel the sediment concentration q_s/q will be constant as well as d. If we consider the resistance equation of H.K.Lin, M.ASCE and S.Hwang, A.M.ASCE [61]

$V = CR^X S^Y$ ...(3.41)

The values of X and Y depend on bed formation and sediment size . Then, q = VR and by using Eq. (3.41) we can write,

$$q = VR = CR^{1+X} \ S^Y \quad ...(3.42)$$

$$\frac{q_s}{q} \propto R^{2-X} \ S^{3-Y} \quad ...(3.43)$$

For any one stable canal system, with d and q_s/q constant it follows that $R^{2-X} \ S^{3-Y}$ is constant . If $R^{1/2}S$ = constannt, then, $(R)^{\frac{2-X}{3-Y}} \ S = \text{constant}^{1/(3-Y)}$ = constant. It follows that $\frac{2-X}{3-Y} = \frac{1}{2}$. This condition is very nearly fulfilled as per table below in which the values of X and Y are given by Lin and Hwang . And is exactly fulfilled by X=3/4 and Y=1/2 which are assumed by Lacey.

TABLE 3.7

X	Y	$\frac{2-X}{3-Y}$	$X-Y\frac{2-X}{3-Y}$	$\left[\frac{2(1+X)(3-Y)}{2-X} - 2Y\right]^{-1}$	Remarks.
5/7	4/7	8/17	0.412	0.188	Plane bed, d<0.2 mm
5/7	0.55	0.525	0.426	0.184	Plane bed, d =0.4mm
2/3	1/2	0.533	0.4	0.190	Manning Eq. d>10 mm
0.35	0.30	0.61	0.167	0.262	Dune bed, d =0.01mm
0.4	0.30	0.59	0.223	0.241	Dune bed d=0.1mm
0.5	0.33	0.56	0.315	0.214	Dune bed d=0.4mm
2/3	0.4	0.513	0.462	0.175	Dune bed d=2mm
0.6	0.35	0.53	0.414	0.216	Dune bed, d=0.7mm

Lacey's first Eq($V \propto \sqrt{R}$) , does not stand up well by the above examination. This equation is derived from Lacey's resistance equation, $V = K R^{3/4} S^{1/2}$ ...(3.44)

and Eqn ($R^{1/2}S$= const). by elimination of S between them,

$$V \propto \sqrt{R}$$

Now, if Lin and Hwang's more general resistance equation is considered and S is eliminated between them, then we have,

$$R^{\frac{2-X}{3-Y}} S = \text{Constant} \qquad ...(3.45)$$

and $V \propto R^X S^Y$ ...(3.46)

leads to $V \propto R^{X-Y\frac{2-X}{3-Y}}$ ...(3.47)

From the table 3.7 , we can say that value $(X-Y\frac{2-X}{3-Y})$ is not constant and approximately equal to 0.5. It is seen that the values vary widely, and approaches the value 1/2 only in certain cases. For providing that $P \propto \sqrt{Q}$ is true, if there is a certain slope discharge relationship, again we will use a channel in which d and q_o/q are constant and Eq. 3.43.

$$R^{2-X} S^{3-Y} = \text{constant} \qquad ...(3.43)$$

$$V = CR^X S^Y \qquad ...(3.41)$$

wetting Q = VPR and eliminating R we can get,

$$P \propto QS^{\frac{(1+X)(3-Y)}{2-X} - Y} \qquad ...(3.48)$$

If the relationship $P \propto Q^{1/2}$ is true, then,

$$S \propto Q^{-\frac{1}{\frac{2(1+X)(3-Y)}{2-X} - 2Y}} \qquad ...(3.49)$$

From Table 3.7 , we can see that this value is quite close to −1/6 , for plane bed, For dune bed, it ranges between −1/4 and −1/6.

On the other hand if we eliminate R between Eq.(3.18) and (3.23) which both are applicable in wide channels we will get,

$$P = \frac{1.14\, Q\, S^{7/6}}{d^{3/2}} \qquad ...(3.50)$$

From these we can say if the relationship $P \propto \sqrt{Q}$ holds in certain canal system there must be a slope-discharge relation

$$S^{7/6} \propto Q^{-1/2}$$

That is $\quad S \propto Q^{-0.46} \qquad ...(3.51)$

Similarly in order that the relation (such as eq.3.27b)

$$P \propto d^{-0.15}\, Q^{1/2} \qquad ...(3.52)$$

can be true, if the relation,

$$S \propto d^{1.15}\, Q^{-0.46} \qquad ...(3.53)$$

would have to be true (Eq. 3.26)

Hence, the dependence of a P-d-Q relation on an S-d-Q relation has the same form for wide channels and narrow Type B channels. On a live bed theory, on other hand,

$$S \propto Q^{-1/6} \qquad ...(3.54)$$

The summary of the result that Mr. F.M. Henderson got is as below.

1. By using Shields criterion in tractive force method, with Lane's theory, there is theoretical basis for the Lacey relation $P \propto \sqrt{Q}$ in special case of the narrowest possible channel with bank stability.
2. The author proved by the help of Einstein bed-load and Lin and Huang's resistance formulae, that in live-bed condition $R^{1/2}S$ = constant is true.
3. By the same analysis he proved that $V \propto \sqrt{fR}$ is not well founded.
4. In both cases (fixed and live bed) only two equations

can be deduced immediately from the considerations of bed movement and flow resistance. A third equation, such as $P \propto \sqrt{Q}$, can only be true if a certain relation between discharge and slope exists.

5. This slope-discharge in threshold theory are similar in wide and narrow channels. It is $S \propto D^{-0.46}$ approximately, and in the case of live-bed theory it is $S \propto Q^{-1/6}$ approximately.

3.15. DR BHARAT SINGH's APPROACH [35]

Dr. Bharat Singh put forward the hypothesis that a bed load transport channel first adjust its cross-section to carry a maximum bed load and then adopts the necessary slope. He assumed rectangular cross-section and exponential relationship between bed load transport and tractive force as

$$G_s \propto (\tau_b \frac{n'}{n} - \tau_c)^{3/2} \; B \qquad \ldots(3.55)$$

he has obtained the equation for width of optimum sediment transporting channel as

$$B = \left[10.4 \frac{n_w^{0.68}}{n_1^{0.136}} \left(\frac{n'}{n}\right)^{0.45} \frac{s^{0.182}}{\tau_c^{0.454}} \right] Q^{0.545} \qquad \ldots(3.56)$$

If we keep the values $n \simeq n_1 = n_w$, approximately the equation will be

$$B = 10.4 \, n^{0.094} \, n'^{0.45} \frac{s^{0.182}}{\tau_c^{0.454}} Q^{0.545} \qquad \ldots(3.57)$$

Dr. B.Singh found that the equation holds satisfactorily with Gilbert's flume data. He used typical values

$n_v = 0.015$, $n_1 = 0.021$ and $n'/n = 0.60$ for regime channel with silt factor $f = 1.0$ in Lacey's slope equation and $\tau_b = 0.006$ lb/ft^2 and he deduced the equation for bed width as below:

$$B = 2.04\ Q^{0.515} \qquad \ldots(3.58)$$

This equation is much similar to Lacey's, $P = \frac{8}{3} Q^{1/2}$ equation.

Where

G_s = Transport of bed load in lb/hr

τ_b = Tractive stress on the bed of the channel, lb/ft^2

τ_c = Critical tractive stresses of the bed material lb/ft^2

n' = Manning resistance coefficient corresponding to size of bed material in the channel (Strichler's equation)

n = working value of Manning Coefficient for the channel as a whole.

n_1 = Actual working value of Manning coefficient for the channel bed.

n_v = Manning coefficient for the sides of the channel

B = Width of the channel, ft.

If all the n values are assumed to be proportiona to $d^{1/6}$ the slope to as proportional to $d^{0.9}$ (Eq.2.30) and τ_c as proportion to d, there will be $d^{1/5}$ left in denominator in equ. 3.6 . This means that the optimum bed load transporting section will be narrow for coarser material.

In 1961 [35] he derived, by experiment from flume data which was carried out by him and other and published data, the following equation.

$$\frac{C}{\sqrt{g}} = 5.65 \log \frac{13 R_b}{K'_s} \qquad \text{...(3.59)}$$

$$K'_s = K_s \left(\frac{67 \tau_b}{F_r^2} \right)^{\alpha}$$

The value of R_b is given by him as below

$$\frac{Q}{B R_b} = 32 \sqrt{R_b S} \left[\log \frac{13 R_b}{K_s} - \alpha \log(12 \gamma_w S R_b^2 S) + 2\alpha \log \frac{Q}{B R_b} \right]$$

...(3.60)

For given values of Q, S and B a value of α obtained from graph, only R_b remain unknown and can be solved by trial and error. This equation has been found to give better result with field data than Lacey's equation.

But Dr. Bharat Singh suggested it to be tested for more extensive data.

where,

C = Chezy constant

R_b = Hydraulic radius of bed

K'_s = equivalent sand grain roughness of ripple bed material. It is defined as $K'_s = K_s \left(67 \frac{\tau_b}{F_r^2}\right)^{\alpha}$ but for the field channel constant (67) is changed to 12 by him.

K_s = equivalent sand roughness for the bed material without ripple and it is equal to d_{65}

τ_b = bed stress in lb/ft^2

α = a function of grain size and grading which can be found from graphical relation between α and

d_{65}

F_r = Froude number = $\frac{V}{\sqrt{g R_b}}$

3.16 NING CHEIN'S RATIONAL APPROACH [62,63]

The regime theory was analysed by Ning Chein on the basis of Einstein's bed load function to attack the problem by a more rational approach to the design of alluvial channels. When the defect of the Lacey's silt factor(f) in the equation (2.15 and 2.31) was known he investigated the problem to overcome this difficiency . He thought that silt factor was dependent not only on the silt grade on which the bank and bed materials are made up but also it depends on the sediment in flow.

The author proposed that the omission of sediment relation in regime equation is because of the following-

1. The silt factors may implicitly include the sediment with bed material size, or
2. There may be a suitable combination of the variables R and S which rarely depend on the sediment load.

The above two ideas were actually Lacey's statement, while discussing the paper he agreed that the dimensions and slope of an alluvial channel having a given discharge must depend on the sediment load, but in certain combination of variables the sediment load was entirely absent, He proved the statement this way.

$$f_{VR} = 0.75 \frac{V^2}{R} \text{ and } f_{RS} = 192\, R^{1/3}\, S^{2/3}$$

Combining f_{RV} and f_{RS} and calling it $f_{\frac{RS}{V}}$, we have,

$$f_{\frac{RS}{V}} = \frac{f_{RS}^{3/2}}{f_{VR}^{1/2}} = 3072\ RS/V,$$

Putting $f_{\frac{RS}{V}}$ as proportional to square root of sediment diameter ($d^{1/2}$) and inserting (g), it can be written

$$V = \text{constant}\ \frac{g^{1/2}}{d^{1/2}}\ RS \qquad ...(3.61)$$

This is the same equation as given by Lacey and Malhotra(2.42) which is dimensionally homogeneous equation and independent of charge.

However, Hing Chien collected data from Indian canals and rivers used by Lacey and utilised his graphical solution of channel based on Einstein's Bed Load function to test those hypotheses. He started the investigation from these equations (Lacey's Regime equations)

$$\frac{V^2}{R} = 1.32\ f_{VR} \qquad (i)$$

$$R^{1/2}\ S^{2/3} = 0.0052\ f_{RS} \qquad (ii)$$

$$P = 2.67\ Q^{1/2} \qquad (iii)$$

The analysis was conducted in following three steps:

a. Bank friction neglected and the conditions as obtained in Indian canals and rivers were investigated and computations made.

b. This type of analysis was extended to conditions different from those prevalent in India and Pakistan.

c. Bank friction was i troduced and its effect was studied.

In the first step f_{VR} and f_{RS} were computed from the following conditions-

(i) Bed material size 0.25 mm (average size in North India and Pakistan).

(ii) R varies between 2ft - 25 ft.

(iii) S varies between 0.001 to 0.004.

Values of f_{VR} and f_{RS} were plotted against sediment concentration and he had obtained the following relations

$$f_{VR} = 0.061 \left(\frac{q_T}{q}\right)^{0.715} \qquad ...(3.62)$$

$$f_{RS} = 1.18 \left(\frac{q_T}{q}\right)^{0.052} \qquad ...(3.63)$$

where q_T is total sediment transport rate / unit width.

In the second step he neglected the bank friction.

Bed material size = 0.25 mm, 2.5 mm and 25 mm.

R = 2 ft to 30 ft.

S = 0.001 to 0.005 for 0.25 mm size.

= 0.003 to 0.0025 for 2.5 mm size.

= 0.001 to 0.01 for 25 mm size.

He had plotted f_{RS} and f_{VR} against q_T/q by keeping R constant and S varying and S constant R varying and found a set of graph showing that : f_{VR} became quite erratic but f_{RS} remains the same as before, or

$$f_{RS} = 2.2\, d^{0.45} \left(\frac{q_T}{q}\right)^{0.052} \qquad ...(3.64)$$

for $\left|\frac{q_T}{q}\right| < 200$ ppm and $0.25 < d < 25$ mm.

In the third step he has introduced bank friction with vertical side wall with n = 0.02 and

R = 2ft - 30 ft

S = 0.001 - 0.005

His results and conclusions are summarized as below-

1. In the conditions pertaining to Indian plains f_{VR} depends on sediment concentration, f_{RS} depends primarily on bed material size.
2. Using the functional relationship given both Einstein bed load function and the regime theory give the same depth and slope of alluvial channel in equilibrium.
3. For conditions different from these f_{VR} depends also on the hydraulic characteristics of the channel.
4. Bank friction has little effect on the overall trends according to which silt factors vary with the sediment concentration.

It will be observed from these findings that Lacey's criterion for a regime channel $R^{1/2}S$ = constant is well founded. But his criterion $V \propto \sqrt{R}$ seems not well stabilised as f_{VR} depends on hydraulic properties as well as sediment concentration.

There will be a possibility of correlation between Lacey's regime equation and the threshold theory.

3.17 LANGBEIN's THEORY [64]

Langbein states that rivers construct their own geometries. The principles of hydraulics, which are applicable in firm boundary channel cannot be applied to determine the form and slope of river channels.

The hydraulic geometry" defined by him is the mean form of various forms of the stream channel.

The author has stated that: stream channels form their hydraulic geometry in response to loads of water and sediment imposed. In accommodating a change in discharge, rivers can change velocity, width, depth and slopes. These changes appear to be uniformly distributed among these components as is permitted by the river mechanics.

As a consequence of the generalisation that the total work tends towards a minimum, because the rate of fluid friction is measured by the energy slope, the rate of work (power) per unit length along a river equal $\gamma_w QS$. Variance of power tends towards zero as the product QS tends towards a constant value along the river. But, as QS tends towards uniformity, total work, as expressed by the integral $\Sigma \gamma_w QS \, \Delta X$, tend toward a minimum.

He mentioned that both of these ideas - equal action between adjustment of variable and principle of minimum work is to be found in geomorphic literature by Gilbert and Ruby and others.

He compared the river section by relating the channel's proportion to bankful discharge, Q, as given below:

V (Velocity) $\propto Q^m$

D (depth) $\propto Q^f$

W (width) $\propto Q^b$

S (Slope) $\propto Q^z$

By using the postulate cited above, and by utilising hydraulic properties of channel and the resistance equation of Manning and principles of statistical analysis he found the most probable state among alternatives.

$b = 0.53$ and $V \propto Q^{0.53}$ ...(3.65)

$f = 0.37$ and $D \propto Q^{0.37}$ ...(3.66)

$m = 0.10$ $W \propto Q^{0.10}$ ...(3.67)

$z = -0.73$ $S \propto Q^{-0.73}$ ...(3.68)

He has pointed out that the analysis is weak in the sense that end points are based on the probable states, rather than points of exact balance between the forces in the action. But the values are given for averaging long reaches and a wide range of observation of bankful discharges.

Comments - It is therefore reasonable to assume that the state of equilibrium will be attained for a channel in regime after long period.

3.18 LABORATORY CHECK

Laboratory experiments have the great advantage that they permit the control of the variables and the study of almost every phase of sediment transport. Nevertheless, the time has not yet come when results obtained by models can be applied with complete confidence to the design of large canals. However, the physical laws involved in the case would be qualitatively good for checking the interrelationship between the variables . Out of these studies two of them are discussed here.

3.19 PETER ACKER [65]

He has conducted his research with sand having median diameters of 0.16 mm and 0.34 mm , with discharges varying from 0.4 cusecs to 5.4 cusec. He selected a small model channels in sand excavated to a s traight trapezoidal channel but free to adjust their cross-sections, areas, alignment, sediment concentrations and gradient until stability was reached. Then he measured the discharge, slope, shape, cross section and sediment charge. During the performance of experiment, he came across the following three types of channels:-

i. Channels whose bed and banks were covered with a deposit of fines and clay making them inerodible. The fines and clay were washed out during the scouring stage, and seepage at initial running, remained in circulation and were deposited on bed and sides, at the equilibrium stage giving rise to these types.

ii. Channels that eroded to a stable section in which significant bed and bank material was fine sand provided in the model.

iii. Channels that developed prominent shale and meander

He has found for class (ii) of channels these equations.

		Lacey's equation
$A = 1.00\ Q^{0.85}$	..(3.69)	$A = 1.126\ (Q/f^2)^{5/6}$
$V = 1.00\ Q^{0.15}$	..(3.70)	
$W = 3.6\ Q^{0.42}$	..(3.71)	$P = 2.67\ Q^{0.5}$
$D = 0.28\ Q^{0.43}$	..(3.72)	

He found little variation of cross-sectional shape with discharge, the ratio W/P and P/R remain constant and independent of discharge. He could not find any correlation between S and Q and found that the power of 1/6 in Lacey's equation is under estimated. He has found out that his observations compare well with Simons and Albertson's equations for comparable canals.

He also found that the resistance equation

$$\frac{V}{\sqrt{gRS}} = 7.13 \quad (R/K)^{1/4} \qquad \ldots(3.73)$$

Adequately describes the log turbulent equation

$$\frac{V}{V^*} = 2\sqrt{8} \quad \log 12.3 \quad \frac{R}{K_s}$$

of Keulegan in the range $1.5 < \frac{R}{K} < 11$

He has used the resistance equation (3.73) and sediment transport equation (3.75) derived from his experimen and third consideration of B/D ratio, obtained relationship to define channel geometry as below:

From resistance considexation he has found

$$V = \alpha_2 \, D^{0.75} \, S^{0.50} \qquad \ldots(3.74)$$

where $\alpha_2 = \frac{7.13\sqrt{g}}{K^{0.25}}$ and K = height of ripple.

From sediment transport aspect he found

$$V = \alpha_1 \, D^7 \, S^8 \qquad \ldots(3.75)$$

$$\alpha_1 = \frac{2400 \, V_s \quad d^{-7}}{\frac{N \, (S_s - 1)8}{S_s}}$$ where d = diameter of particles in mm and N is sediment charge in ppm.

From width-depth relation, he has obtained

$$D/W = \alpha_3 \qquad \ldots(3.76)$$

α_3 = constant for small stream in non-cohesive material or may be a function of cohesive strength of bank material. From consideration of equations 4,5,6 he has obtained the following relationships.

$$D = \alpha_1^{0.03}\ \alpha_2^{-0.47}\ \alpha_3^{0.43}\ Q^{0.43} \qquad \ldots(3.77)$$

$$W = \alpha_1^{0.03}\ \alpha_2^{-0.47}\ \alpha_3^{-0.47}\ Q^{0.43} \qquad \ldots(3.78)$$

$$A = \alpha_1^{+0.06}\alpha_2^{-0.94}\ \alpha_3^{-0.14}\ Q^{0.86} \qquad \ldots(3.79)$$

$$V = \alpha_1^{-0.06}\ \alpha_2^{0.94}\ \alpha_3^{-0.14}\ Q^{0.14} \qquad \ldots(3.80)$$

$$S = \alpha_1^{-0.16}\ \alpha_2^{0.56}\ \alpha_3^{+0.36}\ Q^{-0.36} \qquad \ldots(3.81)$$

He derived from equation (3.74, 3.75) and compared them as below

Peter Acker	Ingis – Lacey
$\frac{V^2}{D} \propto N^{0.08} d^{0.5} S^{0.4}$ ...(3.82)	$\frac{V^2}{D} \propto N^{0.5} d^{0.5}$
$\frac{V^3}{W} \propto N^{0.2} d^{1.2} \alpha_3$ ...(3.83)	$\frac{V^3}{W} \propto d^{0.5}$
$V \propto N^{-0.2}\ d^{-0.2} \left(\frac{D.S}{\sqrt{d}}\right)^2$ ..(3.84)	$V \propto \frac{D.S}{\sqrt{d}}$
	Blench
$\frac{V^2}{DS} \propto N^{-0.25} d^{-0.15} \alpha_3^{0.375} (V.W)^{0.375}$ ...(3.85)	$\frac{V^2}{DS} \propto (VW)^{0.25}$
	Lacey
$V = \alpha_1^{-0.02} \alpha_2^{1.02}\ D^{0.61}\ S^{0.33}$	$V = 16\ R^{0.67}\ S^{0.33}$

Petor Acker

$$V = \alpha_1^{-0.026}\ \alpha_2^{0.71}\ (D^2S)^{0.285} \quad ...(3.86)$$

Simons and Albertson

$$V = 17.9\ (R^2S)^{0.286}$$

(Coarse non cohesive)

3.20 B. SINGH AND SINGHAL'S MODEL STUDY [25]

The authors have studied the behaviour of self-formed channels under control conditions, and to attempt the determination of fundamental correlations of sediment bearing channels specially with reference to width adjustment. In all the previous experiments on the laboratory study of alluvial channels sediment load was not considered as an independent variable. The study of Petor Ackers, though very comprehensive , did not include sediment load parameters in his best fit equations. This useful experiment extended by authors by inclusion of sediment load in their works and making the experiment approach more nearly to field conditions. Experimental river bed sand having a medium size of 0.3 mm and standard deviation of 1.55 specific gravity 2.66 and average fall velocity of grain in clear water 0.05 m/sec. (0.16 ft/sec) was used. The range of variables involved in the experiment is given as below:

Discharge	0.33 - 1.90 cusec.
Sediment concentration	0 - 3100 ppm.
Sediment size d_{50}	0.30 mm
Slope	1/180 to 1/1750
Water surface width	3.33 to 8.1 ft.

Mean depth 0.10 to 0.31 ft.

Mean velocity 0.62 to 2.32 ft/sec.

W-Q relations

By plotting available data they found the following relations

i. B. Singh and Singhal data and Wolman-Brush data

$$W_s = 5.95 \; Q^{0.59} \qquad \ldots(3.87)$$

ii. Peter Acker Data

$$W_s = 3.24 \; Q^{0.42} \qquad \ldots(3.88)$$

iii. Simon -Bender's (U.S.Canals) Indian canals (Field data)

$$W_s = 2.24 \; Q^{0.50} \qquad \ldots(3.89)$$

Thus they observed, experimental channels are wider than the field channels and the effect of sediment concentration on W_s-Q relationship on width is practically negligible, but the effect of discharge is strong. The effect of sediment size also is surmised to be small.

They have found value of the constant in the equation

$$W_s \propto \frac{I^{1/4}}{d^{1/4} \; S^{1/4}} \qquad \ldots(2.56)$$

ranged from 0.27 to 1.21 showing almost a four fold variation. The value of this constant as reported for some of the field data is 0.25 while Chaturvedi and Bharat Singh obtained values of 1.06 and 1.00 for runs in cohesionless sand. They therefore concluded that the Inglis -Lacey equations fail to represent the different conditions of field and laboratory . They have used surface width for perimeter in White's equation $P = 4.04 \; \frac{Q^{0.432}}{S^{0.136} \; V_s^{0.16}}$

$$\ldots(2.89)$$

and found the values of the numerical constant varying from 5.10 - 7.74 . Since the experimental channels are thus 25% to 90% wider than those rivers from which White's numerical constants were taken. Therefore they concluded that White's equation is better fitted than the Inglis-Lacey Equation. They have found that the value of B obtained by equation 3.56 were lower than the values observed. But they have found if the top width is calculated by this method it tallied well with that measured in the model.

A-Q Relation.

The same data as were used in H-Q relation were also used to make an A-Q plot, and they found that unlike the width plot , however, in this case all the data laboratory as well as field, fall on a single line conforming to the the equation

$$A = 0.9\, Q^{0.89} \qquad \ldots(3.90)$$

Since the data contained wide range (0.02 - 0.715 mm) of sediment sizes and intensity (4 ppm to 5,759 ppm) and that a common line has been fitted for the entire data it indicates that cross-sectional area of a channel is not significantly dependent on either the grain size or the amount of sediment flowing.

A-Q relation of White

They have found the constant in equation

$$\frac{A g^{2/5}}{Q^{4/5}} = 2.4 \left(\frac{g^{2/5} Q^{1/5}}{V_s} \right)^{0.22} \qquad ..(2.82)$$

vary from 0.9 to 2.76 . The lower values were however, for isolated runs. Twenty three of the twenty eight points were within 25% of White's constant.

<u>A-Q Relation - Inglis</u>

They have found the constant in Inglis Equation (2.57) for area varied 0.57 to 2.54 for the data of their experiment and concluded that the Inglis number in no case improves the Lacey relationships.

<u>S-Q Relation</u>

A log-log plot of S against Q, made for their data did not give any correlation, there being very wide scatter. The constant of Inglis-Lacey equation obtained by their data ranged from 0.0013 to 0.0042. Therefore it become evident that incorporation of sediment concentration in Inglis equation does not enable it to conform to field data. They plotted V against $g^{1/2} / d^{1/2}$ RS and did not find any correlation.

Thus they concluded that Q, S relation does not represent true regime condition, there may be some other factor which influences the correlation.

<u>Derived Equations</u>

For an analytical solution of the problem they have used the following equations -

(i) Flow Equation

$$V = \frac{1.486}{n} R^{2/3} S^{1/2} \quad \text{(Manning Formula)}$$

(ii) The bed load transport equation assuming that the suspended load concentration is within the carrying capacity of the channel with the help of turbulent energy contained in it. They obtained the bed load transport equation by empirical relation as below:

$$q_s^{0.182} = 2300 R'S$$

where $R' = R\left(\frac{n'}{n}\right)^{3/2}$ ...(3.91)

n' = Manning coefficient for grain resistance, and

n = Manning coefficient for channel as a whole.

q_s = Bed load transport rate per unit width.

(iii) A condition determining or relationship defining the channel geometry in the form of a width-depth ratio. They obtained by their experiment the following equation

$$\frac{W_s}{R} = 18.5\, Q^{0.45}\, N^{0.27} \quad ..(3.92)$$

whereq = discharge in cusecs/ft.

$$N = \frac{q_s W}{Q} \times 10^6$$

From three basic equations obtained by empirically from their data and Wolman and Brush and some algebraic manipulation, they derived the following equations for cohesionless bed and banks.

$$W_s = 19.3\, Q^{0.55}\, n'^{0.286} n^{0.096}\, N^{0.086} \quad ...(3.93)$$

For their data n' = 0.0106, average value of n=0.02 then equation (3.93) becomes,

$$W_{n} = 3.6\, Q^{0.55}\, N^{0.036} \qquad ...(3.93a)$$

$$A = 80\, n'^{0.70}\, n^{0.24}\, Q^{0.9}\, N^{0.06} \qquad ...(3.94)$$

Putting the mentioned values for n and n' we get,

$$A = 1.23\, Q^{0.9}\, N^{-0.06} \qquad ...(3.94a)$$

S-Q relation

Bharat Singh and Singhal have made a statistical correlation for the field data of the Indian canals and their experiment, calculating the possible maximum transport rate of Indian canals by Mayer-Peter's equation and have obtained the following relation

$$SQ^{1.8} \times 10^{4} = 3.4\, (1+N^{0.4})^{0.85} \qquad ..(3.95)$$

They argued that there is a strong relation between S and N and a weak one between S and Q.

C H A P T E R-IV

APPRAISAL OF FINDINGS AND CONCLUSIONS

4.1 GENERAL

Generally, when a sediment bearing canal in alluvium is run steadily with a definit discharge, and silt charge, it tends to adjust width, depth and slope to equilibrium values irrespective of their originally constructed values. This is defined as "regime". This self-adjustment indicates that each of the three dependent variables. width, depth and slope is a function of independent variables the discharge, sediment grade and sediment load.

Under such circumstances the most basic problem of canal engineers as well as research workers is how to find the elements of the channel cross-section and its slope to carry a given discharge and sediment load free of scour and deposit or in-stability condition.

Stability denotes freedom from change but within the annual cycle of many sediment transporting canals there is a cyclic variation in the sediment load. During floods in parent river part of the sediment admitted at the head of the canal may be deposited on the bed and entrained later in the season by water less heavily loaded with sand.

The investigations of the problem which were reviewed in the previous chapters can be categorised as below:

I. Original Regime Approach as Developed in India

These works consists of Kennedy, Lindley, Lacey and Bose. This was based on field measurements of stable canal sections and include only the discharge and sediment grade as independent variables.

II. Modified Regime Approaches

These works consist of investigations by Inglis who included sediment load also, Blench who took into account the difference between bed and bank materials, or its examination in light of bed load equation as done by Ning Chien, Simons and Albertson and Leopold and Maddock and some others have proposed correlations on somewhat similar lines on basis of other river systems, White has made what is perhaps the only reasonably successful dimensional approach to the problem.

III. Rational or Semi Theoretical Approach

This includes the successful methods for designing scour free channels carrying clear water developed by Lane, Henderson and Langbein, and the bed load theories given by Meyer Peter, Einstein and others and some semi theoretical analysis by Dr. Bharat Singh and Ning Chien.

4.2 INADEQUACY OF REGIME METHOD

Defects of Kennedy's Theory:

1. No actual measurements of discharge , velocity and slope were made.
2. He did not correlate n (rugosity coefficient) to m(CVR)

3. The importance of bed width and depth relationship was ignored by Kennedy. Hence by equation (2.2) a narrow deep channel and a shallow wide channel can be designed for the same discharge which is impossible.

4. He did not take account of silt charge . However Lindley's equations combined with Kutter's equation gave a unique solution. But his use of Kutter's equation for determining the velocity was open to question as the value of roughness varied with the discharge.

The Lacey's equations and their modifications have been very popular in India, but have never been used extensively elsewhere in the world. The group of equations undoubtedly provides as sound a basis for design as currently exists if they are used under circumstances similar to those from which they are obtained. The major disadvantages of the method are:

1. It fails to recognise the important influence of sediment charge on design,

2. The derivation of various formulae depends upon a single factor and dependence is not adequate. There are different phases of flow on bed and sides and hence different values of silt factor for bed and sides should have been used. Lacey approach of averaging the effects of sides and bed in a single factor is not justified.

3. It has not been developed based on the wide variety of conditions encountered in practice.
4. It involves factors that require a knowledge of conditions upon which the formulae are based if they are to be applied successfully.
5. When data from one site were inserted in each of the equations derived algebraically from originally Lacey ones different values of f were obtained.

The regime equations presented by Blench modify the Lacey equations in such a way that the effects of side and the bed of the channel can be evaluated separately by means of side factor and bed factor. This approach seems basically more sound than averaging the two effects as the Lacey equations do since effect of side and bed conditions on flow are vastly different. Lacey and Blench claimed that their equations represented fundamental laws with arguments advanced by them in support of this claim. But the general shortcomings of his equations given by some research workers are :

1. The expression for square of the mean tractive force intensity on hydraulically smooth sides is $\frac{\rho\mu V^3}{X}$, where X is measured in the direction of flow, but in his equation $\frac{\rho\,\mu\,V^3}{W}$, W is measured in perpendicular to the flow. Thus shear stress could only be caused by secondary currents.
2. Equation $\frac{V^2}{gDS} = 3.6\left(\frac{VW}{v}\right)^{1/4}$ may be rewritten as $V = \text{Abs. Cons.}\left(\frac{VW}{v}\right)^{1/8}\sqrt{gDS}$, which is exactly

the same in form of Blasius equation for smooth pipe, except that Re (Reynolds Number) is formed with W as the length parameter, which seems of doubtful validity. And also Blasius equation is applied to hydraulically smooth boundaries. Many of the regime channels will have hydraulically rough boundaries.

3. His bed factor F_b is the same as Lacey's constant $(1.55\sqrt{f})^2$. Leliavsky [66] has pointed out that F_b has been found to vary widely between 0.60 to 1.25 i.e., a 100% range, for the same type of channels.

4. It will be seen that the bed factor F_b is indirectly the square root of Froude number and it varies widely for different bed configurations depending on flow conditions, the size distribution of bed load material, sediment intensity and the total wash load. Therefore, F_b cannot be a constant as supposed by Blench in the material.

5. Comparing the $V^2/D = F_b = 9.6\sqrt{d}\ (1+0.012N)$ with Peter Acker Formula $\frac{V^2}{D} \propto N^{0.08} d^{0.5} S^{0.4}$, it will be seen that S is missing in Blench Equation.

6. If we consider side factor of Blench V^3/W, the following defects appear.

   i. The variation in side factor found by Leliavsky [66] ranges from 0.05 in some cases to 0.3 in other cases.

   ii. If we compare the side facto (V^3/W) to Peter Acker equation $\frac{V^3}{W} \propto N^{0.2} d^{1.2} \triangleleft_3$, it can be seen that side factor has much dependence on sediment concentration and considerable dependence on diameter of particles and D/W ratio.

7. Simons and Albertson [43] showed that the slope equation of Blench did not hold good. They showed that the exponent VH/ν is more approximate to 0.37 instead of 0.25 and that the relationship is approximate in the range $10^5 < \frac{VH}{\nu} < 10^7$, beyond which it fails. They also showed that constant 3.63 vary for different types of bed and bank materials. Peter Ackers [70] has also showed that the exponent of VH/ν is 0.37 instead of 0.25, and constant of slope equation has little dependence on sediment concentration ($N^{-0.025}$), some dependence on diameter of bed particle ($d^{-0.15}$) and strong dependence on D/W ratio ($\alpha_3^{0.375}$). The change of exponent and cost suggests that Blench analogy of Blasius form of equation does not hold good.

The credit goes to Inglis also due to including the sediment concentration in regime equations but his equations suffer from the following defects:

1. Constants of proportionality in his equations evaluated by many workers from field and laboratory data, have been found to vary widely as shown below:

Relation	Inglis (Field)	B.Singh and Singhal (Laboratory)	B.Singh and Chaturvedi (Laboratory)	U.P.I.R.I. (Field)
W-Q	0.545	0.27 - 1.21	1.06 - 1.00	0.257
A-Q	3.88	0.57-2.54	3.25 - 3.68	4.05
S-Q	0.00202	0.0013-0.0042	0.0058-0.0072	0.00134

2. Comparing to Peter Acker Formulae

		Peter Acker	Inglis-Lacey
$\frac{V^2}{D}$	$\propto$	$N^{0.08}\ d^{0.5}\ S^{0.4}$	$N^{0.5}\ d^{0.5}$
$\frac{V^3}{W}$	$\propto$	$N^{0.2}\ d^{1.2}\ \alpha_3$	, $d^{0.5}$
V	$\propto$	$N^{-0.2} d^{-0.2} \left(\frac{D.S}{\sqrt{d}}\right)^2$	$\frac{D.S}{\sqrt{d}}$

It can be seen that V^2/D is not so strongly dependent on sediment concentration as shown by Inglis and the slope is also missing in Inglis Equation. Considering V^3/W relation it is seen that this relation depends more on diameter of particles and D/W which does not confirm Inglis-Lacey findings. The V-S-D relationship of Inglis -Lacey is widely different from that obtained from by Peter Ackers. Finally Dr. B. Singh conclu- ded that inclusion of Inglis Number in no case improves Lacey's equations.

It becomes apparent that improvement in Lacey's equ tion by inclusion of various parameters to represent the effects of charge and grade of sediment were in vain. Lacey himself tried to rationalise his equations by modifying them from time to time. He had first replaced his P and R by water surface width W_s and mean depth D_m and has introduced parameters. $e = \frac{0.375}{Q^{1/2}} W_s$ and $E = \frac{P}{W_s}$ or $\frac{D_m}{R}$ to obtain his new set of equations.

In view of the very limited investigation these modified equations cannot be considered superior to his original equations. Only Chitale has verified these relations and has found that his D_m-Q-f-e relation and A-Q-e-f relation are superior to his original equations and has found that W_s-Q and S-E-D_m-W_s equations are inferior to his original equations. On his further modification he showed that V/RS = constant, but no research worker supports this relation. Dr. B. Singh and Singhal [25] in their experiment could not find any correlation. In general we can say that Lacey's original work is a great contribution but his subsequent attempts at rationalising his work have failed.

4.3 THEORETICAL APPROACH ASPECTS

The theoretical approaches evolved principles more capable of adequately predicting channel behaviour than any heretofore. There is no universal acceptable theory to consider all the variables involved in determining the flow characteristics completely. The complexity arises from the number of independent variables which determine the geometry of the channel of given discharge. Therefore as yet these theories are incomplete and should be subjected to further study and investigation in order to broaden their scope and possibly to reduce them into one general comprehensive and complete theory.

Inadequacy of the Tractive Force Method

The tractive-force concept is basically sound in so far as it has been developed. That is, for the design of channels for the conveyance of essentially clear water in coarse non-cohesive materials. However, its application is quantitative for-

1. Design of channels in fine non-cohesive soils(sand range and finer),
2. Design of channels in cohesive material, and
3. Design of channel that are required to transport appreciable (in excess of 500 ppm) sediment load.

In consideration of sediment load theories this point should be kept in mind that if the design of a channel is specified , it is possible by means of these theories to calculate the sediment load it would carry. However, to carry a specified sediment load, many alternative designs can be made in accordance with these theories. On the other hand, observations of natural streams indicate a tendency towards a unique set of channel characteristics with a given combination of independent, variables.

For the comparison of available literature all relationships are profitably tabulated as below-

TABLE 4.1

A-Q RELATIONS

1.	Lacey	$1.26 \dfrac{1}{(f^2)^{5/6}}$	$Q^{0.833}$	Indian canals
2.	Mathews	0.98	$Q^{0.833}$	Sudan canals($f=0.62$)
3.	Simmons and Albertson	1.076	$Q^{0.873}$	Sand bed and cohesive bank.
		0.45	$Q^{0.873}$	Coarse non-cohesive material.
4.	Pettis	1.25	$Q^{0.80}$	Miami river, bed and bank clay with shingle
5.	Leopold and Maddock	$\propto$	$Q^{0.90}$	American rivers, equal frequency discharge.
6.	Mars hall Nixon.	0.90	$Q^{0.833}$	England and Wales rivers equal frequency discharge (bank full discharge)
7.	Peter Ackers	1.00	$Q^{0.85}$	Experimental class II channels ($d=0.16$ mm)
		$\alpha_1^{0.06}\,\alpha_2^{-0.94}\alpha_3^{-0.14}$	$Q^{0.86}$	Empirical.
8.	B.Singh and Singhal	0.94	$Q^{0.89}$	Experimental($d_{50}=0.30$mm)
		$80(n'^{0.7}n^{0.24})N^{-0.06}$	$Q^{0.90}$	Empirical.
12.	Langbein	$\propto$	$Q^{0.90}$	Theoretical.

TABLE 4.2

P – Q RELATIONS

1. Lacey	$2.67Q^{0.50}$	Indian Canals
2. Mathews	$2.72\ Q^{0.50}$	Sudan Canals
3. Simmons and Albertson	$2.51\ Q^{0.512}$	Canals with sand bed and cohesive banks (Regime canals)
4. Pettis	$2.45\ Q^{0.50}$	Miami Rivers, Bed and Banks Clay with shingles
5. Leopold and Maddock	$Q^{0.50}$	American rivers equal frequency discharge.
6. Nixon	$1.65\ Q^{0.50}$	England and Wales rivers, equal frequency.
7. Henderson	$1.03d^{-0.15}Q^{0.46}$	Theoretical for silt-free channels.
8. Langbein	$Q^{0.53}$	Theoretical findings
9. Peter Ackers	$3.24Q^{0.42}$	Class I and II channels
	$\alpha_1^{0.03}\ \alpha_2^{-0.47}\ \alpha_3^{-0.47}\ Q^{0.43}$	Sand bank and bed, empirical
10. B.Singh and Singhal	$5.95\ Q^{0.59}$	B. Singh and Singhal, and Wolman Brush Data.
	$19.3(n^{10.2}n^{0.096})N^{0.086}Q^{0.55}$	Theoretical.
11.		

TABLE 4.3

D - R - Q RELATIONS

1. Lacey (R)	$0.473(1/f)^{1/3}$	$Q^{0.333}$	Indian Canals
2. Mathews		-	-
3. Simmons and Albertson	0.43	$Q^{0.361}$	Sand bed and cohesive banks
4. Pettis (R)	0.511	$Q^{0.30}$	Miami River
5. Marshall Nixon (D)	0.545	$Q^{0.333}$	England and Wales Rivers equal frequency discharge (Bankful discharge)
6. Leopold and Maddock(D) ≏ (R)	$\propto$	$Q^{0.40}$	American rivers
7. Peter Ackers(D)	0.28	$Q^{0.43}$	Experimental(d_{50}=0.16mm) Class II channels
(D) $\propto$	$\alpha_1^{0.03}\ \alpha_2^{-0.47}\ \alpha_3^{0.43}$	$Q^{0.43}$	Empirical
8. B.Singh and Singhal(D)	0.16	$Q^{0.30}$	Experimental(d_{50}=0.30mm)
(D) $\propto$	$(n_s^{0.70}\ n^{0.24})\ N^{-0.06}$	$Q^{0.45}$	Empirical.
9. Langbein (D) (R)	$\propto$	$Q^{0.37}$	Theoretical.

TABLE 4.4

S - Q - f RELATIONS

1.	Lacey	$0.000547\ f^{2/3}\ Q^{-1/6}$	Indian canals
2.	Methews	$0.000254\ Q^{-1/6}$	Sudan canals
3.	Peter Ackers	$\alpha_1^{-0.16}\ \alpha_2^{0.56}\ \alpha_3^{-0.36}\ \ Q^{-0.36}$	Experimental (empirical)
		No correlation	Experimental
4.	Leopold and Maddock	$\propto\ Q^{-0.49}$	American rivers, equal frequency discharge.
5.	B.Singh and Singhal	No correlation	Experimental
		$\frac{3.4(1+N^{0.4})^{0.85}}{10^4} x Q^{-0.18}$	- Empirical (Experimental)
6.	Henderson	$0.44\ d^{1.15}\ \ Q^{-0.46}$	Theoretical silt-free channels
		$\propto\ Q^{-1/4}$ to $Q^{-1/6}$	For plane bed and dune bed respectively(Theoretical).
7.	Langbein	$\propto\ Q^{-0.73}$	Theoretical .

TABLE 4.4a.

FLOW EQUATIONS

1. Lacey	$16\ R^{2/3}\ S^{1/3}$	Indian canals
2. Simmons and Albertson	$17.9\ R^{0.572}\ S^{0.286}$	Coarse non-cohesive
	$16.0\ R^{2/3}\ S^{1/3}$	Sand bed and Cohesive banks.
	$13.86\ R^{2/3}\ S^{1/3}$	Sand bed and banks
3. Henderson	$\propto R^{1/2}\ S^{1/3}$	Theoretical silt-free channel.
4. Peter Ackers	$\alpha_1^{-0.02}\ \alpha_2^{1.02}\ D^{0.61}\ \ S^{0.33}$	Empirical(Experimental)

4.4 CONCLUSIONS

1. For the design of canals carrying sediment laddon water, the Kennedy formula is now practically absolête.
2. Lindley's idea of 'uniqueness' of regime channel geometry for discharge, charge and grade of silt has been supported by field measurements and laboratory checks.
3. Inclusion of various parameters in original regime equation to represent the effect of charge and grade of sediment by Blench and Inglis and Lacey has been a failure as per discussion in Section 4.2.
4. By comparing the indices of the discharge in the regime relations in the A-Q, P-Q , R-Q equation (Tables 4.1, 4.2, and 4.3) given by different authors it is found that variation is very small. These changes may be due to varying

relative roughness in channel system which are investigated.

5. The perimeter relationship of Lacey represents an effort on the part of the channel to attain most efficient bed load transporting section, however, the transporting capacity of Lacey channel is not the optimum it is still close to optimum except for coarser material, the section is likely to be narrow. As concluded from Dr. B. Singh Eq.(3.57, 3.58).

6. The water surface width, W_s, has a strong correlation with Q, but does not depend significantly on sediment concentration or sediment size as concluded from experiment and equation of (3.93) of B. Singh and Singhal. The constants depend on bank erodibility. In channels with truly non-cohesive bed and banks, with little suspended sediment to provide long-term cohesion to the banks, the width is significantly larger than in channels with cohesive banks. The same width relationship cannot be applied to the two types of channels [25] For cohesionless bed and banks, the equation (3.93) looks accurate.

7. There is strong correlation between area and discharge. The area-discharge relationships are practically independent of sediment charge and grade as per conclusion in Section 3.20. The relation may be effected by channel roughness as shown by equations 3.94 and 3.79.

The constant value varies within short limit for canals (0.90– 1.06). It will probably be independent of flow characteristics, sediment charge and grade, and will depend on the properties of bed and bank material as proposed by Simons and Albertsons. As charge increases, the constant has a tendency to decrease resulting in decrease in area. As grade increases area increases for the same discharge and charge, this results in decreasing constant, as shown by Simons and Albertsons.

8. At present, there is no satisfactory slope relation or flow equation for alluvial channel(Table 4.4). Correlation between slope and sediment charge is strong and between slope and discharge is very weak. as per Equations 3.81 , 3.95 and 2.45. If sediment grade increases discharge and charge remaining constant increased slope is required to transport the sediment. If the charge increases sediment grade and discharge remaining constant, slope increases considerably.

9. Lacey's general formula for flow is not founded well, because the variation of Na with bed formation is ignored by Lacey as mentioned in section 2.5.6. But his general flow equation has found much support. The constant depends on bed formation or type of materials, Table 4.4.

10. Lacey's equations are valid for limited range of

condition on which they have been derived.

Leliavsky [66] showed that a slight difference in indices and constant will result in entire elements of the channel cross section. Hence any attempt in extrapolation for wider range and conditions gave little success.

11. It can be concluded that there is very considerable support for the regime approach from data of different part of the world. The fundamentals of the problem still remains to be understood. Until the time when satisfactory solution are available, the use of some empiricism is unavoidable.

12. The basic requirements for a sediment transporting channel are :

    i. Must be able to convey the required amount of water.

    ii. Must be able to transport the sediment load imposed.

    iii. Must not scour or silt either banks or the bed.

Therefore three relationships required for the above purpose

A - Flow equation.

B - A sediment transport equation

C - Bank erosion criterion.

4.5 RECOMMENDATIONS

The reasonable basis for a satisfactory design of stable channel with present stage of knowledge of pertinent literature considered by the author is as below:

4.5.1 For design of channels to convey essentially clear water in coarse non-cohesive materials the tractive-force method is the most suitable.

4.5.2 For the sediment laden channels the following procedure is probably the most suitable method.

As already mentioned, to satisfy a general requirements of a sediment transporting channel there must be three independent equation beside the continuity equation. These equations are : flow equation, bed load transport equation and bank competence criteria.

A procedure of design in this dissertation is recommended by use of these formulae:

I. Flow Equation (Mannings')

Because upto this time there is no satisfactory resistance or flow equation for sediment transporting channel, Manning's formula will serve the purpose suitably. The rugosity coefficient (n) must be carefully chosen.

$$V = \frac{1}{n} R^{2/3} S^{1/2} \text{ (in metric units).}$$

II. Sediment Transport Equation

A bed load transport equation assuming that the suspended concentration is within the carrying capacity of the channel with the help of turbulent energy contained in it is chosen as below.

Meyer - Peter equation reduced to metric units is suggested to calculate the sediment transport capability of channels:

$$q_s = 4{,}700 \left[\tau_b \left(\frac{n'}{n}\right)^{3/2} - \tau_{cr} \right]^{3/2}$$

where,

q_s = rate of bed load transport in kg/m/hour.

n' = Grain resistance calculated from Strickler formula

$$n' = \frac{d_{65}^{1/6}}{24}$$

d = diameter of sediment particle in metres.

n = Manning coefficient of roughness for the channel as a whole.

τ_b = Total tractive force intensity on the bed, and

τ_{cr} = Critical tractive force stress of the material in kg/m^2, equal to 0.07d (d in mm).

III. Bank Competence Criteria

For the bank erosion, a suitable check will be use of critical tractive stress at the bank which is given by Lane with combination of Lacey's P-Q relation with different constant. The stable cross section of a canal can be designed to conform the two above conditions and the stress at the bank must not exceed the limiting tractive force.

4.5.3 For design of canals with sediment concentration less than 500 ppm, the Simons and Albertson's method is suitable.

R E F E R E N C E S

1. Framji and Mahajan, Irrigation and Drainage in the World, International Commission on Irrigation and Drainage, Second Edition, Vol. I, New Delhi, India, 1969.

2. Israelsen O.W. and Harison, V.E., Irrigation Principles and Practices, John Wiley and Sons, Inc., New York , 1962.

3. Blench, T., Regime Behaviour of Canals and Rivers, Butter-worths Scientific Publication, London, 1957.

4. Lacey, G., Regime Flow in Incoherent Alluvial, Central Board of Irrigation and Power, India, Paper No. 20.

5. Kennedy R.G., " The Prevention of silting in Irrigation Canals" Minutes of Proceeding, Institution of Civil Engineers, Vol.CXIX(119),1895,pp281.

6. Buckley, R.C., " Irrigation Pocket Book" Spon, London 1928, pp.178.

7. Woods, E.W., " A new hydraulic Formula (For silting velo-city-Kennedy's data),The Engineer,Vol.143, 17.6.1927.

8. Singh, Bharat, "Fundamentals of Irrigation Engineering" Nem Chand and Bros,Roorkee,India,5th Ed,1972 pp 66.

9. Garrete, A.F.F., "Hydraulic Tables and Diagram for Practical Engineer".

10. Lindley, S.E., "Regime Channels", Proc. Punjab Eng. Congress, Vol.7, 1919.

11. Inglis Claude, "Historical note on Empirical Equations Developed by Engineers in India for Flow of Water and Sand in alluvial channels".Proc. Second Meeting, International Association of Hydraulics Res. Stockholm 1948.

12. Simons, Darley, B., and Miller,Carl, R.,"Sediment discharge in Irrigation Canals". Sixth Con . on Irrig. and Drainage. International Commission on Irrigation and Drainage, New Delhi,1966.

13. Lacey, G., " Stable Channels in Alluvium" Paper No.4736, Min.Proc. ICE, Vol.129,1929-30,pp.259.

14. Lacey, G., "Uniform Flow in Alluvial Rivers and Channels", Min.Proc.ICE,Vol.237,1933-34,pp.421.

15. Lacey, G., " Regime Diagram for the Canals and Distribution" Tech. Paper, U.P.P.W.D.,India (Irrig.Branch),No.1, 1932.

16. Lacey G, " A general theory of flow in Alluvium" Jl. Min.PICE,Vol.27,Nov.1946,pp-16.

17. Lacey,G.,"Flow in Alluvial Channels, with Sandy Mobile Bed",ICE,London,1958,Vol.9,pp145.

18. Lacey,G.,"Sediment as a Factor in the Design of Unlined Irrigation Canals", Sixth, Cong. on Irrigation and Drainage 1966(ICID)New Delhi,1966.

19. Lacey, G., " Discussion on Kinga River Deviation", Paper No. 6644, Min.Proc. ICE,1964.

20. Bose, N.K., "Silt Movement and Designing of Channels" Paper No. 192, Punjab Engg. Congress,1936.

21. Bose, N.K., and Malhotra, J.K., " An investigation of Interrelation of Silt Indices and Discharge Element for some Regime channels in Punjab" Research Publication Vol.II.No.23-1939.

22. Inglis Claude, "Divergence from Regime in Stable channels in Alluvium" Annual Report(Tech.), Central Irrigation and Hydraulic Research Station Poona (1940-41)

23. Inglis, C and Allen F.H., " The Regime of Thames Estuary as Affected by Currents, Salinities and River Flow" PICE,London, 1957.

24. Inglis C.,"Discussion of 'Uniform Water Conveyance in Alluvial material' by Simon and Albertson" Jr. of ASCE(Hydr).Jan, 1961,P-129.

25. Singh B., and Singhal, R.P., "Study of Self-formed channel on a Model Sand Bed"Irrigation and power Journal,Vol.126,No.4, Oct.1969.

26. Blench, T., "A new Theory of Turbulent Flow in Liquid of Small viscosity"Paper No.5185 Jl.ICE, April,1939.

27. Blench, T., and King, C."Effect of Dynamic Shape on Lacey Relations" Annual Report(Tech.)Central Board of Irrigation India, 1941.

28. King, C., "Practical Design Formulae for Stable Irrigation Channels",Ann.Report(Tech.)CBIP(India),1943.

29. Blench, T., "Turbulent Flow Theory from view Point of an Irrigation Engineer" Written for USBR, and Published in French in Houille Blanche, May, 1946.

30. Blench, T., " Canals, Channels and Rivers" Civil Engineering Reference Book (Edited by Probst and Comrie), 1951, London Butter-Worths.

31. Blench, T., "Regime Behaviour of Canals and Rivers", 1957 London Butterworths.

32. Blench, T. and Qureshi, M., " Practical Regime Analysis of River Slopes" Proc. ASCE(Hyd.) 1964.

33. Mathews, I.S.G., " Design of Stable Channel in Coherent Alluvium" IAHR, 1951.

34. White, C.M., "The influence of transported Solids upon Rivers" Proc. International Conference of Scientific Hydrology, Washington, 1939.

35. Singh, B., " Some Implication of Regime Design Channels", Sixth Congress on Irrigation and Drainage, ICID, New Delhi, 1966.

36. Marshal Nixon, "A study of the Bankful Discharge in England and Wales" Jr. ICE, Vol.12, Feb, 1955.

37. Pettis, C.R., Discussion Published in Trans.ASCE, Vol.120, 1937 on the paper No. 1957 "Stable Channels in Erodible, Material" by W.E.Lane in the same Trans. pp.123.

38. Leopold, L.B. and Maddock, T., " The Hydraulic Geometry of Stream channels and some Phisiographic Implications" U.S.Geol.Survey Professional Paper No.252, 1953.

39. Nedoco, "River Studies and Recommendations on the Improvement of Nigor and Bene North Holand" Publishing Comp. Amster dam (India) May, 1965, 1959, pp.510.

40. Simons, D.B. and Albertson, M.L., "Uniform Water Conveyance in Alluvial Material". ASCE(Hyd.) Div. 1960, Vol. 86, No.HY5, pp33 and Trans.ASCE(Hyd) Part I, 1963, pp.65.

41. Simons, D.B., and Richardson, E.V., "Resistance to Flow in Alluvial Channels" ASCE, Vol.86, No.HY5, 1960, pp39-73.

42. Simons, D.B. and Richardson, E.V., "Form of Bed Roughness in Alluvial Channels", ASCE, Proc. Vol.87, No.87, No.HY3, May 1961, pp.87-105.

43. Eanie,R.M. and Simons, D.B., "Design of Stable Channels in Alluvial Material" Trans. ASCE, Vol 94,1968,pp.1399.

44. Torrel, P.W., and Borland, W.M., "Design of Stable canals and Channels in Erodible Material" Trans.ASCE, Hyd. Div. Feb., 1956, p.1-17.

45. William J. " Correspondence on'Stable Channels in Alluvium' by Lacey" G. Paper No. 4736 Proc. of Inst. of Civil Engineer (PICE),Vol.229,1929-30,pp.259.

46. Scholitsch, Armin,"Hydraulic Structures",New York,American Society of Mechnnical, Engineers, 1937 Vol.2,pp.733.

47. Lane, E.W., "Progress report on the des gn of stable channels" by the USBR, Am.Soc.CivilEng. Proc.Vol.79, Separate no.280, p 1-31, 1953,(Irrigation and Drainage Division).

48. Lane, E.W. and Carls on E.J., " Some Factors Affecting the Stability of Canals constru ted in Coarse Granular Materials" Proc. of the Minnisota International Hyd. Convention Proc. IARG, 1953, Sept.1-4, 1953, Minneapolis, Minnisota, or ASCE(Hyd) pp.37-48 Aug, 1953.

49. Lane, E.W., " Progress Report on Results of Studies on Design of Stable channels" USBR,Hydr.Laboratory Report No.Hyd1-352, Denver Colorado June,1952.

50. Du Boys, P., " Le,Rhone et les River a Lit.Affouillable" Cited in Rouse",History of Hydraulics,P 172.

51. Leighly, J.B., "Toward a theory of the morphologic significance of Turbulence in the flow of Water in Stream" University of California, Publication in Geography, Vol. 6, No.1, pp1-22,Berkeley,1932.

52. Olsen O.J., " Florey Q.L.(Compilers),"Sedimentation Study in Open Channels" Boundary Shear and Velocity Distribution by Membrane analogy, analytical and Finite Difference method reviewed by D.McHency and R.E. Glover, USBR, Laboratory Report No.SP.34 Aug, 5, 1952.

53. Forchheimer, P."Hydraulik"(Hydraulics), Teubner Verlagsgesellschaff, Leipzig and Berlin, 1st ed., 1924, P.495, 3ded , 1930, P.551.

54. Chia-Hwa Fan, "A Study of Stable channel cross-section" (in Chinese),Hydraulic Engineering, Chinese, Society of Hydraulic Engineers, Vol.15,No.1, pp.71-79 ,Nanking, 1947.

55. Lane, E.W.,"Design of Stable Channels" ASCE Trans.Vol.120, 1955, pp.1234-1279.

56. Etchevory B.A."Irrigation Practice and Engineering" McGraw Hill Book Co., Inc., New York N.Y.,Vol.II, 1915, pp.57.

57. Fortier S., and Scobey, F.C., " Permissible Canal Velocity" ASCE, Trans., Vol.89, 1926.

58. "Standards Permissible Non Erodible Velocity".Bureau of Methodology of the Hydro Engrg Plan"Gidro Tekhnicheskoyo Stroitel Stro" Obedinennoo Nauchno-Tekhnicheskoo Izdatel'stvo,Moscow,USSR, May, 1936.

59. Glover R.E. and Q.L.Florey"Stable channel profiles" Hydraulic report No. Hyd -325,USBR,Dept.of Interior Washington, D.C.1951.

60. Henderson, M.F.,"Stability of Alluvial channels", Trans., Am.Soc. of Civil. Eng. Vol.248,1963,pp.657, Part I.

61. Liu H.K. and Hwang S.Y., "Discharge Formula for Straight Alluvial Channels", Proceeding, ASCE,Vol.95, No. 1959, P.65.

62. Ning Chien,"A Concept of Regime Theory" Trans.ASCE,1957, Vol.122-p.785.

63. Ning Chien,"A Concept of Lacey's regime theory",ASCE, Sept. 1955.

64. Langbein W.B., "Geometry of River Channels",Proc.ASCE, Hy2 March,1964,pp.301.

65. Ackers, Peter,"Experiments on Small Streams" Proc. ASCE, 1964.

66. Leliavsky, Serge, "An introduction to Fluvial hydraulics" London, Constable and Company.