Guidelines on the study of seawater intrusion into rivers

Editor: H. van der Tuin

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Guidelines on the study of seawater intrusion into rivers

Prepared for the
International Hydrological Programme
by the Working Group of Project 4.4b (IHP-III)

Editor: H. van der Tuin

UNESCO

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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.

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Foreword

At its Sixth Session the Intergovernmental Council for the International Hydrological Programme (Paris, 30 March 1984) gave a working group the task of preparing a publication on guidelines on the study of seawater intrusion into rivers for practising engineers.

This working group consisted of:

Mr. V. Mikhailov	(U.S.S.R.)
Mr. Le Huu Ti	(Mekong Secretariat, Bangkok)
Mr. A. Roelfzema	(Delft Hydraulics, Netherlands)
Mr. A. Volker	(Netherlands)
Mr. H. van der Tuin	(Netherlands) who was elected Chairman of the working group.

During two meetings of the working group (in Dordrecht, the Netherlands, December 1986 and Paris, October 1987) Mr. F. H. Verhoog, UNESCO Division of Water Sciences, provided the secretariat.

The work of IHP-III Project 4.4.b has been carried out mainly by correspondence. The various contributions were prepared by:

Volker and van der Tuin	:	1.1. and 1.2
Le Huu Ti	:	2.1, 2.2, 3.3, 4.3, 4.4, 4.5, 5.1, 5.2 and 5.4.
Mikhailov	:	2.3, 3.2, 5.3, 8.3, 8.4, 8.5 and 8.6
Roelfzema	:	3.1, 4.1, 4.2, 6.1, 6.2, 6.3, 8.1 and 8.2
van der Tuin	:	7.1 and 7.2

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1. Introduction

1.1 Nature of the problem

Where rivers carrying freshwater debouch into a recipient basin with saline water (sea, ocean or inland sea) the saline water tends to propagate into the river mouth, affecting the quality of the water in the lower river reach. This is due to the difference in density between the fresh and the saline water. By virtue of its higher density (plus 2 to 3%) the saline water slides over the bottom of the river and moves in an upstream direction against the river flow. Depending on the specific conditions of the river mouth the saline water mixes to a smaller or larger degree with the freshwater above. This seawater intrusion can manifest itself at great distances from the coastline, especially when the river flow is small. Figures of some tens of kilometres frequently occur, and under extreme conditions the reach affected by the seawater intrusion may extend over more than two hundred kilometres.

The seawater intrusion may render the quality of the water in the lower river reach unsuitable for domestic, agricultural, industrial and other uses, thus damaging the interests of the people of very large areas in the lower portions of a river basin. This is especially the case during dry periods when the needs for freshwater are often maximum.

Seawater intrusion is a material phenomenon and occurs at all meeting places of fresh and saline water, though many human activities in the river basin may aggravate the situation. One of these is the deepening of river channels with a view to receiving deep draught vessels in harbours near the river mouth. The increase in the channel depth facilitates the flow of saline water in an upstream direction and the contaminated river reach extends further. In this respect a controversy exists between the interests of freshwater supply and navigation. In addition, human activities on the upstream portions of a river basin which influence the river flow ('upland discharge') have a direct impact on the seawater intrusion in the river mouth. Thus abstraction of water upstream for irrigation increases the saline intrusion. The same applies to the filling of storage reservoirs upstream. On the other hand releases from such reservoirs during dry periods may lead to a flow augmentation in the river mouth which to some extent pushes the saline water back.

Thus the river mouth system should be viewed as a part of the river basin as a whole. In the ideal situation a balanced development of the upstream parts and the river mouth portion of a river basin should be aimed at in order to obtain optimum benefits. It is therefore essential that tools are available to predict seawater intrusion and its effect under a wide range of conditions pertaining to channel configuration, upland discharge, water abstraction, low flow augmentation, flow diversion, etc. Unfortunately, seawater intrusion is a complex phenomenon and many processes involved have not yet been fully analysed. Although there are many approaches, formulae and mathematical models their validity and applicability in any given case can only be ascertained with reference to the results of measurements in the first instance of salinity and discharge under a wide range of conditions.

For the determination of the parameters involved measurements are again indispensable with respect to the case under consideration.

1.2 Objectives and scope of the publication

The objective of these guidelines on the study of seawater intrusion into rivers is to provide basic practical information for those engineers and scientists who are engaged in the physical development of coastal areas and its relation with the development of the upstream portions of a river basin but who are not necessarily specialized in coastal and estuarine research. This target group includes hydraulic engineers, agronomists, soil scientists and irrigation engineers. It is hoped that the guide-lines will also give useful information to scientists in allied disciplines such as geography, hydrology, oceanography, geology, and so on, and that they will offer a general survey of the topic to economists, social scientists and policy makers.

The study of seawater intrusion into rivers can be considered as a special branch of tidal hydraulics. The practical results of such studies are meant to give basic background information for the design of hydraulic works and, as stated before, to predict the consequences of natural or man-made changes in the river basin and river mouth upon seawater intrusion. These changes also affect the erosion and sedimentation processes at or near the sea–freshwater interface. The work involves the cooperation of many engineers and scientists of various disciplines. The publication at hand is designed to give to all directly or indirectly involved an overview of the problems with which they may be faced, the solutions that may be considered and the detailed studies by specialists that may be required.

The scope of the publication is governed by these objectives as well as by the present state of the art of the process of seawater intrusion. The study of seawater intrusion is still not a well established or documented discipline, and there is a proliferation of formulae and models. The approaches are still largely empirical, making it difficult to predict conditions outside the range of the actual observations. Flow and other transport processes as observed in a given river mouth form an unique combination governed by the unique individual features of a given estuary such as geometry, rugosity, tides, tributaries, flood plains, etc. There is little information on the applicability of formulae and models derived in a given situation to another.

In spite of the great practical significance of the problem of seawater intrusion research in this field is limited and in many cases even basic field data are lacking. This is especially the case in developing countries. In addition, available methods of prediction of changes in seawater intrusion are not entirely adequate for answering practical questions. Although this publication is not a state-of-the-art document we have attempted to take into account the latest developments as well as the experience obtained in many countries. Therefore the publication, although written for non-specialists, will hopefully also be of interest to those specialists practising salt/surface water intrusion studies.

2. Definition of problems of salinity intrusion into rivers

2.1 Salinity intrusion problems in water management

River mouth systems together with the fertile deltaic lands form the most productive areas in terms of agricultural production and fisheries resources. The extensive natural waterways suitable for navigation of the river mouth channels and the relative flat topography of the deltaic areas have also offered the best potentials for other economic development activities.

The multiplicity of exploitation of the rich resources of such an ecosystem often results in adverse effects on the resources and undesirable interactions between activities. This has called for a thorough investigation to determine possible impacts of each development activity on the system with a view to arriving at an optimum plan for development. For this kind of impact assessment study, interaction between the various systems and resources involved should be considered, since the benefits provided by all the resources would be taken into account and possible adverse effects included.

Experience gained in many river mouth systems in the world has shown that an increase in the flow during the dry period can substantially reduce seawater intrusion, as shown in Fig. 2.1. By contrast, reduction of freshwater inputs would aggravate the problem of seawater intrusion as experienced in the delta of the Arvandrud and Karum rivers of Iran (Gholizadeh, 1969).

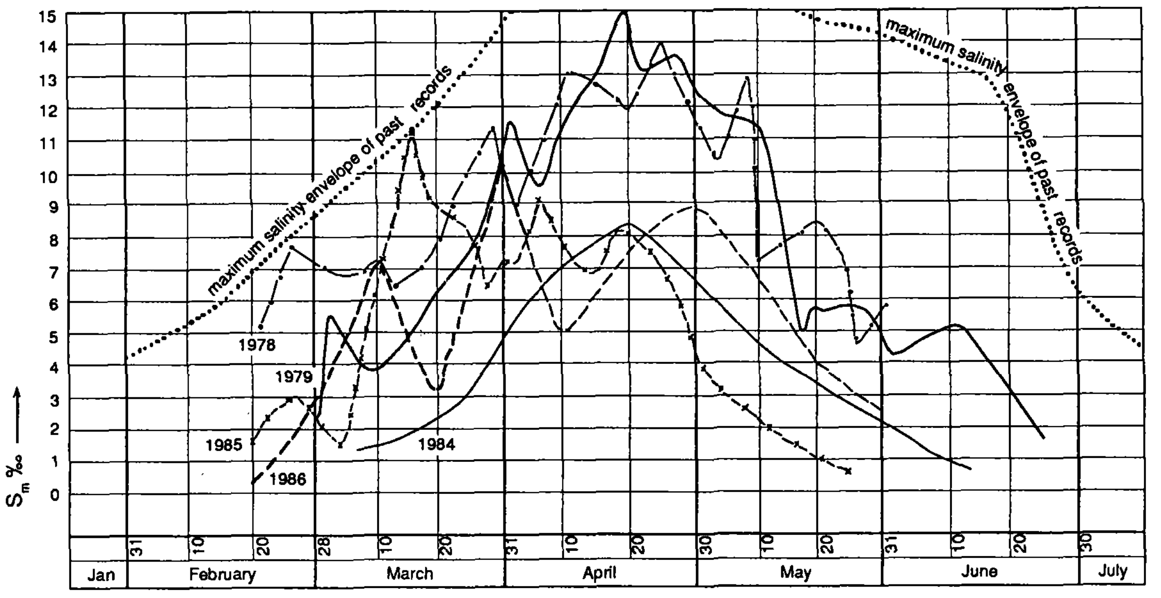


Fig. 2.1 Variation of maximum salinity at Tan An station in the west Vam Co River, one of the drains of the drainage routes of the Mekong Delta

However, management of river mouth water resources requires information not only on the variation in the extent of seawater intrusion, but also on the effects of such changes. Apart from the agricultural benefits, adverse effect can also take place, such as demonstrated by the case of the giant freshwater prawn *Macrobrachium rosenbergii* in the lower reaches of Asian rivers, which spawns in the brackish water of river mouth systems and migrates seasonally 200 km upstream. These giant prawns were abundant ten years ago, but efforts to improve agriculture have adversely affected spawning patterns and natural supplies are now scarce (Mekong Secretariat, 1987).

Water control works in complex river mouth environments, on the other hand, may create severe adverse effects when an annual equilibrium cannot be repeated. For example, in the Mekong delta, leaching of acid sulfate soils by saline water and then by freshwater is much quicker and requires less freshwater than leaching the acid sulfate soil directly by freshwater. The time saving and the reduced freshwater requirement, both important factors in agricultural development, need to be incorporated into a complete study for an overall development plan. A typical organization chart for such a study is shown in Fig. 2.2. In this figure, the hydraulic/water quality model refers to the modelling of hydraulic conditions of the river mouth system with the effects of seawater intrusion (salinity), and drainage of flood and/or acidic waters.

2.2 External factors influencing salinity intrusion in water management

In order to evaluate the impact of development activities on the water resources system, important factors including external ones have to be identified. However, study of such impact in the river mouth ecosystem is normally complicated, as demonstrated by Fig. 2.3 on a typical network

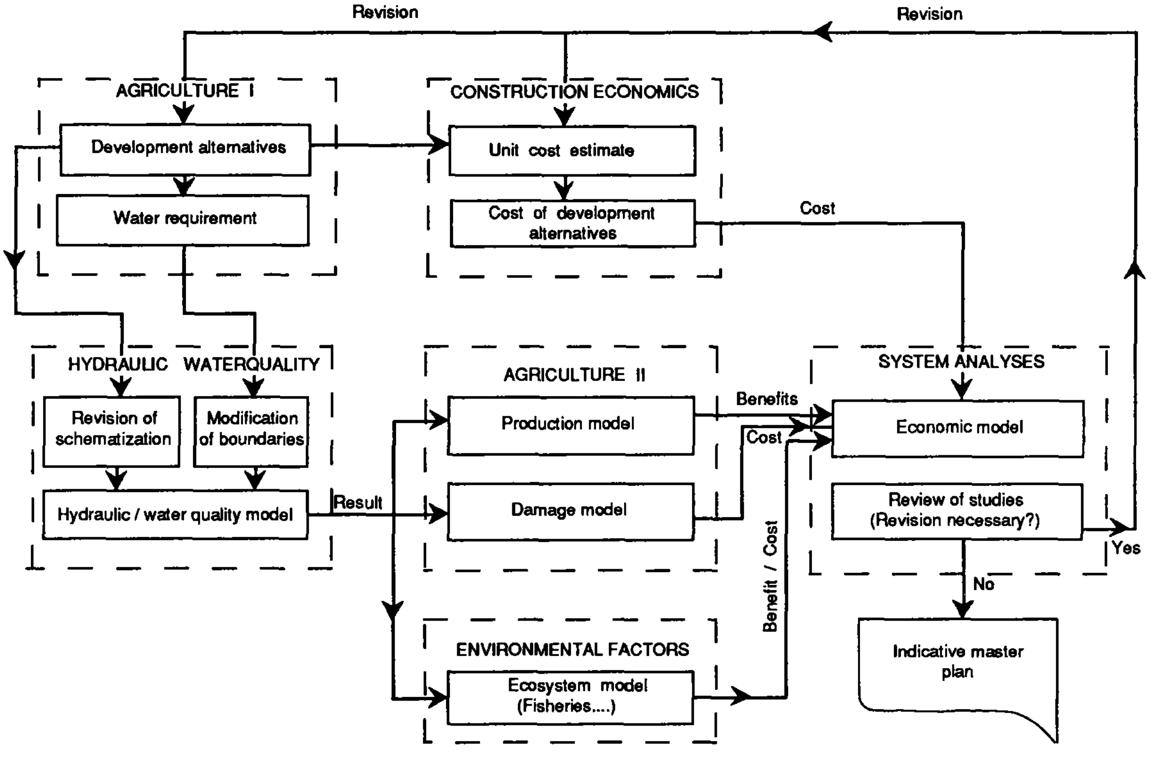


Fig. 2.2 Organisation of an indicative master plan study

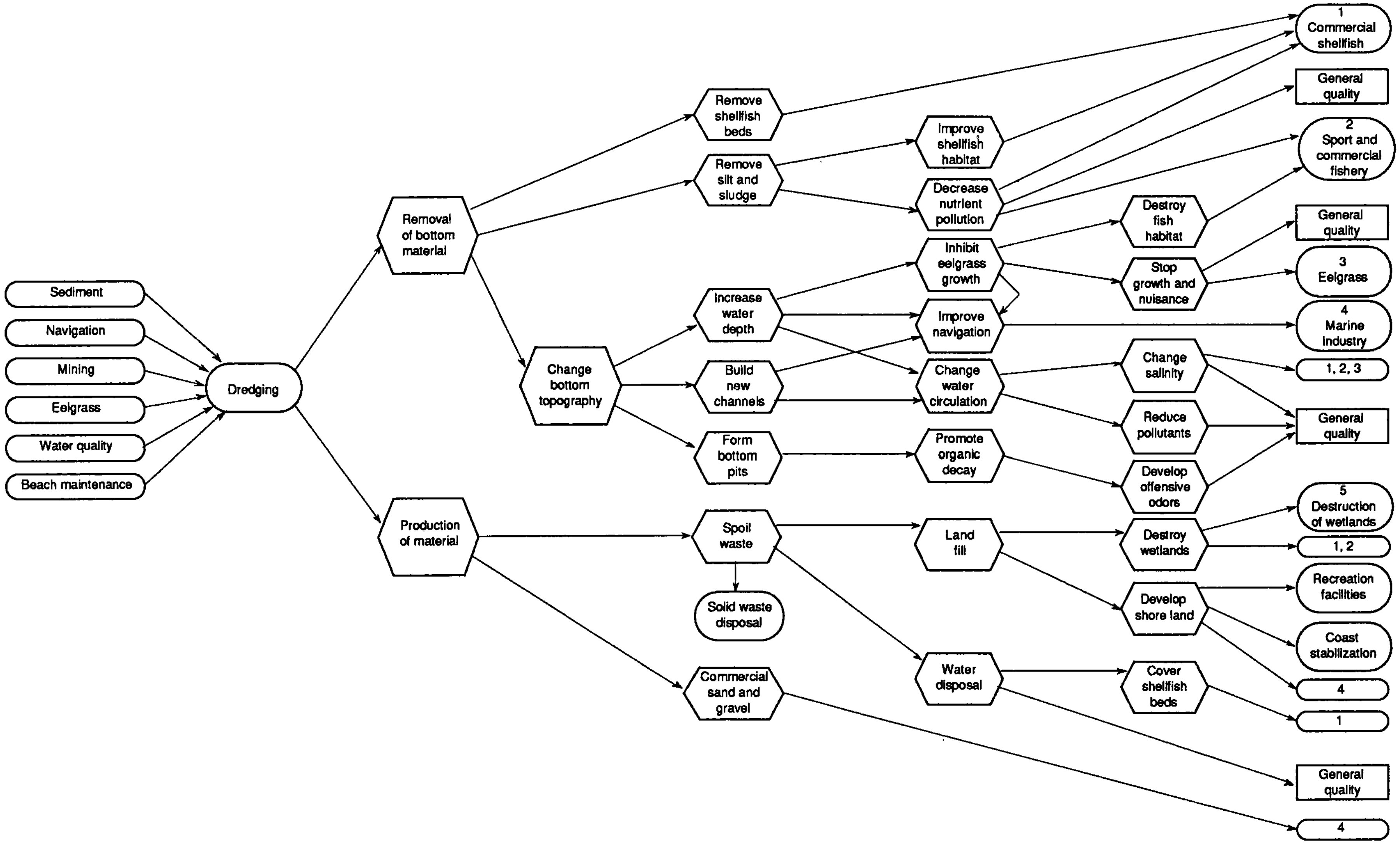


Fig. 2.3 Network analysis of the impacts of dredging (Sorensen, 1971)

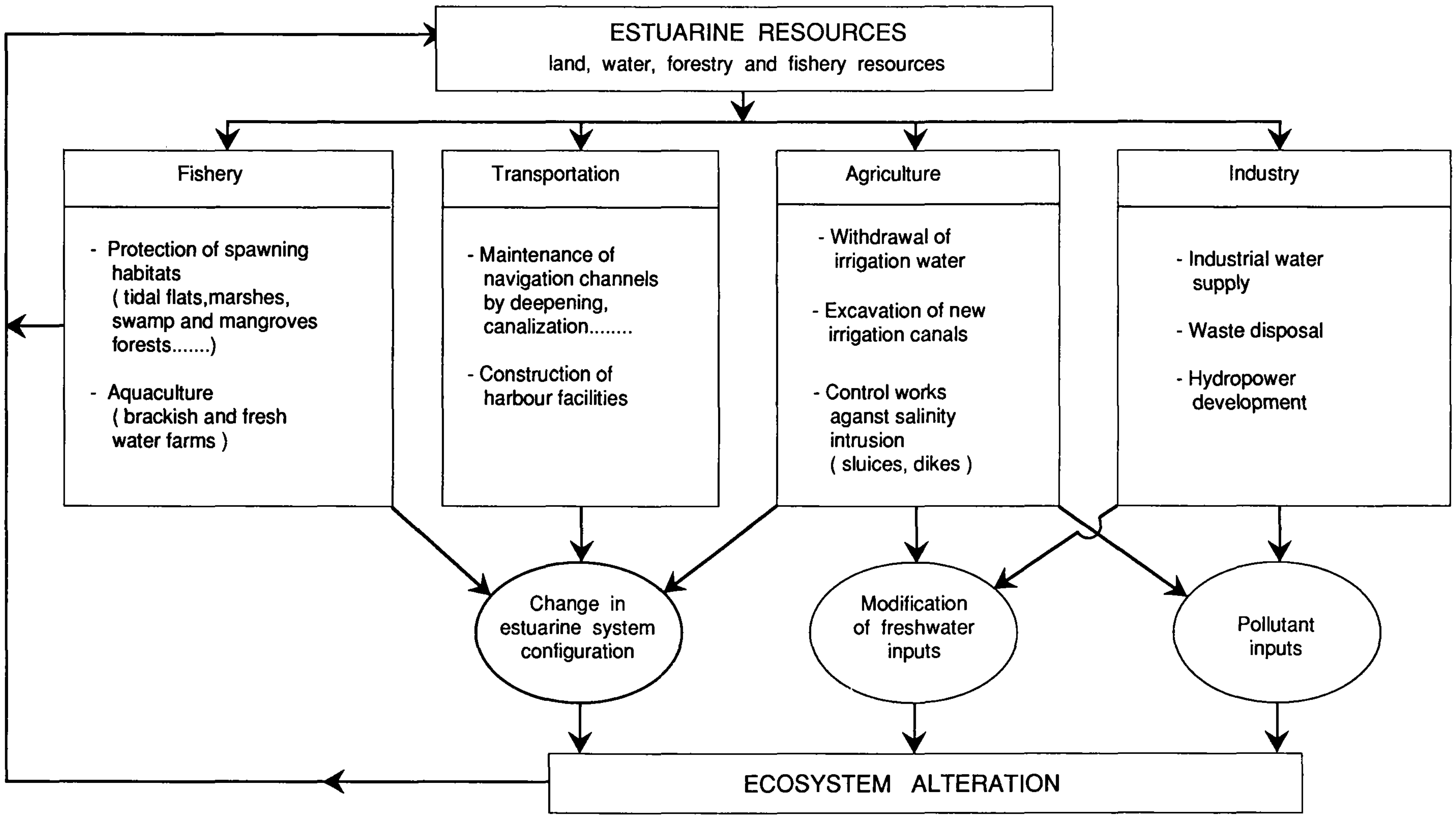


Fig. 2.4 Four major groups of development activities in estuarine ecosystem

analysis of the impacts of dredging. It will therefore be an almost impossible task for the present guidelines to address all these detailed interactions, in view of their complexity and great number of factors involved. It would be more realistic to confine ourselves to the water resources sector of Fig. 2.4, in which impacts can be expressed in terms of changes in the quantity and quality of water for subsequent use as inputs for the evaluation of impacts on the other sectors. For example, the reduction of salinity level of waters (water quality) may allow an increase in water supply for agricultural production and other related development activities but may affect the aquatic biological production systems of the brackish water zone. Changes in water levels (water quantity) can also be used to estimate possible effects on agricultural development and land resources and subsequently terrestrial biological production systems. For more details on the river mouth processes and impact assessment, readers can refer to Wiley (1976), Mekong Secretariat (1982) and Ketchum (1983).

2.2.1 Important factors of the seawater intrusion mechanism in impact assessment

The basic concept of seawater intrusion to be used in impact assessment is shown in Fig. 2.5. In this concept, the two factors that are susceptible to changes either by man-made or natural actions are the river mouth configuration and inputs of water. It should be noted that changes in the water inputs (fresh or salt) can be more descriptively related to the extent of saltwater intrusion than the changes in the river mouth configuration. Further elaboration will be made in the later part of Chapter 5 on changes in these two important factors by development activities and the consequent effects on seawater intrusion.

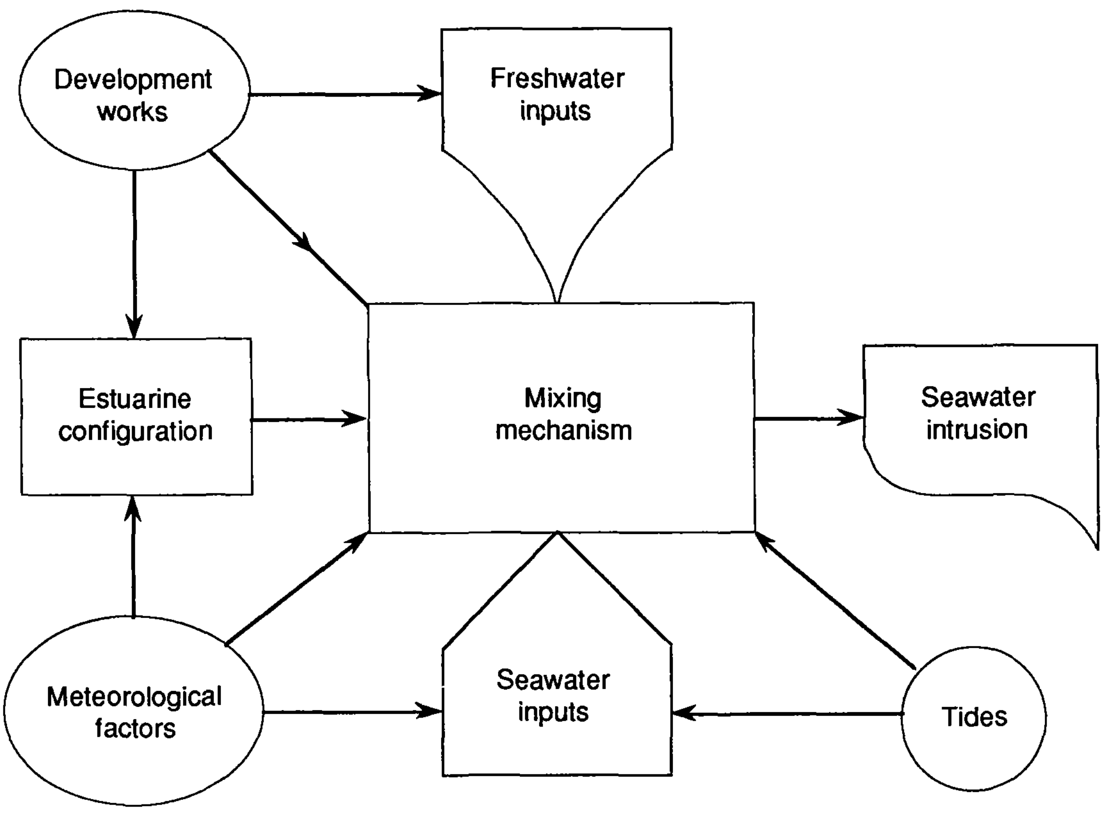


Fig. 2.5 Basic concept of seawater intrusion in impact assessment in the water resource sector

2.2.2 Changes caused by development activities

With respect to the use of the natural resources of a river mouth ecosystem in the developing countries, four most important groups of activities of economic development can be identified as agriculture, fisheries, industries and transportation. These activities are either aimed at controlling seawater intrusion or likely to cause important effects on seawater intrusion. The changes caused by these four groups of activity on the water resources system can be classified into three major areas: changes in the river mouth configuration, modification of freshwater inputs and inputs of pollutants. A summary chart of these activities has been given in Fig. 2.4.

The quality of the water in the river mouth is not only affected by seawater intrusion but also by the adverse effects of the disposal of pollutants. Experience in many river mouths has confirmed that the effects of pollutant inputs are normally profound and long-lasting. These inputs are not confined to chemical disposal from industrial plants but also include the run-off of chemical fertilizers and insecticides from irrigated agricultural sectors or concentrated drainage of acidic waters leached from reclaimed lands. Since this aspect of the exploitation of river mouth systems is beyond the scope of these guidelines, discussion will be only made in Chapter 5 on impacts of changes in river mouth system configurations and the modification of water inputs.

2.2.3 Classification of changes caused by natural factors

There are two groups of changes caused by natural factors: changes in the water inputs and changes in the topographical conditions. The change of water inputs can appear either in the direct form of quantity of water inflow or through the change of water levels. The first group would include the variation of freshwater streamflows, rainfall, evaporation, variation of tidal fluctuation and changes in water level caused by strong winds and storms. The second group is mainly associated with sediment movement in the river mouths and along the coastline and partly with exceptional events such as storm surges.

2.3 Types of river mouth

2.3.1 Classification of river mouth areas

River mouth areas (or more briefly, river mouths) are defined as specific geographical objects situated at places of river inflow into recipient basins (oceans, seas or lakes) and as zones of dynamic interaction, mixing of river and seawater, and of the deposition of riverine and/or marine sediments.

As a hydrographic system a river mouth area can be divided into two parts:

(a) the river part, where a fluvial hydrological regime predominates, though sea influences may occur actively (tidal and storm surge water level variations and intrusion of salt seawater); and
(b) coastal part, where a marine hydrological regime predominates, though river influences may occur actively (significant freshing of seawater due the river flow).

River and coastal parts of the river mouth area are separated from each other by a coastline cross-section of the river (or of a delta branch) and by a delta coastline. River mouth areas can be classified by morphology, river or sea hydrological regime and the character of the mixing of river and seawater.

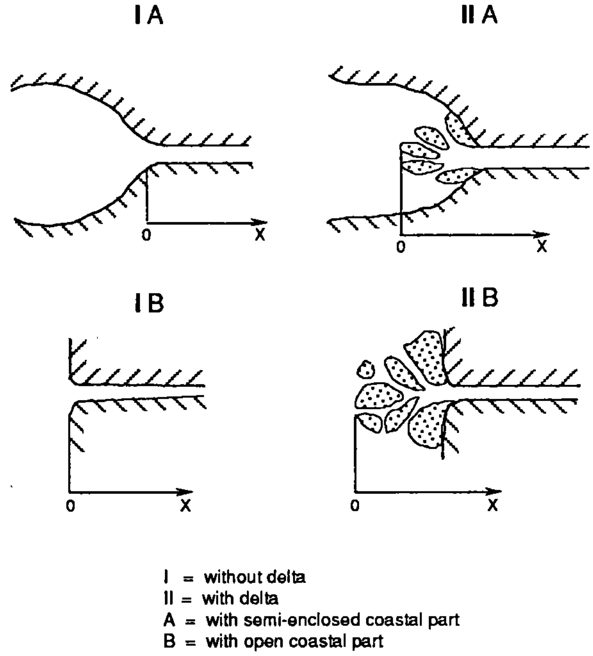


Fig. 2.6 Classification of river mouth areas

Morphologically river mouth areas are divided into two main types: those without deltas (I) and those with deltas (II) (Fig. 2.6). River mouth areas can be also divided into two classes: those with semi-enclosed (A) and those with open coastal parts (B). Semi-enclosed coastal parts of the mouth areas are represented by narrow sea bays, limans, lagoons, estuaries. The deltas developing in semi-enclosed coastal parts of the mouths are called as inner (or filling) deltas; the deltas, developing in open coastal parts of the mouths are called as outer or protruding deltas.

Thus there are four subtypes of river mouth areas: IA – without delta and with semi-enclosed coastal parts (for example the mouths of the South Bug River, the Mesen, the Thames, the Delaware, etc.); IB – without deltas and with open coastal parts (the Rotterdam Waterway, mouths of small rivers flowing into the ocean); IIA – with inner (filling) deltas in semi-enclosed coastal parts (the mouths of the Ob, the Enisei, the Don, the Dnieper, the Vistula, etc.); IIB – with outer (protruding) deltas in open coastal parts (the mouths of the Volga, the Danube, the Mississippi, the Rhone, the Hwang Ho, etc.). In the course of the deposition of river sediments the subtype of river mouth area changes in following way: IA → IIA → IIB and IB → IIB (Fig. 2.6). The evolution of a river mouth area leads to an advance of the delta coastline in a seaward direction.

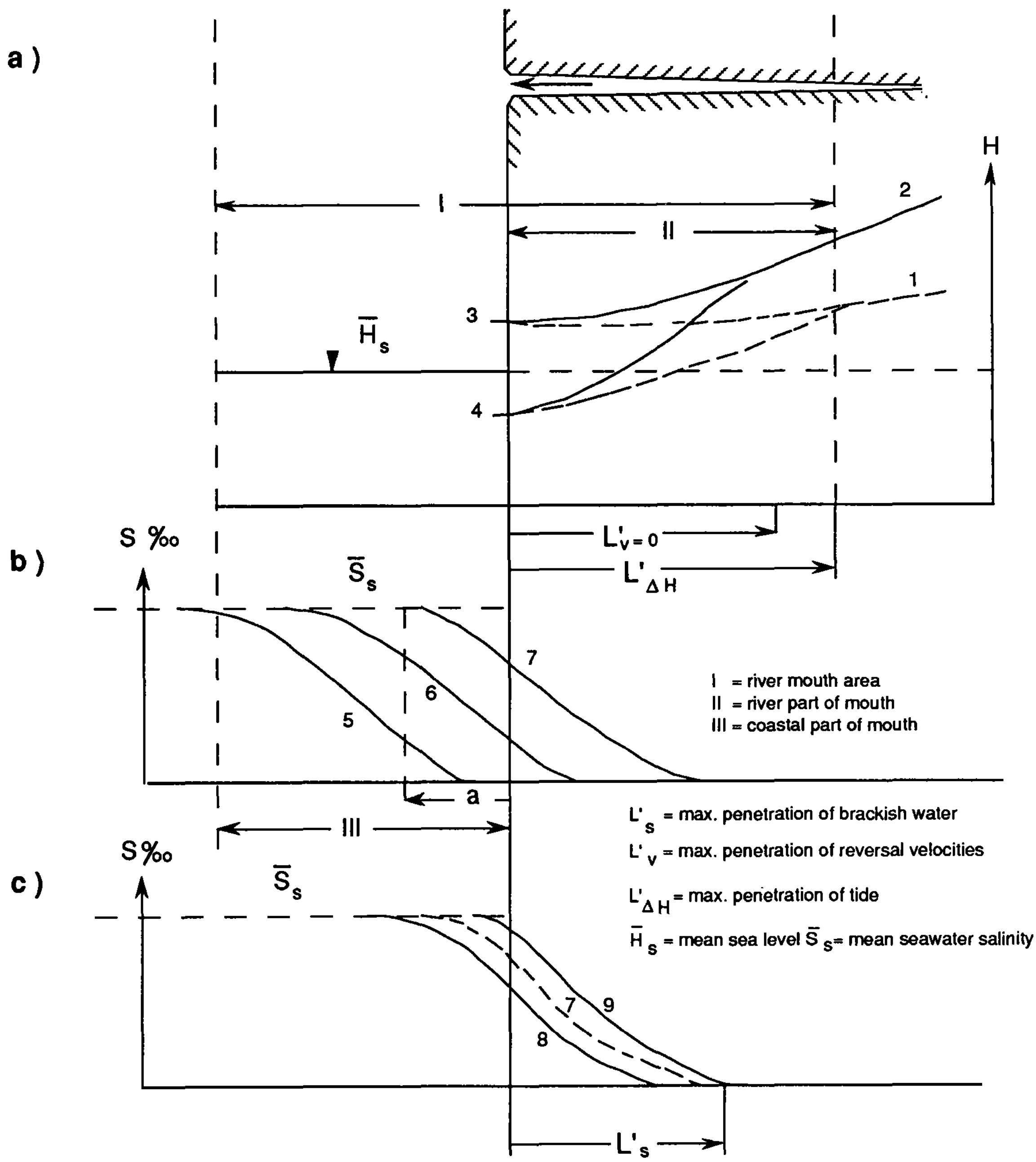
With regard to river (fluvial) hydrological regime, river mouth areas are divided into: (1) river mouths with a well defined river flooding period due to snow melting or long rainfall, and (2) river mouths with flushy rainfall floods.

As far as sea (marine) hydrological regimes are concerned, river mouth areas can be divided into: (1) tidal, and (2) non-tidal. River mouths are considered tidal when the tidal range is above 0.3 m. Storm surges take place in practically all river mouths. Besides that river mouth areas can be divided, by the character of the recipient basin, into: (a) those of a sea type (in this case the river debouches as a rule into a saltwater sea), and (b) those of a lake type (in this case the river generally flows into a freshwater lake).

According to the character of vertical mixing of river and seawater three types of river mouth areas can be distinguished: (1) well mixed river mouths (the water density being practically constant over the depth and varying in longitudinal direction); (2) partially (moderate) mixed river mouths (the water density changing continuously both over the depth and longitudinal direction); and (3) river mouths with a saltwater wedge (the water density changing sharply at the interface of fresh and saltwater).

2.3.2 The mouth mixing zone

Independently of their types, river mouth areas always include a mixing zone of river water and water of the recipient basin, called the mouth mixing zone. A mouth mixing zone is more typical for river mouths debouching into seas with saltwater, where fresh river water and salt seawater with different physical, chemical and biological properties mix. Within the mouth mixing zone the salinity changes from the salinity of river water (usually not more than 1‰) to that approaching the salinity of seawater (as much as 10–40‰ in certain seas).



a) longitudinal profile of water level with a low (1) and a high (2) river discharge at flood (3) and ebb tide (4).
b) longitudinal profile of water salinity averaged over the tidal cycle with a high (5), a moderate (6) and a low (7) river discharge.
c) idem b, with a low river discharge salinity profile averaged over tidal cycle (7), at Low Water Slack (8) and at High Water Slack (9).

Fig. 2.7 Scheme of the mouth mixing zone and the division of the mouth area

A large part of the mouth mixing zone is usually situated in the coastal part of the mouth area. The outer (marine) edge of the coastal part of the mouth area (and of the mouth area as a whole) is determined by the extreme propagation of the sea edge of the belt of maximum water salinity gradients into the sea at large river discharges. This belt corresponds to the most active mixing of river and seawater (Fig. 2.7). It is usually considered that water salinity in this outer edge of the coastal part of the mouth area and mouth mixing zone is equal to 90-95% of the mean seawater salinity.

The upstream limit of the mouth mixing zone (usually determined by a salinity of 1‰) in different river mouths and in different periods of time lies either in the coastal part of the mouth near the delta coastline or in delta branches (or in the narrow part of the estuary). The mouth mixing zone can spread into delta branches during low river discharges (Fig. 2.7b), during tides (Fig. 2.7c) and storm surges. In these cases seawater intrusion occurs into the river part of the mouth.

In the river part of the mouth the following zones can be distinguished according to the character of the dynamic interaction and mixing of river and seawater (Figs 2.7 and 2.8):

1. The zone where water level variations due to tides and storm surges occur. The smaller the river discharge (Q_r) and water slope (I) and the greater the tidal range and storm surge in the sea (ΔH_s), the greater the distance of penetration of tides and storm surges into the river ($L_{\Delta H}$).
2. The zone where reversal currents connected with tides and storm surges are observed. The relationship between the length of this zone ($L_{v=0}$) and the determining factors (Q_r, I, ΔH_s) is qualitatively the same as for $L_{\Delta H}$. The value of $L_{v=0}$ is the greatest at the moment of high water slack (HWS) with a low river discharge.
3. The zone with brackish water. The greater the salinity of seawater (S_s), channel depth (h) and tidal range or storm surge and the smaller the river discharge (Q_r), the larger the penetration of brackish water into the river (L_s). Two subzones (Fig. 2.8) can be distinguished at a fixed Q_r within this zone: (a) where brackish water is always present, and (b) where it occurs only during flood tide or storm surge.

In some river mouths with large river flow and with a narrow coastal part (for example the mouths of the Ob and Enisey) the brackish waters never penetrate into the delta branches, the mixing zone being always situated in the coastal part of the mouth. In many cases the brackish waters only penetrate in extreme situations: namely during very low river discharges with very high tides and/or storm surges. In these mouths subzone 3a is absent. In any river mouth and at any time the following relationship between lengths of three above-mentioned zones can be obtained and established (Fig. 2.7):

$$L_{\Delta H} > L_{v=0} > L_s \tag{2.1}$$

The same relationship can also be attributed to the extreme values of the penetration of tidal (storm surges) water level variations, reversal currents and brackish waters (these values are marked by strokes) (Fig. 2.7):

$$L'_{\Delta H} > L'_{v=0} > L'_s \tag{2.2}$$

2.3.3 Types of mixing of river and seawater in river mouths

There are three types of mixing of river and seawater over the depth in river mouths: (1) well (complete) mixed; (2) partially (moderate) mixed; and (3) saltwater wedge (see Table 2.1).

In the first case the water density changes slightly over the depth; density stratification is

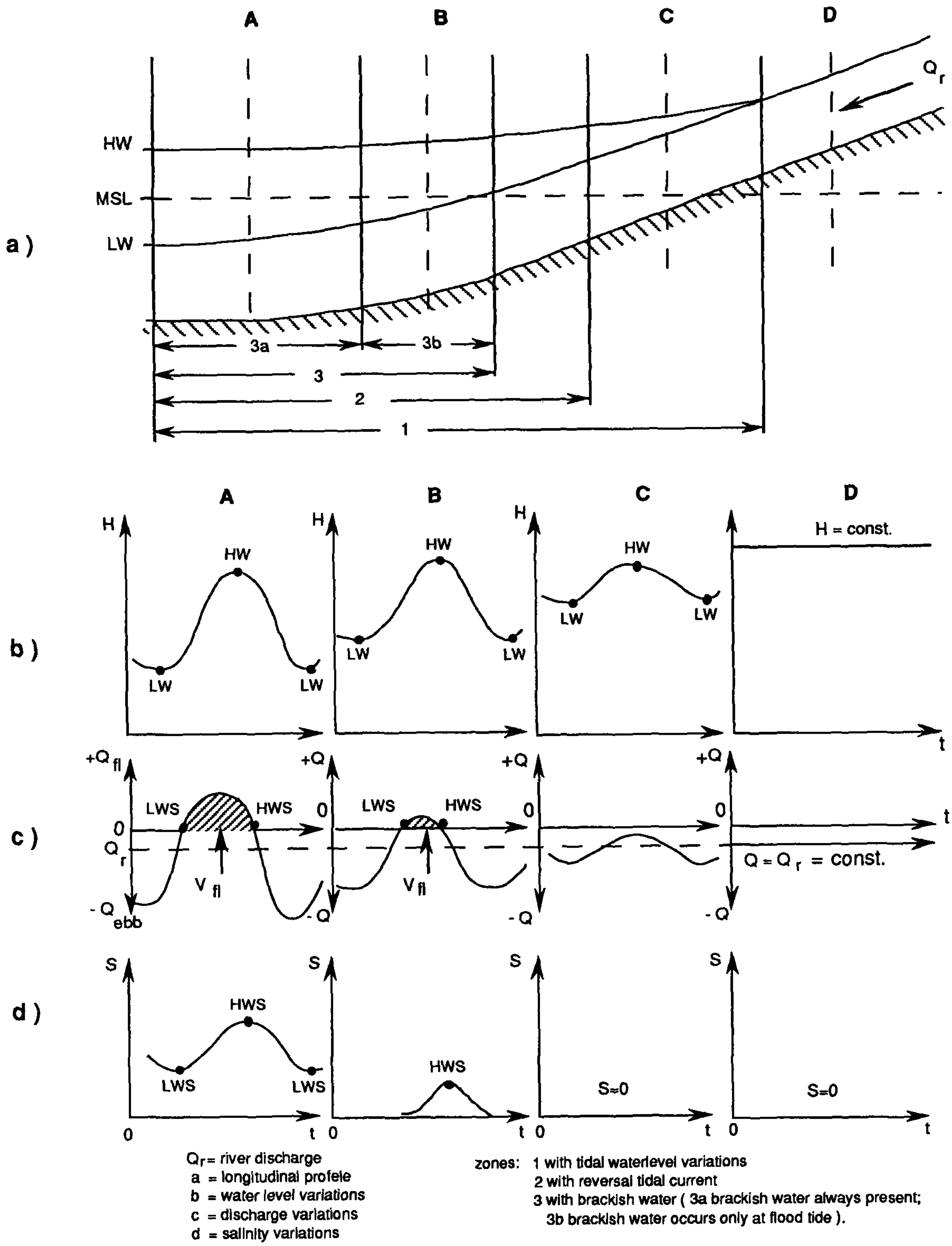
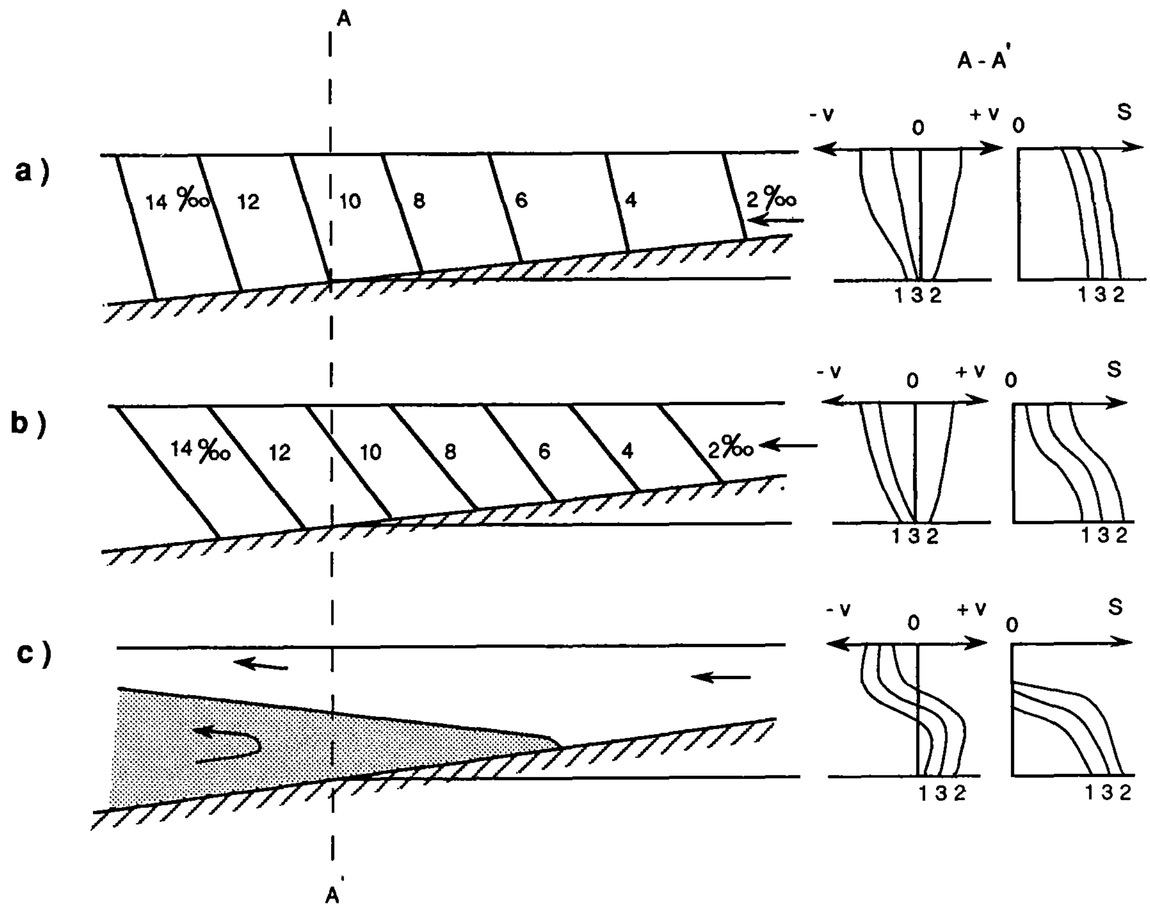


Fig. 2.8 Scheme of tidal aspects in river part of the mouth:
A, B, C are cross sections within the tidal reach and D in the non-tidal part of the river.



a) well mixed
b) partly mixed
c) saltwater wedge

1 = flood tide
2 = ebb tide
3 = averaged over tidal cycle

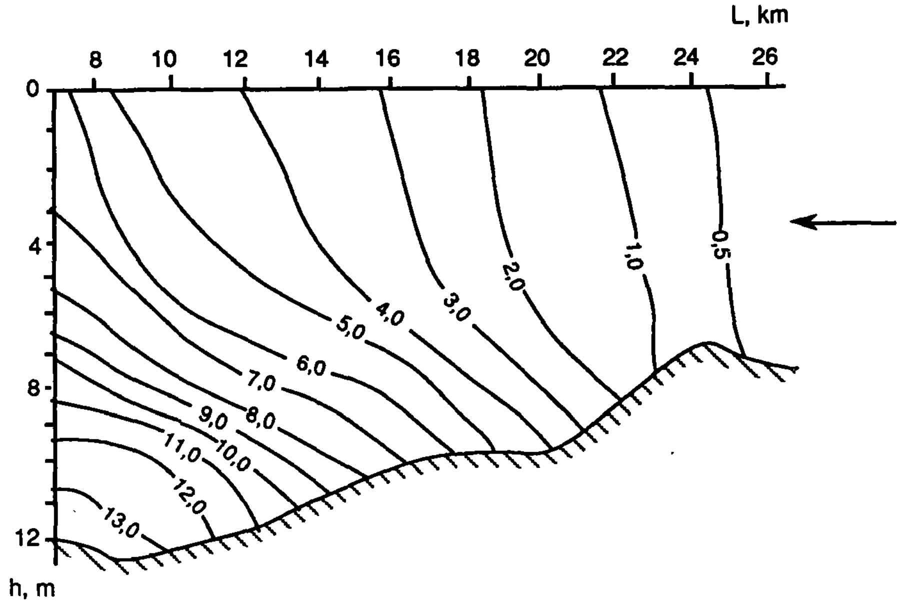
Fig. 2.9 Scheme of river and seawater mixing with corresponding velocity and salinity distribution at cross section A–A'

smooth (Fig. 2.9a). In the second case the water density continuously increases from surface to bottom, and the flow is moderately stratified (Fig. 2.9b). In the third case the water density changes sharply at the interface between upper layer of freshwater and underlying layer of saltwater, the flow being strongly stratified (Fig. 2.9c).

The case of thorough mixing usually corresponds to low river discharges and large tides. The saltwater wedge is usually observed as a contrary during large river discharges and low tides. The saltwater wedge is typical for deep non-tidal river parts of the mouths. The case of partial mixing occupies an intermediate place between the well mixed situation and the saltwater wedge. Wind and waves favour the mixing process.

As a function of governing factors (river discharge, tidal range and tidal phase, wind, waves) the type of mixing of river and seawater can change both in time and over the area of the river mouth. Thus in several mouths during periods of large river discharge the saltwater wedge can be observed, but during low river discharge well or partial mixing may take place. In deep and narrow river mouth branches the rate of density stratification may be stronger than in shallow channels.

a) **Well and partly mixing**
The mouth of the North Dvina River (U.S.S.R.) according to Y. V. Lupachev (1976)



a) **Saltwater wedge**
Sulina Branch of the Danube delta (Rumania) according to C. Bondar (1986)

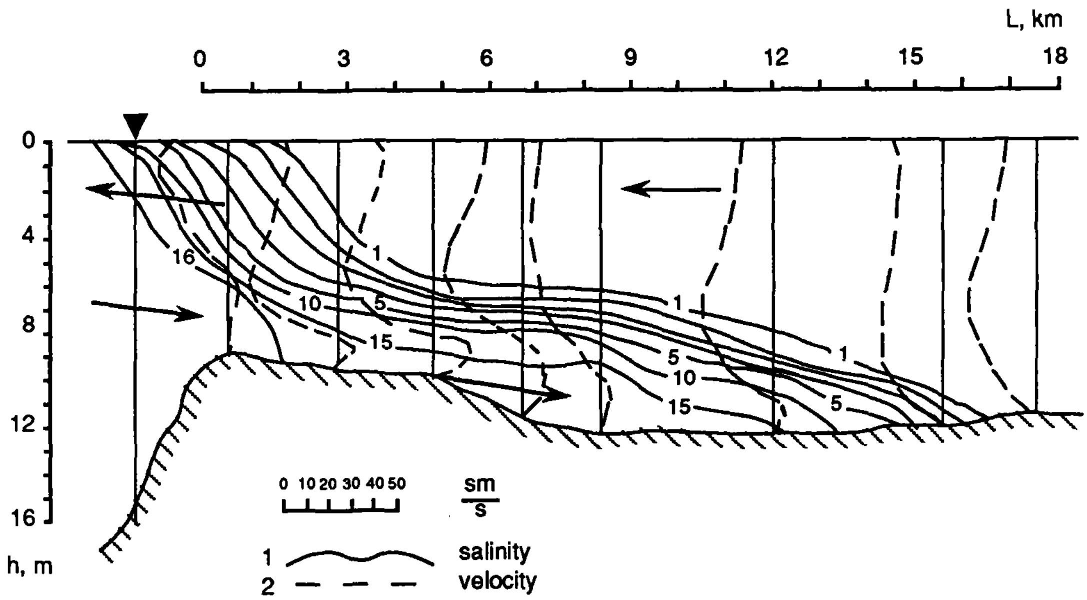


Fig. 2.10 Typical cases of the salinity (‰) distribution along the rivermouth

For a quantitative estimation of the vertical mixing degree and simultaneously of the degree of density stratification one can use the stratification parameter n, which is equal to:

$$n = \frac{\Delta S}{S_m}, \tag{2.3}$$

where $\Delta S = S_{bott} - S_{surf}$, $S_m = 1/2\,(S_{bott} + S_{surf})$,

S_{bott} is water salinity at the bottom, S_{surf} is water salinity at the water surface. Usually it is considered that if $n < 0.1$, well mixing is observed, if $0.1 < n < 1.0$ partial mixing takes place, if $n > 1.0$ the saltwater wedge is observed (Table 2.1).

Equation (2.3) characterizes not only the salinity vertical gradients but also the density vertical gradients, because the density of brackish or saltwater depends primarily on water salinity (Appendix 1).

Examples of different types of mixing and stratification types in river mouths are shown in Fig. 2.10.

The types of mixing and stratification a the consequence of dynamic processes in river mouths, which may be also evaluated by the following parameters: flood parameter α, estuary number E, internal estuary number E_ρ and Richardson's estuary number Ri_E.

Flood parameter α is defined according to Canter Cremers and H. B. Simmons as the ratio of the volume of river water coming down the river mouth during a tidal cycle and the flood volume:

$$\alpha = \frac{Q_r T}{V_{fl}}, \tag{2.4}$$

where Q_r is river water discharge averaged over the tidal period, T is the duration of the tidal cycle (the tidal period), and V_{fl} is volume of the seawater entering into the river mouth during the flood tide (or volume of tidal prism, $V_{fl} = P_t$).

Sometimes this parameter is expressed in another way:

$$\alpha^1 = \frac{Q_r T_{fl}}{V_{fl}}, \tag{2.4a}$$

where T_{fl} is the duration of flood phase of the tide.

Flood parameter α has some inconvenience as it turnes into infinity with the absence of tides. It is therefore more convenient to express this parameter in modified form:

$$\beta = \frac{V_{fl}}{Q_r T}. \tag{2.5}$$

Harleman and Abraham (1966) proposed a parameter for the description of the stratification in river mouths which they called estuary number E:

$$E = \frac{V_{fl} Fr_0^2}{Q_r \cdot T} = \frac{Fr_0^2}{\alpha}, \tag{2.6}$$

where Fr_0 is the Froude number equal to $v_0/\sqrt{gh}$.

Here v_0 is the maximum value of the cross-section averaged velocity during flood tide, h is the time averaged channel depth, g is the acceleration of gravity, and v_0 and h are defined at the coastline edge of the river channel. In equation (2.6) α is the flood parameter according to Canter Cremers and Simmons (2.4).

M. L. Thatcher and D. R. F. Harleman (1972) introduced another parameter called by them as international estuary number E_ρ:

$$E\rho = \frac{V_{fl} Fr^2}{Q_r T} \rho_0 = \frac{Fr}{\alpha} \rho_0 = \frac{E}{\Delta\rho / \rho_m}, \tag{2.7}$$

where $Fr\rho_0$ is the internal densimetric Froude number that is equal to $Fr_{\rho_0} = v_0 \Big/ \sqrt{\frac{\Delta\rho}{\rho_m} \cdot gh}$

Here $\Delta\rho$ is density difference between seawater (ρ_s) and river water (ρ_r): $\Delta\rho = \rho_s - \rho_r$ and ρ_m is equal to $1/2\,(\rho_s + \rho_r)$. v_o and h are as explained above. α resp. E is flood parameter resp. estuary number.

Richardson's estuary number Ri_E was proposed by Fischer (1972) and is equal to

$$Ri_E = \frac{\frac{\Delta\rho}{\rho_m} \cdot g \cdot Q_r}{B \cdot \overline{v}_t^3} \tag{2.8}$$

where Q_r is river discharge averaged over the tidal period, B is the width of the river channel, and v_t is root mean square of the tidal velocity averaged over the cross-section;

$$\overline{v}_t = \left[\frac{1}{T} \int_0^T v_t^2 dt \right]^{\frac{1}{2}}$$

Generalized criterion values of above mentioned parameters for different types of mixing of river and seawater and stratification in river mouths are represented in Table 2.1.

Table 2.1: Types of mixing and stratification in river mouths and respective values of parameters

N	Type of mixing of river and sea water over the depth	Type of density stratification over the depth	River water dis-charges	Tides	n after (2.3)	α after (2.4)	β after (2.5)	E after (2.6)	E_ρ after (2.7)	Ri_E after (2.8)
I	Well (complete) mixed	Weak stratification	small	high	0 – 0.1	0 – 0.1	> 1.0	> 0.2	> 8	< 0.08
II	Partial (moderate) mixing	Moderate stratification	medium	medium	0.1 – 1.0	0.1 – 1.0	1.0 – 0.1	$0.2 - 5 \cdot 10^{-3}$	8 – 0.2	0.08– 0.8
III	Salt water wedge; limited entrainment of water into fresh flow at the interface	Strong stratification	large	low	> 1.0	> 1.0	0.1 – 0	$< 5 \cdot 10^{-3}$	< 0.2	> 0.8

2.3.4 General hydrological information on seawater intrusion into rivers

Hydrological information on seawater intrusion into the rivers is unfortunately very poor. This can be explained by the very complicated character of river mouth processes as a whole and in particular with seawater intrusion. Some hydrological information of seawater intrusion into non-tidal rivers was provided by Scriptunov and Lupachiev (1982). This process was studied in the mouth of the West Dvina (Daugava) River by Rogov, Romashin and Shteinbach (1964), in the mouth of Sulina branch of the Danube delta by Bondar (1972, 1986). General information for non-tidal rivers (critical river water discharges Q_{cr} at which seawater intrusion into rivers begins and the maximum lengths of seawater intrusion L_s^1) is shown in Table 2.2. The most general regularities of seawater intrusion into non-tidal rivers are the following. In many cases this intrusion is in the form of a saltwater wedge entering into the river part of the mouth at the bottom (Figs. 2.9c, 2.10b). The length of this saltwater wedge in a river or in a delta branch increases rapidly at decreasing river discharge.

Table 2.2: Information on seawater intrusion into rivers

River mouth (delta branch)	Critical river water discharge Q_{cr}, in m^3/s	Maximum length of penetration of brackish or saltwater into river L_s', in km
Non-tidal river mouths		
West Dvina (Daugava) (USSR)	1000 - 1200	27 - 28
Danube, Sulina branch (Romania)	1280	20 - 30
Lielupe (USSR)	40	45
Kura (USSR)	-	27
Kuban (USSR)	-	10 - 15
Don (USSR)	-	15
Yana (USSR)	2400	40
Tidal river mouths		
North Dvina (USSR)	10000	43 - 45
Onega (USSR)	750 - 1000	10
Petchora (USSR)	4000	5 - 7
Senegal (Senegal)	750	300
Mississippi (USA)	-	240
Mekong (Viet Nam)	-	40

In many river mouths under natural conditions saltwater does not usually penetrate into delta branches due to the existence of shallow river mouth bars. But after artificial deepening of these river mouth bars for navigation the process of saltwater intrusion begins in many cases at low river discharges. This phenomenon was observed in the mouths of the West Dvina, in several branches of the delta of the Danube (Fig. 2.10b), the Yana, the Don, etc. This kind of phenomenon was also observed in several tidal mouths of large rivers, for example in mouth of the Mississippi. Brackish water usually penetrates into tidal river mouths during flood tide phase and at low river discharges to remarkably longer distances than in non-tidal river mouths (Table 2.2).

In the mouth of the North Dvina River with a tidal range of about 1.0 m seawater intrusion (Figs 2.10a, 2.11a) according to Lupachiev (1976) begins at river discharges below 10,000 m^3/s. In this river mouth at very low river discharges (400 - 500 m^3/s) the maximum lengths of penetration of the tidal level variation $L^1_{\Delta H}$, the reversal flow current $L^1_{v=0}$ and the brackish water L^1_S are equal to 140, 80 and 45 km respectively, which is in good confirmity to expression (2.2). More details of seawater intrusion into the North Dvina River will be described in Chapter 8 (Section 8.3).

Seawater begins to penetrate into the Senegal River according to Gac, M. Carn and Soas (1986) at river discharges below 750 m^3/s. The tidal range in this mouth is about 1.0 m, the length of penetration of tidal water level variations into the river is about 400 km. The length of seawater intrusion increases very quickly with decreasing of river discharge and reaches nearly 300 km (Fig. 2.11c):

Q m^3/s	750	700	600	500	400	300	200	100	50	20	10	5	1	0.5
L_S km	0	7	22	35	50	63	81	120	155	180	195	205	235	270

Within very small river discharges in the mouth of the Senegal River (as also in many others river mouths under very dry climatic conditions) the phenomenon of a 'reverse estuary' can be observed. In this case water salinity in the river part of the mouth at some distance from the sea does not decrease but increases due to strong evaporation of water (Fig. 2.11c).

The most general regularities of seawater intrusion into tidal rivers are the following:

- The distribution of water salinity over the depth is usually more homogeneous than in non-tidal rivers.
- Although at the bottom salinity is usually a bit more than at the surface (Figs. 2.10a, 2.11a) well or partial mixing of river and seawater is usually observed.
- Water salinity at the bottom decreases in an upstream direction usually more rapidly than at the surface (Fig. 2.11a).
- The distribution of water salinity along the river part of the mouth depends at first on the tidal phase (Figs. 2.7c, 2.11b).

In various river mouths the value of the tidal excursion of the water with the same salinity during the tidal cycle is different. In the mouths of the Pungu, the Limpopo, the Incomati and the Maputo in Africa the value of tidal excursion according to Savenije (1986) is from ± 3.5 to ± 7.0 km relatively to the mean position (Fig. 2.11b). In the mouth of the North Dvina River this value is ± 3 km. Storm surges can increase this tidal motion of water still more, for example in the mouth of the North Dvina River to ± 5 to ± 8 km. In tidal river mouths (as well as in non-tidal river mouths) a good dependence of longitudinal salinity distribution on seasonal river flow variations is observed (Figs. 2.7b, 2.11c).

In spite of the very important practical significance of the study of seawater intrusion into rivers the process is not satisfactorily known. Data from field observations are poor. Several hydrodynamic solutions of seawater intrusion problems based on two- and three-dimensional mathematical models of turbulent stratified flow (Thatcher and Harleman, 1972; Fischer, 1972; etc.) may be used in practice with great difficulty due to the absence of experimental data of turbulent diffusion and dispersion coefficients (McDowell and O'Connor, 1977). Therefore to solve engineering problems the application of more simple one-dimensional hydrodynamic models, simple balance approaches and semi-empirical methods is more realistic (see Chapters 3 and 6). Even so, such methods must be applied with care and validated with reliable experimental data.

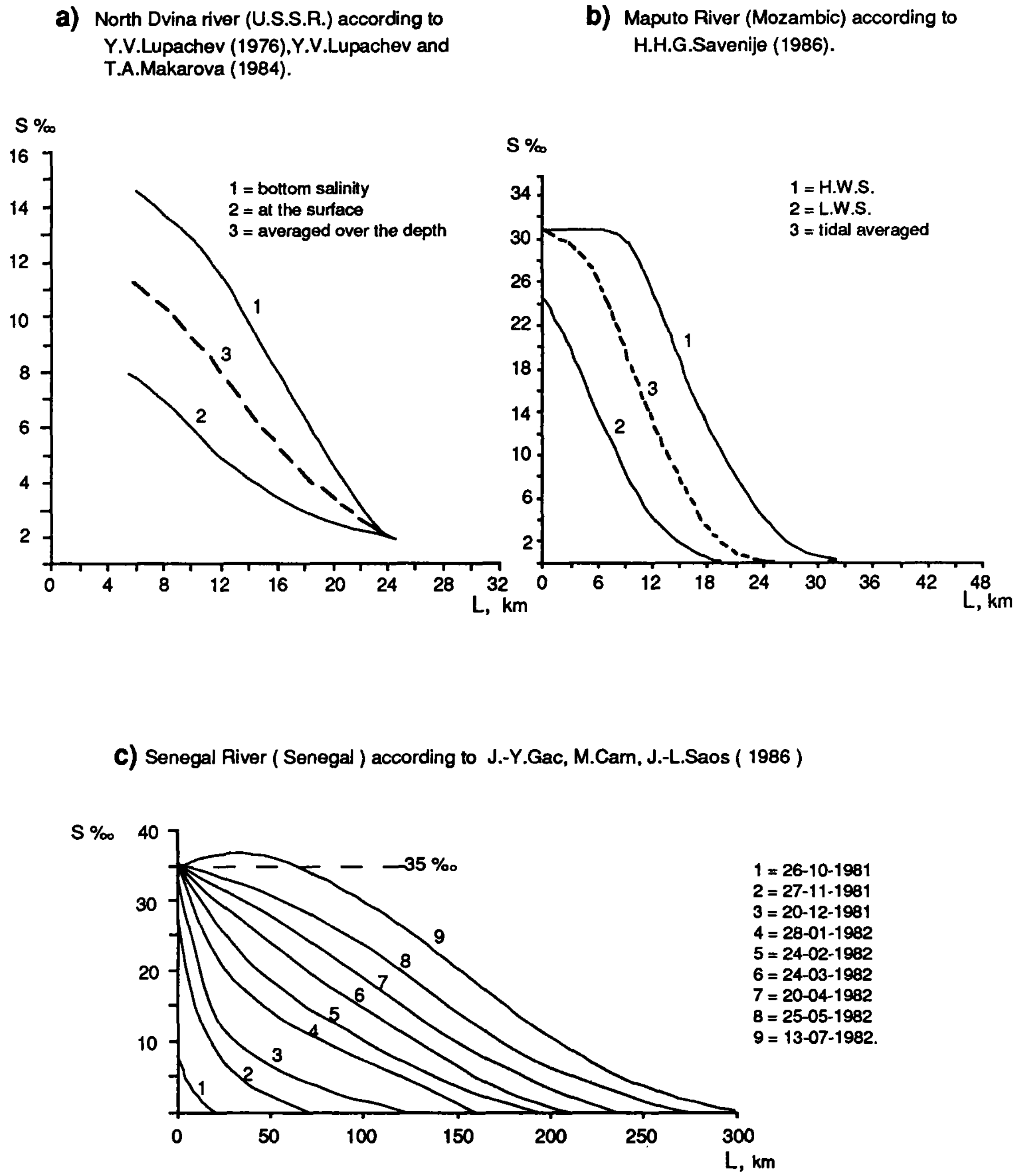


Fig. 2.11 Typical longitudinal salinity profiles along tidal river mouth

List of notations

A	-	cross-sectional area of flow, m^2
a, b	-	numerical constants
B	-	channel width, m
C	-	Chezy's coefficient, $m^{\frac{1}{2}}/s$
D	-	dispersion coefficient, m^2/s
E	-	estuary number (see equation (2.6))
E_ρ	-	internal estuary number
f	-	friction coefficient at interface of an arrested saltwater wedge
Fr	-	Froude number $(Fr = v/\sqrt{gh})$
Fr_ρ	-	densimetric Froude number $(Fr_\rho = v/\sqrt{\frac{\Delta\rho}{\rho} gh})$
g	-	acceleration due to gravity, m/s^2
H	-	water surface elevation, m
HW	-	high tide water, m
HWS	-	high water slack
h	-	water depth, m
K	-	Van der Burgh's constant
k	-	numerical constant
$L_{\Delta H}$	-	length of penetration of tide or stormsurge waterlevel variations into river, km
$L'_{\Delta H}$	-	maximum value of $L_{\Delta H}$, km
$L_{v=0}$	-	length of reach with reversal flow velocity during tide or storm surge, km
$L'_{v=0}$	-	maximum value of $L_{v=0}$, km
L_s	-	length of penetration of brackish or saltwater into the river; length of saltwater wedge, km
L'_s	-	maximum value of L'_s, km
LW	-	low tide water, m
LWS	-	low water slack
m, n	-	numerical constants
n	-	stratification parameter
p_t	-	tidal prism, m^3
Q	-	discharge, m^3/s
Q_r	-	river discharge averaged over the tidal cycle, m^3/s
Re	-	Reynolds number $(Re = \frac{v \cdot h}{\nu})$
Re_ρ	-	densimetric Reynolds number $(Re_\rho = \frac{v_\rho h}{\upsilon})$
Ri	-	Richardson number
Ri_E	-	Richardson estuary number
S	-	water salinity, ‰
S_s	-	seawater salinity, ‰
T	-	tidal period, s
T_{fl}	-	duration of flood phase, s
u	-	flow velocity, m/s
v	-	cross-section averaged follow velocity, m/s
v_r	-	flow velocity averaged over the tidal cycle, m/s

v_ρ	-	densimetric velocity $\left(v_\rho = \sqrt{\frac{\Delta\rho}{\rho} \cdot g.h.}\ , m/s\right)$
V_{fl}	-	volume flood flow, m^3
x, y, z	-	Cartesian coordinates
α	-	flood parameter according to Canter Cremers and H.B. Simmons
ν	-	coefficient of kinematic viscosity, m^2/s
ρ	-	fluid density, kg/m^3
ρ_r	-	density of the fresh river water, kg/m^3
ρ_s	-	density of the saline seawater, kg/m^3
Δ_ρ	-	difference between density of sea and fresh river water ($\Delta_\rho = \rho_s - \rho_r$)
ΔH_s	-	tidal or storm water level variations, m

3. Mechanisms of seawater intrusion

3.1 Basic mechanisms governing seawater intrusion

3.1.1 Seawater being heavier than river water

The salt content of seawater is higher than that of fresh river water. This makes the density of seawater, i.e. its mass per unit volume, a few percent higher than the density of river water. Consequently, seawater is a few percent heavier than river water. Because of this the difference in density between seawater and river water has a large effect on the flow in an estuary, where the seawater is measurably diluted by the freshwater from land drainage (Cameron and Pritchard, 1963).

The density of seawater and river water depends on both the salinity and the temperature (see Appendix 1). In most estuaries the effect of salinity is predominant. Many tropical estuaries, however, have a small river flow entering them during the hot season. Surface heating can then have a significant effect on the density difference between the estuary and the sea.

Density differences have a major effect on estuarine flow since they tend to cause stratification and because of their effect on pressure. Both these mechanisms will be elaborated upon in the following sections.

3.1.2 Stratification of estuaries

Density difference tends to cause stratification, meaning that near the bottom of the body of water under consideration one finds heavier water with a greater density than near the water surface. Energy is needed to overcome the stratification by mixing the heavier bottom water with the lighter surface water. This mixing is effectuated by turbulence, with the required energy being supplied by the tidal flow. Stratification is therefore most pronounced in estuaries through which a river issues into a non-tidal sea, whilst it is weaker when tidal action is strong.

On this basis, Pritchard (1955) and Cameron and Pritchard (1963) have classified estuaries according to their stratification and their salinity distributions. They define the following types of estuaries: highly stratified salt-wedge type estuaries, partly mixed estuaries and well mixed estuaries (see Section 2.3.3). In the salt-wedge type freshwater flows over virtually non-diluted seawater towards the sea (Fig. 3.1). In well mixed estuaries the density varies primarily in the horizontal direction and hardly over the depth of the estuary (Fig. 3.3). A partly mixed estuary is in the intermediate position (Fig. 3.2).

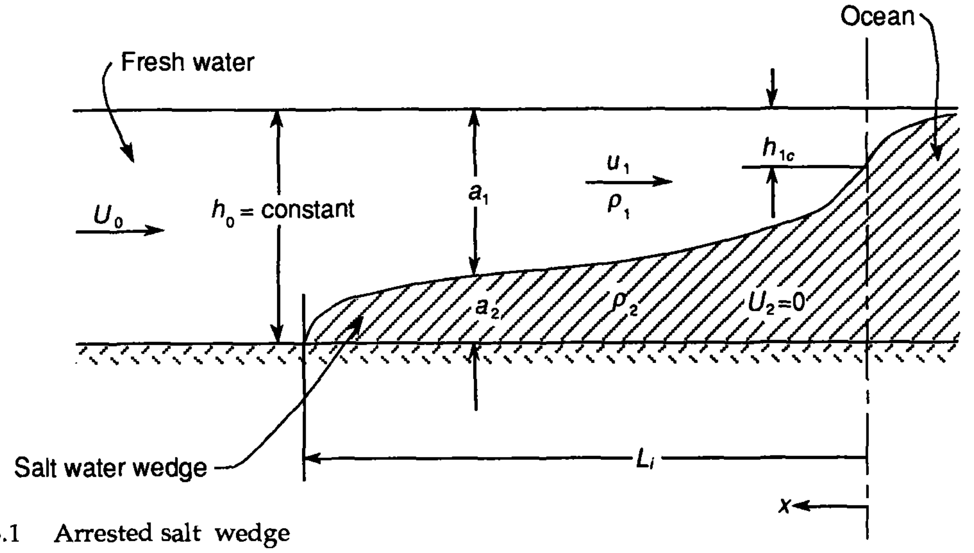


Fig. 3.1 Arrested salt wedge

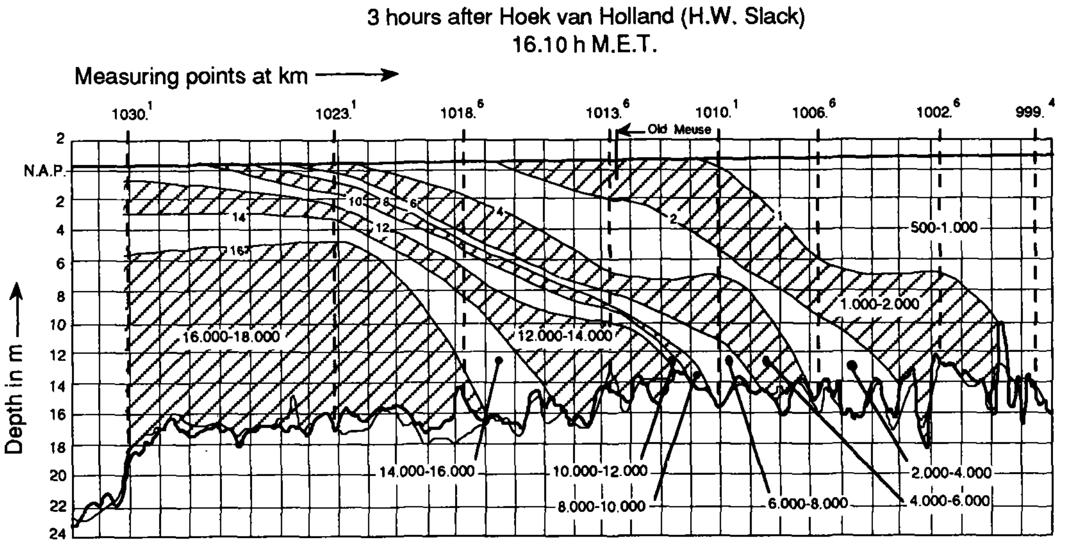


Fig. 3.2 Rotterdam Waterway (partly mixed estuary) 7th April 1971, salinity data (Rijkswaterstaat, 1971); numbers refer to salinity in g/l

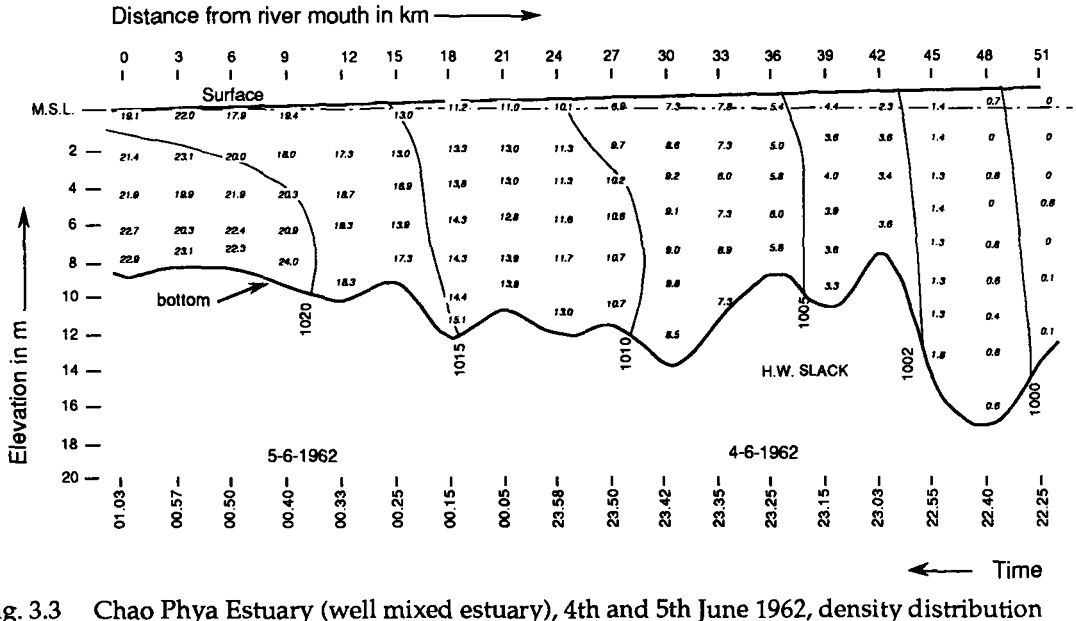


Fig. 3.3 Chao Phya Estuary (well mixed estuary), 4th and 5th June 1962, density distribution (Nedeco, 1965); numbers refer to density in kg/m^3

3.1.3 Effect of density differences on pressure; gravitational circulation

Density differences have a major on the estuarine flow because of their effect on pressure. In an estuary the pressure is hydrostatic, meaning that at a given level the pressure is equal to the weight of the water column above it per unit horizontal area. In formula

$$p = \bar{\rho}\ g\,(h + h_b - z) \qquad (3.1)$$

where p : hydrostatic pressure
g : gravitational acceleration
$(h + h_b - z)$: length of considered water column (see Fig. 3.4)
z : vertical coordinate (see Fig. 3.4)
h : water depth
$\bar{\rho}$: average density of water column with length $(h + h_b - z)$ (see Fig. 3.4)
h_b : vertical coordinate of bottom

In accordance with Figs. 3.1–3.3 and because of tidal action

$$\rho = \bar{\rho} = f\,(x,t) \text{ for well mixed estuary} \qquad (3.2)$$

$$\bar{\rho} = f\,(x,z,t) \text{ for partially mixed estuary} \qquad (3.2a)$$

and

$$h = f(x,t) \qquad (3.3)$$

where x: longitudinal coordinate, measured from mouth of estuary, positive when directed landward
t: time
ρ: density of given mass of water
$\bar{\rho}$: density averaged over depth h (see Fig. 3.4)

If the pressure at level z on the seaward side of a given mass of water is larger than on the landward side, the difference in pressure will cause the mass of water to be subjected to a net landward force and therefore to a landward acceleration. In formula, the net landward force is given by

$$F_p = -\frac{1}{\rho}\frac{\delta p}{\delta x} \qquad (3.4)$$

where F_p: net landward force per unit mass of water caused by a variation of p with x.

If the pressure at the seaward side of a given mass of water is larger than that at the landward side and when x is positive in the landward direction, $\delta p/\delta x$ is negative. This explains the minus sign in (3.4), as both F_p and x are positive in the landward direction.

Differences between the pressure on either side of the considered mass of water may have two causes. Either there is a slope of the water surface or a difference in density. The essential difference between the two causes is that in the former case the difference in pressure is the same over the whole depth, whereas in the latter case the difference in pressure increases as the distance from the water surface (h-z) increases. This difference is illustrated in Fig. 3.5, and it can be derived from (3.4) by differentiating the pressure p in the horizontal direction, keeping z constant.

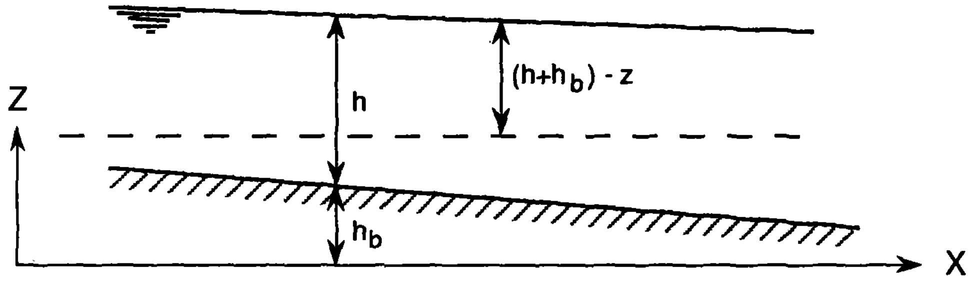


Fig. 3.4 Definition sketch for notation

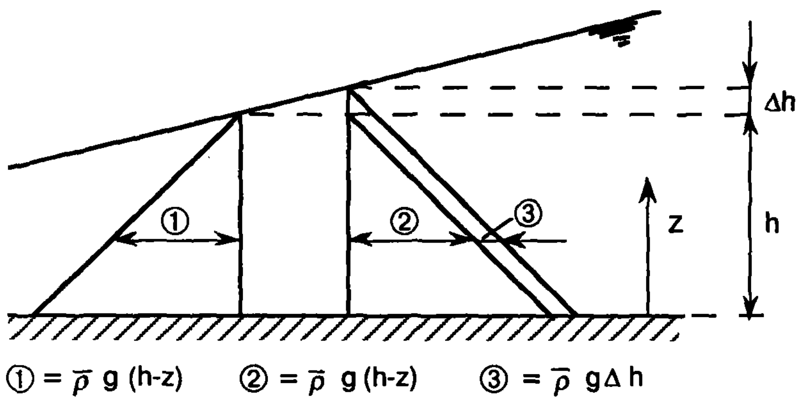


Fig. 3.5a Pressure gradient caused by surface slope (horizontal bottom)

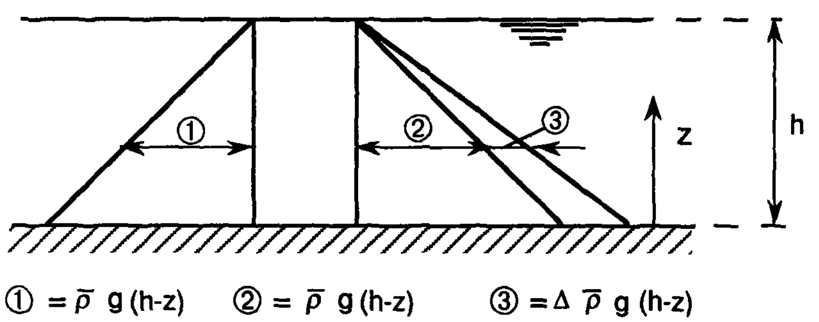


Fig. 3.5b Pressure gradient caused by density gradient (horizontal bottom)

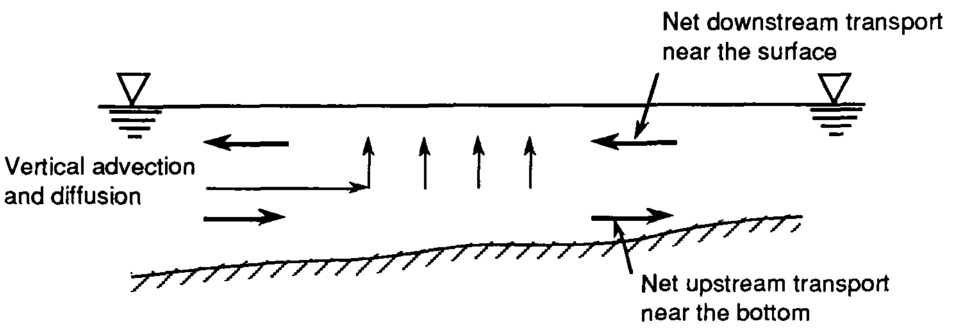
For a well mixed estuary (Fig. 3.3), satisfying (3.2) because of (3.3) and (3.4) this procedure gives at any point in time

$$F_p = -g\frac{\delta(h+h_b)}{\delta x} - \frac{1}{\rho}(h-y)g\frac{\delta\bar{\rho}}{\delta x}$$

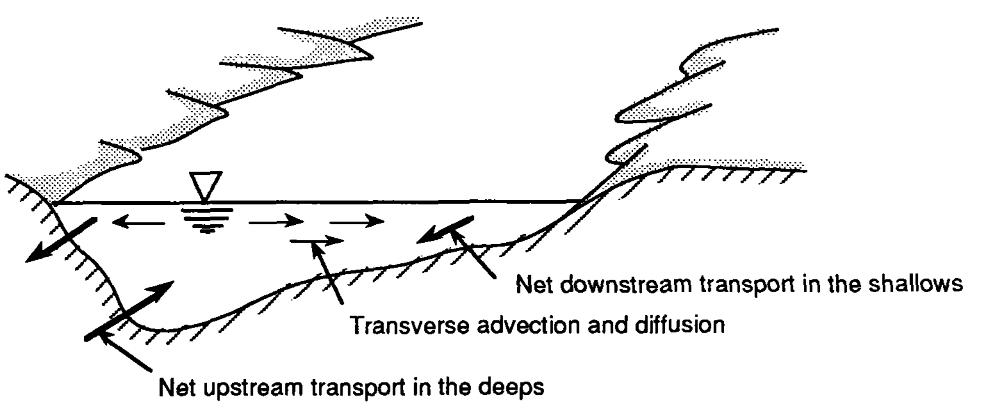
$$= -g\frac{\delta(h+h_b)}{\delta x} - \frac{1}{\rho}\cdot\frac{1}{2}hg\frac{\delta\bar{\rho}}{\delta x} - \frac{1}{\rho}(1/2h-y)g\frac{\delta\rho}{\delta x} \qquad (3.5)$$

(a) (b) (c)

Term (a) represents the effect of the slope of the water surface on the net force acting on a unit mass of fluid. It does not vary over the depth. This explains why the tide, which causes the surface slope to change, accelerates the water equally over the depth. Term (b) does not vary over the depth either. It represents the relative small effect of the density differences on the depth-mean tidal flow.



a) The vertical circulation envisaged by Pritchard in the James

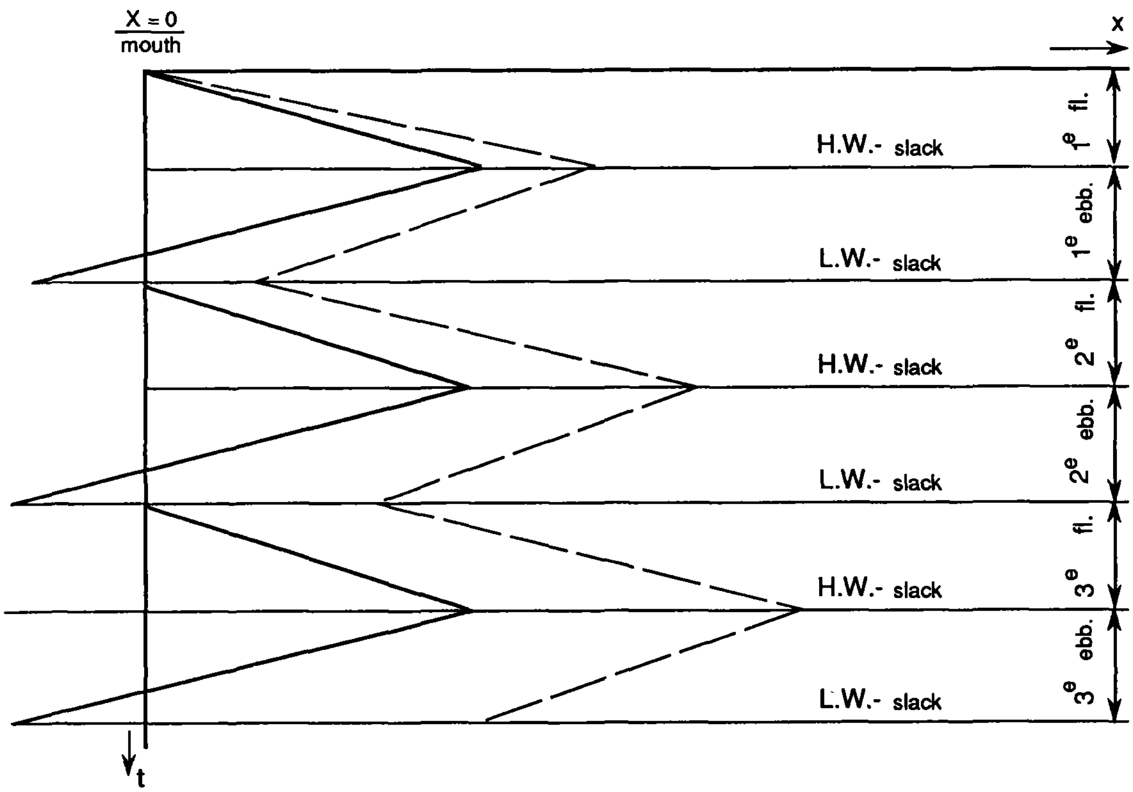


b) A three-dimensional circulation in a nonrectangular channel.
(after Fischer, 1976)

Fig. 3.6 Two views of a salt balance maintained by gravitational circulation

Averaged over the depth, term (c) is equal to zero, meaning that it has no effect on the depth-mean tidal flow. However, it varies linearly over the depth and therefore causes the velocity of flow to vary over the depth. As the density decreases with increasing distance from the sea, $\delta\bar{\rho}/\delta x$ is a negative quantity throughout the tidal cycle. Therefore, due to the influence of the density differences on the hydrostatic pressure, from the bottom to mid-depth ($0 < y < 1/2\ h$) throughout the tidal cycle F_p is positive and the water is subjected to a landward force. From mid-depth to the water surface ($1/2\ h < y < h$) throughout the tidal cycle F_p is negative and the water is subjected to a seaward force. This explains the gravitational circulation as the driving mechanism of salt intrusion into rivers from the sea.

Two views on a salt balance maintained by gravitational circulation are presented by Fischer (1976): a vertical circulation in an estuary of rectangular cross-section (Fig. 3.6a) and a three-dimensional circulation in an irregularly sloped channel (Fig. 3.6b). Equation 3.5 implies that the strength of the gravitational circulation is proportional to $|\delta\bar{\rho}/\delta x|$. Therefore, the strength of the gravitational circulation increases with increasing distance from the sea until $|\delta\rho/\delta x|$ reaches its maximum value at about half the salt intrusion length. Further landinward $|\delta\rho/\delta x|$ decreases to become zero in the river water zone of the estuary. Because of continuity considerations, this



1. x-t diagram of *bottom water particle* coming in from *fresh water sea* ($\Delta_\rho = 0$) at beginning of flood (river discharge has caused particle to go out to sea again before end of ebb).
2. x-t diagram of *bottom water particle* coming in from *salt water sea* ($\Delta_\rho \neq 0$) at beginning of flood.

Fig. 3.7 Effect of velocity distribution and mixing on salt intrusion into rivers

variation of the strength of the gravitational circulation in the horizontal direction is associated with a relatively small vertical flow. Near the mouth of the estuary this is a downward flow. Near the tip of the zone of salt intrusion it is an upward flow, as shown in Fig. 3.6a.
Summarizing, the following phenomena are important in salt intrusion:

- tidal movement;
- the effect of differences in density on the hydrostatic pressure leading to gravitational circulation;
- the vertical flow caused by the variation of the strength of the gravitational circulation with increasing distance from the sea; and
- turbulent mixing induced by the tidal flow.

Because of the differences in density, the landinward flow is stronger near the bottom than if there were no differences in density. Near the water surface, the position is reversed. This causes saltwater to intrude over the bottom further up the river than if there were no differences in density (Fig. 3.7). Factors which limit the distance the saltwater can intrude inland are the vertical turbulent mixing and the vertical flow, which near the tip of the zone of salt intrusion is directed upward. By these mechanisms salt from the bottom layers with a predominantly landward flow is brought into the upper layers with a predominantly seaward flow.

3.1.4 Rotation of the earth

Because of the rotation of the earth, with respect to well mixed estuaries distinction must be made between laterally inhomogeneous and laterally homogeneous estuaries (Dyer, 1973).

Laterally inhomogeneous estuaries
When the estuary is sufficiently wide the Coriolis force will cause a horizontal separation of the flow. The seaward net flow will occur at all depths on the right-hand side in the northern hemisphere and the compensating landward flow on the left. Thus the circulation would be in a horizontal plane rather than in the vertical sense as found in the other estuarine types. The increase of salinity towards the mouth will be regular on both sides of the estuary.

Laterally homogeneous estuaries
When the width is smaller, lateral shear may be sufficiently intense to create laterally homogeneous conditions. Salinity increases evenly towards the mouth and the mean flow is seawards throughout the cross-sections.

3.1.5 Large-scale advective mixing

The interaction between the tidal flow and the large-scale geometry of the estuary induces large-scale advective mixing. Fig. 3.8 is a typical example, and shows temporary storage of a constituent in a side arm (Pritchard, 1955 and Fischer *et al.*, 1979, Section 7.2.2). In the main channel the velocities are larger than in the side arm. Consequently the momentum of the flow in the main channel is larger. This causes a phase difference between the tidal flows in the main channel and the side arm. In the latter the current direction changes at high water, in the main channel some time after high water. This phase difference acts as a 'chopping mechanism', separating fluid particles which at a given moment in time were neighbours. Temporary storage in shallow basins or above tidal flats has a similar effect, as long as there is a phase difference between the main flow and the flow to and from the areas of storage.

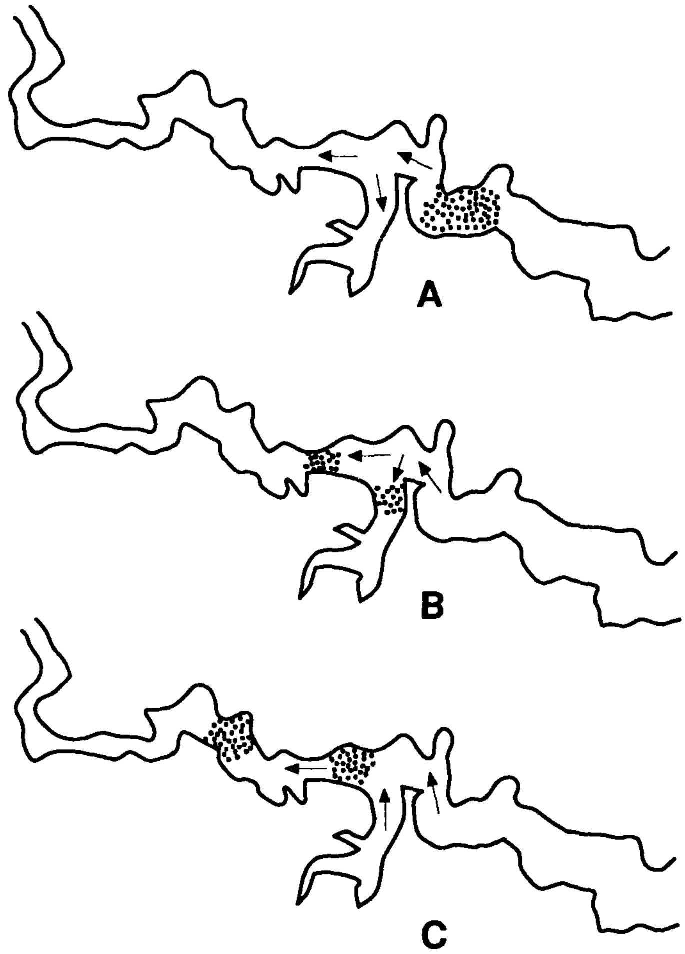


Fig. 3.8 The phase effect in a branching channel (A).
A cloud of tracer being carried upstream on a flooding tide (B).
At high water some of the particles are trapped in the branch (C).

During the early stages of the receding tide the flow in the main channel is still upstream. The particles trapped in the branch reenter the main channel, but are separated from their previous neighbours (after Fischer).

An example related to the above temporary storage mechanism can be found at the junction of two tidal rivers. In this case chopping occurs if there is a phase difference between both rivers, i.e. when the ebb flow in the one tidal river begins earlier than in the other. The mechanism is discussed in further detail in Chapter 8. (Case study: Rotterdam Waterway).

Chopping may also take place when a flood channel (or ebb channel) divides into two channels with different times of current reversal (slack). This is illustrated by Fig. 3.9.

The large scale transports discussed thus far are due to tidal currents. In addition, one has to distinguish those due to wind and residual currents (Fischer *et al.*, 1979, Sections 7.2.1 and 7.2.2.2). In wide estuaries residual currents may be caused by the Coriolis force (see Section 3.1.4) and by the interaction of the tidal flow with the irregular bathymetry.

While the large scale features of the concentration distribution are established by the aforementioned mechanisms, the concentration distribution is further affected by small scale transports caused by turbulence generated by the tidal flow at the bottom, the banks and in the zone of contact between waters with different velocities.

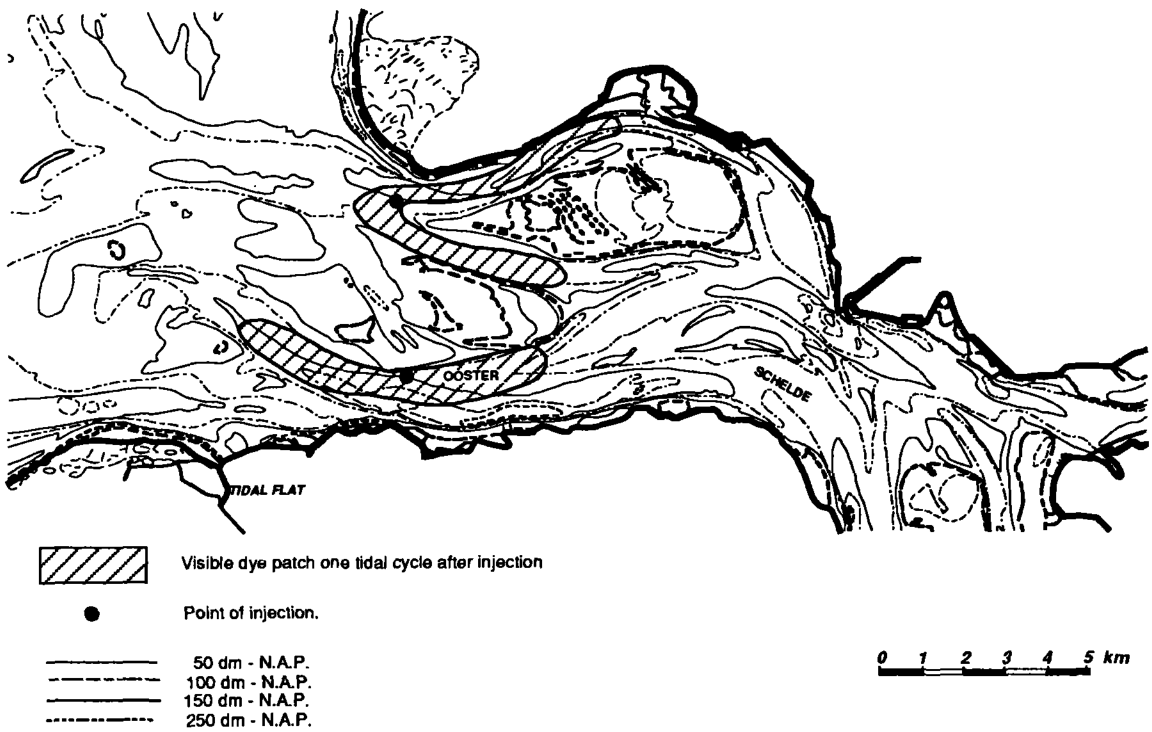


Fig. 3.9 Concentration distribution primarily controlled by large-scale advection induced by combined influence of tidal flow and bathymetry; dye patch separated into two parts

3.1.6 Independent variables

Table 3.1 lists the main independent variables which govern the seawater intrusion and their effect (Rigter, 1973).

Table 3.1: Effect of main independent variables

Independent variable	Effect of increasing independent variable on	
	Stratification	Seawater intrusion
water depth	+	+
river discharge	+	–
driving density difference	+	+
bed roughness	–	–
tidal amplitude (a)	–	– when a is small, + when a is large

+ : stratification or seawater intrusion increases with increasing values of independent variable

– : stratification or seawater intrusion decreases with increasing values of independent variable

Turbulent energy is needed to cause vertical mixing. The amount of energy needed increases with increasing water depth, increasing river discharge and increasing driving density differences, i.e. the density difference between seawater and freshwater. The amount of energy available for mixing increases with increasing bed roughness and increasing tidal amplitude. These observations explain the effect of these independent variables on the stratification as indicated in Table 3.1. The freshwater velocity, i.e. the river discharge divided by the cross section of the estuary, decreases with increasing water depth, while it increases with increasing river discharge.

Therefore, the seawater intrusion increases with increasing water depth, while it decreases with increasing river discharge.

The strength of the gravitational circulation and thus the seawater intrusion increases with increasing driving density difference (see term (c) of 3.5). Vertical mixing is a factor which limits seawater intrusion (see Section 3.1.3). Because of this seawater intrusion decreases with increasing bed roughness.

The effect of the tidal amplitude is determined by two counteracting effects. Large tidal amplitudes at the mouth of the estuary are associated with strong tidal currents, and hence with strong turbulent mixing and weak stratification. The stronger the turbulent mixing, the smaller salt intrusion tends to be. Because of this effect salt intrusion tends to become smaller with increasing tidal amplitude, particularly when the estuary is stratified.

Large tidal amplitudes at the mouth of the estuary are further associated with large tidal excursion paths. The larger the tidal excursion path, the larger salt intrusion tends to be at high water slack. Thus salt intrusion tends to become larger with increasing tidal amplitude, especially when the estuary is mixed.

Which of the above counteracting mechanisms prevails varies with the stratification, i.e. with the strength of the tide. For the schematized estuary studied by Rigter (1973), this is clearly demonstrated by the fact that salt intrusion at high water slack decreases with increasing tidal amplitude when the latter is relatively small, while salt intrusion at high water slack is found to have its smallest magnitude (Rigter, 1973, Fig. 2).

3.1.7 Transient conditions versus equilibrium conditions

The factors governing the seawater intrusion vary with time. Therefore in analysing field data it is important to realize that these data may have been collected under equilibrium or transient conditions.

Equilibrium conditions occur when the factors governing seawater intrusion have been constant over a sufficiently long period. By then seawater intrusion during the one tide is the same as that during the following tide. Transient conditions arise when the factors governing intrusion change with time or have been changing with time in a sufficiently short preceding period. Then the seawater intrusion must follow these changes — either immediately or with a given reaction time.

The duration of the reaction time depends on the relative importance and on the nature of the factor which changes. Advective salt transport reacts almost instantaneously to variations in tidal conditions and mean sea level. Turbulent salt transport needs a longer period of time to adapt to the new tidal conditions and to the new mean sea level. Seawater intrusion reacts slowly to variations in the river discharge.

To illustrate (the effect of) transient conditions some data on the Chao Phya estuary system in Thailand are given below.

3.1.8 Example

Fig. 3.10 shows the type of information that can be obtained when substantial field data are available. This figure summarizes the results of field measurements made in the Chao Phya estuaries over a period of two years (Nedeco, 1965). The available data have been analysed to find a relationship between the salinity distribution in the estuary and the influences of the river

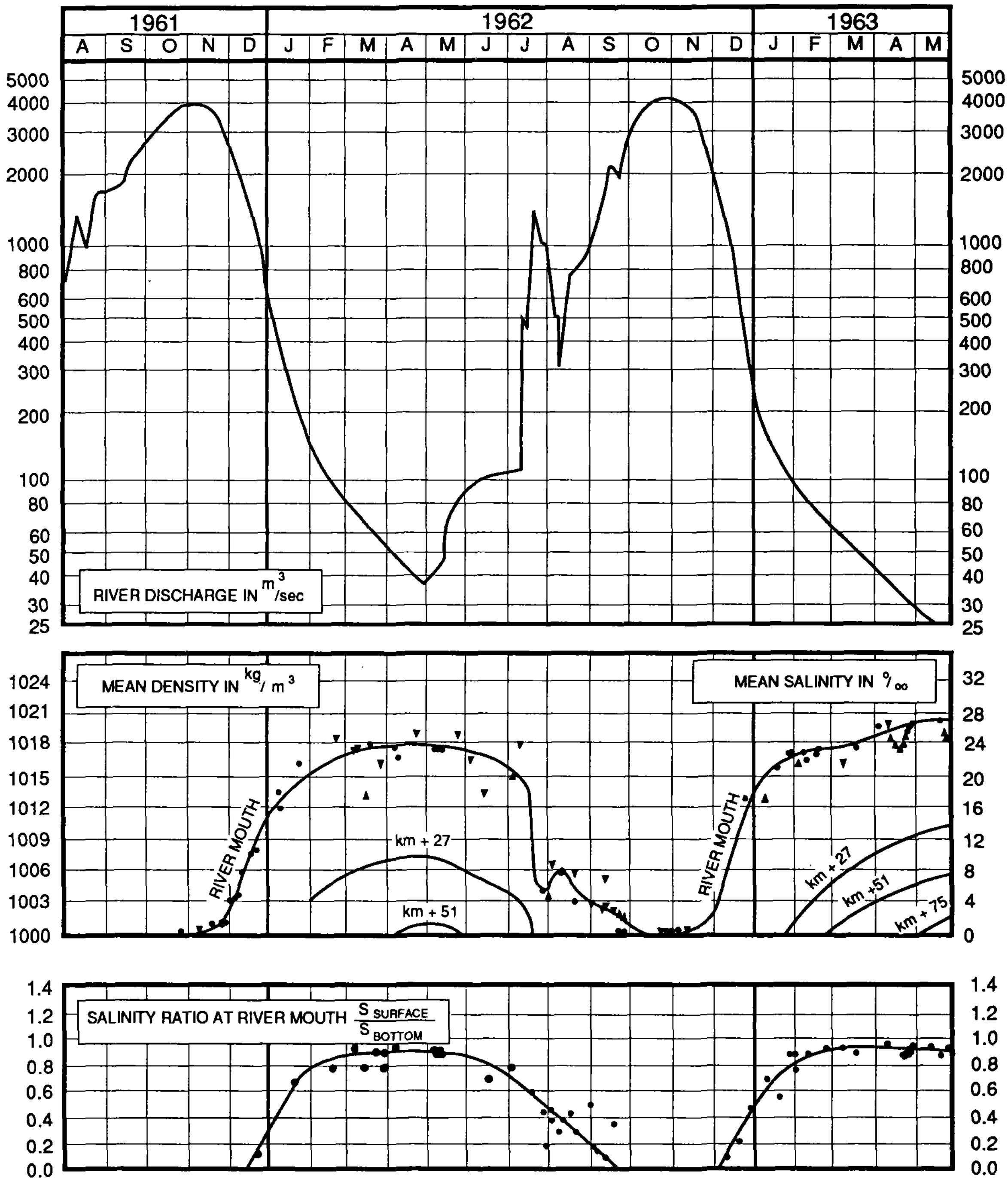


Fig. 3.10 Salinities at four points in the estuary and the salinity ratio at the river mouth, compared with the discharge of the Chao Phya

discharge and tides. In this way it was possible to determine the effect of changes in the conditions by an analysis which was divided into two phases:

- the determination of a correlation between the mean salinity (averaged over the cross-section and over the tidal cycle), the discharge of the river and the location in the estuary, and
- a description of the variations of the salinity around this mean value as a consequence of tidal currents.

Fig. 3.11 shows the relationship between the salinity at the mouth of the estuary, $\bar{S}_m$, and river discharge. Some observations at a discharge of 900 m^3/s show a relatively large deviation from the relationship drawn in the figure. These data were obtained in the short period of relatively small discharges of the river around 10 August, 1962 (Fig. 3.10). The response of the salinity to these changes of the discharge is so slow that the rapid variation of the discharge of the river could not be followed. In Fig. 3.11 S_0 represents the salinity of seawater and Q the discharge of the river.

Fig. 3.12 shows the relationship between the mean salinity, $\bar{S}$, at a distance x from the mouth of the estuary and the mean density at the mouth. The parameter G(x), plotted in the figure is defined by the equation

$$\ln(\bar{S}/\bar{S}_m) = -Q\,G(x),$$

where $\bar{S}$: mean density (averaged over cross-section and tidal cycle) at distance x from mouth of estuary

$\bar{S}_m$: mean density at mouth of estuary

Figs. 3.11 and 3.12 were obtained by trial and error. The relationships represented in these figures do not hold for other estuaries.

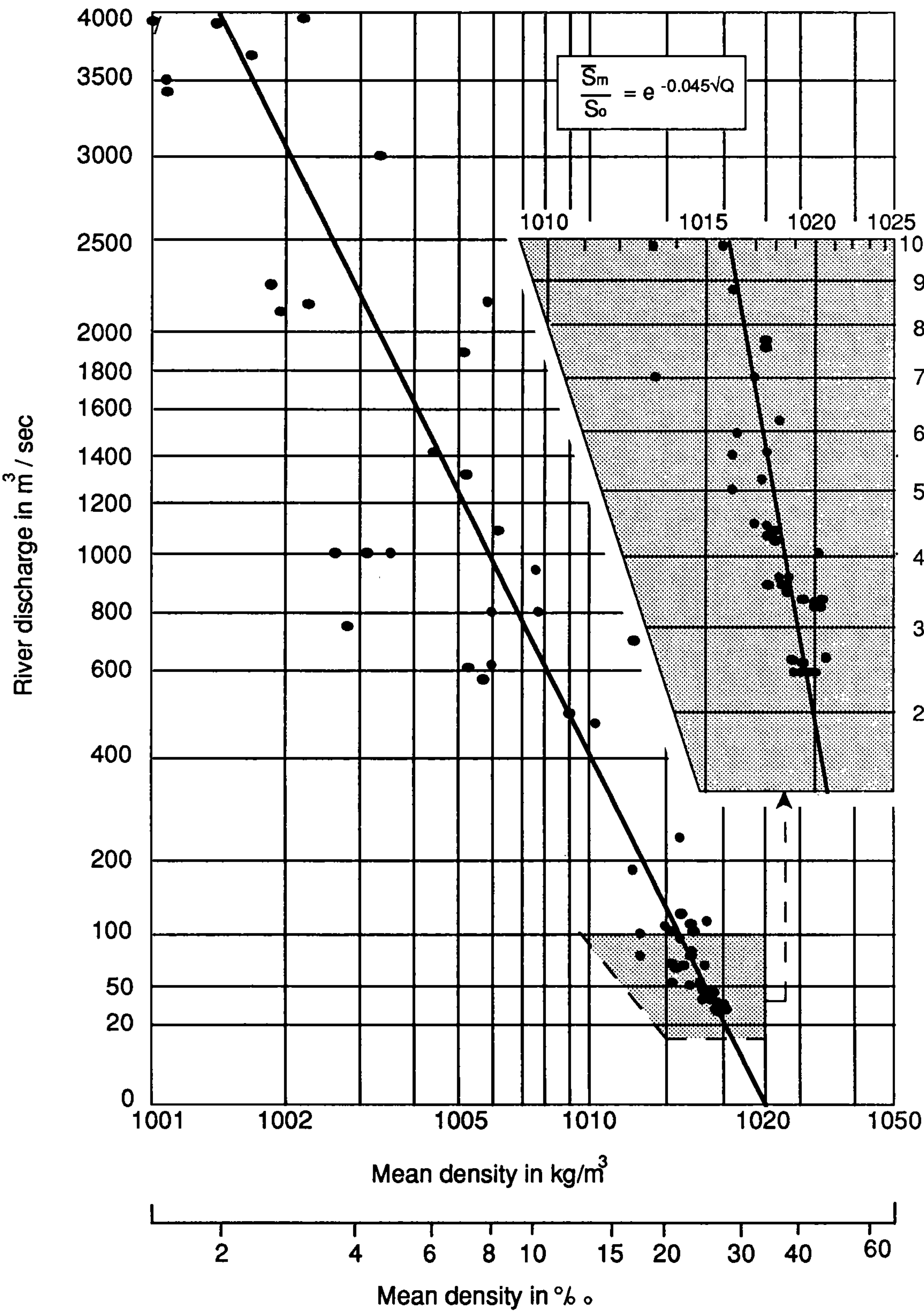


Fig. 3.11 Mean density (salinity) at the river mouth vs. discharge of the Chao Phya

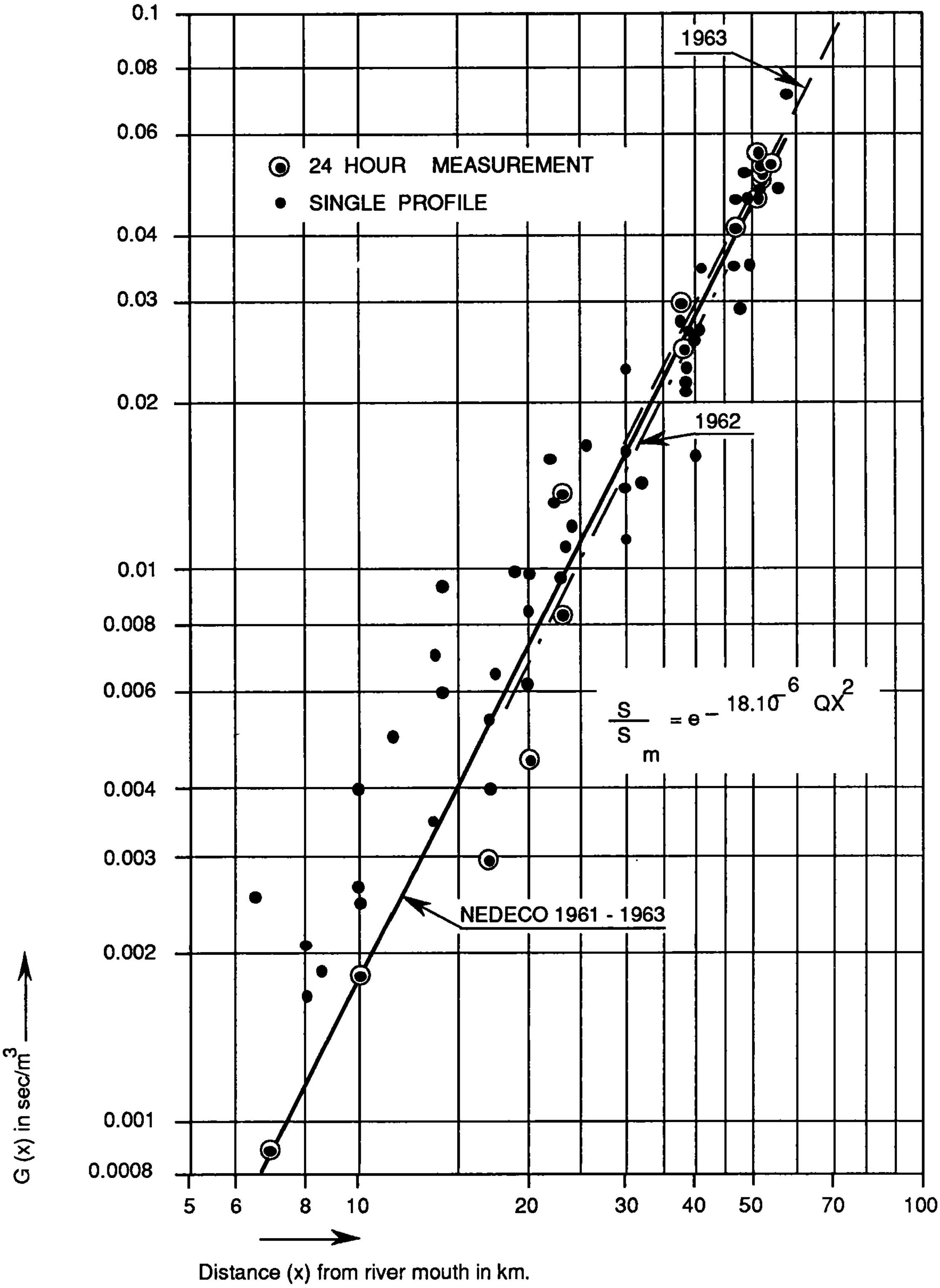


Fig. 3.12 Plots of G (X) vs. X

3.2 Hydrodynamic formulation of types of seawater intrusion

3.2.1 Types of seawater intrusion into rivers

According to the three types of mixing of river and seawater in river mouths (Section 2.3.3) three types of seawater intrusion into rivers can be distinguished: (1) a well mixed situation; (2) a partially mixed situation; and (3) saltwater wedge. The fluvial and dynamic conditions for these types are given in Table 2.1.

3.2.2 Well and partially mixed situations

The hydrodynamic formulation of well (complete) and partially mixed situation are similar and can be carried out on the basis of two concepts: that of advection and dispersion (advection is considered as the averages over the cross-section currents) and secondly tidal prism. The concept of advection and dispersion allows one to consider the longitudinal distribution of water salinity along the river part of the mouth area and to estimate the length of tidal averaged intrusion of brackish water with a given river discharge. The concept of tidal prism permits an estimation of only the maximum (limit) of intrusion of salt or brackish water into the river during the flood tide.

The concept of advection and dispersion (by D.R.F. Harleman)

A one-dimensional equation of advection and dispersion of salt in the river part of the mouth can be written as

$$\frac{\delta S}{\delta t} + v\frac{\delta S}{\delta x} = \frac{\delta}{\delta x}\left(D_x \frac{\delta S}{\delta x}\right); \tag{3.6}$$

where S is water salinity, v is flow velocity, D_x is longitudinal dispersion coefficient. The axis x is from the sea in upstream direction of the river. Assume, that S, v and D_x are averaged over the cross-section and are functions of x and t. The values of the one-dimensional dispersion coefficient D_x lies approximately between the limits of 10 to 5000 m^2/s.

The flow velocity v can be expressed as $v_t - v_r$, where v_t is the tidal velocity variable in value and direction which is measured in the cross-section at any moment of time t, and v_r equals Q_r/A, where A is averaged in time cross-section area.

After averaging of equation (3.6) over the tidal cycle (assuming that averaged values of S and D_x are independent on time) one can write

$$-v_r\frac{d\bar{S}}{dx} = \frac{d}{dx}\left[\bar{D}x\ \frac{d\bar{S}}{dx}\right]. \tag{3.7}$$

Integration of (3.7) in x yields:

$$-v_r\bar{S} = \bar{D}_x\frac{d\bar{S}}{dx} + C_1 \tag{3.8}$$

where the constant of integration C_1 is equal to 0, because

$$\text{for } x \to \infty\ \bar{S} = 0 \text{ and } \frac{dS}{dx} = 0. \tag{3.8}$$

Substitution of v_r by Q_r/A, where Q_r is averaged over the tidal cycle river and A is averaged cross-section area, leads to

$$-Q_r\,\bar{S} = A\bar{D}_x\frac{d\bar{S}}{dx}. \tag{3.9}$$

Equation (3.9) represents the equilibrium between advection and dispersion of salt, in other words it represents the equality between the downstream transport of salt with river water on the one hand, and the upstream transport of salt under the effect of dispersion due to the salinity gradients on the other. Equation (3.9) can be expressed as

$$\frac{d\bar{S}}{\bar{S}} = -\frac{Q_r}{A\bar{D}_x}\,dx. \tag{3.10}$$

The integral of equation (3.10) will depend on the change of $\bar{D}_x$ along the axis x.

There are several versions for the expression of the function

$$\bar{D}_x = f\,(x).$$

1. $\bar{D}_x = \bar{D}_0$ = const. This means that the dispersion coefficient is constant for the whole length of saltwater intrusion and is equal to the dispersion coefficient for the coastal edge of the river channel at $x = 0$. In this case the integration of equation (3.10) with boundary conditions $\bar{S} = \bar{S}_0$ for $x = 0$ and $\bar{S} = 0$ for $x \to \infty$ leads to

$$\bar{S} = \bar{S}_0\exp\left(-\frac{Q_r x}{AD_0}\right). \tag{3.11}$$

Equation (3.11) characterizes the decrease in water salinity along the river channel in accordance with the exponential law. Salinity decreases upstream faster if the value of Q_r (Fig. 3.13a) is greater and the values of A and D_0 are smaller.

The hypothesis of $\bar{D}_x$ to be constant along the axis x is far from being reliable, because it is known that $\bar{D}_x$ decreases along the axis x because the tidal variation of water level decreases. One may therefore suppose that $\bar{D}_x \propto \frac{\bar{D}_0}{x}$, but this expression gives $\bar{D}_x = \infty$ for $x = 0$.

2. In order to avoid this, Ippen and Harleman (1961) offer another expression for $\bar{D}_x$:

$\bar{D}_x = \frac{\bar{D}_0 \cdot a}{x + a}$, where $\bar{D}_0$ is the dispersion coefficient at the coastal edge of the river channel ($x = 0$), a is a constant introduced in order that $\bar{D}_x$ would have a finite value for $x = 0$.

The integration of equation (3.10) with the boundary conditions $\bar{S} = \bar{S}_s$ when $x = -a$ and $\bar{S} = 0$ when $x \to \infty$ leads to

$$\bar{S} = \bar{S}_s\exp\left[-\frac{Q_r}{2aAD_0}(x + a)^2\right]. \tag{3.12}$$

Equation (3.12) corresponds to the transfer of the origin to point $x = -a$ in the coastal part of the mouth, where $\bar{S} = \bar{S}_s$.

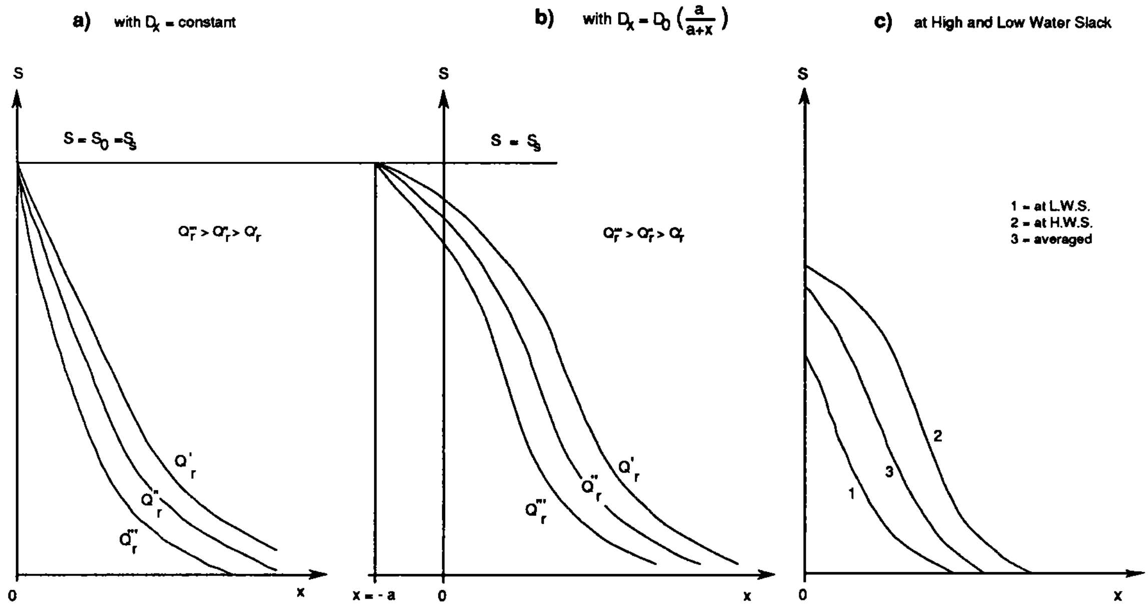


Fig. 3.13 Calculated longitudinal salinity profiles, tidal averaged and in the case of well mixing with different assumptions for dispersion coefficient $\bar{D}_x$.

The change of water salinity along the river channel in this case is represented in Fig. 3.13b.

3. $\bar{D}_x = \bar{D}_0 - K\frac{Q_r}{A}x.$

This expression was proposed by Van der Burgh. Here K is the Van der Burgh's constant, that is often taken to be 0.9.

The integration of equation (3.10) with boundary conditions

$$\bar{S} = \bar{S}_0 \text{ for } x = 0 \text{ and } \bar{S} = 0 \text{ for } x = \frac{A\bar{D}_0}{KQ_r}$$

(for $x > \frac{A\bar{D}_0}{KQ_r}$ $\bar{D}x$ becomes negative)

yields: $$\bar{S} = \bar{S}_0\left(1 - \frac{KQ_r}{A\bar{D}_0}x\right)^{1/K}. \tag{3.13}$$

Equations (3.11), (3.12 and (3.13) also permit one to evaluate the length of intrusion of brackish water into the river. Let us consider that the length of the penetration of brackish water into the river corresponds to the distance from the coastline edge of the river channel where the salinity S is equal to S_r (S_r is salinity of fresh river water that is usually not more 1‰). In this case after logarithmation of equations (3.11) and (3.12) and solving them in relation to the distance $x = L_s$, where $\bar{S} = S_r$, we obtain respectively:

$$L_s = A\bar{D}_0 \ln(\bar{S}_0/S_r), \tag{3.14}$$

$$L_s = -a + \sqrt{\frac{2a\,A\bar{D}_0 \ln(\bar{S}_s/\bar{S}_r)}{Q_r}}. \tag{3.15}$$

From equation (3.13) with $\bar{S} = S_r$ and $x = L_s$ can be obtained:

$$L_s = \left[1 - \left(\frac{S_r}{S_s}\right)^K\right] \frac{A\bar{D}_0}{KQ_r} . \qquad (3.16)$$

Equations (3.14), (3.15) and (3.16) provide qualitatively the same results, namely the length of the intrusion of brackish water into the river L_s is in direct proportional relationship to $\bar{S}_0$ and $\bar{D}_0$ and is an adverse relationship to Q_r. The greater $\bar{S}_0$ and $\bar{D}_0$ and the smaller Q_r (or $v_r = Q_r / A$) the greater is L_s. The above mentioned equations can only have an engineering significance with a reliable estimation of the longitudinal dispersion coefficient D_x (see Section 3.2.4).

Concept of tidal prism (after V.N. Mikhailov)

With some assumptions (Fig. 9.1, Appendix 4) Mikhailov (1971) produced the following equation for the calculation of the length of seawater intrusion into river at HWS for the cases of short river part of the mouth:

$$L_s = L_{\Delta H}\left(1 + \frac{h_0}{\Delta H_0}\right) - \sqrt{\frac{L^2_{\Delta H} \cdot h_0}{\Delta H_0}\left(2 + \frac{h_0}{\Delta H_0}\right) + \frac{2\,Q_r L_{\Delta H} T_{fl}}{B \Delta H}} , \qquad (3.17)$$

where Q_r is river discharge averaged over the tidal period, ΔH_0 is the tidal range at the coastal edge of the river channel, $L_{\Delta H}$ is maximum distance of penetration of tidal water level variations at given Q_r, T_{fl} is the duration of flood tide phase, h_0 is channel depth at the ebb tide, B is averaged channel width.

Equation (3.17) illustrates the following fact: the greater ΔH_0 and $L_{\Delta H}$ and the smaller Q_r the greater is L_s.

3.2.3 Arrested saltwedge

The schemes considered in Section 3.2.2 assumed that the values of water salinity in different points of the flow in a given cross-section have minor deviations from their average values in this cross-section. These conditions are not met when a saltwater wedge is present. This type of mixing is typical of river mouths with a major influence of the river flow and a minor influence of the tides. In such a case two layers are formed in the river part of the mouth: the upper of almost freshwater and the lower of saltwater. In the upper layer freshwater moves towards the sea. The lower layer is usually relative stable and represent a stationary saltwater wedge. Between the layers of fresh and saltwater there is a well defined thin layer with a salinity jump (Fig. 2.9c), the interface of fresh and saltwater. Because of the entrainment of saltwater from the surface of the saltwedge into the freshwater flow a weak compensative upstream current always takes place in the lower layer. At the same time the saltwater wedge and its tip may be motionless or may move slowly upstream or downstream in response to river discharge variation and or the phase of the tide. The movement of saltwater wedge in upstream direction is accompanied by an increase of its thickness and a downstream movement — by a decrease of its thickness. According to Keulegan the shape of the saltwedge can approximately be described as follows (Table 3.2):

Table 3.2: Longitudinal profile of saltwater wedge

x/L_S	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
h_S/h_{S0}	1.0	0.81	0.68	0.61	0.54	0.47	0.41	0.34	0.28	0.19	0

In Table 3.2, x/L_S is the relative distance from the sea edge of the river channel ($x \approx 0$) to the tip of the saltwater wedge, and h_S/h_{S0} is the ratio of the thickness of the saltwater wedge at a given point to the thickness of the saltwater layer in the sea edge of the river channel. The approximate physical interpretation of the process of saltwater wedge formation can be carried out on the basis of densimetric velocity.

The concept of densimetric velocity (by Keulegan)

A more accurate estimation of the saltwater wedge length can be obtained by the analysis based on the concept of densimetric velocity (the velocity of propagation of the internal waves at the interface of fresh and saltwater or the initial velocity of movement of the saltwater front in steady water)

$$v_\rho = \sqrt{\frac{\Delta\rho}{\rho_m} \cdot gh} \tag{3.18}$$

and densimetric Froude number

$$Fr_\rho = \frac{v_r}{v_\rho} = \frac{v_r}{\sqrt{\frac{\Delta\rho}{\rho_m} \cdot gh}}, \tag{3.19}$$

where $\Delta_\rho = \rho_s - \rho_r$, $\rho_m \approx 1/2\,(\rho_s + \rho_r)$,
h is the depth of the channel, assumed to be constant.

Note that the water density depends on salinity and temperature (Appendix 1). The value of the densimetric velocity with different density difference of sea and river water and the depths are given in Appendix 2.

According to Keulegan the thickness of the stationary saltwater wedge in the sea edge of the river channel is equal to

$$\frac{h_{s0}}{h_0} = 1 - Fr_\rho^{2/3} \tag{3.20}$$

Thus if Equation 3.20 is correct the saltwater wedge cannot penetrate into the river, when $Fr_\rho \geq 1$ or $v_r \geq v_\rho$ because $h_{s0}/h_0 \leq 0$.

The river water velocity and the corresponding river discharge are defined as 'critical' when the saltwater wedge begins to penetrate into the river are defined as 'critical'. They are equal to

$$v_{r_{cr}} = \sqrt{\frac{\Delta\rho}{\rho_m} \cdot gh} \tag{3.21}$$

$$Q_{r_{cr}} = Bh^{3/2}\sqrt{\frac{\Delta\rho}{\rho_m} \cdot gh}\,. \tag{3.22}$$

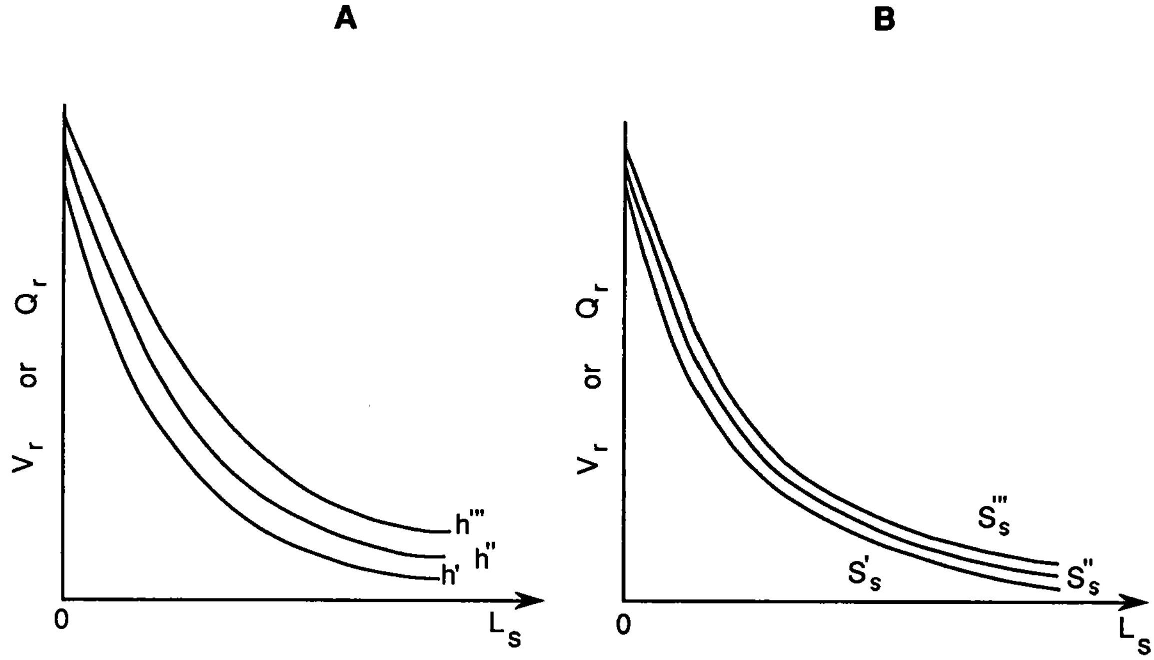


Fig. 3.14 The dependence of the seawater intrusion length on the river discharge Q_r (or flow velocity V_r); the channel depth h (Fig. A) and the seawater salinity S_s (Fig. B), where H''' > h'' > h' and $S'''_s > S''_s > S'_s$.

Measurement data show that actual values of critical densimetric velocity and critical river discharge are a little more than those obtained from Equations (3.21) and (3.22).

The length of the stationary saltwater wedge L_s according to Keulegan depends on densimetric Froude and Reynolds numbers:

$$\frac{L_s}{h} = K\,(Re_\rho)^{1/4}\,(Fr_\rho)^{-5/2} \tag{3.23}$$

where Fr_ρ is the densimetric Froude number (Equation (3.19)) and the densimetric Reynolds number is equal to

$$Re_\rho = \frac{v_\rho \cdot h}{\nu}, \tag{3.24}$$

where ν is the kinematic viscosity coefficient that depends on water temperature (Appendix 3). Comparison of results calculated using Equation (3.23) with measurement data shows that the value of the K coefficient is a little more than 1. The character of the dependence of the saltwater wedge length L_s on the depth h and seawater salinity S_s is given in Fig. 3.14.

The approach based also on the concept of densimetric velocity was used by Schijf and Schönfeld (1953). After studying the interaction of the two layers (fresh and salt) they proposed the equation

$$\frac{L_s}{h} = \frac{2}{f}\left[\frac{1}{5Fr^2_\rho} - 2 + 3\,Fr_\rho^{2/3} - \frac{6}{5}\,Fr_\rho^{4/3}\right], \tag{3.25}$$

where Fr_ρ is densimetric Froude number according to Equation (3.19), f is the average frictional coefficient at the interface, and an empirical function of the turbulence level, expressed by the

Reynolds number. Normal values range between 10^{-2} and 10^{-4}. It was assumed that shear stress at the interface is equal to

$$\tau = f\frac{\rho_r}{8}\left|v_1 - v_2\right| (v_1 - v_2), \tag{3.26}$$

where v_1 and v_2 are the water velocity in the upper freshwater and the lower salt layers respectively, and ρ_r = density of freshwater.

3.2.4 Factors influencing seawater intrusion into rivers and characteristics of this process

3.2.4.1 Well and partial mixing

As was shown in Section 3.2.2 the distance of penetration of saltwater into rivers is larger when the tidal range ΔH_0 and the channel depth h are larger and the river water discharge Q_r and duration of the flood tide phase T_{fe} are smaller.

The character of tidal averaged saltwater distribution along the river depends on:

(1) Q_r or v_r: The salinity decreases rapidly in upstream direction with large values of these factors (Figs. 3.13a, b);

(2) $\bar{D}_0$: the larger the value of the longitudinal dispersion coefficient the slower salinity decreases in upstream direction. The value of $\bar{D}_0$ in the coastal edge of the river channel can be evaluated by the equations of Taylor and Harleman type:

$$\bar{D}_0 \sim k v_{max} h, \tag{3.27}$$

where v_{max} is the maximum water velocity at flood tide, which depends on the tidal range. Van der Burgh proposed the formula

$$\bar{D}_0 = 26\,(\alpha g)^{1/2} R^{3/2}, \tag{3.28}$$

where R is hydraulic radius, α is the flood parameter (Equation 2.4). It is better, however, to evaluate the value of D_x in the river part of the mouth by direct measurement.

From Equation 3.4 one can get an average value of the longitudinal dispersion coefficient for any reach of the channel with brackish water:

$$D_x = \frac{Q_r}{A} \cdot \frac{S_m}{\Delta S/\Delta x}, \tag{3.29}$$

where $\Delta S = S_1 - S_2$, $S_m = 1/2(S_1 + S_2)$,
and S_1 is the water salinity at the lower end of the reach (nearest to sea) of the length ΔX. Thus

$$D_x = \frac{Q_r}{A} \cdot \frac{\Delta_x}{2} \frac{S_1 + S_2}{S_1 - S_2}, \tag{3.30}$$

where Q_r/A can by replaced by the river flow velocity v_r.

The character of longitudinal variation of water salinity depends on the tidal phase (Fig. 3.13c). The curve $S = f(x)$ moves upstream during at the flood tide and downstream during the ebb tide. The range of the movement of these curves is approximately equal to the value of tidal excursion $L_{tide\ exc}$.

Thus the main characteristics that are necessary to study the case of well (complete) and partial mixing of river and seawater in the river part of the mouth are the following:

(1) riverine: Q_r, v_r, A;

(2) marine: S_s, ΔH_0, T_{fl}, D_0;

3) local: averaged values of the depth (h) and patterns of channel relief.

In addition, other factors influence the patterns of water mixing in the river part of the mouth, increasing the intensity of mixing and promoting intrusion of seawater into rivers. Such factors are: the wind and waves caused by the wind; variations of water level induced by storm surges; and hydraulic structures in the channel (bridges, piers etc.), which disturb the flow and causes extra turbulence.

The range of longitudinal dispersion coefficient D_x for tidal river mouths is about 10-5000 m^2/s (McDowell and O'Connor, 1977). According to Abraham the average values of D_x for some tidal river mouths are as follows: Hudson River 500, the Potomac River 20-100, the Delaware Estuary 100-450, the Rotterdam Waterway 800, the Mersey Estuary 136-360, the Severn Estuary 50-500, and the Thames Estuary 50-340 m^2/s. Applying Equation 3.27 the following values of D_x can be obtained: for the North Dvina delta with a river discharge 1000 m^3/s, D_x=250 m^2/s, for the Senegal River with a river discharge 50 m^2/s, D_x=1000 to 2500 m^2/s.

3.2.4.2 The saltwater wedge

The main factors and processes promoting the penetration of the saltwater wedge into the river part of the mouth are the following: decreasing of the river discharge Q_r or the river flow velocity v_r; increasing of the channel depth h and the seawater density ρ_s (see Fig. 3.14).

The main characteristics that are necessary to study the case of a saltwater wedge are the following:

(1) riverine: Q_r, A, h, v_r;

(2) marine: ρ_s and S_s;

(3) local: the river channel relief at the reach of the penetration of the saltwater wedge.

With other conditions being equal the value of river discharge and river channel relief exert the most strong influence on the saltwater wedge regime. The saltwater wedge often stops at a local rise in the river bottom (e.g. shoal, crossing or bar). On the contrary, increased depth as a result of dredging increases sharply the distance of the saltwater wedge penetration.

The saltwater wedge length also increases when the water level rises as a result of the tide or storm surges and during strong winds in an upstream direction.

3.3 Dispersion coefficient

3.3.1 Background

As described earlier in the discussion on basic mechanisms governing seawater intrusion, the mixing process is influenced by many factors. Leaving aside the advective transport component, the mixing process can be mainly described by the dispersion models. There are several dispersion models established for seawater intrusion problems and for the mixing of pollutants in river mouth systems. The most popular type of dispersion model that is used in mathematical modelling of seawater intrusion is based on the concept of diffusion. In this, density gradient plays a major role in mixing and the coefficient (dispersion coefficient) is related to all the actual governing factors and the simplification process of problem solving (e.g. uni-dimensionalization, averaging methods; see Abraham, 1983).

Depending on the characteristics of each river mouth system and the method of simplification, the dispersion coefficient can be modelled in various ways. Since the study of seawater intrusion does not confine itself only to the dispersion coefficient, researchers have had a tendency to search for generally acceptable models of the dispersion coefficient which would simulate the salinity intrusion phenomena with an acceptable accuracy. These models may significantly differ from each other. Nevertheless, they are normally based on the dominant factors of the river mouth system under consideration, since only on this basis can such models be used for simulation as well as prediction.

3.3.2 Models of dispersion coefficient

The earliest efforts assumed a completely well-mixed and uniform flow river mouth system to accept the dispersion coefficient as a constant. In these cases, the constant dispersion coefficient D_x tends to describe the mixing as mainly caused by the turbulence intensity of the flow field. As such, the constant value is not universal for all the river mouth systems and even for one river mouth. In reality, few river mouth systems would satisfy all the above assumptions of mixing and flow conditions. The dispersion coefficient was found to vary significantly from one river mouth system to another. Different models were adopted to explain the large variation of the dispersion coefficient based on the most important factors.

Although the relative importance of each factor would depend on the physical characteristics of each river mouth system (Fischer *et al.*, 1979), besides turbulence intensity of the flow field mentioned earlier the most important factors of these models can be identified as some of the following:

- longitudinal salinity gradient;
- freshwater inputs
- the relative importance of tidal flow;
- irregularity of the channel configurations; and
- vertical and lateral salinity gradients.

It should be pointed out that these present guidelines do not attempt to provide a state-of-the-art review of the models of dispersion coefficient. Rather, it provides some background information on the principles of modelling and typical models for possible application. On the basis of these above-mentioned factors, one can divide the existing models into the following three groups.

(i) $D_x = D_x(x)$

The dominant factors either explicitly or implicitly expressed in this group of models would include the first, fourth and fifth factors. Typical models are

$$D_x = \frac{D_0\, a}{x + a} \qquad \text{(Ippen and Harleman, 1961)}$$

where D_0 : dispersion coefficient at $x = 0$

a : a constant

$$D_x = D_0 + K \frac{(ds)^n}{(dx)}$$ (Prandle, 1981)

where D_0 : constant dispersion coefficient
K : parameter of river mouth system
n = 1 to 2

(ii) $D_x = D_x(x, v_r)$

The most important factors would include the effects of river discharge such as

$$D_x = D_0 - K \cdot \frac{Q_r}{A} \cdot x$$

where K : van der Burgh's constant (normally taken as 0.9)
Q_r: freshwater discharge
A : cross-sectional area of the channel
x : distance from the river mouth end

(iii) $D_x = D_x(x, u, R)$

In this group, all the important factors are included in the models in the two main components: independent of tidal movement and varying with tides. A typical example is

$$D_x = K_1 \frac{d(S/S_0)}{d(x/L)} + K^2 \, nu \, R^{5/6}$$ (Thatcher and Harleman, 1972)

where K : constant longitudinal dispersion coefficient
n : Manning's coefficient
u, R : velocity and hydraulic radius at location x and t
S_0 : salinity at the river mouth end
L : length of tidal propagation limit

It should be pointed out that these models need to be adapted for each given case and the constant(s) in these formulae are obtained from actual measurements. The method of measurement and analysis of dispersion coefficient proposed by Delft Hydraulics (Abraham, 1983) can describe very well the mechanism of the dispersion process.

4. Analysis, field studies and measurements

4.1 Insight to be obtained from analysis

In an analysis of seawater intrusion into a given estuary several items are to be considered. Basic questions are:

- what is the problem to be addressed,
- what is the existing situation, what is known about it,
- what is known about developments in the past, and
- what is the information to be generated, if any.

The problem to be addressed can be:

- how to prevent a seawater intrusion problem in the future, e.g. when constructing a dam to extract river water in the upstream range of the estuary, when deepening a navigation channel for navigation purposes or when developing land for agricultural purposes along the estuary;
- or how to solve an existing seawater intrusion problem, e.g. when the water taken in at a given location cannot be used any longer because of too high a salt concentration.

To address a problem one needs information on the effect of technical measures and/or natural events on the seawater intrusion into the estuary under consideration. This information has to be obtained from:

- an analysis of the seawater intrusion into the estuary in the present situation;
- an analysis of developments that took place in the past; and
- a prediction of future developments on the basis of the above analyses.

Which information one specifically needs depends on the characteristics of the problem addressed and on its scale. For some problems, such as in regions in a monsoon climate, it may be most important to know the seasonal variation of the seawater intrusion. For other such as tidal irrigation it may be most important to know the variation within a tidal cycle. A primary aim of the analysis of the existing situation and past trends is to get an insight into the stratification of the estuary, and its main transport and mixing mechanisms. To obtain this insight one needs information on the geometry of the estuary, river flow and the tidal characteristics.

Within this context, depending on the nature of the problem addressed, it may be important to have an understanding of the time scales of the variation of the seawater intrusion, for example:

- from year to year (because of a variation of the river flow from wet years to dry years),
- within the year from season to season (because of a variation of the river flow from a wet season to a dry season),
- over a month (because of the spring tide-neap tide cycle),
- within a tidal cycle (which may be either diurnal or semi-diurnal),
- at special events (e.g. a temporary increase of mean sea level because of a landward wind).

To get the above insight for the existing situation it is important to collect information on the estuary from all agencies making field measurements in the estuary. The information to be collected includes:

- present geometry of the estuary,
- field data on seawater intrusion and (tidal) flow,
- information on the principal deterministic factors governing the seawater intrusion, e.g. astronomic tide, and
- statistical information on the principal stochastic factors governing seawater intrusion, e.g. river flow, mean sea level at river mouth, prevailing winds.

If available, historic records may be collected in order to get insight into past trends.

In the analysis of the collected information it may be possible to find relevant correlations, e.g. between the river flow and the salt consentration at given locations and times. Looking for these correlations it may be required to eliminate the effect of other parameters, e.g. the variation of the tidal characteristics with time, which may influence the above correlation between river flow and salt concentration. The problem under consideration can be solved on the basis of these correlations provided that they are sufficiently accurate and cover all relevant aspects of the problem.

In particular when information on past trends is available, it may be that the collected field data on salt concentration and flow provide a sufficient basis for the calibration and verification of a hydraulic scale model or a mathematical salinity intrusion model. If so, these engineering tools can be used for the further solution of the problem under consideration.

Additional information is to be generated when the collected information is insufficient to solve the problem under consideration and/or for the calibration and verification of a mathematical or hydraulic scale model that can be used for this purpose. Which additional information to collect depends on the problem to be addressed, on what information could be collected, on what mathematical model one intends to develop, on what hydraulics scale model one wants to apply, and so on.

4.2 Significance of stratification

The stratification of the estuary under consideration is an important feature in the above context. For estuaries of the arrested salt wedge type the salt concentration varies over the depth only. For a well mixed estuary it varies over the tidal cycle, not over the depth. For the intermediate partly mixed estuary the salt concentration varies both over the tide and over the depth. Therefore the stratification of an estuary controls the type of measurement to be made in order to gain insight into its behaviour or to obtain the experimental material needed for the calibration and verification of a mathematical or a hydraulic scale model. Further, the stratification of an estuary controls whether or not the surface water has a lower salt content than the bottom water and therefore can be used for water supply purposes. Seasonal variations of the river flow may result in a variation in the stratification of the estuary over the seasons.

In the case of stratification, not only the salt concentration but also the velocity varies over the depth. Salt concentration and velocity may vary both over the depth and the width of the estuary, for instance when a wide stratified estuary has a pronounced bathymetry with ebb channels which are different from the flood channels.

4.3 Need for measurement

4.3.1 Basic justification

Seawater intrusion into rivers is determined by the related flows and transport processes in the river mouth zone. Although river mouths have basic transport processes in common, the flow and transport processes as observed in a given river mouth, however, form an unique blend of the different basic transport processes, depending on the unique individual properties of this river mouth such as its geometric features, the characteristics of its tributary rivers, etc. Therefore, correlation methods which are found to work well to describe the combined effect of the different basic transport processes in a given river mouth do not necessarily work in another (after Fischer *et al.*, 1979, Section 7.1). Exploitation of the resources of a river mouth system should be based on the characteristics of such a system, and the transfer of empirical formulae from other systems without supporting measurements may lead to dangerous conclusions.

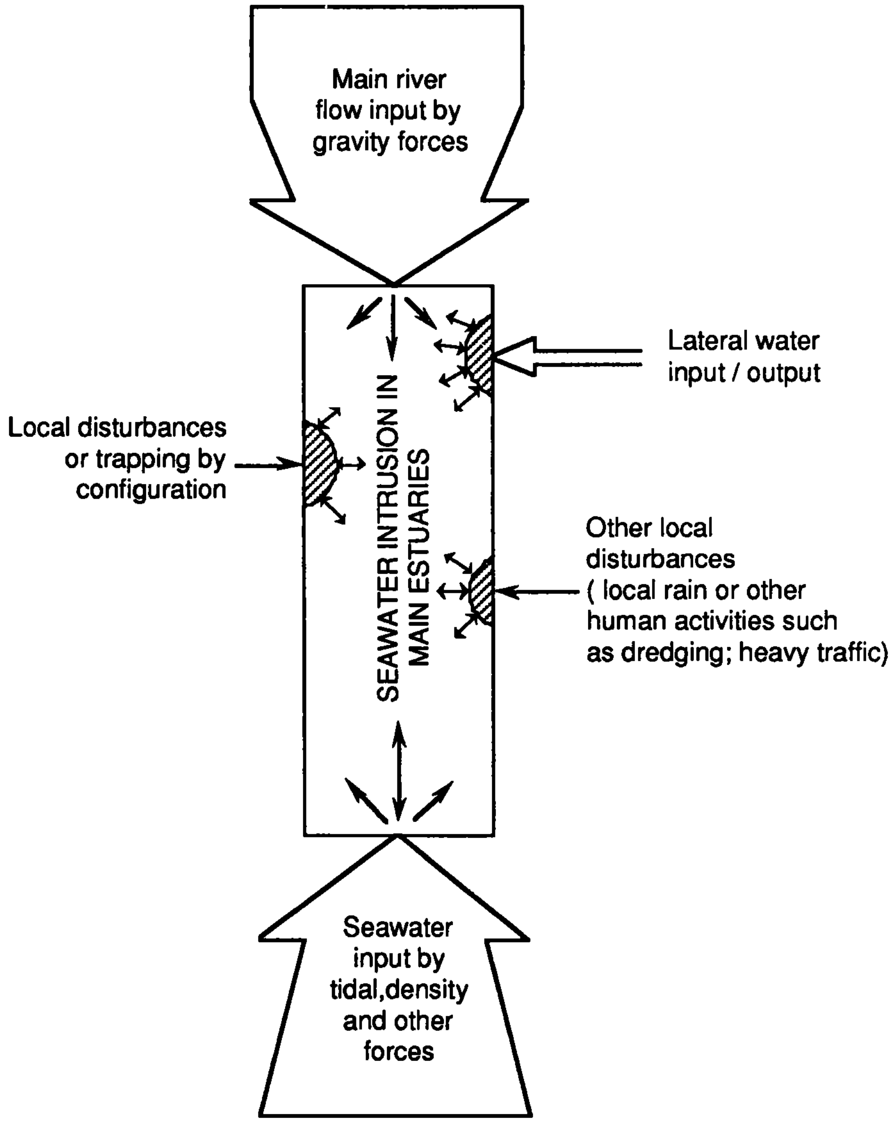


Fig. 4.1 Graphic illustration of seawater intrusion with localized problems

4.3.2 Scales of studies

The intrusion of seawater in a river mouth can normally be described as a macro-scale phenomenon with various localized problems such as described in Fig. 4.1. These localized problems rarely have important effects on the overall properties of seawater intrusion. Instead, the overall phenomenon will decide the magnitude of influence of the local problems. The interactions between local problems and the overall phenomenon are of interest to field engineers and river mouth management experts. The inclusion of these interactions in field investigations depends on the scale of studies as decided by the objectives of the corresponding project. For projects dealing with the overall development of a river mouth system, the macroscopic patterns and parameters should be investigated and the possible effects of local problems identified and excluded. For local development activities, local phenomena should be studied within the framework of the overall system. Examples can be taken from studies on the exchange between harbour basins and river mouths including the effects of salinity intrusion, as described by Abraham (1983, pp. 8.7–8.14). In that study, field investigations were mainly confined to the neighbourhood of the Botlek harbour complex in order to monitor the exchange pattern and mechanism. The volumes of flow through the harbour mouth will enable assessment of the effects of the harbour tidal basin on the overall conditions of seawater intrusion to be made. Although these effects are not readily available in that reference, it can be seen from Fig. 4.2 that the trapping effects of the harbour, as a local phenomenon, may influence the patterns of seawater intrusion around the harbour mouth. Data collected in this neighbourhood do not therefore necessarily reflect the macroscopic behaviour of the river mouth system.

In short, it is always important to identify the major factors governing the overall phenomenon of interest in order to establish the focus of field measurements. In this process of data collection, effects of localized problems should be avoided as far as possible.

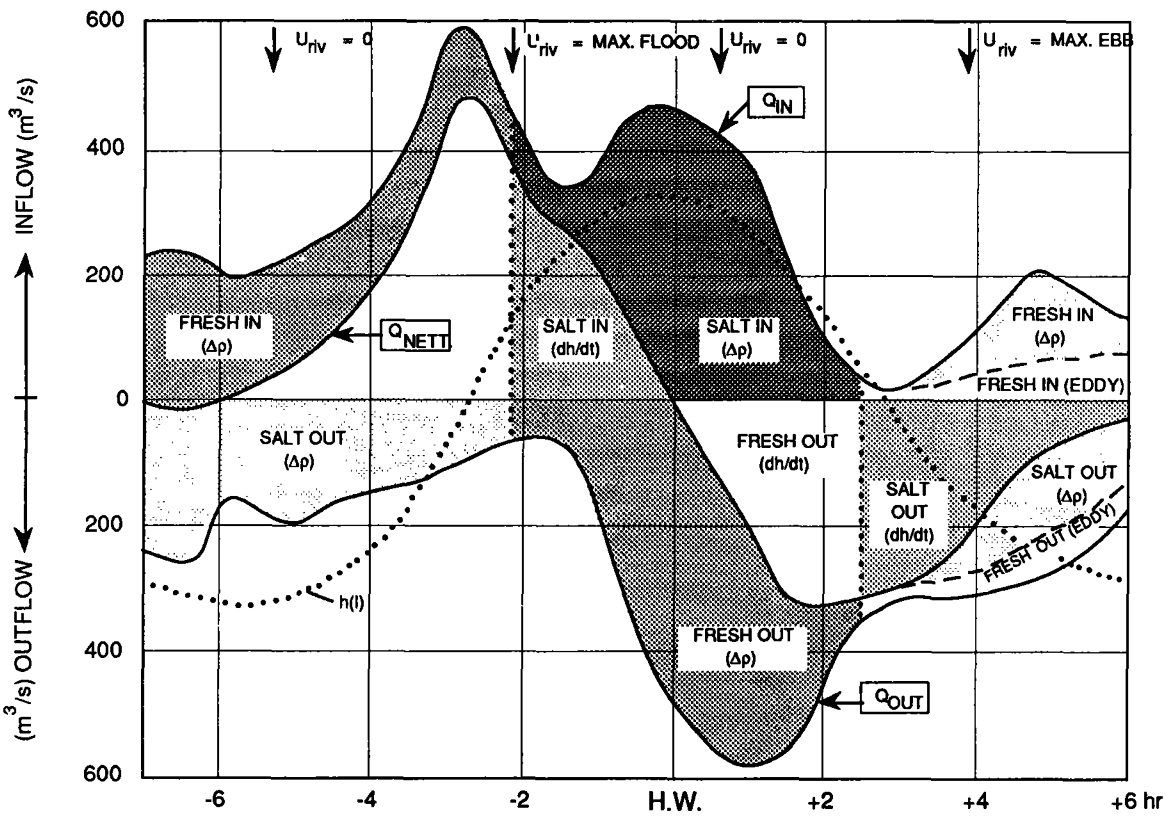


Fig. 4.2 Net flow through mouth of Botlek Harbour (measured values, Abraham, 1983)

4.4 Determination of parameters of processes

4.4.1 Basic time scales

The intrusion of seawater in river mouths is a time-dependent phenomenon. The large variation of river flow inputs, as discussed in Chapter 3, may drastically change the parameters of mixing mechanism and thus the characteristics of seawater intrusion. In general, it is necesesary to observe how the overall phenomenon reacts to different boundary conditions. However, in practice the priority of development objectives and the limitation of financial and human resources would require detailed field investigations be addressed to the most critical period of time, during which the intrusion has its most important impact on the corresponding projects. With respect to agricultural development activities and freshwater supply projects, the most critical period is normally determined by the lowest river flow period.

4.4.2 Parameters of field studies

From the basic concept of seawater intrusion as shown in Fig. 2.5, three major groups of parameters of field studies can be identified: configuration of river mouths; external parameters; and internal parameters of seawater intrusion.

Data on the configuration of river mouths are normally composed of the hydrography of river channels and the coastal zones and land topography of tidal areas, especially those acting as tidal storage. Although these data do not normally vary significantly with time, they may be subject to change under the effects of development works or extreme natural conditions.

The measurement of the configuration of river mouths may be a simple task for a small and stable single-channel river mouth system but difficult work for a large and complex river mouth system. However, initial planning for field investigation can normally be based on the charts used for navigation. These charts usually require further verification and updating with new data to ensure the necessary accuracy of data utilized in the corresponding studies.

Monitoring of the variation and estimation of the parameters of boundary conditions would provide necessary information relating to different conditions of seawater intrusion. The external parameters include inputs of upland freshwater discharge, tidal characteristics and sea salinity, meteorological factors and human intervention if any.

The internal parameters of seawater intrusion are defined by the characteristics of the mixing of fresh and seawaters inside the brackish water zone. These parameters include the effects of convective transport, diffusive and dispersive transport and trapping phenomena, if any.

4.5 Organization of field studies

As discussed above on the need for measurement, any study of seawater intrusion into rivers must be supported by field studies. In order to provide practising engineers with some simple guidelines on field studies, this section will identify the types of measurement involved and their relative importance and possible steps of organization of field studies.

4.5.1 Types of measurement

From the three major groups of parameters identified in Section 4.4.2, five principal types of measurement will be involved in possible detailed studies of seawater intrusion:

- topo-hydrographic;
- water level;
- velocity and discharge;
- salinity and other water quality parameters including temperature; and
- meteorological.

Although these types of measurement are normally well known to practising hydrologists, it should be mentioned here that there is an important difference in the conditions of measurement between the fresh and brackish water zones. The basic difference is the salinity of water, which affects the working conditions of instruments, with its higher conductivity and algal and barnacle growth, and also possibly higher sediment concentration due to flocculation. The conditions together with the presence of tides affect considerably the accuracy of measurements. For practising engineers, it is always worth considering the following factors in field study work: consistency between data; and cost–versus–accuracy.

It is essential that all instruments be calibrated before deployment.

a. Topo-hydrographic measurements

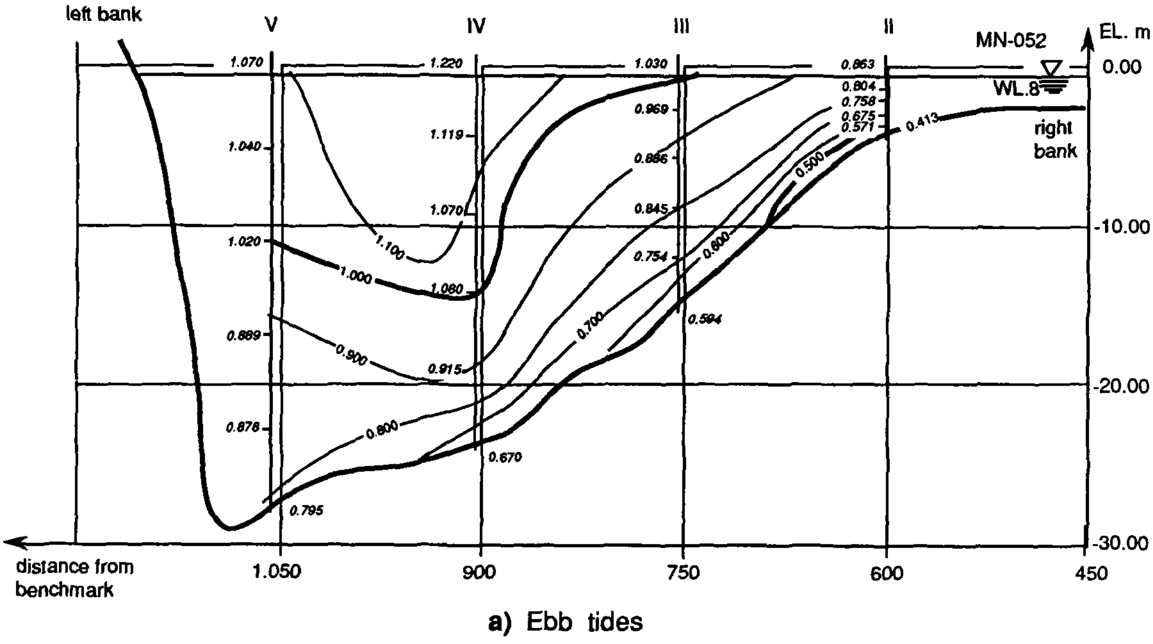
The accuracy of topographic surveys is normally greater than that of hydrographic surveys. Depending on the conditions of each river mouth system, the accuracy of hydrographic measurements is generally determined mainly by the type of depth sounders, channel bottom conditions, variation of water levels (tides and waves), and methods of positioning (excluding human errors and severe climatic conditions). It is always possible to improve the accuracy of measurements with the introduction of more sophisticated and precise instruments. However, the higher costs involved may not be justified by the subsequent utilization of these data and the consistency with other types of data.

b. Water level measurements

This type of measurement may be the simplest and cheapest activity that can be made by either observers or automatic recorders. Care should be paid to the selection of station location, which may be subject to local effects. As an example, the water levels at stations located at a bend may show significant differences between the two opposite banks. On the other hand, stations located at the mouths of creeks or canals flowing into main river channels tend to have smaller tidal amplitudes and higher troughs. With respect to the use of automatic recorders, effects of salinity, tides and ambient conditions are factors in the accuracy of recorded data.

c. Velocity and discharge measurements

Discharge measurements in the tidal reaches are always difficult in view of the different velocity distribution between the flood and ebb tides such as experienced in the Mekong Delta and reflected in Figs. 4.3a and b (Tran Duc Kham *et al.*, 1987). It is all the more difficult in the zone with the effects of important salinity gradients. It is therefore recommended that, as far as is feasible, measurements of upland freshwater discharge be made in the non-tidal zone. The measurement of flow velocity may be required in the brackish water zone for the determination of internal parameters of seawater intrusion.



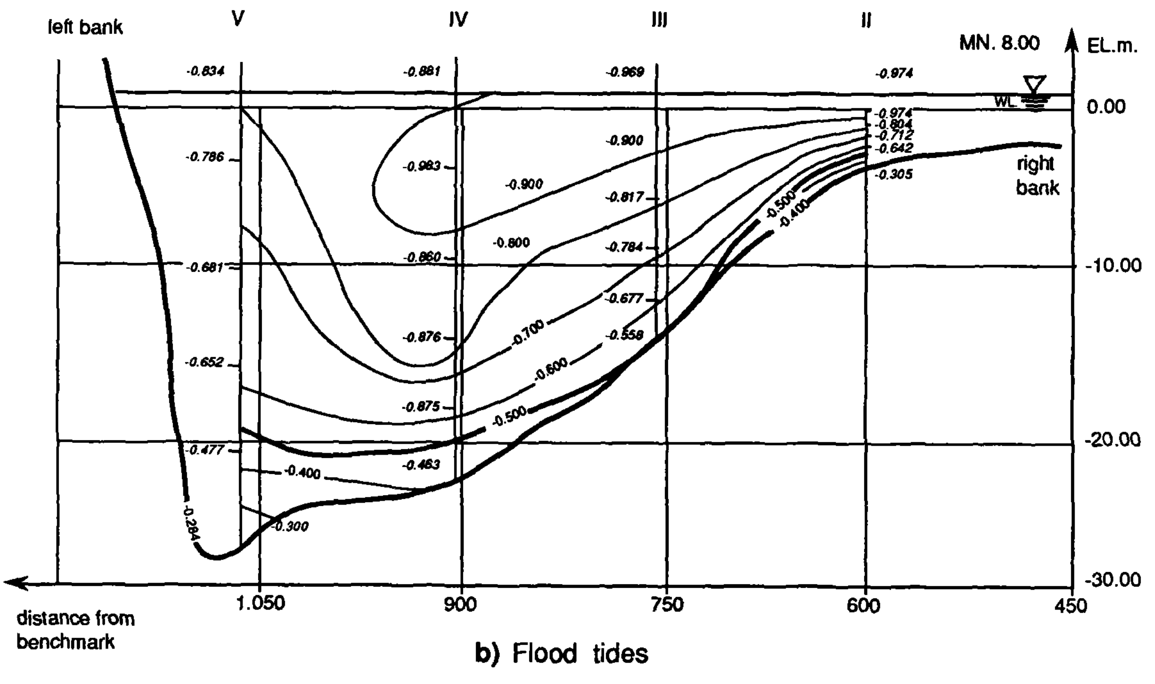


Fig. 4.3 Cross sectional distribution of velocity of the Bassac at Can Tho in the Mekong delta during low flow period (10 April 1985 after Tran Duc Kham *et al.*, 1987)

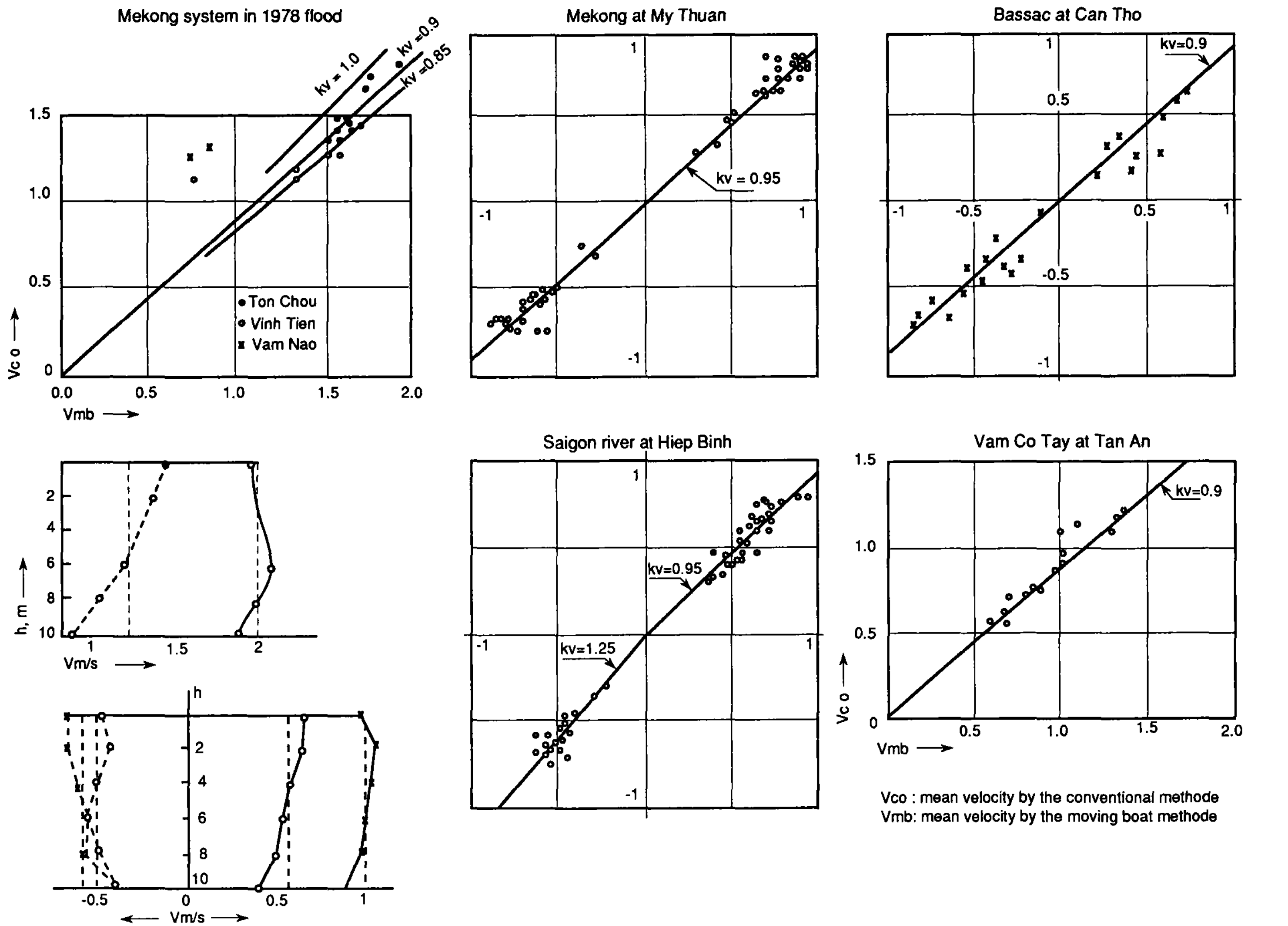


Fig. 4.4 Typical velocity profiles and estimation of velocity correction factor kv (after Nguyen Hac Vu and Chu Thai Hoanh, 1983)

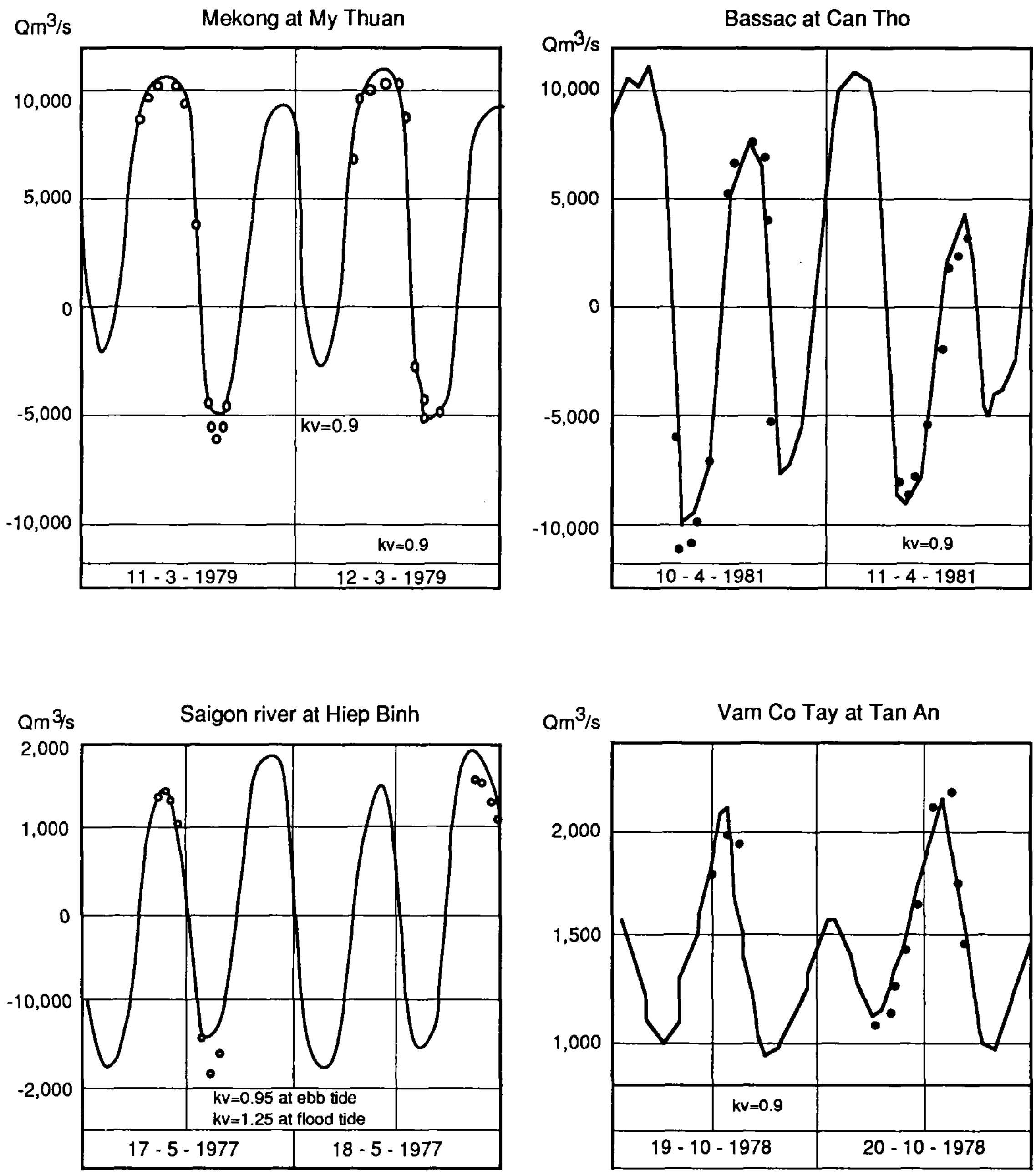


Fig. 4.5 Comparison of discharge data measured by the conventional and moving boat methods (after Nguyen Hac Vu and Chu Thai Hoanh, 1983)

Different methods of discharge measurements — conventional, moving boat, index methods etc. — are well described in the hydrological manuals published by WMO. It should be noted that any coefficients adopted in the various method would need to be determined by the local conditions. This can be demonstrated by the experience in the Mekong delta using the moving boat method, as shown in Figs. 4.4 and 4.5 (Nguyen Hac Vu and Chu Thai Hoanh, 1983).

d. Salinity and water quality measurements

The measurement of water salinity is normally carried out with the use of a conductivity meter. As the conductivity of water does not depend only on salinity but also on temperature, frequent collection of water samples for subsequent calibration is necessary. Furthermore, the salinity of waters depends upon the effectiveness of the mixing process, and the location of sampling stations is always important in the determination of representativeness of data.

4.5.2 Phasing of field studies

In a major and complex river mouth system, a detailed study of seawater intrusion may be a costly undertaking, and in order to make best use of financial and human resources, such studies should be carried out in phases: a preliminary study; a monitoring network; and detailed measurement campaigns.

The most essential condition of measurement in a monitoring network and in measurement campaigns is synchronization.

a. Preliminary study

Within this phase, compilation of existing data and past records on the three major groups of parameters of field studies mentioned in Section 4.4.2 are to be conducted. Subsequently field reconnaissance should be made so as to identify the areas of interest based on the experience of local population, topography and vegetation and to establish a minimum network of monitoring.

b. Monitoring network

This second step will provide a more accurate understanding of the magnitude and the delineation of the problem in the spatial dimension as well as in time. The accuracy of such understanding depends not only on the accuracy of the data but also the representativeness of these data, for which the selection of location of sampling is the deciding factor.

c. Detailed measurement campaigns

The first two steps may provide enough information for a good understanding of the configuration of river mouth systems and the boundary conditions. For a clear determination of the principal factor(s) in the mixing/transport process, detailed measurement campaigns must be carried out. These campaigns are also conducted for a complex river mouth system to determine the partial effects of different boundary conditions.

5. Impacts of changes in river mouth and river basin on salinity intrusion

5.1 Impact of changes in the river mouth system

5.1.1 Description of changes in configuration

As discussed in Chapter 2, changes in the river mouth configuration system can be affected by natural factors and human development activities. In terms of the impacts on seawater intrusion, these changes can be described in four groups, according to their effects: increase of tidal storage, decrease of tidal storage, decrease of tidal range upstream, and increase of tidal range upstream.

The division of changes into these four groups is only aimed at evaluating their principal impacts at a macroscopic point of view based on the basic concept of seawater intrusion presented in Fig. 5.1. Using this concept, one may have to assume that the mixing mechanisms does not change drastically in the cases considered. For example, the change from the well-mixed situation to a stratified one will have a much more important bearing on the study area than in a similar case without the change in the mixing mechanism. In the former case, many mixing factors would have to be studied, since the mixing mechanism cannot be determined by one single factor but rather by interactions between factors.

Under this section, the changes are normally not great and the assumptions presented above would therefore be valid and qualitative effects can be estimated.

5.1.2 Increase of tidal storage

Storage areas located within the tide-affected zone can be increased by the effects of natural changes or hydraulic works. The increase in storage area by natural factors may appear in the form of the destruction of natural levees or existing dikes by disasters or the increase of mean seawater levels by wind set-up or storm surges. An example on the effects of the destruction of dikes on seawater intrusion was well described by Orlob (1976) for the Sacramento-San Joaquin delta under the disaster in 1972. An increase in seawater intrusion from 2 to 3 miles was experienced due to the outbreak of dikes. The increase in mean seawater levels by wind set-up (the Mekong delta with constant eastern winds) or storm surges (typhoon- or cyclone-affected deltas of the tropics) normally results in longer periods of tidal floods because of the larger volume of storage corresponding to higher mean water levels. An increase in tidal storage areas can be also caused by the increase in the farming/aquaculture areas using tidal irrigation as practised in Southeast Asia; by the improved drainage conditions of tidal flats with or without control structures; and by the construction of new harbours.

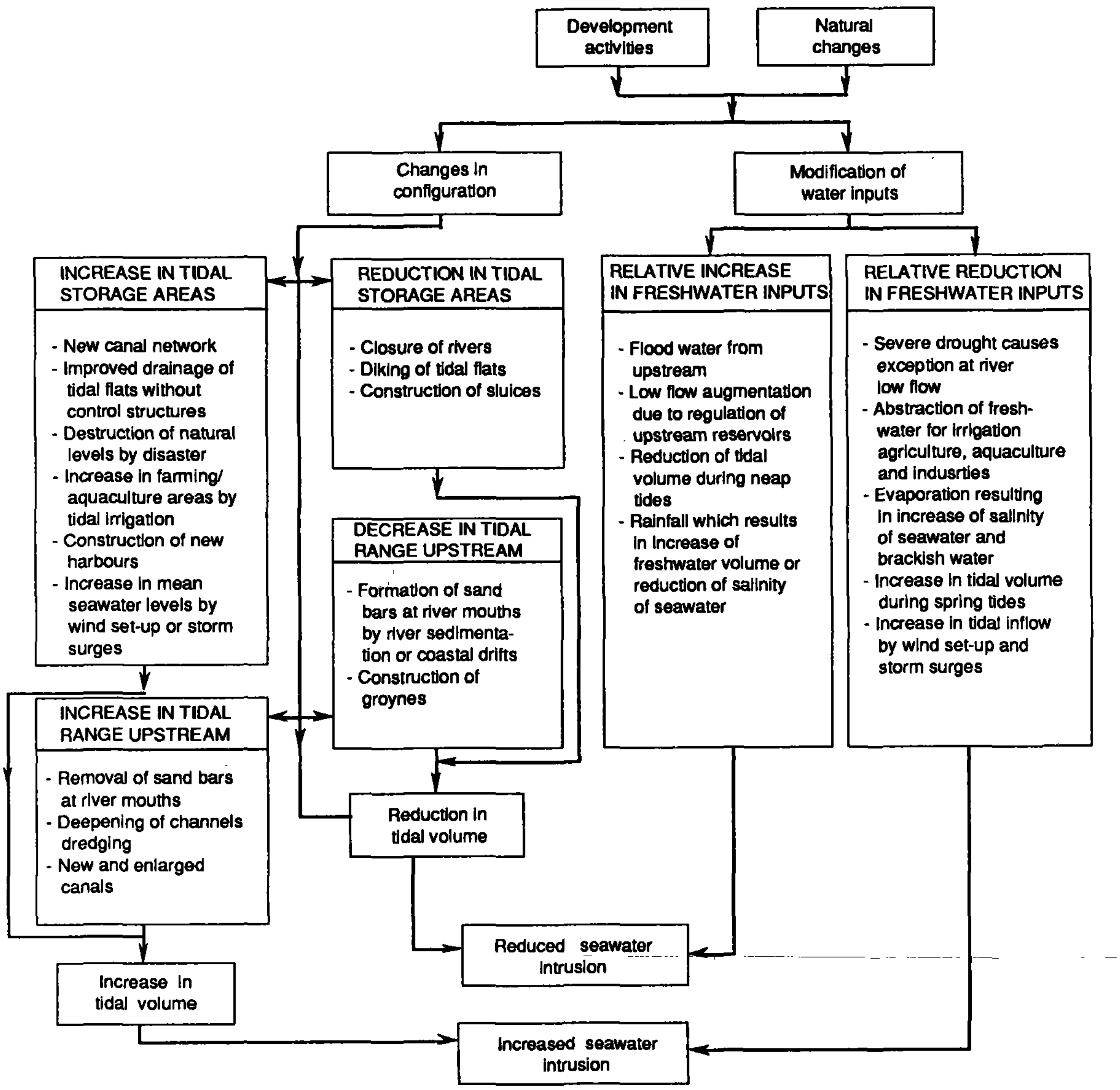


Fig. 5.1 Summary of possible impacts of natural changes and hydraulic works on seawater intrusion

With respect to the improvement of drainage conditions as commonly practised in Southeast Asia, important effects may be expected in view of the large areas involved such as demonstrated by areas of broad depression in the Mekong delta (Fig. 5.2).

With respect to harbour construction, the Rotterdam Waterway shown in Fig. 5.3 represents an intensive exploitation of river mouth systems for transportation. For developing countries, the existing economic conditions are normally not strong enough to support the construction of large harbours that could create a significant increase in the tidal storage area as such.

It should be emphasized here that the effects of increases in the tidal storage areas on the extent of seawater intrusion are felt only when the increase in volume of water caused by the increase in tidal storage areas is significant in comparison with that of the tidal prism (total influx and outflux of tidal flow). Otherwise, the effects are, in general, negligible.

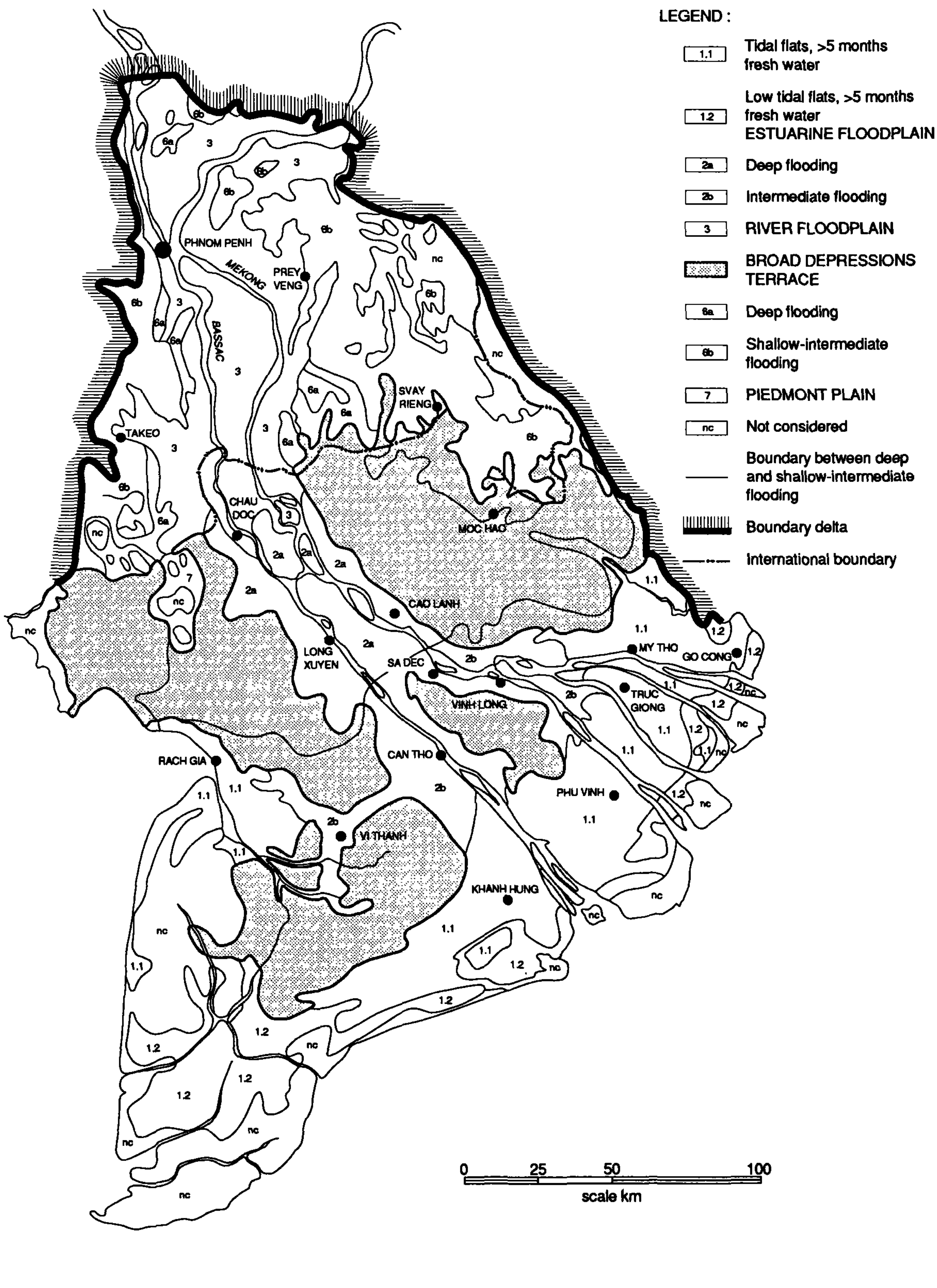


Fig. 5.2 Broad depressions with other agricultural production units in the Mekong Delta (Mekong Secretariat, 1974)

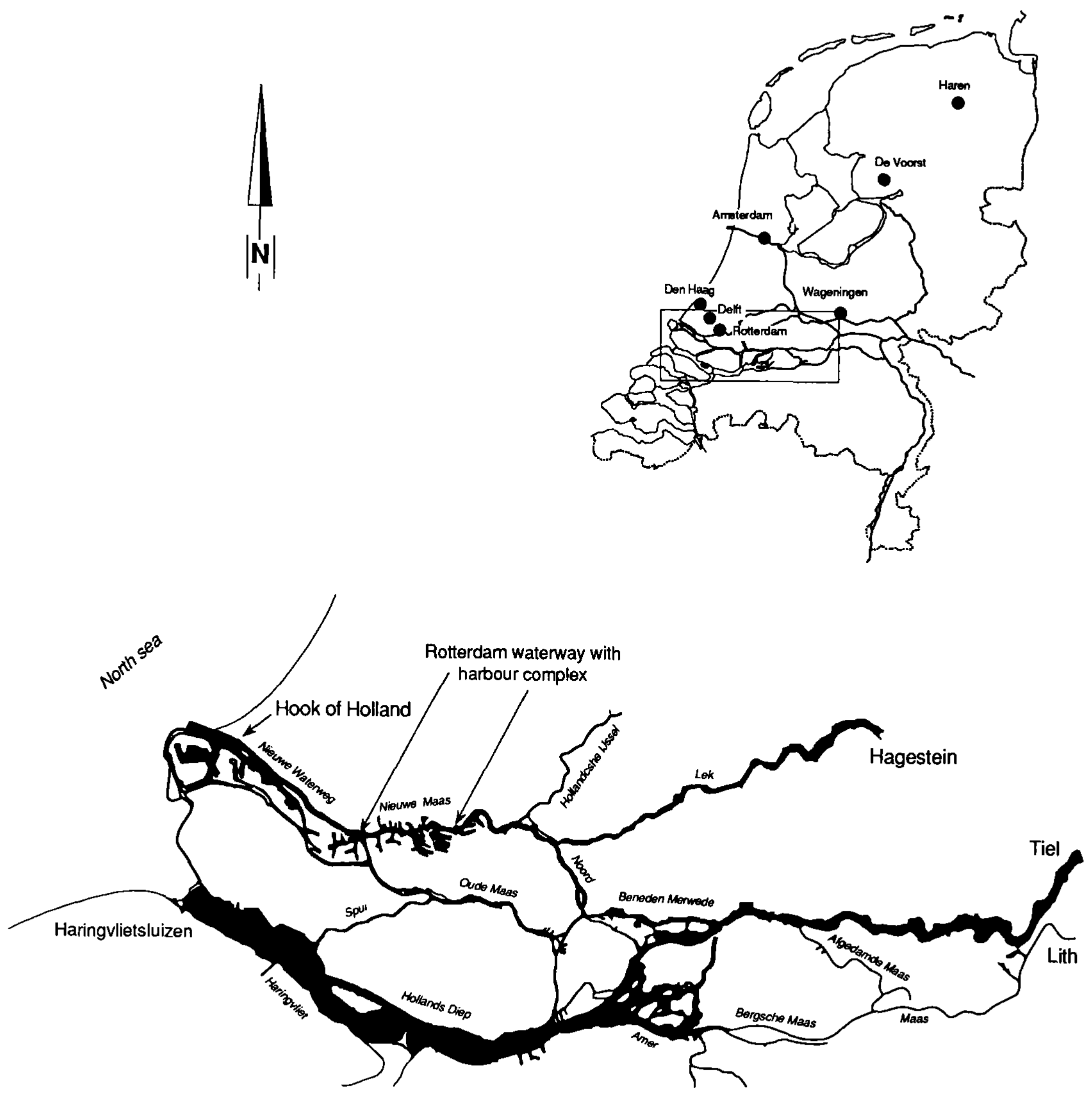


Fig. 5.3 Plan view of Rhine-Meuse estuary with the harbour complex of the Rotterdam waterway (Courtesy by the Delft Hydraulics Laboratory)

5.1.3 Decrease of tidal storage

The reduction in tidal storage areas is rarely caused by natural changes except in the regions of lagoon formation. The reduction is often effected by human actions such as closure of river mouths, diking of tidal flats and swamps, and the construction of sluices. The closure of a river mouth of several distributaries will have two major effects: a relative increase in freshwater inputs, which will be discussed later, and a reduction in the tidal storage areas. The diking of tidal flats is normally aimed at preventing flooding during the high and spring tides. For the Mekong delta, a systematic implementation of this approach may significantly influence the extent of seawater intrusion due to the large areas of tidal flat as shown in Fig. 5.4.

The construction of sluices in deltaic areas is also aimed at controlling seawater intrusion. As such, these sluices will reduce the tidal storage areas, at least during the period of high salinity. Quite often, these sluices are constructed together with dikes against tidal floods. But since they control the major routes of tidal flow, their effects are more important than those caused by diking.

5.1.4 Increase of tidal range upstream

Development activities aimed at improving flow conditions for either navigation or drainage purposes will normally result in an increase in tidal range upstream. In other words, the energy of tidal flow will reduce at a lesser rate when tides propagate inland than before. Consequently the mean water levels and tidal amplitudes would become higher. These effects can also be considered indirectly as the same as the increase in tidal storage areas.

The increase in tidal range upstream can be effected by the removal of sand bars at river mouths, the deepening of channels by dredging for navigation, or the excavation of new canals or the enlargement of existing waterways for drainage improvement. For the Mekong delta, attenuation of tidal propagation can be clearly seen in Fig. 5.5, and introduction of new canals and enlargement of existing waterways will improve considerably the drainage of the hinterlands.

5.1.5 Decrease of tidal range upstream

Decrease in tidal range upstream is hydraulically caused by obstacles to tidal flows, which are normally created by natural factors, especially in the sedimentation processes. These sedimentation processes can be dominated by either riverine factors or coastal (littoral) drifts. Sand bars or shoaling created by the riverine-dominated sedimentation processes usually take place during the high flow period with high sediment concentration. The large reduction in velocity at the river mouth, coupled with the strong influence of flocculation during this high flow period, results in the major deposition of sediments in this neighbourhood. This is a common phenomenon in Southeast Asia, as demonstrated by the related extension of the Bassac river mouth, one of the distributaries in the Mekong delta, by the shoaling process as shown in Fig. 5.6. The coastal drifts have also played an important role in the extension of the Bassac river mouth into the sea. The southern-seasonal currents, which are predominant, carry the sediments discharged by the northern distributaries of the Mekong delta southward.

A decrease in tidal range upstream may also be caused by groynes, which are normally constructed to increase depth required by navigation.

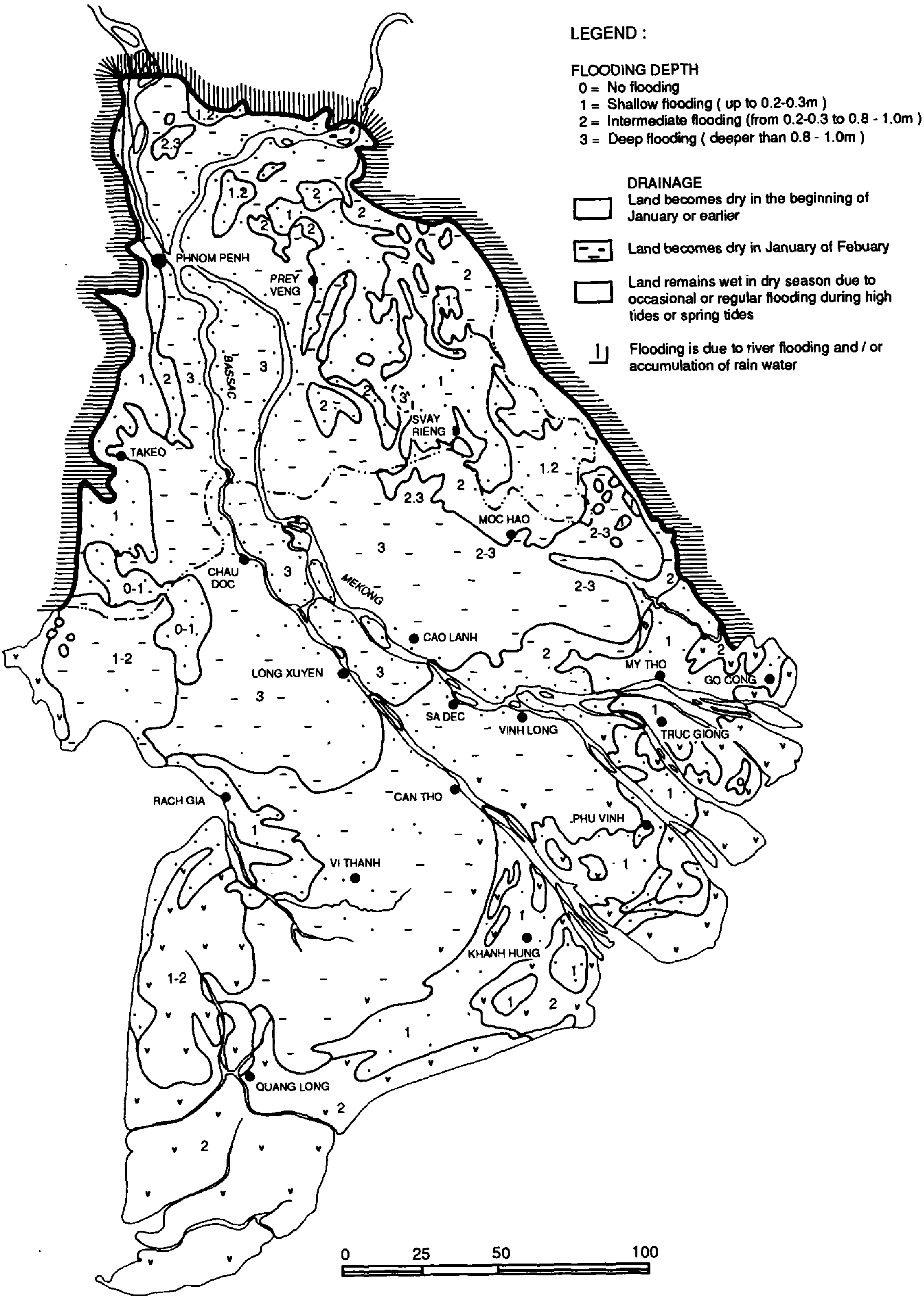


Fig. 5.4 Flooding depth and drainage conditions in dry season in the Mekong Delta (Mekong Secretariat, 1974)

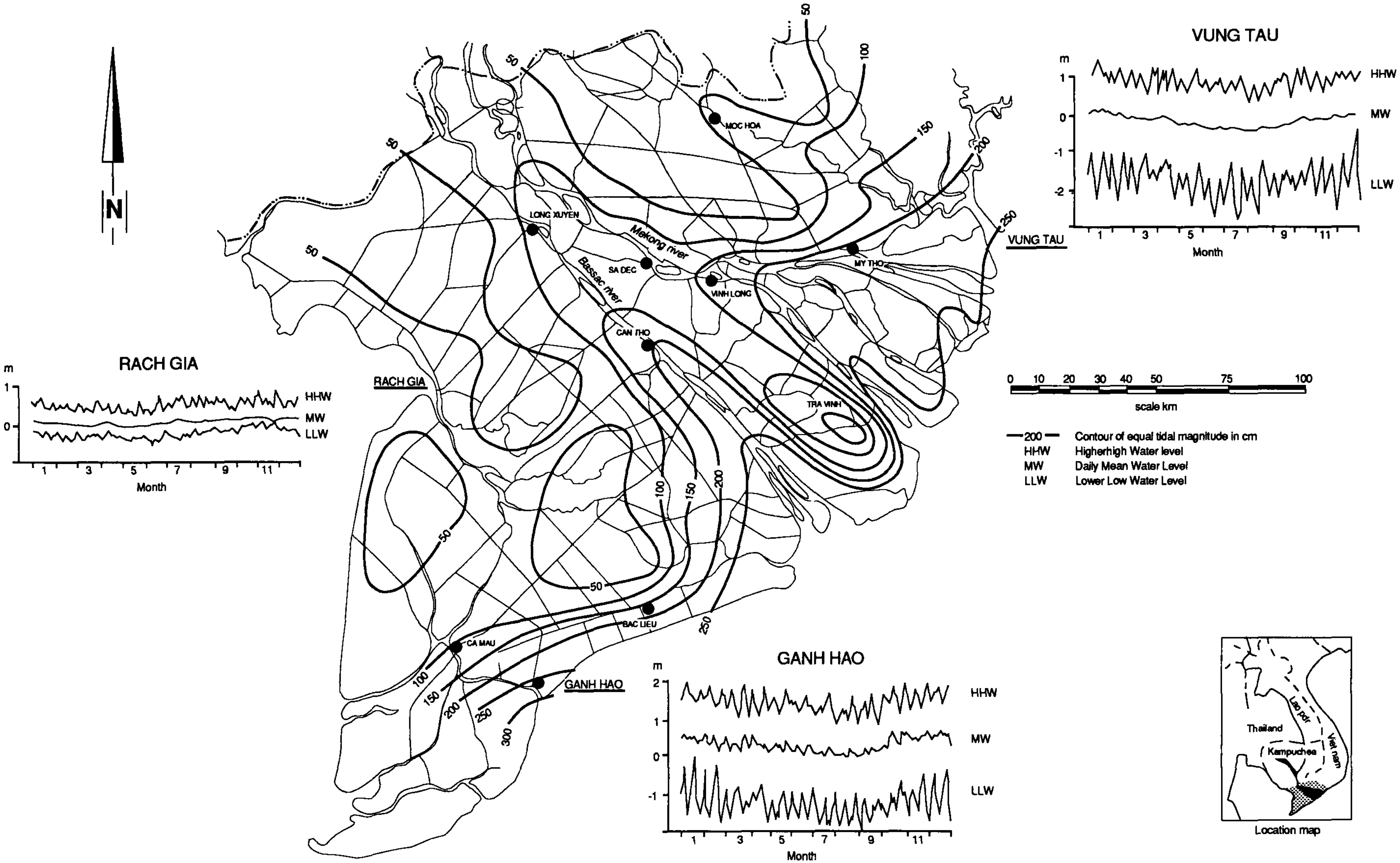


Fig. 5.5 Attenuation of tidal propagation in the Mekong Delta

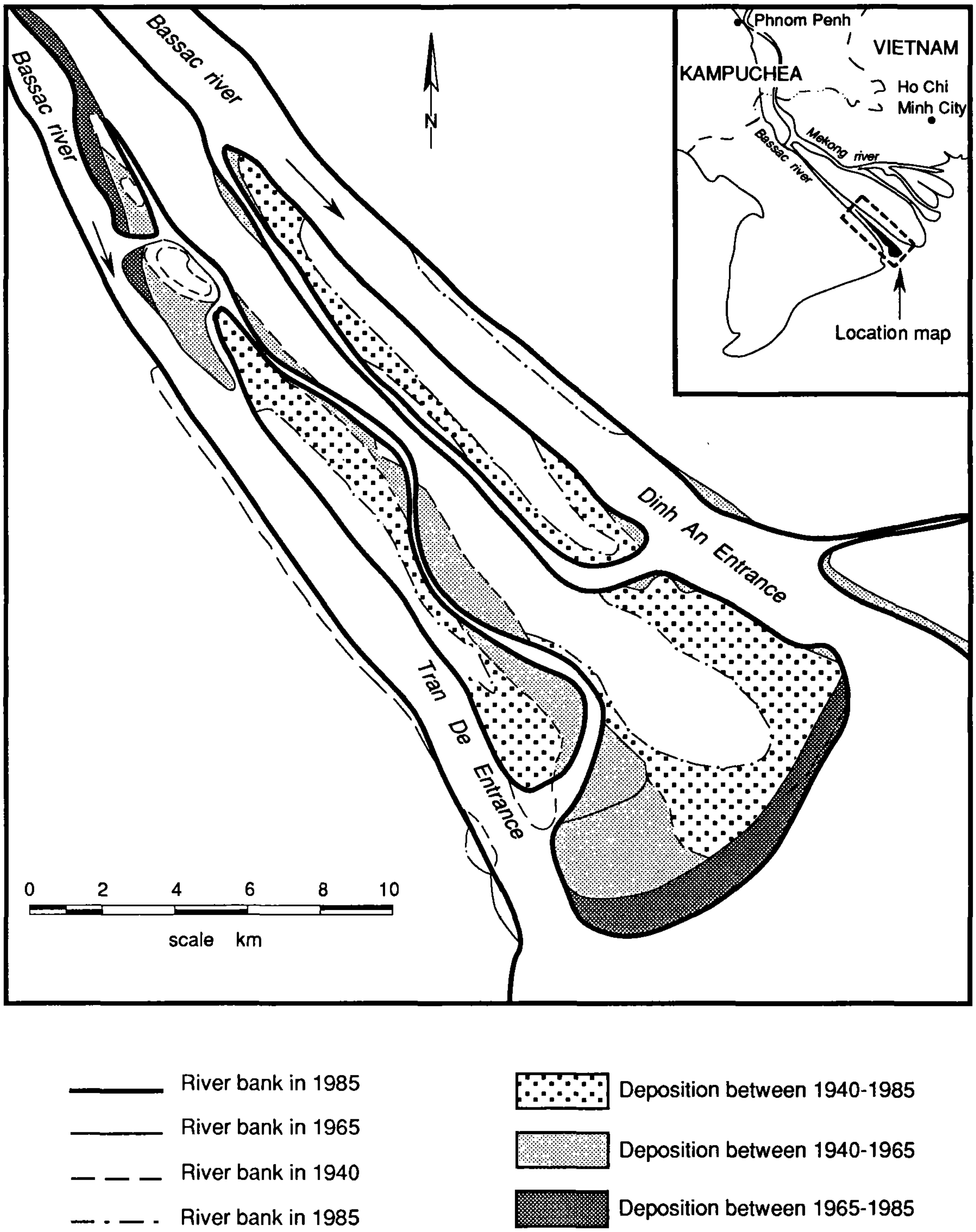


Fig. 5.6 Extension of the Bassac river mouth, one of the Mekong distributaries (Mekong Secretariat, 1976)

5.2 Modification of freshwater inflow

The physical phenomenon of seawater intrusion is mainly controlled by the inputs of freshwater by upstream discharge and of seawater under the effects of tidal fluctuation. The extent of intrusion and also mixing types can normally be correlated with the inputs of freshwater relative to the tidal prism. It can therefore be expected that a substantial change in the relative freshwater inputs by either natural factors or human actions will result in a substantial modification of the seawater intrusion. This section will focus on two categories: a relative increase and a relative reduction in freshwater inflow.

5.2.1 Relative increase of freshwater inflow

The relative increase of freshwater inflow may be the result of an increase in freshwater input or a reduction of tidal inflow. With respect to the reduction of tidal inflow, except for the change of the configuration especially at the river mouths by natural factors or human actions as already discussed in the previous section, it can be seen here that the most important reduction would normally be caused by the changes from spring to neap tides.

The increase of freshwater inflow on the other hand, can be caused either by natural factors such as flood water from the upstream basin and rainfall or by human development activities such as low flow augmentation due to the regulation of upstream reservoirs.

With respect to the reduction of tidal inflow as caused by the neap tides, a reduction in the extent of seawater intrusion would normally be expected. However, in a complex river mouth system with large tidal depression areas, a phase lag may exist between the variation of the astronomical tides and the fluctuation of seawater intrusion. This implies that the neap tides may not correspond to the short distance of seawater intrusion. See also Chapter 8, Case Study 8.1, in which the effect of harbour basins along the Rotterdam Waterway and the effect of the tidal amplitude on the seawater instrusion are discussed in more detail. It is shown that in the Rotterdam Waterway under neap tide conditions the seawater intrusion length is larger than under spring tide conditions. Obviously the degree of stratification strongly determines the intrusion length in the Rotterdam Waterway.

Concerning the natural variation of upstream freshwater inflow, the flood discharges can be as great as over 50 times of those during the low flow period, as experienced in many tropical rivers. In arid regions, this ratio would be much higher. This large variation of inflow may drastically affect the mixing mechanism such as from the well-mixed case to the stratified condition. For large rivers, the increase in upstream freshwater inflow is normally gradual, the extent of seawater intrusion would consequently reduce gradually. For small river basins, the change may take place more quickly. The augmentation of low flow by upstream reservoir regulation would also fall into this category. In terms of the extent of seawater intrusion, the resulting reduction may be similar, but the effects caused by the low flow increase on other sectors or resources systems may be more important due to the different time of occurence.

The relative increase of freshwater inputs by rainfall can influence the extent of seawater intrusion in two ways: the reduction of seawater salinity and the increase of freshwater inputs through the lateral flow. The effects of rainfall through the increase of freshwater lateral flow on the overall extent of sea-water intrusion would normally depend on the concentration time of the corresponding drainage basin and the dispersion of the lateral flow into the main flow. Quite often, low dispersion rates would create important salinity gradient across the channel and a steep salinity gradient in the longitudinal direction of the corresponding drainage sub-basin.

5.2.2 Relative reduction of freshwater inflow

Relative reduction of freshwater inflow may correspond to a reduction in freshwater inflow or an increase in tidal influx. The increase in tidal influx is normally caused by natural factors such as the spring tides or wind set-up and storm surges. The reduction in freshwater inputs, on the other hand, can be caused by natural factors such as severe drought and evaporation and by freshwater abstraction for irrigation, aquaculture and other water supply purposes.

By contrast with neap tides, spring tides would increase the extent of seawater intrusion. Their effects may be delayed in a complex river mouth system by the influence of storage areas. Large wind set-up or storm surges may couple the effects of increase of tidal influx with better mixing processes between the fresh and seawater to significantly increase the extent of seawater intrusion.

The effects of water abstraction for water supply purposes, of severe droughts or of evaporation are similar to the extent that they will worsen seawater intrusion. The local effects may be different depending on the location and time of occurence. The effects of severe droughts are normally felt throughout the river mouth system; those of abstraction may be felt more pronounced locally in the neighbourhood of withdrawal; and evaporation may be more important in the tidal flats. For the arid river mouth systems, the effects of evaporation may be more pronounced since the freshwater inflows are negligible during the dry season.

5.3 Impacts in non-tidal river mouths

In non-tidal river mouths with a small tidal range the most important man-made factor that influences the seawater intrusion is deepening of the river mouth bar.

In many non-tidal deltas in natural conditions seawater usually do not penetrate into the delta branches even with very small river discharges. The shallow river mouth bar with natural depths usually of about 0.5–1.5 m (rarely to 2.0–2.5 m) is a very serious obstacle for seawater intrusion into the river. But dredging of navigation canals through the river mouth bar and deepening them to 4–6 m or more leads to an active penetration of the more dense seawater into the river channel, especially when low river flow is taking place. Therefore seawater intrusion is a very typical process in non-tidal rivers or delta branches with artificially deepened mouth bars and in the period of a low river flow.

In natural conditions seawater intrusion practically was not observed in mouths of such rivers as the Mississippi (USA), the Danube (USSR, Romania), the Rhône (France), the West Dvina (Daugava) (USSR), the Yana (USSR) etc. But after deepening of the river mouth bars for navigation seawater penetrates into the rivers to very long distances when low flow conditions prevail. For example into the Mississippi nearly up to Baton Rouge (more than 200 km), into Salina branch of the Danube delta up to 20–30 km, into the West Dvina River up to 15–20 km, into the Yana River up to 40 km etc. As a rule the process of seawater penetration into the rivers develops in these cases in accordance with the saltwater wedge type of penetration. The scheme of this phenomenon is illustrated in Fig. 5.7.

5.4 Summary of possible impacts

The impacts of natural changes and hydraulic works on seawater intrusion based on discussions presented in Sections 5.1 and 5.2 can be divided into two groups: reduction in the extent of seawater intrusion and increase in seawater intrusion. The reduction in the extent of seawater intrusion can

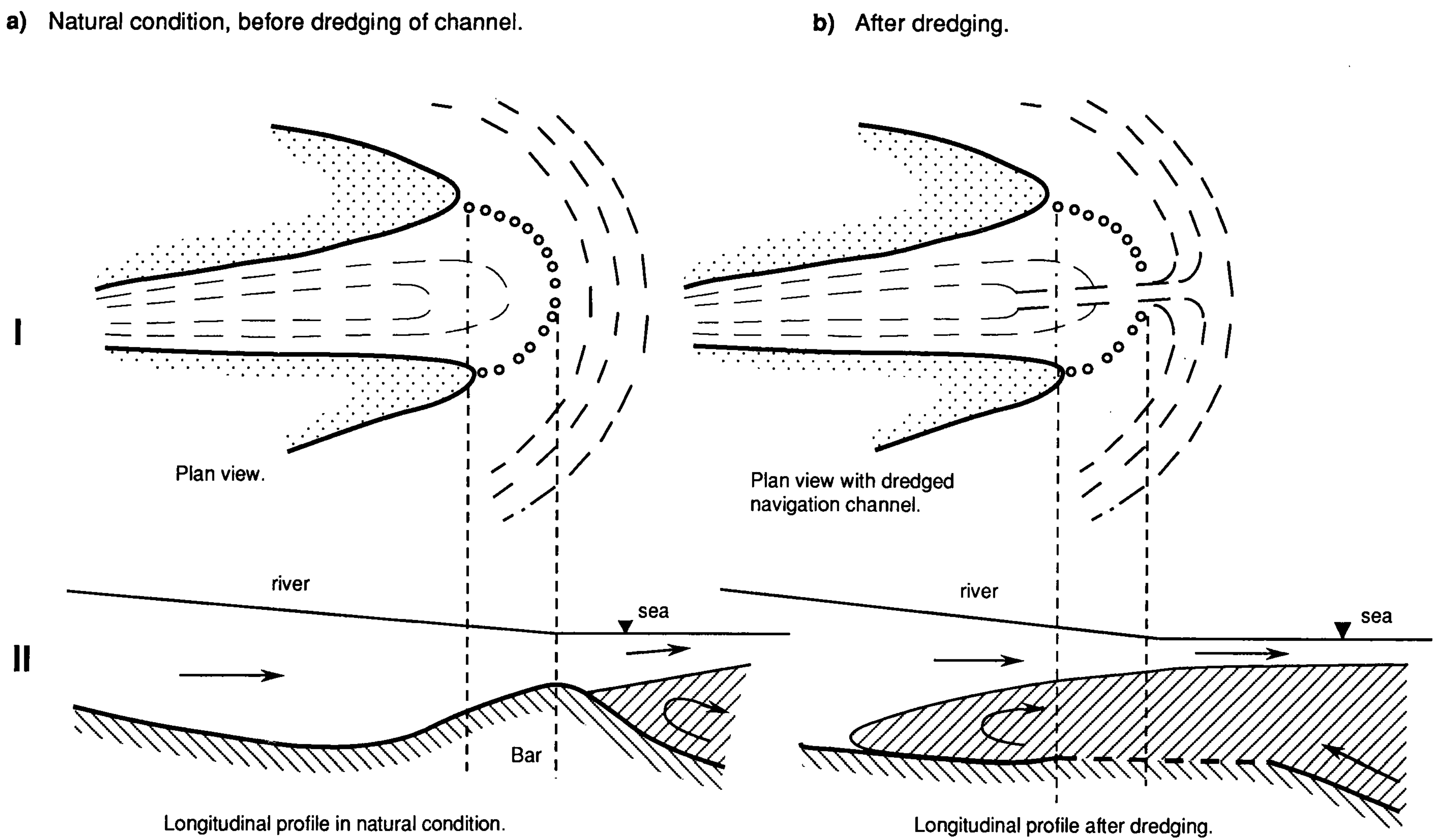


Fig. 5.7 The influence on seawater intrusion when a navigation channel is dredged in a rivermouth bar

result either from the relative increase of freshwater inflow or from a reduction in tidal volume created by a decrease of tidal storage or decrease of tidal range upstream. The increase in seawater intrusion, on the other hand, may be caused either by a relative reduction of freshwater inflow or by an increase in tidal volume as a result of an increase of tidal storage or an increase of tidal range upstream.

The classification of the activities and natural changes together with their impacts are summarized in Fig. 5.1 for easy reference.

6. Methods of prediction

6.1 Review of existing methods

Prediction must be based on a quantitative understanding of the seawater intrusion in an estuary under consideration. This quantitative understanding must be derived from field data collected in this estuary. As a general role one may say that predictions cannot be made unless one has sufficient field data available. There are hardly exceptions to this rule. One cannot study an estuary only from behind one's desk. This is because estuaries only have basic transport processes in common. However, the flow and transport processes as observed in a given estuary form an unique blend of the various basic transport processes, depending on the unique individual properties of the estuary such as its geometric features, the characteristics of its tributary rivers, etc. Therefore, correlation methods which are found to work well to describe the combined effect of the different basic transport processes in a given estuary do not necessarily work in another (after Fischer *et al.*, 1979, Section 7.1).

Suppose that for a given estuary one knows from field observations the seawater intrusion when the river flow is, say, 1000 $m^3 s^{-1}$ and 2100 $m^3 s^{-1}$. Further, suppose that one has to know the seawater intrusion when the river flow is 1450 $m^3 s^{-1}$, in order to solve the seawater intrusion problem under consideration. Then one may expect that the seawater intrusion for the 1450 m^3s^{-1} river flow is somewhere between that for the 1000 $m^3 s^{-1}$ and 2100 $m^3 s^{-1}$ river flows. Then, predicting the 1450 $m^3 s^{-1}$ seawater intrusion involves interpolation between known values.

A different situation arises when, in the above example, one wants to know the seawater intrusion for a river flow of 500 $m^3 s^{-1}$. This situation may arise when a dam is being built in the upstream range of the estuary to extract river water and to use it for agricultural purposes. Then, one may expect that the seawater intrusion for the 500 $m^3 s^{-1}$ river flow is more severe than for the 1000 and 2100 $m^3 s^{-1}$ river flows. Then, predicting the 500 $m^3 s^{-1}$ seawater intrusion involves extrapolation beyond the range of river flows for which the seawater intrusion is known from observations. Other problems which make extrapolations necessary arise when modifications of the geometry of the estuary are being considered, e.g. deepening the navigation channel for navigation purposes.

Predictions are more easily made when they involve interpolation, and do not require extrapolation. Extrapolations require still more care than do interpolations. The crucial question is how far can one extrapolate: to the 500 $m^3 s^{-1}$ seawater intrusion in the above example, or even more to the 250 $m^3 s^{-1}$? Unfortunately, there is no unique answer to this question. Answering it involves engineering judgement, taking into consideration factors like whether or not there is a theoretical support for the extrapolation, experience obtained in other estuaries, and so on.

Prediction methods, which also can be applied in the analysis of available field data, include: correlation methods, mathematical models, or hydraulic scale models.

There is a gradual transition between prediction and analysis, since prediction must be based on a quantitative understanding of seawater intrusion in the existing situation. This understanding has to be derived from the analysis of available field data.

6.1.1 Correlation methods

Correlation methods are a powerful tool, in particular for interpolations and when substantial field data are available. When these conditions are satisfied it may be possible to find a correlation between the maximum and minimum daily bottom and surface salinity, the location along the estuary and the river discharge.

Looking for correlations of the above type it is important to realize that the daily maximum and minimum salinity not only depends on the river discharge, but also on the strength of the tide and on the mean sea level. For instance, when mean sea level starts to rise because a landward wind begins to blow, seawater penetrates further into the estuary than it does in the situation without this rise. In addition, some of the available data may have been collected under equilibrium conditions, others under transient conditions. These factors may blur the correlation between salinity, location along the estuary and river discharge, unless properly accounted for.

The Chao Phya estuaries study, already mentioned in Section 3.1.8 in relation to transient conditions, illustrates the applicability of correlation methods.

6.1.2 Hydraulic scale modelling

In a hydraulic scale model both the geometric features of the estuary under consideration and its flow and saltwater intrusion characteristics are to be reproduced. The latter requirement involves: proper selection of the model scale, proper adjustment of the model, and proper reproduction of the conditions at the boundaries of the model.

The model scale is the ratio between a model value and its corresponding value in the estuary under consideration. When in the model the length of a given stretch of the estuary is 10 m and in the actual estuary it is 10 km, the length scale is 1:1000. When the model depth is 10 cm and in the estuary the corresponding depth is 10 m, the depth scale is 1:100. When in the model a given velocity is 0.1 m s^{-1} and in the actual estuary it is 1.2 m s^{-1}, the velocity scale is 1:12. The time scale is 24 when one day (in reality) lasts 1 hr in the model.

In order to obtain reproduction of the flow there is a fixed relationship (the Froude relationship) between the length scale, the depth scale, the velocity scale and the time scale. When the difference in density between model seawater and model river water is equal to that in the considered estuary, this relationship reads

$$n_u = n_h^{1/2} \qquad n_t = n_l \, n_h^{-1/2} \tag{6.1}$$

where n_l : length scale
n_h : depth scale
n_u : scale for horizontal velocity
n_t : time scale.

Assume that a stretch of an estuary with a length of 20 km and a depth of 15 m has to be reproduced in a hydraulic scale model. Various scale combinations satisfy the conditions imposed by Equation 6.1. Some of these combinations are listed in Table 6.1. Scale combination 1 gives a model length which is large and therefore requires much constructional effort. Combination 2 gives a model depth which is impractible to work with. Both for combinations 1 and 2 one actual day lasts so long that much time is needed for an experiment which lasts one actual week. These drawbacks are eliminated when selecting scale combination 3.

Scale combination 3 shows the advantages of a distorted model, i.e. a model with different length- and depth scales. The model length is relatively small, the model depth is relative large, and the time needed for a given experiment is relatively short. Because of this estuary models are in most cases distorted models. In an distorted model the bottom must be rougher than in a non-distorted model with the same depth scale, in order to reproduce the depth averaged (tidal) flow correctly. This makes it necessary to adjust the model roughness until the model reproduces depth averaged (tidal) flow.

Table 6.1: Model dimensions when reproducing estuary of length L = 20 km and depth h = 15m at different scales.

n_l	n_h	n_u	n_t	L	h	day	comment
1:1	1:1	1:1	1:1	20 km	15 m	24 hr	estuary to be reproduced
1:100	1:100	1:10	1:10	200 m	15 cm	160 min	scale combination 1
1:1000	1:1000	1:31.6	1:31.6	20 m	1.5 cm	45.5 min	scale combination 2
1:1000	1:10	1:10	1:100	20 m	15 cm	14.4 min	scale combination 3

Detailed reproduction of turbulence and turbulent transports of momentum and mass is not feasible in a distorted hydraulic scale model. Nevertheless, the modelling experience is that the main features of seawater intrusion are reproduced satisfactorily, once the hydraulic scale model is properly adjusted for. In addition for the hydraulic scale model one must select the location of the model boundaries and boundary conditions. In this respect the situation is as it is for the mathematical model.

6.1.3 Mathematical modelling

Since the advent of computer, mathematical modelling of complex processes of hydraulic phenomena have become feasible. The rapid progress in computer technology over the past three decades has rendered mathematical models as popular tools not only in developed but also developing countries. In general, a mathematical model can now be adopted for a personal micro-computer without much difficulty. As mentioned in the introduction, since the study of seawater intrusion is still not a well established or documented discipline, mathematical models of this problem cannot be obtained as easily as those of flow simulation. With the efforts made by various researchers around the world, it is hoped that mathematical models of seawater intrusion into river mouths will be accessible by all interested, especially those in the developing countries.

Seawater intrusion can be described in mathematical models as one–, two- or three-dimensional processes depending on each given case. Development of a two- or three-dimensional model is normally more expensive and time-consuming than a one-dimensional model and the higher economic costs involved in the higher dimensions of modelling work cannot normally be justified by the purpose of application for development. At present, the most popular models in this field deal mainly with one-dimensional flow. In many of development studies such as in Europe, America (USA) or Asia (Mekong, Chao Phya, Hooghly, Brahmaputra), the results obtained from one-dimensional models have been quite satisfactory. It must be noted that these results were verified and applied with a good understanding of the actual processes occurring in nature. This implies that these models should be regarded merely as a tool to confirm and systematize the human understanding of the actual phenomena and to reinforce the reasoning of prediction through the extrapolation of existing data. As such, mathematical models will always be the most powerful tools for development planning, operation and management work.

The development of a mathematical model involves the development of:

- a system of equations, which describes the seawater intrusion phenomena to be studied in sufficient detail;
- a numerical code to solve this system of equations; and
- a method to include the specific features of the estuary under consideration in the computation.

Developing the system of equations involves the development of a procedure to select the flow- and mixing parameters which occur in the system of equations. A flow parameter to be described is the wall roughness. Mixing parameters include, depending on the type of modelling, those needed to describe the dispersive transport of salt or the turbulent transport of salt and momentum in mathematical terms.

Geometric parameters are to be prescribed in order to include the specific features of the estuary under consideration in the computation. These geometric parameters may include the horizontal extent and water depth of storage areas along the estuary.

Neither the whole river nor the whole sea into which the river issues is reproduced in a mathematical model. Consequently, the mathematical model has boundaries which do not exist in nature. This leads to the need to select the location of the model boundaries, and the boundary conditions, i.e. the conditions to be imposed at the boundaries of the model.

In nature seawater intrusion reacts to governing factors such as the river discharge, tidal characteristics and the salt content of the seawater.

Reproducing part of the sea into which the considered estuary issues in the model means that the tidal characteristics and salt content at the boundaries of the model sea must be prescribed. This makes it necessary to measure these parameters at sea or to determine them otherwise. When the seaward boundary of the model coincides with the coast line, the tidal characteristics at the mouth of the estuary and the salt content of the water flowing into the estuary from the sea through the mouth must be prescribed. In this case, these parameters must be measured in nature or be described otherwise.

In the development and application of a mathematical model the following stages can be distinguished: calibration, verification, and application for predictive purposes.

Calibration is the process of finding geometric-, flow- and mixing parameters which make the seawater intrusion calculated by the model coincide with that measured in the field. In this process factors such as river discharge and tidal characteristics governing the seawater intrusion during the field survey must be reproduced in the mathematical model. If not, the mathematical model does not work under the conditions prevailing during the field measurements. This means that the field data used for the calibration must include both seawater intrusion as such and the factors by which it is governed. For field data obtained under transient conditions this means that the governing factors must be known over a sufficiently long period prior to the measurements and that the variation of the governing factors over this preceding period must be represented in the mathematical model.

Verification is the process of finding rules for adopting the geometric-, flow- and mixing parameters to new conditions. In this process the seawater intrusion calculated by the model is made to coincide with that measured in the field for other conditions than used in the calibration. Also during this process the factors governing the seawater intrusion during the field surveys involved must be reproduced in the mathematical model.

The final result of the calibration and verification is a mathematical model as such plus the procedures for how to adopt the geometric-, flow- and mixing parameters to new conditions. Then the model can be used to make extrapolations, provided that one is confident that the procedure for adopting the model parameters works over a wider range of conditions than included in the calibration and verification. Another condition to be satisfied is that one knows how to extrapolate the boundary conditions.

6.2 Applicability of methods in a given case

The method to be applied in a given case depends on, as elaborated upon in Section 4.4.1, the insight to be obtained, and the information to be generated. These items determine the time scale of interest, whether interpolation or extrapolation are to be made, whether the stratification (Section 4.4.2) is a significant factor, etc.

Correlation methods are to be considered whenever interpolations may be expected to give enough information to solve the problem to be considered. It may be necessary to supplement the correlation method by mathematical modelling or hydraulic scale modelling. This situation arises when the correlations found are to much blurred by factors not accounted explicitly for in the correlations.

When extrapolation is to be carried out, the crucial question is how far one can go. Mathematical modelling or hydraulic scale modelling may be of help to answer this, but not necessarily in all circumstances.

One-dimensional mathematical modelling may be appropriate when the considered problem can be solved knowing the profile-averaged salinity only. Two-dimensional laterally averaged or three-dimensional mathematical modelling — or hydraulic scale modelling — becomes necessary when the stratification is a feature which controls the solution of the considered problem. Two-dimensional depth averaged mathematical modelling is an appropriate tool when the salinity primarily varies over the width of the estuary, and not over its depth.

One-dimensional real-time modelling is the type of mathematical modelling, which gives the variation of the profile-averaged salinity over the length of the estuary and width the stage of the tide. In accordance with Section 6.1.3 this type of modelling requires specification of: the profile-averaged salt content of the water flowing into the estuary from the sea through its mouth, and the one-dimensional real-time dispersion coefficient.

The one-dimensional dispersion coefficient relates the dispersive transport of salinity with the longitudinal gradient of the profile-averaged salinity. The dispersive transport is the transport of salinity into the estuary from the sea through a reference plane perpendicular to the axis of the estuary, moving at the profile-averaged velocity, u. The dispersive transport of salt is the factor which controls salinity intrusion. Salt would not penetrate further into the estuary from its mouth than the tidal excursion length, if the dispersive transport were zero. If so, saltwater which enters the estuary from the sea on the flood tide will return to the sea before the end of the following ebb tide. This is because, when moving at velocity u, the river flow makes the seaward displacement on the ebb tide larger than the landward displacement on the flood tide. Thus, proper selection of the magnitude of the dispersion coefficient is a crucial item in real time one-dimensional salinity intrusion modelling.

The value of the dispersion coefficient varies with the tidal characteristics (diurnal or semi-diurnal), the riverflow, the stratification, the shape of the cross-section and the location along the estuary. For equilibrium conditions it may be different from that under transient conditions. Therefore, modelling a given estuary, the dispersion coefficient to be applied must be obtained from measurements made in this specific estuary. Making measurements for this purpose, it is necessary to develop a procedure to adopt the dispersion coefficient to conditions other than those of the measurements. Doing so it is necessary to determine whether the extrapolation involved can be made with confidence.

In particular the above observations apply to time-averaged one-dimensional modelling. This type of modelling can be used to determine how the profile-averaged salinity varies over the length of the estuary from one tidal cycle to the other. Both the salt concentration and the above dispersion coefficient are to be obtained from measurements made in the estuary under consideration.

Two-dimensional depth-averaged real time mathematical modelling gives the variation of the depth-averaged salt concentration in the horizontal plane and with time over the tidal cycle. When the estuary under consideration has a pronounced bathymetry, characterized by tidal flats and a system of ebb- and flood channels, as described in Section 3.1.5, it may be so that the salt concentration is primarily controlled by advective processes (pumping, phase effect). If so, this type of modelling requires specification of the depth-averaged salt content of the water flowing into the estuary from the sea through its mouth.

If the above condition is not satisfied, there is a need to specify the dispersion coefficient due to averaging over the depth. Determining the dispersion coefficient poses similar problems as explained above in connection with the one-dimensional dispersion coefficient.

Two-dimensional laterally-averaged real time mathematical modelling gives the variation of the laterally averaged salinity over the depth, over the length of the estuary and with time over the tidal cycle. Three-dimensional real time mathematical modelling gives the variation of the salinity over the depth, over the length and width of the estuary, and with time over the tidal cycle. Both these types of model require specification of the salinity of the water flowing into the estuary from the sea through its mouth, and the turbulence model to be used.

Mathematical turbulence models can be divided into: turbulence models which require the length scale of turbulence, and the effect of stratification thereon, as empirical input; and turbulence models which contain a partial differential equation to determine the length scale of turbulence.

In the former group damping functions are used to express the effect of stratification on the length scale. However, damping functions universally valid for seawater intrusion modelling are not yet available. The latter group of models may offer an alternative, though quite involved and at present still at the stage of development insofar as application to seawater intrusion is involved.

6.3 Practical implications

Besides the aforementioned basic considerations, the choice of methods of prediction will be determined by practical implications. In this respect one has to take into account the terms and costs in making methods operational, and the way methods are meant to be used. This appliesto both the mathematical model and the hydraulic scale model.

For mathematical models the above items require a good understanding of, and insight into, the numerical-related and computer related aspects. As an example, one has to consider the development of a sophisticated three-dimensional model and its operational use in the office on a mainframe or super computer on the one hand, against the development of a simple one-dimensional model, run in the field on a personal computer on the other.

Regarding the different options the possibility of a complementary set of models, including hydraulic models, has also to be considered.

In making a final choice between the various options, the investments involved in the methods have to be compared with terms and intensity of their operational use. A short term, unique application of methods with specific aims may change into a completely different choice when one is aiming at a long-term research programme lasting several years.

7. Structural and non-structural measures

7.1 Structural measures

Various preventive and repressive measures can be taken to check or to reduce the effects of seawater intrusion in rivers or deltas.

Enclosure works
The most drastic means of checking saline intrusion consists of damming off one or more river branches of a delta near the outfalls with enclosing dams, some equipped with drainage sluices. Such works, however, are mostly carried out in the first place to achieve flood protection for a large area. Combatting seawater intrusion is thus something of an extra, secondary goal.

Work of this nature entails a complete change of the environmental and ecological conditions. The enclosure works carried out in the Netherlands and in Japan serve as examples.

Flow diversion
Saline intrusion can be reduced by increasing the upland discharge through the diversion of water from one river branch to another or by release from upstream reservoirs. Such diversion can be achieved by means of a structure in the uplands or near the apex of a delta, e.g. a barrier dam with movable gates. By doing so it is, however, likely that seawater intrusion will increase in those river branches from which the discharge is being diverted. This disadvantage can be overcome by an enclosure dam near the sea, as already mentioned. In this manner freshwater (lowland- or coastal) reservoirs come into being.

River bottom heightening
Seawater intrusion can, under certain circumstances, also be reduced by heightening the river bottom along a certain distance. This method is only effective in places where bottom velocities are low and, at the same time, where the situation is more or less stratified. This undeepening of the river must be extended along the reach of the seawater intrusion.

In the Rotterdam Waterway and Nieuwe Maas bottom heightening has been executed by means of dumping coarse gravel in a regular way. This gravel layer protects the original fine sand bottom against scouring and stops the process of retreating erosion helping in this way to halt the seawater intrusion.

Attention has to be given to the interests of the present navigation and any future developments. An economic analysis of trade-off has to be made to give insight into the possibilities of the measure.

Harbours
The presence of harbours (tidal storage areas) attached to the main river affects the mixing process and so the seawater intrusion in the river. The location of projected new harbours along the river has to be investigated very carefully, because seawater intrusion may be decreased or increased as a result. The rate of siltation of the harbour is also dependent upon its location on the river.

Breakwaters, flow guidance dams
Extension of a river mouth into the sea, naturally by coastal accreation or structurally by dams or (impermeable) breakwaters will shift the saltwedge over about the same direction seaward. Dams or breakwaters however will be carried out for navigational purposes, because accretion, swell and waves or cross-currents hamper safe navigation and in this case reducing seawater intrusion is secondary.

Groynes
Measures which increase the mixing process can also reduce seawater intrusion. This can be done by creating extra roughness along the banks and/or the bottom, e.g. by means of underwater groynes.

These groynes have to be built in the reach where the steepest gradients in density occur in order to be effective for mixing. The saltwedge toe, however, moves to and from during the tidal cycle and its position is also dependent on upland discharge. So the extra roughness must be present along a certain length of the river and the location of groynes is of importance with respect to their salt-combatting abilities. Moreover, the initial degree of stratification of the estuary is relevant. In partly mixed and stratified estuaries groynes will be most effective in the downstream parts of the estuary. If this reach is very long, which is mostly the case in tropical rivers, the cost will be very high and construction of groynes solely for reducing seawater intrusion will not be cost-effective!

A movable pneumatic air bubble screen can also intensify mixing, but the high costs are here also a restraint.

7.1.1 Examples of structural measures

Schemes similar to the Zuiderzee works and the Delta works in the Netherlands have been implemented in many countries and a few cases are mentioned here below. In all cases one of the main objectives was to halt seawater intrusion, along with the shortening of the line of defence against storm surges, typhoons and cyclones. A distinction can be made between schemes *with* a freshwater reservoir (enclosed gulfs, lagoons, wide estuaries) and schemes *without* substantial storage of freshwater.

The Netherlands see Fig. 7.1
reservoir type: Zuiderzee works, Braakman, Lauwerszee
estuary type: Delta Works.

France
estuary type: Vilaine (near Nantes)

Japan
reservoir type: Kojima Bay, Hachiro Gata
estuary type: Kiso river, Tone river

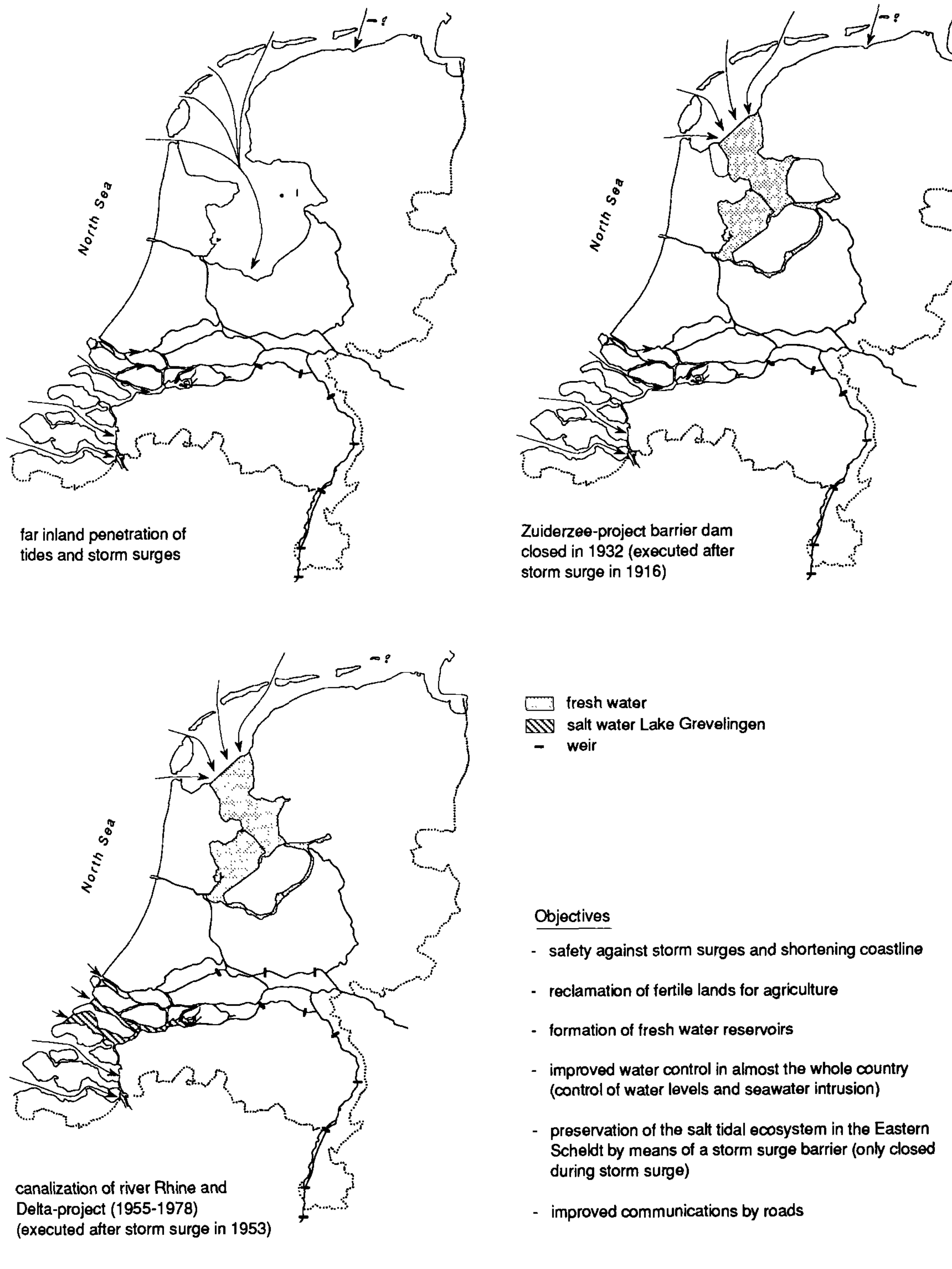


Fig. 7.1. Major structural schemes in the Netherlands

India
reservoir type: Vembanad Lake (Kerala State)

Viet Nam
canal type: Tiep Nhot (Mekong Delta)

7.2 Non-structural measures

The extraction of freshwater can be regulated by legal provisions to achieve an optimal freshwater allocation among different users, but also to reduce freshwater use with the additional effect of less seawater intrusion. In Section 7.1 the technical and managerial measures, which affect the water supply to the water users, are discussed. Legal measures, however, affect water demand and pricing and regulation measures are to be mentioned. Such measures can be: a tax on freshwater intake or on a structure/equipment for water extraction; a subsidy on water saving methods/ equipment; or a licence that restrict water extraction. It will be obvious that the subjects mentioned in this Section can only be exercised by a specific administration responsible for the local or regional water management. An analysis of possible pricing and regulation measures has to be done in which also the technical and managerial measures are included, to find what measures may be beneficial and in what situation. Pricing and regulation measures need not to be imposed permanently but can be used in particular circumstances, e.g. when water demand exceeds water supply. Controlling withdrawals or structures requires a good deal of administration. It will be clear that the benefits of pricing and regulation measures should exceed by far these administration costs.

7.3 Final remark

It will be clear that reduction of seawater intrusion is often difficult and mostly very expensive. Cheaper alternatives for land use and water supply can sometimes be found, being independent of local conditions in the bordering rivers, e.g. water supply via a canal from upstream river reaches.

All above-mentioned measures have to be considered in a more extended and integrated way and this can be covered by a policy analysis for the water management of a whole delta or river basin.

Global warming due to carbon dioxide emission (the greenhouse effect) will result in a worldwide need for improved defences against rising sea level and will result also in an increasing seawater intrusion into rivers and deltas.

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8. Case studies

8.1 Large-scale mixing processes in a partly mixed estuary

Delft Hydraulics Communication No. 371 November 1986 by G. Abraham *et al.*

This paper presents experimental evidence for the considerable effect of the tidal phase difference between two estuary channels on the longitudinal density distribution in the Rotterdam Waterway, a partly mixed estuary. It further shows the effect of harbour basins along the estuary on the salt intrusion to be caused by two mechanisms which in the Rotterdam Waterway are counteracting: the effect of basins on the tidal flow and the temporary storage of saltwater in the basins during part of the tidal cycle.

1. Introduction

This paper deals with the salt intrusion in the Rotterdam Waterway, a partly mixed estuary. It describes large-scale advective mixing processes induced by the large-scale geometry of the estuary.

1. One large-scale mixing process considered is the tidal phase difference between two estuary channels. The tidal phase difference induces advective processes which have a considerable effect on the longitudinal salt distribution. Because of the tidal phase difference and because of a different channel geometry, both estuary channels react in a different manner to variations of the tidal amplitude. In the one estuary channel the salt intrusion at high water slack decreases with increasing tidal amplitude, while in the other channel the smallest intrusion at high water slack occurs with normal tide.
2. Another large-scale mixing process considered is the effect of harbour basins along the estuary. The basins lead to two mechanisms which in the Rotterdam Waterway are counteracting. The tide and density induced exchange flows cause saltwater to be temporarily stored in the basins during part of the tidal cycle. This temporary storage tends to increase salt intrusion. Consequently with harbour basins turbulent mixing is stronger in the main estuary channels than it would be without. This effect tends to reduce salt intrusion. The combined effect of the considered large scale processes depends on the integrated effect of small scale turbulent mixing. The implications of this finding for Rotterdam Waterway salt intrusion modelling are given.

2. General characteristics of Rotterdam Waterway

The Rotterdam Waterway estuary, which is represented in Fig. 8.1.1, is formed by the New Meuse, the Old Meuse and the New Waterway. Freshwater, which flows into the estuary through the New Meuse and the Old Meuse, issues into the North Sea through the New Waterway.

In the Rotterdam Waterway the salt intrusion is influenced by large-scale and small-scale transport processes. On a large scale these processes are primarily advective ones, and include tidal action, freshwater discharge and gravitational circulation. On a small scale these processes are primarily turbulent ones, and include the turbulent transport of momentum and mass. Both the large-scale and the small-scale processes depend on the geometry of the estuary. The combined effect of the large-scale processes depends on the integrated effect of the small-scale turbulent transports. The junction of the New Meuse and the Old Meuse is an important feature of the estuary. There is a phase difference between the tidal velocities in the New Meuse and those in the Old Meuse. This phase difference acts as a large scale mixing mechanism.

Several harbour basins are located along the estuary, and are openly connected to it; the Europoort harbour basins connect with the New Waterway at 1033 km, the Botlek harbour basins connect with the New Waterway at 1014 km, and several harbour basins connect with the New Meuse between 1001 km and 1012 km (see Fig. 8.1.1 and Table 8.1.1).

The basins lead to phase effects. In addition, density currents of the lock exchange flowtype (Schijf and Schönfeld, 1953) cause relatively saltwater to flow into the harbour basins on the flood tide, and out of the basins on the ebb tide. Both the phase effects and the density induced exchange flows cause large-scale mixing to take place in the main estuary channels.

For conditions considered in the paper, high water slack salt intrusion reaches to about 995 km in the New Meuse as well as in the Old Meuse, in each channel with respect to its own coordinate system (see Fig. 8.1.1).

Fig. 8.1.1 and Table 8.1.2 give the dimensions of the channels where salt intrusion takes place. For the New Meuse distinction has to be made between a deep part from 1013 km (the junction with the Old Meuse) to 1001 km, and a shallow part upstream from 1001 km. Table 8.1.2 lists geometric and hydrodynamic characteristics of the separate channels, making a distinction between the deep part and the shallow part of the New Meuse. For the separate channels Table 8.1.2 lists the following quantities: the cross-section below mean sea level (A), the width (B), the depth below mean sea level (h), the freshwater flow rate in the conditions of small discharges which are considered in this study (Q_f), the freshwater velocity ($u_f = Q_f/A$), the maximum flood discharge (Q_0) for respectively spring tide, normal tide and neap tide, and the corresponding maximum flood velocity ($U_0 = Q_0/A$).

The freshwater flow rate as well as the tidal velocities in the Old Meuse are larger than they are in the New Meuse, while the cross-section of the Old Meuse is smaller than that of the New Meuse. Because of these different characteristics, the salt intrudes further into the New Meuse than it does into the Old Meuse, measured from the junction of these rivers. The tidal flow supplies the energy which causes the mixing between fresh and saltwater. The energy input by the tide per unit time and per unit area is propertional to the product of the bottom shear and the tidal velocity, i.e. to the tidal velocity cubed.

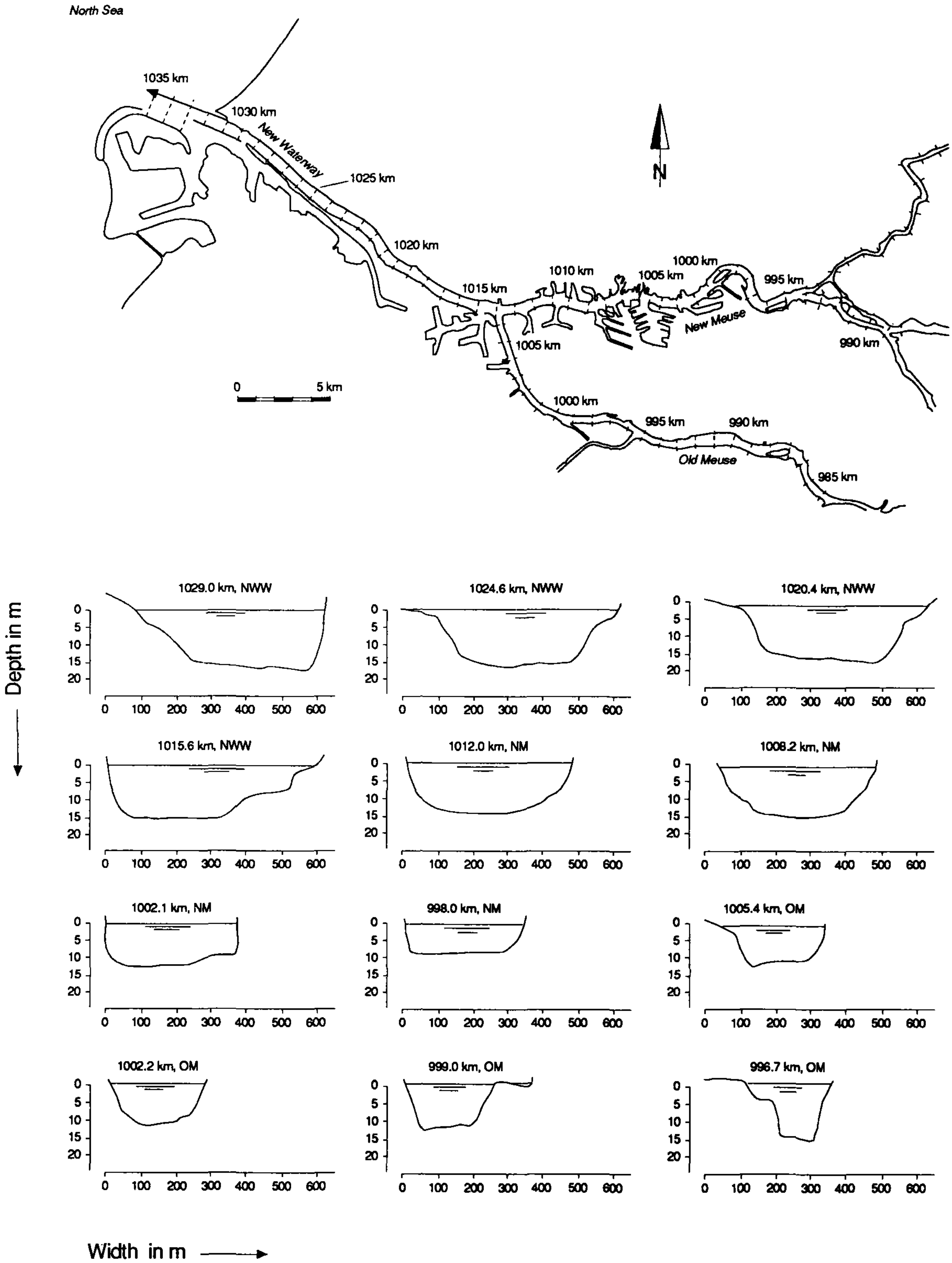

Fig. 8.1.1 Rotterdam Waterway estuary

Table 8.1.1: Dimensions of harbour basins

Harbour basin	Surface area (m^2)		Location (km)
Botlek Harbour	3.6	10^6	1014
Waal Harbour	2.2	10^6	1005
Eem Harbour	1.5	10^6	1008
1st Petrol Harbour	0.5	10^6	1011
Meuse Harbour	0.6	10^6	1003
2nd Petrol Harbour	0.6	10^6	1010
Σ small harbours	3.0	10^6	1002-1012
	12.0	10^6	

Table 8.1.2: Geometric and hydrodynamic characteristics of Rotterdam Waterway (discharge of Rhine 800 m^3/s)

Existing situation									
Channel location (km)		New Waterway		New Meuse		New Meuse		Old Meuse	
		1029	1015	1011	1002	999	995	1005	996
A (m^2)		6000	5600	5500	4200	3000	2700	2700	2300
B (m)		450	450	490	390	380	370	300	260
h (m)		13.5	12.5	11.5	10.5	8	7.5	9	9
Q_f (m^3/s)		640	640	210	210	210	210	430	430
U_f (m/s)		0.11	0.11	0.04	0.05	0.07	0.08	0.16	0.19
Q_0 (m^3/s)	spring tide[1)]	9000	8200	5600	3430	3120	2900	3000	3090
	normal tide	7450	7160	4980	2830	2480	2380	2450	2560
	neap tide	5200	4800	3300	2005	1870	1800	1860	1910
U_0 (m/s)	spring tide	1.50	1.46	1.02	0.82	1.04	1.07	1.11	1.34
	normal tide	1.26	1.28	0.91	0.67	0.83	0.88	0.91	1.11
	neap tide	0.87	0.86	0.60	0.48	0.62	0.67	0.67	0.83
E_{local}	spring tide	100%	100%	100%	100%	100%	100%	100%	100%
	normal tide	59%	67%	71%	55%	51%	56%	55%	57%
	neap tide	20%	20%	20%	20%	21%	25%	22%	24%
$E_{overall}$	spring tide	100%	92%	31%	16%	33%	36%	41%	71%
	normal tide	59%	62%	22%	9%	17%	20%	22%	41%
	neap tide	20%	19%	6%	3%	7%	9%	9%	17%

1) Δh = 2.0 m spring tide Δh: difference between HW and LW and mouth of estuary
= 1.6 m normal tide
= 1.2 m neap tide

Table 8.1.2 gives the following dimensionless parameters as a measure for the energy available to cause mixing

$$E_{local} = \frac{U^3_{0,local}}{(U^3_{0,local})_{spring}} \tag{8.1.1}$$

$$E_{overall} = \frac{U^3_{0,local}}{(U^3_{0,1029})_{spring}} \tag{8.1.2}$$

where the indices local, 1029 and spring refer to respectively the local value of u_0, the value at 1029 km and the value during spring tide. The parameter E_{local} is a measure for the energy available at a given location for different tidal conditions, where the local spring tide value is set at 100%. The parameter $E_{overall}$ is a measure for the energy available at different locations along the estuary, where the spring tide value at 1029 km is set at 100%.

The values of E_{local} pertaining during a spring tide are substantially larger than the values pertaining during a neap tide. Consequently the estuary is less stratified during a spring tide than it is during a neap tide (see Fig. 8.1.3). The estuary number E_D, is a measure of the stratification (Thatcher and Harleman, 1981). It is defined as

$$E_D = \frac{P_t}{Q_f T} \frac{U_0^2}{\frac{\Delta\rho}{\rho} gh} \sim \frac{\rho B U_0^3}{Q_f \Delta\rho g} \text{ (at km 1029)} \tag{8.1.3}$$

where

$\Delta\rho$: density difference between river and seawater
ρ : density of either river or seawater
P_t: volume of seawater entering the estuary on the flood tide
T : duration of tidal cycle
g : gravitational acceleration

For the conditions of small freshwater flow rates considered in this paper the estuary number ranges from 3 (for spring tide conditions) to 0.6 (for neap tide conditions). Estuary numbers of this order of magnitude refer to partly mixed conditions.

In Equation (3) Q_f is a measure of the quantity of freshwater to be mixed in order to obtain mixed conditions, $\Delta\rho$ g is a measure of the force per unit volume of saltwater counteracting the mixing, while $\rho B U_0^3$ is a measure of the energy supplied by the tide. As it is the latter energy which causes the mixing, E_D is a measure of the stratification.

3. Available data

Information on salt intrusion in the Rotterdam Waterway estuary has been derived from several series of field measurements. In addition hydraulic scale model studies and mathematical model studies have been made. A review of current Rotterdam Waterway research is given by Roelfzema *et al.* (1984).

Field measurements, covering a whole tidal cycle of salt concentrations and velocities have been made on several occasions. Rather complete data sets have been obtained for different conditions with respect to the tide and the freshwater flow. The data sets show how the measured quantities vary with time, in the longitudinal direction and over the cross-section. Analysing the available field data by means of the decomposition method (Fischer, 1972), it has been found that in the Rotterdam Waterway the gravitational circulation is primarily in the longitudinal direction (Winter-Werp, 1983) and that both turbulence generated at the solid boundaries and turbulence arising in the interior are to be taken into account (Abraham, 1980). Dronkers (1969) derived the effect of density gradients on the bottom shear from the field measurements.

A hydraulic scale model of the Rotterdam Waterway estuary (vertical scale 1:64, horizontal scale 1:640, distortion 10) has been available since 1965. A survey of the type of engineering studies performed in the model is given by van Rees *et al.* (1972). The hydraulic scale model gives a fair simulation of the physical processes which control salt intrusion, including gravitational circulation and turbulent mixing. This has been demonstrated by comparing the performance of the model with several available field data sets (Breusers and van Os, 1981; van der Heijden *et al.*, 1984). As this stage has not yet been reached in mathematical modelling, predictions to do with salt intrusion under new conditions are primarily derived from the hydraulic scale model.

To obtain basic knowledge on the physical processes related to salt intrusion, flume studies were carried out to study the intrusion in a schematized estuary of rectangular cross-section (see for example Roelfzema and van Os, 1978). In these studies the flume was used as a distorted hydraulic scale model of the schematized estuary.

From the flume studies, dimensionless correlations of the salt intrusion length with the determining quantities have been derived (Rigter, 1973). These correlations may not be applied to the Rotterdam Waterway as they do not incorporate the effect of the junction of the New Meuse and the Old Meuse, the different characteristics of these rivers, and the effect of the harbour basins. In addition, groynes occur along part of the banks of the Rotterdam Waterway. This makes the mixing characteristics of the Rotterdam Waterway different from those of the flume, while in the flume studies salt intrusion was found to vary with the type of flow resistance (bottom roughness, side wall roughness or strips) i.e. with the mixing characteristics (Abraham *et al.,*, 1975).

In one-dimensional, real time mathematical modelling of salt intrusion the effects of stratification and gravitational circulation have to be incorporated into the dispersion coefficient. The correlation between the dispersion coefficient and determining conditions is a unique characteristic of an individual estuary (Abraham *et al.*, 1975, Fischer *et al.*, 1979, Section 7.1). For the Rotterdam Waterway no satisfactory correlation has been found as yet. Given this limitation of one-dimensional modelling, at least a two-dimensional laterally averaged (2DV) model is necessary to reproduce the gravitational circulation in the vertical plane through the axis of the estuary, and its effect on salt intrusion.

Laterally averaged two-dimensional modelling studies have been made by Perrels and Karelse (1981) to reproduce salt intrusion in the schematized estuary of rectangular cross-section. At present, their modelling technique is being applied to the Rotterdam Waterway. Tentative laterally averaged two-dimensional modelling studies of salt intrusion in the Rotterdam Waterway have been made by Hamilton (1975) and Smith and Dyer (1979).

Given the above considerations the information on the vertical and longitudinal density distribution presented in this paper is derived from the hydraulic scale model for different tidal conditions, with the harbour basins openly connected with the Rotterdam Waterway as well as closed.

4. Effect of junction on salt intrusion

Through the New Meuse and the Old Meuse the tide penetrates into different networks of channels and wide basins. This causes phase differences between the tidal velocities in the New Meuse and in the Old Meuse. In the New Meuse the flood tide and the ebb tide begin earlier than in the New Waterway; in the New Waterway these tides begin earlier than in the Old Meuse. The above phase differences act in various ways as large-scale advective mixing mechanisms, causing contacts between waters having a different salt content. The advective processes are the most pronounced when tidal excursion paths are large, and when salt intrusion is beyond the junction. The former condition is satisfied on a spring tide, the latter condition when the freshwater flow rates are small.

Fig. 8.1.2 shows time histories of density at different stations in the vicinity of the junction. It illustrates some consequences of the phase differences. Late on the flood tide (from 14 hours until 15 hours) the salt content of the water, which arrives at the junction from the sea has its peak value. At this phase of the tide, all water reaching the junction from the sea flows into the Old Meuse, not into the New Meuse, where a weak ebb flow starts to develop. This explains why in the New Meuse (1010.1 km) the density remains constant, while in the Old Meuse (1005.8 km) it continues to rise.

Early on the ebb tide (from 15 hours until 16 hours) all water which passes the junction to flow into the New Waterway comes from the New Meuse, where the salt content is relatively small. Later on (from 16 hours) it also comes from the Old Meuse, where the salt content is relatively large.

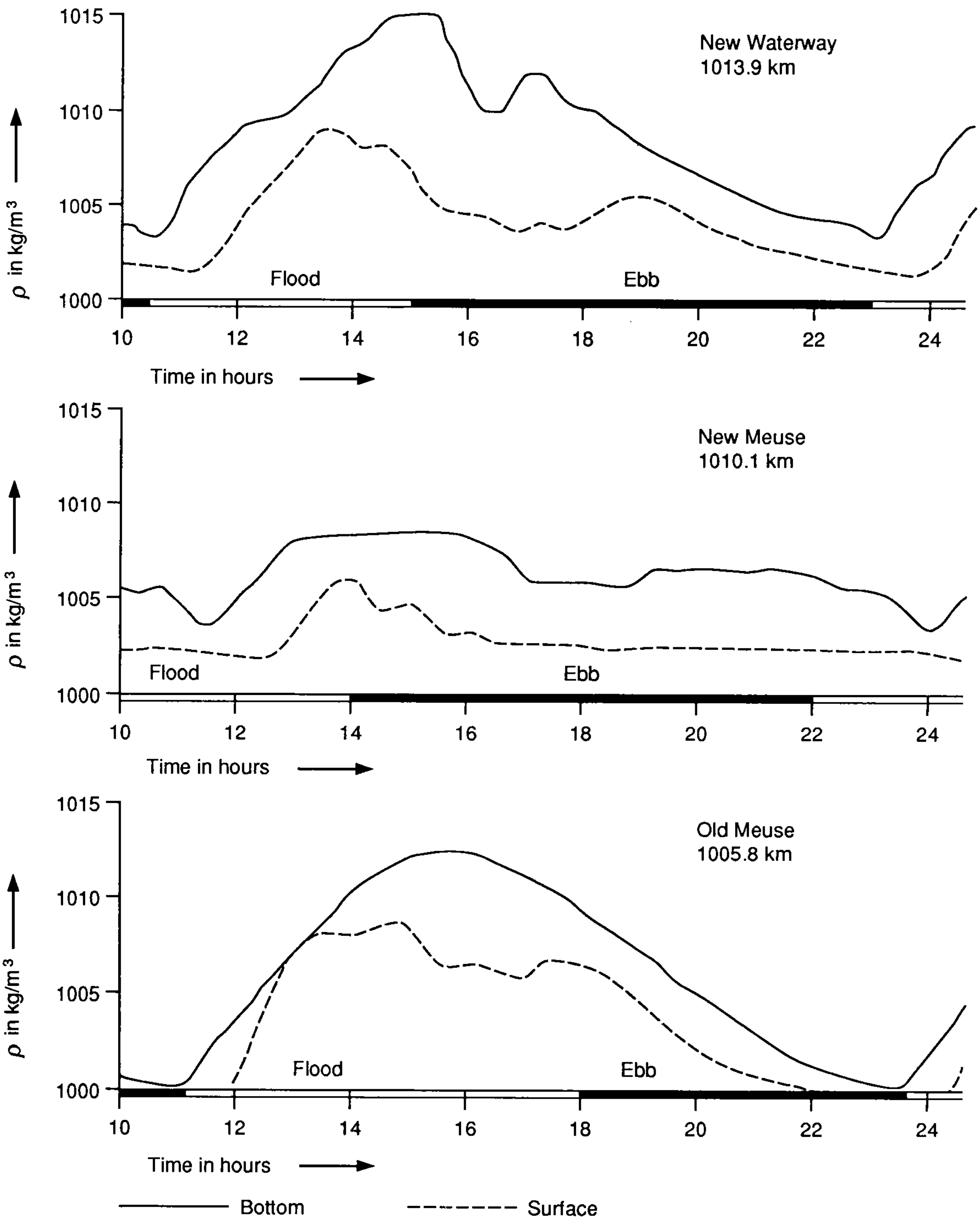

Fig. 8.1.2 Time history of density at junction (from hydraulic scale model)

This explains why in the New Waterway (1013.9 km) the time history of the density exhibits a peak at about 17 hours.

The above phenomena have a significant effect on the longitudinal density distribution in both the New Meuse and the Old Meuse.

5. Effect of tidal amplitude on salt intrusion

Large tidal amplitudes at the mouth of the estuary are associated with strong tidal currents, and hence with strong turbulent mixing and weak stratification. The stronger the turbulent mixing, the smaller salt intrusion tends to be. Because of this effect salt intrusion tends to become smaller with increasing tidal amplitude, in particular when the estuary is stratified.

Large tidal amplitudes at the mouth of the estuary are further associated with large tidal excursion paths. The larger the tidal excursion path, the larger salt intrusion tends to be at high water slack. Because of this effect the salt intrusion tends to become larger with increasing tidal amplitude, in particular when the estuary is mixed. Which of the above counteracting mechanisms prevails varies with the stratification, i.e. with the strength of the tide. For the schematized estuary this is clearly demonstrated by the fact that salt intrusion at high water slack decreases with increasing tidal amplitude when the latter is relatively small, while salt intrusion at high water slack increases with increasing tidal amplitude when the latter is relatively large. For intermediate tidal amplitudes salt intrusion at high water slack is found to have its smallest magnitude (Rigter, 1973, Fig. 8.1.2).

The above counteracting mechanisms also play a role in the Rotterdam Waterway, though modified by the phase effects at the junction. This has been found in the hydraulic scale model, comparing vertical distributions of density measured at high water slack at different locations along the estuary. Fig. 8.1.3 shows vertical distributions of high water slack density for a spring tide, normal tide and neap tide with Δh respectively 2.0, 1.6 and 1.2 m, where Δh represents the difference the mouth of the estuary. The corresponding tidal characteristics are given in Table 8.1.2.

For the New Meuse, Fig. 8.1.3 shows that salt intrusion at high water slack increases with decreasing tidal amplitude. For a normal tide it is larger than it is for a spring tide, while for a neap tide it is larger than it is for a normal tide. For the Old Meuse Fig. 8.1.3 shows the smallest intrusion at high water slack occurring with a normal tide. This difference between the New Meuse and the Old Meuse is due that late on the flood tide there is only inflow of water from the New Waterway into the Old Meuse (see Fig. 8.1.2).

6. Effect of harbour basins on salt intrusion

6.1 Exchange of water between estuary channel and harbour basin

In a harbour basin which is located along and in open connection with an estuary water is stored outside the estuary channel over part of the tidal cycle. The mechanisms behind this temporary storage are tidal filling and emptying, density-induced exchange flows of the lock exchange type, and to some extent eddies with a vertical axis induced by the river flow in the entrance of the basin.

Tidal filling and emptying are associated with the variation of depth within a tidal cycle. Assuming that the water surface remains level and neglecting variations of velocity with depth, the velocity through the entrance of the basin, induced by the tidal filling, v, satisfies:

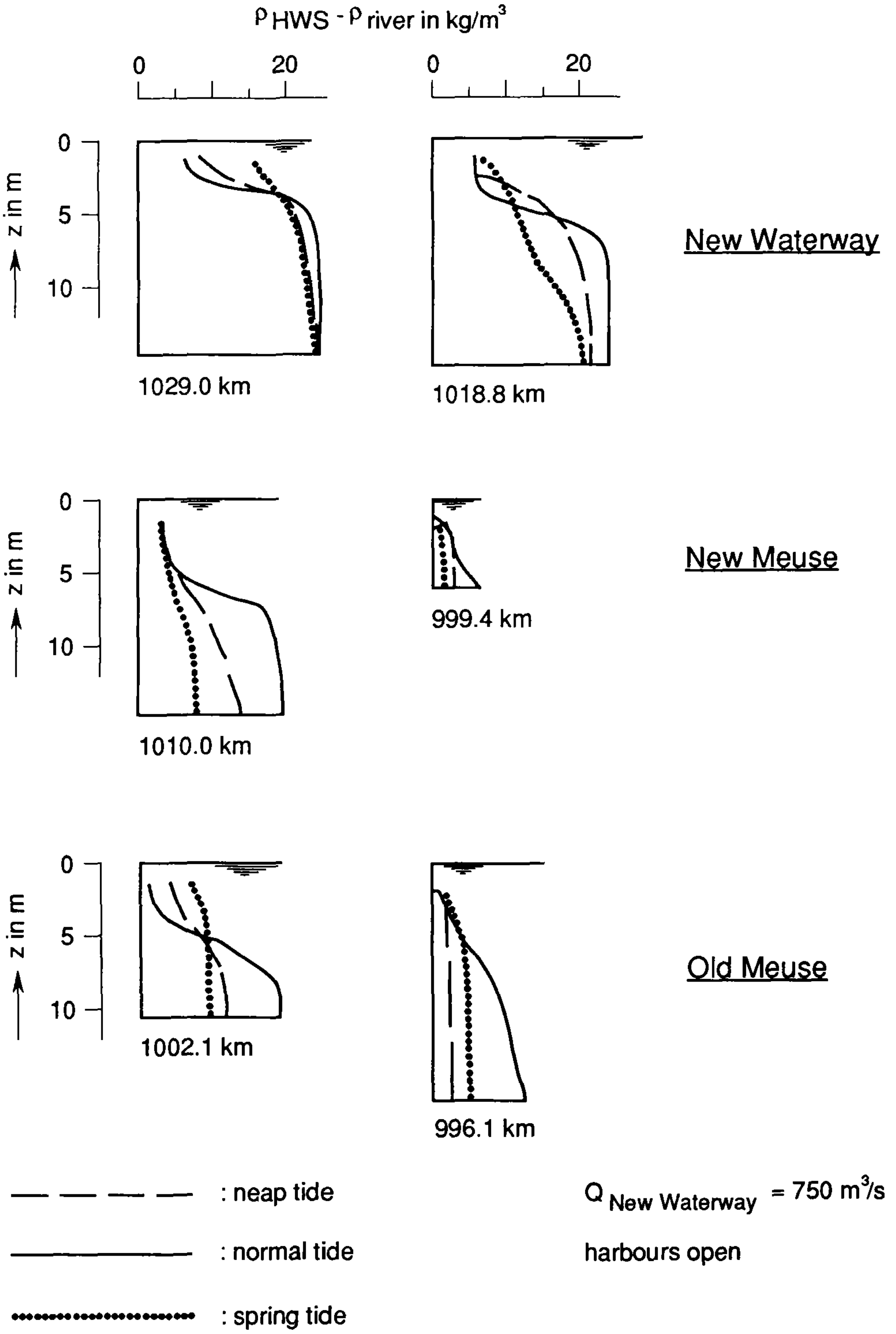

Fig. 8.1.3 Vertical high water slack density distributions comparing conditions with neap tide, normal tide and spring tide (from hydraulic scale model)

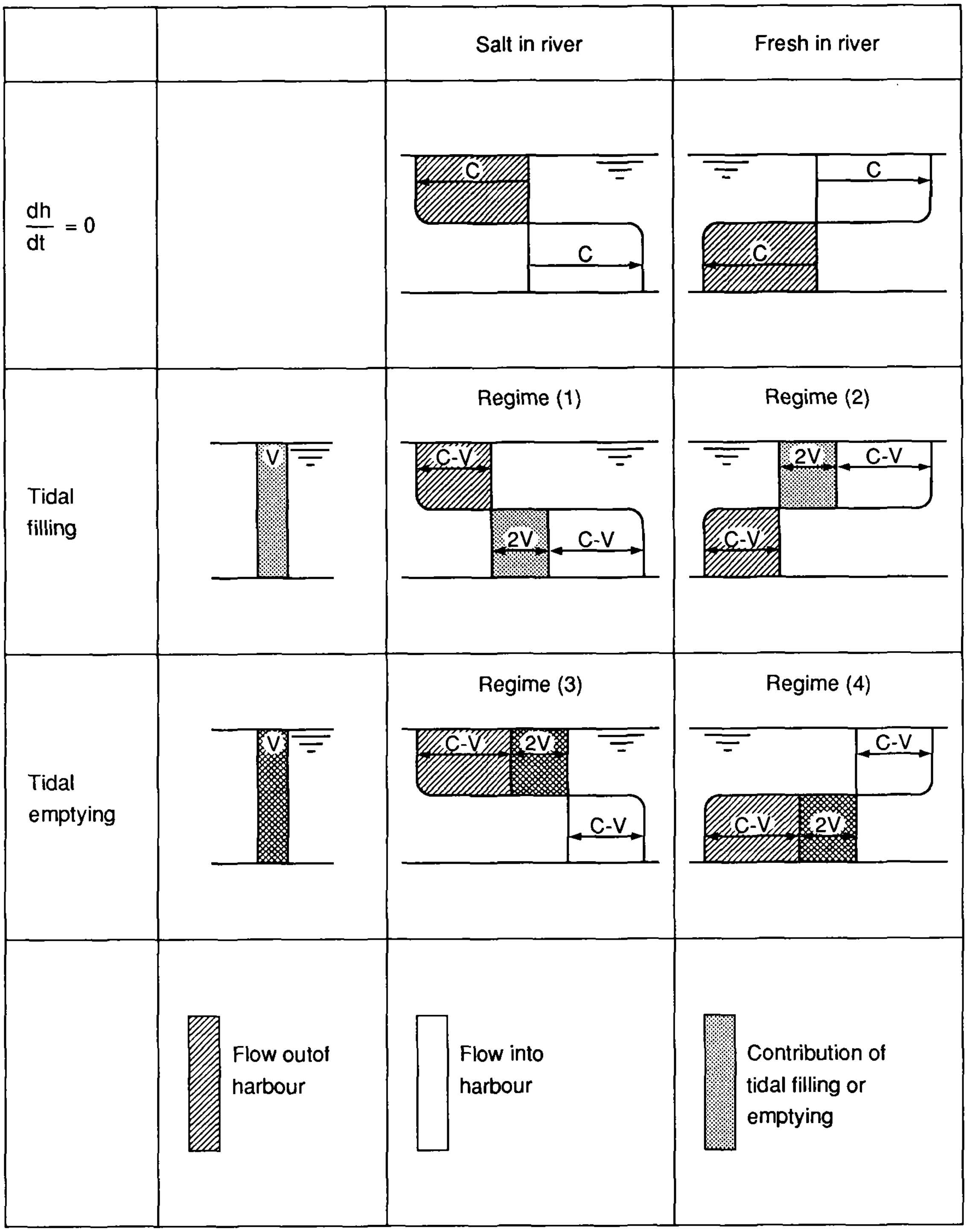

Fig. 8.1.4 Combined effect of tidal filling and density induced exchange flows

$$v = \frac{A\frac{dh}{dt}}{Bh} , \tag{5}$$

where A is the horizontal area of the harbour basin, and where B and h are the width and depth of the entrance of the basin. The density of the water in the estuary channel varies with time. When increasing it is larger than the density of the water in the harbour basin, when decreasing it is smaller than the density of the water in the basin. This is because the mean density of the water in the basin can only increase if water from the basin is replaced by water with a comparatively large or small density from the channel by density induced exchance flows of the lock exchange type.

When v = 0, the density-induced velocity of inflow and/or outflow, c, is in first approximation given by Schijf and Schönfeld (1953).

$$c = 1/2 \left(\frac{\Delta\rho}{\rho} gh\right)^{1/2} , \tag{8.1.6}$$

where $\Delta\rho$ is the driving difference in density between the harbour basin and the estuary channel.

Tidal filling and emptying causes the density induced exchange currents to be reduced to, in first approximation,

$$C_n = c - |v| , \tag{8.1.7}$$

where C_n is the magnitude of the net density induced exchange velocity. Fig 8.1.4 shows the rationale behind Equation (8.1.7).

For Botlek Harbour, Fig. 8.1.5 shows the relative significance of tidal filling and emptying (in the figure referred to as $\delta h/\delta t$), density induced exchange flows (in the figure referred to as $\Delta\rho$) and the exchange induced by eddies in the entrance of this harbour basin (in the figure referred to as eddy). In Fig. 8.1.5 the four regimes distinguished in Fig. 8.1.4, can be recognized. Fig. 8.1.5 was obtained from field measurements.

Further details on the exchange between harbour basins and estuary channels are given by Allen and Price (1959), Vollmers (1976) and Roelfzema and Van Os (1978).

6.2 Effect of harbour basins on salt intrusion mechanisms

Harbour basins influence salt intrusion in two ways:

Firstly, because of the above exchange mechanisms there is a temporary storage of relatively saltwater in the basins on the flood tide. Therefore, they act as a source of relatively saltwater at the end of the ebb tide (see Fig. 8.1.5). Because of this effect closing the basins — which is feasible in the hydraulic scale model only — would tend to reduce the salt intrusion.

Secondly, the total area of the basins is so large that tidal action is much stronger with the basins than it would be without. This applies from the area where the basins are located towards the sea. Tables 8.1.2 and 8.1.3 quantitatively illustrate the above observation. These tables give the tidal characteristics of the separate channels, respectively when the harbour basins listed in Table 8.1.1 are in open connection with the estuary and supposed to be closed. In both tables the parameter $E_{overall}$ is related to its spring tide value at 1029 km, the harbour basins being in open connection with the estuary. This value is set at 100%. From the zone occupied by these harbours towards the sea, spring tide conditions without the basins, which are listed in Table 8.1.1 (i.e. when these basins were closed) are about the same as normal tide conditions with these basins. This can be seen from

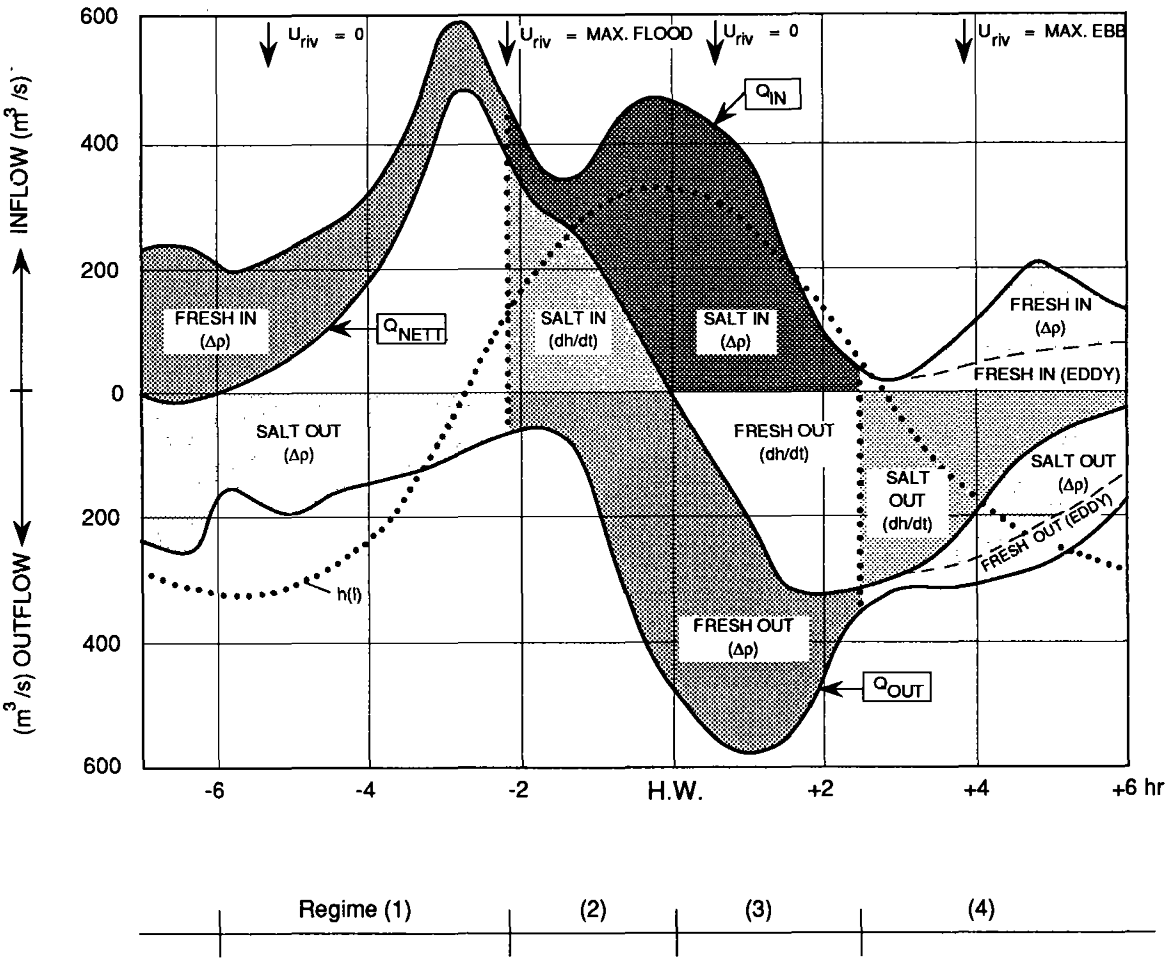

Fig. 8.1.5 Flow through mouth of Botlek Harbour (from field measurements)

the $E_{overall}$ values listed in Tables 8.1.2 and 8.1.3 for 1029 km, 1015 km and 1011 km. The above reduction of tidal action, induced by closing the harbour basins, tends to increase salt intrusion into the New Waterway and into the New Meuse, and hence to increase the salt content at the junction. This follows directly from the information given in Chapter 5. In the Old Meuse the tidal excursion paths do not change when the harbour basins are closed. Therefore, the increase of salt content at the junction would lead to an increase of salt intrusion into the Old Meuse. Hence, both for the New Meuse and the Old Meuse the modification of the tidal action induced by closing harbour basins tends to increase salt intrusion.

Table 8.1.3: Geometric and hydrodynamic characteristics of Rotterdam Waterway (discharge of Rhine 800 m³/s)

Without the harbour basins listed in Table 8.1.1									
Channel location (km)		New Waterway		New Meuse		New Meuse		Old Meuse	
		1029	1015	1011	1002	999	995	1005	996
A (m^2)		6000	5600	5500	4200	3000	2700	2700	2300
B (m)		450	450	490	390	380	370	300	260
h (m)		13.5	12.5	11.5	10.5	8	7.5	9	9
Q_f (m^3/s)		640	640	210	210	210	210	430	430
U_f (m/s)		0.11	0.11	0.04	0.05	0.07	0.08	0.16	0.19
Q_0 (m^3/s)	spring tide 1)	7900	7000	4200	3365	3010	2900	3000	3090
	normal tide	6220	5780	3725	2765	2510	2415	2440	2450
	neap tide	4135	3845	2570	1940	1845	1770	1700	1940
U_0 (m/s)	spring tide	1.32	1.27	0.76	0.80	1.01	10.7	1.11	1.30
	normal tide	1.04	1.03	0.68	0.66	0.84	0.89	0.90	1.07
	neap tide	0.69	0.69	0.47	0.46	0.62	0.66	0.63	0.84
$\overline{E}_{local}$	spring tide	100%	100%	100%	100%	100%	100%	100%	100%
	normal tide	49%	81%	72%	56%	58%	58%	53%	56%
	neap tide	14%	16%	24%	19%	23%	23%	18%	27%
$\overline{E}_{overall}$	spring tide	68%	61%	13%	15%	31%	36%	41%	65%
	normal tide	33%	32%	9%	9%	18%	21%	22%	36%
	neap tide	10%	10%	3%	3%	7%	9%	7%	18%

1) Δh = 2.0 m spring tide
= 1.6 m normal tide
= 1.2 m neap tide

Δh: difference between HW and LW and mouth of estuary

Fig. 8.1.6 shows vertical distributions of high water slack density for a spring tide, both when all harbour basins are in open connection with the estuary, and when the basins listed in Table 8.1.1 are closed. Fig. 8.1.6 has been obtained from the hydraulic scale model. Fig. 8.16 shows that closing the basins would increase salt intrusion in the New Meuse and reduce salt intrusion in the Old Meuse. It appears that for the New Meuse, the effect of reducing tidal action is stronger than that of eliminating the temporary storage of saltwater, in particular between 1002 km and 1012 km, where the harbours are located and the reduction of tidal action is effectuated. For the Old Meuse (996.1 km) it appears that the effect of eliminating the temporary storage of saltwater in the Botlek harbour at 1014 km is stronger than that of reducing the tidal action.

7. Conclusions

The salt intrusion in the Rotterdam Waterway is influenced by the junction of the New Meuse and the Old Meuse and by the harbour basins. The junction and the basins represent unique large scale geometric features of the estuary.

The effect of the tide on salt intrusion is determined by two counteracting mechanisms: the effect of the tide on turbulent mixing and stratification, and the effect of tidal amplitude on the tidal excursion path. For the Rotterdam Waterway a third mechanism has to be added: the phase differences at the junction. The effect of the harbour basins on salt intrusion depends on two mechanisms, which in the Rotterdam Waterway were found to be counteracting: the temporary storage of saltwater in the basins and their effect on the tide. Through their effect on the tide the harbour basins influence turbulent mixing and the stratification of the estuary.

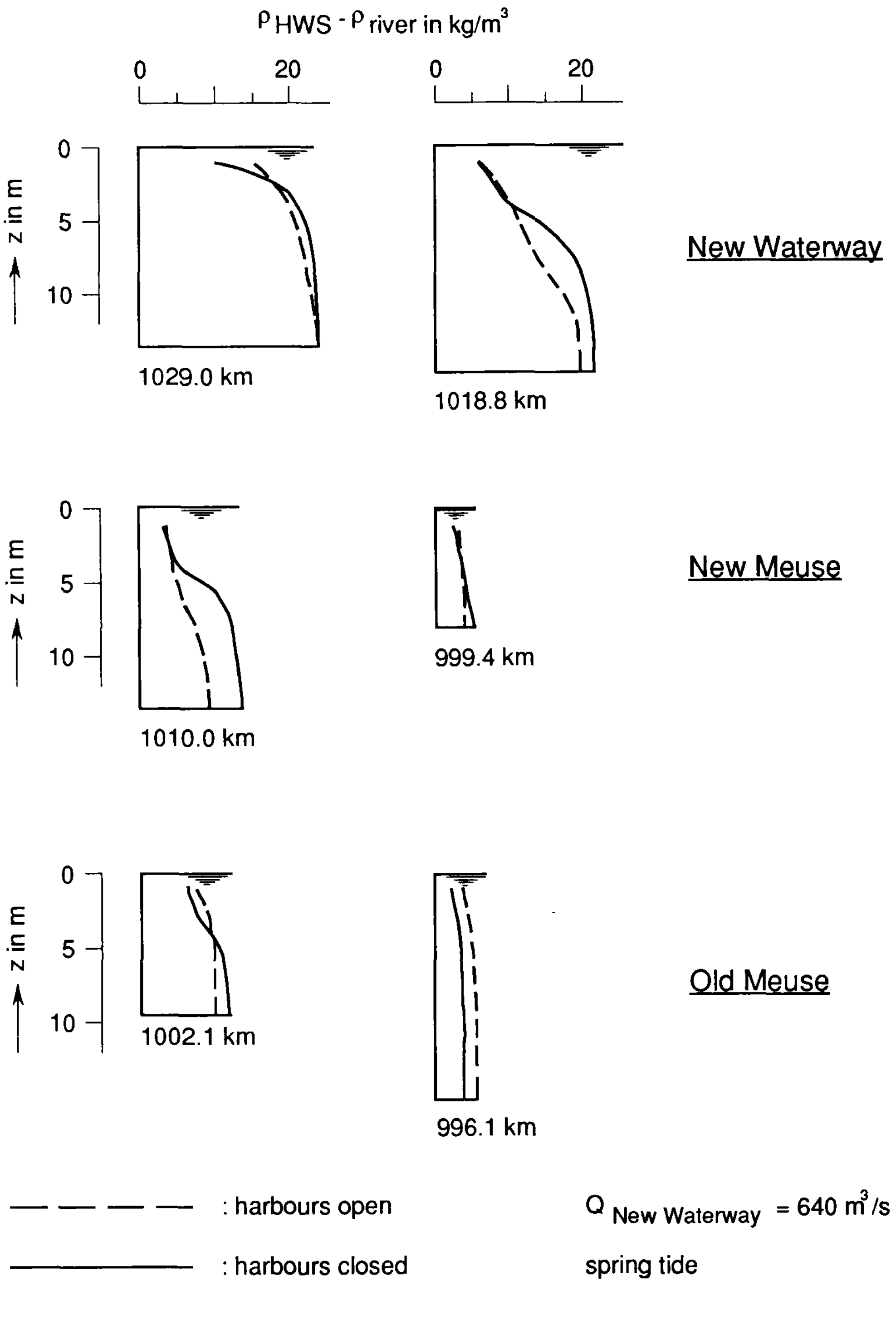

Fig. 8.1.6 Vertical high water slack density distributions, comparing conditions with harbour basins open and closed (from hydraulic scale model)

From a modelling perspective, the above findings imply that the combined effect of the acting large-scale processes depends on the combined effect of the small-scale turbulent transports. Therefore, the small-scale turbulent mixing and the resulting stratification have to be reproduced with sufficient accuracy in salt intrusion modelling including the vertical dimension, whether by a hydraulic scale model or by a laterally averaged two-dimensional (2DV) or a three-dimensional (3D) mathematical. This applies to the Rotterdam Waterway the depth of which varies little over the width and where gravitational circulation is primarily in the longitudinal direction.

Thus far, a satisfactory performance could only be demonstrated for the hydraulic scale model, comparing its performance with several available field data sets (Breusers and Van Os, 1984, van der Heyden *et al.*, 1984). Therefore, given their stage of development, the vertical density distributions supporting this paper were derived from the hydraulic scale model rather than from a 2DV or 3D Rotterdam Waterway mathematical model.

Acknowledgement

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8.2 Developing a mathematical model system for the Rhine-Meuse Estuary

Delft Hydraulics Laboratory and Ministry of Transport and Public Works, Publication No. 384
November 1984 by A. Roelfzema *et al.*

The paper deals with the development of a mathematical model system consisting of flow models, morphological models and water quality models. In such a development automation related aspects and organizational aspects play an essential role. Experiences and recommendations on these aspects are presented.

1. Introduction

At Delft Hydraulics a mathematical system for the Rhine-Meuse estuary (see Fig. 8.2.1a) has been developed for the Ministry of Transport & Public Works, the Netherlands.

The aim of this development is to provide the Ministry, which is responsible for the management of the estuary, with tools to study problems in the field of tidal salt/fresh flows, morphology and water quality. The mathematical models will have to replace the existing hydraulic scale model of the Rhine-Meuse estuary, in which systematic studies were and still are performed on the density induced tidal flows and mixing processes for a wide range of physical and geometrical conditions.

The paper describes difficulties and uncertainties in developing such a mathematical model system with respect to hardware and software and the rapid evolutions in the scope of estuarine management issues, emphasizing more and more the importance of morphological and water quality problems.

2. Physical and numerical aspects

2.1 *Initial set of models*

An inventory of actual and potential problems was made to decide by which set of mathematical models the Rhine-Meuse estuary was to be modelled. The relevant physical processes were analysed and the abilities of numerical models were considered in the light of the foregoing analysis and the accuracy and reliability demanded by the commissioner to solve his practical problems (see Roelfzema *et al.*, 1984). The result was the choice in 1983 for a mathematical model system, consisting of a 3-dimensional, a 2-dimensional width averaged and a 1-dimensional, cross-sectional averaged model, (see Fig. 8.2.1b.). At the internal boundaries the models should be coupled by special interface modules.

Major uncertainties in the development concerned the following: the availability of a reliable, accurate and predictive turbulence model for stratified tidal circumstances, and the availability of adequate coupling modules.

The need for a special 'Unification' project for automation aspects was felt directly at the start, although not in its ultimate form and size (see Section 3).

Together this led to a model system as described in Section 2.2, consisting of the following subjects:

I: Unification project: automation aspects of pre- and postprocessing, programming and documentation (total effort 3.5 man-year).

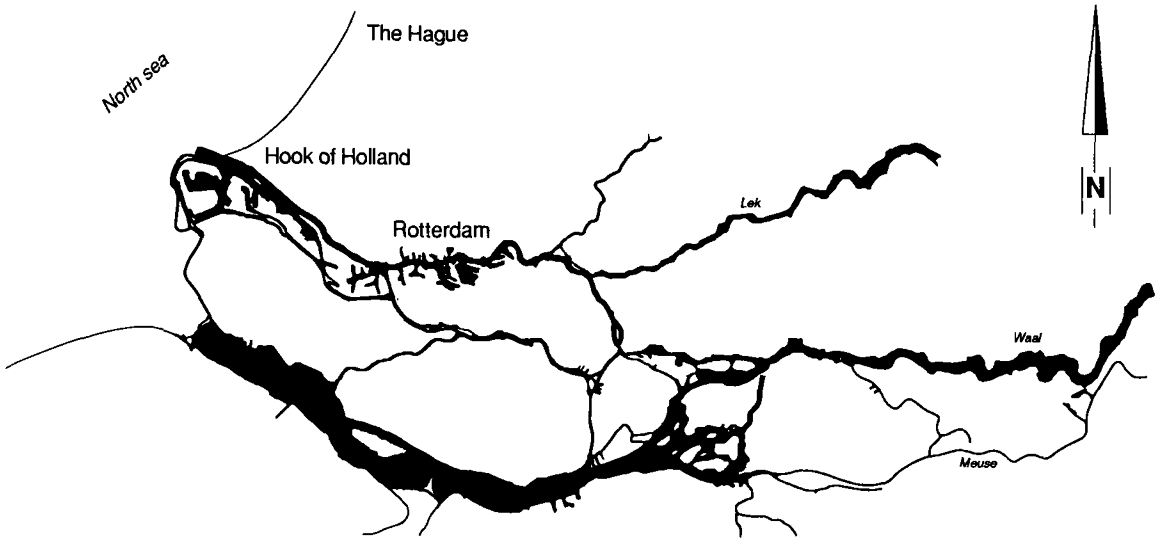
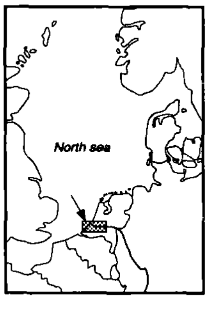


Fig. 8.2.1a Plan view Rhine-Meuse estuary



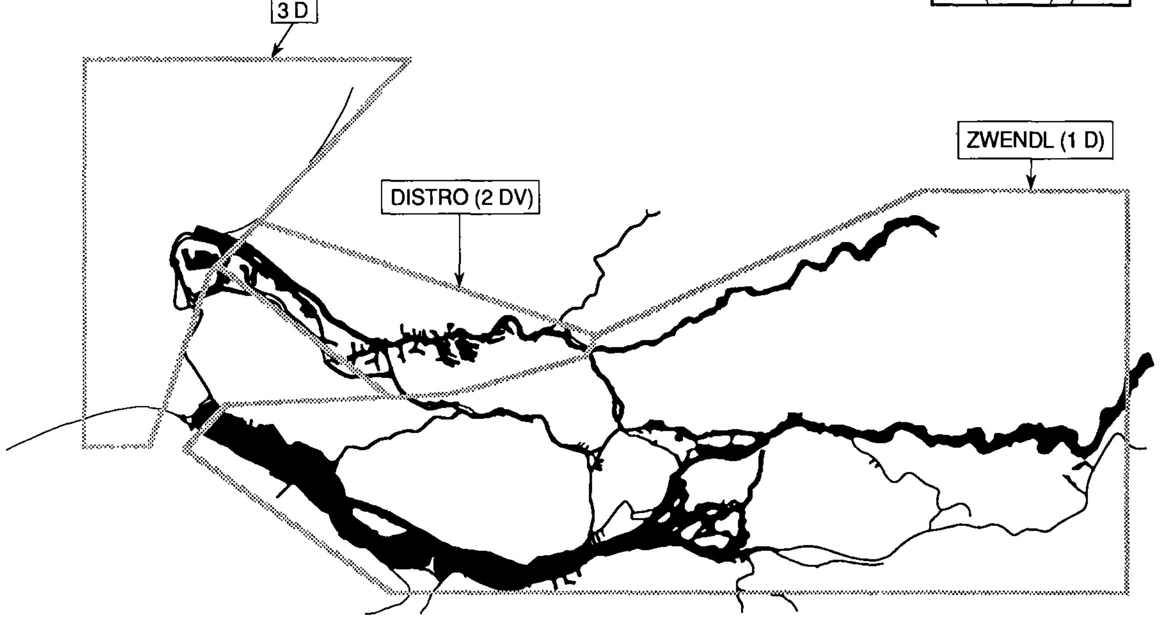


Fig. 8.2.1b Initial set up base model, 1983

II: 1-D model ZWENDL: calibration and verification of dispersion formulation on redundant field data (1.5 man-year).
III: $2D_v$-model DISTRO: standardization, calibration and verification (9.0 man-year).
IV: 3D-model TRISULA: standardization, calibration and verification (11.0 man-year).
V: coupling ZWENDL-DISTRO (2.0 man-year).
VI: coupling DISTRO-TRISULA (3.0 man-year).
VII: coupling with morphology and water quality models (3.0 man-year).
The total extent of these subjects in view of personal and financial capacities made spreading of activities not only desirable but even necessary.

2.2 Experiences up to 1987

In the past three years developments have been started over a broad range. Up to the present the following experiences with respect to physics and numerics have been obtained (the Unification project will be presented in Section 3).

2.2.1 1D-model

To bridge the period before the $2D_v$-model DISTRO was to be operational and to obtain a relative simple operational tool alongside more sophisticated ones, attempts were made to improve the 1D-model ZWENDL for the tidal salt wedge part of the estuary system.

Various formulations and parameter settings for the dispersion coefficient have been tested. In fact two problems had to be solved: firstly the formulation and next the values of the parameters. The first problem was approached on the basis of physical knowledge. Literature furnished several suggestions for dispersion coefficients related to physical quantities. The second problem was tackled with an inverse model. The results were not very satisfactory. The approach of the first problem, which was more or less a judicious trial and error, lacked a standard by which formulations could be compared. With respect to the second problem it was found that an inverse method is a poor one for extracting optimal values from data when they contain errors or are incomplete and inconsistent.

More recently a new attempt was started using optimal control theory. The results are very promising, both with respect to the identification problem and to the calibration problem (see Fig. 8.2.2).

2.2.2 $2D_v$-model

After intensive calibration and verification against laboratory data the DISTRO model has been applied to field situations in both the Eastern Scheldt and the Rhine-Meuse estuary. DISTRO operates on a flexible grid, using a time dependent transformation to follow the free surface elevations and irregular bottom topographies. One of the major experiences was that the discretisation of the transformations which proved to work quite well in regular geometries as tidal flumes, turned out to be quite inaccurate for varying geometries, as met in field situations. In fact the numerical discretisation of the transformation had to be changed rather drastically to get an accurate, fully conservative scheme for the transport equations. The main conclusion is that a box method is preferable to assure conservation.

From earlier tests with laboratory data it was learned that the mixing length concept for modeling turbulent exchange processes had limited predictive abilities. Therefore a k-epsilon model was also implemented in the DISTRO program. Tests in a homogeneous situation with tidal amplitude brought up questions in specifying the seaward boundary condition for the k- and

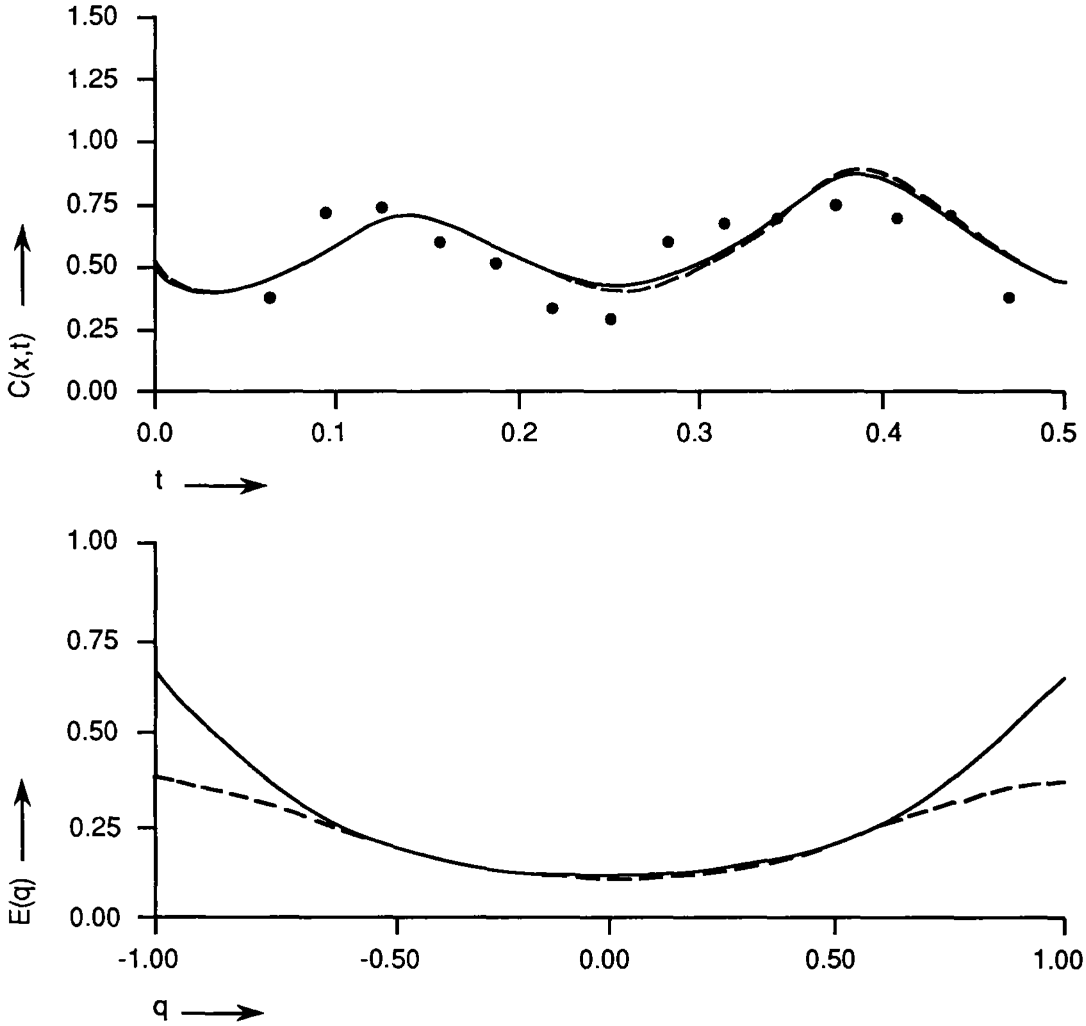


Fig. 8.2.2
A numerical simulation has been performed of a 1-D advection–dispersion equation with a dispersion coefficient E that depends on the discharge q(x, t).

After this simulation the dispersion coefficient was 'forgotten' but could be reconstructed on the basis of a moderate number of noisy point observations of the concentration.

The lower figure shows the identified form (- - -) of E(q) together with its true form. Due to noise and few data at its extrema the estimation of E(q) is less accurate for arguments q close to these extrema. Yet the estimation of the concentration profiles is accurate. See the upper figure that compares

(i) an observed noisy profile (• -points),
(ii) the estimated form of the profile (- - - - -)
(iii) the true form of the profile (———).

epsilon equations, particularly around low water slack. Simply moving the boundary far away from the domain of interest is not sufficient as for a 2 D_v approach of an estuary, the seaward boundary has to be 2 D_v too.

For the inhomogeneous (salt/fresh) situation the treatment of the seaward boundary by a 'Thatcher-Harleman like approach' seems to be obvious.

2.2.3 3D model

The 3D modelling of the outflow area of the Rotterdam Waterway into the North Sea was initially based on a program of the Rand Corporation, developed by Leendertse (1973–1979).

The first release of the 3D model for this area in 1985/86 made use of a 2D vertically averaged pilot model simulating the sea area of the hydraulic scale model. Lay-out, bottom schematization and boundary conditions for the 3D model were derived from this $2D_h$ model. The extension to a 3D model included the application of four layers, using the shallow water equations (hydrostatic assumptions), while density differences were not yet taken into account. The bottom and surface layers have a variable thickness, exchange of mass and momentum between the layers was expressed in simple mixing length assumptions.

The model had a rectangular staggered grid, while an explicit numerical technique in the horizontal direction was applied; only for the vertical diffusion and momentum transfer processes were implicit techniques adopted.

The main purpose of these 3D activities has been to gain experience in the 3D modelling of this area both on numerical and physical aspects. Comparison of 3D model results with the 2D pilot model showed a satisfying consistency. These activities are the basis for further development: incorporating the density differences into the equations, adoption of more sophisticated expressions for exchange of mass and momentum, such as the k-epsilon model and extending the model area in order to provide a sufficient basis for sediment-transport and water quality models.

2.2.4 Coupling of various flow models

The original idea was to couple various models by interface modules, starting with the coupling $1D$-$2D_v$. Experience with coupling existing 1D and $2D_h$ models showed, however, that this probably was not a wise approach (Stelling, 1981). Coupling models of a different dimension and moreover with different numerical schemes and different internal organization brings many problems. The higher order model is described by a set of characteristics different from the lower order one. Coupling implies internal boundaries with all kinds of unwanted reflections. So first of all the coupling must preserve the correct physical behaviour in terms of characteristics.

With flow models of different dimensions it was common practice to develop them independently and not with the idea that some time they had to be coupled. The result is that most models have been equipped with a discretization which is optimal for their class of applications but not necessarily with respect to coupled problems. And finally most programs are organised to some optimum for internal grid generation which may be rather different from one model to another. Considering all the potential problems and developments in numerical techniques it was concluded that coupling of existing, independently developed models was not a wise thing to do. Instead it was recommended to build a fully three dimensional model, organised in such a way that lower-order models could be obtained by degeneration of the 3D model. This approach has as extra benefit that a better connection can be made with the already existing water quality model DELWAQ, as will be explained in Section 2.2.6.

2.2.5 *Coupling of morphological and flow models*

Coupling of flow and morphological models brings along some special problems which will not be met with other transport phenomena such as salt intrusion.
First of all there is an exchange process with the bed, which, depending on flow characteristics, functions as a source or sink of sediment. Secondly there is not one type of sediment involved but a range of grain sizes with varying properties. This becomes apparent in different settling velocities, yield stresses and sorting phenomena and complicates the modelling of bifurcations. In particular, the modelling of silt transport processes in tidal salt/-fresh situations, important from both a morphological and water quality point of view, is still in an early state.

Next there is the problem of the influence of a changing morphology on the flow pattern. Significant bed level changes should be accounted for in the flow field.

And last but not least there is the problem of different time scales such as the interaction of a flow problem with a typical time scale of one day and a morphological process with a time scale which may rather be measured in years or even longer periods. An obvious alternative seems to be the use of time averaged models; however, experiments with residual current models showed that there was no general approach for obtaining the residual transports from the residual currents. This led to the conclusion that residual transports and resultant bedlevel variations had to be computed for some characteristic periods, which should be composed to predict morphological behaviour over longer periods.

2.2.6 *Coupling of water quality and flow models*

The coupling of water quality and flow models differs in essential points from the coupling with morphology, although both are described as transport phenomena. Water quality does not effect the bathymetry or the density and can therefore be computed from a given flow pattern without interaction. By contrast with other constituents, water quality parameters may influence each other significantly, demanding simultaneous solution of the transport equation for several constituents including their interaction. The transport of water quality parameters can also be highly correlated with the silt transport. A typical timescale for water quality parameters is the seasonal timescale with periods in the order of a year. On the other hand, water quality calamities may occur, and must therefore be treated with timescales of hours. Both extremes result in different requirements for the flow model.

The present water quality program DELWAQ has the flexibility of computing a domain of interest with 3D as well as lower dimensional parts. Present flow models do not have this flexibility yet, which is very attractive when handling strongly varying bathymetries. In 3D development, however, such an option is provided presently.

Another special aspect of water quality modelling is that a 3D water quality model does not necessarily demand the details of a 3D flow model. There are several occasions where a 1D flow, which is assumed homogeneous over a cross-section, will satisfy.

3. Automation (related) aspects

3.1 *General considerations*

In considering the selected models, including the morphological and water quality models it was realised that an unambiguous way of data-handling and presentation of results was necessary to obtain a coherent model system.

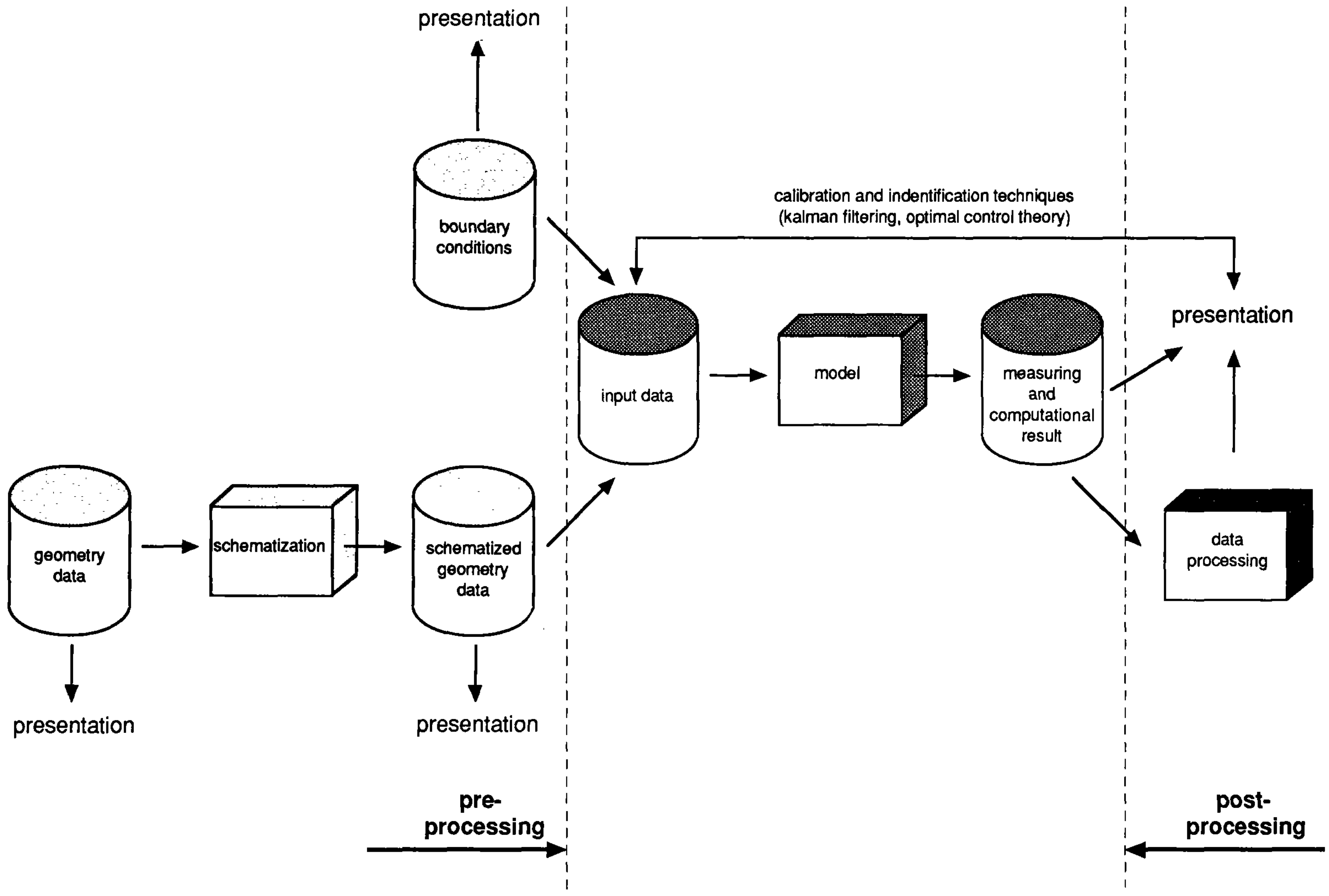


Fig. 8.2.3 Data flow around a model

At the start of the project the state of the models involved was quite different. Each of the models only could be used by the specialists involved in the development of the model. Besides the existing hydraulic scale model of the Rhine-Meuse estuary and observations from nature are of course important in the development process.

An analysis of the data-flow around a model is given in Fig. 8.2.3. The set of elements to the left of the block 'model' is called the preprocessing part, while everything on the right side of the block 'model' belongs to the postprocessing part. The back coupling between the postprocessing part and each of the elements of the preprocessing part, is especially meant for improving mathematical formulation and calibration of the model. This whole data-flow pattern is in principle the same for each model.

A closer look at the data-flow shows what should be done to get proper results from a simulation run of a model.

Pre-processing activities are:

- administration
- lay-out of the model (selecting coordinate system, defining grid points and how to connect them)
- time frame (simulation period, time step)
- boundaries (selecting type of boundary, boundary data)
- schematization of geometry
- defining values for all kind of parameters (global = applicable for all grid points e.g. viscosity, wind stress), local (=applicable for individual grid points e.g. roughness coefficient, dispersion coefficient).

Postprocessing activities are:

- control of output of the simulation run (choice of data-types, selection of grid points, choice of start time, end time and time interval for writing results to result database)
- processing of results (time series analysis, mathematical integration and differentiation, statistics)
- presentation of all possible data in both preprocessing and postprocessing part.

As far as time frame is concerned it should be noted that the length of the simulation period can vary from hours to a number of years. Sometimes only a part of the simulation period is used in a run. Restart of a simulation run from the point where it was stopped is included.

The present solutions to perform all the activities mentioned and their method of automation are quite different. As illustrated in Section 2, this is due to the seperate development of the models. Although from a historical point of view this may be well understood it has led to the use of:

- different numerical techniques,
- different operating methods,
- different software tools, and
- different computer systems, for the various models, with, as a consequence large problems with couplings and presentations.

Regarding this, the problems which have to be solved to get to a coherent system, can be summarized as follows:

- unification and standardization of databases, data processing, presentation facilities, treatment of boundaries, automated schematization procedures and calibration techniques;
- integration of the different models, concerning the operating methods and use of computer systems;
- combining the different demands of developers and end-users on the use and flexibility of the separate models and the system;

- incorporating the development of specifications caused by continuing developments for the separate models and by the increasing number and type of models involved in the project;
- the organization of the developments, because the models involved have their origin in different (parts of the) organisations for example from Delft Hydraulics and the Ministry of Transport and Public Works;
- uncertain future caused by developments in numerical techniques and very fast evolution of automation technology;
- the development and operational use of the separate models should continue on the one hand, while progress in the longer term path to the system must be made visible on the other.

3.2 *Present experiences*

In 1983 it was hardly realised what it meant to obtain a system of models due to the mentioned problems and the state of the models. The effort needed to make a coherent system of the different models was therefore clearly underestimated. At present the way to get to the system is much better understood. An approach is necessary that is very different from just developing seperate mathematical models. A sound understanding of information engineering, software engineering and also of organization is required. Risks are involved, because one is dependent upon developments in computer configurations, data communication facilities and commercially available software. Further characteristics can be mentioned: the large number of persons involved in the use of modern software tools and the use of (still) conventional methods for developing the system.

Concerning the use of computer systems, it has been decided to aim at distributed data-handling. This means that the real simulation runs will be performed on a mainframe computer and in future eventually on a super computer, super micro or a dedicated system. The pre- and postprocessing will take place on a mini computer in combination with personal computers.

The integration of the pre- and postprocessing of the different models is regarded as the most important factor in getting to a coherent system. An analysis of pre- and postprocessing has shown that many functions are common for all the models. However, there exist also functions that are very specific for one model. Based on this analysis, an architecture of the system has been designed that offers the desired flexibility and integration.

It has been decided to start with the presentation facilities and then work backwards to the shown data-flow around the models, Fig. 8.2.3. This choice is based upon:

- concentration on one part of the system at a time,
- the possibility to make progress visible in the path to the system, and
- the fast evolution of the automation technology.

Furthermore, the 2-dimensional laterally averaged model DISTRO, the 2-dimensional vertically averaged pilot model and the hydraulic scale model of the Rhine-Meuse estuary have been used as a test case to show progress and results. This test case shows that also the integration of a mathematical and a physical model is in principle not different from integration of different mathematical models (see Fig. 8.2.4).

It has been decided to buy software tools rather than to build them in house. The latter is possible when functionality and performance are sufficient. The task is then to build the necessary applications with that tool. The consequence of this policy is that one becomes (quite) dependent on the vendor.

As a presentation facility the system PRESENT (with colour facilities) has been built based on the commercially available Uniras package (see Fig. 8.2.4). The system is based on the philosophy that the way of presenting results is very much problem-dependent. The users have near-freedom

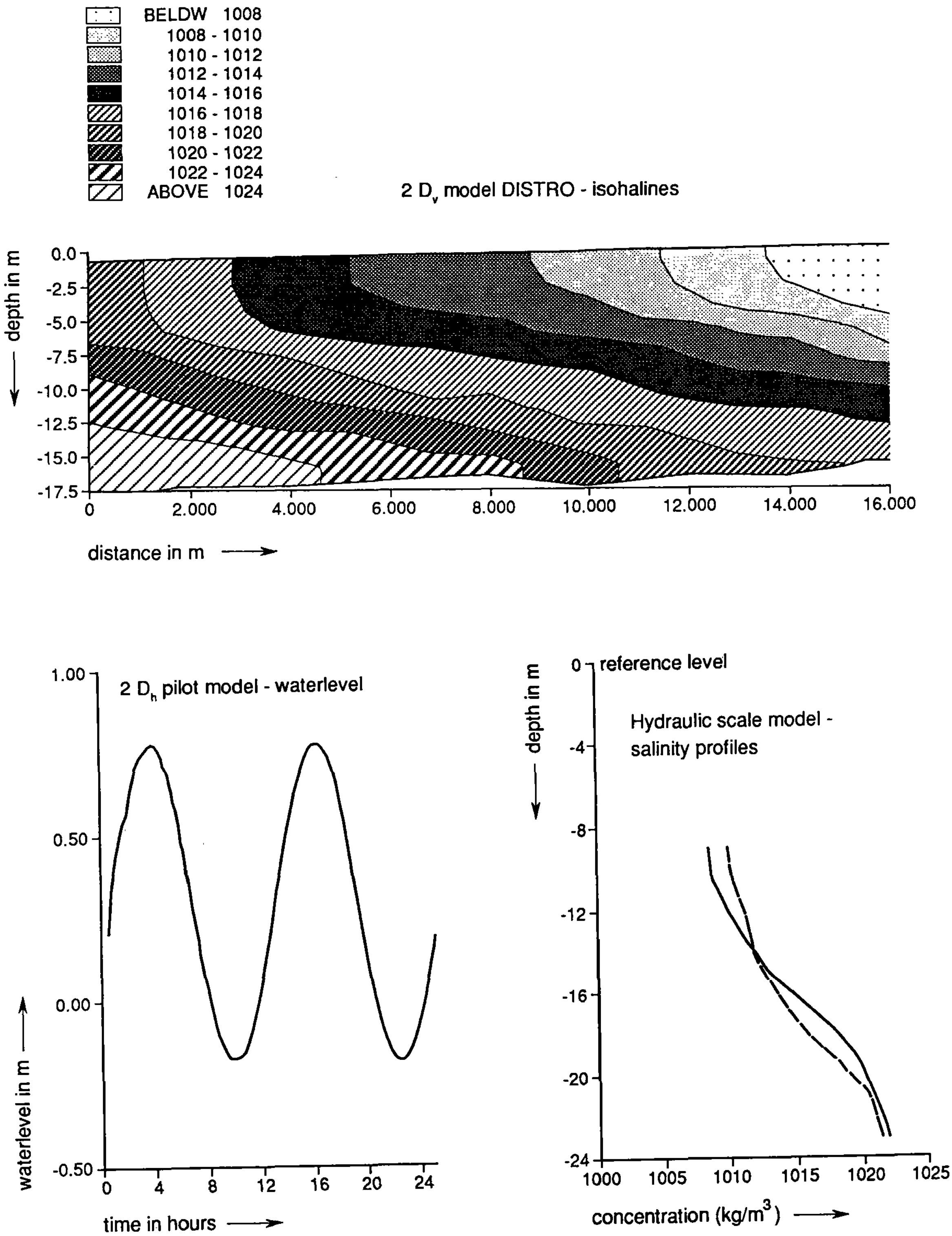


Fig. 8.2.4 Examples of postprocessing

in defining the lay-out of their presentations. It is possible to use the system both in an interactive and a batch-oriented way. The interactive part is meant for experimental presentation and supports 1-, 2- and 3-dimensions, while colours add extra dimensions. The batch-oriented part is meant for the standard presentations (1- and 2-dimensions) like time-series and contouring. The standardization can be reached by agreeing on the lay-out of the presentation. The developers are offered a maximum flexibility. The wishes of the end-user can be fulfilled by building a layer on top of the present user-interface which will decrease the flexibility somewhat, however. The problem of the presentation of 3-dimensional time-dependent results has not yet been solved completely.

It has been decided to use the commercially available relational database management system MIMER as a facility for storing data. The heaviest application seems to be the storage of measuring and computational results. This concerns up to millions of data for every simulation run. In a pilot-project it has been shown that an acceptable performance concerning the storage and retrieval of data can be reached. The use of a relational database management means for the user that knowledge about the structure of the data is no longer necessary while storing the data. This is replaced by powerful tools for data-retrieval whereby much profit is gained by a faster building of new applications. The database management system is now in the phase of implementation. This concerns amongst others a direct coupling to the models and the presentation facility. The mathematical models are built highly around the calculation process. An important question to be answered is the influence of the direct coupling of these calculation processes on databases which are unavoidable for performing pre- and postprocessing.

4. Discussion on organizational aspects

A fundamental choice has to be made between organizational possibilities. The three extremes are: developing models according to their dimensions, so 1-dimensional models for flows, sedimentation/erosion and water quality and the same for 2- and 3-dimensional models. Each model has its own pre- and postprocessing facilities:

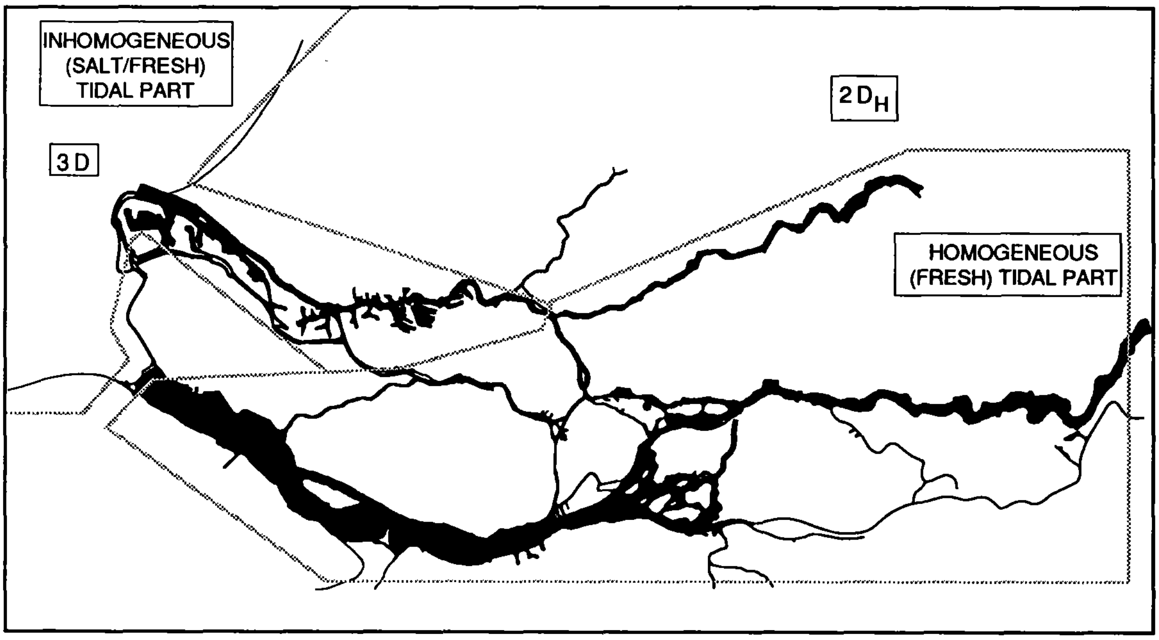


Fig. 8.2.5 Set up base model, 1987. Three-dimensional and two-dimensional curvilinear grid models.

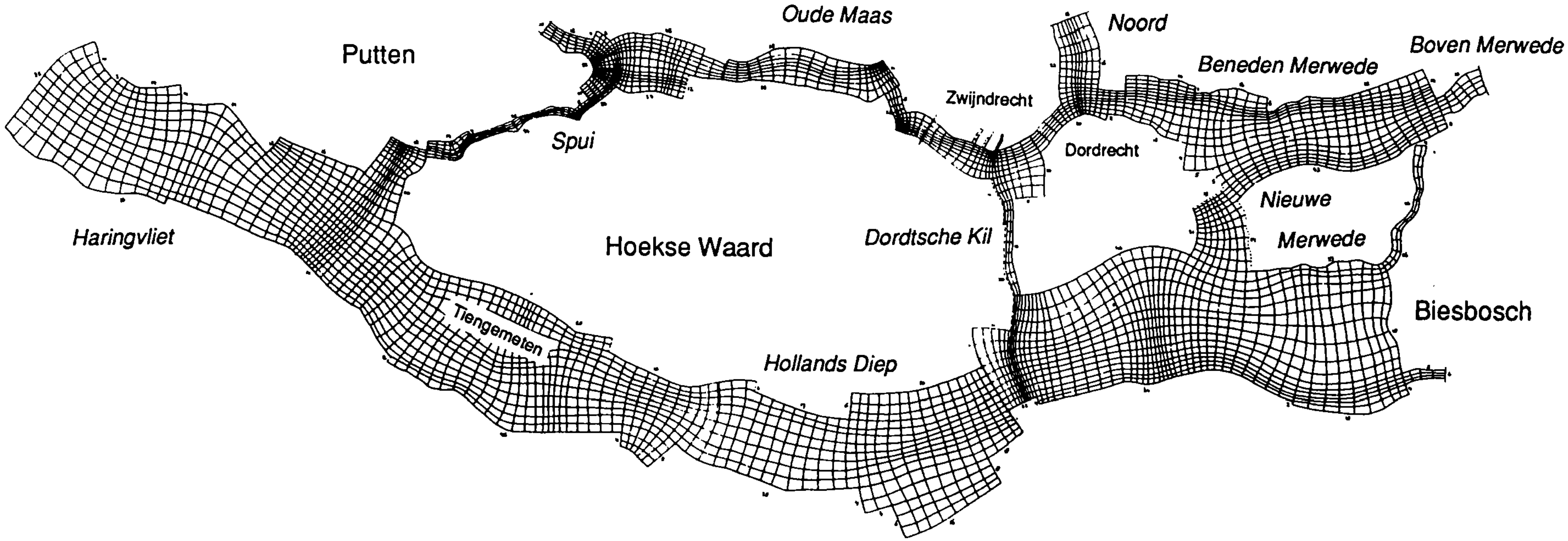


Fig. 8.2.6 Grid for two-dimensional, vertically averaged calculations

- developing models according to their physical applications (salt transport models, morphological models, water quality models);
- developing an integrated 3-dimensional model for both flows, sedimentation/erosion and water quality. This model serves as basis for the other dimensions, which will be obtained by reducing the dimensions ('degeneration' of the 3-dimensional model) and has integrated pre- and postprocessingfacilities.

Originally the aim has been some mixture of the possibilities with the use of integrated pre- and postprocessing facilities. A recent re-analysis of the activities resulted in the choice for the third possibility as a long-term development goal. This long term covers, say, 5 years. This option has been based on considerations on each of the before-mentioned aspects. The present experience with 3-dimensional models, improved numerical techniques and the integration of pre- and postprocessing have played a major role in this choice. The option will be based on the curvilineair grid modelling (see for example Wybenga, 1985) that provides one numerical approach and a unified basis for all dimensions. Moreover the 'degeneration' possibility, for example from a 3-dimensional to a 2-dimensional situation, in principle overcomes the coupling of 3- and 2-, and 2- and 1-dimensional programmes (see Figs. 8.2.5 and 8.2.6).

To meet the short-term requirements of the commissioner, this year a 1-dimensional model for tidal flow, sedimentation/erosion and for water quality will be completed. This serves as a temporary solution looking very much like the first mentioned extreme. It will, however, make use of the available presentation facility PRESENT.

Moreover, to meet the requirements on siltation and water quality problems for the next few years the 2-dimensional, laterally averaged model DISTRO will serve as a special purpose, operational tool for the inhomogeneous salt/fresh part of the estuary, including its harbours, (see Perrels and Karelse, 1981 and Perrels *et al.*, 1987). This model can be characterised as developed according to its dimension with integrated facilities for flow, siltation and water quality. It will make use of the integrated pre- and postprocessing facilities. In the long term a 3-dimensional modelling of this part of the estuary (the Rotterdam Waterway area), rather will be obtained by the extension of the 3-dimensional model for the North Sea part into the Rotterdam Waterway.

5. Conclusions and recommendations

1. With respect to the selection of a set of mathematical flow models for the modelling of the Rhine Meuse estuary an evident evolution has taken place from developing and coupling of three different models respectively a 1D, $2D_v$ and a 3D model to a full 3D approach with degeneration facilities where a 2D or even a 1D approach satisfies.
2. In the scope of estuarine management morphology and water quality have become most important issues. The development of accurate flow models is therefore not an aim in itself but must be related to the transport phenomena they will be applied to. Therefore a time lag between the development of the flow models and the transport models for morphology and water quality is not desirable.
3. The 'unification' of the automation related aspects is an important factor in obtaining a flexible system of models. The integration of the pre- and postpro-cessing is regarded as the most important factor. In a test case it has been shown that this integration, based on a goal architecture of the total system can be reached. The case shows that the integration of a mathematical model and physical model are in principle not different from the integration of different mathematical models.
4. From the 'unification point of view' it is essential to pay attention to:
   - the choice of a goal architecture for the total system as well as for the different parts;

- standardization of the interface between the different parts of the system;
- the use of project organization; and
- splitting the project into smaller parts with clear targets to be reached.

5. Concerning the use of computer systems the aim for developers and final users is at distributed processing. The pre- and postprocessing will take place on a mini computer in combination with personal computers. The real simulation runs will be performed on a mainframe. Developments in computer systems could, however, change this last point. Use will be made of commercially available software rather than build everything in house. This policy leads to a quite different approach in developments than that used until now and to dependency on vendors.
6. Developments have started in a parallel way. This approach would have resulted in long development times with serious risk of producing only outdated products. Nor did it aim to produce intermediate products. Based on the present experiences and insight, the tendency is to go more to a serial and concentrated way of developing. This should overcome the problem of becoming overruled by the fast evolution of software techniques.

Moreover, it does provide the necessary operational tools long before the total development will be finished. From an organizational point of view it is also easier to aim at concrete near goals than at not well defined long-term achievements.

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8.3 The West Dvina (the Daugava) (USSR)

In the mouth of the West Dvina flowing into the Riga Bay of the non-tidal Baltic Sea, brackish seawater penetrates into the river at river discharges below 1000 m^3/s. In these studies the limit of the penetration of the saltwater is defined by the isohaline of 1‰. The natural overdeepening of the river channel, a deep artificial navigation canal through the river mouth bar raises the sea level at storm surges and the wind from the sea promotes seawater intrusion.

For the summer condition the following values of the controlling factors are typical:

(1)riverine: river discharges below 500 m^3/s, river water velocities below 0.4 m/s, water temperature about 20° C;

(2)marine: seawater salinity 4.5–5.5‰, seawater temperature 8-15° C;

(3)local: averaged river channel depth not less 5 m, channel width about 500 m.

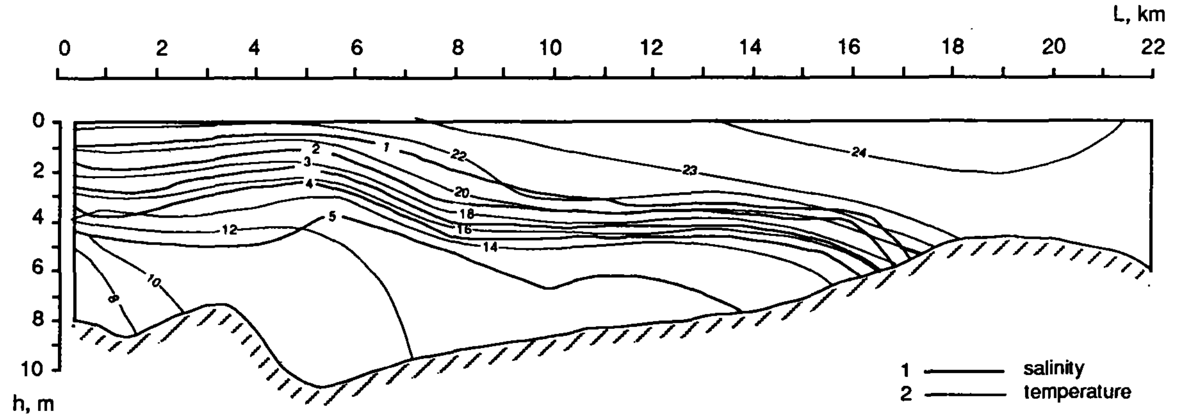

Fig. 8.3.1 The distribution of the water salinity S‰ (1) and water temperature T °C (2) in the river part of the mouth of the West Dvina on 22–23 July 1954 according to M. M. Rogov ($Q = 250\ m^3/s$)

When the river discharge is 250-300 m^3/s brackish water penetrates into the river to 15–17 km (Fig. 8.3.1). The mixing of the river and seawater occurs according to the saltwater wedge type. According to the Equation (2.3) n is much more than 1.0 and according to the Equation (2.4) α is near to ∞.

Calculation of the penetration of the saltwater wedge was carried out according to the formula of Keulegan (Equation 3.19). The base data for the calculation are the following: $T_r = 20 - 23°$ C, $T_s = 8 - 15°$ C, $S_s = 4.5 - 5.5‰$, $\rho_r \sim 997.9\ kg/m^3$, ρ_s 1003.5 kg/m^3, $\Delta\rho \sim 5.6\ kg/m^3$, $\rho_m \sim 1000.7\ kg/m^3$, $\Delta\rho/\rho_m \sim 0.005$, h = 5.0 m, B = 500 m, A = Bh = 2500 m^2. The densimetric velocity is equal to $v_\rho = \sqrt{0.005 \cdot 5 \cdot 9.81} = 0.49$ m/s. The densimetric Reynolds number is equal to

$$Re_\rho = \frac{0.49 \cdot 5}{1.01 \cdot 10^{-6}} = 2.23 \cdot 10^6,\ Re_\rho^{1/4} = 38.6.$$

The critical river discharge according to the calculation is equal to

$$Q_{r_{cr}} = v_{cr} \cdot A = v_\rho \cdot A = 0.49 \cdot 2500 = 1225\ m^3/s.$$

The results of the calculation of seawater intrusion into the West Dvina river according to the Keulegan's method are represented in Table 8.3.1.

Table 8.3.1

Q m^3/s	v m/s	$F_{r\rho} = \frac{v}{\nu\rho}$	$F_{r\rho}^{-5/2}$	L_s, m, by calculation at K= 1	L_s, km, by observation	L_s, km, by calculation at K=1.9
200	0.08	0.16	97.7	19 300	-	36.7
250	0.10	0.20	55.5	10 712	17	20.3
300	0.12	0.24	35.7	6 890	15	13.1
400	0.16	0.33	16.1	3 107	-	5.9
600	0.24	0.49	5.95	1 148	-	2.2
1000	0.40	0.82	1.64	316	0	0.60
1225	0.49	1.00	1.00	193	0	0.37

The comparison of the results of the calculation according to the formula (3.19) and the data of the observations show that the coefficient K in Keulegan's formula should be equal in this case to 1.59–2.18, and on average to 1.9.

The conditions when $Q < Q_{cr}$ and when the saltwater penetrates into the river are typical for all the months except the flood period (April and May) (Table 8.3.2).

Table 8.3.2

Month	I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	Average
Q m^3/s	325	340	565	2085	1320	405	275	305	315	475	506	390	585

The relation of L_s to Q is represented in Fig. 8.3.2.

Under the daily and weekly flow control by the Riga hydropower station since 1974 the regime of saltwater intrusion into the mouth of the West Dvina has a wave and periodical character. In the periods between flash flows due to release the hydropower station the brackish water intrudes nearly to the dam.

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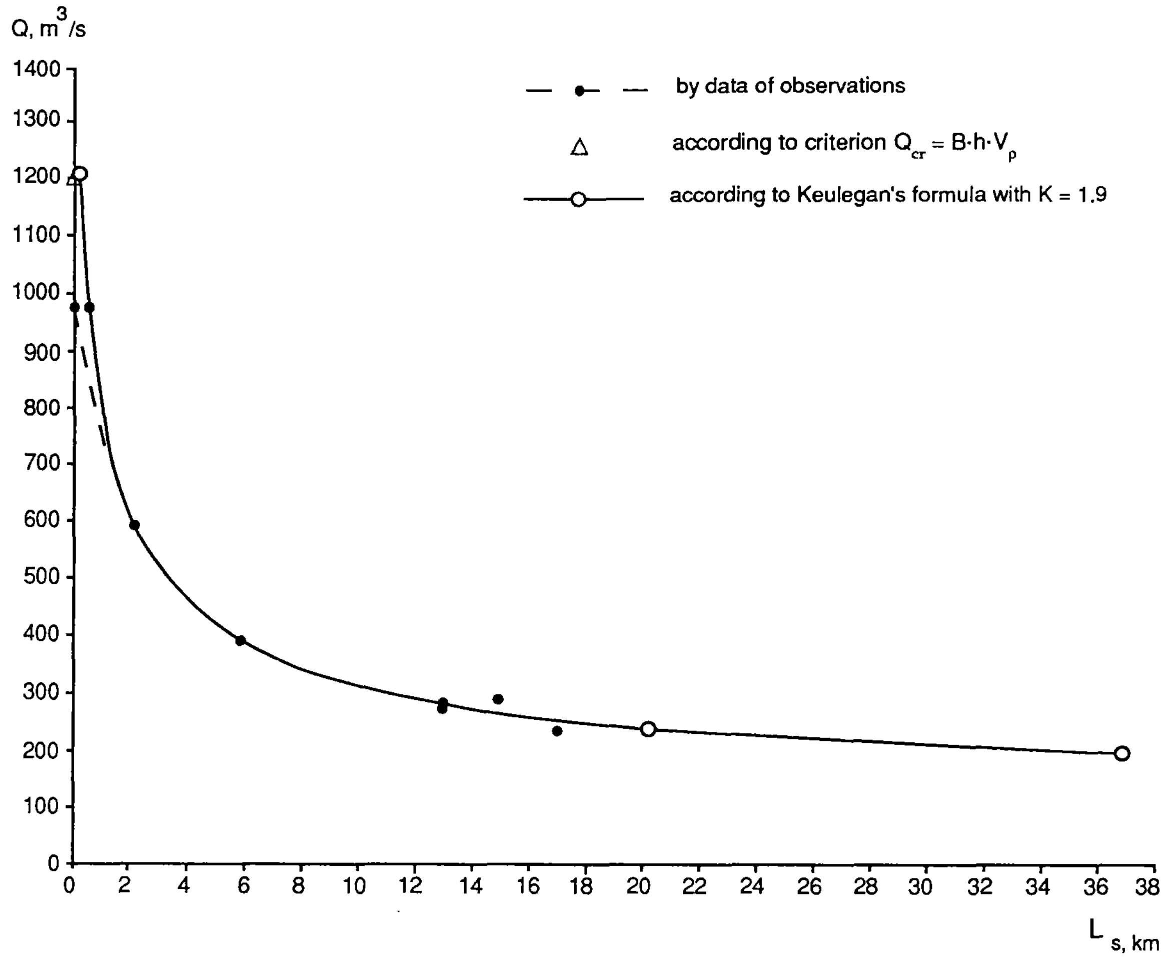

Fig. 8.3.2 The relation of the length of penetration of the brackish water into the mouth of the West Dvina (L_s) to river discharge (Q)

8.4 Sulina branch of the Danube delta (Romania)

The Danube flows into the non-tidal Black Sea. Even in the absence of storm surges salt seawater penetrates into the delta branches deepened for navigation. The mixing of the river and the seawater is of the saltwater wedge type, $n >> 1.0$ or $\alpha = \infty$.

The Sulina branch artificially deepened to 7–10 m is most affected to saltwater intrusion. The length of this penetration at the bottom reaches 20–30 km. The saltwater penetrates into the Sulina branch with discharges of the branch less than 1280 m^3/s. In these cases a storm surge is not a necessary condition. The typical case is represented in Fig. 8.4.1.

Calculation of the length of the saltwater wedge in the Sulina branch of the Danube delta was carried out according to Keulegan's formula (3.19) for the months with the least water discharge (October–November). The base data for the calculation are the following:

$T_{river} = 5 - 15° C$, $T_{sea} = 9 - 13° C$,
$S_s = 15 - 16‰$, $\rho_r \sim 999\sim.7\ kg/m^3$, $\rho_s \sim 1011.7\ kg/m^3$, $\Delta\rho \approx 12\ kg/m^3$
$\rho_m \approx 1005.7\ kg/m^3$, $\Delta\rho/\rho_m \approx 0.012$, $h = 8.0\ m$
$B = 170\ m$, $A = Bh = 1360\ m^2$.

For these conditions the densimetric velocity is equal to
$v_\rho = \sqrt{0.012 \cdot 8 \cdot 9.81} = 0.97\ m/s$, and the critical water discharge is equal to
$Qr_{cr} = v_\rho \cdot A = 1320\ m^3/s$

The densimetric Reynolds number is equal to $Re_\rho = \frac{0.97 \cdot 8}{1.3 \cdot 10^{-6}} = 5.97 \cdot 10^6$ and $Re_\rho^{1/4} = 49.4$.

The results of the calculation are represented in Table 8.4.1.

Comparison of the results of the calculation and data of the observations show that the coefficient K in Keulegan's formula should be equal to 2.80 – 3.00 in this case and on average to 2.9. The conditions favourable for the saltwater intrusion into the Sulina branch are during all months except the flood period (April–June) (Table 8.4.2).

Table 8.4.1

Q m^3/s	v m/s	$F_{r\rho} = \frac{\upsilon}{\nu\rho}$	$F_{r\rho}^{-5/2}$	L_s, m, by calculation at K= 1	L_s, km, by observation	L_s, km, by calculation at K=2.9
350	0.26	0.27	26.3	10 394	-	30.1
400	0.29	0.30	20.4	8 062	-	23.4
450	0.33	0.34	14.9	5 888	16.5	17.1
516	0.39	0.39	10.53	4 161	12.5	12.1
700	0.53	0.53	4.90	1 936	-	5.6
900	0.66	0.68	2.62	1 035	-	3.0
1100	0.81	0.84	1.54	609	-	1.8
1280	0.94	0.97	1.08	427	0	1.2
1320	0.97	1.00	1.00	395	0	1.1

Table 8.4.2

Month	I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	Average
Q m^3/s	971	1003	1229	1432	1508	1438	1223	947	793	742	887	1015	1100

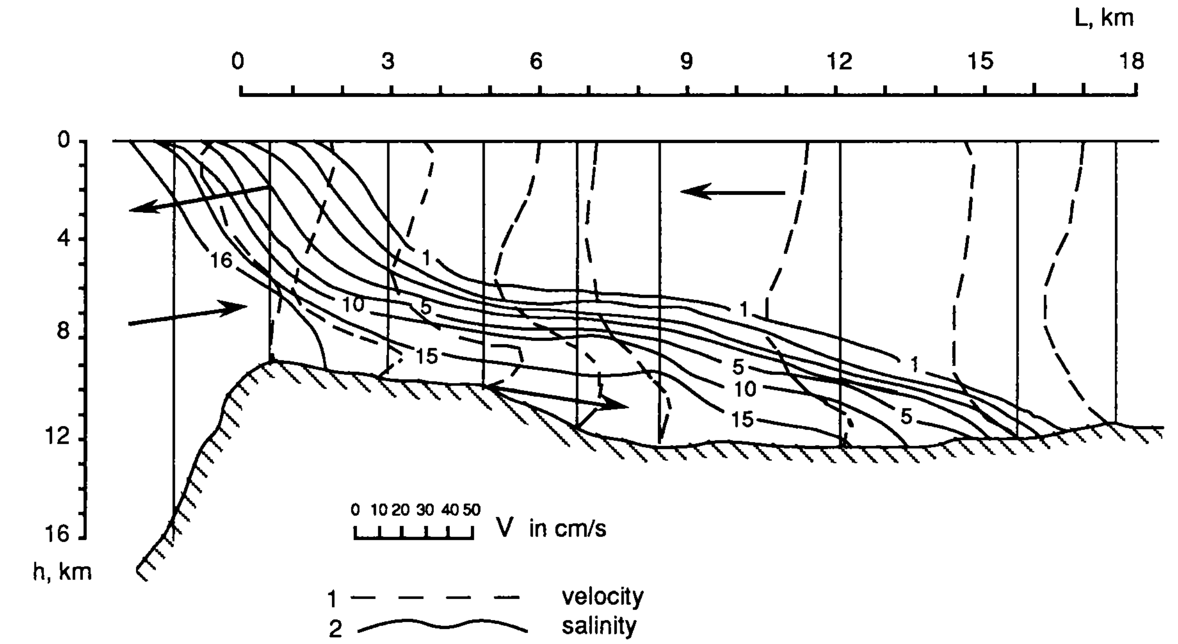

Fig. 8.4.1 The distribution of the water salinity S‰ (2) and water velocity V (1) along Sulina branch of the Danube delta on 22 November 1963 according to C. Bondar ($Q = 450$ m^3/s)

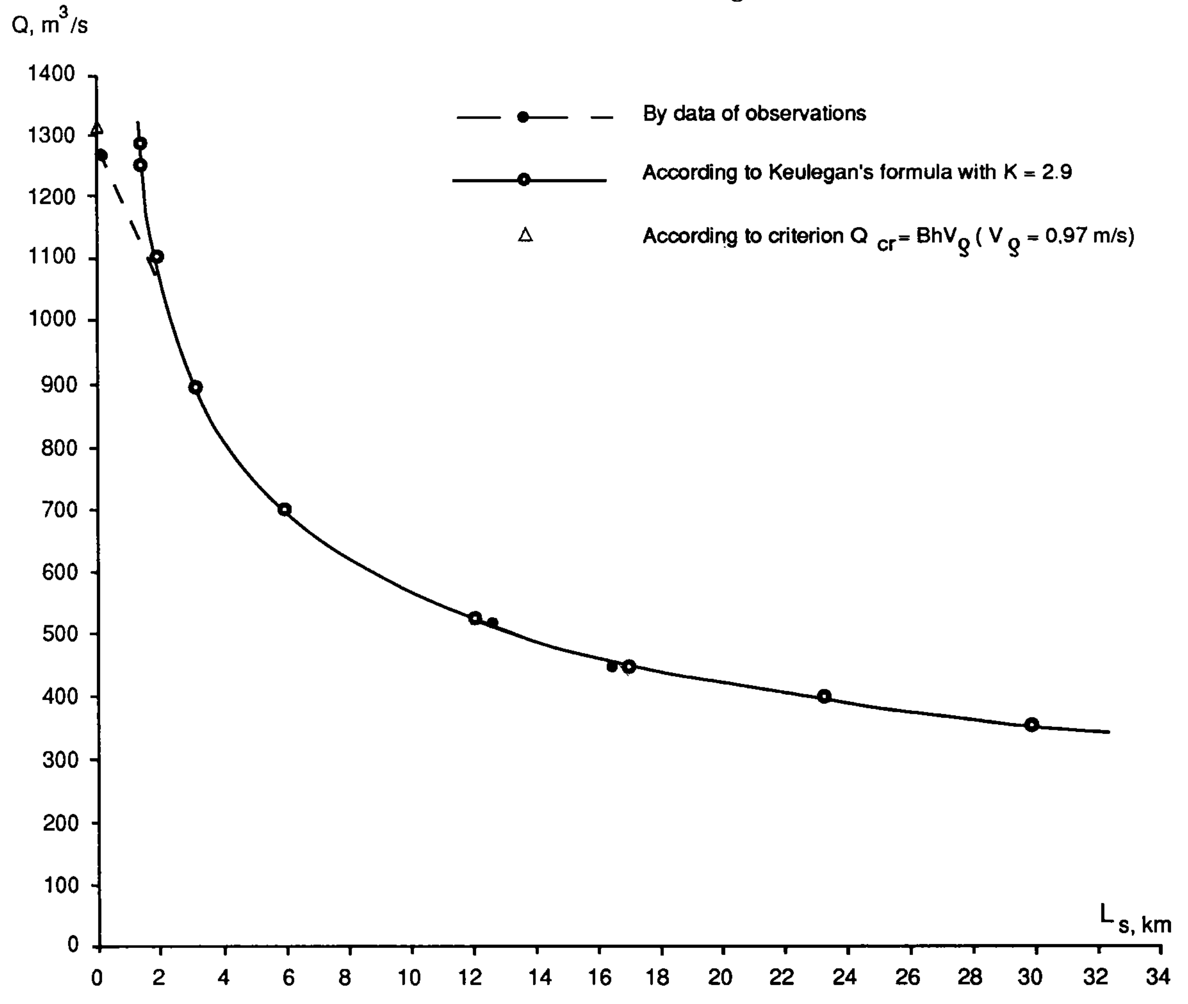

Fig. 8.4.2 The relation of the length of the penetration of the salt water into Sulina branch of the Danube delta (L_s) to the water discharge in Sulina branch (Q).

The data of Table 8.4.2 are obtained by month averaged discharges of the Danube above the top of the delta, taking into account that the discharge of the Sulina branch is equal to 17.5% of the discharge of the Danube.

The relation of L_s to Q is represented in Fig. 8.4.2.

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8.5 The North Dvina (USSR)

The mouth area of the North Dvina, flowing into the White Sea, belongs to the tidal type. The average tidal range at the offshore part of the mouth is about 1.0 m. The tidal variations of the water level propagate upstream about 140 km and the length of the reach with reversal currents is about 80 km. The brackish waters penetrate into the river with a river discharge less than 10000 m^3/s, maximum value of L_s is 40–45 km. The river discharges for the North Dvina are given above the top of the mouth area or at the top limit of the propagation of tidal variations of the water level (135 km from the sea). The type of the mixing of river and seawater depends on the river discharge and the tidal range. On average with river discharges less than 1000-2000 m^3/s, $1.0 > n > 0.1$ and $1.0 > \alpha > 0.1$ which corresponds (see Table 2.1) to the case of partial (moderate) mixing (Fig. 8.5.1). With discharges between 2000 and 10000 m^3/s, $n > 1.0$ and $\alpha > 1.0$, which corresponds (see Table 2.1) to the case of a saltwater wedge. The data of the observations show that the length of the penetration of the brackish water into the river depends generally on the value of the river discharge (Fig. 8.5.2). The greatest penetration of brackish water into the river is typical for deep navigable branches. In rare cases the brackish water penetrates to the top of the delta (45 km from sea).

With a discharge $Q_r = 5000$ m^3/s the length of the penetration of the brackish water (L_s) is about 4 km from the sea. With further decreasing of discharge Q_r the value of L_s quickly increases: to 7 km with 3000 m^3/s, to 11 km with 2000 m^3/s and to 24 km with 1000 m^3/s. The mean value of L_s in the period of summer low flow is equal to 10–15 km.

The greatest penetration of brackish water during the last 50 years was observed in November 1974, when with a discharge of the West Dvina of about 440 m^3/s and a strong storm surge the brackish water reached to the top of the delta (L_s = 43–45 km).

During the whole year the brackish water penetrates into the channel of the West Dvina, except during the flood period in May, when the discharges are above the critical values (Table 8.5.1).

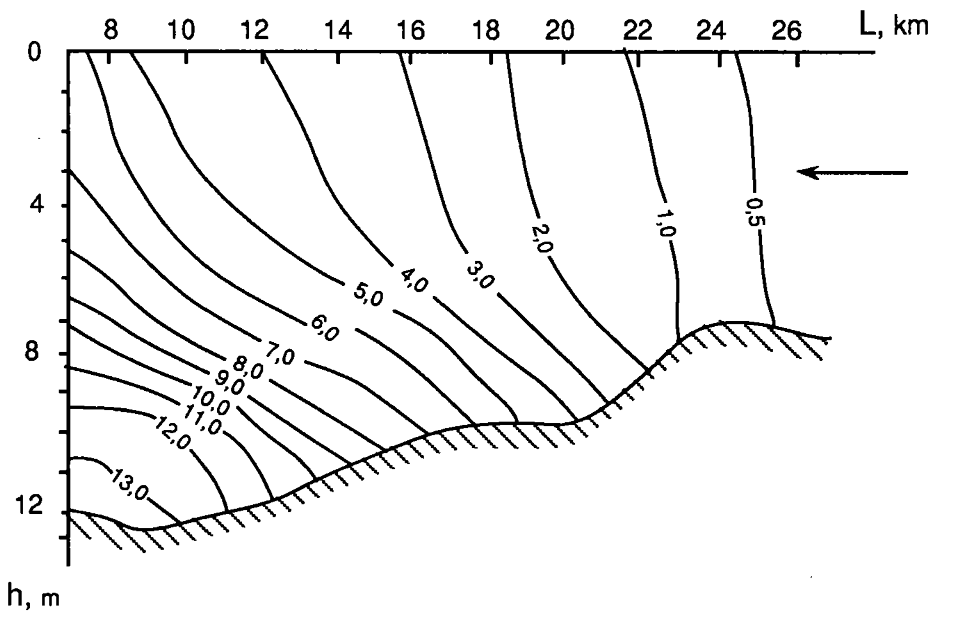

Fig. 8.5.1 The distribution of water salinity S‰ in the main branch of the North Dvina delta with a river discharge of 1000 m^3/s in the top of the mouth area and with an absence of storm surges and internal waves, according to Y. V. Lupachiev

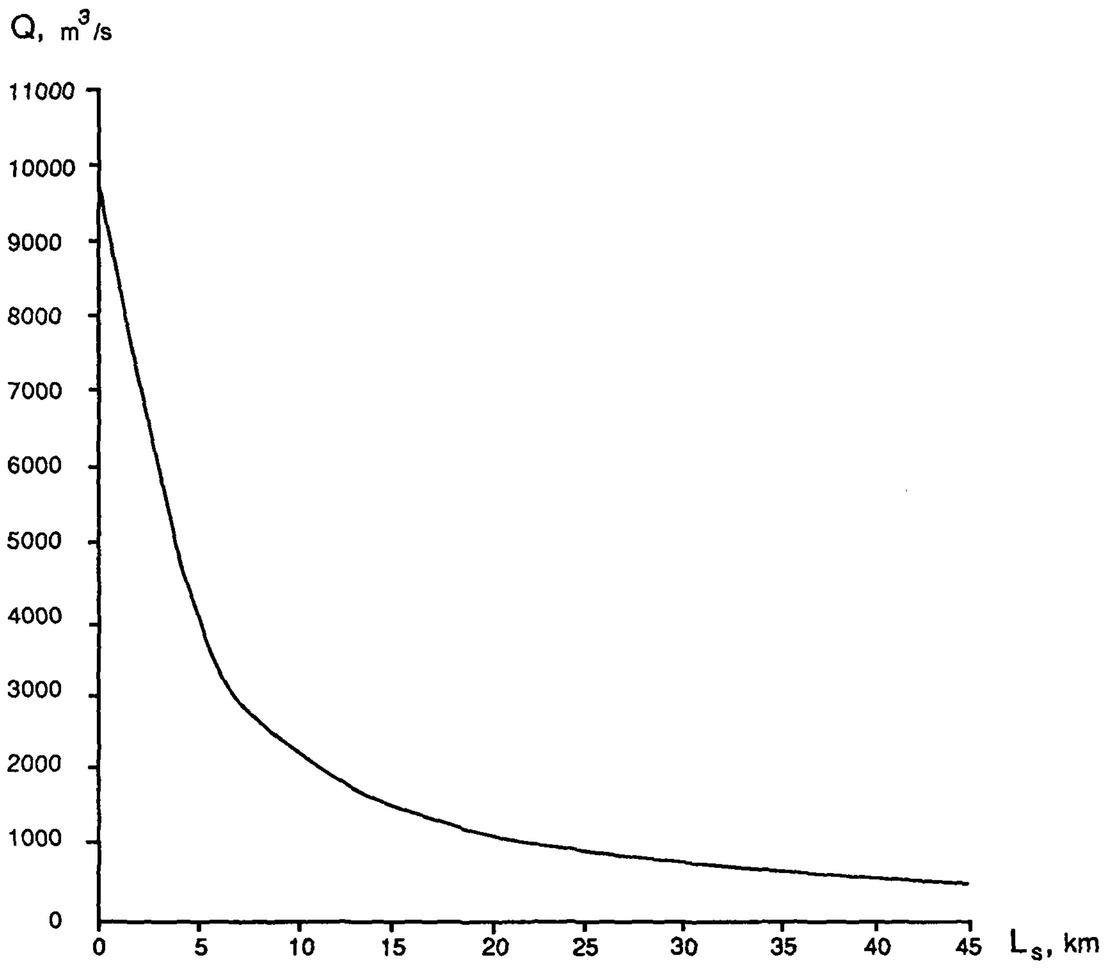

Fig. 8.5.2a The relation of the length of penetration of the salt water into the North Dvina delta (L_s) to the river discharge in the top of the mouth area (Q) by data of the observations

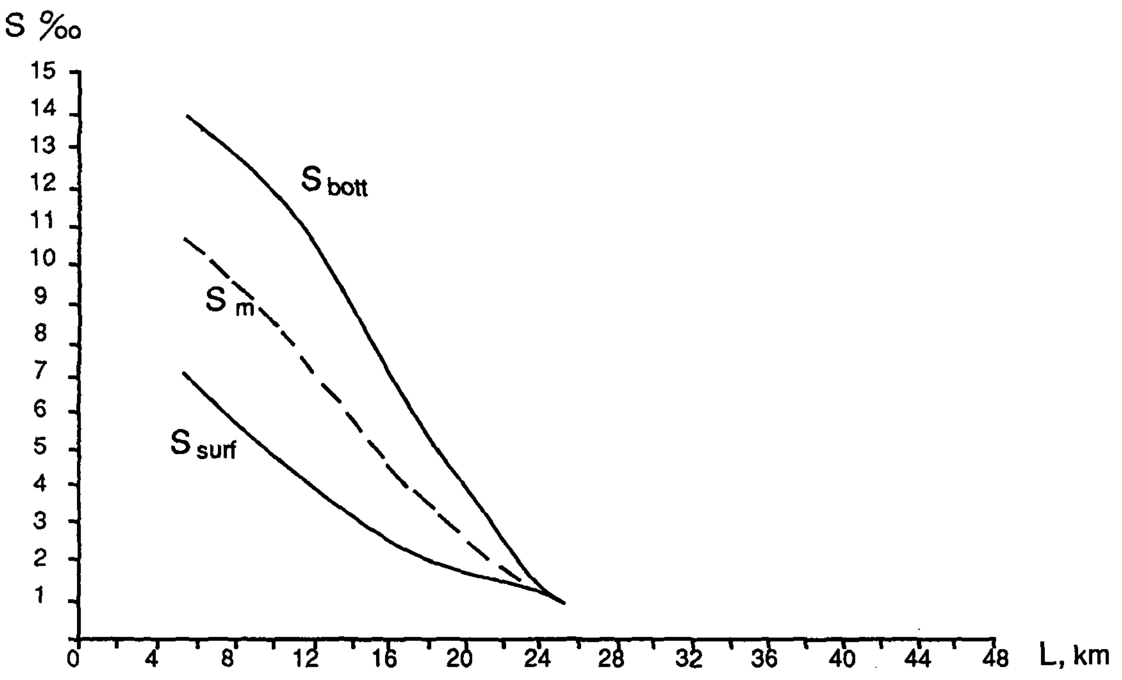

Fig. 8.5.2b The longitudinal distribution of the water salinity at the surface (S_{surf}), at the bottom (S_{bott}) and averaged over the depth (S_m) with a river discharge at the top of the mouth area of 1000 m^3/s

Table 8.5.1

Month	I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	Average
Q m^3/s	1020	813	689	2280	13700	7160	2940	2170	2330	2920	2370	1400	3320

The length of the penetration of the brackish water into the river depends not only on river discharge (Fig. 8.5.2a), but varies in relation to tides, storm surges and internal waves. With the tide the value of L_s, varies in the range of ± 3 km, and with storm surges in the range of ± 5–7 km. The common influence of tides and storm surges leads to variation of the length of the penetration of brackish water into the river in the range ± 8 km on the average. The influence of internal waves on the interface between fresh and saltwater may temporarily increase the value of L_s by 10–15 km.

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8.6 Comparison of data

The comparison of the main hydrological characteristics of the mouth of the West Dvina, Sulina branch of the Danube delta and the mouth of the North Dvina is given in Table 8.6.1.

Table 8.6.1

River (branch)	Mean annual discharge, Q m^3/s	Critical discharge by observation, Qcr m^3/s	Maximum value of Ls, km	Mean depth in the reach of penetration of the salt water, h, m	Water salinity in the offshore part of the mouth with low river flow, S‰	$\frac{\Delta\rho}{\rho_m}$
West Dvina	585	1000	28	5.0	4.5 - 5.5	0.005
Danube, Sulina branch	1100	1280	30	8.0	15 - 17	0.012
North Dvina	3320	10000	45	7.0	20 - 27	0.016 – 0.020

From the engineering point of view it is better to represent the penetration of saltwaters into the rivers in the form of graphs of the length of penetration L_s related to river discharge (or of the delta branch) Q (Figs. 8.3.2, 8.4.2 and 8.5.2a) and also in the form of graphs of longitudinal distribution of the water salinity S (Fig. 8.5.2b). The graphs $L_s = f(Q)$ are suitable for all types of mixing of river and seawater, but the graphs of $S = f(x)$ are generally suitable for the case of well and partially mixed situations.

Another way to represent the graphs of the relation $L_s = f(Q)$ is a non-dimensional form

$$\frac{L_s}{h} = \psi \frac{(Q_i)}{(Q_{cr})}, \tag{8.6.1}$$

where h is the channel depth, Q_i is the river discharge (or branch) and Q_{cr} is the critical river discharge with which the penetration of the saltwater into the river begins.

The relations conform Equation (8.6.1) are represented in ordinary and logarithmic coordinates (Fig. 8.6.1).

As the analysis of data shows the relations conform (8.6.1) are very similar for the mouth of the West Dvina and Sulina branch of the Danube delta. For the mouth of the North Dvina a more slow increase in L_s/h with a decrease in Q_i/Q_{cr} is typical, which must be related to tidal character of the regime of the mouth and more strong vertical mixing of the water.

In logarithmic coordinates the relation conform (8.6.1) is straightened, which allows an analytical representation by equation

$$\frac{L_s}{h} = k \frac{(Q_i)}{(Q_{cr})}. \tag{8.6.2}$$

Equations of this kind are very opportune for engineering calculations.

The parameters of Equation (8.6.2) for the river mouths examined are represented in Table 8.6.2.

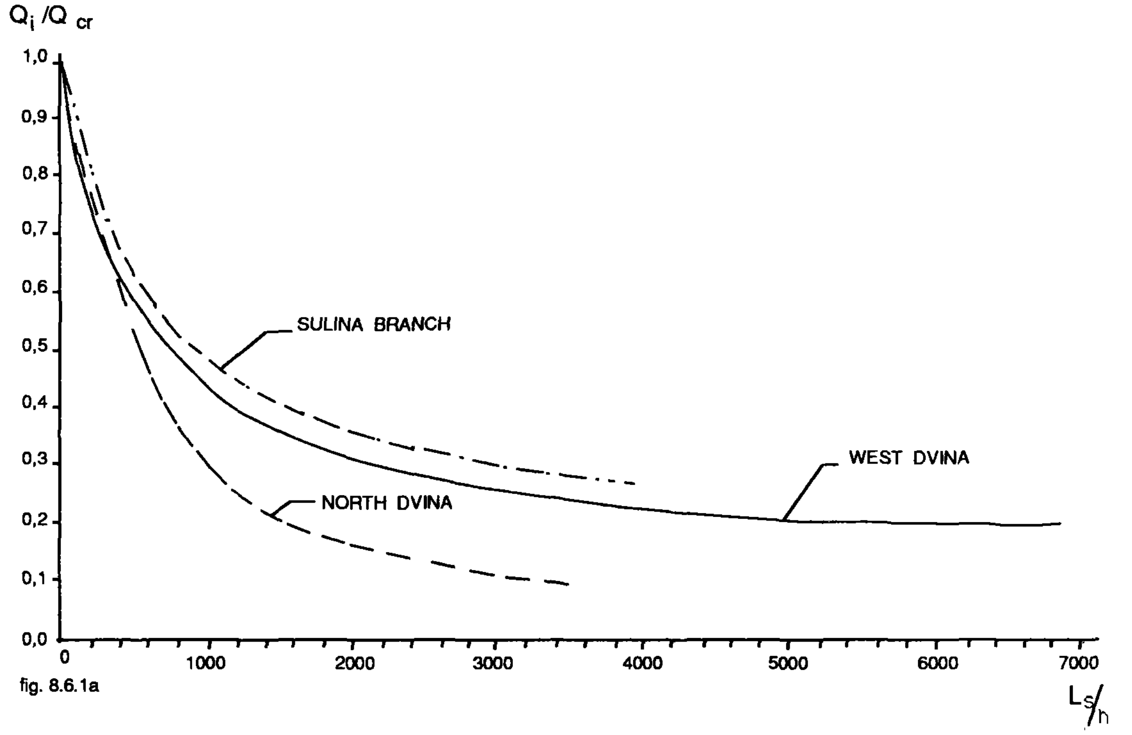

fig. 8.6.1a

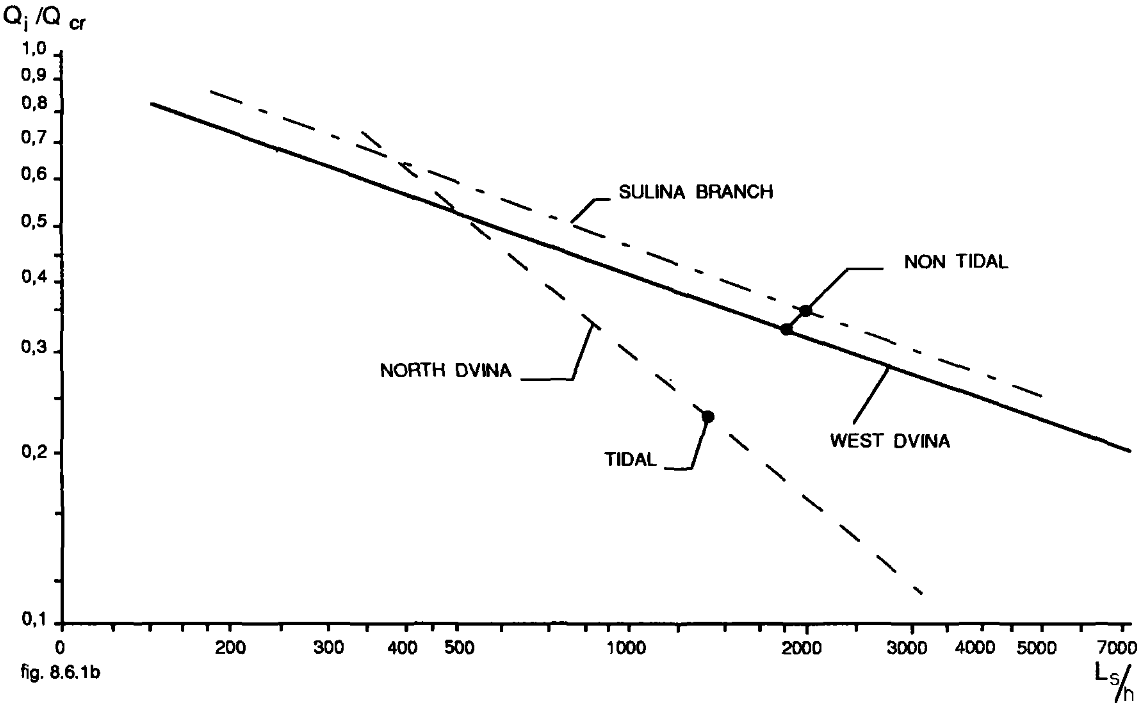

fig. 8.6.1b

Fig. 8.6.1 The relation of the relative length of penetration of the brackish water ($L_{s/h}$) to the relative discharge (Q_i/Q_{cr}) in ordinary (a) and logarithmic (b) coordinates.

Table 8.6.2

River (branch)	Range of Q_i/Q_{cr}	k	m
West Dvina	0.8 - 0.2	76	- 2.94
Danube, Sulina branch	0.8 - 0.2	125	- 2.72
North Dvina	0.7 - 0.1	230	- 1.20

So long as the values of B and h in the mouth part of the channel are practically non-variable with different values of Q, then

$$\frac{Q_i}{Q_{cr}} = \frac{B_i h_i v_i}{B_{cr} h_{cr} v_{cr}} \sim \frac{v_i}{v_p} = Fr_p \tag{8.6.3}$$

when Fr_p is densimetric Froude number (see Section 2.2.3). So for the West Dvina and Sulina branch of the Danube Q_i/Q_{cr} is ~ Fr_p. Hence one can write for these rivers:

$$\frac{L_s}{h} = kFr_p^{\,m} \,, \tag{8.6.4}$$

which approximates the empirical relation to the structure of Keulegan's formula (Equation 3.19).

For the mouth of the North Dvina the substitution $Q_i/Q_{cr} \sim Fr_p$ is impossible for the following reasons:

(1) in this tidal mouth partial mixing is observed more often than the saltwater wedge, and the dependence on the controlling factors of the process of the penetration of the saltwater into river is different;

(2) in this multi-branched deltaic mouth brackish water penetrates into the river through separate branches, but the river discharge in the relations (8.6.1) and (8.6.2) is defined not only above the top of the delta (45 km from the sea), but above the limit of the propagation of the tidal water level variations (135 km from the sea).

Appendices

Appendix 1

Dependence of the water density (kg/m^3) on its salinity (‰) and temperature (° C)

	S‰				
T °C	0	5	10	15	20
0	999.87	1003.99	1008.03	1012.06	1016.08
5	999.99	1004.03	1007.48	1011.93	1015.87
10	999.73	1003.69	1007.58	1011.46	1015.33
15	999.13	1003.03	1006.86	1010.69	1014.51
20	998.23	1002.09	1005.87	1009.65	1013.43
25	997.07	1000.89	1004.63	1008.37	1012.11
30	995.67	999.45	1003.16	1006.87	1010.58

	S‰			
T °C	25	30	35	40
0	1020.09	1024.10	1028.17	1032.22
5	1019.81	1023.75	1027.70	1031.66
10	1019.20	1023.08	1026.97	1030.87
15	1018.33	1022.15	1025.99	1029.84
20	1017.20	1020.99	1024.78	1028.59
25	1015.85	1019.60	1023.36	1027.14
30	1014.29	1018.01	1021.75	1025.50

Appendix 2

Dependence of the densimetric velocity υ_ρ (m/s) on density difference and depth

$\frac{\Delta\rho}{\rho m}$	h, m							
	1	2	3	4	5	10	15	20
0.001	0.099	0.140	0.172	0.198	0.221	0.313	0.384	0.443
0.002	0.140	0.198	0.243	0.280	0.313	0.443	0.542	0.626
0.005	0.221	0.313	0.384	0.443	0.495	0.700	0.859	0.990
0.010	0.313	0.443	0.542	0.626	0.700	0.990	0.213	1.401
0.015	0.384	0.542	0.664	0.767	0.858	1.213	1.486	1.716
0.020	0.443	0.626	0.767	0.886	0.990	1.401	1.716	1.981
0.025	0.495	0.700	0.858	0.990	1.107	1.566	1.918	2.215
0.030	0.542	0.767	0.940	1.085	1.213	1.716	2.101	2.426

Appendix 3

Kinematic viscosity coefficient (m^2/s) versus water temperature (°C) (fresh water)

T °C		0	5	10	15	20	30	40
υ m^2/s	10^{-6}	1.78	1.52	1.31	1.14	1.01	0.81	0.66

Appendix 4

The concept of the tidal prism for the case of well (complete) mixing

(by V. N. Mikhailov).

Assume that in the short river part of the mouth with the invariable width B the tidal range ΔH_x (or the height of the high tide HW above the low tide LW) varies upwards the stream according to the linear law (Fig. A.1):

$$\Delta H_x = \Delta H_0 \left(1 - \frac{x}{L_{\Delta H}}\right) \tag{1}$$

where ΔH_0 is the range of tide at the coastal edge of the river channel, $L_{\Delta H}$ is maximum distance of penetration of tidal water level variations into the river, and x is the distance of the given cross-section from the coastal edge of the river channel.

Usually it is assumed that

$$L_{\Delta H} \sim k \frac{\Delta H_0}{I}, \tag{2}$$

where I is the slope of the water surface at the ebb tide, and $k \sim 1$.

The boundary conditions corresponding to Equation (1) are:

$$\text{at } x = 0 \ \ \Delta H_x = \Delta H_0, \text{ at } x = L_{\Delta H} \ \ \Delta H_x = 0.$$

The volumes of tidal prisms upstream and downstream of the cross-section x (Fig. A.1) P_1 and P_2 are given in TableA.1.

Table A.1: The volumes of tidal prisms

Reach	Average tidal range ΔHm	Area of water surface F	Tidal prisms $P = \Delta Hm\ F$
Downstream of cross-section x	$\Delta H_0 \left(1 - \frac{x}{2L_{\Delta H}}\right)$	Bx	$P_1 = \Delta H_0 Bx \left(1 - \frac{x}{2L_{\Delta H}}\right)$
Upstream of cross-section x	$\frac{\Delta H_0}{2} \left(1 - \frac{x}{L_{\Delta H}}\right)$	$B(L_{\Delta H} - x)$	$P_2 = 1/2\, \Delta H_0 B (L_{\Delta H} - x) \left(1 - \frac{x}{L_{\Delta H}}\right)$

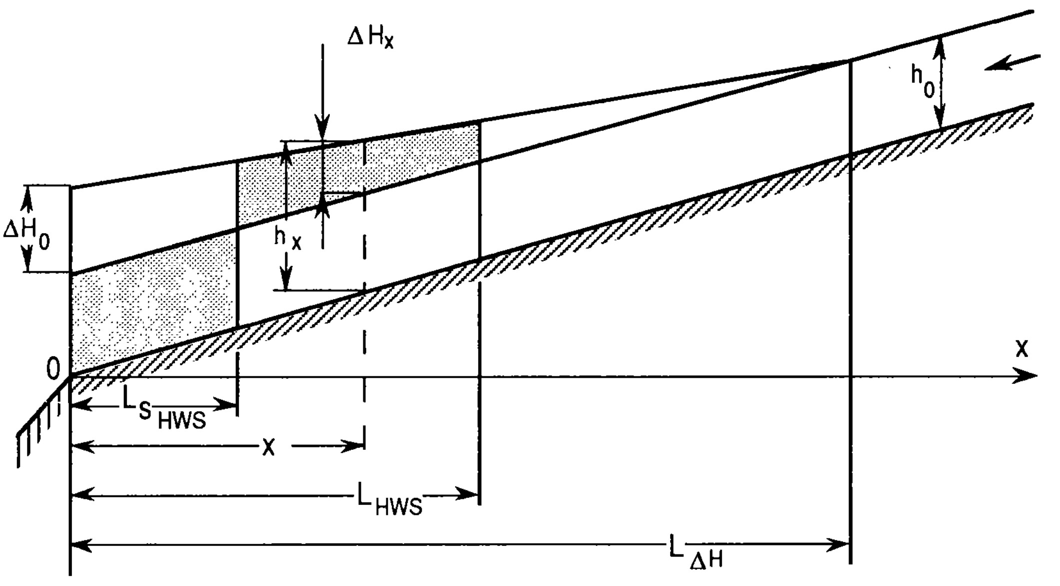

Fig. A.1 Scheme for calculation of tidal prism and penetration length of seawater according to V. N. Mikhailov

In any cross-section in the river part of the mouth area affected by the tidal influence and at any moment of time we have the equality

$$Q = -Q \pm \frac{\Delta P_2}{\Delta t}, \tag{3}$$

where Q_r is mean (averaged over the tidal period) river discharge (axis x is directed landwards), ΔP_2 is the part of the tidal prism upstream of given cross-section, that is water volume accumulated during the flood tide (ΔP_2 is positive) or discharged during the ebb tide (ΔP_2 is negative) in time interval Δt from the beginning of the flood tide or the ebb tide to the given moment.

If there are no tides then $Q = -Q_r$. When the tides present Q has maximum absolute value approximately at the middle of the ebb tide, when $\Delta P_2 \approx P_2/2$ (P_2 is from Table A.1), and $\Delta t \sim T_{ebb}/2$. As P_2 increases towards the sea (with increasing of the length $L_{\Delta H} - x$ and has the maximum value in the coastal edge of the river channel, absolute value of Q increases at the ebb tide towards the coastal edge of the river channel and has its maximum in this point.

Q has its maximum value during the flood tide approximately at the middle of the flood tide when $\Delta P_2 \sim P_2/2$ and $\Delta t \sim T_{fl}/2$.

The largest value of Q is also observed in the coastal edge of the river channel.

Q can become equal to Q twice: at reversal of the ebb current (– Q) into the flood current (+Q) at the moment of low water slack (LWS) and on the contrary at reversal of the flood current (+Q) into the ebb current (–Q) at the moment of high water slack (HWS). In the second case the distance, where the reverse current occurs will be the greatest. In this case Q = 0, if

$$Q_r = \frac{P_2}{T_{fl}}. \tag{4}$$

Let us find the maximum distance L_{HWS} from the coastal edge of the river channel, where at the moment HWS and with a definite Q_r and ΔH_0 the discharge and velocity averaged over cross-section are equal to 0.

The value of P_2 from equation in Table A.1 must be introduced in (4). After the solution of the square equation in relation to x and assuming that $x = L_{HWS}$ we can obtain:

$$L_{HWS} = L_{\Delta H} - \sqrt{\frac{2Q_r L_{\Delta H} T_{fl}}{B \cdot \Delta H_0}} \tag{5}$$

Before the square root we have the negative sign because $L_{\Delta H} > L_{HWS}$. Equation (5) represents the expression for the length of the reach where reverse currents with given Q_r, ΔH_0 and T_{fl} may occur.

Now let us find the distance where salt or brackish waters may penetrate into the river from the sea. Let us assume that water density (and salinity) is homogenous over the cross-section. Let's also assume that the freshwater is present at the end of the ebb tide over the whole river part of the mouth and that the intrusion of salt or brackish water begins through the coastal edge of the river channel just after the reversal of the ebb current into the flood current or at LWS.

In any cross-section x the brackish water may appear if all the freshwater situated before this downstram of this cross-section move from the coastal edge of the river channel to this point. The volume of freshwater downstream of the cross-section x which must move upwards before the appearance of brackish water in the cross-section x, is equal approximately to $Bh_0 x$, where h_0 is the channel depth at the ebb. This water volume will partially form the tidal prism P_2 (equal volume in Fig. A.1 is shaded). The rest of the tidal prism P_2 will be filled by freshwater coming from upstream. This volume is equal to $Q_r T_{fl}$ for the flood tide. Thus for the moment of appearance of salt or brackish water in the given cross-section x we will have for the tidal prism upstream of this cross-section the following condition:

$$P_2 = Bh_0 x + Q_r T_{fl} \,. \tag{6}$$

Putting the value of P_2 from Table A.1 in Equation (6) we find the distance x from the coastal edge of the river channel, where this condition is true. Solving square equation for x and considering that $x = L_{sHWS}$ we shall have:

$$L_{s_{HWS}} = L_{\Delta H}\left(1 + \frac{h_0}{\Delta H_0}\right) - \sqrt{\frac{L^2_{\Delta H} h_0}{\Delta H_0} - \left(2 + \frac{h_0}{\Delta H_0}\right) + \frac{2Q_r L_{\Delta H} T_{fl}}{B \Delta H_0}} \,. \tag{7}$$

Before the square root we have a negative sign because $L_{s_{HWS}} < L_{\Delta H}$. Expression (7) represents in fact also the formula for the estimation of the tidal excursion length ($L_{tide\,exc.}$) at the lower reach of the river part of the mouth.