COASTAL ENGINEERING

Volume I

INTRODUCTION

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"If you stay with a problem long enough you will get the answer. It may not be the one you expected, but the chances are it will be the truth."

Charles M. Allen

COASTAL ENGINEERING

Volume I - Introduction

edited by

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Department of Civil Engineering
Delft University of Technology
DELFT
The Netherlands

1976

Price: ƒ 3,40

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1. INTRODUCTION

W.W. Massie

1.1. Purpose

This set of lecture notes is written primarily to supplement the classes conducted by Prof. E.W. Bijker which are held in Delft, both at the University of Technology and at the International Course in Hydraulic Engineering. The lecture time will be used primarily to discuss and amplify these notes and answer questions.

Some can probably learn much from these books without having attended the classes at all. Questions are often posed within the text; all are intended to stimulate thought and verify understanding.

1.2. Subdivisions

The entire material of coastal engineering presented by Prof. Bijker at the Delft University of Technology is currently divided into three courses:

a. Introduction to Coastal Engineering - required for all hydraulic engineering students.
b. Topics in Coastal Engineering - a more advanced treatment of certain specific more specialized topics, required for all coastal engineering students.
c. Breakwater Design - treats that particular, specialized topic.

This subdivision has been retained in the preparation of these books; the material is divided into three separate volumes, with each volume prepared for one of the three courses listed above.*

Another subdivision is also possible; it is often handy to subdivide the material of coastal engineering into three broad areas according to the types of problems which are treated. These three broad categories are Harbors, Morphology, and Offshore and are discussed further in chapter 2. This division has been retained in the first two volumes of this book. Within each of these volumes material has been grouped in each of these categories. This subdivision is not apparent in volume III since breakwaters fall almost exclusively into the harbor category.

A fourth category of information has been added in these notes to review necessary background theory normally presented in other courses; this is done for completeness. Many can skip over this category completely, others will find it useful. The understanding of this background is, however, of vital importance for the true coastal engineering topics which are

* It has later been decided to separate the offshore engineering in a separate volume. Thus, this appears as volume IV.

built upon this foundation.

1.3. Periodical literature

Specific literature references have been included at the end of each of the four volumes. These are indeed references; they provide background instead of highlighting the most recent developments. Periodical literature provides the best means of keeping up to date. Such literature can be grouped into five sorts, each is described a bit below.

General

Engineering periodical literature of this sort covers a broad spectrum of topics within engineering and, as such, occasionally contains something of direct interest to coastal engineers, even though such articles often lack specific technical detail. Examples of such periodicals are:

a. Engineering New Record, published weekly by McGraw Hill Publications, New York, U.S.A.
b. De Ingenieur, published weekly by the Royal Society of Engineers (Koninklijk Instituut van Ingenieurs), The Hague, The Netherlands
c. Civil Engineering, published monthly by the American Society of Civil Engineers, New York, U.S.A.

General Specific

This group of journals provide general information about a specific topic area. These usually contain information of direct interest but specific technical details are usually still lacking. Examples of this sort of literature are:

a. Ocean Industry, published monthly by the Gulf Publishing Co., Houston, Texas, U.S.A.
b. The Dock and Harbor Authority, published monthly by Foxlow Publications, Ltd., London.

Technical Specific

This group of publications provide, in general, most of the specific technical details of a problem and its solution, and are often found in the references listed in articles found in the above sorts of periodicals. Examples of technical specific literature are:

a. Journal of the Waterways, Harbors, and Coastal Engineering Division, published quarterly by the American Society of Civil Engineers, New York, U.S.A.
b. Shore and Beach, published semiannually by American Shore and Beach Preservation Association, Miami, Florida, U.S.A.
c. Coastal Engineering in Japan, published annually by Japan Society of Civil Engineers, Tokyo, Japan.

Strange Technical

These journals provide the same type of information as the previous sort of journals, but are intended for an entirely different specialty group. It takes a bit of ingenuity on the part of the investigator to discover related topic areas and patience to seek through its literature on the small chance that it contains something useful. Often this searching can be avoided by using an abstract index - see below. The examples listed below serve only to illustrate that useful information can be found in this sort of journal.

a. An article on wave forces: Journal of the Engineering Mechanics Division, published by the American Society of Civil Engineers, New York, U.S.A.
b. An article on wave action in harbors: Journal of the Acoustical Society of America, New York, U.S.A.

Abstracts

Abstracts, indexed in some way, serve to provide easy access and quick reference to the vast domain of literature. Abstracts, of themselves, do not provide any new information; the simply condense and index existing articles. Among the excellent abstract and indexing services are:

a. Documentation Data, published by the Delft Hydraulics Laboratory, Delft, The Netherlands
b. Engineering Index, published by the Engineering Societies Library, New York, U.S.A.

Both of these services are available in the Main Library of the Delft University of Technology. The Engineering Index abstracts can be examined via a display terminal there, even though this type of work is expensive. In addition a file of the Documentation Data is maintained in the Laboratory of Fluid Mechanics of the Civil Engineering Department.

1.4. Reference Books.

A few general reference books of specific interest to coastal engineers are listed here. Each of these will tell something but usually not everything about a wide spectrum of coastal engineering topics.

a. Per Bruun (1973): Port Engineering: Gulf Publishing Company, Houston, Texas, U.S.A.
b. Arthur T. Ippen (1966): Estuary and Coastline Hydrodynamics: McGraw-Hill, New York.
c. H. Lamb (1963): Hydrodynamics (6th edition): Cambridge Univ. Press.
d. Muir Wood, A.M. (1969) Coastal Hydraulics: Macmillan and Co. Ltd., London, England.
e. Robert L. Wiegel (1964): Oceanographical Engineering: Prentice-Hall, Inc., Englewood Cliffs N.J., U.S.A.

1.5. Contributors

These books are prepared by the entire staff of the Coastal Engineering Group of the Delft University of Technology. The primary authors of each section are listed at the beginning. Many others of the staff reviewed each section; final editing and assembly was the responsibility of W.W. Massie. Table 1.1. lists the entire contributing staff for this volume in alphabetical order.

Table 1.1. Contributors to this volume

Ir. E. Allersma, Chief Engineer, Hydrodynamics and Morphology Branch, Delft Hydraulics Laboratory, Delft.

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1.6. Miscellaneous Remarks

The spelling used in this set of books is American rather than English.

A sincere attempt has been made to use consistent, unambiguous notation. Symbols are defined when first introduced in each chapter and a comprehensive list of symbols is provided at the end of each volume.

Literature is cited in the text by author and year date. A complete list of references used is included at the end of each book.

Figures shown are drawn to scale whenever possible. Distorted figures will be specifically pointed out. Many figures in these books are reproduced at 80% of their original size. Their original dimensions can thus be reconstituted by measuring with a 1 : 1250 scale.

Many technical terms used in these notes are listed in a separate glossary giving definitions and Dutch translations.

Since the English system of units is still in common use in the marine industry several tables of units conversion factors are also available separately.

2. OVERVIEW OF COASTAL ENGINEERING

E.W. Bijker

2.1. Definition

Coastal engineering is the collective term encompassing most of the engineering activities related to works along the coasts. In recent years, coastal engineers have often been involved in engineering of structures to be placed offshore as well. It is their primary task to apply technical knowledge to the construction of various works along coasts and offshore. Usually, designs are needed for works for which only incomplete theoretical models are available, thus a fundamental knowledge of the phenomona involved is required as well. Often, coastal engineers must extend the field of technical knowledge.

An additional complicating feature of coastal engineering is that many of the independent variables involved are of a stochastic nature. Statistical computations form the basis for the optimum design techniques applied to many coastal engineering problems.

2.2. Background studies.

Among the fundamental problems facing the coastal engineer are the water movements along a coast, the interactions between moving water and loose beach and sea bed materials, and the hydrodynamic forces exerted by waves and currents on various constructions. These are simply examples for the fundamental phenomona; others will become apparent later. The investigation of these phenomona form the basis for coastal engineering research.

2.3. Subdivisions

Coastal engineering has already been subdivided into main divisions in the general introduction. Here we shall summarize the technical content of each of these divisions.

2.4. Harbors

Harbors have developed along with man's desire to move goods by ship. It is important to develop harbors in such a way that they are both convenient and economical from all points of view. This must obviously result in a compromise. These aspects are treated primarily in volume II. The cooperation of naval architects and mariners is often very helpful when considering this optimization problem.

Many harbor entrances are protected by some form of breakwater; the design of these structures is the main topic of volume III of these notes.

Since many harbors are situated in river mouths or natural estuaries, the formation of shoals and channels in tidal rivers is often included in coastal engineering. Obviously, this aspect is also closely related to river engineering. Special attention is paid to the influence of density currents and time dependent salinity variations on the behavior of silt in harbors. Density currents are approached from a very practical viewpoint in these notes; fundamental theory is handled in other books and courses. The behavior of silt in harbors and river mouths can be of extreme importance since this mud can often dominate the dredging problems of the harbor and can occasionally even dominate the coastal morphology over a considerable distance as well. Harbor design problems are often closely linked to coastal morphological problems. Indeed, it is often impossible to separate these problems. Among the more significant morphological problems directly related to harbors are the siltation of approach channels and the influence of breakwaters on the coastal processes.

2.5. Coastal Morphology

Coastal morphology is the study of the interaction of waves and currents with the coast. Most often this coast will be formed from sandy material; these often respond the most rapidly to the influence of the waves and current. Rocky coasts usually respond very slowly to these influences and as such are more of concern to the geologist than to the coastal engineer. Why do coasts consisting of mud also respond relatively slowly to the action of waves and currents? This is answered in chapter 27 on Mud Coasts.

Luckily, the most common coastal material is sand. We are lucky because it can be moved rather easily by dredging and the changes which occur on sand coasts can be reasonably accurately predicted using mathematical models. These models are briefly described in this volume; more complete information is given in volume II.

It should be clear that one must first understand the motion of water (wave action and other currents) along a coast before he can predict morphological changes. Indeed, many concepts from hydraulics are needed; some of the more specialized topics are reviewed in the immediately following chapters.

The effect of waves and currents on beaches is still not completely understood. Longshore and on and offshore transport of sand is an important topic of coastal engineering research. Results of this research are continually being used to improve the mathematical models used to predict coastline changes.

Since not all natural coastal changes are desirable, coastal defense works can also be needed. Defense works are used to retard the natural coastal processes or, sometimes, simply to neutralize their effects. For example, groins can be constructed perpendicular or parallel to a coast to retard erosion. Another alternative is to artificially move sand from areas of accretion to areas of erosion. Coastal defense works will be considered later in this volume.

Not only harbor breakwaters and approach channels disturb the coastal morphology; natural rivers and estuaries do this as well. This is also discussed in detail later in this volume.

2.6. Offshore Engineering

Until recently, harbors and coastal morphology formed the main topics associated with "conventional" coastal engineering. In recent times man's interest in working at sea has increased rapidly. The offshore branch is developing rapidly as coastal engineers who have worked along relatively shallow coastlines have been asked to solve completely new problems in the deep sea. Indeed, the following chapter on oceanography is included because of an increasing need to understand the processes which take place in the deeper ocean waters. The primary stimulus for offshore engineering has come from the petroleum companies.

The term "offshore engineering" is used, here, to refer to engineering for works which have no direct connection to the mainland. Some people also refer to this topic as "ocean engineering" but the whole study area is too young to have developed a uniform terminology. Confusion of terminology is bound to result; for example, some marine engineers design offshore works while others design power plants for ships!

Ships underway do not have a connection to the mainland, but are still excluded from offshore engineering; these are left for the naval architects. On the other hand, possible impact loads upon offshore structures caused by ships can be very important to us.

The offshore engineer draws on the specialized knowledge of other fields. Mining engineering, Mechanical engineering, and Naval architecture can all contribute to offshore engineering along with Civil engineering. Here in Delft, these departments are now cooperating closely on an interdisciplinary program of offshore engineering.

3. OCEANOGRAPHY

W.W. Massie

3.1. Introduction

Oceanography is study of the oceans. Man has studied the oceans for centuries. Count L.F. Marsigli wrote one of the first books on the subject, published in 1725. A Dutch translation of this book was prepared in 1786 by Boerhaave; a copy exists in the Library of Leiden University.

M.F. Maury, a United States Naval Officer, wrote the first "modern" oceanography book in 1855 while he was Superintendent of the Naval Hydrographic Office. Many of his observations - compiled from ships logs - are excellent; all are interestingly explained, even though he had no knowledge of geophysics.

The first systematic, specific study of the oceans was carried out by the H.M.S. Challenger. The ship sailed from Portsmouth, England on 21 December 1872 and in 3½ years sailed more than 100,000 km producing a 50 volume report. This was also the first report to subdivide oceanography into its four modern major fields: biological, chemical, geological, and physical.

What is the importance of oceanography to the coastal engineer? This will be highlighted in the following more detailed descriptions of each field.

Biological Oceanography

Biological Oceanography concerns itself with living matter in the seas. Coastal engineers are seldom directly involved with biological problems, but biological factors can play important indirect roles. Marine fouling of structures and environmental impact studies can be important, for example.

Chemical Oceanography

The chemistry of sea water is obviously of great importance to the marine biologists but it is becoming more important to engineers concerning with structures in the sea as well. Materials used in construction in the oceans can behave in what seem like strange ways when exposed to sea water under a considerable pressure (depth); Concrete technologists worry about concrete in water depths of a few hundred meters. Special corrosion and fracture problems develop with steel at somewhat greater depths.

Geological Oceanography

The geologists who find commercially valuable minerals on the bottom of and under the sea are indirectly responsible for providing jobs for many coastal engineers. While coastal engineers are not expected to be geologists, themselves, they can certainly get preliminary information about possible foundation problems for a proposed offshore structure from marine geologists.

Physical Oceanography

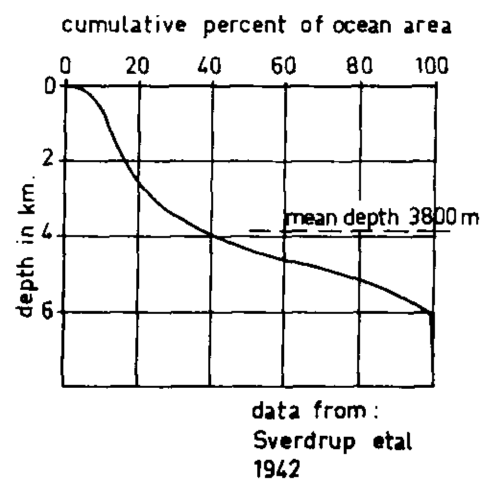
Physical oceanographers are most like the coastal engineers. Both worry about waves, tides and hydrodynamic problems in general. The concern with waves is interesting, if not serious. The oceanographers usually consider waves to be a necessary nusiance; coastal engineers, on the other hand, derive their most challenging problems from them. As offshore work progresses into still deeper water, coastal engineers must also begin to think about a topic which has, in the past, been restricted to physical oceanography: the location and strength of major ocean currents.

3.2. Description of the Oceans

A brief review of the physical features of the oceans will be helpful for our understanding of the dynamic processes which occur in the ocean.

Figure 3.1. shows the depth distribution of the oceans. The mean depth is about 3800 m. and the volume of the oceans is about 1370×10^{15} m^3. By contrast, the North Sea has a mean depth of 94 m and a water volume of 0.054×10^{15} m^3 - pretty insignificant! The shallowest 200 m of the ocean (7.6% of the total area) is called the continental shelf. Only recently have coastal engineers been asked to venture beyond the shelf to the continental slopes; hence, the need to know a bit more about oceanography, now. Shelves border most of the continental coasts and range in width up to about 1200 km.

Figure 3.1
DEPTH DISTRIBUTION OF THE OCEANS



The widest continental shelf is in the Arctic Ocean, north of Siberia; hardly any shelf is present along the west coast of the Americas (east coast of the Pacific Ocean).

The oceans are further divided into a series of interconnected basins in which most of the interesting physical oceanographic activity takes place. These basins are 3 to 5 km deep with occasional deeper or shallower spots.

Most of the interesting activity in the oceans takes place in the upper 1 to 2 km. Deeper than this, the oceans are of rather uniform salinity (35‰ - see section 3.6) and temperature (3^0 - 4^0 C). Also, currents in this deep zone are very weak - often assumed to be zero. Currents in the upper layers are discussed in the next sections, while the physical properties of sea water are treated separately in section 3.6.

3.3. Wind-Driven Ocean Currents

The major driving force for ocean currents results from the wind forces on the ocean surface. The trade winds and the prevailing westerlies result in a generally westward ocean current at low latitudes and an eastward current at high latitudes. This manifests itself in the North Atlantic in the following current pattern:

The North Equatorial Current flows westward from the Cape Verde Islands to the Caribbean Sea. A portion enters this sea and a portion turns northwest east of the Caribbean Islands(Antilles Current) and joins the Florida Current. Water flows out of the Caribbean between Florida and Cuba in the Florida Current. The Florida Current (often called the Gulf Stream) continues north along North America to about 45^0 N. latitude where it turns eastward and spreads out forming the North Atlantic Current. A branch of this turns south, along Portugal to form the Canary Current and close the circuit.

Similar current patterns can be found in the South Atlantic and the other oceans. These major east-west currents correspond in latitude to the prevailing winds. The north-south currents guarantee continuity and conservation of mass.

How do the winds generate these major east-west currents? This is answered later in this chapter but first, the dynamic equilibrium of a flowing ocean current will be examined.

3.4. Dynamics of Ocean Currents

The familiar balance of gravity and friction forces which leads to the well-known Chézy Equation which is used to describe river flows does not work in de deep oceans. Since the oceans are so deep and the velocities are normally small (less than 1 m/s), friction forces become relatively unimportant. On the other hand, since the ocean currents extend over great distances on the surface of a rotating earth, another influence, the Coriolis Force[*], does become important.

Consider a current moving with constant speed along a "straight" path. ("Straight" means that it follows a great circle path.) The Coriolis acceleration acting on a unit mass of this water is:

$$a_c = 2\,\Omega \sin\phi\, V$$

where:

a_c = the Coriolis acceleration
Ω = the angular velocity of the earth = 0.729×10^{-4}/s [†].
V = the current velocity, and
ϕ = the latitude

Further, this acceleration (or force per unit mass) acts toward the right facing in the flow direction in the northern hemisphere. (The direction is opposite south of the equator).

If this current is moving in a "straight" line, then the resultant acceleration perpendicular to the current direction must be zero. The Coriolis acceleration is balanced by a pressure gradient. This is a horizontal gradient also perpendicular to the current direction and counteracting the Coriolis acceleration. Equilibrium of these two components yields:

$$\frac{1}{\rho}\frac{\partial p}{\partial n} = 2\,\Omega \sin\phi\, V \qquad (3.02)$$

where:

ρ is the water density, and
$\frac{\partial p}{\partial n}$ is the pressure gradient normal to the current.

Density differences are not sufficient to cause this pressure gradient, but a water surface slope can, and does provide the equilibrium. Thus, there are differences in mean sea level between points on the ocean surface.

This is demonstrated by computing the mean sea level difference across the Strait of Florida (across the Florida Current). This is located at latitude 26^0 N., the current is about 1.0 m/s., and the strait is about 80 km wide.

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[*] A good review of Coriolis accelerations can be found in chapter 2 of Housner and Hudson (1959).

[†] This angular velocity is the absolute angular velocity based upon the siderial day.

$$\frac{1}{\rho}\,\frac{\partial p}{\partial n} = (2)(0.729 \times 10^{-4})(\sin 26^{o})(1.0)$$

$$= 6.4 \times 10^{-5}\ \frac{m}{sec^{2}}$$

In 80 km there is an elevation difference of:

$$\Delta z = \frac{6.4 \times 10^{-5}}{9.81} \times 80 \times 10^{3} = 52. \times 10^{-2}\ m.$$

This agrees reasonably well with an observed 45 cm value.

The currents just described are commonly called geostrophic currents.

Another interesting, (but less important from an oceanographic viewpoint) result can be obtained if we do allow our current to turn and let the horizontal pressure gradient be zero. In this case, the Coriolis acceleration is balanced by the centripetal acceleration.

$$\frac{V^2}{r} = 2\,\Omega \sin\phi\, V \qquad (3.03)$$

$$\frac{V}{r} = 2\,\Omega \sin\phi \qquad (3.04)$$

where r is the radius of curvature.

Currents of this sort cause little more than minor disturbances in oceanographic measurements; however, they can become a nusiance elsewhere. Such currents caused considerable problems in a sensitive hydraulic model at a lab in the U.S. some years ago. Perfectly quiet water without turbulence was required in a circular tank about 4 m in diameter. After filling the tank and letting it stand overnight, the investigator found a slow circulation current in the tank the next morning. Since the lab was located at latitude 45^{o} N, this current was 0.2 mm/s.

These currents just described are independent of depth; they are constant over the entire depth, since friction has been ignored. This contradicts the earlier observation that there is little activity in the ocean deeper than 1 to 2 km. Actually, there is no real contradiction here, since we have not yet discussed the cause of the geostrophic currents, the wind, which, of course, acts over the surface of the oceans.

3.5. Eckman Wind Drift

Nansen (1902) reported observations of the drift of sea ice in the North Polar Sea. He found that the ice drifted not in the wind direction, but at an angle of 20^{o} to 40^{o} from the wind. He explained this as resulting from the Coriolis effect and further speculated that the current in successively deeper ocean layers, driven by shear stresses from layers above, must deviate even more to the right.

Eckman investigated this mathematically on the suggestion of Nansen. His results, published also in 1902, will not be derived here. We shall concern ourselves only with the basic starting point and the result. His work was done for an infinite ocean (also infinitely deep) with a wind of constant speed and direction over the entire surface. The ocean surface remains horizontal; the only driving force comes from the wind shear stress. In the steady state, (no acceleration) this results in:

$$\frac{\varepsilon_z}{\rho} \frac{\partial^2 u}{\partial z^2} = +2 \Omega \sin \phi \, v \qquad (3.05)$$

$$\frac{\varepsilon_z}{\rho} \frac{\partial^2 v}{\partial z^2} = -2 \Omega \sin \phi \, u \qquad (3.06)$$

where:

- u is the velocity component along a horizontal x axis
- v is the velocity component along a horizontal y axis
- z is the vertical coordinate measured from the ocean surface (+ up), and
- ε_z is the vertical eddy viscosity coefficient.

The further mathematics is given by Neumann and Pierson (1966). When they assume that the wind blows only in the y direction, the shear stress at the water surface is:

$$\tau_s = \varepsilon_z \left.\frac{dv}{dz}\right|_{z = 0} \qquad (3.07)$$

and acts along the y axis.
This all results in the following:

$$u = V_s e^{\frac{\pi}{D} z} \cos \left(45^{\circ} + \frac{\pi}{D} z\right) \qquad (3.08)$$

$$v = V_s e^{\frac{\pi}{D} z} \sin \left(45^{\circ} + \frac{\pi}{D} z\right) \qquad (3.09)$$

which give the velocity components at any depth, once V_s, the velocity at the surface, and D are known.

$$D = \pi \sqrt{\frac{A_z}{\rho \, \Omega \sin \phi}} \qquad (3.10)$$

$$V_s = \frac{\pi \, \tau}{\sqrt{2D\rho \, \Omega \sin \phi}} \qquad (3.11)$$

Eckman calls D the "depth of frictional influence"; the depth over which the turbulent eddy viscosity is important. At this depth the velocity is about 1/23 of its value at the surface, and is directed in the opposite direction. This is in keeping with the hypothesis of Nansen mentioned earlier. D is normally about 50 meters, but increases very rapidly to ∞ at the equator.

Substitution of z = 0 into 3.08 and 3.09 yields a total velocity of magnitude V_s directed 45^0 to the right (in the northern hemisphere) of the wind direction.

The details of the current profile in three dimensions can be examined more conveniently by introducing polar coordinates.

$$V = V_s e^{\frac{\pi}{D} z} \tag{3.12}$$

$$\theta = 45^0 + \frac{\pi}{D} z \tag{3.13}$$

Indeed, the velocity, V, decreases exponentially with depth and the angle between the wind and current direction increases linearly with depth in a clockwise direction. The magnitude and direction of the resultant transport of ocean water is found by integrating 3.08 and 3.09 from z = -∞ to z = 0.

$$q_x = \frac{V_s \; D}{\pi \sqrt{2}} \tag{3.14}$$

$$q_y = 0 \tag{3.15}$$

where q_x and q_y are volume flow rates per unit of ocean width. The resultant transport is perpendicular in the wind direction!

This information does not seem too useful to us as coastal engineers. However, by allowing the ocean to have a coast, a surface slope, and a finite depth it is possible to begin* to attack the problem of predicting storm surges near coasts. Such prediction can be very important especially in light of the devastation that such surges can cause.

Eckman (1905) considered the problem of an enclosed sea of finite, constant depth. An important result is:

$$\beta = \Lambda \frac{\tau_s}{\rho g h} \tag{3.16}$$

---

* This is indeed still only a beginning. Influences of the barometric pressure changes and of complex bottom bathymetry are still being neglected.

where:

β = the water surface slope

h = the depth, and

Λ = a coefficient

Values of Λ vary between 1.0 for very deep water ($h \gg \pi \sqrt{\frac{\varepsilon_z}{\rho\,\Omega \sin\phi}}$), and 1.5 for shallow water or where Coriolis influences are neglected. According to Neumann and Pierson (1966) Coriolis forces can be neglected in wind set-up problems and the direction of the maximum surface gradient does not deviate more than 10° from the wind direction.

If, however, the depth of the body of water varies (as it generally does) and the influence of the storm surge itself on the depth is included then we are forced to carry out a brute force integration of:

$$\frac{dz'}{dx} = \frac{\varepsilon\tau}{\rho g z'} \tag{3.17}$$

where z' is now the depth measured from the actual water surface.

The solution to this is beyond the scope of these lecture notes. Hansen (1956) and Harris (1963) outline an approach to the problem.

3.6. Properties of Sea Water

The most important property of sea water from a coastal engineering point of view is its density. Its density is a function of three variables: salinity, temperature, and pressure. Of these, the pressure influence is least important and we can neglect it unless we are working at depths more than, say, 500 m.

In contrast to pure water, most sea water will continuously increase in density as it cools until it reaches its freezing temperature. Most sea water has a salinity varying between 34 and 36‰ (parts per thousand by weight). Some smaller isolated seas can have significant variations, however. The Baltic Sea, for example, sometimes has a salinity as low as 7‰. The Red Sea, on the other hand, has as much as 41‰ salinity.

Unfortunately, the dependence of density, ρ, on salinity S, and temperature, T, is not simple. Fisher, Williams, and Dial (1970) published an emperically derived equation for the specific volume, v, of water as a function of salinity, temperature, and pressure. Their equation is:

$$v = v_\infty - K_1 S + \frac{K_3}{K_4 + K_2 S + p} \tag{3.18}$$

in which:

K_1 is a temperature dependent coefficient having units of $\frac{cm^3}{g\text{‰}}$,

K_2 is a temperature dependent coefficient with units $\frac{bars}{\text{‰}}$ *

K_3 is a temperature dependent coefficient with units of $\frac{bars\ cm^3}{g}$,

K_4 is a temperature dependent coefficient with units of bars,

p' is the absolute pressure in bars,

S is the salinity in ‰

v is the specific volume in $\frac{cm^3}{g}$, and

v_∞ is a temperature dependent coefficient having units of $\frac{cm^3}{g}$

The five coefficients, K_1, K_2, K_3, K_4 and v_∞ are related to the temperature, T in degrees Celcius, by polynomial equations of form:

$$\sum_{i=0}^{N} a_i T^i$$

The coefficients, a_i, for these polynomials are given in table 3.1. Equation 3.18 is valid for the following ranges:

$$-2^o < T < 100^o\ C; \qquad 0 < p' < 1000 \text{ bars}; \quad 0 < S < 50\text{‰}$$

All of this makes equation 3.18 actually rather cumbersome in use. Therefore, Table 3.2 lists values of coefficients for equation 3.18 evaluated for various temperatures using table 3.1 and equation 3.19.

The water density in $\frac{kg}{m^3}$ can be determined from the specific volume of equation 3.18 as follows:

$$\rho = \frac{1}{v} \times 10^3 \qquad (3.20)$$

in which ρ is the density in kg/m^3.

---

* 1 bar is 10^6 dynes/cm^2 or a pressure of 10^6 N/m^2; about 0.987 atmosphere.

TABLE 3.1 Polynomial Coefficients a_i, for K_1, K_2, K_3, K_4 and V_∞

COEFFICIENT AND UNITS

	K_1	K_2	K_3	K_4	V_∞
i	$\frac{cm^3}{g‰}$	$(\frac{bars}{‰})$	$(\frac{bars\ cm^3}{g})$	(bars)	$\frac{cm^3}{g}$
0	2.679×10^{-4}	10.874	1733.316	5918.499	0.6980547
1	2.02×10^{-4}	-4.1384×10^{-2}	21.55053	58.05267	-7.435626×10^{-4}
2	-6.0×10^{-9}		-0.4695911	-1.1253317	3.704258×10^{-5}
3			3.096363×10^{-3}	6.6123869×10^{-3}	6.315724×10^{-7}
4			-7.341182×10^{-6}	$-1.4661625 \times 10^{-5}$	9.829576×10^{-9}
5					$-1.197269 \times 10^{-10}$
6					1.005461×10^{-12}
7					5.437898×10^{-15}
8					1.69946×10^{-17}
9					$-2.295063 \times 10^{-20}$

TABLE 3.2 Coefficients for

Eqn. 3.18 for various temperatures

COEFFICIENT AND UNITS

T	K_1	K_2	K_3	K_4	V_∞
(^oC)	$(\frac{cm^3}{g^o/oo})$	$(\frac{bars}{^o/oo})$	$(\frac{bars\ cm^3}{g})$	(bars)	$(\frac{cm^3}{g})$
0	2.6790×10^{-4}	10.87400	1788.316	5918.499	0.6980547
2	2.7192×10^{-4}	10.79123	1829.563	6030.156	0.6967108
4	2.7588×10^{-4}	10.70846	1867.201	6133.124	0.6956351
6	2.7930×10^{-4}	10.62570	1901.373	6227.712	0.6948023
8	2.8368×10^{-4}	10.54293	1932.222	6314.225	0.6941902
10	2.8750×10^{-4}	10.46016	1959.885	6392.958	0.6937790
12	2.9128×10^{-4}	10.37739	1984.500	6464.205	0.6935516
14	2.9500×10^{-4}	10.29462	2006.198	6528.253	0.6934924
16	2.9868×10^{-4}	10.21186	2025.111	6585.380	0.6935878
18	3.0232×10^{-4}	10.12909	2041.365	6635.864	0.6938257
20	3.0590×10^{-4}	10.04632	2055.086	6679.793	0.6941953
22	3.0944×10^{-4}	9.96355	2066.396	6717.971	0.6946869
24	3.1292×10^{-4}	9.88078	2075.413	6750.117	0.6952918
26	3.1636×10^{-4}	9.79802	2082.253	6776.663	0.6960021
28	3.1976×10^{-4}	9.71525	2087.030	6797.857	0.6968106
30	3.2310×10^{-4}	9.63248	2089.855	6813.939	0.6977110
32	3.2640×10^{-4}	9.54971	2090.836	6825.146	0.6986973
34	3.2964×10^{-4}	9.46694	2090.076	6831.707	0.6997638
36	3.3284×10^{-4}	9.38418	2087.679	6833.847	0.7009056
38	3.3600×10^{-4}	9.30141	2083.743	6831.785	0.7021179
40	3.3910×10^{-4}	9.21864	2078.365	6825.734	0.7033962

Since the density of salt water is usually a bit more than 1000 kg/m^3, Oceanographers often subtract 1000 from the density values and denote the value by sigma. If this is done for atmospheric pressure, then a subscript t is usually added. Thus:

$$\sigma_t = \rho - 1000 \tag{3.21}$$

in which ρ is evaluated at atmospheric pressure.

Values of σ_t as a function of salinity and temperature are listed in table 3.3. These tables were computed using equation 3.18 with $p'' = 1.0133$ bars = 1 atmosphere.

Since the equations (and their resulting tables) are a bit cumbersome in use, the Delft Hydraulics Laboratory uses a simpler relationship. In the notation already used,

$$\sigma_t = 0.75\ S \tag{3.22}$$

Equation 3.22 neglects influences of temperature and pressure and is therefore more limited in use than equation 3.18. In practice, civil engineers will sometimes find equation 3.22 to be sufficient for problems in which density differences result exclusively from salinity differences and the water temperature is not extreme.

With this information on density we can return briefly to the description of the oceans, themselves. Usually, both salinity and temperature decrease with increasing depth in the ocean. Evaporation is responsible for the higher salinity of the surface layer; how can this float on less saline deeper water? The temperature differences are sufficient to maintain a density profile which increases with depth.

Density variations caused by differences in salinity and temperature can be used in ingenious ways such as to drive a salt fountain, made in the following way:

We take a long (1 km) pipe and extend it vertically down from the ocean surface. Next, we attach a pump and slowly draw up the deep water. We do this slowly so that the water rising in the pipe can be warmed by the surrounding ocean. After deep water reaches the surface we remove the pump and find that the water continues to flow. Why does it flow? No, it is not perpetual motion; the process stops as soon as the upper 1 km layer of the ocean has become mixed.

Currents caused by density differences are discussed in the next section and again, in detail, in chapter 22.

TABLE 3.3 σ_t as Function of T and S

TEMPERATURE IN °C

S ‰	0	2	4	6	8	10	12	14	16	18	20	22	24	26	28	30	32	34	36	38	40
0	-0.16	-0.06	-0.03	-0.06	-0.15	-0.30	-0.50	-0.75	-1.06	-1.40	-1.79	-2.23	-2.70	-3.21	-3.76	-4.35	-4.97	-5.62	-6.31	-7.03	-7.78
1	+0.66	+0.75	+0.78	+0.74	+0.64	+0.49	+0.28	+0.02	-0.28	-0.63	-1.03	-1.47	-1.94	-2.46	-3.01	-3.60	-4.22	-4.88	-5.57	-6.29	-7.04
2	1.48	1.57	1.58	1.54	1.44	1.28	1.06	0.80	+0.49	+0.14	-0.26	-0.70	-1.18	-1.70	-2.26	-2.85	-3.48	-4.13	-4.83	-5.55	-6.30
3	2.30	2.38	2.39	2.34	2.23	2.06	1.85	1.58	1.26	0.90	+0.50	+0.06	-0.43	-0.95	-1.51	-2.10	-2.73	-3.39	-4.08	-4.81	-5.56
4	3.12	3.19	3.19	3.14	3.02	2.85	2.63	2.36	2.04	1.67	1.27	0.82	+0.33	-0.19	-0.75	-1.35	-1.98	-2.64	-3.34	-4.07	-4.83
5	3.94	4.00	4.00	3.94	3.81	3.64	3.41	3.13	2.81	2.44	2.03	1.58	1.09	+0.56	0.00	-0.60	-1.23	-1.90	-2.60	-3.33	-4.09
6	4.76	4.81	4.80	4.73	4.61	4.42	4.19	3.91	3.58	3.21	2.80	2.34	1.85	1.32	+0.75	+0.15	-0.49	-1.15	-1.86	-2.59	-3.35
7	5.58	5.63	5.61	5.53	5.40	5.21	4.97	4.69	4.35	3.98	3.56	3.10	2.61	2.07	1.50	0.90	+0.26	-0.41	-1.11	-1.85	-2.61
8	6.40	6.44	6.41	6.33	6.19	6.00	5.75	5.46	5.13	4.75	4.32	3.86	3.36	2.83	2.25	1.65	1.01	+0.33	-0.37	-1.11	-1.87
9	7.22	7.24	7.21	7.12	6.98	6.78	6.53	6.24	5.90	5.51	5.09	4.62	4.12	3.58	3.01	2.40	1.75	1.08	+0.37	-0.36	-1.13
10	8.03	8.05	8.02	7.92	7.77	7.57	7.31	7.01	6.67	6.28	5.85	5.38	4.88	4.34	3.76	3.15	2.50	1.82	1.12	+0.38	-0.39
11	8.85	8.86	8.82	8.71	8.56	8.35	8.09	7.79	7.44	7.05	6.62	6.14	5.64	5.09	4.51	3.90	3.25	2.57	1.86	1.12	+0.35
12	9.66	9.67	9.62	9.51	9.35	9.13	8.87	8.56	8.21	7.82	7.38	6.90	6.39	5.84	5.26	4.64	4.00	3.31	2.60	1.86	1.08
13	10.48	10.48	10.42	10.30	10.14	9.92	9.65	9.34	8.98	8.58	8.14	7.66	7.15	6.60	6.01	5.39	4.74	4.06	3.34	2.60	1.82
14	11.29	11.28	11.22	11.10	10.93	10.70	10.43	10.11	9.75	9.35	8.91	8.42	7.91	7.35	6.76	6.14	5.49	4.80	4.09	3.34	2.56
15	12.10	12.09	12.02	11.89	11.71	11.48	11.21	10.89	10.52	10.11	9.67	9.18	8.66	8.11	7.52	6.89	6.24	5.55	4.83	4.08	3.30
16	12.92	12.90	12.82	12.69	12.50	12.27	11.99	11.66	11.29	10.88	10.43	9.94	9.42	8.86	8.27	7.64	6.98	6.29	5.57	4.82	4.04
17	13.73	13.70	13.62	13.48	13.29	13.05	12.76	12.43	12.06	11.65	11.19	10.70	10.18	9.61	9.02	8.39	7.73	7.04	6.31	5.56	4.78
18	14.54	14.50	14.41	14.27	14.08	13.83	13.54	13.21	12.83	12.41	11.96	11.46	10.93	10.37	9.77	9.14	8.48	7.78	7.06	6.30	5.52
19	15.35	15.31	15.21	15.06	14.86	14.61	14.32	13.98	13.60	13.18	12.72	12.22	11.69	11.12	10.52	9.89	9.22	8.53	7.80	7.04	6.26
20	16.16	16.11	16.01	15.85	15.65	15.39	15.10	14.75	14.37	13.94	13.48	12.98	12.45	11.88	11.27	10.64	9.97	9.27	8.54	7.78	7.00
21	16.97	16.91	16.81	16.64	16.43	16.18	15.87	15.53	15.14	14.71	14.24	13.74	13.20	12.63	12.02	11.38	10.71	10.01	9.28	8.52	7.74
22	17.78	17.72	17.60	17.44	17.22	16.96	16.65	16.30	15.91	15.47	15.00	14.50	13.96	13.38	12.77	12.13	11.46	10.76	10.03	9.26	8.47
23	18.59	18.52	18.40	18.23	18.00	17.74	17.42	17.07	16.67	16.24	15.77	15.26	14.71	14.14	13.52	12.88	12.21	11.50	10.77	10.01	9.21
24	19.40	19.32	19.19	19.02	18.79	18.52	18.20	17.84	17.44	17.00	16.53	16.02	15.47	14.89	14.28	13.63	12.95	12.25	11.51	10.75	9.95
25	20.20	20.12	19.99	19.80	19.57	19.30	18.98	18.61	18.21	17.77	17.29	16.77	16.22	15.64	15.03	14.38	13.70	12.99	12.25	11.49	10.69
26	21.01	20.92	20.78	20.59	20.36	20.08	19.75	19.38	18.98	18.53	18.05	17.53	16.98	16.39	15.78	15.13	14.45	13.74	13.00	12.23	11.43
27	21.81	21.72	21.58	21.38	21.14	20.86	20.53	20.15	19.74	19.30	18.81	18.29	17.74	17.15	16.53	15.88	15.19	14.48	13.74	12.97	12.17
28	22.62	22.52	22.37	22.17	21.92	21.63	21.30	20.93	20.51	20.06	19.57	19.05	18.49	17.90	17.28	16.62	15.94	15.23	14.48	13.71	12.91
29	23.43	23.32	23.16	22.96	22.71	22.41	22.07	21.70	21.28	20.82	20.33	19.81	19.25	18.65	18.03	17.37	16.69	15.97	15.23	14.45	13.65
30	24.23	24.12	23.95	23.75	23.49	23.19	22.85	22.47	22.05	21.59	21.09	20.56	20.00	19.41	18.78	18.12	17.43	16.72	15.97	15.19	14.39
31	25.03	24.91	24.75	24.53	24.27	23.97	23.62	23.24	22.81	22.35	21.85	21.32	20.76	20.16	19.53	18.87	18.18	17.46	16.71	15.93	15.13
32	25.84	25.71	25.54	25.32	25.05	24.75	24.40	24.01	23.58	23.11	22.61	22.08	21.51	20.91	20.28	19.62	18.93	18.20	17.45	16.68	15.87
33	26.64	26.51	26.33	26.11	25.84	25.52	25.17	24.78	24.34	23.88	23.37	22.34	22.27	21.66	21.03	20.37	19.67	18.95	18.20	17.42	16.61
34	27.44	27.30	27.12	26.89	26.62	26.30	25.94	25.54	25.11	24.64	24.13	23.59	23.02	22.42	21.78	21.11	20.42	19.69	18.94	18.16	17.35
35	28.24	28.10	27.91	27.68	27.40	27.08	26.72	26.31	25.88	25.40	24.89	24.35	23.78	23.17	22.53	21.86	21.17	20.44	19.68	18.90	18.09
36	29.04	28.90	28.70	28.46	28.18	27.85	27.49	27.08	26.64	26.16	25.65	25.11	24.53	23.92	23.28	22.61	21.91	21.18	20.43	19.64	18.83
37	29.84	29.69	29.49	29.25	28.96	28.63	28.26	27.85	27.41	26.93	26.41	25.87	25.28	24.67	24.03	23.36	22.66	21.93	21.17	20.38	19.57
38	30.64	30.48	30.28	30.03	29.74	29.41	29.03	28.62	28.17	27.69	27.17	26.62	26.04	25.43	24.78	24.11	23.40	22.67	21.91	21.12	20.31
39	31.44	31.28	31.07	30.81	30.52	30.18	29.80	29.39	28.94	28.45	27.93	27.38	26.79	26.18	25.53	24.86	24.15	23.42	22.66	21.87	21.05
40	32.24	32.07	31.86	31.60	31.30	30.96	30.58	30.16	29.70	29.21	28.69	28.14	27.55	26.93	26.28	25.60	24.90	24.16	23.40	22.61	21.79

3.7 Density Currents

Horizontal density gradients can also lead to unbalanced pressure forces which result in a current. The mechanics of such currents is the same in a harbor on a tidal river as in the oceans. In chapter 22 of this book, the mathematical details will be explained; here, we shall only describe a significant example which we find in the oceans.

The Mediterranian Sea is more saline, and hence more dense than the Atlantic Ocean. A permanent current in the order of ½ m/s flows outward through the deeper portions of the Strait of Gibraltar. At the the surface, an even stronger current flows inward. The density difference which drives this current is maintained by the evaporation from the Mediterranian Sea.

4. Beaufort Wind Scale

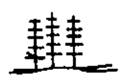
E.W. Bijker

In 1806 Admiral Beaufort of the British Navy devised a wind speed scale which would be helpful to sailors on the large sailing ships of that time, especially the larger men-of-war. On this scale, zero denotes no wind and twelve is the maximum; the scale is shown more explicitly in figure 4.1.

Captains of the large warships were often faced with a difficult choice: if they were cautious with the sails, they would preserve the ship, but might not catch their enemy or could be caught. If, on the other hand, they carried too much sail they had a better chance in battle, but ran a great risk of losing their masts and rigging (and possibly even their ship). Obviously, neither of these extremes is good for a career as navy officer. A bit of this controversy as commander is reflected in the racing sailor's description in the table.

This Beaufort Scale is still in common use, although slight variations in the wind speed limits of each scale division are possible.

TABLE 4.1 BEAUFORT WIND FORCE SCALE

Beaufort Number	Knots	miles per hr. (U.S. Statute)	meters per sec.	km per hr.	Wind Press. N/m^2	Beaufort description for square rigged ships 1806		Racing Sailor's description (C.A. Marchay, 1964)	U.S. Weater Service description	Dutch KNMI description	Beaufort Number
	WIND SPEEDS										
0	0 – 1		0 – 0.5	0 – 2					Calm	Windstil	0
1	1 – 3	1 – 3	0.5 – 1.5	2 – 6	0.14 – 1.4	Just Steerage Way		Boredom	Light air		1
2	4 – 6	4 – 7	2.1 – 3.1	7 – 11	2.4 – 5.7	1-3 knots close hauled		Mild pleasure	Light breeze	zwakke	2
3	7 – 10	8 – 12	3.6 – 5.1	13 – 19	7.7 – 16	4-5 knots close hauled		Pleasure	Gentle breeze		3
4	11 – 16	13 – 18	5.7 – 8	20 – 30	19 – 41	6-7 knots close hauled		Great Pleasure	Moderate breeze	matige	4
5	17 – 21	19 – 24	9 – 11	32 – 39	46 – 67	Hull Speed Full Sail		Delight	Fresh breeze	vrij krachtige	5
6	22 – 27	25 – 31	11 – 14	41 – 50	77 – 115			Delight tinged with anxiety	Strong breeze	krachtige	6
7	28 – 33	32 – 38	14 – 17	52 – 61	125 – 172			Anxiety tinged with fear	Moderate Gale	harde	7
8	34 – 40	39 – 46	18 – 21	63 – 74	182 – 250			Fear tinged with terror	Gale	stormachtige	8
9	41 – 47	47 – 54	21 – 24	76 – 87	270 – 350			Great terror	Strong Gale	storm	9
10	48 – 55	55 – 63	25 – 28	89 – 102	360 – 480			Panic	Whole Gale	zware storm	10
11	56 – 63	64 – 75	29 – 33	104 – 120	500 – 630			I want my mummy!!	Storm	zeer zware storm	11
12	above 63	above 75	above 33	above 120	above 630	bare poles		Yes, Mr. Jones	Hurricane	orkaan	12

Storm warnings are usually issued for winds stronger than Beaufort force 6

5. SHORT WAVES THEORY

W.W. Massie

5.1. Introduction

Some knowledge of the mechanics of short waves is essential for the good understanding of coastal engineering. Since the theory of short waves is not a prerequisite to this course, the more important wave relationships are given in this section. Derivations are not given; these may be found either in specialized courses in short wave theory or in the literature. Kinsman (1965) presents an excellent overview of short wave theory in a very readable fashion.

All of the results presented in this section have been derived using the Airy theory for a linear, sinusoidal wave form. "Ocean waves are not sinusoidal!" one will argue who has ever experienced the actual sea. This is true, but enough important properties of even irregular waves can be discovered by studying a single sinusoidal wave which does not break. This wave will be considered to be two dimensional: it will move along the horizontal x axis while the vertical z axis (positive upward) will have its origin at the still water surface.

5.2. General Relationships

Observation of a float on the surface of waves reveals that its position oscillates horizontally and vertically about a fixed position. This may seem strange since the wave profile moves forward past the float with a definite velocity. Obviously, the velocity of the float (water particle velocity) and the velocity with which the crest moves (phase velocity or wave celerity) are quite different. Let us first examine the motion of the float.

Water Particle Velocities

The horizontal and vertical water particle velocity components are given by:

$$u = \frac{\omega H}{2} \frac{\cosh k(z+h)}{\sinh kh} \cos(kx-\omega t) \qquad (5.01)$$

$$w = \frac{\omega H}{2} \frac{\sinh k(z+h)}{\sinh kh} \sin(kx-\omega t) \qquad (5.02)$$

where:
- H is the wave height
- h is the water depth
- k is the wave number $= \frac{2\pi}{\lambda}$
- λ is the wave length
- t is the time
- u is the instantaneous horizontal particle velocity
- w is the instantaneous vertical particle velocity
- x is the horizontal coordinate

z is the vertical coordinate measured from the still water surface (+ up)

ω is the circular frequency = $\frac{2\pi}{T}$

T is the wave period

Substitution of z = 0 into equations 5.01 and 5.02, yields the instantaneous velocity components of the float.

Water particle displacements

The amplitude of the displacement of the float can be determined by integrating the velocity with respect to time. This yields:

$$\hat{\xi} = \frac{H}{2} \frac{\cosh k\,(z+h)}{\sinh kh} \tag{5.03}$$

$$\hat{\zeta} = \frac{H}{2} \frac{\sinh k\,(z+h)}{\sinh kh} \tag{5.04}$$

where: $\hat{\xi}$ is the horizontal displacement amplitude,
$\hat{\zeta}$ is the vertical displacement amplitude, and
$\hat{}$ denotes "amplitude of".

These define the semi-axes of ellipses. The water particles move along elliptical paths; the size of these ellipses is greatest at the water surface and decreases as the observer moves deeper.

Wave Speed

The speed at which a wave crest moves forward is given by:

$$c = \frac{\lambda}{T} = \frac{\omega}{k} = \sqrt{\frac{g}{k} \tanh kh} \tag{5.05}$$

where: g is the acceleration of gravity, and
c is the wave celerity, or phase speed.

Equation 5.05 is a bit complicated to use in practice. Indeed, since both λ and k are dependent upon the answer, c, we cannot blindly substitute values into this equation for a simple solution. Therefore, the solution of this equation is taken up in section 6 again, where various tricks for its solution are explained.

If, for a moment, we examine a finite number (group) of waves propagating in otherwise still water, we will observe that waves seem to originate at the rear of the group, move forward through the group with speed c, and die out near the front of the group. Certainly this group moves forward as well, but with a smaller speed. The speed with which this group moves forward is given by:

$$c_g = \frac{c}{2} \left(1 + \frac{2\,kh}{\sinh 2\,kh}\right) \tag{5.06}$$

or

$$\frac{c_g}{c} = \frac{1}{2}\left(1 + \frac{2\,kh}{\sinh 2\,kh}\right) = n \tag{5.07}$$

As is indicated in equation 5.07, the ratio of group velocity to phase velocity is often denoted by n.

Wave energy

The energy contained in a wave of unit width (crest length) is:

$$E_T = \frac{1}{8}\rho\, g\, H^2\, \lambda \tag{5.08}$$

where ρ is the mass density of water.

Often, it is more convenient to express energy in terms of energy per unit of water surface area.

$$E = \frac{1}{8}\rho\, g\, H^2 \quad * \tag{5.09}$$

This energy is propagated with the wave group speed, c_g.

Wave Power

Since power is energy per unit time one might attempt to find the power of waves by dividing equation 5.08 by the wave period. Unfortunately, this is incorrect since it was just pointed out that the energy moves forward with the group velocity. Thus, the correct relationship is:

$$U = E\, c_g = E\, n\, c \tag{5.10}$$

where U is the power per unit crest length.

Wave pressure

The presence of the waves shall influence the pressure within our body of water. The pressure under the waves is given by:

$$p = -\rho g z + \frac{\rho g H}{2}\,\frac{\cosh k(z+h)}{\cosh kh}\,\cos(kx-\omega t) \tag{5.11}$$

where p is the instantaneous pressure.

The first term on the right of equation 5.11 is the pressure which would be present in still water. The second term describes the variation in pressure caused by the waves. This pressure variation can be very important when designing a structure to be placed in the sea.

* The reader should verify for himself that the dimensions of equation 5.09 are correct.

5.3. Simplifications

Equations 5.01 through 5.11 can be simplified when certain conditions are satisfied. This will be attempted via the hyperbolic functions. The behavior of the hyperbolic functions is shown in figure 5.1.

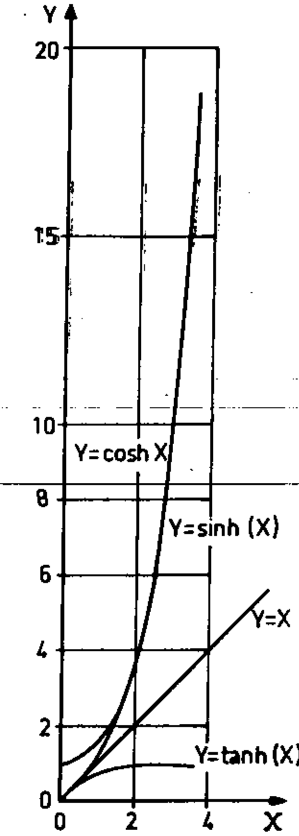


Figure 5.1
BEHAVIOR OF HYPERBOLIC FUNCTIONS

5.4. Approximations for Deep Water

For relatively, deep water ($h > \frac{\lambda}{2}$ *; therefore, $X > \pi$ in figure 5.1):

$$\sinh X \approx \cosh X \gg X \tag{5.12}$$

$$\tanh X \approx 1.0 \tag{5.13}$$

Now, substituting this and doing a bit of algebra with equations 5.01 through 5.11, we get:

$$u_o = \frac{\omega H_o}{2} e^{k_o z} \cos(k_o x - \omega t) \tag{5.01a}$$

$$w_o = \frac{\omega H_o}{2} e^{k_o z} \sin(k_o x - \omega t) \tag{5.02a}$$

$$\hat{\xi}_o = \frac{H_o}{2} e^{k_o z} \tag{5.03a}$$

$$\hat{\zeta}_o = \frac{H_o}{2} e^{k_o z} \tag{5.04a}$$

$$c_o = \frac{\lambda_o}{T} = \frac{\omega}{k_o} = \frac{g}{2\pi} T \tag{5.05a}$$

$$c_{g_o} = \frac{c_o}{2} \tag{5.06a}$$

$$n_o = \frac{1}{2} \tag{5.07a}$$

$$E_{T_o} = \frac{1}{8} \rho g H_o^2 \lambda_o \tag{5.08a}$$

$$E_o = \frac{1}{8} \rho g H_o^2 \tag{5.09a}$$

$$U_o = E_o n_o c_o \tag{5.10a}$$

$$p_o = -\rho g z + \frac{\rho g H_o}{2} e^{k_o z} \cos(k_o x - \omega t) \tag{5.11a}$$

The subscript o has been added to denote deep water conditions; this is fairly common in the literature. This has not been done with T or ω since these parameters remain constant.

* We shall re-examine this criteria in section 5.7.

Substituting values for g and π in equation 5.05a, we get:

$$\left.\begin{array}{lll} c_0 = 1.56\ T & \text{in m kg s units and} \\ c_0 = 5.12\ T & \text{in ft lb s units.} \end{array}\right\} \quad (5.14)$$

From the same equation, it also follows that:

$$\left.\begin{array}{lll} \lambda_0 = 1.56\ T^2 & \text{in m kg s units and} \\ \lambda_0 = 5.12\ T^2 & \text{in ft lb s units.} \end{array}\right\} \quad (5.15)$$

Thus, in deep water, we avoid all the headaches of computing the wave speed using the full equation 5.05.

Note from equations 5.03a and 5.04a that the elliptical particle paths have reduced to circles which decrease in radius exponentially as one moves deeper in the water. Figure 5.2 shows this orbital motion in a deep water wave. In this figure, also, the water depth is equal to half the wave length; under these conditions, the ratio of particle displacement on the bottom to that at the water surface is $e^{-\pi} = 0.043$.

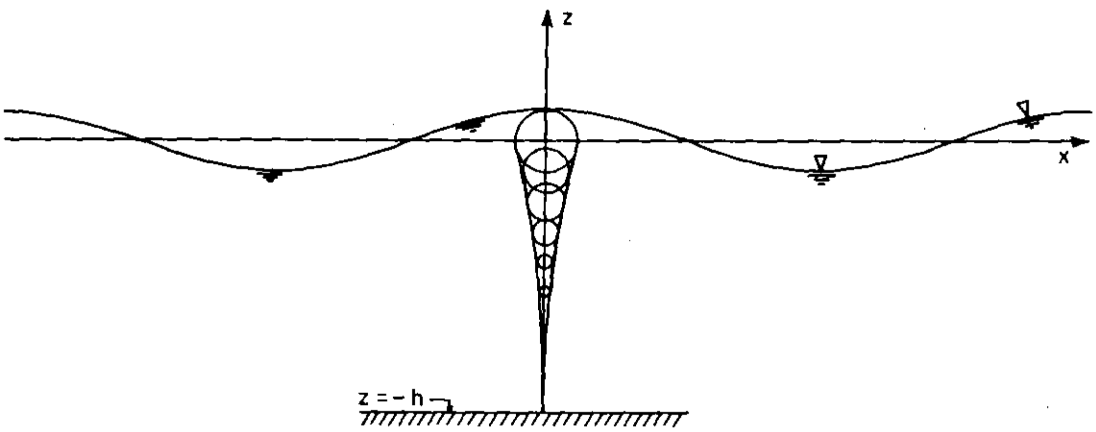


Figure 5.2
ORBITAL MOTION UNDER A DEEP WATER WAVE

5.5. Approximations for Shallow Water

Another set of simplifying approximations can be substituted when the water is relatively shallow ($h < \frac{\lambda}{25}$; $X < 0.25$ in figure 5.1):*

$$\sinh kh \simeq \tanh kh \simeq kh \tag{5.16}$$

$$\cosh kh \simeq 1 \tag{5.17}$$

Again, using these and a bit of algebra in equations 5.01 through 5.11, we get:

$$u = \frac{\omega H}{2kh} \cos(kx - \omega t) \tag{5.01b}$$

$$w = \frac{\omega H}{2} (1 + \frac{z}{h}) \sin(kx - \omega t) \tag{5.02b}$$

$$\hat{\xi} = \frac{H}{2kh} \tag{5.03b}$$

$$\hat{\zeta} = \frac{H}{2}(1 + \frac{z}{h}) \tag{5.04b}$$

$$c = \frac{\lambda}{T} = \frac{\omega}{k} = \sqrt{gh} \tag{5.05b}$$

$$c_g = \frac{c}{2}(1 + 1) = c \tag{5.06b}$$

$$n = 1 \tag{5.07b}$$

$$E_T = \frac{1}{8} \rho g \, H^2 \lambda \tag{5.08b}$$

$$E = \frac{1}{8} \rho g \, H^2 \tag{5.09b}$$

$$U = Ec \tag{5.10b}$$

$$p = -\rho g z + \frac{\rho g H}{2kh} \cos(kx - \omega t) \tag{5.11b}$$

The wave phase velocity is now found to be independent of the wave period; it depends only upon the water depth. Further, the group velocity is equal to the phase velocity, and the horizontal particle velocity, u, is independent of the vertical position, z. Indeed, these equations are the same as those used for long waves.

The wave length can easily be computed using equation 5.05b:

$$\lambda = \sqrt{gh} \; T \tag{5.18}$$

* We re-examine this limit, also, in section 5.7.

Once again, the simple form of equation 5.05b has eliminated the problems associated with the evaluation of equation 5.05.

Figure 5.3 shows the orbital motion under a shallow water wave. In this figure $h = \lambda/25$.

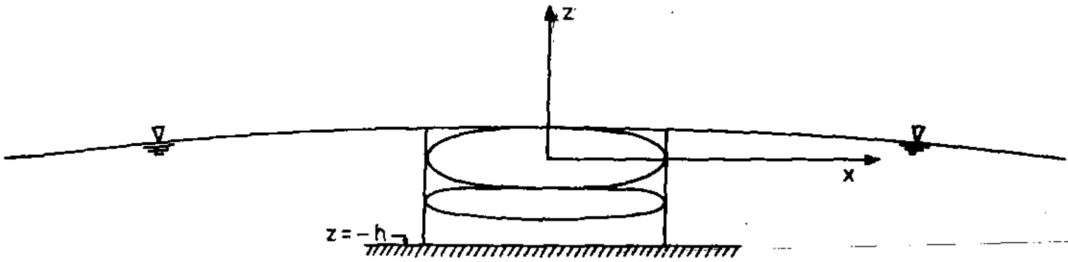


Figure 5.3
ORBITAL MOTION UNDER A SHALLOW WATER WAVE

5.6. Intermediate water depths

For water of all intermediate relative depths $(\frac{\lambda}{25} < h < \frac{\lambda}{2})$*
we are forced to work with the complete equations 5.01 through 5.11. Water particles move along elliptical paths which are nearly circles at the water surface and degenerate both horizontally and vertically to short horizontal lines at the bottom.

Since the use of equations 5.01 through 5.11 is impossible directly when only a water depth, h, wave period, T, and a wave height, H, are known (a very practical situation); special attention will be paid to this problem in section 6.

5.7. A Critical Re-examination

Some important practical questions remain. The first is, "what wave length should be substituted into the ratio of h/λ to determine whether to use shallow, intermediates or deep water wave theory?" One answer is to say, "Use the actual wave length at that depth". This is not too bad, since the wave lengths in deep and shallow water can be computed quite easily using either equation 5.15 or 5.18 respectively. Another approach - which leads to quite different results! - is to always use the deep water wave length, λ_0 from equation 5.15.

Another important related question is:"What is so sacred about the suggested values of h/λ?"

Nothing!!!

---

* subject to review in the next section.

There is certainly discussion and perhaps even disagreement about what these limits should be. Kinsman (1965) pp 129-133 points this out rather colorfully. As he indicates, there are two basic criteria for determining the acceptable accuracy for an approximation: a mathematician's, and an engineer's. The mathematician, worried about computational accuracy accepts an error of the approximation of about 0.5%. The engineer, on the other hand, is more aware of his other limitations and is happy with 5% accuracy. Now, perhaps, we can make a more intelligent appraisal of both our questions.

Table 5.1 lists the relative water depth limits for deep and shallow water according to various criteria.

TABLE 5.1. Comparison of $\frac{h}{\lambda_o}$ and $\frac{h}{\lambda}$ for various criteria

	$\frac{h}{\lambda_o}$	$\frac{h}{\lambda}$
for deep water		
section 5.4:	$\frac{1}{2.01}$	$\frac{1}{2}$
Mathematician's	$\frac{1}{2}$	$\frac{1}{1.99}$
Engineer's	$\frac{1}{4}$	$\frac{1}{3.73}$
for shallow water		
Mathematician's	$\frac{1}{200}$	$\frac{1}{35}$
section 5.5	$\frac{1}{102}$	$\frac{1}{25}$
	$\frac{1}{25}$	$\frac{1}{12}$
Engineer's	$\frac{1}{20}$	$\frac{1}{11}$

Thus, for deep water, the criteria as stated in section 5.4 is very conservative: $h > \frac{\lambda_o}{4}$ would seem more appropriate. For shallow water, on the other hand, the limit criteria given in section 5.5 is not overly conservative, but could still be made a bit more flexible. In keeping with Kinsman's discussion, $h < \frac{\lambda_o}{20}$ is suggested as a limit. Adoption of these suggested limits will greatly reduce the range of relative depths for which the full equations 5.01 through 5.11 must be used, while keeping our error usually less than about 5%.

5.8. Examples

First, let us examine some "typical" waves, and then, using some rather extreme examples, we shall observe that the relative depth $\frac{h}{\lambda_o}$ is more important than the absolute depth.

1. North Sea, H = 0.8 m, T = 8 seconds, and h = 10 m. (this is a very common wave on the North Sea). From 5.15,

   $\lambda_o = (1.56)(8^2) = 100$ m; $\frac{h}{\lambda_o} = \frac{10}{100} = \frac{1}{10}$;

   this is intermediate water depth, we are stuck until after chapter 6. Note, the wave height, H, was not used here at all.

2. Straight of Gibraltar, H = 25 m, T = 15 sec. and h = 1000 m. (This is a severe storm wave in that area). From 5.15,

   $\lambda_o = (1.56)(15^2) = 351$ m; $\frac{h}{\lambda_o} = \frac{1000}{351} > \frac{1}{4}$;

   this is certainly deep water. We can determine the horizontal water particle velocity amplitude at a depth of 100 meters using equation 5.01a:

   $$\hat{U}_o = \left(\frac{2\pi}{15}\right)\left(\frac{25}{2}\right) e^{-\left(\frac{2\pi}{351}\right)(100)}$$

   . The cosine term is dropped for determining the amplitude. Evaluating this, we get:

   $\hat{U}_o = 5.24\, e^{-1.79} = 4.38$ m/s. The speed of this wave (from 5.14) is: $c_o = (1.56)(15) = 23.4$ m/s = 84 km/hr = 45.5 knots!

3. North Sea (Dutch Coast), H = 1.5 m, T = 8 sec., h = 4 m.

   $\lambda_o = (1.56)(8^2) = 100$ m; $\frac{h}{\lambda_o} = \frac{4}{100} = \frac{1}{25}$, this is shallow water.

   Therefore, from 5.05b, $c = \sqrt{(9.81)(4)} = 6.3$ m/s. The wave length is $c\,T = (6.3)(8) = 50$ m. The energy in the wave per unit crest length is (eqn. 5.08b):

   $$E_T = \left(\frac{1}{8}\right)(1030)(9.81)(1.5^2)(50) = 1.42 \times 10^7 \; \frac{N - m}{m}.$$

4. In a model we generate a wave with period 0.6 sec. in water 30 cm deep.

   $\lambda_o = (1.56)(0.6^2) = 0.56$ m; $\frac{h}{\lambda_o} = \frac{30}{56} > \frac{1}{2}$; this is deep water. The wave speed is $c_o = (1.56)(0.6) = 0.94$ m/sec.

6. WAVE SPEED AND LENGTH COMPUTATIONS

W.W. Massie

6.1. Introduction

For intermediate water depths ($\frac{\lambda_o}{20} < h < \frac{\lambda_o}{4}$), there is no simple, direct means of determining the wave length or other related parameters given only the wave period. Two methods are presented here, both are derived from the non-linear equation for wave speed, equation 5.05.

6.2. Iteration Method

Recall equation 5.05,

$$c = \sqrt{\frac{g}{k} \tanh kh} = \frac{\lambda}{T} \qquad (5.05) \quad (6.01)$$

in which c is the wave phase velocity

g is the acceleration of gravity

k is the wave number = $\frac{2\pi}{\lambda}$

h is the water depth

λ is the wave length

T is the wave period.

Substituting various definitions from chapter 5 into equation 6.01 yields:

$$\lambda = \lambda_o \tanh \frac{2\pi h}{\lambda} \qquad (6.02)$$

Since λ, the unknown, cannot be isolated on one side of this equation, a direct solution is impossible. Iterative solution schemes *are* possible. In fact most any iteration will eventually lead to the correct answer since the equation has only one solution for given values of λ_o and h.

One simple but rather inefficient iteration is to resubstitute successive answers from 6.02 (starting with $\lambda = \lambda_o$) into the right hand side of the equation. Thus:

$$\lambda_{i+1} = \lambda_o \tanh \frac{2\pi h}{\lambda_i} \qquad (6.03)$$

where i = 0, 1, 2, ...

A much more efficient iteration is the following:

$$\lambda_{2i+1} = \lambda_o \tanh \frac{2\pi h}{\lambda_{2i}}$$

$$\lambda_{2i+2} = \frac{\lambda_{2i+1} + \lambda_{2i}}{3} \qquad (6.04)$$

$$i = 0, 1, 2, \ldots\ldots.$$

While the algorithm is a bit more complex, it reduces the number of iterations considerably (three or four are usually more than sufficient) and can still be executed on many of the small pocket electronic calculators.

A direct technique attributed to Eckert (unpublished) which usually gives answers correct to within about 5 percent is simply:

$$\lambda = \lambda_o \sqrt{\tanh \frac{2\pi h}{\lambda_o}} \qquad (6.05)$$

Table 6.1 compares the results of these schemes.

Table 6.1. Wave length iterations

T = 19 seconds, h = 50 meters

	eqn. 6.03	eqn. 6.04
i	λ_i (m)	λ_{2i+2} (m)
0	563.8	378.1
1	285.2	382.0
2	451.6	381.6
3	339.2	381.6
4	410.9	
5	362.9	
6	394.2	
7	373.4	
8	387.0	
9	378.0	
10	384.0	
11	380.1	
12	382.6	
13	380.9	
14	382.0	
15	381.3	
16	381.8	
17	381.5	

The superiority of the second iteration scheme is obvious. For comparison purposes, equation 6.05 yields λ = 401.0 which is off by 5.1%.

Obviously, now that the wave length has been determined all of the other related parameters can be easily evaluated.

6.3. Use of Tables

The computations outlined in the previous section were often cumbersome to carry out by hand. For this reason an alternative was developed in the form of a set of tables. By dividing both sides of equation 6.02 into h and doing a bit of algebra:

$$\frac{h}{\lambda_o} = \frac{h}{\lambda} \tanh \frac{2\pi h}{\lambda} \qquad (6.06)$$

in which $\frac{h}{\lambda_o}$ has been conveniently expressed in terms of $\frac{h}{\lambda}$. Thus, by choosing various values of $\frac{h}{\lambda}$, corresponding values of $\frac{h}{\lambda_o}$ can be computed directly and tabulated. Interpolation in this table working either toward values of $\frac{h}{\lambda_o}$ or toward values of $\frac{h}{\lambda}$ is all that is necessary to determine the wave length.

Wiegel (1954) worked out such a table. It is also published in his book *Oceanographical Engineering* (1964) and in the *Shore Protection Manual* (1973). An abbreviated version of this table is included here as table 6.2.

As an example, the previous iteration schemes can be checked. T = 19 sec. and h = 50 m yields λ_o = 563.80 m, and $\frac{h}{\lambda_o}$ = 0.0887. Interpolating in Wiegel (1964) yields $\frac{h}{\lambda}$ = 0.1310 and λ = 381.6 which compares rather favorably to the earlier calculation.

TABLE 6.2. SINUSOIDAL WAVE FUNCTIONS

$\frac{h}{\lambda_o}$	tanh kh	$\frac{h}{\lambda}$	kh	sinh kh	cosh kh	$\frac{H}{H_o'}$
0.000	0.000	0.0000	0.000	0.000	1.00	∞
002	112	0179	112	113	01	2.12
004	158	0253	159	160	01	1.79
006	193	0311	195	197	02	62
008	222	0360	226	228	03	51
0.010	0.248	0.0403	0.253	0.256	1.03	1.43
015	302	0496	312	317	05	31
020	347	0576	362	370	07	23
025	386	0648	407	418	08	17
0.030	0.420	0.0713	0.448	0.463	1.10	1.13
035	452	0775	487	506	12	09
040	480	0833	523	548	14	06
045	507	0888	558	588	16	04
0.050	0.531	0.0942	0.592	0.627	1.18	1.02
055	554	0993	624	665	20	1.01
060	575	104	655	703	22	0.993
065	595	109	686	741	24	981
070	614	114	716	779	27	971
0.075	0.632	0.119	0.745	0.816	1.29	0.962
080	649	123	774	854	31	955
085	665	128	803	892	34	948
090	681	132	831	929	37	942
095	695	137	858	0.968	39	937
0.10	0.709	0.141	0.886	1.01	1.42	0.933
11	735	150	940	08	48	926
12	759	158	0.994	17	54	920
13	780	167	1.05	25	60	917
14	800	175	10	33	67	915
0.15	0.818	0.183	1.15	1.42	1.74	0.913
16	835	192	20	52	82	913
17	850	200	26	61	90	913
18	864	208	31	72	1.99	914
19	877	217	36	82	2.08	916
0.20	0.888	0.225	1.41	1.94	2.18	0.918

$\frac{h}{\lambda_o}$	tanh kh	$\frac{h}{\lambda}$	kh	sinh kh	cosh kh	$\frac{H}{H_o'}$
0.20	0.888	0.225	1.41	1.94	2.18	0.918
21	899	234	47	2.05	28	920
22	909	242	52	18	40	923
23	918	251	57	31	52	926
24	926	259	63	45	65	929
0.25	0.933	0.268	1.68	2.60	2.78	0.932
26	940	277	74	75	2.93	936
27	946	285	79	2.92	3.09	939
28	952	294	85	3.10	25	942
29	957	303	90	28	43	946
0.30	0.961	0.312	1.96	3.48	3.62	0.949
31	965	321	2.02	69	3.83	952
32	969	330	08	3.92	4.05	955
33	972	339	13	4.16	28	958
34	975	349	19	41	53	961
0.35	0.978	0.358	2.25	4.68	4.79	0.964
36	980	367	31	4.97	5.07	967
37	983	377	37	5.28	37	969
38	984	386	43	61	5.70	972
39	986	395	48	5.96	6.04	974
0.40	0.988	0.405	2.54	6.33	6.41	0.976
41	989	415	60	6.72	6.80	978
42	990	424	66	7.15	7.22	980
43	991	434	73	7.60	7.66	982
44	992	443	79	8.07	8.14	983
0.45	0.993	0.453	2.85	8.59	8.64	0.985
46	994	463	91	9.13	9.18	986
47	995	472	2.97	9.71	9.76	987
48	995	482	3.03	10.3	10.4	988
49	996	492	09	11.0	11.0	990
0.50	0.996	0.502	3.15	11.7	11.7	0.990
1.00	1.000	1.000	6.28	268	268	1.000
∞	1.000	∞	∞	∞	∞	1.000

7. EFFECTS OF SHOALING WATER

W.W. Massie

7.1. Introduction

Obviously, a wave breaks sometimes as it progresses from deep through intermediate depths to shallow water. Breaking will be considered later in this chapter and in chapter 8 as well. However, to begin with, consider a wave that is not yet broken, progressing into water which is gradually* becoming shallower. In order to keep things from becoming too complicated, the discussion is still restricted to a two-dimensional case. In a practical sense, this means that the depth contours run parallel to the wave crests. This restriction will not be relaxed until chapter 9.

7.2. Wave Height Changes

Since so many of the relationships in chapter 5 were dependent upon the wave height, H, it seems logical to study how H varies as a wave progresses into shallower - or back into deeper for that matter - water. The relationship between H and h, the water depth, is exposed by applying conservation of energy. The energy transported through a vertical plane parallel to the wave crests is, in fact, the wave power per unit of crest length. This is sometimes called energy flux. Anyway, from equation 5.10:

$$U = E\, c_g = E\, n\, c \qquad (5.10) \quad (7.01)$$

By assuming that this energy flux does not change as the wave progresses through water of varying depth, then:

$$U_2 = U_1 \qquad (7.02)$$

or

$$E_2\, n_2\, c_2 = E_1\, n_1\, c_1 \qquad (7.03)$$

where: c is the wave speed,
- E is the wave energy per unit of surface area,
- n is the ratio c_g/c,
- c_g is the group velocity,
- U is the power or energy flux, and
- 1,2 are subscripts indicating the location at which the parameters are evaluated.

Using equation 5.09 for E, and choosing location 2 to be deep water where the wave properties are easily evaluated, leads to:

$$\frac{1}{8}\rho g\, H_1^2\, n_1\, c_1 = \frac{1}{8}\rho g\, H_0^2\, n_0\, c_0 \qquad (7.04)$$

---

* "Gradually" means less than a few percent bottom slope.

Cancelling out a few unnecessary things and evaluating n_o from 5.07a:

$$H_1^2 n_1 c_1 = \frac{1}{2} H_o^2 c_o \tag{7.05}$$

In another form, this is:

$$\frac{H_1}{H_o} = \sqrt{\frac{c_o}{c_1} \frac{1}{2n_1}} = K_{sh} \tag{7.06}$$

where K_{sh} is often referred to as the shoaling coefficient.

This can be worked out a bit more by substituting for c_o, etc., and doing a lot of algebra; the final result is:

$$K_{sh} = \sqrt{\frac{1}{\tanh kh \left(1 + \frac{2\,kh}{\sinh 2kh}\right)}} \tag{7.07}$$

Also, since $k = \frac{2\pi}{\lambda}$, K_{sh} is purely a function of $\frac{h}{\lambda}$ and, therefore, can be added to table 6.2; indeed, $\frac{H_1}{H_o}$ is listed in the last column.

For completeness, we should check what the extreme values of K_{sh} can be in deep and shallow water. In deep water:

$$K_{sh_o} = 1 \tag{7.07a}$$

Mathematics confirm the result of physical reasoning in this case.

In shallow water, it is easiest to begin with equation 7.06. Using shallow water values of c_1 and n_1:

$$K_{sh} = \sqrt{\frac{c_o}{\sqrt{gh}} \frac{1}{2}} \tag{7.08}$$

With a bit of algebra, this reduces to:

$$K_{sh} = \sqrt{\frac{1}{4\pi} \frac{\lambda}{h}} = 0.2821 \sqrt{\frac{\lambda}{h}} \tag{7.07b}$$

which approaches ∞ as h approaches 0.

Expressed in another form, also for shallow water:

$$K_{sh} = \sqrt[4]{\frac{1}{8\pi} \frac{\lambda_o}{h}} = 0.4466 \left(\frac{\lambda_o}{h}\right)^{\frac{1}{4}} \qquad (7.07b)$$

This gives K_{sh} purely as a function of h for a given wave condition.

7.3. Example

All the information is now available to compute the effect of depth changes on a two dimensional wave as long as the wave does not break. An example of how shoaling affects wave properties is shown in the following table.

In table 7.1. a very common sort of North Sea wave is followed as it progresses from deep water to shallow water. In deep water, equations 5.01a through 5.07a and 7.07a are used. In shallow water, equations 5.01b through 5.07b and 7.07b are used. The complete equations (actually the tables in Wiegel (1964)) are used for intermediate depths. Our wave has the following given properties:

H_o = 2.0 m and T = 7.0 seconds.

From this follows that:

c_o = 10.93 m/s and

λ_o = 76.53 m.

TABLE 7.1. Wave Variations in Shoaling Water

T = 7 s ; H_o = 2.0 m; c_o = 10.93 m/s ; λ_o = 76.53 m

Water depth h (m)	$\frac{h}{\lambda_o}$ (-)	Applicable Theory	$\frac{h}{\lambda}$ (-)	Wave Length λ (m)	n (-)	c/c_o (-)	Phase velocity c (m/s.)	$\frac{H}{H_o} = K_{sh}$ (-)	Wave height H (m)	Wave steepness H/λ (-)	$\frac{H}{h}$ (-)	Amplitude Surface velocity $\hat{u}_s$ (m/s)	Amplitude Bottom Velocity $\hat{u}_b$ (m/s)	Water depth (m)
100	1.307	deep	1.307	76.53	0.500	1	10.93	1	2.0	0.02613	0.0200	0.90	0.00	100
75	0.980	deep	0.980	76.53	0.500	1	10.93	1	2.0	0.02613	0.0267	0.90	0.00	75
50	0.6533	deep	0.653	76.53	0.500	1	10.93	1	2.0	0.02613	0.0400	0.90	0.01	50
36.26	0.500	deep	0.500	76.53	0.500	1	10.93	1	2.0	0.02613	0.05227	0.90	0.04	38.26
		inter.	0.5018	76.25	0.5115	0.9964	10.89	0.9905	1.98	0.02597	0.0518	0.89	0.08	
28	0.366	deep	0.366	76.53	0.500	1	10.93	1	2.0	0.02613	0.0714	0.90	0.09	28
		inter.	0.3728	75.10	0.5433	0.9817	10.73	0.9683	1.94	0.0258	0.0693	0.89	0.17	
19.13	0.250	deep	0.250	76.53	0.500	1	10.93	1	2.0	0.02613	0.1045	0.90	0.19	19.13
		inter.	0.2679	71.41	0.6164	0.9332	10.20	0.9323	1.86	0.0260	0.0972	0.89	0.32	
15	0.196	inter.	0.2218	67.63	0.6724	0.8839	9.66	0.9172	1.83	0.0271	0.1220	0.93	0.43	15
10	0.131	inter.	0.1674	59.74	0.7606	0.7824	8.55	0.9166	1.83	0.0306	0.1830	1.05	0.65	10
5	0.0653	inter.	0.1094	45.70	0.8713	0.5966	6.52	0.9808	1.96	0.0429	0.3920	1.48	1.18	5
3.82	0.0500	inter.	0.09416	40.57	0.8999	0.5310	5.80	1.023	2.05	0.0505	0.5366	1.73	1.47	3.82
		shallow	0.08913	42.86	1	0.5599	6.12	0.945	1.89	0.0441	0.4947	1.51	1.51	
3.00	0.0392	inter.	0.0824	36.41	0.9207	0.4758	5.20	1.068	2.14	0.0588	0.7133	2.02	1.77	3.00
		shallow	0.0790	37.98	1	0.4968	5.43	1.004	2.01	0.0529	0.6700	1.82	1.82	
2.00	0.0261	inter.	0.0663	30.17	0.9466	0.3939	4.31	1.158	2.32	0.0769	1.160	2.64	2.43	2.00
		shallow	0.0645	31.01	1	0.4053	4.43	1.111	2.22	0.0716	1.110	2.47	2.47	
1.00	0.0131	inter.	0.0463	21.60	0.9729	0.2830	3.09	1.348	2.70	0.1250	2.70	4.28	4.11	1.00
		shallow	0.0456	21.93	1	0.2864	3.13	1.321	2.64	0.1205	2.64	4.15	4.15	
0.77	0.0100	inter.	0.0403	19.11	0.9792	0.2480	3.10	1.435	2.87	0.1502	3.73	5.20	5.03	0.77
		shallow	0.0400	19.24	1	0.2516	2.75	1.410	2.82	0.1466	3.66	5.04	5.04	

NOTE: NO CONSIDERATION OF BREAKING IS INCLUDED!

7.4. Review of Example

A lot of useful conclusions can be drawn from table 7.1. For example, the results of the deep, intermediate, and shallow water wave equations can be compared for various values of h/λ_0. This might be helpful for reviewing the criteria for determining which, if any, approximation to use. With $h/\lambda_0 = 0.5$, the amplitude of the horizontal velocity at the bottom $\hat{U}_b$ varies by a factor of two depending upon the approximation used. This may or may not be serious, depending upon the character of the particular engineering problem involved. For a floating structure, then this error at the bottom is certainly unimportant. On the other hand, for erosion prediction around the foot of a gravity construction, this error may be very important.

7.5. Breaking Criteria

Note from equation 7.07b that the theory used does not impose any limit on the height increase of a wave as it approaches a coast. On the other hand, we have all been to the beach at some time* and have not seen any waves of infinite height. What, then, are the practical limitations on wave height? There are two: wave steepness, and wave height to water depth ratio.

Steepness Limit

Using the theory of solitary waves, investigators have shown that the maximum steepness of a non-breaking wave is $0.142 \approx \frac{1}{7}$.
For this, the steepness is defined as the ratio of wave height to wave length, $\frac{H}{\lambda}$. This criteria normally governs the breaking of waves in deep water, and yet as Kinsman (1965) points out, many large storm waves do not break because they are too long†.

Values of wave steepness have been included in table 7.1. This breaking criteria, indicates that the wave broke at a water depth somewhere between 0.77 and 1.00 m.

---

* It is sincerely hoped that this assumption is correct!

† A wave of 15 second period, for example, would have to be 50 m high before it broke according to the steepness criterium. Luckily, we don't find this sort of wave too often.

Wave Height: Water Depth Limit

The second criteria for breaking applies to the ratio of wave height to water depth $\frac{H}{h}$, often called the breaking index. A theoretical limit for this ratio (again for a solitary wave) is 0.78. This limit ratio is sometimes denoted by γ. A practical value for γ is about 0.6. Occasionally, an individual wave has even been observed for which $\frac{H_{br}}{h_{br}} > 1.2$. Thus, the limit is not an absolute surety. Generally, this criteria governs the breaking of waves on a shore. More thorough investigations, to determine γ from know physical parameters of the shore and wave are now being carried out.

Values of $\frac{H}{h}$ have been included for our example in table 7.1 as well. The wave would break according to this criteria in a water depth somewhere between 2 and 3 m. Indeed, the wave has broken by exceeding the maximum ratio $\frac{H}{h}$, rather than the maximum steepness. Now that breaking criteria and the effect of shoaling on a wave are known, we can think about the following questions: "How high is our wave in the example problem after it has passed over a shoal area having a minimum depth of 5 meters and has continued into water which is once again 100 meters deep?" What might be a good answer if the minimum depth of the shoal was 2 m?

In the first of these cases the shoal 5 m deep is not sufficient to cause the wave to break. Since the other effects are reversible, the wave height in deep water after the shoal will be, again, 2 meters.

In the second case, the wave which gets over the shoal will have a height of about 0.6 x 2 = 1.2 m. As this wave progresses on into deeper water its wave height will decrease in the ratio $\frac{2}{2.22}$ (see table 7.1). Thus, the resulting wave height in deep water will be $1.2 \times \frac{2}{2.22} = 1.08$ m.*

* This is only a rough approximation. Other effects of the shoal such as transfer of energy to other (new) waves have been neglected.

8. TYPES OF BREAKERS

W.W. Massie

8.1. Introduction

In chapter 7 the criteria for wave breaking have been presented. Now, the breaking process itself can be examined. Obviously, when a wave breaks, its height dimishes and some of the energy of the wave is dissipated in turbulence and bottom friction; some is reflected back out to deep water, and some of the energy generates sound, other waves, heat, and currents. This last item, currents within the breaker zone, play a very important role in the morphological changes which occur along a coast. This current generation is reviewed in chapter 26 and treated in detail in volume II.

Patrick and Wiegel (1955) list three main types of breakers. These are described in the following paragraph.

8.2. Breaker Types

Spilling Breaker

Spilling breakers are usually found along very flat beaches. Waves begin breaking at a relatively great distance from shore and break very gradually as they approach still shallower water. A foam line develops at the crest during breaking and leaves a thin layer of foam over a considerable distance. Kinsman (1965) shows this very impressively on page 50 of his book. A less spectacular example is shown in figure 8.1. The breaker height decreases rather uniformly, as we approach the coast. There is very little reflection of momentum back toward the sea.

Figure 8.1
SPILLING BREAKER
GENTLE BEACH SLOPE MAY CAUSE SEVERE EROSION

Plunging Breaker

This is type of breaker often found on the travel posters for the Pacific Islands; it is spectacular. The curling top is characteristic of these waves. When one breaks much energy is dissipated in turbulence; little is reflected back to sea, and not much of a new wave is generated in the shallower water. This last is in contrast to what happens with a spilling breaker. Figure 8.2 shows a plunging breaker.

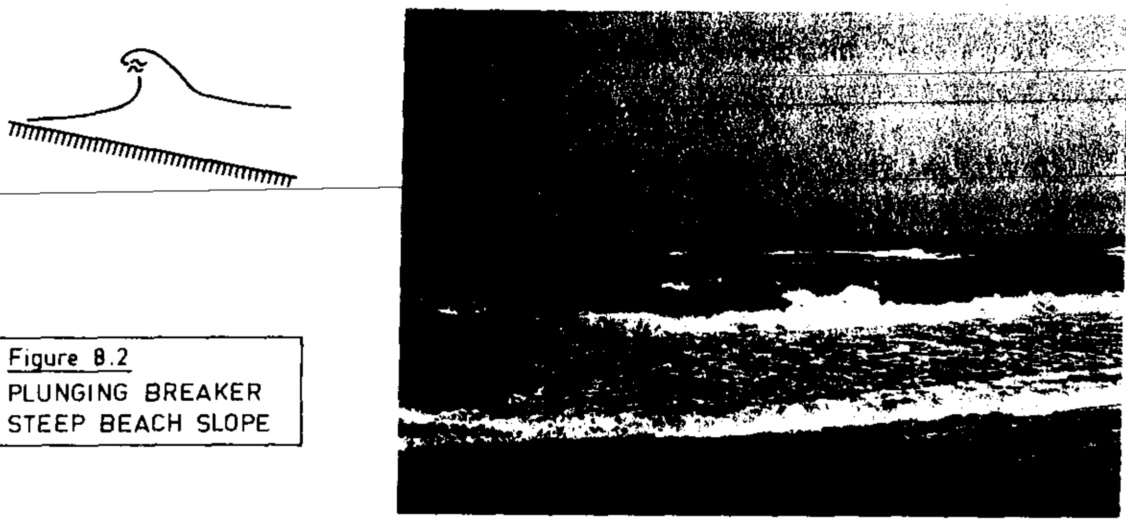


Figure 8.2
PLUNGING BREAKER
STEEP BEACH SLOPE

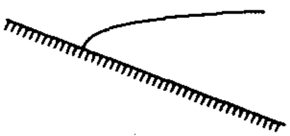


Figure 8.3
SURGING BREAKER
EXTREMELY
STEEP SLOPE

Surging Breakers

Surging breaks occur along extremely steep shores such as might be encountered along rock coasts. The breaker zone is very narrow, and much (more than half, usually) of the wave energy is reflected back out to deeper water. Figure 8.3 shows such a breaker. These breakers form up, much like plunging breakers, but the toe of each wave surges up the beach before the crest can curl over and fall.

8.3. Quantitative Classifications

Galvin (1968) found two parameters which could be used to classify the type of breaking wave via quantative observation. Both parameters are dependent upon wave properties and the beach slope; one is based upon breaker height, H_{br}, while the other is related to deep water conditions. Of these, the first, based upon breaker conditions is probably more practical and dependable. It is practical since it is based upon measurements made in the breaker zone itself where we would probably also be measuring. Galvin did his measurements there, too. It is more dependable, since it is a direct measurement in the breaker zone itself, and cannot be influenced by what happens between deep water and the breaker zone.

More specifically, the two parameters are:

$$\frac{H_o}{\lambda_o m^2} \qquad (8.01)$$

and

$$\frac{H_{br}}{gmT^2} \qquad (8.02)$$

where:

- g is the acceleration of gravity,
- H_{br} is the breaker height (wave height at the outer edge of the breaker zone),
- H_o is the wave height in deep water,
- λ_o is the wave length in deep water,
- m is the slope of the beach, and
- T is the wave period.

These parameters given by (8.01) and (8.02) are emperical (based upon observation). Also, all of the theory of the previous sections is not enough to allow us to derive (8.02) beginning with (8.01).

Another approach used by Swart (1974) defines a parameter p which ranges between 0.0 for spilling breakers and 1.0 for plunging breakers. (It is valid only for these two types). He states that p can be determined with reasonable accuracy but does not elaborate how, expect that he mentions "visual observation".

The relationship between the breaker type and the values of the parameters above is given in table 8.1.

TABLE 8.1. Breaker Type Classification

breaker type	$\frac{H_o}{\lambda_o m^2}$	$\frac{H_{br}}{gmT^2}$	p
	0.0	0.0	
surging			invalid
	0.09	0.003	
Plunging			1.0
	4.8	0.068	
Spilling			0.0
	∞	∞	

8.4. Reexamination of Breaking Criteria

Swart (1974) combines the value p with the deep water wave conditions and the beach slope to determine the value of the breaking index, $\gamma = \frac{H_{br}}{h_{br}}$ in which h_{br} is the water depth at the breaking point of the wave. In equation form:

$$\gamma = 0{,}33\ p + 0.46 \qquad (8.03)$$

Even this equation has some limitations, but may be helpful. Further research to arrive at an improved equation is under way. Putting an "average" value of p of 0.5 in (8.03) yields about the same value of γ as given in chapter 7.

9. WAVE REFRACTION AND DIFFRACTION

L.E. van Loo

9.1. Introduction

Until now, this discussion of waves has been restricted to two-dimensional phenomona; only motions which occurred in the vertical x-z plane were considered. Waves propagating into shallower water were assumed to be moving with their crests parallel to the depth contours. Further, until now, no partial obstacles have been allowed to interrupt the path of the waves. These restrictions will now be relaxed. Consider what happens when waves approach shoaling water with their crests at an angle to the depth contours.

9.2. Wave Refraction

When waves approach shallower water with their crests at an angle to the depth contours, the wave crests appear to curve so as to decrease this angle. This results from the fact that the wave celerity decreases as the water depth decreases - see equation 5.05 or 5.05b. In deep water refraction does not take place, since the wave speed is independent of water depth, there.

This phenomona is much like that in geometrical optics, where Snell's Law describes the behavior of light rays passing from one medium to another having a different transmission velocity. In the present case there is a gradual change in wave speed instead of an abrupt one encountered in optics. This gradual change leads to the curved wave crests shown in figure 9.1.

In figure 9.1, wave orthogonals (always perpendicular to the wave crests) have been sketched as well. These orthognals are sometimes called rays. A bit of geometry quickly reveals that the distance between these rays increases as the water becomes shallower.

The effect of refraction on wave height is computed by assuming that the power transmitted between two adjacent wave orthogonals remains constant. In equation form:

$$U_1 b_1 = U_2 b_2 \qquad (9.01)$$

where U is the power per unit crest length and b is the distance between orthogonals at points 1 and 2 respectively.

This should be compared to equation 5.10. Using this equation, we get:

$$E_1 n_1 c_1 b_1 = E_2 n_2 c_2 b_2 \qquad (9.02)$$

where: E is the wave energy,
n is the ratio of group velocity to wave celerity, and
c is the wave celerity.

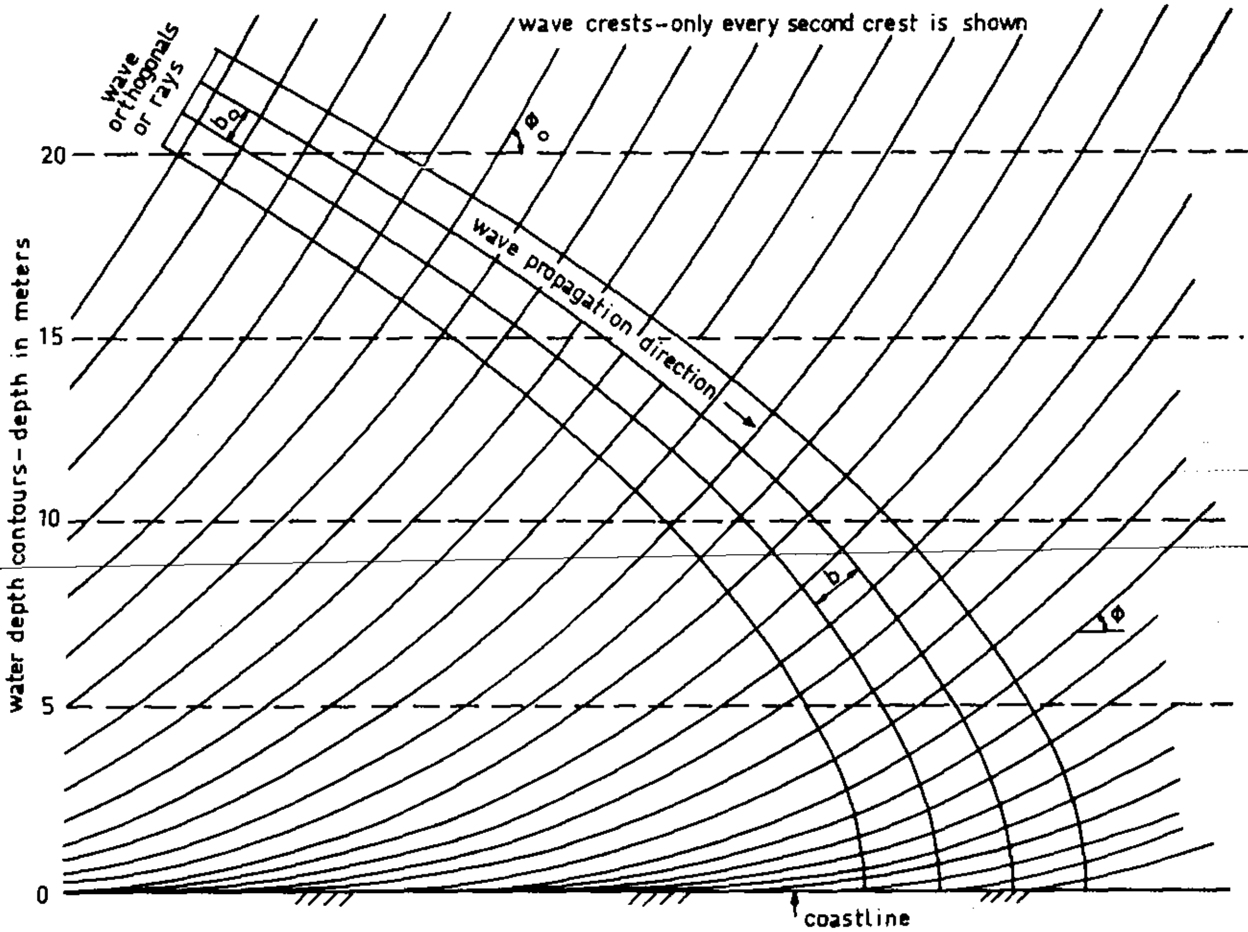


Figure 9.1 REFRACTION PATTERN

Subsituting for E from (5.09) and choosing one measurement point in deep water leads to:

$$\frac{H_1}{H_o} = \sqrt{\frac{1}{2n_1} \frac{c_o}{c_1} \frac{b_o}{b_1}} = K_{sh} K_r \quad (9.03)$$

where: H is the wave height,

K_{sh} is the shoaling coefficient, and

K_r is the refraction coefficient.

$$K_r = \sqrt{\frac{b_o}{b_1}} .$$

Only the problem of evaluating the ratio $\frac{b_o}{b_1}$ remains. This is accomplished using geometery and Snell's Law. In equation form:

$$\frac{\sin \phi_o}{\sin \phi} = \frac{c_o}{c} \quad (9.04)$$

or:

$$\sin \phi = \frac{c}{c_o} \sin \phi_o \quad (9.05)$$

where ϕ is the angle between the wave crest and the depth contour.

From geometry:

$$\frac{b_o}{b_1} = \frac{\cos \phi_o}{\cos \phi_1} \qquad (9.06)$$

These relationships make it possible to complete the computations in a specific case. Table 9.1 shows the effect of refraction on the same wave as was chosen for table 7.1. (T = 7 seconds, H_o = 2 m). Values from this table were used when drawing figure 9.1.

From table 9.1 it is obvious that refraction decreases the wave height as the water becomes shallower. What would be the angle between the wave crest and the depth contour if the depth were allowed to become zero? On the other hand, what happens when our waves pass over a shoal and again into deep water? The refraction process is reversible.

The computation procedure listed above is easily carried out for simple coasts where the bathymetry is simple. For more realistic hydrographic conditions, such a computation can be extremely laborious. For this reason a graphical solution technique has been developed. The construction of the necessary templates and their use is well described in the *Shore Protection Manual*, volume I, chapter 2.

TABLE 9.1. WAVE REFRACTION COMPUTATIONS

T = 7.0 s ; H_o = 2.0 m; → c_o = 10.93 m/s ; λ_o = 76.53 m.
λ_o = 76.53 m

(1) Water Depth h (m)	(1) Wave Length λ (m)	(1) c/c_o (-)	(1) K_{sh} (-)	ϕ (deg)	(1) n (-)	K_r (-)	(2) $\frac{H}{H_o}$ (-)	(2) H (m)	(1) (3) H' (m)	b_1/b_o (-)	
100	76.53	1	1	60.0	0.500	1.00	1.00	2.00	2.00	1.00	
19.13	76.53	1	1	60.0	0.500	1.00	1.00	2.00	2.00	1.00	
15	67.63	0.8839	0.9172	49.9	0.6724	0.8815	0.8085	1.62	1.83	1.29	
10	59.74	0.7824	0.9166	42.7	0.7606	0.8245	0.7558	1.51	1.83	1.47	
5	45.70	0.5966	0.9808	31.1	0.8713	0.7642	0.7495	1.50	1.96	1.71	
3.82	42.86	0.5599	0.9450	29.0	1.000	0.7561	0.7145	1.43	1.89	1.75	
3.0	37.98	0.4968	1.004	25.5	1.000	0.7442	0.7472	1.49	2.01	1.81	
2.0	31.01	0.4053	1.111	20.5	1.000	0.7307	0.8118	1.62	2.22	1.87	Wave Broken!!
1.0	21.93	0.2864	1.321	14.4	1.000	0.7184	0.9490	1.90	2.64	1.94	
0.77	19.24	0.2516	1.410	12.6	1.000	0.7158	1.009	2.02	2.82	1.95	

NOTES
(1) Data taken directly from table 7.1
(2) Includes both refraction and shoaling influences, but not breaking!
(3) Includes only shoaling

A further alternative to hand computations or graphical constructions is the use of large scale digital computer models. These models are currently under development; many still have occasional problems.

9.3. Wave Diffraction

Diffraction is a three-dimensional effect arising as a result of a "shadow" being formed by an obstacle. Diffraction is the phenomona responsible for the spread of waves into this shadow zone.

When diffraction occurs, wave energy seems to be transferred along the wave crests (across the orthogonals). This is in contrast to the assumption made in the previous section of this chapter.

How does diffraction occur? The following physical explanation has some strict theoretical difficulties, but is sufficient to give an insight to the process involved.

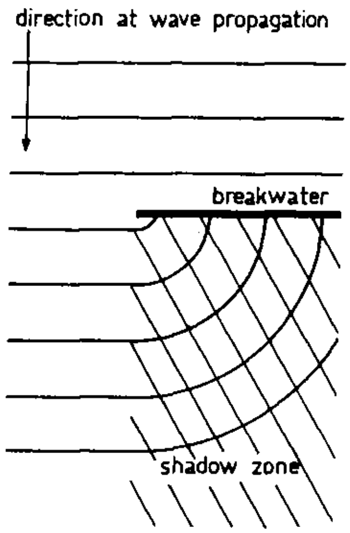


Figure 9.2
WAVE DIFFRACTION PATTERN

As a wave passes the end of the obstacle shown in figure 9.2, the end of the breakwater may be considered as a source which generates arc shaped waves in the shadow zone behind the breakwater. The wave height decreases as we proceed along a wave crest arc in this shadow zone. Diffraction computations based upon this simple model might be easy but would generally be useless. The wave reflected by the seaward side of the breakwater (obstacle) is also partially diffracted into the shadow zone. Further, diffracted waves, hitting the shadow side of the breakwater are also reflected.

Even with all of this, plus a finite width of opening in a breakwater, numerical or graphical computations can often be used to determine wave heights at selected points in the vicinity.
The theory is difficult, and will not be presented here; it is well treated in the courses and literature on short wave theory.
One of the resulting graphical methods is based upon the Cornu Spiral. Graphs, showing wave height ratios as a function of position are given in the *Shore Protection Manual*, volume I, section 2. From these graphs it is obvious that the wave height distribution along a shore within a harbor can be quite irregular.

10. WAVE STATISTICS RELATIONSHIPS

E. Allersma
E.W. Bijker
W.W. Massie

10.1. Introduction

Until this point in our discussion all waves have been considered to be sinusoidal with constant period. It has already been indicated that this is not true in nature. Indeed, the sea surface can appear to be very irregular. Figure 10.1 shows a dramatic example of this.

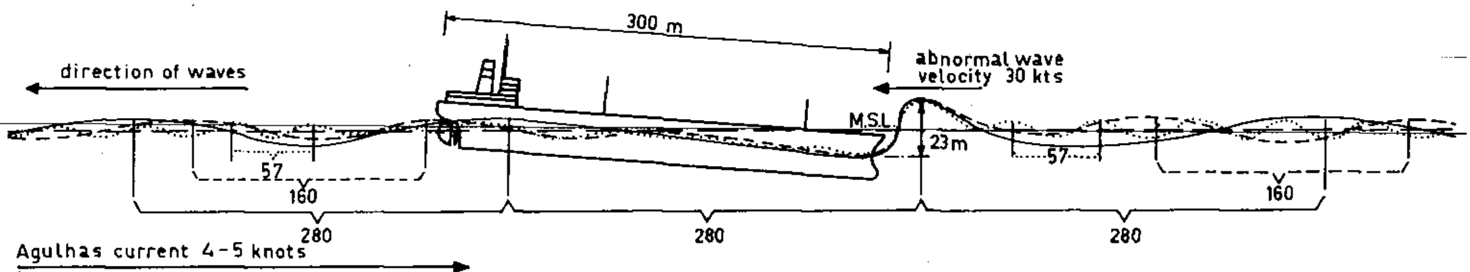


Figure 10.1
DIAGRAMATIC PROFILE SHOWING SINUSOIDAL CURVES OF THREE WAVE TRAINS HAVING WAVE LENGTHS OF 280, 160 AND 57 METERS WHICH BECOME IN-PHASE FOR A SHORT PERIOD THEREBY CREATING AN ABNORMAL WAVE ABOUT 23 METERS HIGH. IN ADVANCE OF THE WAVE IS A LONG DEEP TROUGH.

In this, and the following two chapters, we shall attempt to describe this sea surface irregularity more carefully.

Reference information is provided on these topics by Kinsman (1964), and Neumann and Pierson (1966). This chapter is largely abstracted from Allersma and Massie (1973), in which many more references are also listed.

10.2. The Phenomona and its Characterizations

The motion of the sea surface is irregular. The water level is a stochastic variable. Generally, measurements of water level are made only at a fixed location. This yields a stochastic record of water levels as a function of time. This record could be presented as a graph of measurements carried out over a number of years or even decades. Unfortunately, such a long graph is rather cumbersome to use; statistics can be used to condense this data into more usable form without the loss of valuable details. Thus, the problem becomes twofold:

a. The determination of the statistical parameters necessary to characterize a portion of our wave record representing an interval of constant sea conditions (usually a few hours).
b. The determination of the frequency with which these statistical characterizations of the earlier mentioned sea state occur. When these two things are known, then the theory of the previous chapters (5 through 9) and a statistical model can be used to accomplish the design of our project.

What, specifically, are the statistical parameters needed? Civil Engineers are most often interested in wave heights. Therefore, it would seem most convenient to consider the statistical distribution of wave heights. The common statistical parameters can be used to describe such a distribution; the mean value is the most simple of these. In coastal engineering practice, however, the smaller waves are generally neglected and the mean value of the highest 1/3 of the waves (which are of the most interest) is chosen. This mean is called the significant wave height and is about the same wave height as an experienced observer would estimate visually. This value is denoted by H_{sig}.

Another wave height characterizing parameter of use in energy relationships is the root-mean-square wave height. For a group of N waves it is defined as:

$$H_{rms} = \sqrt{\frac{\sum_{i=1}^{N} H_i^2}{N}} \tag{10.01}$$

Similarly, a characteristic wave period can be determined; the average of the periods of the higher waves in the group is often chosen.

A disadvantage of such a simple parameter such as H_{sig} is that it gives only a very global description of the wave heights in the record. This would indeed be true if the wave height distribution was completely random. Luckily, many stochastic processes can be described by theoretical distributions having certain properties. For example, if a variable has a Gaussian Distribution then all of the statistical information can be condensed into two parameters: the mean, and the standard deviation.

Fortunately, within reasonable accuracy, wave heights of natural irregular waves also can be described via a theoretical distribution model: the Rayleigh Distribution. This distribution is completely characterized by a single parameter. Thus, the significant wave height, H_{sig}, (or any other average such as H_{rms}) is sufficient to completely characterize the distribution. Taking H_{sig} as the characteristic parameter, the Rayleigh Distribution can be described by

$$P(H) = e^{-2\left(\frac{H}{H_{sig}}\right)^2} \tag{10.02}$$

where $P(H)$ is the probability of exceedance of wave height H,

H_{sig} is the significant wave height of the record,

e is the base of natural logarithms.

Values of P(H) versus $\frac{H}{H_{sig}}$ computed from equation 10.02 are given in table 10.1. Special graph paper is available which transforms equation 10.02 into a straight line. This is shown in figure 10.2. Obviously, using either the table, graph, or 10.02 we can determine the probability of exceedance of any desired wave height occurring in an interval characterized by given significant wave height.

Some other handy relationships, also based upon the Rayleigh Distribution are listed below:

$$H_{sig} = 1.414 \, H_{rms} \qquad (10.03)$$

$$\overline{H} = 0.886 \, H_{rms} \qquad (10.04)$$

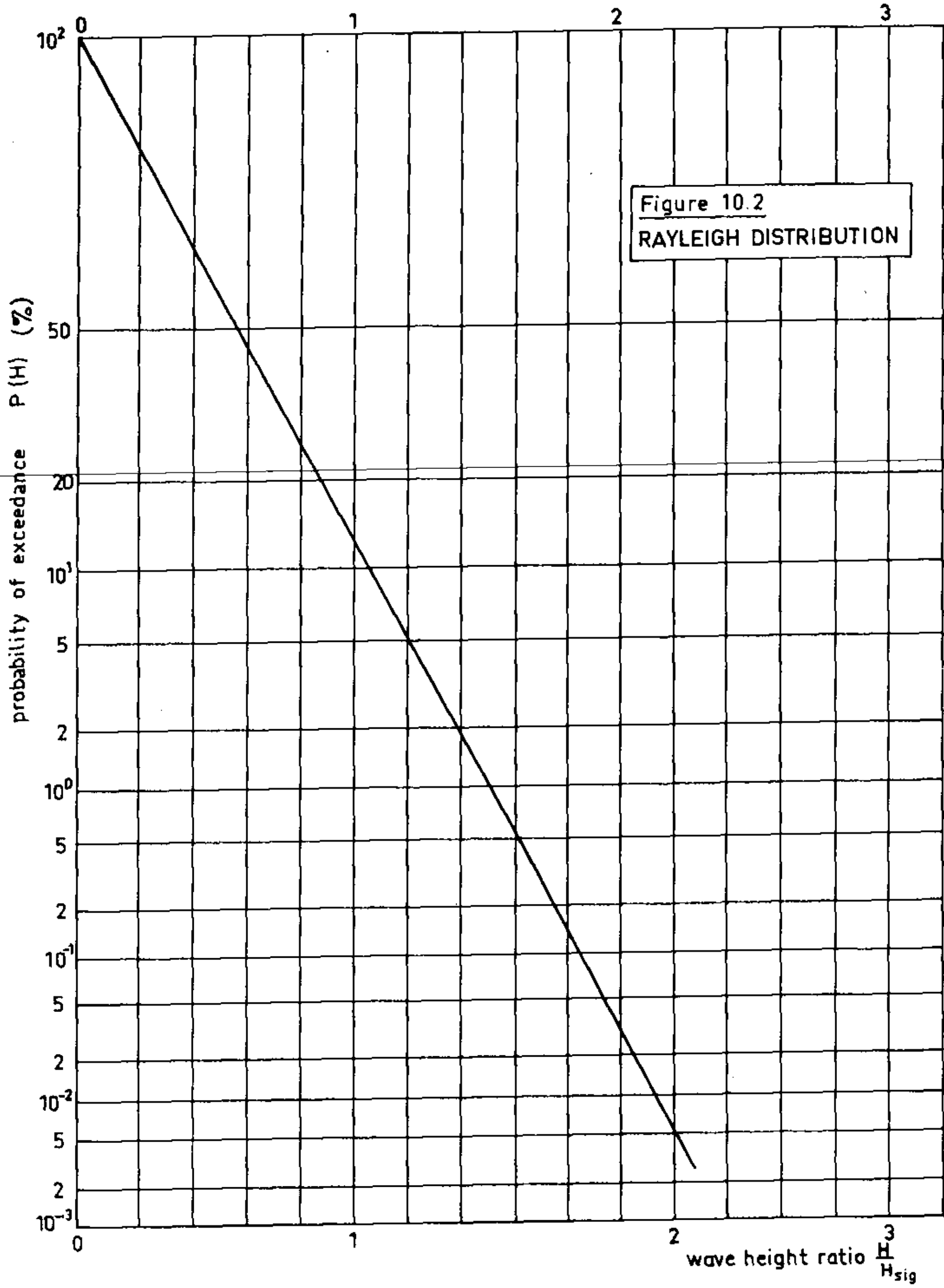
where $\overline{H}$ is the average of all waves.

Table 10.1

Values of $\frac{H}{H_{sig}}$ for various values of P(H)

Probability of exceedance P(H)	$\frac{H}{H_{sig}}$
10^{-5}	2.40
$2x10^{-5}$	2.33
$5x10^{-5}$	2.22
10^{-4}	2.15
$2x10^{-4}$	2.06
$5x10^{-4}$	1.95
10^{-3}	1.86
$2x10^{-3}$	1.77
$5x10^{-3}$	1.63
0.01	1.51
0.02	1.40
0.05	1.22
0.10	1.07
0.125	1.02
0.135	1.00 *
0.20	0.898
0.50	0.587
1.00	0.000

* Follows from definition of H_{sig}



$$\sigma_H = 0.463\ H_{rms} \tag{10.05}$$

where σ_H is the standard deviation of the wave height.

$$H_{sig} = 1.596\ \bar{H} \tag{10.06}$$

We have now answered part of the problem posed at the beginning of this section, namely, we have the statistical parameters necessary do describe an interval of our total wave record.

10.3. Determination of Frequency of Occurrence

Once the statistical characterizations of one interval of the record have been determined, then this process can be repeated for every interval in the record (each producing a single value of H_{sig}) and then the frequency of occurrence of these parameters during the entire long record can be determined. Often times, this frequency is given in the form of number of observations per year. If a single observation characterizes a storm, a scale of storms per year is sometimes used. When this is plotted on semi-logarithmic paper a reasonably straight line usually results.* An example of such a long term distribution is shown in figure 10.3. Extrapolation of this graph is allowed as long as no other boundary conditions such as water depth or wind fetch length limit our wave heights.

10.4. Wave Periods

Just as wave heights have been characterized by statistical parameters, the wave period can also be treated as a statistical variable. The exact theory is a bit too comprehensive to discuss here. Allersma and Massie (1973) do treat this topic more thoroughly.

On the other hand, more or less emperical relationships have been derived which relate wave period to some other easily determined parameter. Many of these relations have been derived for application in a certain geographical area; they are not generally applicable. Examples of such relations are,

For the North Atlantic Ocean:

$$T = 2.5\ H \qquad (10.07)$$

For the Mediterranian Sea:

$$T = 4 + 2\ H^{0.7} \qquad (10.08)$$

For the North Sea:

$$\overline{T} = 3.94\ H_{sig}^{\ 0.376} \qquad (10.09)$$

where $\overline{T}$ is the average of all wave periods.

These equations are *not* dimensionless; the constants stated are for period, T, in seconds and wave heights, H, in meters.

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* The reciprocal of the frequency, the recurrence interval, may also be used on the log scale. This is done in figure 10.3.

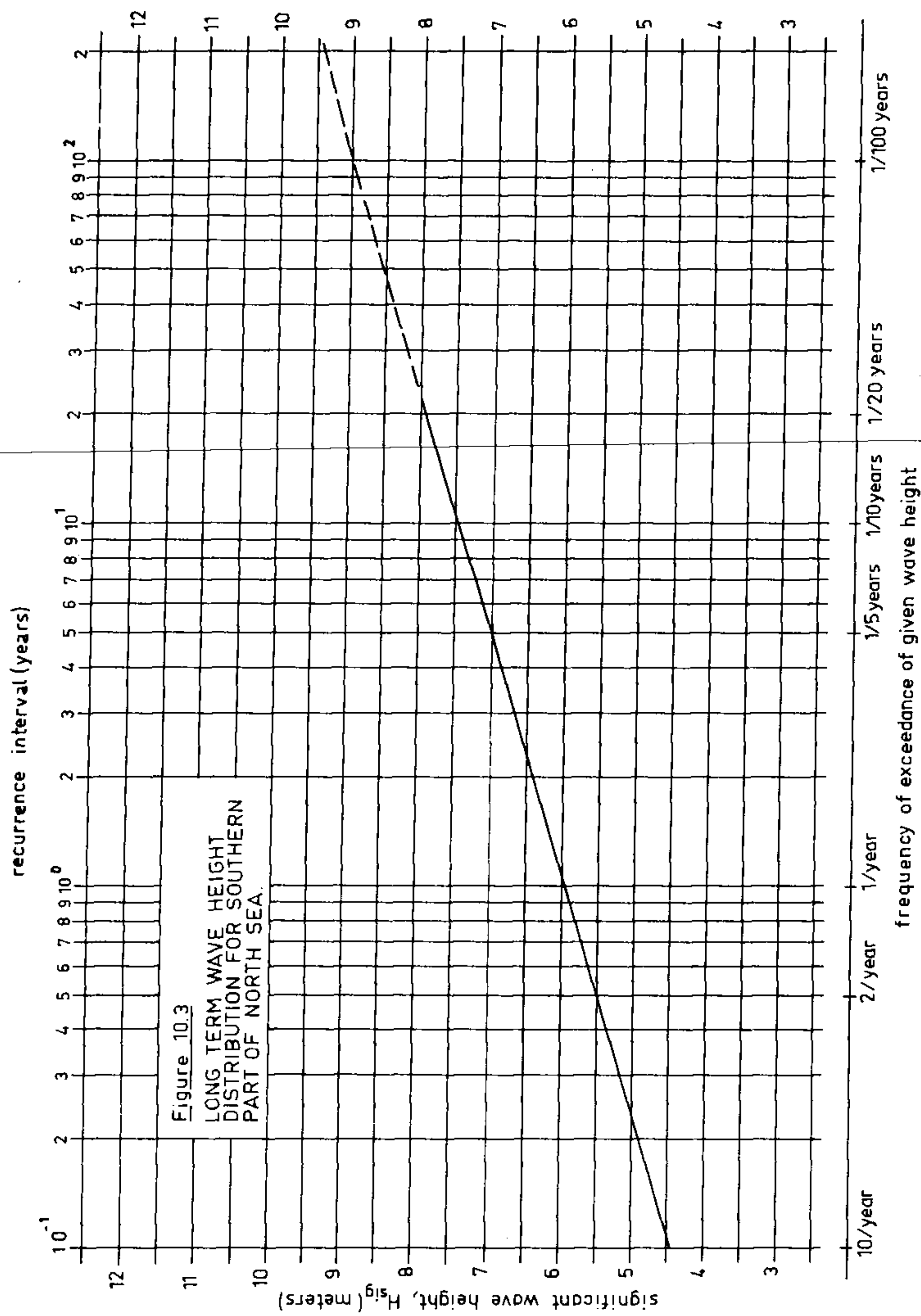

Figure 10.3
LONG TERM WAVE HEIGHT DISTRIBUTION FOR SOUTHERN PART OF NORTH SEA.

Wiegel (1964), on a slightly different path shows theoretically that:

$$T_e = 1.23\,\overline{T} \tag{10.10}$$

where T_e is the wave period of the regular wave train having the same energy as the irregular train having average period $\overline{T}$.

11. APPLICATION OF WAVE STATISTICS

E.W. Bijker
A. Paape

11.1 Introduction

The information provided in the previous chapter is fine for those interested only in condensing wave data. We, as engineers, need to apply that information to make a responsible design.

In this chapter a distinction is made between two similar situations which require different approaches from a statistical point of view.

In the first type of problem, the strength or stability of a structure may be evaluated in terms of a certain characterizing parameter of the waves, such as H_{sig}, the significant wave height. The construction is then subjected, either in a physical or mathematical model, to the entire Rayleigh Distribution determined by the chosen parameter value.

An example of such a design occurs with rubble mound breakwaters. (See volume III of these notes). When the structure is tested in a physical model, then the entire Rayleigh Distribution of wave heights should be reproduced. If, on the other hand, a mathematical model is used, then the fact that our characteristic wave represents an entire population of waves is taken into account in the formula preparation. This is done, for example, in formulas for breakwater armor unit weights.

This type of problem is relatively easy to handle. The probability of occurrence of the design parameter comes directly from the long term wave height distribution. Since this type of problem is treated in detail in the sections on breakwater design (volume III) we shall not consider it further, here.

In the second type of problem the structure is designed using just a single wave, the design wave. The objective in the remainder of this chapter will be to determine the probability of exceedance of a given design wave height. Why do this; don't we want to design a structure to withstand the biggest possible wave? Unfortunately, this is impossible, since it follows from the wave height distributions presented earlier that *any* chosen wave height - no matter how high - has a certain, finite chance that it will be exceeded. *Some* risk must be accepted. The problem of determining how much risk to accept is the subject of chapter 13 of this volume. For now, we shall only attack the problem of determining the chance that a given wave height will be exceeded in a given time interval.

11.2 Problem Statement and Assumptions

The precise problem statement is: "What is the chance that a chosen design wave height, H_d, is exceeded one or more times during the life, ℓ, of a structure?"

Each storm which occurs can be characteristized by a given value of H_{sig}, the significant wave height. This wave height characterizes a set of N waves to which the structure is exposed during the storm.

These N waves are distributed according to a Rayleight Distribution characterized by the single value of H_{sig}.

Lastly, it is assumed that the significant wave heights obey a long - term frequency distribution such as that shown in figure 10.3 of the previous chapter.

11.3 The Numerical Treatment

First, let us discuss how the value of N associated with a particular value of H_{sig} is determined. Sometimes, values of N are tabulated from actual wave records during processing to determine H_{sig}, etc. An alternative is to divide the duration of the storm by the characteristic wave period listed with the reduced wave data. In any event, N is known for each H_{sig}.

First, consider a single storm containing N waves characterized by H_{sig}. We choose an arbitrary design wave height H_d. The chance that H_d *is* exceeded by any given wave is:

$$P(H_d) = e^{-2\left(\frac{H_d}{H_{sig}}\right)^2} \qquad (10.02) \quad (11.01)$$

The chance that this wave *is not* exceeded is:

$$1 - P(H_d) \qquad (11.02)$$

The chance that this wave *is not* exceeded in a series of N waves is:

$$\left[1 - P(H_d)\right]^N \qquad (11.03)$$

Finally, the chance that this wave height, H_d, *is* exceeded at least once in our single storm containing N waves is:

$$E_1 = 1 - \left[1 - P(H_d)\right]^N \qquad (11.04)$$

As an alternative, some prefer to use a Poisson approximation to evaluate E_1. As long as $P(H_d)$ is small (usual for our problems) the difference is not important. In this case:

$$E_1 = 1 - e^{-N\,P(H_d)} \qquad (11.05)$$

The next step is to determine the chance that a single storm characterized by H_s occurs during a given year. This is accomplished by subdividing the cumulative long-term characteristic wave height distribution into intervals called classes. There shall be M chances for our storm to occur in a given year.

$$M = \frac{\text{number of hours per year}}{\text{duration of one storm in hours}} \qquad (11.06)$$

The probability that a given characteristic significant wave occurs in the class interval between H_{sig_1} and H_{sig_2} is:

$$f(H_{sig}) = \text{probability of } (H_{sig_1} \leqq H_{sig} \leqq H_{sig_2}) \quad (11.07)$$

$$= P(H_{sig_1}) - P(H_{sig_2}) \quad (11.08)$$

What is the influence of the width of the interval $H_{sig_1} - H_{sig_2}$ on $f(H_{sig})$? Obviously, as the class interval between H_{sig_1} and H_{sig_2} beomes wider, more waves will fall into the interval and $f(H_{sig})$ will increase. At the same time however, the total number of intervals will decrease such that the this change still has no influence on the end result.

Since there are M intervals per year, the probability that H_{sig} falls in the given interval sometime during the year is:

$$P(H_{sig}) = M\ f(H_{sig}) \quad (11.09)$$

where $P(H_{sig})$ is the chance (probability) that a storm characterized by H_{sig} occurs one or more times during a given year.

Let us now investigate the probability of *non-occurence*. For an individual observation, the probability that this value of H_{sig} falls outside the interval between H_{sig_I} and H_{sig_2} is:

$$1 - f(H_{sig}) \quad (11.10)$$

This time, the chance that the characteristic wave *does not* occur during a year is:

$$[1 - f(H_{sig})]^M \quad (11.11)$$

If the events under study are reasonably rare, then:

$$P(H_{sig}) << 1, \text{ and thus, } f(H_{sig}) << \frac{1}{M}$$

(see equation 11.09), and equation 11.11 can be approximated by:

$$[1 - f(H_{sig})]^M \approx 1 - M\ f(H_{sig}) \quad (11.12)$$

Using 11.09, we find the probability of occurence per year is then:

$$1 - [1 - M\ f(H_{sig})] = P(H_{sig}) \quad (11.13)$$

The chance that *both* this storm occurs *and* the design wave is exceeded in that storm is:

$$E_2 = P(H_{sig}) \cdot E_1 \quad (11.14)$$

The problem is not yet solved, however, since a wave of height H_d can also occur in a different storm characterized by another value of H_{sig}. Since for this different storm the ratio $\frac{H_d}{H_{sig}}$ and N is also different, then both the values of E_1 (eqn. 11.05) and $P(H_{sig})$ (eqn. 11.13) will change. A whole series of chances E_2, some number N' of them in fact, is generated. These can be denoted by E_{2i}, where i = 1 to N'.

Since each value of E_{2i} represents a chance that the design wave *is* exceeded, then the chance that H_d *is not* exceeded in the year during any storm is:

$$E_3 = (1 - E_{21})(1 - E_{22})(\ldots)(1 - E_{2i})(\ldots)(1 - E_{2N'})$$

$$= \prod_{i=1}^{N'} (1 - E_{2i}) \tag{11.15}$$

where $\prod_{i=1}^{N'}$ denotes the product of the N' terms and E_3 represents the chance that H_d *is not* exceeded during one year*.

Since the lifetime of our structure is ℓ years, the chance that the design wave height *is exceeded* one or more times during that lifetime is

$$P(H > H_d) = 1 - E_3^{\ell} \tag{11.16}$$

This is our objective!

11.4 Example Problem

Determine the total chance that a design wave height of 16 meters will be exceeded in the southern portion of the North Sea, for a structure with a life of 20 years. The Rayleigh Distribution is used in conjunction with the long-term distribution shown in table 11.1. This table gives a class interval distribution. The first two columns of this table are represented in figure 10.2 as well.

The computation for the typical row H_{sig} = 8 m in this table goes as follows:

$$\frac{H_d}{H_{sig}} = \frac{16}{8} = 2.00$$

Using 11.01:

$$P(H_d) = e^{-2(2)^2} = e^{-8} = 3.35 \times 10^{-4}$$

---

* Equation 11.15 can be approximated by $E_3 \approx 1 - \sum_{i=1}^{N'} E_{2i}$ if E_{2i} is sufficiently small. This is a similar approximation to that used in equation 11.12.

From 11.04:

$$E_1 = 1 - \left[1 - 3.35 \times 10^{-4}\right]^{1000} = 0.285$$

Then, 11.14 yields:

$$E_2 = 0.05 \times 0.285 = 1.43 \times 10^{-2}$$

We evaluate E_3 using all 5 values of E_2:

$$E_3 = \prod_{i=1}^{5} (1 - E_{2i}) = 0.969455$$

Finally, we determine the required probability from E_3 using 11.16:

$$P(H > 16) = 1 - (0.969455)^{20} = 0.4623$$

or about 46% !

In this example, the chosen design wave height has a chance of being exceeded of 1/1000 for a storm which can be expected, on the average, of once in 50 years. Yet, in the lifetime of the structure, there is a 46% chance (on the average) of encountering this wave; this is a very high risk! In chapter 13, we discuss acceptable risks.

TABLE 11.1 Computation of probability of exceedance of given design wave H_d = 16 m. in 20 years.

Given data			Rayleigh Dist.			
$P(H_{sig})$ $(\frac{\text{storms}}{\text{yr.}})$	N $(\frac{\text{waves}}{\text{storm}})$	H_{sig} (m)	$\frac{H_d}{H_{sig}}$ (-)	$P(H_d)$ (-)	E_1 (-)	E_{2i} (-)
0.002	600	10	1.600	5.98×10^{-3}	0.973	1.95×10^{-3}
0.01	800	9	1.778	1.80×10^{-3}	0.763	7.63×10^{-3}
0.05	1000	8	2.000	3.35×10^{-4}	0.285	1.43×10^{-2}
0.20	1000	7	2.286	2.90×10^{-5}	2.86×10^{-2}	5.71×10^{-3}
1.00	2000	6	2.667	6.66×10^{-7}	1.33×10^{-3}	1.33×10^{-3}

E_3 = 0.969

P (H > 16) = 46%

The computations just shown are very sensitive with regard to accuracy of the intermediate results. These were carried out on a 10 digit electronic calculator. A slide rule is useless for this type of calculation.

11.5 A Second Type of Problem

A different type of problem is sometimes encountered. Sometimes, via a design code, for example, it is specified that a structure be designed to withstand the *maximum* wave occurring in a storm having a given frequency.

The solution is reasonably simple. The significant wave height corresponding to the specified storm frequency of occurrence can be found directly from the long term wave height distribution. Since this storm also has a known number of waves, N, the probability of the maximum wave in that storm is $\frac{1}{N}$.
Putting:

$$P(H_d) = \frac{1}{N} \tag{11.17}$$

into equation 11.01 and then solving for H_d, yields:

$$H_{max} = \frac{1}{\sqrt{2}} H_{sig} \sqrt{\ln N} \tag{11.18}$$

which is the desired solution.

11.6 Example

Applying this idea to a modified form of the question in the previous example, the problem is now: Determine the maximum wave height which can be expected in the southern part of the North Sea in a period of (on the average) 100 years.

Using the data given in table 11.1, we see that the storm with a frequency of once per hundred years, $P(H_{sig}) = 0.01$, has a significant wave height of 9 meters, and contains 800 waves. Putting this into equation 11.18, yields:

$$H_{max} = 0.707 \times 9 \times \sqrt{\ln 800} \tag{11.19}$$

$$= 16.45 \text{ meters.}$$

This is a very interesting result when compared with that of the previous example. What is the explanation for the fact that this wave is only slightly higher than that which had a 46% chance of occurring in 20 years? The reason for this seeming discrepancy is that the H_{max} found in the second example is really an average maximum wave; it, itself, will be exceeded in half of the 100 year periods.

12. WAVE DATA

W.W. Massie

12.1 Introduction

In chapters 10 and 11 we used wave height data without saying too much about the practical problem of obtaining this information. In this chapter we shall indicate briefly how this necessary information can be obtained.

12.2 Existing Data

Government agencies in many countries accumulate data on waves and currents in areas under their jurisdiction. Some of this data is published, most of it is available upon request; occasionally, some is secret. The booklet by Dorrestein (1967) is an excellent example of published data. The information is presented in tabular form.

In general, the type of information needed is accumulated by the weather service or the hydrographic office on a national scale. Locally, measurements made specifically for a given project may be available from local agencies such as departments of public works.

Some of the the major hydrographic offices have wind, wave, and current data for nearly the entire world. Much of this is readily available. Probably the most important source of world-wide hydrographic information is the British Admiralty. The United States Naval Hydrographic Office also has an impressive collection.

12.3 Measurement Program

There are, of course, areas of the world for which no readily available wave data exist. What then? One solution, provided that sufficient time and money is available, is to conduct a specific measurement program. The length of time available for measurements is seldom, if ever, sufficient. Sometimes, the required measurement period can be shortened by correlating our few measurements with simultaneous measurements - part of a much longer record - at a nearby location*.

Various types of wave height meters are available. Some measure water surface elevation directly with reference to a fixed staff, while other ride the waves and record the vertical water surface acceleration. A third type measures pressure differences at some point in the water. It is beyond the scope of this brief summary to discuss these various instruments in detail.

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* The second long-term set of measurements need not even be wave measurements. In many cases, a correlation with local wind data may be possible.

12.4 Use of Substitute Data

It is sometimes advantageous to artificially generate wave data from available meterological data. This information, gathered from ship's logs, is often published in special atlases of wind or barometric pressure data. The wind data can be used in a wave forecasting technique to predict the waves.

Is there *always* a correlation between wind and waves? No. When is it possible to have waves without wind or wind without waves? *

How do we get wave data from barometric pressure information? The wind can often be predicted from the pressure gradients. Equilibrium between pressure gradient, Coriolis, and centripetal forces yields a wind velocity. The computation is similar to that used for ocean currents in chapter 3 of this volume. Once the wind is know, a wave forecasting technique can be used.

The advantage of predictions over our own site measurements is that they can be done more quickly and in a comfortable office. Even so, such a prediction can involve a lot of tedious work, and is probably less dependable that some on-site measurements.

12.5 SMB Prediction Method

Bretschneider (1952) revised the semi-emperical wave forecasting relationships presented by Sverdrup and Munk (1947). The technique is thus called the Sverdrup-Munk-Bretschneider (SMB) method.

Three dimensionless equations form the basis of the method:

$$\frac{g\,H_{sig}}{U_w^2} = 0.283\ \tanh\ [0.0125\ \ \Phi^{0.42}] \qquad (12.01)$$

$$\frac{g\,T_{sig}}{U_w} = 7.540\ \tanh\ [0.077\ \Phi^{0.25}] \qquad (12.02)$$

$$\frac{g\,d}{U_w} \geqq 6.5882\ \exp\ \{[0.0161\ (\ln\ \Phi)^2 - 0.3692\ \ln\ \Phi + 2.2024]^{\frac{1}{2}} + 0.8798\ \ln\ \Phi\} \qquad (12.03)$$

also:

$$\Phi = \frac{g\,F}{U_w^2} \qquad (12.04)$$

---

* Swell is an example of waves without wind. This is most often found along coasts bordering on the larger seas. On the other hand, a wind blowing from the shore will not generate much of a wave along a coast.

in which:

F is the fetch length,
g is the acceleration of gravity,
H_{sig} is the significant wave heigt,
d is the duration of the wind,
T_{sig} is the period of the significant wave,
U_w is the wind speed, and
Φ is the fetch parameter defined in (12.04).

Since equation 12.03 is rather cumbersome to handle and it is often necessary to determine a Φ corresponding to a given value of $\frac{g\,d}{U_w}$, this equation is represented in figure 12.1.

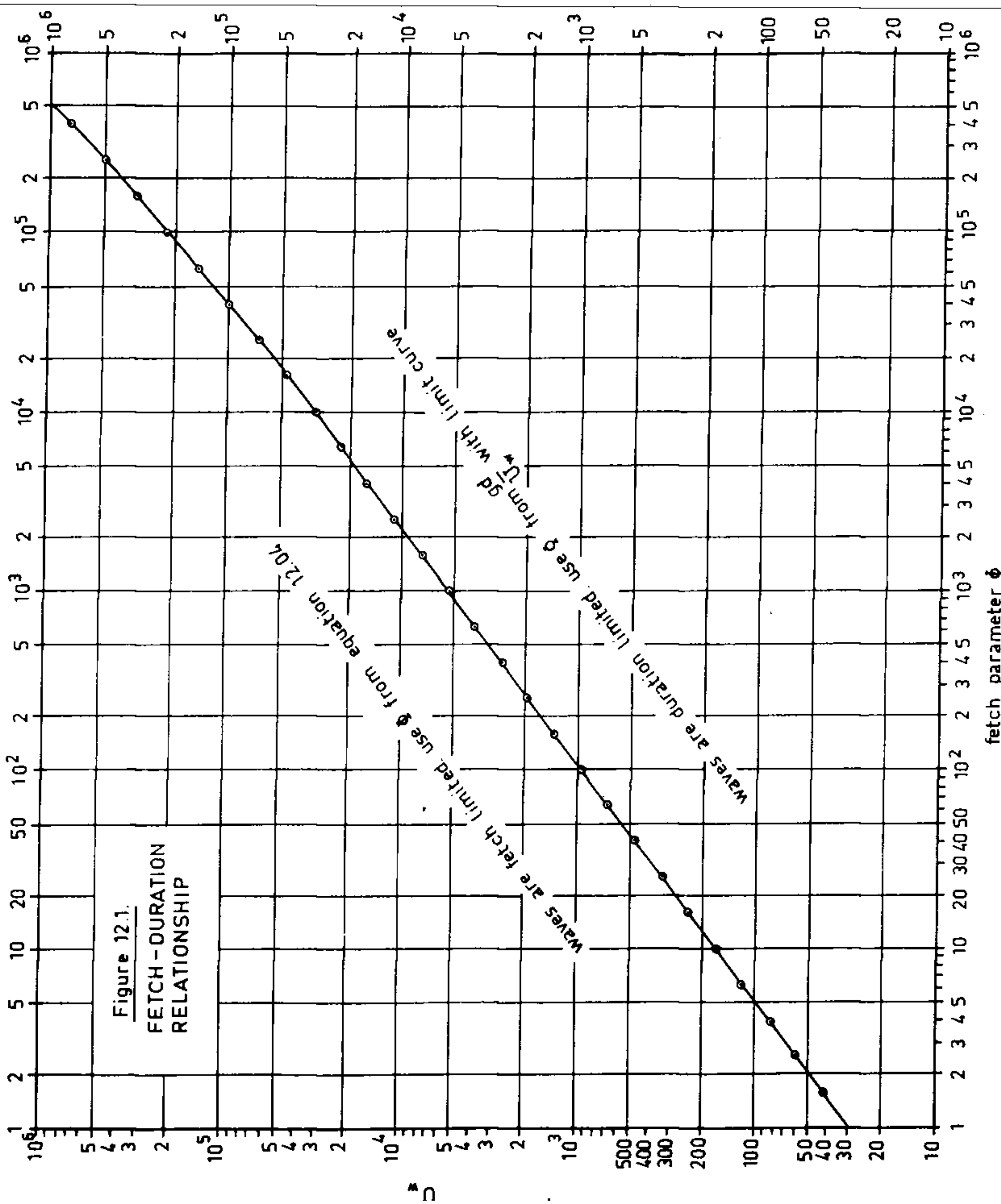

Figure 12.1
FETCH-DURATION RELATIONSHIP

These equations are valid for *deep water* only. Their use, in practice, goes somewhat as follows:

a. The fetch distance, F, the wind speed, U_w, and the duration, d, of this wind are determined from the available data.

b. Φ is the determined from 12.04 - be careful to use consistent units!

c. Compute the parameter $\frac{g\,d}{U_w}$ in the same set of units.

d. Enter figure 12.1 with the parameters evaluated in the two previous steps. If these parameters locate a point above the plotted curve, then the wave height is determined by the fetch - use the value of Φ determined in step b, above. If, on the other hand, a point is below the line, the wind duration determines the wave height - proceed by using a smaller value of Φ determined using the duration parameter and the plotted limit curve.

e. Using the verified (or corrected) value of Φ, the wave parameters, H_{sig} and T_{sig} can be determed via equations 12.01 and 12.02.

f. Wave heights with other probabilities of occurrence can easily be determined using methods outlined in chapter 11.

Another, purely graphical solution to this problem is given in volume I of *Shore Protection Manual* (1973).

A limiting assumption made in the SMB forecasting method is that the wind velocity and direction is constant for the storm duration over the entire fetch length. This is an important limitation if one is making a wave forecast for a large body of water. Research is now being conducted to develop better methods for use on extensive seas.

E. Allersma
E.W. Bijker
A. Paape

13. OPTIMUM DESIGN

13.1 Introduction

It has become obvious in chapter 11 that some risk in the design of engineering works must be accepted. The prime question, now, is, "what is the most responsible risk to assume?"

In this chapter we shall only discuss this problem in a general way. Specific coastal engineering applications will be taken up in later volumes of these notes. The optimum design technqiue can be used for a broad spectrum of projects.

13.2 Project Criteria

The project suitable for the optimum design technique must satisfy certain criteria:

1. There must be alternative solutions for the design available. It is sufficient to have similar structures which vary only in some detail such as size or strength.
2. It must be possible to evaluate the economic construction cost of each project alternative.
3. It must be possible to determine the chance of damage or failure of each alternative.
4. The economic loss resulting from damage to or the failure of our construction must be determinable.

We have seen in chapter 11 how item 3 of the above list can be evaluated for certain types of offshore constructions. The most difficult decision making is involved in the evaluation of item 4, above. The technical consequences of a "failure" are reasonably easy to evaluate; the social, environmental, or esthetic consequences are usually much more difficult to express in economic terms. However, we shall proceed, for now, assuming that the necessary costs can be expressed in economic units.

13.3 Optimization Procedure

The optimization process proceeds as follows:

a. A design from among the alternatives available from step 1, above, is chosen.
b. For this design, the total capital investment involved in construction is determined in convenient units such as current money value.
c. By multiplying the chances of damage or failure found in item 3, above, by the economic consequences of such damage we can obtain the current capitalized monetary value of total damage to be expected during the lifetime of the project.
d. we can repeat these three steps for each of the alternative designs available.

Once these steps have been carried out, we can choose that design which has the lowest total (sum of construction plus capitalized damage) cost. This is then our optimum.

An alternative reasoning is also possible which proceeds more or less as follows:

a. Choose a design from among the alternative designs available from step 1, above.
b. For this design determine the capitalized annual cost of the damage (failure) and the construction cost. Express both quantities in equivalent economic units.
c. Choose a second design from among those available and determine its damage cost and construction cost.
d. Evaluate the second design with respect to the first by comparing the *change* in capitalized construction cost to the *change* in capitalized damage cost.
e. Obviously, only if the capitalized construction cost increase is less than the capitalized damage saving is it economical to choose the second design, assuming that the second design is the more costly to build. Repeated application of this procedure should lead to the same optimum as the first procedure listed.

Specific examples of the optimum design technique are given in volumes II, III and IV of these notes.

13.4 Implicit Assumptions

It should be fully understood that the damage costs include both direct and indirect costs. Not only the structure must be repaired or replaced; there will usually be other losses due to interruption of production or even loss of human lives.

Also, the assumption has been made above that sufficient money was available to carry out the optimum solution. It is possible that when only a limited amount of capital is available now, one must choose a solution which costs less to build, but which has a greater capitalized damage cost. Since such evaluations involve some more complex economic and financial principles such as cash flow, we shall not develop this discussion further.

Another boundary condition to consider in optimilization problems is the existence of design codes. Obviously, all designs must satisfy all applicable design codes specified by law. It is conceivable that such a code may dictate the design of a construction which is too conservative (overdesigned) to meet our optimization criteria.

14. HISTORY OF HARBOR DEVELOPMENTS

E.W. Bijker

14.1 Introduction

Now that the background information has been presented in the first 13 chapters, we are prepared to start applying this to specific coastal engineering problems. We shall begin this study of applications by examining the oldest of the three major subdivisions mentioned in chapter 1, the problem of providing safe harbors for ships. Various details of harbor problems form the subject of this and the next 10 chapters of this volume. Some further details are deferred to volume II; breakwaters form the topic of volume III of these notes.

The remainder of this story is adapted from Bijker (1974).

14.2 Early History

Originally, harbors were built at locations where both good hinterland connections and protection from the evils of the sea were naturally available. These evils of the sea include both natural (waves and currents) and human (pirates) enemies. Since settlements developed around the harbors, sites were usually inland at least far enough to assure dry land. Harbors developed sometimes well inland along rivers or estuaries. New Orleans, for example, is more than 100 km up the Mississippi River from its mouth.

Since ships were small some hundreds of years ago, their shallow draft allowed them to navigate easily over and around the numerous shoals found in these natural watercourses. This meant, even so, that local knowledge of the waterway was needed. Was this a disadvantage or an advantage for shipping? The use of pilots did hinder commerce somewhat, but it hindered pirates even more![*]

As time passed, and ships became larger and deeper, the difficulties with shoals increased. The use of pilots became more common; they knew the deepest channels. Inventive people even developed strange-seeming devices to reduce the draft of ships. One of the successful devices, a "ship camel" was designed and used to help ships cross the bar near the island of Pampus as they approached the port of Amsterdam. Such a camel is shown schematically in figure 14.1; this was really the predecessor of the floating drydock.

14.3 The influence of Dredging

More than 100 years ago another inventor came upon the idea of artificially deepening the shallow areas by means of underwater excavation - dredging. Sometimes even whole new channels were excavated. Both major ports in The Netherlands, Rotterdam and Amsterdam, have experienced this. Both have dredged entirely new artificial channels or canals to reach the sea. Interestingly, both harbors have abandoned their original artificial channels in favor of larger, improved ones.

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[*] Lord Nelson's fleet shipyard and harbor at Buckler's Hard on the Beauly River would have been a juicy prize if it could have been successfully attacked.

Amsterdam abandoned the North Holland Canal linking it with den Helder in favor of the present North Sea Canal forming a much shorter link to IJmuiden. The Rotterdam Waterway is also a major man-made construction.

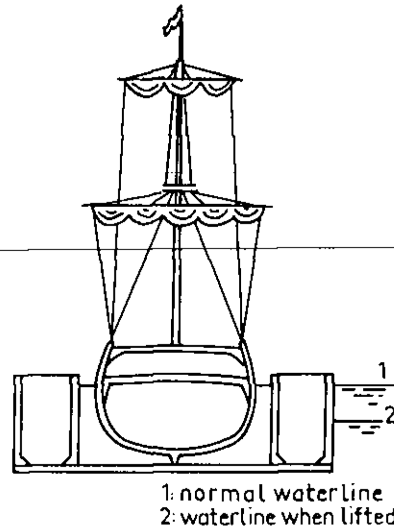

Figure 14.1.
SHIP CAMEL

14.4 Modern Developments

As ships become even larger the dredging required to open and possibly maintain such a channel over a long distance becomes a formidable economic burden. Also, the long sailing distance through such canals by the modern very large oil tankers presents a hazard to navigation. Since these canals pass close by densely populated areas, as well, the risk of social damage from calamities increases. Moreover, as ships carry higher value cargo such as containers, the time lost in navigating along a long canal has an increasing economic impact. In general, goods move faster over land than over water.

These factors, along with the decreasing threat by pirates have led to decisions to expand the harbors nearer to the shoreline. The Maasvlakte and Europoort in Rotterdam are examples of this. Many other harbors, such as London, Amsterdam, and Hamburg, are also at least planning similar developments. Many of these new harbor areas are developing on artificially filled land. The scarcity of land in the older urban areas has contributed to this harbor migration toward the sea.

This migration has not ended. Specialized facilities offshore are also developing rapidly. We need only to think of the plans for island harbors at sea or of the development and use of Single Buoy Moorings which can replace conventional harbors for some cargos.

In the next 10 chapters we shall examine some of the more specialized harbor problems in more detail.

15. APPROACH CHANNELS

W.W. Massie

15.1 Introduction

We might easily conclude from the previous chapter that all necessary dredging work needed to accommodate even larger ships takes place within the harbor or estuary. This is certainly not the case. Considerable dredging work is now carried out in more or less open sea in order to provide safe approaches for the largest ships. The improved channel for the Port of Rotterdam, for example, extends more than 35 km seaward from the harbor entrance; this is a rather extreme example. What are the consequences of such channels in sea? This is answered below.

15.2 Problems Encountered

When a river channel is deepened in order to accommodate larger ships, we need to contend with currents and sediment transport which are directed along the axis of the channel. While some sedimention can be expected, much and often most of the sediment which enters a channel reach is merely swept along by the current and out the other end. Ships navigate easily either with or against the current and have little difficulty manoeuvering.

Does this situation change at sea? Yes, usually it does. Currents and sediment transports are often directed at a considerable angle to the axis of the channel. Sediment transport rates perpendicular to the channel axis are often very high where the channel cuts through the shallowest coastal regions. This results from the high sediment transport caused by the breaking waves along the adjacent shores. These breaking waves can also generate a crosswise current in the approach channel which can be dangerous for shipping, especially if the ships are moving slowly - they are harder to steer, then. One of the reasons to construct breakwaters out from the coast at a harbor or river entrance is to cut off, or at least reduce or possibly divert, the longshore current and sediment transport. Breakwaters are given a separate discussion in chapter 18 of these notes. The causes and effects of longshore current and sediment transport are highlighted briefly in chapter 26. Detailed treatment of this is taken up in volume II.

Farther at sea, outside the area where waves are breaking nearby, the problems are less severe. There are still currents and thus sediment transport caused by tides, but these usually do not create such serious problems as we encounter nearer to shore.

Other problems can become increasingly important farther offshore however. Position determining and navigation systems become less accurate; wider channels are needed to assure that ships do not run aground. Wave action can be more severe. This leads to more severe ship motion which requires a deeper channel to assure that the ship does not hit the bottom. Dredging is hampered in the same way by these problems too, of course.

15.3 The Optimazation Problem

It should be obvious that design decisions must be made with regard to approach channels. Alternative solutions - various widths and depths - are available for a given problem. This problem lends itself well to an optimization procedure. This optimization will be discussed in detail in volume II. A constructional aspect, dredging, is taken up in the next chapter.

16. DREDGING EQUIPMENT

J. Brakel

16.1 Introduction

Dredging is necessary at sea and in rivers and harbors for nearly all of the modern ports of the world today.* What equipment is best suited to carry this out? After briefly explaining some general principles common to most all sorts of dredges, the most commonly available types will be listed and described. Neither part of this discussion is intended to be complete. Other courses specificly on dredging serve their purpose well.

16.2 General Principles

Most dredging is done by hydraulic dredgers. These machines are equipped with centrifugal pumps capable of pumping a mixture of soil and water. The soil is moved as a suspension in the moving water.

The ratio of soil to water in this suspension is an important factor establishing the efficiency of the operation. This ratio depends both upon the equipment used and the soil material. The amount of mixture required to move one part by volume of in situ soil varies from 1 to 2 for mud to 3 to 5 for medium sand ($d \approx 250\ \mu m$). For gravel and rock this ratio can be as high as 10 to 12.

Since the type of pump and the installed power more or less fix the quantity of mixture that can be moved, the productive output of the dredge is strongly dependent upon the ratio listed above.

The head loss in the pipelines increases with increasing soil grain size. The maximum length of discharge pipeline decreases with increasing grain size, since the maximum head (pressure) of the dredge pump is more or less constant. Some rather inaccurate emperical formulas have been developed to predict the head loss in dredge pipelines. See Führböter (1961), for example.

The head loss can be reduced, in theory, by reducing the flow velocity. This can only be done to a certain extent, since a minimum velocity must be maintained in order to assure that the dredged material remains in suspension.

This minimum velocity increases with increasing grain size, grain specific weight, and pipe diameter.

16.3 Plain Suction Dredge

This is one of the most common types in Holland. Fig. 16.1 shows such a dredge designed to discharge through a floating pipeline. Fig. 16.2 shows a modified form designed for loading barges alongside.

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* Another purpose of dredging can be to obtain fill material for land reclamation. This type of dredging is often conducted completely independently of harbor development.

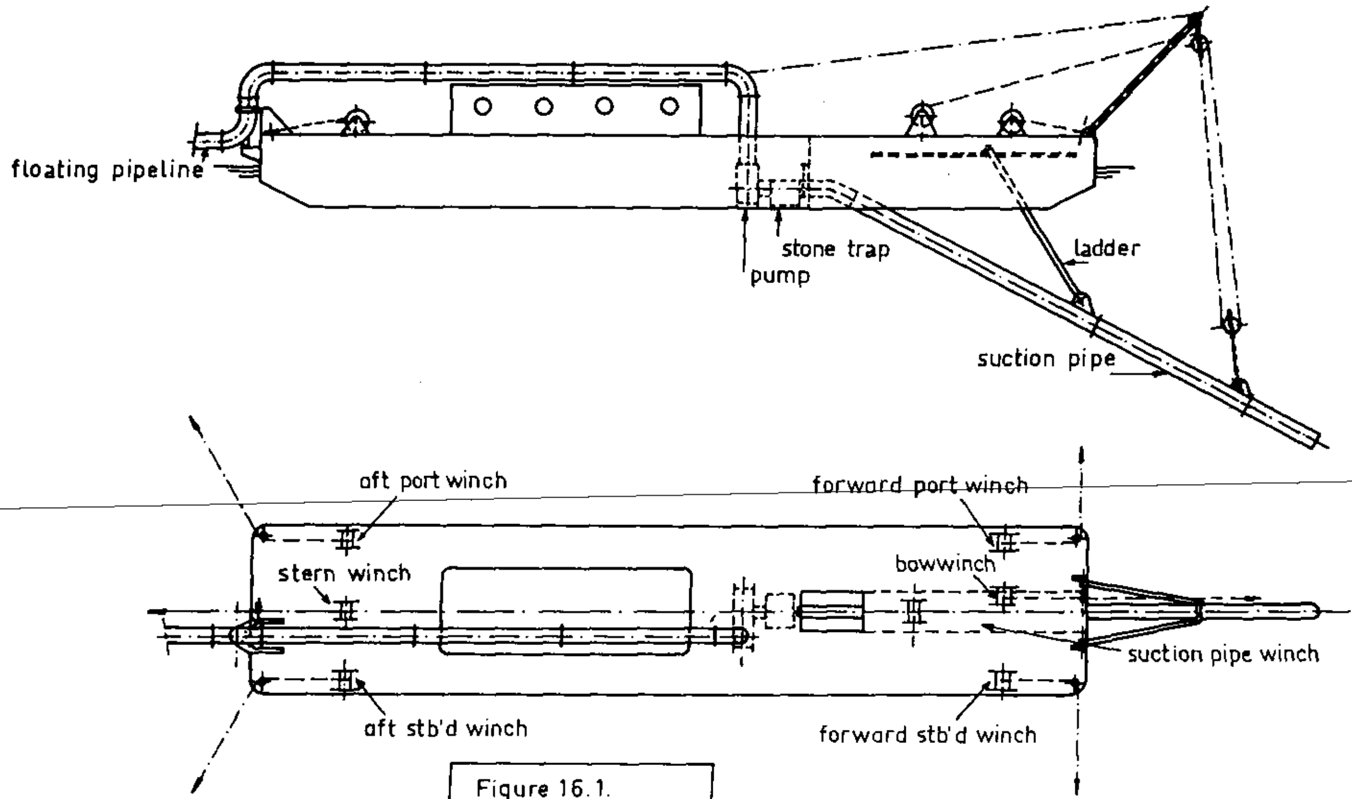

Figure 16.1.
SUCTION DREDGE

These dredges are very efficient when dredging loose material such as sand. If a water jet is added at the end of the suction pipe to help loosen the in - situ material, then layers of clay can be penetrated to reach deeper sand layers.

These dredges are kept in place by six anchors and move very slowly in the direction of the bow anchor. This type of operation results in a very uneven bottom bathymetry making these dredges more useful for reclamation works.

The cost of transporting one unit of soil with this type of dredge is relatively low.*

The production from such a dredge depends upon many factors including the sand grain size and porosity as well as the suction pit geometry.

In order to increase the depths attainable with suction dredges, it has become necessary to place the pump deeper under water. Often, the pump is located along the suction pipe instead of in the hull as shown in the figures. Sometimes, even, two pumps are used in series - one on the suction pipe and one in the hull. The interrelationship between the factors affecting dredge performance is revealed by the so-called "suction equation". It is derived by examining the pressure change along the inside of the suction pipe between the inlet and the pump. See figure 16.3.

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* Exact cost figures are not given since they can vary too rapidly.

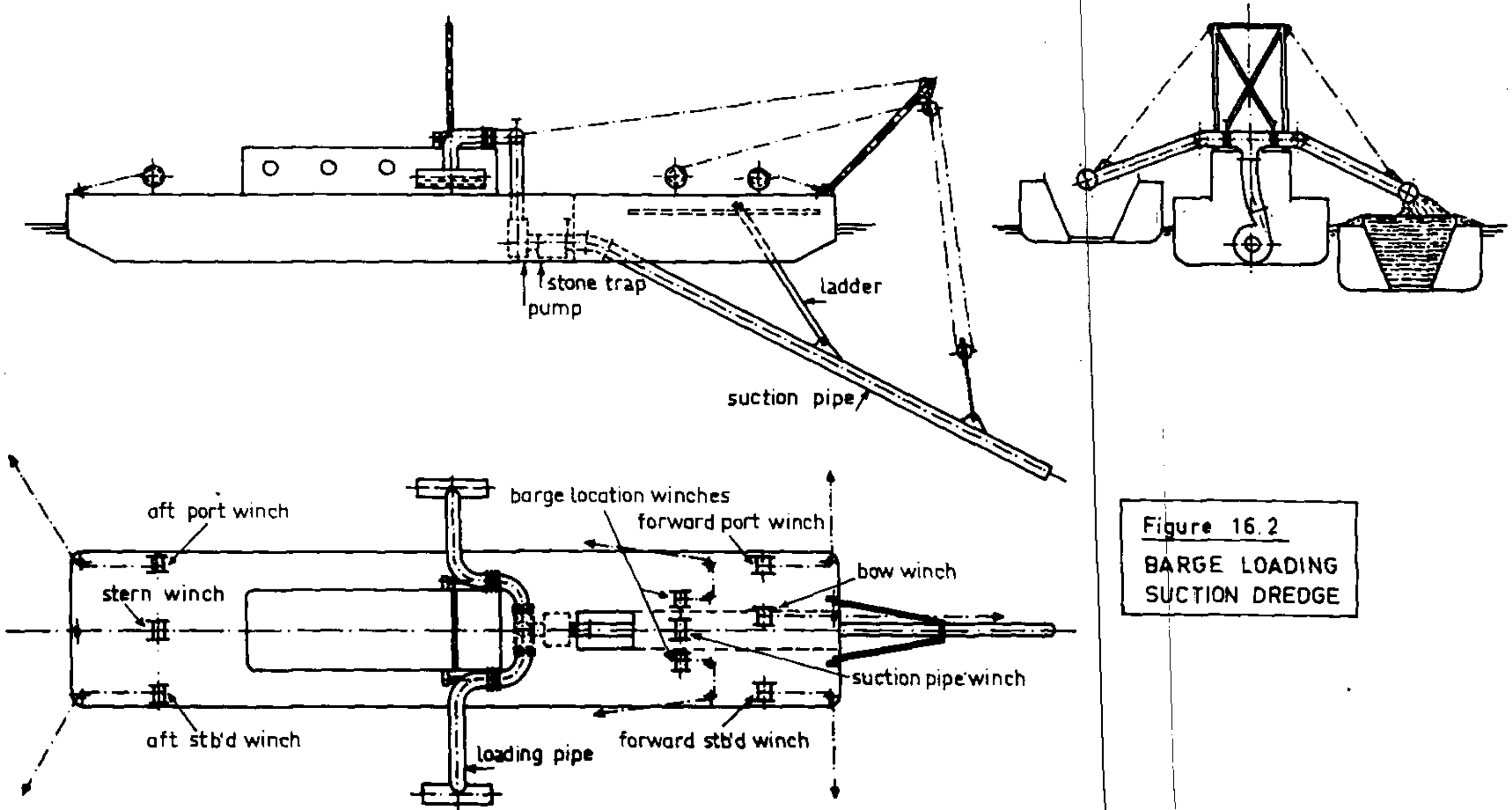


Figure 16.2
BARGE LOADING
SUCTION DREDGE

$$(p^* + Z_s)\ \gamma_w = (Z_s - Z_p + f\,\frac{V_s^2}{2g})\ \gamma_m \qquad (16.01)$$

where:

- f is the hydraulic loss coefficient from suction pipe entrance to pump,
- g is the acceleration due to gravity,
- V_s is the flow velocity in the suction pipe,
- p^* is the vacuum at the pump entrance expressed as head of water,
- Z_p is the depth of submergence of the pump,
- Z_s is the depth of the suction pipe extrance,
- γ_m is the specific weight of the mixture, and
- γ_w is the specific weight of water.

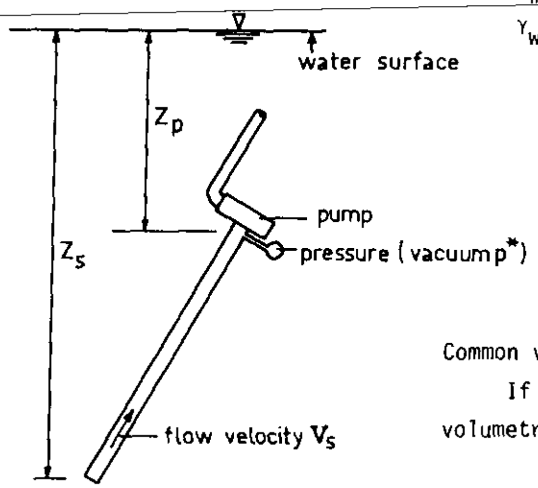


Figure 16.3
DEFINITION OF SUCTION EQUATION TERMS

Common values for f range between 2.5 and 3.5 .

If the concentration of soil in the mixture measured on a volumetric basis is c_v, then:

$$\gamma_m = c_v\ \gamma_g + (1 - c_v)\ \gamma_w \qquad (16.02)$$

where γ_g is the specific weight of the dry sand grains.

Solving for c_v yields:

$$c_v = \frac{\gamma_m - \gamma_w}{\gamma_g - \gamma_w} \qquad (16.03)$$

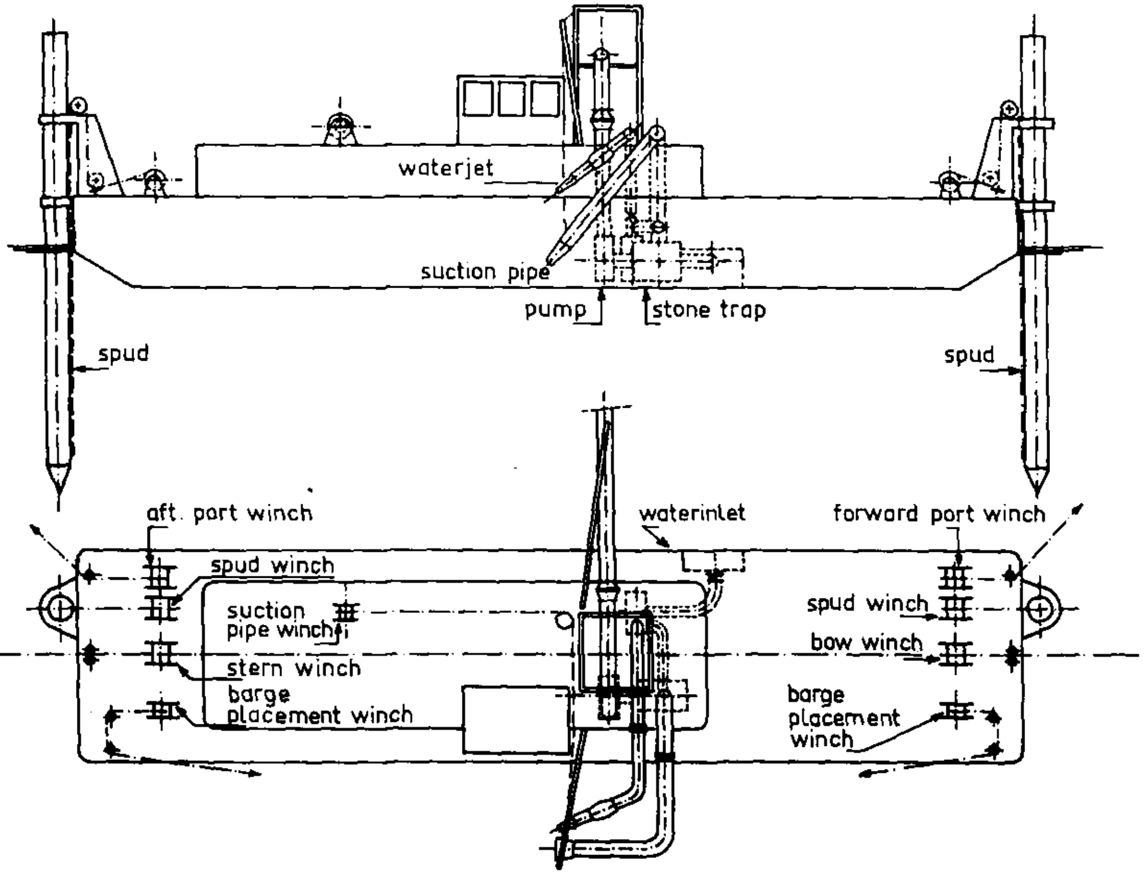
Combining equations 16.01 and 16.03 yields:

$$c_v = \frac{\gamma_w}{\gamma_g - \gamma_w}\left[\frac{p^* + (Z_p - f\,\frac{V_s^2}{2g})}{Z_s - (Z_p - f\,\frac{V_s^2}{2g})}\right] \qquad (16.04)$$

The capacity of the dredge increases as the vacuum, p^*, and the depth of pump submergence, Z_p, increases. Increasing suction submergence, Z_s, and friction factor, f, tend to decrease capacity.

With submerged pumps, these dredges can work with very deep suction depths (70 meters can be reached when necessary).

A slight modification of the standard suction dredge is the barge unloading dredge shown in figure 16.4. Their principle of operation is the same as that for a suction dredge; their use is also very common in Holland.



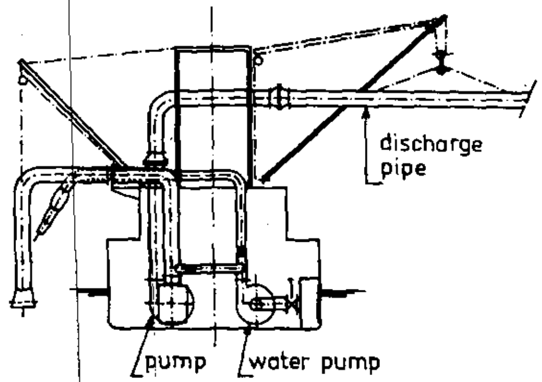


Figure 16.4
BARGE UNLOADING
DREDGE

16.4 Cutter Suction Dredge

The cutter suction dredge is a more versatile form of suction dredge and is shown in figure 16.5. It can handle materials ranging from mud to soft rock. The soil in front of the suction tube is loosened by the cutter. The dredge moves into new material by swinging back and forth about one of the two anchoring spuds using the port and starboard forward winches and anchors. The dredge moves forward by changing the position of one spud while the dredge is held in place by the other. This moves its center of rotation forward a bit.

Equations 16.01 through 16.04 apply to the cutter suction dredge as well. Additional limiting factors are possible, however. These include the cutting capacity of the cutter and the hauling force and speed of the forward winches.

Since the suction dredge takes its material from a small area, it is capable of being accurately controlled. A depth accuracy of $\pm$ 0.25 m can be achieved, and side slopes can be cut.

Smaller cutters are used in soils such as sand, peat, and soft clay* which require relatively little cutter or side winch power. Bigger, more powerful cutters are used with stiff clay and soft rock. Rock having a compressive strength of up to about 5×10^7 N/m^2 can be attacked succesfully with a cutter suction dredge. Harder rock must be handled with explosives.

The maximum depth which can be obtained is about 25 meters. This limit is imposed by structural stiffness limits of the spuds and ladder.

Typical production capacities for a "big" cutter suction dredge are listed in table 16.1.

To achieve the productions listed in this table, the dredge with submerged pump was operating as follows:

dredging depth : 20 m
pipeline diameter : 0.8 m
installed power : 5600 kw (7500 HP)

Table 16.1 Typical Cutter Suction Dredge Production

Material (see text)	Production (m^3/hr in situ)	Maximum discharge distance (km)
sand	1500	3 to 6
soft clay	1750	6
stiff clay	750	3
soft rock	400	1.5

* Soft clay has a cohesion of less than 3×10^4 N/m^2.

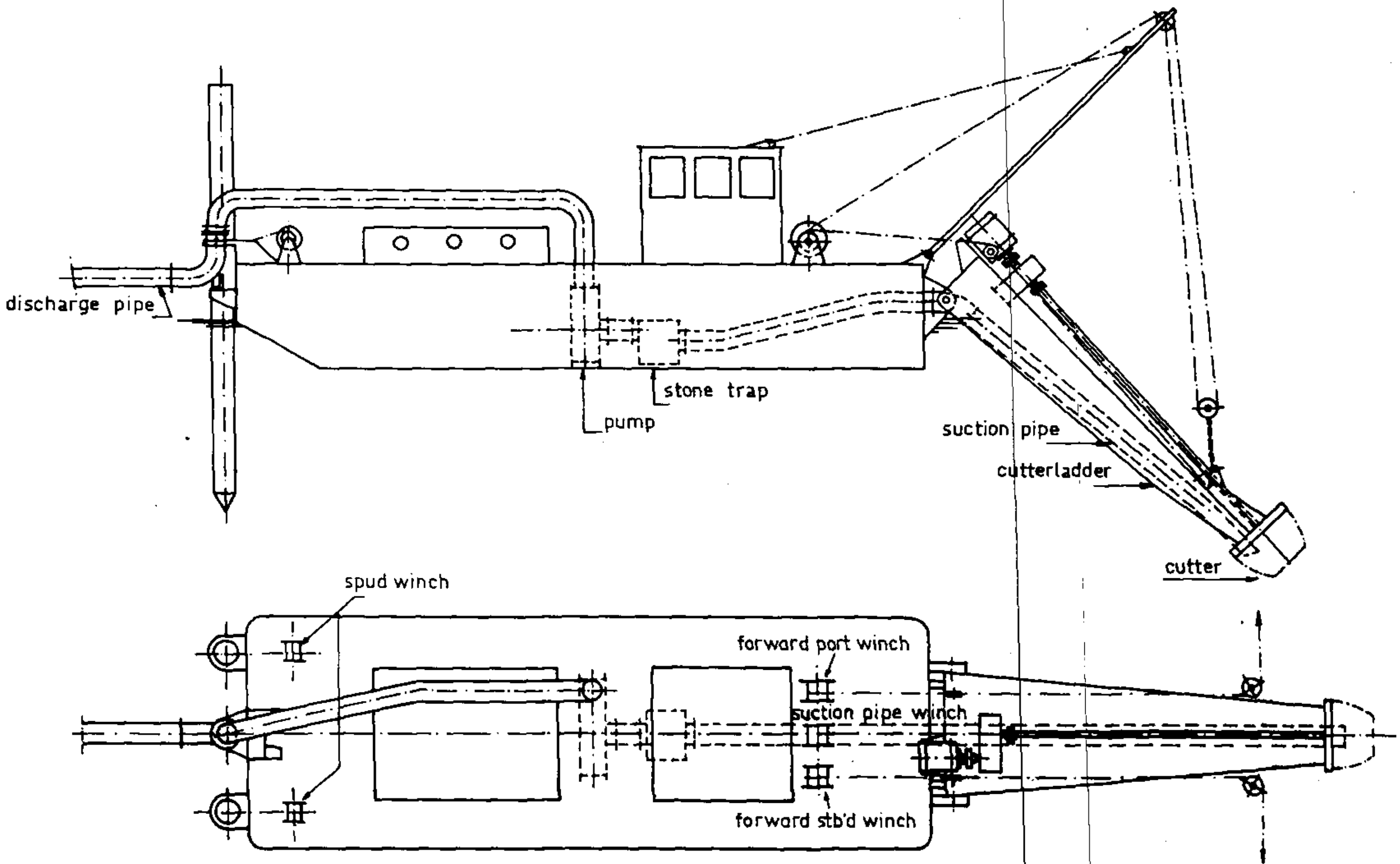


Figure 16.5 CUTTER SUCTION DREDGE

16.5 Trailing Suction Hopper Dredge

This type of dredge, shown in figure 16.6, is a ship with towed suction tubes. These ships pick up their load while travelling slowly (a few knots) forward. This ability to load while underway makes it an ideal dredge for work in waterways with heavy traffic. Furthermore, unlike stationary dredges, it can operate in moderate to high seas*. Thus, this is the most suitable dredge for work in exposed harbor approach channels and at sea. These dredges can handle either mud or sand.

It usually takes 1 to 3 hours to fill the hopper of such a dredge with sand. This time is dependent upon the dredging depth, and grain size and porosity of the sand layer. Only ½ to 1 hour is needed to fill the hopper with mud.

After filling the hopper, the ship sails to a dumping site at a speed of about 11 knots (20 km/hr). The production achieved by such a dredge is dependent upon its pumping and hopper capacity, sailing distance to the unloading site, and the time required to unload. The maximum hopper capacity available in 1975 is about 10000 m^3.

These dredges can work with a depth accuracy of about $\pm$ 0.5 m. up to a maximum depth of about 35 m.

16.6 Bucket Dredge

Unlike the preceeding types, this is a purely mechanical dredge. It loosens the soil and transports it upward by means of a continuous conveyor chain of buckets. See figure 16.7. At the top of the chain of buckets, they dump their load of material into a chute which directs it into a barge moored along side.

The bucket dredge is positioned by six anchors. It moves in an arc around its bow anchor, drawn by its side winches, in order to reach new material. The bottom profile accuracy is of the order of $\pm$ 0.2 m.

A great variety of soils can be excavated with such a dredge. All materials ranging from mud to soft rock can be handled directly. Also, they are well suited for cleaning up the broken stone after explosives have been used. This is because they can handle larger pieces than other types of dredges (up to about 1 m diameter).**

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*A very few of the large, modern plain suction dredges can operate even when they are exposed to light wave action.

**By comparison, other dredges are limited by the pump impeller dimensions to about 0.4 m.

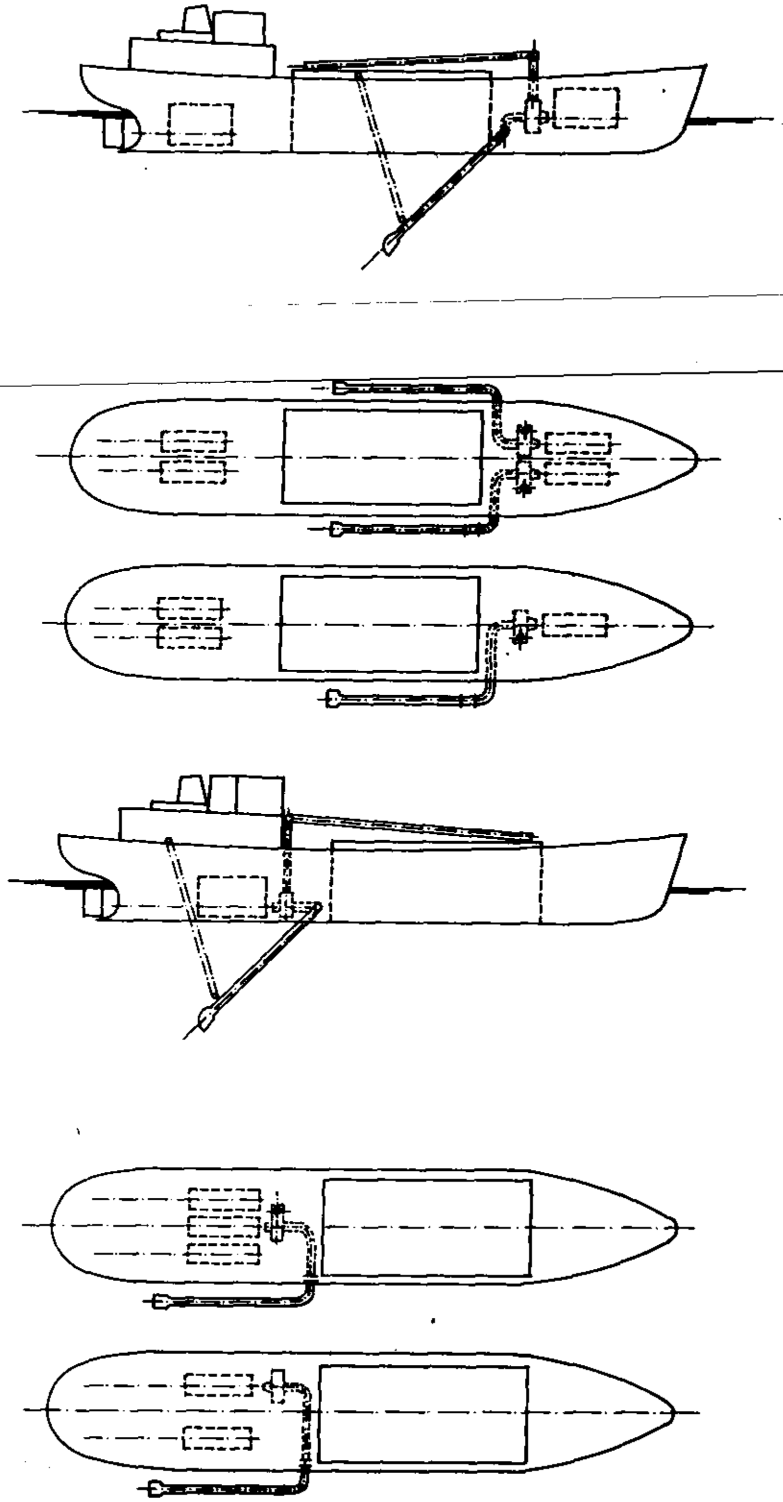

Figure 16.6
TRAILING SUCTION HOPPER DREDGES

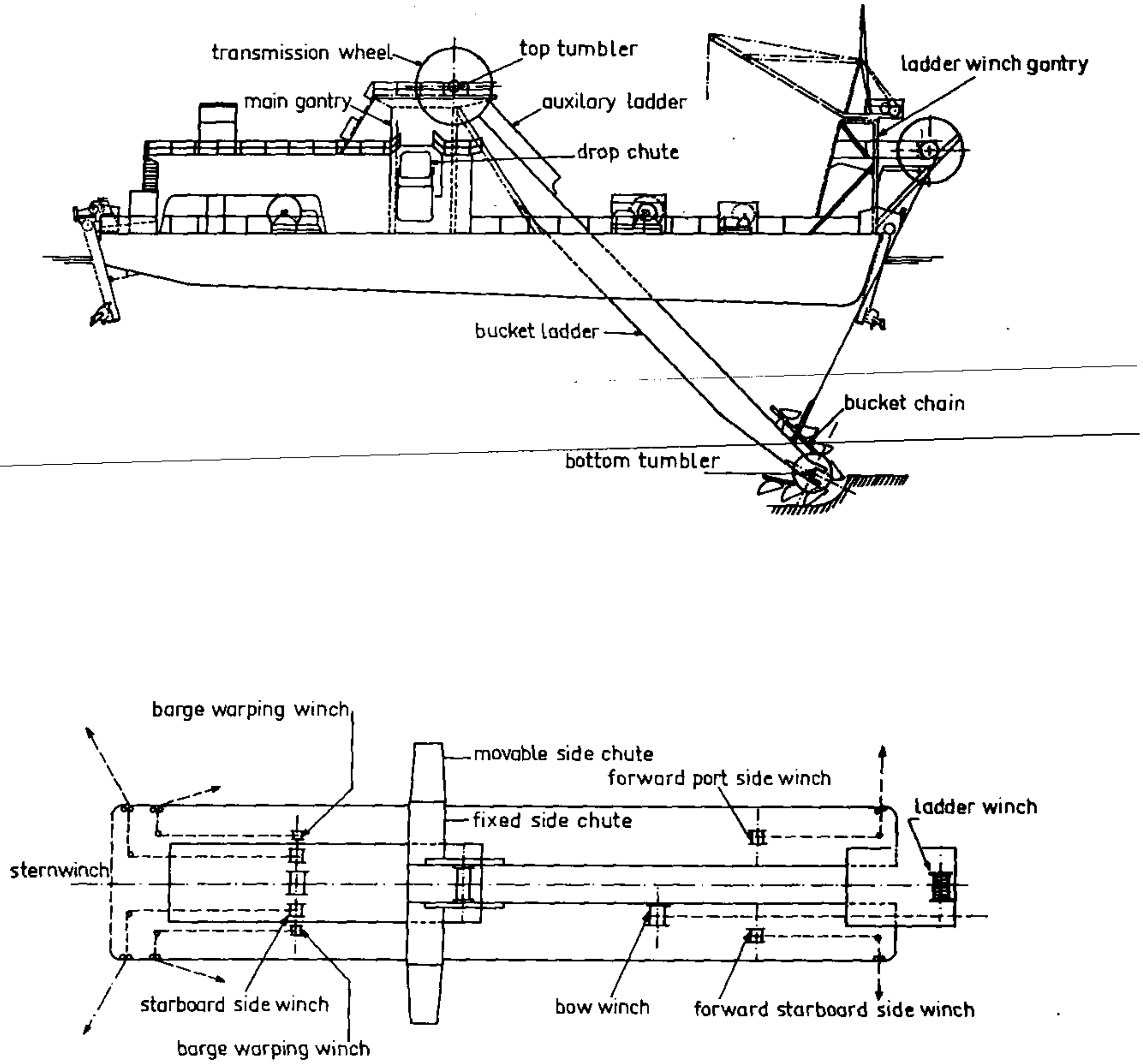

Figure 16.7
BUCKET DREDGE

The maximum depth attainable with these dredges is about 40 m. Bucket sizes range up to about 1 m^3 volume, and a bucket chain speed of about 30 buckets per minute is a maximum. Typical production capacities for a bucket dredge are listed in table 16.2. These productions are listed for the following condition:

bucket volume : 0.75 m^3
dredging depth : 15 m
installed power : 375 kw (500 HP)

Table 16.2 Typical Bucket Dredge Production

Material	production in m^3/hr in situ
mud	1000
clay	500
sand	350
broken rock	75
soft rock	50

16.7 Further Developments

Only the most commonly used dredges have been listed. Other types such as side casting and dipper dredges are sometimes used for special applications. Developments in dredging are aimed at achieving economics of scale or at increasing the productivity of equipment under adverse conditions.

Recently, concern for the environmental consequences of dredging and spoil disposal has increased sharply, especially in the United States. Spoil disposal is the subject of the next chapter.

The proceedings of the various World Dredging Conferences (WODCON) are helpful when attempting to keep abreast of the latest developments.

17. DREDGING SPOIL DISPOSAL

J. de Nekker

17.1 Introduction

It has been described in the previous chapter how material can be dredged from the oceans, lakes, and waterways. When this spoil material is obtained for a specific purpose such as land reclamation or use as concrete aggregate, then there is usually little direct spoil disposal problem. Cressard (1975)* describes some of the indirect environmental consequences of such dredging, however. The dredging operation itself is usually carried out at a location dictated by the quality of the bottom material rather than by navigational considerations.

When navigational considerations do dictacte the site of the dredging, we are often faced with a substantial spoil disposal problem. Because the type and quality of material dredged is no longer a chosen variable, we are often faced with the problem of disposing of a relatively poor quality spoil (one for which there is no immediate use). This material may be disposed of on land or dumped into another body of water.

17.2 Marine Disposal

Obviously, the disposal of a large quantity of foreign material in a body of water - even an ocean - can have serious consequences for the marine biology+. Many of the deleterious consequences can be avoided by disposing of the material only in the deepest ocean trenches. Unfortunately, the prohibitive shipping costs make transport to such sites uneconomical except for disposal of the most lethal materials such as certain radioactive wastes.

Research is only now beginning on the impact of the disposal of dredged material in the areas nearer to shore. This work is beging carried out primarily by the biological oceanographers.

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*The journal in which this article appears, serves as a current awareness journal for the dredging industry.

+This is an example of how coastal engineering relates to biological oceanography. See chapter 3.

17.3 Land Disposal

It is sometimes more economical or desirable to dispose of mud, clay, and sand dredged from harbors reasonably nearby on land rather than at sea. In this case, the spoil material mixed with water is pumped ashore via a pipeline. The further disposal procedure is dependent upon the type of material.

If the material is sand having a grain size greater than about 100 μm, then it is possible to simply discharge the mixture at the disposal area and let the excess water flow away directly. This technique works well, provided that the layer of sand to be built up is at least 0.75 m thick. The sand layer develops sufficient bearing capacity almost immediately to allow construction equipment to work and extend the discharge pipeline.

With finer sands containing some mud and clay, the bearing capacity of the filled area often does not develop rapidly enough to allow construction equipment to work. Work must sometimes be stopped for a time before the discharge pipeline can be extended.

As the dredged material becomes finer, it has a greater tendency to be washed away with the drainage water. Sometimes, it is sufficient to use a bigger discharge area or to take special measures to prevent the discharge of fine material in suspension in the drainage water.

When very fine material such as mud or clay is being handled, it is often discharged into an area which has been dammed off from its surroundings. Only after most of the solid material has settled is the surplus water allowed to flow away. After the surface water has flowed away, a layer, optimally about 1.3 m thick, of saturated mud remains. Drainage channels are made through this to allow the remaining pore water to escape. After a period of a bit more than a year, the layer has consolidated to a thickness of about 0.9 to 1.0 m. At this time new dikes can be built in order to add an additional layer of mud, or the created land surface may be developed for other uses such as agriculture or recreation.

18 BREAKWATERS

J.J. van Dijk

18.1 Introduction

We have just seen in the previous three chapters that as ships become larger the dredging problems associated with harbors become greater. This often necessitates that we continue dredging operations in much more open water. Since the wave action in this open water can complicate the dredging problem, it is often economical to take measures to protect the harbor entrance channel. Protection can be provided to make dredging operations more efficient or to reduce the amount of dredging necessary. Of course, breakwaters can occasionally serve other purposes as well such as providing quay facilities for ships.

Details of breakwater construction are the subject of volume III of these notes. We are concerned here only with their use with regard to morphological problems.

18.2 Morphological Functions of Breakwaters

How can breakwaters be used to help solve approach channel dredging problems; There are several ways:

a. They can reduce the wave action in the approach channel so that dredging equipment can operate more efficiently. Most dredging equipment must remain stable in the waves in order to operate effectively. Reduced wave action may make it possible to select and use more efficient equipment for the dredging operation.

b. By extending through the breaker zone breakwaters can block the longshore transport of sand which could otherwise settle into the dredged channel. By blocking this transport of material by wave action, the amount of necessary maintenance dredging can be significantly reduced in many cases. Obviously, we must be careful when we interrupt this longshore sand transport. Material which once simply passed along the coast (before the channel was built) will now pile up against the breakwater; coast erosion can be expected on the opposite site of the approach channel.

c. When there is a large sediment supply from a natural river flowing through the harbor, then a shoal can often be expected slightly offshore from the river mouth*. This shoal can form a major obstacle for shipping, especially during a severe storm when waves can be breaking on this shoal. Many ships have been wrecked attempting to cross such a bar under these circumstances. Breakwaters built out in such a way that the entrance is kept narrow until deeper water is reached increase the entrance current velocities; the resulting increased sediment transport capacity tends to keep the entrance open. Where does this material then go? Some of it will probably be

*Such a shoal can also form after the accretion mentioned in item b, above, has reached the end of the breakwater and material passes around the end.

deposited in deeper water offshore while some may settle out further upstream in the river. Indeed, since the river has in effect been made longer by building the breakwaters, the slope must slightly decrease and a siltation problem can appear inland from the entrance. At least, this material can be dredged out under excellent conditions - no wave action and often less shipping traffic. An example of a harbor entrance where breakwaters are being built to constrict the entrance is shown in figure 18.1. The channel dredged through the shoal is shown by the dashed lines. In this case, the river is the Columbia discharging into the Pacific Ocean on the west coast of the United States.

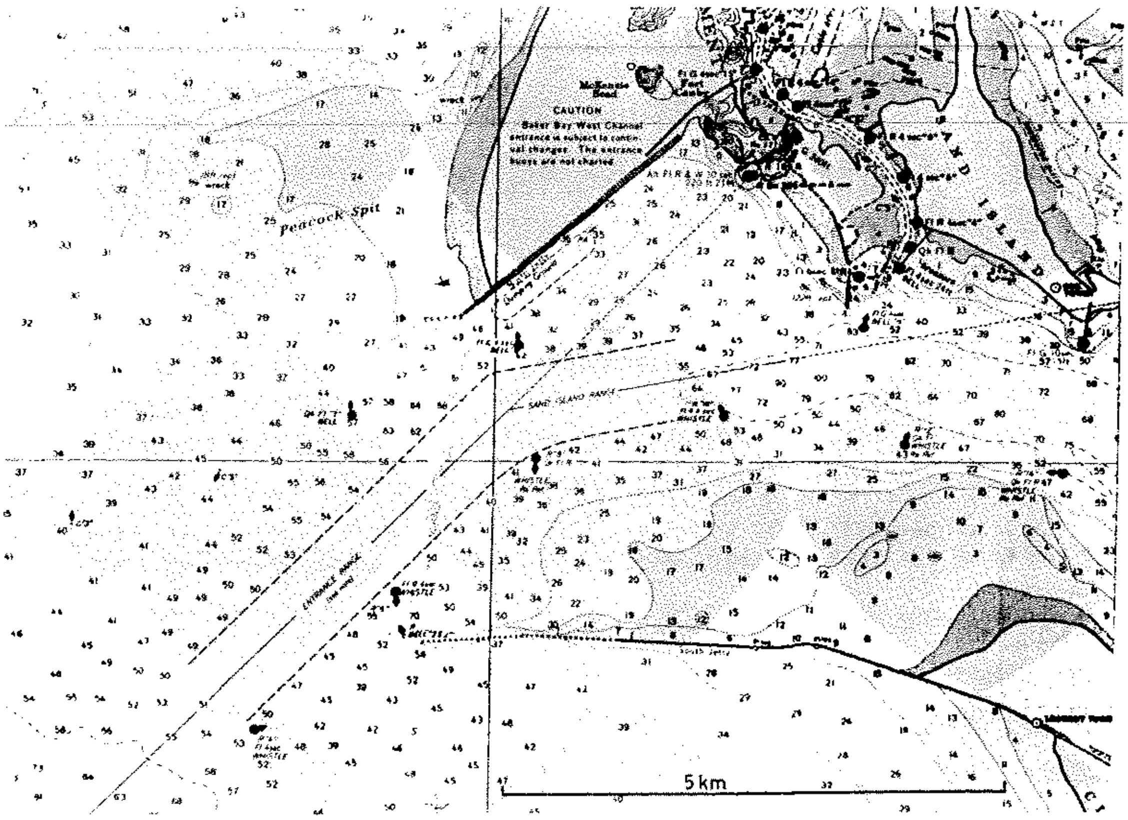

Figure 18.1 COLUMBIA RIVER ENTRANCE, U.S. WEST COAST
Scale - as shown depths in feet

18.3 Other Considerations

Another interesting question is, "What is the optimum depth of such an approach channel?" This is the topic of discussion in chapter 5 of volume II. By reducing wave action in a channel, ship motion is reduced and it can be possible to let ships navigate in a bit shallower channel than would otherwise be possible.

Of course, breakwaters extend and alter the shape of our harbor. These changes will, thus, also modify the natural resonant period of standing waves in the harbor. If the new resonant period should happen to correspond with that of a tidal component, a multiple of a persistent ocean well, or a gust oscillation, then some significant problems can be expected in the harbor. These resonances, or seiches, are treated in the following chapter. Their effect on moored ships is discussed in chapter 4 of volume II.

19. SEICHES

L.E. van Loo

19.1 Definition

Strictly speaking, seiches are free standing wave oscillations in a closed body of water. The surges occasionally experienced in Lake Geneva fit this strict definition, for example. This type of phenomona can be caused by, for example, atmospheric pressure variations, or abrupt in or out flow of large quantities of water.*

The term seiche is also used to describe standing wave action sometimes found in harbors. These waves have relatively long periods and low amplitudes when compared to the waves described in chapter 5. In harbors which are not completely closed, other driving forces can be present. Tidal influences and long period swell in the adjacent ocean can excite seiches in harbors.

Another term,"range action",is often used to refer to seiches. This term is also used to refer to the motion of moored ships resulting from seiches.

19.2 Simple Cases

The simplest true seiche (in a closed water body) is a standing wave with a node in the middle of the basin and an antinode at each end. The basin length is then equal to one half the wave length as shown in figure 19.1. For such a long wave:

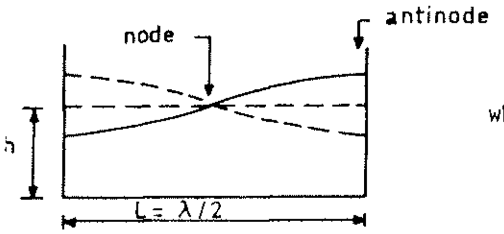

Figure 19.1
STANDING WAVE IN CLOSED BASIN (distorted scale)

$$c = \sqrt{gh} \qquad (5.05\ b) \qquad (19.01)$$

where: c is the wave speed,
g is the acceleration of gravity, and
h is the mean water depth.

Applying this to the basin in the figure, the wave period ,T, can be computed:

$$T = \frac{2\ L}{\sqrt{gh}} \qquad (19.02)$$

Using Lake Geneva as an example, we find, with

L = 90 km and h = 200 m that T is about 1 hour 8 minutes.

When a harbor is connected to the sea, a node can be found at the entrance and an antinode will be found at the end of the harbor basin. In this case, the wave length can be 4 times the harbor basin length as shown in figure 19.2. In this case:

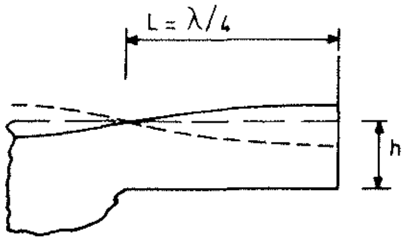

Figure 19.2
SEICHE IN HARBOR BASIN (distorted scale)

$$T = \frac{4\ L}{\sqrt{gh}} \qquad (19.03)$$

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* Seiches are often observed in shipping locks on a small scale.

Other possibilities are also possible, however. In general:

$$T_i = \frac{4\ L}{(i)\ \sqrt{gh}} \qquad (19.04)$$

where i is an odd integer: 1, 3, 5, ... note that as i increases the period of the i^{th} harmonic wave decreases. Also, equation 19.03 is the same as 19.04 with i = 1; this yields the first harmonic, or primary wave. An example with i = 5 is shown in figure 19.3. Thus, seiches having several periods can be sustained in a given harbor basin. In practice, i in equation 19.04 will usually be small; most often 1.

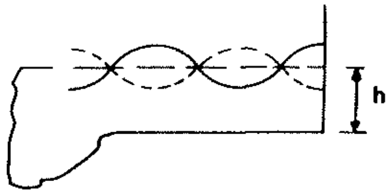

Figure 19.3
FIFTH HARMONIC SEICHE
(distorted scale)

19.3 Effects of Seiches

Usually, the vertical amplitude of a seiche, even at an antinode, is small. However, especially at a node, the *horizontal* displacement of the water can be very significant. Since moored ships are simply drawn along with the water, they can have mooring difficulties if they happen to be near a seiche node. Another related influence on the large ships is the effect of the water surface slope.*

19.4 Seiche Prevention

Seiches have long periods and small amplitudes. The waves cannot be broken on a shore and since they have so much momentum, there is little that can be done to damp them in an existing basin. Often, too, the driving force for a seiche cannot be removed either, since it can well be a tidal wave component or very long natural swell which drives it. However, since we are dealing with a resonant vibration with little damping, we may remember from dynamics that we need change the natural period only slightly to cause a significant reduction in the response. From equation 19.04, it is seen that the periods of the seiches in the harbors is dependent upon both the depth and the length. Either, or both, may be varied to change the response of the harbor to a given driving period. As another trick, many harbors are laid out with irregularly shaped basins, in the hope that direct reflections will be reduced.

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* This is discussed in more detail in chapter 4 of volume II.

20. TIDAL RIVERS

L.E. van Loo

20.1 Introduction

It has already been indicated in chapter 14 that harbors were often originally developed along rivers, sometimes rather well inland. London, England, Portland, Oregon, U.S.A., Antwerp, Belgium, Rotterdam, The Netherlands and Hamburg, Germany are obvious examples of such harbors. In some cases the distance to the sea has become too much of an obstacle for shipping and the prosperity of the harbor has suffered. Deventer, in The Netherlands exemplifies this.

In this chapter, we examine the effects of tides on the lower reaches of rivers and the consequences for dredging and navigation.

20.2 River Mouths

River mouths in flat coasts handle not only the fresh water run-off from their drainage basin, there is also a tidal flow through the mouth. M. P. O'Brien (1969) presented measurements made on 28 river mouths and estuaries on sandy coasts. He found that the minimum equilibrium cross sectional area of the entrance was linearly related to the volume of the tidal prism. In equation form:

$$A = 6.56 \times 10^{-5} \mathcal{P} \tag{20.01}$$

where A is the minimum equilibrium cross section area of the entrance in m^2, and

$\mathcal{P}$ is the tidal prism volume in m^3.

In this equation, $\mathcal{P}$, the tidal prism, is the storage volume of the estuary between the low tide and high tide levels.
The coefficient in equation 20.01 is *not* dimensionless. In foot -lb-sec. units:

$$A = 2 \times 10^{-5} \mathcal{P} \tag{20.01}$$

where A is in ft^2 and $\mathcal{P}$ in ft^3.

Tidal prism volumes in O'Brien's data ranged from about 1.4×10^7 m^3 (5×10^8 ft^3) to about 3×10^9 m^3 (1.1×10^{11} ft^3). There is some indication that equation 20.01 would tend to yield too great a cross section area for smaller tidal prisms.

O'Brien found, as well, that there was little influence on equation 20.01 from the bed material size. Further, the equation seemed equally valid for both large river mouths and for bays and tidal lagoons. A restriction, however, is that (20.01) is valid only for inlets having a predominantly semi-diurnal tide.

Obviously, the *form* of the mouth *is* influenced by many other factors. The discussion of these is postponed until chapter 29, however.

20.3 River channels

The geometry of natural channels and shoals in rivers is discussed in courses in river engineering. The deepest channel sections develop along the outside of the river bends with the channel shifted somewhat down stream from the shoreline bend. How is this modified by the presence of tidal action?

When there is an alternating current direction in a narrow channel the only influence of the tide on the river bathymetry is to make the location of the deeper channel in the river bends correspond more closely to that of the shoreline bend. The position represents a compromise between the development to be expected with only an ebb current and that expected with only a flood current. An example of such a channel development is shown in figure 20.1 - the Schelde River at Antwerp in Belgium.

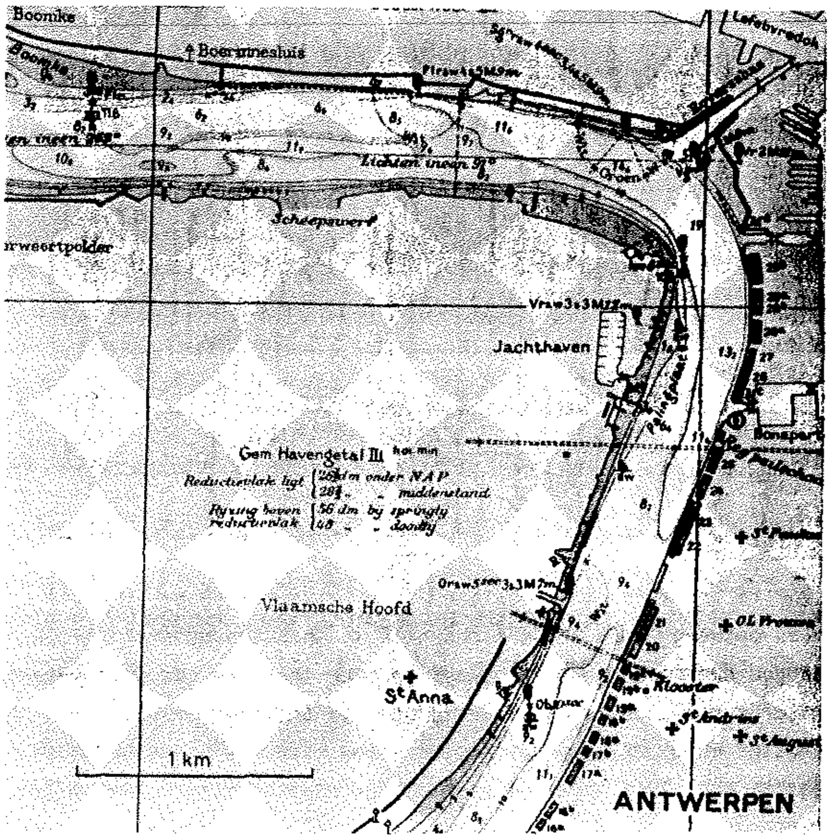

Figure 20.1
THE SCHELDE RIVER AT ANTWERP
Depth in meters scale as shown

In areas where the river width is not restricted, an entirely different pattern can develop. These tidal river reaches often have two rather independent channel systems. The ebb current concentrates in one set of channels while the flood current is often strongest in another set of channels. This pattern is most completely developed near river bends. Figure 20.2 shows this pattern clearly at another section of the Schelde River less than 50 km downstream from Antwerp (fig. 20.1).

Flood channels can usually be recognized by the following characteristic features:

a. They tend to be shallower than ebb channels.

b. They tend to "die out" - they lead to progressively shallower water and finally spread out on a shoal.

Ebb channels, on the other hand, are continuous and tend to be deeper. The reasons for these characteristic differences are explained in the following section.

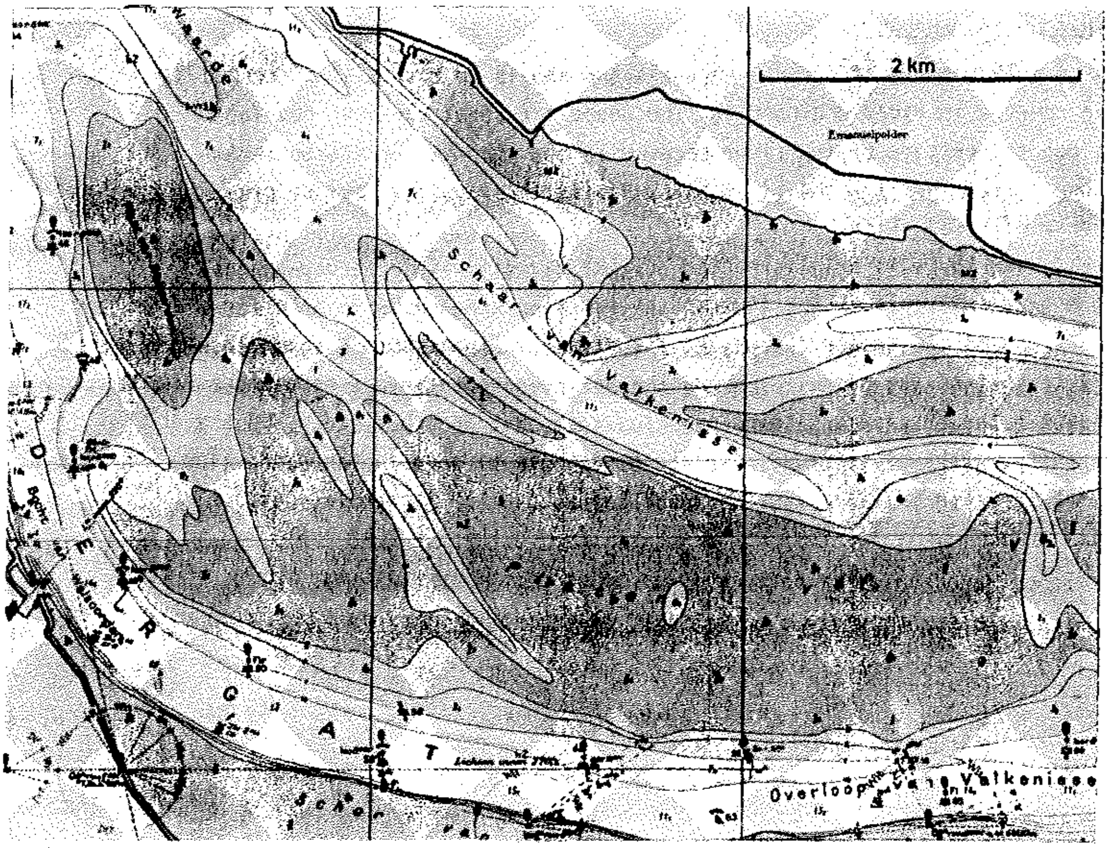

Figure 20.2
THE SCHELDE RIVER EAST OF HANSWEERT
Depth in meters scale as shown

20.4 Tidal Currents

One factor which helps to account for the characteristic differences between flood and ebb channels is that the quantity of water discharged during the ebb is always greater than the quantity entering the river during the flood. This is because the river runoff plus the ocean water which entered during the flood tide must be discharged during the ebb. This tends to make ebb currents stronger, and hence, ebb channels deeper. An example of this is shown in table 20.1 and figure 20.3 which shows the current at Rotterdam. If a similar graph were made of the current some greater distance upstream, the ebb current would become more important. As some point, even, the current would always flow downstream with a velocity which varied according to the tide. How far up a river can a tidal influence still be detected? Thereotically all the way unless there are rapids (reached with supercritical flow).

TABLE 20.1 TIDE AND CURRENT DATA ROTTERDAM.

Time (hrs)	Average Current* (m/s)	Tide level (m)
0	-0.15	-0.69
1	+0.08	-0.50
2	0.60	-0.03
3	0.75	+0.52
4	0.44	0.91
5	+0.07	1.04
6	-0.44	0.91
7	-0.73	0.61
8	-1.03	+0.25
9	-1.05	-0.15
10	-0.85	-0.47
11	-0.52	-0.58
12	-0.30	-0.62

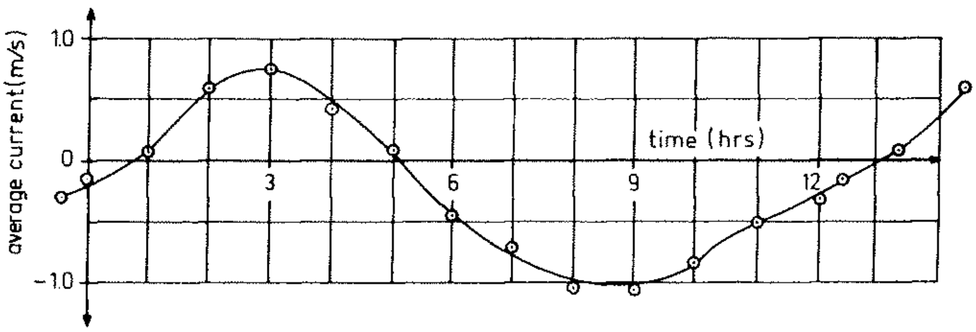

Figure 20.3 CURRENT AT ROTTERDAM

Another phenomona in a tidal river is a tide-dependent variation in water level. The current and the level are related by the equations used to describe long waves. In such a case, conservation of momentum yields:

$$V \frac{\partial V}{\partial x} + \frac{\partial V}{\partial t} = - g \frac{\partial z}{\partial x} - g \frac{V|V|}{C^2 h} \qquad (20.02)$$

in which:

C is the chezy friction coefficient,
g is the acceleration of gravity,
h is the depth,
t is time,
V is the flow velocity,
x is the coordinate measured along the river, and
z is the absolute water surface elevation.

---

* Flood currents are considered positive.

In this equation, it has been assumed that the river slope is small and the runoff is negligible. For a hypothetical case in which the last term of equation 20.02 is zero, the vertical tide (water level) and horizontal tide (current) are in phase with one another as is shown in figure 20.4.

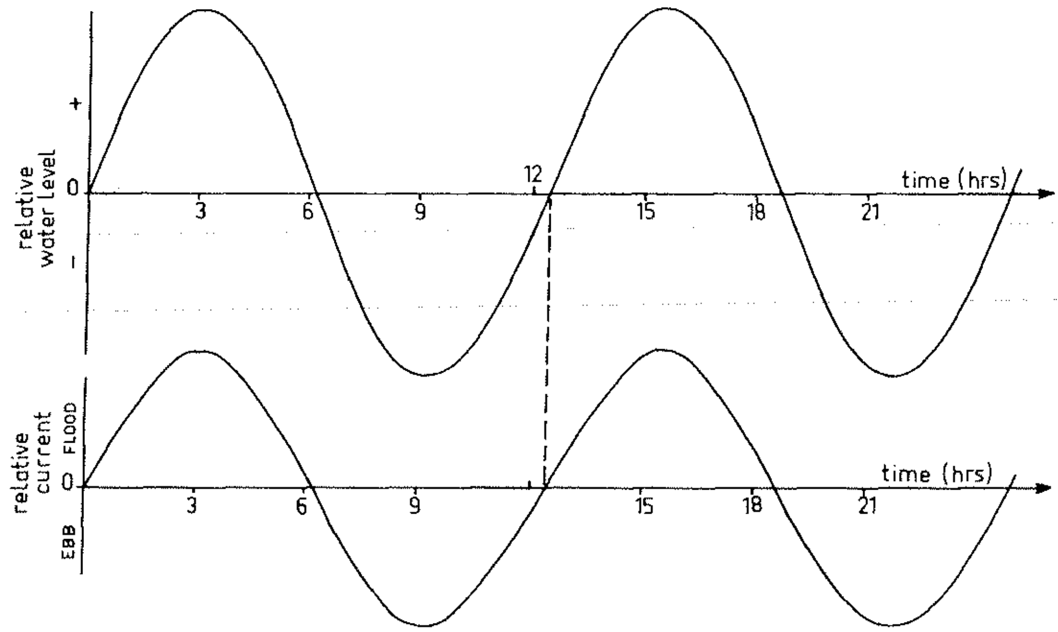

Figure 20.4 IDEALIZED VELOCITY-LEVEL RELATIONSHIP

In a real situation, the friction term in equation 20.02 will be relatively large with respect to the inertia terms. Since some of the momentum is then lost to friction, the velocity will be reduced. Figure 20.5 shows the relationship between the vertical and horizontal tides at Rotterdam. The current curve is the same as that in figure 20.3. The right hand portion of these graphs is plotted by adding one tide period ($12^h\ 25^m$) to the times listed in table 20.1. The time of high tide (high water) and low water are indicated along with the times of slack water (zero current).

Note that the low tide slack comes much later relative to low water than is the case at high tide. This is partially caused by the fresh water river flow acting to fill the most inland portion of the tidal prism during a rising tide. This enhances the development of a water surface slope to retard the tide wave, while at low water on the other hand, the river flow tends to prolong the ebb current.

A second reason why ebb channels are deeper and more continuous is indicated in figure 20.5. Note that the maximum ebb current occurs when the tide level is lower than that corresponding to the maximum flood current. The combined effect of higher total ebb flow and the lower stage during this flow tend to increase velocity and enhance erosion in ebb channels.

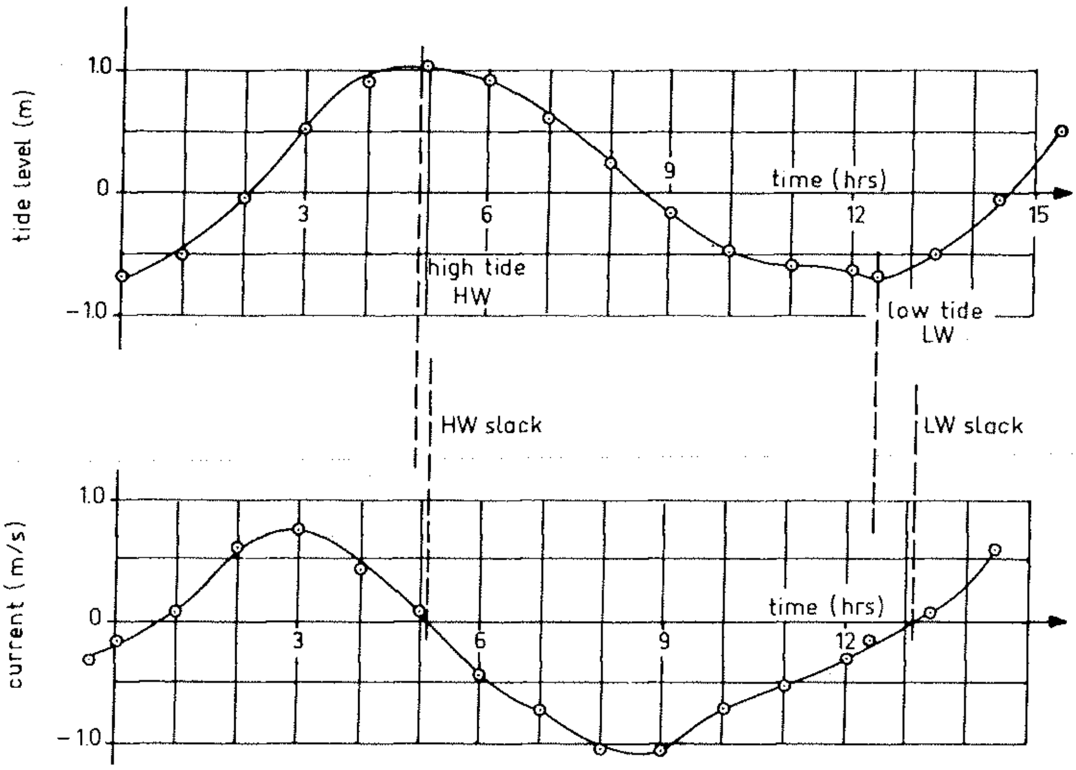

Figure 20.5 VERTICAL AND HORIZONTAL TIDE AT ROTTERDAM

This discussion has been limited until now to measurements made at a single river cross-section. A tidal wave also travels along a river. As an example of this, the high tide wave reaches Antwerp about $1^h\ 45^m$ after it passes Vlissingen. Tidal data for the Western Schelde Estuary between Vlissingen and Schelle, Belgium, are presented in table 20.2 and figure 20.6. This estuary has a very small fresh water flow, in contrast to the situation in Rotterdam. Figure 20.6 gives a clear indication of how a tide wave propagates along the estuary, This phenomona can sometimes be utilized by ships, as outlined in the following section.

Why are the maxima of the flood currents in table 20.2 greater than the ebb current maxima? This is caused, in this case by the distortion of the vertical tide curves; for a rising tide the sharp increase in water level in a short time causes higher velocities.

TABLE 20.2 Tidal Data for Western Schelde

Time (hrs)	Vlissingen level (m.)	Vlissingen current (m/s)	Hansweert level (m.)	Hansweert current (m/s)	Schelle, Bel. level (m.)
0	1.72	-0.14	2.43	+0.50	1.60
1	1.14	-0.66	1.76	-0.95	2.83
2	+0.20	-0.94	+0.80	-1.07	2.92
3	-0.80	-0.87	-0.30	-1.07	2.00
4	-1.63	-0.71	-1.29	-0.99	0.90
5	-2.07	-0.44	-2.08	-0.72	+0.03
6	-1.78	+0.04	-2.32	-0.28	-0.75
7	-1.33	0.32	-1.60	+0.60	-1.44
8	-0.81	0.45	-0.90	0.66	-2.04
9	-0.10	0.63	-0.28	0.84	-1.85
10	+1.30	1.24	+0.73	1.14	-0.65
11	2.10	0.88	2.11	1.74	-0.17
12	1.77	+0.09	2.52	+0.80	+1.10
13	1.35	-0.53	2.00	-0.70	2.60

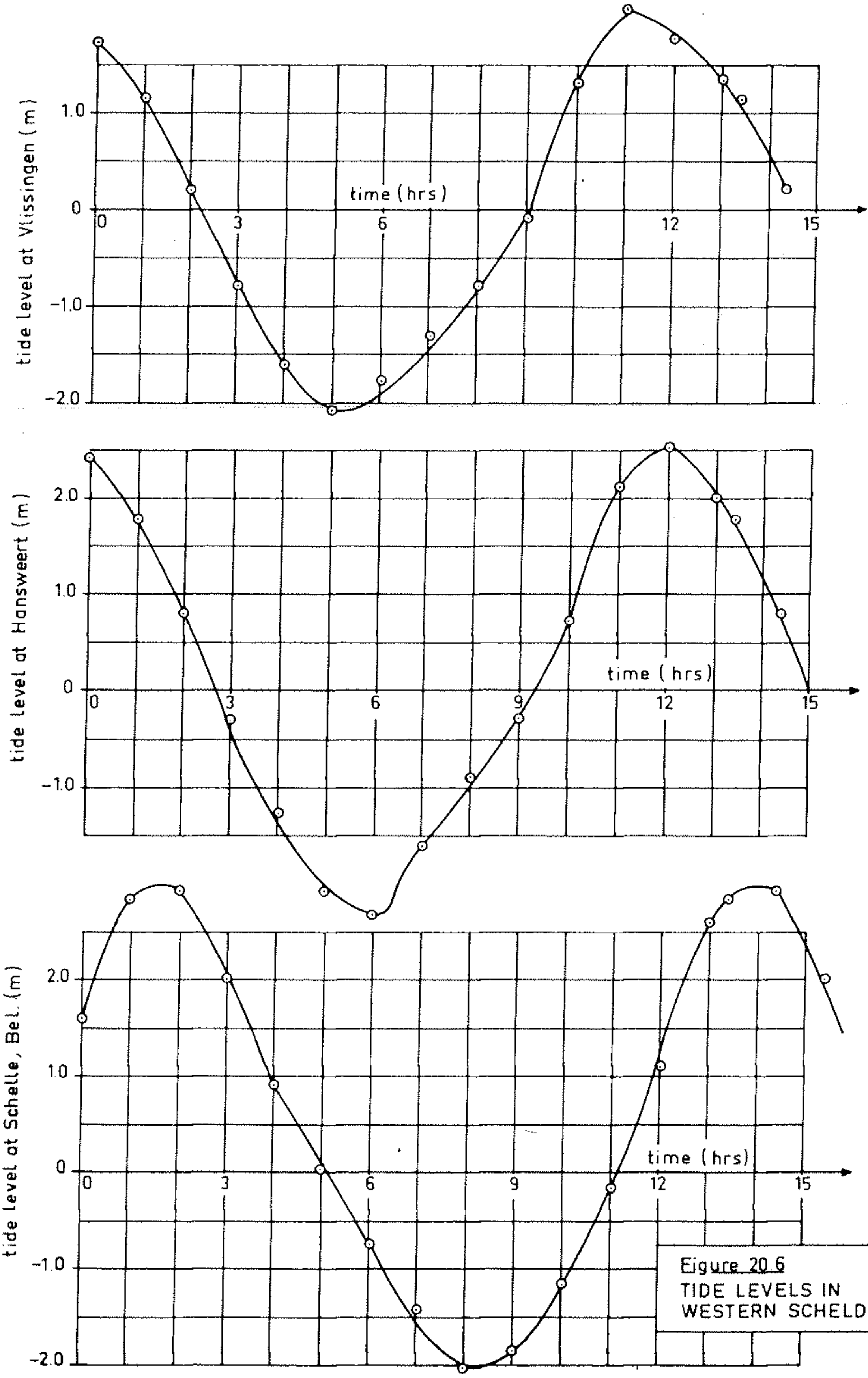

Figure 20.6
TIDE LEVELS IN WESTERN SCHELDE

20.5 River Navigation

The reasons for locating harbors well upstream along rivers have been indicated in chapter 14. The dredging problems have also been indicated. How can the tides be used to reduce the amount of necessary dredging and to help the shipping on a river? After all, it is not always economical to dredge away shoals along a river to sufficient depth so that *all* ships can navigate at *all* tide stages. It may be possible to allow larger, deeper ships to wait at the entrance for high tide and then to "ride" this high tide wave up the river. Unfortunately, the ships of concern here are not able to attain a speed as high as that of the tidal wave. Thus, even though they may enter a river at high tide, they cannot keep up with the tide wave as they progress up the river; interruption of their travel may be necessary, therefore. In order to determine the best strategy for a pilot on a ship, it is necessary to predict the water depth at each shallow channel reach at the time of arrival of the ship. To accomplish this the propagation velocity of the tide wave must be determined. Since friction tends to slow the tide wave, equation 5.05 b is not sufficiently accurate. Instead:

$$c = \sqrt{g\bar{h}\,(1 - \tan^2\theta)} \tag{20.03}$$

where:

$\bar{h}$ is the average depth
θ is a friction factor computed from:

$$\theta = \frac{1}{2}\tan^{-1}\left\{\frac{T'}{6\pi^2}\,\frac{8g}{C^2}\,\frac{V_{max}}{\bar{h}}\right\} \tag{20.04}$$

where:

C is the Chézy friction factor,
T' is the tide period,
V_{max} is the maximum flood current, and
$\tan^{-1}\{\,\}$ indicates the angle whose tanget is .... .

The procedure outlined above will be demonstrated via an idealized example.

20.6 Example

A pilot needs to bring a ship needing a minimum channel depth of 11.5 meters up a 250 km channel to a harbor. Three shoals are located along this channel as shown in figure 20.7. The depth over each of these shoals is only 11.0 m relative to the mean water level. The rest of the river is 13.0 m. deep. The Chézy friction factor for this river is 60 $m^{\frac{1}{2}}/s$.

The tide is semidiurnal (period = $12^h\,25^m$) and is assumed to be sinusoidal. The tide range is 3 meters and the maximum current is 1.2 m/s. It is assumed that this tidal form is valid for the entire river reach. The tidal information is also shown in figure 20.7.

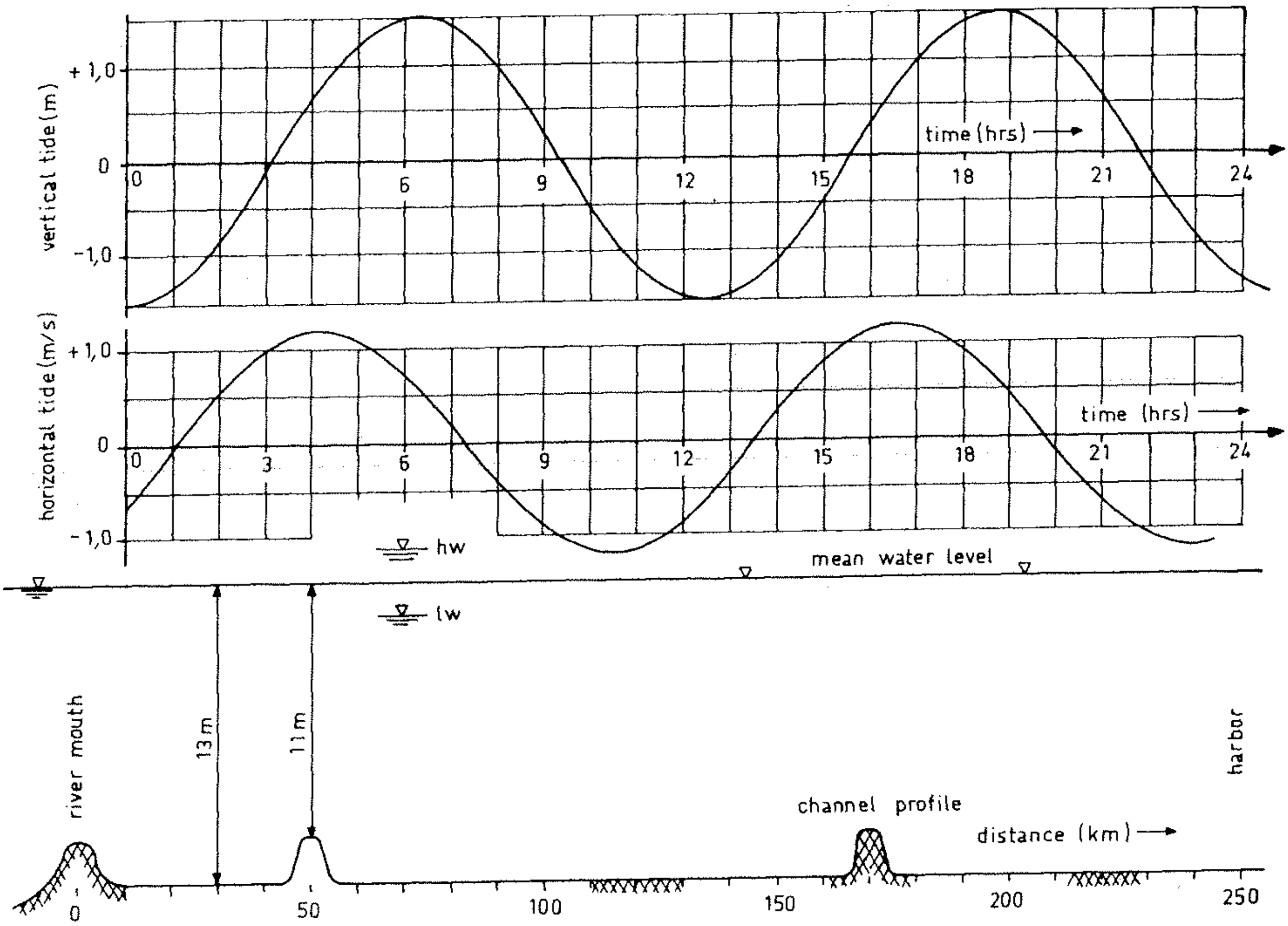

Figure 20.7 EXAMPLE PROBLEM DATA

As an additional limitation, the ship must maintain a minimum speed of 5 kt. (9.26 km/hr) relative to the water. Her maximum speed is 8 kt (14.82 km/hr).

The solution begins by determining the speed of the flood tide wave through the channel. Using 20.04,

$$\theta = \frac{1}{2} \tan^{-1} \left(\frac{(12.42)(3600)}{6\,\pi^2} \; \frac{(8)(9.81)}{60^2} \cdot \frac{1.20}{13} \right) \qquad (20.05)$$

$$= \frac{1}{2} \tan^{-1} \; (1.52) \qquad (20.06)$$

$$= 28.33^o. \qquad (20.07)$$

Thus, from (20.03):

$$c = \sqrt{(9.81)(13)(1 - \tan^2 28.33^o)} \qquad (20.08)$$

$$= 9.51 \text{ m/s} = 34.2 \text{ km/hr} \qquad (20.09)$$

The determination of the location of the ship at any time can be accomplished by integrating the velocity of the ship with respect to time. This can be most easily done by numerical integration in a tabular form, as shown in table 20.3. In this table two identical ships are considered, one moving at a minimum and one at maximum speed through the water.

In table 20.3, the ship velocity with respect to the ground is integrated with time steps of one hour. In order to determine the absolute velocity of the ship, the tidal current *at the ship's location* at the end of each hour must be known. Since the horizontal and vertical tides are known only at the entrance, the assumption is made that the tide wave propagates along the channel at speed c and is unchanged in amplitude and form. (Figure 20.6 shows that this last part of the assumption is not always true in practice). The depth and current at the ship can be obtained from the tidal curves in figure 20.7 by converting the distance between the ship and the crest of the tide wave into an equivalent time. Thus, the time listed in the left hand column of the table is an absolute time, while the "tide time" listed in the first column for each ship is determined by this distance between ship and tide wave crest. Since all tide times are related to a single high water crest, the times can have values greater than one tide period. Obviously, adding or subtracting a multiple of the tide period to these times would not affect the result of the computation.

The first line of table 20.3 is computed as follows:

The start time is arbitrarily chosen as zero. Since the water depth over the first shoal must be 11.5 m, the corresponding tide level must be +0.5. From figure 20.7 this tide corresponds to a tide time of 3.8 hours. At this time, the current is +1.2 m/s, from fig. 20.7. The time interval between the tide time and High Water is 6.21 - 3.8 = 2.41 hrs. with a tide wave speed of 34.2 km/hr, The tide wave crest is located at - 82.4 km at the time the ship crosses the first shoal.

In each succeeding hour, the tide wave progresses 34.2 km.

A typical line, for the time interval 16 -17 hrs goes as follows: At t = 16 hrs, the ship is at 151.0 km and the tide crest is at 464.8. The tide phase in figure 20.7 is then:

$$\frac{464.8 - 151.0}{34.2} + 6.21 = 15.4 \text{ hrs.} \tag{20.10}$$

Entering figure 20.7 with a time of 15.4 hrs yields a tide level of - 0.1 m and a current of + 1.0 m/s. The tide level gives a depth of 12.9 m. The incremental distance for the ship in one hour is:

$$(1.0 \text{ m/s})(3.6) + 9.3 = 12.9 \text{ km.} \tag{20.11}$$

resulting in a distance after 17 hours of 163.9 km.

The results of table 20.3 can be visualized more easily in a graph of position versus time. In figure 20.8, the positions of the ships and of the tide wave crest are shown, along with the positions of the three shoals. The time intervals during which the shoals can be crossed are also shown.

TABLE 20.3 TABULATED INTEGRATION COMPUTATION

Time (hrs)	Pos. of H.W. (km)	SHIP MOVES 9.3 km/hr. Tide Time (hrs)	Tide Level (m)	Depth at ship (m)	Current at ship (m/s)	Ship Δx (km)	Ship x (km)	SHIP MOVES 14.8 km/hr Tide Time (hrs)	Tide Level (m)	Depth at ship (m)	Current at ship (m/s)	Ship Δx (km)	Ship x (km)	Comments
0	-82.4	3.8	+0.5	11.5	+1.2		0.0	3.8	+0.5	11.5	+1.2		0.0	Both ships start when depth is 11.5 m with rising tide.
						13.6						19.1		
1	-48.2	4.4	+0.9	13.9	+1.2		13.6	4.2	+0.8	13.8	+1.2		19.1	
						13.6						19.1		
2	-14.0	5.0	+1.2	14.2	+1.1		27.2	4.7	+1.1	14.1	+1.2		38.2	
						13.3						19.1		Fast ship passes second shoal.
3	20.2	5.6	+1.4	14.4	+0.9		40.5	5.1	+1.3	12.3	+1.1		57.4	
						12.5						18.8		
4	54.4	6.2	+1.5	12.5	+0.7		53.0	5.6	+1.4	14.4	+0.9		76.1	Slow ship passes second shoal.
						11.8						18.0		
5	88.6	6.9	+1.4	14.4	+0.3		64.9	6.0	+1.5	14.5	+0.8		94.2	
						10.4						17.7		
6	122.8	7.6	+1.2	14.2	-0.2		75.2	6.5	+1.5	14.5	+0.5		111.8	
						8.6						16.6		
7	157.0	8.3	+0.8	13.8	-0.6		83.8	7.0	+1.4	14.4	+0.2		128.4	
						7.1						15.5		
8	191.2	9.1	+0.2	13.2	-0.9		91.0	7.6	+1.2	14.2	-0.2		144.0	
						6.1						14.1		
9	225.4	10.0	-0.5	12.5	-1.1		97.0	8.2	+0.8	13.8	-0.5		158.0	
						5.3						13.0		
10	259.6	10.8	-1.0	12.0	-1.2		102.4	8.8	+0.4	11.4			171.0	Fast ship arrives just too late to cross third shoal. Must wait until next tide!
						5.0								
11	293.8	11.7	-1.4	11.6	-1.0		107.3							
						5.7								
12	328.0	12.5	-1.5	11.5	-0.6		113.0							
						7.1								
13	362.2	13.3	-1.4	11.6	-0.2		120.2							
						8.6								
14	396.4	14.0	-1.1	11.9	+0.3		128.8							
						10.4								
15	430.6	14.7	-0.6	12.4	+0.7		139.1							
						11.8								
16	464.8	15.4	-0.1	12.9	+1.0		151.0							
						12.9								
17	499.0	16.0	+0.3	11.3	+1.1		163.9	Slow ship arrives at third shoal just too early to cross						

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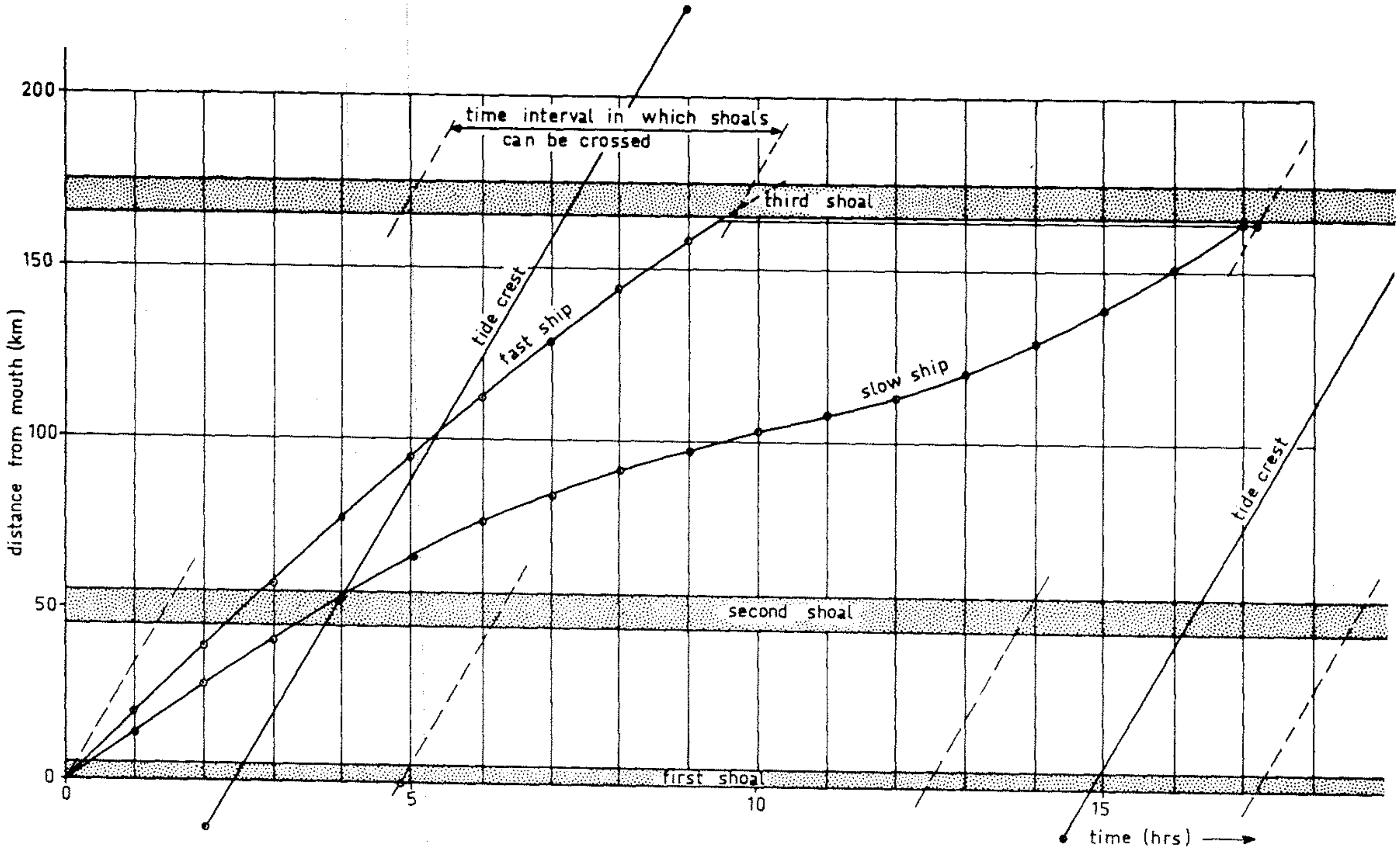

Figure 20.8 DISTANCE – TIME CURVES FOR TIDE AND SHIPS

Several interesting conclusions can be drawn from figure 20.8:

a. Both ships must wait for the second tide to cross the third shoal. The extra speed of the fast ship makes no difference to the time needed to navigate the first 170 km of the estuary.

b. The slow ship could avoid stopping at all by delaying her departure a short time. This would (approximately) move the curve for this ship a bit to the right in figure 20.8. The second shoal would still be cleared on the first tide and the ship would arrive late enough at the third shoal to navigate it easily as well.

c. Dredging away the third shoal would be nearly as effective as dredging out both the first and second shoals for improving the navigation.

d. Dredging only the outer bar would allow the fast ship to cross both the second and third shoals on the first tide.

e. Dredging of the second bar would not change the pilot strategy for either ship.

The choice of which shoals and bars to dredge and how deep to make the channel through them is a problem lending itself well to economic optimilization techniques as were outlined in chapter 13. The problem of determining the optimum depth for a channel is taken up in detail in chapter 5 of volume II.

20.7 Other Tidal Effects

When the fresh water of a river meets salt sea water, the density differences caused by variations in salinity cause additional currents. Also, the salinity variations can affect the physical chemistry of fine sediments. All of these phenomona related to salinity are discussed, together, in chapters 22 and 23.

21. RIVER TIDE MEASUREMENTS

E.W. Bijker

21.1 Introduction

One of the most important bits of data needed for surveying work in a tidal river is a datum elevation. Measurements of tides, channel depths and terrain topography can all be related to this datum.

Such a datum is easily determined from a coastal tidal record from which a mean sea level datum can be determined. Of course, this datum can be transferred inland along a river by conventional differential leveling techniques, but such work can be tedious, especially in rather inaccessable tropical areas where, it seems, an accurate datum is most often lacking.

An alternative methode for transferring a datum level inland along a river is described in the next sections of this chapter. It uses the river, itself, as a level.

An important assumption in this whole chapter is that the fresh water flow in the river may be neglected.

21.2 Precise Problem Statement

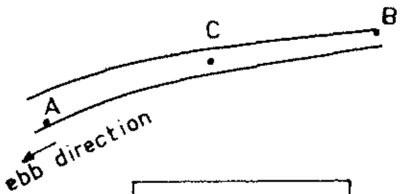

Figure 21.1
RIVER PLAN

It is a simple matter to determine a mean sea level (M.S.L.) datum at the river mouth (coast) from a tide record. We can also easily measure water level changes using a tide gauge some distance upstream from the river mouth (A in figure 21.1). Our problem is one of determining the datum level for this second tide gauge located at B in that figure. The time scales of these two records agree.

This problem really reduces to determining the time at which the river reach A-B has no surface slope. Then, the absolute levels of the two tide records would be identical.

The horizontal tide at C, midway between A and B is also needed for this problem solution. All of this data is shown in table 21.1 and figure 21.2 for a hypothetical reach of river.

21.3 A Simple Method of Solution

The tidal motion in the river reach is governed by:

$$V \frac{\partial V}{\partial x} + \frac{\partial V}{\partial t} = - g \frac{\partial z}{\partial x} - \frac{g V|V|}{C^2 h} \tag{21.01}$$

where:

C is the chézy friction coefficient,
g is the acceleration of gravity,
h is the depth,
t is time,
V is the flow velocity,
x is the coordinate measured along the river, and
z is the absolute water surface elevation.

This equation is the same as equation 20.02 in the previous chapter.

TABLE 21.1 Tide and Current Data

Time (hrs)	Tide level at A (m)	Relative Tide level at B (m)	Current at C (m/s)
0	+0.15	+0.99	-0.10
1	-0.35	+0.69	-0.35
2	-0.75	+0.22	-0.52
3	-1.00	-0.06	-0.60
4	-1.00	-0.21	-0.57
5	-0.75	-0.13	-0.45
6	-0.20	+0.15	-0.24
7	+0.30	+0.49	+0.10
8	+0.72	+0.91	+0.38
9	+0.95	+1.14	+0.50
10	+0.98	+1.32	+0.50
11	+0.80	+1.31	+0.37
12	+0.37	+1.11	+0.07
13	-0.18	+0.77	-0.25
14	-0.68	+0.39	-0.44

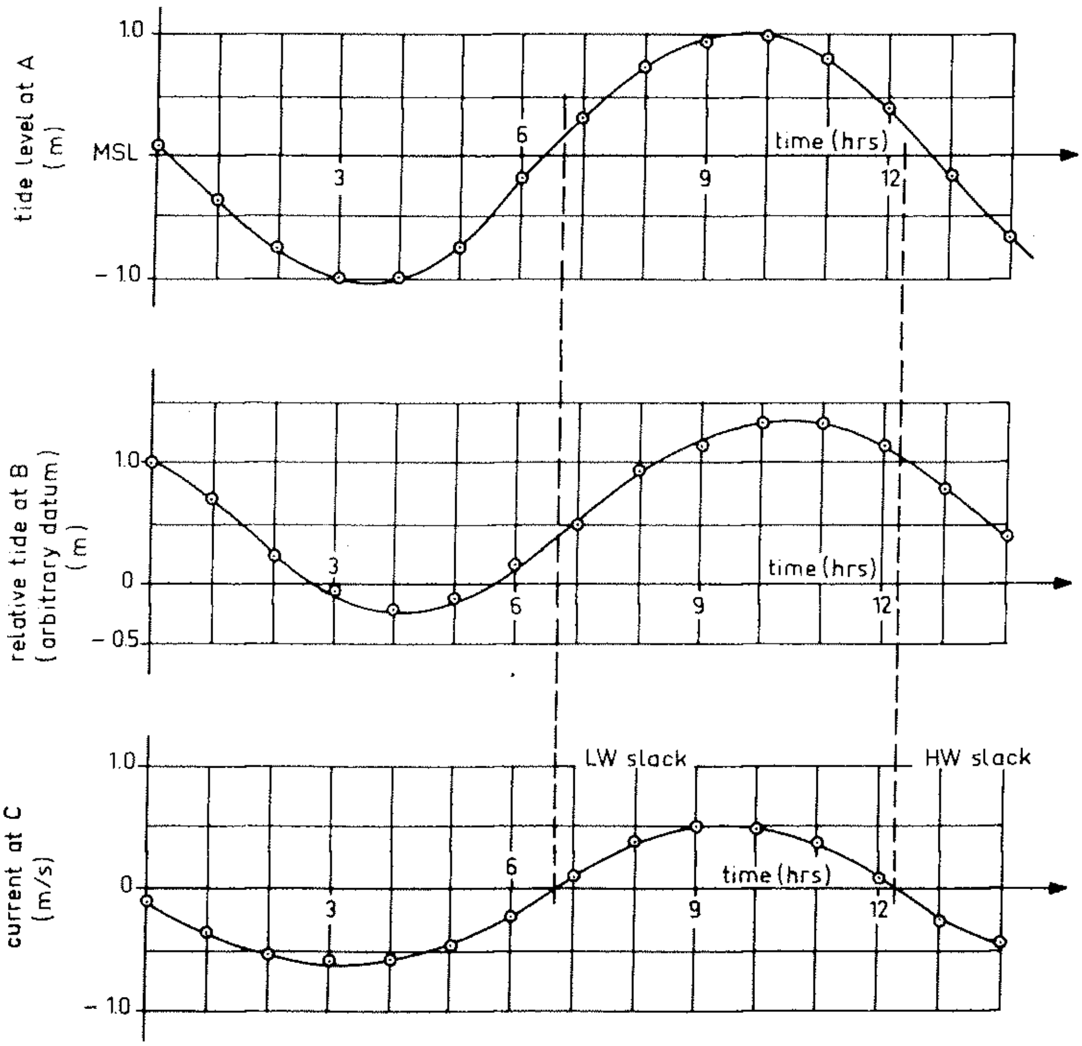

Figure 21.2 TIDE AND CURRENT DATA

If the inertia terms on the left in equation 21.01 could be neglected, then the water surface slope would be zero at the time of slack water.

Unfortunately, this is too simple. Since inertia is important, the water will continue flowing until an adverse surface slope has been generated. This means that the the water surface slope will be zero some time, Δt, before slack water. Making a further essential assumption that the inertia influence at high tide is the same as at low tide leads to a conclusion that:

$$\Delta t_f = \Delta t_e \tag{21.02}$$

In other words, at some time, Δt_f before the flood slack *and* Δt_e before the ebb slack the absolute water levels of the two tide curves must be the same (They must cross when superimposed).

The assumption stated by equation 21.02 is valid if there is no fresh water flow in the river. A relatively large runoff flow can upset this assumption very much. Probably, in such a case, the improved method outlined in the following section will yield somewhat better results. Even so, such a flow can disturb the results appreciably.

The problem is solved graphically by moving the curve of the tide at B vertically over the tide curve at A. Moving curve B vertically over curve A will increase one value of Δt while decreasing the other. When the position is found yielding equal values of Δt, the water levels at A and B are equal and the arbitrary vertical scale at B can be related to the scale at A. The two tide curves, in their proper relative positions, are shown in figure 21.3.

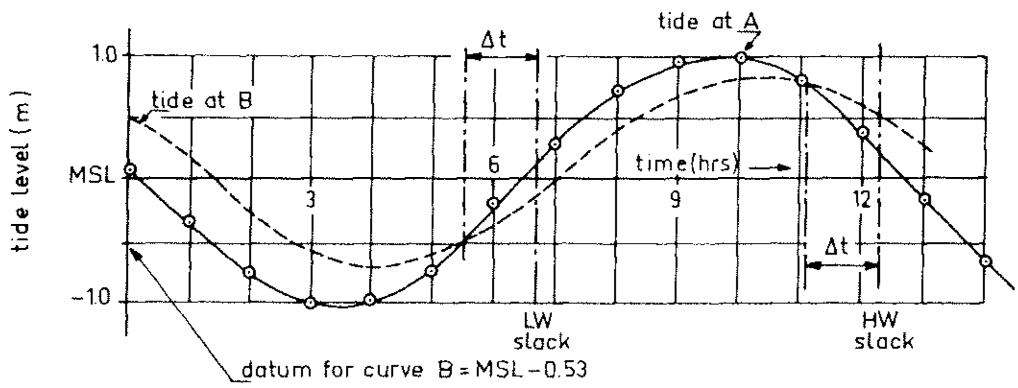

Figure 21.3 ADJUSTED TIDE CURVES

In this figure the tide curve at A is shown with the solid line. The superimposed tide curve for point B is drawn as a dashed line and times of slack water are the same as in figure 21.2. The actual values of Δt found are not important. What is important is that the zero of the arbitrary scale used for the tide record at B corresponds to - 0.53 m with respect to M.S.L. Thus, the tide record at B can be related to M.S.L. by subtracting 0.53 m from the given values tabulated in table 21.1.

21.4 A Better Solution

An essential assumption in the solution just presented was that the two time intervals Δt_f and Δt_e were equal (eqn. 21.02). This assumption is often invalid, especially when the current curve at point C in figure 21.1 is asymetrical.

The theory is based, again, upon equation 21.01 which is repeated here for convenience:

$$V \frac{\partial V}{\partial x} + \frac{\partial V}{\partial t} = - g \frac{\partial z}{\partial x} - \frac{g\, V|V|}{C^2 h} \qquad (21.01)$$

For tidal problems:

$$V \frac{\partial V}{\partial x} \ll \frac{\partial V}{\partial t} \qquad (21.03)$$

This is especially true when the velocity is low near slack water. Also, at the time of interest:

$$\frac{\partial z}{\partial x} = 0 \qquad (21.04)$$

Thus, when we substitute 21.04 and neglect the smaller acceleration in equation 21.01, it becomes:

$$\frac{\partial V}{\partial t} = - \frac{g\, V|V|}{C^2 h} \qquad (21.05)$$

The partial derivative can now be replaced by a total derivative in 21.05:

$$\frac{dV}{dt} = - \frac{g\, V|V|}{C^2 h} \qquad (21.06)$$

The variables can be separated:

$$\frac{dV}{V|V|} = - \frac{g\, dt}{C^2 h} \qquad (21.07)$$

Integration yields:

$$V\, t = \frac{C^2 h}{g} \qquad (21.08)$$

Equation 21.08 gives the relationship between V and t near the time of slack water and when $\frac{\partial z}{\partial x} = 0$. Obviously, it gives a relationship for $\frac{dv}{dt}$ as well.

Since the water depth, h, at both points A and B can be measured at the time of local slack water, we need only to estimate the Chézy friction factor in order to work with equation 21.08. After this is estimated a graph of $V\,t$ = constant = $\frac{C^2 h}{g}$ for a high tide slack at B and a low tide slack at A can be constructed. These curves can then be placed on the measured velocity curve at C such that the origins of the velocity axes are the same and the slopes of the two curves are equal where they are tangent to each other (osculate). The V t curve from equation 21.08 is moved horizontally along the time axis in order to accomplish this osculation. The time on the horizontal scale of the current curve corresponding to the point of osculation is the time at which $\frac{\partial z}{\partial x} = 0$ and hence, the tide curves for A and B must cross.

This procedure is demonstrated in the following hypothetical example.

21.5 Example

This improved method will be applied to the same type of river as shown in figure 21.1. This time, however, the tidal data is given in table 21.2 and figure 21.4. Further the Chézy coefficient is 60 $m^{\frac{1}{2}}/s$.

TABLE 21.2 TIDE AND CURRENT DATA

Time (hrs)	Time level at A (m)	Relative Tide level at B (m)	Current at C (m/s)
0	+0.45	0.75	+0.21
1	0.00	+0.33	+0.07
2	-0.33	-0.04	-0.08
3	-0.67	-0.35	-0.21
4	-0.90	-0.55	-0.37
5	-0.98	-0.62	-0.50
6	-0.79	-0.47	-0.60
7	-0.30	-0.15	-0.62
8	+0.40	+0.35	-0.20
9	+0.83	+0.75	+0.40
10	+1.00	+0.98	+0.50
11	+0.87	+1.05	+0.42
12	+0.60	+0.87	+0.32
13	+0.25	+0.55	+0.14
14	-0.15	+0.15	-0.05

The depth, h, at B during the high tide slack is 8 m and the depth at A during low tide slack is 7 m. Thus, for point A:

$$\frac{C^2h}{g} = \frac{(60)^2(7)}{9.81} = 2568 \text{ m} \tag{21.09}$$

and for B:

$$\frac{C^2h}{g} = \frac{(60)^2(8)}{9.81} = 2935 \text{ m} \tag{21.10}$$

Curves of equation 21.08 using the constants evaluated in the two above equations are shown in figure 21.5.

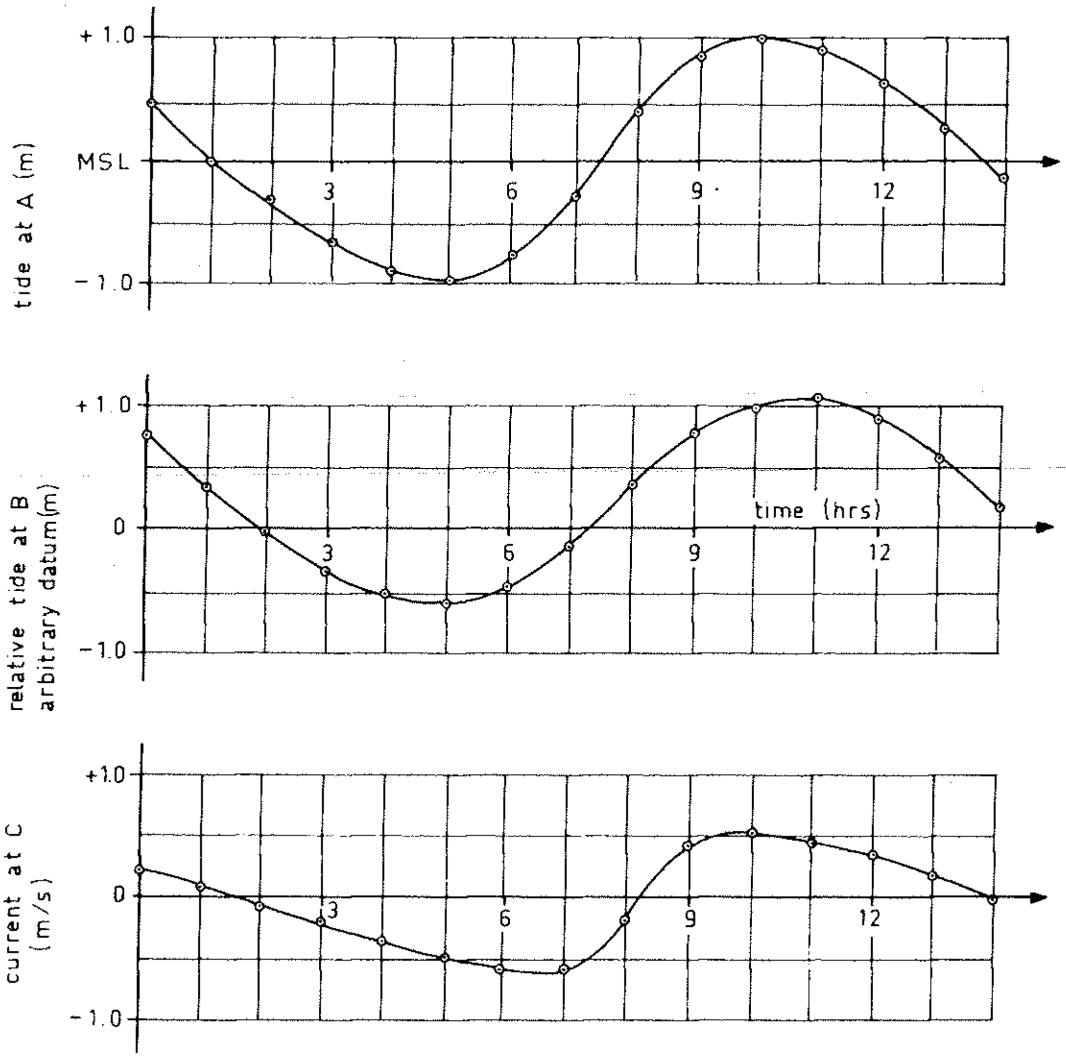

Figure 21.4 TIDE AND CURRENT DATA

Each of the curves in figure 21.5 is then placed individually on the current curve in figure 21.4. The V-t curve is moved *horizontally* along the current curve time axis until the two curves are just tangent to each other at one point (osculate). This point of osculation is projected upward to the tide curve at A. The same procedure with the second V-t curve yields a second point on the tide curve at A.

If all of the theory and assumptions were excatly correct, then the tide curve at B could be superimposed on the curve at A such that the two curves intersected at the two points just determined. Generally, this will not be possible; we make a final adjustment by moving the tide curve at B *vertically* such that the time intervals between the actual and theoretical crossing points are equal.

Figure 21.6 shows the current curve with the superimposed V-t curves. The osculating points are projected on to the properly superimposed tide curves. The two equal time intervals are also shown. The datum for curve B turns out to be 0.21 m below M.S.L.

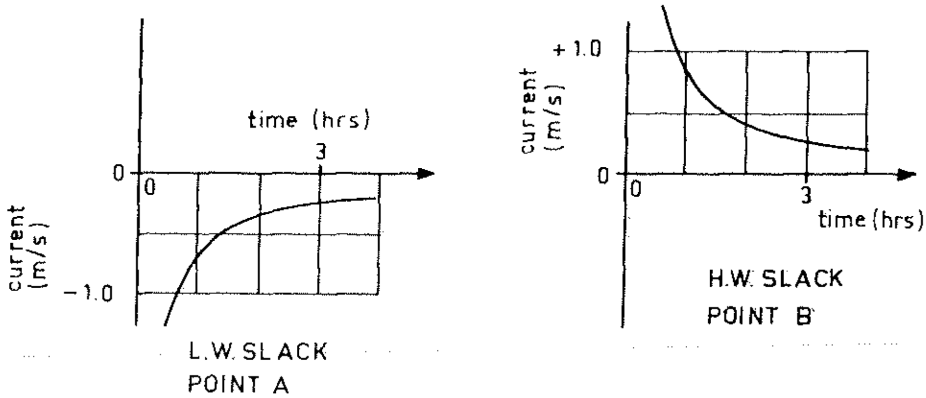

Figure 21.5 VELOCITY TIME CURVES

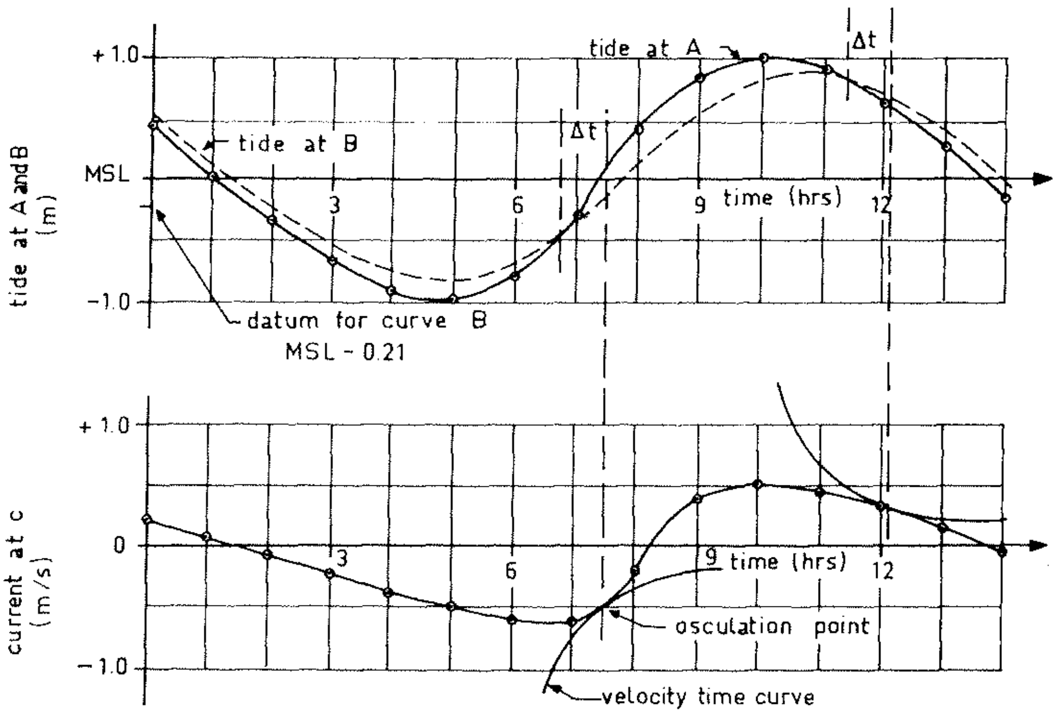

Figure 21.6 GRAPHICAL SOLUTION

21.6 A Reexamination

It must be remembered that even at its best, the methods outlined in this chapter give only approximate results. Even so, these results are usually of sufficient quality for at least preliminary surveys.

The distance along the river between points A and B in figure 21.1 can vary between a few kilometers and more than 100 kilometers. It is important, however, that the current measurement station, C, be located midway between A and B.

As the depth becomes shallower, the importance of the friction term in equation 21.01 increases relative to that of the inertia terms. When the river is shallow enough, in fact, the inertia influences can be neglected and the water surface will be horizontal exactly at the moment of slack water. Thus, the vertical and horizontal tide curves will be $\frac{1}{4}$ period out of phase.

Effects of density differences in estuaries are discussed next in the following chapter.

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22. DENSITY CURRENTS IN RIVERS

22.1 Introduction

The previous two chapters have considered tidal influences on rivers without regard to the fact that the river water is relatively pure while the ocean water is essentially salty. The effects of these salinity differences between ocean and river water form the major subject of this chapter. Chapter 23 will consider the additional influences on harbors located along rivers.

The approach used in these chapters will be purely practical. Derivations of many of the equations used can be found in the literature or other courses on theory of density currents.

22.2 Salinity Variations with Tide

Salt water enters an estuary during a rising tide unless there is more than enough fresh water flow in the river to completely fill the entire tidal prism during the rising tide phase. Few rivers have sufficient flow over the entire year to prevent the intrusion of salt water at least occasionally. Indeed, the opposite is more often true, there is seldom sufficient flow to prevent the intrusion of salt water.

The salinity at some point in a river can be expected to vary according to the tide. Also, since the salt water comes from the sea, the maximum salinity should be expected at about the time of the high tide slack. This is illustrated in table 22.1 and figure 22.1 for Rotterdam. The current data is the same as that listed in table 20.1. Again, flood currents are considered positive.

TABLE 22.1 Tidal Data for Rotterdam

Time (hrs)	Current (m/s)	River Salinity (^{o}/oo)
0	-0.15	2.38
1	+0.08	2.47
2	+0.60	2.83
3	+0.75	3.64
4	+0.44	5.08
5	+0.07	7.25
6	-0.44	8.06
7	-0.73	7.16
8	-1.03	6.08
9	-1.05	4.90
10	-0.85	3.64
11	-0.52	2.65
12	-0.30	2.38

Note: The symbol ^{o}/oo denotes parts per thousand.

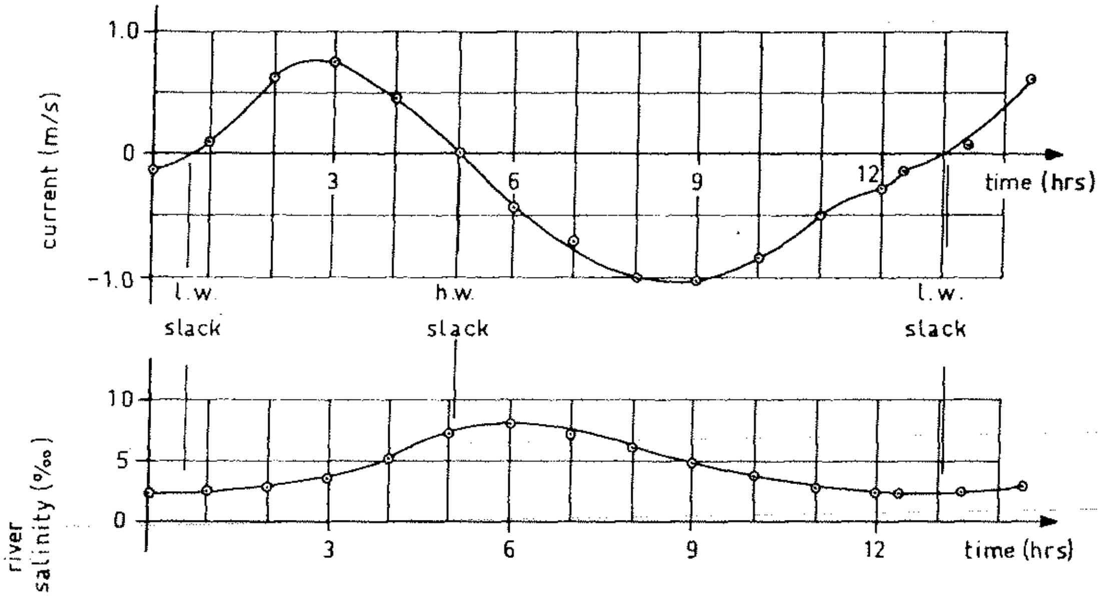


Figure 22.1 CURRENT AND SALINITY AT ROTTERDAM

In this example, the salinity maximum is reached shortly after the high water slack. The explanation for this is given in section 6 of this chapter.

Recalling from chapter 3 that sea water has a salinity of about 35 $^{0}/oo$, we see that pure sea water never really reaches Rotterdam. Mixing has already dispersed the incoming sea water through the fresh river water forming a brackish mixture. If we were to measure salinities at a point nearer to the sea, then we could expect to find higher maximum salinity values. Further inland, the maximum salinity becomes still lower.

The degree of mixing in an estuary can be approximately related to the ratio between the volume of the tidal prism and the river flow. In table 22.2, the mixing parameter, M, is:

$$M = \frac{Q_r \, T'}{P} \qquad (22.01)$$

where: M is the mixing parameter,

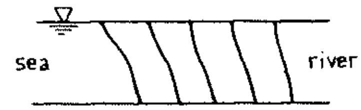
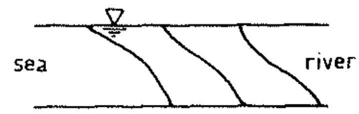
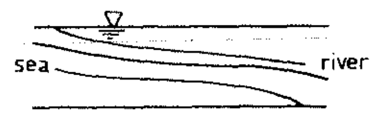
P is the volume of the tidal prism,

Q_r is the fresh water river flow, and

T' is the tide period.

In each of the sketches of river profiles, the sea is assumed to be to the left, haloclines (lines of constant salinity) are shown.

TABLE 22.2 River Mixing Criteria

M	description	sketch
0		
	Mixed Estuary	
0.1		
	Partially Mixed Estuary	
1.0		
	Stratified Estuary	

Note: Sketches are not to scale.

In each sketch, salinity increases toward the left.

A more fundamental approach to the problem used by Ippen and Harleman (1961) investigates the mixing process through use of a dimensionless stratification number. This is defined as:

$$\frac{\text{rate of energy dissipation}}{\text{rate of potential energy gain}}$$

where this is done for a unit mass of fluid. The energy dissipation in the numerator results from the damping of the tidal wave in the estuary; the denomonator reflects the potential energy gain as water increases in density (salinity) moving downstream.

Harleman and Abraham (1966) related the stratification number uniquely to a dimensionless estuary number, defined by

$$E = \frac{P\,F^2}{Q_r\,T'} = \frac{F^2}{M} \qquad (22.02)$$

where:

F is the Froude number based upon the maximum flood current velocity at the estuary mouth.

The estuary number has the advantage over the stratification number that its parameters can be rather easily evaluated. In contrast to the mixing parameter, estuary mixing increases with increasing estuary number values. Well mixed estuaries have estuary numbers greater than about 0.15.

Another independent problem related to estuaries is the determination of the salinity distribution within the estuary. Harleman and Abraham (1966) attempted this determination using a one-dimensional theoretical model for the salinity distribution in an estuary. In their model; the x axis extends positively upstream from the estuary entrance along the channel axis. In keeping with a one-dimensional model, the salinity is assumed to vary only as a function of time and position along this channel axis. Further, they assume that the salinity distribution is determined by an equilibrium between inward diffusion and outward convection with the fresh water flow.

Since the extreme situations of salinity distribution (maximum and minimum intrusion) occur at moments of slack water (high water slack and low water slack, respectively, we can examine these situations using a simplified ordinary differential equation:

$$v_r \frac{d\,S_s}{dx} = \frac{1}{A}\frac{d}{dx}\left(A\,D\,\frac{d\,S_s}{dx}\right) \qquad (22.03)$$

where:

A is the cross sectional area of the channel at point x,
S_s is the salinity at the moment of slack water,
v_r is the fresh water flow velocity,*
x is the coordinate along the channel, and
D is the apparent dispersion coefficient which included all mixing effects.

When we further assume that the cross sectional area of the estuary is independent of x, A falls out of equation 22.03. Integration with respect to x and substitution of the boundary condition

$$S_s\Big|_{x=\infty} = \frac{d\,S_s}{dx}\Big|_{x=\infty} = 0 \qquad (22.04)$$

yields:

$$V_r\,S_s = D\,\frac{d\,S_s}{dx} \qquad (22.05)$$

To integrate this further, an equation for D as a function of x is needed. The following function was assumed:

$$D = \frac{D_o\,B}{x + B} \qquad (22.06)$$

where:

B is the distance outside the estuary at which the salinity reaches that of the ocean, and
D_o is the diffusion coefficient at x = 0.

At x = -B, D = ∞, which is not inconsistent; infinite mixing would be required to maintain a constant salinity.

Substituting 22.06 into 22.05 and integrating yields:

$$\ln S_s + \text{constant} = \frac{V_r(x + B)^2}{2\,D_o\,B} \qquad (22.07)$$

---

* v_r is negative in agreement with the sign convention for x.

The constant is evaluated using the definition of B:

$$S_s\Big|_{x=-B} = S_0 = \text{ocean salinity} \tag{22.08}$$

Thus, 22.07 becomes:

$$\frac{S_s}{S_0} = \exp\left[\frac{V_r}{2\,D_0\,B}\,(x+B)^2\right] \tag{22.09}$$

S_s decreases with increasing x, since V_r is negative.

For a given estuary, the two unknows in 22.09, D_0 and B, can be evaluated if values of S_s are known - from measurements - at two different locations. It is sometimes even possible to reduce 22.09 further by including the dependence of D_0 and B on V_r. This has been done, for example, for the Chao Phya Estuary in Thailand. The resulting equation for the low water slack is:

$$S_s = S_0 \exp\left[-\,(18)(10^{-6})Q_r\,x^2 - 0.045\,Q_r^{1/2}\right] \tag{22.10}$$

Equation 22.10 is *not* dimensionless. Q_r is in units of m^3/s and is positive, x is in km.

22.3 Density - Salinity Relationship

Salinity variations cause variations in water density, just as do temperature variations. The relationship between water density and temperature and salinity is given in chapter 3. The influence of salinity on density is greater than that of temperature, at least over the range of values normally encountered.

Density differences within bodies of water will be the independent variable for the rest of our discussion. These differences may result from either temperature or salinity variations. While the cause of the density differences can be of importance from a thermodynamic or pollution point of view, it is unimportant for the mechanics of the flow. Therefore, little further consideration will be given to the cause of the density variations, except in certain specific instances.

22.4 Statics of Stratified Water Masses

Two limiting cases of static equilibrium between bodies of water having different densities can be considered, depending upon the orientation of the separating surface.

The simplest case has a horizontal interface between the two layers. If the upper layer is less dense than the lower layer this stratification will be in equilibrium. In fact, such an interface can remain stable even though both layers of water are in motion. This stratification, caused either by salinity or temperature differences is found in the oceans and in all but the shallowest lakes.

The second case has a vertical interface, and is unstable. Such interfaces do exist, however; across the door of a lock, for example. Figure 22.2 shows the pressure distribution on such a door. If the resultant horizontal force on the door is zero, then:

$$\frac{1}{2}\rho_1 g h_1^2 = \frac{1}{2}\rho_2 g h_2^2 \qquad (22.11)$$

where:

g is the acceleration of gravity,
h is the depth,
ρ is the mass density of water, and
the subscripts, 1, 2, refer to the two water masses.

When $\rho_2 > \rho_1$, then 22.11 yields:

$$\frac{h_1}{h_2} = \sqrt{\frac{\rho_2}{\rho_1}} \qquad (22.12)$$

while the resultant horizontal force is zero, figure 22.2 shows clearly that the resultant moment on the door is not zero.

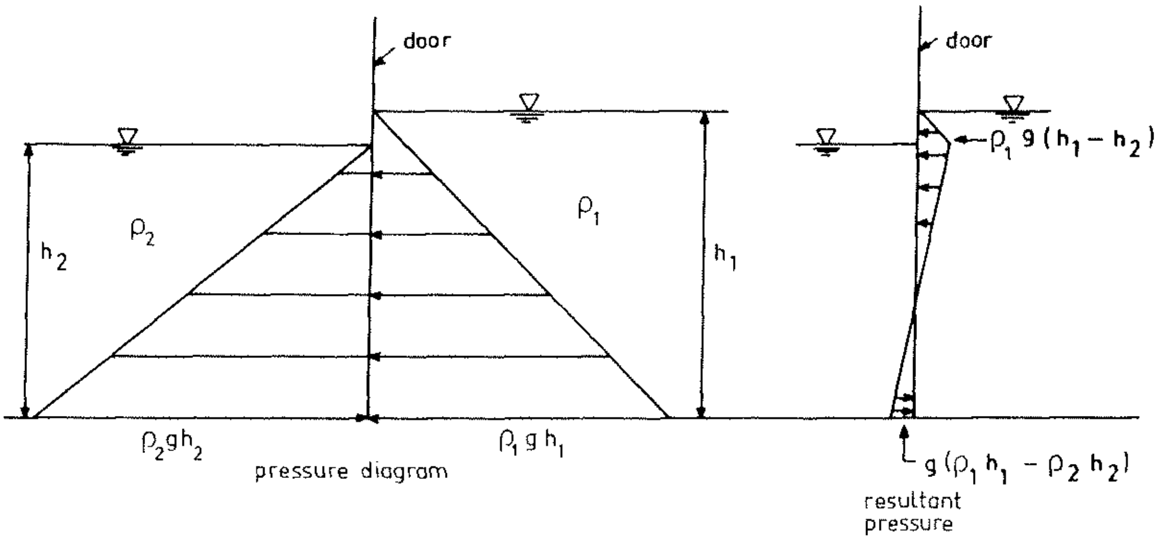


Figure 22.2 PRESSURES ON VERTICAL INTERFACE

This form of density stratification also represents an idealization of a phenomona in nature when the salinity in a river at the mouth of a harbor or tributary suddenly changes at some time during a tide cycle. This phenomona, its theory, and consequences will be the subject of chapter 23.

22.5 Internal Waves

When a horizontal stratification surface exists within a body of water (case one of the previous section), then waves can be generated at this interface, just as on the upper surface. Indeed, the upper surface of a body of water is also on interface between two fluids-water and air. However, for internal waves on an interface between water layers, the density of the upper fluid is nearly the same as the density of the lower fluid. The resulting low density difference will have a strong influence on the phenomona involved, especially when these are compared to wind waves.

Internal waves can be caused by a disturbance such as a ship, earthquake or underwater landslide. They can also result from shear forces along an interface between two layers in relative motion.
The celerity of a wave on an interface is given by:

$$c = \sqrt{\frac{(\rho_2 - \rho_1) g\, \theta_1 \theta_2}{\rho_2 \theta_1 + \rho_1 \theta_2}} \tag{22.13}$$

where:

c is the wave speed,
θ is the layer thickness, and
subscripts 1, 2, refer to the two layers.

See figure 22.3, in which the arrows show the direction of water movement. Equation 22.04 reduces to equation 5.05b when $\rho_1 = 0$.

Since ρ_2 is nearly equal to ρ_1 in equation 22.13, it can be approximated by:

$$c \approx \sqrt{\frac{(\rho_2 - \rho_1) g\, \theta_1 \theta_2}{\rho_1\, h}} \tag{22.14}$$

$$c \approx \sqrt{\frac{\delta\, g\, \theta_1 \theta_2}{h}} \tag{22.15}$$

in which:

$\delta = \frac{\rho_2 - \rho_1}{\rho_1}$ is the relative density of the water masses,
and
h is the total depth $= \theta_1 + \theta_2$.

These waves can be very high, since the gravitational influence on them is small. They are accompained by much smaller negative waves on the water surface, as shown in figure 22.3. Indeed, as a first approximation, the ratio of surface wave height to internal wave height is equal to δ.

These internal waves can absorb a considerable energy from a ship causing the so-called "dead water". This is explained via an example.

A ship of 4 m draft sails into a stratified harbor having a surface layer 3 m thick of relatively fresh water (salinity, S, = 5 ^{0}/oo and temperature, t, = 2^0C) above a deeper layer 7 m thick with S = 35 ^{0}/oo and T = 4^0C*. What is the maximum speed that this ship can attain?

* This stratification is quite common in northern harbors during the spring snow-melt runoff.

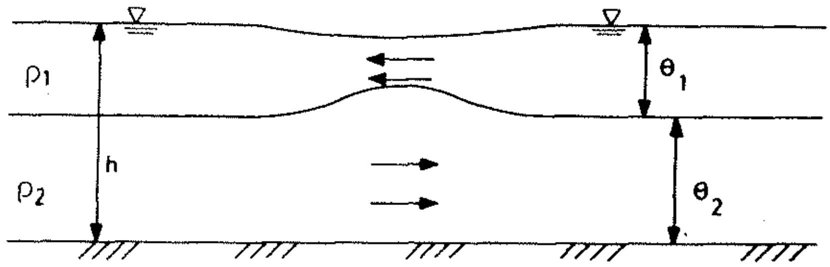


Figure 22.3 INTERNAL WAVE

From table 3.3, chapter 3,

$$\left.\begin{array}{l}\sigma_{t1} = 4.00 \; : \; \rho_1 = 1004.00 \; \frac{kg}{m^3} \\ \sigma_{t2} = 28.70 \; : \; \rho_2 = 1028.70 \; \frac{kg}{m^3}\end{array}\right. \qquad (22.16)$$

Further, $\theta_1 = 3$ m, $\theta_2 = 7$ m. Using (22.13):

$$\left.\begin{array}{l} c = \sqrt{\dfrac{(1028.7 - 1004)(9.81)(3)(7)}{(1004)(7) + (1028.7)(3)}} \\ \quad = 0.709 \text{ m/s} = 1.38 \text{ kt.}\end{array}\right\} \qquad (22.17)$$

The only way the ship can move faster than this wave is to cut through it or climb over it; neither is very likely!

This dead water phenomona also played an important role in a naval battle near Copenhagen some centuries ago. In this area the rather fresh Baltic Sea water flows over more dense water from the Skagerrak.

22.6 The "Static" Salt Wedge

A salt wedge occurs in a fresh water river which discharges into a saline sea. The sea water intrudes along the river bottom under the fresh discharge water. The length of the intruding wedge is determined by an equilibrium between the friction, τ_I, along the interface and the horizontal pressure gradient resulting from inclination of the interface. When this equilibrium is strictly satisfied, the salt wedge will be in a stable position with the fresh water flowing seaward on the surface and spreading out in a thin surface layer at sea. The length of this wedge is of great importance as will be pointed out in more detail later in this chapter.

Schijf and Schönfeld (1953) derived an expression for the length of such a wedge in a prismatic, horizontal, rectangular channel discharging into an infinite, non-tidal sea. If no mixing occurs across the interface, then their equation is:

$$L_w = \frac{2h}{f_I}\left[\frac{1}{5F^2} - 2 + 3\,F^{2/3} - \frac{6}{5}\,F^{4/3}\right] \qquad (22.18)$$

$$\text{with: } f_I = \frac{8\,\tau_I}{\rho\,|V_1 - V_2|\,V_1 - V_2} \qquad (22.19)$$

$$\text{and: } F = \frac{V_r}{\sqrt{\Delta g\; h}} \qquad (22.20)$$

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in which:

L_w is the length of the wedge,
V_r is the velocity in the river upstream from the wedge,
V_1 is the velocity in the fresh water above the wedge,
V_2 is the velocity in the salt wedge, and
τ_I is the friction stress along the interface.

This is all shown in figure 22.4.
This expression illustrates the influence of water depth, h, the river discharge velocity, v_r, and the density differences on the salt intrusion. A reasonable value for f_I is in the order of 0.1. Of course, in the idealized equilibrium state, $V_2 = 0$. This is why no friction stress on the bottom is shown in figure 22.4. The data used to plot this figure were: $f_I = 0.08$; $h = 10.$ m; $V_r = 0.2$ m/s; and $\delta = 0.0246$, giving $L_w = 2689$ m. The figure is drawn with a distortion of 1 : 100.

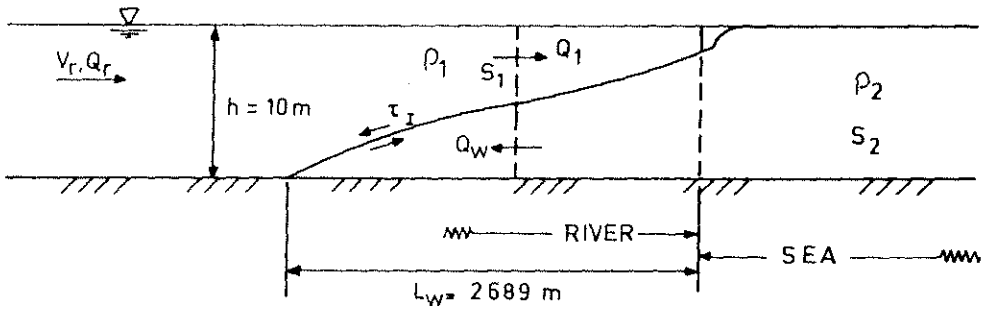

Figure 22.4 "STATIC" SALT WEDGE IN RIVER MOUTH (distortion 1:100)

The word static is enclosed in quotation marks since there is, in a real situation, more a state of dynamic equilibrium. Mixing will take place along the interface between the water masses. Salt and sea water will be transported along with the river water back to the sea. This is indicated in figure 22.4 at the vertical dashed line part way along the wedge. Since the total net flow out the river must be equal to the fresh water runoff:

$$Q_1 = Q_r + Q_w \tag{22.21}$$

in which: Q_w is the inflow flow in the wedge,
Q_r is the fresh water river flow, and
Q_1 is the net outflow through the cross section.

Continuity of salt must also be maintained. This implies that:

$$Q_1 S_1 = Q_w S_2 \tag{22.22}$$

where S_1 and S_2 are the respective salinities.

If one substitutes different values of V_r into equation 22.09 (via 22.11) he will find that L_w decreases as V_r increases; indeed, F = 1 yields $L_w = 0$. Remembering that increasing V_r implies also an increasing Q_r we seem to discover a contradiction with the rules of thumb presented in equation 22.01 and table 22.2. According to that table, increasing Q_r should lead to a more stratified estuary, and hence, a longer instead of a shorter salt tongue (wedge). This dilemma is explained by realizing that all tidal influences have been neglected in formulating equation 22.18; thus, this comparison is invalid.

In a real estuary the salt wedge intrusion problem is much more complex. The river flow, Q_r, varies, tidal influences are present, and the estuary is certainly not prismatic.

Generally, the tidal influence is most important - it leads to an incessant oscillatory motion of the entire two-layer system over an uneven bottom. This motion, of course, increases mixing across the interface. Indeed, in estuaries with strong tidal influence and little fresh water flow the stratification can be essentially destroyed, leading to a well mixed estuary. The Western Schelde is an example of such an estuary. At a given time and place there is little vertical salinity gradient. In such an estuary, the average seaward transport of salt by the river flow is an equilibrium with transport of salt into the estuary by diffusion.

The effect of this diffusion (which is always present to some extent) combined with the momentum of a possible inward flowing salt tongue can delay the time of maximum average salinity at a point on a tidal river until a bit later than the H.W. slack, as observed with the data from Rotterdam, figure 22.1.

As has already been indicated, the tides cause the salt tongue or the haloclines to move back and forth in the river as a function of the tide. The consequences of this presence and movement of the salt tongue for the river and its surroundings are discussed in the next section.

22.7 Siltation Problems

The most direct consequence of a salt tongue in a river is its effect on the siltation pattern of the estuary. Obviously, from figure 22.4, the current along the estuary bottom is drastically changed by the presence of the salt tongue. Upstream from the tip of the tongue, the velocity along the bottom is toward the sea, while within the wedge there is often a small velocity into the estuary. Since the bottom velocity at the tip of the tongue must be zero, it can be expected that material will be deposited there. In estuaries where there is little tidal influence and the position of the salt wedge remains relatively stable, this local sedimentation can form a pronounced shoal in the river. While the cause of this tongue has been attributed to salt, above, this phenomona can also be found in an estuary having a density difference caused by other factors such as thermal gradients. This phenomona might, for example, also be observed in the cooling water discharge channel from a power station, even one located on a fresh water lake.

When the suspended sediment in a river consists of clay and the density tongue is caused by salinity differences, then physical chemical processes can also strongly influence the siltation pattern in the estuary.

Suspended clay in fresh water consists of flat or needle - shaped particles having a maximum dimension less than a few micrometers. Because of their form, large surface area and the crystal structure of the clay minerals, these particles are negatively charged on the surface. Since the particles are so small, the electrostatic forces rather than the gravity forces control the behavior of the clay particles, and work to keep the particles separated and in suspension.

As the salinity of the water increases, the positive ions (Na^+, Mg^{++}, Ca^{++}, etc.) present tend to neutralize the electrostatic forces, thus allowing the clay particles to flocculate, and settle. A salinity of about 3 ^{o}/oo is critical in this process. The physical chemical influences are only important for salinity variations below this value.

The flocculation caused by an increase in water salinity is at least partially reversible. When, later in the tide cycle, the salinity decreases, the flocs of clay particles exposed to the fresh water can "explode" dispersing the individual particles once again in suspension. This process can provide disturbing influences on suspended sediment measurements in areas where low, variable salt concentrations can be found.

An impression of the magnitude of this influence on siltation can be gained by comparing the fall velocity* of clay particles in fresh water to the fall velocity of flocs of particles in salt water ($S > 5$ ^{o}/oo). Allersma, Hoekstra and Bijker (1967) report that the apparent ratio between these fall velocities was more than 1 : 50.

The quality of the material forming the river bed in such an area is not the same as the usual form of compact clay. Indeed, the sediment which forms as a result of flocculation contains a large quantity of water. The volume of the sediment (solid particles plus water) can be 5 to 10 times the volume of the particles. (In soil mechanics terminology, the void ratio can be as high as 10). Obviously, such a high volume of water will keep the sediment density low - usually between 1100 and 1250 kg/m^3. The material behaves as a viscous fluid with a viscosity of in the order of 100 to 5000 times that of water; this is comparable to yoghurt (except for color!).

This material, often called sling mud, is difficult to detect when making soundings. It appears as a faint reflection on an echogram. The sediment is so soft that ships can often sail through it.

The consolidation process for such a soft silt is very slow. Layers up to 2.5 m thick remain fluid for several weeks - even in a laboratory settling tube.

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* The fall velocity is the velocity at which sediment particles drop through still water.

This sling mud can be brought into suspension once again when the current velocity above it reached a critical value ranging between 0.2 and 1.0 m/s.

Sling mud will be discussed again in more detail in chapter 27 about the morphology of mud coasts.

The influence of the salinity on the suspended silt concentration at Rotterdam is demonstrated in figure 22.5. Data used to plot this figure for Rotterdam is listed in tables 22.1 and 22.3.

TABLE 22.3 Suspended Load At Rotterdam

Time (hrs)	Suspended material (mg/l)
0	60
1	58
2	68
3	90
4	116
5	95
6	61
7	54
8	77
9	90
10	93
11	81
12	66

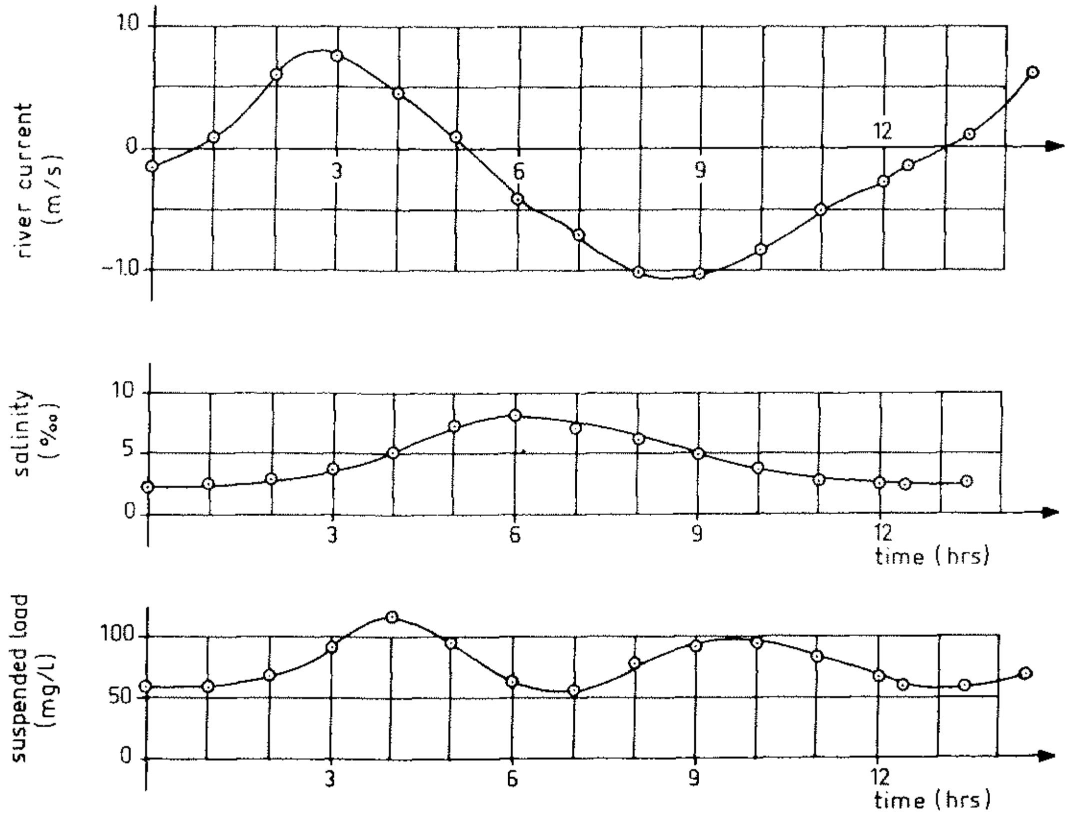

Figure 22.5 SUSPENDED LOAD AT ROTTERDAM

In the time interval between 5 and 7 hours the relatively high salinity causes the suspended sediment concentration to decrease, even though the current is becoming stronger. Between 8½ and 10 hours, the reverse is true; decreasing salinity increases the suspended sediment even though the velocity is becoming weaker. On the other hand, near 0.6 hours and again at 13 hours, the sediment concentration minima are caused by the low current velocity.

22.8 Pollution Problems

In addition to increasing the siltation problems in an estuary, density currents can also cause problems of environmental pollution.

The most obvious source of environmental pollution is the infiltration of salt water into the surrounding ground water along a river. The deleterious effects of water salinity on the growth patterns of plants have been well documented by agricultural specialists. The prediction of the severity of saline pollution for a given location is a topic of study for specialists in ground water hydrology.

Another, often less obvious, pollution problem can be caused by the presence of thermal density currents. Marine life such as shellfish often is unable to adapt to rapidly varying water temperatures experienced when the edge of a thermal plume drifts over it at some time in a tide cycle. Several large and elaborate model studies have been conducted in various laboratories, both in the United States and the Netherlands to determine the extent and severity of thermal plumes from steam power stations to be located along estuaries. Demonstration that the plume of discharged cooling water will not harm the surrounding marine life is often required before a construction permit will be granted.

Techniques used to combat the deleterious effects of density currents are discussed in the next section.

22.9 Methods to Combat Density Current Influences in Rivers

There are relatively few techniques which are economical for combating the intrusion of a salt tongue in a river. Many more technique are available for more restricted areas such as harbor basins and canals; these will be discussed in the following chapter.

It has been indicated via equations 22.09 through 22.11 that the length of the salt wedge can be reduced by decreasing the water-depth and by increasing the fresh water flow. In the Netherlands, the discharge of fresh water through the New Waterway has been increased as a result of the completion of the northern part of the Delta Project (Volkerak dam and locks, and the Haringvliet Sluice). In addition, the development of the Europoort harbor area has eliminated the necessity for bringing large, deep ships into the New Waterway past the Europoort entrance. In recent times, therefore, parts of the New Waterway in Rotterdam have been partially filled in order to decrease their depth and drive the salt water tongue back toward the sea to a greater extent.

Thermal density currents can be combated by either enhancing the mixing of the two water layers or stimulating the heat transfer process between layers or to the atmosphere.

Although not too common in use, mixing can be enhanced, for example, by increasing the turbulence in the thermal discharge or artificially generating an unstable stratification. Increasing discharge velocity and construction of a pile supported jetty in front of the discharge flume of a powe station have been suggested as means to increase mixing by increasing turbulence.

Naturally unstable stratifications are often artificially generated when warm, low salinity sewage is discharge near the bottom of the sea. As the lighter sewage rises through the sea water the resulting turbulence helps to disperse it.

Obviously, another solution to thermal pollution problems is to re-cool the discharge water before it is released. This may be accomplished by retention is shallow pools or by circulation through a cooling tower. Sometimes, simply a long wide discharge channel can serve the purpose. The objective in all of these solutions is to transfer the heat to the atmosphere.

E.W. Bijker
J. de Nekker

23. DENSITY CURRENTS IN HARBORS

23.1 Tide Flow in Harbor

In this chapter the tide and density current influences on a harbor built along a tidal river will be discussed. The information presented in this section, however, will be of general use, even for harbors located along a coast far away from a river or on an estuary without fresh water runoff.

The construction of a harbor along a tidal river will obviously increase the tidal prism of the estuary. Usually, unless there is a very significant and extensive harbor development, the influence of the additional harbor area on the total tidal prism will not be enough to cause significant changes in the river itself.

We have already seen in chapter 20 (figure 20.5) how inertia effects maintain a flood current in a river even after high water. For a harbor, on the other hand, the inertia terms are much less important and the current in the harbor mouth will be slack just at the time of high and low water. This is true when no density effects are involved. Table 23.1 lists the data used to plot figure 23.1 showing this phenomona for the 2e Petroleumhaven in Rotterdam. (The density current influences have been eliminated from the data listed in the table). Since the currents are so small, they are listed in centimeters per second.

TABLE 23.1 HARBOR TIDE AT ROTTERDAM (2e Petroleumhaven)

Time (hrs.)	Harbor Tide level (m)	River Current (m/s)	Harbor Filling Current (cm/s)
0	-0.69	-0.15	0.9
1	-0.50	+0.08	2.2
2	-0.03	0.60	3.2
3	+0.52	0.75	2.2
4	0.91	0.44	1.1
5	1.04	+0.07	0
6	0.91	-0.44	-1.5
7	0.61	-0.73	-2.1
8	+0.25	-1.03	-1.6
9	-0.15	-1.05	-1.1
10	-0.47	-0.85	-1.5
11	-0.58	-0.52	-0.8
12	-0.62	-0.30	0

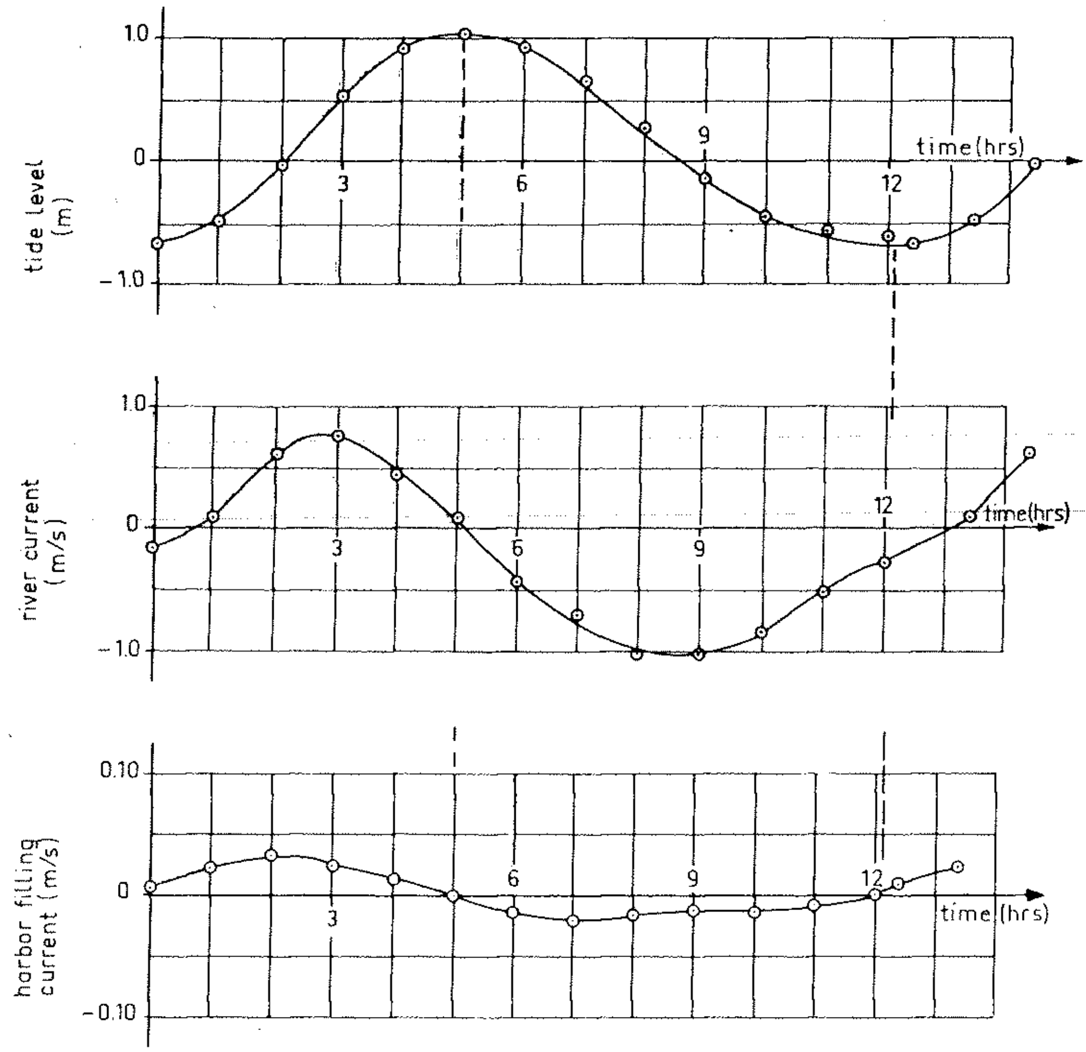

Figure 23.1 HARBOR LEVEL AND CURRENT, 2[e] Petroleumhaven, Rotterdam

23.2 Density Currents in Harbors

The density stratification at the mouth of a harbor basin just after the river salinity has changed can be schematized by a vertical interface such as was shown in figure 22.2 in the previous chapter. As was already pointed out, there, this condition is unstable and leads to a current pattern as is shown in figure 23.2. The flow of the more dense layer can be compared to the flow of water down a river valley just after a dam has burst. Such a profile of the interface is, therefore, sometimes called a dry bed curve. The toe of the dry bed curve is held back slightly by the friction along the bottom.*

Since the volume of water in the harbor remains constant - neglecting filling or emptying - the harbor inflow must equal the outflow caused by the density difference. Since the usual assumption is that the flow in each direction occurs over half the depth, then the two flow velocities must be equal for a rectangular channel.

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* Compare to figure 22.4 in which there is no bottom friction.

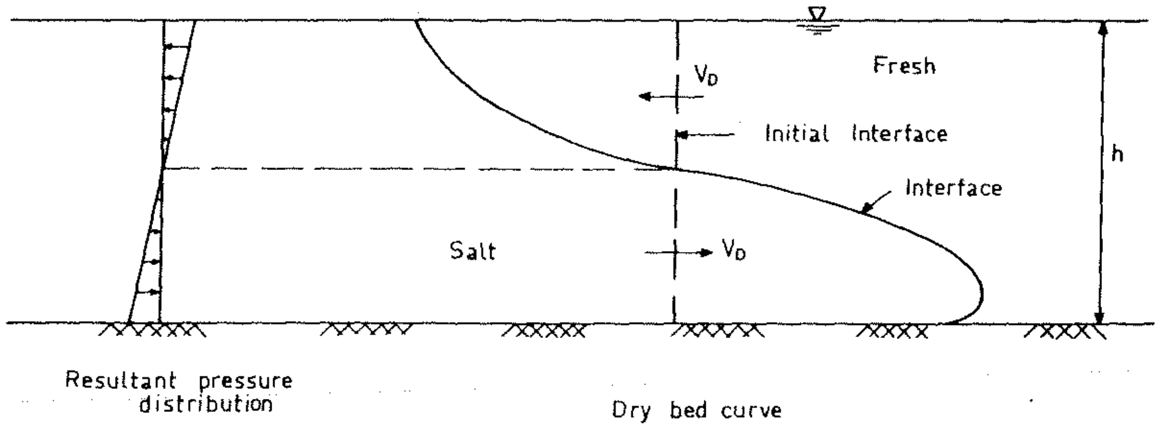

Figure 23.2 DENSITY CURRENT FORCES AND MOTION

Theoretically:

$$V_D = 0.45\sqrt{\delta\, g\, h} \qquad (23.01)$$

in which:

δ is the relative density (ch. 22),

g is the acceleration of gravity,

h is the water depth, and

V_D is the velocity in the dry bed curve.

In practice, the coefficient, 0.45, is a bit too large; a value somewhere between 0.3 and 0.4 usually gives better results. Equation 23.01 compares favorably, but not exactly, with equation 22.13 when $\theta_1 = \theta_2 = \frac{h}{2}$.

Table 23.2 lists river and harbor salinities for Rotterdam as well as the measured density current velocity. The values of δ listed are computed from the salinity data assuming that both the river and the harbor are at a uniform temperature of 16°C. The density current velocities are given for the surface current with positive indicating a flow into the harbor. By symmetry, as already explained, the flow in the lower layer must be in the opposite direction with the same speed. Some of the data from this table are plotted in figure 23.3.

We see from table 23.2 that the magnitude of the density current velocity more or less follows the value of δ. If theory and practive always agreed, then there should be a perfect correlation between $|V_D|$ and $\sqrt{\delta}$ (from equation 23.01). The correlation coefficient for $|V_D|$ against $\sqrt{\delta}$ for the data in table 23.2 is only 0.58, however. This does not make the theory look too good, but this comparison shall be re-examined in section 23.4.

TABLE 23.2 SALINITY AND DENSITY CURRENTS AT ROTTERDAM

Time (hrs.)	River S ($^o/oo$)	Harbor S ($^o/oo$)	δ	V_0 at surface (cm/s)
0	2.38	3.96	1.224×10^{-3}	3.0
1	2.47	3.30	5.952×10^{-4}	4.0
2	2.83	3.04	1.619×10^{-4}	1.2
3	3.64	2.63	7.830×10^{-4}	-5.0
4	5.08	3.01	1.600×10^{-3}	-8.0
5	7.25	3.91	2.567×10^{-3}	-10.7
6	8.06	5.23	2.180×10^{-3}	-10.3
7	7.16	6.56	4.616×10^{-4}	-1.4
8	6.08	6.69	4.679×10^{-4}	+2.1
9	4.90	6.37	1.128×10^{-3}	2.5
10	3.64	5.43	1.379×10^{-3}	2.5
11	2.65	4.36	1.325×10^{-3}	2.1
12	2.38	3.82	1.116×10^{-3}	2.1

δ computed from salinities at T = 16 C. using table 3.3.

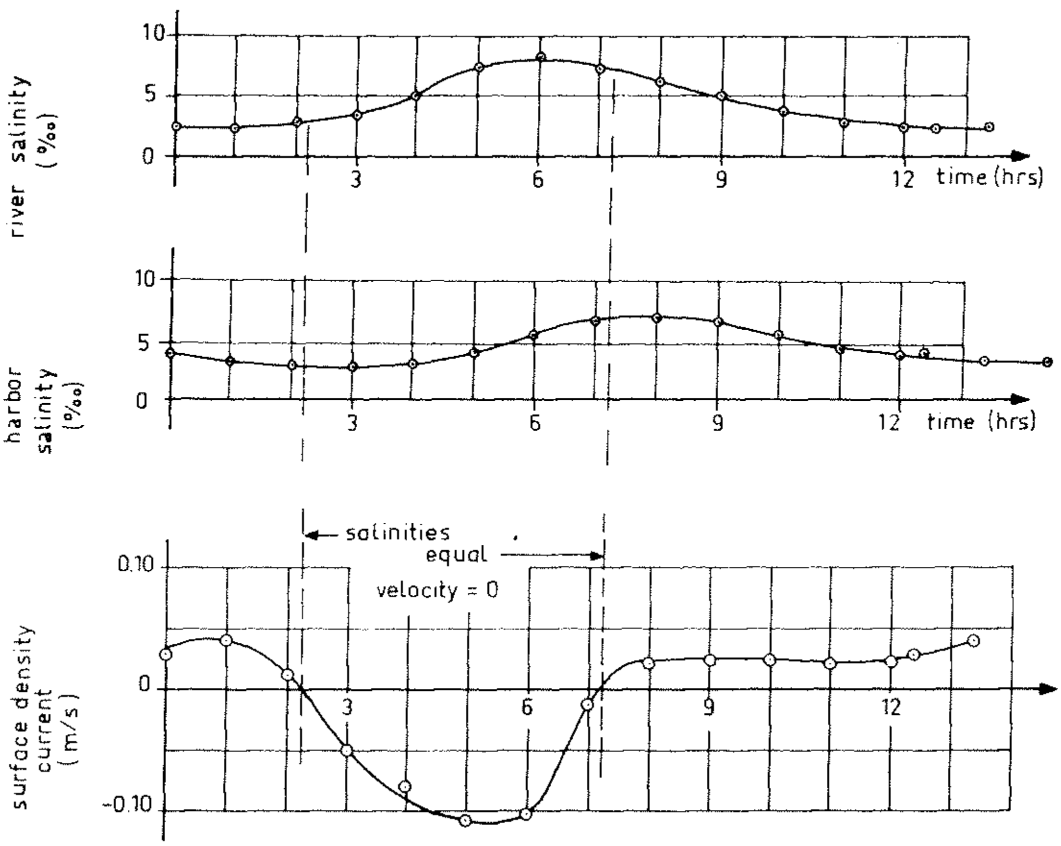

Figure 23.3 HARBOR SALINITY AND DENSITY CURRENT
(2e Petroleumhaven, Rotterdam)

23.3 Superposition of Current Components

In a real harbor on a tidal river, the flow in the harbor will be a superposition of the filling flow and that caused by the density current. Figure 23.4 shows the idealized current profiles and their superposition for various times listed in tables 23.1 and 23.2. When $|V_D| < V_f$, the presence of a density current component does not affect the total volume of water entering the harbor. This is demonstrated in figure 23.4 by the velocity profiles for time equals 2 hours. The implication of this observation is that velocity distributions can be superimposed while the sediment transports cannot be simply added except when the sediment concentration is constant over the entire depth. This discussion comes up again in later sections of this chapter.

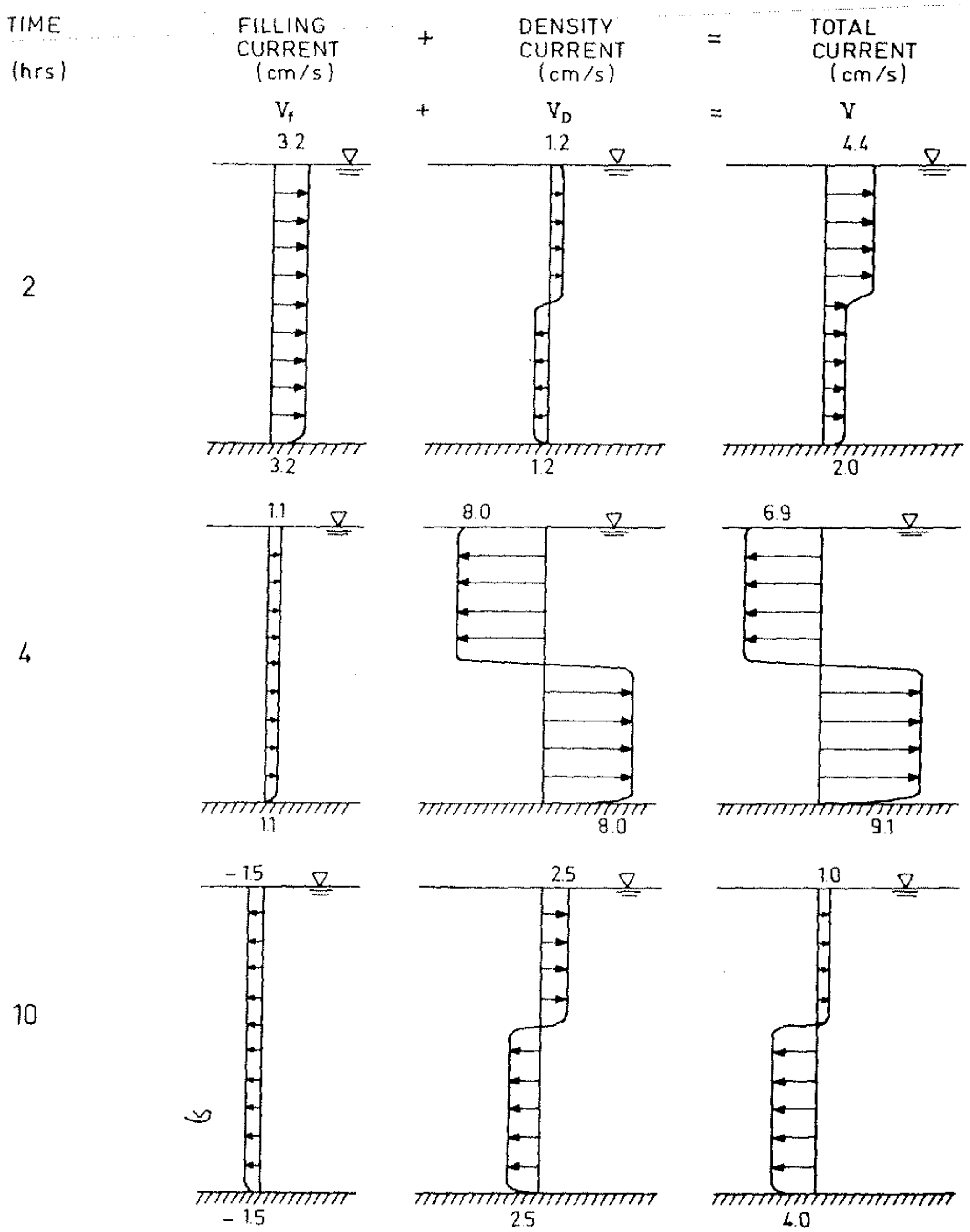

Figure 23.4 HARBOR ENTRANCE VELOCITY PROFILES

Until now, the assumption has been made that the harbor had an infinite length. In the next section, we examine the extra conditions imposed upon this theory by the finite length of a harbor.

23.4 Currents in Finite Harbors

The dry bed curve of a density tongue rushing into a harbor has been shown in figure 23.2. Equation 23.01 described its velocity. How far does such a tongue penetrate into a harbor?

Two conditions must be satisfied for a density tongue to continue progressing in a harbor basin:

a. it must have somewhere to go, and

b. the driving force (density difference) must still exist.

The first of these is dependent only upon the harbor geometry while the second criteria depends upon the water alone. In order to separate these conditions for discussion, let us first assume that initially all of the water in a harbor basin and the adjacent river has a density of 1005 kg/m^3. At some instant, the density of the water in the river increases to 1015 kg/m^3, and maintains that value indefinitely; thus, the driving force (item b, above) is maintained. There is no tide. The harbor has a rectangular form and has depth h = 7 m and length L = 2500 m. (see figure 23.5).

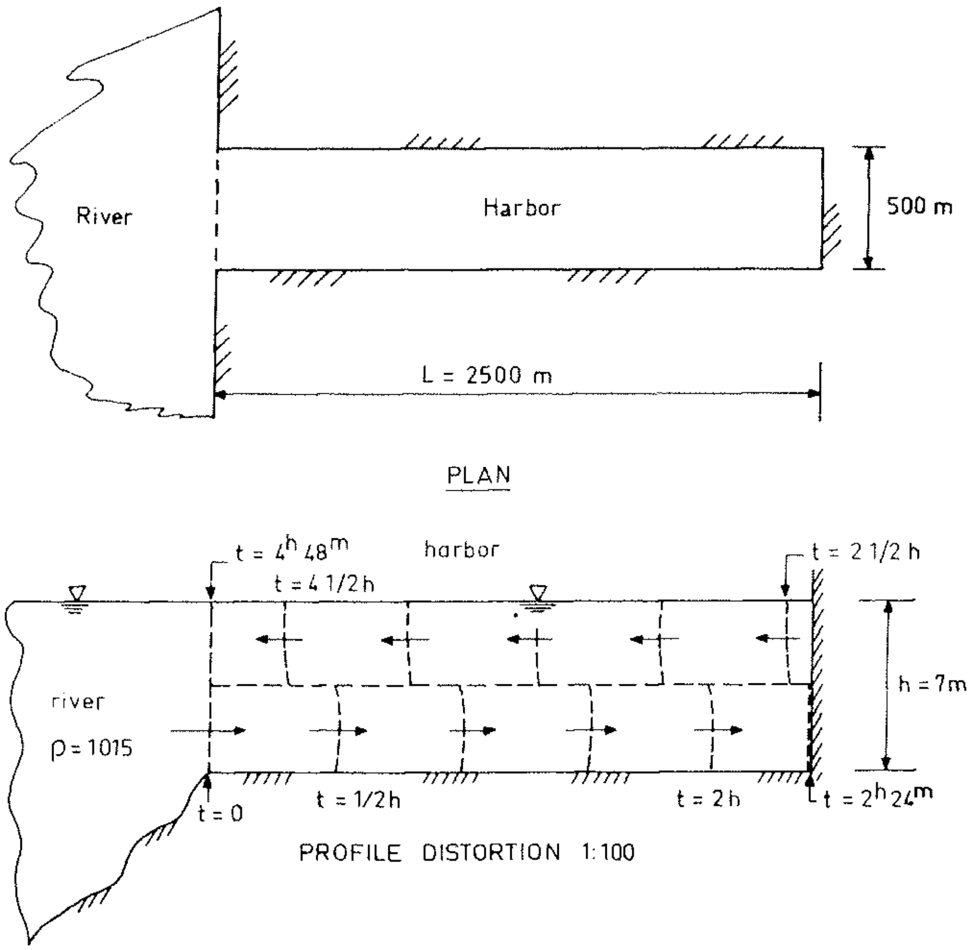

Figure 23.5 PROGRESS OF DENSITY CURRENT IN HARBOR

Using 23.01, we find the density current speed:

$$V_D = 0.35 \sqrt{(\frac{1015 - 1005}{1005})(9.81)(7)} \tag{23.02}$$

$$= 0.289 \text{ m/s} \tag{23.03}$$

$$= 1042 \text{ m/h} \tag{23.04}$$

With this speed, the tongue progresses without hinderance over the length of the basin - 2500 m - arriving at the inner end in 2^h 24^m. The wave then reflects from the inner end of the harbor, just as does any other long wave, and propagates back toward the entrance at the same speed arriving there 4^h 48^m after the cycle started. The progress of the tongue after each half hour interval is shown by the dashed lines in figure 23.5.

After 4^h 48^m the tongue has returned to the harbor entrance. The harbor is now filled with more dense water - the same density as the river - and the process stops, since there is no longer a density difference across the harbor entrance.

What has happened to the less dense water that was originally in the harbor? That water has spread over a large area of the river in a thin layer, where wave action enhances its mixing with deeper water.

This example also yields some additional insight into the data presented in table 23.2 and figure 23.3. The average salinity (if the density difference is of saline origin) in the harbor increases linearly with time during 4^h 48^m in the example above, but the density current remains constant over this time period; it is completely determined by the density difference at the harbor entrance. Thus, the direct correlation between $\sqrt{\delta}$ and $|V_D|$ is really incorrect when δ is determined based upon *average* salinities.

The time required for the density current to enter a harbor and exchange the contents explains the phase lag between the peak salinities in a river and in an adjacent harbor - see figure 23.3. Does a complete water exchange take place? Most likely, it does in this case even though the maximum harbor salinity is less than that in the river. By the time the harbor exchange has taken place, the river salinity is no longer maximum. Additional evidence that a complete exchange takes place is given by the abrupt change in density current velocity in figure 23.3 between 6½ and 7 hours. Since there is no abrupt change in salinity at that time, the velocity decrease must be caused by removal of the effective driving force.

The second type of problem, in which there is insufficient time for a complete exchange, is somewhat more complex. This is illustrated via the following example.

This example is exactly the same as the previous one in that the harbor initially contains water having ρ = 1005 kg/m^3 and the river abruptly changes density from 1005 to 1015 kg/m^3. This time however, this higher density will be maintained in the river for only 1^h 12^m, after which the river density will again become 1005 kg/m^3. Indeed, the problem is exactly like the previous example in all respects for the first 1^h 12^m. This is shown in figure 23.6A.

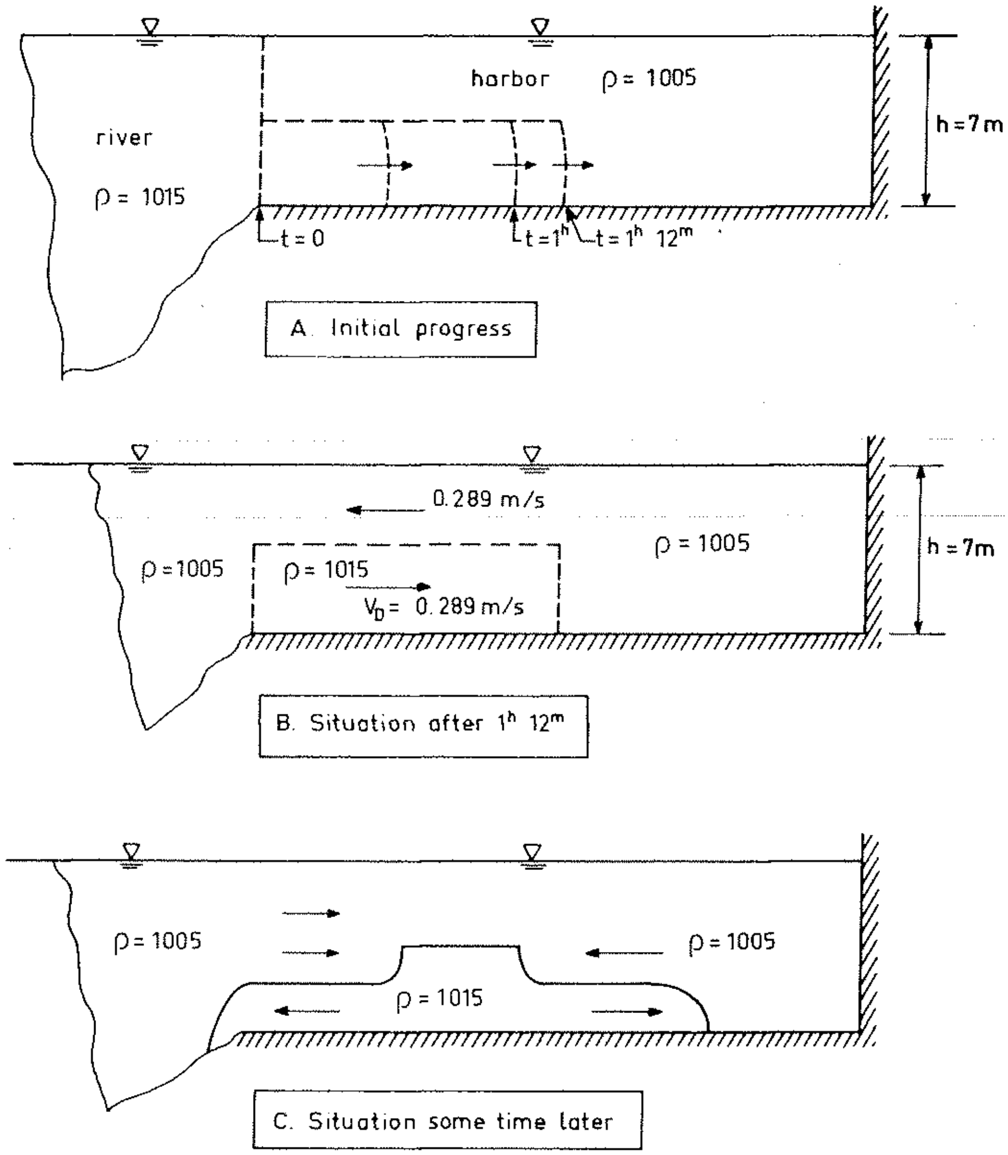

Figure 23.6 DENSITY CURRENTS IN HARBOR

After 1^h 12^m the situation will be as shown in figure 23.6B. The driving force is no longer present. Momentum will keep the slug of salt water moving for a short time, but other influences become important since the ends of the slug of dense water are unstable. Dry bed curves will develop at the ends of the 3.5 m thick lower layer causing the slug of water to spread out in a thinner layer along the harbor bottom. Ultimately, of course, this thin layer could retreat entirely to the deeper river. Quantitative evaluations of all these processes are beyond the scope of this course and are not necessary for our main purpose - the determination of the quantity of silt which enters the harbor along with the more dense water. An impression of the form of the interface between the two water masses at some later time is shown in 23.6C.

23.5 The Practical Problem

The discussion just presented holds true only to the extent that its assumptions are satisfied. A glance at figure 23.3 is sufficient to see that the assumption that the river density changes abruptly does not hold true in nature. Secondly, many harbors are not rectangular in form. A dependable theoretical computation of the water exchange in a harbor of arbitrary shape on a given river is extremely time consuming at best. For this reason, physical model studies are often used; a significant portion of the Delft Hydraulics Lab is devoted to the modeling of saline density currents.

A second approach to the problem is to develop a semi-emperical equation for the water exchange having coefficients based upon experience with existing harbors. This equation can then be used to predict the exchange taking place in a similar harbor under the same conditions. Such an approach has been taken at the harbor of Rotterdam. Using measurements made in several of the larger harbors, (Botlek, 1[e], 2[e] Petroleumhaven, Europoort) it was determined that:

$$\forall_D = G\, A_E \sqrt{\delta'\, \bar{h}} \qquad (23.05)$$

in which:

A_E is the cross sectional area of the entrance in m^2,

G is a coefficient having a value of 8000 $m^{\frac{1}{2}}$/tide period for these harbors,

$\bar{h}$ is the average depth of the harbor in meters, and

δ' is the relative density defined as:

$$\delta' = \frac{\rho_{max} - \rho_{min}}{\bar{\rho}} \qquad (23.06)$$

in which

ρ_{min} is the minimum river density,

ρ_{max} is the maximum river density, and

$\bar{\rho}$ is the average river density over one tide period, and

$\forall_D$ is is the *total* water volume exchanged during an *entire* tide period.

The method just described depends upon having an existing harbor along the tidal river. Further, the size and geometry of a projected harbor is not always comparable to that of an existing harbor. In such cases, the above scheme is of little help, since the coefficient G cannot be determined.

The density current can be of major importance for harbor siltation resulting in maintenance dredging costs. An estimate of the density current, therefore, can be of vital importance for feasability studies. Even a crude computation can be helpful in such cases. The following approach is suggested in an anonymous report (1960) by the Delft Hydraulics Laboratory. The expected accuracy of the method is in the order of ± 50%.

The anonymous author's - in fact Gersie and Bijker - attack the problem by defining a coefficient, α, which gives the ratio of total water exchange volume to harbor volume. Since there are two influences involved, filling plus density current, α is split into two components:

$$\alpha = \alpha_f + \alpha_D \tag{23.07}$$

where:

α is the ratio of water volume entering the harbor per tide to the harbor volume,

α_f is that portion caused by the filling, and

α_D is caused by the density influence.

α_f can be evaluated by comparing the tidal prism of the harbor to the total harbor volume.

$$\alpha_f = \frac{P}{\text{\(V\)}_H} = \frac{\Delta h}{\bar{h}} \quad * \tag{23.08}$$

where:

$\bar{h}$ is the average harbor depth

Δh is the difference between the high and low tide levels,

V_H is the total harbor volume based upon depth $\bar{h}$, and

P is the tidal prism of the harbor basin.

α_D is not independent of the harbor filling - see section 23.3 - but is dependent upon the filling current component as well.

$$\alpha_D = \frac{(V_D - |V_f|)\, T_D}{2\, L} \tag{23.09}$$

where:

V_D is the density current velocity,

V_f is the filling current velocity,

L is the length of the harbor, and

T_D is the time interval over which the density difference exists.

Now some problems begin to appear! What density ratio, δ, should be used to compute V_D? How is T_D determined? What is L for a complex harbor?

Starting with this last question, we are working, in fact, with a schematized rectangular harbor with cross sectional area equal to that of the entrance. The length follows from our schematization, and is usually just about the longest distance from the entrance to an extremity of the harbor. A bit of experience is very helpful in making this schematization.

T_D is the time during which a density difference is present. It is related only to the density - time curve for the estuary and is not necessarily directly related to the tide level. Also, T_D during the time of increasing harbor density may be different from that for decreasing density.

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* The second step is valid if the sides of the harbor are vertical.

The most difficult question is one of determining the proper value of V_D. The following approach is suggested:

a. Compute V_D based upon a δ corresponding to the extreme values of the schematized density in the river using 23.01.

b. Using this value of V_D, compute α.

c. If $\alpha < 1$, then the maximum harbor density will be less than the river maximum and our assumption in step a is violated. In that case, repeat steps a and b using a new value of δ one half as large as the original one.

In equation 23.09 the absolute value of the filling current has be averaged over the time T_D. Note that the absolute value has been taken before the average. The value of V_f for use here is determined from the vertical tide in the river.

$$V_f = \frac{A_H}{A_E} \frac{dh}{dt} \tag{23.10}$$

where:

A_H is the surface area of the harbor, and

A_E is the cross sectional area of the entrance.

Actually, equation 23.09 does not tell the whole story. The following inequality must also be satisfied:

$$0 \leqq \alpha_D \leqq 1 \tag{23.11}$$

Thus, the influence of the density current may not be negative; it may be zero. The upper limit on α_D is imposed by the schematization of the density - time curve.

The value of α_D can be corrected for the fact that the harbor has been schematized by multiplying it by the ratio of surface areas of the schematized and actual harbors, respectively.

The volume of water entering the harbor during a complete tidal period can best be determined as the sum of its components. The harbor filling current contributes a volume of water equal to α_f times the harbor volume. This same volume of water flows out when the water level falls. During the period that the river density is high, α_D* times the harbor volume flows into the harbor. When the river density is low, α_D* times the harbor volume flows out.

Both computational techniques outlined in this section are illustrated in section 23.7.

23.6 Other Current Influences

The current pattern in a harbor mouth can be even more complicated than has already been described. The complication can exist in the form of an eddy rotating about a vertical axis in the harbor entrance. Water exchange between the harbor and eddy on one side and between river and eddy on the other can increase the transport of salt and suspended sediment into the harbor.

* These two values of α_D are not generally equal.

When a harbor is small, the density current can usually carry out a complete water exchange rather quickly, but then stops transporting silt laden water into the harbor. The eddy, on the other hand, continues functioning exchanging sediment laden river water for clearer harbor water. This cause can be the most important of all three causes for the transport of sediment into a small harbor.

Eddies form at the entrance to larger harbor basins as well. However, these tend to be excited by the other current components in the harbor entrance rather than the river current. As such, they contribute little to the supply of sediment to the harbor.

An attempt will be made in the next sections to quantify the amount of siltation to be expected in a harbor. Before attacking that, however, we should consider the effects of the presence of the harbor on shipping in the river.

It takes little imagination to realize that near the mouth of a harbor, where eddies, density currents, river currents and harbor filling currents are all competing with one another, the current pattern can be rather confused. Small, shallow draft ships will only be concerned with the surface currents. Larger, deeper ships which penetrate the interface between layers are subjected to an even more complex pattern of current forces. Add to this the dead water phenomona described in the previous chapter, and we should realize easily the respect with which harbor pilots are usually treated. An enumeration of the various ways various ships can react to various current patterns would be too voluminous to include here. It is sufficient for our purpose to recognize that such ship maneuvering problems can and do occur and to have the sense to ask a pilot's advice about any extensive harbor changes.

23.7 Harbor Siltation

The same processes of siltation described for a tidal river in the previous chapter occur in adjacent harbors as well. Variations in salinity cause floculation and rapid settlement of fine material in harbors just as in rivers. In addition, however, the settlement of material in harbors proceeds even faster because of the relative tranquility of the water in the basin. Obviously, all of the phenomona which cause water exchange between the harbor and river also increase the supply of sediment to the harbor.

The harbor siltation is computed by multiplying the volume of water exchanged in one tide cycle in the basin by the difference in sediment concentration between inflowing and outflowing water. The role of each of the current components will be examined in the following example.

A harbor is located along a river in which the average suspended sediment concentration is 77 mg/l (this agrees with the data in table 22.3). The harbor is 2000 meters long and has a prismatic cross section with side slopes of 1:4. The tide range is 1.7 meters and the harbor depth at low water is 13.5 meters. Figure 23.7 shows such a harbor, having a bottom width of 400 m. Again using data from Rotterdam, the river has a maximum salinity of 8.06 $^{0}/oo$ and a minimum salinity of 2.47 $^{0}/oo$ (table 22.1) With a water temperature of 16^{0}C and table 3.3, we find that the maximum density in the river is 1005.18 kg/m^3 and the minimum density is 1000.85 kg/m^3, yielding:

$$\delta' = \frac{1005.18 - 1000.85}{1003.02} = 4.32 \times 10^{-3} \tag{23.12}$$

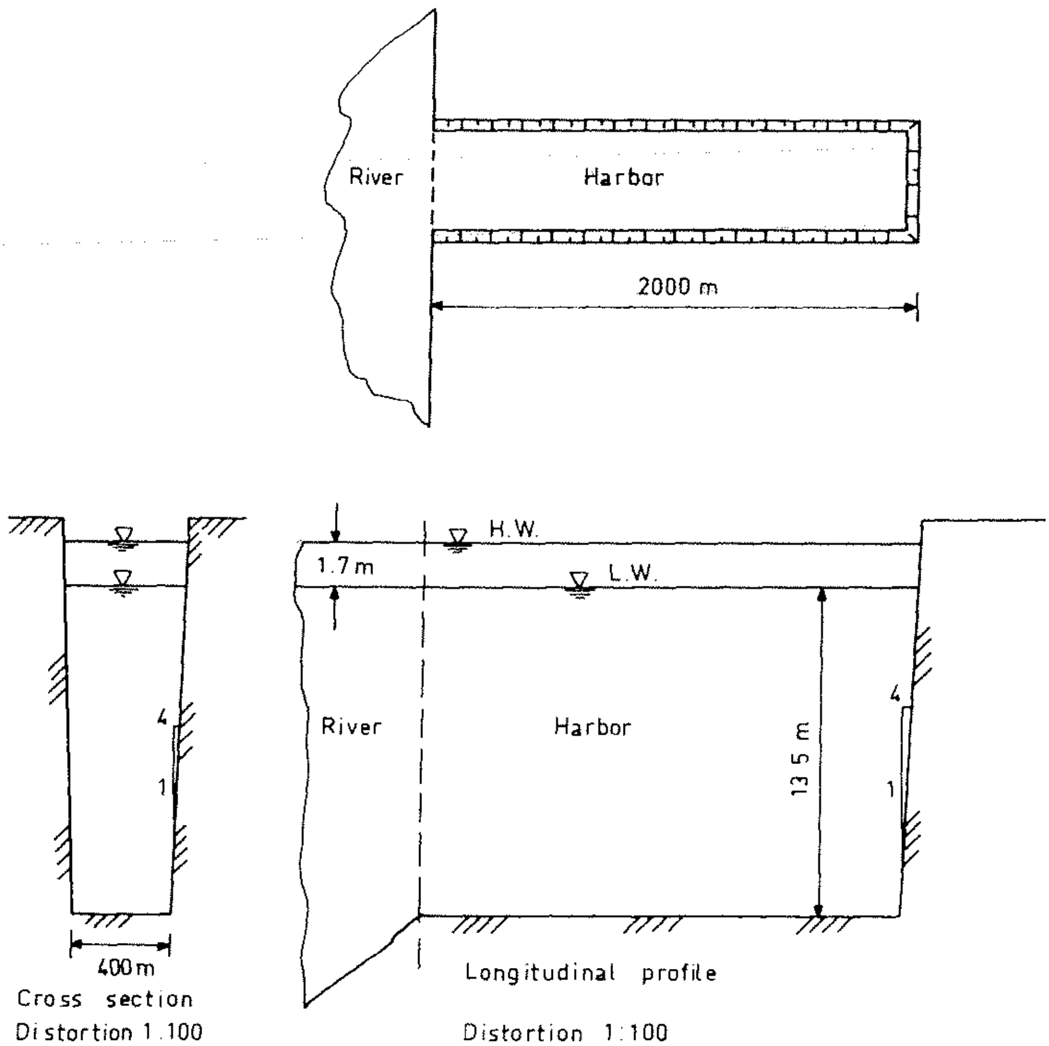

Figure 23.7 HARBOR EXAMPLE SKETCH

The average water depth in the harbor is:

$$\overline{h} = 13.5 + (\tfrac{1}{2})(1.7) = 14.35 \text{ m} \tag{23.13}$$

Yielding a top width of:

$$400 + (14.35)(8) = 515 \text{ m} \tag{23.14}$$

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The average flow area in the entrance is, then:

$$A_E = (\tfrac{1}{2})(400 + 515)(14.35) = 6565 \text{ m}^2 \qquad (23.15)$$

The tidal prism, P, of the harbor is the volume of water supplied per tide by the filling current.

$$P = (515)(2000)(1.7) = 1.75 \times 10^6 \text{ m}^3 \qquad (23.16)$$

Each liter of this water carries 77 mg of dry sediment into the harbor. Probably not all of this material will settle in the limited retention time. The concentration of sediment in discharged water can be estimated from laboratory tests or from experience in similar local harbors. For this problem, let us assume that the discharge water from the harbor carries an average of 10 mg/l of dry silt. Thus, 67 mg/l is retained in the harbor.

The amount of sediment transported into the harbor by the filling current* is then (with units conversion):

$$s_f = (1.75 \times 10^6)\ (67)(10^{-3}) = 1.17 \times 10^5 \text{ kg/tide} \qquad (23.17)$$

The influence of the density current is computed using equation 23.05. The total volume of water exchanged by the density current during a tide period is:

$$V_D = (8000)\ (6565)\ \sqrt{(4.32 \times 10^{-3})(14.35)} \qquad (23.18)$$

$$= 1.31 \times 10^7 \text{ m}^3\text{/tide} \qquad (23.19)$$

Half of this water, 6.53×10^6 m^3/tide, enters along the harbor bottom with the intruding salt tongue and brings:

$$s_{D_1} = (6.53 \times 10^6)(67)(10^{-3}) = 4.38 \times 10^5 \text{ kg/tide} \qquad (23.20)$$

sediment with it.

The other half of the exchange water enters the harbor along the surface as the salt tongue retreats. Since the surface water in a river usually contains less sediment, it transports relatively less sediment into the harbor. For Rotterdam, it is assumed that this surface current transports only 20% of the sediment found in the other currents into the harbor. Since this material will be finer than the average of all the material, it will settle more slowly. Thus, we can still assume that 10 mg/l leaves the harbor later. These considerations yield:

$$s_{D_2} = (6.53 \times 10^6)\ [(0.2)(77) - 10]\ \ (10^{-3}) \qquad (23.21)$$

$$= 3.53 \times 10^4 \text{ kg/tide} \qquad (23.22)$$

* Because the density current component dominates the velocity profile, this current is concentrated in the lower layer of the harbor.

The sedimentations are compared in table 23.3. We see that more than 80% of the harbor siltation is caused by the density current.

TABLE 23.3 HARBOR SEDIMENTATION SUMMARY

Component	Quantity (kg/tide)	Percent of total
Filling Current	1.17×10^5	19.8
Salt Inflow	4.38×10^5	74.2
Salt Outflow	3.53×10^4	6.0
Density Subtotal	4.73×10^5	80.2
Grand Total	5.90×10^5	100

A very pratical question remains for those responsible for the maintenance of the harbor. How much shallower will the harbor become as a result of siltation over the course of one year? This can be answered if the density of the dry sediment particles and that of the in situ sediment are known. Reasonable values for these are 2650 kg/m^3 and 1200 kg/m^3, respectively. Then, if v_v denotes the volume of water filled voids in 1 m^3 of sediment, then:

$$1200 = (2650)(1 - v_v) + (1000)(v_v) \tag{23.23}$$

from which $v_v = 0.88$. Therefore, 1 m^3 of sediment contains

$$(1 - 0.88)(2650) = 318 \text{ kg} \tag{23.24}$$

of dry sediment particles. 5.9×10^5 kg of sediment particles occupies a volume of:

$$\frac{5.90 \times 10^5}{318} = 1855 \text{ m}^3 \tag{23.25}$$

This volume of sediment accumulates in one tide period. There are:

$$\frac{(365.25)(24)}{12.42} = 706 \tag{23.26}$$

tides per year, so that in one year, the accumulation of sediment in the harbor is:

$$(1855)(706) = 1.31 \times 10^6 \text{ m}^3\text{/year!} \tag{23.27}$$

This volume is spread over the harbor bottom in a layer which is:

$$\frac{1.31 \times 10^6}{(2000)(400)} = 1.64 \text{ m} \tag{23.28}$$

thick.

It is usually not economical to dredge out a sediment layer less than about 2.5 m thick. In this case the harbor could be dredged about once every 1½ years.

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This last figure dramatizes the importance of the density current. If the density current could be eliminated in the harbor, then the interval between dredgings could be increased by about a factor 5 (see table 23.3) or to about 7½ years. The economic savings involved are obvious.

As a check, the computations just carried out will be repeated using the second technique of section 23.5.

α_f can be computed from the data using 23.08:

$$\alpha_f = \frac{1.75 \times 10^6}{(6565)(2000)} = 0.133 \tag{23.29}$$

To compute α_D we must first schematize the river salinity curve. We can attempt this by schematizing figure 23.3 for river salinity as being $S = 2.5$ $^o/oo$ from $t = 0$ to $t = 3$ hrs and from $t = 10$ to $t = 12.4$ hrs. From $t = 4.5$ to $t = 7.5$ hrs S is assumed to be equal to $7.5^o/oo$. This yields $T_D = 3$ hrs for increasing harbor salinity and $T_D = 5.4$ hrs for decreasing salinity. (We assume that nothing happens during the rest of the tide period).

Since we are not aiming for high accuracy, values of ρ for computing v_D can be determined using equation 3.22. Thus:

$$\delta = \frac{0.75(7.5 - 2.5)}{1000 + (0.75 \times 2.5)} = 3.74 \times 10^{-3} \tag{23.30}$$

This results in a value of v_D from equation 23.01 with improved coefficient of:

$$v_D = 0.35 \sqrt{(3.74 \times 10^{-3})(9.81)(14.35)} \tag{23.31}$$

$$= 0.254 \text{ m/s} \tag{23.32}$$

$\overline{|V_f|}$ follows using 23.10 with data from table 20.1.

Time interval (hrs)	$\lvert\Delta h\rvert$ (m)	Δt (hrs)
0-3	1.21	3
10-12.4	0.22	2.4
4.5-7.5	0.55	3

Also, from figure 23.7, the area of the harbor is:

$$A_H = (2000)(515) = 10.3 \times 10^5 \text{ m}^2$$

and

$$A_E = 6565 \text{ m}^2$$

Thus, using 23.10 for increasing density:

$$|V_f| = \frac{10.3 \times 10^5}{6565} \times \left(\frac{0.55}{3}\right) \tag{23.33}$$

$$= 28.8 \text{ m/hr} = 8 \times 10^{-3} \text{ m/s.} \tag{23.34}$$

and for decreasing density:

$$|V_f| = \frac{10.3 \times 10^5}{6565} \times \frac{1.43}{5.4} \tag{23.35}$$

$$= 41.5 \text{ m/hr} = 1.15 \times 10^{-2} \text{ m/s} \tag{23.36}$$

The two values of α_D can now be computed using 23.09, in which L = 2000 m. For increasing density:

$$\alpha_D = \frac{(0.254 - 8 \times 10^{-3})(3)(3600)}{(2)(2000)} = 0.664 \tag{23.37}$$

and for decreasing density:

$$\alpha_D = \frac{(0.254 - 1.15 \times 10^{-2})(5.4)(3600)}{(2)(2000)} = 1.179 \tag{23.38}$$

Since one value of α_D is somewhat less than 1, our assumption about the salinity (density) differences is too extreme. Following the suggestion in section 23.5, we can recompute V_D and α_D for a reduced δ. Reducing δ by 50% as suggested yields:

$$V_D = 0.35 \sqrt{(1.87 \times 10^{-3})(9.81)(14.35)} \tag{23.39}$$

$$= 0.180 \text{ m/s} \tag{23.40}$$

Since V_f remains the same, we can proceed directly to the computation of α_D.

For increasing salinity:

$$\alpha_D = \frac{(0.180 - 8 \times 10^{-3})(3)(3600)}{(2)(2000)} = 0.464 \tag{23.41}$$

and for decreasing salinity:

$$\alpha_D = \frac{(0.180 - 1.15 \times 10^{-2})(5.4)(3600)}{(2)(2000)} = 0.819 \tag{23.42}$$

This seems reasonable.

The total harbor volume, V_H, is:

$$V_H = (6565)(2000) = 1.31 \times 10^7 \text{ m}^3 \tag{23.43}$$

Using the same silt concentrations as previously, the silt transport into the harbor by the filling current is:

$$s_f = (0.133)(1.31 \times 10^7)(67)(10^{-3}) = 1.17 \times 10^5 \text{ kg/tide} \tag{23.44}$$

The density current during increasing salinity transports:

$$s_{D_1} = (0.464)(1.31 \times 10^7)(67)(10^{-3}) = 4.07 \times 10^5 \text{ kg/tide} \qquad (23.45)$$

and for decreasing salinity using the lower sediment concentration just as with the previous version:

$$s_{D_2} = (0.819)(1.31 \times 10^7)[(0.2)(77) - 10](10^{-3}) \qquad (23.46)$$

$$= 5.79 \times 10^4 \text{ kg/tide} \qquad (23.47)$$

These values are compared in table 23.4.

TABLE 23.4 Habor Siltation Summary

Component	Quantity (kg/tide)	Percent of total
Filling Current	1.17×10^5	20.1
Salt inflow	4.07×10^5	69.9
Salt outflow	5.79×10^4	10.0
Density Subtotal	4.65×10^5	79.9
Grand total	5.82×10^5	100.0

The remaining problem of determining the amount of siltation is exactly the same as was previously done and will not be repeated here.

The amazing agreement between the results of the two methods should be attributed more to luck than to accuracy of the method.

Methods to eliminate or reduce density current influences in a harbor are discussed in the next section.

23.8 Methods to Combat Density in Harbors

Since it is not necessary to pass a runoff flow through a harbor entrance - in contrast to a river mouth, more technical possibilities are available to reduce the influence of density currents.

One of the simplest methods to reduce the density current water exchange in a given harbor basin is to narrow the entrance. As was shown in equation 23.05, the volume of water exchanged is directly proportional to the entrance area, A_E. Thus, reducing the entrance width should reduce the volume of exchanged water in direct proportion. In practice, such a narrowing will not be quite that effective. The intruding density current stream will spread in both horizontal directions in the wider harbor basin; this tends to increase the effective driving force by increasing the slope of the interface between the water masses. The density current flow will be greater than might otherwise be expected. This effect is difficult to quantify, however.

Another technique is to install a single set of doors at the entrance to the harbor. The harbor level is then maintained at a constant level - even the filling current is eliminated. The water level in the harbor remains constant; this is handy for the cargo handling operations. Does a density current cause a water exchange? It does not have to. If the doors are opened only once during the tide cycle and at the same time in the cycle when the water levels are equal, then the harbor water will eventually have the same density as the river water and no dredgino problems will be experienced. On the other hand this means that the doors are opened only once every tide period and it may be unacceptable to force the shipping to wait so long to pass through the entrance.

What would happen if the doors were opened twice per tide cycle while the water levels were the same - once on a rising tide and once on a falling tide? There will still be no filling current, but there is no quarantee that the density in the river will be the same at both times. In general it will not be, and a density current and water exchange will take place during the time that the doors are open. Indeed, such a solution is of little value except when very great tide level variations might make cargo handling inefficient in an open basin.

If the single set of doors were replaced by a lock, then ships could enter and leave the harbor at any time irrespective of the water levels. Each locking operation can be accompanied by a water exchange within the lock, however. Since the lock is relatively small, this exchange progresses rather rapidly - 27 minutes for the large lock at IJmuiden, for example. Special facilities have been built at IJmuiden to trap this intruding salt water and retain it for later disposal. These special facilities consist of a deep pit just inside the lock connected via an equally deep channel to a sluice. Salt water coming though the inner door opening of the lock falls into the pit. Later, during low tide at sea, this salt water can be discharged through the sluice.

An additional device used at IJmuiden to reduce the salt intrusion is an air bubble curtain. This is a stream of rising air bubbles released from a perforated submerged pipeline at the end of the lock near the door. The rising bubbles increase turbulence and hence mixing. The mixing reduces the driving force of the density tongue and reduces the intrusion.

Such a device could also have been used, of course, in combination with the single set of doors mentioned earlier. Its use for a harbor entrance which is always open is not usually economical, because of power consumption of the air compressors which drive the system.

Other more exotic devices have been proposed from time to time to combat density current intrusions into harbors. For example, a device looking like a giant brush with vertical bouyant rubber bristles fixed to the bottom has been conceived. The bristles bend in order to allow a ship to pass. Many other similar devices can be conceived using a bit of ingenuity.

23.9 Review

Many relationships between phenomona which take place in rivers and harbors have been presented in this and the preceeding three chapters. It is instructive as review to gather together all of the information presented on a single graph sheet. This has been done in figure 23.8. All of the data, with the exception of the tide data for Hook of Holland has been presented earlier in these chapters. The tidal data for Hook of Holland is included for completeness in table 23.5.

TABLE 23.5 TIDAL DATA FOR HOOK OF HOLLAND

Time (hrs)	Water level (m)	Average Current (m/s)
0	-0.53	+0.13
1	-0.21	0.80
2	+0.40	1.17
3	0.80	1.43
4	0.88	0.80
5	0.67	+0.26
6	+0.19	-0.44
7	-0.25	-0.94
8	-0.58	-1.16
9	-0.58	-1.13
10	-0.63	-0.93
11	-0.69	-0.62
12	-0.62	-0.08

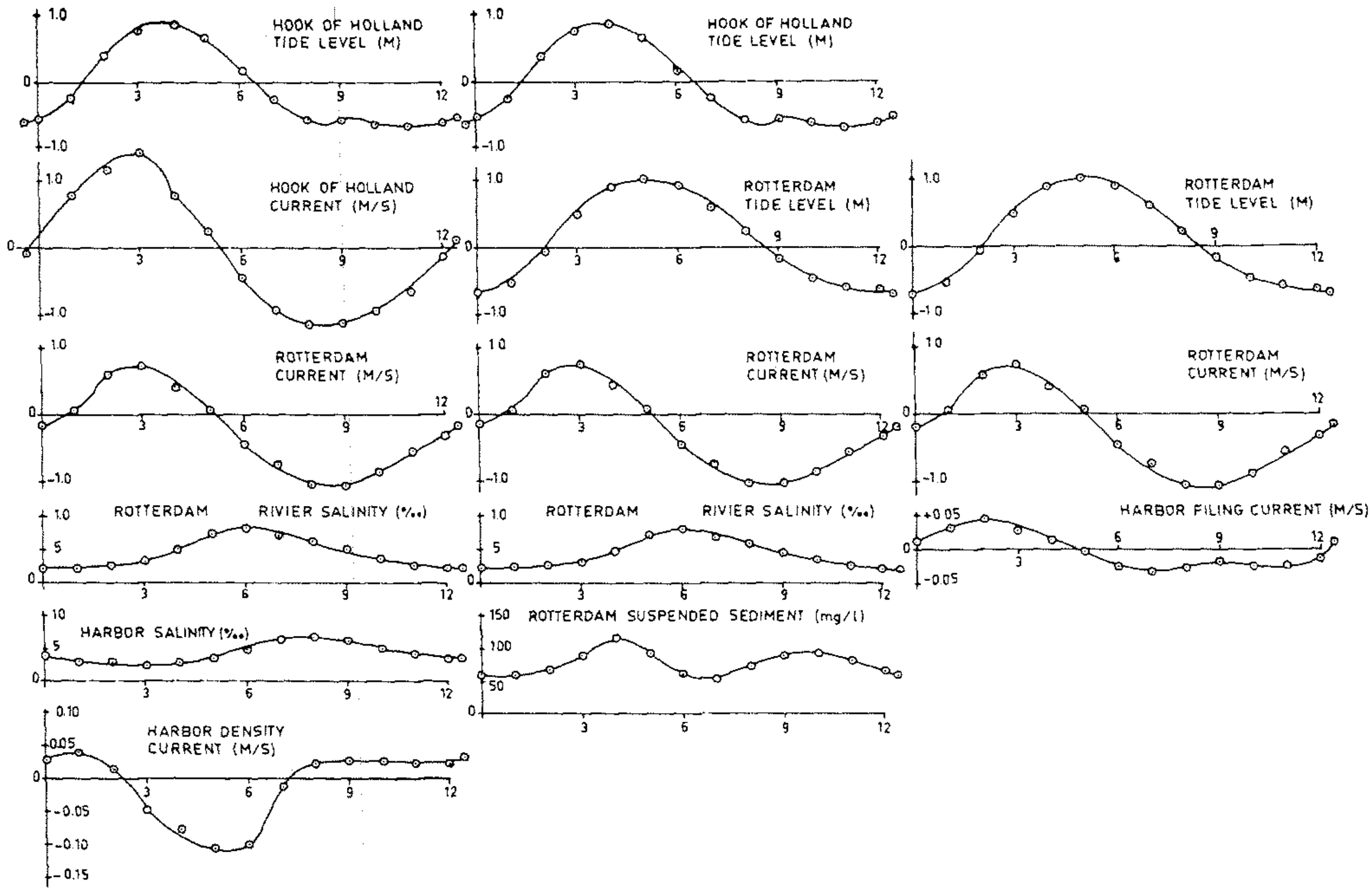

Figure 23.8 SUMMARY OF ALL DATA PRESENTED FOR ROTTERDAM

24 POLLUTION

J. de Nekker
W.W. Massie

24.1 Definition

Pollution is defined by A.F. Spilhaus as "anything animate or inanimate that by its excess reduces the quality of living".* This definition is extremely general; even overpopulation can be seen as a pollution problem under this definition. The important word in the above definition is excess. We often forget that many polluting substances occur and are transported naturally as well as by man.

A more restricted definition is provided by a report to the President and Congress by the U.S. National Water Commission in June, 1973: "Water is polluted if it is not of sufficiently high quality to be suitable for the highest use people wish to make of it at present or in the future".*

This definition for use with regard to water quality is good in that it allows for variations in quality dependent upon water use.

It is the purpose of this chapter to create an awareness of marine pollution problems. Hopefully, an emotional discussion of this topic can be avoided.

The degree to which disagreement can develop is exemplified by two opposing articles which appeared in *Civil Engineering* - Gould (1973) and Thomas (1974). Both quote factual information and neither, really contradicts the other except on matters purely based upon opinion.

24.2 Polluting Materials

The materials causing marine pollution can be grouped into seven main categories. These are: human wastes, oil, halogenated hydrocarbons, other organic materials, heavy metals, heat, and radioactive materials. Each of these is described a bit below.(Dredging spoil material has been discussed separately in chapter 17.).

Human fecal waste is often first considered, since it raises such a great aesthetic problem - people do not like to see or smell it. On the other hand, it is certainly a natural product and fecal wastes are also produced in great quantity by marine life. Six million tons of anchovies off the California (U.S.A.) coast produce as much fecal material as 90 million people, according to Bascom (1974-1). Two aspects of the disposal of fecal wastes remain important, however: Fecal wastes can consume oxygen from the water and these wastes contain bacteria. The oxygen demand can lower the disolved oxygen level below that needed by marine life. While most bacteria are killed soon by contact with sea water (within hours), it is not sure that this is true for all types, thus epidemiological problems can be conceived.

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* quoted by Bascom (1974-1)

Oil and petroleum products are perhaps the most controversial pollutants. The public reaction to oil spills by ships is usually emotional and vehement. Shipping is not the only source of marine oil pollution, however. Unknown quantities of it seep naturally into the oceans. A report compiled for the Connecticut (U.S.A.) State Legislature concludes that more than two thirds of the oil discharged by man into the seas comes from the crankcases of automobile engines and oil sumps of other machines.* This oil causes no great problems, however, since its rate of input is low enough and it is sufficiently dispersed to be broken down by natural processes.

Oil pollution from major spills is usually a local and often temporary problem. The short term biological and esthetic influences can be severe, but the pre-existing natural situation usually restores it self without the intervention of man within a few years. This is not true of the next category of pollutants.

Halogenated hydrocarbons include the most common organic pesticides. While a few of these chemicals, such as TEPP lose their lethal properties rather quickly, others such as DDT seem to be virtually undestructable in nature. The process of concentration of pesticides in certain types of marine life is rather well known. Because of their indestructability, disposal of these types of materials should be especially carefully controlled.

The adverse effects of the discharge of nutrients into restricted bodies of water such as lakes are well documented elsewhere. On the other hand, the effect of such nutrients on the ocean can be beneficial. According to Isaacs, "The sea is *starved* of the basic plant nutrients, and it is a mystery to me why we should be concerned with their thoughtful introduction into coastal seas in any quantity that man can generate in the forseeable future." **
With proper management, nutrients may be successfully disposed of along the coasts. A by-product of this disposal would be the stimulation of marine life and thus of the fish life dependent upon these plants. This artificial nutrition amounts to the simulation of an upwelling responsible for the proserous fishing industry in certain parts of the world (Japan, for example).

Since oxygen is consumed in the biodegradation of the nutrient materials, their discharge must be managed in such a way that disolved oxygen levels do not become too low to support fish life.

Heavy metals such as copper and zinc are naturally present in sea water and in bottom sediments. Low concentrations of certain of these elements are beneficial or even essential for a number of organisms. For example, copper is an essential nutritional element for crabs.

Marine sediments contain much higher concentrations of heavy metals than sea water (Table 24.1). Heavy metals, ionicly bound to sediment particles tend to go into solution in sea water as the ionic constituents of sea water itself disturb the physical chemistry of the sediment

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* Reported in *Scientific American*, Vol. 228, no. 2, Feb. 1973 page 48.

** quoted in Bascom (1974-1)

particles - see section 7 of chapter 22. Table 24.1 compares heavy metal concentrations in sediments and sea water. Heavy metals also enter the sea from the atmosphere. Forest fires, for example, add metalic oxides to the atmosphere which deposits them over the whole world.

Just as with discharges of many pesticides, the influence of heavy metal discharges is cumulative. The indiscriminate addition of heavy metals to the sea should be avoided. An example of the cumulative action as influenced by man is shown in figure 24.1, which shows the lead concentration in layers of sediment in the ocean near Long Beach, California, U.S.A. The sharp rise in concentration in recent years is attributed to airborne lead from automotive emissions.

TABLE 24.1 Concentrations of Heavy Metals in sea water and sediments.

Element	Concentrations in parts per million at stated locations					
	Sea water Average	Ocean Sed. California upper 10 cm	Europoort silt	Botlet to Emshaven silt	Waalhaven to Rijnhaven silt	Rhine River Silt
Cadmium	$1x10^{-4}$	0.3	2.7	19.	36.	45.
Chromium	$4.5x10^{-4}$	42.	185.	435.	870.	1240.
Cobalt	$4x10^{-4}$	7.				
Copper	$3x10^{-3}$	16.	55.	250.	450.	600.
Lead	$3x10^{-5}$	8.	96.	304.	545.	800.
Manganese	$1.8x10^{-3}$	290.				
Mercury	$2x10^{-4}$	0.04				
Nickel	$6.6x10^{-3}$	13.				
Silver	$3x10^{-4}$	1.				
Zinc	0.01	32.	350.	1300.	2150.	2900.

Data from: Bascom (1974-1), and
de Nekker & In't Veld (1975)

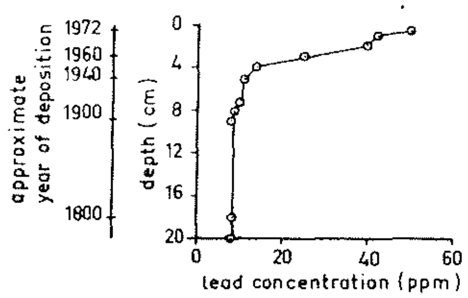

Figure 24.1 LEAD CONCENTRATION IN SEDIMENT BASCOM (1974-1)

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Thermal discharges may be either warmer (power station cooling water) or cooler (liquified natural gas conversion) than the surrounding water. Most marine life can adapt to the modified thermal climate near such a heat source or sink, but are often killed either mechanically or as a result of abrupt temperature and pressure changes as they are drawn through the plant. Heat discharged into the oceans is only of local biological significance. It might well be combined in the future with the discharge of nutrients to stimulate marine life for the benefit of man.

Radioactive wastes form the seventh category of pollutants. Since water forms relatively good shielding radioactive moderator, disposal of such wastes at sea can seem attractive. The direct danger to marine life is less than that to man because marine life can tolerate a larger radiation dose before it becomes fatal - van Staveren (1974). Such a reasoning can be dangerous, however, since man can conceivably ingest a fatal dose of radio poisons from seemingly health fish.

24.3 Control Measures

The most common control measures are legal sanctions applied against those who cause pollution. Starbird (1972) describes the success possible with a river. When rivers cross international boundaries, their cleanup can only be accomplished by all of the bordering nations working together. The attempts to clean up the Rhine exemplefy the frustrations; the British have had much more success with the Thames.

Legal restrictions must be realistic, however. Pollution reduction levels must be attainable and consistent with other standards. Bascom (1974-2) points out an example of unrealistic restriction. "In Los Angeles (U.S.A.), the level of 'pollutants', such as arsenic and copper, in municipal drinking water is higher than can be legally discharged into the ocean." This is an example of a ridiculously strong restriction.

On a more humorous level, a common joke in yacht clubs in the U.S. in the early seventies was that anti-pollution laws were becoming so strict that diapers would soon be required on seagulls.

24.4 Proposed Disposal Systems

It has already been alluded that the oceans can be an ideal disposal place for some wastes. The most promising are heat and nutrients which could be harnessed in the sea to increase food production via fish farming.

Whether or not other wastes are dumped into the oceans, they must be disposed of in some way. Some people propose disposal of the more undesirable wastes by dumping in the deep sea. The consequences of such actions deserve careful study; unexpected things can happen as is reported by Jannasch and Wiersen, cited in *Scientific American* (1973). It appears that biochemical decay processes are greatly retarded by pressure. Thus, "since natural re-cycling processes are nearly at a stand still in deep water, these areas are not appropriate dumping grounds for organic wastes".

The above quotation is in contrast with the proposal of Bostrom and Sherif, also reported in *Scientific American* (1972). They propose compacting and then disposing of all sorts of wastes in ocean regions called subduction sinks. At these locations - usually deep ocean trenches - crust material is being drawn down to the earth's mantle. They indicate that all of man's wastes amount to less than 1/250 of the volume of material drawn into the mantle. They admit that there are several details yet to be worked out for this plan, however.

One of the more important of these problems is that this large volume of material is drawn downward over a large area at a very low rate - millimeters per decade. Thus, any unit of waste material deposited on one of these subduction sinks will remain exposed for such a long time that containment of the waste remains a serious problem.

25. Beach Profile

L.E. van Loo

25.1 Introduction

The cross sectional form of a beach (profile) is strongly influenced by the wave action along the coast. More factors help to determine the actual form of the profile. These factors include:

material properties such as density, and erosion resistance,
particle size and shape,
wave and current conditions, and
coastal geography and bathymetry.

By popular definition, a beach can refer to an entire coastal area having a bottom consisting of particles ranging in size from sand to gravel. The technical definition is a bit more stringent; a beach extends from the coast outward to the normal low tide level, as shown in the sketch in figure 25.1. This sketch shows a typical profile for a sandy beach. In this figure, the beach is subdivided into a backshore and a foreshore. The boundary between these zones is at the crest of the berm - the point of maximum wave run-up under normal wave conditions. The wave run-up reaches the boundary between beach and coast only during the more severe storms.

The surf or breaker zone extends from the point where waves first break to the point of maximum wave run-up. A bar is usually formed near the outer edge of the breaker zone.

The average beach slope is largely dependent upon the grain form and size of the beach material. Coarse, irregularly shaped particles form the steepest slopes. Gravel or shingle beaches can be stable with a slope as high as 1:4 (vertical:horizontal). Sandy beaches usually have slopes ranging between 1:25 and 1:150. Silt and mud shores are usually even flatter having slopes as low as 1:5000. Since mud shores exhibit some special characteristics, they are discussed separately in chapter 27; attention will remain on sandy beaches in this chapter.

In the next sections we discuss first the dynamic equilibrium of a beach under normal wave conditions, and later, the effect of storms on this equilibrium.

25.2 Beach Dynamic Equilibrium

During "ordinary" weather conditions (no storm waves) a beach is in a state of dynamic equilibrium. Considerable quantities of sand are moving over the beach profile, but there is little net gain or loss of material at any location.

As waves break either by spilling or plunging (the only types of breakers commonly found along beaches - see chapter 8), their energy is dissipated largely in turbulence. Sand grains are stirred from the bottom and held temporarily in suspension by this turbulence. A portion of the mass of water of the former wave crest rushes toward the shore in the upper layers of the breaker zone carrying this sand with it. This water dissipates its remaining energy by running up on the beach. Some of this run-up water returns to the sea by percolation through the beach, most flows back along the beach surface. Since the backflow is less turbulent, less sand is transported back in the offshore direction than was brought in; the foreshore builds up slowly during this calm weather. The return flow of water and sand continues along the bottom to the bar at the outer edge of the breaker zone completing the flow circuit.

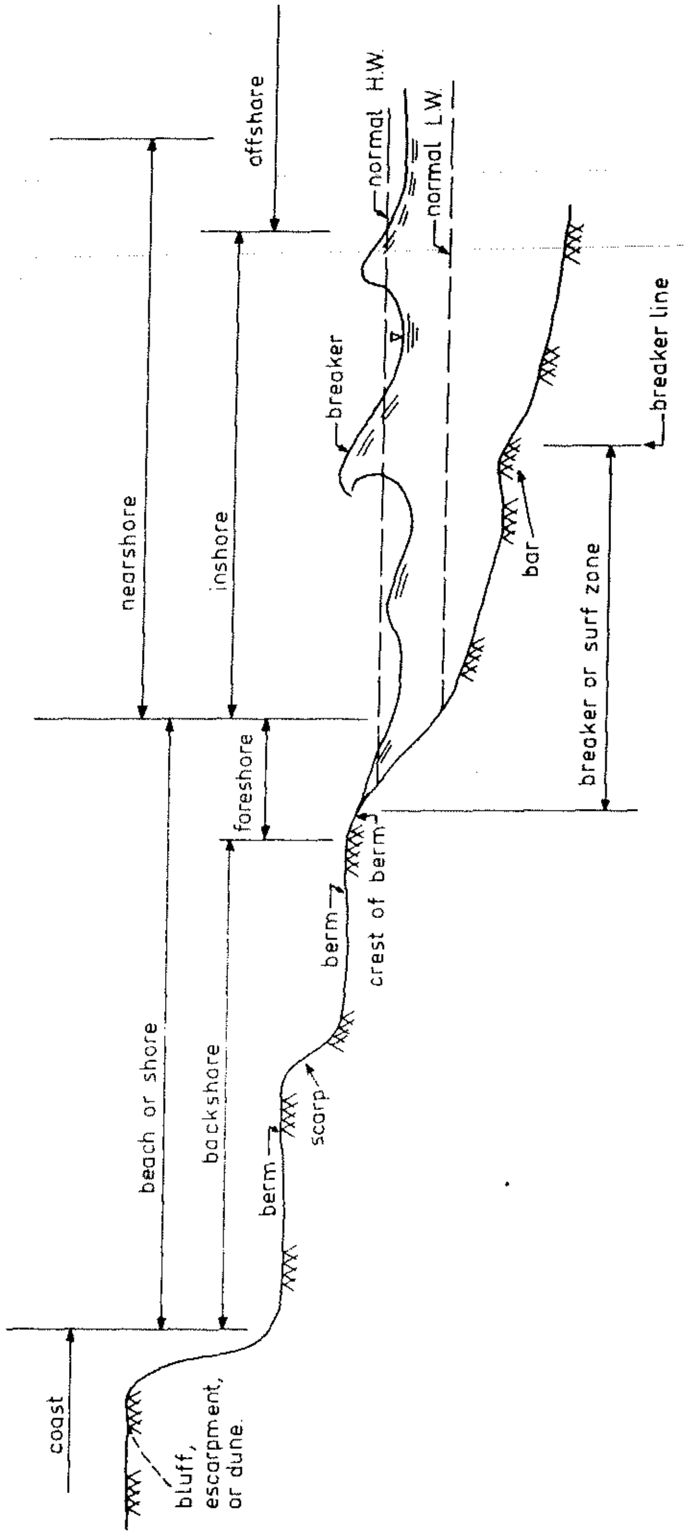

Figure 25.1 SANDY BEACH PROFILE NOMENCLATURE (scales are distorted)

Outside the surf zone, the wave action usually causes a small sand transport toward the coast. This small supply of sand to the bar compensates for the sand deposited on the foreshore, thus maintaining the equilibrium of the bar. Figure 25.2 shows this schematically. The erosion of the material offshore from the bar is a slow process spread over a large area, so that the loss of material in this zone has no further consequences for the beach stability. This loss is compensated during storms as indicate in section 4 of this chapter.

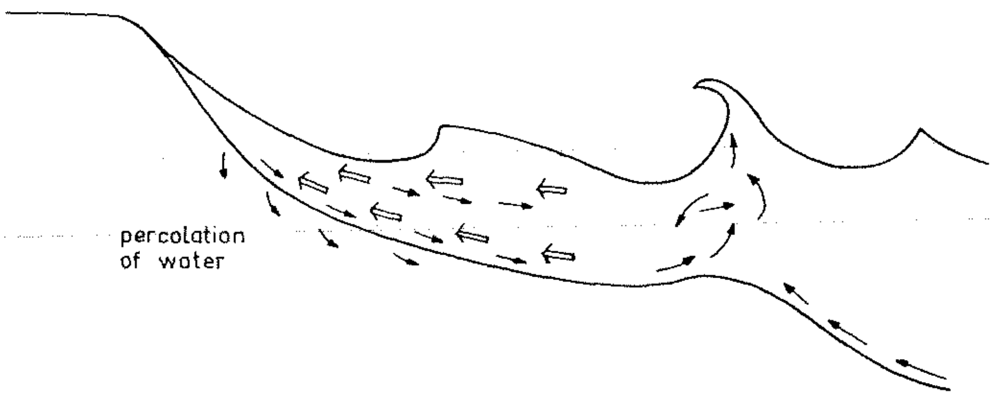

Figure 25.2 MOVEMENTS OF SAND AND WATER DURING CALM WEATHER IN THE BREAKER ZONE
⇐ movement under crest
→ movement under trough
sketch not to scale

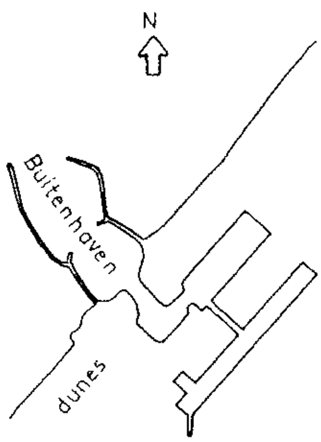

Figure 25.3
PLAN OF SCHEVENINGEN

25.3 Dunes

If the water level varies in time such as happens when a vertical tide is present, the upper portions of the foreshore can be built up during high tide, but become dry during low tide. An onshore wind can then transport this dry sand further inland to the backshore or even further to the coast forming dunes.

Such transport by wind can have unexpected influence on harbor entrances built through a row of dunes, for example. Slijkhuis (1974) reports that in Schevingen sand transported from the dunes south of the harbor by wind must be dredged from time to time from the entrance channel between the buitenhaven and the voorhaven. (figure 25.3). This happens even through the dunes in The Netherlands are rather well stabilized by vegatation. This is not always the case, however. Figure 25.4 shows some much larger, less domesticated dunes along the coast of Oregon, U.S.A. The scale may be appreciated by noting the group of people near the center of the picture. The dunes are encroaching on the wooded area to the left in the photo. The Pacific Ocean is in the distance.

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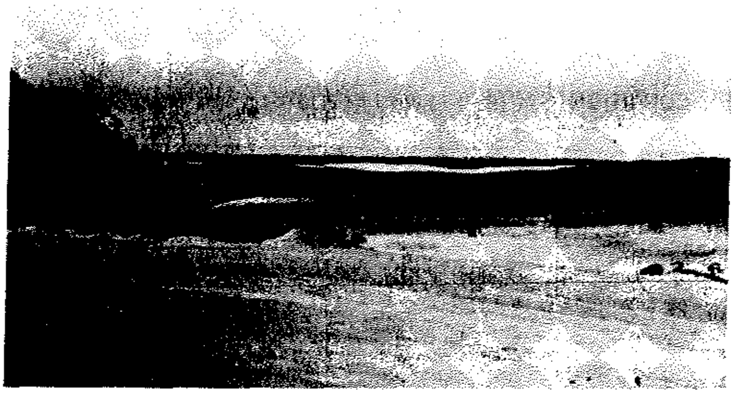

Figure 25.4
DUNES ALONG THE COAST OF OREGON, U.S.A

25.4 Influence of Storms

One might expect that the process just described is only amplified during a storm; more things happen however. Not only the waves are higher during a storm, but the still water level is usually higher as well due to wave and wind set-up. (See chapter 3 of this book and chapter11 of volume II). If this set-up is high enough (a few meters is not exceptional) the wave run-up can reach the coast line and attack the dunes or bluff. Material is transported from the coast and beach to the nearshore zone as shown in figure 25.5. The dune attack can result in a crest lowering and a crest recession. If sufficient material is present in the dunes, the sea will not destroy them completely and the calm water profile will be slowly restored after the storm. If the storm has been exceptionally severe, this natural restoration process can take some years.

If, on the other hand, insufficient material is available in the dunes, then a series of storms occurring within a few months can break through the dunes having a disasterous effect on the hinterland.

In addition to this transport of sand along a coast profile (perpendicular to the coast line), waves can also move sand along the shore parallel to the coast. This is further explaned in the following chapter.

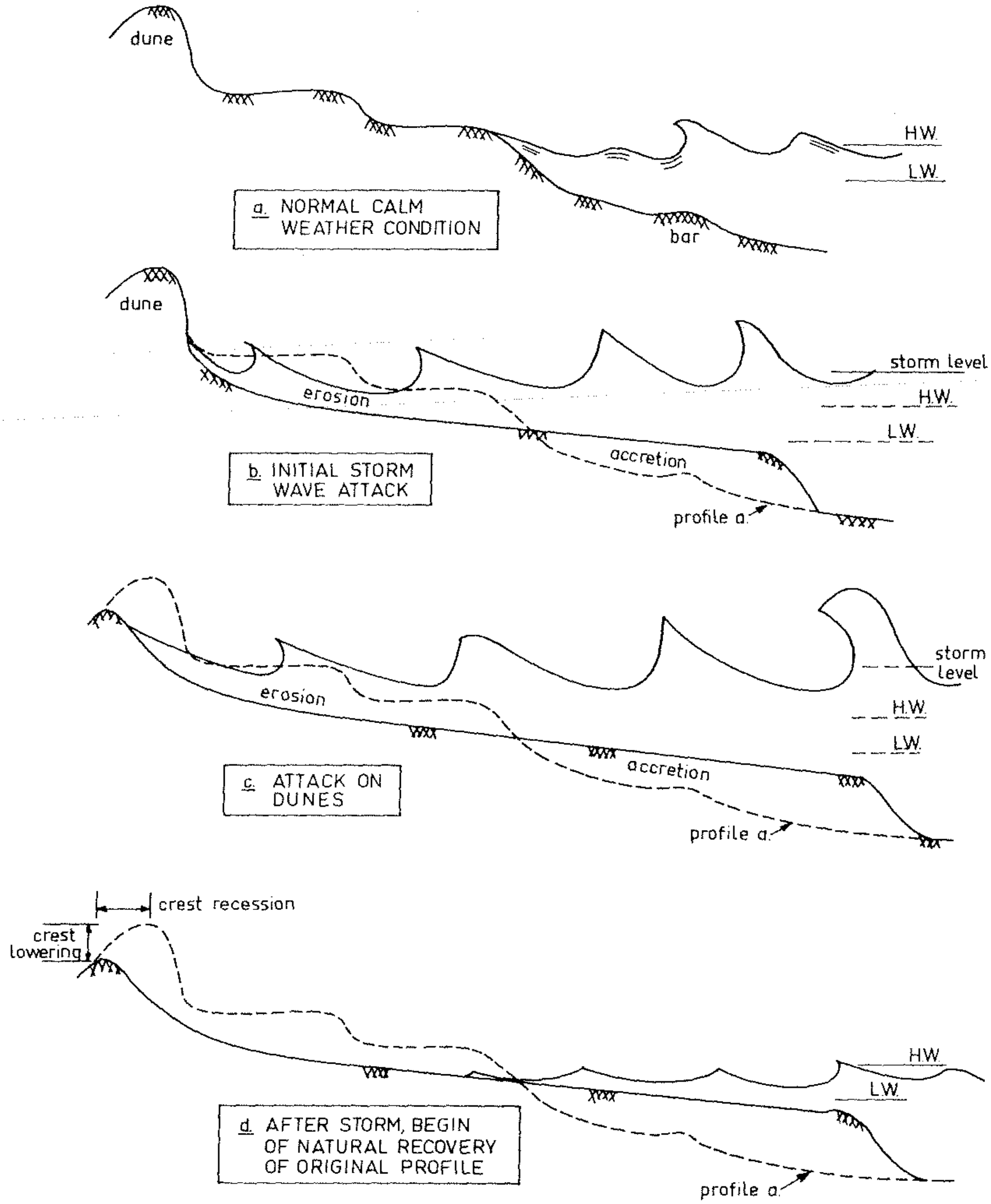

Figure 25.5
BEACH DEVELOPMENT DURING STORM

26. SEDIMENT TRANSPORT ALONG COASTS

E.W. Bijker

26.1 Definitions

A transport of bottom material parallel to a sandy coast (longshore transport, or littoral drift) occurs whenever a current component is present parallel to the coast (longshore current). In general this current can be the resultant of several influences such as permanent ocean currents, tides, obliquely approaching waves, or in some cases, river currents.

26.2 The CERC Formula

Since obliquely approaching waves are usually the most important cause of a longshore current - and hence, a longshore transport - the most common simple model relates the longshore transport to the wave properties. All of the other influences are neglected. Under these conditions, the longshore current is concentrated in the breaker zone and the wave conditions will be determined at the outer edge of this zone - the breaker line.

The wave energy flux (power per unit crest length) in deep water can be expressed as

$$f(H_0^2, c_0) \tag{26.01}$$

where:
c_0 is the wave speed in deep water,
H_0 is the wave height in deep water, and
f( ) denotes some function.

This can be reviewed in chapter 5.

We, however, are interested in the energy flux crossing a unit length of the breaker line, the energy supply to the transport process. This energy flux entering the breaker zone per unit length of coastline can be expressed as:

$$f(H_0^2, c_0, K_{rbr}, \cos \phi_{br}) \tag{26.02}$$

where:
K_{rbr} is the refraction coefficient at the breaker line,
ϕ_{br} is the angle of wave incidence at the breaker line, and
f( ) again denotes some function.

The component of this flux parallel to the coast can be expressed in terms of:

$$f(H_0^2, c_0, K_{rbr}, \cos \phi_{br}, \sin \phi_{br}) \tag{26.03}$$

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Some years ago, the U.S. Coastal Engineering Reseach Center (CERC) studied a great quantity of model and prototype measurements and determined the following best-fit formula:

$$S = 0.014\ H_o^2\ c_o\ K_{rbr}\ \sin\phi_{br}\ \cos\phi_{br} \qquad (26.04)$$

where:

S is the total littoral transport in the entire breaker zone, and
0.014 is a dimensionless coefficient.

If we wish to express S in units of length^3/year, and leave the units of c_o in length/sec, then the equation takes the form:

$$S = 0.44 \times 10^6\ H_o^2\ c_o\ K_{rbr}\ \sin\phi_{br}\ \cos\phi_{br} \qquad (26.05)$$

For a real beach and real waves, the wave height will not be constant but will vary according to the Rayleigh Distribution - chapter 10. For such conditions the root mean square wave height characterizing the record should be used, since this wave represents the energy correctly. Equations 26.04 and 26.05 (often called the CERC Formula) give not only the quantity but also the direction of the longshore transport.

Galvin (1972) examined the data in another way and came to the following formula:

$$S' = 1.65 \times 10^6\ H_o^2 \qquad (26.06)$$

where:

S' is the total* transport in m^3/yr., and
H_o is the deep water wave height in m.

This equation is not dimensionally consistent. Galvin's implied conclusion is that the total yearly net transport along a coast is independent of the wave direction or period.

Of these two formulas, the CERC formula is to be preferred. Its background is rather straightforward, and it is even reasonably reliable in use. Even so, it does have some disadvantages and weaknesses. The first weakness has already been mentioned; effects of all driving forces except waves are neglected.

A second limitation is that the sand transport is independent of the sand properties such as grain size and density. Also, the beach slope, and hence the type of breakers, is ignored. This has happened because the observed data which led to the CERC Formula were made on sandy beaches having more or less the same properties. The accuracy of the data was not sufficient to allow inclusion of these variables in the computational model.

A last major limitation is that only the total sand transport in the breaker zone is given. It is often handy to know how this transport is distributed over the width of the breaker zone. Bijker and Svasek

* This total is the sum of the absolute values of the transports in each direction.

(1969) solved this by assuming that the longshore transport in some element of breaker zone width is directly proportional to the wave energy dissipation - or better, energy transformation - within that width element. This assumption exposes another basic objection to the CERC Formula (and all other energy - based formulas, too): Only a few percent of available energy is actually used to transport the sediment along the coast. Small changes in this percentage can result in large changes in sand transport; this is not healthy for the formula.

Another, entirely different approach has been developed in an effort to overcome the limitations mentioned above. This approach is described briefly in the next section.

26.3 The Bijker Formula

Bijker (1967) proposed that the combined effect of all of the possible force components mentioned in section 26.1 be determined and that the longshore current and littoral transport be determined based upon this. The modern development of this idea is outlined very briefly here and is described in detail in volume II.

The influence of waves on the longshore current along a coast manifests itself via the gradient of the longshore component of momentum flux of the waves. This is explained in detail in volume II. In principle, this gradient of the longshore component of momentum flux, combined with other driving fluxes resulting from tides, etc., provides the during driving force acting on a water mass. In a steady state condition, these forces are balanced by a bed friction force acting on the longshore current as it is disturbed by the waves. The development of this equilibrium is discussed in detail in volume II as well.

This technique allows all driving force components to be included in the longshore current determination, and its velocity distribution within the breaker zone is also revealed. With the details of the current known, it is a reasonably simple matter to combine this with a sediment transport formula in order to obtain a littoral drift prediction. Obviously, the distribution of this transport of sand is also revealed using such a method.

26.4 Sediment Transport Along the Profile

In addition to the sediment transport parallel to the coast, there is also movement of water and sediment perpendicular to the coast. This sand transport component, often called on and offshore transport has been indicated in the previous chapter.

Bakker (1968) attempted to quantify this transport by postulating that it is proportional to the difference between the actual beach slope and the equilibrium beach slope corresponding to the given wave and current conditions.

Swart (1974) proved this hypothesis and quantified it further. This development is the subject of a chapter in volume II.

This approach to the on and offshore movement of material is only very crude. A major research effort is now being devoted to the study of the water and sediment motions within the breaker zone in an attempt to determine

27. MUD COASTS

E. Allersma

27.1 Physical Description

Mud coasts occur near the mouths of rivers which discharge great quantities of fine (clay) sediments into the sea. This supply must be greater than the capacity for the sea to disperse these materials toward deeper water offshore.

A pure mud coast is very low - seldom higher than a spring high tide level. Since the slope of the shore profile is very flat - 1:1000 is not uncommon - vast mud flats are found before the coast. Accretion, if present, leads to the formation of a broad, flat, poorly drained, swampy coastal plain. Vegetation eventually forms a layer of peat on the surface. These coastal plains can become very fertile agricultural land if drainage and protection from flooding are provided.

Some sand is also present along most mud coasts. Since the transport process for sand is basically different from that for mud (see section 2 of this chapter) the two materials tend to segregate. The sand can be found as isolated segments of beach, perhaps even with a berm and dunes as described in chapter 25. These berms and dunes are sometimes found isolated in the coastal plain - evidence of past coastal development.

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Clay particles (smaller than 2 μm) are generally transported in suspension. Waves and strong currents can bring compacted clay into suspension, while only a very weak current is necessary to maintain this suspension. Flocculation can accelerate the deposition process (see ch. 22 section 7). Deposition of flocculated clay forms a very soft material commonly called sling mud. Its classification based upon silt concentration is shown in table 27.1. For convenience, some of the data is plotted in figure 27.1.

As has already been pointed out in chapter 22, sling mud, a viscous water-clay mixture, can occur in large quantities along the coast and in channels. Ships can, with care, sail through it. It may even clean the barnacles off the ship's bottom! Its viscosity damps waves rapidly; even surface water waves are "absorbed" by it. Given enough time, this mud will eventually consolidate into a very soft soil.

Sand, on the other hand, is primarily transported as bed load rather than suspended load. This explains its tendency to segregate along a coast. Transport processes for sandy portions of mixed mud-sand coasts are the same as have been described in the previous two chapters. If much sand is present, it can form a nearly continuous layer covering a clay subsoil. Such a formation can be found, for example, in the Gulf of Venezuela.

Table 27.1 Properties of Sling Mud

Concentration of solids (mg/l)	Mass density (kg/m^3)	Water Content by volume (%)	Water Content by Weight (%)	Material classification (-)
0	1000.00	100.00	100.00	
100	1000.06			
200	1000.12			
500	1000.31			
1 000	1000.62	99.96	99.90	
2 000	1001.25			
5 000	1003.11			
10 000	1006.23	99.62	99.01	suspended silt
20 000	1012.45			
50 000	1031.13			
100 000	1062.26	96.23	90.59	
200 000	1124.53	92.45	82.21	
300 000	1186.79	88.68	74.72	
400 000	1249.06	84.91	67.98	sling mud
500 000	1311.32	81.83	61.87	
600 000	1373.58	77.36	56.32	
700 000	1435.85	73.58	51.25	
800 000	1498.11	69.81	46.60	
900 000	1560.38	66.04	42.32	
1 000 000	1622.64	62.26	38.37	clay
2 000 000	2245.28	24.53	10.92	
2 650 000	2650.00	0.00	0.00	

27.3 Influence of Rivers

Rivers and estuaries discharging on a mud coast are characterized by deep (about 20 m) tidal estuaries which discharge over shallow bars at their mouths where the maximum depth is usually only 3 to 5 m.

Both the bars and the estuary are important for determining the sediment transport. Tidal currents, fresh water river discharge and its seasonal variations, as well as density currents are important. Generally, sediment is transported from the bars into the estuary during the dry season as the density tongue penetrates further upstream. As the river flow increases at the start of the high runoff season, these sediments are again spewed out of the estuary along with the sediment discharge of the upper river. This can lead to very rapid accretion in the bar area with all of its subsequent dredging and navigation problems.

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This approach to the on and offshore movement of material is only very crude. A major research effort is now being devoted to the study of the water and sediment motions within the breaker zone in an attempt to determine a better formulation of the equations describing the sediment movements in the breaker zone.

The general research philosophy parallels that used succesfully for th longshore transport: describe the current and combine it with an appropriat sediment transport formula. Further development of this is postponed for volume II.

26.5 Computation of Coastal Changes

The sediment transport formulas indicated above can be used as a basis from which it is possible to compute changes in a coastline.

In a most simple model coastal erosion or accretion involves only a horizontal displacement of the equilibrium profile of the beach. (There is no net on or offshore transport). Figure 26.1 shows the actual and schemati meaning of such an assumption. In this figure, the actual and schematized profiles at some time are indicated by the solid lines. The accretion after some interval of time is shown by the dashed lines.

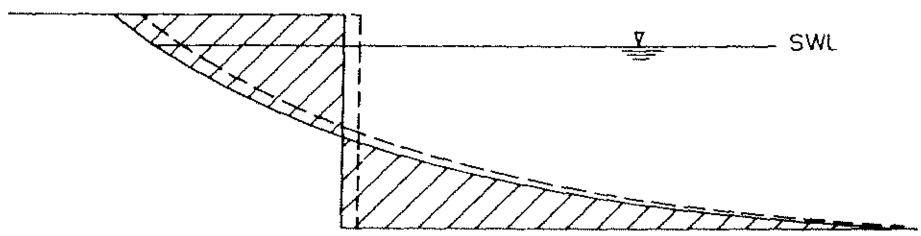
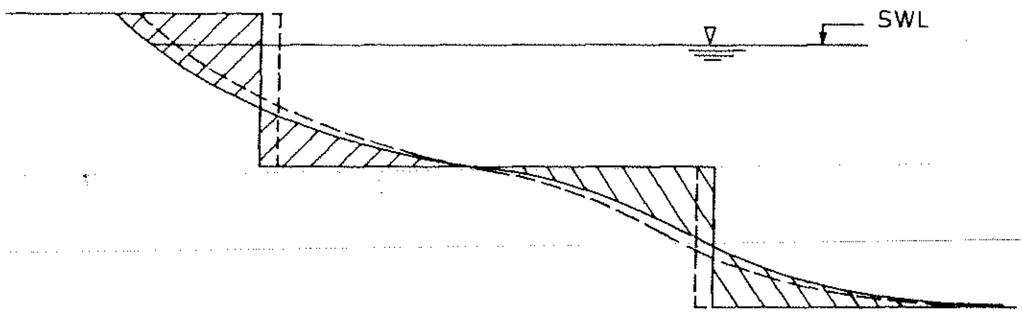


Figure 26.1 BEACH PROFILE AND ITS SCHEMATIZATION. THE SHADED AREAS ARE EQUAL (no scale)

When a continuity relationship is combined with a longshore transport formula, a second order partial differential equation for the coastline can be determined. This equation, its boundary conditions and solution are discussed further in volume II.

The beach profile does not always remain unchanged during accretion or erosion. In such a case a schematization as shown in figure 26.2 can be used. Again, changes occuring during some time interval have been shown with a dashed line; each set of shaded areas is equal. In the diagram, the upper portion of the beach is accreting while the lower portion is eroding.

This schematization results in two, coupled second order partial differential equations - one for each schematization line. Separate boundary conditions can be specified for each equation. This could be done, for example, if the transport along the higher portion of the beach is interrupted by a groin. Indeed, Bakker, Klein Breteler, and Roos (1970) developed this method to study coastal changes along just such groin protected shores. The coupling between the differential equations comes via the on and offshore sediment transport discussed earlier.



26.2 TWO LINE SCHEMATIZATION (no scale)

Coastal formations such as deltas and spits result from pseudo-equilibrium of the forces acting on the coastal materials. These formations and the explanation for their shapes are given in chapters 28 and 29. First, we discuss the morphology of mud coasts in the following chapter.

27. MUD COASTS

E. Allersma

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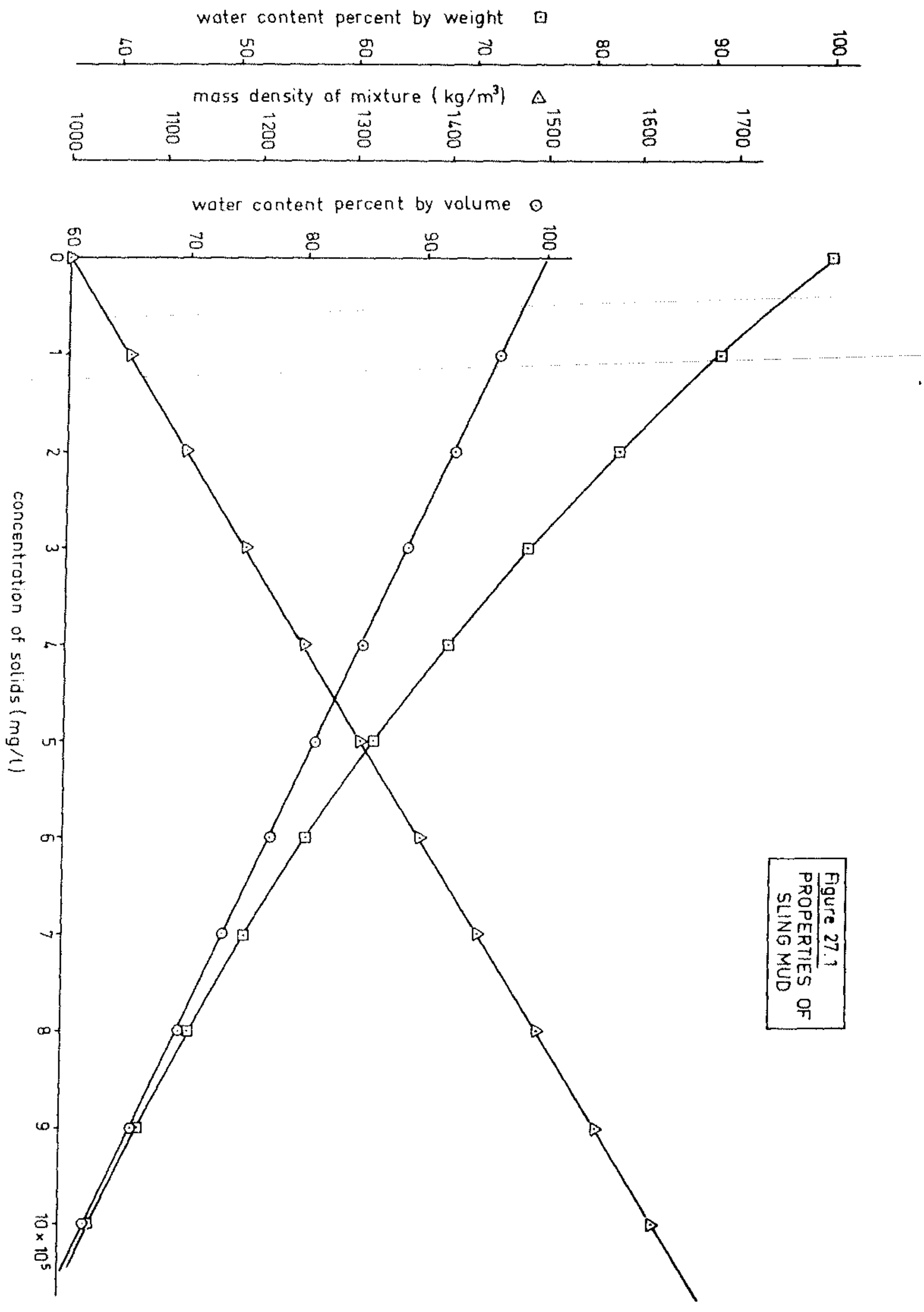

Figure 27.1
PROPERTIES OF
SLING MUD

27.4 Examples

Mud coasts can be found in many parts of the world, examples include:

a. The coast of Languedoc, west of the mouth of the Rhône in southern France.
b. The coast of Louisiana, west of the Mississippi River Delta on the Gulf Coast of the U.S.A. A chart of this delta is shown in chapter 29.
c. The northern coast of the Gulf of Siam, near the mouth of the chao Praya River in Thailand.
d. The coast of the Gulf of Martaban with sediment supplied by the Irrawaddy, Sittang, and Salween Rivers in Burma.
e. The northeast of South America, between the mouths of the Amazon and Orinoco Rivers. Some aspects of the coast of Suriname, a portion of this 1600 km coast, are so special they are discussed in the following section.

27.5 The Coast of Suriname

The coast of Suriname is unique in that an enormous supply of sediment from the Amazon River is distributed by a relatively calm sea. This description is abstracted from Allersma (1968).

The flow in the Amazon is in the order of 200 000 m^3/s (compared to the Rhine - 2200 m^3/s). The sediment supply is in the order of 7×10^8 tons per year. Along the coast of Suriname, 98% of the bottom material has a diameter less than 50 μm, with a mean diameter of about 1 μm. The total transport of sediment toward the west along this coast is estimated to be 100×10^6 tons per year. The mud coast extends to a depth of about 20 meters and is about 30 km wide with an average slope of about 1:1500.

This coast has a remarkable pattern of wave - like depth contours. Huge shoals extend from the shore at more-or-less regular intervals of about 45 km - figure 27.2A. Bottom slopes on these shoals range from 1:500 to 1:3000. The shoals move along the coast toward the west with a speed of about 1.5 km/year. This is accomplished by erosion of material from the eastern side of the shoals combined with deposition on the western side. About 100×10^6 m^3 of sediment move along the coast in this way each year. This erosion and deposition is shown in figure 27.2C. The influence of waves on this process is evidenced by comparing figures 27.2C and 27.2D. Deposition generally can be associated with areas of lower wave height - indicated by more widely spaced wave orthogonals (See chapter 9).

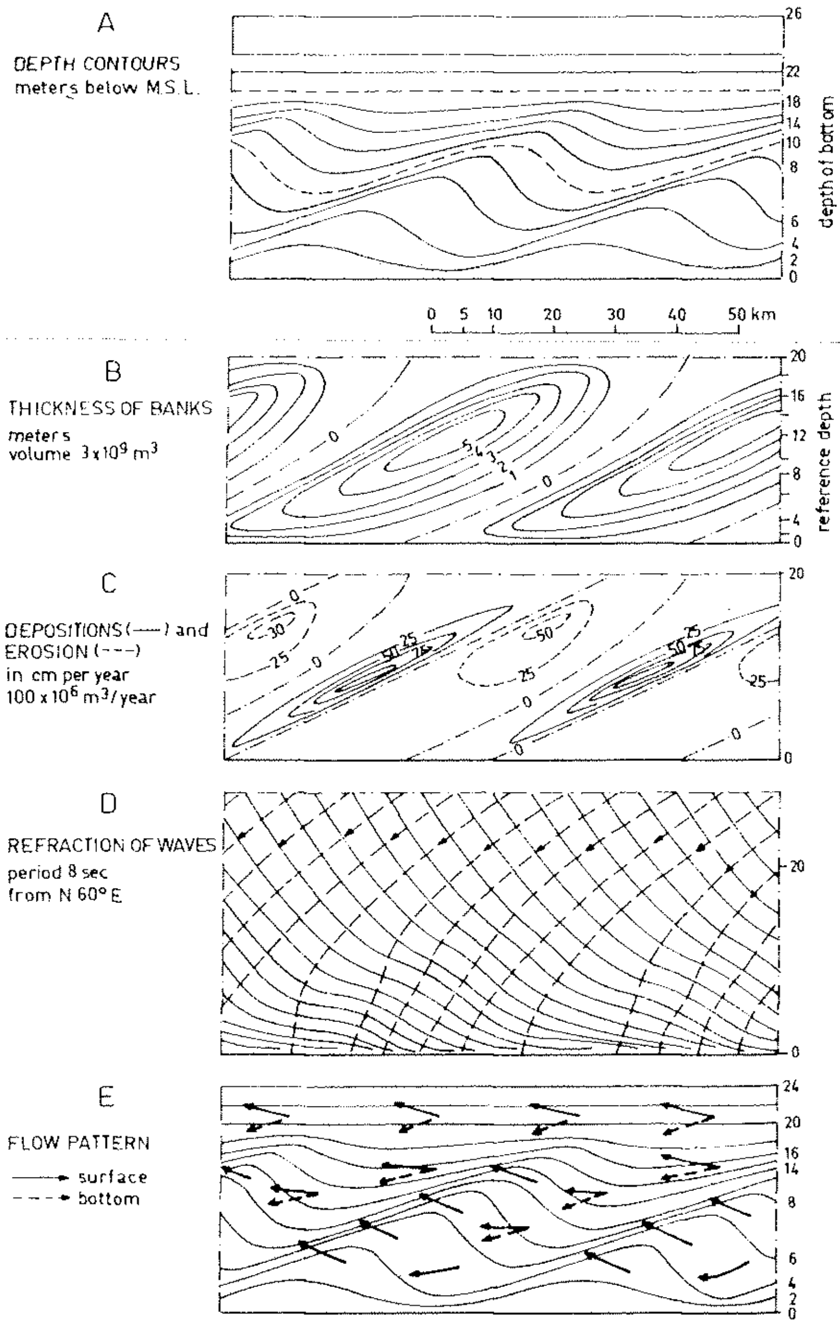

Figure 27.2 SCHEME OF FEATURES ABOUT MUD SHOALS

28. COASTAL FORMATIONS

L.E. van Loo
W.W. Massie

28.1 Introduction

The purpose of this and the following chapter is to illustrate the various coastal formations found in the world and to explain the reasons for their existence. Ideas developed in previous chapters describing the water and sediment movements in rivers and along coasts will now be combined to provide the necessary explanations.

Several additional photos and descriptions of coastal formations illustrated in this and the following chapter are included in the *Shore Protection Manual*. In addition, Shepard and Wanless (1971) provide a large collection of spectacular photos along with descriptions of the physical processes.

Also, much can be learned from a careful study of seamen's navigation charts. Portions of such charts are used in these chapters to illustrate many of the coastal formations.

One additional principle remains to be explained, however. Consider an infinitely long, straight sandy coast having parallel depth contours. Such a coast was sketched in chapters 9, figure 9.1. If waves approach this coast at a uniform angle along its entire length, and there are no other current driving forces such as tides, then there will be a constant, uniform transport of sand along this coast. There will be no erosion or deposition even though a continuous flow of sand passes along the coast. What, then, causes erosion or deposition? This is caused by a *change* in transport or transport capacity along a coast. This change may result from changing any of the factors influencing sand transport, such as wave height and direction of approach - see chapter 26.

Natural, continuous beaches will not get our main attention in this chapter even though some of the formations to be discussed will look - at first glance - like long, uninteresting beaches. Beaches can develop even along rocky coasts, however. Figure 28.1 shows such a beach, nestled between rock outcrops on the eastern shore of the Adriatic Sea in southern Yugoslavia.

28.2 Spit

A spit is a pointed tongue extending into the sea. Its direction is usually a continuation of the shoreline from which sediment is supplied. Such a spit is shown in figure 28.2a, the north end of Block Island on the Atlantic Coast of the United States. Waves coming predominantly from the southwest cause a sand transport toward the north along the western shore of the island. As the water becomes deeper at the north end of the island, the waves no longer break, the sediment transport decreases and the spit builds out further. Figure 28.2b shows an oblique aerial photograph of this island, showing the spit.

Sandy Hook, near the entrance to New York Harbor (U.S.A.) is also a spit; they are rather common.

Spits can also form where a river mouth interrupts an otherwise straight coast. This will be discussed in the following chapter.

Figure 28.1
A NATURAL BEACH ON THE ADRIATIC COAST OF YUGOSLAVIA

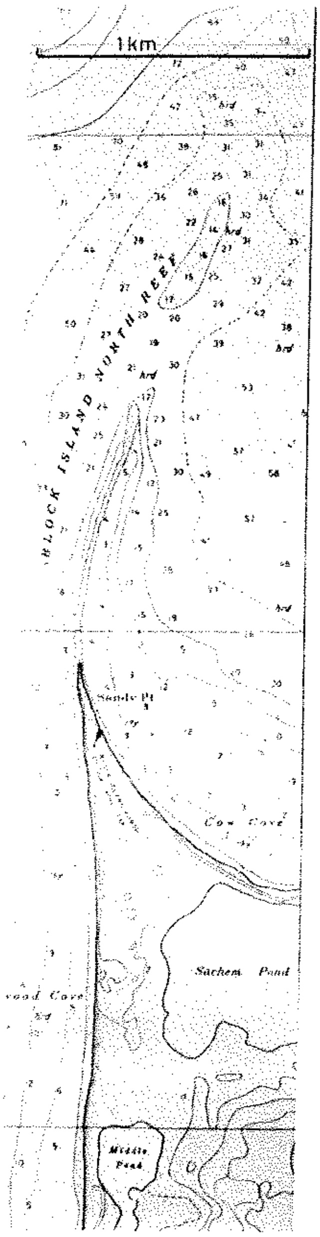

Figure 28.2a
SPIT AT NORTH END OF BLOCK ISLAND, R.I., U.S.A.
(depths in feet, scale as shown)

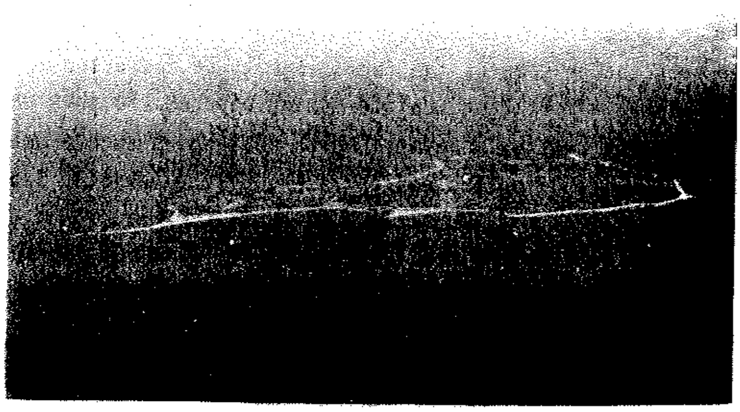

Figure 28.2b
AERIAL PHOTO OF BLOCK ISLAND

28.3 Barrier

In contrast to a spit which is formed from material moving along the coast, barriers are built from material moving perpendicular to the coast - review chapter 25.

Barriers can form when there is sufficient supply of beach material from offshore, and the bottom bathymetry is such that the waves break at some distance from the coast, because of a broad shallow foreshore zone. A barrier will form at the outer edge of this shallow zone where the waves break; the supply of sand will eventually build up a berm - isolated from shore - which becomes the barrier. Storm waves can break over this barrier transporting sand into the shallows behind it. Severe storms can even break gaps in the barrier. If the variations in tide level are sufficient for the berm to become dry, then the wind can also transport material forming dunes along the barrier.

An excellent example of a barrier which has been broken is the string of Wadden Islands in the north of the Netherlands. (figure 28.3). A much more extensive nearly continuous barrier is found along the nortwest coast of the Gulf of Mexico - figure 28.4. Cape Hattaras, North Carolina, on the U.S. Atlantic coast is another example of a barrier. When a barrier completely encloses an estuary, a salt or brackish lake usually results. Figure 28.5 shows such a barrier along the south coast of Martha's Vinyard Island on the U.S. Atlantic coast.

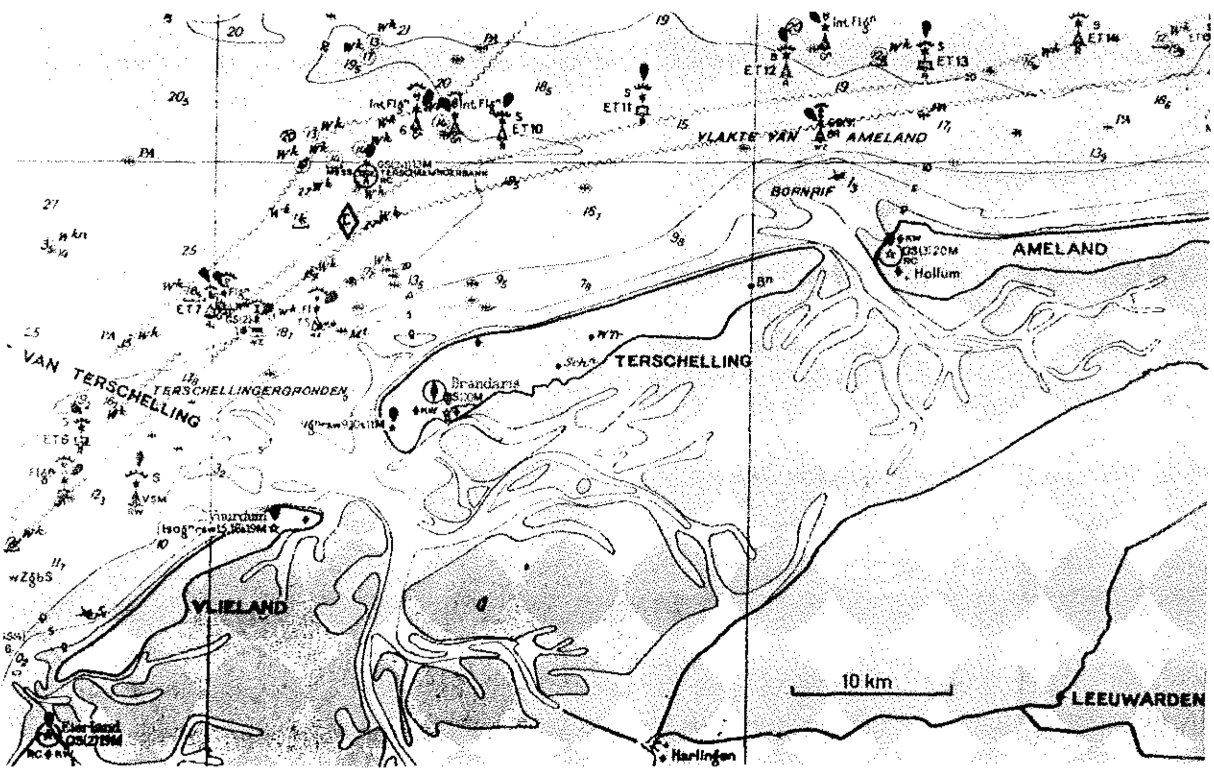

Figure 28.3
PORTION OF WADDEN SEA AND ISLANDS, FRIESLAND, THE NETHERLANDS
(scale as shown, depths in meters)

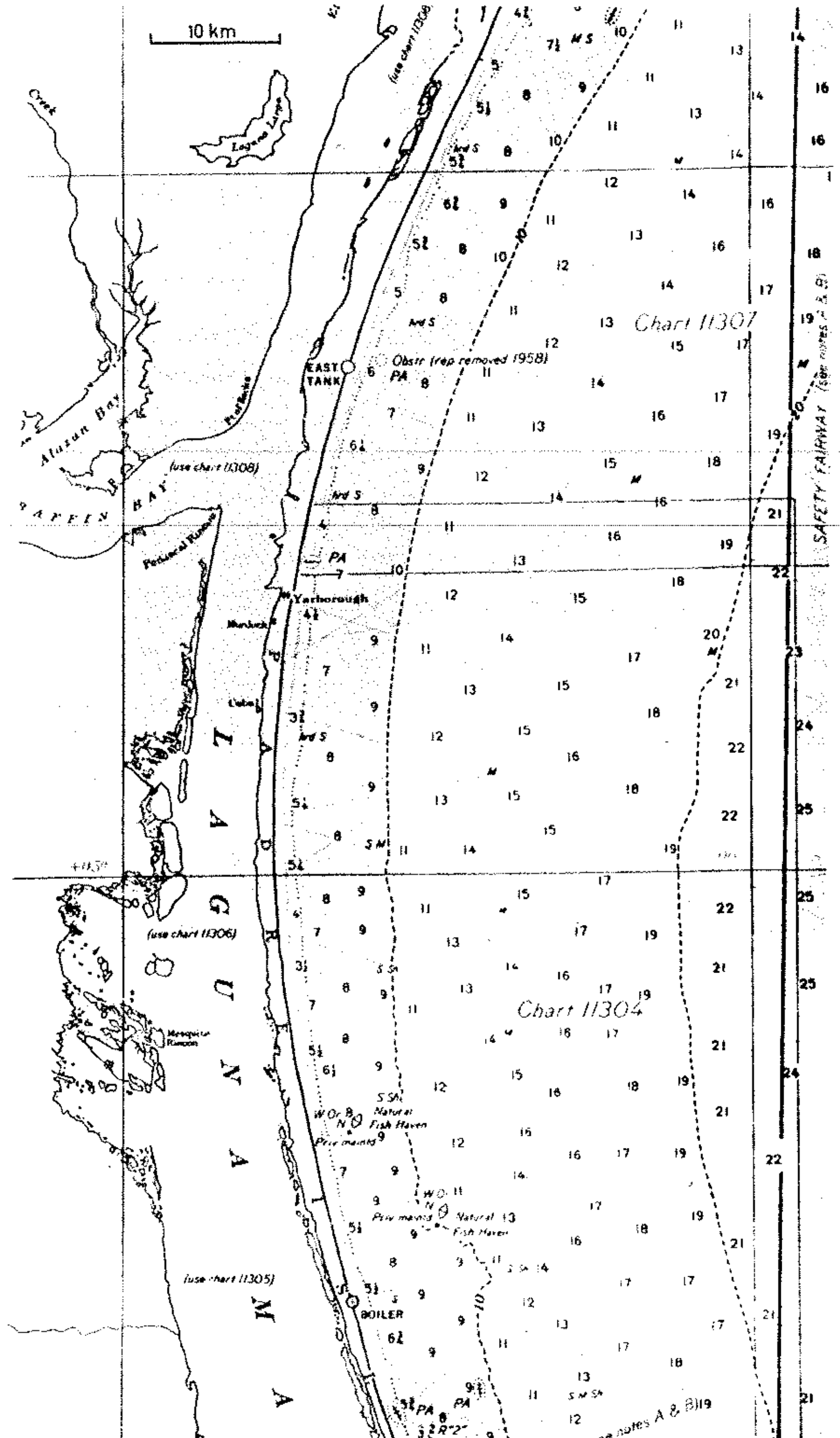

Figure 28.4

BARRIER ALONG THE COAST OF TEXAS, U.S.A.
(scale as shown, depths in fathoms)

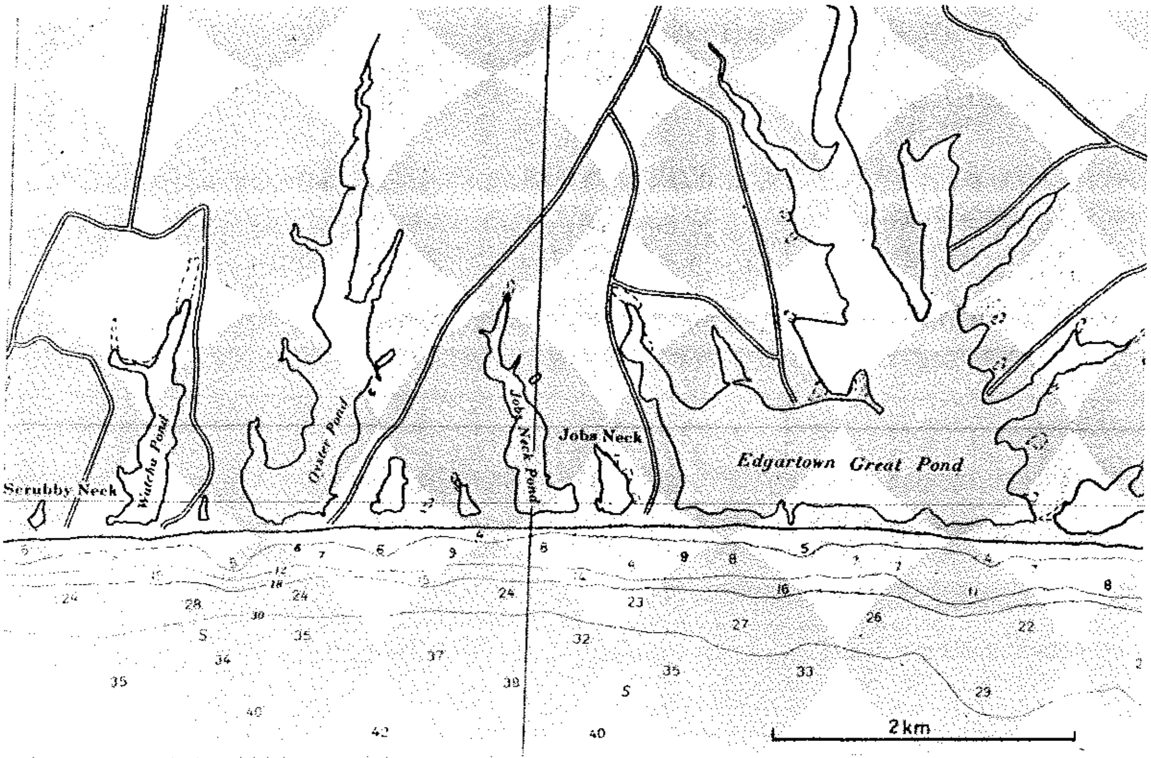

Figure 28.5
BARRIER ENCLOSING POND ON MARTHA'S VINYARD IS.
(scale as shown, depths in feet)

28.4 Tombolo

A obstacle before a coast such as a rock outcropping, an off-shore breakwater, or even a shipwreck will reduce the wave activity in the zone of wave shadow between the object and the shore. Since the reduced wave activity in the shadow will result in a reduced sediment transport capacity, material being carried along the shore will be deposited in the shadow zone forming a tombolo. Initially, only a shoal will form. This can, however, develop into a point of land connecting the original shoreline to the obstable. As with a spit, the development of a tombolo depends upon a transport of material parallel to the coast.

A small natural tombolo has developed behind Ram Island in Buzzards Bay on the North Atlantic coast of the United States. This area is protected from wave attack except from the south. This tombolo is shown in figure 28.6.

Figure 28.7a shows the start of a tombolo formation behind a series of offshore breakwaters. These were built, in this case, to stimulate and preserve a recreational beach. Figure 28.7b shows an aerial photo of this area, near Boston, on the U.S. Northeast coast The pattern of wave diffraction is especially prominent in this photo.

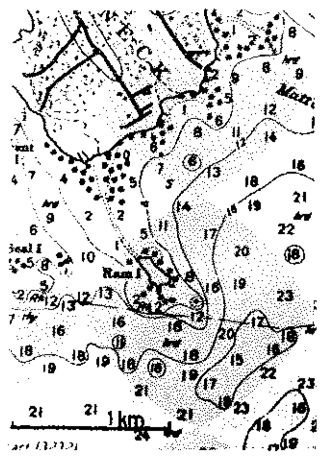

Figure 28.6
TOMBOLO BETWEEN RAM ISLAND AND MATTAPOISETT NECK, BUZZARDS BAY, USA
(scale as shown, depths in feet)

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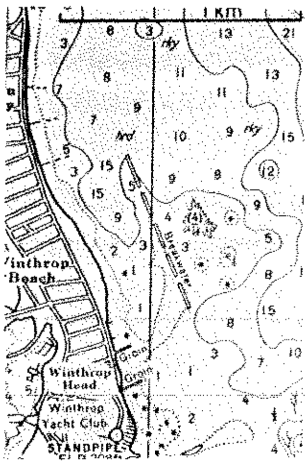


a. PLAN
(scale as shown, depths in feet)

b. OBLIQUE AERIAL PHOTO

Figure 28.7
WINTHROP BEACH, MASSACHUSETTS, U.S.A.

29. DELTAS

L.E. van Loo
W.W. Massie

29.1 Introduction

A delta develops around a location where an estuary discharges water and sediment through a coast. Because rivers transport more sediment, the most spectacular deltas develop near river mouths rather than at other forms of estuaries.

Several factors contribute to the form of a delta. Among these are,

Tide and fresh water runoff currents in the estuary,

Estuary sediment discharge and properties,

Coastal waves and currents,

Coastal sediment material transport and properties, and

Water level variations in the sea and estuary.

Most of these factors can be combined in a qualitative descriptive factor: the ratio between the supply of river sediment and the distribution capacity of the coastal processes. It seems most logical to start the discussion with an estuary discharging sediment and water into a body of still water; the distribution capacity of the coast is zero.

29.2 Deltas on Quiet Coasts

Consider a river discharging a constant flow Q_r and a constant sediment load S_r through an initially straight coastline. There are no waves, and only a river current. Such a situation is shown in figure 29.1a.

As the discharge passes the mouth, the flow will spread out, reducing the current strength and hence the sediment transport capacity. Material will be deposited in areas where the current is weakest - at the sides of the discharge. Shoals, which will eventually come above water level, will develop projecting into the sea. These can be compared to spits described in the previous chapter except that in this case the supply of material now comes from the river.
Such a development is shown in figure 29.1b.

This cannot go on forever, however. The consequences of such a formation are a topic for river engineering. The most important consequence for the delta is that the water level at the location of the orginal mouth becomes higher. Eventually, this will cause too high an hydraulic gradient across the spit and the river will break through, forming a new mouth - see figure 29.1c.

This process, of course, repeats itself. Figure 29.2 shows a natural example of such a delta - the Lyêna River delta on the north coast of Siberia, U.S.S.R. Both Érgye - Muóra - Sîssye Island and Bărkin Island are parts of this delta.

Figure 29.3 shows a detail of a portion of the Mississippi River Delta on the Gulf of Mexico coast of the U.S.A. The pattern of repeated shoal breaks is evident. Only isolated portions of this delta remain dry during storm surges.
Such delta formations are often called "birdfoot deltas" because of their form. This especially obvious in figure 29.4 which shows a major portion of the Mississippi River Delta.

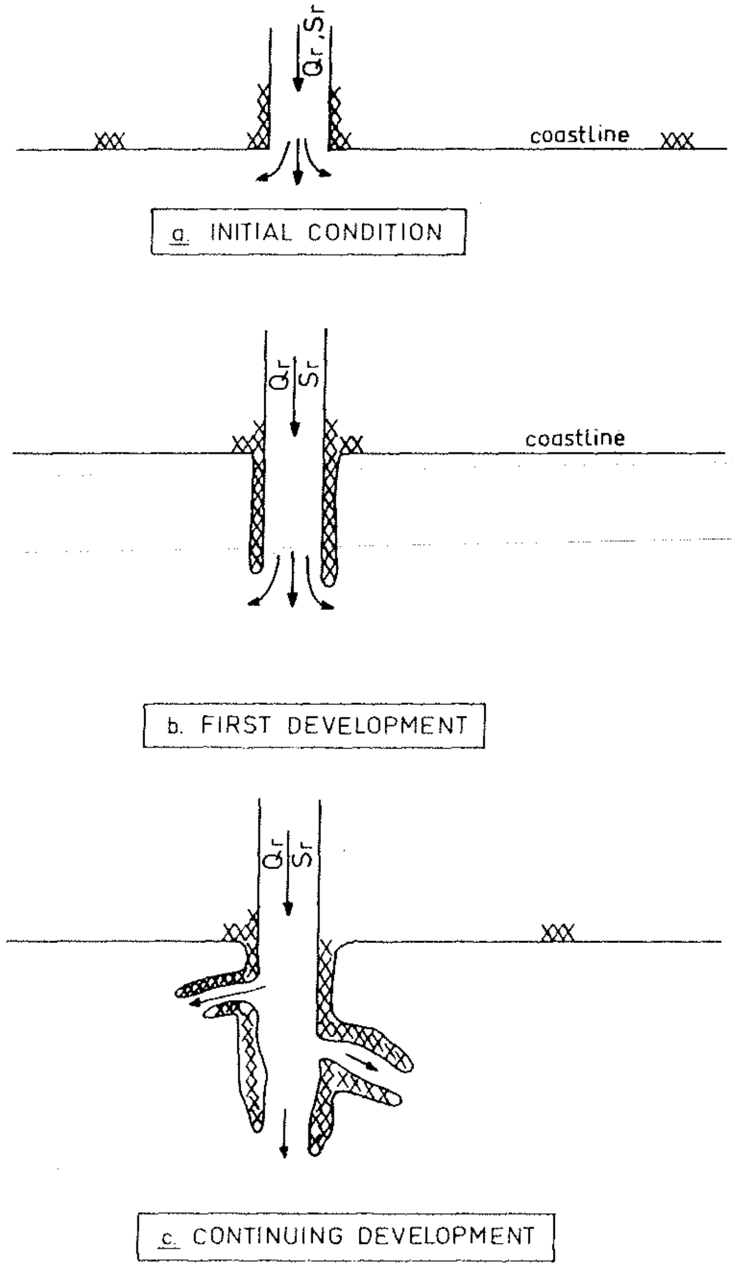


Figure 29.1
DEVELOPMENT OF DELTA IN ABSENCE OF WAVE
(no scale)

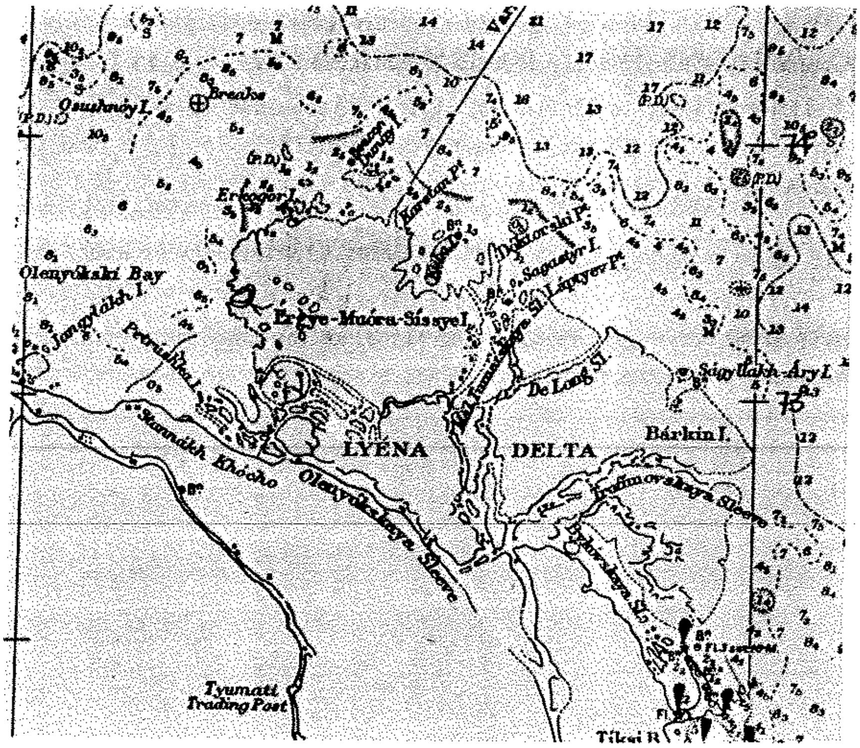


Figure 29.2
LYÉNA RIVER DELTA, SIBERIA, U.S.S.R.
(scale as shown; depths in fathoms and feet)

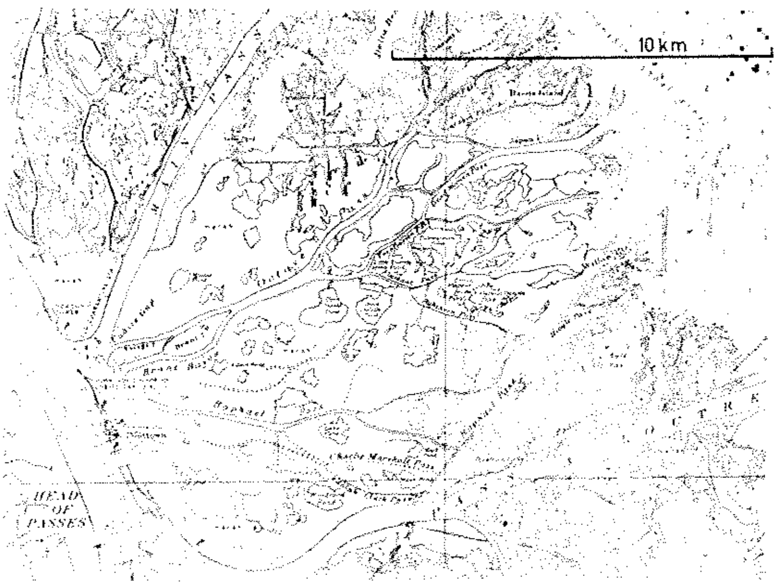


Figure 29.3
DETAIL OF MISSISSIPPI RIVER DELTA, LOUISIANA, U.S.A.
(scale as shown, depths in feet)

Figure 29.4
MISSISSIPPI RIVER DELTA REGION, LOUISANA U.S.A.
(scale as shown depths in fathoms and feet)

29.3 Deltas with Moderate Distributing Influences

If waves were to attack one of the deltas just mentioned, what would be their effect on the delta form? For simplicity, let us assume that the dominant wave propagation direction is perpendicular to the original straight coast, thus, there will be no sand transport along the original coast. Why? - see chapter 26!

The wave action will obviously attack the ends of the projecting shoals shown in figure 29.1b. Material from the shoals is transported toward and along the coast. Figure 29.5 shows the development with and without wave action.

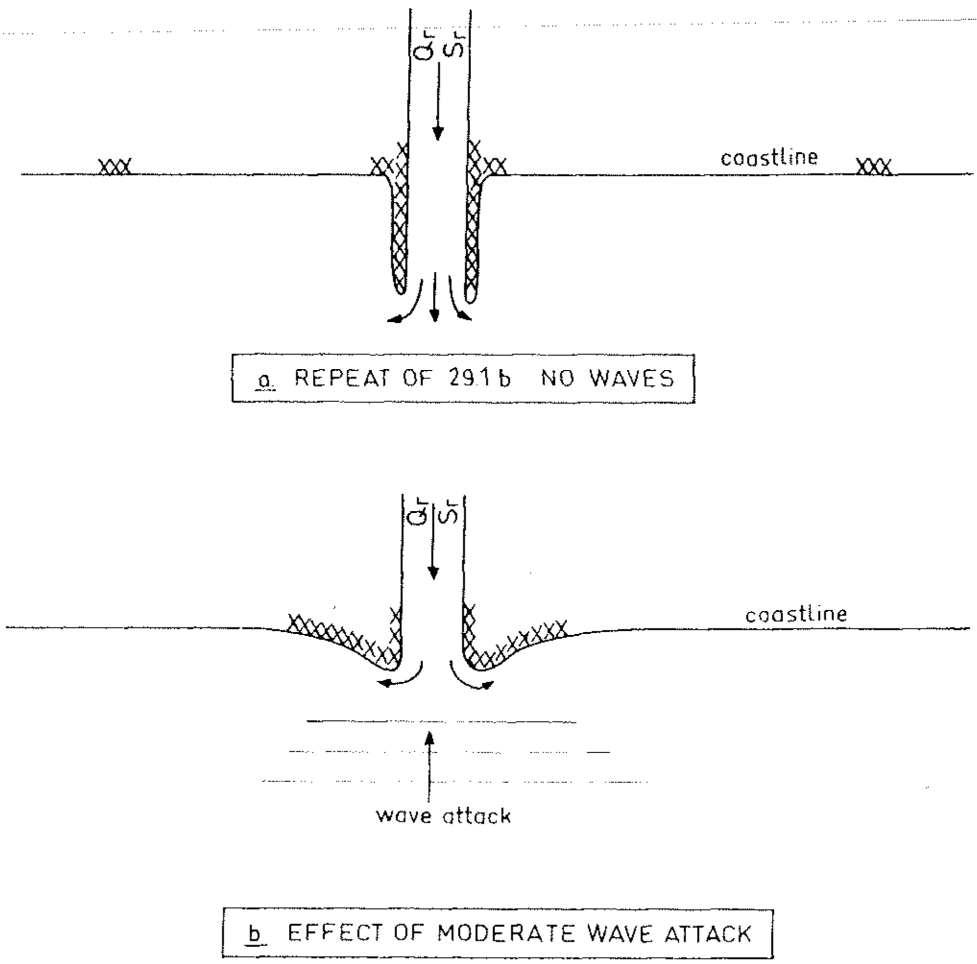


Figure 29.5
DELTA WITH MODERATE WAVE ATTACK
(no scale)

The form of the delta can be explained by determining the sediment transport at each location along the coast, remembering to include refraction influences where the waves no longer approach parallel to the new coast. Figure 29.6 shows an actual example of such a delta portion - the Rosetta Mouth of the Nile on the Mediterranean Sea coast of Egypt. In this example, the river supply of sediment is relatively greater than that shown in figure 29.5; this results in a more pointed delta. This figure, by the way, is only a small portion of the entire delta; just as with a birdfoot delta, many discharge branches can still develop although they tend to be more limited in number than when no waves are present.

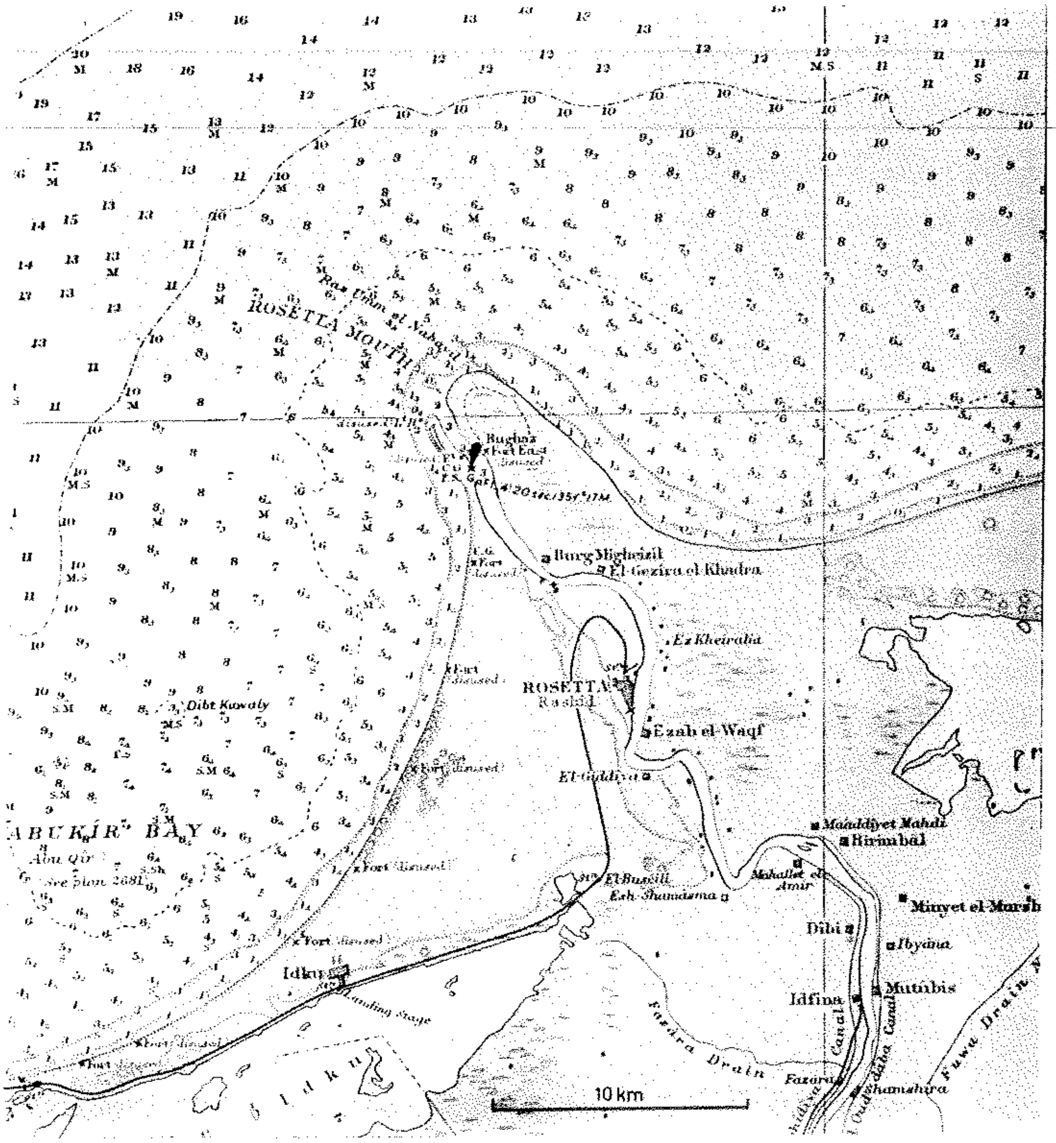


Figure 29.6
PORTION OF NILE DELTA, EGYPT
(scale as shown, depths in fathoms and feet)

Another, ideal example of such a delta is that of the Niger River in Nigeria. Here, waves approach from the southwest with crests parallel to the coast near the mouths of the Sengana River (figure 29.7).

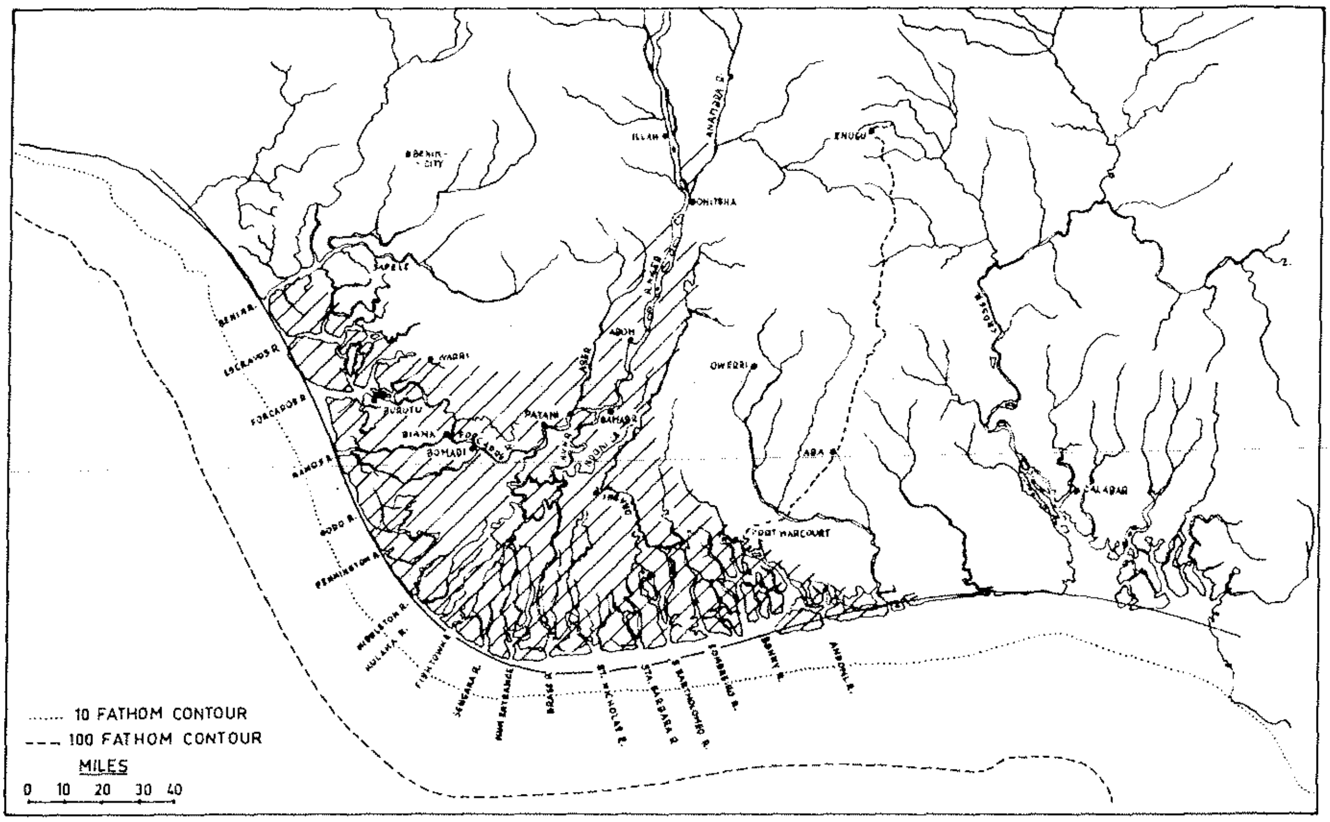


Figure 29.7 NIGER RIVER DELTA, NIGERIA

The maximum sand transport then occurs near the inflection points of the shore line. Material is supplied by the river through the mouths between these two points. More information about this delta is given by Frijlink (1959).

29.4 Deltas with Strong Distributing Influences

As the distributing capacity of the coastal processes becomes relatively more important, the delta protrudes less and less into the sea. Even a river having a very large sediment transport can form such a delta if the coastal distribution capacity is high and/or the material is easily eroded.

An excellent example of such a delta is shown in figure 29.8 - the Amazon River in Brazil. Even though the sediment supply is enormous (ch.27) it is immediately swept away to the northwest. In this case a longshore current of up to 4 knots is caused by the South Equatorial current (see ch. 3). The current approaching the delta area from the southeast is not fully loaded with sediment as evidenced by the relatively steep shore slope in that area. To the north of the river mouth, the mud coast has developed as indicated in chapter 27.

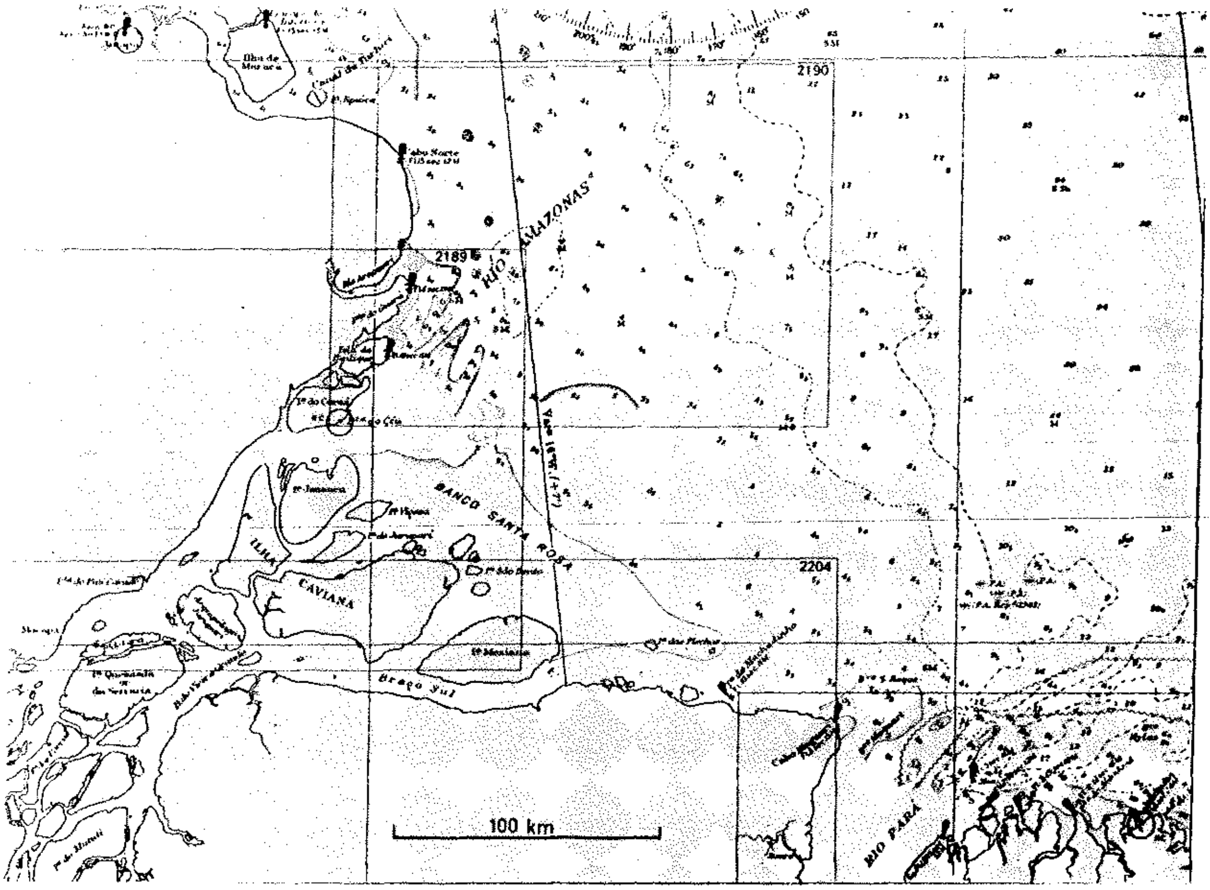


Figure 29.8
AMAZON RIVER DELTA, BRAZIL
(scale as shown, depths in fathoms and feet)

If the river sediment is heavier (sand instead of clay) even severe wave attack may not be sufficient to distribute this material along the coast. In such cases a somewhat horseshoe shaped shoal will develop just outside the river mouth. (This usually happens to some extent for every estuary). An example of such a river mouth bar has already been shown in figure 18.1, the mouth of the Columbia River, Oregon, U.S.A. As discussed in that chapter, breakwaters are being built here to narrow the river mouth and stimulate natural erosion of this bar. In effect, such works force the bar farther offshore.

In this and the previous sections,.the sand transport along the coast independent of the river supply has been negligible. In the next section we examine the influence of this longshore sediment supply.

29.5 Influence of Longshore Transport

When a significant longshore transport is present, still other delta forms can develop. The estuary mouth is often sufficient to block the longshore transport. Coastal material is deposited on the up-drift side of the entrance, narrowing the mouth. This enhances erosion resulting in a slow displacement of the entire river mouth sideways in the direction of the longshore transport. Figure 29.9 shows such an estuary - the Coos River (Coos Bay), on the Pacific Coast of the U.S.A. A dominant longshore transport from the north has displaced the estuary mouth several kilometers toward the south so that it is now confined by the rock outcrop (Cape Argo; Coos Head).

What is the source of the material in the North Spit in this figure? This has all been supplied by the longshore transport from the north. The relatively unimportant sediment supply of the river has either remained within the estuary as it slowly became longer or has been swept away past Cape Argo.

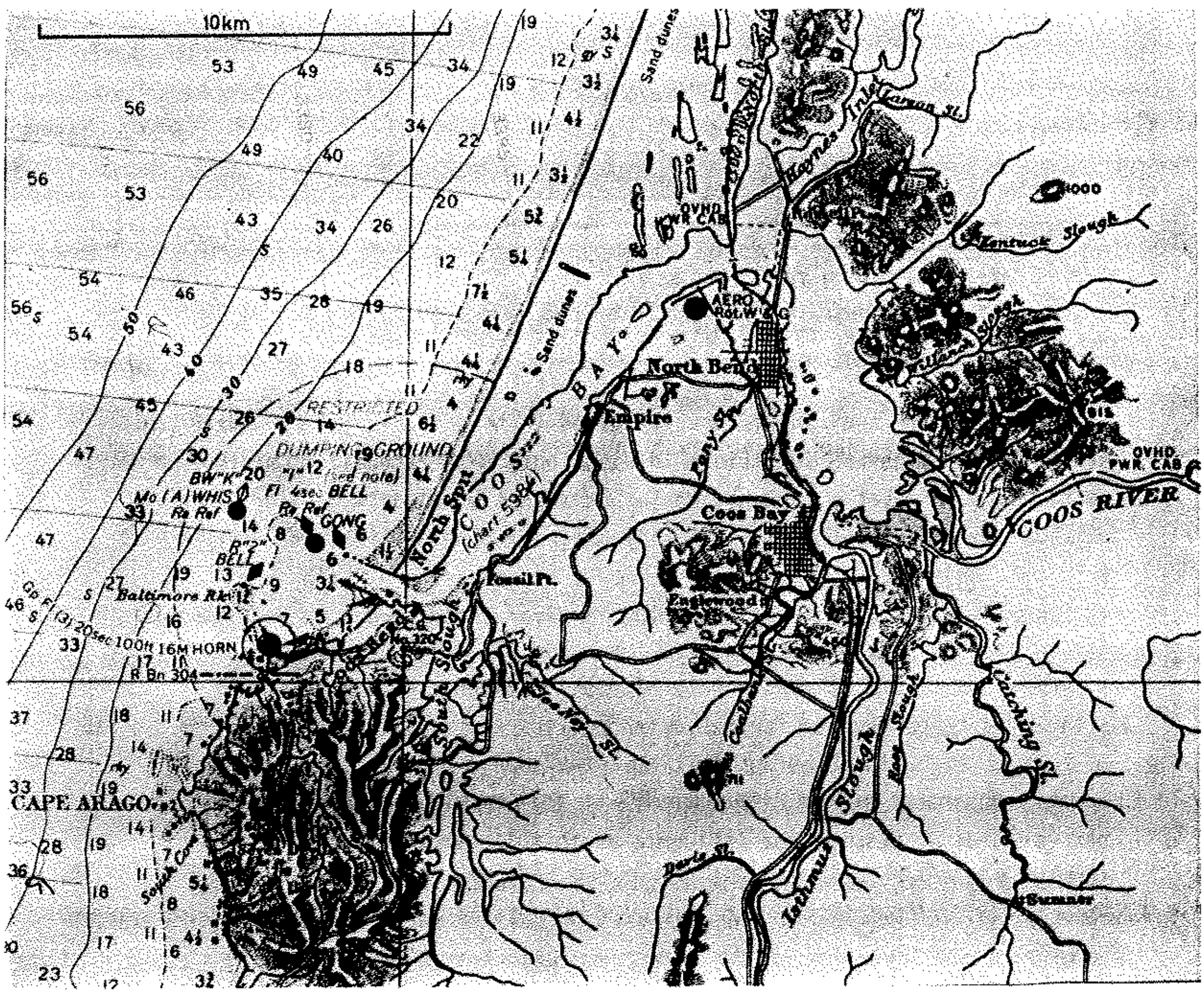


Figure 29.9
COOS BAY, OREGON, U S A
(scale as shown, depths in fathoms)

A river flow is not neccessary for such a spit development. At Netharts Bay, also on the Oregon Coast, the sand transport is from south to north. The tidal prism for this estuary is much smaller than for Coos Bay; a shoal has formed before the entrance and there is even a small chance that a heavy storm will close the entrance completely. This area is shown in figure 29.10.

One last comment about these last figures seems appropriate: both spits are covered with dunes of blowing sand. A photo made along this coast was included earlier - figure 25.4.

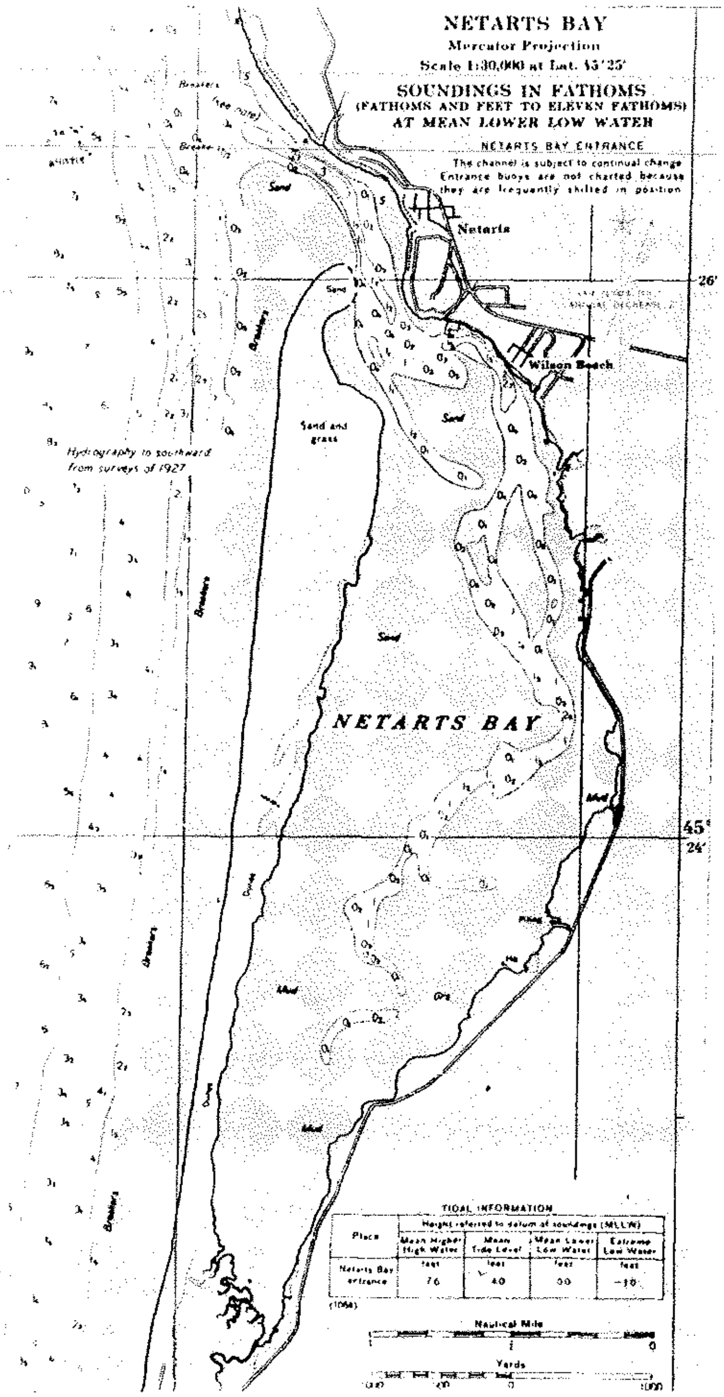


Figure 29.10
NETHARTS BAY, OREGON, U.S.A.
(scale as shown, depths in fathoms and feet)

FIGURE 30.2
BEACH ACCRETION NEAR BROUWERSDAM, NETH.

30.3 Jetties

When a longshore transport threatens to cause a shallowing of a harbor entrance, for example, this process can be interrupted by constructing a jetty perpendicular to the coast slightly "up-drift" from the harbor entrance. This jetty or breakwater should extend at least through the breaker zone, even during storms, and even after the coast has moved forward from accretion. Material transported along the coast will accumulate against the jetty on the "up-drift" side, opposite the channel.*

Most breakwaters attached to land can be considered to be jetties. Their design is the topic of volume III of these notes.

30.4 Groins

A jetty only prevents accumulation of material in a small area or stimulates accumulation in another rather restricted area; its influence is purely local.

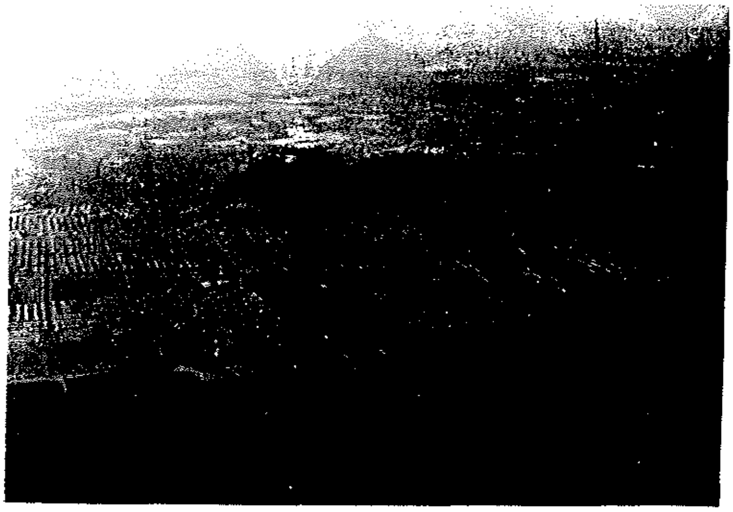
Groins, on the other hand, are a series of smaller jetties spaced at relatively short intervals along a coast. They tend to stabilize the entire coast along which they are built by keeping the coastal sand trapped between adjacent groins. As such, they can be used to defend an eroding coast.

Figure 30.3 shows an oblique aerial photograph of a portion of the coast of New Jersey, U.S.A. The groins on this coast are reducing the transport of sediment in the souterly direction - to the left in the photo. These groins are built much farther apart than is common practice.

* A case is known to one of the authors of a jetty being built on the wrong side of the channel; the consequences were not happy!

9

FIGURE 30.3
GROINS ALONG NEW JERSEY COAST, U.S.A.



A spacing equal to a few times the length of the groins is more common as shown in figure 30.4 along the coast of Scheveningen, The Netherlands.

FIGURE 30.4
GROIN PROTECTED COAST SCHEVENINGEN, NETH.

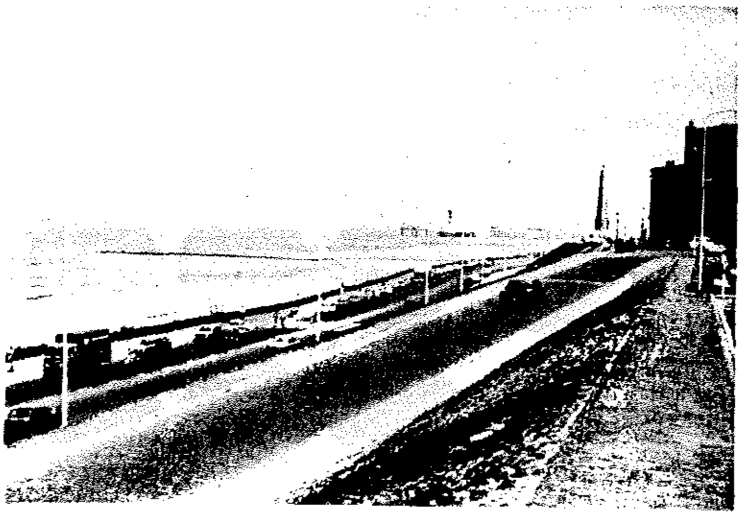


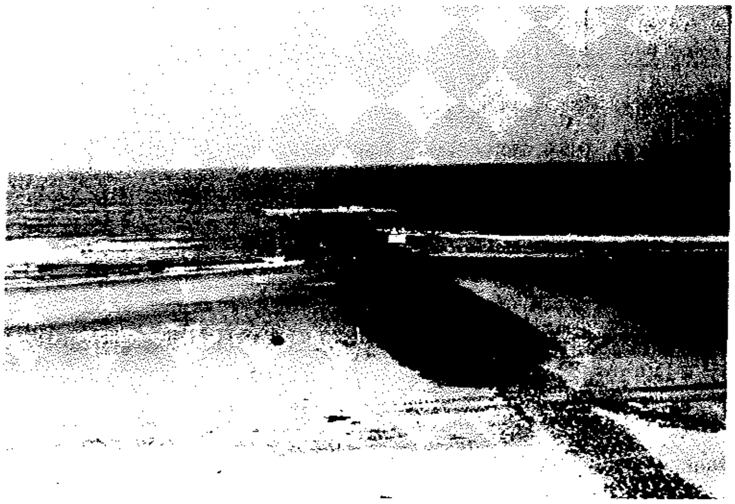
Figure 30.5 shows two of the many types of groins; more are illustrated in the *Shore Protection Manual*.

Unfortunately, neither a jetty nor a set of groins does anything to prevent material transport perpendicular to the coast. This was dramatically demonstrated late in 1973 when several severe northwest storms caused a significant coastal erosion near Scheveningen, The Netherlands. This was an example of the development depicted earlier in figure 25.5.

Groins derive their effectiveness by reducing or even halting the longshore transport at the locations along the beach where they are built.

Such coastlines can be analyzed most effectively using the two-line sand transport theory of Bakker - chapter 26. This analysis is discussed in detail in volume II.

a. DOUBLE PILE ROW FILLED WITH STONE VLISSINGEN, NETH.



b. BASSALT STONE GROIN SCHEVENINGEN NETH.

FIGURE 30.5
EXAMPLE OF GROIN STRUCTURES.

30.5 Detached Breakwaters

We have already seen how a segmented breakwater parallel to the coast can be used to stimulate a tombolo development - figure 28.7. As explained in section 28.4, such breakwaters reduce the longshore transport capacity in their shadow resulting in sediment deposition and the tombolo formation. Obviously, since such a breakwater is rather impermeable, the offshore transport of sand is also restricted.

This has led in the past to proposals by some that continuous breakwaters be built at the outer edge of the breaker zone in order to prevent offshore transport of sand. Unfortunately, proponents failed to realize that such structures also prevent the onshore transport of sand even more effectively. Any sand lost in the offshore direction now never returns; the net effect can be worse than doing nothing! In addition, such dams can suffer from severe foundation problems just as do seawalls. This is described in the following section.

30.6 Seawalls

Since offshore breakwaters are expensive (difficult) to build, especially near the breaker line, an alternative might be to construct an impermeable seawall on the beach parallel to the coast. The philosophy behind such a concept is that erosion will be prevented by simply cutting off the local supply of material.

Unfortunately, a rigid massive seawall tends to reflect the incoming waves. The increased turbulence resulting from this reflection stimulates the erosion of a deep trench before the seawall. The presence of this trench endangers the foundation so that a risk exists that the wall will fail by collapsing into the scour trench. This can be prevented, of course, by maintaining a beach in front of the seawall using some other means. If this is to be done, however, the logical question is, "why build a seawall, then?" It can be very effective for keeping people from using the beach!

30.7 A Remaining Problem

What are consequences for *adjacent* sections of beach caused by the various structures just described? Consider, for example, a long slowly eroding coast, a portion of which is protected by groins. Let us assume that the section protected by the groins has become stable - there is no more erosion along that beach segment. What, now, happens on the up-drift side of the *first* groin? Since the sand transport along the unprotected shore is greater than along the protected shore, accretion will take place against the first groin. This may or may not have deleterious consequences, but a new problem has been introduced.

What, on the other hand, happens on the down-drift side of the *last* groin? Since the coastal sand transport capacity abruptly increases here, a severe erosion can be expected. This usually does have deleterious consequences. Thus, construction of one of the shore protection works described may solve a local problem, but usually creates another one somewhere nearby.

Construction of other structures such as harbor breakwaters can involve the same coastal morphological problems.

30.8 Sand By-Passing

The "displacement of problems" described in the previous section often has the particular characteristic that our structure (a harbor entrance, for example) generates two complementary problems - an accretion and an erosion. In such a case, both problems can be solved by simply moving sand from the accretion area to the erosion area.

If the distance between the accretion and erosion areas is not too great an artificial by- passing operation may be economical. Sand accumulated by a jetty or single tombolo is moved to the eroding beach using some type of dredge. For very short distances, a pipeline on a suction dredge may be used. Occasionally, such a suction dredge is built on a fixed platform in the accretion area. More about such installations is told in the *Shore Protection Manual*.

31. COASTAL MORPHOLOGISTS' TEN COMMANDMENTS W.W. Massie

We have just seen in the previous chapter how many shore protection works displace problems to adjacent areas. Knowing this, and with tongue in cheek, Per Bruhn (1972) presented his version of the Ten Commandments as applied to coastal morphology. These are reproduced here with only minor editorial changes in tabel 31.1. Any resemblance to the original version of the Ten Commandments is intentional.

This concludes our discussion of coastal morphology. Coastal morphology is a major topic of volume II of these notes. This present volume continues in the next chapter with an introduction to the problems associated with offshore engineering.

Table 31.1. THE TEN COMMENDMENTS FOR COASTAL PROTECTION

1) *Thou shalt love thy shore and beach.*
2) Thou shalt protect it gainst the evils of erosion.
3) Thou shalt protect it wisely, yea, verily and work with nature.
4) *Thou shalt avoid that nature turns its full forte gainst ye.*
5) Thou shalt plan carefully in thy own interest and in the interest of thine neighbour.
6) Thou shalt love thy neighbour's beach as thou lovest thine own beach.
7) Thou shalt not steal thy neighbour's property, neither shalt thou cause damage to his property by thine own protection.
8) *Thou shalt do thy planning in cooperation with thy neighbour and he shalt do it in cooperation with his neighbour and thus forth and thus forth. So be it.*
9) Thou shalt maintain what thou has built up.
10) Thou shalt show forgiveness for the sins of the past and cover them with sand. So help thee God.

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32. OFFSHORE ENGINEERING

C.J.P. van Boven
W.W. Massie

32.1 Disciplines Involved

Offshore engineering refers to the engineering work related to all sorts of structures located offshore - see chapter 2 and figure 25.1. This definition includes work done by many other branches of engineering as well as civil engineering. While the major emphasis within this chapter will be on civil engineering aspects of offshore work, the ties to other specialized engineering fields will also at least be indicated.

Since many offshore structures are used by the petroleum industry, a strong relationship to mining and mechanical engineering is obvious. In general, the mining engineers determine what operations need to be carried out and where this must be done. Mechanical and electrical engineers determine what mechanical and electrical equipment is needed and translate this into a necessary work area and loading condition on this area.

Naval architects are concerned with floating structures whether they are real ships or fixed structures which are being towed to the construction site. Also, they provide valuable information about characteristics of ships which will be used in conjunction with fixed structures.

Oceanographers provide information on waves, sea currents, chemical and biological conditions - related to material properties and marine fouling - as well as sea bottom conditions so important to the adequate foundation or anchoring of a structure. Indeed, many disciplines are drawn together with civil engineering to execute offshore engineering works. The specification of the various civil engineering specialties which are involved in offshore engineering is postponed to a later section of this chapter. First, we discuss some of the various types and uses of offshore structures.

32.2 Types of Offshore Structures

Offshore structures can be grouped by form roughly into three categories: fixed, anchored, and free floacting constructions.

Fixed Structures

Fixed structures are most suited for heavy loads and uses for which the platform must be stable - radar islands, for example. These fixed structures can again be subdivided into three groups: gravity, jacket, and jack-up structures.

Gravity structures are the heaviest of the offshore structures and derive their stability againt overturning from their weight combined with their broad base. (The word gravity is used in the same way to describe a type of dam.) Offshore gravity structures are commonly built from concrete at some protected location and then floated and towed the the desired site where they are sunk and placed on the bottom. Figure 32.1 shows a sketch of the ANDOC (Anglo Dutch Offshore Concrete) platform being built near Rotterdam in 1975-76. The base (in prototype) is about 100 m square and 30 m high; the structure will be located in about 150 m water depth in the northern portion of the North Sea.

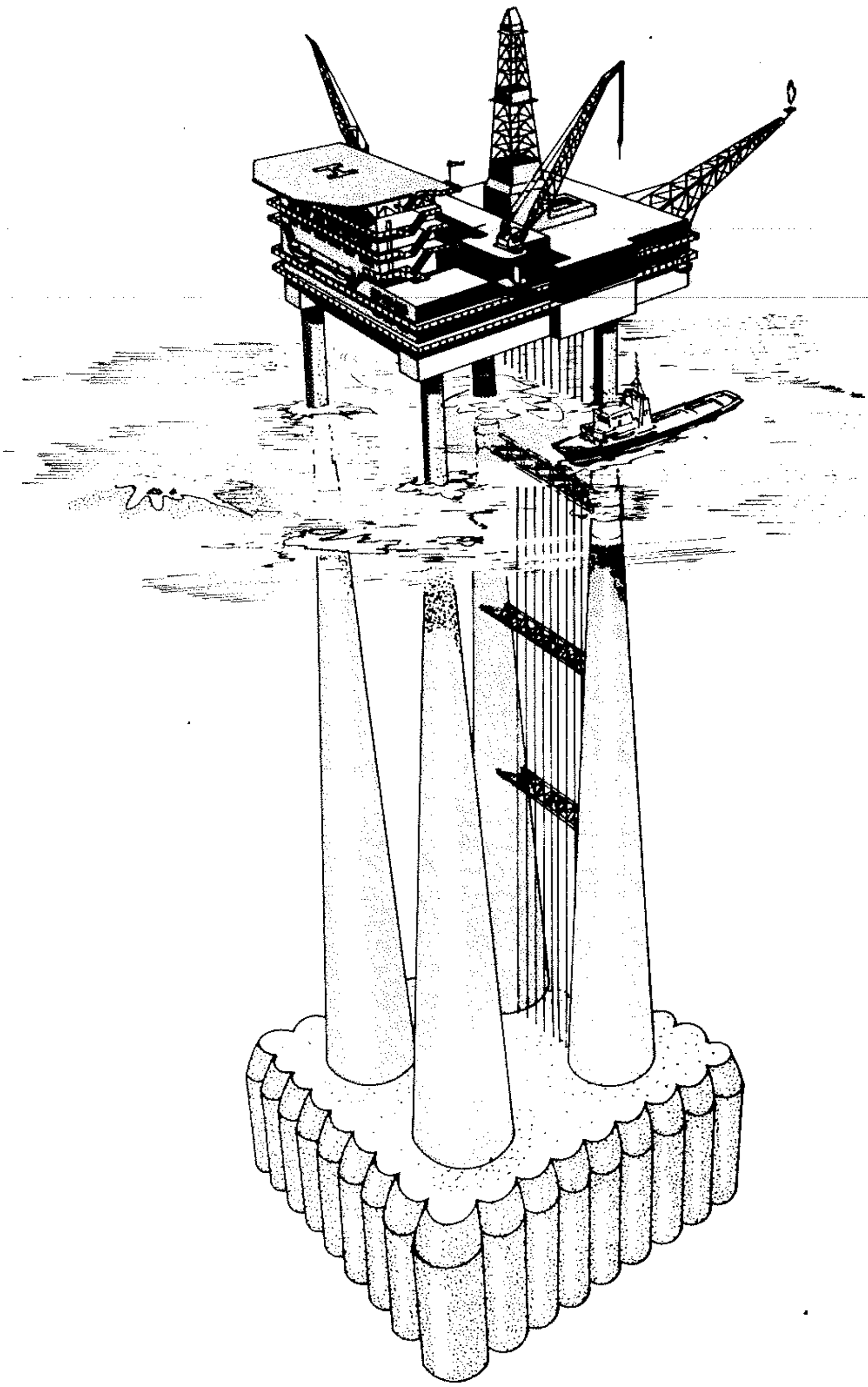


Figure 32.1
SKETCH OF ANDOC GRAVITY STRUCTURE

Jacket structures are a space frame constructed from hollow tubular elements. In order to appreciate the scale of such structures we must realize that these tubes for a jacket in the northern North Sea can be up to 10 meters in diameter with wall thicknesses of up to 100 mm. Such a structure to be placed in the Thisle Oil Field in a water depth of about 160 m will have a dead weight of about 30 000 tons. Complete and ready for use, the estimated cost of such a structure is about 300 million guilders*. The largest of the models in figure 32.2 shows such a structure. These are built on land and are either moved to place by ship or are floated and towed to position. In contrast to gravity structures, they are dependent upon pile foundations for their stability.

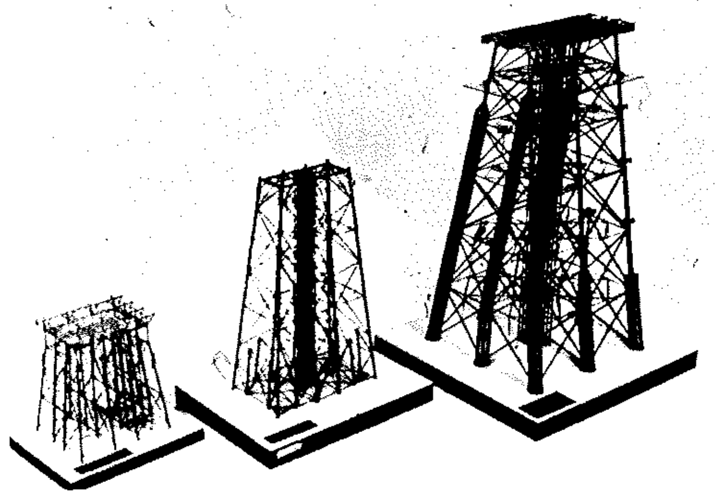
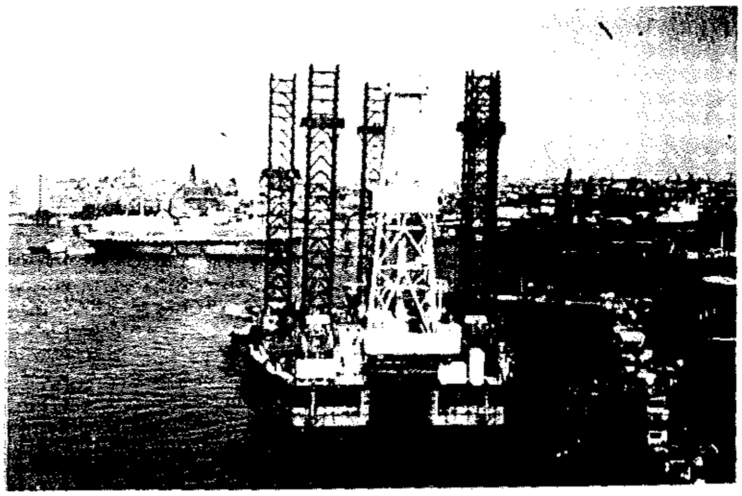


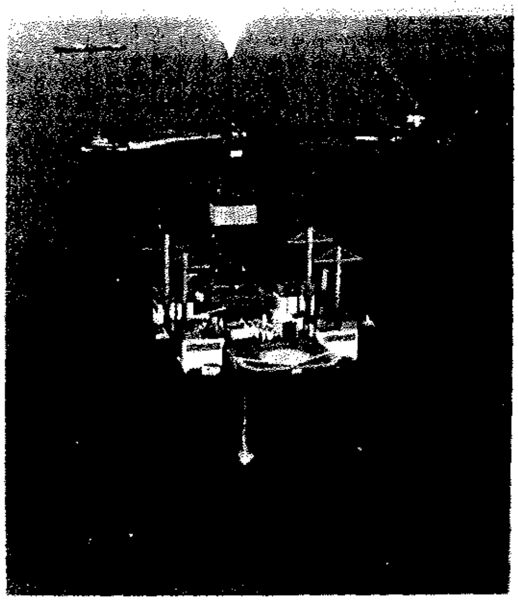
Figure 32.2
MODELS OF JACKET CONSTRUCTIONS

The third type of fixed structure, the jack-up platform, consists of a floating pontoon which raises itself above the sea surface by jacking itself along legs which are lowered to the sea bottom after the structure has been floated into position. Since jack-up platforms can be relatively easily moved from place to place they are best suited for temporary projects. Unfortunately, the raised legs have an adverse effect on the stability (sea-worthiness) during transport so that limitations on size and leg length restrict their normal use to shallow water locations (up to 40 m) and to light loads (a few hundred tons). They derive their stability in use from their weight; the bearing capacity of the sea bottom is obviously important for their optimum use. Figure 32.3 shows a jack-up platform floating with its legs raised.

* Such large quantities of steel or money are difficult to visualize. It can be instructive, therefore, to note that this structure is cheaper than beefsteak.



a. FLOATING



b. IN USE AS PRODUCTION PLATFORM
EKOFISK FIELD, NORTH SEA.

Figure 32.3
JACK-UP PLATFORM

Anchores Structures.

Anchored structures depend upon bouyant forces acting against some form of anchor force to provide their stability and maintain their position on the working site. Several subtypes are available: ships, semi-submersibles, articulated platforms and buoys.

We are all familiar with ships. Figure 32.4 shows one of the larger crane ships at work in the North Sea assembling a jacket structure. The analysis of such ships is primarily the responsibility of the naval architects. Civil engineers become involved with the anchoring, primarily.

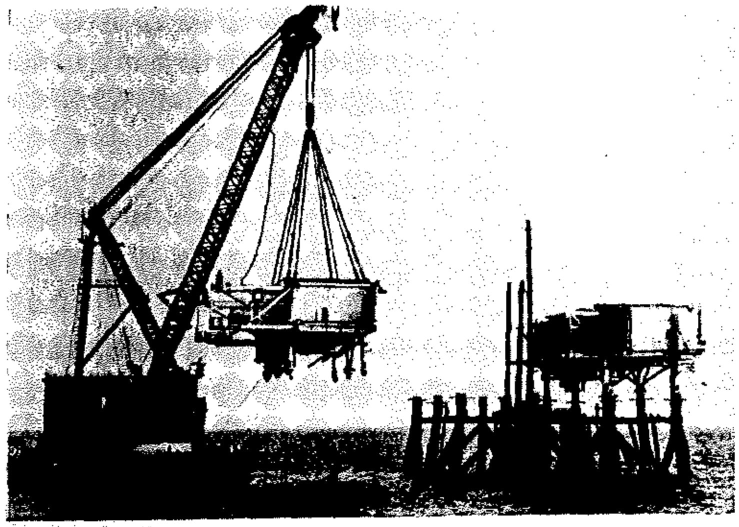


Figure 32.4
CRANE SHIP AT WORK.
1750 TON LIFT, B.P. FORTIES FIELD, NORTH SEA

Semi-submersibles are floating working platforms consisting of a deck supported above the waves by a group of submerged floatation chambers, as shown in figure 32.5. This particular general form is chosen in order to reduce the hydrodynamic forces and motions in waves. In an attempt to limit the vertical motions of a semi-submersible, it has been suggested that they be connected to heavy anchored with highly tensioned vertical cables. Such a tension leg platform has little vertical motion, but its inverted pendulum sidewise motion can be most unpleasant. This special type of anchored floating structure is not proving to be very popular.*

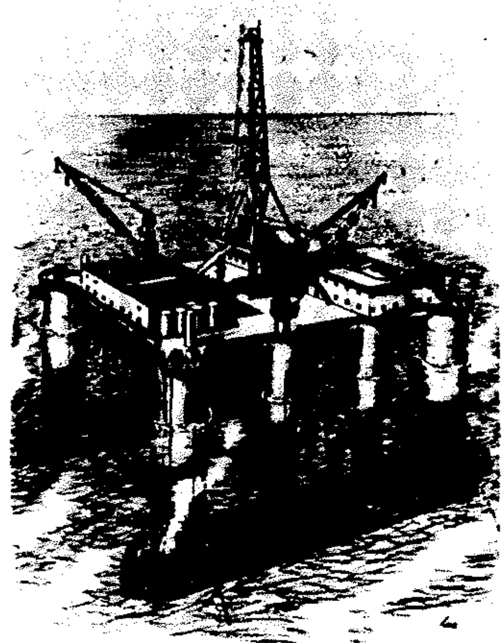
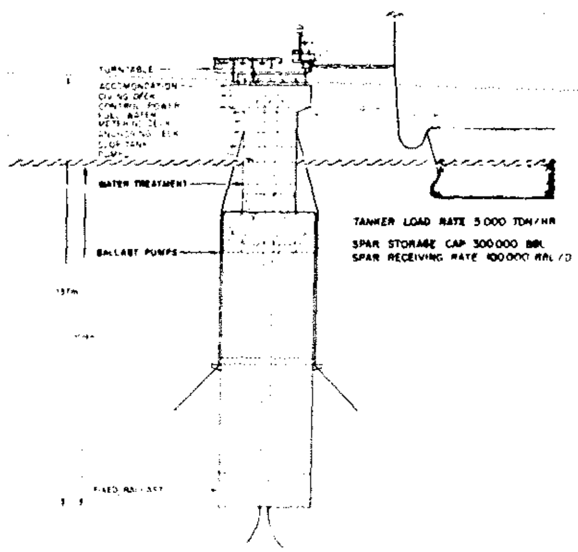


Figure 32.5
SKETCH OF A SEMI - SUBMERSIBLE PLATFORM

* Improvements in boring equipment technolgy have made the restriction of vertical motion less necessary, too.

Articulated platforms consist of a single vertical bouyant cylinder extending over the entire water depth hinged to a heavy anchor at its lower end. They are a reasonably new development sometimes used for offshore moorings.

Buoys have been used for a long time for navigational aids and for moorings. More recently, however, oil storage has been proposed in large, mostly submerged floating buoys. Figure 32.6 shows a concept sketch for one having a capacity of about 150 000 m^3.



SPAR TYPE STORAGE TERMINAL

Figure 32.6
SKETCH OF FLOATING OIL STORAGE BUOY

Free Floating Structures

Neither ships nor semi-submerisbles need to be anchored in position. Special propulsion systems have been developed which allow such a floating body to propel itself in various directions with forces just sufficient to counteract quasi-static force components resulting from currents or wind. Such systems, called dynamic positioning, are most adapted for use in very deep water where anchoring would be prohibitively expensive. A dynamically positioned ship has been used to carry out geological borings in some of the deeper portions of the Pacific Ocean - depths over 5000 meters, see chapter 3.

32.3 Uses of Offshore Structures

The various structures just described can be applied for many purposes. Several of these uses will be highlighted here along with the associated special structure characteristics required.

Navigational Aids

Navigational buoys and lightships were the first offshore structures. For economic reasons and increased dependability, many offshore light vessels are being replaced by fixed offshore structures. Figure 32.7 shows the construction of the jacket structure which replaces the light ship Goeree in the Dutch portion of the North Sea.

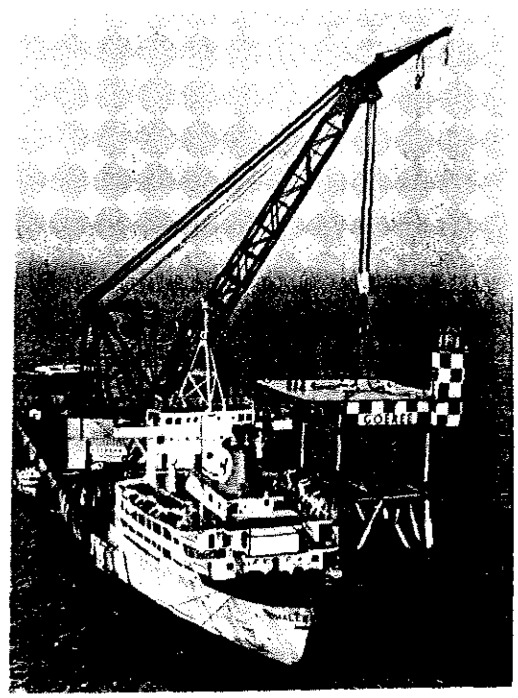


Figure 32.7
THE LIGHT TOWER GOEREE

As approach channels to harbors have become longer (chapter 15) and ships have become larger, it has become necessary to place more sophisticated navigational aids such as high resolution radar offshore. A special requirement of such a radar platform is that it have a high torsional stiffness so that the reference direction of the radar remains constant.

Moorings

Offshore moorings have been developed for use in areas where it is uneconomical te develop conventional harbors of sufficient depth for the larger ships. For the oil industry, such moorings provide for both ship anchoring and connection to a pipeline. Fixed structures are *not* generally used - they suffer too much damage from a collision with the ship and often do not let the ship swing into the on-coming waves. Buoys and articulated platforms are most suited to this work. Ships can even be moored to some large oil storage buoys. Moorings are discussed in more detail in volumes II and IV.

Oil Exploration

Initial oil exploration works carried out from the sea surface usually use ships. Later exploratory borings are carried out from anchored or dynamically positioned ships or semi-submersibles. Jack-up platforms can be used in relatively shallow areas. The choice among types depends largely upon water depth. Since exploratory borings usually do not take too long at one site (a few months, perhaps) portability is important for this type of equipment.

Oil Production

Once the extent of an oil or gas field has been defined, *production platforms* can be designed and installed. In contrast to exploration platforms, these have a much longer useful life at a single site (a few decades, hopefully). Fixed jacket or gravity platforms are then usually the most economical.

An idea of the production capacity of a production platform can be obtained by noting that the steel jacket construction described in the previous section is intended for a production of 300 000 barrels of oil per day from 60 wells.

Oil Storage

One of the simplest forms of oil storage uses a ship more or less permantly moored at the oil field. Gravity structures and an occasional steel structure are also used. The inclusion of oil storage in a mooring buoy is still uncommon as of late 1975. Gravity oil production structures usually include a storage resevoir as part of their base. This is true for the ANDOC structure shown in figure 32.1. Figure 32.8, on the other hand, shows a gravity structure designed exclusively for oil storage.

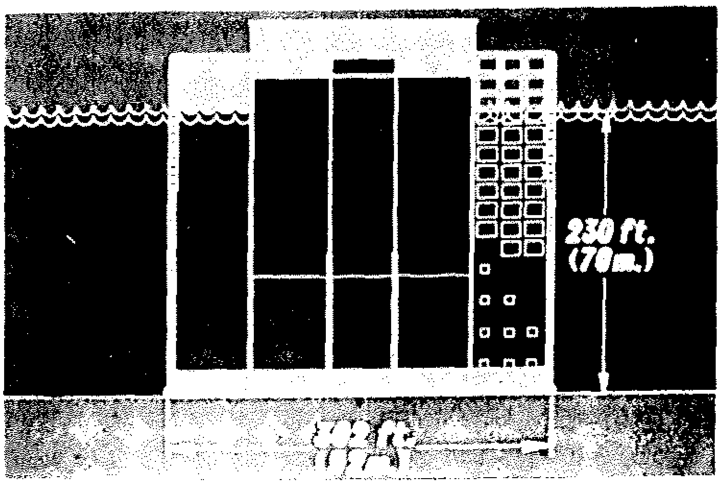


Figure 32.8
EKOFISK OIL STORAGE TANK

Pipelines

While it may seem at first glance that pipelines are rather unimportant, their dependable function if often critical. Submarine pipelines are used not only in the oil industry, they also serve as sewer outfalls, for example. Once of the major problems with pipelines is keeping them on the sea bottom. Ideally, they are buried deep enough over their entire length to hold them in position and protect them from ship anchors. Unfortunately, the sea bottom is not smooth; large areas are covered with irregular humps of sand similar to sand dunes, called megaripples. Experts (morphologists) are about equally divided over whether these ripples are stable. There is a good chance that under such conditions a pipeline will be buried in the crests of the megaripples and be hanging free between them. Hydrodynamic forces acting on the exposed pipeline can cause vibrations. If a resonant vibration occurs metal fatigue and failure can result. This is discussed further in volume IV. Living evidence of the fact that problems still exist with pipelines was provided when an oil pipeline in the North Sea unexpectedly floated to the surface late in 1975.

Construction Equipment

Various types of fixed and floating equipment are needed to install and service offshore facilities. Figure 32.4, for example, shows a very large crane in operation. Jack-up cranes are sometimes used to good advantage in very shallow water for construction of breakwaters, for example. These cranes can be floated into place in calm weather and can elevate themselves above breaking waves.

Ships and semi-submersibles are usually used for laying pipelines. Because they must exert a strong tension on the pipeline during the pipelaying operation (to prevent buckling) these units must be anchored.

32.4 Civil Engineering Aspects

Civil engineering aspects of offshore engineering can be subdivided to some extent along the lines of specialization within the civil engineering field. In this section, however, a subdivision involving the problem characteristics will be used.

Environmental Loads

The determination of environmental loads on an offshore structure can be subdivided into two sub-problems: the determination of environmental conditions, and the translation of these conditions into loads.

Environmental conditions result from nature. These include wind, waves, currents, ice formation, and earthquakes. A major problem is the determination of the probability with which a given environmental condition - or combination of conditions - will be exceeded within the lifetime of the structure. A bit of the technique of this has been indicated earlier in chapter 11.

Luckily, not all of the environmental conditions listed above are universally found. Ice, either drifting on the water surface or freezing on the superstructure, is only a problem in colder climates. The load resulting from ice frozen on the superstructure is not usually important for the design of the structure as a whole; it can be very important for individual parts of the structure, however. Earthquakes can present design problems for structures to be located in the Pacific Ocean basin and in the eastern parts of the Mediterranean Sea.

The second part of this problem is equally complex. Because the environmental loads on a structure are not, in general, directly proportional to the environmental conditions (such as wave height or current velocity) which cause them, the transformation of conditions into loads is not simple. The common technique of multiplying a conditions spectrum by a response function to determine a loading spectrum is not, in general, adequate. This problem is treated in depth in volume IV. The special topic of impact forces resulting from breaking waves is treated in volume III.

The determination of the environmental conditions is a task of coastal engineers and oceanographers. The transformation of conditions into loads is a current research topic for coastal engineers as well as fluid mechanics specialists.

The importance of the total of environmental loads should not be underestimated. They form, by far, the most critical loadings, and have even led to failure of offshore structures. Figure 32.9 shows a jacket type production platform during a storm.

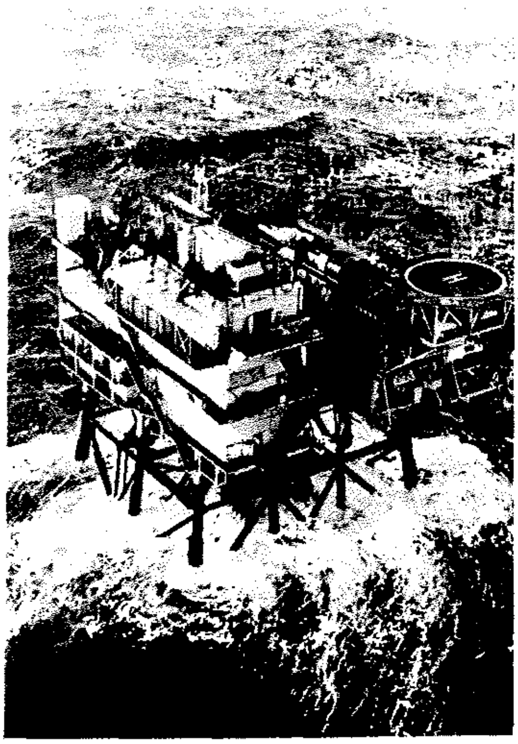


Figure 32.9
PRODUCTION PLATFORM DURING A STORM
FORTIES OILFIELD, NORTH SEA WIND FORCE 12

Structural Design

Once the environmental loads have been estimated, the detailed structural design in steel or concrete can be started. Obviously, since the environmental loads are dependent upon the size and location of construction elements, load determination and structural design are closely related, in practice.

Special structural problems are encountered with offshore structures, however. First, as already indicated in chapter 3, material properties can be affected by the sea water environment. Corrosion of steel is only one of the more obvious problems. However, corrosion combined with large dynamic environmental loads requires the modification of traditional fatigue stress relationships for use offshore.

The physical size of elements and the complexity of connections leads to complex stress concentration problems. Research on this topic by structural engineers is badly needed. A chapter in volume IV highlights the problems in more detail.

Foundations

The foundation design is also related to the structural design. One of the first problems of foundation design is the determination of bottom material properties in situ. Recent technical developments provide apparently good data now, however. Even so, foundations for fixed offshore structures must sustain very large static and dynamic loads when compared to land-based foundations. When the structure is in shallow water, waves can cause additional short period dynamic fluctuations in the soil pore water pressure which complicate the foundation design problem.

The anchoring of moored floating structures provides another specialized facet of offshore foundation engineering. For economic reasons, it is desirable to obtain a maximum anchor force with a minimum anchor weight.

Erosion near footings of structures and near pipelines can complicate foundation design. The foundation engineer and the coastal engineering morphologists should attack this problem cooperatively.

Corrosion and Fouling

Corrosion has already been indicated with regard to fatigue in structural design. Materials engineers are, of course, also concerned with the problem as they seek to improve materials used in offshore structures.

Fouling by the accumulation of marine growth on an offshore structure can have significant consequences. Since a fouling layer increases the effective size of a structural element, it increases the environmental load. A layer of marine growth over 20 cm thick has been found on a 100 cm diameter element of an offshore platform after 10 years. This layer has increased the effective diameter of the element by 40%! Such marine growths develop relatively rapidly on offshore structures as compared to coastal structures. Offshore, the marine life has less hinder from pollution and less competition for the available food resources of the ocean.

Obviously, accumulations of marine growth on structures also hinder periodic inspections which are often required by insurance underwriters.

Pollution Control

6 Both domestic and industrial wastes are produced on offshore structures. Sanitary engineers are beginning to attack the problem of disposal of these often relatively small quantities of environmentally damaging wastes.

Construction

Construction aspects of the fabrication of an offshore structure on land or in a drydock are not considered here. Problems associated with the placing of a structure on a site at sea are of interest to us, however.

Various problems arise during the transport of a large structure to its operating site. The floating stability and towing problems are handled in cooperation with the naval architects. The mobilization of the necessary number of sea-going tugboats to bring a structure to its location can present a challenging management problem. The precise determination of position of structures - necessary for the installation of connecting pipelines, etc. - provides an interesting geodesy problem.

Once the site has been reached other equipment must be mobilized to complete the construction at the site. The lifting capacity of seaworthy floating cranes can be an important limitation even though this capacity seems high - a few thousand tons. Piles must be driven at sea in order to guarantee the stability of jacket structures. Divers may be needed for a whole variety of underwater operations.

Since operations carried out at sea are so expensive compared to similar operations elsewhere, careful selection of construction techniques and management of equipment can be economically rewarding. Construction limitations and problems are discussed further in volume IV.

32.5 Other Problems

Several rather strange seeming problems arise which have special significance for offshore structures. The isolation of personnel on board the structures can lead to sociological problems not unlike those experienced on ships.

Since many offshore structures are located outside the area of the legal territorial waters, legal questions concerning taxes and customs can be expected. The threat of armed take-over of an offshore structure by pirates poses legal as well as strategic defense questions.

SYMBOLS AND NOTATION

W.W. Massie

The symbols used in this set notes are listed in the table. International standards of notation have been used where available except for occasional uses in which direct conflict of meaning would result. Certain symbols have more than one meaning, however this is only allowed when the context of a symbol's use is sufficient to define its meaning explicitly. For example,T is used to denote both wave period and temperature.

Functions are denoted using the British and American notation. The major discrepancy with European continental notation occurs with the inverse trigonometric functions. Thus, the angle whose sine is y is denoted by:

$\sin^{-1} y$ instead of arc sin y.

Possible confusion is avoided in these notes by denoting the reciprocal of the sine function by the cosecant function, csc, or by $\frac{1}{\sin}$. This same rule applies to the other trigonometric and hyperbolic functions as well.

In the table a meaning given in capital letters indicates an international standard. The meaning of symbols used for dimensions and units are also listed toward the end of the table.

Roman Letters

Symbol	Definition	Equation	dimensions	Units
A	AREA	20.01	L^2	m^2
A_E	of harbor entrance	23.05	L^2	m^2
A_H	of harbor surface	23.10	L^2	m^2
a	acceleration		LT^{-2}	m/s^2
a_c	Coriolis acceleration	3.01	LT^{-2}	m/s^2
a_i	Coefficient			
B	distance of river influence in ocean	22.06	L	m
b	distance between wave orthogonals	9.01	L	m
C	Chézy Friction Coefficient	20.02	$L^{\frac{1}{2}}T^{-1}$	$m^{\frac{1}{2}}/s$
c	wave speed	5.05	LT^{-1}	m/s
c_g	wave group velocity	5.06	LT^{-1}	m/s
c_v	volume concentration	16.02	-	-
D	depth of frictional influence	3.08	L	m
	apparent diffusion coefficient	23.03	LT^{-1}	m/s
D_o	diffusion coefficient at x=o	22.06	LT^{-1}	m/s
d	storm duration	12.03	T	hr

Symbol	Definition	Equation	dimensions	Units
E	expected chance	11.04	-	-
	estuary number	22.02	-	-
	Wave Energy per unit surface area	5.09	MT^{-2}	N/m
E_T	Wave energy per unit width	5.08	MLT^{-2}	N
e	BASE OF NATURAL LOGARITHMS		-	-
F	Froude Number	22.20	-	-
	fetch length	12.04	L	m
f( )	function of ( )			
f	hydraulics loss coefficient	16.01	-	-
G	coefficient	23.05	$L^{\frac{1}{2}}T^{-1}$	$m^{\frac{1}{2}}$/tide period
g	ACCELERATION OF GRAVITY		LT^{-2}	
H	wave height	5.01	L	m
H'	wave height neglecting refraction, diffraction	table 9.1	L	m
H_{rms}	root mean square wave height	10.01	L	m
$\overline{H}$	average wave height	10.04	L	m
h	water depth	5.01	L	m
$\overline{h}$	average water depth	20.03	L	m
i	subscript index			
K	coefficient	3.18	varies	varies
K_r	refraction coefficient	9.03	-	-
K_{rbr}	refraction coefficient at breaker line	26.02	-	-
K_{sh}	shoaling coefficient	7.06	-	-
k	wave number	5.01	L^{-1}	1/m
L	harbor length	'	L	m
L_w	length of intrusion wedge	22.18	L	m
ℓ	life of structure	11.16	T	yr
M	number of storms per year	11.06	-	-
	mixing parameter	22.01	-	-
m	beach slope	8.01	-	-
N	number of terms in series		-	-
	number of waves in record		-	-
N'	number of terms in series		-	-

Symbol	Definition	Equation	dimensions	Units
n	normal direction notation	3.02	L	m
	ratio of group velocity to wave velocity	5.07	-	-
P()	probability of ( )	10.02	-	-
P	volume of tidal prism	20.01	L^3	m^3
p	pressure	5.11	$ML^{-1}T^{-2}$	N/m^2
p'	absolute pressure	3.18	$ML^{-1}T^{-2}$	N/m^2
p^*	vacuum head	16.01	L	m
p	wave breaking parameter	ch.8.3	-	-
Q	volume flow rate		L^3T^{-1}	m^3/s
Q_w	volume flow rate in salt tongue	22.21	L^3T^{-1}	m^3/s
q	volume flow rate per unit width		L^2T^{-1}	m^3/s m
r	radius of curvature	3.03	L	m
S	salinity	3.18	-	$^o/oo$
S_s	salinity at moment of slack water	22.03	-	$^o/oo$
S	sand transport	26.04	L^3T^{-1}	m^3/yr
S'	total sand in motion	26.06	L^3T^{-1}	m^3/yr
s	sedimentation	23.20	L^3T^{-1}	m^3/tide
T	wave PERIOD	5.01	T	s
T_e	equivalent wave period	10.10	T	s
T_i			T	s
$\bar{T}$	average wave period		T	s
T'	tide period	20.04	T	hr
T	TEMPERATURE		degrees	^oC
t	TIME		T	s;hr
U	wave power per unit crest length	5.10	ML^2T^{-3}	N/s
U_w	wind speed	12.01	LT^{-1}	m/s
u	component velocity in x direction	5.01	LT^{-1}	m/s
V	total velocity		LT^{-1}	m/s
V_s	suction pipe velocity	16.01	LT^{-1}	m/s
v	component velocity in y direction		LT^{-1}	m/s
v	specific volume	3.18	$M^{-1}L^3$	cm^3/g
v_∞	coefficient	3.18	$M^{-1}L^3$	cm^3/g
v_v	volume of voids	23.23	L^3	m^3

6

Sym- bol	Definition	Equa- tion	dimensions	Units
w	component velocity in z direction		LT^{-1}	m/s
X	COORDINATE DIRECTION		L	m
x	COORDINATE DIRECTION		L	m
Y	COORDINATE DIRECTION		L	m
y	COORDINATE DIRECTION		L	m
Z	VERTICAL COORDINATE DIRECTION		L	m
Z_p	depth of submerged dredge pump	16.01	L	m
Z_s	depth of dredge suction pipe	16.01	L	m
z	VERTICAL COORDINATE DIRECTION		L	m
z'	VERTICAL COORDINATE DIRECTION	3.17	L	m

GREEK LETTERS

α	coefficient	23.07	-	-
β	water surface slope	3.16	-	-
γ	wave breaking index	ch.7.5	-	-
γ	unit weight		$ML^{-2}T^{-2}$	N/m^3
γ_g	unit weight of sand grains	16.02	$ML^{-2}T^{-2}$	N/m^3
γ_m	unit weight of suspension	16.01	$ML^{-2}T^{-2}$	N/m^3
γ_w	unit weight of water	16.01	$ML^{-2}T^{-2}$	N/m^3
δ	relative density of water masses	22.15	-	-
ε	eddy viscosity	3.05	$ML^{-1}T^{-1}$	Ns/m
ζ	vertical displacement of water particle	5.04	L	m
Θ	POLAR COORDINATE	3.13	-	rad.
	layer thickness	22.13	L	m
θ	phase angle	20.04	-	rad.

	Definition	Equation	dimensions	Units
.	coefficient	3.16	-	-
λ	WAVE LENGTH	5.01	L	m
ξ	horizontal displacement of water particle	5.03	L	m
Π	PRODUCT notation			
π	3.1515926536		-	-
ρ	density of water	3.20	ML^{-3}	kg/m^3
$\bar{\rho}$	average density of water	23.06	ML^{-3}	kg/m^3
Σ	THE SUM OF			
σ	NORMAL STRESS		$ML^{-1}T^{-2}$	N/m^2
σ_H	STANDARD DEVIATION of wave height	10.05	L	m
σ_t	ρ-1000	3.21	ML^{-3}	kg/m^3
τ	SHEAR STRESS	22.19	$ML^{-1}T^{-2}$	N/m^2
Φ	fetch parameter	12.04	-	-
ϕ	latitude	3.01	-	deg.
	angle of wave incidence	9.04	-	deg.
Ω	angular velocity of earth	3.01	T^{-1}	rad/s
ω	circular frequency	5.01	T^{-1}	rad/s

Special symbols

$\hat{}$	amplitude of	tab.7.1		
$\overline{}$	time average of			
$\forall$	volume		L^3	m^3
$\forall_H$	volume of harbor	23.08	L^3	m^3
∞	infinity			
$^0/oo$, %o	parts per thousand by weight	3.18	-	-

Subscripts

Symbol	Definition
b	bottom; evaluated at $z = -h$
br	breaker; evaluated at outer edge of breaker zone
D	density; caused by density influence
d	design
e	equivalent
f	filling; caused by harbor tide
g	group; wave group
I	interface
i	index counter
0	evaluated for ocean conditions
o	evaluated in deep water
p	pump
r	river
	refraction
s	surface
	suction
sh	shoaling
sig	significant
x	component in x direction
y	component in y direction
z	component in z direction
1, 2, 3, 4	used to distinguish similar values actual meaning from context.

Functions used

	Trigonometric functions
sin( )	sine of ( )
cos ( )	cosine of ( )
tan ( )	tangent of ( )
$\sin^{-1}$( )	angle whose sine is ( ) *
$\cos^{-1}$( )	angle whose cosine is ( )
$\tan^{-1}$( )	angle whose tangent is ( )

* The reciprocal of sin( ) would be denoted by csc( ) cosecant ( ).

	hyperbolic functions
	yperbolic sine of ( )
	hyperbolic cosine of ( )
	hyperbolic tangent of ( )
n^{-1}( )	argument whose hyperbolic sine is ( )
$\cosh^{-1}$( )	argument whose hyperbolic cosine is ( )
$\tanh^{-1}$( )	argument whose hyperbolic tangent is ( )
	logarithmic functions
log( )	logarithm to base 10 of ( )
ln( )	logarithm to base e of ( )
exp( )	e raised to the power ( )
P( )	probability of exceedance of ( )
f( )	general function of ( )
Π( )	product of ( )
Σ( )	sum of ( )

Dimensions and units

Symbol	Definition
^{o}C	degree celsius
cm	centimeter = 10^{-2} m
ft	foot
g	GRAM
h	hour
hr	hour
kg	KILOGRAM
km	kilometer = 10^{3} m
kt	knot = nautical miles per hour
L	LENGTH DIMENSION
lb	pound force
M	MASS DIMENSION
m	METER
mg	miligram = 10^{-3} g
mm	milimeter = 10^{-3} m
µm	micrometer = 10^{-6} m
N	NEWTON
rad	radians
s	SECOND
T	TIME DIMENSION
yr	year
o	degree temperature degree angle
$^{o}/oo$, ‰	parts per thousand

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