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BED LOAD MEASUREMENT AND SAMPLING

by

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SOCIETE CENTRALE
POUR L' EQUIPEMENT
DU TERRITOIRE
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Introduction

The bedload or contact load is commonly referred to as the kind of sediment transport which takes place near the bottom and carries particles which belong to the bed material. The motion of entrained particles is one of rolling, sliding and sometimes jumping (saltation) whereas in the so-called suspended-load the particles are carried along with the same velocity as the flow. (In fact, the transition between the two modes of transport is gradual. For clarity sake, we consider that particles of a given size fraction will or will not move with a bedload type of motion according to the prevailing hydraulic conditions).

Although there does not exist a sharp line of demarcation, the distinction between the two modes of transport becomes very important for two reasons:

1. two physically different models are used to describe each of them
2. the two loads are predominantly measured by different methods. The bedload is measured with suitable traps whereas the suspended load is obtained from water-mixture samples.

Chapter 1

Scour Criteria

Before dealing with the bedload equation, it seems right to the point to try and answer the question: when and why does bed-erosion begin to occur.

Consider a channel which fulfills the following requirements:

- a plane stationary bed consisting of loose and cohesionless solid particles of uniform size
- a constant cross-section area throughout a given distance
- a constant longitudinal slope
- a liquid flowing through the channel.

It is obvious that hydrodynamic forces are exerted upon the solid particles of the bed and that beyond a certain level there will be a motion of the particles, that is, a bedload transport phenomenum will take place. The problem is to determine which hydraulic conditions (called the critical condition or initial scour) for the given particles bring about this initial movement of the bed.

Numerous attempts were made to relate the initial scour to the flow velocity or to the shear stress or tractive force, both factors being in fact related to each other.

1.1 Critical Velocity

In theory the bottom velocity, U_b, has to be considered since it is the one on which depends the initial movement of the particles. Theoretical considerations lead to an extremely involved relationship which is of no practical use, this is due in part to the difficulty of both clearly define and measure the bottom velocity and to determine such parameters as the particles shape factors, etc....

So far it is widely accepted that the study called "Permissible Canal Velocities" by Fortier and al. is a safe criterion and has formed the basis for canal design for many years.

The following table summarizes the results of this study.

Table 1 Permissible canal velocities [*after* FORTIER *et al.* (1926)]

	*Velocity, m/s, after aging, of canals carrying**		
Original material excavated for canal (1)	*Clear water, no detritus* (2)	*Water-transporting colloidal silts* (3)	*Water-transporting noncolloidal silts, sands, gravels, or rock fragments* (4)
Fine sand (noncolloidal)	0.45	0.75	0.45
Sandy loam (noncolloidal)	0.55	0.75	0.60
Silt loam (noncolloidal)	0.60	0.90	0.60
Alluvial silts when noncolloidal	0.60	1.10	0.60
Ordinary firm loam	0.75	1.10	0.70
Volcanic ash	0.75	1.10	0.60
Fine gravel	0.75	1.50	1.15
Stiff clay (very colloidal)	1.15	1.50	0.90
Graded, loam to cobbles, when noncolloidal	1.15	1.50	1.50
Alluvial silts when colloidal	1.15	1.50	0.90
Graded, silt to cobbles, when colloidal	1.20	1.70	1.50
Coarse gravel (noncolloidal)	1.20	1.80	2.00
Cobbles and shingles	1.50	1.70	2.00
Shales and hard pans	1.80	1.80	1.80

* Depth of 0.9 m or less.

It is worth noting that in most cases the presence of colloidal silts increase the permissible mean critical velocities, $\bar{U}_{cr}$, this is sometimes explained by the damping of the turbulence due to the presence of the suspended sediment, one can also assume that the fluid properties are not the same. This interesting consideration makes the determination of the bedload all the more difficult when a sediment laden stream is considered.

HJULSTROM proposed the following figure (1) where it is clearly shown that the velocity required to ravel and scour a bed in any material is greater than the velocity required to maintain movement of particles of the same material.

The mean velocity, $\bar{U}$, used here is assumed to be about 40% greater than the bottom velocity for depths exceeding 1 m whereas in the Fortier's table, though the mean velocity was also used, depths were below 0.90 m and so the difference is not so great.

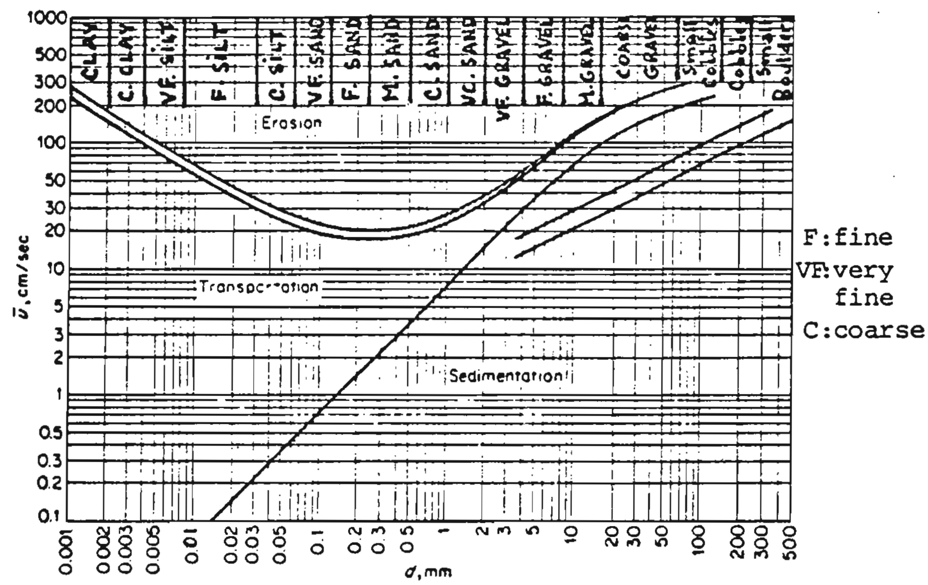


Fig. 1 Erosion-deposition criteria for uniform particles. [*After* HJULSTRÖM (*1935*).]

Further remarks are in place here:

- very small velocities are sufficient to maintain silt particles in suspension
- the great resistance to erosion in the smallest particles range which must depend on cohesion and adhesion forces. Therefore once a bed made of small particles has begun to be scoured it is unlikely that the removed layer might be replaced by particles belonging to the same range unless there is a drastic decrease of the velocity, so such a bed runs the risk of being continuously eroded.

The fact that Fortier's results are based on observations made in real canal with original soil material whereas Hjulstrom's ones were carried out on "monodisperse material on a bed of loose material of the same size of particles" might account for the discrepancies between the two approaches.

To end with the velocity as the leading factor, here are a few formulae:

$$(U_b)_{cr} = 4\sqrt{d} \quad \text{where } U_b \text{ in m/s and d in m (Forchheimer 1914)} \qquad (1)$$

$$(U_b)_{cr} = 3.3\, d^{4/9} \sqrt{\frac{\rho_s - \rho}{\rho}} \qquad \text{(Mavis et al.1948)} \qquad (2)$$

Note:

- If we put $\rho_s = 2650$ kg/m^3 and $\rho = 1000$ kg/m^3 we obtain

$$(U_b)_{cr} = 4.2\, d^{4/9} \quad \text{which does resemble the preceding} \qquad (2')$$

Both following formulae take into account the ratio of the mean depth, $\bar{D}$, of the stream to the particle diameter d. It is the mean velocity, $\bar{U}$ which goes on record here:

$$\bar{U}_{cr} = 1.4\sqrt{gd}\ \ln\frac{\bar{D}}{7d} \quad \text{for } \frac{\bar{D}}{d} > 60 \qquad \text{(Jarocki 1963)} \qquad (3)$$

or

$$\bar{U}_{cr} = 4.38\sqrt{d}\ .\ \ln\frac{\bar{D}}{7d} \quad \text{for } g = 9.81 \text{ m/s}^2$$

$$\bar{U}_{cr} = \left(2.50\,\frac{\rho_s - \rho}{\rho}\, gd\right)^{1/2} \left(\frac{\bar{D}}{d}\right)^{0.1} \qquad \text{(Neill 1967)} \qquad (4)$$

or with $g = 9.81$ m/s^2 , $\rho_s = 2650$ kg/m^3 and $\rho = 1000$ kg/m^3

$$\bar{U}_{cr} = 6.32\sqrt{d}\ \left(\frac{\bar{D}}{d}\right)^{0.1} \qquad (4')$$

The values found for different sizes of particles are lower with Neill's formula than with the Jarocki's one. This could be explained by the fact that Neill presents his formula as a "conservative design curve" for coarse uniform sediment.

Note: in all the foregoing formulae velocities $\bar{U}$, are expressed in meter per second and diameters in meter.

1.2 Critical Shear Stress

Due to the difficulties to relate the mean velocity, $\bar{U}$, to the bottom velocity, U_b , the shear stress, τ_o , was prefered by most researchers as a scour criterion.

The critical shear stress (or tractive force) $(\tau_o)_{cr}$ per unit surface is defined as the shear stress beyond which the particles start to move.

$$\tau_o = \gamma R_h S$$

γ = specific weight of the fluid in kg/m^3

R_h = hydraulic radius, that is, the ratio of the stream cross-sectional area to the wetted perimeter, in meter

S = slope of the energy line, that is, the tangent or sine if the angle is small.

therefore (τ_o) is expressed in kg/m^2.

Numerous experiments were carried out to relate $(\tau_o)_{cr}$ to the particle diameter. Unfortunately agreement among the shear stress formulae is not very good. Thus, care must be taken to specify the problem then select a formula which suits its specifications best.

Figure (2) summarizes the important work made by Lane on a considerable amount of field data. It is helpful and widely accepted though its design does not make it clear to read.

From the figure it is seen that the critical shear stress is considerably lower for clear water than for sediment laden water. This is consistent with the remark made about the critical velocity, see table (1) (Fortier).

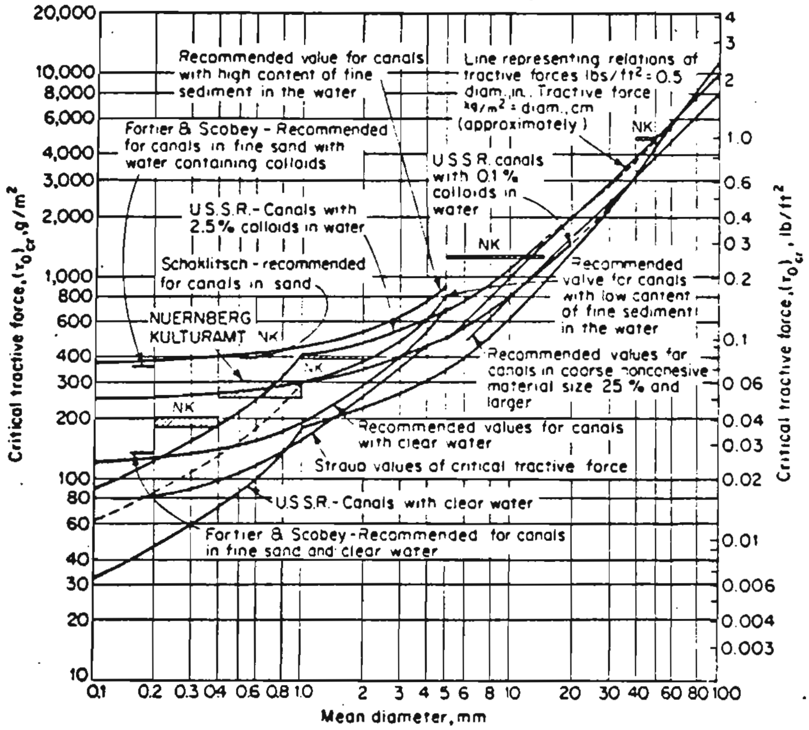
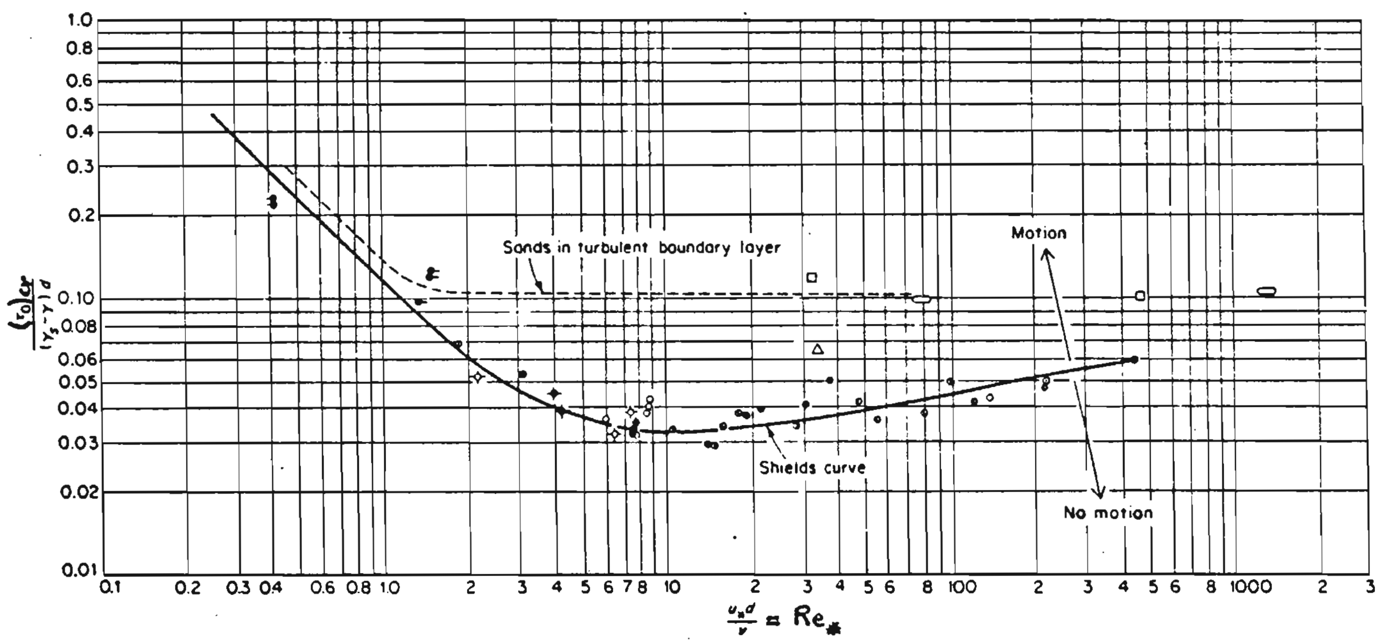


Fig. 2 Critical shear stress as function of grain diameter. [*After* LANE (*1953*).]

A useful guide is the Shield's diagram presented in Fig. 3



Fully developed turbulent velocity profile

Sym	Description		γ_s, g/cm³
o	Amber	(Shields)	1.06
•	Lignite	(Shields)	1.27
•	Granite	(Shields)	2.7
•	Barite	(Shields)	4.25
•	Sand (Casey)		2.65
✧	Sand (Kramer)		2.65
✦	Sand (U.S.W.E.S)		2.65
•	Sand (Gilbert)		2.65

Turbulent boundary layer

Sym	Description	γ_s, g/cm³
•-	Sand (Vanoni)	2.65
-•	Glass beads (Vanoni)	2.49
□	Sand (White)	2.61
▭	Sand in air (White)	2.10
△	Steel shot (White)	7.9

Fig. 3 Shields' diagram; dimensionless critical shear stress vs. shear Reynolds number. [*After* VANONI (*1964*).]

d (or d_{50}) is the median diameter, that is, the Sieve diameter for which 50% per weight of material is finer.

The shape similarity between the Shield's diagram and the Hjulstrom's one is obvious.

In the abscissa appears along with the diameter, d , the shear velocity U_* , and the kinematic ~~velocity~~, ν .
viscosity

The shear velocity or friction velocity is defined by:

$$U_* = \sqrt{\frac{\tau_o}{\rho}} = \sqrt{\frac{\gamma R_H S}{\rho}} \quad , \rho \text{ being the specific mass of the fluid} \quad (6)$$

About the matter of units we must take care to express, γ , specific weight thus a force unit in Newton per m^3 or better to take for U_*

$$U_* = \sqrt{g\ Rh\ S}$$

Usually γ and ρ are both expressed in kg/m^3 (or g/cm^3 , or t/m^3) and this is confusing for they are numerically equal.

The dimensionless quantity $\frac{U_* d}{\nu}$ is called the "shear Reynolds number", Re_* , it is related in Shield's diagram to the quantity: $\frac{(\tau_o)_{cr}}{(\gamma_s - \gamma)d}$ which is called the dimensionless critical shear stress.

Shield's diagram was established for uniform sand mixtures so the values for $(\tau_o)_{cr}$ for non-uniform grain material as well as sticky ones are in fact higher. Furthermore, shear stress is liable to vary and it was shown that for a given average shear stress the momentary shear stress may be three times higher, in Shield's diagram are given the average values of the critical shear stress.

Chapter 2

Bedload Formulae

Numerous bedload formulae have been proposed. The ultimate goal is to predict the amount of bedload in a natural water course but most of the time the bedload motion has been studied in small-scale laboratory flumes thus the applications of bedload equations to field studies are hazardous.

The empirical or semi-empirical character of most of the equations entails for the application of each method to remain limited to similar hydraulic conditions and similar sedimentary material as originally used in the development of the equation.

On the grounds on a statistical analysis Johnson concluded that the choice of a formula could be made on the basis of convenience in measuring the variables appearing in it.

Two widely used formulae are presented here, namely, the Meyer-Peter and al. and the Einstein's bedload equations.

2.1 Meyer-Peter and al. Formula

The experiments were performed in a laboratory flume with a cross section of 2 m^2 and a total length of 50 m. The water discharge could be varied up to 5 m^3/s and the sediment discharge up to 4.3 kg/sec.m. The size of the grain varied from 3 mm to 29 mm.

Remarks:

1) The meaning of such a symbol as d_n is: n per cent in weight of the sediment mixture is finer than d. d_n is determined by use of the grain size distribution curve, see Fig.(9). d_{50} is called the median diameter.

2) $\bar{d}$ is the mean diameter, that is, for non-uniform material the weighted diameter. For example, consider a mixture divided in n (equal) classes with each having

a median diameter d_i , w_i being the weight of the class of same index i, i ranges from 1 to n. Then the mean diameter can be expressed with:

$$\bar{d} = \frac{\sum_1^n d_i w_i}{\sum_i^n w_i} \quad \text{so } d \text{ is weight-weighted}$$

If each class has the same weight (uniform material), w , then

$$\bar{d} = \frac{w \sum_1^n d_i}{nw} = \frac{\sum_1^n d_i}{n} = d_{50} .$$

The first attempt resulted in the following equation for uniform sand.

$$0.4 \frac{g_s^{2/3}}{d} = \frac{g^{2/3} S}{d} - 17 \qquad (7)$$

which does justice to most of the data except the ones of small grain diameters, Fig. (4).

THE BEDLOAD

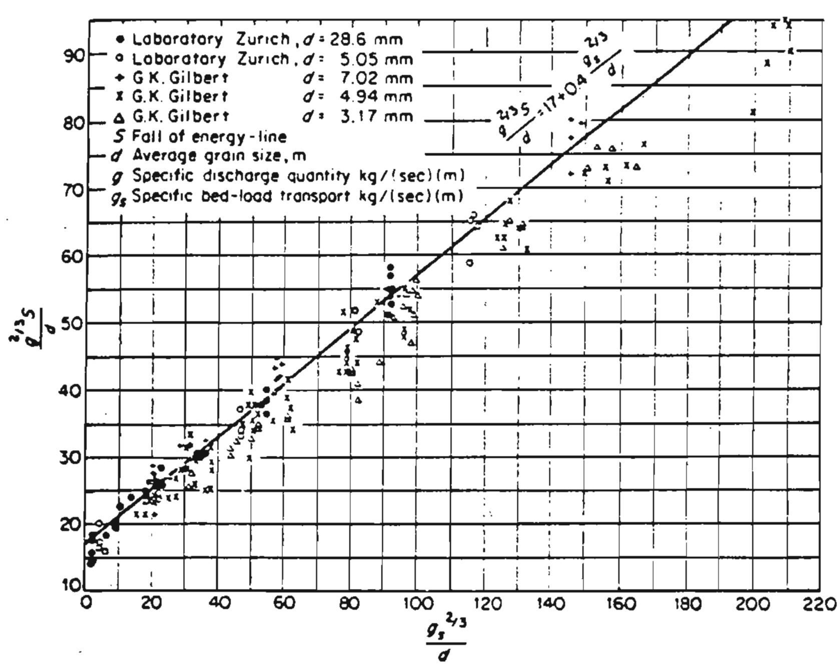


Fig. 4 The E.T.H. bedload equation. [*After* MEYER-PETER *et al.* (*1934*).]

Further experiments led to the formula:

$$\frac{g^{2/3}\,S}{d} - 9.57\left(\frac{\gamma_s-\gamma}{\gamma}\right)^{10/9} = 0.462\,\frac{(\gamma_s-\gamma)^{1/3}}{\gamma^{1/3}d} \times \left(\frac{\gamma_s-\gamma}{\gamma_s} \times g_s\right)^{2/3} \tag{8}$$

which is the formula used for uniform grain.

The symbols used are:

kg

g	:	water discharge per unit width in m^3/s per meter
g_s	:	bedload discharge per unit width in kg/s per meter
γ, γ_s	:	specific weights of fluid and particles respectively in kg/m^3
d	:	diameter in m
S	:	slope of the energy line

Eventually the following formula (9) was derived for sediment mixtures.

$$\frac{\gamma\,Rh\left(\frac{n'}{n}\right)^{3/2} S}{(\gamma_s-\gamma)\,d} = 0.047 + 0.25\left(\frac{\gamma}{g}\right)^{1/3} \times \left(\frac{\rho_s-\rho}{\rho_s}\right)^{2/3} \times g_s^{2/3} \times \frac{1}{(\gamma_s-\gamma)d} \tag{9}$$

Some more symbols are used, viz,

ρ, ρ_s	:	density of fluid and particle respectively in kg/m^3
d	:	or d_{50} median diameter in meter
g	:	gravitational constant equal to 9.81 m/s^2
R_H	:	hydraulic radius which equals the mean depth $\bar{D}$ when bank resistance is negligeable, in meter
n'	:	Manning roughness coefficient for a plane bed
n	:	actual Manning coefficient.

As can be seen from Fig.5, formula (9) fitted all of the data pretty well.

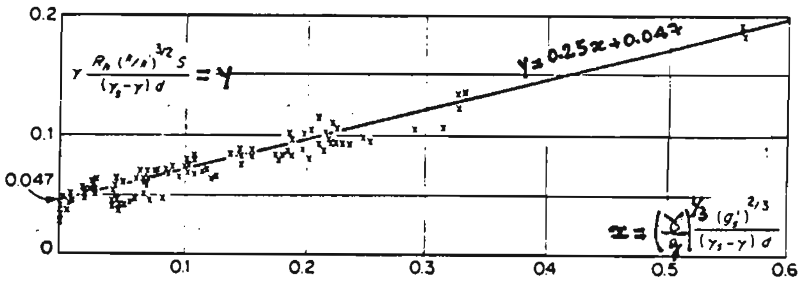


Fig. 5 The E.T.H. bedload equation. [*After* MEYER-PETER *et al.* (*1948*).]

This formula established for grain mixtures takes into account the bedform through the $\frac{n'}{n}$ factor.

Remark: If we consider a plane bed the resistance to the flow is due only to the grain roughness referred to as, the surface drag, whereas with a bed which presents ripples of dunes, an additional resistance, the form drag, takes place. This can be expressed in terms of shear stress by the equation:

$$\tau_0 = \tau_0' + \tau_0'' \tag{10}$$

Since the surface drag is mainly responsible for the bedload motion the problem is to divide the bed resistance into its two components. Meyer-Peter used the Manning formula.

$$\bar{U} = \frac{1}{n} \times R_h^{2/3} \times S^{1/2} \qquad S : \text{total energy loss (slope)} \tag{11}$$

and assumed that the total energy loss, S , could be divided. Keeping the hydraulic radius constant we get:

$$\bar{U} = \frac{1}{n'} R_h^{2/3} S'^{1/2} \qquad S': \text{being the energy loss due to the grain resistance} \tag{12}$$

thus

$$\frac{n'}{n} = \left(\frac{S'}{S}\right)^{1/2} \quad \text{transformed into } \frac{n'}{n} = \left(\frac{S'}{S}\right)^{2/3} \text{ for fitness' sake with experimental results} \tag{13}$$

In fact, the ratio of $\frac{S'}{S}$ is difficult to measure and an easier way is to determine the ration of $\frac{n'}{n}$. n is known through equation (9) and for n' the Strickler's formula can be used.

$$n' = \frac{1}{21} (d_{50})^{1/6} \qquad (14)$$

or $$n' = \frac{1}{24} (d_{65})^{1/6} \quad \text{(Einstein)} \qquad (14')$$

or $$n' = \frac{1}{26} (d_{90})^{1/6} \quad \text{(Muller)} \qquad (14'')$$

The ratio of $\frac{n'}{n}$ as far as sand and gravel mixture are concerned, varies from 0.5 for strong bedforms to 1 when no bedform exists.

2.2 Einstein's Bedload Equation

Einstein departed from the previous bedload formula. He has developed a new function where he assumed that the bedload transport is related to the fluctuations of the velocity rather than to the average value of the velocity and that the probability of a particle commencing to move could be expressed in terms of the rate of transport, the size and the relative weight of the particle and a time factor equal to the ratio of the particle diameter to its fall velocity. The same probability was expressed again in terms of the ratio of forces exerted by the flow to the resistance of the particle to motion. The two forms of probability and relationship were then equated to yield the function:

$$\Phi = f\ (\psi) \qquad (15)$$

where the quantity

$$\Phi = \frac{g_s}{\gamma_s} \left(\frac{\rho}{\rho_s - \rho} \times \frac{1}{g d^3} \right)^{1/2} \qquad (16)$$

is called the intensity of the bedload transport

and $$\psi = \frac{\rho_s - \rho}{\rho} \times \frac{d}{S R'_H} \qquad (17)$$

is called the flow intensity or shear intensity

The bedform, if any, is taken into account through R'_H which is the hydraulic radius with respect to the grains. Unlike in the Meyer-Peter's formula the slope is kept constant whereas the hydraulic radius is divided.

An empirical relationship was first set up between Φ and ψ. Figure (6) curve (1) shows that for $\Phi < 0.4$ the relationship can be expressed with:

$$0.465\ \Phi = e^{-0.391\ \psi} \qquad \text{Curve (1)} \qquad (18)$$

Curve (2) though it explains the data for $\Phi > 0.4$ better than curve (1), however still deviates from the data. This was attributed to the fact that the experimental data included suspended-load material.

Remark : Φ and ψ being dimensionless quantities any consistent set of units may do.
will

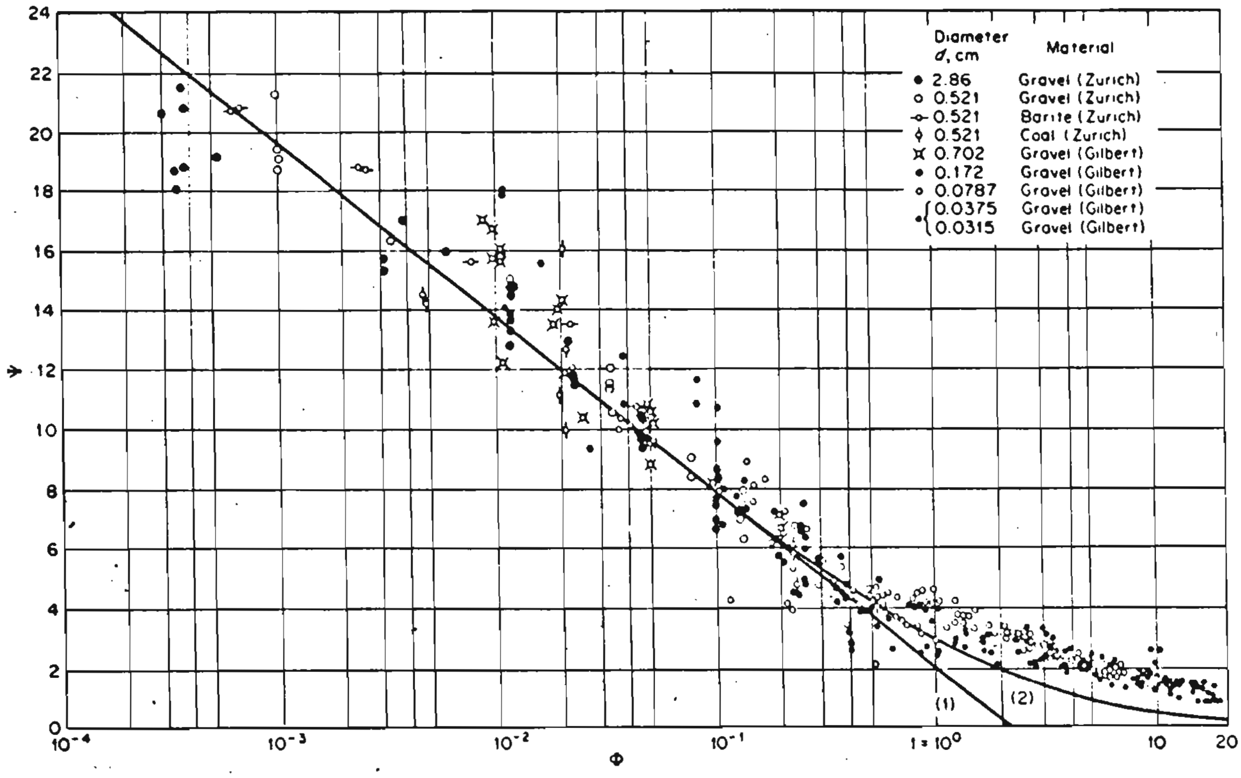


Fig. 6 Einstein's bedload equations. [*After* EINSTEIN (*1942*).]

Evantually, Einstein developed an analytical relationship which is certainly the most generally applicable but also the most involved bedload equation, namely,

$$1 - \frac{1}{\sqrt{\pi}} \int_{-B_* \psi_* - 1/\eta_o}^{+B_* \psi_* - 1/\eta_o} e^{-t^2} dt = \frac{A_* \Phi_*}{1 + A_* \Phi_*} \qquad (19)$$

Where A_*, B_* and η_o are universal constants to be determined from experimental data. The following values were obtained for uniform sediments.

$A_* = \frac{1}{0.023} = 43.5$

$B_* = \frac{1}{7} = 0.143$

$\eta_o = \frac{1}{2}$ $\quad$ η being a random function distributed according to the normal-error law (Gauss) whose standard deviation is η_o

In theory the transport rate of each individual component has to be calculated, then by summing up the total transport rate is obtained. Fortunately for mixtures with small size spread, the total transport of the mixture can be determined directly by using d_{35} as the effective diameter.

In Fig. (7) is shown a graphical form equation (19).

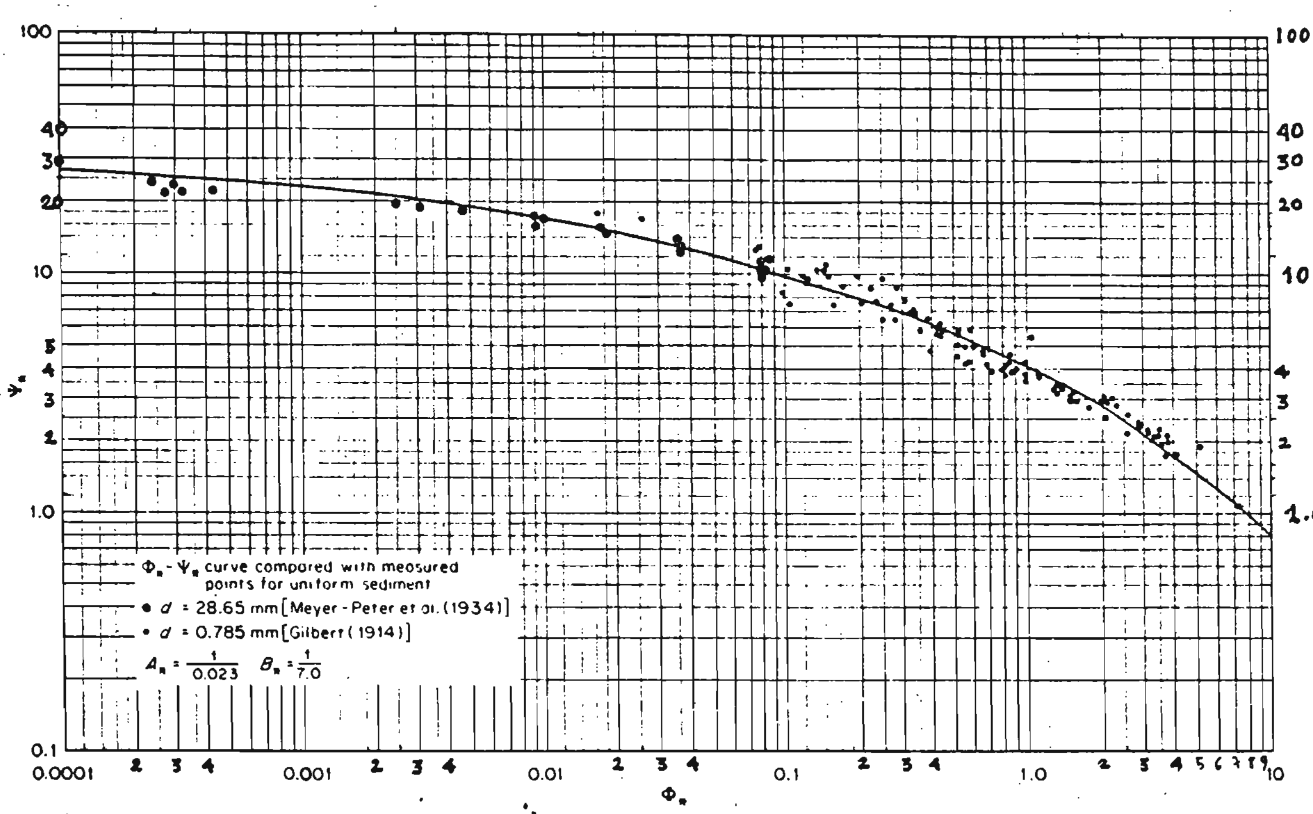


Fig. 7 Plot of Einstein's functions; Φ_* vs. Ψ_*. [*After* EINSTEIN (*1950*).]

It is interesting to rewrite the Meyer-Peter's formula using the Einstein's notation. In order to do that, we have to assume that R'_H can be expressed as the product of R_H and $(\frac{n'}{n})^{3/2}$, that is, dealing with R_H as was done with S in Meyer-Peter's formula.

$$\frac{\gamma R_H \left(\frac{n'}{n}\right)^{3/2} S}{d\,(\gamma_s - \gamma)} = 0.047 + \frac{0.25 \left(\frac{\gamma}{g}\right)^{1/3} \left(\frac{\rho_s - \rho}{\rho_s}\right)^{2/3} g_s^{\,2/3}}{d(\gamma_s - \gamma)} \qquad (9)$$

with the foregoing assumption we have:

$$\psi = \frac{\rho_s - \rho}{\rho} \times \frac{d}{S\left(\frac{n'}{n}\right)^{3/2} R_H} \qquad (17')$$

and

$$\Phi = \frac{g_s}{\gamma_s}\left(\frac{\rho}{\rho_s - \rho} \times \frac{1}{gd^3}\right)^{1/2} \qquad (16)$$

with

$$\frac{\rho_s - \rho}{\rho} = \frac{\gamma_s - \gamma}{\gamma}$$

combining (9) with (17') and (16) we obtain:

$$\frac{1}{\psi} = 0.25\,\Phi^{2/3} + 0.047 \qquad \text{or} \qquad (20)$$

$$\Phi = \left(\frac{4}{\psi} - 0.188\right)^{3/2} \qquad (20')$$

Equations(19) and (20) are presented in Fig.(8) for uniform material and they show very good agreement. For sediment mixture by using d_{35} in the Einstein relation and d_{50} in the Meyer-Peter et al. agreement was found equally good.

SEDIMENT TRANSPORT IN OPEN CHANNELS

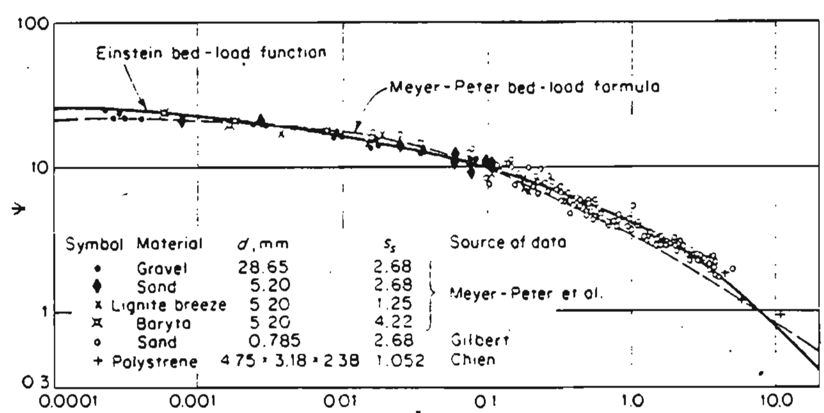


Figure 8

Chapter 3

Example of Bedload Calculation

Given a river equipped with a gauging station. Assuming the channel geometry to be stable and long enough to measure the slope with sufficient accuracy and the bed material composition to be uniform throughout the cross section.

The water discharge rating curve is available and so is the curve cross-section area versus gauge-height, that is, a survey of the cross section was carried out recently.

The following data will be used.

Gauge Height	:	1.80 m	
Q	:	152 m^3/s	through the rating curve, Q versus G.H.
A	:	116 m^2	through the cross-section area versus G.H. curve
Width	:	103 m	through the cross-section area versus G.H. curve
Bottom width	:	101 m	through the cross-section area versus G.H. curve

Shape of the wetted perimeter : rather trapezoidal with steep-slope banks and almost plane bed.

$\bar{D}$ (mean depth): 1.13 m $\bar{D} = \dfrac{A}{\text{width}}$ most generally D # R_H as far as rivers are concerned

R_H : 1.07 m $R_H = \dfrac{A}{\text{wetted perimeter}}$

$\bar{U}$ (mean velocity) : 1.31 m/s $\bar{U} = \dfrac{Q}{A}$

S : 0.0007

From the following grain size distribution of bed material.

(A density of 2650 kg/m^3 (quartz) is assumed)

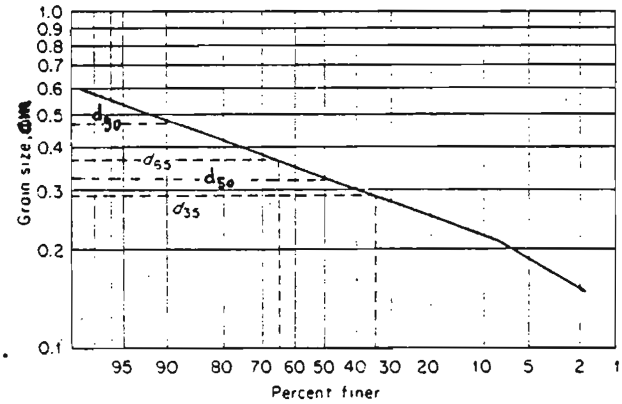


Fig. 9. Grain size distribution of bed material.

We obtain:

d_{90} = 4.7 mm d_{65} = 3.5 mm d_{50} = 3.2 mm d_{35} = 2.9 mm

Compute the Manning coefficient, n , with formula (11).

$$\bar{U} = \frac{1}{n} R_H^{2/3} S^{1/2} \quad \text{we get} \quad n = 0.0212$$

Compute the Manning coefficient n', that is, if the resistance to flow was due solely to the roughness of the grains.

$$n' = \frac{d_{90}^{1/6}}{26} = 0.0157 \quad \text{(Muller)} \qquad (14")$$

3.1 Meyer-Peter and al. Approach

$$\psi = \frac{\rho_s - \rho}{\rho} \times \frac{d_{50}}{R_H \left(\frac{n'}{n}\right)^{3/2} S} = \frac{2650 - 1000}{1000} \times \frac{3.2 \times 10^{-3}}{1.07 \times \left(\frac{0.0157}{0.0212}\right)^{3/2} \times 7 \times 10^{-4}} \qquad (17')$$

ψ = 11.07 dimensionless value

$$\Phi = \left(\frac{4}{\psi} - 0.188\right)^{3/2} = \left(\frac{4}{11.07} - 0.188\right)^{3/2} \qquad (20')$$

Φ = 0.072 dimensionless value

and we get g_s through the formula (19)

$$\Phi = \frac{g_s}{\gamma_s}\left(\frac{\rho}{\rho_s - \rho} \times \frac{1}{gd^3}\right)^{1/2} = \frac{g_s}{2650} \times \left(\frac{1000}{1650} \times \frac{1}{9.81 \times 3.2^3 \times 10^{-9}}\right)^{1/2} \qquad (16)$$

g_s = 0.139 kg/ms, that is, the bedload rate per unit width

and for the whole bottom width

$G_s = g_s \times w_{bottom} = 0.139 \times 101 \# 14$ kg/s

3.2 Einstein Approach

Here we consider d_{35} as the representative diameter and assume that the hydraulic radius, R'_H , due to the grain roughness may be computed through the formula $R'_H = R_H \left(\frac{n'}{n}\right)^{3/2}$.

We have for ψ :

$$\psi = \frac{\rho_s - \rho}{\rho} \times \frac{d_{35}}{R_H \left(\frac{n'}{n}\right)^{3/2} S} = \frac{1.65 \times 2.9 \times 10^{-3}}{1.07 \times 0.74^{3/2} \times 7 \times 10^{-4}}$$

$$\psi = 10.04$$

The figure (6) shows that for $\psi = 10.04$ we have a Φ value of 0.09. We follow the same procedure, that is, we obtain g_s through formula (16). Thus,

$$0.09 = \frac{g_s}{2650} \times \frac{(10^9)^{1/2}}{(1.65 \times 9.81 \times 2.9^3)^{1/2}} \qquad \text{and}$$

$$g_s = 0.150 \text{ kg/ms}$$

and for the whole bottom width

$$g_s = 0.150 \times 101 = 15.2 \text{ kg/s} \quad \# \quad 15 \text{ kg/s}$$

So, the two approaches give similar results. The closeness of the results must not hide the fact that any bedload equation applied to field studies remain a mere estimate.

However, Meyer-Peter's equation tested with large grains does represent an interesting approach. Einstein's relationship tested with a number of experimental data is considered to represent the most complete equation of all.

A further remark is in place here. All the bedload equations are supposed to predict the maximum bedload or transporting capacity that a stream in equilibrium can possibly carry at the given hydraulic and sedimentary conditions. The bedload being a highly unsteady phenomena the actual bedload may or may not be equal to the transporting capacity. Given below in Figure (10), an example where fluctuations with respect to time of both bedload and velocity are shown. As could be expected bedload and velocity are in phase but the variations in amplitude are much greater for the bedload than for the velocity.

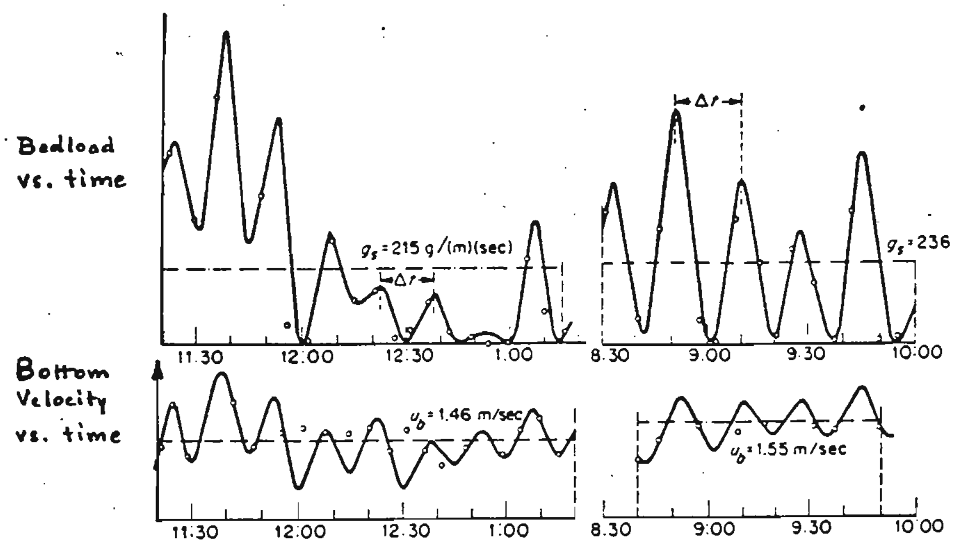


Fig. 10 Unsteady bedload transport. [*After* EHRENBERGER (*1931*).]

DANUBE RIVER.

Chapter 4

Direct Measurements

Bedload is difficult to measure for several reasons. Any mechanical device placed in the vicinity of the bed will disturb the flow and hence the rate of bedload movement.

The sediment movement and the velocity of water close to the bed vary considerably with respect to both space and time. See fig. (10). (In testing one bedload sampler on the Middle-Loup river at Dunning, Nebraska, Hubbel found the sampling rate to range from 11 to 120 cc per minute at one point).

So far, published data from field measurements are scarce and also hard to work with.

Furthermore, if a sand bed presents dunes, which is often the case, the results are completely different according to the location of the device with respect to the crest of the dune.

For the time being, there exists no generally agreed upon definition for the demarcation between the two forms of transport. This makes the problem of measuring difficult and at times dubious.

Obviously, the more heterogenous the bed material size the higher the number of samples required.

4.1 Box and Basket-Type Samplers

Bedload samplers of this type consist of a pervious container where bedload accumulates, of a supporting frame and cables to make the sampler portable and of vane to give the sampler the appropriate direction.

The sampling operation consists of lowering the sampler to the bed and, on contact with the bed, the front gate of the sampler opens and a timer is released. The water and the bedload enter the box, experience a velocity reduction, and thus the bedload is deposited in the trap. At the end of the measurement, the gate is closed, the measuring time is recorded and the sampler lifted.

A typical box-basket sampler is presented below Figure (11).

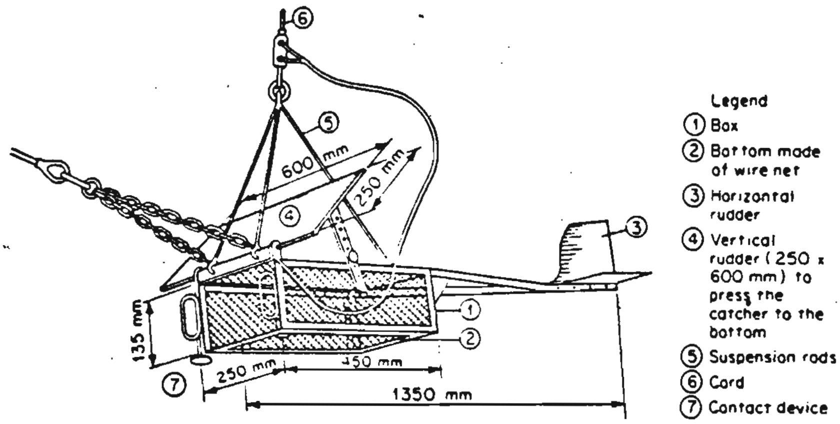


Fig. 11 Mühlhofer sampler. [*After* JAROCKI (*1963*).]

A major drawback is that due to the presence of the sampler the velocity reduction may take place at the entrance to the sampler instead of inside. So accumulation of material may prevent the bedload from entering the sampler.

With the pressure-difference samplers this drawback is avoided by making the side walls diverge toward the rear so that the intake velocity and stream velocity are identical. The most notable and used, the Arnhem or Dutch pressure-difference sampler is presented in Figure (12). The pressure drop to maintain sampling velocity is obtained by use of a flared section at the rear of a rubber connection. It has been designed to trap and measure coarse sand and fine gravel.

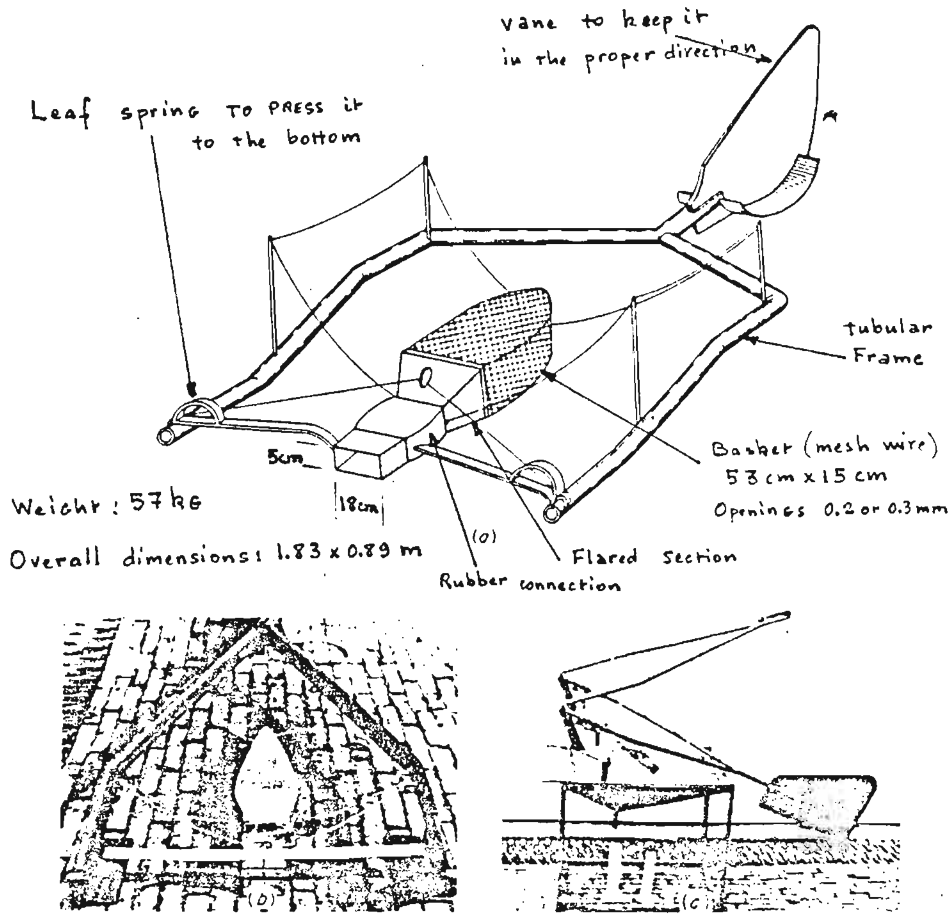


Fig. 12 Arnhem sampler (BTMA). (*a*) The Arnhem sampler [*after* HUBBELL (*1964*)]. (*b*) The new Arnhem sampler with an improved frame construction. (*c*) The emptying of the instrument; the catch is measured volumetrically. [The photographs are provided by Diephuis (1969) from the Delft Hydraulics Laboratory and are made available by the Van Essen N.V. company in Delft. This modified version is not yet in use in the Netherlands; however, application is expected.]

In the Karolyi sampler presented in Figure (13) pressure drop is obtained by use of an increasing height toward the rear of the sampler. The rear section has a dividing wall. Beneath this wall the bedload gets trapped, and the clear water rises and leaves at the exit of the sampler.

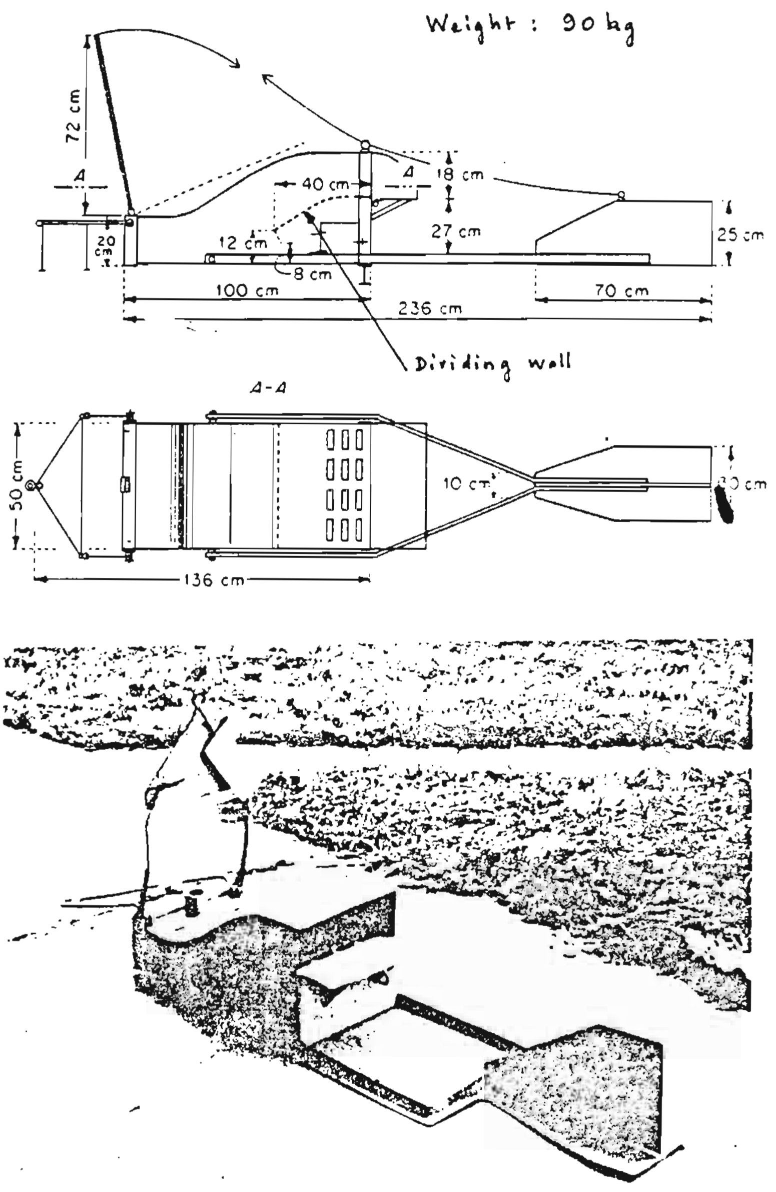


Fig. 13 Karolyi sampler [*after* NOVAK (1959)]. [The photograph is of a sampler presently at use at the *River Training Experimental Station* and was provided by Stelczer (1969).]

The VUV sampler which obeys the same principles as the Karolyi one was designed by NOVAK to improve the efficiency of the latter. (Fig.14)

SEDIMENT TRANSPORT IN OPEN CHANNELS

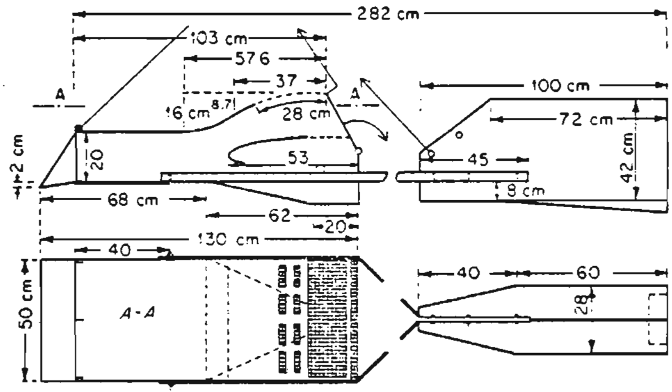
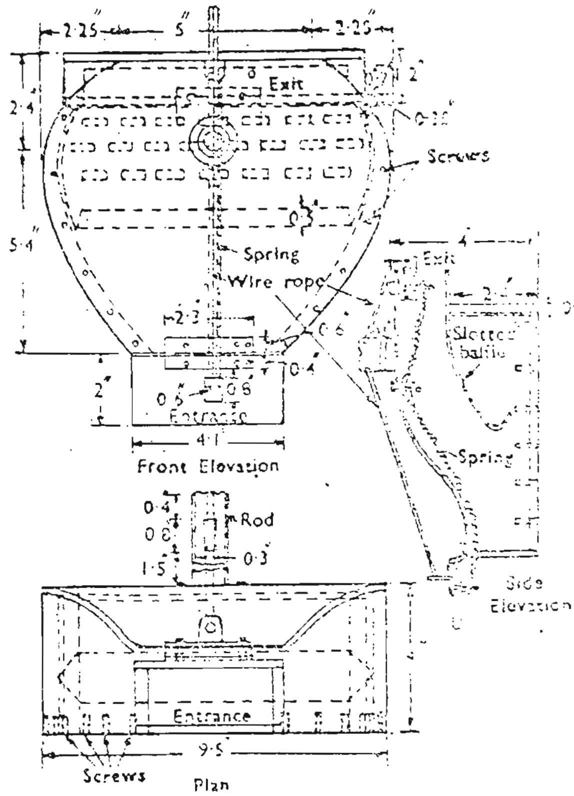


Fig. 14 VUV sampler. [*After* NOVAK (*1959*).]

Rather similar to the Karolyi sampler, Uppal sampler presented here in Figure (15). Uppal claims a 90% efficiency for this kind of sampler. Dimension in Figure (15) are given in inches (1 inch = 2.54 cm).

SEDIMENT OBSERVATIONS ON RIVERS AND CANALS

Figure 15 : *Uppal Bed load sampler.*



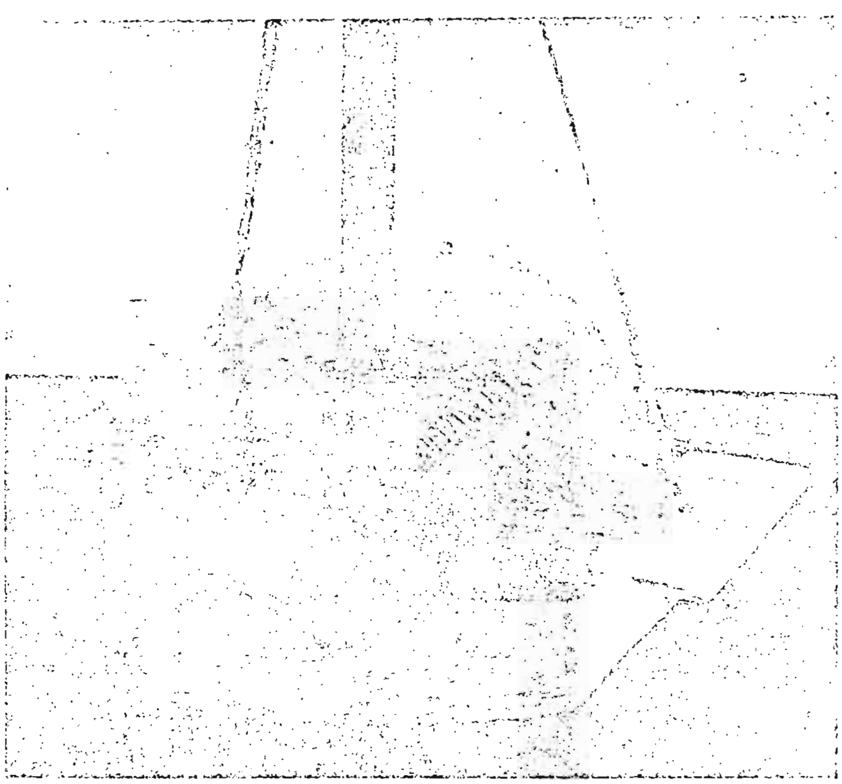


Figure 16 : *Uppal Bed load sampler.*

4.2 Pan-Type Samplers

These are shaped like a wedge with downstream half of the top surface sopen so that the bedload will ascend the incline and drop into the interior. Although water does not flow through such a sampler, there is considerable disturbance of the flow which may result in selective sorting of the coarse material. Samplers of this type have been advanced by Soviet researchers but outside the Soviet Union there exists little experience with such samplers. Use of that kind of samplers must be limited to stream which present smooth bed and both moderate velocities and bedload.

A sketch Fig. (16).

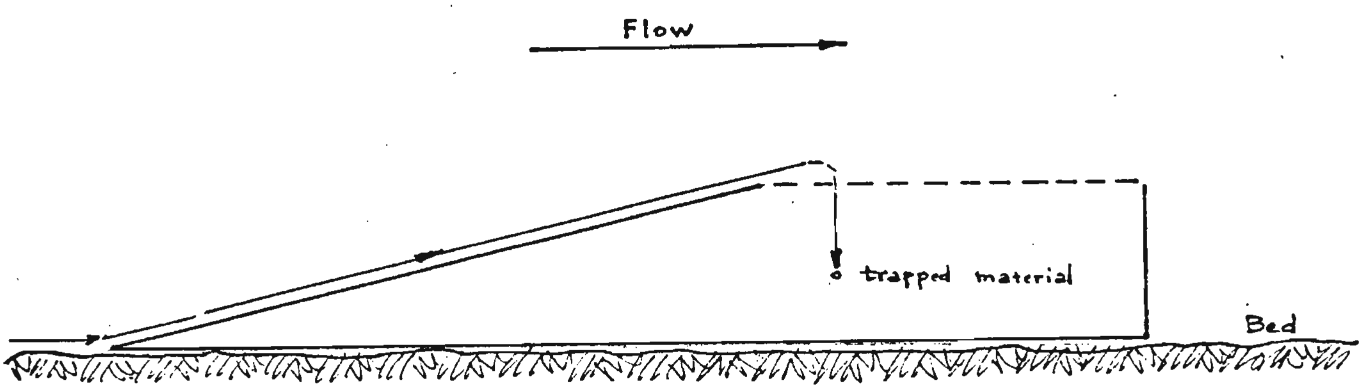
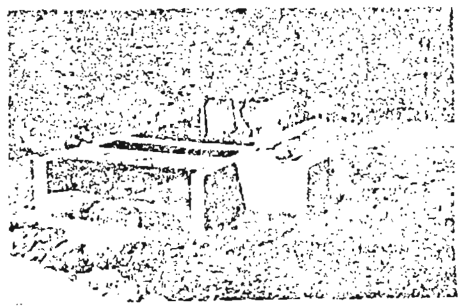


Fig.16

4.3 Pit-Type Samplers

Considerably different from the foregoing samplers which are introduced into the flow for only short intervals of time, are structures built permanently into small streams and canals. These may consist of open or grated depressions (pits) extending entirely across the channel near the end of a uniform reach. If a mechanical device is installed which removes continuously the accumulated sediment, a continuous record of the bedload rate is obtained. The efficiency of such a sampler is rather high.

For small watersheds, less than 1 km^2 when there exists a strong erosion, a joint pit-flume structure may be built. The pit is located upstream close to the flume, the velocity reduction allows the bedload to drop and be trapped into the pit and both suspended-load and water discharge are measured at the flume. In Figure (17) is shown such a structure built by the ORTSOM in Guinea.



4.4 Sound Samplers

The bedload scraping along the bottom creates audible sound waves. Acoustic instruments are designed to pick up these waves. The equipment consists of an underwater microphone located at a certain distance from the bottom and housed in a streamlined body, an amplifier, and a recorder. So far, only qualitative information has been given by sound samplers.

4.5 Tracing Methods

Information about the movement of sediments can be obtained through tracing of labeled particles, i.e. an artificial sediment mixture presenting the same density and dimensions as the natural bed material is supplied with particles activated by neutron irradiation in a nuclear reactor so that they can emit detectable radiations, the mixture is then released into the stream and the motion of the labeled particles is studied and measured by use of a Geiger counter for example.

So far, important conclusion on sediment motion has been drawn from tracer studies but most of them have as their scope the providing of qualitative answers and determination of bedload through tracing of radioactive isotopes is not to the best of our knowledge a current practice.

Chapter 5

Bed-Material Samplers

Data on the size of material making up the streambed are necessary for computation of bedload and for study of the long-range changes in channel conditions as well.

Three bed samplers capable of collecting bed-material samples consisting of particles finer than about 30 or 40 mm in diameter are presented here.

5.1 For wadable streams the US-BMH-53 may be used. Shown in Fig.(18), it is a boring type samplers of 1.17 m total length. The bed sample is collected in a cylinder in which vacuum is created by means of a tight-fitting brass piston. This partial vacuum retains the sample in the cylinder while the sampler is being removed from the bed. The piston is used to force the sample out from the cylinder.

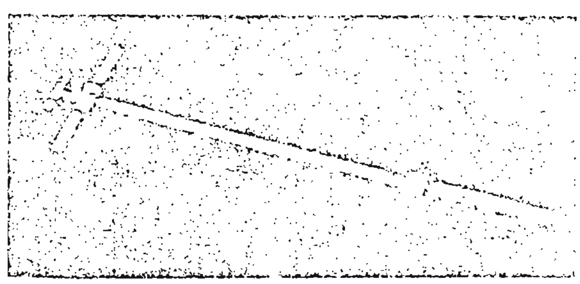


Figure 18.—Hand-held piston-type bed-material sampler, US BMH-53.

5.2 For no wadable streams, several samplers have been designed by the Interagency Committee on Water Resources. Based on the same principle that is, a streamlined body houses a mechanism consisting of a rotating scoop bucket kept open during the lowering of the sampler. Upon contact with the bed, through the release of tension on the hanger rod, the scoop is rotated by a heavy coil spring thus, it penetrates the bed and traps a sample. A rubber gasket prevents trapped material from being washed out of the bucket when it is closed.

The US BM(H) 60 designed for streams of moderate depths (max # 4 m) can be hand-manipulated, it is available in weights of approximately 13 kg, 16 kg or 18 kg. Figure (19).

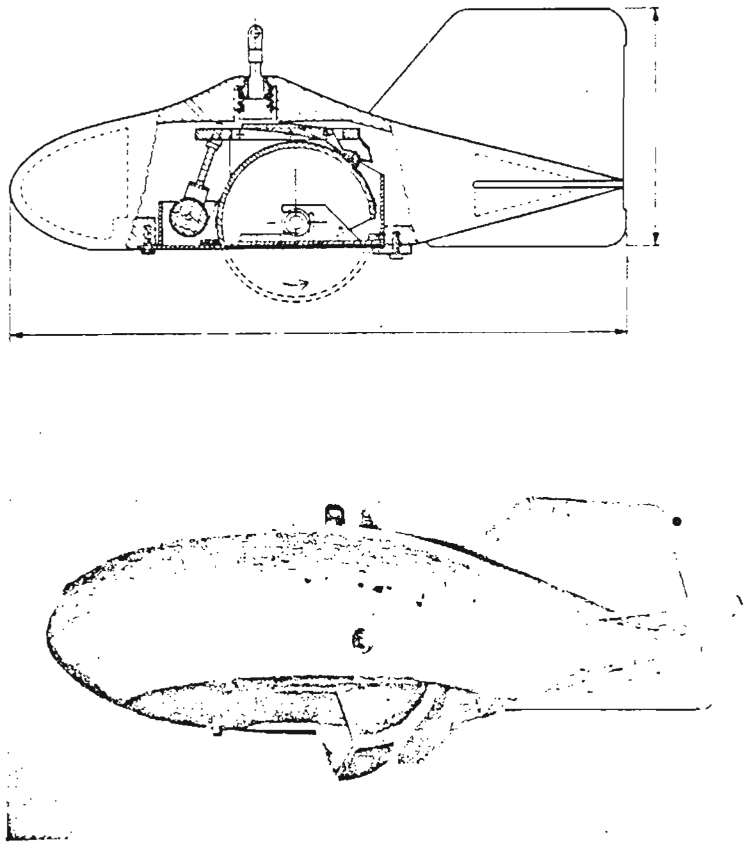
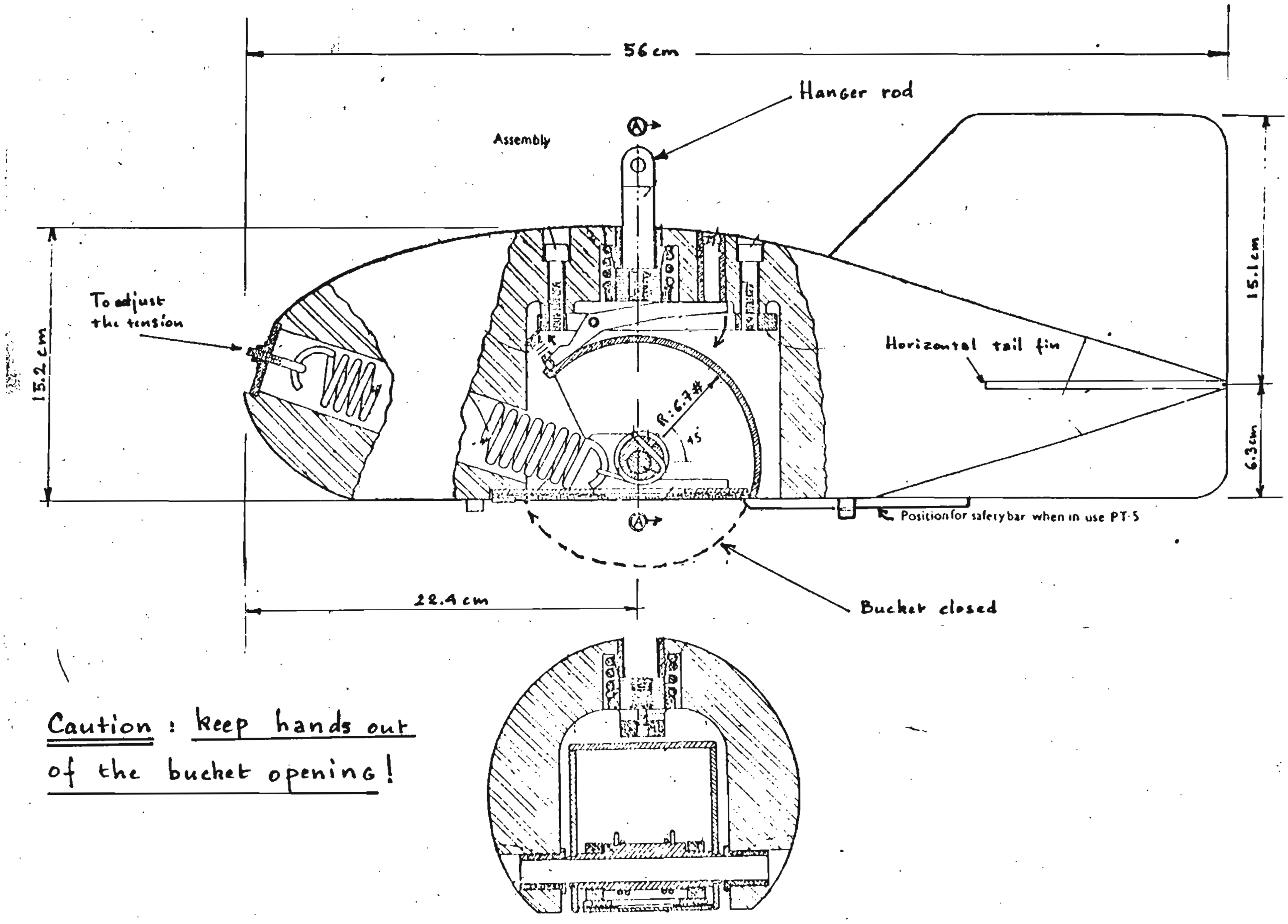


Fig. 19 Bed material sampler US BM-60. [*After* INTERAGENCY COMMITTEE ON WATER RESOURCES (*1963*).]

5.3 The US BM 54 cable-and-reel bed sampler weighs 45 kg and the tension on the spring which is different from the US BM 60's one can be adjusted, a useful feature when the streambed is very firm. Details are given in Figure (20). Approximately the top two inches of the streambed are taken (# 5 cm) and the bucket can hold 175 cc of material.



Section AA

Figure 20 : *Bed material sampler U.S.B.M.-54.*

5.4 When high velocities are encountered the following procedure may be used. A white petroleum jelly (vaseline) disk is strapped onto a heavy sounding weight (up to 100 kg) so that the disk is on the bottom side of the weight. The sampler is lowered to the streambed at any speed desired. Upon striking the bed, the sounding weight forces the disk against the streambed and the vaseline assures adherence of the surface bed particles. (Guy-Field methods for measurement of fluvial sediment, Chapter C 2).

5.5 A gravel sampler designed by UPPAL is presented in Figure (21).

SEDIMENT OBSERVATIONS ON RIVERS AND CANALS

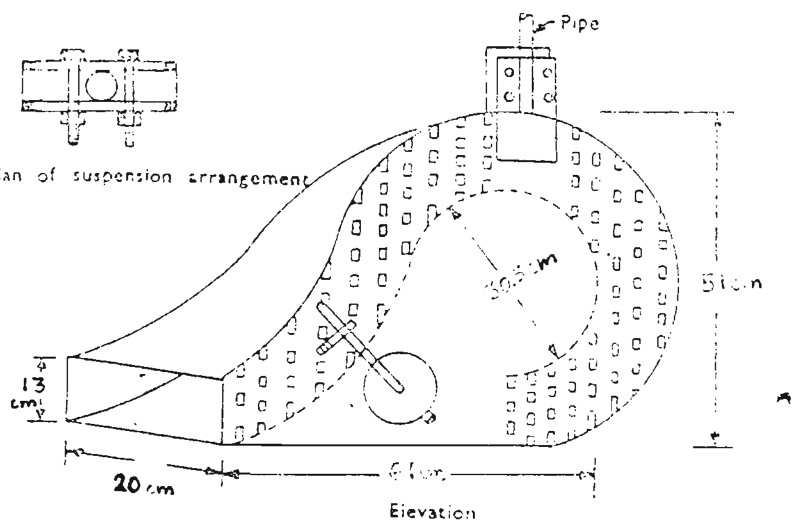


Figure 21 : *Gravel sampler.*

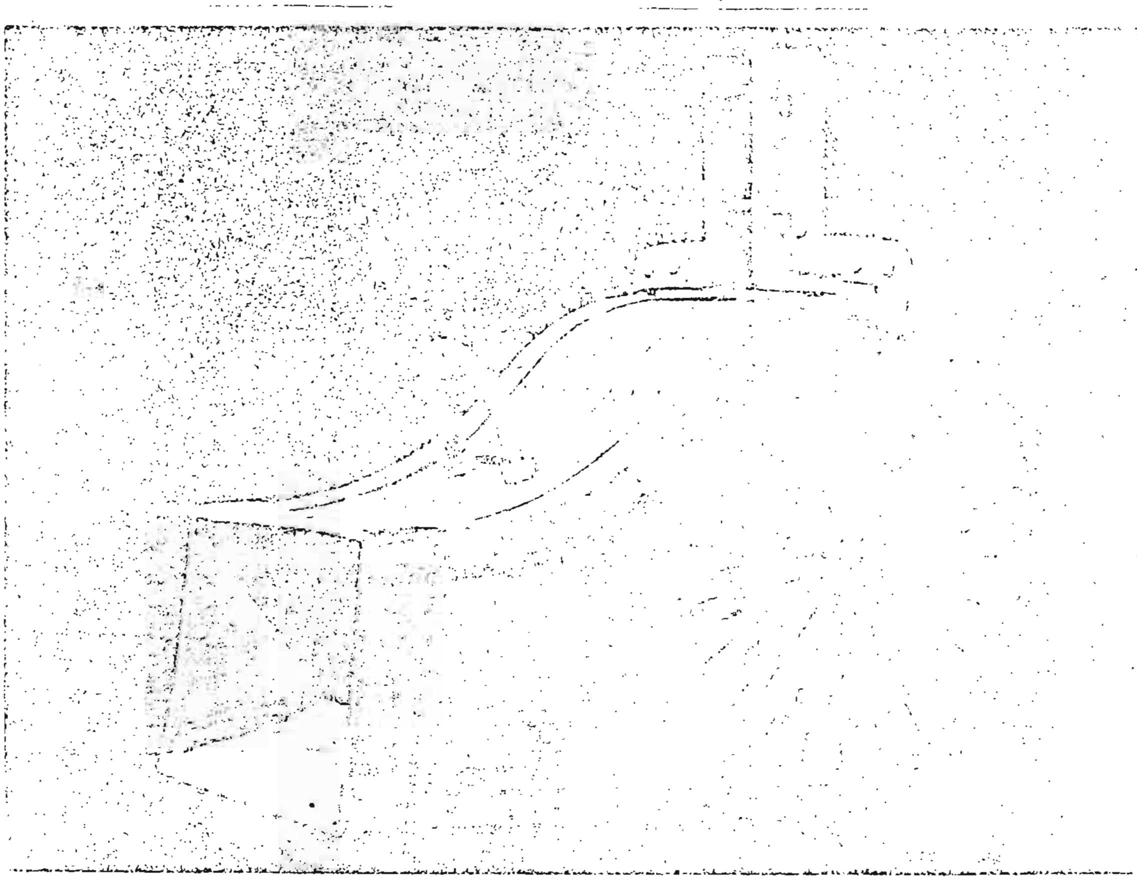


Figure 21 : *Gravel sampler.*

Any gravel sampler is awkward to handle and quite often streams having gravel, cobble and boulder beds spell high velocities, that makes measurements all the more difficult, if not impossible.

Remark: As was mentioned in "direct bedload measurements" changes in bed material composition entail to get more bed material samples. Sub-division of the streambed might be required, the chosen bedload formula being applied to each subdivision.

Chapter 6

Concluding Remarks

It stands to reason that either measuring bedload or sampling bed material may turned out to be a very hard task.

For example, if high velocities are encountered to make a bedload sampler penetrate the flow is nearly impossible, let alone the fact that such a device under very turbulent conditions is not likely to stay on the surface bed but rather to be dragged away in an erratic movement. Given the same conditions though streamlined a bed material sampler may prove to be difficult to handle.

In fact, most sediment transported by alluvial streams is washload which is made up of particles finer than the bulk of the bed material ones. These fine particles are supplied from the watershed thus the rate of washload passing through a channel reach is unaffected by the prevailing hydraulic conditions of the reach but depends both on the hydrological (rainfall, slopes, vegetations, etc...) and geological conditions prevailing throughout the watershed. So, knowledge about the washload can be reached only through direct measurements during the period of flow.

The bedload is maximum in streams with sandy bed where the size of the suspended material closely approaches the size of the bed material and where quantity of the suspended material is low, that is, where erosion from the watershed does not play a leading part.

Generally, the bedload is taken as a certain percentage of the suspended material. This percentage veries from 3 to 5 percent of the total suspended load, depending in part upon the nature of the bed material. A percentage of 10 is the more commonly accepted figure.

The following table (2) due to Borland and Maddock (Sedimentation studies for planning of reservoirs by U.S.B.R., IVth Congress on Large Dams 1951) gives data on situation of bedload.

Table 2

Suspended load concentration	Nature of bed Material	Nature of Suspended Material	Percentage of bedload in terms of total suspended load
Low - 1000 ppm or less	Sand	About the same as bed	Up to 50%
	Gravel or Rock	Clay, Silt plus small amount of sand	5%
Medium - 1000-7500 ppm	Sand	About the same as bed	10 - 20%
	Gravel or Rock	Clay, Silt, 25% sand or less	5 - 10%
High - 7500 and more	Sand	About the same as bed	10 - 20%
	Gravel or Rock	Clay, Silt, 25% sand or less	2 - 8%

It must be pointed out that during a flood at a given location in a river changes both in the flow regime and in concentration of suspended material occur. These changes will in turn affect bedload rates. For example, in the lower regime taking place during the falling limb of the flood hydrograph the sediment supply from the watershed is usually low and so is the concentration of fine suspended sediment (washload), in that case the bedload may represent an important factor in the sediment transport process. On the contrary, in the upper regime taking place during the rising limb of the hydrograph, concentration is high consequently the bedload plays a very minor part.

Therefore, we must keep in mind that under drastic hydraulic conditions the fact that direct bedload measurements cannot be carried out is not all that important since bedload is likely to be a slight percentage of the total load.

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