Use of models for river problems

Prepared for the
International Hydrological Programme
within Project M-3-5(a) (IHP-IV)

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UNESCO Publishing

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Published in 1993 by the United Nations
Educational, Scientific and Cultural Organization
7, place de Fontenoy, 75700 Paris

Printed by:
Imprimerie de la Manutention, Mayenne

ISBN 92-3-102861-8

Printed in France

Studies and reports in hydrology 51

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Preface

Although the total amount of water on earth is generally assumed to have remained virtually constant, the rapid growth of population, together with the extension of irrigated agriculture and industrial development, are putting stress on the quantity and quality aspects of natural systems. Because of the increasing problems, society has begun to realize that it can no longer follow a 'use and discard' philosophy — either with water resources or any other natural resources. As a result, the need for a consistent policy of rational management of water resources has become evident.

Rational water management should be founded upon a thorough understanding of water availability and movement. Thus, as a contribution to the solution of the world's water problems, UNESCO, in 1965, began the first world-wide programme of studies of the hydrological cycle — the International Hydrological Decade (IHD). The research programme was complemented by a major effort in the field of hydrological education and training. The activities undertaken during the Decade proved to be of great interest and value to Member States. By the end of that period, a majority of UNESCO's Member States had formed IHD National Committees to carry out relevant national activities and to participate in regional and international co-operation within the IHD programme. The knowledge of the world's water resources had substantially improved. Hydrology became widely recognized as an independent professional option and facilities for the training of hydrologists had been developed.

Conscious of the need to expand upon the efforts initiated during the International Hydrological Decade, and following the recommendations of Member States, UNESCO launched a new long-term intergovernmental programme in 1975: the International Hydrological Programme (IHP).

Although the IHP is basically a scientific and educational programme, UNESCO has been aware from the beginning of a need to direct its activities toward the practical solutions of the world's very real water resource problems. Accordingly, and in line with the recommendations of the 1977 United Nations Water Conference, the objectives of the International Hydrological Programme have been gradually expanded in order to cover not only hydrological processes considered in interrelationship with the environment and human activities, but also the scientific aspects of multi-purpose utilization and conservation of water resources to meet the needs of economic and social development. Thus, while maintaining IHP's scientific concept, the objectives have shifted perceptibly towards a multidisciplinary approach to the assessment, planning, and rational management of water resources.

As part of UNESCO's contribution to achieving the objectives of the IHP, two publication series are issued: 'Studies and reports in hydrology', and 'Technical papers in hydrology'. In addition to these publications, and in order to expedite exchange of information in the areas in which it is most needed, works of a preliminary nature are issued in the form of technical documents.

The purpose of the continuing series 'Studies and reports in hydrology', to which this volume belongs, is to present data collected and the main results of hydrological studies, as well as to provide information on hydrological research techniques. The proceedings of symposia are also sometimes included. It is hoped that these volumes will furnish material of both practical and theoretical interest to water resources scientists and also to those involved in water resources assessment and planning for rational water resources management.

Acknowledgements

The author would like to express his gratitude to Mr. J.J. van der Zwaard, Director of the Rivers, Navigation and Structures Division of Delft Hydraulics for his assistance in the production of this report. The co-workers of the Division who actually worked on its preparation are acknowledged likewise.

Contents

Main symbols

Symbol	*Description*	*Dimension*
a	water depth	[L]
A	cross-sectional area	[L^2]
B	width	[L]
c	celerity	[LT^{-1}]
C	Chézy coefficient	[$L^{\frac{1}{2}}T^{-1}$]
D	grain size	[L]
Fr	Froude number = $u/\sqrt{ga}$	–
g	acceleration of gravity	[LT^{-2}]
h	water level	[L]
H	energy head	[L]
	dune height	[L]
i	slope	–
K	diffusion/dispersion coefficient	[L^2T^{-1}]
n	exponent of transport (power) law	–
n_x	scale of $x = x_p/x_m$	–
q	discharge per unit width	[L^2T^{-1}]
Q	discharge	[L^3T^{-1}]
r	distortion = n_L/n_a	–
R	radius of curvature	[L]
Re	Reynolds number	–
s	sediment transport per unit width (bulk volume)	[L^2T^{-1}]
S	sediment transport over the entire width (bulk volume)	[L^3T^{-1}]
t	time	[T]
u	flow velocity (x-direction)	[LT^{-1}]
v	flow velocity (y-direction)	[LT^{-1}]
w	flow velocity (z-direction)	[LT^{-1}]
W_s	fall velocity	[LT^{-1}]
x	ordinate in flow direction	[L]
y	horizontal ordinate perpendicular to main flow direction	[L]
z	vertical coordinate	[L]

Symbol	*Description*	*Dimension*
$z_{(b)}$	bed level	[L]
Z	$= W_s/\kappa u_*$	–
δ	thickness viscous sublayer	[L]
Δ	relative density $= (\rho_s - \rho)/\rho$	–
ε	eddy viscosity	[L^2T^{-1}]
κ	von Kármán constant	–
Λ	$= x.i/a$	–
ρ	density of water	[ML^{-3}]
ρ_s	density of sediment	[ML^{-3}]
σ	Courant number $= c\Delta t/\Delta x$	–
ϕ	relative celerity (= c/u) concentration	–
ϕ_s	transport parameter $= s/\left\{D^{\frac{3}{2}}\sqrt{g\Delta}\right\}$	–
ψ	a^{-1} ds/du	–

1. Introduction

1.1 General

Phase IV (1990–1995) of the International Hydrological Programme of UNESCO contains the Project M-3-5 entitled: 'Promotion of environmentally sound water resources management'. Sub-project M-3-5(a) concerns 'Development of manual and guide related to promotion of environmentally sound river management', and the present publication represents one of its planned products: '*A state-of-the-art and report on the use of scale models for studying rivers, including movable bed models and a guide for calculating two- and three-dimensional open channel flow with special regard to sediment and pollution transport and ice phenomena*' (IHP-IV plan).

The development and use of river models has been extensive during the course of the last three decades, the period the writer is able to review because of personal involvement. It is not possible to give full account of all past developments and those taking place at present (1992) worldwide. The limits given to the size of this publication would restrict its coverage to a bibliography alone. Consequently the writer has been obliged to make a selection and express a somewhat personal view. His objective is a well-balanced approach to the use of river models in order to obtain predictions of sufficient accuracy for particular cases.

1.2 On the use of models

The general philosophy on the use of models is the same for scale models and numerical models (Fig. 1.1).

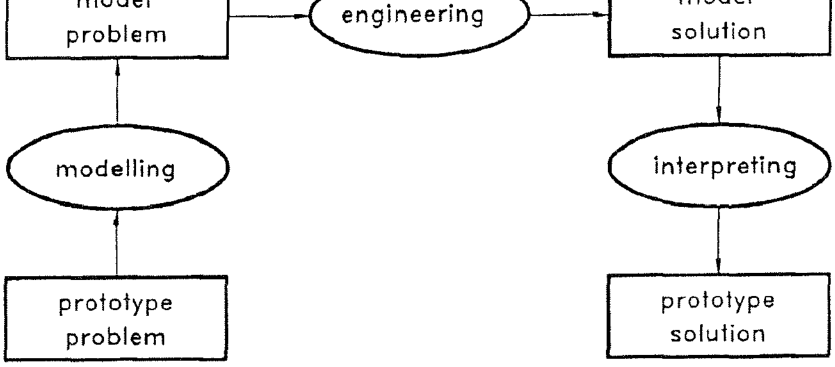


Fig. 1.1 Use of models as a detour

The real problem for '*the prototype*' cannot be solved directly; a *detour* is made via the model. This contains three related steps. The schematisation phase separates the model problem from the prototype problem. The model solution is obtained in the engineering phase. Interpretation of the model results enable the prototype solution to be obtained. Some remarks are needed in this respect:

- *Schematisation* involves the relevant physical processes, the geometry involved and also boundary conditions to be applied.
- *Engineering phase*: the model is no more than a tool. It is the skill of the user that determines the quality of the final results.
- *Interpretation* of the model results leads to the anticipated results for the prototype. The quality of the schematisation determines the accuracy of the final results.

In order to check the quality of the model *calibration* is required. By means of field data, coefficients are adjusted to match the model solution with the prototype solution measured in the field. *Verification* of the model is also strongly advised. This implies comparing model results with prototype data that have not yet been used. In the verification no additional adjustment of coefficients should take place. In this report both scale models and numerical models are discussed.

- *Scale models* are copies of the prototype in a hydraulics laboratory where the model results are obtained by measurement. Since complete similarity cannot usually be obtained (a full-scale model being out of the question) *scale effects* are present to a certain extent.
- *Numerical models* give computer simulations of the relevant physical phenomena of the prototype. The physical phenomena are usually described by (partial) differential equations. *Numerical errors* may be present particularly since the equations can only be solved via discretisation.

There is apparently a large similarity in design and use of the two types of models; the underlying philosophy is basically the same.

Remarks

(i) In some cases the boundary conditions for a scale model of a (tidal) river are supplied on-line via a numerical model. Such a combination (a *hybrid model*) is not considered in this report.

(ii) In an ideal situation both types of model are based on partial differential equations of the relevant fluvial processes. If these equations are not available, a scale model can be designed in principle via *dimensional analysis*. However, in such a case no information is obtained beforehand on possible scale effects (see Section 3.1).

1.3 Outline of the report

In Chapter 2 of this report an overview is given on fluvial processes involved in the modelling discussed. Chapter 3 is concerned with scale models, whilst Chapter 4 deals with numerical models. The main problems treated are, firstly, modelling of the water movement, followed by sediment transport and morphology. Attention is also given to the dispersion of dissolved matter and problems associated with ice.

The 'tools' for scale models and numerical models are given only little attention. For scale models the tools are instruments and control systems. For numerical models numerical schemes are involved. A good introduction can be found in Vreugdenhil (1989).

In Chapter 5 guidance is given on the selection of the type of model for river problems, though this has to be of a general nature.

2. On fluvial processes

2.1 Introduction

For the construction of a model for a particular river problem it is necessary to have a description in mathematical terms of the fluvial processes involved. This usually implies differential equations. As river processes are of a dynamic character, both time and space will be involved; hence *partial* differential equations are implicated.

The equations contain coefficients that can only be adjusted experimentally. An obvious example is the (alluvial) roughness of a river, for instance expressed in terms of the Chézy coefficient (C). For a particular river this can only be achieved through *field measurements*.

To study the link between a coefficient and other hydraulic parameters *laboratory experiments* are required. They have the advantage that the conditions can be controlled (which is not usually the case in nature). Moreover, the experiments can be designed according to the specific relationships that are anticipated. The relations between *dispersion coefficients* and hydraulic parameters can be given as examples.

Consequently both field measurements and laboratory experiments are necessary tools in the further development of the description of fluvial processes to be used in the design and application of river models.

Within the framework of this report the description of the fluvial processes can only be rather sketchy. Many references, however, are given to the existing literature.

Section 2.2 deals with water movement over a rigid bed (free surface). In Section 2.3 some aspects of sediment transport in rivers are discussed, whereas in Section 2.4 morphological processes are treated. This is a basis for morphological models. Section 2.5 presents the mathematical background for dispersion processes, and finally Section 2.6 deals with the formation and transport of ice in rivers.

2.2 Water movement

2.2.1 General

Water movement in a river is essentially three-dimensional in space and time-dependent. Moreover, in the case of an alluvial river the flow takes place over a mobile (changing) bed and the presence of suspended sediment may influence the hydraulic roughness. However, this does not mean that all river problems have to be tackled by means of a 'complete' description of the water movement; a fair degree of schematisation is possible in many cases. For instance, the determination of flow profiles for steady uniform flow can be made using a one-dimensional (1-D) model assuming a fixed bed and a constant discharge. The accuracy of the results of such a model will mainly be governed by the accuracy of the available geometric data and the hydraulic roughness.

This type of model can be extended for non-steady flow still using the one-dimensional approach of a fixed bed. The propagation of flood waves can, for instance, be studied in this way.

A two-dimensional (horizontal) description is necessary when the distribution of the depth-averaged velocity across the river is required. This is, for instance, the case if the velocity near embankments of the flood plains of the river have to be known in order to establish the kind of bank protection to be selected. An example of this is given in Sub-section 4.2.2. A full three-dimensional description of (time-dependent) water movement seems only necessary near hydraulic structures in rivers where the hydrostatic pressure distribution is no longer present.

A description of water movement in mathematical terms can be found in many textbooks on hydraulics, and the flow over a rigid bed will therefore not be treated here in any detail. For a systematic approach from 3-D to 1-D descriptions special reference is made to Jansen *et al.* (1979).

There are, however, a number of cases in which it is not sufficient to use the above mentioned flow over a rigid bed. This concerns specifically the flow in a bend of a meandering river. The bend induces a helical flow which is of importance when the bed is mobile. The characteristics of the bed levels in river bends are determined by the helical flow. The flow in a bend is given attention in Sub-section 2.2.2, whilst the response of the river bed is discussed in Sub-section 2.4.5.

2.2.2 Flow in river bends

In an x, y, z-coordinate system the velocity components are u, v and w respectively. Here x is taken to be in the main direction of flow, y perpendicular to x in the horizontal plane and z in the vertical direction.

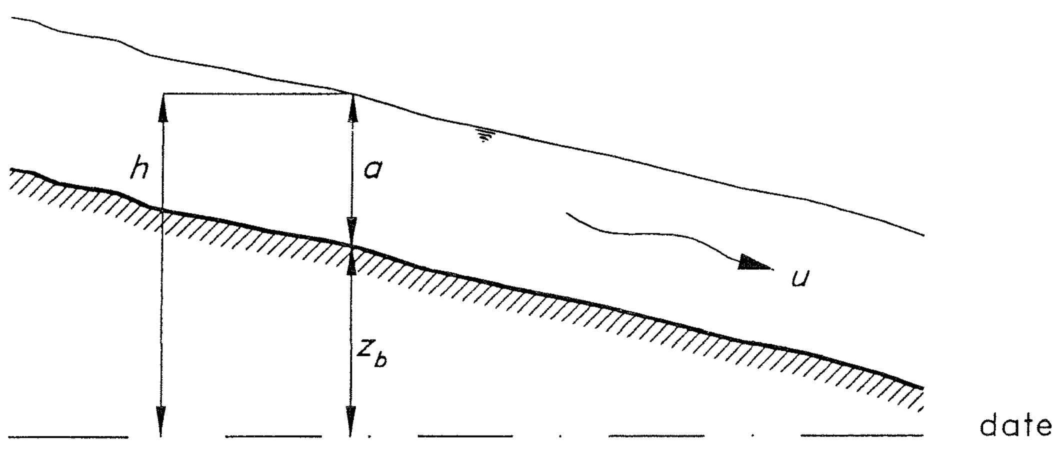


Fig. 2.1 Definition sketch

The bed level (z_b) plus the water depth (a) give the water level (h). These parameters are basically functions of x, y and t.

For this sub-section the flow will be assumed steady and the bed rigid.

The following simple example gives some insight into the interaction of the velocity distribution in the y-direction and the bed topography.

Consider a straight, wide laboratory flume with a rigid, horizontal bed. The depth-averaged flow velocities ($\overline{u}$) are considered. At $x = 0$ the introduced velocity distribution is $\overline{u}$ $(0,y)$. For large values of x the value of $\overline{u}$ (x,y) will no longer be a function of y if wall effects are neglected (Fig. 2.2).

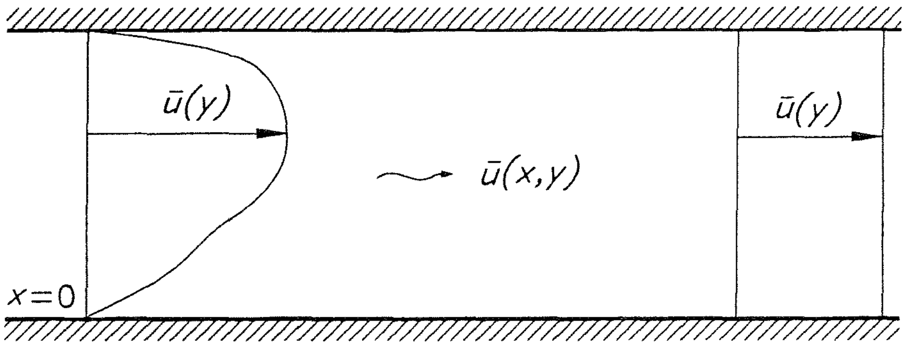


Fig. 2.2 Adaptation of velocity field

It is convenient to describe the problem with natural coordinates (s, n, b). In the (vertical) b-direction the hydrostatic pressure distribution is present.
In the s- and n-directions the momentum equations are

$$\overline{u}\frac{\partial \overline{u}}{\partial s} + g\frac{\partial h}{\partial s} = -g\frac{\overline{u}^2}{C^2 a} \tag{2-1}$$

and

$$g\frac{\partial h}{\partial n} = \frac{\overline{u}^2}{R} \tag{2-2}$$

in which R is the (local) radius of curvature.
For this case $R \to \infty$, hence $\partial h/\partial n = 0$
Differentiation of Eq. (2-1) gives

$$\frac{\partial^2}{\partial n\,\partial s}\left[\tfrac{1}{2}\overline{u}^2\right] + \frac{g}{C^2 a}\frac{\partial \overline{u}^2}{\partial n} = 0 \tag{2-3}$$

Integration of Eq. (2-3) in s-direction yields

$$\frac{\partial \overline{u}^2}{\partial n} = f(n) * \exp(-s/\lambda_w) \tag{2-4}$$

in which $f(n)$ is the integration constant and

$$\lambda_w = \frac{C^2 a}{2g} \tag{2-5}$$

Integration in the n-direction of Eq. (2-5) gives

$$\overline{u}^2 = \exp(-s/\lambda_w) * \int f(n)\mathrm{d}n \tag{2-6}$$

This simple example shows that the *transverse adaptation* of $\overline{u}$ (s,n) and hence of $\overline{u}$ (x,y) is governed by the *length scale* λ_w. The greater the roughness (or the smaller the C-value) the quicker the adaptation takes place.

N.B. A similar length scale (λ_s) but responsible for the adaptation of the mobile bed to the velocity field is treated in Sub-section 2.4.5.

The flow through a river bend is basically of a three-dimensional nature: the velocity has a component in the plane perpendicular to the river axis. This component is directed to the inner bend along the river bed and directed to the outer bend near the water surface. A detailed analysis is given by Rozkovskii (1957) and De Vriend (1981). This 'helical flow' is of great importance to the formation of the bed in an alluvial river, hence its reproduction is of importance (both in numerical and in scale models).

For the special case of axi-symmetric uniform flow (which is not always present in practice) Jansen *et al.* (1979, p. 60) give the following expression for the radial component (v) of the flow velocity.

$$\frac{R}{a}\cdot\frac{v}{U}=\frac{1}{\kappa^2}\left[F_1(\eta)+\frac{\sqrt{g}}{\kappa C}F_2(\eta)-2+\frac{2g}{\kappa^2C^2}-\frac{\sqrt{g}}{\kappa C}\left\{1-\frac{\sqrt{g}}{\kappa C}\right\}\ln\eta\right] \tag{2-7}$$

with

$$F_1(\eta)=\int_{\eta_0}^{\eta}\frac{2\ln\eta^1}{\eta^1-1}\mathrm{d}\eta^1 \text{ and } F_2(\eta)=\int_{\eta_0}^{\eta}\frac{\ln^2\eta^1}{\eta^1-1}\mathrm{d}\eta^1 \tag{2-8}$$

Here R is again the radius of curvature and $\eta = z/a$. The reproduction of this flow pattern in a scale model is discussed in Sub-section 3.2.2.

However, 3-D computations are not necessary if 2-D morphological predictions have to be made in a numerical model. This avoids too large computation costs.

Depth-averaged equations are used to compute the flow field (Fig. 2.3). The effect of the helical flow is added.

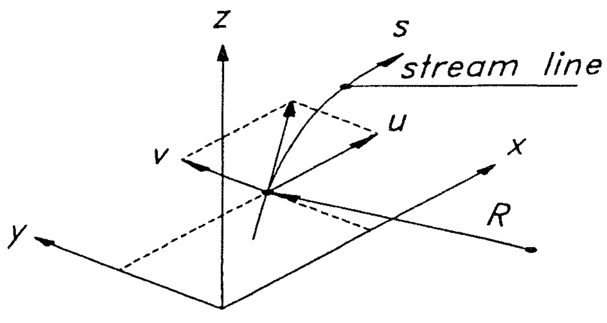


Fig. 2.3 Coordinate system depth-averaged flow

The equations read (e.g. Jansen *et al.*, 1979, p.59)

$$u\frac{\partial u}{\partial x}+v\frac{\partial u}{\partial y}+g\frac{\partial h}{\partial x}+\frac{1}{\rho a}\cdot\tau_{bx}=0 \tag{2-9}$$

$$u\frac{\partial v}{\partial x}+v\frac{\partial v}{\partial y}+g\frac{\partial h}{\partial y}+\frac{1}{\rho a}\cdot\tau_{by}=0 \qquad (2\text{-}10)$$

$$\frac{\partial(au)}{\partial x}+\frac{\partial(av)}{\partial y}=0 \qquad (2\text{-}11)$$

The components τ_{bx} and τ_{by} of the bed shear-stress can, for example, be expressed using the Chézy coefficient (C)

$$\tau_{bx}=\left(\rho\, g\, u\sqrt{u^2+v^2}\right)/\,C^2 \qquad (2\text{-}12)$$

$$\tau_{by}=\left(\rho\, g\, v\sqrt{u^2+v^2}\right)/\,C^2 \qquad (2\text{-}13)$$

Strictly speaking Eqs. (2-10) and (2-11) apply only for straight flow-lines. For curved flow-lines an approximation is involved.

Based on De Vriend (1981), the relation between the radius of curvature of the stream-wise coordinate lines (R_c) and the radius of curvature of the stream line (R_f) was simplified by Olesen (1982, 1987) according to

$$\frac{1}{R_f}=\frac{1}{R_c}-\frac{1}{u}\frac{\partial v}{\partial s} \qquad (2\text{-}14)$$

Here u = stream-wise flow velocity and v = transverse flow-velocity.
The effect of the secondary current can be incorporated into the depth-averaged flow equations via the secondary flow intensity I_s (De Vriend, 1981 and Olesen, 1987), using the equation

$$\lambda_r=\frac{\partial I_s}{\partial s}+I_s=\frac{au}{R_f} \qquad (2\text{-}15)$$

In this equation λ_r = adaptation length for secondary-flow adaptation.
Here

$$\lambda_r=\beta\frac{aC}{\sqrt{g}} \qquad (2\text{-}16)$$

with
β = 1.3 for I_s
β = 0.6 for bed shear-stress associated with I_s

Reference is also made to Struiksma *et al.* (1985).

2.3 Sediment transport

2.3.1 General

The transport of sediment is an essential process in an alluvial river, and its understanding, preferably in a quantitative sense, is of paramount importance for adequate morphological modelling. In Fig. 2.4 a classification of the transport components is given. This *qualitative* classification is according to the ISO-standard 4363-(1977) (see ISO, 1983)

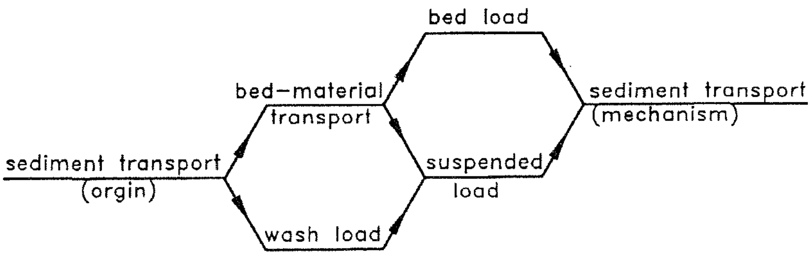


Fig. 2.4 Classification of sediment transport

- *Wash load* is defined as the transport in suspension of material finer than that of the (alluvial) bed. It is simply washed through the river reach.
- *Bed-material transport* takes place close to the bed, i.e. via the bed forms.
- *Suspended load (transport)* is the transport of sediment which is suspended in the fluid for some time. It involves wash load plus part of the bed-material transport.

By definition, wash load is not determined by the hydraulic characteristics of the river *reach* considered, hence it cannot be computed.

Sediment transport formulae concern only bed-material transport.

There are at least two reasons why, in addition to Fig. 2.4, a *quantitative* distinction between wash load and bed-material transport is necessary.

(i) For the comparison of transport predictions with values measured in the field it is necessary to subtract the wash load component.

(ii) A reduction of the flow velocity in the direction of the current will make that part of the wash load become bed-material transport (e.g. for sedimentation in reservoirs).

Vlugter (1941, 1962) argues that fine particles being moved downstream add part of their potential energy to the river system. On the other hand coarse grains require kinetic energy from the river system to stay in (quasi-) suspension. Based on the author's analysis it can be assumed that particles with a fall velocity $W_s < W_c$ form the wash load.
For W_c is given

$$\frac{\rho_s - \rho}{\rho_s} W_c = u \cdot i \tag{2-17}$$

in which ρ_s is the density of the sediment.
Note that wash load is via W_c not only linked to the grain diameter (D) but also to the flow characteristics.

Remarks

(i) The classification of sediment transport given above is based on the assumption of an alluvial river. For a *non-alluvial* river reach (e.g. a gorge with a rigid bed) the following modification applies. In such a case sedimentation is possible. However, erosion below the rigid bed is obviously not possible. If the sediment is simply flushed over the rigid bed, i.e. without sedimentation, then the transport has to be considered to be wash load only. Consequently sediment transport formulae cannot be applied in such a case, since they are only valid for an alluvial bed.

(ii) The writer avoids the terminology 'total load transport' as in the literature this is sometimes used for bed-material transport alone and sometimes for bed-material transport plus wash load.

(iii) In this report the sediment transport is taken as bulk volume (i.e. with pores as the sediment settles). This has the advantage that no porosity need be incorporated into the equation of continuity of the sediment.

2.3.2 On suspended sediment

Here we shall restrict ourselves to remarks on the transport of sediment in suspension. The basic differential equations for the sediment concentration $\phi\,(x, y, z, t)$ in a flow field are derived in many handbooks (e.g. Graf, 1971). These remarks will be used in the discussion on scale models and numerical models for rivers with suspended sediment.

For *steady uniform flow* for water and sediment in suspension the differential equation reads:

$$W_s \phi + \varepsilon_s \frac{\mathrm{d}\phi}{\mathrm{d}z} = 0 \qquad (2\text{-}18)$$

Here W_s is the fall velocity of the sediment and ε_s is the diffusion coefficient for the sediment. Rouse (1936) was the first to solve Eq. (2-18) by supposing $\varepsilon_s = \varepsilon_m$ where ε_m is the transfer coefficient for momentum:

$$\varepsilon_m(z) \approx \kappa u_* z \left\{1 - z / a\right\} \qquad (2\text{-}19)$$

Integration of Eq. (2-18) then leads to the concentration distribution over the vertical, ϕ (z):

$$\frac{\phi(z)}{\phi(z_1)} = \left[\frac{a - z}{a - z_1} * \frac{z_1}{a}\right]^Z \qquad (2\text{-}20)$$

Here z_1 is a reference level where the sediment concentration has to be known (integration constant). The exponent Z (Rouse parameter) is given by

$$Z = \frac{W_s}{\kappa u_*} \qquad (2\text{-}21)$$

Many references are available with respect to the relation between ε_s and ε_m. Here mention is made only to the work of Coleman (1970), who derived ε_s from flume experiments (Fig. 2.5).

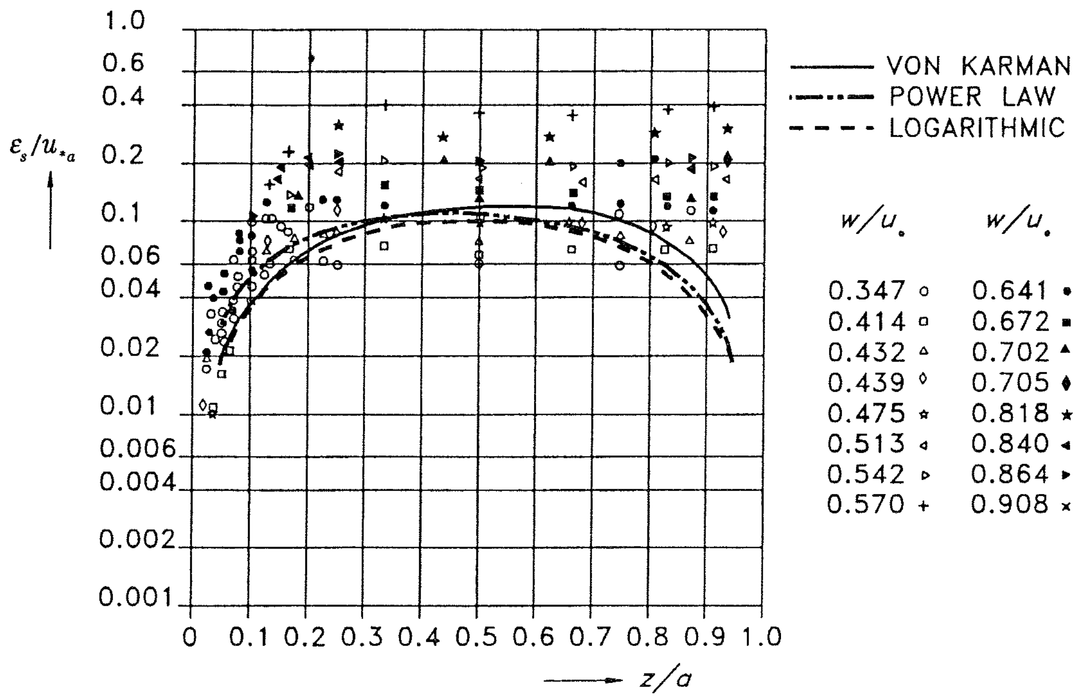


Fig. 2.5 Diffusion coefficient for sediment (ε_s), after Coleman (1970)

In spite of much scatter (due to the fact that the measured values of $\phi(z)$ had to be differentiated) two conclusions can be drawn.

(i) Apparently the assumption $\varepsilon_s = \varepsilon_m$ is not valid.

(ii) The tendency is present that

$$\frac{\varepsilon_s}{u_* a} = f\left\{\frac{z}{a}, Z\right\} \tag{2-22}$$

This has led to the use of approximations (Kerssens *et al.*, 1977).

$$\begin{aligned} \varepsilon(z) &= \varepsilon_{\max} = \left\{0.13 + 0.2\,(W_s/u_*)^{2.12}\right\} && \textit{for } z/a \geq \tfrac{1}{2} \\ \varepsilon(z) &= 4\{z/a\}\{1 - z/a\}\varepsilon_{\max} && \textit{for } z/a \leq \tfrac{1}{2} \end{aligned} \tag{2-23}$$

In addition some attention has to be paid to uniform flow for water only. For the concentration $\phi(x, z, t)$ in the 2-DV case holds

$$\frac{\partial \phi}{\partial t} + \frac{\partial}{\partial x}\{u\phi\} - \frac{\partial}{\partial z}\left\{W_s\phi + \varepsilon\frac{\partial \phi}{\partial z}\right\} = 0 \tag{2-24}$$

With $\partial u/\partial x$ the equation can be written in dimensionless parameters using for the velocity scale U the expression $u = Uu^1$.

Similarly

$$z = a\zeta;\ \varepsilon = E\,\varepsilon^1;\ t = T\tau \text{ and } x = L\xi \tag{2-25}$$

Inserting this in Eq. (2-24) gives

$$\left[\frac{a}{W_s T}\right]\frac{\partial\phi}{\partial\tau}+\left[\frac{aU}{LW_s}\right]\left\{u^1\frac{\partial\phi}{\partial\xi}\right\}=\frac{\partial\phi}{\partial\zeta}+\left[\frac{E}{W_s a}\right]\frac{\partial}{\partial\zeta}\left\{\varepsilon^1\frac{\partial\phi}{\partial\zeta}\right\} \qquad (2\text{-}26)$$

The relevance of Eq. (2-26) is demonstrated for scale models in Sub-section 3.3.8 and for numerical models in Sub-section 4.3.3.

2.3.3 Predictors for transport and roughness

Basically the assumption underlying transport formulae is the presence of steady uniform flow over an alluvial bed. Einstein (1950) was the first to put forward an overall concept.

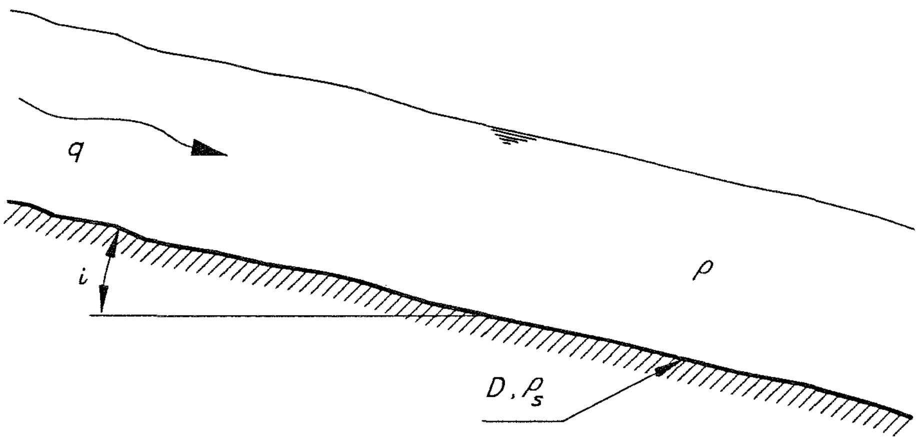


Fig. 2.6 Basic concept (Einstein, 1950)

Einstein considered an alluvial channel with a given bed slope (i) of granular material (diameter D and relative density $\Delta = (\rho_s - \rho)/\rho$). A constant discharge per unit width (q) is introduced over the bed (Fig. 2.6)

Bed forms are developed leading to a bed roughness (C). This determines the water depth (a) via $q = C\, a^{3/2}\, i^{1/2}$. Under these conditions there will be a transport of sediment per unit width (s), provided the shear stress at the bed ($\tau_b = \rho g a i$) is larger than the critical one (τ_c) for initiation of motion (e.g. according to Shields, 1936).

Water movement and sediment movement are obviously closely interrelated. Consequently two predictors have to be used. A *roughness predictor* to predict C and a *transport predictor* to predict s. Modern transport formulae are based on this concept.

Remarks

(i) Obviously there are two types of transport formula. Firstly, those that assume the roughness to be known. These can be used to estimate the transport in an existing alluvial channel, of which the composition of the bed and the water movement are known. This is, for instance, the case for the Meyer-Peter and Mueller (1948) formula. This paper, written prior to Einstein (1950), does not even mention how the roughness value to be used in the MPM-formula is obtained.
A second example is the Ackers and White (1973) formula, which can only be applied to *existing* channels. However, in this case the *transport predictor* can be combined with a later published *roughness predictor* (White *et al.*, 1980). With these predictors combined it is possible to forecast the sediment transport if the composition of the

bed material, the overall bed-slope and the discharge per unit width are known. It then becomes a tool for the *second* category of predictors, viz general ones that can be used in numerical morphological models.

(ii) A 'complete' formula for sediment transport is therefore a combination of a *roughness predictor* and a *transport predictor*. The first roughness predictor was described by Einstein and Barbarossa (1952). It seems to be now of historical value only, since it was based on just a few measurements.

(iii) The original concept of Einstein (1950) was later also used as a framework for processing river measurements with respect to discharge and sediment transport (Colby and Hembree, 1955). To call this a *modified Einstein procedure* is somewhat misleading, as it seems a different use than that given in the original Einstein concept.

The writer does not try to give a full overview here of the more modern transport predictors and roughness predictors, since a rather complete picture is given in Raudkivi (1990). Older books are Graf (1971), Yalin (1972) and Bogárdi (1974).

Many transport formulae can be expressed as a function of two dimensionless parameters ϕ_s and θ with

$$\phi_s = \frac{s}{D^{3/2}\sqrt{g\Delta}} \text{ and } \theta = \frac{\mu a i}{\Delta D} \tag{2-27}$$

For D a characteristic grain diameter (different definitions are in use) is taken. The coefficient μ does take care of the influence of the bed forms; although here again different definitions are in use.

For general considerations of morphological changes either by mathematical analysis or by scale modelling it is attractive to apply the generic equation

$$s = mu^n \tag{2-28}$$

or in its dimensionless form

$$\phi_s = \alpha\,\theta^\beta \tag{2-29}$$

by assuming m and n to be locally constant; Eq. (2-28) takes into consideration that, of all parameters involved, the variation of u contributes to the greatest extent to the variation in transport. As $\theta \approx u^2$ it can easily be shown that $n = 2\beta$.

For the Engelund-Hansen (1967) formula it can easily be found that n = constant = 5. For the Meyer-Peter and Mueller formula n varies with θ.

In general, when the transport formula can be written as a unique function

$$\phi_s = f(\theta) \tag{2-30}$$

then the values of α and β can be obtained by equalising the function values and the first derivatives of Eqs. (2-29) and (2-30) respectively.

This procedure gives, for example for the Meyer-Peter and Mueller formula, the following expression for n

$$n = \frac{3}{1 - 0.047\,\theta^{-1}} \tag{2-31}$$

Besides the older formulae quoted above, some newer ones can be mentioned here (Brownlie, 1981; Karim and Kennedy, 1983; Van Rijn, 1984; and Parker and Klingeman, 1982). As stated above, details and other formulae can be found in Raudkivi (1990).

2.3.4 Accuracy of predictors

Transport predictors and roughness predictors have a limited accuracy. A transport predictor is accurate if a measured transport can be predicted within a factor of two. Predicting the alluvial roughness (C-value) within ± 20 per cent accuracy is already a success.
The influence of the inaccuracy of predictors on morphological forecasts can be studied in two ways.

(i) Accuracy of predictors proper

A study of the accuracy of the predictors proper can be carried out by comparing *measured* and *predicted* values of s and C. Using the data from the compendium of Peterson and Howells (1973) the writer compared some formulae for transport and roughness (De Vries, 1983). Some results are given below. The scores (in %) for the ratio r_s for predicted and measured transport ($r_s = s_p/s_m$) are given for four formulae in Table 2.1.

TABLE 2.1 Scores in the interval $\frac{1}{2} < r_s < 2$

Source	No. of data	Transport predictor			
		Engelund and Hansen (1967)	Ackers and White (1973, 1980)	Van Rijn (1982, 1984)	Karim and Kennedy (1983)
CSU-data	162	89%	85%	70%	74%
USA-rivers	299	67%	61%	79%	53%
Pakistan canals (ACOP)	142	71%	71%	56%	45%

Similarly the scores for the ratio r_C for predicted and measured Chézy roughness ($r_C = C_p/C_m$) are given in Table 2.2.

TABLE 2.2 Scores in the interval $0.8 < r_C < 1.2$

Source	No. of data	Roughness predictor			
		Engelund and Hansen (1967)	Ackers and White (1973, 1980)	Van Rijn (1982, 1984)	Karim and Kennedy (1983)
CSU-data	162	59%	36%	63%	53%
USA-rivers	299	38%	56%	68%	34%
Pakistan canals (ACOP)	142	42%	80%	84%	81%

The reader can judge for him/herself whether or not the newer predictors are necessarily better than the older ones. The relatively simple predictor of Engelund and Hansen (containing many fewer experimental coefficients than the Ackers and White method) is still doing surprisingly well.

(ii) Accuracy of morphological predictions

For many morphological predictions the accuracy of the transport is not the ultimate goal (sedimentation of reservoirs perhaps excluded). Therefore research has also been carried out to study the propagation of inaccuracies in the prediction of s and C into morphological predictions (e.g. of the water depth).

Insight into this aspect can be obtained using analytical solutions of the one-dimensional morphological equations (De Vries, 1982a; 1983). Although analytical solutions give insight into the influence of errors in the separate parameters there is the disadvantage that the basic equations have to be linearised.

An alternative method is the repeated use of a numerical model. Random selection of the parameters from their respective probability distributions can lead to, for example, the probability distributions of the bed level as a function of time and space. An example is given by Van Rijn (1987). Obviously this method can only be applied when a single computation is not too time-consuming.

2.4 Morphology

2.4.1 General

The combined movement of water and sediment determines the river's morphology. Modelling of the morphological processes is required to make forecasts of the changes of the river's morphology due to natural causes or human interference.
Within the framework of this report only a rather sketchy treatment on river morphology can be given, and attention paid to only some aspects.

In Sub-section 2.4.2 the question whether the discharge variation, thus $Q = Q(t)$, can be replaced by one single discharge is answered in the negative. Consequently Sub-section 2.4.3 on 'regime concepts' is also rather critical.

Sub-section 2.4.4 summarises the one-dimensional time-dependent approach, whereas in Sub-section 2.4.5 some remarks are made on the mathematical description of the combined movement of water and sediment in meandering rivers.

2.4.2 'Dominant' discharge?

One of the essential features of rivers is the variation of the discharge with time. Many attempts have been made to schematise this into one discharge if morphological descriptions of rivers are given. Such a drastic schematisation of the varying discharge in nature is also sometimes used in river models based on the mathematical description of the physical processes in alluvial rivers (1-D and 2-D).

Such a schematisation into a single discharge ('dominant' discharge) stems typically from the time when the computational capacity was very much smaller than at present (1992). Even now especially two-dimensional morphological numerical models are used with constant discharges to save computer time. This seems especially possible when bed-level variations in time are relatively small, as with the River Rhine in the Netherlands. For other rivers the application of a constant discharge can lead to errors, the Mekong River being an example (*cf.* Sub-section 4.3.4).

A single discharge cannot be found for more than one morphological parameter. This statement can be made plausible for some simple geometries. It is then unlikely that one single discharge can be used for more complicated geometries. The writer is used to demonstrating with a simple example that one discharge cannot reproduce the morphological parameters of a river as caused by the varying discharge in nature (De Vries, 1990).

Consider therefore a river discharging into a lake with a constant water level. The river banks are assumed to be non-erodible. The sediment is uniform and the width (B) is constant.

'Dynamic equilibrium' means in this case that through each cross-section the total sediment transport over a sufficiently long time-interval, due to a varying discharge with probability density $p\{Q\}$, is constant

or
$$\int_0^\infty S(Q) * p\{Q\}\, \mathrm{d}Q = \text{constant} \qquad (2\text{-}31)$$

Note that $S(Q)$ does *not* mean that S is a function of Q *alone*.

Consider for instance with $s = mu^n$ the relation $S(Q, i_b, B)$ in which i_b is the overall bed-slope. Using the Chézy equation leads to

$$S \sim B^{1-n/3} * Q^{n/3} * i_b^{n/3} \qquad (2\text{-}32)$$

Combining Eqs. (2-31) and (2-32) gives, assuming n = constant,

$$B^{1-n/3} * i_b^{n/3} \int_0^\infty Q^{n/3} p\{Q\} * \mathrm{d}Q = \text{constant} \qquad (2\text{-}33)$$

Here the logical assumption is made that the overall slope i_b does not vary with Q in the case of 'dynamic equilibrium'.

Consider, secondly, a case in which $s = mu^n$ the relation $S(Q, a_0, B)$ in which a_0 is the depth in the mouth of the river.

Now similarly

$$S \sim B^{1-n} * Q^n * a_0^{-n} \qquad (2\text{-}34)$$

A combination of Eqs. (2-31) and (2-34) yields for this case, again assuming n = constant.

$$B^{1-n}\, a_0^{-n} \int_0^\infty Q^n * p\{Q\} * \mathrm{d}Q = \text{constant} \qquad (2\text{-}35)$$

in which as an approximation it is assumed that a_0 does not vary too much with the discharge.

Obviously the integrals of Eqs. (2-33) and (2-35) cannot be replaced by an expression with the *same* representative discharge. Either i_b or a_0 can be reproduced correctly by a single discharge ('dominant' discharge).

The writer has come across on various occasions a 'dominant' discharge concept based on the following reasoning. The average yearly sediment transport due to a varying discharge is reached by the (constant) 'dominant' discharge *in one year*. This is questionable. In fact a model is made of the river (*cf.* Figure 1.1) via this schematisation. There is, however, no reason why the time scale of this model should equal unity.

Finally, the definition of a 'dominant' discharge by NEDECO (1959) has to be mentioned. The derivation is lengthy, caused by the exclusive use of the Meyer-Peter and Mueller (1948) formula. If the general approximation $s = mu^n$ is used then the derivation becomes more transparent (Jansen *et al.,* 1979, p. 128). For n = constant the 'dominant' water depth (a_D) is

$$a_D = \frac{\int_0^\tau S(t)\,\mathrm{d}t}{\int_0^\tau a^{-1}(t) * S(t)\,\mathrm{d}t} \qquad (2\text{-}36)$$

The derivation is, however, questionable due to the assumptions made (Jansen *et al.*, 1979). For example, the assumption is made that $\partial S_D / \partial x = 0$ which holds for *every* constant discharge.

From the considerations given above it can be concluded that schematisation of the varying discharge of a river into a single discharge to describe the morphological properties of the river is generally impossible (see also Prins and De Vries, 1971).

2.4.3 On 'regime concepts'

Some attention has to be given to 'regime concepts' being simple 'models' to describe major overall parameters of alluvial rivers. The term 'regime theory' also used in this respect has to be avoided since there is hardly theory involved in these concepts. Summaries of 'regime concepts' can, for example, be found in Graf (1971) and Raudkivi (1990).

The concepts are in principle based on a statistical analysis of overall river parameters. The method was originally designed for irrigation canals on the Indian subcontinent by British engineers (Lacey, Inglish a.o.). Later it was extended to rivers (see for example Blench, 1957, 1969). Whereas the discharge in irrigation canals seems to be more constant than in rivers, the shift to rivers raises the question which (constant) discharge has to be used in the 'regime concepts' to represent the variable $Q(t)$. It has been shown in Subsection 2.4.2 that such a constant discharge is unlikely to exist. This is also stated by Blench (1969, p.30).

The 'regime concepts' are hardly based on physical notions. Even a strong advocate of the 'regime concepts' like Blench gives warnings in this respect. One of the strongest seems to be: *"formulas* [. . .] *cannot be expected to represent physical laws except by accident'*. (Blench, 1969, p. 22). Attempts have been made, however, to give the 'regime concepts' a physical basis (Stevens and Nordin, 1987).

By their nature the equations forming a specific 'regime concept' are largely based on experimental data. Consequently there is much scatter. It is remarkable that statistical methods are hardly used to investigate the significance of the numerical values of the coefficients and exponents of the regime relations. Hence also the significance of the numerical values derived with the regime relations cannot be indicated.

This statement can be demonstrated with an example. White *et al.* (1981a) present a design concept for stable channels. The many figures only give a visual indication of the scatter, for example in the plot of *measured* and *observed* values on log-log paper. In a subsequent report (White *et al.*, 1981b) tables are presented to be used in the actual design. There, for example, the slopes are given to three digits. It is questionable whether three digits are significant. Nevertheless in the report quoted here on page 2 a numerical example is given in which as result of depth and surface width are given to three digits. A sentence is added: 'Interpolation within the table could be used to refine the estimate' (!?).

From the remarks given above it will be clear that the writer is reluctant to incorporate 'regime concepts' into numerical models for morphological river problems. The same holds for scale models, as is outlined in Sub-section 3.3.6.

2.4.4 One-dimensional morphology

In the one-dimensional morphological approaches time-dependent equations for the width-averaged dependent variables depth (a), velocity (u), bed level (z_b) and transport (s) are considered. Only one space dimension (x) is taken into consideration.

Taking the width (B), the grain size (D) and the roughness (C) constant, the two equations for the water movement and the two equations for the sediment movement there are just four equations for the four dependent variables.

This system of equations was analysed by De Vries (1959, 1965). It was shown that it contains three celerities (c) governing the propagation of disturbances in the dependent variables. These celerities are obtained from the cubic equation:

$$- c^3 + 2uc^2 + (ga - u^2 + g\, \mathrm{d}s/\mathrm{d}u)c - ug\, \mathrm{d}s/\mathrm{d}u = 0 \qquad (2\text{-}37)$$

Or in dimensionless form

$$\phi^3 - 2\phi^2 + \left(1 - Fr^{-2} - \psi Fr^{-2}\right)\phi + \psi Fr^{-2} = 0 \qquad (2\text{-}38)$$

Here ϕ = c/u; Fr = Froude number = $u/\sqrt{ga}$ and $\psi = a^{-1}\, \mathrm{d}s/\mathrm{d}u$.
The analysis can also be found in Jansen *et al.* (1979, p. 93).
Usually $\psi = n\, s/q$ is much smaller than unity. For moderate Froude numbers (say $Fr < 0.6$) it holds $|\phi_{1,2}| >> \phi_3 >> \phi_3$ with

$$\phi_3 = \frac{\psi}{1 - Fr^2} \qquad (2\text{-}39)$$

In this case $\phi_{1,2}$ are the celerities for small disturbances at the water level, as is the case of a fixed bed, the relative celerity ϕ_3 can be identified as the one for propagation of a small disturbance at the bed.

This basic analysis contains a number of assumptions to be discussed here.

(i) *Non-erodible banks* are essentially assumed. It requires an additional equation to incorporate $B = B(x,t)$.

(ii) *Constant C-value*; this assumption is not essential. If needed, a suitable roughness predictor can be used (see also Sub-section 4.3.2).

(iii) *Constant D-value;* this restricts the application of the above given system. A thorough theoretical and experimental study on the extension for non-uniform sediment by Ribberink (1987) has indicated a way to overcome this restriction.

(iv) *Application of* $s = s(u)$ implies that the sediment transport is a function of the *local* hydraulic condition.

The above analysis has the advantage that it is a basis for 1-D morphological computations. For many practical problems it holds $Fr < 0.6$. In their case the values of $\phi_{1,2}$ are not influenced by the mobility of the bed. Hence the system of equations can be decoupled and the water movement and bed-level changes can be computed alternately. If $Q(t)$ does not vary rapidly in time the water movement can be computed assuming steady non-uniform for time t. This is discussed in Sub-section 4.3.2.

For quasi-steady flow, $B(x)$ = constant and $s<<q$ the governing equations reduce to

$$u\frac{\partial u}{\partial x}+g\frac{\partial a}{\partial x}+g\frac{\partial z_b}{\partial x}=g\frac{u^2}{C^2a} \tag{2-41}$$

$$u\frac{\partial a}{\partial x}+a\frac{\partial u}{\partial x}=\frac{\partial q}{\partial x}=0 \text{ or } q(x,t)=q(t) \tag{2-42}$$

$$s=f(u) \tag{2-43}$$

$$\frac{\partial z_b}{\partial t}+\frac{\partial s}{\partial x}=0 \tag{2-44}$$

Under certain restrictions analytical solutions can be obtained for this (hyperbolic) system. Raudkivi (1990, p. 294) argues that analytical solutions do not have a high accuracy. The writer agrees, but considers that analytical models do give much insight; therefore a number of remarks on analytical solutions of Eqs. (2-41 . . . 2-44) are made.

(i) Analytical solutions can only be obtained if the equations are linearised. The equations can then be used to find variations on a steady uniform base situation (subscript 0).

(ii) If the x-axis is taken along the original bed slope (positive downstream) then two equations can be written on the *variation* of the water level, $\eta(x,t)$ and the *variation* of the bed level, $z_b(x,t)$ as deduced by Ribberink and van der Sande (1984, 1985).

$$\alpha_0\frac{\partial \eta}{\partial x}+(1-\alpha_0)\frac{\partial z_b}{\partial x}+A_0(z_b-\eta)=0 \tag{2-45}$$

$$\frac{\partial z_b}{\partial t}+c_{b_0}\frac{\partial z_b}{\partial x}-c_{b_0}\frac{\partial \eta}{\partial x}=0 \tag{2-46}$$

in which

$$A_0=3i_0/a_0$$

$$C_{b_0}=\left(\frac{\mathrm{d}s}{\mathrm{d}u}\right)_0\cdot\frac{U_0}{a_0}$$

and $\alpha_0=1-Fr_0^2$

(iii) For *large values* of u and t the (*hyperbolic*) system becomes *parabolic* (Vreugdenhil and De Vries, 1973). This is the case when backwater effects can be neglected, or if $A_0=xi_0/a_0>3$.

The governing equation is (De Vries, 1975)

$$\frac{\partial z_b}{\partial t}-K(t)\frac{\partial^2 z_b}{\partial x^2}=0 \tag{2-47}$$

with $$K(t)=\frac{1}{3}\frac{\mathrm{d}s}{\mathrm{d}u}\cdot\frac{C^2a}{u} \tag{2-48}$$

This reduces for $s = mu^n$ into (see also Jansen *et al.*, 1979, p. 122)

$$K(t) = \frac{1}{3}\frac{ns}{i_b} \tag{2-49}$$

An interesting application of the parabolic model is the definition of a morphological time-scale (De Vries, 1975). A sudden drop Δh in the (hypothetical) erosion base is assumed and the time T_m computed at which at a distance L_m upstream the bed level is lowered to $z_b(L_m, T_m) = \frac{1}{2}\Delta h.$

The analytical solution of Eq. (2-47) for this problem makes it possible to express T_m in a number of years (N_m) with

$$N_m = \frac{L_m^2}{Y} \tag{2-50}$$

with

$$Y = \int_0^{1\,\text{year}} K(t)\,\mathrm{d}t \approx \frac{1}{3}\frac{n}{Bi_b}\int_0^{1\,\text{year}} S(t)\,\mathrm{d}t \tag{2-51}$$

TABLE 2.3 Morphological time-scale for various rivers (after De Vries, 1975)

River	Station (approx. distance from sea)	D (mm)	$i_b \times 10^{-4}$	$3a/i_b$ (km)	N_m (centuries)
Waal River (Netherlands)	Zaltbommel	2	1.2	100	20
Magdalena River (Colombia)	Puerto Berrío (730 km)	0.33	5	30	2
Danube River (Hungary)	Dunaremete (1826km)	2	3.5	40	10
	Nagymaros (1695km)	0.35	0.8	180	2.6
	Dunaujvaros (1581km)	0.35	0.8	180	1.5
	Baja (1480km)	0.26	0.7	210	0.6
Tana River (Kenya)	Bura	0.32	3.5	50	2.0
Apure River (Venezuela)	San Fernando	0.35	0.7	200	2.2
Mekong River (Viet Nam)	Pa Mong	0.32	1.1	270	1.3
Serang River (Indonesia)	Godong	0.25	2.5	50	2.0

To fulfil the requirement $A_0 \geq 3$ a large length L_m had to be selected, leading to large values of N_m. In Table 2.3 the values of N_m are listed for some river stations based on the selection L_m = 200 km. Relatively few river data are necessary to establish the value of N_m for a particular river.

(iv) On the other hand for *small values* of x and t the friction term of Eq. (2-41) can be neglected. This means that for small Froude numbers the *rigid lid approximation* can be applied. The hyperbolic system is reduced to a *simple-wave equation.* Expressed in the water depth (a) under the (horizontal) water level

$$\frac{\partial a}{\partial t} + c(a)\frac{\partial a}{\partial x} = 0 \qquad (2\text{-}52)$$

with

$$c(a) = -\frac{\mathrm{d}s}{\mathrm{d}a} \qquad (2\text{-}53)$$

It has to be realized that Eq. (2-52) holds for the case of the applicability of $s = f(u)$. The interesting aspect of Eq. (2-53) is its non-linearity on the one hand. On the other it is possible despite the non-linearity to get analytical solutions. This is the reason why Vreugdenhil (1982) applied Eq. (2-52) to test the performance of various numerical schemes to solve one-dimensional problems.

(v) A typical example of the use of analytical models to gain insight into morphological phenomena is the study by Ribberink and van der Sande (1984, 1985) of the morphological reactions of a river on local overloading. They used the laboratory data of Sony *et al.* (1980).

The overloading at $x = 0$ for $t > 0$ creates a bed-wave front travelling according to $\tilde{x} = \frac{1}{2}\tilde{t}$. Here $\tilde{x}$ and $\tilde{t}$ are dimensionless independent variables with

$$\tilde{x} = 3\frac{i_0}{a_0}x \qquad (2\text{-}54)$$

and (for $n = 5$, Engelund-Hansen)

$$\tilde{t} = 30\frac{s_0 i_0}{a_0}t \qquad (2\text{-}55)$$

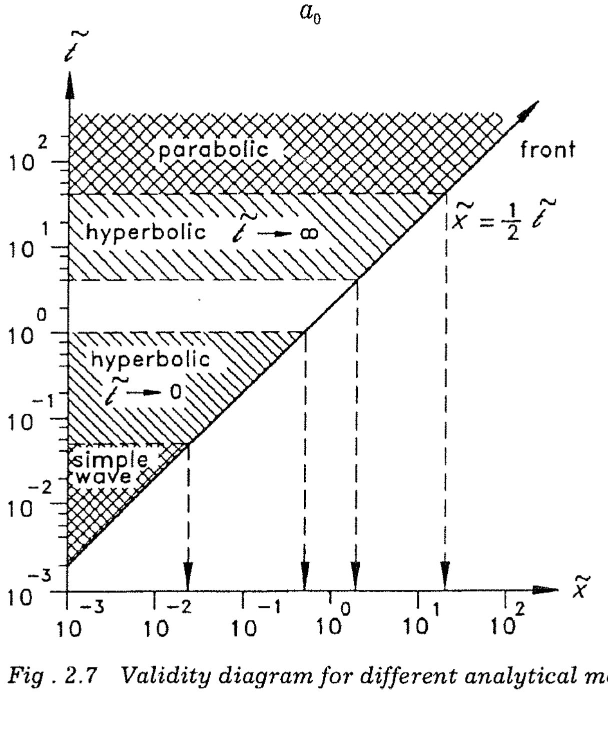


Fig . 2.7 Validity diagram for different analytical models (Ribberink and van der Sande, 1984)

In Fig. 2.7 the range of validity of the three models is indicated. It is to be noted that the general hyperbolic model is only valid for a constant discharge. It is not always possible to find an analytical solution with the hyperbolic model. That is why Fig. 2.7 shows a gap for intermediate values of $\tilde{x}$ and $\tilde{t}$.

2.4.5 Two-dimensional morphology

In this sub-section two (horizontal) space-dimensions are considered. A number of practical problems can be found in this category:

(i) *Meander migration* has to be forecast if river banks are not protected and the future course of the river needs to be estimated. A relatively simple theory is given by Ikeda *et al.* (1981). Application of this method is discussed in Sub-section 4.3.3. In practice it is difficult to express the erodibility of the river banks. Crosato (1987, 1990) treats this mathematical problem in more detail.

(ii) *Two-dimensional models with fixed banks.* In this case the plan form of the river is fixed, and $z_b\,(x, y, t)$ needs to be described. This requires a detailed description of the movement of water and sediment in a curved (meandering) channel. For description with bed load only, reference can be made to Olesen (1987). An extension with suspended load is given by Talmon (1989).

Between the two cases described above there is the intermediate one in which $z_b\,(x, y, t)$ is anticipated in the presence of erodible banks (Mosselman, 1989, 1992). Since the early work of Van Bendegom (1947) the 2-D morphology has become better understood by analysing the basic equations in simplified form (Struiksma *et al.*, 1985; Struiksma and Crosato, 1988). See also Engelund (1974). In Sub-section 2.2.2 the adaptation of the velocity distribution $\overline{u}(x,y)$ to the bed topography has been mentioned, leading to a *length scale* (λ_w) for this process.

In a similar way there is an adaptation of the bed topography to the velocity field. The length scale of this process is

$$\lambda_s = \frac{1}{\pi^2}\frac{B_0^2}{a_0^2} f(\theta_0) a_0 \qquad (2\text{-}56)$$

The subscript $_0$ refers to the (uniform) base flow.

The function $f(\theta_0)$ is a measure of the influence of the bed slope perpendicular to the transport direction. Odgaard (1981) gives a review on $f(\theta_0)$.

A linear analysis of the 2-D model given by Struiksma *et al.* (1985) shows that for steady state the zero-order solution describes the bed level of the fully developed bend. This is the axi-symmetrical case, i.e. an infinitely long bend where the subsequent cross-sections have the same characteristics. The first-order solution is wave-like around the zero-order solution; this wave has a length L_p (roughly the meander length) and a damping length (L_D). Both L_p and L_D are dependent on λ_w and λ_s. Struiksma (1986) gives the expressions

$$\frac{\lambda_w}{L_p} = \frac{1}{4\pi}\sqrt{(n+1)\frac{\lambda_w}{\lambda_s} - \left(\frac{\lambda_w}{\lambda_s}\right)^2 - \frac{n-3}{2}} \qquad (2\text{-}57)$$

and

$$\frac{\lambda_w}{L_D} = \frac{1}{2}\left(\frac{\lambda_w}{\lambda_s} - \frac{n-3}{2}\right) \tag{2-58}$$

Here n is again the exponent of the power-law for the transport. It has great influence. Figure 2.8 gives a presentation for $n = 5$ (Engelund-Hansen).

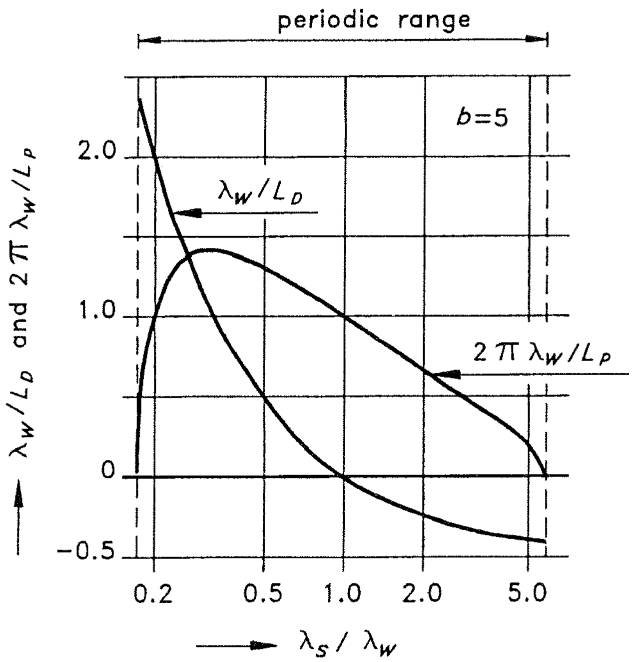


Fig. 2.8 Wave length and damping length (after Struiksma et al., 1985)

The influence of the ratio λ_s/λ_w ('interaction parameter') is great. The bed-level variation along the banks of a river is shown in Fig. 3.1 where a circular bend between two straight river-reaches is taken.

The analysis quoted above has given substantial insight into the results of numerical models. The analysis is also valuable for scale models (Struiksma, 1986) (see Sub-section 3.3.3).

2.5 Dispersion

2.5.1 General

The (turbulent) flow in rivers is able to transport dissolved matter. The transport of the matter is governed by two processes, *convection* by the mean flow-velocity and *dispersion* caused by the variation of the velocity field around the mean flow velocity.

Practical problems that are solved by models based on the mathematical description of dispersion concern:

(i) The prediction of the concentration of the dissolved matter ϕ in the river water due to an outlet structure for effluent at a known location. This can also involve cooling water.

(ii) The presence of an *accidental spill* in the river also requires the prediction of the concentration (ϕ) in time and space. This may involve dissolved matter which above a certain concentration may harm the aquatic environment.

As a side line the dispersion process is also used if intentionally dissolved matter is injected in a stream in order to derive via the measured concentrations the discharge (*dilution method*).

Here the basic equations will be discussed only in general terms; during the last decades much research has been carried out on dispersion.

2.5.2 Basic equations

Molecular diffusion can be neglected in rivers compared to the other mixing processes present. It is the turbulence of the flow that is responsible for the mixing. In the one-dimensional approach the transport of dissolved matter is governed by the *convection* by the time-averaged flow velocity and *diffusive transport* in the x-direction due to the gradient $\partial\phi/\partial x$ (Taylor, 1953, 1954). Considering a 2-DV situation, there are in fact three processes that determine the mixing:

(i) convection with the depth-averaged flow velocity ($\overline{u}$)
(ii) dispersion due to the time-averaged flow-velocity distribution $u(z)$
(iii) diffusive transport due to the turbulence.

It appears that the effect of (iii) is small compared with that of (ii). It is assumed that the total effect of the dispersive transport is proportional to $\partial\phi/\partial x$ (Fisher, 1966). The 2-DV dispersion is demonstrated in Fig. 2.9.

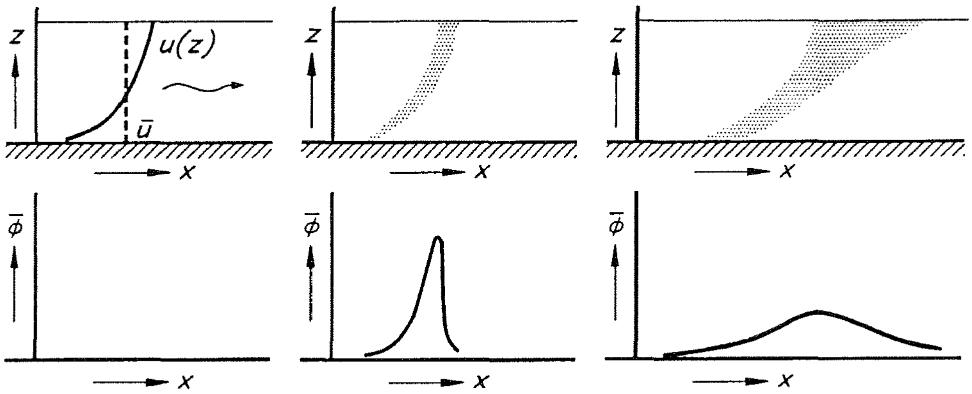


Fig. 2.9 Dispersion mechanism (after Fischer, 1966)

This approach leads to the one-dimensional dispersion equation

$$\frac{\partial\overline{\phi}}{\partial t}+\overline{u}\frac{\partial\overline{\phi}}{\partial x}-K\frac{\partial\overline{\phi}}{\partial x^2}=0 \qquad (2\text{-}59)$$

Here K is the dispersion coefficient. A similar approach can be used for a 2-DH problem. If the x-axis is taken along the direction of the main stream then

$$\frac{\partial\overline{\phi}}{\partial t}+\overline{u}\frac{\partial\overline{\phi}}{\partial x}-K_1\frac{\partial\overline{\phi}}{\partial x^2}-K_2\frac{\partial\overline{\phi}}{\partial y^2}=0 \qquad (2\text{-}60)$$

Here K_1 and K_2 represent dispersion coefficients in *longitudinal* and *transverse* directions respectively.

In Eqs. (2-59) and (2-60) conservation of the dissolved matter is assumed. If there is decay then

$$\frac{\partial \overline{\phi}}{\partial t} + \overline{u}\frac{\partial \overline{\phi}}{\partial x} - K_1 \frac{\partial^2 \overline{\phi}}{\partial x^2} - K_2 \frac{\partial^2 \overline{\phi}}{\partial y^2} + \frac{\overline{\phi}}{t_r} = 0 \qquad (2\text{-}61)$$

in which t_r in the *relaxation time* of the dissolved matter considered.
A thorough derivation of the basic equations is given by Fisher *et al.* (1979).
The problem is now to find the dispersion coefficients. For the coefficient K in Eq. (2-59) Fischer *et al.* (1979) give the semi-empirical expression

$$K = 0.011\, \frac{\overline{u}^2 \cdot B^2}{a\, u_*} \qquad (2\text{-}62)$$

For the 2-DH model described by Eq. (2-61) the following information on K_1, and K_2 can be given.

(i) The *longitudinal dispersion coefficient* (K_1), gives

$$K_1 \sim a\, u_* \qquad (2\text{-}63)$$

The proportionality factor is about 6.

(ii) The *transverse dispersion coefficient* (K_2) can be written as

$$K_2 = \alpha\, a\, u_* \qquad (2\text{-}64)$$

Fisher *et al.* (1979) suggest $\alpha = 0.6 \pm 0.3$. This fits with the values found from measurements in the Rhine branches by Holley and Abraham (1973). For the River Rhine between Ruhrort and the border between Germany and the Netherlands, Van Mazijk (1987) found $\alpha = 0.54$ to 0.81.

When an effluent is released at one point in a vertical then mixing over the vertical takes place initially. If that situation is reached then in the second place there is mixing across the width. This is illustrated in Fig. 2.10 where a release is induced at the bank of a river. Complete mixing can be defined if the concentration at the opposite bank reaches 99% of the width-averaged concentration. The length L required for this complete mixing is given by

$$L = 0.55\frac{\overline{u}B^2}{K_2} \qquad (2\text{-}65)$$

This involves the mixing of a conservative substance. For non-conservative substances with a small relaxation time (t_r) it is possible that complete mixing does not take place at all.
The value of L is of importance for two problems:

- If a dispersion problem is of importance for $x >> L$ then instead of the 2-D model it is possible to use the one-dimensional model of Eq. (2-59).
- 'Complete mixing' is a prerequisite for the application of a 'dilution method' to measure the discharge of a natural stream (see Sub-section 4.4.2)

Although the dispersion coefficients may in practice vary in space and time, the inaccuracy by which these coefficients can be established in a particular case justify that they are taken to be constant. This has the advantage that provided that also $\overline{u}$ = constant, analytical solutions of the basic dispersion equations can be reached.

As the basic equations are linear in the dependent variable ($\overline{\phi}$) the principle of superposition can be applied.

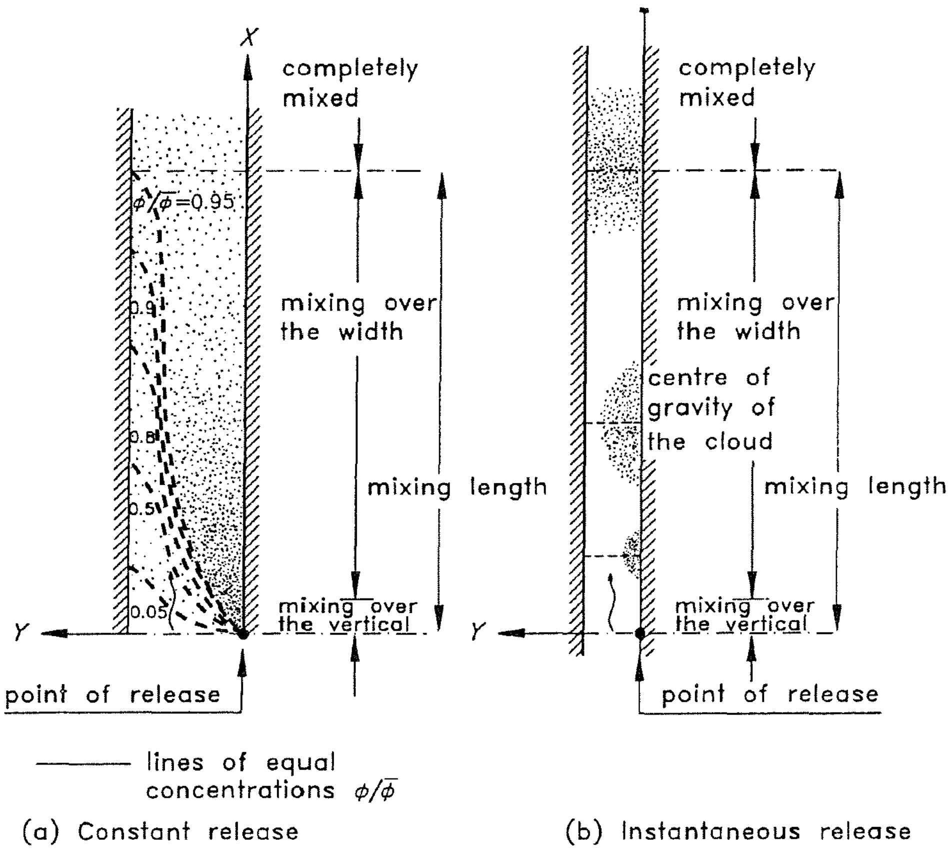


Fig. 2.10 Two phases in the mixing process

2.5.3 Dead zones

The dispersion model discussed so far (the 'Taylor model') gives a prediction that often differs from measurements (Fig. 2.11).

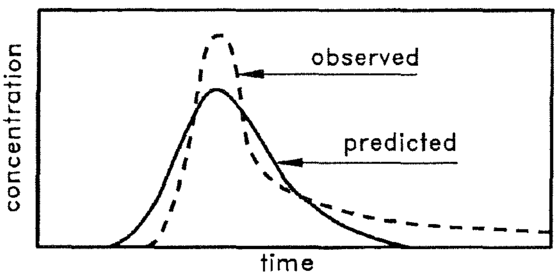


Fig. 2.11 Difference between measured and predicted concentrations (Thackston and Schnelle, 1970)

These differences are partly caused by convective and dispersive phenomena near the point of release ('near-field' effects). The main cause, however, is an exchange of dissolved matter between the mainstream of the river and 'dead zones' adjacent to the river. These dead zones may involve the areas between groynes or parts of the river where the water is nearly stagnant due to bed- and wall roughness, meandering, vegetation, etc. The presence of dead zones means that the convective velocity of the dissolved matter (c_s) becomes smaller than the mean flow-velocity (u). (Valentine and Wood, 1977; Van Mazijk and Verwoerdt, 1989).

$$c_s = \frac{\overline{u}}{1+\beta} \qquad (2\text{-}66)$$

For instance, when the dead-zone cross-section (A_b) occupies 10% of the mainstream cross-section (A_s) then as $\beta = A_b/A_s$ the arrival time of the dissolved matter will be 10% later. For rivers without groynes values of β = 0.03 to 0.05 are found (Nordin and Troutman, 1980; Purnama, 1988).

2.6 Ice problems

2.6.1 General

Ice in rivers is only of importance in certain areas, mainly rivers in the northern part of the Northern Hemisphere. Consequently, ice research on rivers is being performed by a limited number of researchers.

Much information on ice research can be found in the proceedings of the regular symposia organized by the Section on ice research and engineering of the International Association for Hydraulic Research (IAHR). The *Journal of Hydraulic Research* of the IAHR, Vol. 28, No. 6 of 1990 was a special issue dealing almost entirely with ice problems.

Since 1987 when it was first published, the *Journal of Cold Regions Engineering* of the ASCE has also been a potential source of information.

2.6.2 Formation and transport of ice on rivers

The processes in rivers with respect to the transport of water and sediment become even more complicated when ice is involved. Ashton (1986) has summarized our knowledge on ice problems in rivers.

Shen *et al.* (1984, 1988) published a number of papers on ice problems. Shen *et al.* (1990) treat the coupled ice transport and channel flow in a mathematical formulation. Obviously a number of parameters (coefficients) have to be established experimentally as for other fluvial processes.

Not only is laboratory research necessary for studying details of the processes involved, but also entire processes are studied experimentally in order that a conceptual mathematical model can be constructed. A good example is the laboratory study by Ettema (1990) on jam initiation. Figure 2.12 shows his illustrative sketch using floating beads to reproduce the ice-phase in the process.

Laboratory studies and mathematical formulation alone can never lead to a model that can be applied to the field. Field observations are also necessary. Matoušek (1990) describes field studies on ice jams in flood control. Starösölszky (1990) gives another example of field observations.

Attention to the mobility of the river bed and hence to the influence of an ice cover in the morphological processes is given by Lau and Krishnappan (1985) and by Krisknappan (1983). An illustrative example of the practical relevance of ice on the morphological processes in rivers is taken from Maas and Roukema (1991).

There is a story from the past of ice management on the Rhine River in the Netherlands when it was once decided to break an ice cover (obviously from the downstream side). It was found that the ice-breakers could not reach the site due to the sedimentation downstream of an ice cover. Figure 2.13 illustrates this. It is a product of the author's 1-D mathematical model. It shows the substantial reduction of the available depth for navigation.

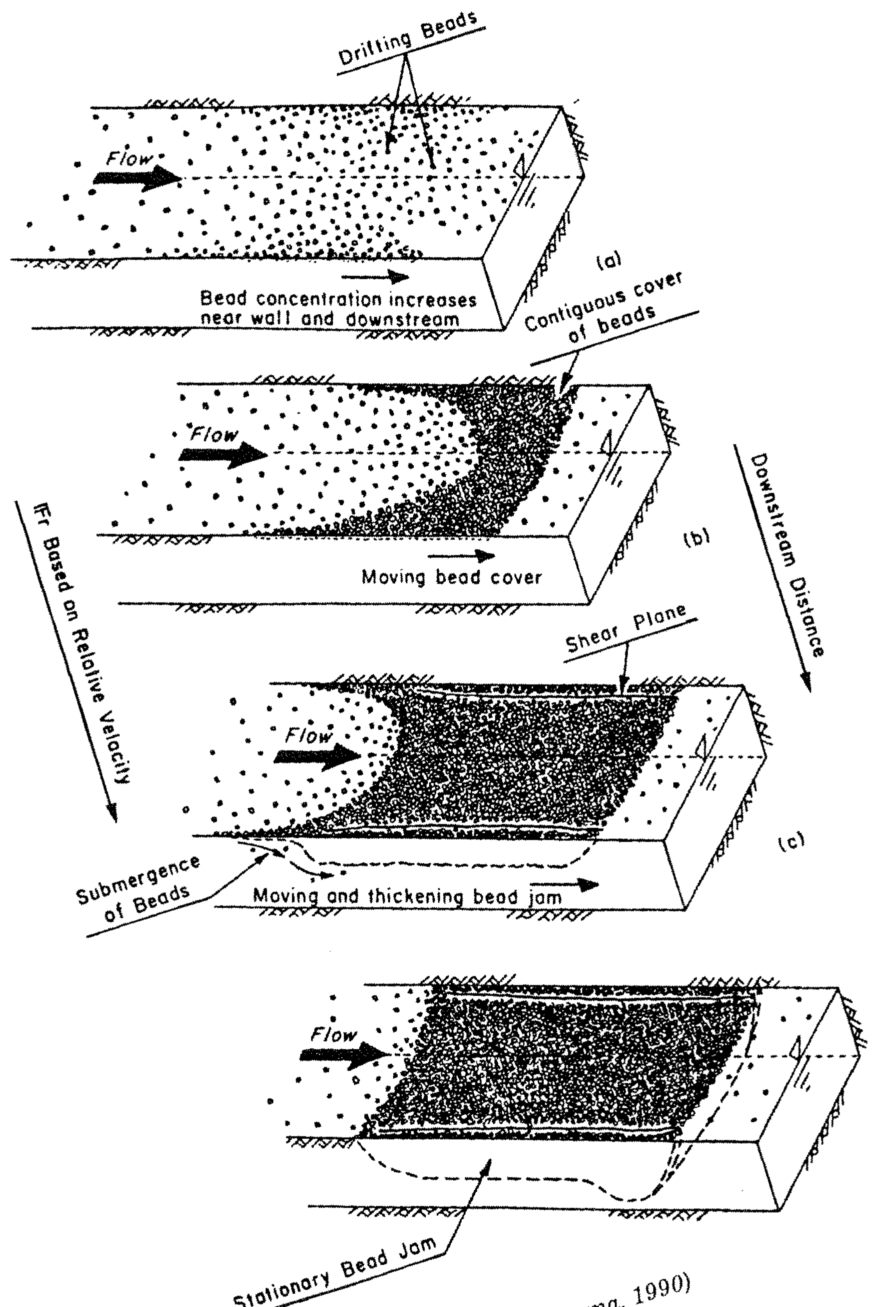


Fig. 2.12 Ice jam initiation (Ettema, 1990)

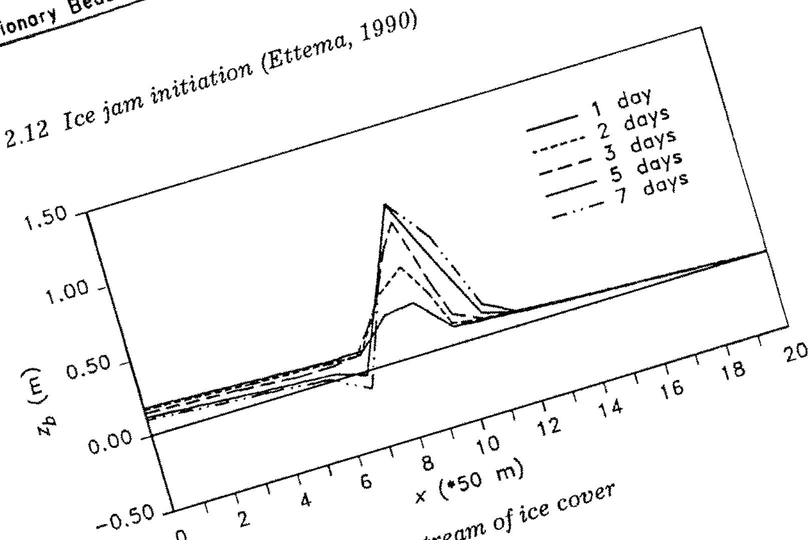


Fig. 2.13 Aggradation downstream of ice cover

3. Scale models

3.1 General

It should be recalled that with a scale model a problem is solved by means of measurement of the relevant parameters. This implies two things:

(i) the scale model has to be reliable, i.e. the fluvial processes have to be *similar* in model and prototype;

(ii) instrumentation has to be available to measure with sufficient accuracy.

Prior to the design of the various possible scale models for fluvial processes some general remarks are made here as to how to achieve sufficient similarity.

The scale of a parameter (x) is defined by

$$\text{scale of } x = n_x = \frac{x_{\text{prot.}}}{x_{\text{model}}} = \frac{x_p}{x_m} \qquad (3\text{-}1)$$

The scales of all parameters involved cannot be selected at will; they are determined from *scale relations*. These are derived from the hydrodynamic processes involved.

The derivation of scale relations can be carried out using three general rules:

Scale of a product: the scale of a product of two parameters is equal to the product of the scales of these two parameters.

Example: as by definition $L = u.t$ this gives by simple algebra the scale relation:

$$n_L = n_u \cdot n_t \qquad (3\text{-}2)$$

Scale of a sum: the scale of a sum of two parameters is equal to the scales of these parameters (only if they are equal).

Example: For free surface flow the energy head (H) is equal to the sum of the piezometric head (h) and the velocity head ($s = u^2/2g$)

$$H = h + s \qquad (3\text{-}3)$$

Using Eq. (3-1), simple algebra gives

$$n_H = \frac{n_h + n_s\{S_m / h_m\}}{1 + \{S_m / h_m\}} \qquad (3\text{-}4)$$

If the selection is made $n_s = n_h$ then $n_H = n_h$. Otherwise *scale effects* are present, i.e. the value of n_H varies in the model. The selection $n_s = n_h$ implies as $h = a + z$ that the *Froude condition* is fulfilled: the velocity scale has to be selected equal to the square root of the vertical scale.

Scale of a function: the scale of a function of a parameter is unity if the parameter (i.e. the argument of the function) is selected equal in model and prototype.
Example: Note that the argument of a function has always to be dimensionless. For instance the Meyer-Peter and Mueller (1948) sediment transport formula reads

$$\phi_s = \alpha\{\theta - \beta\}^{\frac{3}{2}} \tag{3-5}$$

Applying the rule of a function gives (α and β being constants):

$$n_{\phi_s}^{\frac{2}{3}} = \frac{n_\theta - \beta\,\theta_m^{-1}}{1 - \beta\,\theta_m^{-1}} \tag{3-6}$$

Hence n_{ϕ_s} is only constant in the model if $n_\theta = 1$ is selected. Otherwise scale effects are present, i.e. n_{ϕ_s} varies in the model.
It appears from the above examples that two types of scale *relations* exist

- *Scale laws* are scale relations that *must* be fulfilled. They usually come from equations that imply a definition. Equation (3-2) is an example.
- *Scale conditions* are scale relations that *have to be* fulfilled in order to avoid scale effects. Equations (3-4) and (3-6) are examples.

The more complicated the fluvial process is, the more scale relations are found. If all scale relations are fulfilled usually only the *trivial solution* of a full-scale model seems possible. However, in deviating to a certain extent from the scale *conditions* a set of scales can be obtained. Obviously the scale effects induced have to be kept to an acceptable minimum.

Preferably the expected scale effects have to be quantified, and this implies that preference has to be given to derive the scales from the hydrodynamic equations rather than by dimensional analysis. Examples of the first method are given in the following sections of this chapter.

Dimensional analysis consists of deriving dimensionless products (Π) from the parameters involved in the physical process described. If a physical phenomenon is described by parameters p_i with $i = 1,\ldots, n$ and if m elementary quantities are involved then $n - m$ dimensionless products result.

In river problems there are usually three elementary quantities (*mass*, *length* and *time*) thus $n - 3$ values of Π result. The technique for a systematic derivation of relevant dimensionless products Π can, for example, be found in Langhaar (1956).

The book on scale models by Yalin (1971) uses mainly dimensional analysis to arrive at a consistent set of model scales. With this method, however, no insight is gained into the magnitude of possible scale effects. This can be demonstrated by means of an example. Consider the flow over a sill. The derivation of the scale condition from the basic hydrodynamic equation (in this case: Bernoulli law) leads to Eq. (3-4). From this equation the Froude condition can be found. Moreover, if for some reason or another $n_{Fr} \neq 1$ is selected then Eq. (3-4) gives insight in the magnitude of possible scale effects. This analysis uses *a priori* the notion that viscous effects can be neglected as long as the model is not too small.

If the same problem is treated with dimensional analysis, it is found that the dimensionless products *Fr* and *Re* are relevant. If for the same reason as argued above *Re* is supposed to be irrelevant, then only the condition $n_{Fr} = 1$ results. No insight in possible scale effects due to $n_{Fr} \neq 1$ is attained. The wish to adopt $n_{Fr} \neq 1$ may be present if not only the water movement but also the sediment movement has to be reproduced (Sub-section 3.3.3).

There are books dealing specifically with scale models (ASCE, 1963; Ippen, 1968; Yalin, 1971; Ivicsisc, 1975; Kobus, 1980; Sharp, 1981). In Jansen *et al.* (1979) attention is paid to both scale models and numerical models.

3.2 Water movement

In river problems for which only the water movement has to be reproduced, the scaling procedures are relatively simple. The hydrodynamic equations are relatively well-known (Section 2.2). However, if the water movement is not the only process that has to be reproduced, very accurate reproduction is necessary. For instance, in mobile-bed river models both water movement and sediment movement have to be reproduced at the same time. In bends of meandering rivers the helical flow has a large influence on the bed topography. A good reproduction of the helical flow is therefore essential for obtaining similarity with respect to the bed topography.

For many river problems the horizontal dimensions involved are much larger than the vertical dimensions. This will lead to unrealistic scale models if the vertical scale (n_a) is selected equal to the length scale (n_L). These models are therefore usually *distorted* ($n_L > n_a$). This topic has been given some attention below.

The scaling for the water movement can best be understood by analysing the one-dimensional long-wave equation. For a wide river with a constant width the momentum equation reads

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + g\frac{\partial a}{\partial x} + g\frac{\partial z_b}{\partial x} = -g\frac{u|u|}{C^2 a} \tag{3-7}$$

The continuity equation is

$$\frac{\partial a}{\partial t} + u\frac{\partial a}{\partial x} + a\frac{\partial u}{\partial x} = 0 \tag{3-8}$$

Applying the *rule of a sum* gives the condition that all terms in one equation have to be reproduced on the same scale. Equalling the scales of the first two terms of Eq. (3-8) gives the scale *law* that was deduced in Section 3.1

$$n_L = n_u \cdot n_t \tag{3-9}$$

In addition, from Eq. (3-7) two scale *conditions* can be derived. From the second and the third term the *Froude condition* is obtained

$$n_u = \sqrt{n_a} \tag{3-10}$$

From the second and the fifth term the *roughness condition* is obtained

$$n_C^2 = \frac{n_L}{n_a} = \text{'distortion'} = \text{r} \tag{3-11}$$

In addition, the third and the fourth term give the *geometric condition* $n_a = n_{z_b}$.

Remarks

(i) The analysis of Eq. (3-7) leaves open the possibility of *distorted models*. This is logical since the basic assumption underlying the long-wave equation is the presence of a hydrostatic pressure distribution (i.e. the flow lines are not curved in the vertical plane).

(ii) Above it is assumed implicitly that the flow is sufficiently turbulent in the model. Actually the main reason for distortion is to obtain turbulent flow.

(iii) The distortion leads to the fact that geometric *similarity* is replaced by geometrical *affinity*.

(iv) The distortion leads to scale effects in the flow field. For instance a local depression in the bed level may have such a shape that the flow lines follow the bed. Distortion, however, may lead to the situation that flow separation at the upstream side of the depression takes place. Another example can be taken from the velocity distribution in an axi-symmetrical bend. For a logarithmic velocity distribution in the vertical this distribution is (Jansen *et al.*, 1979, p. 61)

$$\frac{R}{a} \cdot \frac{v}{U} = \frac{1}{\kappa^2}\left[F_1(\eta) + \frac{\sqrt{g}}{\kappa C}F_2(\eta) - 2 + \frac{2g}{\kappa^2 C^2} - \frac{\sqrt{g}}{\kappa C}\left\{1 - \frac{\sqrt{g}}{\kappa C}\right\}\ln \eta\right] \tag{3-12}$$

in which R is the radius of curvature, v the radial component of the flow velocity at the distance z from the bed and $\eta = z/a$. The mean flow-velocity in the vertical is U. Examination of Eq. (3-12) leads to the conclusion that each term is only to be reproduced on the same scale for $n_C = 1$. Together with the roughness condition (Eq. 3-11) this leads to the requirement of an undistorted model.

(v) For two-dimensional horizontal flow problems for which the hydrostatic pressure distribution holds, the same scale relations are found as following from Eqs. (3-7) and (3-8). For these problems the roughness condition of Eq. (3-11) is not only a requirement for the correct reproduction of the water levels but also for the curvature of the stream lines (Bijker *et al.*, 1957).

3.3 Sediment transport and morphology

3.3.1 Introduction

Mobile-bed river models have additional scale relations to the ones present for water movement only. A typical example of the conflict that may arise involves the reproduction of the (alluvial) roughness. In fixed-bed models the bed roughness can be adjusted by adding artificial roughness to the bed in order to fulfil the roughness condition (Eq. 3-11). For mobile-bed models, however, the roughness of the model bed is a result of the flow velocity and the selected bed material. Further interference is difficult if not impossible. Numerical models do not have this difficulty.

In this section some scaling methods are first treated in principle and a comparison is then given (Sub-section 3.3.6). These scaling methods are based on a number of (limiting) assumptions:

(i) one-dimensional steady flow of water and sediment;

(ii) non-erodible banks (constant width);

(iii) moderate Froude numbers in model and prototype;

(iv) turbulent flow in prototype and model;
(v) neglect of the spatial variation of the roughness coefficient.

3.3.2 Einstein-Chien method

The method given by Einstein and Chien (1956) is based on the hydrodynamic description of the fluvial process involved. Basically it is a one-dimensional approach. There is hardly any further literature on this method. Shen (1979) is only a repetition. No practical applications have been found by the writer. Einstein and Chien (1956) only contains a design of a fictitious scale model of the Big Sand Creek in Mississippi. However, an outline of the method is given here, to enable one to make a comparison with other available methods.

The Einstein-Chien method consists of a set of relevant scale relations including factors Δ which may be different from unity (for $\Delta = 1$ the scale relation is fulfilled exactly). The following scale relations are mentioned:

(i) The '*friction criterion*' is derived from the generalised Manning equation

$$u = \frac{c\sqrt{g}}{D^n} i^{\frac{1}{2}} a^{\left(\frac{1}{2}+m\right)} \tag{3-13}$$

in which c and m are 'constants'.
From Eq. (3-13) is derived

$$n_u^2 \cdot n_i \cdot n_a^{-1-2m} \cdot n_D^{2m} \cdot n_c^{-2} = \Delta_u \tag{3-14}$$

(ii) The '*Froude criterion*' gives

$$n_u \cdot n_a^{-\frac{1}{2}} = \Delta_{Fr} \tag{3-15}$$

(iii) The '*sediment-transport criterion*' gives

$$n_s = n_\Delta \cdot n_D^{\frac{3}{2}} \tag{3-16}$$

(iv) The '*laminar-sublayer criterion*' leads for a relatively wide river to

$$n_D \cdot n_a^{\frac{1}{2}} \cdot n_i^{\frac{1}{2}} = \Delta_\delta \tag{3-17}$$

(v) A *tilted model* is obtained for $\Delta_N \neq 1$ in the equation

$$n_i \cdot n_L \cdot n_a = \Delta_N \tag{3-18}$$

In studying the method once again, the writer found (De Vries, 1982b) a remarkable error. Equation (3-14) suggests that basically the selection $\Delta_u \neq 1$ is possible. However, this is an impossibility since Eq. (3-14) with $\Delta_u = 1$ is a scale *law*. Hence deviation from $\Delta_u = 1$ is not possible.

In their 1956 paper the authors did not use the opportunity of taking the remaining Δ-values (notably Δ_{Fr}, Δ_δ and Δ_N) unequal to unity. In their fictitious example this was possible by a theoretical selection for the scale of the relative density ($n_\Delta = 0.041$).

The Einstein-Chien method will be considered in Sub-section 3.3.6 where various possible methods are compared.

3.3.3 Delft method

The method applied by Delft Hydraulics has been described over the years in a number of publications (Bijker *et al.*, 1957; Bijker, 1965; Jansen *et al.*, 1979; Struiksma, 1986). The method can be outlined in principle as below.

For the basic equations for water movement the *Froude condition* (Eq. 3-10) and the *roughness condition* are derived (Sub-section 3.2.2.). The morphological scaling is based on the notion that sediment transport formulae have the general shape.

$$\phi_s = f(\theta) \tag{3-19}$$

Applying the *rule of a function* (Section 3.1) it is then argued that the selection n_θ *equals unity* makes also the scale of ϕ_s equal to unity. This is similar to what was proposed by Einstein and Chien (1956), as can be seen from Eq. (3-16).
From the requirement $n_\theta = 1$ follows

$$n_u^2 \approx n_\Delta \cdot n_D \tag{3-20}$$

For $n_\Delta = 1$ this demonstrates the conflict that may arise from the *two* conditions for selecting the velocity scale:
(i) The *Froude condition* requires $n_u^2 = n_a$;
(ii) The *transport condition* requires $n_u^2 \approx n_D$.
For coarse gravel, etc. the two conditions can only be fulfilled at the same time by selecting $n_D = n_a$. However, for fine gravel and sand this would lead to an unrealistically small value of the grain size in the model, in spite of selecting $n_\Delta > 1$ (i.e. light material).

Preference is then given to fulfilling Eq. (3-20). The thus-derived velocity scale is called the *ideal velocity scale*. Hence $n_u^2 \neq n_a$ is attained leading to errors in the reproduction of the waterlevels.

These errors are corrected by *tilting* the model. This means nothing other than constructing the fixed parts of the river model according to a *sloping reference level* (slope i_t). Apparently

$$i_t = i_m - r\, i_p \tag{3-21}$$

or after some rearrangement

$$i_t = i_p\left[\frac{n_C^2 \cdot n_a}{n_u^2} - r\right] \tag{3-22}$$

Note that $i_t = 0$ if both Froude and roughness conditions are fulfilled.

As the value of i_t has to be known *before* the construction of the scale model, it has to be based on an *estimate* of the roughness of the model (C_{me}). This implies that in spite of tilting an error (Δh_m) in the waterlevel may be present at both ends of the model. If the correct waterlevel is introduced in the middle of the model then (model length L_m).

$$\Delta h_m = \tfrac{1}{2} L_m\left(i_{ma} - i_{me}\right) \tag{3-23}$$

in which i_{ma} and i_{me} represent the *actual* model slope and the *estimated* model slope respectively.

The relative error $\Delta h_m / a_m$ can, after some algebra, be expressed as

$$\frac{\Delta h_m}{a_m} = \tfrac{1}{2}\left[\frac{Li}{a}\right]_p * \left[\frac{n_a}{n_u^2}\right] * \left[\frac{C_{me}^2}{C_{ma}^2} - 1\right] \tag{3-24}$$

Hence this error depends on:

(i) the dimensionless length $\Lambda_p = L_p\, i_p/a_p$ of the prototype reach that has to be reproduced;

(ii) the degree to which $n_u^2 < n_a$;

(iii) the difference between C_{ma} and C_{me}.

The latter depends on the quality of the available alluvial roughness data at the design stage of the model.

In practice the scaling procedure consists of selection from available bed material (prototype data to be known) the scale combination that leads to the smallest scale effects.

Moreover the morphological time scale (n_{tm}) is also taken into consideration. The value of (n_{tm}) follows from the continuity equation for the sediment:

$$n_{tm} = \frac{n_a \cdot n_L}{n_s} = \frac{n_a \cdot n_L}{n_D^{\frac{3}{2}} \cdot n_\Delta^{\frac{1}{2}}} \tag{3-25}$$

The optimal choice of n_{tm} is between two extremes: (i) too small values lead to a 'slow' model so the tests may take too much time, and (ii) too large values lead to a 'quick' model; there may be not sufficient time to measure bed levels during time-depending morphological processes.

Remarks

(i) The selection of the *ideal velocity scale* may lead to deviation from the Froude condition and hence to an incorrect local reproduction of the water *levels* in spite of the tilt. However, the water *depths* are reproduced correctly, as can be demonstrated as follows:

With $s = mu^n$, thus $ds/du = ns/u$ and assuming steady flow or $Q = Bua$ = constant and $S = Bs$ = constant, for non-uniform flow because of $B = B(x)$ it follows that

$$Bu\frac{\partial a}{\partial x} + Ba\frac{\partial u}{\partial x} + ua\frac{\partial B}{\partial x} = 0 \tag{3-26}$$

and

$$B\frac{\partial s}{\partial x} + s\frac{\partial B}{\partial x} \approx B\frac{ds}{du}\frac{\partial u}{\partial x} + s\frac{\partial B}{\partial x} = 0 \tag{3-27}$$

From Eqs. (3-26) and (3-27) it is possible to eliminate $\partial u/\partial x$ leading to

$$Bu\frac{\partial a}{\partial x} + Ba\left[\frac{-u}{Bn}\frac{\partial B}{\partial x}\right] + ua\frac{\partial B}{\partial x} = 0 \tag{3-28}$$

or

$$\frac{\partial a}{\partial x} = \frac{1-n}{n}\cdot\frac{a}{B}\cdot\frac{\partial B}{\partial x} \tag{3-29}$$

Note that Eq. (3-29) is independent of the flow velocity. For a correct reproduction of $\partial a/\partial x$ (and hence of a) the scale of the factor $(1-n)/n$ should equal unity. This is, however, exactly the basis of the concept of the *ideal velocity scale,* i.e. in the (dimensionless) graph $\phi_s = f(\theta)$the average value of ϕ_s and hence of θ is at the same point for the prototype and the model. Hence the scale of n equals unity; the same holds then for the factor $(1-n)/n$.

(ii) The morphological timescale will in general differ from the timescale of the water movement. From Eq. (3-25) it follows that only in the case of an undistorted model ($n_a = n_L$) for which is selected $n_D = n_L$ (only possible for very coarse bed material in the prototype) and for $n_\Delta = 1$ the result is $n_{tm} = n_L^{\frac{1}{2}}$. Hence $n_{tm} = n_{tw}$ if in addition the ideal velocity is equal to the velocity scale following from the Froude condition.

(iii) Most of the analysis given above is based on one-dimensional approaches for water and sediment movement. For the morphology of a bend in a meandering river a two-dimensional approach in necessary. An introduction is given by Struiksma (1986), which shows the crucial role played by alluvial roughness. On the one hand the roughness condition has to be fulfilled to reproduce the flow pattern correctly. However, given the relatively large roughness of the model bed (leading to $n_C > 1$) distortion is necessary. This leads on the other hand to errors in the 2-D bed topography.

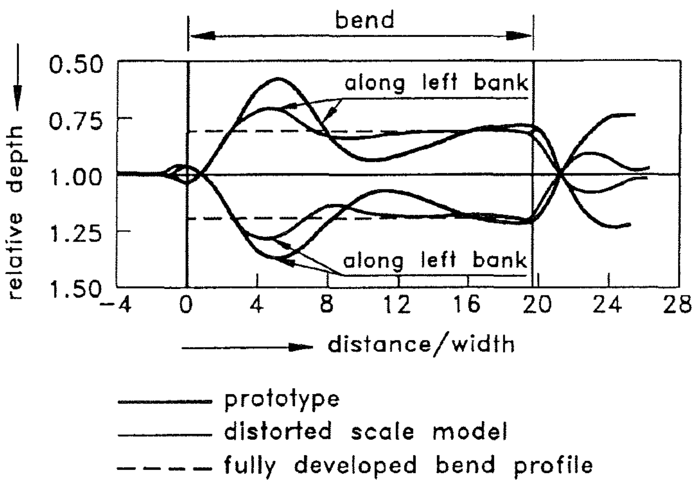


Fig. 3.1 Scale effect in bend topography (Struiksma, 1986)

Figure 3.1 shows the tendency of the scale effects present due to distortion. The (circular) bend is situated between two straight reaches. The longitudinal profiles along both banks are given. The tendency is that the location and the magnitude of the deepest (shallowest) point at the outer (inner) bend are not correct.
This has two consequences:

- for *navigation* the scale model will be too optimistic with respect to the depth available at an *inner* bend;
- For *bank protection works* the scale model is too optimistic with respect to the level at which the toe of the protection at the *outer* bend can be placed.

(iv) Another aspect that is given attention by Struiksma (1986) for the 2-D bed topography is the important role played by the exponent n in the equation $s = mu^n$. It stresses the remark made under (i) that the value of n should be equal in model and prototype, as follows from Eq. (3-29) for the one-dimensional approach.

3.3.4 Chatou method

The method applied at the Laboratoire national d'hydraulique (Chatou, France) is, for example, described by Chauvin (1962) and Ramette (1981). A summary of the method is found in Abdalla (1990).

The scale relations for the water movement follow from the equation for steady uniform flow (Manning-Stickler) and the definition of the Froude number.

For the sediment movement the scaling is not only based on $n_\theta = 1$ from $\phi_s = f(\theta)$ but tries to take into account the *type* of bed form (plane bed, ripples or dunes). Consequently the parameter $Re_* = u_* \, D/v$ also plays a role, since Ramette(1981) shows that the bedform classification can be indicated in a relationship between θ and Re_*. In the θ-Re_* plot lines of equal D_* can be drawn with

$$D_* = D\left[\frac{g\Delta}{v^2}\right]^{\frac{1}{3}} \tag{3-30}$$

The scaling procedure is best be illustrated by Fig. 3.2. The following remarks can be made (Abdalla, 1990):

(i) The three similarity equations in Fig. 3.2 contain five unknown scales (n_L; n_D; n_Δ; $r = n_L/n_a$ and n_{Fr}) in terms of the scale of n_{Re*}. This means that two unknowns can be selected freely (two degrees of freedom).

(ii) Where n_{Fr} is rigidly held to unity, the number of unknowns is reduced to four and there is only one degree of freedom. In practice this one degree will be taken by n_L, following from the length (L_p) to be reproduced and the length (L_m) dictated by the space available.

3.3.5 Wallingford method

The scaling method used at Hydraulic Research Ltd, Wallingford, U.K., as described by White (1982), is based on 'rational relationships for width, depth and slope of an equilibrium channel'.

(i) The Ackers-White (1973) sediment transport formula

(ii) The White, Paris and Bettess (1980) roughness predictor

(iii) 'The assumption that equilibrium occurs when the sediment transport capacity is at a maximum' (White, 1982)

(iv) The continuity equation for water ($Q = u \cdot A$)

The method is outlined in White, Paris and Bettess (1981a, b). White (1982) claims the method to be very flexible as the model discharge, bed material and longitudinal slope can be selected freely.

The Wallingford method differs essentially from the three methods quoted in the earlier sub-sections. The writer feels that critical remarks can be made with respect to the basic assumption (iii) quoted above.

(i) Is a river ever in equilibrium? The timescales quoted in Table 2.3 suggest that due to natural causes and human interference a river will hardly ever be in equilibrium.

(ii) The assumption is based on the belief that nature maximises the sediment transport, or what is basically the same : minimises the 'stream power'. The writer is not a believer of this concept. There are examples which show that nature 'does not work that way'. One example is be given here: the human liver normally has a capacity which is far beyond the capacity needed.

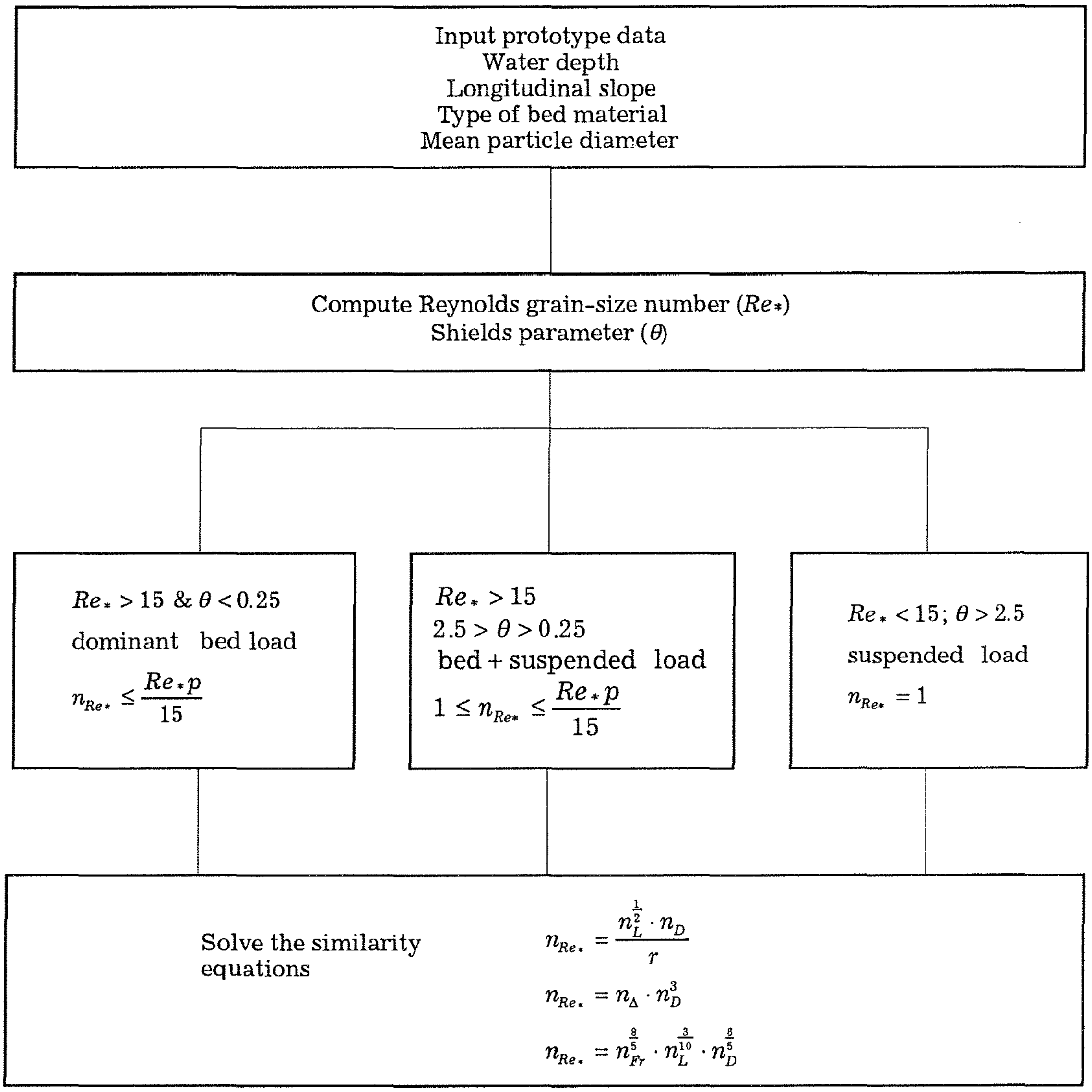


Fig. 3.2 Scale selection, Chatou method (after Abdalla, 1990)

Most likely, the flexibility claimed by White (1982) of the large freedom in selecting scales is due to the fact that the physical basis of the scaling is incomplete. The most striking omission regards the reproduction of the water movement in one and two (horizontal) dimensions. In other words the *roughness condition* is missing.

In White (1982) the design of a scale model for the Sabi River (Zimbabwe) is treated. The model has been given a distortion $r = 6$ ($n_L = 120$; $n_a = 20$) whereas from the roughness data given $r = 1.3$ should have been selected if the *roughness condition* had been applied. In addition, the remark can be made that with a distortion $r = 6$ the bed topography cannot be locally correct because of the angle of repose being the same in this model (using sand) as in the prototype.

Application of the Delft method means that, most likely for a scale model of the Sabi River, lightweight bed material would have been a must.

3.3.6 Comparison of scaling methods

A complete comparison of the four scaling methods is out of question financially. It would require the construction and operation of four scale models for each prototype river selected (preferably more than one).

Abdalla (1990) therefore as a substitute compared the four methods in a desk study. He considered three rivers:

(i) the River Waal, in the Netherlands (Struiksma, 1980);
(ii) the River Nile, near Beni-Mazar; and
(iii) the River Sabi, Zimbabwe (White, 1982).

There were sufficient data to apply scaling methods. For each, river sand ($n_\Delta = 1$) and bakelite ($n_\Delta = 4$) were used for the scale model. Since the Einstein-Chien method gives many possible scale combinations, it is impossible to select one combination without an additional criterion. Abdalla therefore added the roughness condition (Delft method) to the Einstein-Chien method.

In the second part of his study Abdalla (1990) concentrated on the River Waal for which sufficient detailed (bed-level) measurements were available. Applying a simplified 2-D numerical model (calibrated for the prototype), he carried out morphological computations using scale-model data. Hence for four scaling methods and two bed materials (sand and bakelite) a total of eight computations were performed.

The 2-D numerical model was simplified as in Crosato (1990), but with a fixed bank as present in the prototype.

TABLE 3.1 River data († = predicted)

		R. Waal	R. Nile	R. Sabi
length	m	10 000	6000	4100
depth	m	5.5	2.97	2.26†
width	m	240	477	320
discharge	m^3/s	1500	1200	1000
grain size	mm	1.5	0.34	1
velocity	m/s	1.14	0.82	1.39†
slope	-	$1.2 * 10^{-4}$	$0.85 * 10^{-4}$†	$9.6 * 10^{-4}$†
C-value	$m^{\frac{1}{2}}/s$	47	52	30†

In Table 3.1 some general data are given for the three rivers. For the River Sabi some of the data were predicted with the regime relations in White *et al.* (1981a,b).

Abdalla used these relations to predict similar river data for the river Nile near Beni-Mazar. The values predicted ($a = 8.4$ m; $B = 160$ m; $u = 1$ m/s and $C = 42$ $m^{\frac{1}{2}}/s$) seem to differ significantly from the values measured (Table 3.1). Consequently, the River Sabi data most likely do not have the accuracy of measured data.

Abdalla (1990) computes longitudinal profiles of the river bed as a pertubation of the mean bed-level. In Fig. 3.3 the reproduction by the computations of the prototype measurements are given. Figures 3.4 to 3.7 compare the prototype simulation with the computer simulations for the (eight) scale models.

The figures show that the (extended) Einstein-Chien method and the Delft method give *in this case* the best results. The Chatou method merits third place, and the Wallingford method gives the largest difference between 'model' and prototype.

The reason for the deviations found with the Wallingford method may be the incomplete physical bases (Sub-section 3.3.5). It is also possible that the fact that the River Waal has fixed banks is one of the causes of the differences. The 'regime concepts' deal with erodible banks.

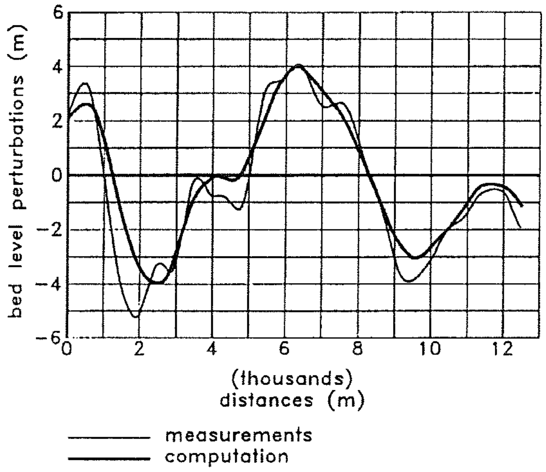


Fig. 3.3 Measured and computed bed level (prototype)

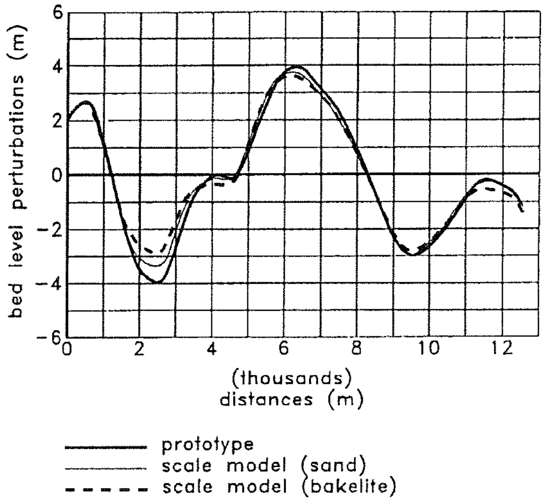


Fig. 3.4 Einstein-Chien method

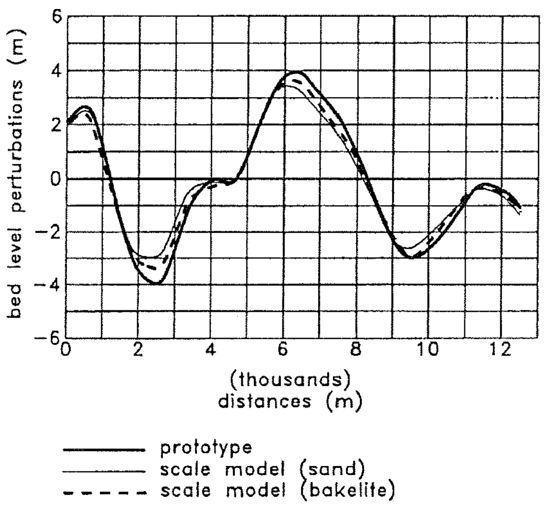


Fig. 3.5 Delft method

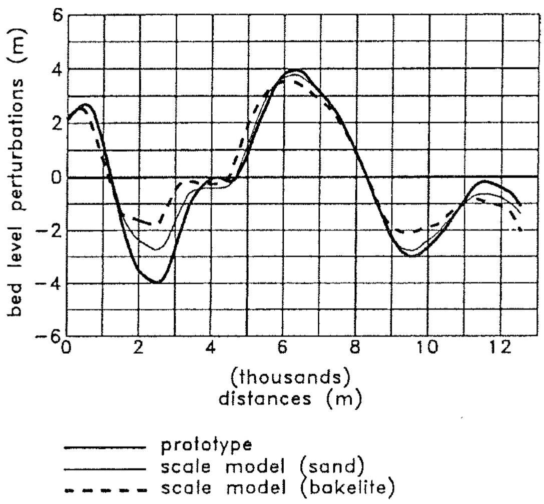


Fig. 3.6 Chatou method

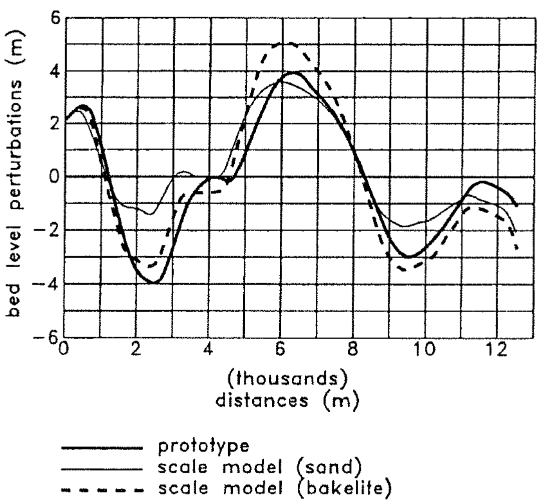


Fig. 3.7 Wallingford method

3.3.7 Scaling of suspended load

The methods discussed in the previous sub-sections are based on steady uniform flow for water and sediment. For non-uniform flow some additional remarks have to made, in case a substantial part of the bed-material is transported in suspension.

Three scale relations can be derived for such a case:

(i) Similarity with respect to the vertical concentration distribution for steady uniform flow is obtained if the Rouse-parameter (Z) is equal in model and prototype. From Eq. (2-21) follows for $n_Z = 1$ the requirement (scale condition)

$$n_{W_s} = n_{u*} \tag{3-31}$$

(ii) For steady non-uniform flow there follows from Eq. (2-26) the condition

$$n_{au} = n_{LW_s} \tag{3-32}$$

Introducing the distortion (r):

$$n_{W_s} = n_u \cdot r^{-1} \tag{3-33}$$

(iii) For non-steady flow the following scale condition follows from Eq. (2-26) and Eq. (3-2):

$$n_{W_s} = n_a \cdot n_t^{-1} = n_a \cdot n_L^{-1}\, n_u = n_u \cdot r^{-1} \tag{3-34}$$

Obviously, (ii) and (iii) lead to the same scale condition. On the other hand (i) and (ii) are only identical if the scale model is undistorted and if the roughness condition is fulfilled. Hence the scale model of a sand trap, which has to be undistorted, can be similar to the prototype. This holds even for $n_C \neq 1$, since in this case the roughness of the bed seems not to determine the flow pattern to a large extent.

However, a river model with substantial suspended sediment cannot obtain complete similarity. Such a model has to be distorted because of the large horizontal dimensions compared to the vertical dimensions. A logical selection of scales follows from Eq. (3-33). In this case Eq. (3-31) reads as $u/u_* = C/\sqrt{g}$ and if the roughness condition is fulfilled:

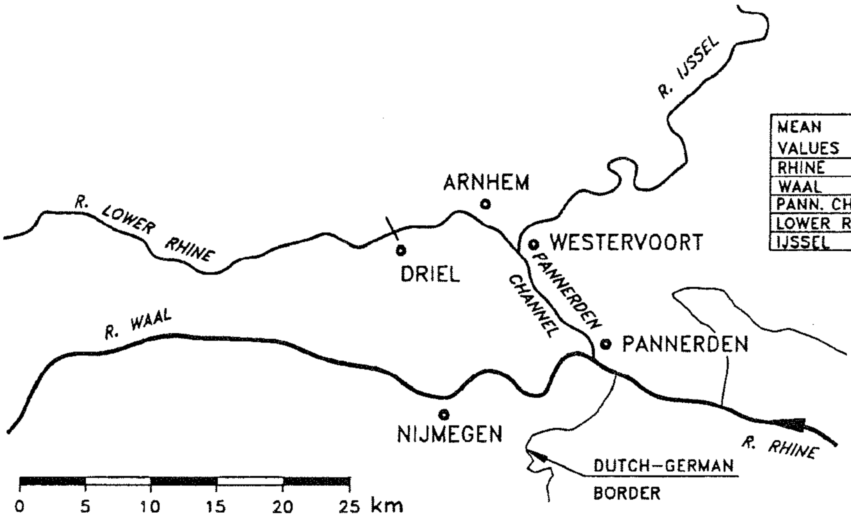
$$n_{W_s} = n_{u*} = n_u \cdot n_C^{-1} = n_u \cdot r^{-\frac{1}{2}} \tag{3-35}$$

Logically, as $r > 1$ Eqs. (3-33) and (3-35) are conflicting. Eq. (3-33) implies that the condition of Eq. (3-31) is violated. The tendency is that the selected W_{sm} is *too large* to obtain similarity for the vertical concentration distribution.

3.3.8 Scaling of sediment mixtures

In the previous sub-section the scaling of the bed material was carried out under the implicit assumption that the bed material is (nearly) uniform. When this assumption is not justified, the modelling becomes more complicated. The first question is: 'What is *the* grain-size distribution of the prototype?' The second question concerns the reproduction of this distribution in the scale model. The possible approach to these questions can best be illustrated by means of an example. Figure 3.8 shows the Rhine branches in the Netherlands.

The system contains two bifurcations. At a bifurcation grain sorting takes place. Careful study is needed to demonstrate this, since sorting also takes place due to bed forms and river bends. The following systematic sampling was applied:



MEAN VALUES	Q m³/s	S m³/day	D_m mm
RHINE	2140	550	6
WAAL	1490	700	4
PANN. CHANNEL	650	300	4
LOWER RHINE	390	270	2
IJSSEL	260	130	5

Fig. 3.8 Bifurcations at Pannerden and Westervoort, the Netherlands

(i) For each branch samples were taken over a length of some kilometres in cross-sections at regular intervals of 250m.
(ii) In each cross-section three samples were taken at the axis and at $y = \pm\frac{1}{4}B$
(iii) The samples were taken with a grab from the active transport layer.
(iv) The sample size was large enough to be representative (cf. Jansen *et al.*, 1979, p. 220)
(v) From each sample D_m was determined; together these values gave $p\{D_m\}$ for each branch.

In Fig. 3.9 the result is given for the bifurcation at Westervoort.

In the sixties a scale model study on the Pannerden Birfucation was started and, taking into account grain-sorting at a bifurcation, it was judged essential to use non-uniform bed material in the scale model. Samples were therefore taken in the (upstream) River Rhine as indicated above. The prototype distributions $p\{D_m\}$ showed a greater deal of scatter (Fig. 3.10). In this particular case the same diameter scale was taken for almost all fractions. In this example the grain-size of the prototype was rather large.

The use of non-uniform sediment in the scale model increases the operation costs substantially. The sediment supplied to the model has to have a prescribed distribution. This implies the installation of drying and sieving apparatus for handling substantial quantities of sediment. Moreover, the sediment could not be circulated but sediment traps at the end of the two downstream branches had to be used.

For river reaches without bifurcations or confluences automatic circulation of the sediment has advantages. The effective time of operation of the model is larger. The disadvantage is that the sediment transport in the model is more difficult to determine.

As shown in Sub-section 4.3.4 it seems at present (1992) that numerical (2-DH) morphological models can produce results with the same accuracy as in scale models in the case of the bifurcation at Pannerden.

In Jansen *et al.* (1979, p. 315) an example is given where the scale model of part of the River Waal (see Fig. 3.8) was supplied with relatively uniform sediment (bakelite) (see also De Vries and van der Zwaard, 1975). A comparison of sand and bakelite as sediment in a scale model was made by Struiksma and Klaassen (1986).

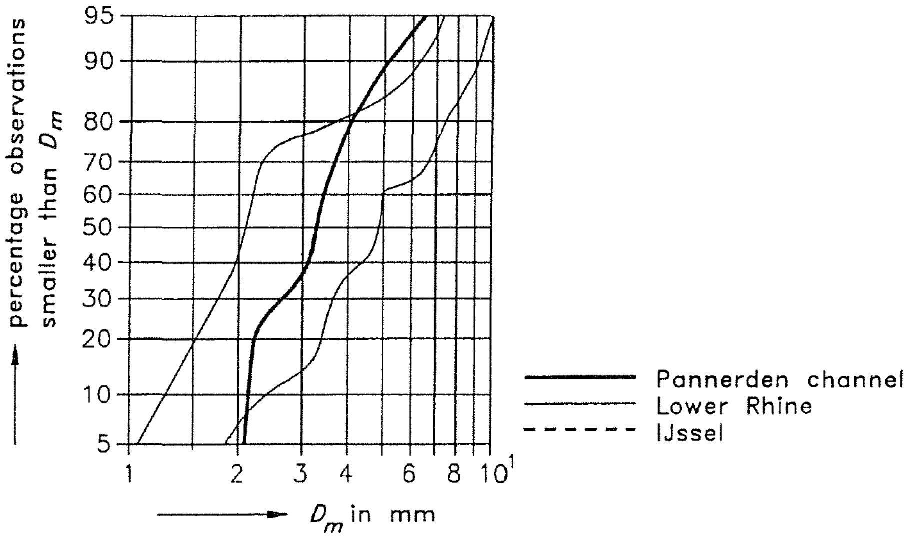


Fig. 3.9 Grain-sorting at the Westervoort bifurcation

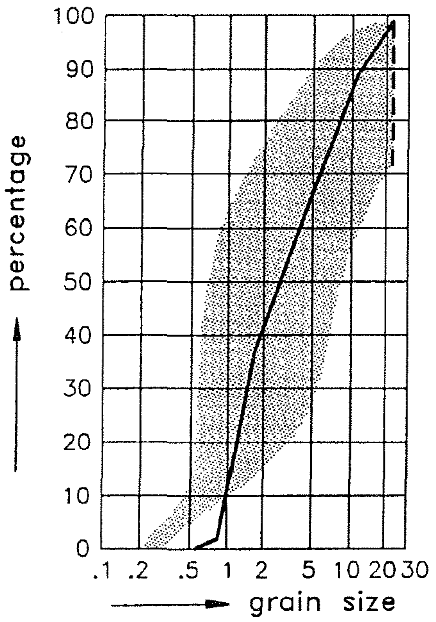


Fig. 3.10 Example of grain-size distributions

3.3.9 Miscellaneous

A number of additional remarks can be made with respect to mobile-bed scale-models of rivers.

(i) In spite of tilting the model errors are present in the water level due to the fact that before building the model the value of C_m has to be estimated. From Eq. (3-24) the implication is that the length of the river reach (L_p) to be reproduced is restricted. This restriction is not present in a numerical model if the latter is carefully calibrated mainly with respect to the alluvial roughness.

(ii) The methodology discussed here is concerned with banks or cases in which the erodibility of the banks is much lower than that of the river bed. The writer is not aware of the use of scale models to predict the change in time of the planform of a meandering river. This seems to be due to the difficulty in establishing the erodibility of the banks (*cf.* Sub-section 2.4.5). For a scale model there is then, in addition, the difficulty of finding a suitable material to reproduce the erodibility properly. Moreover, the effects mentioned under (i) play a role.

(iii) No attention has been paid here to many practical aspects that are of importance in the successful use of a mobile-bed scale model. A number of these can be cited merely as examples.
- The model is no more than a tool. In addition to schematisation of the river reach the boundary conditions have also to be established. In other words the user of the model has to be a *river engineer*. This obviously also holds for a numerical model.
- The measuring programme for the model has to be fixed. It is advisable to use a flexible programme, which implies that the results of each test have to be analysed properly before the next one is defined.
- Sufficient prototype measurements (of sufficient quality) have to be available for reliable model results. A model can never replace field studies. It can, however, give guidance in the measuring programme of the prototype.

3.4 Three-dimensional scale models

3.4.1 General

It would appear that at present (1992) scale models are still competitive or even superior to numerical models when the flow has a strongly three-dimensional character. Some remarks on this aspect are perhaps in order here.

3.4.2 Hydraulic structures

The flow via hydraulic structures is usually of a strongly three-dimensional nature. This still creates a domain for scale modelling. As an example, a weir with a downstream stilling basin can be taken. The shaping of the structure in such a way that it meets requirements is an occasion on which the researcher combines science and art.

The scaling is based on the Froude condition and to a lesser extent on the roughness condition. Obviously no distortion should be applied. The Reynolds condition dictates here that the scale model should be large enough to avoid viscous effects. If the thickness of the viscous sublayer at the structure is of importance to the flow pattern than scale effects cannot be avoided (*cf.* Jansen *et al.*, 1979, p. 308).

Special attention has to be paid to the correct reproduction of the velocity upstream and of the scale model.

3.4.3 Sedimentation at intakes

If for a hydraulic structure (e.*g.* an intake) the sediment movement has also to be considered, then in addition to the requirements given in Sub-section 3.4.2 the reproduction of the sediment requires careful attention. According to Eq. (3-35) the requirement in this case is

$$n_{W_s} = n_u \qquad (3\text{-}36)$$

providing that for this undistorted model the condition $n_C = 1$ is fulfilled.

As n_u is here determined by the Froude condition, the value of W_{sm} is fixed for given

W_{sp}. For fine sediment in the prototype this leads in practice to the need for light-weight material in the model. If available, light-weight material should be used for preference and two ways are then open to meet the condition of Eq. (3.36).

(i) Between certain limits the length scale can be changed to fulfil

$$n_{W_s} = n_L^{\frac{1}{2}} \tag{3-37}$$

Available space and discharge capacity may block this way out.

(ii) By changing the density of the water (e.g. by means of salt) it is possible to change Δ_m and hence of W_{sm}. Obviously this solution (which has been used in the past) does have some consequences. In the first place the model requires its own circulation system, and secondly the water becomes aggressive to piping and pumps.

3.4.4 Near-field dispersion

The strong 3-D character of near-field dispersion especially when a hydraulic structure is concerned, also makes the use of a scale model a good choice. Dissolved matter (or heat) has to be reproduced in the scale model. For the model it is important that concentrations can be measured easily. This is the case for a salt solution, dilute enough to avoid density currents. However, then the water becomes aggressive to the circulation system. For any selection of dissolved matter it holds that after a number of tests the concentration of 'clear' water gradually increases. This means that the background concentration of the water has to be measured as well, and this also incorporated into the interpretation of the results of the scale model.

3.4.5 Structures and ice

When a structure is subject to floating ice additional forces act on it. Again, this seems a topic for an (undistorted) scale model. The requirements outlined in Sub-section 3.4.2 are of importance, but, in addition, the elastic characteristics of the ice-floats need to be reproduced properly in order to get similarity with respect to the forces acting on the structure (which might also be a bridge pier).

4. Numerical models

4.1 General

Increased computational capacity together with progress made with respect to numerical analysis has meant that in recent years more river problems have been tackled using numerical models. Whereas a good mathematical description of the physical processes involved is wanted for a scale model, for a numerical model it is an absolute prerequisite. The mathematical description is the basis for a numerical model, which when properly calibrated and verified can be used for the solution of practical river problems.
In this publication the *use* of numerical models rather than their design forms the main aim. Consequently the numerical model (computer code) is considered here as a tool.
For effective use it is important that the user considers a number of questions.

(i) What is the nature of the problem involved and how can it be schematised?
(ii) Is there a computer code available for solving the problems?
(iii) Are there sufficient data available to calibrate and verify the model?
(iv) What interpretation can be given to the results of the computations?

4.2 Water movement

4.2.1 One-dimensional models

A large number of computer models is available for solving one-dimensional water equations. Usually these involve models for subcritical flow (see, for example, Jansen *et al.*, 1979), and these are generally used when not much detailed information is required. For flood forecasting it is mainly waterlevels that are of importance. For this type of problem a 1-D numerical model is relevant, since a scale model is out of question because of the large distances involved.

Moreover, for large Froude numbers there may be problems. For a *critical slope* the flow is inherently unstable. Hence it cannot be expected that numerical stability will be reached.

It has to be recalled that for *subcritical flow* a *downstream* boundary condition is required for the water level. This can be a steady-discharge relationship (Q-h curve) for a gradual variation of $Q(t)$. However, if Q and h do not have a unique relationship during a flood wave (hysteresis) the Jones formula has to be applied (Henderson, 1966).

$$Q \approx Q_s \sqrt{1 + \frac{1}{i_b c}\frac{\partial h}{\partial t}} \qquad (4\text{-}1)$$

in which Q_s is the steady discharge and $c \approx \frac{3}{2}u$ is the celerity of the flood wave. Details can be found, for example, in Jansen *et al.* (1979). On the other hand for *supercritical flow* the water level has to be known at the *upstream* end.

For rather regular channels reliable computer codes are available. The application involves the proper schematisation of the geometry of the channel into one space dimension and the selection of the roughness parameter (e.g. Chézy coefficient, C). The latter is preferably based on measurements in the channel studied. For alluvial channels an alluvial roughness predictor can be used if sufficient measurements are not available. For natural channels data given for example by Barnes (1987) can be of much help. A reliable model can then be prepared via calibration and verification.

At least in two respects more research is needed to improve the performance of 1-D models for water movement.

(i) For flood forecasting of *extreme situations* the model cannot be calibrated properly, since no measurements are usually available for rare events. In particular alluvial roughness predictors are largely empirical. Then extrapolation outside the range of experimental data creates uncertainty. A solution can be obtained by sensitivity analysis. This means that the probability of the water level at a certain station is established through repeated computation using different roughness parameters taken from the probability distribution of the roughness parameter (Monte Carlo method).

(ii) Very *irregular channels,* notably ones with a steep average slope, are not very well treated with models for rather regular channels. The difference is that discontinuities occur in irregular channels (e.g. mountain streams). Research is going on (Savic, 1991) to extend in this respect the applicability of 1-D models.

The geometry of a river with flood plains may give reason to use basically a one-dimensional model rather than a two-dimensional one. This is, for instance, the approach used by Zanobetti and Lorgeré (1968) for the Mekong Delta. The flood plain is schematised into a system of channels (with friction) and boxes (with storage) (see also Cunge *et al.*, 1980). A similar approach is reported by Grijsen and Vreugdenhil (1976) for the Quad Sebou River and the Plain of the Rharb (Morocco); see also Meijer *et al.* (1965).

This schematisation was applied as long ago as 1926 (Staatscommissie Zuiderzee, 1926) by a State Commission chaired by the famous physicist H. A. Lorentz, for a model to predict the changes in the Wadden area due to the enclosure of the Zuider Zee in the Netherlands. This led to very time-consuming computation, since only simple mechanical tools were available at the time.

4.2.2 Two-dimensional computation

For certain flood problems a 1-D computation does not give sufficient information, and a 2-DH model is then needed. These models are based on the assumption of hydrostatic pressure distribution in the vertical, being time-dependent or regarded as in steady state. Whenever possible the steady-state case is attractive: it simply saves computer time. Moreover, the computer results are much easier to present.

The generally irregular geometry in the horizontal plane requires a curvilinear grid in order to follow the fixed boundaries with sufficient accuracy. This is the case if the basic equations are solved with finite differences. If the equations are solved with a finite element method (which is usually not the case) the selection of the elements can be such that the fixed boundaries are covered properly.

An example is given in Fig. 4.1A to F (Wijbenga, 1985) concerning a reach of the River Waal (the main branch of the River Rhine in the Netherlands). One of the objectives of this model study was the velocity distribution across the river and along the banks. It describres an extreme situation (Q = 10 500 m^3/s, recurrence interval 10^4 years). The downstream boundary (water level) was obtained from a 1-D computation. The 2-D computation involves steady-state at the top of the flood wave.

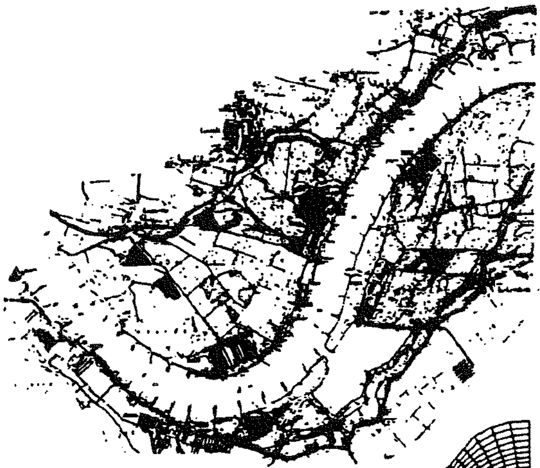


Fig. 4.1A Plan form

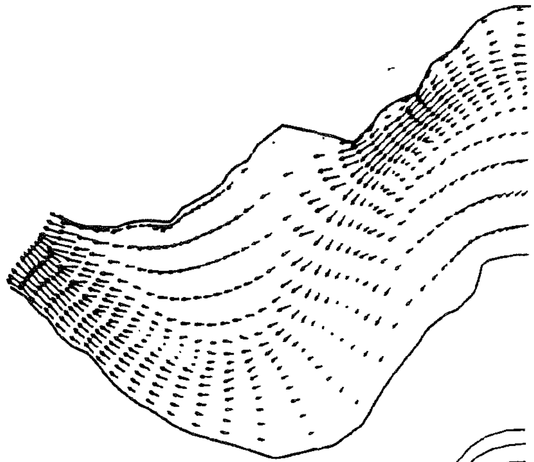


Fig. 4.1D Velocity vectors

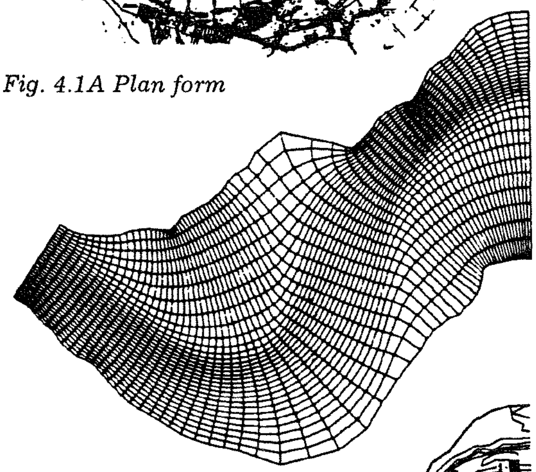


Fig. 4.1B Curvilinear grid

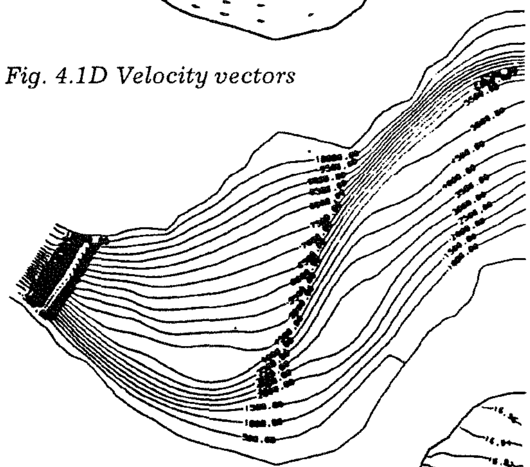


Fig. 4.1E Flow lines

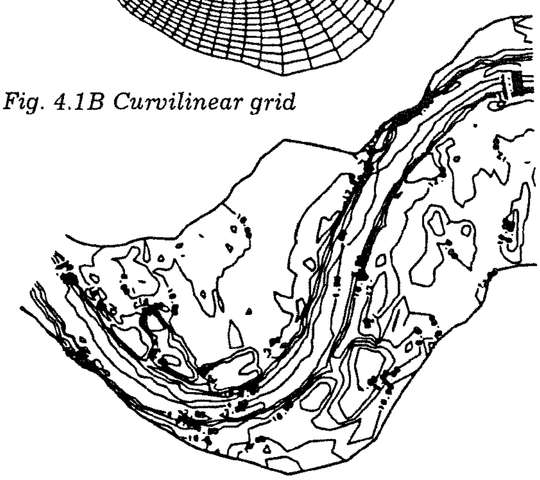


Fig. 4.1C Bed elevations

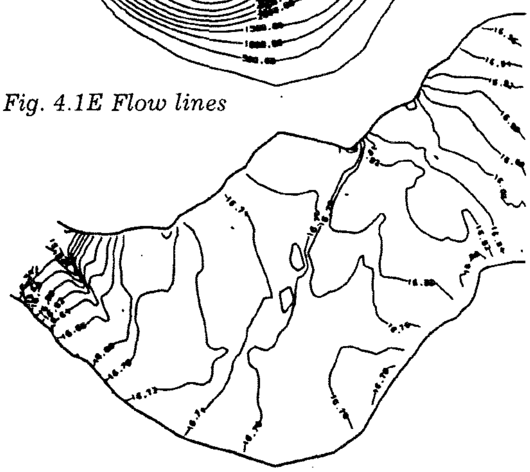


Fig. 4.1F Piezometric heads

Figures 4.1A to F are self-explanatory.

The following remarks need making.

(i) In Sub-section 4.2.1 the difficulty of establishing roughness parameters for an extreme situation was remarked upon. Obviously this also holds here, only twice so. First of all, it influences the downstream boundary condition. In the second place it governs the distribution of the flow velocity in the river reach.

(ii) Automation of the computation can sometimes lead to strange results. In Fig. 4.1E it may be noticed that the stream line bordering the first stream lane at the left-hand side of the river starts at the river bank. This is, by definition, not possible.

4.3 Morphological computations

4.3.1 General

The use of morphological models has increased widely during the last three decades or so. There are three important aspects to this development. In the first place the physical processes involved are much better understood, albeit that in a quantitative sense the picture is far from complete. In the second place the developments in numerical analysis have contributed to a large extent to practical applicability. Thirdly the increase of the computer capacity both in speed and memory size is of importance. This holds also for desk-top computers, used now extensively.

4.3.2 One-dimensional models

A one-dimensional approach is possible when the width-averaged values of the dependent variables give sufficient information for the problem to be solved. Large reaches of a river are usually considered. A number of aspects are discussed here.

Boundary conditions

If the river reach involved covers the interval $0 < x < L$ then the following conditions have to be known (for details see Jansen *et al.*, 1979).

(i) The *initial condition* $z_b\,(x,0)$ has to be obtained from soundings.

(ii) For subcritical flow the *downstream condition* $h(L,t)$ has to be given. This usually comes from a known Q-h rating curve.

(ii) The *upstream conditions* are in the first place the discharge $Q(0,t)$. This can be obtained from previously recorded discharges or from rainfall statistics using a hydrological model. To solve the sediment equations either $S(0,t)$ or $z_b(0,t)$ has to be known. This is not easily obtained, since S is *not* a function of Q *alone.*

(iii) *Internal boundaries* are to be used if discontinuities are present in the dependent variables. This can involve the discharge (withdrawal of water), the sediment transport (sand-mining at a point), the water level (presence of a weir) or the width.

Special conditions have to be formulated as internal boundaries when *confluences* and *bifurcations* are concerned. A confluence does not create a difficulty since *at* the confluence

$$\sum^{i} Q_i = 0 \text{ and } \sum^{i} S_i = 0 \quad i = 1,2,3 \tag{4-2}$$

Whilst the upstream discharges and transports are known Eq. (4-2) is just sufficient to derive the values of Q and S downstream.

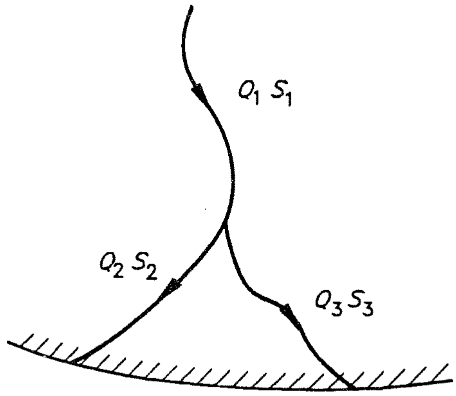


Fig. 4.2 Bifurcation.

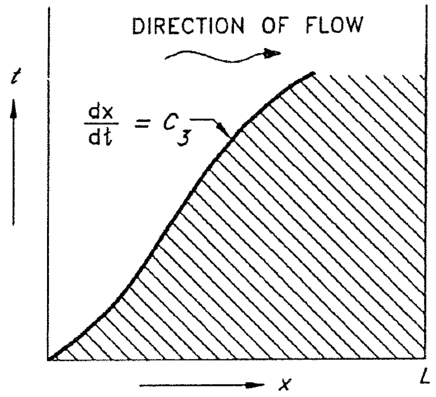


Fig. 4.3 Upstream boundary

For a *bifurcation* the values of Q and S for *two* downstream branches have to be known. This implies that besides Eq.(4-2) two more equations have to be found (Fig. 4.2).

One equation has to be found by considering that the distribution of Q_1 in Q_2 and Q_3 has to be such that at the bifurcation only one water level is present. The distribution of S_1 into S_2 and S_3 is not so easy to find. Only for fine sediment (wash load) it can be assumed $Q_2/Q_3 = S_2/S_3$. For coarser sediment, however, the distribution of S is determined by the *local* flow-field at the bifurcation (Bulle, 1926).

Special attention has to be paid to the upstream sediment condition i.e. the transport $S(0,t)$ or the bed level $z_b(0,t)$. This information is not easily obtained except in special cases. Here, however, use can be made of the fact that the morphological processes are usually slow. In Fig. 4.3 the x-t diagram for the interval $0 < x < L$ is given.

The characteristic through the origin (following from c_3, Eq. (2-39)) separates the regime for which $S(0,t)$ is influencing the results of the computations. Moving the boundary more upstream enlarges the region in which the influence of $S(0,t)$ is not present. A similar procedure can be applied at the downstream boundary. At $x = L$ the value of $h(L,t)$ has to be given. If this is done via a discharge-rating curve then this curve is no longer appliciable when bed-level changes reach $x = L$. By moving the downstream boundary more downstream this can be circumvented.

Sub-models

One-dimensional computations are based on the (four) equations of motion and continuity of the two phases (water and sediment). These equations require two sub-models: relations to express transport and roughness as functions of the local hydraulic conditions including the characteristics of the sediment. A proper *transport predictor* has to be selected but whether or not a *roughness predictor* has to be selected is questionable. It might be preferable not to introduce a roughness predictor in the modelling but to use experimental data on the roughness gathered for the particular river involved. This is an example of the fact that a model is no more than a tool in the hands of the user.

Numerical schemes

For quasi-steady flow the selection of a numerical scheme focusses on the numerical solution of the continuity equation for the sediment.

$$\frac{\partial z_b}{\partial t} + \frac{\partial s}{\partial x} = 0 \qquad (4\text{-}3)$$

A number of numerical schemes are in use, mainly of an explicit nature. An implicit scheme is proposed by Holly and Rahuel (1990). No practical applications are as yet reported by the authors.

Vreugdenhil (1982) has compared a number of schemes. This has also been done by Croad (1986), who warns against the use of the HEC-6 scheme (Thomas, 1979) as it appeared to be unstable.

The writer agrees with the statement of Vreugdenhil (1982) that one should not claim one particular method to be the 'best'. Vreugdenhil gives the following warning: *'However, searching for an optimal method does not seem very useful; the important thing is to avoid an uncritical use of any numerical model.'*

Non-uniform sediment

For river beds with a large range of grain sizes the morphological model cannot be based on a single representative grain size. During morphological changes grain sorting can occur. The extention of the 1-D model is thus necessary via the transport equation and the continuity equation.

Egiazaroff (1965) describes transport for separate grain fractions. This work was used by Ribberink (1982, 1983, 1987) to model the morphological process for non-uniform sediment. Ribberink carried out flume tests with two fractions (D_1 = 0.78 mm and D_2 = 1.29 mm). These careful and time-consuming measurements have given much insight into the complex morphological process. As an example Fig. 4.4 gives the vertical sorting after the flume has been in operation with the supply of 50% of each grain size ($p_1 = p_2 = 0.5$).

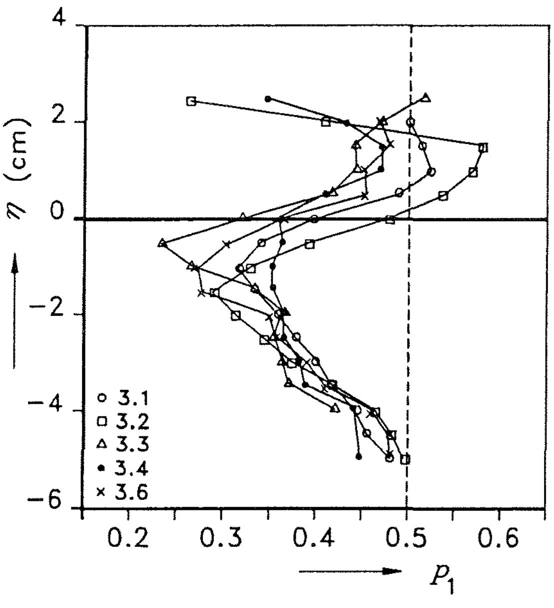


Fig. 4.4. Vertical distribution of p_1 (Ribberink, 1983)

The figure shows for five independent sample runs the percentage (p_1) of the fine fraction as a function of the bed level. A relatively fine upper layer and a relatively coarse lower layer is present. The findings of Ribberink have been used by Olesen (1987) for his two-dimensional model.

Schematisation of cross-sections

The schematisation of cross-sections for the application of a one-dimensional morphological model is a problem in itself. What is required is a simulation of actual wetted bed-level $z_b(y)$ with $0 < z_b(y) < B(h)$ by a kind of average $z_b(x)$ over a width $B(x)$. There is no standard method for the solution of this problem. In Cunge *et al.* (1980) some suggestions are given. A logical basis for the schematisation of the *real* cross-section into the *schematised* one seems to be that both should be able to transport the same Q and S. A related problem is how the bed-level changes are distributed over the real width. Two remarks have to be made.

(i) This problem demonstrates again that a model is only a tool in the hands of the river engineer. The model cannot provide the schematisation.

(ii) If it is really important how the bed-level changes vary across the width then the limit of the applicability of a 1-D morphological model has been reached and the use of a 2-D model is recommended.

Example: Tana River (Kenya)

Let us take as an example of a one-dimensional computation the study of the Tana River, Keny (DHV-DHL, 1986). Upstream of the Koreh Falls (Fig. 4.5) there is hydropower potential and downstream of these falls the water can be used for irrigation. Moreover, wildlife is an important factor. Artificial interference in the river will lead to morphological changes.

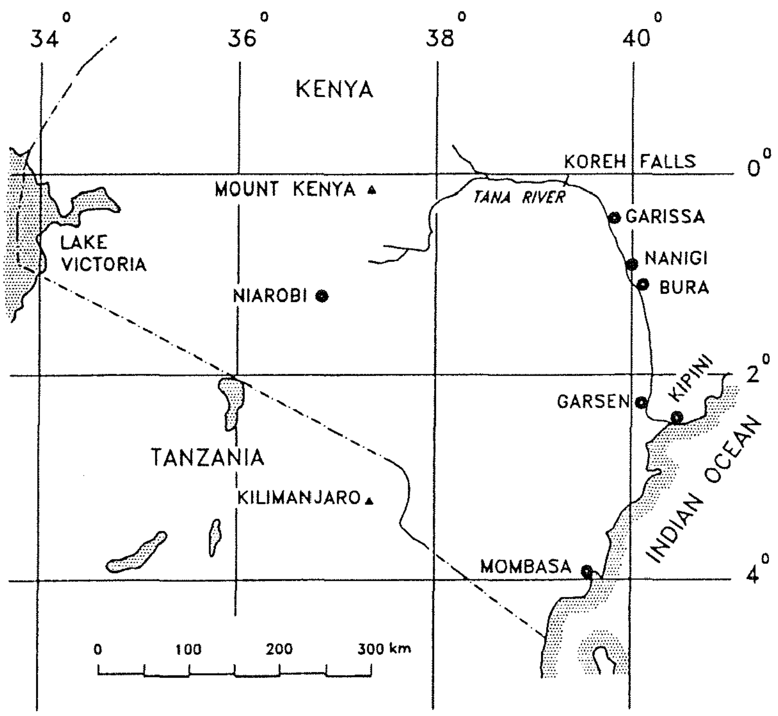


Fig. 4.5 Tana River Basin

Building of dams and reservoirs will reduce the sediment supply to the lower reach of the river. Construction of weirs for the intake of water for irrigation will introduce sedimentation upstream of the weirs and erosion downstream. Erosion will lead to lowering of the water levels and consequently also to the lowering of the ground-water levels. This, in its turn, may lead to a change in vegetation and hence in wildlife, and finally the latter may lead to a reduction in tourism.

In Jansen *et al.* (1979, p.433) information is given on the preliminary model study for the Tana River carried out in the 1970s. A second study was conducted in the 1980s (DHV-DHL, 1986), based on many more data.

The collection of data is time- and money-consuming. This is especially so for the initial condition $z_b(x,0)$. In practice this means that the space step Δx is of great importance. As in this case the total length of the river reach is some 600 km the space step should not be too small, otherwise the amount of (field) work becomes too great. On the other hand, a large value of Δx also leads via the celerity of disturbance on the bed (c_b) to a large time step (Δt). This value may be too large to follow $Q(t)$ adequately. Fortunately, the variation of Q is rather slow for the Tana River.

Different scenarios where envisaged, with two barrages at Nanigi (km 430) and Sailoni (km 150). The following general information can be given on the computations.

(i) It appeared that the river from km 510 (Garissa) to the river mouth (km 0) loses a substantial part of its discharge. This is reflected in a reduction of the width in the downstream direction (Fig. 4.6).

(ii) None of the following roughness predictors were applicable: Engelund-Hansen (1967); White *et al.* (1980) and Van Rijn (1982). A special relation for the Tana River was therefore deduced from measurements (*cf.* Sub-section 4.3.2).
The following relation was used in the morphological computation.

$$\theta = 3.27\left(\theta^1\right)^2 + 6.58\,\theta^1 - 0.27 \qquad (4\text{-}4)$$

(iii) From transport measurement at four stations it appeared that the formula of Engelund-Hansen (1967) was the most suitable one, in spite of the fact that the formula over-estimated the measured transport on average by a factor two. This is acceptable (see also Table 2.1). This factor was incorporated in the Engelund-Hansen formula. Over the reach considered the D_{50} changes roughly from 0.2 to 0.4 mm.

(iv) The computations were (after calibration for the existing situating over the period 1944-1985) carried out for the reach $0 < x < 650$ km in general with $\Delta x = 10$ km (Δt = 5 days) but in the vicinity of the weirs with $\Delta x = 2$ km ($\Delta t = 1$ day). The Courant number applied was in this way smaller than unity.

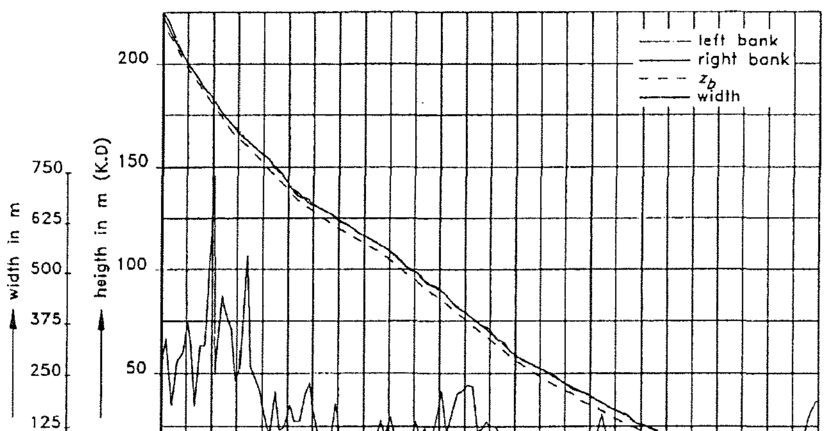
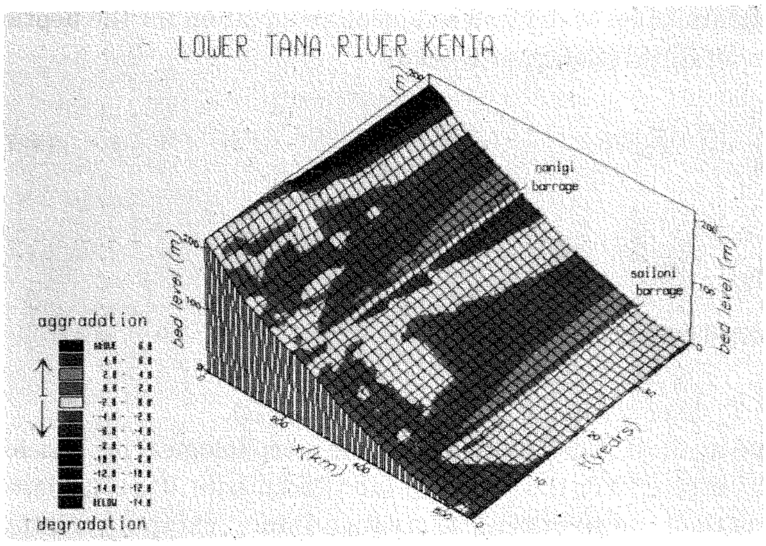


Fig. 4.6 Longitudinal profile Tana River

Logically a large number of data was needed for input into the standard program RIVMOR (Delft Hydraulics), but also a large number of data forms the output of the computations. Special attention must be paid in showing policy-makers the results of a specific scenario as far as morphological changes are concerned. Figure 4.7 gives an example (the normally multicolour image being reproduced here in black and white).



4.3.3 Quasi one-dimensional models

One-dimensional models are based on the assumption that the sediment transport is determined by the *local* hydraulic conditions (Sub-section 2.4.4). For bed-load transport this is a fair assumption considering the length-scale of the problems that are tackled with a one-dimensional model (*cf.* Sub-section 4.3.2).

In the case of suspended load, especially where the *adaptation length* of the transport to the hydraulic conditions is not small compared to the space step (Δx) of the morphological computation, a quasi 1-D model can be used.

Basically, a 2-DV model can be used. Equation (2-26) can be solved numerically if the water movement is known. Then the local transport can be determined via $s \sim \int u\phi \, dz$ and the new bed level can be found through integration of the continuity equation of the sediment:

$$\frac{\partial z_b}{\partial t} + \frac{\partial s}{\partial x} = 0 \tag{4-5}$$

Although an integration of Eq. (2.26) is not too complicated, morphological computations for a large number of years would still be too time-consuming. Therefore Galappatti (1983) has used an asymptotic approach to solve Eq. (2-26) for steady state $(\partial \phi / \partial t \approx 0)$. This method is applicable when the concentration profiles do not differ too much from the equilibrium profile (see also Galappatti and Vreugdenhil, 1985).

The assumption is made that the parameter $\delta = aU/LW_s$ is much smaller than unity. Hence the concentration ϕ can be written as

$$\phi = \phi_e + \delta\phi_1 + \delta^2\phi_2 + \ldots \tag{4-6}$$

Here ϕ_e is the equilibrium concentration distribution and ϕ_1, ϕ_2 etc. are first order, second order, etc. corrections to it. The method reduces the computational time for the solution of

Eq. (2-26) if not too many values of ϕ_i have to be taken into consideration (roughly speaking if only ϕ_1 and ϕ_2 have to be considered in addition to ϕ_e). The validity of the approach was tested experimentally by Wang and Ribberink (1986). The application of the asymptotic approach was extended by Wang (1989) to two space dimensions (x, y) and time (t), making it possible to model the morphological processes in a tidal estuary.

If it is assumed that for a certain case it suffices to include only ϕ_1 in addition to ϕ_e then besides the four basic equations for water and sediment an equation for the depth-averaged concentration $\overline{\phi}$ can be added (Galappatti, 1983).

$$T_A \frac{\partial \overline{\phi}}{\partial t} + L_A \frac{\partial \overline{\phi}}{\partial x} + \overline{\phi} - \overline{\phi}_e = 0 \tag{4-7}$$

Here

T_A = adaptation time ~ a/W_s
L_A = adaptation length ≈ aU/W_s ~ $u\ T_A$
$\overline{\phi}_e$ = equilibrium depth-averaged concentration

The proportionality factor in T_A depends on the shape of $u(z)$ and $\varepsilon_s\ (z)$.

Equation (4-7) was used by Sloff (1990) in modelling the morphological process for fine sediments and steep slopes as present on the Kelud Volcano (Indonesia). It is considered outside the scope of the present report to give details of the Kelud study, except to mention that the large concentrations require additional terms in the continuity equations for water and sediment. Moreover, supercritical flow was present which required as a boundary condition the bed level at the *downstream* end of the interval $0 < x < L$. In addition, $\overline{\phi}(0,t)$ has to be given.

A completely different type of quasi one-dimensional model concerns the meander model as proposed by Ikeda *et al.* (1981). This mathematical model contains two basic equations, one for the tangential flow velocity at the river bank and the other describing the erosion of the river bank due to the tangential flow velocity. One differential equation gives the tangential flow velocity as a function of the local curvature, the Froude number, a friction factor, the river width and a bed-form factor. The erosion equation contains an *erosion factor*.

In practice, the equations have to be solved numerically, involving a two-step computation, solving the two equations alternate. The initial bed topography has to be given and also the velocity distribution at the upstream end of the reach considered. Unfortunately the upstream end has to be fixed in place. This implies that this has to be a fixed place in reality too, or that it has to be placed so far upstream of the reach of interest that its fixation does not (yet) influence the results.

In spite of the rather crude schematization of the basic equations, including linearization, the model seems useful for freely meandering rivers. Naturally sufficient historical data have to be available to calibrate the model.

A numerical model has been applied by Van der Linden (1985) for the Tana River in Kenya (see also DHV-DHL, 1986). Figure 4.8 shows the calibration results for the period 1960 to 1975. The river axis is indicated.

The hindcasting given in Figure 4.8 was followed by a forecasting. In this case the practical problem is linked to the presence of an irrigation scheme (ADC-scheme) near the river. The investment in the scheme was relatively large, since it involved sprinkler irrigation. Figure 4.9 gives a detail of the forecast. Note the large horizontal movement of the river banks (up to 17 m/a).

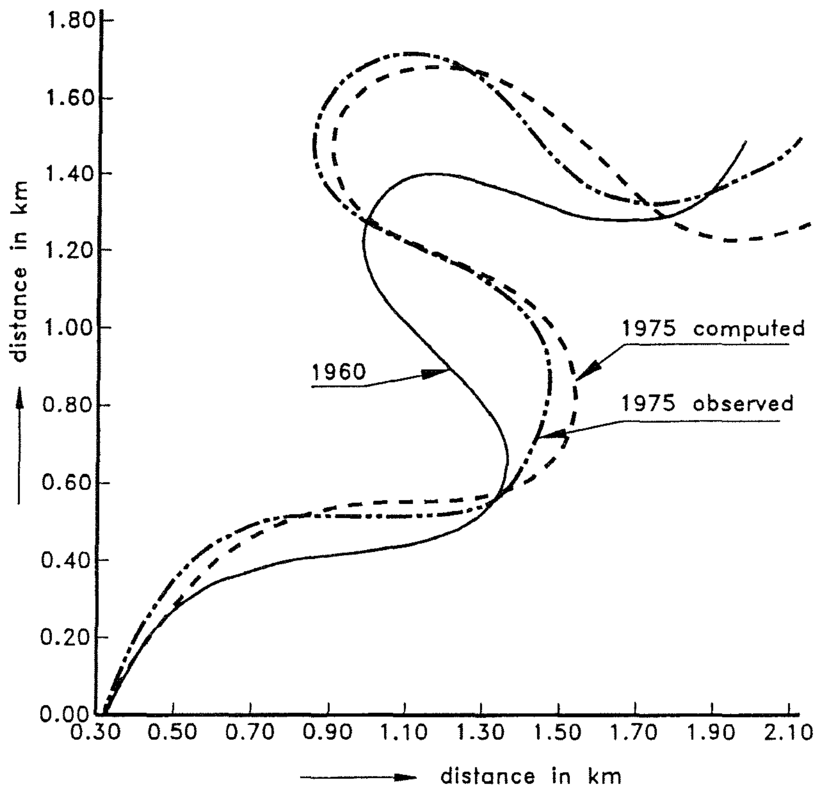


Figure 4.8 Calibration meander model (Tana River)

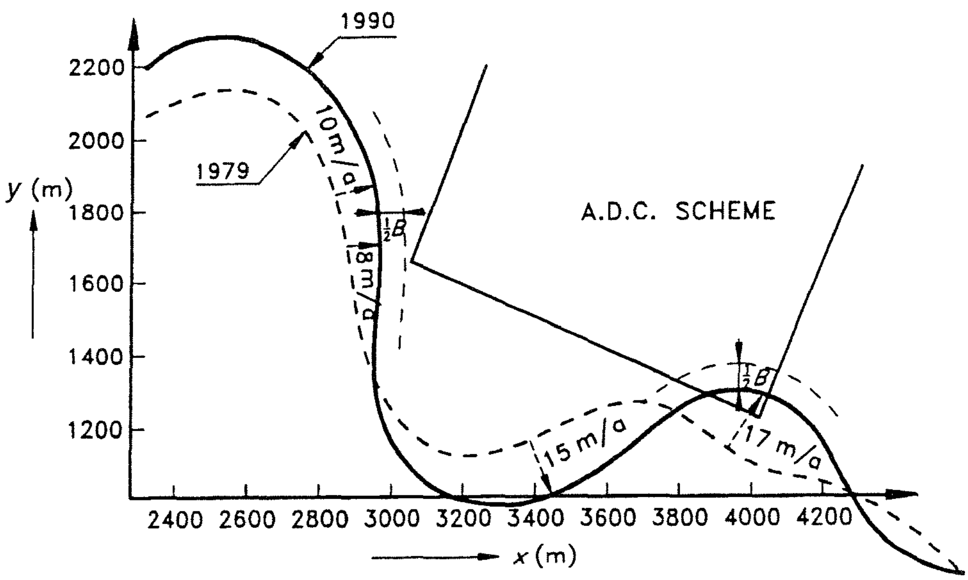


Figure 4.9 Prediction meander model (Tana River)

Johannessen and Parker (1985) report on the numerical simulation of the meander migration of rivers in Minnesota.

4.3.4 Two-dimensional models

The transition between one- and two-dimensional models is gradual. The meander-migration models of the previous sub-section already take into account the horizontal dimensions to a certain extent.

Linearization of the two-dimensional equations (as in Struiksma *et al.*, 1985) makes it possible to treat the equation perpendicular to the river axis analytically, whereas along the river axis a numerical solution is sought.

This principle was applied by Taal (1989) for the Vientiane-Nong Khai reach of the Mekong River (40 km). The Mekong has a large variation Q(t) and from year to year large variations can be seen (Figure 4.10).

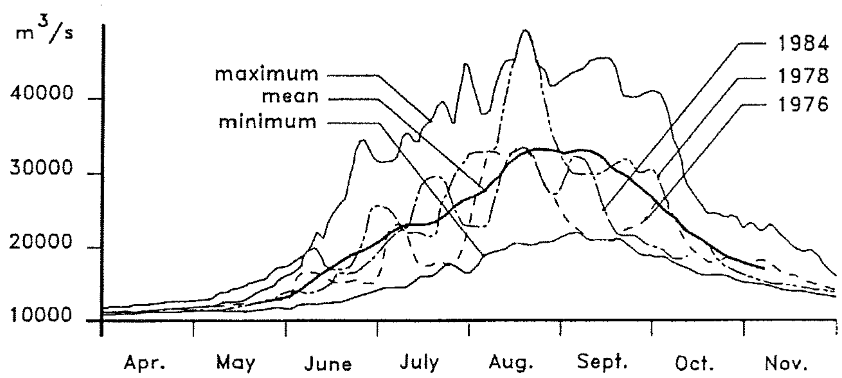


Figure 4.10 Discharge Mekong River (Vientiane)

It appears, however, that the yearly sediment transport does not vary a great deal. In Figure 4.11 the relative yearly sediment transport per year has been indicated. The Engelund-Hansen formula was applied.

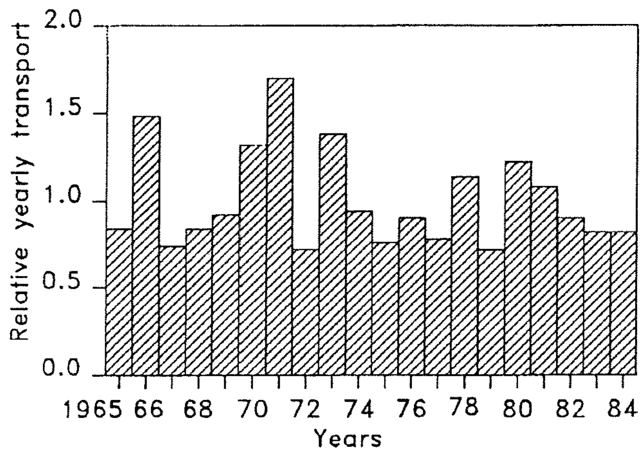


Figure 4.11 Yearly transport (Mekong River)

The objective of the study by Taal was the prediction of the bed level at the toe of the outer bends, necessary for the design of bank-protection works. The numerical model did not include bank erosion. In Figure 4.12 some typical results are depicted. Variation around the average depth due to the bends is indicated. Logically the variation of the discharge in time has been incorporated. The variation in time of the bed level at the toe of a bank can be some metres; this is in accordance with observations in the field.

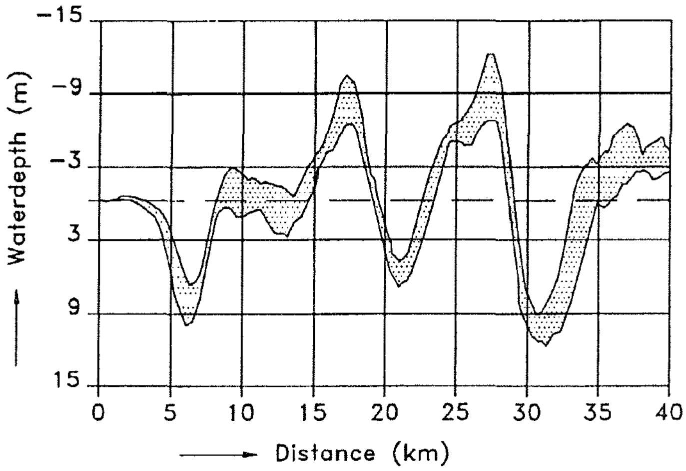


Figure 4.12 Bed-level variation near banks, Mekong River (after Taal, 1989)

Crosato (1987, 1990) has incorporated in her 2-D meander model MIANDRAS the erodibility of the river banks, and here again linearized equations are used. A description of the erodibility in a quantitative sense is difficult in practice; the historical record has to be used in the calibration.

Figure 4.13 is indicative of the results of Crosato's work. The river axis is indicated as a function of time.

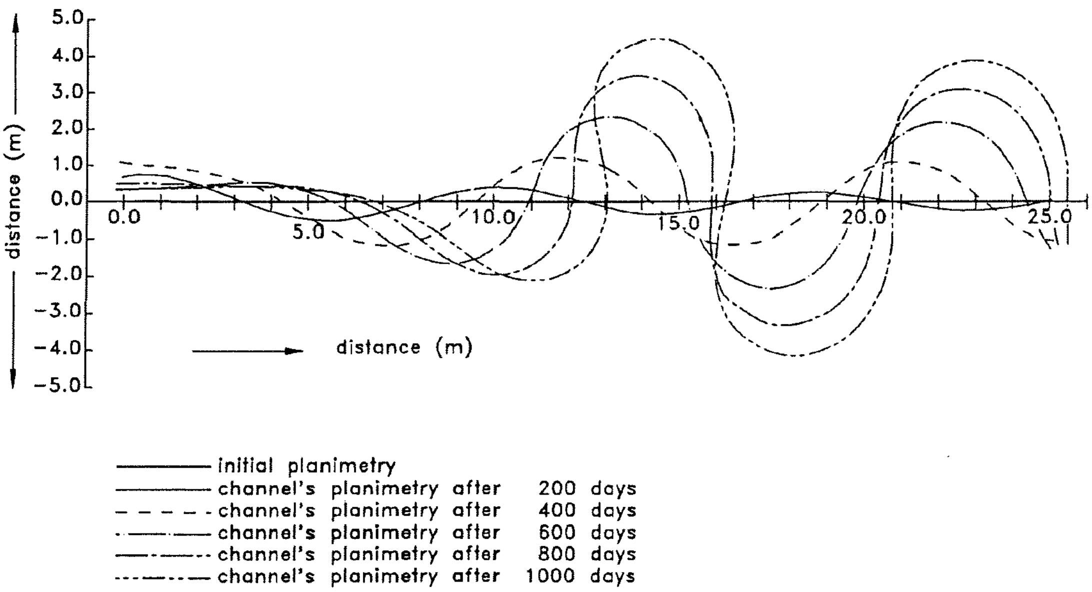


Fig. 4.13 Meander migration (after Crosato, 1990)

Murshed (1991) has extended this model by allowing for the fact that part of the eroded bank-material becomes bed material, i.e. it participates in the morphological process. Murshed applied his model to the Dhaleswari River (Bangladesh). An interesting aspect of his study in that the present course of the river was taken directly from satellite images (see cover of this publication). The starting point is the situation in 1986 and the predicted course (white line) concerns 1993. The model gives unreal results if too much of the eroded bank material is considered as becoming bed material.

As an example of 2-D morphological modelling the study of the Pannerden Bifurcation is given here (*cf.* Figure 3.8). The 2-D morphological model described by Struiksma *et* *al.* (1985) was extended for use in the case of a bifurcation. The study was carried out for a constant discharge, non-uniform bed material and bed load only. Reference can be made to De Vries *et al.* (1990).

In Figure 4.14 contour lines for water depth are given for a prototype situation (1966), reproduction by a scale model and one via a numerical model. It appears that the accuracy of a scale model and of a numerical model are comparable.

Remark

In the 2-D morphological models discussed here 3-D effects have been included in an approximative way. The quasi 3-D model developed by Van Rijn (1987) should also be mentioned; this was not composed especially for morphological predications but for bends in alluvial rivers. The 2-DH model was developed assuming a logarithmic velocity distribution in the vertical direction. In Van Rijn *et al.* (1990) the validity of 2-D and 3-D models for suspended sediment is studied by field measurements. For complete 3-D modelling reference can be made, for example, to Wang and Adeff (1986).

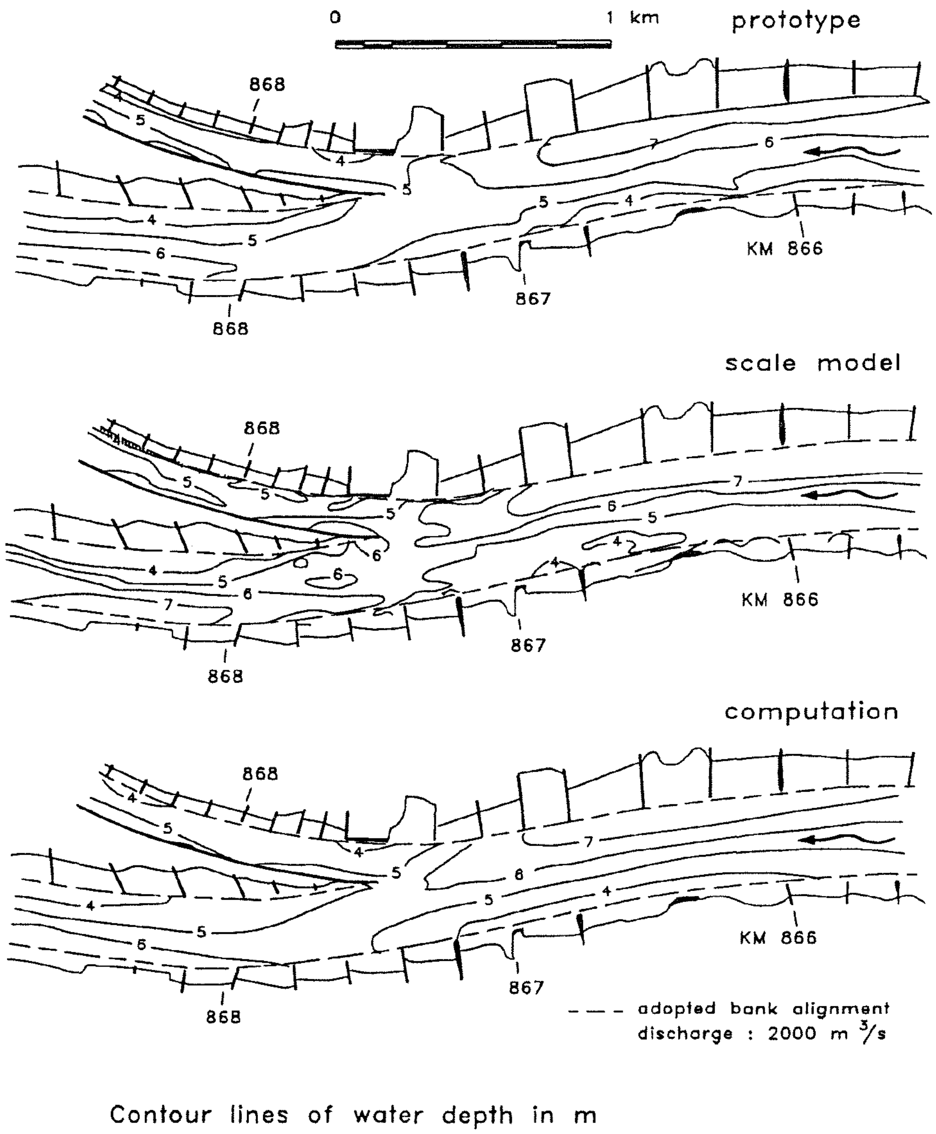


Figure 4.14 Model studies of the Pannerden Bifurcation

4.4 Computations on dispersion problems

4.4.1 General

As has been indicated in Sub-section 2.5, the dispersion equations are basically linear in the concentration ϕ. This implies that the principle of superposition is applicable. This seems to be the reason why computations in this field are based on a mixed analytical/numerical approach. In this section a number of solutions to practical problems are presented.

4.4.2 Measuring discharges by dilution methods

The observation of the dispersion of a dissolved tracer is a method used to determine a discharge. For a constant (unknown) discharge (Q) the method is quite simple (Figure 4.15).

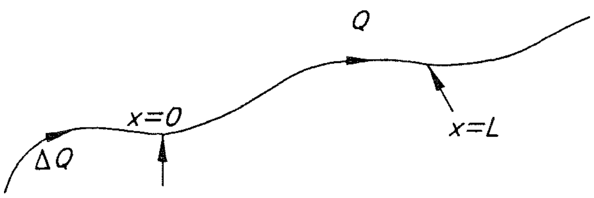


Figure 4.15 Dilution method

At $x = 0$ a tracer solution ΔQ is introduced at a constant rate into the river. At $x = L$ the tracer concentration (ϕ) is measured. If ΔQ is released for a sufficiently long time then at $x = L$ the equilibrium concentration $\phi_e = \Delta Q/Q$ is reached.

If ϕ_e is measured and ΔQ is known then Q can be determined is a simple way (if $\Delta Q << Q$). The method has been standardised for a constant Q (ISO, 1983). One condition is that L has to be sufficiently large to enable *complete mixing* to occur. In ISO (1983) a recommendation on the minimum length of L is given:

$$L \geq 0.13\, K' \frac{B^2}{a} \tag{4-8}$$

in which

$$K' = \frac{C\left(0.7\, C + 2\sqrt{g}\right)}{g} \tag{4-9}$$

for

$$15\ m^{\frac{1}{2}}/s < C < 50\ m^{\frac{1}{2}}/s \tag{4-10}$$

The value of L is said to be within 1 % of complete mixing. The equation for L is of an experimental nature and not based on many data.
It has been shown that this value of L is too pessimistic. Van Mazijk and De Vries (1990) proposed, on the basis of the dispersion equations, that it is better to use Eq. 2-65:

$$L \geq 0.55 \frac{u \cdot B^2}{K_y} \tag{4-11}$$

in which according to Fischer *et al.* (1979):

$$K_y = \alpha \cdot a u_* \text{ with } \alpha = 0.15 \qquad (4\text{-}12)$$

Figure 4.16 gives a representation of Eqs. (4-8) and (4-11) showing that ISO (1983) overestimates L.

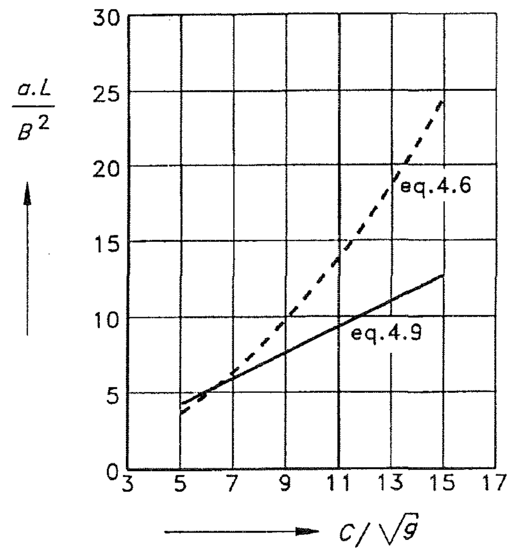


Figure 4.16 Complete mixing

A restriction in the method is that the discharge Q has to be constant. This can be a serious drawback since mountain streams seem especially suitable for the *dilution method* due to the irregular shape leading to large mixing and consequently relatively small values of L. However, a mountain stream with a constant discharge seems not to be very common. This is the reason why Noppeney and Kranenburg (1989) have proposed to make the steady-state method applicable for the non-steady situation.

4.4.3 Accidental-spill models

River basins situated in densely populated and/or industrialised areas require water-*quality* management in addition to water-*quantity* management. An accidental-spill model therefore represents an indispensable tool. By monitoring water quality at stations along the river it can be established whether or not an accidental spill has taken place. The model is used to answer two questions:

(i) Where did the accidental spill take place and what was its magnitude?
(ii) What will be the concentrations downstream (in time and place)?

The answers to these questions can lead to the following:

(i) Legal measures can be taken with respect to the industry that caused the accidental spill.
(ii) Measures can be taken downstream to mitigate the damage harm. This can, for example, include the closure of intakes for water supply and for agriculture.

Some comments may be made here on the construction of accidental-spill models based on the design of such a model for the River Meuse (van Mazijk *et al.*, 1988; Veldkamp and Van Mazijk, 1990; Van Mazijk and De Vries, 1990).

In the first place it has to be recalled that the basic equations are linear in the concentrations ϕ (*cf.* Sub-section 2.5.3). This implies that analytical solutions possible for simple geometries can be used in a summation for a channel system consisting of different reaches. The models are therefore of a combined analytical/numerical nature. In this way *numerical diffusion* likely to occur in a completely numerical solution of the basic equations is suppressed.

Secondly, the construction of an accidental-spill model requires a large amount of geometric data on the specific channel system. This makes the model costly.

Logically the model has to be (like all models) *calibrated* and *verified.* Special measurements of the dispersion of induced 'spills' are essential. In the course of time the model can be improved, guided by its *required* accuracy. Since this *required* accuracy will increase over the years work on a suitable accidental-spill model will be on a relatively long time scale.

5. On the selection of models

5.1 Introduction

Logically this report ends with a chapter on the selection of models for specific cases. In reflecting on this topic the writer feels that the subject can only be treated in a rather general way. The selection not only depends on the type of problem at hand, but also on when and *where* the solution has to be found: in other words, the stage of development of the Institute that has to solve the problem also plays a role.

The selection generally involves either a mathematical model or a scale model. Combinations are also possible. In the case of the combination of two mathematical models (one for the 'far field' and one for the 'near field') it is common practice to call these 'nested models'. Here, this term will also be used if one of the models is a scale model.

In addition to the terms 'far field' and 'near field' the term 'very far field' will be used; the need for this will become clear in Section 5.2, where a case study is presented.

5.2 Case study: morphological models for the River Rhine (The Netherlands)

The selection of models was said above to depend on *when* and *were* the problem has/had to be solved. In this sub-section a historical note will be given on the change in the selection of models over time, i.e. depending on the possibilities available.

To understand this case study it is important to know that for the River Rhine as far as navigation is concerned the *Mannheim Act* of 1868 states that the riparian countries have committed themselves to change the river only with the consent of the other countries. For the Netherlands this concerns the main branch, the River Waal (see Figure 3.8).

When the plans were presented to canalise the Lower Rhine in the Netherlands it was clear that detailed studies had to be carried out to try and establish how this would interfere in the morphology of the River Waal. Moreover, it was important to study the behaviour of the two bifurcations and three reaches of the Lower Rhine where adjustable weirs were planned. An overview of the canalisation works is given in De Gaay and Blokland (1970). The development of mathematical models and scale models for morphological river problems in the Netherlands has been strongly influenced by the canalisation of the Lower Rhine. Progress has been gradual but a number of phases can be distinguished.

Preparatory phase

The planning of the works took place in the 1940s. However, the morphological processes were also studied, as is demonstrated in the pioneering paper of Van Bendegom (1947). His basic idea was that the 'far field' should be covered by means of scale models whereas mathematical means would have to be used to make corrections on detail ('near field'). The restricted computational capacity at that time made this, however, not practically possible.

Scale model phase

In the 1950s and early 1960s the focus was on scale models. However, the scale model of the Westervoort Bifurcation (which has been in operation for more than a decade) was already provided with boundary conditions obtained via computations, a kind of primitive mathematical model (see Sybesma and De Vries, 1961). This was necessary because of the fact that the morphological impact due to the canalisation was expected to take place over larger river reaches than could be covered by a scale model. This is due to the fact that in spite of tilting, uncertainty about actual alluvial roughness restricts the length of river to be reproduced with acceptable errors in the water levels at the boundaries of the scale model (see Sub-section 3.3.9). In fact the 'very far field' was in this way covered by means of a (primitive) mathematical model.

During this period the construction of time-depending mathematical models for morphological processes was started (De Vries, 1959). Restricted computer capacity at the time prevented an integral use (De Vries, 1965).

Combined models phase

In the late 1960s and 1970s a scale model study was performed for the Pannerden Bifurcation (see Fig. 3.8). The set-up of this scale model study differed in two ways from the earlier similar study of the Westervoort Bifurcation.

(i) The scale model study now basically had the objective of finding the sediment distribution at the bifurcation depending on the water distribution and the geometry (in a general sense); the data were meant to be used in a one-dimensional mathematical model with mobile bed. Reference can be made to Sybesma *et al.* (1970).

(ii) This scale model was operated with a prescribed grain-size distribution (see also Sub-section 3.3.8), leading to a substantial increase in costs.

An overview of the sediment transport studies in relation to the canalization of the Lower Rhine is given in Sybesma et al. (1970). The development of the mathematical modelling is described by De Vries (1969, 1973).

Numerical model phase

In the 1970s and 1980s, over a period of mroe than ten years, a basic research programme was in operation in the Netherlands on morphological processes in rivers. The partners were the Public Works Department, Delft Hydraulics and the Delft University of Technology. The objective was to obtain a better *quantitative* insight into the morphological processes, which might then lead to the further development of numerical models, i.e. to avoid the use of expensive scale models for morphological predictions.

This example is a demonstration of the statement made in Section 5.1 about *when* and *where* the morphological problem has to be solved. The gradual switch for the Rhine

Branches in the Netherlands from scale models to numerical models is typical of the situation:

(i) The size of the branches is relatively small compared with the requirements with respect to the cross-sections as put forward by the intense navigation.

(ii) This leads to large accuracy requirements with respect to morphological predictions.

(iii) Increased insight in a quantitative sense into the morphological processes in rivers has shown there to be a limit to the accuracy of morphological scale models. The application of sediment mixtures can increase the accuracy, albeit at the relatively high cost of equipment and manpower.

(iv) The critical factor in the limited accuracy of scale models is alluvial roughness: it cannot be controlled. There is therefore a restriction in the length of reach in spite of tilting the scale model (see Eq. 3-24). A numerical model does not have this problem.

5.3 Criteria for the selection

In the previous chapters already indications have been given as to whether a numerical model or a scale model is preferred for a particular type of river problem. Between extreme cases, where a clear preference is possible, there is an intermediate range in which both model types are, in principle, relevant. For this intermediate range the following remarks may be helpful.

Facilities

The availability of laboratory facilities and/or computer codes may influence the selection. A computer code is usually easier to obtain than suitable experimental facilities, and it is therefore expected that the trend noticeable during the last decades – the increased use of numerical models – will continue. The user is advised to be very critical of the results of computer codes, especially when they are made available at low cost. One should never believe the output of a computer!

Prototype data

Prototype data do not give in the writer's opinion a criterion with respect to the selection of the model type. Almost by definition the available prototype data are always incomplete. A model can never be a substitute for prototype data. Not only the amount of data but also their quality is important. A scale or numerical model cannot give results that are more accurate than the underlying prototype data. This brings forward the difference between *available accuracy and required accuracy. The researcher should* be aware that if too great an accuracy is required then the amount and quality of the field data has to be increased. The study of this topic has only just started. The writer has tried to analyse this problem for 1-D morphological modelling in a general sense (De Vries, 1982a, 1983).

Experience and cost

A typical difference between a scale model and a numerical model concerns the treatment of the geometry. Provided that for a certain river problem both model types can be used then a completely new scale model has to be built for another geometry.

For a numerical model another geometry can be treated more easily albeit that the proper schematization of the geometry is necessary for each new problem. Generally speaking for cases in which both a scale and numerical model are possible the latter seems to become the cheaper alternative.

The model type can be selected because experience is available in this respect. On the other hand, a particular model type may be selected precisely because the experience is not present, in order that it may be gained in the given field.

5.4 Outlook

The gradual increase in the use of numerical models instead of scale models – the trend observed over the last few decades – is likely to continue into the future. The main reason is that the better the mathematical description of fluvial processes becomes, the better the numerical models that can be developed. For scale models better understanding of the fluvial processes implies that more constraints are present in scaling.

This development does not mean that there is no further need of experimental research in hydraulic laboratories. However, the character of this research is changing. Rather than carrying out measurements in a specific geometry, the experimental research is directed towards the study of specific fluvial processes in specially designed research facilities.

Such experimental research is an essential complement to field studies. There the phenomena take place at the full scale; however, in the field conditions cannot generally be controlled.

Finally a remark on education is called for. The switch from scale models to numerical models has meant that young hydraulic engineers are less familiar with flow patterns than those of previous generations. The size of a prototype is usually such that the eye cannot catch flow patterns such as eddies, mixing layers, water-level variations and the like. Flow patterns shown on a monitor are poor representations of reality. The reproduction of flow patterns in hydraulic laboratories are therefore still needed, especially for teaching.

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