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1. THE DEVELOPMENT OF CONCENTRATION PROFILES IN A STEADY, UNIFORM FLOW WITHOUT INITIAL SEDIMENT LOAD

by

L.C. van Rijn*)

1. Introduction

A fundamental problem in the field of sediment transport is the description of the processes which govern the pick-up of sediment particles from a loose bed and the development of concentration profiles in a steady, uniform flow, which is initially free of sediment as shown in Figure 1.
Usually, these erosion processes are described by a convection-diffusion type equation. In 1970, Hjelmfelt and Lenau presented an analytical solution assuming a parabolic distribution for the diffusion coefficient and a constant flow velocity over the flow depth while the longitudinal diffusion was neglected (Hjelmfelt et al, 1970).
In this paper a numerical solution using a finite element method is given, while also a function to compute the reference concentration at the bed-boundary is proposed. The numerical method is compared with the analytical solution of Hjelmfelt and Lenau. Finally, the mathematical model is verified using experimental results.

2. Equations

The transport processes which govern the development of concentration profiles in non-equilibrium conditions can be described by (two-dimensional, steady and uniform flow):

$$\frac{\delta(uc)}{\delta x} - \frac{\delta(w_s c)}{\delta z} - \frac{\delta}{\delta x}\left(\varepsilon_{s,x}\frac{\delta c}{\delta x}\right) - \frac{\delta}{\delta z}\left(\varepsilon_{s,z}\frac{\delta c}{\delta z}\right) = 0 \qquad (1)$$

in which: c = concentration, u = flow velocity, w_s = fall velocity of sediment particles, $\varepsilon_{s,x}$ and $\varepsilon_{s,z}$ = diffusion coefficient, x = longitudinal coordinate, z = vertical coordinate.

Assuming a scalar diffusion coefficient, a constant particle fall velocity and neglecting the longitudinal diffusion, equation (1) can be simplified to:

$$u\frac{\delta c}{\delta z} - w_s\frac{\delta c}{\delta z} - \varepsilon_s\frac{\delta^2 c}{\delta z^2} = 0 \qquad (2)$$

Diffusion coefficient

The diffusion coefficient is described by a parabolic-constant distribution:

$$\varepsilon_s = 0.25\,\beta\,K\,u_*\,h \quad , \quad \text{for } \frac{z}{h} \geq 0.5 \qquad (3)$$

$$\varepsilon_s = \beta\,K\,u_*\,z\left(1-\frac{z}{h}\right), \quad \text{for } \frac{z}{h} < 0.5 \qquad (4)$$

$$\beta = 1 + 2\left(\frac{w_s}{u_*}\right)^2 \qquad (5)$$

in which: β = ratio of sediment and momentum diffusion coefficient, K = constant of Von Karman, u_* = bed-shear velocity, h = flow depth.

The β-factor was evaluated from data given by Coleman (1970). According to equation (5) the β-factor is dependent on the ratio of the fall velocity and the bed-shear velocity and is always larger than 1. The influence of the particles on the turbulence (dampening) has not been taken into account.

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Flow velocity

The vertical distribution of the flow velocity is described by a logarithmic function (Nikuradse) for rough flow conditions:

$$\frac{u}{u_*} = \frac{1}{K} \ln \left(\frac{z}{z_o}\right) \qquad (6)$$

in which: $z_o = 0.033\ k_s$ = zero-velocity level, k_s = equivalent roughness of Nikuradse.

3. Boundary conditions

Water surface

At the water surface it is assumed that no sediment particles can pass the surface, which can be formulated as:

$$\left(w_s c + \varepsilon_s \frac{\delta c}{\delta z}\right)_{z=h} = 0 \qquad (7)$$

Sediment bed

At the bed boundary a reference concentration is specified:

$$c_{z=a} = c_a \qquad (8)$$

At the Delft Hydraulics Laboratory, a theoretical study (Rijn, 1981) was initiated to develop a function which relates the reference concentration to local flow and sediment parameters. For that purpose, firstly the motion of bed-load particles was studied using the equations of motion for a solitary particle which can be described by (see also Figure 2):

$$m\ddot{x} - L\left(\frac{\dot{z}}{v_r}\right) - D\left(\frac{u - \dot{x}}{v_r}\right) = 0 \qquad (9)$$

$$m\ddot{z} - L\left(\frac{u - \dot{x}}{v_r}\right) + D\left(\frac{\dot{z}}{v_r}\right) + G = 0 \qquad (10)$$

in which: m = particle and added fluid mass, L = lift force, D = drag force, G = submerged particle weight, u = local flow velocity according to equation (6), $\dot{x}$ and $\dot{z}$ = longitudinal and vertical particle velocity, $\ddot{x}$ and $\ddot{z}$ = longitudinal and vertical particle acceleration, $v_r = \{(u - \dot{x})^2 + (\dot{z})^2\}^{0.5}$ = particle velocity relative to the flow.

The drag force is described by:

$$D = \tfrac{1}{2}\,\alpha_D\,\rho\,A\,v_r^2 \qquad (11)$$

in which: α_D = drag coefficient, ρ = density of water, $A = \frac{1}{4}\pi d^2$ = cross-section of particle, d = particle diameter. The drag coefficient is a function of the particle Reynolds' number. The values given by Morsi and Alexander (1972) were used.

The lift force in a viscous flow has been given by Saffman (1968):

$$L = \alpha_L'\,\rho\,\nu^{0.5}\,d^2\,v_r\left(\frac{\delta u}{\delta z}\right)^{0.5} \qquad (12)$$

in which: α_L = 1.6 = lift coefficient, ν = viscosity coefficient, $\delta u/\delta z$ = velocity gradient. For a turbulent flow, the lift coefficient is unknown and has, therefore, been used as a calibration parameter.

Equations (9) and (10) can be transformed to a system of ordinary simultaneous differential equations of the first order. Assuming an initial particle position and initial particle velocities (see Figure 2), both equations can be solved numerically.
To calibrate the unknown lift coefficient, some experiments of Fernandez Luque were used. He carried out laboratory tests on bed-load particles using film techniques to determine the particle trajectories and velocities. Figure 3a shows some trajectories for a sand particle of 1800 μm. Figure 3b and 3c show computed trajectories for varying lift coefficients (α_L) and roughness heights (k_s). The best agreement can be observed for $\alpha_L = 20$ and $k_s/d = 1$. It must be stressed that the calibrated lift coefficient reflects all influences which are not taken into account (turbulent fluctuations, additional pressure forces in the proximity of the wall).

The calibrated equations (9) and (10) were used to determine functions for the saltation height (δ_b) and transport velocity (u_b) of the particles:

$$\delta_b = 0.3\ d\ D_*^{0.7} \left[\frac{\theta' - \theta_{cr}}{\theta_{cr}}\right]^{0.5} \qquad (13)$$

$$u_b = u_*' \left[9 + 2.6 \log(D_*) - 8 \left(\frac{\theta_{cr}}{\theta'}\right)^{0.5}\right] \qquad (14)$$

in which: $D_* = d\left[\frac{\rho_s - \rho}{\rho}\frac{g}{\nu^2}\right]^{1/3}$, ρ_s = density of sediment, $\theta' = \frac{\rho(u_*')^2}{(\rho_s - \rho)gd}$, u_*' - bed-shear velocity related to grain roughness, θ_{cr} = critical mobility parameter according to Shields.

Using equations (13) and (14) and measured bed-load rates (s_b), a bed-load concentration was defined as:

$$c_b = \frac{s_b}{u_b\ \delta_b} \qquad (15)$$

The computed bed-load concentrations according to equation (15), which are an estimate for the sediment concentration in the saltation layer, were related to flow and sediment parameters resulting in:

$$c_b = \frac{0.12}{D_*}\left[\frac{\theta' - \theta_{cr}}{\theta_{cr}}\right] \qquad (16)$$

To determine a <u>reference concentration</u> (c_a) at a specific reference level (a), the sediment transport below the reference level was assumed to be equal to the bed-load transport resulting in:

$$s_b = c_b\ u_b\ \delta_b = c_a\ \bar{u}_a\ a \qquad (17)$$

Assuming $\bar{u}_a = \alpha\ u_b$ ($\bar{u}_a$ = average flow velocity below reference level) and substituting the equations (13), (14) and (16) in equation (17), the reference concentration can be expressed as:

$$c_a = \frac{0.036}{\alpha}\ \frac{d}{a}\ D_*^{-0.3}\left[\frac{\theta' - \theta_{cr}}{\theta_{cr}}\right]^{1.5} \qquad (18)$$

Using equation (18) as a reference concentration for concentration profiles in equilibrium conditions and fitting with measured concentration profiles, the α-factor was found to be 2.3 (Rijn, 1981).

4. Solution method

To solve equation (2), a finite element method based on weighted residuals according to the Galerkin method is used.
Firstly, the continuous solution (two-dimensional) domain is divided into a set of quadrangular elements. The vertical dimensions of the elements decrease towards the bed to provide a greater resolution in the zone where large velocity and concentration gradients exist. Between the nodes of the elements the unknown variable is represented by linear interpolation functions. Then, for each element the coefficients corresponding to the unknown variable at each node are determined. Finally, the (tri-diagonal) coefficient matrix for the complete solution domain is developed, from which the coefficients can be solved.
To evaluate the accuracy of the solution method, a numerical solution of the problem as shown in Figure 1 was compared with an analytical solution according to Hjelmfelt and Lenau (1970). They presented an analytical solution of equation (2) assuming a parabolic diffusion coefficient over the flow depth (equation 4) and a constant flow velocity equal to the depth-averaged flow velocity. Both the analytical and the numerical solution are shown in Figure 4. The inaccuracy incraeases towards the water surface because the relative distance between the nodes increases in vertical direction. The maximum error is about 5% which is quite acceptable for sediment transport problems.

5. Verification

To verify the mathematical model, a laboratory experiment according to Figure 1 was carried out. The flow depth was 0.25 m, while the average flow velocity was 0.67 m/s. The bed material had a d_{50} = 230 μm. Pitot-tubes were used to determine the vertical distribution of the flow velocity. Water samples were collected simultaneously at four locations downstream of the rigid bed to determine the spatial distribution of the sediment concentrations. At each location five water samples were taken using the siphon method (5 intake tubes). The concentrations were determined as the ratio of the dry weight of the sediment particles and the volume of the water sample.
Although the measuring period was made as short as possible, a small scour hole was formed directly downstream of the rigid bed, thereby disturbing the flow conditions.
Figure 5 shows measured and computed concentrations as a function of longitudinal and vertical distance. The reference level was assumed at 0.010 m, which was the average height of the bed forms. Using equation (18), a correction coefficient of about 0.5 was necessary to reproduce the measured bed concentrations at 0.015 m above the bed. The applied distribution for the diffusion coefficient was not modified.
The agreement between measured and computed concentrations in the developing zone is rather good, particularly at 0.015, 0.025 and 0.05 m above the bed. Only at a height of 0.1 m the development of the computed concentrations proceeds somewhat too rapidly.
At the most downstream meausuring location, where an equilibrium concentration profile is approached, the agreement between computed and measured concentrations is extremely good, which is also a support for the validity of the applied diffusion coefficient (equation (3), (4) and (5)).

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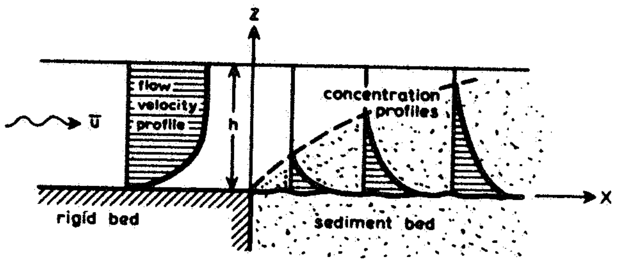


FIG. 1 FLOW, INITIALLY FREE OF SEDIMENT, OVER A SEDIMENT BED

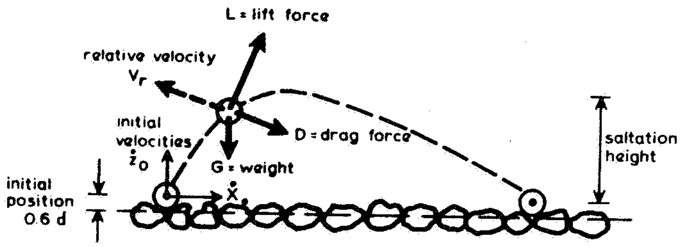


FIG. 2 TRAJECTORY OF A SALTATING PARTICLE

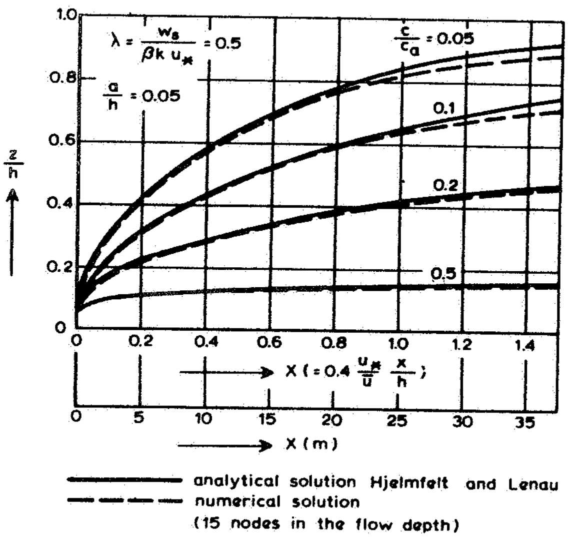


FIG. 4 CONCENTRATION DISTRIBUTIONS

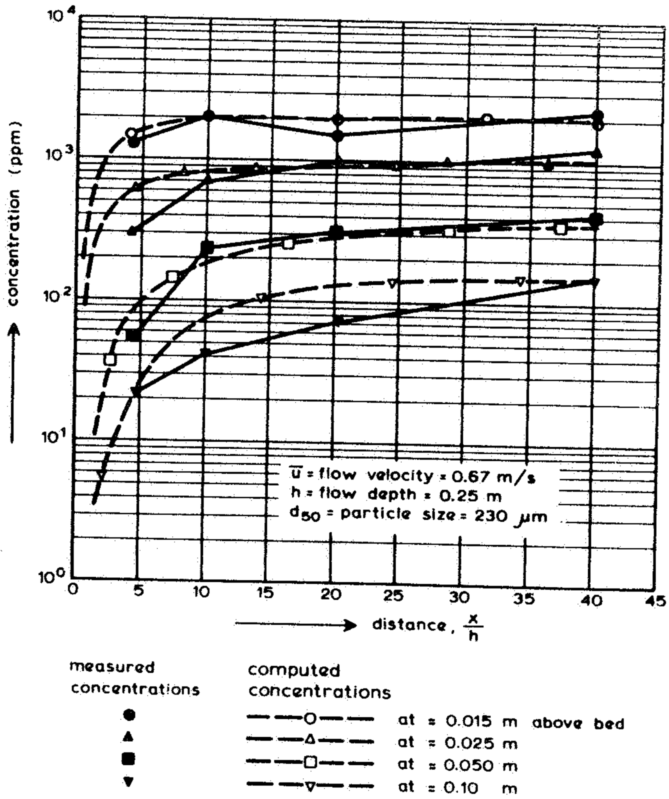
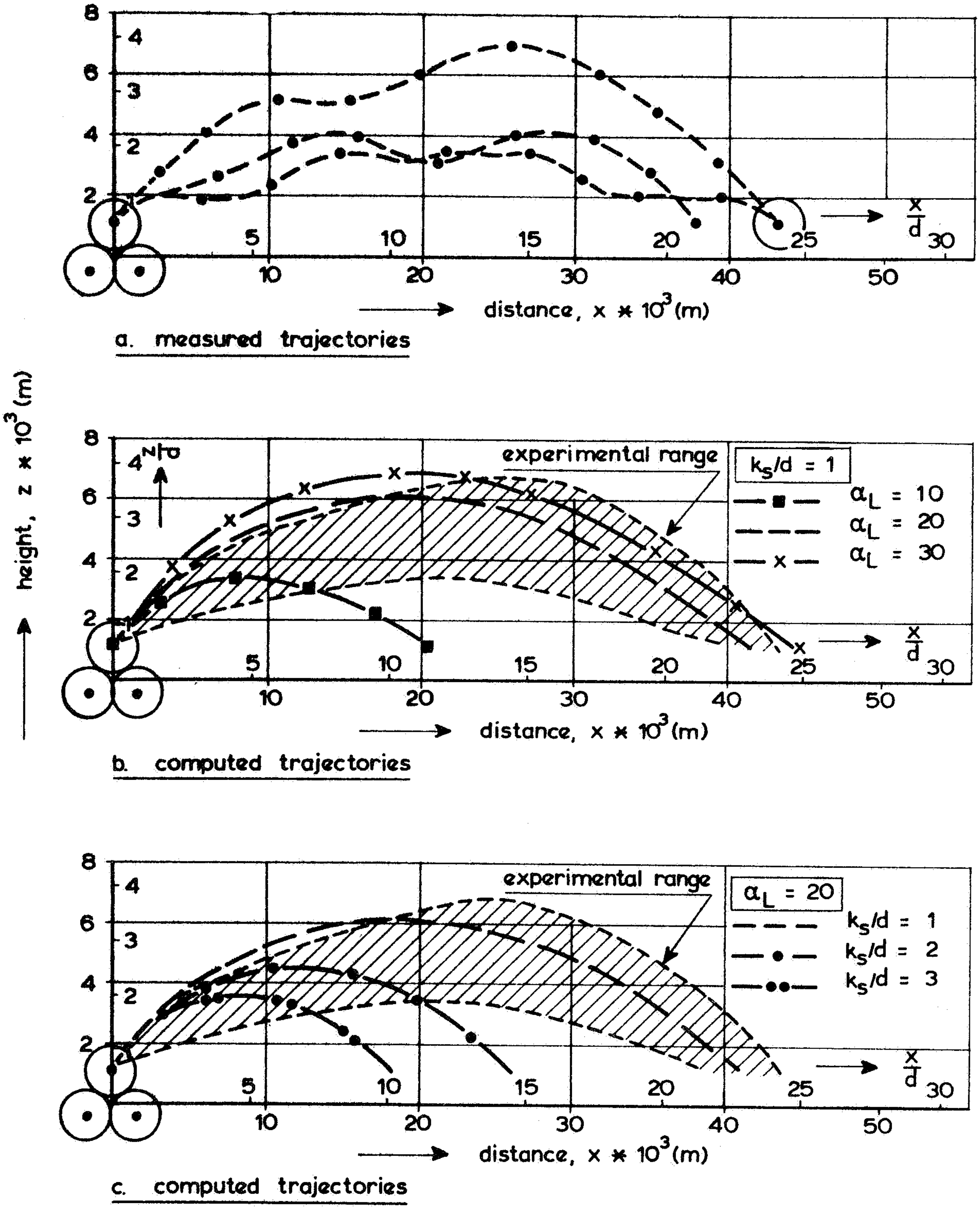


FIG. 5 LONGITUDINAL DISTRIBUTION OF SEDIMENT CONCENTRATIONS



d = 1800 (μm)

u_* = 0.04 (m/s)

$\dot{x}(o)$ = 2 u_* (m/s)

$\dot{z}(o)$ = 2 u_* (m/s)

FIG. 3 MEASURED AND COMPUTED PARTICLE TRAJECTORIES (LARGE PARTICLE)

2. EXPERIENCE WITH STRAIGHT FLUMES FOR MOVABLE BED EXPERIMENTS

by

L.C. van Rijn* and G.J. Klaassen*

1. Introduction

Flume studies may increase the understanding of the interrelationship between the bed material and the sediment transport on the one hand and the hydraulic parameters on the other hand. During a flume experiment the conditions can be controlled and the relevant parameters can be measured with sufficient accuracy. Experiments in flumes, however, are influenced by the presence of the side-walls, which dissipate a part of the available flow energy. To reduce this effect, it may seem attractive to do experiments with relatively large width-depth ratios only. Under these conditions, however, three-dimensional bed form patterns and even alternate bars may occur. Although these phenomena are also present in nature, the ratio of the characteristic length of these macro-scale phenomena and the length of the average bed forms is much more unfavourable in flumes and therefore, the influence of the three-dimensional phenomena should be limited.

In this paper some of the experience gained at the Delft Hydraulics Laboratory is discussed concentrating mostly on the influence of the flume width on the bed forms. The implications of the observed phenomena are reviewed together with some preliminary results of presently on-going research.

2. Types of sediment flumes

Usually two types of flumes are being used for experiments with movable beds, viz. a sand-recirculation system and a sand-feed system:

Sand-recirculation system	Sand-feed system
the sand particles are returned continuously (Figure 1)	the sand particles are separated from the flow and returned by means of some mechanical method (Figure 2)
advantages	advantages
* the system tends to act like an infinite flow (natural channels)	* possibility to impose a specific rate of sediment transport,
* applicable for fine sediments	* no limitations on flume dimensions
* relatively easy to operate	* accurate determination of sediment transport rate
* equilibrium conditions are established rapidly, if pre-set slope is correct	
imposed variables	imposed variables
* discharge	* discharge
* slope (tilting flume)	* sediment transport
adjusted variables	adjusted variables
* flow depth	* flow depth
* sediment transport	* slope

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According to Guy et al (1967) the two systems do not show significant differences if the imposed boundary conditions are similar.

For experiments with a movable bed, presently two flumes are being used at the Delft Hydraulics Laboratory.
The smallest flume (test length = 10 m, width = 0.5 m and depth = 0.7 m, maximum discharge = 0.25 m^3/s) is mainly used for research in the field of suspended sediment transport. The largest flume (measuring length = 30 m, total length = 100 m, width variable between 0.3 and 1.5 m, depth = 1.0 m, maximum discharge = 0.8 m^3/s) has been built especially for fundamental research into bed-load transport.
The main features of the large flume are:

- the sediment transport is measured and regulated at the beginning and end of the flume by means of hydrocyclones enabling the determination of the submerged weight of the bed load (no influence of voids ratio), (Figure 3),
- longitudinal records of the sand-bed in three profiles (middle profile and two profiles on each side at one-sixth of the width) are measured by means of profile indicators, mounted on a measuring carriage; the measurements are made and stored automatically on pre-set hours,
- the discharge in the flume can vary in time according to a pre-set function by means of a computer programme.

Also the other relevant hydraulic and sediment parameters are sampled automatically. All collected data are stored on tape in a mini-computer, which also checks the collected data after each measurement. Next, the tapes are processed on a large computer system.
Originally, the large flume was built as a sand-feed system, but recently a slope-control system was installed to obtain the advantages of the sand-recirculation system. The slope-control system consists of regulating the height of the tailgate of the flume until the actual slope of the flume is equal to a pre-set value within narrow limits, while the sand particles are recirculated directly (see Figure 3).

3. Influence of the flume width

One of the problems in interpreting flume data is a correct elimination of the side-wall effects, because the side-walls influence both the bed-shear stress and the dimensions of the bed forms and hence the sediment transport.

Bed-shear stress

For large values of the width-depth ratio the influence of the side-walls can be neglected and the bed-shear stress can be assumed to be equal to the value ρghi (ρ = density of fluid, g = acceleration of gravity, h = flow depth, i = slope).
Some insight in the value of the bed-shear stress for small width-depth ratios can be obtained from the experiments of Knight and Macdonald (1979), who determined the bed-shear stresses from special Pitot-tube measurements for different bed roughnesses. Figure 4 shows values of $\tau_{bed}/\rho ghi$ (τ_{bed} = bed-shear stress) as a function of the width-depth ratio and the bed roughness.
Assuming a rough bed for experiments with a mobile bed, a width-depth ratio larger than 3 should be taken to reduce the side-wall influence to less than about 20%. This is in accordance with the observations of Williams (1970), that at the same flow conditions the energy gradient remains nearly constant for a width-depth ratio larger than 3 indicating a vanishing influence of the side walls (Figure 5).
Initially the design of the large flume in the Delft Hydraulics Laboratory was based on a minimum width-depth ratio of 4 (Struiksma et al, 1971); lateron also tests with smaller ratios were carried out.

Bed-form dimensions

Experiments have shown that the flume width also influences the dimensions of the bed forms. Crickmore (1970) and Williams (1970) reported an increase of both the length and height of the bed forms for increasing values of the width-depth ratio at the same flow conditions (discharge per unit width).
Statistical analysis of the sand waves measured in the large flume of the Delft Hydraulics Laboratory (Bogirski, 1977) and some tests with reduced widths in a flume of the Delft University of Technology (Vermaas, 1980) show similar results. For each experiment the bed was sounded in 3 longitudinal profiles at regular time intervals of 2 hours (to ensure statistical independent measurements). In all about 400 bed forms for each profile were analyzed. The data were used to compute the average bed slope of each profile, the bed form lengths defined as the distance between two successive zero-upcrossings with the average bed slope, and the bed form heights defined as the distance between the highest and lowest point between two successive zero-upcrossings.
Figure 6 shows the average bed form length and height (in the middle profile) as a function of the width-depth ratio for the same flow depth and discharge per unit width.
Clearly, an increase of the bed form dimensions for increasing values of the width-depth ratio can be observed, while both the value of the energy-gradient and the Chézy-coefficient show a decrease. Figure 7 represents the probability density function of the bed form length which shows a shift to the larger bed-forms for larger width-depth ratios, while also the variation in the bed form length shows an increase.

4. Three-dimensional effects

Three-dimensional bed forms have been reported by Guy et al (1966), who carried out a series of experiments in a 8-foot wide flume in the years 1956-1961. They described the development of alternate bars in some runs which caused the flow to meander. Also Williams (1970) reported three-dimensionals bed forms for a widht-depth ratio larger than about 3.
Statistical analysis of sand waves measured in three longitudinal profiles in the large flume of the Delft Hydraulics Laboratory also show the existence of predominantly three-dimensional bed forms for large width-depth ratios. Figure 8 represents the ratio of the bed form dimensions in the side profiles and the middle profile as a function of the width-depth ratio (same run as in Figure 6). As can be observed, the length and height of the bed forms in the side profiles are considerably larger than the values in the middle profile. Particularly, the ratio of the bed-form length in the side profiles and the middle profile seems to increase with the width-depth ratio.
It should be stressed that the present results relate to average values obtained from 15 to 20 independent soundings of the bed profiles. Individual measurements show considerable scatter in the observed energy gradients, Chézy-coefficients and bed form dimensions. This may, at least partly, be caused by the presence of three-dimensional phenomena, which may migrate in downward direction thereby causing a continuous change in the flow conditions in the flume.

Present research

At present, research is carried out at the Delft Hydraulics Laboratory into these three-dimensional phenomena. The purpose of this research is to establish criterions for the selection of an optimal flume width for particular tests conditions, taking into account the effect of both the wall roughness and the occurence of three-dimensional phenomena. Within this frame-work, methods are studied to eliminate the effect of the latter phenomena from the bed-level recordings.

Some preliminary results of the application of a high-pass digital filter to the recordings is presented in Figure 9. This figure is related to a test with a width-depth ratio of about 15. The mean waterdepth is about 0.10 m. In the figure the original signal, the removed trend and the resulting signal are shown for the middle profile and the side-profiles. The filter characteristics are such that all wave-lengths larger than 3 m have been removed from the signal. From the results it can be concluded that for this particular test a definite alternate bar pattern is present. After filtering, the ratio of the dune-length of the side and the middle profile decreased from 1.56 to 1.04.

The research programme will be continued (i) by selecting the most appropriate filter, (ii) by applying the filtering method to other tests with smaller width-depth ratios, (iii) by estimating the effects of the three-dimensional bed-level pattern on hydraulic roughness and sediment transport.

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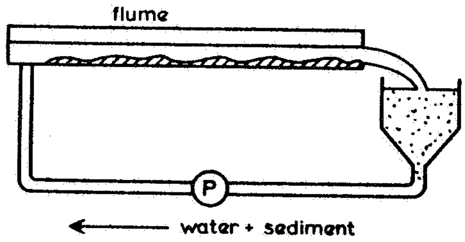


FIG. 1 SAND-RECIRCULATION SYSTEM

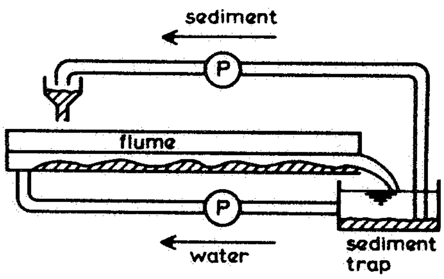


FIG. 2 SAND-FEED SYSTEM

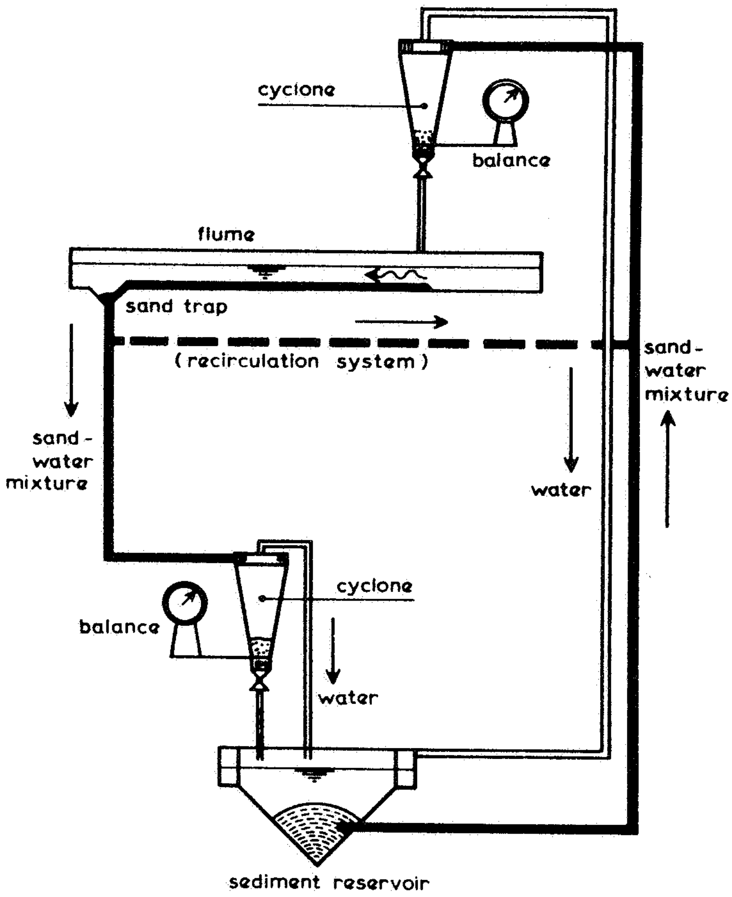


FIG. 3 SAND-FEED SYSTEM OF THE LARGE FLUME AT THE DELFT HYDRAULICS LABORATORY

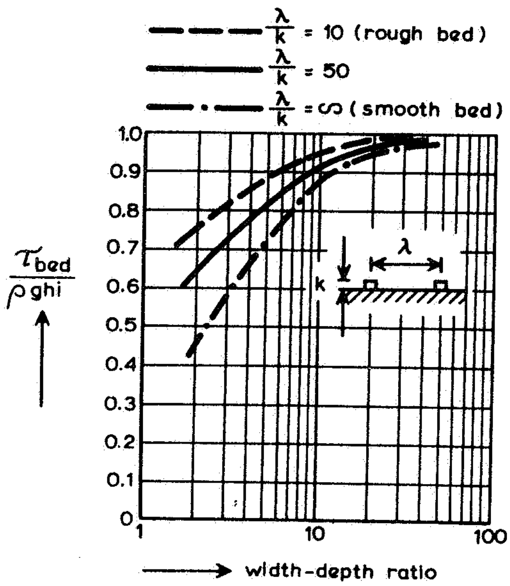


FIG. 4 BED-SHEAR STRESS AS A FUNCTION OF FLUME WIDTH AND BED-ROUGHNESS (KNIGHT ET AL., 1979)

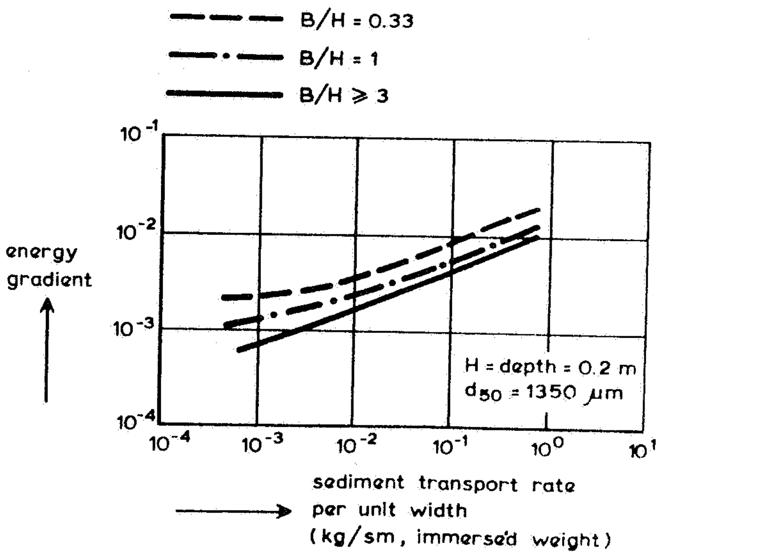
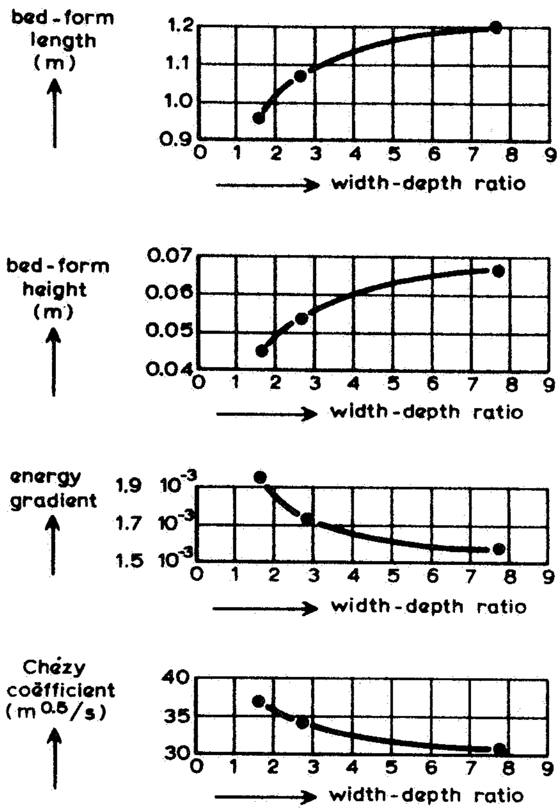
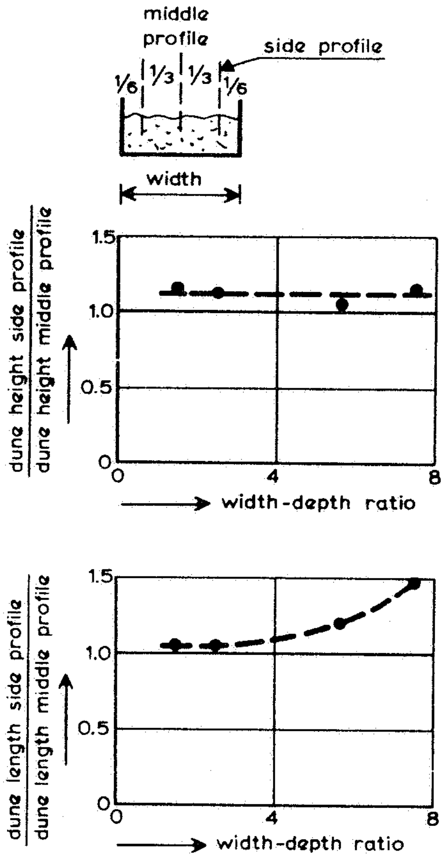


FIG. 5 ENERGY GRADIENT AS A FUNCTION OF THE FLUME WIDTH AND SEDIMENT TRANSPORT RATE (WILLIAMS, 1970)



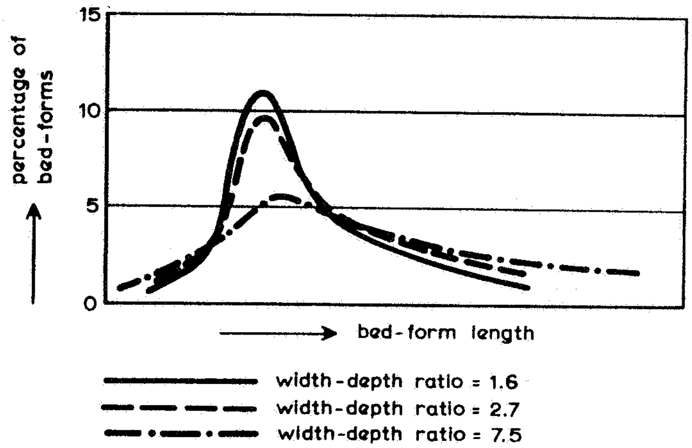
q = water discharge = 0.1 m^2/s
s = sand discharge = 0.0122 10^{-3} m^2/s
H = flow depth = 0.19 m
D_m = sand diameter = 770 μm

FIG. 6 BED-FORM DIMENSIONS IN MIDDLE PROFILE, ENERGY GRADIENT AND CHÉZY - COËFFICIENT AS A FUNCTION OF FLUME WIDTH



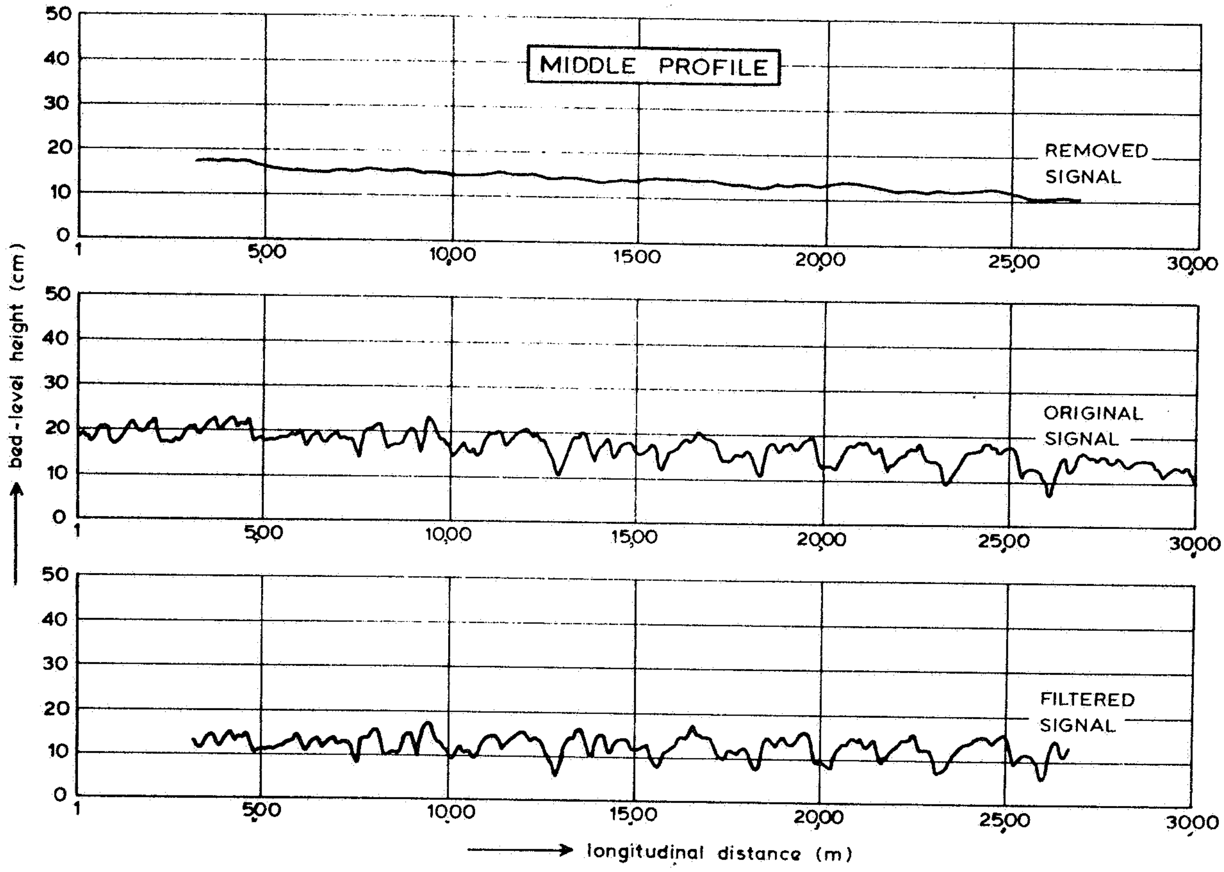
q = 0.1 m^2/s
s = 0.0122 10^{-3} m^2/s
H = 0.19 m
D_m = 770 μm

FIG. 8 RATIOS OF BED FORM LENGTH AND HEIGHT IN MIDDLE AND SIDE PROFILES AS A FUNCTION OF FLUME WIDTH



q = 0.1 m^2/s
s = 0.0122 10^{-3} m^2/s
H = 0.19 m

FIG. 7 PROBABILITY DENSITY FUNCTION OF THE BED-FORM LENGTH IN MIDDLE PROFILE



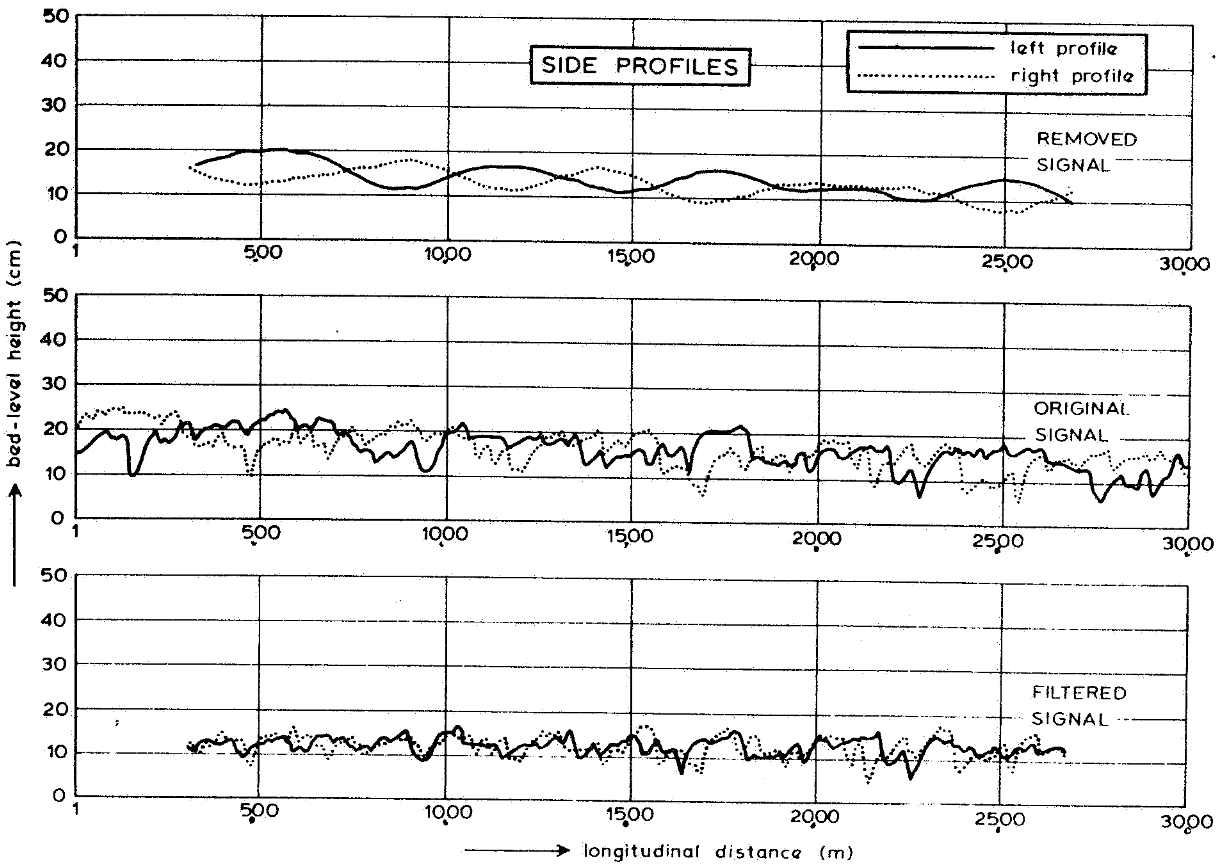


FIG. 9 BED LEVEL RECORDINGS