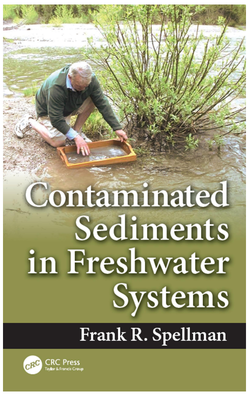

Contaminated Sediments in Freshwater Systems

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Transport of Sediment by Water

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6 Transport of Sediment by Water

We can't fully understand freshwater sediments unless we wholly understand them.

INTRODUCTION

Understanding the principles of sediment transport by flowing water is essential to interpreting and solving many problems. The individual characteristics of water and sediment and their interaction directly affect the type and volume of material eroded and transported and the place and time of deposition. Evaluating channel instability, including erosion or aggradation, and predicting the performance of proposed channel improvements are problems that require knowledge of sediment transport and the use of procedures pertaining to it. Information derived from applying sediment transport prediction procedures can be used to determine the requirements of storage of coarse sediment in debris basins and other types of structures. This chapter includes a discussion of the characteristics of water as a medium for initiating the movement and transport of sediment. The reaction of material on the streambed to the hydraulic forces exerted and the effect of velocity and flow depth on the rate of bed material transport are described. Formulas and procedures designed to predict the rate of bed material transport are given and evaluated. Recommendations are made for applying these formulas and procedures to channel problems. The chapter concludes with a discussion of the mechanics of suspended load transport and a description of a method for computing suspended load yield from concentration and flow duration data.

FACTORS AFFECTING SEDIMENT TRANSPORT

The mechanism of entrainment and the rate at which sediment is transported depend on the characteristics of the transporting medium and on the properties and availability of particles.

Characteristics of Water as the Transporting Medium

The interrelated characteristics of water that govern its ability to entrain and move sedimentary particles are *density*, *viscosity*, and *acidity* (pH):

- Density is the ratio of mass to volume. Increasing the temperature of water increases its volume and decreases its density. With an increase in temperature from 40°C to 100°C (104°F to 212°F), water will expand to 1.04

times its original volume. When working with large volumes of moving water, the slight variations in density resulting from temperature change are usually ignored.

- Viscosity is the cohesive force between particles of a fluid that causes the fluid to resist a relative sliding motion of particles. Under ordinary pressure, viscosity varies only with temperature. A decrease in water temperature from 26.7°C to 4.4°C (80°F to 40°F) increases viscosity about 80%. Changes in viscosity affect the fall velocity of suspended sediment and thereby its vertical distribution in turbulent flow (Colby and Scott, 1965). Increasing the viscosity lowers the fall velocity of particles, particularly very fine sands and silts. A substantial decrease in water temperature and the consequent increase in viscosity smooth the bed configuration, lower the Manning *n* roughness coefficient,* and increase the velocity over a sand bed (USACE, 1968).
- The pH value is the negative logarithm (base 10) of the hydrogen ion concentration. Neutral water has a pH value of 7.0. Acid water has a pH value lower than 7.0, and alkaline water has a pH value higher than 7.0. In acid waters, sediment deposition may be promoted by the formation of colloidal masses of very fine sediments (flocculation) that settle faster than their component fine particles.

Laminar Sublayer

In turbulent flow, a thin layer forms adjacent to the bed in which the flow is laminar because the fluid particles in contact with the bed do not move. This is the laminar sublayer; the higher the velocity or the lower the viscosity, the thinner the sublayer. If the boundary is rough enough, its irregularities may project into the theoretical laminar sublayer, thereby preventing its actual development. Although laminar flow is primarily related to fluid viscosity, turbulent flow is affected by a number of factors. In laminar flow filaments of water follow parallel paths, but in turbulent flow the paths of particles crisscross and touch, mixing the liquid. A criterion defining the transition from laminar to turbulent flow is the Reynolds number (*Re*), which is the ratio of inertial force to shear force on the fluid particle. If the Reynolds number is low, shear forces are dominant, but as the Reynolds number increases they decline to little significance, thereby indicating the dominance of inertial forces. In other words, the Reynolds number is a dimensionless quantity that expresses the relative importance of inertial forces compared to viscous forces in a flow system. A small Reynolds number is associated with laminar flow; a large Reynolds number is associated with turbulent flow. The association of laminar flow with viscosity and that of turbulent flow with inertia are the same whether the fluid is moving or at rest. A small particle of sediment, such as a very fine sand, settling in still or flowing water moves slowly enough to sustain laminar flow lines in relatively viscous media. Inertial forces become increasingly important as grain size increases and are dominant when the particle size exceeds 0.5 mm.

* Manning's equation can be written as $V = (k/n)R_h^{2/3}S^{1/2}$, where V is the average flow velocity; k = 1.49 for English units or 1.0 for SI units; n is the Manning roughness coefficient; R_h is the hydraulic radius; and S is the slope of the water surface.

Characteristics of Transportable Materials

The characteristics of separate or distinct in form (discrete) particles were discussed in Chapter 4. The entrainment and transport of granular materials depend on the size, shape, and specific weight of the particles and their position with respect to each other. The resistance of cohesive materials depends largely on the forces of interparticle bonding. Cohesive forces can be attributed to several factors, including the amount and kind of clay minerals, the degree of consolidation or cementation, and the structure of the soil mass.

MECHANISM OF ENTRAINMENT

Forces Acting on Discrete Particles

Turbulence is a highly irregular motion characterized by the presence of eddies. The degree to which eddies form depends on the boundary roughness and geometry of the channel, and eddies are sustained by energy supplied by the flow. The eddies penetrate the laminar sublayers formed along the bed. Discrete particles resting on the bed are acted on by two components of the forces associated with the flow. One component force is exerted parallel to the flow (drag force) and the other is perpendicular to the flow (lifting force). Drag force results from the difference in pressure between the front and the back sides of a particle. Lifting force results from the difference in pressure on the upper and lower surfaces. If the lifting force exceeds the particle's immersed weight and the interference of neighboring grains, the particle goes into suspension. Because turbulence is random and irregular, discrete particles tend to move in a series of short, intermittent bursts. In each burst, particles move a short distance and many grains move simultaneously. The movement then subsides until another burst occurs. The frequency and extent of movement increases with the intensity of turbulence, and above a certain intensity some particles may be projected into the flow as suspended load (Sutherland, 1967). The coarser and rounder the particles are, however, the greater the possibility that they will begin to roll and continue rolling.

Tractive Force

Experiments to determine the forces that act on particles on a streambed were performed mainly to predict channel stability. More advanced methods are necessary to describe transport. The instantaneous interactions between turbulent flow and discrete sediment particles resting on the bed were described briefly in the preceding paragraphs. In practical application, however, it is more convenient to deal with time-average values of the force field generated by the flow near the bed. Here, the forces normal to the bed having a time average equal to zero can be eliminated, and only those forces parallel to the bed need to be considered. The time average of these forces is the tractive force. The tractive force measured over a unit surface area is the tractive stress. In a prismatic channel reach of uniform flow bounded by two end sections, the mean value of tractive stress is equal to the weight of the water prism in the reach multiplied by the energy gradient and divided by the wetted boundary surface in the reach. Shear stress or force per unit area of bed is expressed as $\tau_0 = \gamma R S_e$.

Determining Critical Tractive Stress

The most widely used and most reliable evaluation of tractive stress related to the initiation of motion is that developed by Shields (1936). The theoretical concepts, supported by experiments, resulted in a plot of $\tau_c/\{\gamma[(\gamma_s - \gamma) - 1]d_s\}$ against $(U_* d_s)/\nu$. The first expression is an entrainment function and the second is the boundary Reynolds number, indicating the intensity of flow turbulence around the particle. The Shields data are based on particles of uniform size and a flat bed. The Shields experiments indicate that beyond a certain value of the boundary Reynolds number, $(U_* d_s)/\nu$, the value of the parameter $\tau_c/\{\gamma[(\gamma_s/\gamma) - 1]d_s\}$ remains constant. Within these limits, the critical tractive stress is therefore proportional to grain size.

Data on critical tractive stresses obtained in a number of investigations were assembled by Lane (1955). These data show that the critical tractive stress in pounds per square foot is equal to $\tau_c = 0.5d_{75}$, where d_{75} is the size in inches of the bank material at which 25% by weight is larger. The limiting (allowable) tractive stress was determined from observations of canals (Land, 1955). The recommended limiting tractive stress in pounds per square foot is equal to 0.4 of the d_{75} size in inches for particles that exceed 0.25 in diameter. Results of experiments on finer particles vary considerably, probably because of variations in experimental conditions. These include differences in interpreting the initiation of sediment movement, in temperature of the water, in concentration of colloids, and in configuration of the bed. Critical conditions for initiating movement sometimes are determined by the number of particles or the frequency with which the particles start to move. For example, one observer's criterion was the time at which particles begin to move every 2 seconds at a given spot on the bed (Sutherland, 1967).

Determining Critical Velocity

Determining the critical velocity (the velocity at which particles in the bed begin to move) is another method for establishing stability criteria. Critical water velocity is a function of mean grain size. There has been less agreement on critical velocity than on critical tractive stress, probably because bottom velocity increases more slowly with increasing depth than does mean velocity. Critical conditions for initiating movement can be expressed directly in terms of tractive stress, but critical mean velocity must be related to variation in velocity with depth. Determining the correct critical value for tractive stress or velocity is important when considering stability problems in channels in which there is to be no significant movement of the boundary material. The significance of the critical value is determined by the magnitude and duration of flows that initiate sediment movement. A prolonged flow slightly exceeding the critical value may have little significance in terms of the volume of bed material transported. On the other hand, a brief flow substantially exceeding the critical value could transport a large volume of sediment.

HYDRAULIC CONSIDERATIONS

Fixed Boundaries

The relationships of velocity, stage, and discharge for stream channels with fixed boundaries have long been satisfactorily predicted by selected the appropriate n value in Manning's and other related formulas.

Movable Boundaries

The study of the hydraulics of movable boundaries has been directed toward two general problems. Primary interest has been in determining methods for predicting the friction coefficient and thereby the correct velocity, stage, and discharge relationships for channel design. The need for these data as a key element in predicting sediment transport has added incentive to the investigations. The changes in bed form produced on a movable bed and the consequent change in friction characteristics of the bed are among the most intensively studied flow phenomena. The literature on this subject generally describes the sequence of changes in bed configuration that can occur as the flow and transport intensity increase. Ripples, ripples on dunes, or dunes may form at a low transport rate, and antidunes or a flat bed may form at a high transport rate. These bed forms have been observed in sand-bed flumes and streams with a d_{50} size finer than 1.0 mm. The variety of bed forms in coarser material seems to be smaller.

Pioneering efforts in investigating the hydraulics of movable beds led to dividing the hydraulic radius into two parts. One part is the radius resulting from the roughness of the grain size of the individual particles (R'), and the other is the radius resulting from the roughness of the bed configuration (R'') (Einstein, 1950; Einstein and Barbarossa, 1952). From field observations, Einstein and Barbarossa developed a graph relating the dimensionless ratio V/U_*'', where $U_*'' = (gR''S_e)^{1/2}$, to Einstein's flow intensity parameter, ψ. The graph they developed indicates that for a given set of conditions it is possible to develop a unique stage–discharge relationship and thus to predict the hydraulics of a channel with movable boundaries. Vanoni and Brooks (1957) presented a graphical solution to the friction equation from which R' is determined.

Another procedure for predicting hydraulic behavior in movable channel beds is based on the division of slope (S) into two parts, S' and S'' (Meyer-Peter and Müller, 1948). In this procedure, S' is the energy gradient associated with the grain size of the bed material under a certain velocity and depth, excluding form resistance, and

DID YOU KNOW?

Whenever I present the observations of Hans Albert Einstein and his co-authors to my students or in my guest presentations here and there, I am always asked if Hans Albert Einstein is related to the genius Albert Einstein of $E = MC^2$ fame. Yes, he is. In fact, Hans Albert Einstein is the oldest son of Albert Einstein and is renowned for his doctoral thesis, "Bed Load Transport as a Probability Problem," which is considered the definitive work on sediment transport.

S'' is the additional gradient pertaining to bed form resistance. This division of slope was adopted by Alam and Kennedy (1969). A similar hydraulic consideration sometimes used as part of the preliminary procedure in sediment transport computations is the treatment of bank friction as completely distinct from bed friction. One such approach, involving the use of Manning's friction equation, is included as part of the procedure in the Einstein bedload function.

Movement of Bed Material

In this discussion the term *bed material load* is defined as that part of the total sediment load (suspended load plus bedload) that is composed of grain sizes occurring in appreciable quantities in the bed material. The part of the total load that consists of grain sizes not present in the bed material in significant quantities is the *washload*. Sand-size particles that constitute all or the major part of the bed material travel either on the bed as bedload or in suspension. Figure 6.1 illustrates how the total sediment load is classified: bedload, bed material load, or washload. Evaluation techniques are not refined enough to predict accurately what part of the bed material load moves in suspension or what part moves as bedload under specific hydraulic conditions. Establishing this separation does not seem essential to the general solution of sediment transport problems.

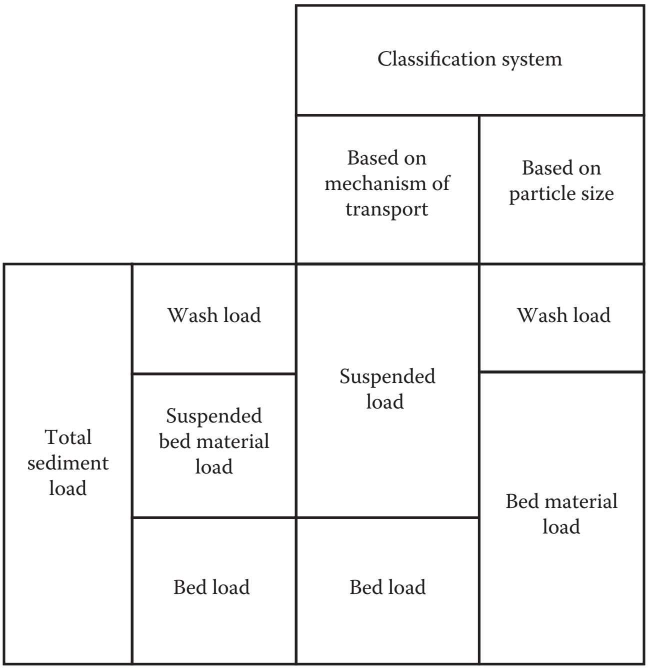

FIGURE 6.1 Sediment load classification. (Adapted from Cooper, R.H. and Peterson, A.W., *Journal of the Hydraulics Division*, 96(HY9), 1880–1886, 1970.)

Transport rates for sand and gravel have been determined by both direct measurement and computation. Measurements of the transport rate in natural streams have been few, chiefly because of the difficulty in getting representative measurements. Sampling equipment established in or on the bed tends to alter the direction of flow filaments and the sediment concentrations. The more accurate measurements have been made by using equipment installed to withdraw representative samples of the water sediment mixed during specific periods. Another method is to sample total load as the flow moves over a sill at an elevation the same as that of the slope upstream.

The existence of many procedures for predicting transport rates indicates both the difficulty of obtaining measurements and the influence of many variables on the consistency of results. Because flume studies are the most easily controlled and exclude some variables, they have become the primary means of establishing relationships between stream discharge and bed material load. The earliest bed material transport formula still in use is that of DuBoys, who published results of studies of the Rhone River in 1879. DuBoys originated a concept common to many later formulas when he assumed in his derivation that the rate of sediment transport is proportional to the tractive stress in excess of the critical value required to initiate motion:

$$q_T = \psi\tau_0(\tau_0 - \tau_c) \tag{6.1}$$

where

q_T = Rate of sediment transport per unit width of stream.
ψ = A coefficient that depends on characteristics of the sediment (not to be confused with Einstein's ψ).
τ_c = A value established by experiment (not the same as that of Shields).

Early in the 20th century, several flume studies of sand transport were started, including that of Shields. He is best known for developing criteria for the initiation of movement. Probably the most extensive early investigation of sediment transport in flumes was Gilbert's in about 1910 (Gilbert, 1914). Descriptions of a number of transport phenomena resulted from those experiments, but no general formula was derived. Of the formulas that follow, those of Schoklitsch, Meyer-Peter, Haywood, and Meyer-Peter and Müller are bedload formulas. The Einstein bedload function, the Engelund–Hansen procedure, and the Colby procedure determine the rate of bed material transport, both bedload and suspension load.

Schoklitsch Formula

Schoklitsch developed one of the more extensively used empirical formulas (Shulits, 1935; Shulits and Hill, 1968). He used his own experimental data and also data from Gilbert's flume measurements. The 1934 Schoklitsch formula in English units is

$$q_B = [86.7/(d_{50})^{1/2}]S_e^{3/2}(q - q_0) \tag{6.2}$$

where

q_B = Unit bedload discharge (pounds per second per foot of width).
d_{50} = Medium size of sediment (inches).
$q_0 = 0.00532(d_{50}/S_0^{4/3})$.

In describing the formula, Shulits recommended using a cross-section in a straight reach of river where the depth of water is as uniform as possible and the width changes as little as possible with stage. As described by Shulits, the Schoklitsch formula fits Gilbert's measurements for uniform particle sizes of about 0.3 to 7 mm and slopes ranging from 0.006 to 0.030 ft/ft for small particles and 0.004 to 0.028 ft/ft for larger particles.

Meyer-Peter Formula

In 1934, the Laboratory for Hydraulic Research at Zurich, Switzerland, published a bedload transport formula based on flume experiments with material of uniform grain size. The original analysis of the Zurich and Gilbert data for uniform particles ranging from about 3 to 28 mm in diameter was supplemented by studies of mixtures of various-size particles up to 10 mm and having various specific gravities. The Meyer-Peter formula in English units is

$$q_B = (39.25q^{2/3}S_0 - 9.95d_m)^{3/2} \tag{6.3}$$

where d_m is expressed in feet. The new term in this formula is d_m, the effective diameter of the bed material, which identifies the characteristic size of a sample. To determine this value, divide the size distribution curve of a bed material mechanical analysis into at least 10 equal-size fractions and determine the mean size and weight percentage of each fraction.

Haywood Formula

The Haywood formula is based on Gilbert's flume data and data from the U.S. Waterways Experiment Station, Vicksburg, Mississippi. In his evaluation, Haywood (1940) adjusted Gilbert's data to account for sidewall resistance. He assumed that the discharge effective in moving bedloads is midway between the discharge of walls offering no resistance and that of walls offering the same resistance as the bed. Haywood demonstrated the close relationship of his formula to the Schoklitsch formula, which is based on some of the same data. Haywood believed that his formula substantially agreed with Schoklitsch's formula for relatively large rates of bedload movement and that it was much more accurate for very small rates of movement. Haywood considered 3 mm to be the maximum particle size for application of his formula. He regarded his formula as a modification of the Meyer-Peter formula:

$$q_B = [(q^{2/3}S_0 - 1.20d^{4/3})/0.117d^{1/3}]^{3/2} \tag{6.4}$$

where d is d_{35} expressed in feet.

Meyer-Peter and Müller Formula

The Meyer-Peter and Müller formula is based on data obtained from continuing the experiments that resulted in the Meyer-Peter formula. The range of variables, particularly slope, was extended. A few tests were run with slopes as steep as 20% and sediment sizes as coarse as 30 mm. Meyer-Peter and Müller stated explicitly that their work was on bedload transport, by which they meant the movement of sediment that rolls or jumps along the bed. Transport of material in suspension was not included (Meyer-Peter and Müller, 1948). The Meyer-Peter and Müller formula as translated by Sheppard (1960) is

$$q_B = 1.606[3.306(Q_S/Q)(d_{90}^{1/6}/n_s)^{3/2}(DS_e - 0.627d_m]^{3/2} \tag{6.5}$$

where d_{90} and d_m are expressed in millimeters.

Nomographs are available for determined Q_S/Q (a ratio of the discharge quantity determining bedload transport to the total discharge) and n_s (a Manning n value for the streambed). The formula, a significant departure from the previously cited formulas, includes a ratio of the form roughness of the bed to the grain roughness of the bed surface.

Einstein Bedload Function

In 1950, Einstein's bedload function had a major effect on investigations of the hydraulics and sediment transport characteristics of alluvial streams. Einstein (1950) described the function as "giving rates at which flows of any magnitude in a given channel will transport as bed load the individual sediment sizes of which the channel bed is composed." It was developed on the basis of experimental data, theory of turbulent flow, field data, and intuitive concepts of sediment transport. The Einstein bedload function first computes bedload and then, by integrating the concentration at the bed layer with the normal reflection of that concentration in the remainder of the flow depth, determines the total bed material load.

Einstein introduced several new ideas into the theory of sediment transport. Included were new methods of accounting for bed friction by dividing it into two parts, one pertaining to the sand-grain surface and the other to the bed form roughness, such as ripples or dunes. An additional friction factor, that of the banks, was included in the procedure for determining hydraulic behavior before computing bed material transport.

Another idea introduced by Einstein to explain the bedload function is that the statistical properties of turbulence govern the transport of particles as bedload. This statistical character is reflected in the structure of the dimensionless parameter ϕ, defined as the intensity of bedload transport. The relationship between this factor and the dimensionless flow intensity (ψ, another dimensionless parameter reflecting the intensity of shear on the particle) is used in the procedure. The ϕ–ψ relationship has subsequently been tested by others and found to be an appropriate determinant of bedload transport.

Engelund–Hansen Procedure

Engelund and Hansen (1967) developed a procedure for predicting stage–discharge relationships and sediment transport in alluvial streams. They introduced a parameter θ (the reciprocal of Einstein's ψ) to represent the ratio of agitating forces (horizontal drag and lifting force) to the stabilizing force (immersed weight of the particle). This parameter is a dimensionless form of the bed shear, τ_0, to be divided into two parts: τ', the part acting directly as traction on the particle surface, and τ'', the residual part corresponding to bed form drag. This division is similar to that of

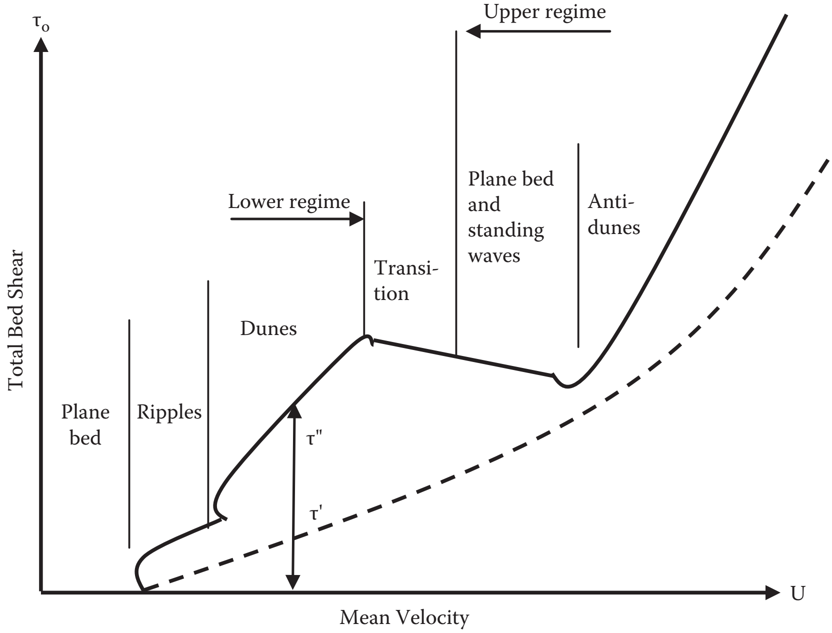

FIGURE 6.2 Relationship between grain roughness (τ′) and form drag (τ″). (Adapted from Engelund, F. and Hansen, E., *A Monograph on Sediment Transport in Alluvial Streams*, Teknisk Forlag, Copenhagen, Denmark, 1967.)

the Einstein–Barbarossa R' and R''. The authors' diagram of the relationship of bed forms to the two separations of total bed shear and to velocity is shown in Figure 6.2. Principles of hydraulic similarity were used to develop a working hypothesis for describing total resistance to flow, specifically for dune-covered streambeds and bed material discharge.

The steps used in applying the Engelund–Hansen procedure are given here in some detail because the procedure demonstrates the impact of changing bed forms on bed material transport and because it was published in a foreign journal not readily available for reference. Data from flume experiments by Guy et al. (1966) were used to test the Engelund–Hansen theories. The mean sizes used in these experiments were 0.19, 0.27, 0.45, and 0.93 mm. Transport of the bed material, both in suspension and moving along the bed, was measured. It was found that the Engelund–Hansen procedure does not provide a means for determining the bed material discharge at lower flow regimes of plane beds and ripples. These regimes are not significant in terms of the volume of sediment transported.

Colby Procedure for Relating Mean Velocity to Sand Transport

The Colby procedure was developed by correlating mean velocity with sediment concentration in a sand bed stream. The procedure, partly empirical and partly derived from Einstein's bedload function, is based on measurements in flumes and channels.

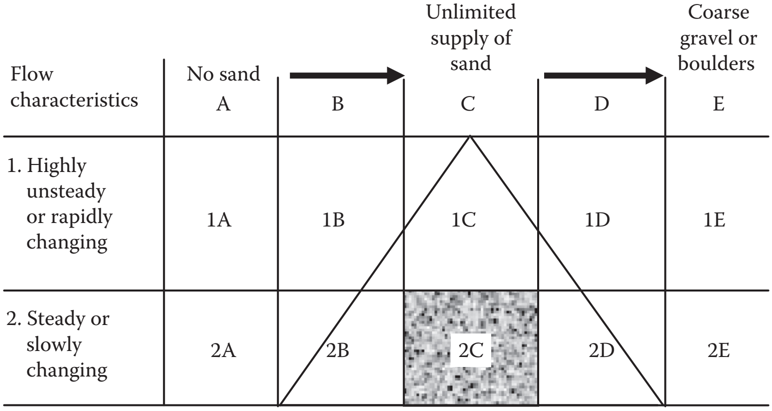

FIGURE 6.3 Characteristics of bed material.

Application and Limitations of Formulas

The lack of certainty in solving specific sediment transport problems is in part a result of the extremely limited number of situations in which predictive techniques, such as bedload or bed material transport formulas, have been substantiated by field measurement. Even for techniques that have been substantiated, little information is available about the specific hydraulic characteristics for comparison with conditions for the problem to be solved (Cooper et al., 1972). Figure 6.3 illustrates a few of the major factors that can be considered in the application and limitations of sediment transport formulas. The availability of bed material ranges from no sand (column A), to an unlimited supply of sand in sizes less than 1 mm (column C), to bed material of gravel and boulders (column E). Flow characteristics range from highly unsteady or rapidly changing to steady and slowly changing.

Of the possible conditions illustrated by this diagram, the condition in box 2C most nearly fits the flow and sediment conditions used in developing transport formulas. Box 1C pertains specifically to the smaller streams with which we are concerned, not to rivers in which deep steady flows may transport gravel as they do sand. Through limited reaches and during high flows, shallow streams may also transport gravel and boulders. Frequently there is a transition from scour to deposition over a relatively short reach. Boxes adjacent to 2C (1C, 2B, 2D) can be considered a "gray" area from which correct solutions to sediment transport problems can be obtained by including the appropriate modifiers, such as changes in slope to match variations in discharge.

The effect of rapidly changing flow (top line on the chart) on bedload transport was the subject of a flume study by DeVries (1965). The mean grain size was 2.5 mm. After an equilibrium rate of transport was attained, the tailwater was suddenly lowered while other factors were kept constant. DeVries computed the lowering of the bed level from scour and the change in rate of sediment transport during the transition to a new state of equilibrium by using several procedures, including the Meyer-Peter and Müller formula. He concluded that establishment and damping of a steady state are slow and that steady-state formulas are unreliable for predicting local, temporary transport for an unsteady state.

A subsequent flume study was made of the effect of introducing a substantial increase (65%) in bed material load into a run where equilibrium flow and transport had been established (Rathbun and Guy, 1967). The medium size of the sand used was about 0.30 mm. This increase in load increased slope, decreased depth, and increased the transport rate. In another run, the rate of sediment input was reduced to about 50% of the equilibrium rate. At first the transport rate was about the same as during equilibrium flow; then, with degradation of the upper end of the sand bed and a decrease in slope, the transport rate also decreased.

Aggradation occurs in some channels even through hydraulic computations indicate that sediment should not deposit. It is not always known whether the aggradation occurred in the rising or falling stage of the hydrograph. Some of the unpredicted changes can be explained by variable bed roughness not accounted for in conventional hydraulic computations. Variable bed roughness does not necessarily explain all of the inaccuracies in predicting the effects of hydraulic change on sediment transport, however, because some procedures do take into account the changes in bed roughness with various flows. Part of the problem may be due to unsteady flow, as steady-flow procedures fail to account for differences between stage and discharge.

When using computational procedures, it is very important that the supply of bed material just satisfies the capacity for transport under existing hydraulic conditions; that is, there can be neither a deficiency, resulting in scour, nor an excess, resulting in aggradation. A sand bed satisfies the necessary requirements for using bedload or bed material transport formulas and that of bed material availability if the bed is sand from bank to bank throughout the reach.

When he considered the availability of bed materials, Kellerhals (1966) made a distinction between channels with a sand bed and channels with a gravel bed. According to his studies, channels with a gravel bed cannot be expected to obey the same laws as channels with a sand bed. One distinction is that ripple and dune formations are less significant in channels with a gravel bed.

In terms of particle size, the scarcity of particles in the 2- to 4-mm size fraction, as described by Sundborg (1956), creates a sharp division between predominantly sand-bed streams and predominantly gravel-bed streams. This division has been substantiated by data on sizes of bed material in various parts of the United States.

The segregation of particles in a mixture of sizes, including gravel, and the depth of scour before the formation of armor were the subjects of flume studies by Harrison (1950). The purpose was to determine the most critical condition for segregation and for building an armor during degradation. Harrison used the Einstein bedload function to calculate the limiting grain diameter for equilibrium flow. He determined that a value of ψ (a dimensionless parameter of transport capability) above 27 indicates negligible transport of bed material.

Harrison (1950) found that the representative grain roughness, k_s (assumed to be d_{65} in his procedures), increases during segregation and armor formation. On the basis of data from field and laboratory studies, Kellerhals (1967) computed the k_s

values after armor formation to be the d_{90} size. On the basis of these considerations, the following treatment is suggested for sediment problems in streams as categorized in Figure 6.3:

1A, 2A—For cohesive soil, cemented gravel, and rock, initiation of movement is the important factor in channel scour or bank erosion. Critical tractive force is related to the d_{75} of bank materials. Undisturbed cohesive soil exhibits erosion resistance that may result from one or several characteristics such as structure, permeability, consolidation, cementation, or cohesion. The influence of each of these characteristics has not been identified. Their cumulative effect on erosion resistance, however, can be determined by shear strength tests on undisturbed soil that has been saturated to duplicate moisture conditions during channel flow (Flaxman, 1963).

1B, 2B—A bed only partially covered with sand and exposing different material (e.g., cohesive soil, rock) as the fixed channel boundary indicates a limited sand supply at this specific location. Sediment transport formulas applied to this condition usually yield computed rates that exceed the actual rate. Test the potential for back erosion by tractive force theory if the bank is composed of noncohesive materials; otherwise, use the procedures for cohesive soils.

1C, 2C—A sand-covered bed is the condition used in sediment transport formulas if the problem to be solved requires (1) estimating the volume of bed material transport during a specific interval of time and at a specific level of discharge or (2) comparing the bed material transport in a reach with that in another reach in which changes in slope, cross-section, or discharge may influence the design of a channel. If flow is unsteady, replace the steady-state procedures with the proper unsteady flow relationships, as previously mentioned.

2D—Techniques for predicting transport rates of sand–gravel mixtures allow estimates of the potential for scour or aggradation. The probable depth of scour can be estimated by determining whether the maximum tractive force for a given flow will exceed the critical for the coarsest 5 to 10% of bed material. If the maximum tractive force exceeds the critical for the d_{90} to d_{95}, the depth of scour cannot be predicted unless still coarser material underlies the bed surface material. The amount of scour necessary to develop armor formed of the coarsest fraction can be determined from either the depth of scour or the volume of material removed in reaching this depth.

1D, 1E, 2E—For gravel and gravel–boulder mixtures, the technique used for determining depth of scour and volume of material produced by scour is similar to that for sand–gravel mixtures (2D). Do not use bedload formulas for this type of material unless confined flow, steepness of slope, and uniformity of cross-section provide relatively uniform discharge per foot of width. The highly variable velocity and discharge per foot of width in many alluvial channels is particularly conducive to deposition alternating with scour of coarse bed material.

Conditions favoring bed material transport at or near a constant and predictable rate do not include delivery in slurries or other forms that change the viscosity and natural sorting processes of flow. Alluvial fills of mountain or foothill canyons are typical of conditions favoring viscous flow. Heavy storm runoff after many years of fill accumulation may produce debris or mud flows whose volume can be predicted only by field measurement.

Example Channel Problem

Applying different procedures to determine sediment transport capacity will likely produce similarities and differences in the results obtained. In the following example, the Schoklitsch formula and the Colby procedure are used to illustrate this point. An existing channel 20 ft wide with a bed slope of 0.002 ft/ft has inadequate capacity for controlling flooding of adjacent lands. It has been proposed that the width of this channel be increased to 30 ft to provide the necessary capacity. Field investigations show that an unlimited supply of sand is available for transport in the bed of the channel and that this sand has a d_{50} size of 0.30 mm. Water temperature is 60°F, and the concentration of fine sediment does not exceed 5000 ppm. For purposes of simplification, it is assumed that the banks have no effect on depth–discharge relationships, but the roughness of the banks and differences in roughness of the banks in both unimproved and improved reaches can in fact affect depth and velocity for a given discharge and thereby affect the rate of bed material transport. The hydraulics of the flow, including distribution of shear on the banks as well as on the bed, must be determined by an established procedure before computing the bed material transport. The hydrograph used in this example is divided into segments to determine the discharge per foot of stream width as required for the computational procedures. The mean discharge and duration for each of the hydrograph segments are shown in Table 6.1.

The Schoklitsch formula requires data only for the amount of discharge per foot of width. The Colby procedure requires velocity and depth of flow. To determine velocity and depth for a given discharge (unless they are available from stream-gauge records), it is necessary either to assume an n roughness coefficient for use in the Manning equation or to obtain such values empirically. For solution of the

TABLE 6.1
Discharge Data for Example Channel Problem, High Flow

	Discharge per Foot of Width	
Hydrograph Segment	**20-ft Channel (ft³/s)**	**30-ft Channel (ft³/s)**
Rising stage		
(a) Mean flow for 2 hours, 90 ft^3/s	4.5	3.0
(b) Mean flow for 2 hours, 280 ft^3/s	14.0	9.333
Falling stage		
(c) Mean flow for 3 hours, 240 ft^3/s	12.0	8.0
(d) Mean flow for 3 hours, 180 ft^3/s	9.0	6.0
(e) Mean flow for 3 hours, 40 ft^3/s	2.0	1.333

TABLE 6.2
Sediment Transport (lb) Computed for Various Flows

	Schoklitsch Formula		Colby Procedure: Using n = 0.020		Colby Procedure: Using Alam and Kennedy Friction Factors	
Discharge Segment	**20-ft width**	**30-ft width**	**20-ft width**	**30-ft width**	**20-ft width**	**30-ft width**
a	44,135	42,840	97,285	86,720	109,270	103,225
b	142,760	141,470	347,085	344,210	412,425	543,140
c	182,995	181,060	442,745	426,435	590,170	564,565
d	136,280	134,340	328,735	310,100	516,280	431,920
e	27,270	25,330	50,710	42,765	46,180	31,190
Total	533,440	525,040	1,226,560	1,210,230	1,674,325	1,674,040
Ratio (30-ft width/ 20-ft width)	525,040/533,440 = 98.43%		1,210,230/1,226,560 = 95.55%		1,674,040/1,674,325 = 99.98%	

example problem by the Colby procedure, two approaches are used. In one, a constant assumed n of 0.020 is used. In the other, the most recent and perhaps the most reliable procedure (Alam and Kennedy, 1969) for predicting friction factors (and thereby depth, velocity, and discharge relationships) is used.

The data in Table 6.2 indicate that in the stated problem the Schoklitsch formula predicts considerably less sediment transport than either of the Colby approaches. This difference may be due to the fact that the Schoklitsch formula predicts bedload and the Colby procedure accounts for suspended bed material as well as bedload. The difference between the two Colby predictions can be attributed to the different approaches for estimating the depth of flow. The first assumes $n = 0.020$ and a normal depth based on bed slope equal to friction slope; the second assumes a depth based mostly on grain roughness for friction slope. The Alam and Kennedy friction factors are never in the lower flow regime for this set of calculations; therefore, bedform changes had little effect on the results. All three results indicated a slight, but negligible, reduction (less than 5%) in sediment transport capacity for the water channel.

The next step in the analysis is to determine whether lower flows give different results. For this computation, 20% of the discharges indicated in Table 6.1 are used in Table 6.3. Table 6.4 shows the amount of sediment transported as computed by the two procedures. Table 6.4 again indicates considerable difference between the Schoklitsch and Colby predictions, but less than that shown in Table 6.2. This smaller difference can be attributed to the smaller loads in suspension for the lower flows. All three predictions, however, indicate greatly reduced sediment transport capacity for the wide (30-ft) channel (9, 17, and 32%, respectively). The most significant reduction, almost one-third, is predicted by the Colby procedure using the Alam and Kennedy friction factors. It is believed that the Colby procedure using the Alam and Kennedy factors most closely reflects the influence of variable bed forms that are more pronounced during low to moderate flows.

TABLE 6.3
Discharge Data for Example Channel Problems, Lower Flow

Hydrograph Segment	Discharge per Foot of Width: 20-ft Channel (ft^3/s)	Discharge per Foot of Width: 30-ft Channel (ft^3/s)
Rising stage		
(a) Mean flow for 2 hours, 18 ft^3/s	0.9	0.6
(b) Mean flow for 2 hours, 56 ft^3/s	2.8	1.87
Falling stage		
(c) Mean flow for 3 hours, 48 ft^3/s	2.4	1.6
(d) Mean flow for 3 hours, 36 ft^3/s	1.8	1.2
(e) Mean flow for 3 hours, 8 ft^3/s	0.4	0.267

This example clearly shows that estimates of the absolute rates of sediment transport vary according to the procedure. But, the study also shows that the relative rates can be insensitive to choice of procedure if variation in bed forms is not a factor as for channel performance at peak discharge. In many stability problems, however, the performance of the channel during one or more low to moderate flows must be considered. Formulas and procedures that determine the effect of variable bed forms on depth, velocity, and discharge relationships and, thus, on bed material discharge afford greater flexibility for all purposes.

Procedures for Evaluating Bed Material Transport Problems

Problems of bed material transport requires consideration of three elements: (1) existing conditions, (2) availability of bed material, and (3) natural or artificial changes in stream or watershed conditions. The existing conditions can be best determined

TABLE 6.4
Sediment Transport (lb) Computed for Lower Flows

	Schoklitsch Formula		Colby Procedure: Using n = 0.020		Colby Procedure: Using Alam and Kennedy Friction Factors	
Discharge Segment	20-ft width	30-ft width	20-ft width	30-ft width	20-ft width	30-ft width
a	6760	5470	9970	7195	450	700
b	26,485	25,195	53,280	46,705	61,225	41,645
c	33,500	31,560	67,580	54,615	66,255	46,245
d	24,155	22,220	43,710	36,000	39,245	24,500
e	2355	415	3315	2525	940	415
Total	93,255	84,860	177,855	147,040	168,115	113,505
Ratio (30-ft width/ 20-ft width)	84,860/93,255 = 91.00%		147,040/177,855 = 82.67%		113,505/168,115 = 67.52%	

DID YOU KNOW?

Whenever I discuss rivers and streams with students or others, invariably the term *river reach* or *reach* comes up. Many are confused as to its exact meaning—that is, its English definition. Simply, think of a river reach as an arm, which can refer to an extended portion or stretch of land or water, to a straight stretch of the stream (from one turn to another), or to a level stretch.

by field investigation and analysis. Surveys of old and new cross-sections, use of techniques for identifying depth of scour or aggradation, and comparison of series photographs all facilitate definition of the problems. Although the correct identification and analysis of existing bed material transport conditions are important, most problems require projections of what will or can occur rather than what is now occurring. The availability of bed material and the impact of change are the key elements of such projections.

Equilibrium can be achieved only if bed material is being introduced into the reach at a rate comparable to that at which bed material moves out of the reach. Problems arise when the amount introduced is greater or less than the transport capacity of the flow. In other words, equilibrium transport seldom causes problems but a change from equilibrium to nonequilibrium transport often does.

The supply of bed material can exceed transport capacity during unusually high discharges. This excess can be caused by development of new and substantial sources of bed material within or adjacent to the problem reach or by channel changes that may increase transport capacity in the upstream reach but not in the downstream reach. Determining the availability of bed material is largely a field problem. To be readily available to channel flow, sediment must be in the stream system. The coarse particles in an upland soil tend to lag behind during erosion. Gullies that feed directly into the stream system and that expose soils with a large proportion of particles of bed material size can be major contributors but do not in themselves constitute an immediate and unlimited stream channel supply.

Streambanks that have, at least in part, soil textures comparable to those in the bed, can be a ready source of supply, depending on the ease with which the flow can erode the material. A frequently used emergency flood-protection measure is to bulldoze streambed materials to each side to form banks or levees. These banks are a ready source of supply. Their erosion and the consequent deterioration of channel alignment result in overloading the flow and downstream aggradation.

Scour of bed material can result from an undersupply of sediment in an alluvial reach. Upstream changes in watershed or stream conditions that can reduce the supply of incoming bed material include the removal of supply by major flood scour and the construction of reservoirs, debris basins, or other structures.

In addition to cutting off the supply of bed material to the reach downstream, a reservoir can materially influence the stability of the channel bed and banks by modifying the flow. For example, a detention structure that controls a high flood peak can thereby extend the duration of released flows by days. The resulting bed and bank scour may be extensive because of the energetic discharge of clear water.

TABLE 6.5
Checklist of Procedures for Solving Bed Material Transport Problems

	Analysis Procedure			
Item	**Tractive Stress**[a]	**Comparative Hydraulics**[b]	**Bed Material Formulas**	**Field Evaluation**
Problem Characteristics				
Erodibility of bed	X			X
Erodibility of bed and banks	X			X
Erodibility of banks	X			X
Channel aggradation	X	X	X	
Volume of bed material			X	X
Effects of channel change		X	X	X
Channel Boundary Characteristics				
Cohesive soils	X			X
Cohesive soils or rock with intermittent deposits of sand or gravel	X			X
Sand ≤1.0 mm	X			X
Sand ≤1.0 mm with <10% gravel	X	X	X	X
Gravel, gravel mixed with sand	X	X	X	X
Gravel and boulders	X			X
Hydraulic Characteristics				
In problem reach				
Steady state or slowly changing	X	X	X	X
Rapidly changing	X	X		X
Cross-section—slope upstream vs. problem reach				
About the same	X	X	X	X
Steeper slope	X	X	X	X
Wider channel	X	X	X	X
Narrower channel	X	X	X	X

[a] For cohesive soil boundaries, analysis may include tractive power (tractive stress times mean velocity).
[b] Comparison of relationships between depth, velocity, and unit discharge in two or more reaches.

Table 6.5 provides a checklist of procedures to consider when solving problems of bed material transport. The last column in this table indicates that a field evaluation is important to the solution of any such problem. Because of the variety of factors that can influence their solution, most problems are not routine, and solving them requires the assistance of well-trained and experienced personnel. The first step should always be a field evaluation of existing or potential problems related to sediment transport. With experience, well-trained personnel frequently can find answers to questions of stability, degradation, or aggradation by relating the availability of bed material to proposed changes in the hydraulics of the flow without resorting to formulas. If formulas must be used, it should be recognized that the results are qualitative and not quantitative. Observations of similar streams having comparable drainage areas, geology, soils, topography, and runoff often provide guidance on the probable stability.

TRANSPORT OF SUSPENDED SEDIMENT

Suspended sediment load includes both the bed material load in suspension and the washload, as shown in Figure 6.2. If erosion of fine-textured soils is the chief source of sediment, the washload, not the bed material load, usually constitutes the bulk of the sediment discharge. No method exists for predicting rates of washload transport unless there is a substantial amount of data on concentrations of suspended sediment during measured discharges.

Suspension Mechanism

Bagnold (1966) explained the suspension mechanism as follows:

> Isotropic turbulence [i.e., equal physical properties along all axes] cannot by definition be capable of exerting any upward directed stress that could support a suspended load against gravity. For any suspended solid must experience over a period of time a downward flux of eddy momentum equal on the average to the upward flux. A swarm of solids would be dispersed equally in all directions by diffusing along uniform concentration gradients, but the center of gravity of the swarm would continue to fall toward a distant gravity boundary.
>
> The center of gravity of a swarm of solids suspended by shear turbulence, on the other hand, does not fall toward the gravity shear boundary. The excess weight of the solids remains in vertical equilibrium. It follows therefore that the anisotropy [property of being directionally dependent] of shear turbulence must involve as a second-order effect a small internal dynamic stress directed perpendicularly away from the shear boundary. In other words, the flux of turbulent fluid momentum away from the boundary must exceed that toward it. … The turbulence appears to be initiated and controlled by a process akin to the generation of surface waves by a strong wind. An upwelling on the part of a minor mass of less turbulent boundary fluid intrudes into an upper, faster moving layer, where its crust is progressively torn off, like spray, and mingles with the upper layer. Corresponding motions in the reverse sense are absent or inappreciable.
>
> Since there cannot be a net normal transport of fluid, the returned flow must be effected by a general sinking toward the boundary on the part of a major mass of surrounding fluid.

The settling rate for sediment particles of uniform density increases with size, but not proportionally. The settling rate for particles smaller than about 0.062 mm varies approximately as the square of the particle diameter, whereas particles of coarse sand settle at a rate that varies approximately as the square root of the diameter. The settling rate for particles of intermediate size varies at an intermediate rate. The dividing line between sediments classed as silts and those classed as sands is

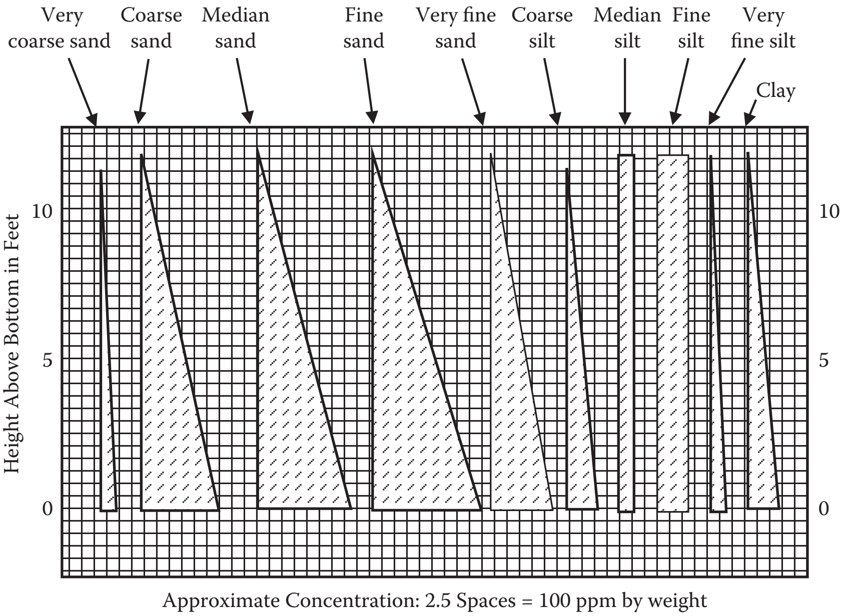

FIGURE 6.4 Approximate vertical distribution of sediment in Missouri River at Kansas City, Missouri. (Adapted from FIRBC, *A Study of Methods Used in Measurement and Analysis of Sediment Load of Streams*, Report No. 14, Federal Interagency River Basin Committee, Subcommittee on Sedimentation, Washington, DC, 1963.)

the 0.062-mm size. Clay and silt particles usually are distributed fairly uniformly in a stream, but sand particles usually are more concentrated near the bottom. The degree of variation is a function of the coarseness of the particle (see Figure 6.4).

The lateral distribution of suspended sediment across a stream is fairly uniform in both deep and shallow flows except below the junction of a tributary carrying material at a concentration substantially different from that of the main stream. The flow from the tributary tends to remain on the entrance side of the channel for some distance downstream.

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