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CHAPTER 4

SEDIMENT TRANSPORT

by

H. W. Shen, Professor of Civil Engineering, Colorado State University, Fort Collins, Colorado

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CHAPTER 4

4.1 DEFINITION

Total sediment transport can be divided three different ways:

1) By the mechanics of movements. The total sediment transport can be divided into the bed load and the suspended load. Bed load is the movement of sediment particles very close to the beds; most of the time these particles are supported by the sediment bed. The suspended load consists of those sediment particles supported by the moving fluid.

2) By methods of measurement. In this case, total sediment transport consists of the measured load and the unmeasured load. Since bed load is very difficult to measure, the measured load consists entirely of suspended load. It does not include the entire suspended load because we can only measure from the water surface to a distance about 10 centimeters from the top of the bed. Therefore, the measured load is that part of the suspended load which is at least 10 centimeters above the top of the bed. The unmeasured load consists of the entire bed load plus the suspended load within 10 centimeters of the top of the bed.

3) According to the method of calculations. In this case, total sediment transport it is equal to the wash load plus the bed material load. The concepts of wash load and bed material load may not be clear to many geologists and an explanation may be necessary. In general, the wash load is limited by the supply of particles from the watersheds to the stream. The bed material load is entirely determined by the capability of the flow to transport sediments.

The best that a sediment transport equation based on river flow conditions can do is to predict the sediment transport capability of a given flow for a certain sediment mixture. For instance, one may hope to obtain a relationship between transport capability and sediment size for a flow discharge q on a particular river as shown by curve COD in Figure 4.1. One may also plot the available supply of various sediment sizes from the upslope area for the same river discharge q, as indicated by AOD on Figure 4.1. The intersection of these two curves may not be

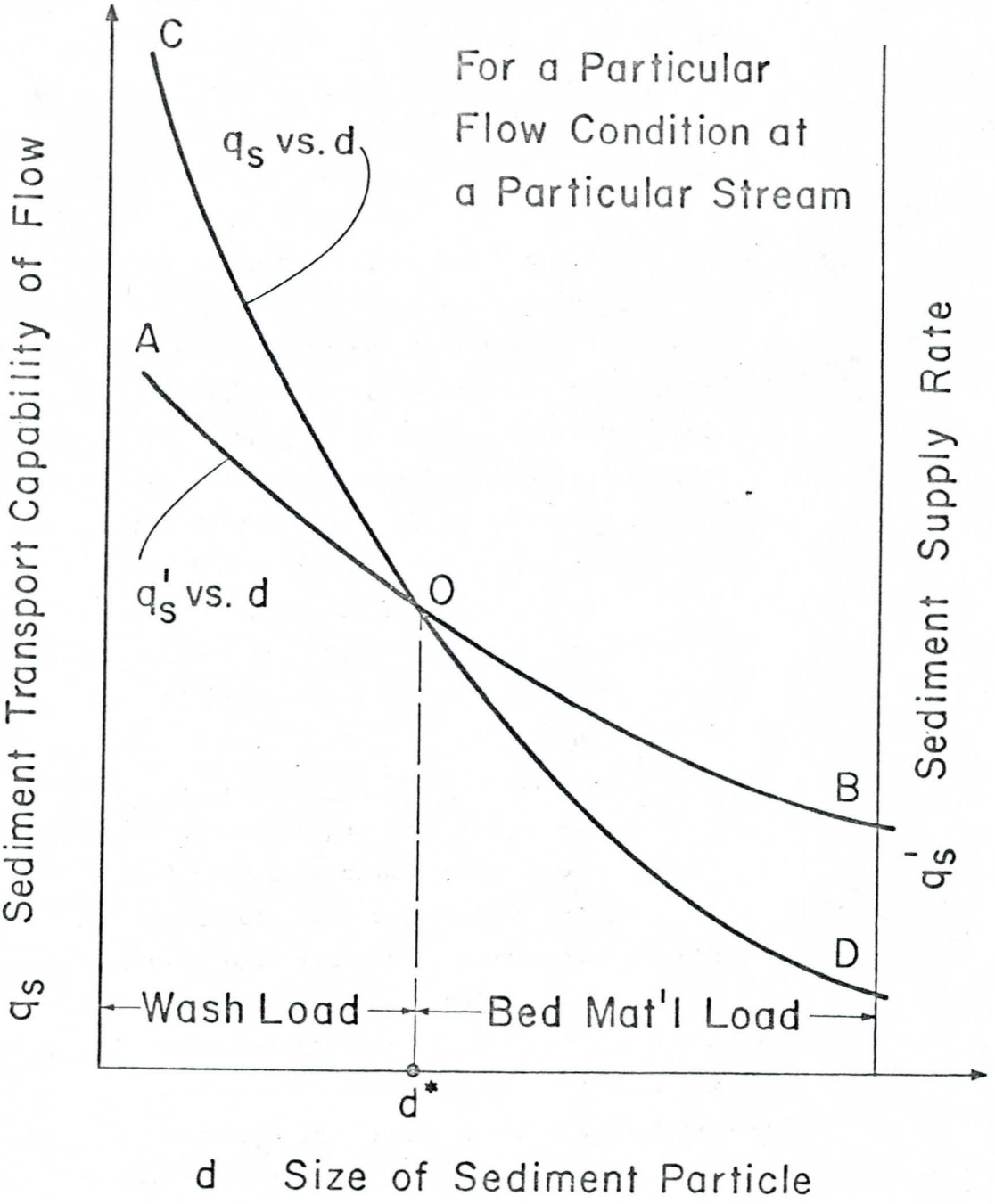

Figure 4.1 Division between wash load and bed material load.

distinct as shown; however, let us assume that they intersect at point 0. The sediment size at point 0 is d_s^*. For sediment sizes larger than or equal to d_s^*, the available supply from the upslope area is equal to or greater than what the river can carry, and therefore deposition of these sizes will occur in the upstream reach; the river's ability to transport these sediment sizes is dictated by its transport capability in the lower reach. One may hope to establish a sediment transport equation, based on the sediment transport capability of the river, that agrees with the actual sediment transport in the lower reach. However, for sediment sizes smaller than d_s^*, the transport capability of the river exceeds the supply from upslope, the actual amount of sediment of this size transported is determined by the rate of upslope supply or production, and not by the capability of the river to transport. Obviously, in order to predict the actual amount of this sediment transported, one must study the upslope erosion rate and not the sediment transport capability of the river. Knowledge of d_s^* is important because for a sediment size equal to or greater than d_s^*, the actual transport rate equals the sediment transport capability and may be determined by a sediment transport equation (based on river flow condition). For sediments smaller than d_s^*, the actual transport rate equals the upslope sediment supply rate, and therefore must be estimated by a different technique. Consequently, the best that a sediment transport equation based on river flow can do is to predict sediment transport rates of sizes equal to and larger than d_s^*.

Wash load is defined as that portion of the sediment load governed by the upslope supply rate, and is considerably less than the sediment transport capability of a river. Wash load size is less than d_s^*, as defined in Figure 4.1. Bed material load is defined as the portion of the sediment load that is governed by the sediment transport capability of a river and is less than or equal to the upslope supply rate. The sizes of sediment in the bed material load is equal to or larger than d_s^*. The sum of the wash load and the bed material load equals the total sediment load.

Since d_s^* is determined by the intersection of the sediment transport capability curve and the available sediment supply from the upslope curve, the value of d_s^* will certainly be changed if either or both of these curves change.

The remainder of this chapter is devoted to a discussion of wash load, bed load and bed material load.

4.2 WASH LOAD

By definition, wash load is that portion of the sediment transport rate which is supplied to the stream from the watershed. Soil erosion formulae must be used to estimate the erosion rate, and thus the supply rate to the stream. Almost all of these equations were obtained after years of data collection from different watersheds.

The U.S. Department of Agriculture began to study soil loss in about 1930, when the first ten Federal-State Cooperative Stations began operation. Thirty-two additional stations were established in the next 25 years. Measurements of precipitation, runoff and soil loss at these 42 stations in 23 states east of the Rocky Mountains were collected continuously for periods of from 5 to 30 more years. The field plots were rectangular to facilitate normal flow row spacing for cultivated units. A "unit plot" 22.1 meters long on a nine percent uniform slope, continuously in bare fallow soil and tilled to break surface crusts was arbitrarily selected to serve as a common reference point for evaluation. The dimensions were chosen because most of the plots in U.S Department of Agriculture erosion studies from 1930 to 1960 were 22.1 meters long and nearly nine percent slope.

Six major factors describing rainfall (R), soil erodibility (K), slope length (L), slope steepness (S), cropping and management (C), and supplemental erosion-control practices such as contouring, terracing, etc. (P) were used to develop the following Universal Soil Loss Equation based on the field data described above and rainfall simulation data by a multiplicative model:

$$\underline{E} = RKLSCP \qquad (4\text{-}1)$$

where $\underline{E}$ is the computed soil loss per unit area, expressed in the units selected for K and for the period selected for R. In practice, these units are usually so selected that they compute $\underline{E}$ in tons per acre per year, but other units can be selected. R, the rainfall and runoff factor, is the number of rainfall erosion index units, plus a factor for runoff from snowmelt or applied water where such runoff is significant. K, the soil erodibility factor, is the soil loss rate per

erosion index unit for a specified soil as measured on a unit plot, which is defined as a 72.6-ft length of uniform 9 percent slope continuously in clean-tilled fallow. L, the slope-length factor, is the ratio of soil loss from the field slope length to that from a 72.6-ft length under identical conditions. S, the slope-steepness factor, is the ratio of soil loss from the field slope gradient to that from a 9 percent slope under otherwise identical conditions. C, the cover and management factor, is the ratio of soil loss from an area with specified cover and management to that from an identical area in tilled continuous fallow. P, the support practice factor, is the ratio of soil loss with a support practice, such as contouring, stripcropping, or terracing, to that with straight-row farming up and down the slope.

Numerical values for each of the six factors were derived from analyses of the assembled research data and from National Weather Service precipitation records. For most conditions in the United States, the approximate values of the factors for any particular site may be obtained from Wischmeier and Smith (1978). Localities or countries where the rainfall characteristics, soil types, topographic features, or farm practices are substantially beyond the range of present U.S. data will find these charts and tables incomplete and perhaps inaccurate for their conditions. However, they will provide guidelines that can reduce the amount of local research needed to develop comparable charts and tables for their conditions. Smith and Wischmeier (1957), Meyer (1971), Wischmeier (1973), and Wischmeier and Smith (1978) gave detailed analyses of recommended values for these factors.

The Universal Soil Loss Equation is designed to predict average annual soil losses on sheet and rill erosion on uplsope areas such as farmland and construction sites. It can be helpful for prediction of contributions from these sources to downstream sediment loads, but its capabilities and limitations for this use must be recognized. It is an erosion equation and is not designed to predict deposition. Its predictions do not include sediment contributions from gully erosion, and it does not include factors to account for sediment losses or gains between the field and the stream or reservoir. These items must be evaluated separately.

Many investigators have also formulated difficult equations to estimate soil erosion from experimental data. Table 4.1 gives a brief summary of some available soil loss equations, where L_o is the plot length in feet, S_o is the bed slope, and S_c and L_c are the values of S_o and L_o at which erosion begins.

Shen and Li (1976) presented a theoretical analysis of these soil loss regression equations. Of the six factors affecting soil erosion, it is impossible at this stage to make a theoretical analysis of the cropping management factor C and the erosion control factor P. It is also difficult to analyze the soil erodibility factor K, because the relationship between soil characteristics and soil erosion loss was not given in a closed form.

Assume that: 1) the overland flow reach is wide and one-dimensional; 2) hydrostatic pressure distribution is valid across the flow depth; 3) the variation of the momentum coefficient β along the direction of flow is negligible; 4) the soil layer is loose and uniform, with fine-sediment size; 5) the sediment concentration is small so the equation of motion for sediment-laden water can be approximated by the equation of motion for water only; 6) the variation of bottom slope S_o is negligible; 7) the rainfall intensity and the infiltration rate are constant and the approximate solutions to the mechanics of steady sheet flow can be developed; 8) the continuity equation for sediment can be expressed as

$$\frac{dq_s}{dx} = p_s \tag{4-2}$$

where q_s is the sediment discharge per unit width of channel and p_s is the fine-sediment pickup rate per unit area; and 9) the rate of fine-sediment pickup is assumed to be a power function of τ, the boundary shear stress of surface runoff. The relation is

$$p_s = a\,\tau_o^b \tag{4-3}$$

where a is a constant describing the erodibility of a specific soil and b is an exponent which, judging from the existing sediment transport equations, is assumed to be 2.0. However, a and b can be determined

Table 4.1 Soil loss equations.

Investigator	Relationship presented
Zingg (1940)	$\underline{E} \alpha L_o^{0.66} S_o^{1.37}$
Musgrave (1947)	$\underline{E} \alpha RKL_o^{0.37} S_o^{1.35} C$
Wischmeier and Smith (1965)	$\underline{E} \alpha RKL_o^{0.5} (0.00076 S_o^2 + 0.0053 S_o + 0.0076) CP$
Meyer and Monke (1965)	$\underline{E} \alpha L^{0.9} S_o^{3.5}$
Meyer (1965)	$\underline{E} \alpha (L_o - L_c)^{1.\sim 1.5}$, $E \alpha (S_o - S_c)^{1.5}$
Meyer and Kramer (1968)	$\underline{E} \alpha L^{0.5} (S_o - S_c)^{1.4}$
Young and Mutchler (1969)	$\underline{E} \alpha L^{1.24} S_o^{0.74}$
Kilinc (1972)	$\underline{E} \alpha L^{1.035} S_o^{1.664}$

by an optimization scheme. Shen and Li found that for a small Reynolds number of the flow

$$\underline{E} = R_k L_o^{0.67} S_o^{1.33} \tag{4-4}$$

and for a large Reynolds number of the flow

$$\underline{E} = R_k L_o^{1.17} S_o^{1.33} , \tag{4-5}$$

where R_k is a factor describing the rainfall characteristics, soil erodibility, and fluid properties.

The exponents of slope length and percent slope in the above equations are consistent with the regression equations given in Table 4.1. The analytical results indicate that the exponents of L_o and S_o for most of these empirical equations are reasonable, and that these exponents should be different for different Reynolds numbers.

Although these equations seem to be reasonable, one must be very cautious in using any of them. Erosion of soil in watersheds is an extremely complex matter and there are so many factors involved that these equations can only provide qualitative answers at best.

4.3 BED LOAD

Generally the bed load is small compared to suspended load in the main flow. In most of the canals and rivers in West Pakistan the bed load is only about five percent of the total load, and may even be neglected for computing the total sediment load. If this is the case, why should one even consider bed load? The answer is that although the bed load may be small compared to the total sediment load, it is nevertheless important because it shapes the bed and is a major factor in determining the stability of the channel, the form of the sediment bed surface, etc.

The bed load equation can be classified into the following four categories. From a great deal of practical experience, engineers have found that bed load is a function of the flow velocity, V, and the characteristics of sediment. Thus, the following type of equation is developed:

$$q_B = \text{bed load (rate)} = k_1 V^{k_2} \tag{4-6}$$

They have also found that velocity is much more important than flow depths or slopes or other parameters in the determination of the bed load. However, it was later found that this equation does not satisfy the condition that there is a critical velocity below which no bed load movement would occur. In order to satisfy that condition, engineers have proposed a second equation

$$q_B = k_1 (V-V_c)^{k_2} \tag{4-7}$$

where V_c is defined as the critical velocity below which no sediment transport would occur. This equation does satisfy the initial condition that when $V \leq V_c$, q_B is zero or even negative, which is impossible. Although this equation does satisfy the initial condition for incipient motion, there is no physical reason to believe that if $V > V_c$, q_B should be a function of $V - V_c$ rather than a function of V as indicated in Equation 4-6. In recent decades it has been found that shear stress on the bed may be a better parameter than flow velocity to indicate the bed load rate. Thus, engineers propose the equation

$$q_B = k_1 \tau^{k_2} \tag{4-8}$$

In order to satisfy the initial condition for incipient motion, engineers propose Equation 4-9 where

$$q_B = k_1 (\tau - \tau_c)^{k_2} \qquad (4\text{-}9)$$

In the above equation τ is the shear stress on the bed and τ_c is the shear stress on the bed for incipient motion. Similar to Equation 4-7, this Equation 4-9 also satisfies the initial condition that when $\tau \leq \tau_c$, q_B is zero or negative, which is impossible. Laursen (1956) reviews several well-known bed load equations as follows:

	Original Form	Reduced Form
DuBoys (Straub)	$q_B = A_1(\tau-\tau_c)$	$= B_1 n^4 \frac{V^4}{d^{2/3}}$
Schoklitsch (Shulits 1935)	$q_B = \frac{A_2}{d_s^{1/2}} S^{3/2}(q-q_c)$	$= B_2 \frac{n^3}{d_s^{1/2}} \frac{V^4}{d}$
Meyer-Peter et al. (1934)	$q_B = (A_3 q^{2/3} S - A_4 D)^{3/2}$	$= B_3 n^3 \frac{V^4}{d}$
Wes (1935)	$q_B = \frac{A_5}{n} (\tau-\tau_c)^m$	$= B_5 n^{2m-1} \frac{V^{2m}}{d^{m/3}}$
Shields (1936)	$q_B = \frac{A_6}{d_s} q\, S(\tau-\tau_c)$	$= B_6 \frac{n^4}{d_s} \frac{V^5}{d^{m/3}}$
Brown-Einstein (1950)	$q_B = \frac{A_7}{d_s^{3/2}} \tau^3$	$= B_7 \frac{n^3}{d_s^{3/2}} \frac{V^6}{d}$
Brown-Kalinske (1950)	$q_B = \frac{A_8}{d_s} \tau^{5/2}$	$= B_8 \frac{n^5}{d_s} \frac{V^5}{d^{5/6}}$

where m, A_1 through A_8 (inclusive) and B_1 through B_8 (inclusive) are constants, n is Manning's roughness, d is the flow depth, and d_s is the sediment size.

Of these equations the one developed by Einstein (1950 probably is the most reliable because it is based on a very comprehensive study and has been most extensively tested. It should also be pointed out that these equations are applicable only to cohesionless material in steady and uniform flow. Another limitation could probably be added--that

these equations are not useful if the ratio between the flow depth and the sediment particles size is too small. A possible improvement over these equations is through the formulation of stochastic bed load. For a general discussion of stochastic bed load models the reader is referred to Shen (1976).

Meyer-Peter and Müller (1948) developed the following equation based on experiments with sand particles of uniform size, sand particles of mixed sizes, natural gravel, ligmite, and baryta:

$$\left(\frac{Q_b}{Q}\right)\left(\frac{K_b}{K_r}\right)^{3/2} \gamma y_o S = B'(\gamma_s - \gamma)D_m + B\left(\frac{\gamma}{g}\right)^{1/3}\left(\frac{\gamma_s - \gamma}{\gamma_s}\right)^{2/3} q_B^{2/3} \qquad (4\text{-}10)$$

where q_B is the bed load rate in weight per unit time and per unit width, Q_b is water discharge quantity determining bed load transport, Q is total water discharge, y_o is the depth of flow, S is the energy slope and B' and B are dimensionless constants. B' has the value 0.047 for sediment transport and 0.034 for the case of no sediment transport. B has a value of 0.25 for sediment transport and is meaningless for no transport, since q_B is zero and the last term drops out. Equation 4-10 is dimensionally homogeneous so that any consistent set of units may be used.

The quantities K_b and K_r are defined by the expressions

$$V = K_b R_b^{2/3} S^{1/2} \qquad (4\text{-}11)$$

and

$$V = K_r R_b^{2/3} S'^{1/2} \qquad (4\text{-}12)$$

where S' is the part of the total slope, S, required to overcome the grain resistance and S-S' is that part of the total slope required to overcome form resistance. Therefore,

$$\frac{K_b}{K_r} = \sqrt{\frac{f_b'}{8}}\ \frac{V}{\sqrt{g\ R_b S}} \qquad (4\text{-}13)$$

where f_b' is the Darcy-Weisbach bed friction factor for the grain roughness. If the boundary is hydraulically rough ($V_* D_{90}/\nu \geq 100$), K_r is given by

$$K_r = \frac{26}{D_{90}^{1/6}} \tag{4-14}$$

in which D_{90} is in meters and K_r is in $m^{1/3}$ sec. The quantity D_m is the effective diameter of the sediment given by

$$D_m = \frac{\sum_i p_i D_i}{100} \tag{4-15}$$

where P_i is the percentage by weight of that fraction of the bed material with geometric mean size, D_i.

The Meyer-Peter and Müller formula (Equation 4-10) is often written in the form

$$q_b = K (\tau - \tau_c)^{3/2} \tag{4-16}$$

where

$$K = \left[\frac{1}{B(\frac{\gamma}{g})^{1/3} (\frac{\gamma_s - \gamma}{\gamma_s})^{2/3}} \right]^{3/2}$$

4.4 SEDIMENT BED MATERIAL LOAD

Rational Approaches

Lane and Kalinske (1941) and Kalinske (1947) probably made the first successful attempts to determine suspended sediment discharge by integrating Equation 4-17 below. They first found the bed load sediment concentration and then, using that as the reference sediment concentration at a certain level, integrated Equation 4-17 to give the total bed material load.

Einstein (1950) also integrated Equation 4-17 to obtain suspended load from bed load.

$$\frac{C_y}{C_a} = \left(\frac{d-y}{y} \quad \frac{a}{d-a}\right)^Z \tag{4-17}$$

where C_y and C_a are suspended sediment concentrations at distances y and a from the top of the bed surface and Z is a function of shear velocity and sediment size. He first determined the bed load for a particular particle size and assumed from experimental evidence that the bed load concentration occurred at two grain diameters from the bed

in order to integrate Equation 4-17 to obtain the suspended load for that size. Einstein gave an example to illustrate the application of his procedure in calculating the total sediment transport rate for Big Sand Creek, Mississippi.

Einstein's procedure, although theoretically sound, does involve many assumptions. Some of these assumptions are necessary if no other data are available. However, if sediment concentration and stream flow can be measured in a river, one may find the total load according to a modified procedure.

Colby and Hembree (1955) proposed a modified Einstein procedure to obtain the total sediment transport rate in a river. The term "modified Einstein procedure" usually gives the false impression that it serves the same purpose as Einstein's procedure. Actually, these two procedures, although based on similar principles, serve entirely different purposes. As stated in the previous paragraph, Einstein's procedure estimates total sediment bed material load for different river discharges based on the channel cross section and a sediment bed sample from a selected river reach with uniform flow. This procedure is mainly for design purposes. The modified Einstein procedure developed by Colby and Hembree only estimates the total sediment load (including wash load) for a given discharge from the measured depth integrated suspended sediment load, the stream flow measurements, the bed material samples, and the water temperature for this discharge at a given cross section.

The major modifications from Einstein's original procedure used in the modified Einstein procedure are:

1. The calculation is based on a measured mean velocity rather than on the slope, and the depth is observed for each velocity.
2. The friction velocity and the corresponding suspended load exponent Z in Equation 4-17 are determined from the observed Z value for a dominant grain size and are assumed to change with the 0.7 power of the settling velocity.
3. A slight change in the hiding factor is introduced.
4. Flow depth replaces the hydraulic radius.
5. The value of Einstein's intensity of bed load transport is arbitrarily divided by a factor of two to fit the data more closely. Since the modified Einstein procedure essentially

estimates the total sediment load from the measured sediment load, it no doubt can give better agreement with field data than the Einstein procedure, which is based on more assumptions and fewer data. Since the modified Einstein procedure relies on depth-integrated suspended sediment samples, it should be more reliable with shallow streams where the sediment concentration variation is less than in deep rivers.

Bishop, Simons, and Richardson (1965) present other modifications of the procedure presented by Einstein (1950).

Toffaleti (1968, 1969) presents a procedure for the analytical determination of sediment transport based on the concepts of Einstein (1950) and Einstein and Chien (1953). First, he establishes that Einstens's ψ versus ϕ curve may be represented by the equivalent expression,

$$\psi = \frac{TA}{V^2} 10^4 d \qquad (4\text{-}18)$$

in which ψ is Einstein's transport parameter, T (unit in length/time^2) is a parameter that includes constants and those components of shear force that are functions of water temperature, A is a dimensionless correction factor to replace Einstein's correction factors for sediment of mixed sizes, V (unit in length/time) is the mean flow velocity, and d (unit in length) is the grain diameter. He divides the flow depth into three zones: the lower zone where the depth of flow is less than R/11.24 (R is the hydraulic radius), the middle zone where the depth is between R/11.24 and R/2.5, and the upper zone, for a depth greater than R/2.5. Toffaleti further states that the G_F, or nucleus load, in tons per day, computed for a 1-foot width in lower zone and assuming the bed is composed entirely of one sand, can be represented by

$$G_F = \frac{0.600}{\left(\frac{TA}{V^2}\right)^{5/3} \left(\frac{d}{0.00058}\right)^{5/3}} \qquad (4\text{-}19)$$

For very fine sand ($d_s < 0.00029$ feet) the equation would be

$$G_F = \frac{1.905}{\left(\frac{TA}{V}\right)^{5/3}} \qquad (4\text{-}20)$$

The form of Equations 4-14 and 4-20 is based on Equation 4-18. The exact relationships are determined from field data as measured in a large river (the Atchafalaya River). These data were taken over a 20-year period covering a range of discharges of 20,000 cfs to 500,000 cfs. (The above information was obtained by private communication with Toffaleti.) The sediment concentration distribution is expressed as

$$C_y = C_a \left(\frac{R}{y}\right)^z \tag{4-21}$$

in which C_y is the sediment concentration at y, C_a is the sediment concentration at a level a, and y is the vertical elevation. For his middle zone,

$$Z = \frac{VW}{C_z SR} \tag{4-22}$$

in which W is the fall velocity of the particle, C_z is a temperature correction factor which equals 260.67 - 0.667x (degrees Fahrenheit), and S is the energy or water surface slope. The exponents, Z, of the sediment concentration distribution at the lower and upper zones are, respectively, 0.756 and 1.5 times that of the middle zone.

After knowing G_F and the sediment concentration distribution in the lower zone, one can proceed to determine the sediment concentration at the upper edge of the lower zone. With G_F and the sediment concentration distribution in the middle zone, one can determine the sediment concentration load in the middle zone, and similarly, one can obtain the total sediment load in the upper zone. The summation of the total sediment loads in the three zones is of course the total sediment load in the entire flow depth.

Bagnold (1966) introduced the concept that the sediment transport mechanism is related to the availability and the efficiency of stream power to transport sediments.

Empirical Approaches

Because most of the rational approaches described previously involve rather complex procedures and many questionable assumptions, there is definitely a need to develop a simple relationship between the sediment transport rate and the flow condition, based entirely on available data,

to provide the design engineer with an estimation that is of the correct order of magnitude. In order to achieve a successful empirical approach, one must first select all the important factors involved. It is generally agreed that the flow velocity, the flow depth (or hydraulic radius), the energy slope, the characteristics of the sediment, and the temperature of the fluid are important factors; whether or not all these factors must be included in the analysis is, of course, subject to interpretation.

In making the analysis, one must recognize that the sediment concentration increases much faster than does of flow discharge. There is a generalized rule that sediment transport rate increases with flow velocity to the fourth power at low flow discharges, and increases with flow velocity to the eighth power at high flow discharges. In other words, there is not much hope of finding a single combination of flow and sediment characteristics to describe sediment concentration for all flow conditions. Recognizing this fact, Colby (1964) developed different sediment discharge relationships with flows for 0.1-foot, 1-foot, 10-foot, and 100-foot flow depths; Maddock (1969) provided different sediment discharge relationships with flow for low-, mid-, and high-velocity ranges, and Shen and Hung (1972) constructed a sediment-transport parameter and determined the concentration as a function of this parameter.

Colby (1964) investigated the effect of mean flow velocity, shear, shear velocity computed from mean velocity, stream power of flow, flow depth, viscosity, water temperature, and concentration of fine sediment on the discharge of sand per foot of channel width. He recommended three diagrams. In spite of many inaccuracies in the available data and uncertainties in the graphs, Colby found that "...about 75 percent of the sand discharges that were used to define the relationships were less than twice or more than half of the discharges that were computed from the graphs of average relationship. The agreement of computed and observed discharges of sands for sediment stations whose records were not used to define the graphs seemed to be about as good as that for stations whose records were used." Note that all curves of 100-ft depth, most curves of 10-ft depth and part of the curves of 1.0-ft and 0.1-ft depths for Colby's diagrams are not based on available data and are extrapolated. His curves are given in Figure 4.2 and the corrections for temperature and time suspended sediment are given in Figure 4.3.

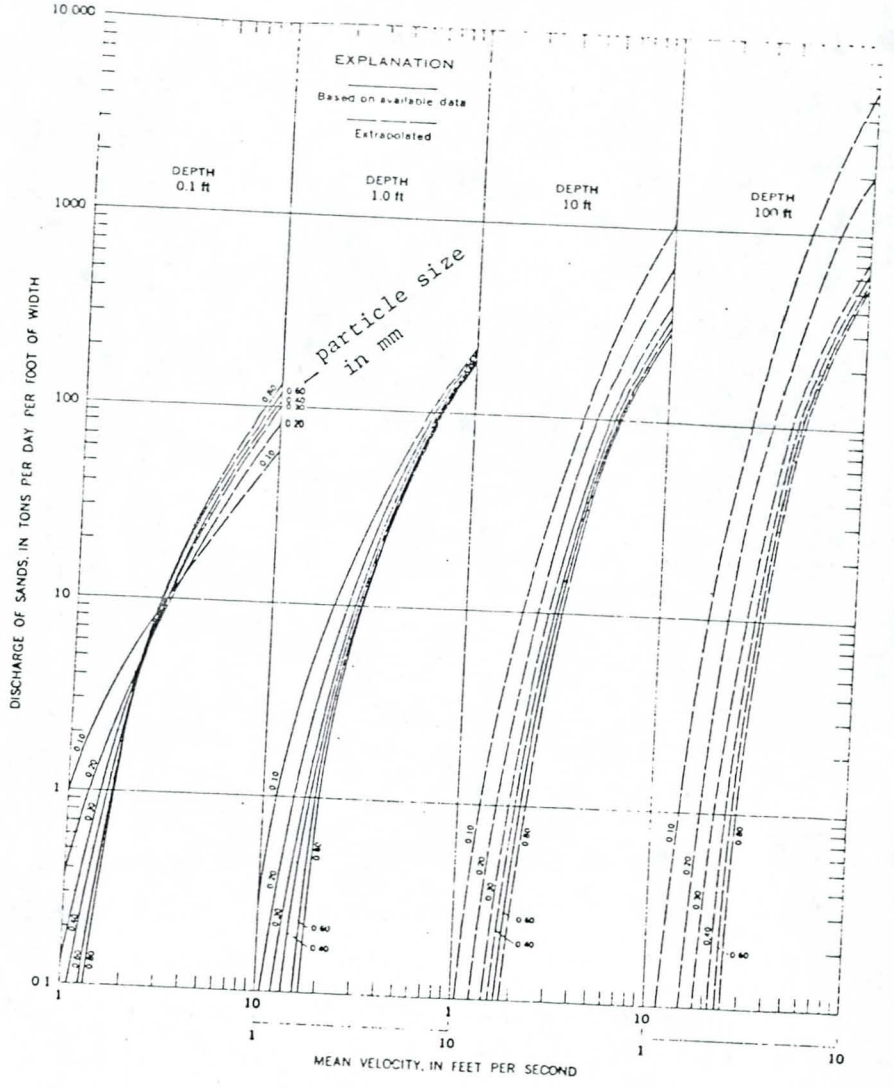

Figure 4.2 Relationship of discharge of sands to mean velocity for six median sizes of bed sands, four depths of flow, and a water temperature of 60°F, after Colby (1964).

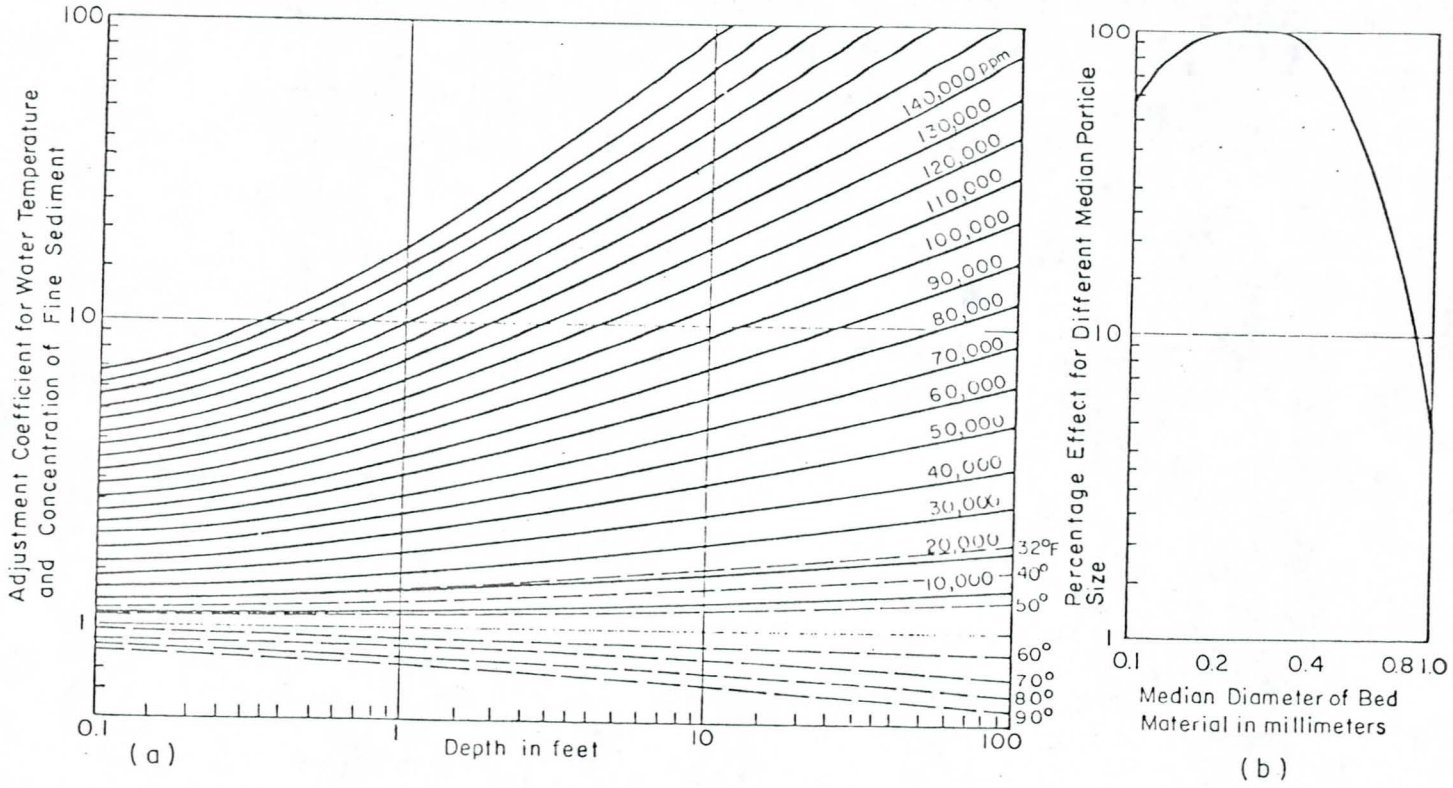

Figure 4.3 Approximate effect of water temperature and concentration of fine sediment on the relationship of discharge of sands to mean velocity, after Colby (1964).

Shen and Hung (1972) began with the assumption that sediment transport is such a complex phenomenon that no single Reynolds number, Froude number, or combination thereof can be found to describe sediment motion under all conditions. They recommended use of a regression method to develop a formula based on all available data for immediate engineering purposes. The disadvantage of this approach is, of course, that the final flow parameter will probably be dimensional, but the approach has merit in that if all previous data are found to correlate well, it is likely that other data within the same range will follow the same trend. They selected the sediment concentration (bed material load) as the dependent variable and the fall velocity of the median sediment particle of the bed sample, the flow velocity, and the flow depth as the independent variables.

Based on available reliable flume data and a few river data, they found that the sediment concentration is a function of $V^{0.57}\ S/w^{0.32}$ where V is the flow velocity in feet per second, S is the energy slope, and w is the fall velocity in feet per second of the median sediment size. Since this is an empirical dimensional equation it is not easy to convert it to cgs units. This curve is shown in Figure 4.4.

Yang (1972) presented another regression equation based on his stream power concept. He found that the sediment concentration C is a function of stream power (product of flow velocity and energy slope), sediment particle size, shear velocity, fall velocity of sediment particle, etc. Table 4.2 gives a comparison of many equations by Yang (1977) based on all data available to him.

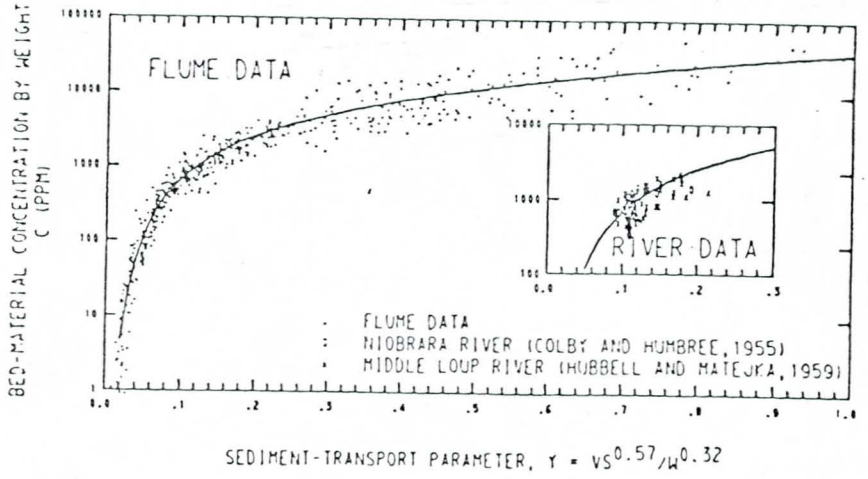

Figure 4.4 Bed-material load vs flow (all primary data).

Table 4.2 Summary of accuracies of different equations.

Equations	Date	Data with Discrepancy Ration Between 1/2 and 2
Yang	1973	91 percent
Shen and Hung[a]	1972	85 percent
Ackers and White	1973	68 percent
Engelund and Hansen	1967	63 percent
Rottner	1959	56 percent
Einstein	1950	46 percent
Bishop et al.	1965	39 percent
Toffaleti	1969	37 percent
Bagnold	1966	22 percent
Meyer-Peter and Müller	1948	10 percent

[a]Should not be applied to large rivers.

4.5 CONCLUSIONS

Because of the tremendous uncertainties, it is difficult to make any final recommendations on the estimation of sediment transport which is, unfortunately, an extremely important problem. However, the following procedures are suggested for analyzing field data:

1. Use the modified Einstein's method (Colby and Hembree, 1955) to estimate the unmeasured suspended load and bed load based on measured data. There is a question on whether Einstein's intensity of bed load transport should be arbitrarily divided by a factor of two.
2. Separate bed material load and wash load and analyze them separately.
3. Decide which available sediment transport equation best agrees with the measured data and use it to estimate the sediment transport load for design flow where actual measurement is not available.
4. If the measured data do not agree with any available sediment transport equation, use a simple power relationship between flow discharge and sediment transport rates. These terms are given as Equations 4-6, 4-7, 4-8, and 4-9.

When no measured data is available, the writer is inclined to:

1. Use Einstein's (1950) procedure if bed load is a significant portion of the total bed material load. Meyer-Peter's (1948) equation should be used if bed load consists of very large-sized material.
2. Use Colby's (1964) method for rivers with flow depth up to about 10 feet; also see 4 below.
3. Use Toffaleti's (1969) method for large rivers.
4. Use methods by either Shen and Hung (1971) or Yang (1972) for reference purposes.

4.6 APPENDIX. SEDIMENT MEASURING EQUIPMENT

Introduction

The most complete set of reports to describe flow- and sediment-measuring equipment were prepared by the U.S. Inter-Agency Committee on Water Resources. Since the U.S. Geological Survey is the official data collection agency for the U.S. government, any problem related to measurements should be related to them. A series of manuals entitled "Techniques of Water Resource Investigations of the United States Geological Survey" can be purchased from the U.S. Printing Office, Superintendent of Documents, Washington, D. C. Section Three of Book A is written on surface water measuring techniques.

Site Selection for Gaging Stations

This is an extremely important item for consideration because the usefulness of this data depends a great deal on choosing the proper sites.

Consideration should be given to the following items:

1. Natural controls or opportunity to install artificial controls so that the relationship between stage and discharge is constant.
2. Accessibility, especially during flood.
3. No backwater so that the relationship between stage and discharge is distinct.
4. Uniform reach so that the effect by secondary currents is not strong.
5. Suitability for installing measuring structures.

Stream Flow Measurements

1. Current meter: A price current meter is the most commonly used method to measure flow velocities. The meter usually consists of six cups and the velocity is measured by noting the number of revolutions in a given time interval. It is a durable piece of equipment. The total flow discharge is the sum of the product of velocity and flow area for each subsection.

2. Dilution techniques: The steady-feed method (continuous source) and the instantaneous source method are used. The steady-feed method is illustrated below.

   As shown in Figure 4.5: a constant injection of tracer $C_1 Q_T$ is introduced at x_1. (C_1 is the tracer concentration at 1 and Q_T is the tracer design). By continuity relationsips between x_1 and x_2

$$Q_T C_1 = (Q + Q_T)C_2 \tag{4-23}$$

   where Q is the flow discharge and C_2 is the concentration of tracer at status x_2. Thus,

$$Q = \frac{Q_T C_1}{C_2} \tag{4-24}$$

   if $Q_T << Q$.

   Thus the flow discharge Q can be obtained if Q_T, C_1 and C_2 are known.

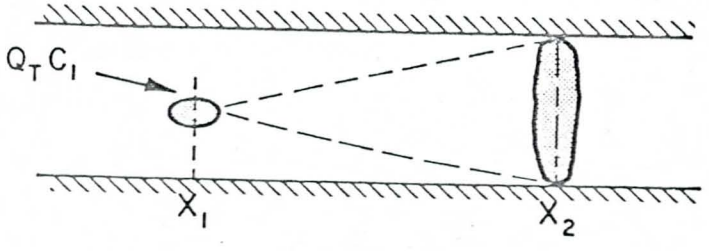

Figure 4.5 Use of dye-dilution method to estimate flow discharge.

The main assumption there is that 1) the total Q at x_1 and x_2 is the same, and 2) C_2 is well mixed in the flow $Q + Q_T$.

The instantaneous source method is to inject a quantity of tracer W at x_1. Since

$$W = \int_0^\infty QCdt \tag{4-25}$$

and the flow discharge Q is approximately constant,

$$Q = \frac{W}{\int_0^\infty cdt} \tag{4-26}$$

3. Other methods: If a stream is relatively narrow, a weir can be constructed for flow measurements. Another method would be to continue the flow discharge from Manning's equation that

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2} \tag{4-27}$$

where n is the Manning's roughness, A is the estimated flow area, R is the estimated flow depth, and S is the estimated channel bottom slope.

Sediment Discharge

1. Bed load is the most difficult item to measure. There are four methods to measure it and these are listed below according to their relative accuracies.
   A. Bed load sampler. This is useful if the stream bed surface is relatively flat (no dunes or ripples), and if the particles are not too big. The Helly-Smith bed load sampler is a popular instrument.
   B. Turbulent flume. The basic principal here is to stir up the flow so that the bed load is carried in suspension and can be measured as part of the suspended load.
   C. Bed load trap. Measuring the deposit in a trap at the stream bottom is a method that can be used to measure bed load in mountain streams. However, the suspended load deposition in the trap cannot be estimated.

D. Monitor the movements of dunes and other bed forms. Usually this method would overestimate the bed load movement because some sediment in the bed form is stationary even though the entire bed form progresses downstream.

2. Suspended load. All of the standard U.S. sediment-sampling equipment is described in the Inter-Agency Reports. The three commonly used depth integrated samplers are the U.S. DH-48, DH-59 and D-49 Samplers. DH-48 is used in shallow streams. The sampler is calibrated with a 1/4 or 3/16 inch diameter nozzle and can sample to within 3-1/2 inches of the streambed. A standard stream-gaging wading rod is threaded into the top of the sampler body.

The U.S. DH-59 is used with a hand-line suspension in streams that cannot be waded but have flow velocities less than about 6 feet per second. It is similar to the DH-48, except that tail vanes orient this sampler into the flow. The sampler weighs 24 pounds.

The U.S. D-49 sampler has a cast bronze streamlined body and weighs 62 pounds. The hose of the sampler is drilled and tagged for intake nozzles, 1/4, 3/16 and 1/8 inch inside diameter. It is suitable to a flow depth of about 15 feet and up to a flow velocity of about 7 feet per second.

The new P-61 and P-63 point-integrated samplers have replaced the old P-46 and P-50. Maximum sampling depth reaches 150 to 200 feet. All of the samplers are streamlined and cast in bronze, with tail fins to orient the sampler into the flow. They have either a two- or three-position solenoid-activated valves that closes the nozzle, equalize the pressure in the sampler container to avoid an initial surge when sampling starts, and open the sampler nozzle.

3. Bed material size. The size distribution of bed material is an important property of a stream, especially if the bed load or the bed material load is to be estimated by computational methods.

The BMH-53 is designed for shallow streams that can be waded and collects a core 2 inches in diameter and 8 inches

deep. The BM-59 can be used to collect bed material samples from a stream or reservoir at any depth. It is 22 inches long and made of cast steel. When it is supported by a cable, a scoop bucket in the bottom can be set in an open position and when the cable tension is released by resting the sampler on the stream bed, the bucket snaps shut, collecting a sample from the top two inches of the stream bed. The BMH-60 is similar to the BM-59, except it is designed to operate from a handline rather than a cable suspension.