Flow in Alluvial Channels

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1. BASIC EQUARIONS IN HYDRAULICS

- Conservation of Mass
- Conservation of Linear Momentum
- Conservation of Energy

1.1 Conservation of Mass (Continuity Equation)

Mass flux into the system - Mass flux out of the system
= Time rate of change in mass in the control volume

$$\mathbf{Q = A_1V_1 = A_2V_2}$$

1.2 Conservation of Linear Momentum

Flux of momentum out of the control volume – Flux of momentum into the control volume + Time rate of change of momentum in the control volume = Sum of the forces acting on the fluid in the control volume

1.3 Conservation of Energy (Bernoulli Equation)

Flux of energy out of the control volume – Flux of energy into the control volume + Time rate of change of energy in the control volume = Rate at which heat is added to a fluid system – the rate at which a fluid system does work on its surroundings

$$\mathbf{\frac{V_1^2}{2g}}$$

1.4 St. Venant Equations

- **Continuity Equation**

$$\mathbf{v}\,\frac{\partial \mathbf{A}}{\partial \mathbf{x}} + \mathbf{A}\frac{\partial \mathbf{v}}{\partial \mathbf{x}} + \mathbf{b}\frac{\partial \mathbf{h}}{\partial \mathbf{t}} = \mathbf{0}$$

- **Momentum Equation**

$$\mathbf{g}\frac{\partial \mathbf{h}}{\partial \mathbf{x}} + \mathbf{v}\frac{\partial \mathbf{v}}{\partial \mathbf{x}} + \frac{\partial \mathbf{v}}{\partial \mathbf{t}} = \mathbf{g}(\mathbf{S_i} - \mathbf{S_e})$$

S_i Bed Slope
S_e Energy Slope

Celerity

$$\mathbf{c} = \sqrt{\mathbf{gh}}$$

$$dh = \frac{2c}{g}\,dc$$

From Continuity Equation A = bh

- **From the momentum equation**

Adding the energy and momentum equations and subtracting them

$$(\boldsymbol{v}+\boldsymbol{c})\frac{\partial(v+2c)}{\partial x}+\frac{\partial(v+2c)}{\partial t}=\boldsymbol{g}\ (\mathbf{S_i}-\ \mathbf{S_e})$$

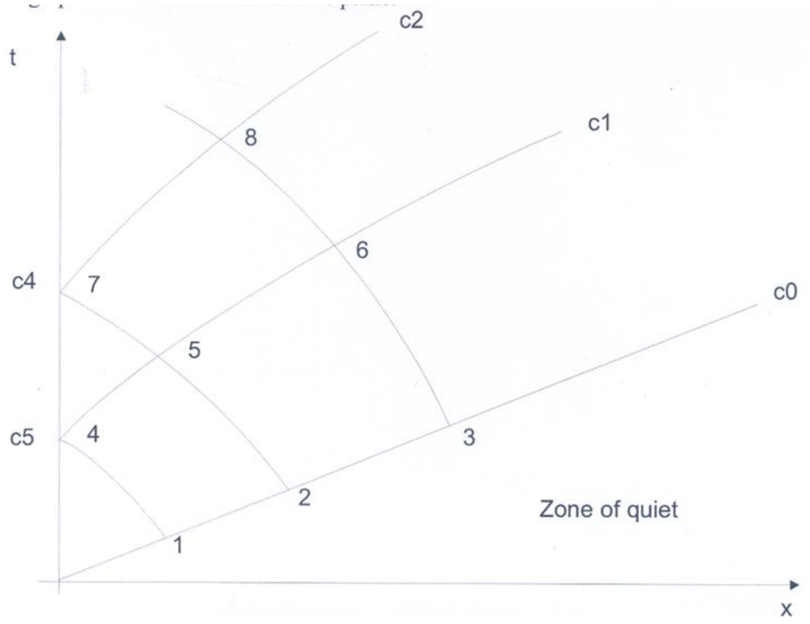
$$(\boldsymbol{v}-\boldsymbol{c})\frac{\partial(v-2c)}{\partial x}+\frac{\partial(v-2c)}{\partial t}=\boldsymbol{g}\ (\mathbf{S_i}-\ \mathbf{S_e})$$

St. Venant Equations

For

And

For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2. SEDIMENT PROPERTIES

2.1 Introduction

River sediments can be carried in suspension as a suspended load or rolling over the bed and saltation as a bed load. Sediment have physical and chemical properties. Watersheds which are usually the source of sediment shape the sediment characteristics. In alluvial rivers and streams such as Nile River, the materials of bed and banks are formed from the sediment material, which are carried through the river and originated in the watershed. In few cases wind blown dump sediments in rivers.

Since sediment in alluvial rivers shape the banks and the bed of these rivers, it is well known that the physical properties of sediment influence the river morphology. Herein, the physical properties of sediment well be explained and only those properties which are of interest for studying river morphology.

2.2 Sediment Size Classification

One of the sediment classification systems which are broadly known is the one proposed by the subcommittee on Sediment Terminology of the American Geophysical Union which is shown in **Table 2.1** below.

Table 2.1 Sediment Classification

Size				Approximate Sieve Mesh Openings per Inch		
Millimeters		Microns	Inches	Tyler	U.S. Standard	Class
4000–2000			160–80			Very large boulders
2000–1000			80–40			Large boulders
1000–500			40–20			Medium boulders
500–250			20–10			Small boulders
250–130			10–5			Large cobbles
130–64			5–2.5			Small cobbles
64–32			2.5–1.3			Very coarse gravel
32–16			1.3–0.6			Coarse gravel
16–8			0.6–0.3	$2\frac{1}{2}$		Medium gravel
8–4			0.3–0.16	5	5	Fine gravel
4–2			0.16–0.08	9	10	Very fine gravel
2–1	2.00–1.00	2000–1000		16	18	Very coarse sand
$1-\frac{1}{2}$	1.00–0.50	1000–500		32	35	Coarse sand
$\frac{1}{2}-\frac{1}{4}$	0.50–0.25	500–250		60	60	Medium sand
$\frac{1}{4}-\frac{1}{8}$	0.25–0.125	250–125		115	120	Fine sand
$\frac{1}{8}-\frac{1}{16}$	0.125–0.062	125–62		250	230	Very fine sand
$\frac{1}{16}-\frac{1}{32}$	0.062–0.031	62–31				Coarse silt
$\frac{1}{32}-\frac{1}{64}$	0.031–0.016	31–16				Medium silt
$\frac{1}{64}-\frac{1}{128}$	0.016–0.008	16–8				Fine silt
$\frac{1}{128}-\frac{1}{256}$	0.008–0.004	8–4				Very fine silt
$\frac{1}{256}-\frac{1}{512}$	0.004–0.0020	4–2				Coarse clay
$\frac{1}{512}-\frac{1}{1024}$	0.0020–0.0010	2–1				Medium clay
$\frac{1}{1024}-\frac{1}{2048}$	0.0010–0.0005	1–0.5				Fine clay
$\frac{1}{2048}-\frac{1}{4096}$	0.0005–0.00024	0.5–0.24				Very fine clay

Normally, sediment particles have irregular shape which is usually expressed by the diameter of equivalent sphere. Therefore, there are common definitions for particle size as follows:

Sieve diameter: It is the diameter of sphere equal to the length of the side of a square sieve opening through which the given particle will just pass.

Sedimentation diameter: It is the diameter of a sphere having the same specific weight and the same terminal velocity as the given particle in the same fluid under the same conditions.

Fall diameter: It is the diameter of a sphere having a specific gravity of 2.65 and the same terminal velocity as the particle when each is allowed to settle alone in quiescent water of infinite extent at 24^oC.

Usually, sediment sample is composed different size fractions. Through the mechanical sieve analysis, every size fraction can be separated and percentage of size fraction by weight or volume can be determined.

The percentage by weight of sediment finer than a certain size is plotted against the grain size on log-normal size distribution. **Figure 2.1** shows an example of grain size distribution plotted on log-normal curve. The physical characteristics of the sample can be obtained through this plot. The median grain size can be obtained as d_{50}. Also, the geometric mean can be obtained as $d_m=(d_{15.9}d_{84.1})^{0.5}$. The median diameter is widely used as a representative of the sample. The standard deviation is obtained as:

$$\sigma_g = (\frac{d_{84.1}}{d_{15.9}})^{0.5}$$

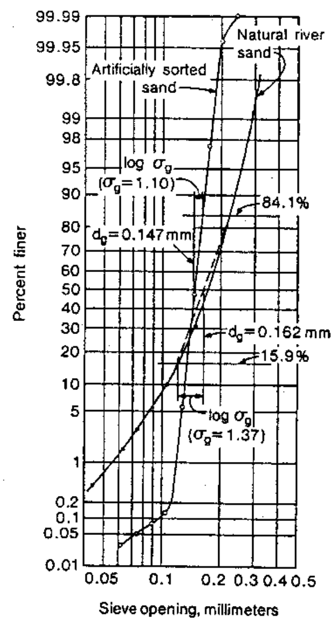

Figure 2.1 Grain Size distribution

2.3 Fall Velocity

The Standard fall velocity of a particle is defined as the average rate of fall that a particle would attain if falling alone in quiescent distilled water of infinite extent at 24°C.

Fall velocity of a particle is affected by water temperature, concentration of sediment, flow turbulent, flow boundaries and sediment size.

Considering a particle in a flow, it comes under the balance of its submerged weight and upward drag force. The drag coefficient is defined as:

$$C_D = \frac{F_D / A}{\frac{1}{2} \rho \omega_s^2}$$

Where,

F_D is the drag force

$$F_d = \frac{1}{6} \pi d^3 g (\rho_s - \rho)$$

d is the diameter of sphere
ρ is the flow density
ρ_s is the sphere density
g is the gravitational acceleration
ω_s is the settling velocity, and
the term ½ $\rho \omega_s^2$ is the dynamic pressure

C_D varies with Reynolds number (R_e), $R_e = \omega_s d / \nu$. For $R_e<1$, $C_D = 24/R_e$ (Stoke's law).
When $R_e>1$, C_D can be obtained through **Figure (2.2)**.

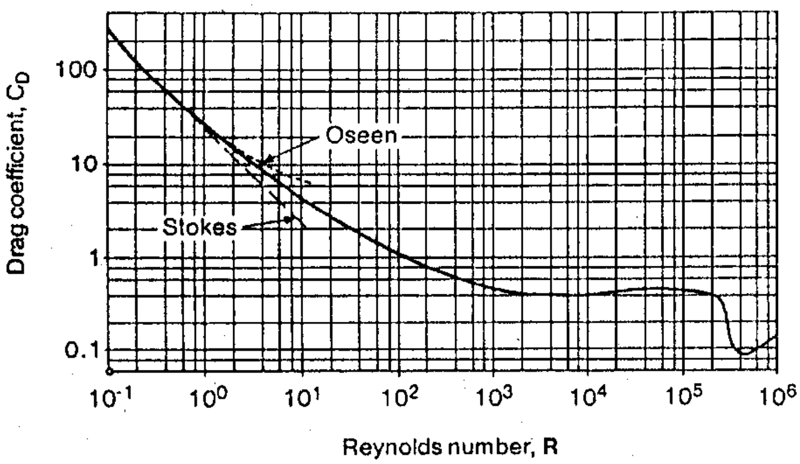

Figure (2.2) C_D versus R_e

The fall velocity (ω_s) can be obtained as follow;

$$\omega_s = \frac{gd^2}{18}\frac{(\rho_s - \rho)}{\mu}$$

Where;

μ   is the dynamic viscosity

3. INCIPIENT MOTION OF SEDIMENT PARTICLES

3.1 Introduction

Considering a sediment particle resting on bed slope of a stream, the water flow around the particle exerts forces which tend to initiate their down-slope motion. The resisting forces of non cohesive material are related to the weight of the particles.

Threshold conditions for initiation of motion is reached when the hydrodynamic force acting on a particle equal the resisting forces.

Figure 3.1 shows the forces acting on a sediment particle which are submerged weight, buoyancy, and lift and drag forces.

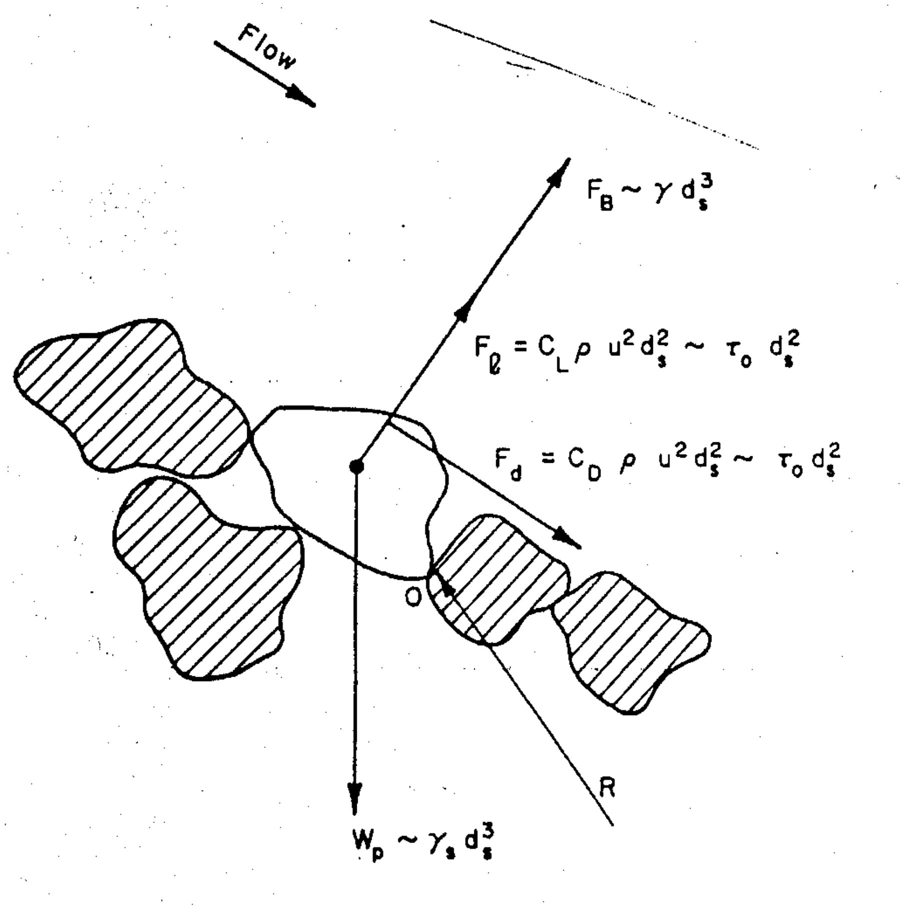

Figure 3.1 Forces Acting on a Particle

3.2 Shields Diagram

Under creeping flow conditions, no lift forces take place. In turbulent flow, lift forces depend on the circulation around the particle and it is assumed that lift and drag forces are proportional.

Considering **Figure 3.1**, the sum of the moment around the point of contact O is:

$$\tau_o C_1 d_s^2 = C_2(\gamma_s - \gamma) d_s^3$$

Where;

τ_o is the bed shear stress
γ_s Specific weight of a sediment particle
γ is the specific weight of the fluid
d_s Sediment size
C_1 , C_2 are products of moment arms and function of the shape of the particle and the geometry of the channel.

The ratio of the two moments is defined as Shields Number (τ_*). Shields (1936) found that τ_* is a function of flow conditions around the particle whether it is laminar or turbulent conditions. The value of Shields Number corresponding to beginning of motion ($\tau_o = \tau_c$) becomes a function of the ratio of sediment size to the laminar sub-layer thickness d_s/δ , or,

$$\delta = \frac{U_* d_s}{\upsilon}$$

Where,

$$\delta = \frac{11.6\upsilon}{U_*}$$

That is:

$$\tau_* = \frac{\tau_c}{(\gamma_s - \gamma) d_s} = f\left(\frac{U_* d_s}{\upsilon}\right)$$

Analysis and experiments of Shields led to a widely accepted diagram shown in **Figure 3.2**

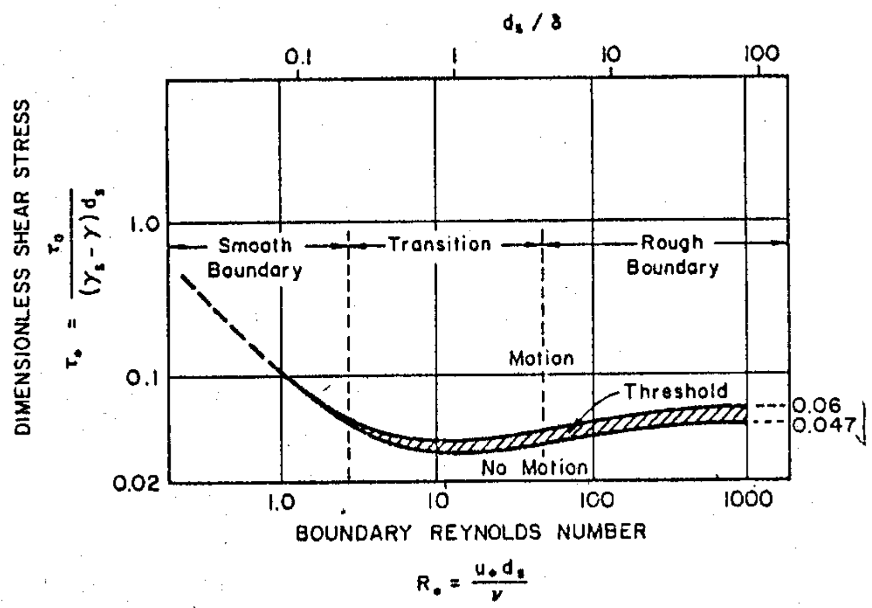

Figure 3.2 Shields Diagram

Shields curve determines the threshold conditions for incipient motion. The area above the curve indicates particles in motion. While, area under curve indicates no motion.

4. SEDIMENT TRANSPORT IN RIVERS

4.1 Introduction

Sediment transport in rivers effect the dynamic stability of the river. The science of sediment transport persists a challenge for engineers studying river engineering since the late of eighteen century. When the flow carrying out sediment moves along the river, it may either cause a deposition of sediment or erosion. Both cases may have a negative or positive impact on local communities and the stabilization of the river itself.

Sediment transport known as sediment load can be classified by two methods:

Method A:
Bed load: This is a part of the load that moves on the bed by rolling, sliding or saltation.
Suspended load: This is part of the load which moves in suspension.

Method B:
Wash load: Finest portion of sediment (mainly silt and clay) that is washed in the channel and barely exists in the bed.
Bed material load: Particles that are found in the bed.

4.2 Sediment Transport Formulas

There are a large number of sediment transport equations. Since sediment transport process is very complex and is affected by many variables, most of these equations do not suit every river. These equations were driven based on flume experiments of ideal conditions or field data.

Sediment transport formulas can be summarized as follows:

1. Suspended load formulas: Einstein (1950)
2. Bed load formulas: DuBoys (1879), Meyer-Peter-Muller (1948), Shields (1936), Parker et al. (1982), and Einstein (1950).
3. Bed material load: Englund-Hansen (1967), Ackers-White (1937), Yang (1937), and Colby (1964).

Sediment transport formulas can be classified according to three different approaches used for defining these formulas:

1. Shear stress approach: DuBoys formula, Shields formula, Einstein bed load function, and Meyer-Peter-Muller formula.
2. Power approach: Yang formula, Engelund-Hansen formula, and Ackers-White formula.
3. Parametric approach: Colby relations

4.3 Colby's Method for Estimating Total Bed Sediment Discharge

The method proposed by Colby in 1964 when tested in Niobrara River of Nebraska, USA, showed good agreement between the estimated and actual data as reported by the Sedimentation Engineering (1977) **Figure 4.1**.

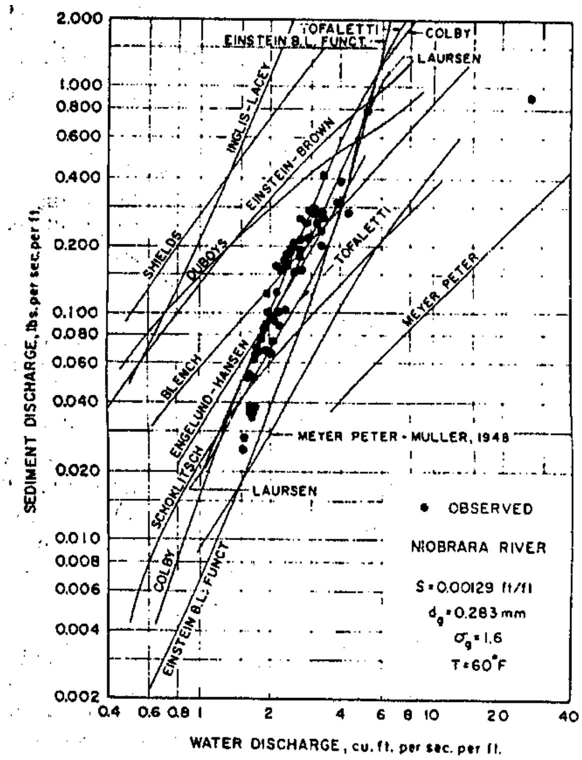

Figure 4.1 Comparisons Between Different Methods for Estimating Total Bed Discharge

Colby's method is based on large amount of field and experimental data. This method covers a wide range of flow depths as will be shown. It has to be noted that this method suites best streams and river having flow depths range from 10 to 100 feet.

Two figures are used in this method, **Figures 4.2 and 4.3**. The method can be summarized as follows:

1. Required data to be known: mean velocity v, depth y_o, median size of bed material d_{50}, Water temperature T^o, and fine sediment concentration C_f..
2. Uncorrected sediment discharge Q_n can be found from **Figure 4.2** for given v, y_o, and D_{50} by reading first q_n knowing v and d_{50} for two depths that bracket the desired discharge and then interpolating on a logarithmic graph of depth versus q_n to get bed sediment discharge per unit width.

3. The correction factors K_1, K_2 in **Figure 4.3** are for the effect of water temperature and fine suspended sediment on bed sediment discharge. The obtained values are good for d_{50}= 0.2 – 0.3 mm, and temperature = 60^oF

4. If bed sediment size is outside 0.2 mm to 0.3 mm range, the factor k_3 in **Figure 4.3** is applied to correct the effect of sediment size.

5. The unit bed sediment discharge q_T corrected to the effect of temperature, presence of fine suspended sediment and sediment size is given by the equation:

$$q_T = (1 + (K_1 K_2 - 1)0.01 K_3) q_n$$

Where;
K_1 is the correction for temperature,
K_2 is the correction for concentration, and
K_3 is the correction for sediment size

The total sediment discharge is given by:

$$Q_T = W q_T$$

Where W is the width of the stream.

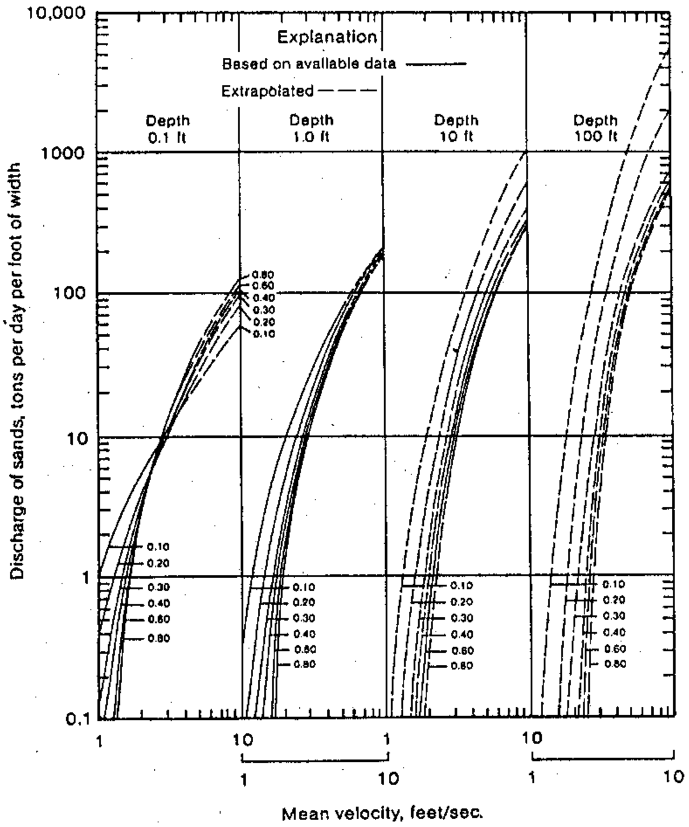

Figure 4.3 Factor k_2

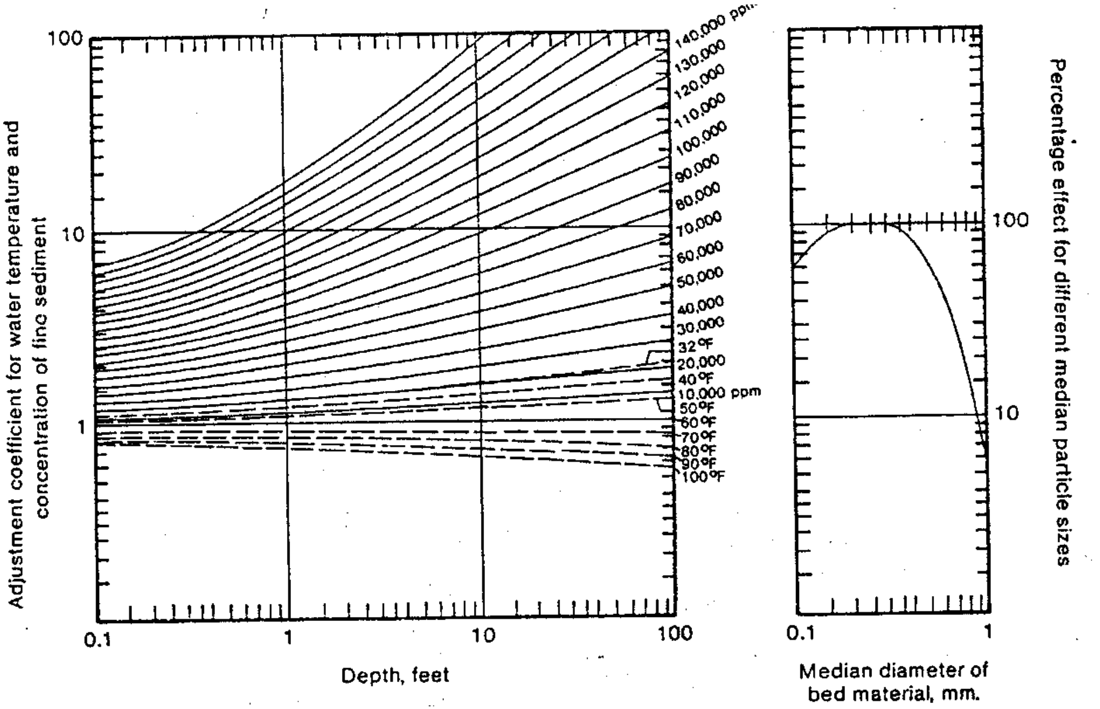

Figure 4.2 Factors K_1, K_3

5. LOCAL SCOUR

5.1 Introduction

Local scour happens around hydraulic structures such as bridge piers and embankment due to hydrodynamic action of running flow. The existence of obstructing structure in the stream forms a vortex at the base of the structure. The vortex is caused by the pile up of water on the up-stream face and subsequent acceleration of the flow around the pier or embankment. If the bed material which is removed away is more than sediment deposition, local scour takes place.

In the followings, local scour caused by the existence of bridge piers and embankments will be focused on since they are the most common types of local scour.

5.2 Factors Influencing Local Scour

The local scour is generally affected by many factors such as:

1. Physical dimension of the piers.
2. Hydraulic characteristics of the flow (depth, velocity,......etc.).
3. Sediment characteristics of the flow (Concentration, sediment size).
4. One of the most important factors is the approach angle of the flow to the structure.

5.3 Types of Local Scour

There are two types of local scour; 1) Clear water scour, and 2) Live bed scour.

1) **Clear water scour:**
This type of scour occurs when there is no movement of bed material of the stream up-stream of the crossing. The acceleration of the flow and vortices created by the piers or abutments cause the material at their bases to move away.

2) **Live bed scour:**
This type occurs when the bed material up-stream of the crossing is moving and bed material moves away the down-stream base of the structure.

Figure 5.1 shows the difference between live bed scour and clear water scour as a function of time.

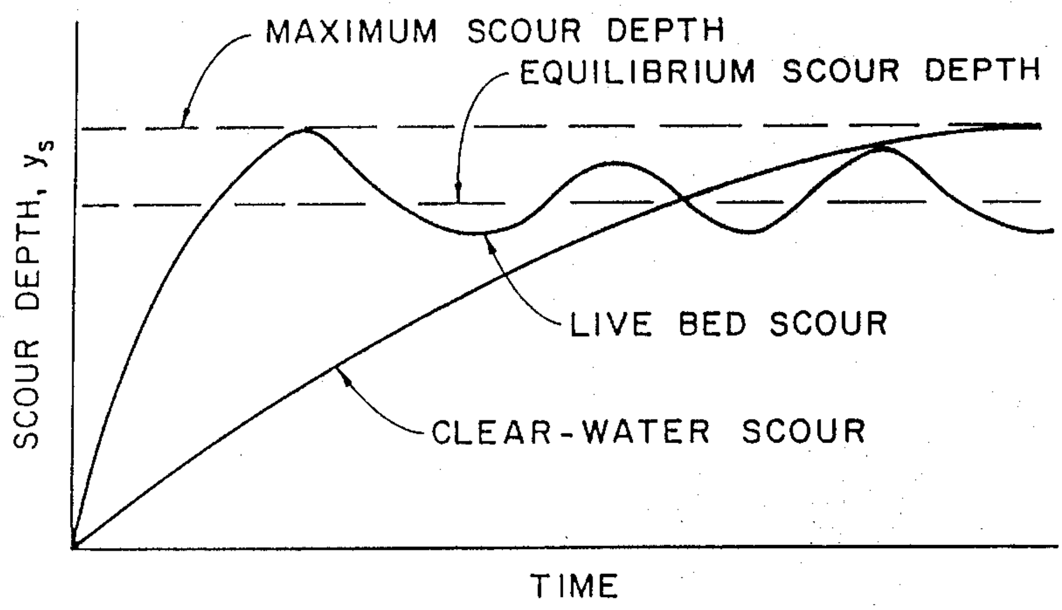

Figure 5.1 Live Bed Scour and Clear Water Scour as a Function of Time

5.4 Estimation of Local Scour

There are large number of formulas developed for estimating the scour depth at piers or abutments. These formulas are based on experimental data with limited field data.

5.4.1 Local Scour at Piers

There are many equations derived for live bed scour in cohesionless sand bed streams. In this section, the equation developed by Colorado State University (CSU) will be presented. CSU equation which is based on experimental data, is widely acceptable and can be used for live bed and clear water scour.

The CSU equation is as follows:

$$\frac{Y_s}{Y_1} = 2.0 K_1 K_2 (a/Y_1)^{0.65} F_r^{0.43}$$

Where:

Y_s is the scour depth
Y_1 is the flow depth just up-stream of the pier
K_1 is the correction for pier shape from Table 4.1 and Figure 4.2
K_2 is the correction for angle of attack of flow from Table 4.2

a is the pier width

F_r is the Froude Number

$$F_r = \frac{V}{\sqrt{g y_1}}$$

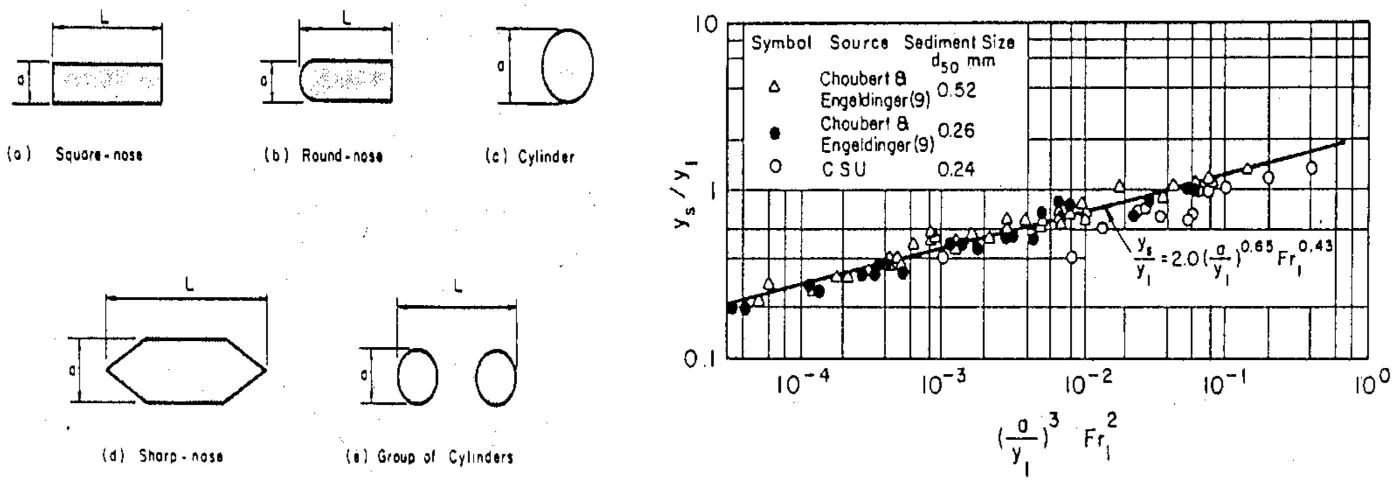

Figure 5.2 Coefficient K_1

Table 5.1

K_1 for pier type

	Type of Pier	K_1
(a)	Square nose	1.1
(b)	Round nose	1.0
(c)	Circular cylinder	1.0
(d)	Sharp nose	0.9
(e)	Group of cylinders	1.0

Table 5.2

Correction factor

Correction factor, K_2 for angle of attack of the flow

Angle	L/a=4	L/a=8	L/a=12
0	1.0	1.0	1.0
15	1.5	2.0	2.5
30	2.0	2.5	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5.0

Angle = skew angle of flow

L = length of pier

5.4.2 Local Scour at Abutments, Embankments, and Spur Dikes

Liu et al. (1961) presented their equation for live bed scour which is a common case. The equilibrium scour depth for local live-bed scour in sand is given by:

$$\frac{Y_s}{Y_1} = 1.1\left(\frac{a}{Y_1}\right)^{0.4} F_{r1}^{0.33}$$

Where,

Y_s is the equilibrium depth of scour (measured from the mean bed level to the bottom of the scour hole).

Y_1 is the average up-stream flow depth in the main channel.

a is the abutment and embankment length (measured at the top of the water surface and normal to the side of the channel.

F_{r1} Froude Number at the up-stream.

6. RIVER DEGRADATION AND AGRADATION

6.1 Introduction

The instability of river bed with time causes the process of aggradation and degradation. Aggradation occurs when sediment deposits in the stream bed are larger than sediment removed away over a period of time. Thus, resulting in an increase in bed elevation of the river or the stream. If the rate of sediment removed from the stream is larger than the rate of sediment deposited, then, the river bed is degrading.

Degradation and aggradation may cause serious problems if they take place at high rate. For example, unfavorable conditions may resulted near hydraulic structures or changes in river morphology may happens.

Aggradation takes place mainly in reservoir up-stream of dams. Therefore, it will be discussed separately in reservoir sedimentation. In the followings, degradation in rivers and streams will be highlighted.

6.2 River Degradation

Usually, Degradation occurs as a result of constructing a dam and retaining of sediment material up-stream the dam. In this case, the degradation occurs down-stream the dam. Just after the construction of the dam the degradation rate becomes very high. But, it slows down afterward.

Taking the Nile River as an example, the Nile down-stream the Aswan High Dam suffered from bed degradation . According to Shalash (1974), field observations showed significant degradation between Aswan and Esna as shown in **Figure 6.1**.

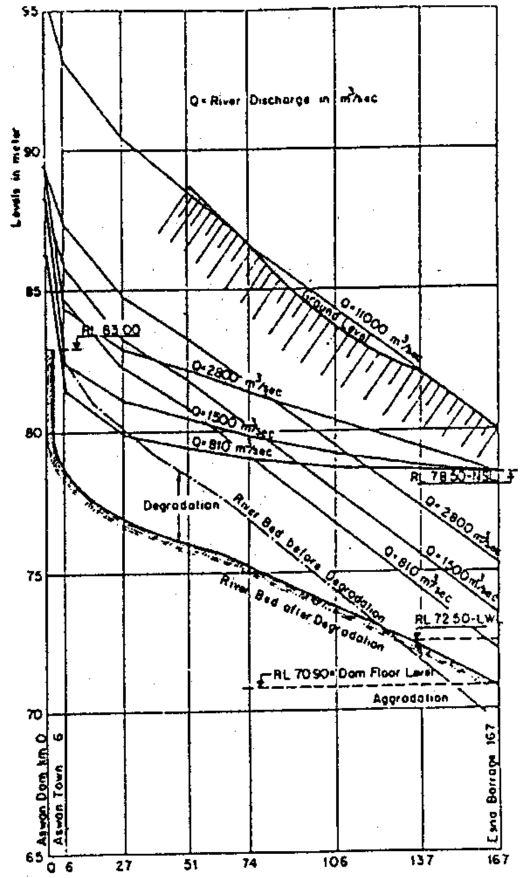

Figure 6.1 Degradation in the Nile After Aswan High Dam

Degradation process is a function of the following variables:

1. Flow characteristics
2. Concentration of suspended material in the flow
3. Characteristics of river bed sediment
4. Man made structures and developments along the river
5. River channel geometrical properties
6. The existence and location of the controls in the down-stream channel

In order to simplify the analysis of degradation, the following assumptions are usually made:

1. Water released from the reservoir is clear and sediment is transported as bed load
2. Irregularities of river course are ignored
3. Sediment characteristics of degrading reach are constant

6.3 Estimation of Degradation

Gamal Mostafa (1957) proposed a formula for defining the equilibrium profile of river bed. Mostafa utilized Einstein's Formula:

$$\frac{U}{U_*} = 5.75\log(12.27\frac{R}{k_s}x)$$

and Shield's relation for threshold conditions

$$\frac{\tau_c}{\gamma_s^{\backslash} D_{50}} = 0.06$$

Mostafa suggested that

$$\frac{Q}{A} = 5.75(\frac{0.06\gamma_s^{\backslash} k_s / Y}{\rho})^{0.5} \log(12.27\frac{R}{k_s}x)$$

Where;

Q	is the flow discharge
A	is the wetted area
γ_s'	is the specific weight of submerged particles
k_s	is bed roughness which is equal to D_{98} size of bed material
R	is the hydraulic radius

$$Y = \frac{0.06}{k}$$

$$k = \frac{\tau_c}{\gamma_s^{\backslash} k_s}$$

τ_c is the critical shear stress as defined by Shields
x is the Einstein correction factor as in **Figure 5.2**.

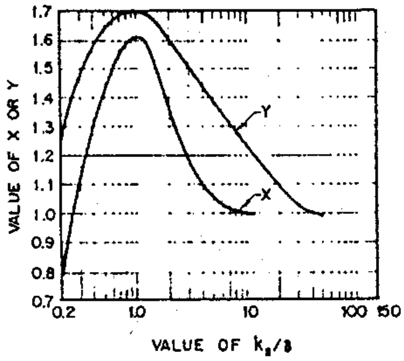

Figure 6.2 Einstein Correction Factor

Finally, Mostafa obtained the following formula for slope relation:

$$S = \frac{0.06\gamma_s^{\prime} k_s / y}{\gamma R}$$

Method of Mostafa can be summarized as follows:

1. Assume x=y=1
2. Compute the corresponding value of R related to each value of Q
3. From Figure 5.2 check the values of x and y
4. If x and y are not equal to unity, choose another values for x and y so that the assumed values are equal to the computed values
5. If the computed x , y equal to the assumed values, calculate S

7. DEPOSITION IN RIVER CHANNELS

7.1 Introduction

As stated earlier in section 3.1, sediment transport plays a pivotal role in shaping the channel morphology. Term channel morphology usually refers to the general configuration of the channel bed and branches which takes place due to erosion of the primary elements and deposition of debris and sediment.

Sediment deposition may have negative and/or positive impact on river system. Deposition of sediment may equalize the scour which takes place around hydraulic structures and may provide the required material for river mining which can be used for agriculture purposes and other municipal works. However, deposition in the river channel may have a negative impact on navigation.

7.2 Channel Classification

Deposition in rivers is a result of low stability of river system. It occurs when there are high bed load transport, large sediment size, large sediment load, and high flow velocity and stream power.

There are more than channel classification proposed by different researchers. Some of these classifications take into account rates of erosion and deposition. Classification which is proposed by Shen et al. 1981 is widely used by river engineers. This classification is shown in **Figure 7.1**. The classification indicates stages of river development from straight channel to meander to braided channel.

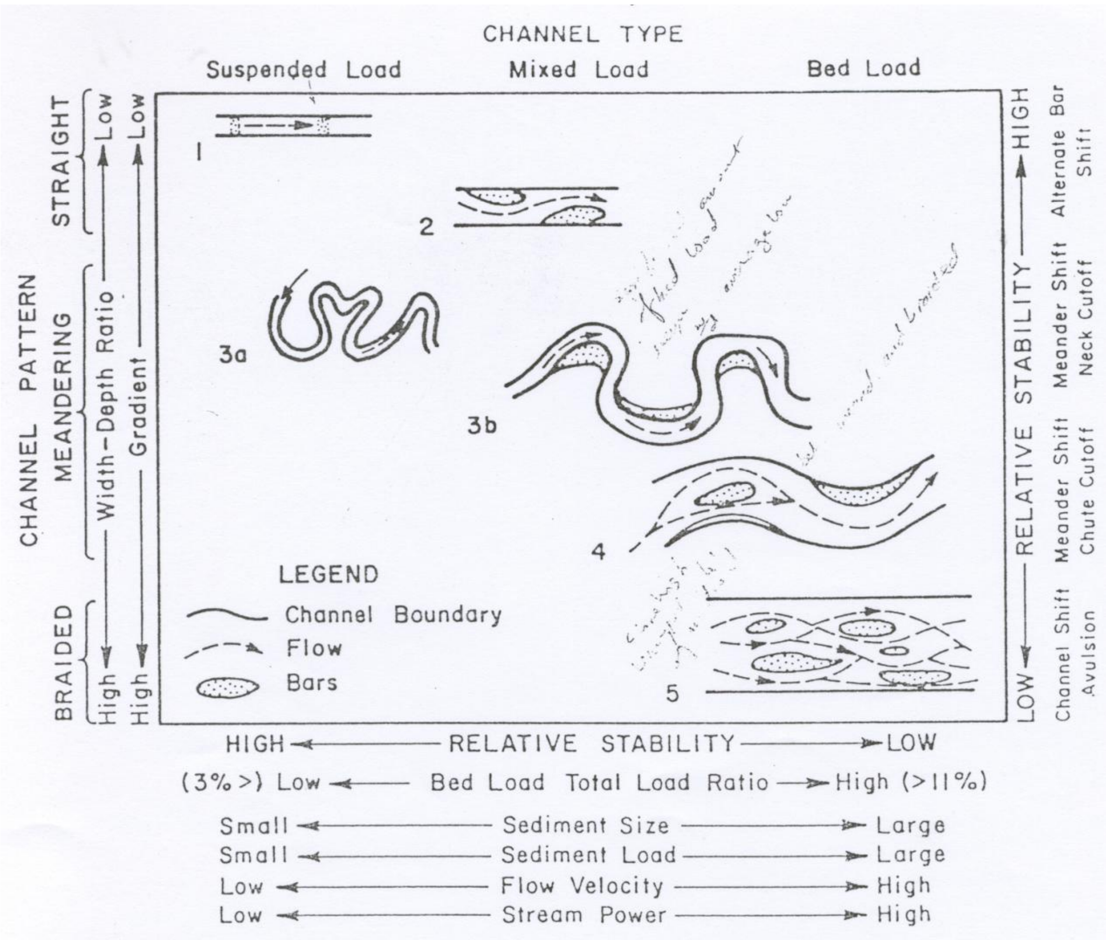

Figure 7.1 Channel Classification After Shen et al. 1981

7.3 Types of Depositions

As bed load increases with suspended sediment, the channel form changes from meandering towards braiding gradually until the bed load becomes dominant over the suspended load. In this case, the channel becomes fully braided. This section focuses on types of depositions take place as river changes from meandering to braided. These depositions can be classified as follows;

1. Alternate bars
2. Point bars
3. Middle bars

The difference between bars is outlined in **Figure 7.2**.

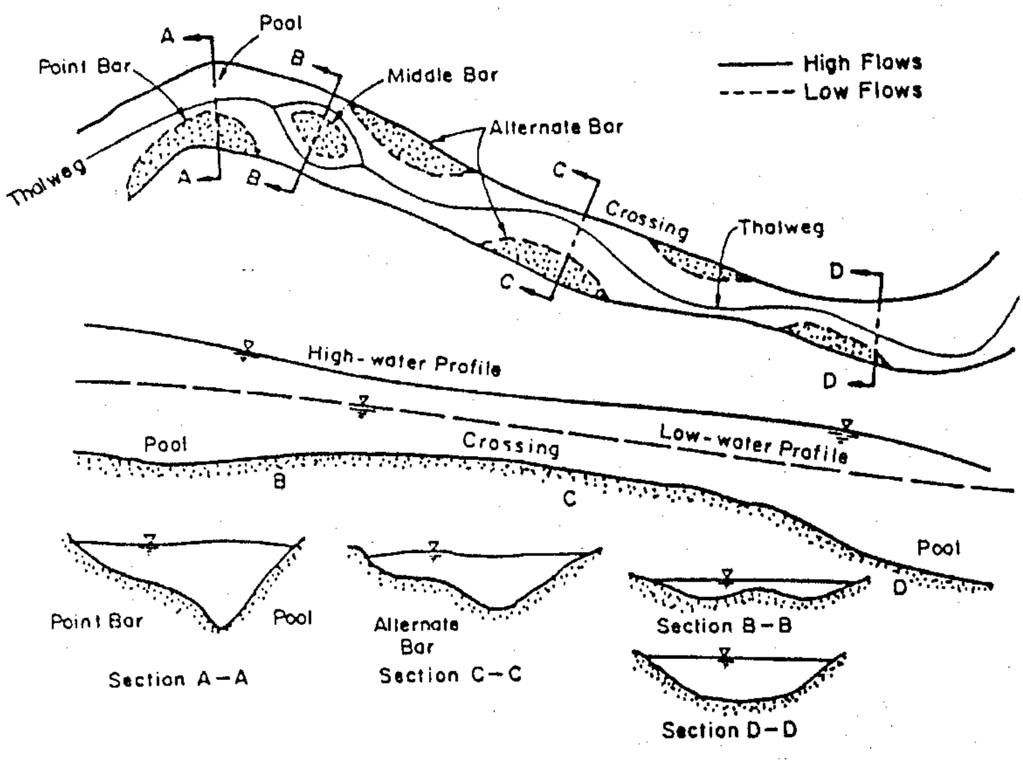

Figure 7.2 Different Types of Bars

1. Alternate bars:
   Alternate bars are formed when the channel has a mix of suspended load and bed load. These bars are formed on a transverse configuration on each side of the channel.

2. Point bars:
   As bed load developed above suspended load, the point bars are formed in the inner side of bends.

3. Middle bars:
   As bed load fully developed and suspended load is reduced, the channel becomes relatively unstable. In this case, the channel is characterized by middle bars which dramatically reduces the channel efficiency for navigation and flow of water.

8. STABLE CHANNELS AND REGIME EQUATIONS

8.1 Introduction

The term of stable channel indicates the channel does not undergo silting or scouring. In reality, alluvial channels are subjected to processes of scours and depositions. Design methods of stable channel are based on utilizing field and experimental data.

In designing stable alluvial channels, the designed discharge and terrain slope are defined. Factors to be determined are width, depths, and channel slope. The design velocity is based on non silting non scouring criteria.

Methods for the design of stable channels follows either rational approach or regime approach. The rational approach is based on theories of hydraulics and sediment equilibrium for defining hydraulic geometry. The rational approach does not rely on real field data.

On the other hand, the regime which is developed in India and Pakistan for Indian and Pakistani canals in the nineteen century is based on measurements from alluvial canals and streams.

8.2 Regime Equation by Blench (1961)

There are numerous equations proposed for the design of stable channels based on regime concept. The proposed equations by Blench (1961) are widely acceptable since it has the greatest generality. Blench's equations are limited in its application to canals only.

Blench's equations are applicable to canals which have the following characteristics; straight channels, boundaries behave as hydraulically smooth boundary, bed width>3 times flow depth, steady discharge and sediment load, operating velocity is less than the critical velocity, and width, depth and slope adjusted to their final values.

Blench method consists of three independent conditions; bed factor, side factor, and flow resistance equations. Through these equations, width, depth and slope of the channel may be determined.

$$F_b^{\backslash} = \frac{U^2}{D}$$

The bed factor $F_b^{'}$,

Where;

U is the mean velocity in feet per second
D is the depth of flow in feet

The side factor F_s

$$F_s = \frac{U^2}{B^\backslash}$$

B' is the average width equal to the cross-sectional area divided by D
The flow resistance equation;

$$\frac{U^2}{gDS} = 3.63(1 + \frac{C}{2330})(\frac{U^\backslash B}{\upsilon})^{\frac{1}{4}}$$

Where;

ν is the kinematic viscosity
C is the suspended sediment concentration in part per million by weight]

Empirical values for the bed factor and side factor as provided by Blench are:

$F_b = 1.9\ d^{1.9}$
$F_s = 0.1$ for slightly cohesive banks
$F_s = 0.2$ for medium cohesive banks
$F_s = 0.3$ for highly cohesive banks

where d is the medium grain size in millimeter

From above equations, the channel width, depth, and slope are obtained as follows;

$$B^\backslash = (\frac{F_b Q}{F_s})^{\frac{1}{2}}$$

Where;

Q is the flow discharge in cubic feet per second

$$D = (\frac{F_s Q}{F_b^2})^{\frac{1}{3}}$$

and

$$S = \frac{F_b^{5/6} F_s^{1/12} \upsilon^{1/4}}{3.63(1 + \frac{C}{2330}) g Q^{1/6}}$$

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