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A discussion of the considerations required for the design of sediment transporting channels as components of a branching canal system. Attention is given to the selection of compatible approaches for the determination of channel geometry and bed material transport capacity to be used in conjunction with sediment routing relations. A generalized procedure is presented which includes sediment equilibrium considerations as a part of the system design criteria. The development of methods currently in use for the design of individual channels in erodible material is reviewed as are various computational techniques for the estimation of bed material transport capacity. In general, no specific approach or computational technique may be considered best for all applications due to the large number of variables involved, the complexity of their inter-relation, and the variability of field conditions. Emphasis therefore is placed on the concepts, assumptions, and data on which a specific method is based, rather than on the mechanics of its application.

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THESIS

SEDIMENT STABLE CANAL SYSTEMS

Submitted by

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WE HEREBY RECOMMEND THAT THE THESIS PREPARED UNDER OUR SUPERVISION

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ABSTRACT OF THESIS

SEDIMENT STABLE CANAL SYSTEMS

The considerations required for the design of sediment transporting channels as components of a branching canal system are discussed. Attention is given to the selection of compatible approaches for the determination of channel geometry and bed material transport capacity to be used in conjunction with sediment routing relations. A generalized procedure is presented which includes sediment equilibrium considerations as a part of the system design criteria.

The development of methods currently in use for the design of individual channels in erodible material is reviewed as are various computational techniques for the estimation of bed material transport capacity. In general, no specific approach or computational technique may be considered best for all application due to the large number of variables involved, the complexity of their interrelation, and the variability of field conditions. Emphasis is therefore placed on the concepts, assumptions, and data on which a specific method is based rather than on the mechanics of its application.

Relations describing the requirements for sediment equilibrium within a branching irrigation canal system or subsystem are presented and their implications with respect to individual channel design are discussed. In general, equilibrium considerations will require that relatively higher sediment concentrations be allocated to diversions from larger channels of the system if sediment removal by bed clearance is to be minimized or eliminated.

Successful application of any of the techniques discussed is dependent on the proper evaluation of field conditions unique to the specific situation and on consideration of each component in relation to the overall system.

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF FIGURES (continued)

LIST OF SYMBOLS

Symbol	Definition
A	Area of channel cross section
B	Representative channel width
C	Chezy discharge coefficient, empirical constant, or sediment concentration (defined as used)
C_b	Mean concentration parameter (Mahmood's transport relations)
C_R	Ratio of bed material concentration in specific channel or turnout to concentration at the head of the system
C_{RAT}	Ratio of bed material concentration in branch channel or turnout to concentration in the parent channel
D	Channel depth
ΔD	Depth correction to smooth or grain rough channel
d_m	Geometric mean diameter of bed material
d_i	Diameter of bed material for which i percent by weight if finer
F_b	Blench bed factor
F_s	Blench side factor
f	Lacey silt factor
f_r	Lacey silt factor related to hydraulic radius
f_s	Lacey silt factor related to slope
G	Channel bed material transport capacity
G_c	Average rate of bed material removal by mechanical means
g	Gravitational constant
g_c	Percent of bed material in flow removed from the system by mechanical means
K	Tractive force correction for side slope
K_i	Empirical coefficient, defined as used (i = variable subscript)
K_s	Roughness coefficient ($K_s = 1/n$)

Symbol	Definition
K_r	Roughness coefficient related to grain roughness
n	Manning roughness coefficient
P	Channel wetted perimeter
Q	Volumetric water discharge
q	Volumetric water discharge per unit width
q_T	Total bed material discharge per unit width
q_s'	Bed load discharge weighted under water
R	Channel hydraulic radius
R'	Hydraulic radius associated with grain roughness
ΔR	Hydraulic radius correction to smooth or grain rough channel with equal volume of flow
$\mathbf{R}$	Reynolds number $(\frac{VD}{\nu}$ or $\frac{VR}{\nu})$
$\mathbf{R}$	Reynolds number with respect to shear $(\frac{V_* D}{\nu}$ or $\frac{V_* R}{\nu})$
S	Slope of energy grade line
S_h	Shields parameter $(\rho\ V_*^2/(\gamma_s-\gamma)d_{50})$
T	Water temperature
U_{*e}	Effective shear velocity
u	Local point velocity
V	Average velocity
V_*	Shear velocity $(\sqrt{gDS}$ or $\sqrt{gRS}$
ΔV	Velocity correction to smooth or grain rough channel with equal volume of flow
W	Channel width
x	Distance in direction of flow
y	Vertical distance from channel bed
z	Exponent describing suspended sediment distribution

Symbol	Definition
α	Exponent describing variation of bed material size with distance
γ	Specific weight of water sediment complex
γ_s	Specific weight of sediment particle
γ'	Bouyant specific weight of sediment particle $(\gamma_s - \gamma)$
η	Dimensionless depth (y/D)
ν	Kinematic viscosity
θ	Angle of side slope with horizontal
ρ	Density of water sediment complex
ρ_s	Density of sediment material
τ_B	Maximum time average tractive force on channel bed
τ_c	Critical tractive force
τ_o	Average tractive force on channel perimeter
τ_s	Maximum time average tractive force on channel side slope
ϕ_*	Sediment transport intensity parameter
ϕ	Angle of repose of granular material
ψ_*	Particle shear intensity factor
ω	Particle terminal fall velocity

CHAPTER I

INTRODUCTION

The problem of sediment routing within a canal system becomes a design consideration whenever material of a size capable of being transported by the flow is available to the flow field. This material may be introduced into the system at the headworks, eroded from channel boundaries within the system, or both. The basic principle to be satisfied for system equilibrium and therefore channel stability is that of mass continuity, which requires that in each segment of the system the sediment inflow over time equal the outflow.

In plan view, an irrigation canal network for distribution of water to the land closely resembles a natural system of streams and rivers with the flow direction reversed. Viewing a canal system in this fashion, as a reversal of nature's water collection system, can provide a qualitative model useful in understanding the difficulties involved in satisfying the requirements of continuity with respect to sediment, since in the natural system, a relative state of equilibrium has been reached over geologic time.

In natural river systems, small streams flow down relatively steep slopes converging into larger streams having smaller energy gradients, giving the system a generally concave profile [58][1] as illustrated in Fig. 1.1. Water lost to seepage, evaporation, etc., causes the sediment concentration to increase in the downstream direction. The general result is then an increase in concentration of sediment with increasing discharge.

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[1]Numbers in brackets refer to items in the bibliography.

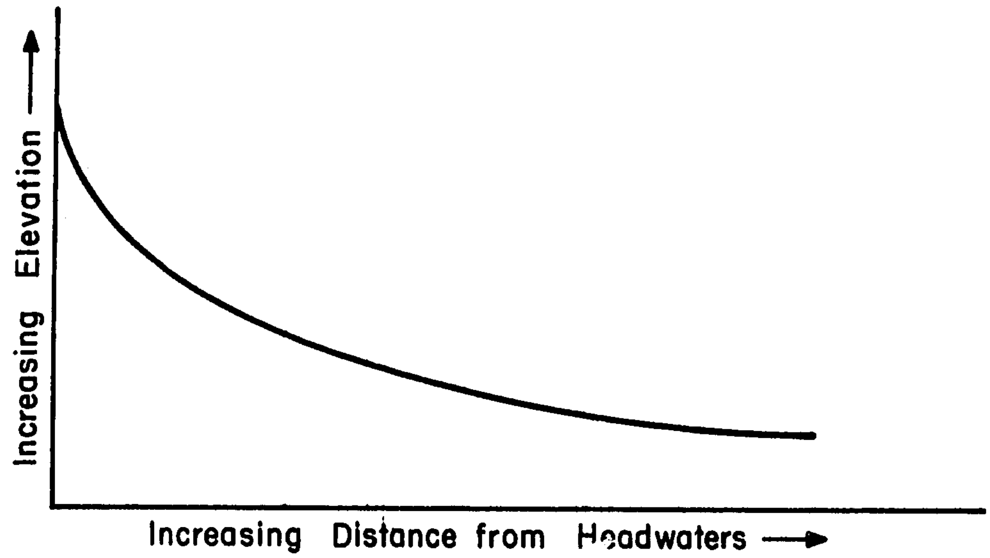

Figure 1.1. Qualitative Representation of River System Profile.

In man-made water distribution systems, however, the capability for slope variation may be limited, with the maximum available slope often being less than the slope of the parent channel at the canal headworks. Smaller individual channel discharges associated with branching of the system for distribution purposes result in a significant decrease in the total sediment transport capacity of a constant slope canal network as illustrated in Fig. 1.2. Examination of this figure shows that, with all variables other than individual channel width and discharge held constant, a single bifurcation of a channel into two channels of equal discharge results in a decrease in sediment transport capacity on the order of twenty percent. A system with eight channels of equal discharge has a sediment transport capacity less than half that of a single channel having the same slope, bed material, and total discharge. In addition, losses due to seepage and evaporation tend to increase sediment concentrations as water discharge is decreased

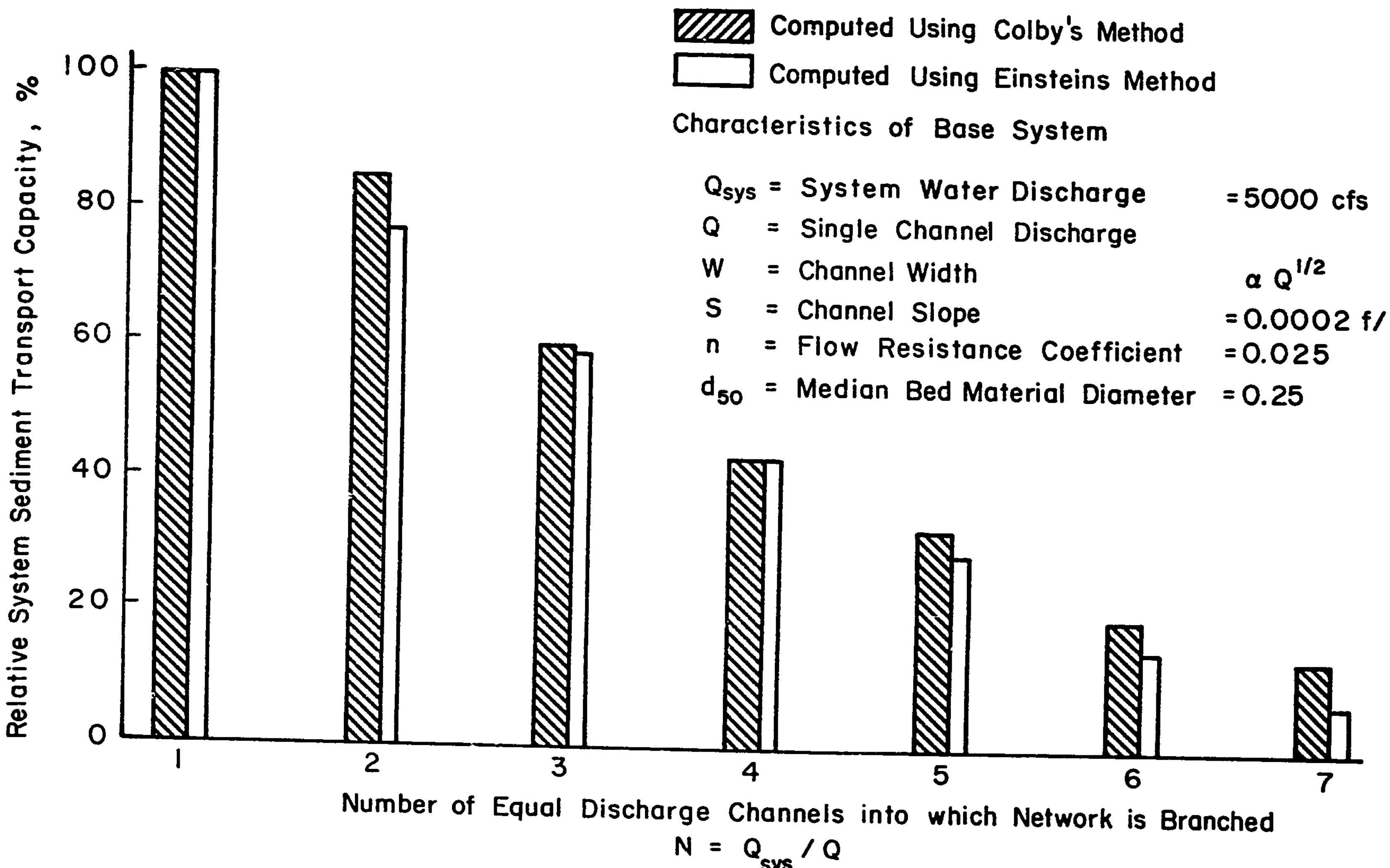

Figure 1.2 Relative Sediment Transport Capacity of Branching Canal Network.

in the downstream direction. This results in a tendency for sediment to be deposited in the middle and lower reaches of the system as individual channel discharges are reduced through branching. This has been found to be a serious problem in many existing irrigation canal systems [67].

Approaching a solution to this problem requires that each portion of the canal network be considered in relation to the overall system. An understanding of the implications of the geometric variations observed in the natural system is also required so that proper geometric design of the channels may be combined with headworks and bifurcation design for optimal routing of the sediment through the system. Although much can be learned from study of the parent system, it would in most cases be infeasible and ineffective to attempt to merely duplicate the geometry of the parent system in reverse.

The purpose of this study is to review the various approaches available to the engineer faced with the problem of canal system design and to relate these to the problem of sediment routing within the system. Since many of the physical laws related to the interaction of flow with erodible boundary materials are only imperfectly understood, most of the techniques presented are semiempirical. For this reason, the emphasis throughout the paper is placed on understanding the assumptions and data on which an approach is based rather than on quantitative design procedures. After selecting the approach most applicable to his particular problem, the engineer may wish to refer to items in the reference list for a more complete presentation of the computational technique.

Chapter II, on stable channel design techniques, follows the historical development of these techniques as applied to individual channels. In general, terms are defined and concepts discussed where they are first encountered. Following the historical presentation is a discussion of the various approaches with the attempt made to show both their interrelation and the area of most direct applicability of each.

The greatest portion of the material presented is applicable to straight channels with erodible boundaries. References are provided which deal with the more specific problems associated with such items as seepage, channel curvature, structural encroachment on the channel, etc.; however, these items are not dealt with directly in the text to any significant extent.

Chapter III reviews briefly the basic concepts and approaches to computation of the sediment transport capacity of a channel. Since a presentation of all computational techniques available in this area would be impractical due to space considerations, specific techniques are selected to represent each of the identified approaches to the problem. It is not intended to imply that those presented are the only techniques available. Indeed, the engineer may be familiar with other computational techniques more directly applicable to his specific problem. The attempt is made, however, to present techniques representative of the primary approaches to sediment transport computations, and it is believed that those discussed are as dependable for general usage as any presently available. Chitale [14] and Bogardi [9] present more thorough reviews of techniques currently in use.

It must be emphasized that due to their empirical nature and the large number of interrelated variables involved, the success of any of the techniques presented in Chapters II and III is dependent on the judgment and ability of the design engineer. Simons [90] describes the design of erodible channels as "...still something of an art, and for this reason, the engineer as an artisan plays a most important role." An understanding of the principles underlying specific computational techniques is therefore essential.

Chapter IV addresses itself to application of the technology outlined in Chapters II and III to the problem of sediment routing within a canal system. Existing models for solution of the problem are presented and briefly discussed. Procedures are outlined for application of the concepts in the design situation and for analysis of existing systems.

In the overall design, not only the channels within the system must be considered, but also the characteristics of the parent channel, the nature of bifurcation and turnout structures, and the use of sediment exclusion and/or ejection devices in headworks design. Again, logical limitations on space and scope prevent complete discussion of each of these facets of design, and the reader is referred to pertinent items in the reference list. Melone, Richardson, and Simons [74] provide a recent review of sediment exclusion and ejection techniques, and Mahmood [67] presents a model for use in turnout design. These references, as well as others, are referred to later in the text.

As was previously indicated, the concepts are generally presented in the context of application to channels constructed through noncohesive materials. It should be realized, however, that sediment routing may also need to be considered in lined or cohesive boundary channels if the flow introduced into the system carries with it significant quantities of sand size materials. Deposition of these materials on the channel bed may result in a large increase in flow resistance with a corresponding decrease in channel capacity. This may occur even when sediment exclusion or ejection devices are utilized. The concepts used in routing sediment through a system of this type are the same as those applied to the erodible boundary system.

CHAPTER II

REVIEW OF STABLE CHANNEL DESIGN TECHNIQUES

Problems related to the interaction of a flow field with erodible boundary materials involve the complex interrelation of a large number of variables. Current levels of understanding regarding these relations are the result of a logical evolution of ideas as the knowledge base was expanded through observation and experimentation. Computational methods presently in use represent the combined efforts of a large number of engineers and scientists over time, but must be considered as still in the evolutionary stage since many related physical laws are still only imperfectly understood. Ongoing research in the areas of hydraulics and fluid mechanics continues to contribute to the knowledge base.

As a means of placing in proper context the concepts and approaches to stable channel design, the major points of development during the last century are briefly reviewed. The chapter is divided into two parts. The first portion presents the historical development of the techniques currently in design usage. Although the material in this portion is subdivided according to the general approach represented, an attempt is also made to follow the chronological development of the concepts. The second portion of the chapter consists of a discussion of the approaches to clarify their interrelation and areas of most direct applicability.

HISTORY

Throughout most of the 1800's, canals were designed by selecting the geometry more or less arbitrarily and computing the capacity using a formula credited to Chezy in 1775.

$$V = C\sqrt{RS} \tag{2.1}$$

in which V is the average velocity, R is the hydraulic radius, S is the channel slope, and C is an empirical constant depending upon the nature of the channel boundary. The value of Chezy's C for various materials and conditions has been tabulated by numerous engineers, and the formula still finds some use today.

In 1870, Kutter and Ganguillet [29] proposed a formula for computing the value of C in the Chezy equation as:

$$C = \frac{41.65 + \frac{0.00281}{S} + \frac{1.811}{n}}{1 + \frac{n}{\sqrt{R}}\left(41.65 + \frac{0.00281}{S}\right)} \tag{2.2}$$

where n is a roughness coefficient, and the other variables are as previously defined.

In 1890, Manning [72] suggested the flow equation which, in its present usage, takes the form:

$$V = \frac{1.486}{n} R^{2/3} S^{1/2} \tag{2.3}$$

where n is Kutter's roughness coefficient and the constant 1.486 is a conversion factor for the English system of units. The Manning equation is probably the most widely used flow equation in the U.S. at the present time, but may be applied to the case of a mobile boundary only with great care for reasons which will be discussed.

It is of interest to note that both the Chezy and Manning equations may be related to the dynamics of the flow after being reduced to dimensionless form through the use of the near constant terms of gravitational acceleration and kinematic viscosity. In the case of the Chezy equation:

$$C/\sqrt{g} = V/V_* = \frac{V}{\sqrt{\tau_o/\rho}} \tag{2.4}$$

and in the case of the Manning equation:

$$\left(\frac{V}{V_*}\right)^2 \propto \left(\frac{VR}{\nu}\right)^{-2/9} \left(\frac{V^2}{gR}\right)^{1/9} \tag{2.5}$$

in which:

V_* = shear velocity.

τ_o = average boundary shear.

ρ = mass density of the fluid.

ν = kinematic viscosity of the fluid.

VR/ν = Reynolds number in terms of hydraulic radius.

$(V^2/gR)^{1/2}$ = Froude number in terms of hydraulic radius.

Since the time of Manning, attempts to refine the flow formulae have met with only limited success. More recent work in this area includes that of Liu and Hwang [60] and that of Mahmood [69]. Whereas the Manning and Chezy coefficients were originally assumed constant for a given channel, these later relations recognized the variation in flow resistance with discharge resulting from the interaction of the flow with the boundary.

Mahmood's [69] resistance function was developed in conjunction with his sediment transport relation and is dependent on advances made in the 1960's with respect to bed form prediction. Discussion of the approach is therefore placed in Chapter III with Mahmood's sediment transport relations. In general, use of the method requires a relatively large number of computations, but also provides an estimate of velocity distribution as well as average velocity for sand bed channels.

The approach of Liu and Hwang was empirical since, as they stated, "At present, there is no satisfactory theory of turbulent flow available."

They arrived at the general equation:

$$V = C_1 R^x S^y \tag{2.6}$$

Guides were given for the selection of the coefficient and exponents. In functional form, these were defined as:

$$C_1 = f(D, \rho_s, \rho, g, \nu, \Omega) \tag{2.7a}$$

$$x \;\&\; y = \text{pure numbers} = f(\Omega, D) \tag{2.7b}$$

where

D = depth of flow.

ρ_s = density of cohesionless bed material.

ρ = fluid density.

g = gravitational constant.

ν = kinematic viscosity.

Ω = parameter describing bed form.

It may be noted that the general equation is of the same form as both the Chezy and Manning equations, with the exponents allowed to vary. In this way, account is made for the changes in turbulent structure of the flow due to changes in boundary configuration and sediment transport. To fully treat the development of these and other flow formulae would be beyond the scope of this paper. The reader is therefore referred to items in the bibliography for more comprehensive treatment.

Critical Velocities

For use in conjunction with the flow formulae in the design of stable channels, engineers developed tabulations of critical or permissible velocities based on experience and observation. These velocities were assumed to be those at which canals in a given material would operate without excessive scour or deposition.

Perhaps the earliest published values of permissible velocities were those of DuBuat in 1786 [21]. Utilizing data from flume measurements, he gave scouring velocities for various materials from potter's clay to flint the size of an egg or larger. As more experience was gained, more listings were published, two of the more notable of these being the works of Etcheverry in 1915 [25], and Fortier and Scobey in 1926 [28]. These later works followed the initial developments in regime theory, and included not only critical velocities according to the type of material, but also guidelines for adjusting tabulated values according to flow depth. As late as 1936, the USSR introduced guidelines for the design of channels in granular material which listed permissible velocities and a table of correction factors according to depth of flow.

Although the critical velocity approach is seldom used directly in present day design of larger canal systems, the tabulated data can provide valuable information on the expected behavior of various materials when subjected to flowing water. The prime reason for the continued value of these tabulations is the extensive experience base on which they were developed. Table 2.1, for example, reproduces the recommendations of Fortier and Scobey and is based on answers to a questionnaire sent to a number of practicing engineers and therefore

Table 2.1. Permissible Canal Velocities (after Fortier and Scobey [28]).

Original material excavated for canal	Velocity, in Feet per Second, After Aging, of Canals Carrying:		
	Clear water, no detritus	Water transporting colloidal silts	Water transporting non-colloidal silts, sands, gravels, or rock fragments
(1)	(2)	(3)	(4)
Fine sand (non-colloidal)	1.50	2.50	1.50
Sandy loam (non-colloidal)	1.75	2.50	2.00
Silt loam (non-colloidal)	2.00	3.00	2.00
Alluvial silts when non-colloidal	2.00	3.50	2.00
Ordinary firm loam	2.50	3.50	2.25
Volcanic ash	2.50	3.50	2.00
Fine gravel	2.50	5.00	3.75
Stiff clay (very colloidal)	3.75	5.00	3.00
Graded, loam to cobbles, when non-colloidal	3.75	5.00	5.00
Alluvial silts when colloidal	3.75	5.00	3.00
Graded, silt to cobbles, when colloidal	4.00	5.50	5.00
Coarse gravel (non-colloidal)	4.00	6.00	6.50
Cobbles and shingles	5.00	5.50	6.50
Shales and hard-pans	6.00	6.00	5.00

represents a composite of their experience. The values given are for clear water flowing at a depth of three feet or less in a straight channel. Lane [52] used these and other critical velocity tabulations in arriving at recommended limiting tractive force values in the early 1950's.

Regime Theory

Regime theory may be thought of as the process of taking data from real alluvial channels observed to be "in regime" and applying it to the development of ideal channels. A channel is said to be in regime if it tends to neither aggrade nor degrade over time periods which include all water and sediment discharges the channel may be reasonably expected to experience. Blench [4] compares the concept of regime to that of climate in that at any point in time or space, the channel may experience deposition or scour and slow changes may occur over time, but a relative equilibrium condition may be conceived. Perfect regime in which no erosion or deposition takes place in either time or space does not exist in real channels, but is the design base for the ideal channel carrying the formative or dominant discharge of water and sediment of the real channel.

Regime theory began to develop in India when the use of flow formulas and critical velocity tabulations proved to be unsatisfactory for design problems encountered. In 1895, R. G. Kennedy [43] proposed a formula relating the depth of flow with the velocity which would neither silt nor scour. This equation took the form:

$$V = C\,D^{n} \tag{2.8}$$

and is credited as being the beginning of the regime approach to the design of stable channels. In Kennedy's original equation, C and n took on the values of 0.84 and 0.64 respectively. The values of both the coefficient and exponent were subsequently computed for many of the canals then in operation in India and found to vary over a broad range.

Lindley (1919) [59] introduced width as a variable and, using the Kutter formula for velocity computations, gave the relations:

$$V = 0.95\ D^{0.57} \tag{2.9a}$$

$$V = 0.59\ B^{0.355} \tag{2.9b}$$

and the dependent relation

$$B = 3.80\ D^{1.61} \tag{2.9c}$$

These relations implied that there was a natural combination of width (B) and depth (D) which a channel must attain for stability at a given velocity.

Building on these ideas, Lacey, beginning in 1929, presented a complete set of regime equations for the design of unlined channels in "incoherent alluvium" [48,49,50]. These relations were based on data collected from successfully operating canals in the Punjab (India), and may be summarized as:

$$V = 1.16\ \sqrt{fR} \tag{2.10a}$$

$$P = 2.67\ Q^{1/2} \tag{2.10b}$$

$$S = 370.68 \times 10^{-6}\ f^{3/2}\ R^{-1/2} \tag{2.10c}$$

where f is a silt factor assumed to be primarily dependent on the nature of the boundary material. Lacey proposed the relation:

$$f = 1.76 \sqrt{d_{50}} \tag{2.10d}$$

as a "rough" guide in the selection of the silt factor where d_{50} is the median size of the bed material in millimeters. This was not claimed by Lacey to be an exact relation. Experience and comparison with similar systems was the main criteria to be used in the selection of a silt factor for design.

In 1939, N. K. Bose [10] published the results of a study performed for the Punjab Irrigation Research Institute, India, in which he presented relations similar to those of Lacey, but without the use of the silt factor. These relations may be summarized as:

$$P = 2.8\ Q^{1/2} \tag{2.11a}$$

$$V = 1.12\ R^{1/2} \tag{2.11b}$$

$$R = 0.47\ Q^{1/3} \tag{2.11c}$$

$$S = 2.09\ d_{50}^{0.86}/(1000\ Q^{0.21}) \tag{2.11d}$$

Bose, however, placed the following limitations on these relationships.

1) They apply to particular channels in the Punjab. No data have been examined for channels outside the Punjab, and therefore it is not claimed that they are applicable beyond the canal systems from which they have been derived.

2) They apply to channels in which the bed silt lies between 0.075 and 0.6 mm in diameter.

3) The entire inquiry is dependent on the method adopted for securing samples of the bed silt, and the analysis of this silt.

Blench and King (1941) [8] studied Lacey's work, and broke Lacey's silt factor into two parts f_s and f_r. f_s was based on Lacey's slope equation:

$$S = f_s^{5/3}/(1788\ Q^{1/6}) \tag{2.12a}$$

and f_r on Lacey's turbulence factor:

$$f_r = 0.75\ V^2/R \tag{2.12b}$$

They pointed out that Lacey's silt factor was the square root of the product of these two factors.

In an Addendum to this paper, Blench modeled an idealized regime channel as having coherent banks and incoherent bed. From this model, he introduced a bed factor and a side factor for use in equations similar to those of Lacey. Blench's equations, as summarized in 1969 [4] for a "trifling bed load charge," are:

$$B = \sqrt{F_b\ Q/F_s} \tag{2.13a}$$

$$D = \sqrt[3]{F_s\ Q/F_b^2} \tag{2.13b}$$

$$\frac{V^2}{gDS} = 3.63\left(\frac{VB}{\nu}\right)^{1/4} \tag{2.13c}$$

where

$$F_b = V^2/D = \text{Blench bed factor} \tag{2.13d}$$

$$F_s = V^3/B = \text{Blench side factor} \tag{2.13e}$$

D = mean channel depth

B = mean bed width.

The physical significance of these relations is summarized by Blench [4] to be:

1) Channels with the same water sediment complex tend to acquire the same Froude number in terms of a suitable depth.
2) The erosive attack on sides that behave as if hydraulically smooth can be measured in terms of the well known criterion $\rho\mu V^3/B$ where V is mean velocity of flow, μ is dynamic viscosity, and B is a suitable breadth.
3) Channels with the same water sediment complex and the same measure of erosive attack on sides, tend to adjust to the same dissipation of energy per unit mass per unit time.

In his 1957 and later publications [4,5], Blench introduced the term C (bed load charge in parts per hundred thousand by weight) into his relations to account for varying sediment transport requirements. Including this term, Eq. (2.13c) becomes:

$$V^2/gDS = 3.63(1 + C/233)(VB/\nu)^{1/4} \tag{2.13f}$$

and the bed factor is modified according to the relation:

$$F_b = F_{bo}(1 + 0.12\ C) \tag{2.13g}$$

in which F_{bo} is the bed factor for an equivalent channel transporting a "vanishingly small" bed load charge.

Sir Claude Inglis [39] had previously, in 1948, proposed a set of regime equations which included a term for "silt charge" based on data from the Lower Chenab Canal System (India). These relations, referred to as the Inglis-Lacey equations, were given as:

$$B = K_1 \frac{Q^{1/2}}{g^{1/3}\ \nu^{1/12}} \left[\frac{C\omega}{d_{50}}\right]^{1/4} \tag{2.14a}$$

$$A = K_2 \frac{\nu^{1/36}\ Q^{5/6}}{g^{7/18} (C\omega d_{50})^{1/12}} \tag{2.14b}$$

$$V = K_3 \frac{g^{7/18}}{\nu^{1/36}} Q^{1/6}\ (C\omega d_{50})^{1/12} \tag{2.14c}$$

$$D = K_4 \frac{\nu^{1/9}\ Q^{1/3}\ d_{50}^{1/6}}{g^{1/18}\ (C\omega)^{1/3}} \tag{2.14d}$$

$$S = K_5 \frac{(C\omega d_{50})^{5/12}}{\nu^{5/36} g^{1/18}\ Q^{1/6}} \tag{2.14e}$$

In these relations, K_i represents a constant coefficient, ω is the representative sediment fall velocity, and the other terms are as previously defined. Since the coefficients in these equations were never defined, they were not used much in practice. Inglis did however, suggest that the relations could be used in the form of ratios by utilizing measurements on similar systems[38].

Simons and Albertson, in 1963 [95], expanded the scope of the regime type relations by incorporating additional data and separating the channels into five classifications by type of bed and bank material. The effect of making these divisions was to incorporate to some extent the equivalent of a bed and side factor as used by Blench directly into the relations, and eliminate some of the uncertainty involved in their estimation. These relations were summarized by Henderson in 1966 [36] as:

$$P = K_1 Q^{1/2} \tag{2.15a}$$

$$b = 0.9 P \tag{2.15b}$$

$$b = 0.92 B - 2.0 \tag{2.15c}$$

$$R = K_2 Q^{0.36} \tag{2.15d}$$

$$y = 1.21 R \quad \text{for} \quad R \leq 7 \text{ ft} \tag{2.15e}$$

$$y = 2 + 0.93 R \quad \text{for} \quad R > 7 \text{ ft} \tag{2.15f}$$

$$V = K_3 (R^2 S)^m \tag{2.15g}$$

$$C^2/g = V^2/gys = K_4 \left(\frac{Vb}{\nu}\right)^{0.37} \tag{2.15h}$$

In these relations, b = mean width, B = surface width, y = depth, C = Chezy discharge coefficient, P, Q, R, S, V, ν, and g are as previously defined, and the coefficients, K_i, and exponent, m, are given in Tables 2.2 and 2.3 for the English system of units.

Table 2.2. Channel Types According to Bed and Bank Material (after Simons and Albertson [95]).

1.	Sand bed and banks
2.	Sand bed and cohesive banks
3.	Cohesive bed and banks
4.	Coarse noncohesive material
5.	Same as for 2, but with heavy sediment loads, 2000-8000 ppm

Table 2.3. Coefficients and Exponents for Eqs. 15 (after Henderson [36]).

Coefficient	Channel Type (from Table 2.2) 1	2	3	4	5
K_1	3.5	2.6	2.2	1.75	1.7
K_2	0.52	0.44	0.37	0.23	0.34
K_3	13.9	16.0	--	17.9	16.0
K_4	0.33	0.54	0.87	--	--
m	0.33	0.33	--	0.29	0.29

Tractive Force Approach

Through work done primarily in the early 1950's, E. W. Lane [51-55] approached the problem of stable channel design from the concept of tractive force. Tractive force is defined as being that force which is exerted on the periphery of the channel due to the motion of the fluid. Although, as noted by Lane, the idea of tractive force was not new and could be traced to duBoys in 1879 [20], Lane's work appears to be the most significant in developing the concept into a rational design procedure.

Lane classed unstable channels in three categories, and concentrated his efforts on the first. The classes of instabilities given by Lane are:

1) Channels in which bed and banks are scoured without objectionable deposits being formed;

2) Channels where objectionable sediment deposits occur without scour being produced;

3) Channels in which scour and objectionable deposits are both present.

In arriving at an effective value of tractive force for use in design, the average tractive force distribution over the channel perimeter was computed for trapezoidal channels using a membrane analogy. Maximum values of tractive force occurring on the bed and side slopes were expressed as ratios to the average tractive force exerted on the bed of an infinitely wide channel having the same depth, and plotted as functions of width to depth ratio and side slope (Fig. 2.1). The values of tractive force for use in design then became:

$$\tau_B = C_1 \gamma DS \tag{2.16a}$$

$$\tau_s = C_2 \gamma DS \tag{2.16b}$$

where

τ_B = maximum time average tractive force on channel bed.

τ_s = maximum time average tractive force on channel side.

γDS = time average tractive force on bed of wide channel.

C_1 and C_2 = coefficients from Fig. 2.1.

Lane found that for channels normally used in design, the values of C_1 and C_2 tended to 1.0 and 0.76 respectively.

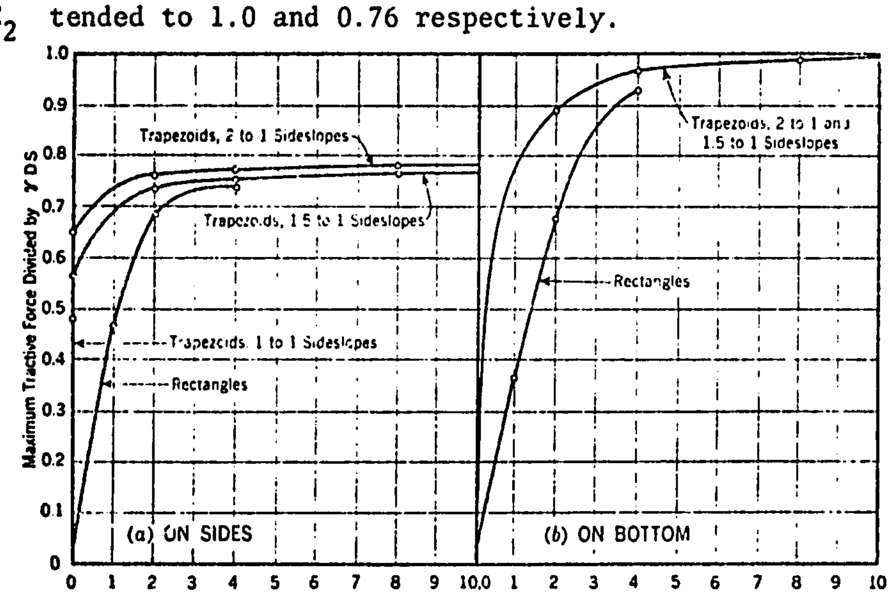

Figure 2.1. Maximum Tractive Forces on Channel Boundaries (after Lane, 1955 [52]).

From field data and published tables of critical velocities, Lane determined a relation relating critical tractive force on the bed to bed material size for coarse noncohesive material (d_{50} greater than 5 mm). This relation took the form:

$$\tau_c = C_3 \, d_{75} \tag{2.16c}$$

In this relation, τ_c = critical tractive force which material on the channel bed may withstand and d_{75} = diameter of bed material in inches for which 75 percent by weight is finer. The value of the coefficient C_3 was determined to be approximately equal to 0.5, but a value of 0.4 was recommended for use in design. These values of the coefficient were determined for material having a specific gravity of 2.56, but may be corrected by the ratio of specific gravities for other materials.

To determine a value of critical shear applied to the side slope, a force balance on the particle was made considering the effects of drag and gravity. The correction factor derived using this force balance is given as:

$$K = \cos\theta \sqrt{1 - \frac{\tan^2\theta}{\tan^2\phi}} \tag{2.16d}$$

with the resulting critical tractive force on the side slope given by:

$$\tau_{cs} = K \, \tau_c \tag{2.16e}$$

where

τ_{cs} = critical tractive force on channel bank.

θ = angle of side slope with horizontal.

ϕ = angle of repose of bank material.

For use in the preceding relations, Lane recommended a maximum value for ϕ of 41° for very angular material, and 39° for rounded material. Figure 2.2 developed by Simons and Albertson [95] may serve as a guide in estimation of the angle of repose for granular materials.

In computing depth for use in the tractive force relations, Lane used Manning's equation and gave three relations which could be used in the estimation of the roughness coefficient n.

$$n = \frac{d_{50}^{1/6}}{44.4} \tag{2.17a}$$

$$n = \frac{d_{75}^{1/6}}{39} \tag{2.17b}$$

$$26n = \left(\frac{d_{65}}{R}\right)^{1/6} \tag{2.17c}$$

d_{50}, d_{75}, and d_{65} are the bed material diameters for which 50 percent, 75 percent, and 65 percent by weight of the material is finer. It is noted that relation (2.17b) is applicable only to coarse noncohesive material. Which, if any, of these relations is used is left to the discretion of the individual engineer.

In Lane's consideration of forces acting on an individual particle, lift is accounted for only in that the parameters governing the magnitude of the lift force are the same as those governing the drag force. Simons [90] presents a similar analysis for use in the sizing of riprap, in which lift is considered separately and the angle of the flow with the horizontal is added as a variable. Using the Meyer-Peter criteria for the initiation of motion, four simultaneous equations are developed which may be solved for the factor of safety against side slope failure.

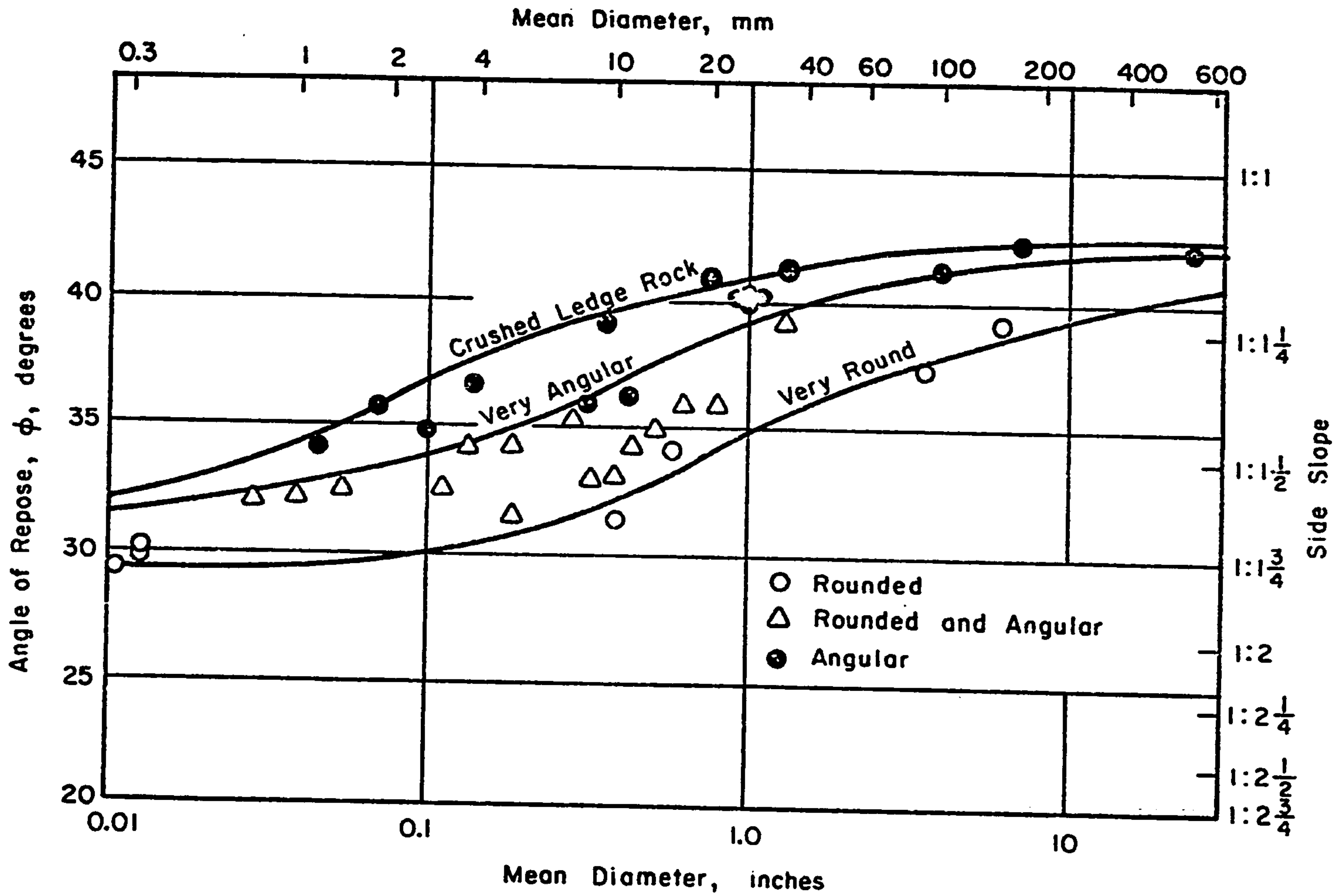

Figure 2.2. Angle of Repose of Noncohesive Material (after Simons and Albertson, 1963 [95]).

$$S = \frac{\cos\theta \tan\phi}{\eta'\tan\phi+\sin\theta\cos\beta} \tag{2.18a}$$

$$\beta = \tan^{-1}\left\{\frac{\cos\lambda}{\frac{2\sin\theta}{\eta\tan\phi}+\sin\lambda}\right\} \tag{2.18b}$$

$$\eta = \frac{21\ \tau_s}{(S_s-1)\gamma d} \tag{2.18c}$$

$$\eta' = \eta\left\{\frac{1+\sin(\lambda+\beta)}{2}\right\} \tag{2.18d}$$

in which

S = factor of safety against erosion

θ = side slope angle with the horizontal

ϕ = angle of repose of bank material

λ = angle of velocity field with horizontal

τ_s = tractive force produced by velocity field

S_s = specific gravity of bank material

In general, when applied to coarse noncohesive material on a side slope, Simons relations will lead to a more conservative design than will Lane's method. It is worthwhile to note, however, that Eqs. (2.18) were designed to preclude all motion of the bank material, whereas Lane's criteria is that of channel stability in homogeneous material and does not necessarily imply the absence of all boundary motion.

Lane, in his 1955 paper [52], also gave tentative recommendations for limiting tractive forces applicable to fine noncohesive and cohesive materials. These recommendations were based primarily on published tabulations of critical velocities whereas the values derived for coarse noncohesive materials included significant field data from the canals of the San Luis Valley.

Recent Advances in Channel Design Criteria

During the late '50's and '60's, the knowledge base regarding flow in alluvial channels was extended through continuing research in several areas. One of these areas was the study of bed forms generated in sand bed channels and their effect on flow characteristics. The brief outline which follows is based on the work of Simons and Richardson [92,94].

Simons and Richardson divided the flow in sand bed channels into two regimes connected by a transition phase. In the lower flow regime, resistance to flow is large and sediment transport capacity small, while in the upper flow regime, resistance is comparatively small and transport capacity large.

The lower flow regime consists of three phases. Given in order of increasing stream power (γDSV or $\tau_o V$), these are:

1) Plane bed without sediment motion (i.e., flow with insufficient energy or tractive force to initiate motion of the bed material).
2) Ripple phase in which the bed forms are triangular with heights of less than 0.1 ft and lengths of less than 1.5 ft. Ripple dimensions appear to be independent of depth. Ripples are not formed in material with a mean diameter greater than 0.6 mm.
3) Dune phase in which the dimensions of the triangular form are depth dependent and flow resistance tends to increase with depth.

Upper regime flow may also be divided into three major components using the stream power criteria.

1) Plane bed with sediment motion.
2) Standing waves (flow resistance approximately the same as for plane bed).

3) Antidunes which are similar in form to dunes but differ in the mechanics of formation. At high energy levels, breaking antidunes are formed causing increased turbulence and associated high local concentrations of suspended sediment and increased flow energy losses. Chutes and pools may result from breaking antidunes in a high energy level situation.

Although the bed form is a function of a large number of variables, Simons and Richardson indicated good correlation of bed form with stream power (γDSV) and sediment mean fall diameter. The plot developed from this correlation is shown in Fig. 2.3. Although the plot appears to give reasonably reliable results, it should be realized that the transition from one defined bed form to another is gradual, and therefore the enveloping lines cannot be taken as defining exact and sudden changes in form. It should also be noted that more than one form of roughness may be present in a single channel cross section due to variations in local flow conditions.

The relationship of flow resistance to bed form is illustrated qualitatively in Fig. 2.4 after Simons et al. [91], and summarized in Table 2.4 after Simons [90]. It is seen that flow resistance varies over a broad range both between various bed phases and within a single bed form. In canal design, primary concern is directed toward variations within the dune phase since most canals constructed for irrigation and power purposes fall in this category.

The idea that flow resistance is related to bed form in a direct fashion is not new; Einstein (1950) [22,23] utilized this concept in the

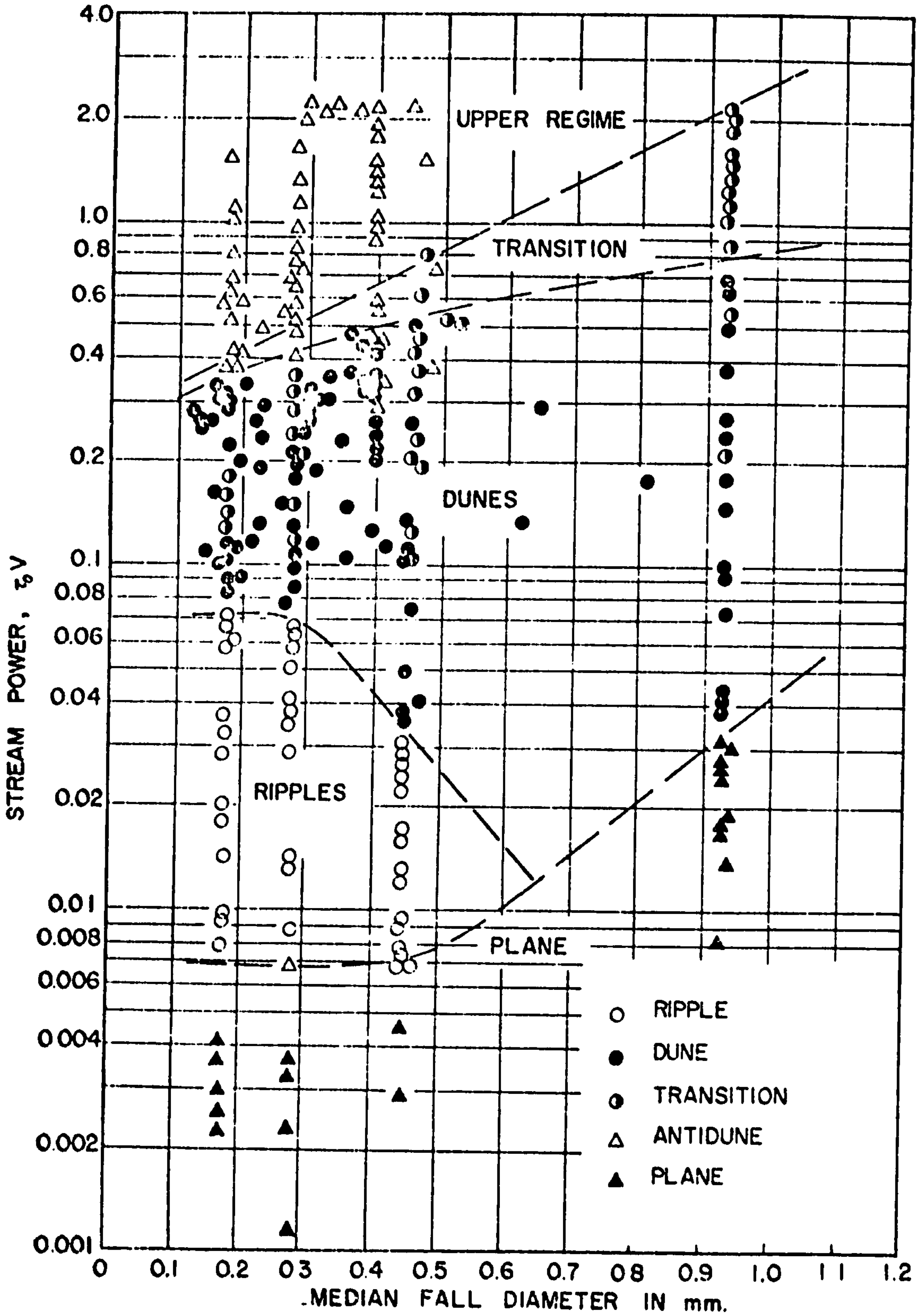

Figure 2.3. Relation of Stream Power and Median Fall Diameter to Form of Bed Roughness (after Simons and Richardson, 1963 [94]).

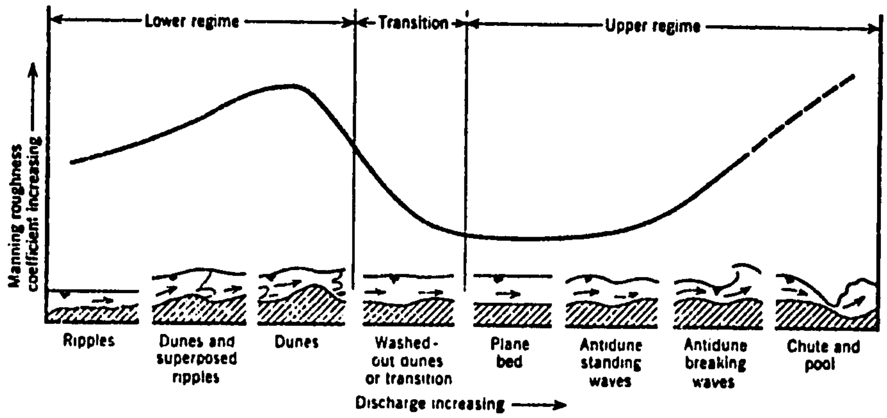

Figure 2.4. Bed Forms and Associated Roughness (after Simons et al. [91]).

development of his now familiar theory of sediment transport. In Einstein's method, the hydraulic radius, and thereby the flow resistance, was broken into a portion dependent on form roughness and a portion dependent on grain roughness, with the actual hydraulic radius being the sum of these two parts. Working along these lines, Haynie [34,35], and Simons and Richardson [92], proposed design procedures which eliminated the necessity of estimating such factors as Manning's rugosity coefficient, Lacey's silt factor, or Blench's bed and side factors.

Table 2.4. Relation Flow Resistance to Bed Roughness (after Simons [90]).

	Plane	Ripples	Dunes	Antidunes	Chute & Pool
$C/\sqrt{g}$	15-23	6-7-12	8-12-15	10-20	9-16
n	0.012-0.016	0.018-0.050	0.018-0.035	0.012-0.028	0.015-0.031

Haynie approached the problem from a point of view analogous to the boundary layer displacement thickness concept of fluid mechanics, and the friction factor vs. Reynold's number diagram used in pipe flow. The logarithmic flow equation,

$$\frac{V}{V_*} = 5.75 \log_{10}\left(\frac{V_* D}{\nu}\right) + 2.5 \tag{2.19}$$

was assumed to correctly describe flow over a smooth rigid boundary, where V = average velocity, $V_* = \sqrt{gRS}$ = shear velocity, D = flow depth, and ν = kinematic viscosity.

Values of V/V_* based on data from existing canal systems were plotted vs. the log of the shear Reynold's number (V_*D/ν) on the same diagram with the curve described by Eq. (2.19). This plot resulted in the determination of a velocity difference between flow over a smooth rigid boundary and flow over a mobile bed. From this, a depth correction or effective smooth boundary displacement thickness could be obtained. Values of velocity and depth corrections to smooth boundary conditions were then correlated with parameters describing channel geometry based on flume, canal, and river data from the U.S. and Pakistan. From this, an iterative design procedure was devised with the initial channel geometry estimated using the regime type plots of Simons and Albertson [95]. The steps to design and necessary plots as given by Haynie are reproduced in Appendix A. For channels operating in the plane bed phase, Haynie recommended use of the relations developed by Liu and Hwang [60] for velocity computations.

Simons and Richardson in 1963 [94] had suggested the possible use of a depth correction to account for variable flow resistance due to bed forms, and in 1966 [92] presented a design procedure similar to that of Haynie utilizing this idea. The procedure differs from

Haynie's in that rather than correcting to a smooth boundary, Simons and Richardson corrected to grain rough plane bed conditions. The correction therefore represents only that portion of the flow resistance attributable to form roughness, and is similar in nature to Einstein's R'' term. Also, where Haynie used the velocity correction as the base from which to develop his relationships, Simons and Richardson used depth or hydraulic radius correction as the prime parameter. The necessary material for application of this procedure is also reproduced in Appendix A.

DISCUSSION OF DESIGN METHODS

The single channel design methods which have been presented may in general be divided into four categories: critical velocities, regime theory, tractive force theory, and depth correction. The engineer faced with a design problem must select from these, either individually or in combination, a method for the solution of his own unique problem. Each of these methods have at some time been used successfully, yet the resulting designs may differ significantly and no one method is generally accepted as being absolutely correct. It would therefore seem worthwhile to review some of the implications, similarities, and differences of the various approaches.

Since average velocity is a variable entering most computations in canal work, and may easily be seen to relate to the sediment carrying capacity of the channel, its selection for use as a criteria for design was logical. The problem with this lies in the fact that average velocity alone fails to account for the forces acting on individual particles which are dependent on other hydraulic and geometric parameters. Critical velocity tabulations do, however, constitute a large and valuable data base representing the experience of many engineers over time. Large deviations from accepted critical velocity values should be regarded with suspicion, and the reason for such deviation ascertained.

Regime theory attempts to account for variations in the parameters not considered in the use of critical velocity alone by relating velocity to the geometric properties of the channel and bed material. Implicit in the use of regime theory is the assumption of three degrees of channel freedom. These are the freedom to adjust in width, depth, and slope. Further implied is the idea that for a given discharge, and bed material, a unique equilibrium value of each of these variables exists [45]. The depth correction technique may be considered as an extension of conventional regime theory and assumes the same three degrees of freedom, but implies an interrelation between these variables which allows a range of equilibrium values for each.

Blench [4], when discussing the applicability of regime concepts, gives the following criteria for <u>exact</u> applicability of the <u>basic</u> relations.

1) Steady discharge.
2) Steady bed sediment discharge of too small an amount to appear explicitly in the equations.
3) Duned sand bed.
4) Suspended load insufficient to affect equations.
5) Steep cohesive sides that are erodible or depositable from suspension and behave as hydraulically smooth.
6) Straightness in plan, so that the smoothed dune bed is level across the cross section.

7) Uniform section and slope.

8) Constant water viscosity.

Additionally, Lacey [45] points out that the regime channel must be formed of the same material as that transported by the flow. This condition is normally satisfied by sand bed channels due to the interrelation of bed material and transported sediment to be discussed in Chapters III and IV.

Since the regime formulas are primarily based on correlation of data from operating canals, care must be taken in extrapolating the relations to conditions not adequately represented in the formulative data. Significant work has been accomplished [4,5,36,45,70] in relating the form of the regime equations to theoretical bases, but the understanding is still imperfect.

The major source of data for the formulation of regime theory was the canals of the Indo-Gangetic plane. Blench summarizes the range of variables represented by these canals as given in Table 2.5.

It is not meant to imply that use of the regime equations has been, or should be, strictly limited to channels which satisfy all of the above criteria. However, as in all work of this type, judgment must be used, and an understanding of the conditions on which a method is based forms a point of beginning. The greater the departure from ideal conditions, the greater the part played by experience and judgment in the design procedure.

Attempts have been made to directly compare Lane's tractive force method with regime equations [37,80]. Insight into why these attempts have not resulted in determining one method "right" and the other

Table 2.5. General Data Range of Indian Canals (after Blench [4]).

Variable	Range	Remarks (not given by Blench)
d	0.1-0.6	Diameter of bed material in millimeters
Gradation	log prob.	Sediment has log-normal size distribution
C per 10^5	0-3	Bed load charge in parts per hundred-thousand by wt.
Suspended Sediment Conc.	0-1%	Wash load
Water Temp.	50-86°F	
Sides	clay, smooth	Canals are "aged" with suspended load settling to form cohesive layer
B/D	4-30	Width to depth ratio
V^2/d	0.5-1.5	Implies Froude number of 0.12-0.22
VB/ν	10^6-10^8	Reynolds number with respect to width
Q	1-10,000	Discharge in cubic ft per sec
Bed phase	dunes	
D/d	>1000	Measure of relative grain roughness

"wrong" may be gained by comparing the approach and the data base of each.

Lane's primary source of data in the initial development of the tractive force approach was the San Luis Valley canals. His primary concern was in the design of channels to carry relatively clear water without scour. In order to contrast the differences in the prime data, a portion of Table 2.5 is reproduced along with comparable parameters from the San Luis Valley canals [55] in Table 2.6.

Table 2.6. Comparison of Indian and San Luis Valley Canals.

Variable	Range Indian Canals	Range San Luis Valley Canals
d_{50}	0.1-0.6 mm	0.79-3.23 in (20-82 mm)
Suspended Sediment	0-1%	relatively clear
Sides	clay, smooth	granular
Bed Phase	dunes (fully active bed)	armored with coarse material (little sediment motion)

It may be seen from the comparison that the problems attacked initially by the two methods, although similar in nature, differ significantly. Both approaches have been expanded for use outside of their original data bases (regime - ref. 4 and 70, tractive force - ref. 52); however, their areas of most direct applicability remain separate.

The critical velocity, tractive force, and regime approaches all suffer from the common fault of requiring the estimation of one or more empirical factors. In the case of critical velocities or tractive force, a flow equation must be selected. The equation used most commonly is that of Manning which requires the selection of the rugosity coefficient $\underline{n}$. In the case of the Lacey relations, the silt factor $\underline{f}$ must be estimated, and in Blench's relations, both side and bed factors must be obtained. Although guides exist for the selection of each of these, their ultimate determination must be based on judgment and experience.

The selection of a value for Manning's $\underline{n}$ is probably the least difficult due to its long and widespread use. A number of tables, some of which include photographs [5,15], have been published giving values of $\underline{n}$ for various materials and conditions. Empirical relations such as those given in conjunction with the presentation of Lane's tractive force method may be considered as reasonably reliable in situations where grain roughness is dominant.

The selection of factors for use in regime equations may present somewhat more of a problem. These factors encompass both flow resistance and stability considerations. Lacey [50] gives a "rough" relation for estimating the silt factor used in his relations as:

$$f = 8\sqrt{d_m} \tag{2.20}$$

where d_m is the mean diameter of the bed material in inches. This relation, however, was never claimed by Lacey to be exact.

Blench [4] also gives guides to the selection of bed and side factors used in his relations. These are:

$$F_b = 1.9\sqrt{d_m} \tag{2.21a}$$

or

$$F_b = 0.58\ \omega^{11/24}\ (\nu_{70}/\nu)^{11/72} \tag{2.21b}$$

and

$$\left.\begin{aligned} &F_s < 0.1 \quad \text{for sandy loam} \\ &F_s \leq 0.20 \quad \text{for silty clay loam} \\ &F_s \leq 0.3 \quad \text{for cohesive banks} \end{aligned}\right\} \tag{2.21c}$$

in which

F_b = Blench bed factor

F_s - Blench side factor

d_m = mean diameter of bed material in millimeters.

ω = fall velocity of mean diameter particle at 70°F in cm/sec.

ν_{70} = kinematic viscosity of water at 70°F.

ν = kinematic viscosity of water in the channel.

Relation (2.21a) is approximately equivalent to Lacey's relation given as Eq. (2.20). Blench indicates this relation to be applicable to sand size particles only. Again, final selection must be based on experience with Eqs. (2.21) serving only as a guide.

The methods involving depth, hydraulic radius, or velocity correction were initiated primarily in an attempt to avoid the uncertainties involved in estimating these factors for sand bed channels. The correction methods cited utilize the regime approach in the initial estimation of channel dimensions and in this respect may be considered an extension of regime methods. The roughness or resistance factor is integrated into the correction plots and need not be calculated directly so long as the representative bed form is maintained. These methods must be considered empirical in the sense that, although the significant parameters were obtained from theoretical considerations, the final curves represent the "best fit" of data points and apparently do not directly represent physical laws.

The correction procedures offer an additional advantage over regime equations in allowing a limited flexibility in the selection of canal geometry. The freedom is limited in that the dune phase of flow must be maintained, sediment transport capacity must be compatible with parent river characteristics and headworks design, and large deviations from regime widths and depths are not advisable.

The first restriction may be seen as a data base limitation which is in turn a result of natural channel behavior characteristics. When the flow passes into upper regime with respect to bed form, the resistance to flow is greatly decreased with a resulting velocity increase. Few natural bank materials can withstand the velocities of upper regime flow without protection. For this reason, care must be taken when designing a channel to operate near the transition region. The occurrence of upper regime flow at some stage of operation may cause severe bank erosion in a short time.

The second and third limitations hinge on the principle of self-adjustment or the three degrees of freedom as applied to canals and rivers. The sediment transported and the balance thereof is the means by which slope adjustment is made. If the quantity of sediment entering the canal exceeds the transport capacity, deposition will occur in the head reaches resulting in an increase in slope. Conversely, if the flow is capable of transporting more material of the sizes making up the boundary than is otherwise available, degradation will occur resulting in a decrease in slope.

The width and depth are controlled by the fact that if the channel is significantly narrowed and deepened, the velocity adjacent to the banks will be increased and erosive forces will tend to widen the channel, while if width is significantly increased, the channel will exhibit a tendency to meander. Alternate bars may be formed with the main current attacking the banks at intervals.

In using the limited freedom provided by the use of depth or velocity correction methods, an understanding of the principles of channel behavior is essential. For the purpose of reviewing these principles, the Lacey-Inglis [39] regime relations are restated.

$$B = K_1 \frac{Q^{1/2}}{g^{1/3}\ \nu^{1/12}} \left[\frac{C\omega}{d_{50}}\right]^{1/4} \tag{2.14a}$$

$$V = K_3 \frac{\nu^{1/36}\ Q^{5/6}}{g^{7/18}(C\ w\ d_{50})^{1/12}} \tag{2.14b}$$

$$D = K_4 \frac{\nu^{1/9}\ Q^{1/3}\ d_{50}^{1/6}}{g^{1/18}(Cw)^{1/3}} \tag{2.14c}$$

$$S = K_5 \frac{(C\ w\ d_{50})^{5/12}}{\nu^{5/36}\ g^{1/18}\ Q^{1/6}} \tag{2.14d}$$

Viewing these relations qualitatively it is seen that if the discharge is increased, other things remaining constant, the width to depth ratio will tend to increase, the average velocity will tend to increase, and the slope will tend to decrease. It was the observation of these tendencies which led to the mathematical formulation of regime theory. Evaluating the effect of sediment concentration qualitatively, it is seen that an increase in sediment concentration will tend to result in an increase in width and slope, and a decrease in depth, with slope being the most sensitive to changes in concentration.

Since increasing the sediment concentration term in the Inglis-Lacey relations is in effect designing the canal for a larger sediment transport capacity, the general behavior indicated may be used to advantage even if the numerical values attached to the coefficients and exponents are not accepted as precise. Adjustment of these variables in the methods given by Simons and Richardson or Haynie, within the limits indicated and in conjunction with acceptable sediment transport computations, provides a logical approach to the design of sand bed channels.

Little has been said about the design of channels in cohesive material. Since research in this area has been limited, large amounts of data do not exist. Lane's tractive force approach and critical velocity tabulations may form a point of beginning for work in this area. Lane [52] computed tentative values of critical tractive force for a number of materials including clays from critical velocities given by Etcheverry [25] and Fortier and Scobey [28] by assuming a depth of three feet, a bottom width of ten feet, and side slopes of 1-1/2:1. The values of critical tractive force obtained from the critical velocities of Fortier and Scobey are given in Table 2.7 and may be used as a guide in design for the case of cohesive materials.

As with any design, it must be realized that the canal is a part of a larger system and its behavior influenced by the other components of that system. If the water entering a canal constructed in cohesive material carries significant sediment in the sand size range as bed load, the final behavior of the canal may be that of a sand bed channel with associated flow resistance and capacity.

In the implementation of any design procedure, available data from similar systems in the area should be utilized. Data taken from these systems may serve both to point out problems unique to the particular area and to provide a check on the applicability of the selected method of approach.

Table 2.7. Comparison of Limiting Velocities Determined by S. Fortier and F. C. Scobey with Values of the Tractive Force for Straight Channels After Aging (after Lane [52]).

Material	Value of n	Clear Water		Water Transporting Colloidal Silts	
		Velocity, in feet per second	Tractive force, in pounds per square foot	Velocity, in feet per second	Tractive force, in pounds per square foot
Fine sand, colloidal	0.020	1.50	0.027	2.50	0.075
Sandy loam, noncolloidal	0.020	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 noncolloidal	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	0.30	6.00	0.67
Cobbles and shingles	0.035	5.00	0.91	5.50	1.10

CHAPTER III

REVIEW OF SEDIMENT TRANSPORT COMPUTATIONAL TECHNIQUES

To anticipate the sediment transport capacity of a channel is perhaps the most difficult problem encountered in canal design. As stated by Bogardi [9]:

> "The problem is a highly complex one so that, in spite of the great number of investigations and observations conducted for obtaining a clear picture, no satisfactory solution has been found as yet."

The computational methods presented must therefore be used with proper caution and judgment and should, as much as possible, be supplemented by data from similar existing systems.

To undertake a complete discussion of the development of sediment transport theories would be beyond the scope of this paper. The presentation here will therefore be limited to three of the more accepted points of approach to the problem through computational techniques representative of each.

Before proceeding with a discussion of the various approaches to sediment transport computations, selected terms are defined for clarification.

Definitions

Bed layer: The flow layer immediately above the bed. (Usually taken as two grain diameters thick.)

Bed load: Sediment which moves essentially in contact with the bed within the bed layer.

Bed material: That sediment material of which the streambed is composed.

Bed material discharge: That part of the total sediment discharge which is composed of grain sizes found in the bed. Bed material discharge is usually considered to be equal to the transport capacity of the flow.

Sediment concentration: The quantity of sediment relative to the quantity of transporting fluid or fluid sediment mixture. When expressed in parts per million (ppm), the concentration is the ratio by weight.

Suspended load: Sediment that is supported by the upward components of turbulence and remains in suspension for an appreciable length of time.

Wash load: That part of the sediment discharge which is composed of particle sizes finer than those found in appreciable quantities in the bed material. Wash load is primarily dependent on the upslope supply rate.

Tractive Force Approach

One of the earliest models for computing bed material transport was that advanced by duBoys in 1879 [20] based on tractive force as the governing flow parameter. duBoys modeled the channel bed as moving in layers with the velocity of each layer decreasing linearly downward.

Although duBoys' model was proved to be incorrect in its assumptions, more refined formulae, based on the idea of shear stress in excess of that required to initiate motion being the governing parameter for sediment transport, have survived. As an example of a method utilizing this basic approach, the Meyer-Peter and Müller [75] relations are selected. These relations have received extensive use in Europe, and were converted to English units for use in the U.S. by the U.S. Bureau of Reclamation. In the following summary however, the units required are those of the metric system.

Meyer-Peter and Müller based their relations on studies done from 1934 to 1948 and concerned themselves with bed load only. Bed material transported in suspension was not considered in their computations. The relations were developed in three steps. The effect of variations in depth and discharge on the transport of uniform material was studied first. The experiments were then repeated with different bed materials to determine the influence of specific gravity on transport, and with natural sand mixtures to determine the effect of gradation. The general range of data covered by the study was:

Energy slope from 0.4% to 20%

Mean diameter of bed material from 0.4 mm to 30 mm

Water depths from 1 cm to 120 cm

Discharge quantities from 2 lit/sec to 4 m^3/sec.

The final relation derived from these studies to describe bed load transport was given as:

$$\gamma(K_s/K_r)^{3/2}\ R_s S = 0.047\ \gamma'\ d_m + 0.25\ \rho^{1/3}\ q_s'^{2/3} \tag{3.1}$$

in which

K_s = roughness coefficient equivalent to that developed by Strickler (K_s may be taken as $1/n$ where n is the roughness coefficient used in Manning's equation).

K_r = roughness coefficient due to skin friction (given by Meyer-Peter and Müller as $K_r = 26/d_{90}^{1/6}$ where d_{90} is the diameter in meters for which 90 percent of the material is finer).

$\gamma' = \gamma_s - \gamma$ = bouyant specific weight of sediment particle.

ρ = density of water sediment complex.

q_s' = rate of sediment discharge weighed under water.

d_m = algebraic mean diameter of bed material in meters.

R_s = bed hydraulic radius.

S = slope of energy grade line.

Examination of Eq. (3.1) reveals the left-hand side to be the average tractive force exerted on the bed of the channel corrected by the ratio of grain resistance to total flow resistance raised to the 3/2 power. The right-hand side of the equation then breaks this effective tractive force into two parts, that required to initiate motion ($q'=0$), and that part effective in transporting the material. The prime difference between this and more primitive forms based on excess tractive force therefore lies in the factor $(K_s/K_r)^{3/2}$, which attempts to account for the effect of form roughness in the channel.

Stochastic Approach

Einstein [23] approached bed material transport as a probability problem. He reasoned that in an equilibrium condition, the number of particles deposited on the bed of a channel must be equal to the number of particles of the same size eroded from the bed in the same time period. The probability of a particle being eroded was taken as that portion of the time which, at any one spot, the local flow conditions cause a sufficiently large lift on the particle to remove it from the bed. With all points on the bed statistically equivalent, this was shown to also be equal to the fraction of the bed on which, at any time, the lift on a given size particle was sufficient to cause motion.

Through the second interpretation and existing fluid mechanics concepts of the behavioral characteristics of turbulent flow, the probability of a particle being eroded was then related to parameters describing the hydraulic and geometric properties of the channel and the bed material. For computational purposes, the bed material is divided into size fractions and the transport computed for each fraction. In this way, the gradation of the transported material as well as the total quantity is estimated.

The resulting relations lead to a primary dependence of the method on individual particle shear and make it, in this respect, similar to those methods previously discussed. The nature of this dependence is presented in abbreviated form by Eqs. (3.2) in which the subscript i refers to a given size fraction of the bed material.

$$q_b = \sum_i f(\Phi_{*i}) \qquad (3.2a)$$

$$\Phi_{*i} = f(\psi_{*i}) \qquad (3.2b)$$

$$\psi_{*i} = \xi_i \, Y(\beta/\beta_x)^2 \, \psi_i \qquad (3.2c)$$

and

$$\psi_i = \left(\frac{\rho_s - \rho}{\rho}\right) \frac{d_i}{R'S} \qquad (3.2d)$$

in which

q_b = bed load transport per unit of channel width

Φ_{*i} = Einsteins transport intensity parameter for size fraction i. (Relation of Φ_{*i} to q_b developed from theoretical considerations.)

ψ_{*i} = intensity of shear on individual grain size. (Relation of ψ_* to ϕ_* developed from theoretical considerations.)

ξ_i = correction factor for particle hiding. (Developed from empirical data, primarily flume experiments.)

Y = lift correction factor. (Developed from empirical data, primarily flume experiments.)

$(\beta/\beta_x)^2$ = parameter related to the hydraulic roughness of the boundary. (Dependent on Nikuradse's data using sand roughened pipes.)

ψ_i = intensity of particle shear.

ρ_s = mass density of particle.

ρ = mass density of fluid-sediment complex.

d_i = geometric mean size of bed material fraction.

R' = hydraulic radius associated with grain roughness (discussed further below).

S = slope of energy grade line.

The use of R' in relation (3.2d) is a result of Einstein's assumption that only the turbulence generated by the grains was in close enough proximity to the particles to be effective in generating bed motion. He then developed a flow resistance function by dividing the hydraulic radius into two parts, a portion associated with grain roughness (R') and a portion associated with form roughness (R''). The actual hydraulic radius is the sum of these two parts. The division was made such that the logarithmic velocity equation in terms of grain roughness was satisfied, i.e.:

$$\frac{V}{u_*'} = 5.75 \log_{10} \left(12.27 \frac{R'x}{k_s}\right) \tag{3.3a}$$

giving a velocity distribution in the vertical of:

$$u_y = u'_* \, 5.75 \log_{10} (30.2 \frac{yx}{k_s}) \tag{3.3b}$$

in which

V = average velocity.

u_y = local velocity at a distance y above the bed.

$u'_* = \sqrt{gR'S}$ = shear velocity associated with grain roughness.

R' = hydraulic radius associated with grain roughness.

x = correction factor relating to the transition from smooth to rough boundaries

k_s = representative grain size, taken as d_{65} by Einstein.

The resistance relation was completed by relating the hydraulic radius associated with form roughness to a representative boundary shear. This relation was given as:

$$\frac{V}{u''_*} = f(\psi') \tag{3.3c}$$

where

$$\psi' = \frac{\rho_s - \rho}{\rho} \cdot \frac{d_{35}}{R'S} \tag{3.3d}$$

with u''_* being the shear velocity associated with form roughness, and all other variables being as previously defined. The functional relation given by Eq. (3.3c) was developed graphically from existing river measurements. Einstein [23], in his original publication, indicates this to be probably the least reliable of the relations developed.

Given d_{35} of the bed material and the relation of cross-sectional area to hydraulic radius for the channel, the total hydraulic radius and corresponding discharge may be obtained from Eqs. (3.3) by assuming a value of R'. If a specific discharge is desired, solution is by trial and error.

Einstein included the suspended portion of the bed material load in his calculations by assuming the bed load to be limited to a layer two grain diameters above the bed, and that the concentration at the upper limit ($y = 2d_i$) was equal to the average concentration within this layer. The distribution of suspended material in the vertical was taken to be of the form given by Rouse [84] as:

$$C_{yi} = C_{ai} \left(\frac{D-y}{y} \frac{a}{D-a}\right)^{z_i} \tag{3.4a}$$

where

$$z_i = \omega_i/\beta\kappa V_* \tag{3.4b}$$

and

y = distance above the bed.

C_{yi} = concentration of material having a geometric mean diameter $\underline{d_i}$, as a function of $\underline{y}$.

C_{ai} = concentration at a distance $\underline{a}$ above the bed.

D = depth of flow.

ω_i = fall velocity of sediment particle of diameter $\underline{d_i}$.

κ = von Karman's coefficient.

β = a coefficient relating diffusion coefficients.

V_* = shear velocity.

Einstein then numerically integrated the produce of concentration and velocity over the channel depth to obtain the suspended sediment discharge for a given particle size. In performing the integration, β and κ were assigned values of 1.0 and 0.4 respectively, and the shear velocity (V_*) was replaced by the grain shear velocity (u_*').

For a full derivation of Einstein's method, the reader is referred to Einstein's original work [23] which also contains a discussion of the methods limitations and example computations. Due to the number of computations required, the method is seldom applied in a form other than that adapted to the use of high speed computers. One such program to perform the required computations along with directions for its use is included as Appendix B.

A number of modifications to Einstein's method have been made by various researchers. One such modified approach is that devised by Colby and Hembree in 1955 [17] and often referred to simply as the "Modified Einstein Procedure." The technique was developed to utilize the principles developed by Einstein for estimating the total sediment discharge (including wash load) from stream flow measurements, depth integrated suspended sediment samples, bed material samples, and temperature. In application, the methods differ in that the modified procedure computes total load, but was developed for, and limited to, use on an existing system, whereas the original procedure computes bed material load only and may be used as a design tool.

The major computational differences between the two methods are:

1. In the modified procedure, shear velocity associated with grain roughness is computed using measured values of average velocity and depth in the relation:

$$u_m = \frac{V}{5.75 \log_{10} \{\frac{12.27\ Dx}{k_s}\}} \tag{3.5}$$

This corresponds to Eq. (3.3a) with u_*' replaced by u_m and R'

replaced by the measured mean depth (D). All other variables are as previously defined.

2. The suspended load exponent, z, in Eq. (3.4a) is determined from suspended sediment measurements for a dominant grain size. Values of z for other than the dominant size are assumed to vary with the 0.7 power of the particle fall velocity. The relation used by Einstein, given as Eq. (3.4b) implies a variation of z with particle fall velocity to the first power.

3. The intensity of individual particle shear, ψ_*, in the original procedure is replaced in the modified procedure by the larger value of ψ_m computed by the following:

$$\psi_m = \frac{\left(\frac{\rho_s - \rho}{\rho}\right) d_{35} \sqrt{g}}{u_m} \tag{3.6a}$$

$$\psi_m = \frac{0.4 \left(\frac{\rho_s - \rho}{\rho}\right) d_i \sqrt{g}}{u_m} \tag{3.6b}$$

where all variables are as previously defined. This in effect assigns a value of 0.4 to the expression $\xi Y(\beta/\beta_x)^2$ in Eq. (3.2c) for grain sizes greater than $2.5\ d_{35}$ and assigns a constant value of shear intensity to particles smaller than $2.5\ d_{35}$.

4. Einstein's transport intensity parameter, Φ_*, is arbitrarily divided by two in the modified procedure to more closely fit the available data.

Colby and Hembree provide a discussion of the logic underlying these modifications, as well as graphical aids and example computations illustrating the uses of the method, in their original publication [17].

A computer program to perform the required computations has been published by Mahmood and Ponce [62].

Mahmood, in 1971 [69], developed a method for estimating bed material transport in sandbed channels based on the work of Einstein, but incorporating later advances in bed form prediction and the effect of sediment motion on the turbulent structure of the flow. A model of two-dimensional time and space average velocity distribution for sandbed channels was first developed. This model divides the flow into three regions as shown in Fig. 3.1, using the mean bed elevation as a reference.

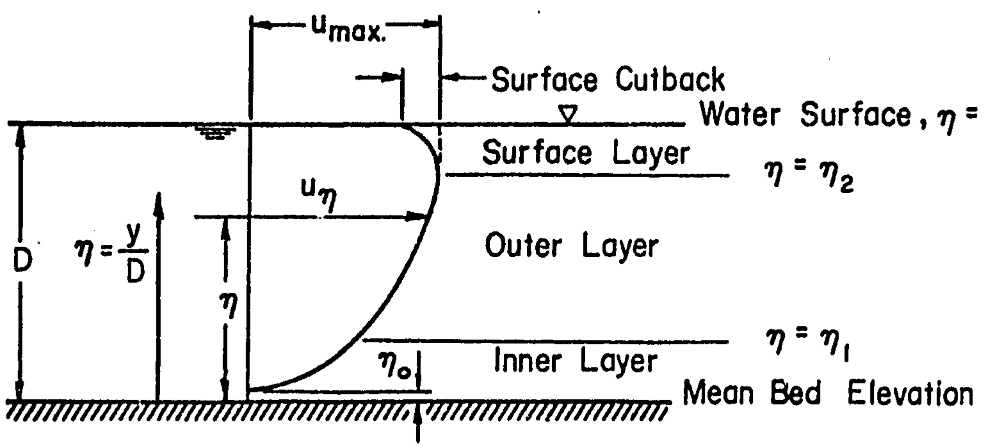

Figure 3.1. Velocity Distribution in Sandbed Channels (after Mahmood [69]).

Using the analogy to the inner and outer layers in turbulent flow over hydraulically rough rigid boundaries, the velocity profile was described mathematically in terms of the dimensionless distance from the mean bed as:

$$u_\eta = \frac{u_{*e}}{\kappa_o} \ln\left(\frac{\eta}{\eta_o}\right) \quad \text{for} \quad \eta_o \leq \eta \leq \eta_1 \tag{3.7a}$$

and

$$u_\eta = \frac{u_{*e}}{\kappa_o} \ln \left(\frac{\eta_1}{\eta_o}\right) + \frac{V_*}{\kappa_o\, g_1} \ln \left(\frac{\eta}{\eta_1}\right) - I(\eta_2)\, V_*\, a_1 \left(1 - \frac{\eta}{\eta_2}\right)^2 \qquad (3.7b)$$

for $\eta_1 \leq \eta \leq 1.0$

in which

u_η = local average velocity.

u_{*e} = effective shear velocity. Effective rather than actual shear velocity is used to account for the effects of bed form and boundary movement on the average velocity profile of the inner layer.

κ_o = 0.4 = von Karman's constant.

V_* = $\sqrt{gRS}$ = actual shear velocity

g_1 = a correction factor applied to von Karman's constant to account for the effect of suspended sediment and local accelerations caused by the bed forms on the turbulent structure of the flow in the outer layer. For practical application in his resistance and transport functions, Mahmood assigned a value of $g_1 = 1.0$.

$I(\eta_2)$ = unit step function used to combine relations for the outer and surface layers.

a_1 = an empirical coefficient describing surface cutback. A value of $a_1 = 35.0$ was found to fit Mahmood's flume data.

η_o = dimensionless depth at which the velocity is zero. Related to relative roughness of the boundary after Keulegan as $\eta_o = (1/33.35) \cdot (k_s/D)$. Mahmood used this relation taking $k_s = d_{84}$ of the bed material.

η_1 = dimensionless depth at which both Eqs. (3.7a) and (3.7b) apply. Based on measured profiles, Mahmood assigned a value of $\eta_1 = 0.15$.

η_2 = dimensionless depth describing the lower limit of surface cutback influence. In fitting Eq. (3.7b) to flume data, η_2 was found to be approximately 0.8. Surface cutback was neglected in the derivation of resistance and transport relations, however, thereby assigning an effective value of $\eta_2 = 1.0$.

Substituting in the constant values indicated, Eqs. (3.7a) and (3.7b) reduce to:

$$u = 2.5\ u_{*e}\ \ln\left(\frac{33.5\ y}{d_{84}}\right) \quad \text{for} \quad y \leq 0.15\ D \tag{3.7c}$$

and

$$u = 2.5\ u_{*e}\ \ln\left(\frac{5.03\ D}{d_{84}}\right) + 2.5\ u_*\ \ln\left(\frac{6.67\ y}{D}\right)$$

$$\text{for} \quad 0.15\ D < y \leq D \tag{3.7d}$$

In Eqs. (3.7c) and (3.7d), all values are directly obtainable except the effective shear velocity u_{*e}. Although arrived at differently, u_{*e} is similar in nature to Einstein's u_*'. Since it is always less than the actual shear velocity, it may be expressed as:

$$u_{*e}^2 = V_*^2 - \Delta u_*^2 \tag{3.8a}$$

A resistance function was developed based on Eqs. (3.7) by graphically correlating Δu_* with Shield's parameter expressed as:

$$S_h = \frac{\rho u_{*e}^2}{(\gamma_s - \gamma)\ d_{50}} \tag{3.8b}$$

in which all terms are as previously defined. Separate plots of $V/\Delta u_*$ versus $1/S_h$ were developed for the upper and lower flow regimes as defined by Simons and Richardson [94] and determined using Fig. 2.3. Since effective shear velocity enters both parameters, solution is by trial and error. Example computations illustrating the use of this approach are given in Mahmood's original publication [69].

Equations (3.7) were also used in the development of a method for estimating the bed material discharge. Like Einstein, Mahmood assumed the bed load to be limited to a bed layer two grain diameters thick, and the suspended load for a given size fraction to be determined by the integral over the remaining depth of the product of the velocity and the concentration. To more closely conform to the assumptions of Eqs. (3.7), the concentration was assumed to vary as:

$$C_{\eta_i} = C_{\eta_{ai}} \left(\frac{\eta_{ai}}{\eta}\right)^{z_i} \qquad (3.9a)$$

in the inner layer ($\eta_{ai} \le \eta \le \eta_1$), and:

$$C_{\eta_i} = C_{\eta_{1i}} \left(\frac{\eta_1}{1 - \eta_1} \cdot \frac{1 - \eta}{\eta}\right)^{z_i} \qquad (3.9b)$$

in the outer layer ($\eta_1 < \eta \le 1$), with $\eta_{ai} = 2d_i/D$ and z_i computed from Eq. (3.4b) using the total shear velocity and a value of $\beta = 1.0$. Except for the determination of z_i, Eq. (3.9b) is identical to Eq. (3.4a) used by Einstein.

Bed load transport per unit of channel width for a size fraction (q_{bi}) was related to the reference concentration such that:

$$q_{bi} = u_{\eta_{ai}} C_{\eta_{ai}} \eta_{ai} D \qquad (3.10)$$

in which $u_{\eta_{ai}}$ is the velocity at the outer limit of the bed layer and other variables are as previously defined.

The reference concentration for a given size fraction was in turn assumed to be related to a mean concentration parameter in proportion to excess particle shear as:

$$C_{n_{ai}} = \left(\frac{\tau_o - \tau_{ci}}{\tau_o}\right) \frac{C_b}{n} \tag{3.11a}$$

in which

τ_o = γRS = average shear stress on the boundary.

τ_{ci} = critical shear stress as determined by Shield's criteria for the geometric mean diameter of the size fraction.

n = number of equal size fractions into which the bed material is divided.

C_b = mean concentration parameter.

Through analysis of flume data, the mean concentration parameter was developed as:

$$C_b = f\left(\frac{\gamma_s S_h^{3/4} \omega_{50}}{u_{*e}}\right) \tag{3.11b}$$

with the functional relationship being empirically defined. The data used in determining this relationship were primarily from flume and canal measurements. The range of variables included median bed material sizes from 0.19 to 0.93 mm, total bed material transport rates from 0.00024 to 8.06 pounds per second per foot of channel width, and bed forms from ripples to antidunes. Mahmood expressed this relation graphically by plotting the total bed material transport per unit width, q_B, versus a transport parameter, G_m. The relation of these variables to those of Eq. (3.11b) is given by:

$$\frac{C_b}{\left(\frac{\gamma\, S_h^{3/4}\, w_{50}}{u_{*e}}\right)} = \frac{q_B}{G_m} \tag{3.11c}$$

With the mean concentration parameter known, the bed material transport for a given size fraction may be expressed as:

$$q_{Bi} = \left(\frac{\tau_o - \tau_{ci}}{\tau_o}\right) \frac{C_b}{n} D \left[u_{\eta_{ai}} C_{\eta_{ai}} \eta_{ai} + \int_{\eta_{ai}}^{1.0} u_\eta C_{\eta i}\, d\eta\right] \tag{3.12}$$

with the functional relationship of the variables defined in Eqs. (3.7) and (3.9).

A more complete discussion concerning the development of these resistance and transport relations as well as graphical aids and example computations may be found in Mahmood's original publication [69]. A computer program, written in Fortran IV, for performing the required computations has been developed and published by Mahmood and Ponce [62].

Direct Data Correlation

Colby (1964) [16] presented plots by which he stated a "reasonable estimate" of the bed material discharge could be made with a minimum of time and expense. Colby recognized the overall complexity of the problem and initially stated:

> "The sediment discharge at a cross section of a stream may be considered to depend on depth, width, velocity, energy gradient, temperature, and turbulence of the flowing water; on size, density, shape and cohesiveness of the particles in the banks and bed at the cross section and in upstream channels; and on geology, meteorology, topography, soils, subsoils, and vegital cover of the drainage area."

Colby then limited his study by considering only the bed material discharge of sandbed streams to reduce the influence of some of these variables.

Four related primary parameters were selected for comparison in their ability to predict bed material discharge. These parameters were total shear (γRS) or shear velocity ($\sqrt{gRS}$), mean velocity (V), shear velocity computed from mean velocity ($\sqrt{g(RS)_m}$, approximately equivalent to Einstein's $\sqrt{gR'S}$), and stream power (γRSV).

After plotting each of these parameters against bed material discharge for a number of measured points, Colby concluded that the relation developed using mean velocity was as accurate as any tried, except in the antidune region, and was more convenient to apply. Within the antidune region, stream power appeared to have the advantage over the other parameters in accurately predicting sediment discharge, but suffered from the fact that the required evaluation of energy slope was often difficult.

In his final plots, Colby incorporated the "secondary" effects of depth, temperature, and concentration of fine sediment. High fine sediment concentrations were assumed to affect bed material discharge in the same fashion as low temperatures, through a change in the effective viscosity of the water sediment complex. The resulting curves developed using mean velocity as the prime governing variable are shown in Figs. 3.2 and 3.3. These curves were based on available data with some portions being extrapolated.

To use Colby's curves for estimation of total bed material discharge, mean velocity, mean diameter of bed material, temperature, concentration of fine sediment, depth, and width of channel must be known. Figure 3.2 is entered with mean velocity, mean diameter of bed material, and depth, to obtain bed material discharge per unit width for a temperature of 60°F and a negligible concentration of fine sediment.

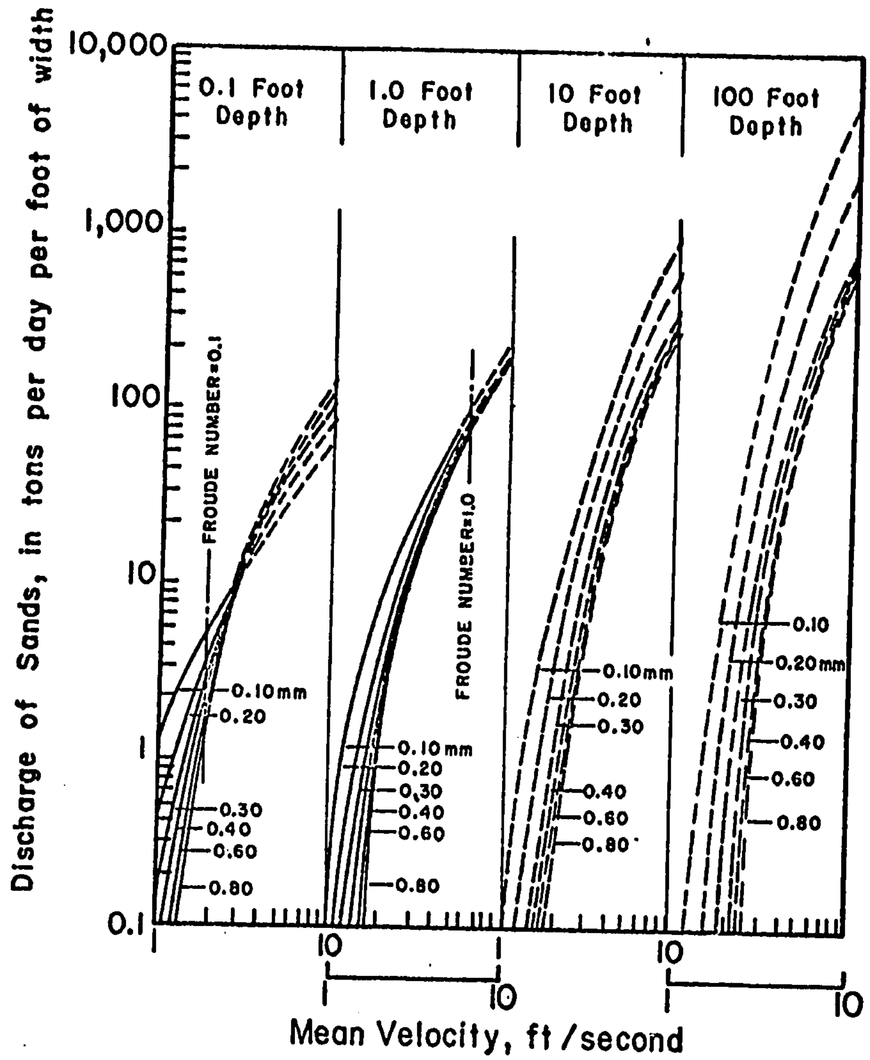

Figure 3.2. Bed Material Discharges as Experimentally Defined Functions of Mean Velocity, Depth, and Median Particle Size for a Water Temperature of 60°F (after Colby [67]).

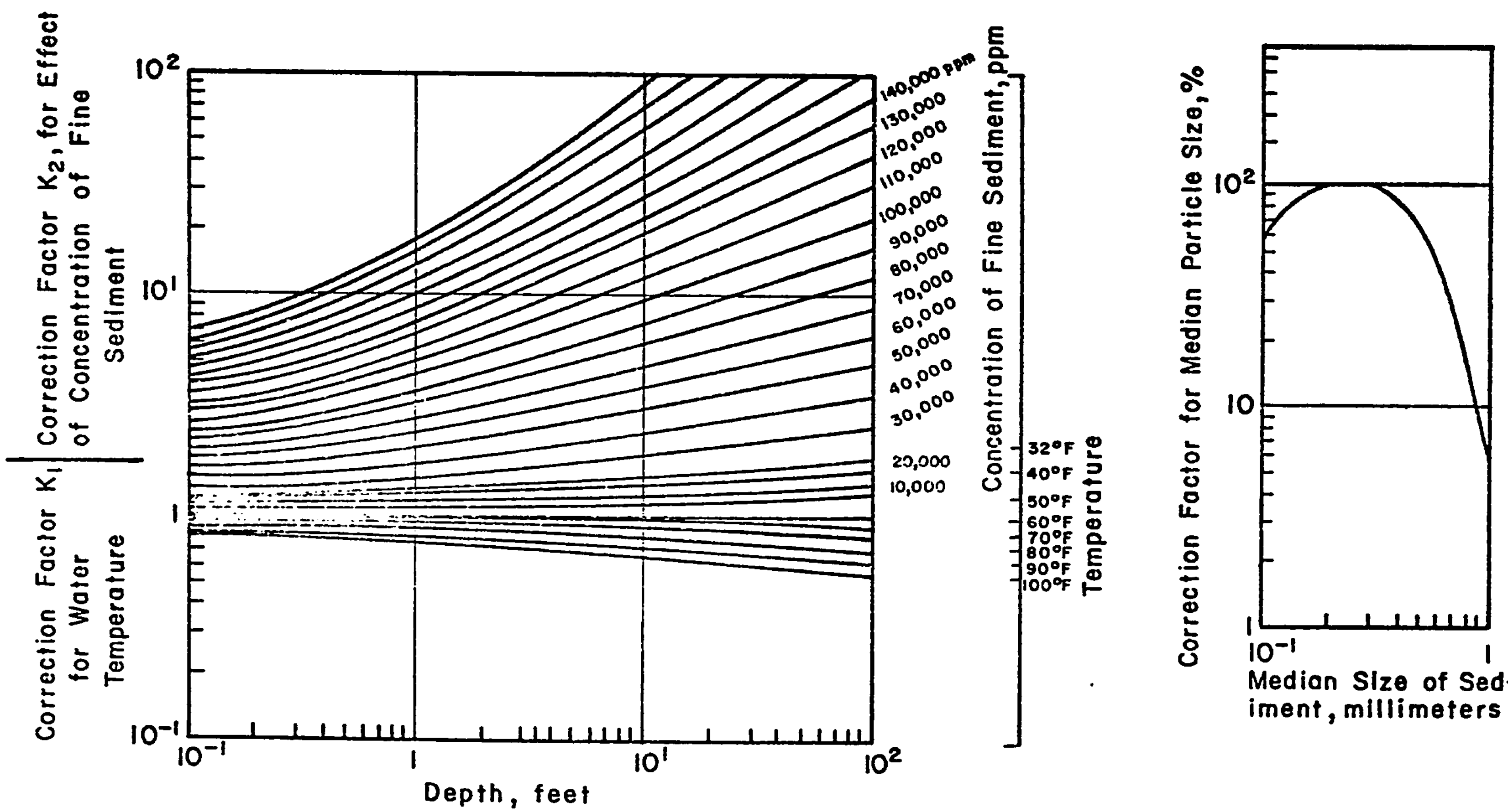

Figure 3.3. Approximate Effect of Water Temperature and Concentration of Fine Sediment on the Relationship of Bed Material Discharge to Mean Velocity (after Colby [16]).

Correction factors k_1 and k_2 are obtained from Fig. 3.3 for temperature and fine sediment concentration respectively. Correction factor k_3 adjusts the effect of viscosity changes on bed material discharge according to bed material size, making it a correction to the k_1 and k_2 factors. The bed material discharge is then given by the relation:

$$q_T = q_{BM} \left(1 + (1 - k_1 k_2)\ 0.01\ k_3\right) \tag{3.13}$$

where

q_T = bed material discharge in tons per day per ft of width.

q_{BM} = bed material discharge @ 60°F with negligible wash load.

k_1, k_2, and k_3 are correction factors as defined above.

In discussion of the method, Colby indicated that

> "...about 75 percent of the sand discharges that were used to define the relationship were less than twice or more than half of the discharges that were computed from the graphs of the average relationship. The agreement of computed and observed discharges of sands for sediment stations whose records were not used to define the graphs seemed to be about as good as that for stations whose records were used."

Another empirical approach to the problem has been to apply mathematical regression analysis to large quantities of data. This approach has been used by a number of researchers since the advent of the high speed digital computer. Although the relations derived by this method do not appear to show general agreement, the approach may prove particularly useful if the system in question is very similar to the system(s) from which primary data is available.

Shen and Hung [89] applied regression analysis to 587 data points from flume, canal, and river measurements to arrive at the relation:

$$\log_{10} C = -107404.459 + 324214.747\ X - 326309.589\ X^2 + 109503.872\ X^3 \tag{3.14a}$$

in which

$$X = \left(\frac{V\, S^{0.57}}{\omega^{0.32}} \right)^{0.00750189} \qquad (3.14b)$$

and

C = concentration in parts per million by weight.

V = average velocity in ft/sec.

ω = sediment fall velocity in ft/sec.

S = slope of energy grade line.

They found that 95 percent of their primary data fell within 0.37 and 2.72 of the value expressed by this relation.

Summary of Sediment Transport Computational Methods

Three separate, but not unrelated, approaches to the computation of sediment transport in a channel have been presented. As with the relations presented in Chapter II, no single approach or computational method may be designated as being the "best" for all application. The design engineer must again rely on judgment and experience in selecting the method most applicable to his particular problem. However, a knowledge of the assumptions made and data used in the development of the methods may serve as a valuable guide in the selection process.

The method devised by Meyer-Peter and Müller based on excess shear stress [75], utilized data involving relatively coarse material and steep energy slopes. Since they concerned themselves with only that material transported as bed load, Eq. (3.1) is primarily applicable to channels in which little or no material is transported in suspension. As a result, its use is most often associated with gravel and cobble bed streams.

The stochastic approach as presented by Einstein [23] is seen to also result in a primary dependence on individual particle shear. The method computes total bed material load and the gradation of transported material. The procedure was initially intended for general usage, but has not always been found to give reliable results.

Mahmood [69] modified Einstein's approach for application to sandbed channels using a two layer model of velocity distribution. The plots developed by Mahmood relating flow resistance to Shield's parameter are identical in nature to that developed by Einstein. Mahmood's resistance function, however, was developed from canal and flume data and directly considers variations in bed form, whereas Einstein used river data which may have included multiple bed forms within a single cross section.

Mahmood's original work showed a close comparison of computed values with primary data for resistance, total bed material transport, and gradation of transported material. Sufficient field use of the method has not been made to determine its general applicability.

Colby and Hembree [17] modified Einstein's computational procedure for application to existing channels on which velocity and suspended sediment measurements are available. The method computes total load and gradation of transported material.

Of the methods discussed, this is the only one which includes wash load (that portion of the total load too small to be found in significant quantities in the bed). Since this portion of the load is dependent on supply rather than channel transport capacity, it must be considered separately in the usual design situation. Calculations which include this portion of the sediment load must be based on measurements, upstream availability, or both.

Since the techniques utilizing Einstein's approach involve a large number of calculations, they are often applied through use of the high speed computer. Programs have been developed for this purpose [62].

Colby [16] developed a quick graphical method for estimating the total bed material discharge of sandbed channels. The limits of application are implied by the median diameter and mean velocity range of the curves. Simons [90] suggests that the gradation of the transported material may also be estimated by dividing the bed material into size fractions and applying Colby's curves to each fraction. In his original publication, however, Colby did not discuss use of the curves in this fashion.

In addition to those mentioned, a number of other formulae have been developed and used in the computation of sediment discharge. Most, however, may be seen to depend on approaches similar to one of the preceding examples. At the present time, the methods presented here may be considered as applicable for general usage as any available. In selecting any technique for application to a particular problem, it is desirable that the characteristics of the system lie within the data base used in development of the method, or within the range for which the method has been shown to reasonably apply. When available, data from similar systems should be utilized. In some cases, mathematical regression analysis of this data may provide effective transport relations. Chitale (1966) [14] and Bogardi (1974) [9] present rather thorough reviews of sediment transport theory and methods of computation. These references may serve as a point of beginning for the individual interested in pursuing further information in this area.

CHAPTER IV

SEDIMENT DISTRIBUTION IN BRANCHING SYSTEMS

The problems associated with sediment in a canal system are primarily related to the stability of the channels within the system. If neither aggradation nor erosion is to occur, the principle of mass continuity with respect to sediment must be satisfied within each subsystem contained in the network. Application of this principle implies that the sediment inflow to any subsystem over finite time periods must equal the sediment outflow. In order to achieve this goal, each component of the system must be considered in relation to the overall system.

Sediment inflow to the system is dependent on the transport characteristics of the parent channel and on headworks design. At low head diversions, inflow concentrations may be expected to vary seasonally with variations in water and sediment discharge in the parent channel. For systems with relatively constant water discharge, however, a time average value for sediment inflow may be used for design if provision is made for channel storage of bed sediments during periods of high inflow. This sediment is then removed by the flow during periods of low sediment inflow concentrations, resulting in satisfaction of mass continuity with respect to bed sediment for time periods which include one or more seasonal cycles of operation [67].

Both quantity and quality of sediment entering the system may be influenced by the location and design of the headworks. Sediment exclusion and/or ejection devices incorporated into headworks or bifurcation structures may reduce the quantity and mean size of sediment entering the system or subsystem. In the case of low head diversions from alluvial channels, however, significant quantities of sand size material may enter the system even when these devices are used. Melone, Richardson, and Simons [74], among others, present a review of current sediment exclusion and ejection techniques. Computational methods discussed in Chapter III combined with proper headworks design computations may be used to construct a sediment inflow hydrograph and to determine a sediment inflow quantity for use in canal network design.

Sediment entering the system is disposed of through distribution to water users with the water supply, by allowing it to accumulate at designated areas within the system and periodically removing it by mechanical means, or by a combination of the two. Efficient disposal of the sediment by either means requires consideration be given to sediment routing within the system as a part of the design procedure.

Location of points within the system for the storage and removal of sediment should be selected in advance based on equipment available for sediment removal and the availability of disposal areas. The siting of a disposal area is often a critical factor, since the removal of 50 ppm of sediment from a channel with a 5000 cfs discharge would amount to 2.5 million tons in a ten year period, requiring approximately 1200 acre feet of storage area [67].

The feasibility of disposing of sediment through distribution with water supplies in an irrigation system is determined by the capacity of on-farm water courses to handle sediment loads, and on the ability of the farmland to absorb the sediment. Sand size materials may be beneficial to clay soils but reduce productivity of more open textured soils. In the initial design of the system, an attempt should be made to route the bed material load to those lands most capable of absorbing it without detrimental effects.

The routing of sediment to the desired disposal areas is complicated by the tendency for sediment concentrations to increase in the downstream direction due to seepage and evaporation losses, while channel transport capacities tend to decrease in the downstream direction due to the reduced individual channel discharge in a branching system. The principles and computational methods presented in Chapters II and III form a basis for the evaluation of this problem and thus for the development of a system design which minimizes the detrimental effects of sediment disposal. The remainder of this chapter is devoted to the development of a rational system design procedure based on the previously discussed concepts and methods and application of the principle of continuity with respect to sediment.

Continuity Considerations

The requirement for mass continuity of water and sediment in a well maintained irrigation canal system made up of channels CH(i,j) may be expressed as [68]:

$$Q(K,L) = \sum_{i,j \varepsilon N} \{ \sum_{ir \varepsilon M} [Q_{ir}(i,j)] + Q_s(i,j)\} \tag{4.1a}$$

$$G(K,L) = \sum_{i,j \varepsilon N} \{ \sum_{ir \varepsilon M} [Q_{ir}(i,j) \cdot C_{ir}(i,j)] 62.4 \times 10^{-6} + G_c(i,j)\} \tag{4.2a}$$

$$G(K,L) = Q(K,L) \cdot C(K,L) 62.4 \times 10^{-6} \tag{4.2b}$$

where

$Q(K,L)$ = Water discharge at the head of $CH(K,L)$ in cfs.

$G(K,L)$ = Bed material load at the head of $CH(K,L)$ in lbs/sec.

$C(K,L)$ = Bed material concentration in $CH(K,L)$ in ppm by wt.

$Q_{ir}(i,j)$ = Water discharge of irrigation turnout (ir) from $CH(i,j)$ in cfs.

$C_{ir}(i,j)$ = Concentration of bed material carried by $Q_{ir}(i,j)$ in ppm by wt.

$Q_s(i,j)$ = Seepage and evaporation loss from $CH(i,j)$ in cfs.

$G_c(i,j)$ = Average quantity of bed material removed from $CH(i,j)$ by mechanical means in lbs/sec.

The summation for $\underline{i}$ and $\underline{j}$ is over the subset N of the channels supplied by and including $CH(K,L)$, and the summation for $\underline{ir}$ is over all irrigation diversions from $CH(i,j)$.

Equation (4.1a) defines the condition for mass continuity with respect to water within the system. Turnout discharges ($Q_{ir}(i,j)$) are determined from irrigation requirements and seepage and evaporation losses ($Q_s(i,j)$) from the characteristics of the area through which the channels are constructed. $Q(K,L)$ is therefore uniquely determined for all values of $\underline{K}$ and $\underline{L}$, and in general, not subject to manipulation in the application of Eqs. (4.2).

Equations (4.2) define the condition for mass continuity with respect to bed material sediment within the system. With $Q_{ir}(i,j)$ and $Q(i,j)$ taken as given for all $\underline{i}$ and $\underline{j}$, $G(i,j)$, $C_{ir}(i,j)$, and $G_c(i,j)$ may be varied within feasible limits so as to satisfy the required relationship. After performing the indicated summations, Eq. (4.2a) may be viewed as a single equation in three unknowns which must be satisfied for all channels within the system. To arrive at a unique solution, two of the unknown values must be determined from other considerations.

Considerations influencing the economics of sediment disposal with the irrigation supplies and by mechanical bed clearance have already been discussed. In addition, recent investigations concerning the design of farm turnout structures [67,76,81] indicate a practical upper limit on bed material concentrations in irrigation discharges of approximately four times the concentration in the main channel. In terms of previously defined variables, this limitation may be expressed as:

$$C_{ir}(i,j) \leq 4.0\ C(i,j) \tag{4.3}$$

In order to determine bed material transport capacities (G(K,L)) using the concepts developed in Chapter III, it is first necessary to determine the nature of the bed material. Initially, the bed material of an unlined channel is determined by the material through which the channel is constructed. If, however, the initial boundary material is finer than the incoming sediment, or if the final channel bed is formed by aggradation, the bed material will change in time to reflect the nature of the transported material. At present there is little information available to assist the designer in predicting the nature of the final bed material from sediment inflow data. Sediment transport concepts may be applied in combination with judgment and experience to arrive at design values.

In general, the final bed material may be expected to be slightly coarser and to have a slightly smaller gradation coefficient than the incoming sediment [67]. Investigations have shown that hydraulically formed sand mixtures will have an approximately log-normal gradation [4] with the mean size decreasing with distance from the headworks. This tendency helps to offset the decrease in transport capacity associated with decreasing discharge.

The variation in size of bed material with distance from the channel head has been found to follow the general relation:

$$d_{50} = (d_{50})_0 e^{-\alpha x} \tag{4.4}$$

in which $(d_{50})_0$ is the geometric mean diameter at a reference section in the channel, and d_{50} is the geometric mean diameter at some distance x downstream. α appears to be a function of the sediment transport characteristics of the flow and, in general, increases with decreasing discharge [67].

Rana [79] studied the variation of size with flow distance using a model based on Einstein's transport function [23]. The values of α obtained varied widely with an order of magnitude of approximately 0.002/mile. In systems with irrigation turnouts which draw relatively high concentrations of coarse material, α may be expected to be slightly greater. At the present time, no direct method for the determination of α for design is known. Analysis of similar systems on the basis of previously discussed concepts forms the best available guide.

Sediment Routing Models

Mahmood [66,67,68] applied the relations given as Eqs. (4.1) and (4.2) to the design of the simulated irrigation canal system shown in Fig. 4.1. Characteristics of the system, which was fashioned to be similar to those currently operating in Pakistan, are given in Tables 4.1 and 4.2.

Table 4.1. Overall Characteristics of Simulated Canal System (After Mahmood [68]).

Discharge at Headworks	5000 cfs
Total Length of Channels (10 mi per channel)	1100 mi
Irrigation Diversions at Turnouts	3388 cfs
Seepage Loss (8 cfs per 10^6 ft^2 channel perimeter)	1612 cfs
Irrigable Area	1.13×10^6 acres
Irrigation Channel Density	0.47 mi/mi^2

Table 4.2. Individual Channel Characteristics for Simulated Canal System (after Mahmood [68]).

Channel Discharge Q(i,j) cfs	Rate of Irrigation Diversions cfs/mi	Bed-sediment Size Reduction Exponent, α $mile^{-1}$
> 2500	0	0.0030
1200 - 2500	1	0.0035
500 - 1200	2	0.0040
50 - 500	3	0.0045
< 50	4	0.0050

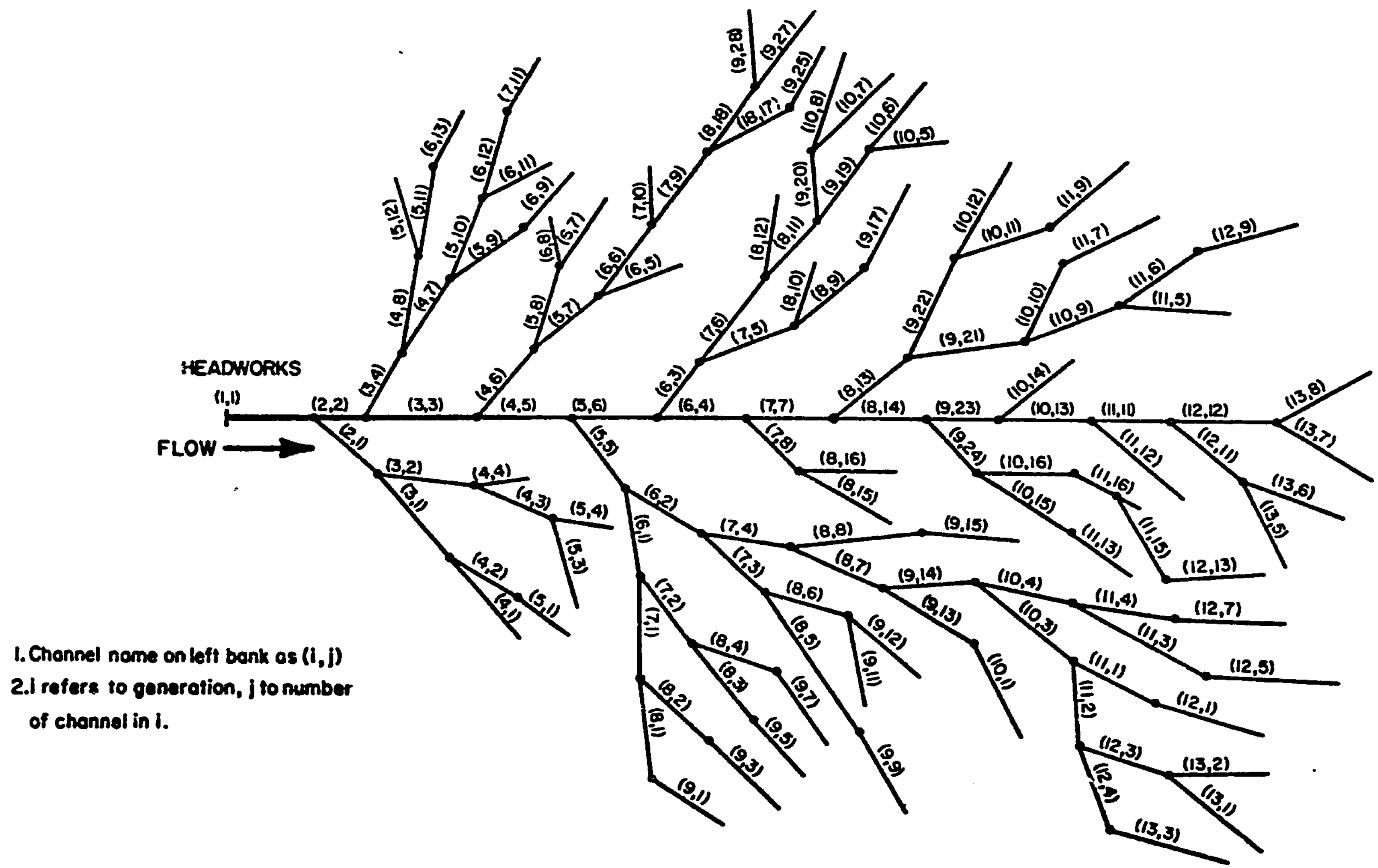

Figure 4.1. Simulated Irrigation Canal System with 5,000 cfs Discharge at Headworks (after Mahmood [67].

Application of Eq. 4.2a to the simulated system was carried out by assuming all of the transported sediment was to be disposed of through the irrigation supplies ($G_c(i,j) = 0$, for all $\underline{i}$ and $\underline{j}$), and assigning relative sediment concentrations to the farm turnouts a priori. Equation 4.2a was then solved for the required bed material transport of each channel and the channels designed accordingly. Size and gradation of the bed material at the headworks was assumed known.

In order to assign sediment discharge concentrations in the turnouts and evaluate the system on a relative basis, it was necessary to define the following additional parameters.

1) The required average concentration in irrigation diversions,

$$\bar{C}_I = C(1,1) \frac{(1 - g_c/100)}{(1 - s/100)} \qquad (4.5a)$$

in which s is the percent of the head discharge which is lost to seepage and evaporation, $\underline{g_c}$ is the percent of entering bed material sediment removed by mechanical means, and $C(1,1)$ is the bed material concentration at the head of the system,

2) Weight factors describing the relative bed material concentration in individual turnouts. $W_{ir}(i,j)$ is defined so as to satisfy the relation;

$$C_{ir}(i,j) = W_{ir}(i,j) \cdot \left[\frac{\sum Q_{ir}(i,j)}{\sum W_{ir}(i,j) \cdot Q_{ir}(i,j)}\right] \bar{C}_I \qquad (4.5b)$$

in which the summations are carried out over all irrigation diversions in the system.

3) The ratio of bed material concentration in irrigation supplies to the concentration at the head of the system,

$$C_{ir_R}(i,j) = \frac{C_{ir}(i,j)}{C(1,1)} \tag{4.5c}$$

4) The ratio of bed material concentration in irrigation supplies to that in the channel from which they are diverted,

$$C_{ir_{RAT}}(i,j) = \frac{C_{ir}(i,j)}{C(i,j)} \tag{4.5d}$$

and

5) The ratio of bed material concentration in channel CH(i,j) to that at the head of the system,

$$C_R(i,j) = \frac{C(i,j)}{C(1,1)} \tag{4.5e}$$

The simulated system was evaluated for various combinations of weighting factors, $W_{ir}(i,j)$. It was found impractical to design channels to carry the required sediment loads with $W_{ir}(i,j) = 1.0$ ($C_{ir}(i,j) = \overline{C}_I$) for all i, j, and ir. The system of weighting factors which was found to "yield a better overall design" is shown in Table 4.3. Under this system, $C_{ir}(i,j)$ is a function only of the required average irrigation discharge concentration for the system, and the water discharge of the channel farm which the irrigation supply is diverted.

Application of the weighting factors given in Table 4.3 to the simulated system resulted in the following range of sediment concentration ratios.

$$1.00 \leq C_R \leq 1.22$$

$$1.15 \leq C_{ir_R} \leq 1.91$$

$$1.15 \leq C_{ir_{RAT}} \leq 1.86$$

Table 4.3. Selected Weighting Factors for Bed Sediment Concentrations in Irrigation Diversions (after Mahmood [68].

Discharge at Channel Head Q(i,j) cfs	Weight Factor to Apportion Bed Material Concentrations $W_{ir}(i,j)$
> 200	1.00
100 - 200	0.90
40 - 100	0.75
20 - 40	0.60
< 20	0.50

Bed material transport capacity (G(i,j)) of the individual channels within the system was computed using Mahmood's transport function [69] described in Chapter III. Flow resistance and channel dimensions were determined using the regime relationships of Lacey [47] with the coefficients allowed to vary. These relations restated in variable coefficient form as used by Mahmood [66,67,68], are:

$$F_{vr} = a_1 \sqrt{d_{50}}$$

$$V = a_2 \sqrt{f_{vr} R}$$

$$P = a_3 \sqrt{Q}$$

$$f_{sq} = a_4 f_{vr}$$

$$S = 5.47 \times 10^{-4} Q^{-1/6} f_{sq}^{\ 5/3}$$

In using these relations to design channels of the simulated system to carry the required bed material loads, coefficient a_1 was allowed to vary from 1.76 to 2.60 with coefficients a_2, a_3, and a_4 assigned values of 1.15, 2.67, and 1.10 respectively. (For a discussion regarding the use of the Lacey relations in this manner the reader is referred to Ref. 70.)

Application of Sediment Routing Models

Although Mahmood's investigations were limited to a simulated system, it would appear possible to apply the model developed to a real irrigation canal system design problem directly by carrying out the steps outlined below.

1) Determine the general layout of the system, locations of water use, quantity of water required at each location, and estimated seepage and evaporation losses from each channel.

2) Select the computational techniques considered most applicable to the system for the determination of channel geometry and bed material transport capacity. Concepts discussed in Chapters II and III combined with analysis of available data from similar systems will provide a guide to the selection of applicable methods.

3) Apply Eq. (4.1a) to determine the required water discharge at the headworks for each channel within the system.

4) Determine design values for quantity and quality of incoming sediment from water and sediment hydrographs of

the parent channel and tentative headworks design computations.

5) Determine mean diameter and gradation of the bed material at the headworks and the anticipated variation of bed material properties within the system. Simple methods for the determination of these variables are not available. However, their determination should be compatible with assumptions inherent in Step 2.

6) Select disposal areas, if any, for sediment removed by bed clearance and desilting works, and estimate the rate at which bed sediments are to be delivered to these areas.

7) Select weighting factors for the relative allocation of sediment to irrigation diversions. Values given in Table 4.3 may provide a guide if channel discharges are expressed as a percent of head discharge, but final selection should consider the ability or inability of the lands within the system to absorb bed sediments.

8) Compute the required average concentration in irrigation supplies from Eq. (4.5a).

9) Compute the concentration in individual irrigation turnouts from Eq. (4.5b).

10) Compute the required bed material transport capacity of each channel from Eq. (4.2a).

11) Design the channels within the system to carry the bed material load computed in Step 10. For some channels, it may not be possible to design a stable section with the

required transport capacity. In this case, modification of the values selected in Steps 6 and/or 7 is required with subsequent steps repeated.

12) Analyze and modify the design as necessary based on the economics and objectives of the specific system.

13) Design turnout and bifurcation structures to obtain the design sediment distributions.

One problem which may be encountered in extending the results of Mahmood's investigation from the simulated system to a real system results from the fact that energy slopes in the real system are often confined between narrow limits, whereas the model developed assumes the slope to be variable within the limits of channel stability. This becomes significant when it is realized that slope of the energy grade line is the single most important geometric parameter governing channel transport capacity. It is suggested, however, that by changing the roles of the dependent and independent variables in Eq. (4.2a), the model may be applied to the analysis of existing systems and to the design of systems in which the hydraulic gradient is strictly limited.

Analysis of an existing system is more easily carried out if a a slight change is made in the form of Eqs. (4.1) and (4.2). By successive application to channels within the system, these relations may be rewritten as:

$$Q(K,L) = \sum_{ir \varepsilon M} Q_{ir}(K,L) + \sum_{i,j \varepsilon n} Q(i,j) + Q_s(K,L) \qquad (4.1c)$$

$$G(K,L) = \sum_{ir \varepsilon M} Q_{ir}(K,L) \cdot C_{ir}(K,L) 62.4 \times 10^{-6} + \sum_{i,j \varepsilon n} G(i,j) + G_c(K,L) \qquad (4.2c)$$

where the summation for $\underline{i}$ and $\underline{j}$ is over the subset $\underline{n}$ of the channels which offtake directly from channel CH(K,L) and all other variables are as previously defined.

By treating the channel transport capacity G(i,j) as known between relatively narrow limits, Eq. (4.2c) may be applied iteratively to determine the best possible sediment disposal scheme within the limits of the system. Application of the principles in this fashion may be carried out by replacing Steps 7 through 11 as previously outlined by the following alternate steps.

7a) Determine feasible upper and lower limits on the energy gradient for each channel.

8a) Develop a trial design of the system based on feasible channel slopes. Relations such as that developed by Bose [10] (Eq. (2.11d)) may be used as a guide for the selection of individual channel slopes in the design situation. For a more equitable distribution of sediment among water users, the larger channels will normally require slightly flatter slopes than indicated by regime relations, while the smaller channels will normally require steeper slopes.

9a) Compute the bed material transport capacity of each channel using methods selected in step 2.

10a) Determine the quantity of sediment which is to be disposed of in irrigation supplies diverted from each channel using Eq. (4.2c).

11a) Distribute the quantities computed in Step 10a to the water users according to the ability of the water courses and the land to handle bed sediments.

After the first trial design has been evaluated, it may be possible to use these results to develop a sediment disposal scheme judged to lie within the limits of the available energy gradients and proceed with the steps initially outlined to arrive at the final design. In determining such a scheme, use may be made of the Lacey-Inglis slope relation [39] (Eq. 2.14e) to approximate bed material concentration limits from slope limitations. For a channel with constant discharge and bed material characteristics, this relation may be expressed as:

$$C \propto S^{12/5}$$

and may be applied in ratio form. So long as this relation is treated as being only approximate, it may be used effectively regardless of the computational techniques selected in Step 2 of the outlined procedure.

A sample problem illustrating the compuations required for a trial design using the alternate steps is given in Appendix C.

CHAPTER V

SUMMARY AND RECOMMENDATIONS FOR FURTHER STUDY

The object of this study was to develop a rational approach to the solution of sediment problems associated with branching canal systems. Such a generalized approach for the routing of sediment through a system has been presented in Chapter IV based on existing concepts and computational procedures. Due to the complexity of the interaction of the flow with boundary materials, a "cookbook" solution to the problem is not considered feasible at this time. Selection and application of specific computational methods for channel design and sediment transport capacity estimation is dependent on field conditions unique to the specific problem as judged by the engineer.

To assist the engineer in making this selection, a significant portion of the text has been devoted to the presentation of the concepts and data on which the various computational procedures are based. Since different computational techniques may lead to significantly different results, the selection of an approach compatible with field conditions is essential for successful application of the routing procedure. Whenever possible, data from similar systems in the local area should be used to aid in the selection process. In general, no single approach may be considered "right" or "wrong" for all applications. Rather, the data and assumptions on which a method is based should be examined to determine similarity to the problem at hand. The more recent computational techniques do, however, have the advantage of having incorporated into them later contributions to the understanding of the interaction of the flow and sediment. As further advances are made in this area, these too should be incorporated into the design procedure.

Certain specific areas in which further study is needed may be identified through examination of the outlined sediment routing procedure. Perhaps the most critical of these is the determination of bed material size and gradation and its variation within the system. Both flow resistance and bed material transport capacity are significantly influenced by the nature of the bed material.

In cases where the material through which the channel is constructed is finer than the transported sediment, or where the channel is formed through aggradation, the bed material will adjust in time to reflect the nature of the incoming sediment. Very little information is available to assist the engineer in estimating either the final bed material size and gradation or the time required for the adjustment. Further study relating these variables to the quantity and quality of incoming sediment is needed.

The related problem of estimating the variation in bed material characteristics with location in the system also needs study. The works of Rana [82], Rafay [79], and others provide a base from which to work. These studies have not, however, resulted in relations which are easily applied to design problems.

In order to be directly applicable to irrigation canal systems, the influence of selective sediment withdrawal from turnouts on size and gradation variations would also need to be considered. Since these problems are directly related to bed material transport, such studies could tend to contribute significantly to the knowledge base in this area.

In Chapter IV, the economics of sediment disposal was discussed in general terms. A model through which the economic values of water, conveyance channels, channel lining, desilting works, etc. could be accounted for more directly would be a valuable tool in canal system design. Such a model would necessarily be general in nature due to the large variation in the relative economic value of the variables from one location to another.

In application of the sediment continuity relations, the assumptions of steady flow and constant discharge were made. Further study is needed to determine the effects of unsteady flow and intermittent operation on the relative transport characteristics of channels within a system.

REFERENCE LITERATURE

1* Alghita, B. K., "The Problem of Freeboard in Irrigation Channels, Drainage Channels, Embankments, Levees, and Reservoirs," Transactions of the Second Congress on Irrigation and Drainage, R4Q3, 1954.

2** Anderson, A. G., et al., "Tentative Design Procedure of Riprap Lined Channels," Project Report No. 96, St. Anthony Falls Hydraulic Lab., Minneapolis, Minnesota, NCHRP Report 108.

3* ASCE Task Committee on Preparation of Sedimentation Manual, 1972, "Sediment Control Methods: B. Stream Channels," Journal of the Hydraulics Division, ASCE, Vol. 98, No. HY7. Proceedings Paper 9071, July 1972, pp. 1295-1326.

4 Blench, T., Mobile-Bed Fluviology, The University of Alberta Press, Edmonton, Alberta, Canada, 1969.

5 Blench, T., Regime Behavior of Canals and Rivers, London, Butterworths Scientific Publications, 1957.

6* Blench, T., "Regime Theory for Self-Formed Sediment-Bearing Channels," Transactions of the American Society of Civil Engineers, Vol. 117, 1952, pp. 383-408.

7* Blench, T., Discussion of "River Channel Roughness" by Einstein and Barbarossa, Transactions of the American Society of Civil Engineers, Vol. 117, 1952.

8 Blench, T. and King, C., "Effect of Dynamic Shape on Lacey Relations," Central Board of Irrigation, India, Publication No. 27, Annual Report, 1941.

9 Bogardi, Janos, Sediment Transport in Alluvial Streams, Akademiai Kiado, Budapest, 1974.

10 Bose, N. K. and Malhotra, J. K., "An Investigation of the Interrelation of Silt Indices and Discharge Elements for Some Regime Channels in the Punjab," Punjab Irrigation Research Institute, Research Publication, Vol. II, No. 23, 1939.

11* Brooks, N. H., "Boundary Shear Stress in Curved Channels," Proceedings of the ASCE, Journal of the Hydraulics Division, Vol. 89, No. HY3, 1963.

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* Item reviewed, but not referenced directly in text.

** Item unavailable for review at the time of writing, but may contain relevant information.

12** Brooks, N. H., "Mechanics of Streams with Movable Beds of Fine Sand," Transactions of the American Society of Civil Engineers, Vol. 123, 1958.

13* Chitale, S. V., "Design of Alluvial Channels," Transactions of the Sixth Congress of the International Commission on Irrigation and Drainage, Vol. 3, 1966, pp. 20.363-20.427.

14 Chitale, S. V., Hydraulics of Stable Channels, Central Water and Power Research Station, Poona, India, Government of India, Ministry of Irrigation and Power, Central Water and Power Commission, 1966.

15 Chow, V. T., Open Channel Hydraulics, McGraw-Hill Book Company, New York, 1959.

16 Colby, B. R., "Practical Computations of Bed Material Discharge," Journal of the Hydraulics Division, ASCE, Vol. 90, No. HY2, 1964.

17 Colby, B. R. and Hembree, C. H., "Computation of Total Sediment Discharge, Niobrara River near Cody, Nebraska," U.S. Geological Survey, Water Supply Paper 1357, 1955.

18* Doubt, P. D., "Design of Stable Channels in Erodible Materials," Proceedings of the Federal Inter-Agency Sedimentation Conference, Miscellaneous Publication No. 970, Agricultural Research Service, USDA, 1963, Paper No. 43, pp. 373-376.

19* Doubt, P. D., "Stabilized Channels," Agricultural Engineering, Vol. 43, No. 2, February 1962, pp. 76-85.

20** duBoys, M. P., "The Rhone and Streams with Movable Beds," Annales des Ponts et Chaussées, Vol. XVII, 1879, cited by Lane, ref. #52.

21** DuBuat, L. G., "Principles d'Hydraulique," Paris, 1786, cited by Bogardi, Ref. #9.

22 Einstein, H. A. and Barbarossa, N. L., "River Channel Roughness," Transactions of the American Society of Civil Engineers, 1952, Vol. 117, p. 1121.

23 Einstein, H. A., "The Bed Load Function for Sediment Transportation in Open Channel Flows," USDA Technical Bulletin No. 1026, September 1950.

24 Engineering Hydraulics, Proceedings of the Fourth Hydraulics Conference, Iowa Institute of Hydraulic Research, June 12-15, 1949, edited by Hunter Rouse, John Wiley and Sons, Inc., New York, 1950.

25 Etcheverry, E. A., Irrigation Practice and Engineering, Vol. II, "Conveyance of Water," New York, McGraw-Hill Book Company, 1915.

26 Federal Highway Administration, "Highways in the River Environment, Hydraulic and Environmental Design Considerations," Federal Highway Administration, National Highway Institute, Office of Research and Development, Office of Engineering, 1974.

27** Flaxman, E. M., "Channel Stability in Undisturbed Cohesive Soils," Proceedings of the ASCE, Journal of the Hydraulics Division, Vol. 89, No. HY2, Pt. 1, Paper 3462, March 1963, pp. 87-96.

28 Fortier, S. and Scobey, F. C., "Permissable Canal Velocities," Transactions of the American Society of Civil Engineers, Vol. 89, 1926.

29** Ganguillet, E. and Kutter, W. R., A General Formula for the Uniform Flow of Water in Rivers and Other Channels, John Wiley and Sons, New York, 1889, cited by Haynie, ref. #35.

30* Garde, R. J. and Rangarajan, K. G., "Analysis of Resistance to Flow over Alluvial Beds," Transactions of the Sixth Congress of the International Commission on Irrigation and Drainage, Vol. 3, 1966, p. 20.113.

31** Garde, R. J. and Rangarajan, K. G., "Regime Criteria for Alluvial Streams," Proceedings of the ASCE, Journal of the Hydraulics Division, Vol. 89, No. HY6, November 1963.

32** Glover, R. E. and Florey, Q. L., "Stable Channel Profiles," U.S. Bureau of Reclamation, Hydraulics Laboratory Report, Hyd. 325, 1955.

33* Green, K. D., "Considerations Affecting the Freeboard Problem in Irrigation Channels with Special Reference to Present Practice in Victoria," Transactions of the Second Congress on Irrigation and Drainage, R2Q3, 1954.

34 Haynie, R. M. and Simons, D. B., "Design of Stable Channels in Alluvial Materials," Proceedings of the ASCE, Journal of the Hydraulics Division, Vol. 94, No. HY6, Paper No. 6217, November 1968.

35 Haynie, R. M., "Design of Stable Channels in Alluvial Material," Ph.D. Dissertation, Colorado State University, Fort Collins, Colorado, June 1964.

36 Henderson, F. M., Open Channel Flow, The Macmillan Company, New York, 1966.

37 Henderson, F. M., "Stability of Alluvial Channels," Transactions of the American Society of Civil Engineers, Vol. 128, Paper No. 3440, 1963.

38 Inglis, C. C., Discussion of "Uniform Water Conveyance Channels in Alluvial Material," by D. B. Simons and M. L. Albertson, Transactions of the American Society of Civil Engineers, Vol. 128, 1963.

39 Inglis, C. C., "Historical Note on Empirical Equations Developed by Engineers in India for Flow of Water and Sand in Alluvial Channels," IAHSR, Second Meeting, Stockholm, 1948.

40** Ippen, A. T. and Drinker, P. A., "Boundary Shear Stress in Trapezoidal Channels," Proceedings of the ASCE, Journal of the Hydraulics Division, No. HY5, September 1962.

41** Joglekar, D. V. and Gole, C. V., "Sand Control of Channels Taking off from Alluvial Rivers," Proceedings of the Regional Technical Conference on Water Resources Development in Asia and the Far East, Flood Control Series No. 9, 1956.

42* Kennedy, J. F., "Laboratory Study of an Alluvial Stream at Constant Discharge," Proceedings of the Federal Inter-Agency Sedimentation Conference, 1963, USDA, Agricultural Research Service, Miscellaneous Publication 970.

43** Kennedy, R. G., "The Prevention of Silting in Irrigation Canals," Proceedings, Institution of Civil Engineers, Vol. CXIX.18, cited by Blench, ref. #4.

44* Lacey, G., "Sediment as a Factor in the Design of Unlined Irrigation Canals," Transactions of the Sixth Congress of the International Commission on Irrigation and Drainage, No. 3, 1966, pp. 20.1-20.20.

45 Lacey, G., "Flow in Alluvial Channels with Mobile Sand Beds," Proceedings of the Institute of Civil Engineers, Vol. 9, February 1958.

46 Lacey, G., "The Problem of Freeboard in Irrigation and Drainage Channels," Transactions of the Second Congress on Irrigation and Drainage, R1Q3, 1954.

47 Lacey, G.,"A General Theory of Flow in Alluvium," Journal, Institution of Civil Engineers, Vol. 27, 1946.

48 Lacey, G., "Uniform Flow in Alluvial Rivers and Canals," Minutes of Proceedings of the Institution of Civil Engineers, Vol. 237, Pt. 1, p. 421, 1933-34.

49 Lacey, G., "Regime Flow in Incoherent Alluvium," Control Board of Irrigation, India, Publication No. 20, 1939-40.

50 Lacey, G., "Stable Channels in Alluvium," Institution of Civil Engineers, Minutes of Proceedings, Vol. 229, p. 33, 1929.

51 Lane, E. W., "Stable Channels in Erodible Material," Transactions of the American Society of Civil Engineers, Vol. 102, Paper No. 1957, 1937.

52 Lane, E. W., "Design of Stable Channels," Transactions of the American Society of Civil Engineers, Vol. 120, pp. 1234-1260, 1955.

53 Lane, E. W., "Change as a Factor in Stable Irrigation Canals," Journal of the Central Board of Irrigation and Power (India), Vol. 12, No. 2, April 1955.

54 Lane, E. W. and Carlson, E. J., "Some Factors Affecting the Stability of Canals Constructed in Coarse Granular Materials," Proceedings of the Minnesota International Hydraulics Convention, 1953.

55 Lane, E. W., "Progress Report on Results of Studies on Design of Stable Channels," U.S. Department of the Interior, Bureau of Reclamation Hydraulic Report No. Hyd-352, June 1952.

56* Laursen, E. M., "Scour at Bridge Crossings," Proceedings of the ASCE, Journal of the Hydraulics Division, Vol. 86, No. HY2, February 1960.

57** Leliavsky, S., An Introduction to Fluvial Hydraulics, London, Constable and Company, 1955.

58 Leopold, B. L., Wolman, M. G., and Miller, J. P., Fluvial Processes in Geomorphology, W. H. Freeman and Company, San Francisco, California, 1964.

59** Lindley, E. S., "Regime Channels," Punjab Engineering Congress 7, 1919, cited by Bogardi, ref. #9.

60 Liu, M. H. and Hwang, S., "Discharge Formula for Straight Alluvial Channels," Proceedings of the ASCE, Vol. 85, No. HY11, November 1959.

61** Maddock, T., Jr., "Intermediate Hydraulics of Alluvial Channels," Proceedings of the ASCE, Journal of the Hydraulics Division, Vol. 96, No. HY11, November 1970.

62 Mahmood, K. and Ponce, V. M., "Computer Programs for Sediment Transport," Colorado State University, Engineering Research Center, Fort Collins, Colorado, 1975.

63* Mahmood, K., "Flow through Vortex Tube Sediment Ejectors," Reprinted from the Proceedings of the ASCE Irrigation and Drainage Division Specialty Conference, Logan, Utah, August 1975.

64* Mahmood, K., "Mathematical Modeling of Morphological Transients in Sandbed Canals," Proceedings, XVIth Congress, IAHR, Sao Paulo, Brazil, 1975.

65 Mahmood, K., "Variation of Regime Coefficients in Pakistan Canals," Meeting Reprint No. 2039, ASCE National Transportation Engineering Meeting, Tulsa, Oklahoma, 1973.

66 Mahmood, K., "Sediment Routing in Irrigation Canal Systems," Meeting Reprint No. 1905, ASCE National Water Resources Engineering Meeting, Washington, D.C., 1973.

67 Mahmood, K., "Planning Sediment Distribution in Surface Irrigation Systems," CUSUSWASH Water Management Technical Report No. 26, Colorado State University, Fort Collins, Colorado, 1973.

68 Mahmood, K., "Sediment Equilibrium Considerations in the Design of Irrigation Canal Networks," Paper No. A-65, International Symposium on River Mechanics, International Association of Hydraulic Research, Bangkok, 1973.

69 Mahmood, K., "Flow in Sand Bed Channels," CUSUSWASH Water Management Technical Report No. 11, Colorado State University, Fort Collins, Colorado, 1971.

70 Mahmood, K. and Shen, H. W., "The Regime Concept of Sediment Transporting Canals and Rivers," Chaper 30, River Mechanics, Vol. II, edited by H. W. Shen, Fort Collins, Colorado, 1971.

71* Malhotra, J. K., "Comparison of the Barnes, Kutter, Lacey, and Manning Rugosity Coefficients for 42 Regularly Observed Canal Sites in the Punjab," Central Board of Irrigation, India, Publication No. 27, Annual Report 1941.

72** Manning, R., "On Flow of Water in Open Channels and Pipes," Institution of Civil Engineers, Ireland, Transactions, 1890, cited by Haynie, ref. #35.

73* Masch, F. D., Espey, W. H., Jr. and Moore, W. L., "Measurements of the Shear Resistance of Cohesive Soil," Proceedings of the Federal Inter-Agency Sedimentation Conference, USDA, Agricultural Research Service, Miscellaneous Publication 970, 1973.

74 Melone, A. M., Richardson, E. V., and Simons, D. B., "Exclusion and Ejection of Sediment from Canals," Colorado State University, Fort Collins, Colorado, April 1975.

75 Meyer-Peter, E. and Müller, R., "Formulas for Bed-Load Transport," Proceedings, 2nd Meeting of International Association Hydraulic Research, Stockholm, Sweden, 1948.

76 Nazar, A., "A Laboratory Study of Bed Material Withdrawal in Farm Turnouts," M.S. Thesis, Colorado State University, Fort Collins, Colorado, 1973.

77** Neill, C. R., "Mean Velocity Criterion for Scour of Coarse Uniform Bed Material," IAHR, 12th Congress, Fort Collins, Colorado, 1967.

78* Parsons, D. A., "Vegetative Control of Streambank Erosion," Proceedings of the Federal Inter-Agency Sedimentation Conference, 1963, USDA, Agricultural Research Service, Miscellaneous Publication 970.

79 Rafay, T., "Analysis of Change in Size of Bed Materials along Alluvial Channels," M.S. Thesis, Colorado State University, Fort Collins, Colorado, 1964.

80 Raju, B. C., "Correlation of Regime Theory and Tractive Force Theories of Stable Channel Design," Master's Report, Colorado A&M, 1955.

81 Rakha, A., "Sediment Conduction of Turnouts," M.S. Thesis, Colorado State University, Fort Collins, Colorado, 1971.

82 Rana, S. A., "Sediment Sorting in Alluvial Channels," M.S. Thesis, Colorado State University, Fort Collins, Colorado, 1971.

83** Raudkiwi, A. J., Loose Boundary Hydraulics, The Commonwealth and International Library, Publisher, R. Maxwell.

84 Rouse, H., "Modern Conceptions of the Mechanics of Fluid Turbulence," Transactions of the American Society of Civil Engineers, Vol. 102, 1937.

85 Rouse, H. and Ince, S., History of Hydraulics, Dover Publications, Inc., New York, 1957.

86* Sayre, W. W. and Albertson, M. L., "Roughness Spacing in Rigid Open Channels," Transactions of the American Society of Civil Engineers, Vol. 128, pp. 343-371, 1963.

87* Schiller, Robert E., "A Study of Sand Transport in the Indus Basin Canals," Ph.D. Dissertation, Colorado State University, 1969.

88** Schumm, S. A., "The Shape of Alluvial Channels in Relation to Sediment Type," U.S. Geological Survey, Professional Paper 352-B, 1960.

89 Sedimentation, Symposium to Honor Professor H. A. Einstein, Edited and published by H. W. Shen, Colorado State University, 1972.

90 Simons, D. B., Notes from Classroom Instruction, CE 812, Sedimentation, Colorado State University, 1974.

91 Simons, D. B., Stevens, M. A., and Duke, J. H., Jr., "Predicting Stages on Sand-Bed Rivers," ASCE, Journal of the Waterways, Harbors, and Coastal Engineering Division, Vol. 99, No. WW2, May 1973.

92 Simons, D. B. and Richardson, E. V., "Resistance to Flow in Alluvial Channels," U.S. Geological Survey Professional Paper 422-J, 1966.

93* Simons, D. B. and Miller, C. R., "Sediment Discharge in Irrigation Canals," Transactions of the Sixth Congress of the International Commission on Irrigation and Drainage, No. 3, pp. 20.275-20.307, 1966.

94 Simons, D. B. and Richardson, E. V., "Forms of Bed Roughness in Alluvial Channels," Transactions of the American Society of Civil Engineers, Vol. 128, 1963.

95 Simons, D. B. and Albertson, M. L., "Uniform Water Conveyance Channels in Alluvial Material," Transactions of the American Society of Civil Engineers, Vol. 128, 1963.

96 Simons, D. B. and Richardson, E. V., "A Study of Variables Affecting Flow Characteristics and Sediment Transport in Alluvial Channels," Proceedings of the Federal Inter-Agency Sedimentation Conference, USDA, Agricultural Research Service, Miscellaneous Publication No. 970, 1963.

97 Simons, D. B., "Theory and Design of Stable Channels in Alluvial Materials," Ph.D. Dissertation, Colorado State University, Fort Collins, Colorado, 1957.

98* Singh, B., "Some Implications of Regime Design of Channels," Transactions of the Sixth Congress of the International Commission on Irrigation and Drainage, No. 3, pp. 20.96-20.11, 1966.

99** Steed, C. V., "Hyperbolic Channel Bends," U.S. Army, Corps of Engineers, Civil Works Bulletin 56-15, August 3, 1956.

100 U.S. Bureau of Reclamation, "Canals and Related Structures," Design Standards No. 3, 1961.

101** Vanoni, V. A., Brooks, N. H., and Kennedy, J. F., "Lecture Notes on Sediment Transport and Channel Stability," Report No. KH-R-1, California Institute of Technology, W. M. Keck Laboratory, January 1961.

102* Vanoni, V. A. and Nomicos, G. N., "Resistance Properties of Sediment Laden Streams," Transactions of the American Society of Civil Engineers, Vol. 125, 1960.

103* Vanoni, V. A. and Brooks, N. H., "Laboratory Studies of the Roughness and Suspended Load of Alluvial Streams," California Institute of Technology, Sedimentation Laboratory, Report No. E68, 1957.

APPENDIX A

DESIGN BY VELOCITY OR DEPTH CORRECTION

APPENDIX A

DESIGN BY VELOCITY OR DEPTH CORRECTION

The design methods presented by Haynie [34,35] and by Simons and Richardson [92] are similar both in approach and in results obtained. Both are primarily applicable to sand bed channels operating in the dune phase of flow. Their development appears to have been, to some extent, complimentary with much of the same data used in the development of each. The steps to design presented along with the required figures are taken directly from the original works. The terminology used is that defined in the list of symbols and definitions are not repeated here. Figures A-1 to A-3, taken from the work of Simons and Albertson [95], are common to both methods as is Fig. 2.3 from Simons and Richardson [94].

Initial data required for both methods are the discharge (Q), the median size of bed material (d_{50}), the type of bed and bank material, and the kinematic viscosity (ν). In determining the type of bed and bank material and size of bed material, conditions in the parent channel and at the headworks should be considered to determine possible changes in channel character during the aging process. In situations where a high concentration of fine sediment exists, the apparent viscosity should reflect its influence. Changes in bed and/or bank material along the route may require repeated application of the procedure with transitions designed between each reach.

In the methods as originally presented, the initial estimate of the slope is left to the designer based only on experience and the requirements of local topography. In some cases, the number of iterations required may be reduced by estimating the slope with the aid of regime relations. The relation developed by Bose [10] is convenient for this purpose since no additional data is required. This relation, repeated here for convenience, is:

$$S = 2.09\ d_{50}^{0.86}/(1000\ Q^{0.21})$$

<u>Haynie's Design Procedure</u>

1) With Q known a tentative value of hydraulic radius (R) is selected from the plot of R vs Q (Fig. A-1).
2) Using this value of R, select a value of depth (D) from the plot of R vs D (Fig. A-2).
3) Select an initial trial slope (S) based on experience, the sediment load to be transported, the slope of the surrounding terrain, the slope of existing canals which are operating successfully at the selected R, etc.
4) From the plot of velocity correction (ΔV) vs R (Fig. A-4) using the selected values of R and S, select the value of ΔV.
5) Compute shear velocity $(V_*) = \sqrt{gRS}$.
6) Compute $\frac{\Delta V}{V_*}$.
7) Compute the shear Reynolds number $\frac{(V_* D)}{\nu}$ and using this value go to the plot of $\frac{V}{V_*}$ vs. log $\frac{V_* D}{\nu}$ (Fig. A-5) and from the curve for a smooth boundary select the corresponding value of $\frac{V'}{V_*}$.
8) Compute $\frac{V}{V_*} = \frac{V'}{V_*} - \frac{\Delta V}{V_*}$ and then compute V, the average velocity to be expected in the channel being designed.
9) Compute the stream power $(\tau V = \gamma VDS)$.

10) Using the stream power (τV) and the d_{50} of the bed material go to the plot of τV vs. d_{50} (Fig. 2-3) and see whether the channel designed will be in the regime in which dunes exist. If so, proceed to the next step; if not, return to step 2 and select a new R and/or S and repeat the design procedure.

11) Using the values of D, S, and V obtained compute the width of the channel under design.

Simons and Richardson's Design Procedure

1) Determine R from Q (Fig. A-1).
2) Determine A from Q (Fig. A-3).
3) Compute V from $V = Q/A$.
4) Assume an initial value of S_o, (determined from physical constraints) and using R obtained above, determine hydraulic radius adjustment (ΔR) from Fig. A-6.
5) Compute $\Delta R/R$.
6) Compute $V_* = \sqrt{g\,D\,S_o}$ with $D = R$, $\therefore V_* = \sqrt{g\,R\,S_o}$.
7) Compute shear Reynolds number $\mathbb{R}_* = V_* D/\nu$ with $D = R$ $\therefore \mathbb{R}_* = V_* R/\nu$.
8) Using values of $\mathbb{R}_*$ and $\Delta R/R$ obtained, determine $C/\sqrt{g}$ from Fig. A-7.
9) Compute $V_c = (C/\sqrt{g})\ V_*$.
10) Compare V_c and V values, and if not approximately equal, recompute using either a new depth or slope value.
11) Compute tractive force: $\tau = \gamma\ D\ S$.
12) Compute stream power τV .
13) For the d_{50} value and τV obtained above, determine the flow regime from Fig. 2-3.

14) For stable canal design, the flow regime should be no higher than the upper dune range. If the resulting regime is above this, recompute with modified D and S_o values.

15) Compute Mannings value: $n = \frac{1.486}{V} R^{2/3} S^{1/2}$.

16) Compute total sediment load.

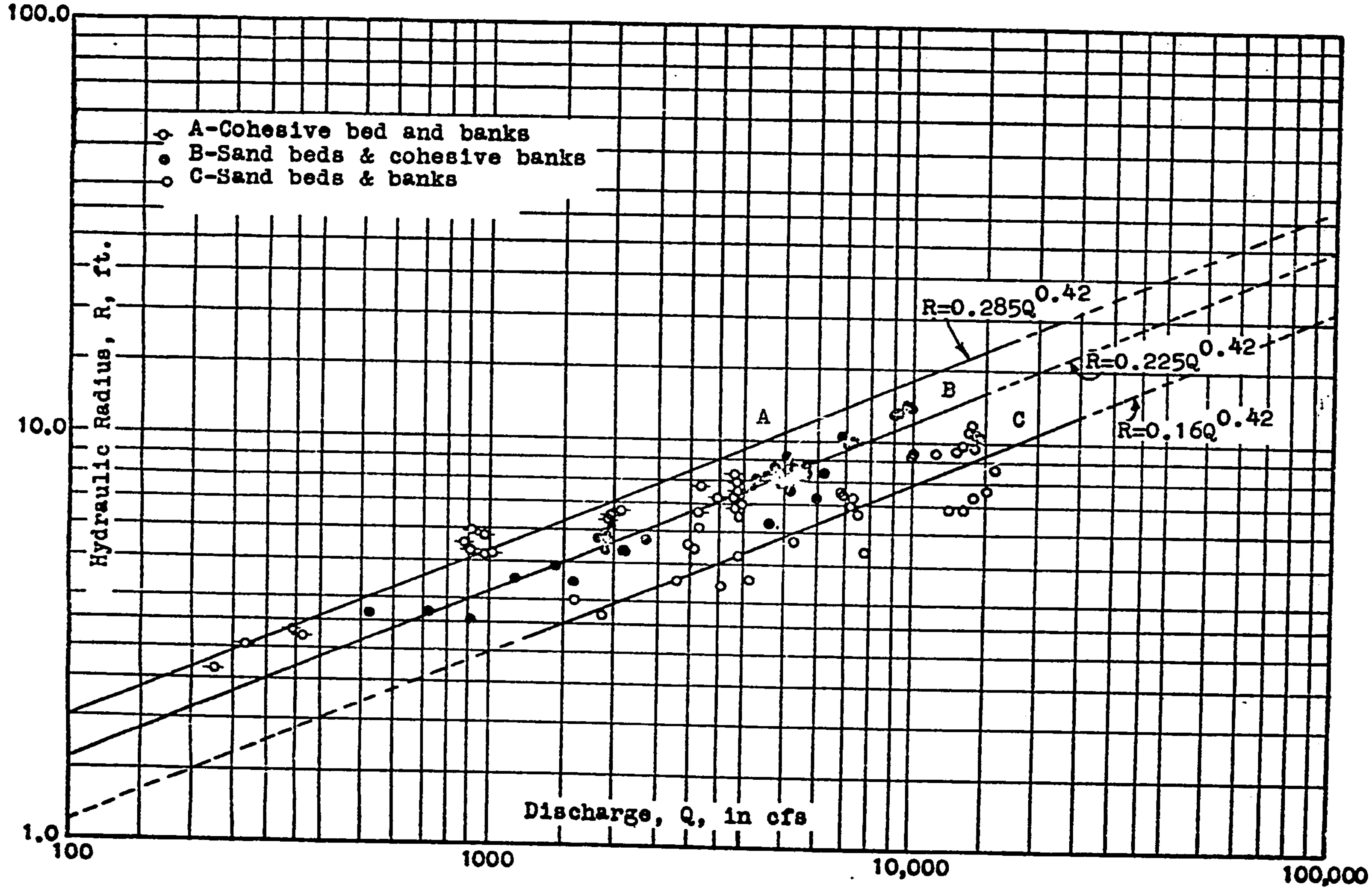


Figure A-1. Variation of Hydraulic Radius R with Discharge Q and Type of Channel (after Simons and Albertson [95]).

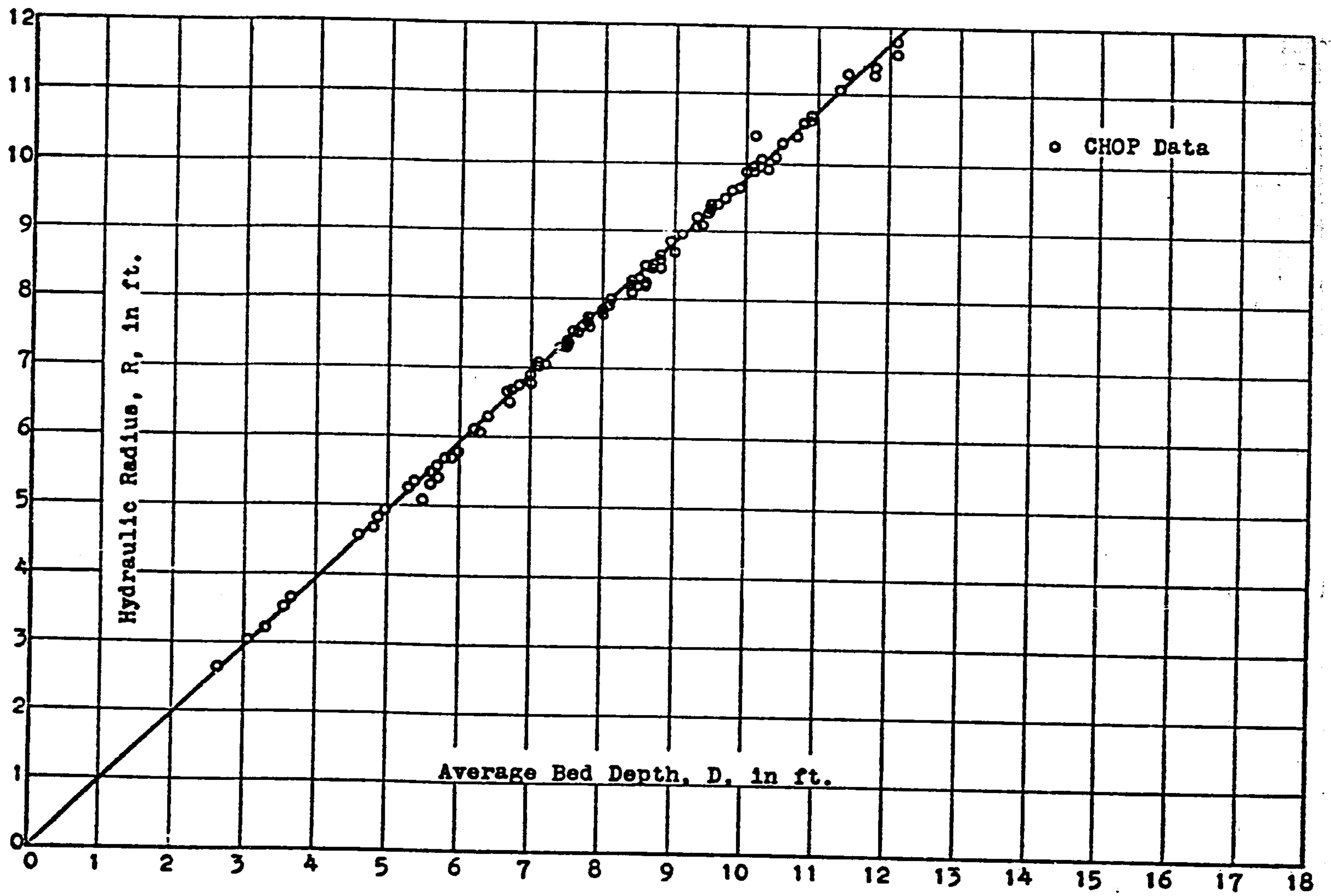


Figure A-2. Variation of Hydraulic Radius R with Depth D (after Simons and Albertson [95]).

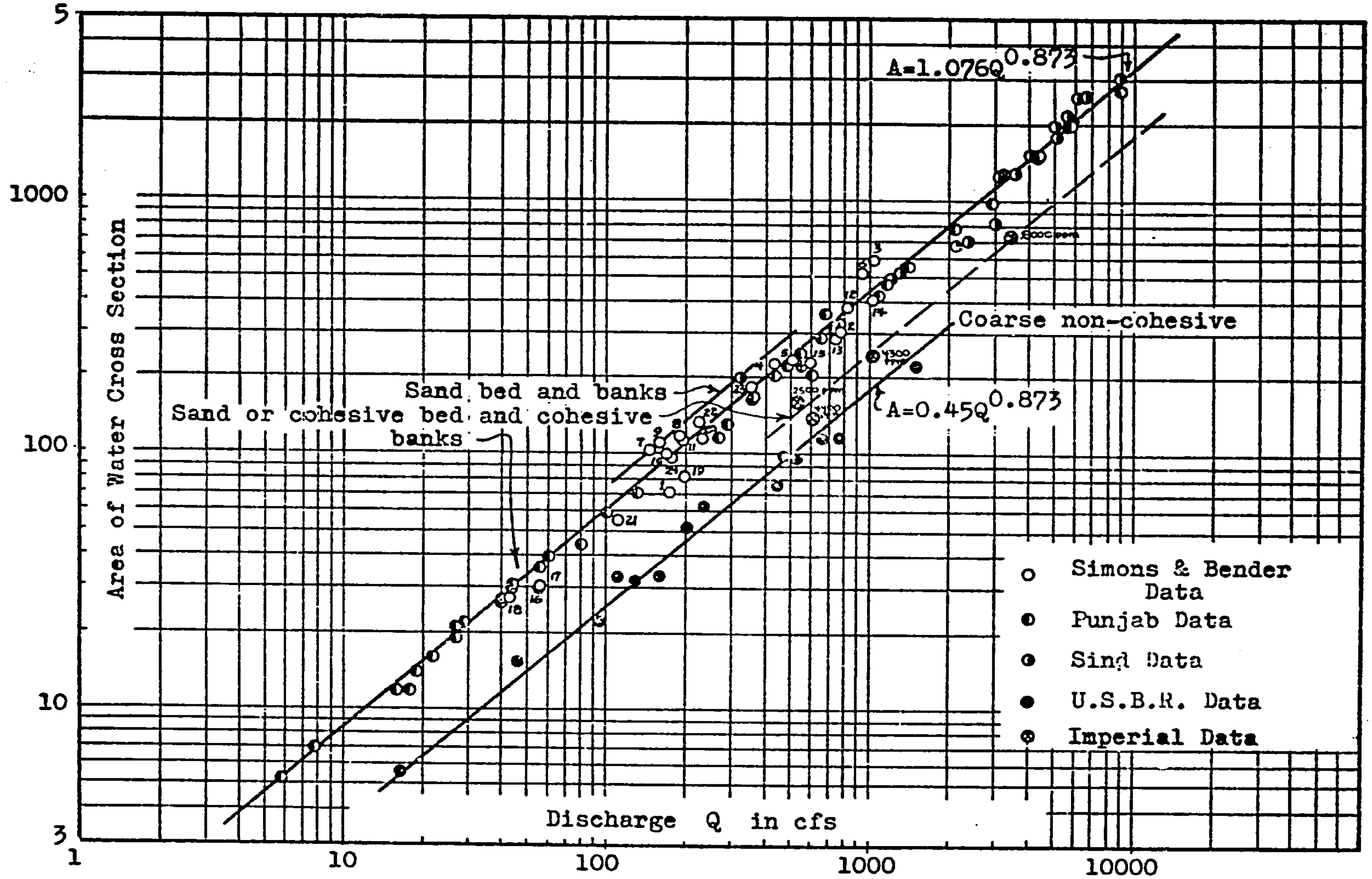


Figure A-3. Variation of Area of Water Cross-Section A with Discharge Q and Type of Channel (after Simons and Albertson [95]).

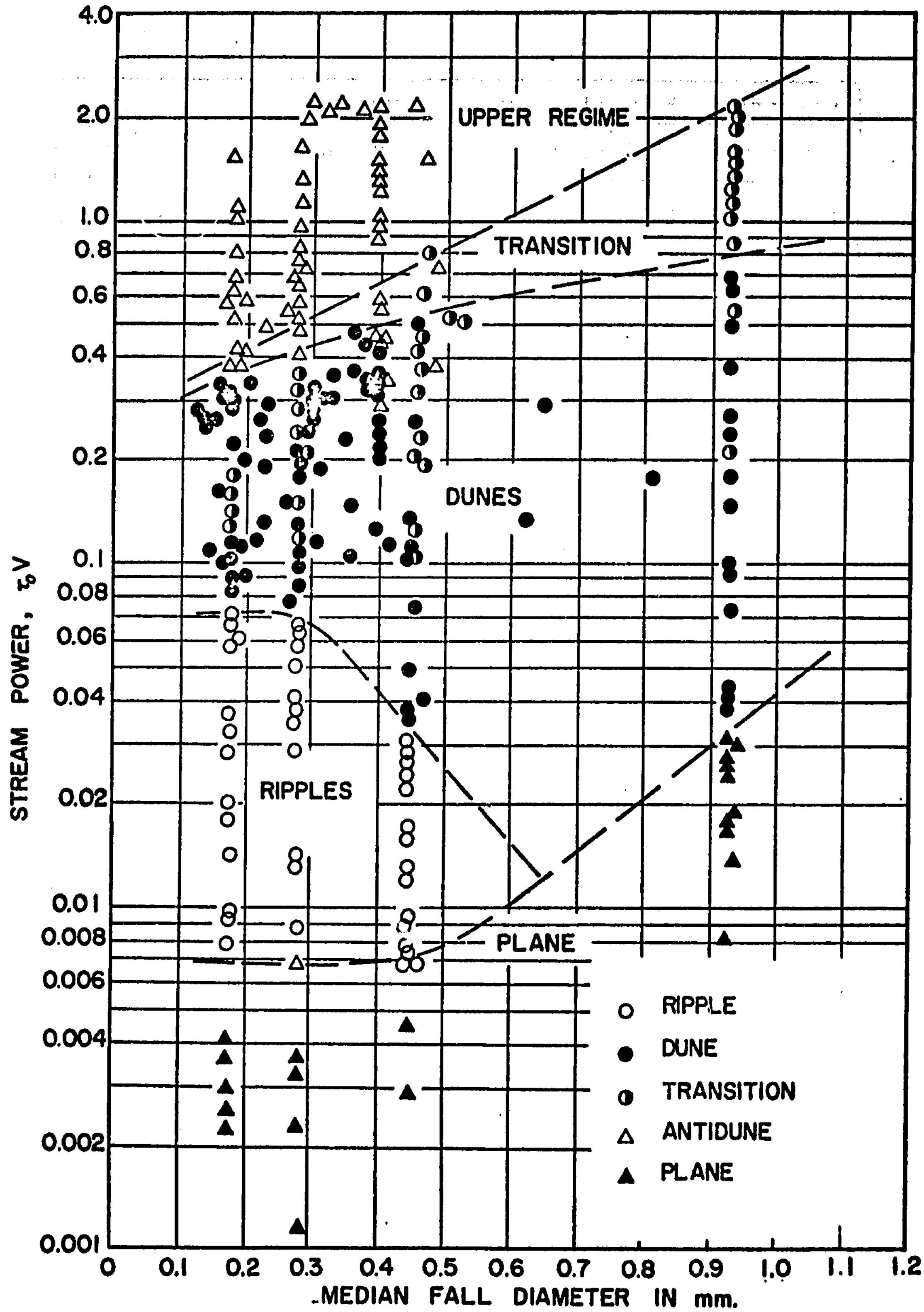


Figure 2-3. Relation of Stream Power and Median Fall Diameter to Form of Bed Roughness (after Simons and Richardson [94]).

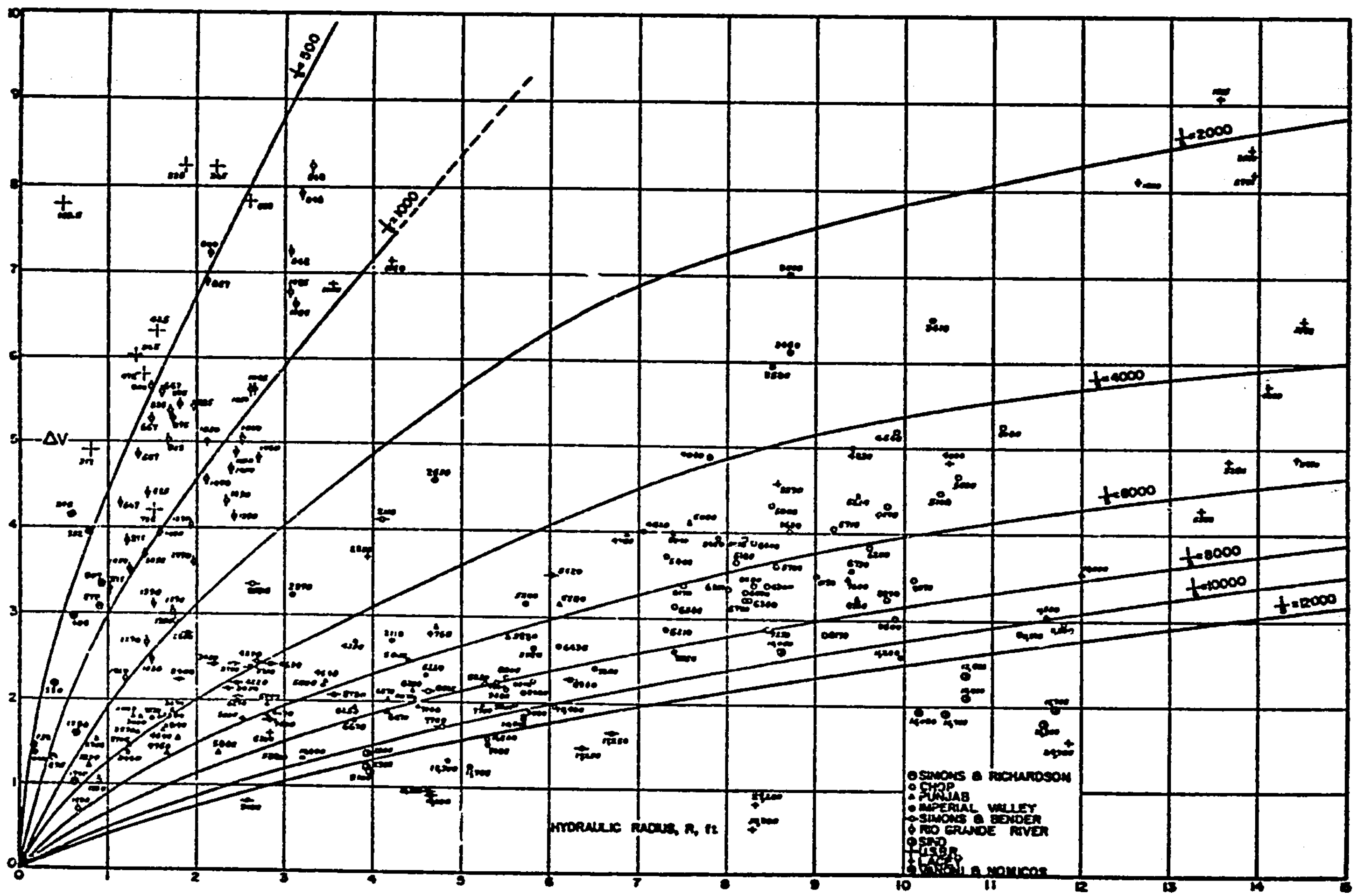


Figure A-4. Variation of ΔV with Hydraulic Radius and Slope (after Haynie [35]).

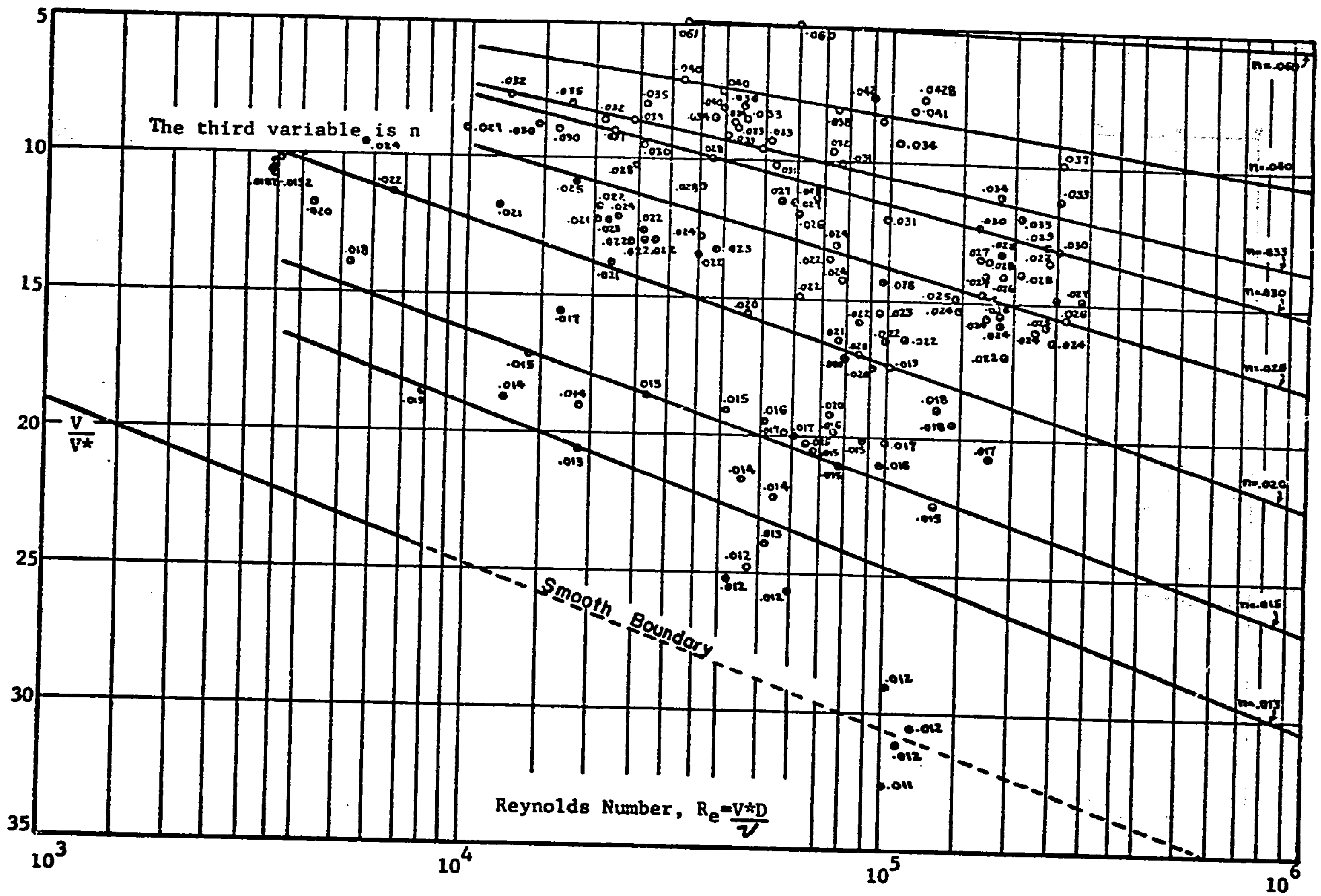


Figure A-5. Flow Resistance Diagram Relating V/V_* to R_* (after Haynie, [35]).

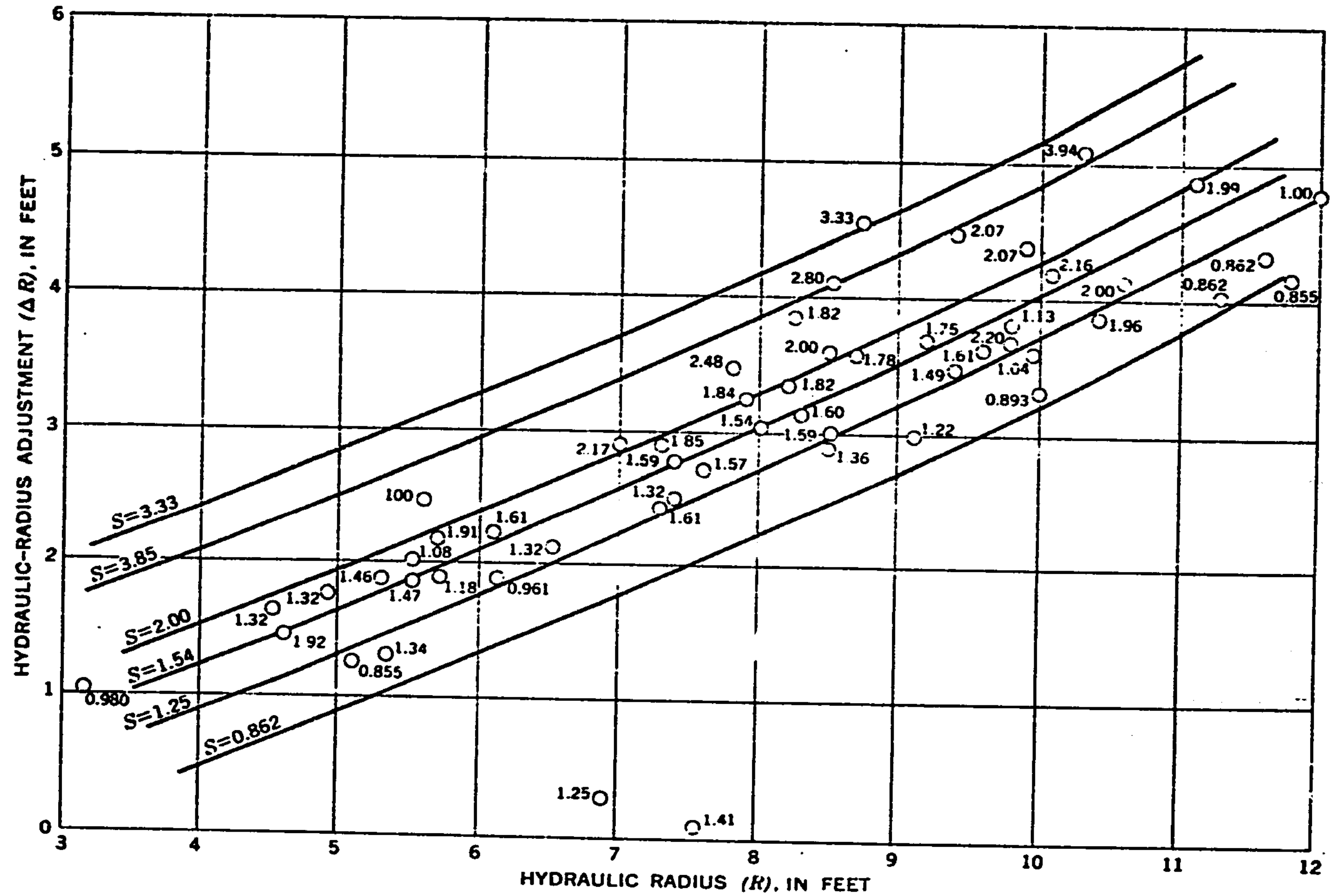


Figure A-6. Relation Between the Hydraulic-Radius, Adjustment (ΔR), Hydraulic Radius (R), and Slope ($S \times 10^4$) for Pakistan Canals (after Simons and Richardson [92]).

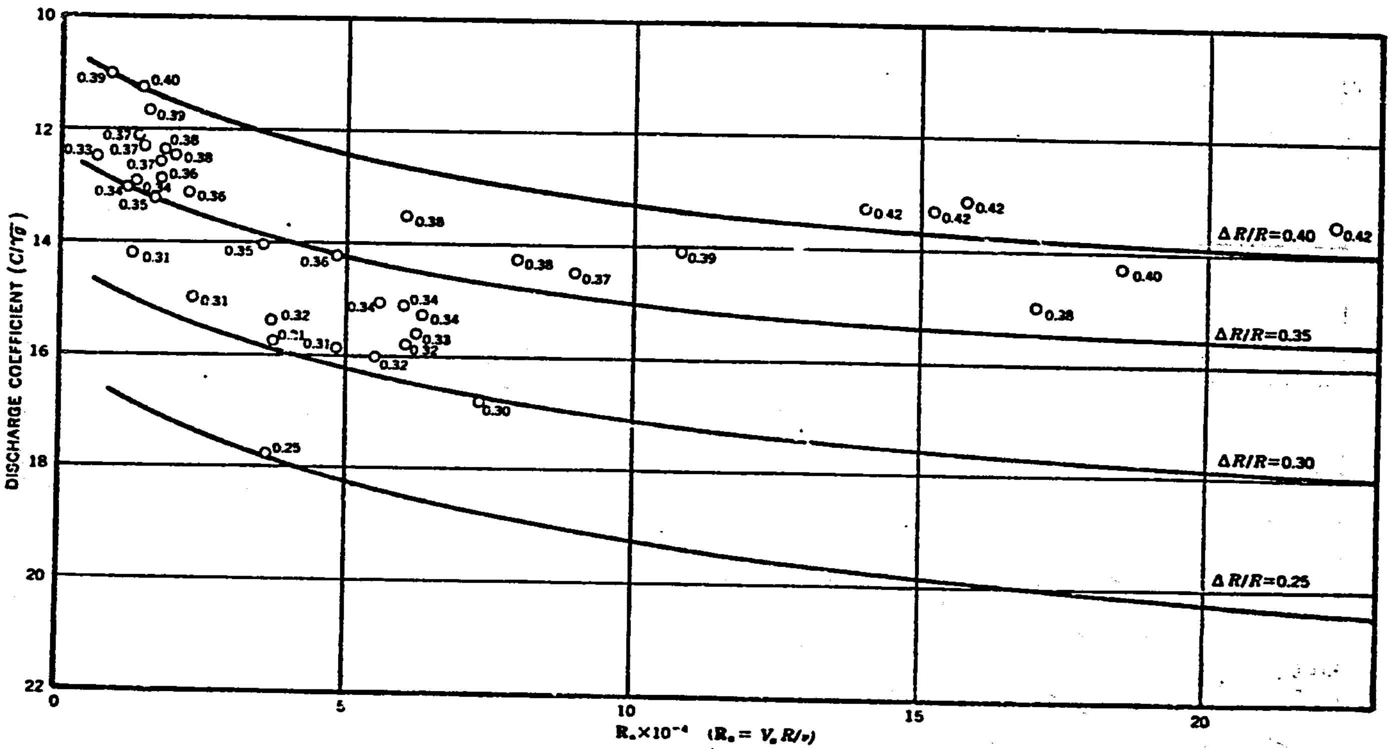


Figure A-7. Resistance Diagram Relating $C/\sqrt{g}$, R_*, and $\Delta R/R$ for Punjab Canals (after Simons and Richardson [92]).

APPENDIX B

BED MATERIAL DISCHARGE BY EINSTEIN'S METHOD

COMPUTER PROGRAM EIINS

APPENDIX B

BED MATERIAL DISCHARGE BY EINSTEIN'S METHOD COMPUTER PROGRAM EIINS

Computer program EIINS was developed to perform the required calculations for application of the Einstein [23] procedure for computation of total bed material load. The program is written in Fortran IV and was tested on the CDC 6400 computer.

For use in the program, Einstein's correction factor curves were segmented with a parabolic or log-parabolic equational form used to approximate each segment. The same procedure was used for the plot of transport intensity (Φ_*) vs. shear intensity (ψ_*). Integrals which sum the product of local velocity and local concentration over depth are computed directly using Simpson's rule with a variable increment for numerical integration.

Data Input

The first card in the input data deck contains three path control parameters in the format 3I3. The first parameter (NC) is the number of sets of input data to be read by the computer. The second (MIO) selects the way in which bed sediment data will be entered and the third (MRC) relates to the use of Einstein's resistance function. The use of MIO and MRC is explained further in the following paragraphs.

The second card in the input deck contains information describing channel geometry. Water discharge (Q), cross-sectional area (A), hydraulic radius (R), bed width (BW), energy slope (S), and channel bank slope (ZS) are entered in that order under the format 4F9.2, E10.3, F5.2. Units are those of the foot-pound-second system.

For a value of the path control parameter MRC = 1, the values entered for hydraulic radius and cross-sectional area are taken as estimates with the actual values being computed internally using Einstein's resistance function. In this case, a trapezoidal channel is assumed having the average width and side slope indicated. Side slope is entered as the cotangent of the angle made with the horizontal. When MRC is given the value of zero, side slope need not be entered. Flow resistance is assumed known, and geometric variables are taken as exact.

Entry of variables describing water and sediment properties is made according to the value of path control parameter MIO. Input option one (MIO = 1) is used when it is desired to enter each bed sediment fraction separately. Using this input form, water temperature (T) in degrees Fahrenheit, sediment specific gravity (SG), and the bed material diameters for which sixty-five percent (D65) and thirty-five percent (D35) of the sample is finer by weight are entered using the format F7.2, F6.3, 2E10.3. The next card enters the number of size fractions into which the sample has been broken (format I3), and the remaining entry cards in the data set contain the fractional mean diameters and the decimal percent represented by each in the format 2(E10.3, F6.3). All bed material diameters are entered in feet.

Input option two (MIO = 2) is used for bed sediment having a lognormal gradation. In this case, water temperature and sediment specific gravity are entered in the format F7.2, F6.3, followed by a card in the format 2F7.3 containing the mean diameter of the bed material in millimeters and the gradation coefficient. The bed material is divided into nine equal fractions internally with five percent increments at each end of the gradation curve neglected.

With the exception of the path control card, all input is repeated for each set of data indicated by the variable NC. The same input and resistance computation options must be used for each data set in a given run of the program.

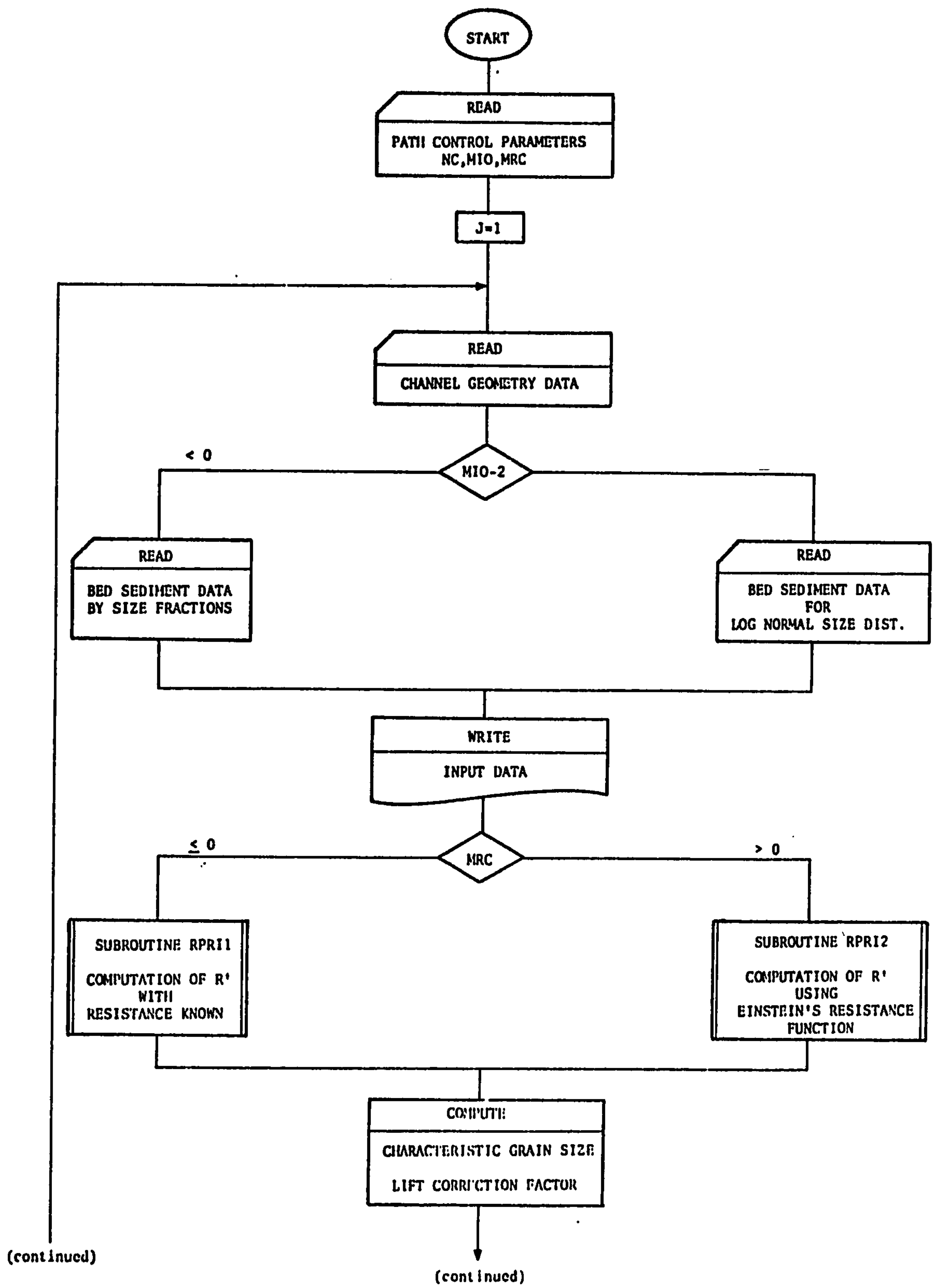
Table B-1 contains input variable names, definitions, and units in the order in which the variables appear in the program. The remainder of this appendix consists of a computational flow chart and listing of program EIINS followed by a sample printout for a single channel run.

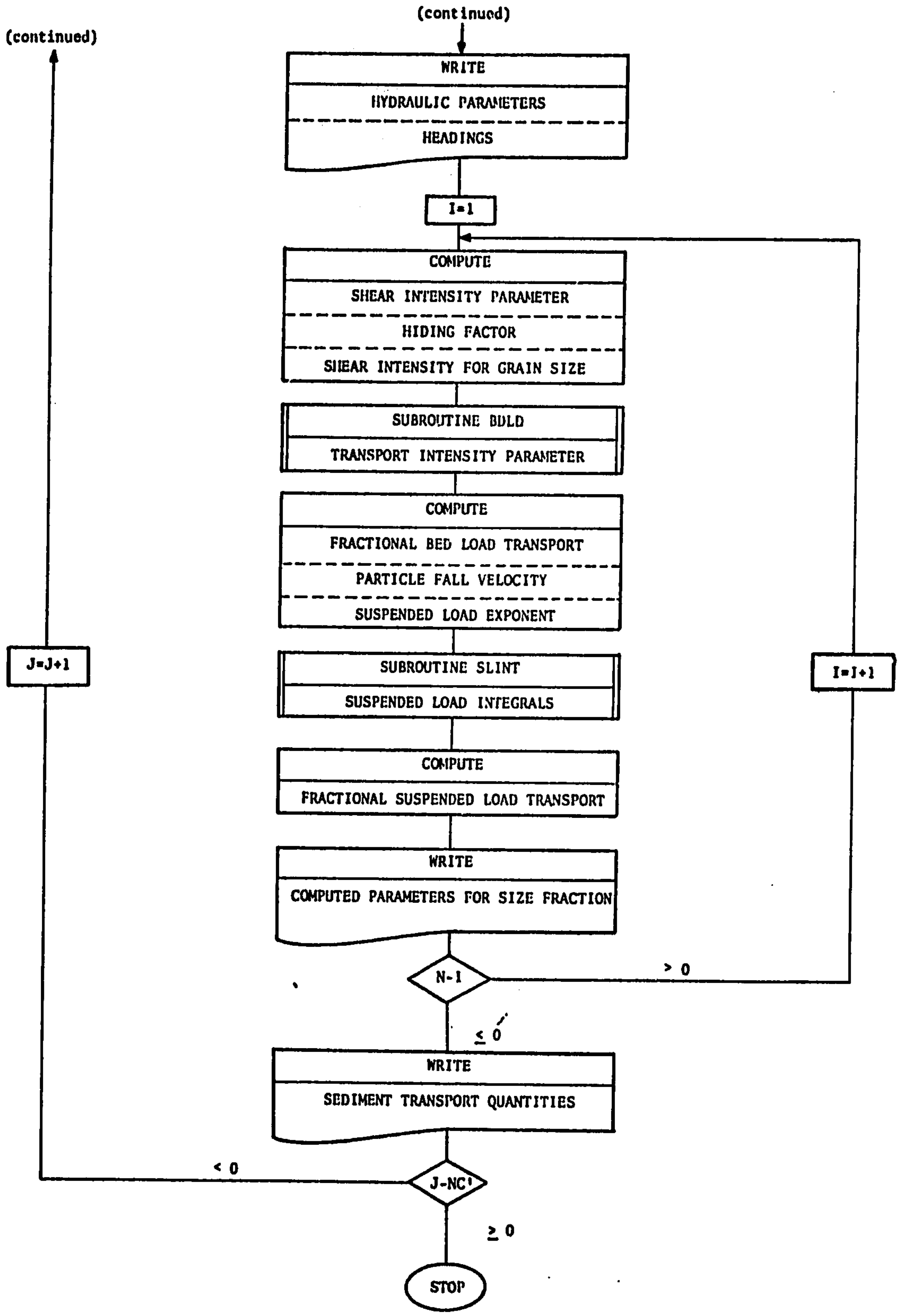
Table B-1. Input Variables for Program EIINS

Fortran Name	Variable Definition	Units	Input Option
NC	Number of data sets	--	1, 2
MIO	Input option control	--	1, 2
MRC	Resistance computation control	--	1, 2
Q	Water discharge	c.f.s.	1, 2
A	Cross-sectional area	sq. ft	1, 2
R	Hydraulic radius	ft	1, 2
BW	Bed width	ft	1, 2
S	Channel slope	--	1, 2
ZS	Bank slope	--	1, 2
T	Water temperature	°F	1, 2
SG	Sediment specific gravity	--	1, 2
D35	Diameter for which 35% by wt. of the bed material is finer	ft	1
D65	Diameter for which 65% by wt. of the bed material is finer	ft	1
N	Number of sediment size fractions	--	1
D(I)	Geometric mean diameter of the size fractions	ft	1
BI(I)	Decimal fraction of sample represented by D(I)	--	1
D50	Mean diameter of bed sediment	mm	2
GC	Bed sediment gradation coefficient	--	2

Program EIINS

(Computational Flow Chart)


START
READ
PATH CONTROL PARAMETERS NC,MIO,MRC
J=1
READ
CHANNEL GEOMETRY DATA
< 0
MIO-2
READ
BED SEDIMENT DATA BY SIZE FRACTIONS
READ
BED SEDIMENT DATA FOR LOG NORMAL SIZE DIST.
WRITE
INPUT DATA
≤ 0
MRC
> 0
SUBROUTINE RPRI1
COMPUTATION OF R' WITH RESISTANCE KNOWN
SUBROUTINE RPRI2
COMPUTATION OF R' USING EINSTEIN'S RESISTANCE FUNCTION
COMPUTE
CHARACTERISTIC GRAIN SIZE
LIFT CORRECTION FACTOR
(continued)
(continued)


(continued)
(continued)
WRITE
HYDRAULIC PARAMETERS
HEADINGS
I=1
COMPUTE
SHEAR INTENSITY PARAMETER
HIDING FACTOR
SHEAR INTENSITY FOR GRAIN SIZE
SUBROUTINE BDLD
TRANSPORT INTENSITY PARAMETER
COMPUTE
FRACTIONAL BED LOAD TRANSPORT
PARTICLE FALL VELOCITY
SUSPENDED LOAD EXPONENT
SUBROUTINE SLINT
SUSPENDED LOAD INTEGRALS
COMPUTE
FRACTIONAL SUSPENDED LOAD TRANSPORT
WRITE
COMPUTED PARAMETERS FOR SIZE FRACTION
N-I
> 0
≤ 0
I=I+1
WRITE
SEDIMENT TRANSPORT QUANTITIES
J-NC'
< 0
≥ 0
J=J+1
STOP

Program EIINS

(listing)

```
      PROGRAM EIINS(INPUT,OUTPUT,TAPE5=INPUT,TAPE6=OUTPUT)
      DIMENSION D(20),BI(20),BF(20),SF(20),TF(20)
C
C     NC=NUMBER OF CHANNELS FOR  WHICH COMPUTATIONS ARE TO BE MADE
C     MIO SELECTS MODE OF INPUT
C     MRC SELECTS METHOD OF HYDRAULIC RADIUS BREAKDOWN
C     MRC=0 IMPLYS CHANNEL FLOW RESISTANCE IS KNOWN PRIOR TO INPUT
C     MRC=1 IMPLYS FLOW RESISTANCE IS TO BE COMPUTED FROM EINSTEINS
C     RELATIONS.  A TRAPEZOIDAL CHANNEL IS ASSUMED WITH SIDE SLOPE AND
C     AVERAGE WIDTH GIVEN
C
      READ(5,5)NC,MIO,MRC
    5 FORMAT(3I3)
      DO 150 J=1,NC
C
C     CHANNEL DATA INPUT
      READ(5,10)Q,A,R,BW,S,ZS
   10 FORMAT(4F9.2,E10.3,F5.2)
      DE=A/BW
      IF(MIO-2)7,8,8
    7 CONTINUE
      READ(5,11)T,SG,D65,D35
   11 FORMAT(F7.2,F6.3,2E10.3)
C
C     FRACTIONAL BED SEDIMENT DATA INPUT
C     ENTER ALL BED MATERIAL DIAMETERS IN FT.
      READ(5,12)N,(D(I),BI(I),I=1,N)
   12 FORMAT(I3,/(2(E10.3,F6.3)))
C
      GO TO 20
    8 CONTINUE
C     INPUT OPTION 2 PROGRAM EIINS
C     SEDIMENT HAVING LOG NORMAL DISTRIBUTION
C     INPUT D50 IN MM AND DIMENSIONLESS GRADATION COEFFICIENT
C     BED SEDIMENT DATA INPUT
      READ(5,600)T,SG
  600 FORMAT(F7.2,F6.3)
      N=9
      READ(5,601)D50,GC
  601 FORMAT(2F7.3)
      CF=1./(25.4*12.)
      DPA=ALOG(GC)
      DPB=ALOG(D50*CF)
C
      D(1)=EXP(DPA*(-1.64485)+DPB)
      D(2)=EXP(DPA*(-1.03652)+DPB)
      D(3)=EXP(DPA*(-0.67452)+DPB)
      D(4)=EXP(DPA*(-0.38540)+DPB)
      D(5)=EXP(DPA*(-0.12564)+DPB)
      D(6)=EXP(DPA*( 0.12564)+DPB)
      D(7)=EXP(DPA*( 0.38540)+DPB)
      D(8)=EXP(DPA*( 0.67452)+DPB)
      D(9)=EXP(DPA*( 1.03652)+DPB)
      D(10)=EXP(DPA*(1.64485)+DPB)
      D35=D(4)
      D65=D(7)
      DO 602 I=1,N
      D(I)=SQRT(D(I)*D(I+1))
  602 BI(I)=0.1
```

```
   20 CONTINUE
C     END INPUT OPTION 2
C
      WRITE(6,505)J,Q,A,R,BW,DE,S,ZS,T,SG
  505 FORMAT(1H1/26X"INPUT DATA CHECK"/28X"CANAL NO."I3,/,1H0,/2X,"DISCH
     1ARGE  AREA  HYD.RAD.  WIDTH  DEPTH  SLOPE  S-SLOPE TEMP.  SP.GR.
     1",/6X,"Q        A         R        BW     DE        S          ZS      T
     1     SG",/1H0,F9.1,F8.1,F8.2,F8.1,F7.2,1PE10.2,0P2F7.2,F7.3)
      WRITE(6,506)D35,D65
  506 FORMAT(1H0,/8X,"D35        D65          FRACT. MEAN                PERCEN
     1T IN",/8X,"(FT)"5X"(FT)"9X"DIAMETER(FT)"10X"FRACTION"/2X1P2E11.3)
      DO 507 I=1,N
  507 WRITE(6,508)D(I),BI(I)
  508 FORMAT(1X,T31,1PE10.3,6X,2PF11.2)
C
      TL=0.
      RL=0.
      SL=0.
      G=32.174
      VISC=1.05943E-05*ALOG10(T)*ALOG10(T)-6.1145E-05*ALOG10(T)
     1+8.7341E-05
      RO=-2.75E-06*T*T+1.725E-04*T+1.9375
      ROS=SG*1.94
      SGA=ROS/RO
C
C     SUBROUTINE RPRI COMPUTES EINSTEINS HYD. RAD. ASSOCIATED WITH GRAIN
C    1ROUGHNESS
      IF(MRC)103,103,102
  102 CALL RPRI2(R,R1,R11,D65,D35,A,Q,VISC,X1,SGA,U1S,DE,BW,ZS,S,U,U11S)
      GO TO 104
  103 CALL RPRI1(R,R1,R11,S,D65,D35,A,Q,VISC,X1,SGA,U1S,U,U11S)
  104 CONTINUE
C
      DLP=11.6*VISC/U1S
      DLK=D65/X1
      DV=DLK/DLP
C     COMPUTATION OF CHARACTERISTIC GRAIN SIZE OF MIXTURE
      IF(DV.GT.1.8) GO TO 105
      X2=1.39*DLP
      GO TO 106
  105 X2=0.77*DLK
  106 CONTINUE
C
C     COMPUTATION OF PRESSURE OR LIFT CORRECTION
      YP=D65/DLP
      IF(YP.GT.1.70) GO TO 109
      IF(YP.GT.0.68) GO TO 108
  107 A1=-0.23972
      B1= 1.0256
      C1=-0.02215
      GO TO 110
  108 A1=-2.76548
      B1= 0.249404
      C1=-C.081306
      GO TO 110
  109 A1=0.802826
      B1=-.970331
      C1=0.01028
  110 CONTINUE
```

```
      DV1=ALOG10(YP)
      Y=10.**(A1*DV1*DV1+R1*DV1+C1)
C
      WRITE(6,500)J,Q,VISC,HO,SGA,X1,U,U1S,U11S,R1,R11,DLP,DLK,X2,Y
  500 FORMAT("1",/,"0",48X,"COMPUTED PARAMETER CHECK",/55X,"CANAL NO.",
     1I4,/1H0,30X,"ADJ.  LOG COR AVERAGE  SHEAR VELOCITY    HYD. RADIUS
     1    DELTA       CAP.     CHARACTERISTIC LIFT COR"/1X"DISCHARGE VISC
     1OSITY DENSITY  SP.GR.  FACTOR VELOCITY  GRAIN   FORM   GRAIN    FO
     1RM     PRIME      ",
     1" DELTA      GRAIN SIZE    FACTOR",/5X,"Q",7X"VISC",7X,"RO",6X,"SGA
     1",5X,"X1",6X,"U",6X,"U1S",5X"U11S",5X,"R1",6X,"R11",5X,"DLP ",7X
     1"DLK ",11X,"X",10X,"Y",/"0"F9.2,1PE10.3,0P8F8.3,1P2E11.3,E13.3,
     10PF9.4)
      WRITE(6,501)
  501 FORMAT("0",/,"0FRACTIONAL  PARTICLE   HIDING     GRAIN   TRANSPORT
     1   RED LOAD  PART.FALL"13X" EINSTIENS"/," DIAMETER   SHEAR       F
     1ACTOR     SHEAR   INTENSITY  THICKNESS   VELOCITY    EXP.",5X," IN
     1TEGRALS",/,"      D(I)          ZII         SQI         ZIS       PHS
     1    E            W            Z          I1          I2         PE",/)
C
C     COMPUTATION BY SEDIMENT SIZE FRACTIONS
      DO 140 I=1,N
      ZII=(SGA-1.)*D(I)/(R1*S)
C
C      COMPUTATION OF HIDING FACTOR
      DV2=D(I)/X2
      IF(DV2.GT.1.5) GO TO 125
      IF(DV2.GT.0.8) GO TO 122
      IF(DV2.GT.0.6) GO TO 121
  120 A2=-0.498372
      B2=-2.912252
      C2=-0.250483
      GO TO 124
  121 A2=5.968915
      B2=0.052227
      C2=0.08883
      GO TO 124
  122 A2=1.509921
      B2=-.631933
      C2=0.064458
  124 CONTINUE
      DV3=ALOG10(DV2)
      SQI=10.**(A2*DV3*DV3+B2*DV3+C2)
      GO TO 126
  125 SQI=1.0
  126 CONTINUE
      ZIS=ZII*Y*SQI*(ALOG10(10.6)/ALOG10(10.6*X2/DLK))**2.
C
  127 CALL BDLD(ZIS,PHS)
C
      BF(I)=BI(I)*PHS*ROS*((G*D(I))**1.5)*SQRT(SGA-1.)
      BL=BL+BF(I)
      W=(SQRT((2./3.)*G*(SGA-1.)*(D(I)**3.)+36.*VISC*VISC)-6.*VISC)/D(I)
      Z=W/(0.4*SQRT(G*R1*S))
C
      CALL SLINT(D(I),DE,Z,SI1,SI2,E)
C
      PE=2.303*ALOG10(30.2*DE/DLK)
      SF(I)=BF(I)*(PE*SI1+SI2)
```

```
      TF(I)=BF(I)+SF(I)
      SL=SL+SF(I)
      TL=TL+TF(I)
      WRITE(6,502)D(I),ZII,SQI,ZIS,PHS,E,W,Z,SI1,SI2,PE
  502 FORMAT(" ",1PE10.3,0P3F10.4,1P3E11.3,0PF8.3,3F10.4)
  140 CONTINUE
      WRITE(6,998)
  998 FORMAT("0"/"0"/" NOTE - ALL PARAMETERS GIVEN IN THE FOOT-POUND-SEC
     1OND SYSTEM")
      BLT=BL*BW*43.2
      SLT=SL*BW*43.2
      TLT=BLT+SLT
      CONC=(TL*BW/(Q*G*RO))*1.E 06
  149 CONTINUE
C
      RC=R1+R11
      WRITE(6,999)J,Q,A,BW,DE,R,RC,R1,R11,U
  999 FORMAT(1H1,/35X,"OUTPUT",/"0"/32X"CHANNEL NO.",I3,/"0",31X"HYDRAUL
     1IC DATA",/"0",46X,"HYDRAULIC RADIUS",/20X,"AVERAGE",2X,"AVERAGE",
     13HX,"AVERAGE",/," DISCHARGE",2X,"AREA",5X,"WIDTH",4X,"DEPTH",4X,
     1"ACTUAL",2X,"COMPUTED",3X,"GRAIN",4X,"FORM",3X,"VELOCITY",/4X,
     1"(CFS)"2X,"(SQ.FT)",6(4X,"(FT)",1X),2X,"(FT/SEC)",/"0",F9.2,8(F8.2
     1,1X))
      WRITE(6,1000)BLT,SLT,TLT,CONC
 1000 FORMAT("0"/"0"/48X"TRANSPORT  DATA"/48X,"(BED MATERIAL)",/"0"/17X,
     1"TRANSPORT BY SIZE FRACTIONS",39X,"TOTAL TRANSPORT"/"0",2X,"MEAN"
     15X,"PERCENT",4X,"BED LOAD",4X,"SUSP. LOAD",3X,"TOTAL LOAD",13X,"BE
     1D",8X,"SUSP.",8X,"TOTAL",8X,"CONC.",/" DIAMETER",3X,"IN FRACT.",
     150X,"LOAD",7X,"LOAD",9X,"LOAD",9X,"BY WT",/3X,"(FT)",16X"(POUNDS P
     1ER SECOND PER FOOT WIDTH)",23X,"(TONS  PER  DAY)",12X"(PPM)",/"0",
     165X,4(1PE11.3,2X))
C
      SUP=0.
      DO 1001 I=1,N
      SUP=SUP+BI(I)
 1001 WRITE(6,1002)D(I),BI(I),BF(I),SF(I),TF(I)
 1002 FORMAT(1H0,1PE10.3,2PF9.2,1PE12.3,2E13.4)
C
      WRITE(6,1003)SUP,BL,SL,TL
 1003 FORMAT("0"5HTOTAL,5X,2PF9.2,1PE12.3,2E13.4)
C
      IF(MRC.EQ.0)WRITE(6,1006)
 1006 FORMAT("0",/,"0NOTE-FLOW RESISTANCE KNOWN PRIOR TO BEGINNING COMPU
     1TATIONS"/,6X"R1 IS COMPUTED FROM GIVEN VELOCITY TO SATISFY LOGRITH
     1MIC VELOCITY DISTRIBUTION"/,6X"COMPUTED VALUES OF R11 AND R NOT US
     1ED IN TRANSPORT COMPUTATIONS")
      IF(MRC.EQ.1)WRITE(6,1007)
 1007 FORMAT("0",/,"0NOTE-FLOW RESISTANCE COMPUTED BY EINSTIENS RELATION
     1S (RC=R)",/,6X,"THIS RESISTANCE MAY TEND TO BE HIGH FOR CANAL WORK
     1")
      IF(MIO.EQ.2)WRITE (6,1008)GC
 1008 FORMAT("0",5X,"SEDIMENT SIZE BREAKDOWN ASSUMES LOG NORMAL DISTRIBU
     1TION WITH A GRADATION COEFFICIENT OF",F5.2)
C
  150 CONTINUE
C
      STOP
      END
```

Program EIINS--Subroutine RPRI1

```
      SUBROUTINE RPRI1(R,R1,R11,S,D65,D35,A,Q,VISC,X,SGA,U1S,U,U11S)
C
C     COMPUTATION OF R PRIME FOR USE IN EINSTEIN BED LOAD FUNCTION
C     WHEN DISCHARGE AND RESISTANCE ARE KNOWN
C
      QE=0.0
      R1=.8*R
      G=32.174
C
   10 R1=R1-QE*R1
      U1S=SQRT(G*R1*S)
      DDE=U1S*D65/(11.6*VISC)
C
C     COMPUTATION OF X CORRECTION FACTOR IN LOG VELOCITY FORMULA
C     EINSTIENS PLOT REPRESENTED BY PARABOLIC SEGMENTS
C
      IF(DDE.GT.8.3) GO TO 15
      IF(DDE.GT.3.) GO TO 14
      IF(DDE.GT.1.5) GO TO 13
      IF(DDE.GT.0.5) GO TO 12
C
   11 ACO=-.36379
      BCO=1.345
      CCO=1.8278
      GO TO 16
C
   12 ACO=-2.482
      BCO=-0.04578
      CCO= 1.615
      GO TO 16
C
   13 ACO=0.394
      BCO=-1.4068
      CCO= 1.7655
      GO TO 16
   14 ACO= 0.79255
      BCO=-1.52256
      CCO= 1.73
      GO TO 16
C
   15 ACO=0.
      BCO=0.
      CCO=1.
C
   16 CONTINUE
      DV=ALOG10(DDE)
      X=ACO*DV*DV+BCO*DV+CCO
      U=U1S*5.75*ALOG10(12.27*X*R1/D65)
      QC=U*A
      QE=(QC-Q)/Q
      QEA=ABS(QE)
      IF(QEA.GE.1.)QE=QE/2.
      IF(QEA.GE.2.)QE=QE/2.
C
      IF(QEA.GT.0.005)GO TO 10
C     COMPUTATION OF HYD RADIUS ASSOCIATED WITH FORM ROUGHNESS
C     EINSTIENS PLOT REPRESENTED BY LOG PARABOLIC SEGMENTS
   17 F=(SGA-1)*D35/(R1*S)
      IF(F.GT.7.0) GO TO 21
```

```
      IF(F.GT.2.5) GO TO 20
      IF(F.GT.1.0) GO TO 19
   18 ACO2= 0.45859
      BCO2=-1.14249
      CCO2= 1.59439
      GO TO 22
C
   19 ACO2= 0.60401
      BCO2=-1.15492
      CCO2= 1.59439
      GO TO 22
C
   20 ACO2= 0.2056
      BCO2=-0.78001
      CCO2= 1.50838
      GO TO 22
C
   21 ACO2= 0.0
      BCO2=-0.39476
      CCO2= 1.3295
C
   22 CONTINUE
      DV2=ALOG10(F)
      U2=10.**(ACO2*DV2*DV2+BCO2*DV2+CCO2)
      U11S=U/U2
      R11=U11S*U11S/(G*S)
C
      RETURN
      END
```

Program EIINS--Subroutine RPRI2

```
      SUBROUTINE RPRI2(R,R1,R11,D65,D35,A,Q,VISC,X,SGA,U1S,DE,BW,ZS,S,U,
     1U11S)
C     COMPUTATION OF R PRIME WITH DISCHARGE, AVERAGE WIDTH, AND BANK
C     SLOPE KNOWN
C     RESISTANCE TO FLOW COMPUTED USING EINSTEIN RESISTANCE FUNCTION
C
      QE=0.0
      R1=.5*R
      G=32.174
C
   10 R1=R1-QE*R1
      U1S=SQRT(G*R1*S)
      DDE=U1S*D65/(11.6*VISC)
C
C     COMPUTATION OF X CORRECTION FACTOR IN LOG VELOCITY FORMULA
C     EINSTIENS PLOT REPRESENTED BY PARABOLIC SEGMENTS
C
      IF(DDE.GT.8.3) GO TO 15
      IF(DDE.GT.3.) GO TO 14
      IF(DDE.GT.1.5) GO TO 13
      IF(DDE.GT.0.5) GO TO 12
C
   11 ACO=-.36379
      BCO=1.345
      CCO=1.8278
      GO TO 16
   12 ACO=-2.482
      BCO=-0.04578
      CCO= 1.615
      GO TO 16
   13 ACO=0.394
      BCO=-1.4068
      CCO= 1.7655
      GO TO 16
   14 ACO= 0.79255
      BCO=-1.52256
      CCO= 1.73
      GO TO 16
   15 ACO=0.
      BCO=0.
      CCO=1.
C
   16 CONTINUE
      DV=ALOG10(DDE)
      X=ACO*DV*DV+BCO*DV+CCO
      U=U1S*5.75*ALOG10(12.27*X*R1/D65)
C     COMPUTATION OF HYD RADIUS ASSOCIATED WITH FORM ROUGHNESS
C     EINSTIENS PLOT REPRESENTED BY LOG PARABOLIC SEGMENTS
   17 F=(SGA-1)*D35/(R1*S)
      IF(F.GT.7.0) GO TO 21
      IF(F.GT.2.5) GO TO 20
      IF(F.GT.1.0) GO TO 19
   18 ACO2= 0.45859
      BCO2=-1.14249
      CCO2= 1.59439
      GO TO 22
   19 ACO2= 0.60401
      BCO2=-1.15492
      CCO2= 1.59439
```

```
      GO TO 22
   20 ACO2= 0.2056
      BCO2=-0.78001
      CCO2= 1.50838
      GO TO 22
   21 ACO2= 0.0
      BCO2=-0.39476
      CCO2= 1.3295
C
   22 CONTINUE
      DV2=ALOG10(F)
      U2=10.**(ACO2*DV2*DV2+BCO2*DV2+CCO2)
      U11S=U/U2
      R11=U11S*U11S/(G*S)
      R=R1+R11
      A=RW*1./(1./R+1./RW*(ZS-2.*SQRT(ZS*ZS+1.)))
      QC=U*A
      QE=(QC-Q)/QC
      QEA=ABS(QE)
C
      IF(QEA.GE.2.)QE=QE/2.
      IF(QEA.GE.1.)QE=QE/2.
C
      IF(QEA.GT.0.005)GO TO 10
      DE=A/RW
      RETURN
      END
```

Program EIINS--Subroutine BDLD
(listing)

```
      SUBROUTINE BOLD(ZIS,PHIS)
C
C     REPRODUCES EINSTEINS  ZI* VS PHI* PLOT BY PARABOLIC SEGMENTS
C
      IF(ZIS.GT.20.) GO TO 132
      IF(ZIS.GT.10.) GO TO 131
      IF(ZIS.GT.4.) GO TO 130
      IF(ZIS.GT.1.) GO TO 129
C
  128 A3=-0.067134
      B3=-1.0897
      C3= 0.875061
      GO TO 133
C
  129 A3=-0.669469
      B3=-1.008352
      C3= 0.875061
      GO TO 133
C
  130 A3=-1.999368
      B3= 0.436293
      C3= 0.487355
      GO TO 133
C
  131 A3=-6.994067
      B3=11.160671
      C3=-5.242325
      GO TO 133
C
  132 A3=-26.449074
      B3= 61.225849
      C3=-37.447539
C
  133 CONTINUE
      DV5=ALOG10(ZIS)
      PHIS=10.**(A3*DV5*DV5+B3*DV5+C3)
C
      RETURN
      END
```

Program EIINS--Subroutine SLINT
(listing)

```
      SUBROUTINE SLINT(D,DF,Z,SI1,SI2,EA)
      DIMENSION DEL(4),F1(4),F2(4),AI1(11,4),AI2(11,4),AY(4)
C
C     INTEGRATES CONCENTRATION OVER DEPTH FOR EINSTEINS SUSPENDED
C     LOAD INTEGRALS
C     EA MUST BE LESS THAN 0.2
C
      EA=2*D/DF
      N=11
      AN=N
      BI1=0.
      BI2=0.
      IF(EA.GE.0.1)GO TO 51
      II=1
      JJ=0
C
   31 GII=II
      GJJ=JJ
      DLT=(2.*GII/(AN-1.))*EA
      AY(1)=(1.+GJJ/(AN-1.))*EA
      AY(2)=AY(1)+DLT/2.
      AY(3)=AY(1)+DLT
C
      DO 32 KK=1,3
      F1(KK)=((1.-AY(KK))/AY(KK))**Z
      F2(KK)=F1(KK)*ALOG(AY(KK))
   32 CONTINUE
      RIA=((F1(1)+4.*F1(2)+F1(3))/6.)*DLT
      RIB=((F2(1)+4.*F2(2)+F2(3))/6.)*DLT
C
      BI1=BI1+RIA
      BI2=BI2+RIB
      IF(AY(3).GE.0.1)GO TO 50
C
      JJ=JJ+2*II
      II=II+1
      GO TO 31
C
   50 CONTINUE
      E=AY(3)
      GO TO 52
   51 CONTINUE
      E=EA
   52 CONTINUE
      M=(N-1)/2
      DEL(1)=(.2-E)/(N-1)
      DEL(2)=.3/(N-1)
      DEL(3)=.5/(N-1)
C
      DO 111 J=1,3
      DO 110 I=1,M
      DO 106 K=1,3
      IF(J-2)101,102,103
  101 IA=2*I-3+K
      IB=0
      IC=0
      GO TO 104
  102 IA=N-1
      IB=2*I-3+K
      IC=0
      GO TO 104
  103 IA=N-1
      IB=N-1
      IC=2*I-3+K
  104 CONTINUE
      A=IA
      B=IB
      C=IC
      Y=E+A*DEL(1)+B*DEL(2)+C*DEL(3)
      F1(K)=((1.-Y)/Y)**Z
      F2(K)=F1(K)*ALOG(Y)
  106 CONTINUE
      AI1(I,J)=((F1(1)+4.*F1(2)+F1(3))/3.)*DEL(J)
      AI2(I,J)=((F2(1)+4.*F2(2)+F2(3))/3.)*DEL(J)
      BI1=BI1+AI1(I,J)
      BI2=BI2+AI2(I,J)
  110 CONTINUE
  111 CONTINUE
C
      DV=0.216*(EA**(Z-1.))/((1.-EA)**Z)
      SI1=DV*BI1
      SI2=DV*BI2
      RETURN
      END
```

Sample Output--Program EIINS

INPUT DATA CHECK
CANAL NO. 1

DISCHARGE Q	AREA A	HYD.RAD. R	WIDTH BW	DEPTH DE	SLOPE S	S-SLOPE ZS	TEMP. T	SP.GR. SG
5000.0	1764.0	9.00	196.0	9.00	9.00E-05	10.00	70.00	2.650

D35 (FT)	D65 (FT)	FRACT. MEAN DIAMETER(FT)	PERCENT IN FRACTION
7.016E-04	9.589E-04		
		4.763E-04	10.00
		5.798E-04	10.00
		6.616E-04	10.00
		7.395E-04	10.00
		8.202E-04	10.00
		9.097E-04	10.00
		1.017E-03	10.00
		1.160E-03	10.00
		1.413E-03	10.00

COMPUTED PARAMETER CHECK
CANAL NO. 1

DISCHARGE Q	VISCOSITY VISC	DENSITY RO	ADJ. SP.GR. SGA	LOG COR FACTOR X1	AVERAGE VELOCITY U	SHEAR VELOCITY GRAIN U1S	SHEAR VELOCITY FORM U11S	HYD. RADIUS GRAIN R1	HYD. RADIUS FORM R11	DELTA PRIME DLP	CAP. DELTA DLK	CHARACTERISTIC GRAIN SIZE X	LIFT COR FACTOR Y
5000.00	1.059E-05	1.936	2.655	1.593	2.823	.101	.207	3.530	14.825	1.215E-03	6.018E-04	1.689E-03	.7308

FRACTIONAL DIAMETER D(I)	PARTICLE SHEAR ZII	HIDING FACTOR SUI	GRAIN SHEAR ZIS	TRANSPORT INTENSITY PHS	BED LOAD THICKNESS E	PART.FALL VELOCITY W	EXP. Z	EINSTIENS INTEGRALS I1	EINSTIENS INTEGRALS I2	PE
4.763E-04	2.4818	15.8480	13.9182	2.367E-02	1.058E-04	5.289E-02	1.308	.6372	-4.2458	13.0230
5.798E-04	3.0214	9.8707	10.5534	7.150E-02	1.288E-04	7.096E-02	1.755	.2845	-2.1811	13.0230
6.616E-04	3.4477	7.1169	8.6828	1.365E-01	1.470E-04	8.484E-02	2.098	.1965	-1.5560	13.0230
7.395E-04	3.8535	5.3702	7.3230	2.343E-01	1.643E-04	9.749E-02	2.411	.1530	-1.2253	13.0230
8.202E-04	4.2742	4.1118	6.2191	3.750E-01	1.823E-04	1.099E-01	2.719	.1256	-1.0089	13.0230
9.097E-04	4.7408	3.1337	5.2571	5.797E-01	2.022E-04	1.230E-01	3.041	.1058	-.8496	13.0230
1.017E-03	5.2988	2.3292	4.3673	8.855E-01	2.260E-04	1.375E-01	3.401	.0900	-.7182	13.0230
1.160E-03	6.0465	1.7338	3.7096	1.213E+00	2.578E-04	1.555E-01	3.845	.0760	-.6013	13.0230
1.413E-03	7.3610	1.3262	3.4544	1.375E+00	3.139E-04	1.834E-01	4.536	.0612	-.4761	13.0230

NOTE - ALL PARAMETERS GIVEN IN THE FOOT-POUND-SECOND SYSTEM

OUTPUT

CHANNEL NO. 1

HYDRAULIC DATA

DISCHARGE (CFS)	AREA (SQ.FT)	AVERAGE WIDTH (FT)	AVERAGE DEPTH (FT)	HYDRAULIC RADIUS ACTUAL (FT)	COMPUTED (FT)	GRAIN (FT)	FORM (FT)	AVERAGE VELOCITY (FT/SEC)
5000.00	1764.00	196.00	9.00	9.00	18.35	3.53	14.82	2.82

TRANSPORT DATA
(BED MATERIAL)

TRANSPORT BY SIZE FRACTIONS

MEAN DIAMETER (FT)	PERCENT IN FRACT.	BED LOAD	SUSP. LOAD	TOTAL LOAD
		(POUNDS PER SECOND PER FOOT WIDTH)		
4.763E-04	10.00	2.969E-05	1.2034E-04	1.5003E-04
5.798E-04	10.00	1.205E-04	1.8370E-04	3.0419E-04
6.616E-04	10.00	2.805E-04	2.8121E-04	5.6170E-04
7.395E-04	10.00	5.667E-04	4.3649E-04	1.0052E-03
8.202E-04	10.00	1.063E-03	6.6715E-04	1.7305E-03
9.097E-04	10.00	1.920E-03	1.0172E-03	2.9373E-03
1.017E-03	10.00	3.466E-03	1.5731E-03	5.0389E-03
1.160E-03	10.00	5.789E-03	2.2479E-03	8.0367E-03
1.413E-03	10.00	8.809E-03	2.8221E-03	1.1631E-02
TOTAL	90.00	2.205E-02	9.3492E-03	3.1395E-02

TOTAL TRANSPORT

BED LOAD	SUSP. LOAD	TOTAL LOAD	CONC. BY WT
(TONS PER DAY)			(PPM)
1.867E+02	7.916E+01	2.658E+02	1.976E+01

NOTE-FLOW RESISTANCE KNOWN PRIOR TO BEGINNING COMPUTATIONS
R1 IS COMPUTED FROM GIVEN VELOCITY TO SATISFY LOGRITHMIC VELOCITY DISTRIBUTION
COMPUTED VALUES OF R11 AND R NOT USED IN TRANSPORT COMPUTATIONS

SEDIMENT SIZE BREAKDOWN ASSUMES LOG NORMAL DISTRIBUTION WITH A GRADATION COEFFICIENT OF 1.50

APPENDIX C

SAMPLE SEDIMENT ROUTING COMPUTATIONS

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SAMPLE SEDIMENT ROUTING COMPUTATIONS

To illustrate the application of the concepts presented in Chapter IV, the required computations for an initial trial design are carried out for the irrigation canal sub-system shown in Fig. C-1. Overall characteristics of the system are given in Table C-1. Channel geometry is determined using the depth correction method developed by Simons and Richardson [92] (see Appendix A). The geometry resulting from these computations is given in Table C-2. Bed material transport capacity of the channels is computed by Einstein's method [23] using the computer program presented as Appendix B.

Slope limitations for the system are arbitrarily taken as a maximum available slope of 3.33×10^{-4} and a minimum slope for channel CH(1,1) of 9.0×10^{-5} relating to a minimum sediment inflow at the headworks. Individual channel slopes were selected using the Bose [10] slope equation (Eq. 2.11d) as a reference and departing from the computed value according to relative channel discharge.

Equation 4.2c was applied to those channels represented by solid lines on Fig. C-1 with the entire bed material load assumed to be disposed of with the irrigation supplies. Results are given in Table C-3 with parameters and computations explained column by column in the footnotes.

Since all parameters computed for the trial design lie within feasible hydraulic limits, no modification of the design is required from this standpoint. Further modification would necessarily be based on the feasibility of introducing the computed concentrations into the farm watercourses.

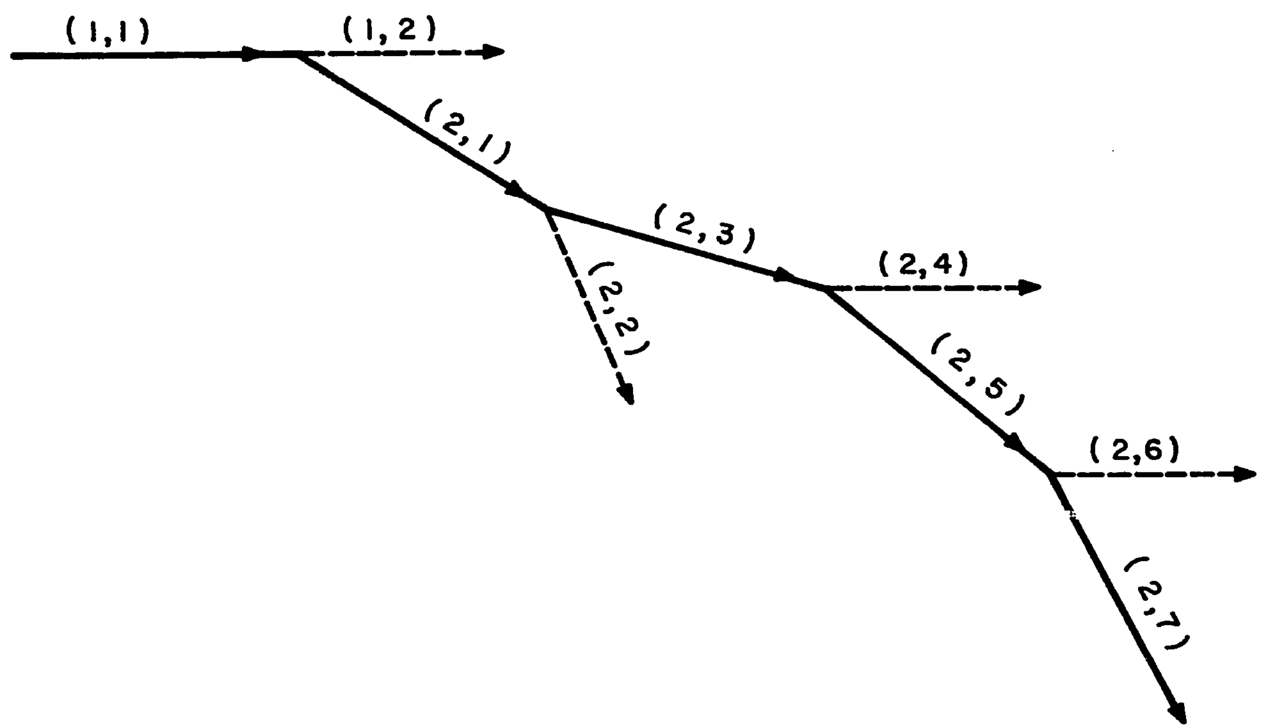

Figure C-1. Irrigation Canal Subsystem for Sample Computations.

Table C-1. Characteristics of Irrigation Canal Subsystem, Sample Problem

(1)	(2)	(3)	(4)	(5)	(6)	(7)
CH(i,j)	Q(i,j) cfs	$Q_s(i,j)$ cfs	$\sum_{ir} Q_{ir}(i,j)$ cfs	L(i,j) mi	d_{50} mm	α mi^{-1}
1, 1	5,000	80	0	10	0.250	0.0020
1, 2	4,420	--	--	--	0.245	--
2, 1	500	20	20	10	0.245	0.0030
2, 2	60	--	--	--	0.240	--
2, 3	400	20	30	10	0.240	0.0035
2, 4	250	--	--	--	0.230	--
2, 5	100	10	30	10	0.230	0.0035
2, 6	30	--	--	--	0.220	--
2, 7	30	8	22	10	0.220	0.004

(1) CH(i,j) = Channel number designation (see Fig. C-1)

(2) Q(i,j) = Channel discharge at head of channel CH(i,j)

(3) $Q_s(i,j)$ = Seepage and evaporation loss from channel CH(i,j). Taken as 8 cfs per 1 million sq. ft of wetted perimeter.

(4) $\sum_{ir} Q_{ir}(i,j)$ = Total irrigation discharge from channel CH(i,j)

(5) L(i,j) = Length of channel CH(i,j)

(6) d_{50} = Mean diameter of the bed material at head of channel

(7) α = Bed material size reduction exponent for the channel. Assumed values based on the work of Rana [82]. $d_{50} = (d_{50})_o \, e^{-\alpha x}$

Table C-2. Computed Channel Geometry for Sample Problem

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
CH(i,j)	Q(i,j) cfs	$S_R x10^4$	$S_o x10^4$	P ft	R ft	V ft/sec	$\tau_o V$ ft-lb/sec	Bed Form
1, 1	5,000	1.06	0.90	196.0	9.00	2.83	0.143	Dunes
1, 2	4,420	1.07	1.00	184.0	8.60	2.79	0.150	Dunes
2, 1	500	1.69	1.50	60.0	3.85	2.16	0.078	Dunes
2, 2	60	2.59	3.00	20.0	1.75	1.71	0.056	Ripples
2, 3	400	1.74	1.68	54.0	3.53	2.09	0.077	Dunes
2, 4	250	1.89	2.00	42.0	2.92	2.03	0.074	Dunes
2, 5	100	2.25	2.36	26.4	2.12	1.78	0.056	Ripples
2, 6	30	2.78	3.33	14.3	1.40	1.49	0.043	Ripples
2, 7	30	2.78	3.33	14.3	1.40	1.49	0.043	Ripples

(1) CH(i,j) = Channel number designation (see Fig. C-1)

(2) Q(i,j) = Water discharge at head of channel CH(i,j)

(3) S_R = Channel slope computed by Bose [10] relation.
$S_R = 2.09\ d_{50}^{0.86}/(1000\ Q^{0.21})$

(4) S_o = Selected channel bed slope

(5) P = Channel wetted perimeter taken from plots developed by Simons and Albertson [95]. See Appendix A.

(6) R = Channel hydraulic radius computed using method developed by Simons and Richardson [92]. See Appendix A.

(7) V = Average velocity in channel computed by method developed by Simons and Richardson [92].

(8) $\tau_o V$ = γDSV = Stream power.

(9) Bed Form predicted from plot of stream power versus mean bed material diameter [94] (Fig. 2.3).

Table C-3. Bed Sediment Distribution for Sample Problem

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
CH(i,j)	Q(i,j) cfs	G(i,j) Ton/day	C(i,j) ppm	$\sum_{ir} Q_{ir}(i,j)$ cfs	$\sum_{ir} \frac{Q_{ir}(i,j) \cdot C_{ir}(i,j)}{370.96}$ Ton/day	$\bar{C}_{ir}(i,j)$ ppm	$\bar{C}_{RAT}(i,j)$	$\bar{C}_R(i,j)$	$\bar{W}(i,j)$
1, 1	5,000	265.8	19.76	0.	0.00				
1, 2	4,420	241.3	20.29	--	--				
2, 1	500	25.1	18.67	20	3.04	56.39	3.02	2.85	1.00
2, 2	60	2.5	15.71	--	--				
2, 3	400	19.6	18.17	30	2.89	35.74	1.97	1.81	0.63
2, 4	250	13.3	19.69	--	--				
2, 5	100	3.4	12.73	30	2.44	30.17	2.37	1.53	0.53
2, 6	30	0.5	6.08	--	--				
2, 7	30	0.5	6.08	22	0.49	8.26	1.36	0.42	0.15

(1) CH(i,j) = Channel number designation

(2) Q(i,j) = Channel discharge at head

(3) G(i,j) = Bed material transport capacity of CH(i,j) as computed by Einstein's method [23]. See Appendix B

(4) C(i,j) = Bed material concentration in CH(i,j). $C(i,j) = [G(i,j)/Q(i,j)] \cdot 370.96$

(5) $\sum_{ir} Q_{ir}(i,j)$ = Total irrigation diversion from CH(i,j)

(6) $\sum_{ir} \frac{Q_{ir}(i,j) \cdot C_{ir}(i,j)}{370.96}$ = Bed material load in irrigation diversions from CH(i,j). Computed by Eq. 4.2c.

(7) $\bar{C}_{ir}(i,j)$ = Average bed material concentration in irrigation diversions from CH(i,j).

$$\bar{C}_{ir}(i,j) = \frac{\sum_{ir} Q_{ir}(i,j) \cdot C_{ir}(i,j)}{\sum_{ir} Q_{ir}(i,j)}$$

(8) $\bar{C}_{RAT}(i,j)$ = Ratio of average bed material concentration in irrigation diversions from CH(i,j) to concentration in CH(i,j). $\bar{C}_{RAT} = \bar{C}_{ir}(i,j)/C(i,j)$.

(9) $\bar{C}_R(i,j)$ = Ratio of average bed material concentration in irrigation diversions from CH(i,j) to concentration in CH(1,1). $\bar{C}_R = \bar{C}_{ir}(i,j)/C(1,1)$

(10) $\bar{W}(i,j)$ = Relative weighting factor for concentration in irrigation diversions from CH(i,j).