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Hydraulics of Sediment Transport

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1. Introduction

This chapter deals with the theoretical and experimental considerations of hydraulics of sediment transport, involved in identifying the hydraulics formulas for fluid flow and sediment computation in open channels, and analyzing the flow and sediment characteristics of the water motion. In general, the field of sediment transport is very complex, and the engineers in this field should refer to more comprehensive works to better understand the computational basis.

The hydraulics of flow in a river and its sediment transport characteristics are the two basic phenomena that determine it's geometric and plan form shape. There are many variables that affect the hydraulics of flow and the nature of sediment transport in a natural stream. As indicated Yang et al. (1996), the Yellow River in China is notorious for enormous amount of sediment it carries. The total average annual sediment discharge to the sea in China is about 1.94×10^9 t. of which 59% comes from the Yellow River. A concentration of $911 kg/m^3$ was measured on September 7, 1977, at the Saumenxia Station near the entrance of the lower Yellow River.

The condition of incipient sediment motion is important in a large variety of problems associated with sediment transport. For more than two centuries workers in this field have attempted to formulate the conditions of incipient motion.

Many research programs have been devoted to the study of the sediment transport in channels. Extension can be found by Vanoni (1984), Yallin (1963, 1972) and Yang (1972, 1973). Yallin (1963, 1972) developed a bed load equation incorporating reasoning similar to Einstein (1942, 1950), but with a number of refinements and additions. Yang (1972, 1973) approached the total transport from the energy expenditure point of view and related the transport rate to stream power. Shen and Hung (1972) derived a regression equation based on laboratory data for the sand-sized particles. Using the same concept, Ackers and White (1973) defined sediment transport functions in terms of three dimensionless groups namely, size, mobility and transport rate of sediments. His functions are based on flume data carried out with flow depths up to 0.4 meters. One of the most extensive field and laboratory studies of sediment transport is that by Van Rijn (1984). He has presented a method which enables the computation of the bed load transport as the product of the saltation height, the particle velocity and the bed load concentration.

More recently, Hassanzadeh (2007) based on the dimensional analysis and the Buckingham Π-theorem in reasoning and discussion of bed load phenomenon has presented a dimensionless semi-empirical equation on the bed load. Comparisons have been made between this formula with common ones on sediment hydraulics after their unified descriptions. It is showed that, the latest formula agrees well with the measured data and could be regarded as optimal, compared with other common formula.

The theoretical equation for the distribution of suspended sediment in turbulent flow has been given by H. Rouse. Further useful information on the modification of the theory can be found in Einstein and Chien (1955), Vanoni (1984), Hassanzadeh (1985, 1979), and many others (Graf 1971, 1998 and Raudkivi 1976). A special study on the hyperconcentrated fluid mud in rivers is also reported by Mei et al. (1994).

Utilizing the data obtained from the Vanyar gauging station on Adji-chai River in Tabriz, Iran, and by means of regression analysis on the relationship between the suspended load and water discharge, Hassanzadeh (2007) has given two empirical equations with high regression coefficient to calculate the river's suspended load for the wet and dry seasons. This chapter presents a comprehensive analysis of the hydraulics of sediment transport. The effort is focused on those aspects of the study that will produce the best overall results within given constraint of time. This chapter briefly reviews hydraulics formulas for fluid flow in open channels and several fundamental sediment computations and contains the following subjects:

1. Sediment properties
2. Threshold of Particle Transport
3. Channel Roughness and Resistance to Flow
4. The Sediment Load

2. Sediment properties

The dynamic problems of liquid - solid interaction are greatly influenced by the sediment properties. The description of the latter, however, is exceedingly complex and one is forced to make many simplifying assumptions. The first of which is the subdivision into cohesive and non- cohesive sediments.

In cohesive sediments the resistance to erosion depends on the strength of the cohesive bond between the particles which may far outweigh the influence of the physical characteristics of individual particles. The problem of erosion resistance of cohesive soils is a very complex one and at present our understanding of the physics of it is very incomplete.

The non- cohesive soils generally consist of larger discrete particles than cohesive soils and the movement of these particles depends on the physical properties of the individual particles, such as size, shape and density.

2.1 Particle size, shape and density

Particle size. The most important physical property of a sediment particle is its size.

It has a direct effect on the mobility of the particle and can range from great boulders, which are rolled only by mountain torrents, to fine clays, which once stirred uptake days to settle.

The size of particles can be determined in a number of ways. The nominal diameter refers to the diameter of a sphere of same volume as the particle, usually measured by the displaced volume of a submerged particle. The sieve diameter is the minimum length of the square sieve opening through which a particle will fall. The fall diameter is the diameter of an equivalent sphere of specific gravity δ= 2.65 having the same terminal settling velocity in water at 24˚C.

Shape. Apart from size, shape affects the transport of sediment but there is no direct quantitative way to measure shape and its effects. McNown (1951) suggested a shape factor S.F. = $c/\sqrt{(ab)}$, where c is the shortest of the three perpendicular axes (a, b, c,) of the particle. The shape factor is always less than unity, and values of 0.7 are typical for naturally worn particles.

Density. Density of the particles is important and must be known. A large variation in density affect sediment transport by segregation, e.g. the armoring effect of the heavy minerals on dune crests. The mass density of a solid particle, ρ_s, describes the solid mass per unit volume. The particle specific weight, γ_S, corresponds to the solid weight per unit volume of solid. The specific weight of a solid, γ_S, also equals the product of the mass density of a solid particle, ρ_S, times the gravitational accelerating, thus:

$$\gamma_S = \rho_S \cdot g \tag{1-1}$$

Submerged specific weight of a particle, γ_S'. Owing to Archimedes' principle, the specific weight of a solid particle, γ_S, submerged in a fluid of specific weight, γ, equals the difference between the two specific weights, thus,

$$\gamma_S' = \gamma_S - \gamma = (\rho_S - \rho)g \tag{1-2}$$

Where ρ is the mass density of fluid.

The ration of the specific weight of a solid particle to the specific weight of a fluid at a standard reference temperature defines the specific gravity δ. With common reference to water at 4˚C, the specific gravity of quarts particles is

$$\delta = \frac{\gamma_S}{\gamma} = \frac{\rho_S}{\rho} = 2.65 \tag{1-3}$$

The specific gravity is a dimensionless ratio of specific weights, and thus its value remains independent of the system of units.

2.2 The fall velocity

The fall or settling velocity of a particle is assumed to be a steady- state motion. It is also a function of size, shape, density and viscosity of fluid.

In addition it depends on the extent of the fluid in which it falls, on the number of particles falling and on the level of turbulence intensity. Turbulent conditions occur when settling takes place in flowing fluid and can also occur when a cluster of particles is settling. For grain diameter d greater than 2mm, the fall velocity w can be approximated by the following equation:

$$w = 3.32\sqrt{\mathrm{d(mm)}} \tag{1-4}$$

Falling under the influence of gravity the particle will reach a constant velocity named the terminal velocity, when the drag equals the terminal velocity, i. e. difference of the solid and fluid velocities, V_s- V= w; we obtain the following equation:

$$C_D \frac{\pi d^2}{4}\frac{\rho w^2}{2} = \frac{\pi d^3}{6} g(\rho_S - \rho) \tag{1-5}$$

or

$$w^2 = \frac{4}{3}\frac{1}{C_D} gd(\frac{\rho_S - \rho}{\rho}) \tag{1-6}$$

Thus, the problem reduces to finding the value of the drag coefficient, C_D, for the particle in question. For spherical particles of diameter d in a viscous fluid of dynamic viscosity μ, the drag coefficient is fairly well defined. In laminar flow region, for Re < 0.5 and approximately for up to 1.0, where $\mathrm{Re} = wd / \nu$, we have the stokes' solution of

$$F_D = 3\pi\mu dw \text{, and } C_D = \frac{24}{\mathrm{Re}} \tag{1-7}$$

The Stokes' solution may be considered if either the viscosity of fluid is very large (heavy oil) or the particle is very small (dust particle). Stokes, in solving the general differential equation of Navier- Stokes, neglected the inertia terms completely. Oseen (1927) seems to have been the first who successfully included, at least partly, the inertia terms in his solution of the Navier- Stokes equation. The Oseen's solution in approximate form is

$$C_D = \frac{24}{\mathrm{Re}}(1 + \frac{3}{16}\mathrm{Re}) \tag{1-8}$$

Goldstein (1929) provides a more complete solution for Oseen approximation and gives the drag coefficient in the form of

$$C_D = \frac{24}{\mathrm{Re}}(1 + \frac{3}{16}\mathrm{Re} - \frac{19}{1280}\mathrm{Re}^2 + \frac{71}{20480}\mathrm{Re}^3 + ...) \tag{1-9}$$

For $\mathrm{Re} \le 2$.

The value of drag coefficient, C_D, depends strongly on the level of free stream turbulence, apart from turbulence caused by the particle itself. It also depends on whether or not the surface of the sphere is hydraulically smooth or rough.

Schiller and Naumann (1933) suggest a formula that gives good results for Re <800, or

$$C_D = \frac{24}{\mathrm{Re}}(1 + 0.150\,\mathrm{Re}^{0.687}) \tag{1-10}$$

Schiller and Naumann (1933) also multiplied Eq. (1-6) by $(d / \nu)^2$ and obtained

$$C_D\,\mathrm{Re}^2 = \frac{4}{3} g \frac{\rho_S - \rho}{\rho} \frac{d^3}{\nu^2} \tag{1-11}$$

Olson (1961) suggests that the drag coefficient can be well represented by the following equation, for Re <100,

$$C_D = \frac{24}{\mathrm{Re}}(1 + \frac{3}{16}\mathrm{Re})^{\frac{1}{2}} \tag{1-12}$$

2.3 Effect of viscosity

The effect of the viscosity of the fluid on the dray coefficients, fall velocities, etc., enters through the Reynolds number. However, when dealing with suspensions it may be necessary to consider the effective viscosity of the suspension rather than that of the fluid. For dilute suspensions of spheres, Einstein developed the following equation.

$$\frac{\mu_{susp}}{\mu} = 1 + k_e C \tag{1-13}$$

Where

μ_{susp}= Viscosity of the suspension
μ = Viscosity of the liquid medium
k_e= Einstein's viscosity constant
C = Volumetric concentration of the solid phase.
k_e = 2.5 for C < 2-3%.

The kinematic viscosity of a Newtonian mixture, ν_m, is obtained by dividing the dynamic viscosity of a Newtonian mixture, μ_m, by the mass density of the mixture, ρ_m, or

$$\nu_m = \mu_m / \rho_m \tag{1-14}$$

2.4 Colloids and flocculation

Suspensions of fine particles are known as colloidal system. These very small particles are made up of cohesive material. If many of the very small particles come together and form flocs, the effective weight of such an agglomerate would increase and sedimentation will occur. This entire process is frequently referred to as flocculation. This phenomena plays an important part in the formation of estuarine muds and deltas. These fine materials form soil-water complexes which have physical properties quite different from those of their elementary particles. The behavior of these materials is controlled by electro-chemical forces and most the clay-water phenomena are interpreted in terms of these electro-chemical forces.

3. Threshold of particle transport

The Motion of a fluid flowing across its bed tends to move the bed material downstream. A submerged grain on the surface is subjected to a weight force and the hydrodynamic forces. For analysis all forces are resolved into normal and tangential components. The tangential components maintain the forward motion. Below some critical hydraulic condition, the hydrodynamic forces will be so small that particles submerged weight will move very rarely or not at all. However, a slight increase in flow velocity above this hydraulic critical condition will initiate appreciable motion by some of the particles on the bed. This hydraulic critical condition is termed the condition of initiation of motion and is computed in terms of either mean flow velocity in the vertical or the critical bed shear stress (also known as the tractive force or the drag force).

3.1 Velocity criteria

In the study of hydraulics of alluvial channels the engineers are often interested in finding the quantity of water and the sediment load carried by the stream under given hydraulic conditions.

The condition of incipient motion for an assembly of solid particles is given in terms of the forces acting on the particle by the following relation

$$\tan\phi = \frac{F_t}{F_n} \tag{1-15}$$

Where F_t and F_n are the forces parallel and normal to the angle of repose ϕ. In this study F_t and F_n are resultants of the hydrodynamic drag F_D, the lift force F_L and the submerged weight.

Equation (1-15) for the condition of incipient motion under the action of these three forces becomes

$$\tan\phi = \frac{W\sin\alpha + F_D}{W\cos\alpha - F_L} \tag{1-16}$$

Where angle α is the inclination of the bed from the horizontal at which incipient sediment motion takes place. Fig. 1.1 shows the situation of these three named forces. The drag F_D and lift F_L forces may be expressed as

$$F_D = C_D K_1 d^2 \frac{\rho u_b^2}{2} \tag{1-17}$$

$$F_L = C_L K_2 d^2 \frac{\rho u_b^2}{2} \tag{1-18}$$

Where u_b = fluid velocity at the bottom of the channel

C_D, C_L= drag and lift coefficients, respectively
d= particle diameter
K_1, K_2 = particle shape factors
ρ = liquid density

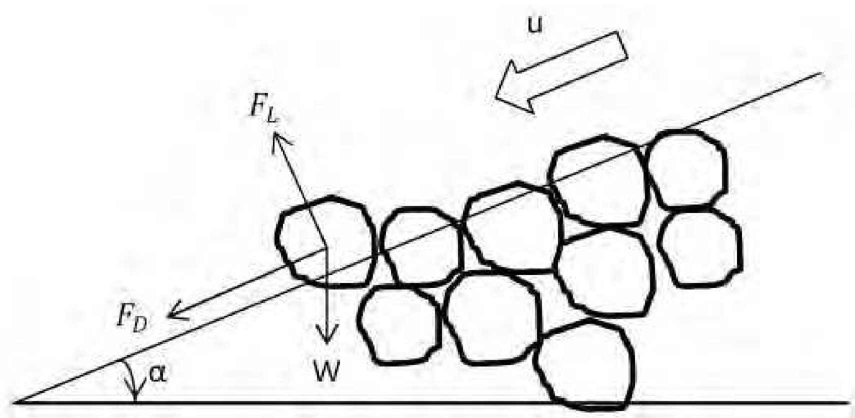

Fig. 1.1. Force diagram on particles in a cohesionless sediment

The submerged weight of the particle is given by

$$W = K_3(\rho_S - \rho)gd^3 \tag{1-19}$$

With K_3 being another shape factor and ρ_s being the solid particle density.

Introducing Eqs. (1-17), (1-18), and (1-19) into Eq. (1-16) yields

$$\frac{(u_b^2)_C}{(\rho_S/\rho - 1)gd} = \frac{2K_3(\tan\phi\cos\alpha - \sin\alpha)}{C_D K_1 + C_L K_2 \tan\phi} \tag{1-20}$$

Where $(u_b)_c$ is the critical bottom velocity at which incipient sediment motion takes place. The quantity of the right- hand side in Eq. (1-20) is referred to as the sediment coefficient A',

$$A' = \frac{2K_3(\tan\phi\cos\alpha - \sin\alpha)}{C_D K_1 + C_L K_2 \tan\phi} \tag{1-21}$$

The sediment coefficient A' depends on the particles properties, the dynamics of the flow, the channel slope, and the angle of repose. The angle of repose is the slope angle formed with the horizontal by granular material at the critical condition of incipient sliding. The angle of repose ϕ which is given by Lane (1953), in Figure 1.2 depends on particle size.

For more than two centuries the hydraulicians have attempted to formulate the conditions of incipient motion. One of the earliest relations is due to Brahms (1753). Brahms gave the critical velocity V_c of water as

$$V_c = kW^{1/6} \tag{1-22}$$

Where k is an empirical constant and W is the weight of the grain.

Fortier et al. (1926) report on a most extensive study on "permissible canal velocities", the maximum permissible value of the mean velocities.

Erosion, transportation, and deposition phenomenon have studied by Hjulström (1935) for uniform material on a loose bed (Fig. 1.3).

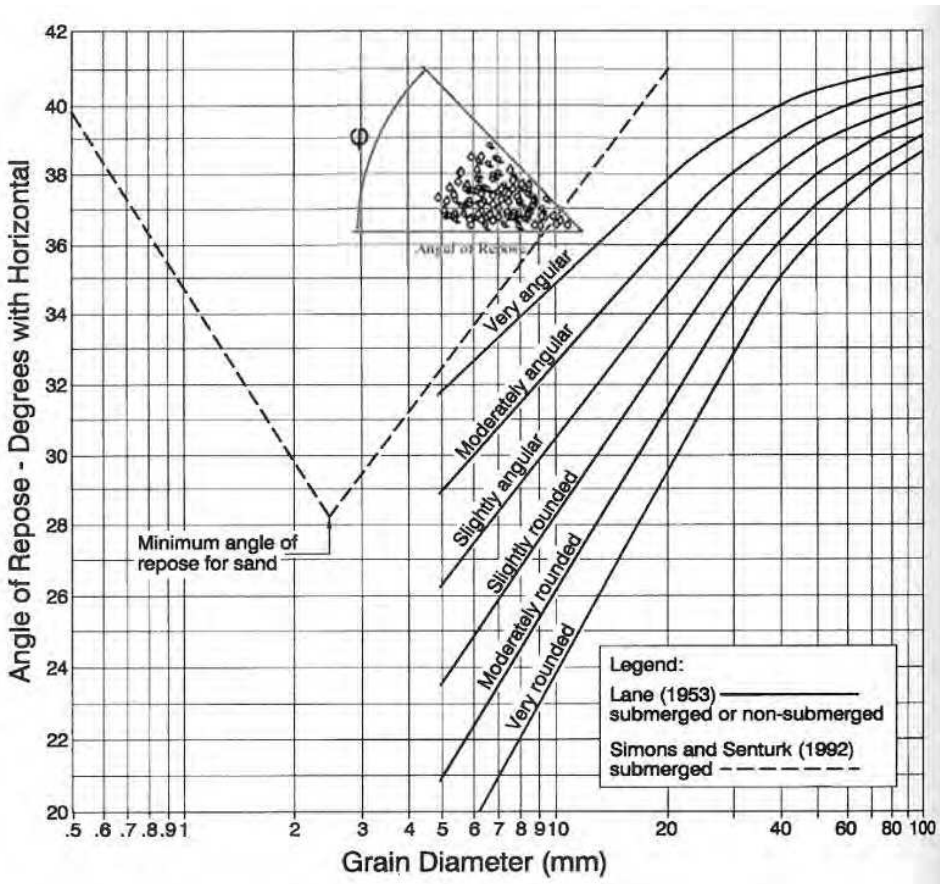

Fig. 1.2. Angle of repose for uniform non cohesive sediment (Lane, 1953)

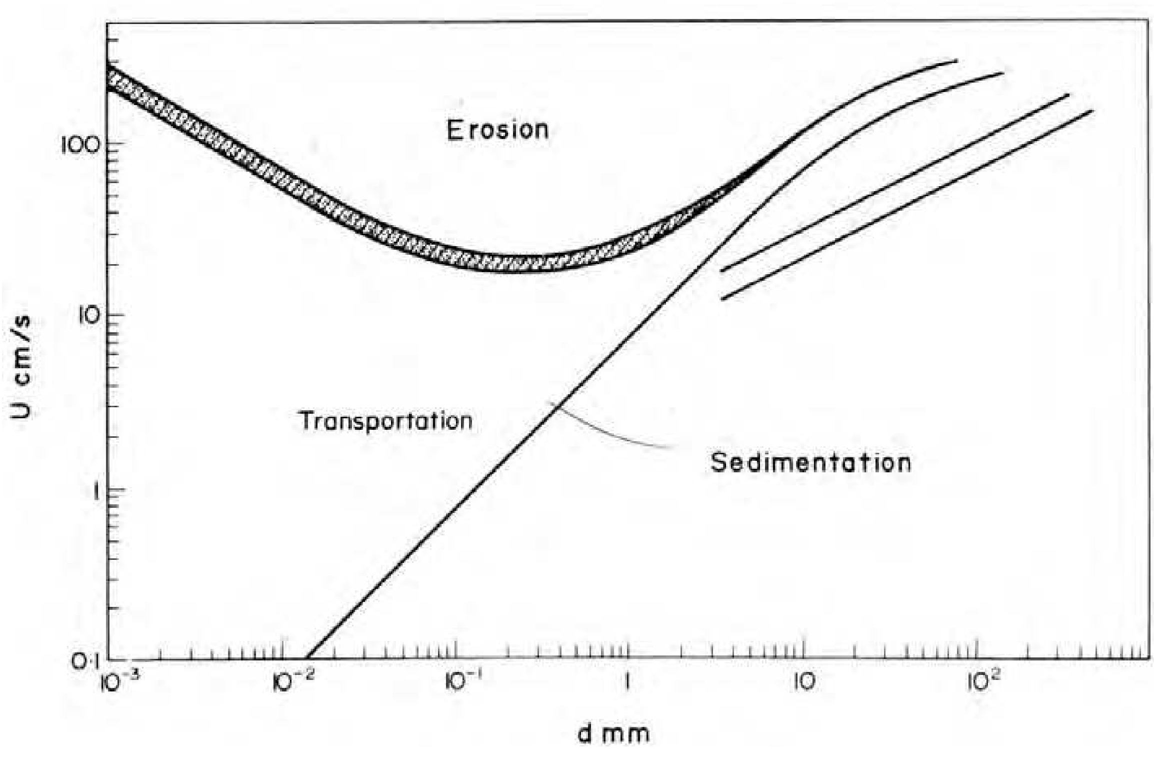

Fig. 1.3. Erosion and deposition criteria for uniform particles (Hjulström, 1935)

Neil (1967) gave the following relation for uniform material.

$$\frac{u_c^2}{(\rho_S/\rho - 1)gd} = 2.50(d/h)^{-0.20} \tag{1-23}$$

Where h is the depth of uniform flow.

Based on the incipient motion data, Carstens (1966) proposed an equation such as

$$\frac{u_c^2}{(\rho_S/\rho - 1)gd} = 3.61(\tan\phi\cos\alpha - \sin\alpha) \tag{1-24}$$

The latest is by Yang (1973, 1996) who used the conventional drag and lift concepts combined with the logarithmic velocity distribution and arrived at

$$\frac{V_c}{w} = \frac{2.5}{\log\frac{u_* d}{\nu} - 0.06} + 0.66 \quad ; \quad 0 < \frac{u_* d}{\nu} < 70 \tag{1-25}$$

Where the numerical constants are from empirical curve fitting. The above equation is valide for the hydraulically smooth and transition zones and for the hydraulically rough region the relationship is:

$$\frac{V_c}{w} = 2.05 \qquad , \qquad R_{e*} > 70 \tag{1-26}$$

It should be noted that, Eq.1.25 yields $V_c/w \approx \infty$ when $u_* d/\nu \approx 1.48$ .This would means that particles just a little finer than 100 μ_m will behave as a fixed boundary because $w \neq 0$ for this grain size. It is likely that the formula will give acceptable results for the shear velocity Reynolds number $R_{e*} > 1.5$ or 2.

In the turbulent range, for $R_{e*} > 70$, Yang's expression states that particles on a bed will begin moving when the average velocity is twice the particle settling velocity.

3.2 Bed shear stress or tractive force criteria

In a steady uniform flow, the component of the gravitational force exerted along the slope direction which causes downstream motion is balanced by the bed shear stress or tractive force per unit surface τ_0, which is the frictional force exerted on the moving fluid at its boundary. For the small slopes, the bed shear stress in a channel can computed as

$$\tau_0 = \gamma R_h S = \rho g R_h S \tag{1-27}$$

Where R_h= hydraulic radius, and S = bed slope. In a wide open channel, the hydraulic radius R_h is equal to the depth of flow h; hence

$$\tau_0 = \gamma h S = \rho g h S \tag{1-28}$$

The relationship between the friction velocity u_* and shearing stress τ_0 is given by

$$u_* = \sqrt{\tau_0 / \rho} = \sqrt{R_h g S} \tag{1-29}$$

The first research in the mechanics of sediment transport, using the foregoing concept, was reported by Shields (1936). Shields determined that the critical condition could be related to two dimensionless parameters: the dimensionless shear stress or the Shields parameter F_*, and the boundary Reynolds number or the shear Reynolds number $\mathrm{Re}_* = \frac{u_* d}{\nu}$. The Shields parameter F_* reflects the ratio of the force producing sediment motion to the force resistance motion and may be computed by:

$$F_* = \frac{\tau_0}{(\gamma_S - \gamma)d} \tag{1-30}$$

Use of the Shields diagram requires that the critical value of the Shields parameter F_{*c} be determined. To facilitate computations when grain size is known, the dimensionless diameter d_* may be computed by Jullien (1995) equation:

$$d_* = d\left[\frac{(\delta - 1)g}{\nu^2}\right]^{\frac{1}{3}} \tag{1-31}$$

Where d= sediment diameter, δ= specific gravity of sediment, and ν = kinematic viscosity.

The Shields diagram (Fig. 1.4) is a widely used method to determine the condition of incipient motion based on bed shear stress. Points lying above the curve representing the critical condition correspond to sediment motion, and points below the curve correspond to no motion. Three somehow distinct zones can be noticed in Fig. 1.4. It should be mentioned that, many hydraulic engineering problems deal with flow in the turbulent range and sediment having d >1 mm, the value of Shields parameter may be considered $F_{*c} = 0.047$ for critical condition in the range of boundary Reynolds number $\mathrm{Re}_* > 40$ (Yalin and Karahan, 1979). For this condition Eq. (1-28) is rearranged as

$$\tau_c = F_*(\gamma_S - \gamma)d = 0.047(\gamma_S - \gamma)d \tag{1-32}$$

In Shields diagram, a value for $\mathrm{R_{e^*}} \geq 400$ is obtained as

$$\frac{\tau_c}{(\gamma_S - \gamma)d} = 0.06 \tag{1-33}$$

Zeller (1963) finds this constant to be too high, and he obtained a value of 0.047.

The simplest method for estimating the critical condition for the movement of cohesionless sediment is using the linear relationship between critical bed shear stress τ_c and grain size d given by Julien (1995).

$$\tau_c(g/m^2) = 80 d_{50}(mm), \quad \text{for } \mathrm{d}_{50} > 0.30mm \tag{1-34}$$

A similar linear relationship is also suggested by Leliavsky (1955):

$$\tau_c(g/m^2) = 166 d_{50}(mm), \quad \text{for} \quad d_{50} \langle 3.4mm \qquad (1\text{-}35)$$

These equations are approximately valid for d_{50} > 0.3 mm, and can be used as a quick check against other methods and to help determine the hydraulic roughness regime.

Considerable field data are used by Lane (1953) to establish the critical tractive force diagram on function of grain size, as shown in Fig. 1.5. This diagram concern of allowable tractive force in channels as a function of grain size, for the range of fine to coarse noncohesive materials. As can be seen in Fig. 1.5, the critical shear stress τ_c for clear water is considerably lower than for water with a low or high content of sediments. The Lane diagram summarizes much of the important research done and, hence, should prove very helpful for the hydraulician engaged in stable channel design.

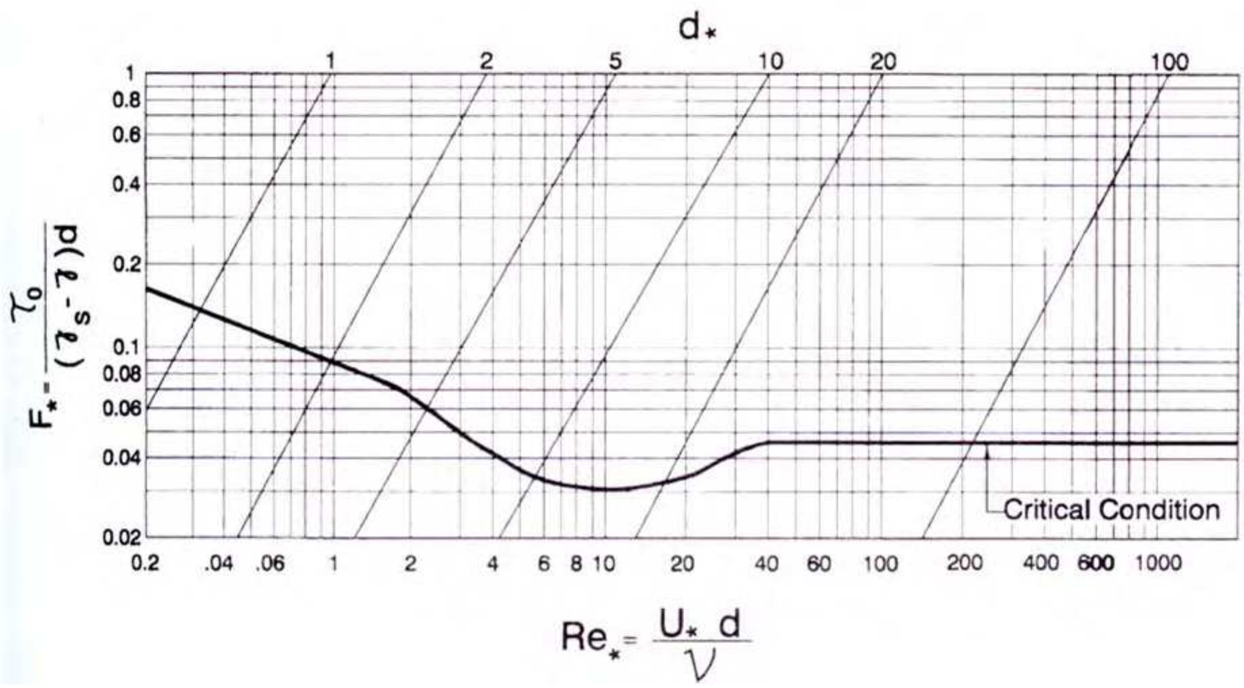

Fig. 1.4. Modified Shields diagram (Yalin and Karahan, 1979).

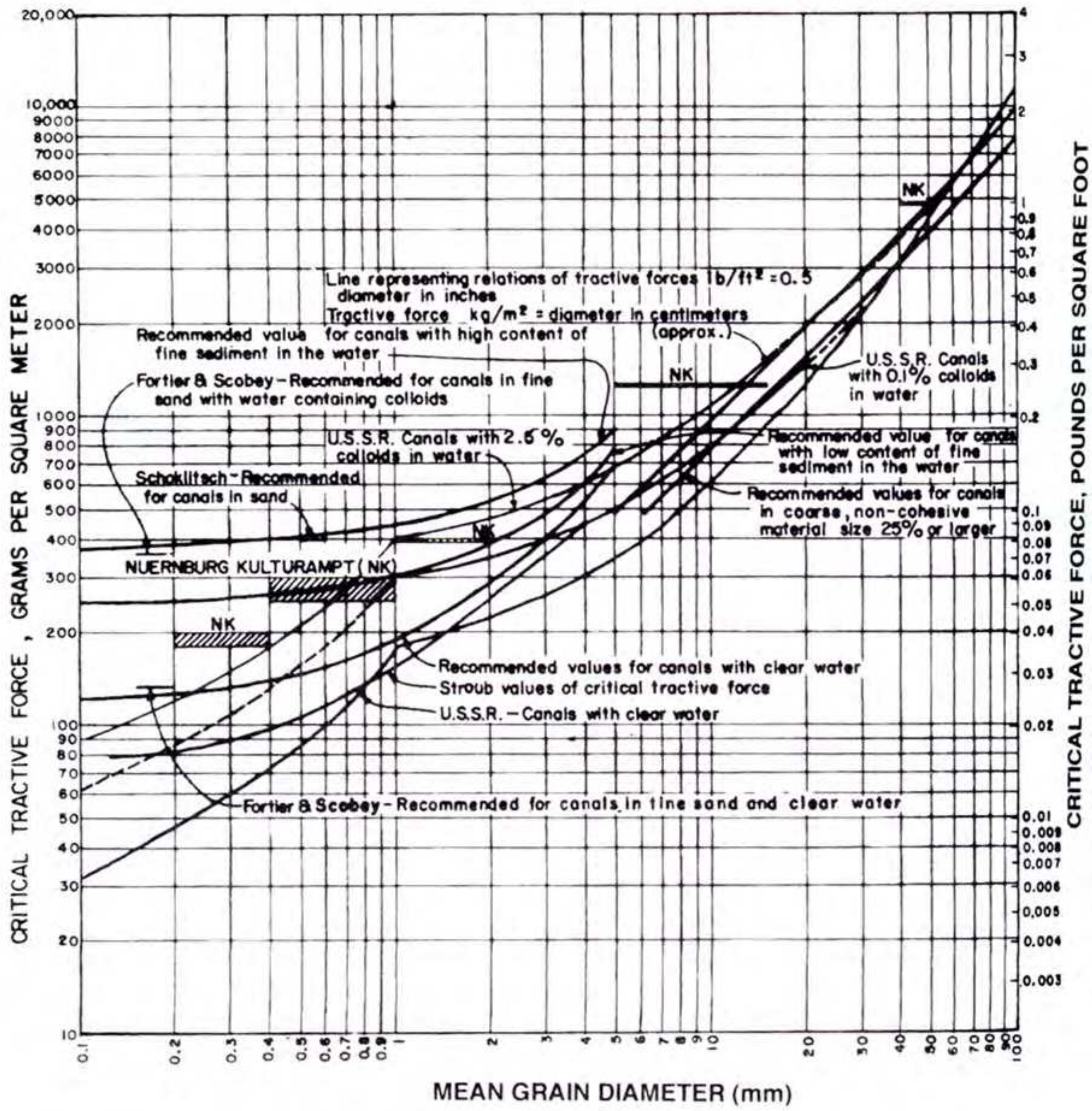

Fig. 1.5. Allowable tractive force in channels as a function of grain size (Lane, 1955).

It should be noted that the initiation of significant motion within a bed of mixed sediment can be affected by factors such as hiding of smaller grain by the larger ones and armoring. Graf (1971) indicated that for materials which are not uniformly sized or contain cohesive materials, the critical bed shear stress for incipient motion should be higher than predicted in the Shields diagram. Also, the shear stress is not distributed evenly accros a cross section. For a straight prismatic trapezoidal channel, Lane (1953) determined the shear stress distribution shown in Fig. 1.6 and concluded that in trapezoidal channels maximum shear stress for the bottom and sides is approximately equal to $0.97\,\gamma hS$ and $0.75\,\gamma hS$, respectively.

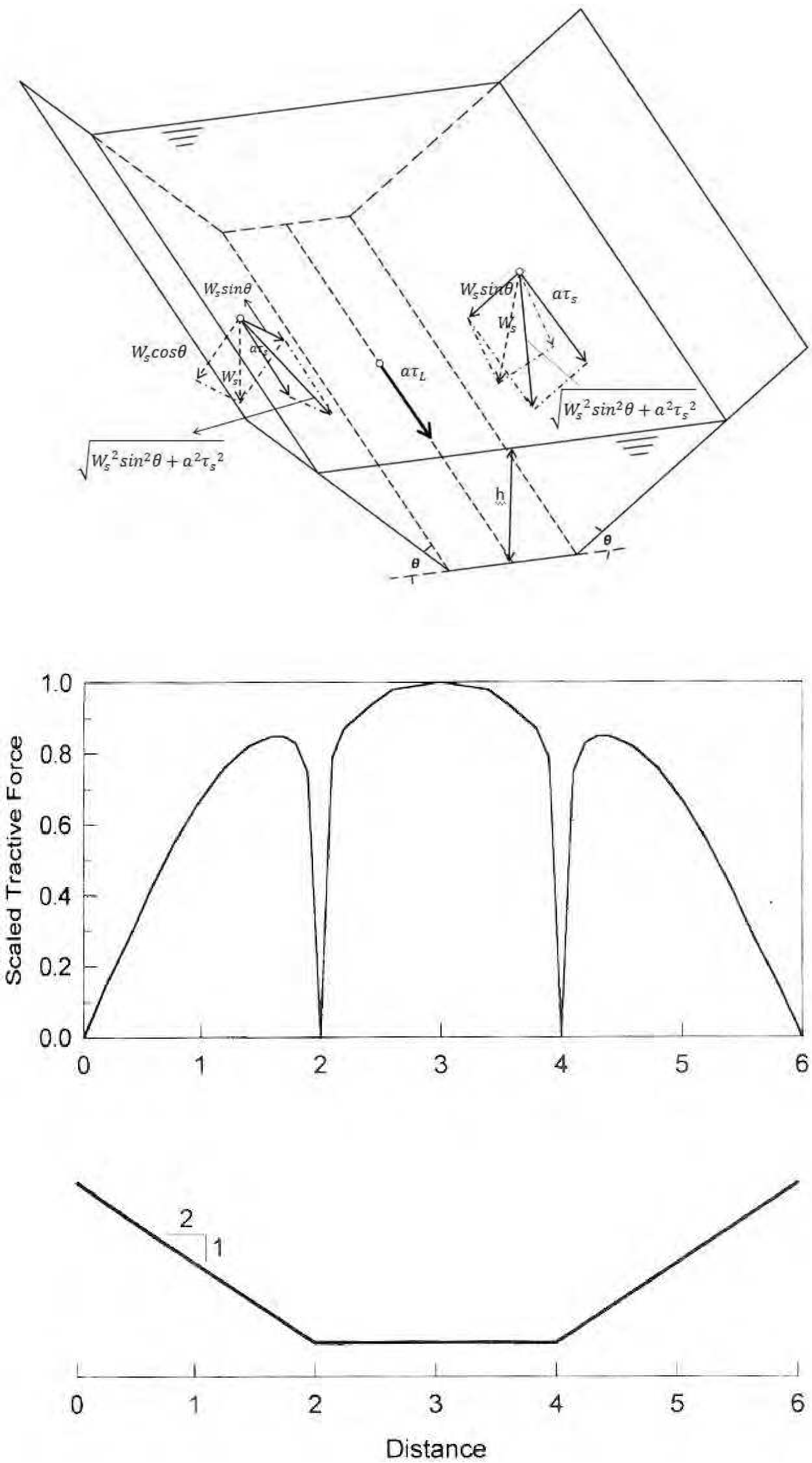

Fig. 1.6. Shear stress distribution in a trapezoidal channel section (Lane, 1955).

The boundary shear stress distribution for a curved trapezoidal channel was experimentally measured by Ippen and Drinker (1962) who found that the maximum shear stresses τ_{max}, occur at the outer toe of the bank immediately downstream of the curve. Shear stress in the curved reach will be 2 to 3 times greater than the shear in a straight channel. The ratio of maximum local boundary shear stress, τ_{max}, in a curved reach to the average boundary shear stress in a approach straight channel τ_0, is given by

$$\frac{\tau_{\max}}{\tau_0} = 2.65(\frac{r_c}{B})^{-0.5} \tag{1-36}$$

Where r_c = centerline radius of the bend, and B = water surface top width at the upstream end of the curved reach.

3.3 Shear stress ratio

It is remarkable that, on a channel bank, the gravity force that causes the particle to move down the sloping sides of the channels must be considered. On a soil particle resting on the sloping side of a channel section (Fig. 1.6) in which water is flowing, two forces are acting: the tractive or shear stress force $a\tau_s$, which try to move the sediment particle down the channel in the direction of flow, and the gravity- force component $W_s \sin\theta$, which tends to cause the particle to roll down the side slope. The resultant of these two forces, which are at right angles to each other, is

$$\sqrt{W_S^2 \sin^2\theta + a^2\tau_S^2}$$

where a = effective area of the particle, τ_s = unit tracrive force or shear stress on the side of the channel, W_s = submerged weight of the particle, and θ = angle of the side slope. When this force is large enough, the particle will move. On the other hand, the resistance to motion of the particle is equal to the normal force $W_s \cos\theta$ multiplied by the coefficient of friction, or $\tan\phi$, where ϕ is the angle of repose. Hence, by the principle of friction motion in mechanics, we have

$$W_S \cos\theta \tan\phi = \sqrt{W_S^2 \sin^2\theta + a^2\tau_S^2} \tag{1-37}$$

Solving for the unit tractive force τ_c that causes impending motion on a sloping surface,

$$\tau_S = \frac{W_S}{a}\cos\theta\tan\phi\sqrt{1-\frac{\tan^2\theta}{\tan^2\phi}} \tag{1-38}$$

Similarly, when motion of a particle on the level surface is impending owing to the tractive force $a\tau_L$, the following is obtained from Eq. (1-37) with θ = 0:

$$W_S \tan\phi = a\tau_L \tag{1-39}$$

Solving for the unit tractive force τ_L that causes impending motion on a level surface

$$\tau_L = \frac{W_S}{a}\tan\phi \tag{1-40}$$

The ratio of τ_s to τ_L is called the tractive or shear stress force ratio; which is an important ratio for design purposes. From Eqs. (1-38) and (1-40) the ratio is

$$K = \frac{\tau_S}{\tau_L} = \cos\theta\sqrt{1 - \frac{\tan^2\theta}{\tan^2\phi}} \tag{1-41}$$

Simplifying

$$K = \sqrt{1 - \frac{\sin^2\theta}{\sin^2\phi}} \tag{1-42}$$

It can be seen that this ratio is a function only the inclination of the sloping side θ and of the angle of repose of the material ϕ. The Eq. (1-41), is the form given by Lane (1953), has been suggested for the use in channel designs. Both of these expressions give the ratio of shear stress required to start motion on the slope to that required on the level surface of the same material.

By knowing the critical shear stress on the bottom, with the aid of Shields diagram the critical wall shear stress can be calculated, provided information on the angle of repose is available.

It is evident that, for a bank to be stable, the angle of the bank θ, must be less than of the angle of repose ϕ, or in other words, for stability reasons, $\phi > \theta$.

3.4 Annandale's erodibility index method

The erodibility index method has been developed recently by Annandale (1995) and may be used to compute the hydraulic conditions under which erosion will be initiated in a wide range of materials.

The erodibility index method is based on unit stream power, τV, the erosive power of water, according to the relationship:

$$\tau V = f(K_h) \tag{1-43}$$

Where τV = unit stream power (W/m^2), and $f(K_h)$ is the erodibility index. Erosion will occur when $\tau V > f(K_h)$, but not when $\tau V < f(K_h)$. The rate of energy dissipation per unit of bed area is determined by:

$$\tau V = \gamma h S V \tag{1-44}$$

Where γ = unit weight of water, h = depth, V = velocity and S = energy slope. For loose granular material, $0.1 < d_{50} < 100$ mm, the relationship between unit stream power at the critical erosion condition τV_c and the erodibility index K_h may be given by

$$\tau V_c = 480 K_h^{0.44} \tag{1-45}$$

Erosion occurs if unit stream power exceeds the value of τV_c. For cohesionless granular sediment, the erodibility index value may be related to grain diameter by:

$$K_h = 20d_{50}^3 \tan\phi \quad (1\text{-}46)$$

Where ϕ = angle of repose (Annandale, 1994).Fig. 1.7 shows the eroding and noneroding conditions based on the erodibility index method given by Annandale (1996).

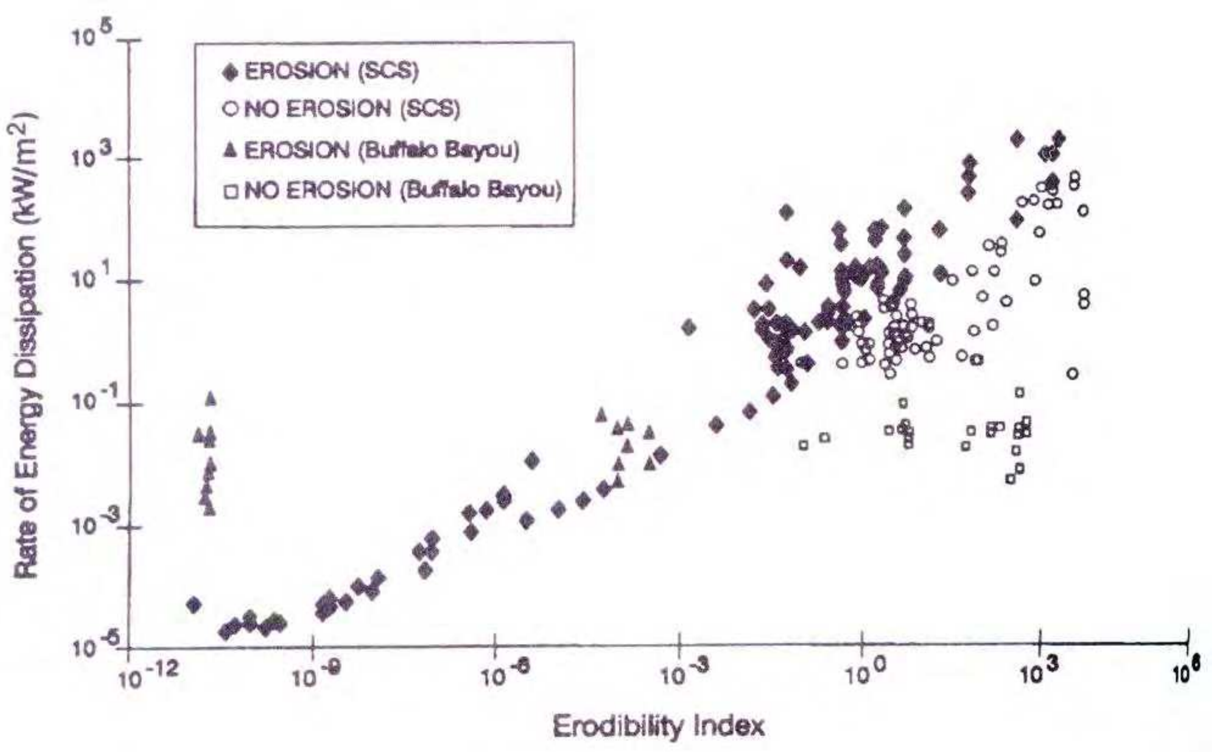

Fig. 1.7. Eroding and noneroding conditions based on the erodibility index method (Annandale, 1996).

4. Channel roughness and resistance to flow

The resistance to flow in a channel of fixed geometry carrying clear water in steady uniform flow can be predicted quiet accurately. But when the same channel carries clear water in non-uniform state of flow the resistance problem becomes very complicated. The shape of the channel in alluvium changes with flow conditions; bed features may form and the cross-section of the channel may become displaced laterally. These changes affect the drag caused by surface roughness and introduce form drag caused by the bed features, as well as energy losses due to secondary currents. The problem is further complex by the sediment motion along the bed and in suspension, since the mixture of water and sediment does not behave as clear water.

4.1 Flow resistance equations

4.1.1 Chézy equation

In 1775, the French engineer Chézy has given the relationship bearing his name, the first equation to successfully relate uniform open-channel flow to bed resistance. Chézy related the average velocity V of steady uniform open-channel flow to three parameters: channel slope S the hydraulic radius R_h, and a coefficient which expresses the boundary roughness. The Chézy equation is usually written in the form:

$$V = C\sqrt{R_h S} \tag{1-47}$$

in which C=Chézy coefficient of friction with the dimensional equation of $[C] = L^{1/2}T^{-1}$.

4.1.2 Manning equation

The equation developed in 1889 by the Irish engineer Robert Manning may be derived from the Chézy equation if the Chézy friction coefficient is set equal to $C = \frac{1}{n}R_h^{1/6}$. The Manning equation has the following form in SI units:

$$V = \frac{1}{n}R_h^{2/3}\sqrt{S} \tag{1-48}$$

In which n= Value in the Manning equation shows the roughness or flow-resistance characteristics of the boundary surface with the dimensional equation of $[n] = L^{-1/3}T$.

By multiplying both sides of the equation by the wetted cross-sectional area A, Manning's equation can be solved for discharge in SI units:

$$Q = \frac{1}{n}AR_h^{2/3}\sqrt{S} \tag{1-49}$$

This equation is widely used in open channel water flow computations.

4.1.3 Darcy-Weisbach equation

The nature of the boundary resistance in open channel flow is identical with that of full pipe flow and the Darcy-Weisbach and Colebrook-White equation for non-circular section may be applied. Noting that the energy gradient S_f is equal to the bed slope S in uniform flow:

$$V = \sqrt{\frac{8g}{\lambda}}.\sqrt{R_h S} = C\sqrt{R_h S} \tag{1-50}$$

or

$$V = \sqrt{\frac{8}{\lambda}}\quad u_* \tag{1-51}$$

With

$$C = \sqrt{8g/\lambda} \quad , \quad \text{and} \quad \lambda = 8g/C^2 \tag{1-52}$$

Unlike the friction coefficient in the Chézy and Manning equations, the Darcy-Weisbach friction factor λ is dimensionless, and can be read from Moody diagram.

4.2 Hydraulic flow-resistance factors

Hydraulic flow resistance has two components: grain roughness and form loss. Grain roughness is due to the tractive force created by sediment materials, vegetations, or other roughness elements at the flow boundary surface.

Form loss is the large-scale turbulent loss caused by the irregularity of the channel geometry along its length because of bed forms, expansions, constrictions, bends, and similar geometric features. Both types of flow resistance are important in natural channels.

A large information on n value determination for common engineering materials and natural channels may be found in chow (1959), Barnes (1967), and Arcement and Schneider (1989).

4.2.1 Grain roughness

The Strickler formula, developed in 1923, defined Manning's n value for grain roughness as a function of particle size:

$$n = \frac{d_{50}^{1/6}}{21.1} \tag{1-53}$$

Where d_{50}= median sediment particle diameter in meter (Morris and Fan, 1998).

Muler (1943) proposes to calculate the roughness coefficient K which is a result of the friction of the top layer of the grains,

$$K = \frac{26}{d_{90}^{1/6}} \quad (m^{1/3} / \sec) \tag{1-54}$$

Where d_{90} represents the size of the sediment in the bed for which 90 percent of the material is finer. This is certainly a reasonable diameter, since the top layer, being made up of the largest grains and is armoring the bed.

Specific procedures that can be used to determine Manning's n values for channels and flood plains, described by Arcement and Schneider (1989), are based on Cowan's (1956) method, research on channel roughness by Aldridge and Garrett (1973) and flood plain studies.

Cowan's procedure determines roughness values using a base n value which is then modified to incorporate additional factors which influence flow resistance.

4.2.2 Effect of vegetation on flow resistance

Vegetation presents a special problem in hydraulic computations because the boundary roughness changes greatly as a function of factors such as flow depth, velocity, and the biomechanical characteristics of the vegetation.

The sediment roughness value n can change remarkably during flow events as vegetation is inclined of flattened by the flow. Torres (1997) collected data from 452 experiments in grassed channels with flow depths from 0.018 to 0.81 m and vegetation height ranging from

0.05 to 0.9 m. The variation in Manning's n value with respect of channel mean velocity V, and submerged ratio (h/h_v), for grass channels is given by Torres (1997) in Fig 1.8. In which h and h_v represents water depth and the vegetation height, respectively.

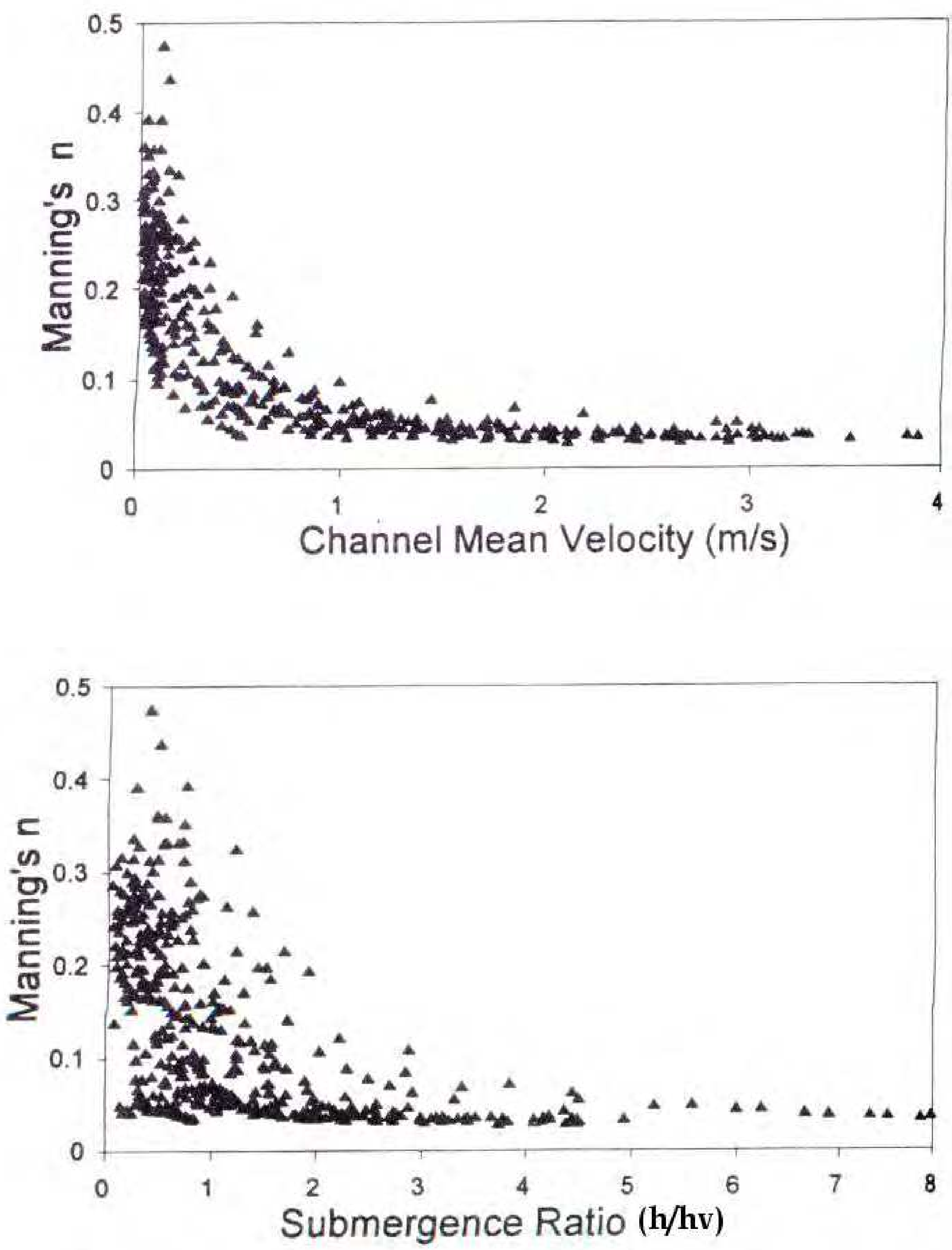

Fig. 1.8. Variation in Manning n value for grass channels (Torres, 1997)

4.3 Bedforms

When the sediment materials enter motion, the random patterns of erosion and sedimentation generate very small perturbations of the bed surface elevation. In many instances, these perturbations grow until various surface configurations known bed forms cover the entire bed surface. Resistance to flow, which depends largely on bed form configuration, directly affects water surface elevation in alluvial channel. As flow velocity increases, an initially flat sand bed develops first ripples and then dunes. With additional velocity, as illustrated in Fig.1.9, the stream subsequently transitions into a plain bed form, and finally forms antidunes with standing waves which may or may not crest and break. Transition from the lower flow regime, where bed forms dominant roughness, to the upper regime produces a dramatic drop in roughness and will produce a discontinuous discharge rating curve.

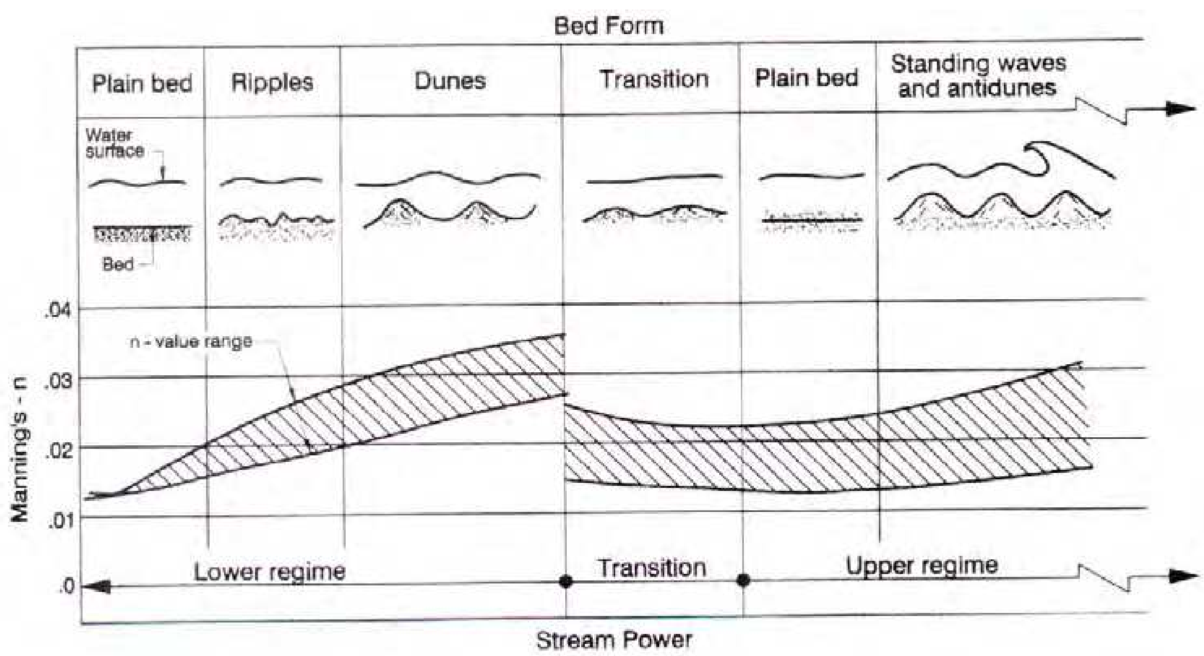

Fig. 1.9. Bed forms encountered in movable bed streams.

Various bed form configurations and geometry define the boundary roughness and resistance to flow in alluvial channels. The primary variables that affect bed form configuration and geometry are the slope of the energy grade line, flow depth, bed particle size, and particle fall velocity. Flat bed, or plain bed, refers to a bed surface without bed forms. Ripple shapes is small bed form, vary from nearly triangular to almost sinusoidal. Dunes are larger than ripples and are out of phase will the water surface waves.

Bedforms are classified into lower and upper flow regimes based upon their shape, resistance to flow, and mode of sediment transport (Simons and Richardson, 1963, 1966). A transition zone exists between the two flow regimes, where bedforms range from washed-out dunes to plain bed or standing waves. The relationship between bedform and stream power developed by Simons and Richardson (1966) shown in Fig. 1.10 can be used to determine the flow regime. Because of the non-uniform conditions in natural channels, different flow regimes and bed forms can coexist in different areas of the same channel.

The Simons and Richardson (1966) proposed predictor bed form encompassing both lower and upper regimes when plotting the stream power (τ_0V) as a function of particle diameter d. Their bedform predictor based on extensive laboratory experiments is quite reliable for shallow stream but deviates from observed bed forms in deep streams.

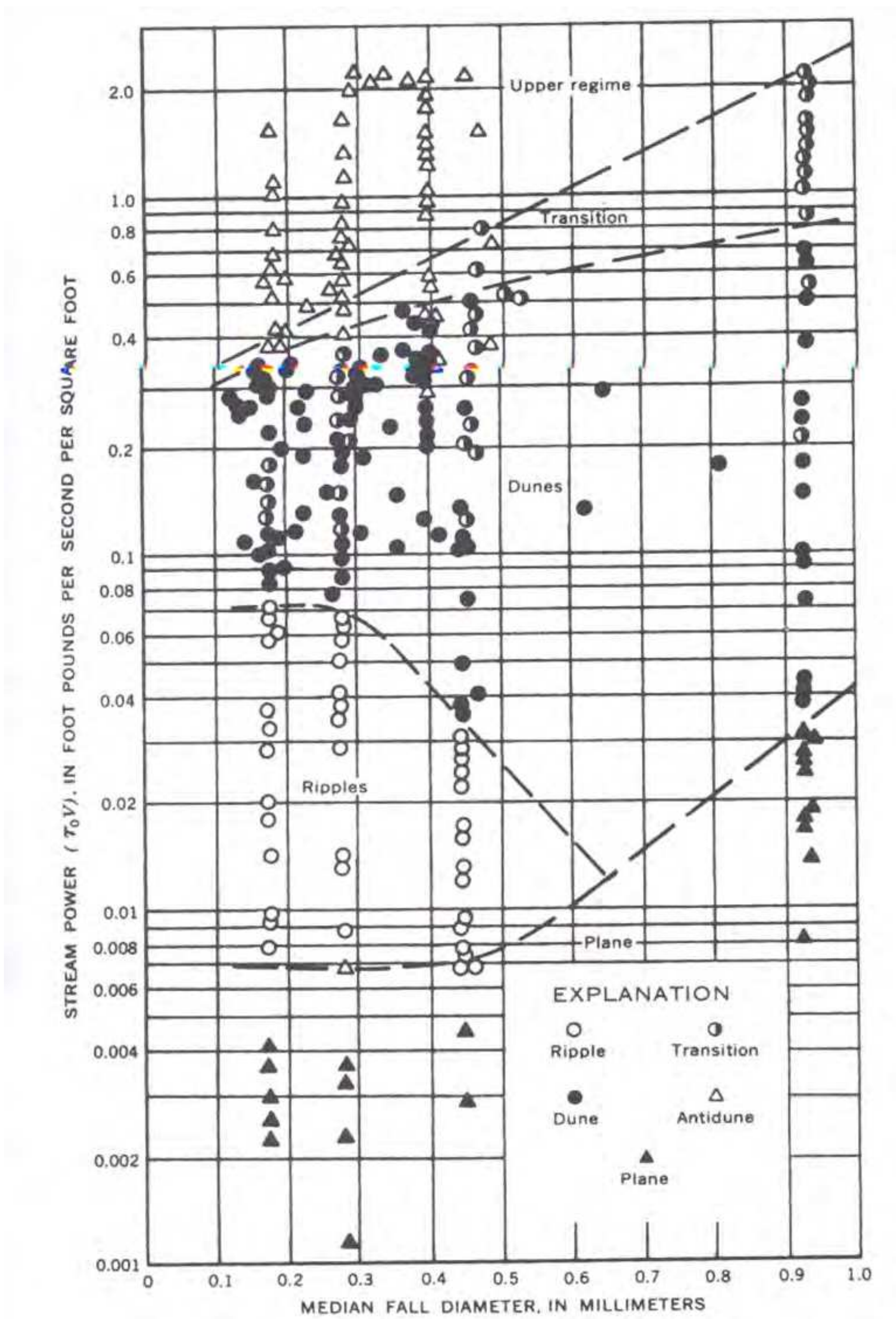

Fig. 1.10. Predictional bed form type based on stream power (Simons and Richardson, 1966)

Example 1. 1. Determine critical grain size for noncohesive sediment in a wide channel with the following characteristics:

T= 20°C $(\nu = 1\times10^{-6}\, m^2/s)$
h= 0.8 m ; V= 1.25 m/s ; n=0.03

$$\gamma_S = 2650 kg/m^3 = 25996\ \mathrm{N/m^3};\ \ \gamma = 1000 kg/m^3 = 9810\ \mathrm{N/m^3}$$

Solution. Use Manning's equation and assume R_h=h, since the channel is wide, and rearrange to compute: $V = S^{1/2}h^{2/3}/n$

$$S = \frac{V^2.n^2}{h^{4/3}} = \frac{(1.25)^2(0.03)^2}{0.8^{4/3}} = 0.0019$$

Solve for bed shear stress:

$$\tau_0 = \gamma h S = 9810\times0.8\times(0.0019) = 14.9 N/m^2 \cong 1520\, gr/m^2$$

Empirical Relationships:

Use Eq. 1.34 by letting $\tau_0=\tau_c$, compute d_{50}:

$$d_{50} = \tau_c/80 = 1520/80 = 19 mm$$

From the Fig. 1.5, a diameter of approximately 20 mm is obtained.

Using the known values, determine the flow regime.

$$\mathrm{Re}_* = \frac{u_* d}{\nu} = \frac{(gR_hS)^{1/2} d}{\nu} = \frac{(9.81\times0.8\times0.0019)^{1/2}\times0.02}{1\times10^{-6}} = 2442$$

The flow regime is strongly turbulent; i.e., $\mathrm{Re}_* \rangle\rangle 70$.

Yang's Method: In the turbulent range rearrange the Yang's Eq. 1.24 to obtain:

$$w = \frac{V_c}{2.05}$$

Let V=V_c ; solve for $w = 1.25/2.05 = 0.61 m/s$

Estimating that the particle diameter is greater than 2 mm, rearrange Eq. 1. 4 to solve for the particle diameter:

$$d = (\frac{w}{3.32})^2 = (\frac{0.61}{3.32})^2 = 0.034 m = 34 mm$$

It should be noted that, for particle in this grain size, viscosity effects are negligible.

Using Shields diagram and letting $\tau_0=\tau_c$, in the turbulent flow range, from Fig. 1.4, we have F_{*C}= 0.047. Rearrange Eq. 1.32 to solve for diameter:

$$d = \frac{\tau_c}{F_{*C}(\gamma_s - \gamma)} = \frac{14.9}{0.047(25966 - 9810)} = 0.019m = 19mm$$

Erodibility Index Method. Compute stream power:

$$\tau V = \gamma hSV = (9810)(0.8)(0.0019)(1.25) = 18.64 \quad W/m^2$$

Determine the critical K_h value from stream power by rearranging Eq. 1.45:

$$K_h = (\frac{\tau V}{480})^{2.27} = (\frac{18.64}{480})^{2.27} = 6.3 \times 10^{-4}$$

Determine the value of the critical grain diameter by letting tanϕ=0.8 and rearranging Eg.1.46 to solve for diameter:

$$d_{50} = (\frac{K_h}{20 \tan \phi})^{1/3} = (\frac{6.3 \times 10^{-4}}{20 \times 0.8})^{1/3} = 0.034m = 34mm$$

Example 1.2. Determine critical grain size for noncohesive sediment in a wide channel at 20°C $(\nu = 10^{-6} m^2 / s)$ with the following characteristics:

h=5 m ; V= 1.25 m/s ; n= 0.03

$$\gamma_S = 2650 \quad kg_f / m^3 \quad ; \quad \gamma = 1000 \quad \text{kg} / m^3$$

Solution. Using the Manning's equation to solve for S:

$$S = \frac{V^2.n^2}{R_h^{4/3}} = \frac{(1.25 \times 0.03)^2}{(5)^{4/3}} = 0.00016$$

Solve for bed shear stress:

$$\tau_0 = \gamma hS = (9810)(5)(0.00016) = 7.85 \quad \text{N/m}^2 = 0.8kg / m^2 = 800 \quad \text{g/m}^2$$

Solve as before using the Shields diagram:

$$d = \frac{\tau_c}{F_{*c}(\gamma_S - \gamma)} = \frac{7.85}{0.047(25996 - 9810)} = 0.01m = 10mm$$

A similar value is obtained from Fig 1.5, and using Eq. (1.32). For the erodibility index method, compute stream power:

$$\tau V = \gamma hSV = (9810)(5)(0.00016)(1.25) = 9.81 \quad \text{W/m}^2$$

$$k_h = (\frac{\tau V}{480})^{2.27} = (\frac{9.81}{480})^{2.27} = 1.46 \times 10^{-4}$$

$$d_{50} = (\frac{K_h}{20 \tan \phi})^{1/3} = (\frac{1.46 \times 10^{-4}}{20 \times 0.8})^{1/3} = 0.021m = 21mm$$

The above approaches all predict a reduction in the critical grain size as flow depth increases, with average velocity remaining unchanged. However, mean velocity approaches will predict a grain size identical to that in Example 1.1 because the mean velocity remain unchanged between the two examples.

5. The sediment load

The analysis of hydraulic sediment transport is usually seperated into two sediment parts: suspended load and bed load. The total load could be obtained by summing these two loads.

It should be noted that the fine fractions flowing into the river bed from its catchment area are usually transported in suspension and are defined as wash load. The latter is a very fine organic material transported by rivers over long distances and then deposited at the river mouth as a result of energy dissipation.

Suspended load is defined as that part of sediment load which is being remained in suspension for a considerable time by upward components of turbulent flows.

Bed load is defined as part of sediment within the bed layer moved by saltation (jumping), rolling, or sliding. The bed layer is a flow layer in several grain diameters thick immediately above the bed. Its thickness is usually taken as 2 times of grain diameters. Bed load transport occurs when the flow conditions exceed the criteria for incipient motion.

This subdivision does not rest on a physical basis and, in practice, it is at times very difficult to define where the suspended load starts and bed load stops (Vanoni 1984).

In the past, numerous bed load equations have been proposed, but some of them are very similar. There are essentially four slightly different approaches to the bed load problem. They are:

i. The Du Boys-type equations

Du boys (1879) introduced the tractive force or bed shear stress which was an entirely new concept. He expressed transport rate in terms of shear stress and the critical shear stress for initiation of sediment motion.

ii. The Schoklitsch-type equations

The bed load formula of A Schoklitsch (1926) is based on discharge relationship, and represents essentially the same form as the Du Boys formula.

iii. The Einstein-type equations

Introducing probability concepts of sediment movement and statistical considerations of the lift forces, Einstein (1942-1950) developed his empirical relationship. The contributions by Einstein (1942-1950) to the problem of bed load transportation represent also somehow a departure from the Du Boys-type and Schoklitsch-type equations.

iv. The Hassanzadeh-type equations

More recently, based on the dimensional analysis and the Buckingham Π – theorem, Hassanzadeh(2007) has presented a dimensionless semiempirical equation on the bed load.This latest dimensionless bed load equation has been given as a function of the hydrodynamic-immersed gravity force ratio.

Extensive discussions on this subject has been given by Vanoni (1984), Yallin (1963, 1972) and Yang (1972, 1973). Yallin (1963, 1972) developed a bed-load equation incorporating reasoning similar to Einstein but with a number of refinements and additions. Yang (1972, 1973) approached the total transport from the energy expenditure point of view and related the transport rate to stream power. Shen and Hung(1972) derived a regression equation based on laboratory data for the sand-sized particles. Using the same concept, Ackers and White(1973) defined his sediment transport functions in terms of three dimensionless groups namely, size, mobility and transport rate of sediments. His functions are based on flume data carried out with uniform or near uniform sediments with flow depths up to 0.4 meters. One of the most extensive field and laboratory studies of sediment transport is that by Van Rijn(1984). He has presented a method which enables the computation of the bed-load transport as the product of the saltation height, the particle velocity and the bed-load concentration.

5.1 Bed load

Bedload particles roll, slide, or saltate along the bed. The sediment transport thus occurs tangential to the bed. For determiniation of bed load several empirical equations from laboratory flume data have been given by many investigators with the basic assumptions that the sediment is homogeneous and noncohesive. The results differ appreciably but it is not recommended in practice to transfer the information to outside the limits of the experiments. However, one can discern general trends of the sediment transport rate by using several formulae, with some theoretical background.

The dimensional analysis and the Buckingham Π-theorem in reasoning and discussion of bed load phenomenon has been used here.

Mathematically, the physical problem of bed load per unit width q_s in turbulent free surface flow depends upon bed shear stress τ_0 , sediment diameter d, gravity acceleration g, slope S, immersed specific weight of solid γ'_s and specific weight of fluid γ. It would be written then:

$$F(q_s, \tau_0 , d , \gamma'_s , g , \gamma , S) = 0 . \qquad (1\text{-}55)$$

In the Eq.1.55, $\tau_0 = \gamma hS$ and $\gamma'_s = \gamma_s - \gamma = (\rho_s - \rho)\, g$

are used to replace respectively depth of flow h and specific weight of solids γ_s.

Chosen γ'_s , d and g as the repeating variables and using the Buckingham Π-theorem procedure, we obtaine the following expression:

$$\psi\left(\frac{q_s}{\sqrt{gd^3}}, \frac{\tau_0}{\gamma'_s d}, \frac{\gamma}{\gamma'_s}, S\right) = 0. \qquad (1\text{-}56)$$

So, the Π-independent, dimensionless and significant terms are as follows:

$$\Pi_1 = \frac{q_s}{\sqrt{gd^3}} \quad ; \quad \Pi_2 = \frac{\tau_0}{(\gamma_s - \gamma)d} = \frac{U_*^2}{(\frac{\gamma_s}{\gamma} - 1)gd}$$

$$\Pi_3 = \frac{\gamma}{\gamma'_s} = \frac{\gamma}{(\gamma_s - \gamma)} = \frac{\rho}{\rho_s - \rho} \; ; \; \Pi_4 = S$$

where ρ_s and ρ represent density of solids and fluid respectively, and $u_* = \sqrt{\tau_0/\rho}$ is shear velocity. Using the Buckingham Π-theorem procedure, the bed load equation may be expressed as:

$$\frac{q_s}{\sqrt{agd^3}} = \phi(\frac{\tau_0}{\gamma_s - \gamma)d}) \ , \qquad (1\text{-}57)$$

where $a = (\gamma_s - \gamma)/\gamma = \gamma_s'/\gamma$ shows the immersed sediment specific gravity. The hydrodynamic-immersed gravity force ratio is obtained from:

$$f = \frac{\tau_0}{(\gamma_s - \gamma)d} = \frac{U_*^2}{(\frac{\gamma_s}{\gamma} - 1)gd} \ . \qquad (1\text{-}58)$$

Based on the properties of the Buckingham Π-theorem and neglecting the mild slope S, $q_s/\sqrt{agd^3}$ may be obtained as follows:

$$\frac{\Pi_1}{\sqrt{\Pi_3}} = \frac{q_s}{\sqrt{gd^3}} \cdot \frac{1}{\sqrt{\gamma_s'/\gamma}} = \frac{q_s}{\sqrt{agd^3}} \ .$$

The variation of dimensionless sediment discharge in unit width, $q_s/\sqrt{agd^3}$ with respect of $f = \tau_0/(\gamma_s - \gamma)d$ has been presented in Figure 1.11, and compared with field measured data obtained from Vanyar guaging station on Adji - chai river (with a = 1.65 , d = 2.5 – 10 mm, S = 1.1 *10^{-3} and width of B=29.9-39.35m).

The Hassanzadeh (2007) bedload equation which agrees closely with the measured data has been expressed as follows:

$$\frac{q_s}{\sqrt{agd^3}} = 24 f^{2.5} \ . \qquad (1\text{-}59)$$

Comparison have been made between the last proposed equation (1.59) with common ones on sediment hydraulics after their unified descriptions.

For comaprison reason the common formulae on sediment hydraulics after their unified description are given in Table 1.1 and Fig.1.11.

As Figure 1.11 and Table 1.1 show, the Hassanzadeh (2007) equation (1.59) agrees well with the measured data and could be considered as an optimum one compared with the formulae given by the others (Graf, 1971, Julien, 1995 and Larras, 1972).

Example 1.3. Determine the rate of bed load transport in a rectangular cross section river with the following hydraulic characteristics:

Average flow depth h= 5.87 m
River bed slope S= 6.5 × 10^{-4}

Width B= 46.52 m
Chézy coefficient C= 56.
Median size of bed materials d_{50}= 0.012 m =12 mm

$$\gamma_S = 2650 kg / m^3, \qquad \gamma = 1000 \mathrm{kg/m^3}.$$

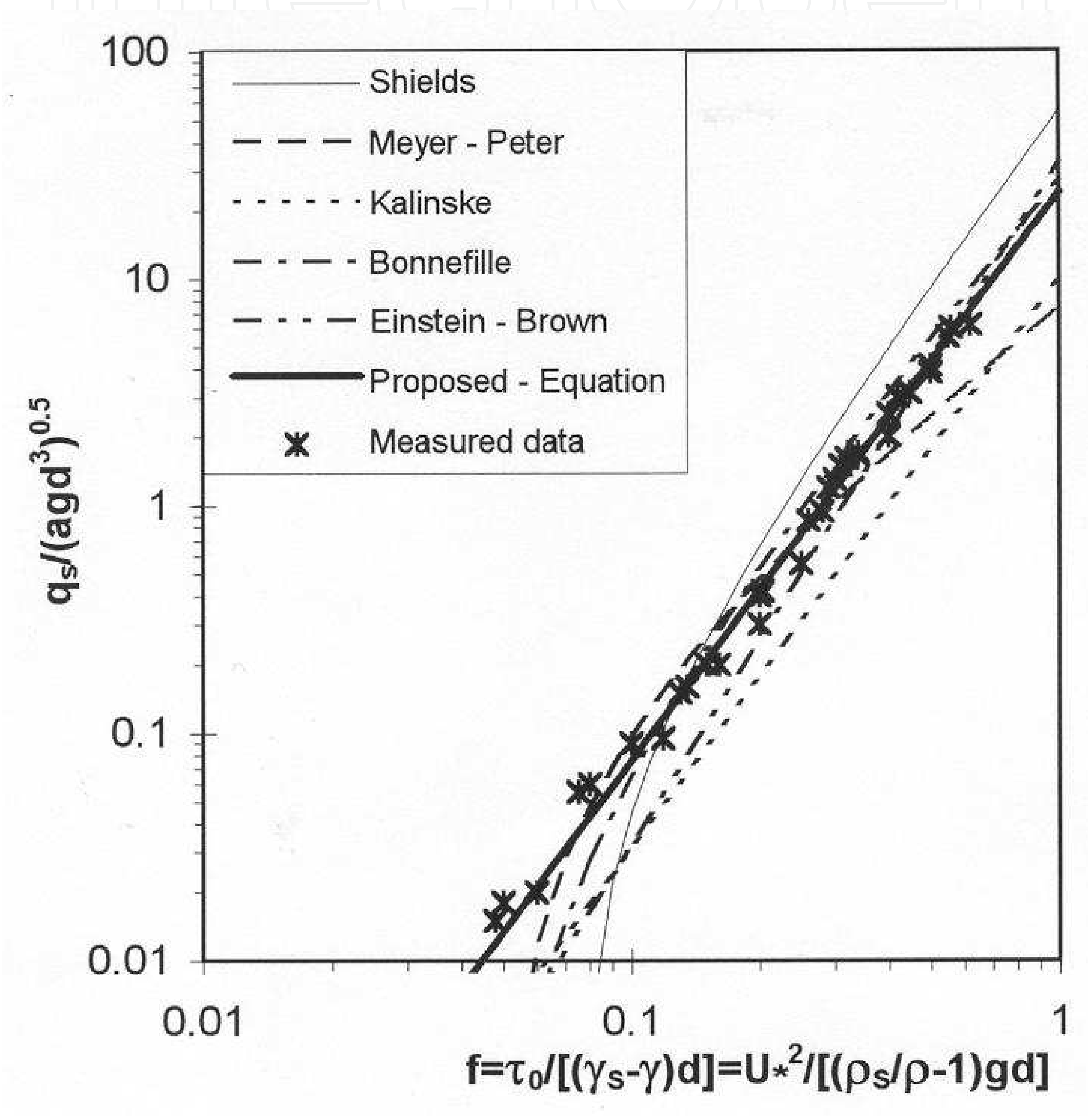

Fig. 1.11. Comparison of dimensionless bed load formulas (Hassanzadeh, 2007)

Solution: Use the Hassanzadeh-type equations. Assume R_h=h, since the river is wide. Determine the values of: f, a, $\sqrt{agd^3}$ and C_* as bellows respectively:

$$f = \frac{\tau_0}{(\gamma_S - \gamma)} = \frac{\gamma h S}{(\gamma_S - \gamma) d} = \frac{(1000)(5.87)(6.5\times 10^{-4})}{(2650-1000)(0.012)} = 0.193$$

$$a = \frac{\gamma_S - \gamma}{\gamma} = \frac{2650-1000}{1000} = 1.65$$

$$\sqrt{agd^3} = \sqrt{1.65(9.81)(0.012)^3} = 5.29\times 10^{-3}\, m^2/s$$

$$C_* = \frac{C}{\sqrt{g}}.\frac{\gamma}{\gamma_S} = \frac{56}{\sqrt{9.81}}.\frac{1000}{2650} = 6.75$$

Using the Hassanzadeh (2007) type of common dimensionless formulas on the hydraulics of sediment transport, the rate of bed load has been calculated and given in Table 1.1.

Author	Formula	$\frac{q_S}{\sqrt{agd^3}}$	$Q_S(kg/s) = \gamma_s B q_S$
Meyer-Peter (1934)	$\frac{q_S}{\sqrt{agd^3}} = 8(f-0.047)^{1.5}$	0.446	291.05
Shields (1936)	$\frac{q_S}{\sqrt{agd^3}} = 10C_*(f-0.076)f^{1.5}$ $C_* = \frac{C}{\sqrt{g}}.\frac{\gamma}{\gamma_S}$	0.67	436.68
Einstein-Brown (1942)	$\frac{q_S}{\sqrt{agd^3}} = 23.6f^3$	0.17	110.64
Kalinske (1947)	$\frac{q_S}{\sqrt{agd^3}} = 10f^{2.5}$	0.164	106.72
Bonnefille (1963)	$\frac{q_S}{\sqrt{agd^3}} = 5.5f^{1.5}(4.26f^{0.5}-1)^{1.25}$	0.393	256.08
Hassanzadeh (2007)	$\frac{q_S}{\sqrt{agd^3}} = 24f^{2.5}$	0.393	256.12

Table 1.1. Dimensionless existing formulas to estimate bed load (Hassanzadeh, 2007)

5.2 Suspended load

The mechanics of the process of suspension of particles more dense than liquid is still inadequately explained. Engineers and scientists had long been interested in the phenomenon of sediment suspension in water flows. The theoretical equation for the vertical distribution of suspended sediment in turbulent flow has been given by H. Rouse. Vertical distribution of sediment concentration under condition of steady state uniform flow, the tendency for sediment to settle under the influence of gravity, is offset by the vertical component of turbulence in the water column, a process called as turbulent diffusion. The upward-moving water comes from the deeper zone with higher concentration, partially of setting the settling of the sediment and creating a vertical concentration gradient which can be described using the Rouse (1937) equation:

$$\frac{C}{C_a} = (\frac{h-y}{y} \times \frac{a}{h-a})^Z \ , \tag{1-60}$$

$$Z = \frac{w}{ku_*} \qquad \text{or} \qquad Z = \frac{\mathrm{w}}{\beta \mathrm{ku}_*} \tag{1-61}$$

Where C and C_a= concentration of sediment having fall velocity w at vertical distances y and a above the bed and h=total depth, k=0.4 is von Karman's constant. β is of the order of unity for fine sediment and appears to decrease with increasing particle size. If β=1 and K=0.4 the ratio $w/u_* = 1$ corresponds to Z=0.4. Eq.1.60 represents the state of equilibrium between the upward rate of sediment motion due to turbulent diffusion and the downward volumetric rate of sediment transfer per unit area due to gravity. This equation can be used to determine the concentration C at any height y above the bottom relative to the known concentration Ca at height a above the bed. The value of Z will decrease as fall velocity w decreases; producing a vertical distribution will exist for each size class. Vertical concentration distributions for several values of Z are illustrated in Fig 1.12, assigning a concentration distribution C_a=1 at height a=0.05 h. For given shear stress, Z is proportional to w, which means that fine-grained material has small values of Z and the particles are fairly uniformly distributed throughout the depth, whereas coarse grains will be near the bed.

Equation (1.60) has been found that to give a better fit to the observed distribution β must be taken to 0.6 (Hassanzadeh, 1985 and 1979).

To formulate river sediments, data from Vanyar guaging station on Adji-chai river, near Tabriz city, were collected for 8 years. All the suspended sediment data were collected by the surface water departement of Tabriz water organization. The collected data consist of the instantaneous water discharges and the corresponding sediment load.

Through the use of the instantaneous values of suspended sediment load, Q_s , and water discharge, Q, regression equations for dry and wet seasons were developed. Fig.1.13 represents the average monthly water discharge Q_m (m^3/s), and suspended load, Q_{sm} (tons/day) with respect of time in months from October 1.

The inspection of this figure shows that reasonable correlations do exist between the average monthly water discharge Q_m and suspended load Q_{sm} for a period of 8 years, from october

1974 through september 1982 water years. It is obvious from this figure that at some time during the year a good correlation between the average monthly water discharge Q_m and the sediment load Q_{sm} exists . However, for 6 months, the sediment load was quite low even though the water discharge remained fairly high. Thus an exact correlation between water discharge and sediment load did not exist throughout the whole water year. It should be stated that, in Fig.1.13 the values of monthly and annual water discharge Q_m (m^3/s) and the corresponding suspended sediment load Q_{sm} (tons/day) are related respectively to the left and right axis.

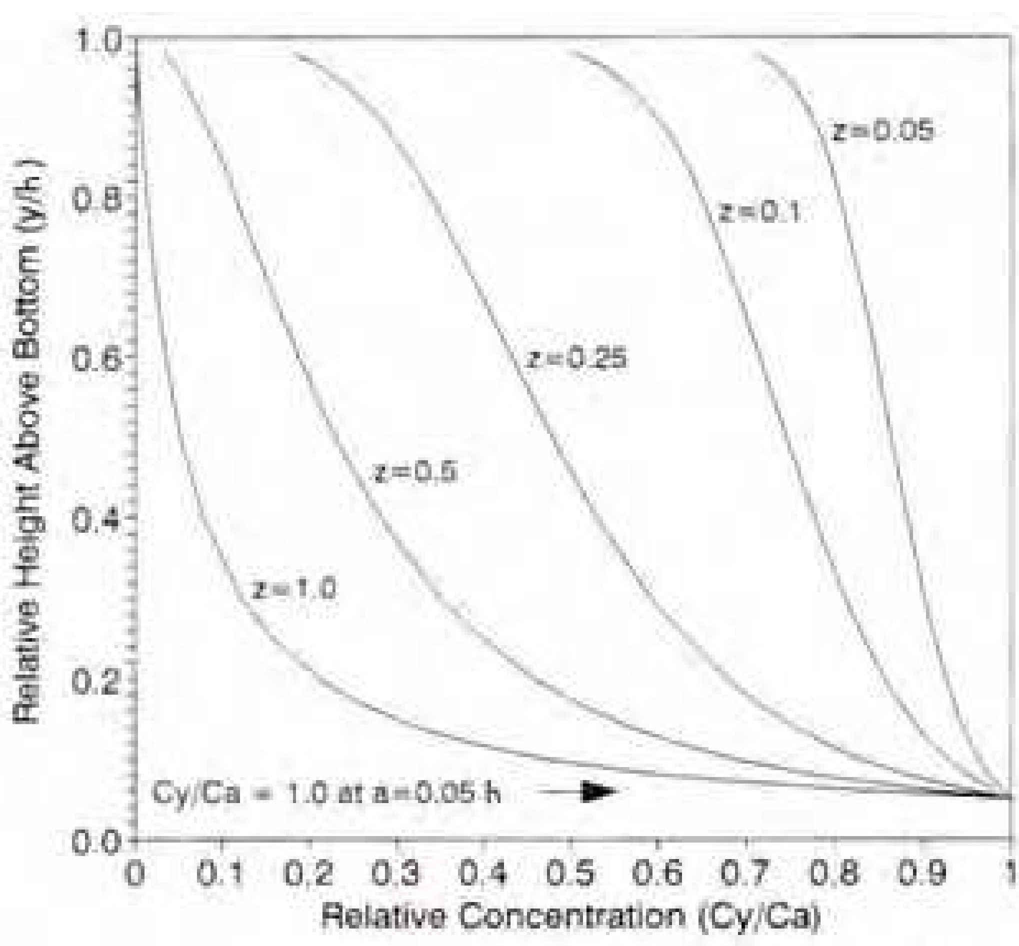

Fig. 1.12. Distribution of suspended sediment in a flow according to Eq.1.60

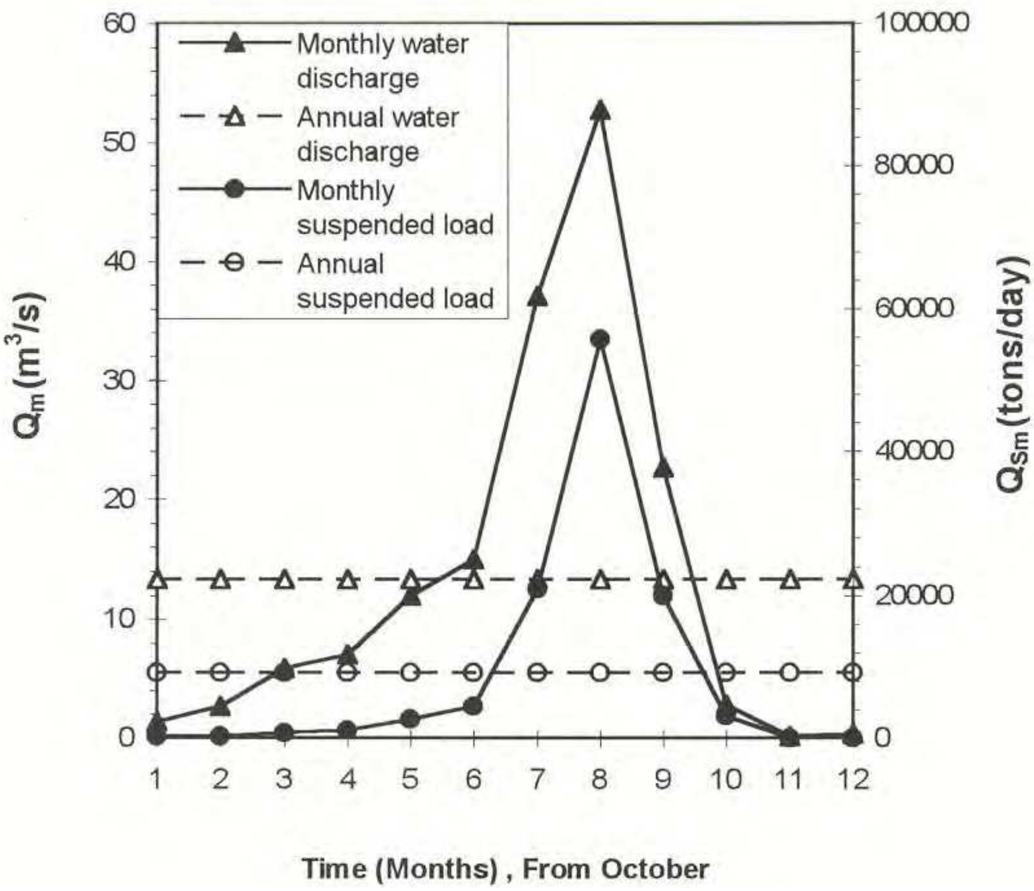

Fig. 1.13. Relationship between Q_{sm} and Q_m with time in months for a period of 1974-1982 (Hasaanzadeh, 2007)

Fig.1.14 shows the relationship between the percentage of the cumulated annual suspended load moving past the Vanyar guaging station in a given number of days for the four water years. An examination of Fig.1.14 will reveal that the bulk of the sediment load has been moved during storm events. Since the number of storm events in a water year is small and the duration of the storm events are generally short, the bulk of the suspended load passes through station during the relatively small number of days in a water year.

Fig.1.15 shows the variation of daily suspended load Q_s (tons/day) with the corresponding water discharge Q (m^3/s) for the dry and wet seasons of the 1974-1982 water years, for the Vanyar guaging station on the Adji-chai river. Throught the use of these instantaneous values of suspended load, Q_s , and water discharge, Q, regression equations for the wet and dry seasons were developed.

The resulted equations are as follows:

For wet seasons ($Q>15m^3/s$):

$$Q_s \text{ (tons/day)} = 16.67Q^{1.91} \tag{1-62}$$

For dry seasons ($Q\leq 15m^3/s$):

$$Q_s \text{ (tons/day)} = 47.48Q^{1.42} \tag{1-63}$$

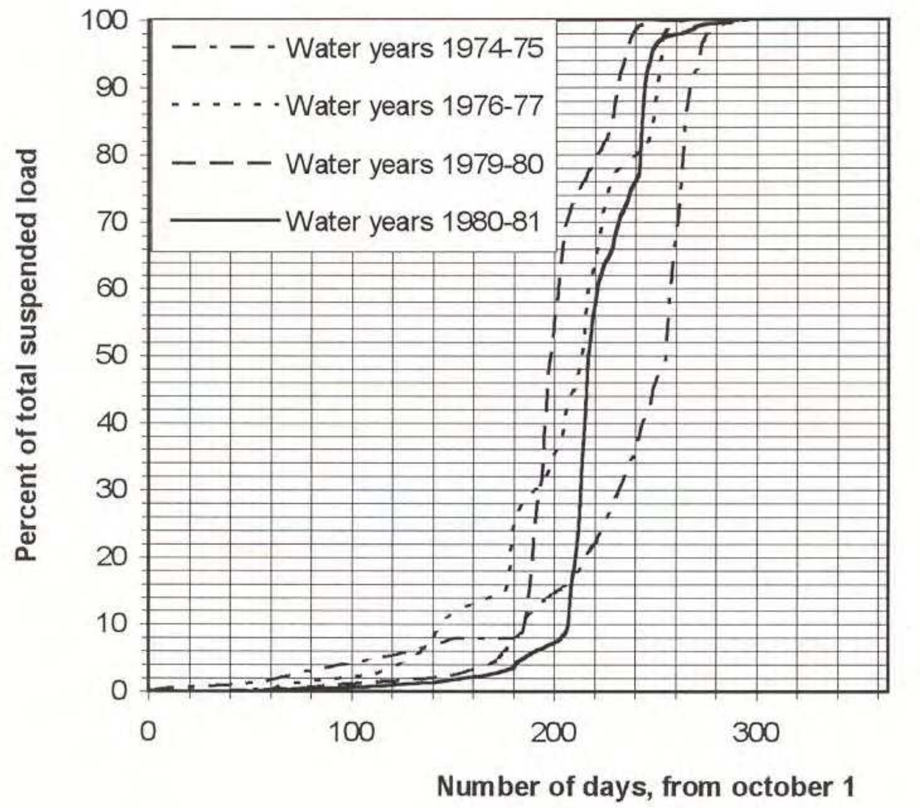

Fig. 1.14. Annual suspended load carried in a given number of days (Hasaanzadeh, 2007)

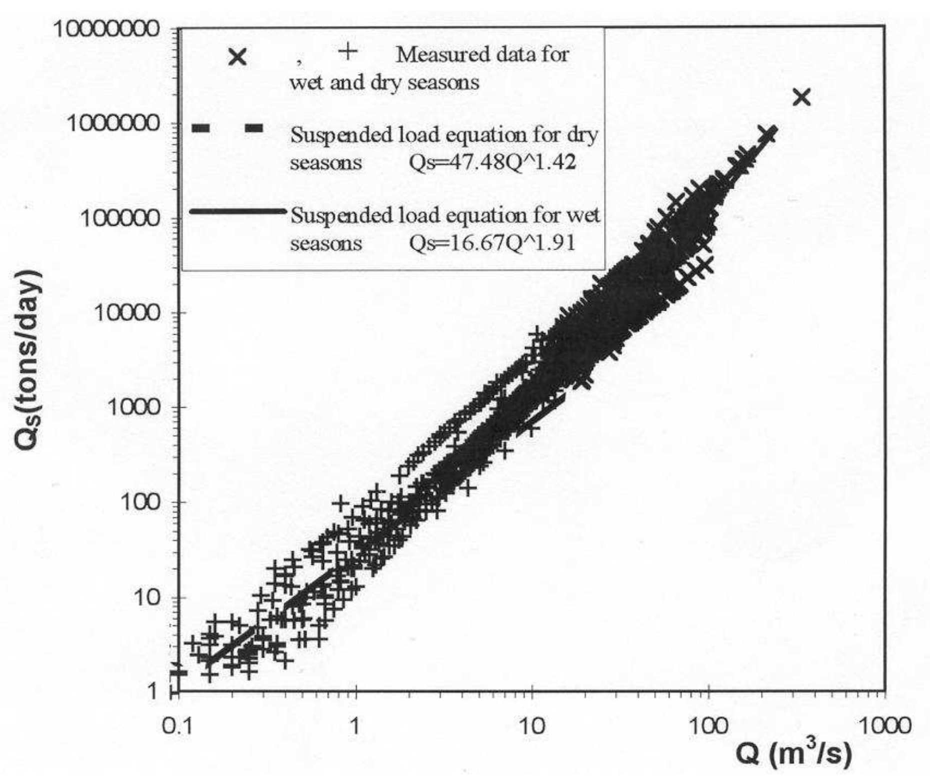

Fig. 1.15. Variation of suspended load Q_s with respect to water discharge Q (Hasaanzadeh, 2007)

6. Nomenclatuers

a	reference level; radius of particle; particle effective area; one of the triaxial dimensions of a particle; immersed sediment specific gravity
A	wetted cross-section area
A′	sediment coefficient
b	one of the triaxial dimensions of a particle
B	water surface top width
c	one of the triaxial dimensions of a particle
C	concentration of sediment at vertical distance y, Chézy coefficient
C_a	concentration of sediment at vertical distance a
C_D	drag coefficient
C_L	lift coefficient
C_*	Chézy coefficient en dimensionless form
d	particle diameter
d_*	dimensionless diameter
F	force
F_D	drag force
F_L	lift force
F_n	normal force to the angle of repose
F_t	parallel force to the angle of repose
F_r	Froude number
F_*	Shields parameter
f	hydrodynamic-immersed gravity force ratio
g	gravitational acceleration
h	water depth
h_v	vegetation height
k	constant, von Karmman’s constant
K_1, K_2, K_3	particle shape factor
K	Strickler coefficient, tractive- force ratio
K_h	erodibility index
n	Manning coefficient
P	length of wetted perimeter, pressure
Q	water discharge
q_s	rate of bedload in volume per unit time and unit width
Q_s	rate of bedload in weight per unit time, daily suspended load
Q_m	average monthly water discharge
Q_{sm}	average monthly suspended load
r	radius
Re	Reynolds number
Re_*	boundary Reynolds number
Re_c	critical Reynolds number
R_h	hydraulic radius
S	bed slope
S.F.	shape factor
S_f	energy gradient
t	time

T	temperature
$\tan\phi$	coefficient of friction
u	velocity in x-direction
u_*	shear velocity
u_b	liquid velocity at the bottom of the channel
V	mean velocity over depth
V_c	liquid critical velocity
V_s	velocity of solids
w	fall or settling velocity
W	grain weight
x	coordinate direction
y	coordinate direction
z	coordinate direction
Z	exponent in the suspension distribution
α	angle of the inclination of the bed form from the horizontal
β	constant
γ	liquid specific weight
γ_s	solid particle specific weight
γ_s'	solid particle submerged specific weight
δ	specific gravity
θ	angle of the side slpe
λ	friction factor
μ	liquid dynamic viscosity
μ_{susp}	dynamic viscosity of the suspension or mixture
ν	liquid kinematic viscosity
ν_m	kinematic viscosity of the suspension or mixture
π	dimensionless group
ρ	liquid mass density
ρ_s	solid particle mass density
τ	unit shear stress or tractive force
τ_c	critical unit shear stress
τ_0	unit shear stress at a solid boundary
τ_L	unit shear stress on the level surface
τ_s	unit shear stress on the side of the channel
τV	unit steam power
ϕ	angle of repose of the materials

7. Subscripts

none	liquid phase
b	at bed or at bottom
c	critical condition, center line
L	level surface

susp	suspension, mixture
m	mixture, suspension, average monthly
max	maximum value
v	vegetation
s	solid phase, side of channel, suspension
sm	average monthly suspended load
y	at a distance y
0	at boundary (y=0)
*	shear value
1,2,..	index
∞	at infinity

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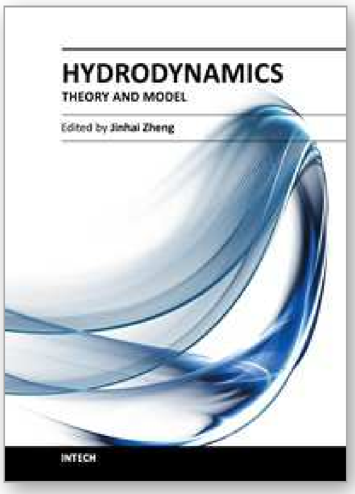

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