1. CHAPTER ONE - INTRODUCTION
1.1. River Characteristics

1.1.1 Introduction

The primary function of a river channel is the conveyance of water and sediment. It should be appreciated that this primary function cannot be stopped. Nor can the long-term average be changed by measures carried out in the riverbed. Thus alterations in space and time can only be made within the context of ultimate equilibrium.

The most conspicuous aspect of a river channel, apart from its size, is the amount of water it carries. This is best shown in a hydrograph. A hydrograph is a time series of water level data or discharge data. Changes in discharge cause changes in water level in the river channel. At very high discharges a river channel overflows its banks on to the adjacent land. This periodically flooded land is called the flood plain. Whilst in the upper reaches the flood plains are usually narrow or even non-existent, in the lower reaches of a river the flood plains could be tens of kilometers wide.

1.1.2 The Catchment Area

Total area from which surface runoff flows to a given point of concentration is called a *catchment area, drainage basin, drainage area, or a watershed*. Hence, a catchment area is always connected to a certain point of concentration, the lowest point of the respective basin. Therefore, whenever specifying a watershed area of a given stream, it should always be clearly stated upstream of which point on the stream course it is related to (Ref. Fig. 1.1). By summing the partial watershed areas of all the tributaries, and by adding the areas draining directly into the stream, total area of the watershed above the concentration point is obtained.

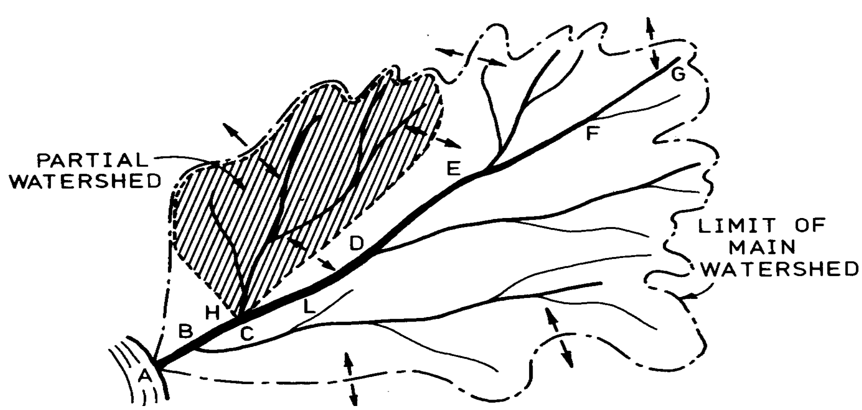

Figure 1.1: Watershed boundaries

The imaginary line delimiting various watersheds is known as *water dividing line* or *water-divide.* Its configuration depends on the topography only, and it runs along the highest points of the surrounding area. Precipitation falling outside the area enclosed by this line will form a runoff flowing to another stream, over another catchment area.

The farther downstream along the stream the point of concentration, the more tributaries will join the stream and the larger the respective watershed. Sudden increase of area indicates the inflow of a large tributary, the point at which the whole partial watershed of the tributary joins the watershed of the main watercourse. Gradual increases are derived mainly from overland flow areas or small gullies.

1.1.3 Watershed Forms

Form of a watershed varies greatly, however, and is tied to many factors including climatic regime, underlying geology, morphology, soils, and vegetation.

Drainage Patterns: One distinctive aspect of a watershed when observed in planform (map view) is its drainage pattern (Figure 1.2). Drainage patterns are primarily controlled by the overall topography and underlying geologic structure of the watershed.

Stream Ordering: A method of classifying, or ordering, the hierarchy of natural channels within a watershed was developed by Horton (1945). Several modifications of the original stream ordering scheme have been proposed, but the modified system of Strahler (1957) is probably the most popular today.

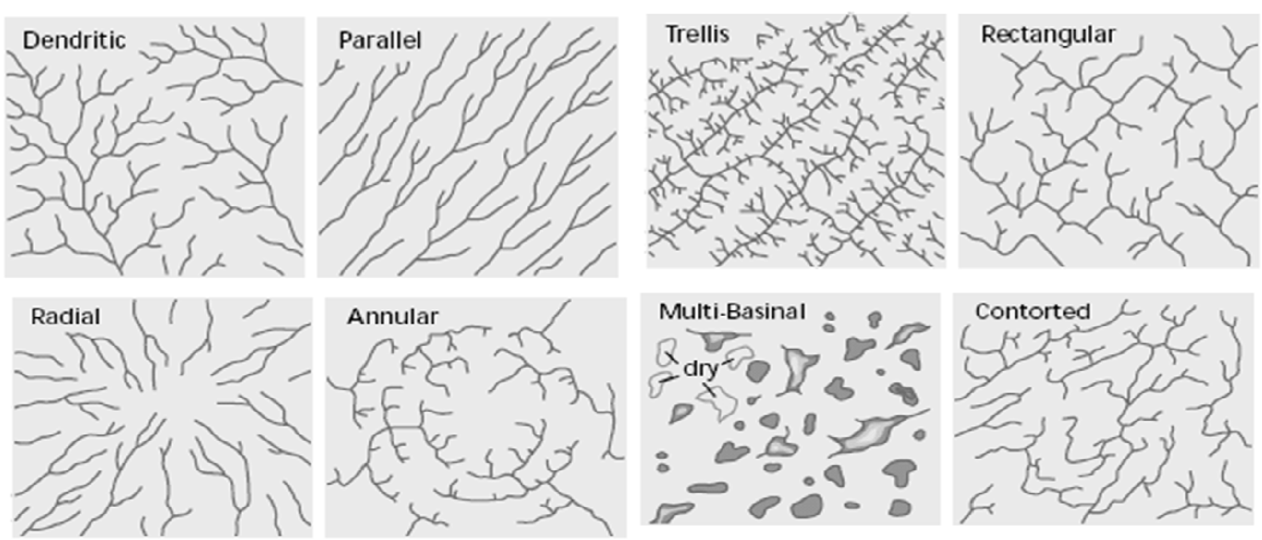

Figure 1.2: Watershed drainage patterns. Patterns are determined by topography and geologic structure.

Strahler's stream ordering system is portrayed in Figure 1.3. The uppermost channels in a drainage network (i.e., headwater channels with no upstream tributaries) are designated as *first-order* streams down to their first confluence. A *second-order* stream is formed below the confluence of two first-order channels. *Third-order* streams are created when two second-order channels join, and so on. Note in the figure that the intersection of a channel with another channel of lower order does not raise the order of the stream below the intersection (e.g., a fourth-order stream intersecting with a second-order stream is still a fourth-order stream below the intersection).Within a given drainage basin, stream order correlates well with other basin parameters, such as drainage area or channel length. Consequently, knowing what order a stream is can provide clues concerning other characteristics such as which longitudinal zone it resides in and relative channel size and depth.

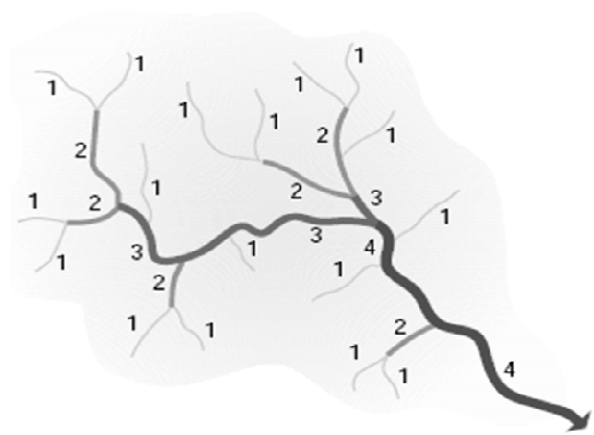

Figure 1.3: Stream ordering in a drainage network

Channel and Ground Water Relationships: Interactions between ground water and the channel vary throughout the watershed. In general, the connection is strongest in streams with gravel riverbeds in well-developed alluvial floodplains.

Figure 1.4 presents two types of water movement:

- *Influent or "losing" reaches* lose stream water to the aquifer.
- *Effluent or "gaining" reaches* receive discharges from the aquifer.

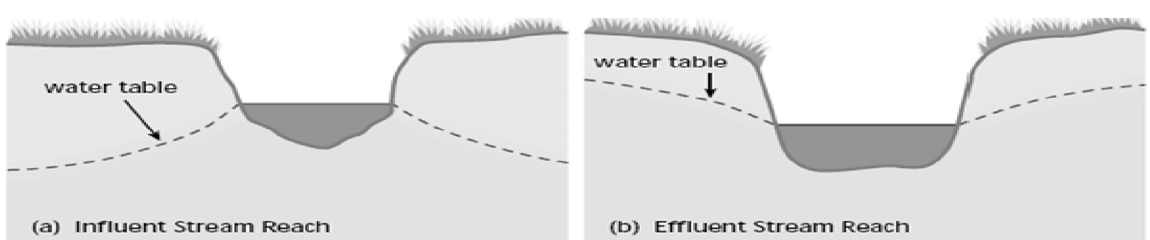

Figure 1.4: Cross sections of (a) influent and (b) effluent stream reaches.

Practitioners categorize streams based on the balance and timing of the stormflow and baseflow components. There are three main categories:

- *Ephemeral streams* flow only during or immediately after periods of precipitation. They generally flow less than 30 days per year.
- *Intermittent streams* flow only during certain times of the year. Seasonal flow in an intermittent stream usually lasts longer than 30 days per year.
- *Perennial streams* flow continuously during both wet and dry times. Baseflow is dependably generated from the movement of ground water into the channel.

Discharge Regime: Streamflow is one of the variables that determine the size and shape of the channel. There are three types of characteristic discharges:

- **Channel-forming (or dominant) discharge:** If the streamflow were held constant at the channel-forming discharge, it would result in channel morphology close to the existing channel. However, there is no method for directly calculating channel-forming discharge. An estimate of channel-forming discharge for a particular stream reach can, with some qualifications, be related to depth, width, and shape of channel. Although channel-forming discharges are strictly applicable only to channels in equilibrium, the concept can be used to select appropriate channel geometry for restoring a disturbed reach.

- **Effective discharge***:* The effective discharge is the calculated measure of channel-forming discharge. Computation of effective discharge requires long-term water and sediment measurements, either for the stream in question or for one very similar. Since this type of data is not often available for stream restoration sites, modeled or computed data are sometimes substituted. Effective discharge can be computed for either stable or evolving channels.
- **Bankfull discharge***:* This discharge occurs when water just begins to leave the channel and spread onto the floodplain. Bankfull discharge is equivalent to channel-forming (conceptual) and effective (calculated) discharge.

1.1.4 Longitudinal View along a Stream

Channel width and depth increase downstream due to increasing drainage area and discharge. Related structural changes also occur in the channel, floodplain, and transitional upland fringe, and in processes such as erosion and deposition. Even among different types of streams, a common sequence of structural changes is observable from headwaters to mouth.

The overall longitudinal profile of most streams can be roughly divided into three zones:

- Zone 1, or *headwaters (or upper course)*, often has the steepest gradient. Sediment erodes from slopes of the watershed and moves downstream. Typically erosive stream characteristics.
- Zone 2, *the transfer zone (or Middle course)*, receives some of the eroded material. It is usually characterized by wide floodplains and meandering channel patterns. Longitudinal slope of the stream gradually eases; tributaries join the main stream, and therefore often sudden changes of flow regime. Although stretches of erosion and deposition frequently exchange, both in space and time, this transitional reach of the stream is on the whole generally the most stable and balanced part. Stream characteristics obtained from the middle course are frequently used as basis for design of stream training projects.
- *Zone 3, the depositional zone (or Lower course)*: Longitudinal slope flattens; discharge increases in Zone 3, the *primary depositional zone (ref Fig. 1.5 a and 1.5b)*. gradual deposition of sediment eroded upstream, hence relatively short-period shifting and changing of the main stream channel.

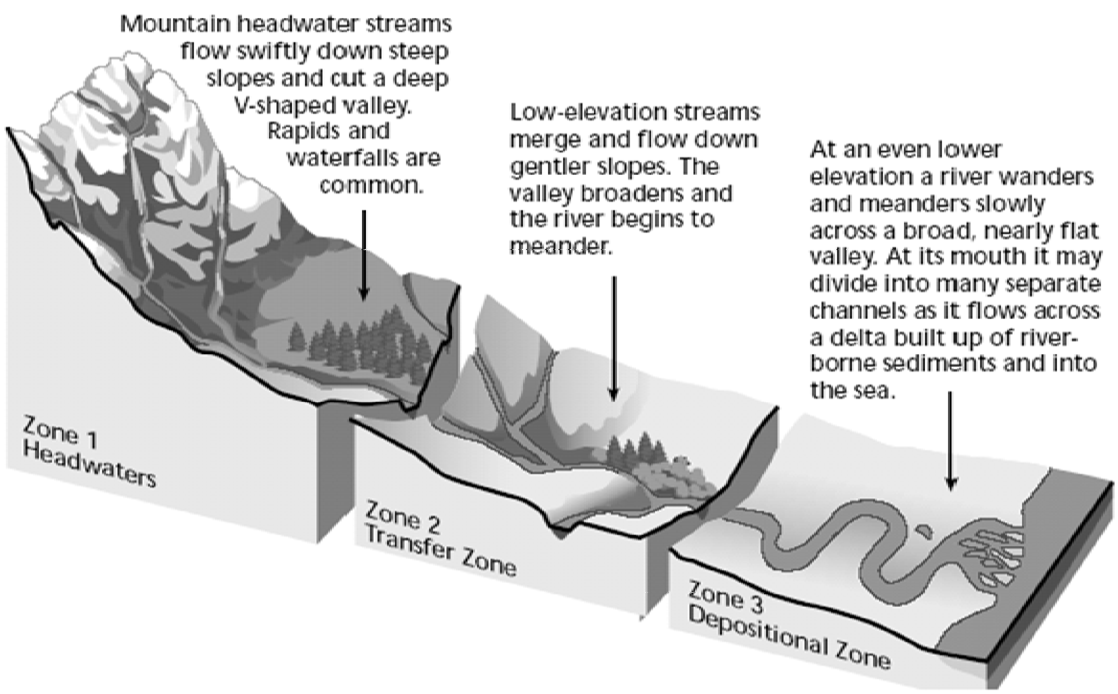

Figure 1.5: Three longitudinal profile zones. Channel and floodplain characteristics change as rivers travel from headwaters to mouth.

Though the figure displays headwaters as mountain streams, these general patterns and changes are also often applicable to watersheds with relatively small topographic relief from the headwaters to mouth. It is important to note that erosion, transfer, and deposition occur in all zones, but the zone concept focuses on the most dominant process.

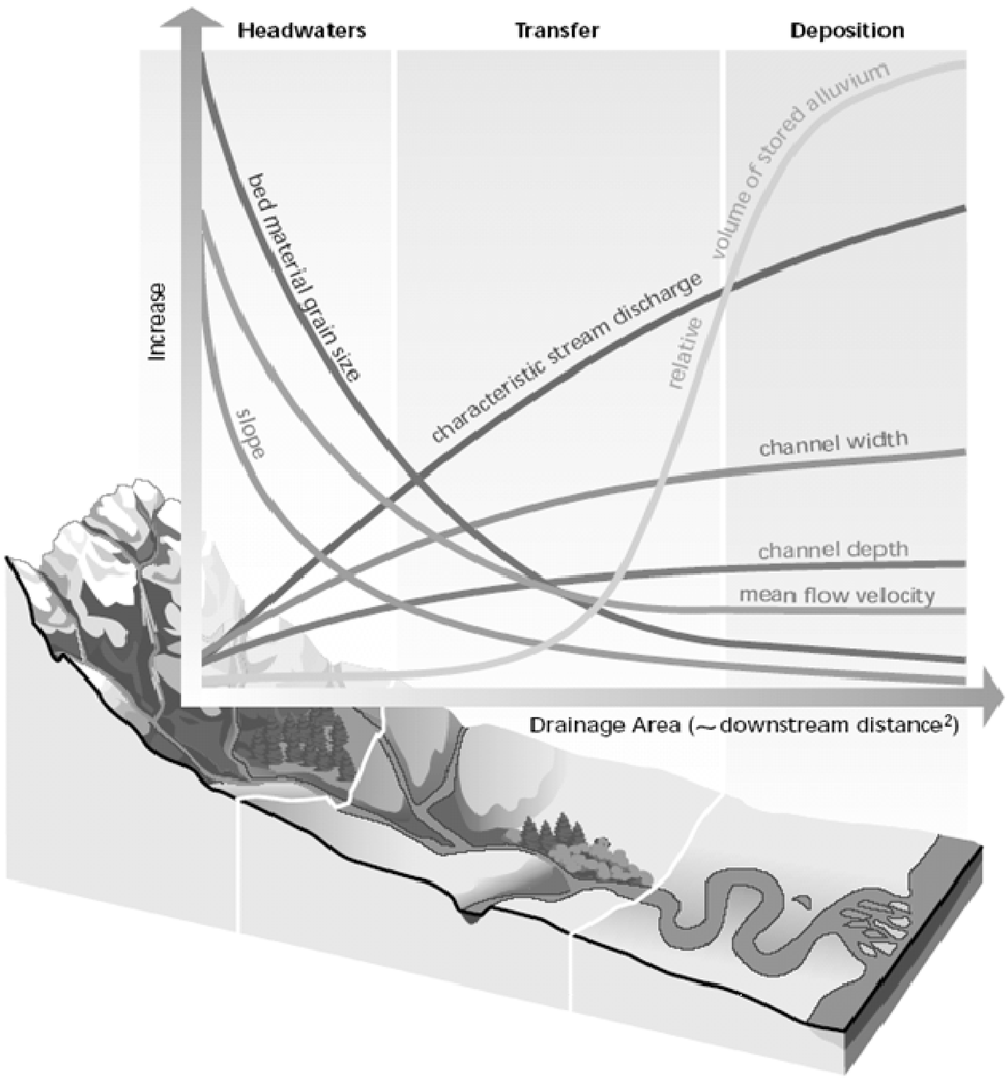

Figure 1.6: Changes in the channel in the three zones. Flow, channel size, and sediment characteristics change throughout the longitudinal profile.

1.2. River Hydraulics

1.2.1 Introduction

Although the hydraulic behavior of alluvial channels is known to be in many respects different from the behavior of rigid-boundary channels, there is to this day no theoretical set of equations applying specifically to the flow in movable-bed channels. In general, the accepted procedure is to use the kinematic and dynamic equations as developed for rigid-boundary channels, and to introduce empirical coefficients or relationships supposed to suitably modify them in order to better fit movable-bed conditions. This is certainly not a satisfactory solution but it is the only feasible one with the present state of the art.

It can hardly be sufficiently stressed that the limitations and constraints of theoretical considerations, when applied to alluvial streams, should always be borne in mind by practicing engineers, if a correct understanding of actual physical processes is to be reached. It has already been pointed out in the opening sentences of the present chapter that substantial differences exist between the rigid-boundary and movable-bed hydraulics, and at this point it may be of interest to mention some of them.

As soon as the flow has started, an alluvial channel begins to continuously adapt and change its deformable boundaries. Indeed, its characteristic roughness is determined not only by some mean grain size protruding into the water, but usually by bed forms as well, or often even more by the latter factor than by the former. In the latter case of form roughness, the ratio between the roughness height (the effective height of the roughness elements, a measure of linear dimension designating their effect upon the flow) and depth of flow is generally several orders of magnitude larger than for fixed-bed channels.

Moving sediment elements and their continuous shifting of position are likely to induce additional shear stresses; progressive movement of bed forms, on the other hand, may cause disturbances in flow pattern due to changing water depth. Moreover, suspended sediment carried by the water, especially when in higher concentrations, often has also an influence upon the turbulence level of the flow.

Although the quantitative analysis and the extent of the mentioned and other influences due to the movable-bed conditions of alluvial channels largely elude our present knowledge, they should be carefully kept in mind whenever applying flow concepts developed for rigid-boundary conditions to flows in deformable conduits.

1.2.2 Types of flow and Water Movement in Rivers

Descriptions of various types of flow are given in the following.

Laminar versus turbulent: Laminar flow occurs at relatively low fluid velocity. The flow is visualized as layers, which slide smoothly over each other without macroscopic mixing of fluid particles. The shear stress in laminar flow is given by Newton's law of viscosity as

$$\tau_v = \mu \frac{du}{dz} = \rho\, \nu \frac{du}{dz} \tag{1.1}$$

Where μ is dynamic viscosity, *ρ* is density of water and *ν is kinematic* viscosity ($\nu = 10^{-6}\ m^2/s$ at 20^0C).

Most flows in nature are turbulent. Turbulence is generated by instability in the flow, which trigger vortices. However, a thin layer exists near the boundary where the fluid motion is still laminar. A typical phenomenon of turbulent flow is the fluctuation of velocity

$$U = u + u' \quad ; \quad W = w + w' \tag{1.2}$$

Where *U* and *W* are instantaneous velocity, in x and z directions, respectively

u and w are time-averaged velocity, in x and z directions, respectively

u' and w' are instantaneous velocity fluctuation, in x and z directions, respectively

Turbulent flow is often given as the mean flow, described by *u* and *w*.

In turbulent flow the water particles move in very irregular paths, causing an exchange of momentum from one portion of fluid to another, and hence, the turbulent shear stress (Reynolds stress). The turbulent shear stress, given by time-averaging of the Navier - Stokes equation, is

$$\tau_t = -\rho\, \overline{u'\, w'} \tag{1.3}$$

Note that $\overline{u'w'}$ is always negative. In turbulent flow both viscosity and turbulence contribute to shear stress. The total shear stress is

$$\tau = \tau_v + \tau_t = \rho\nu \frac{du}{dz} + \left(-\rho\overline{u'w'}\right) \tag{1.4}$$

***Steady versus unsteady*:** A flow is steady when the flow properties (e.g. density, velocity, pressure etc.) at any point are constant with respect to time. However, these properties may vary from point to point. In mathematical language,

$$\frac{\partial\ (\text{any flow property})}{\partial t} = 0 \qquad (1.5)$$

In the case of turbulent flow, steady flow means that the statistical parameters (mean and standard deviation) of the flow do not change with respect to time. If the flow is not steady, it is unsteady.

Uniform versus non-uniform: A flow is uniform when the flow velocity does not change along the flow direction.

Boundary layer flow: Prandtl developed the concept of the boundary layer. It provides an important link between ideal-fluid flow and real-fluid flow. Here is the original description. *For fluids having small viscosity, the effect of internal friction in the flow is appreciable only in a thin layer surrounding the flow boundaries.* However, we will demonstrate that the boundary layer fills the whole flow in open channels.

The boundary layer thickness (δ) is defined as the distance from the boundary surface to the point where $u = 0.995\ U$. The boundary layer development can be expressed as

$$\text{laminar flow} \quad \frac{\delta}{x} = 5\left(\frac{U\,x}{\nu}\right)^{-0.5} \qquad \text{when } Re_x = \frac{U\,x}{\nu} < 5 \times 10^5$$

$$\text{turbulent flow} \quad \frac{\delta}{x} = 0.4\left(\frac{U\,x}{\nu}\right)^{-0.2} \qquad \text{when } Re_x = \frac{U\,x}{\nu} > 5 \times 10^5$$

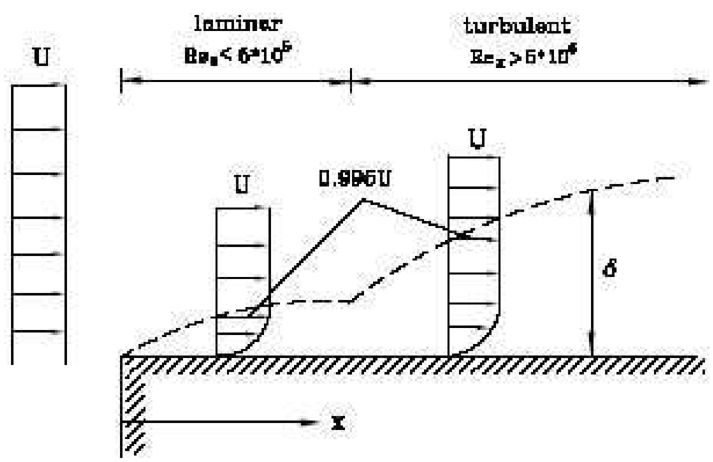

Figure 1.7: Development of the boundary layer.

1.2.3 Prandtl's Mixing Length Theory

Prandtl introduced the mixing length concept in order to calculate the turbulent shear stress. He assumed that a fluid parcel travels over a length l before its momentum is transferred.

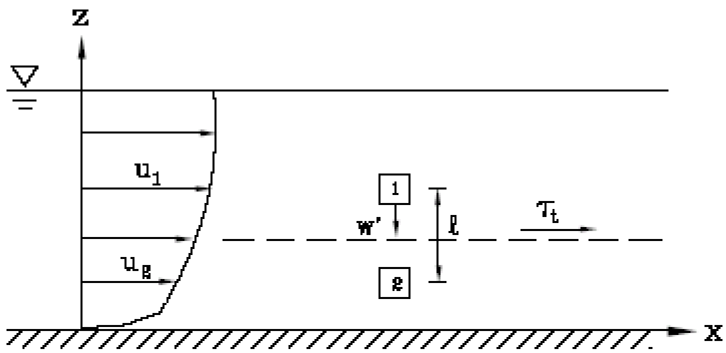

Figure 1.8: Prandtl's mixing length theory.

Fig.1.8 shows the time-averaged velocity profile. The fluid parcel located in layer 1 and having the velocity u_1, moves to layer 2 due to eddy motion. There is no momentum transfer during movement, i.e. the velocity of the fluid parcel is still u_1 when it just arrives at layer 2, and decreases to u_2 sometime later by the momentum exchange with other fluid in layer 2. This action will speed up the fluid in layer 2, which can be seen as a turbulent shear stress τ_t acting on layer 2 trying to accelerate layer 2, cf. Fig.1.8.

The horizontal instantaneous velocity fluctuation of the fluid parcel in layer 2 is

$$u' = u_1 - u_2 = \ell \frac{\mathrm{d}u}{\mathrm{d}z} \tag{1.6}$$

Assuming the vertical instantaneous velocity fluctuation having the same magnitude

$$w' = -\ell \frac{\mathrm{d}u}{\mathrm{d}z} \tag{1.7}$$

Where negative sign is due to the downward movement of the fluid parcel; the turbulent shear stress now becomes

$$\tau_t = -\rho\, u'\, w' = \rho\, \ell^2 \left(\frac{\mathrm{d}u}{\mathrm{d}z}\right)^2$$

If we define kinematic eddy viscosity as

$$\varepsilon = \ell^2 \frac{\mathrm{d}u}{\mathrm{d}z} \tag{1.8}$$

The turbulent shear stress can be expressed in a way similar to viscous shear stress as follows

$$\tau_t = \rho\, \varepsilon \frac{\mathrm{d}u}{\mathrm{d}z} \tag{1.9}$$

1.2.4 Fluid Shear Stress and Friction Velocity

Fluid shear stress: The forces on a fluid element with unit width are shown in Fig.1.9. Because the flow is uniform (no acceleration), the force equilibrium in x-direction reads as

$$\tau_z \, \Delta x \; = \; \rho \, g \, (h - z) \, \Delta x \; \sin\beta$$

For small slope we have $\sin\beta \approx \tan\beta = S$. Therefore

$$\tau_z \; = \; \rho \, g \, (h - z) \, S$$

The bottom shear stress is

$$\tau_b \; = \; \tau_{z=0} \; = \; \rho \, g \, h \, S \qquad (1.10)$$

Bottom shear stress: In the case of arbitrary cross section, the shear stress acting on the boundary changes along the wetted perimeter, cf. Fig.1.9. Then the bottom shear stress means actually the average of the shear stress along the wetted perimeter. The force equilibrium reads

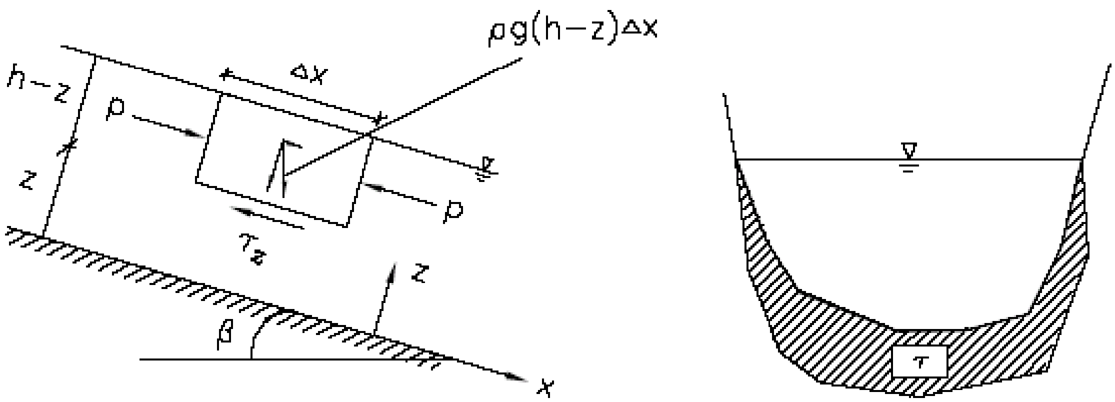

Figure 1.9: Fluid force and bottom shear stress.

$$\tau_b \; P \, \Delta x = \rho \, g \, A \, \Delta x \, \sin\beta$$

Where *P* is the wetted perimeter and *A* the area of the cross section. By applying the hydraulic radius ($R = A/P$) we get

$$\tau_b \; = \; \rho \, g \, R \, S \qquad (1.11)$$

In the case of wide and shallow channel, R is approximately equal to h; eq (2.11) is identical to eq (1.10).

Friction velocity: The bottom shear stress is often represented by friction velocity, defined by

$$u_* \; = \; \sqrt{\frac{\tau_b}{\rho}} \qquad (1.12)$$

The term *friction velocity* comes from the fact that $\sqrt{\frac{\tau_b}{\rho}}$ has the same unit as velocity and it has something to do with friction force.

Inserting eq (2.11) into eq (2.12), we get

$$u_* = \sqrt{g\ R\ S} \qquad (1.13)$$

Viscous shear stress versus turbulent shear stress: Eq (1.10) states that the shear stress in flow increases linearly with water depth. As the shear stress is consisted of viscosity and turbulence, we have

$$\tau_z = \tau_\nu + \tau_t = \rho\ g\ (h - z)\ S \qquad (1.14)$$

On the bottom surface, there is no turbulence ($u = w = 0$, $u' = w' = 0$), the turbulent shear stress

$$\tau_t = -\rho\ \overline{u'\ w'} = 0$$

Therefore, in a very thin layer above the bottom, viscous shear stress is dominant, and hence the flow is laminar. This thin layer is called ***viscous sub-layer***. Above the viscous sub-layer, i.e. in the major part of flow, the turbulent shear stress dominates.

Measurements show the shear stress in the viscous sub-layer is constant and equal to the bottom shear stress, not increasing linearly with depth.

Classification of flow layer: Figure 1.9 shows the classification of flow layers. Starting from the bottom we have

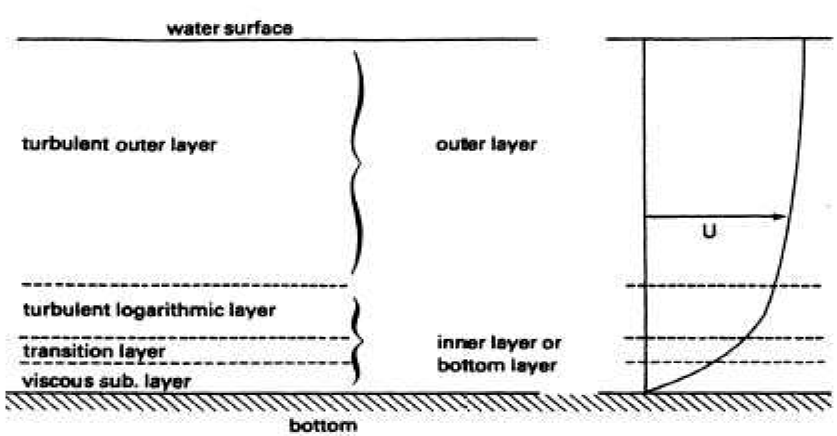

Figure 1.9: Classification of flow region

1. ***Viscous sublayer***: a thin layer just above the bottom. In this layer there is almost no turbulence. Measurement shows that the viscous shear stress in this layer is constant. The flow is laminar. Above this layer the flow is turbulent.
2. ***Transition layer***: also called buffer layer. Viscosity and turbulence are equally important.
3. ***Turbulent logarithmic layer*: ** viscous shear stress can be neglected in this layer. Based on measurement, it is assumed that the turbulent shear stress is constant and equal to bottom shear stress. It is in this layer where Prandtl introduced the mixing length concept and derived the logarithmic velocity profile.
4. ***Turbulent outer layer***: velocities are almost constant because of the presence of large eddies which produce strong mixing of the flow.

In the turbulent logarithmic layer the measurements show that the turbulent shear stress is constant and equal to the bottom shear stress. By assuming that the mixing length is proportional to the distance to the bottom (l = kz), Prandtl obtained the logarithmic velocity profile.

Various expressions have been proposed for the velocity distribution in the transitional layer and the turbulent outer layer. None of them are widely accepted. However, by the modification of the mixing length assumption, the logarithmic velocity profile applies also to the transitional layer and the turbulent outer layer. Measurement and computed velocities show reasonable agreement.

Therefore, from engineering point of view, a turbulent layer with the logarithmic velocity profile covers the transitional layer, the turbulent logarithmic layer and the turbulent outer layer.

At the viscous sublayer the effect of the bottom (or wall) roughness on the velocity distribution was first investigated for pipe flow by Nikurase. He introduced the concept of equivalent grain roughness k_s (Nikurase roughness, bed roughness). Based on experimental data, it was found

1. ***Hydraulically smooth flow***: $\frac{u_* k_s}{\nu} \leq 5$; bed roughness is much smaller than the thickness of viscous sublayer. Therefore, the bed roughness will not affect the velocity distribution.

2. ***Hydraulically rough flow***: $\frac{u_* k_s}{\nu} > 70$; bed roughness is so large that it produces eddies close to the bottom. A viscous sublayer does not exist and the flow velocity is not dependent on viscosity.

3. ***Hydraulically transitional flow**:* $5 \leq \frac{u_* \, k_s}{\nu} \leq 70$; the velocity distribution is affected by bed roughness and viscosity.

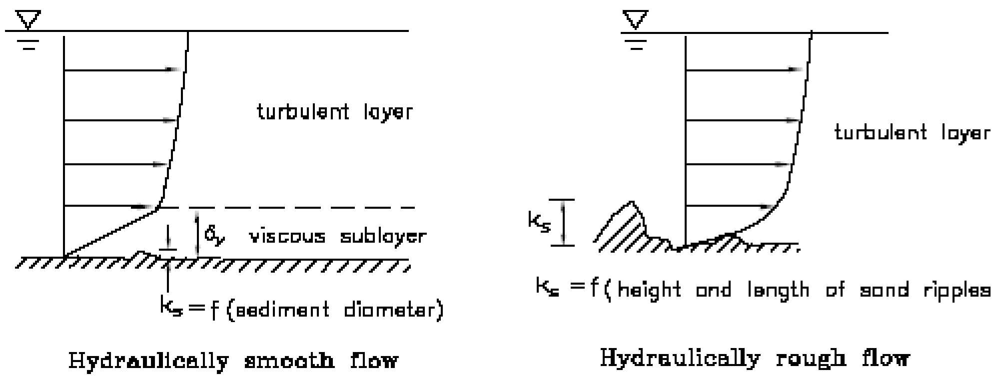

Figure 1.10: Hydraulically smooth and rough flows

1.2.5 Velocity Distribution for Rigid-Boundary Channels

In the following, only velocity-distribution concepts for turbulent uniform flow will be briefly reviewed. Experimental evidence from which the constants used in the velocity-distribution equations have been obtained relates mainly to data on the flow in pipes. However, it is generally accepted that in spite of this fact, universal velocity-distribution equations thus arrived at may also be applied to turbulent flow in open channels.

In analogy with the kinematic theory of gases, Prandtl assumed that a particle of fluid in turbulent flow is displaced at distance *l*, called mixing length, before its momentum is changed by the new environment. Hence, turbulent velocity fluctuation in both x- and z- directions is proportional to *l.du/dz.* From the above mixing-length theory, a useful expression can be derived for the turbulent shear stress. If furthermore it is assumed that near the boundary the mixing length, *l*, is proportional to the distance from the boundary, *z*, then

$$l = kz \tag{1.15}$$

Where k denotes a dimensionless constant to be deduced from experiments; and if the shear velocity, u_* is introduced, Prandtl's differential equation for the turbulent shear stress can be integrated. The integration yields

$$u = \frac{u_*}{k} \ln z + C \tag{1.16}$$

in which u denotes average point velocity at a distance z from the boundary (the averaging is related to turbulent fluctuations in time). C is a constant of integration, which must be determined from the boundary conditions, requiring that close to the boundary the turbulent and the laminar velocity distributions must join each other. On the other hand, since within the laminar sublayer turbulent-flow conditions are no longer valid, the constant of integration may be adjusted to give a zero velocity at some distance z_o within the sublayer (see Fig. 1.11). The distance z_o is presumed chosen in such a way as to ensure a smooth blending to the profiles somewhere in the transition zone.

Hence, when $z = z_o$, $u = 0$, and the constant of integration is obtained $C = -(u_*/k\ (\ln z_0))$. Eq (1.16) can, therefore, be written in the form

$$u = \frac{u_*}{k}(\ln z - \ln z_0) \tag{1.17}$$

Order of magnitude of the laminar sublayer is about υ/u_*, υ denoting kinematic viscosity. Accordingly, $z_0 = m\upsilon/u_*$, in which m is a dimensionless constant. Eq. (2.17) can now be written in the dimensionless form

$$\frac{u}{u_*} = \frac{1}{k}(\ln \frac{z\,u_*}{\upsilon} - \ln m) \tag{1.18}$$

Finally, denoting $A = l/k$ and $B = -1/k(\ln m)$, the resulting logarithmic equation is

$$\frac{u}{u_*} = A \ln \frac{zu_*}{\upsilon} + B \tag{1.19}$$

Eq. (1.19) is generally known as the *Prandtl – von Karman universal velocity distribution law*, valid for all types of conduits.

Laboratory studies have shown that velocity distribution is to a great extent influenced by the roughness of the boundary. Consequently, distinct expressions are obtained for different ranges of roughness conditions.

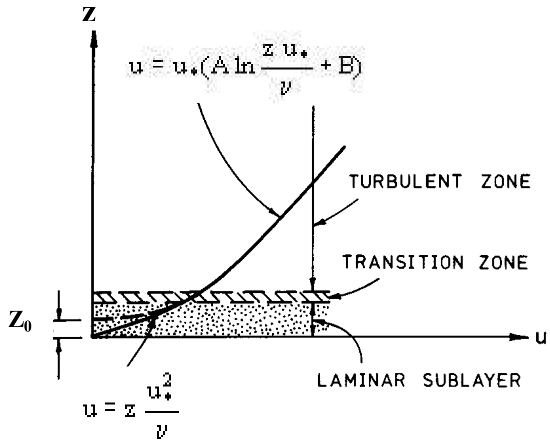

Figure 1:11: Turbulent velocity distribution.

For *hydraulically smooth boundaries*, Niduradse's experiments indicate the constants in Eq. (1.19) to be A = 2.5 and B = 5.5.This means that the constant k in Eq. (1.15) is equal to 0.4. The numerical value of k for a long time was thought to be universally constant for the turbulent flow, regardless of the boundary configuration or value of Reynolds number. It has later been argued, however, that the constant is not independent of the boundary configuration and probably not of the Reynolds number either. Introducing the above constants leads to the expression

$$\frac{u}{u_*} = 2.5\ln\frac{vu_*}{v} + 5.5 = 5.75\log\frac{zu_*}{v} + 5.5 \cong 5.75\log 9\frac{zu_*}{v} \qquad (1.20)$$

In order to derive a suitable expression of Eq. (1.17) for *hydraulically rough boundaries,* we have to introduce the *roughness height, K_s*, having the dimension of length, Fig. 1.12.

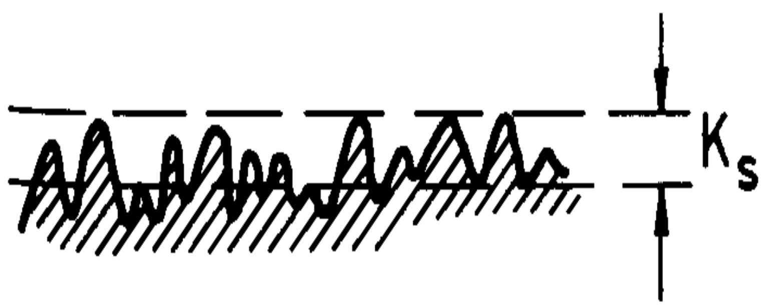

Figure 1.12: Definition of the "roughness height" K_s.

Starting from Eq. (2.17) again and applying dimensional analysis, a similar reasoning as adopted for the smooth–boundary conditions leads to the conclusion that $z_0 = \beta\upsilon/u_*$, in which $\beta = f\,(K_s u_*/\upsilon)$. Introducing as previously the constants A = 1/k and B' = -1/k(lnβ), Eq. (1.19) is obtained.

From Nikuradse's experiments it can be deduced that as long as $K_s u_* / v \leq 3.6 \sim 5$, B' is independent of the function *f* above, i.e. β= m, and it has a constant value of 5.5. This conclusion means that as long as the thickness of the laminar sublayer is sufficient to completely cover the roughness height K_s, Eq. (1.20) will be valid also for rough boundaries. When, on the other hand, $K_s u_* / v \geq \sim 70$, Nikuradse's experiments indicate that

$$B' = 8.5 - 2.5 \ln \frac{K_s u_*}{v} \tag{1.21}$$

Hence, using again Eq. (2.19)

$$\frac{u}{u_*} = 2.5 \ln \frac{z u_*}{v} + 8.5 - 2.5 \ln \frac{K_s u_*}{v} \tag{1.22}$$

Or finally

$$\mathbf{\frac{u}{u_*} = 5.75 \log \frac{z}{K_s} + 8.5 \cong 5.75 \log 30 \frac{z}{K_s}} \tag{1.23}$$

This is the velocity-distribution equation for flow over **rough boundaries**.

For the t*ransition zone* between the smooth and completely rough rigid boundaries (~ $5 < K_s u_* / v \leq \sim 70$), velocity-distribution equation is found also to be of the general form

$$\frac{u}{u_*} = 5.75 \log \frac{z}{K_s} + B' \tag{1.24}$$

Here B' has not a constant value, but is according to Nikuradse a function of the dimensionless parameter $K_s u_* / v$. Subsequent investigations have shown, however, that the transition curve obtained by Nikuradse for uniform-grain distribution does not hold for non-uniform roughness conditions generally encountered in engineering practice. Indeed, transition curve giving B' values for non-uniform roughness seems to be much more gradual.

A more simplified approach regarding velocity distribution for flow over rigid boundaries was later proposed by H.A. Einstein. It is supposedly valid for all three boundary conditions (hydraulically smooth, rough or in transition region), and is given by

$$\mathbf{\frac{u}{u_*} = 5.75 \log \left(30.2 \frac{z}{K_S} x\right)} \tag{1.25}$$

Roughness height K_s denotes here d_{65}, i.e. grain diameter determined so that 65% are equal to it or smaller, and x is a correction factor, which depends on the ratio K_s/δ, where δ stands for the thickness of the laminar sublayer, Fig 1.13.

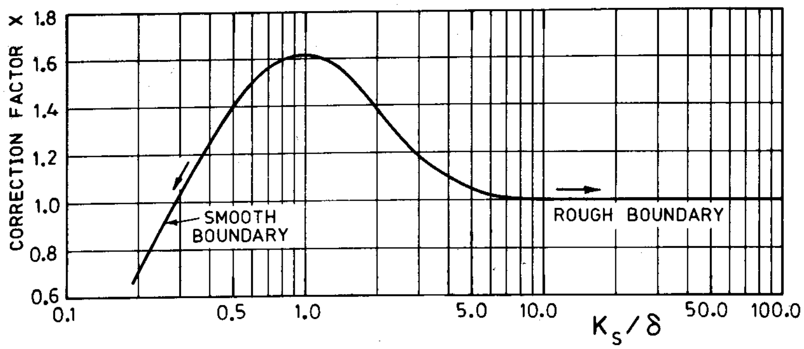

Figure 1.13: Correction factor x in the logarithmic velocity- distribution Eq. (1.25)

By integrating Eq. (2.25) over the vertical section, Einstein, following previous Keulegan's deductions, has proposed expressions for the average velocity in turbulent flow,

Smooth boundary:

$$\frac{\overline{V}}{u_*} = 5.75 \log\left[3.67 \frac{Ru_*}{v}\right]$$

Rough boundary and transition

$$\frac{\overline{\mathbf{V}}}{\mathbf{u}_*} = \mathbf{5.75 \log}\left[\mathbf{12.27} \frac{\mathbf{R}}{\mathbf{K_s}} \mathbf{x}\right] \tag{1.26}$$

Where R denotes the hydraulic radius.

1.2.6 Thickness of the Laminar Sublayer

Theoretical classification of boundary roughness is largely dependent on the relation between roughness protrusions and the thickness of the laminar sub-layer.

As already mentioned, large volume of experimental study has shown that the three roughness regimes can best be distinguished in relation to a dimensionless parameter $K_s u_*/v$, also called **Reynolds number related to grain size**. Summarizing,

Smooth boundary $\Rightarrow \frac{\mathbf{K_s u_*}}{\mathbf{v}} \leq \mathbf{3.6 \sim 5}$

Rough boundary $\Rightarrow \frac{\mathbf{K_s u_*}}{\mathbf{v}} > \sim \mathbf{70}$

Transition $\Rightarrow$ $\mathbf{3.6 \sim 5 < \dfrac{K_s u_*}{v} \le \sim 70}$

In order to obtain the thickness of the laminar sublayer, it is assumed that at the boundary between the laminar and turbulent layer, the shear stress of the laminar flow, $\tau = \mu(du/dz)$, is equal to that of the turbulent flow. Hence

$$\mu \frac{\mathbf{du}}{\mathbf{dz}} = \rho\, \mathbf{u}_*^{\mathbf{2}} \tag{1.27}$$

Let us next denote the velocity at the boundary between the two layers as u_o; then du/dz may be approximately written as u_o/δ, where δ is the thickness to the sublayer. If the velocity-distribution curve within the thin laminar sublayer is represented by a straight line, then it is possible to write

$$\frac{u}{u_o} = \frac{z}{\delta} \tag{1.28}$$

Experiments have shown that the thickness δ is related to a constant value of the dimensionless parameter $u_o\delta/v =$ const., and the numerical value of the constant has been found to be about 135. Since within the sub-layer $\tau = \mu(du/dz) \cong \mu(u_o/\delta)$, it follows that $u_o = \tau(\delta/\mu)$; substituting this into the former expression for the constant parameter, it can be deduced that $\tau\ \delta^2/\rho v^2 = 135$, whence $(u_*\ \delta/v) = (135)^{1/2} = 11.6$. Finally from this:

$$\mathbf{\delta = 11.6 \frac{\upsilon}{u_*}} \tag{1.29}$$

In order to compare the thickness of the sublayer with the height of a roughness element, let us write Eq. (1.29) in a different form,

$$\frac{K_s}{\delta} = \frac{1}{11.6} \frac{K_s u_*}{v} \tag{2.30}$$

If now for *smooth boundary* $k_s u_*/v \le 3.6 \sim 5$, it follows that

$$\frac{K_s}{\delta} = \frac{1}{11.6}(3.6 \sim 5) = \frac{1}{3.2} \sim \frac{1}{2.3} \tag{1.31}$$

For completely *rough boundary*, the parameter $K_s u_*/v \ge \sim 70$. Hence similarly,

$$\frac{K_s}{\delta} = \frac{1}{11.6} 70 \cong 6 \tag{1.32}$$

In the *transition region* the ratio K_s / δ must be between the limits given by Eqs. (1.31) and (1.32).

From the above approximate comparisons it can be concluded that for smooth boundaries the thickness of the laminar sublayer is not less than about three times the roughness height K_s. For rough boundaries, the thickness is not more than about one sixth of the roughness height.

1.2.7 Effect on Velocity-Distribution of the Movable Boundary in Alluvial Channels

All the theoretical velocity-distribution equations discussed in the preceding paragraphs presuppose clear water and rigid boundaries. These conditions can but very exceptionally be met with in sediment-laden alluvial watercourses. Not only is sediment particles carried along in suspension, thus changing probably some of the basic parameters of the turbulent flow; the situation is further complicated by the fact that movable and ever changing bed forms are also likely to have influence upon the velocity distribution structure of the stream.

A logical analysis of the dynamic equilibrium conditions in the turbulent flow leads to the conclusion that the energy spent on the entrainment of sediment particles by the water should somehow damp the turbulent momentum-transfer mechanism, and hence the random velocity fluctuations, as compared to the clear water. Following this reasoning, one is likely to conclude that sediment particles should lag behind the water particles, and that their turbulent mixing length should be shorter on the average. It implies as well that the von Karman universal constant for turbulent flow *k*, so far assumed to be 0.4 would be reduced.

Laboratory experiments reported by the ASCE Task Committee show a reduction of the constant *k* as large as about 50% for a suspended sediment concentration of 15,800 ppm, see Fig. 1.14. The diagram clearly shows that for the same depth, slope and bed surface, velocity for sediment-laden stream is greater than for the clear water. Taking into account Eq. (1.16), this implies that the constant *k* must be smaller.

However, these and more recent experiments have not been conclusive, since they also have shown that this reduction of flow resistance can be more than offset by the increase in resistance caused by bed formations. This fact may well explain many field observations which tend to indicate that sediment-laden water brings about more resistance to flow, and not less as suggested by carefully executed experiments.

While experiments by Vanoni et al., suggest that for clear water the value of the constant k always tends to be close to 0.4, whatever the bed formation, some later experiments, cited by the Task Committee, seem to indicate that the bed formations may reduce k-values even for clear water flows. Some field observations have also hinted that the sediment-transfer coefficient may in some cases be greater than the momentum-transfer coefficient for water particles.

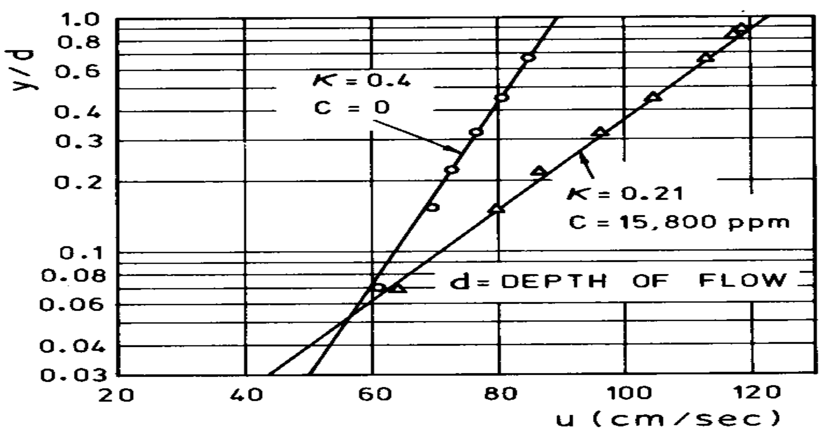

Figure 1.14: Comparison of the velocity profiles for clear water and suspended sediment, after ASCE Task Committee; d = 9 cm, d_s = 0.1 mm. S = 0.0025.

Einstein et al. have tried several times to experimentally determine the variation of the von Karman coefficient k in dependence of characteristic sediment parameters, and subsequently to derive an equation for the velocity distribution in alluvial streams. These equations have been unduly complicated for engineering applications. Adopting the view that there should exist an average value of k for both the velocity and the suspended-load distributions down to a point close to the bed, Einstein and Abdel-Aal finally proposed an experimental correlation curve between the coefficient k and sediment parameters (see Fig.1.15). Here w_{35} denotes the settling velocity for d_{s35} –size of sediment (35% by mass smaller than the given size); d_{s65} – grain size for which 65% by mass of all grains are smaller; q – specific discharge (discharge per unit of width); *J* – hydraulic gradient.

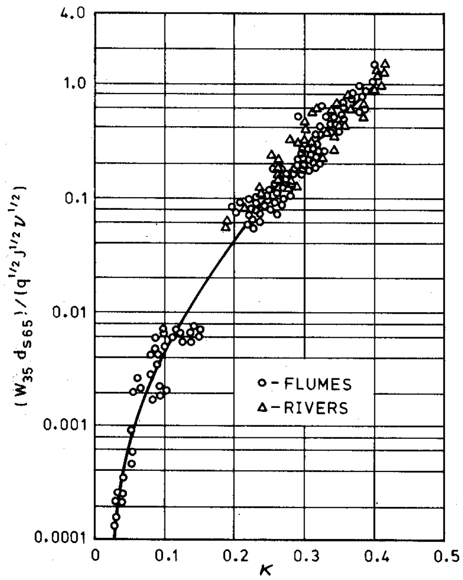

Figure 1.15: Coefficient k as fun of dimensionless sediment parameter ($w_{35}\ d_{65}) /(q^{½} J^{½}\ \upsilon^{½})$.

They further put forward a set of relatively simple equations for the vertical velocity distribution and for the average velocity. It was claimed that good agreement had been found between experimental values from both laboratory and field measurements, and values predicted by the equations.

Equation proposed for the vertical velocity distribution is similar to Eq. (1.25), and it reads:

$$\frac{u}{u_*} = \frac{2.3}{k} \log\left[30.2\frac{z}{k_s}.x\right] \qquad (1.33)$$

Here x denotes the correction factor to be taken from the diagram on Fig. (1.13). Average velocity is expressed in a similar manner,

$$\frac{\bar{V}}{u_*} = \frac{2.3}{k} \log\left[30.2\frac{d}{k_s}.x\right] \qquad (1.34)$$

where d denotes the depth of water.

1.3.1 River morphology and regime

1.3.2 Introduction

Rivers are the natural canals which carry a huge quantity of water drained by the catchments as runoff. They take off From Mountains, flow through plains and finally join the sea or an ocean. Rivers are important arrangements of the hydrological cycle. In addition to water, rivers carry a large amount of silt or sediment which is washed down from the catchments area and also eroded form the bed and banks of river. The silt plays an important role in the behavior of rivers in alluvial soils.

Floods in rivers cause tremendous devastation and miseries to human beings. In the primitive times there was no control on river. With the development of science and technology, the behavior of rivers is now better understood, and various river training methods are used.

1.3.2 Types of rivers

Rivers can be classified according to different criteria:

A. Classification based on variation of discharge

1. **Perennial River**: Perennial Rivers have adequate discharge throughout the year.
2. **Non-perennial rivers**: their flow is quite high during and after rainy seasons and reduces significantly during dry seasons.
3. **Flashy rivers**: in these rives, there is a sudden increases in discharge. The river stage rises and falls in a very short period.
4. **Virgin rivers**: these are those rivers which get completely dried up before joining another river and sea.

B. Classification based on the location of reach:

1. **Mountainous rivers:** they flow in hilly and mountainous regions. These rivers are further divided into rocky rivers and Boulder Rivers.
2. **Rivers in flood plains:** after the boulder stages, a river enters the flood plains having alluvial soil. The bed and banks of river are made up of sand and silt.
3. **Delta Rivers**: when a river enters a deltaic plain, it splits into a number of small branches due to very flat slopes. There is shoal formation and braiding of the channels in the delta rivers.
4. **Tidal rivers**: just before joining a sea or ocean, the river becomes a tidal river. In a tidal river, there are periodic changes in water level due to tides.

C. Classification based on plan-from:

1. **Straight rivers:** these rivers are straight in plain and have cross-sectional shape of a trough. The maximum velocity of flow usually occurs in the middle of the section.
2. **Meandering Rivers**: follow a winding course. They consist of a series of bends of alternate curvature in the plain. The successive curves are connected by small straight reach of the river called crossovers or crossings.
3. **Braided rivers**: flow in two or more channels around alluvial islands developed due to deposition of silt.

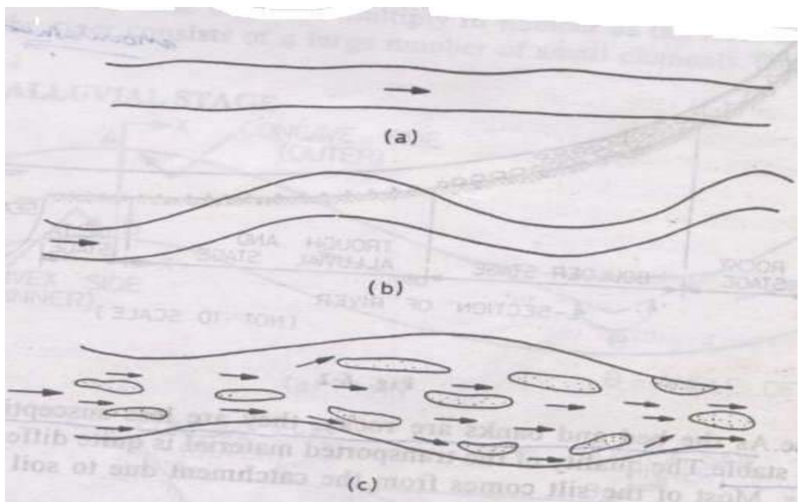

Figure 1.16: Straight, meandering, and braided rivers

1.3.3 Stages of rivers

As the river flows from its origin in a mountain to a sea, it passes through various stages. A river generally has the following 4 stages:

1. Rocky stages
2. boulder stage
3. alluvial stage
4. deltaic rivers

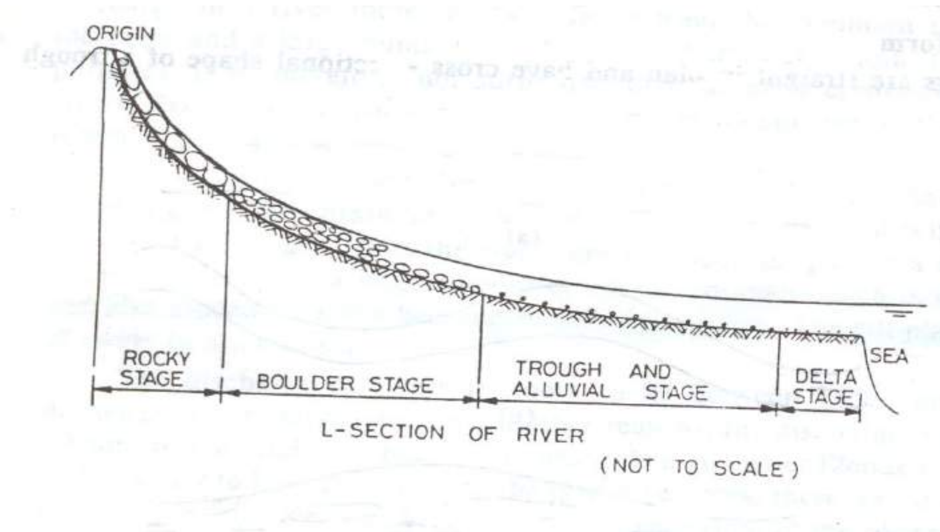

Figure 1.16: Stages of rivers

1. **Rocky stage**: it is also called the hilly or mountainous stage or the incised stage.
   - The flow channel is formed on the rock by degradation and cutting.
   - The cross section of the river is usually made up of rock.
   - The river has a very steep slope and velocity of water is quite high.
   - As the beds and banks are rocky, erosion hazard is less.
   - It is ideal for the construction of dam.
2. **Boulder stage**: the bed and banks are usually composed of large boulders, gravels and shingles.
   - The bed slop is quite steep
   - The river first flows through wide shallow and interlaced channels and then develop a straight course.
   - Most of the diversion head works are constructed in this stage.
3. **Alluvial stage**: the river in this stage flows in a zig- zag manner known as meandering.
   - The cross section of the river is made up of alluvial sand and silt.
   - The materials get eroded form the concave side (the outer side) of the bend and get deposited on the convex side (inner side) of the bend.
   - The bed slope is flat and consequently the velocity is small.
   - The behavior of the river in this stage depends up on the silt charge and the flood discharge.
   - River training works are required in the alluvial stage.

4. **Deltaic stage**: is the last stage of the river just before it discharge into the sea.
   - The river is unable to carry its sediment load. As a result, It drops its sediments and gets divided into channels on either side of the deposited sediment and form the delta.

1.3.4 Types of alluvial rivers

Rivers in flood plains (alluvial stage) is future sub divide into the following classification:-

1. aggrading
2. degrading
3. stable deltaic
4. deltaic

If the river is collecting sediment and is building up its bed it is called an aggrading (accreting) type. If the bed is getting scoured year to year, it is called a degrading type. If there is no silting or scouring, it is called a stable river. It is not necessary that a river reach should be of one type the entire alluvial length; rather it is generally of more than one type of reach, in its length. In other words, the same river reach may behave as aggrading, degrading, or stable type. However under what circumstances the river my change its type will be prominent after the following discussion:

a. **Aggrading or accreting type**: is a silting river.
   - It builds up its slope.
   - The silting is mainly due to various reasons, such as: heavy sediment load, construction of an obstruction across a river, sudden intrusion of sediment from a tributary, etc.

b. **Degrading type**; if the river bed is constantly getting scoured, to reduce and dissipate available excess land slope as shown in the figure below, then it is known as degrading.
   - It is found either above a cutoff or below a dam or weir

c. **Stable type**: a river that does not change its alignment, slope and its regime significant is called Stable River.

d. **Deltaic River**: is as discussed in the proceeding sections.

1.3.5 Behaviors of rivers in alluvial stages

The behaviors of alluvial rivers depend to a large extent on the sediment carried by it. The sediment carried by the river poses numerous problems, such as:

- increasing of flood levels
- sitting of reservoirs
- silting of irrigation and navigation channels
- splitting of a river into a number of interacted channels
- meandering of rivers

Especially the meandering causes the river to leave its original course and adopt a new course. An alluvial river usually has the following three stages:

1. flow in a straight reach
2. flow at bends
3. development of meanders

1. **Flow in a straight reach**: the river cross section is in the shape of a trough, with high velocity flow in the middle of the section.

- Since the velocity is higher in the middle, the water surface level will be lower in the middle and higher at the edges. (see fig.)
- Due to the existence of this transverse gradient from sides towards the center, transverse rotary currents get developed. However,
  straight reaches are very few in alluvial channels.

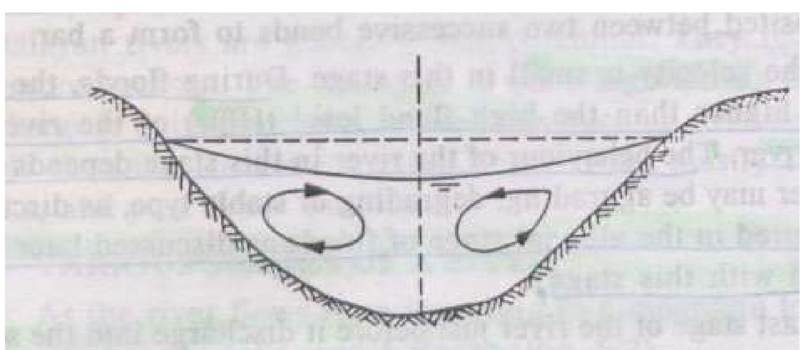

Figure 1.17: Flow in a straight reach

2. **Bends**: every alluvial river tends to develop bends, which are characterized by scouring on the concave side and silting on the convex side. (see fig. below)

- The silting and scouring in bends may continue due to the action of centrifugal force.

Process: when the flow moves around a bend, a centrifugal force is exerted upon the water, which results in the formation of transverse slope of water surface from the convex edge to the concave edge, creating greater pressure near the convex edge. To keep its level, water tends to move from the convex side towards the concave side. However, the top most water surface movement is prevented by the centrifugal force. Moreover, towards the bottom, the velocities are much less than towards the top; and enough centrifugal force is not available to counteract the tendency of water at the top to move inwards. Hence, the water dives in, from the top at the concave end, and moves at the bottom towards the convex end. These rotary currents cause the erosion of concave edge and deposition on the convex edge forming shoal on this edge. When once the bend forms, it tends to make the curvature large and larger.

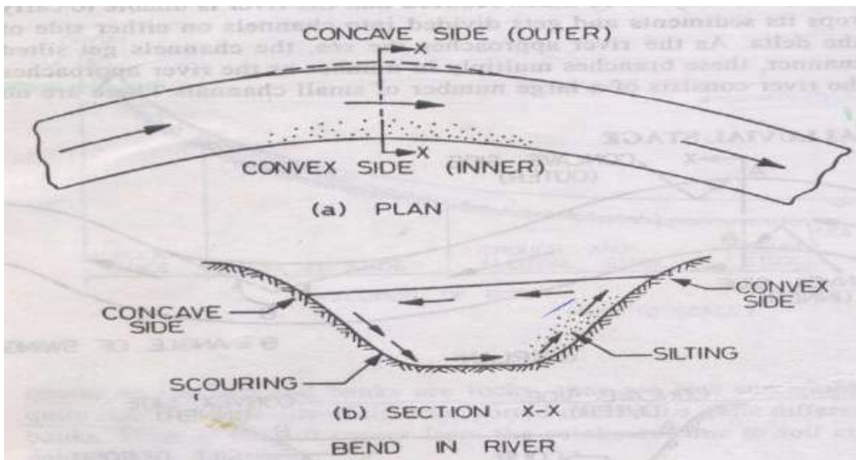

Figure 1.18: Bends

3. **Development of Meanders:** Once a bend in the river has been developed, either due to its own characteristics or due to the impressed external forces, the process continues furthest downstream. The successive bends of the reverse order are formed. It ultimately leads to the development of a complete S-curve called a meander.

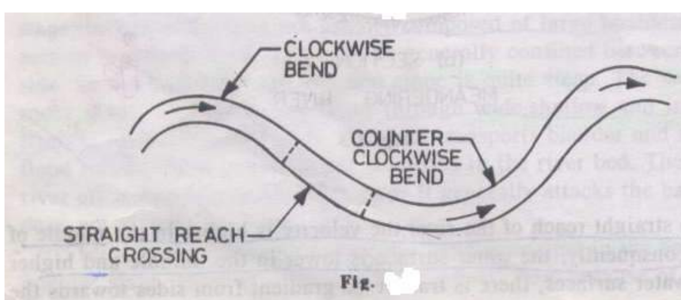

Figure 1.19: Meandering River