# "A CONTRIBUTION TO THE TOTAL SEDIMENT LOAD TRANSPORTATION"

## A THESIS

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## ABSTRACT

This thesis deals with the estimation of the sediment charge rates in fluvial open channels flow . The bed load , the suspended load and the total load rates were studied based on laboratory measurements and compared to some existing previous investigations .

The main purpose of this research is to find a correlation between the bed load concentration and that of the suspended load.The effect of sediments on flow characteristics was studied , too .

It was found that the bed load concentration equals the extrapolated suspended load concentration at a depth equals half the thickness of the moving bed layer , based on this conclusion , a step by step method was introduced to estimate the total sediment load rate given the sediment flow parameters only .

In studing effect of sediment on flow characteristics ,it was found that: first the sediment reduces the Von Karman's constant "k" ; the value of k decreases as sediment concentration increases , second the value of the sediment transfer coefficient $\epsilon_s$ is equal to 1.28 the value of momentum transfer coefficient $\epsilon_m$ for the uniform sand used in the present experiments with 0.15 mm grain size.

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# CONTENTS

# LIST OF SYMBOLS

The following are the main symbols which have been used in this thesis.Any other symbols are defined wherever they appeared:-

| SYMBOL | MEANING | DIMENSIONS. |
|---|---|---|
| $A$ | Cross-sectional area | $L^2$ |
| $A_b$ | Cross-sectional area associated with the channel bed. | $L^2$ |
| $A_w$ | Cross-sectional area associated with the side walls. | $L^2$ |
| $A_*$ | Constant. | _ |
| $B$ | Bed width. | $L$ |
| $C$ | *Chezy's* coefficient. | $L^{0.5}T^{-1}$ |
| $C$ | Sediment concentration. | _ |
| $C_a$ | Reference concentration. | _ |
| $C_D$ | Drag coefficient. | _ |
| $C_L$ | Lift coefficient. | _ |
| $D$ | Water depth. | $L$ |
| $d$ | Grain diameter | $L$ |
| $Fe$ | Froude number. | _ |
| $i_b$ | Bed-load fraction in a given grain size. | _ |
| $i_s$ | Suspended load fraction in a given grain size. | _ |
| $i_t$ | Total load fraction in a given grain size. | _ |
| $g_B$ | Bed-load rate in weight per unit width | $M\ T^{-3}$ |
| $g_S$ | Suspended load rate in weight per unit width. | $M\ T^{-3}$ |
| $g_T$ | Total load rate in weight per unit width. | $M\ T^{-3}$ |
| $g$ | Accelaration due to gravity. | $L\ T^{-2}$ |
| $h$ | Ripple height. | $L$ |
| $M$ | *Kramer* uniformity coefficient. | _ |
| $m$ | Mass. | $M$ |
| $N$ | Volume concentration of moving solids. | _ |

| SYMBOL | MEANING | DIMENSIONS |
|---|---|---|
| $N_*$ | Maximum grain concentration by volume. | _ |
| n | *Manning-Kutter* rugosity factor. | _ |
| L | Length. | L |
| P | Wetted perimeter. | L |
| Q | Water discharge. | $L^3T^{-1}$ |
| q | Water discharge per unit width. | $L^2T^{-1}$ |
| $q_B$ | Volume rate of bed-load discharge per unit width. | $L^2T^{-1}$ |
| $q_s$ | Volume rate of suspended load dischrge per unit width. | $L^2T^{-1}$ |
| $q_T$ | Volume rate of total load discharge per unit width. | $L^2T^{-1}$ |
| R | Hydraulic mean depth of the channel. | L |
| $R_b$ | Hydraulic mean depth associated with the bed. | L |
| $R_w$ | = = = = = = walls. | L |
| $R_h'$ | = = = due to grain roughness. | L |
| $R_h''$ | = = = = = bed-forms. | L |
| Re | Reynold's number. | _ |
| $Re_*$ | Particle Reynold's number. | _ |
| S | Channel slope,or specific gravity of solids. | _ |
| t | Time. | T |
| u,v | Local velocity at depth y . | $L\ T^{-1}$ |
| U,V | Mean velocity . | $L\ T^{-1}$ |
| $u_*,v_*$ | Shear velocity. | $L\ T^{-1}$ |
| $V_g$ | Grain speed. | $L\ T^{-1}$ |
| x | Coordinate direction. | _ |
| X | A function of $k_s/\delta'$ | _ |
| y | Coordinate direction , elevation, depth. | L |
| Y | A function of $d/\delta'$. | _ |
| z | Exponent in the suspended load distribution. | _ |
| ........ | | |
| $\alpha$ ,$\beta$ | constats. | _ |
| $\gamma$ | Specific weight of fluid. | $ML^{-2}T^{-2}$ |

| SYMBOL | MEANING | DIMENSIONS |
| --- | --- | --- |
| $\gamma_s$ | Specific weight of solids. | $ML^{-2}T^{-2}$ |
| $\Delta\gamma_s$ | $= \gamma_s - \gamma$ ,Immersed specific weight of solids | $ML^{-2}T^{-2}$ |
| $\delta$ | Boundary layer thickness. | L |
| $\delta'$ | The laminar sub-layer thickness. | L |
| $\epsilon_m$ | Momentum transfer coefficient. | - |
| $\epsilon_s$ | Sediment transfer coefficient. | - |
| $\eta$ | Relative depth, effeciency. | - |
| $\theta, \tau_*$ | Dimensionless shear stress, angle. | - |
| k | *Von-Karman's* constant. | - |
| $\lambda$ | Ripple wave length. | |
| $\mu$ | Dynamic viscosity. | $ML^{-1}T^{-1}$ |
| $\upsilon$ | Kinamatic viscosity. | $L^{2}T^{-1}$ |
| $\rho$ | Density of fluid. | $M L^{-3}$ |
| $\rho_s$ | Density of solids. | $M L^{-3}$ |
| $\tau$ | Shear stress. | $ML^{-1}T^{-2}$ |
| $\tau_b$ | Average shear stress on the bed . | $ML^{-1}T^{-2}$ |
| $\tau_c$ | Critical shear stress. | $ML^{-1}T^{-2}$ |
| $\phi, \phi_*$ | Intensity of bed-load transport. | - |
| $\phi$ | Angle of repose of the bed material. | - |
| $\psi$ | Shape factor, Einstein's shear parameter. | - |
| $\omega, v$ | Fall or terminal velocity of grains. | $LT^{-1}$ |
| o | Subscript o signifies the value at the bed. | - |
| c | Subscript c signifies the critical value. | - |
| s | Subscript s refers to solid or suspended. | - |
| - | Bar over a quantity signifies mean values. | - |
| JHD | Journal of hydraulic division ASCE. | - |

## LIST OF FIGURES AND PLATES

# CHAPTER 1

# INTRODUCTION

The transport of sediment by water is a problem of great importance to mankind. Silt deposition reduces the capacity of reservoirs, interferes with harbour operation and modifies the path of water courses. Erosion or scour may undermine structures such as bridge piers, which in-turn threaten the structure itself.

Clearly, sediment transport occurs only if there is an interface between a moving fluid and an erodible boundary. The activity of this interface is extremely complex. Once sediment is being transported, The flow is no longer a simple fluid flow, since two materials are involved.

Sediment transport in open channels may be conceived of as occuring in one of two modes;

(a) *The bed-load.* (b) *The suspended load.*

The bed-load is usually defined as that portion of sediment that is moving in the immediate vicinity of the bed; This is the material that rolls,slides or saltates along the bed. Whereas the suspended load is that portion of total load that all surrounded by fluid (by suspension in the moving fluid). There is continual exchange of material between the region of the suspended load and the bed-load, also between the stationary bed and the transported sediment.

The sum of the bed-load and the suspended load is referred to as the *bed material load*. Besides the bed material load, there may exist the so-called wash load, which is made up of grain sizes and materials that is rarely found in the bed. The wash load and the bed material load together compose the total load. When wash load is not present, the terms bed material load and total load are often used interchangeably.

Numerous bed-load equations have been proposed to calculate the rate of bed-load transport, but some of them look very similar. In fact there are essentially four slighty different approaches to the bed-load problem. These are:

1- The *Du Boys*-type equations, considering shear stress relationship.

2- The *Schoklitsch*-type equations considering discharge relationship.

3-The *Einstein*-type equations based on statistical considerations of the lift forces.

4- Bed forms type, bed-load equations which consider the bed form motion.

Regarding the suspended load transport rate, the common equations are considered to be analogous to the diffusion dispersion process. This model is referred to as diffusion-dispersion model. Although such a model doesn't account for all influences, it has been found to explain satisfactorily many suspension problems. In this model a 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 is assumed for steady state condition and uniform turbulence distribution.

The basic equation for steady vertical diffusion is of the form;

$$\omega \, dc/dy + d/dy(\epsilon_s dc/dy) = 0 \qquad \ldots(1.1)$$

where $\omega$ is the settling velocity of the particle, c is the concentration at a distance y from the bed, $\epsilon_s$ is the sediment transfer coefficent.

Being less popular in evaluating suspended load rate of transport;the energy or gravitational models, as well as the statistical model are also reviewed .

The total sediment load is simply the sum of the suspended load rate and the bed-load rate excluding any wash load, this concept was probably first done by *Lane and Kalinske*[1]. Then *Einstien*[2] introduced an approach which gives the total sediment load ; other workers followed the same approach, which is called the indirect determinations of total load or Microscopic methods .Another group of researchers feel that, in calculation of total load,no need exists to distinguish bed-load from suspended load,because the hydrodynamic forces involved in both cases are the same. Thus it is not necessary to define the thickness of the bed-layer and the proper line of demarcation between bed-load and suspended load. Instead they relate the total load to the shear and other flow parameters.This is the direct determination of total load or Macroscopic methods.The relationships proposed by these direct methods are based on dimensional analysis, intuition or complete empiricism.

The main purpose of the present work is to study the concentration of the bed-load and that of the suspended load, trying to find a general correlation between them based on experimental measurements. If such a relation exists, the total load can be evaluated by only estimating one of its concentrations i.e.either the bed-load or the suspended load without using any reference concentration or field measurments.

General review of the previous work related to the bed-load, the suspended load, and the total load is presented in chapter (2),hoping that these information will serve as a useful suplement.

# HISTORICAL REVIEW

# CHAPTER (2)
# HISTORICAL REVIEW

## 2.1 INTRODUCTION :-

The problem of sediment transport in open channel has attracted the attention of hydraulic engineers for many centuries due to its connection with river control,reservoirs capacity,and the design of irrigation projects.

The transport of sediment is also important in land erosion, soil conservation and transportation by water and wind,undermining of hydraulic structures, and in industrial processes where solids are transported by fluid.

The loose noncohesive material interacts with the water flow is called sediment;the subcommittee on sediment Terminology of The *American Geophysical Union* has accepted the following definition given by the new Standard Dictionary; "Fragmental material transported by, suspended in, or deposited by water or air, or accumulated in the beds by other natural agent".

The analysis of sediment transport load is usually separated into two parts: Bed-load and suspended load, the two parts form the bed material load,or the total load in the absence of wash load.The bed-load,the suspended sediment load ,and the total sediment load formulae are introduced in sequence after a brief review of incipient sediment motion.

## 2.2. INCIPIENT OF SEDIMENT MOTION:-

Incipient of sediment motion, is a subject of great importance to be studied in evaluating the transport rate of certain sediment flow.Incipient conditions of sediment particle means the hydraulic conditions at which sediment particles of a given size just start moving. Three major approaches have been used to establish the condition for incipient motion of sediment particles composing the bed; these are:

1- Competency: in which the size of bed material d is related to either bed velocity (bottom velocity) or mean velocity of flow,which just causes the particle to move

2- Lift concept:in which, it is assumed that the motion starts when the upward force due to fluid,lift force,is just greater than the submerged weight of the particle.

3- Tractive force approach, which is based on the concept that,the tractive force exerted by the flowing fluid on the channel bed in the direction of flow is mainly responsible for the starting of sedimen particles motion.

The three approaches will not be reviewed in detail, however,the important and useful relations will be highlighted. The critical tractive force approach seems to be more rational and sound than other,although the competent velocity approach is much simpler.

One of the earliest expressions is due to *Brahms*[3], in 1753, the critical velocity of water is given by;

$$u_c = K\ W^{1/6} \qquad ...(2.1)$$

where K is an empirical constant and W is the weight of the grain.

*Mavis et al*[4], in 1937, obtained experimentaly the competent bottom velocity as:

$$(u_b)_c = 1/2\ d^{4/9} \sqrt{\rho_s/\rho - 1} \quad \text{fps} \qquad ...(2.2)$$

where d is in mm.

*Jarockv*[5], in 1963, deduced a formula as

$$u_c = 1.4 \sqrt{gd}\ \ln D/7d \qquad ...(2.3)$$

which is applicable for D/d > 60 only.

*Neill*[6], in 1968, has related the parameters of sediment flow over a rough boundary as:-

$$u_c/[(\gamma_s-\gamma)d/\rho]^{1/2} = 1.414(D/d)^{1/6} \qquad ...(2.4)$$

*Garde*[7], in 1970, analyzed the available data and found that for hydrodynamically rough surfaces:

$$u_c/[(\gamma_s-\gamma)d/\rho]^{1/2} = 0.5 \log D/d + 1.63 \qquad ...(2.5)$$

where d is the grain size and D is the depth of the flow.

*Yang*[8], in 1973, found that for rough boundary, when $Re_* = u_* d/\nu > 70$ then

$$u_c/\omega = 2.05 \qquad ...(2.6)$$

and when $u_* d/\nu < 70$, he found that

$$u_c/\omega = 2.5/(\log u_* d/\nu - 0.06) + 0.66 \qquad ...(2.7)$$

these equations are subjected to doubt as $u_c/\omega = \infty$ for

$u_* d/\nu = 1.148$.

Concerning the lift concept, *White*[19], in 1940, came to the conclusion that the lift is very small compared to the weight of the particle. However *Einstein and El Samni*[9], in 1949, found that:

$F_L$ = Lift force per unit area of the particle is given by

$$F_L = C_L \rho U^2/2 \qquad \text{...(2.8)}$$

where $C_L$, the lift coefficient, given by them to be 0.178, and U is the velocity of flow at 0.35 $d_{35}$ from the bed. Here $d_{35}$ is the grain size at which 35% of the material by weight is finer, the above value of $C_L$ is valid for rough bed only.

The critical tractive force concept has gained more attention among the others.

The bed shear stress due to flowing fluid is given by.

$$\tau_o = \rho g R S \qquad \text{...(2.9)}$$

where $\rho g$ is the unit weight of fluid, R is the hydraulic radius and S is the channel slope. Many empirical formulae for the critical shear stress $\tau_c$ were given according to experimental data.

*Kramer*[10], in 1939, defined the critical shear stress as,

$$\tau_c = (10^{-4}/6)\ (\gamma_s - \gamma) d/M \qquad \text{...(2.10)}$$

where M is the Kramer's uniformity coefficient varying from 0.256 to 1.00, $\tau_c$ in $N/m^2$, $\gamma$ in $N/m^3$, and d in mm.

United States Waterways Experiment *Station*[11], has proposed the formula;

$$\tau_c = 0.285 \; [(\rho_s-\rho)d/(\rho M)]^{1/3} \qquad \ldots(2.11)$$

where $\tau_c$ in $N/m^2$, $\rho$ in $kg/m^3$, and d in mm.

*Chang*[12], in 1939, found that the relation between $\tau_c$ and d is different for different ranges of grain sizes. According to him for $(\rho_s-\rho/\rho)(d/M) > 2.0$ ;

$$\tau_c = 0.216 \; [(\rho_s-\rho/\rho)(d/M)]^{1/2} \qquad \ldots(2.12)$$

and for $[(\rho_s-\rho)/\rho](d/M) < 2.0$,

$$\tau_c = 0.304[(\rho_s-\rho)/\rho](d/M) \qquad \ldots(2.13)$$

the units are the same as equation (2.11) above.

*Krey* [13] found that

$$\tau_c = 0.754 \; [(\rho_s-\rho)/\rho]d \qquad \ldots(2.14)$$

where $\tau_c$ in $N/m^2$ and d in mm.

*Schoklitsch*[14], in 1914, advanced the formula

$$\tau_c = \sqrt{0.201 \; \gamma_s(\gamma_s-\gamma)\eta d^3} \qquad \ldots(2.15)$$

where $\eta$ is an empirical factor depends on the shape of sediment taken 1.0 for spheres, $\tau_c$ is expressed in $N/m^2$, $\gamma$ in $N/m^3$ and d in meters (SI Units).

*Kalinske*[15], in 1947, expressed $\tau_c$ as

$$\tau_c = 0.232 \; (\gamma_s-\gamma)d \qquad \ldots(2.16)$$

this will be discussed later as well.

*Leliavsky*[16], in 1955, found a simple relationship between the shear stress and the grain size as:-

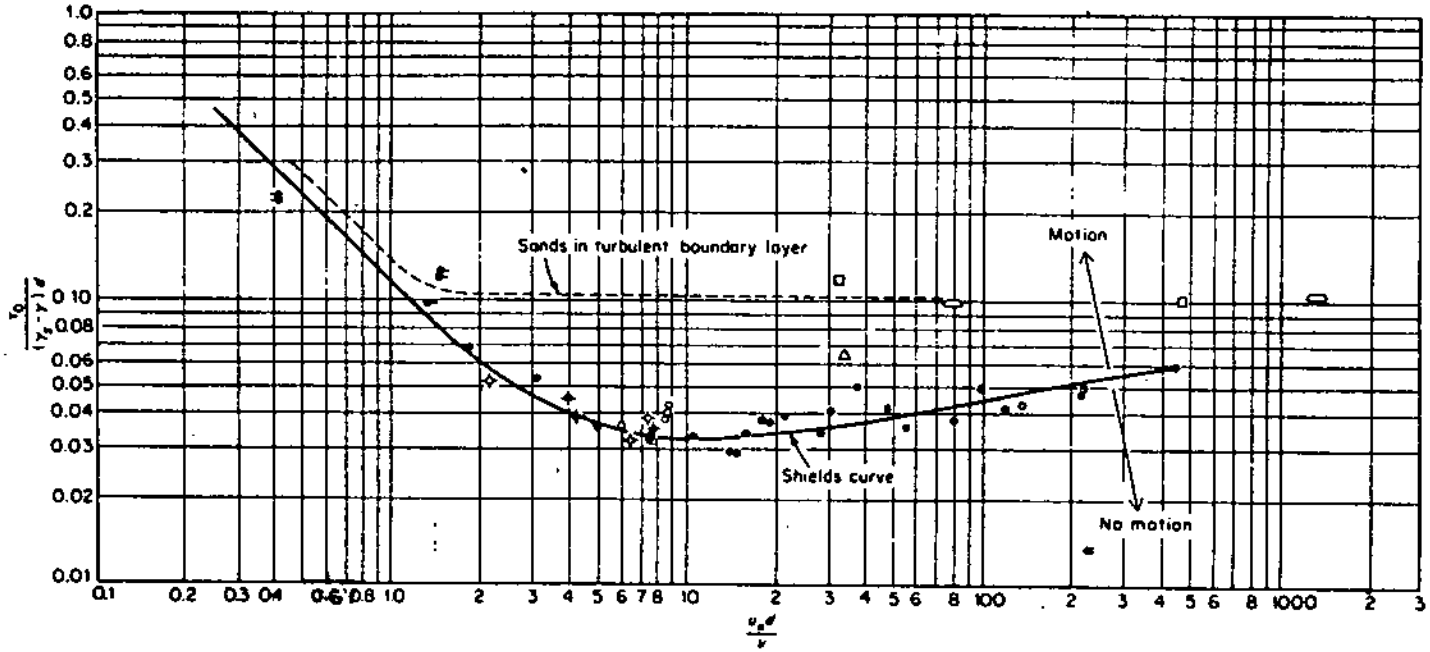
$$\tau_c = 166d \quad , \; g/m^2 \qquad \ldots(2.17)$$

in which d is given in mm.

Modern advancement in fluid mechanics was due to expressing the term friction or shear velocity, $u_*$, to represent a measure of turbulent fluctuation. *Shields*[17], in 1936, using the concept of dimensionless shear stress, found that dimensionless shear stress is a function of Shear Reynold's number $Re_*$ as ;

$$\tau_c/(\gamma_s-\gamma)d = f(du_*/\nu) \qquad ...(2.19)$$

where the left side of the above equation is the dimensionless shear stress and the $du_*/\nu=Re_*$. Shields plotted the above relation based on experimental data and obtained the famous "*Shields' Curve*", Fig. (2.1), which can be divided into three zones according to the value of $Re_*$.




Fully developed turbulent velocity profile

| Sym | Description | $\gamma_s$, 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

| | | |
| --- | --- | --- |
| •- | Sand (Vanoni) | 2.65 |
| -• | Glass beads (Vanoni) | 2.49 |
| o | Sand (White) | 2.61 |
| ⊂⊃ | Sand in air (White) | 2.10 |
| Δ | Steel shot (White) | 7.9 |

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

FIG.(2.1) SHIELDS' DIAGRAM

It was shown that at high $Re_*$ values, the dimensionless shear stress is independent of the shear Reynold's number, i.e for $R_* \geq 400$ ,$\tau_c$ is given by ;

$$\tau_c/[(\gamma_s-\gamma)d] = 0.06 \qquad ...(2.20)$$

*Zeller* [18], in 1963,found that equation (2.19) above can be written as ;

$$\tau_c/(\gamma_s-\gamma)d= 0.047 \qquad ...(2.21)$$

Generally speaking,Shields' diagram has been checked by many experimentalists and is, by now, widely accepted.

*White* [19],in 1940,studied the equilibrium of a single particle resting on a granular horizontal bed to obtain the critical tractive shear stress. He classified the flow into two categories for purpose of analysis:-

High speed case in which $Re_* > 3.5$, he defined $\eta$ to be the packing coefficient as $d^2$ times the number of particles per unit area, (i.e $\eta = Nd^2$), the critical tractive stress is given by ;

$$\tau_c= \eta(\pi/6)d\ (\gamma_s-\gamma)\ \tan\phi \qquad ...(2.21)$$

where $\phi$ being the angle of repose for the sediment grains.

For low speed case in which $Re_* < 3.5$ ;

$$\tau_c= \alpha\eta(\pi/6)d\ (\gamma_s-\gamma)\ \tan\phi \qquad ...(2.22)$$

where $\alpha$ is a coefficient contributing for low velocities. The above two equations of *White* have to be modified for a particle resting on a sloping bed, respectively, as ;

$$\tau_c= \eta(\pi/6)d\ (\gamma_s-\gamma)\ (\tan\phi - S) \qquad ...(2.23)$$

and $\tau_c = \alpha\eta(\pi/6)d\ (\gamma_s-\gamma)\ (\tan\phi - S)$ ...(2.24)

Where S is the bed slope of the channel.

For $Re_* < 3.5$, the factor $\alpha\eta$ was found to be 0.34, from which the critical tractive stress equation become for $Re_* <$ 3.5 ;

$$\tau_c/(\gamma_s-\gamma)d = 0.18 \tan\phi = \text{const.} \qquad ...(2.25)$$

This result doesn't agree with Shields' diagram as for $Re_* <$ 3.5 the tractive shear stress is almost inversly proportional to $Re_*$.The equation (2.25) was checked experimentally in a converging nozzle design, such that the shear stress remains constant along the length. For open channel a turbulent factor has been defined to be the ratio of the instantaneous shear stress and the average shear stress;the value of $\eta$,the packing factor,equal to 0.4 and $\tan\phi = 1$.Equation (2.24) above reduces to (for $Re_* > 3.5$.)

$$\tau_c/(\gamma_s-\gamma)d = 0.052 \qquad ...(2.26)$$

which agrees to some extent with Shields.

*Yalin and Karahan*[20], in 1979, developed a relation similar to Shields' Curve but with a constant value of ;

$$\tau_c/(\gamma_s-\gamma)d = 0.045 \qquad ...(2.27)$$

at large $Re_*$, instead of 0.06 of Shields.

## 2.3 THE BED-LOAD:-

The bed-load is the mode of transport in which the particle mode of movement is by rolling, sliding or sometimes saltating close to the bed of the channel, so the bed-load is commonly referred to as to be both the contact load and the saltation load. For a particular ratio of mass densities of the sediment and the fluid, the bed-load transport rate generally depends on the shear stress on the bed, residual discharge or the lift forces for a certain bed form.

A century ago *Du Boys*[21] was the first to propose a bed-load relation,which has been used subsequently by many investigators. He assumed that the bed sediment moves in a series of layers parallel to the bed, the velocity of each layer varies linearly from maximum for the top layer to zero for the lowest layer. Hence the bed-load rate on volume bases per unit width, $q_B$, is given by

$$q_B = A\,\tau\,(\tau-\tau_c) \qquad \ldots(2.28)$$

where A is a function of the sediment in motion, $\tau = \rho g R S$ is the average shear on the bed, $\tau_c$ is the critical shear stress at the point of incipient; $\rho g$ is the unit weight of water, R is the mean hydraulic radius and S is the channel gradient.

The assumptions made by *Du Boys* in derivation of equation (2.28) have been argued by *Schoklitsch*[14], in 1914, they were in disagreement with observations.

*Straub*[22], in 1935, determined the values of A and $\tau_c$ in the *Du-Boys'* equation for different sediment sizes by examining the observations of previous investigators.(in the British system of units )

$$A = 0.17/d^{3/4} \qquad \ldots(2.29)$$

For metric system *Zeller*[18], in 1963, gives a graph of conversion.

Straub's work has been criticized mainly on two accounts: (1) All the data are obtained in laboratories having small flume dimensions and were taken over a small range of grain sizes, and (2) field measurements have apparently not clarified the applicability of equation (2.29).

*O'Brien and Rindlaub*[23], in 1934, argued the *Du-Boys'* assumptions on the basis that it would produce a continuous acceleration, and the sliding layer should be kept in motion by the shear of the moving fluid which should be transfered to the bottom unchangeable;but they overlooked the frictional resistance of the lower layers which should be increased due to the weight component of the material above. Accordingly they introduced the bed-load transport rate to be in the form:-

$$q_B = a(\tau - \tau_c)^B \qquad \ldots(2.30)$$

both a and B are constants depending on sediment size. Quite independently the *U.S. Water Ways Experiment Station*[24], in 1935, found an empirical relation, taking into account, that the growth of the bed ripples leads to a reduction in the charge, whence ;

$$q_B = (1/n)[(DS - D_oS_o)/a]^m \qquad \ldots(2.31)$$

where n in the denominator stands for the reduction in the sediment charge when the bed roughness increases due to the growth of bed undulation; a and m are constants for given sand mixture, $D_oS_o$ is the depth slope product for a given sand mixture at the commencement of movement and was determined from a linear plot of $q_B^n$ against DS. For sand mixtures (0.025 mm < d < 0.56 mm) m varies from 1.5 to 1.8.

Further experiments reported by *Chang*[25], in 1939, could be represented by *DuBoys'* equation, and it was suggested to express the characteristic sediment coefficient A as a function of Manning roughness coefficient to account for the changes in the rate of transport due to the bed roughness.

*Shields*[17], in 1936, presented a useful model for sediment transport using the concept of excess shear stress in relation to the critical one. The semiempirical tractive force equation is given by:-

$$(g_B/\gamma q)(\gamma_s-\gamma/\gamma S) = 10[(\tau_o-\tau_c)/(\gamma_s-\gamma)d] \qquad ...(2.32)$$

where $g_B$ is the rate of bed-load transport on weight basis, q is the flow rate per unit width, $\gamma$ and $\gamma_s$ are the unit weight of water and sediment respectively, and S is the channel gradient.Equation (2.32) is dimensionally homogeneous and is based on data in a range of $\gamma_s/\gamma$ from 1.06 to 4.2 and sediment size from 1.56 mm to 2.47 mm. Data indicates deviation up to 200 percent[25] in the bed-load predicted by equation (2.32).

*Kalinske*[15], in 1947, proposed a rational equation for bed-load based on three important premises. Firstly, he considered that a critical tractive force is required to start the movement of sediment. Secondly, he considered the bed-load rate to be a function of the number, size and average speed of the particles in motion. Kalinske wrote the critical tractive shear stress as;

$$\tau_c = 0.233(\gamma_s-\gamma)d \qquad ...(2.33)$$

A correction factor of 0.5 was introduced to take care of the fluctuations of pressure in the wake of the particle. Accordingly

$$\tau_c = 0.116(\gamma_s-\gamma)d \qquad ...(2.34)$$

another correction was introduced in the case of turbulent flow considering that the maximum instantaneous velocity close to the bed is 1.75 times the time average velocity.Since the shear stress might be approximated to be thrice the average value, i.e

$$\tau_c = 1/3[0.116(\gamma_s-\gamma)d]$$

$$= 0.039(\gamma_s-\gamma)d \qquad ...(2.35)$$

The instantaneous velocity of a grain at the grain level $U_g$ was written as

$$U_g = b(u- u_c) \qquad ...(2.36)$$

where u is the instantaneous fluid velocity, $u_c$ is the critical fluid velocity at which the grain starts moving, the constant b was found to be unity according to experimental evidence. The rate of movement in dry weight per unit width is given by:-

$$g_s = [p_1/(\pi d^2/4)]\ (\pi/6)d^3U_g\gamma_s \qquad ...(2.37)$$

since $(p_1/(\pi/4)d^2)$ gives the number of grains per unit area so that $p_1$ is the portion of the bed taking the shear. Assuming that u varies according to the normal error law;

equation (2.36) was simplified to get

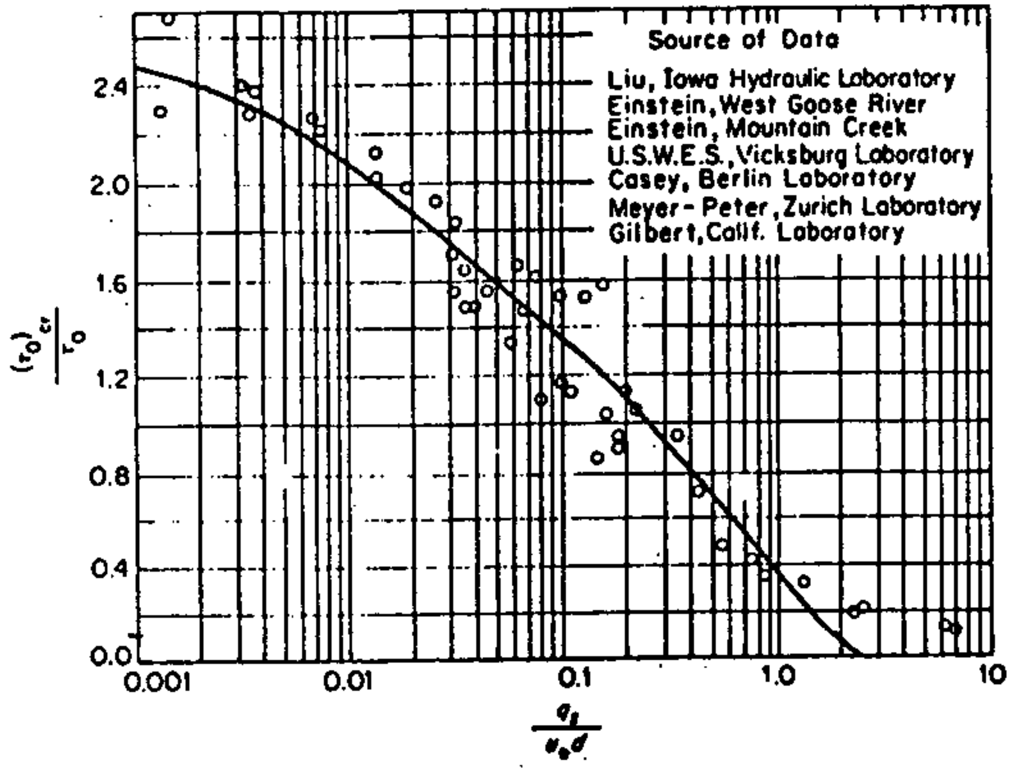
$$U_g/u = f(u_c/U,\ r) \qquad \ldots(2.38)$$

where $r = \sigma/U$, U is the average velocity at grain level and $\sigma = ((u-U)^2)^{1/2}$. But $u_c/U$ can be expressed as $(\tau_c/\tau)$ as given by Kalinske, for laminar flow r= 0, and for turbulent flow r= 0.25, assuming that $U/u_* = 11.0$ from the laws of velocity distribution, yields:-

$$g_B/u_* d\,\gamma p_1 = f(\tau_c\ /\tau) \qquad \ldots(2.39)$$

where $p_1$ is a pure number depending on the number of grains in motion relative to the number of static grain per unit area of the bed and was, unexplained, taken = 0.35.

This equation is reasonably supported by experimental data from different sources as was plotted by Kalinske. Equation (2.39) could be considered more advanced than any other *DuBoys-type* equations.See Fig.(2.2).




Kalinske's bedload equation. [*After* KALINSKE (*1947*).]

FIG. (2.2) KALINSKE'S BED-LOAD FUNCTION.

*Brown*[20] has, however, shown that the data can be described by the relation

$$g_b/u_* d = 10[\tau/(\gamma_s - \gamma)d]^2 \qquad \ldots(2.40)$$

which holds well over the upper range, but considerably deviates at the lower range of stress which conform well to Kalinske's equation.

Kalinske's assumptions that the grain speed is equal to that part of the local water speed which is in excess of the speed necessary for the commencement of movements seems to miss the facts, as shown by *Khalil*[28], in the next pages, that the grains tend to slip relative to the local fluid velocity. The analysis of Kalinske also missed the effect of the grain concentration on the speed of transport. Again the use of turbulent factor equal to 0.25 seems not to be justified in veiw of *Tison's*[27] work which indicated that the turbulent ratio is not constant but depends on many factors among them is the water depth.

*Khalil*[28], in 1963, treated the hydraulic transport of solids in an open channel as a special case of the general problem of transport by traction, in analogy with the "draw-bar pull" of the locomotive.The tangential stress at the bed of channel $\tau_b$,for two dimensional parallel flow of steady mean velocity,is due to weight component of the fluid, that is:-

$$\tau_b = \rho g \text{ D Sin S} \qquad \ldots(2.41)$$

whereas the total resistance contributed by the various element is partly due to the frictional resistance $r_f$ of moving grains, and partly due to the weight component $r_w$ of the grains themselves. To which also is added the portion of the stress $r_*$ carried directly by the static grains without contributing to transport. On this basis each resistance can be expressed as;

$$r_f = (\rho_* - \rho) g \cos S \tan \phi \int_0^y N dy \qquad ...(2.42)$$

$$r_w = -(\rho_* - \rho) g \sin S \int_0^y N dy \qquad ...(2.43)$$

$$r_* = \tau_c \; (1-(N/N_*) \qquad \text{for } \int_0^y N dy < N_* d \qquad ...(2.44)$$

where tan $\phi$ is the coefficient of friction of moving grains, N is the volume concentration,a measure to the volume of moving grains per unit volume of space,and $N_*$ is the maximum volume concentration in a single active grain layer which he found to be 0.46. The (-ve) sign in equation (2.43) is associated with downward gravity slope. The total resistance $r_t$, therefore, is simply the sum of $r_f$, $r_w$, and $r_*$, i.e

$$r_t = (\rho_* - \rho) g \cos S(\tan \phi - \tan S) \int_0^y N dy + \tau_c \; (1-(N/N_*) \qquad ...(2.45)$$

Regarding the applied stress and the resulting force their equilibrium at the surface of a flat grain bed can be used to predict the bed-load within the bed layer thickness y.

When less than one complete layer of grains is in transport, i.e. $\int_0^y N dy < N_* d$, the volume of bed-load per unit area of space is,

$$\int_0^y N dy = [\tau_b - \tau_c \; (1-(N/N_*)] / [(\rho_* - \rho) g \cos S \; (\tan \phi - \tan S)] \qquad ...(2.46)$$

When there is one or more complete layer in motion.

i.e. $\int_0^Y N dy = \tau_b / [(\rho_s - \rho) g \cos S (\tan \phi - \tan S)]$ ...(2.47)

which gives the bed-load as predicted from equilibrium of the grains just above the stationary surface.

He proved the stability of the grains just below the stationary surface by introducing the relation given by:-

$(\rho_s - \rho) g \cos S \; N_b d(\tan \phi - \tan S) > \tau_c \quad (1-(N/N_*)$ ...(2.48)

which indicates that static resistance in excess of the applied stress and the bed grains are in stable equilibrium.

The rate of bed-load transport on mass basis is defined as the total mass of grains which in unit time passes a unit width of the channel. To evaluate the average rate of transport, it is necessary to find the average speed of movement of grains. *Malouf's*[29] and then the *Khalil's* experimental work showed that the speed of a moving grain driven by a fluid stress depends not only on the grain diameter but also upon the local velocity of flow. The effective local velocity itself depends on the nature of the motion whether smooth, rough or transitional turbulent.

For smooth turbulent flow the velocity distribution has the form;

$u / u_* = 5.75 \log[y / (0.1 \nu / u_*)]$ ...(2.49)

This relation is valid when the laminar shear stress can be neglected in comparison with the turbulent stress. In the immediate vicinity of the bed where the turbulent shear stress diminishes and laminar stress predominates, *H. Reichardt*[30], who extended his measurements to include very small distance from the bed, observed that for values of $yu_*/\nu < 5$, the contribution of turbulent friction is negligible compared with laminar friction, thus the velocity distribution has the form;

$$u/u_* = y\,u_*/\nu \qquad \ldots(2.50)$$

whereas for $5 < (u_*y/\nu) < 70$, the transitional range, both contribution are of the same order. On the other hand for rough flow when $du_*/\nu > 70$ the velocity distribution is expressed in the form;

$$u/u_* = 5.75 \log (33y/d) \qquad \ldots(2.51)$$

Regarding the speed of single grains for rough flow *Khalil* calculated $V_I$ at depth equal d/2 from;

$$V_I = 5.75 \log [\{33(d_{50}/2 + d/2)\}/d_{50}] \qquad \ldots(2.52)$$

where $d_{50}$ is the median diameter of the bed material (0.705 mm in his case). According to experimental evidence, *Khalil* found that $V_g$ is proportional to $V_I$ with the average value of proportionality factor 0.85, i.e.

$$V_g = 0.85\, V_I \qquad \ldots(2.53)$$

For transitional range of $5 < (yu_*/\nu) < 70$, although the points were grouped with some scatter, the speed of the grains to water remains constant and equal to 0.85. Whereas at high value of $yu_*/\nu > 70$, the curve approach the smooth law where:-

$$V_g = 0.85\,[5.75 \log\{(d/2)/(0.1\nu/u_*)\}]\, u_* \qquad \ldots(2.54)$$

He also studied the effect of the grain concentration on their average speed,and concluded that;

$$V_g = 0.85\ [1-(0.6N/N_*)^{3/7}]\ u_* \qquad \ldots(2.55)$$

Experimental verification of equation (2.43) showed that the bed-load rate on flat bed is of the form;

$$\int_0^y N dy = [\tau_b - 0.526(1-N/0.46)]/[(\rho_s - \rho) g \tan\phi\ ] \ldots(2.56)$$

where 0.526 $N/m^2$, is the critical shear stress for the sand used (d= 0.705 mm). Thus the general form of *Khalil's* bed-load rate of transport on flat bed can be written as;

$$g_B = [\{\tau_b - \tau_c(1-N/N_*)\}/\{\cos S\ (\tan\phi - \tan S)\}] A(1-\alpha N^{3/7}) V_I \qquad \ldots(2.57)$$

when $\int_0^y N dy < N_* d$, whereas when $\int_0^y N dy \geq N_* d$ equation (2.57) reduces to,

$$g_B = [A\ \tau_b (1-\alpha N^{3/7}) V_I]\ /[\cos S\ (\tan\phi - \tan S)] \qquad \ldots(2.58)$$

where A & $\alpha$ are constants.

For small channel gradient S is small, tan S may be neglected and Cos S can be approximated to unity.

Both equations (2.56) and (2.57), were derived to estimate the bed-load transport rate for a flat bed, when ripples are formed on the bed a corrective coefficient has to be introduced. Sine on rippled surface, the total stress is transfered to the bed partly as tangential stress and partly as normal stress. According to *Einstein*(2) and *Meyer-Peter*(39), the normal stress plays nopart in the support of the bed-load and they regarded the part of the stress effective in the transport of sediment as the part of the total in excess of the form drag. However, according to *Khalil* himself, the part of the stress important in the movement of bed-load seems to be that part acting tangentially on the gentle slope of the ripple, which, no doubt, exceeds the average due to the reversed stress acting at the leeward. From which is to be deducted the part of the stress consumed in accelerating the grain up to the ripple crest; since this part of the stress cannot be regained; as the grains, in general, decelerate in the regions of no transport.

Unlike the rough flat bed where the local velocity is a function of $u_*$ , on rippled beds the local velocity as seen by Khalil and others is not a function of $u_*$ only but also of the ripple dimensions.

In view of these factors, the transport rate $g_B$ was plotted versus the value of,

$[\tau_b - \tau_c(1-N/0.46)][u_*\{1-0.6(N/0.46)^{3/7}]$

as shown in Figs. (2.3a) and (2.3b).

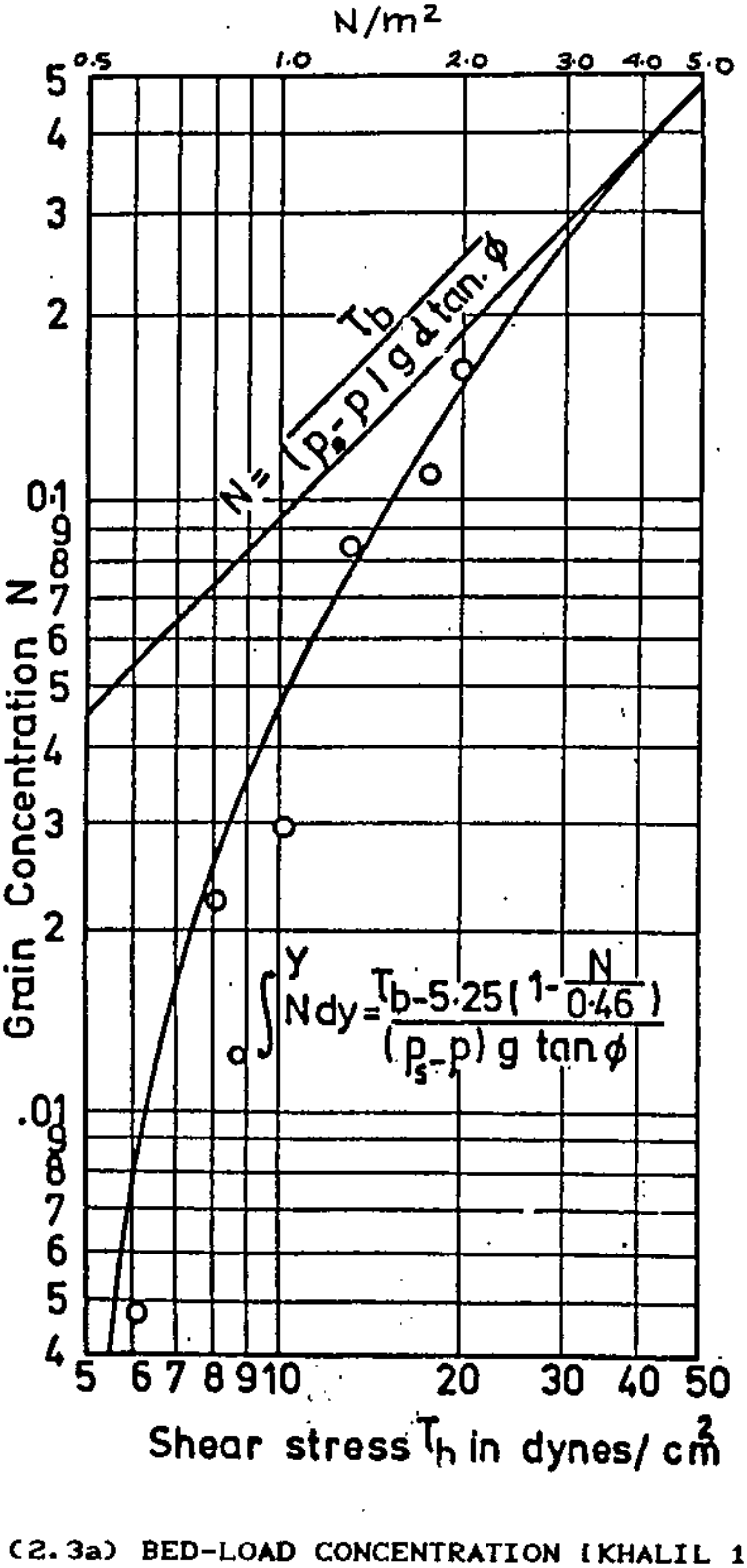



FIG. (2.3a) BED-LOAD CONCENTRATION [KHALIL 1963].

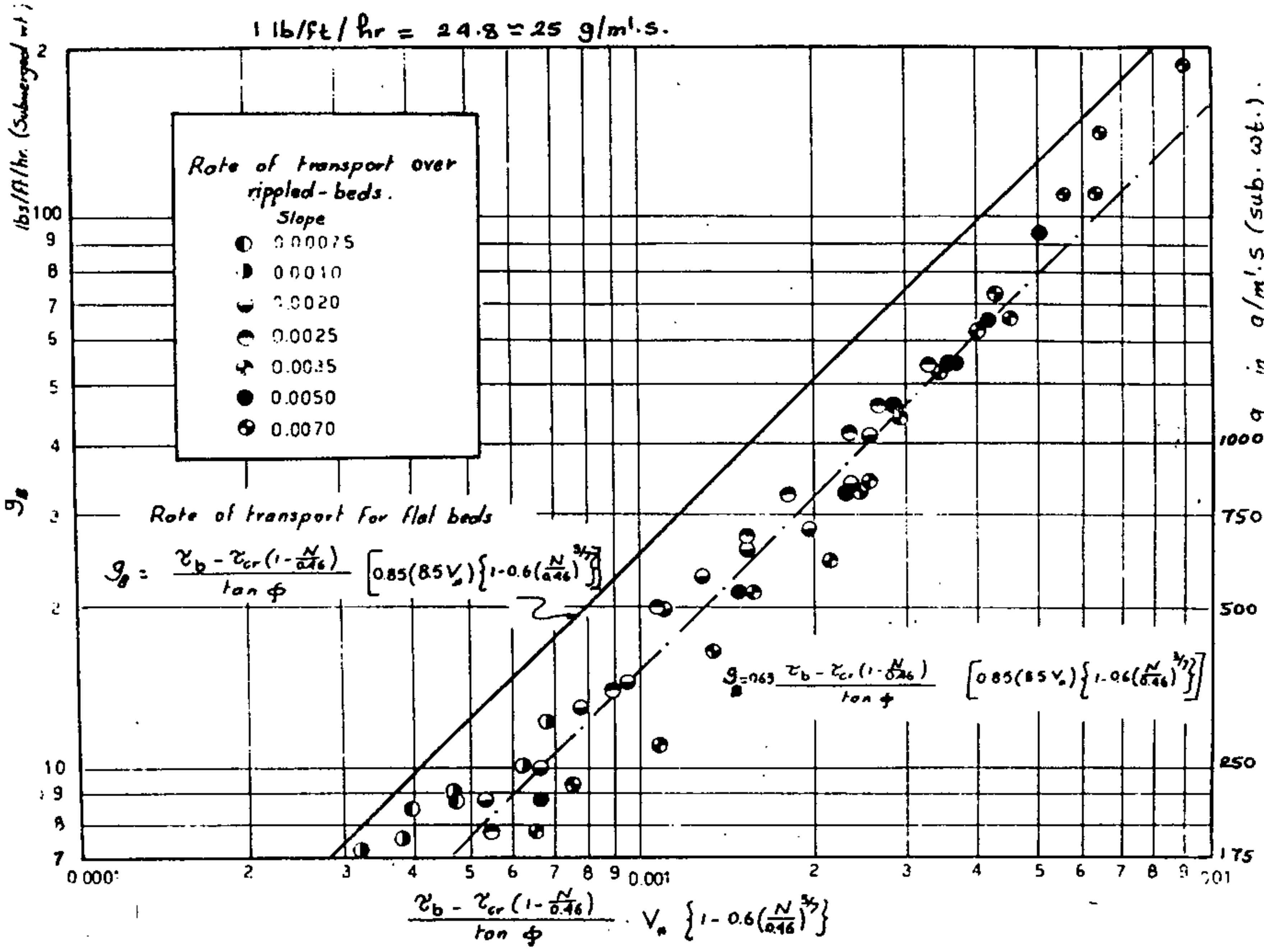



FIG. (2.3b) KHALIL'S BED-LOAD EQUATION.

A correction factor was introduced by *Khalil* to be 0.63, i.e.

$$g_B = 0.63[(\tau_b - \tau_c) / \tan \phi \, (1-N/0.46)]^* \quad [0.85 \, (8.5u_*) \, \{1-0.6(N/0.46)^{3/7}\}] \quad ...(2.59)$$

It may be concluded that the transport on a rippled bed is not like the flat as form drag, reversed stress, accelerated stress and deviation in velocity distribution all add to the deviations from the formula given for transport on flat beds.

*Rottner*[31] has also approached the problem from dimensional considerations. Five dimensionless numbers were developed for the similarity in sediment transport, using the principle of dimensional analysis and with some physical consideration,the parameters were reduced to;

$g_B/[\rho_s\{(\rho_s-\rho)/\rho(gd^3)\}^{1/2}]$ ,D/d, $U/[gd(\rho_s-\rho)/\rho]^{1/2}$, $S/[(\rho_s-\rho)/\rho]$

Rottner classified a large number of data into categories in-terms of D/d , plotted against the cube root of the grain Froude number $U/[gd(\rho_s-\rho)/\rho]^{1/2}$ and the result was represented algebraically in the form;

$$[g_B/\rho_s(gd^3\rho_s-\rho/\rho)^{1/2}]^{1/3} = a \, U/(gD \, \rho_s-\rho/\rho)^{1/2} + b \quad ...(2.60)$$

where a and b are functions of D/d. The analysis indicated an importance of the Froude number on the sediment transport but the role is not clear due to nonlinearity of the solution.

A major contribution to the knowledge of the transport of solids was made by *Gilbert and Co-workers*[32]. Their experiments carried out almost 75 years ago have been the starting point of many of the sequence works. Gilbert demonstrated the individual effect of each of the factors; slope, discharge, fineness (1/d), and the aspect ratio on the sediment charge ($r/r_m$); flow depth and velocity were considered in so far as to influence the former factors. The general relation was given as:-

$$g_B = b(S-S_c)^n(Q-Q_c)^o(1/d-1/d_c)^p(1-(m/m+1)(r_m/r)).r_m \quad ...(2.61)$$

where b is a constant, $S_c$, $Q_c$, $d_c$ are values of the respective quantities at start of motion ; n,o,p and m are indices for the various factors, $r_m$ is the optimum form ratio at which, when other things being equal, maximum transport can be obtained. Simpler form of Gilbert equation is given in *some books*[33] as:-

$$g_B = (a/d^{0.58})\ S^{1.59}\ q^{1.02} - b \quad ...(2.62)$$

*Schoklitsch*[34] *and Mac-Dougall*[35] seperately developed more than one empirical formula supported by Gilbert data as well as some data measured by them. *Schoklitsch* gave his formula based on data using uniform sand in the form;

$$g_B = (7.00/d^{1/2})\ S^{1.5}\ (q - q_c) \quad ...(2.63)$$

equation (2.36) is used in metric unit system; where $q_c$ is the critical flow rate at which bed-load starts to move given by *Schoklitsch* as;

$$q_c = 1.944\ (10^{-5}d)/S^{4/3} \quad m^3/Sec/m \quad ...(2.64)$$

also *Mac Dougall*[35], in 1934, gave his Schoklitsch type equation of the form ;

$$g_B = a\ S^b(q - q_c) \quad ...(2.65)$$

*Casey*[36], in 1935, analyzing his own laboratory data and

*Jorrissen*[37], in 1938, found that Schoklitsch's equation (2.63) is quite suitable.

Based on numerous experiments in laboratory flumes and field measurement of bed-load, *Schoklitsch*[38], in 1950, suggested a modification of his equation, the new equation is essentially of simple form, but also incorporates implicitly the critical shear stress concept. The final form of this equation is:

$$g_B = 2{,}500\ S^{3/2}(q - q_c) \qquad \ldots(2.66)$$

where $q_c$ is given by

$$q_c = 0.26\ (\gamma_s - \gamma/\gamma)\ \ d^{3/2}/S^{7/6} \qquad \ldots(2.67)$$

both equations (2.66) and (2.67) are in metric units.

*Meyer-Peter and Muller*[39], in 1948, after extensive research in E.T.H. in Zurich found an empirical relation for sand, barite and lignite; The equation for sand given in metric system is:-

$$0.4\ g_B^{3/2}/d = g_v^{2/3}\ S/d - 17 \qquad \ldots(2.68)$$

where $g_v$ is the flow rate of water in weight per unit time per unit width. For sand mixtures, the representative grain diameter was expressed as;

$$d = 1/100\ \Sigma\ d_i \Delta p_i \qquad \ldots(2.69)$$

where $\Delta p_i$ is the percent of particle size fraction of mean diameter $d_i$

*Meyer-peter et al.* found that the total shear stress is not available for sediment transport in the case of an undulated bed;a part of the shear stress is used up in overcoming the form resistance of the undulations and the bed-load transport is a function of the shear stress due to grains only. Thus they split up the slope S as;

$$S = S' + S'' \qquad ...(2.70)$$

in which S' is the slope required to overcome the grains resistance and S'' is the slope required to overcome the bed form. The value of S' was estimated using the *Manning-Strickler* formula,

$$U = 1/n_s \; R_b^{2/3} \; S'^{1/2} \qquad ...(2.71)$$

where

$$n_s = d_{90}^{1/6}/26 \qquad ...(2.72)$$

where $d_{90}$ expressed in meters

Also $$U = 1/n \; R_b^{2/3} \; S^{1/2} \qquad ...(2.73)$$

Therefore;

$$S'/S = (n_s/n)^2 \qquad ...(2.74)$$

where the ratio $n_s/n$ represents the ratio between the value of Manning-Strickler coefficient as it would be obtained on a plane bed to the actual value on a rippled bed. Accordingly *Meyer-peter and Muller* obtained the non-dimensional equation given by ;

$$[(n_s/n)^{3/2} \gamma R_b S/(\gamma_s-\gamma)d_a] - 0.047 =$$
$$[0.25\rho^{1/3}/(\gamma_s-\gamma)d_a][(g_B(\gamma_s-\gamma/\gamma)]^{2/3} \qquad ...(2.75)$$

where $d_a$ is the median diameter of the mixture.

A furthermore notice, that zero bed-load transport occured when $(n_s/n)^{3/2} \gamma R_b S/(\gamma_s-\gamma)d_a = 0.047$, which may be explained as a dimensionless critical shear stress; Thus *Chien*(40), in

1954, has shown that equation (2.75) reduced to the form of a *Du-Boy s'* type equation, and he also showed the similarity of this equation to the bed-load equation given by *Einstein*[2], 1950.

An empirical relation similar of that of Meyer-Peter and Muller was given by *Bharat Singh*[41], in 1961, as;

$$g_B = A\ [(\tau_o(n_s/n)) - B)]^{3/2} \qquad \ldots(2.76)$$

where A and B are constants depends on sediment size and relative density.

*Einstein*[42], in 1942, in formulation of his bed-load equation departed radically from the concept of the critical quantities and adopted instead,the probability concept. He introduced the idea that the grains move in steps of definite length L proportional to the diameter of the grain irrespective of the hydraulic conditions and that the rate of transport depends on the number of grains per second that pass a unit width. The latter is assumed equal to the number of grains in the surface area of unit width and length L times the probability of the local drag or lift to be strong enough to put the grains in motion. On this basis, and through combination of dimensional analysis and statistical considerations ,he deduced the transport equation of the form;

$$\phi = f(\psi) \qquad \ldots(2.77)$$

where $\phi$,the intensity of bed-load transport, is proportional to $g_B$ and involves the grain diameter, its immersed weight,and a dimensionless function of the fall velocity as;

$$\phi = g_s/\gamma_s[(\rho/(\rho_s-\rho))(1/gd^3)]^{1/2} \qquad ...(2.78)$$

and $\psi$ ,the flow intensity, is inversely proportional to $\tau$ and involves the same grain characteristics as;

$$\psi = (\rho_s-\rho/\rho)\ d/RS \qquad ...(2.79)$$

Experimental data plotted as $\psi$ versus $\phi$ on semilog scale. Data for $\phi < 0.4$ are represented by;

$$0.465\ \phi = e^{-0.391\psi} \quad , \quad \phi < 0.4 \qquad ...(2.80)$$

as shown in Fig.(2.4). For large rate of transport the data deviates from the straight line relationship.

Brown[26], in 1950, by logarithmic plotting of $\phi$ against $\psi$ showed that,whereas Einstein found that uniform materials yielded a linear semi-logarithmic function, the same can be expressed as ;

$$\phi = 40(1/\psi)^3 \qquad ...(2.81)$$

*Einstein*[2], in 1950, further developed his theory and gave a new equation based on the concept that the continuous interchange between the active and static grains should be in such a way that the rate of deposition is equal to the rate of erosion.He also defined the exchange probaliblity, p,to be dependent on the fraction of the moving grains of a given grain diameter in the mixture, $i_B$, on the average distance travelled by the grain which was taken ,as before, to be proprotional to the grain size, and on the fraction of a given grain size existing in the bed, $i_b$.

On,these bases,and other considerations of the lift being the sole agent in initiating the grain motion ; also the probability of the lift to exceed the grain submerged weight is in accordance with the normal error law, Einstein deduced that;

$$p/1-p - A_*(i_B/i_b)\ \phi = A_*\phi_* \qquad ...(2.82)$$

where p is the probability of exchange (erosion). $A_*$ is a constant to be determined by experiments. and $\phi_*$ is the intensity of transport for an individual grain size,other terms are defined earlier in this review.

The probability of erosion p may be expressed as the probability of the ratio of the effective weight to the instantaneous lift, which has to be smaller than unity:

$$1 > (k_2(\rho_s-\rho)gd^3)/c_L(1/2\rho k_1 d^2 u_b^2(1+\eta)) \qquad ...(2.83)$$

In an extensive investigation, the coefficient of lift, $c_L$, was found to be 0.178, and the random function, $\eta$, is distributed according to the normal error law, where the standard deviation, $\eta_o$, is a universal constant of $\eta_o$= 1/2. The velocity, $u_b$, was found to be at a distance of 0.35 X from the theoretical bed, where X is the characteristic grain size of the mixture ,and $k_1$,$k_2$ are constants of the particle area and volume; respectively. If $u_b$ is expressed with the law of logarithmic velocity distribution, and introducing a new definitions , as the laminar sublayer thickness, $\delta$ = 11.5 $\nu/u_*$; the apparent grain roughness diameter, $\Delta$ which was given by a graph due to Einstien, $\Delta$= $d_{65}$ for rough wall. Hence X was defined to be X= 0.77 $\Delta$ if $\Delta/\delta$ > 1.8, and X= 1.39$\delta$ if $\Delta/\delta$ > 1.8, then $u_b$ may be expressed as;

$$u_b = u_* \; 5.75 \log [30.2(0.35)X/\Delta] \qquad ...(2.84)$$

Then introducing all the values in equation (2.83) with some abbreviations, yields;

$$1 > 1/(1+\eta) \; \psi \; B \; \beta_*^{-2} \qquad ...(2.85)$$

where $\psi$ is the flow intensity, B is a constant of $\psi$ scale, and $\beta_* = \log(10.6 \; X/\Delta)$. Furthermore *Einstien* suggested to introduce two correction factors, namely $\xi$ the factor account for sheltering of smaller particles and known as hiding factor, which was taken to be unity for d/X > 1.5; and Y is the factor describing the change of the lift coefficient in mixtures with various roughness. It is a function of $d_{65}/\delta$ and known as pressure correction factor. For uniform grains the two factors have a value of unity. Introducing these factors in equation (2.85) Einstein deduced that;

$$|1+\eta| > \xi \; Y \; B'\beta^2/\beta_*^2\psi \quad , \qquad ...(2.86)$$

whereas $\eta$ may be either positive or negative and $\beta = \log 10.6$

Rewriting equation (2.86) with $\eta = \eta_o \eta_*$.

$$\beta_* = B'/\eta_o, \qquad \psi = \xi \; Y(\beta/\beta_*)^2 \; \psi$$

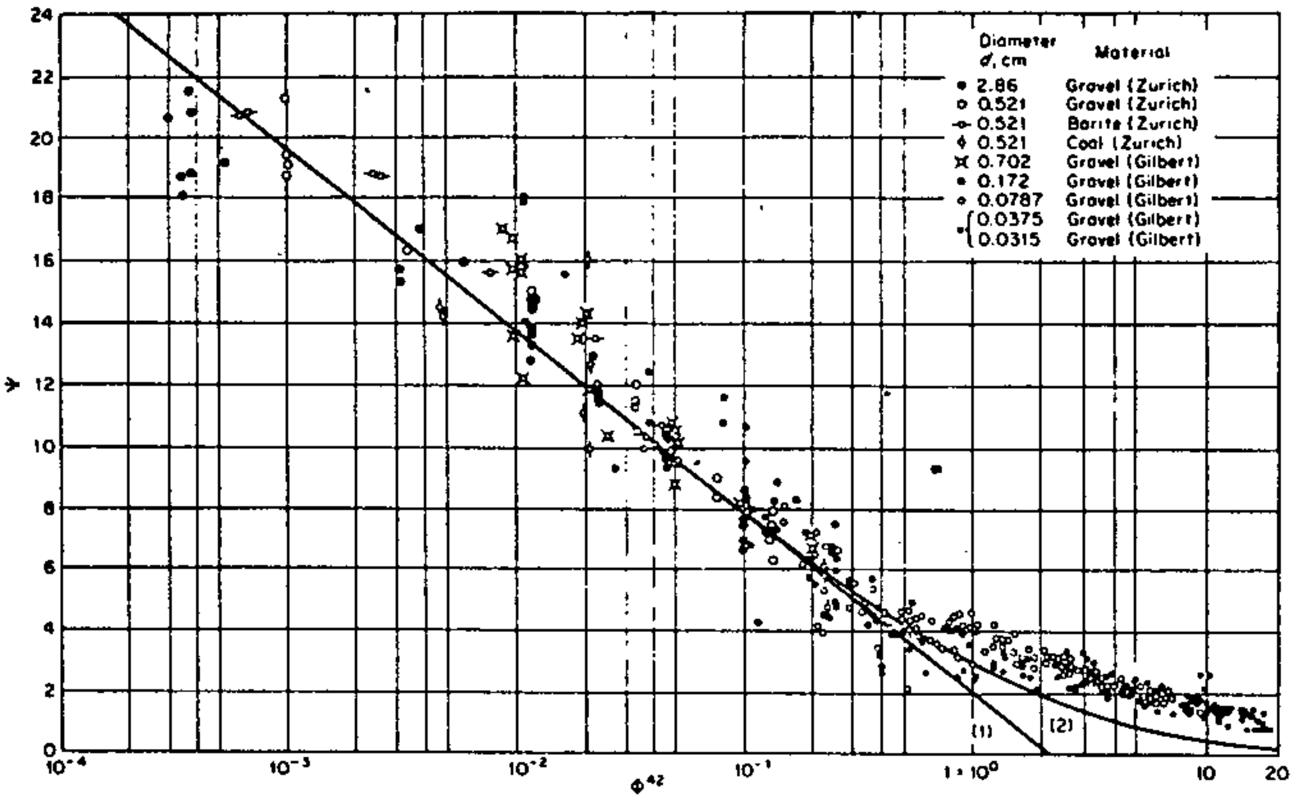
and, as mentioned before,that probability distribution is according to the norml error law ; The second bed-load equation suggested by *Einstein (1950)*, was of the form;

$$1 - (1/\sqrt{\pi}) \int_{-\beta_*\psi_* - 1/\eta_o}^{+\beta_*\psi_* - 1/\eta_o} e^{-t^2} \, dt = A_*\phi_*/1+A_*\phi_* \qquad ...(2.87)$$

where t is the only variable of integration, $A_*$, $\beta_*$, and $\eta_o$ are universal constants determined by experiments to be 43.5, 0.143 and 0.5 respectively and $\psi_*$ and $\phi_*$ as defined earlier as:

$$\phi_* = (\iota_m/\iota_b)\ \phi \qquad \text{...(2.88)}$$

$$\psi_* = \xi\ \Upsilon(\beta/\beta_*)^2\ \psi \qquad \text{...(2.89)}$$




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

FIG. (2.4) EINSTEIN'S BED-LOAD EQUATION.

Although many verifications on Einstein's method have been made in the last four decades by comparing observed and computed bed-loads; *Khalil*[28], in 1963, found that the physical reasoning of Einstein's theory is some what difficult to visualise; as for a rippled bed the scour exceeds the deposition over the gentle slope while the deposition exceeds the scour at the steep side and therefore the basic concept is not as envisaged by Einstein. Also the attribution of the initiation of grain motion to be solely due to lift forces seems not to be justified; the drag and the friction stress, no doubt, have their important role.

Another point, which *Khalil*[28] showed its disagreement with observations, regarding the grains moving in steps of definite length depending only on its diameter, he showed that the path of grains coated with fluorescent and photographically recorded indicated that the length of these steps for small grains statistically increased with $u_*$.

*Pemberton*[43], in 1972, has used Einstein's relationship to predict the total sediment transport rates of three American rivers. The rates of sediment transport thus computed were not in agreement with the measured ones. The computed transport rates were higher depending on the size fraction.

*Chien*[40], in 1954, studied Einstein's bed-load equation and Meyer-Peter et al. equation, being the most complete equations available, and showed how they do compare. As far as hydraulics is concerned, the difference lies in how the total frictional resistance is divided. Einstein divided the hydraulic radius, $R_h = R'_h + R''_h$, and kept the slope constant, whereas Meyer-Peter et al. divided the slope, $S = S' + S''$, and kept the hydraulic radius constant. Both assumptions facilitate the understanding of superposition of the effects due to grain roughness and bedform. Under these conditions *Chien, 1954*, has shown that the Meyer-Peter and Muller equation can be written in terms of $\phi$ and $\psi$ as ;

$$\phi = [(4/\psi) - 0.188)]^{3/2} \qquad ...(2.90)$$

Agreement between the two relations was found to be equally good.

Based on analysis simillar to Einstein's; *Sato, Kikkawa, and Ashida*[44], in 1958, developed semitheoretical equation for bed-load transport of uniform material. The final equations obtained by plotting data collected by them and those collected by Gilbert are as follows:

$$q_B/\tau_o u_* f(\tau_c\ /\tau_o) = (1/40n)^{3.5} \quad , \text{ for } n \le 0.025 \qquad ...(2.91)$$

$$q_B/\tau_o u_* f(\tau_c\ /\tau_o) = 1 \quad , \text{ for } n \ge 0.025 \qquad ...(2.92)$$

where the values of $\tau_c$ and $f(\tau_c/\tau_o)$ were given in tabular form. For large shear stresses, $(\tau_c\ /\tau_o)$ will be very small and in such case equation (2.92) can be approximated by:-

$$q_B = \tau_* u_* \qquad ...(2.93)$$

*Bagnold*[45], in 1956, used the principles of mechanics to present a semitheoretical approach. He considered the concept of dispersion of solid particles under shear, and that the total resistance to be the sum of the fluid shear stress at the boundary and the shear stress due to the collision of sediment particles. Based on these premises he used the concept useful work done by the fluid to transport a mass of grains per unit time per unit width and introduced an efficiency factor to account for the slip of the grain relative to the fluid.

The mass of grains passing the cross section per unit width and time is:-

$$g_{sm} = \rho_s \int_o^{y_o} cu\,dy \qquad \ldots(2.94)$$

and the available work is proportional to $(\tau_o - \tau_c)u_*$, and the useful work done is

$$W = A_b(\tau_o - \tau_c)u_*\ \eta \qquad \ldots(2.95)$$

where $\eta$ is the efficiency of transport, A is a constant depending on the Reynolds number, the friction factor between the grains and the bed, and the efficiency. His final equation converted to Einstein's coordinates becomes of the form:-

$$\phi = A^1 \Theta^{1/2}(\Theta - \Theta_c) \qquad \ldots(2.96)$$

for transport by wind $\Theta_c$ becomes zero as soon as saltation starts.

*Bagnold*[46], in 1966, simplified his (1956) earlier approach so that it is more directly applicable. This will be discussed in more details under the "total load" review.

*Yalin*[47], in 1963 and 1972, expressed the bed-load transport rate as

$$g_s = W_b U_g \quad ...(2.97)$$

where $W_b$ is the weight of granular material moving over a unit area of the bed surface at a velocity $U_g$. He developed expressions for $U_s$ and $W_s$, his transport equation is:

$$g_s/\gamma_s^* d u_* = \text{const } \{s[1-(1/as)\ln(1+as)]\} \quad ...(2.98)$$

where $s = u_*^2/u_{*c}^2 - 1$,

and $\theta = \rho u_*^2/\gamma_s^* d$

$$a = 2.45(\rho u_{*c}^2/\gamma_s^* d)^{0.5}/(\rho_s/\rho)^{0.4} = 1.66\sqrt{\theta}$$

when $\rho_s/\rho = 2.65$, the constant was determined experimentally to be 0.635. His formula can be converted to Einstein's coordinates as

$$\phi = f(\psi, \theta_c, \rho_s/\rho) \quad ...(2.99)$$

For $\rho_s/\rho$= const., and at the beginning of transport $\theta - \theta_c$ is very small, then ;

$$1/as \ln(1+as) = 1-(0.5)as$$

Thus *Yalin's* equation becomes;

$$\phi = 1/2 \text{ const. } as^2/\sqrt{\psi} = (a.\text{const.}/2\ \theta_c^2)\theta^{1/2}(\theta-\theta_c)^2 \quad ...(2.100a)$$

At large values of shear stress $\theta_o$, as $\rightarrow \infty$ and the term [1/s ln(1+as)]approaches zero. Then Yalin's equation becomes:-

$$\phi = \text{const}/\theta_c\ \theta_o^{1/2}(\theta_o-\theta_c) \quad ...(2.100b)$$

The latter is identical to Bagnold equation (2.96).

*Engelund and Fredsoe*[48], in 1976, followed some of the concepts introduced by Khalil[28], in 1963, by equating the activating force on the grain and the resisting force, in that case the grain moves at a constant velocity, the grain velocity $U_g$ is taken as:

$$U_g/u_* = \alpha(1-0.7\ (\tau_{*c}/\tau_*)^{1/2}) \qquad ...(2.101)$$

with $\alpha = 9.0$ and $\tau_{*c}$ obtained from Shield's curve. Further they obtained a bed-load transport equation as:

$$g_B = (\pi/6)d^3(p/d^2)\ U_g\gamma_* \qquad ...(2.102)$$

in which $g_B$ is the transport rate on weight bases, p is the probability of the movement of grains in the surface layer, and $1/d^2$ represents the number of grains per unit area. Using Einstein's coordinates equation (2.102) can be written as:

$$\phi = 5p(\tau_*-0.7\tau_{*c}) \qquad ...(2.103)$$

An expression for p was obtained on the assumption that for plane bed, $(\tau_o-\tau_c)$ is transmitted as the drag on the moving particles, if N is the number of particles moving per unit area then, they wrote,

$$(\tau_o-\tau_c) = N(\pi/6)d^3(\rho_*-\rho)\ g\ \beta \qquad ...(2.104)$$

where $\beta$ is the coefficient of dynamic friction and was taken to be 0.51, or equation (2.104) can be written as:

$$\tau_*-\tau_{*c} = (\pi/6)\beta\ Nd^2 \qquad ...(2.105)$$

or $$\tau_*-\tau_{*c} = (\pi/6)\beta p \qquad ...(2.106)$$

Hence

$$\tau_* = \tau_{*c} + 0.2668p$$

in which $\tau_*$ is taken as 0.05, which is a conservative value from Shield's curve.

By equating the energy available for bed-load transport to the rate of work done on the sediment particles and assuming that the particles travelling as bed-load expended maximum energy *Garg and Co-worker*[44], in 1971, derived the following equation

$$q_B = 3(\tau_o' - \tau_c) / (C_D \rho u)) d \gamma_s \qquad ...(2.107)$$

where u is the velocity of the stream at a distance 0.35 of the depth from the bed, and $C_D$ is the drag coefficient for the sediment particle. By using the vlaue of $C_D$ for a sphere placed in an infinte uniform flow, they obtained fair agreement between the above equations and fields data for a sediment size from 0.33 mm to 0.6 mm. However, the value of $C_D$ used by them needs more verification since the drag coefficient is different for the rolling sphere on the bed from that assumed by them.

*Amin and Murphy*[50], in 1981, evaluated two bed-load formulae, namely the *Meyer-Peter and Muller*[39] formula, and the *Toffaleti*[51] formula, which will be reviewed later, for predicting the bed-load The evaluation is done by measuring both the hydraulical data needed for the formula and also the actual sediment load transport rate, and then comparing the predicted and the observed values.

They related the bed-load transport rate to some important hydraulic parameters as the flow rate q, the bed shear stress $\tau_o$, and the flow velocity V, the resulting power equations were (in SI units):-

$$q_B = 0.0594\ q^{4.27} \qquad ...(2.108)$$

$$q_B = 0.00394\ (\tau_o V)^{2.79} \qquad ...(2.109)$$

and

$$q_B = 5.25\ V^{9.86} \qquad ...(2.110)$$

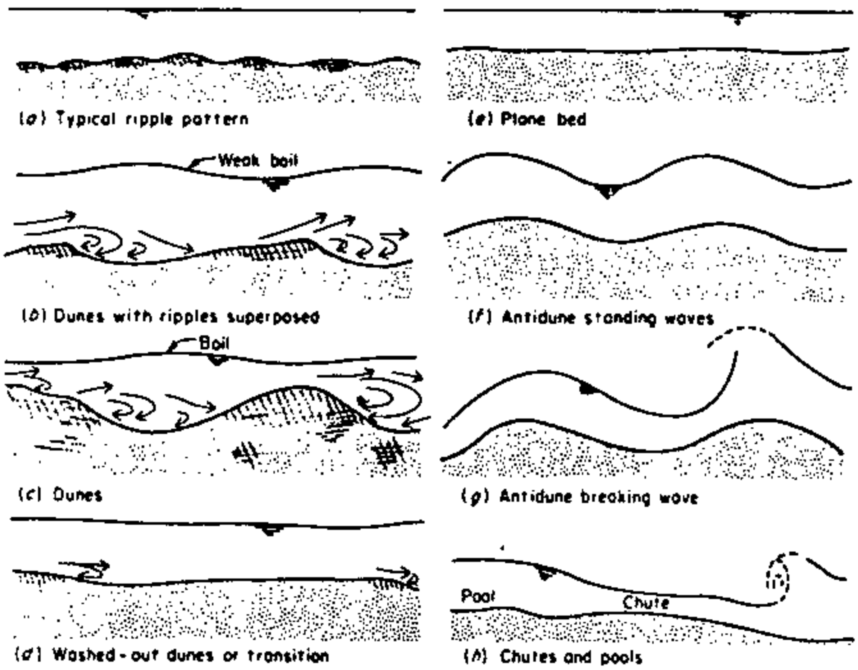
They concluded, according to experimental evidence, that the Toffaleti method predicted the bed-load rate more accurately than the Meyer-Peter and Müller equation for sand bed channels.

Some of the bed-load equation are based on the bed-form motion. Bed-forms can be defined as the features developed on the bed of an alluvial channel due to the flow of water. These bed features are sometimes called the regimes of flow, or bed irregularities.

*Alberston, Simons and Richardson*[52], in 1961, and later *Simons and Richardson*[53,54], in 1961 and 1962, have given a complete description of the different regimes of flow that are observed in an alluvial channel. For convenience these regimes can be divided into three categories:-

Lower regime at which the Froude number= $u/((gD)^{1/2})$ is less than 1, upper regime of which Fe >1 and a transitional regime in between with Fe $\simeq$ 1. A fairly widely used classification for description of bed feature ranges from ripples, dunes, wavy bed, transition flat bed and antidunes.

Idealized sketches of various bed forms are shown in Fig. (2.5)




Idealized bedforms in alluvial channels. [*After* SIMONS *et al.* (*1961*).]

FIG.(2.5) IDEALIZED BEDFORMS IN ALLUVIAL CHANNELS.

Ripples are bed features which are formed at relatively low values of shear stress in excess of the critical shear stress, usually ripples are linked with fine grained materials (d less than 0.7 mm).

The ripple height h, and the ripple length $\lambda$, was studied by *Yalin*(55), in 1972, using dimensional reasoning as;

$$h/\lambda = f(\theta/\theta_c , Re_* , D/d) \qquad ...(2.111)$$

where $\theta$ is the dimensionless shear stress= $\tau_o/\gamma(s-1)d$, as stated earlier, the role of D/d can be neglected and for $Re_* < 14$, the above equation reduces to;

$$h/\lambda = f(\theta/\theta_c) \qquad ...(2.112)$$

The length of the ripple, by observation was found to be in the order of a thousand grain diameter, i.e.

$$\lambda_r = 1000\ d \qquad ...(2.113)$$

When the discharge is further increased, the ripples grow into dunes, which are triangular undulations on the bed but they are much larger in size than ripples.

The bed with dunes, is generally very soft with considerable segregation of the bed materials, with some stratifications. The size of dunes is strongly related to the depth of flow. Yalin(55), in 1972, also discussed the conditions of dune formation, being out of phase with the water surface, he found that the wave length $\lambda$ is proportional to the flow depth D, say:

$$\lambda = 2\pi D \qquad ...(2.114)$$

A family of curves, for different $Re_*$, were plotted to relate $\lambda/d$ and $D/d$. Another plot showed the dune steepness $h/\lambda$ VS. $\theta/\theta_c$. the maximum steepness observed to be around $h/\lambda = 0.06$.

Anti dunes are bed features which are uniquely associated with free surface waves, these are more symmetrical than dunes, its crest is observed to migrate upstream.

The effects of flow regimes on hydraulic characteristics is of great importance in studying the resistance to flow, stage discharge relation and the sediment transport rate.

In the lower regime,the bed material moves mainly as bed-load; whereas in the upper regime the main mode of transport is suspension.

The basic differential equation of sediment transport with bed forms is given by *Exner*[56],in 1925,known as the erosion equation.

$$\partial y/\partial t + [1/(1-m)]\partial q_B/\partial x = 0 \qquad ...(2.115)$$

where y= elevation of the sand bed above a horizontal datum.

x = direction of flow.

m = porosity of the sand.

using a transformation given by

$$\delta = x - V_B t \qquad ...(2.116)$$

where $V_B$ is the velocity of sand ridges. Rearranging the above two equations, we have;

$$q_B = (1-m)V_B y + C \qquad ...(2.117)$$

Where C is a constant of integration, assuming ridges to be triangular of a height $\Delta H$, the above equation becomes;

$$q_B = (1-m)V_B(\Delta H/2) + C \qquad ...(2.118)$$

The above equation was given by *Hansen*[57], in 1966.

*Khalil*[28], in 1963, introduced a direct relation between the ripple velocity and the rate of bed-load transport as

$$g_B = \beta\ V_r\ y + const. \qquad ...(2.119)$$

where $\beta$ is the bulk specific submerged weight of the bed material, $V_r$ is the ripple velocity.

To evaluate the constant of integration, the origin is selected at the point of zero scour, which defines the demarcation border between forward surface creep and backward surface creep. Thus equation (2.119) can be written as

$$g_B = \beta . V_r y \qquad ...(2.120)$$

The rate of transport per unit width is defined as the mass average of grains which in unit time passes a fixed cross section. According to this definition, y is supposed to vary for a fixed cross section with respect to time from a minimum = 0 to a maximum = h. The elevation y varies with respecct to x according to the ripple shape as

$$\Sigma\ yx\ /\Sigma\ x = \alpha\ h. \qquad ...(2.121)$$

where $\alpha$ is a constant for a particular shape of ripple, varies from 0.5 for triangular shape to 0.67 for parabolic shape of bed-form. According to the above considerations, and neglecting any saltating rate, the observation of Khalil gave the following equation:

$$g_B = 0.565\ \beta . V_r\ h \qquad ...(2.122)$$

indicating that the ripple shape factor $\alpha = 0.565$

Thus he concluded that the bed-load transport rate due to wave motion, when there is no saltation, can be predicted from a simple measurement of a ripple height and its velocity.

Recently, *Engel and Lau*[58,59], in 1980 and 1981, introduced the concept of bed-load discharge coefficient, K, to compute the bed-load transport rate from bed profiles of migrating dunes, their final equation can be written as:

$$q_B = K \bar{\xi} U_w \qquad \ldots(2.123)$$

where $\bar{\xi}$ = the average departure of the bed elevation about the mean bed elevation, $U_w$=the average dune migration speed.

The value of K depends on the dune shape; dune steepness and less sensitive to change in grain size of the bed material. For maximum dune steepness of 0.06, the value of K was found to be 1.32.

## 2.4. SUSPENDED LOAD TRANSPORT:-

### 2.4.1. INTRODUCTION:-

Suspended load, as defined earlier, is the material moving in suspension, and being kept in suspension by the turbulent fluctuations of the flowing fluid. These particles held into suspension travel with a velocity almost equal to the flow velocity. Suspended load transport is an advanced stage of the bed-load transport. Thus in case of uniform sediment, one would expect only bed-load transport at low shear stress; while at higher shear stress both bed-load and suspended load transport would occur. In case of nonuniform sediment, the finer sizes of the bed material may be thrown into suspension, while the coarser sizes may move mostly as bed-load, if they move at all. An active interchange between the bed-load and suspended load occurs.

### 2.4.2. THE FALL VELOCITY:-

Before going further, it seems useful to have a brief review about the *fall velocity or the settling velocity*.

The fall or settling velocity of the sediment particles is one of the most important parameters in all sediment transport problems. This subject will be discussed briefly without going into analytical details.

It was found that the fall velocity of a particle is a function of size, shape and density of the sediment particle as well as the viscosity and extent of the fluid in which it falls; in addition to that, fall velocity depends on the number (concentration ) of the particles falling, and on the level of turbulence intensity which occurs when settling takes place in flowing fluid. Falling under the influence of gravity the particle will reach a constant velocity, called the terminal velocity, when the drag equals the submerged weight of the particle. In the case of spherical particle the equilibrium equation is :.

$$C_D \pi (d^2/4)\ \rho(\omega^2/2) = \pi/6\ d^3\ g(\rho_s - \rho) \qquad ...(2.124)$$

or, $$\omega^2 = (4/3)(1/C_D)\ gd(\rho_s - \rho) \qquad ...(2.125)$$

where $\omega$ is the terminal velocity, $C_D$ is the coefficient of drag. Thus the problem reduces by equation (2.125) to find the value of $C_D$ for the particle in question.

*Stokes*[60] found that for spherical particles of diameter d falling in an infinite liquid, the drag coefficient is fairly well defined. In Laminar flow region, where $Re < 0.5$ the Stokes' solution is:

$$C_D = 24/Re \qquad ...(2.126)$$

where $Re = \omega(d/\nu)$, the Stokes' range was extended by *Oseen*[61], in 1927, in which he took partly the inertia term of his solution to Stokes' equation as;

$$C_D = (24/Re)\ [1 + (3/16)\ Re] \qquad ...(2.127)$$

valid for $Re < 5.0$.

The complete solution was given by *Goldstien*[62], in 1929, as;

$$C_D = 24/Re[1+(3/16)Re-(19/1280)Re^2+(71/20480)Re^3+...] \quad ...(2.128)$$

which is valid for $Re \leq 2.0$.

*Shiller and Naumann*[63], in 1933, suggested a formula that gives good results for $Re < 800$ as;

$$C_D = (24/Re) + 2 \quad ...(2.129)$$

Also *Rubey*[64], in 1933, generalized Stokes' law into a general form of:-

$$C_D = (24/Re)(1+0.150\ Re^{0.687}) \quad ...(2.130)$$

*Dallavalle*[65], in 1943, suggested that $C_D$ can be expressed for different values of Re as;

$$C_D = (24.4/Re) + 0.4 \quad ...(2.130')$$

*Torobin et al*[66], in 1959, equation reads as;

$$C_D = (24/Re)(1 + 0.147\ Re^{0.69} + 0.0026\ Re^{1.38}) \quad ...(2.131)$$

which is valid accurately for $1 < Re < 100$.

*Olson*[67], in 1961, suggested , for $Re < 100$, an equation of the form;

$$C_D = 24/-Re(1 + (3/16)Re)^{1/2} \quad ...(2.132)$$

All the above mentioned equations are based on data considering a single sphere falling in an infinite extent calm fluid. In practice this is not the normal case; so correction factors will be introuduced to account for the various effects. The effect of particle shape on fall velocity was studied by *Mc Nown et al*[69], in 1950, by comparing the $C_D$ for a sphere and circular disc in the Stokes' range as;

Sphere, $C_D = 24/Re$ }
Circular disc, $C_D = 20.37/Re$ } ...(2.133)

*Lamb*[69], in 1954, introduced a correction factor for circular cylinder drag coefficient as;

$$K = 1/(2.0 - \log Re) \qquad ...(2.134)$$

which is valid in the Stokes' range.

Recently, many investigators, as reviewed by *Graf*[72], in 1971, considered the effect of shape factor and sphericity on the fall velocity; shape factor is defined by *Mc Nown et al.*[68] as, $a/\sqrt{bc}$ where a being the shortest of the three perpendicular axes. The sphericity, $\psi$, is defined as the surface area of the particle, A, to that of a sphere having the same volume, $A_s$, i.e. $\psi = A/A_s$.

If the fluid is extermely bounded, then the value of drag coefficient depends on the distance between the particle and the boundaries. *Brenner*[70], in 1961, showed that when the sphere is approaching the bottom of the container, in the Stokes' range, the drag force has to be multiplied by a factor K as;

$$K = 1 + (9/8)(r/s) \qquad ...(2.135)$$

where r is the radius of the sphere, s is the distance from the center of the sphere to the fixed boundary. Equation (2.135) is valid for solid bottom plane. If the plane is not solid, like an interface between two liquids; the K value is given by Brenner as ;

$$K = 1 + (3/4)(r/s) \qquad ...(2.136)$$

For a sphere: falling near a single vertical wall, in the Stokes' range *Mc Nown et al*[71], in 1951, gave that;

$$K = 1 + (18\ r)/(32\ s) \qquad \ldots(2.137)$$

and half-way between two plane walls K is given by

$$K = 1 + 1.006\ r/s \qquad \ldots(2.138)$$

For spherical particles falling on the axis of the cylinder, *Happel and Brenner*[73], in 1965, showed that;

$$K = 1 + 2.1\ r/R \qquad \ldots(2.139)$$

where R is the radius of the cylinder.

The next complication arises from the effect of concentration on the fall velocity. Let $K_c$ be the correction factor for the effect of concentration on the fall velocity so that $K_c = \omega_o/\omega$ where $\omega_o$ is the single particle fall velocity, and $\omega$ is the observed fall velocity during the fall of number of particles. *Maude and Whitmore*[74], in 1958, presented the most extensive study on this topic, commonly referred as hindered settling, regardless the Re value $K_c$ is given by:-

$$K_c = (1 - C)^{-\beta} \qquad \ldots(2.140)$$

or $$\omega = \omega_o\ (1 - C)^{+\beta} \qquad \ldots(2.141)$$

where C is the concentration per volume of solid particle and $\beta$ is a function of particle shape, size distribution and Re. $\beta$ ranges from 4.65 for Re $<$ 1 to 2.32 for Re $>$ 1000.

For dilute suspensions *Happel et al*[73], in 1965, discussed various models; the results are best summerized by;

$$K_c = 1 + h\, C^{1/3} \qquad \ldots(2.142)$$

where h varing from 1.30 to 1.91 with an average value of 1.56. Equation (2.142) stated that a 1% volume concentration will yield $K_c$=1.336 or there will be a reduction in the fall velocity by 25%. Average experimental values for fall velocities of quartz grains in water at 20°c in *Laminar* motion are given by Raudkivi[75], for $d \le 0.15$mm.

$$\omega = 660\, d^{2.022} \qquad \ldots(2.143)$$

or,
$$\omega = 663\, d^{2} \qquad \ldots(2.144)$$

and for turbulent motion , with $d \ge 1.5$ mm

$$\omega = 134.5(d)^{1/2} \qquad \ldots(2.145)$$

Combination of various effects can be done according to the principle of superposition, however no clear experimental evidence seems to exist.

## 2.4.3. THE SUSPENDED LOAD EQUATIONS:-

Observations have shown that the concentration of suspended load vertically decrease with the increase in the distance from the bed. The concentration of the suspended load can be expressed in various ways as:-

(1) Absolute volume of solids per unit volume of water-sediment mixture, the volume of solids can be obtained by determining the dry weight of solids dividing this by the specific weight of solids, for example part per million or percent.

(2) Dry weight of solids per unit volume of mixture for example gram per litre.

(3) Dry weight of solids per unit weight of mixture, this is customarily expressed in parts per million (ppm), one percent equals 10,000 ppm.

As mentioned earlier, three approaches will be considered in the analysis of suspended load mechanism,namely the diffusion-dispersion model, the gravitational (energy) model ,and the statistical model.

The majority of the analytical treatments are based on the concept of diffusion. Diffusion is the spreading of a fluid in another fluid of the same or smaller density , neutrally bouyant particles caused by random molecular action or by turbulent mixing. In continuum physical, molecular diffusion is governed by Fick's law as ;

$$P = -D\ (\partial c / \partial y) \qquad \ldots(2.146)$$

where P= the rate at which the quantity or property is transported across unit area normal to the y-direction; D= coefficient of diffusion,or diffusivity; c = concentration of some quantity transported by diffusion.

The simple diffusion equation of foreign particles (sediment) in a fluid was derived by *Dobbins*[70], in 1943, and other reseachers as follows:

Consider an elementary cube of sides $\delta x$, $\delta y$, and $\delta z$ and let $\epsilon_x$, $\epsilon_y$ and $\epsilon_z$ be the sediment diffusion coefficients for the diffusion along the x,y and z axes respectively. Further, let $v_1$, $v_2$, $v_3$ be the time-averaged velocities in the three directions, Let c be the sediment concentration and $\omega$ is the settling velocity. The inflow and outflow of sediment flux per unit time through various faces will be as shown in Fig .(2.6).

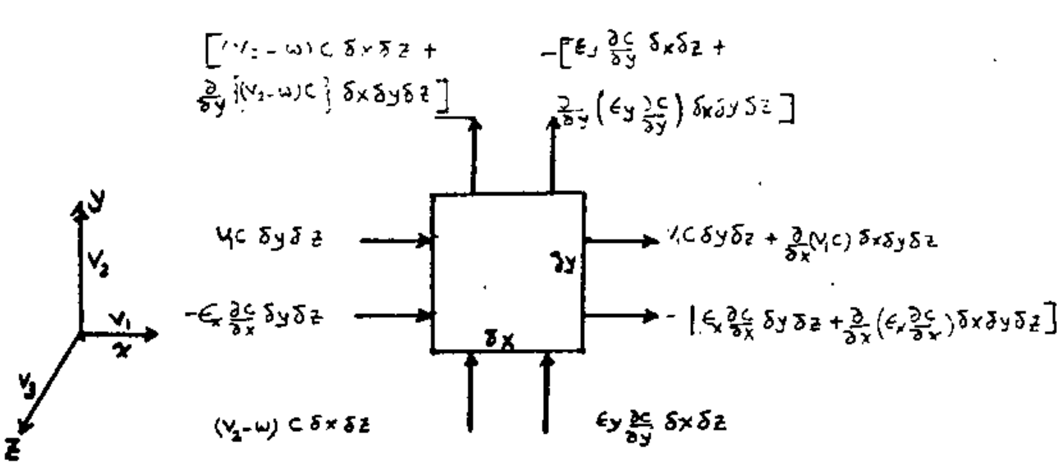



Fig.(2.6):Inflow and outflow of sediment flux.

Then equating the total rate of change of sediment in the volume ($\delta x \delta y \delta z$) to the change per unit time due to diffusion, the result is;

$$\partial c/\partial t \; \delta x \delta y \delta z = [-\partial/\partial x \; v_1 c + \partial/\partial x(\epsilon_x \partial c/\partial x) - \partial/\partial y \; \{(v_2 - \omega)c\} + \partial/\partial y(\epsilon_y \partial c/\partial y) - \partial/\partial t(v_3 c) + \partial/\partial z(\epsilon_z \partial c/\partial z)] \quad ...(2.147)$$

Rearranging equation (2.147) above and dividing by $\delta x \delta y \delta z$, one gets:

$$\partial c/\partial t + \partial/\partial x(v_1 c) + \partial/\partial y(v_2 c) + \partial/\partial z(v_3 c) = \partial/\partial x(\epsilon_x \partial c/\partial x) + \partial/\partial y(\epsilon_y \partial c/\partial y) + \partial/\partial z(\epsilon_s \partial c/\partial z) + v \; \partial c/\partial y \quad ...(2.148)$$

The foregoing equation was also given by *Hayami*[77], in 1938.

Assume steady-state conditions, $\partial c/\partial t = 0$ and no variation of the concentration with either x direction, $\partial c/\partial x = 0$, or the z direction, $\partial c/\partial z = 0$; and furthermore $\epsilon_y$ is considered independent of elevation. In such a case $v_1$ and $v_3$ are zero. Hence equation (2.148) can be reduced to:-

$$\partial/\partial y(\epsilon_y \partial c/\partial y) + \omega \; \partial c/\partial y = 0 \quad ...(2.149)$$

integrating equation (2.149) above, one gets;

$$\epsilon_y \partial c/\partial y + \omega = C_1(x,z,t) \quad ...(2.150)$$

where $C_1$ is the constant of integration, which will be either constant or zero for steady, uniform, two dimensional flow.However since there is no sediment transfer across the free surface, $C_1=0$. Replacing $\epsilon_y$ by $\epsilon_s$, the sediment transfer coefficient, we get:

$$\omega_o c + \epsilon_s dc/dy = 0 \quad ...(2.151)$$

equation (2.151) was first used by *Schmidt*[78], in 1925, while studying suspension of dust particles in atmosphere.

*O'Brien*[79], in 1933, derived this equation by equating the upward rate of sediment motion due to turbulent diffusion and the downward volumetric rate of sediment transfer due to gravity.

Equation (2.151) above is the differential equation for distribution of suspended material in the verticals. This equation can be rewritten as ;

$$dC/C = (-\omega / \epsilon_s)\ dy \qquad \ldots(2.152)$$

This can be integrated between any two limits a and y to give

$$\ln C/C_a = -\int_a^y (\omega / \epsilon_s)\ dy \qquad \ldots(2.153)$$

in which C and $C_a$ represent the concentrations of suspended load at distances y and a respectively from the bed. The expression on the right-hand side of equation (2.153) can be integrated if the variations of w and $\epsilon_s$ with y are known. As an approximation it can be assumed that both $\omega_o$ and $\epsilon_s$ remain constant with respect to y. Therefore;

$$\ln C/C_o = -\omega/\epsilon_s (y-a) \qquad \ldots(2.154)$$

or

$$C/C_a = \exp\ [-\omega(y-a)/\epsilon_s] \qquad \ldots(2.155)$$

This equation was first obtained by *Schmidt*[78].

Both *Hurst*[80], in 1929, and *Rouse*[81], in 1938, have investigated experimentally equation (2.153) and its solution, their observations indicated that the general shape of equation (2.155) is almost correct for fine sediment, but the agreement was not quite perfect for larger particles. The assumption that $\epsilon_s$ is constant in the vertical direction is a rather bold one. However while the variation of $\epsilon_m$, the momentum transfer coefficient, with y is known for clear water; It is not known how $\epsilon_s$ varies with y. *Rouse*[82], in 1937, assumed $\epsilon_s = \epsilon_m$, also he assumed a linear distribution of shear stress, and assuming logarithmic velocity distribution, that is;

$$\epsilon_s = \epsilon_m = (\tau/\rho)/(du/dy) \qquad ...(2.156)$$

and

$$\tau/\tau_o = D - y/D \qquad ...(2.157)$$

where

$$\tau_o = \rho g\ D\ S \qquad ...(2.158)$$

also

$$du/dy = (\tau_o/\rho)^{1/2}/ky = u_*/ky \qquad ...(2.159)$$

where k is Karman's universal constant.

Combining the above equations back into equation (2.156), $\epsilon_s$ can be expressed as

$$\epsilon_s = ku_*(D-y)\ y/D \qquad ...(2.160)$$

introducing equation (2.160) into equation (2.153) and seperating variables, yields;

$$\int_a^y dc/c = \int_a^y (\omega/ku_*)\ (D/y)[dy/(D-y)] \qquad ...(2.161)$$

The quantity $\omega/ku_* = z$ ...(2.162)

where z is frequently referred to as the exponent of sediment distribution equation. Integration gives;

$$C/C_a = [\{(D-y)/y\}\{\ a/(D-a)\}]^z \qquad ...(2.163)$$

This suspended load distribution equation was introduced by *Rouse*[82], in 1937. It may be used for calculation of the concentration of a given grain size, if a reference concentration $C_a$ at a distance "a" is availble.

It may be worthwise to mention that equation (2.163) was independently derived by Ippen earlier, in 1936, at the suggestion of *Von Karman*[83].

*Vanoni*[86], in 1946, carried an integrated experimental analysis of suspended sediment load transport in water. In his discussion of the results, he explored the effect of suspended load on velocity distribution; As the concentration was increased the k value decreased, and concluded that the k value is a function of $\omega$, c, and $u_*$. He also studied the effect of suspended load on the resistance to flow, and showed that the suspended load reduces the resistance to the flow,as it decreases the turbulence with regard to the effect of sediment size on the distribution of suspended sediment. Vanoni showed good agreement between the measured suspended load distribution and the sediment distribution equation given by *Rouse*[82] for coarse sediment, but for fine sediment the measured values of z, the exponent of equation (2.163) above, was found to be about 20% less than the theoretical value for 0.160 mm sand. Also for fine sediment $\epsilon_s$ is observed to be greater than $\epsilon_m$. Vanoni explained that the difference between the transfer mechanism for sediment and momentum is attributed to two reasons :

The first reason is due to the absence of a correlation between the horizontal and vertical turbulence fluctuations, since uncorrelated or random fluctuations don't contribute to the momentum transfer. For sediment transfer the existence of such a correlation is not necessary, but the random fluctuations of transfered sediment tend to make $\epsilon_s$ larger than $\epsilon_m$. The second reason is due to the slip between the fluid and the sediment as the sediment is accelerated. To sum up, Vanoni's conclusions, the sediment in suspension affects the flow in three different ways:

1- The sediment appears to damp out the turbulence which in-turn reduces the momentum trnasfer.

2- Random turbulence, which is not a factor in the transfer of momentum, contributes to the transfer of sediment.

3- The slip between the fluid and the sediment tends to make the sediment transfer coefficient less than the momentum transfer coefficient.

*Ismail*[87], in 1951, used experimental data in close channel to study (a) the effects of the presence of sand in suspension on the characteristics of the flow, and (b) the relation between the sediment transfer coefficient $\epsilon_s$ and the momentum transfer coefficient $\epsilon_m$ for two grain sizes of fine sand. Observations showed that the value of the universal turbulent constant, k, decreases with the increase in suspended material. The value of k was 0.20 when the average concentration is 4.3% by weight. Ismail also observed that, the sediment transfer coefficient $\epsilon_s$ is equal to 1.5 times the momentum transfer coefficient $\epsilon_m$ for the 0.10 mm sand, and $\epsilon_s = 1.3\ \epsilon_m$ for the 0.16 mm sand, both $\epsilon_s$ and $\epsilon_m$ follow the normal parabolic form at the outer two thirds of the channel. According to these conclusions, the measured values of z were found to be less than the calculated values, i.e;

$$z_m = z\ /\beta = [\ w/(\beta\ k\ u_*)] \qquad \ldots(2.164)$$

the value of $\beta$ was found to be, as mentioned earlier, 1.5 for 0.1 mm sand and 1.3 for 0.16 mm sand. The present auther gives support to the conclusions of Ismail since for the sand used ($d_{50}$=0.15 mm), the value of $\beta$ is (1.28) which is in good agreement with *Ismail*[87] , and confirmed Vanoni's findings.

*Einstien and Chien*[88], in 1954, proposed a second order approximation to the suspended load theory by modifying some of the assumptions upon which the derivation was based. They assumed the mixing length to vary according to probability distribution.

*Tanaka and Sugimoto*[89], in 1958, have also proposed an exponential form of sediment distribution equation as;

$$\frac{C}{C_a} = \left\{\left(\frac{\sqrt{D} + \sqrt{(D-y)}}{\sqrt{D} - \sqrt{(D-y)}}\right)\left(\frac{\sqrt{D} - \sqrt{(D-a)}}{\sqrt{D} + \sqrt{(D-a)}}\right)\right\}^{\omega/(u_* k)} \qquad \ldots(2.165)$$

*Paintal and Garde*[90], in 1964, showed that *Hunt*[89], *Tanaka and Sugimoto*[89], *and Rouse*[82] equations gave more or less the same distribution of suspended materials in the vertical.

*Navntoft*[91], in 1970, treated the flow of sediment water mixture as a one-phase flow of fluid with density gradient. He assumed the fall velocity of the sediment to be unaffected by the concentration,but he assumed that the fluctuation in concentration with time at any level to be linearly related to the average concentration at that level. Using the mixing length hypothesis for the turbulent shear stress and the equation of continuity, Navntoft gave his equation in the form:-

$$C = B_1 \ln [\{A_1 + (y/D)\}/\{A_1 + (y/d)\}]_{c=o} \quad \ldots(2.166)$$

where $A_1$ and $B_1$ are empirical constants and $(y/D)_{c=o}$ is the relative depth at which concentration is zero;It should be noticed that the above equation gives zero concentration some distance below the free surface. *Navntoft* found that his equation is in good agreement with the observations of *Einstien and Chien*[88].

*Levelle and Thacker*[92], in 1978,considered the variation in fall velocity due to change in sediment concentration. If $\omega$ is the fall velocity at concentration C, its relation with $\omega_o$, the fall velocity in clear water, was assumed to be;

$$\omega/\omega_o = (1-C)^5 \quad \ldots(2.167)$$

further, the value of $\epsilon_s$ was related to distance from the boundary as;

$$\epsilon_s = (a+by)(1-(y/D)) \quad \ldots(2.168)$$

where a and b are empirical constants. According to these assumptions, they derived a suspended load equation that fits the data of *Einstein and Chein*[88]. However no predictors are available for the coefficients a and b.

*Willis*[93], in 1978, assumed that $\epsilon_s = \epsilon_m$ and a distribution of $\epsilon_s$ as;

$$6\epsilon_m/(k u_* D) = (2)^{1/2} e^{-p^2/2} \qquad \ldots(2.169)$$

where p is obtained from

$$y/D = 1/(2\pi)^{1/2} \int_{-\infty}^{p} e^{-r^2/2}\, dr \qquad \ldots(2.170)$$

The distribution of suspended materials was then derived as;

$$C/C_a = e^{-(3/(\pi))^{1/2} (p-p_a)(v_o/u_* k)} \qquad \ldots(2.171)$$

where $p_a$ is the value of p at a reference level y= a, Willis recommended the adoption of a= 2d.

*McTigne*[94], in 1981, visualised the flow depth as consisting of an inner layer and an outer layer. The inner layer extended from the bed up to 0.2D, while the outer layer lies above it. The sediment transfer coefficient $\epsilon_s$, was assumed to be equal to $k_1 u_* D$ in the outer region and equal to $k_2 u_* y$ in the inner region. The concentration distribution equation becomes:-

$$C/C_a = \exp\,[-\,\omega/k_1 u_* D\,(y-a)] \quad \text{for } y < 0.2D \qquad \ldots(2.172)$$

and $$C/C_a = (y/a) - (\omega/k_2 u_*) \quad \text{for } y \geq 0.2D \qquad \ldots(2.173)$$

The values of $k_1$ and $k_2$ were evaluated for one of the experimental runs as 0.11 and 0.35 respectively.

*Rubey*[64], in 1933, approached the sediment transport by flow as a problem of expenditure of the stream energy and these ideas were further extended by *Knapp*[95], in 1938, However the major development of the concept is due to *Bagnold*[45,46], in 1956, 1966, although *Velikanov*[96,97], in (1954, 1955, 1958) proposed his gravitational theory which leads to similar results.

*Bagnold*[45] found that the rate of work done by the shear flow turbulence of the fluid is;

work rate of suspended load = $g_{is}(\omega/U_s)$

where $g_{is}$ is the immersed weight of sediment and $U_s$ is the mean transport velocity of suspended solids. Hence $\omega/U_s$ is analogus to the friction factor tan $\alpha$. The available power supply per unit area is:-

$$P = \rho g\ D\ S\ U = \tau_o U \qquad ...(2.175)$$

of which $\eta_b P$ is dissipated in the bed-load transport, leaving $P(1-\eta_b)$ for suspended load. Hence

$$g_s\ \omega/U_s = \eta_s\ P(1-\eta_b) \qquad ...(2.176)$$

or

$$g_s = \eta_s P\ U_s/\omega\ (1-\eta_b) \qquad ...(2.177)$$

*Velikanov*[95], in 1954, introduced his gravitational theory. For two dimensional flow and using energy concept, he wrote;

$$\omega\ c + \bar{u}\tau/\gamma_s * \ dc/dy = 0 \qquad ...(2.178)$$

The above equation is of the same form as the diffusion equation, except the exchange coefficient has a different form. with the same assumptions of linear shear stress and logarithmic velocity distributions, Velikanov obtained;

$$C/c_a = \exp\left[-(w/u_*)(k\gamma_s^* /\gamma S) \int_{\eta_a}^{\eta} d\eta/\{(1-\eta)\ln(\eta/\alpha)\}\right] \quad ...(2.179)$$

where $\eta = y/D$, $\eta_a$ is a reference level, $\alpha$ = ds/30D and $\gamma_s^* = \gamma_s - \gamma$.

Since suspension is maintained by turbulence which is random by nature, it is only natural that the distribution of suspended sediment should be subjected to decription by probabilistic methods. The studies of suspension which have led to the statistical models starts with attempts to relate the particle motion to turbulence.

So far the review of suspended load has been devoted to the discussion of concepts and ideas, it seems appropriate to explore the quantitative approaches for calculating the suspended sediment load.

The suspended-load rate per unit width is obtained by integration the product of velocity and concentration over the suspension depth, i.e.

$$q_s = \int_{y=a}^{D} C\, u dy \quad ...(2.180)$$

*Lane and Kalinske*[1], in 1941, suggested a simplified form for the integration by introducing a mean value of diffusion coefficient $\epsilon_s$, from the general diffusion equation ;

$$\omega c + \epsilon_s dc/dy = 0 \quad ...(2.181)$$

$$\int_{c_a}^{c} dc/c = -\omega \int_{a}^{y} dy/\epsilon_s \quad ...(2.182)$$

for constant $\epsilon_s$, we have

$$C/C_a = \exp -[w(y-a)]/\epsilon_s \quad ...(2.183)$$

since the assumption of $\epsilon_s$ being constant is incorrect, but rather varies with depth as;

$$\epsilon_s = k u_* \, y/D(D-y) \qquad \ldots(2.184)$$

an average value $\bar{\epsilon}_s$ is given by

$$\bar{\epsilon}_s = \int_0^D \epsilon_s dy/D = (k u_* /D^2) \int_0^D (Dy-y^2)\, dy \qquad \ldots(2.185)$$

which becomes for k= 0.4,

$$\epsilon_s = (1/15)\, u_* D \qquad \ldots(2.186)$$

Lane and Kalinske suggested to use the $\bar{\epsilon}_s$ in evaluating the integration of equation (2.182) above, the result is ;

$$C/C_a = \exp[\{-15\, w(y-a)\}/Du_*] \qquad \ldots(2.187)$$

and for the velocity distribution, they took;

$$u/u_* = (5.75 \log y/k) + 8.5 \qquad \ldots(2.188)$$

$$\bar{u}/u_* = (5.75 \log D/k) + 6.5 \qquad \ldots(2.189)$$

The last two equations lead to:-

$$u-\bar{u}/u_* = 5.75\, y/y_o + 2.5$$

$$= 1/k(\ln y/y_o + 1)$$

and $$u/\bar{u} = 1+(u_*/k\bar{u})(\ln y/y_o + 1) \qquad \ldots(2.190)$$

then $q_s = \int C\, udy$ , from which

$$q_s = q\, P_L C_a \exp[15\, \omega a/Du_*] \qquad \ldots(2.191)$$

with a factor $P_L$, a function of $\omega/u_*$ and the relative roughness $n/D^{1/6}$, and was given graphically by Lane et al.as in Fig.(2.7) , but converted to SI units.

The value of $C_a$ is the reference concentration at y= a; The above equation has to be solved for each size fraction. *Einstein*(2), in 1950, expressed the suspended sediment load rate by introducing the logarithmic velocity distribution as;

$$u/u_* = 5.75 \log(30.2y/\Delta) \quad \ldots(2.192)$$

and the *Rouse*[82] suspension distribution equation given by;

$$C/C_a = ((D-y/y)(a/D-a))^z \quad \ldots(2.193)$$

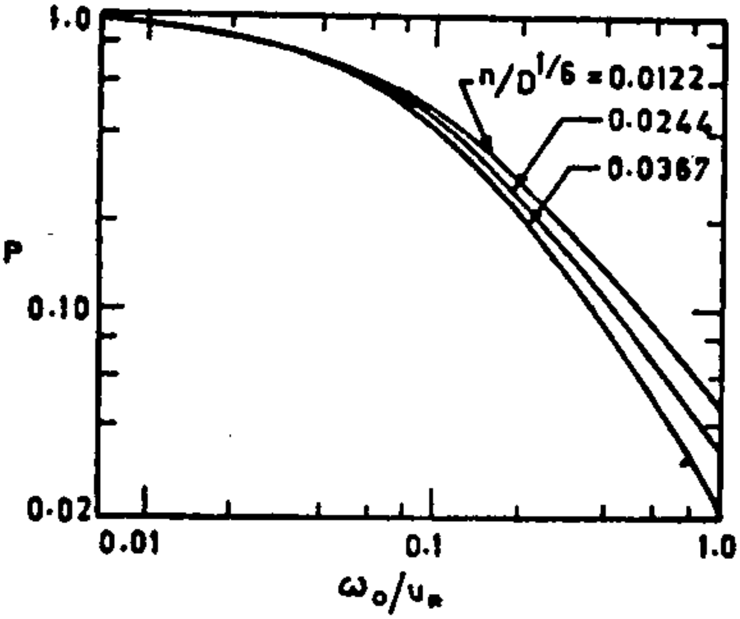
where $\Delta = k_s/X$, $k_s$ being the bed roughness and X is a correction factor. Let $A_E = a/D$ and performed the numerical integration of the equation $\int_{A_E}^{1} CuD\,dy$,

The values of $I_1$ and $I_2$ were evaluated for different $A_E$ and z values as follows;

$$I_1 = 0.216\, A_E^{z-1}/(1-A_E)^z \int_{A_E}^{1}((1-y)/y)^z\, dy \quad \ldots(2.194)$$

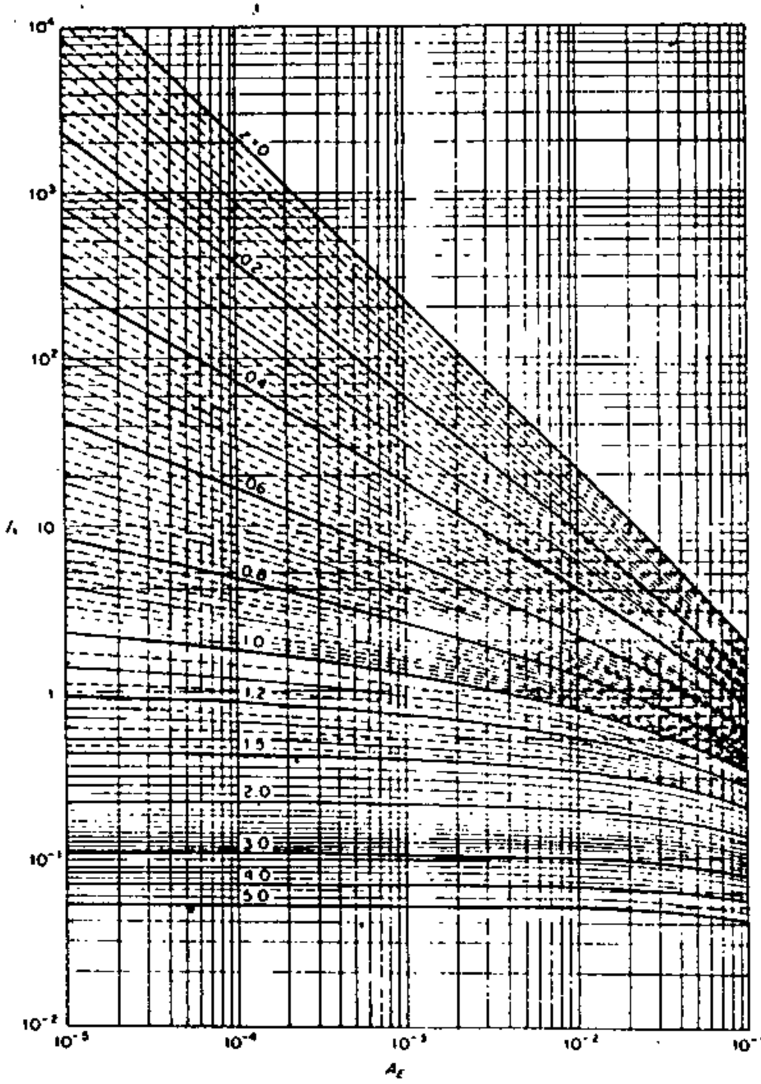
$$I_2 = 0.216\, A_E^{z-1}/(1-A_E)^z \int_{A}^{1}((1-y/y))^z \ln y\, dy \quad \ldots(2.195)$$

The values of $I_1$ and $I_2$ were given graphically by Einstein as a function of $A_E$ and z.as shown in Fig.(2.8).

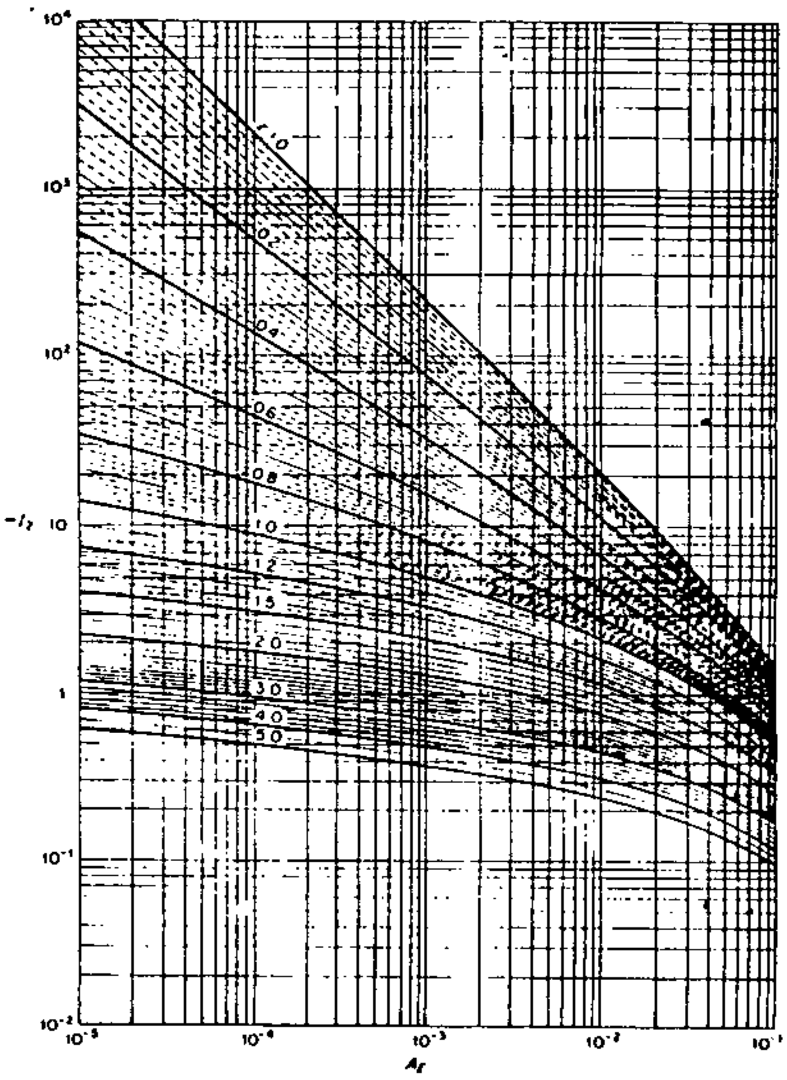



Variation of $P$ with $\omega_0/u_*$ and $n/D^{1/6}$

FIG.(2.7) LANE AND KALINSKE'S SUSPENDED LOAD FUNCTION [SI UNITS].




Function $I_1$ in terms of $A_E$ for values of z. [After EINSTEIN (1950).]




Function $I_2$ in terms of $A_E$ for values of z. [After EINSTEIN (1950).]

FIG. (2.8) EINSTEIN'S INTEGRAL $I_1$ AND $I_2$ .

The Einstein's suspended load equation can be written as;

$$g_* = 11.6\ C_a\ u'_* \quad a\ [2.303\ \log(30.2D/\Delta) I_1 + I_2] \qquad ...(2.196)$$

which gives the suspended load for a given size fraction of a given $\omega$.

The reference concentration $C_a$, is that concentration occurs at the top of the bed layer. The thickness of the bed layer, due to Einstein, was taken to be 2d. Since there will be continuity in the distribution of suspended load and bed-load, it was assumed, by Einstein, that the average concentration of the bed-load in the bed layer must be equal to the concentration of suspended load at y= 2d, the average concentration in the bed layer is given by

$$C_{2d} = i_B g_B / 23.2\ u'_* d \qquad ...(2.197)$$

where $g_B$ is the bed-load rate of size fraction $i_B$ on weight basis.

Thus the bed-load rate can be introduced to the suspended load rate equation as;

$$i_* g_* = i_b g_B [2.303\ \log(30.2D/\Delta)\ I_1 + I_2] \qquad ...(2.198)$$

introducing $p_E$ as a transport parameter given by

$$p_E = 2.303\ \log(30.2D/\Delta) \qquad ...(2.199)$$

a relationship between the bed-load transport and suspended load transport exits as

$$i_s g_S = i_b g_b (p_E I_1 + I_2) \qquad ...(2.200)$$

This equation is dimensionally homogenous. With a slightly different approach *Brooks*[98], in 1963, has developed an equation to determine the suspended load, assuming the law of logarithmic velocity distribution, and the typical concentration distribution. Brooks obtained the relationship given by,

$$q_s = C_{md} q \, [1 + u_*/k\bar{u} \int_{A_E}^{1} (1-y/y)^{z} dy$$

$$+ \, u_*/k\bar{u} \int_{A_E}^{1} (1-y)/y)^{z} \ln y \, dy] \qquad \ldots(2.201)$$

The above equation can be rewritten in terms of transport function $T_B$ as;

$$q_s/qC_{md} = T_B(k\bar{u}/u_*, \, z, \, A\ ) \qquad \ldots(2.202)$$

where $C_{md}$ is the reference concentration at y= D/2, the choice of lower limit of integration is suggested to be at u= 0, and $A_E$ becomes

$$A_E = \exp[-(k\bar{u}/u_*)-1] \qquad \ldots(2.203)$$

The equation (2.204) above reduces to

$$q_s/qC_{md} = T_B^*(k\bar{u}/u_*, \, z) \qquad \ldots(2.204)$$

or

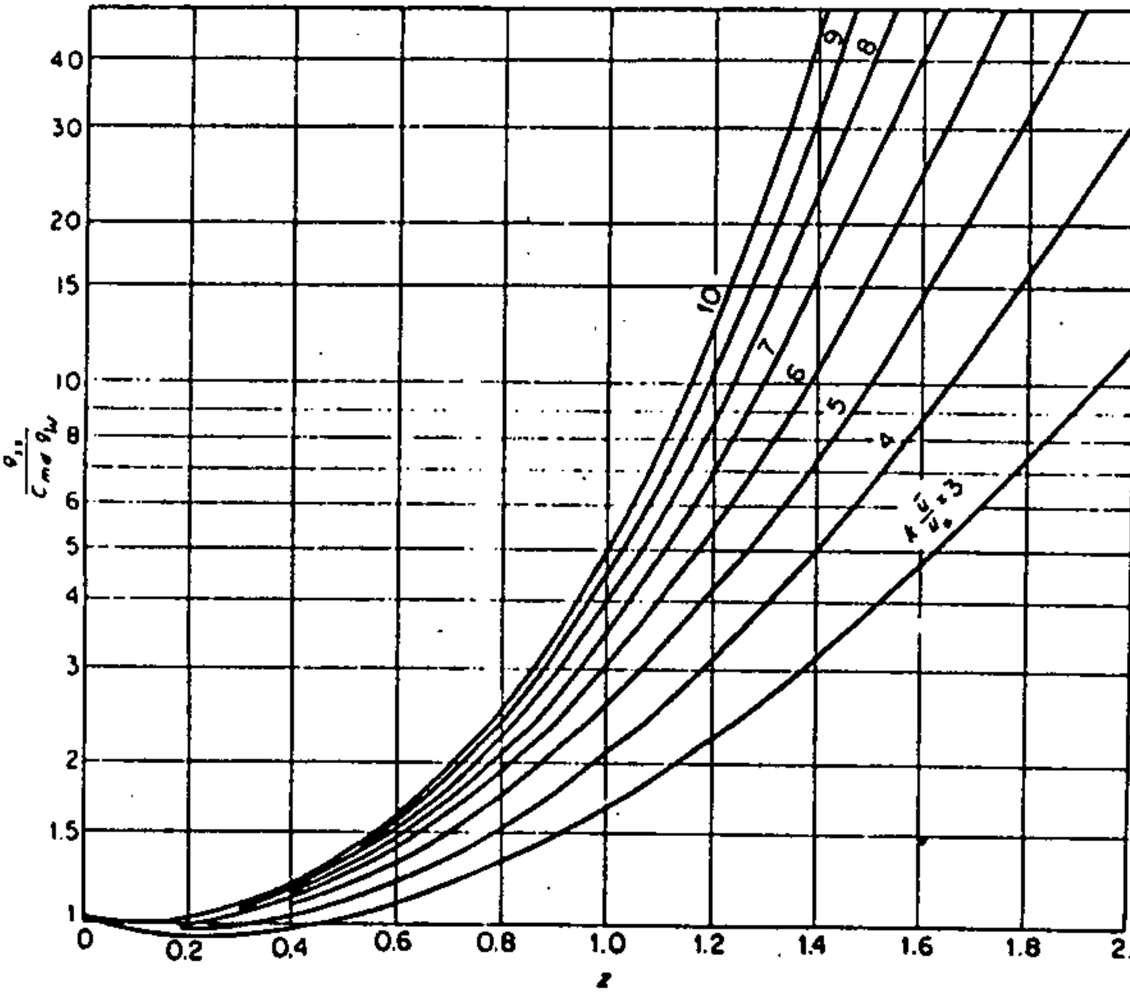
$$\bar{C}/C_{md} = T_B^*(k\bar{u}/u_*, \, z) \qquad \ldots(2.205)$$

from which the relationship to $C_a$ is,

$$\bar{C}/C_{md} = \bar{C}/C_a(y\ -a/a)^{z} \qquad \ldots(2.206)$$

thus $2d/D = (C_{md}/C_b)^{1/z}$ ...(2.207)

where $C_b$ is the average concentration in the bed layer. The values of $T_B^*$ is given by Brooks as shown in Fig.(2.9).




Function $g_{ss}/gC_{md}$ in terms of $k\bar{u}/u_*$ and of $z$ values. [*After* BROOKS (*1963*).]

FIG. (2.9) BROOKS' SUSPENDED LOAD EQUATION.

From the concentration distribution equation given by *Rouse*[82] as,

$$C/C_a = (D-y/y \ . \ a/D-a)^z \qquad ...(2.208)$$

for $a = D/2$ then $C_a = C_{md}$

$$C/C_{md} = [D-y/y \ (D/2)/(D/2)]^z$$

$$C/C_{md} = [D-y/y]^z$$

or $$z = \log(C_{md}/C \quad)/\log(D-y/y) \qquad ...(2.209)$$

i.e z is the slope of $C/C_{md}$ Vs. D-y/y on log-log scale.

The reference level is important because near the bed the concentration is relatively high and small change in elevation will have a large effect on the total suspended sediment discharge. Brooks showed an example where $y_1 = 0.1D$ and the unmeasured suspended load was twice the measured load.

There exists some simple relations for suspended load, *Rubey*[64], found that the average suspended load concentration $\bar{C}$, is proportional to $R^{1/2}S^{2/3}$. Since q is proportional to $R^{3/2}S^{1/2}$ from the Chezy equation, it can be written as:

$$q_s = \bar{C}q \ \alpha \ R^2 S^{7/6} \qquad ...(2.210)$$

Several field engineers have reported a relationship between $q_s$ and q for different rivers as,

$$q_s \ \alpha \ q^b \qquad ...(2.211)$$

the value of b was found to have an average of 2.00.

*Samaga*[99], in 1984, proposed a method of calculation of suspended load by considering the individual fraction in a mixture.He found that the parameters $\phi_*$ and $\tau_*$ are uniquely related by the equation

$$\phi_* = 30\ \tau_*^{\sigma} \qquad \ldots(2.212)$$

in case of uniform sediment, he reported that,

$$\phi_* = (q_*/\gamma_* d)\ [(\gamma/\gamma(s_*-1))\ (1/gd)^{1/2}] \qquad \ldots(2.213)$$

where

$$\tau_* = \tau_o/(\gamma_*-\gamma)d \qquad \ldots(2.214)$$

This method seems to be similar to the bed-load transport equation introduced by *Einstein*[42] *and Co-worker.*

## 2.5. THE TOTAL SEDIMENT LOAD :-

The total load rate $g_T$ is obtained by the addition of the bed-load rate $g_B$ and the suspended load rate $g_s$. each of the bed-load rate and suspended load rate may be obtained from some equations. At low transport rates,where most of the sediments move in contact with the bed, the bed-load approximates, sufficiently well, the total load.

*Lane & Kaliske*[1], in 1941, were probably the first to calculate the total load , by summing their calculated suspended load to that obtained from a bed-load equation.

A more correct name of the total load is,actually,the Bed Material Load.Since the bed-load and suspended load equations were derived such that the particle supply is found within the bed material; This then implies that the term total load is not identical to bed material load, because of another so-called wash-load, which is made up of grains finer than the bulk of bed material. Wash-load usually is caused by land erosion and not by channel errosion, so it depends on the hydrological & geological conditions, this is of some concern to the agricultural engineers.

The methods of computation of the total sediment transport rate can be broadly classified into two categories. The first category make use of the subdivision of the total Sediment load into suspended load and bed-load. The addition of the two gives the total load in absense of the wash-load,these methods are referred to as the direct mothods or microscopic methods. Under the second category, no distinction needs to be trade between bed-load and suspended load. The second group are referred to as the direct methods or macroscopic methods.

*Einstein*[2], in 1950, advanced the bed-load and suspended load concept. The bed-load rate is given by

$i_B\ g_B$ , for a size fraction of $i_B$.

And the suspended-load rate is given by

$i_s\ g_s$ , for a size fraction $i_s$, thus the resulting total load for a size fraction $i_T$ is:

$$i_T\ g_T = i_B\ g_B + i_s\ g_s \qquad ...(2.217)$$

where all the rates are given in weight per unit time and unit width, simplifying equation (2.217) above i.e.

$$i_T\ g_T = i_B\ g_B(1 + P_E\ I_1 + I_2) \qquad ...(2.218)$$

where $P_E$: is a transport parameter given by;

$P_E = 2.303 \log (30.2\ D / \Delta)$

D = depth of the flow (uniform and steady flow)

$\Delta$ = apparent roughness = $K_s/x$

x = a correction factor.

$I_1$, $I_2$ = are integrals evaluated by Einstein (1950), and given graphically in Fig.(2.8).

Modified Einstein procedures as outlined by *Colby and Hembree*[100], in 1955, adapted for computation of total load.

The *Einstein*[2] procedure estimates the bed-load and suspended load for any selected discharge from data on the geometry of the river and sediment grading. It is a design procedure.

The modified Einstein procedure estimates the total load discharge for a given stream at a given discharge from a measured depth integrated suspended load sample, including sometimes the wash-load. The calculation is based on measured mean velocity, and depth instead of hydraulic mean raduis.

*Einstein*[101], in 1964, wrote the equation given by Colby et al[100], in a simpler form, as;

$$(i_T g_T)/(i_{sm} g_{sm}) = (\eta_1/\eta_o)^{z-1}((1-\eta_o)/(1-\eta_1)^z) \qquad \ldots(2.219)$$

$$(1+P_E I_1+I_2)\eta_o/(P_E I_1+I_2)\eta_1$$

where: $g_T$ is the total sediment discharge per unit width

$g_{sm}$: the measured suspended sediment discharge per unit width.

$\eta_o = (a/D) = A_E$

$\eta_1 = (y_1/D) =$ unmeasured depth/total depth of flow.

z: modified exponent

$I_1, I_2$ & $P_E$: as defined earlier by Einstein.

Equation (2.219) is dimensionally homogeneous and can be used in any system of units.

From the point of general physics, *Bagnold*[40], in 1966, argues that the existance and maintenance of upward supporting stresses equal to the immersed weight of the solids. The dry mass m and the immersed mass of the solid $m^1$ are related by:

$$m^i g = (\rho_s - \rho/\rho_s) m g \qquad \ldots(2.220)$$

Thus the bed-load mass $m^i b$ is defined as that part of the total load mass which is supported by a solid-transmitted stress $m_b^i g$, while the suspended load mass $m_s^i$ is supported by the fluid-transmitted stress $m_s^i g$. The transport rate of solids by immersed weight per unit width is given as $i_T$

$$i_T = i_b + i_s = (\rho_s - \rho/\rho_s) m g U = m_b^i g U_b + m_s^i g U_s \qquad \ldots(2.221)$$

where U: mean transport velocity of solids.

$U_b$: mean transport velocity of solid moving as bed-load

$U_s$: mean transport velocity of solid moving as suspended load.

The important point made by Bagnold is that equation (2.221) above gives dynamic transport rates which have dimensions of work rates, but however stresses and velocities are not in the same direction, thus a correction factor was introduced as follows:

$$\text{The bed-load work rate} \equiv i_b(\tan\alpha) = m_b^i g U_b(\tan\alpha) \qquad \ldots(2.222)$$

where $\tan\alpha$ = coefficient of solid friction ,

and the suspended load work rate is:

$$i_s(w/U_s) = m_s^i g U_s(w/U_s) \qquad \ldots(2.223)$$

where w= settling (terminal) velocity.

Furthermore, Bagnold introduced the power equation which relates the rate of doing work with the avaliable power by means of efficiency.

The avaliable power per unit length per unit width is

$$P = (\gamma\ QS/B) = \gamma\ DSU = \tau_o U \qquad \ldots(2.224)$$

where $D \equiv$ channel depth

$B \equiv$ channel width

$U$ = average velocity of fluid

$S$ = slope of the channel

Accordingly

$$i_b \tan\alpha = e_b P$$

$$i_s (w/U_s) = e_s P(1-e_b) \qquad \text{...(2.225)}$$

where:

$e_b$ , $e_s$ represent the bed-load and suspended-load efficiency, respectively. Introducing equations (2.221) and (2.225) together;

$$i_T = i_b + i_s = P[(w_b/\tan\alpha) + (e_s U_s/w)(1-e_b)] \qquad \text{...(2.226)}$$

Thus the total load rate may be obtained if four parameters namely $e_b$, $e_s$, $\tan\alpha$, and $U_s$ are known. Equation (2.226) can be applied to laminar flow but in this case the second term of the equation disappears. Bagnold showed graphically that $e_b = f(u,d)$ . From flume studies it was found that, $e_s(1-e_b) = 0.01$ , and assuming that the mean velocity of fluid and the suspended solid velocity are equal introducing these into equation (2.226); yields:-

$$i_T = [P(e_b/\tan\alpha + 0.01\ (U/w))] \qquad \text{...(2.227)}$$

It should be stressed that equation (2.227) is applied to fully turbulent flow conditions with adequate depths, where $\tan\alpha$ being the solid friction coefficient of grains.

Considering $P = \tau_o U$ and $\tau_o = \rho u_*^2$ then according to Bagnold,

$$q_T \propto u_*^4 \quad \text{or} \quad q_T \propto \tau_o^2$$

which is the case of many field observations carried out for flows transporting heavy suspended load.

*Chang, Simons, and Richardson*[102], started from the concept that the total load may be given as:

$$g_T = \int_0^a Cu_s\,dy + \int_a^y Cu_s\,dy \qquad \ldots(2.228)$$

in which the first term represents the bed-load and the second term represents the suspended load, and a is the bed layer thickness.

They employed the Du Boys'[21] relationship, to express the bed-load, in the form;

$$g_s = K_T U\,(\tau_o - \tau_c) \qquad \ldots(2.229)$$

where $K_T$ is the bed material discharge coefficient.

and U is the mean flow velocity.

It was found experimentally that for natural rivers $0.72 < K_T < 1.10$. For suspended load, they introduced $g_s = g_b R_s$ which is, similar to Einstein's equation. Thus the total load;

$$g_T = g_s + g_b = K_T [\tau_o - \tau_{cr}]\ U(1+R_s) \qquad \ldots(2.230)$$

where $R_s \equiv$ constant containing the two integrals of Einstein.

*Colby*[123], in 1964 , prepared a set of graphs for calculation of sediment transport in sandy rivers. He studied the effect of mean velocity, depth, shear stress, stream power, viscosity, temperature and concentration of fine sediment on the discharge of sand per unit width of the channel. The estimated sediment discharge $g_T$ is given by:

$$g_T = [1+(K_1K_2-1)0.01\ K_3]\ g_{T1} \qquad \ldots(2.231)$$

where $K_1$ is correction factor for temperature.

$K_2$ is correction factor for concentration.

$K_3$ is correction factor for median particle size.

$g_{T1}$ is the uncorrected discharge of sand per unit width

Values of $K_1$, $K_2$, $K_3$ are known graphically for any given conditions.

*Laursen*[104], in 1958, advanced parameters to explain the relation between the flow and sediment transport rate. One parameter is the ratio

$$(\tau_o/\rho)^{1/2}/w = u_*/w \qquad \ldots(2.232)$$

this ratio expresses the effectiveness of mixing action of turbulence in the suspended load concept.

Using *Manning and Strickler's* relation, Laursen[104] gave

$$\tau_o' = u^2 d^{1/3}/30\ D^{1/3} \quad ,\ lb/ft^2 \qquad \ldots(2.233)$$

where $\tau_o'$ is the boundary shear associated only with the sediment particle resistance.

also $\tau_o^1 \simeq \rho\ U^2/58(d_i/D)^{1/3}$, (any system of units) ...(2.234)

Accordingly the following empirical relationship was suggested

$$C = \Sigma i(d_i/D)^{7/6}((\tau_o^1/\tau_{c_i})-1)\ f(u_*/w_i) \qquad \ldots(2.235a)$$

for quartz sand, It is found that ;

$$C = 256\ (q_s/q) \qquad \ldots(2.235b)$$

where C ≡ the cross sectional mean concentration by weight in percent.

It is worthwhile to mention investigation of *Bogardi*[105], in 1965, which gave similar equation to (2.235) as;

$$C= (d/R_h)^{7/6}[(\tau_o/\tau_c)-1]\ f(gd/u_*^2,d) \qquad ...(2.236)$$

where $R_h$= hydraulic mean raduis.

In a discussion of Laursen's contribution, *Garde et al*, 1983, suggested another empirical relationship:

$$(u_*D\ /\nu)*\ (1/C)^{1/3}= [(y_o/d)\ (1/f(d))]^{3/2} \qquad ...(2.237)$$

which was found to be in good agreement with observations.

*Bishop, Simons and Richardson*[106], in 1965,used of the $\phi_*$ Vs $\psi_*$relation but rather than predicting the bed-load transport, it was remodeled such that it predicted the total load discharge, as Einstein[2],they reported that;

$$\phi_*= f(\psi_*) \qquad ...(2.238)$$

where $\psi_*$= intensity of shear = $(\rho_s-\rho)/\rho\ (d/(S^1R_h^1))$

$\phi_*$= intensity of transport= $g_b/\gamma_s[(\rho/(\rho_s-\rho))(1/gd)^3]^{1/2}$

*Bishops et al*[106] reason that the shear intensity parameter $\psi_*$ may be used to predict immediately and directly the intensity of transport for total load rate $g_T$ by introducing the intensity of transport for total load as $\phi_T$, where $\phi_T$ is given by;

$$\phi_T= g_T/\gamma_s[(\rho/(\rho_s-\rho))(1/d^3g)]^{1/2} \qquad ...(2.239)$$

Using flume data for four different sands the $\phi_T$ Vs. $\psi_*$ relationship was established. Although the curves for each grain size exhibit the same general trend; they differ by a considerable degree. To remedy this effect, the scale constants $A_*$ and $B_*$ were introduced and found to be in functional relationship with the median diameter of the sand.

The $\phi_T$ Vs. $\psi_*$ relationship can be divided into three segments corresponding to the lower, transition and upper regimes of the bed configurations. The lower part of it represent the bed forms such as ripples and/or dunes, this part is fitted to Einstein's relationship $\phi_*$ Vs $\psi_*$. The second segment or the inflection of the curve in which bed forms ranging from dunes to plane beds to antidunes. The upper most part of the curve represents data with bed forms of plane beds and antidunes. Data in the second and upper part of the curve cannot be predicted with *Einstein's* relationship because, most likely, the better part of the total load is by now in suspension.

A physical model was proposed by *Graf and Acaroglu*(107), in 1968, for sediment transport in conveyance systems, both open channel and for closed conduits.They used the entire hydraulic radius $R_h$ rather than that associated with the grain roughness $R_h^{\prime}$ used by Einstein(2).

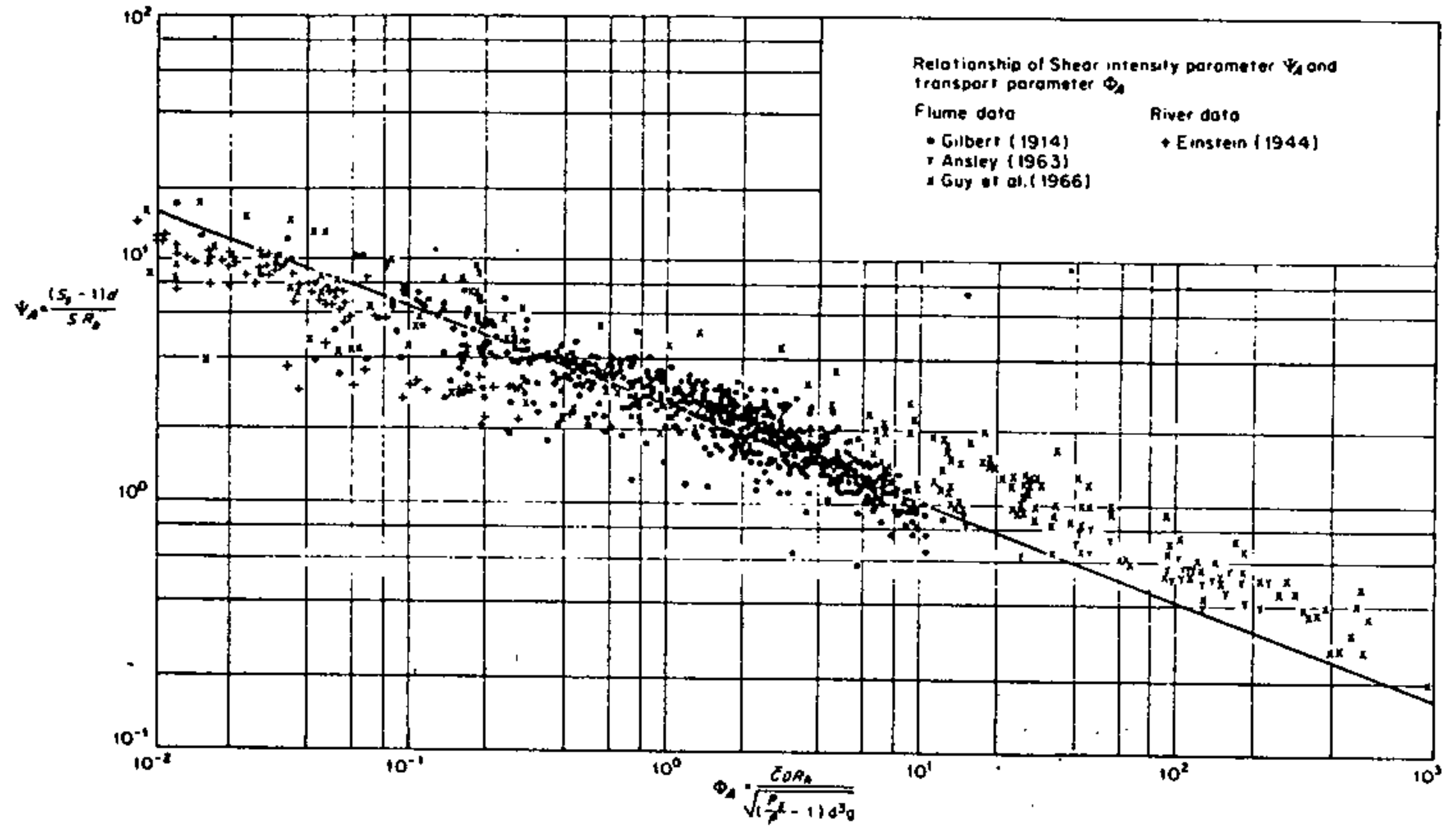
A shear intensity parameter $\psi_A$ is given by

$$\psi_A = [((\rho_s-\rho)/\rho)/SR_h]d \qquad ...(2.240)$$

Based on a work rate concept; a transport parameter was established such as;

$$\phi_A = (C_v U R_h)/[((\rho_s-\rho)/\rho)dg^3]^{1/2} \qquad ...(2.241)$$

where $C_v$ is the volumetric concentration of the transported particles.




$\Phi_A$ vs. $\Psi_A$ relation; with open-channel data only. [*After* GRAF *et al.* (*1968*).]

FIG.(2.10) GRAF'S ET. AL. TOTAL LOAD EQUATION.

Using experimental data from laboratory and field measurements, the relationship $\phi_A = f(\psi_A)$ was evaluated by regression analysis as;

$$\phi_A = 10.39(\psi_A)^{-2.52} \qquad \ldots(2.242)$$

which is applicable for both open channel and close conduits.

*Engelund and Hansen*[108] proposed a method for determination of the total sediment discharge in streams with dune-covered beds for which $u_* d_{50}/\nu > 12$ and $d_{50} > 0.15$ mm.

The computations depends on the Engelund resistance formula. The relationships are:

$$f\phi = 0.1\ \theta^{2.5} \qquad \ldots(2.243)$$

and $$\phi = q_{Ts}/[(S_s-1)g\ d_{50}^3]^{1/2} \qquad \ldots(2.244)$$

where $$f = 2gD_oS/U^2 \qquad \ldots(2.245)$$

and $$U/(gD_oS)^{1/2} = 0.6 + 2.5\ \ln y_o/k \qquad \ldots(2.246)$$

in which $k = 2.5\ d_f$, $d_f$ is the fall diameter of sediment, and

$$\theta = \tau_o/[\gamma(S_s-1)d] = D_oS/(S_s-1)d \qquad \ldots(2.247)$$

$$\theta^1 = 0.06 + 0.4\ \theta^1 = D_o^1S/(S_s-1)d \qquad \ldots(2.248)$$

the last equation proposed by Engelund et al. for flow over dune covered bed can be expressed as:

$$q_{Ts} = 0.05\ U^2[(d_{50}/g(S_s-1)(\tau_o/(\rho_s-\rho)gd_{50})^{1/2}]^{3/2} \qquad \ldots(2.249)$$

*Engelund*[109], in 1973, also proposed a method for calculation of sediment transport when the bed material is graded. He defined a critical size below which the particles will be in suspension & above which the particles will move as bed-load. This size is given by the empirical relation

$$2.5\ w_c/u_* = 2 \qquad \ldots(2.250)$$

where $w_c$ is the fall velocity when suspension exists.

*Toffaleti*[51], in 1969, proposed a procedure for calculation of total load based on concepts of Einstein. He divided the sediment into fractions and the stream depth into four zones: bed, lower, middle and upper zone.

Whereas Einstein obtained the reference concentration from the bed-load, Toffaleti proceeded in the opposite direction and calculated the bed-load on the basis of suspended sediment concentration curve. He defined an exponent $Z_t$ similar to that given in Rouse[81] equation as

$$Z_t = w\, U/(C_z RS) \qquad \ldots(2.251)$$

where $C_z$ is a temperature dependent coefficient given by

$$C_z = 260.67 - 0.667\, T \qquad \ldots(2.252)$$

and $T$ is the temperature in °F.

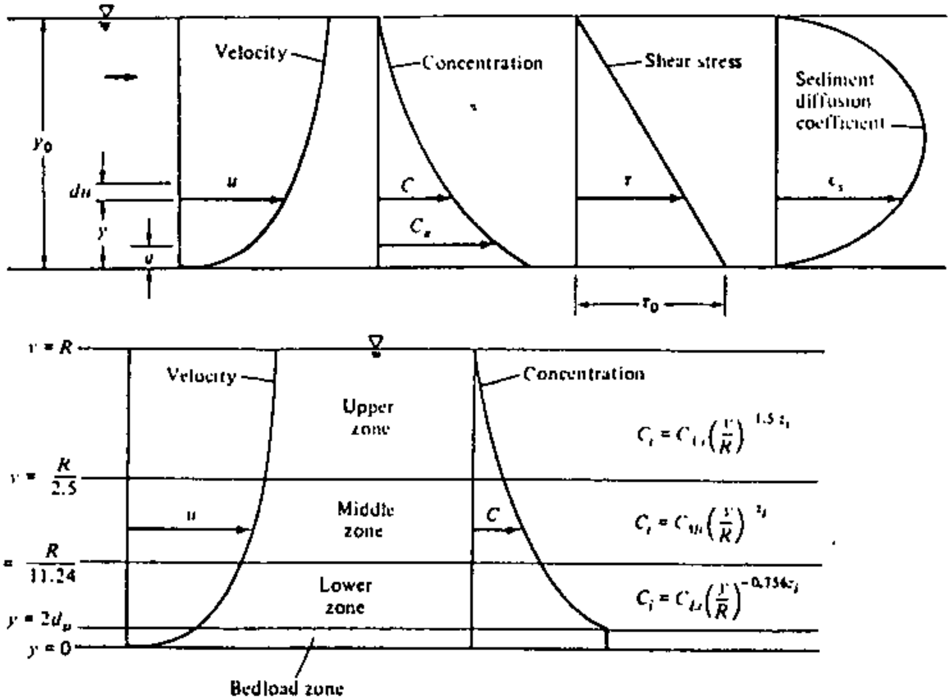
The velocity distribution was assumed to follow the one-seventh power law as

$$u = U(1+n_v)\,(y/R)^{n_v} \qquad \ldots(2.253)$$

where $n_v = 0.1198 + 0.00098\, T \qquad \ldots(2.254)$

and again T is in degree F.

Toffaleti found that the concentration curve in the three regions could be fitted to the following equations, as shown in Fig.(2.11).




Definitional sketch for the Toffaleti method.

FIG.(2.11) TOFFALITI'S CONCENTRATION MODEL.

$$C_i = C_{L_i} (y/R)^{-0.756 z_i} \qquad \text{(lower zone)} \qquad ...(2.255)$$

$$C_i = C_{M_i} (y/R)^{-z_i} \qquad \text{(middle zone)} \qquad ...(2.256)$$

$$C_i = C_{U_i} (y/R)^{-1.5 z_i} \qquad \text{(upper zone)} \qquad ...(2.257)$$

where $C_{L_i}$, $C_{M_i}$ and $C_{U_i}$ are the concentration coefficients evaluated from the continuity of the suspended sediment concentration profiles.

The suspended load can be found by adding the calculated suspended load in the upper, middle and lower zone. Meanwhile the bed-load is found by taking the product of bed layer concentration and the bed velocity (both at y = 2d). Finally the bed material load is the direct sum of bed-load and suspended load .

This method is rather lengthy and is best handled by a computer program. The results gave reasonable estimate of sand transport, particularly in large rivers, but it is not recommended in case of gravels transport.

*Shen & Hung*[110], in 1971, accepted the fact that the total sediment transport is, in the present state of knowledge, not subjected to a formal description in terms of a few parameters. They, therefore, recommended a relationship to be fitted to the available data by regression techniques.

They selected the bed material transport concentration $C_T$ by weight as the dependent variable and w (fall vel.), $d_{50}$ in mm, U(ft/sec), $y_o$, and the energy slope S as the independent variables. The fall velocity is corrected for water temperature, Their equation is

$$\text{Log } C_T = a_o + a_1 y + a_2 y^2 + a_3 y^3 \qquad \ldots(2.258)$$

in which

$a_o$ = -107404.45938164

$a_1$ = 324214.74734085

$a_2$ = -326,309.58908739

$a_3$ = 109,503.87232539

and; $y = U\, S^{0.57}/w^{0.32}$ ...(2.259)

This was tested for 587 sets of laboratory data in the sand size range. Any wash load should be added separately.

*Ackers & White*[111], in 1972, based on Bagnold's stream power concept, applied dimensional analysis to express the mobility and transport rate in terms of some dimensionless parameter. They postulated that only a part of the shear stress in the channel bed is effective in the movement of course sediment, while in the case of fine sediment, suspended load movement predominates and the total shear stress is effective in causing the movement of sediment. They proposed a design chart of mobility number $F_{gr}$ versus a dimensionless grain size $D_{gr}$ where ,

$$F_{gr} = u_*^n/[gd(S_s-1)]^{1/2}[U/(32)^{1/2} \log(10D/d)]^{1-n} \qquad \ldots(2.260)$$

in which $u_*$= shear velocity, n= transition exponent depending on the grain size; For course grains n=0, and for fine grains n= 1, and d = $d_{35}$.

Also;

$$D_{gr} = d\ [g(S_s - 1)/\nu^2]^{1/3} \qquad ...(2.261)$$

Then a general dimensionless sediment transport function can be expressed as;

$$G_{gr} = XD_o/S_s d\ (U_*/U)^n \qquad ...(2.262)$$

in which

X= the rate of sediment transport in terms of mass flow per unit mass flow rate. The transport rate is related to $F_{gr}$ by

$$G_{gr} = [C(F_{gr}/A)-1]^m \qquad ...(2.263)$$

where A is the value of $F_{gr}$ at a nomial initial movement; and the four constants A,C,m,and n are all functions of $D_{gr}$.These constants were determined by optimization techniques applied to the existing data; The values of C= 0.025 and m= 1.5 were assumed to be constant.

Their equations can be applied to the lower flow regime with different bed forms such as plane, ripple and dune.

*Yang* [112,113], (in 1972,1976), approached the total transport from the energy rate or stream power concept.

The unit stream power can be expressed by the velocity and slope product. Yang's dimensionless unit stream power equations can be expressed in basic form;

$$\log C_t = \alpha + \beta \log(US - US_c) \qquad ...(2.264)$$

where $US_c$ is the critical unit stream power required to start the sediment movement. This is made dimensionless by writing

$$\log C_t = I + J \log(U_s/w - U_c\ S/w) \qquad ...(2.265)$$

where $I = A_1 + A_2 \log(vd/\nu) + A_3 \log(U_*/v)$ ...(2.266)

$J = B_1 + B_2 \log(vd/\nu) + B_3 \log(U_*/v)$ ...(2.267)

Then using available data (laboratory) and multiple regression techniques *Yang*[113] power equation (1976) is:

$$\log C_t = 5.435 - 0.286 \log(vd/\nu) - 0.457 \log(U_*/v)$$
$$+((1.799-0.409 \log(vd/\nu)-0.314 \log(U_*/v)) \log[(US/v)-(U_cS/v)]$$ ...(2.268)

The values of $U_c/v$ are given by

$U_c/v = 2.5/(\log(U_*d/\nu)-0.06)+0.66;\ 0<(U_*d/\nu)<70$ ...(2.269)

and $U_c/v = 2.05;\ 70 \le (U_*d/\nu)$ ...(2.270)

in which: $C_t$ = the total sediment weight concentration (ppm).

$v$ = the average fall vel.

$d$ = the median diameter $d_{50}$.

$U_*$ = shear velocity.

$\nu$ = the kinetic viscosity.

$U$ = the average flow velocity.

$U_c$ = the average flow velocity at incipient motion.

$S$ = the energy slope.

$US$ = the unit stream power.

$US/v$ = the dimensionless unit stream power.

*Maddock*[110], in 1969, used an empirical type approach to express the total sediment concentration which includes the wash load as a function of unit stream power

$$C_t = \{10^3\ US/\phi(d) - 60(S_s-1)g^{1/2}d/\phi(d)\ D^{1/2}$$

$$[(S_s-1)gd/v^2]^{1/4}\}^{4/3} \qquad \ldots(2.271)$$

in which $\phi(d)$ = a function of median diameter of sediment. a graphical solution of $\phi(d)$ was given by Maddock ,the above equation is dimensionally non-homogenous.The dimensions of

$C_t$ is in part per million by weight.

U is in ft/sec; g is in ft/sec$^2$

d is in mm, D in ft, and w in ft/sec.

*Yang*[114] later, in 1979, introduced a unit stream power equation for total load without using any criterion for incipient motion.This equation was compared with a similar dimensionless unit stream power equation introduced by him before, with the inclusion of criteria for incipient motion. Comparision between the measured results from laboratory data and natural rivers data with the computed results from the two stream power equations indicated that they are equally accurate in predicting the total sediment concentration in the sand size range. For the 1259 sets of data compared, the average computed result from these equations is only 3% higher than the measured results. A simplified unit stream power equation can be written as

$$\log C_t = 5.165 - 0.153\log(wd/\nu) - 0.297\log(U_*/w)$$
$$+ [1.78 - 0.360\log(wd/\nu) - 0.480\log(U_*/w)]\log(US/w) \quad \ldots(2.273)$$

*Hideo Kikkawa and T. Ishikawa*[117], from Tokyo Institute of Technology, in 1978; obtained a mathematical expression of the mutual relation between bed-load and suspended load by developing a model of sediment particle motion.

As a particle moves downsteam, it turns from a component of suspended load into one of bed-load and vice versa. The characteristics of motion seems to be very different in each case; In the case of suspension, particles motion is affected by the fluctuations of fluid dynamic force or the "diffusion effect". On the other hand, the bed-load is subject to upward force whose average is equal to the gravity acting downward or the "upward force effect". For this purpose, a stochastic model of particle motion is proposed based on observation and physical considerations.They concluded the following results:

1- An expression of concentration distribution is derived without introducing the reference concentration.

2- The total load equation was found to be in good agreement with the flume data.

The eddy viscosity $\epsilon_m$ is represented by

$$\epsilon_m = k/6\ U_*\ D \qquad \ldots(2.274)$$

the ratio of the eddy diffusion coefficient of mass to eddy viscosity $\epsilon_s/\epsilon_m$ was examined experimentally by *Ismail*[87], its value is in the range 1.2-1.3. The following relation is assumed by them :

$$\epsilon_s = 1.2\ \epsilon_m$$

Considering the velocity distribution in an idealized flow as;

$$u/U = a(y/D - (y/D)/2) + b \qquad \ldots(2.275)$$

in which $a = (6/k)(u_*/U)$

$b = 1 - (2/k)(u_*/U)$

If this equation is considered, the total load of bed material $q_T$ is calculated from ;

$$q_T = \int_0^D u\,C\,dy \qquad ...(2.276)$$

The average concentration is ;

$$C_T = (d/D)\, f_2(\tau_*)\, [\,a\alpha\{(\beta - 1/\beta) + (-\beta^2 + \beta + 1)/2\beta^3 \exp(-\beta)\} + \alpha b/\beta\, (1 - \exp(-\beta) + b] \qquad ...(2.277)$$

where

$$f_2(\tau_*) = 0.88\, \tau_*\{\phi((1.52/\tau_*) - 2) + 0.199 \exp\, [-1/2(1.52/\tau_* - 2]^2\} \qquad ...(2.278)$$

in which,

$$\tau_* = U_*^2/(\rho_s/\rho - 1)gd \text{ and } \phi(x) = 1/(2\pi)^{1/2} \int_x^\infty \exp(-t^2/2)$$

The dimensionless total load is expressed as;

$$q_* = q_T/u_* d \qquad ...(2.279)$$

*Ranga Raju, J. Garde et.al.*[118], in 1981, used the concept of effective shear stress,and applied this concept to data collected in flumes and natural streams which was available.

According to *Vital et.al*[119], the dimensionless sediment transport parameter $\phi_T$ was closely related to the dimensionless shear stress for flow with a plane bed as:

$$\phi_T = g_T/\gamma_s(\rho/\Delta\gamma_s)^{1/2}(1/(gd^3)^{1/2}) \qquad ...(2.280)$$

and $\tau_* = \tau_o/\Delta\gamma_s d$ $\qquad$ ...(2.281)

in which $g_T$ = rate of total load transport (by weight) per unit width and $\Delta\gamma_s = \gamma_s - \gamma_f = g(\rho_s - \rho)$

*Vital et al*[119], defined the shear stress $\tau_t$ for ripple and dune beds as the shear stress required to give the same total load transport of the same size material on a plane bed.

Investigation done before showed that $(\tau_t/\Delta\gamma_s.d)$ is uniquely related to $\tau_*^1$ here $\tau_*^1=\tau_o/\Delta\gamma_s.d$ and $\tau_o^1$=grain shear stress = $\gamma_f\ R_b^1$, where $R_b^1$ is given by

$$U= 24/d^{1/6}\ R_b'^{2/3}S^{1/2} \quad \text{(S.I units)}$$

Raju[118] et al. investigations showed that no unique relaion exists between $\tau_*^1$ and $\tau_t/\Delta\gamma_s.d$. So they try to provide a prediction of $\tau_t$ by introducing a functional relationship as

$$\tau_t/\tau_o^1= f(\tau_o^1/\tau_o,\ U_*/w) \qquad \text{...(2.282a)}$$

From the given data the above equation can be expressed as;

$$\tau_t/\tau_o^1= (\tau_o/\tau_o^1)^{-m} \qquad \text{...(2.282b)}$$

in which m is a function of $(u_*/w)$. In the case of no suspension , m= 0 and $\tau_t = \tau_o^1$, this is known when $u_*/w \le 0.5$. Otherwise the value of m is given by;

$$m = 0.2(u_*/w) - 0.1, \quad \text{for } u_*/w \ge 0.5 \qquad \text{...(2.282c)}$$

Since $\phi_T$ is a function of $\tau_t/\Delta\gamma_s.d$ .

Then, $\phi_T= f[\tau_*^1(\tau_o^1/\tau_o)^{-m}]$ ...(2.283)

and this is valid for ripple-dune and plane bed regimes.

The last equation (2.283) can be expressed as

$$\phi_T= 60\ {\tau_*^1}^3\ (\tau_o^1/\tau_o)^{-3m} \qquad \text{...(2.284)}$$

in the range of

$$0.05 \le \tau_*^1\ (\tau_o/\tau_o^1)^{-m} \le 1.0$$

Based on regression analysis of laboratory and field data, *Brownlie*[120], in 1981, obtained the following equation for concentration of total load ppm by weight:

$$C_T = 7115\, C_F \left(U/((\Delta\gamma_s . d)/\rho)^{1/2} - u_c/((\Delta\gamma_s . d)/\rho)^{1/2}\right)^{1.978}$$

$$* S^{0.6601} (R/d)^{-0.3301} \qquad ...(2.285)$$

in which $C_F$ is a coefficient equals unity for laboratory data and 1.268 for field data.

*Karim and Kennedy*[121], in 1983, carried out a regression analysis of the sediment data. Their equation is:-

$$\log q_T/[(\rho_s/(\rho-1))gd^3]^{1/2} = -2.786 + 2.9719\, V_1 + 0.2989\, V_1 V_2 + 1.06\, V_1 V_3 \qquad ...(2.286)$$

where $V_1 = \log U/[(\Delta\gamma_s/\rho)d]^{1/2}$

$V_2 = \log (u_* - u_{*c})/[(\Delta\gamma_s/\rho)d]^{1/2}$

However *de Vries*[122], in 1983, found that the accuracy of the above method to be less than those of Engelund[108] and Ackers-White[111].

*Yang and Molines*[115], in 1982, compared seven total load equations derived from the concept that the rate of sediment transport should be related to the rate of energy dissipation of the flow,these equations were Colby[103] approach *Ackers and White*[111], *Engelund and Hansen*[108], *Shen and Hung*[110], *Yang*[115] and *Maddock's*[116] approach.

Using 1,259 sets of data in the sand size range indicated that the equation proposed by Yang, Engelund and Hansen, and

Ackers and White are more accurate than others under laboratory and field conditions,mean while Shen and Hung's equation and Maddock's equation showed good agreement in case of laboratory flumes: finally Colby approach should not be applied to the laboratory flumes, also this method underestimates total sediment load in natural rivers.

# APPARATUS AND EXPERIMENTAL PROCEDURE

# CHAPTER 3

# APPARATUS AND EXPERIMENTAL PROCEDURE

## 3.1. DESCRIPTION OF THE APPARATUS

### 3.1.1. INTRODUCTION

The experimental work was conducted in a 10-m long,glass sided tilting flume, located in the Hydraulics and Fluid Mechanics Laboratories in The Civil Engineering Department, at The University of Jordan. All experiments were carried during Summer, 1988, although preparations and modification of the apparatus started one year earlier. A general view of the apparatus is shown in plate (3.1).

### 3.1.2. THE TILTING FLUME:

The glass sided tilting flume, as illustrated diagramiti-cally,is a fully self-contained 10m in length, 0.3 m in width as 0.45 m in depth. The base frame is a steel box section, bolted together through end flange plates, the channel bed is manufactured from a cold rolled steel, fully machined for accuracy. Pressure tappings are provided in the bed of the flume.

The sides are manufactured from toughened glass and are supported by cast aluminium cantilevers connected to the bed. The flume, before some adjustments, was fed through an inlet tank. After passing through the working rectangular section, the water travels by way of an adjustable overshot weir (tail gate).

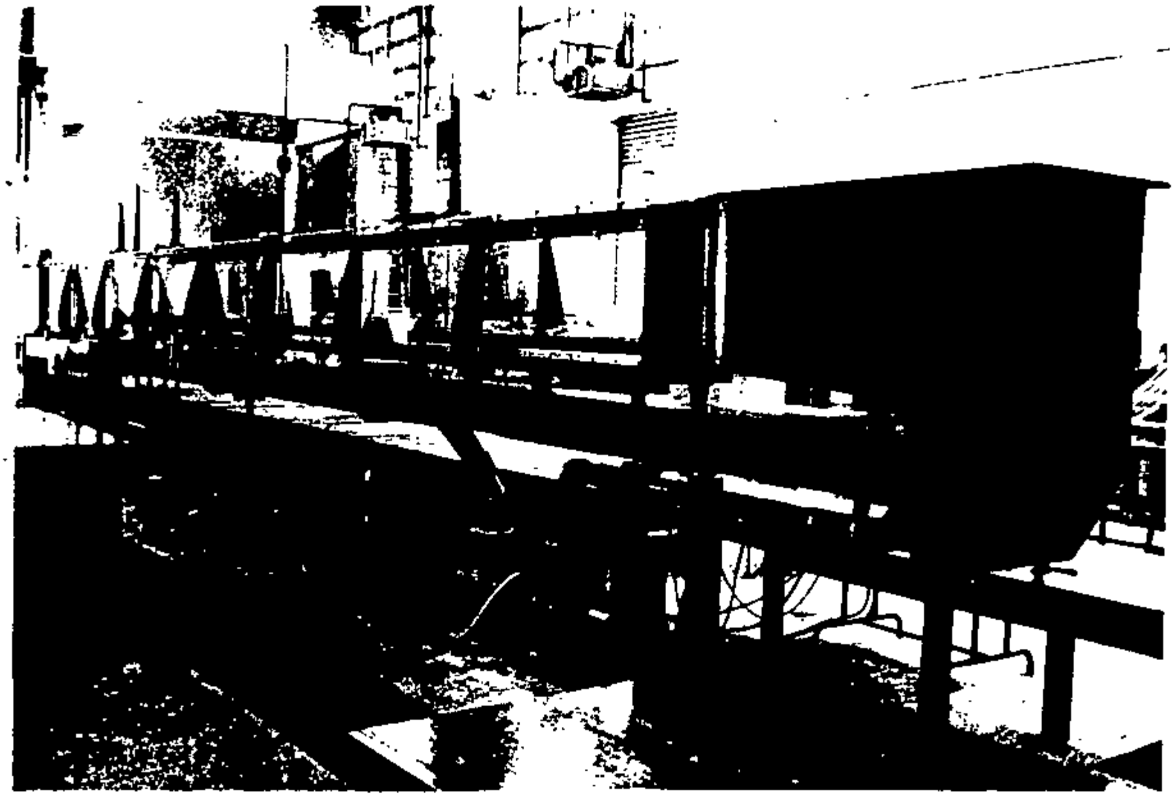


PLATE (3.1) GENERAL VIEW OF THE FLUME.

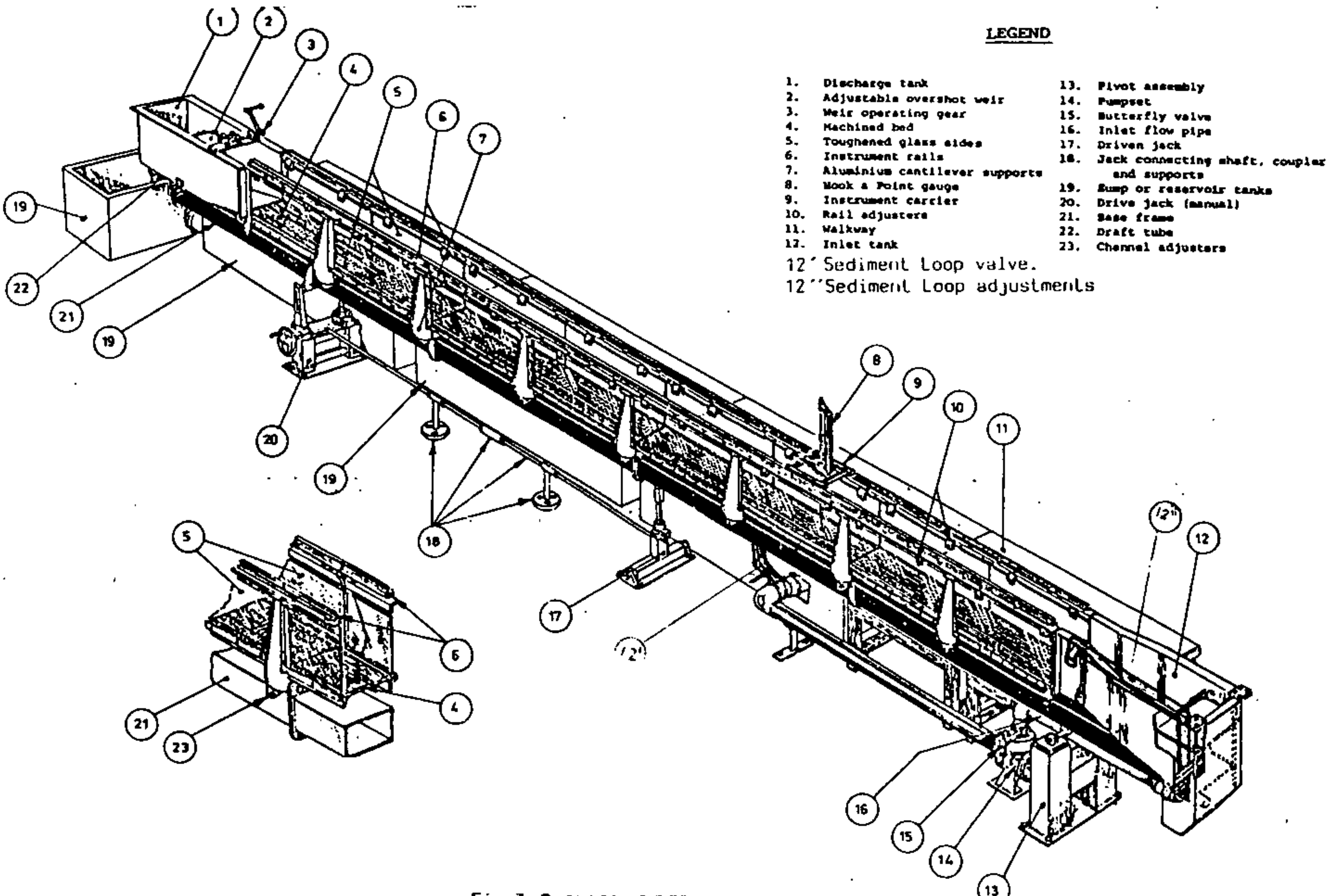



Fig.3.2 GLASS SIDED TILTING FLUME

Some adjustments are needed to establish a close circuit sediment loop, through which the sediment circulates without having any possibility of deposition. Thus the down stream side of the flume was fully seperated from the down stream tank the tail-gate in semi-vertical position (closed). The circuit was then changed to be a closed loop with a control volume flow rather than taken the water from the tanks, this was done by closing the tank valve and oppening the close circuit valve before the pump.

To avoid deposition in the inlet side of, the upstream, of the flume, a PVC. pipe connections of 10 cm in diameter were provided to ensure that water, at the inlet side, came directly to the flume working section, through these pipe connections, ( Fig. 3.2.).

Both manually and electrically operated screw jacks are provided for bed slope variations,which can easily give accurate slopes in both the positive and negative range. The slope indicator scale is provided to give slopes up to 1:40 and negative slopes up to 1:200 as shown in Plate (3.3).

The top flanges of the flume working section carry a pair of accurately aligned instrument rails. To one of these rails is affixed a longitudinal positioning scale calibrated in millimeters. On these rails the instrument carriers move along the working section carring either depth gauge, scraper or any other instruments.

The control board contains bottons which electrically operate the pumpset, the valve, and a meter for measuring flow rate.

### 3.1.3. MOVABLE CARRIAGES:

The flume is equipped with four carriages which can be moved on rails along the flume length. Each carriage has a jockey which moves in the transverse direction. On these carriages the point gauge, the levelling scraper, the static pitot tube set and suspended load sampler can be mounted easily. These apparatus can be moved easily to any location on the flume working section.

### 3.1.4. OTHER EQUIPMENTS:

1- The Point Gauge:

Water surface and bed levels are measured by a point gauge which is supported on a sliding carriage. The pointed end, shown in plate (3.4) is mounted on graduated red actuated by a slow motion screw equiped by a vernier for accurate reading to 0.1 mm.

2- The Levelling Scraper:

For levelling the sand bed, a scraper was made of a perspex angle, just shorter than the flume width. Its lower surface was carefully smoothed to a straight edge.

The perspex angle, is clamped to a depth gauge which was adjusted to the level of a marble apron, and suspended from a carriage travelling on the flume rails. The sand was levelled before each run by sliding the scraper over the wet sand with a thin film of water for several times,this scraper is shown in plate (3.5).

3- The Static Pitot Tube:

The velocity profiles were measured by the use of Prandtl-type pitot tube with internal diameter of 1.0 mm. The pitot tube is clamped to a depth guage and connected to a differential inclined manometer using water as a manometer liquid; as shown in Plate (3.6).

4- The Manometer Board:

The pitot tube assembly is connected through a flexible tubes of 5.0 mm internal diameter to an inclined differential water manometer .

The inclination of the manometer is made to magnify the differential manometer reading at low velocities. This magnification is about 4.31 times greater than the vertical normal position of the manometer as shown in Plate (3.7). Both of the manometer tubes are of 8.0 mm internal diameter which seems large enough to eleminate capilary effect, and their ends are opened to its atmosphere. Readings was taken to the nearest 1.0 mm on the inclined manometer

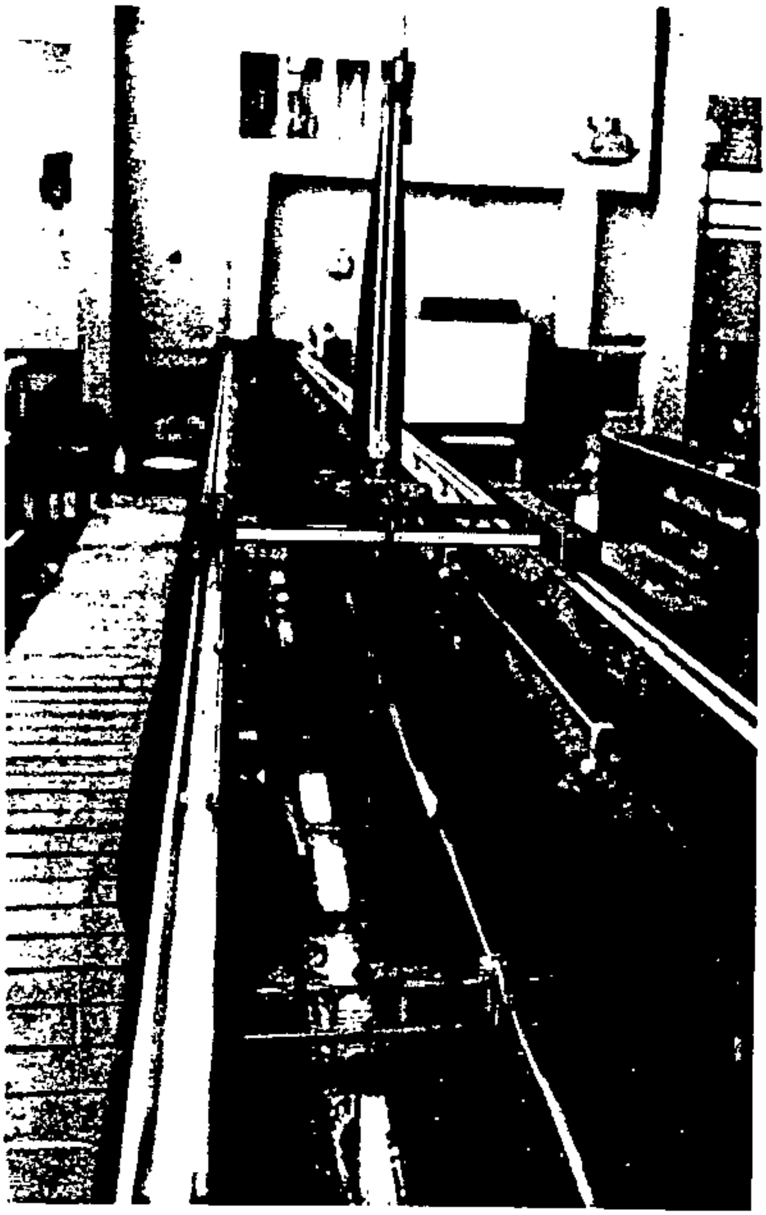


PLATE (3.5) THE LEVELLING SCRAPER.

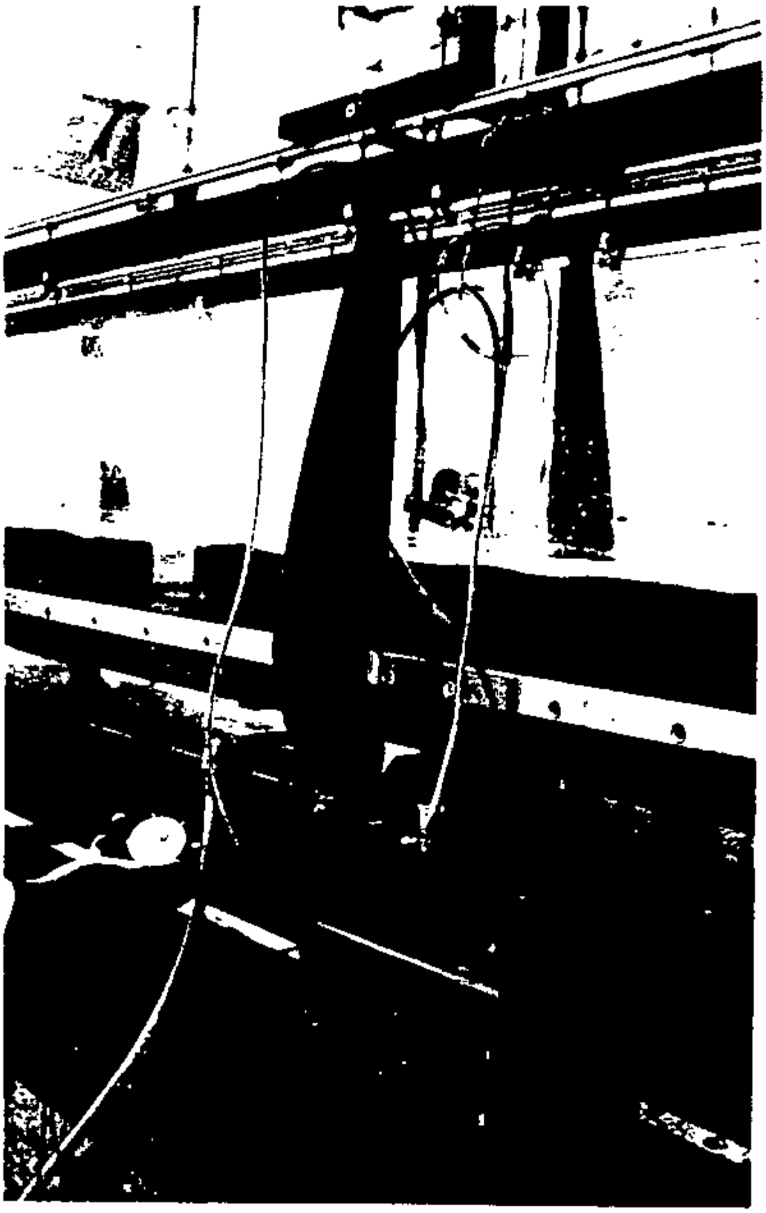


PLATE (3.6) THE PITOT TUBE & SUSPENDED LOAD SAMPLER [GENERAL ARRANGEMENT].

5- Suspended Sediment Samplers

The distribution of sediment was determined from samples siphoned from the flow through a glass tube of 3.0 mm internal diameter, shaped much like a pitot tube. The tip of this sampler was sharpened from the outside, with the inside diameter being unchanged, this was done to minimize the disturbance to the flow. The average velocity at which that sediment-laden water entered the tip of the sampler was made equal to the local stream velocity at the sampling point by adjusting the head on the siphon, the local stream velocity was measured momentarily using a pitot tube assembly mentioned above.Then the time required to fill a 154 ml volume was calculated using the local velocity of the stream and the cross sectional area of the sampler tip, thus the head on the siphon was adjusted by trials so that the actual time of filling is the same of the pre-calculated value .The suspended load sampler is shown in Plate (3.6) above.

6- The Bed-Load Trap and Samplers:

Making use of the pressure tapping holes in the bed of the flume, a bed-load trap of a funnel shape was designed to measure the bed-load rate.This trap as shown in Plate (3.8). was located at station 7.80 m from the start of the working section.It was made of stainless steel with 80mm * 80 mm plan and a 50 mm depth, the sides were sloping at an angle of 40° which is greater than the angle of repose of the experimental sand (30°) to ensure instability of sand grains on the sides.

The exit of this trap was connected to a pipe of 12.5 mm internal diameter with a ball type valve, this pipe leads to the bed-load samplers or bottles. A brass wire mesh was used on the top of the trap as a baffle to eleminate vortices formation during bed-load sampling.

7- Other Sampling Apparatus:

3 bottles of 1.0 L each were sampled for bed-load at each trial, the valve of bed-load sampling was opened for about 15 min.before each trial and was allowed to discharge in an external container to avoid non-uniformily of sampling due to valve closure. The sampled sediment laden water bottles were weighed after being filled using a balance of 1.0 gm (least count).These bottles were filled with clear water and weighed again.The difference between the two weights is the submerged weight of the sediment being traped.

6 bottles of 154 ml volume each were used in suspended load sampling. The submerged weight of the suspended sediment was found in similar way to that of the bed-load but using an electrical balance of 0.0001 g accuracy. A stop-watch was used to record the actual time of sampling.An air jet and a cotton cloth are used to dry the bottles from the outside.

## 3.2. General Arrangements:

The sand bed 5 cm in thickness occupied the recess, about 9.0 m in length, formed by the marble aprons placed at the upstream and the downstream ends of the working section.The upstream apron was very effective in damping the turbulence of the water and preventing the local scour and irregularities of the sand bed. The 5 cm layer of sand mentioned above was formed of 2.5 cm of fixed bed sand treated by special type of varnish emulsion overlaid by another 2.5 cm of uniform experimental sand of median diameter of 0.15 mm.

### 3.3. The Experimental Sand

#### 1- Characteristics of Sand

The sand used for these experiment was brought by the present auther from the sand dunes piles found in Wadi Arabah South of Jordan, about 200 Km south of Amman. It is dark and yellowish in colour. The sand is predominatly Silica sand and the specific gravity of it was measured experimentally in the Soil Mechanics lab. and was found to be 2.65 .The sand is quite subrounded as shown in plate (3.9),since it was tranported by air from the arabian desert for many thousands of kilometers.

#### 2- Sieve Analysis Procedure:-

The natural sand was sieved using six standard sieves which were shaked mechanically for twenty minutes . Sand contained between successive sieves is collected and stored in marked containers.The experimental sand used in these experiments was that portion which passed sieve size 0.202 mm (sieve # 70) and retained on sieve size 0.104 mm (sieve #140) .

Thus the experimental sand used in our case could be considered as uniform sand of mean diameter 0.15 mm.

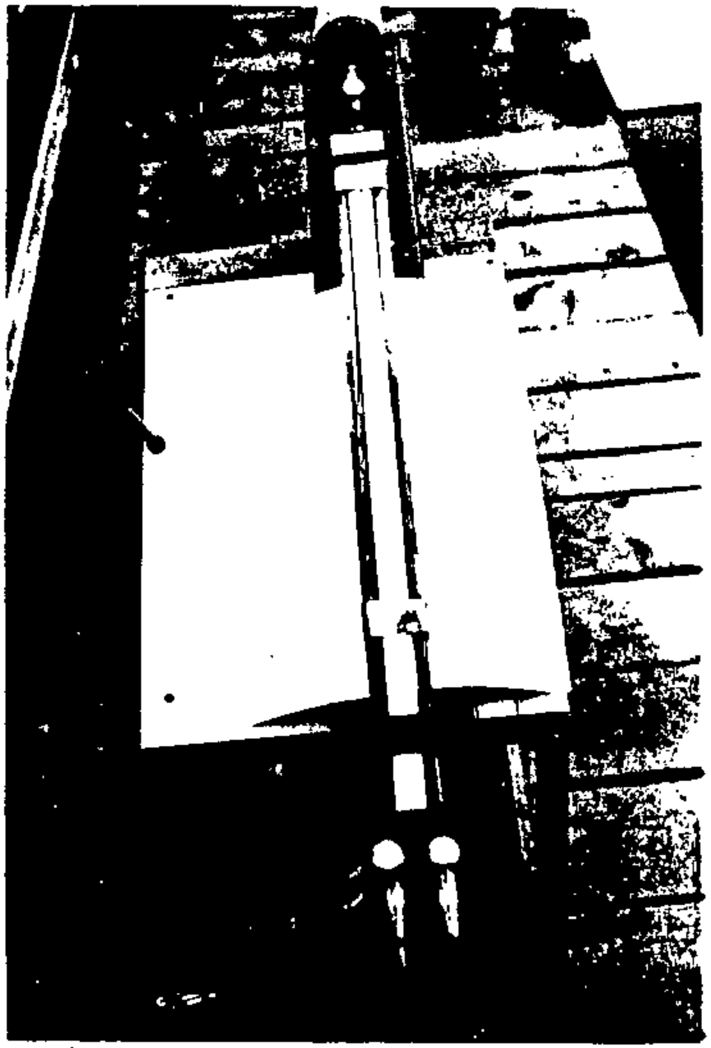


PLATE (3.7) THE INCLINED MANOMETER BOARD.

PLATE (3.8) THE BED-LOAD TRAP.

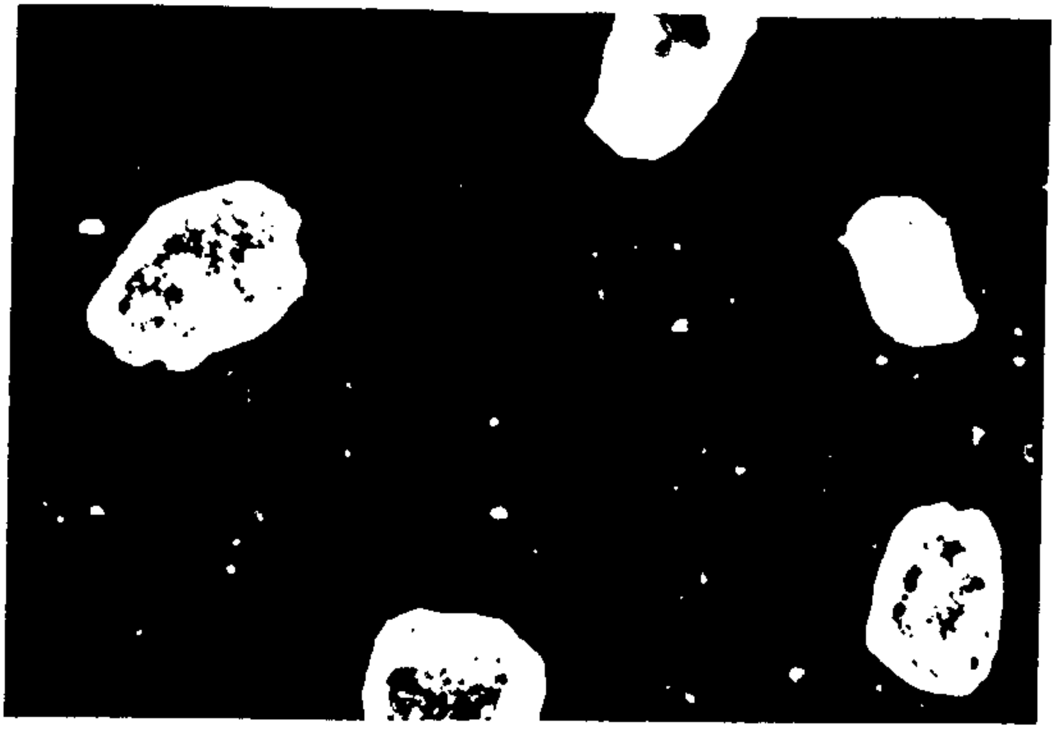


PLATE (3.9) MICROSCOPIC VIEW OF THE EXPERIMENTAL SAND [$d_{50}$ = 0.15mm].

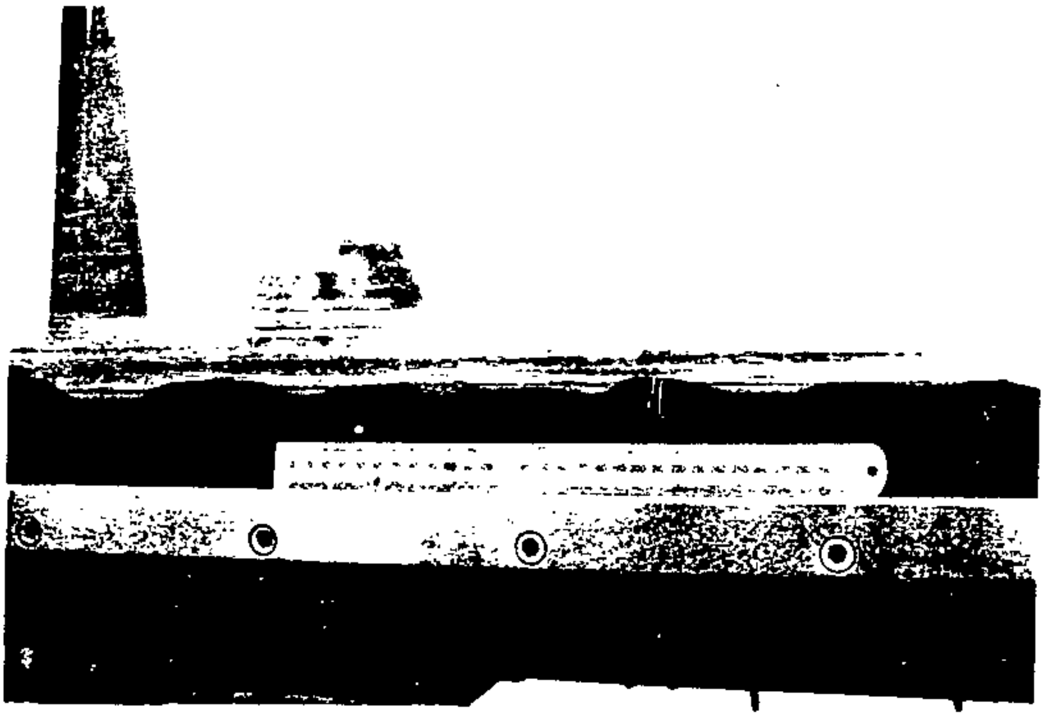


PLATE (3.10) TYPICAL VIEW OF RIPPLES.

### 3.4. Experimental Procedure:-

Intially the flume was set to a horizontal position (zero slope).Then it was checked for being horizontal by keeping an amount of water in it and read the water depth at a number of station ,these readings were found to be constant.

#### 3.4.1. Preparation of the Sand Bed:-

With the flume horizontal, a well dressed two marble sills of 50 mm thickness each was prepared and placed horizontally on the upstream and downstream ends of the working section. These sills were fixed to the channel bed by some type of mastic.

A 2.5 cm layer of a wet sand was placed between the two sills, compacted and levelled by a trowel and then using a a levelling scraper.

This layer was stablized, to prevent mixing between its material and the uniform sand layer on its top. Stabilization was done by spreading a typical nitro-cellutose lacquer with a natural resin, namely tinner, all mixed with some amount of adhesive. Trial mixes of these materials were sprayed over an external sand layer and then checked. The best mix was found to be 45% lacquer, 40% tinner and 15% adhesive all by volume.

This stabilizer was sprayed at uniform rate using pressure paint-sprayer gun, keeping the spraying gun parallel to the surface at a distance of 15 to 20 cm.When the distance was less, the sprayer tended to dislodge the grains.

No difficulties were faced since the stabilized layer was thin and horizontal. This layer was left to dry for 48 hours.

Another layer of 2.5 cm uniform washed sand, with a mean diameter of 0.15,was placed. Additional amount of sand was added to ensure effective scrapering. Thus the whole recess between the two marble aprons was filled with sand;a fixed layer of 2.5 cm overlaid by a 2.5 cm loose layer of the experimental sand having mean size of 0.15 mm.

3.4.2 Experimental Runs:-

In performing the experiments, the slope, and the water depth are the independent variables, all other hydraulic parameters are the dependent variables.

After levelling the loose sand bed using s scraper with a thin film of water, the flume circuit was filled with some water.The pitot tube and manometer assembly were primed also.

The flume is set to the required slope by means of electrically operated screw jacks, once the pump started, the valve was opened gradually till uniform flow is achieved. This was checked by measuring the water depth at two stations 6.0 m apart. This depth was recorded, the flow rate was then measured.

Near the bed-load trap, the velocity profile was measured using the static pitot tube and inclined differential water manometer, these measurements extended over the whole depth of water at a 5 mm step in the vertical directin.

Once the velocity was determined at any level, the suspended load sampler was adjusted to that level, and the sediment-laden water was siphoned through a 3.0 mm internal diameter glass tube, described above. The average velocity at which the sample entered the tip of the sampler was made equal to the stream velocity at the sampling point by adjusting the head on the siphon.This was done as follows:

The time to remove 154 ml of sample was calculated for each point from the measured stream velocity and the cross sectional area of the sampler nose.The head on the siphon for that rate of flow was determined by trial,till the difference between the actual measured time and the calculated time of sampling was within 0.5 sec. 3 samples were taken at each point.

These bottles, with the sediment-laden water,were weighed using an electrical balance of 0.0001 g accuracy. This weight was recorded and called $W_{sw}$. The bottles weight filled with clear water were taken also using the same balance, and this weight was called $W_{w}$, then the submerged weight of the sediment is simply equal to ($W_{sw}$-$W_{w}$).

Also for the bed-load measurements the value of the bed-load trap was opened first for 15 min. to eleminate the effect of valve opening and closure. Then three sample of one litre each were taken and the time required for these bottles to be filled was recorded. The submerged weight of the sediment was measured in the same way followed in the suspended sediment sampling but using a balance with 1.0g accuracy.

Another set of bed-load samples was taken after one hour from the first set. The average value of these 6 samples was considered in calculations.

Additional water was added at each sampling operation, to keep a constant uniform depth of the flow (constant volume).

Visual observation of bed-forms was made with simple measurement of ripple dimensions.

The water temperature was taken at each run.

In this work eleven experiments were conducted following the above mentioned procedure. Any suspicious observations were repeated.

All the experimental results and calculations are summarised and tabulated in Appendix A.

These experiments covered a wide range of slopes 1:200, 1:300, 1:400, 1:500 and depthes ranges from 3.0 cm to about 6.0 cm.

Using the Sediment Particle Apparatus the terminal velocity for a single grain of the experimental sand was measured in clear water .This was done by measuring the distance travelled by the grain ,which is fixed to one meter,per unittime.The average value of the fall velocity for the 0.15 mm single sand grain in clear is 0.0147 m/s.

# ANALYSIS AND DISCUSSION OF EXPERIMENTAL RESULTS

# CHAPTER 4

# ANALYSIS AND DISCUSSION OF EXPERIMENTAL RESULTS

## 4.1. INTRODUCTION

The present work describes an experimental investigation concerning the sediment load in the open channel flow. In these experiments the total load is estimated by measuring the two parts composing it, namely the bed-load and the suspended load.

The main objective of this study is to find a correlation between the suspended load concentration and that of the bed-load. Establishing such a relation may be helpful in estimating the sediment discharge of any stream flow based on hydraulic measurements and sediment properties. The measured values of both bed-load and suspended load are compared with the corresponding values estimated by other investigators.

In this chapter the relationship between the bed-load concentration and the suspended load concentration at certain reference level will be discussed.The effect of sediment concentration on the velocity distribution profiles, namely on the turbulent coefficient (k), and on the suspended sediment distribution exponent (z), will be also discussed.

## 4.2. THEORETICAL APPROACH

### 4.2.1. THE BED-LOAD

The theoretical derivation introduced by *Khalil*[28], in 1963, will be followed in the present work. This derivation has been reviewed in Chapter (2).

For the experimental sand used in the present experiments $d_{50}$ equals to 0.15 mm and its specific gravity = 2.65. The critical shear stress $\tau_c$ to initiale motion of grains was evaluated using the *Shields'* curve given in (Fig.2.1). This was estimated to be 0.194 $N/m^2$

According to *Khalil*, when there is less than one complete layer of grains in transport i.e $\int_o^y N\,dy < N_* d$ , then

$$\int_o^y N\,dy = \tau_b - \tau_c(1-(N/N_*))/(\rho_s-\rho)g\cos S(\tan\phi - \tan S) \quad ...(4.1)$$

and when there is a complete layer of grains or more are in transport, i.e $\int_o^y Nd \geq N_* d$ , then

$$\int_o^y Nd = \tau_b/(\rho_s-\rho)g\cos S(\tan\phi - \tan S) \quad ...(4.2)$$

where $N_*$ is the maximum volume grain concentration which was taken to be 0.46 according to *Khalil's* work, $\phi$ is the angle of repose of the experimental sand and it equals 30°, y is the thickness of the moving layer, and S is the channel gradient.

The rate of transport $g_B$ is defined as the total mass of grains which in unit time passes a unit width of the channel. To evaluate the rate of transport knowing the volume of moving grains per unit area, it is necessary to find the average velocity of the moving grains.

In his study, *Khalil* found that the grain speed $U_g$ is in linear relation with $U_I$, the local velocity of flow calculated at the mid depth of the moving bed-layer. The constant of proportionality was found to be 0.83 for 1.79 mm sand and 0.88 for 0.65 mm sand; extrapolating this to the present sand, this factor of proportionlity is found to be 0.96 i.e.

$$U_g = 0.96 \quad V_I \qquad ...(4.3)$$

this means that the grain speed is almost equal to the local stream velocity.

The effect of grain concentration on their average speed was studied by *Khalil*. He made a plot of $N/N_*$, i.e the volume concentration of active grains per grain layer to the maximum volume grains concentration, versus the percentage reduction in the speed of free grains. The plot confirmed the relation given by percentage reduction $= 0.60(N/N_*)^{3/7}$ ...(4.4)

Now the grain speed can be evaluated with respect to the local velocity of flow and can be expressed as

$$U_g = 0.96 \, [1 - 0.6(N/N_*)^{3/7}] \, V_I \qquad ...(4.5)$$

Since $R_{**} = d(u_*/\nu)$, ranges from 4 to 7. It was shown that the grain size has little effect on the bed roughness.

Thus the bed-load transport rate can be expressed as

$$g_B = \{\tau_b - \tau_c(1-(N/N_*))\}/\tan\phi$$

$$* \, [0.96(8.5U_*)(1-0.6(N/N_*)^{3/7}] \qquad ...(4.6)$$

The above equation is used to estimate the transport rate on a flat bed (neglecting the bed form effect).

On rippled surface, the total shear stress is transformed to the bed as tangential and normal stresses. The part of stresses which contributes to the bed-load transport seems to be the one acting tangentially on the gentle slope of the ripple which exceeds the average stress due to reseversed stress acting on the leeward. The stress consumed to accelerate the grains up to the ripple crest should be subtracted from the tangential stress available, since this part of stress cannot be regained as the grains decelerate in the regions of no transport.

The velocity distribution on a rippled surface is not a function of $U_*$ only as the case of flat rough surface, but also depends on the ripple dimensions.

In the view of these factors, *Khalil* introduced a correction factor to equation (4.6) above to account for the effect of ripple formation on the bed-load transport. This empirical constant was estimated experimentaly by him to be 0.63.

### 4.2.2. THE SUSPENDED LOAD,

Starting from the difussion equation:

$$\omega C + \epsilon_s (\partial C/\partial y) = 0 \qquad ...(4.7)$$

and assuming a linear shear stress distribution as;

$$\tau_o/\tau_y = (D-y)/y \qquad ...(4.8)$$

with a logarithmic velocity distribution,a further relation can be written as;

$$du/dy = u_*/ky = ((\tau_o/\rho)^{1/2})/ky \qquad ...(4.9)$$

where k is the Karman's constant.

Due to Reynold's analogy, the shear stress can be expressed as;

$$\tau_y = -\rho\epsilon \, du/dy \qquad \ldots(4.10)$$

Using these equations,the general suspended load distribution equation can be derived.This is shown in Chapter (2) that is;

$$C/C_a = [(D-y/y)(a/D-a)]^z \qquad \ldots(4.11)$$

Taking $a = D/2$ (mid depth) and $C_a = C_{md}$ then the last equation reduces to;

$$C/C_{md} = (D-y/y)^z \qquad \ldots(4.12)$$

where $z = \omega/ku_*$,

The suspended load rate is given by

$$g_s = \int_a^D C \, U \, dy \qquad \ldots(4.13)$$

where a is the bed-layer thickness.

The suspended load calculation was carried out based on two approaches, namely:

(a) *Lane* and *Kalinske*[1] and (b) *Brooks*'[98] approach .

4.2.3. THE TOTAL LOAD

The total load is simply taken as the sum of the observed bed-load and the calculated suspended load. The total load will be compared with the results of different approaches namely;

(a) Engelund and Hansen's

(b) Yang et al.

(c) Graf's et al. .

and (d) Einstein's approach.

These approaches are selected because they are the most popular in use recently .They need a few sediment laden flow parameters to be measured and they have the least restrictions upon applicatin.All these approaches were reviewed earlier in chapter (2).

### 4.3. EXPERIMENTAL VERIFICATIONS,

#### 4.3.1. VERIFICATION OF THE BED-LOAD RELATION:

In the present experiments since $\tau_c$ is found to be 0.194 $N/m^2$ from Shields' diagram (Fig. 2.1). The number of layers set in motion, m,can be found from :

$$N\ md = \tau_b - \tau_c(1-(N/0.46))/(\rho_s - \rho)\ g\ \tan\phi$$

assuming that m = 1 layer

then $$N(0.15*10^{-5}) = \tau_b - 0.194(1-(N/0.46))$$

$$N = [\tau_b - 0.194(1-(N/0.46))][0.713].$$

Let N = 0.46 = $N_*$ ( the maximum possible concentration) then

$$0.46 = [\tau_b - 0.194(0)][0.73]$$

$\therefore\ \tau_b = 0.645\ N/m^2$ which is the necessary shear stress to set one complete layer in motion.

This value is less than the minimum value of available shear stress at the bed for all the experiments conducted in the present work (see table A.3). Then it seems that at least one complete layer has been set in motion.

Now moving in the opposite direction of *Khalil's*[28] evaluation for N, that is by assuming $N = N_*$ ,the number of moving layers m can be estimated as:

$$m = \tau_b / N(d)(\rho_s - \rho)\ g\ \tan\phi$$

for $\phi$ = angle of repose = 30°, $\rho_s$ = 2650 kg/m$^3$ then

$$m = \tau_b / 0.46(0.15*10^{-3})(1.65)(10^3)(0.81)\ \tan 30$$

$$m = \tau_b / 0.645 \qquad \ldots$$

The above relationship is valid for $N = N_* = 0.46$ ,one complete layer or more are set in motion.

After making some modifications so as to suit the present work Khalil's equation (4.6) reduces to :

$$g_B = A\ \tau_b / \tan\phi\ [0.96(8.5\ U_*)(4)]$$

$$g_B = A(5.650)(\tau_b U_*)$$

where A is an empirical constant taken as 0.63 for Khalil's work and is confirmed by the present work.

In the actual measurements the bed-load was measured using 8 cm sediment trap. Thus the bed-load rate per meter width = 100/8 * measured weight/sec.

The measured values are compared with the estimated value according to *Khalil's*[28] and *Kalinske's*[15] approaches outlined earlier. The comparasion is shown graphically in Fig.(4.1) and Fig. (4.2) respectively.

The estimated values and the measured values are found to be in good agreement.

In analogy to *Khalil's* empirical constant, the present constant is found to be 0.63 as shown in Fig. (4.3).

These results confirms *Khalil's* finding and verifies the soundness of his theoretical approach.

To calculate the bed-load concentration ,the flow and grains speed are considered to be equal, since the volume concentration was checked to be 0.46 by volume. so the bed-load concentration can be evaluated as

$$C = 0.46 \ m^3 \ grains/m'/(m^3 \ volume/m')$$
$$= 0.46 * 2.65 * 1000/(m^3/m')$$
$$= 1219 \ kg \ of \ sediment/m^3 \ of \ volume$$
$$= 1219 \ g \ of \ sediment/L \ of \ volume$$
$$= 1219 \ g/L.$$

The concentration value, although seems to be constant for all the experimental runs,it was estimated at different levels for each experimental test,since the number of moving layers in each test is variable.

4.3.2. VERIFICATION OF SUSPENDED-LOAD RELATIONS:-

1- The Effect of Suspended-Load on the Turbulent Constant:

Before the discussion of suspended-load analysis,it is convenient to study the effect of suspended load concentration on the flow characteristics.

a- Effect of Suspended-load on velocity distribution:

The measured velocity profiles are plotted on a semi-logarithmic scale as in Fig.(4.4), Fig.(4.5) and Fig.(4.6). It is shown that the velocity distribution fits, very well, the logarithmic law. The effect of suspended-load on velocity distribution is found to reduce the value of the von Kármán's turbulent constant, k,.

The value of k is evaluated experimentally from the slope of the relation log y versus the local stream velocity u, i.e

$$\text{Slope} = k/2.3\, U_*$$

or $$k = 2.3\, U_*\ (\text{slope})$$

The values of k for each experimental test are listed in Table (A.3) of the Appendix.These values are plotted versus the average concentration c as shown in Fig.(4.7) . This relationship between the values of k and the concentration can be presented on a semi-log scale by:

$$k = 0.3 - 0.025 \log C$$

the value of k according to the above equation reaches 0.4, when

$$\log C = -4$$

or $C = 1*10^{-4}$ g/L, which is the case of silt free water (clear water). This confirms the observation of many previous researchers; that k value is taken to be 0.4 for clear water.

The values of k ranges from 0.258 when the average concentration is 37.6 g/L to a value of 0.295 when the average concentration equals 2.47 g/L.Another point to be noticed is that values of k decreases as concentration of the sediment increases. This can be explained as, outlined earlier, to be due to the damping of turbulence. According to Vanoni[80], and *Ismail*[87] the decrease in the k value corresponds to an increase in velocity gradient.Thus for a plane bed, with a particular value of shear stress acting on the bed, the increase in suspended load causes the velocity gradient to increase near the bed, since

$$\tau = \rho \epsilon_m du/dy$$

Then the value of momentum transfere coefficient i.e turbulence is damped. Another way to explain the decrease in the k value, due to the presence of suspended-load , is based on energy concept; that is the power needed to keep the sediment particles in suspension is provided from the energy of turbulent fluctuations, which results in daming of the turbulence. These results conform the findings of *Vanoni*[86],*Ismail*[87], *Einstein and Chein*[88], *Vanoni and Nomicos*[39], *Itakura and Kishe*[33], *Goleman*[33], and many other investigator[33].

Another explanation to the decrease of the *Von Karman's* turbulence constant is due to the bed-forms.*Ali*[126] correlates the value of k with the ripple steepness (h/λ) assuming that when (h/λ) is around zero,the value of k approaches 0.4, and k was found to be (0.15) when h/λ= 0.08. The last concept seems to be not applicable to the present work,since in these experiments for (h/λ) > 0.08, yet k < 0.25.

From all the experiments carried by the author and others It has been found that the *Von-Karman's* constant is no longer a universal constant but a variable value which depends on the sediment concentration.

4.3.2.2. Distribution of the Suspended-Load Concentration:-

The suspended-load concentration equation (2.210) was verified by plotting the dimensionless depth (D-y)/D versus concentration on log-log scales. This relation was found to be a linear one. Thus with $C_a = C_{md}$ at y=D/2 the resulting equation is;

$$C/C_{md} = [(D-y)/y]^{z_1}$$

$$\text{or} \quad C = C_{md}\,[(D-y)/y)]^{z_1}$$

$$\text{then} \quad \log C = \log C_{md} + z_1 \log[(D-y)/y]$$

The value of $z_1$ was estimated from the slope of this linear relation, i.e

$$z_1 = [\log(C/C_{md})]/\log[(D-y)/y]$$

These relations are shown in Figs.(4.8) through (4.11).

On the other hand the values of z can be calculated from the relation given by,

$$z_{cal.} = \omega(u_* k)$$

where $\omega$ is the terminal settling velocity as affected by the grain concentration. The value of $\omega_o$ which is the settling velocity for a single grain in clear water is measured experimentally using the fluid particle system apparatus. For the experimental sand used ($d_{50}$= 0.15 mm), the value of $\omega_o$ is found to be 0.0147 on average. This value of $\omega_o$ is corrected for the effect of concentration according to the *Happel et al*[78] equation (2.142), given by $\omega_o$ as;

$$\omega_o = K_3 \omega$$

$$\text{where} \quad K_3 = 1 + 1.56(C_v)^{1/3}$$

and $C_v$ = is the volume concentration of sediment.

The volume concentration for suspended-load was measured and then the value of $\omega$ was evaluated for example for 1% volume concentration (Test number 6);then the resulting $\omega$ is

$$\omega = 0.0147/1.336 = 0.011 \text{ m/s}.$$

The value of k, the *Von-Karman's* turbulent constant, is found as outlined in section (4.3.2.1) earlier.

The two values of the exponent z ,mentioned before, $z_1$ for measured value and z for calculated one, are compared and plotted as shown in Fig.(4.12). The experiments showed that the calculated values of z is higher than the measured ones. The relation can be expressed as;

$$z = \beta \cdot z_1.$$

Also the sediment transfer coefficient $\epsilon_s$, was assumed to be constant in *Lane et al* approach and equals to the momentum transfer coefficient $\epsilon_m$. The present author found that these values are not equal. Since $\beta > 1.0$,they can be related as

$$\epsilon_s = \beta\, \epsilon_m.$$

The value of $\beta$ was found to be, (1.28) in the present experiment, which shows a good agreement with the previous investigations done by *Vanoni*[86] and *Ismail*[87],since for the present experimental size ($d_{50}$ = 0.15 mm), $\beta$ = 1.28 and *Ismail*[87] found that for sand size of 0.16 mm the value of $\beta$= 1.3 and $\beta$= 1.5 for sand size of 0.10 mm.This can be explained as;

1- Damping the fluid turbulence due to concentration gradient.

2- Changing in the turbulence characteristics due to the changing of bed configurations.

3- Changing in the fall velocity due to turbulence and concentration.

4- Slipping between the fluid and sediment particles due to flow acceleration.

5- Secondary circulations (secondary currents).

According to observations, it is not easy to correlate quantitatively the values of $\beta$ and the grain size, although it's clear that $\beta$ decreases as the grain size increases.

The suspended sediment distribution curves based on the present measurement are in a good agreement with those introduced by previous investigators such as *Vanoni*, *Ismail*, *Brooks* and others.

4.3.2.3: The Suspended-Load Rates:-

The suspended-load rate is evaluated by direct integration according to the equation:

$$g_s = \int_a^D C\ u\ dy$$

where a is the bed layer thickness. This integration is carried out graphically as shown in Figs. (4.13) through (4.23). In these graphes, the values of c are in g/L, u in m/s and y in cm, then the units of $g_s$ are;

$$g_s = g/L * m/s\ (cm)$$

or $g_s = g/0.001\ m^3 (m/5)(1/100) = 10\ g/ms$

thus each square unit represents a suspended-load of 10g/m.s.

These results are compared to those given by *Lane and Kalinske*[1] and *Brooks*[98]. The comparision is given in table (A.5) and represented graphically in Fig.(4.24) and Fig. (4.25) respectively.

In these two approaches, the $C_{md}$ was taken to be the reference concentration at y= D/2 and according to *Lane* and *Kalinske* equation (2.193) above;

$$g_s = q_\omega * V_{avr}$$

where $q_\omega$ = is the water discharge per unit width and $C_{avr} = C_{md}\ P_L e^{15(\omega/u_*)(0.5)}$.

The value of the factor $P_L$ is found from the graph supplied by Lane et al, this factor is a function of $\omega/u_*$ and the relative roughness, Fig.(2.7). The units of the above factor has been converted to SI units to be homogenous with the present measurements as;

$q_w$ in L/s.m' ,

$C_{avr}$ , $C_{md}$ in g/L

and $g_s$ in g/s.m'

In *Brooks'* method, his equation (2.206), is;

$$g_s/g_w C_{md} = C/C_{md} = T^*_B(kU/u_*,\ Z).$$

or $$q_s/q_w C_{md} = T^*_B(kU/u_*, Z)$$

The value of $T^*_B$ is given graphically by Brooks as a function of z and $kU/u_*$. These values are taken based on experimental measurements.

The relationships between the measured suspended-load as evaluated by the present auther and those values evaluated by *Lane et al.* and *Brooks'* approaches are shown graphically in Fig.(4.24) and Fig.(4.25), respectively.

It is found that both approaches are in agreement with the present results of the suspended-load rates. However *Brooks'* results are found to be closer to the present observations. This can be explained as the present work refers to the measured mid-depth concentration as reference concentration which is the same reference adopted by *Brooks'* ; in *Lane et al.* approach the reference concentration depth is not specified.

**4.3.3: Verification of the Total Load Relations:-**

In the present experiments the total load is taken to be the sum of the measured bed-load and estimated suspended load. This is called the measured total load.

These measured values of total loads are compared to three direct methods and one indirect method adopted for calculations of the total load.As these methods are recently the most popular,they don't require extensive information about flow and sediments,and they have the least restrictions on application.

The first method is the *Yang*[115], approach, in 1979, using the regression techniques,that is equation (2.273) which was mentioned earlier as;

$$\log C_T = 5.165 - 0.153 \log(vd/\nu) - 0.297 \log(u_*/v)$$
$$+ [1.78 - 0.36 \log(vd/\nu) - 0.480 \log(u_*/v)] \log(US/v)$$

The relationship between *Yang's* values and the present measured values is shown graphically in Fig.(4.26). It was found that *Yang's* values were almost less than one third of the actual measured values.Although they follow the same general trend.

The second method is the *Engelund and Hansen*(108) approach as outlined in equation (2.243) through (2.247) from which Engelund et al stated that:

$$f\ \phi = 0.1\ \theta^{5/2}$$

where $f = 2gSD/U^2$

and $\theta = \tau_o/(\gamma_s-\gamma)d$

also $\phi = q_r/[((\gamma_s-\gamma)/\gamma)\ gd^3]^{1/2}$

the values of f and $\theta$ can be estimated for each test,then the values of $\phi$ is estimated from which $q_r$ can be evaluated.

The relationship between the observed values and those estimated by Engelund et al is shown graphically in Fig.(4.27). Again the measured values are almost ten times the estimated values.This may be due to the restrictions outlined by Engelund et al as their equation should be applied to the flows with dune beds and particle size greater than 0.15 mm; since the bed forms are mostly observed to be ripples.

The third direct approach is that introduced by *Graf and Acaroglu*(107), outlined earlier in equations (2.240), (2.241) and (2.242).

The values of $\psi_A = [\{(\rho_s-\rho)/\rho\}d]/SR_h$ and

$\phi_A = (C\ U\ R_h)/[\{(\rho_s-\rho)/\rho\}\{gd^3\}]^{1/2}$

are evaluated for each experiment.

The $\psi_A$ versus $\phi_A$ relation is plotted on a log-log scale which is similar to graph given by Graf et al as;

$$\phi_A = 10.39(\psi_A)^{-2.52}$$

This is shown graphically in Fig.(4.28).

The observed relationship is nicely fitted by the relation

$$\phi_A = 32.9\ \psi_A^{-2.52}$$

which means that the measured value of total load is almost three times that predicted by Graf et al. It is important to notice that the observed values and Graf et al values are following the same trend. Therefore the two methods can be related by introducing an empirical factor. This factor can be taken roughly around three.

A final note on the comparision between the observed values of the total load and the estimated total load using direct methods is that all these methods gave values significantly smaller than the observed ones.This may be due to the limitations,assumptions and different sediment flow parameters on which each method is based, as well as the experimental errors in the present measurements.

The last method to verify the present total load is the famous *Einstein's*[2] procedure, which evaluate the total load indirectly by adding the bed-load and suspended load. The calculations according to Einstein's metdod are lengthy but systematic.

The present results of the total load and those due to Einstein were compared graphically as shown in Fig.(4.29). It is observed that these results show a good agreement,but with some scatter. This scatter may be due to the involved assumptions and the correction factors introduced by Einstein as well as the usual experimental errors involved in the present measurements.

## 4.4: RELATION BETWEEN THE SUSPENDED LOAD CONCENTRATION AND THE BED-LOAD CONCENTRATION:-

The bed-load concentration has been estimated, as outlined before, to be a constant concentration of 1219 g/L based on dry weight of sediment per unit volume of water. Then the suspended load concentration distribution has been evaluated according to experimental measurements. This distribution is extrapolated to the bed layer trying to find a depth at which the suspended sediment concentration meets the same value of the bed-load concentration.Utilizing the measured concentration at the mid-depth $C_{md}$,the actual value of exponent z,and the bed-load concentration,$C_b$,then equation (2.208) can be written as;

$$C_{md}/C_b = ((D-(D/2)/(D/2))(a/(D-a)^z) = (a/(D-a)^z)$$

where a is the depth at which the suspended load concentration equals that of the bed-load.The value of a can be calculated directly for given values of Z, $C_{md}$, $C_b$ and D as

$$(a/(D-a)) = (C_{md}/C_b)^{1/Z}$$

when a is small compared to D the above equation reduces to:-

$$a/D \simeq (C_{md}/C_b)^{1/Z}$$

or $$a = D \ (C_{md}/C_b)^{1/Z}$$

The suspended load concentration at the top of the bed-layer thickness has been estimated.This was found to be less than half the given bed-load concentration.It is important to note that the bed-layer thickness varies for each run, according to the number of layers set in motion.

In the same way, the suspended load concentration has been evaluated at the mid-depth of the bed layer, i.e at a depth of half the thickness of the moving layers of the bed. This value of concentration has been found to represent nicely the bed-load concentration with a maximum deviation of 3%, as shown in the table (A.7) in Appendix A.

These results are different from the *Einstein's*[2] assumptions,where he assumed that the average concentration of the bed-load in the bed-layer must be equal to the concentration of suspended load at a depth = 2d. This was chosen arbitrarily by him and has been subjected to further investigations.*Garde*[33],found that Einstein's assumption of thickness 2d is reasonable in the case of a plane bed but the physical significance of the bed layer thickness in a dune bed channel is rather elusive. *Brooks*[98] and other group of researchers adopted that Einstein's suggestion of a reference concentration at 2d is reasonable for the plane bed. Also they suggested that the bed layer thickness at which the concentration of the suspended load represents the concentration of the bed-load is evaluated directly in the same concept used in this research. Studies by *Bagnold*[45], and *Einstein and Chien*[88] indicate that the suspended load tends to reach a limiting concentration $C_b$ of 4.8 $KN/m^3$ which is about 500 g/L being less than one half the values predicted by *Khalil*[28]. *Toffaleti*[51] suggested a limiting value of bed-load concentration of 100 $lb/ft^3$ which is 1600 g/L. Thus the value adopted in the present work, 1219 g/L, which was examined experimentally by *Khalil*[25], is a reasonable one.

According to the relation established between the bed-load concentration and that of the suspended load the total load can be estimated for given flow parameters in the following sequence:-

A- Based on *Khalil's* bed-load model, (1) calculate the bed shear stress $\tau_b$ from the flow parameters. (2) estimate the bed-load concentration as given by *Khalil*[28] based on a graph introduced in Fig.(2.3b). If the value of the bed shear stress is greater than that required to set more than one layer in motion (i.e $N > N_*$), then calculate the number of moving layers. (3) Calculate the bed-load rate on rippled bed according to equation (2.59).

B- According to the present work, find the bed-load concentration $C_b$ at the mid-depth of the moving layers called y=a. The limiting maximum value of N is 0.46 or 1219 g/L.

This concentration fits the extrapolation of the suspended load concentration at this depth a.

C- According to the suspended load concentration distributions equation, calculate $C_{md}$ as,

$$C_{md}/C_b = (a/(D-a))^z$$

where z is the exponent given by $\omega/(\beta u_* k)$ also $\omega, k, u_*$ and $\beta$ can be calculated from the sediment flow parameter. However for these values the value of k is assumed arbitrary and then adjusted according to the concentration by a successive iterations with the aid of Fig.(4.7) shown earlier. The value of $\omega$ for a given grain size should be corrected for the effect of grain concentration in successive iteration as given in equation (2.142).

The recommended value of $\beta$ is taken in the range 1.2 to 1.5 according to the grain size.

The concentration at the mid-depth $C_{md}$ of the flow (y=D/2), is the only unknown in the above equation.

D- Using *Brooks'* approach, outlined earlier, find the value of $g_s$, given by:

$$g_s/g_\omega C_{md} = T_B^*(kU/u_*, Z)$$

E- Add the calculated $g_B$ in step A to $g_s$ found in step D in order to estimate the total load rate $g_T$ .

The above step by step approach will be illustrated by the an example taken from experimental data (Test no.2) shown in Appendix C.

FIG.4.1. ESTIMATED BEDLOAD Vs. OBSERVED VALUES

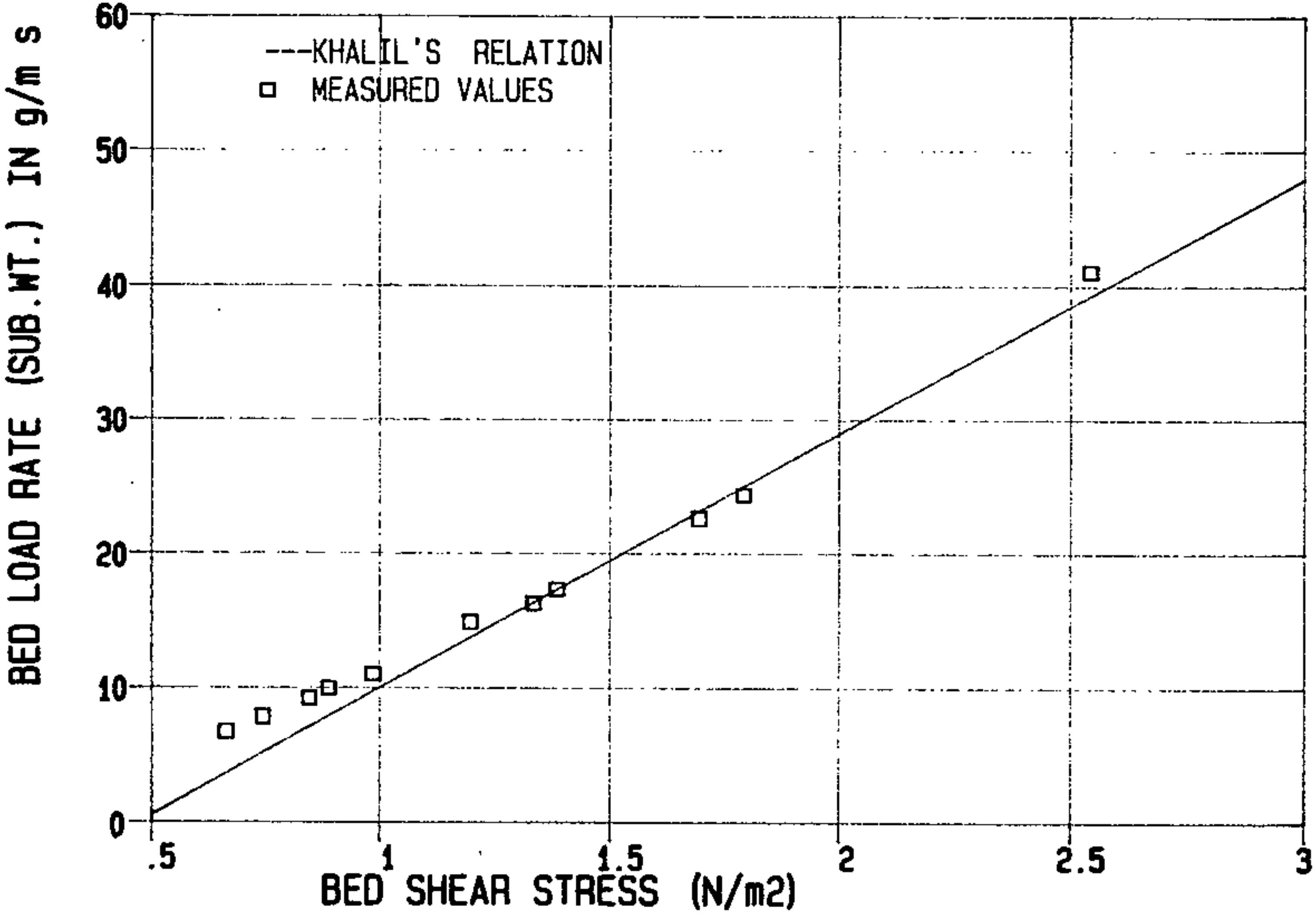



FIG. (4.1)

FIG.4.2. ESTIMATED BEDLOAD Vs.OBSERVED VALUES

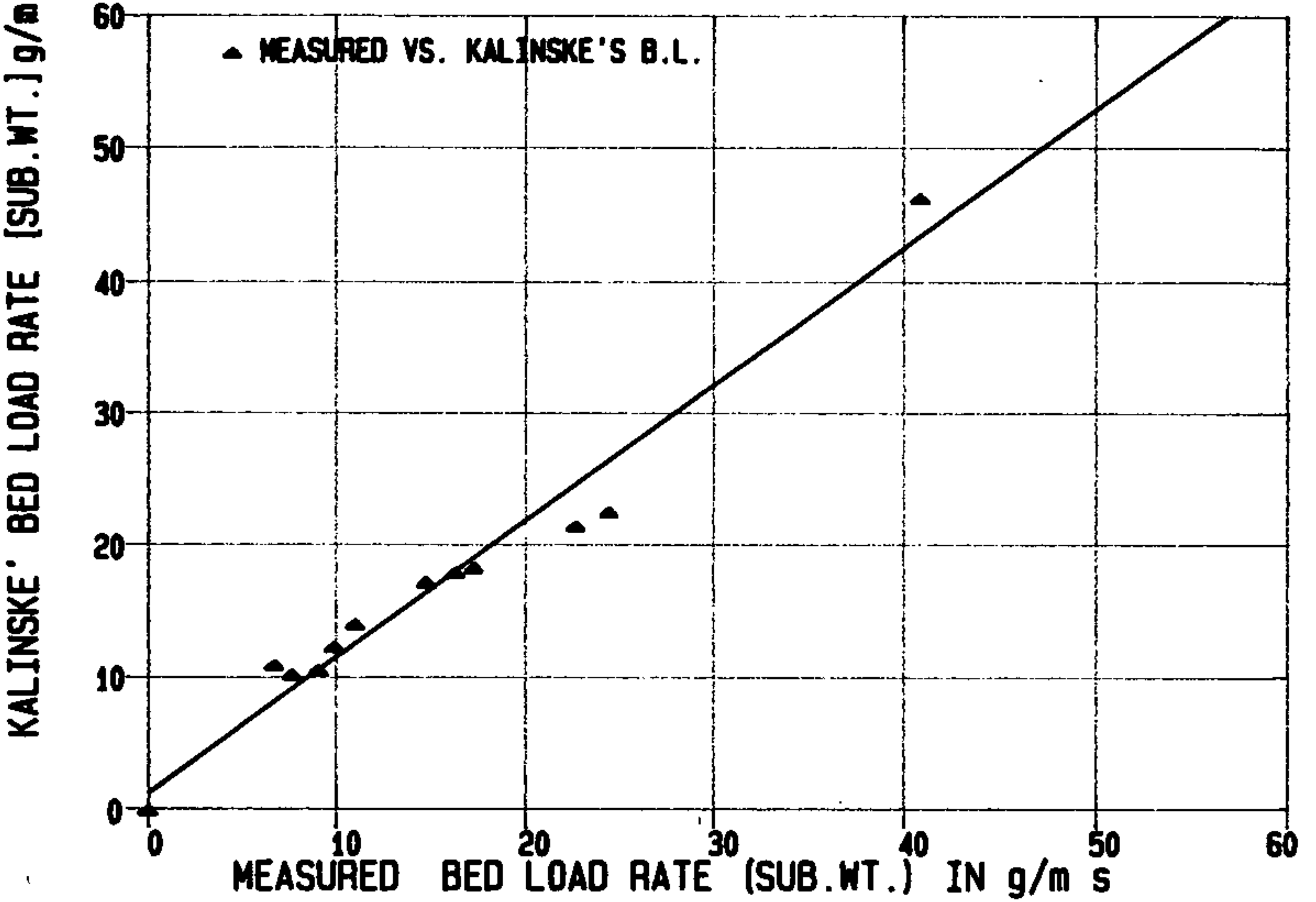
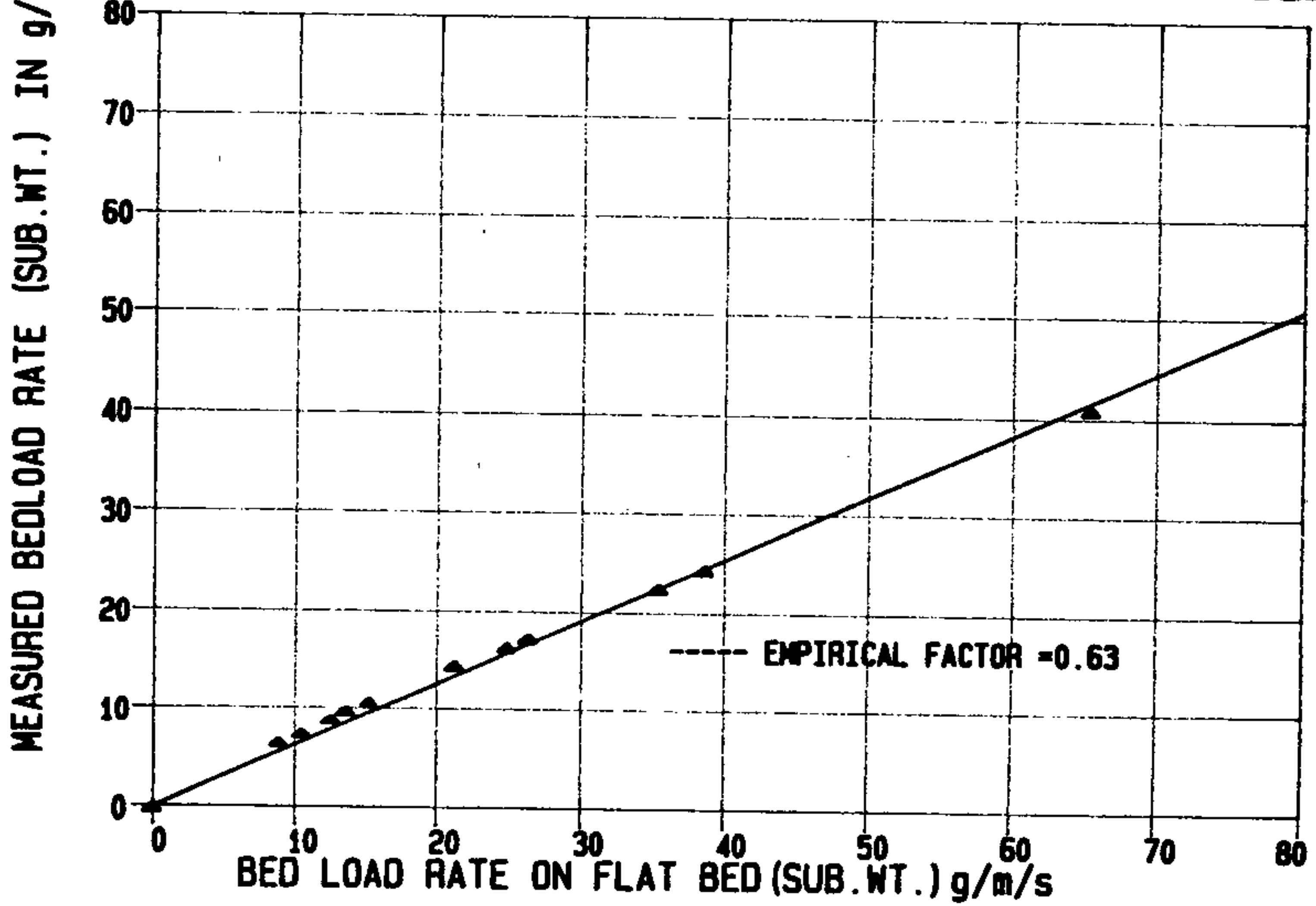



FIG. (4.2)

FIG.4.3. BEDLOAD RATES ON RIPPLED VS. FLAT BEDS.



VELOCITY PROFILE IN OPEN CHANNEL FLOW

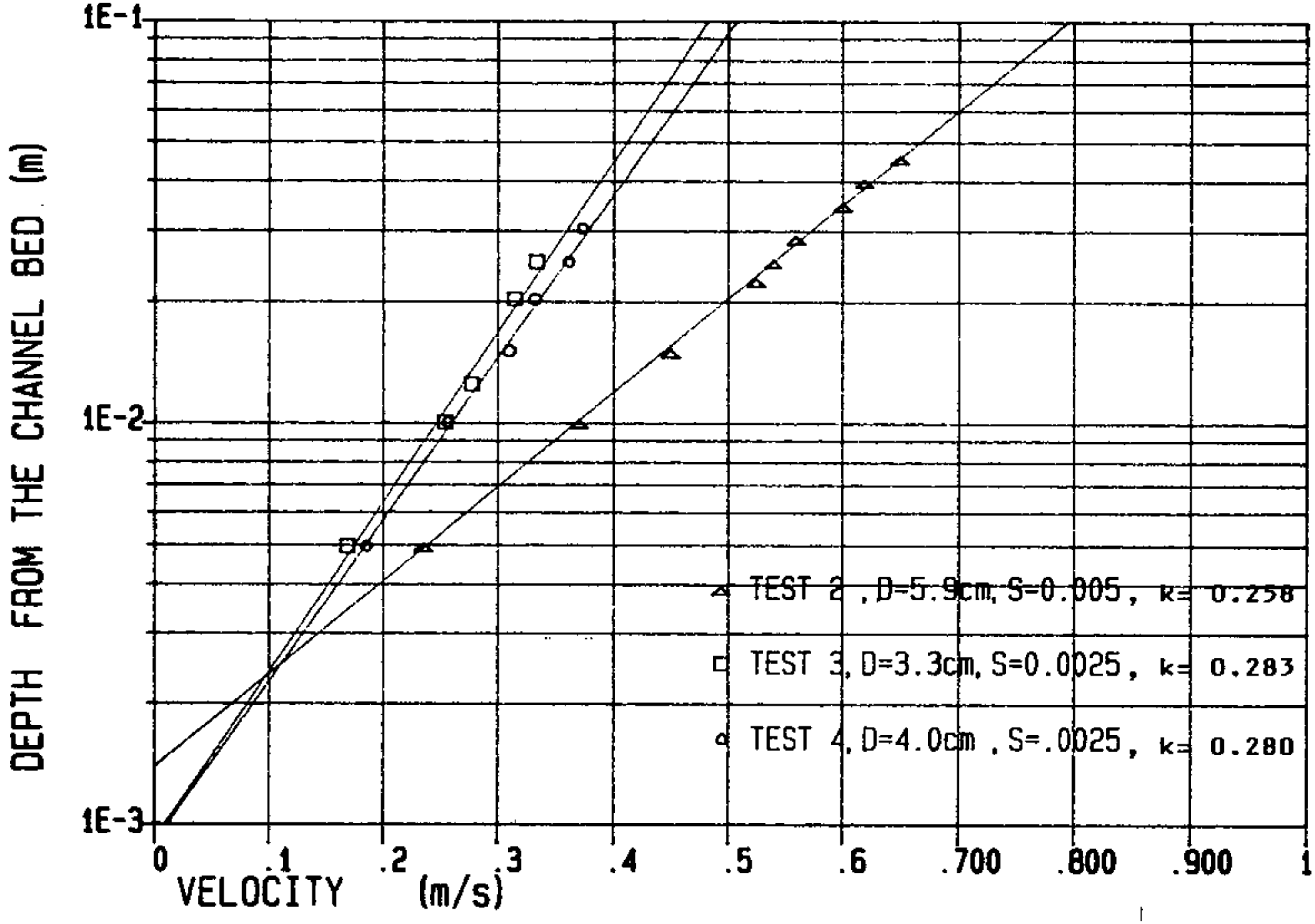



FIGURE 4.4

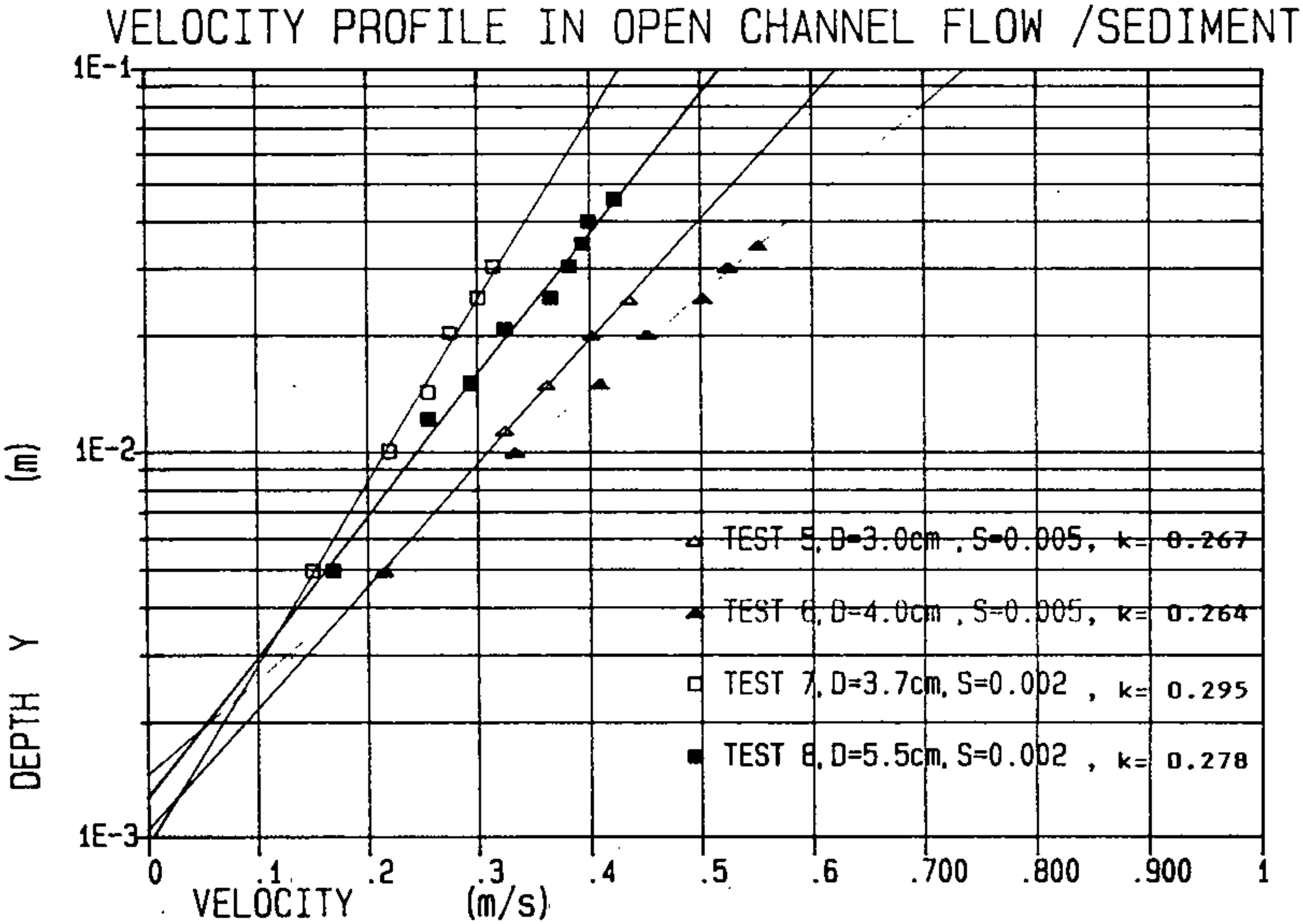

VELOCITY PROFILE IN OPEN CHANNEL FLOW /SEDIMENT
1E-1
1E-2
1E-3
DEPTH Y (m)
0
.1
.2
.3
.4
.5
.6
.700
.800
.900
1
VELOCITY (m/s)
TEST 5, D=3.0cm , S=0.005, k= 0.267
TEST 6, D=4.0cm , S=0.005, k= 0.264
TEST 7, D=3.7cm, S=0.002 , k= 0.295
TEST 8, D=5.5cm, S=0.002 , k= 0.278

FIG. (4.5)

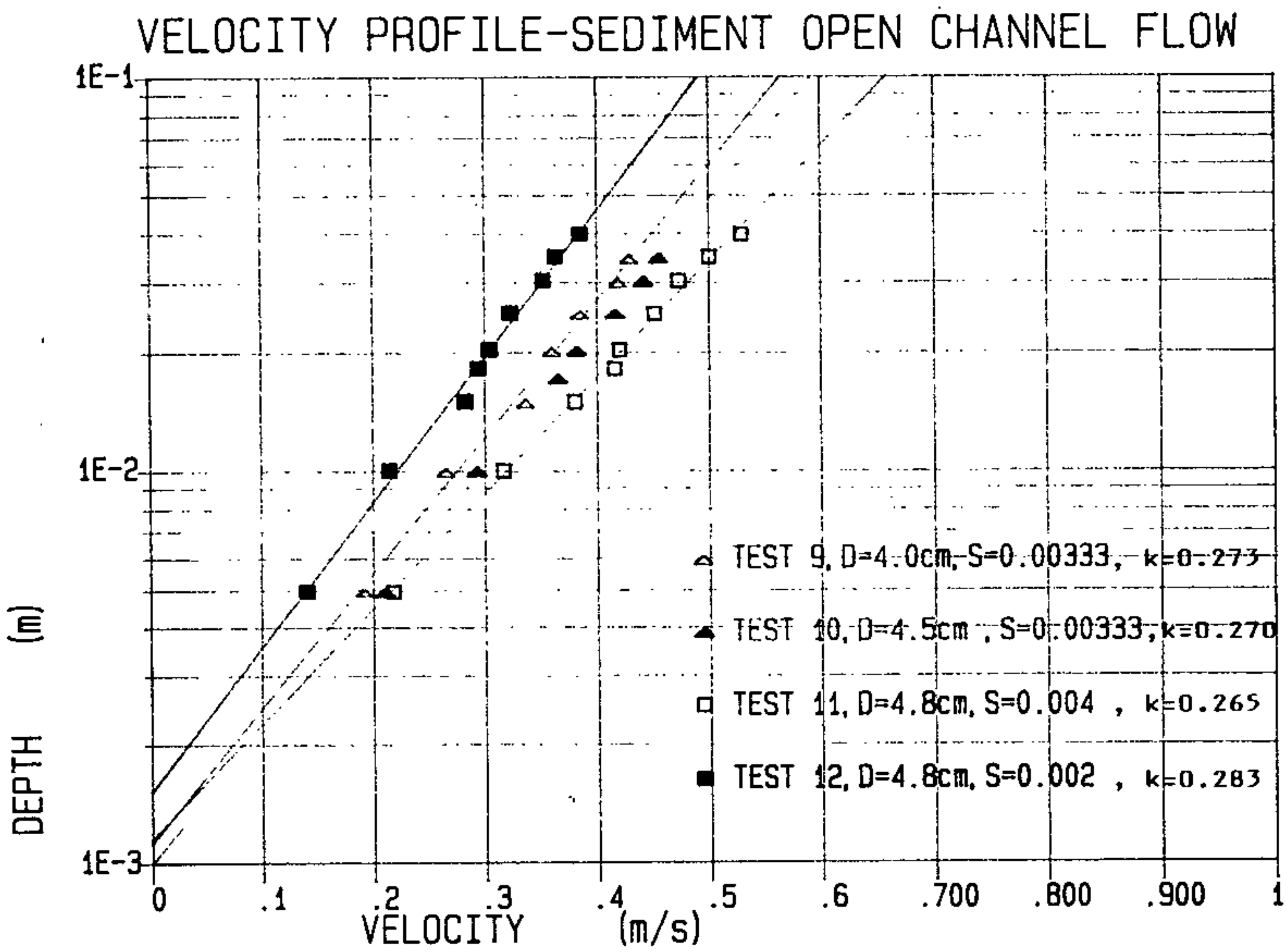

VELOCITY PROFILE-SEDIMENT OPEN CHANNEL FLOW
1E-1
1E-2
1E-3
DEPTH (m)
0
.1
.2
.3
.4
.5
.6
.700
.800
.900
1
VELOCITY (m/s)
TEST 9, D=4.0cm, S=0.00333, k=0.273
TEST 10, D=4.5cm, S=0.00333, k=0.270
TEST 11, D=4.8cm, S=0.004, k=0.265
TEST 12, D=4.8cm, S=0.002, k=0.283

FIG. (4.6)

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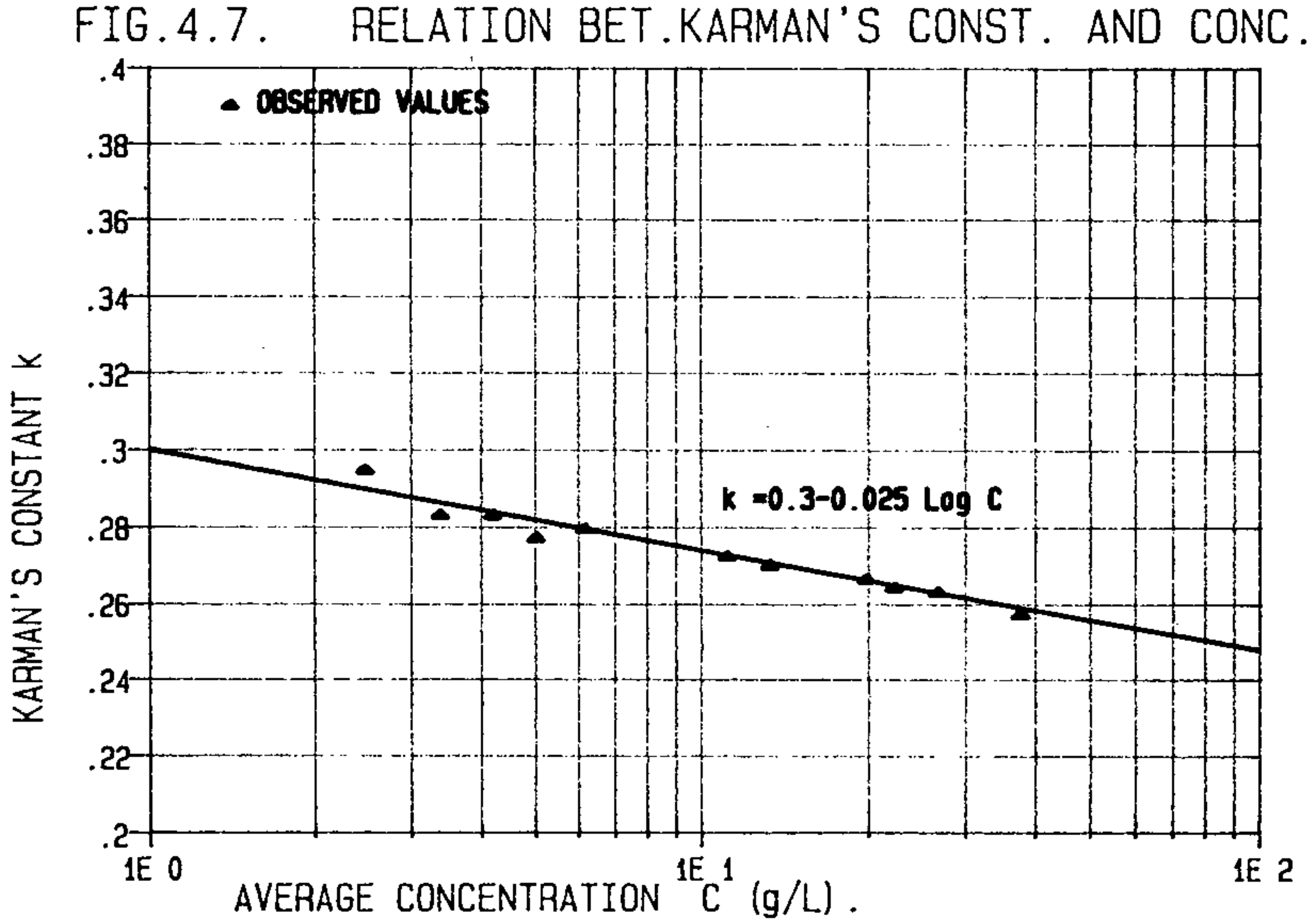
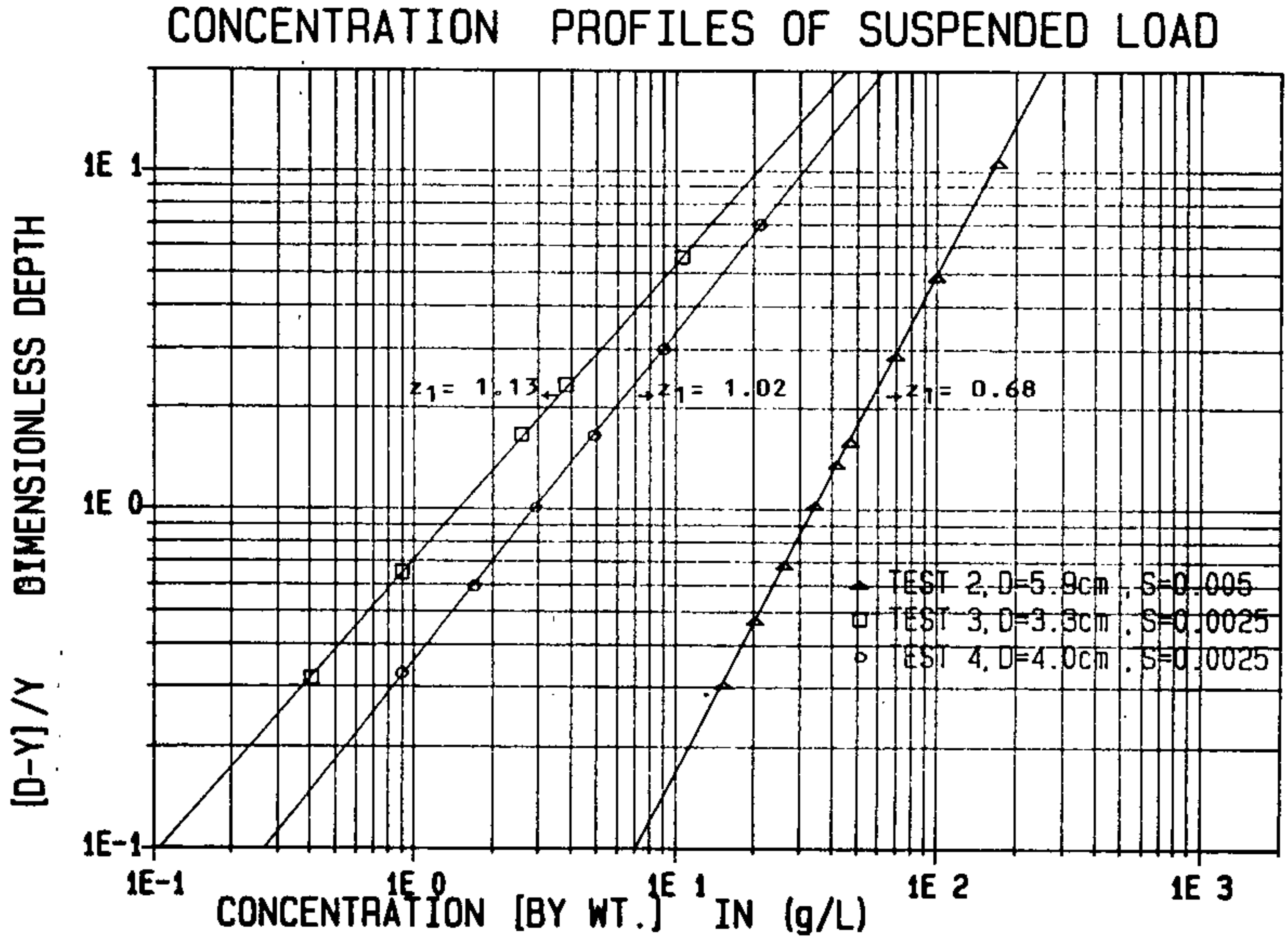


FIG.4.7. RELATION BET.KARMAN'S CONST. AND CONC.
OBSERVED VALUES
k =0.3-0.025 Log C
KARMAN'S CONSTANT k
.4
.38
.36
.34
.32
.3
.28
.26
.24
.22
.2
1E 0
1E 1
1E 2
AVERAGE CONCENTRATION C (g/L).

FIG. (4.7)



CONCENTRATION PROFILES OF SUSPENDED LOAD
1E 1
1E 0
1E-1
1E-1
1E 0
1E 1
1E 2
1E 3
[D-Y]/Y DIMENSIONLESS DEPTH
CONCENTRATION [BY WT.] IN (g/L)
$z_1$ = 1.13
$z_1$ = 1.02
$z_1$ = 0.68
TEST 2, D=5.9cm , S=0.005
TEST 3, D=3.3cm , S=0.0025
TEST 4, D=4.0cm , S=0.0025


FIG. (4.8)

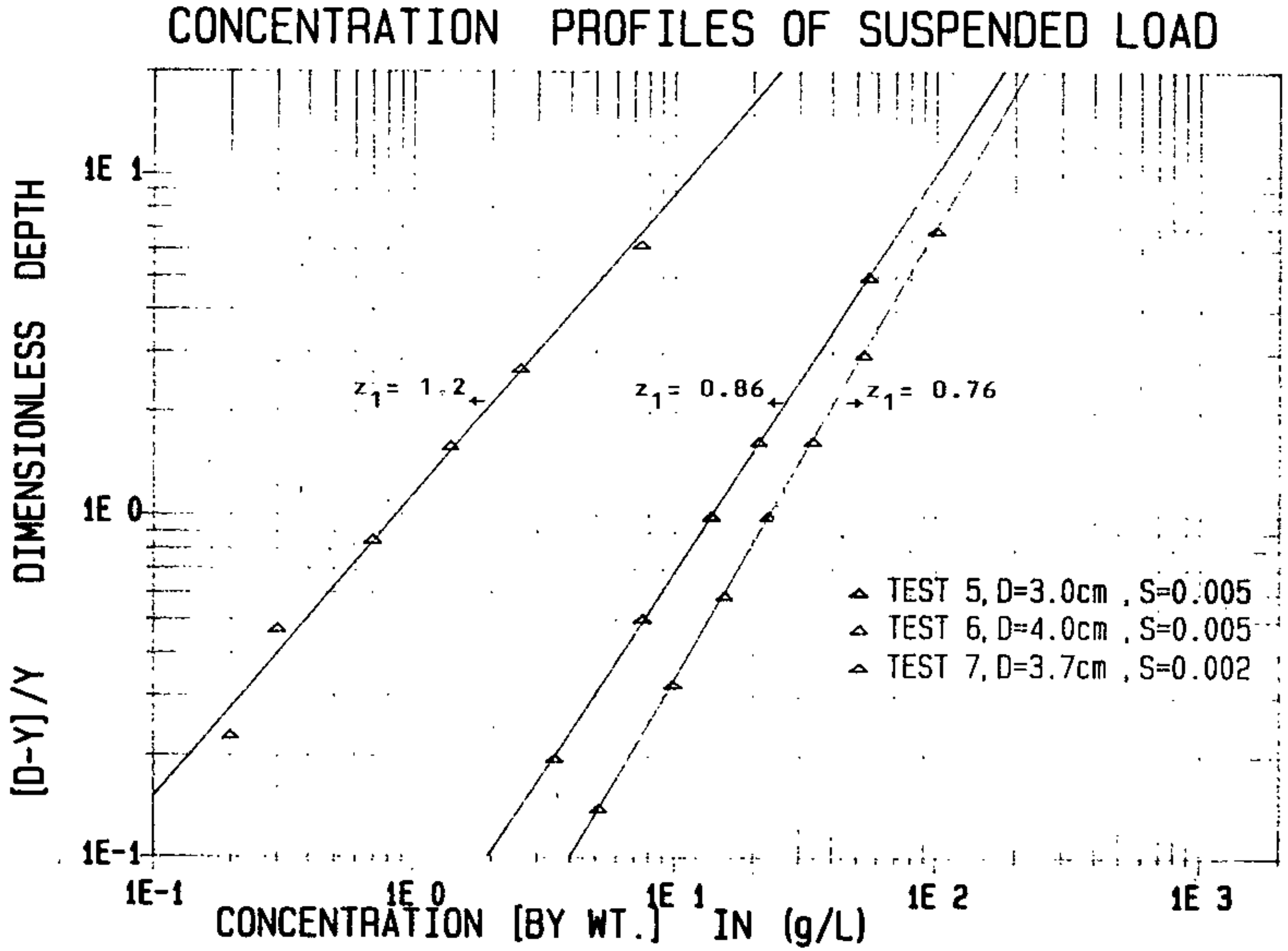

CONCENTRATION PROFILES OF SUSPENDED LOAD
[D-Y]/Y DIMENSIONLESS DEPTH
1E 1
1E 0
1E-1
1E-1
1E 0
1E 1
1E 2
1E 3
CONCENTRATION [BY WT.] IN (g/L)
z1 = 1.2
z1 = 0.86
z1 = 0.76
TEST 5, D=3.0cm , S=0.005
TEST 6, D=4.0cm , S=0.005
TEST 7, D=3.7cm , S=0.002

FIG. (4.9)

CONCENTRATION PROFILES OF SUSPENDED LOAD

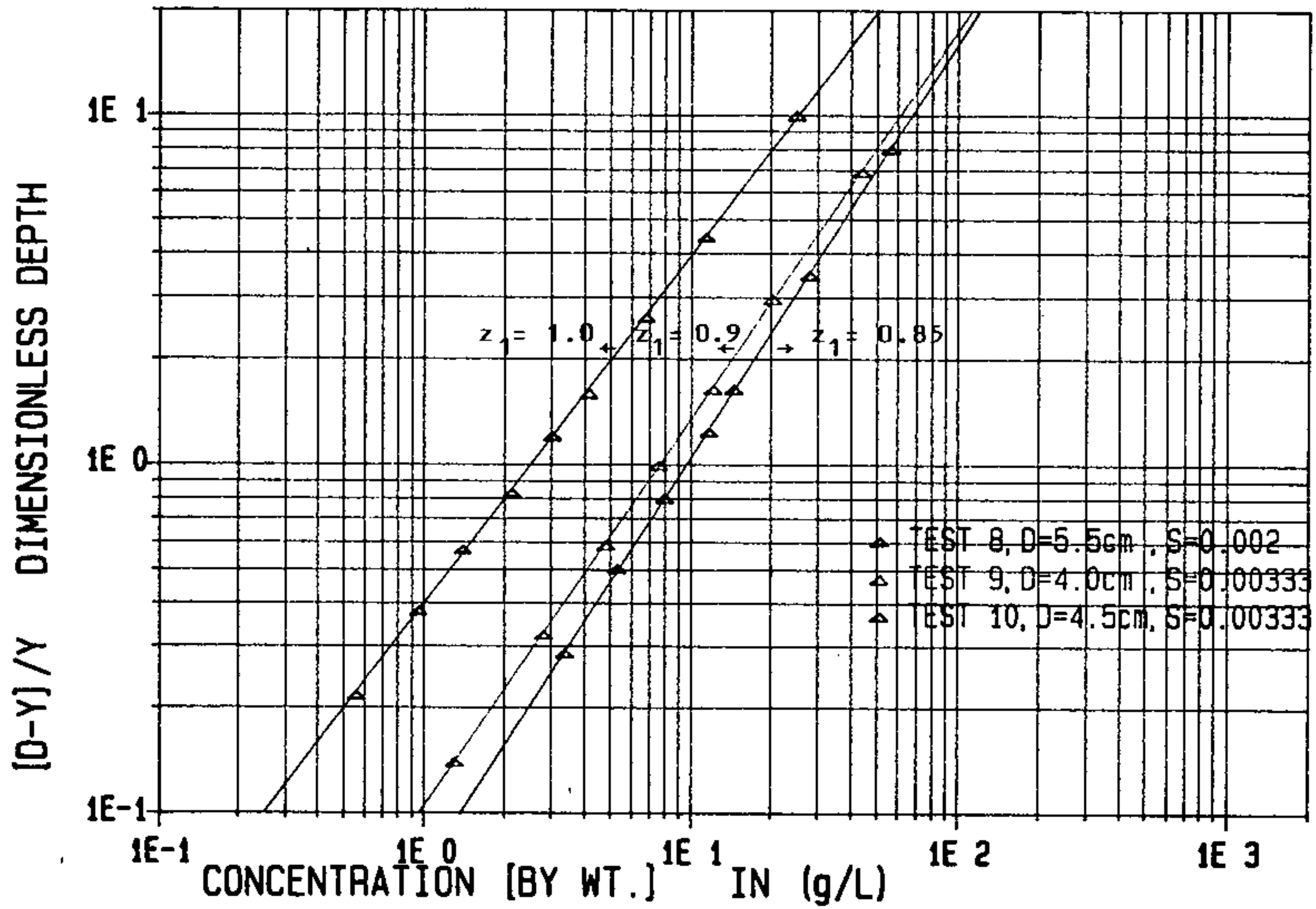



FIG. (4.10)

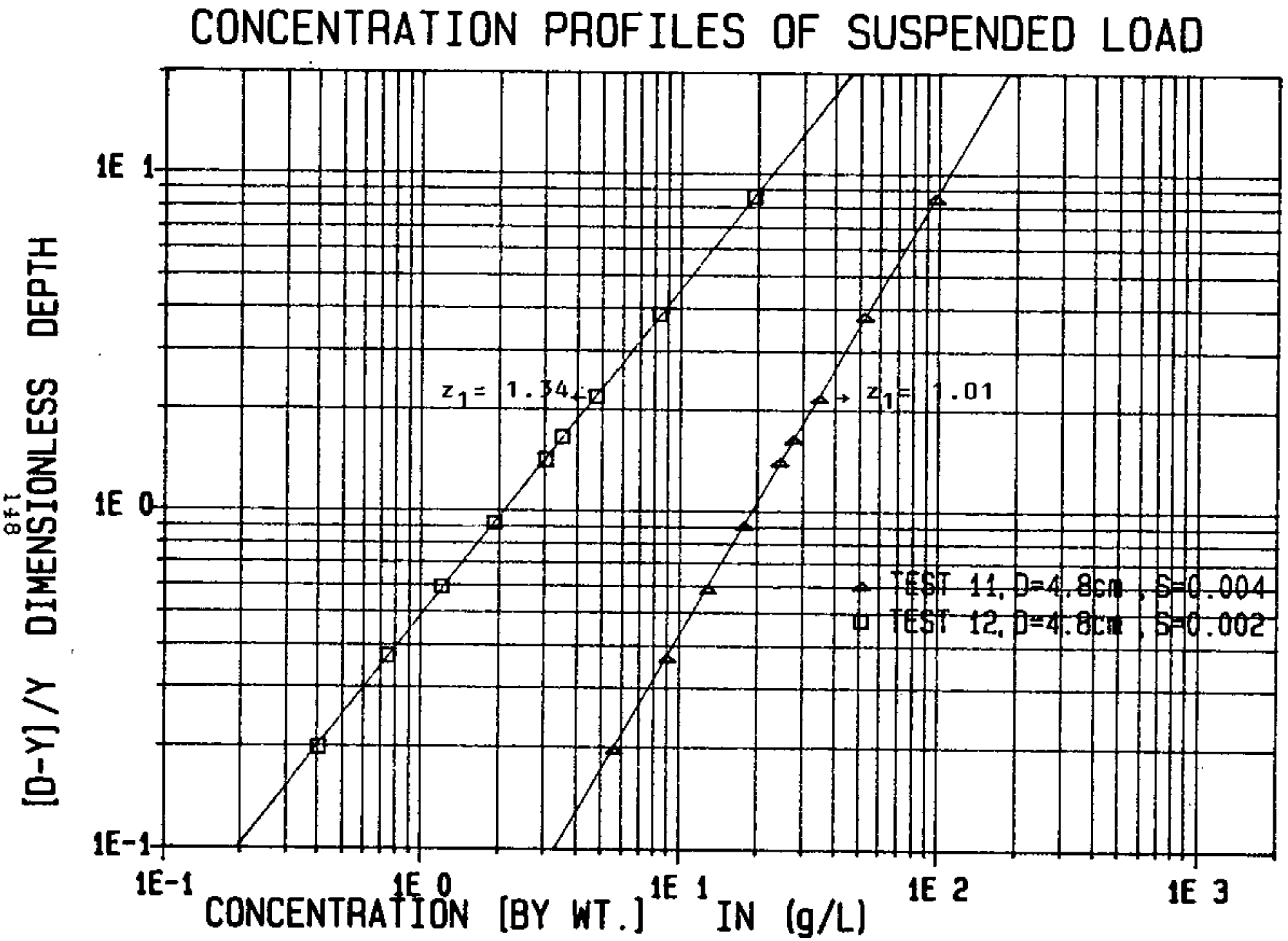

CONCENTRATION PROFILES OF SUSPENDED LOAD
1E 1
1E 0
1E-1
[D-Y]/Y DIMENSIONLESS DEPTH
1E-1
1E 0
1E 1
1E 2
1E 3
CONCENTRATION [BY WT.] IN (g/L)
z1 = 1.34
z1 = 1.01
TEST 11, D=4.8cm , S=0.004
TEST 12, D=4.8cm , S=0.002

FIG. (4.11)

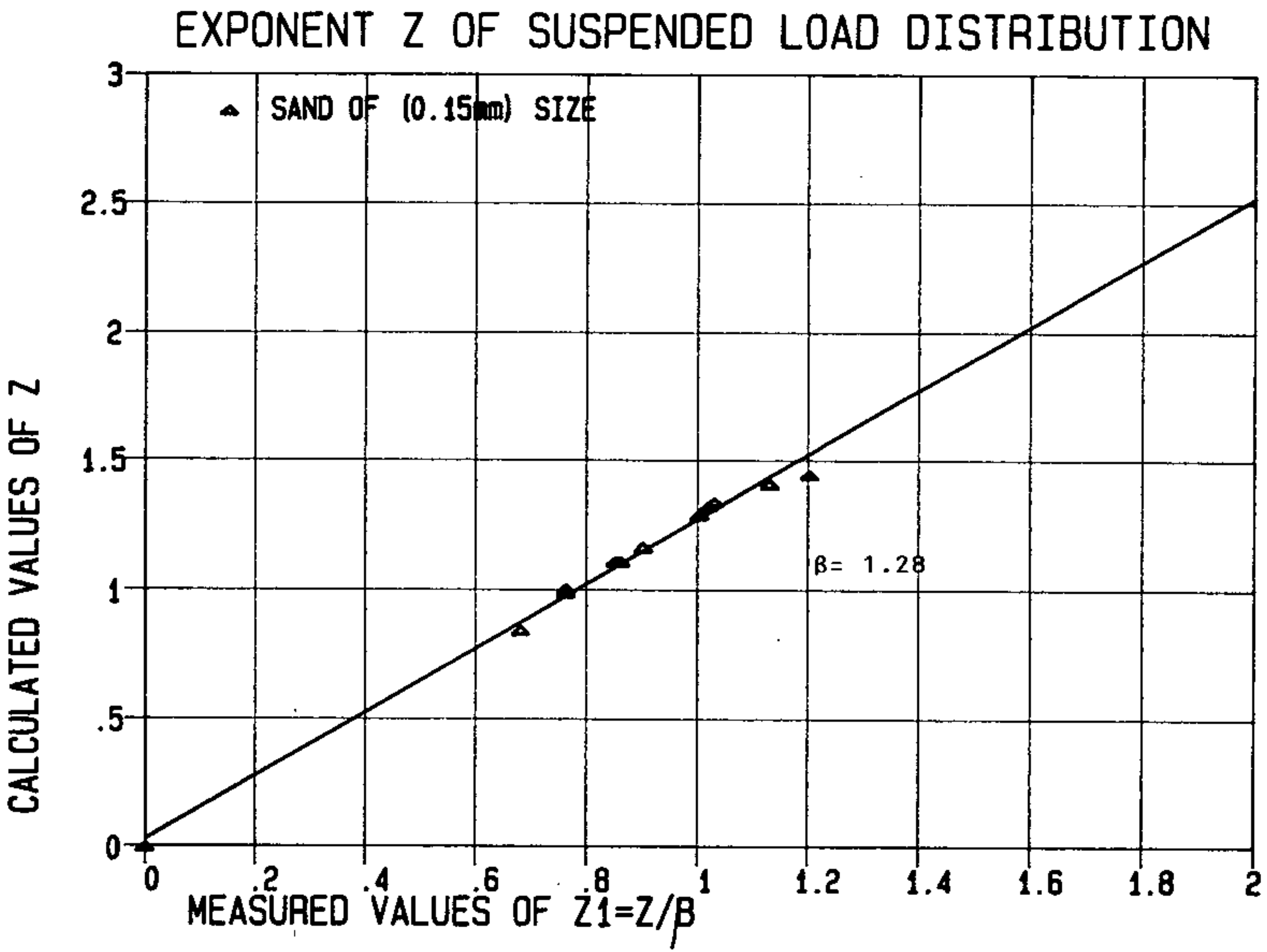
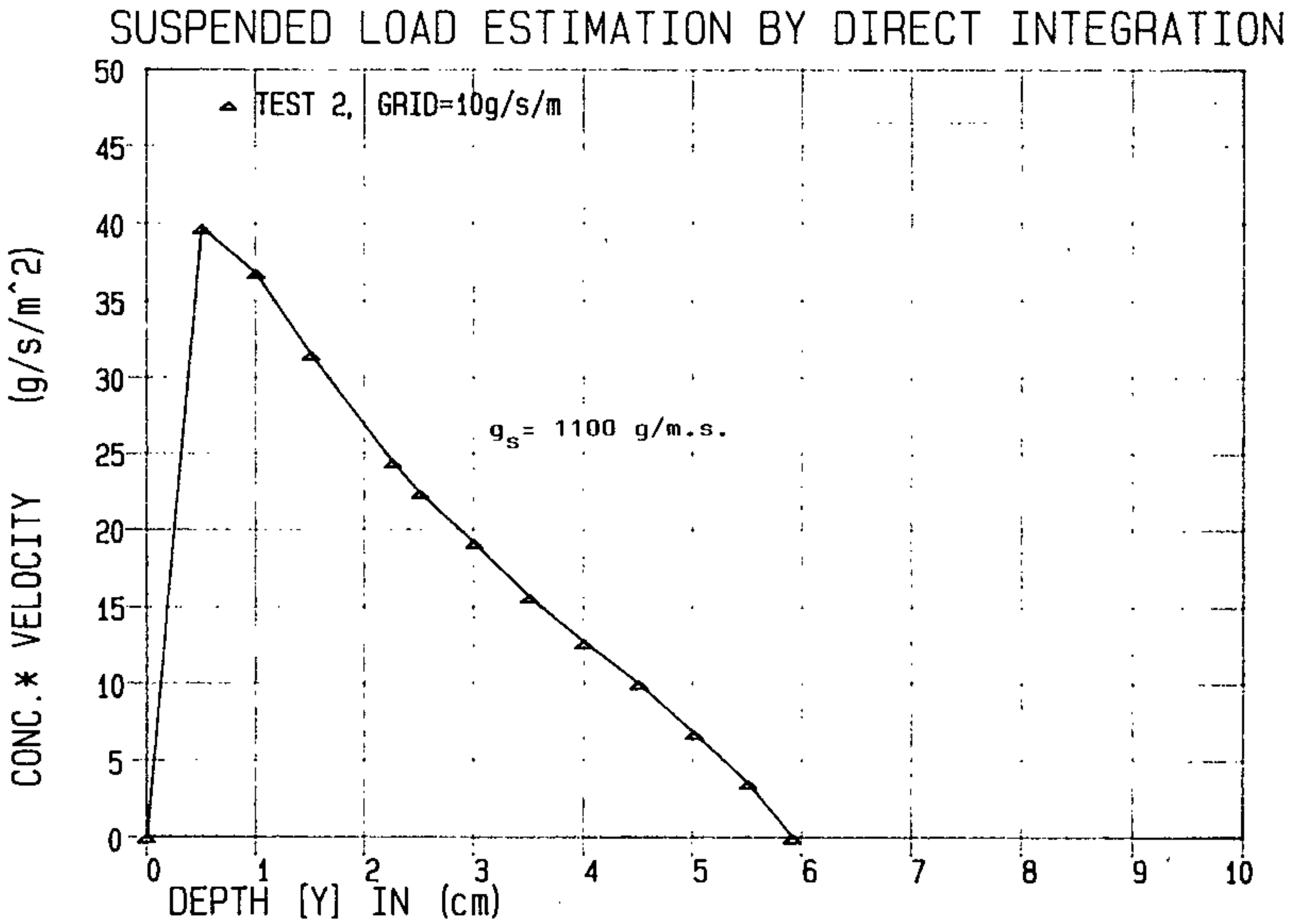

EXPONENT Z OF SUSPENDED LOAD DISTRIBUTION
SAND OF (0.15mm) SIZE
β= 1.28
CALCULATED VALUES OF Z
MEASURED VALUES OF Z1=Z/β
0
.5
1
1.5
2
2.5
3
.2
.4
.6
.8
1.2
1.4
1.6
1.8

FIG. (4.12)



SUSPENDED LOAD ESTIMATION BY DIRECT INTEGRATION
TEST 2, GRID=10g/s/m
g_s= 1100 g/m.s.
CONC.* VELOCITY (g/s/m^2)
DEPTH [Y] IN (cm)
0
5
10
15
20
25
30
35
40
45
50
0
1
2
3
4
5
6
7
8
9
10


FIG. (4.13)

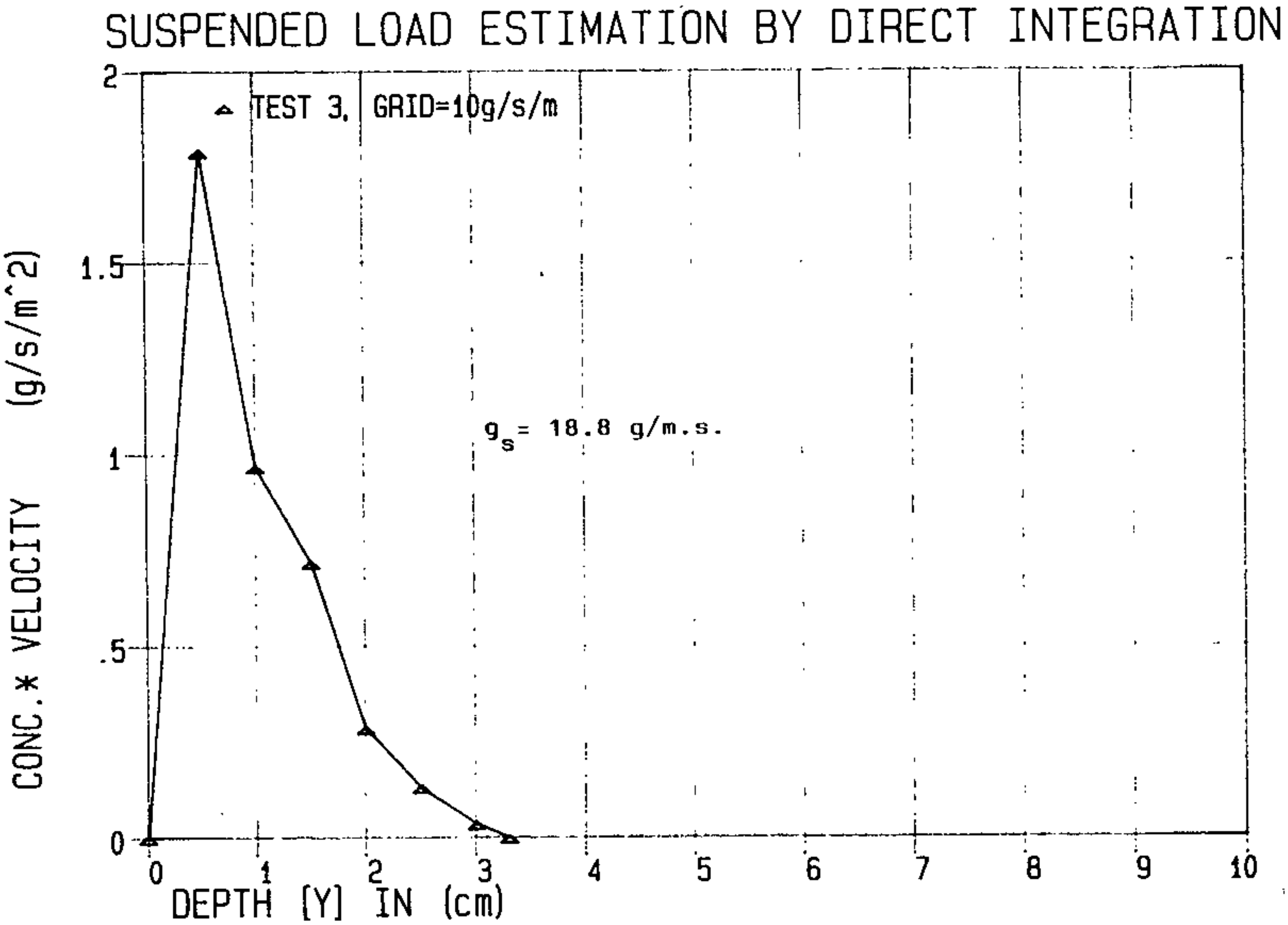

SUSPENDED LOAD ESTIMATION BY DIRECT INTEGRATION
TEST 3, GRID=10g/s/m
$g_s$ = 18.8 g/m.s.
CONC.* VELOCITY (g/s/m^2)
DEPTH [Y] IN (cm)
0
.5
1
1.5
2
0
1
2
3
4
5
6
7
8
9
10

FIG. (4.14)

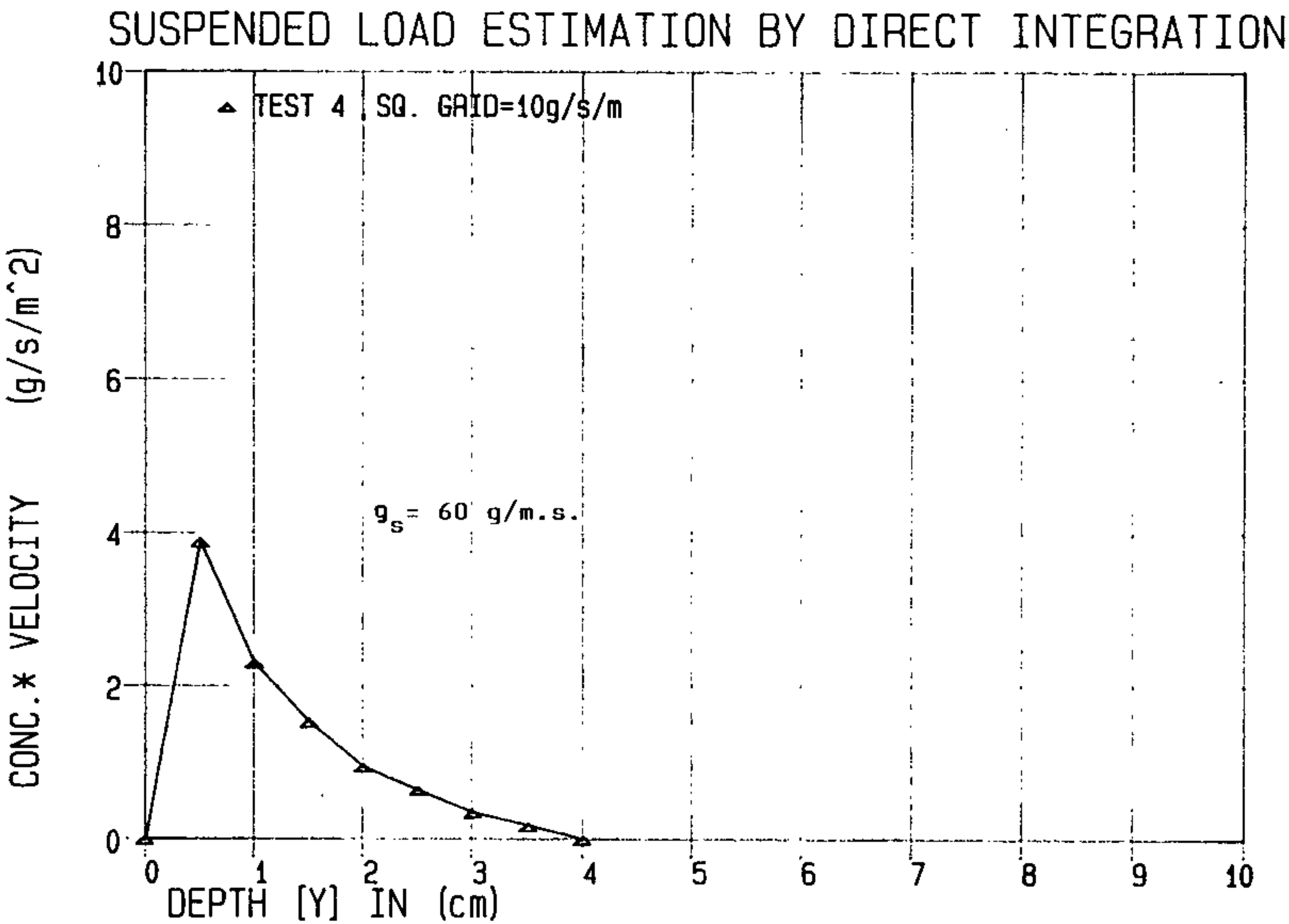
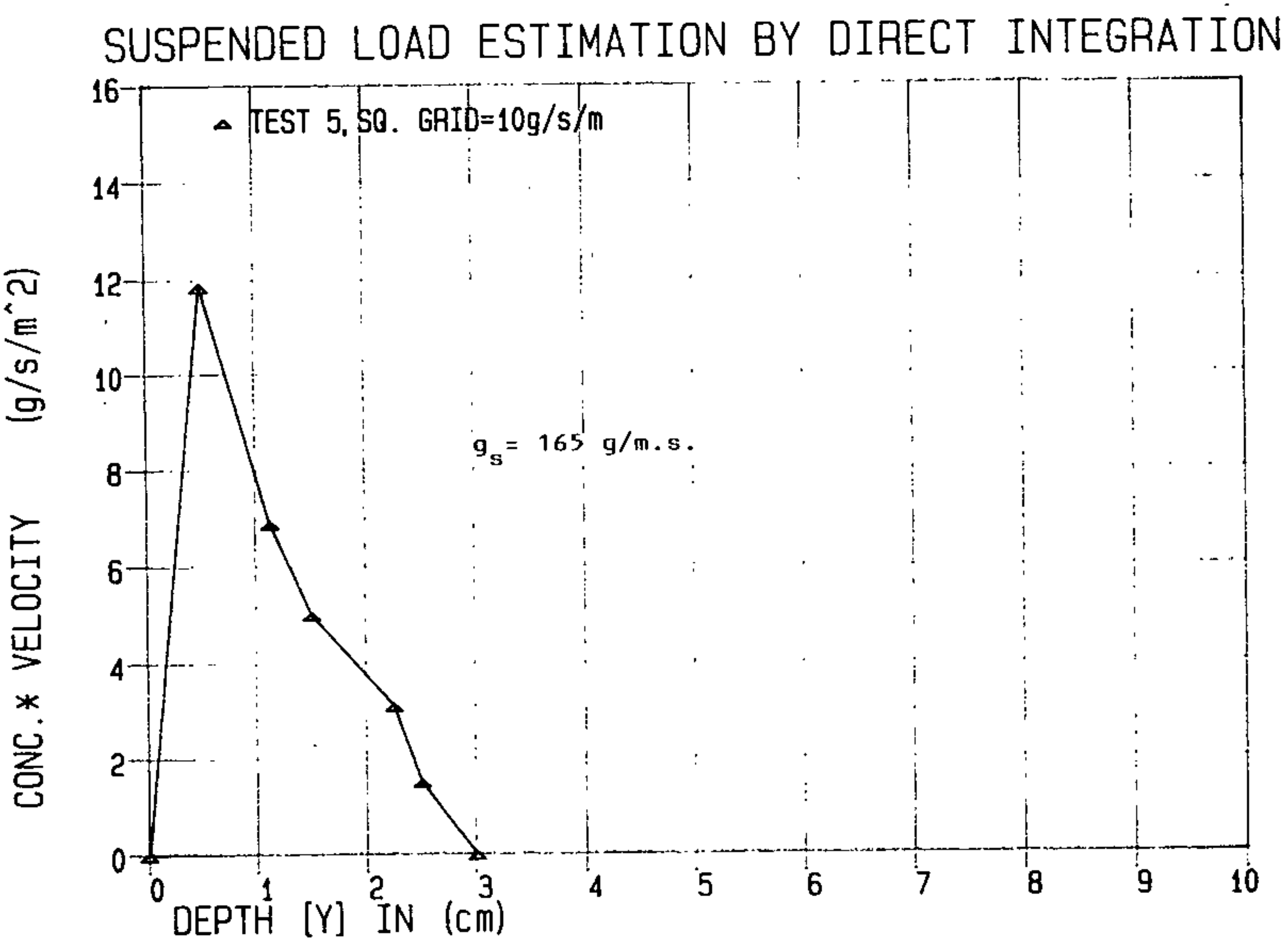

SUSPENDED LOAD ESTIMATION BY DIRECT INTEGRATION
TEST 4 SQ. GRID=10g/s/m
$g_s$= 60 g/m.s.
CONC.* VELOCITY (g/s/m^2)
DEPTH [Y] IN (cm)
0
2
4
6
8
10
1
3
5
7
9

FIG. (4.15)



SUSPENDED LOAD ESTIMATION BY DIRECT INTEGRATION
TEST 5, SQ. GRID=10g/s/m
g_s = 165 g/m.s.
CONC.* VELOCITY (g/s/m^2)
DEPTH [Y] IN (cm)
0
2
4
6
8
10
12
14
16
0
1
2
3
4
5
6
7
8
9
10


FIG. (4.16)

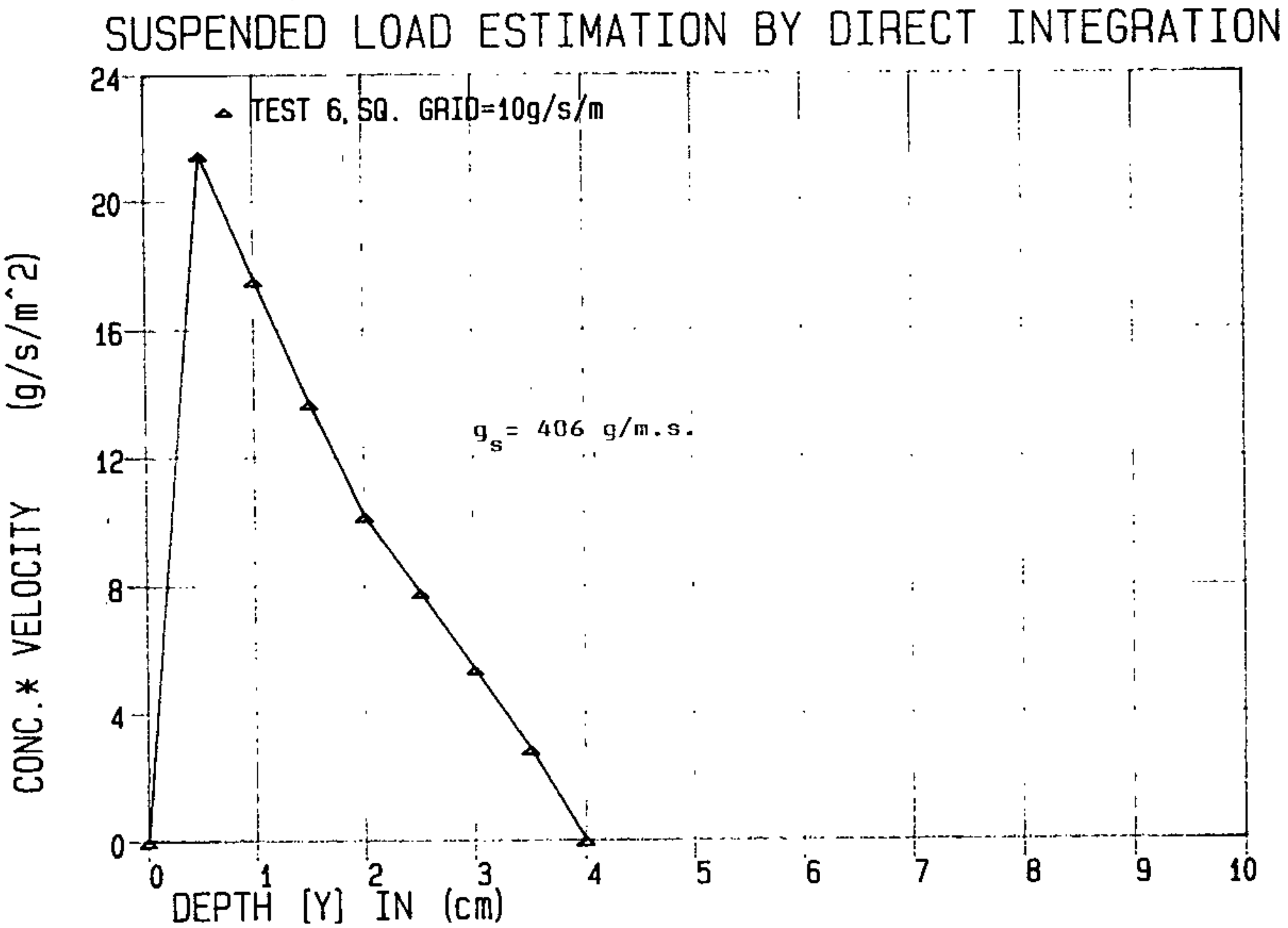

SUSPENDED LOAD ESTIMATION BY DIRECT INTEGRATION
TEST 6, SQ. GRID=10g/s/m
g_s = 406 g/m.s.
CONC.* VELOCITY (g/s/m^2)
0
4
8
12
16
20
24
DEPTH [Y] IN (cm)
0
1
2
3
4
5
6
7
8
9
10

FIG. (4.17)

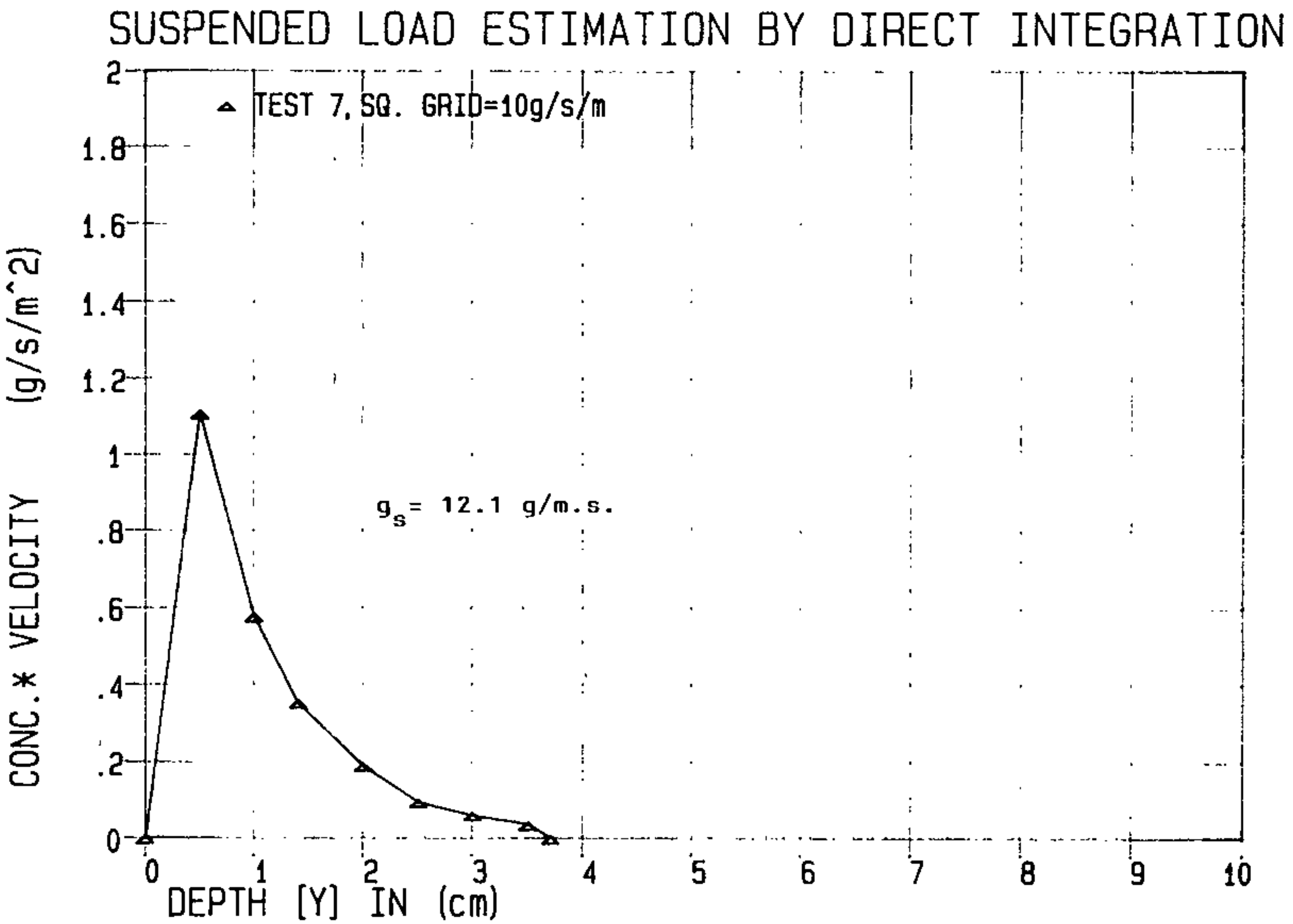

SUSPENDED LOAD ESTIMATION BY DIRECT INTEGRATION
TEST 7, SQ. GRID=10g/s/m
g_s= 12.1 g/m.s.
CONC.* VELOCITY (g/s/m^2)
DEPTH [Y] IN (cm)
2
1.8
1.6
1.4
1.2
1
.8
.6
.4
.2
0
1
2
3
4
5
6
7
8
9
10

FIG. (4.18)

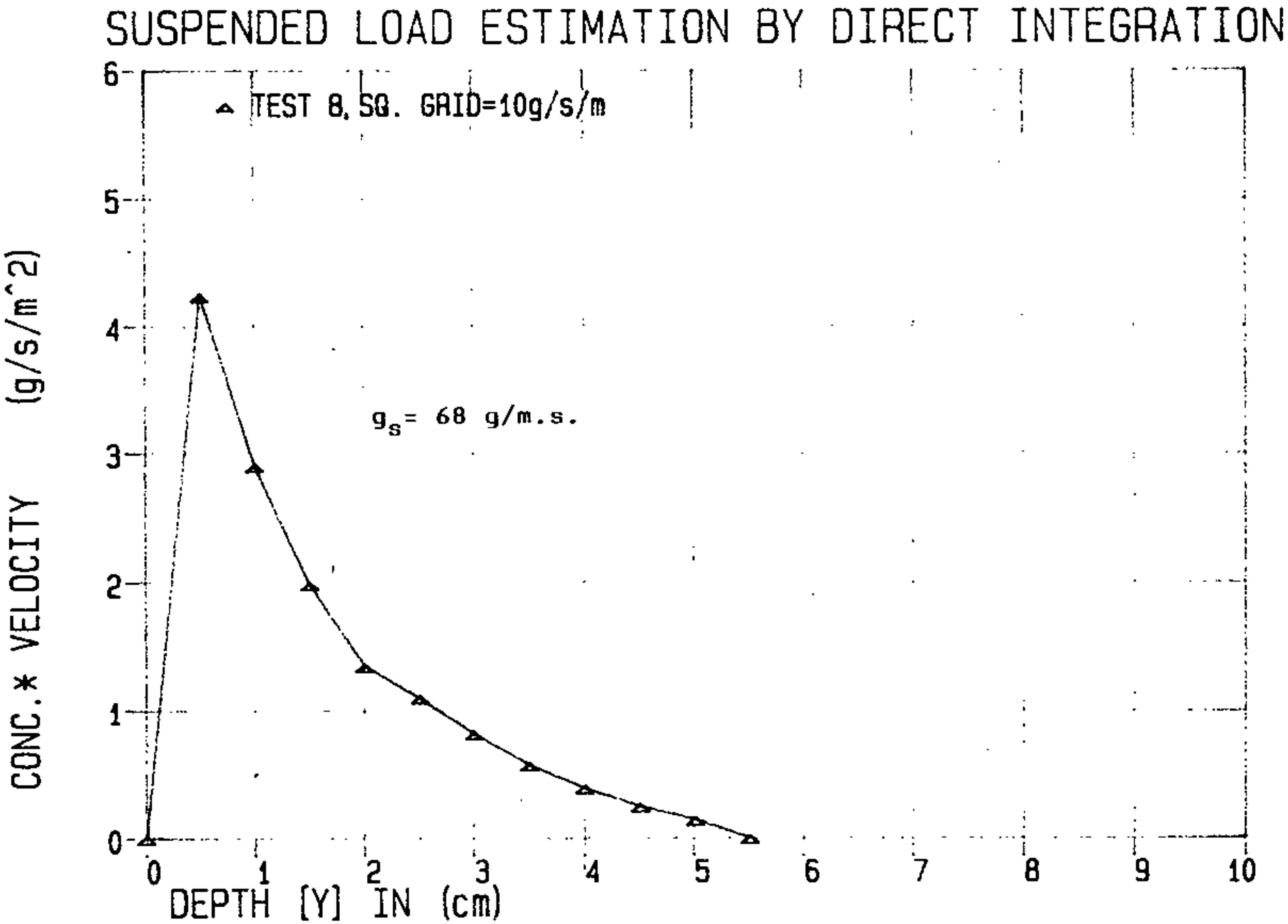

SUSPENDED LOAD ESTIMATION BY DIRECT INTEGRATION
TEST 8, SQ. GRID=10g/s/m
g_s= 68 g/m.s.
CONC.* VELOCITY (g/s/m^2)
DEPTH [Y] IN (cm)
0
1
2
3
4
5
6
7
8
9
10

FIG. (4.19)

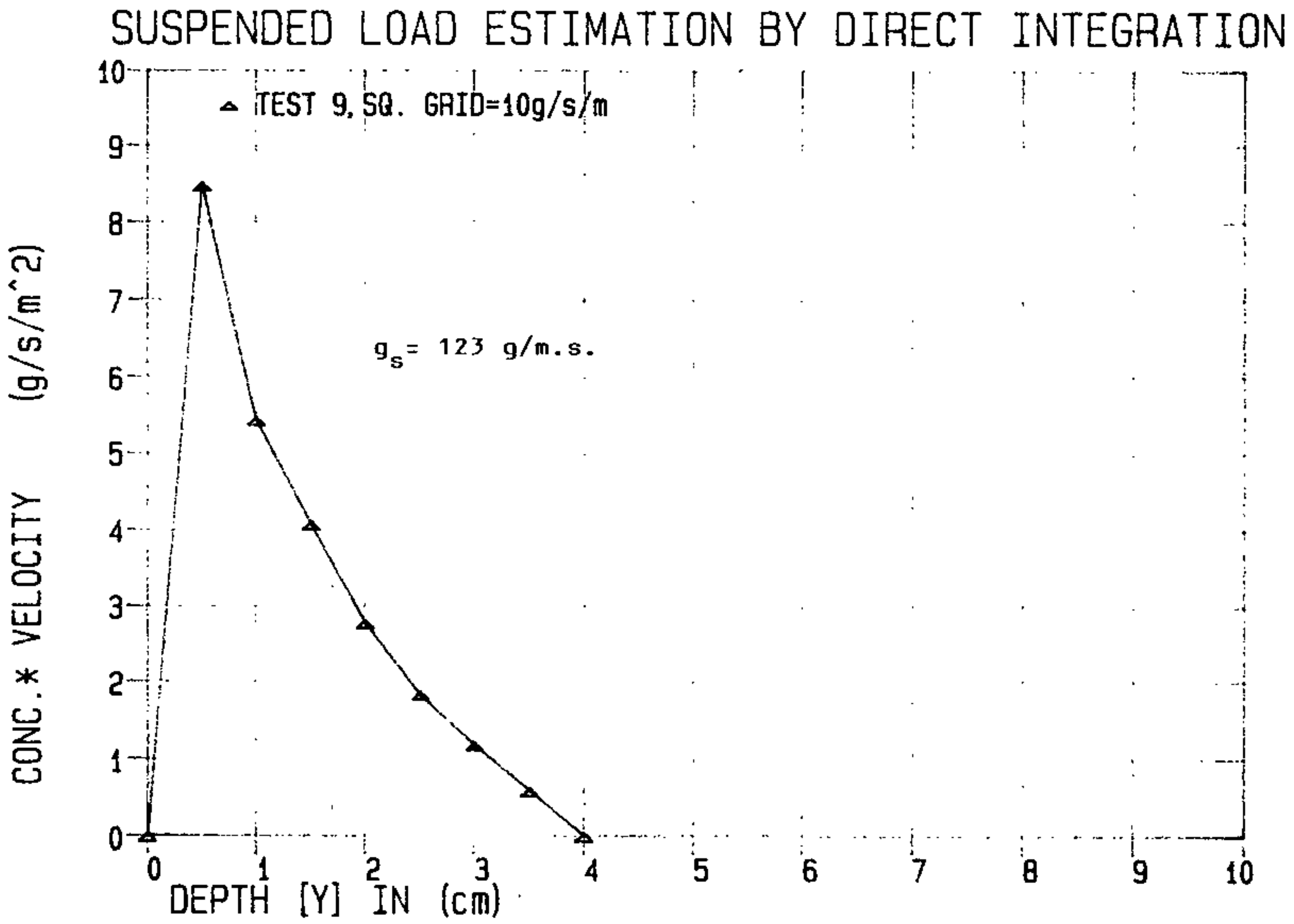

SUSPENDED LOAD ESTIMATION BY DIRECT INTEGRATION
TEST 9, SQ. GRID=10g/s/m
g_s= 123 g/m.s.
CONC.* VELOCITY (g/s/m^2)
DEPTH [Y] IN (cm)
0
1
2
3
4
5
6
7
8
9
10

FIG. (4.20)

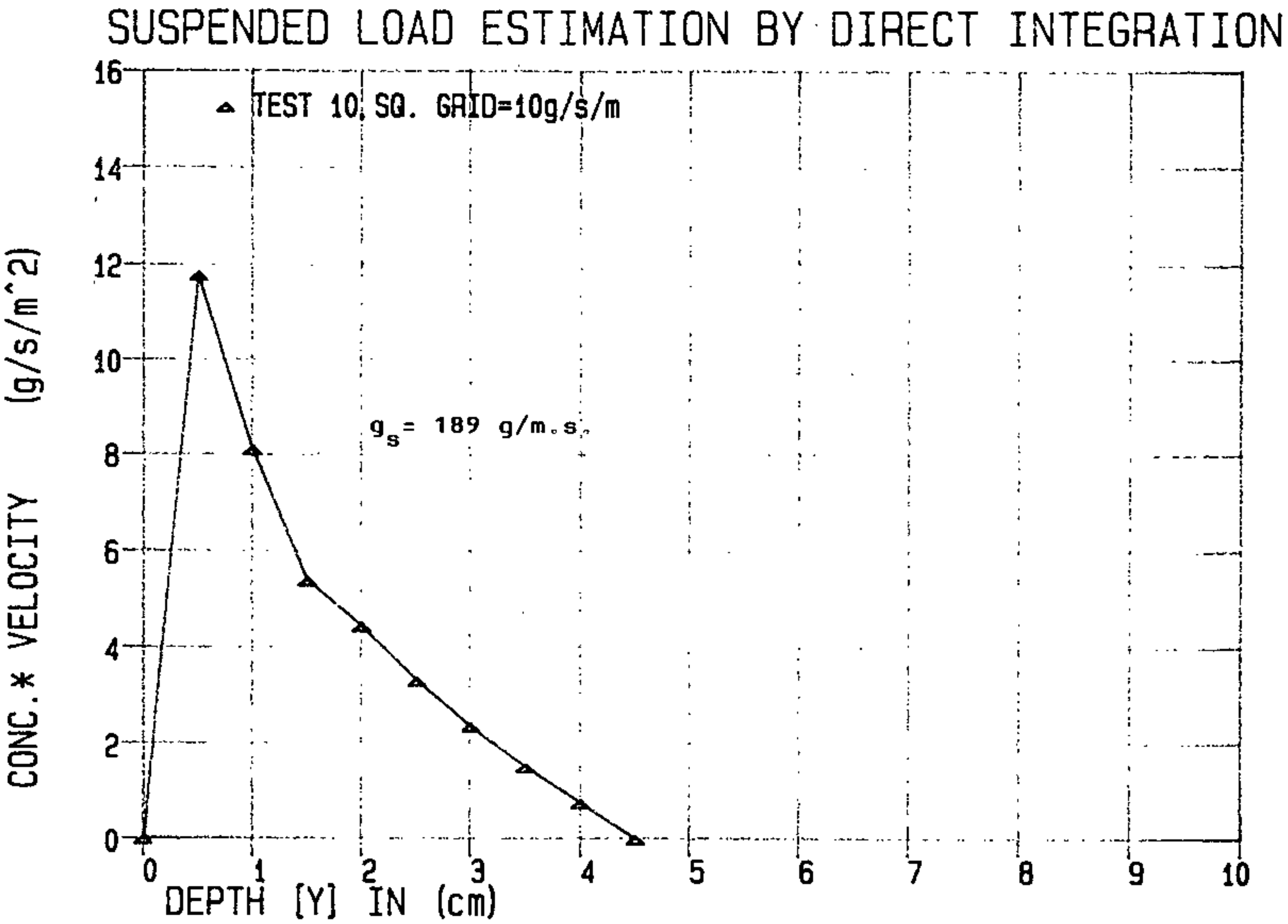

SUSPENDED LOAD ESTIMATION BY DIRECT INTEGRATION
TEST 10, SQ. GRID=10g/s/m
$g_s$ = 189 g/m.s.
CONC.* VELOCITY (g/s/m^2)
DEPTH [Y] IN (cm)
0
2
4
6
8
10
12
14
16
0
1
2
3
4
5
6
7
8
9
10

FIG. (4.21)

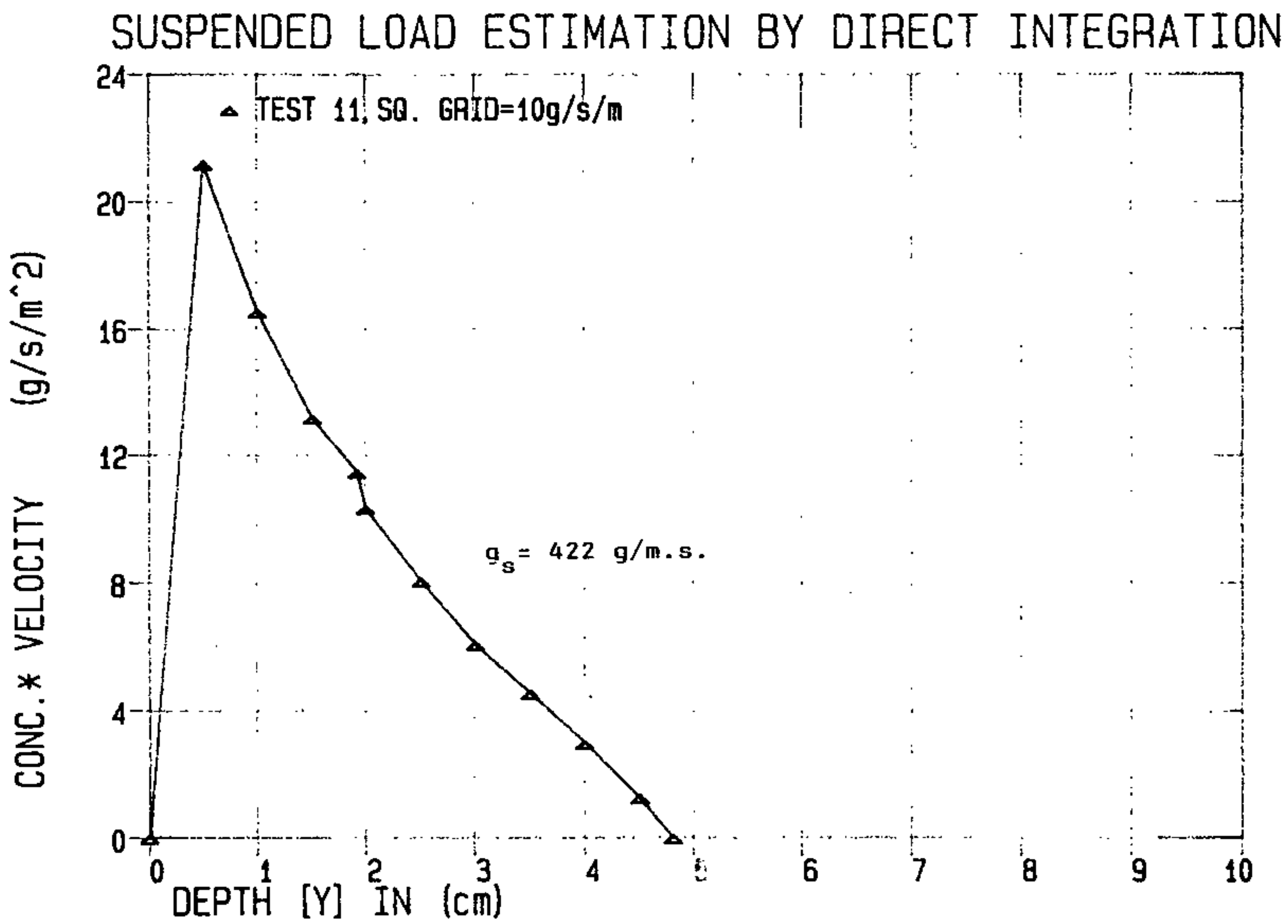

SUSPENDED LOAD ESTIMATION BY DIRECT INTEGRATION
TEST 11, SQ. GRID=10g/s/m
q_s = 422 g/m.s.
CONC.* VELOCITY (g/s/m^2)
24
20
16
12
8
4
0
0
1
2
3
4
6
7
8
9
10
DEPTH [Y] IN (cm)

FIG. (4.22)

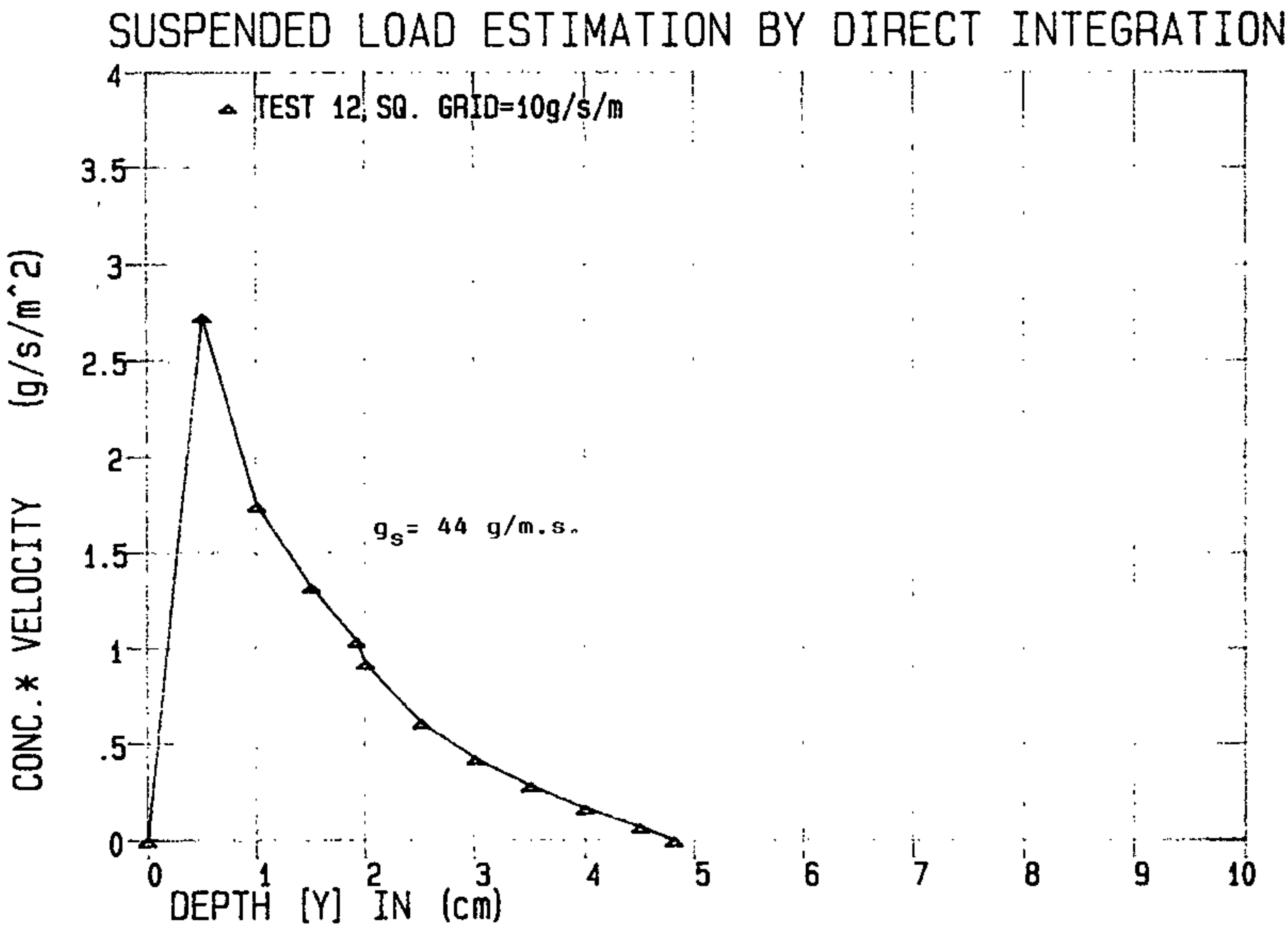

SUSPENDED LOAD ESTIMATION BY DIRECT INTEGRATION
TEST 12, SQ. GRID=10g/s/m
$g_s$= 44 g/m.s.
CONC.* VELOCITY (g/s/m^2)
DEPTH [Y] IN (cm)
0
.5
1
1.5
2
2.5
3
3.5
4
0
1
2
3
4
5
6
7
8
9
10

FIG. (4.23)

FIG. 4.24 SUSPENDED LOAD MEASUREMENTS [DRY WT.]

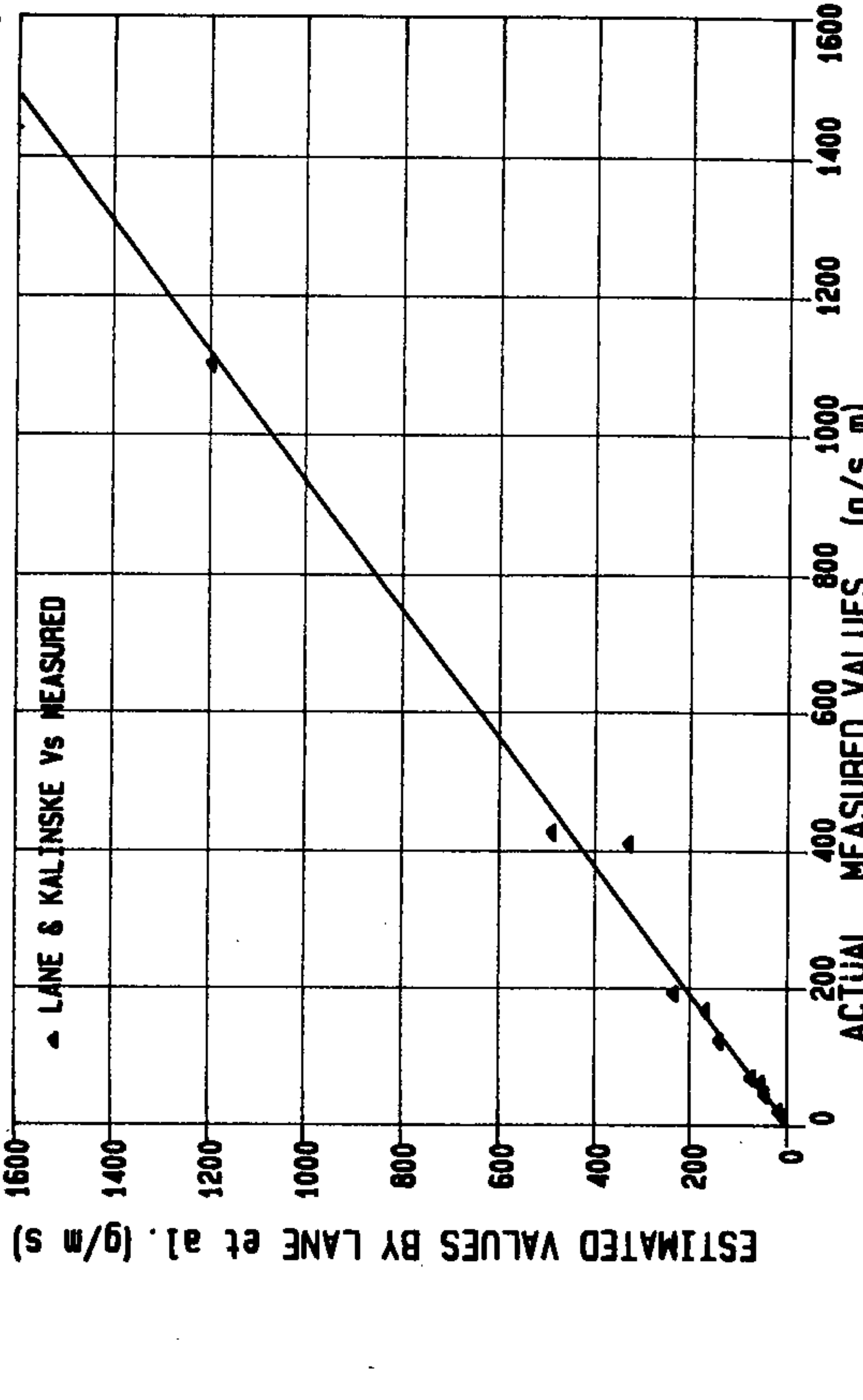


FIG. 4.25 SUSPENDED LOAD MEASUREMENTS [DRY WT.]

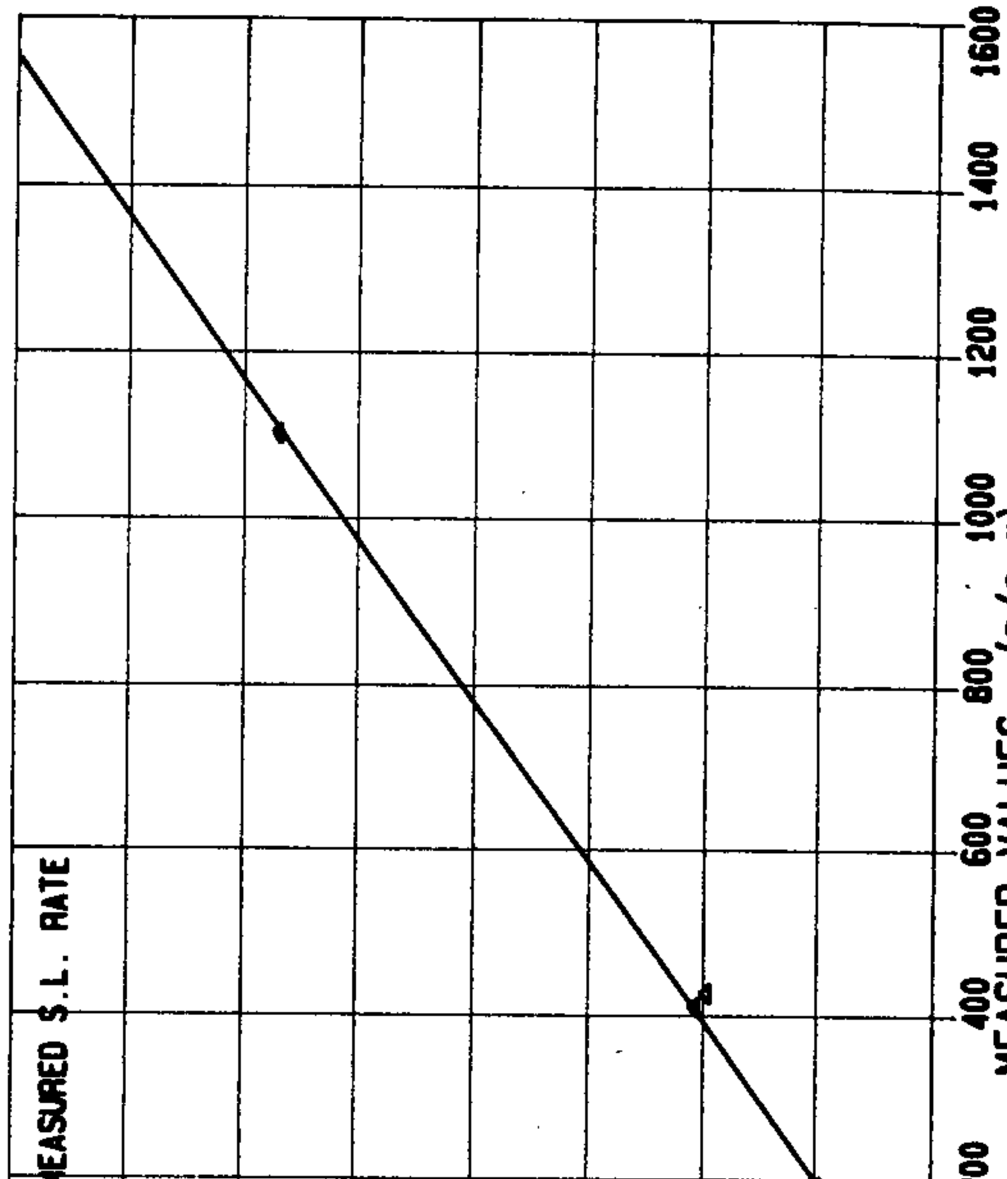


FIG.4.26.TOTAL LOAD MEASUREMENTS (DRY WT.)

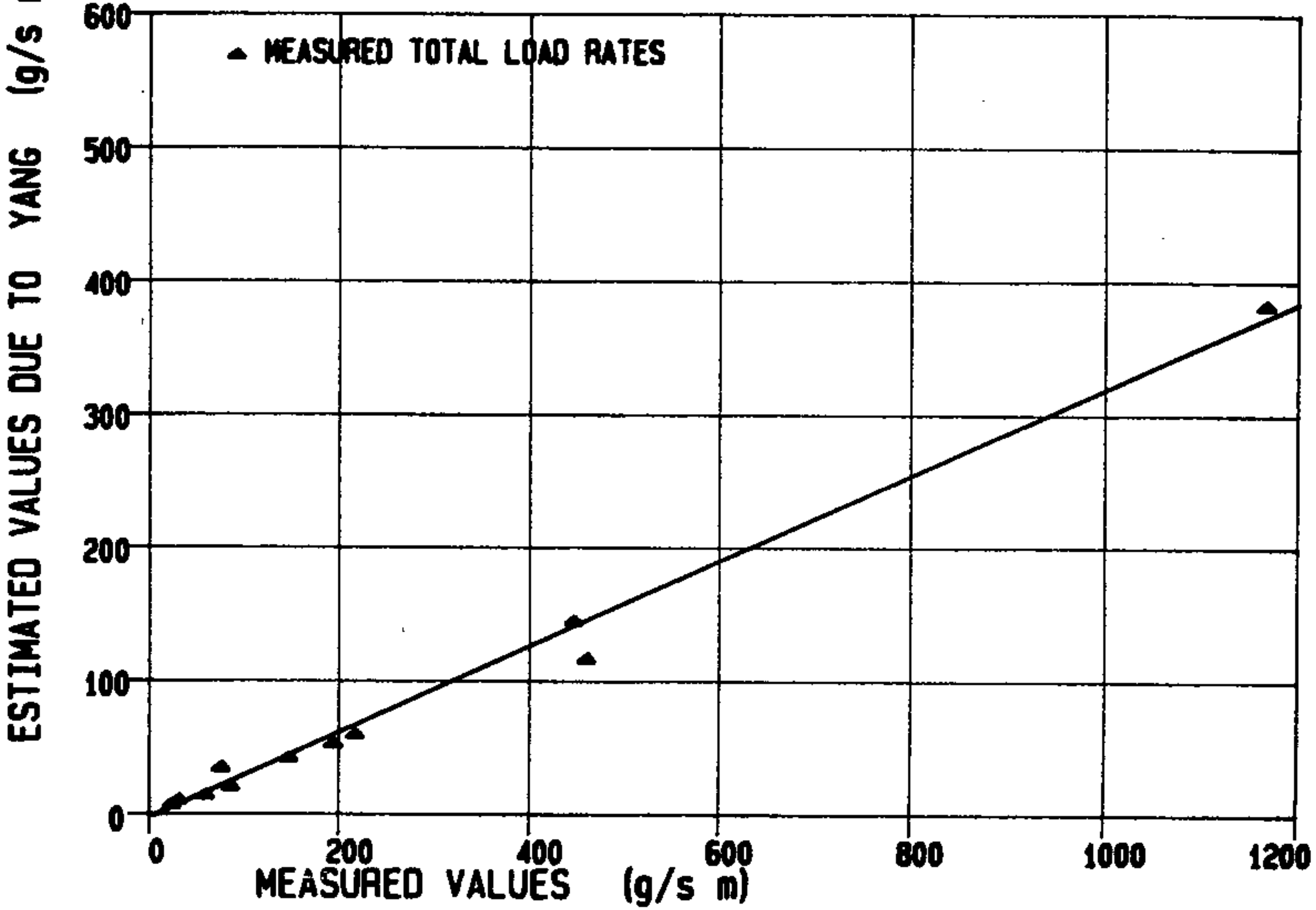


FIG. 4.27. TOTAL LOAD MEASUREMENTS [DRY WT.]

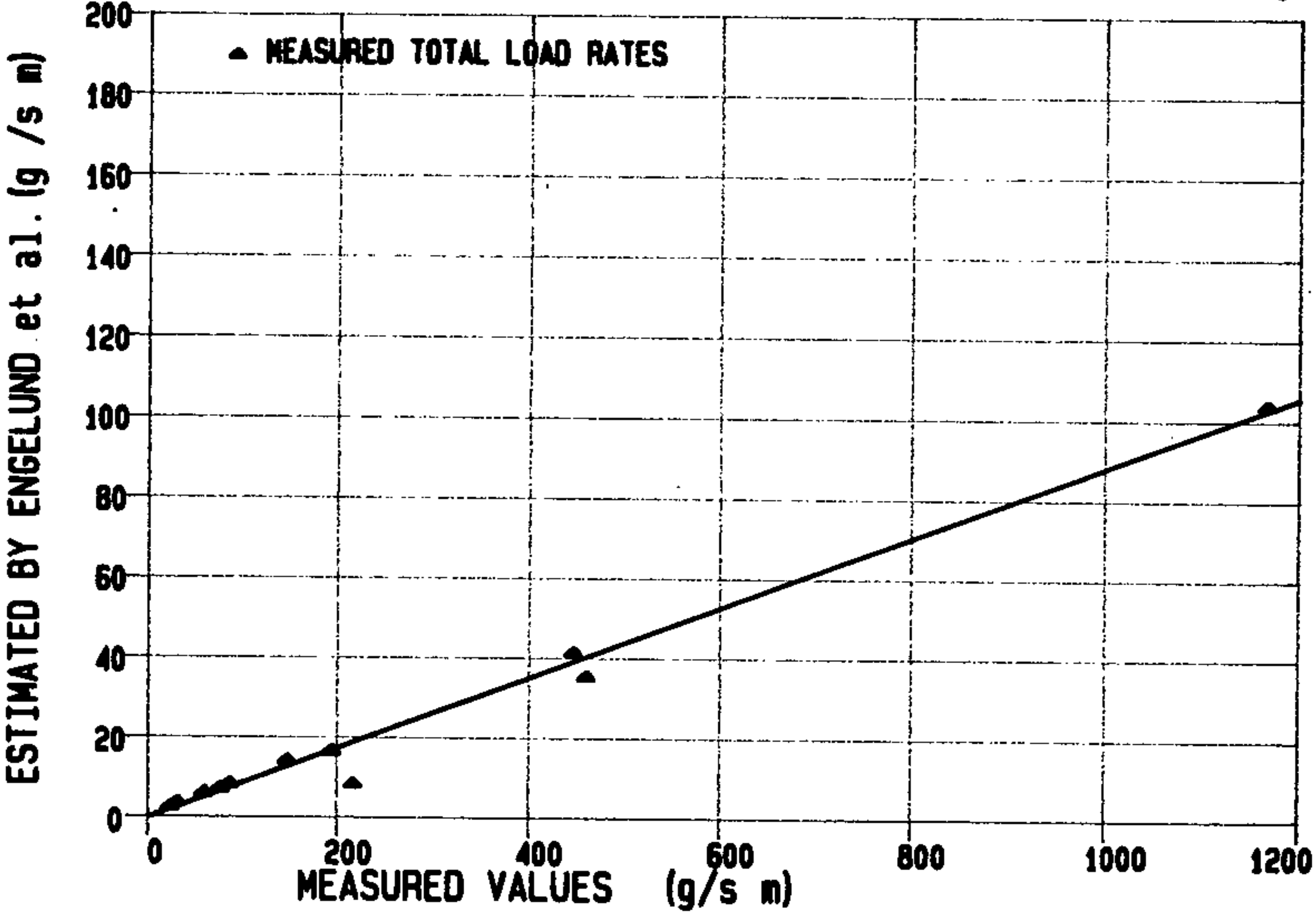


FIG.4.28. MEASURED & GRAF'S TOTAL LOAD RELATIONS

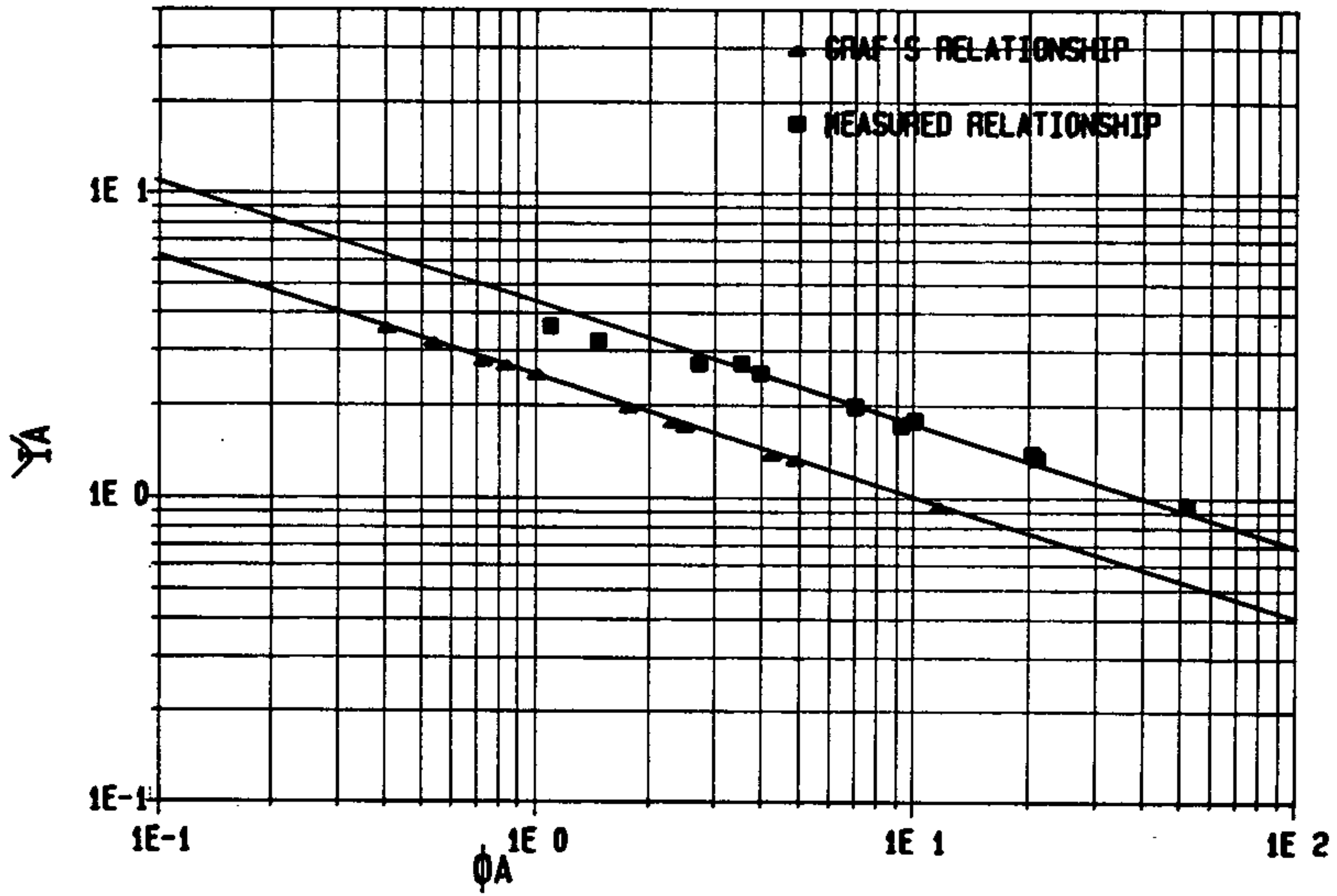


FIG.4.29. TOTAL LOAD MEASUREMENTS [DRY WT.]

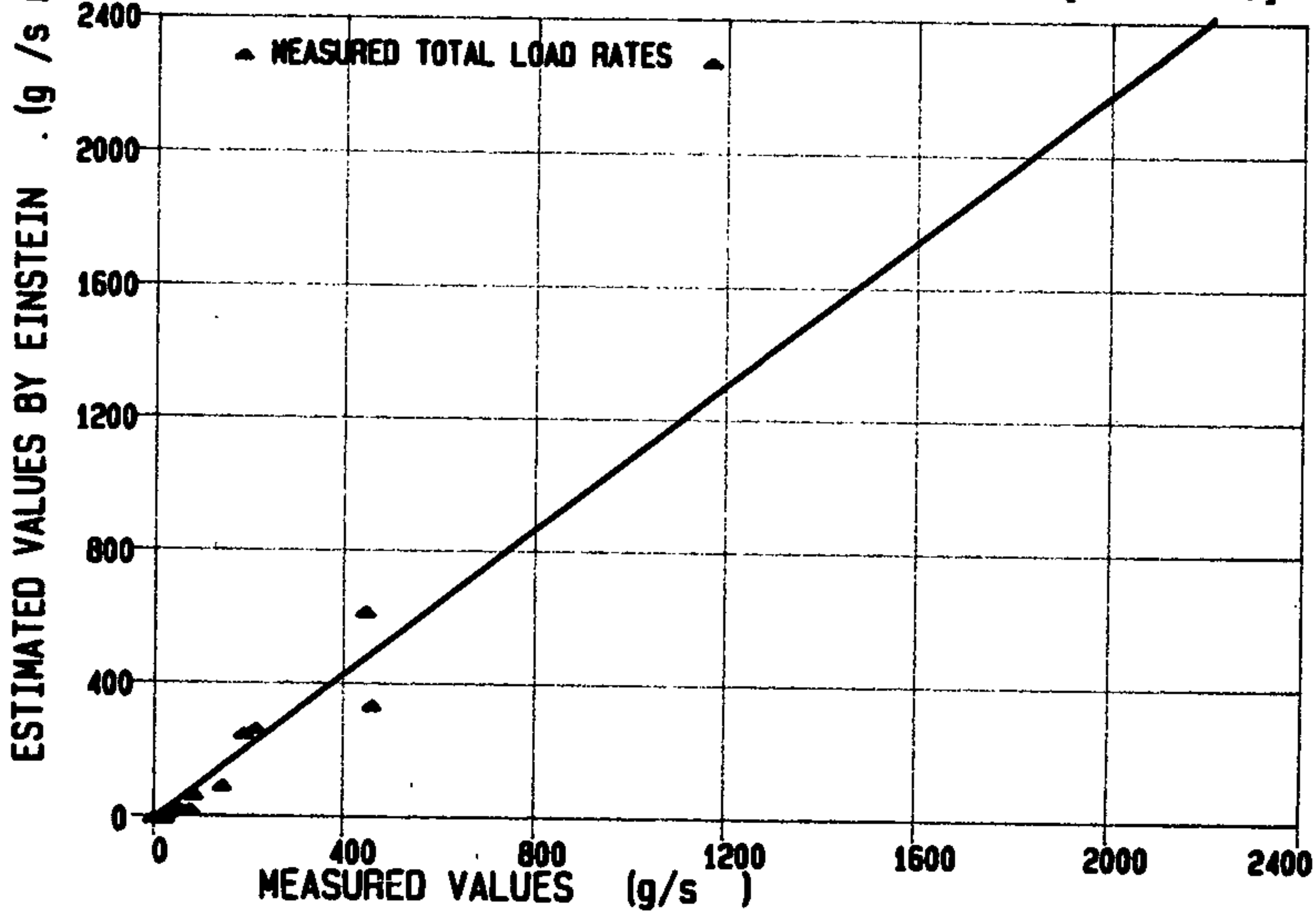



FIG. (4.29)

# CHAPTER 5

## CONCLUSIONS

Based on the foregoing analysis and discussion of the experimental work, which is carried out to study the relationship between the bed-load concentration and that of the suspended load. The main conclusions can be summarized as follows:

1. The experimental results showed that for the experimental sand used in the present work the average value of the ratio between the bed-load transport for rippled bed and that for flat bed is (0.63).
2. The analysis of the experimental results confirmed that the maximum concentration of the bed-load rate is about 1219 g/L which corresponds to a volume concentration of 0.46.
3. The measured bed-load rates are found to be in good agreement with the rates estimated by *Khalil*[28] and *Kalinske*[15].
4. The *Von-Karman's* turbulent constant is indeed a variable constant which depends on the suspended load concentration. The value of k, from the analysis of experimental results, decreases as the concentration increases. The relation is expressed as;

$$k = 0.3 - 0.025 \log C$$

This k value reaches 0.4 for silt free water.

5. The value of the sediment transfer coefficient $\epsilon_s$ is found to be 1.28 the momentum transfer coefficient $\epsilon_m$, for the experimental sand used in the present work ($d_{50}$ = 0.15 mm). i.e $\epsilon_s = 1.28\ \epsilon_m$.
also for the suspended sediment distribution exponent the theoretical values of z were found to be 1.28 times the values based on experimental results.

6. The observed suspended load rates based on experimental measurements are in good agreement with those evaluated by *Brooks*[98] *and Lane et al*[1]. The observations also confirm the findings of *Vanoni*[86] *and Ismail*[87] and some other investigators.

7. Based on experimental observations, the total load rate is found to be around three times the rate estimated by *Yang*[115], and *Graf et al*[107]. However the present results are in fair agreement with the total load rate estimated using *Einstein's*[2] approach.

8. In analogy to *Graf et al*[107] approach the present results fit the relation

$$\phi_A = 32.9\ \psi^{-2.52}$$

9. Based on experimental evidence the bed-load concentration is found to fit the suspended load concentration at a depth equal half the thickness of the moving bed layer.

10. According to the experimental analysis, the total load rate can be estimated from the hydraulic parameters of the

sediment flow, without introducing any reference concentration for the suspended load. This method of calculation is a step by step approach confirmed by experimental measurements.

11. In the present work, with uniform sediment size of 0.15 mm, the suspended load is found to contribute 90% to the total load, on average.

# REFERENCES

# REFERENCES


1-*Lane,E.W. ,and A.A.Kalinske (1940):*
"Engineering Calculations of Suspended Sediment", Trans.Am.Geophys. Union,22,pp.603-607.

2-*Einstein,H.A. (1950):*
"The Bed-Load Function for Sediment Transportation in Open Channel Flows",Tech.Bull. No.1026, U.S.Dept.of Agriculture,Soil Conservation Services,Washington,D.C.

3-*Brahms,A.* :As Quoted from Reference (72),p.86.

4-*Mavis,F.T. , T.Liu,and E.Souced (1937):*
"The Transportation of Detritus by Flowing Water", Univ. of Iowa, Studies in Eng. , No.341.

5-*Jarocki,W. (1963) :*
"A Study of Sediment",Translated from Polish ,Nat. Sci. Found. and U.S. Dept.of Int. Washington.

6-*Neill,C.R. (1968) :*
"Notes on Initial Movement of Course Uniform Material",JHR, Int.Assoc.for Hyd. Research,Vol.6, No. 2.

7-*Garde,R.J. (1970) :*
"Initiation of Motion on Hydrodynamically Rough Surface -Critical Velocity Appraoch", Jour. of Irrigation and Power (India),Vol.27, No.3.

8-*Yang,C.T. (1973) :*
"Incipient Motion and Sediment Transport", JHD., Pro.ASCE., Vol.99, No.HY10.

9-*Einsteien, H.A. ,and El-Samni (1940) :*
"Hydrodynamic Forces on a Rough Wall",Review of Modern Physics,Am.Instit. of Physics,Vol.21,No.3.

10-*Kramer,H. , (1935):*
"Sand Mixture and Sand Movement in Fluvial Modles",Trans.ASCE.,Vol. 100.

11-*U.S. Waterways Experiment Station, (1935):*
" Studies of River Bed Materials and Their Movement," USWES, Vicksburg, paper 17.

12-*Chang, Y.L., (1939):*
" Laboratory Investigation of Flume Traction and Transportation," Tranc ASCE., Vol 104, 1939.

13-*Krey* Quoted from Reference (33) ,p. 64.

14-*Schoklitsch, A., (1914):* Qouted from Reference (33), p.65.

15-*Kalinske, A.A., (1947):*
" Movement of Sediment as Bed-Load in Rivers", Tran. Am. Geophys.Union, Vol.28, No.4, Aug. 1947.

16-*Leviavsky, S., (1955):*
"An Inroduction to Fluvil Hydraulics ", Constable London, 1955

17-*Shields, A., (1936):*
Quoted from Reference (72), p. 95, 96.

18-*Zeller, J., (1963):*
Quoted from Reference (72), p. 97

19-*White, C.M., (1940)*
" The Equilibrium of Grains on the Bed of a Stream", Proc. Roy. Soc, London, Vol. 174A, pp. 322-338

20-*Yalin, M.S., and E.Karahan (1979)*
" Incipient of Sediment Transport", JHD, Proc. ASCE, Vol. 105, No. HY11, Nov. 1979

21-*Du-Boys, M.P. (1879):*
Quoted from Reference (72), pp. 124-125 .

22-*Straub L.G. (1939):*
" Laboratory Inverstigation of Flume Traction and Transportation " A Discussion, Trans. ASCE., Vol.65

23-*O'Brien, M.B. and B.D. Rindlaub (1934):*
" The Transportation of bed-Load by Streams ", Trans Am.Geophys. Union, 1934.

24-*The Same as Reference (11)*

25-*The same as Reference (12)*

26-*Brown.C.B. (1950)*

" Sediment Transportation ", in Engineering Hydraulics ( H.Rouse, ed.) Wiley, New York. Chap. XII.

27-*Tison, L.J. (1953):*

" Studies of the Critical tractive Force of Entrainment of Bed Material", Int. Assoc. Hyd. Research, 5th Congr. Minneapolis, pp. 21-35.

28-*Khalil M.B. ( 1963 ):*

" Mechanics of Bed-Load Transport and the Characteristics of Rippled-Beds with special Reference to Channel Roughness", Thesis Presented to the University of London for the Degree of Ph.D., March 1963.

29-*Malouf,K.M. (1950):*

"The Movement of Detritus in Models of River and Channel Beds", Thesis Presensed to the University of London for the Degree of Ph.D..

30-*Reichard. H. (1940):*

As Quoted from Reference (28) Above.

31-*Rottner, J.A .. (1959):*

"Formula for Bed-load Transportation" as Quoted from Reference (28).

32-*Gilbert, G.K.,(1914):*

"The Transportation of Debris by Running water", Prof.Paper,No.86,U.S.Ged.Suivey,PP.228-229.

33-*Garde R.J.and K.G.R.Raju (1987):*

Mechanics of Sediment Transportation and Alluvial Stream Problems,Second Edition,First Reprint Wiley Eastern Limited ,India .

34-*Schokhitsch,A.,(1934):*

As Quoted from Reference(72).

35-*Mac Dougall,C.H.(1933):*

"Bed sediment Transportation in Open Channels"Trans. Am.Geoplys.Union,Vol.14,PP.491-495.

36-*Casey.H.J.,(1935):*

Quoted from Reference(72).

37-*Jorrissen.A.,(1938):*

Quoted from Reference (72).

38-Quoted from Reference(72),P.134 .

39-*Meyer-Peter.E.and R.Muller, (1948):*

" Formulas for Bed-Load Transport"Proc . Inter.Assoc.for Hyd.Research,2nd Congress Stockholm,June 1948 .PP.39-64.

40-*Chien .N. (1954):*

"Meyer-Peter Formula for Bed-Load Transport and Einstein Bed-load Function",Univ of Calif .Ins.of Eng. Reseorch ,No.7.

41-*Bharat.S. (1961):*

"Bed-Load Transport in Channels",Jour. of Irrigation and Power,India ,Vol.18,No.5.

42-*Einstein, H.A. (1942):*

"Formulas for the Transportation of Bed-Load", Trans ASCE.,Vol.107,pp.561-573.

43-*Demberton. E.L. (1972):*

"Einstein's Bed-Load Function Applied to Channel Design and Degradation", Sedim. Symp. to honour Prof. Einstein, Fort Collins, Colorado U.S.A.

44-*Sato. S., H-Kikkawa and K. Ashida (1985):*

"Research on the Bed-Load Transportation", Jour. of Research, Public Works Research Instit. (Japan) Vol.28.

45-*Bagnold. R.A. (1956):*

"Flow of Cohesionless Grains in Fluids", Proc Roy. Soc. (London), Vol.249, No.964.

46-*Bagnold, R.A. (1966):*

"An Approach to the Sediment Transport Problem From General Physics", U.S. Geol. Survery Prof. paper 422-I.

47-*Yalin, M.S. (1972):*
Mechanics of Sediment Transport Pergmon Press, Oxford, 2nd Edition.

48-*Engelund, F. and J. Fredsoe (1976):*
"A sediment Transport Model for Straight Alluvial Channels", Noidic Hydrology. Vol.7.

49-*Garg, S.P. A.K. Agarwal and P.R. Singh (1971):*
"Bed-Load Transportation in Alluvial Channels", JHD, Proc ASCE, Vol.97, No. HY5, May 1971.

50-*Amin, M.I. and P.J. Murphy. (1981):*
"Two Bed-Load Formulas", JHD, Proc ASCE, Vol.107, No HY8, August 1981. pp.961-973.

51-*Toffaleti, F.B. (1969):*
Definitive Computations of Sand Discharge in Rivers, JHD, Proc. ASCE., Vol.95, No. HY1.

52-*Simons, D.B., E.V. Richardson and M.L. Albertson (1961a):*
"Flume Studies Using Medium Sand (0.45 mm)", U.S. Geol Survey Water-Supply, Paper 1948-A.

53-*Simons, D.B. and E.V. Richardson (1961):*
"Forms of Bed Roughness in Alluvial Channels", JHD Proc ASCE., Vol.87, No. HY3, May 1961.

54-*Simons, D.B. and E.V. Richardson (1962):*
"Resistance to flow in Alluvial Channels", Trans ASCE, Vol.127, Part 1, 1962.

55-*Yalin, M.S.* Included in Reference (47).

56-*Exner, F.M. (1925):*
Quoted from Reference (72).

57-*Hansen, E. (1966):*
"Bed-Load Investigation in Skive-Karup River", As Quoted from Reference (72).

58-*Engel, P. and Yi Lamb Lau (1981):*
"Bed-Load Discharge Coefficient", JHD, ASCE, Vol. 107, No. HY11, Nov. 1981, pp.1445-1455.

59-*Engel. P and Y. Lamb Lau (1980):*
"Computation of Bed-Load Using Bathy-Metric Data", JHD,ASCE, Vol.106, No.HY3, March 1980, pp.369-381.

60-*Stockes*, As Quoted from Reference (33), pp. 21.

61-*O'Seen*. As Quoted from Reference (33), pp. 22.

62-*Goldstien*, As Quoted from Reference (75), pp.6.

63-*Shiller and Naumann*, As Quoted from Reference (75), pp. 7.

64-*Rubely, W.W. (1933):*
"Settling Velocity of Gravels, Sands and Silt Particles", Am Jour of Sc. Vol.25, No. 148, April.

65-*Dallavalle. J. (1943):*
Micromeritics, Pitman, New York.

66-*Torobin, L.B. and W.H. Gauvin (1959):*
"Fundamental Aspects of Solid-Gas Flow", Part 1, Canad. Jour. of Chem. Eng. Vol.37.

67-*Oslon, R. (1961):*
Essential Engineering Fluid Mechanics, Chap 11, Scranton, Pa. U.S.A.

68-*Mc Nown, J.S. and J. Malaika (1950):*
Effect of Particle Shape on Settling Velocity at Low Reynolds Numbers", Trans. Am. Geophys. Union, Vol.31.

69-*Lamb, H. (1945):*
Hydrodynamics , Dover, New York.

70-*Brenner. H. (1961):*
The Slow Motion of a Sphere Through a Viscous Fluid Towards a Plane Surface", Chem. Eng. Sci. Vol.16.

71-*Mc Nown, J.S. (1951):*
"Particles In Slow Motion", La Houile Blanche, 6, No.5.

72-*Graf, W.H. (1971):*
Hydraulics of Sediment Transport , McGraw Hill, New York.

73-*Happel, J. and H. Brenner, (1965):*
Low Reynold Number Hydrodynamics, Prentice Hall, N.J., U.S.A.

74-*Maude, A.D. and R. Whitmore (1958):*
"A Generalized Theory of Sedimentation", Brit. J. of Appl. Phys. 9, 477-482.

75-*Raudkivi, A.J. (1976):*
Loose Boundary Hydraulics ,Pergamon Press Oxford, England, 2nd Edition.

76-*Dobbins, W.E. (1943):*
"Effect of Turbulence on Sedimentation", Trans ASCE, Vol. 109.

77-*Hayami, S. (1938):*
"Hydrological Studies on the Yangtze River ..." Jour. of Shanghi Sci. Instit. China, Sec.1, Vol.1.

78- As Quoted from Reference (33).

79-*O'Brien, M.P. (1933):*
"Review of the Theory of Turbulence Flow and Its Relation to Sediment Transportation", Trans. Am. Geophys. Union.

80-*Hurst, H.E. (1929):*
"The suspension of Sand in Water", Roy. Soc. London, Series A, Vol.157.

81-*Rouse, H. (1938):*
"Experiments on the Mechanics of Sediment Suspension", Proc. 5th Inter. Congr. Appl. Mech.,Cambridge, Mass.

82-*Rouse, H. (1937):*
"Modern Conceptions of the Mechanics of Turbulence", Trans ASCE. Vol.102.

83- As Quoted from Reference (33).

84-*Hunt, J.N. (1954):*
"The turbulence Transport of Suspended Sediment in Open Channels", Roy Soc., London A, 224, No.1158. pp. 322-325.

85-*Hunt, J.N. (1969)*:
"On The Turbulence Transport of Hetrogeneuous Sediment",Quart J.Mech. Appl.Math. 22, pp.234-246.
86-*Vanoni, V.A. (1946)*:
"Transportation of Suspended Sediment by Water", Trans. ASCE, Paper No. 2267, pp. 67-133.
87-*Ismail, H.M. (1952)*:
"Turbulence Transfer Mechanism and Suspended Sediment in Closed Channels",Trans. ASCE. Vol.117, Paper No. 2500, pp. 409-446.
88-*Einstein, H.A. and N. Chien (1954)*:
"Second Approximation To Solution of Suspended-Load Theory", Univ. of Calif. Inst. of Eng. Res., No.8.
89-*Tanak, S. and S. Sugimoto (1958)*:
As Quoted from Reference (33).
90-*Paintal, A.S. and R.J. Garde (1964)*:
"Discussion of Sediment Transport Mechanics: Suspension of Sediment", JHD,ASCE, Vol.90, NO.HY4, July 1964.
91-*Navntoft, E.A. (1970)*:
"A Theory for the Velocity and Suspended-Load Distribution in a Two-Dimensional Steady and Uniform Open Channel Flow", Report No.3, Danish Centre for Applied Mechanics, Tech. Univ. of Denemark.
92-*Lavelle, J.W. and W.C. Thacker (1978)*:
"Effect of Hindered Settling on Sediment Concentration Profiles", Jour. of Hyd. Research, Int. Assoc. of Hyd. Research, Vol.16, No.4.
93-*Willis, J.C. (1978)*:
"Suspended-Load From Error-Function Model", JHD, ASCE, Vol.105, No. HY7.

94-*McTingue, D.F. (1981):*
"Mixture Theory for Suspended Sediment Transport", JHD, ASCE, Vol.107, No. HY6.
95-*Knapp, R.T. (1938):*
"Energy Balance in Stream Flows Carring Suspended-Load", Am. Geophys. Union, Trans., pp. 501-505.
96a-*Velikanov, M.A. (1954):*
"Principle of the Gravitational Theory of the Movement of Sediments", As Quoted from Reference (75).
96b-*Velikanov, N.H. (1963):*
As Quoted from Reference (75).
97-*Velikanov, M.A. (1958):*
As Quoted from Reference (75).
98-*Brooks, N.H. (1963):*
"Calculations of Suspended-Load Discharge from Velocity and Concentration Parameters", Proc. Fed. Interag. Sedim. Conf., U.S. Dept of Agr., Misc. Publ. No. 970.
99-*Samaga, B.R. (1984):*
"Total Load Transport of Sediment Mixtures", Ph.D. Thesis Presented to Univ. of Roorkee, India.
100-*Colby, B.R. and C.H. Hombree (1955):*
"Computation of Total Sediment Discharge, Niobrara River Near Cody, Nebraska", U.S. Geo. Survey, Water-Supply Paper 1357.
101-*Einstein, H.A. (1964):*
"River Sedimentation", in "Handbook of Hydrology" (V.T. Chow edit.) McGraw Hill, New York.
102-*Chang, F.M., D.B. Simons, and E.V. Richardson (1967):*
"Total Bed Material Discharge in Alluvial Channels", Inter. Assoc. Hydr. Res; 12th Congress Fort Collins, Coloradi.

103-*Colby, B.R. (1964):*
"Partical Computation of Bed Material Discharge", Proc. ASCE, Vol.90, No. HY2.

104-*Laursen, E.M. (1958):*
"The Total Sediment Load of Streams", Proc. ASCE, Vol.84, No. HY1.

105-*Bogardi, J.L. (1965):*
"European Concepts of Sediment Transportation", Proc. ASCE, Vol.84, No. HY6.

106-*Bishop, A.A., D.B. Simons and E.V. Richardson (1965):*
"Total Bed Material Transport",Proc. ASCE, Vol.91, No. HY2.

107-*Graf, W.H. and E.R. Acaroglu (1968):*
"Sediment Transport in Conveyence Systems",Part 1, Bull Int. Assoc. Sci. Hydrology, Vol.13, No. 2.

108-*Engelund, F. and E. Hansen (1967):*
"A Monograph on Sediment Transport in Alluvial Streams", Teknish Vorlag, Compenhagen, Denemark.

109-*Engelund, F. (1973):*
"Steady Transport on Moderatly Graded Sediment", Tech. Univ. of Denemark, Progress Report 21.

110-*Shen, H.W. and C.S. Hung (1971):*
"An Engineering Approach to Total Bed Material Load by Regression Analysis", Proc. Sedimentation Symp., Berkeley, U.S.A.

111-*Acker, P. and W.R. White (1973):*
"Sediment Transport-New Approach and Analysis", JHD, ASCE, Vol.99, No. HY11, pp. 2041-2058.

112-*Yang, C.T. (1972):*
"Unit Stream Power and Sediment Transport", Proc. ASCE, Vol98, No. Hy10, pp. 1805-1826.

113-*Yang, C.T., and J.B. Stall (1976):*
"Applicability of Unit Stream Power Equation", ASCE, Vol.102, No. HY5, pp. 559-568.

114-*Yang, C.T. (1979):*
Unit Stream Power Equation for Total Load", Jour. of Hydrology 40, pp. 128-138.

115-*Yang, C.T. and A. Molinas (1982):*
"Sediment Transport and Unit Stream Power Function", JHD, ASCE, Vol.108, No. HY6, pp. 774-793.

116-*Maddok, T.Jr. (1969):*
"The behaviour of Straight Open Channels with Movable Beds", U.S. Geol. Surve Prof. Paper,662-A.

117-*Kikkawa, H. and T. Ishikawa (1978):*
"Total Load of Bed Material in Open Channels", JHD, ASCE, Vol.104, No. HY7, pp. 1045-1059.

118-*Raju, K.G. Garde and R.C. Bhardwaj (1981):*
"Total Load Transport in Alluvial Channels", JHD, ASCE, Vol.107, No. HY2, pp. 179-191.

119-*Vittal, N., R. Raju and R.J. Garde (1973):*
"Sediment Transport Relations Using Concept of Effective Shear Stress", Proceeding Int. Assoc. of Hyd. Res. Int. Symp. on River Mechanics, Bangkok, Thailand pp. 489-499.

120-*Brownlie, W.R. (1981):*
"Prediction of Flow Depth and Sediment Discharge in Open Channels", Report No. KH-R-43A, W.M. Keck Lab, Caltec, U.S.A.

121-*Karim, M.F. and J.F. Kennedy (1983):*
"Computer-Based Predictors for Sediment Discharge and Friction Factor of Alluvial Channels",Nanjing, China.

122-*De Vries, M. (1983):*
"Keynote Address on (Hydraulics of Alluvial Rivers)", Proc. of 2nd Int. Symp. on River Sedim., Nanjing, China.

123-*Johnson, J.W., (1942):*

"The Importance of Side-Wall Friction in Bed-Load Investigations", Civil Engineering, Vol.12, No. 6, pp. 329-331, As Quoted from References (126).

124-*Khalil, M.B., (1969):*

"Hydraulic Roughness of Channels with Rippled-Beds", University of Assiut Bulletin of Science and Technology, Vol.12.

125-*Yassin, A.M., (1953):*

"Mean Roughness Coefficient in Open Channels with Different Roughness of Bed and Side Walls", Thesis Swiss Federal Institute of Technology, Zurich. As Quoted from Reference (126).

126-*Ali, N.A., (1978):*

"A Contribution to Sediment Transportation with Reference to Hydraulic Resistance",A Master Thesis Submitted to the Faculty of Engineering, Assiut University. Egypt.

# APPENDICES

## SUMMARY OF EXPERIMENTAL RESULTS

THE RESULTS OF ALL EXPERIMENTAL MEASUREMENTS AND CALCULATINS ARE LISTED IN TABULAR FORM .THE ENTERIES OF EACH COLUMN ARE WELL DEFINED AS SHOWN BELOW.

(TABLE A.1)

(1) Test number
(2) Flow depth in (cm).
(3) Channel slope
(4) The flow rate Q , L/S
(5) The mean velocity = Q/A = Q * 100/D.(0.3)1000 m/s
(6) The hydraulic radius associated with the bed corrected for side wall effect as explained in Appendix B.
(7) The bed shear stress = $\rho g\ R_b S$ in $N/m^2$
(8) Shear velocity $U_* = (\tau_b/\rho)^{1/2} = (gR_b S)^{1/2}$, m/s
(9) The particle Reynold's Number = $U_* d/\nu$
(10) The Von Karman's turbulent constant estimated from the slope of semilog plot of velocity profile measured.
(11) The suspended sediment concentration distribution exponent as estimated from the slope of the log-log graph of the measured concentration profiles.
(12) Bed-load rate per unit width based on submerged weight as measured using the bed-load trap of 8 cm width converted to unit width.
(13) Number of moving layers as a bed-load estimated as explained earlier to be:

$$m = \tau_b/0.645$$

(14) The mid depth concentration measured using the suspended load sampler based on submerged wt per litre.

TABLE A.1.

SUMMARY OF EXPERIMENTAL WORK

| (1) Test No. | (2) D (cm) | (3) S (m/m) | (4) Q (L/s) | (5) U (m/s) | (6) $R_b$ (cm) | (7) $\tau_b$ (N/m²) | (8) $U_*$ (m/s) | (9) $Re_*$ | (10) k (actual) | (11) $z_1$ measured | (12) Bed-Load g/s m' (Sub wt.) | (13) No. of moving layers | (14) $C_{md}$ g L (Sub wt.) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 2 | 5.9 | 1/200 | 9.30 | 0.525 | 5.19 | 2.544 | 0.0504 | 7.57 | 0.258 | 0.68 | 40.77 | 3.95 | 20.9 |
| 3 | 3.3 | 1/400 | 2.75 | 0.278 | 3.04 | 0.746 | 0.0273 | 4.10 | 0.283 | 1.13 | 7.59 | 1.16 | 0.9 |
| 4 | 4.0 | 1/400 | 3.71 | 0.309 | 3.63 | 0.891 | 0.0298 | 4.48 | 0.280 | 1.02 | 9.85 | 1.83 | 1.8 |
| 5 | 3.0 | 1/200 | 2.93 | 0.325 | 2.82 | 1.385 | 0.0372 | 5.58 | 0.267 | 0.86 | 17.26 | 2.15 | 8.6 |
| 6 | 4.0 | 1/200 | 5.04 | 0.424 | 3.65 | 1.790 | 0.0423 | 6.35 | 0.264 | 0.76 | 24.33 | 2.78 | 14.1 |
| 7 | 3.7 | 1/500 | 2.77 | 0.256 | 3.40 | 0.666 | 0.0258 | 3.87 | 0.295 | 1.2 | 6.67 | 1.03 | 0.5 |
| 8 | 5.5 | 1/500 | 5.12 | 0.315 | 4.88 | 0.958 | 0.0310 | 4.65 | 0.278 | 1.0 | 10.98 | 1.49 | 1.6 |
| 9 | 4.0 | 1/300 | 3.96 | 0.336 | 3.66 | 1.198 | 0.0346 | 5.19 | 0.273 | 0.9 | 14.72 | 1.86 | 4.7 |
| 10 | 4.5 | 1/300 | 4.86 | 0.365 | 4.07 | 1.332 | 0.0365 | 5.47 | 0.270 | 0.85 | 16.29 | 2.07 | 6.0 |
| 11 | 4.8 | 1/250 | 5.90 | 0.416 | 4.32 | 1.694 | 0.0411 | 6.16 | 0.265 | 0.76 | 22.65 | 2.63 | 11.8 |
| 12 | 4.8 | 1/500 | 4.18 | 0.195 | 4.31 | 0.846 | 0.0290 | 4.35 | 0.283 | 1.03 | 9.03 | 1.31 | 1.3 |

TABLE (A.2)

Summary of Bed-Load Measurements and Analysis

all rates are in g/m$^{1}$.s based on sub. wt.

| 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|
| Test no | Measured Rate | Estimated Rate on flat bed | Estimated Rate by Khalil | Estimated Rate by Kalinske |
| 2 | 40.77 | 73.31 | 41.17 | 46.06 |
| 3 | 7.59 | 11.64 | 6.54 | 10.13 |
| 4 | 9.85 | 15.18 | 8.52 | 12.5 |
| 5 | 17.26 | 29.40 | 16.55 | 18.41 |
| 6 | 24.33 | 43.29 | 24.31 | 22.45 |
| 7 | 6.67 | 9.81 | 5.52 | 10.85 |
| 8 | 10.98 | 16.96 | 6.92 | 14.1 |
| 9 | 14.72 | 23.70 | 13.33 | 17.12 |
| 10 | 16.29 | 27.75 | 15.61 | 18.06 |
| 11 | 22.65 | 39.82 | 22.27 | 21.36 |
| 12 | 9.03 | 14.0 | 7.88 | 10.76 |

1- Test number

2- Measured bed-load rate based on sumberged wt. per unit width.

3- Estimated bed-load rate on flat bed based on equation (4.6).

4- Estimated bed-load rate on rippled bed based on equation (4.6), taking on empirical factor = 0.63, or based on graphical relation supplied by Khalil Fig.(2.3) based on submerged wt. basis.

5- Estimated bed-load rate based on Khalil approach and graphical relation Fig.(2.1) based on submerged wt.basis.

TABLE (A.3)
Summary of Suspended-Load Measurement and Analysis
(all rates are in g/m.s based on dry wt.)

| 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|
| Test no | measured concent. $C_{md}$ (g/L) | measured $g_s$ rate per unit width | estimated rate by Brooks | estimated rates by Lane et al (g/L) |
| 2 | 33.6 | 1100.0 | 1145.76 | 1202.0 |
| 3 | 1.5 | 18.80 | 22.00 | 25.41 |
| 4 | 2.9 | 60.0 | 58.10 | 61.89 |
| 5 | 13.8 | 165.0 | 183.30 | 178.29 |
| 6 | 22.7 | 406.0 | 419.50 | 337.86 |
| 7 | 0.8 | 12.10 | 14.77 | 16.27 |
| 8 | 2.5 | 68.0 | 71.68 | 82.44 |
| 9 | 7.6 | 123.0 | 140.45 | 146.97 |
| 10 | 9.6 | 189.0 | 209.95 | 241.49 |
| 11 | 18.9 | 422.0 | 408.87 | 497.97 |
| 12 | 2.1 | 44.0 | 46.82 | 58.88 |

1- Test number.

2- Measured concentration of mid depth of the flow.

3- Measured suspended load rate estimated based on concentration and velocity measurements. The graphical Integration method is used.

4- Estimated due to Brooks' approach based on measured values of $C_{md}$ and q, u, $z_1$, k and $U_*$ given in table (A.1). with these values known, the Brooks' function $T_B^*$ in equation is estimated graphically based on the graph shown in (Fig.2.9).

5- Estimated due to Lane etal approach based on measured values of D, ω, $U_*$ and q, taking $n = 24/d^{1/6}$.
the function D of lane is evaluated from the graph given in Fig.(2.7) after being converted to SI units.

TABLE (A.4)

Total Load Rates

(dry wt. bases in g/m.s)

| 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|
| Test no. | measured Total load rate . | Estimated by Yang's approach | Estimated by Engelun approcah | Estimated by Einstein's |
| 2 | 1165.48 | 383.0 | 104.9 | 2280.0 |
| 3 | 30.99 | 14.54 | 4.89 | 11.27 |
| 4 | 75.82 | 39.98 | 7.78 | 22.1 |
| 5 | 192.72 | 57.94 | 17.28 | 257.4 |
| 6 | 445.08 | 147.07 | 41.9 | 619.4 |
| 7 | 22.81 | 9.06 | 3.48 | 0.116 |
| 8 | 85.63 | 24.28 | 9.44 | 64.76 |
| 9 | 146.64 | 45.05 | 15.0 | 96.4 |
| 10 | 215.16 | 62.82 | 19.50 | 267.0 |
| 11 | 458.38 | 121.69 | 36.50 | 336.2 |
| 12 | 58.50 | 17.61 | 6.51 | 21.6 |

1- Test number.

2- Measured total load rate = bed-load rate + suspended load rate per unit width.

3- Estimated by Yang's equation (2.273).

4- Estimated by Engelund's etal approach outlined in equations (2.243) to (2.247).

5- Estimate by Einstein approach outlined earlier in equation with the aids of graphes in Fig.( 2.4) and Fig. (2.9).

TABLE (A.5)
Measured Total Load Compared to
Graf's etal Method

| 1 | 2 | 3 | 4 |
|---|---|---|---|
| Test No. | $\psi_A = (\rho_s-\rho)/\rho\ d/SR$ | $\phi_A = CUR\ /(\rho_s-\rho/\rho)gd^3$ | $\phi_A$ given by Graf et al |
| 2 | 0.954 | 51.94 | 11.70 |
| 3 | 3.256 | 1.457 | 0.53 |
| 4 | 2.727 | 3.51 | 0.83 |
| 5 | 1.755 | 9.23 | 2.50 |
| 6 | 1.356 | 20.93 | 4.82 |
| 7 | 3.64 | 1.09 | 0.40 |
| 8 | 2.536 | 3.93 | 1.00 |
| 9 | 2.03 | 6.99 | 1.75 |
| 10 | 1.82 | 10.0 | 2.30 |
| 11 | 1.43 | 20.47 | 4.20 |
| 12 | 2.87 | 2.71 | 0.72 |

1- Test number.

2- $\psi_A = (\rho_s-\rho)/\rho\ d/SR_h$ based on measured values.

3- $\phi_A = C_v U\ R_h/[\ ((\rho_s-\rho)/\rho\ dg^3)^{1/2}]$

based on measured values, where

$C_v$ = measured total load volume/volume flow rate.

other terms are explained earlier.

4- $\phi_A$ given by Graf etal as shown in Fig.(2.10).

TABLE (A.6)
Effect of Sediment Load on Flow Characteristics

| 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|
| Test no. | Average Conc. (g/L) | Von Karman's Const. k | measured Exponent ($z_1$) | Calculated Exponent z |
| 2 | 37.6 | 0.258 | 0.68 | 0.85 |
| 3 | 3.38 | 0.283 | 1.13 | 1.42 |
| 4 | 6.13 | 0.280 | 1.02 | 1.32 |
| 5 | 19.73 | 0.267 | 0.86 | 1.11 |
| 6 | 26.49 | 0.264 | 0.76 | 0.985 |
| 7 | 2.47 | 0.295 | 1.2 | 1.44 |
| 8 | 5.02 | 0.278 | 1.0 | 1.28 |
| 9 | 11.12 | 0.273 | 0.9 | 1.16 |
| 10 | 13.28 | 0.270 | 0.85 | 1.12 |
| 11 | 22.31 | 0.262 | 0.76 | 1.01 |
| 12 | 4.20 | 0.283 | 1.03 | 1.34 |

1- Test number

2- The dry wt of the total load rate per unit width divided by the flow rate per unit width.

3- Determined from the slope of velocity profiles on semi-log scale log y Vs. U for each run.

4- Estimated from the slope of concentration profiles given in Figs. (4.8) through (4.11) for each run.

5- Estimated as

$z = \omega / U_* k$ where

$\omega$ measured fall velocity after being correct for grain concentration. Other values are explained earlier.

TABLE (A.7)

Bed-Load Concentration Compared to Suspended Load Concentration at the Mid Thickness of the Bed Layer (based on dry wt.)

| 1 | 2 | 3 | 4 | 5 | 6 | 7 |
|---|---|---|---|---|---|---|
| Test No. | $z_1$ value | $C_{md}$ (g/L) | a=md (mm) | Suspended load concent. at a/z $C_s$ (g/L) | Bed-load concent. $C_b$ (g/L) | % deviation |
| 2 | 0.68 | 33.6 | 0.593 | 1229.9 | 1219 | 0.9 |
| 3 | 1.13 | 1.5 | 0.174 | 1230.9 | 1219 | 0.9 |
| 4 | 1.02 | 2.9 | 0.207 | 1255.7 | 1219 | 3.0 |
| 5 | 0.86 | 13.8 | 0.322 | 1223.5 | 1219 | 0.3 |
| 6 | 0.76 | 22.7 | 0.417 | 1224.3 | 1219 | 0.3 |
| 7 | 1.20 | 0.8 | 0.155 | 1310.0 | 1219 | 7.4 |
| 8 | 1.0 | 2.5 | 0.224 | 1230.0 | 1219 | 1 |
| 9 | 0.9 | 7.6 | 0.279 | 1234.0 | 1219 | 1.2 |
| 10 | 0.88 | 9.6 | 0.310 | 1224.1 | 1219 | 0.4 |
| 11 | 0.76 | 18.9 | 0.395 | 1225.1 | 1219 | 0.5 |
| 12 | 1.03 | 2.1 | 0.196 | 1233.9 | 1219 | 1.2 |

1- Test number.

2- Measured suspended load exponent estimated from the slope of log D-y/D Vs. Log C.

3- Measured concentration at D/2 in g/L (dry wt.).

4- The bed layer thickness = number of layer in motion times the mean grain size.

5- The suspended load concentration extrapolated to the distance a/2 from the bed, given by:

$$C_s = C_{md}(D-(a/2)/a/2)^{z_1} \quad \text{in g/L (dry wt.)}$$

6- The bed-load concentration based on 0.46 volume concentration in (g/L), dry weight.

7- % Deviation = $|C_s - C_b/C_b| * 100\%$ .

## APPENDIX (B)

### SIDE WALLS CORRECTION PROCEDURE FOR OPEN CONDUIT

The flow in open conduit with finite width, is resisted partly by the bed and partly by the sides. Since the flume is covered by sand, it will generally be much rougher than the flume walls and thus will be subjected to higher value of shear stress. The problem considered is the development of a calculation procedure for determining the average shear stress on the bed.

The method used here to correct for the effect of side walls, is proposed by *Einstein*[42] and *Johonson*[123]. The principal assumption is that the cross-sectional area can be divided into two parts $A_b$ and $A_w$ in which the component of gravity force is resisted by the shear force exerted on the bed and the walls respectively.

It is further assumed that the mean velocity and the energy gradient are the same for $A_b$ and $A_w$. *Khalil*[124], in 1969 from analysis of *Yassin's*[125] experiments, found that the velocity factor designated by $\alpha = V_w / V$ is considered to be unity when the width-depth ratio $>7$ provided that $f_b / f_w < 2.3$. Thus for rectangular cross-section as shown in Fig (B.1) the correction is achieved as:

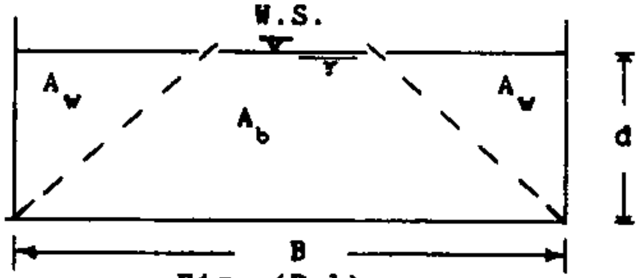


Fig. (B.1)

The hydraulic radius for the wall $R_w$ is calculated using Manning's formula as;

$$R_w = (V\ n_w/S)^{1.5} \qquad \dots (B.1)$$

Where $n_w$ is the *Manning* coefficient for the sides and V is the mean velocity. The sub-area of the flow taken by the walls is

$$A_w = 2\ d\ R_w \qquad \dots (B.2)$$

Thus, the sub-area for the bed is

$$A_b = A - A_w \qquad \dots (B.3)$$

So the hydraulic radius for the bed only is given by;

$$R_b = A_b/B \qquad \dots (B.4)$$

And the corresponding bed shear stress is obtained as ;

$$\tau_b = \rho . g . R_b . S \qquad \dots (B.5)$$

The value of $n_w$ used here is taken (0.0093) for steel bed and glass sided flume.This was verified by *N. Ali*[(126)] in 1978.

# APPENDIX C

**WORKED EXAMPLE USING THE PRESENT APPROACH TO CALCULATE THE TOTAL LOAD RATE, BASED ON SEDIMENT FLOW PARAMETERS.**

**Sediment Flow Parameters:**

Flow rate = 9.3 L/s. ,Flume width=0.3m.

$q = 31$ L/s. m',

Flow depth D = 5.9 cm

Slope = 1/200 = 0.005

Sediment size = 0.15 mm (uniform)

$\rho_s = 2650\ kg/m^3$

$\nu = 1 * 10^{-6}\ m^2/s$

**Calculations:**

$U = Q/A = q/D = 0.525$ m/s

$R_b = A/p = 4.23 * 10^{-2}$ m.(this value may be correct for the side wall effect as outlined in Appendix B, thus the value of $R_b$ shown in Table (A.1) is 5.186 cm after correction also for wide channels $R_b \simeq D$).

$\tau_b = \rho\ g\ R_b S$

$= 2.544\ N/m^2$

$u_* = (\tau_o/\rho)^{1/2} = 0.0504$ m/s

The turbulent constant k may be easily evaluated from velocity profile or otherwise taken arbitrary. $\omega$ is also known for a given sediment grain size.

Thus z thus can be calculated simply as $z = \omega/\beta u_* k$, where $\beta$ is taken as recommended in the range 1.0 to 1.5 according to grain size of sediment.

For this example, $k = 0.258$, $\omega = 0.011$ m/s and $z = 0.68$ (with $\beta = 1.28$).Following the steps outlined before yields:

A- $\tau_b$ needed to set a complete layer in motion is given by

$$\tau_b = 0.46(d)(\rho_s - \rho)g \tan\phi$$

$$= 0.645 \ N/m^2 < \tau_b \text{ available; i.e } N = N_s = 0.46$$

The bed-load rate is given by *Khalil's* equation,with a correction factor to contribute for the rippled bed .This factor is 0.63 as taken by *Khalil*,

$$g_s = 0.63 \ \tau_b/\tan\phi \ [0.96(8.5 \ u_*)(1-0.6(N/N_*))]$$

$$= 3.46 \ \tau_b u_*$$

$$= 0.443 \ N/m.s$$

$$= 45.22 \ g/m'.s.$$

B- The number of moving layers, m equals to

$$m = \tau_b/0.645 = 3.944 \text{ layers}$$

The thickness of the moving layers = 0.15(3.944)

= 0.592 mm

The mid depth of this layer is a= 0.592/2= 0.296 mm.

The bed-load concentration = 1219 g/L which is equal to the suspended load concentration at a = 0.296 mm.

C- From the suspended load concentration equation, calculate $C_{md}$, as

$$C_{md}/1219 = (0.296 \text{ mm}/59.0 - 0.296)^{0.68}$$

$$C_{md} = 1219(0.0274) = 33.4 \text{ g/L}$$

D- From *Brooks'* equations, for a given values of $kU/u_*$ and z, the suspended load rate can be obtained as;

$$g_s/q_v C_{md} = 1.1 \Rightarrow q_s = 31 * 33.4 * 1.1$$

$= 1138.94$ g/m' s

E- The total load rate, $g_T = 45.22 + 1138.94$

$= 1184.16$ g/m'.s

compared to a measured values of 1165.48 g/m'.s.

$\therefore$ % Error = $|1184.16 - 1165.48/1165.48| * 100\%$

$= 1.6$

which is an acceptable deviation for the field measurements.

Thus this systematic approach is a useful tool in calculating the total load rate in open channel flow with sandy bed of uniform grain size, without introducing any sediment load measurement in the field. This method makes use of the continuity of the bed-load and suspended load concentration profiles.

" بسم الله الرحمن الرحيم "

رسالة ماجستير مقدمة من الباحث المهندس رضوان عبدالله الوشاح .

## ملخص الرسالة


**عنوان الرسالة : " اسهام في حمل النقل الرسوبي الكلي "**

ان موضوع النقل الرسوبي في القنوات الرسوبية المكشوفة ، يعتبر موضوعا ذا أهمية كبرى ليس فقط للمهندسين العاملين في مجال المياه والري وانما لجميع المهتمين من جيولوجين وجغرافيين وغيرهم.

وقد استحوذ هذا الموضوع في القرنين الماضيين على اهتمام الكثير من الباحثين الذي استفاضوا فيه بحثا وتحليلا محاولين ايجاد علاقات تمكنهم من حساب حمل النقل الرسوبي. وقد قسم هذا الحمل الى شقين :

١- حمل النقل القاعي وهو تلك المواد الرسوبية التي تنجر أو تتدحرج وتكون على مقربة من القاع.

٢- والنوع الثاني هو حمل النقل المعلق والذى يمثل المواد الرسوبية التي تكون مدفوعة الى أعلى بقوة تعادل وزنها المغمور مما يبقيها في حالة تعليق. ومما تجدر ملاحظته أن هناك تبادل مستمر بين هذين النوعين من الاحمال الرسوبية واللذين يشكلان بمجموعهما حمل النقل الكلي للمواد الرسوبية التي مصدرها القاع المتحرك للقناه.

والغرض الرئيس من هذا البحث هو ايجاد علاقة بين تركيز الحمل الرسوبي القاعي والحمل الرسوبي المعلق ، مما يساعدنا على حساب الحمل الرسوبي الكلي بمعرفة الخصائص الهيدروليكية للجريان في القناة الرسوبية. وقد وجد أن هذه الطريقة تتفق مع التقياسات المخبرية.

وقد تطرق هذا البحث الى اختبار عينات من الدراسات الموجودة في مجال حساب الحمل الرسوبي القاعي والمعلق والكلي . وكذلك تأثير المواد الرسوبية على خصائص الجريان الهيدروليكية.

وتحتوى الرسالة على خمسة فصول بالاضافة الى الملخص والمراجع والملاحق ٠

١- **الفصل الاول :** مقدمـــــة٠

٢- **الفصل الثاني:** ويشمل عرض موجز لاهم النظريات والابحاث والاختبارات الميدانية والمخبرية والمعادلات التجريبية والنظرية الخاصة بهذا الموضوع مع بيان أهم ماتوصل اليه الباحثون من نتائج٠

٣- **الفصل الثالث :** وهو خاص ببرنامج التجارب العملية التي اجريت في هذا البحث ، شامـــلا شرحا وافيا للاجهزة والادوات والمواد المستخدمة في هذا البحث وطريقة العمل.وكلها مدعمة بالرسومات التوضيحية والصور اللازمة٠
وقد بوبت خلاصة ونتائج الاختبارات والبيانات في جداول أدرجت ضمن الملاحق٠

٤- **الفصل الرابع:** ويحوى عرضا شاملا لنتائج التجارب وتحليلها ومناقشتها ضمن المداخـــل النظرية المعروضة ٠ مدعمة بالرسومات البيانية المفصلة للعلاقات بيـــن النتائج المقاسه مخبريا وتلك المحسوبة بطرق مختلفة٠

٥- **الفصل الخامس:** ويحوى ملخص النتائج التي تم التوصل اليها استنادا الى القياسات المخبريـــة ، والتي يمكن تلخيصها بما يلي :-

١- أوضحت التجارب أن النسبة بين حمل النقل القاعي على سطح قاعي متمـوج وبين ذلك الحمل على سطح مستو تساوى ٦٣ر٠

٢- اثبت تحليل النتائج أن أقصى تركيز ممكن للحمل القاعي هو ١٢١٩ غم/لترا وهذا يعادل تركيز ا حجميا يساوى ٤٦٪٠

٣- اوضح التحليل المقارن للنتائج المخبرية أن هذه النتائج تتسق وتنسجـم مع مثيلاتها المحسوبة وفق نظريات وطرق الباحثين الاخرين٠

٤- اثبتت التجارب المخبرية أن ثابت كارمن ( k ) هو ثابت متغير يقل عـــن قيمته المعروفة وهي ٤ر٠ في حالة الجريان الرسوبي٠ كذلك فان هذا الثابـت يتناقص مع ازدياد تركيز الحمل ( C ) للمواد الرسوبية في الجريـــان ويمكن ربط ذلك بالعلاقة٠ $k = 0.3 - 0.025 \, Log \, C$



وهو يصل الى قيمته المعروفة ٤ر٠ في حالة الجريان الصافي الخالي مـــن الرسوبيات٠

٥- بين تحليل النتائج أن قيمة معامل نقل المواد الرسوبية ( $\epsilon_s$ ) تزيد عن قيمة نقل الزخم في القنوات الرسوبية ( $\epsilon_m$ ) ، وفي حالة الرمل المتجانس ذو الحجم الجيبي المتوسط = ١٥ر٠ ملم وجد أن هذه العلاقة هي بالصورة $\epsilon_s = 1.28\ \epsilon_m$

وكذلك الحال بالنسبة للاس z الخاص بتوزيع تركيز الحمل المعلق في الاتجاه الرأسي وجد أن قيمة $z_1$ المقاسه هي أقل من قيمة z المحسوبة ويمكن الربط بينهما بالعلاقة. $z = 1.28\ z_1$

٦- من تحليل نتائج قياسات الحمل المعلق ومقارنتها بتلك المحسوبة وفق نظريات الباحثين الاخرين وجد أن هذه النتائج متفقة مع مثيلاتها.

٧- أثبت تحليل النتائج الخاصة بالحمل الرسوبي الكلي أن القيم المقاسه تزيد عن ثلاثة امثال القيم المعطاه ببعض الطرق المباشرة امثال Yang و Graf. وان هذه النتائج في نفس الوقت تتفق جيدا مع تلك المحسوبه بطريقة ( Einstein) المعروفة.

٨- واذا ما رُسمت النتائج المخبرية للحمل الكلي على نمط العلاقة المعطاه من قبل ( Graf et al. ) فستكون على الصورة:

$$\Phi_A = 32.9\ \Psi_A^{-2.52}$$

٩- وجد من تحليل النتائج المخبرية ومقارنة تركيز الحمل القاعي مع امتداد تركيز الحمل المعلق أن هذين التركيزين يتساويان عند عمق يساوي نصف سماكة الطبقة القاعية المتحركة.

١٠- استنادا الى الاستنتاج الوارد في البند (٩) ، فيمكن حساب حمل النقل الرسوبي الكلي بمعرفة المتغيرات الهيدروليكية وخواص المواد الرسوبية بطريقة نظامية ، تعطي نتائج متفقة مع القياسات دون الحاجة الى اجراء قياسات ميدانية لاى من حملي النقل القاعي أو المعلق.

١١- لوحظ من استعرض النتائج الحالية أن الحمل المعلق يساهم بالمتوسط بحوالي ٩٠٪ من حمل النقل الكلي. وهذا ينطبق على الرمل المتجانس ذو الحجم الجيبي المتوسط (١٥ر٠) ملم.