WORLD BANK TECHNICAL PAPER NUMBER 77

Integrated Resource Recovery

Wastewater Management for Coastal Cities

The Ocean Disposal Option

Charles G. Gunnerson, editor

UNDP Project Management Report Number 8

A joint contribution by the United Nations Development Programme and the World Bank to the International Drinking Water Supply and Sanitation Decade

1981-1990

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Integrated Resource Recovery

UNDP Project Management Report Number 8

INTEGRATED RESOURCE RECOVERY SERIES
GLO/80/004
Number 8

This is the eighth in a series of reports being prepared by the Resource Recovery Project as part of a global effort to realize the goal of the United Nations International Drinking Water Supply and Sanitation Decade, which is to extend domestic and community water supply and sanitation services throughout the developing world during 1981 to 1990. The project objective is to encourage resource recovery as a means of offsetting some of the costs of community waste management.

Volumes published to date include:

1. RECYCLING FROM MUNICIPAL REFUSE: A State-of-the-Art Review and Annotated Bibliography (Technical Paper No. 30) by S. Cointreau et al.

2. REMANUFACTURING: The Experience of the United States and Implications for Developing Countries (Technical Paper No. 31) by R. T. Lund.

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Technical Notes published to date include:

Aquaculture with Treated Wastewater: A Status Report on Studies Conducted in Lima, Peru (Technical Note No. 3). Compiled and edited by Sandra Johnson Cointreau.

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with contribution from
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ABSTRACT

This manual presents information on wastewater management practices for coastal cities and the options available for protecting the ocean coastal environment. It reviews the current state of knowledge on the planning, design, and construction of ocean outfalls for ultimate disposal in the marine environment. The paper is geared to engineers, planners, municipal authorities, and others interested in the subject.

The scope of the manual includes the scientific basis for understanding what happens to wastes discharged into the nearshore marine environment and the implications for appropriate ecological design. It also covers the engineering framework for outfall planning and site selection, hydraulic design of outfall pipelines and diffusers, selection of pipeline material, analysis of on-bottom stability and stresses on pipelines, and determination of shore approaches. Other engineering topics include corrosion control; state of the art in marine outfall construction, trenching, backfilling; and construction monitoring and inspection. Finally, consideration is given to performance monitoring for ocean outfalls, both for engineering and ecological purposes.

The report rounds out the presentation of scientific and technological information with a series of case studies from both developed and developing countries which illustrate the application of these principles under a variety of oceanographic, ecological, and socioeconomic conditions.

This manual is part of the Integrated Resource Recovery (Waste Management and Recycling) series published by the World Bank as an executing agency for the United Nations Development Programme (UNDP).

<u>Cover photographs</u>
Selection of appropriate systems and technologies for wastewater management in coastal cities depends on policy issues in environmental protection, water conservation and reclamation, and ocean disposal. Photographs (clockwise from top): Pair of brown pelicans, California (courtesy of Alan J. Mearns); photomicrograph of the dinoflagellate <u>Ceratium tripos</u>, Pacific Ocean; trickling filter treatment plant effluent being used for irrigating recreation area, Cyprus; testing corrosion coating on welded joints of 21-inch (53cm) steel pipe before adding concrete coating, California; harvesting portion of 8-ton per hectare annual yield from aquaculture system using mixture of primary effluent and river water, India; fishing fleet at home port on the Bosporus, Turkey.

FOREWORD

In 1981, the Global Research and Development Project on Integrated Resource Recovery (Waste Management and Recycling) was initiated as Project GLO/80/004, later superseded by Project GLO/84/007, by the United Nations Development Programme through its Division for Global and Interregional Projects. The World Bank, via its Infrastructure and Urban Development Department (formerly Water Supply and Urban Development Department), agreed to act as executing agency.

Increasing recognition of both the need for technical and economic efficiency in the allocation and utilization of resources and the role that appropriate technology can play in the water and sanitation sector has led to the inclusion of this project in the formal activities of the United Nations International Drinking Water Supply and Sanitation Decade.

The ocean margin is the most vulnerable and heavily utilized part of the world oceans. Most large cities of the world are located along estuaries, embayments, or the open ocean. National security, commerce, industry, fisheries, recreation, water supply, and waste disposal activities are served by the ocean margin. The value of the environment and other resources at the ocean margin largely depends on appropriately planned and executed systems for wastewater disposal. These systems are the subject of this manual.

Storage and reuse of liquid wastes could significantly improve the situation; however, in most cases, adequate disposal facilities have to be constructed. Marine outfalls can therefore also be an integrated part of a coastal waste reuse system for discharging unused quantities of wastewater properly, especially when sufficient storage is not available or when problems arise within the treatment or reuse cycle.

This manual is intended as an introduction to oceanography for sanitary and environmental engineers and, for oceanographers, as an overview of environmental assessment and decision making by engineers. Both disciplines are required in marine ecosystem analysis and protection. The scope of the manual includes the scientific and technological framework for selecting appropriate ocean outfall systems. These outfalls may be added to existing systems for collecting, treating, utilizing, or disposing of wastewaters. Alternatively, they may be a component of an entirely new system for waste management. In either case, the framework for decisionmaking includes the same elements of water supply and sanitation service levels, technological appropriateness, costing, and environmental protection.

We greatly appreciate the efforts devoted to this study by Charles Gunnerson and the guidance of John M. Kalbermatten for this important manual.

We shall be grateful for any comments you may have and for calling to our attention such errors of fact or interpretation as may be appropriate.

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Water and Urban Technologies and Assessment
Infrastructure and Urban Development Department
The World Bank

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PREFACE AND ACKNOWLEDGMENTS

The ocean margin is both the most vulnerable and the most heavily utilized part of the world ocean. Most large cities of the world are located along estuaries, embayments, or the open ocean, and national security, commerce, industry, fisheries, recreation, water supply, and waste disposal activities are served by these waters. The value of the environment and other resources at the ocean margin depends in large part on appropriately planned and executed systems for wastewater recycling and disposal. These systems are the subject of this manual. This manual is directed toward principals involved in identifying, preparing, appraising, and implementing water supply, community, and environmental sanitation projects for coastal cities in developing countries.

The scope of this manual includes the scientific and technological framework for selecting appropriate ocean outfall systems. These outfalls may be added to existing systems for collecting, treating, utilizing, or disposing of wastewaters. Alternatively, they may be a component of an entirely new system for waste management. In either case, the framework for decisionmaking includes the same elements of water supply and sanitation service levels, technological appropriateness, costing, and environmental protection.

Experts from many disciplines have contributed to this volume. Responsibility for Chapters 2 and 3, which deal with the physical and living framework of the ocean margin, is shared by D. A. Segar, P. G. Davis, and E. Stamman, SEAMocean Consultants, and the editor. Chapter 4, Hydraulic Design, was prepared by Jonathan French, CDM International. Chapters 5-13 on design and construction are the work of J. T. Powers and D. R. Miller, Daniel, Mann, Johnson, and Mendenhall, Inc. The editor and D. A. Segar prepared Chapter 14 on operational monitoring. Case studies presented in Chapter 15 were prepared by the editor and D. A. Segar (Thames Estuary, New York Bight, Southern California Bight, and Hiroshima), and M. Cuellar (Istanbul and Manila). Chapter 16 was prepared by the editor. Special thanks go to R. Walton, CDM International, for portions of Chapter 4 on numerical modeling; to Frederick Schremp, consulting corrosion engineer, for portions of Chapter 9 on cathodic protection; to B. Wright, Suffolk County Department of Public Works, for information on one of the largest ocean outfall systems on the U.S. East Coast; to V. J. Cabelli, University of Rhode Island, for his evaluations of microbiological and epidemiological information; to J. S. O'Connor, U.S. National Oceanic and Atmospheric Administration, for his insight into the ecology of the ocean margin; and to W. F. Garber, consultant, for guidance in evaluating state-of-the-art systems for wastewater treatment.

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I am most grateful to John Kalbermatten and DeAnne Julius for their insights and guidance in both the original and continuation of the World Bank research which are revealed in this manual. Finally, I owe a special debt to my wife, Betty Gunnerson, for her support and patience throughout the many years of my preoccupation and absences on land and at sea during which my appreciation and respect for the ocean margin have developed (C.G.G., Boulder, Colorado, 1987).

CHAPTER 1

OVERVIEW

The certain fate of wastewaters in coastal communities is ocean disposal, whether through natural streams, ocean outfalls, or over the beach. Wastewater disposal is expensive, exceeding from about one-tenth to ten times the cost of supplying water. Added to the costs of the system itself are the environmental costs of improper disposal, although these are difficult to quantify. The social costs and the remedial costs can be determined, however. When the latter two become equal, either because of increasing pollution or decreasing remedial costs, decisions are made to assume the remedial ones, although the cost-benefit ratio may be marginal in comparison with other investments. This manual identifies for engineers, scientists, economists, planners, and other advisors the principal factors, constraints, and options available for protecting the health of the ocean margin in the face of competing demands upon it.

This document is intended to support policy decisions for wastewater management in coastal communities, specifically those relating to wastewater reclamation in which a certain amount of the wastewater will have to be disposed of at sea. It is intended as an introduction to oceanography for sanitary and environmental engineers and, for oceanographers, as an introduction to environmental assessment and decision making by engineers. Both disciplines are required in marine ecosystem analysis and protection.

Chapters 2 and 3 look at the physical, chemical, and biological structure of coastal lands and waters and explain the scientific basis for the engineering and financial decisions that must be made in managing wastewater systems. Chapters 4-13 deal with the mathematical modelling, engineering, planning, design, and construction of submarine outfalls. Chapter 14 presents the scientific rationale for operational monitoring of outfalls and receiving waters; the emphasis here is not so much on essential research as on the question, "What do you do when you find out?"

Chapter 15 examines information gained from selected case studies in both industrial and developing countries. The case of the Thames Estuary, for example, illustrates the effectiveness of a simple predictive model in the hands of careful observers and committed institutions in restoring a polluted ecosystem. In the case of New York Bight, the costs and benefits reveal the limits of conventional, generally well-executed waste disposal planning and implementation. Total pollutant discharges in this area are expected to remain at about 1950 levels even though annual capital and recurrent costs for the period 1950 to 2000 are estimated at $1.1 billion* (1980 dollars) per year. Another interesting case is that of Southern California Bight, which is

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* Unless otherwise indicated, $ = US$.

reviewed in the light of what is probably the most complete, competent, and cost-effective marine pollution research in the world, and in the light of the pollution indices and the diagnostic and predictive models now coming into wider use. The fourth example from an industrial country is Minimata Bay, where careful chronicling and research have revealed the fragility of the environment and of industrial and government institutions in the face of acute toxicity in wastes and the time that it takes for previously unknown impacts to be identified and corrected.

Developing country examples are drawn from the area around the Bosporus and the Sea of Marmara, and from Manila Bay. Because of the unique geographic and hydrographic regimes of the Turkish Straits, difficult choices have to be made with respect to outfall locations and details, separate or combined sewers, interceptor costs, and construction technologies. Treatment requirements are not a major issue, however. After minimum flotation and grit removal, sewage discharged with minimum dilution will flow with the lower layer of the Bosporus into the lower anaerobic layer of the Black Sea, where the organic portion will be stabilized in the largest anaerobic digestion system in the world. If near-term interceptor investments are kept to a minimum, flexibility in implementing future wastewater reclamation schemes will be assured.

Manila has one of the more viable systems with respect to extending household and environmental sanitation to the urban poor. The problems connected with Manila's humid tropical climate, persistent seasonal monsoons, and the need to protect large fishery resources of Manila Bay and the fresh-water Laguna de Bay and Pasig River are being addressed by a number of increasingly coordinated agencies in a series of World Bank, Asian Development Bank and bilaterally supported projects concerned with water supply, sewerage, solid waste management, and urban development.

CHAPTER 2

OCEANOGRAPHY AT THE MARGIN

2.1 Introduction

Ocean science has long been concerned with global navigation, fisheries, climate, geology and geophysics, water and energy balances, national security, and the origin of life. Most of the economic value of the ocean lies along its margin--in its fisheries, recreation, tourism, transportation, placer deposits, cooling water, and waste assimilative capacity. To make the best of these ocean resources, we must assess, appreciate, allocate, and efficiently conserve them. This chapter identifies the nonliving and living frameworks of the ocean margin into which marine outfalls discharge, whether over the beach, onto tidal flats, or into or beyond the surf zone. These areas were generally shaped by Pleistocene geology and climate, and their responses to waste discharges must be taken into account in engineering, environmental, and investment decisions.

2.2 The Physical, Geological, and Chemical Framework

Many of the world's peoples live within 100 km of the ocean, which provides them with food, transport, and communication. The oceans cover nearly 71 percent of the earth's surface and contain 86.5 percent of the earth's water. Their total volume of water is approximately 1,350 million km^3 and their average depth almost 4 km. (The geography, principal surface currents, and continental shelf and margin areas to depths of 1,000 m are shown in Figures A1 and A2 of Appendix A.)

Solar energy heats the ocean, which in turn supplies energy for atmospheric circulation. Much of this energy is used to evaporate water, which is returned to the earth as rain. Approximately 10 million tons/sec of freshwater flow into the oceans, mostly from river runoff and melting ice (71). The areas and depths of the major ocean basins are listed in Table 2-1.

Light, temperature, and salinity are the primary variables that control biological and chemical interactions in the ocean. Figure 2-1 shows typical temperature, salinity, and density depth distributions. In the surface layer, temperature, salinity, and density are determined by heating, cooling, evaporation, and precipitation. The permanent pycnocline (there is also a shallower, seasonal one) is where the water density changes rapidly with depth at the base of the surface layer. This layer tends to be stable and prevents deep ocean waters from mixing with surface waters. Most ocean water lies below the pycnocline, is cold, and receives no light; almost all of the organisms that live there depend on food sources that have settled into this layer from the upper layers.

TABLE 2-1

Areas and Depths of the World Oceans (65)

Ocean Area	Water Area		Land Area Drained		Ratio, Water/Land	Average Depth	
	(10^4 km^2)	(10^4 mi^2)	(10^4 km^2)	(10^4 mi^2)		(m)	(miles)
Pacific	180	69.5	18	6.95	10	3940	2.44
Atlantic	107	41.3	67	25.8	1.6	3310	2.06
Indian	74	28.6	17	6.57	4.3	3840	2.38
World Ocean	361	130	102	39.4	3.6	3730	2.32

Surface waters are driven by winds, tides, and density differences. Bottom currents are caused by density differences. Wind forces mix shallow coastal waters, which sometimes move to the bottom as a result. Solar energy, which drives the principal surface currents shown in Figure A-1, acts through both heating and evaporation. The close linkage between large-scale weather, wind, and current patterns--important to both sailors and oceanographers--is revealed in Figure 2-2.

2.2.1 Coastal Waters

In general, oceanographic processes in coastal waters are similar to those in the open ocean, except that in coastal waters they occur over a much shorter period of time. As a result of the shallow depth and the influence of the adjacent land masses, much of the circulation of coastal waters is wind-driven. Diurnal land and sea breezes, wave movement, storm surges, coastal currents, and upwelling all affect the water column and, therefore, sediment transport.

Water temperature, more than any other factor, controls the physical, chemical, and biological conditions in the coastal ocean (6, 14). Whereas bottom waters in deeper ocean areas generally undergo very little temperature change, coastal water masses are subject to seasonal and diurnal temperature fluctuations. Thus the highest and lowest ocean surface temperatures are found in coastal waters. During the day, surface waters are warmed by the sun to depths up to 10 m. These waters reach their highest temperature in midafternoon and their lowest at dawn.

A thermocline is a layer of water with a large vertical temperature gradient that separates a warmer, upper layer from a cooler, bottom layer (see Figure 2-1). Permanent thermoclines are resistant to change and are the major stabilizing feature in the water column. Shallower, seasonal thermoclines are less stable and can disappear for short periods following major storms.

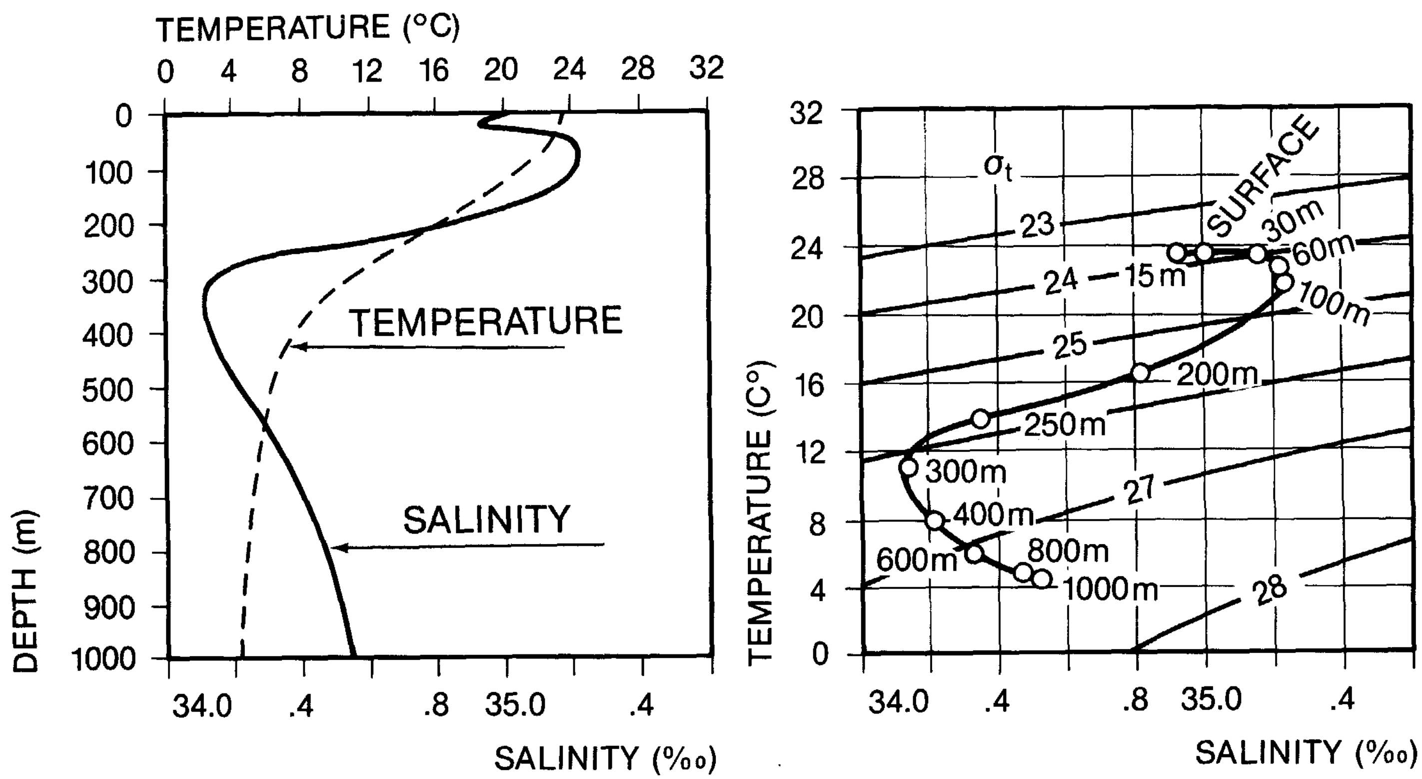


Fig. 2-1. Temperature and Salinity, Hawaiian Islands. The left diagram illustrates the variation of temperature and salinity with depths for an area in the vicinity of the Hawaiian Islands. The T-S curve (right diagram) illustrates temperature vs. salinity in profile with depth. This curve identifies a water mass which helps in tracing the origins of distinct layers that mix along layers of equal density or sigma-t. Source: Adapted from Smith and Brown (62).

Haloclines, which are similar to thermoclines, are layers in the water column characterized by a strong vertical salinity gradient. Salinity varies greatly in coastal waters because of the freshwater coming in from rivers and land runoff. Strong haloclines develop where river discharges of low salinity flow out over heavier, more saline coastal ocean waters. Since density is a function of both temperature and salinity, the pycnocline may be a thermocline, a halocline, or a combination of both. Thermoclines in coastal waters are often seasonal, developing in spring as waters are warmed, becoming stronger in the summer, and disappearing in the fall and winter as the waters become thoroughly mixed. This seasonal temperature cycle of waters is strongest at midlatitudes where changes in air temperatures are also greatest. Salinity changes--which result from varying amounts of precipitation, evaporation, and freshwater inputs--are also seasonal.

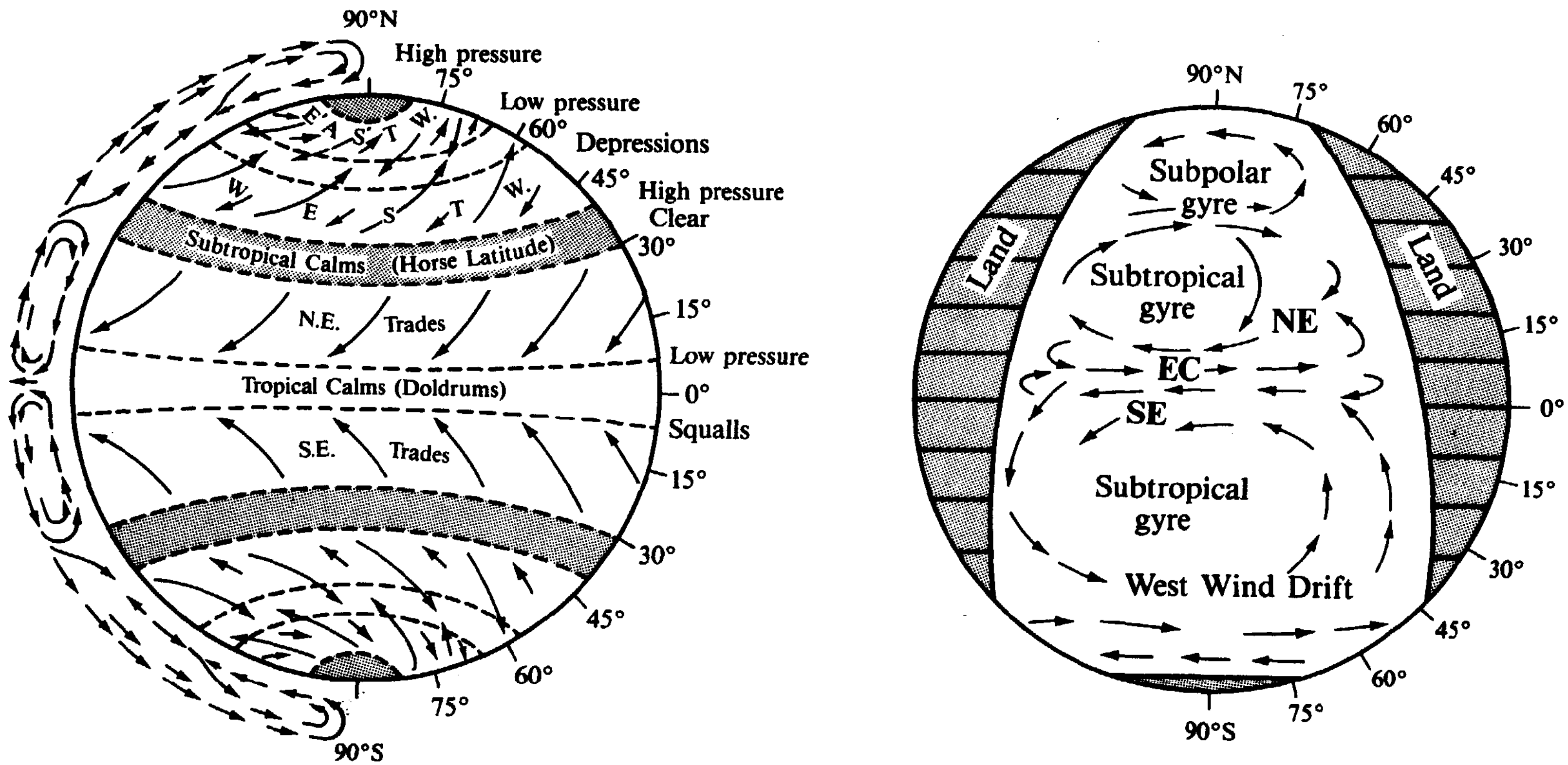
Currents in open coastal waters are a result of winds, tides, and geostrophic forces. Geostrophic forces are generated by the earth's rotation. Currents may be local, regional, seasonal, and/or permanent. Near the sea surface and in shallow water, currents are directly related to geostrophic winds (such as trade winds), to seasonal (monsoon) winds, and to diurnal land breeze and sea breeze systems. When winds blow the surface waters, high and low points are formed on the sea surface. As the sea surface slopes downward, water gravitates downhill and flows in a direction influenced by the Coriolis effect of the earth's rotation. As a result, water is deflected up to 45^{o} to the right of the wind direction in the northern hemisphere and to the left in the southern hemisphere (Figure 2-2). The Coriolis effect increases with increasing latitude, but is constrained in coastal waters by the shoreline and other factors. Geostrophic currents are strongest when river runoff is large and when strong winds parallel the coast. However, even weak geostrophic currents will move wastewaters counterclockwise along the right bank of an embayment in the northern hemisphere (clockwise in the southern), a factor to be considered in outfall siting.

Geostrophic forces also control the large, near-coastal currents such as the Gulf Stream and the Peru Current, and are reflected in the counterclockwise circulation of the Mediterranean and Black Seas).

Surface waters moved away from the coast directly or by the Coriolis effect are replaced by subsurface waters that upwell to replace them (Figure A-1). Upwelled waters usually form small, cold, nutrient-rich surface water masses notably along arid coasts in subtropical areas, such as those of Peru and Ecuador. Since freshwater runoff is not present in such areas, there is no low-salinity surface layer, and cool, upwelled waters readily rise to the surface. In areas with large freshwater discharges, the low-salinity surface layer inhibits wind-induced upwelling.

Tidal currents are caused by the pull of the sun and the moon on the earth. They normally move in a rotary pattern in the open ocean, but, in the coastal ocean, the typical pattern is elliptically reversing (see Figure 2-3). The tidal currents may, or may not, be locally dominant. On essentially open coastlines, currents generated by small tidal ranges (up to 0.2 m)

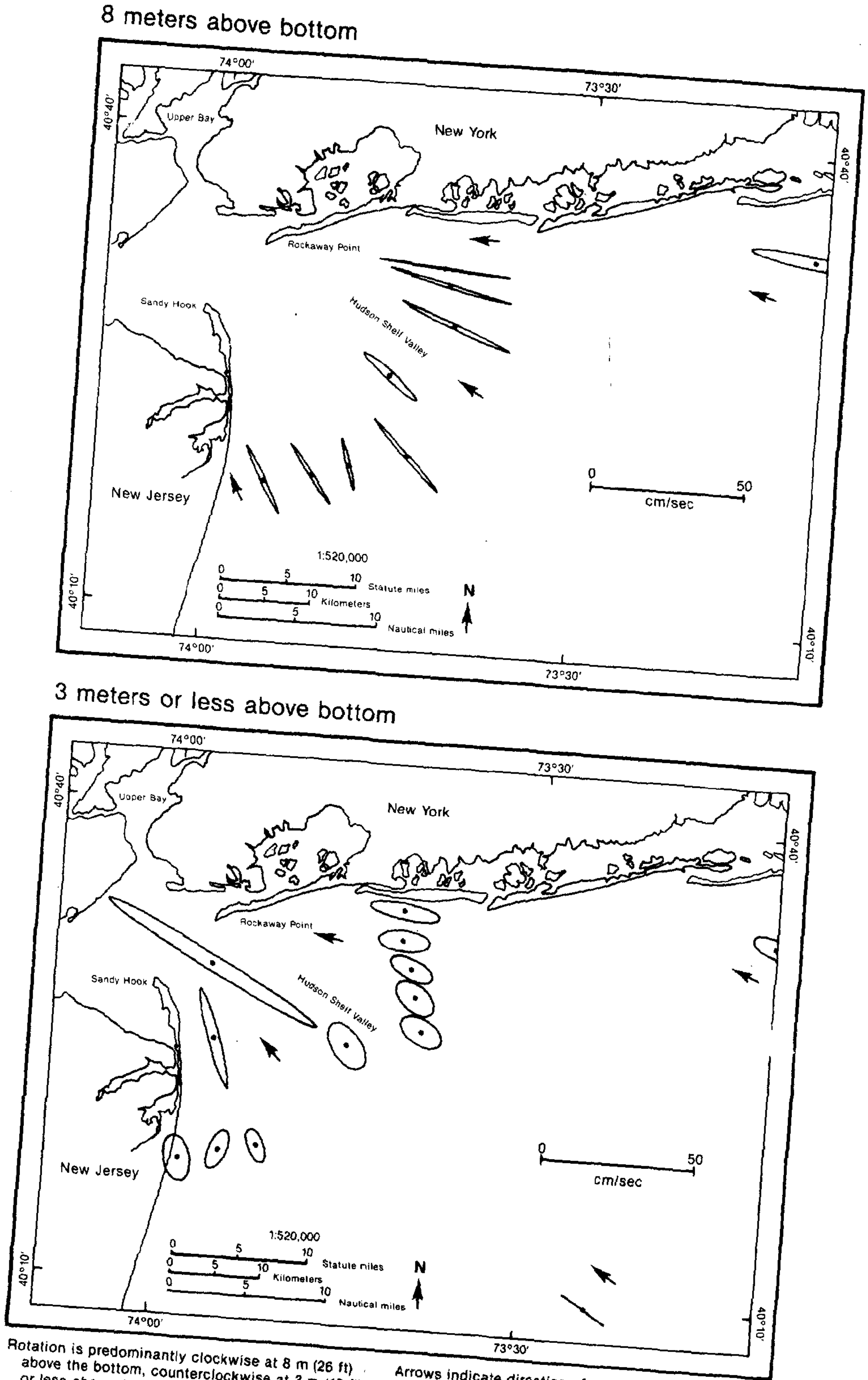
Fig. 2-2. Planetary Atmospheric and Oceanic Circulation.
Source: Adapted from Fleming (23) and Gross (29).



Atmospheric Oceanic

Legend - NE - north equatorial current
EC - equatorial countercurrent
SE - south equatorial current

Fig. 2-3. Tidal Currents in New York Bight Apex.
Source: Hansen (33).



Rotation is predominantly clockwise at 8 m (26 ft) above the bottom, counterclockwise at 3 m (10 ft) or less above the bottom.
The centers of ellipses are the station locations.

Arrows indicate direction of progress of the maximum M_2 flood current velocity.
Current velocity vectors rotate 360° in 12 to 42 hours.

are only a minor component of the total currents. However, in semi-enclosed basins such as Long Island Sound, even small ranges result in locally strong currents. In either situation, large local tidal ranges (1 m or more) result in significant components of coastal currents.

2.2.2 Waves, Tides, and Sea Level

Although waves move such small amounts of water that the Coriolis effect is negligible, they do transmit energy from winds to the shore areas (where wastewater outfalls are built). This energy ranges from 1.4 mw/km (2.2 mw/mi) during relative calms with 0.6 m (2 ft) waves to 40 mw/km (65 mw/mi) during storms. Their height, measured from trough to crest, may reach 30 m (100 ft), depending upon wind velocity, duration, and fetch (distance). As waves enter depths less than about half their length, their velocity and length begin to decrease, their steepness and height increase, and, when the water depth is about four-thirds to twice their height, the waves begin to break. Spring dynamometers have measured pressures up to 70 tonnes/m^2 (7T/ft^2) to which must be added wave momentum (16, 41).

Wave energy is a major design consideration. At the shoreline, surf forces have broken off and moved reinforced concrete harbor sections weighing over 1200 tonnes (1320 tons). Bigelow and Edmondson (4) and Kinsman (41) report that wave action has been known to move rocks weighing over 100 kg (200 lb) at depths of 30 m (100 ft), and that even much less noticeable swells have moved 0.5 kg (1 lb) rocks into lobster pots at depths of 55 m (180 ft). Thus the selection of design waves for outfall construction is clearly critical.

Rigorous analysis of wave forces requires a combination of empirical observations and simplifying assumptions as to water surface profiles, particle velocities, particle accelerations, and particle movements. Gravity waves--that begin as small capillary waves clutched by the wind from the sea surface, grow to seas with significant heights, and continue as low swells after the winds die down--are most important because of the energy they transmit (see Figure 2-4).

Particle movements within gravity waves are determined from observations at specific points (Eulerian) or from their trajectory (Lagrangian) (see Figure 2-5). Although many series of hydrodynamic equations for waves have been developed during the nineteenth and twentieth centuries, the linear approximations of Airy in 1845 are adequate for the design and construction of coastal structures (13). These are summarized in Table 2-2 and compared with nonlinear theories in Figure 2-6.

The elevation or datum from which waves depart is also variable. Tidal ranges are shown in Figure A-8 (54); long-term rises in sea level due to global warming and ice melting are shown in Figure 2-7. Local sea level changes are due to isostatic adjustment caused by the melting of ice sheets, local vertical displacement caused by tectonic or seismic activity, consolidation of deltaic sediments, or withdrawal of groundwater or oil and gas. The

(text cont. on p. 15)

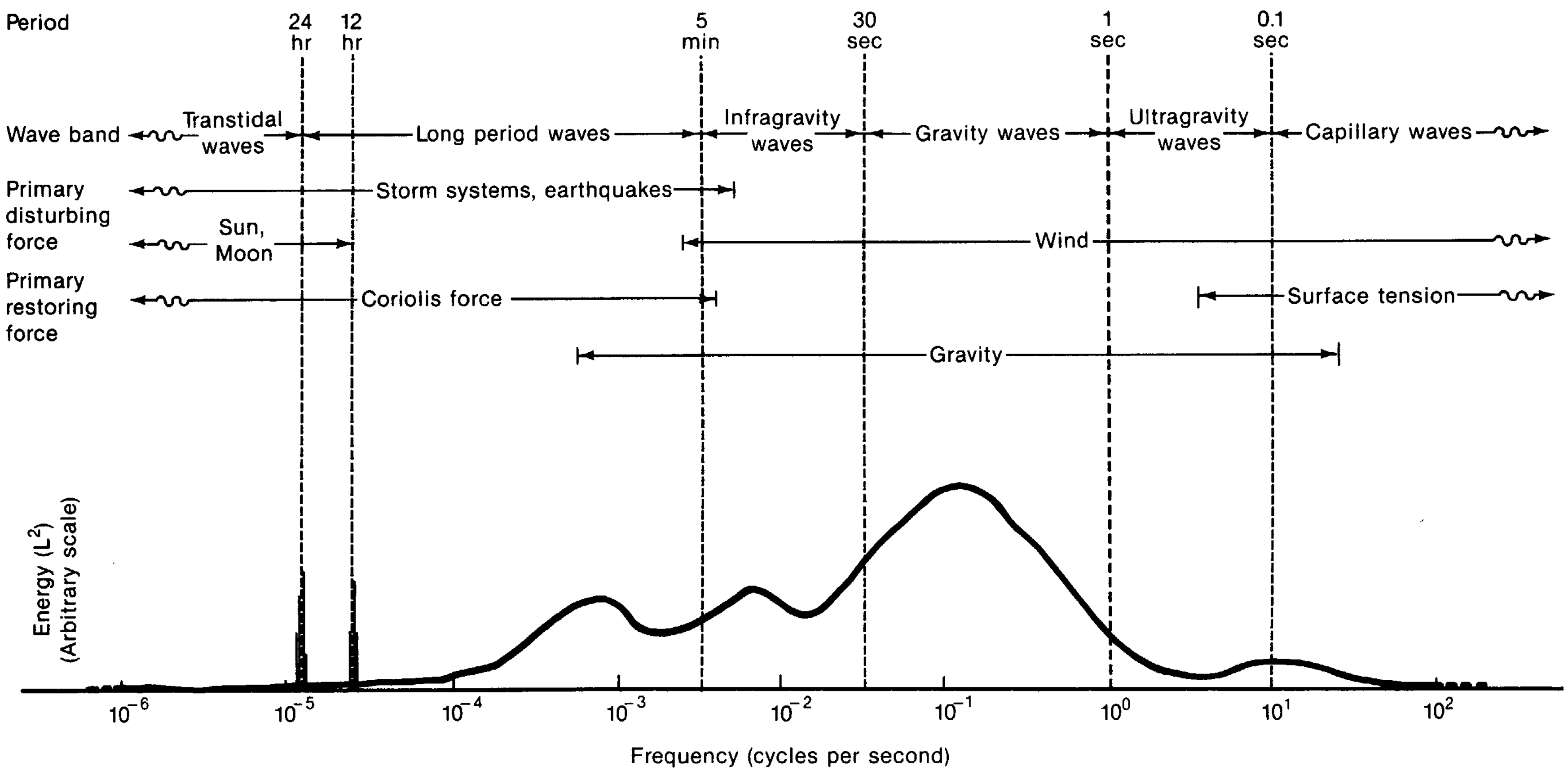


Fig. 2-4. Estimated Energy Spectrum for Ocean Surface Waves.
Source: Kinsman (41).

Fig. 2-5. Particle Paths (Lagrangian viewpoint) and Streamlines (Eulerian viewpoint) for a Sinusoidial Progressive (gravity) Wave. Source: Adapted from Kinsman (41).

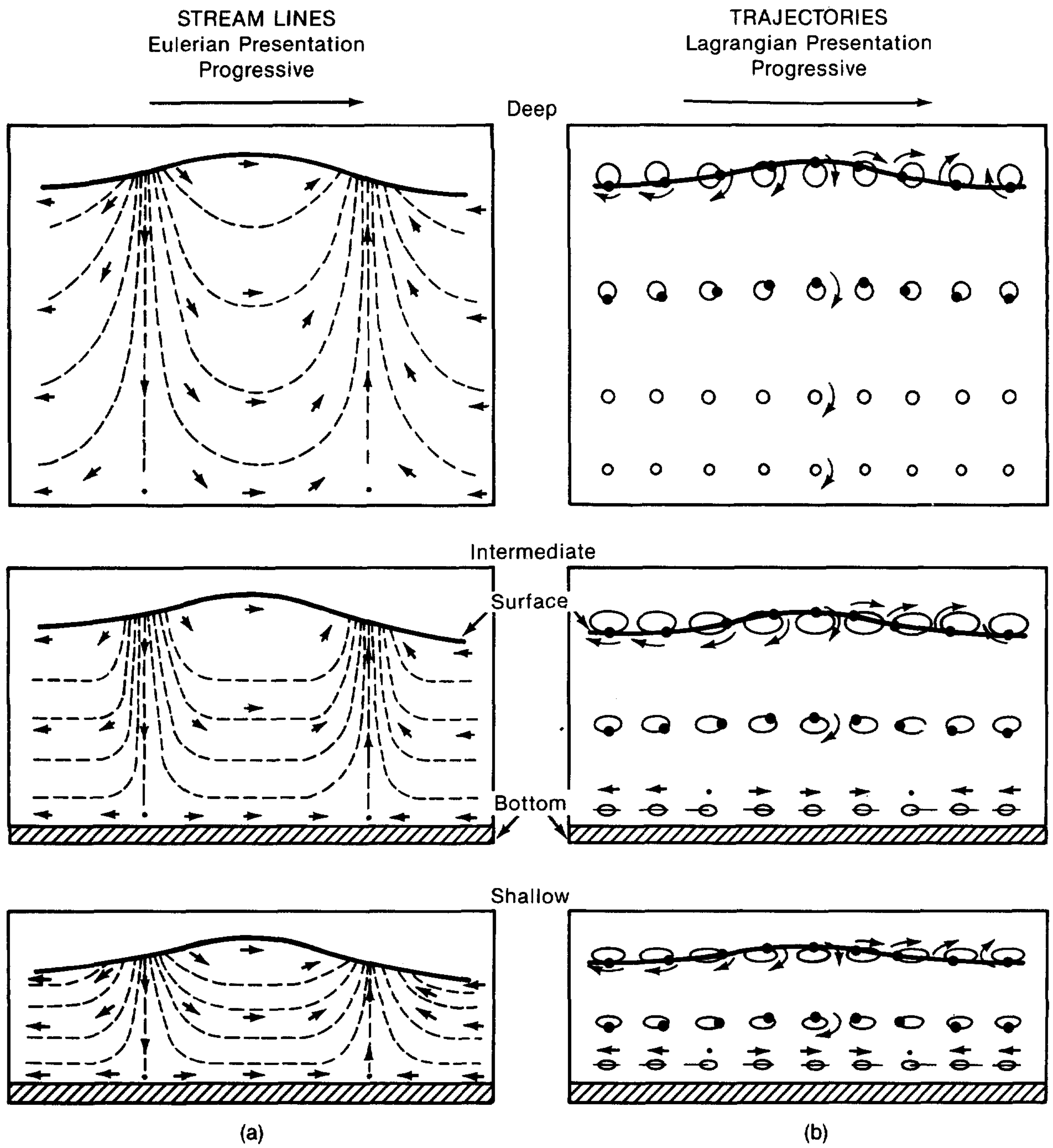


TABLE 2-2

Summary of Linear (Airy) Wave Theory--Wave Characteristics

RELATIVE DEPTH	SHALLOW WATER $\frac{d}{L} < \frac{1}{25}$	TRANSITIONAL WATER $\frac{1}{25} < \frac{d}{L} < \frac{1}{2}$	DEEP WATER $\frac{d}{L} > \frac{1}{2}$
1. Wave profile	Same As →	$\eta = \frac{H}{2} \cos\left[\frac{2\pi x}{L} - \frac{2\pi t}{T}\right] = \frac{H}{2} \cos\theta$	← Same As
2. Wave celerity	$C = \frac{L}{T} = \sqrt{gd}$	$C = \frac{L}{T} = \frac{gT}{2\pi} \tanh\left(\frac{2\pi d}{L}\right)$	$C = C_0 = \frac{L}{T} = \frac{gT}{2\pi}$
3. Wavelength	$L = T\sqrt{gd} = CT$	$L = \frac{gT^2}{2\pi} \tanh\left(\frac{2\pi d}{L}\right)$	$L = L_0 = \frac{gT^2}{2\pi} = C_0 T$
4. Group velocity	$C_g = C = \sqrt{gd}$	$C_g = nC = \frac{1}{2}\left[1 + \frac{4\pi d/L}{\sinh(4\pi d/L)}\right] \cdot C$	$C_g = \frac{1}{2}C = \frac{gT}{4\pi}$
5. Water Particle Velocity (a) Horizontal	$u = \frac{H}{2}\sqrt{\frac{g}{d}} \cos\theta$	$u = \frac{H}{2} \frac{gT}{L} \frac{\cosh[2\pi(z+d)/L]}{\cosh(2\pi d/L)} \cos\theta$	$u = \frac{\pi H}{T} e^{\frac{2\pi z}{L}} \cos\theta$
(b) Vertical	$w = \frac{H\pi}{T}\left(1 + \frac{z}{d}\right) \sin\theta$	$w = \frac{H}{2} \frac{gT}{L} \frac{\sinh[2\pi(z+d)/L]}{\cosh(2\pi d/L)} \sin\theta$	$w = \frac{\pi H}{T} e^{\frac{2\pi z}{L}} \sin\theta$
6. Water Particle Accelerations (a) Horizontal	$\alpha_x = \frac{H\pi}{T}\sqrt{\frac{g}{d}} \sin\theta$	$\alpha_x = \frac{g\pi H}{L} \frac{\cosh[2\pi(z+d)/L]}{\cosh(2\pi d/L)} \sin\theta$	$\alpha_x = 2H\left(\frac{\pi}{T}\right)^2 e^{\frac{2\pi z}{L}} \sin\theta$
(b) Vertical	$\alpha_z = -2H\left(\frac{\pi}{T}\right)^2\left(1 + \frac{z}{d}\right) \cos\theta$	$\alpha_z = -\frac{g\pi H}{L} \frac{\sinh[2\pi(z+d)/L]}{\cosh(2\pi d/L)} \cos\theta$	$\alpha_z = -2H\left(\frac{\pi}{T}\right)^2 e^{\frac{2\pi z}{L}} \cos\theta$
7. Water Particle Displacements (a) Horizontal	$\xi = -\frac{HT}{4\pi}\sqrt{\frac{g}{d}} \sin\theta$	$\xi = -\frac{H}{2} \frac{\cosh[2\pi(z+d)/L]}{\sinh(2\pi d/L)} \sin\theta$	$\xi = -\frac{H}{2} e^{\frac{2\pi z}{L}} \sin\theta$
(b) Vertical	$\zeta = \frac{H}{2}\left(1 + \frac{z}{d}\right) \cos\theta$	$\zeta = \frac{H}{2} \frac{\sinh[2\pi(z+d)/L]}{\sinh(2\pi d/L)} \cos\theta$	$\zeta = \frac{H}{2} e^{\frac{2\pi z}{L}} \cos\theta$
8. Subsurface Pressure	$p = \rho g (\eta - z)$	$p = \rho g \eta \frac{\cosh[2\pi(z+d)/L]}{\cosh(2\pi d/L)} - \rho g z$	$p = \rho g \eta e^{\frac{2\pi z}{L}} - \rho g z$

Source: CERC (13).

Fig. 2-6. Regions of Validity for Linear and Nonlinear Wave Theories.
Source: Adapted from CERC (13).

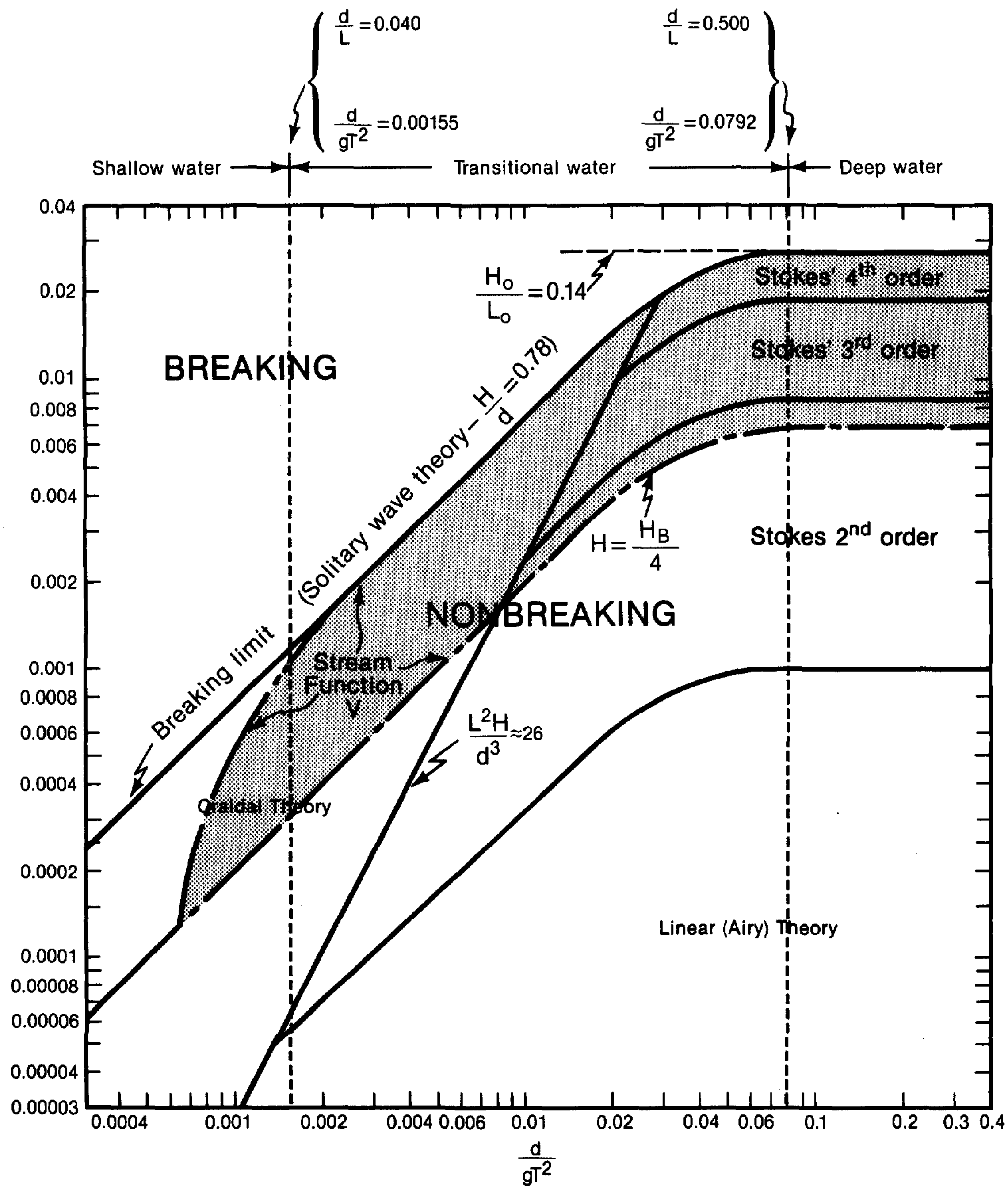
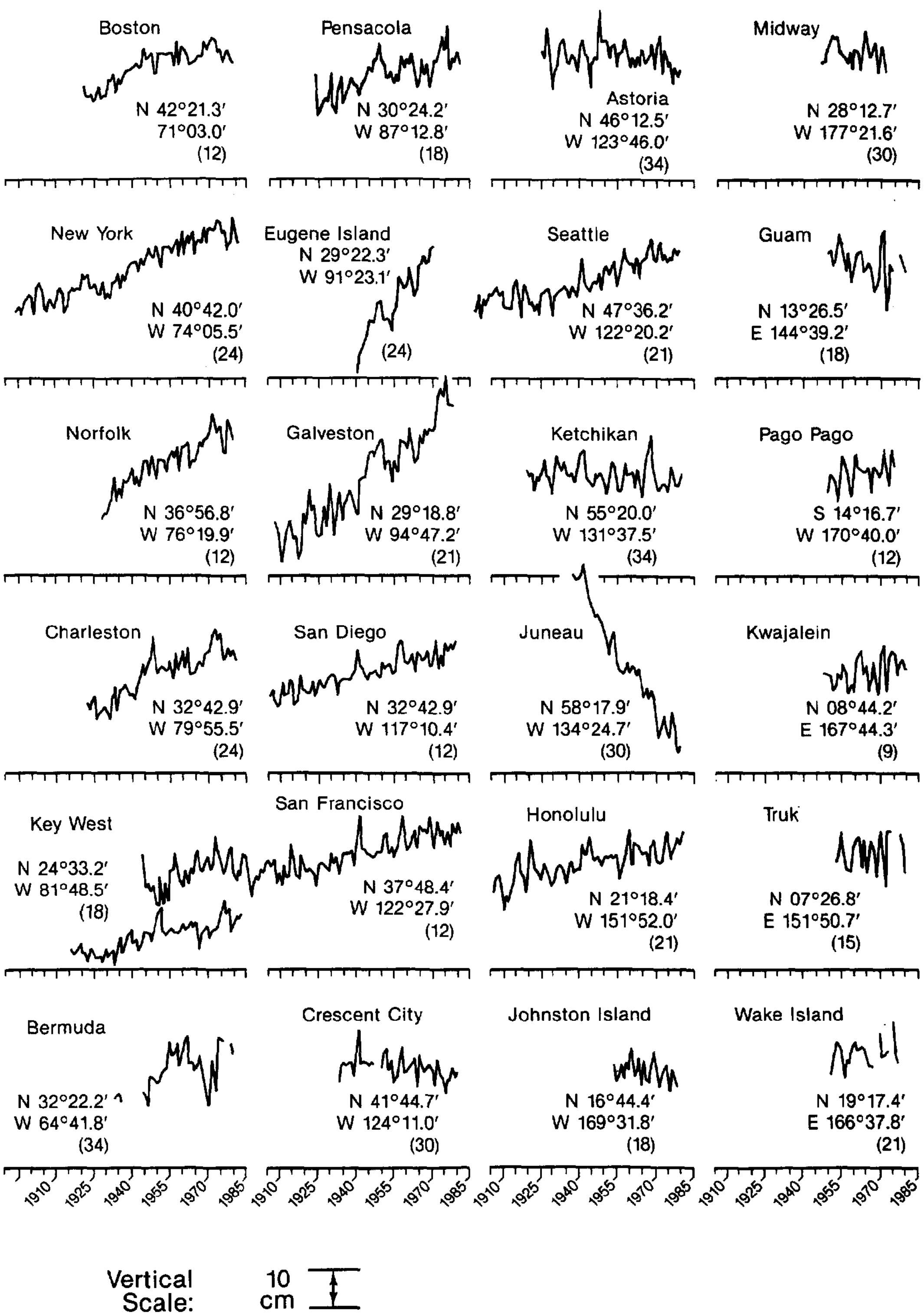


Fig. 2-7. Annual Average Sea Levels and (in parentheses) Fortnightly Ranges in cm at U.S. Tide Stations. Source: Adapted from Hicks (35).



local movements are ordinarily the most important, and local tide-gauge measurements are needed to assess their engineering significance. Many of these displacements vary widely over short distances. According to Fleming (24), archaeological evidence from ancient harbors indicates that the average uplift at the western end of Crete over the last 2,000 years has been 50 cm (20 in) per century and average sinking has been 20 cm (8 in) per century at the eastern end. Such movements are large enough to affect the designs for gravity sewers and interceptors.

In any event, all continents are most unstable at their margins (which is why the margins are located where they are). Most sea level changes are small (35). There are, however, large-scale land surface movements, which range from 40 cm per century rise in Scandinavia (where they are the result of melting and removing the weight of continental ice sheets) to 200 cm per century fall in Japan (where they are caused by tectonic movements) (K. O. Emery, personal communication, 1985). The effects of these movements are multiplied by slopes of coastal plain areas. Pilkey (55) demonstrates that each meter of sea level rise along the U.S. East Coast will result in a 100 to 10,000 (or average 1,000) meter advance of the surf zone. Similar ratios apply to other midlatitude coastal plains within about 20 km (12 mi) of the shoreline. Coastal engineering and sanitation works are accordingly temporary solutions to environmental problems. Storm surge is a major consideration. Other factors that contribute to changing water levels include semi-diurnal (lunar), diurnal (solar), and fortnightly (lunar) tides; direct winds; barometric pressure; Coriolis effect; rainfall; gravity waves and associated setup; and storm motion. For example, maximum recorded departures from mean high water (MHW) and mean low water (MLW) levels are listed in Table 2-3.

Highly destructive waves are generally caused by submarine landslides associated with earthquakes or by volcanic explosions. The Lisbon earthquake of November 1, 1755, for example, caused a locally destructive wave that reached a height of 12 m (40 ft) and that continued across the Atlantic and reached the West Indies with a still disastrous height of 4 to 6 m (12 to 18 ft) (65). In 1958, a rockfall triggered by an earthquake caused a local surge up to 530 m (1,140 feet) on the opposite side of Lituya Bay, Alaska (59). Volcanic explosions that were responsible for the formation of the caldera at Thera (Santorini) in the Aegean Sea during the thirteenth or fourteenth century B.C. are credited with causing a major and perhaps fatal disruption of Minoan civilization, the Exodus, and a series of destructive waves from Minoan Crete to the Phoenician ports of the eastern Mediterranean. A similar but smaller explosion on Mount Krakatoa in Indonesia in 1883 caused a destructive wave that traveled from the Sunda Strait across the Indian Ocean into the Atlantic Ocean as far as the English Channel (44, 51, 65).

The accepted approach to selecting a design wave for coastal engineering works at present is to consider predicted rather than historical conditions. The best available working bases for design are the simplifying assumptions and worked examples using linear wave theory and the storm surge predictions of the U.S. Army Corps of Engineers and the National Oceanic and Atmospheric Administration, respectively (13).

TABLE 2-3

Maximum Recorded Departures from Mean High Water and Mean Low Water along U.S. Coastlines

Location	Meters (feet) above MHW	Meters (feet) below MLW
North Atlantic (Maine to North Carolina)	1.04 to 3.99 (3.4 to 13.1)	0.52 to 2.04 (1.7 to 6.7)
South Atlantic (So. Carolina to Florida)	0.91 to 2.35 (3.0 to 7.7)	0.49 to 1.34 (1.6 to 4.4)
Gulf Coast (Florida to Texas)	0.67 to 3.08 (2.2 to 10.1)	0.46 to 1.62 (1.5 to 5.3)
Pacific Coast (California to Washington)	0.46 to 0.88 (1.5 to 2.9)	0.79 to 1.43 (2.6 to 4.7)
Alaska	1.01 to 5.09 (3.3 to 16.7)	0.73 to 1.83 (2.4 to 6.0)

Source: CERC (13).

Standing waves or seiches may also be locally important. These occur when a single energy input such as a landslide is applied, in which case the wave is damped, as in Lituya Bay (59), or when the natural period of an embayment is essentially equal to a tidal period, in which case the wave is magnified, as in the Bay of Fundy (54).

2.2.3 Marine Geology and Sedimentation

The principal bathymetric features of the continental border and ocean floor are shown in Figure 2-8 from Smith and Brown (62). Liquid wastes and sludges discharged onto the continental shelf or barged further offshore are integrated into the hydrographic and sedimentary regimes of the sea.

Approximately 70 percent of continental shelf areas consist of terrestrial deposits laid down between 10,000 and 25,000 years ago on river and coastal plains (10). These deposits were subsequently overlain by more recent marine deposits, most of which are riverborne bedload and suspended sediments that flocculate, settle, and add to the bedload when the salinity reaches about 4 parts per thousand ($^{o}/oo$) in estuaries (72). The usually polluted bedload is dredged from navigation channels and harbors and is often dumped offshore together with sewage sludge, building and construction debris, and industrial wastes.

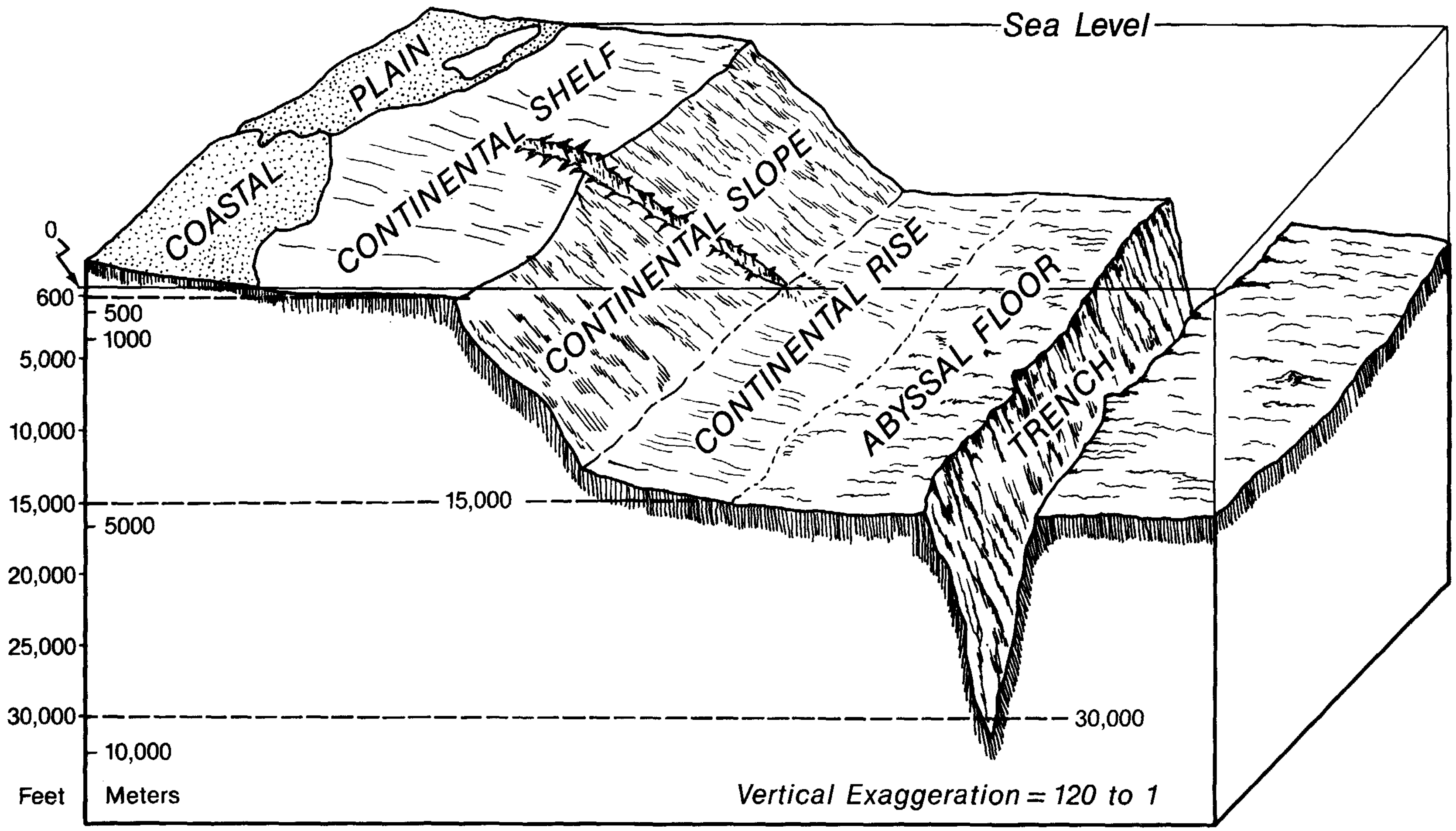


Fig. 2-8. Relationship of Major Continental Border and Oceanic Features. Note large vertical exaggeration. The Continental Shelf actually averages less than one degree gradient, the Slope about 4 degrees, and the Rise somewhat less. Source: Adapted from Smith and Brown (62).

Occasionally, bottom sediments accumulate on an increasing slope, which becomes unstable and slumps under static or seismic conditions or forms a fast-moving turbidity current along the bottom. The latter can reach estimated velocities of some 30 km/h (15 knots), and are credited with sustaining, if not creating, submarine canyons (59). Turbidity currents also can remove sewage sludge downslope from an outfall (30, 37).

The seafloor is composed primarily of unconsolidated sediments of organic and inorganic origin. The inorganic fraction of continental shelf sediments originates predominantly from the land and is introduced by a variety of means: rivers and streams carrying both particulate and dissolved material; sheet runoff; shoreline erosion; glaciers and sea ice; winds; volcanoes; and biological activity (65).

There are three classifications of naturally occurring marine particles: lithogenous, biogenic, and authigenic. Lithogenous particles are those derived from the weathering of rock and soil. These may be sand and silt sediments formed by the disintegration and mechanical breakdown of rock into smaller fragments, or they may result from the chemical decomposition of rock particles by air and water. Biogenous particles are derived from marine plants and animals. Various organisms remove calcium carbonate and silica from seawater to build their skeletons, which, upon the organism's death, become the principal constituents in sediments known as oozes, defined as sediment containing more than 30% biogenous constituents by volume (29). Insoluble fragments of bones, teeth, and shells account for the larger particles in biogenic sediments. Authigenic minerals such as manganese or phosphorite particles or nodules are formed by chemical precipitation that occurs in seawater or within sediments. Flocculation, which occurs when very fine clay particles come in contact with the dissolved salts in seawater, is responsible for much of the sediment deposited near river mouths.

Sediments are generally classified according to their grain size either as clay, silt, sand, gravel, cobbles, or boulders. If their densities are uniform, size and shape determine where they will be found in the water column during transportation and the distance that they will travel before settling to the bottom. In addition, size determines the degree of sorting that can be accomplished by the physical processes involved in their transport, such as waves and currents. A well-sorted sediment such as beach sand contains particles of a limited size range since the other sizes have been removed, usually by physical means. Poorly sorted sediments indicate that little mechanical energy was available to separate the different sizes.

The distribution of sediments throughout the ocean is related to their origin. Land-derived sediments accumulate fairly rapidly (in geologic time) near the continents and cover about 25 percent of the ocean bottom. The remainder of the ocean floor is covered primarily by pelagic deposits consisting largely of biogenous particles. Most land-derived sediments are not transported to deep ocean basins. Sewage effluents contain organic particles of varying density and size, mostly with low settling velocities. Effluents from different sources and treatment plants can vary substantially in their distribution of particle settling rates (see Chapter 3). The speed at which particles settle depends on their specific gravity, size, and shape, and on the specific gravity and viscosity of the water (65).

Waves and currents are the primary mechanisms for sediment transport in coastal waters, either in suspension or as bed load carried by traction (rolling and sliding) and saltation (bouncing). When current velocities are no longer able to maintain the particles in suspension, sedimentation takes place. Silt and clay will occur in suspension near the seafloor, with larger particles near the bottom, depending upon the current velocity and the roughness of the bottom (65).

Land-derived sediments are introduced not only by rivers, but also by wave action along the coasts, which causes slumping, sliding, grinding, and breaking of rock along the shoreline. Durable rocks are slowly eroded to sedimentary material, whereas sandstones and glacial drift are readily eroded in large quantities. Along limestone coasts, terrigenous sediments are generally less abundant than the skeletal remains of calcareous marine organisms.

Coarse sediment is dragged out to sea by rip currents, backwash, and undertow and is deposited in shallow depths offshore. As each wave crest passes a given reference line on the bottom, the forward drag of the wave crest moves sand grains shoreward; with each passing wave trough, sand is dragged seaward. This back-and-forth motion causes sand ripples to be formed. Rip currents occur when waves cause a buildup of water near the beach. Returning flows can channel large amounts of sedimentary material seaward from the shore.

The landforms found along the shore are shaped largely by the movement of water and the amount and type of sediment transported. Refracting waves together with longshore currents tend to transport sand and pebbles laterally along the coast. This littoral drift plays a role in the formation of beach ridges, sand spits, attached bars, and coastal dunes.

The degree of erosion (scour) and deposition (fill) of sediments near river mouths varies according to season. Scour is more prevalent during periods of increasing riverine discharge, whereas fill occurs with decreasing discharge. Offshore deposition of sediments occurs most often after storms, whereas inshore deposition is most common during periods of low wave energy.

Seafloor topography is another factor influencing the deposition of sediments within the oceans. High topographic features, on the one hand, generally attract relatively coarse or unconsolidated material transported by waves and currents. Depressions and basins, on the other hand, contain fine-grained material that has been transported from the continental shelf to deeper water at a slow rate. Thus, the rate of sedimentation is small on isolated topographic highs, whereas it is relatively great on, and immediately below, the slopes of major ocean basins and in nearshore basins (65).

In the open ocean, the character of a sediment in a given location is determined by the topography surrounding the site of deposition, the depth, the physical and chemical conditions in the overlying water, and the distance to the sources of inorganic material. Sediment samples can therefore provide considerable information on bottom currents and other environmental factors in the area of deposition.

After deposition, sediments can be resuspended by wave action or tidal currents with competent velocities (see Figure 2-9). For preliminary design purposes, rough estimates of these velocities can be made from grain-size distributions of the sediments (16, 63, 69).

2.2.4 Chemistry of Seawater and Suspended Sediments

Seawater is a complex solution of elements, organic and inorganic ions and compounds, and gases. The total dissolved inorganic salts (salinity) are conventionally expressed in parts per thousand (^{o}/oo or, in some recent reports, ppt). Salinity may be calculated from its chlorinity (where S, ^{o}/oo = 1.805 x Cl ^{o}/oo + 0.03). Alternatively, the accuracy and precision of conductivity measurements are sufficient for outfall design and monitoring purposes. Standard seawater is conventionally taken as having a salinity of 35.00 ^{o}/oo, which is typical for the open ocean. Locally, seawater salinity varies from a few parts per thousand near major rivers or melting ice to about 37 ^{o}/oo in the eastern Mediterranean and Aegean seas and 40 ^{o}/oo in the Red Sea.

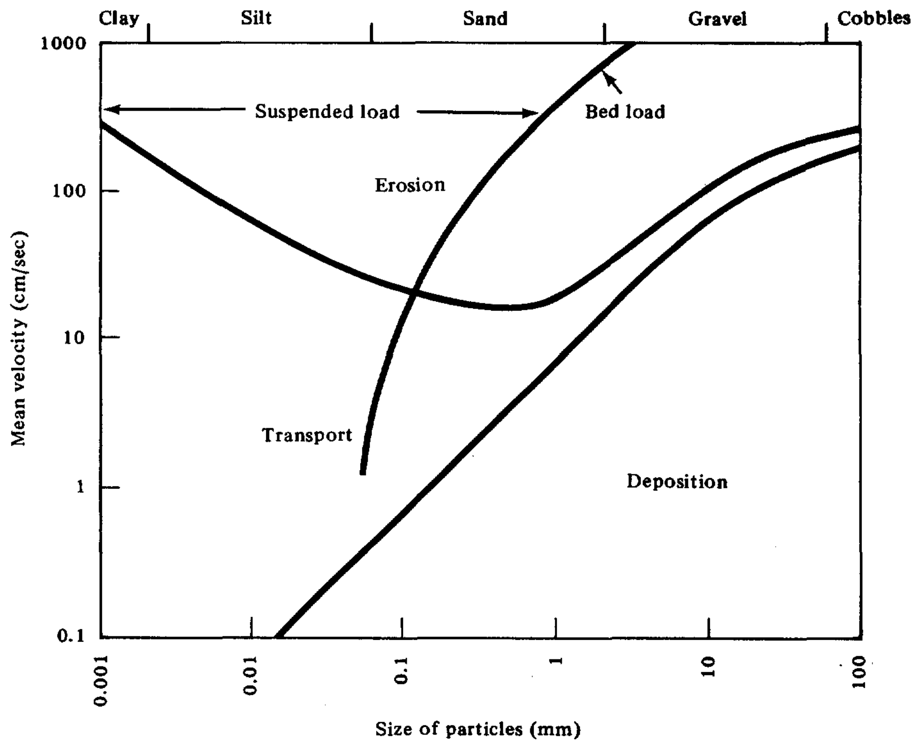
Seawater density (ρ) is a nonlinear function of its salinity (S) and temperature (T), and is conventionally expressed as sigma-T, where $\sigma_t = (\rho-1)\,1000$, so that for $\rho = 1.02468$, $\sigma_t = 24.68$. Because of density stratification in the sea, waters with different salinities and temperatures tend to mix along surfaces or layers of equal σ_t. (Nomographs for determining seawater density are presented in Appendix D.)

Carbon dioxide, oxygen, and nitrogen are the principal dissolved gases in seawater. These gases are present in the atmosphere and are dissolved into surface waters by the constant stirring of winds and waves at the air-sea interface. Temperature and salinity control the amount of gas that can be dissolved in seawater: when either of these increase, the amount of gas that can be dissolved decreases. Tables listing oxygen solubility values for ranges of 0-50^o C and 0 to 20 ^{o}/oo Cl (36 ^{o}/oo S) are presented in Standard Methods (1).

Seawater contains trace amounts of the majority of elements known to man (see Figure 2-10 and Table 2-4). Although most elements occur in very small concentrations in seawater, these concentrations can vary, depending on natural and cultural inputs. The residence times of different elements in the water column depend upon reactions between the element and other physical, chemical, and biological factors in the marine environment.

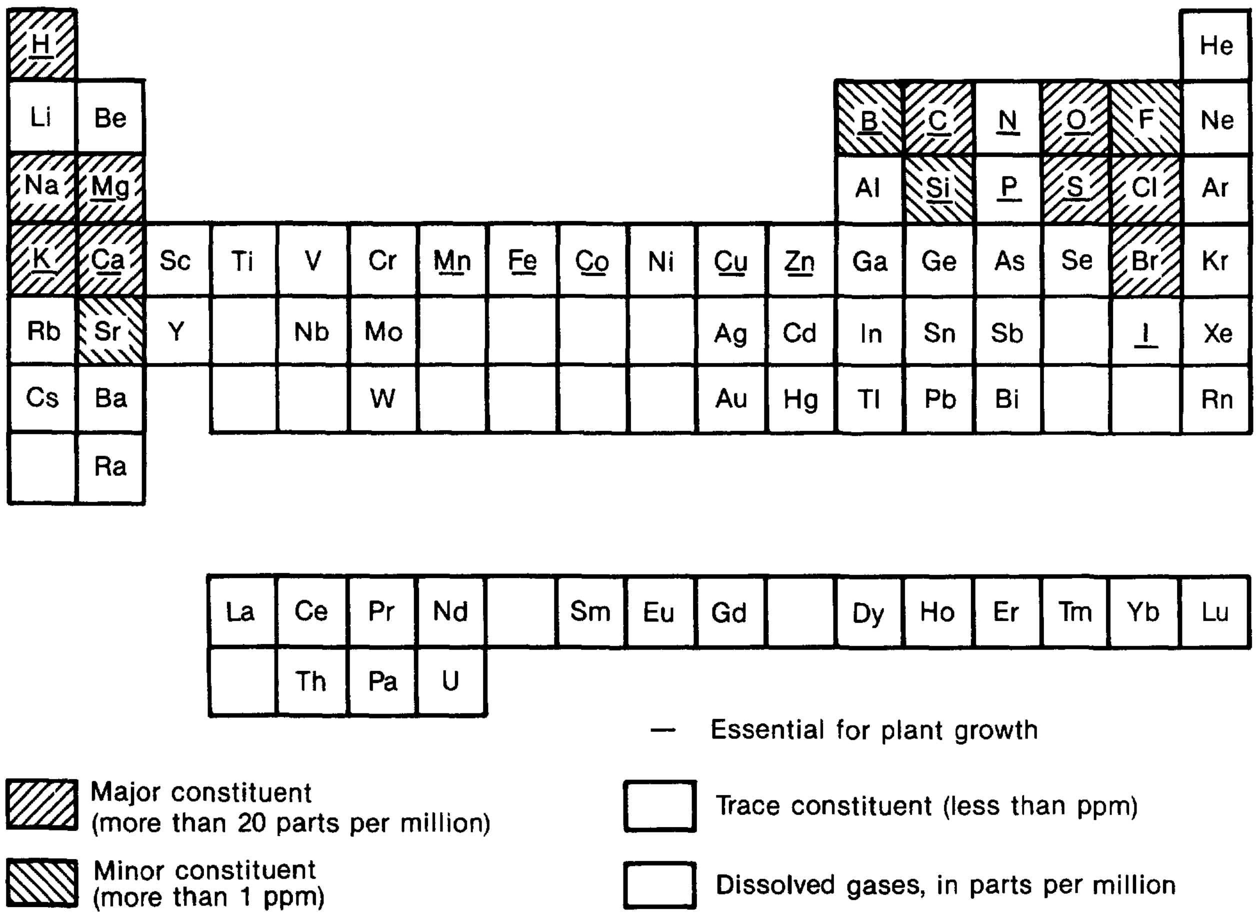
The dissolved organic matter in seawater comes from decaying plants and animals and from waste substances discharged by animals, people, cities, agriculture, and industries. Dissolved organics are present in seawater in small but variable concentrations, ranging from 0 to 6 mg/l (58). Concentrations are higher in areas of high biological productivity or waste discharges. Dissolved organic matter includes organic carbon, carbohydrates,

Fig. 2-9. Effects of Current Velocity on Sediments



Source: Adapted from Trask (69, p. 10) and Sternberg (63).

Fig. 2-10. Elements detected in sea salts. Most non-conservative elements are involved in biological processes (underlined).



Source: Gross (29).

TABLE 2-4

The Constituents in Solution in Ocean Water Having a Salinity of 35.00 $^{o}/oo$

	Grams per Kilo	Grams per liter at 20° C (specific gravity 1.025)
Total salts	35.1	36.0
Sodium	10.77	11.1
Magnesium	1.30	1.33
Calcium	0.409	0.42
Potassium	0.388	0.39
Strontium	0.010	0.01
Chloride	19.37	19.8
Sulphate as SO_4	2.71	2.76
Bromide	0.065	0.066
Boric acid as H_3BO_3	0.026	0.026
Carbon:		
Present as bicarbonate, carbonate, and molecular carbon dioxide	0.023 g at pH 8.4 0.025 g at pH 8.2 0.026 g at pH 8.0 0.027 g at pH 7.8	
As dissolved organic matter	0.001-0.0025 g	
Oxygen (where in equilibrium with the atmosphere at 15° C)	0.008 g. = 5.8 cm^3 per l	
Nitrogen (where in equilibrium with the atmosphere at 15° C)	0.013 g. = 10.5 cm^3 N_2 per l + 0.28 cm^3 argon, etc.	
Other elements	0.005	

Note: Calculated from a chlorinity 19.37, where S = 1.805 Cl + 0.03.

Source: Harvey (34).

amino acids, organic acids, proteins, vitamins, and nutrients (nitrogen and phosphorus), which are later oxidized.

Suspended particulate matter in seawater includes organic detritus, some complexes of organic material, and fine-grained minerals. The amount of suspended material in any particular location varies greatly because it is influenced by local geography, biological productivity, and atmospheric conditions. Suspended materials are unevenly distributed throughout the water column, but are especially prevalent at the surface layer. Essentially all of the heavy metals, toxic hydrocarbons, and microorganisms associated with waste disposal are adsorbed onto the suspended solids. Very fine sand, silt, and clays often remain suspended near the bottom in a nepheloid layer.

Seawater also contains a number of precipitated materials. One of these is calcium carbonate, which tends to precipitate when salinity and temperature increase and carbon dioxide content decreases. This reaction is most likely to occur in areas of active photosynthesis and high or rising temperatures, as in coastal tropical waters. Conversely, low photosynthesis and low temperatures favor the solution of these elements. The other precipitation reactions that occur in seawater are less common.

The chemical interactions that occur between the seawater and sediments or suspended sediments can be summarized as follows: (1) dissolution of certain constituents of the sedimentary particles; (2) adsorption of seawater constituents onto the sediments; (3) ionic exchange; and (4) reactions that form new substances. The variations in the minor and trace elements and organic chemical composition of water in the coastal ocean depend upon the local rocks, soils, and plant and animal life, as well as the anthropogenic inputs that enter the sea. Whether suspended or dissolved, nutrient elements are taken up by marine plankton communities consisting of both phytoplankton and zooplankton in ratios of approximately 41:7:1 by weight and 106:16:1 by atoms for organic carbon:nitrogen:phosphorus, respectively (65).

2.2.5 Stirring and Mixing

Eckert's (18) explanation of dilution and dispersion in the sea--which he compares to adding cream to a cup of coffee--is particularly applicable to ocean outfalls. Three stages appear to take place when one fluid (cream) is introduced into another fluid of differing density (coffee). In the first stage, large volumes of the two fluids are distinctly visible, and there is a sharp gradient at the interface. If no motion is introduced, the boundaries persist for some time. The concentration gradient is high but, since the interfacial area is small, the average concentration gradient throughout the cup is small. The second, or intermediate, stage begins with stirring. The two fluids are distorted and the interfacial areas are increased. Since the concentration gradients are still visible and high, the average concentration gradient throughout the cup is high. In the final stage, the concentration gradient disappears rapidly and reduces to zero. This is mixing.

Space and time scales for stirring and mixing in the ocean are much larger than those in a coffee cup. Nonetheless, Eckert's coffee cup gives an idea of the difference between near-field (mostly advective stirring) and far-field dispersion and diffusion phenomena (mostly mixing that extends down to the molecular scale). Density stratification and vertical stability in the sea being what they are (see Figure 2-1), horizontal diffusion coefficients are many orders of magnitude greater than vertical ones, except near an outfall where stirring takes place and establishes the thickness and vertical stability of the initial sewage field. These factors are discussed in Chapter 4, which also presents examples of outfall design and calculations, some of which are quite complicated because of the assumptions on which they are based. However, for small open-end outfalls of, say, less than 1 m diameter, Brooks's analysis (8) shows that initial (stirring) dilution is essentially the depth divided by the diameter. Subsequent dilution takes place slowly, and site-specific rates for the disappearance of bacteria in the sea become the principal factor in functional design and operation (see the discussion below as well as that in Chapters 3 and 4). This is particularly true in developing countries where the following sequence of events seems to be typical: water supply improvements overtake the capacity of household or community sanitation systems, local nuisances are created, local sewers are built to discharge at the shoreline, larger community nuisances are created, health is threatened (though fortunately seldom affected), and a decision is made to collect and discharge the wastes beyond the surf zone. Thus, additional improvements in sewerage and wastewater reclamation become increasingly important.

In recent years, our understanding of and ability to predict physical dilution processes in the coastal and shallower areas of the sea have improved considerably. Garrett (27) points out, for example, that empirical observations and dimensional analysis of mixing in the English Channel, the Gulf of Maine, and the Bay of Fundy reveal that a critical value for H/U^3 (where H is the depth and U the velocity) of about 70 $m^{-2}s^3$ marks the transition between stratified and vertically mixed conditions. The H/U^3 parameter, originally suggested by Simpson and Hunter (61), appears to have bearing on waste discharge problems in estuarine and coastal waters, although the numerical threshold values are likely to be site-specific.

2.2.6 Bays, Estuaries, and Straits

A bay may be defined as "a well-marked indentation [in the coast] whose penetration [into land] is in such proportion to the width of its mouth as to contain land-locked waters and constitute more than a mere curvature of the coast" (70). Bays may be considered to be intermediate between estuaries and the coastal ocean. However, many so-called bays are actually estuaries, whereas others are merely indentations in the coast.

Because bays are somewhat isolated from the main body of ocean water, they provide a certain degree of protection from the effects of coastal storms. Thus, over the centuries, people have chosen to settle on bays. The size of these settlements, which has ranged from small waterfront communities to heavily populated port cities, has generally depended on a bay's aesthetic appeal and available resources. The physical and hydrographic processes that occur in bays are usually less dynamic than those that take place along open coastlines. Therefore, bays are usually characterized by comparatively restricted flushing, low current velocities, and high rates of sedimentation. As a result, bays tend to retain dissolved and suspended solid contaminants introduced into their waters for longer periods than do waters in open coastal areas. Since the organisms in bays may therefore be at greater risk, bays--particularly embayments that have a highly limited exchange with oceanic waters--are generally not suitable locations for outfalls.

Pritchard (56) defines an estuary as a semi-enclosed coastal body of water that has a free connection with the open sea and within which seawater is diluted with freshwater derived from land drainage. More simply, estuaries can be described as the tidal mouths of rivers. Many of the world's major ports are located on estuaries. Since they are typically areas of high biological productivity, estuaries usually sustain substantial fisheries.

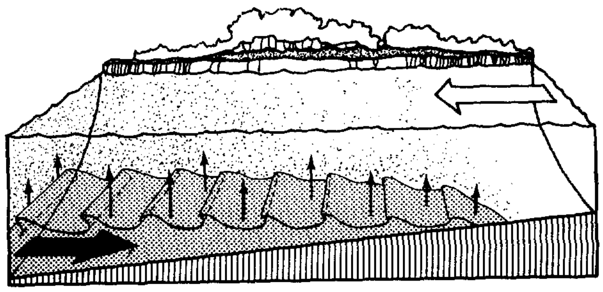
Estuarine conditions vary seasonally, particularly diurnally (2). The dynamic nature of an estuary is principally a function of its salinity regime. Whereas salinities in the open ocean typically average about 35 $^{o}/oo$, salinities within most estuaries tend to decrease steadily as one moves inland, the highest salinity being about 30 $^{o}/oo$ at the estuary mouth. The upper limit, or head, of an estuary is defined as the most landward intrusion of sea-derived salt. This is usually somewhere beyond the point at which salinity drops below 1/10 $^{o}/oo$. This area is characterized by a short transition zone in which the ratios of the major dissolved constituents change rapidly and chlorinity (the most common measure of salinity) drops to about 0.01 $^{o}/oo$. Although sea salts no longer influence the freshwater flows beyond this point, rivers can be affected by tides, most notably when river outflows, tidal ranges, and hydraulic geometry of the stream channel combine in the upstream movement of a steep, destructive wave or (tidal) bore (16).

Estuaries can be classified according to their water budgets, their geomorphology (drowned river valley, fjord, tectonic, or bar-built), or their hydrography.

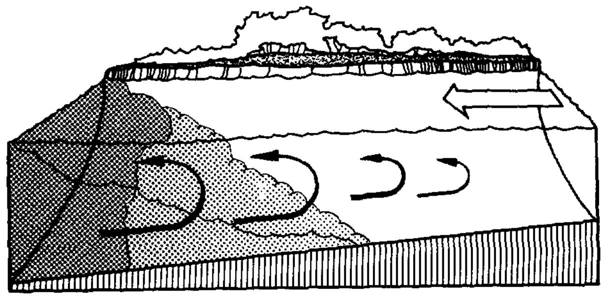
Pritchard's (56) hydrographic classification into (1) salt wedge, (2) partially stratified, or (3) vertically mixed estuaries is as useful as any. Their vertical structures, averaged and idealized for purposes of numerical modeling (see Section 4.4), are shown in Figure 2-11. In real estuaries, where no two successive tidal cycles are alike and where flows are modified by Coriolis effect and tidal pumping (22), current and salinity distributions are both seasonally and site-specific (see Appendix D).

Casual observations and interpretations of net downstream tidal flows within estuaries have led many to recommend that wastes be released only during ebb tide. This doesn't work. Both field and hydraulic model observations confirm the basic models of advection and mixing in estuaries, which indicate that average concentrations are fixed by the tidal excursion and volume and total daily waste loadings. The effects of discharging twice as much during half the time cannot be distinguished from the effects of a continuous discharge into an estuary (the ocean constitutes a discharge at the mouth). A putative exception may exist for a discharge at the mouth of an estuary into a long-shore current that is much larger than the tidal flow.

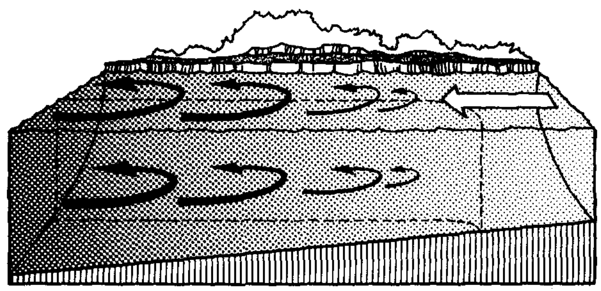
Fig. 2-11. A Hydrographic Classification of Estuaries.



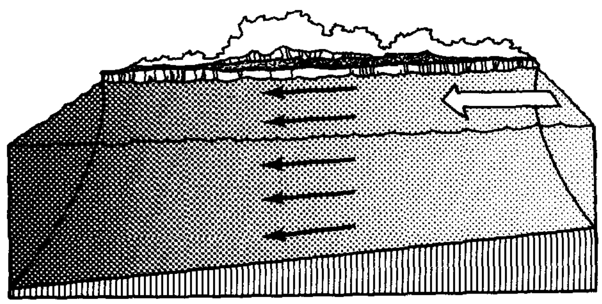
(1) Salt Wedge Estuary



(2) Partially Stratified Estuary



(3a) Vertically Homogeneous Estuary



(3b) Sectionally Homogeneous Estuary

Source: Pritchard (56).

A salt wedge is formed in estuaries where the river flow dominates the circulation pattern. If friction were absent, the freshwater/saltwater interface would be horizontal and extend upriver to where the riverbed was at sea level. However, because there is a small amount of friction between the layers, the interface slopes slightly downward as the wedge points upstream. Where there is a steep density gradient at the interface, mixing between the layers is restricted (56).

If the velocity of the seaward-moving freshwater exceeds a certain value, internal waves form at the interface. As these waves break at their crests, saltwater is entrained, or captured, into the upper (freshwater) layer. This is commonly known as a two-layer flow with entrainment, or a partially mixed estuary. In entrainment, saltwater mixes with the freshwater in the upper layer, but freshwater is not incorporated into the denser saltwater layer. In other words, the fluxes of both water and salinity are from the lower into the upper layer. Since the depth of the upper layer does not increase, the velocity of this seaward-flowing water is increased. A slow movement of the lower salt layer upstream compensates for the water lost by entrainment. Since there is a net flow of water out of the estuary, most mixing of surface water with seawater occurs near the mouth of the estuary. Fjords typically exhibit this type of saltwater-freshwater relationship.

In shallow estuaries, tidal currents often create vertical mixing that may extend throughout the water column. Even though salinity is relatively uniform vertically, there are still two layers separated by an area of no net motion between the seaward-flowing upper layer and the landward-flowing bottom layer. No marked salinity interface exists in these cases and salinities continuously increase from the surface to the bottom, the greatest difference occurring near the level of no net motion. The degree of stratification can vary greatly in this type of estuary, depending on the magnitude of tidal currents relative to the strength of the river flow. If tidal currents are very strong compared with the river flow, an estuary may exhibit vertically homogeneous properties with no measurable variation in salinity from top to bottom. Nonetheless, higher salinities will still be found at the mouth of the estuary, and will decrease toward the head.

Salinities may differ laterally in estuaries that are fairly wide relative to their depth as a result of the Coriolis effect. As the river flows toward the sea, the river water is deflected to the right in the northern hemisphere. Thus average salinities on the right are lower and there is a compensatory flow of more saline waters on the left. Where the ratio of width to depth is relatively small, salinities may not vary greatly across the estuary; such an estuary is said to be laterally homogeneous. These are for time-averaged conditions; the fine current detail can be very complex.

Another type of circulation may exist where a number of small estuaries are tributaries to a larger estuarine system. In Baltimore Harbor and Raritan Bay in the eastern United States, for example, a three-layer pattern of flow occurs. Water moves inland both at the surface and in the bottom layers and moves seaward at middepths (32).

Estuaries are important areas for the propagation, movement, and harvesting of marine and freshwater fish. Most estuarine organisms are euryhaline, that is, tolerant of a wide range of salinities. However, some totally estuarine species cannot survive in either freshwater or seawater (29, 58, 65).

Because of their limited mobility, the benthic communities within estuaries must be able to survive large extremes in hydrographic changes. However, species composition may change significantly between seasons if there is extreme variation in environmental conditions. Generally, the total number of species within the estuary declines to a minimum as salinities decrease to approximately 5 $^{o}/oo$; but then the number of species rises with the increase in freshwater. The nutrients in estuaries come from several sources: oceanic waters carried into the estuary; rainwater or the soils in the drainage basin; and waste products discharged or washed into the estuary.

Because the circulation within an estuary tends to keep the nutrients in the system, ample food is available for the biota. Estuaries can support large phytoplankton populations because of the nutrient availability and because periodic changes in current directions prevent the plankton from being washed out to sea. These phytoplankton populations support zooplankton populations, which in turn provide nourishment for higher trophic levels, including larval and juvenile fish.

Eutrophication is caused by nutrient enrichment of waters, particularly by nitrogen and phosphorous compounds. This enrichment stimulates an array of symptomatic changes, including: (1) increased production of algae and macrophytes and, possibly in the short term, fisheries; (2) in extreme cases, the deterioration of fisheries; and (3) deterioration of the physical water quality, characterized by an overgrowth of plant life and reduced oxygen levels (43).

Estuarine eutrophication depends on nutrient inputs, water conditions, and the estuary's sensitivity to change. In estuaries with clear water and limited vertical mixing, most of the nutrients are in the sediments and are used by the benthic communities. Although these estuaries may be considered biologically productive, a lack of nutrients in the water column often limits the total potential productivity. In contrast, turbid estuaries tend to have greater vertical mixing and are more productive; in this case the excess nutrients are in solution or are suspended along with sediment, organic detritus, and plankton.

The standing stock of nutrients in estuarine waters depends on: (1) streamflow and quality, anthropogenic sources, rain, and influx of seawater; (2) metabolic processes of estuarine populations; and (3) the role of sediments as sinks and sources of available nutrients (43).

The decomposition of plants, whose growth was stimulated by eutrophication, may give rise to oxygen deficiencies. More commonly, partial or total depletion of oxygen (anoxia) is caused by the decomposition of organic matter in sewage. Increased organic production may also be due to inputs of wastes containing inorganic nutrients. Although phytoplankton and other plants produce oxygen, the respiration of consumers eating the primary biomass eventually causes oxygen deficiencies.

Straits connect adjacent seas with each other or the ocean. Surface salinities on either side of a strait reflect differences in relative precipitation, runoff, and evaporation, and hence density. Straits are similar to estuaries in that their salinities and flows are highly stratified, but they differ in that tides are not an important factor in their stratification and in their general lack of harborage. Instead, their flows are governed by the slopes of the surface and interface, modified by the Coriolis effect, which, according to Defant (16), provide sufficient elevation heads to force the circulation. The Bosporus is the principal strait in which the two-layer flow has been considered important in sewage disposal (see Chapter 15). However, Marchetti (46) has proposed that gaseous material from European power plants be discharged into the Mediterranean outflow, which sinks and spreads in the Atlantic, in order to divert the carbon dioxide from the atmosphere.

2.3 The Living Sea

The living sea is the product of the tendency of living and nonliving matter to assume every possible form, and the capacity of living matter to utilize food in any form as a source of energy and materials for survival and multiplication (14).

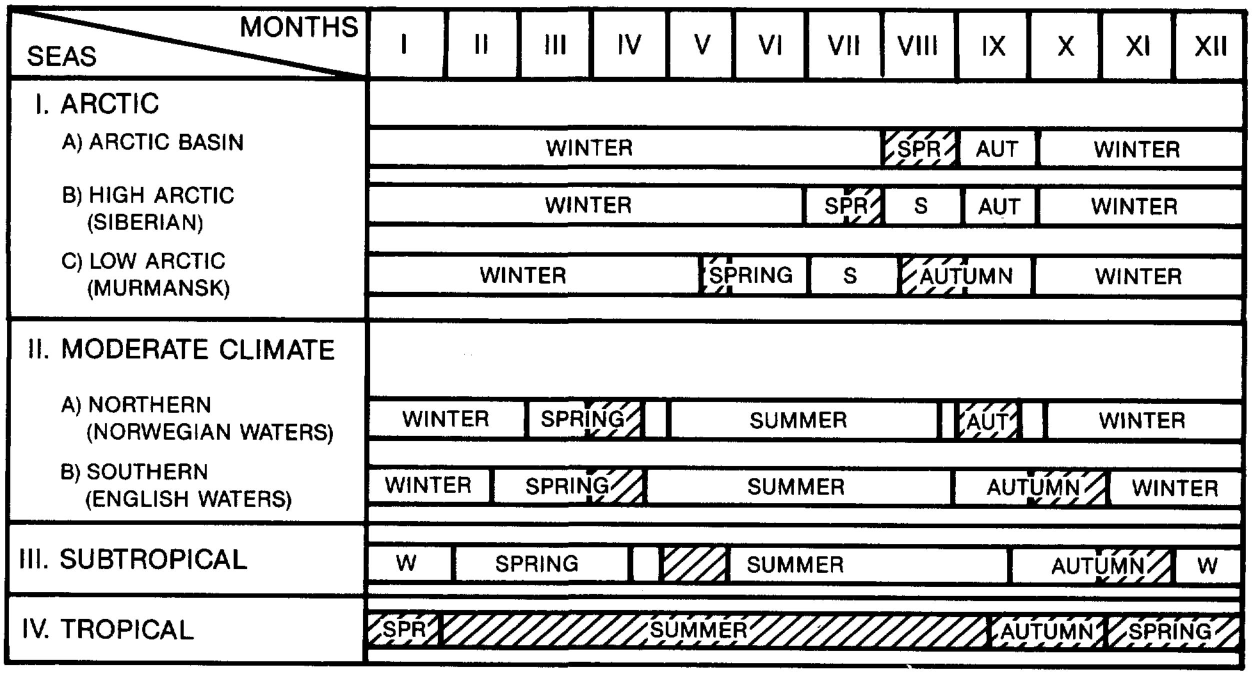
Productivity depends upon temperature, salinity, and nutrients. These factors in turn depend upon latitude (5, 38) and the average and seasonal variations in marine, coastal, and inland climates. The latter are summarized in Appendix Figures A-3 through A-6. The effects of climate and of upwelling (see Figure A-1) on the general geographic distribution of marine productivity are shown in Figure A-7.

The effects of latitude on the time of onset and duration of marine productivity rates and standing crop have been studied by Bogorov (5) (see Figure 2-12). Comparisons of various determinants in tropical and temperate ecosystems (see Table 2-5) have led to increasing concern over the effects of economic development on tropical ecosystems.

2.3.1 Ecological Relationships and Trophic Levels

The distribution and abundance of organisms in the sea depend on interactions between the organisms and their environment. Essentially all life in the sea, as on land, depends on the sun for energy. In the marine food chain, energy is transferred from plants through herbivores (grazers) to carnivores (animal eaters). In the ocean, tiny floating plants (phytoplankton) are the primary producers and thus constitute the first tropic level; herbivorous grazers (zooplankton) are the second; and the larger predators, such as fishes, are the third and higher levels. Consumer organisms store some of this energy in order to carry out life functions; the rest is passed

Fig. 2-12. Duration of Biological Seasons. Shaded areas indicate periods of high plankton production.



Source: Bogorov (5).

TABLE 2-5

Differences between Shallow Tropical Marine Ecosystems and Their Temperate Counterparts

Physical and Chemical		
Temperature	- mean	higher
	- range	much lower
Light	- mean	higher
	- range	lower
Salinity		higher
Oxygen		lower
Carbon dioxide		lower
Phosphorus		lower
Nitrogen		lower
Transparency		higher
Tides		lower
Community Structure		
Species diversity		
benthic invertebrates		higher
fish		higher
zooplankton		higher
phytoplankton		higher
macrophytes		lower
Mean size		smaller
Biomass		lower
Population density		lower
Population size		smaller
Predator population		higher
Colonial life forms		more
Zooplankton/phytoplankton ratio		greater
Eggs		smaller
Planktonic larvae		more
Meristic counts		lower
Macrophyte tax		more red and green algae
Phytoplankton tax		more flagellates
Zooplankton tax		more copepods
Lipids in plankton		lower
Biological functions		
Metabolic rates		higher
Benthic productivity		higher
Phytoplankton productivity		lower (except in upwelling areas)
Thermal tolerance		smaller range maximum nearer ambient
Breeding seasons		longer, successional
Asexual reproduction		higher
Growth rates		higher, more variable
Larval development		faster-fish, zooplankton slower-benthic invertebrates
Feeding habits		more specialized (esp. fish)
Niche width		smaller
Space sharing		higher
Minimum oxygen		closer to ambient
Algal invertebrate symbiosis		greater incidence
Poison defense		greater incidence
Cleaning symbiosis		greater incidence
Biological $CaCO_3$		greater precipitation
Color polymorphesin		more conspicuous
Evolution		higher rates

Source: Johannes and Betzer (38).

along to the next trophic level when the organism is eaten by another consumer. "Ecological efficiency" refers to the 5 to 50 percent of energy transferred by organisms between and within trophic levels. Thus organisms found at the bottom of a food chain are much more abundant than those at higher trophic levels. At each trophic level, energy is: (1) used to increase the size of the individuals in the population, (2) recycled by the decomposer chain, or (3) lost to the system in respiration (43).

Structure and organization within a community are determined by a number of factors, including productivity, diversity, dominance, and stability. Dominance can be measured in relation to biomass or standing crop, which is the amount of living matter expressed in terms of total weight per unit area. Species diversity may be measured simply by counting the number of different species in a collection or by weighting each species by its relative abundance (43). Species diversity is greater in coastal regions than in the open ocean because physical and chemical conditions are more variable and food is more abundant near the coasts (29). In the open ocean, temperature and latitude have the largest influence on diversity; thus, the warmer, tropical waters generally support a larger variety of lifeforms. Diversity may also vary with depth in the water column and the abundance of available nutrients. In benthic communities, different substrates also affect the diversity of habitats. If a system is perturbed, community structure may be altered by certain fast-growing species that can force out previously dominant residents (29).

2.3.2 Marine Productivity

The balance, succession, and distribution of marine life are governed by seasonal changes in temperature (6) and nutrient availability. In areas where nutrient supplies are low, productivity is also low. As in the case of species diversity, productivity in shallow coastal waters is much greater than in the open ocean.

Although nutrients are essential for primary production, excessive amounts may ultimately reduce local diversity. The excessive buildup of nitrogen and phosphorus compounds can lead to eutrophication, which is characterized by increased production of a few species of algae and phytoplankton and subsequent decline in other types of species. Because much of this plant material cannot be consumed by predators, it is instead decomposed by bacteria. This process reduces the available oxygen in the water column, and as the oxygen supply decreases, predatory species disappear. Sewage effluents can place a high demand on the available oxygen if disposed of in areas with limited water exchange. In areas of extreme nutrient loading and poor flushing, anoxic conditions may develop and biological life become limited primarily to anaerobic bacteria.

2.3.3 Plankton and Neuston

Plankton consists of marine organisms that float and drift with the currents and tides. Most marine organisms are planktonic during at least the early stages of their life cycle. Planktonic organisms that live at the ocean surface are called neuston.

Phytoplankton in the sea are composed principally of the one-celled diatoms, coccolithophorids, chysophyceans, and dinoflagellates (see Figure 2-13). Other phytoplankton include the blue-green algae, which are sometimes classified with bacteria.

Zooplankton consist of protozoans (foraminiferans, radiolarians, and ciliates), and nongelatinous and gelatinous metazoans. Nongelatinous forms include larval stages of crustaceans, copepods, euphausids, mysids, chaetognaths, molluscs, annelids, and rotifers. Gelatinous forms include the coelenterates, ctenophores, and tunicates. The smaller zooplankton include the flagellated protozoa and amoebae, which feed on bacteria. Selective grazing by zooplankton may govern the distribution of certain phytoplankton species.

Although many pollutants have the potential to influence plankton growth in the laboratory, field observations demonstrate that phytoplankton growth and reproduction are not noticeably affected by domestic sewage discharges. In extreme cases, natural phytoplankton blooms may deplete oxygen or produce toxic chemicals that may kill off fish and shellfish or possibly cause paralytic shellfish poisoning (7).

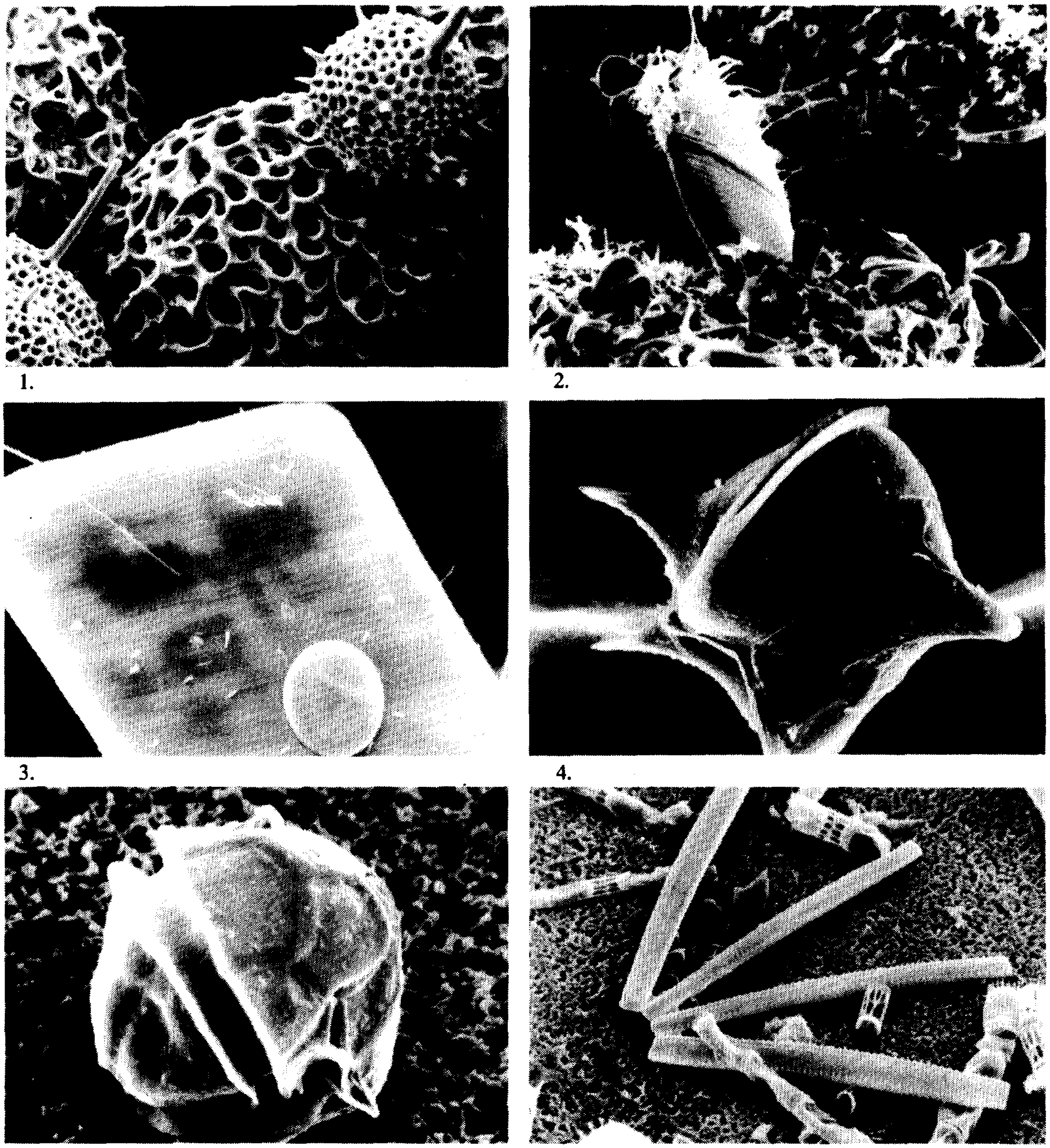
2.3.4 Bacteria

Although terrestrial bacteria may be a public health concern (11), marine bacteria are of greater ecological importance, because they are the primary decomposers of organic matter in the oceans (76). Marine bacteria utilize particulate organic matter found in the remains of plants and animals as well as dissolved organic matter, which originates, in part, during photosynthesis and excretion. With the breakdown (mineralization) of organics by bacteria, carbon dioxide and nutrients are released in the form of simple, soluble inorganic ions that can be utilized by plants and phytoplankton. The rates of mineralization of sewage constituents can be quite rapid, exceeding those for marine nutrients severalfold (31).

Two major types of bacteria are involved in the breakdown of various compounds in the oceans. Organotrophic bacteria use organic compounds as an energy source, while chemolithotrophic bacteria oxidize reduced inorganics such as ammonia and hydrogen sulfide into nitrate and sulfate for energy.

The numbers of bacteria present in the oceans are directly proportional to the amount of food (organic matter) in their environment. Most bacteria are found in the upper waters, or euphotic zone, where photosynthesis occurs and organic matter is produced, and at the bottom sediment surface, where organic materials accumulate. In general, the average concentrations of bacteria in sediments are greater than those in the euphotic coastal waters (approximately 10^8 - 10^9/ml in interstitial waters of nearshore sediments versus approximately 10^6/ml in coastal waters). Seasonal variations of bacteria (and zooplankton) in seawater tend to lag slightly behind phytoplankton concentrations.

Fig. 2-13. Variety of Plankton that May Be Affected by Pollution.



1—Radiolarian skeletons (210×).
2—Stalked diatoms, *Liconophora sp.* on whelk egg (265×).
3—Rare bacterial *(Flexibacterium sp.)* attachment to penuate diatoms, the rod-shaped *Rhabdonema adriaticum,* and the disc-shaped *Cocconeis, sp.* (950×).
4—The dynoflagellate *Peridinuum sp.* (485×).
5—The red-tide dynoflagellate, *Gonyaulaux tamarensis* (2100×).
6—Mixed plankton from Narragensett Bay, Rhode Island (700×).

Sewage and other natural organic materials discharged to the ocean are mostly dilute organic matter subject to degradation by bacteria. As particulates from sewage accumulate on the seafloor surface, they are mineralized at and below the surface of the sediments. Below the surface of the sediments, rapid bacterial metabolism of organic matter often results in depletion of the available oxygen. In such anaerobic conditions, organisms with greater oxygen requirements will be eliminated in favor of anaerobic bacteria. These anaerobic bacteria produce hydrogen sulfide, ammonia, and methane as by-products of anaerobic degradation of organic matter. In most coastal sediments, the number of aerobic bacteria declines sharply downward from the surface, and the proportion of anaerobic bacteria increases.

Given sufficient time, bacteria break down petroleum hydrocarbons, chlorinated hydrocarbons, and other complex organic compounds into products some of which are more toxic than the original compound. Bacteria have been implicated in the methylation of mercury, whereby the inorganic form of this element is converted into an organic form (see Chapter 15). Mercury in the marine environment originates from both natural sources and man's waste discharges. The toxicity of this otherwise relatively benign metal is increased by methylation (see Chapter 15).

2.3.5 Nekton

The nekton include free-swimming fish, squid (molluscs), and cetaceans (mammals). An enormous diversity of forms, sizes, and shapes have evolved as a result of the competition between these various animals for food and habitat.

The larger nekton require greater food supplies for energy but are usually fewer in number. Since man is a competitor in the marine food chain, the natural balance in fishery populations is upset by overfishing certain commercially important species. Ocean disposal of wastes may also affect the production and growth of fish. Some waste inputs, particularly toxic chemicals, could affect local fish distribution and productivity, but the mobility of the nekton allows them to avoid affected areas. Many other fish prefer to feed on food associated with disposal areas, and productivity and growth may actually be enhanced by nutrient and organic carbon inputs associated with domestic waste discharges.

2.3.6 Benthos

Sessile (attached), creeping, or burrowing organisms living on or in the sea bottom are known as the benthos. The numbers of benthic organisms generally decrease with depth below the high-tide level. Since most benthic organisms have limited mobility, they are particularly susceptible to and hence reliable indicators of pollution changes (see Chapter 14).

Sessile members of the benthos include sponges, barnacles, mussels, oysters, crinoids (feather stars and sea lillies), and some worms. Attached plants such as kelp, eel grasses, and some diatoms are often included in this group of organisms. Benthic organisms that move at or over the sediment surface include echinoderms (starfish and urchins), molluscs (snails, clams, and scallops), and crustaceans (lobsters, shrimp, some copepods, and amphipods). Some fish with limited mobility are also considered to be benthic animals. Burrowing forms of benthic life include most worms and clams, and some crustaceans.

Some members of the benthos are important to man as a source of food or income, but most benthic forms are of little direct value. Nonetheless, they play an important role in marine food chains. The benthos constitute a substantial proportion of the marine biomass of the lower trophic levels and therefore provide a large amount of food for the higher trophic levels in most marine ecosystems.

The distribution of benthic forms is largely influenced by the substrates (bottom types) where they occur. These substrates vary from clean, firm rocks to shifting sands to soft muds, and they support an enormous diversity of organisms.

The three main life style strategies of benthic biota are: attachment to a firm surface, free movement on the bottom, or concealment below the surface. These strategies correspond to the organism's method of obtaining food. A sessile life style is possible only where water movements provide floating microscopic food that can be filtered from the seawater and a medium for the dispersal of metabolic products and of reproductive and larval life stages. Turbidity is a problem for these benthic filter-feeders, since a high concentration of very fine particles suspended in near-bottom waters can clog the filtration devices that these animals use to collect food particles. Most of the free-moving benthic organisms obtain their food by catching it, but others are scavengers of bottom detritus.

Almost all shallow-water marine communities have abundant sources of food. These communities are generally dominated by sessile filter feeders and substrate grazers. Sessile organisms commonly shed their gametes (eggs and sperm) into the water and thus ensure widespread dispersal. Many free-swimming larvae of the benthos have the ability to postpone metamorphosis (that is, the change from larval to adult form) until a suitable substrate for survival is found. Some species prefer to colonize a substrate similar to themselves (e.g., oysters settle on oyster shells), but this strategy can result in overcrowding and mortalities. Reef communities may develop when one organism predominates and creates its own environment. Colonial coelenterates living in shallow tropical marine waters at temperatures greater than 20° C, for example, can form coral reefs with their skeletons. These reefs support a large diversity of life since they provide: (1) a firm substrate for attachment, (2) hiding places, (3) calcareous rock that can be readily penetrated by boring or corrosive organisms, and (4) abundant calcareous sediment that burrowing animals can inhabit. A variety of predatory animals from many trophic levels may use the coral reefs as hunting grounds. All coral reefs support symbiotic relationships in which one organism benefits from, or in some cases may even demand, the presence of another. Coral itself participates in such a relationship, since internal symbiotic photosynthetic organisms (zooxanthellae) provide coral with its needed oxygen and metabolize the coral's waste products so that these do not reach toxic levels. Reef communities occur where many elegantly balanced conditions exist (see Figure 2-14). The disruption of any of these conditions can upset this delicate balance, destroy the reef assemblages and, ultimately, kill the coral itself.

Since benthic organisms are unable to escape the contaminants entering their environment, these organisms are often used to monitor the effects of waste discharges. Adverse effects occur if a completely different substrate is imposed upon a benthic community, or if anoxic conditions resulting from the accumulation of excessive amounts of organic material deprive the community of needed oxygen. Even so, some benthic life forms still manage to exist in highly contaminated bottom sediments. Although their diversity may be diminished, their biomass is often comparable to or greater than that found in uncontaminated areas with more diverse assemblages.

A number of approaches have been developed to measure the effects of waste inputs on benthic populations. These include the sentinel organism approach, which measures changes in contaminant concentrations in a stationary species over a period of time, and the infaunal trophic index, which examines the feeding strategies of benthic organisms, of which 90 percent was marine.

2.3.7 Fisheries

Most harvesting of fish and shellfish occurs along the ocean margins at depths of less than 100 m. Although fishery harvests provide only a relatively small percentage of the total calorie requirements of the world's population, they are an excellent source of animal protein. It has been determined that about 10 grams of animal protein per person daily can prevent protein deficiency, while 36 grams daily are suggested for minimum good nutrition (43). The 1980 fishery harvest of 72 million tons (equivalent to about 12 million tons of pure protein) could have provided about 19 percent of the animal protein requirements for the world population (4.4 billion at that time), given an equal distribution of fish protein. Marine fish harvests are increasing (Table 2-6), and it has been suggested that 100 million tons of fish could be harvested for human consumption through increased fishing effort and improved management of fishery stocks (57). This harvest could become even greater as current noncommercial species gain consumer acceptance and more aquaculture is practiced. Alternatively, waste-fed aquaculture systems can provide additional protein for animals or people (20).

As already mentioned, sunlight is the ultimate source of energy for most marine life. Depending upon turbidity, light is sufficient in the upper tens to hundreds of meters for plant growth. Most fish are found here. However, because plants and animals sink when dead, the lower levels of the water column are able to support fish life even where light cannot penetrate. Light also influences the different biological functions of fish, the diurnal vertical migration of some crustaceans whose growth is promoted by darkness, the reproductive activities, and other marine invertebrates.

TABLE 2-6

Summary of World Fishery Statistics

	Total	Marine
1980	71 996 300	64 393 200
1981	74 850 400	66 712 400
1982	76 590 100	68 135 300
1983	76 845 900	67 714 400
1984	82 769 800	73 053 600

Source: Personal communication, Armin Lindquist, FAO, 1986.

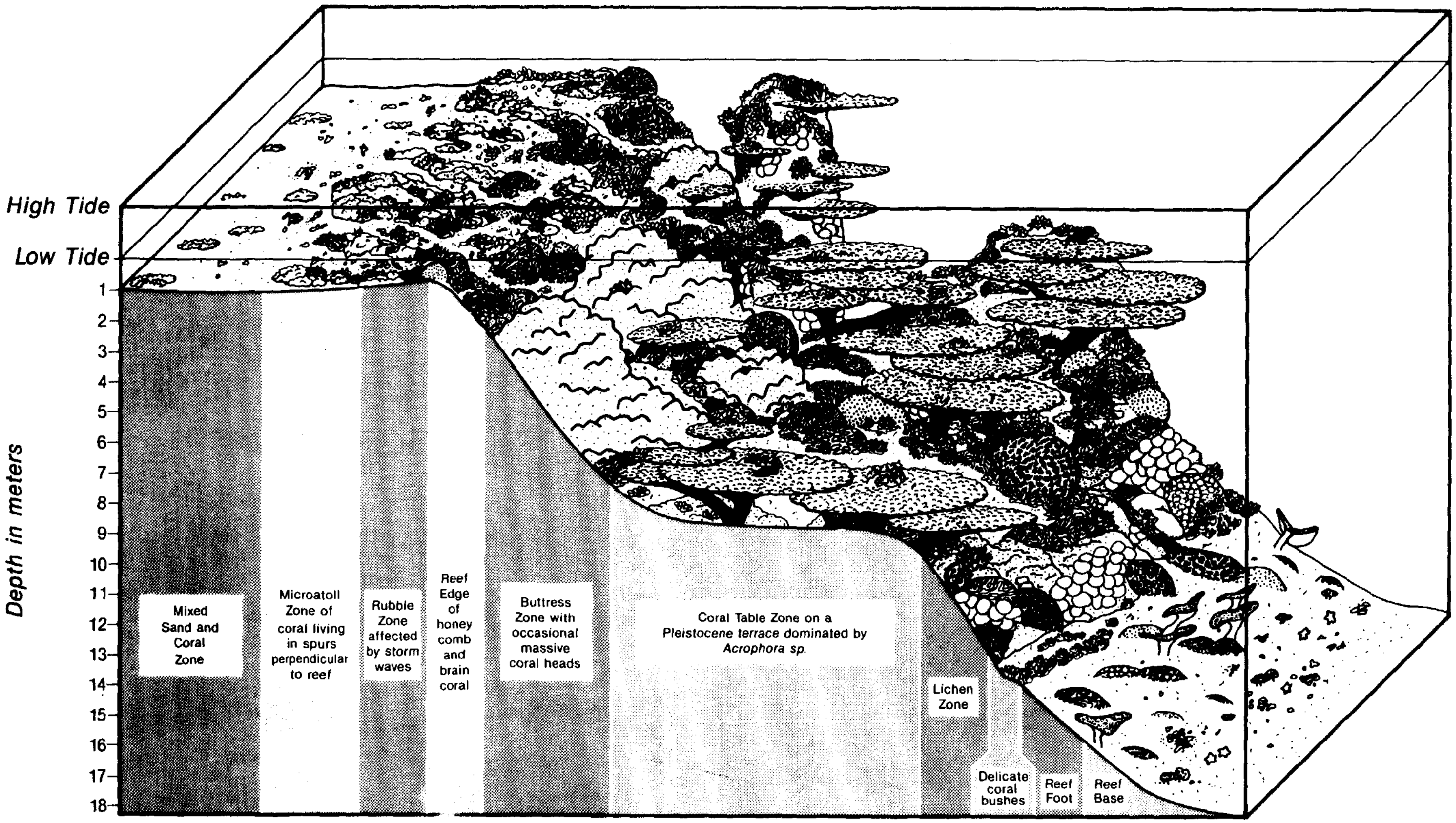


Fig. 2-14. Elegance of Life on a Coral Reef in the Arabian Gulf. Source: Basson et al. (1977). Courtesy of the Arabian-American Oil Company, Dharan, Saudi Arabia.

As mentioned previously, temperature greatly affects plant productivity and fisheries in the sea. Temperature can: (1) control the life or death of an organism, (2) control its rates of metabolism and development, (3) limit its activity and movement, and (4) stimulate sensory perception and oriented response (6).

Besides light, plants require nutrients. Areas of high primary productivity have been grouped into three categories by the Food and Agricultural Organization of the United Nations (21): (1) upwelling areas found primarily off the western subtropical continental coasts (Peru, California, North and Southwestern Africa) and along the equator, where cold, nutrient-rich waters rise to the surface; (2) temperate and sub-Arctic waters off the Southern Ocean, North Atlantic, and North Pacific; and (3) shallow waters over those parts of the continental shelves that receive stream runoff.

Some fish stocks are distributed over fairly large areas and migrate great distances whereas others are less mobile. Therefore, exploitation of different stocks can produce either widespread or local effects. For some parts of their lives, most animals in the sea are mobile and are subject to waves, currents, and other water movements. For shellfish, this mobile phase occurs in the larval stage, from which they later settle to the bottom and inhabit particular areas. Others, such as the fauna of coral reefs, are capable of greater mobility but tend to remain in one locale. Most fish stocks that are commercially harvested move for distances of tens to hundreds of kilometers. Some stocks, such as bluefin tuna, migrate across one or more oceans.

2.3.8 Effects of Waste Inputs

The effects of waste inputs on fisheries depend on the nature of the wastes, the transport of the wastes in the marine environment, the types of fisheries involved, and the residence time of the stocks in the area of discharge. Marine organisms deal with stresses by (1) escape--benthic species can escape by burrowing into the sediments, while finfish may escape by swimming; (2) reduction of contact--some fish species secrete a mucous layer, while shellfish species may close their valves and stop feeding to avoid changes in their environment; (3) regulation--many fish are able to control their environment by regulating chemical concentrations, volumes of body fluids, or fluid pressures within their bodies; (4) acclimation--a number of species have adapted to man-induced changes in their environment with no apparent harm because they are used to large changes in their coastal-zone environment.

In general, a sewage outfall will attract large numbers of free-swimming fish, although some species may choose to avoid the immediate area of the discharge. Since the discharge provides an ample source of food, energy that would ordinarily be expended in obtaining food is instead used for enhanced growth and recruitment of the species. If a discharge is discontinued, the enhanced species will return to normal size and abundance. In contrast, local populations of shellfish are likely to be adversely affected by pollutant inputs from sewage outfalls. The effects on mobile species, particularly contaminant concentrations within the fish themselves, depend on their residence time in the discharge area. As fish move in and out of the discharge area, effects may be short-lived and transitory. However, the potential for bioaccumulation of organic chemicals in finfish is increased with prolonged residence in an affected area. The effects of discharges are not always local or easily identified. For example, the release of a persistent insecticide into a coastal river in the eastern United States resulted in the contamination of fish hundreds of kilometers from the discharge site (64). In another case, in the southwestern United States, bird mortalities were attributed to consumption of fish contaminated by a pesticide discharged from an outfall (75).

Tastes and odors (taints) can often be traced to the bioaccumulation of pollutants. Crustaceans, fish, and molluscs exposed to oil can acquire an objectional, oily taste. With increased inputs of oil and petroleum by-products into the marine environment, the likelihood of fish tainting increases. Susceptibility to tainting varies among species and depends on the condition of the fish, the nature of the oily compound, and the exposure time.

2.4 Special Topics

2.4.1 Public Health

Both natural and man-made conditions in the sea give rise to certain public health concerns (73). Some marine plankton, nekton, benthos, and bacteria are naturally toxic or infectious. In a few areas of the world ocean, seasonal occurrences and occasional dense blooms of some dinoflagellates, "red tides," contain neurotoxins that can be concentrated by shellfish and that have caused human paralysis or death if the shellfish are eaten. It has been hypothesized that some dinoflagellate blooms may be induced by nutrient enrichment of open coastal waters from sewage. There is no evidence for this (7, 48), although some investigations have yielded putative evidence from embayments with limited circulation (17, 68). Other natural toxins are found in some jellyfish, bottom fish, and sea urchins.

A few marine bacteria that live and multiply in seawater and/or sediments can cause illnesses. Foremost among these is Vibrio parahaemolyticus, which often causes minor cases of food poisoning and, occasionally, death. Consumption of raw fish and shellfish is one of the most common causes of Vibrio-associated illnesses. Another microorganism, Mycobacterium marinum, causes skin lesions. Many of these pathogens are opportunistic microbes that can adapt to a tissue or host other than the one in which they are normally found.

Chemical contaminants of interest to health officials include certain inorganic and organic chemicals and radionuclides. Acids, alkalis, and cyanides discharged to the ocean are quickly neutralized and are only locally significant. Except for artificial radionuclides, the metals discharged into the ocean are also normal constituents of the sea and are not of great concern except in extreme instances. Earlier concerns about the toxicity of heavy metals in the marine environment have been reduced. Recent evidence shows that, in general, concentrations of these metals do not increase upward in the food chain. They are not known to pose a threat to people eating seafood, except in rare cases such as the methyl mercury contamination that occurred in Minimata Bay, Japan (2), although they may exceed the precautionary levels established by various standards or guidelines. In contrast, toxic synthetic organic chemicals are persistent in the sea, are known to accumulate in food chains, and are found in increasing amounts in the marine environment. These chemicals have been linked to sublethal effects on marine life and to mortalities in birds that feed on fish with high levels of these chemicals (74, 75). Careful research and monitoring have revealed no evidence of human diseases or mortalities from the consumption of fish or shellfish with high levels of synthetic organic contaminants. Nevertheless, economic losses have been substantial in cases where fishery areas have been closed because organic contaminant levels exceeded national safety limits (36).

Radionucleides are introduced into the marine environment by both nature and man. Natural sources include the interaction of cosmic rays with atoms in the atmosphere and the weathering of rocks. Artificial radionuclides come from nuclear power production, nuclear-detonations, and pharmaceutical and industrial inputs. Even in the most radioactively contaminated areas there has been no measurable harm to marine organisms or to man (52).

Pathogens that on land cause salmonellosis, typhoid fever, hepatitis, dysentery, and other gastrointestinal illnesses are discharged with sewage into the sea. Although most of these survive only for short times in the marine environment, a few species and subspecies persist longer.

It is essential to know the transmission routes by which pathogens reach human populations in order to predict their effects on human health. By far the most important route of transmission is the consumption of contaminated shellfish. Recreational use of contaminated waters raises an important aesthetic issue but is a much lower-priority public health problem. Contaminated shellfish that have accumulated pathogens to infective levels are sometimes eaten raw. Well-managed harvesting, depuration, and sanitary handling of shellfish can prevent disease outbreaks caused by eating raw shellfish; of course, adequate cooking of shellfish destroys all accumulated pathogens. Swimming in polluted waters can result in mild gastrointestinal symptoms. Increasing numbers of antibiotic-resistant microbes are being found on land, particularly in hospitals where they have caused staphylococcal disease outbreaks. They have also been found in the sea (28, 42, 67), but are not suspected to have caused similar problems.

2.4.2 Toxicity

The toxicity of a substance depends on its concentration in the environment, its availability to an organism, and the susceptibility of the organism. Any substance can become toxic when its concentration exceeds some threshold. Persistent substances are most likely to accumulate to toxic levels, although others also accumulate if they settle quickly to the ocean bottom and thereby avoid the great ability of the ocean to neutralize or disperse them. These pollutants do not always settle permanently since storms and currents can resuspend them.

While no environmental entity wants to see marine organisms killed with toxicants, neither does it want them to become sick. The problem is that sublethal or chronic effects are not revealed by the conventional bioassay tests used to assess acute toxicity. In the latter, organisms are subjected to a range of concentrations of a known or suspected pollutant. Results are reported as an LD_{50} or LC_{50}, which is the lethal dose or the concentration at which half of the test specimens are killed within a specified time limit. Because reported LC_{50} values for the same species tend to vary widely, such tests are of questionable value. Changes in salinity, pH, turbidity, temperature, concentrations of other substances, and the health of the organisms themselves can cause the toxicity of a pollutant to change substantially. For example, chlorinated hydrocarbon pesticides are more toxic in summer; detergents are more toxic at higher salinities; and some organisms are more sensitive during their reproductive or early developmental stages. In addition, organisms may be able to adapt to gradual changes in a toxicant level in their environment, but exposure concentrations are increased abruptly during bioassays.

Some toxicants accumulate in sediments or biological systems, where they may be toxic both to marine organisms and man. Bioaccumulation is assessed by comparing concentrations of the substance in the water column, the sediments, and certain organisms. The bioconcentration factor (BCF), or the ratio of the concentration in the organism to the concentration in the surrounding water, indicates pathways of a toxicant in the marine environment. Flocculation and sedimentation of particles with adsorbed toxicants may cause pollutants to become entrapped in the sediments. Some of these pollutants may reenter the water column by physical resuspension, by metabolic activity (excretion of the benthos), or by accumulation within benthic organisms that serve as food for pelagic species.

Wastes discharged into the marine environment undergo various biochemical transformations, such as decomposition of organic matter, alteration of the physical or chemical form of the constituents, adsorption onto particles and sediments, and incorporation into living matter. Some of these transformations and interactions increase toxicity, whereas others decrease it. Where mixing and circulation are limited, and where added nutrients and organics increase the biological activity, the oxygen content of the water is often decreased.

Toxic inorganic chemicals include acids, alkalis, cyanides, and metals. The first three are quickly neutralized when dispersed and diluted in seawater, but the effect of metals is difficult to demonstrate. Metal compounds form a major portion of the earth's crust and are transported to the ocean by rivers, glaciers, and wind. These metals are naturally occurring constituents of the sea, and it is difficult to distinguish them from metals in wastes. Man-made sources of metals include industrial and sewage effluents, combustion of fossil fuels, and ocean dumping. When the normal background concentrations of certain metals in seawater are substantially increased, the potential for toxicity exists. Several factors influence the toxicity of metals to marine organisms:

1. The physiochemical form of the metal

2. The synergistic or ameliorative action of other metals or compounds

3. The physical properties of the surrounding seawater

4. The physiological condition of the marine organisms.

Acute toxic effects have been demonstrated by laboratory experiments, primarily through the interaction of metals with animal tissue enzymes and resulting alteration of enzyme activity. These effects are most likely to occur near large sources, where the highest pollutant concentrations are found (53). Opinions differ as to which metals are more toxic, but measured inputs of specific contaminants indicate that the following are of primary concern (52, 53):

Mercury(Hg)*	Silver(Ag)	Tin(Sn)
Cadmium(Cd)	Copper(Cu)	Chromium(Cr)
Lead(Pb)	Zinc(Zn)	Selenium(Se)*

Of secondary concern are:

Antimony(Sb)	Nickel(Ni)
Arsenic(As)	Vanadium(V)

Other trace metals such as thallium (Tl) may be potentially damaging to marine ecosystems, but they are not yet used to any great extent by man and, therefore, inputs are low. Although elevated metal levels have been detected frequently in organisms from contaminated areas, there are very few documented cases of acute trace metal toxicity in organisms in the marine environment. One such case involved a substantial abalone kill attributed to copper pollution from the discharge of seawater that had been held for some time in copper condensors at a coastal power plant (45).

Laboratory experiments indicating that elevated concentrations of certain trace metals in seawater are toxic to marine organisms are difficult to interpret since toxic effects have been reported even at concentrations near or within natural background variations. This discrepancy is due either to the inability to reproduce field conditions in the laboratory, or to the natural detoxification processes of organisms that have had time to adjust to these levels in the environment, but not in the laboratory. Recent studies conducted on both the east and west coasts of the United States indicate that many previously accepted beliefs concerning the toxicity of metals to marine

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* See Section 2.4.2.3 for ameliorative effects of Se on Hg toxicity.

life are wrong. These studies have now shown that high concentrations of toxic metals in tissues are not necessarily the cause of metabolic disorders in marine life (12) and that, in general, concentrations of metals do not increase with higher trophic levels in the food chain (75). Instead, the animals detoxify themselves by sequestering (chemically binding and storing) the toxic metal with other substances found within their bodies.

Organic chemicals are those containing carbon, although a few compounds with one carbon atom, such as carbon monoxide (CO) and carbon dioxide (CO_2), are considered inorganic. Organic chemicals ultimately decompose into carbon dioxide, water, and other simple inorganic compounds. However, many synthetic organic chemicals, some of which are used in anti-fouling paints, are extremely persistent in the environment and often accumulate in the food chain.

The synthetic organic chemical industry has grown phenomenally in recent decades and has produced substantial quantities of hundreds of thousands of new compounds. Among these, chlorinated hydrocarbons are of particular concern. They have lower molecular weights and lower bioaccumulation factors than most other halogenated hydrocarbons, but are in widespread use throughout the world. Kepone, DDT, and PCBs are the chlorinated hydrocarbons that have received most notoriety as a result of their impact on coastal or estuarine environments (39, 45, 47, 60, 64, 66). These persistent chemicals are only slightly soluble in water, are found in the food of living organisms, and tend to remain in the fatty tissues for a long time. Laboratory tests for lethal toxicity do not reflect the effects of these substances in the marine environment. Various sublethal effects have been documented in marine organisms in association with exposure to these chlorinated hydrocarbons (60). For example, DDT has been correlated with a decrease in the reproductive success of spotted sea trout in the southeastern United States (9) and of brown pelicans in California (39). Although the United States has banned the use of DDT, many other nations have not because its low price and overall benefits to public health and food production are thought to outweigh any environmental costs (66). To some extent, organophosphate pesticides, such as Malathion and Parathion, are replacing the chlorinated hydrocarbons in agricultural use. They attack the nerve systems of insects and, although they are relatively biodegradable, sufficiently high doses will affect the nervous systems of marine animals and humans (53).

Sewage effluents may be chlorinated to meet end-of-pipe bacteriological standards, but this may make the effluent more toxic. Chlorine interacts with the organic materials in seawater to produce chlorinated organic compounds such as chloroform, a known carcinogen and mutagen.

Although oil spills contribute about 12 percent to the total petroleum in the oceans, the remainder derives from river runoff (35%), normal shipping operations (34%), natural seeps (8%), atmospheric fallout (9%), and offshore oil production (2%) (19). Small amounts derive from atmospheric inputs, waste discharges, and urban runoff (14). Locally high concentrations of petroleum hydrocarbons in the parts-per-million range can kill most marine species, but sublethal effects of petroleum residues are largely unknown. Research is now being directed toward the effects of polynuclear aromatic hydrocarbons (PAHs) because of their carcinogenicity.

2.4.2.1 Measurements of Toxicity

Laboratory determinations of the LC_{50} (or LD_{50}) are the most readily available, although inconsistent, source of data on the relative toxicities of various chemicals. The great variability in these test results and their limited applicability to the marine environment are due to several factors:

1. Organisms used in the tests have very different physiologies from those found in the discharge area.

2. Test conditions--including time, temperature, pH, and salinity regimes--often vary and make it difficult to compare the data. Some tests use the same water (static system) throughout the time period and some tests use flowing water. Some are short-term acute tests, whereas others are chronic tests, which are more sensitive and more closely approach environmental conditions. (The latter tests do not attempt to destroy the organism, but rather are aimed at examining the sublethal effects over a longer period of time.)

3. The developmental stage of the test species often varies in separate experiments. Fry and larval stages tend to be more sensitive than adults to toxic substances.

4. Different forms of the toxin are used for different experiments. The ionic form of the inorganic metals used in laboratory toxicity tests are often substantially different from the form of the metals found in the sea and from those metals found in effluents, which are usually attached to particulates.

Thus, the best that can be expected from laboratory studies of toxins are suggestions of relative toxicities of materials. They cannot be used to predict effects in the marine environment.

2.4.2.2 Bioaccumulation

Bioaccumulation is the ability of an organism to concentrate, often by many orders of magnitude, the chemical components of its environment in its tissues. The accumulation of contaminants by an organism is dependent upon exposure time, concentration, and chemical form of the contaminant. Uptake of contaminants from all available sources (water, food, and sediments) determines the bioconcentration factor (BCF).

Bascom (2, 3) reports that, contrary to conventional theory, studies now show that metals in their usual inorganic forms are not concentrated through the marine food web. In areas near sewage discharge, bioaccumulation of trace metals or organic compounds, such as PCBs or aromatic hydrocarbons, in marine biota may be observed. However, elevated tissue concentrations of these substances in a discharge area are not a signal of significant adverse effects on the marine biota. For example, caged mussels placed in highly contaminated areas of the New York Bight apex were shown, through a variety of biochemical and growth rate measurements, to suffer no negative biochemical effects, even though these mussels did bioaccumulate significantly higher concentrations of several metals (Zn, Pb, and Cd), PCB, and polyaromatic hydrocarbons than did mussels grown at cleaner reference sites. In fact, the biological measurements indicated that the mussels at the contaminated (and enriched) sites were probably growing more rapidly than mussels at the cleaner sites (2). Bioaccumulation of trace metals and toxic organics alone cannot, therefore, be considered an indication of adverse effects of sewage disposal unless concentrations in seafood species approach maximum safe levels for human intake.

The accumulation of toxic substances in people depends on the amounts of particular animal tissues consumed, the concentrations in the seafood (which depend upon the pollutant concentration and the proximity and residence time of the species in the discharge area), and the method in which the seafood is prepared. Although sedentary shellfish might appear to be of particular concern, both laboratory and field studies of kepone contamination have shown that oysters do not bioconcentrate organic toxins. Furthermore, when relocated to a cleaner environment, organisms are able to purge organic toxins from their systems within a relatively short time (26).

2.4.2.3 Ameliorative Effects

Marine animals have evolved natural protective mechanisms to survive gradual variations in the amounts of metals present in seawater. One such mechanism is a sequestering protein, metallothionein, which is produced within organisms to maintain a reserve supply of essential metals (e.g., zinc and copper) required for enzymatic activity (2, 3). In addition to holding essential metals for future use, the metallothionein also holds excess or otherwise nonessential metals and thus prevents them from reaching the enzymatic and genetic systems, where toxic effects can occur.

A number of other interactions of a protective nature are known to occur in the marine environment. Mercury, certainly the most studied and possibly the most toxic of all metals, is a normal constituent of seawater and is accumulated in fish. Selenium, another potentially toxic metal, is required by most animals and is also a trace element in the sea. Experiments with a number of animals have confirmed an antagonistic interaction between these two metals that limits the toxicity of both the mercury and the selenium (25). In one study, a group of quail were fed tuna contaminated with methyl-mercury with no apparent detrimental effects, whereas another group was poisoned by an equivalent amount of methyl-mercury in a corn/soya diet. These results support the contention that selenium, which occurs naturally in high levels in tuna, suppresses the toxicity of mercury.

Although the research is still in an early stage, it appears that selenium may have similar ameliorative effects on other heavy metals, such as silver. A number of other compounds also have been found to suppress the toxicity of metals. Vitamin E, which contains selenium, has been shown to be effective against silver toxicity; and sulfur amino acids, in combination with forms of selenium and arsenic, are especially effective against methyl-mercury toxicity. Moreover, recent research indicates that there are other ameliorative interactions within the marine environment.

2.4.2.4 Red Tides

Dense, short-term blooms of various plankton, commonly referred to as "red tides," occur naturally, particularly in enriched coastal waters (7). There are two principal problems associated with red tide blooms. Shellfish may become contaminated during blooms of the dinoflagellate, Gonyaulax sp., and subsequently rendered unsuitable for human consumption as a result of accumulation of the biotoxin responsible for paralytic shellfish poisoning. The usual consequence of dense blooms, however, is oxygen consumption during the decomposition of the dead plankton cells, which can create hypoxic or anoxic conditions inimical to marine life in the water column and near the sea bottom.

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CHAPTER 3

ECOLOGICAL DESIGN

3.1 Introduction

This chapter looks at the rationale and selection of design criteria for the protection of public health and of the natural ecological systems of which man is a part. Annex A contains global-scale information on these natural systems.

3.2 Public Health

Municipal sanitation and sewerage systems provide health benefits by separating people from their excreta at the household level. The myriad household problems thus solved are aggregated into a larger problem of treatment and disposal in which the whole is essentially the sum of its parts. Conventional primary and secondary sewage treatment was developed to protect ecological systems by reducing suspended solids and oxygen demand of organic wastes that would otherwise create fish kills and nuisances. Such treatment is not particularly effective in removing pathogens, however (see Table 3-1). Effluent chlorination can be used to destroy bacteria and protozoa in trickling filter or activated sludge effluents containing low, say 15 mg/l, concentrations of suspended solids, but this is an expensive technique and requires foreign exchange, which is always limited in developing countries.

TABLE 3-1

Pathogen Removal Effectiveness Of Wastewater Treatment Processes (in $\log_{10}$ units)

Process	Viruses	Bacteria	Protozoa	Helminths
Primary sedimentation	0-1	0-1	0-1	0-2
Septic tanks	0-1	1-2	2-3	3-4
Trickling filters	0-1	0-2	0-2	0-1
Activated sludge	1-3	1-3	1-3	1-4
Stabilization ponds (20 days, 4 cells)	2-4	4-6	4-6	4-6

Source: Feachem et al. (6) and Shuval et al. (35).

In any event, large numbers of every pathogen known to man are being discharged to marine and estuarine waters. The reason why more people aren't sick is that certain intervening factors reduce the incidence of infection. These have been identified by Shuval et al. (35) and Feachem et al. (6) as pathogen persistence in the environment, minimum infective dose, host immunity, and the relative importance of alternative pathways of infection.

The World Bank studies by Shuval et al. (35) of the excess incidence of infection due to occupational or consumer exposure to raw sewage used for irrigating vegetables revealed that the intervening factors resulted in a low incidence of excess infection due to viruses, medium excess infection due to bacteria, and high incidence due to helminths. These findings have made it possible to set priorities on investments in wastewater treatment based on health benefits.

3.2.1 Marine Recreational Water

The same intervening factors apply in marine waters used for recreation. Because the incidence of excess infection is even lower here, many authorities base marine pollution control activities on aesthetic rather than bacteriological standards (25). Figure 3-1 summarizes the recent intensive efforts of the U.S. Environmental Protection Agency to identify excess incidence of gastrointestinal upsets among New York City residents; Alexandria, Egypt, residents; and Alexandria visitors, mostly from Cairo (3). The results of this work indicate that (1) New York City marine recreational waters are less polluted than those of Alexandria; (2) New York City residents are more susceptible to infection from a given level of exposure than are Egyptians, presumably because of a higher level of immunity among the latter group, and (3) visitors are more susceptible to infection than are residents. It is thus evident that people develop greater immunities to locally dominant strains of pathogens. Cabelli (3) also reported that incidence among children (aged $\leq$ 10 years) swimming in polluted water was about 1.5 times that for adults; he therefore concluded that much of the immunity was acquired during childhood. All of these findings are consistent with those reported by Shuval et al. (35).

The minimum infective dose for pathogens varies from a single egg for helminths to one million or more for salmonella (see Figure 3-2). Such large numbers would be extremely difficult to remove from the water, artificial methods, but a shellfish can injest a sufficient portion of them to become contaminated.

Earlier field observations on the disappearance of coliforms from recreational marine waters reveal that sedimentation of the solids with which the microorganisms are associated is always an important and often a dominant intervening factor. The results of several studies (see Table 3-2) indicate that disappearance rates are site specific and related to the concentrations of suspended sediments in the effluent. Times for 90 percent reduction (T_{90}'s) due to mortality plus sedimentation range from 0.2 to 2 h for raw sewage, from about 2 to 4 h, for primary effluent, and to 9.6 h for secondary effluent. Chlorination reduces concentrations of coliforms, but those

Fig. 3-1. Reported Gastro-Intestinal Effects of Swimming in Polluted Marine Waters in New York City, USA and Alexandria, Egypt (after Calelli, 1983).

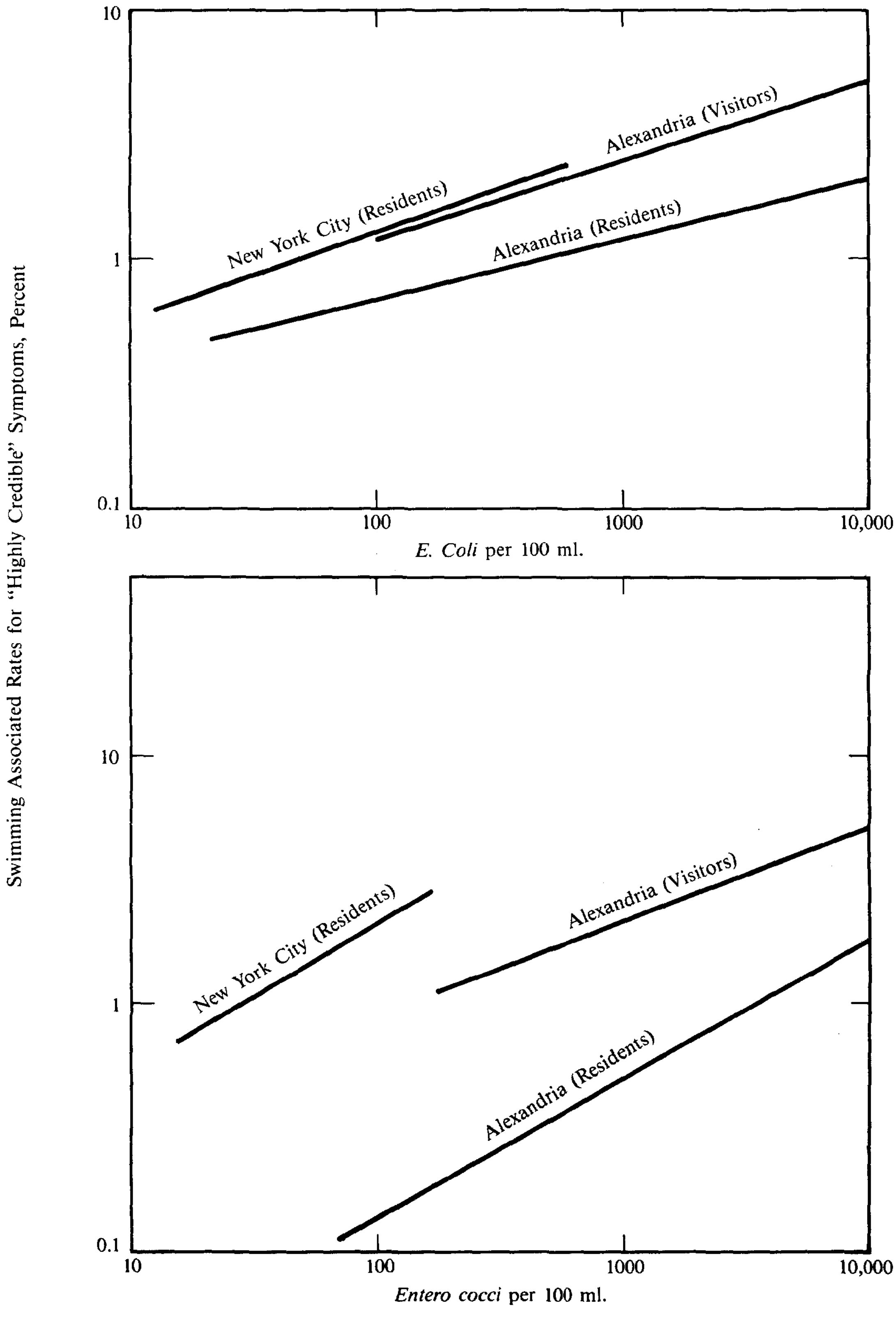
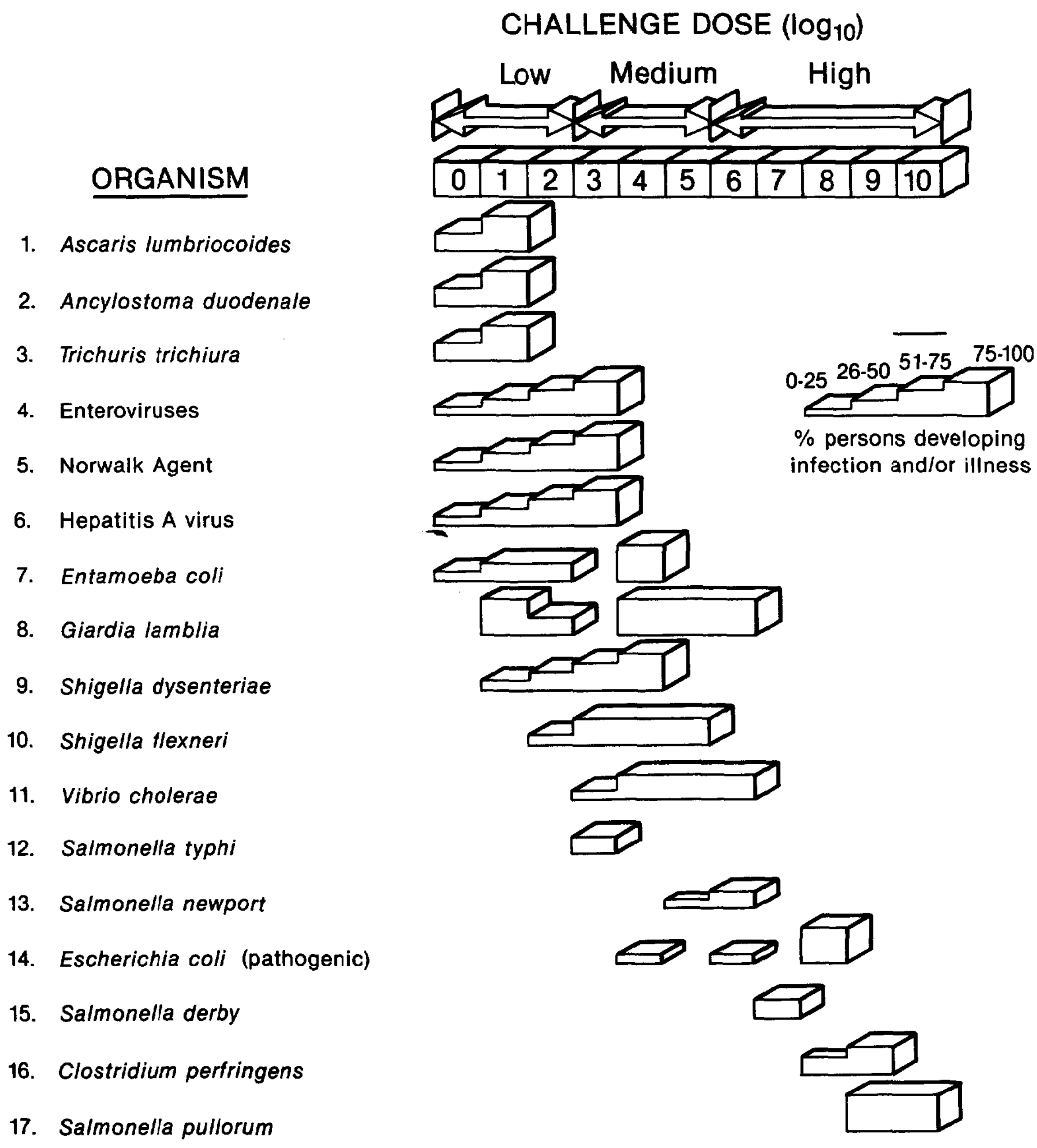


Fig. 3-2. Minimal Infective Dose of Enteric Pathogens - Clinical Response of Adult Humans to Varying Challenge Doses



Source: Shuval et al 1986

TABLE 3-2

Reported Rates of Reductions of Coliform Bacteria due to Mortality and Sedimentation in Ocean Waters in Times for 90 Percent Reductions and as k_e Where $k_e = \ln 10/T_{90}$

Location	Date	T_{90}(h)	k_e(h^{-1})	Remarks
Raw sewage				
Honolulu	1970	$\leqq 0.75$	$\geqq 3.1$	Reported T_{90}'s varied from 0.2 h during trade wind weather when there was no reflux of coliforms to 0.75 h during other periods. The higher value has been adopted.
Titahi Bay, New Zealand	1959-60	0.65	3.5	
Rio de Janeiro	1963	1.0-1.2	1.9-2.3	
Israel	-	<1.0	>2.3	Poliovirus T_{90} > 24 h
Istanbul	1968	0.8-1.7	1.4-2.9	Mean value 1.1 h
Genofte, Denmark	-	1.2	2.0	
Tema, Ghana	1964	1.3	1.7	
Nice, France	-	1.1	1.1	
England	1965	0.78-3.5	0.66-2.9	
Manila, Philippines	1968-69	1.8-3.4	0.67-1.3	Higher T_{90} value may represent mixing with old effluent
England	1969-73	1.4-5.3	0.43-1.6	Median of 11 results 3.2 h.
Mayaguez Bay, Puerto Rico	-	0.7	3.3	
Montevideo, Uruguay	-	1.5	1.5	
Santos, Brazil	-	0.8-1.7	1.4-2.9	
Porlaleza, Brazil	-	1.1-1.5	1.5-2.1	
Maceio, Brazil	-	1.2-1.5	1.5-1.9	
Primary effluent				
Ventura, California	1966	1.7	1.4	
Seaside, New Jersey	1966	1.8	1.3	
Orange Co., California	1954-56	1.8-2.1	1.1-1.3	
Santa Barbara, California	1967	2.4	0.96	
Los Angeles, California	1954-56	4.1[a]	0.56	
Secondary effluent				
Los Angeles, California	1954-56	9.6[b]	0.24	
Combined				
The Hague	1968-69	5-175	0.01-0.5	Higher T_{90} values may represent mixing with old effluent, resuspended sediments, or other sources

[a] T_{90} due to mortality 17.8 h, and to sedimentation 5.3 h; higher T_{90} after chlorination.
[b] T_{90} due to mortality 17.8 h, and to sedimentation 21.0 h.

Source: Gunnerson (12) and R.G. Ludwig (personal communication 1986).

remaining are more persistent; with Hyperion, primary effluent increased the T_{90} due to mortality alone from 17.8 to 39.5 hours, where mortality is the combined effect of competition, predation, sunlight, and other factors (10).

The site- and effluent-specific nature of sedimentation factors has been confirmed in the laboratory by Hering and Abati (13). (Figure 3-3 shows differences both in initial flocculation times and in subsequent sedimentation for six different Southern California effluents.) Their results are qualitatively consistent with earlier empirical work on effluents (11, 22, 36) and more recent work on sludge and artificial seawater mixtures by Koh (20, 40), Hunt (16), and their associates at the California Institute of Technology and the University of California, respectively.

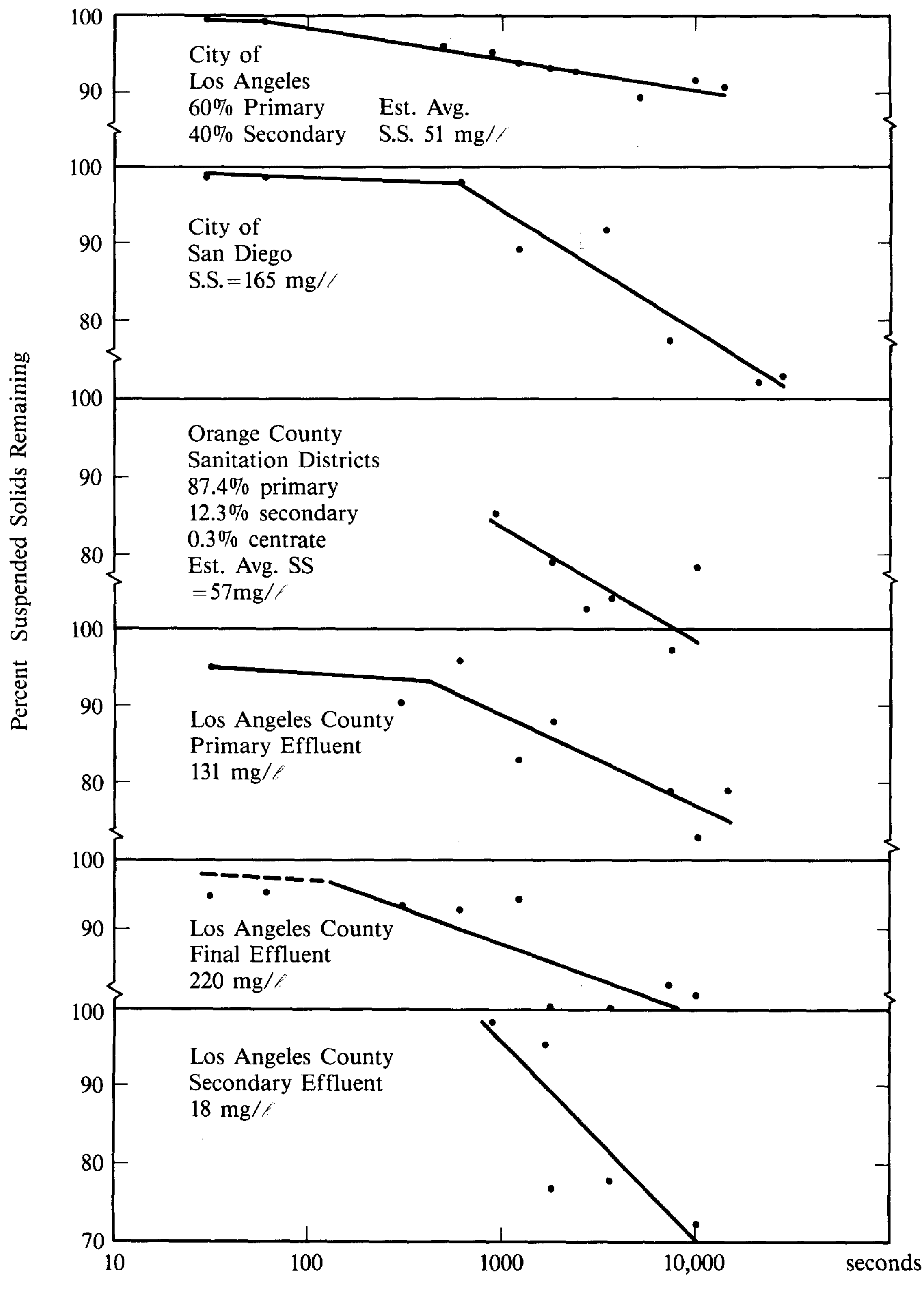
There are numerous objections to the use of coliforms as indicators of pollution and water-quality standards (3, 6, 35). Although alternatives to coliforms, have been recommended the latter continue to be used for marine outfall design because the available evidence indicates that they provide both epidemiological and ecological protection. However, their specificity to both location and level of treatment is indicated in Table 3-2. Thus, extending decay coefficients from one location to another opens the door to error and to the assumption of factors of safety or conservative use of arbitrarily high values for T_{90}. Although travel times and, to a first approximation, outfall lengths are directly proportional to design T_{90}'s, outfall costs are not. The latter increase by step functions that are nonlinear owing to construction difficulties, which increase with both length and depth. It is less expensive to determine site-specific T_{90}'s (see Chapters 15 and 16). More important, most experience confirms that, when bacteriological criteria are met, aesthetic and other requirements are also met (7, 17, 29, 30, 33, 41) and that meeting numerical receiving-water standards is more cost effective than applying uniform effluent standards. However, ecological problems, such as eutrophication--for which there are no satisfactory numerical criteria--are more difficult to resolve.

3.2.2 Shellfisheries and Finfisheries

Oysters, clams, and mussels are particularly susceptible to waste discharges since they are attached or sedentary during their adult life and concentrate pathogens and other contaminants from their surrounding environment. Since sedimentation creates a sink for infectious agents attached to large particulates, it makes these pathogens more available to the benthic shellfish located near the contaminant source (9). The shellfish themselves are not harmed by sewage microorganisms, but humans become exposed to a number of diseases--for example, typhoid fever, salmonella, dysentery, and other gastrointestinal illnesses--if they consume contaminated shellfish. Cooking can destroy any pathogens of concern, but many people prefer to eat raw shellfish, particularly bivalve molluscs.

Most shellfish are filter-feeding animals, many of which have the ability to pump large amounts of water (up to 500 liters per day) over and through their gills, often concentrating contaminants from the environment in the process. Cabelli et al. (4) note that concentration factors of seven- to tenfold have been demonstrated for various microorganisms in shellfish.

Fig. 3-3. Laboratory Settling Velocities for Southern California Effluent Solids in Seawater at 80:1 Dilution (after Hering and Abati, 1978).



Existing water-quality standards for shellfish growing areas are not 100 percent effective, and even "acceptable" areas may contain contaminated shellfish. Even though pathogens and other contaminants in shellfish may exceed standards for human consumption, these levels may be brought into tolerable ranges by removing the shellfish to clean waters for a period of time. The two methods commonly used to rid shellfish of contaminants are relaying and depuration. Relaying involves placing contaminated shellfish in clean areas and then reharvesting them, usually several months later. This method was used in the mid 1970s, for example, to treat oyster beds in a small portion of the eastern United States that had been closed because they were contaminated by a synthetic organic pesticide. The young seed oysters were relaid in uncontaminated waters, where they matured and were later harvested with no trace of chemical contamination (15). Depuration involves placing the contaminated shellfish in tanks with clean water. The time required for purification is determined by water temperature, salinity, initial bacteriological quality (or contaminant level), and species of shellfish (14). For sewage-contaminated shellfish, 24-72 hours may be adequate to remove pathogens prior to human consumption. The Minimata incident has given rise to concern, sometimes exaggerated (see Chapter 14), over the potential effects of toxic elements and substances in seafood (see the guidelines listed in Table 3-3).

TABLE 3-3

Guideline Values for Toxic Materials in Foods

Element or Compound	WHO-FAO Provisional Tolerable Weekly Intake (mg/person)	USFDA Action levels (ppm) in food
Cadmium	0.4-0.5	--
Lead	3.0	--
Mercury (total)	0.3 (seafood, edible portion)	1.0
Methyl mercury	0.2	--
Chlorinated pesticides		0.03-10 (varies with type of compound and food)
PCB's	--	5.0 (fish)

Note: Not applicable for infants or children.

Source: WHO (39).

Table 3-3 shows that even extremely small amounts of some toxic materials are considered hazardous. However, daily exposures to lower levels in food are considered normal for some or all of these toxic elements and do not result in any known health effects (19). The listed safety limits are almost universally met in seafood, although fish and shellfish in certain limited urban coastal areas occasionally surpass these levels for some chemicals and, as a result, fishing is restricted.

Human populations with a high seafood intake are obviously the ones most likely to be affected by any toxic materials present in edible fish and shellfish tissues. A recent study (34) investigated the possible health effects of the consumption of fish from an area of the United Kingdom with known sewage sludge inputs containing mercury. A population with an above-normal seafood intake for the United Kingdom was chosen. Their fish consumption averaged 0.36 kg/person/week. During the study, the blood levels of mercury in this population rarely exceeded 20 mg Hg/liter, which is only one-tenth of the level that the World Health Organization has reported is the minimum necessary to cause an adverse health effect. Although toxic metals other than mercury in seafood are believed to be a lesser hazard to human health, another recent study indicated that safe intake levels of cadmium may occasionally be exceeded. In Hong Kong, where both concentrations of metals in seafood and seafood consumption are relatively high, a sampling of fish from retailers indicated that cadmium levels in that case were approaching safety limit levels (31).

A number of naturally occurring interactions between heavy metals and other metals or compounds take place in the water, sediments, or marine organisms. Some of these interactions significantly reduce the toxicity of the metals. For example, selenium and mercury, both of which exist as trace elements in the sea and in fish, function as antagonists; their interaction decreases the toxicity of either one or both (8). Vitamin E, a nutrient found in fish, has also been shown to be effective in decreasing the toxicity of both mercury and silver.

In summary, it would appear that acute or significant sublethal toxic effects of heavy metals or synthetic organics on marine organisms and their predators are unlikely to occur if outfalls are properly located and contaminant concentrations in the effluent are controlled at their points of manufacture and utilization. Excessive concentrations are more likely to be found where there are major industrial releases of toxic compounds to the sewer systems.

By and large, the consumption of toxic compounds in seafood can be avoided if proper care is taken in the harvesting and handling of fish and shellfish. This implies that the presence of contaminants in a discharge area or, more specifically, in the seafood itself has been detected. If shellfish are harvested near a contaminant source, depuration or relaying in uncontaminated waters may allow the shellfish to purge themselves of many contaminants, although these treatments are not equally effective for all contaminants.

Thorough cooking of fish or shellfish eliminates pathogens (6) and may reduce the concentrations of some toxics due to volatilization.

Because finfish are mobile, it is unsafe to assume that those harvested far away from a contaminant source are uncontaminated. The presence of toxic contaminants in seafood and, hence, the likelihood of human illness resulting from its consumption, can only be determined by periodic sampling of the various fish tissues normally consumed.

3.3 Ecological Design

It is now widely recognized that ecological systems must be protected if we are to ensure the health and economic well-being of future generations. Evidence in support of the need for remedial and preventive environmental programs comes from natural and man-made ecological disasters such as that at Bhopal, India (isocyanates), Seveso, Italy (dioxin), Minimata, Japan (methyl-mercury), Kesterson, California (selenium), and from longer-term conditions arising from acid rain, overfishing, or hyper-eutrophication. Generic remedial measures tend to be as excessive and recursive as their causes. Except for public health aspects, there are at present no numerical standards or criteria for marine ecological protection from waste discharges. Nonetheless, it is important to consider what is currently known about eutrophication and chronic toxicity.

3.3.1 Eutrophication

Eutrophication has been defined by Likens (21) as "the nutrient enrichment of waters which results in stimulation of an array of symptomatic changes, among which, increased production of algae and macrophytes, deterioration of water quality and other symptomatic changes are found to be undesirable and interfere with water uses".

Although this definition is widely accepted, it implies that enrichment is bad and fails to acknowledge that some nutrient stimulation may be useful, particularly in waters that are naturally deficient in nutrients.

Some examples of acute eutrophication and anoxia are found in marine waters where a restricted water circulation pattern limits the dilution of nutrients and oxygen-demanding material and the resupply of dissolved oxygen. These areas include estuaries and semi-enclosed seas (e.g., the Baltic Sea), fjords, archipelagos, bays, lagoons, and some continental shelf regions. They are not known to ever extend to the open ocean (32).

When the enhanced plant production causes excessive levels of organic matter to concentrate in the water column and sea bottom, bacterial decomposition may severely deplete the oxygen levels. Although mobile organisms may leave the low-oxygen water, sedentary organisms usually succumb and die. Under conditions of more moderate eutrophication, however, overall system productivity of some planktivorous, commercially valuable species (e.g., menhaden) may benefit from the abundance of phytoplankton, while others may suffer declines.

Algae utilize carbon, nitrogen, and phosphorous in specific ratios that are related to their metabolic requirements. Therefore, algal production is regulated by the nutrient in least abundance, which is called the limiting nutrient. In general, nitrogen is not the limiting nutrient since some blue-green algae common to marine and fresh waters are able to fix (i.e., convert into a usable form) nitrogen from the atmosphere. Phosphorous may be a limiting nutrient since, in well-aerated waters, it is often bound to oxygen in the sediments in the form of phosphates and is not immediately available for use by the algae. As organic matter in the sediments is decomposed by bacteria, the phosphates may be released to combine with nitrogen and stimulate plankton growth. Other factors that can limit plankton growth, are temperature and light. Light availability may be limiting when turbidity is high.

The effect of nutrient enrichment on algal production depends on three factors:

1. Whether the added nutrient is limiting. If so, its addition will increase algal production.

2. Whether luxuriant uptake occurs. This occurs when a nutrient is taken up by an organism but is not used immediately. In this case, the addition of even a nonlimiting nutrient might increase production, if the luxuriant nutrient then becomes limiting.

3. Whether the nutrient is in a preferred form. For example, an organism expends more energy in the uptake of nitrogen in the form of nitrates or nitrites rather than as ammonium.

Blue-green algae dominate when phosphate enrichment occurs, particularly in the fresher regions of an estuary, but blue-green dominance is less common in areas of higher salinity. When blue-green algae are dominant, the diversity and stability of the normal phytoplankton community decline particularly in higher trophic levels since blue-green algae are inedible to many plankton-feeders.

Attempts to control eutrophication should begin with an assessment of the cause of eutrophic conditions--whether man-induced or natural. Although this is a somewhat difficult task, it is a necessary first step, since little can be done to prevent natural eutrophication. Man-induced eutrophication occurs principally through two pathways: (1) land runoff, which contains large amounts of nutrients, particularly from fertilized agricultural lands, and (2) nutrient-rich wastes discharged to sensitive waters. Although the first pathway may be controlled by the use of vegetation buffer zones or engineered land-treatment systems, the second is likely to require higher levels of treatment before discharge or moving the discharge location. Tertiary sewage treatment can remove large amounts of nutrients from waste discharges at high costs but the benefits are uncertain in most cases.

The natural concentration of dissolved oxygen is, in most areas, adequate to support marine life. Hypoxia occurs when the dissolved oxygen level reaches a point at which organisms can no longer survive. In marine waters, mortalities generally occur at oxygen concentrations below 2 mg/l. Hypoxic conditions often degenerate to anoxic conditions. Anoxia occurs below the level at which oxygen can be measured. When conditions become anoxic, sulfate-reducing bacteria thrive and release toxic hydrogen sulfide into the water. A number of basins in the world--such as the Black Sea--are characterized by poor circulation and naturally occurring anoxic conditions.

Some fjords have natural anoxic basins owing to reduced circulation and the resultant stratified conditions below the depth of the sill at the mouth of the fjoord. As organic matter accumulates on the bottom, the available dissolved oxygen is metabolized by bacteria until none remains. The anoxic bottom waters can occasionally affect the oxygenated surface waters when coastal storm surges cause large intrusions of oxygenated seawater to flow over the sill. This forces toxic, deoxygenated bottom waters rich in hydrogen sulfide toward the surface, which may kill off organisms living there. In certain temperate locations, anoxia occurs seasonally. Coastal waters become stratified only in summer. Phytoplankton productivity increases in the spring as a result of increased temperatures and solar radiation, and an abundance of nutrients. Subsequently, organic matter accumulates on the bottom, and as it decomposes oxygen concentrations in the bottom waters decrease. In severe cases, this gives rise to restricted circulation, and to anoxic conditions. Nutrient and organic inputs from waste discharges and runoff can contribute to the formation of anoxic conditions. As in the case of eutrophication, it is difficult to distinguish between anoxic conditions caused by waste discharges and runoff from those that occur naturally since waste discharges and runoff may in fact contribute monotonically to naturally existing hypoxic or anoxic conditions or tendencies (see Sec. 15.2).

Whereas eutrophication and anoxia may cause permanent damage to the ecosystems of small, land-locked freshwater bodies, these conditions are more likely to be transient in coastal waters. In many areas where eutrophic and hypoxic conditions have led to deterioration in water quality, attempts have been made to correct these problems by controlling pollutant (nutrient) inputs to each of the various segments of the water body. Segmentation can be used as an analytical tool since it treats the water body as an interrelated ecosystem composed of physically, chemically, and biologically diverse areas (37, 38). This concept of segmentation is being applied to water-quality management in, among other areas, (1) the Great Lakes, which have been divided into zones having similar nutrient and chlorophyll a levels for the purpose of monitoring eutrophication, (2) San Francisco Bay, which was separated into six major areas according to flushing characteristics, to determine acceptable sites for discharging treated sewage effluents, and (3) Chesapeake Bay, which has been divided into forty-five segments on the basis of circulation, salinity, and geomorphological characteristics. The segmentation concept has also been applied to the tidal portion of the Thames River, and it can be used in bays, estuaries, and other coastal waters where circulation is restricted.

Natural systems, particularly tidal and estuarine marshes, are important in removing nutrients from receiving waters. Such areas are characterized by high plant productivity and incorporate the wastes into their yield of organic plant material that supports estuarine species and provides food and cover for waterfowl and mammals. Because of the special value of these natural systems in the role of pollution assimilation, it is important to avoid expedient filling of these systems.

3.3.2 Toxic Wastes

Waste discharges include inorganic and organic compounds that are either suspended in the water column or in the sediments. Some of these chemicals, particularly certain trace metals and organic compounds, can be toxic to marine life. As yet, little is known about the physical, chemical, and biochemical processes controlling transfers of toxic compounds within marine ecosystems (28). (Table 3-4 summarizes recent information on the relative importance of contaminants in the New York metropolitan area.) It is known, however, that the concentrations of most toxic metals do not increase as they move upward through the food web (biomagnification), but that the concentrations of many organic chemicals do. Furthermore, the uptake of contaminants by organisms does not necessarily harm them nor does increased time in the discharge area always result in elevated contaminant levels. Many marine organisms have developed detoxification mechanisms for metals in their environment, since metals occur in the marine environment naturally, as well as being transported there by waste discharges. Few marine organisms have mechanisms to detoxify synthetic organic compounds, although some may excrete organics and others may detoxify them by various metabolic processes. In contrast, the toxicity of some organics may be increased in marine organisms as they are metabolized and converted to other compounds.

Benthic organisms immediately adjacent to an outfall are most likely to be affected. The size of the area affected depends on the degree of flushing and dispersion in the discharge area, the topography of the seafloor, the sedimentation characteristics of the waste, and the quantity of waste discharged. As the deposited organic matter degrades, dissolved oxygen decreases in the pore waters of the surface sediments. Species of benthos unable to adapt to the altered substrate are replaced by opportunistic species that prefer more organic-rich sediments. In situations where an existing low-organic substrate is replaced by organic-rich sediments, the diversity of benthic species decreases, whereas the abundance and biomass of opportunistic species increase. The overall effect of such an alteration of the benthic community is generally unknown. Exclusion of some benthic species may adversely or beneficially affect food-specific carnivores at higher trophic levels, whereas other carnivores may not be affected at all. Researchers in Southern California have observed some changes in the species composition of predatory bottomfish in the vicinity of a treated sewage discharge; this may be a result of a change in benthic species (26). Whether this change was beneficial or detrimental could not be determined.

TABLE 3-4

Classification of Contaminants of the Hudson-Raritan Estuary With Respect To Their Importance in Ecosystem Protection

Contaminant	Relative Importance (number 1 is high)
Metals	
Arsenic	3
Cadmium	2
Chromium	3
Copper	1
Lead	2
Mercury	1
Nickel	3
Selenium	1
Silver	2
Zinc	2
Petroleum Hydrocarbons	
Alkanes	3
Polycyclic aromatic hydrocarbons	1
Total hydrocarbons (oil and grease)	1
Pesticides	
Aldrin	3
α-BHC	3
Chlordane	1
Dieldrin	1
DDT and metabolites	1
Endosulfan	3
Endrin	1
Heptachlor	2
Heptachlor epoxide	2
Kepone	3
Lindane (γ-BHC)	2
Mirex	3
Trans-nonachlor	1
Toxaphene	3
2,4-D	3
2,4,5-T	3
Halogenated Hydrocarbons	
Brominated benzenes	3
Brominated diphenyl ethers	3
Chlorinated anilines	3
Chlorinated benzenes	3
Chlorinated dibenzodioxins	2
Chlorinated dibenzofurans	2
Chlorinated diphenyl ethers	3
Chlorinated ethanes	3
Chlorinated ethylenes	3
Chlorinated methanes	3
Chlorinated paraffins	3
Chlorinated phenols (other)	3
Chlorinated styrenes	3
Chloronitrobenzenes	2
Dechloranes	3
1,2 dibromo-3-chloropropane	3
Dichlorobenzidine	3
Halogenated alkyl esters	3
Hexabromobenzene	3
Hexachlorobenzene	2
Hexachlorobutadiene	3
Hexachlorocyclopentadiene	3
Pentabromotoluene	3
Pentachlorophenol	3
Polybrominated biphenyls	3
Polychlorinated biphenyls	1
Polychlorinated naphthalenes	3
Polychlorinated terphenyls	3

Source: Adopted from Breteler (2).

Finfish can swim away from discharge areas and avoid irritants. However, since waste discharges typically offer readily available food sources, many finfish are usually attracted to the discharge area. Productivity and growth of certain finfish species may actually increase as they expend less energy in obtaining food.

Measurable effects of waste discharges are limited to the immediate area around an outfall. Far-field effects are extremely rare and any such effects are likely to occur among organisms that have spent time in the discharge area. For example, migratory fish may accumulate toxic substances in a discharge area and transport trace amounts of them elsewhere. Any effect that this has on the fishery, and subsequently on ecosystems, is almost certainly negligible where only a single outfall is concerned. However, multiple outfalls in an enclosed region might give rise to a more significant broad-scale effect.

Despite the concern regarding the toxicity of heavy metals, there is little documented evidence from field observations that metals have a significant adverse impact on marine organisms or their predators (including man), except in the extreme hydrographically and geographically unique case of methyl-mercury poisoning at Minimata, Japan. The only evidence that correlates marine species mortality with metal concentrations in discharge plumes is laboratory data that have not been, and probably cannot be, normalized to actual marine environmental conditions.

A heavy metal (indeed, any substance) is toxic to an aquatic organism when present in levels in excess of an organism's tolerance. These levels are ordinarily greater than those found even in waters adjacent to point sources of pollution (1). Moreover, the tolerance levels of organisms are highly variable between and within species.

3.3.3 Minamata--A Special Case

The potential for problems due to release of large amounts of toxic heavy metals into the marine environment first gained worldwide public attention when a number of deaths in Japan were linked to the consumption of mercury-contaminated seafood. (The incident is summarized in Chapter 15.)

It is now known that this incident is not typical of mercury contamination in marine waters. Very large quantities of mercury in the form of methyl-mercury were released into Minamata Bay. This form is much more toxic to, and more easily accumulated by, marine organisms than inorganic mercury, which is commonly found in recent and ancient terrestrial and marine sediments, including coal and petroleum formations. In addition, the Minimata discharges were flushed into estuarine waters and the mercury contamination was confined to a relatively small area with limited dilution and dispersion; as a result the mercury concentrations reached very high levels.

Mercury levels in the ocean appear to have remained nearly constant over many years, and fish such as tuna and swordfish contain naturally high levels of mercury (23). The incident in Minimata is instructive and, because of the publicity surrounding it, discharges of large quantities of methylmercury or other toxic metals into the estuarine environment are not likely to occur again. Although some fishing areas in Japan, Sweden, and Italy have been closed because of high mercury concentrations in fish populations, no recently reported levels approach those found in the Minamata incident.

Other toxic elements in the marine environment that are generally considered to be of concern to public health include cadmium, lead, silver, arsenic, chromium, and selenium. None of these has been studied to the same extent as mercury. Elevated levels of these and several other elements have been reported in certain marine coastal areas. Meanwhile, all reported health effects associated with these metals apparently have been caused by food grown in contaminated soil or direct human exposure in the work or living environment (33).

3.3.4 Synthetic Organics

Recent evidence has shown that man-made synthetic organic chemicals pose greater health risks than metals. Many of these compounds persist in the marine environment and tend to accumulate in the fatty tissues of organisms. Although synthetic organics include an enormous variety of compounds, the chlorinated insecticides and polychlorinated biphenyls (PCB's) have received the most attention. Most of these chemicals enter the marine environment in agricultural runoffs of pesticides and herbicides and industrial discharges. Only a small fraction derive from municipal discharges, except where there are major industrial or chemical inputs to the sewage system.

Reports of mortalities of marine species caused by organic chemicals are rare, with the most notable exception of the death of some captive seabirds, which was later ascribed to the consumption of fish with a high DDT content (see Chapter 2). These substances can produce sublethal effects, however, which may alter metabolic, reproductive, physiological, and behavioral responses in marine species. Fin rot disease, which occurs primarily in bottomfish or flatfish, has been observed in some discharge areas (28). The distribution and incidence of this disease is highly variable and it has been found in fish far removed from discharge areas. There is some evidence to suggest that the increased incidence of this disease may be correlated with the presence of chlorinated hydrocarbons and high levels of toxic metals in sediments (28).

Petroleum hydrocarbons constitute another group of persistent organic compounds, some of which are carcinogenic and appear to be increasing in the marine environment. Major sources of these compounds are shipping operations and land runoff, rather than municipal discharges.

3.4 Ocean Dumping

For more than a century, the coastal oceans have been used to varying degrees to dispose of dredged material, industrial wastes, and sewage sludge. Although ocean waste disposal practices are largely up to the discretion of individual nations, the London Dumping Convention was adopted in 1972 to control ocean dumping of certain wastes from vessels. Several similar regional agreements exist throughout the world. As of January 1, 1983, fifty-two states had ratified or acceded to the convention (18). These parties have agreed not to dump certain substances in the sea, and only to dump others with special care. Under the convention, permits for ocean dumping off their shores are issued by the participating states under the auspices of the International Maritime Organization (IMO).

The major dumpsites of the contracting parties have remained at more or less the same locations since the London Dumping Convention was introduced. The English Channel, coastal areas on the southern side of the North Sea, and the New York Bight in the United States (see Section 15.2) contain the greatest number of industrial waste and sewage sludge dumpsites (5). Dredged spoils and industrial wastes are dumped in areas throughout the world. In addition, four countries (United States, United Kingdom, Japan, and Federal Republic of Germany) currently dump large quantities of sewage sludge into the ocean.

The methods by which various wastes are discharged from a disposal vessel vary with the type of waste being dumped and the ship's design. Dilution and dispersion depend on the waste, the disposal site, and the time scale of the physical processes involved. Sewage sludge is typically released in the wake of a moving vessel, which promotes rapid initial dispersion and dilution. Industrial and acid wastes are normally towed in special rubber-lined barges and, similarly, are pumped through discharge pipes at keel level, to provide rapid dilution in the turbulent wake. Dredge spoils, the largest source of solid wastes to the ocean on a mass basis, are usually released en masse through doors in the dumping vessel. These dredged materials often contain large quantities of coarse sand that accumulate in a pile at the dumpsite, even though the fine-grained, contaminant-rich particles tend to escape and be dispersed during disposal and through later resuspension.

The variability of natural factors within marine ecosystems, makes it difficult to establish precise cause-and-effect relationships involved in ocean dumping. In general, it is accepted that the effects of ocean dumping of sewage sludge are similar to those observed in the vicinity of sewage sludge outfalls, although the latter may be more acute since the initial dilution achieved by dumping is greater. The effects of both techniques depend largely on the dispersive ability of the disposal site. Wastes are swept away from a dumpsite or outfall location in directions and at speeds determined by the mean flow of the long-term current. However, dispersion of the sewage particles is also controlled by tidal currents and wave-induced turbulence. In areas where tidal velocities or wave action, or both, are strong, dispersion is more rapid, and any particles in the sewage reaching the sediments near the outfall or at the dumpsite are effectively resuspended and transported away from the discharge site. Sites with strong tidal and wave-induced mixing are often referred to as dispersive sites since little or no accumulation of organic particles occurs in the sediments. In contrast, sites with poor mixing are often referred to as containment sites since particles of the sewage tend to accumulate in the sediments at the discharge site. Even these sites are leaky buckets--those that receive the largest loadings of contaminants in the sediments near the discharge site retain only a very small fraction of the waste, and the sewage particles are widely dispersed and assimilated. The current practice in selection of disposal sites is to provide the best dispersion characteristics and, therefore, to minimize accumulation of sewage particulates in the sediments at the discharge site.

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CHAPTER 4

HYDRAULIC DESIGN FOR OCEAN OUTFALLS

4.1 Introduction

This chapter explains the terms and the processes by which an appropriate discharge site is selected. The discussion then proceeds to the hydraulic design of the diffuser and other parts of the outfall system.

Sections 4.2 and 4.3 introduce the terms and conditions, while Section 4.4 describes analytical tools and approaches and presents some basic formulas. For complete descriptions of these tools, see Fischer, et al. (6).

The central questions of outfall siting are addressed in Section 4.5. The techniques used to answer these questions and to arrive at the engineering decisions as to where the outfall is to lie are explained in terms of the concepts and techniques introduced in the earlier sections.

Outlet design and initial dilution computations are presented in Section 4.6. Other important hydraulic considerations, including internal hydraulics upstream of the outlet section and external hydrodynamics forces due to waves and earthquakes, are presented in Section 4.7. This order of presentation follows the principal sequence of steps of proper ocean outfall design. However, the design process is iterative, in that it generally involves compromise among design factors, and one may have to consider siting, outlet design, and other hydraulic and nonhydraulic factors (such as structural, financial, or public pressure) several times in turn before arriving at a final design.

4.2 Concepts and Definitions

Listed below are brief definitions of the most common terms and concepts encountered in outfall siting and hydraulic design. Further analytical discussion of some of the concepts is presented in later sections.

Dilution and relative concentration. Dilution is defined herein as the ratio: total volume of seawater-effluent mixture/volume of effluent that would be found in a sample of seawater-effluent mixture at a particular location. The reciprocal of the dilution ratio is the relative volumetric concentration,

$$p = 1/S$$

Hydraulic performance of an outfall is usually quantified in terms of the dilution, S, with a large number such as S = 100 or S = 500 implying more complete mixing of effluent with ocean water than a low number such as S = 10. To compute the multiplicative effects of successive dilutions and concurrent decay of an effluent constituent, it is usually more convenient to work in terms of relative concentration, p.

Effluent plume. Wastewater discharged from an ocean outfall mixes with surrounding seawater and is swept away by ambient currents. When the effluent discharge is continuous, the mixing and drifting effluent appears as a continuous plume, much like a plume of smoke from a chimney. Effluent dilution increases and concentration decreases with increasing distance from the discharge. At any one instant, the effluent concentration in a plume cross section roughly follows a normal (or Gaussian) distribution about the centerline, modified by a patchiness due to turbulence. On a larger scale, the plume path also fluctuates because of turbulence in the same manner that an atmospheric smoke plume from an open fire surrounded by seated people blows in the face of first one person and then another, although not in the faces of all people at once.

Buoyant plume; initial dilution. In many cases, an essentially freshwater effluent discharged into seawater will form a plume of water less dense than the surrounding water and will rise toward the sea surface in a buoyant plume. In the process, the effluent undergoes initial dilution with surrounding seawater. After the effluent reaches the sea surface (or other terminal rise height if kept submerged by stratification) the initial dilution phase ends; it is followed by a gravitational spreading phase with dispersion by the turbulence and shear flow of the ambient current.

Disappearance/Decay/Dieaway; T_{90}. Coliform bacteria are the most common nonconservative sewage effluent constituent considered in outfall design. Bacterial concentrations in an effluent plume decrease with increasing time and distance from the discharge point. Bacteria concentrations are reduced not only by physical dilution, but also by mortality and by sedimentation, as discussed in Section 3.1.3. The rate of reduction in numbers is usually described by a first-order decay function of the form

$$C = C_o e^{-k\,t} = C_o\ 10^{-t/T_{90}} \tag{4.1}$$

It is customary to express the decay rate as the time required in hours for 90 percent of bacteria in a sample to disappear; where $k_e = \ln 10/T_{90}$. For a T_{90} of 0.5 hr., a bacteria concentration at time zero of 10,000,000 /100 ml will be reduced to 1 /100 ml in 3-1/2 hours.

Advection, dispersion, shear (stirring), and diffusion (mixing). These are described in Chapter 2 and defined analytically in subsequent sections of this chapter. Typical advection patterns for ocean outfall effluents are shown in Figure 4-1.

Density stratification. Density stratification (see Chapter 2) often blocks the rise of a buoyant effluent plume, forcing it to spread laterally, so that subsequent lateral spreading of the effluent plume is confined to a subsurface water layer (Figure 4-2). This subsurface plume submergence is a desired design objective if it keeps a wastewater plume away from recreational surface waters. On the other hand, it may be a condition to avoid if oxygen depletion of subsurface waters is an essential design concern.

Fig. 4-1. Advection and Dispersion of Effluent Discharges

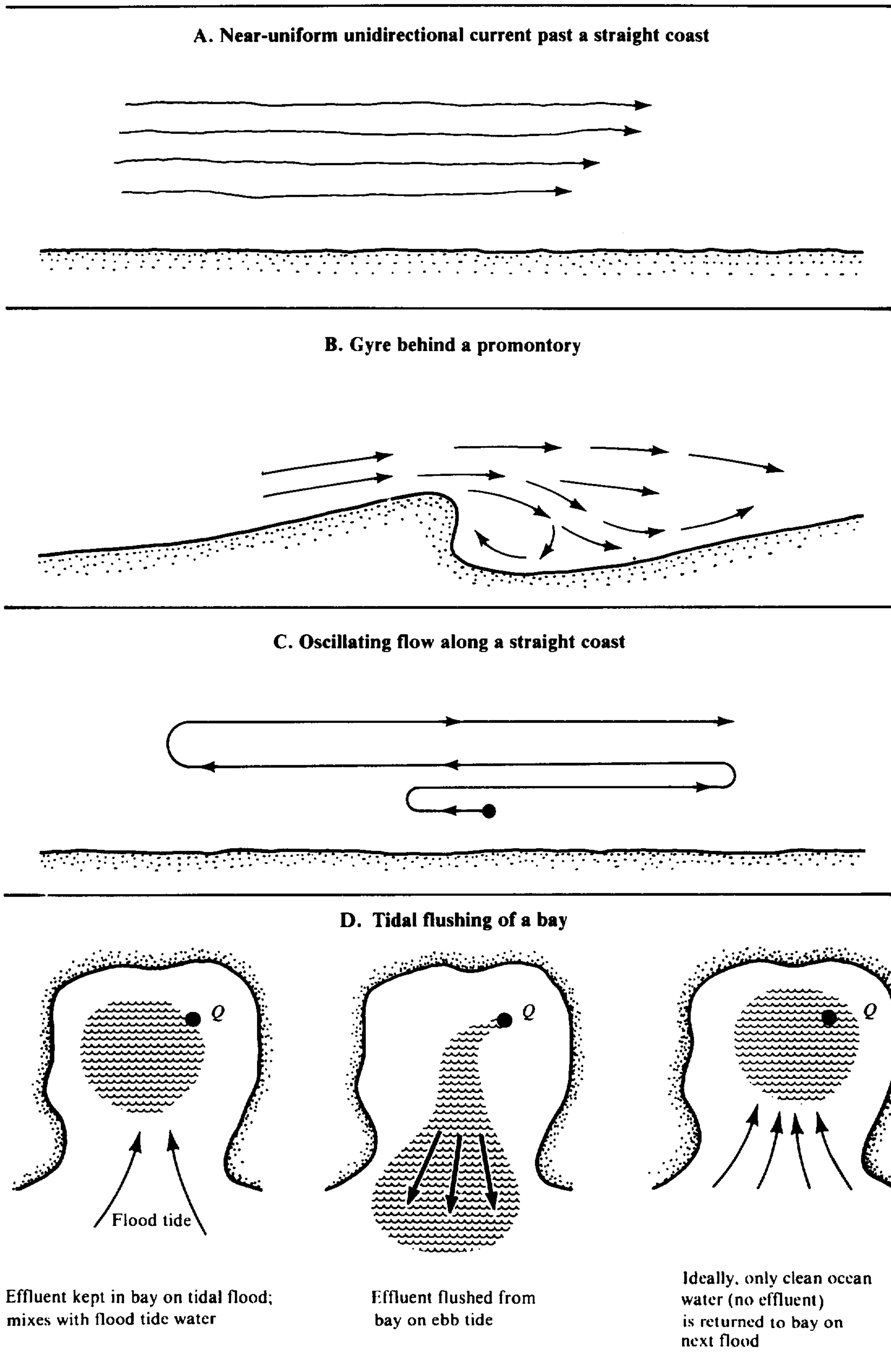
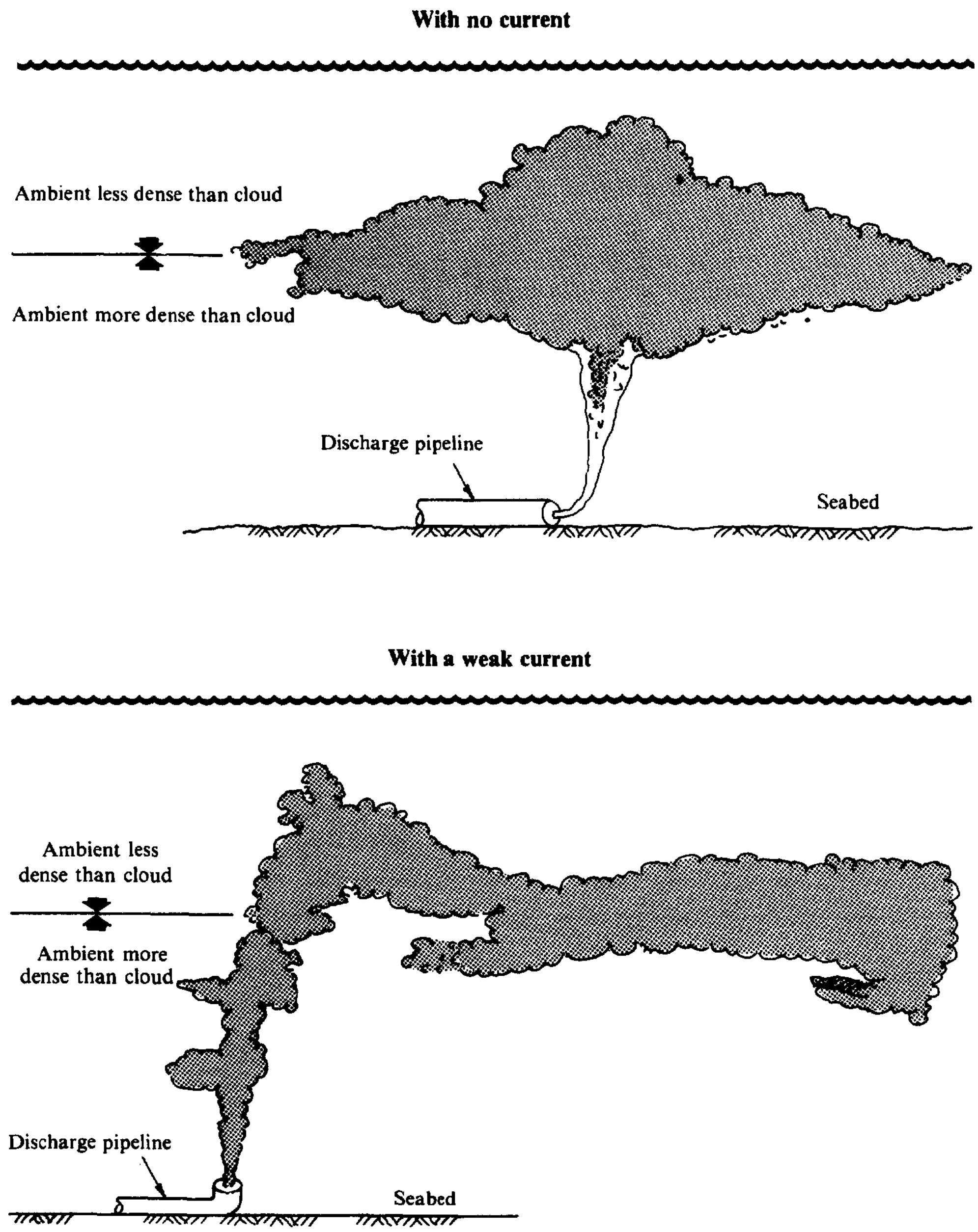


Fig. 4-2. **Buoyant Plumes Kept Submerged by Ambient Density Stratification.** Note that submergence is independent of angle of jet.



Source: Top, Fan (5); bottom, Wright (15).

Particle Settling. Most wastewater effluents contain solid particles that flocculate in seawater and settle at rates controlled by density, currents, and turbulence. Some remain on the bottom while others are moved by saltation or resuspension. When accumulation is sufficiently rapid, sediments become anaerobic and support an abundance of specialized pollution-tolerant species that feed on the detritus. Effects of sediment oxygen demand are ordinarily negligible, except, for example, when an untreated papermill waste with thousands of mg/l suspended solids discharges to a shallow, sheltered bay with little current or wave action.

Typical settling velocities for sewage particles and the current velocities needed to suspend the particles are given in Table 4-1.

TABLE 4-1

Settling Velocities and Current Velocities
For Resuspension Of Sewage Effluent Particles

	Percentage of Settling Particles with Settling Velocities Greater than Indicated	
Particle Settling Velocities, cm/sec	Raw Sewage	Primary Effluent
1.0	5	-
0.5	20	-
0.1	40	5
0.05	-	-
0.01	60	20
0.005	-	30
0.001	85	50
Bottom Current Velocity, cm/sec, for resuspension		
Resuspension Unlikely	0-6	0-6
Resuspension Possible	6-30	6-20

Source: Adapted from USEPA (14).

4.3 Qualitative Descriptions of Receiving Waters

Receiving water characteristics described in Chapter 2 are summarized below with respect to outfall siting and design. Quantitative analytical and engineering procedures for outfall siting and hydraulic design are presented in the sections that follow.

4.3.1 Coastal Waters

Whatever the source of current motion, average currents flow parallel to the coastline and to depth contours, but onshore-offshore components arise periodically owing to waves, tides, and diurnal land-sea breezes. This means that effluent plumes discharged offshore are seldom carried directly out to sea or directly onshore, but usually tend to drift parallel to the shore.

Shallow coastal embayments often attract development because they offer a protected anchorage and because the adjacent land is usually flat and is easy to build upon. Many such bays are now heavily polluted by industrial and domestic effluents. Unfortunately, the flushing capacity of embayments is less (sometimes by orders of magnitude) than that of the adjacent sea. New York Bight (United States), Izmir Bay (Turkey), Abu Kir Bay (Egypt), and Cockburn Sound (Australia) are examples of such bays. There are structural advantages in building outfalls discharging to such bays. The wastewater collection points are on the bayshore, bottom conditions are often structurally favorable, and wave action is minimal. However, because of their limited assimilative capacities, such bays require either a small discharge rate, or a high degree of treatment.

Coastal lagoons shoreward of barrier beaches occur along much of the world's coastlines. Here, brackish water bodies connect to the sea via occasional inlets through the barrier beach. Cities and towns are often built on the barrier beaches themselves (e.g., Lagos, Alexandria, and Miami Beach), or on the mainland shoreward of the lagoons (Titusville, Miami). Sewage from such cities is often discharged to the lagoons through outfalls easily constructed in calm and shallow water.

However, the environmental balance of coastal lagoons is even more sensitive than that of the less constricted coastal embayment. Lagoons also tend to harbor intense ecological activity, and at the same time have much less dispersive capabilty than the open ocean.

4.3.2 Rivers

This report is concerned primarily with ocean disposal of wastes, and secondarily with estuarine disposal. River disposal is included for purposes of comparison. A river is a freshwater stream with unidirectional flow and with velocities that are affected by tides in lower reaches. The plume from a wastewater outfall will be diluted and dispersed much as in the open ocean, except that (1) buoyancy of the plume is less because, unlike the ocean, the stream is not more saline and hence significantly heavier than the effluent, and (2) dispersion due to turbulent diffusion will be enhanced by the shear flow because of the channel curvature.

Plume analysis in rivers attempts to determine how far downstream the plume will be advected before it is more or less completely mixed across the river cross section. Complete mixing across the cross section achieves the maximum dilution, and justifies one-dimensional analyses of chemistry or biochemistry such as the Streeter-Phelps equation. Where there is a water supply intake on the opposite shore downstream from an outfall, however, a design criterion may be that the wastewater plume should not be fully mixed across the stream at the water intake. Some computational techniques for plume dispersion in rivers are presented in Section 4.4.5.

4.3.3 Estuaries

Most large coastal cities are located on estuaries that provide harbors for naval and commercial shipping, water transport routes to the interior, and, unless they are too badly polluted, valuable fisheries. Wastewaters are almost always discharged to these estuaries, unless major engineering measures are undertaken to discharge the water elsewhere.

Islands in estuaries strongly affect tidal flows. Sewage discharged at a given point near an island may be pumped entirely around an island by tidal action rather than flushed directly out to sea. Similarly, circulatory pumping around a submerged shoal will cause a net upstream flow in one channel and a net downstream flow in the other.

4.4 Methods of Analysis for Outfall Siting

Field studies, physical models, and numerical models all play a role in outfall siting decisions. Each should be preceded by an initial study to establish perspectives and proportion for their planning.

4.4.1 First Approximation

Among the questions to be answered in planning ocean disposal systems are: (1) How do outfall alternatives relate to the sewage collection and treatment options, present and future? (2) How many outfalls should there be? (3) Should the discharge be to the estuary or to the ocean? (4) How far offshore and to what depth should the outfall extend? (5) Should a diffuser be provided? (6) Is plume submergence likely?

It usually takes months of study to arrive at the final answers. In the meantime, reasonable approximations may be provided after only a few days of study, to determine what will be needed for scoping, scheduling, budgeting, and conducting the full study program.

The first step is to collect all the readily available tables, charts, and atlases pertaining to the area's hydrography, topography, geology, meteorology, land use, and fisheries (see Chapter 2). Sources include engineering, earth sciences, and geography departments of universities, ship supply stores, nearby airports, harbormasters, and government agencies of various maritime nations. Many of the latter publish their own versions of the nautical charts, borrowing from one another for correcting and updating. (As of 1983, the U.S. Defense Mapping Agency, British Admiralty, and Commonwealth country charts show soundings in feet and fathoms; most others show soundings in meters.) Previous engineering feasibility studies, plans, and reports for water supply, waste disposal, cooling water discharges, harbor or shoreline development, or oil pipelines are particularly valuable.

Civil engineers, scientists, and others engaged in an outfall study are seldom the first and never the last to study a given site. It is arguably a professional obligation to report study results so that they may be easily found and completely understood by those who follow. The results and conclusions from such studies--including details on construction methods and costs, particularly on operating experience and costs--should be made widely available.

The next step is to determine the design flow rate, by determining the tributary areas, populations, and design flows for the outfall system. Sewage flows may range from 100 to 800 liters per capita per day, depending upon economic development and land use. Groundwater infiltration and stormwater contributions should be accounted for.

Advection. To determine where the effluent from an outfall will go, obtain all previous current data. If conditions permit, estimate current speeds from tidal prism considerations. Apply information gained from Chapter 2 and Section 4.3. Query local boatmen.

Initial Dilution. For a simple open-ended pipe of diameter d, discharging into water of depth D, the initial dilution of the resultant plume is essentially D/d, assuming a discharge velocity of the order of 1 m/sec. (2).

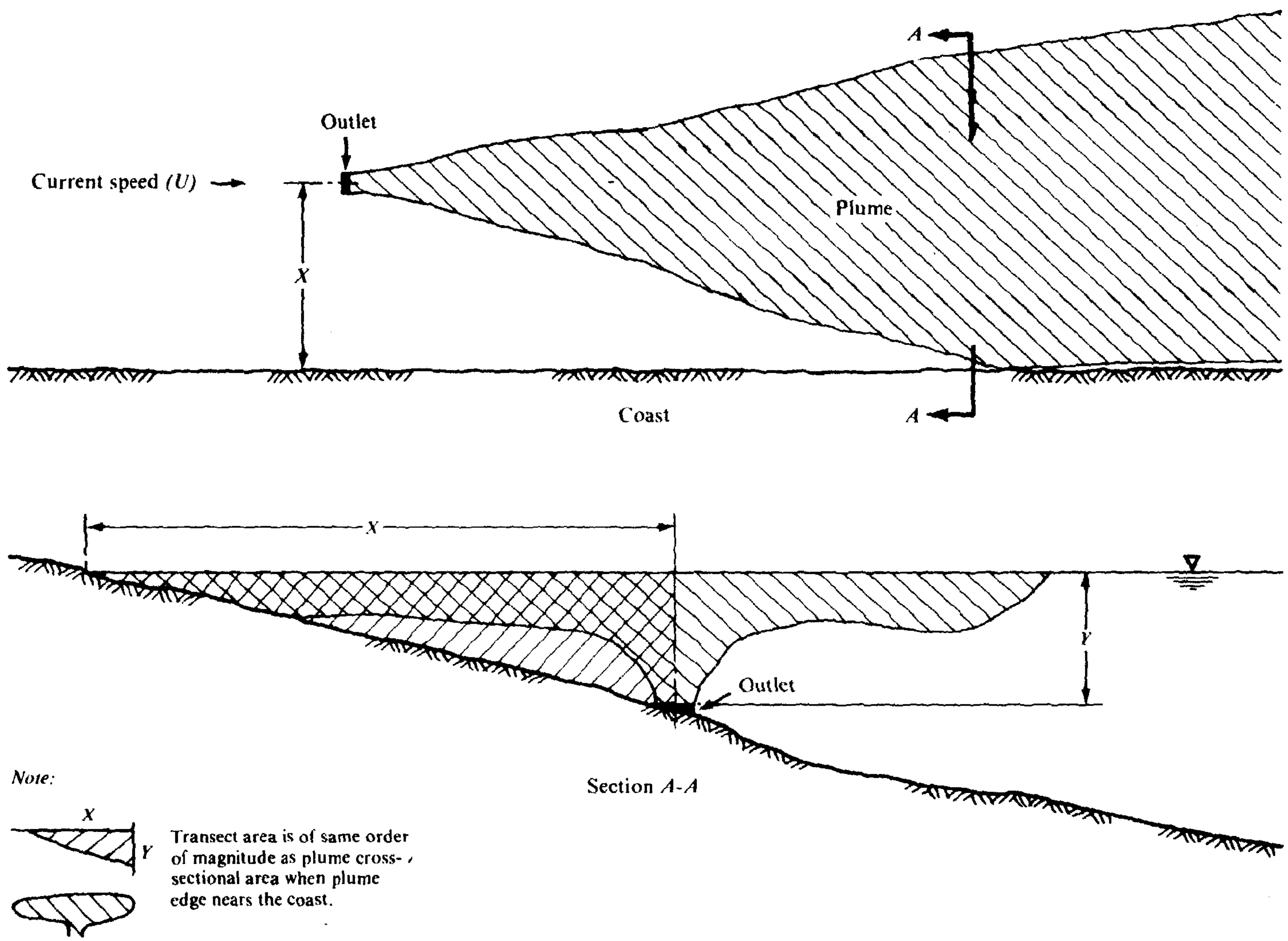
For a multiport diffuser of length b aligned perpendicular to an ambient current of velocity U, in water of depth D, the ambient water flow available for diluting the effluent is of the order of UbD, (a flow velocity multiplied by an approximate flow depth multiplied by an approximate flow width). The initial dilution is therefore of the order of UbD/Q_e, in which Q_e is the effluent flow rate.

These rough approximations may be compared with the more precise relationships given in Section 4.6.

Ultimate Dilution. In a river, the ultimate physical dilution of the discharge cannot exceed Q_r/Q_e, the ratio of the river's flow rate to the effluent flow rate. In an estuary, the ultimate dilution cannot exceed $(Q_r + Q_t)/Q_e$ where Q_t equals the volume of the tidal prism divided by the ebb flow period.

In unidirectional flow along an open coast, the estimate of ultimate dilution is similar to that of a river, but is somewhat more complicated. An effluent plume entrained in flow parallel to a coast will be dispersed across a cross-sectional area that gradually increases with distance and time. The diffuser is located x m offshore, in y m depth of water (Figure 4-3). The plume is advected along a path parallel to the shoreline, and spreads with respect to time and distance from the discharge point. At some point, the plume edge will reach the shoreline, and the shore will be lapped by waters containing a seawater/effluent ratio whose value one wishes to know. That ratio is of the order of uA/Q_e, in which u is the ambient current speed, A is the plume cross-sectional area at the point of interest, and Q_e is the discharge flow rate. The approximate value of A is likely to be of the same

Fig. 4.3. Estimate of Significant Far-field Dilution off an Open, Straight Coast



magnitude as the cross-sectional area of flow between the shore and the discharge point:

$$A \approx \int y(x)\, dx$$

where y is the water depth at a distance x from shore.

Example: If the effluent discharge, Q_e , equals 10 m^3/sec and the current speed u is 0.1m/sec, roughly how far offshore must be the discharge site be to ensure dilution of the order of 1,000 by the time the effluent plume reaches the beach? The shore and bathymetric contours are straight and parallel, and the bottom slope is constant at 1:50. Note that there is nothing magic about 1,000:1. It corresponds to both a 99.9 percent removal and one cupful in a bathtub.

Try an offshore distance of 1 km. For this trial, A is on the order of 1,000 m x 10 m average depth, or 10,000 sq m. Thus uA/Q_e is 0.1 m/sec x 10,000 m^2/(10 m^3/sec), or only 100. This distance will not give the desired dilution of 1,000. This result has nothing to do with diffuser design; changing the diffuser design will do nothing to alter the fact that there is simply inadequate dilution water available at that site.

Try an offshore distance of 4 km. For this trial, A is of the order of 4,000 m x 40 m average depth, or 160,000 sq m. Thus uA/Q_e is of the order of 1,600, which is of the order of magnitude desired.

4.4.2 Field Surveys

For outfall siting and design, field studies most commonly include current measurements, density profiles, bathymetry and other seabed surveys, water quality sampling, sediment sampling, wave climate measurements, and diver surveys and photographs. Wind data and water level data are usually measured near the project area by government agencies, but may be taken during the survey as well, or instead.

Sampling for water quality is not discussed here but is easily fitted in with other survey tasks described below.

4.4.2.1 Current Measurements

Current measurements are made by either "Lagrangian" or "Eulerian" methods. Lagrangian measurements include studies with drogues, drifters, or dye in which the path and velocity of a particle are traced in the flow field. Eulerian measurements provide large quantities of data from current meters at fixed points in space and where data are subsequently transformed to Lagrangian data and ultimately to circulation patterns and flows. Each type of measurement has its place in a complete set of field studies.

Aerial surveys may provide valuable qualitative insight into flow patterns where water bodies with differing densities or turbidities converge. Tidal outflows, inlet jets, and promontory gyres may stand out sharply. Photographs should be taken with appropriate light filters to maximize visual contrast between waters of differing color. Photos from a series of flights at hourly or two-hourly intervals over a tidal cycle provide a valuable qualitative understanding of surface flows.

A drogue consists of a weighted sail, a tether, and a surface marker float (Figure 4-4). The sail presents a large surface area to the current that carries it along. The surface marker float enables the surveyor to track and retrieve the drogue; the float also keeps the drogue sail from sinking to the bottom. The sail depth is essentially equal to the length of the weighted tether. A good design has a large sail area, a small surface marker and float so as to minimize wind effects, and a slender tether to minimize near-surface current effects.

There are several satisfactory sail designs. The details apparently matter little; it is reported that lightweight aluminum-and-woven-plastic lawn chairs have been used satisfactorily as sails. Important practical criteria are ease of folding for transport, ease of unfolding for deployment, and low cost (because some will probably be lost).

In a typical deployment, five or six drogues, each with a different tether length, are deployed at one time at a proposed outfall site. Their locations are fixed every half hour or so by bringing the survey vessel close to the drogue (but not touching it), and taking a position reading. This continues usually until sunset or drogue grounding, or until the drogue leaves the study area. Lighted drogues for nighttime deployments require larger surface marker floats, which present more windage and thus are more susceptible to error. Radar reflectors attached to drogues can be tracked at night as well as day from a shore radar station set up for the survey. First, however, the minimum size and weight of the reflectors must be determined as well as the size of the surface marker buoys to support them, and possibly larger sails to minimize windage. Aerial photographic tracking of drogues with large surface floats and fixed reference points is another option. Drogue paths may be related to larger scale river or other shoreline outflows in the area. Recording current meters implanted at a site and recovered after at least two weeks during critical seasons, provide valuable data. Indicating or recording meters deployed over-the-side-of a double-anchored boat for at least a tidal cycle can provide some data but these may be subject to contamination by boat movement. Long-term recording meter implants or drogue studies are preferable.

There are several current meter mooring strategies. One with a good successful record is the J-format shown in Figure 4-5. The meter (or meters) is attached to a taut wire between an anchor weight and a submerged buoy. The submerged buoy minimizes the effects of wave motion to the meters, interference by boats or flotsam, and the risk of theft or vandalism. A marker buoy possibly carrying a radar reflector and a flashing light is moored

Fig. 4-4. Typical Cruciform Drogue

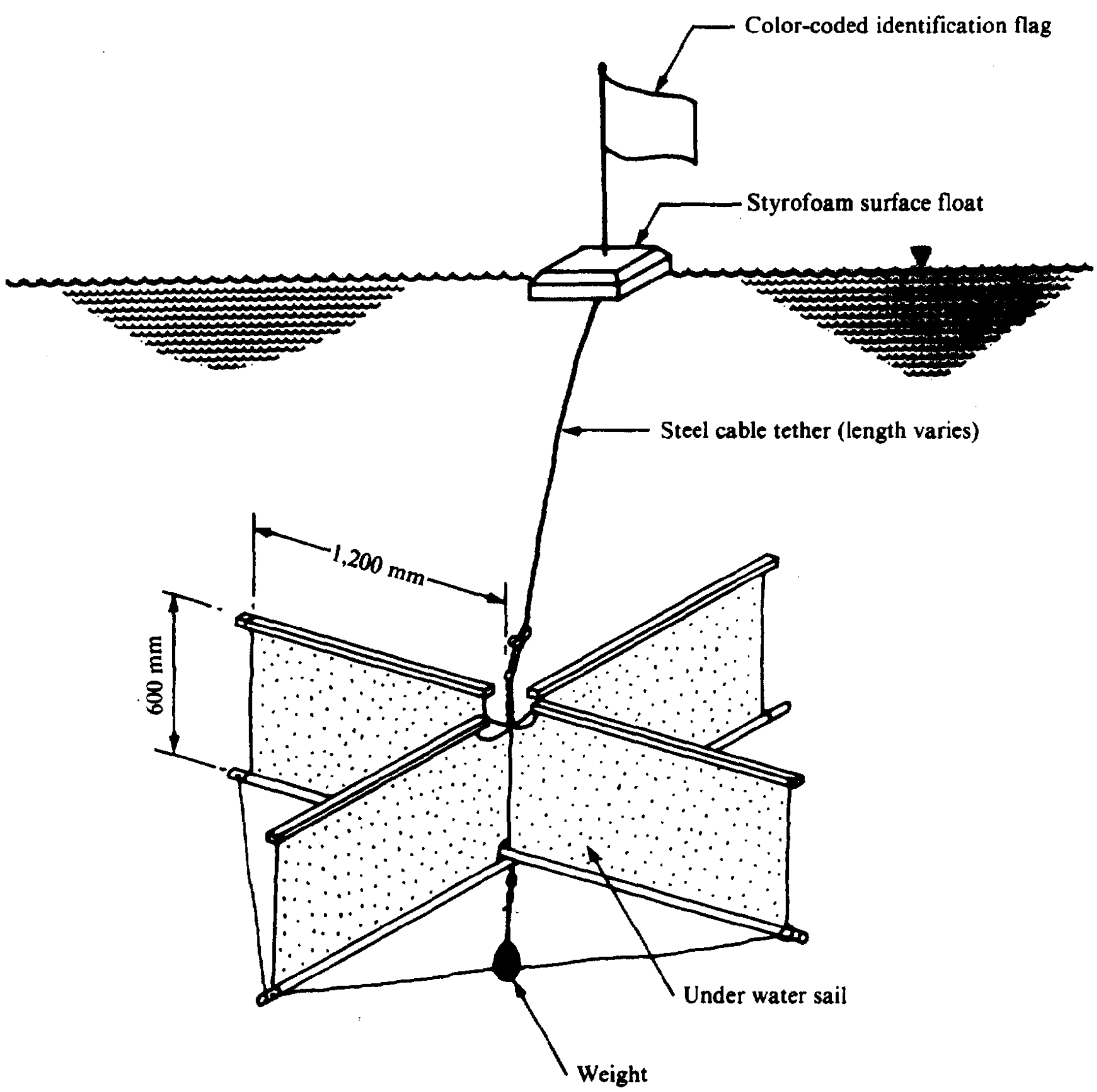
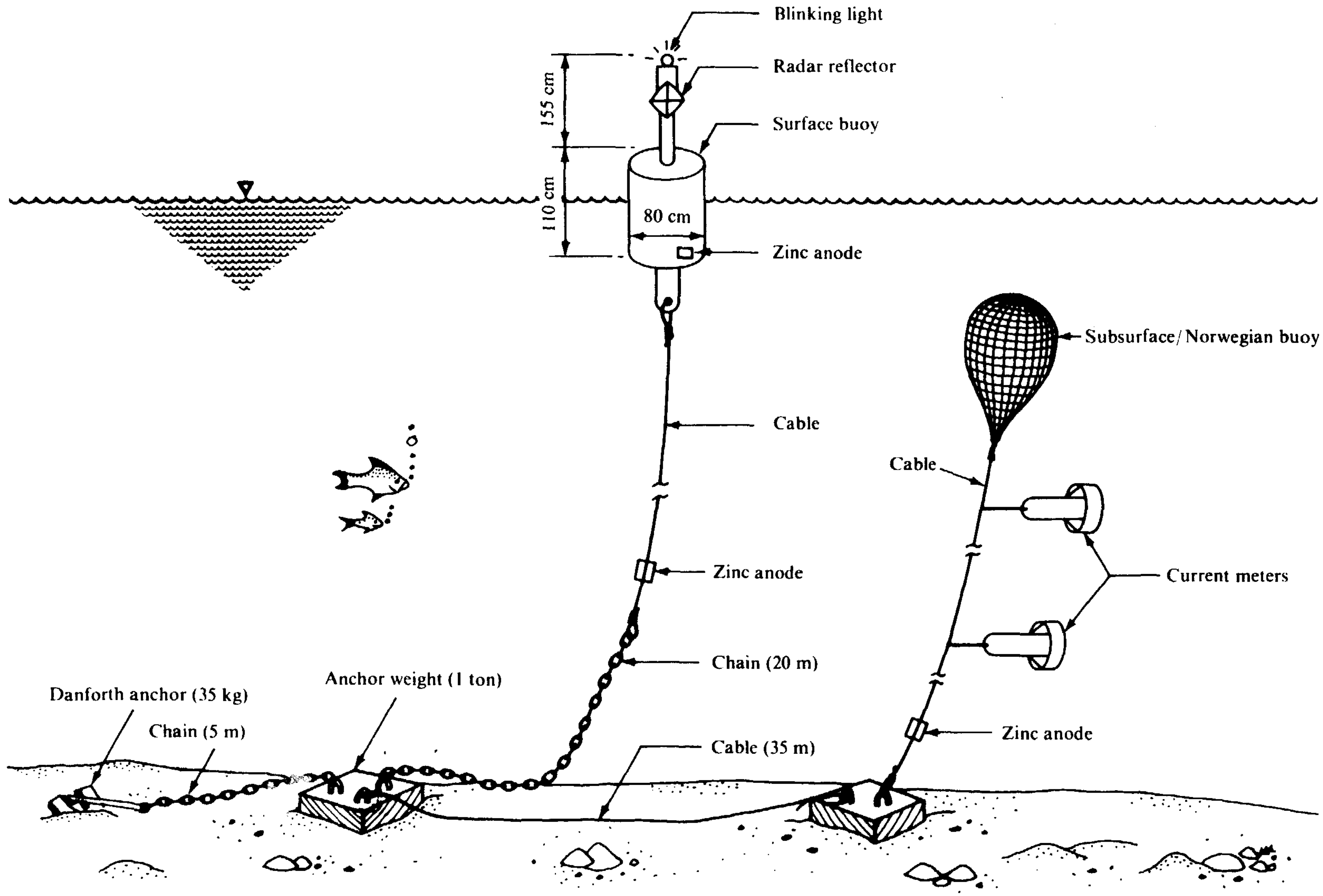


Fig. 4-5. Configuration of Current Meter Mooring



Source: CDM (4).

a few tens of meters away. The marker buoy's anchor is usually a large dead weight, often a railroad wheel with a small Danforth anchor for added insurance against drag. The marker buoy anchor and the meter mooring anchor are joined by a cable strong enough to hoist the meter mooring anchor.

Types of current meters include Savonius rotors, ducted impellers, inclining meters, and vector meters. A Savonius rotor turns about a vertical axis in the same direction for all current directions. A separate vane and compass measures direction. This type is very useful in deep water. In shallow water near shore, it is subject to wave motions that result in erroneously high current speed values. This type of meter is usually capable of recording speeds as low as 20 mm/sec.

The tethered ducted impeller rotates on an essentially horizontal axis parallel to the current. The entire meter unit is field-adjusted to be neutrally buoyant in seawater and is loosely connected to the mooring cable by a tether about 1 m long. The tether is connected to one end of the current meter, which aligns itself with the current direction. Wave motion is largely filtered out because (1) for motions for which displacement is less than the tether length, the meter is carried to and fro so that the wave motion is essentially unconstrained by the tether, with little or no impeller rotation, and (2) for motions of larger displacement, the to-and-fro wave motion induces some to-and-fro impeller rotation, which is mutually cancelling in the Endeco design for averaging and recording current speed. The threshold speed is approximately 20 mm/sec.

The inclining meter has no external moving parts and can be made to measure veolocities as low as 5 mm/sec. The meter is suspended at arm's length from the mooring cable and tilts in response to current direction and magnitude. With the inclining meter built by General Oceanics, wave motion can be assessed and distinguished from net current speed by "burst" sampling, which means taking a large number of samples within a short time. The vector-measuring current meter has two sets of impellers with mutually perpendicular horizontal axes. The version by EG&G reportedly has a speed threshold of less than 10 mm/sec. It has many of the advantages of the tethered ducted impeller meter, and is relatively insensitive to surface and/or mooring motion. It is insensitive to vertical motion. The EG&G instrument, of high quality, also has a high price, two or three times that of some of the other widely used meters.

4.4.2.2 Temperature, Salinity, and Density Profiles

Seawater density is determined by temperature and salinity (T and S) field probes lowered from a survey boat. Recording continuous-profile probes can take a continuous record of temperature, salinity, and depth from sea surface to sea bed. However, for outfall planning and design an instrument that merely indicates T and S is adequate. Readings are observed as the probe is lowered manually from the surface to intermediate levels where T or S gradients change, and to the bottom. Density is conventionally stated as σ_t where $\sigma_t = 1,000\ (\rho-1)$ and may be determined from nomographs (see Appendix D) or, if greater accuracy is required, may be calculated from T and

S data using published tables. Section 4.6.4 describes how density profiles are related to effluent plume submergence. The precision of density measurements should be within one-tenth of the smallest surface-to-bottom density difference that is likely to keep a proposed effluent plume submerged. For example, if a density difference of at least 1 $kg/m^3/m$ ($\Delta\sigma_t = 1.0$) is required to submerge a particular plume, the density measurements should be within $\pm$ 0.1 kg/m^3. The salinity probe should be periodically (if possible, before and after every voyage) calibrated against a laboratory-grade salinometer, which is in turn calibrated with standard Copenhagen seawater.

The probe cable should be weighted sufficiently to hang essentially vertically from the survey vessel, whether drifting or moored. The weights should be at least 1 m from the probes to avoid electrical interference with the salinity (conductivity) probe.

4.4.2.3 Seabed Surveys

Seabed surveys are made to determine topography, texture, and soil characteristics of the seabed. Four classes of survey are: (1) preliminary reconnaissance, (2) systematic broad surveys to select alignment corridors, (3) detailed studies for the final alignment, pipeline design, and construction cost estimates, and (4) construction control. A preliminary reconnaissance is based on published nautical charts showing depth contours and values with some indications of bottom conditions (rocky, sandy, soft, shells, hard, sticky), and by use of a recording fathometer to locate sampling or current meter stations and to reveal unsuspected submarine ridges, canyons, cliffs, or soil conditions.

Systematic surveys along alternative candidate outfall alignment corridors are used to confirm bathymetry and subbottom soil conditions, and to compare probable construction requirements for each alignment. A "corridor" is a swath 100 to 200 m-wide within which an alignment can be specified. An ideal survey mode is to proceed along the alignment at a slow, steady speed while simultaneously and continuously recording bathymetry, seismic subbottom profiles, side-scan sonar, and vessel position. Tide elevation must also be noted. Bathymetric surveys should be made shortly before the winter storm season and immediately after a major storm period to determine the annual range of fill and scour. Seismic subbottom profiling should give a good record resolution at subbottom depths of up to 10 m. Deeper penetration is not necessary and loses resolution of shallower detail. Side-scan sonar provides a continuous acoustical image on a wide strip chart of seabed surface features such as rocks, cables, sand ripples, pipelines, or shipwrecks within several tens of meters to either side of the vessel track.

The systematic surveys should include bottom sediment sampling with a gravity corer for silts, clays, or soft rock. Sand and rock samples can be taken with a bottom grab sampler, or they can be collected by hand by a diver.

Under conditions of sufficient light and water clarity, a diver with an underwater camera with color film can provide valuable records and photographs of typical rock outcrops, sand waves and ripples, or reef structure. A series of photographs taken in a slow panoramic sweep often form a montage of considerably more interest than an equal number of stand-alone photos.

Detailed studies on one or more alignment corridors most likely to be used serve to indicate the best choice of alignment to within 2 m along any section within the corridor. Uniform seabed conditions are preferred. Rock bottoms, cliffs, and ravines present difficult construction problems. Cables, other pipelines, and shipwrecks should be avoided entirely.

Detailed studies, using fathometer, subbottom profiles, and side-scan sonar are made on a tighter survey grid with lateral spacing of 25 to 50 m and numerous cross-tie runs perpendicular to the corridor, at intervals of 50 to 100 m. When later mapping the data, where cross-tie paths cross the paths along the corridor, check to see if bathymetric and other data from two runs at the same point give consistent results; gross lack of consistency indicates positioning error.

It is essential that detailed studies also include borings or corings at frequent intervals along one or more alternative alignments within a corridor. The borings provide the ground truth necessary for confident, unambiguous interpretation of the stratigraphic profiles recorded by the acoustic subbottom profiler, as well as the soil samples for grain size analysis and dynamic soils testing for structural design. Intervals of borings vary from 10 m to 1000 m, depending upon bottom variability.

Construction control consists of high-resolution bathymetric mapping capable of monitoring, say, the completeness of trench excavation and backfill.

4.4.3 Numerical Modeling

With the development of high-speed computers, numerical simulation of current patterns, and other physical processes for outfall studies has become feasible. Programs have been developed for thermal discharges, sewage discharges, and current patterns at offshore oil fields. The number and variety of available modeling programs are growing rapidly, and the designer is faced with the increasingly difficult task of selecting the program most appropriate for simulation of the important advective, dispersive, and perhaps biochemical processes in his study area.

At the core of any mathematical analysis of ocean outfall discharges is a series of equations that describe the pertinent physical, chemical, and biological processes of a study area. For example, physical processes that define circulation or current patterns are described by differential equations for conservation of water mass and momentum in three spatial dimensions:

$$\nabla \cdot \bar{u} = o; \text{ and } \frac{\partial \bar{u}}{\partial t} + \bar{u} \cdot \nabla \bar{u} + \frac{1}{\rho} \nabla \cdot \bar{P} + \bar{f} \cdot \bar{u} = \frac{1}{\rho} \nabla \cdot \bar{\tau} \tag{4.2}$$

where $\bar{u}$ = Mean velocity

t = Time

ρ = Mass density

$\bar{P}$ = Mean pressure

f = Coriolis acceleration, and

$\bar{\tau}$ = stress tensor

Chemical and biological processes are described by second-order partial differential equations that conserve constituent mass, called mass transport equations, of the form:

$$\frac{\partial C}{\partial t} + \bar{u} \cdot \nabla C = \nabla \cdot (\bar{E} \nabla C) + r \tag{4.3}$$

where C = Constituent concentration

$\bar{E}$ = Diffusion tensor, and

r = Source/sink terms which describe constituent addition, loss, and reaction kinetics

The techniques needed to use such systems of equations may deal with the three spatial dimensions explicitly, but very often the equation systems are simplified by integrating the equations over one or two spatial dimensions. For example, transverse and vertical movement and diffusion may be considered negligible compared to longitudinal movements and gradients, and thus may be said to integrate the governing equations over the transverse and vertical dimensions to obtain a one-dimensional mathematical description. In a shallow estuary, one might choose to neglect vertical variations and develop a vertically integrated two-dimensional model that describes movements in the two horizontal dimensions.

Solution techniques are analytical, empirical, or numerical. Analytical techniques use closed-form analytical solutions to the equations, such as tidal prism analysis and steady-state mass transport in a uniform flow field. Empirical techniques provide regression analyses of field or laboratory data which reveal functional relationships between a dependent variable, such as initial dilution, and one or more independent variables, such as outfall discharge or local depth. The equations of Section 4.4.5, 4.6.2, and 4.6.3 represent analytical techniques using empirical coefficients.

Numerical approximation techniques render the governing differential equations in an approximate finite-difference form, and solve repeatedly through time and space taking into account all the variability of, say, depth or shoreline geometry. Three types of numerical approximation techniques often used in circulation studies are called link-node, finite-difference, and finite-element techniques. In a link-node model, the study area is divided into a series of storage areas called nodes, at which water volumes and constituent concentrations are determined. These storage areas are interconnected by flow channels called links, along which constituent mass and ambient flow are transported. In Figure 4-6a, a one-dimensional analysis of a narrow channel is represented by a single strand of links and nodes, while a wide bay is represented by a two-dimensional link-node matrix.

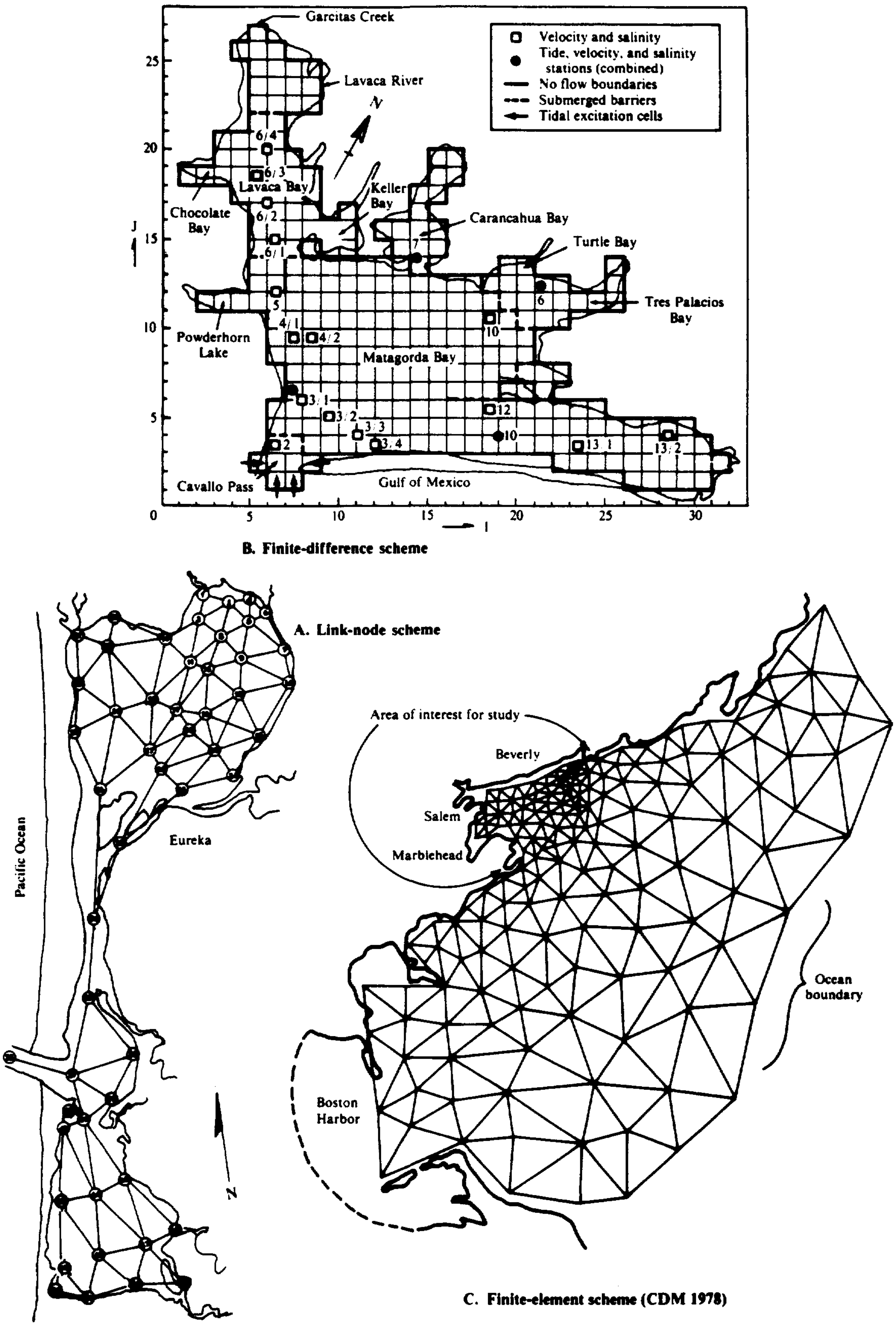
The finite-difference technique is the oldest and most widely used scheme. The water body under study is divided into a matrix of quadrilateral cells (usually square or rectangular), as shown in Figure 4-6b. Water surface elevation is commonly defined at the center of each cell, and normal velocity, or inflow per unit width, is defined for the side wall of each cell. An advantage of this technique is that it is conceptually relatively simple and easy to program. A disadvantage is that a grid of rectangles may permit only a crude representation of a complex shoreline with many promontories or dendritic arms.

The finite-element technique, already common in structural analysis, is relatively new to circulation analysis. The water body under study is divided into a series of elements, which may be triangles, rectangles, or even higher-order polygons (Figure 4-6c). Within each element, solution variables are defined as nodes, usually at element vertices or at midside. The finite element technique employs a method of weighted residuals defining the distributions of the independent variable within each element as a particularly efficient computational procedure for solving the governing equations. The triangles can be designed to simulate a tortuous coastline much more gracefully and faithfully than can the rectangles of the finite-difference technique.

Many numerical models are available. To select the most appropriate numerical approach for a particular study, one must consider: water quality standards, scales of the analyses, characterization of the physical system, data availability, dominant physical processes, and model availability. Where there are regulatory standards for water quality, the degree of required compliance within narrowly defined geographical areas determines the level of analytical effort and the choice of model. Most standards are written in terms of near-field and far-field regulations, and these define the scales of analyses to be performed and physical process considered. Another scale is intermediate. These scales are defined below.

The near-field region is immediately adjacent to the outfall in which the hydraulics of the outfall jet dominate ambient circulation. It is sometimes referred to as the zone of initial dilution or the region of stirring (see Sec. 2.2.5). The region is characterized by jet momentum effects, buoyancy due to thermal or salinity differences, and entrainment.

Fig. 4-6. Numerical Models for Estuaries



The intermediate-field region lies immediately beyond the near-field region. Ambient processes take over the provision of spatial mixing over one or more directions. For example, in shallow estuaries, ambient processes may mix the effluent vertically; and in a river, ambient processes will mix the effluent plume thoroughly across the river cross section.

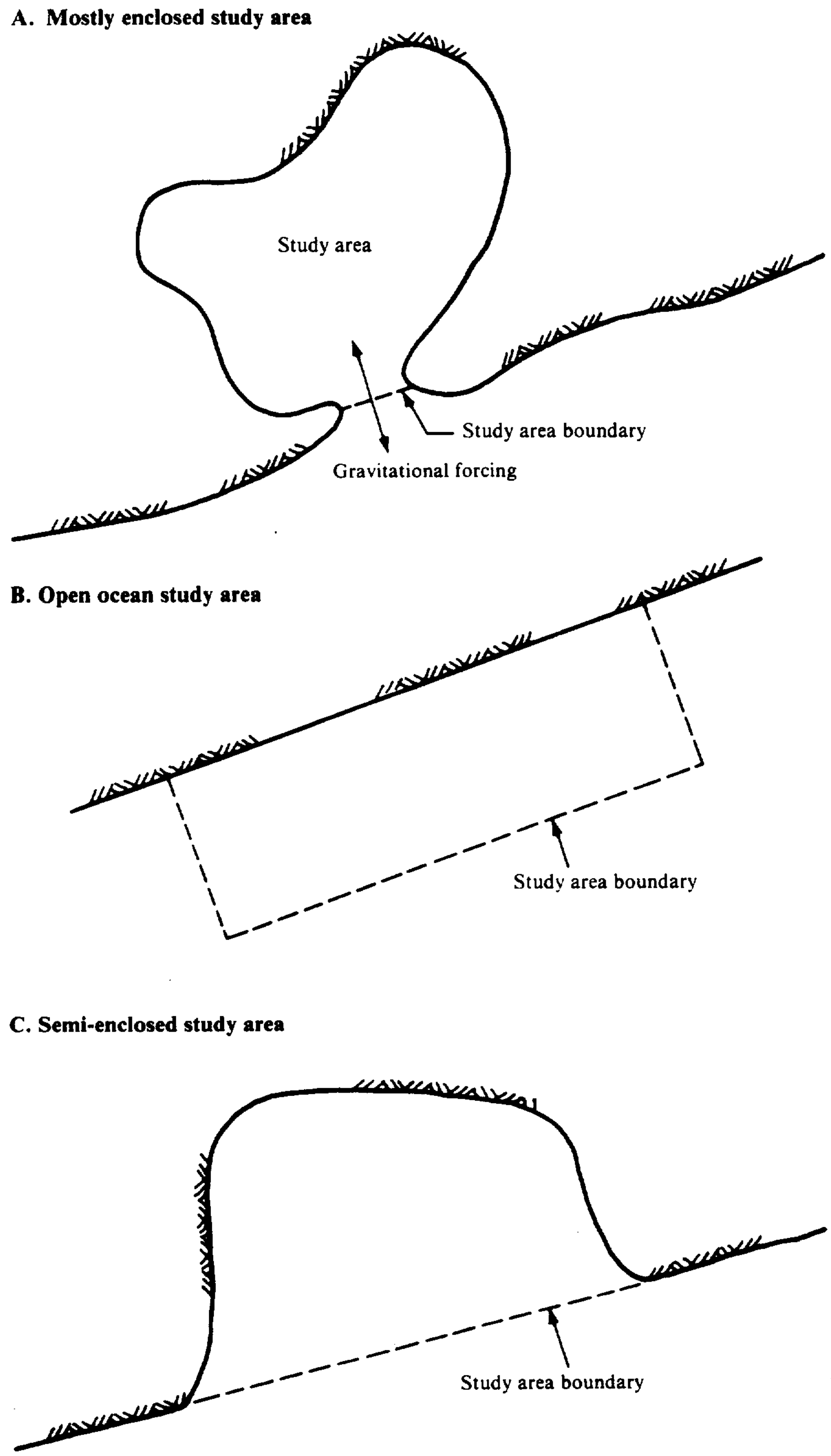
Large-scale processes of the ambient waterbody dominate the far-field region. This region includes longitudinal distributions in rivers and horizontal movement in shallow estuaries.

Model characteristics often indicate what type of model should be used, even before other aspects are considered. These include geometric length scales, complexity of land boundaries, and extent of the open water boundary. The ratio of length scales in the physical system indicate the dimensionality of model required. In a river, for example, after the initial dilution phase, the plume will generally become well mixed both laterally and vertically within a short distance downstream. Dominant variation occurs only in the longitudinal direction, and a one-dimensional model best simulates far-field concentrations. In a shallow, wide estuary, such as Tampa Bay, Florida, a plume is quickly vertically well mixed, and in the far-field, concentrations are determined by horizontal processes. Here, a two-dimensional (longitudinal and transverse) model should be used. A fjord, by contrast, may first become laterally well mixed, and be best modelled using a two-dimensional (longitudinal and vertical) model. Finally, in a deep, open water body such as Monterey Bay, California, far-field processes occur in all spatial dimensions, and thus requires a three-dimensional model.

For very complex land boundaries such as Chesapeake Bay, it is often desirable to use a finite-element representation rather than finite-difference (see Figure 4-6c). If the shoreline is more regular, a finite-difference grid may be adequate with the added advantage that this model is simpler to develop and use.

In modeling hydrodynamics and transport processes, special care must be taken in considering boundary conditions in larger open-water regions. It is relatively straightforward to model these processes in a water body that is almost entirely enclosed (Figure 4-7a), such as Chesapeake Bay where boundary conditions are established at the entrance. Usually a stage (water surface elevation) boundary condition is imposed with implicit assumption that momentum fluxes are reflected at the entrance. Internal tidal hydrodynamics are driven by gravitational forcing. When an open coastal region (Figure 4-7b), such as Key Largo in the Florida reefs, is being modeled, specifying open ocean conditions is much more complicated and resulting models much more costly. Aside from gravitational forcing, momentum effects from eddies and density-driven currents can enter the region and dominate the local circulation patterns calculated from long-term current meter records. For such areas, programs can be used that interpolate current meter data onto a numerical grid while maintaining mass continuity. For semi-enclosed study areas (Figure 4-7c), appropriate model selection and verification is much more difficult, and a thorough investigation and understanding are required before choosing and applying a model whose value for design and operation may by then have been reduced or preempted.

Fig. 4-7. Types of Ocean Boundaries for a Numerical Model



Another limiting factor in the selection of a numerical model is the availability of prototype data. For most sites, there is usually an adequate description of meteorologic, streamflow, and outfall discharge data. Current and water quality data are usually sparse, however. In most cases, there is a nearby tide gauge, a few salinity/temperature/depth measurements and some isolated velocity and water quality measurements. These data should be sufficient to determine the dominant physical processes and to be used for calibration/validation studies of the numerical models selected. A preliminary survey is needed to determine the important physical processes, and a second larger study to obtain a set of synoptic data for model calibration and validation. Note, however, that there are cases where the extensive empirical and statistical data needed for model verification are in themselves sufficient for outfall design without further recourse to the numerical model.

In any investigation of initial and ultimate consituent data, it is necessary to understand the dominant physical processes with respect to their effects on near-field and far-field circulation. This is usually done by either reviewing available data, from experience with similar systems, or occasionally by undertaking a sensitivity analysis of a numerical model--the latter requires at least some data to establish ranges of conditions. In the near-coastal ocean, the circulation is a combination of forcing due to waves, tides, wind, atmospheric pressure gradients, fresh water inflows, density induced currents, and Coriolis acceleration. For a given site, certain processes dominate, whereas others may be neglected. In a fjord, for example, Coriolis effects can be neglected and emphasis placed on the effects of tide and density forcing. In a wide, shallow estuary, Coriolis and wind effects are important.

Once the means to evaluate and simulate hydrodynamic processes has been established, it must be determined what constituents and reaction kinetics should be included in the transport or water quality models. The most common ocean outfalls discharge heated water or freshwater sewage. In at least the initial dilution region, jet momentum and buoyancy effects dominate, and the concentration of the constituent discharged is large relative to both ambient conditions and processes. For example, wastewater in an estuary may be modeled as a single constituent such as BOD with a first-order decay specified to approximate reaction kinetics. In the far-field such a simplified approach may not be justified as lower concentrations define more subtle water quality reactions that must be accurately modeled to evaluate compliance with standards. Instead, a conceptual model of pertinent water quality processes must be developed and quantified reaction kinetic terms through field data or estimate must be developed from the literature.

Having accomplished a preliminary investigation of the study area, physical processes, and conceptual constituent fate processes, the design engineer can select a numerical model to address both the geometric features and the dominant processes of the area. Table 4-2 summarizes the types of models and processes that may be simulated and indicates the general availability of models in the cross classification.

TABLE 4-2

Availability of Numerical Models

Model Type	Near-Field (initial dilution)	Far-Field	Thermal	Single Constituent	Single Water Quality	Complex Water Hydrodynamic	Observed Hydrodynamic
One-dimensional	-	many	several	many	many	many	few
One-dimensional network	-	several	few	several	several	several	few
Two dimensions (length and width)	-	many	several	many	several	many	few
Two dimensions (length and depth)	-	several	few	several	few	several	few
Three dimensions	many	several	few	few	very few	several	few

More specific advice on model availability may be obtained from the Technical Committee on Computational Hydraulics, American Society of Civil Engineers, 345 East 47th Street, New York, NY 10017-2398, USA.

In general, only large ocean outfall projects may justify the use of highly technical numerical model approaches. It is essential that the responsible designer understand the procedures for model selection and application. Only an experienced engineer should be asked to apply a highly complex computer/numerical model.

Running numerical models involves several steps, some of which are optional, depending on data availability. These include model mobilization on the computer system, initial calibration, sensitivity analysis, design of additional field programs, final calibration, validation, and production runs.

Once the model(s) has been adapted to a given computer system, the first step is to use available data to perform an initial calibration by adjusting model parameters, including friction factors for hydrodynamic models, and dispersion and reaction rate coefficients for water quality models. The model may then be used to perform a sensitivity analysis to determine the relative importance of various terms and processes within its description.

Often overlooked is the numerical model's usefulness in designing field programs. In a region with less than adequate descriptive data, a model may predict advection patterns, such as pumping and gyres that were previously unrecognized. A field program could be designed to both investigate such effects and obtain data on general processes for final model calibration and validation. Validation here is defined as running the model on a previously unused set of data to determine the adequacy of model coefficients established during calibration. Model credibility requires that calibration coefficients remain unchanged during this phase.

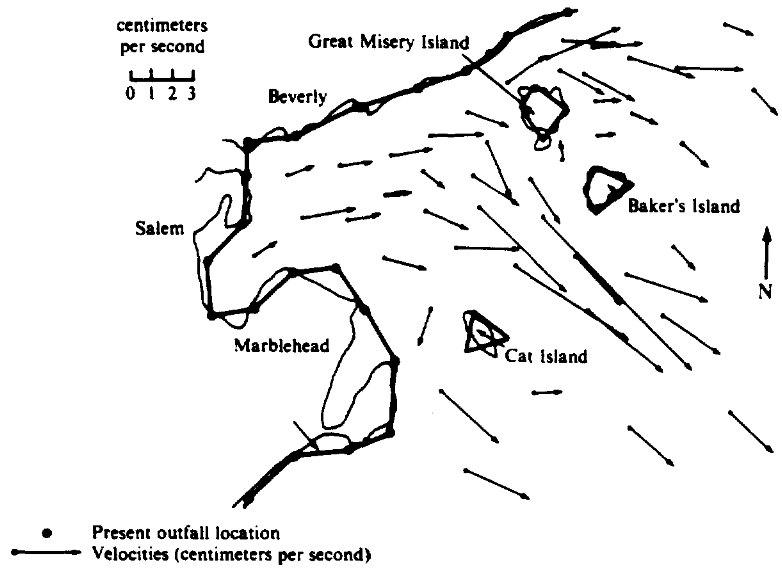
The model is now ready to be run in production mode to analyze the effects of conceptual outfall designs. Production runs indicate where an outfall may be located to minimize the impacts on receiving water uses. This is achieved by modeling several outfall locations and developing contour plots of areas of poor water quality, or perhaps the time history of coliform concentrations in a particular beach area. By simulating discharge from each outfall site under a variety of hydrodynamic conditions, a frequency distribution curve of anticipated duration of outfall effects may be developed so that it can be compared with regulatory standards or design criteria. Aside from the vast amounts of numbers generated, the most useful, and readily understandable, form of output is the graphic display. Current patterns, water quality distributions (Figure 4-8), and other data presented this way are used by officials authorizing and financing ocean outfalls.

4.4.4 Physical Hydraulic Models

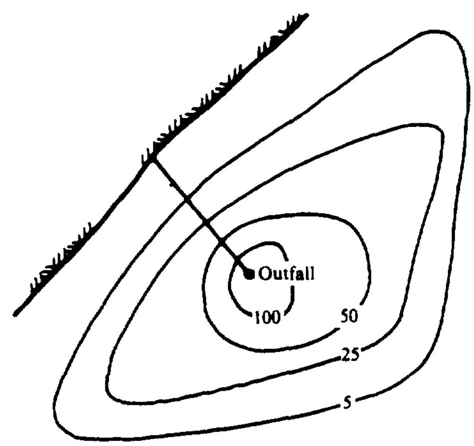
The cost of a physical hydraulic model typically runs from US$30,000 to $50,000 for a pump intake study, to millions of dollars for a model of a major estuary. For major urban outfall systems, physical models are sometimes justified either as a guide in unusual small-scale internal hydraulic design

Fig. 4-8. Types of Computer Output

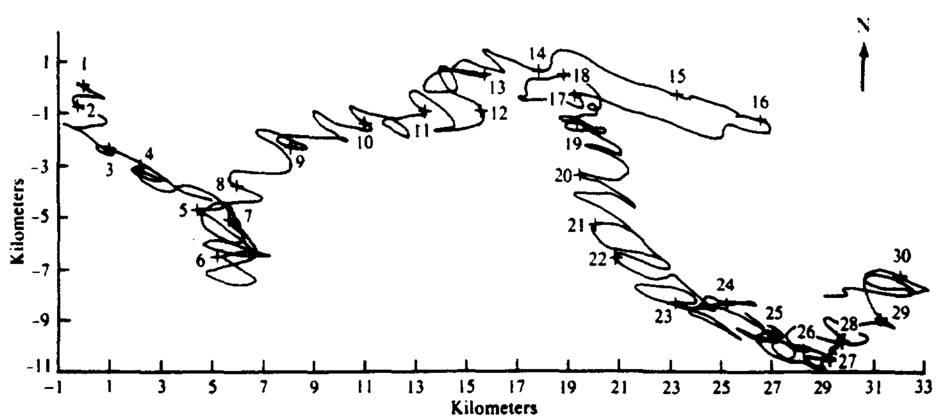
A. Velocity field plot (CDM 1978)



B. Concentration contours



C. Progressive vector diagram (CDM 1978)



problems such as a unique outlet design, or as an aid in predicting dispersion of effluent through a complex system such as Puget Sound, San Francisco Bay, the Scheldt, or the Singapore Straits.

For smaller outfalls, the design and construction of a new physical model will seldom be economically feasible. However, if the proposed outfall site is within the study area of an operating model, a small program of studies with that model can probably be justified. If the model has been decommissioned and more runs are not easily made, the published or unpublished results from earlier tests may be useful.

Geographical physical models are most useful as indicators of mixing from point sources, of flow phenomena such as eddies shed by a promontory at a particular tide stage, or of the pattern of net tidal circulation around a large island in a braided estuary. As with numerical models, phenomena observed in the hydraulic model must be validated by field tests.

4.4.5 Equations for Estimating Turbulent Dispersion

The method of calculating the rate at which a turbulent plume spreads in width and increases in centerline dilution with increasing time and distance from the point of discharge is treated briefly. Several practical equations are presented, together with an indication of the appropriate application and the shortcomings of each. For a definitive discussion, see Fischer et al. (6).

The rate of lateral dispersion is the principal factor in turbulent dispersion of an effluent plume as it moves away from outfall. Vertical dispersion of a plume (mixture of a plume thoroughly between the sea surface and the sea bottom) is of limited interest, as it can only increase the dilution ratio by a factor of about 3, because the initial plume ordinarily occupies some 30 percent or more of the water depth. Longitudinal dispersion of an isolated "puff" of effluent will dilute the puff, but longitudinal dispersion in a plume, a continuous chain of puffs, succeeds only in mixing puffs with each other, and not with new dilution water. Therefore transverse dispersion, or the spreading of effluent out to either side of the mean plume path, is of prime interest.

The theoretical analysis is based on the assumption, fairly well-supported by experiment, that a cloud of particles, or parcels, of effluent in a turbulent flow will tend to have a transverse (y-direction, in Figure 4-9a) spatial distribution that is normal, or Gaussian, centered on the mean plume path:

$$p(x,y) = \frac{Q_e}{d\bar{u}} \frac{1}{\sqrt{(2\pi)}s} e^{-\frac{y^2}{2s^2}} \qquad (4.4)$$

in which: p = The relative volumetric effluent concentration at any longitudinal distance, x;

Q_e = Volumetric effluent discharge rate;

d = Water depth;

$\bar{u}$ = Mean current speed;

$Q/(du)$ = Volume of effluent per unit length found along the plume x-axis;

s = Standard deviation of the plume cross-sectional distribution.

If C_o mg/L is the mass concentration of a contaminant in the undiluted effluent, then that contaminant's mass concentration at point (x, y) in the plume is p/C_o mg/L if the contaminant is conservative, or $p\ C_o\ e^{-kt}$ if the contaminant decays at rate k with respect to time, t. The remainder of this subsection will be in terms of the relative volumetric concentration, p, which will simply be called the concentration.

The centerline concentration is:

$$p(x,0) = \frac{Q_e}{du} \frac{1}{\sqrt{(2\pi)}s} \tag{4.5}$$

The centerline concentration decreases with increasing "s". The rate at which s increases is of major interest in plume studies. It is customary to work in terms of the variance, s^2; its time rate of increase is defined as

$$\frac{ds^2}{dt} = 2\varepsilon \tag{4.6}$$

in which ε is the dispersion coefficient.

Where the plume size, as characterized by s, is greater than the size of the largest turbulent eddies promoting the dispersion, ε is observed to be a function of water depth, d, and shear volocity, u*:

$$\varepsilon = K\ d\ u^* \tag{4.7}$$

in which u* is related to the shear stress exerted on the stream bed by the flow; this may be calculated in terms of the friction slope of the flow, or in terms of the mean flow speed, $\bar{u}$, and the Manning friction factor, n:

(4.8) $u* = \sqrt{\tau_o / \rho}$

(4.9) $u* = \sqrt{gd \times slope}$

(4.10) $u* = \bar{u}\, n\, g^{1/2}\, d^{-1/6}$

Suggested values for the constant, K, are given in Table 4-3. Values between 0.1 and 0.2 have been obtained in straight, uniform laboratory channels; but higher values are obtained with increased channel tortuosity, or "curviness." In natural streams, K is rarely found to be less than 0.4. Values measured in the Missouri River range between 0.6 and 10.0. In straight, unstratified reaches of tidal estuaries, values of about 1.2 are appropriate (6).

TABLE 4-3

Transverse Mixing in Open Channels (6)

Channel	$K = \varepsilon/(du*)$
Straight uniform laboratory channels	0.1 to 0.2
Straight natural streams or meandering natural streams	0.4 to 0.8
Straight, unstratified reaches of tidal estuaries	1.0 to 1.4

Where the cloud size, L_s, is smaller than the size of the largest turbulent eddies, as is the case for many outfalls to the open ocean, ε is observed to increase with cloud size, s, according to what is called the "4/3 law":

(4.11) $\varepsilon \sim L_5^{4/3}$

4.4.5.1 Applications

(1) The simplest application is for a point discharge to a wide river before the plume reaches the river banks:

(4.12) $$p\ (x,y,t) = \frac{Q}{d\sqrt{4\pi\bar{u}\varepsilon x}}\ e^{-(y^2\bar{u})/(4\varepsilon x)}$$

in which C/C_o is the concentration of an effluent constituent, Q is the mass discharge rate of an effluent constituent, d is the water depth, and $\bar{u}$ is the mean advective current speed, as illustrated in Figure 4-9. This simplest case is used for analyzing point discharges, and is a building block for more complex cases, using the principle of superposition and the method of images.

(2) If the stream is of width, w, Equation 4.12 is useful until one or both edges of the plume touch the banks; downstream of that point, the density distribution within the steam behaves as it would in an infinitely wide stream with "image" plume sources abreast of the true source. If the plume is at midstream, the method of superposition gives the downstream concentration distribution as:

(4.13) $$p = \frac{Q}{d\sqrt{4\pi\bar{u}\varepsilon x}} \sum_{n=-\infty}^{\infty} e^{-(y-nw)^2\, u/(4\varepsilon x)}$$

The situation is illustrated in Figure 4-10. The plume trailing from the coordinate origin, confined between the sides w apart, is represented as a composite of plumes at y = 0, y = +w, y = +2w,...,y = +nw for n up to ∞. Two results of the summation are plotted in Figure 4-10a. The centerline concentration (y = 0), initially very high near the source (small x) decreases and gradually approaches $C = Q/(\bar{u}dw)$, the value for a complete mixture of effluent across the stream. At the side (y = w/2), the concentration is initially zero, but rises to approach $C_o = Q/(\bar{u}dw)$, again the value for complete mixture. The plot shows that the centerline and side concentrations differ by less than 10 percent when

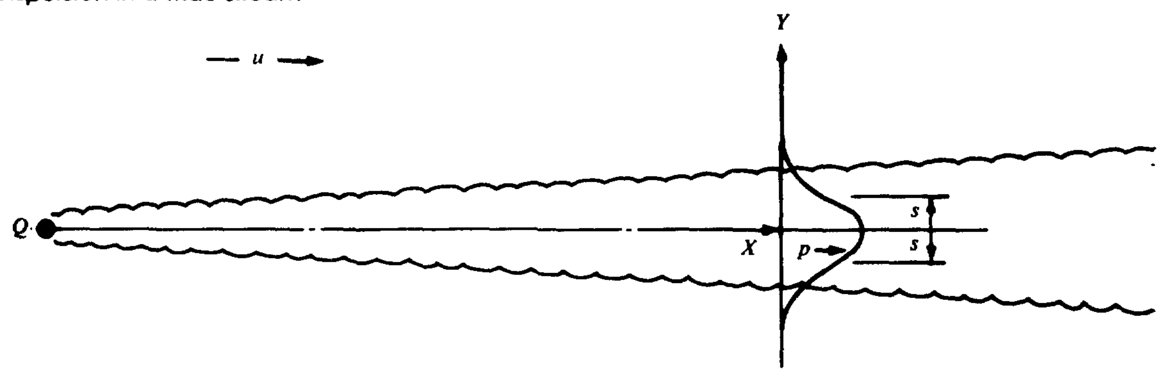
(4.14) $$x = 0.1\ \bar{u}\ w^2/\varepsilon = L$$

Equation 4.14 may be used to calculate the downstream distance x = L, which is required for complete mixture of centerline discharge across a stream.

(3) Essentially the same mathematics may be used to analyze the spread of a plume from a shoreline discharge across a stream. In Figure 4-9b imagine the "centerline" to be a channel side, the other channel side being the side shown at y = w/2. We can thus study the spread of a bankside discharge across a stream of breadth B = w/2. The plot of Figure 4-10a is repeated in 4-10b, but is relabeled for the case of bankside rather than centerline discharge. The downstream distance required for complete across-stream mixture is:

Fig. 4-9. Plan Views of Point Source Plumes in Streams

A. Dispersion in a wide stream



B. Wall effects in a narrow stream are modeled by image sources and plumes

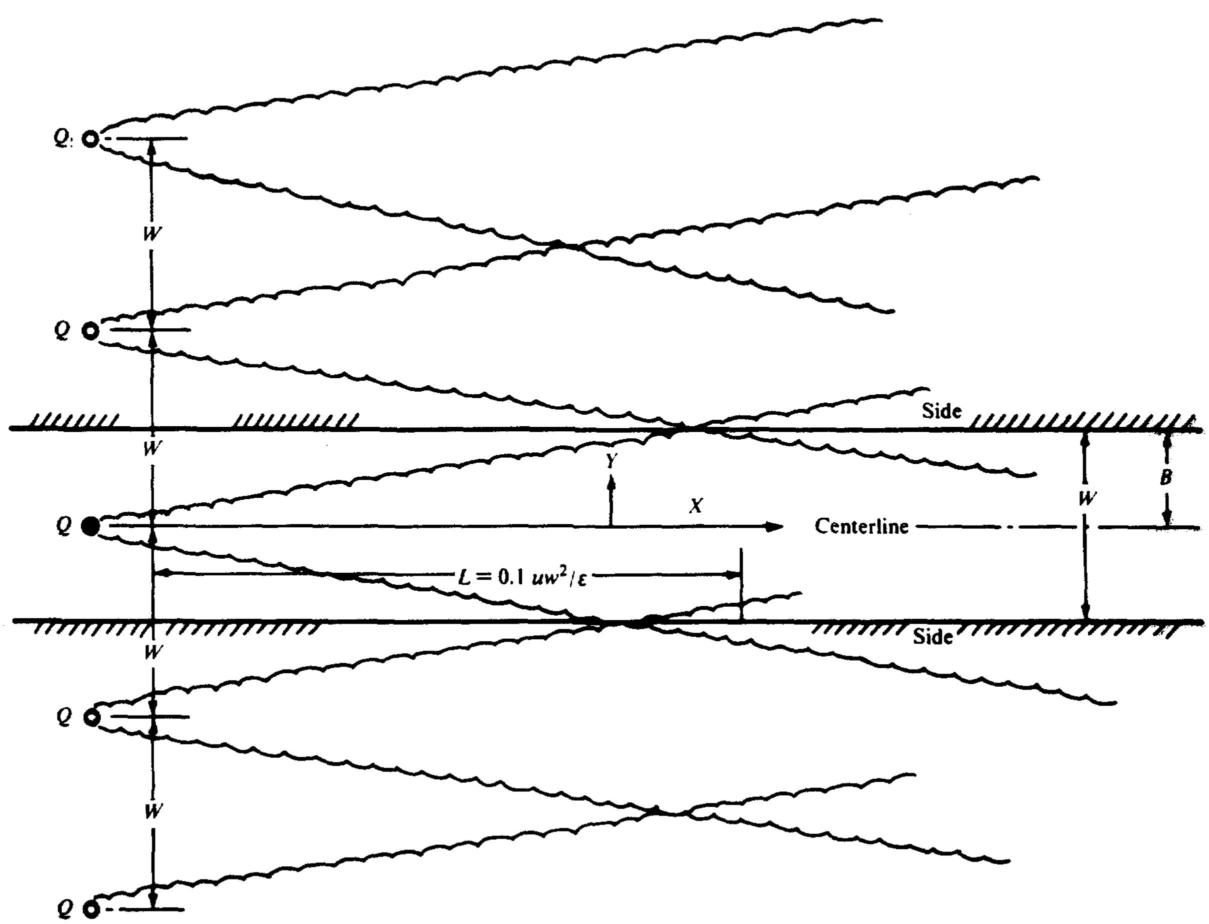
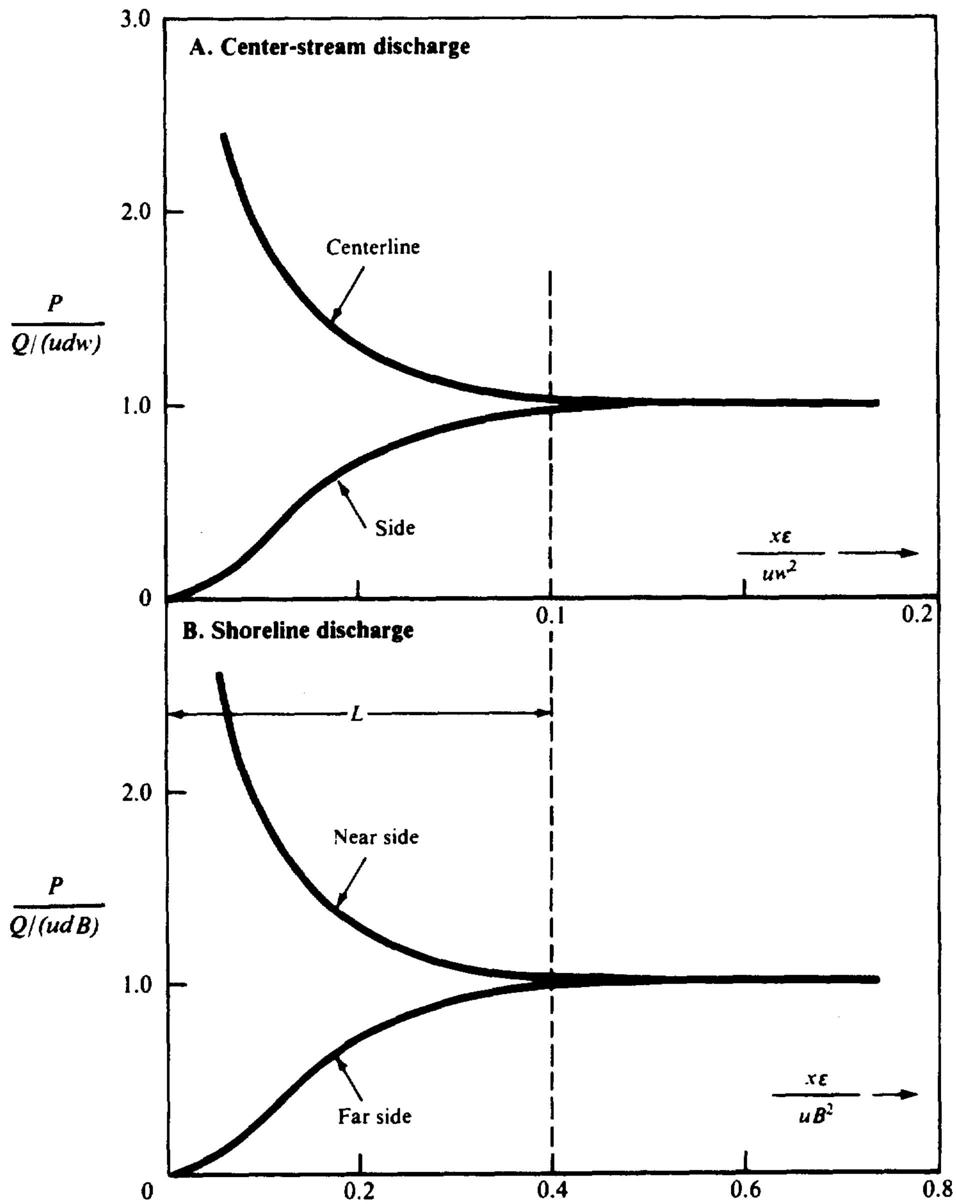


Fig. 4.10. Effluent Concentration as a Function of Downstream Distance for a Point Source Discharged at Midstream (A) and a Point Source Discharged at One Shore (B).
<u>Source</u>: Fischer et al. (6).



(4.15) $x = 0.4\ u\ B^2/\varepsilon = L$

(4) Discharge from a diffuser of finite length, b, in a river is computed in theory by superimposing the plumes, each described by Equation 4.12, from many small sources distributed along the diffuser. The mathematical expression for plume centerline concentration for such a superposition is:

(4.16) $$p = \frac{Q}{ubd}\ \mathrm{erf}\ \left(\frac{u\ b^2}{16\ \varepsilon x}\right)^{1/2}$$

in which erf is the standard error function

(4.17) $$\mathrm{erf}(M) = \frac{2}{\sqrt{\pi}} \int_0^M e^{-v^2}\ dv$$

Equation 4.16 is plotted in Figure 4-11 in normalized form.

Note that for large x (hence small M), far downstream from the diffuser, erf(M) approaches $2M \sqrt{\pi}$, so that the centerline concentration approaches:

(4.18) $$p \rightarrow \frac{Q}{ubd}\ 2\ \sqrt{\frac{u\ b^2}{16\ \pi\ \varepsilon x}}$$

or (4.19) $$p \rightarrow \frac{Q}{d \sqrt{4\pi\ \varepsilon\ ux}}$$

Note that this is identical to Equation 4.12 for plume centerline concentration (y = 0). Figure 4-11 shows that Equation 4-16 can be well approximated by Equation 4.19 for y = 0 whenever x exceeds about $0.5\ ub^2/\varepsilon$.

(5) Turbulent dispersion from a diffuser of length b in the open ocean, where the "4/3-law" applies, can be analyzed using the classic formula for the centerline concentrations developed by Brooks (1):

(4.20) $$p = \frac{Q}{ubd}\ \mathrm{erf} \left| \frac{1.5}{\left(1 + \frac{8\ Ex}{u\ b^2}\right)^3 - 1} \right|^{1/2}$$

This expression is plotted in normalized form in Figure 4.11. The diffusivity, ε, increases with the width of the plume, according to Equation 4.11. The initial value, E, is related to diffuser length:

(4.21) $E, m^2/sec = 5 \times 10^{-4} (b,m)^{4/3}$

As with dispersion in a river from a diffuser, the length of the diffuser becomes less and less important to the dilution value the farther downsteam one proceeds. For large x (small M), Equation 4.20 can be approximated by:

(4.22) $$p \rightarrow \frac{Q}{ubd} \frac{2}{\sqrt{\pi}} \quad \frac{1.5}{(1 + \frac{8\,Ex}{ub^2})^3 - 1}$$

Equation 4.22 is plotted with dashed lines in Figure 4-11; it is a good approximation to Equation 4.20 for x larger than about 0.25 ub^2/E. Substituting Equation 4.21 into Equation 4.22 (using meter-kilograms-second units), the limit for large x is:

(4.23) $$p \rightarrow 5463 \frac{Q}{d} u^{1/2} x^{-3/2}$$

in which the diffuser length does not appear at all. Thus diffuser length, which is a very important factor in initial dilution computations, is essentially unimportant in estimating dispersion further down-plume from the diffuser. An exception is the case in which there is potential for plume submergence (see Sec. 4.6.4 and 4.6.5) with different advection-dispersion patterns beneath the surface and at the surface.

4.4.6 Comparisons of Results

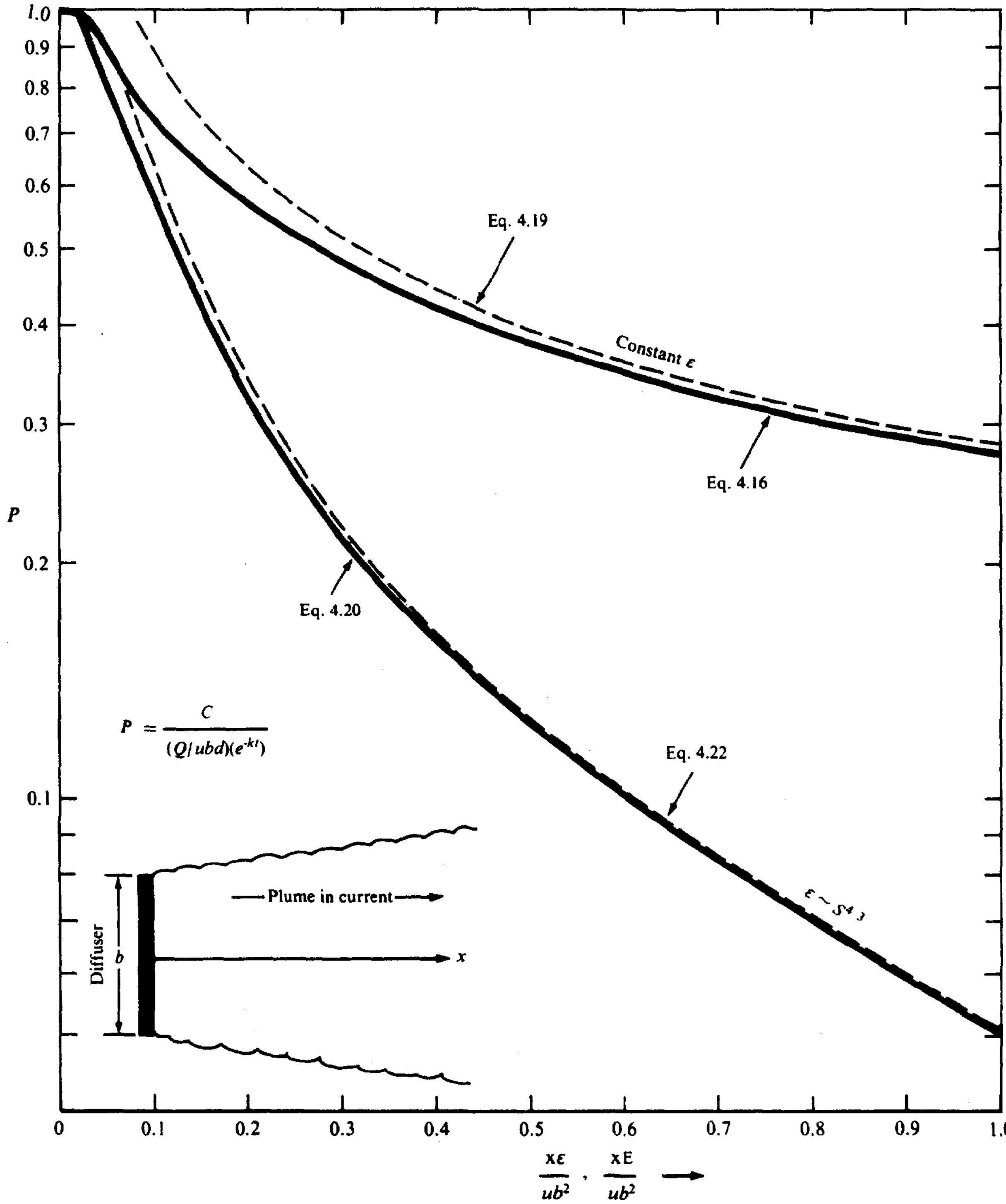
Current patterns may be predicted by field studies using drogues or moored meters, by numerical models, or by physical models.

Generally, there is little agreement between, say, drogue and meter values, for instantaneous current velocities in one locality, but considerable agreement between velocity integrals, such as flux over a tidal cycle, or trends, such as seasonal speed direction histograms.

Example 1. A drogue deployed near and at the depth of a recording current meter may show little correlation with the velocity measured simultaneously by the meter. However, after a year of nearly continuous current meter recordings and dozens of drogue deployments, the sum of the drogue measurements will show a range of speeds and current directions that is in close agreement with a similar summary of meter data.

Example 2. Tidal current charts for a major seaport (Figure 4-12) suggest that for three hours of a flood tide, effluent from a particular sewage outfall is entrained into a branch estuary (A); for the next three hours the effluent is carried up the stem of the main estuary (B); and during the six hours of ebb, the effluent is carried out of the harbor (C). A link-node

Fig. 4-11. Relative Concentration at the Centerline (p) Downstream of a Line Source of Length b



numerical model of the same estuary system predicts that for each of the six hours of flood, roughly half the effluent flows to A, and half to B; and that during the ebb, all effluent goes toward C. Again, the charts and the model disagree on flood current patterns on an hour-by-hour basis, but both agree that over one tide cycle, 25 percent of the effluent is drawn into branch estuary A.

4.4.7 Level of Effort

Because outfall studies can be expensive, one should carefully select an appropriate scale of survey effort. Table 4-4 presents a proposed year-long field study program of open coastal waters off four major districts within one waste management jurisdiction. The districts are separated by several tens of kilometers, and have a total population over 2 million. Bids from marine survey contracting firms ranged from USD 0.8 to 1.4 million in 1982. The program included "big ticket" items such as geophysical studies and numerous recording current meters.

Adequate surveys can often be conducted at much less cost. For a population of, say, 150,000 on the same open coastline with a vigorous local current regime, the effluent load is less by an order of magnitude, yet the receiving water has no less assimilative capacity. The study area need not be as large since less flux water and less volume will be required to assimilate wastes. The outfall, or outfalls, will probably be shorter. Fewer subsurface borings will be needed; the outfall will be of smaller diameter, with the result that construction and wave protection problems will be reduced. A well-executed drogue study costing perhaps US$100,000 without moored current meters or numerical modeling will probably be adequate to describe current patterns. Per capita costs are from half a dollar to one dollar, in each example.

4.5 Outfall Siting

In this section, outfall siting will be discussed by including considerations of bathymetry, waves, and water quality standards, and several worked examples provided.

4.5.1 Information Needs

The central questions of ocean outfall siting are (1) Where and how may one discharge a given effluent so as to meet given water quality criteria throughout a given study area? (2) Of the sites that satisfy conditions of the first question, which are the most cost effective? and (3) What level of treatment should be provided to the effluent?

The information needed to answer these questions includes: (1) effluent quantity and characteristics, (2) topographic and bathymetric maps and charts of the proposed service and discharge areas and environs, (3) identification of sensitive areas, such as bathing beaches, shellfish areas, and, in estuaries, migratory pathways for fish, (4) local patterns of maritime traffic, and anchorages, (5) locally applicable water quality criteria or standards, (6) predominant current patterns in the proposed discharge areas (tidal, wind-driven, river outflows, eddies, stratification effects), (7)

Fig. 4-12. Hypothetical Estuary for Example 4.4.6-2

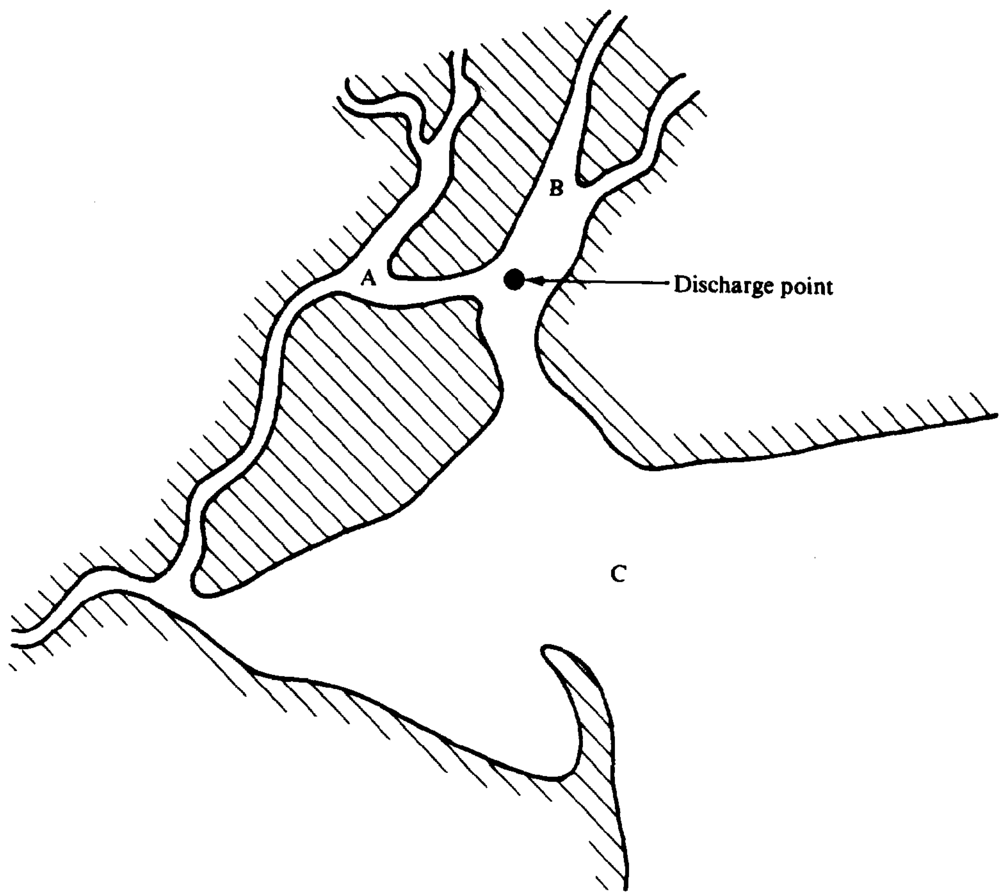


TABLE 4-4

Marine Survey Program

Activity	Location and Depth		Part A 1982 j	a	s	o	Part B 1983 n	d	j	f	m	a	m	j
I. PHYSICAL OCEANOGRAPHY	Key: The 4 survey districts are denoted M, J, N, and S													
A. Currents														
i) Drogues	In each area, select three deployment sites, two of which are on 60 m contour. At each site, deploy up to six drogues simulaneouly, each at a different depth.	M	•	•		•		•		•		•		•
		J	•	•	•	•	•	•	•	•	•	•	•	•
		N	•	•	•	•	•	•	•	•	•	•	•	•
		S	•		•		•		•		•		•	•
ii) Recording Current Meters	In each area, select two deployment sites, implant two meters at different depths at each site. Service fortnightly.	M	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
		J	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
		N	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
		S	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
iii) Direct Readout Current Meters	(Each day at sea that conditions and time permit anchoring, obtain current profile when at anchor.)		- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
iv) Wind Data	In each area, continuous recording at shoreline, instrument 10 m above sea level.		—	—	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
B. Temperature/Salinity														
i) Daily Opportunity	(Each day at sea, whatever the activity, take at least one T/S profile over the deepest site occupied.)		- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
ii) Array Surveys	In each area, select one array of up to ten sites. At each site, sample T and S at 5 m depth intervals.	M	•	•		•		•		•		•		•
		J	•	•	•	•	•	•	•	•	•	•	•	•
		N	•	•	•	•	•	•	•	•	•	•	•	•
		S	•		•	•	•		•		•		•	•

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TABLE 4-4 (cont'd)

Activity	Location and Depth	Part A 1982 j a s o	Part B 1983 n d j f m a m j
2. BACTERIAL DISAPPEARANCE			
i) Effluent Concentrations	In each area, take at least 50 samples from the principal existing discharge.		
ii) 24-Hour Bag Studies in Harbor	At one location, run 30 to 50 tests.		
iii) In-situ disappearance rate studies	(To be determined by survey team.)		
3. GEOPHYSICS			
i) Bathymenty			
a. Preliminary	On first visit to each area, run two or three bathymetric profiles to determine location of 60 m contour and other important features.		
b. Systematic	Operate simultaneously with side-scan sonar and sub-bottom profiler.		
ii) Side-Scan Sonar iii) Subbottom Profiler	In each area, run transects perpendicular and parallel to shoreline. Run along prospective outfall pipe alignment in conjunction with fathomether and horizontal control system. Schedule for autumn. M J N S		
iv) Diver Observations	In each area, at several sites, as schedule permits.		
4. WATER QUALITY			
i) Secchi Disc	Each day at sea, at each station.		
ii) Water Quality Sampling	In each area, one day per three months, from two depths at each site.		

seabed conditions along alternative pipeline alignments (sediments, rock, bearing capacity, relief), and (8) seasonal wave climate effects on design criteria and on construction (4, 8).

This information will facilitate the selection of the least-cost system that will protect water uses, with due regard for construction and maintenance costs and minimum risk of wave, shipping, or other damage.

A smooth, sandy seabed with good bearing capacity not subject to major erosion or accretion, is an ideal bottom condition. A rock bottom with reefs, cliffs, and canyons makes construction difficult and much more expensive. Very soft marine clays are also difficult. In general, greater depths better promote initial dilution. Thus if for advection/dispersion reasons an outfall should be 1,500 m long, choose a site in 30 m of water over one in 10 m of water, where both sites lie 1,500 m from land. On the other hand, deeper water increases construction costs. A short outfall with a deep discharge and plume submergence may result in occasional plume upwelling along the shoreline. In any event, it is necessary to compare marginal costs and benefits of alternative systems.

Waves may impose construction problems, induce dynamic forces on a completed pipeline, or cause seasonal scour or accretion of sand or silt on the seabed alongside the pipeline (see Chapters 6 and 8).

4.5.2 Water Quality

Water quality standards and criteria recognize a wide distribution of values, and recognize that a standard will be met when a particular fraction of the samples does not exceed a limiting value. For example, a coliform bacteria standard may state that no more than 10 percent of the samples collected in a prescribed schedule may contain more than 1,000 fecal coliforms per 100 ml. This is important, for with a variable input to a fluctuating current climate, there is no plausible outfall length beyond which a discharge will never violate standards, or within which a discharge will always violate standards. Rather, the frequency with which a limiting nearshore value is exceeded will decrease monotonically with increasing outfall length. For design purposes, the outfall length/violation frquency function, and the acceptable frequency of violation, each of which may vary seasonally, are determined.

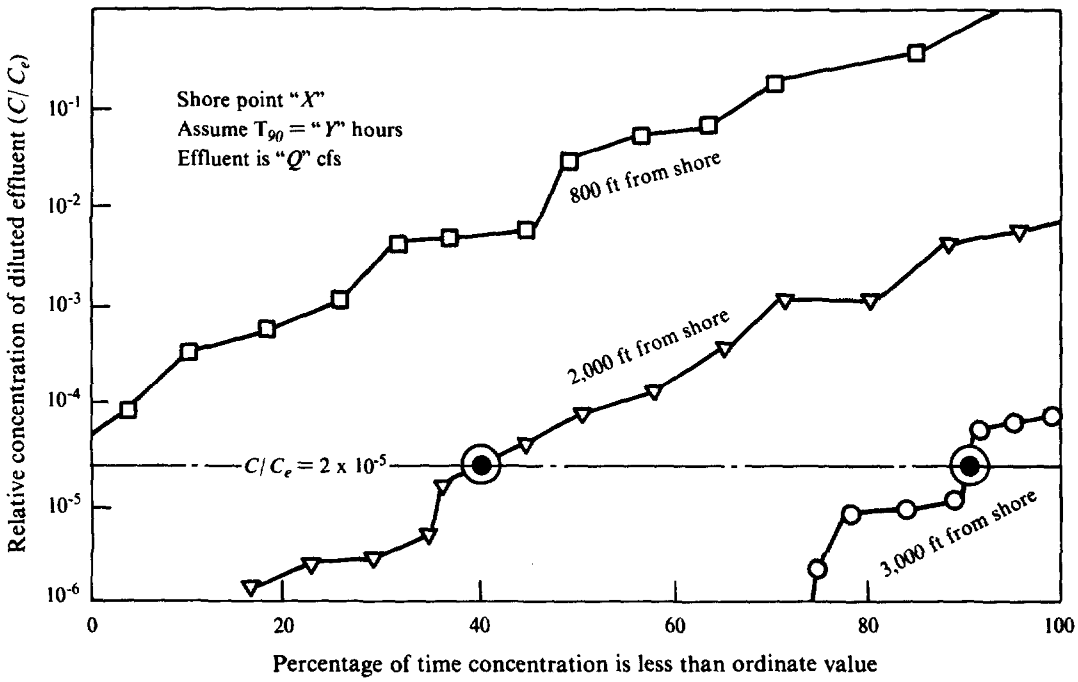
Figure 4-13a shows how field data and modeling results may be used to predict cumulative frequencies of relative concentrations of an effluent contaminant measured at a shore point "X", for several discharge distances from shore, and for a given decay rate of the contaminant.

For a given effluent concentration (such as 10 fecal coliform per 100 ml of effluent) and for a given limiting values (such as 200 per 100 ml), the curve in Figure 4-13b shows the frequency of violation (or satisfaction) of the standard as a function of discharge distance from shore.

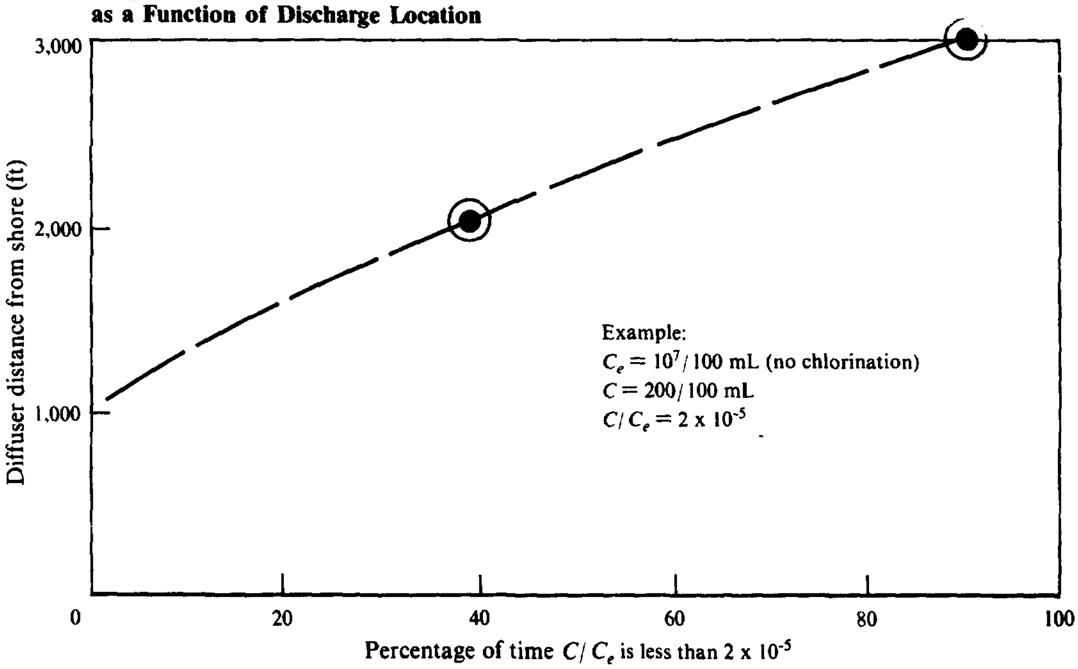
Note that some water quality factors are relatively insensitive to outfall length, and are most effectively controlled by wastewater treatment rather than by siting of the discharge point. Surface slicks of oil and

Fig. 4-13. Typical Relations between the Effluent Concentration Measured at the Shore and the Distance of Discharge from the Shore

A. Cumulative Frequency Curves for Concentration for Several Discharge Locations



B. Percentage of Time That Nearshore Concentration is Less Than a Standard Value, as a Function of Discharge Location



grease can blow onshore in an afternoon seabreeze even from a discharge far offshore, and are best controlled by removing from the waste stream. Coastal upwelling of nutrient-rich and oxygen-deficient seawater may impose an obligation to remove BOD from the waste stream before discharge, no matter what discharge site is chosen. The acceptable violation frequency establishes the required discharge distance from shore.

4.5.3 Examples of Outfall Siting

Example 1. Consider a primary effluent discharge of 100 L/sec, with 120 mg/L SS, 140 mg/L BOD, and 1 million fecal coliforms per 100 mL. The effluent could be chlorinated, with a residual chlorine concentration of 2 mg/L; chlorination would reduce the fecal coliform count to 1,000/100 mL. The open coast is essentially straight; 800 to 1,200 m offshore lies a narrow coral reef running parallel to the coastline. The current speed averages 0.1 m/sec between the shore and the reef, and averages 0.5 m/sec beyond the reef. In-situ bacterial disappearance tests carried out for study of another site 100 km away with essentially identical effluent and receiving water characteristics show a fecal coliform T_{90} of 1 hour.

The water quality criteria to be met at least 95 percent of the time are (1) settleable solids to accumulate not more than 0.5 $kg/m^2/year$, (2) dissolved oxygen to be depressed no more than 1 mg/L below ambient concentration, (3) fecal coliform concentration to be not more than 200 per 100 mL at shore, or over the coral reef, and (4) chlorine residual concentration to be not more than 0.002 mg/L over the reef.

With a straight coastline and reef, the position of the outfall on the shore will probably be near the treatment plant following a favorable routing over the seabed. The siting question is thereby reduced to, "How far offshore?" First, check the overall dilution flux inside the reef. The average current velocity is 0.1 m/sec; the cross-sectional area is about two-thirds of the width (1,000 m) times the maximum depth (8 m) or 5,000 m^2. The dilution flux is therefore 500 m^3/s, which, divided by the effluent discharge rate of 100 L/sec, gives a dilution ratio of 5,000. How does this value, representing the ideal, complete mixing of effluent with seawater throughout the region shoreward of the reef under average conditions, compare with dilution ratios required for meeting standards?

Constituent	Effluent, C	Standard C	Required Dilution Ratio, Ce/Cs
SS	120 mg/L	(Study separately)	
BOD	140 mg/L	"1 mg/L"	140
Cl	2 mg/L	0.002 mg/L	1,000
Fecal	1,000,000	200	5,000
Coliform	1,000 if chlorinated	200	5

The required dilution ratios, with one exception, are much less than the ideally attainable value of 5,000. As for the fecal coliform without chlorination, the concentration will be reduced by a factor of 10 after 1 hour due to bacterial disappearance; during this time the plume will travel an average of 360 m.

The suspended solids mass discharge rate is 120 mg/L x 100 L/sec = 375,000 kg/year. According to Table 4-1, about 5 percent or 18,900 kg/year will have particle settling velocities greater than 0.1 cm/sec. If these settle out of a plume within 8 m of the bottom, they will reach the bottom within 8,000 sec; the 0.1 m/sec current will carry the particles up to 800 m in this time.

The particles will settle in a swath, the width of which depends initially on outlet diffuser length, but which after 800 m of drift will be several times greater than the water depth, and, in this case of unidirectional current next to a straight shoreline, will be an order of magnitude less than the travel distance. Thus it is assumed that the plume path is much greater than 8 m and much less than 800 m. Use 80 m for the width, for order-of magnitude computations.

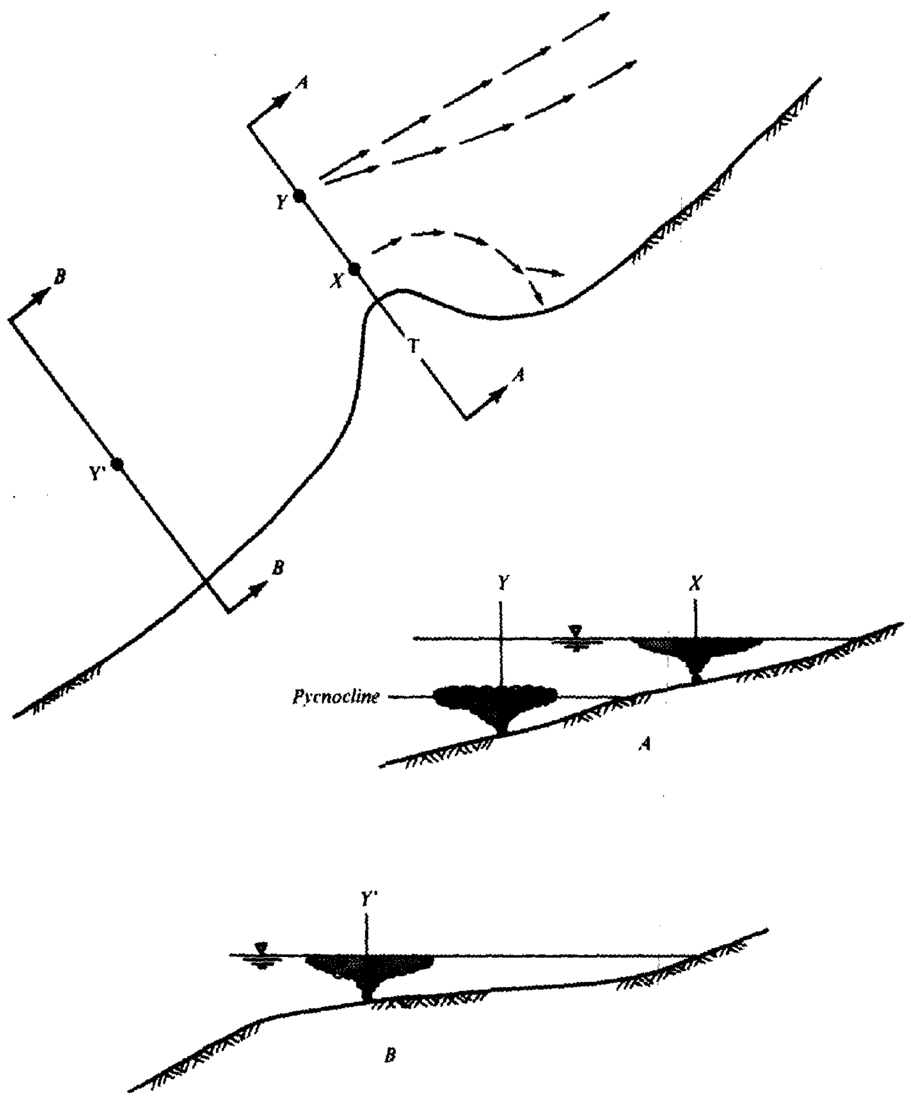
For 18,900 kg/year settling in a swath 800 m long by 80 m wide, the accumulation rate is 0.3 kg/m /year, within the standard. With current speeds of 0.1 m/sec, and with such wave activity as can pass over the reef, frequent resuspension of these settled particles is probable.

For this small outfall, discharge shoreward of the reef is feasible. With an outlet at the deepest point between the reef and the shore, the above ratios are recalculated for slow (say, 10-percentile) current speeds and for the peak rate of discharge, to confirm that standards will be met even under unusual conditions. With or without chlorination, coliform standards and chlorine standards will be met on the beach and over the reef.

Example 2. Assume the same conditions and contaminant concentrations as in Example 1, but with a tourist development that increases the discharge to 1,000 L/sec.

The dilution flux of 500 cms, divided by the discharge rate, gives a dilution ration of 500. This is adequate for BOD dilution; but insufficient for chlorine dilution. Fecal coliform standards will be met because of bacterial disappearance. The suspended solids accumulation rate will now be 3 $kg/m^2/year$, exceeding the standard, particularly during slow currents and peak discharge rates. Accordingly, it will be necessary to extend the outfall beyond the reef so that the transect cross-sectional area of ocean between the reef and the chosen discharge point is greater than A, say 2A, in which A is the area required to obtain ocean water flux sufficient to provide adequate effluent dilution at the reef. If A = 1,250 m with current speed 0.8 m/s, the ocean flux of 1,000 m^3/s will provide a dilution capacity of the order of 1,000. Doubling the area will double the dilution capacity.

Fig. 4-14. Optional Discharge Locations for a Hypothetical Outfall



Note: Discharge at *X*: effluent plume rises to the surface and is entrained in the gyre behind the point. Discharge at *Y*: effluent remains submerged and is carried past the point. Discharge at *Y'*: effluent plume rises to the surface and will be carried past the point.

Example 3. A 5 m^3/s paper mill effluent has 3,000 mg/L suspended solids (SS). Sedimentation tests show that 50 percent of the particles have settling rates in seawater in excess of 0.1 cm/sec. Consider discharging this effluent to the sheltered bay shown in Figure 4-7a, the average depth of which is 10 m, and in which the 80 percentile current speed is 0.08m/s. The bay area is about 100 km^2. Here, the question is whether there is an appropriate site such that the rate of accumulation of solids on the seabed does not average more than 1 $kg/m^2/year$. The bay will have too little wave action or currents seldom exceeding 8 cm/s, to resuspend sediments. What settles on the seabed will remain there.

What area will be required to accommodate this discharge, at 1 $kg/m^2/year$? 5 m^3/s x 3,000 mg/L x 86,400 sec/day x 365 days/year equals 4.73 x 10^5 kg/year, requiring 473 km^2, far more area than is contained in the bay. Therefore discharge to the bay should not be permitted without treatment to greatly reduce the SS load. This conclusion is reached even without considering current speeds or particle settling velocity.

Example 4. An outfall is to be sited in a deep, seasonally stratified tidal estuary, near the treatment plant sited at T in Figure 4-14. Drogues deployed at point X tend to be entrained in a gyre induced during the seasonal flood flow. Those deployed at Y, generally drift parallel to the coastline.

Transect A is typical of transects near the point of land on which the treatment plant (T) is located. An effluent plume from a diffuser at X would float. A plume from Y would be below the pycnocline, or depth of strongest density gradient, and so would be held submerged. In any event, X is considered too close to shore because the effluent plume will be entrained by the gyre. If we discharge at point Y on Transect A, any effluent that reaches the surface will be efficiently advected away. If the plume is trapped below the pycnocline, it will be kept away from the shoreline; it may also cause local oxygen depletion at or below the pycnocline.

Check the likelihood of plume submergence by using the methods of Section 4.6.4: (1) Obtain density profiles at all seasons for the area. (2) Measure the density difference and depth difference between discharge depth and the water surface. (3) For projected discharge rates, use equations 4.25 to 4.30, as appropriate to determine the density difference required for plume submergence. (4) If this density difference exists or is exceeded between discharge depth and the surface during any portion of the year the plume will be submerged beneath the surface for that part of the year. Determine the portion of the year for which such a density difference exists. (5) Repeat steps 2, 3, and 4 but for the pycnocline rather than the sea surface, to determine the seasons, if any, that the plume will be submerged beneath the pycnocline.

If the plume will indeed be submerged a significant fraction of the year, determine the extent to which dilution alone will be sufficient to maintain acceptable dissolved oxygen levels. This may be accomplished with a sufficiently long diffuser (but check to see that any mandated initial dilution is obtained) or by going farther offshore to deeper waters for greater initial dilution. Note here the increased construction costs for depths in excess of about 60 m.

An alternative is to seek another alignment. In this example, Transect B has a much flatter profile, so that point Y' is as far offshore as point Y, but is in water shallow enough to be above the pycnocline. The extra costs of an outfall to point Y' must be justified by the extra benefits accruing when gyre entrapment is avoided and the plume is submerged.

Example 5. Is it feasible to discharge 1 m^3/s of secondary effluent to the coastal lagoon shown in Figure 4-15? The mean tide range is 1 m. The area of the lagoon is 100 hectares. Tides are semidiurnal to diurnal, depending on the lunar phase.

The tidal prism is 100 ha x 1 m = 1,000,000 m^3. Diurnal tides have a period of about 24.8 hours. At best, if the effluent plume is mixed uniformly throughout the lagoon all of it will be exported from the lagoon on an ebb tide. With a sufficiently strong permanent longshore current, seawater entering the lagoon on the flood will have no sewage content at all. That is, there is complete exchange of water between lagoon and ocean during a tide cycle.

The mass of sewage entering per tide cycle is Q x T = 1 m^3/s x 24.8 hr x 3,600 s/hr = 89,280 m^3. The volume of seawater that leaves the inlet is the tidal prism, one million m^3. The ratio of these two numbers is 11.2. This is the far-field tidal prism dilution, the theoretically greatest dilution value attainable. Actual dilution will be less because complete mixing with lagoon contents does not generally occur.

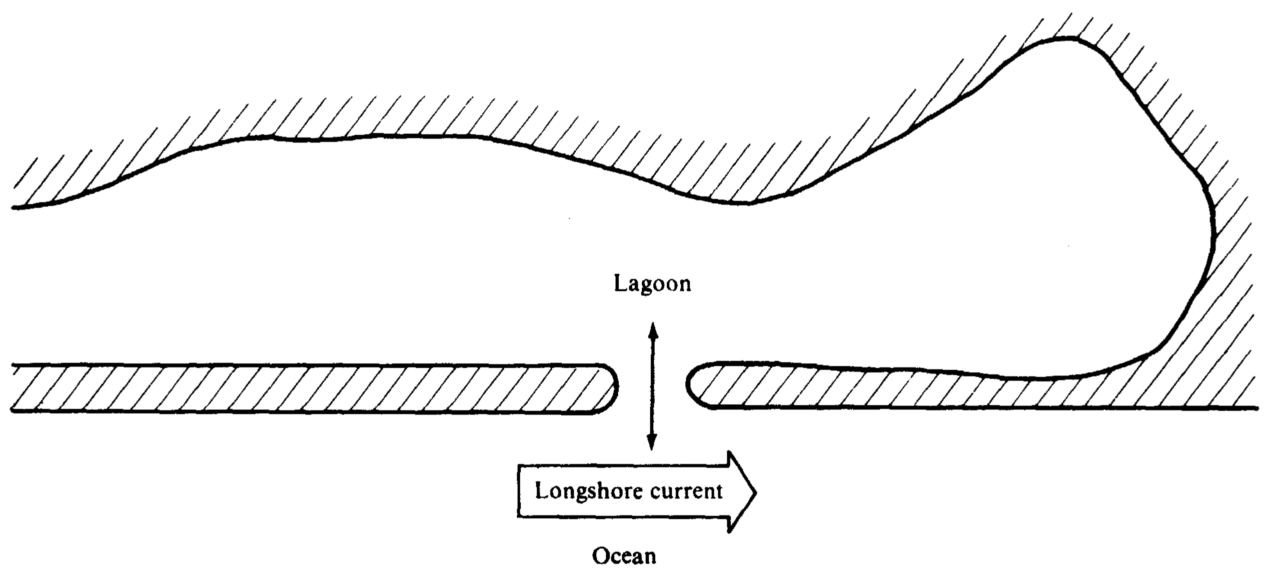
If the effluent constituents and the water quality standards are those of Example 1, a dilution ratio of 11.2 is clearly insufficient to satisfy the water quality standards. Furthermore, the addition of 1 m^3/s of freshwater effluent will reduce salinity in the lagoon, which may have an impact on the health and composition of lagoon biota.

4.6 Outlet Design and Initial Dilution

4.6.1 Simple Open Ends vs. Multiport Diffusers

The outlet of an ocean outfall may consist of a simple open end, perhaps with a slight upward turn, or it may consist of a multiport diffuser, in which the end of the outfall pipe is capped, and the last sections of pipe contain a regularly spaced line of relatively small ports. The purpose of a multiport diffuser is to ensure a much greater initial interception of ambient dilution water by the effluent stream for greater initial dilution. Most small outfalls, as well as larger ones built before 1950, have simple open ends. In recent decades, the multiport diffuser has become a conventional design feature for large-diameter outfalls for sewage and cooling water discharges. Some multiport diffusers have been constructed with from two to as many as six branching arms. These are unnecessary; by the time the sewage field has moved one or two diffuser lengths downstream, average

Fig. 4-15. Hypothetical Lagoon into Which Sewage is Discharged (Example 5)



dilution is the same whether the current is normal or parallel to the diffuser section (6). Thus, most diffusers are simple extensions of the outfall pipe itself, sometimes with changes in diameter.

A multiport diffuser provides increased initial dilution within a small mixing zone near the diffuser, increasing the possibilities of plume submergence due to ambient stratification, whereas a plume from a simple open pipe end would be more likely to rise to the sea surface. By the time the diluted effluent has traveled a few diffuser lengths downstream, particularly when there is little stratification and no plume submergence, the plume dilution distribution becomes independent of the diffuser length (see Section 4.4.5). Provision of even the most elaborate diffuser at a poor site is likely to be worse than a modest diffuser or possibly an open pipe end at a good site.

Use of the simple open end is recommended in cases where it will provide adequate initial dilution to meet water quality standards. It is also recommended in cases where plume submergence due to a diffuser is unattainable or undesirable. A simple open end is obviously the easiest terminus to build and maintain.

4.6.2 Initial Dilution for Plumes from Single Round Ports

Over the past quarter century there has been extensive theoretical and experimental research on the movement and dilution of buoyant plumes from round discharge ports, such as an open pipe end. The trajectory and dilution pattern have been related to orientation of the port (vertical, horizontal, or intermediate), to the density difference between the effluent and the ambient fluid, to the mass flux and the momentum flux, and to the density stratification. The accuracy of dilution predictions by these analytical procedures is considered to be within 15 to 20 percent. Orientation (aim) of the discharge port is an important factor for dilution when the discharge momentum is large and the discharge is jet-like; but orientation is of negligible importance when momentum is small and the plume motion is driven by its buoyancy. For a more complete discussion, see Fischer et al. (6).

In most practical cases encountered in ocean outfall diffuser port design, the port emission Froude number

$$F = u/\left(g \frac{\Delta\rho}{\rho} D_{\rho}\right)^{1/2} \qquad (4.24)$$

is less than 4, in which case the momentum can be neglected in the analysis. Here, u is the velocity of flow leaving the port, D is the port diameter, $\Delta\rho$ is the density difference between effluent and ambient fluid, ρ is the ambient fluid density, and g is the gravitational acceleration.

When momentum is neglected, the principal dilution equation for a single, round-port discharge into an unstratified ambient is:

$$(4.25) \qquad S = 0.089 \frac{\left(g \frac{\Delta\rho}{\rho}\right)^{1/3} y^{5/3}}{Q^{2/3}}$$

in which S is the plume centerline dilution at elevation y above the discharge point, and Q is the discharge rate.

When there is a linear ambient density gradient $d\rho_a/dy$, that is a constant rate of change of seawater density with respect to height, y, the plume will rise to a maximum elevation, y_{max}, above the outlet:

$$(4.26) \qquad y_{max} = 3.98 \left(Qg \frac{\Delta\rho}{\rho}\right)^{1/4} \left(- \frac{g}{\rho} \frac{\Delta\rho_a}{dy}\right)^{-3/8}$$

The centerline dilution at the height, y_{max}, the elevation below which the plume spreads out, is:

$$(4.27) \qquad S = \frac{0.071 \left(g \frac{\Delta\rho}{\rho}\right)^{1/3} (y_{max})^{5/3}}{Q^{2/3}}$$

a result identical in form to Equation 4.25, but whose constant coefficient is 20 percent smaller. The reduction in dilution for y_{max} is due to the fact that for the last part of its buoyant rise, the plume passes up through its own submerged cloud, and entrains itself rather than clean ambient seawater.

These equations are for the case of no ambient current and for linear density gradient (or zero gradient). In a current, the dilution, S, attained by the time the plume has attained a given height, y, will be greater than with no current. With stratification, y_{max} is less with a current than with no current.

Results obtained by assuming a linear density gradient can be also applied in the more common case of a non-linear density gradient discussed in Section 4.6.4.

4.6.3 Initial Dilution for Plumes from a Line Source

The merging of individual round-port discharge plumes issuing from many ports evenly spaced along a diffuser can be approximated by a "line plume" or "two-dimensional plume." Equations for the dilution and rise height of line plumes are analogous to those for round plumes in a still ambient (no current), given in the previous section.

When the ambient is of uniform density,

$$(4.28) \qquad S = 0.38 \frac{\left(g \frac{\Delta\rho}{\rho}\right)^{1/3} y}{q^{2/3}}$$

in which q is the effluent discharge per unit length of the line source. When the ambient has a linear density gradient $d\rho/dy$, the maximum height of plume rise is

$$(4.29) \qquad y_{max} = 2.84 \frac{\left(g \frac{\Delta\rho}{\rho} \cdot q\right)^{1/3}}{\left(- \frac{g}{\rho} \frac{\Delta\rho_a}{dy}\right)^{1/2}}$$

and the dilution ratio at the height of rise is

$$(4.30) \qquad S = 0.31 \frac{\left(g \frac{\Delta\rho}{\rho}\right)^{1/3} y_{max}}{q^{2/3}}$$

a result similar to that of Equation 4.28, but with dilution reduced owing to the plume's entraining its own spent cloud, as in the case of Equation 4.27.

These results are for a line source infinitely long, in an ambient with no current. Following earlier work by Fan (5) and Wright (15), Roberts (13) studied the case of a line source of finite length in a current of uniform density, a case of great practical importance in outfall diffuser design. Roberts' results are summarized in Figure 4-16, a plot of the parameter group Sq/ud versus the line emission Froude number. Here, S is the minimum initial dilution (greatest relative effluent concentration) found at the end of the period of plume rise, at the sea surface, and U is the current speed. These are curves for line plumes aligned perpendicular to, parallel to, and at 45 degrees to the current direction. Roberts' studies showed that the ratio of line-length to water-depth is not important for the results presented. In his studies, this ratio varied between 3.7 and 30.

For F less than about 0.1, the effect of current appears to be negligible, and for all plume/current alignments.

$$(4.31) \qquad Sq/ud = 0.27F^{-1/3}$$

which reduces to

$$(4.32) \qquad S = 0.27 \frac{\left(g \frac{\Delta\rho}{\rho}\right)^{1/3} d}{q^{2/3}}$$

which is essentially identical to that of equation 4.30 with d equal to y_{max}. The 15 percent difference between the coefficients 0.31 and 0.27 is well within the margin of uncertainty for these analyses.

For F greater than 0.1, the effect of current begins to be seen, and with increasing F, the ratio Sq/Ud tends to the following constant limits:

a) Line perpendicular to current:		tends to 0.58
b) Line 45` to current:	$\frac{Sq}{ud}$	tends to 0.37
c) Line parallel to current:		tends to 0.15

(Note that for the ideal case of effluent mixed uniformly from surface to bottom, and uniformly across a swath just equal to the length of a line source perpendicular to the current, the dilution would be S = Ud/q; or Sq/Ud = 1. This "ideal" value may be compared with the three cases above.)

The Roberts' analysis provides an exceptionally clear and thorough means by which to estimate initial dilution from a multiport diffuser in an ambient current.

4.6.4 Quick Estimate of the Likelihood of Plume Submergence

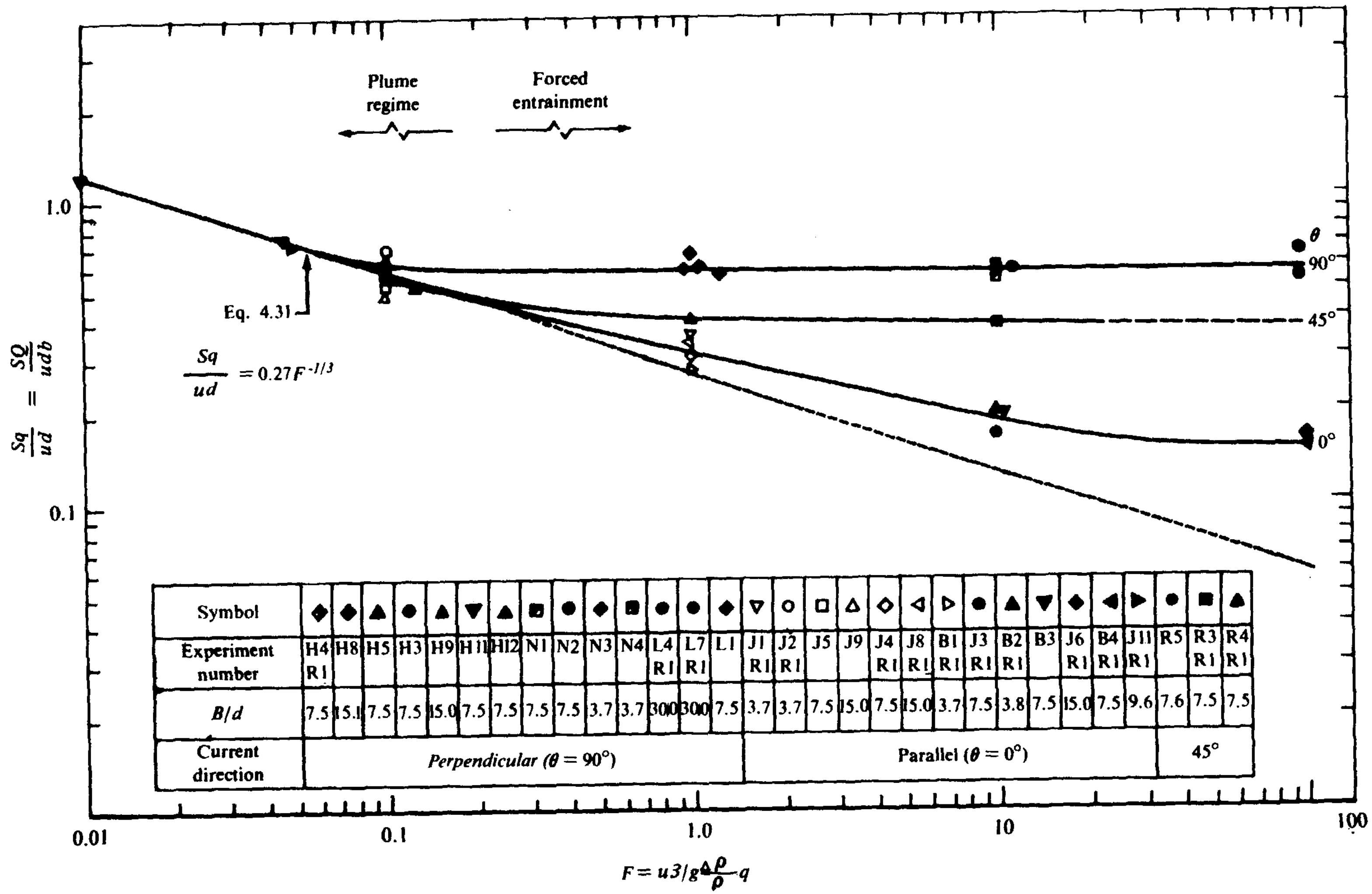
Equations 4.26 and 4.29 in the foregoing section are used to predict the rise height, y_{max}, of a plume rising in a linearly stratified environment. If y_{max} is less than the water depth, d, the plume is submerged.

These equations are good as far as they go, but in general, the density profiles in the ocean are not linear; they change constantly, especially seasonally; and sometimes density data are reported not as continuous surface-bottom profiles, but as only a surface value, a bottom value, and possibly a mid-depth value.

Because of such complications, it may be necessary to compare many seasonal profiles, construct a composite design profile(s), make linear approximations to them for use with equations presented in Sections 4.6.2 and 4.6.3, or use numerical models with the nonlinear profiles as discussed in Section 4.6.5.

However, the following procedure is relatively easy to apply, and makes good and complete use of available data. It takes advantage of two features:

Fig. 4-16. Experimental Measurements of Minimum Surface Dilution for a Finite Line Source of Buoyancy Flux in a Current



Symbol	◈	◆	▲	●	▲	▼	▲	▣	●	◆	▣	●	●	◆
Experiment number	H4 R1	H8	H5	H3	H9	H11	H12	N1	N2	N3	N4	L4 R1	L7 R1	L1
B/d	7.5	15.1	7.5	7.5	15.0	7.5	7.5	7.5	7.5	3.7	3.7	300	300	7.5
Current direction	Perpendicular ($\theta = 90°$)													

Symbol	▽	○	□	△	◇	◁	▷	●	▲	▼	◆	◀	▶	●	■	▲
Experiment number	J1 R1	J2 R1	J5	J9	J4 R1	J8 R1	B1 R1	J3 R1	B2 R1	B3	J6 R1	B4 R1	J11 R1	R5	R3 R1	R4 R1
B/d	3.7	3.7	7.5	15.0	7.5	15.0	3.7	7.5	3.8	7.5	15.0	7.5	9.6	7.6	7.5	7.5
Current direction	Parallel ($\theta = 0°$)													45°		

Source: Roberts (12).

1. Linearization of a density profile between the height of rise, y_{max}, and the discharge elevation, $y = 0$, introduces only modest error into computations. In essence, the height of rise of a plume depends much more on the ambient density values at $y = 0$ and at $y = y_{max}$ than on the details of the density distribution in between.

2. For outfall planning purposes, it is just as useful to ask whether a plume will be kept submerged beneath the surface and whether it will be kept submerged beneath middepth as it is to ask precisely what y_{max} will be, for a given plume and ambient.

The procedure illustrated in Figure 4-17 for a 400-meter multiport diffuser section in water 40 m deep shows the conditions under which the plumes will (1) rise the full 40 m to the sea surface ($y_{max} > 40$ m), (2) rise to y_{max} more than 30 m but less than 40 m, or (3) rise to y_{max} less than 30 m.

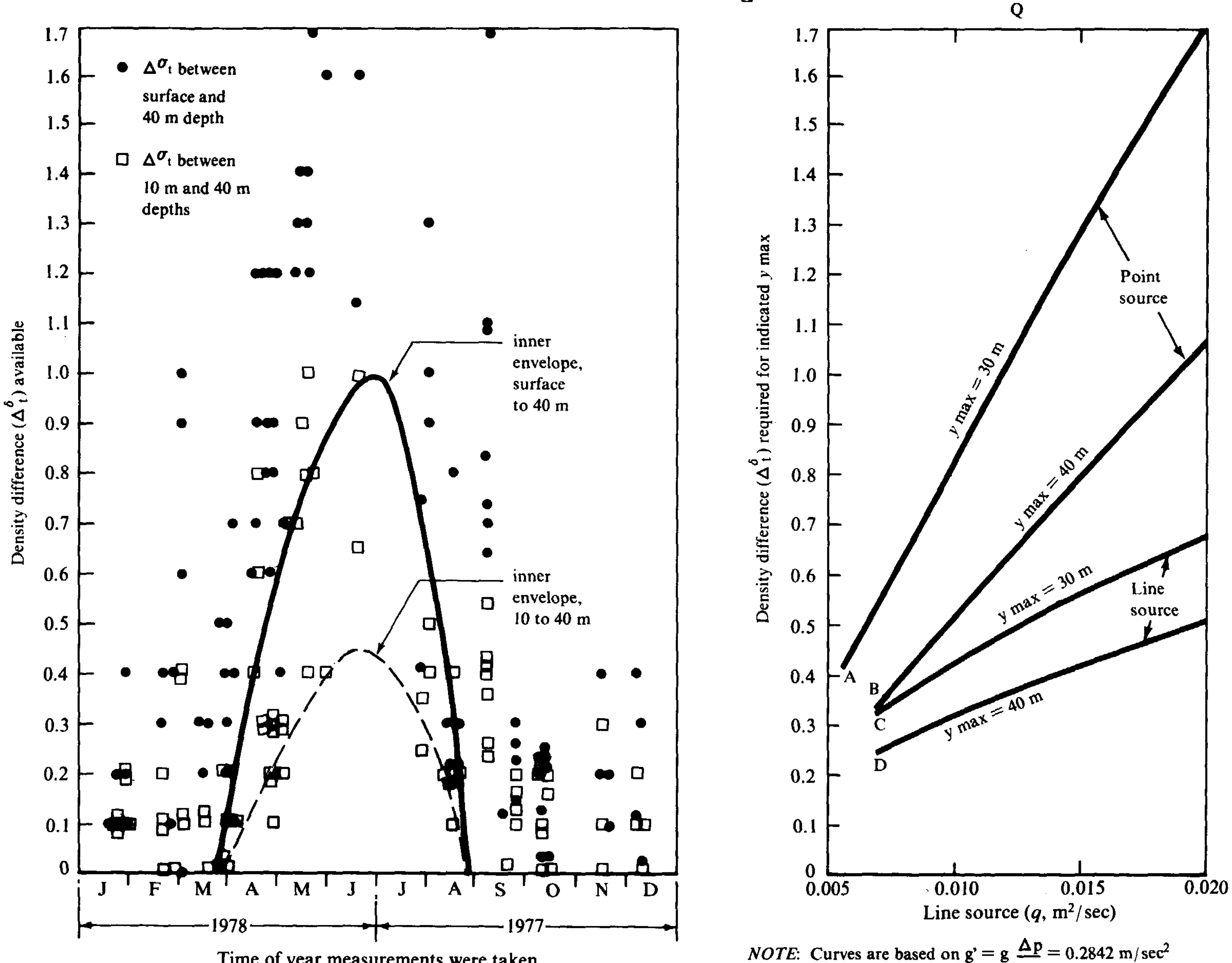
The left part of the figure contains a plot of data from all available density profiles for the example study area, expressed as the ambient water density difference between the surface and 40 m depth (solid circles) and between 10 m depth and 40 m depth (open squares). These density differences are plotted against calendar date to show seasonal trends. Inner envelope curves are fitted, such that few solid circles fall beneath the "surface to 40 m" curve, and few open squares fall beneath the "10 m to 40 m" curve. The plot shows that from April through August there is a fair guarantee of up to 0.4 kg/m of density difference to keep a plume submerged beneath the sea surface.

On the right side of Figure 4-17, the required density difference for plume submergence is plotted as a function of discharge rate Q from a point source using Equation 4.26, for y_{max} = 30 m and 40 m; and as a function of discharge per unit length, q, from a line source, using Equation 4.29 for y_{max} = 30 m and 40 m. The values of $g\Delta\rho/\rho$ and of ρ are noted on the figure.

Now: the right side of Figure 4-17 shows that a line source discharge rate of $q = 0.010$ m^3/s requires a density difference (between $y = 0$ and $y = y_{max}$) of at least 0.32 kg/m^3 in order to be submerged beneath the surface, and of at least 0.42 kg/m^3 in order to be submerged beneath the 10 m depth. Moving to the left side, one sees that the available density difference, 40 m to surface, exceeds 0.32 kg/m^3 consistently from mid-April to mid-August. The available density difference 40 m to 10 m consistently exceeds 0.42 kg/m^3 during parts of June and July.

The point-source curves can be used in the same way, either for simple open-end of pipe outfalls, or for the individual plumes from each of the many ports of a diffuser. Brooks (3) advises that in some situations y_{max} may be attained before the component port plumes have risen and spread to merge into what is effectively a line plume. Therefore, it is well to compute the rise heights both of an idealized line plume from a diffuser, and from a typical single component port, and use the more conservative of the two results.

Fig. 4-17. Case Study of the Density Difference Required and Available for Plume Submergence



NOTE: Curves are based on $g' = g \frac{\Delta p}{p} = 0.2842$ m/sec^2 and $p = 998$ kg/m^3

In summary, when this method is used to survey the likelihood of seasonal plume submergence, the following steps should be taken:

1. Collect density profile data, and plot density differences versus month or season as shown on the left side of Figure 4-17. (Note that the data need not be full profiles; simple pairs of, say, surface-bottom or middepth-bottom density values can be used.)

2. Using Equations 4.6.2 and 4.6.5, plot required density difference versus Q or q, for chosen values of y_{max}. In the equations, replace $d\rho_a/dy$ with the linearized form, density difference/y_{max}.

3. For any q or Q, examine the seasonal prospects for submergence for each y_{max} considered.

4.6.5 Numerical Models for Computing Initial Dilution

The equations presented in Sections 4.6.2 and 4.6.3 are derived from systems of governing differential equations describing buoyancy, mass, and momentum balances, integrated over plume rise height. To perform the integrations in closed form, simplifying assumptions, such as those of linear density gradient or no current, were made. Experience has shown that the integrated equations of the preceding sections yield answers of adequate practical accuracy in most sewer outfall design applications, certainly within the margin of confidence the results deserve. For smaller, shallower wastewater outfalls in particular, greater detail is generally not necessary, particularly when far-field results are independent of initial dilution. However, where local conditions warrant more detailed calculations, one or more numerical plume models may be used that have at their core the same fundamental governing equations as the closed-form equations of the preceding sections. For example, the United States Environmental Protection Agency Office of Water Program Operations uses the models listed in Table 4-5 (14). The model ULINE is a generalization of Roberts' analysis (12), described in Section 4.6.3, to an arbitrary ambient density profile. All models are available from USEPA.

4.6.6 Diffuser Design Pointers

4.6.6.1 Orientation

It is usually desirable to align a diffuser perpendicular to the current direction, as this orientation provides greater initial dilution than any other alignment direction, when current speeds are significant (See Figure 4-16). Furthermore if, as usual, the prevailing currents are parallel to the shoreline and to depth contours, and if the outfall line from shore to the discharge site is perpendicular to shoreline and current direction, there will be no need to install a bend at the diffuser section. However, for essentially uniform distribution of discharge along the diffuser length at all rates of flow, it is essential that the diffuser alignment be as horizontal as possible. For this, the diffuser should run along a depth contour if at all possible. If not, the depth factor must be included in the calculation of port sizes.

TABLE 4-5

Summary of Numerical Model Characteristics for Buoyant Plumes

Parameter	UPLUME	UOUTPLM	UMERGE	UDKHDEN[a]	ULINE
Port[b]	single	single	multiple	multiple	slot/closely spaced
Discharge angle[c]	-5^{o} to 90^{o}	-5^{o} to 90^{o}	-5^{o} to 90^{o}	-5^{o} to 130^{o}	assumes 90^{o}
Density profile	arbitrary	arbitrary	arbitrary	arbitrary	arbitrary
Current speed	no	constant with depth	arbitrary	arbitrary	arbitrary
Current angle relative to the diffuser[d]	n/a	assumes 90^{o}	assumes 90^{o}	45^{o}-135^{o}	0^{o}-180^{o}

[a] For a single port discharge the current angle may be in the range of 0^{o} to 180^{o}. For an angle greater than 90^{o} the program converts it to the supplementary angle. (Note: 0^{o} and 180^{o} give the same results.)

[b] All the models except ULINE reduce the data to a single port discharge. UPLUME and UOUTPLM detect merging of adjacent plumes and alert the user, but do not account for this in the remainder of the calculations whereas UMERGE and UDKHDEN do. ULINE converts the data to a slot discharge.

[c] The discharge angle limits are those allowed by the subroutines LIMITS in each of the programs. They are not necessarily the theoretical limits associated with these models. Caution should be exercised when using the models for angles beyond these limits.

[d] 90^{o} is perpendicular to the diffuser. At a discharge angle of 0^{o} (horizontal) and a current angle of 90^{o}, the discharge and the current are parallel and in the same direction.

Source: Muellendorff, W.F., et al. (12).

The relevant reports and programs by Muellendorf, et al., may be obtained from the National Technical Information Service, U.S. Department of Commerce. Title: Initial Mixing Characteristics of Municipal Ocean Discharges. Vol. 1 (PB 86-137478 @ $16.95) and Vol. 2 (PB 86-137486 @ $70.00, 1986 prices). The latter includes floppy disc programs for IBM or IBM compatible computers.

Where there is significant sediment bedload past an unburied diffuser, accumulating sediment may block or enter the diffuser ports; on the other hand, sediment may be scoured from beneath the diffuser pipe. Such problems will be avoided by having the diffuser lie parallel to the direction of bedload travel. In this case, the initial dilution can be achieved as well as with a perpendicular alignment by using a somewhat longer diffuser section. In any event, the diffuser orientation does not affect initial dilution when current speeds are small (Figure 4-16).

4.6.6.2 Diffuser Length

For a given outlet site, the next step is to use a multiport diffuser or a simple open pipe end. If, for a given discharge rate Q, site conditions and currents are such that a diffuser rather than a simple open-ended pipe is called for, then determine the appropriate diffuser orientation and select a diffuser length, b, that yields $q = Q/b$ small enough to meet near-field water quality standards and to achieve (or avoid) plume submergence.

4.6.6.3 Port Size and Spacing for Uniform Discharge Distribution

It is desirable to have a uniform discharge per unit length issuing from the ports along the diffuser length. For many wastewater outfalls, this may be achieved largely by following two hydraulic guidelines: (1) keep the diffuser section at a constant depth, and (2) ensure that the sum of the cross-sectional areas of the ports is less than about 60 percent of the cross-sectional area of the diffuser pipe.

If effluent is less dense than the ambient water and the diffuser slopes, the effluent at low flows will favor the higher ports, with no flow (or even inflow) through the lower ports (most likely those farthest from the offshore end). Note that the port area criterion dictates that the end of the diffuser be closed.

Ideally, the pipe cross-sectional area should taper continuously and linearly from its greatest size, at its connection with the outfall pipe, down to zero at its end. This, with uniform discharge per unit length, will provide constant velocity and nearly uniform hydraulic conditions within the pipe (conducive to uniform discharge), and will allow scour velocities to be maintained for any settleable solids in the effluent. However, it is neither practicable nor necessary to provide a continuous taper of pipe area. Large outfalls have been designed with one or more decrements in diffuser diameter. Many smaller outfalls operate well with constant diameter diffuser sections.

Detailed computation of discharge distribution in a multiport diffuser follows the procedure proposed by Brooks (1,11), starting at the offshore, or downstream, end, where the pipe flow velocity is close to zero:

1. Assume a discharge rate through the furthest downstream port in the diffuser.

2. Using a stage-discharge relationship involving port area and shape, calculate the excess energy head, E, within the diffuser pipe at its downstream end, required to achieve the discharge rate in 1. Brooks (11, 3) conducted laboratory studies to determine the stage-discharge relationship for several different port designs. The studies consist of determining the function C ~ C $(V^2/2gE)$, where C is the discharge coefficient for the port and

$$p = C \frac{\pi}{4} a^2 (2g\ E)^{0.5} \quad (4.33)$$

in which p is the discharge rate for a port, a is the port diameter, V is the average pipe flow velocity, and E is the total energy head in the diffuser pipe, relative to the ambient outside the pipe. For smooth bellmouth ports, flowing full:

$$C = 0.975\ (1 - V^2/(2gE))^{0.375} \quad (4.34)$$

For sharp edged ports, flowing full:

$$C = 0.63 - 0.58\ V^2/(2gE) \quad (4.35)$$

Other port designs have their own functional relationships for C.

3. Calculate the average pipe flow velocity, V, in the section of diffuser pipe feeding the last port.

4. Move up the pipe to the next port. Compute the energy head there by adding the friction head lost beween ports, to the energy head at the last port.

5. Compute the discharge through the next port. Compared to the last port, the flow at the next port may be greater, because of increased energy head required to overcome friction losses; or less, because of the increased velocity in the pipe effectively reduces the discharge coefficient in the total head-discharge equation.

6. Compute the increased pipe velocity in the pipe section leading to the next port upstream, etc. BASIC computer program for this is listed in Table 4-6.

This analysis proceeds iteratively, and can accommodate any changes in port size, port spacing, pipe diameter, pipe internal friction, or pipe elevation (for a nonhorizontal pipe alignment). By the time the discharge for the upstream port has been calculated, the discharge through each port and the total head (relative to the ambient pressure outside the diffuser) required to drive the diffuser are known. The absolute value of these quantities depends entirely on the discharge rate assumed for the first port, so several computations are usually necessary to get a satisfactory result for a particular design Q.

This diffuser analysis procedure can be easily computerized, as shown by the simple BASIC program listed in Table 4-6. (Users will wish to modify and extend this simple illustrative version of the program.) Use of such a program is advisable where changes in diffuser pipe diameter are contemplated, or where the diffuser alignment has appreciable slope. Alternatively, for the many cases in which a constant-diameter diffuser pipe can be contemplated and where a horizontal alignment can be found, one may use the summary curves developed by French (7), presented in Figure 4-18. The latter procedure is swift, and involves little or no iteration. Each figure contains two families of curves. The top family are plots of port discharge ratio R to port area ratio, X. The port discharge ratio is the port discharge per unit length of diffuser at any point, X, divided by the port discharge per unit length at the offshore end of the diffuser, where X = 0. The abscissa, X, is the distance, x, from the offshore end, multiplied by the aggregate port area per unit length between 0 and x, and divided by the diffuser pipe area. In other words, X is the ratio of aggregate port area to pipe cross-sectional area. The variable parameter, W, on the curves is a normalized Darcy-Weisback friction factor. For a great number of practical cases, its value ranges between 1 and 6.

In order that the sum of the cross-sectional areas of the ports may be less than about 60 percent of the cross-sectional area of the diffuser pipe, X should not exceed 0.6. In both figures, note that for X less than 0.6, and for W between 1 and 6, R remains within 20 percent of unity (perfectly uniform distribution), dipping below 0.8 only for X = 0.6 and W = 1 for sharp-edged ports.

The upper families of curves indicate how to select port size, number, and spacing, as well as port hydraulic design, for adequately uniform R throughout its length. The lower families of curves indicate B, thus the total head, E, required to drive the flow through the diffuser, for a given pipe velocity, V.

For well-screened or well-treated effluent, port diameters may be as small as 5 cm. However, if the effluent is only given coarse screening, the port diameter should be no less than about twice the screen bar spacing.

4.6.6.4 Simple Ports vs. Ports with Risers

The diffuser pipe may be laid either exposed on the seabed, with a properly prepared bed and with revetment; or it may be buried in a backfilled trench. In the former case, ports are simply holes in the sidewall of the pipe. In the latter case, the ports necessarily consist of small pipes rising from the diffuser pipe crown through the backfill and extending to 0.5 m or more above the finished grade of the backfilled pipe trench. There are pros and cons to each type of port, and selection is largely a function of local

TABLE 4-6

Basic Program to Compute Diffuser Port Flow Distribution

```
100  PRINT "DIFFUSER FLOW DISTRIBUTION"
110  PRINT "ASSIGN VARIABLES STARTING WITH DOWNSTREAM"
111  PRINT "END OF THE DIFFUSER.  USE SI UNITS."
120  PRINT
130  PRINT "BELLMOUTH PORTS ASSUMED.  IN THIS EXAMPLE, GEOMETRY"
131  PRINT "CHANGE WILL BE LIMITED TO DIAMETER."
140  PRINT "INITIAL PIPE DIAMETER.  D1=";
141  INPUT D1
150  PRINT "NUMBER OF PORT STATIONS AT THIS DIAMETER.  I1=";
151  INPUT I1
160  PRINT "ALTERED PIPE DIAMETER.  D2=";
161  INPUT D2
170  PRINT "PORT SPACING.  K=";
171  INPUT K
180  PRINT "PORT DIAMETER.  A=";
181  INPUT A
190  PRINT "NUMBER OF PORTS PER STATION.  P=";
191  INPUT P
200  PRINT "TOTAL NUMBER OF PORT STATIONS.  T=";
201  INPUT T
210  PRINT "MANNING FRICTION FACTOR.  N=";
211  INPUT N
220  PRINT "DIFFUSER SLOPE.  S=";
221  INPUT S
230  PRINT "DENSITY DIFFERENCE BETWEEN EFFLUENT AND AMBIENT.  KG/CU M.Z=";
231  INPUT Z
240  PRINT "ASSUMED DISCHARGE THROUGH LAST PORT.  D1=";
241  INPUT D1
250  REMARK:  INITIAL VALUE OF PIPE DIAMETER.  D. IS D1
260  D=D1
270  REMARK:  GRAVITATIONAL ACCELERATION G
280  G=9.81
290  REMARK:  DARCY-WEISBACH FRICTION FACTOR
300  F=12.7*G*N↑2/D↑0.3333
310  REMARK:  PIPE AREA IS A1.  PORT AREA IS A2
320  A1=D*D*3.1416/4
330  A2=A*A*3.1416/4
339  PAGE
340  PRINT "PORT STATION", "PORT D1", "PIPE D", "PIPE DIAMETER"
349  PRINT
350  PRINT
351  PRINT "1",D1,D1*P.D
360  REMARK:  CALCULATE INITIAL DOWNSTREAM TOTAL HEAD.  E.
370  C=0.975
380  E=(C1/C/A2)↑2/(2*G)
390  REMARK:  CALCULATE INITIAL PIPE DISCHARGE AND VELOCITY
400  D=D1*P
410  V=D/A1
420  REMARK:  PROCEED WITH PORT-BY-PORT ITERATION
430  FOR I=2 TO T
440  E=E+K*F*V↑2/(2*G*D)+K*S*Z/1000
450  C=0.975*(1-V↑2/(2*G*E))↑0.375
460  C1=C*A2*SDR(2*G*E)
470  C=D+D1*P
480  V=D/A1
490  PRINT I,D1,D,D
500  IF I<I1 THEN 520
510  D=D2
511  A1=D*C*E.1416/4
520  NEXT I
530  END
```

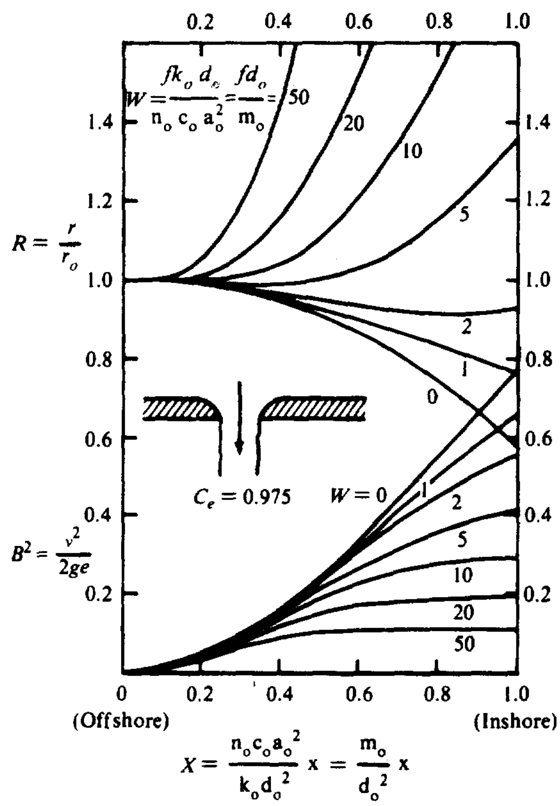
TABLE 4-6
continued

DIFFUSER FLOW DISTRIBUTION
ASSIGN VARIABLES STARTING WITH DOWNSTREAM
END OF THE DIFFUSER. USE SJ UNITS.
BELLMOUTH PORTS ASSUMED. IN THIS EXAMPLE. GEOMETERY
CHANGE WILL BE LIMITED TO DIAMETER.
INITIAL PIPE DIAMETER D1=1.1
NUMBER OF PORT STATIONS AT THIS DIAMETER. I1=6
ALTERED PIPE DIAMETER, D2=1.5
PORT SPACING, K=5
PORT DIAMETER, A=.2
NUMBER OF PORTS PER STATION, P=4
TOTAL NUMBER OF PORT STATIONS, T=20
MANNING FRICTION FACTOR, N=016
DIFFUSER SLOPE, S=01
DENSITY DIFFERENCE BETWEEN EFFLUENT AND AMBIENT', KG/CU M.Z=24
ASSUMED DISCHARGE THROUGH LAST PORT, Q1=.1

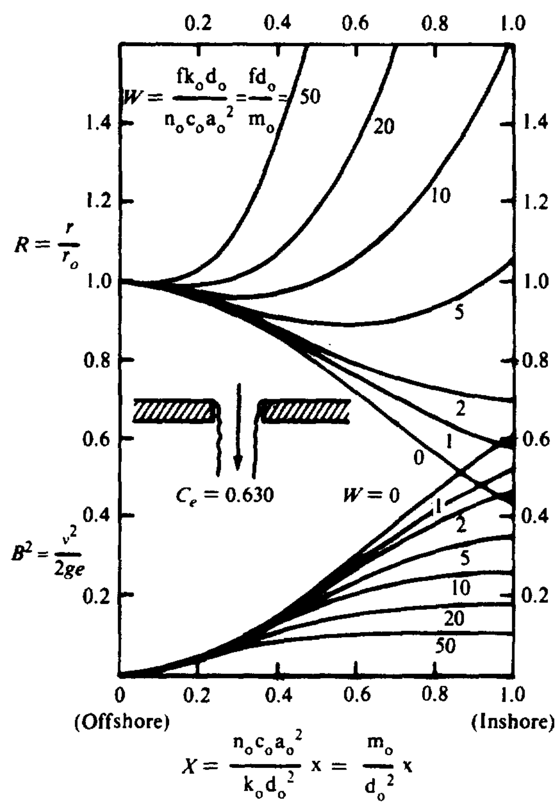
PORT STATION	PORT DISCH AT THIS STATION	PIPE FLOW AT THIS STATION	PIPE DIAMETER
1	0.1	0.4	1.1
2	0.099601758804	0.798407035216	1.1
3	0.0982837532893	1.19154284837	1.1
4	0.0962426866920	1.57651279514	1.1
5	0.0936656390584	1.95117535135	1.1
6	0.0907358996952	2.31411895013	1.1
7	0.0864096232476	2.65975744312	1.5
8	0.102744415488	3.07073510507	1.5
9	0.101540269793	3.47689618424	1.5
10	0.100323096425	3.87818856994	1.5
11	0.0991382303975	4.27474149153	1.5
12	0.0980315385712	4.66686764582	1.5
13	0.0970496643247	5.05506630312	1.5
14	0.0962399076893	5.44002593387	1.5
15	0.0956495798966	5.82262425346	1.5
16	0.0953247533348	6.2039232668	1.5
17	0.0953084683474	6.58515714019	1.5
18	0.0956386293511	6.96771165759	1.5
19	0.0963459690823	7.35309553392	1.5
20	0.0974525150554	7.74290559414	1.5

Fig. 4-18. Selection of Diffuser Port Sizes

A. ***R*** **and** ***B***^{2} **for bell-mouthed ports, uniform port parameter**



B. ***R*** **and** ***B***^{2} **for sharp-edged ports, uniform port parameter**



Note:

a = port diameter
B = square root of ratio of velocity head to total head
C = normalized discharge coefficient
c = discharge coefficient
D = normalized diffuser pipe diameter
d = diffuser pipe diameter
E = normalized total head
e = total head
f = Darcy-Weisbach friction factor
g = acceleration of gravity
K = a constant
k = longitudinal port spacing
M = normalized port parameter
m = port parameter
n = number of ports at a given longitudinal position
p = port discharge
q = flow within diffuser pipe
R = normalized port discharge per unit length
r = port discharge per unit length of diffuser pipe
V = normalized velocity in diffuser
v = velocity in diffuser pipe
W = dimensionless ratio
X = normalized distance from offshore end
x = distance from offshore end of diffuser pipe

Source: French (1972).

conditions. If diffuser pipe is laid on the seabed, the cost of riser pipes is avoided; there are no riser pipes to break off if snagged by trawls or anchors; and the diffuser pipe is relatively accessible for cleaning or repair. However, the pipe itself is subject to damage from anchors, trawls, or waves; if there is sediment bed load transport at the site, the pipe may cause sediment to be deposited on one side, and may possibly allow sediment to enter the pipe via the ports. On the other hand, sediment may be eroded out from under the pipe; but damage to the pipe from this may be avoided by bedding the pipe adequately on crushed stone.

A buried diffuser with riser pipes costs more to build, clean, and repair, but may be the desired alternative when there is significant shipping, anchorage or sediment transport due to the longshore drift or (orbital) wave movement. Sediment bedload can pass the riser pipes without difficulty. The riser pipe can usually be designed long enough to accommodate any seasonal aggradation and degradation of the seabed. The probability of anchor snags is reduced, since there is one riser pipe of relatively small diameter (0.2 to 0.5 m) every several meters instead of a continuous pipeline.

It is well to give the riser the simplest exterior form possible, that is without a crown or T-head of any sort, so that an anchor or trawl caught on a riser can be freed by hauling it straight up (Figure 4-19).

An important design criterion for diffusers with riser pipes is to ensure that the pressure loss through the risers, that is between the buried diffuser pipe and the outside ambient fluid, exceeds the product of riser pipe height, gravitational acceleration, and the density difference between the sewage effluent and the ambient seawater. This condition must be met if the effluent is to successfully purge seawater from all the risers.

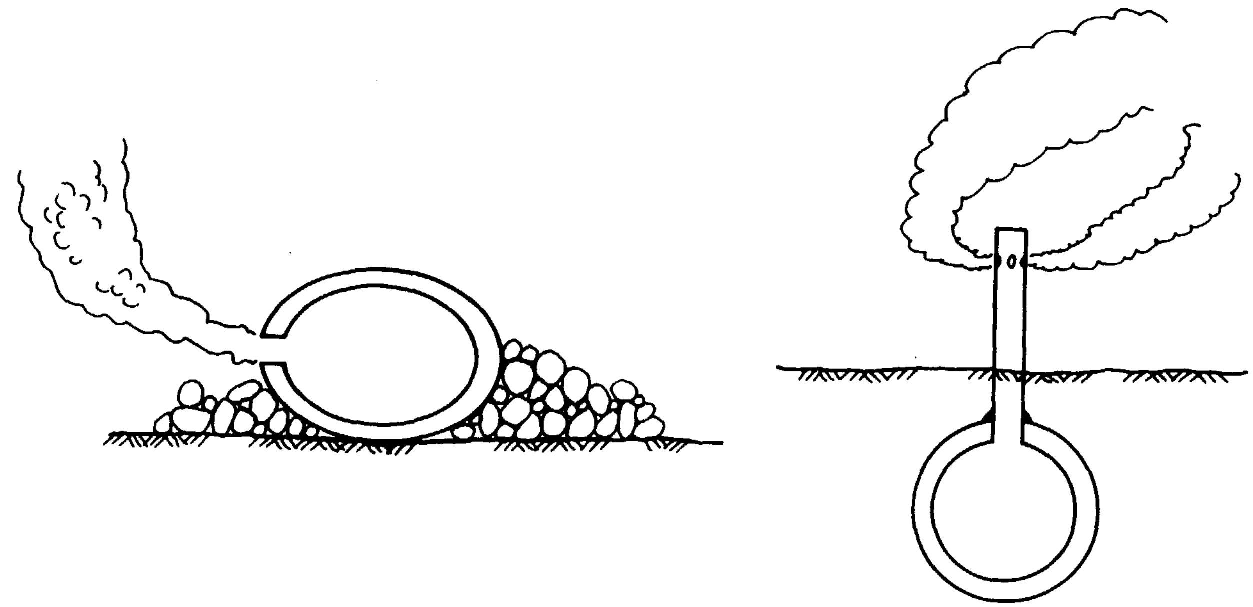
In sum, pipes with simple open ends or ports, properly laid on the sea bed, are generally to be preferred to buried pipes with risers; but the latter may be advisable in some areas.

4.7 Other Hydraulic Design Considerations

4.7.1 Pipe Diameter

Selection of an outfall pipe diameter involves striking a balance among a number of considerations. Decreasing the pipe diameter usually reduces construction costs, reduces incident wave forces, and increases the flow velocity in the pipe, which is important if settleable solids in the effluent are to be kept in suspension or to be resuspended. However, decreasing the pipe diameter also increases the required driving head for a given discharge rate. Often, the balance struck is one in which the average flow velocity is not less than 0.6 m/sec in the early design life of a project, yet the system head is not excessive at peak flow in the mature years of the project.

Fig. 4-19. Alternative Diffuser Configurations



4.7.2 Thrust at Bends

Should there be a bend in the pipeline to provide uniform diffuser depth, it must be provided with a resistance against a pipe motion due to the change in momentum of the flow within the pipe. At a 90-degree bend, the thrust in each direction is equal to QV; the resultant combined thrust is $\sqrt{2}$ QV.

4.7.3 Hydraulic Transients

Hydraulic transients must be considered for a pumped discharge to a long outfall. If there is a sudden shutdown of the pumps, a means for gradual deceleration of the flow within the pipe must be provided, to avoid damaging downsurges, and subsequent reflective upsurges, within the pipeline. Where pumping heads are not excessive, an open surge chamber just downstream of the pumps is appropriate. The chamber freeboard should be high enough to accommodate the pump shutoff head, and the chamber volume should be great enough to continuously feed water into the pipeline from the moment of pump shutdown (when the head is at its level pump-rated for discharge, Q) to the moment that flow in the pipe comes to rest. It may be advisable to throttle the flow to and from the surge chamber to dampen the downsurge somewhat. For design details on surge tanks and other control devices such as air chambers an air relief valves, see a standard text on surge analysis, such as that by Parmakian (10).

4.7.4 Hydrostatic Head

Excess hydrostatic head must be considered when discharging a freshwater effluent to a sea with significantly different density. Suppose an outfall discharges to a saltwater ocean at a depth of 10 m. The hydrostatic pressure at the outlet is 10 m x g x the density of saltwater, say 1025 kg/m^3. This hydrostatic pressure, imposed by the seawater outside the pipe, is balanced by the freshwater effluent inside the pipe--but the freshwater effluent is less dense, say 998 kg/m^3. Therefore the depth required to balance the saltwater pressure is 10 m x (1025/998) = 10.27 m. With no flow in the outfall filled with freshwater effluent, the water level found in an outfall manhole at the shoreline would be 0.27 m above sea level. The excess hydrostatic head, 0.27 m in this example, must be considered when designing manholes and surge tanks.

4.7.5 Drop Structures

It is desirable to keep all but very small quantities of air from being carried through the outfall. Air would impede the flow of effluent, and might cause unsightly bubble boils when released through the outlet. Sometimes the treatment plants discharging through the outfall is on a cliff or bluff, or there may be a high overland force main between the treatment plant and the outfall. In either case, there is potential for high-velocity flow in a partially full pipe to go through a hydraulic jump in the pipe and entrain large quantities of air into full-pipe flow in the outfall. In some cases, the pipe would be so steep as to merit the term "cascade" or "waterfall" to describe the flow in the pipe.

There are two design approaches to limit the entrainment of air into the outfall:

a) Allow the hydraulic jump or waterfall to occur, but provide a stilling section downstream to allow the entrained air bubbles to rise to the crown of the section for venting before the flow proceeds into the outfall pipe; or

b) Provide a vertical vortex drop structure in which the water swirls in a helix as it falls, in which the water is centrifuged to the walls and air is collected at the core of the shaft, and purged up the shaft counter to the water flow direction.

The second approach has been used successfully on very large outfall systems (3); but the first approach, with good design, will probably be adequate for most small outfall system. The rise velocity of bubbles larger than a millimeter in diameter is 0.1 m/sec or greater.(9) Therefore a stilling section following a section where entrainment occurs in a jump or cascade should have a length:

$$L \geq \frac{V\ (m/sec)}{0.1\ m/sec} \times \text{flow depth}$$

in which V is the flow velocity in the section. This will allow any bubble layer than a millimeter to rise from the invert to the water surface and escape, before being swept out of the stilling section. Obviously, the stilling section should be designed with appropriate crown venting.

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4.9 NOTATION FOR CHAPTER 4

A plume cross-sectional area (m^2)

B stream breadth (m); ratio of velocity head to total head in a diffuser

b diffuser length (m)

C concentration; port discharge coefficient

C_o initial concentration

C_t concentration at time t

D depth of discharge (m)

D_p	port diameter
d	water depth (m); pipe diameter (m); sill depth in a strait (m)
E	Initial value of diffusivity; energy head (m)
$\overline{E}$	diffusion tensor
ε	dispersion coefficient
F	Froude number
f	Coriolis acceleration (m/sec ^{2})
g	gravitational acceleration (m/sec^2)
h	difference in water surface elevation between two points (m)
k, k_e	decay rate coefficient, 1/hours or 1/days
L	distance downstream (m); length of stilling section (m)
L_s	cloud size or scale
n	Manning friction factor
p	pressure, port discharge rate
Q	discharge rate (m^3/sec)
Q_e	effluent discharge rate (m^3/sec)
Q_r	river discharge rate (m^3/sec)
Q_t	volume of tidal prism divided by the ebb low period (m^3/sec)
q	discharge rate per unit length of line source (m^2/sec)
R	port discharge ratio
r	source and sink terms in mass transport equation
S	dilution ratio; friction slope; salinity (parts per thousand)
S_c	plume centerline dilution ratio
s	standard deviation of plume cross-sectional concentration distribution
T	tidal period; temperature (degrees C)

T_{90}	time required for 90 percent reduction in concentration (hours)
t	time
U,u	ambient current velocity (m/sec)
u*	shear velocity (m/sec)
V	mean flow velocity in a pipe (m/sec)
W	friction parameter
w	stream width (m)
X	port area ratio
x	distance from shore (m); distance from the offshore end of a diffuser (m); distance downstream from a discharge point (m)
y	water depth (m); height of plume rise (m)
ε	dispersion coefficient
ρ	fluid density (kg/m^3)
ρ_a	ambient fluid density (kg/m^3)
σ_t	measure of density (equal to density in kg/m^3, less one thousand
$\bar{\upsilon}$	stress tensor
$\Delta\sigma_t$, $\Delta\rho$	density difference (kg/m^3)

CHAPTER 5

CONSTRUCTION MATERIAL

5.1 General

The materials needed for an outfall pipeline include those for the pipe coating and lining. Coating and lining materials are discussed in Chapter 9, and pipe materials are discussed in subsequent paragraphs. Outfall materials establish the options of construction methods, mobilization requirements, and resistance to failure during either construction or operation. Their properties are accordingly more significant than their costs. In any event, project costs are site specific and highly dependent upon service period, staging, and demand projections (see Sec. 15.8).

5.2 Pipe Materials

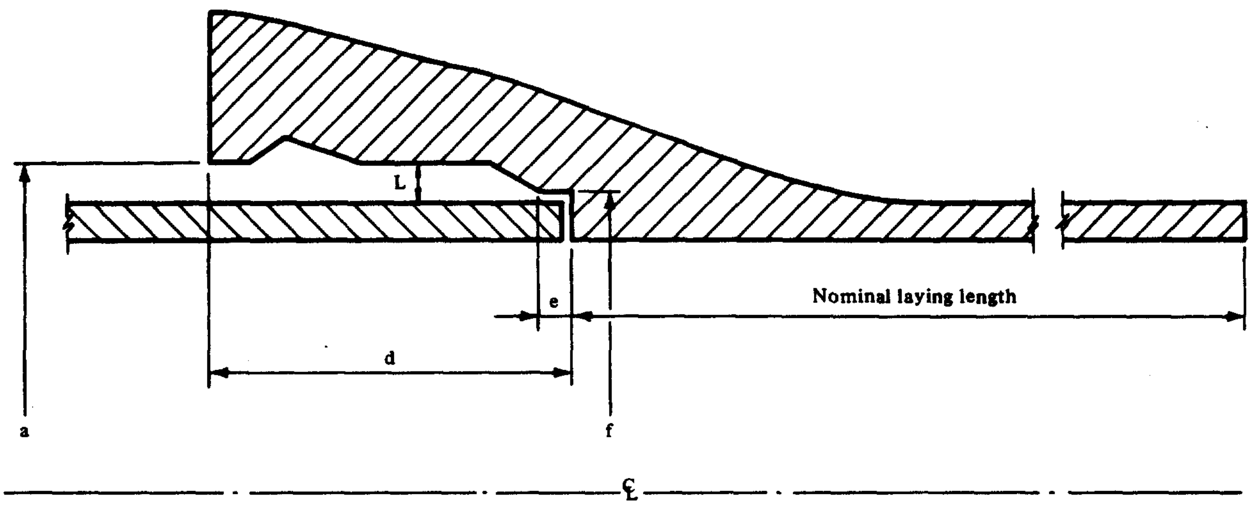
Early outfalls used either cast iron or concrete pipe, and occasionally wrought iron. More recent outfalls have used steel or plastic to reduce material or construction costs. Other industries, such as the gas and oil industry, have used steel pipe exclusively for offshore installations for about a quarter of a century. The materials of construction for pipe discussed in this chapter include cast iron, wrought iron, asbestos bonded corrugated metal, plastic, reinforced concrete, and coated steel.

5.2.1 Cast Iron

Historically, cast iron has been one of the most popular of all pipe materials. Its strength approaches that of steel pipe and it has corrosion-resistant properties that are superior to unprotected steel pipe. However, it has much poorer flexibility and impact resistance and it is more expensive than other types of pipe. It is seldom used as an outfall pipe material today. Cast iron, manufactured in accordance with AWWA Standard C106-75, is available in diameters through 54 in (135 cm). Standard lengths vary depending upon the process used to make the pipe and the pipe end conditions specified (i.e., flanged or otherwise); however, 20-ft (6-m) lengths are most common. Joint design is of critical importance to the integrity, stability, efficiency, and useful life of the pipe. Joints must provide a tight seal; they should prevent pullout; and in many instances they should be flexible. Flanged, bell-and-spigot, and ball-and-socket joints are the most common (see Figure 5-1).

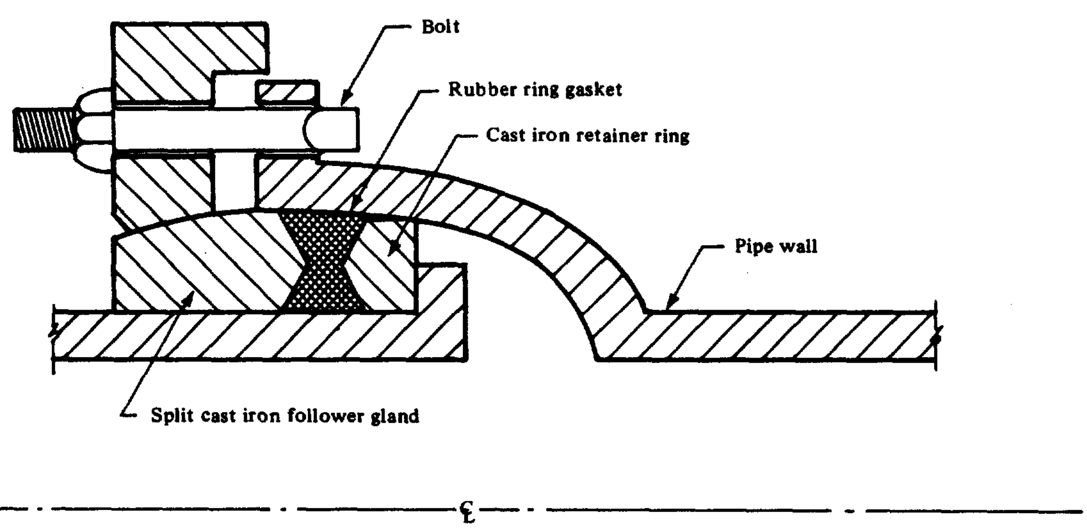
Older outfalls used bell-and-spigot joints, but these are less suitable for offshore outfall pipelines because they require accurate line and grade, and in many cases the driving of pile bents. More recent pipelines use ball-and-socket joints, which permit up to 15 degree seal-tight deflections between adjacent lengths of pipe, without shear stress on the bolts. Flexibility of the joints compensates for soil settlement and minor undercutting and will allow the pipe to follow some variations in topography.

Fig. 5-1. Standard Pipeline Joint Connections



Where a = socket diameter, L = joint thickness, d = socket depth, e = centering shoulder depth, and f = centering shoulder.

Bell-and-Spigot Joint



Ball-and-Socket Joint

The relatively weak joint designs and short manufactured lengths limit the installation of cast iron pipe to the joint-by-joint method where each joint is connected on the seabed. This method of construction makes cast iron outfalls an expensive option.

5.2.2 Wrought Iron

Wrought iron pipe has been used for several ocean outfalls on the United States West Coast. Welded joints and mechanical couplings have been used. It lends itself to construction by either the short or long length methods but it is relatively expensive to purchase. Manufacturers claim that its corrosion-resistant properties are superior to those of cast iron, particularly in saltwater environments. The maximum diameter currently available in the United States is 105 cm (42 in).

5.2.3 Asbestos Bonded Corrugated Metal

Asbestos bonded corrugated pipe has been used for many land sewage installations over the past 35 years and has provided excellent service life. Recent improvements include a smooth interior with the same or lower friction characteristics as other smooth-walled pipes. The pipe is relatively inexpensive, requires a firm bed and good grade, and may be suitable for short length construction methods.

5.2.4 Plastic

Recent developments in the technology of plastics have made possible the manufacture of some plastic pipe in diameters up to 144 in (365 cm) and larger on special request. However, for outfall lines the practical upper diameter limits are lower because acceptable joint connection techniques are not currently available. Smaller diameters may be formed and pulled into place from an extrusion machine on the beach. There are two main groupings of pipe in this category:

Thermosetting Resin - commonly called FRP or fiberglass pipe. The resin used for this plastic is a thermosetting type that maintains its strength and shape through its range of temperature. The resin is reinforced with glass fiber either by filament winding, contact hand-lay-up molding, or casting. This pipe is available in diameters up to 365 cm (144 in) with pressure capability up to 105 kg/cm^2 (150 psi). The temperature limitation is 200` F. It has a specific gravity of about 1.73 and has been pulled from shore. Polyvinyl chloride pipe (PVC) is available in diameters up to 30 cm (12 in). It has a specific gravity of 1.38.

Thermoplastics - made from plastic resins that are heated to a critical temperature and extruded. The pressure-temperature capabilities vary according to the type of plastic. Being more flexible than FRP or PVC these pipes have a potential advantage in being able to respond to wave and current forces and to resulting scour without failure.

Characteristics of plastic materials for outfall pipes are listed in Table 5-1.

TABLE 5-1

Plastic Pipe Characteristics

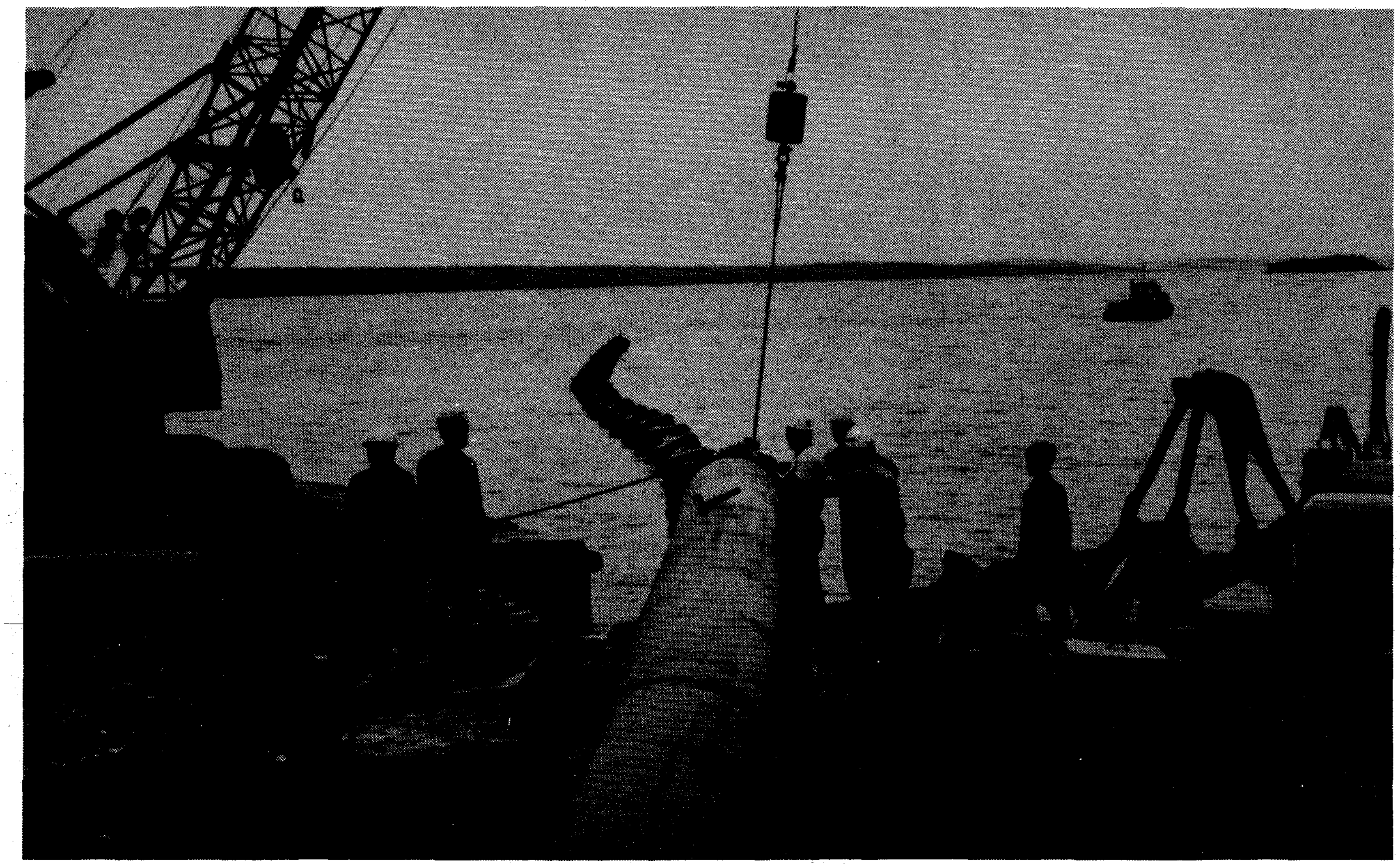
Plastic Material	Maximum Diameter for Acceptable Joints, inches (centimeters)	Standard Lengths, feet (meters)	Specific Gravity
Fiberglass Reinforced Epoxy (FRP)	36 (90)	various	1.73
High Density Polythylene (HDPE)	48 (120)	40 (12)	0.95
Polyproplyene	48 (120)	various	-
Polybutylene (PB)	24 (60)	various	-
Polyvinyl Chloride (PVC)	24 (60)	30 (9)	1.38

A principle advantage of plastic pipe is its excellent corrosion resistance. Thermoplastic pipe should not be used for hot effluents since heat weakens the mechanical properties. Initial costs of plastic pipe are competitive with coated steel; it is more expensive than RCP, and less expensive than cast or wrought iron pipe. Thermoplastic has a specific gravity of about 0.925-0.955 and floats, even when filled with water. Its lightness facilitates handling and may reduce costs of installation in areas where its lower strength and lighter weight are not a determent. It is not ordinarily suitable in areas of high currents.

Plastic pipe is almost always installed by the surface pull method. It is jointed on shore; weights are added as it is pulled into place, and sunk by controlled flooding into the empty pipe. Addition of weights or anchors may be required for on-bottom stability.

The joints are critical factors in plastic pipelines. HDPE pipe sections used for outfalls are usually joined by butt-fusion. Here, the ends of the two pipe sections are squarely machined by a special planing tool, softened by heating, and pressed together to form a butt-fusion joint. Curing of the joint consists of cooling, which is completed in a matter of minutes. Mechanical joints, such as steel or alloy flanges, can be fitted onto special molded pieces of HDPE pipe. Figure 5-2 shows a 24-in (60-cm) diameter HDPE outfall line being towed onto the water. The pipeline was formed by butt fusing 40-ft (12-m) lengths of HDPE pipe. A representative pipe joint is seen in the foreground.

Fig. 5-2. High-Density Polyethylene Outfall Line Being Towed



Polyethylene is available in three density grades: high (HDPE), medium (MDPE), and low (LDPE) density. Only the high density is discussed here; however, the general methods described also apply to the lower densities.

Polybutylene utilizes the same type of butt joints as HDPE but they require 24 hours to 7 days' curing time for their chemical bonding rather than a fusion or heat bond. The pipe is very similar to HDPE pipe within a temperature range up to 180° F.

Fiberglass-reinforced pipe (FRP) joint connections are usually the built-up type. Pipe joint ends are butted, the irregular exterior pipe surfaces are ground down, and a coupling is fabricated in place with alternate layers of epoxy and fiberglass. Because this is a slow process that cannot be done underwater, the use of FRP for outfalls is limited. FRP sizes up to 16 in (41 cm) can be purchased with bell-and-spigot connections and O-ring seals. Also, flanges can be molded to FRP pipe ends as connectors.

Polyvinyl chloride pipe joints are connected with flanges, bell-and-spigot, or slip-on cemented connectors. It is primarily a small-diameter pipe available for pressures up to 150 psi (105 kg/cm^2) and temperatures under 150° F. Because it is available only in smaller sizes and is reported to be susceptible to attack by marine life, PVC is less often used for marine outfalls.

Specifications for plastic pipe are published by the American Society of Testing and Materials (ASTM), the Plastic Pipe Institute (PPI), American Water Works Association (AWWA), National Sanitation Foundation (NSF), Canadian Government Standard Board (CGSB), and others. Plastic pipe has many positive features, but it has not been used long enough to adequately judge its performance. A point of concern with FRP, for example, is delamination of the resin and fiber layers. In addition, experience with FRP marine outfalls in British Columbia has shown that these pipes can suffer rapid wear from abrasion on the sea bottom if they are not properly bedded and anchored. Rock backfill has also been known to puncture FRP.

5.2.5 Reinforced Concrete

Most submarine outfalls greater than 84 in (215 cm) in diameter have been built of reinforced concrete pipe (RCP). The special care required to make underwater joints has led to the use of cast iron or wrought iron in the past and coated steel for smaller outfalls. Even though concrete pipe materials are less expensive, installation costs can be quite high. Concrete pipe is highly resistant to corrosion and to attack by seawater or marine organisms.

RCP is commercially available for submarine service in sizes of 24 in (60 cm) to 156 in (400 cm) and weights of 7.38 to 340 lb/ft (11 to 506 kg/m), respectively. In the United States it is manufactured in accordance with AWWA specifications C300, C301, and C302 (latest revisions). RCP is available in standard 16-ft (5-m) lengths. Pipe 36 in (85 cm) diameter and greater is available in 32-ft (10-m) lengths. For larger, say 114-in (365-cm) diameter concrete outfalls, sections up to 32 ft (10 m) in length can be specified. RCP operating under relatively high pressures is manufactured with a watertight flexible-expansion joint. The joints are the bell-and-spigot type, with the joint surfaces formed by steel rings in the ends of the pipe. Pipe operating at moderate pressures requires a rubber gasket to make the joint watertight. The gasket is placed in a rectangular groove on the spigot end and as the pipes are pushed together, this gasket is compressed into the groove by the flared portion of the bell. With the pipes pushed into position, the gasket is confined on all sides. Figure 5-3 illustrates reinforced concrete pipe sections being lowered into position.

A double rubber-gasket joint is sometimes used with certain types of RCP under relatively low pressure. In this case, the pipe has a spigot on each end. The rubber gasket is placed in the spigot groove, and a double bell-ring of steel or reinforced plastic is placed over the end of the pipe. Half the double bell-ring extends beyond the end of one pipe.

Long-radius curves can be made with joint openings on straight pipe or with beveled-end pipe or bevel adapters. Elbows for short-radius deflections, reducers, tees, wyes, and closures are standard items. Special fittings tailored to suit specific requirements can be supplied or cast in place.

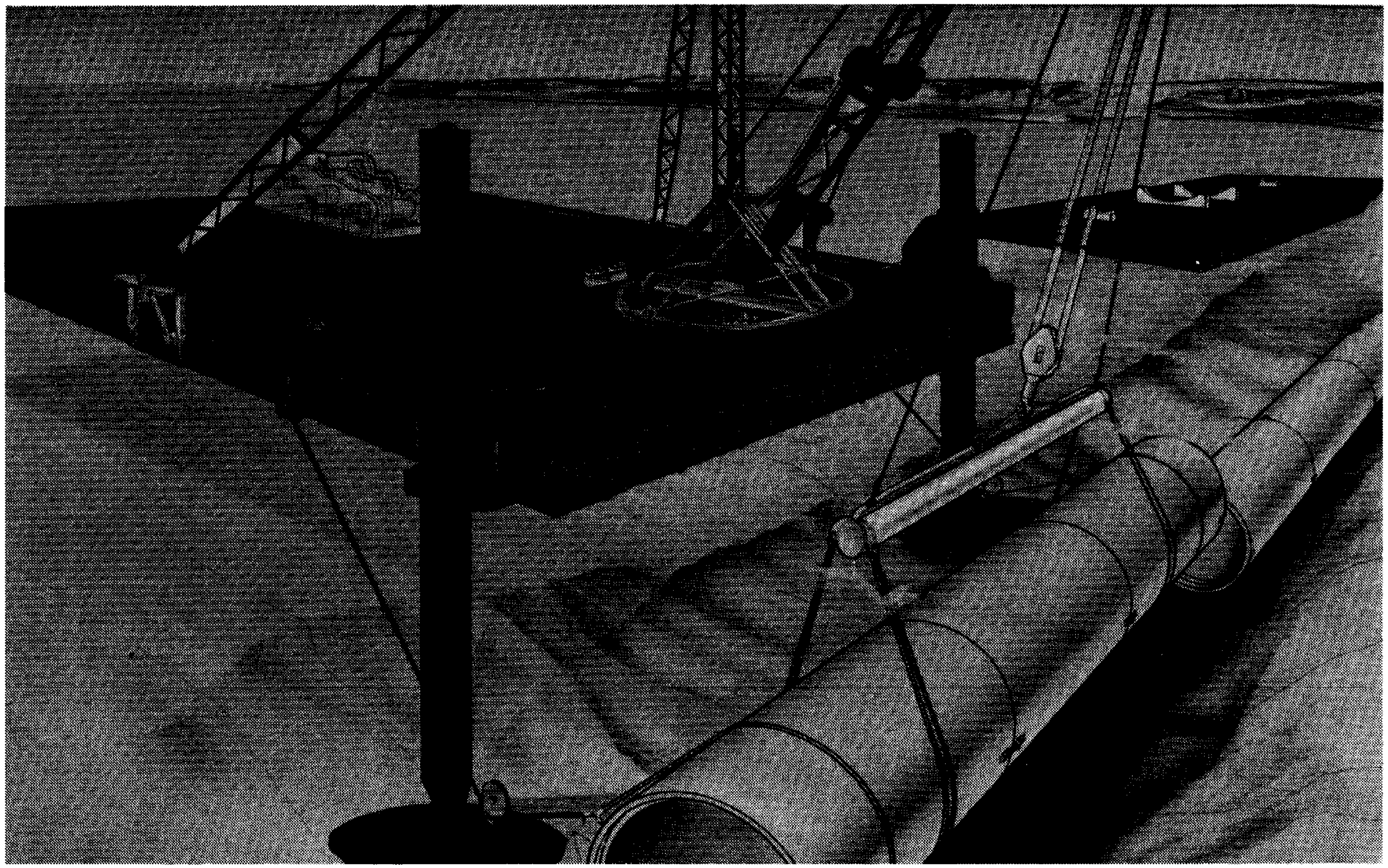
5.2.6 Coated Steel

Steel pipe has been used extensively by the oil and gas industry for subaqueous work because it allows a great degree of flexibility in construction techniques. Steel pipe has also been used in numerous outfalls. Steel pipes are generally lighter, less expensive, easier to transport, and easier to construct than cast iron pipe. Steel pipe is available in standard diameters up to 92 in (235 cm). Larger-diameter spirally welded steel pipe can be obtained. Standard lengths of the steel pipe sizes range from 20 to 40 ft (6-12 m). Steel pipes are manufactured in accordance with AWWA Specification C200-75 with plate steel, which conforms to API standards.

Steel pipes for marine use are usually joined by welding, although flanged connections are sometimes used at the ends of an outfall to permit extension of the line or to add a diffuser at a later time. Welded and flanged connections are illustrated in Figure 5-4. Welded joints are usually checked by X-ray radiography.

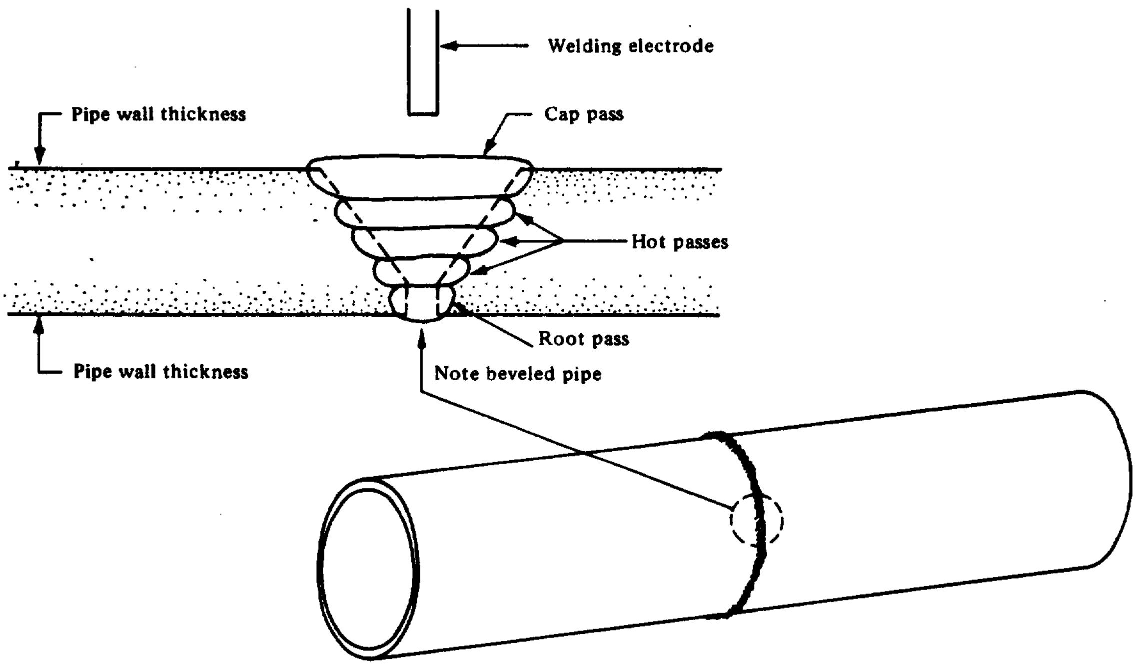
Steel is susceptible to corrosion in seawater. However, experience in the oil and gas industry and the water supply industry has proved that a well-coated pipeline with cathodic protection has a reasonably long life (Chapter 9). The initial costs of steel pipe and cathodic protection may be higher than for concrete or plastic pipe. The advantages of steel pipe include its adaptability to rapid fabrication and installation, its joint tightness, inherent structural integrity, and its higher head capacity. This ability to operate at higher pressures allows the flexibility to convert from

Fig. 5-3. Reinforced Concrete Pipe Being Laid

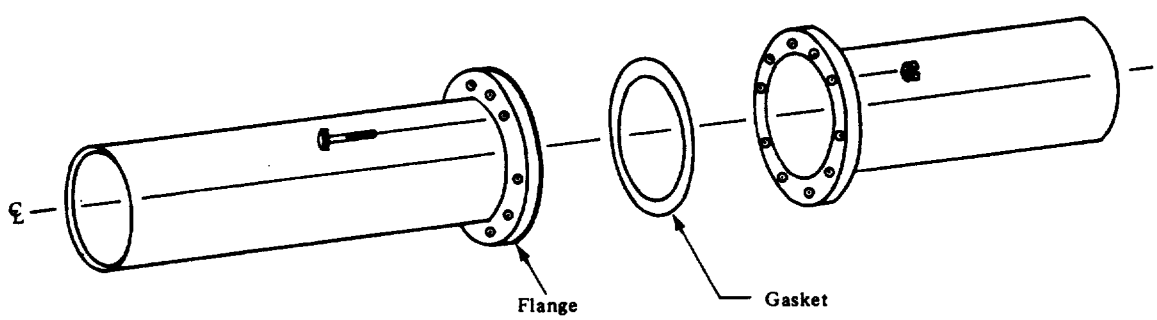


Source: Flexifloat Systems Catalog,
Robishaw Engineering, Inc.,
Houston, Texas.

Fig. 5-4. Steel Pipeline Joint Connections



Welded Joint



Flanged Joint

a gravity outfall to a pumped or pressured outfall in the future. Initial costs of purchase and installation of a smaller diameter outfall operating under pressure are lower than for a gravity outfall. This advantage must be balanced against the higher cost of discharge pumps, increased operating costs, and the cost of providing overflow facilities in the case of power failures or pump malfunctions.

CHAPTER 6

ON-BOTTOM STABILITY

6.1 General

On-bottom stability is one of the most critical elements in the design of outfall pipelines. Designers are concerned with vertical and horizontal stability and with special conditions of the outfall pipeline during its installation and operation. This applies to both buried and unburied marine pipelines or pipeline segments. The line will generally be empty during installation and filled with water during operation. The pipeline weight in both conditions must be calculated and then used in various stability calculations. In general, outfall pipelines are buried near the shoreline and unburied at the effluent end. Therefore, both buried and unburied stability must be calculated.

6.2 Forces

The forces that act on outfall pipelines depend on the method of construction, geological conditions, and oceanographical conditions. These forces can be classified into soil (or geologic) forces, hydrodynamic forces, and installation forces.

Soil forces include frictional resistance, bearing capacity, soil liquefaction potential, and the potential for slumping or sliding. The last element is characteristic of delta areas where mud slides are frequent. Most outfalls are in locations where the other factors are dominant.

Buried pipelines are not directly subjected to hydrodynamic forces. However, the effluent ends of almost all outfalls are unburied and must be designed to resist oceanographical forces.

Six major forces act on an unburied underwater pipeline resting on a seabed: submerged weight, buoyancy, lift, drag, frictional resistance, and inertia.

Weight is the only force that can be controlled without changing the circular shape of the pipe. The weight may be varied by changing wall thickness of the pipe, by changing the material or thickness of the coating, or by adding auxiliary weights.

The buoyancy of a pipeline equals the weight of fluid it displaces. The fluid may be freshwater, seawater, silt, or in some instances, liquefied soil.

When currents pass over an outfall, they create an area of low pressure on the downstream side. In turn, this pressure difference creates a lift force normal to the current.

Drag is due to the vector sum of current and wave forces acting perpendicularly (normal) to the pipeline. The resistance to horizontal movement is a function of the weight and coefficient of friction of the supporting material. It varies with the composition of the supporting material, which varies from place to place. If the pipeline is partially embedded, frictional resistance is increased.

Inertial forces are caused by wave-induced oscillatory water movement on the bottom. They are modified by the pipeline diameter above the bottom. Inertial force exceeds the drag force when there are large accelerations owing to heavy swell and surf or to tidal waves. Acceleration due to surface waves decreases exponentially as water depth increases and is most important in water less than 30 m (100 ft) deep.

The vector sum of the above forces determines whether the pipeline is stable. This procedure is strictly valid for the inertial force only when the acceleration is constant. It provides an approximate solution with nonconstant accelerations; no better method is available at present.

Another effect of oceanographical forces is vortex shedding, when currents force water around both sides of a suspended pipeline; this is discussed more fully in Chapter 7.

6.3 Vertical Stability of Unburied Outfall Pipelines

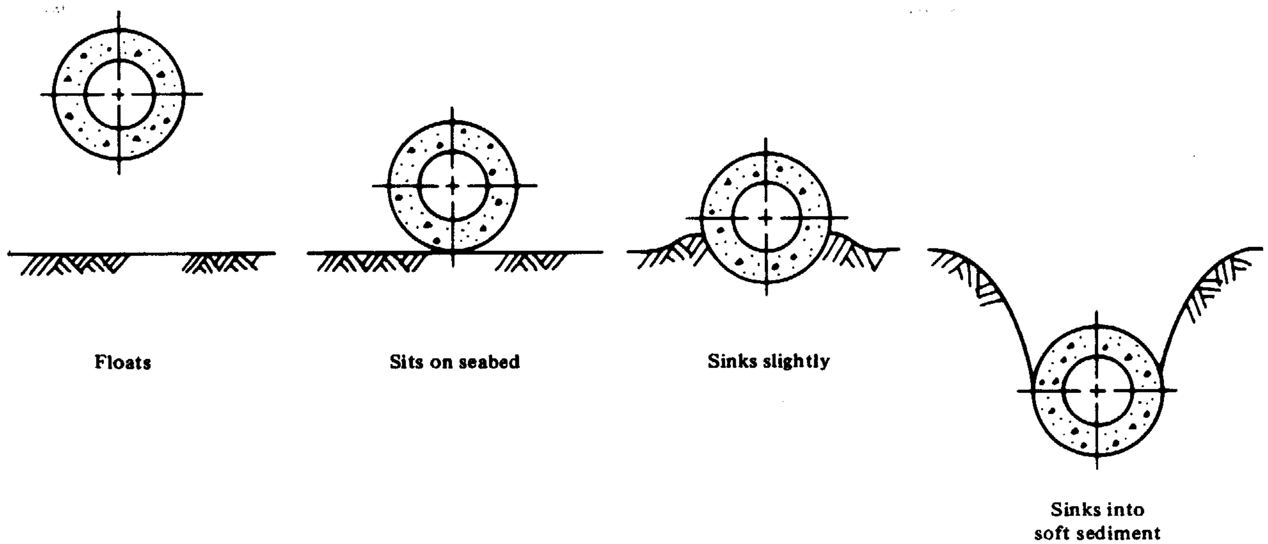
When an outfall pipeline is laid on the seabed it will: (1) float to the surface because of buoyancy; (2) lie on top of the seabed; (3) sink slightly into the bottom until the bearing area of the pipe is sufficient to provide support, or (4) sink substantially into the soft bottom sediment. Figure 6-1 illustrates these conditions.

Vertical instability of an unburied outfall pipeline causes it to float to the water surface or to sink substantially into soft bottom sediment and form a catenary that exceeds design stress limits. Vertical stability of concrete-coated steel pipe is assured by controlling the total weight to within ±7.5 kg/m (±5 lb/ft).

A pipeline may be filled with air to make it lighter and easier to handle in the water while it is being installed. In one case its weight, which was to provide a minimum negative buoyancy of 12 to 18 kg/cm (8 to 12 lb/ft), during installation was specified. However, it is very difficult to maintain pipe joint weights within this tolerance. As a result, the pipeline weight was less than the design weight, the line floated to the surface, was caught in cross currents, buckled, and broke into pieces that either sank or were washed up on the beach. It was not economical to recover and repair the pipe. As a result there were delays, high cost overruns, and legal actions brought by the principals (owner, constructor, and engineer) against each other.

The most common design consideration is the sinking of the pipeline into the bottom sediments. If a short segment of the outfall settles or "sags," a low spot is formed where solids tend to settle and reduce the flow

Fig. 6-1. Possible Results of Laying an Outfall on the Seabed



capacity. If the effluent end of an outfall sinks into the bottom it is again probable that the flow capacity will be reduced, either because the pipe is blocked with sediment or is bent or broken. Soil survey data are used to predict the depths to which a pipeline will sink into the seabed.

If the seabed is mostly sand, adequate bearing capacity may be assumed. Sand will normally support the anticipated loads except in areas where it may become "quick" or liquefied by an upward flow of water, seismic shock, or wave action.

Most of the methods used to analyze the pipeline settling problem are based on indefinitely long strip footing theories. Two commonly used methods for determining the bearing capacity of soils are contained in the works of Karl Terzaghi (4) and L. Reese and A. Casbarian (3). Figure 6-2 presents the results of analyzing Terzaghi's method, where B is the buoyancy, R is bearing capacity of the soil, D is the outside diameter, and W is its weight. This figure can be used as a quick reference to decide whether a more detailed analysis of seabed bearing capacity is required. To use the figure one must know the pipeline's external diameter (including coating), the pipeline specific gravity (in-water weight divided by weight of water displaced) and the cohesive shear strength (C) of the seabed soils. If the intersection of the pipeline specific gravity and pipeline diameter falls well below the applicable C curve, there is little need to make an in-depth analysis of the soil bearing capacity. If the intersection falls near the C curve or above it, an in-depth analysis must be made.

Another method of estimating the bearing capacity of cohesive soils is based on Reese and Casbarian (3):

(6.1) $$Q_u = kcd$$

Where:

Q_u = Ultimate soil bearing capacity per unit length of pipeline

c = Soil cohesive shear strength

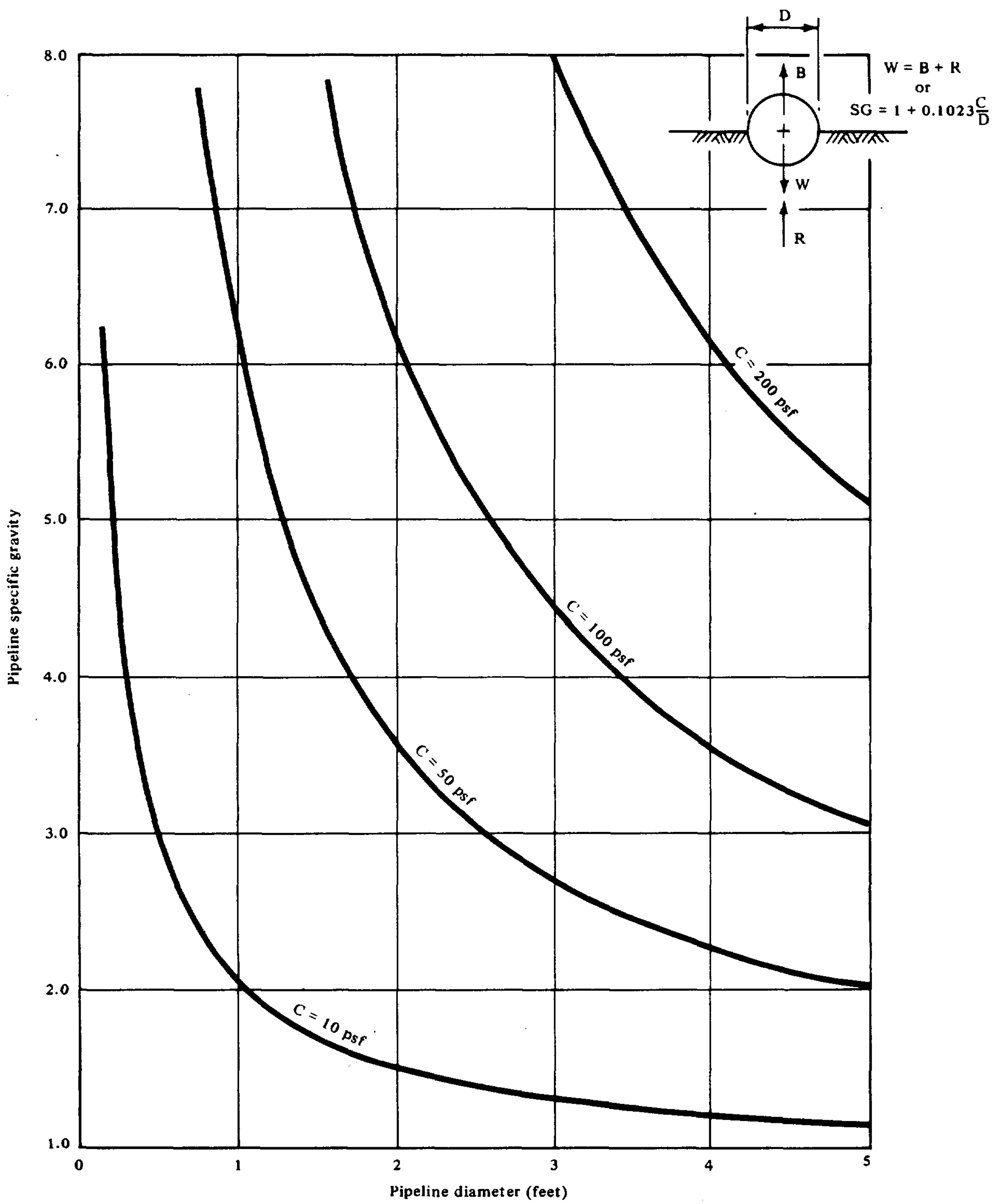
d = Outside diameter of pipeline

k = Bearing capacity coeffient

Suggested values of the variable k are a linear function between 5.7 at the soil/water interface and 11.0 at a depth of four times the pipe diameter. Below this depth, k is assumed to be constant at 11.0.

When calculating the pipeline weight that the soil must support, the designer must remember that the maximum weight is usually after installation when the line is filled with water.

Fig. 6-2. Specific Gravity to Cause Bearing Capacity Failure of an Outfall



Note: C = Cohesive strength.

The vertical stability of an outfall line also depends upon currents flowing across or beneath the line (Figure 6-3). Scour is difficult to predict accurately. It may be minimized by armoring the pipe with coarse stone on a triangular cross-section. Anchoring with large saddle-type weights or collar weights tends to increase scour between anchors.

In soft sediments it is advisable to provide some antiscour measures at the pipeline's effluent end. This can be as simple as digging a large hole at the terminus and filling it with stone 0.3 to 1 m (1 to 3 ft) in diameter. It should extend at least one pipe diameter on each side of the pipeline. If an annual or semiannual inspection detects scour or settlement, the problem can be solved by installing heavy-duty inflatable plastic liners under the pipe span and inflated with mortar until the pipeline is supported, or by sandbags or fabricated supports placed by divers.

6.4 Vertical Stability of Buried Outfall Pipelines

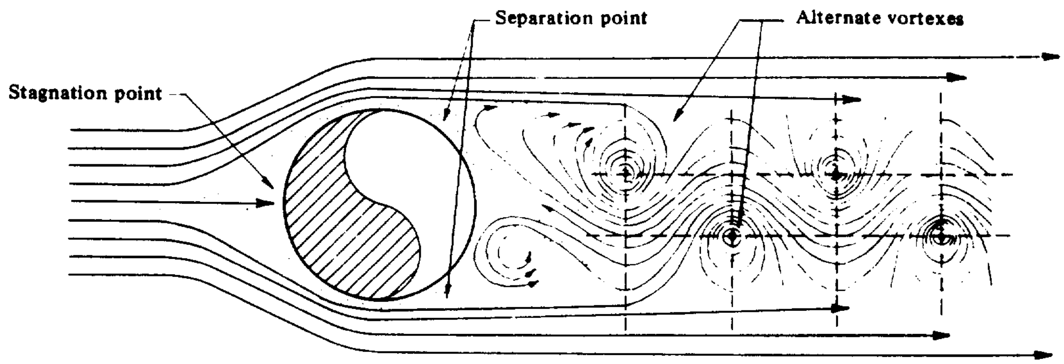
A buried pipeline becomes unstable when it: (1) floats to the soil-water interface and is exposed to wave and current forces that it was not designed to resist, or (2) continues to sink into the trench bottom owing to soil bearing failure.

It is not uncommon to find buried pipelines "floating" out of their trenches. When outfall pipelines are to be buried, designers tend to select a pipe material that is as light as possible. The assumption is that the overburden or backfill in the trench will add sufficient weight to keep the pipeline buried. Whether this happens depends on the soil conditions, backfill material, and method of burial. For example, some designers choose a pipeline weight that will allow the line to be floated into place and filled with water so that it can be sunk in marshes or periodically flooded areas. Often the line is covered or backfilled with the same material excavated from the trench. Generally it is a heavy (specific gravity approximately 1.3) liquid-mud with little or no shear strength because it has been handled twice and replaced in a flooded ditch. Alternatively, the trench may not have been backfilled under the assumption that natural sediment will fill the trench. If the buried line is lighter than the liquid-mud, it will float to the interface. Note that delayed liquefaction may be due to shock or agitation from earthquakes or unusually heavy surf. Pipelines that are light enough to be floated into place should be analyzed for these conditions.

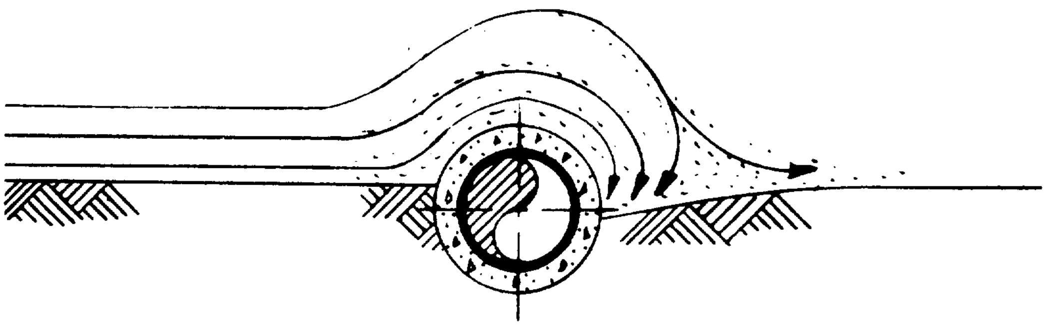
All trenching methods that produce slurries contribute to vertical instability, particularly the "jet-sled" method of trenching offshore pipelines that uses high volume water jets to loosen and liquefy soils underneath the pipeline. This liquefied soil is removed from underneath the pipe with short eductors or air lifts on the jet-sled and the pipe settles into the trench. However, as the excavated material sinks back into the ditch as a high density slurry, it can displace light pipelines.

In soft clay sediments, the design problem is to select a weight that is heavy enough to remain buried but not so heavy as to cause soil bearing

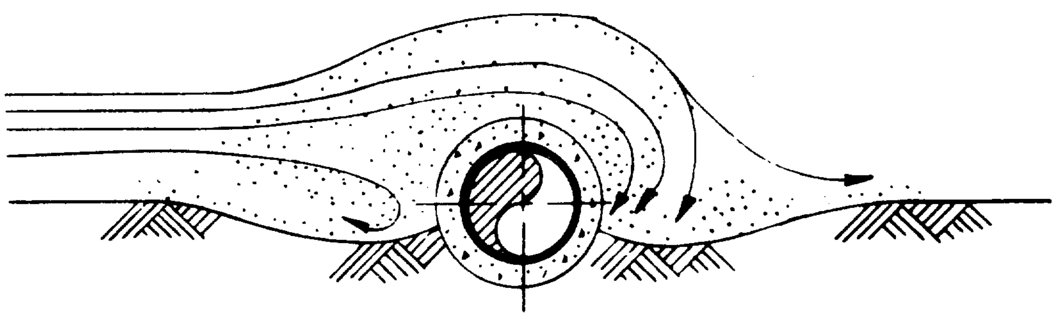
Fig. 6-3. Vortexes and Drag Coefficients for Pipelines



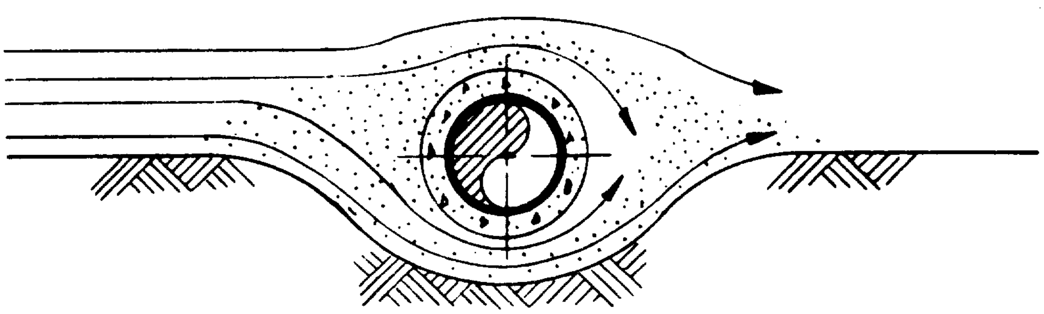
(1) Fully Suspended, (C_D = 1.0)



(2) Partial Scouring (C_D = 0.25)



(3) Partial Scouring (C_D = 0.50)



(4) Complete Scouring of Sediment from under Pipeline (C_D = 1.0)

failure and continued sinking into the trench bottom. The key is to select a pipe weight that will be heavier than the liquefied soil. One approach for clay and silt soils is to determine the liquid limit as outlined in ASTM Specification D423. Here an Atterberg Cup is filled with soil at specified water contents, the sample is divided with a groove, and the cup subjected to a series of blows or taps. The number of blows at which the two portions become one is determined for each water content. The number of blows (usually in the range of 10-50) is plotted and extrapolated as shown in Figure 6-4. Water content percent (%) means the weight (in percent) of the water divided by the weight of the soil in a specific volume.

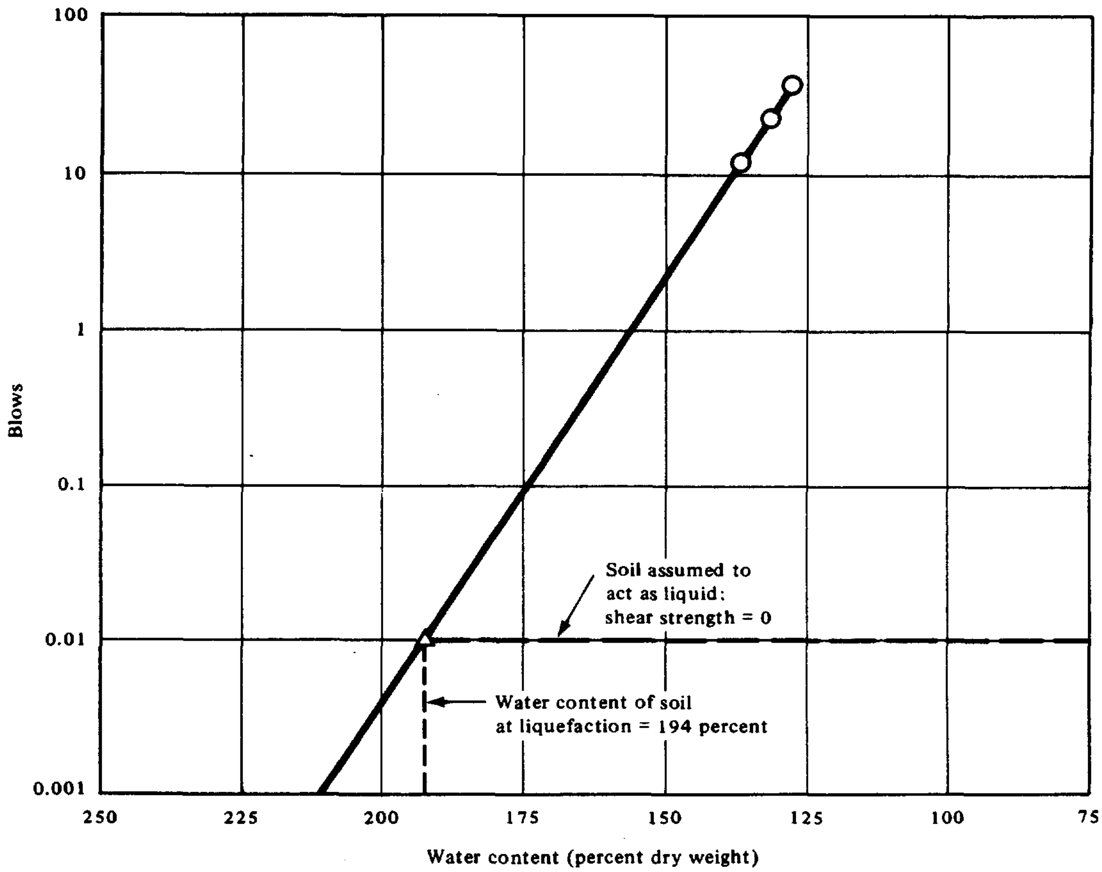
The water content at 0.01 blows is then estimated from Figure 6-4. It has been experimentally determined that at this water content the soil and liquid mixture acts as a viscous, dense fluid that produces maximum buoyancy on the pipeline. A plot of moisture content versus soil density (Figure 6-5) determines the maximum liquefied mud density used in buoyancy calculations. The rationale for this procedure and a list of experimental results are presented in a 1961 ASCE committee report entitled "Rational Design for Pipeline Across Inundated Areas" (1). Nearshore sands present special problems. A striking example of pipeline failure occurred in Australia when a pipeline with a specific gravity of 1.3 was trenched across a sand shore approach and backfilled with sand. After a severe storm, the pipeline was lying exposed on the beach. An analysis indicated that the basic cause was the inertial or cyclic shock force caused by the breaking waves at the shoreline. This force liquefies the soil and imparts a cyclic vertical force to some depth below the seabed. The pipe tends to rise slightly each time a wave breaks and sand particles move under the pipe. Eventually the pipe jacks its way up through the sand. Designers can avoid this by specifying deeper burial, a heavier pipe through the surf zone, or by constructing the shore approach as a groin.

6.5 Lateral Stability of Unburied Outfall Pipelines

When a pipeline is exposed and unanchored on the seabed, it must weigh enough to resist lift, drag, and inertial forces. The wave forces and design current velocities used in the design are based on a statistical analysis of historical weather records and on-site measurements. Two sets of outfall design values are normally selected--one for installation and one for operation. Because installation is of relatively short duration, a set of design values based on wave heights and current velocities with a probability of occurrence of 5% during any one year or season, i.e., a return period of 20 years may be used. Design values for operation are usually based on a 1% probability of occurrence, i.e., 100-year storm, a return period of 100 years.

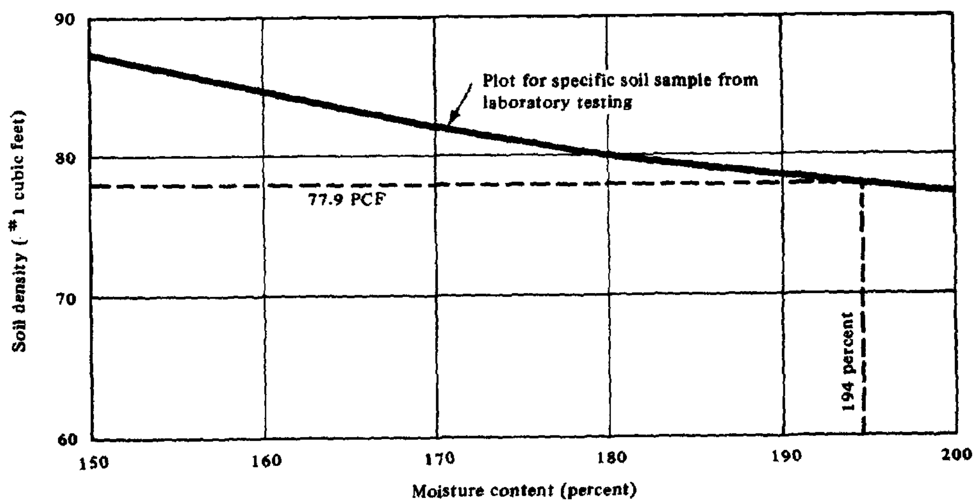
Drag, inertial and frictional forces are of primary concern in horizontal stability calculations. Buoyancy, submerged weight, and lift forces contribute indirectly since the frictional force is generally assumed to be directly proportional to the net downward force. Although this is not precisely true, it is within the accuracy of the other variables.

Fig. 6-4. Typical Liquid Limit Curve (ASTM D 423)
for Determining Water Content



Note: O Laboratory test results; △ point obtained by extrapolation.

Fig. 6-5. Typical Moisture Content versus Soil Density for Submarine Soils



Inertial forces are much less than drag forces for nonbreaking shallow-water waves and of the same order for deep water waves (see Table 2-2 and Figure 2-6). Detailed analyses of these and of breaking wave forces have been found necessary by some designers (I. Wallis, personal communication) and have been published by the U.S. Army Corps of Engineers (4).

For the purposes of this report, inertial forces are neglected and the basic stability equation, if inertia is neglected, in foot-pound-second units, is

$$W/D_c = (C_l + C_d/f)\ (wV^2/2g) \tag{6.2}$$

where: W = Submerged weight of pipeline, pounds per foot

D_c = Outside diameter of pipeline including coating, feet

C_l = Lift coefficient

C_d = Drag coefficient

f = Friction coefficient

w = Water density, pounds per cubic foot (64 pounds per cubic foot for saltwater)

V = Current velocity, feet per second

g = Gravitational acceleration, 32.2 feet per second

The minimum specific gravity required for equilibrium is relatively insensitive to large variations in all of the parameters appearing in the stability analysis except the current velocity. Therefore, a nominal set of numerical values can be chosen for most design conditions.

Coefficients for lift, drag, and friction have been the subject of much research, particularly by the offshore oil and gas pipeline industry. Coefficients have also been presented in many engineering handbooks; however, designers are cautioned that these generally prove inadequate, since many of the handbook coefficients have not been specifically established for the special problem of a pipeline on the sea floor.

Lift and drag coefficients for a submarine pipeline are usually set with respect to a dimensionless parameter called the Reynolds Number. The Reynolds Number (Re) is defined by:

$$Re = Vd/y$$

where V = Current velocity, feet per second d = Outside diameter of pipe and coating, feet

y = Kinematic viscosity of sea water, feet squared per second

Figure 6-6 shows drag and lift coefficients versus Reynolds Number for pipelines lying on the seabed. Conventional values are generally used for outfall design calculations. It is recommended that no values higher than those in Figure 6-6 be used in design, because higher values generate unnecessarily strong stability designs and higher costs.

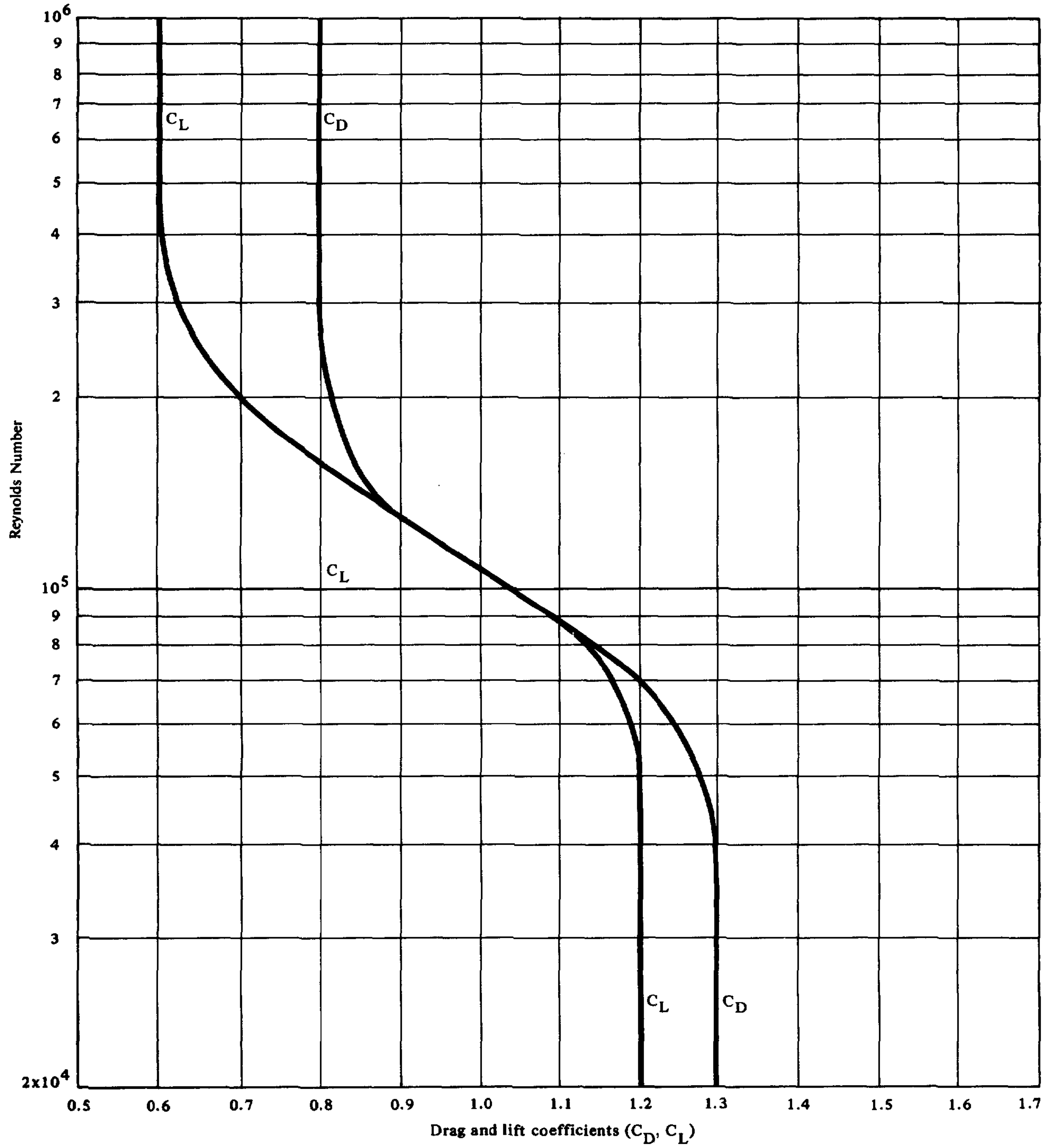
Friction coefficients between pipeline and soil are amply discussed in technical literature. Laboratory values can be used as a guide but they should not be used as an absolute design criteria. It should also be noted that the friction coefficient used for calculating axial pulling forces for the pull method of construction are usually less than the friction coefficients used in calculating lateral stability. Laboratory values vary with soil type. However, this difference may not be reflected in the effective friction coefficient that should be used for on-bottom lateral stability calculations. A friction coefficient of 0.65 to 0.75 is used for sand, depending on the designer's degree of confidence in the sampling and testing procedures. The effective friction coefficient for clay is generally higher than a laboratory value and varies with pipe weight and soil bearing capacity. As the submerged weight of the pipeline increases the pipe tends to settle into the clay, and thus more pipe surface is covered and a soil wedge is formed on each side of the pipe, both of which increase the effective resistance to sliding. Values used for friction coefficients vary between 0.5 and 1.0, depending on soil type and the degree of safety or conservatism desired.

Although most discussions in the literature on lateral stability deal with coefficients, sensitivity analysis of the stability equation reveals that the most critical variable is the current velocity. If the selection is too conservative, very heavy concrete coating weights and unreasonable construction restraints are derived. For example, a recent outfall pipeline design in Western Canada required up to 320 cm (12.6 in) of concrete on a 1067-cm (42-in) diameter pipeline.

6.6 Lateral Stability of Buried Outfall Pipelines

Lateral stability is a problem for buried lines only when the pipeline cover is scoured or removed under normal or storm conditions. High nearshore currents produced by seasonal severe storms can cause large sediment shifts that may uncover pipelines and expose them directly to current and wave forces. Further offshore, inertial forces due to long waves can often cause damage at depths less than 30 m (100 ft) and occasionally at depths to 60 m (200 ft). Either current or wave forces can suspend or liquefy sediments in a trench. Long-term records as well as site-specific data collected over two storm seasons should be used to determine the burial depth required for stability.

Fig. 6-6. Drag and Lift Coefficients for Pipe Laying on an Ocean Floor



Reinforced concrete weight coating or anchors stabilize exposed pipelines. The two basic types of anchors are density anchors and mechanical anchors.

6.7 Density Anchors

Density anchors consist of weight added to the pipeline to increase its average density or negative buoyancy to a level that will be stable under the design criteria. Continuous reinforced concrete weight coatings are normally used. These may be augmented by set on or bolted on reinforced concrete anchors with densities of 2,800-3,500 kg/cm^2 (4,000-5,000 psi). 2,200, 2,600, and 3,000 kg/m^3 (140, 165, and 190 lb/ft^2) are used during engineering analysis and optimization. These densities are obtained by substituting heavy aggregate, such as iron ore, for the conventional aggregate. A maximum thickness of 14 to 15 cm (5-1/2 to 6 in) is recommended because of the danger of the concrete spalling at greater thicknesses.

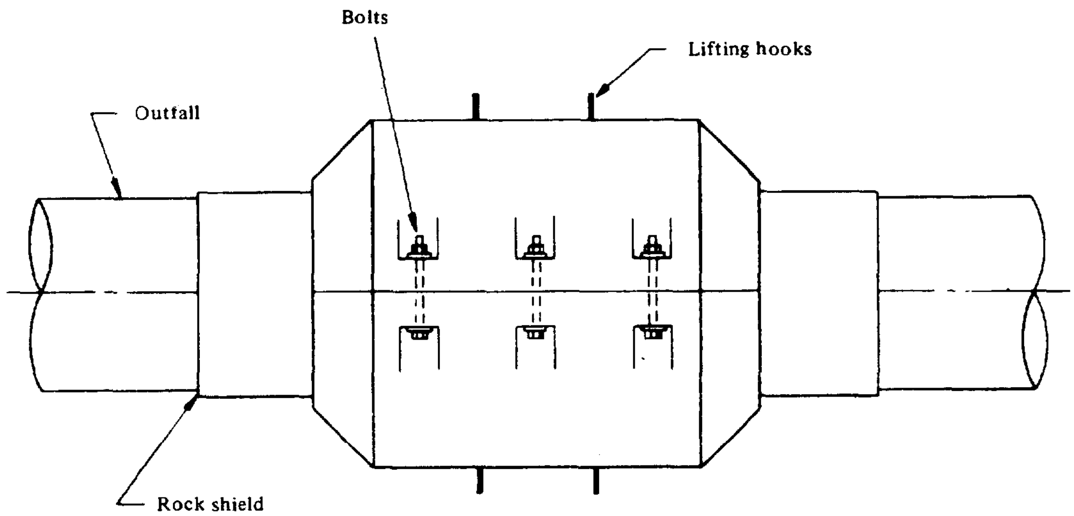
Concrete weight coatings may be applied by machine or a form may be built around the pipe and concrete poured into the form. Forming and pouring are normally used for small quantities of pipe. Forms are normally thin sheet metal. Small spacers are used to hold the form off the pipe the required distance. A slot is cut midway into the spacer and wire mesh [usually 15 x 15 cm (6 x 6 in) 10-gauge reinforcing mesh commonly used in concrete foundations on grade] is fixed into the slot. This holds the reinforcing steel midway between the form and the pipe. Slightly wet concrete is poured through a slot left open at the top of the form. Forms are vibrated to ensure a uniform coating of 2,100 kg/cm^2 (3,000 psi): note that the ultimate strength of concrete is inversely proportional to the amount of water in the mixture. Since the form-and-pour method is used primarily for short outfalls, 2,100 kg/cm^2 (3,000 psi) is normally sufficient.

Machine-applied concrete coating is very dry and can therefore achieve a higher strength. During application the pipe is continuously rotated about its longitudinal axis and moved axially past the applicator. The applicator is either a belt or set of brushes that rotate at a high speed, throwing the concrete onto the rotating pipe, where it is dry enough to stay in place without forming.

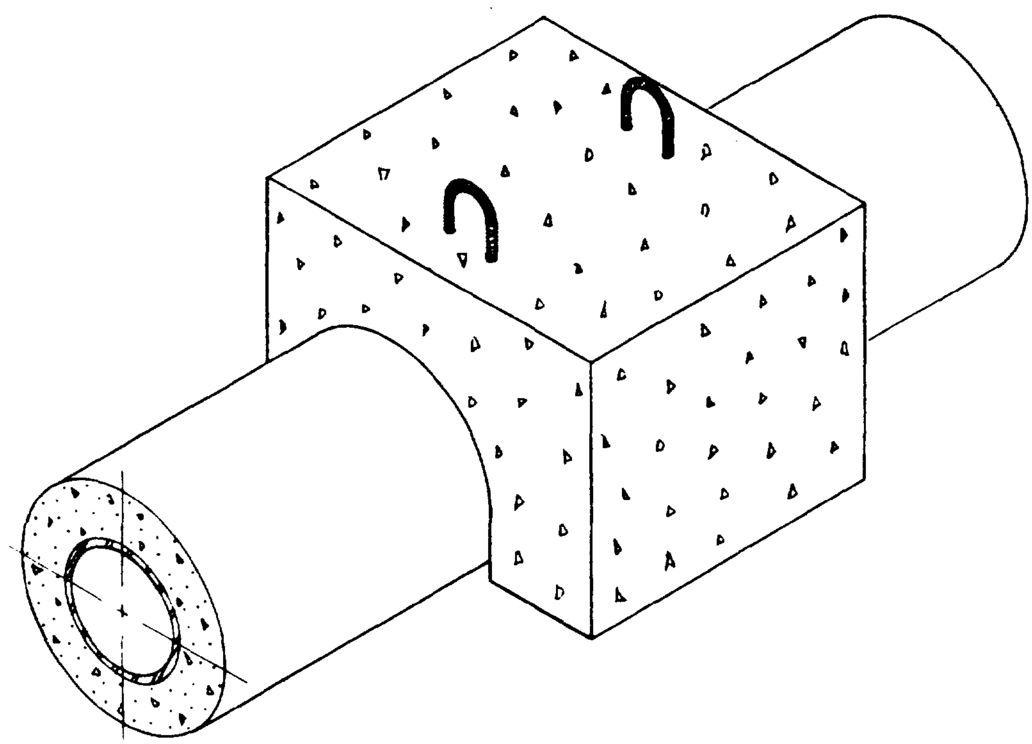
Reinforcing wire mesh is automatically embedded into the concrete during application. This reinforcing mesh is 6 cm (2-1/4 in), 17-gauge wire mesh known as "Chicken Wire." If the concrete coating thickness is more than 6.4 cm (2-1/2 in), two layers of mesh are used.

Bolt-on concrete weights (Figure 6-7) are semicircular sections that are clamped onto the pipeline with long bolts. Bolt-on weights are made by pouring concrete into molds with reinforcing bar in place. Each half has lifting hooks. Rock shield is sometimes attached to the inside of the weights to protect the coating on the pipeline from damage during construction. Bolt-on weights have bevels on each end to prevent snagging on obstacles if they are used on a pull section.

Fig. 6-7. Typical Bolt-on and Set-on Weight Gravity Anchors for an Outfall



Bolt-on Anchor



Set-on Anchor

The least expensive density anchors are U-shaped set-on weights that are set over the pipeline after it is in the trench (Figure 6-7). They should have as low a center of gravity as possible. The legs are 5 to 7.5 cm (2 to 3 in) longer than the pipe diameter to prevent the weight from rolling off the pipe and to enable the ditch bottom to take the load. Large-diameter thin-wall pipelines can be overstressed if the pipeline has to support the set-on weight in addition to resisting the stresses due to pressure and bending. Rock shield is sometimes attached to prevent damage to the pipe coating. Set-on weights are usually poured on the job site. They are reinforced with steel and have lifting hooks. Both the bolt-on and the set-on weights are installed with a side boom, dragline, or other machine capable of lifting the weight.

6.8 Mechanical Anchors

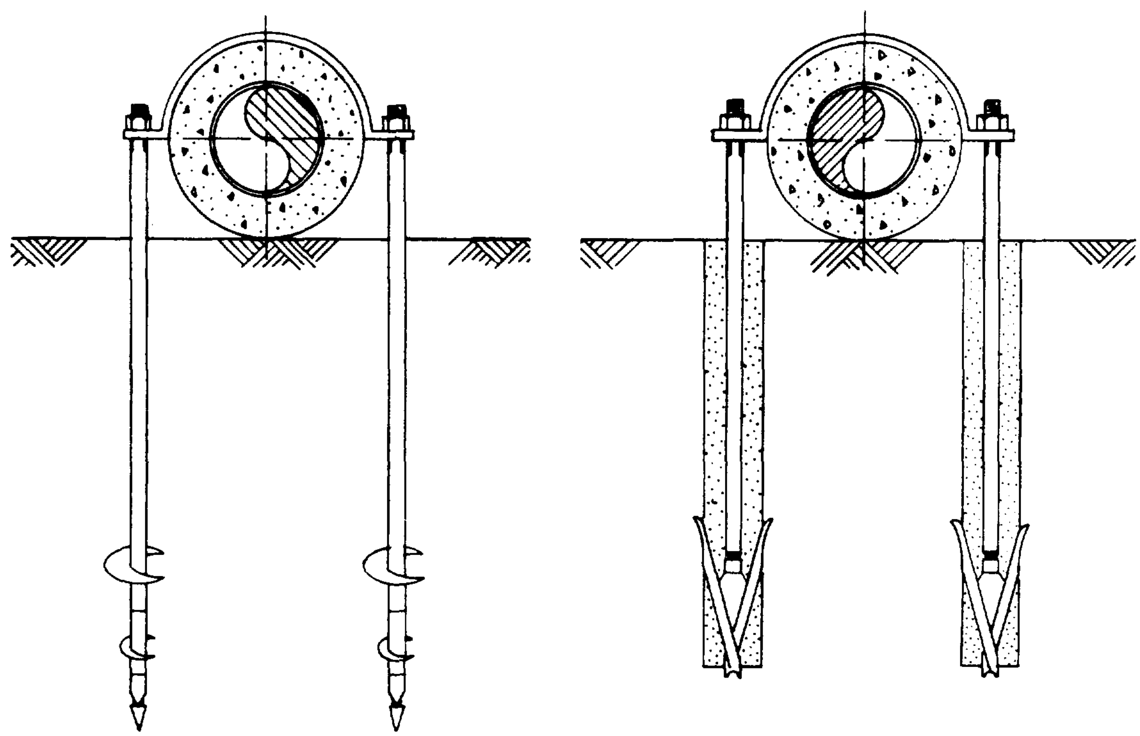
Mechanical anchors are usually steel and are not designed to add weight. They maintain a minimum hold-down force on the pipe when properly installed in the soil. The holding power of mechanical anchors is considered to be more efficient than that for density anchors, particularly on large-diameter lines.

The most commonly used mechanical anchor is the auger-type (Figure 6-8). This anchor consists of a round steel plate shaped like an auger and attached to the end of a long steel rod. The other end of the rod is threaded so that it can be attached to a clamp. This system consists of two augers and strap shaped to fit the pipeline.

Installation consists of installing an anchor on each side of the pipeline and attaching the strap snugly over the pipeline to both anchors. The strap is usually padded to protect the pipe coating. The anchors and strap are hot-dipped galvanized to extend their service life. Small magnesium anodes can be attached to each anchor to provide corrosion protection. In most applications the anchor will last as long as necessary for the backfill to compact and gain sufficient shear strength to hold the pipeline in place.

Expanding mechanical anchors (Figure 6-8) are used in the same manner as the auger type. The anchor rod has hinged flukes on one end that expand outward by turning the threaded anchor rod which is run through a nut in the center of the flukes. The anchor is either expanded in a previously bored hole or forced to the desired depth and expanded. The expanding anchor can be made with tandem flukes so as to increase the effective area where the soil has low shear strength.

Fig. 6-8. Typical Auger-Type and Expanding-Type Mechanical Anchors



Auger-type Anchor

Expanding-type Anchor

6.9 References

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CHAPTER 7

STRESS ANALYSIS

7.1 General

The design of marine outfall pipelines must take into account four structural factors: (1) installation stress, (2) stress induced by anchoring (if applicable), (3) collapse/buckling analysis, and (4) unsupported pipe spans.

7.2 Installation Stress

Installation stress can be analyzed by calculating the estimated magnitude of forces acting on the pipeline induced by the selected construction method and equipment. For instance, if the pipeline is to be pulled into place, the maximum expected pulling force can be divided by the pipe wall thickness area to determine stress levels.

7.3 Stress Induced by Anchoring

When anchors are used to provide on-bottom stability to an outfall pipeline, they become supports for uniformly loaded beams where the loading is the drag force caused by water currents. Figure 7-1 illustrates the principle used.

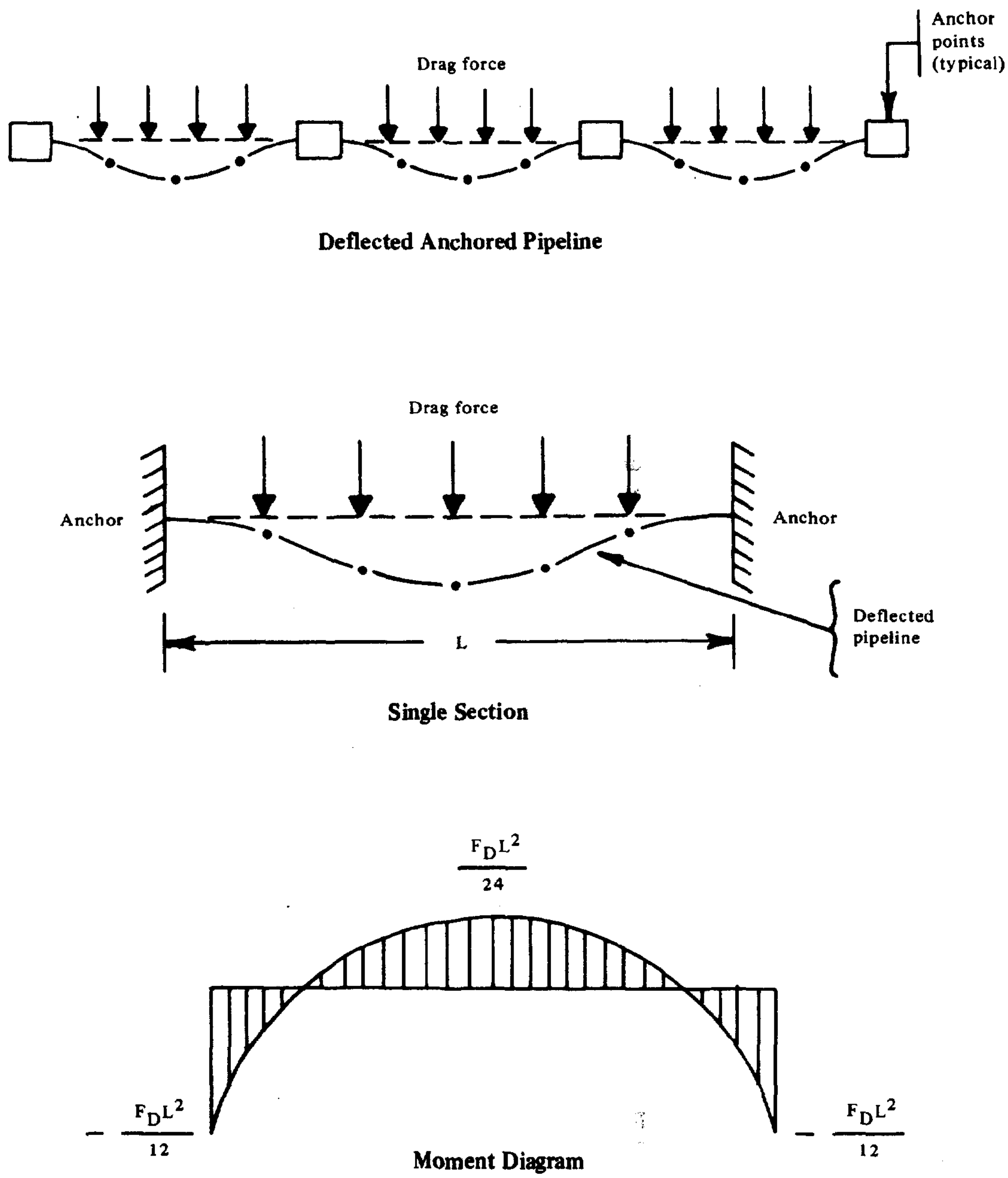
7.4 Collapse/Buckling Analysis

When a marine pipeline fails because of excessive external hydrostatic pressure, the result is defined as collapse. Air-filled, moderate-sized lines installed in deep water or large lines in moderate depths may be subject to collapse. If the pipeline is to be installed by a method that requires the line to be open ended, such as a joint-by-joint bottom assembly method, a collapse analysis is unnecessary, since the external and internal pressure will equalize.

Failure by buckling occurs when there is excess bending of the pipe. Buckling and collapse can occur at the same time under conditions of excess bending and external pressure. In the case of an offshore pipeline, two conditions should be investigated. First, the vertical position (or water depth) of maximum bending during installation should be located and quantified. The combination of maximum bending and external pressure should be checked for buckling failure. A second check should be made for maximum moment near the seabed at maximum water depth. This bending moment value can then be combined with the value due to maximum hydrostatic pressure.

A structure's buckling characteristics depend on both its material properties and its shape. The physical properties of a steel pipe are characterized by the D/t ratio (the outside diameter divided by the wall

Fig. 7-1. Deflection of an Anchored Submarine Pipeline



thickness) and the material properties defined by the stress-strain curve for the material. The elastic limit for buckling in steel pipe occurs at a D/t of approximately 250 or higher, depending on the pipe material. Below this value, buckling occurs in the plastic range of the material and is accompanied by a flattening of the pipe's cross-section. The resulting ovalization lowers the bending resistance by reducing the section modulus of the pipe. For steel pipe with a D/t ratio in excess of approximately 60, buckling occurs before the pipe develops its full resisting moment. For pipe with a D/t ratio less than 60, the resisting moment tends to drop off before buckling. An alternative approach for determining the critical collapse pressure, P_c, is given by Reynolds (1):

(7.1) $$p_c = [2\ E/(1 - R^2)]\ [t^2/D]$$

where E = Young's modulus

R = Poisson's ratio

t = Thickness, and

D = Average diameter

Another type of failure is the propagating buckle in which an air-filled pipeline buckle failure changes its geometry from a transverse dent or crease to a longitudinal collapse, which then propagates along the pipeline. The external pressure required to maintain the propagation of the collapse is less than that required to initiate it. Initiating pressure, propagating pressure, and theoretical collapse pressure should be computed and plotted for different D/t ratios for the pipe material under consideration. Because such equations to analyze collapse and buckling take time, most qualified consultants use computerized solutions. A rule of thumb for conservative design is:

Maximum Water Depth ft (m)	D/t
1000 (300)	25 or less
500 (150)	35
250 (75)	45
125 (38)	55

7.5 Unsupported Span Analysis

When an offshore pipeline is suspended on seabed obstructions or across scour, unsupported pipe spans are exposed to water particules flowing above and below them. Water particles flowing around the pipeline produce vortex shedding. The weight of the unsupported pipe creates bending stresses in the pipe. A sound design of the maximum allowable span of unsupported pipe should therefore include both static and dynamic analysis.

In static pipeline analysis all static forces acting on the pipeline must be considered when calculating the maximum pipe span allowable without exceeding the design stress. Span ends can be assumed to be pinned or fixed, depending on soil characteristics and external supports.

Vortex shedding produced by the water particules flowing around the pipeline induces a large transverse force in the line that causes lateral vibration unless there is adequate restraint. The frequency of the vortex shedding depends on pipe diameter, flow velocity, and a nondimensional parameter called Strouhal number. The Strouhal number can be experimentally determined with respect to the Reynolds number since both are functions of the pipe diameter and the flow velocity (2,3). The Strouhal, S, number may be estimated as:

(7.2) $$S = FD/U$$

where: F = Exciting frequency of vortex shedding, cycles per second

D = Pipe diameter

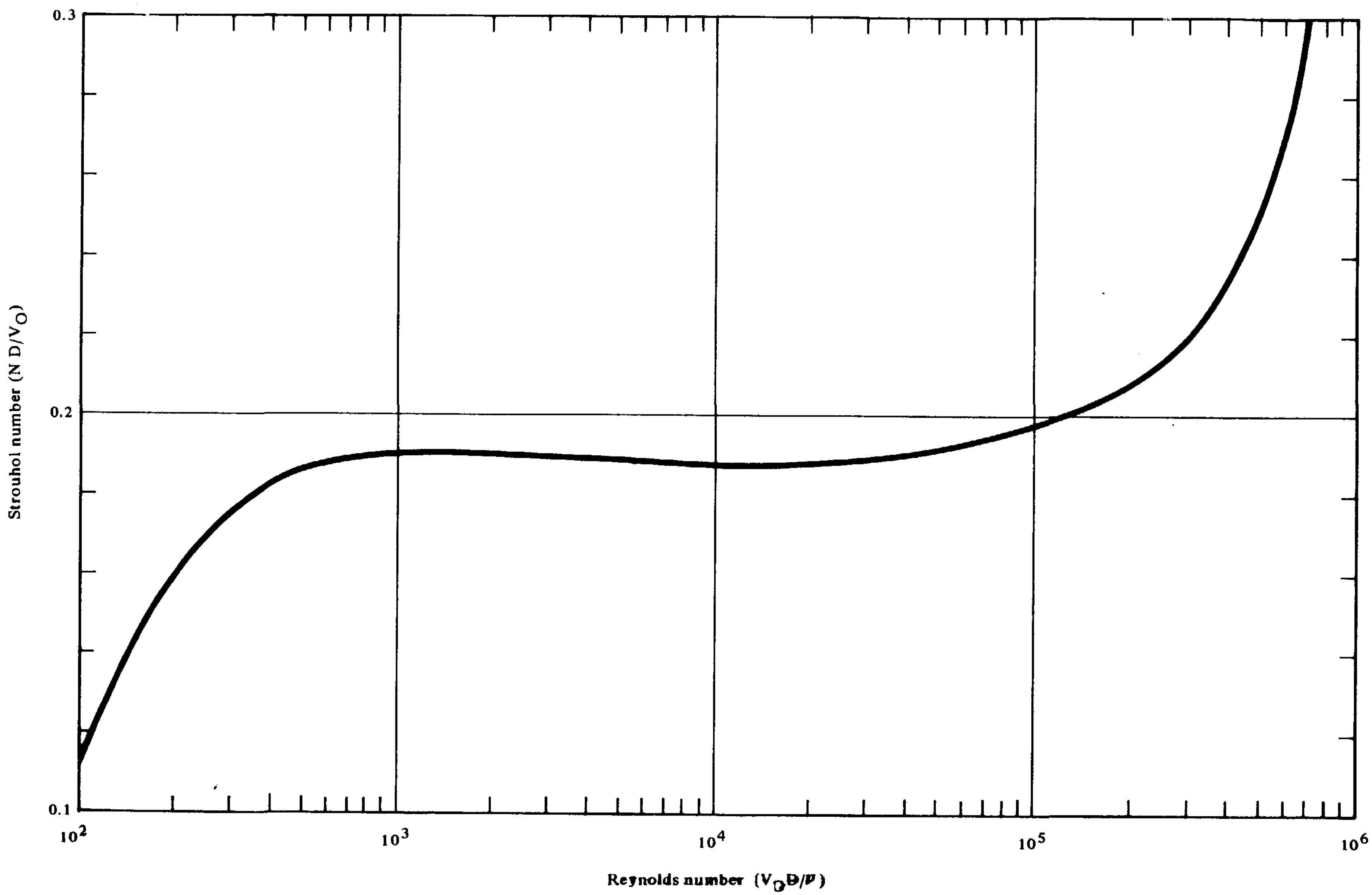
U = Flow velocity

The excitation frequency of vortex shedding can then be determined by calculating the Reynolds number, estimating the Strouhal number from Figure 7-2, and solving the above equation for F.

If the frequency of vortex shedding coincides with a natural mode of oscillation for the pipeline, resonance and amplitude amplification occur. To avoid this, the natural frequency of the unsupported pipe span should be made higher than the highest anticipated vortex shedding frequency. This can be done by increasing the stiffness of end restraints on the pipe, or in the event that resonance develops during construction (see Chapter 11, Section 11.3), the pipe can be lowered quickly so as to shorten the unsupported span.

During installation of an offshore pipeline, the unsupported spans may be reduced by leveling the pipeline route or by providing intermediate supports. Construction specifications should limit the allowable unsupported span length and should specify acceptable methods of correcting excessive pipe spanning. If the span occurs in an area or trench that is to be covered with gravel or rock, or to be backfilled with soil, the supports should be installed before covering. Consideration should be given to the impact load of placing the cover plus the material weight (which could overstress the pipe) across its unsupported length.

Fig. 7-2 Variation of Strouhal Number with Reynolds Number



Source: Steinman (3).

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CHAPTER 8

SHORE APPROACHES

8.1 General

A substantial part of the cost of most outfall pipeline projects lies in the construction of the shore approach. Special attention must be given to protecting the outfall from mechanical damage, hydrodynamic forces, stability in a soil that liquefies in heavy surf, methods of construction, and effects on the usefulness and aesthetics of the beach area or shoreline. These factors are briefly reviewed below. For a current state-of-the-art treatise on coastal engineering, the reader is referred to the Corps of Engineers Shore Protection Manual (1).

8.2 Design Considerations

Governmental regulatory requirements must be considered early in the planning phase. In some parts of the southern United States, for example, all pipelines must be at least 12 ft (4 m) below mean low water level throughout the shallow water shore approach, at the shoreline, and for a minimum distance of 1,000 ft (300 m) on the land side of the shoreline. Designers must also check with local planners and property owners to identify potential impacts of the outfall on future coastal zone development.

Shorelines are not static. They are either receding inland or building seaward, often at accelerated rates because of shoreline construction or changing sediment discharges from drainage basins undergoing development. Seasonal removal of several meters of nearshore sediment during winter storms and its redeposition during spring is common. If a design specifies one meter of cover over the pipeline in such areas, it must also specify the season to which this applies. Estimates of nearshore scour and fill may be obtained from historical charts and records, local pilots and harbor masters, nearby university departments, ocean-oriented government agencies or seasonal bathymetric surveys over two or more winters.

Designers and the constructors must know quantities and types of soil materials to be excavated, particularly rock, before they can select construction methods and equipment. Unfortunately, subbottom data are more difficult to obtain in shallow water than further offshore. Vessels used offshore are seldom suitable for shallow-water work. Shore-based operations using divers are limited in effectiveness because of breaking waves or surf. Nevertheless, it is important to obtain soil samples in the shallow-water zone.

One method of obtaining samples is to load a small (10-20 ton) crane on a flat barge, ground the barge in shallow water, and use it as a work base to either core the soil or dig a test pit. Divers can work in slack tide or calm periods and obtain useful data with a hand-held water jet and a sharp probe.

A sandy shoreline is particularly subject to continuous long- and short-term change. Many pipelines buried across sandy shorelines have gradually become exposed even when soil transport by littoral draft was insignificant because breaking waves liquefied the soil and the pipeline gradually jacked its way through the sand. To combat this phenomenon, some designers increase the empty specific gravity of the pipeline to 2.0. Others bury the line deeper to avoid the zone of soil liquefaction.

Factors that have to be established during the design phase include:

1) The depth of pipeline burial through the shore approach necessary to protect the line from potential shoreline movement and from damages caused by outside sources throughout the design life.

2) The depth of pipeline burial necessary to prevent instability resulting from breaking waves.

3) The weight the pipeline needed for vertical stability, which may be different from the offshore portions.

4) The compatibility of the proposed outfall with future plans for the shoreline.

8.3 Shore Approach Construction Considerations

The construction method (Chapters 11 and 12) used for the shore approach is often different from that for the offshore portion, particularly if the outfall is over 3,300 ft (1,000 m) in length. Predredged trenches are often prepared through the shore approach zone and the line laid or pulled into the trench before it fills with sediment moving with littoral drift. Attempts to lay pipe across the shoreline and bury it later generally do not work well (pipeline trenching and backfilling are discussed in Chapter 12).

Several factors to be weighed during the selection of construction methods include the availability of essential skills in the local labor market, availability of construction equipment from local or distant sources (the latter have greater mobilization costs), available work space on shore, blasting reqirements and effects on nearby structures and residents, effects of weather changes on construction, construction equipment requirements and availability (either locally or mobilized from a remote location), the shoreline condition after construction, access to the onshore work for heavy equipment, adverse reactions from nearby establishments or residents during heavy construction activity, and the increases in water turbidity which may cause local problems.

8.4 References

1. Corps of Engineers. United States Army. 1984. Shore Protection Manual. 2 vols. Waterways Experiment Station, Vicksburg, Miss. 39180.

CHAPTER 9

CORROSION CONTROL

9.1 General

Concrete pipe, cast iron pipe, plastic pipe, and wrought iron pipe are generally considered corrosion resistant. Corrosion is difficult to control, however, on steel outfalls. Corrosion control, in this case, is usually accomplished by a combination of pipe coatings and cathodic protection.

9.2 Corrosion of Steel in Seawater

There are two types of corrosion of steel outfalls--oxidation and galvanic corrosion. Oxidation forms rust from the steel. Galvanic corrosion occurs when a small electric current flows between dissimilar metals or between local cells on the same metal which have dissimilar characteristics. It is also caused by stray electric currents. Various factors that affect the corrosion of steel in seawater include dissolved oxygen (most common factor), surface films (rust or corrosion products), pH (depends on the dissolved carbon dioxide and/or hydrogen sulfide), salinity, and temperature (corrosion rates approximately double with each 10` C increase in temperature). Alternate wetting and drying is especially corrosive. Marine organisms also can influence corrosion rates by penetrating any protective coating. Sulfide corrosion attack is also common under barnacle encrustations.

Oxidation-type corrosion is normally controlled with pipe coatings. Care must be taken to protect these coatings during pipe handling and installation. Galvanic corrosion is induced by the flow of electric current from more positive areas on a metal surface (anodes) through the electrolyte (soil, seawater), to the less positive (cathode). This electrochemical action removes metal at anode areas where metallic ions leave the metal and enter the electrolyte. Corrosion is usually prevented by means of both pipe coating and cathodic protection.

9.3 Cathodic Protection

Cathodic protection imposes an electric current that reverses the natural electrolytic action of corrosion. The basic principle is that the entire metallic surface is made a cathode in an electric circuit with an external anode. The external anode must have sufficient driving potential (electromotive force) to deliver the required current to the metallic surface through the electrolyte (seawater) so that corrosion ceases.

There are two common sources of protective current, both of which are used on offshore pipelines. One is direct current obtained from rectification of alternating current (AC). The rectified current is distributed to the electrolyte by means of semi-inert, long-life anodes such as graphite, high silicon cast-iron, or lead-silver alloys. The other common source of current is from sacrificial anodes distributed in the electrolyte and connected to the pipeline. The anodes are consumed in the process.

Anodes for rectifer/groundbed installations are normally arranged in a long "groundbed" parallel to the pipeline but seldom closer than 200 ft (60 m) to the pipeline. The 200-ft (60-m) spacing between pipeline and groundbed provides better current distribution along the pipeline. A groundbed may consist of two or three anodes to as many as forty, depending on soil resistivity and local conditions. Anode spacing may vary from 10 to 20 ft (3 to 6 m). Each anode is connected to a common electrical cable leading to the rectifier. Soil resistivity governs the depth to which anodes are buried. The anode is buried in the lowest resistance soil available. Soil resistance can be measured on location with special instruments or from test samples sent to a soils laboratory.

Figures 9-1 and 9-2 illustrate sacrificial anode and rectifier cathodic protection systems, respectively. System selection is based on soil resistivity, local costs of materials and labor, available energy sources, and maintenance requirements. The present trend is toward the use of long-life, low-maintenance sacrificial anodes to protect offshore pipelines. Suitable sacrificial anodes are made from magnesium, zinc, or aluminum alloys. The relative degree of protection is measured in ampere-hours of protective current per pound of metal consumed.

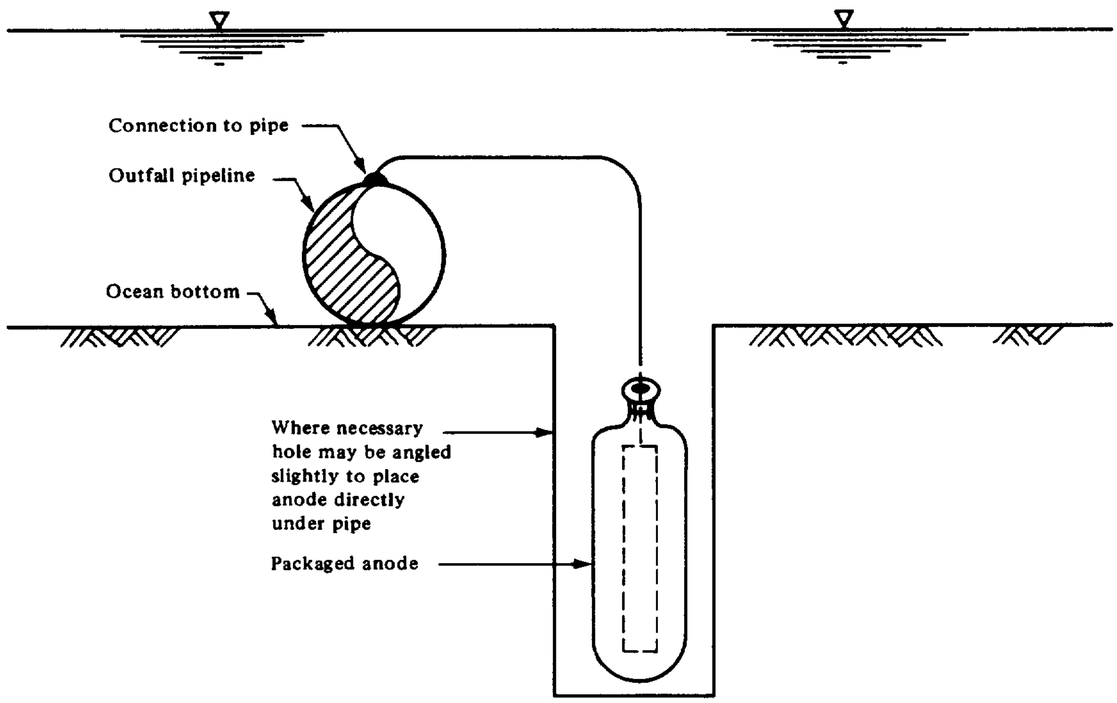
Magnesium was widely used for marine sacrificial anode installations during the 1950s and early 1960s. Magnesium has a high (driving) potential (-1.5 volts open circuit to silver/silver chloride half-cell reference) and a resultant high current availability. However, the high current output causes rapid depletion of the anodes which must be frequently replaced at considerable expense.

Zinc also has a long history as a sacrificial code and has an open circuit potential of -1.05 to -0.5 volts (Ag/AgCl). The relatively low driving potential of zinc against steel, as compared with magnesium, has been proven advantageous in extending the life of sacrificial anode systems.

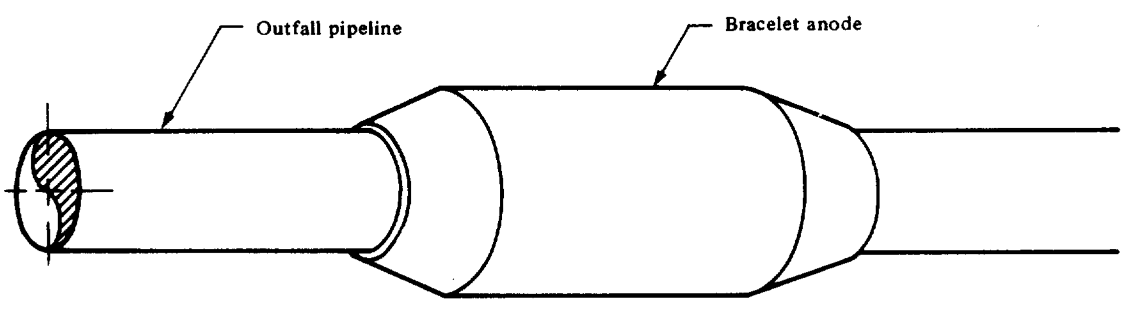
According to theoretical electrochemical performance and cost criteria, aluminum is the most attractive material for a long-life galvanic anode. However, unalloyed aluminum has little use because the normally passive surface film limits current output. Alloyed aluminum overcomes this limitation and has an open circuit potential of -1.0 to -1.05 volts (Ag/AgCl).

One of the earlier aluminum anodes (Al-Type A) to be used contained 5-6% zinc, maximum iron 0.17%, maximum copper 0.02%, maximum silicon 0.10% and the remainder aluminum. Properties claimed for this alloy are: energy delivery of 700-725 ampere-hours per pound alloy, current efficiency of 55% and consumption rate of 12.2-12.5 pounds per ampere-year. One of the latest aluminum alloys, called Al-Type B, contains 0.045% mercury, 0.045% zinc and

Fig. 9-1. Sacrificial Anode Cathodic Protection System for a Marine Pipeline

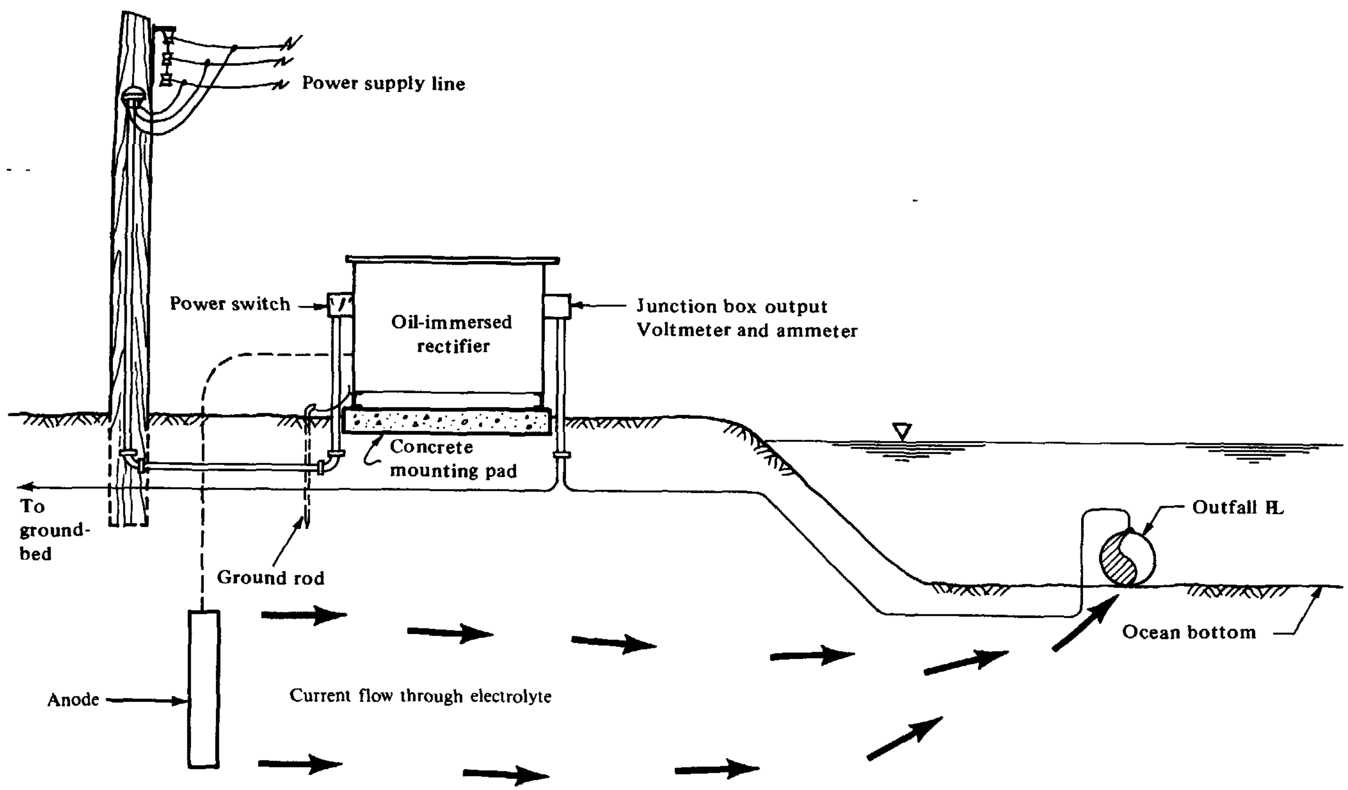


Packaged Anode



Bracelet Anode

Fig. 9-2. Rectifier Cathodic Protection System for a Marine Pipeline



aluminum 99.90%. Its current efficiency is 95% with an energy output of 1280 ampere-hours per pound. Energy capacities and consumption rates of representative anodes are compared in Table 9-1.

9.4 External Coatings

External coatings are not required for concrete or plastic pipe outfalls, or normally for cast iron outfalls. Steel outfalls almost always have external and internal protective coatings in addition to cathodic protection systems.

Two types of external coatings are included in the design of most steel outfall pipelines: a corrosion prevention coating and a weight coating. Weight coating provides stability (Chapter 7) and protects the external corrosion-prevention coating from damage. Pipelines with diameters less than 12 in (30 cm) are often installed with only a corrosion prevention coating. When on occasion larger diameter steel lines are installed with no concrete weight coating, the corrosion prevention coating is usually damaged.

Most types of corrosion preventive coatings are suitable for marine pipeline applications. Tape coatings are normally not used offshore or in areas where the pipe is submerged because they are easily damaged by the weight coating and tend to detach from the pipe when submerged.

Thin film epoxy coatings are occasionally used but they are expensive. The most popular coating materials are coal tar enamel, asphalt enamel, polyethylene, and polypropylene.

Coal tar enamel and asphalt enamel coatings are the most popular because they are known to withstand the marine environment. These are normally 3/32 in (2.4 mm) thick and are overlaid with a felt outerwrap.

The American Water Works Association (AWWA) publishes the C203-73 AWWA Standard for Coal Tar Enamel Protective Coatings for Steel Water Pipelines for both external and internal use. Surface preparation includes sand or shot blasting to provide a clean surface. A high-quality primer is used to ensure a strong bond between the pipe metal and coating. The manufacturers' recommendations regarding the primer and coating material should be followed exactly with respect to shelf life, surface preparation, application temperature, and ambient conditions during application. An inspector representing the owner should be present during all coating operations.

Polyethylene and polypropylene are normally used as thin films around the pipe and the annulus between film and steel filled with mastic. An extrusion process is used to apply the coating. This type of coating is generally limited to 10-in (25-cm) or smaller-diameter pipelines that require no weight coating. Concrete coatings are not applied over such coatings. Field joints are usually coated with heat sensitive tapes or heat shrinkable sleeves.

TABLE 9-1

Energy Capacities and Consumption Rates of Galvanic Anodes in Sea Water

Energy Capacity Anode Material	Consumption Rate Amp-hr/kg	 Amp-hr/lb	Potential kg/amp-yr	 lb/amp-yr	Volts (Ag/AgCl)
Magnesium (H-1 Alloy)	1100	500	8.0	17.5	-1.4 to -1.6
Zinc (MIL-A-18001h)	735-815	335-370	11.9-10.7	26.1-25.7	-1.0 to -1.05
Aluminum-Zinc-Mercury	2750-2840	1250-1290	3.2- 3.1	7.0- 6.8	-1.0 to -1.05
Aluminum-Zinc-Indium	2290-2600	1040-1180	3.8- 7.4	8.4- 7.4	-1.05 to -1.1
Aluminum-Zinc-Tin	925-2600	420-1180	9.5- 3.4	20.8- 7.4	-1.0 to -1.05

Source: Lennox et al. (1).

Factors that govern the selection of the proper corrosion coating materials for marine pipeline projects are availability of coating materials and coating yards in the geographical area, temperature conditions, and unusually corrosive conditions.

Some materials should be carefully evaluated for their costs and service life before imported materials are called for. A survey of coating yard capabilities is also necessary to ensure that the specified coating can be properly applied. Normally the corrosion prevention coating and weight coating are applied at the same coating yard to prevent damage to the corrosion prevention coating during shipment. The coating yard should be as close to the project location as possible to reduce freight costs on the heavy concrete coating.

High ambient temperatures can cause some coatings to become soft and make them susceptible to damage. If it is very cold, coatings become brittle and crack during installation, when the line is subjected to bending stresses. For these reasons, coating manufacturers market several types of materials.

If the outfall is for chemical process wastes, the maximum temperature of the waste must be known. If the maximum rated service temperature is exceeded, disbonding and coating deterioration will occur. An alternate coating, such as a heat-cured epoxy, may be selected for high operating temperatures.

Unusually corrosive conditions occur at the shore approach of a marine pipeline. The soil may be oil soaked or contain other materials that are detrimental to a particular coating. Thus it is necessary to check the site visually and to discuss with local residents or officials past uses of the land.

Coatings are normally applied in a permanent coating yard well in advance of construction to protect the pipe from oxidation-type corrosion in storage and to prevent delays during construction owing to coating yard breakdowns. Because the steel pipe is usually welded, about 6 to 8 in (15 to 20 cm) at each end of each pipe length is left uncoated. After welding, this uncoated area must be cleaned with a wire brush, primed, and coated with materials compatible with the coating previously applied. It is essential that an inspector be present during joint coating operations to ensure protection against corrosion.

Coating integrity can be inspected with a portable, battery-operated electrical resistance monitor, commonly called a "jeep" or "holiday detector." A flexible spring completely encircling the pipe is pulled over the entire length of pipe. If a hole in the coating (often called a "holiday") is found, the detector will sound an alarm at the exact location. This device should be used to inspect both yard-coating application and field joint coating operations. The machine can be adjusted to allow for different types and thicknesses of coating.

9.5 Internal Linings for Outfall Pipelines

Noncorrosive pipe materials such as concrete, plastic, and cast iron do not require linings. Steel outfalls should always be lined with a corrosion prevention coating. A wide variety of coating materials, including baked-on epoxy, or other resins and paints can be used. However, the most commonly used linings for water and wastewater service are coal tar enamel, coal tar epoxy, and cement mortar because of their low costs. Cement mortar lining is not recommended for seawater exposure because of its rapid deterioration in seawater. It is also more brittle than coal tar. Most lined outfalls use coal tar.

Other internal lining materials that have been used for potable water, oil, and gas include zinc silicate-epoxy (potable water), polyamide-cured epoxy (oil and gas), epoxy-phenolic (oil and gas), and polyurethane (abrasive slurries). Whether the extra expense of these coatings is warranted will depend on the corrosive or abrasive characteristics of the wastewater and possibly on the improvements to flow characteristics of the pipe.

During welding operations the lining will be "burned-back" or charred for two to three inches. This area must be cleaned and recoated with a compatible primer and coating material. On large pipelines this presents few problems since a man can crawl into the pipe and coat the joints. On smaller lines an automatic cleaning and coating machine is sometimes used but the results have not been consistent. Sometimes the welded joint area is left uncoated in smaller lines.

9.6 References

1. Lennox, T.J., Jr., R.E. Groover, and M.H. Peterson. 1971. "Electrochemical Characteristics of Six Aluminum Galvanic Nodes in the Sea." Materials Protection and Performance, 10(9):39-44.

CHAPTER 10

THE STATE OF THE ART OF MARINE OUTFALL CONSTRUCTION

10.1 General

A number of methods have been used to construct outfall pipelines, most of which have been successful. However, failures have occurred when previously successful methods have been improperly applied.

Some pipelines have been assembled onshore and others offshore, either above water or underwater on the seabed. In some cases the pipe has been assembled in a remote location and floated to the job site.

The following paragraphs present a brief summary of the more commonly used methods, and the present state-of-the-art constraints. Chapter 11 describes some of the details of construction methods.

10.2 Parameters for Method Selection

Factors to be considered when a construction method is being selected include pipe size, pipe material, outfall length, shore-site topography, tidal and storm currents, and construction equipment availability and location.

Pipe diameter limits alternatives with respect to both construction materials and methods. Outfalls with diameters greater than 60 in (150 cm) are normally reinforced concrete pipe. Smaller lines can be reinforced concrete pipe (RCP), plastic, steel, or cast iron. Those with diameters of 12 in (30 cm) or less are lighter and more flexible and provide for a wider range of construction options. Lines 24 in (60 cm) and larger in diameter are increasingly rigid and require more attention to equipment-lifting capacity and pipe-bending stress analysis.

RCP sections are usually installed by connecting individual sections 10-30 ft (3-9 m) long on the seabed with a crane barge. Joints are usually a bell-and-spigot design with gaskets. An above-water assembly technique is not practical for RCP as the joint is weaker than the pipe. Steel pipe and plastic pipe have joint designs as strong as the pipe, and therefore allow more flexibility in selecting the construction methods. Cast iron is not a popular outfall pipe material but it has been used in some cases. Its weak joints, availability only in small diameters, brittle nature, and higher costs are inherent disadvantages.

Short outfall lengths of 300-1,300 ft (100-400 m) lend themselves to site-specific construction methods depending on local conditions, local contractor experience, and equipment. Outfalls 2,000 ft (600 m) and longer have either been pulled into place from onshore fabrication sites by the pull method or installed from a lay barge. Recent developments in construction technology and pipe materials make other methods practical, such as directional drilling or reel barges for small diameter lines.

A flat open area at the shore end of the outfall provides an ideal pipeline fabrication site for conventional pull methods of outfall construction. Figure 10-1 shows an onshore layout for pipe fabrication for a pulling operation. If the onshore area is steep, crowded, or otherwise unusable for a fabrication site the contractor can use a floating work platform, usually a barge (Figure 10-2), or he can fabricate elsewhere and float the outfall into place.

The availability and location of construction equipment may be a significant factor in selecting pipe materials and the construction method. The costs of marine equipment mobilization and demobilization on a smaller outfall project may exceed 30 percent of the total cost.

Strong tidal and storm currents may be an asset in dispersing the outfall effluent, but they are a liability during construction of the outfall. Strong tidal currents can rule out construction methods that involve floating sections of pipe into place or the surface pull method.

10.3 Construction Method Classification

Construction methods include bottom (seabed) assembly, surface assembly from offshore lay barge, bottom pull from onshore, offshore, or opposite shore, surface pull (flotation), remote assembly, and directional drilling or boring.

Bottom assembly refers to assembling and connecting short lengths 10-300 ft (3-100 m) on the seabed from crane barges or jack-up barges. Divers assist in making the connections.

With surface assembly, pipes are joined on a floating or jack-up barge that lowers the assembled pipeline to the seabed as it progresses along the pipeline route. Lay barges range from very simple units used for small-diameter or shallow-water projects to elaborate floating assembly plants. Lay barges used on offshore oil and gas pipelines and some longer outfalls are capable of fabricating 3,000-10,000 ft (1,000-3,000 m) of pipeline per day with a crew of 100 to 200. Substantial auxiliary floating equipment is necessary to support the operation of larger barges. Most lay barges are located near offshore oil or gas activities and are feasible for nearby outfalls of 3,000 ft (1,000 m) or more in length. Many lay barges can install pipes up to 48 in (120 cm), and with minor modifications larger lines can be installed.

In bottom pull, the line is pulled along the final route of the pipeline. This method requires a minimum amount of floating construction equipment and may be used in many geographical locations and sea conditions.

The surface-pull method is essentially the same as the bottom-pull method except that the pipeline is light enough to float either by itself or with auxiliary buoyancy. After it is pulled into place, water is allowed to enter and sink the line into the predredged trench or along the specified route.

Fig. 10-1. Onshore Layout for Pipe Fabrication

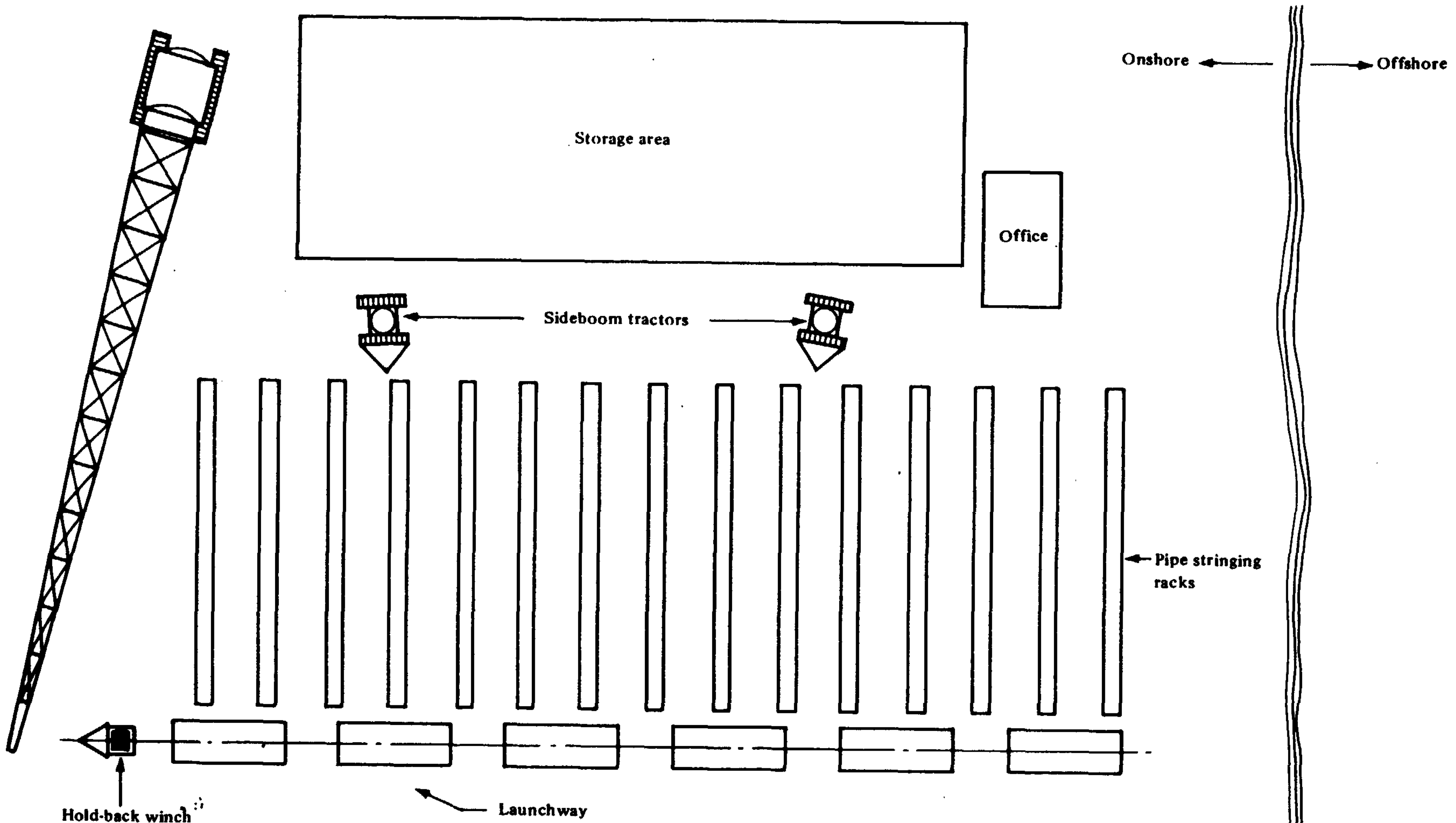
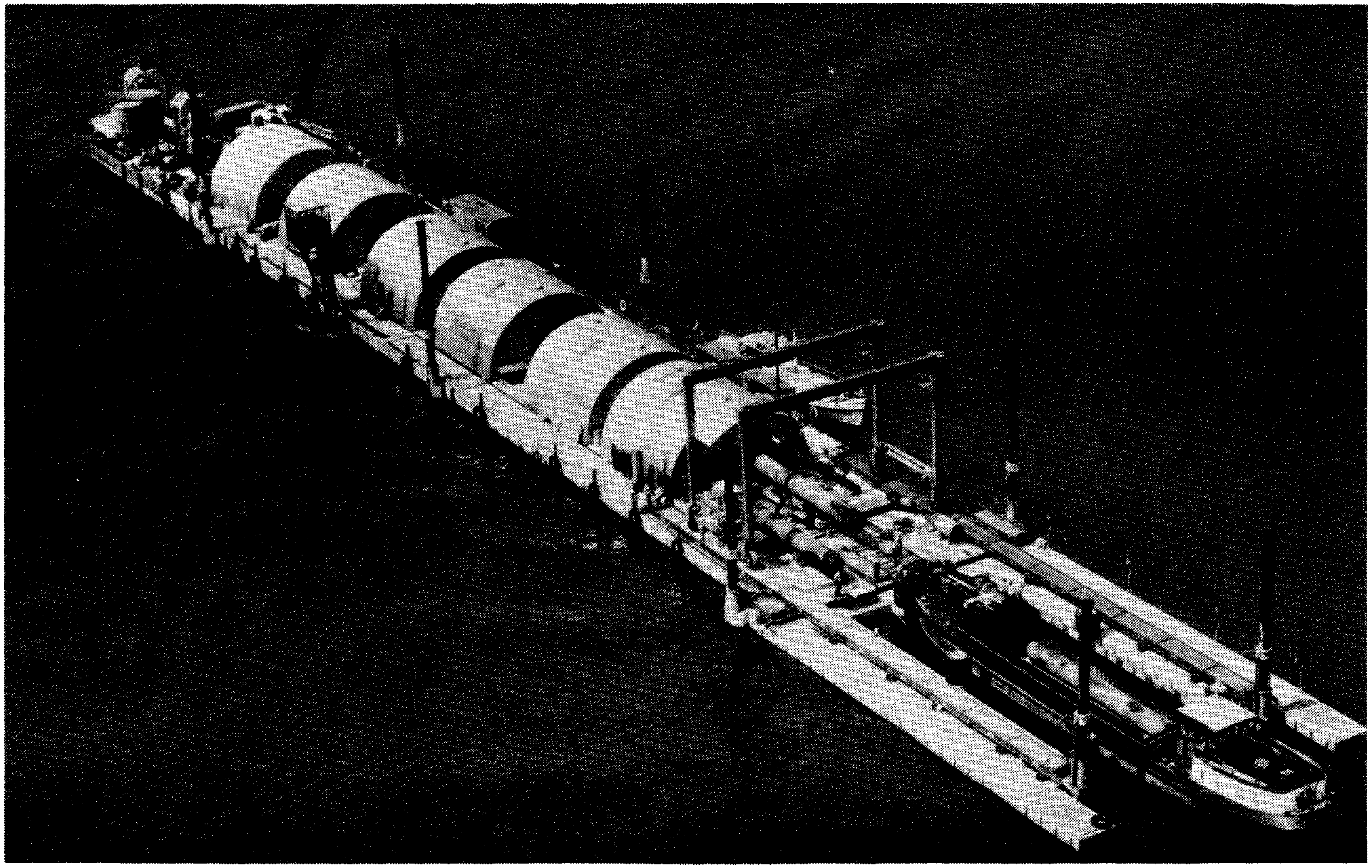


Fig. 10-2. Barge for Pipeline Fabrication



Source: Flexifloat Systems Catalog, Robishaw Engineering, Inc., Houston, Texas.

An example of remote assembly is the reel barge where lengths of small diameter pipe of 30 cm (12 in) or less are welded together at an onshore fabrication site, reeled onto a floating reel barge, and transferred to the job site for laying. It is also possible to assemble long sections (or the entire line in the case of short lengths and no tidal current) at a remote location, by floating the line into place and sinking it along the route. Connections are made on the seabed as necessary.

Boring or directional drilling follows the directional drilling technique developed in the oil and gas industry. Use of an on-site drilling rig is expensive, but it minimizes construction area requirements and eliminates marine traffic interference. A boring unit is shown in Figure 10-3.

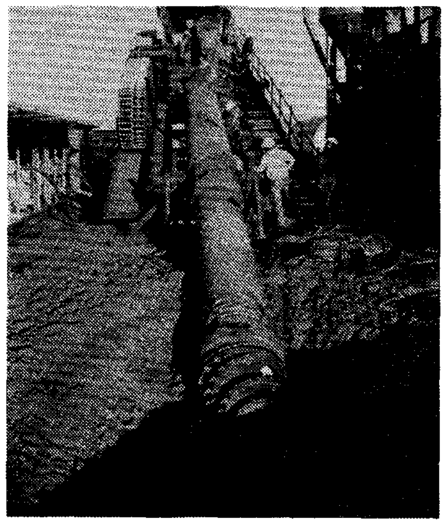
Both the reel barge and directional drilling technologies are for small pipelines carrying oil, water, or other high-value products. They are mentioned accordingly suited to water supply rather than wastewater pipelines.

For outfalls over 1,000 m (3,000 ft) long, one construction method may be used for the shore approach and another for the offshore portion. Predredged trenches are often prepared through the shore approach zone and the line laid or pulled into the trench. Predredged trenches in surf zones tend to silt in quickly; therefore, construction scheduling and proper planning is important. Attempts have been made to lay pipe across the shoreline and then bury it later; most of these operations have failed. Trenching and backfilling are discussed in Chapter 12.

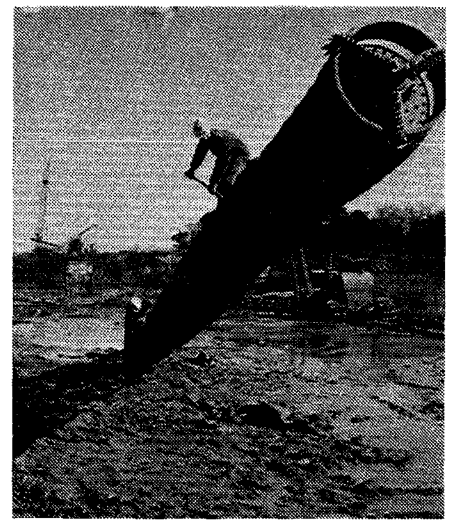
10.4 State-of-the-Art Constraints

Innovative and improved outfall constructions are constantly being developed. Nevertheless, it is important to recognize the present (early 1980s) limitations to alternative construction methods listed in Table 10-1. These factors are in addition to those listed in paragraphs 8.3 and 10.3.

Fig. 10-3. Directional Drilling Machine



Rig with Bit



Bit Exits on Opposite Bank

Source: River Crossings Brochure, Reading and Bates Construction Company, Houston, Texas.

TABLE 10-1

Practical Limitations of Outfall Construction Technologies

	Maximum Diameter in	(cm)	Minimum Length ft	(m)	Maximum Length ft	(m)	Maximum Depth ft	(m)	Maximum Current knots	(m/sec)
SHORE APPROACH										
Pretrenching										
Explosives	--	--	--	--	--	--	--	--	6	(3)
Mechanical Dredge										
Dipper	--	--	--	--	--	--	100	(30)	6	(3)
Dragline	--	--	--	--	--	--	330	(100)	6	(3)
Clamshell	--	--	--	--	--	--	180	(60)	6	(3)
Continuous	--	--	--	--	--	--	180	(60)	6	(3)
Hydraulic										
Hopper	--	--	--	--	--	--	--	--	6	(3)
Dustpan	--	--	--	--	--	--	60	(20)	6	(3)
Cutterhead	--	--	--	--	--	--	600	(200)	6	(3)
Post-trenching										
Jet sled	--	--	--	--	--	--	300	(100)	6	(3)
Cutterhead sled	--	--	--	--	--	--	--	--	6	(3)
Plow	NOT RECOMMEMDED									
Sheetpile channel/ cofferdam	--	--	--	--	1,000	(300)	60	(20)	--	--
Trestle	--	--	--	--	1,000	(300)	60	(20)	--	--
OFFSHORE INSTALATION										
Bottom assembly										
RCP, Bell-and-spigot	144	(365)	--	--	Unlimited		830	(250)	2	(1)
Steel, alignment frame	92	(235)	--	--	Unlimited		830	(250)	2	(1)
Cast iron	54	(135)	--	--	Unlimited		830	(250)	2	(1)
Lay barge	18	(45)	3,000	(1,000)	Unlimited					
	30	(75)	3,000	(1,000)	Unlimited		600	(200)	6	(3)
	48	(120)	3,000	(1,000)	Unlimited		300	(100)	6	(3)
Reel barge	12	(30)	30,000	(10,000)	Unlimited				6	(3)
Bottom pull--steel pipe	120	(325)	--	--	5,000	(500)	500	(150)	5	(2.6)
	42	(105)	--	--	100,000	(1,000)	--	--	5	(2.6)
	--	--	--	--	15,000	(5,000)	--	--	--	--
	--	--	--	--	75,000	(25,000)	--	--	--	--
	--	--	--	--	100,000	(1,000)	--	--	--	--
Surface pull--steel pipe	--	--	--	--	300	(100)	--	--	5	(2.6)
	--	--	--	--	1,000	(300)	--	--	1	(.5)
Directional drilling	50	(150)	--	--	5,000	(1,500)	--	--	--	--

Note: Minimum lengths are based on mobilization costs while maximum lengths and other characteristics are based on technological constants. Plastic pipe laying technologies are not included because of limited experience and information on service life. Descriptive information on selected technologies is in Chapter 11.

CHAPTER 11

CONSTRUCTION METHODS

11.1 General

Descriptions and design factors of the construction methods identified in Chapters 8 and 10 are presented in the following paragraphs. Construction costs usually account for 60 to 80 percent of the overall installed cost of an outfall line. Both the costs and methods are site specific. The ability to select the least-cost method from among those described below depends on familiarity with and experience in marine and coastal construction practices.

11.2 Bottom Assembly Methods

Final connections of pipe lengths (joints) are made on the seabed. Variations of this concept include laying from a mobile jack-up platform, a trestle, or a crane barge. All the variations require lifting capacity with sufficient precision to lower pipe lengths into place, alignment and on-bottom connection of pipe lengths by divers. Research on and prototype testing of unmanned, remote-controlled connection systems are being carried out by oil and gas companies on steel pipe in deep water (greater than 300 m or 1,000 ft). Meanwhile bottom assembly methods are limited to water depths and site conditions in which divers can perform useful work, which is presently less than 100 m (300 ft) for conventional diving and less than 250 m (800 ft) for saturation diving.

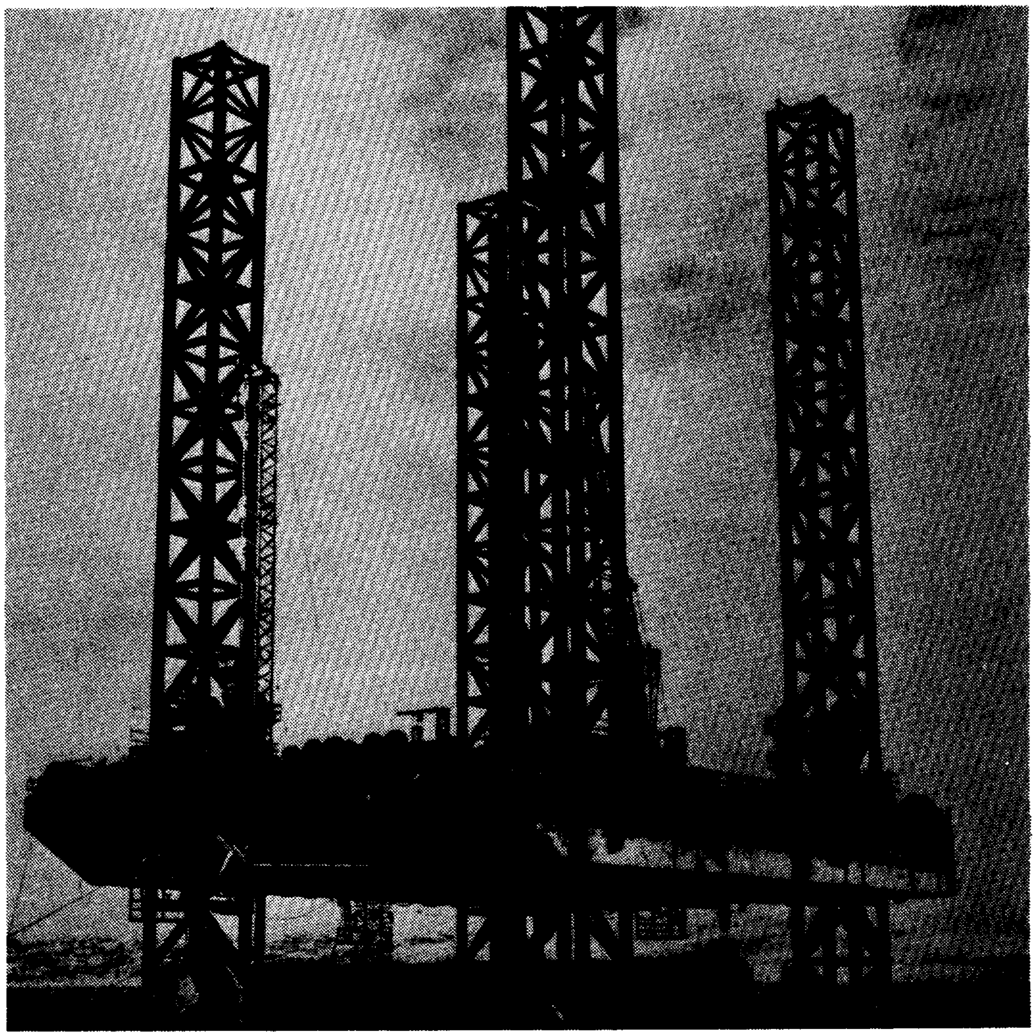
11.2.1 Pipe Laying from a Mobile Jack-up Platform

Mobile jack-up platforms have been built for specific projects that used large diameter reinforced concrete pipe. The platform is floated into place, large anchors (either gravity anchors or drilled-in) are placed to maintain the barge on location, and the legs are lowered to the seabed. Large jacks are used to raise the work platform out of the water free from the effects of normal waves and currents. After each pipe section is added to the line, the platform is advanced by lowering its buoyant deck until the whole structure is floated, pulling itself on its anchor lines to the next position and jacking the platform out of the water to continue laying operations. Figure 11-1 shows a photograph of the mobile platform used to install the Hyperion outfall. Figure 11-2 is a simplified drawing showing the platform operation.

11.2.2 Pipe Laying From a Trestle

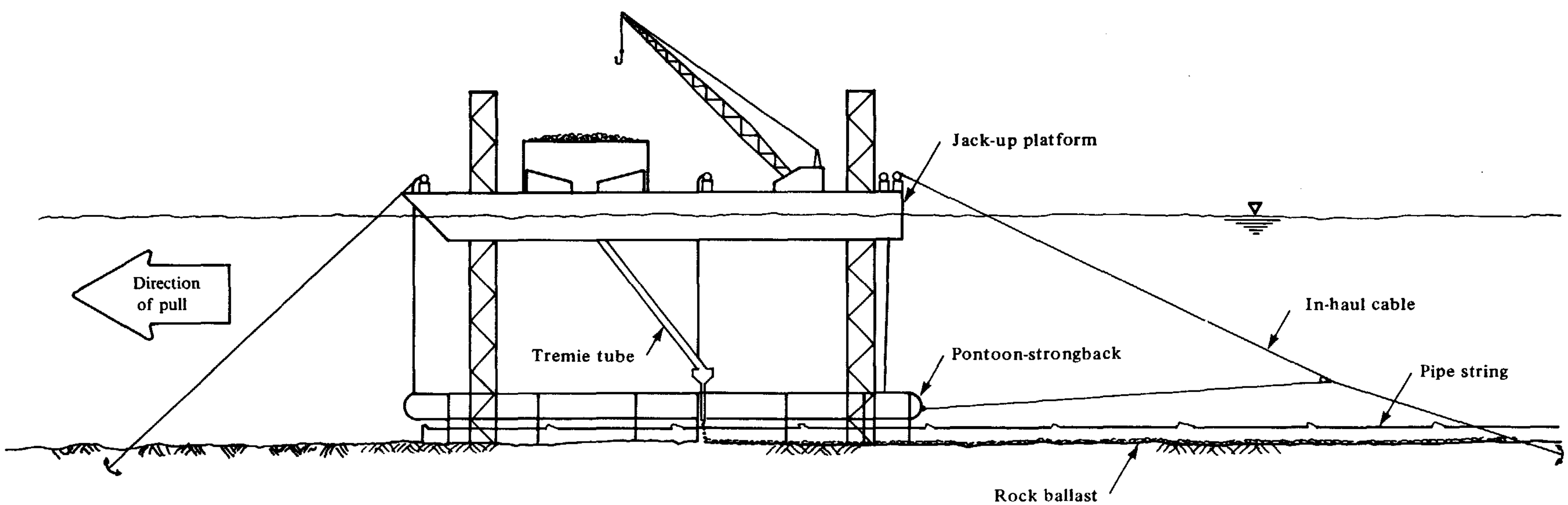
The inshore sections of many outfalls have been laid from trestles, or temporary piers. The trestle is used in waters that are too shallow for pipe-laying barges or where wave and surf conditions might rock a floating platform excessively. Although the trestle does not eliminate all wave surge problems in inshore waters, it ensures that the work can be performed from a fixed platform.

Fig. 11-1. Mobile Pipe-laying Platform



Source: Photograph from D.R. Miller.

Fig. 11-2. Mobile Platform Operation



Trestles are begun by driving H-piles into the sea floor. They are usually not grouted, so that the piles can be retrieved later. A pile bent is made by setting beams across pairs of piles. Bolting rather than welding eases recovery. Longitudinal beams are then laid across the pile bents, and rails for the crane, pile-driver, and pipe-laying equipment are placed to complete the structure. The crane advances seaward for pile driving and follow-up work, and when the work is completed, returns to shore, taking up the piles as it progresses. While the trestle is being extended by one crane, another crane can be driving sheet piling further inshore, (preferably with a vibratory hammer) or excavating with a clamshell to prepare the seabed for the pipe laying operations.

Either one or two standard cranes or a gantry crane (Figure 11-3) can be ued to lower the pipe, supported by two or three slings, into place on the seabed.

11.2.3 Pipe Laying from a Floating Crane Barge

This method requires a large barge and auxiliary pipe transport and supply barges. The crane on the lay barge lowers a length of pipe horizontally to the bottom, where it is joined to the existing line. Pipe lengths can vary from 3 to 100 m (10 to 330 ft). This method can be used for small to very large pipelines.

In protected waters, a crane barge and a strongback (heavy beam supporting the length of a pipe) can pull a pipe joint into the bell of the completed pipe section with the aid of cables. Final seating may be done with a vacuum in the space between the pipe ends, or by using flanged connections and bolts.

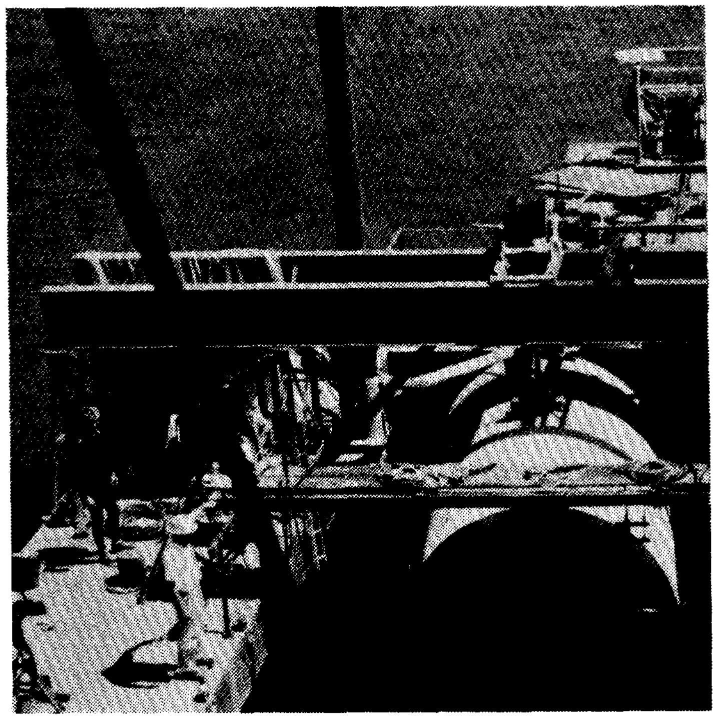
In rough waters, if there is poor control over the position of the section being laid, pipe ends can be damaged and it is extremely difficult to insert a new section into the completed line. This problem can be overcome by using a pipe handling frame. A typical handling frame is shown in Figures 11-4 and 11-5. Handling frames such as these can assemble lengths of pipe and install the preassembled section underwater. Hydraulic rams provide vertical control at quarter points, transverse adjustment at each end, and longitudinal motion for inserting the new section into a previously laid section. The frame works independently of the crane barge. The frame is supported by the seabed and controlled by a console aboard the barge. The crane is free to prepare and preassemble joints of pipe.

11.3 Surface Assembly with an Offshore Lay Barge

This method is used in deep water by the offshore oil and gas industry, but it can also be employed in laying sewer outfalls. Although cast iron pipe has been laid from a lay barge, the pipe usually laid by this method is coated steel pipe.

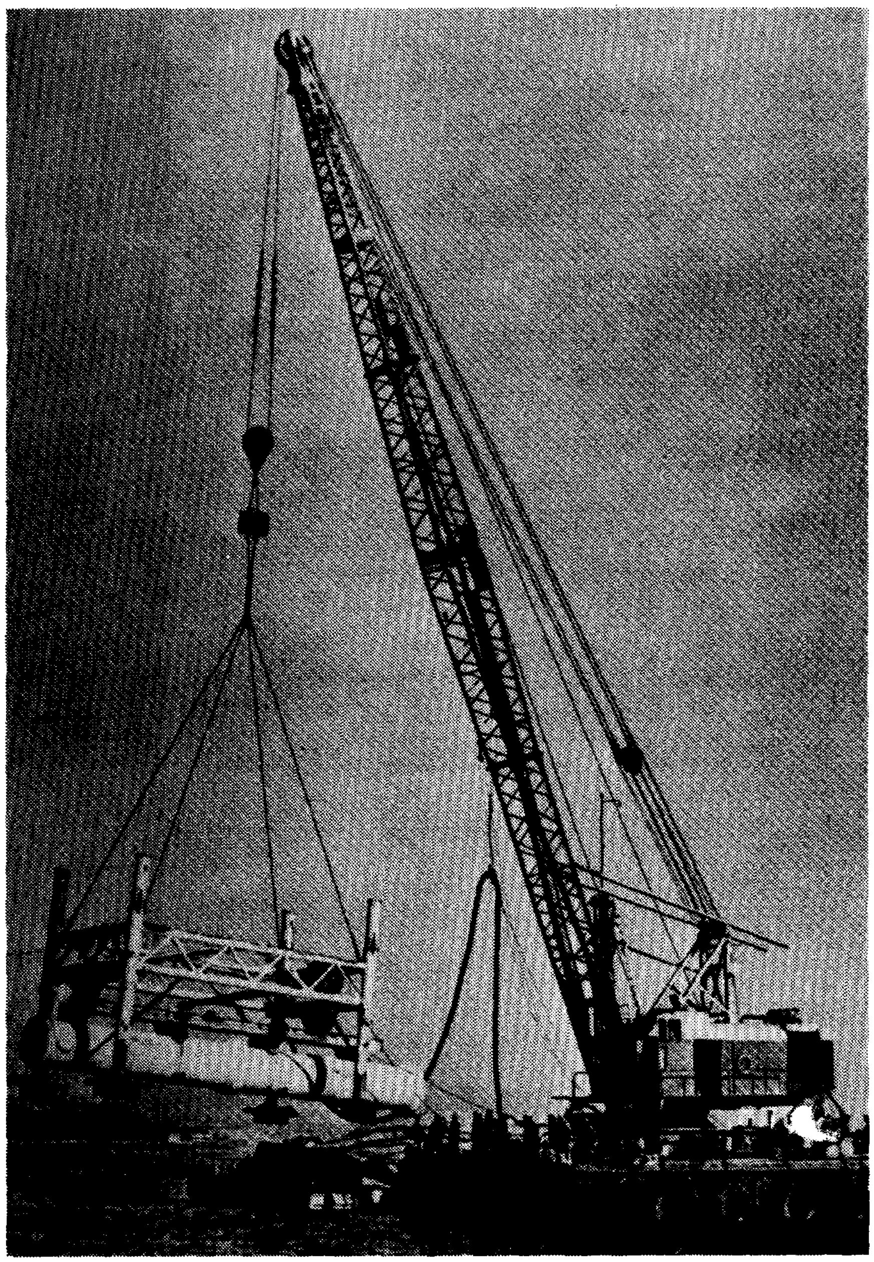
Lay barges can be single or multiple barge units with fabrication facilities, automatic positioning systems, and living accomodations. Simple barge arrangements are usually suitable for nearshore, shallow water, 6 to 60 m (20 to 200 ft) deep.

Fig. 11-3. Reinforced Concrete Pipe Being Lowered by a Crane



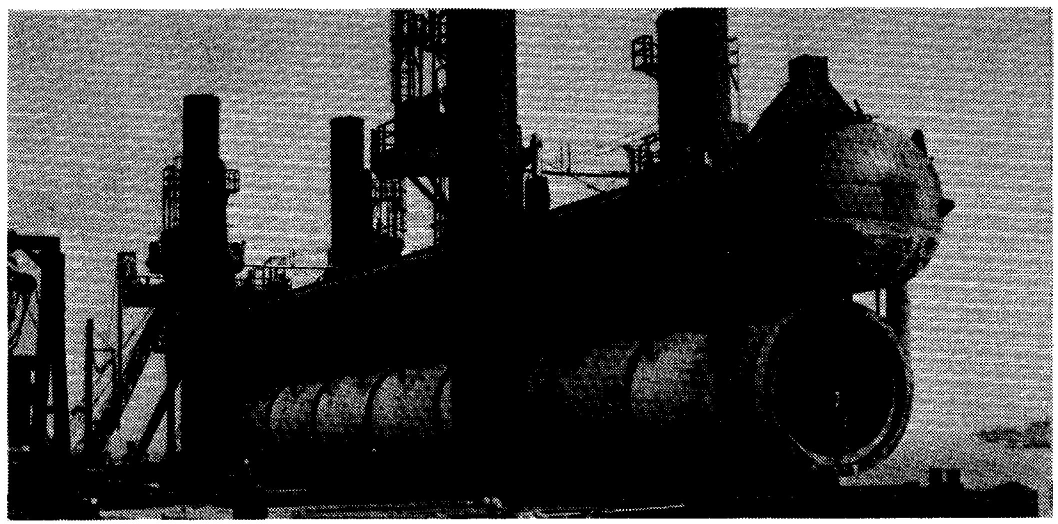
Source: Flexifloat Systems Catalog, Robishaw Engineering, Inc., Houston, Texas.

Fig. 11-4. Pipe Handling Frame



Source: Western Construction, vol. 48, no. 5 (May 1973).

Fig. 11-5. Pipeline Horse



Source: Daniel, Mann, Johnson, and Mendenhall, Los Angeles, California.

Lay barges used by the oil and gas industry are large floating pipeline construction facilities, with four to seven welding stations, a radiographic weld inspection station, and a joint coating station. The conventional lay barge method of construction in waters less than 60 m (200 ft) deep is illustrated in Figure 11-6. The pipe configuration consists of a straight portion, supported by rollers on the barge assemblyway, an overbend at the end of the barge, support of the descending pipe by rollers on a buoyant ramp (the stinger), an unsupported sagbend between the end of the stinger and the seabed, and support of the pipe by the seabed.

Pipe lengths are joined together on the lay barge supplied by attending pipe barges. As the pipe is joined, the lay barge pulls itself forward on its anchor lines, which are progressively moved forward by an attending tugboat. The pipe moves down the barge assemblyway and down the stinger, which extends close to the bottom so that large stresses are not built up in the unsupported pipe section between the end of the stinger and the seabed. Stinger length and angle are designed to keep the pipe stress below approximately 85 percent of the minimum specified yield stress. This is done by controlling the radius of curvature of the pipe in the overbend and sagbend regions. At water depths of 60 to 200 m (200 to 600 ft) the length of the unsupported pipe between the end of the stinger and the seabed increases. This results in overstressing the pipe unless axial tension is applied to the pipeline. For instance, at 160 m (540 ft) of depth, a 90 cm (36 in) diameter pipeline could have an unsupported sag bend up to 65 m (2,110 ft) long.

A lay barge for use in deep water is shown in Figure 11-7. The pipe configuration is similar to that of Figure 11-6, except that the tensioning device is installed in the assemblyway before the end of the barge, and a curved or articulated stinger is used instead of a straight stinger. Articulated stingers are composed of several structural sections, usually 20 to 40 m (65 to 130 ft) long, hinged at the joints to allow a greater flexibility than is possible with straight or fixed curvture stingers.

Straight stingers up to about 140 m (450 ft) in length provide good support and protection for the pipe over most of its path from the barge to the ocean floor at 60 m (200 ft). Longer stingers up to 215 m (700 ft) have been used but are unwieldy, more susceptible to breakage, and more difficult to maintain. They have the advantage that when rough weather approaches and it is necessary to drop the pipe and stinger to the ocean floor to prevent breakage, the operation can be performed quickly and with much less risk than is the case with the shorter straight or articulated stingers.

Articulated stingers are shorter and more mobile. Also, the stinger configuration can be altered to meet changing conditions. They are difficult to lower to the bottom with the pipe, especially in deep water; time must first be taken to alter the stinger configuration so that it is straight.

Curved stingers offer the same advantages and disadvantages as articulated stingers, except that their radius of curvature is fixed during a

Fig. 11-6. Conventional Lay Barge Pipe Configuration

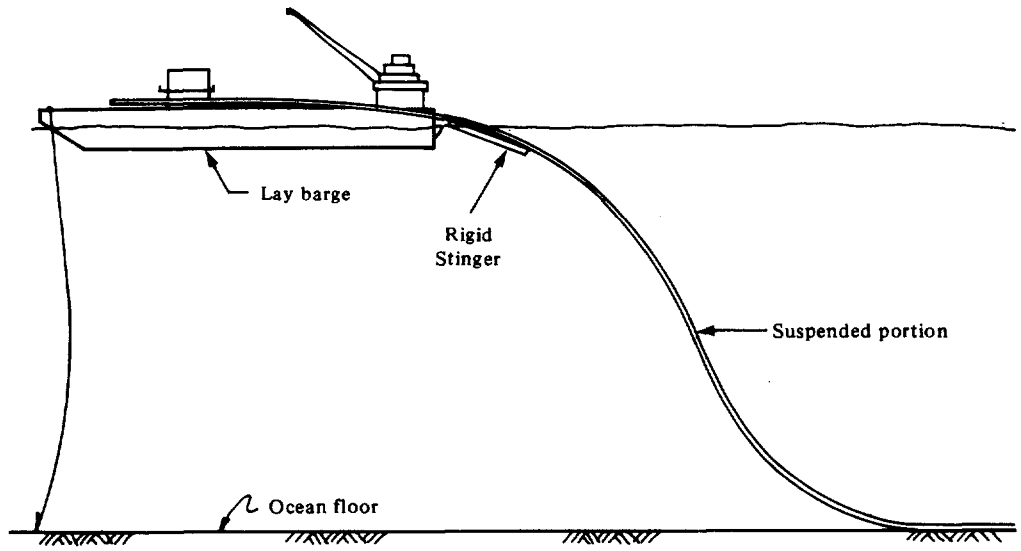
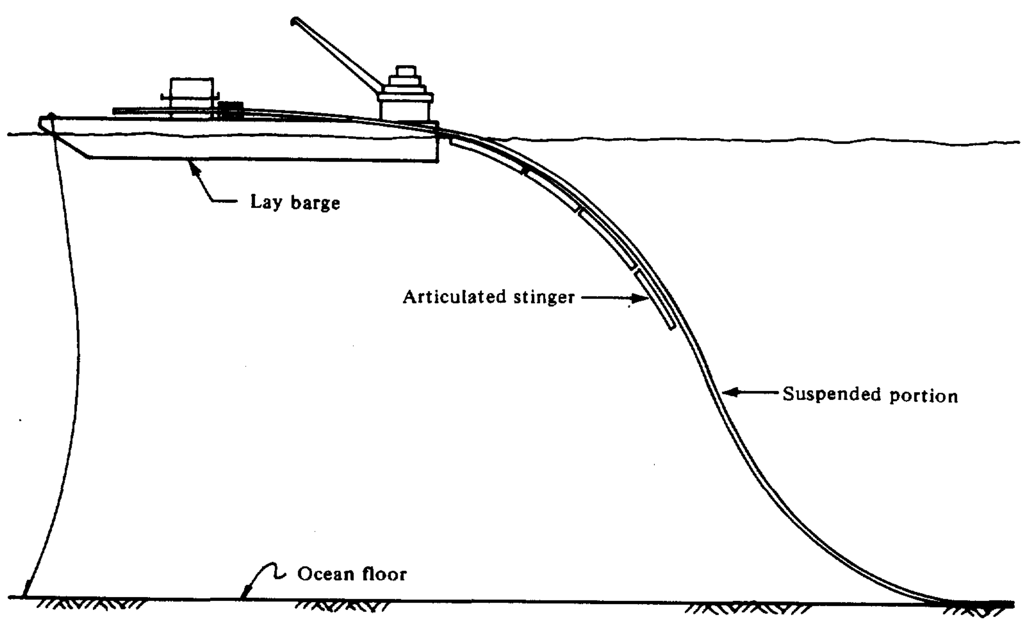


Fig. 11-7. Deepwater Lay Barge Pipe Configuration



job and can only be altered by disconnecting it and towing it back to a shore-based facility.

Anchoring of the lay barge is critical to successful pipe laying, since excessive or sudden barge motion due to anchor slippage can cause the pipe to become overstressed or the stinger to fail. Problems occur where anchors are too light or improperly designed for the particular bottom conditions. At present, anchoring is normally accomplished by using from 8 to 12 anchors in deep water and 6 to 8 anchors in calm weather environments such as estuaries and bays. Anchors are positioned by tugs and attached to the barge winches by steel cables.

Laying pipe by lay barge is fast but costly. Lay barge construction spreads require auxiliary tugs and barges and tugs for moving anchors and supplying pipe, supplies, and personnel. In 1980 the costs for typical lay barge spreads ranged from approximately US$100,000 per day in the calm water areas of the world to more than US$300,000 per day in the North Sea. Apart from cost considerations of the lay barge method, there is considerable difficulty in accurately positioning the barge along a predetermined alignment. Most modern barges are equipped with computerized automatic positioning systems. An outfall less than about 1,000 m (3,000 ft) is normally too short for the lay barge to be a viable alternate. Many lay barges can install pipe sizes up to 120 cm (48 in). With minor modifications to rollers and tensioners larger pipe (up to 150 cm or 60 in) can also be installed.

11.4 Bottom-Pull Methods

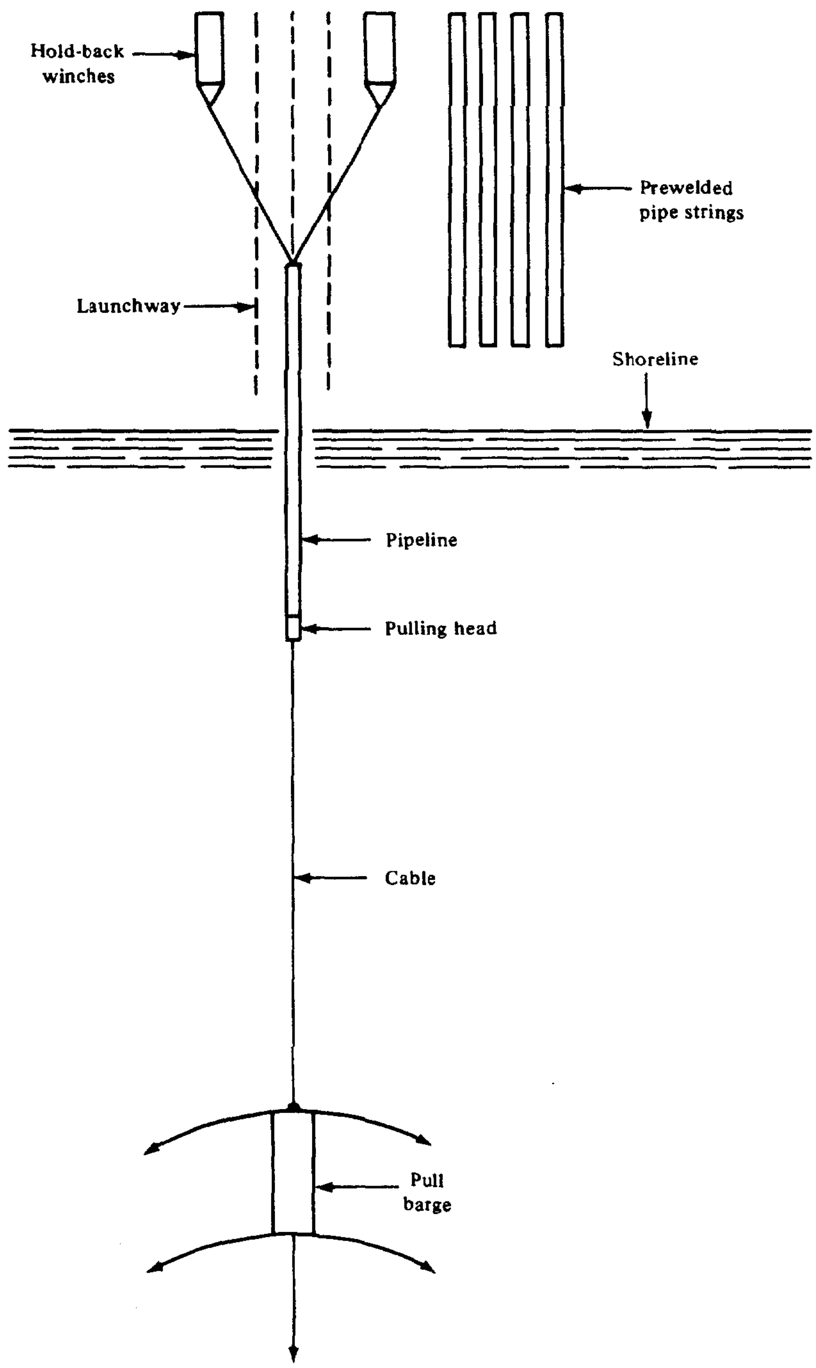
These methods are particularly suitable for smaller diameter outfalls in developing countries and use a minimum of floating equipment. The pipe configuration for the bottom-pull method is shown in plan view in Figure 11-8. Steel and high-density polyethylene plastic pipe have been installed by this method. Several variations of the bottom-pull method have been successfully used when site conditions have been favorable and construction equipment has been available.

Sections of precoated steel pipe are assembled onshore in 20 to 500-m (60 to 1,650-ft) lengths, welds are nondestructively tested, and the bare pipe at the field joint is given a corrosion-resistant protective coating. Each length is set aside on skids or runners parallel to the route of the pipeline to await the actual installation.

The pulling winch may be on a barge anchored several hundred meters beyond the pipe terminus and directly in line with the route of the outfall line. Alternatively, the winch could be located onshore with a pile cluster and sheave located offshore or on the opposite shore of an estuary.

Before the pull, one end of a cable or wire rope is connected to a pulling head welded to the leading section of pipe to be pulled. An onshore track or roller system may be needed. As the pipe enters the water, buoyancy relieves some of the pipe weight, but it is often necessary to add more buoyancy to the outfall line, particularly if the line is to be pulled over a rough or rocky bottom. If a pipeline weighs more than 15-30 kg/m (10-20 lb/ft)

Fig. 11-8. Bottom-Pull Method



in the water, the external coating could be damaged by rough or rocky bottoms. Pipes are pulled empty and often fitted with timber floats or buoyancy tanks (pontoons). Such tanks can be permanently sealed or fitted with ports and valves for controlled buoyancy. It may be difficult to release buoyancy devices at depths greater than 60 m (200 ft). The usual practice is to provide 8-30 kg/m (5-20 lb/ft) of negative buoyancy for a pipeline when it is being bottom-pulled. Maximum pulling speed is of the order of 6 m (20 ft) per minute.

Under no circumstances should this method be used when the line must be left lying immobile between tides, particularly where the tide might come in laterally to the line.

With too little negative buoyancy, a line being pulled can easily wander off course, and attempts to straighten the line by pulling usually result in breaking the pull cable, the pipe, or the pipe coating material.

A serious problem that arises from the effects of friction and cohesion during pulling is that on resumption of pulling operations following a pipe section tie-in, the starting force may be of such magnitude that the elastic limit of the pipe may be exceeded, with consequent damage to the pipe and its coating. A series of excessive stress applications could ultimately rupture the pipe. Starting forces from two to five times the continuous pulling load have been encountered.

The bottom-pulling method is not compatible with laying lines around obstacles such as rock outcrops, reefs, wrecks, or isolated deeps. This is not to say that bottom-pulled lines cannot be laid on broad sweep curves. However, a great deal depends on pipe stiffness and bottom conditions. The best practice is to pull the line in straight route alignments.

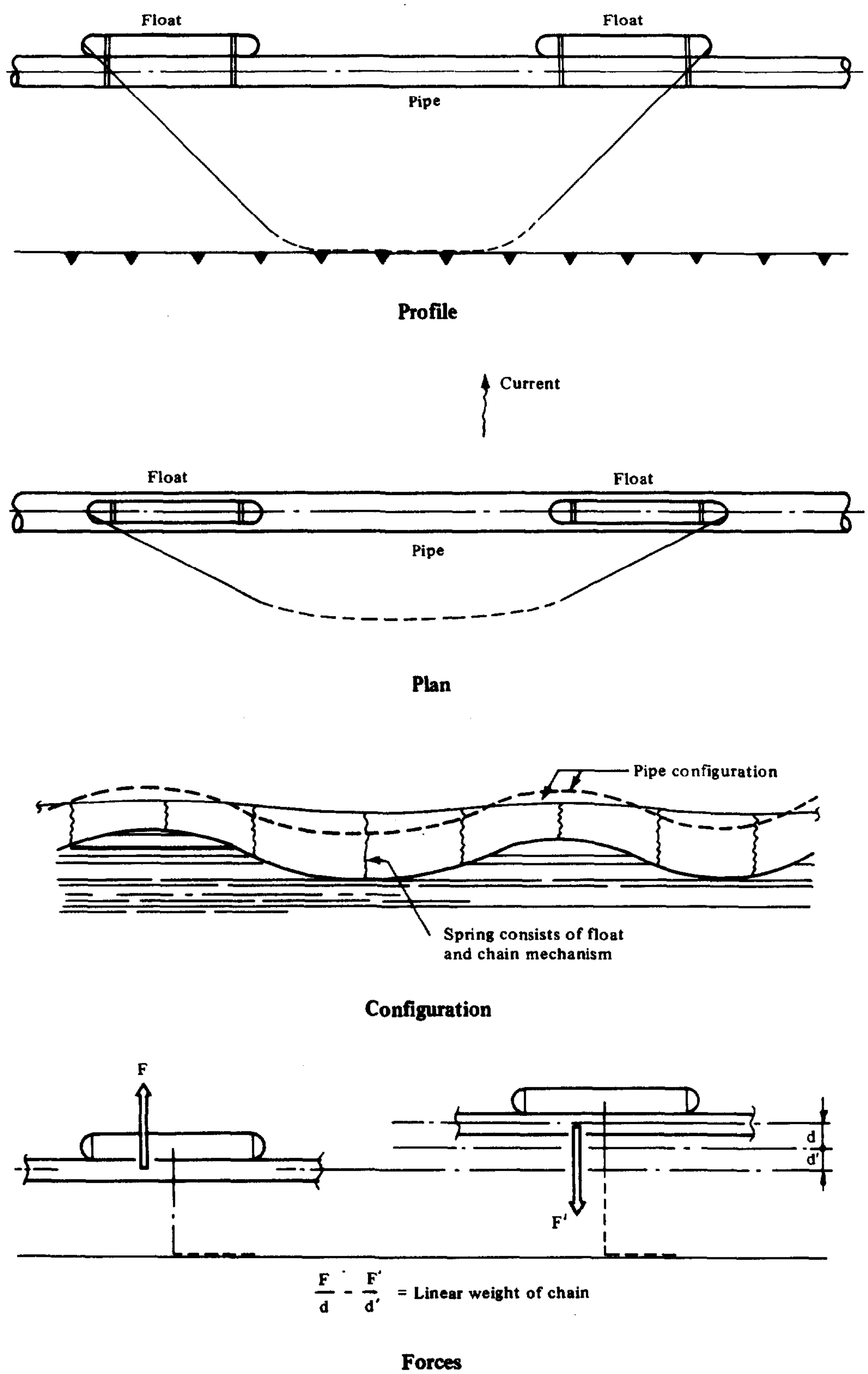
The method requires a relatively large, level land or barge area near the shoreline and in line with the proposed offshore route. This allows onshore storage and assembly of pipe sections prior to pulling.

11.4.1 Floats-and-Chains Method

This method has been developed for towing long sections of pipe that are fitted with buoyant units and chains. The pipe is towed while being suspended a few meters above the seabed. It requires surface facilities that are less expensive to operate than lay barges. This method has been used to lay very long sections of pipe, over 10 km (6 miles) in length at depths up to 100 m (300 ft). A diagram of the pipe fitted with its buoyant units and chains is shown in Figure 11-9.

The weight of the raised chains balances the amount of the thrust load of the buoyant units and the weight of the pipe. The pipe is towed empty. Where the seafloor is flat, the chains maintain the pipe at a constant level above the seabed. Where the seafloor is uneven the effect of the chains and buoyant units is similar to that of an elastic mattress placed between the pipe and the seabed. The length of the chains pulled on the seabed is calculated so that pipe stability is ensured in the maximum lateral current

Fig. 11-9. Floats-and-Chains Method



likely to be encountered on the towing route. The tensile force required for towing a section of line is directly related to the maximum force exerted by the lateral current on the section of line over a flat area. This method is of great interest, especially in areas with small currents. The method differs from conventional bottom pull in that a retaining force is exerted on the shore end of the pipeline to avoid buckling.

11.4.2 On-Bottom Connection of Short Lengths

This method is a variation of the bottom-pull method. Several long sections of pipe are bottom-pulled and connected underwater by welding or mechanical means such as flanges. The first section is pulled into place at the most distant location offshore. Then a second section is pulled by the same method and axially aligned with the first section. Final alignment is made by an alignment frame that is lowered onto the pipe ends from the surface and is operated by divers. Alignment frames also contain a chamber that enables a diver-welder to join the sections. Succeeding strings follow the same procedure of pulling, aligning, and joining until the job is completed at the onshore end.

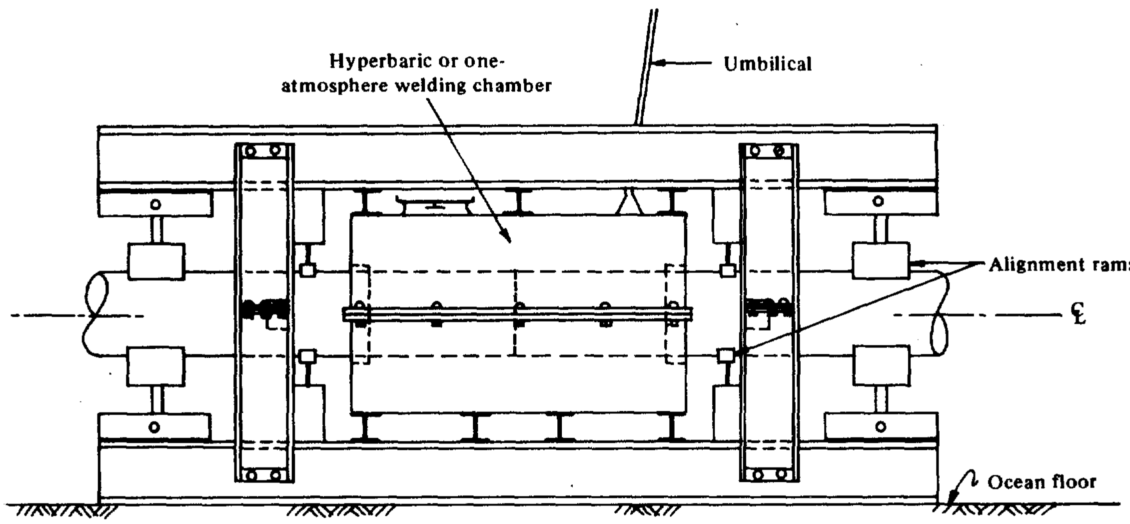
While rough positioning of the pipe is accomplished by conventional pipe pulling procedures, final alignment of the joint is made in the alignment frame with tie-in fixtures. Pipe strings that are made up on shore are fitted with weld-neck anchor flanges on the leading and trailing ends of each string. These flanges are used to attach pulling heads, to cables and anchors for rough alignment, and to secure the alignment-welding fixture. After rough positioning, the alignment frame is lowered to the pipe on taut lines. Divers remove the pulling heads and attach the frame fixtures to the pipe. The hydraulically actuated system of the frame is used to position the pipe in the frame and to pull the pipe ends into a chamber for sealing, cleaning, and final alignment. Welding, coating, and inspection take place in a dry, controlled atmosphere. The welding and alignment fixture is also able to cut pipe and install valves. Figure 11-10 shows an arrangement for such a fixture.

11.4.3 Bottom Pull from an Offshore Work Platform

This configuration of the bottom-pull method is used where the entire offshore operation is conducted from three floating platforms or barges. One barge, the pulling barge, is used to pull the pipeline offshore. A second platform is made up of one or more barge units designed to operate near the shoreline in shallow water. This work platform is used for blasting, excavating, and making up the pipe joints. A third utility barge is used in mounting a crane for offshore excavation and for pipe handling.

With the work platform and utility barge in position, the utility barge crane supplies pipe lengths to the work platform for welding and field jointing. The fabricated pipe length on the work platform is pulled into the sea by the bottom-pull method. Another pipe joint is set into position on the work platform by the crane. It is welded, field joint coated, and the pull cycle repeated. Figures 11-11 and 11-12 show various barge unit configurations used with this method.

Fig. 11-10. Underwater Welding and Alignment Fixture



11.5 Surface-Pull (Flotation) Method

The pipe is assembled in long sections and buoyancy pontoons are attached on a launchway parallel to the direction of the assembly. Each string is pulled into the water and towed into position as a floating unit. The tie-in barge, which holds the offshore end of a previous string, makes the connection. Then the pontoons are released except for those near the end of the completed pipe string. Another string is pulled into the water, floated to the site, and connected. The process continues with the remaining pipe strings. The method is illustrated in Figure 11-13.

This method can be hazardous even in moderate seas because the pipe can oscillate even under small wave conditions. Currents tend to push the pipe off-line. For these reasons this method is generally used only in protected waters. Since the pipe hangs between pontoons, large pipe stresses can be built up if this spacing is too great, especially during lowering operations.

In a variation on the above method, the sections are joined onshore so that the need for a tie-in barge is eliminated.

11.6 Remote Assembly Methods

Remote assembly means that the pipe sections are joined together in a location away from the job site. In some cases this may involve joining the entire line together at a remote work site, floating it into place over the route and sinking the line to a prepared seabed. This is applicable to short outfalls of less than 300 m (1,000 ft). It requires calm weather and currents of less than one knot.

The reel barge (Figure 11-14) is suited to small lines greater than 6 miles (10 km) in length. The pipe is plastically deformed for storage on a large reel or spool. The remaining steps are the same as those used with a lay barge except that the pipe is unwound from the reel in a continuous, uninterrupted fashion. Holding tension on the pipeline is applied to the reel mechanism as required.

This approach is attractive because:

1. Work stoppages due to weather are practically eliminated onshore and greatly minimized offshore.

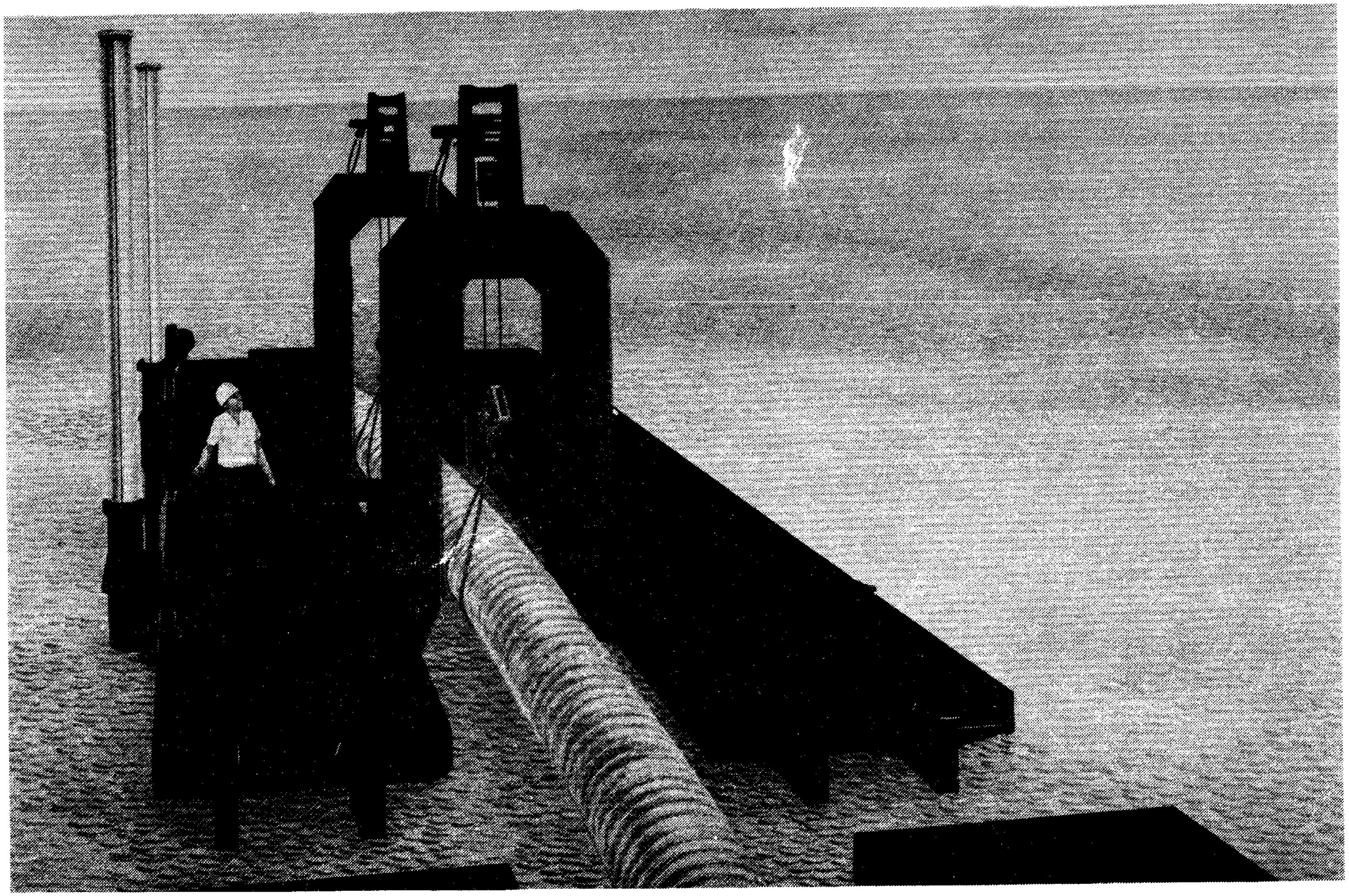
2. Capital investment can be reduced.

3. Welds and protective coating of long lengths of pipeline can be fully tested onshore.

4. Large volumes of pipe can be transported and handled with ease.

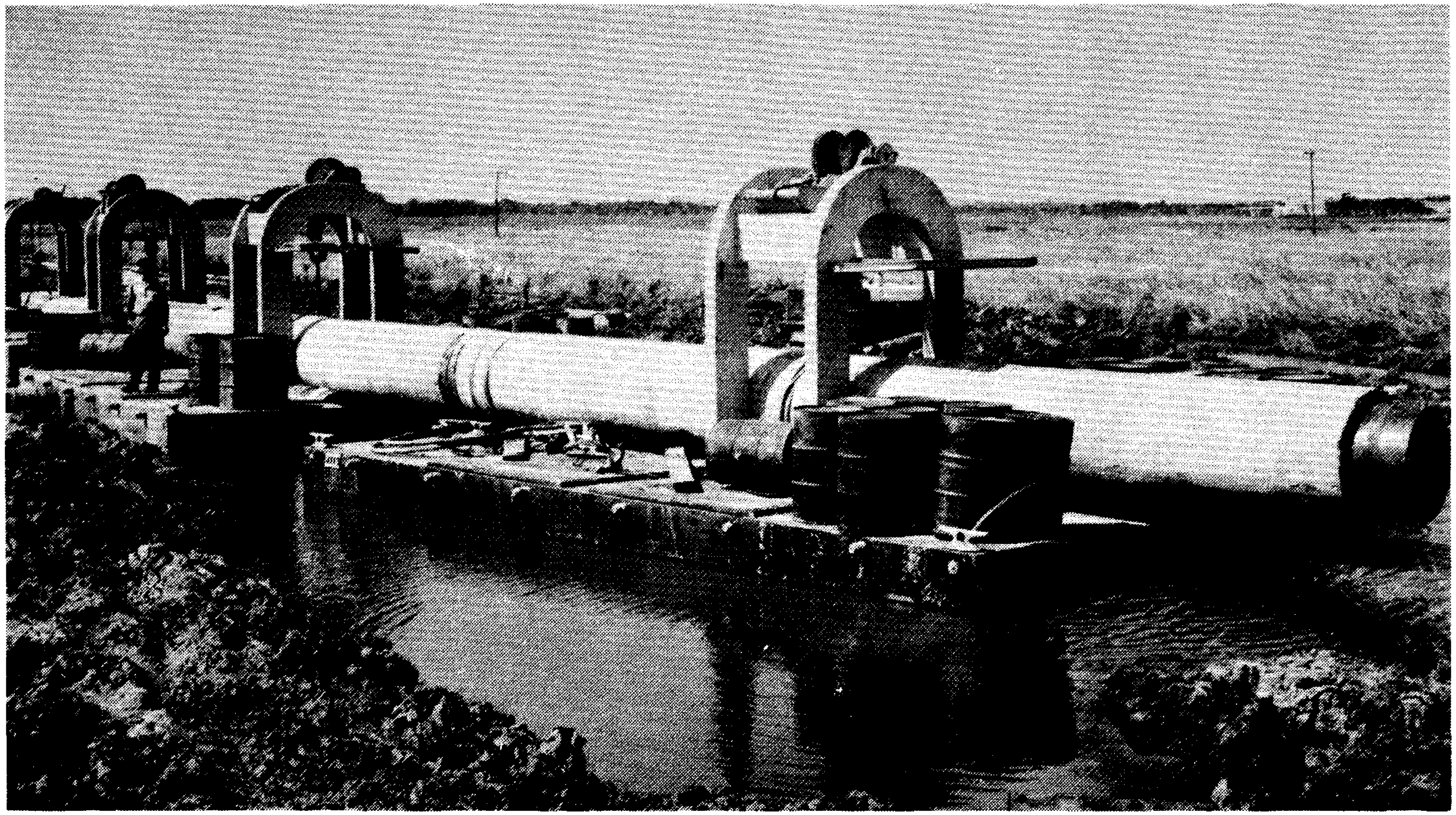
5. Very high laying speed can be achieved (1,500 to 3,000 m or 5,000 to 10,000 ft per hour) with steel pipe.

Fig. 11-11. Flexifloat Platform



Source: Flexifloat Systems Catalog, Robishaw Engineering, Inc., Houston, Texas.

Fig. 11-12. Floating Platform



Source: Flexifloat Systems Catalog, Robishaw Engineering, Inc., Houston, Texas.

Fig. 11-13. Flotation Method

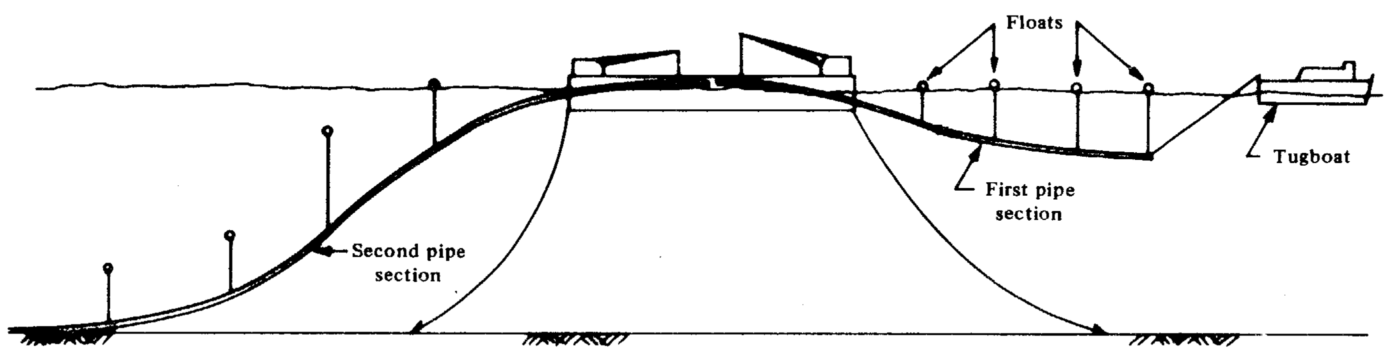
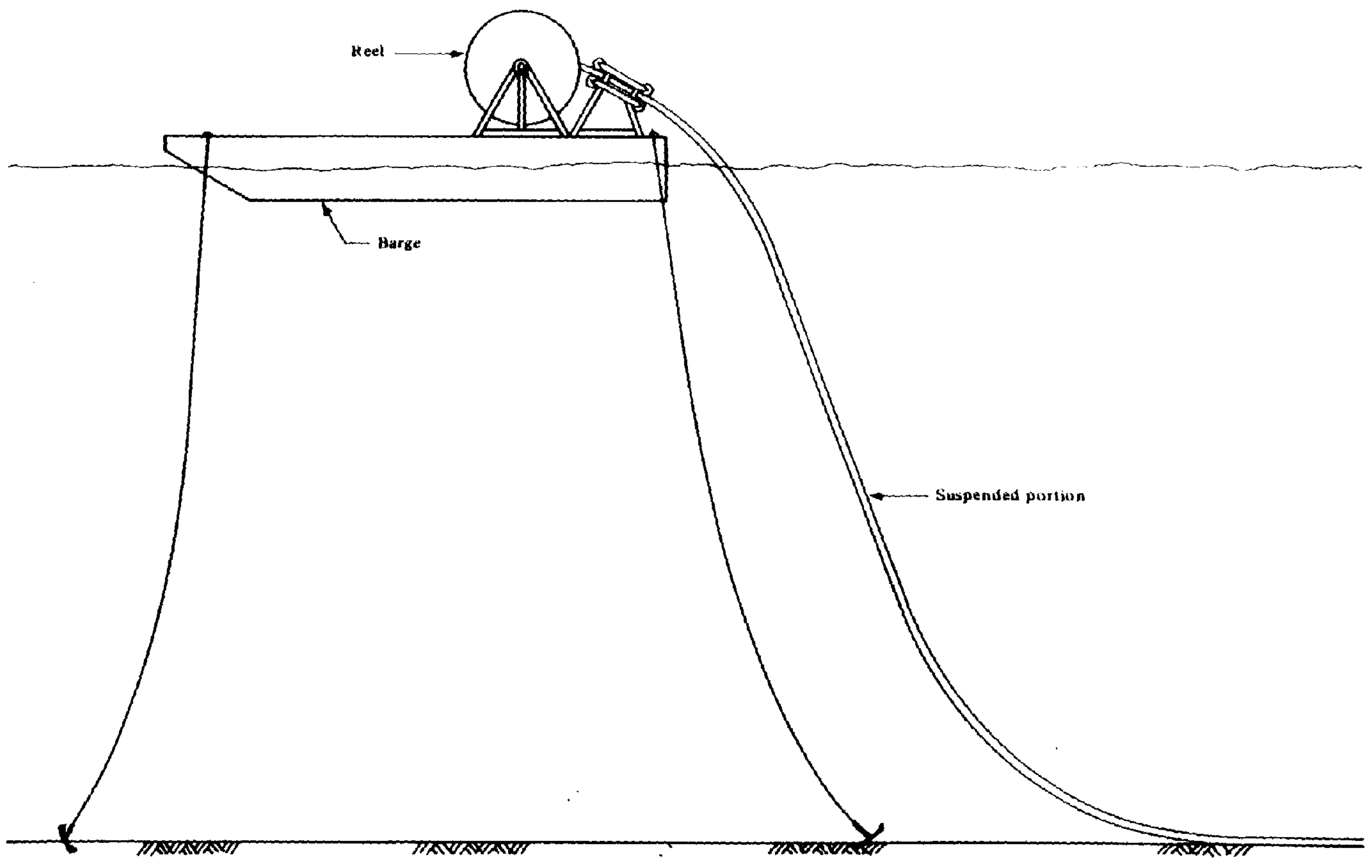


Fig. 11-14. Real Barge Method



Steel pipe laid with the reel-barge method is normally coated with either a polyethylene or epoxy coating. Heavy wall pipe is normally used to attain sufficient negative pipe buoyancy with this technique.

11.7 Directional Drilling (Boring) Method

A relatively new and innovative concept for a directionally controlled, near-horizontal drilling process has been developed by Titan Contractor Corporation. Although the largest market for this technique is in the installation of river crossings, drilled installation of outfalls up to a length of 3,000 ft (1,000 m) is technically feasible.

A specifically designed drill rig is disassembled into several components for highway transport and reassembled on the site. The heart of the rig is a self-contained hydraulic power unit that travels up and down a ramp, the slope of which is determined by the pipeline profile.

The first stage in construction is the drilling of a small-diameter pilot hole beneath the outfall route, following the preplanned course as closely as possible. The drill bit is powered by an in-hole hydraulic motor attached to the end of a nonrotating drill string. The drill string is composed of 30 ft (10 m) lengths of lightweight threaded drill pipe. The in-hole hydraulic motor is attached to a curved section of pipe called a bent sub. This unique feature of bit rotation without pipe rotation and the bent sub makes it possible to achieve the directional control required to produce a curved hole.

The drill bit progresses downhole and curves in the direction of the bend. The drilling rate is a function of the composition of the material through which the drill is passing, the amount of thrust applied to the drill string, and the speed of rotation of the motor.

A pipe spread is set up on the shore site and the pipeline is welded together. If space permits, the entire line is preassembled into one continuous unit; more often, it is made up into several long sections. In either case, the preassembly process facilitates pressure testing, a procedure normally conducted while the pilot hole is being drilled.

After the pilot hole has be completed, it is enlarged by a full-size bit running just in front of the leading edge of the pipeline. The entire assembly is actually jacked or pushed from the shore site. Several roller stands are set up along the centerline of the outfall route to support the pipeline as it is being pushed into the hole. This reduces friction and protects the pipe from being damaged.

11.8 Reference

1. Titan Contractor Corporation, Houston, 1979. Personal communication.

CHAPTER 12

TRENCHING AND BACKFILLING

12.1 General

Marine outfalls may be trenched and buried for almost their entire length or only throughout the shore approach, or they may be unburied for their entire length. This choice depends on many factors including:

- Local regulations for controlling and protecting the shoreline.
- Esthetic considerations (for instance, a scenic beach that would be adversely affected by an exposed pipeline).
- Fishing activities that might be adversely affected (for instance, bottom nets or trawls might hang or snag on an exposed pipeline).
- Heavy breaking surf that may cause bottom instability and damage to an exposed outfall line.
- Danger of damage to an exposed pipeline by wave-borne debris or potential damage from anchors of ships or barges during severe storms.
- Uneven terrain that requires trenching to maintain a uniform alignment in the pipeline.
- Construction method and pipe material selection constraints.

The best practice is to bury the outfall through the surf zone to protect it from damage. In this way the shoreline can be returned to near its original condition and use.

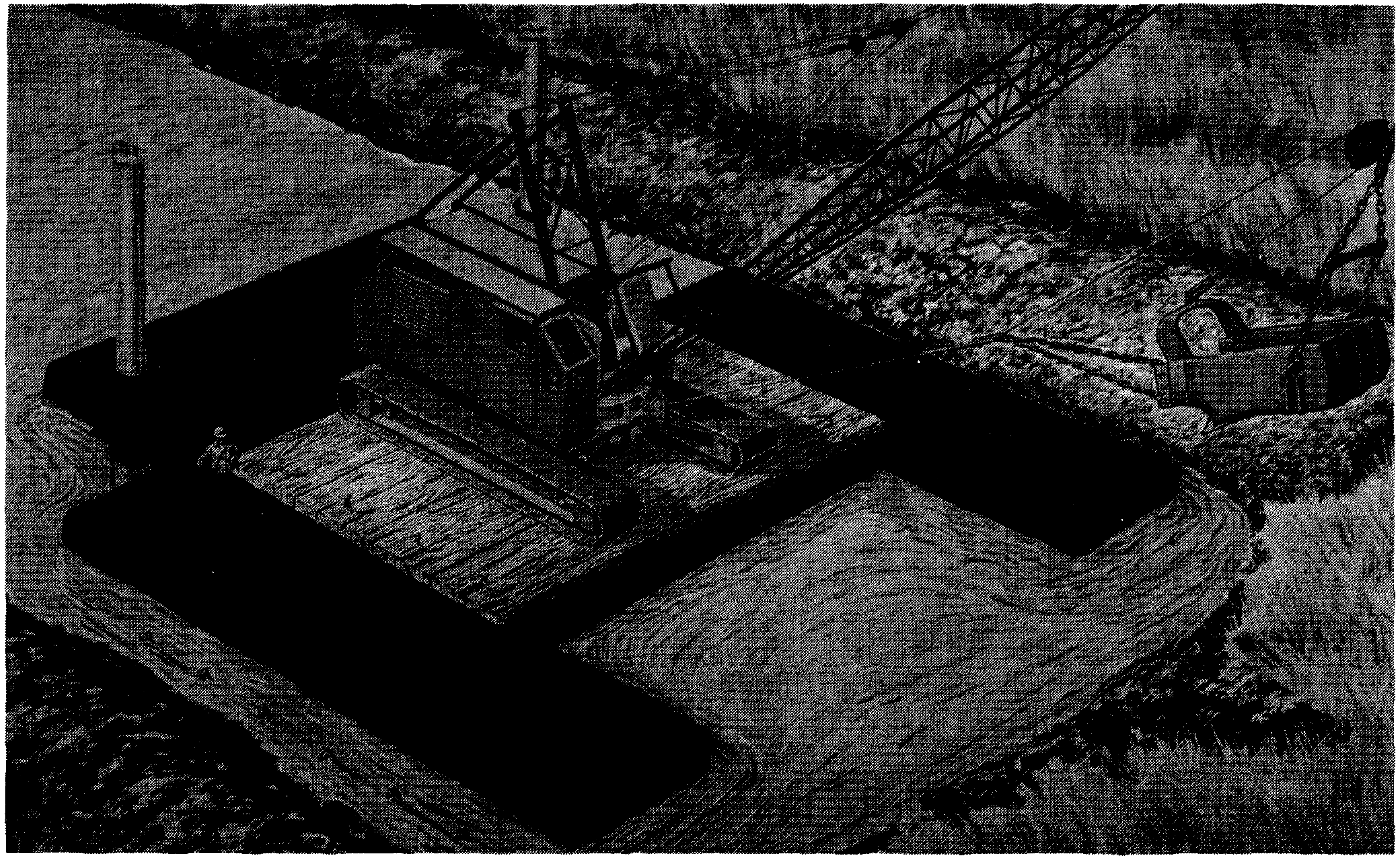
At the discharge, trenched and buried outfalls emerge from the bottom just upstream from the end of the diffuser section. The exposed end must be designed to be stable under the design storm-current conditions.

12.2 Trenching Methods

Trenching can be done either before or after the pipe is laid. The installation method, joint connections, and the pipe material all have an effect on the choice of trenching and pipeline burial methods.

Pre-installation trenches can be dug by explosives, dredges, excavation between two parallel rows of sheet piling, and ploughs. It is usually necessary to blast through rock or coral, either by drill hole explosives or by using shaped explosive charges. Shaped charges require no drill holes and direct most of the energy into the rock. After the initial

Fig. 12-1. Dragline Dredge



Source: Flexifloat Systems Catalog, Robishaw Engineering, Inc., Houston, Texas.

explosives have fractured the rock along the ditch line, a string of explosives or "bangalore torpedos" is used to clean out the ditch.

Bangalore torpedo strings also have been used to excavate trenches in silts, clays, and sands. A problem with blasting or with trenches excavated prior to pipe laying is that the sediment is disturbed, so that if the sea is turbulent (as in a surf zone), much of the sediment settles back into the trench before pipe installation is finished.

Dredges used offshore include bucket-type dredges such as dippers, draglines (Figure 12-1), clamshell dredges and continuous mechanical dredges such as the bucketline dredge. Continuous-type dredges are seldom suitable for outfall line trenching because of the set-up time required and the fact that they cannot work nearshore. Hydraulic or suction dredges (Figure 12-2) include hopper dredges and cutterhead dredges; their efficiency is significantly reduced if large rocks are present. Suction dredges operate best on loose material, while bucket dredges are more effective in consolidated soils. Unless the trench is being excavated between parallel sheet piling, side slopes on most excavated trenches vary from a slope of 1:5 to 1:20 so that large quantities of soil must be moved and deposited elsewhere.

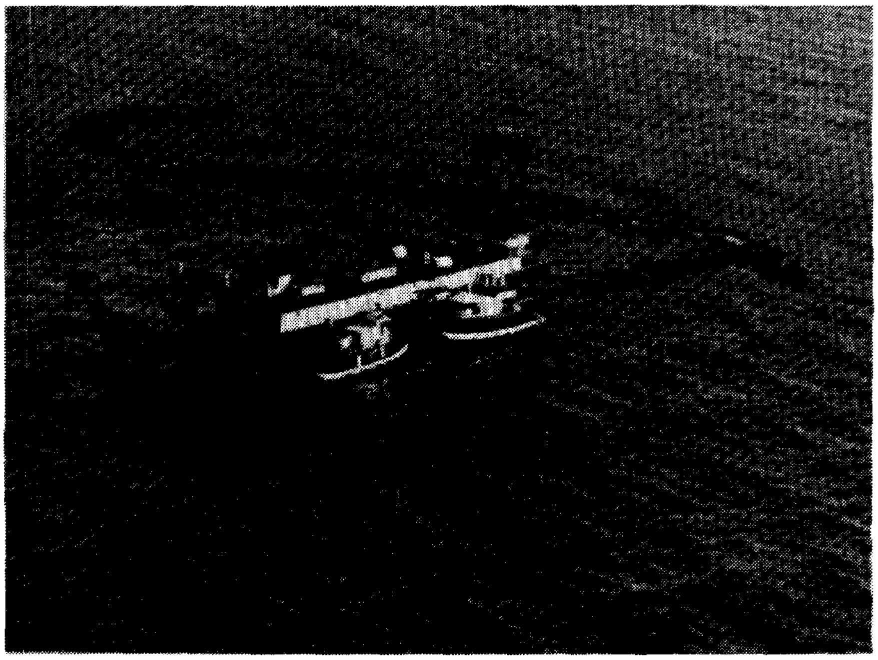
Large towed ploughs have been used offshore in Australia and in the North Sea for postinstallation trenching of oil and gas lines. These reportedly worked well in both sands and clays (1). Alignment control, (that is, providing a straight ditch) may be a significant problem because the plow will follow the line of least resistance. If sufficiently hard pockets of soil are encountered, the plow will shift laterally.

Postinstallation trenching works best when the pipe material is steel with welded joints. It is not used when rock is present. The most common postinstallation burial device is a "jet-sled" with high velocity (and pressure) jets that can loosen the soil immediately under the pipeline. Air lifts or water eductors (vacuum devices) lift and remove the loosened soil and spread it on each side of the ditch. Jets and eductors are mounted on the sled that is pulled along the pipeline by a surface barge that supports the large pumps and prime movers required to operate the water jets. As the sled is pulled along the pipeline, the pipe sags or settles into the trench behind it. Figure 12-3 shows a jet-sled in operation.

Rollers guide the sled along the outfall during operation. Sensors on the rollers relay signals to the barge so that the alignment of the sled can be constantly monitored and corrected when necessary to avoid undue stresses on the pipe and coating.

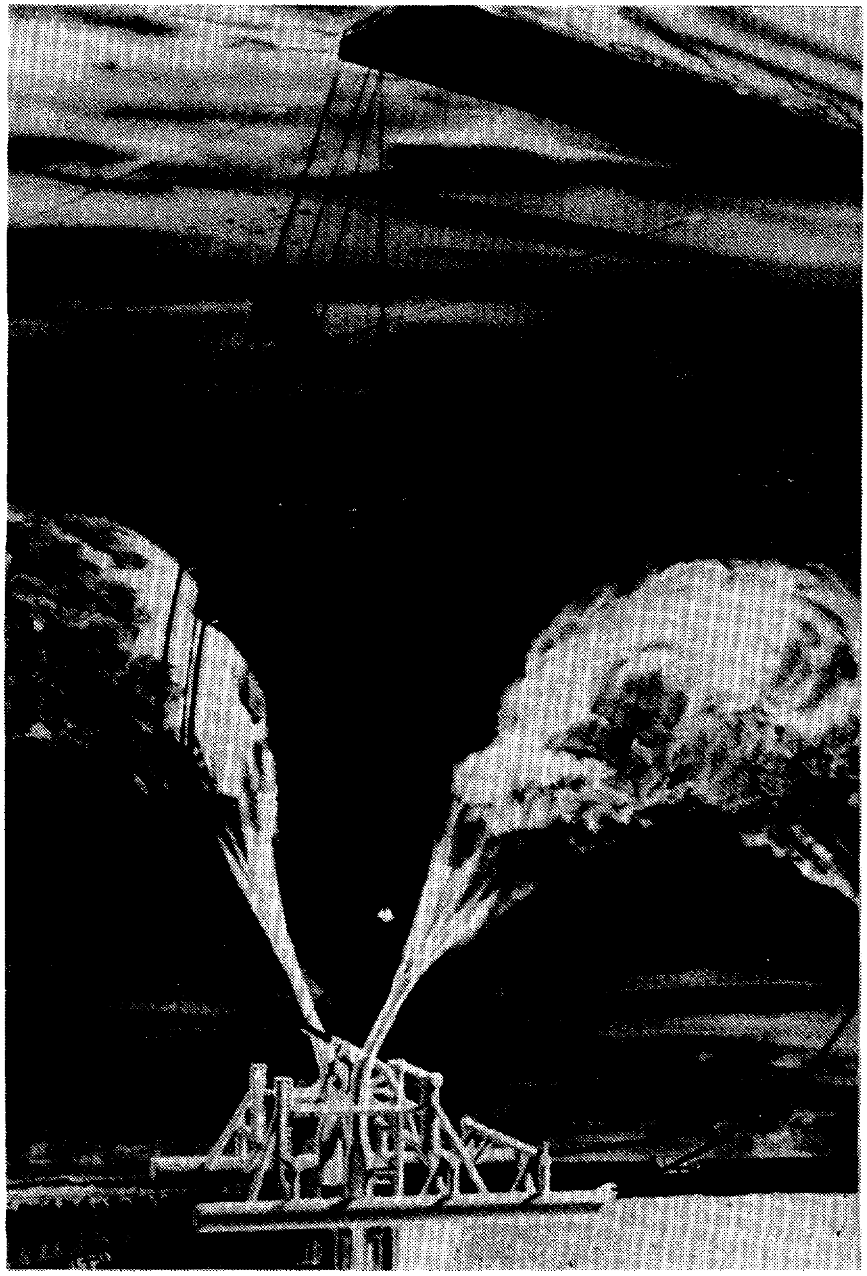
Jet-sleds work best in silts and medium clays in favorable soil conditions. They can excavate as much as 2 m (7 ft) deep on a single pass. In hardened formations or for deep burial, additional passes are necessary. Because of pressure losses in the hose and water back pressure, this method is less efficient at depths greater than 100 m (300 ft), although there are reports of eductor systems working at 300 m (1,000 ft).

Fig. 12-2. Hydraulic Dredge



Source: Dredging Brochure, C.F. Bean Corporation, New Orleans, Louisiana.

Figure 12-3. Jet Sled in Operation



Another postinstallation trenching machine uses dual hydraulic-powered cutterhead dredges mounted on a sled that travels along the pipe similar to the jet-sled. This machine requires much less energy to obtain the same depth. Additional advantages include a cleaner ditch, less soil disturbance, ability to handle a wider range of soil conditions, less support equipment, and unlimited water depth. Some cutterhead sleds have been equipped with self-propulsion devices which reduce the possibility of damage to the pipeline.

12.3 Backfilling Methods

Backfilling requirements are site-specific. Where there are persistent longshore currents, the trench will backfill itself. In most cases, it is best to include backfill in the construction contract. High current velocities require gravel, rock, or riprap. For example, one outfall in an area with design currents of 3 m (11 ft) per second was backfilled with 5.1 cm (2 in) crushed stone to a height of 0.6 m (2 ft), then the remainder of the ditch (0.3 to 1.0 m or 1 to 3 ft) was filled with 40 to 60 cm (16 to 24 in) riprap to prevent the lower backfill from being swept away. The placement method used should be determined before construction starts and should be done so as not to damage the pipe or coating.

If current velocities are less than 1 knot (0.5 m/sec) on the seabed, the excavated material may be flushed back into the ditch. In such cases the excavated spoil will have to be stockpiled on the seabed or on barges. Owing to the high costs of stockpiling, it is seldom practiced except in very shallow waters with no current.

Paragraph 6.3 describes pipeline failure due to jacking by wave or surf action and liquefaction of the bottom sediments. This can be avoided if the proper pipe weight, depth, backfilling, and armoring are selected.

12.4 Reference

1. Anon. 1977. "7200 Foot Loading Line Plowed in at Statfjord," Petroleum Engineer (September), p. 10.

CHAPTER 13

CONSTRUCTION MONITORING AND INSPECTION

The objective of construction monitoring and inspection is to ensure that the outfall is installed in accordance with the engineer's specifications and with good workmanship. This means that all of the construction contractor's activities must be inspected by the owner's project management team, often furnished by the consultant who designed the outfall. If the project is small, the project management team may only consist of a project manager/engineer and a craft inspector. Except for outfall sections constructed alongside a trestle or in a cofferdam, the project management team must have an inspection boat and access to qualified divers to inspect the completed outfall.

During the bidding phase, contractors should be placed on notice that the successful bidder will submit detailed construction (safety, welding, installation, etc.) procedures before beginning construction. The project management team is responsible for reviewing and either approving or rejecting these procedures before installation begins.

Pipe coating or other corrosion prevention measures are usually the first field activities requiring inspection. Inspectors are assigned to the yard to ensure that the surface is properly prepared for coating, and that the coating is installed in accordance with the engineering specifications and the coating materials manufacturer's recommendations. Battery-operated holiday detectors that measure the electrical resistance of the coating detect holes (or "holidays") in the coating are used to ensure coating integrity.

Concrete weight coating is also applied and monitored in advance of pipe installation. Application of the weight, including concrete thickness and pipeline weight per unit length, is very critical in some construction methods. Construction specifications should spell out the critical elements to be monitored during weight-coating application.

During preparation of the offshore route, which includes trenching and grading, it is necessary to monitor the route cross sections and profiles. A small boat with a fathometer is used to establish the original cross sections. After trenching begins, subsequent cross sections are made by the construction management team on the same tracks to check excavation progress. It is the team's responsibility to approve a final trench prior to pipe installation.

Construction management and inspection includes inspecting the welding (or other jointing), installation of pipe, backfilling, and cleanup. The number and specialities of the construction management team members depend on the pipe material, installation method, and magnitude of the project. In general, these include a project manager; project engineer to oversee the marine survey and trenching operations; craft inspectors to inspect coating, onshore preparation, and welding; and divers to make a trench inspection and a final inspection.

Certain specialized equipment is necessary, such as a holiday detector for coating inspection, fathometer, survey boat, tender boat for divers, and specialized equipment for radiographic joint inspection.

There is a tendency to rely on the contractor's divers and marine surveyors to reduce inspection costs. This is a poor practice because contractor employees and subcontractors are loyal to the contractor and can almost never be depended on to represent the best interests of the owner. Since the area of inspection is subsea, the only information that the project manager has for decisionmaking comes from diver reports and instrument interpretations. It follows that this information should be gathered by individuals representing and obligated to the outfall owner.

Other more sophisticated inspection techniques such as video tape records and inspection with a submersible are available for large deep water outfall projects. Factors to consider in outfall inspection that affect diving time include water visibility, support equipment required for the submersible, mobilization time, and comparative costs.

Although both owners and contractors benefit by minimizing construction time, there are unique incentives. In one celebrated case that occurred when the storm season was at hand, friction during a bottom pull approached the theoretical limit of the winch and cable system. The contractor elected to push on the shore end of the pipe with a bulldozer. It didn't work. The pipe buckled and broke. Much additional time was lost.

CHAPTER 14

OCEAN OUTFALL PERFORMANCE MONITORING

Ocean and estuarine monitoring data provide descriptive information for research and, when combined with operational data, are used for the functional design of waste disposal facilities and operations. Monitoring data are time-series observations used for research, prediction, assessment, and/or control. However, the high costs of monitoring require that priority uses of the information be rigorously established in advance and periodically reviewed.

14.1 Performance Monitoring Principles

The operational definition of performance monitoring of an ocean outfall is that which provides the minimum information to be used for (1) operating an existing facility, (2) managing commercial or recreational fisheries or tourist resources in the vicinity of the waste discharge, (3) evaluating compliance with guidelines, criteria, or standards, or (4) modifying or expanding the waste disposal facilities. Each set of uses has characteristic response times that establish the frequency with which useful monitoring data are obtained. When and where continuous monitoring of tidal estuaries is warranted, data should be collected at 2- or 3-hour intervals. Shorter-interval sampling will not provide additional information, and 6-hour or greater intervals will provide spurious data. These and selected quantitative aspects of time series, particularly those with tidal components, are presented in the addendum to this chapter.

Monitoring tends to create large data-gathering programs that are difficult to change or discontinue even if the data are not found to be useful (13). This inertia, which results in continuing collection of unused or unusable data, increases with the amount of money invested in monitoring hardware and facilities, the number of careers involved, the length of record, and the uncertainty as to how the data will be used.

For effective monitoring, sampling intervals, and interpretation of time series, data must depend upon the nature of the variance. Periodic events in tidal estuaries, bays, inlets, and some straits are due to semidiurnal (lunar), diurnal (solar), fortnightly (lunar), and annual (solar) events (7). Diurnal land-sea breezes affect the distributions of nearshore surface water movements and properties (24). As noted above, filtering of (Eulerian) measurements taken at fixed stations and at intervals of 1.67 to 6.00 minutes in tidal estuaries revealed that (Lagrangian) estimates of advection and other properties related to 6-hour changes in current direction could have been determined from observations taken at 2- to 3-hour intervals (13). Alternatively, measurements taken essentially simultaneously at a single, lower-low water slack are used to measure longitudinal distributions of estuarine pollution when tidally induced temporal changes are unimportant (8).

Optimum sampling, digitizing, averaging (which may lose some of the information), or otherwise filtering of a time series for descriptive or predictive models can be based on the spectral analysis approach presented in the addendum to this chapter, or other assessment approaches, including single or multiple regression analysis or probability distributions. In general, spectral and cross-spectral analyses appear to be most useful. Similar optimizing techniques can be applied to space fields or sampling station locations (see Appendix B).

Navigational requirements for sampling stations depend upon the utilization of the data. Position accuracies for representative optical and microwave systems are listed in Table 14-1.

TABLE 14-1

Navigation System Accuracies

Sextants	10 m over 5 km
Theodolites/lase	5 m over 5 km
Artemis III	16 m over 30 km
Motorola	4 m over 30 km
Decca transponder	4 m over 30 km
Hydrodist	1.5 m over 30 km
Decca Main Chain	100 m over unlimited area

Source: Willis (41).

The choice of optimum sampling station locations and the issue of replication of biological samples are closely related considerations in monitoring program design. Both numbers and associations of individual species tend to vary significantly over short distances. Accordingly, many ecologists, regulatory agency officials and their contractors (38), and attorneys representing dischargers insist on three, five, or even more, closely spaced replicates from the same water mass or benthic sampling station to impute statistical elegance. However, the issue has been shown by Bascom (2, 3) and Word et al. (43) to be irrelevant when the objective of the sampling is to determine the areal effects of a discharge. Both found that more information for the same effort was obtained by increasing the number of sampling stations rather than replicating samples from fewer stations. In any event, optimum design of monitoring programs with respect to sample replication is facilitated by appropriate analysis of variance analogous to that for time series (Appendix B).

Monitoring program design includes selection of appropriate parameters for measurement and sampling station locations. In general, these selections are based upon environmental and sanitary surveys of the area that reveal locations and probable characteristics of waste discharges and their effects upon the uses of the receiving waters. Other sources of information include university engineering, physical science, biology, and geography departments; government agencies; ferry pilots; fishermen; and residents. New releases from new or expanded industries that include characteristic wastes (e.g., mercury from chlor-alkali plants). Occasionally, a preproject outfall survey or an academic research project will identify a specific need for monitoring (e.g., the reported finding in the muds of Manila Bay (see Chapter 15) of increasing concentrations of dieldrin, a pesticide long since proscribed in the United States because of its persistence and toxicity).

Environmental monitoring data are easier to collect than to analyze. Excess data are often justified because (1) their marginal cost is low, (2) another agency (or country) is collecting them, (3) "We've always collected them," (4) the data provide management and funding officials with a simple quantitative (but not qualitative) measure of accomplishment, (5) large amounts of data confuse critics by their bulk, and (6) the data may be useful someday. However, these uncertain marginal benefits are rarely known to justify the marginal costs of excess data storage, retrieval, and analysis. A series of six regional marine pollution monitoring workshops in the United States revealed (1) little utilization of costly monitoring data, (2) a need to reallocate resources so that data assessment receives essentially the same funding as data collection and (3) a need for research to demonstrate that benefits derived from institutionalized analytical quality control justify its cost in environmental monitoring systems (4, 15, 19, 20, 33, 34, 35). The last need was identified since no instance could be cited where the presence of expensive state-of-the-art analytical quality control procedures had affected any operating or management decision.

Sample replication, institutionalized analytical quality control, and other approaches to scientific certainty can have academic value. With respect to operational monitoring, Kamlett (23) advised that "while accurate and precise data are obviously preferable to inaccurate and imprecise data, we do not require scientific certainty (if such is attainable) as a predicate for action. Reasonable trends, projections, and potentials are usually sufficient to allow us to respond in the administrative, legislative, and political spheres."

Careful selection of procedures and their calibration within the particular environment to be monitored are fundamentally important. For example, Rhodamine B, a dye measured by fluorometry and used to observe and model wastewater diffusion in coastal areas (28), works well in clear water. However, since it is also a very good dye, it adsorbs onto sediments suspended by tidal or other currents (12), and local calibration of the procedure is essential to determine the significance of dye losses to sediments. When the losses are unacceptable, another dye such as Rhodamine WT can be used.

14.2 Structural Monitoring

Evidence of actual or imminent structural or hydraulic failures of outfalls is provided by inspection by divers or submersibles, by excessive current requirements for cathodic protection systems (see Section 9.3), or by high operating pressures resulting from reduced cross sectional area. Most failures are due to storms, ship anchors, or heavy fishing gear that bend, move, or break the pipe (9, 10, 21, 40). Partial failures, which may go unnoticed from the surface, include cracking or spalling of concrete weight-coats caused by movement during storms or by anchors, loss of diffuser risers (9, 10, 40), gradual accumulation of grease in the line (21), or loss of anchor blocks near the outlet so that the pipe can move laterally and is difficult to locate for inspection (40). Breakages occurred in three of twenty-nine recently constructed British outfalls (10), two of them were in PVC pipe. The remaining steel or concrete pipes and the six plastic pipes functioned satisfactorily.

European and California experiences indicate that a minimum of one inspection annually by direct observation is needed, particularly on those outfalls extended beneath shipping lanes, anchorages, and trawling areas, or over shallow, broad continental shelf areas with heavy surf. Plastic pipe sections, which are weaker than steel sections, fail more frequently and therefore require more inspection and maintenance (9, 10).

14.3 Discharge Monitoring

The appropriate frequency and type of discharge monitoring depend upon how the data are used to control and schedule waste collection, treatment, and disposal operations. Monitoring information includes wastewater flows, average concentrations and mass loadings of selected dissolved and suspended constituents, and frequencies of extreme values related to spills or weather. Information needs increase with the complexity of the system. In some areas, seasonal or annual observations of nearby shoreline conditions may suffice to monitor impacts of raw, screened, or comminuted sewage. In contrast, daily end-of-pipe monitoring of chlorinated secondary effluent for BOD, volatile and total suspended solids, nitrogen and phosphorus, ether solubles, detergents, heavy metals, and selected hydrocarbons may in some sensitive areas be mandated, supplemented by hourly monitoring of chlorine residuals for operational control during bathing seasons.

14.4 Ecological Monitoring

In contrast to end-of-pipe monitoring where information can be acted upon quickly, ecological changes that take place only slowly in response to waste inputs do not provide a basis for routine discharge management. Responses to biological monitoring observations, such as remedial project identification, preparation, authorization, financing, construction, associated training, and initial operation, usually require many years. Other approaches include the occasionally obscure adaptive environmental assessment and management tools developed by C.S. Hollings and his associates (18), for application to biological monitoring and its management responses. His concepts provide for due regard to the scale and complexity of a particular problem and for the participation of the people who will ultimately have to pay for maintaining or improving their environment.

The urgency with which routine performance monitoring is expanded to research monitoring and to implementation of remedial measures will depend upon expert scientific, technological, financial, and economic appraisals. Local benefits, such as protection of a bathing beach or shellfishery, can be readily assessed. However, the regional implications of locally caused change, particularly those of multiple discharges and other inputs are more difficult to assess. These broader implications are examined by various regional, national, and bilateral or multilateral international organizations.

14.4.1 Public Health

Most outfall designs are based upon the need to satisfy bacteriological criteria in affected areas. However, untreated or conventionally treated discharges from industrial areas may also contain toxic materials that may accumulate in seafood. Bacteriological water quality of ocean bathing or diving waters has not been demonstrated to be a high priority public health concern (26, 31), even in industrial countries (see Sec. 3.2.1). Here pollution-caused disease is generally restricted to occasional gastrointestinal upsets in children and other non-immune populations and to minor ear, eye, nose, and upper respiratory complaints (5, 6, 16, 25). In contrast, aesthetic matters may be a very high-priority public concern, especially in tourist areas. Observations of these incidences in polluted coastal areas and extrapolated "risks" of infection have provided the basis for WHO, EEC, U.S., and other microbial standards for ocean bathing waters. In practice, designing and operating municipal waste systems to meet applicable microbiological receiving-water criteria normally also result in the discharge meeting aesthetic criteria.

Treatment plant failures can cause the stranding of fecal, plastic, and rubber materials on bathing beaches. In two highly publicized cases, one in California and one in New York, beach quarantines were established and, in due course, terminated with no regard whatever for the monitoring data that showed that microbiological quality standards were met throughout both incidents (11, 16, 37). These events demonstrate that daily microbiological sampling along surf-zone bathing waters is unnecessary; weekly or monthly sampling during critical seasons is sufficient to confirm satisfactory plant performance. Although it has been argued that increased microbiological surveillance should be scheduled during storm, heavy runoff, or other unusual periods, the marginal benefits of the additional data remain unexplored.

Public health considerations should include exposure time in the water, the general health of the discharging population, and, for tourist areas, the health of the visiting populations. For example, the United States in 1960 had one typhoid case per 22,500,000 people. Istanbul, Turkey, in 1968 had one case per 10,000 people (14). Conceptually, visitors from the United States swimming in the Sea of Marmara would be afforded protection from typhoid equivalent to that afforded swimmers in U.S. waters only by promulgating coliform standards for the Sea of Marmara that were 1/2250 of U.S. standards. Corresponding ratios could be developed for hepatitis and other waterborne diseases. However, comparison of Cabelli's (6) and Hakim's (17) results on gastroenteritis in the United States and Egypt, using enterococci as an indicator, does not indicate that such an effort would provide a worthwhile improvement in public health protection. Fortunately for swimmers (but not for eaters), gastroenteritis and similar infections of foreign tourists are predictably derived from food rather than swimming.

In any event, microbiological monitoring data are assessed against local guidelines, criteria, or standards. Recreational water quality standards based on fecal indicators vary from nation to nation, and even within a country. A national standard of 200 fecal coliforms/100 ml has been recommended for the United States. Another set of bacterial standards for bathing waters as proposed bv the European Economic Community (effective in 1985) is listed in Table 14-2. On a worldwide basis, the World Health Organization has determined that "highly satisfactory bathing areas should show E. coli counts of consistently less than 100 CFU (colony forming units)-per 100 milliliters and to be considered acceptable bathing waters should not give counts consistently greater than 1000 E. coli per 100 milliliters" (44). The lack of consensus in setting bacteriological standards presumably reflects differences in immunological, environmental, epidemiological, and economic determinants of disease in different places.

TABLE 14-2

EEC Bacteriological Standards for Bathing Waters

Parameter	Guideline (% of samples)	Limit (%)	Sampling Frequency
Total coliforms/100 ml	500 (80)	10,000 (95)	every 2 weeks
Fecal coliforms/100 ml	100 (80)	2,000 (95)	every 2 weeks
Fecal streptococci/100 ml	100 (90)	-	daily

Note: limiting standards for salmonellae and enteroviruses have also been proposed. The daily samples for fecal streptococci are to be taken on evidence of deterioration of water quality.

Source: Official Journal of the European Communities, No. L31/L S.2.76, Brussels.

Contaminated shellfish, usually oysters, clams, or mussels, eaten raw or undercooked, cause typhoid and paratyphoid fevers, infectious hepatitis, cholera, and a variety of other gastrointestinal illnesses. Contributing factors include the following: (i) shellfish are often harvested illegally from quarantined waters; (ii) shellfish concentrate microorganisms from their environment, often by 5- to 10-fold; (iii) people ingest perhaps 150 grams of shellfish at a sitting, almost certainly more than the salt water they would swallow during a swim; and (iv) contaminated particulates tend to settle to the seafloor where survival rates may be greater (32) so that pathogens are readily available for uptake by the shellfish.

The World Health Organization has suggested that, wherever possible, molluscan shellfish resources should be harvested only from those areas where the product is acceptable for human consumption without further treatment (44). Otherwise shellfish should be boiled, steamed, or depurated (maintained in clean water for several days after harvesting to allow the shellfish to eliminate any pathogens).

Paralytic shellfish poisoning (PSP) is occasionally caused by blooms or "red tides" of toxic dinoflagellates that result in shellfish contamination. These blooms are limited geographically, and although their seasonal preference is known, the years of their outbreaks are unpredictable. For frequently affected areas, routine assays using mice as indicators may be appropriate to warn communities of potential PSP outbreaks and to suspend shellfishing.

Pathogens in finfish caught in marine or estuarine waters are not routinely monitored. Most of the fish will be cooked and the pathogens destroyed. Fish to be eaten raw are selected for firmness and taste and ordinarily come from offshore waaters. In contrast, monitoring for toxic hydrocarbons in the North American Great Lakes and other areas has resulted in closure of waters to fishing as a precautionary measure. This has been supported by findings of PCB's (which may be considered as indicators of dioxins and furans) in New Jersey finfish (46), Wisconsin human body fluids (47), and teratogenic responses to consumption of Lake Michigan finfish (45). An example of the costs of such precautionary measures is provided by the 1969 United States Food and Drug Administration's mercury standards for fish. Although this standard was a response to exaggerated public furor in the late 1950s over methyl mercury being the cause of Minamata disease (22), subsequent research revealed that the high mercury levels in tuna that led to its prohibition were, in fact, natural and that consumption of the fish imposed no danger (29).

14.4.2 Ecological Interactions

Monitoring of ecological interactions is designed to protect the long-term productivity of marine and estuarine areas from the effects of waste discharges. These effects are most readily revealed in animals that live with or are attached to the bottom sediments and that accumulate waste constituents slowly over time. Since ecological changes take place slowly in response to waste inputs, data from benthic stations sampled over periods of several weeks are considered synoptic. After a baseline survey, areal sampling may be conducted at 5- to 10-year intervals and supplemented by surveys immediately preceding and one year following major structural or other technological changes.

A number of indices have been designed to reflect environmental degradation. The diversity of benthic species is usually reduced in the area immediately surrounding a discharge after a period of continuous discharge, whereas total numbers of adaptive organisms and biomass increase (3, 30). Similarly, where fish may have been scarce or absent prior to a discharge, substantial populations of finfish may be attracted to the discharge as a food source. Environmental effects also depend upon the nature of the discharge. Monitoring for diversity and abundance of biota alone may not provide information on some subtle ecosystem alterations. Determination of community structure and of sublethal effects on reproduction, feeding, growth, endocrine functions, and tissue pathology requires extensive benthic, sediment, and fish sampling; water column chemistry; and determination of toxic substances in selected organisms.

The three environmental characteristics associated with waste discharges that have been demonstrated to have the greatest influence on the benthic community are (1) the concentration and biological availability of organic particulates in the waste, (2) the velocities of the bottom currents, and (3) the settling rates of particulates through the water column (43). Virtually all pollutants found in discharged wastes are attached to particles that eventually settle to the seafloor.

Benthic sampling and monitoring programs can vary greatly in complexity and cost. Generally, samples of the macrobenthos (> 1 mm) are taken because analyses of smaller benthic organisms are more costly and their community structure is less well-known. Conditions should be maintained as uniformly as possible when sampling, since variables such as depth and time of year can significantly affect the results. The sampling device should sample an effective area of the sediment (0.1 m^2 is suggested) and should penetrate at least 5-10 cm into the sediment, except for rocky bottoms where this is not possible. The sample should reach the deck of the boat undisturbed, and at least two subsamples (one for biological examination and the other for chemical analysis) should be taken. Samples should always be sorted with the same screen size (1.0 mm suggested) and preserved on the day of collection. Replicate samples from a particular location may be advocated for special statistical research purposes.

Although pollution-tolerant species may serve as general indicators of the extent of contamination in the sediments, they do not offer a sensitive measure of change. The Infaunal Trophic Index (ITI), used in conjunction with other physical and chemical data, promises to determine accurately the true extent of effects from organic-rich wastes (42), although it is probably too costly to be applied to more than a few discharges that enter sensitive areas. The ITI was designed to examine feeding strategies of benthic species off the west coast of the United States, but can be employed elsewhere by using local indigenous species at different trophic levels. By examining the percentage of organisms using either suspended material or deposited material as a food source, an indication of the amount of particulate organic material present in the benthic environment can be obtained. Organisms are placed in one of four groups:

Group I - suspension feeding organisms,
Group II - organisms feeding on suspended material or detritus on the sediment surface,
Group III - organisms feeding on surface detritus, and
Group IV - organisms feeding on detritus below the surface.

The individuals in each group are counted and put into a simple formula to derive a value of 0-100 that indicates the response of the benthic community to pollution inputs:

$$(14.1) \qquad ITI = 100 - \left[33 - 1/3 \; \frac{(0n_1 + 1n_2 + 2n_3 + 3n_4)}{n1 + n2 + n3 + n4}\right]$$

where n = the number of individuals in the group and coefficients in numerator are an arbitrary, evenly increasing scaling factor.

The ITI values can then be lumped into groups that reflect the type of organism that dominates. Group I organisms dominate in a community with an ITI value of 78-100, Group II's dominate ITI's of 58-77; Group III from 25-57; and Group IV from 0-24. Group I organisms predominate in least-affected areas where benthic oxygen demand is low. Conversely, Group IV organisms dominate in areas that are most impacted by organic material in the sediments or that have a high concentration of hydrogen sulfide. The dominance of a particular group indicates the degree to which the normal sediments and their associated biota have been altered. These groupings can then be plotted on a bathymetric chart to give a graphic representation of the range of contamination (see Chapter 15).

Total organic carbon or total volatile solids in the sediments generally increase substantially if there is accumulation of sewage-derived material in the sediments near an outfall. However, in some areas the background level of organic matter in the sediments is naturally equally high.

The size distribution of sediment particles, total organic carbon, total volatile solids, and occasionally other variables may be measured as indicators of the presence of sewage organic particles and associated toxic contaminants. Beyond this, measurements of these parameters serve little purpose. Chemical analyses of sediment samples are costly and many routine analyses are not warranted. Therefore, analyses should not be performed for those contaminants that are known to exist in insignificant concentrations in the waste discharge. Further, a frequency of once every 5 to 10 years is generally adequate for analyses of those contaminants measured.

Fish are mobile and their community structure is complex and dynamic. Natural variations, daily vertical migrations through the water column, and seasonal migrations make monitoring strategies for fish especially difficult, and a large number of samples are necessary. Furthermore, selecting the appropriate equipment for sampling is difficult, since each gear type is designed to operate at a specific depth in the water column and since healthy fish ordinarily avoid many types of gear. Depending upon the depth and species to be sampled, samples can be taken by otter trawls, gill and trammel nets, various seines, traps, submersibles, divers, and hook-and-line methods. Length, weight, and ageclass should be measured for the various species collected. Histopathological examinations of fish that exhibit tissue abnormalities can provide supplemental information on population structure and health. Interpretation of fish monitoring results must include consideration of commercial and, occasionally, sport fishing in the area and of any bias due to the relative ease of netting disproportionate numbers of disabled or sick fish. Because of the difficulty in identifying changes resulting from sewage input within the context of a much larger natural variability, fish population monitoring is usually ineffective and generally should not be performed.

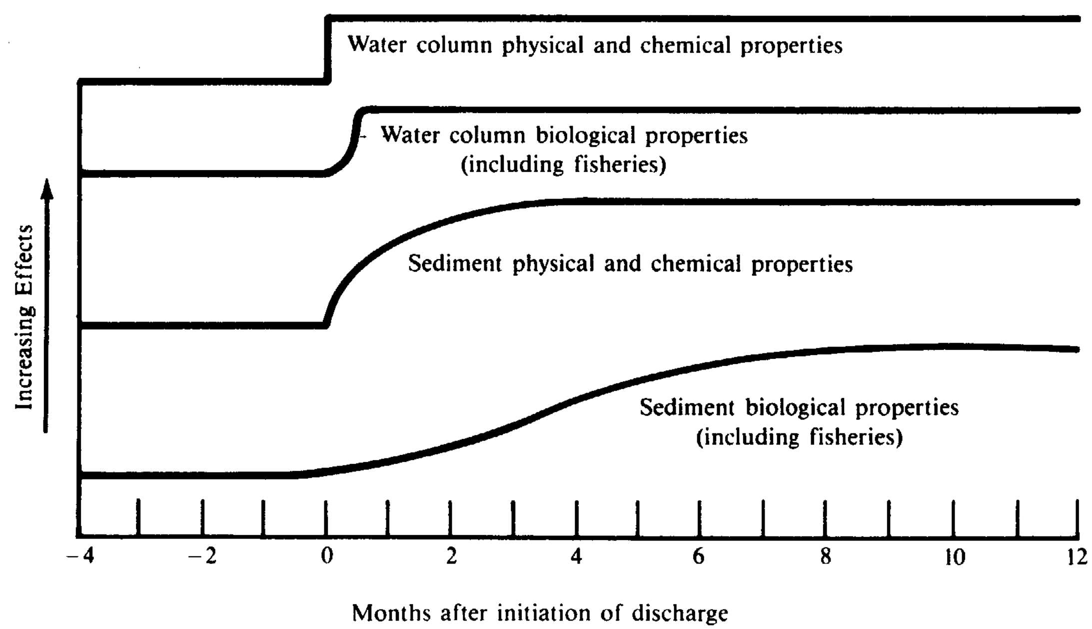
Phytoplankton populations are transient and subject to water movements and seasonal cycles. Phytoplankton monitoring for operational purposes is yet to be justified. Many previous studies based on first principles have assumed measurable effects of waste discharges on phytoplankton populations in the ocean; these studies have uniformly failed to reveal any adverse effects directly attributable to waste discharges. Where circulation is restricted, information on phytoplankton populations may identify eutrophication caused by nutrient enrichment of receiving waters, which stimulates plankton growth and possibly leads to depletion of oxygen. Similarly, field studies have revealed no adverse effect on zooplankton resulting from sewage discharges. For research purposes, chlorophyll may be measured as an indirect indicator of the standing stock of phytoplankton.

Since bivalves characteristically concentrate marine pollutants, they are frequently used as "sentinel organisms" to warn of potentially hazardous levels of various contaminants. The bivalves are placed in a cage, or similar device, at various locations in the discharge area and also at uncontaminated control stations. They are analyzed on a regular basis for contaminant levels and toxic effects and for comparison with data obtained from similar regional or international activities, such as Mussel Watch (27) and the Mediterranean Action Plan (39).

14.4.3 Equilibrium Response Times and Monitoring Design

When a new discharge to the ocean or an estuary is initiated, the physics, chemistry, and biology of the discharge region will be altered. Monitoring is usually intended to identify and measure these changes over time. It is often concluded that monitoring of a specific discharge must continue for the lifetime of the discharge or at least for many years. However, this conclusion is based on the erroneous assumption that physical, chemical, and biological changes caused by the discharge are progressive and/or cumulative over periods of several decades or more. In practice, once a discharge is initiated, changes will take place over a limited period of time during which the discharge ecosystem adjusts to a new dynamic equilibrium state with respect to the discharge. Once this new dynamic equilibrium is reached, no further changes can or will take place unless (i) the quantity or nature of the discharge is substantially altered, (ii) the area is subject to the influence of other contaminant inputs which are changed, or (iii) the ocean climate, that is best measured by other than operational performance monitoring, changes so that natural shifts in the equilibrium occur. The

Fig. 14-1. Model of Marine Environmental Response to Continuous Waste Discharges



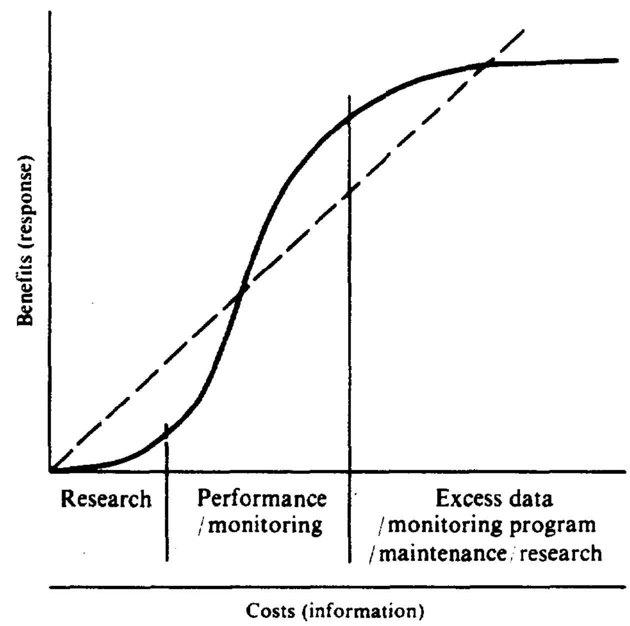
equilibrium reached is a dynamic one because, in all marine ecosystems, natural fluctuations in physical, chemical, and biological characteristics occur whose magnitudes are ordinarily comparable to or greater than the magnitudes of any changes caused by sewage inputs.

Accordingly, it is necessary to continue monitoring only for a sufficient length of time after initiation of the discharge to characterize the new dynamic steady-state that can then be evaluated in terms of the design and operational criteria. No further monitoring is necessary until such time as the quantity or nature of the discharge is substantially altered. The time required for an ecosystem to respond to a discharge and reach a new dynamic equilibrium depends upon the nature of the ecosystem and the specific characteristics of the ecosystem monitored. For example, when a waste is discharged into a large waterbody, physical and chemical changes in the water column caused by the discharge occur rapidly and dynamic equilibrium is reached within a day or two of discharge initiation (the time needed for effluent discharged during the first tidal cycle to be mixed below detectable levels). In an enclosed bay, chemical equilibrium may not be reached for several days or weeks until the effluent input to the enclosed area is in physical equilibrium with outputs from the bay. This effect is more pronounced in an estuary where seasonal extremes in temperature and outflow result in qualitative and quantitative differences in equilibrium response times and characteristics. In contrast, chemical changes in sediments near the discharge will proceed slowly in response to inputs integrated over several years or, in areas of low sedimentation of effluent solids, decades. In addition, biological changes will generally lag somewhat behind physical and chemical changes particularly where these changes take place quickly compared to the generation times of the affected biota.

Figure 14-1 provides a generalized representation of the changes taking place in water column and sediment physical/chemical and biological characteristics over a period of a decade following initiation of a sewage effluent discharge to open coastal waters. Note that for most parameters substantial natural temporal variations occur. Figure 14-1 represents smoothed mean values of parameters based upon basic oceanographic science principles and particularly upon published information concerning temporal response of sedentary infauna to sewage effluents (30).

It is concluded from the information summarized in Figure 14-1 that the effects of a new discharge into the marine environment on water column chemistry, physics, or biology can be effectively monitored by taking sufficient relevant measurements for about 15 months before and during the first year or so after discharge initiation to establish the predischarge and postdischarge mean (temporal) values of the monitored parameters. Similarly, the sedimentary effects can be effectively monitored by taking sufficient measurements before discharge initiation to establish the mean (temporal) value of the monitored parameter and sufficient relevant measurements 5 to 8 years after discharge initiation to establish the mean value at the new equilibrium level. In practice, the time needed for water and sediment parameters to reach equilibrium with respect to the new discharge varies somewhat depending upon the specific discharge environment. Therefore,

Fig. 14-2. Costs and Benfits of Monitoring



monitoring program design should provide for (1) establishing the predischarge mean value and variance of monitored parameters, (2) measuring the parameter at seasonal intervals after discharge initiation, and (3) when a new dynamic equilibrium has been determined, terminating or reducing the monitoring program to limited observations taken every 5 to 8 years to reveal unexpected additional changes. For most discharges, the volume of discharge will increase progressively over a period of several years. In such cases, the monitoring program need only be continued for sufficient additional time until the volume of discharge becomes essentially constant and dynamic equilibrium is reached. Additional monitoring may be needed after any major changes in waste quantities, characteristics, and treatment.

In summary, the numbers of parameters and sampling points and the analytical quality control procedures determine the costs of monitoring. The value of management actions supported by the monitoring information determines the benefits of monitoring. The wide variety of health-related issues for which this is true can be developed from the classic concepts of Shuval et al. (36) who showed that the marginal benefit-cost ratio of health investments followed a logistic curve. The same is shown in Figure 14-2 as it applies to numbers of parameters selection, particularly to the intervals at which they are sampled (13).

Although benefits are not easily quantified and the value of excess information is arguable, Figure 14-2 confirms the consensus of the U.S. regional monitoring workshops (4, 15, 19, 20, 33, 34, 35). Well-planned research projects and feasibility studies (see Chapter 15 for examples) that follow standard methods (1) and that discriminate are cost-effective in supporting engineering, operational, and management decisions; long-term monitoring programs adopted for reasons of political legitimacy (38) are both more expensive and less effective.

14.5 References

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CHAPTER 15

CASE STUDIES

15.1 Introduction

There is no shortage of solutions to sanitation problems. Ancient records include the admonition in Deuteronomy (21) to bury the stuff, Homer's eighth-century description of Herakles flushing the Augeian stables (19), and Socrates' fifth-century B.C. zoning restriction on night-soil dumping (87). The attraction of flushing continues today, often with ultimate disposal to the sea. The case studies summarized in this chapter cover different waste, hydrographic, and ecological characteristics. They include estuarine waters of the Thames, Manila Bay, and Minamata Bay; open coastal waters of the Southern California Bight, New York Bight, and Sea of Marimara; and the Bosporus portion of the Turkish Straits.

15.2 Thames Estuary

The Thames River estuary is a case study of the restoration of a damaged ecosystem. Thirty years ago, much of the estuary was devoid of fish life. Now, more than eighty species of fish can be found there. This recovery is attributed to (1) identification, quantification, and modelling of the factors controlling concentrations of dissolved oxygen and other determinants of survival of the biota, (2) determination of the pollutant inputs causing those effects in the Thames, and (3) effective control through waste regulation and treatment based on minimum dissolved oxygen requirements for segments of the river.

15.2.1 History

Pollution of the Thames has been documented since the seventeenth century (29). Even so, there was a large fishery based on whitebait, shad, smelt, salmon, and sea trout. However, nineteenth century increases in population, importation of water, and invention of the flush toilet caused accelerating degradation of the estuary and its environs. The fishery was failing and drinking water from the river was a source of cholera. Attempts to remedy the situation came about largely because of aesthetic rather than health reasons. The sulphurous odors coming from the river in the summer, caused people to complain of headaches and nausea, and in London the situation was severe enough to disrupt the workings of government. Gameson and Wheeler (28) relate how by mid-century sheets soaked in disinfectant were eventually hung in the Houses of Parliament in an attempt to counteract the stench.

In 1856, the Metropolitan Board of Works was established by Parliament and was charged with preventing any sewage from flowing into the river within the Metropolitan District. A comprehensive system of drainage was constructed, which, when opened in 1865, diverted the sewage downstream to be discharged during the ebb tide from outfalls at Beckton on the north bank and

Crossness on the south bank (see Figure 15-1 and Table 15-1). This improved conditions within London proper where the old outfalls had been located, and moved the waste downstream to the vicinity of the new outfalls. (Unfortunately, discharging twice the quantities of wastes into estuaries during half the time into ebb tides that become flood tides during the other half increases hydraulic requirements and disposal costs without corresponding benefits.) In 1882, a further attempt was made to rectify the situation with the creation of two sedimentation channels at the outfall sites, and the sewage was treated with coagulants so that the solids would settle and not disperse. The treated sewage sludge was then periodically dredged, transported to sea, and dumped there. Although the water quality of the Thames improved slightly, there were still numerous complaints of offensive odors, especially during the dry summer period. Some fish life reestablished itself, with whitebait reappearing at Gravesend in 1892 and Greenwich in 1895; in the latter year, flounders were caught in the upper reaches for the first time in twelve years (28). Freshwater fish also began to move downstream, but several attempts to reestablish the commercially important salmon fishery were unsuccessful.

Interceptor sewers were constructed between 1900 and 1910, and the emphasis of pollution control shifted to improving the quality of treatment. A paddle-aeration, activated-sludge plant was built at Beckton in the 1930s to treat about one-quarter of the total flow. In 1936, a new sewage works was built upstream at Mogden to replace twenty-eight smaller treatment works. Despite these improvements, the period from 1930 to 1950 was one of progressive deterioration in water quality, particularly dissolved oxygen levels. This was caused by several factors: (1) the population was still increasing, and thus inputs of sewage also increased; (2) during World War II bombs damaged the sewage treatment works; and (3) nondegradable detergents were introduced, and they increased the nutrient loading and reduced the capacity of the river for self-purification.

After the war, improvements in sewage facilities were postponed for several years while the city concentrated on other reconstruction efforts. Then, in 1955, a new, efficient primary settlement plant was constructed at Beckton, followed by additional activated-sludge plants, which brought the portion of total flow receiving biological treatment to about 50 percent by 1960 (28). With the further improvements in treatment since that time, the water quality in the Thames Estuary has become much better.

15.2.2 The DSIR Dissolved Oxygen Model

In 1949, the Water Pollution Research Laboratory of the Department of Scientific and Industrial Research (DSIR) began a fifteen-year study of the factors affecting the water quality of the Thames Estuary. This effort was stimulated by complaints of offensive sulphurous odors and reports that the fumes were causing brass to tarnish rapidly and lead-based paints to discolor. Because sulfide was present in the water only when anaerobic conditions prevailed, the investigation was aimed primarily at studying the factors affecting the distribution of dissolved oxygen (78). The study had three parts: (1) river water was sampled and analyzed throughout the length

Fig. 15-1. The Thames Estuary and the major sewage discharges to tidal reaches in 1978. Source: Adapted from Norton (78).

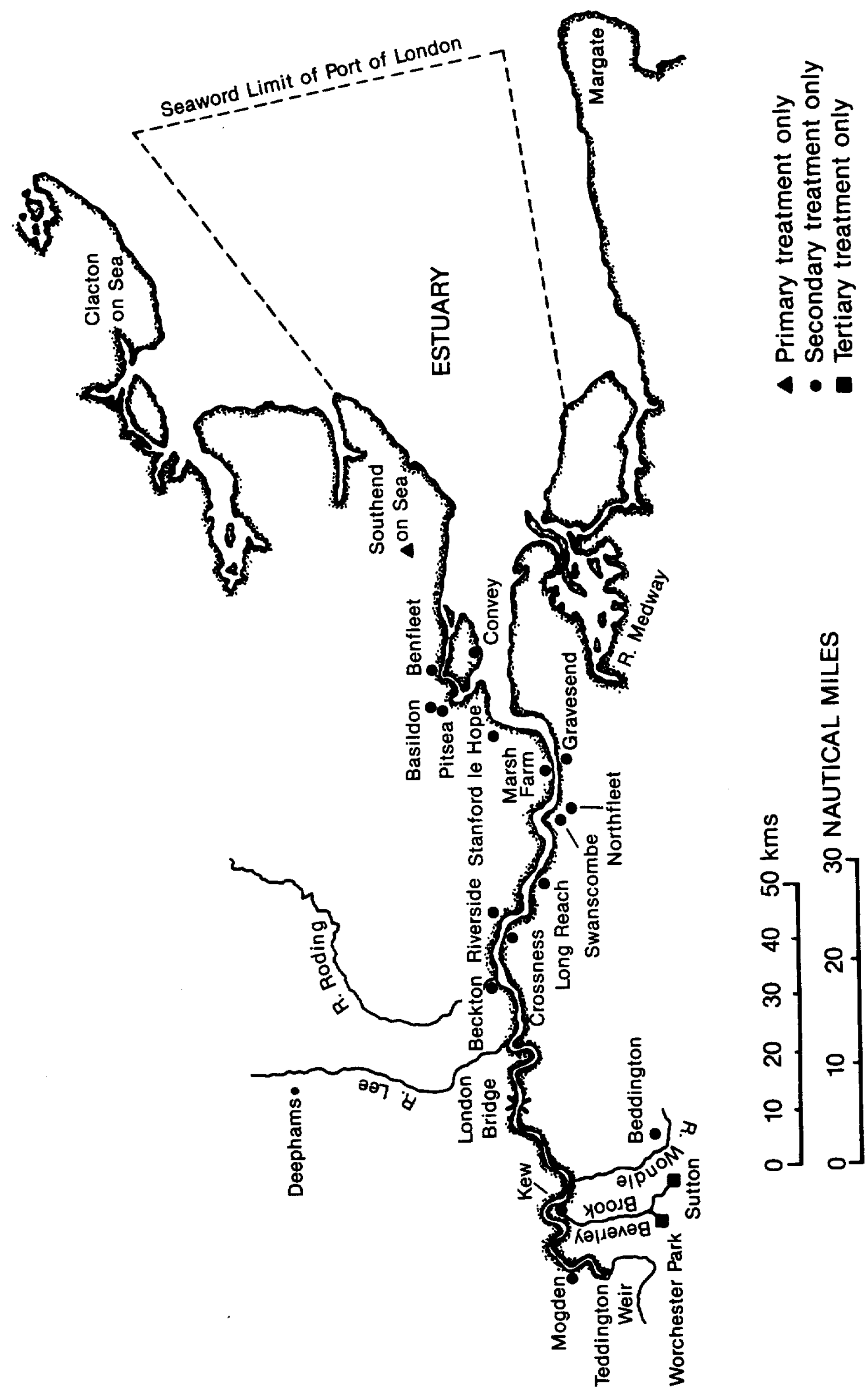


TABLE 15-1

Municipal Discharges to the Thames Estuary

Sewage Treatment Works	Flow (m^3/d)	EOD [a/] (tonnes/d)	Type of Treatment
Mogden	446,000	18.8	Secondary
Worcester Park	16,400	0.8	Tertiary
Sutton	7,950	0.4	Tertiary
Beddington	86,400	3.8	Secondary
Deephams	185,000	5.2	Secondary
Beckton	818,000	23.5	Secondary
Crossness	500,000	32.8	Secondary
Kew	34,400	0.8	Secondary
Riverside	86,800	14.8	Secondary
Long Reach	186,000	96.8	Secondary
Basildon	18,200	2.1	Secondary
Benfleet	6,140	0.5	Secondary
Canvey	4,500	1.1	Secondary
Marsh Farm	31,500	16.0	Primary + some Secondary
Pitsea	5,750	0.3	Secondary
Southend	32,700	15.0	Primary
Stanford-le-Hope	4,660	0.6	Secondary
Gravesend	7,200	3.1	80% Primary, 20% Secondary
Northfleet	7,450	1.1	Secondary
Swanscombe	1,000	0.2	Secondary

a/ Effective oxygen demand = 1.5 (B + 3N), where B = 5-day BOD and N = oxidizable nitrogen.

Source: Norton (78).

of the estuary; (2) water quality records were examined to reveal changes in the condition of the estuary during the previous fifty years; and (3) models were developed to predict the movements of effluents discharged into the estuary.

As these models were developed, they were validated by comparisons between predicted and observed values of dissolved oxygen. Later, attempts were made to predict the condition of the estuary if certain changes were made in a number of the variables (e.g., amounts of pollutants discharged, source locations, different freshwater flows). The model resulting from this study has withstood the test of time--the agency responsible for water quality (presently, the Thames Water Authority)-- continues to use it in managing the estuary.

The mathematical model that is used assumes the estuary to be a barrier-free tidal river with boundaries at the tidal limit (Teddington Weir) and the sea. This stretch of about 150 km is divided into 3.3-km segments, each considered to be of uniform chemical composition throughout. The composition is determined by (1) the inputs of pollutants from all sources; (2) the movements of these pollutants by diffusion and advection into freshwater sources, including the river and groundwater, and by tidal movements and exchange with seawater; and (3) the rate of decay or removal of the constituents by chemical, physical, or biological means (78). Although simple in nature and selective in the variables it takes into consideration, the model can accurately predict dissolved oxygen (see Figure 15-2), ammonia, and oxidized nitrogen concentrations, as well as the temperature of the river along its length, all on the basis of established decay characteristics of these water quality parameters.

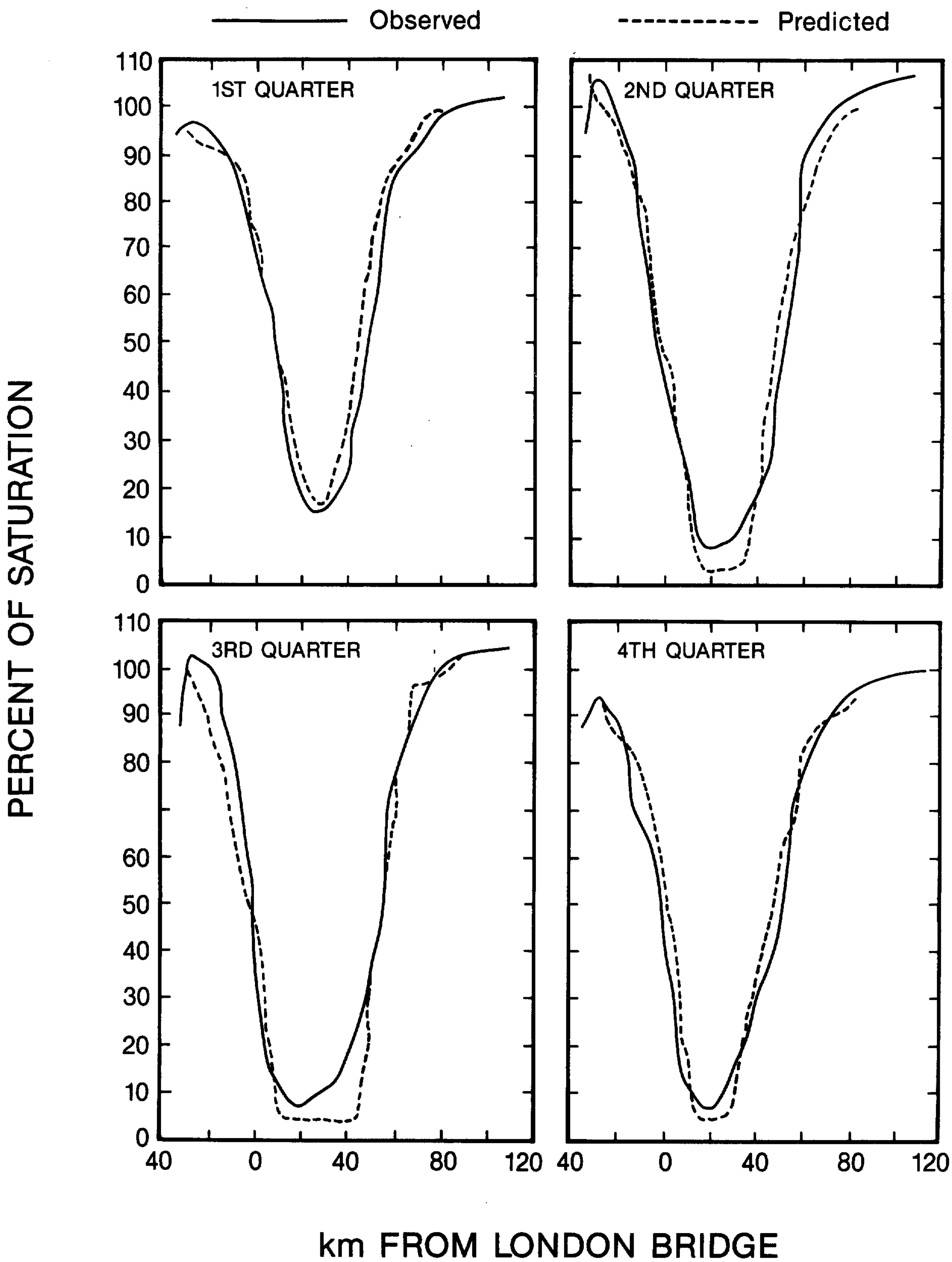
15.2.3 Hydrography of the Thames Estuary

Water movements and the distribution of dissolved oxygen in the Thames are dominated by freshwater flow from upstream and by the effects of tides. Low freshwater flows lead to lower oxygen reserves and reduce the estuary's assimilative capacity for sewage effluents.

Over a fifty-year period (1925-74), river flows measured daily ranged from 0.9 to 709 m^3/s. Withdrawals from upstream of Teddington for municipal water supplies have increased from 4.3 m^3/s in 1885 to 17.0 m^3/s in 1970-74. During periods of low river flow, this removal significantly reduces the freshwater flow into the estuary, and causes oxygen concentrations to drop (28).

The average tidal range at Teddington is 2 m; it increases to 6 m at London Bridge, and then gradually decreases to 4m at Southend. Depending on the freshwater flow, tidal state, and tidal range, pollutant inputs to the estuary are dispersed considerable distances upstream and downstream during successive tidal cycles. Recognizing that the dissolved oxygen contents of two samples taken at the same point in the estuary under different tidal states would be likely to differ considerably, the DSIR model reduces the DO data to a common predicted tidal state by replacing the true sampling position

Fig. 15-2. Seasonal concentrations of dissolved oxygen (expressed as a percentage of saturation) in the Thames Estuary in 1971. Source: Norton (78).



with the location of the water at "half-tide." Half-tide is defined as the instant when the volume upstream (to Teddington Wier; see Fig. 15-1) is the mean value for the average tidal cycle (28). The result is that samples taken at low water are, in effect, moved upstream and samples taken at high water are moved downstream. Statistical analysis of dissolved oxygen measurements has revealed an essentially linear relationship between dissolved oxygen and freshwater flow. Effects of temperature photosynthesis, and seasonal loading upon dissolved oxygen content have been more difficult to determine.

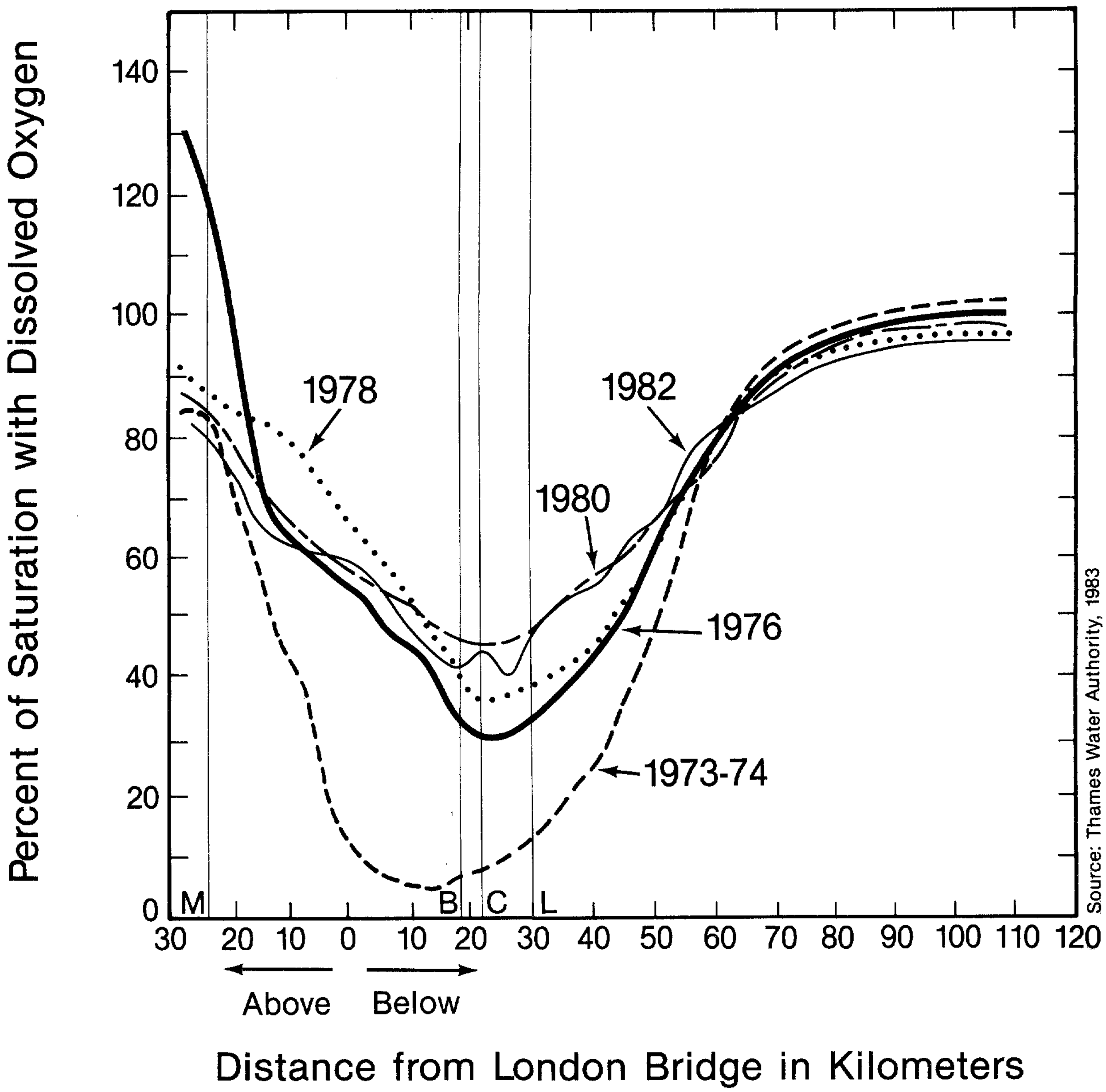
DSIR studies found that when the oxygen concentration falls below 10 percent saturation, nitrification (oxidation of nitrogen compounds to nitrites and nitrates) ceases, and denitrification (reduction of nitrates to nitrogen) occurs. When reserves of nitrates are exhausted and fully anaerobic conditions are established, sulfates are reduced and give rise to offensive odors. Dissolved oxygen levels have continually improved since 1950 and have not fallen below 10 percent saturation in almost a decade. Figure 15-3 shows these levels for the third quarter (July-September) for the past ten years. Although sewage effluents still exert the greatest polluting load to the tidal Thames, improved treatment has reduced the average daily oxygen demand from over 800 tons O_2/day in the early 1950s to 300 to 400 tons O_2/day in the early 1970s and to the present 130 tons O_2/day (28). There have been no complaints of offensive odors from the river for many years and dissolved oxygen levels routinely exceed those necessary to allow the passage of migratory fish under normal conditions (50).

15.2.4 Fish Populations

Early fish population data are sparse and based largely on incidental observations. There is no evidence of any fish in the late 1950s for some 68 km upstream of Gravesend, except for eels that breathe at the surface. There were no commercial fisheries on the river and sportfishing was limited to only a few isolated areas. Evidence that the river was finally on its way to recovery first appeared in 1964 with the capture of fish on power station cooling water intake screens within this previously fishless zone. The number of freshwater and marine species has increased (Figure 15-4) to over eighty. Seasonal lows in dissolved-oxygen levels decrease the range of distribution of all species during these warmer months. Many migratory euryhaline species capable of survival in a wide range of salinities are commercially important. The recent introduction of young salmon into the upp reaches of the river has proven successful in that a number of marked fish have been taken from the tidal Thames on their return to spawn (78).

The tremendous improvement in the Thames Estuary did not come about by applying administratively and legally simple uniform treatment standards. Rather, the needed reductions of inputs in specific segments of the estuary, and alternative levels of biological treatment of sewage have been applied to domestic and industrial effluents throughout the estuary. Inputs have been regulated only to the extent that assures adequate oxygen supplies for fish life. The extent of treatment necessary to achieve the desired water quality was determined by the use of the DSIR model. The model predicted that significantly improved dissolved oxygen levels would be achieved by higher levels

Fig. 15-3. Annual Changes in Dissolved Oxygen during July to September in the Thames Tideway, 1973-82. Source: Jennings (50).



M—Mogden
B—Beckton S.T.W.
C—Crossness S.T.W.
L—Long Beach S.T.W.

Fig. 15-4. Number of Species of Fish Caught on Intake Screens of West Thurrock Power Station. Total number of fish species identified 1963-1978 was 66. (Source: Norton 1983.)

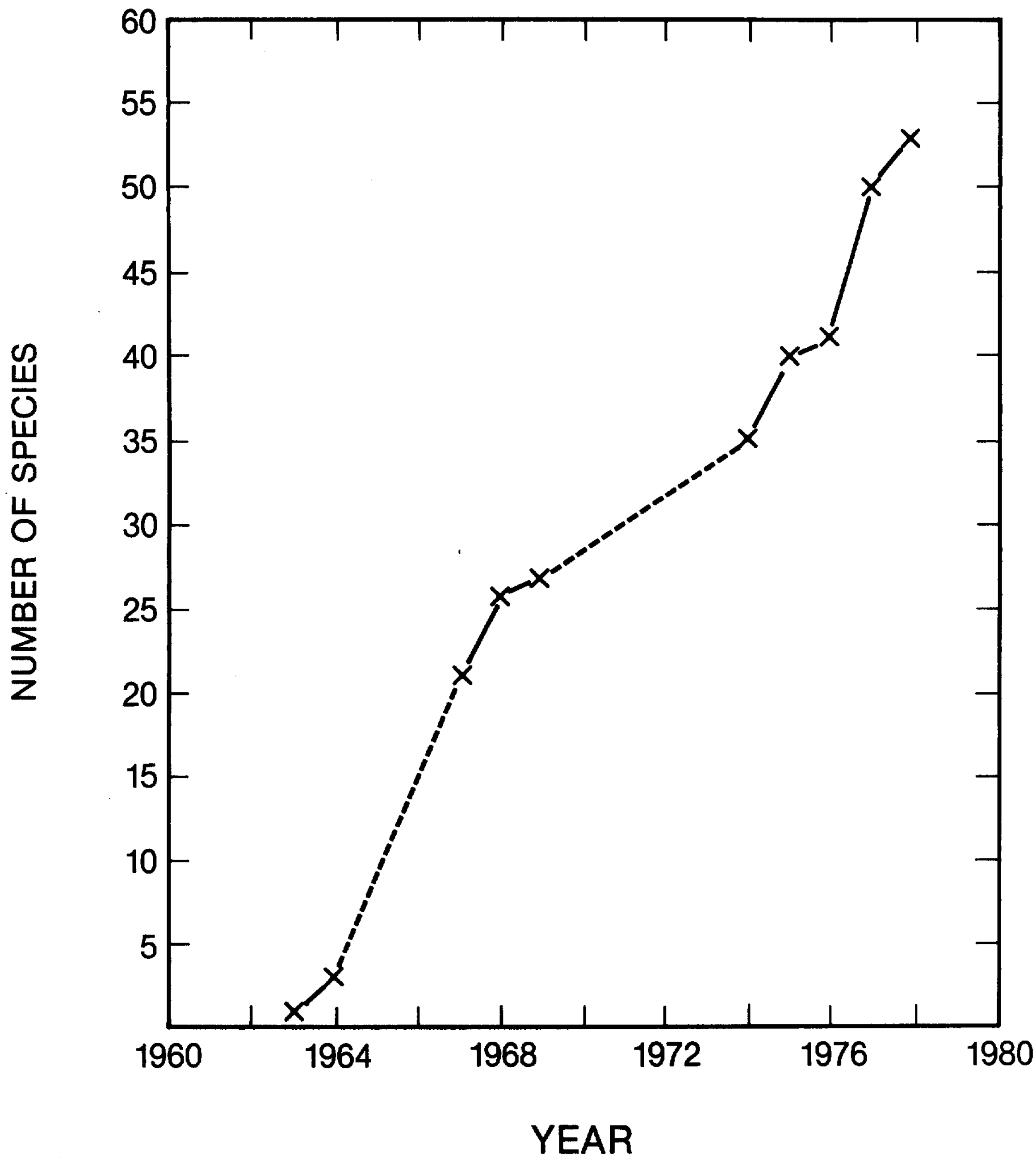


Fig. 15-5. The Effect of Beckton Sewage Treatment Works Effluent Quality upon Dissolved Oxygen (maintaining Long Reach sewage treatment works at 1974 quality of 150 mg/l BOD and 32 mg/l NH_3). Source: Norton (78).

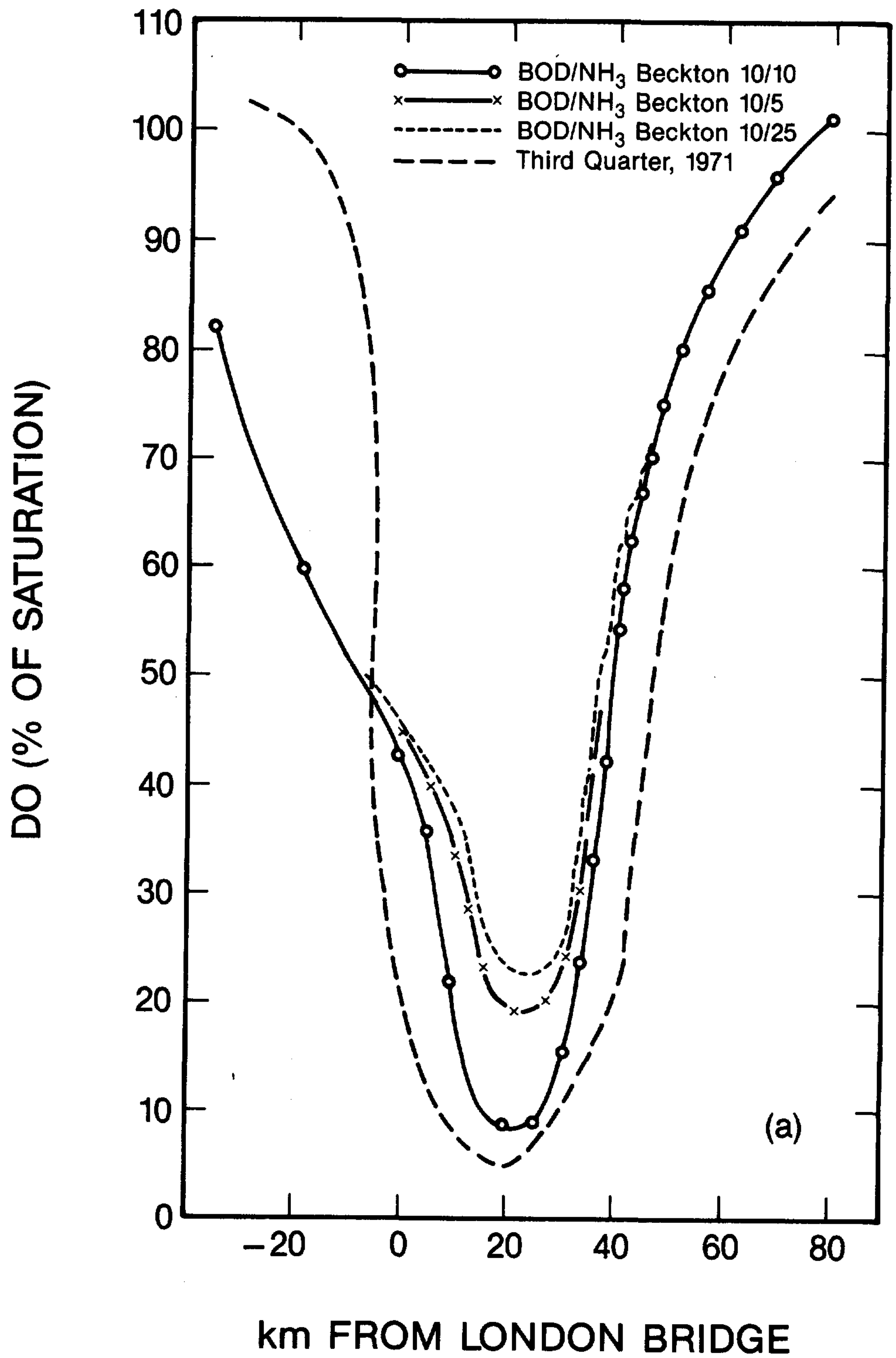


Fig. 15-6. The Effect of Long Reach Sewage Treatment Works Effluent Quality upon Dissolved Oxygen (maintaining Beckton sewage treatment works at 10 mg/l BOD and 2.5 mg/l NH_3). Source: Norton (78).

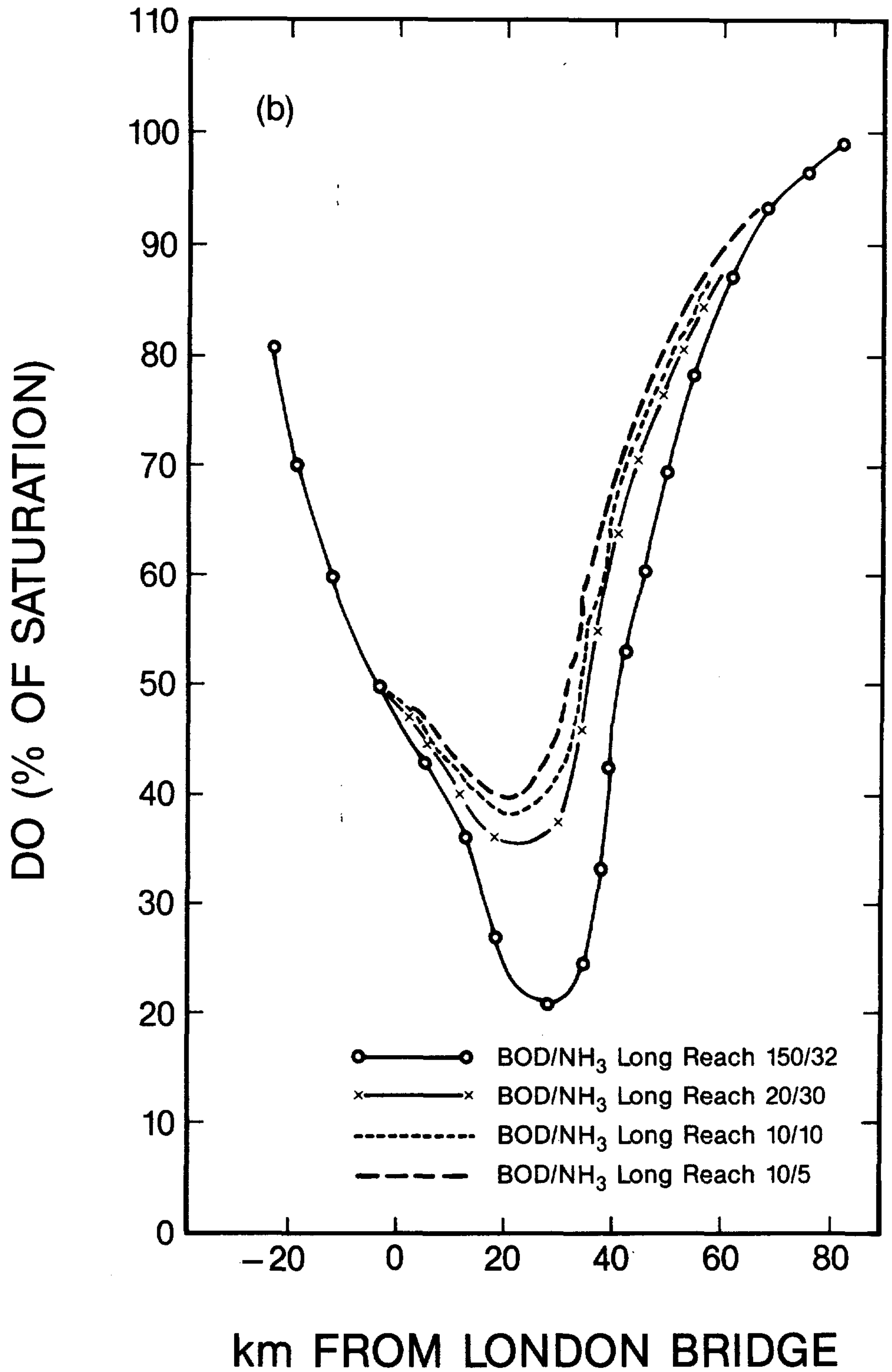


Fig. 15-7. Contaminants in Sediments of Thames Estuary
Source: Norton (83).

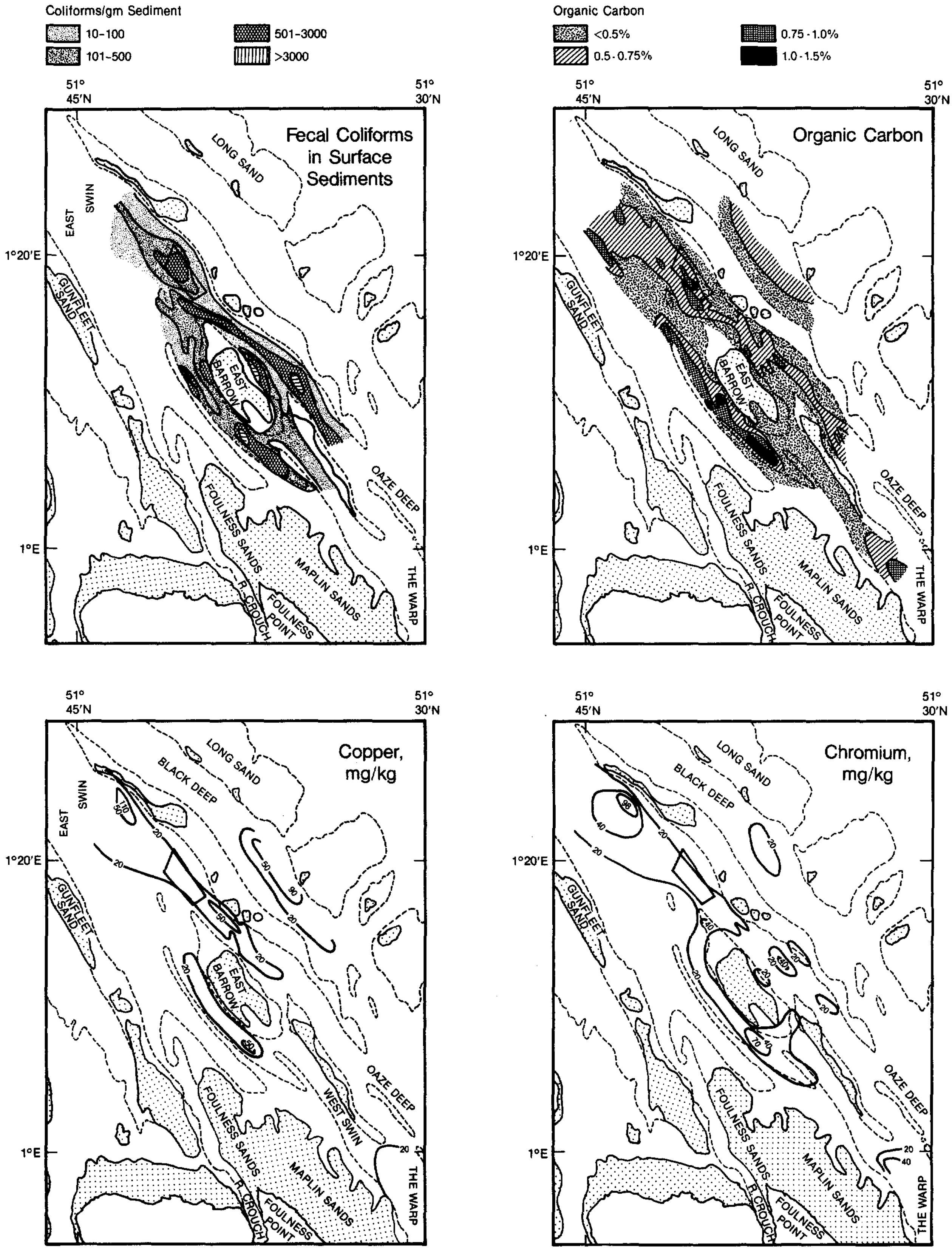
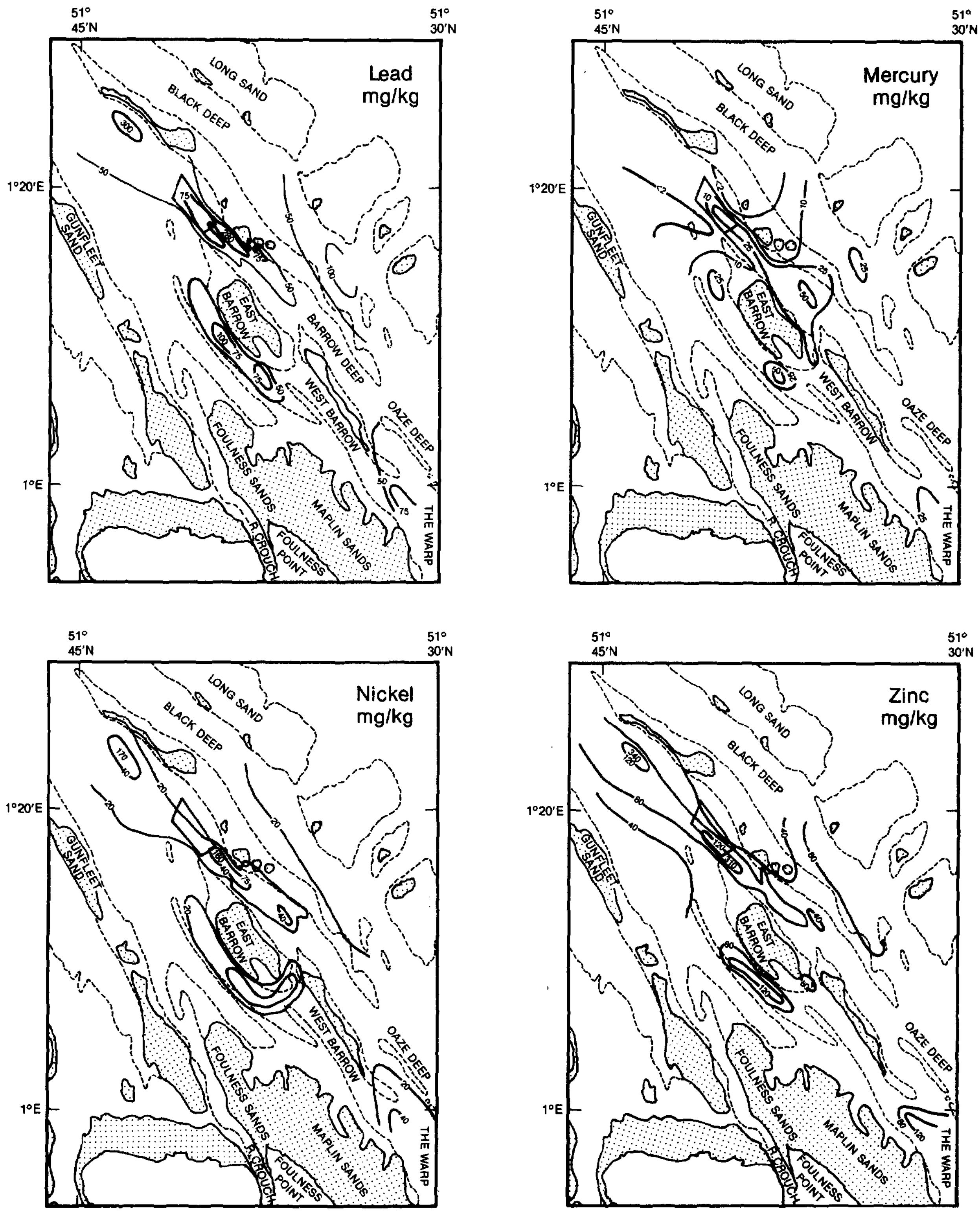


Fig. 15-8. Contaminants in Sediments of Thames Estuary (cont.)
Source: Norton (83).



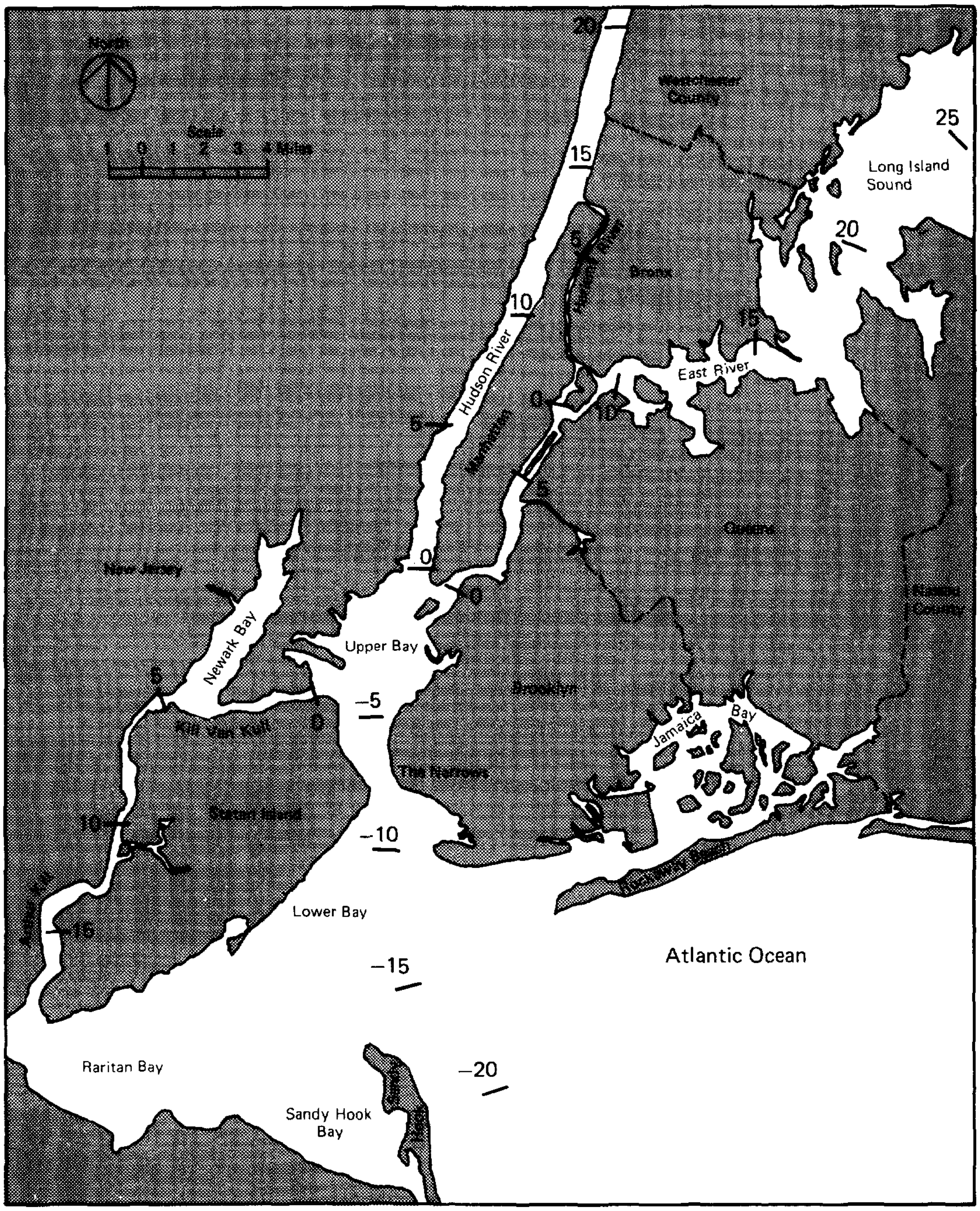
of treatment at the Beckton plant (Figure 15-5) but only marginal benefits would be achieved by improvements at the Long Reach plant (Figure 15-6), downstream which discharges four times the oxygen demand that Beckton does (see Table 15-1). Application of model findings has resulted in desired water quality levels throughout the river using different levels (i.e., primary, secondary, or tertiary) at different plants. The finding that generally higher levels of treatment are required with increased distance from the estuary mouth is particularly significant. The current emphasis in the efforts to improve the quality of the Thames is on the control of toxic industrial wastes.

Approximately 5 million tonnes/year of sewage sludge with 2.7 percent solids are barged to the mouth of the estuary (Point A; Figure 15-1). Tidal action moves some of the sludge up to 35 km upstream. Increased levels of fecal coliforms, organic carbon, chromium, copper, lead, mercury, nickel, and zinc are found in bottom sediments (Figures 15-7 and 15-8). Most of the sludge moves seaward, and does not appear to accumulate in the discharge area (78). The distances noted are roughly comparable to those for movement of copper and zinc in the New York Bight, where circulation is limited. By contrast, elevated concentrations of heavy metals in Santa Monica Bay sediments are generally closer to the discharge point (see Sections 15.2 and 15.3).

15.3 The New York Bight

Waste disposal in the New York metropolitan area (Figure 15-9) developed along conventional, ancient lines described by Tarr (93). Because of periodic flooding, which has always been a problem in low-lying metropolitan areas, stream channels were gradually improved to handle the runoff. As population densities increased a series of alternating technological and environmental responses developed: (1) local water supplies became inadequate and were supplemented with increasingly distant supplies; (2) manual conservancy systems for utilizing night soil from buckets or pit latrines became overloaded and so people began using drainage channels for disposal; (3) ordinances were passed forbidding but not preventing people from placing human wastes in drains; (4) with the increased availability of water it became possible to introduce flush toilets and cesspools; (5) pollution of local groundwater supplies from increasing amounts of water discharged to overloaded cesspools "constituted a means of distributing fecal pollution over immense areas," according to Lee (56); (6) with the loss of local water supplies the dependence on imported water became even greater; (7) dilute household wastes were piped to storm drains and inadvertently created combined sewers discharging into the Hudson, Harlem, and East Rivers, Raritan River, and the Kills (Figure 15-9); (8) separate and combined sewers were gradually constructed so that by 1977 they were serving 18,032 hectares (44,557 acres) and 63,543 ha (157,015 acres), respectively (2); (9) local nuisances and public health risks, losses of fisheries, and other ecosystem damage first appeared in the smaller rivers and embayments and continued until today it is present throughout the estuary; (10) as a result of the treatment and removal from dry weather flows of putrescible materials that began in 1924, increasing amounts of sludge were disposed of at a convenient site about 12 miles seaward of New

Fig. 15-9. New York Harbor Area. Numbers show miles from the Battery. Source: Hazen and Sawyer (44).



York harbor; (11) federal and state legislation was enacted mandating increased removal of solids (sludge) that must be disposed of; (12) with increasing competition for space, more and more people objected to having other people's wastes dumped on or near them (their own wastes are bad enough); (13) environmental protection mandates became increasingly restrictive; and (14) remedial technological responses and proposals became increasingly expensive and energy intensive (40).

Technological development has followed a similar path in industrial waste treatment. In both the municipal and industrial approaches, water supplies have been developed and rate structures set with no regard for the costs of getting rid of the water; sewerage and industrial waste systems are designed and optimized with no regard for the cost of flushing water (note that the costs of sewage treatment are not those of getting the solids out of the water, but of getting water out of the solids -- the greater the dilution with flushing water, the greater the cost). The consequences of these conventional approaches are summarized in this case study.

15.3.1 Geography

The location of the New York Bight is shown in Figure 15-10 (44). After five centuries of immigration, growth, and development of natural resources such as harbors, and soils, effective land and water resource management is sorely needed. Seventy-three percent of the average 790 m^3/s (27,900 cfs) freshwater entering the New York Bight is from the 35,000 square km (13,500 sq mi) of the Hudson River Drainage Basin. Total flows vary from 210 m^3/sec (7,500 cfs) in summer to 1,300 m^3/sec (47,400 cfs) in April.

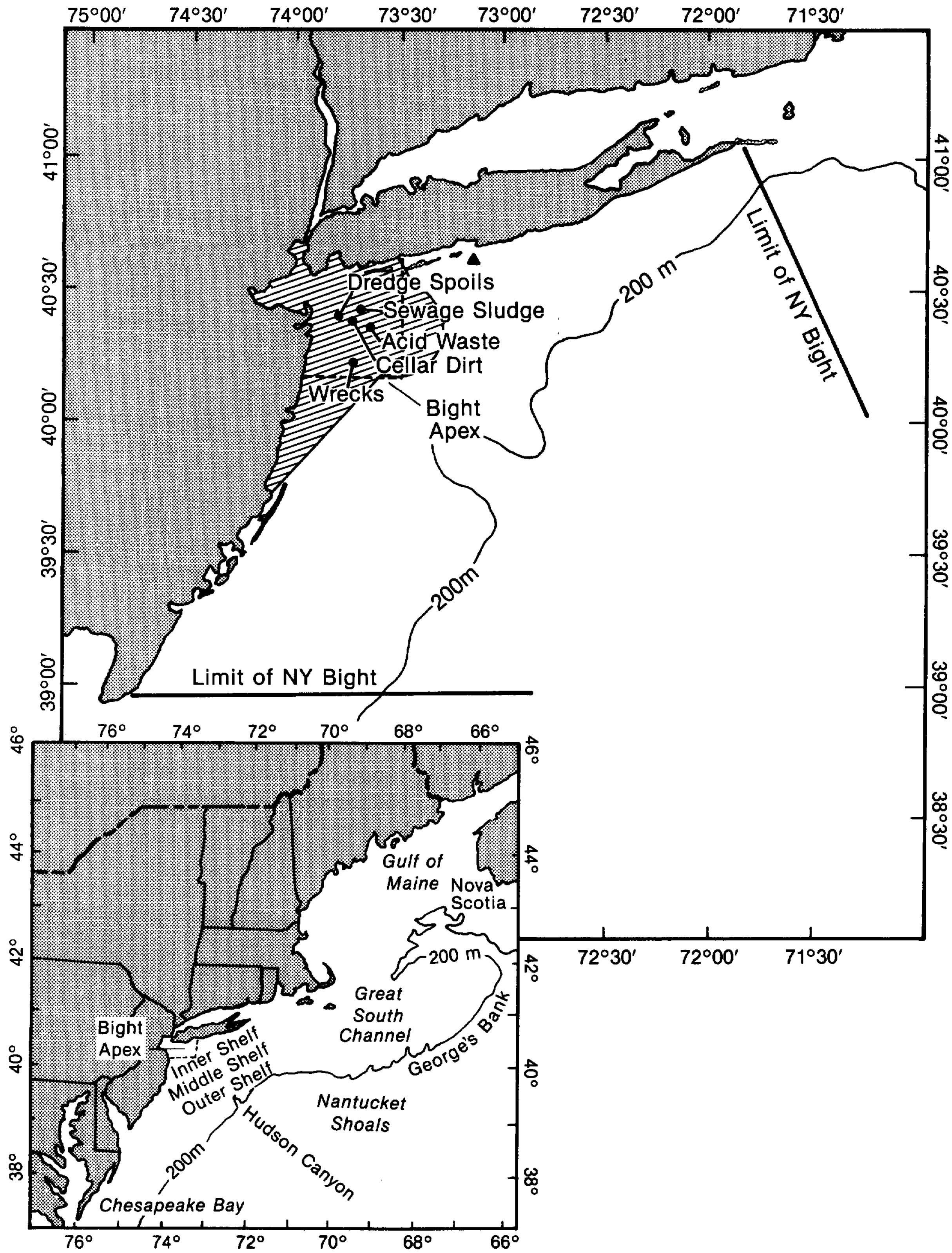
The bathymetry (Figure 15-11) is irregular, and is marked by the Hudson Shelf Valley, which leads to Hudson Canyon; the Christiansen Basin, where sediments are affected by waste discharges; and the local mound of dredge spoil.

Mean semidiurnal tides range from 0.8 to 1.4 m (2.8 to 4.6 ft) along the coast and reach 2.2 m (7.2 ft) in Long Island Sound (90). These are superimposed upon the annual cyclical variations of approximately 1/4 m (8 in) and a secular increase in mean sea level of about 35 cm (1.1 ft) per century. The storm tides in the area can be up to 4 m high and have periods of 3 hours to 6 days (83).

Tidal currents reported by Hansen (43) increase near the entrance to the harbor as shown in Chapter 2, Figure 2-3. Net surface outflows average >16 cm/sec near the center of the Sandy Hook-Rockaway Point transect, while average inflows reach 12 cm/sec near Rockaway Point and 4 to 6 cm/sec within the deeper channels off Sandy Hook.

Within New York Bight, surface flows are generally southwesterly but highly variable. Along the bottom near the river mouth, estuarine circulation causes a net shoreward and upstream movement. The aperiodic up- or down-slope movements in the Hudson Canyon and Hudson Shelf Valley are the result of regional meteorological conditions; monthly average velocities in the Valley

Fig. 15-10. New York Bight Showing Ocean Dumping Locations. Shaded area is closed to shellfishing. Source: Gunnerson (37).



are as much as 5 cm/sec (0.1 kt) up-valley (43). From mean velocities calculated by Beardsley et al. (7) over the 180-km (100-n.mi.) wide continental shelf, transport is estimated at 2 x 10^5 m^3/sec within the 100-m isobath and residence times at 3/4 year. Residence times in the Apex vary from about 4 to 12 days (92), with less flushing during the September-October minimum Hudson River outflow period. Currents over the shelf are strongly affected by surface wind speed and direction over the Bight, which have pronounced seasonal variations but generally little spatial variation; thus observed air movements appear to be governed by large-scale meteorological systems (57).

Mean water temperatures are generally 1 to 4^o C cooler than air temperatures in spring and early summer and up to 6^o C warmer than air in winter (57). The springtime air-sea temperature differentials, local wind mixing, and outflows from the Hudson River influence the rates of development and the depth and strength of the summer pycnocline over the inner shelf.

Fisheries are sensitive both to pollution and to fishing pressures. Edwards (23) has shown that reductions between 1963 and 1974 are due in large part to overfishing of standing crops of commercial species off the northeastern United States. Depletions in stocks of haddock, herring, and other commercially important species have been noticeable. However, the numbers of mackerel, some of which spawn in the New York Bight, have increased. Some of the decline in commercial finfishing has been offset by sportfishing (61). In this regard, Jensen (50) notes that with the increasing competition for particular species such as striped bass, some habitats are being destroyed by fishing gear and disputes have arisen over areas, access, traditional rights, and legal prerogatives.

Shellfisheries also change with time. Commercial landings of the American oyster in the New York Bight have decreased from about 7,000 metric (or long) tons per year in 1950 to a low of 46 tons/year in 1967, followed by an increase to 960 tons in 1975. Hard clam landings dropped from 6,000 tons in 1950 to 2,500 in 1960, to 4,700 tons with a value of $16 million in 1975. McHugh reports that the surf clam fishery expanded from about 3,000 tons in 1950 to about 16,000 tons from 1968 through 1975 (60), but has been overexploited in the past few years.

15.3.2 Economic Development

The New York metropolitan area began with 24,000 people in 1786 in lower Manhattan. From that point on, it grew steadily. Within New York City, the most rapid growth took place between 1900 and 1930. By 1970 the metropolitan area supported 19.3 million people. This figure may reach 25.8 million by the year 2000 (40). Population densities in 1977 ranged from approximately 1/km^2 (3/mi^2) in the lower Hudson Valley to 21,000/km^2 (67,000/mi^2) in New York City.

Although much of the economic activity in the New York Metropolitan Area is in the commercial, transportation, and service sectors, manufacturing has increased steadily since 1939, with a shift toward fewer numbers of establishments and employees since about 1950. By 1972, 1.8 million manufacturing employees in the metropolitan area contributed more than $28 billion worth of value added, approximately half of which has come from the chemical, publishing, and textile industries (39).

15.3.3 Water Supply

Most of the municipal water supply is not metered (New York City is unique among major cities of the world in this regard) and per capita consumption has increased from 380 liters per capita per day, lcd (100 gcd) in 1900 to 660 lcd (175 gcd) in 1977 (16). Per capita consumption in New York City is relatively high in comparison with other U.S. cities. Of 30 cities with populations of 250,000 or more, the 1960 domestic consumption in New York (337 lcd, or 89 gcd) was exceeded only by Cincinnati (348 lcd, or 92 gcd), Los Angeles (365 lcd, or 96 gcd), and Denver (550 lcd, or 96 gcd). Like New York City, Los Angeles and Denver rely largely on imported supplies but receive less than 40 cm (16 in) of precipitation per year compared with about 120 cm (48 in) in the New York metropolitan area. Other major U.S. cities use from 138 to 291 lcd (36 to 77 gcd), with mean and median values of 212 lcd, or 56 gcd (16).

15.3.4 Waste Disposal

Major municipal and industrial charges leading directly or indirectly to the New York Bight are shown in Figures 15-11 and 15-12, respectively (2, 39, 70). Not shown is the 5.5-km (3.5-mi) Suffolk County outfall designed to divert 1.33 m^3/s (30.5 mgd) of sewage from Hempstead Bay through a 1.80 cm (72-inch) diameter pipe into about 16 m (53 feet) through a 1 km (0.62 mi) diffuser section. The pipe is reinforced concrete with a cathodically protected embedded steel cyclinder (see Chapters 5-13 for comparisons).

Recent (1972) information on waste loadings to the New York Bight (32, 44, 68, 69, 91) indicates 127 major municipal discharges of 114 m^3/sec (2,600 mgd) of sewage, of which 18 percent was untreated. By 1978, the untreated sewage had been reduced to 9 m^3/sec (295 mgd); 1972 industrial process discharges amounted to 27 m^3/sec (610 mgd), of which 47 percent went through municipal systems. Another 210 m^3/sec (4,800 mgd) of cooling waters were discharged.

Ocean dumping at locations shown in Figure 15-2 is the most visible discharge to the Bight. Mueller et al. report an annual total of 27.6 x 10^6 m^3 (21.7 x 10^6 yd^3) dumped, of which 53 percent was dredge spoils, 26 percent was sewage sludge with about 5 percent solids, 15 percent was acid waste, 3 percent was chemical wastes, and 3 percent was building and construction debris. Dredge spoils account for most of the sediments entering the Bight; of these, about half are sands moved by littoral drift and estuarine bottom flow into the harbor (32). Because of sediment contamination by waste discharges to upstream dredged channels, at least half of the cadmium, chromium, copper, and iron entering the Bight is associated with dredge spoils (69).

Fig. 15-11. Municipal Discharges. Source: Gunnerson (39).

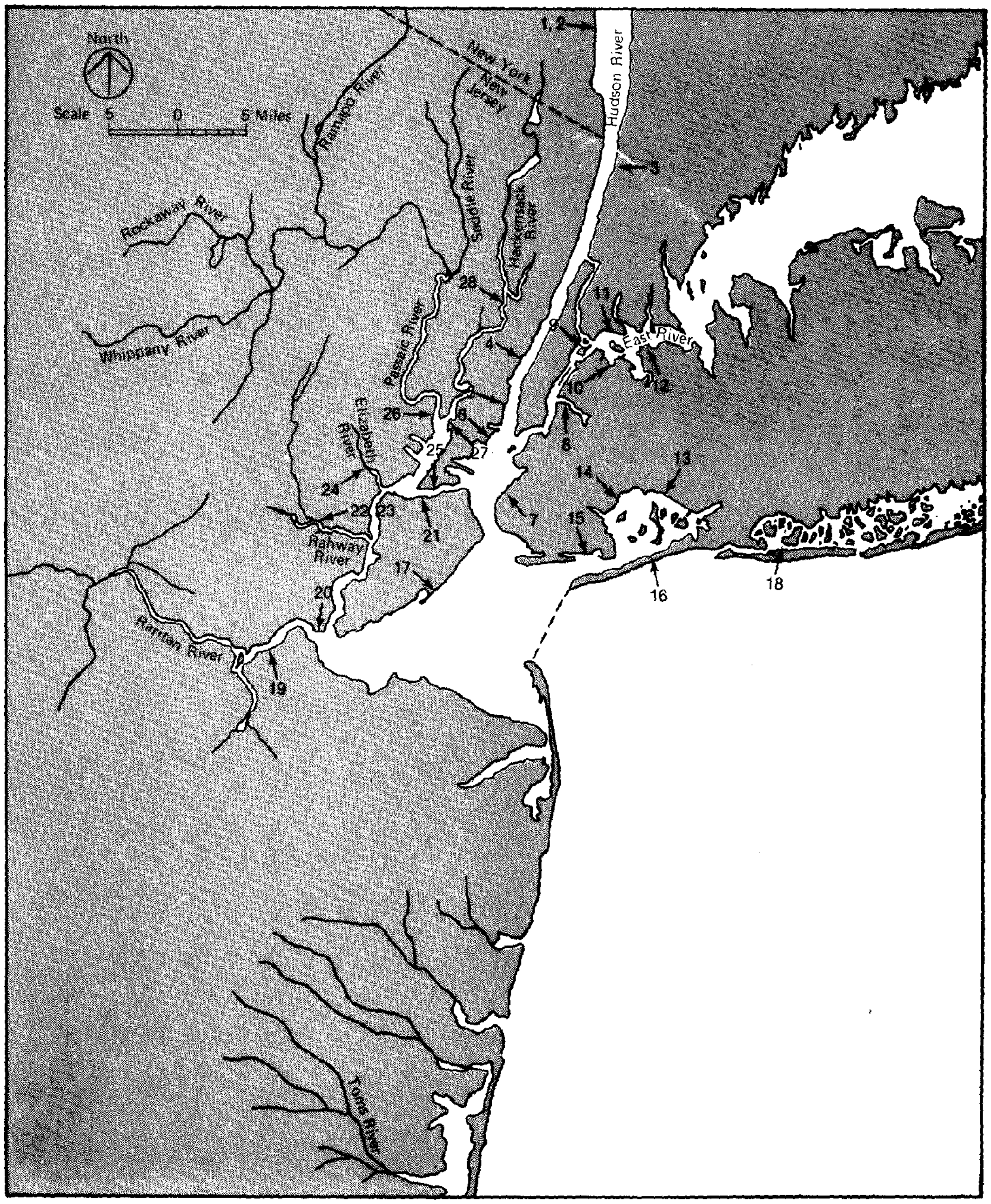


Fig. 15-12. Industrial Discharges. <u>Source</u>: **Gunnerson (39).**

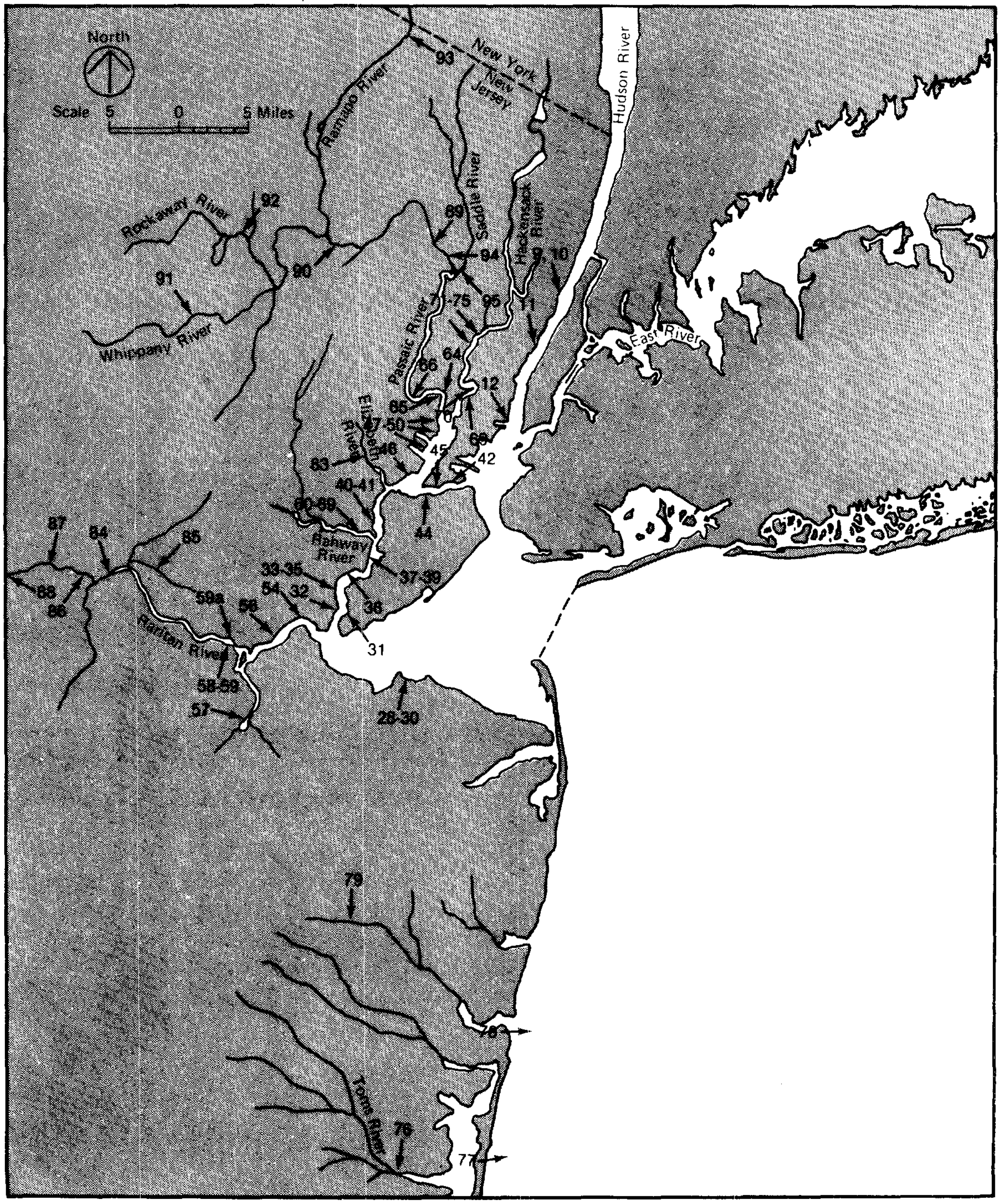


Table 15-2 lists sources and loadings of materials entering the New York Bight in 1976. Future quantities of wastes will reflect wastewater reclamation, additional areas sewered, sewage treatment plants and effluent outfalls constructed. Consideration is also being given to the removal of floatables from stormwater overflows for aesthetic reasons (there is no evidence of a health hazard from this source). At present 6.6 m^3/s (150 mgd) of raw sewage continues to be discharged from Manhattan's West Side (one major hotel recently advised its guests to flush razor blades down the toilets).

Anderson and Mueller (1) have estimated loadings for the year 2000 (Table 15-3) assuming secondary (biological) treatment and effluent chlorination of municipal sewage. They further assume that mass loadings of industrial, atmospheric, ocean dumping, and urban runoff loadings will remain the same as at present. Although it can be argued that implementation of regulations that limit discharge of toxic or hazardous substances will reduce the amounts of materials entering the Bight, the large reservoir of these materials in estuarine and river sediments and on land surfaces continues to be a source of Bight pollution. Table 15-3 shows that little change may be expected in loadings to the Bight.

15.3.5 Effects of Waste Disposal

15.3.5.1 Estuarine Pollution

Figure 15-9 shows locations of estuarine areas and transects for which water quality values are listed in Table 15-4. Mean values for dissolved oxygen concentrations are as low as 0.7 mg/l in the Kills and 1.0 mg/l in the Harlem River. High concentrations of Kjeldahl nitrogen, phosphorus, cadium, copper, lead, and zinc are scattered throughout the area. Bacteriological pollution is greatest in the Harlem River, where up to 100,000 fecal coli/100 ml have been recorded. Steady-state levels of coliform pollution are explained by times of 40 to 70 hours for 90 percent reductions due to sedimentation and mortality. Coliforms ordinarily disappear from marine surface waters much more quickly, except where sewage is conventionally treated or chlorinated (36), or in areas where they are resuspended by waves, currents, and ships' propellers from contaminated bottom sediments (33).

15.3.5.2 Shoreline Pollution

Coastal surf zone and beach pollution includes enteric bacteria in the water and stranding flotsam. Data for 1973-77 on coliform bacteria concentrations in the surf zone are summarized in Table 15-5. Numbers of samples per year varied from 6 to 48, mostly in the summer. Variations between stations reflect proximity to waste discharges (Coney Island is within the estuary) and differences in sampling, analytical procedures, and methods of data analysis and reporting. Differences between years at individual stations are real and reflect changes in waste discharges and treatment and in runoff.

TABLE 15-2

Percentage of Waste Loadings to New York Bight by Source

	Direct Bight		Coastal Zone				
			Wastewater		Runoff		
Parameter	Barge	Atmospheric	Municipal	Industrial	Gaged	Urban	Groundwater
Flow	0.02	59	5	0.4	33	2	0.4
Suspended solids	63	5	4	0.2	16	12	Nil
Alkalinity	1	Nil	35	0.3	59	5	0.03
5-day BOD	21	9	48	2	11	9	0.01
Chemical oxygen demand	32	10	35	1	13	9	0.01
Total organic carbon	25	12	29	1	18	15	0.02
MBAS (detergent)			86		5	9	0.05
Oil and grease	38		22	0.7	16	23	
Ammonic nitrogen	24	4	55	3	10	4	0.04
Organic nitrogen	19	9	45	2	21	5	0.02
Kjeldahl nitrogen	21	6	51	2	15	5	0.02
Ortho-phosphorus		1	72		18	9	Nil
Total-phosphorus	50	0.7	35	1	9	4	Nil
Cadmium	82	2	5	0.6	5	5	0.001
Chromium	50	1	22	0.8	10	16	Nil
Copper	51	3	11	9	10	16	0.006
Iron	79	3	5	0.5	6	6	0.01
Mercury	9		71	2	13	5	
Lead	44	9	19	3	6	19	0.004
Zinc	29	18	8	2	21	22	0.009
Fecal coli							
winter	<0.01	Nil	87	0.2	0.01	13	Nil
summer	<0.01	Nil	85	0.2	0.01	15	Nil
Total coli							
winter	<0.01	Nil	91	0.1	0.05	9	Nil
summer	<0.01	Nil	84	0.2	0.1	16	Nil

Source: Mueller et al. (69).

TABLE 15-3

Present and Future Municipal Influent, Effluent, and Sludge Loads

	1972-73 Loadings (metric tons/day)			Future (2000) Loads (metric tons/day)					
				Influent		Effluent		Sludge	
Parameter	Influent	Effluent	Sludge	Load	% Change	Load	% Change	Load	% Change
Flow, m^3/s (mgd)	114 (2,600)	114 (2,600)	0.13	3,200	+23	3,200	+23	8	+170
Suspended solids	1,700	854	450	2,200	+29	200	-74	1,220	+171
Organic carbon	1,070	735	110	1,340	+25	400	-46	290[3]	+164
Total nitrogen	230	208	17	290	+26	220	+ 6	72	+324
Total phosphorus	52	48	4.7	70	+35	49	+ 2	21	+347
Lead	3.4	2.7	0.72	4.07	+20	3.26	+21	0.81	+ 12

Note: Effluent coliform concentrations are expected to remain at present levels so that future loadings will be proportional to flow.

Source: Anderson and Mueller (1).

TABLE 15-4

Average and Worst Mean Values of Pollutant Concentrations in NYC208 Study Area Transects

Transect	D.O, mg/l Avg.	D.O, mg/l (Min.)	BOD_5, mg/l Avg.	BOD_5, mg/l (Max.)	E. Coli/100 ml Avg.	E. Coli/100 ml (Max.)	F. Coli/100 ml Avg.	F. Coli/100 ml (Max.)	TKN, mg/l Avg.	TKN, mg/l (Max.)	NO_3-N, mg/l Avg.	NO_3-N, mg/l (Max.)	TSS, mg/l Avg.	TSS, mg/l (Max.)	T-PO_4-P,1 Avg.	T-PO_4-P,1 (Max.)
Hudson River	5.0	(2.0)	1.5	(2.0)	1500	(70000)	800	(10000)	0.8	(1.0)	0.4	(0.7)	10	(28)	0.3	(0.)
Kills	2.5	(0.7)	2.3	(4.6)	50000	(100000)	8000	(50000)	1.9	(2.3)	0.1	(0.2)	8	(10)	0.6	(0.)
Harlem River	2.0	(1.0)	2.2	(2.8)	100000	(500000)	10000	(100000)	1.3	(1.3)	0.2	(0.2)	10	(14)	0.8	(1.)
East River	4.0	(2.0)	1.8	(2.2)	5000	(40000)	900	(7000)	0.8	(1.0)	0.1	(0.2)	9	(14)	0.6	(0.)
Hackensack River	3.0	(2.5)	2.5	(2.6)	20000	(40000)	7000	(10000)	1.8	(1.9)	0.2	(0.2)	6	(7)	0.5	(0.)
Raritan Bay	6.0	(4.0)	2.0	(7.2)	800	(7000)	100	(500)	1.1	(1.5)	0.1	(0.2)	7	(13)	0.4	(0.)
Jamaica Bay	6.0	(4.0)	2.0	(7.2)	2000	(10000)	300	(1100)	1.0	(2.0)	0.1	(0.1)	5	(7)	0.3	(0.)
Rockaway Beach	7.0	(7.0)	1.0	(1.0)	60	(100)	30	(70)	0.3	(0.3)	No data		4	(5)	0.3	(0.)

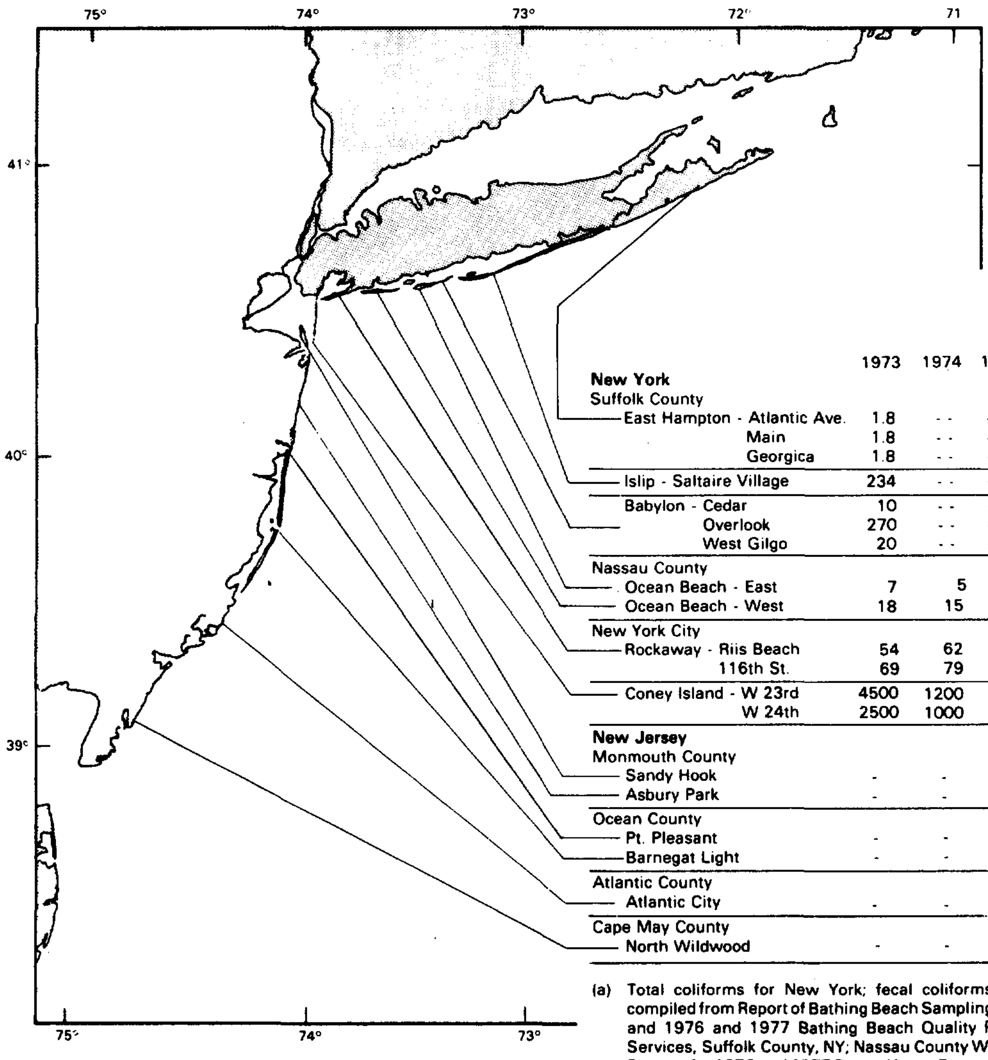
Transect	Zn,)/1 Avg.	Zn,)/1 (Max.)	Cd,)g/1 Avg.	Cd,)g/1 (Max.)	Cr,)g/1 Avg.	Cr,)g/1 (Max.)	Cu,)g/1 Avg.	Cu,)g/1 (Max.)	Pb,)g/1 Avg.	Pb,)g/1 (Max.)	Ni,)g/1 Avg.	Ni,)g/1 (Max.)	Hg,)g/1 Avg.	Hg,)g/1 (Max.)	PCB,)g/1 Avg.	PCB,)g/1 (Max.)
Hudson River	40	(140)	2.0	(5.5)	<25	(<25)	15	(27)	5	(10)	5	(10)	<1	(<1)	0.1	(0.2)
Kills	80	(110)	1.8	(4.0)	<25	(<25)	30	(54)	15	(22)	30	(50)	<1	(<1)	0.02	(<0.02)
Harlem River	5	(7)	<1	(<1)	<25	(<25)	12	(14)	10	(11)	10	(10)	<1	(<1)	No data	
East River	30	(60)	<1	(<1)	<25	(<25)	10	(12)	<5	(10)	6	(18)	<1	(<1)	0.15	(0.30)
Hackensack River	75	(90)	1.8	(2.5)	<25	(<25)	30	(47)	25	(37)	25	(25)	<1	(<1)	No data	
Raritan Bay	50	(130)	<1	(<1)	<25	(<25)	12	(20)	5	(9)	15	(29)	<1	(<1)	No data	
Jamaica Bay	35	(50)	1.0	(2.0)	<25	(<25)	9	(18)	<5	(5)	5	(10)	<1	(<1)	No data	
Rockaway Beach	20	(20)	4.0	(8.2)	<25	(<25)	10	(140)	5	(15)	3	(3)	<1	(<1)	No data	

Note: D.O., BOD_5, T. Coli, F. Coli, NO_3-N, PCB data obtained during Summer, 1977. TSS, T-PO_4-P data obtained during September, 1975. Heavy metals data obtained during Winter, 1976. Any "<" implies that the data obtained was below the limit of detectability.

Source: Hydroscience (45).

TABLE 15-5

Annual Geometric Mean Coliform Concentrations in Waters along Long Island South Shore (40).



	1973	1974	1975	1976	1977
New York					
Suffolk County					
East Hampton - Atlantic Ave.	1.8	- -	- -	b/60	b/10
Main	1.8	- -	- -	b/120	b/170
Georgica	1.8	- -	- -	b/20	b/370
Islip - Saltaire Village	234	- -	- -	b/20	b/120
Babylon - Cedar	10	- -	- -	b/150	b/70
Overlook	270	- -	- -	b/10	b/110
West Gilgo	20	- -	- -	b/240	b/100
Nassau County					
Ocean Beach - East	7	5	6	7	10
Ocean Beach - West	18	15	26	22	25
New York City					
Rockaway - Riis Beach	54	62	- -	9	22
116th St.	69	79	45	16	15
Coney Island - W 23rd	4500	1200	300	900	1000
W 24th	2500	1000	1000	1900	9000
New Jersey					
Monmouth County					
Sandy Hook	-	-	25	38	c/20
Asbury Park	-	-	39	30	c/170
Ocean County					
Pt. Pleasant	-	-	c/330	65	29
Barnegat Light	-	-	c/20	c/70	c/20
Atlantic County					
Atlantic City	-	-	c/130	c/20	34
Cape May County					
North Wildwood	-	-	33	17	28

(a) Total coliforms for New York; fecal coliforms for New Jersey. Data compiled from Report of Bathing Beach Sampling Analyses 1970 to 1973, and 1976 and 1977 Bathing Beach Quality Reports, Dept. of Health Services, Suffolk County, NY; Nassau County Water Quality Assessment Reports for 1976 and 1977 Report Years, Dept. of Health, Nassau County, NY; Beach and Harbor Water Sampling Program, Reports for 1973 through 1977, Bureau of Public Health Engineers, City of New York, NY; and annual reports of counties to State of New Jersey Cooperative Coastal Monitoring Program 1975 through 1977.

(b) Geometric means for all stations <10. Maximum value is included for year to year comparisons only.

(c) Geometric mean fecal coliform concentration <20. Maximum value is included for year to year comparisons only.

15.3.5.3 Nearshore Effects

As a result of the bacteriological contamination of shellfisheries, a quarantine has been placed on the shaded area shown in Figure 15-10. Some finfisheries have also been stressed, abandoned, or quarantined.

Other nearshore effects are evident in the sediments. Bathymetric surveys in 1936 and 1973 reveal an accumulation of 93 x $10^6 m^3$ (122 x 10^6 cu yd) of dredged material and up to 10 m (30 ft) of shoaling (see Figure 15-13). Elsewhere over the 718 km^2 (277 sq mi) Bight Apex, there has been general erosion, so that the net change is 1 x $10^6 m^3$ (1.3 x 10^6 cu yd) of deposition (25, 26).

Most of the 15 x $10^6 m^3$ (20 x 10^6 cu yd) of dredge spoils dumped annually is dispersed and moves down slope. Figure 15-14 shows higher concentrations of mercury, cadmium, copper, and zinc in superficial sediments. Concentrations in sediments are found in and near topographic lows (37). Mercury is uniquely distributed, presumably because it is bound to finer sediments influenced more by near-surface coastal currents while they settle to the bottom than by subsequent movement. Concentrations of 1500 ppb of polychlorinated biphenyls are also found in sediments between the sewage sludge and dredge spoil dumping sites.

15.3.5.4 Offshore Effects

The effects of ocean dumping on biological communities are limited to the Christiansen Basin and the nearshore zone. Sludge dumping is a minor factor (59). Most effects on the biota of effluents are due to contaminants from the Hudson, Raritan, and Passaic rivers; commercial and sportfishing, and the large natural variations. In 1983, liquid municipal discharges accounted for 35-70 percent of individual contaminant inputs, sludge dumping accounted for 5-15 percent, and the remainder came from direct industrial discharges and runoff. As the contaminants flow seaward, the assimilative capacity of the waters increases and these effects decrease as they move into nearshore coastal waters and thence to the open ocean.

Fine-grained and low density sewage sludge and effluent particles tend to accumulate only in topographic depressions of low energy where biota are changed by (1) organic enrichment, (2) the accumulation of fine-grained sediments, (3) low dissolved oxygen levels, and (4) toxic metals and hydrocarbons associated with contaminated sediments (84). However, attempts to isolate a dominant causal factor for the observed benthic alterations have not been successful (91).

Winter flounder and other fishes, rock crabs, lobsters, and other shellfish stressed by contamination of the Bight may be prone to disease. The problem of fin-rot in fishes has gained widespread public attention; fortunately, fin-rot has declined recently (72). The shellfish diseases, "black gill" and "exoskeletal erosion," affect lobsters, shrimp, rock crabs, horseshoe crabs, and other organisms with hard outer skeletons, and may be linked to the organic content of underlying sediment.

Fig. 15-13. New York Bight Apex Showing Depths in Meters.
Source: Freeland, et al. (25).

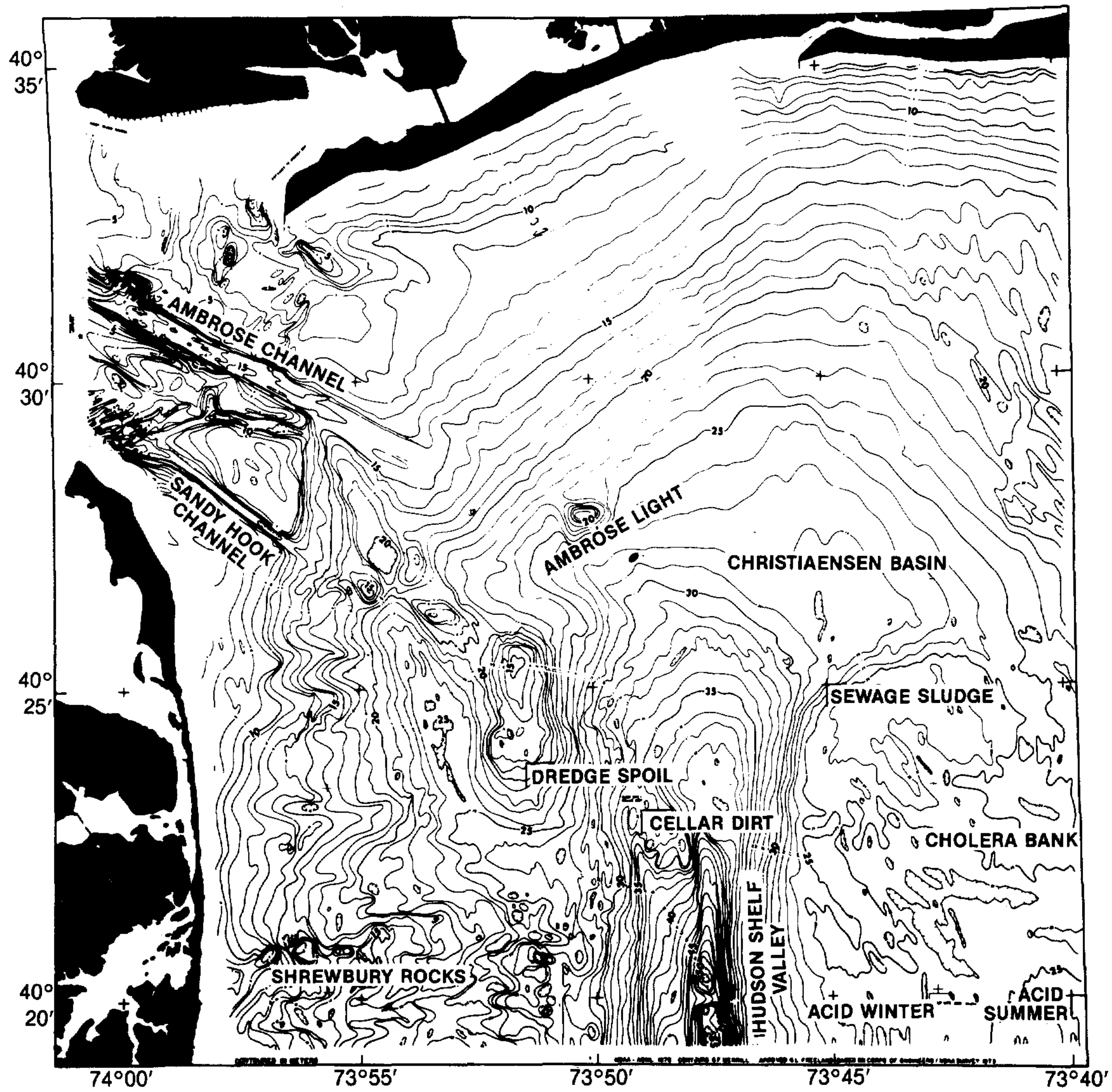
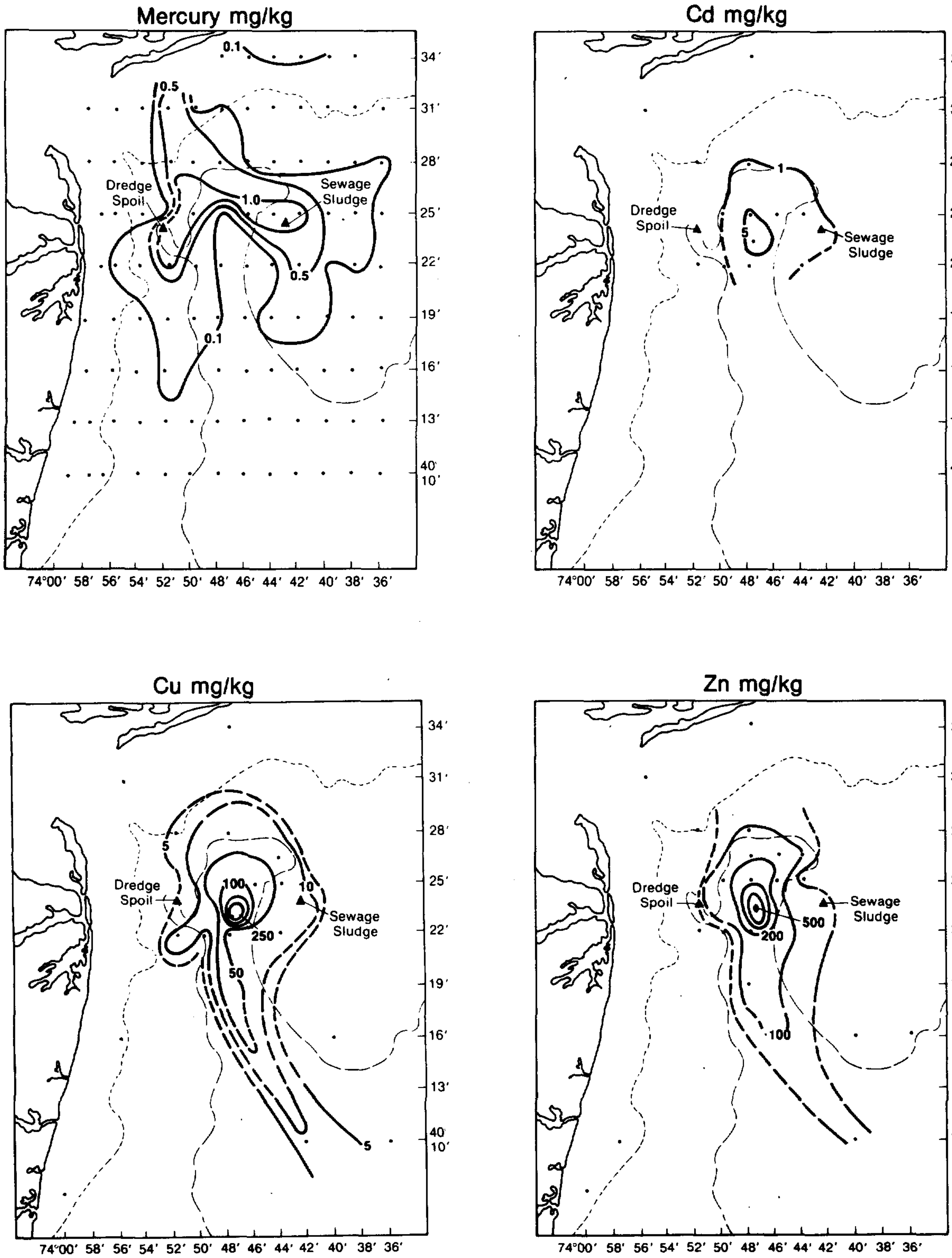


Fig. 15-14. Heavy Metals in New York Bight Sediments (37).



Koditschek (54), Timoney et al. (95), and Litchfield (58) have reported abnormal tolerances of bacteria from sediments for some metals and antibiotics. Large numbers of Bacillus from the most contaminated area of the inner Bight were resistant to mercury and ampicillin in concentrations that would kill normal Bacillus strains. Bacillus from inshore coarse-grained less contaminated sediments were less resistant; still less resistance was found in samples from near the outer edge of the continental shelf. Furthermore, one-third of the Bacillus strains from the dump site area metabolized mercury and released it as elemental mercury. This could be a significant mechanism for promoting a flux of mercury from sediments to the water column (95).

15.3.5.5 Acute Effects of Waste Discharges

The chronic effects of pollution of the New York Bight are serious enough but the occasional severe episodes attract public attention and demand scientific and engineering responses. Two examples include the sudden stranding of grease balls and plastics from sewage on Long Island beaches; and the slow development of anoxic bottom waters and fish and shellfish kill on the New Jersey shore, initially assumed to be due solely to nutrients in waste discharges.

In June 1976 almost all of Long Island's major public ocean beaches were temporarily closed to swimmers because of the stranding of floating trash of obvious sewage origin. The event took place with the following stages: (1) Throughout May, the Hudson River discharge was far above normal. (2) As a result of an oil spill in Upper New York Bay in early May, tar balls washed up on beaches from Jacob Riis Park to Fire Island. On May 26, a storage tank ruptured at Jersey City, New Jersey, and more oil was spilled into the Hackensack River and into the wetlands of the Hackensack Meadows. (3) On June 2, two sewage sludge storage tanks on Pearsalls Hassock exploded; 1 million gal of sewage sludge flowed into the water, and 1.1 million gal spilled onto the land. These tanks had been routinely emptied from the bottom for twelve years, leaving an unknown amount of floating material. (4) Pier fires on June 3 and 11, at Weehawken, New Jersey, and Manhattan, New York, dumped large amounts of wreckage and debris into the water that escaped recovery. (5) Southerly winds persisted throughout most of June. (6) Meanwhile, the usual discharges of raw and treated sewage, occasional spills from refuse disposal operations, commercial shipping, recreational and commercial boating, and ocean dumping continued. (7) Twenty miles of Fire Island beaches were closed on June 15 and the remaining coastal beaches by June 22. (8) All beaches were reopened by July 1.

There was no correlation between quantities of stranded grease balls, plastics, rubber, and other materials and bacteriological pollution. Water samples collected along the beaches during the peak of the pollution problem showed total coliform levels well within the New York State standard for swimming (2,400 Most Probable Number [MPN]/100 ml). Fecal coliform MPN levels were less than 100 /100 ml, and usually less than 10 /100 ml. High concentrations in individual water samples were caused by known local sources. Beaches were closed to swimming as a precautionary measure, pending results of bacterial analyses, because of the suspected origin of the waste materials.

Although grease and tar balls contained up to 1 million total and fecal coliform bacteria per 100 grams of sample, no Salmonella or other pathogens were found.

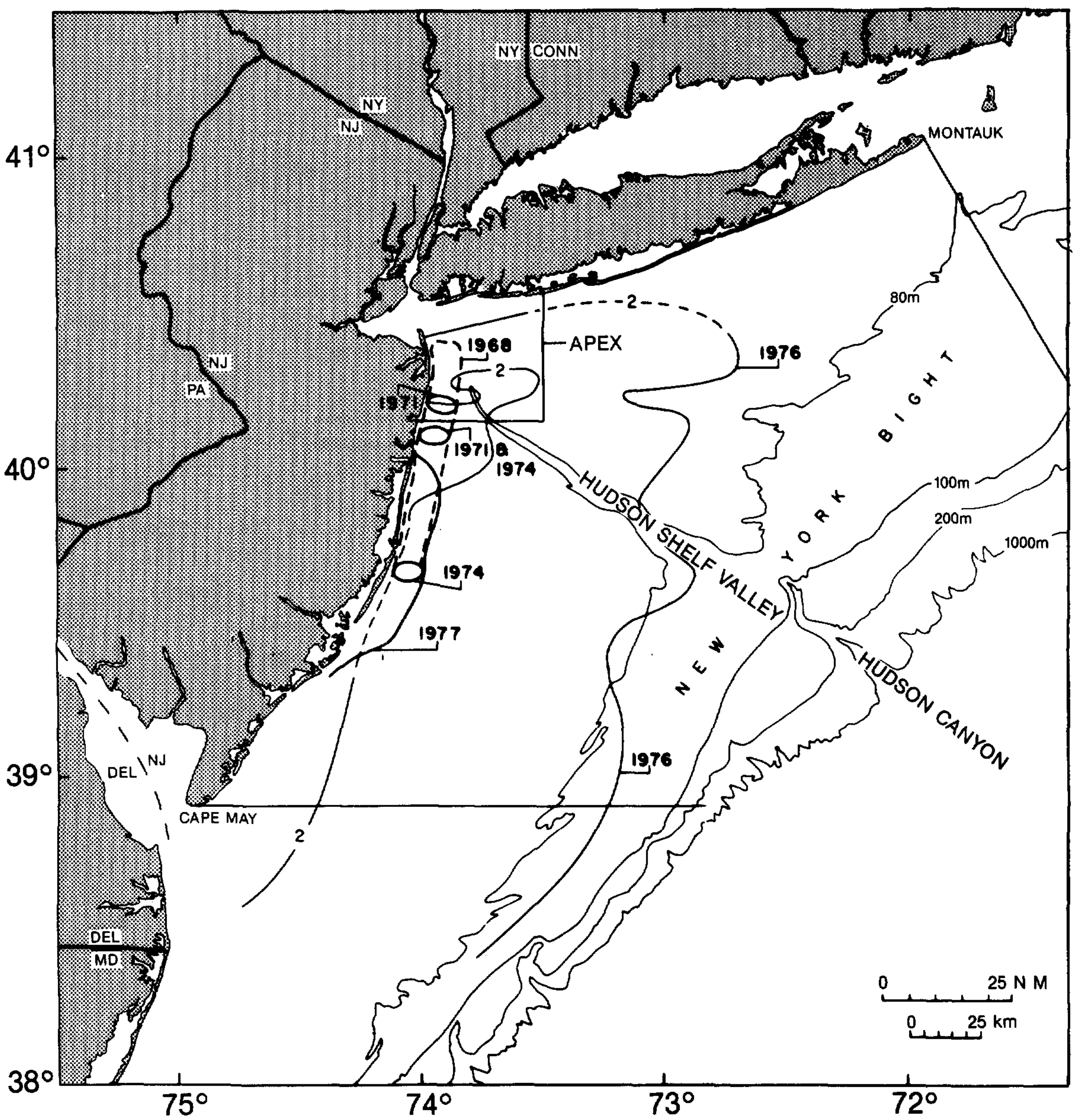
Within Hempstead Bay, the explosion of the sludge storage tanks had little impact upon bay water quality. Daily sampling at twenty-six stations, from June 3 to 7, revealed that coliform levels were generally normal or below normal, except on June 3, one day after the explosion, when coliform levels were three to five times above normal. Dissolved oxygen levels were even less affected. A smaller sludge accident occurred on June 18, 1976, when 10,000 gallons of sludge was spilled from a storage trough at the Lawrence sewage treatment plant and eventually entered the western end of Reynolds Channel. This accident also produced a temporary increase in coliform bacteria levels in the bay, with a quick return to normal levels soon after the incident (74).

In sum, there were multiple sources for the grease balls and for the plastic sanitary materials found on the beaches, and there were wind and surface current mechanisms to transport them there. Although the absolute contributions from individual sources is unknown, the timing of the sludge tank explosions followed by reporting of unprecedented amounts of floatables, whose sewage origin was incontestable, brought to public and official attention the narrow limits within which present waste disposal systems operate satisfactorily. Minimizing the future probability of such incidents will require a combination of consumer materials selection, waste segregation, and removal at the sources of materials such as grease presently discharged through garbage grinders.

The 1976 anoxia and fish kill event was the most severe on local record and possibly the first incident of its kind along an open coastline where waste discharges were implicated. Approximately 8,600 km^2 (Figure 15-15) along the continental shelf off the New Jersey coast experienced mass mortalities of benthic organisms from July through October, 1976 (92), moving southerly from near Monmouth Beach in early July to Atlantic City by late July. Surf clams, ocean quahogs, finfishes, lobsters, and sea scallops (in decreasing order) were the commercial species most affected. Figley et al. (24) estimated losses to the fishing, processing, and marketing industries of $7.9 million during the year and calculated that the total near-term losses would reach about $62.5 million. Figley further assumed up to seven years for recruitment so that potential losses would exceed $550 million. The latter estimate has proven to be exaggerated. While surf clam harvests (the major component of the industry) were reduced from 1976 through 1978, subsequent harvests of the 1976 year class have sustained and are expected to improve the industry for another five to ten years (71, 80). It is believed that the 1976 anoxia event eliminated that year's predators so that more clams survived. Meanwhile, the immediate economic loss to the sport fishery was estimated to be $3.7 million. Summer flounder fishing along the New Jersey coast was excellent, because their normal movement to offshore areas was prevented by the anoxic water.

Operational monitoring (see Chapter 14) can be based on 1976 and subsequent events (see Figure 15-15) for assessing the probability on a time

Fig. 15-15. Oxygen Depletion in New York Bight, 1968-76. Isopleths show areas where bottom waters contained less than 2 ml/l dissolved oxygen. Source: Swanson and Sinderman (92).



scale of about two months, of future mass benthic mortalities. Swanson and Sinderman (92) reviewed the 1976 series of anomalous natural events that extended from January through August. They included large-scale blooms of the demoflagellate ceratuim tripos, early unseasonal warming of the atmosphere and ocean, large and extended river runoff, early development of density stratification and deoxygenation of the lower larger, atmospheric pressure gradients and wind fields, together with abnormally northerly currents and upwelling, lower-layer respiration of sinking C. tripos, deoxygenation and sulfide generation in the bottomwaters. There is no evidence to show that waste inputs triggered the anoxic event. However, there had been slight evidence of a gradual decrease over the preceding years in bottom DO which prompted O'Connor (79) to report that

> the sensitivity of the system might therefore be changing such that a slight imbalance (either due to natural causes or increases in waste loadings) in the 'normal' cycle of environmental conditions is sufficient to drive the system towards anoxia with increasing frequency. If this is true and if the slight decreasing trend in bottom DO over the last three decades is real and monotonic, then there is an increasing probability of low DO events in any year.

15.3.6 Monitoring and Reappraisal

O'Connor's warning is sobering. It comes at a time of increasing competition for resources and a decreasing margin for error in allocating them. These include construction of conventional sewers and treatment plants that are variously estimated to have cost some $2 billion since 1950 and to require at least $3 billion (1980 dollars) more by the year 2000 (44, 48, 98). Total costs of sewerage for New York City may be estimated assuming (i) annual investment costs of $500 million per year, (ii) an average capital recovery factor of 10 percent, (iii) annual operation maintenance costs of 30 percent of capital costs, and (iv) marginal costs of 65 to 95 lcd (17 to 25 gcd) of flushing of 400 million per year (16). Thus, total estimated average costs of sewerage (not including plumbing costs) are $500 million capital recovery, $150 million O&M, and $400 million flushing water totaling $1,050 million per year in 1980 dollars (37, 38, 39).

Receiving water benefits from these costs are elusive. The Mueller and Anderson data previously cited reveal that future waste loadings to the New York Bight will remain at essentially present levels. These are not much different from 1950 levels, although local improvements within the harbor in the coliform and dissolved oxygen levels have been observed since 1974 except for anoxia in Arthur Kill (48). Meanwhile, fisheries have been lost owing to degraded water quality for some 120 years and more recently because of high concentrations of PCB's and other chemicals in the fish (31, 81).

Probably the main, and certainly the most appreciated, improvement in the Bight, came with the elimination of garbage dumping onto surface waters in 1934.

15.4 Southern California Bight

In 1906, the City of Los Angeles constructed the first major ocean outfall in what has been named the Southern California Bight (Figure 15-16). This 6-ft diameter outfall was laid on a 500-ft trestle and discharged beyond the surf zone onto the sea surface. Since then, successively larger outfalls have been built to discharge wastes as much as 7 mi (11.2 km) offshore into bottom waters as deep as 330 ft (100 m). The total 1984 discharge from seven outfalls within the purview of the Southern California Coastal Water Research Project, SCCWRP (6), was 1166 mgd (51 m^3/s). These outfall systems intercept what would otherwise be a larger number of smaller, diffuse waste discharges along the shoreline, treat the wastes at central facilities and then partially rediffuse wastes through outfall systems into coastal receiving waters.

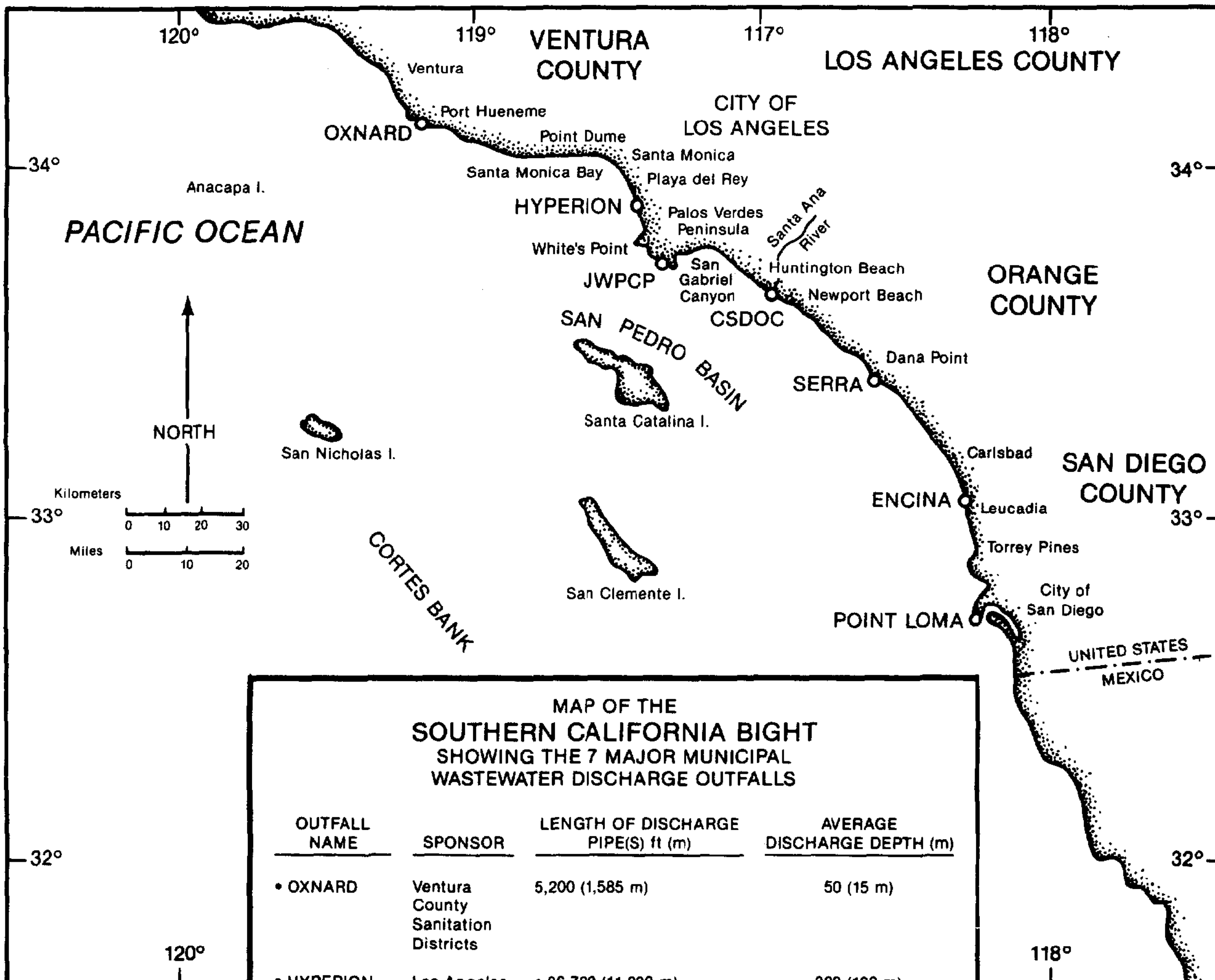
Table 15-6 reveals the changes in waste flows and characteristics from 1971 through 1983. Whereas flows have increased by about 15 percent, effluent suspended solids and BOD have decreased by about 15 and 10 percent, respectively, with upgraded municipal treatment and disposal facilities with an estimated 1983 replacement value of about $1.2 billion and another $2 billion projected by 1990 for the Los Angeles metropolitan area alone (29). Meanwhile, improved source controls have reduced the discharges of DDT by 99 percent and PCB's by 80 percent. Arsenic, cadmium, chromium, copper, mercury, nickel, and lead have been reduced by 50 to 70 percent. Concentrations of wastewater constituents for 1983 are listed in Table 15-7. The effects of waste discharges into the Bight have been characterized by SCCWRP since 1971 and reported on in a series of annual reports edited by Bascom (4, 5, 6). Figure 15-17 shows variations in bottom sediments and fauna along the coastline related to waste discharges. The figure also shows that constituents regarded as contaminants are present at all control locations; natural background levels are always reported (4).

Figures 15-18 A and B and Figure 15-19 show in greater detail the effects of waste discharges in the vicinity of Los Angeles. The infaunal trophic index (ITI, see Chapter 14), is considered a sufficiently sensitive parameter to be used for predicting changes resulting from increasing or decreasing annual loadings of wastes. The results of Mearns, Word, and Young (63, 64, 100) are summarized in Figure 15-20. In contrast to the effects of waste discharges to estuarine or coastal areas with limited circulation, the effects here extend over small areas. The prediction methodology appears valid for other marine environments and pollutants, although the numerical values for ITI or other parametric thresholds may be different. For example, Mearns and O'Connor (62) have related lengths of coastlines affected by oil spills, areas of water column eutrophication, and areas of receiving water toxicity to invertebrate embryos to the total mass of oil, nitrogen, and BOD, respectively. The effects of the discharge upon the benthic biomass (Figure 15-18B) correspond well with those of the ITI. In addition, high standing crop areas in shallow water near Oxnard and in deep water in Santa Monica Bay are shown which have no relation to waste discharges. The inverse relationship between the ITI and biomass is predictable and consistent with fertilized areas such as a rice paddy or other monocrop activity.

The results of the outfall design and source control measures in the Southern California Bight can be viewed from both socioeconomic and

(text cont. on p. 285)

Fig. 15-16. Locations and Characteristics of the Seven Discharges Summarized in This Report. Source: Bascom (5).



MAP OF THE
SOUTHERN CALIFORNIA BIGHT
SHOWING THE 7 MAJOR MUNICIPAL WASTEWATER DISCHARGE OUTFALLS

OUTFALL NAME	SPONSOR	LENGTH OF DISCHARGE PIPE(S) ft (m)	AVERAGE DISCHARGE DEPTH (m)
• OXNARD	Ventura County Sanitation Districts	5,200 (1,585 m)	50 (15 m)
• HYPERION	Los Angeles City Bureau of Sanitation	• 36,722 (11,200 m) ("7-mile pipe") • 27,525–4,000* (9,600 m) ("5-mile pipe")	320 (100 m)
• JWPCP	Los Angeles County Sanitation Districts	• 8,000–1,200* (2,800 m) (90-in) • 12,000 (3650 m) (120-in)	200 (60 m) 200 (60 m)
• CSDOC	Orange County Sanitation Districts	27,400 (8,350 m)	185 (55 m)
• SERRA	South East Regional Reclamation Authority	8,700 (2,650 m)	170 (50 m)
• ENCINA	Encina Water Pollution Control Facility	5,300 (1,600 m)	100 (30 m)
• POINT LOMA	City of San Diego	11,500–1,360* (4,000 m)	200 (60 m)

*Pipe length—length of each arm of a Y shaped diffuser

TABLE 15-6

Combined Annual Mass Emission Rates for Seven Southern California Municipal Wastewater Discharges, 1971-1983[a]

	1971	1972	1973	1974	1975	1976	1977	1978	1979	1980	1981	1982	1983
Flow													
MDG	931	922	955	967	985	1,027	966	1,015	1,054	1,097	1,097	1,134	1,166
L/day x 10	3,524	3,490	3,615	3,360	3,728	3,889	3,658	3,840	4,000	4,160	4,160	4,292	4,414
General constituents (mt/year)													
Total sus. sol.	288,000	279,000	270,000	264,000	287,000	288,000	244,000	256,000	243,000	233,000	226,000	227,000	247,000
5-Day BOD	283,000	250,000	217,000	222,000	237,000[b]	259,000[b]	244,000	237,000[b]	246,000[b]	260,000[b]	264,000[b]	269,000[b]	255,000[b]
Oil and grease	63,500	60,600	57,400	54,700	57,420	59,100	49,000	49,000	45,000	39,000	37,000	31,900	36,300
NH-3-N	56,600	39,900	45,900	37,000	36,620	37,350[b]	41,200	39,500	41,200	42,000[b]	41,000[b]	44,000	40,600[b]
Trace metals (mt/year)													
Silver	17.7	21.1	29.0	21.7	25.7	20.2	34.3	32.3	42.2	30.8	27.9	25.9	25.6
Arsenic	ND	ND	ND	20.9[c]	11.9[c]	10.5[c]	14.0	14.5	15.4	10.6	12.2	8.7	10.1
Cadmium	57.3	33.8	48.3	55.4	50.0	45.0	42.4	44.8	42.3	39.5	31.7	21.2	23.6
Chromium	676	673	695	690	580	593.0	366	280	237	275	187	203	164
Copper	559	485	509	575	511	507	412	417	359	336	339	286	247
Mercury	ND	ND	ND	3.1[c]	2.2[c]	2.6[c]	2.8	1.9	2.5	1.9	1.8	1.2	1.2
Nickel	339	273	318	314	234	307	264	320	256	224	167	169	165
Lead	243	226	180	199	198	191	152	219	223	175	130	123	99.7
Selenium	ND	ND	ND	17.75[b]	16.9[d]	22.0[d]	23.0[d]	23.0[d]	7.7[d]	10.5[d]	15.3[d]	9[d]	10[d]
Zinc	1,880	1,210	1,360	1,320	1,142	1,064	837	905	724	730	540	549	505
Chlorinated hydrocarbons (kg/year)													
Total DDT	21,700	6,600	4,120	2,120	1,989	1,673	920	1,110	760	644	474	289	218
Total PCB	8,730	9,830	4,620	9,390	6,011	4,310	2,183	2,510	1,190	1,129	1,250	857	1,440

a. Oxnard included only since 1975. Serra and Encina included since 1982.
b. Hyperion 7-mile effluent excluded.
c. CSDOC data not included.
d. Total for Hyperion and JWPCP only.

Source: Bascom (5).

TABLE 15-7

1983 Effluent Characteristics
(concentrations in mg/l, except as noted)

	JWPCP	Hyperion 5	Hyperion 7	CSDOC	PL Loma	Oxnard	Encina	SERRA
Flow (mgd)	353.3	411	4.23	223.7	129.6	19.5	14.2	10.5
General constituents								
Suspended solids	188.6	102	8,100[a]	95.8	98.4	34.3	75.8	10
Settlable solids	0.908	1.3		2.1	0.9	<0.1	1.01	0.7
BOD_5	175.5	183		129	124	26.8	105	22.2
Oil and grease	27.94	19.0		15.8	23.8	9.5	18.0	1.7
NH3-N	40.71	14.5		22.8	26.2	11.6	22	
Organic-N	12.02	8.37	314			5.2		
Total-P	8.78	6.46	181					
MBAS[b]	5.60	3.22			3.61			
CN-	0.038	0.044	0.289	<0.02	0.007	0.038	0.014	0.01
Phenols	2.573	0.064	0.177	0.039	0.033	0.010	0.031	<0.01
Turbidity (JTU)	131.3	69		61	71.3	18.2	40.7	3.8
Toxicity (TU)	5.06	0.83		0.38	1.19	0.79	1.14	0.847
Metals								
Silver	0.0104	0.015	0.756	0.015	0.016	0.009	0.0029	0.03
Arsenic	0.0063	0.007	0.183	0.0036	0.003	0.006	<0.002	<0.01
Cadmium	0.0152	0.013	0.480	0.0108	0.012	0.011	<0.0009	<0.01
Chromium	0.144	0.08	4.21	0.0632	0.02	0.013	0.009	<0.01
Copper	0.127	0.15	9.28	0.172	0.10	0.013	0.063	<0.01
Mercury	0.00088	0.0006	0.029	0.00038	0.0004		<0.0001	0.0007
Nickel	0.153	0.09	1.76	0.06	0.048	0.044	0.014	<0.01
Lead	0.083	0.03	1.9	0.06	0.06	0.020	0.060	<0.01
Selenium	0.0134	<0.005	<0.05					
Zinc	0.521	0.17	10.9	0.153	0.20	0.017	0.178	0.016
Chlorinated hydrocarbons ()/L)								
Total DDT	0.375	0.03	0.55	0.02	0.046			
Total PCB	0.508	<0.2	<2.0	1.23	<0.002	<10		ND[c]
TICH[d]	1.323	4.9	5.07	1.41	0.107	<1	<1	ND

a. Total solids.

b. Methylene blue active substances.

c. Not detected.

d. Total identified chlorinated hydrocarbons.

Fig. 15-17. Regional Effects of Discharges to the Southern California Bight
Source: Word and Mearns (100).

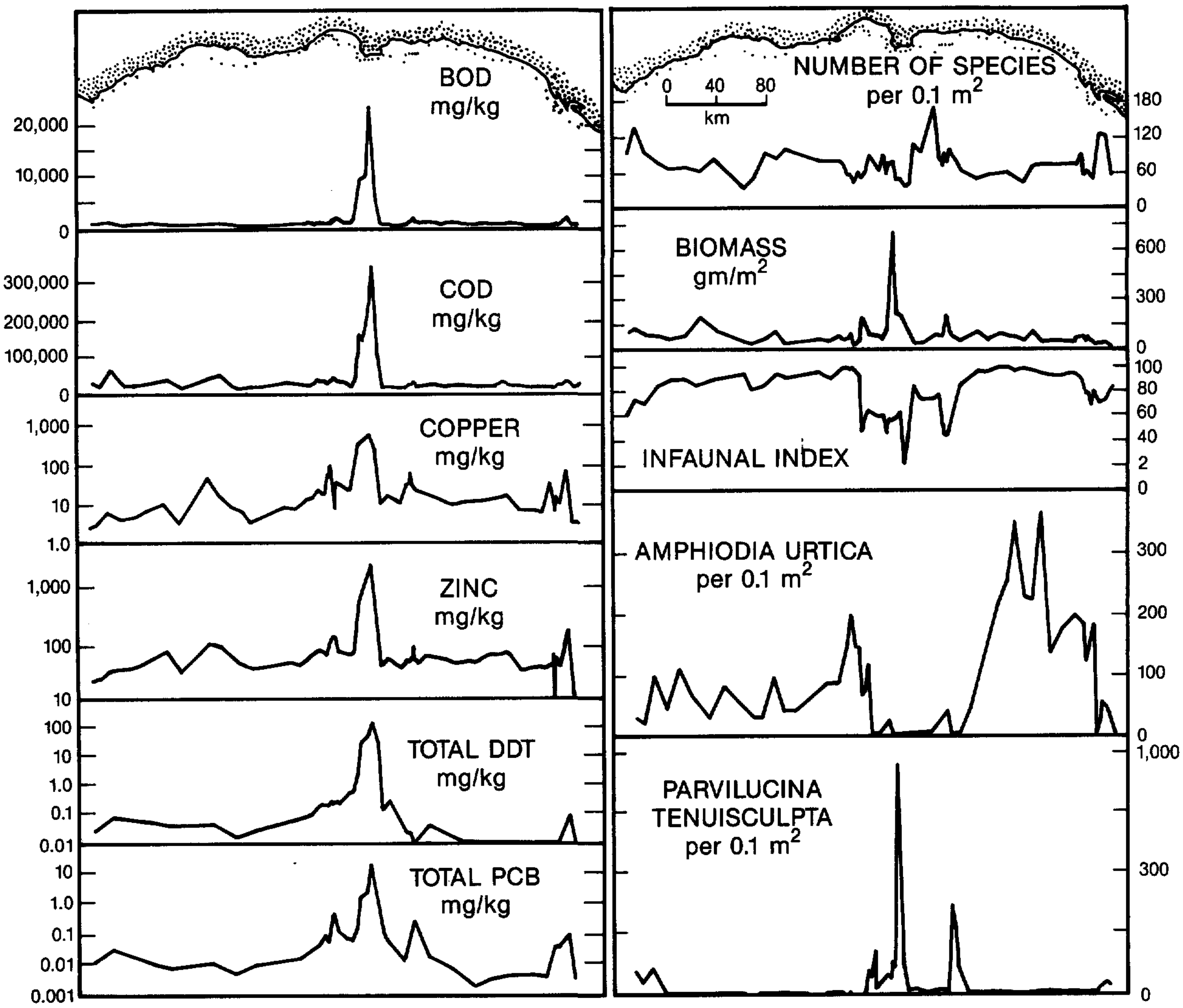
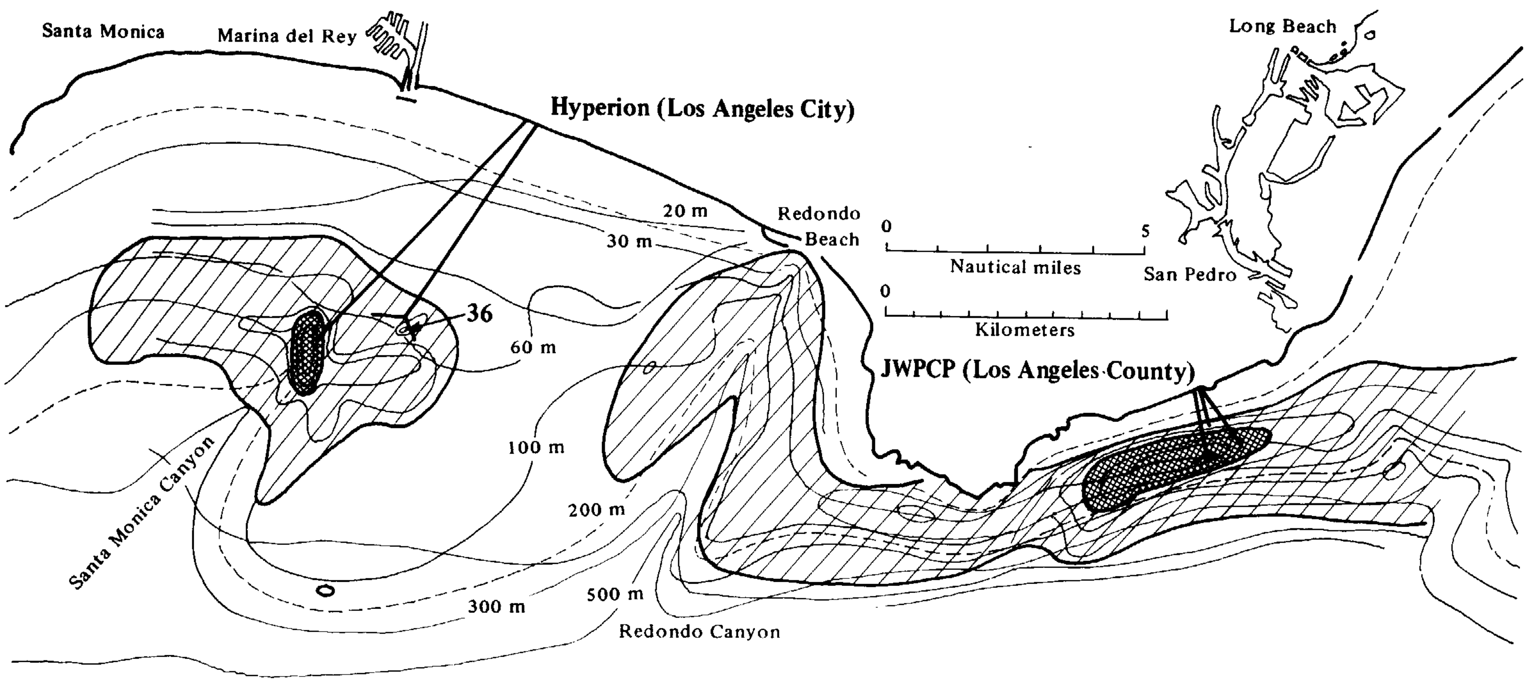
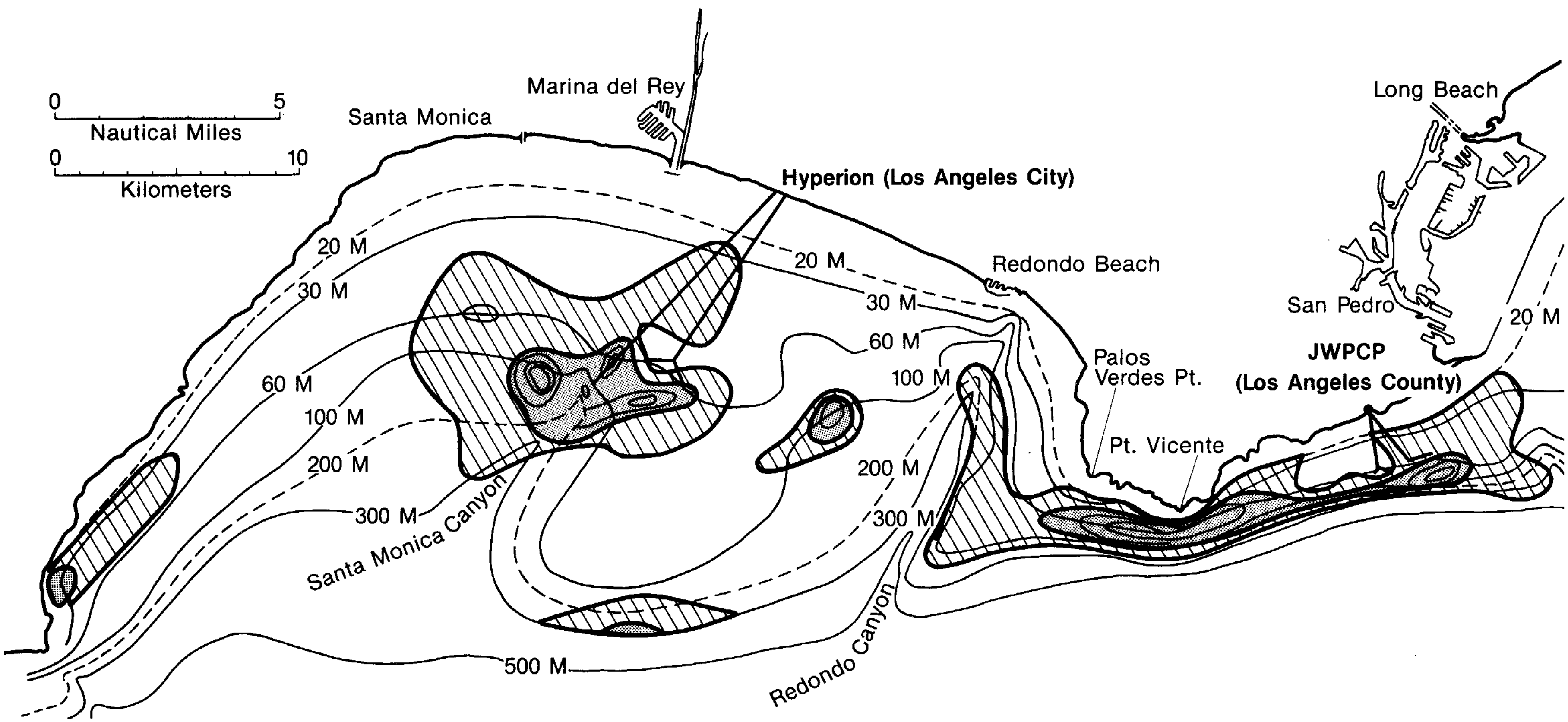


Fig. 15-18A. Effects of Los Angeles County and City Waste Discharges on Bottom Life in the Pacific Ocean
Source: Bascom (4).



Note: Degraded area (I.T.I., 0 to 30); nged area (I.T.I., 30 to 60). Depths are in meters.

Fig. 15-18B. Effects of Los Angeles County and Los Angeles City Waste Discharges on Bottom Biomass in the Pacific Ocean
Source: Bascom (4)



Note: Infaunal biomass ▩ Highest standing crop (>200 gm/²); ▧ high standing crop (100 to 200 gm/m²); ☐ background (~5 to 40 at < 20m depth; ~20 to 100 at > 20m depth except near JWPCP outfalls).

Fig. 15-19A. Effects of Waste Discharges on Sediments in Santa Monica Bay
Source: Bascom (4).

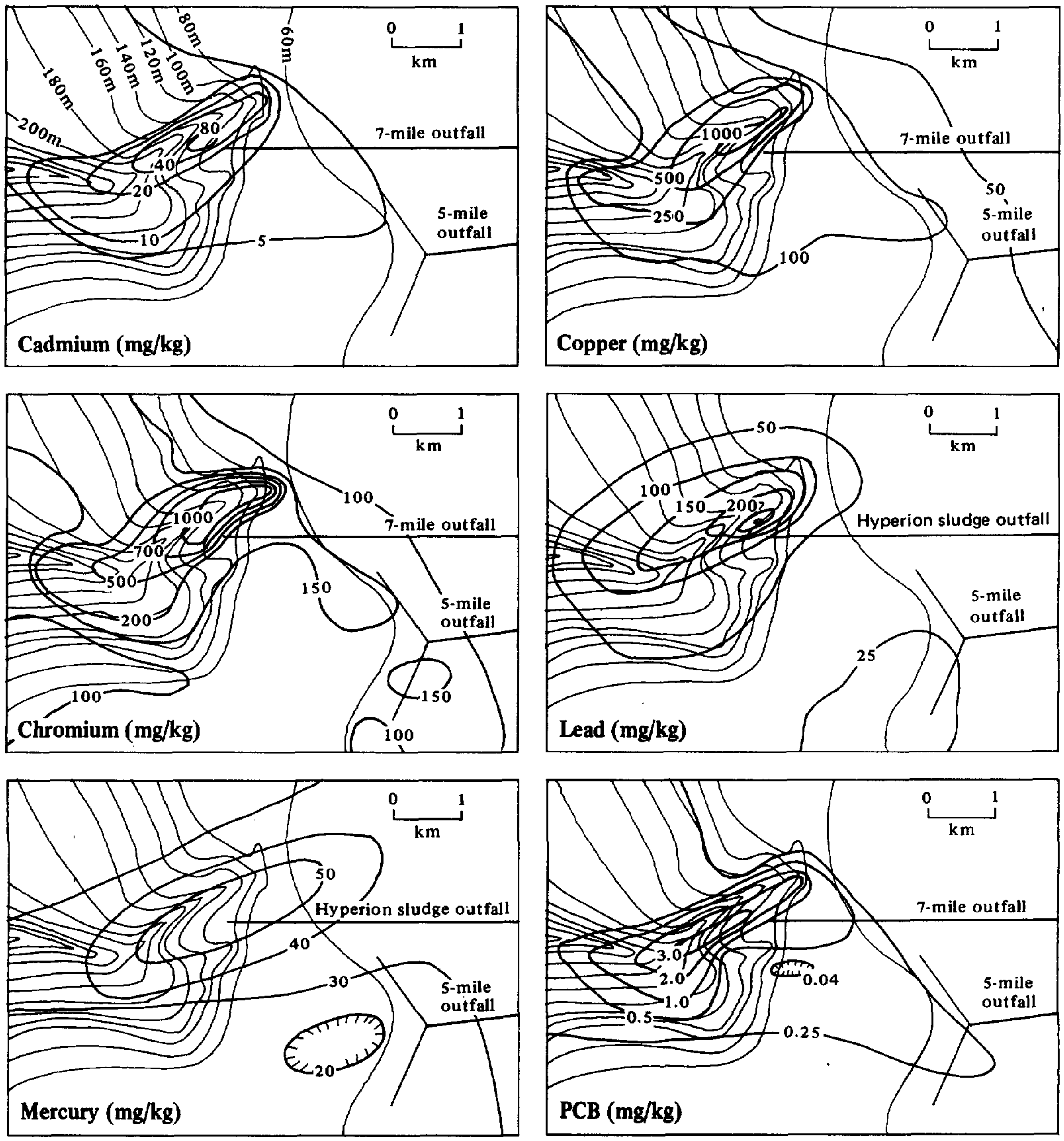


Fig. 15-19B. Effects of Waste Discharges on Sediments in Santa Monica Bay
Source: Bascom (4).

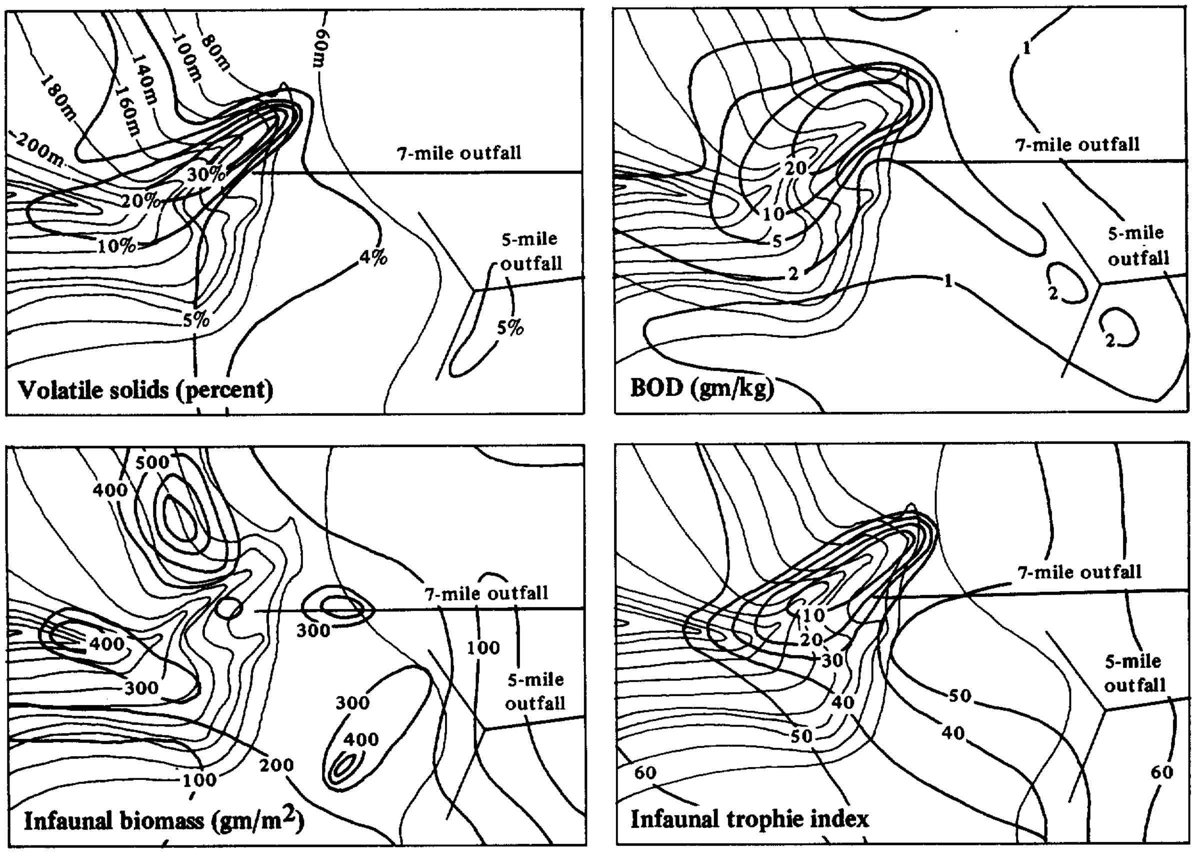
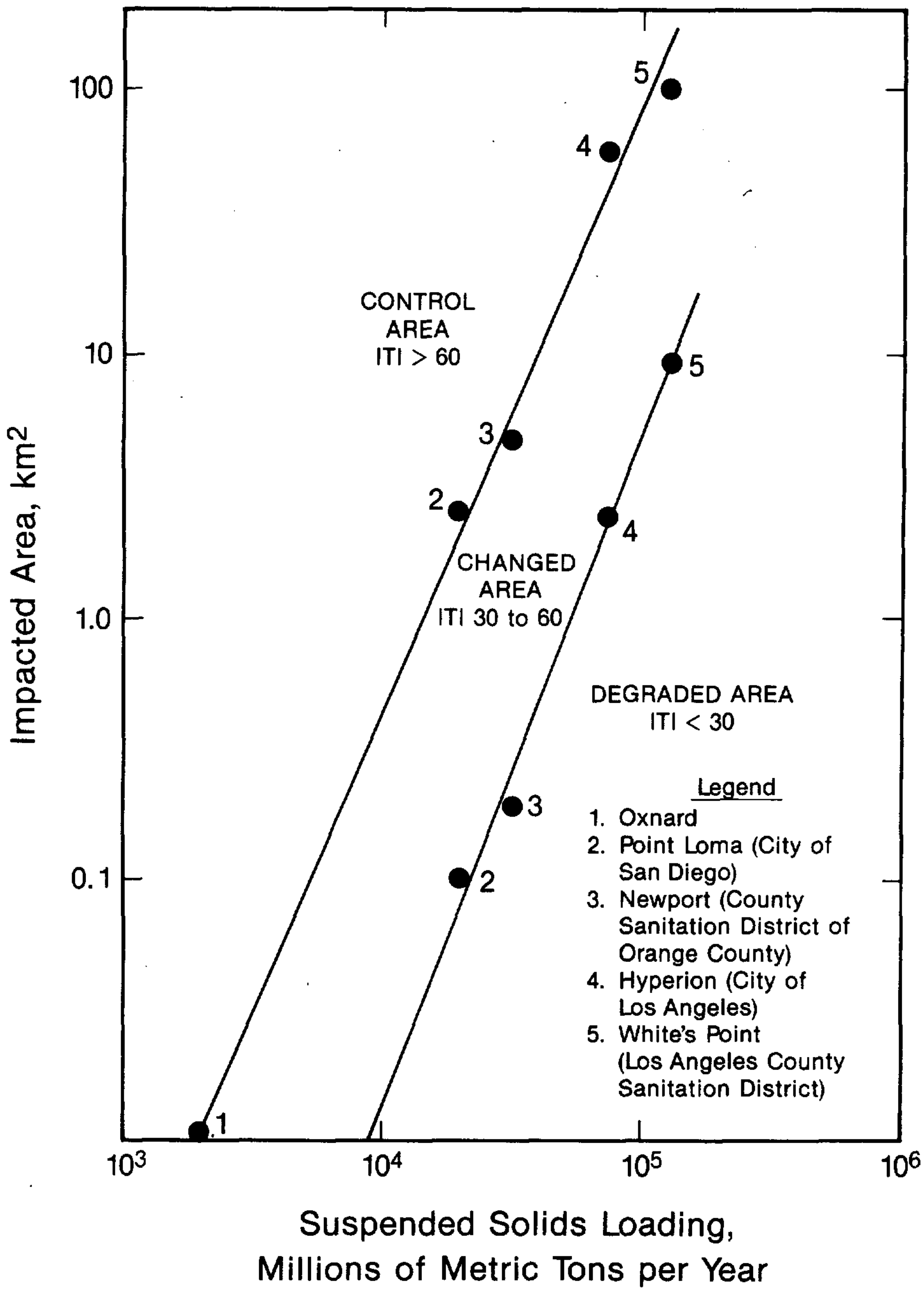


Fig. 15-20. Areal Extent of Impacts of Municipal Waste Discharges on Infauna of the Southern California Bight.
Source: Mearns and Word (63).



environmental perspectives. Beaches that were quarantined prior to the 1950s are now among the least contaminated in the world and have generated substantial tourism revenues for the coastal communities. The offshore discharges have not adversely affected fisheries since the Bight continues to support a large commercial fishing industry--more than 70 percent of all commercial fish landed in Southern California are caught within 50 km of the three largest outfalls. Fish stocks in the Southern California Bight have remained essentially constant except where overfishing or climatic change has caused declines or where increases have been attributable to sewage discharges (64).

From a public health perspective, there has never been a documented case of human illness caused by bacteria or viruses from outfalls in the open coastal waters of Southern California (4, 30). Daily monitoring of beach and near-shore locations has shown that coliform levels rarely exceed those established for recreational usage and thus no beaches or shellfish harvesting areas have been closed in recent years (see Chapter 14). Because mixing characteristics are different in the winter months, discharges are occasionally chlorinated during these months to maintain state water quality standards for shellfish.

Long-term monitoring in the Southern California Bight supports the conclusion that the treated sewage effluent outfall discharges are not causing unacceptable effects on the marine environment. In areas where adverse environmental changes have occurred in the past, there has been a measurable recovery once the source was identified and control measures were put in place. For example, during the 1950s and 1960s, beds of giant kelp decreased in size at many Southern Californian coastal locations and became almost extinct near White's Point, where the diversity and abundance of other seaweeds, macroinvertebrates, and fishes were also greatly reduced. These effects were attributed to inshore accumulation of wastewater solids, overgrazing by sea urchins, and some abnormally warm surface temperatures. By 1973, these conditions began to reverse and by 1978, there was a marked increase in diversity and abundance of the biota. Transplanting combined with reductions of 38 percent for waste suspended solids and up to 98 percent for DDT has resulted in 1.2 square kilometers of kelp.

Residual inputs of persistent organic chemicals are expected to continue, with possible sublethal effects on marine organisms. The loadings of these chemicals and of metals are expected to decline further with improved industrial and municipal pretreatment.

Meanwhile, additional facilities for waste treatment and ocean disposal are under consideration or construction for the Los Angeles Metropolitan area. These include a state-of-the art (Carver-Greenfield patent) sludge-processing plant and additional secondary treatment at the City of Los Angeles Hyperion Treatment Plant, expanded secondary treatment capacity at the Los Angeles County Sanitation Districts Joint Water Pollution Control Plant, and a sludge outfall discharging at 400-m depth from the County Sanitation Districts of Orange County. Numerical modeling of the deoxygenation effects of the latter was roughly analogous to that for a vertical oxygen sag curve (10, 49).

15.5 The Bosporus and Sea of Marmara

Istanbul, the largest city in Turkey, was founded by Megarian Greeks in 657 B.C. as fabled Byzantium. In A.D. 330 it became Constantinople, capital of the Eastern Roman (later Byzantine) Empire and in 1453 of the Ottoman Empire. The city is on the northern shore of the Sea of Marmara and lies on both sides of the Bosporus, the 31-km-long (17 nm) strait that joins the Marmara with the Black Sea and separates Europe and Asia.

European Istanbul is divided by the Golden Horn, a tidal estuary and excellent harbor. The Golden Horn watershed includes rolling hills, valley-floor villages, and the Kagithane and Alibey rivers (55, 94). Westward, along the Marmara, the terrain ascends gently from the coast to a plateau cut by narrow valleys perpendicular to the shoreline. Northward, along the Bosporus, the coastline is steep and cut by sharp, narrow valleys.

Asian Istanbul has a more rugged topography. The steep Bosporus coastline continues southeasterly along the Marmara for about 20 km to flat coastal areas of varying widths. The five Prince's Islands lie about 7 km offshore.

The climate is mild--with average summer and winter air temperatures of 50^{o} C and 25^{o} C, respectively. Approximately 70 percent of the average rainfall (726 mm) occurs from October through March.

Istanbul lies in a second degree seismically active zone and the city has suffered extensive earthquake damage in the past. Two faults have been located within the city and a third, showing signs of recent activity, crosses the Bosporus 5 km south of the Black Sea.

Most of the population growth has occurred since about 1920, when it was around 500,000. It reached 1 million in 1940, 2.8 million in 1970, and 4.6 million in 1980 over an area of about 300 sq km. Rural immigration since 1960 has resulted in the tenured establishment of large squatter settlements (gecekondu, meaning overnight-built) that must meet certain legal criteria. The growth was accompanied by industrial expansion from 1,100 ha in 1970 to about 2,200 ha in 1980. The population is expected to reach 6.3 million by 1990 and 7.8 million by 2000 (47).

Since Roman times most of the water for the European side of Istanbul has come from a number of reservoirs northwest of the city. The present average annual yield of Istanbul's existing water supply is 375 Mm^3, evenly distributed between European and Asian sources. Of the rated treatment capacity of 905,000 m^3/d (330 Mm^3/y), 62 percent is on the Asian side. Twin, 1.9 km-long, 1,000 mm-diameter pipelines under the Bosporus connect the European and Asian systems. In 1980, 40 percent of the municipal water supply was non-revenue-producing. An additional 82,000 m^3/d was extracted by industry from private wells. It was estimated that 79 percent of the 4.6 million population was served by house connections, 15 percent by public fountains, and 6 percent from private supplies. Domestic water consumption averaged 82 lcd for the population served, an estimated 94 lcd for the population served by house connections, and 20 lcd for those using public fountains (47).

15.5.1 Sewerage and Environmental Sanitation

Sewers conform to the hilly topography and follow the natural drainage pattern, with trunk mains discharging along the shore lines. About 50 km of Byzantine or Ottoman masonry sewers known as "black channels" are in use. Combined sewers have been built in the older part of the city since 1920 and separate sanitary sewers in the newer urban areas since around 1950.

In 1971, the DAMOC (Daniel, Mann, Johnson, and Mendenhall, USA; Alvord, Burdick and Howson, USA; Motor-Columbus, Switzerland, and Checchi and Co., USA) water supply and sewerage master plan provided the first blueprint for a modern and well-integrated sewer system (17). Although sewer locations were poorly known, rigorous adherence by city and private constructors to regulations on antiquities provided a working basis for mapping the existing system. When a sewer excavation revealed an ancient artifact, the location was reported and subsequently cataloged by the German Archaelogical Institute. These files were made available and were the most reliable source of sewer location data (52). Implementation of the master plan included the construction in 1978 of a fully automated factory for concrete pipe to replace the previous system of hand-casting in place.

At present, Greater Istanbul has about 2,500 km (1,500 mi) of secondary sewers, 200 km (120 mi) of sewer mains and collectors, and 80 km (50 mi) of storm sewers. The 1980 connected population was estimated at 2.1 million (46 percent of the total). The rest depend on septic or holding tanks that often overflow to the nearest watercourse.

Because much of the soil is unsuitable for septic tanks, the city requires holding tanks to be constructed with a nominal one-month storage capacity; The tanks are emptied by vacuum trucks (vidanjors). The number of vidanjors is limited, the practical maximum emptying interval is about ten days because after that the nightsoil congeals and resists pumping, and holding tanks designed for 30 days frequently overflow (53).

The estimated 1980 average dry weather sewage flow was 333,000 m^3/d (67 mgd), of which about 44 percent was industrial. The vidanjor service handled an additional 3,000 m^3/d. Total BOD_5 and suspended solids (SS) loadings (estimated at 122 and 164 ton/d, respectively) indicated rather strong sewage with 336 mg/l BOD_5 and 492 mg/l SS, respectively.

The present system of dispersed discharges into the well-oxygenated waters of the surf zone produces small sleek and floatable fields except for (1) the anoxic upper reaches and heavily degraded bottom layer of the lower Golden Horn, where 1980 BOD_5 loadings into the Golden Horn were estimated at 38 ton/d or 152 kg/ha/d, over twice recommended loadings of oxidation ponds; and (2) an area of the Sea of Marmara, near Yenikapl where heavily discolored and ill-smelling tannery wastes are discharged.

Coliform testing at most public beaches has revealed concentrations from 4,000 to 24,000 per 100 ml and, in the Golden Horn, 10^7 to 10^8/100 ml near the sewage outfalls and 10^4-10^6/100 ml in the midchannel. Considerable amounts of industrial toxicants are discharged into the coastal waters. Testing of ocean bottom sediments has shown high concentrations of lead and other heavy metals in the upper Golden Horn and chromium in the Marmara around the outlets of tanneries.

15.5.2 Sewerage Master Plan

A succession of agencies and consultants have been involved in the sewerage master plan (47). In 1967 and 1968 DAMOC together with the State Hydraulic Works (Devlet Su Isleri) and Istanbul Municipality with UNDP/WHO support prepared a basic plan consisting of a series of independent sewer systems discharging degritted sewage to the lower layer of the Bosporus or Sea of the Marmara through deep ocean outfalls. A supplemental plan for storm water was prepared in 1973 by Scandiaconsult (Sweden). During 1972-74, Camp-Tek-Ser (Camp, Dresser & McKee, Ltd., U.S.A., and Tek-Ser Teknik Servis A.S., Turkey) carried out additional oceanographic and wastewater surveys and a full review of the Istanbul portion of the DAMOC master plan for the Provincial Bank (Illerbankasi) (12, 47). In 1979-81, Nedeco (Netherlands) reviewed and updated the sewer system plans for priority construction (75) for the newly established Istanbul Water Supply and Sewerage General Directorate (ISKI). Both reviews supported DAMOC's conceptual approach and analysis. Both recommended additional costs for providing greater marginal factors of safety in eliminating Marmara coastal water pollution. A summary of alternative sewerage schemes is presented in Table 15-8. The current (1983) revision to the sewerage master plan contemplates the construction of twelve separate sewer systems, eight discharging into the Bosporus, and four into the Marmara (47).

15.5.3 Regional Geography and Oceanography

The unique geographical and oceanographic situation of Istanbul establishes the environmental design of its sewerage system (Figure 15-21). The Turkish Straits extend about 300 km (187 miles) from the Aegean Sea through the Dardanelles (60 km, 37 miles), Sea of Marmara (210 km, 131 miles), and Bosporus (31 km, 19 miles) to the Black Sea. There is a two-layer current system. Mediterranean water increases in density from the excess of evaporation over precipitation during its circulation from the Strait of Gibraltar along the African and Levantine coasts to the Aegean Sea. From the Aegean Sea, this heavy, highly saline water flows northerly through the Turkish Straits to the anaerobic lower portion of the Black Sea. Less dense, brackish surface waters carry runoff from tributaries to the Black Sea (9). Of these, the most notable are the Danube and Dneiper Rivers, which drain areas of 840,000 km^2 and 502,000 km^2 (325,000 square miles and 194,000 square miles) with average flows of 6,200 m^3/s and 1,700 m^3/s (221,000 cu ft/sec and 61,000 cu ft/sec), respectively (35, 41, 42). The outflow from the Black Sea also includes any excess of precipitation over evaporation from the 423,000 km^2 (166,000 sq miles) of the sea itself. The salinity of the outflow is due to vertical mixing within the Black Sea of salts from the Bosporus undercurrent inflow.

Fig. 15-21. Regional Geography of Eastern Mediterranean, Aegean Sea, Turkish Straits, and Black Sea. Generalized surface circulation is shown by arrows. Source: Gunnerson (35).

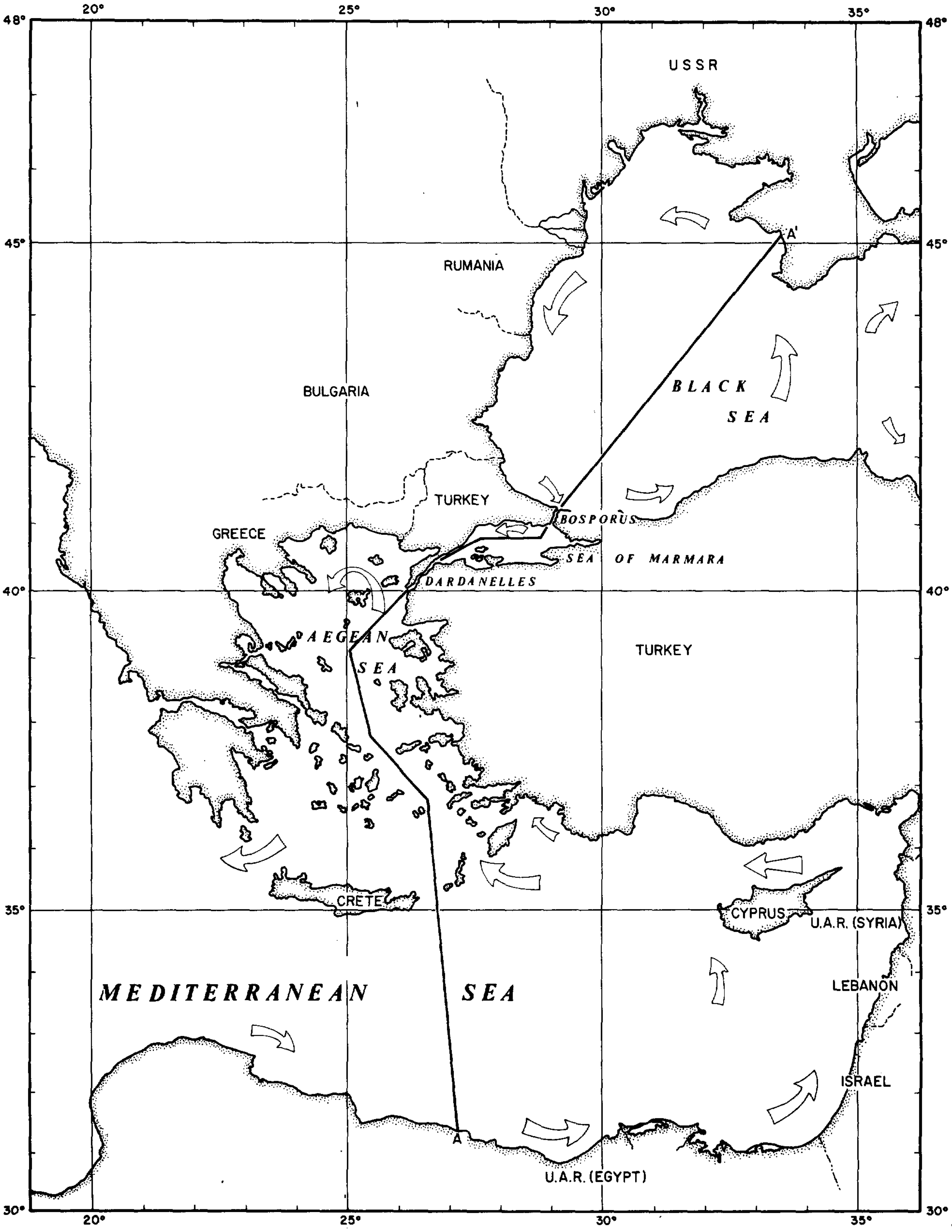


TABLE 15-8

Alternative Bosporus - Sea of Marmara Outfall Systems

			Preliminary Design Data				Alternative Outfall System (a)					
Outfall or alternative	Depth to Lower Layer m	Distance to 10 m Depth in Lower Layer m	Outfall Length m (b)	Outfall Diameter cm (c)	Diffuser Length m (d)	Maximum Depth m	I	II	III	IV	V	VI
1. Buyukcekimece	15	1250	1225	25	75	22		•				
2. Kumburgaz	15	1250	1225	35	75	25		•				
3. Kucukcekmece	15	400	375	70/70	75	27	•	•		•	•	
4. Florya	15	900	-	-	-	-	•		•			
5. Atakoy	15	2800	2750	100/120	150	27	•	•	•	•	•	
6. Yesilkoy	15	1250	-	-	-	-	•					
7. Zeytin Burnu	15	600	-	-	-	-	•		•			
8. Kumkapai	15	1350	-	-	-	-	•					
9. Ahirkapi/Yenikapi (e)	15	1200	1000	220/220	600	27	•	•	•	•	•	•
10. Fatih Tunnel (f)			2510	300	-	-		•		•	•	•
11. Golden Horn Siphon (g)			430	140/160	-	-		•		•		
12. Saray Burnu	20	80	320	-	-	-	•		•			
13. Kabatas	23	350	750	-	-	-	•		•		•	
14. Arnavutkoy	25	100	-	-	-	-	•		•		•	
15. Rumeli Hisar	40	230	-	-	-	-	•					
16. Baltalimani	45	280	400	70/70	100	63	•	•	•	•	•	
17. Tarabya	50	120	600	60	100	56	•	•	•	•	•	
18. Sariyer	50	~200	650	-	-	-	•					
19. Beykoz/Selvi Burnu	50	800	1000	-	-	-	•					
20. Cubuklu	50	~100	600	-	-	-	•					
21. Pasabahce	50	200	1020	60/60	30	75	•	•	•	•	•	
22. Kucuksu	50	230	280	70/70	70	82	•	•	•	•	•	
23. Beylerbeyi	25	100	-	-	-	-	•					
24. Uskudar	23	240	300	90	90	48	•	•	•	•		
25. Buyukada	15	200	760	60	160	42	•	•	•	•	•	
26. Heybeliada	15	780	780	45	40	25	•	•	•	•	•	
27. Burgazada	15	200	780	40	50	33	•	•	•	•	•	
28. Kinaliada	15	500	460	35	45	25	•	•	•	•	•	
29. Kadikoy	15	1740	1740	120/100	150	27	•	•	•	•	•	
30. Fenerbahce	15	1450	-	-	-	-	•					
31. Caddebostan	15	2200	-	-	-	-	•					
32. Bostanci	15	1400	-	-	-	-	•					
33. Kartal	15	300	500	80/90	150	35	•	•	•			
34. Pendik	15	375	-	-	-	-	•					
35. Tuzla	15	900	880	50	30	25	•	•	•	•	•	
36. Darica	20/40(h)	250	430	30	20	65	•	•				
37. Gebze	20/40(h)	480	480	35	20	50	•	•				
38. Hereke	20/40(h)	(i)	380	25	20	35	•	•				
39. Yarimca	20/40(h)	(i)	750	60	50	27	•	•				
40. Izmit	20/40(h)	(i)	1750	90/110	25	35	•	•				
41. Golcuk	20/40(h)	(i)	380	40	20	30	•	•				

TABLE 15-8 (cont.)

TABLE 15-8 (cont.)

(a) Principal differences in selection criteria for alternative systems include:

I. Maximum number of discharge points for dispersion into the environment, minimum outfall, lengths, and diameters for local fabrication and construction; nominal 1150:1 dilution in Marmara upper layer; minimum lengths and sizes of interceptors; maximum discharge direct to lower layer of Bosporus for rapid transport to lower anaerobic layer of Black Sea; minimum investment in interceptors prior to assessment of wastewater recovery and utilization schemes (34).

II. Same as I, except for applying more advanced construction technologies; minimum direct discharges to Bosporus; larger investments in interceptors (17).

III. Same as I, except simplify outfall construction to eliminate diffusers except where operating experience shows less than the 10:1 initial dilution adequate to retain essentially all of the discharges, within the lower layer; environmental optimum (35).

IV. Same as II, except moving discharges from Asian shore of Sea of Marmara to southerly entrance to Bosporus, where Coriolis effect will promote flow into lower layer of Bosporus (Figure 15-22); increased factors of safety (12).

V. Same as IV, except for increasing discharges from European side into lower layer of Bosporus (12, 75).

VI. First phase construction, including interceptor (not shown) for European Istanbul south of Golden Horn and last of Bakirkoy, pumping plants, and pretreatment works to serve 1987 population of 3.5 million (61 percent of Greater Istanbul Municipality; increased factors of safety for discharges to Marmara coastal waters; marginal cost:benefit ratio to be determined (12, 75).

(b) Not including diffusers.

(c) Two figures indicate diameters of initial and second-phase construction to meet 2020 requirements.

(d) Total lengths of Y-diffusers in Marmara and I-diffusers in Bosporus.

(e) Ahirkapi/Yenikapi system for European Istanbul south of Golden Horn.

(f) Includes interceptors along south side of Golden Horn are along Marmara coastline to Bakirkoy.

(g) Includes interceptors along north sides of Golden Horn on west side of Bosporus to Atakoy.

The large-scale features of this two-layer circulation are shown in Figure 15-22. Bottom waters move north through the Bosporus into the anaerobic lower portion of the Black Sea. This lower portion has been anaerobic for some 7,000 years and at present occupies 88 percent of the 543,000-km^3 (128,000-cu miles) volume of the sea (18, 20).

15.5.4 Morphology and Oceanography of the Sea of Marmara, the Bosporus, and the Golden Horn

Emphasis during the DAMOC study was on the northerly Sea of Marmara, the southerly two-thirds of the Bosporus, and the Golden Horn where most of the population and pollution are. The morphology of the area is shown in Figures 15-22, 23, and 24.

The Bosporus is a meandering strait about 31 km (20 statute miles) in length. It varies in widths from 0.7 km to 3.5 km (0.4 miles to 2.2 miles) and averages about 1.6 km (1 mile). Average and maximum depths are 35.8 m and 110 m (118 ft and 360 ft), respectively. A sill between 32 m and 34 m (105 ft and 112 ft) depth about 3 km (2 miles) from the southerly entrance strongly affects the two-layer current system.

Dry northerly to northeasterly winds prevail approximately 50 percent of the time in winter and 80 percent in summer. Southerly to southwesterly winds occur about 2 percent of the time in August and bring warm, humid weather to the area. During winter they occur approximately one-third of the time and, when they are sufficiently strong, bring storm waves and increased sea levels to the Marmara coastline, the southerly entrance to the Bosporus, and the Golden Horn.

Approximately 70 percent of the 726-mm (29-in.) average annual precipitation falls during October through March. Long-term mean air temperatures vary from about 5° C (41° F) in winter to 25° C (77° F) in summer. From July 1966 to December 1967, mean air temperatures varied from about 26° C ± 5° C down to 3° ± 1° C (79° ± 9° F down to 37° ± 2° F). Mean water temperatures rose from 4° C (39° F) in winter to 25° C (77° F) in summer and followed mean air temperatures by about 1 month (Figure 15-25). Occasionally, low winter temperatures cause drift ice, most recently in 1954 when the Bosporus was so blocked with ice that one could walk across it (65). Chihatchef (15) reported that ice formation was even more severe in earlier times. The most extensive freezing recorded was that of A.D. 732 when the Black Sea froze solid to within 90 km (55 miles) of the Bosporus and drift ice reached the Dardanelles.

Southerly flows of water from the Black Sea are related to surface slopes between the north and south ends of the Bosporus. Sea level data obtained at Üsküdar, Çubuklu, and Kavak, located 7 km, 15 km, and 24 km (4.4 miles, 9.3 miles, and 15 miles), respectively, from the southerly entrance to the Bosporus showed effects of tides, winds, and seasonal changes in outflow from the Black Sea (Figure 15-25). In the Bosporus, the semidiurnal tide has a range of about 2-10 cm (1-4 in.) with a large diurnal inequality. The lunar

Fig. 15-22. Vertical Salinity, in Parts per Thousand. Section from Mediterranean to Black Sea, showing two-layer circulation in Turkish Straits. Location of Section A-A' is shown in Figure 15-21.

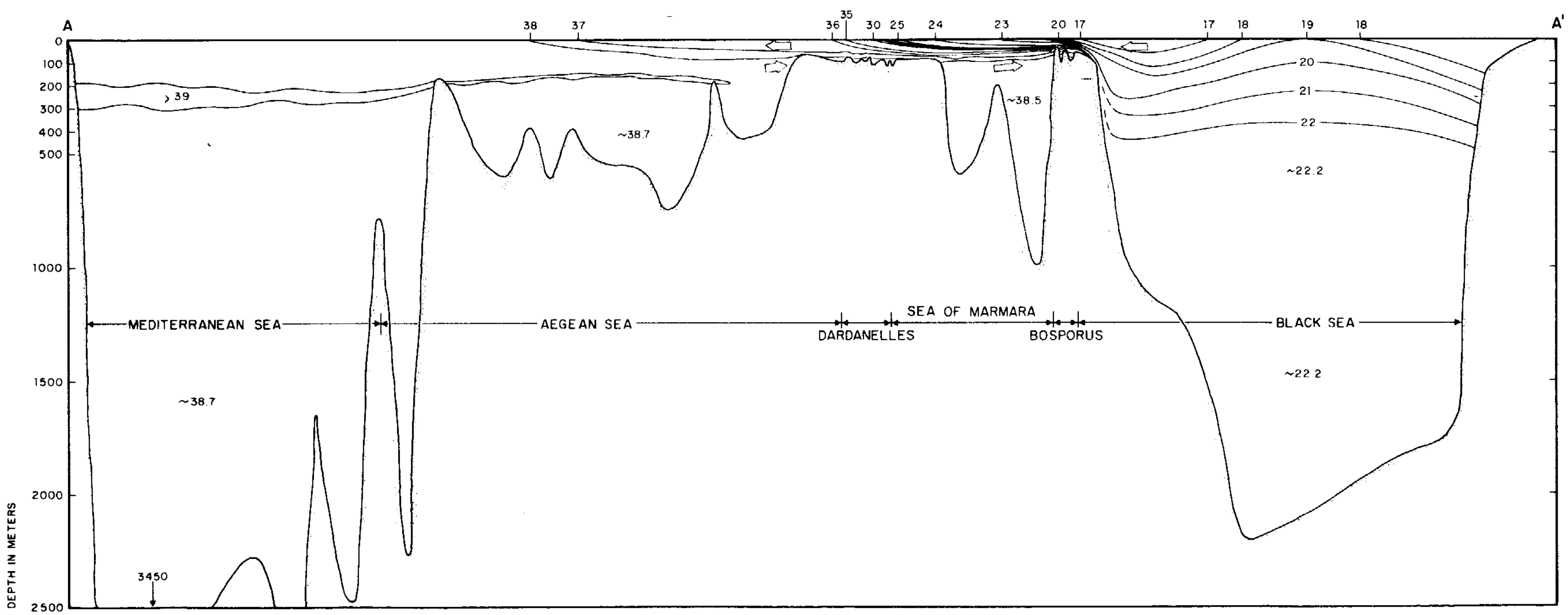
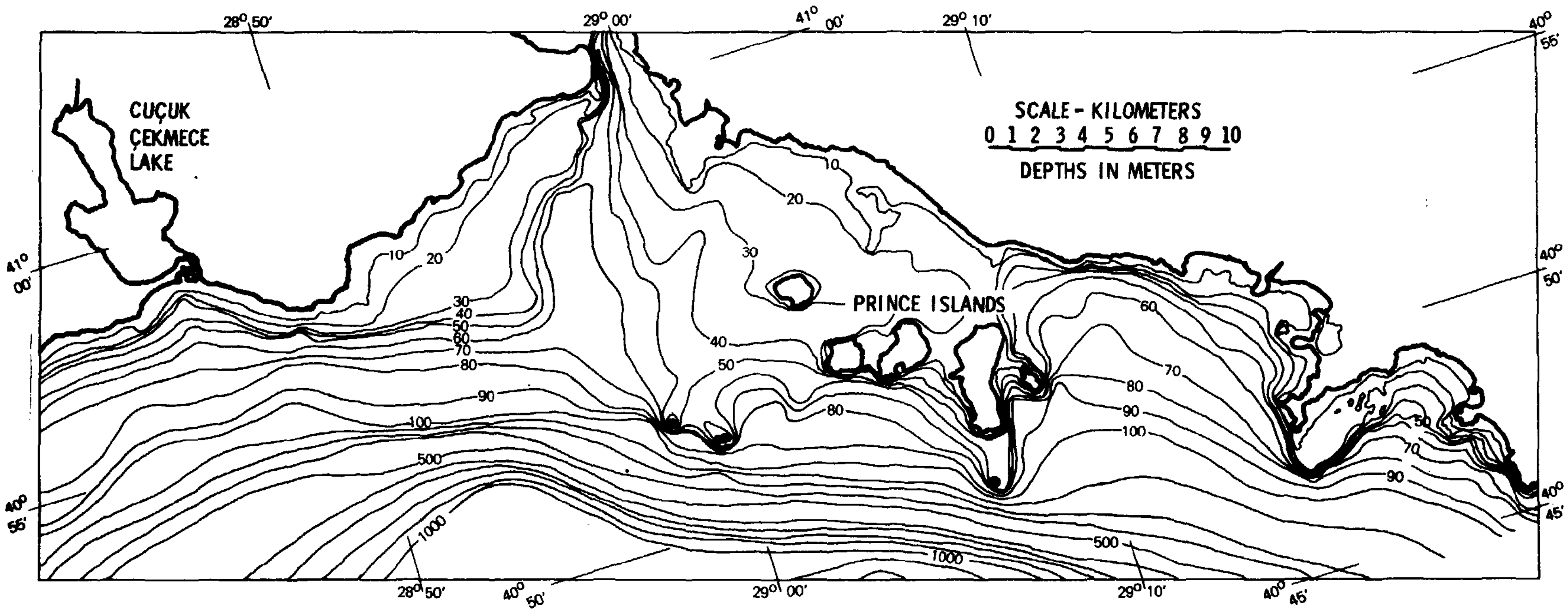
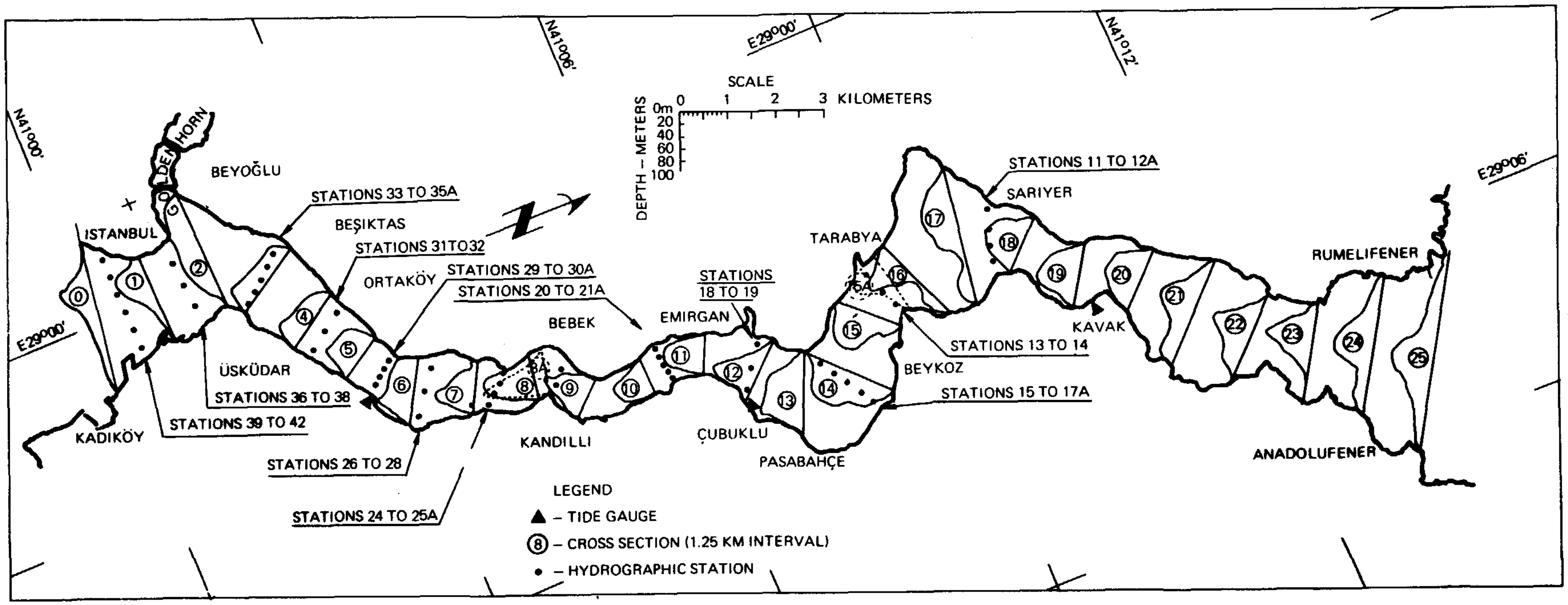


Fig. 15-23. Southerly Approaches to the Bosporus



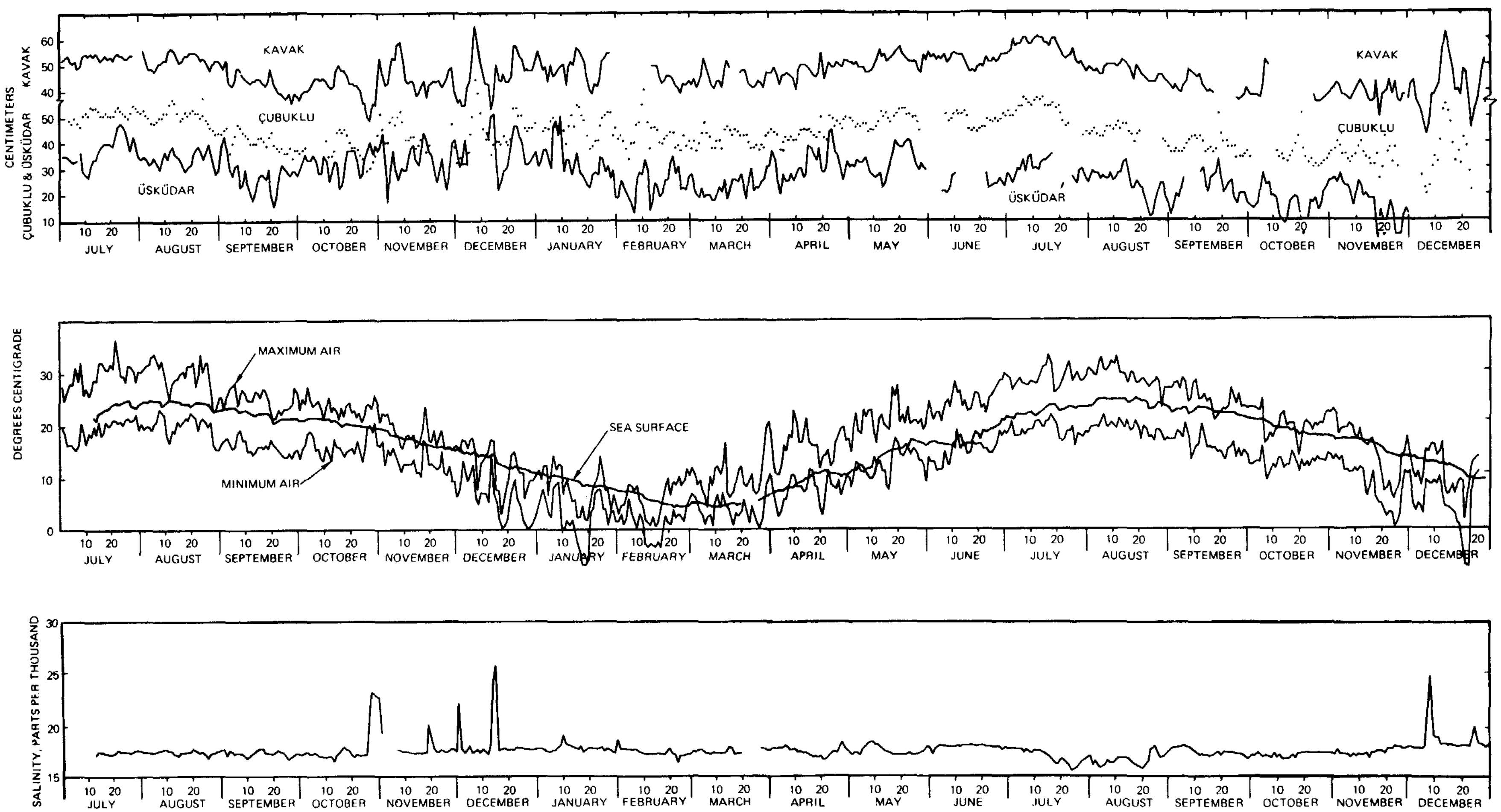
Source: Gunnerson and Ozturgut (41).

Fig. 15-24. Bosporus Morphology and Sampling Stations



Source: Gunnerson et al. (40).

Fig. 15-25. Bosphorus Stage Temperature and Salinity, July 1966-December 1967



Source: Gunnerson et al. (40).

fortnightly tide has an estimated range of about 5-20 cm (2-8 in.). These tides are often obscured by wind setup or storm tides.

During July 1966 to December 1967, monthly average water surface elevations varied from about 40 cm (16 in.) in late autumn to 55 cm (22 in.) in July 1967 at Kavak, and from 38 cm to 52 cm (15 in. to 20 in.) for the same months at Çubuklu. These elevations and the surface salinities at Çubuklu correspond with seasonal stream runoff into the Black Sea. Short-term variations at the three locations are due mostly to winds. Bosporus surface currents increase from close to 100 cm/s (2 knots) near the Black Sea to 250-350 cm/s (5 to 7 knots) near Istanbul.

The average salinity of Black Sea waters at Çubuklu from July 1966 to December 1967 was approximately 17.5 parts per thousand (^{o}/oo). The 16 to 17 0/00 values between July 19, 1967 and August 23, 1967 (Figure 15-25) reflect peak runoff from Black Sea tributaries. During the same period in 1966, 17.0 to 17.5 ^{o}/oo salinities indicate lower seasonal precipitation and runoff. Higher average salinities of 17.5 to 19.0 ^{o}/oo were found throughout the rest of the year, with occasional values up to 25 parts per thousand during winter months. Surface salinity increases by an average 2 ^{o}/oo in the Bosporus, mostly in the southerly 10 km (6.3 miles), where currents and mixing are greatest (35, 41).

Infrequent reversals of surface slopes and currents that cause high salinities at Çubuklu are short-lived. They follow strong, persistent southerly winds during winter when Black Sea outflows are low. Surface slope reversals are followed within one day by salinity increases proportional to wind stresses. Ordinarily, the wind changes after a day or so, and the sea surface slopes and salinity return to normal. Even if the wind continues, salinities at Çubuklu begin to decrease by the third day, and thus indicate a new equilibrium slope.

The wind-driven current reversals bring Sea of Marmara surface water into the Bosporus and may also increase vertical mixing due to wave action. In all cases, high-salinity Mediterranean water remains in the deepest parts of the Strait.

The Golden Horn is approximately 7 km (4.5 miles) long. Its maximum depths is 1 m (3 ft) at the upper end and 40 m (130 ft) at the mouth; its hydrography has been summarized by Kor (55). Planned development of local water resources will reduce tidal flushing of the upper reach of the Golden Horn by about half, and will increase the need for removal of wastewaters from the estuary.

15.5.5 Two-Layer Current System in Bosporus

Bosporus currents, salinity, and morphometry are intimately related. Möller's (66) estimates of 6,100 m^3/s (215,000 cu ft/sec) in the lower layer and 12,600 m^3/s (440,000 cu ft/sec) in the upper layer have been accepted as a first approximation, although data published by the United States Navy Oceanographic Office (99) indicate flows of 3,000 m^3/s to 30,000

m^3/s (100,000 cu ft/sec to 1,000,000 cu ft/sec) in each layer. Figure 15-26 shows the slopes of the boundary layer in 1917-18 according to Möller (66), the Baltalimani Hydrobiological Institute of the University of Istanbul series of 1959-60, and the 1961 Turkish Navy study (97). Figure 15-26 indicates that, during winter months, the cross-sectional area of the upper layer is reduced by about half, so that average net velocities are increased as the current flows through the southerly 24 km (15 miles) of the Bosporus. In the lower layer, cross-sectional areas are reduced between Istanbul and Tarabya at km 20 from 15-23 thousands of square meters at Istanbul to 3-4 thousands of square meters at Tarabya, 20 km to the north. This implies a five- to sixfold increase in average velocity. While Möller (66) reported currents in the lower layer of 1.0 m/s to 1.5 m/s (2-3 knots), Carruthers (13) found bottom velocities of 4 cm/s (0.09 knots) at the southerly entrance to the Bosporus, which increased to as much as 75 cm/s (1.5 knots) 1 km or more to the north. Effective cross-sectional areas for both upper and lower layers are further reduced, particularly in embayments, by stationary vortices. Here, near-shore countercurrents up to 25 cm/s (1/2 knots) are followed to advantage by ferry and other small boat traffic.

15.5.6 Persistence of Coliform Bacteria in Seawater

Times for 90 percent reduction (T-90) of coliforms in marine receiving waters have been routinely included in outfall design criteria for treatment and disposal ever since the 1955 in situ studies for the Los Angeles Hyperion Treatment Plant showed that coliforms disappeared from surface waters by a combination of dilution, sedimentation, and mortality (11, 33). Dilution is easily measured and can be predicted by standard diffusion equations or, in the Bosporus, by continuity considerations. The combined effects of sedimentation and mortality have been measured with the results listed in Table 15-9.

The values of T_{90} for the Bosporus and Sea of Marmara averaged 1.1 hr, whereas those from the Golden Horn were 2 to 3 times this value, presumably because of older sewage in the diluting water. Table 15-9 is based on sampling sewage fields up to 3-1/2 hr. Confirming surveys at the Dolmabahçe outfall on April 10, 1968, and April 17, 1968, showed a T-90 of 1.2 hr over 7 hr. This agrees with the earlier value. No effect of temperature--which varied from about 22° C (72° F) in September to about 10°C (50°F) in April--was observed.

15.5.7 Environmental Design Criteria

15.5.7.1 Depth of Discharge

The basic decision in designing sewage treatment and disposal into the Turkish Straits is whether to: (1) discharge to the surface layer of brackish water flowing to the Marmara and Aegean seas; (2) discharge to the lower, highly saline layer flowing to the Black Sea; or (3) discharge to both layers. Rational designs can be developed for the first two alternatives but not for the third (35).

Fig. 15-26. Boundary layers.

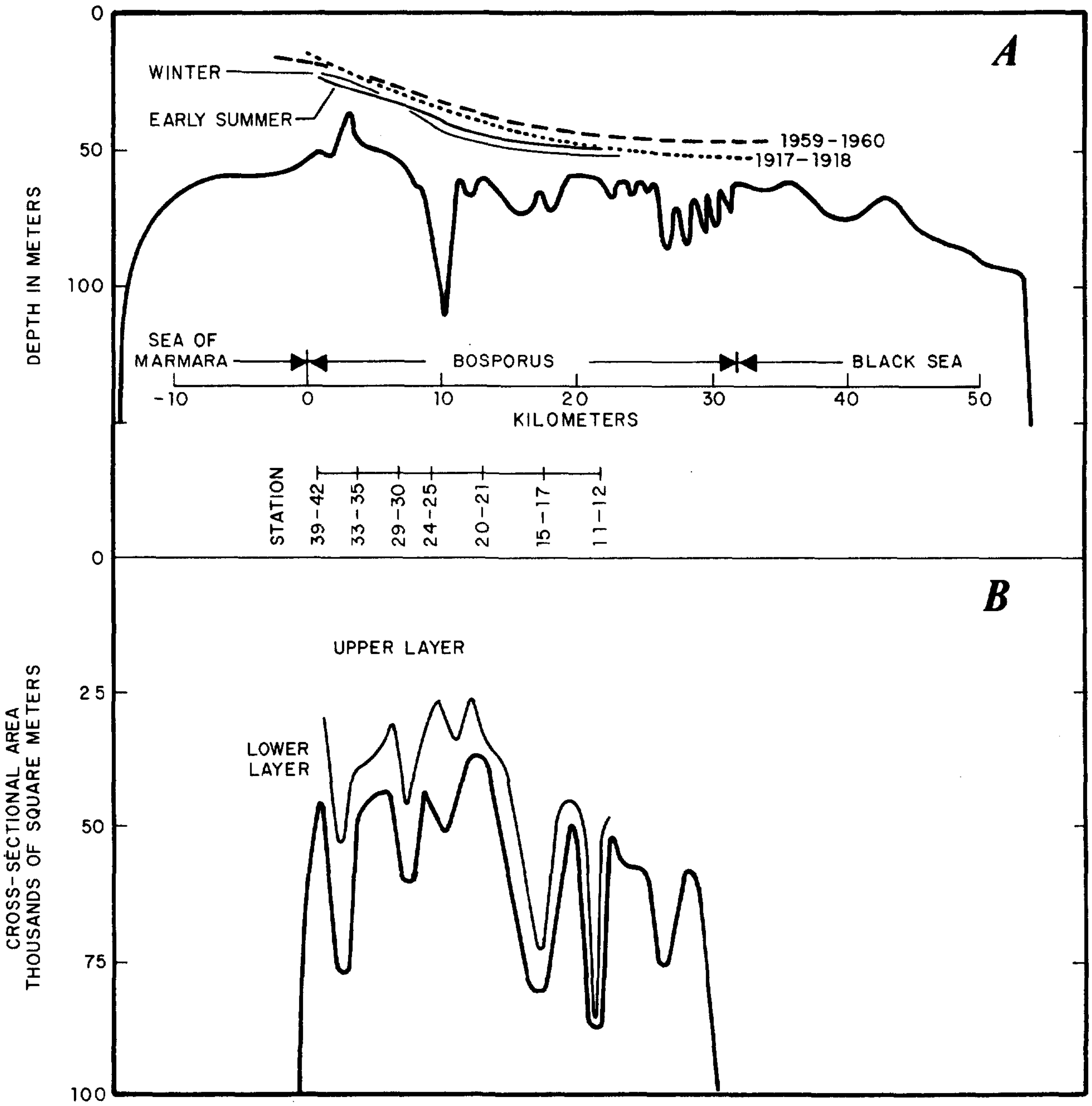


TABLE 15-9

Coliform Decay Rates For Istanbul Raw Sewage

Outfall	Date	T_{90}, in hours
Dolmabahçe-Bosporus	October 23, 1967	0.8
Kumkapi-Sea of Marmara	October 28, 1967	0.9
Kumkapi-Sea of Marmara	October 19, 1967	0.9
Kumkapi-Sea of Marmara	October 16, 1967	1.0
Dolmabahçe-Bosporus	November 29, 1967	1.1
Kadiköy-Bosporus	November 20, 1967	1.2
Kumkapi-Sea of Marmara	September 28, 1967	1.2
Kadiköy-Bosporus	October 9, 1967	1.3
Caddebostan-Sea of Marmara	October 5, 1967	1.4
Caddebostan-Sea of Marmara	November 9, 1967	1.7
Kasimpaşa-Golden Horn	October 25, 1967	2.0
Kasimpaşa-Golden Horn	November 6, 1967	2.5
Kasimpaşa-Golden Horn	December 4, 1967	3.0

There is a stable boundary layer between the lower layer of approximately 15.5° C (59.9° F), 38.5 0/00 salinity, and 1.028 density, and the upper layer with seasonal variations in temperature and salinity and in densities ρ of 1.011 to 1.014. The boundary layer generally lies below 10 m (33 ft) and is 10-m to 15 m (33-49 ft) thick near the southerly entrance to the Bosporus. Twenty kilometers or more (12 miles or more) to the north in the Bosporus, the boundary layer is 10-25 m (33-82 ft) thick and lies below depths of 35-50 m (115-164 ft).

Discharges to the upper layer would necessarily be located near the shore and would require a high degree of treatment followed by large diffusers providing initial dilutions of, say, 150:1, or more. Since the upper layer is homogeneous, the diluted sewage would rise to the surface.

In contrast, sewage ($\rho \approx 0.998$) discharged to the lower layer ($\rho = 1.028$) with an initial dilution of only 2 parts seawater to 1 of sewage will have a density of 1.0167, and will stratify within the boundary layer. Theoretical studies (Chapter 4) and field observations of point discharges through pipes with diameters on the order of 1 m (3 ft) into well-mixed California surface waters with depths of 10 m (33 ft) or less typically have shown average initial dilutions of 20:1 or more (11). This is an order of magnitude greater than the 2:1 mixture assumed for the example. A 20:1 mixture would have a density of 1.022. For design purposes, discharge into 10 m or more (33 ft or more) of boundary and lower-layer (Mediterranean) water without a diffuser section would provide for spreading of the sewage:seawater mixture within or below the boundary layer. Discharge through diffuser sections at greater depths would provide even greater factors of safety.

Sewage discharged into the lower layer of the Bosporus would mix rapidly with the lower-layer flow and reach the Black Sea within 18 hours. Its impact upon the Bosporus lower layer has been estimated in terms of BOD_1 loadings (12). Total BOD_5 loading for the eight systems discharging into the Bosporus is expected to be 527.6 ton/d by the year 2020. This corresponds to a BOD_1 loading of 173 ton/d, which, for the average flow of 6,000 m^3/s, would depress oxygen concentrations in the lower layer by only 0.33 mg/l, without measurable impact upon fisheries. Meanwhile, bottom sediment size distributions reveal that currents are sufficient to prevent deposition of sewage solids in the Bosporus and any consequent impact on fisheries (17).

Mediterranean water of the lower layer and the heavier portion of the boundary layer containing a 20:1 bottom water:sewage mixture will flow into the Black Sea. Further sinking and spreading of water from the Bosporus into the Black Sea has been studied by Bogdanova and his associates since 1961 (9) and detailed by Tolmazin (96).

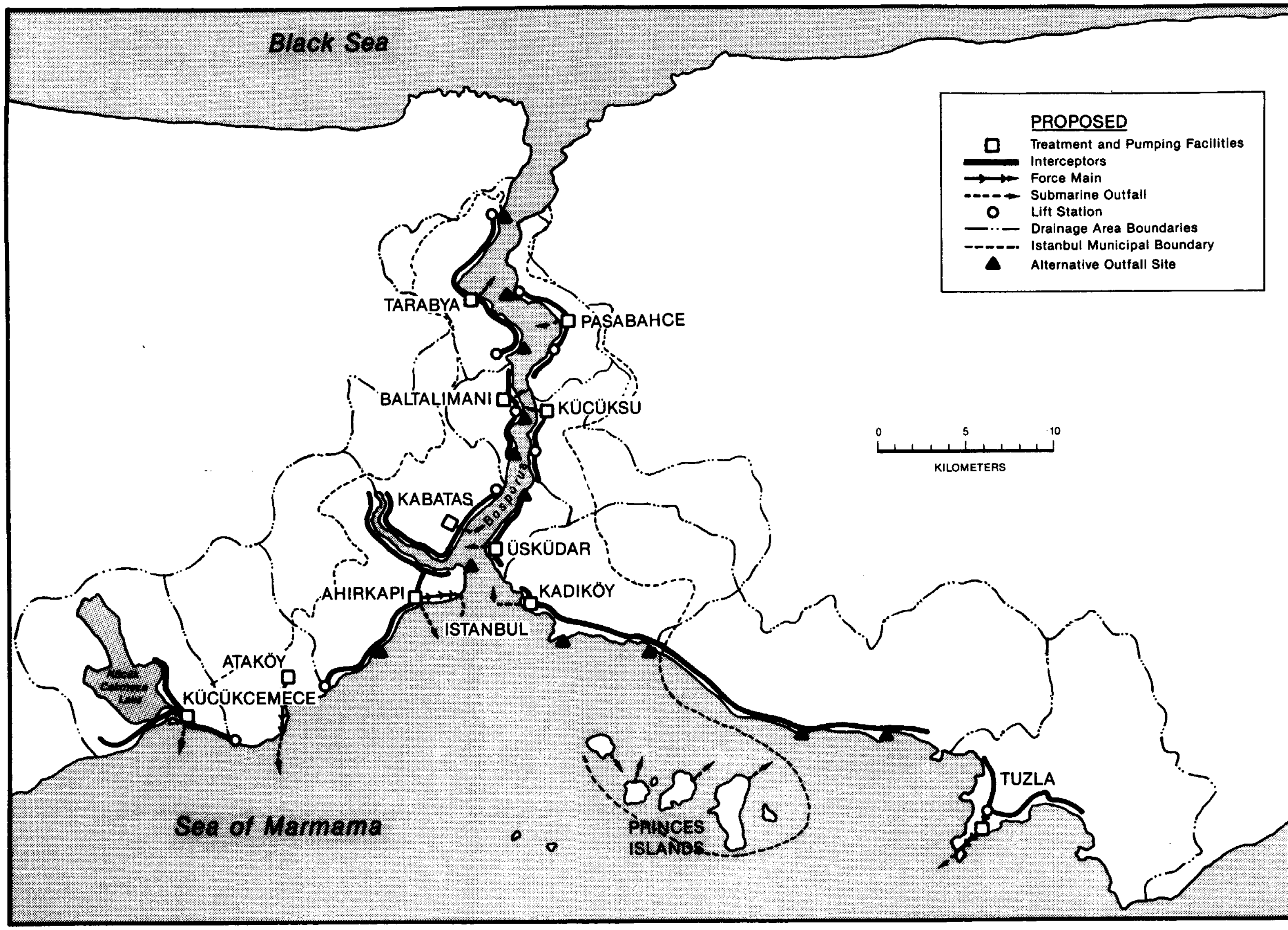
15.5.7.2 Outfall and Treatment Requirements

The minimum 3,000 m^3/s (106,000 cu ft/sec) flow in the lower layer would provide dilution water at 20:1 for 52,000,000 people at 250 liters per capita per day (lcd) (65 gallons/capita/day). For a population of 6,000,000, 3,000 m^3/s (106,000 cu ft/sec) would provide a 170:1 dilution and the average 6,100 m^3/s (215,000 cu ft/sec) would provide 350:1.

Locations and numbers of outfalls discharging to the lower layer in either the Bosporus or Sea of Marmara are determined by topography and the availability of space for outfall and headworks construction; an example of outfall locations is shown in Figure 15-27. Flotation and grit removal systems are sufficient. Therefore, space for flotation equipment is required, but the actual installation can be deferred. Relative costs of interceptors, pumping plants, energy, land, rights of way, headworks and flotation facilities, and outfalls may favor the construction of longer interceptors and fewer outfalls. Marginal costs should be justified by marginal environmental benefits, particularly for discharges into the lower layer at or near the southerly entrance to the Bosporus, with due regard for potential future wastewater reclamation (22, 40).

Sewage within the boundary layer will diffuse upward at the rate at which up to 20 percent of the salinity in the lower layer is diffused upward. During the 9-hr average residence time in the Bosporus, dilution and T-90 will combine to reduce average concentrations of bacteria to approximately 1×10^{-10} of initial concentrations in the sewage. The assumption that sewage discharged to the lower layer will not cause significant effects in the upper layer is accordingly conservative. Similar assurance based on numerical modeling of vertical flux was reported by Camp-Tek-Ser (12).

Fig. 15-27. Alternative Location Schemes for Istanbul Outfalls



15.5.7.3 Environmental Impact of Sewage Disposal into Lower Layer of Bosporus

The composition and distribution of planktonic and benthic organisms in both the surface and lower layers of the Sea of Marmara, Bosporus, and Black Sea have been summarized by Sverdrup et al. (89), Caspers (14), and Zenkevitch (101). The gradual lowering of salinity in the surface layer from the Aegean to the Black Sea is accompanied by an impoverishment of both planktonic and benthic organisms. Nevertheless, an important fishery is found entirely within the upper layer of the Black Sea, Bosporus, and Sea of Marmara.

The lower layer of the Black Sea is anaerobic. Large populations and biomasses of anaerobic bacteria occur here in a climate of hydrogen sulphide. There is a stable boundary between the upper and lower layers at depths established by freshwater inflow and by the Coriolis effects of surface circulation patterns. The boundary layer marks the lower limit of plankton and benthos generally at a depth of less than 200 m (650 ft).

There is a continuous flux of salinity and of dissolved nutrients into the upper layer. Doubling times for phosphorus concentrations in the lower layer due to Istanbul waste discharges are estimated at 700 yr to 16,000 yr (35) and concentration increases in the upper layer are expected to occur very slowly. It is reasonable to assume that the agricultural value of sludge and the more immediate value of water for irrigation or industrial uses will result in their reclamation long before the phosphorus increases become significant.

Available data on heavy metals are also reassuring. Recent studies by Spencer and Brewer (88) indicate that copper and zinc are precipitated from the lower layer as insoluble sulfides. Nickel and cobalt tend to remain in solution, presumably because of their tendency to form soluble thio complexes. A somewhat similar fractionation also occurs in anaerobic digestion and is seen in the relative concentrations of copper, zinc, and nickel in City of Los Angeles effluents and sludges (29).

An overall negligible impact of Istanbul sewage discharged into the lower layer of the Bosporus is accordingly indicated. This reinforces the decision not to remove solids from Istanbul sewage, which would presumably then require conventional anaerobic digestion. These solids, which will remain in the sewage, will be stabilized in the largest anaerobic digester in the world--the Black Sea.

15.5.7.4 Wastewater Disposal in the Sea of Marmara

Southeast of Istanbul, increasing discharges of industrial and domestic wastes into the restricted circulation of Izmit Bay for over two decades has led to a serious degradation of its waters, which has led to concern that the assimilative capacity of the lower layer of Marmara is similarly limited. Accordingly, Camp-Tek-Ser (12) recommended substituting a costly 25.5 km long interceptor for the Kartal outfall (Table 15-8 and Figure 15-27), but one that would concur with DAMOC recommendations for the Atakoy and Kucukcekmece outfalls.

The Ahirkapi System described below and the proposed future Kadiköy system are based on coastal interceptor networks totaling about 40 km in length, discharging into two large outfall and diffuser systems. The corresponding scheme shown in Figure 15-27 would have provided for eight outfalls and about 27 km of interceptors.

15.5.8 Construction of the Ahirkapi Sewer System

In November 1981, the Government created the Istanbul Water Supply and Sewerage General Directorate (ISKI), to manage the planning, design, construction, and operation of all water supply and sewerage facilities in Greater Istanbul. The Sewer Department of Istanbul Municipality became the nucleus of ISKI's sewer division.

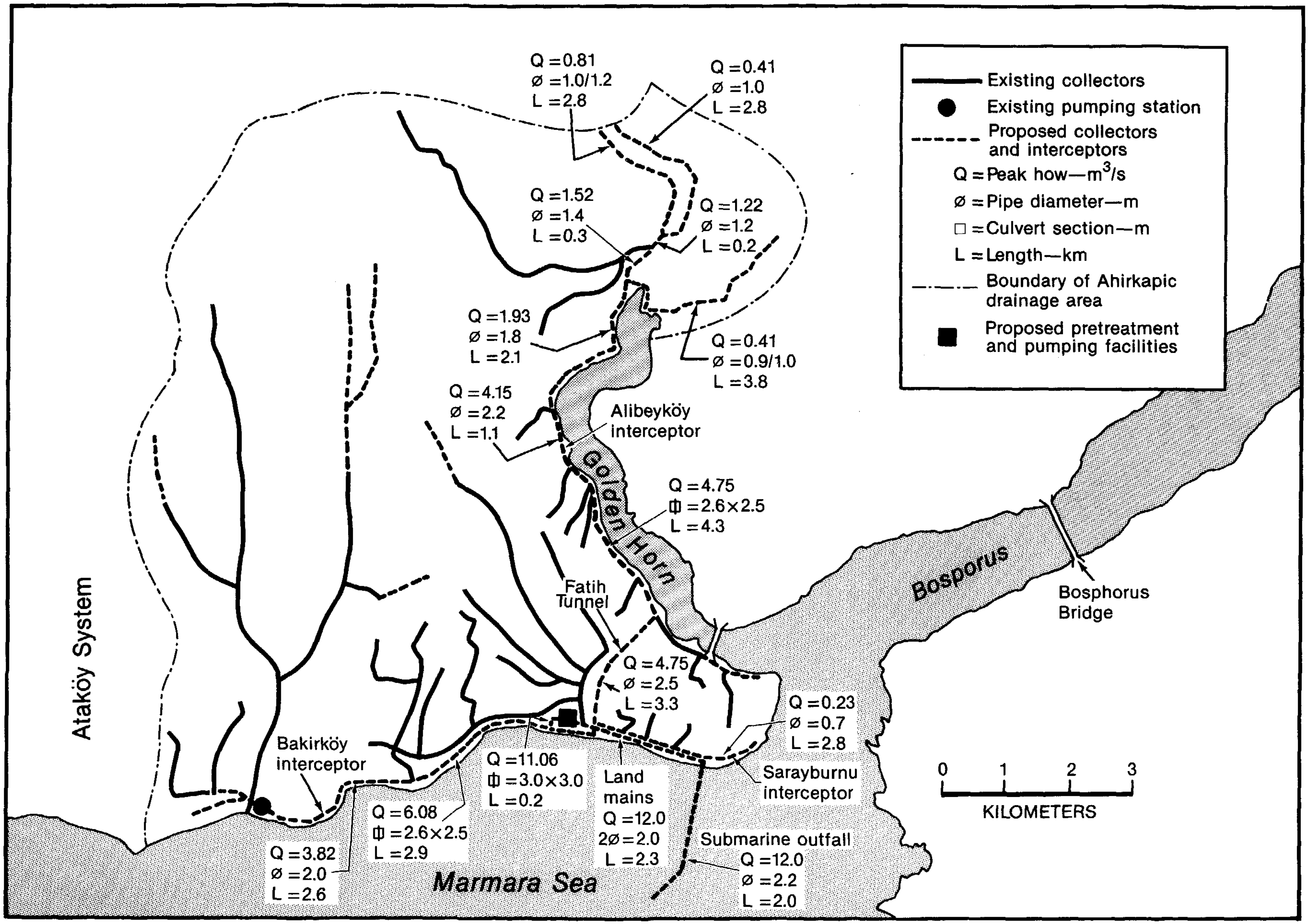
With World Bank assistance, ISKI is initiating construction of the Ahirkapi system to serve about 32 percent of Greater Istanbul population, which discharges about 50 percent of the waste and storm waters presently being discharged in the Golden Horn and which includes the most contaminated portion of the Sea of Marmara. The project also includes industrial wastewaters and oceanographic surveys; upgrading of the pipe factory equipment and manufacturing processes; new laboratory facilities, and a regional training center.

Figure 15-28 shows the location of the existing collectors and of the reconstruction proposed for the Ahirkapi system, as well as the peak design flows, diameters, and approximate lengths of the proposed interceptors and outfall. The Bakirkoy and Sarayburnu interceptors along the Marmara coast were designed for twice the average dry weather flow and the Alibeykoy interceptor along the southerly coast of the Golden Horn for four times the average dry weather flow. The pretreatment plant and outfall were designed for a maximum flow of 12 m^3/s, some 1.6 times the average daily flows. The pretreatment plant to be located at Yenikapi, about 100 m from the coast will include screening, influent pumping (capacity of 3 m^3/s at 7.2 m head), aerated grit chambers, a grease removal channel and effluent pumping of flows larger than about 5 m^3/s.

The proposed outfall will terminate near the entrance to the Bosporus and includes a land main composed of twin 2.0-m diameter, 2,260-m long pipelines; a 2.2-m diameter 1,400-m long submarine outfall, and a 600-m long diffuser section with diameters decreasing from 2.2 to 0.50 m. The diffuser section is designed to provide a 100:1 initial dilution and lies at 30 to 37 m depths, in a zone where the lower layer is homogeneous, is about 20 m thick, and moves steadily toward the Bosporus at velocities from 5 to 10 cm/s.

The consultants (75) recommended the outfall be constructed of 22-mm thick steel pipe and have a coal tar enamel corrosion coating, a 280 mm thick concrete weight coating, and a cathodic protection. The empty pipe will have

Fig. 15-28. Ahirkapi Sewer System, Existing and Proposed Structure.



a negative buoyancy of 196 kgs/m and when full of water, a submerged weight of 3.960 kgs/m.

The design wave was estimated from wave-fetch curves for winds of 25-30 m/s (Beaufort 10) and has a height of 10.5 m, a length of 189 m, and a period of 11 seconds.

To avoid possible damage by anchors, the outfall will be buried along its entire length with a minimum 2-m cover. The diffusers will consist of 3.5-m long, 0.30-m diameter risers provided with 0.20-m diameter bell mouth laterals for horizontal jet discharges. Spacing between the diffusers will decrease from 12 m in the initial portion to 9.5 m in the final section.

The ocean bottom along the outfall route is sandy, with rock outcrops near the shore, high gravel content to a depth of about 30 m, and silty sediments at greater depths. The consultants have recommended that the outfall be installed by the bottom-pull method.

Construction costs, including contingencies, for the main structures of the Ahirkapi system, estimated by Nadeco, are listed in Table 15-10.

TABLE 15-10

Estimated Ahirkapi System Costs

Structure	1981 US$ Million Equivalent
Extension of the Secondary Sewer System	36.2
Alibeykoy Interceptor	35.6
Bakirkoy and Sarayburnu Interceptors	7.1
Fatih Tunnel Interceptor	11.2
Pretreatment and Pumping Facilities	65.6
Outfall	44.4

Source: IBRD (47).

Estimated per capita investment costs will be about US$126. Istanbul (1983) service charges of US$0.17/$m^3$ for water and $0.05 for sewerage were expected to increase to a total of US$0.31/$m^3$ by the following year. The new tariff represents about 3.6 percent of their typical monthly income of US$77 equivalent for a family of four.

In view of the different populations benefiting from the collection and disposal facilities, their average incremental costs (AIC) were computed separately. For a 30-year term and a 10 percent discount rate, the AICs are about US$3.15 and US$2.90 per capita per year for collection and disposal facilities, respectively.

The high economic efficiency of the densely populated Ahirkapi system does not extend to the entire sewerage wastes plan. Approximate per capita investment costs (1990 populations) for the other sewer systems, as compared with that for Ahirkapi, are listed in Table 15-11.

TABLE 15-11

Outfall System Per Capita Costs

System	Comparative Per Capita Investment Cost
Ahirkapi	1.0
Kabatas and Uskudar	1.3
Remaining Bosporus Systems	2.0-2.4
Kucukcekmece	2.9
Tuzla	4.5
Prince's Islands	9.0

Source: IBRD (47).

15.6 Manila Bay

The importance of climate is revealed in the name, Manila, a contraction of "may nilad" or "here is a marsh plant" (82). Manila is surrounded by water (see Figure 15-29), lies in the humid tropics (Figures A3 and A5), and receives an average of 2,070 cm (81 in) of rainfall, mostly between June and October during the southwest monsoon (Figure 15-30) (Table 15-12). The maximum tidal range of 1.7 m occurs in January (77). Sanitation, sewerage, and storm drainage are major problems. Population densities average 193 persons per hectare (46), and are much higher in areas of the Tondo where some of the squatter housing extends over tidal flats.

Metro Manila includes four cities (Caloocan, Manila, Pasay, and Quezon), and twenty-three municipalities, and has an area of some 1,500 km^2 (one-third of which is urban). Its population in 1980 was 6.7 million. An estimated 2 million people live in 415 blighted areas throughout Manila where poor sanitation, overflowing sewers, and other hallmarks of poverty are found. Few of these people can afford sewerage. Many are willing to pay for household sanitation (defined here as separating people from their excreta) provided by household or community pit latrines, vault and cartage systems, or septic tanks (46, 53). Community or environmental sanitation consists of household sanitation and hygiene, solid waste management (including

Fig. 15-29. Manila Bay Drainage Area (47).

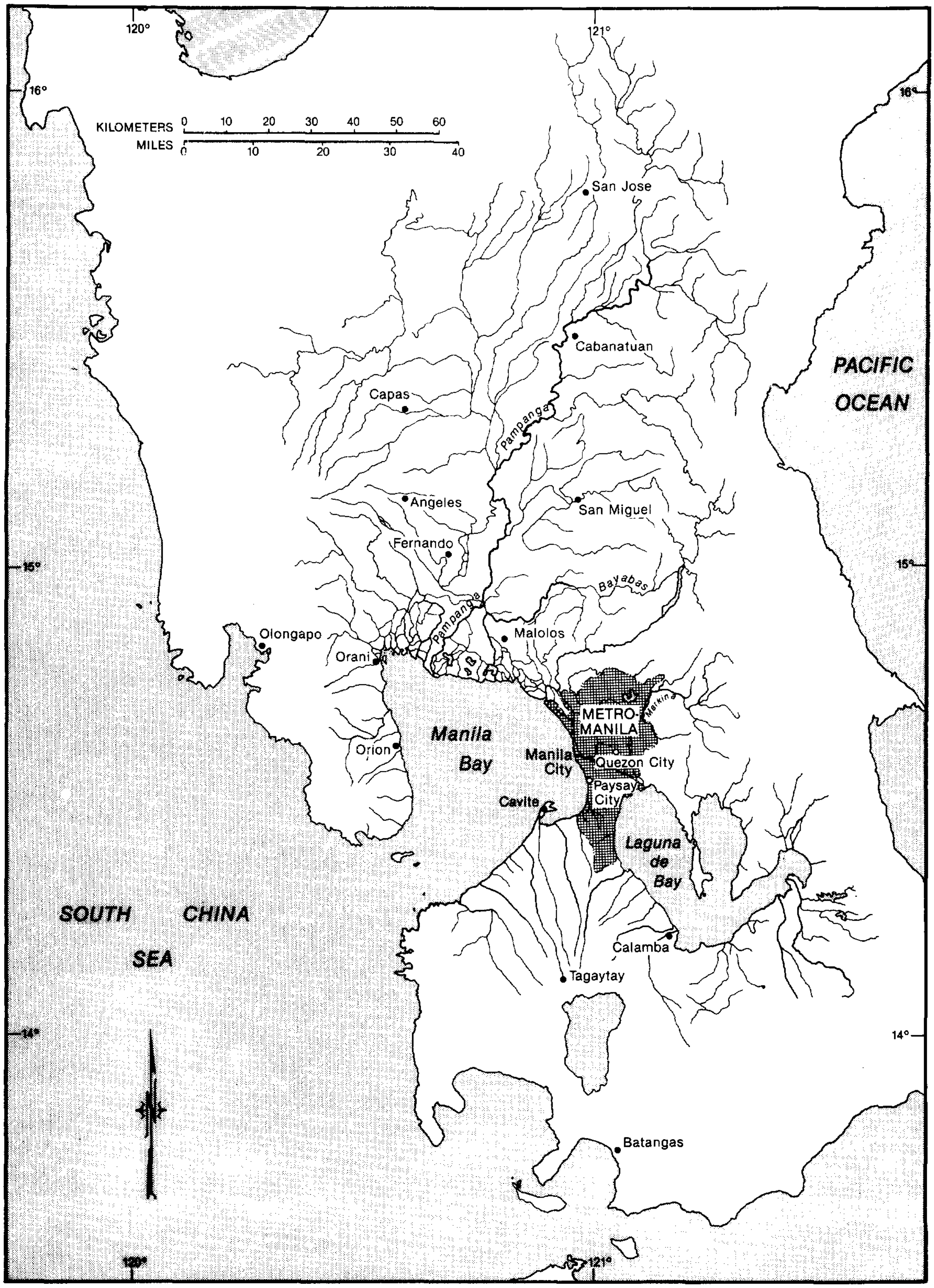


Fig. 15-30. Metropolitan Manila Sewer and Sanitation Project (47).

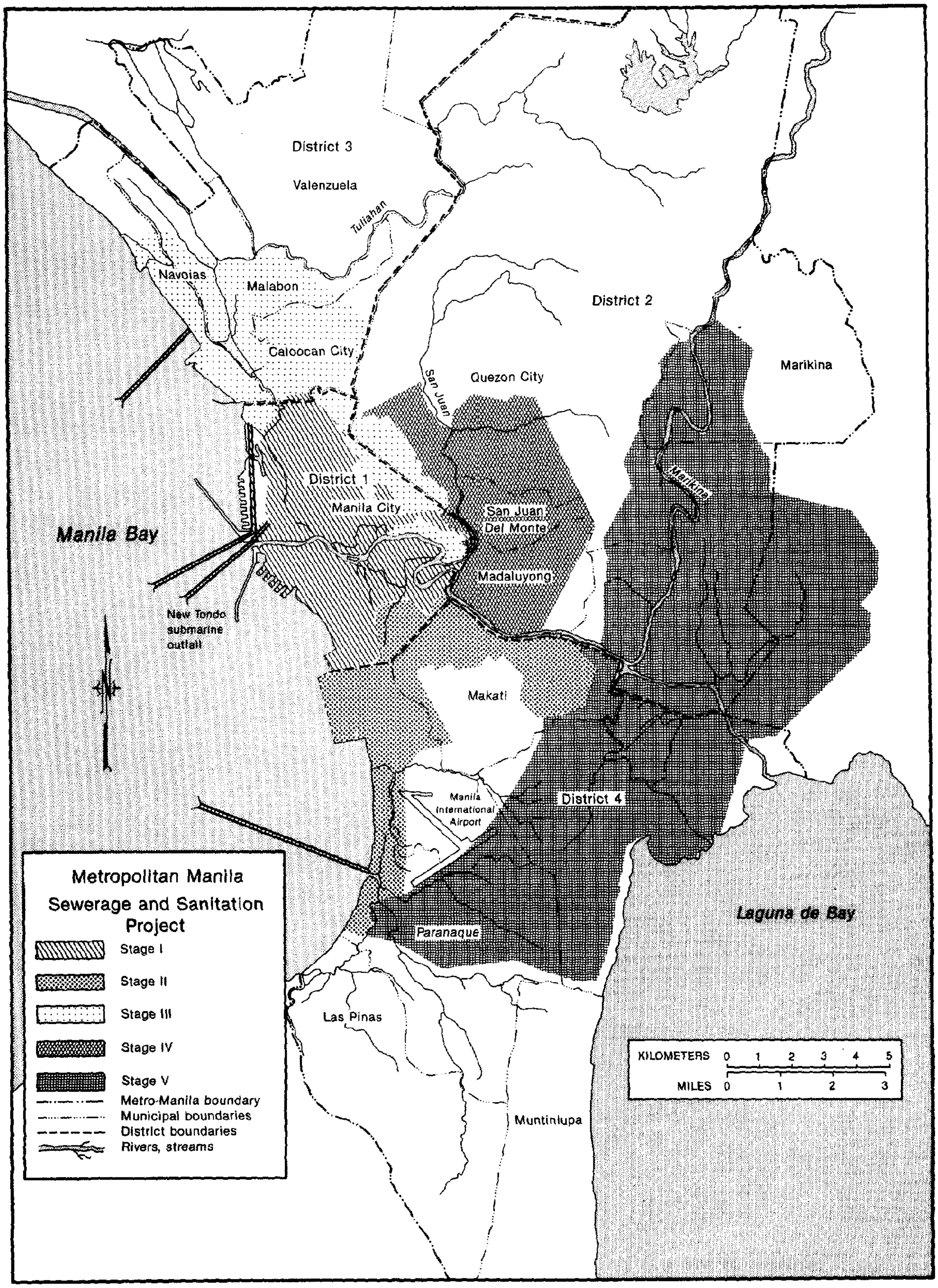
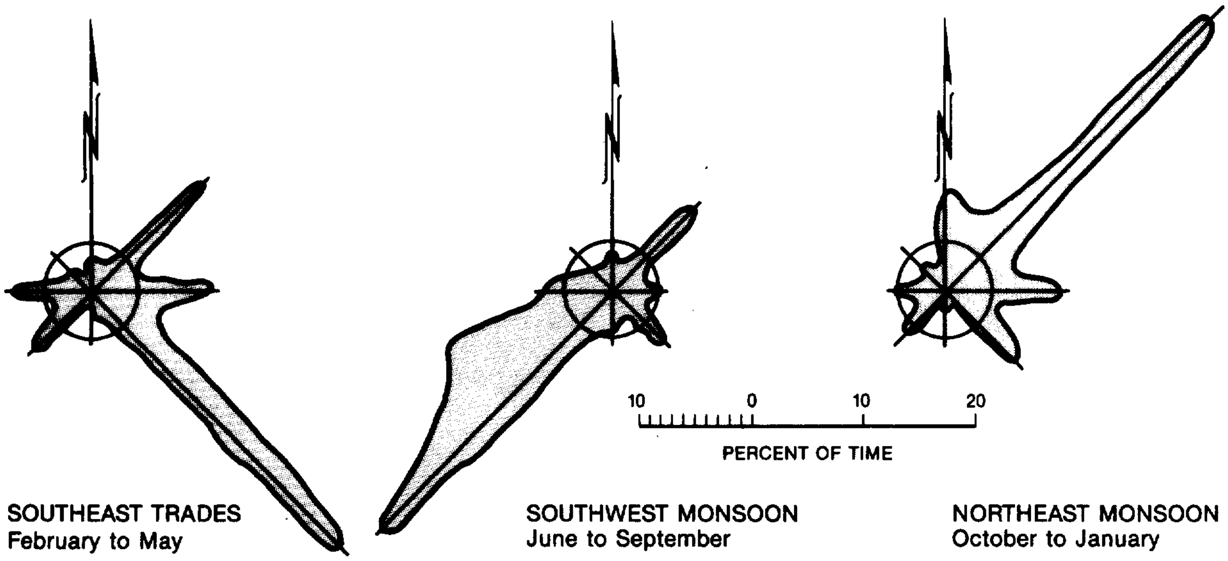


Table 15-12. Metropolitan Manila Climate Data (47).



PAMPANGA RIVER INFLOW
Cubic meters per second

	Feb	Mar	Apr	May	June	July	Aug	Sep	Oct	Nov	Dec	Jan
Southeast Trades	100	80	86	178								
Southwest Monsoon					444	737	938	754				
Northeast Monsoon									615	367	214	138

PASIG RIVER INFLOW
Cubic meters per second

	Feb	Mar	Apr	May	June	July	Aug	Sep	Oct	Nov	Dec	Jan
Southeast Trades	173	136	75	12								
Southwest Monsoon					41	149	212	250				
Northeast Monsoon									276	278	234	205

NET FRESH WATER FLUX
Cubic meters per second

	Feb	Mar	Apr	May	June	July	Aug	Sep	Oct	Nov	Dec	Jan
Southeast Trades	267	158	102	258								
Southwest Monsoon					844	1416	1858	1596				
Northeast Monsoon									1247	868	563	368

OTHER RIVERS INFLOW
Cubic meters per second

	Feb	Mar	Apr	May	June	July	Aug	Sep	Oct	Nov	Dec	Jan
Southeast Trades	77	66	62	116								
Southwest Monsoon					256	350	459	413				
Northeast Monsoon									296	208	121	85

recycling), street and channel cleaning, effective sewerage and water reclamation, and occupational safety and health.

15.6.1 Water Supply

The average yield of the existing water supply is estimated at 766 million cubic meters (Mm^3)/yr, but it is not fully utilized owing to delays in the construction of transmission mains and treatment facilities. In 1980, the average water production from municipal sources was 1,536,000 m^3/d and average water sales 860,000 m^3/d for 44 percent non-revenue-producing water. The total population served by house connections was estimated at 4.6 million (68 percent of the population) and the average volume of water sold to domestic consumers at 480,000 m^3/d, for an average per capita consumption of 104 lcd. In addition, an estimated 830,000 persons (12 percent of the population) were served by public fountains.

Water service varies widely throughout the city, but in general is extremely deficient. In most low-income areas it is provided almost exclusively by water vendors.

15.6.2 Sewerage and Sanitation in Metropolitan Manila

The first sewer system was constructed in the early 1900s to serve about 1,800 ha in the central part of Manila. The sanitation and public health conditions in Manila vary widely. The upper-income residents have piped water, septic tanks with effluent dispersed into covered drains, and regular garbage collection. In contrast, about 2 million poor people live in dense and abject conditions in blighted areas. There most housing is makeshift; water is carried from distant sources; and the areas are subject to deep flooding during rains. Sewerage, where it exists, is provided through a manual flush toilet to septic tanks, cesspools, vaults, and pits, which are often improperly designed. Effluent from these facilities (black water) and grey water from kitchen, bath, and laundry wastes drain to the nearest watercourse. Impervious and saturated soils prevent the proper functioning of subsurface drainage and seepage pits. Typhoid, cholera, and gastroenteritis are endemic. More recent statistics are unavailable and will be compiled under a proposed project.

In 1957, there were 172 public toilets in Manila; in 1980, there were only 54, most of them poorly maintained by the cities and municipalities. Public toilets are used by a large number of people: along the waterfront (the Tondo) one such public toilet served some 3,000 people despite neglect, strong odor problems, and lack of water. The Metro-Manila Commission (MMC) with technical assistance from the Metropolitan Waterworks and Sewerage System (MWSS) and the Ministry of Public Works (MPW) constructs and maintains public toilets in some areas.

15.6.3 Sewerage Systems in Metropolitan Manila (46)

The sewage generated in Metropolitan Manila at present (1980) is about 760 Mld. About 1.14 million of the population living in 2,450 ha of the service area in 1980 were served by public sanitary or combined sewer systems. Another 2.28 million of the population living in 10,970 ha of the service area at that time were served by communal or single septic tanks of varying standards of efficiency. The remainder had pit privies or, as in some poverty pockets, no service at all.

In poverty areas, sewage is commonly discharged into street gutters, open canals, esteros (tidal inlets), or rivers. Because of the lack of services and public facilities, streets in poverty areas are littered with vermin-infested garbage and excreta, and esteros are often clogged with wastes. Waterways are biologically degraded and septic conditions prevail 9 to 10 months in the year.

Sewage from about 190,000 households is discharged into individual septic tanks that are without subsurface filters and are not cleaned on a regular basis. Thirty-seven community systems with septic tanks that are pumped regularly serve approximately 320,000 people. There are about 200,000 vaults or pits in lower-income areas. Because of bad design, irregular cleansing, and poor maintenance, most septic tanks function poorly. Improved septic tank emptying services and alternative low-cost appropriate sanitation technologies are being implemented.

The major sanitary sewer system serves 828,000 people living in 1,880 ha, called MWSS's "central service area." It includes a collection network, seven lift stations, trunk sewers and an outfall pump station, force mains, and an ocean outfall at Tondo, the submarine part of which has been washed away. In 1980, the land-based part of the broken outfall discharged raw sewage near the mouth of the Pasig River. The only other sanitary sewer system is a privately owned collection system and treatment plant, which serves Makati, a residential and commercial development with 650 ha and 56,400 people.

The central service area sewer system is overloaded and badly deteriorated, and requires rehabilitation. Otherwise, the additional sewage generated under the previously funded Manila Water Supply Project II will overflow through the manholes into the streets in many parts of this highly congested area, where most of the country's trade, commerce, and some industry are centered. Most of the lift stations are obsolete and worn out.

Sewage collected within the central area collection system flows to the Tondo pumping station; from there it is pumped into Manila Bay at the mouth of the Pasig River through a badly eroded 1,050 mm outfall. By 1980, breaks--two of which are beyond repair--occurred at several points along the route of the outfall.

Laguna de Bay (see Figure 15-29), with a surface area of 890 km^2, is the largest lake in the Philippines and a major center of the fishing industry. Its waters are used for irrigation, industry, hydropower generation, and recreation, and they are expected to serve as an additional source of water supply for Manila in the future. Because of the increasing discharges of nutrients from domestic wastewaters and agricultural activities, Laguna de Bay is reportedly undergoing eutrophication. Discharges of industrial wastes and pesticides carried by runoff from agricultural areas further contribute to the lake's pollution.

By the end of the dry season, the level of Laguna de Bay has usually dropped below mean sea level and the Pasig River reverses its course, carrying into the lake polluted and saline waters. Chloride concentrations in excess of 15,000 mg/l have been recorded at the confluence of the Pasig and Marikina Rivers. Conversely, during the wet season, the lake level may rise as much as 5 m above mean sea level, far exceeding the carrying capacity of the Pasig River (700 m^3/s) and flooding extensive lakeshore areas as well as the low areas along the Pasig and Marikina Rivers.

Flood control structures are being constructed to divert the excess waters of the Marikina to Laguna de Bay and to expand the carrying capacity of the Pasig to 1,200 m^3/s. Also, a dam is being built at the outlet of the Pasig into Laguna de Bay to control the lake elevation and stop the dry season inflow of Pasig waters into the lake. These structures will considerably reduce the flows in the Pasig River during the dry season and will decrease its waste assimilative capacity.

15.6.4 Wastewater Discharges into Manila Bay

Manila Bay opens onto the South China Sea and receives all the agricultural runoff from the Pampanga River watershed, sewage and urban runoff generated in Manila, and runoff from the smaller catchments around the bay. The bay has a coastline of approximately 190 km and a surface area of about 1,800 sq km. It is a major economic asset to the inhabitants of the bay area, who use it for commercial fishing (fishponds, shellfish harvesting, fish traps), shipping, recreation (beach resorts), boating, and salt production. The increasingly uncontrolled pollution of the bay by pesticides and fertilizers from the Pampanga River by sewage discharged into the Pasig River, and by urban runoff from Manila is damaging shell fish harvesting, bathing, and boating in the bay.

The level of pollution of the bay is not fully known. Dissolved oxygen in open surface waters of the bay is generally good, varying between 6.3 mg/l (saturation value) and 8.5 mg/l. Occasional daytime values up to 11 mg/l at mouths of rivers indicate local eutrophication. Oxygen depletion occurs at depths during the wet season owing to organic sediments from the Pasig and Pampanga Rivers.

Waters in the open bay are generally clear. The east and south shoreline waters, especially during the wet season, have high turbidity and total coliform bacterial counts that often exceed standards for bathing water quality and fish farming.

Trace metals (mercury, copper, lead, zinc, cadmium) occur occasionally in water samples in concentrations above anticipated natural background levels. At times cadmium from one or more point sources exceeds 0.5 mg/l, 50 times the National Pollution Control Commission (NPCC) limit of

0.01 mg/l. Mercury is present in Manila Bay fish and other marine products consumed by humans and, with few exceptions, concentrations are within the limiting value of 2,000 mg/l set by NPCC. NPCC is planning to assist industry with the construction of facilities to reduce the discharge of wastes containing mercury, cadmium, and other heavy metals into MWSS's sewer system and the Pasig and Paranaque Rivers as part of the Manila Solid Wastes Management Study financed by the World Bank (IBRD Loan 1415-PH.).

Chlorinated hydrocarbon pesticides from agricultural runoff are present in Manila Bay surface waters and rivers flowing into the bay. However, the observed concentration levels are, with the exception of endrin, below the acceptable upper NPCC limits. Endrin exceeded the allowable limit of 0.02 mg/l, and maximum concentrations coincided with the May to September planting season. Dieldrin concentrations in the sediments are increasing. The pH in open bay waters varies between 7.5 and 8.5, and in shoreline areas between 6.8 and 8.7. The upper values indicate algal activity. Shoreline BODs vary between 5 mg/l and 8 mg/l in the dry and wet seasons. In the open waters, BODs are generally less than 1 mg/l, with no seasonal trends.

Tidal currents are most significant at the entrance to the bay. A wind-induced south to southwest net movement during the dry season (from the proposed Tondo Outfall alignment to Cavite) carrying floatables and greases to the shore at Cavite would not occur more than 10 percent of the time in a year. It is calculated that the dispersion induced by wind and tide in the inner part of the Bay provides about 1,000 m^3/s for dilution; in the wet season freshwater runoff increases this rate to about 2,000 m^3/s (46). Because of the complexity of the currents, further studies are required to obtain more data for future design.

Estimates of nutrient loads from watershed areas with a population of 8.4 million along with livestock and fertilizer loadings indicate that discharges of sewage into Manila Bay through well-designed ocean outfalls equipped with diffusers would be possible without seriously or irreversibly degrading water quality. The initial dilution is expected to preclude a near-field reduction in dissolved oxygen. A decrease in diversity of flora and fauna and an increase in biomass are expected near the diffusers. However, both the bay and the proposed Tondo outfall would be monitored by MWSS and NPCC to guard against any unforeseen situation that might degrade water quality and reduce beneficial uses of the bay. MWSS is making arrangements to acquire sites at the general locations of three proposed outfalls for future sewage treatment plants should they be needed.

15.6.5 Relationship to the IBRD Third Urban Development Project

Three components of this urban project provide water supply, combined sewers, public toilets with bathhouses, and community septic tanks: (1) the Zonal Improvement Program (ZIP) which involves comprehensive slum upgrading; (2) the Metro Manila Infrastructure Utilities and Engineering Program (MMINUTE), which includes water supply, construction of public toilets, combined and sanitary sewers, sewage disposal facilities, and road improvements; and (3) a sites-and-services program to encourage local entrepreneurs in developing housing sites. MWSS is responsible for water supply, sewerage services, construction of public toilets in the Third Urban Project, and the construction of combined sewers in the Manila Sewerage and Sanitation Project. Arrangements have been made for coordinating the two projects (see Figure 15-31).

15.6.6 Development of a Sewerage Master Plan

The development of Manila's master plan for sewerage reflects one of the more successful developing country experiences. The first sewerage master plan for Metropolitan Manila provided for a conventional sanitary sewer collection system, a main interceptor below the bed of the Pasig River, and submarine outfalls into Manila Bay (8). The Philippine authorities considered that construction and operation costs had been underestimated. More important, the plan could not succeed unless other public services were improved at the same time and urban and housing upgraded considerably.

During the 1970s, sanitary and environmental conditions worsened with the accelerated growth of the population and the urban area. Institutional responses to these changes included:

1. The Zonal Improvement Program (ZIP) to upgrade water and power supply, sanitary and storm drainage, housing, and roads and to provide health and sanitation services in selected blighted areas.

2. The Metro Manila Infrastructure Utilities and Engineering (MMINUTE) program to improve deficiencies and establish maintenance in various sectors of urban infrastructure in depressed areas through low-cost technologies.

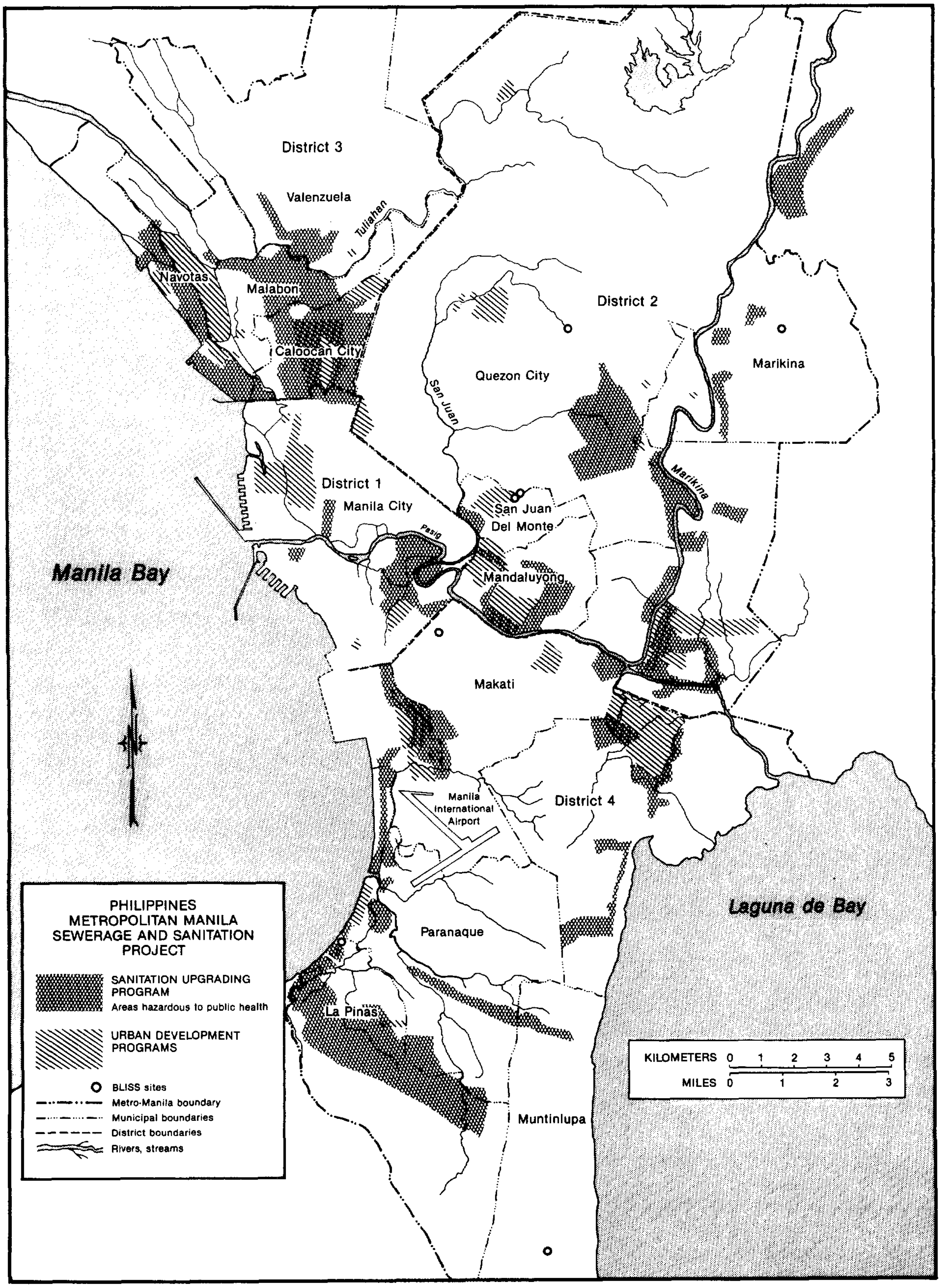
3. The Program to Reduce and Eliminate Sewage from Streets (PROGRESS) to construct minor drainage works in blighted areas.

4. The Manila-Cavite Coastal Road and Reclamation Project (MCCRRP) to develop a planned community in about 2,900 ha to be reclaimed from the sea along the southern Manila coastline. This community is expected to have a conventional sewer system and to discharge its wastewaters into Manila Bay after secondary treatment.

5. A project for protecting Laguna de Bay, which includes lakeshore interceptors to capture the industrial effluents and dry weather flows of the rivers discharging into the northern shore of Laguna de Bay. These flows would be discharged into Laguna de Bay and the Marikina River after treatment in aerated lagoons.

These programs and projects are being carried out by agencies that are difficult to coordinate and that occasionally overlap in their jurisdictions, have technical conflicts, and have difficulty understanding and predicting environment impacts of coastal zone development.

Fig. 15-31. Metropolitan Manila Sanitation Upgrading and Urban Development Programs (47).



In 1977, the city engaged the services of a consortium composed of James M. Montgomery (United States), DCCD Engineering Corporation (Philippines), and Kampsax-Kruger (Denmark) to prepare a new sewerage and sanitation master plan. The consultants made a detailed comparison of twenty-one alternatives, assuming that discharges of raw wastewaters through submarine outfalls were equal in environmental benefits to discharges into the rivers after secondary treatment (46).

The plan chosen gives priority to the sanitation of the urban area, deferring to the next century improvements in the quality of the coastal waters. It maintains and slightly expands the existing sewer system serving the older part of the city and divides the remaining urban area into three (North, Central, and South) systems, each one discharging into Manila Bay through submarine outfalls (Figure 15-29). The plan aims at serving almost half of the population with conventional sewers by the year 2000 and includes:

1. Rehabilitation of the existing sewer system and the construction of a new Tondo submarine outfall and effluent pumping station

2. Installation of conventional sanitary sewers along the coastal areas and throughout the more hilly northern part of Manila

3. Retention of most of the existing septic tank system and construction of combined sewers designed to collect dry weather storm flows and the septic tank's effluents

4. Improvement of the existing facilities for removal of septage (septic tank sludge) and pilot projects on communal toilets and other low-cost sanitation technologies (53).

The preliminary design for the new Tondo outfall in effluent pumping station calls for an 1,800-mm diameter outfall with a total length of 3.6 km, of which 1.2 km would be on land and 2.4 km under water. Average and peak design flows are 3.5 and 5.5 m^3/s, respectively.

The diffuser section would be 320 m long and about 10 m deep with ninety-seven 150-mm diameter ports to achieve an initial dilution of 1:43. The new pumping station would be provided with self-cleaning screens. Degritting facilities are not contemplated, although the failure of the old outfall is attributed to abrasion.

The total costs (including price and physical contingencies) for the outfall and pumping station are estimated at US$35.7 and US$9.7 million equivalent, respectively.

Areas and populations to be served by the master plan are tabulated in Table 15-12 and outfall characteristics on Table 15-13.

The master plan proposes that conventional sewerage be installed throughout Manila after the year 2000. This would include primary treatment prior to discharge through outfalls into Manila Bay. It considers, also, the

TABLE 15-13

Areas and Populations to Be Served by the Year 2000

System	Served by Conventional Sewerage Area (ha)	Served by Conventional Sewerage Population (000)	Served by Septic Tanks Area (ha)	Served by Septic Tanks Population (000)
North	7,331	1,157	-	-
Central	2,234	1,500	9,472	3,206
South	13,471	2,737	16,752	2,419
Totals	23,036	5,394	26,224	5,625

TABLE 15-14

Manila Bay Outfalls Characteristics

System	Length (m) a/	Diameter (mm)	Flow (m^3/s) b/	Diffuser depth (m)
North	5,700	2,300	8.4	10
Central c/	3,900	3,500	21.0	11
South	5,000	4,650	34.9	10

a. Outfall length including diffuser.

b. Projected peak flow for the year 2000.

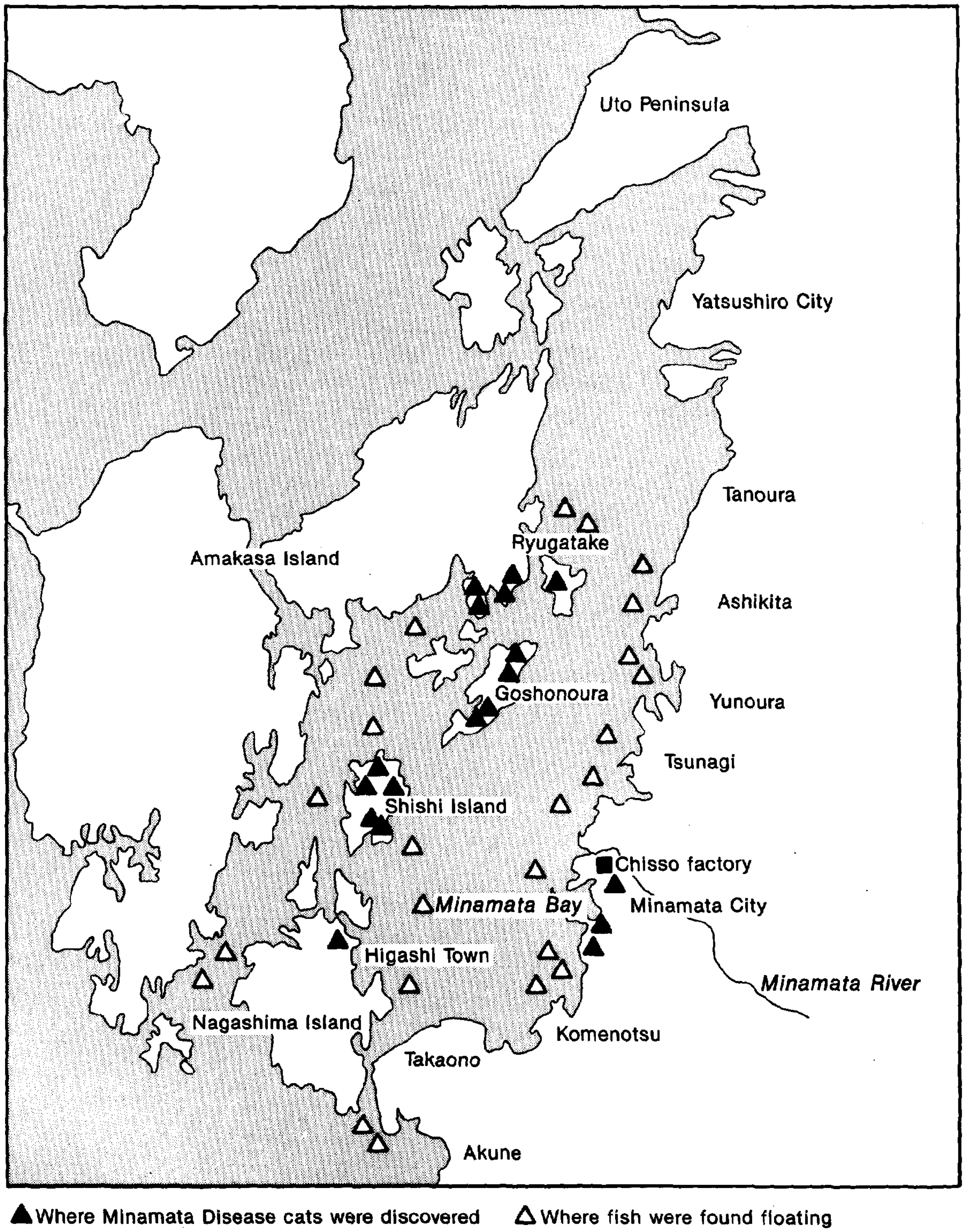
c. It does not include the flows from the existing sewer system which would be discharged through the reconstructed Tondo outfall.

possibility of constructing two additional submarine outfalls south of the present metropolitan boundaries.

15.7 Minamata Bay

The story of Minamata disease has become a classic tale of chemical product development; manufacturing and marketing; environmental, health, and economic impacts of waste disposal; efforts to link the waste to its impacts; directed research funding and findings; corporate and governmental concerns over financial implications; state-of-the-art treatment systems that didn't meet performance criteria; judicial and legislative decisions; labor violence;

Fig. 15-32. Yatsushiro/Shiranui Sea.
Source: Smith and Smith (86).



job security problems; disclosure; compensation for damages; and process or product changes. It also illustrates the length of time during which the events take place in the marine environment. It is an instructive example of inadequate and often detrimental management responses to environmental monitoring and study data. The Minamata story is told by Smith and Smith (86).

Minamata Bay is located on the edge of the Yatsushiro Sea, an inland sea connected to the East China Sea by several deep, narrow passages. (Figure 15-32) Yatsushiro Sea supports a diverse and abundant population of fish and shellfish, which are the basis of Minamata's traditional fishing economy. In 1907, the Nihon-Chisso Corporation built a chemical manufacturing facility in Minamata and began discharging its effluents into Minamata Bay through a drainage channel. The chemical and fishing industries coexisted in Minamata without major problems for several decades, although Chisso began making small indemnity payments to fishermen in 1925 to compensate them for the apparent loss of, or damage to, traditional fishing grounds at and near the mouth of the discharge channel. Acetaldehyde production, later linked to Minamata disease, began in 1932 and rapidly increased during the 1950s. Birth defects began to show up in 1947, followed by a variety of unusual changes in the area, including numerous fish and shellfish kills, and declines in both shellfish and seaweeds. By 1952, some land and seabirds began to drop into the sea while flying; many cats and some pigs and dogs in the region were found staggering, suddenly convulsing, and dying. So many cats were affected that, by 1958, virtually no cats could be found in the Minamata area.

In April 1956, four members of one family and two from a neighboring family were admitted to the Chisso Corporation's factory hospital with symptoms of brain damage, including delirium and impaired speech and balance. The disease was recognized to be similar to that affecting the local cats and initial investigations quickly uncovered thirty additional cases. Many of these people had been ill for three years or more and their illnesses had been ascribed to various causes. Research on the disease proceeded rapidly and, by October 1956, a report was released correctly concluding that the "disease" was noninfectious and that it was heavy metal poisoning caused by eating the fish and shellfish of Minamata Bay. However, the metal or metals responsible and their source had not yet been identified, although the Chisso effluent, being the only large industrial source in Minamata Bay, was implicated early in 1957.

Early recommendations that consumption of fish and shellfish from Minamata Bay be restricted and the Chisso effluent tested were not accepted. Chisso-sponsored research reported a lack of certainty concerning the linkage of its effluent with the disease, and put forth hypotheses attributing the cause to "putrified amines" from decaying fish or the dumping of explosives by a then defunct army as causes. Further, many residents did not favor implementation of the remedial actions since the people relied upon fishing and the chemical plant for their employment. In view of the lack of public support, the government took no action. In 1957, proof was obtained that fish and shellfish incorporated a poison or poisons from the waters and food chain of Minamata Bay and that this poison was responsible for the "disease." Fish from an uncontaminated area were kept in Minamata Bay for several days and were then fed to cats from uncontaminated areas; these cats acquired the disease within a matter of weeks. Meanwhile, the industrial effluent continued to flow untreated into Minamata Bay, fish and shellfish from the Bay continued to be eaten without any harvesting or consumption controls, and new cases of the "disease" continued to appear. Finally in 1958, the local (prefecture) government placed a ban on the sale (but not on consumption) of Minamata Bay fish. In addition, Chisso diverted its acetaldehyde plant effluent to the nearby Minamata River, which discharges to the north of Minamata Bay. Within a few months, cases of Minamata disease began to appear in the river area.

By 1959, Chisso had learned through its subsequently published research that its acetaldehyde plant effluent caused Minamata disease and installed an ineffective "Cyclator" pollution control device on the waste stream. At the same time, methyl mercury was identified as the toxic organic metal compound causing the disease, and extremely high concentrations of mercury were found in the marine environment (sediments and biota) of Minamata River and Bay, and in affected cats and humans. Following initial claims that only inorganic mercury was in the acetaldehyde waste stream, it was learned that mercury was methylated both during the acetaldehyde process itself and also in the marine environment after release as inorganic mercury.

Despite the evidence, the acetaldehyde process effluent was continued, people continued to eat seafood from the polluted area, and new cases of the "disease" continued to appear. By 1962, there had been 121 confirmed cases, 46 of whom died. In 1959 Chisso did, however, pay small indemnities to victims of the disease through a consent decree according to which they were not required to admit their legal guilt. The acetaldehyde effluent continued to flow into the sea until 1968 and might have continued much longer if it had not been for a second outbreak of the disease discovered in 1966 in Niigata, where mercury contamination from another chemical plant affected almost 500 people and led to a lawsuit that was accompanied by extensive publicity. As a result of lawsuits and settlements, the Chisso Corporation had by 1975 paid eighty million dollars in indemnities to Minamata area victims. In addition to these costs, there had been the loss of many years of fishery resources in Minamata Bay, which was still badly polluted with mercury in 1981 (76), more than a decade after the discharge of mercury was stopped. The Japanese government has considered remedial dredging or impoundment and burial in place of approximately 50 km^2 of Minamata Bay sediments (27). No such action has yet been taken, however, because of the extreme cost and uncertainty that it would be effective.

Meanwhile, the rest of the world, although fully informed about Minamata disease, did not initially take any appropriate action to prevent occurrences elsewhere. During the early 1970s, putative evidence of a mild Minamata disease outbreak was found in Canada (86) and natural methylation of mercury with adverse effects on fish and birds was reported in many more environments near chemical plants worldwide. When governments did take notice, many overreacted, and some entities, including the United States and the World Health Organization, set stringent standards for mercury concentrations in seafood. These standards caused a near-global panic so that some seafood, no matter where it was caught, was thought to be contaminated so badly with mercury that it was unsafe to eat (see Chapter 14). Because methyl mercury is biomagnified up the marine food chain, it is found in relatively high concentrations in top pelagic (deep open ocean) predators such as tuna. In many countries health authorities and the public alike panicked, even though the concentrations observed did not approach the concentrations found in Minamata seafood. Overenthusiastic environmentalists caused the public to fear that man had seriously polluted the entire world ocean with mercury. It took several years of research to demonstrate that the observed concentrations in tuna or other pelagic fish had been there for almost a century, that they were natural and not caused by massive global mercury contamination (3), and that normal consumption of fish was quite safe (85).

The lessons to be learned from the mercury poisoning of Minamata are many, but perhaps the most important are as follows:

1. Effluents from man's activities can pollute the local marine waters enough to cause severe local health and environmental hazards, particularly in areas with restricted circulation.

2. Effluent discharges should be carefully located so that the effluent can be adequately dispersed and flushed where possible, in areas remote from major fisheries. Even a cursory examination of Figure 15-34 shows that the Yatsushiro Sea and Minamata Bay are poorly flushed and so protected from ocean waves that dispersion is poor. In addition, it is clear from the consequences that the simple effluent channel utilized for many years and the subsequent, short-lived discharge into the Minamata River provided very poor dispersion and flushing of the wastes. Proper siting of effluent discharges is essential and fundamental to safe operation.

3. Monitoring for unexpected effects of marine discharges can be most effectively conducted by gathering observations and information from fishermen and coastal residents. Although the appearance of floating fish and the bizarre behavior of cats and birds were noted in Minamata, action was not taken until after some years.

4. Techniques for mercury analysis in environmental samples were not reliable when Minamata disease first appeared. Therefore, effluent monitoring would not have led to early detection of the problem, since mercury, and particularly methyl mercury, would not have been measured. Although mercury can now be readily assayed, no generally accepted techniques exist for many other metals, such as thallium, selenium, and many synthetic organics, which, therefore, cannot be included in a field monitoring program. In all cases, anecdotal data and observations of physiological aberrations, and information on existing and proposed manufacturing processes and production are essential monitoring tools.

5. The scientific evidence that finally confirmed the connection between the Chisso acetaldehyde waste and Minamata disease came from a relatively simple set of laboratory bioassays. In this case, the critical food chain was simulated in the laboratory by exposing fish to high concentrations of the effluent and feeding those fish to cats. The cats, of course, became sick and died. Fish and other animal bioassays should be a fundamental part of industrial effluent-monitoring and assessment programs.

6. Once the "disease" had been recognized in Minamata, scientific studies quickly identified the Chisso acetaldehyde waste as the cause of the problem, but the management and political systems to which that information was first revealed were slow to respond because of financial and hence political considerations.

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CHAPTER 16

COSTS

The initial costs of submarine pipelines include those for (i) engineering, rents, licenses, and other legal costs and institutional, economic, and environmental advisory services; (ii) mobilization and demobilization; and (iii) materials, equipment, and construction. Reported costs for ii and iii above for 33 completed municipal wastewater outfalls and 7 completed cooling water discharge lines are listed on Table 16-1. Also listed are estimated costs for three tunnel outfall systems under construction at Sydney, Australia. Unit construction costs per meter have been normalized to an Engineering News Record (ENR) construction index of 4300 (5). Most costs were reported in U.S. dollars. Costs originally reported in other currencies were converted according to IMF exchange rates (9).

The tabulated costs do not include those for engineering, legal and related services; these are site-specific and typically range from 10 to 25 percent of construction costs (6, 14). Mobilization costs vary from 10 to 50 percent of total bid prices (1, 6, 12). Conventional engineering feasibility study estimates explicitly exclude mobilization because of its uncertainty (8), and are accordingly biased in favor of apparent economies of scale.

Cost escalation factors are often based on extrapolation of ENR indices. As with other indexing systems, these tend to be both self-fulfilling and inflationary. They may also promote institutionalizing excess capacity and legitimizing underestimated costs. The ENR index essentially doubled between 1950 and 1967, quadrupled between 1967 and 1982, and increased at a lower rate to 1986 for annual compounding rates of 4.2 percent, 10.5 percent, and 1.7 percent, respectively (5). Cost figures for this discussion have been normalized to an ENR index of 4300, a figure expected in 1987.

Cost escalation figures for Sand Island, Honolulu (Hawaii), Manila (Philippines) and Sydney, show probable ranges of percentage increases. In Honolulu, the September 1962 estimate for the Sand Island outfall was 9.6 million (1986 dollars) but was bid in October 1973 at $13.57 million (6). The Sydney outfall cost increases (Table 15-14) reveal the uncertainty in estimating hard-rock construction (2). Wet tunneling is particularly difficult, whether in fractured rock or muds, and is from 10 to 50 percent more expensive than work in dry, competent rock (4).

Unit cost data listed in Table 16-1 are presented in Figure 16-1. The figure reveals two populations of costs, one for diameters of up to about two meters diameter where costs quadruple for each doubling of the diameter. For larger diameters, the costs increase by a factor of about 6 as the diameter is doubled. Outfall lengths, which here include diffuser sections, do not appear to be a significant factor for a given diameter (13). A similar discontinuity in the diameter:cost function using a smaller data set was reported in 1968 (3).

TABLE 16-1. Construction Costs for Selected Submarine Pipeline

No.	Location	Year	ENR Index	Diameter, m	Length, m (incl. diffusers)	ENR Factor	Reported Cost, USD, millions	Normalized Unit Cost, USD/m	
1	Dubrovnik YU	1974	3900	0.45	2 @ 1500	2.13	5.5	2,020	HDPE
2	Coos Bay OR	1972	1753	0.52	1440	2.45	1.8	3,062	
3	Toledo OR	1965	971	0.53	1130	4.43	0.96	3,763	
4	Los Angeles CA	1957	724	0.56	11260	5.94	2.6	1,372	7-mile sludge outfall
5	Passaic NJ	1920	357	0.60	460	17.1	0.1	3,717	Raritan Bay
6	Carmel CA	1971	1581	0.61	270	2.72	0.41	4,130	
7	San Elejo CA	1965	1020	0.76	820	4.43	0.96	1,188	Cooling water
8	Seattle WA	1962	872	0.76	640	4.93	0.27	1,509	
9	Seattle WA	1962	872	0.84	490	4.93	0.15	2,079	
10	San Mateo CA	1962	872	0.84	490	4.93	0.84	2,660	
11	Oceanside CA	1972	1753	0.91	2500	2.45	1.9	1,862	Extension
12	Watsonville CA	1959	797	1.0	1170	5.40	0.47	2,169	
13	Encina CA	1964	926	1.2	1370	4.59	0.35	1,172	Cooling water
14	San Francisco CA	1974	2020	1.2	180	2.13	0.57	6,745	
15	Mokapu HI	1977	2577	1.2	1547	1.67	6.2	6,687	
16	San Francisco CA	1966	1019	1.4	250	4.22	0.46	7,765	San Francisco Bay
17	Bellingham WA	1973	1895	1.5	850	2.27	0.44	1,180	Puget Sound
18	Istanbul TU	1986	4300	1.6	2 @ 1162	1.00	12.9	5,555	Bid price
19	Hampton NC	1981	3600	1.7	2930	1.19	12.3	4,996	
20	Sao Paulo BR	1976	2401	1.75	4000	1.79	31.7	14,175	
21	Contra Costa CA	1959	797	1.8	520	5.40	0.17	1,765	San Francisco Bay
22	Ponce PR	1972	1753	1.8	1550	2.45	3.3	5,216	
23	Encina CA	1973	1895	1.8	700	2.27	1.05	3,405	Cooling water, ext.
24	Suffolk NY	1981	3600	1.8	5577	1.19	28.8	5,141	
25	Manila PI	1985	4220	1.8	3600	1.00	13.0	3,632	Manila Bay
26	Sand Island HI	1976	2401	2.1	3816	1.79	13.6	6,379	
27	L.A. County CA	1954	628	2.3	3170	6.85	2.2	4,754	Force Account
28	Seattle WA	1964	986	2.4	1110	4.59	1.2	5,508	
29	Ipenema BR	1975	2212	2.4	4325	1.94	22.0	9,888	
30	Bombay IN	1985	4220	2.4	6000	1.00	65.0	10,833	Delta muds
31	San Diego CA	1962	872	2.7	4340	4.93	10.5	11,930	
32	San Francisco CA	1970	1385	2.7	90	3.10	0.40	13,778	
33	L.A. County CA	1964	986	3.0	3620	4.59	4.5	5,706	Force Account
34	Orange Co. CA	1969	1269	3.0	8350	3.30	9.0	3,665	
35	Redondo Bch	1947	413	3.2	2650	10.4	2.2	10,473	Cooling water
36	El Segundo CA	1954	629	3.3	2620	6.85	2.3	6,013	Cooling water
37	Los Angeles CA	1950	570	3.6	1610	8.43	2.0	10,473	Hyperion 1-mile
38	Huntington Bch CA	1957	724	3.6	820	5.94	1.5	10,866	Cooling water
39	Los Angeles CA	1957	724	3.6	8046	5.94	20.2	14,912	Hyperion 5-mile
40	San Onofre CA	1965	971	3.6	1740	4.43	3.3	8,402	Cooling water
41	Sydney, Austr.	1975	2212	2.4	3400	1.94	28.0	15,976	
	Bond	1986	4300	2.5	2300	1.00	55.0	24,000	
42	Sydney, Austr.	1975	2212	2.8	3400	1.94	25.0	14,265	Tunnel systems
	North Head	1986	4300	3.5	3400	1.00	68.0	19,400	under construction
43	Sydney, Austr.	1975	2212	3.1	3100	1.94	25.0	15,645	
	Malabar	1986	4300	3.5	3500	1.00	55.0	19,200	

Note: Both length and cost figures include those for diffuser systems.

Sources: Clancy and Carroll (2), ENR (5), Grace (6), Hennessy (8), Wallis (13), World Bank (14).

The higher costs for larger diameter outfalls presumably reflect costs of materials and their transportation, constraints on construction methods (see Chapters 5 through 12), greater sophistication in diffuser systems, increased demand for scarce professional and technical skills, and higher mobilization and infrastructure costs.

Some provisional conclusions may be drawn from Table 16-1 and Figure 16-1. First, the greater departures from the median are related both to location and to function. The high cost at Sao Paulo presumably reflects mobilization. Cooling water lines tend to be less expensive than outfalls. Force account projects utilizing owner staff and construction crews tend to be lower in cost. Larger California cities report generally higher unit costs. The data do not support a rational characterization of the expensive, ordinary, and expensive categories published by Wallis (13). However, the discontinuity in the median cost function is an important determinant in evaluating the marginal costs of increasing outfall capacities.

Figure 16-2 shows the costs of each additional cubic meter per second capacity for 100, 500, and 1,000-meter long outfalls with an initial (elevation) head of 10 meters, based on the Manning equation. The Manning formula is appropriate for pipes with diameters over 200 mm and velocities of about 1 m/s (the Hazen-Williams formula is generally preferred for 100 to 200 mm diameter smooth pipes).

$$Q = d^{8/3} s^{1/2}/n$$

where
Q = flow, m^3/sec,
d = pipe diameter, meters,
s = slope = length/elevation head, and
n = roughness coefficient

Incremental capacities in m^3/sec are shown for "n" values of 0.011 (plastic pipe, smooth lined steel pipe) and 0.015 (concrete pipe) as functions of pipe diameter (6, 11). Unlined steel pipe has an "n" value of about 0.012.

Figure 16-2 shows the higher marginal costs of additional outfall capacities that are associated with smaller diameters, greater lengths, and higher "n" values. However, the discontinuity in unit costs is demonstrated in the 1.2 to 3.2 diameter range. Within this range, additional capacities may be obtained at the same cost with either smaller or larger pipes. The significance of this choice is that the smaller pipes more readily lend themselves to closer matching of capacity to demand and hence lower costs of unused capacity during the early years of the project life. Procedures for evaluating these costs are presented by Kalbermatten, et al (10). Institutional advantages of the closer match include realization of full project benefits during shorter periods, which more closely correspond to political schedules.

In sum, a wide range of site-specific environmental characteristics, institutional constraints, costs, and benefits of ocean outfall systems is apparent. Specific needs for competent external or peer review of planned systems where outfall diameters exceed about 1.5 m also exist. These issues emphasize the need for early identification and comparative economic costing of all feasible, technological, and institutional options for water supply and wastewater management and for environmental protection in coastal cities.

Fig. 16-1. Unit Construction Costs for Submarine Pipelines

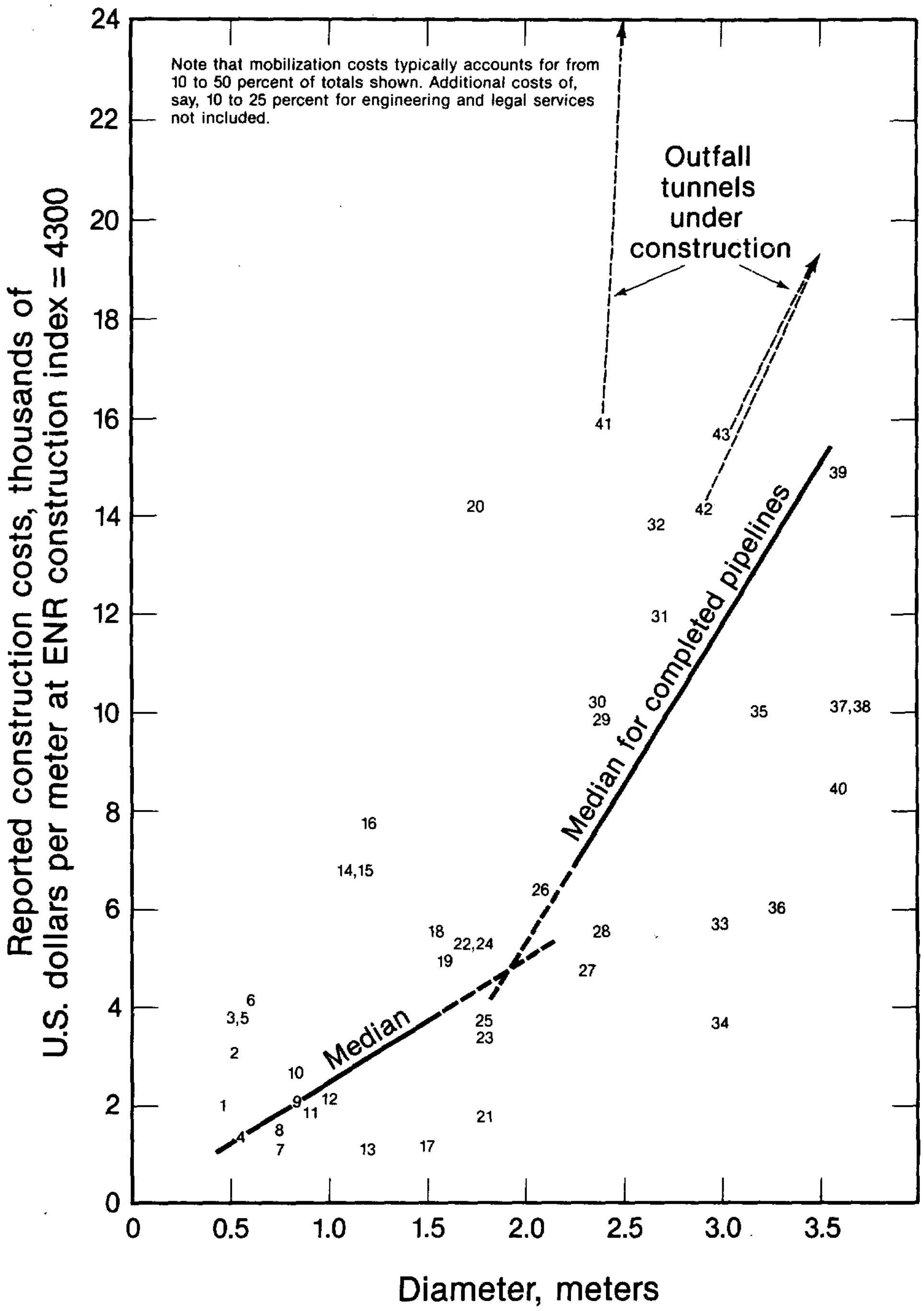
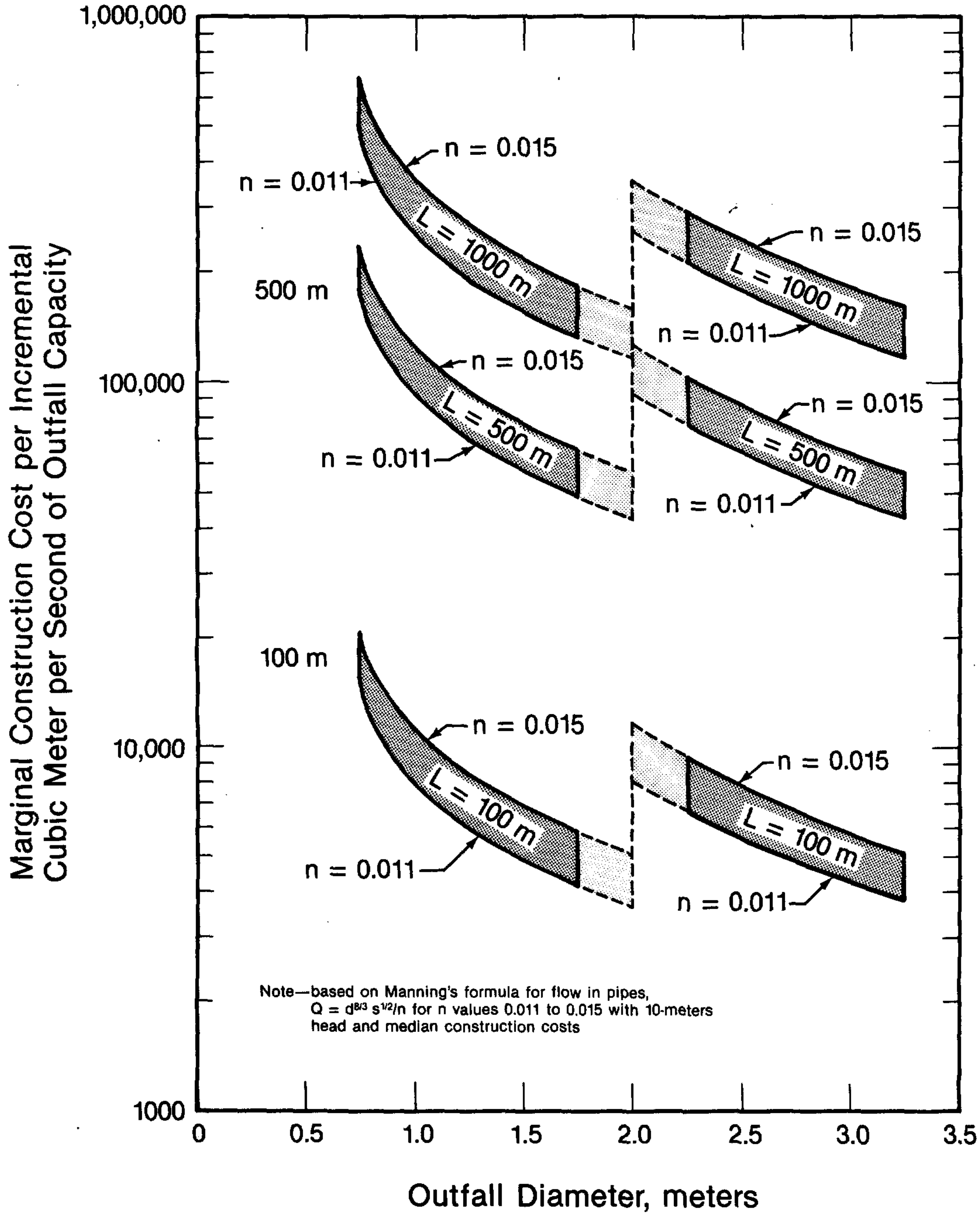


Fig. 16-2. Costs of Incremental Capacities Achieved by Increasing the Diameters of 100, 500, and 1,000 Meter Outfalls



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APPENDIX A

ATLAS SECTION

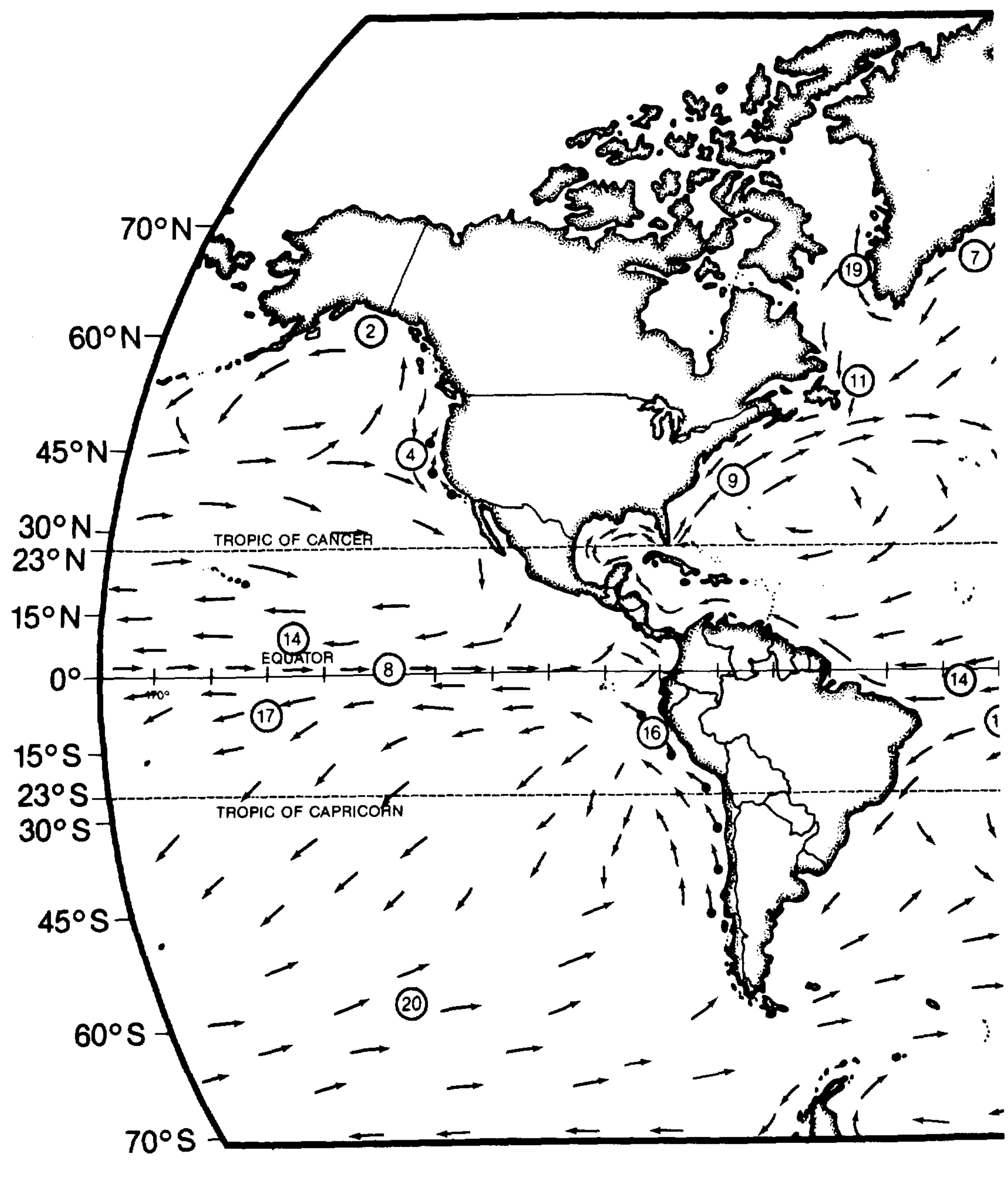
Figures A-1 through A-6 present environmental information on the ocean margin throughout the world to be considered in wastewater management system selection. Figure A-1 shows locations of major ocean surface currents and of coastal upwelling. Where currents diverge from each other or from a coastline, nutrient-rich deepwater comes to the surface and supports high production of marine plants and animals (Figure A-7). The latter are particularly exploited in depths to 150 meters over continental shelf areas (Figure A-2). Global-scale climatic zones are shown on Figure A-3.

The ecological climate diagrams on Figures A-4 through A-6, adapted from H. Walter, E. Harnickell and D. Mueller-Dombois, Climate-Diagram Maps, 1975, Springer-Verlag, Berlin, reveal both site-specific and regional factors which are most important for plant growth. They are accordingly reliable indicators of the eventual need for wastewater reclamation in coastal cities. The diagrams include station elevation, average annual temperature and precipitation, and monthly average temperature and precipitation. The latter are plotted on vertical scale divisions of 10°C and 20 mm/month. Above 100 mm/month, the precipitation scale division is 100 mm/month. Months are shown for January through December in the northern hemisphere and July through June in the southern. Thus summer always falls in the middle of the period shown. Relatively humid months are shown by vertical lines and relatively arid seasons by dots. Both are functions of potential evaporation. Note that the effects of long-term drought are most severe in arid climates and that drought resistance of economically important plants is generally on an annual cycle.

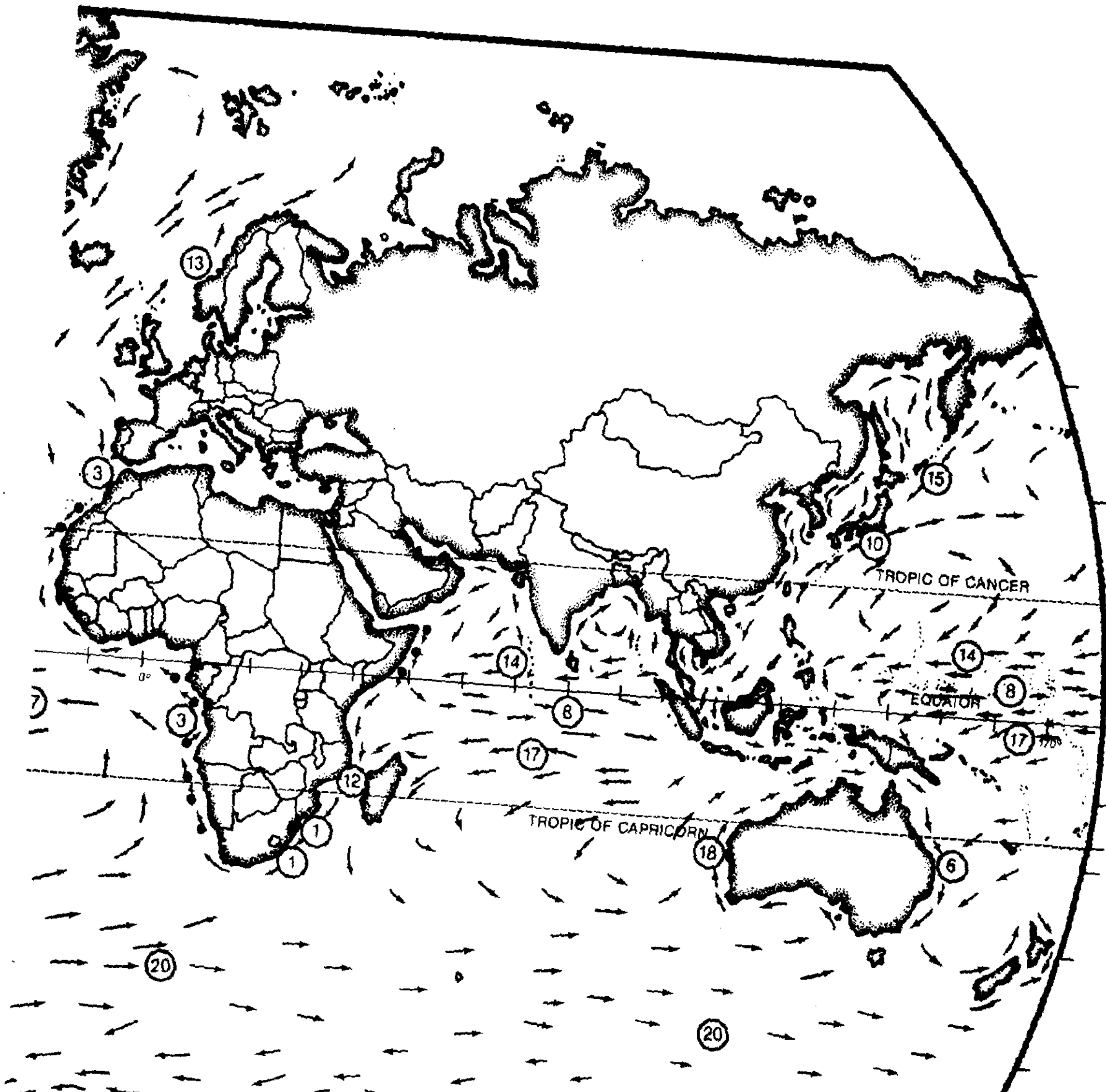
Figure A-8 illustrates variations of tidal ranges which affect construction along shorelines. Their consequences are discussed in Chapter 8 and 11.

Table A-1 is supplemental to Figures A-4 through A-6 and presents numerical data on climate for sites considered in development of the algorithm in Table 16-7. Table A-2 is supplemental to Figures A-3 through A-6 and lists native vegetation types for different climate zones.

Fig. A-1. The World Ocean and Its Currents. • indicates coastal upwelling (after Defant, A. 1961. Physical oceanography, 2 vols. Pergamon Press, Oxford; and Weisman, W.J., Mooers, C.N.K., and Forrestall, G.Z. 1983. Ocean Sciences and Technology, Vol. 8 No. 4. pp. 367-459).

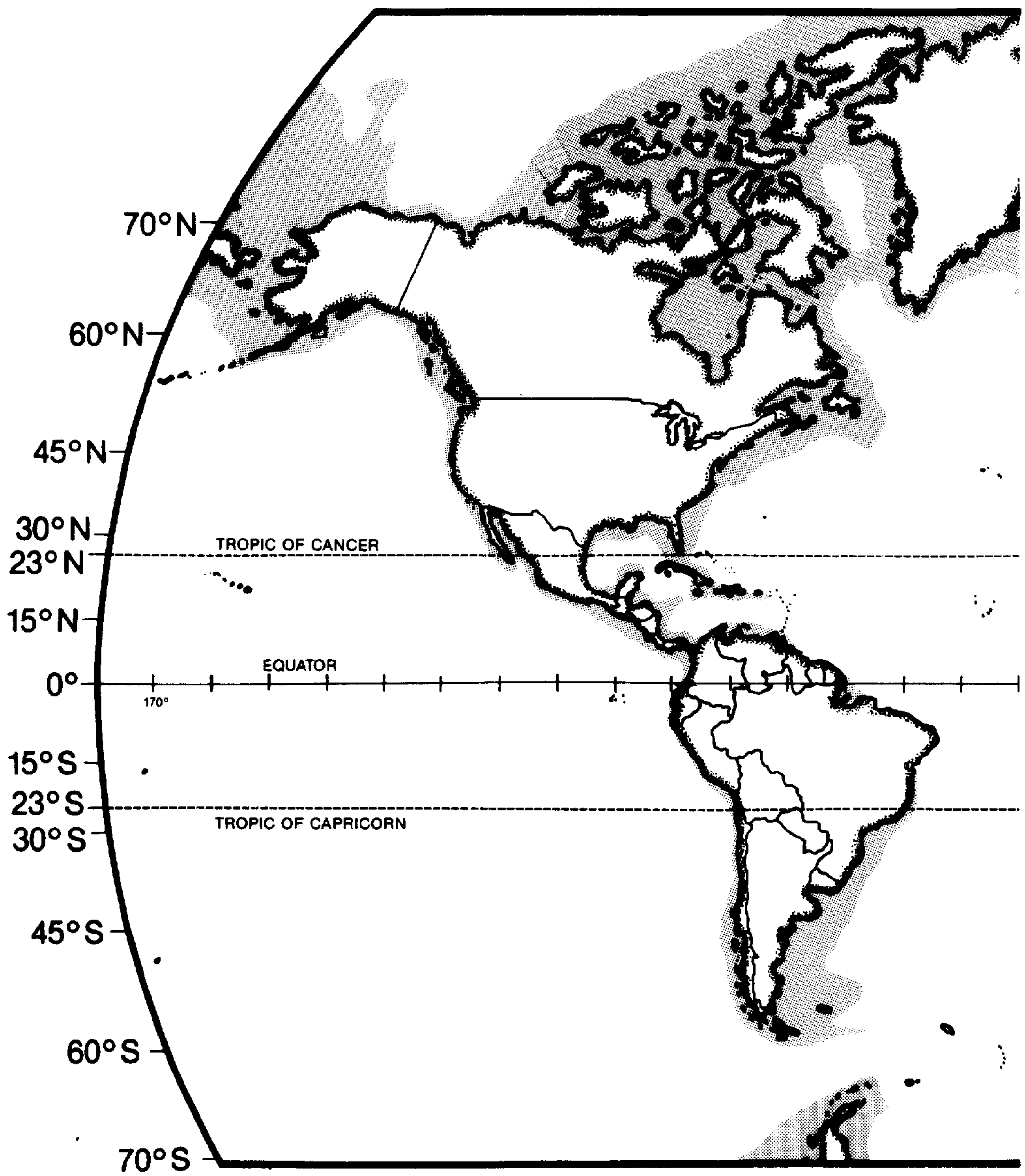


1	Argulhas C.	6	East Australia C.
2	Alaska C.	7	East Greenland C.
3	Benguela C.	8	Equatorial Counter C.
4	California C.	9	Gulf Stream
5	Canary C.	10	Kuroshio C.



11 Labrador C.
12 Mozambique C.
13 Norwegian C.
14 North Equatorial C.
15 Oyashio C.
16 Peru C.
17 South Equatorial C.
18 West Australia C.
19 West Greenland C.
20 West Wind Drift

Fig. A-2. Continental Shelf Areas of the World (after Anderson, E.R., 1964. Single depth charts of the world's ocean. Report 1252/Ses 1. U.S. Navy electronics laboratory, San Diego). Shaded areas indicate depths to 150 meters.



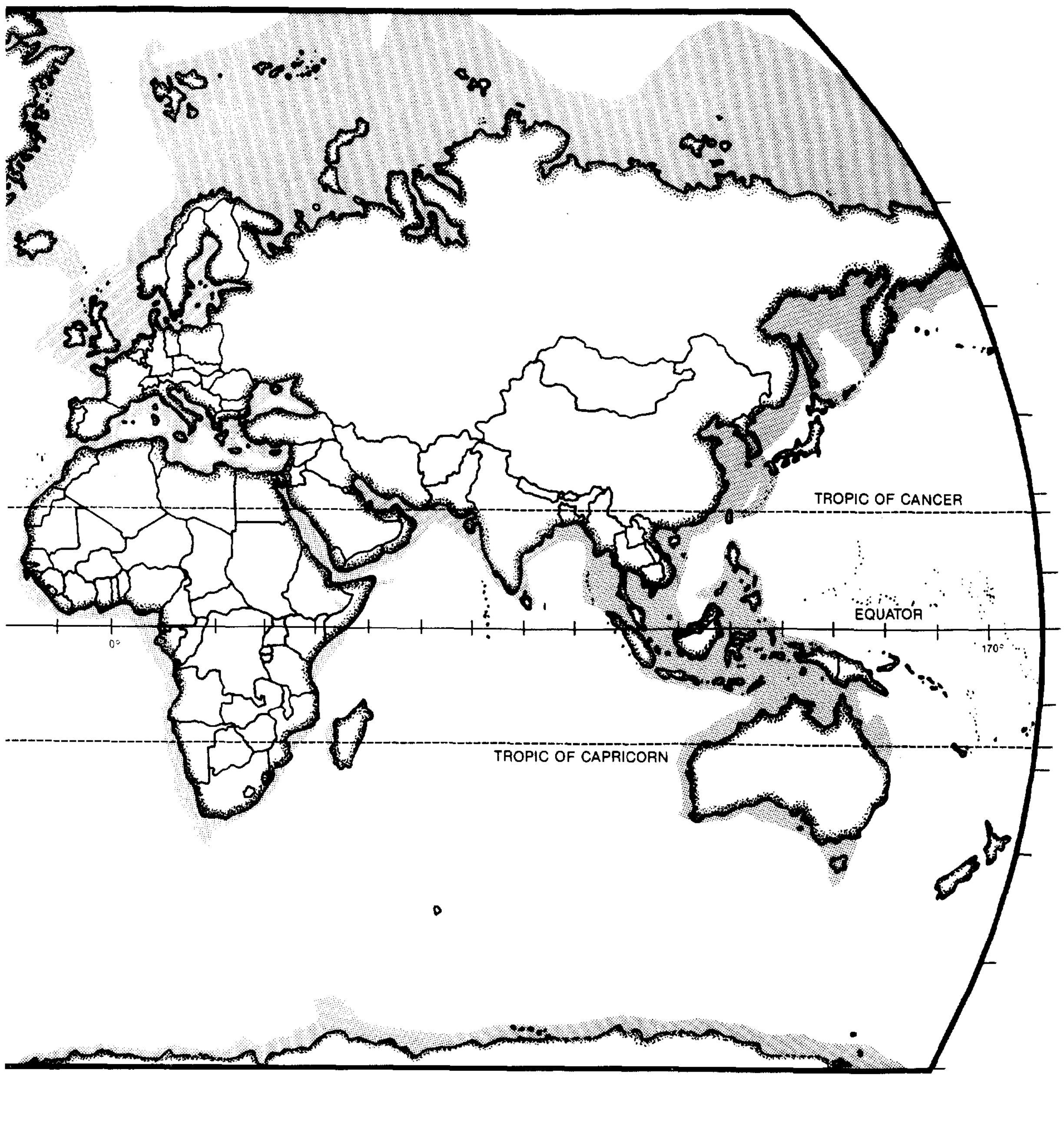
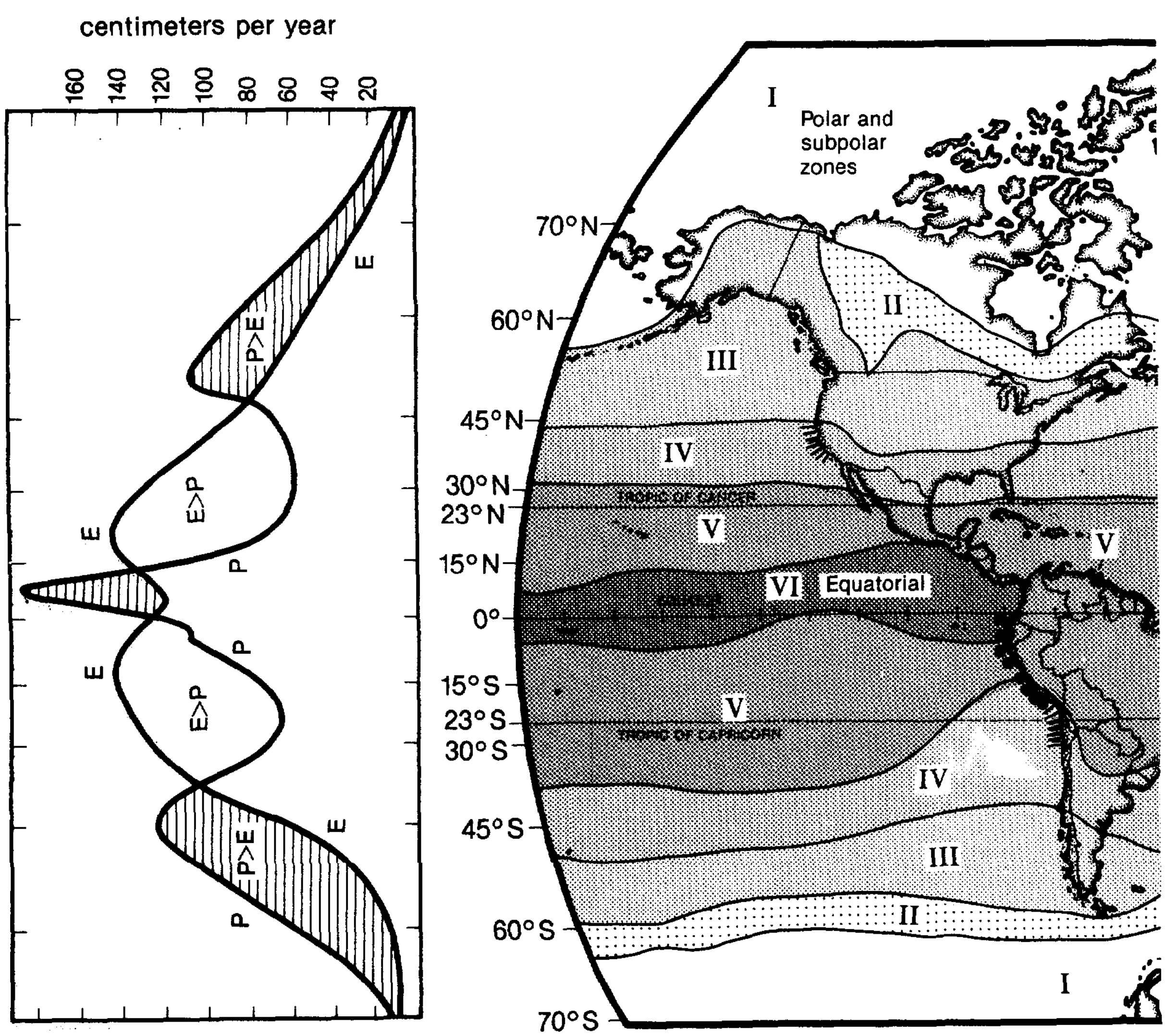

TROPIC OF CANCER
EQUATOR
0°
170°
TROPIC OF CAPRICORN

Fig. A-3. Global Seasonal Climate Zones and Latitudinal Oceanic Precipitation, P, and Evaporation, E (after Landsberg, H.E., Lippman, H., Pfaffen, K.H., and Troll, C. 1966). Weltkarten zur Klimakunde, Springer Verlag, Berlin; Gross, M.G. 1980, Oceanography, Charles E. Merrill Publishing Company, Columbis, Ohio), and Collier, A.W. 1950. "Oceans and coastal waters and life-supporting environments," in Kinne, O. Marine Ecology, 2 vols., Wiley interscience, London). Note: Coastal mist: ••• - winter; - summer.



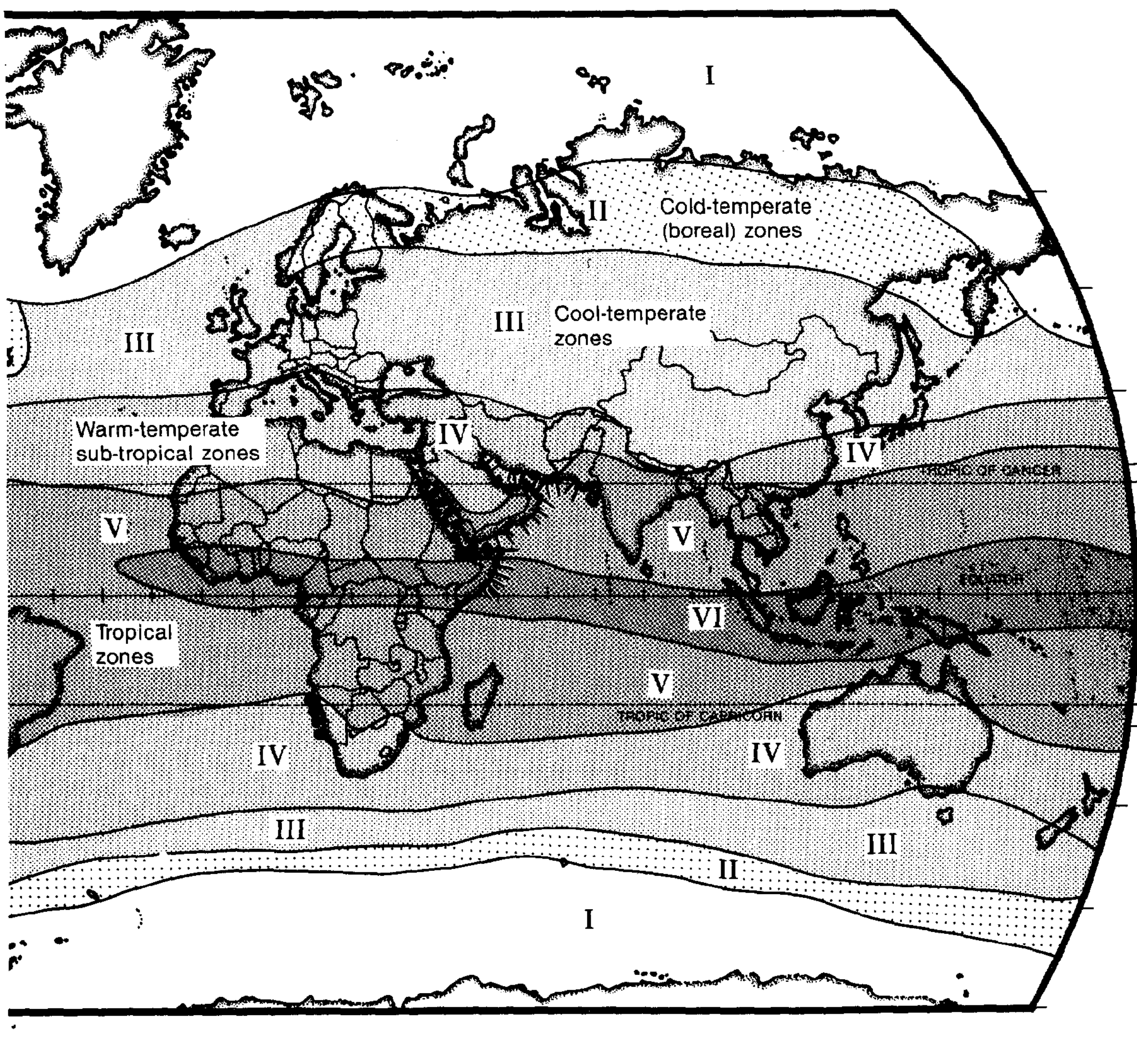

I
II
Cold-temperate
(boreal) zones
III
III
Cool-temperate
zones
Warm-temperate
sub-tropical zones
IV
IV
V
V
TROPIC OF CANCER
EQUATOR
VI
Tropical
zones
V
TROPIC OF CAPRICORN
IV
IV
III
III
II
I

Fig. A-4A. Coastal Climate Diagram - North America.

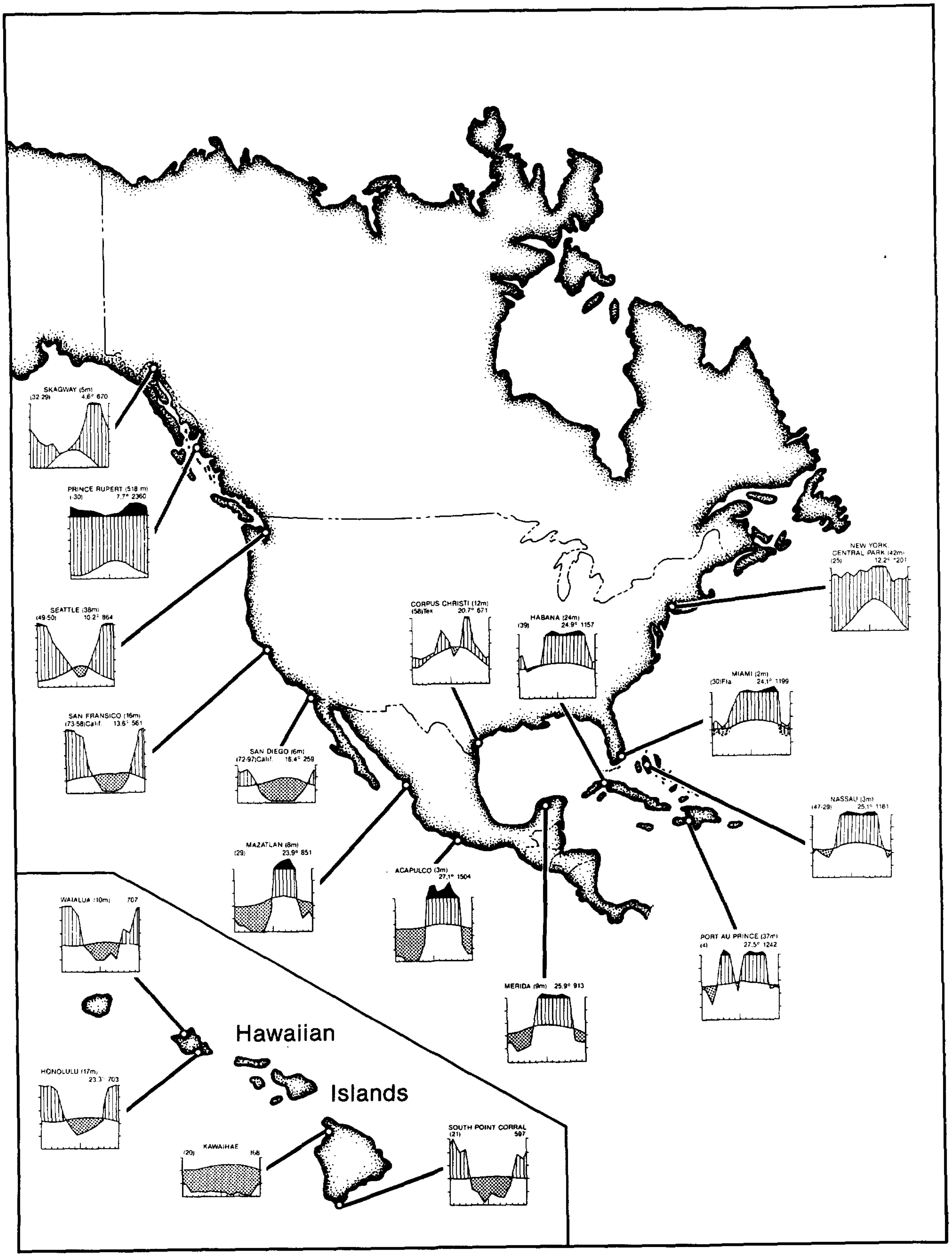


Fig. A-4B. Coastal Climate Diagram - South America.

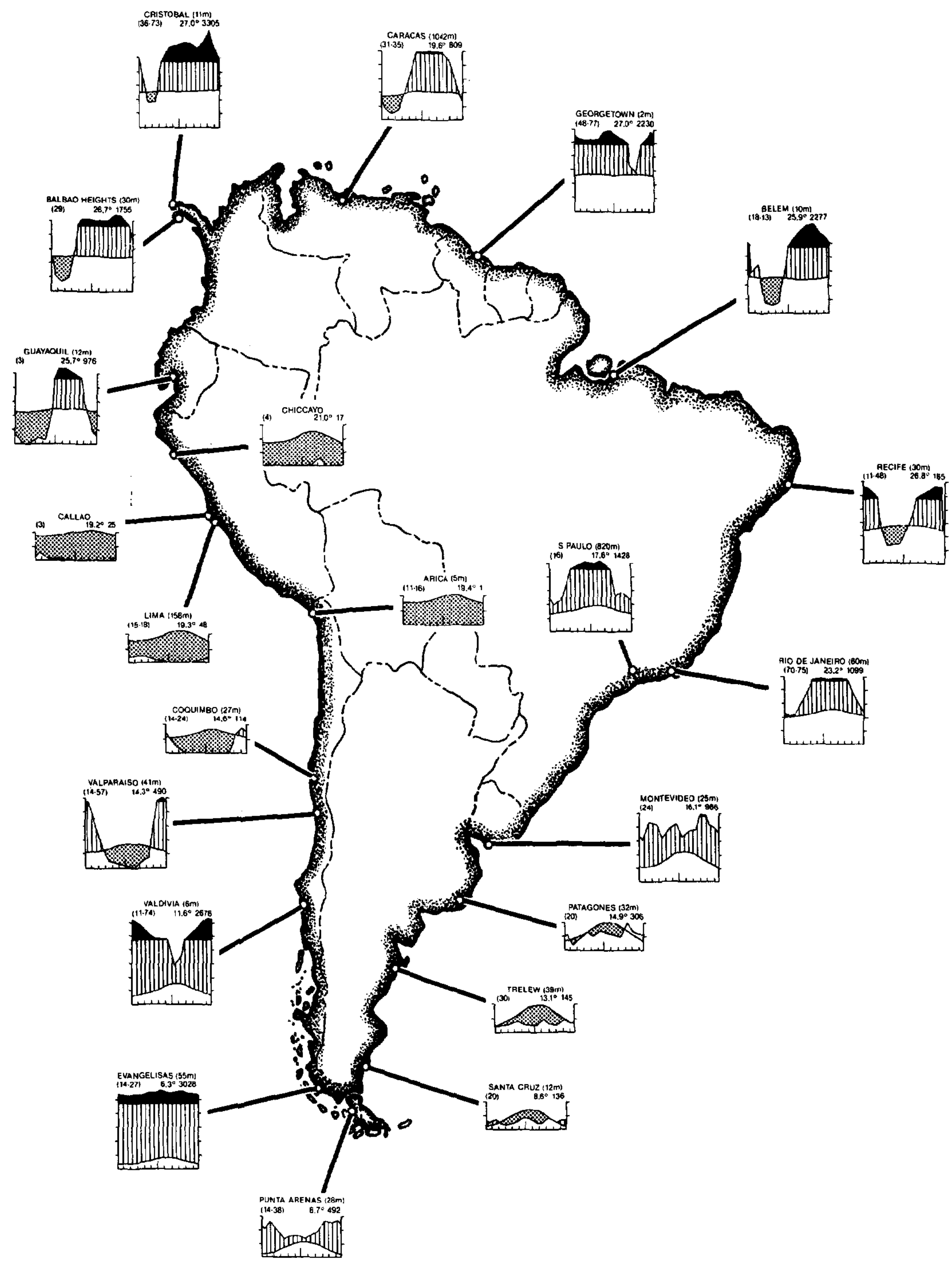
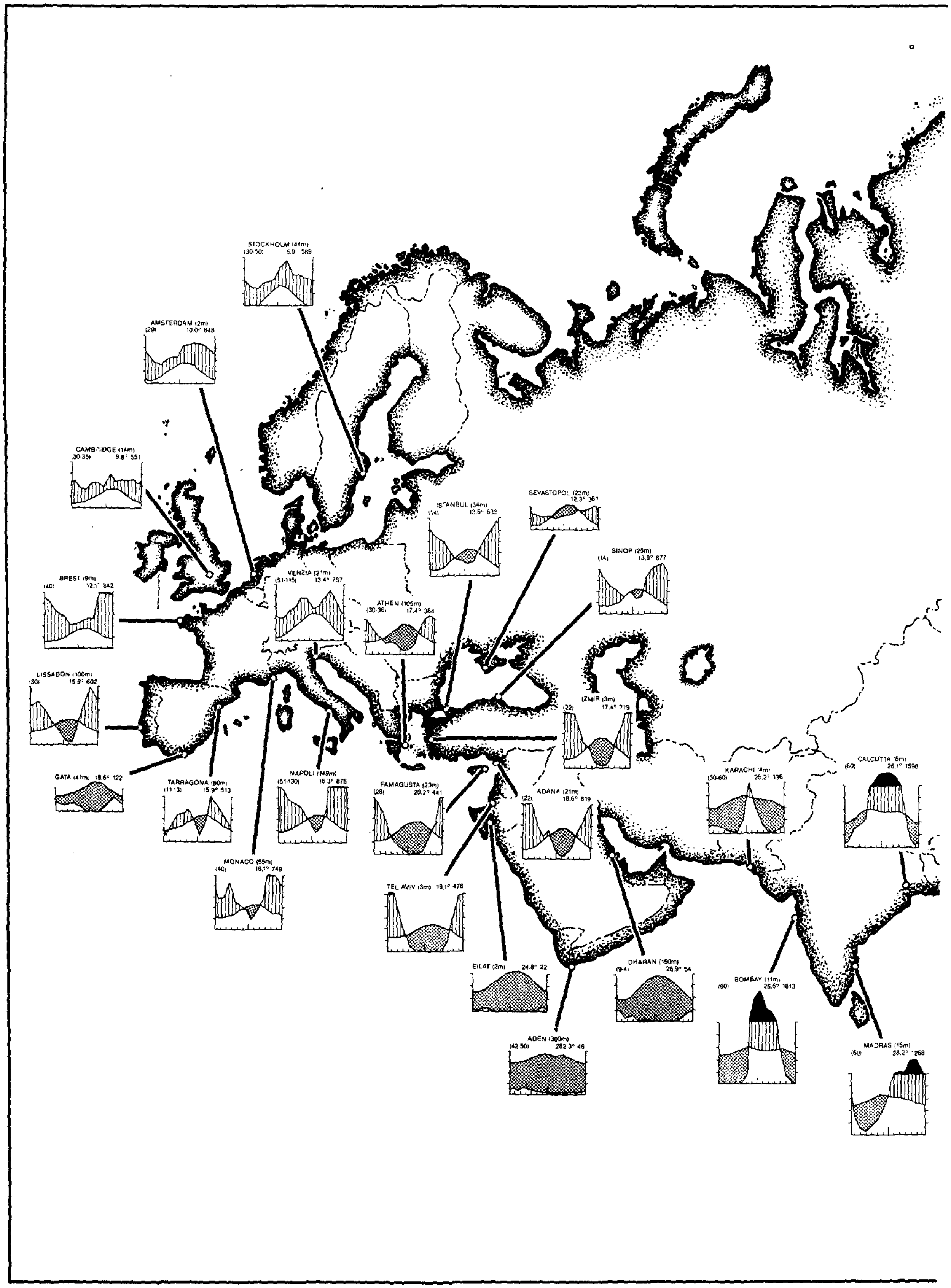


Fig. A-5. Coastal Climate Diagram - Europe and Asia.



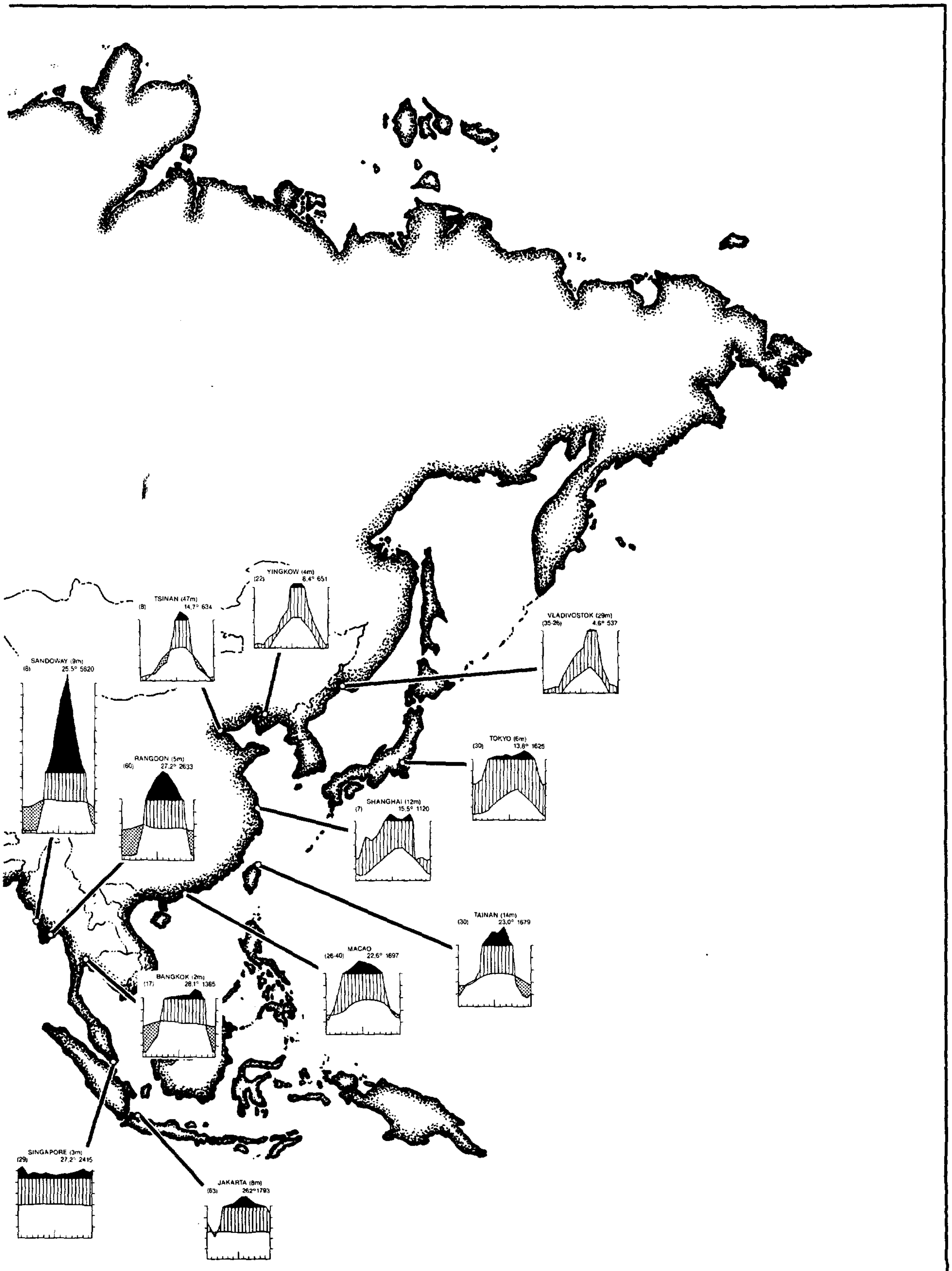

YINGKOW (4m)
(22) 8,4° 651
TSINAN (47m)
(8) 14,7° 634
VLADIVOSTOK (29m)
(35-26) 4,6° 537
SANDOWAY (9m)
(8) 25.5° 5620
TOKYO (6m)
(30) 13,8° 1625
RANGOON (5m)
(60) 27,2° 2633
SHANGHAI (12m)
(7) 15,5° 1120
TAINAN (14m)
(30) 23,0° 1679
MACAO
(26-40) 22,6° 1697
BANGKOK (2m)
(17) 28,1° 1365
SINGAPORE (3m)
(29) 27,2° 2416
JAKARTA (8m)
(63) 26,2° 1793

Fig. A-6A. Coastal Climate Diagram - Australia.

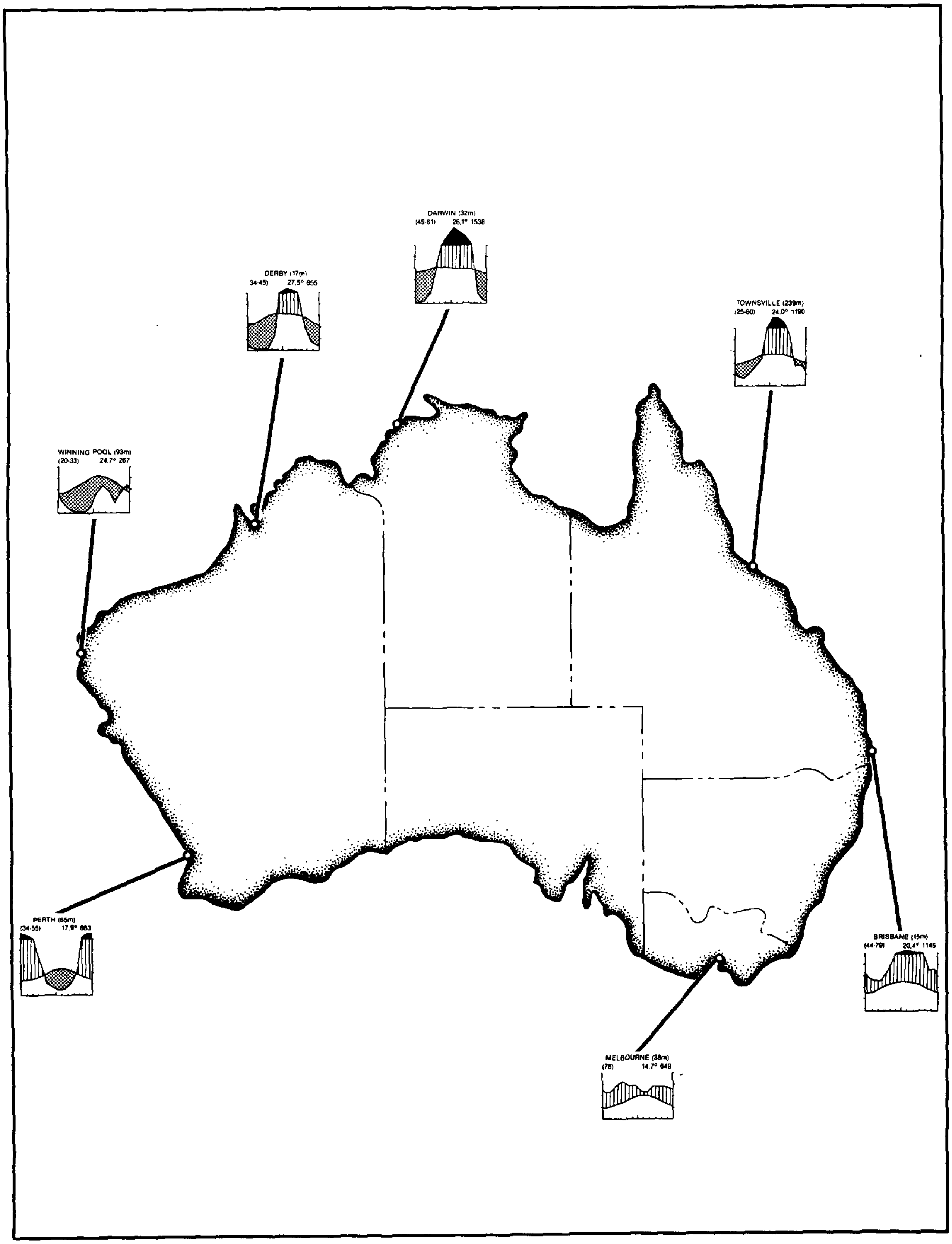


Fig. A-6B. Coastal Climate Diagram - Africa.

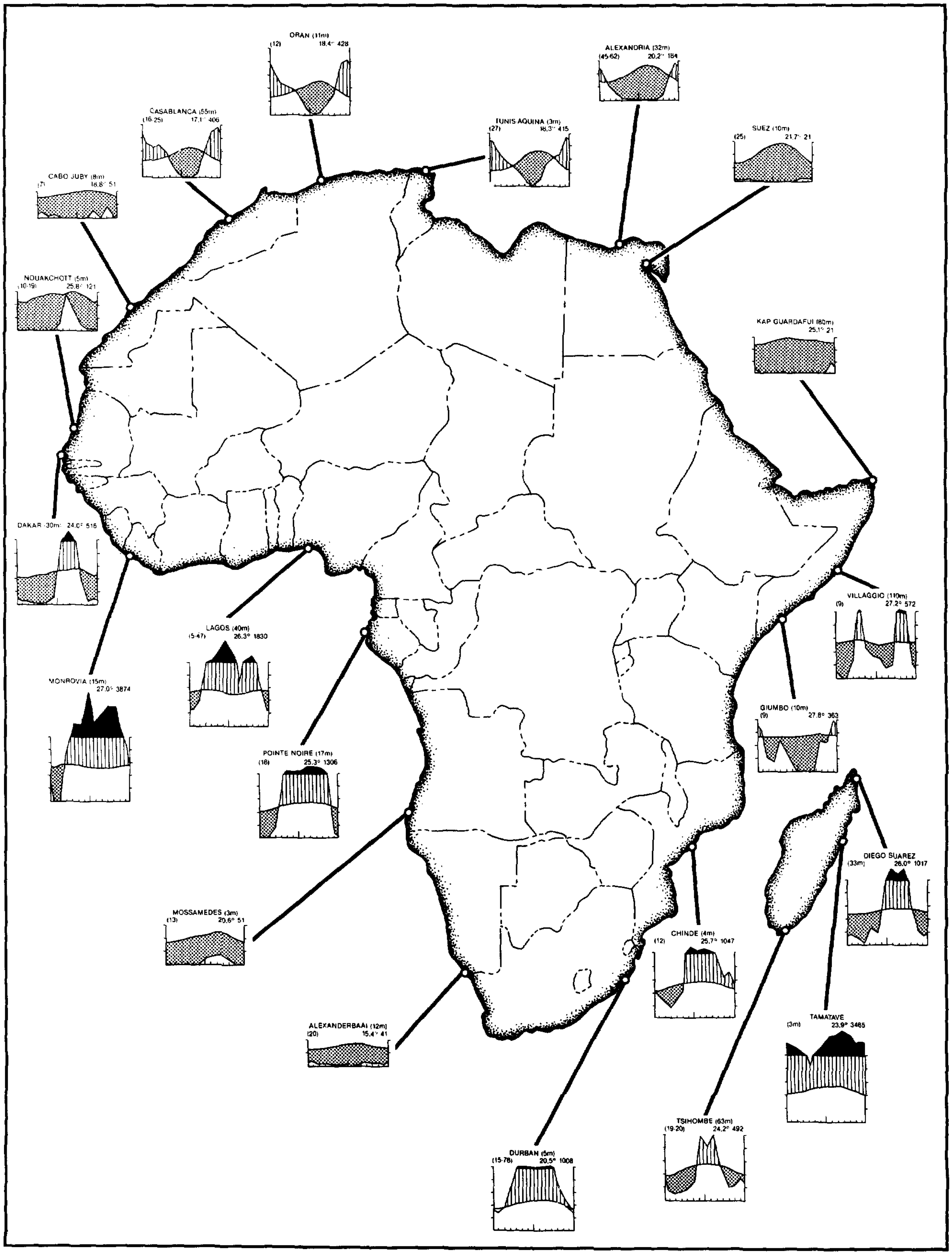
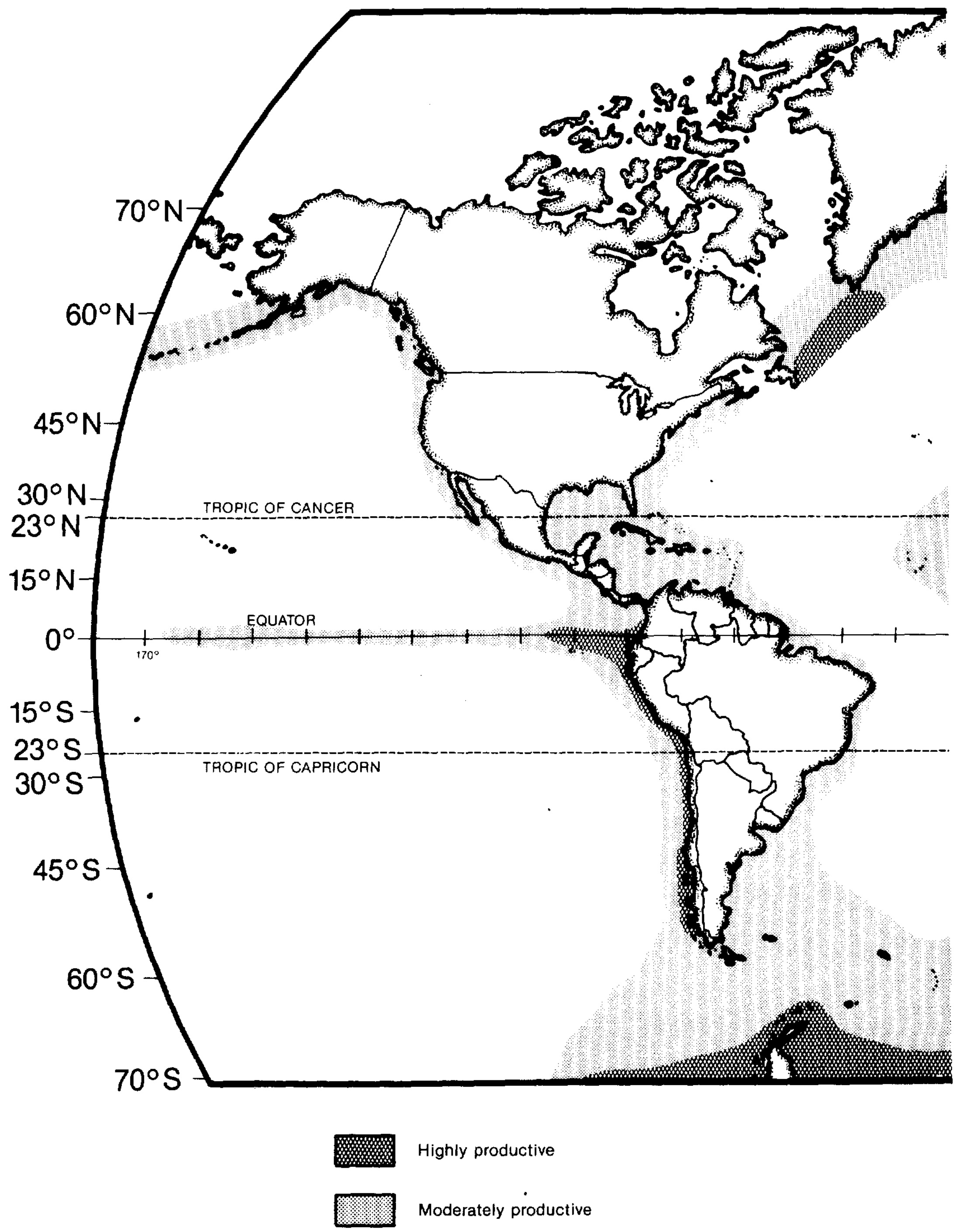


Fig. A-7. Areas of High Productivity in the World Ocean (after Gross, M.G., 1980. Oceanography. Charles E. Merrill Publishing Company, Columbus, Ohio).



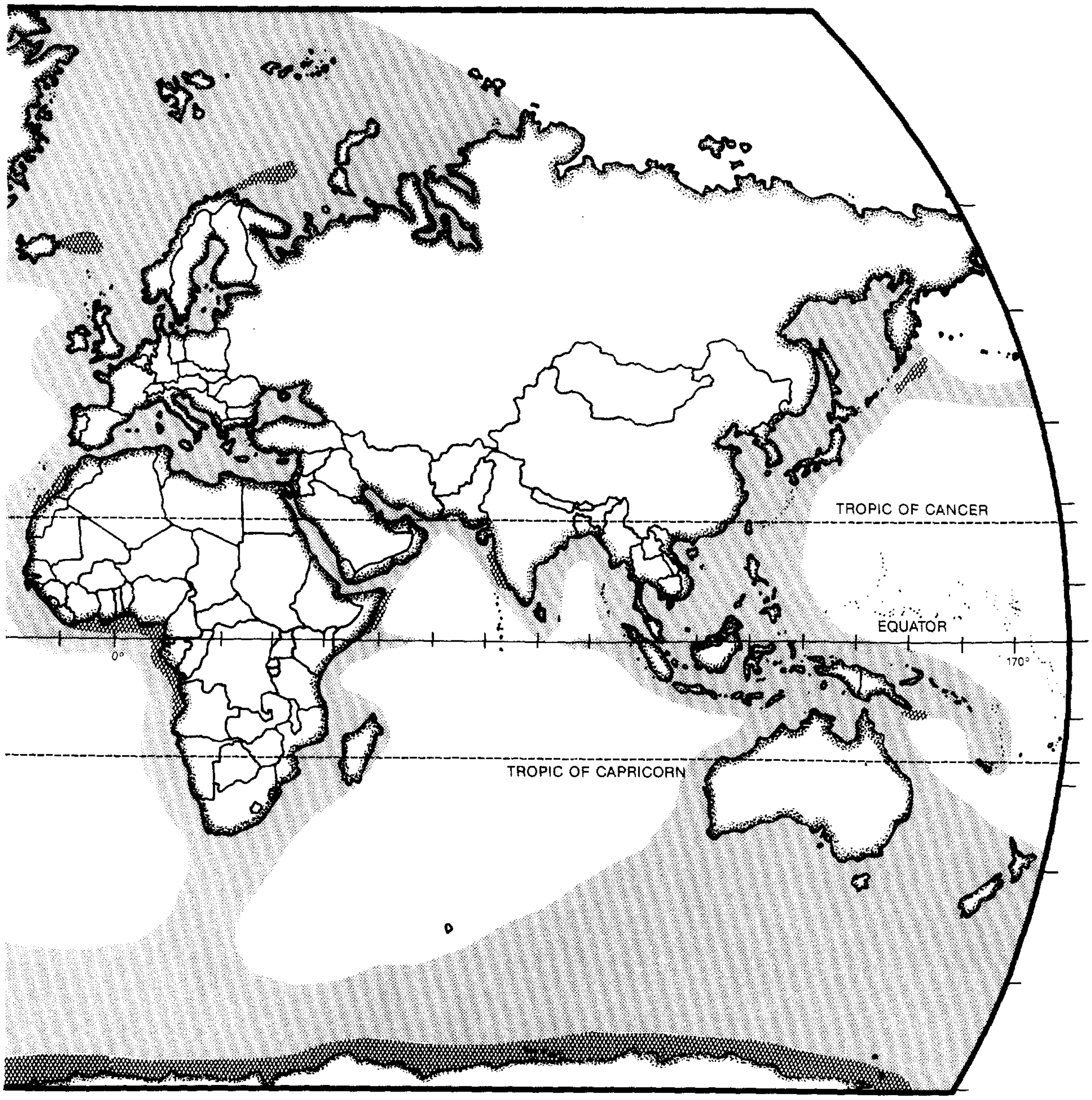
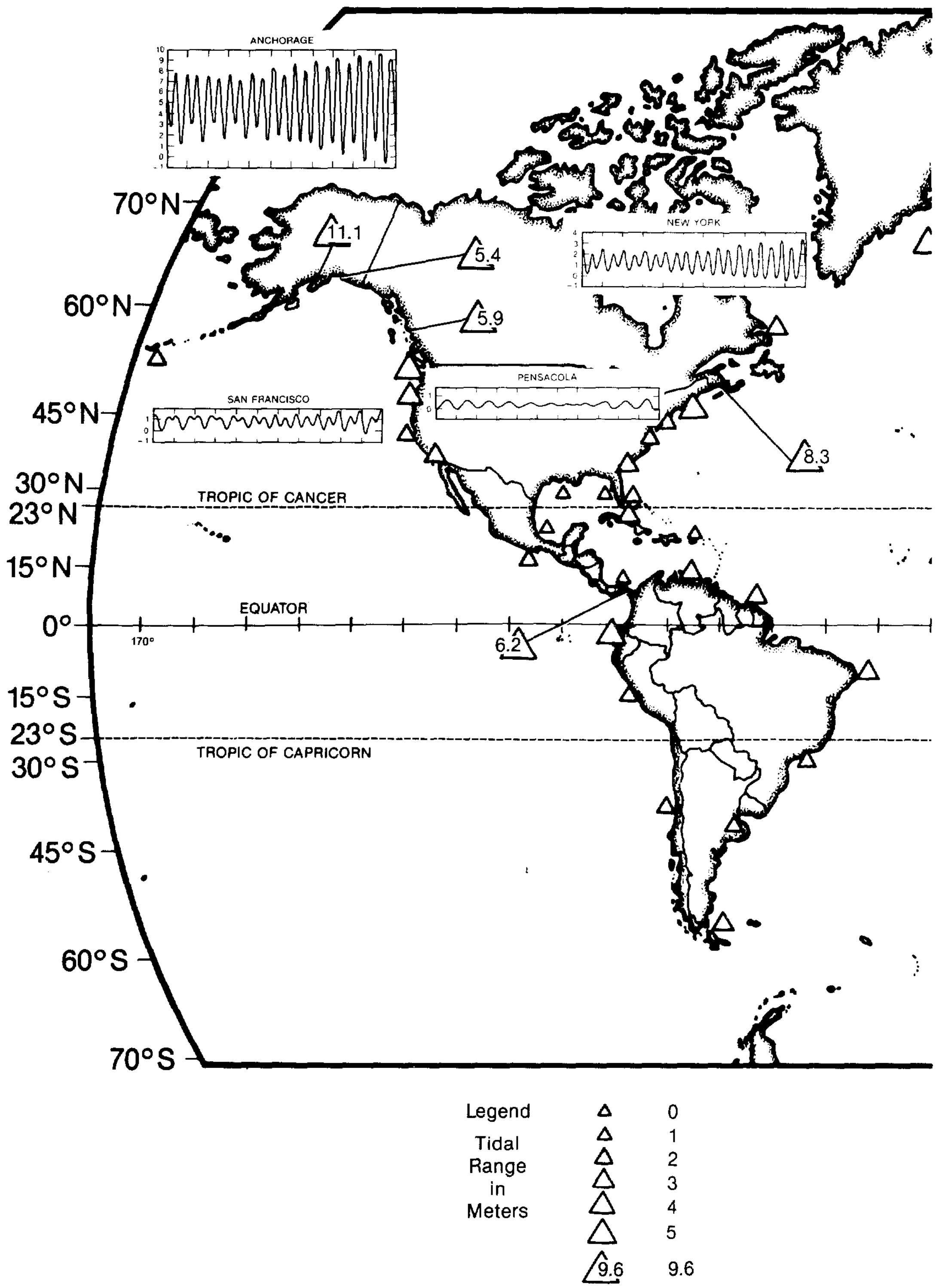

TROPIC OF CANCER
EQUATOR
0°
170°
TROPIC OF CAPRICORN

Fig. A-8. Representative Tidal Characteristics and Ranges.



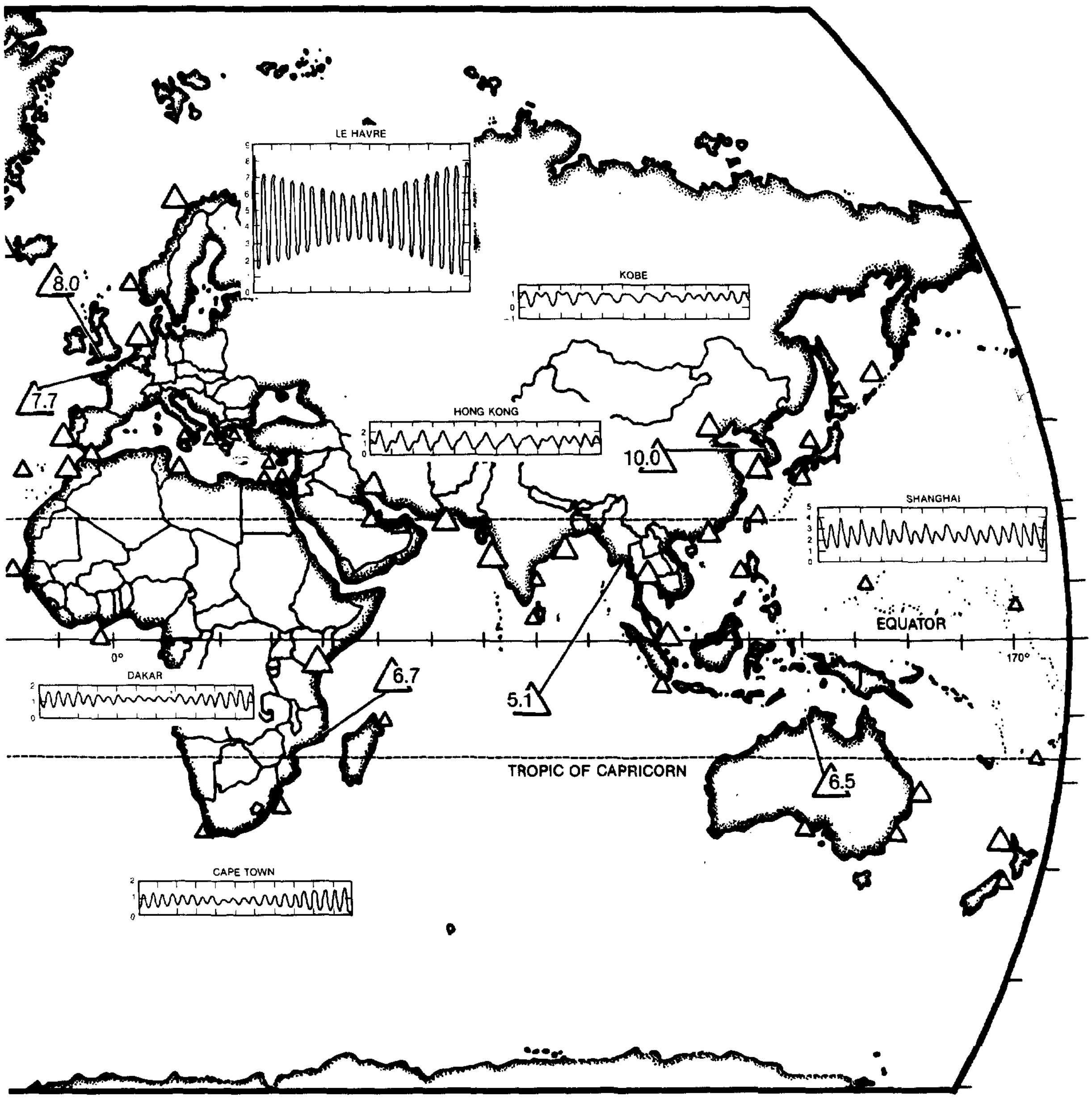

LE HAVRE
KOBE
HONG KONG
SHANGHAI
DAKAR
CAPE TOWN
8.0
7.7
10.0
6.7
5.1
6.5
EQUATOR
TROPIC OF CAPRICORN
0°
170°

TABLE A-1

Site-specific Climate Factors Relevant to Outfall System Design

Ref	Location	Total Precip mm	Months of Greatest Precip	Irri-gation Months	Frost Months	Freeze Months	Coldest Month °C	Warmest Month °C	Months <5°C	Months <10°C
UNITED STATES										
11/312	New York	1076	-	-	X-V	XII-II	0.7	24.9	4	5
/274	Los Angeles	373	XI-IV	IV-XI	-	-	13.2	22.8	0	0
/330	Brownsville	680	VI-IX	VI	-	-	16.3	28.9	0	0
/281	Miami	1520	V-X	-	XI-II	-	19.4	27.9	0	0
MEDITERRANEAN										
	Cyprus-Kyrenia	534	XI-III	nd	nd	-	12.6	27.9	0	0
	-Famagusta	403	XI-III	IV-X	XII-III	-	11.4	28.1	0	0
	-Nicosia	369	XI-III	nd	nd	-	10.2	28.9	0	0
	-Limmasol	457	XI-III	IV-X	XII-III	-	12.1	26.1	0	0
	-Morphou/Paphos	498	XI-III	III-IX	X-III	-	-	-	-	-
9/229	Turkey-Trabzon	831	IX-II	-	-	-	7.0	23.2	0	4
to 240	-Samsun	719	IX-III	VII-VIII	XI-IV	-	6.7	23.3	0	4
	-Florya (1st)	651	X-III	V-VIII	XI-V	-	5.2	23.3	0	4
	-Ankara	358	XII-VI	VI-X	IX-V	XII-III	-0.1	23.3	3	5
	-Izmir	704	XI-III	V-IX	XI-III	-	8.5	27.6	0	2
	-Antalyc	1030	XI-III	V-IX	XII-III	-	10.0	28.2	0	0
	-Adana	625	XII-III	VI-IX	XII-III	-	9.2	28.1	0	1
6/233	Greece-Saloniki	449	X-IV	VI-IX	XI-III	-	9.2	24.7	0	1
234	-Larisa	522	X-III	nd	nd	-	5.8	28.0	0	4
235	-Patras	749	X-III	nd	nd	-	9.7	26.4	0	1
235	-Athens	402	X-III	IV-IX	XI-III	-	9.3	27.6	0	2
	Iraq-Baghdad	145	XII-II	IV-XI	X-III	-	10.0	34.8	0	0
	Syria-Damascus	232	XI-III	III-X	XI-IV	-	7.4	26.8	0	2
	Iran-Tehran	165	I-IV	nd	nd	-	4.5	30.0	1	3
	Lebanon-Tripoli	955	XI-III	nd	nd	-	12.8	26.8	0	0
9/229	-Beirut	743	XI-III	IV-X	I-III	-	12.7	26.1	0	0
/-240	Jordan-Amman	272	XI-III	nd	nd	-	8.1	25.6	0	2
	-Aqaba	29	XI-V	I-XII	-	-	15.6	33.0	0	0
	Occupied-Gaza	313	XI-III	nd	nd	-	13.7	26.5	0	0
	-Jerusalem	627	XI-III	nd	nd	-	8.3	23.7	0	2
	Israel-Tel Aviv	529	XI-III	IV-X	XII-II	-	13.8	27.7	0	0
	Arabian Peninsula	37-166	-	I-XII	-	-	11.6	35.7	0	0
10/128	Egypt-Cairo	24	XII	I-XII	-	-	19	35	0	0
	-Alexandria	143	XII-I	II-XI	-	-	18	31	0	0
60	Libya-Tripoli	286	XI-II	III-X	-	-	17	31	0	0
74	Tunisia-Tunis	461	IX-III	IV-X	-	-	15	32	0	0
73	Algeria-Algiers	641	IX-IV	V-VIII	-	-	15	29	0	0
69	Morocco-Tangier	895	IX-IV	VI-VIII	-	-	16	29	0	0
64	-Rabat	496	X-III	V-IX	-	-	17	27	0	0
62	-Casablanca	511	X-IV	V-IX	I	-	17	26	0	0
116	Mauritania-Nouackchott	156	VII-IX	I-XII	-	-	29	34	0	0
6/174	Italy-Venice (Traviso)	1056	I-XII	-	-	-	2.5	23.4	3	5
179	-Rome	881	IX-VI	VI-VII	XI-III	-	6.9	24.7	0	3
176	-Genoa	1294	IX-III	nd	nd	-	7.9	24.1	0	3
5/193	France-Marseilles	546	IX-V	VI-VIII	X-IV	-	5.5	23.3	0	3
230	Spain-Barcelona	594	I-XII	IV-IX	-	-	9.4	24.4	0	2
-	Gibraltor-see Tangier									
235	Portugal-Lisbon	708	X-IV	V-IX	XII-II	-	10.8	22.5	0	0
231	-Oporto	1150	X-III	VI-VIII	XI-III	-	9.0	19.8	0	3
102	UK-London (Kew)	594	I-III	-	X-VI	-	4.2	17.6	2	6
BLACK SEA										
7/370	USSR-Batumi	2508	V-III	-	-	-	6.4	23.2	0	6
397	-Novorassysk	688	I-XII	V-IX	-	-	2.5	23.7	3	4
398	-Odessa	389	I-XII	IV-X	IX-IV	XII-II	-2.8	22.1	5	6
6/227	Romania-Constanta	413	I-XII	V-X	nd	nd	-0.8	22.2	4	6
6/229	Bulgaria Varna	474	I-XII	VII-IX	nd	-	1.2	22.9	4	6
SOUTH ASIA										
9/298	Pakistan-Karachi	196	VI-IX	VII-V	-	-	19.2	30.2	0	0
323	India-New Delhi	714	VI-IX	X-V	I	-	14.3	34.3	0	0
311	-Bombay	549	VI-X	X-V	-	-	19.7	31.3	0	0
315	-Madras	1215	VII-XII	I-V	-	-	24.5	32.7	0	0
305	-Calcutta	1582	V-X	X-IV	-	-	20.2	31.1	0	0

(Cont.)

TABLE A-1 (cont.)

AFRICA										
R/372	Somalia-Mogadishu	422	IV-VI	XII-III	-	-	26	29	0	0
			X-XI	VII-VIII						
10/341	Kenya-Mombasa	1191	IV-VI	I-II	-	-	29	33	0	0
			X							
344	Tanzania-Dar-es-Salaam	1179	III-V	VII-IX	-	-	29	31	0	0
			XII							
405	Mozambique-Beira	1429	XI-IV	V-IX	-	-	25	32	0	0
578	So Africa-Durban	1003	XI-III	VII	-	-	22	28	0	0
584	-Capetown	506	IV-IX	X-III	-	-	17	26	0	0
556	Angola-Luanda	367	III-IV	V-II	-	-	24	31	0	0
301	Cameroon-Douala	4150	III-XI	I	-	-	27	32	0	0
242	Ghana-Accra	787	V-VI	VII-VIII	-	-	27	32	0	0
				XII-II						
240	Ivory Coast-Abidjan	2144	III-VI	VII-VIII	-	-	28	32	0	0
			X-XII	I-II						
309	Sierra Leone-Freetown	2530	II-XI	XII-III	-	-	28	34	0	0
218	Senegal-Dakar	540	VII-IX	X-VI	-	-	26	32	0	0
EAST AND SOUTHEAST ASIA										
7/422	USSR-Vladivostok	721	VI-IX	-	nd	XI-III	-14.7	20.0	4	8
8/ 81	N. Korea-Wonsan	1308	VI-IX	-	nd	nd	-3.8	23.4	4	6
92	S. Korea-Pusan	1381	IV-IX	-	nd	-	1.8	25.4	2	4
87	-Seoul	1259	VI-IX	-	nd	nd	-4.9	25.4	4	5
82	China-Tienjin	527	VI-IX	-	X-IV	XII-II	-4.1	26.8	5	5
97	-Shanghai	1143	V-IX	-	-	-	3.4	27.2	2	4
114	-Guangzhon	1722	IV-VIII	-	-	-	13.6	28.8	0	0
R/292	Hong Kong	2265	IV-IX	XII-I	I	-	15	28	0	0
9/ 53	Thailand-Bangkok	1492	V-X	XII-III	-	-	25.4	30.3	0	0
9/ 66	Singapore	2430	I-XII	-	-	-	25.6	27.6	0	0
R/305	Indonesia-Jakarta	1755	XII-III	-	-	-	26	27	0	0
R/294	Burma-Rangoon	2618	V-IX	I-II	-	-	32	39	0	0
304	Bangladesh-Chittagong	2734	V-IX	XII	-	-	19.4	27.9	0	0
SOUTH AMERICA										
12/209	Ecuador-Guayacil	843	I-IV	VI-XII	-	-	29	31	0	0
212	Peru-Lima	10	I-I	I-XII	-	-	18	26	0	0
			VI-IX							
132	Chile-Arica	0.7	-	I-XII	-	-	15.8	22.3	0	0
138	-Valparaiso	459	V-VII	XI-IV	-	-	11.8	18.0	0	0
139	-Santiago	356	V-VIII	nd	nd	-	8.1	20.0	0	3
141	-Valdivia	248o	IV-IX	-	-	-	7.7	17.0	0	5
145	-Punte Arenas	448	I-XII	-	-	-	2.5	11.7	5	9
407	Columbia-Baranquilla	356	V-IX	II	-	-	8.1	20	0	3
383	Venezuela-Caracas	835	V-XI	I-III	-	-	19.2	22.0	0	0
390	Guyana-Georgetown	2420	XII-VIII	-	-	-	26.3	27.7	0	0
392	Surinam-Paramibo	2208	XI-VIII	-	-	-	26.4	28.5	0	0
395	Fr. Guiana-Cayenne	3744	XII-VI	IX-X	-	-	25.1	26.2	0	0
273	Brazil-Belem	2732	XII-VI	IX-XI	-	-	25.0	26.3	0	0
277	-Recife	1501	III-VI	X-XII	-	-	24.3	27.1	0	0
283	-Salvador	1912	III-VII	-	-	-	23.0	26.3	0	0
291	-Rio de Janeiro	1093	XII-IV	-	-	-	22.2	25.6	0	0
293	-Porto Alegre	1297	IV-X	-	-	-	14.3	24.7	0	0
111	Uruguay-Montevideo	1012	-	-	-	-	10.5	22.5	0	0
91	Argentina-Buenos Aires	1027	VIII-IV	-	-	-	10.5	23.7	0	0
96	-Mar del Plata	768	-	-	-	-	7.7	19.3	0	4
R/549	-Bahia Blanca	558	X-VI	-(a)	-	-	8	23	0	3
CENTRAL AMERICA-CARIBBEAN										
R/486	Mexico-Mexico City	726	VII-IX	XI-IV	-	-	12	17	0	0
11/400	-Mazatlan	805	VII-IX	XI-VI	-	-	19.8	28.1	0	0
11/392	-Acapulco	1379	VI-X	XI-V	-	-	26.5	28.8	0	0
R/486	-Vera Cruz	1672	VI-X	II-IV	-	-	28	33	0	0
11/396	-Merida	929	VI-IX	XI-IV	-	-	23.0	27.8	0	0
12/470	Belize-Belize	1895	VI-XII	III	-	-	23.3	27.2	0	0
466	Guatemala-Guatemala	1281	V-X	XI-IV	-	-	16.3	19.6	0	0
478	Panama-Colon	3424	V-XII	-	-	-	26.1	27.2	0	0
R/495	Cuba-Havana	1167	V-XI	-	-	-	21	27	0	0
496	Haiti-Port au Prince	1163	IV-X	XI-II	-	-	25	29	0	0
496	Dominican Rep-St Dmgo	1386	V-XI	-	-	-	24	27	0	0
493	Puerto Rico-San Juan	1631	V-I	-	-	-	24	27	0	0
494	Trinidad-Piarco	1772	V-XII	II-III	-	-	24	27	0	0

(a) year-round irrigation 42 to 50^{o}S

Sources:

R/page number - Rudloff, W., World Climates, Wissenschaftlich Verlag gesellschaft Gmbh, Stuttgart, 1981, 632 pp.

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TABLE A-2

Terrestrial Characteristics of Climate Zones

		Warmest Month °C	Coldest Month °C	Annual Monthly Range °C	Diurnal Range °C	Number of Humid Months	Remarks: Representative Locality	Remarks: Vegetation
I.	Polar Zone							
	1. Ice cap/ice desert						Arctic/Antartic	--
	2. Frost-debris belt	<6					Novaya Zemla, USSR	--
	3. Tundra	10	<-8				Nome, Alaska	--
	4. Oceanic sub-polar	5-12	-8 to 2	22	0.6-1.8		McMurdo Sound, Antarctica	Tussock grassland moors
II.	Boreal Zone							
	1. Oceanic climate	10-15	-3 to 2	13-19			Aleutian Islands	Humid coniferous woods
	2. Continental climate	10-20		20-40			Mcakenzie Valley, Canada	Continental coniferous woods
	3. Highly continental	10-20	<-25	>40			Newfoundland, Canada	Dry coniferous woods
III.	Cold Temperature Zone Woodland Climate							
	1. Highly oceanic	<15	2 to 10	<10			Olympic Peninsula, WA, USA	Evergreen broad-leaved woods
	2. Oceanic	≤20	>2	<10	4		England	Oceanic deciduous woods
	3. Sub-oceanic		-3 to 2	16 to 25	2 to 9		Central Europe	Oceanic deciduous woods
	4. Sub-continental	≤20	-13 to -3	20-30			Montreal, Canada	Sub-continental deciduous woods
	5. Continental	15-20	-20 to -10	30-40	≤12		Minneapolis, USA	Wooded steppe
	6. Highly continental	>20	-20 to -10	>40	>12		Trkutsk, Canada	Wooded steppe
	7. Humid summer	20-26	-8 to 0	25-30			Chicago, USA	Cold-tolerant wooded steppe
	7a. Arid summer	20-26	-6 to 2	25-30			Kayseri, Turkey	Thermophile dry woods
	8. Permanently humid	20-26	-6 to 2	20-30			Vladivostok, USSR	Humid broadleaved woods
	Steppe Climate							
	9. Humid-cold		<0				Odessa, USSR	High grass steppe perennial herbs
	9a. Humid-mild		>0				Baku, USSR	
	10. Arid summer		<0				Central Anatolia, Turkey	Short grass, dwarf shrub
	10a. Dry		0-6				Tierra del-Fuego, Argentina	Dwarf shrub, thorns
	11. Humid summer		<0				Llahsa, Tibet	--
	12. Desert-cold winters		<0				Atrakhan, USSR	--
	12a. Desert-mild winters		0-6				Western Patagonia	--
IV.	Sub-Tropical							
	1. Mediterranean		2 to 13			>5	Athens, Greece	Sub-tropical hardwoods
	2. Dry summer steppe		2 to 13			<5	Baghdad, Iraq	Sub-tropical grass and shrub
	3. Dry winter steppe		2 to 13			>5	Monterey, Mexico	Sub-tropical thorn, succulents
	4. Dry winter		2 to 13	22	12 to 14	6 to 9	El Paso, USA	Hard-leaved monsoon wood
	5. Desert		2 to 13			<2	Sahara Desert	
	6. Permanently humid		2 to 13			10 to 12	Montevideo, Uruay	Sub-tropical grasses
	7. Permanently humid		2 to 13			10 to 12	Eastern China	Laurel and coniferous forests
V.	Tropical Zone							
	1. Tropical rainy			1.5	6 to 13	9-1/2 to 12	Amazon Basin, Brazil	Rain forest
	2. Summer humid					7 to 9-1/2	Calcutta, India	Humid forest and savannah
	2a. Winter humid					7 to 9-1/2	Recife, Brazil	Half deciduous woods
	3. Wet and dry			11	5 to 15	4-1/2 to 7	Poona, India	Dry wood and savannah
	4. Dry					2 to 4-1/2	Ahmedahad, India	Tropical thorn and succulents
	5. Desert					<2	Guayacil, Ecuador	--
IV/V.	Seasonally Humid coastal desert						Lima, Peru	

Source: Landsberg, H.E., Lippman, H., Paffen, K.H., 1966. Weltkarten zur Klimakunds, Springer-Verlag, Berlin.

APPENDIX B

EXAMPLE OF DESIGN CRITERIA CHECK LIST

Preliminary design criteria serve as a checklist of information to be obtained or resolved before detailed deign can proceed. This information is normally obtained from the client, local government agencies, local marine operators and contractors, interviews with individuals knowledgable of the local marine environment, and by direct field observation.

The following pages present an example of a preliminary design criteria checklist. Since outfall pipelines are used to dispose of chemical wastes and slurries, some of the items listed are applicable to waste products other than sewage. Final design criteria, which contain exact values are prepared for the detailed design.

1. GENERAL

1.1 Purpose of Project:
1.2 Location of Project:
1.3 Scope of Project:

2. SCOPE OF GOVERNMENTAL REGULATIONS

2.1 Political jurisdictions and boundaries to be considered:

2.2 International ecological requirements and restrictions:

2.2.1 Offshore:
2.2.2 Onshore:

2.3 National ecological requirements and restrictions:

2.3.1 Offshore:
2.3.2 Onshore:

2.4 Local ecological requirements and restrictions:

2.4.1 Offshore:
2.4.2 Onshore:

2.5 Permits required:

2.5.1 Offshore:
2.5.2 Onshore:

NOTE: The above information is required to establish regulatory constraints on the design.

3. CLIENT STIPULATIONS

3.1 Specified design codes:
3.2 Client design standards:
3.3 Construction limitations and safety constraints:

4. REMARKS

5. PIPELINE PARAMETERS

5.1 Pipe Diameter

5.2 Pipe Materials to Consider: (yes or no) Pipe Manning Number "n"

Steel
Cast Iron
Concrete
Plastic
Other(list)

5.3 Pipe Coatings (if required)

5.3.1 Corrosion resistant coating types acceptable
5.3.2 Weight coatings to be considered:
5.3.3 Is internal lining required (yes or no)

5.4 Cathodic protection (yes or no)

Rectifier groundbeds
Bracelet-type anodes

5.5 Waste sources, quantities, and qualities (sewage, chemical wastes (etc.)

5.5.1 Sources
5.5.2 Temperature
5.5.3 Specific gravity
5.5.4 Viscosity
5.5.5 Design flow rate

Daily average
Daily maximum
Maximum design flow

5.6 Basis for effluent flow (measurements, projections, etc.) and possible ranges

5.7 Elevations (state datum)

Inlet (soffit)
Outlet

5.8 Critical flow velocities

Maximum allowable
Minimum allowable

5.9 In-line equipment requirements and restrictions (describe)

Pumps
Comminutors
Bar screens
Grit chambers
Flow measuring devices
Diffusers

NOTE: Above data are used to verify hydraulic calculations, to design coatings, and to select a compatible construction method and materials.

6. ROUTE SELECTION

6.1 Where will the line begin onshore? (Attach sketch)

6.2 What is expected total length of line?

6.3 What is required in route survey? (yes or no)

Precision Depth Finder Profile
Side Scan Sonar Profile
Channel Location Identified
Route Obstruction Identification
Surf Zone Identification
Subbottom Profile
(ATTACH APPROPRIATE MAPS)

6.4 What is correct designation of nautical charts covering area of concern?

6.5 General bottom conditions (smooth, rough, boulders, etc.)

6.6 Obstructions along or near pipeline route

7. RIGHT OF WAY (R-O-W)

7.1 Ownership

7.2 Expected problems in obtaining R-O-W

7.3 What is anticipated onshore width of R-O-W?

During Construction
Permanent Easement

7.4 What is anticipated offshore width of R-O-W?

During Construction
Permanent Easement

8. POTENTIAL FOR ONSHORE OR OFFSHORE DAMAGE TO LOCAL PROPERTY DURING CONSTRUCTION:

8.1 Onshore/Offshore aesthetic requirements (appearance, odors, etc.)

8.2 Nearby aesthetic distractions

9. METEOROLOGICAL DATA

9.1 Design Wind Speed
9.2 Average and ranges of precipitation
9.3 Peak rainfall per unit time
9.4 Monthly average and ranges of temperature
9.5 Monthly average and ranges of winds from critical directions
9.6 Design wind direction and fetch
9.7 Diurnal wind variations
9.8 General weather description (summer vs. winter, prevailing winds, storms, etc.)

NOTE: Wind information is to be used for evaluating wave generation and criteria constructing constraints as well as outfall operation. If possible, provide detailed wind and weather history over past 20 years from government weather sources. Rainfall data are necessary for small estuaries with large drainage basins.

10. WAVE AND CURRENT

10.1 Tides (state datum)
10.2 Mean Sea Level
10.3 Mean Higher High Water by Month
10.4 Highest High Water
10.5 Chart Datum
10.6 Tidal Currents

Direction: Flood Ebb
Number per day
Slack tide lasts to minutes

10.7 For Pipeline Installation: (Probability of occurrence or return period)

10.7.1 Design wind driven wave (height and period)
10.7.2 Design swell (height and period)
10.7.3 Design total wave (height and period)
10.7.4 Design tidal current velocity
10.7.5 Design water particle velocity
10.7.6 Design particle acceleration
10.7.7 Breakers and surf (describe)

10.8 For Pipeline Operation: (Probability of occurrence or return period)

10.8.1 Design wind driven wave (height and period)
10.8.2 Design swell (height and period)
10.8.3 Design total wave (height and period, including storm surge)
10.8.4 Design tidal current velocity
10.8.5 Design water particle velocity
10.8.6 Design particle acceleration
10.8.7 Water temperature range (°C) to
10.8.8 Bathymetry of nearshore/surf zone area

11. GEOTECHNICAL DATA

11.1 Soil classification
11.2 Design cohesive strength
11.3 Design angle of internal friction
11.4 Design shear strength
11.5 Design unconfined comprehensive strength
11.6 Design Atterburg limits (clays only)--Attach curves
11.7 Density vs water content--Attach curves
11.8 Design bulk density
11.9 Design in-situ water content
11.10 Rock classification
11.11 Rock hardness

12. SCOUR

12.1 History
12.2 Potential
12.3 Probability of occurrence

13. SEDIMENT TRANSPORT

13.1 History
13.2 Probability of occurrence
13.3 Possible effects on pipeline

14. ROCKSLIDE POTENTIAL

14.1 History
14.2 Probability of occurrence
14.3 Possible effects on pipeline

15. MUDSLIDE POTENTIAL

15.1 History
15.2 Probability of occurrence
15.3 Possible effects on pipeline

16. SEISMIC FACTORS

16.1 Seismic history
16.2 Probability of occurrence of earthquake
16.3 Possible effects on pipeline
16.4 Design acceleration

17. ON-BOTTOM STABILITY DURING INSTALLATION (VERTICAL AND LATERAL)

17.1 Design nearbottom water particle velocity
17.2 Seafloor conditions (coral, mud and sand)
17.3 Surf zone
17.4 Offshore
17.5 Design parameters to be used
17.5.1 Reynolds number (pipe w/concrete coating)
17.5.2 Lift coefficient
17.5.3 Drag coefficient
17.5.4 Lateral friction factor
17.5.5 Soil bearing capacity
17.5.6 Estimated unit weight of concrete

18. ON-BOTTOM STABILITY DURING OPERATING LIFE (VERTICAL AND LATERAL)

18.1 Design water particle velocity - near-bottom
18.2 Sea floor conditions (coral, mud, and sand)
18.3 Surf zone
18.4 Offshore
18.5 Design parameters to be used
18.5.1 Reynolds number (pipe w/concrete coating)
18.5.2 Lift coefficient
18.5.3 Drag coefficient
18.5.4 Lateral friction factor
18.5.5 Soil bearing capacity
18.5.6 Estimated unit weight of concrete
18.5.7 Design line unit weight (full or empty)

19. TRENCHING REQUIREMENTS

19.1 What is required depth of burial at the shoreline?
19.2 Is burial required by client or regulatory offshore?
19.2.1 To what depth?
19.3 Will rock blasting be required?
19.3.1 Length and depth of trench
19.3.2 Depth of water(attach maps and charts)

20. CONSTRUCTION CONSIDERATIONS

20.1 What construction methods can be used?
20.1.1 Pull method
20.1.2 Lay barge
20.1.3 Float & sink (1 section)
20.1.4 On-bottom joining of short sections
20.1.5 Other--list

21. WHAT SITE CONDITIONS WILL LIMIT OR RESTRICT CONSTRUCTION?

21.1 Onshore
21.2 Surf zone
21.3 Offshore

22. CONSTRUCTION AREA REQUIRED

23. WHAT IS AVAILABILITY OF CONSTRUCTION EQUIPMENT?

23.1 Barges
23.2 Winches
23.3 Anchors
23.4 Dredges
23.5 Onshore trenching
23.6 Dozers
23.7 Cranes
23.8 Coating yards

APPENDIX C

THE ARITHMETIC OF POWER SPECTRUM ANALYSIS

This material is based on T.A. Wastler's primer ("Application of Power Spectrum Analysis to Stream and Estuary Field Surveys," Publ. 999-WP-7, U.S. Public Health Service, Washington, D.C., 1963) of the mathematical basis for, and application of, spectral analysis. Power spectrum analysis is one method for time series analysis. The term is frequently shortened to spectral analysis. Spectral analysis provides measures of the variance of a record. Variance is expressed as the mean square or, following usage, the power of the units being measured: hence, "power spectrum analysis."

In recent years, the analysis of sequential data has been greatly facilitated by the use of power spectrum analysis. The application of this tool to geophysical problems was first described by J.W. Tukey in 1949 ("The sampling theory of power spectrum analysis," Symposium on Application of Autocorrelation Analysis to Physical Problems, Woods Hole, Massachusetts, Office of Naval Research, U.S. Navy, Washington, D.C., pp. 47-67). Originally developed for analyzing noise in communications systems, spectral analysis has since been applied to a variety of problems in meteorology, oceanography, and engineering where the basic data were arranged in time series. The most obvious benefit of this application, paraphrasing W.H. Munk ("Long Ocean Waves," in M.N. Hill, ed., The Sea. Interscience, 1962, pp. 647-663), has been the condensation of "miles of wiggly curves in the time domain to a few simple traces in the frequency domain."

One feature of the method (the number of lags) will be referred to repeatedly. Lags are defined explicitly. Their importance lies in the fact that total computation effort time is a linear function of the record length (number of data points) multiplied by the number of lags. The reason for using larger numbers of lags is to improve the resolution of the calculation. This means that the frequencies (or periods) of cyclic events in the record can be more precisely defined and separated. For example, it becomes possible to measure and compare the effects of the semidiurnal tide with the solar day on dissolved oxygen in estuaries.

The dependence of computation effort on the record length and the number of lags needed to unscramble the record suggests that when more data are collected, more work is needed to examine them. In this respect, power spectrum analysis is similar to simple averaging. It takes more effort to determine the average of many numbers than for a few numbers.

A time series is a record of repeated observations made at a particular location. Each observation is a momentary summation of the effects of everything happening to the particular parameter. These effects may be caused by cyclic or random phenomena. A trend throughout a given record length may be real or apparent; for example, a short segment of a sine wave will appear as a trend.

Power spectrum analysis identifies the frequencies at which different factors cause the record to vary. The analysis also provides estimates of the variances that derive from each of these factors. Figure C-1 shows some simple spectra (more precisely, estimates of spectral density) from several types of curves. Where the original curve is made up of more than one factor, as in Cases V and VI, the power spectrum clearly reveals the nature of the components.

Four steps in the computation of individual power spectra are as follows:

1. The mean and the square of the mean of the record are determined.

2. The autocorrelation function of the record is formed. This is the basic operation in spectral analysis and is described in detail later.

3. The Fourier cosine transform for each autocorrelation is computed. This defines the difference between spectral analysis and standard Fourier analysis because in the latter, the Fourier transform is applied to the raw record rather than to the autocorrelation function. In spectral analysis, the Fourier transform serves to smooth out some of the fluctuations present in the autocorrelation function.

4. A second weighting operation provides the estimate of spectral density.

The Autocorrelation Function

Wastler's description is essentially as follows: the autocorrelation function is obtained by first multiplying each number in the record by another number in the record. From the mean of the sum of these products is subtracted the square of the arithmetic mean of the entire series. In determining the autocorrelation at lag 0, the record is multiplied by itself so that the result is the variance as defined in statistics. The autocorrelations at lag 0, lag 1, lag 2, and lag 3 are computed as shown in Table C-1.

The entire operation may be expressed mathematically as

$$\text{(C-1)} \qquad C_r = \frac{1}{(n-r)} \sum_{1}^{n-r} x_t x_{t+r} - \left[\frac{1}{n} \sum_{1}^{n} x_t\right]^2$$

in which C_r = Autocorrelation at lag r

x_t = Record value at t

t = 0,1,2,...

n = Sequential index of values

r = 0,1,2,...

m = Lag number

m = Total number of lags

Fig. C-1. Spectral Arithmetic. Source: Gunnerson (1975).

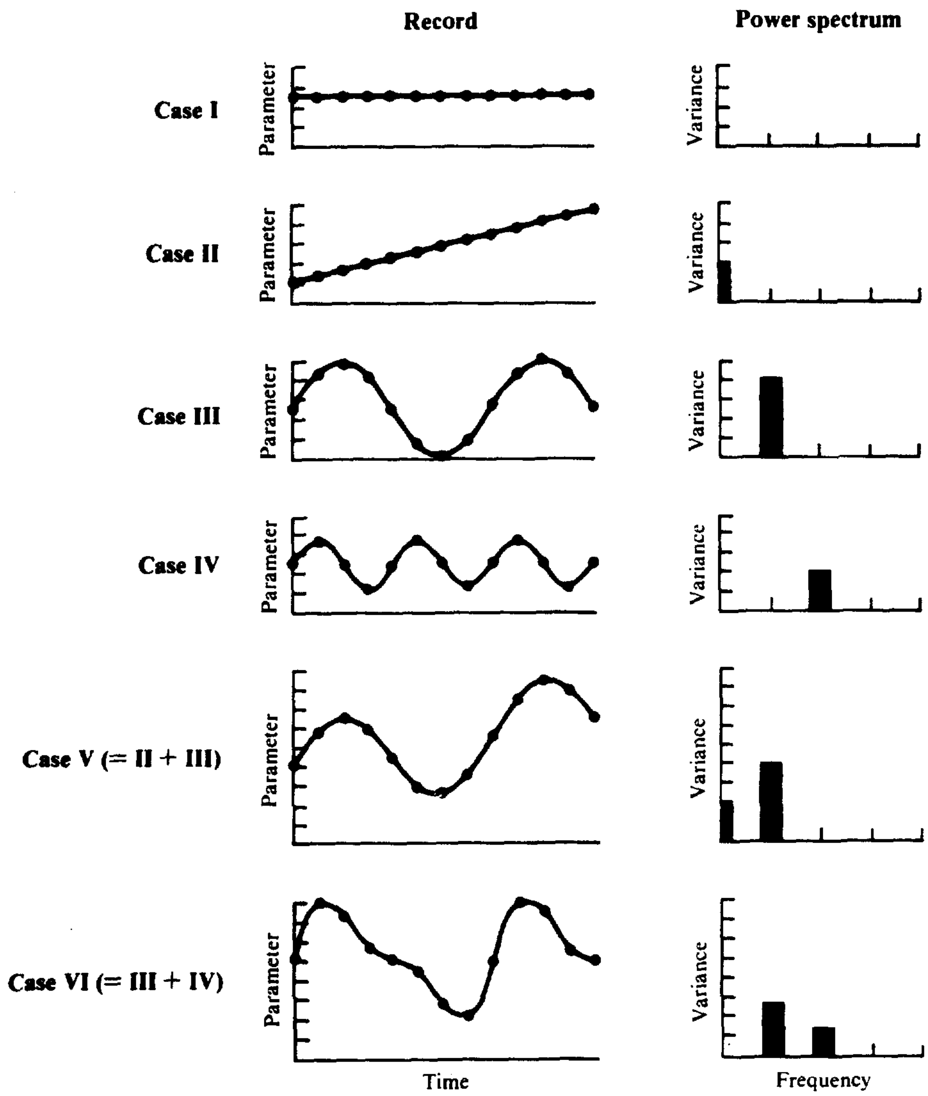


TABLE C-1

Typical Computation of Autocorrelation Function[a]

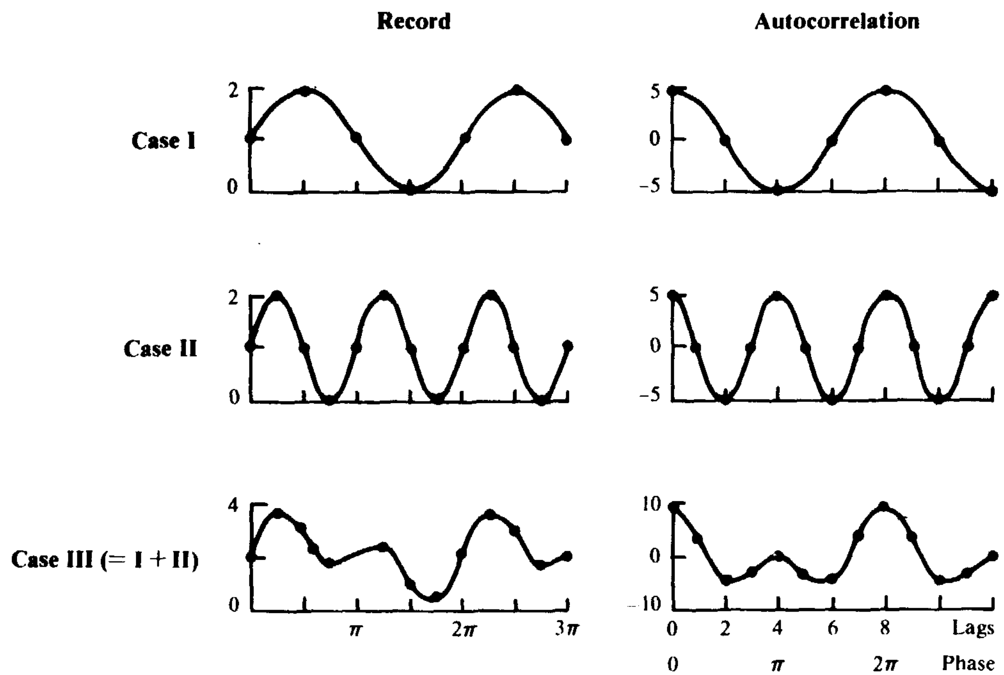
C = Autocorrelation at Lag 0	C = Autocorrelation at Lag 1	C = Autocorrelation at Lag 2
1.30 x 1.30 = 1.69	1.30 x 2.57 = 3.34	1.30 x 3.79 = 4.93
2.57 x 2.57 = 6.60	2.57 x 3.79 = 9.74	2.57 x 1.49 = 3.83
3.79 x 3.79 = 14.36	3.79 x 1.49 = 5.65	3.79 x 2.30 = 8.72
1.49 x 1.49 = 2.22	1.49 x 2.30 = 3.43	1.49 x 4.73 = 7.05
2.30 x 2.30 = 5.29	2.30 x 4.73 = 10.88	2.30 x ... ...
4.73 x 4.73 = 22.37	4.73 x	... ...
... ...	... ...	... ...
... ...	... ...	... ...
... ...	... ...	... ...
... ...	... ...	... ...
3.10 x 3.10 = 9.61	3.10 x 1.46 = 4.53	3.10 x 3.16 = 9.80
1.46 x 1.46 = 2.13	1.46 x 3.16 = 4.61	1.46 x 3.30 = 4.82
3.16 x 3.16 = 9.99	3.16 x 3.30 = 10.43	3.16 x 1.41 = 4.46
3.30 x 3.30 = 10.89	3.30 x 1.41 = 4.65	3.30 x 2.35 = 7.76
1.41 x 1.41 = 1.99	1.41 x 2.35 = 3.31	1.41 ...
2.34 x 2.34 = 5.82	2.35	2.35 ... ...
Sum = 1046.9	Sum = 804.67	Sum = 764.62
$\frac{1046.9}{145} = 7.22$	$\frac{804.67}{144} = 5.588$	$\frac{764.62}{143} = 5.347$
−5.90	−5.90	−5.90
$C_0 = 1.32$	$C_1 = -0.312$	$C_2 = -0.553$

[a]Number of observations = 145; mean of observed values = $\sqrt{5.90}$.

In the simple case of a pure sine wave, the computation is physically analogous to looking at a white picket fence through one of a series of vertical gratings. Both the fence and the gratings are constructed so that the bar width always equals the slot width. The pickets are a square-wave approximation of the sine wave. The resolution of detail with which the fence may be seen is a function of the spacing between grates. Similarly, the effective resolution (R) of the sine wave is a function of the number of lags (m) used for computing the autocorrelation function and of the sampling interval (T), where $R = 1/2mT$.

If the spacing of the grates is too large, some of the pickets will not be seen. The total amount of light seen by the viewer will be less than

Fig. C-2. Autocorrelation Functions of Simple Sine Waves. <u>Source</u>: Gunnerson (1975).



the amount reflected by the fence. When the grating with the optimum spacing is selected, the fence can be described precisely. This also happens when the optimum number of lags is applied to the sine wave record. As the grating spacing is decreased, all of the pickets are still seen, but some light is again lost to the viewer. Soon, only half the reflected light is available, and, eventually, the visual resolution of the fence is destroyed entirely because the grating dimensions approach those of light waves. Therefore, interference and diffraction patterns are set up. Something comparable, although not strictly analogous, happens when the autocorrelation is computed to too many lags for the particular record length. The computation becomes unstable. A common rule of thumb is that the number of lags should not exceed 10 percent of the number of data points. A smaller number of lags will often suffice.

Figure C-2 illustrates the autocorrelation functions of two pure sine waves and of their sum. Although the particular example is simpler than those found in nature, it shows how the number of lags (in this case, eight) relates to the description of the record by means of an autocorrelation function. It can be seen intuitively that any variance due to a secular trend in the record will be measured at lag 0. This is because a secular variation has an infinite period (zero frequency). This variance will, of course, be added to that due to harmonic motion.

Fourier Transform and Smoothed Spectrum

The Fourier cosine transform is calculated as

$$V_r = \frac{k}{m}\left[C_o + C_m \cos r\pi + 2\sum_{q=1}^{m-1} C_q \cos\frac{q\,r\,\pi}{m}\right] \tag{C.2}$$

in which V_r = Fourier cosine transform of the autocorrelation at lag r, q = lag number having values between 1 and m-1, and k = constant,

(C.3)
$k = 1$ for $r = 1,2,\ldots m-1$
$k = 1/2$ for $r = 0$
$k = 1/2$ for $r = m$

and the other letters have the definitions previously given.

The smoothed estimate of spectral density is obtained from a final weighting function, expressed mathematically as

$$U_0 = 0.54\ V_0 + 0.46\ V_1 \tag{C.4}$$

$$U_r = 0.23\ V_{r-1} + 0.54\ V_r + 0.23\ V_{r+1} \tag{C.5}$$

for $r = 1,2,3,\ldots m-1$, and

(C.6) $U_m = 0.46\ V_{m-1} + 0.54\ V_m$

in which U_0, U_r, U_m = the power spectrum estimates corresponding to the respective lags, and the remaining symbols have the meanings previously assigned.

Sample calculations of the smoothed spectrum are presented by Wastler.

Aliasing

The designs of both the sampling interval and the subsequent statistical or spectral analysis require consideration of aliasing. Aliasing, defined graphically in Figure C-3, results from the high-frequency events that add variance to the record but that are not "seen" by the particular sampling interval. Figure C-3 illustrates how the variance from this type of event is folded into the record and reappears at a lower frequency. Where the period of the high-frequency event is known, the period of the aliased record can readily be determined analytically.

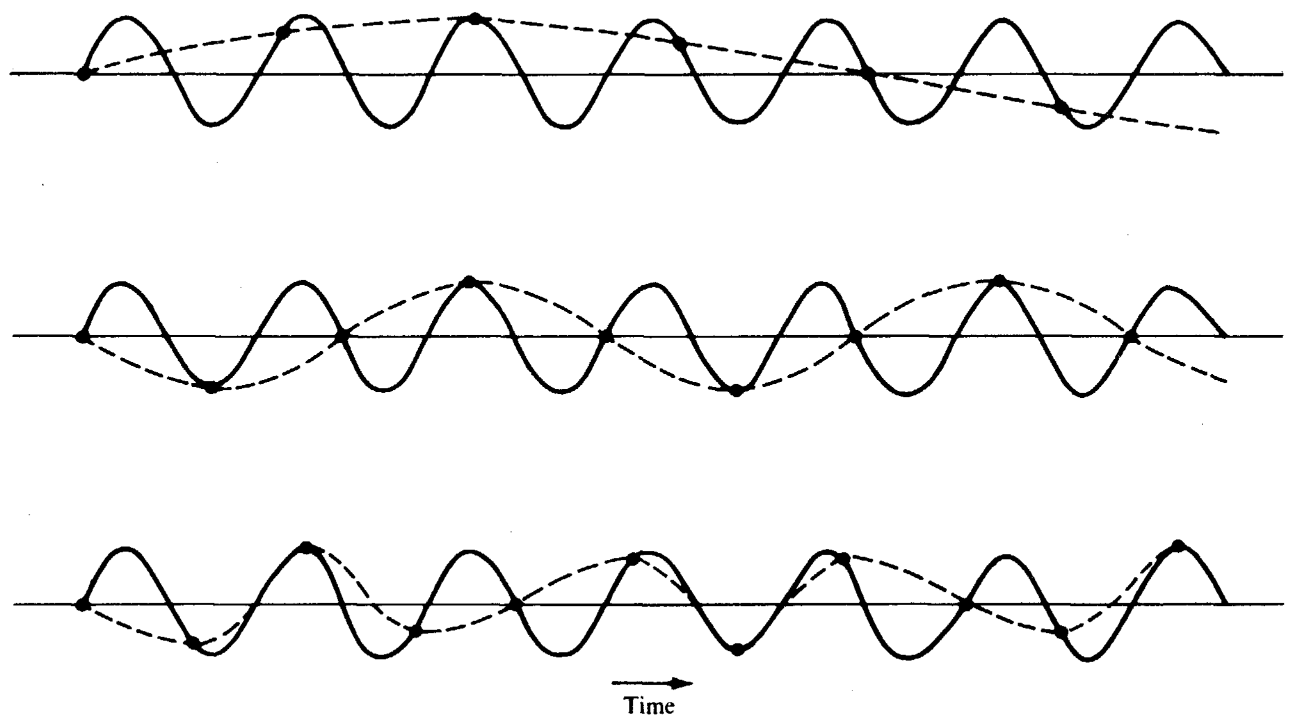
Inspection of Figure C-3 reveals that any cyclic event that occurs at a period less than twice the sampling interval will result in aliasing. Where the period equals twice the sampling interval, the event will never be seen. The same is true for any event whose period (P) is related to the sampling interval (I) by $P = 2T/n$, in which n is a positive integer. In the design of sampling programs, those cases in which $n > 1$ are not normally considered. Only when the period exceeds twice the sampling internal can the event be measured. As the period of the cyclic event approaches a value of twice the sampling interval, the record length necessary to describe the event increases. The corresponding frequency, f_n, which limits the events seen by a particular sampling frequency, f_s, is known as the Nyquist frequency, $f_n = 1/2\ f_s$.

General

The interrelationships among sampling interval, record length, number of lags, degrees of freedom, and confidence limits are examined briefly by Wastler. Definitive treatments are presented in several of the works listed in the bibliography.

The bibliography also includes works that describe the calculation and interpretation of cross spectra. These are basically a comparison of pairs of individual power spectra, which produce the equivalent of correlation coefficients between time series measuring different events (for example, tidal stage and salinity). In addition to providing a measure of the correlation or coherence between the two events, the phase relationship is established and other criteria are determined.

Fig. C-3. Typical Effects of Observing Periodic Events at Intervals of More Than One-half the Period.
Source: Gunnerson (1975).



Note: True record; aliased record; ● sampling time.

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APPENDIX D

DETERMINATION OF SEAWATER DENSITY

Gravimetrically determined seawater density has been found to be a non-linear function of temperature and salinity. While conversion of density values to five significant figures to specific volumes at various depths is necessary for computation of oceanic-scale (geostrophic) currents, seawater density at atmospheric pressure is used in ocean outfall design. This density, ρ_o, is expressed as σ_o where $\sigma = (\rho_o - 1)\ 1000$. Salinity is usually measured as chlorinity, Cl (or indirectly as specific conductance). At 0^oC and atmospheric pressure, the effect of chlorinity on density is:

$$\sigma_o = -0.069 + 1.4708\ Cl - 0.001570\ Cl^2 + 0.0000398\ Cl^3$$

and the effect of temperature is:

$$\sigma_\zeta = -\textstyle\sum_t + (\sigma_o - \sum_o)[1 - A_t + B_t\ (\sigma_o + \sum_o)]\ \text{where}$$

$$\textstyle\sum_t = -\ (\zeta - 3.98)^2\ (\zeta + 283)/503.570\ (\zeta + 67.26)$$

$$\textstyle\sum_o = -0.1324,\ \zeta = {}^oC,$$

$$B = 1 - 4.7867 \times 10^{-3}\ {}^oC + 9.8185 \times 10^{-5}\ {}^oC^2 - 1.0843 \times 10^{-6}\ {}^oC,\ \text{and}$$

$$C = 1.8030 \times 10^{-5}\ \zeta - 8.164 \times 10^{-2}\ \zeta^2 + 1.667 \times 10^{-8}\ \zeta^3.$$

A number of tables showing σ_t for different values of temperature and salinity (S = 1.203 Cl) are available. For outfall design purposes, the nomographs shown on Figures D-1 through D-6 are sufficiently accurate.

Fig. D-1. Nomograph for Determining Density from the Salinity and Temperature of Sea Water, for 0.00 to 7.50% S and 2.00 to 33.00°C

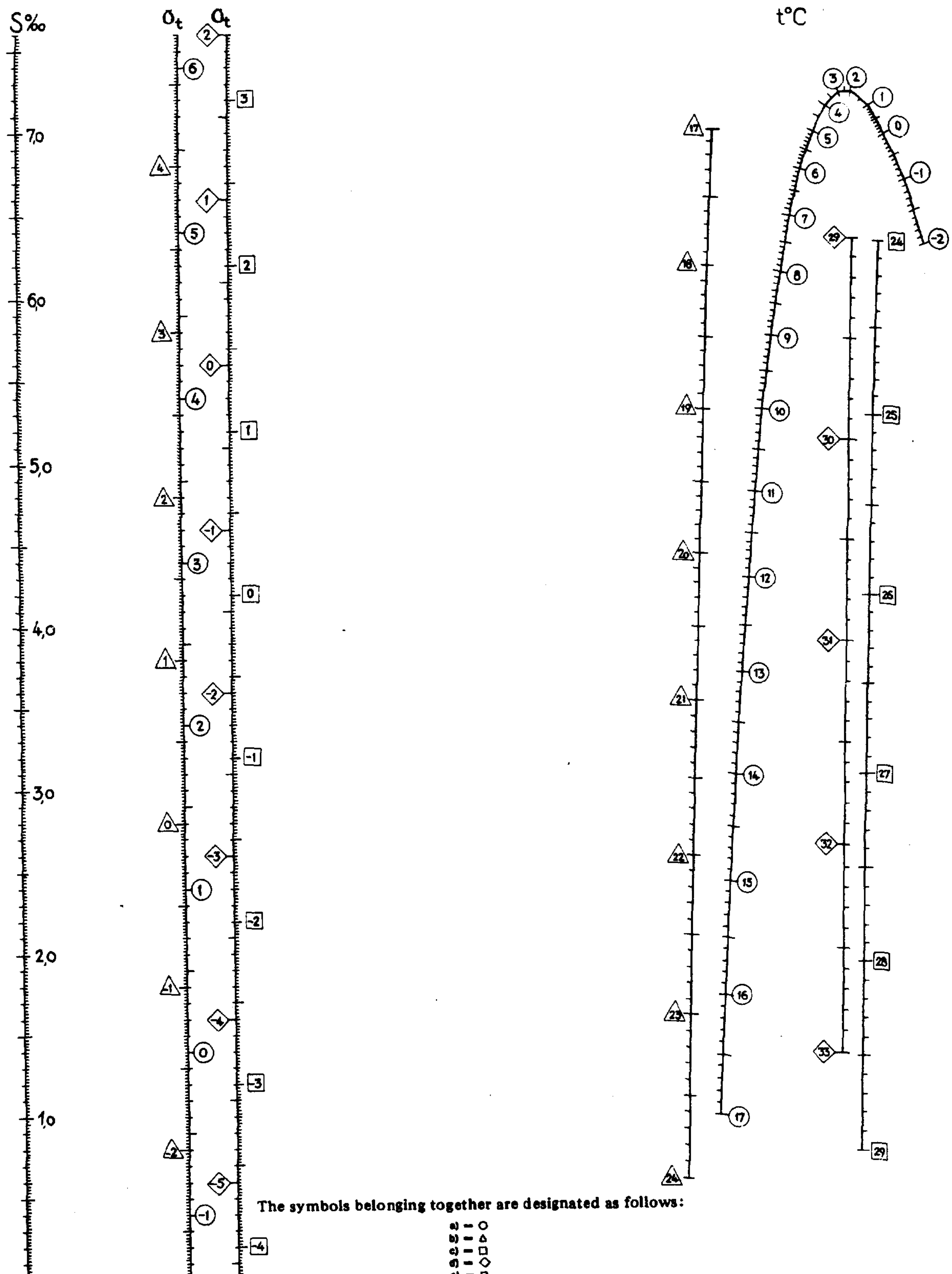


Fig. D-2. Nomograph for Determining Density from the Salinity and Temperature of Sea Water, for 7.00 to 14.50 S% and 2.00 to 33.00°C

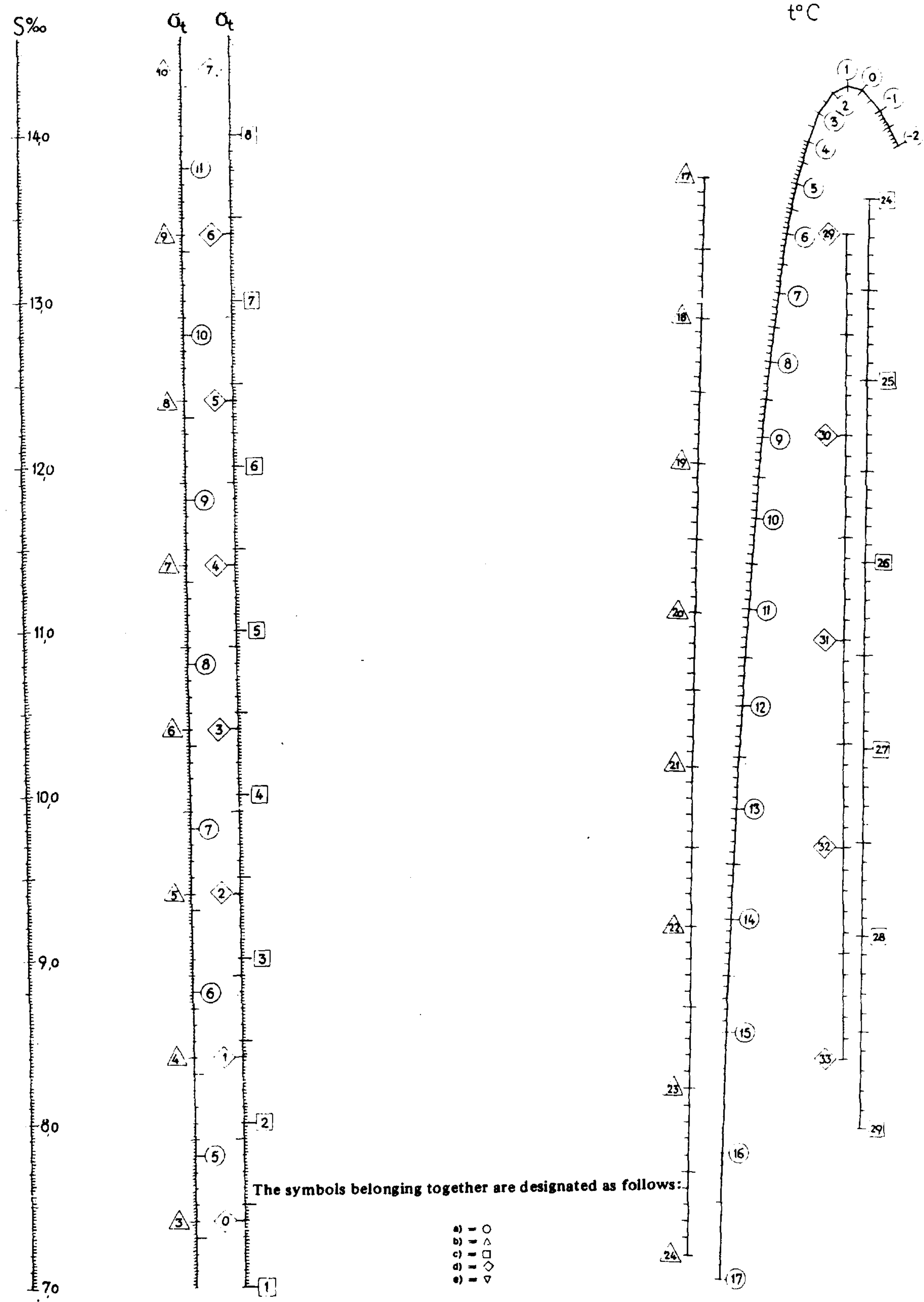


Fig. D-3. Nomograph for Determining Density from the Salinity and Temperature of Sea Water, for 14.00 to 21.50 S% and 2.00 to 33.00°C

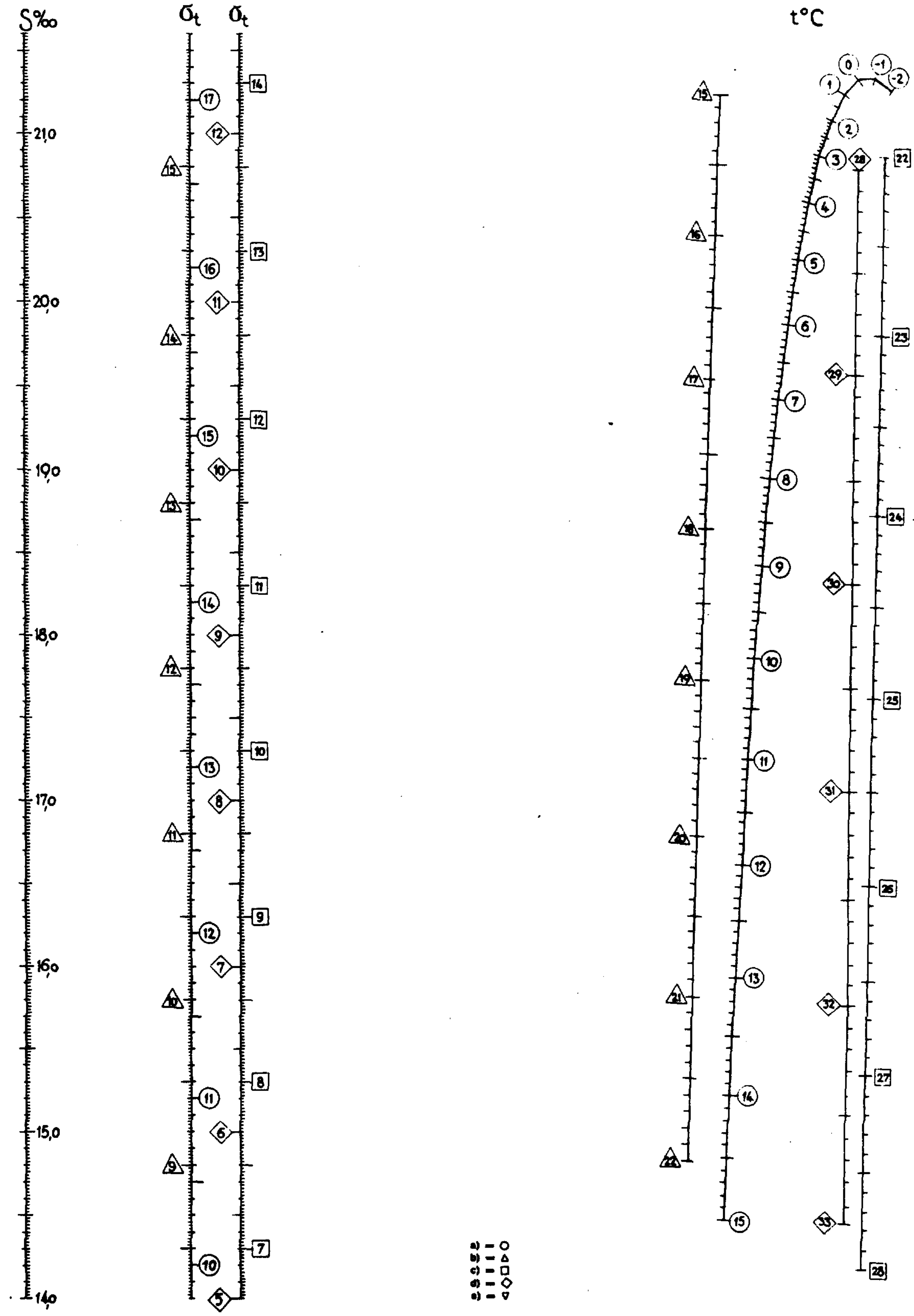


Fig. D-4. Nomograph for Determining Density from the Salinity and Temperature of Sea Water, for 21.00 to 28.50 S% and 2.00 to 33.00°C

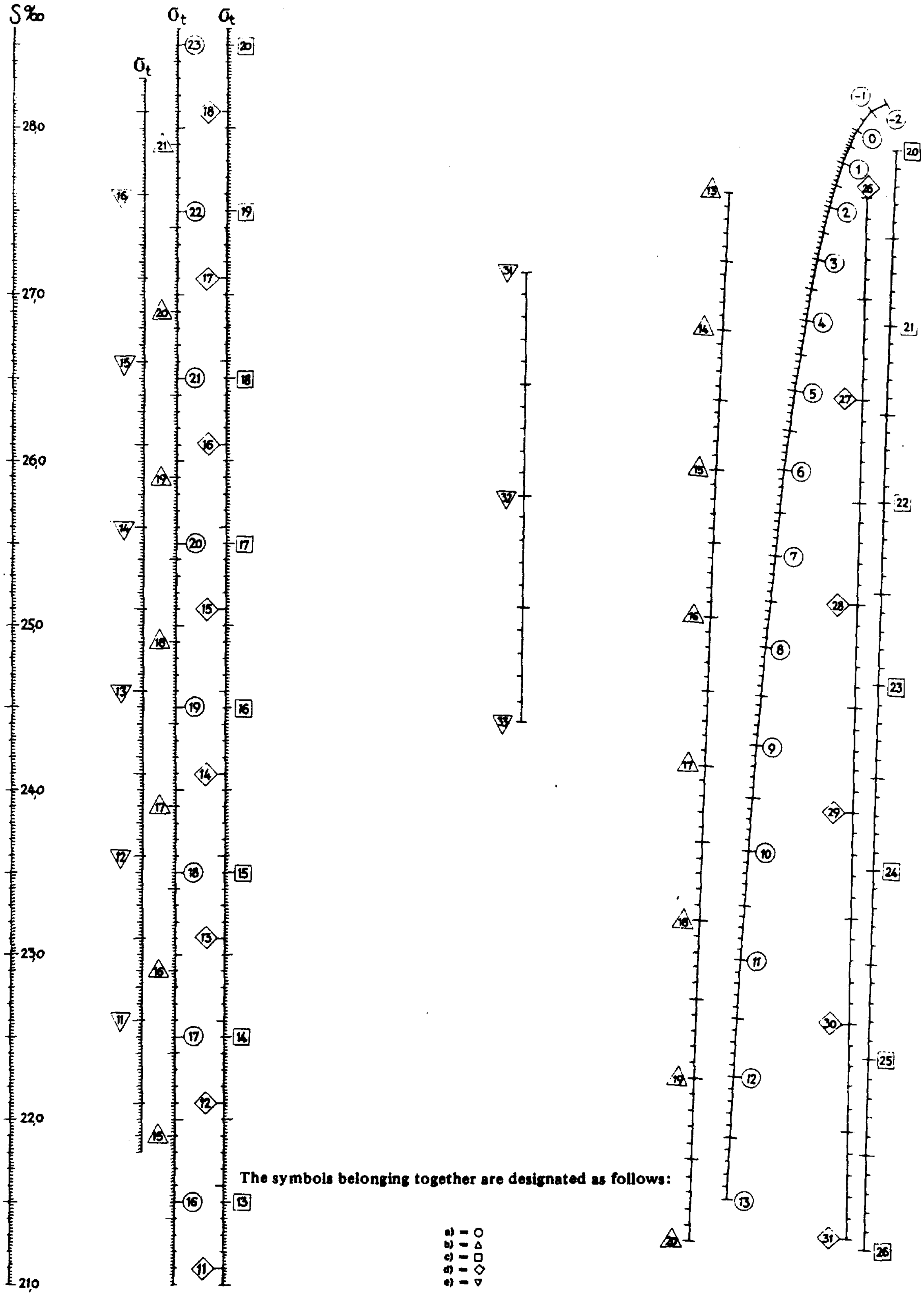


Fig. D-5. Nomograph for Determining Density from the Salinity and Temperature of Sea Water, for 28.00 to 35.50% S and -2.00 to 33.00°C

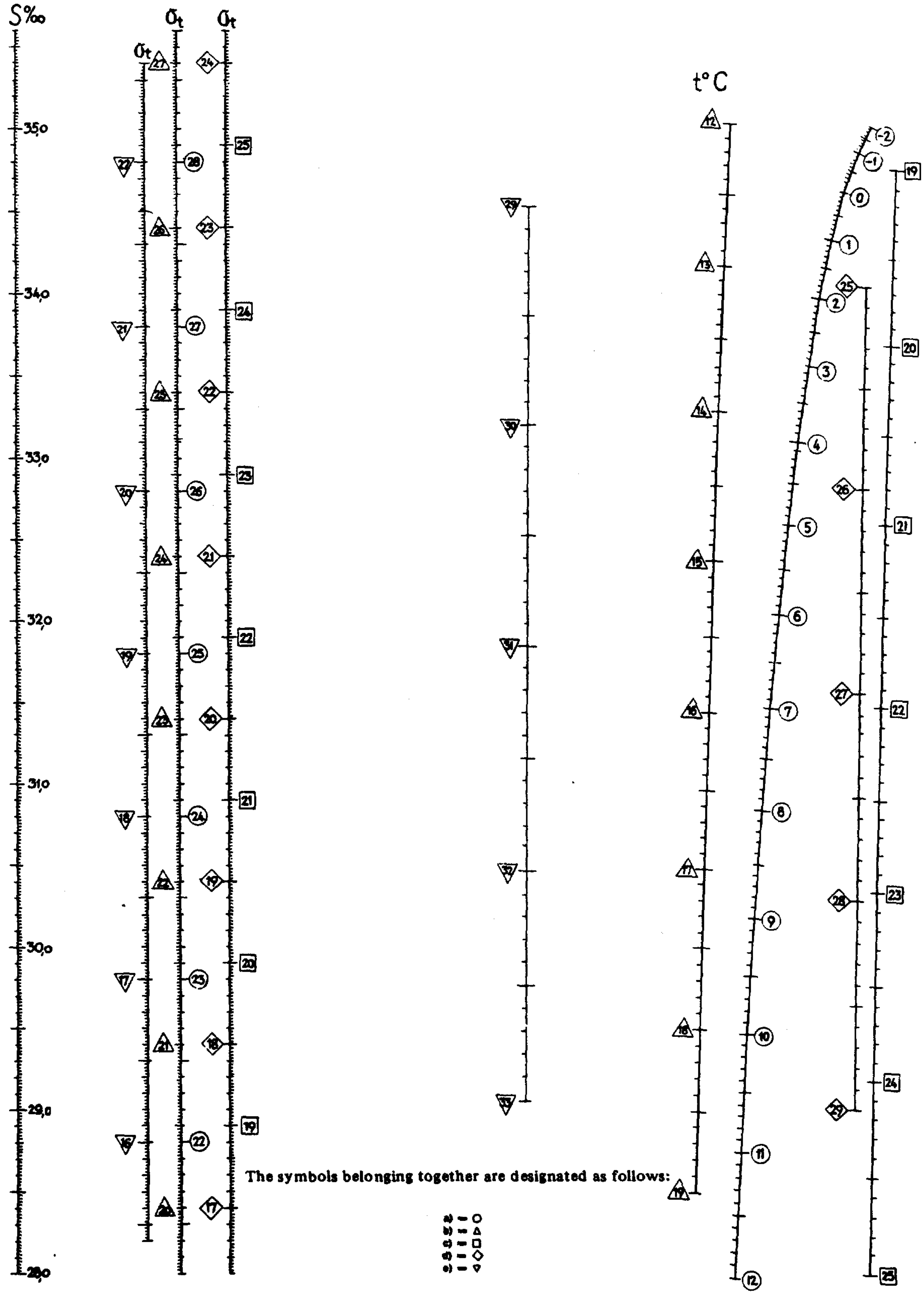
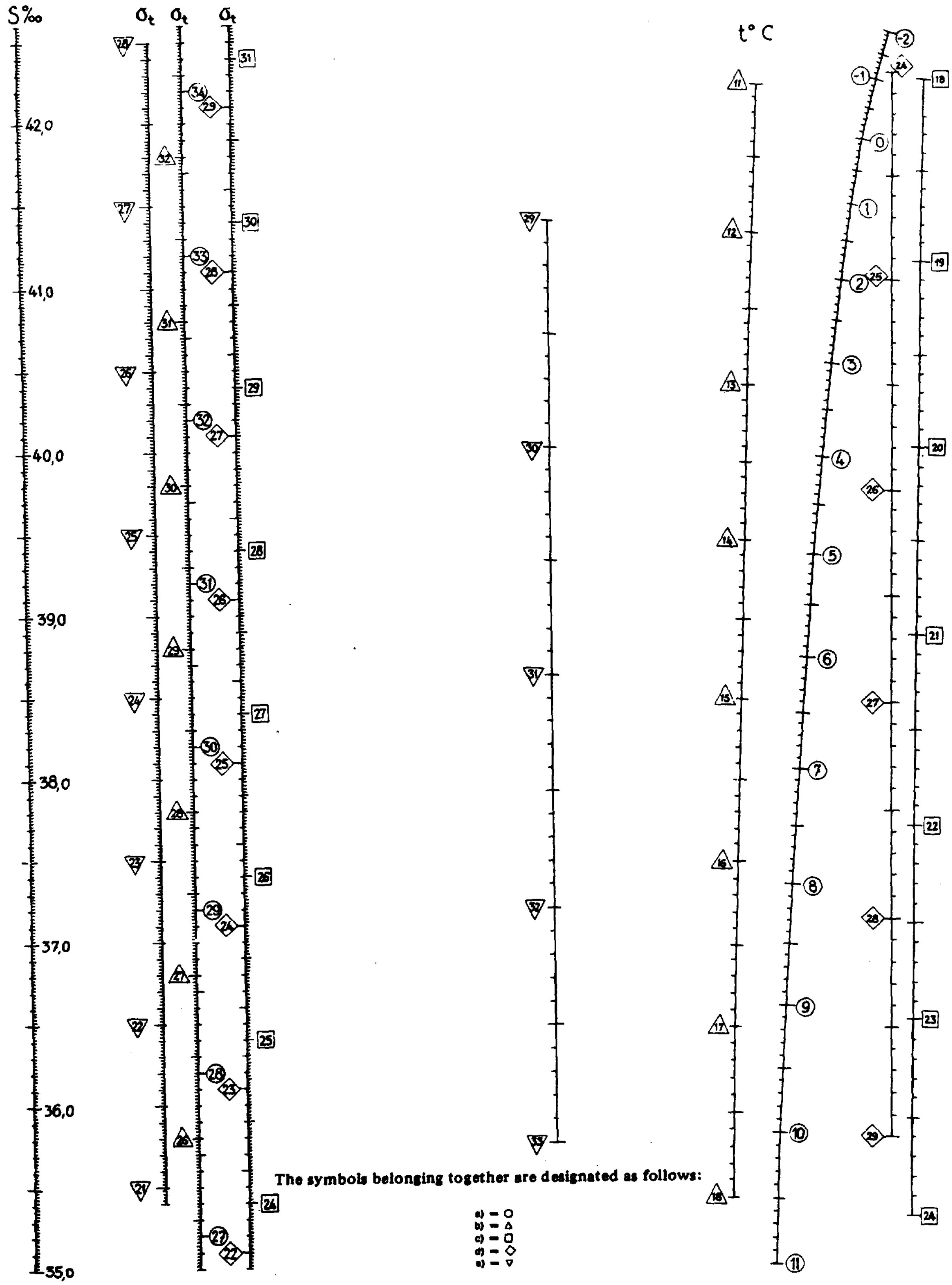


Fig. D-6. Nomograph for Determining Density from the Salinity and Temperature of Sea Water, for 35.00 to 42.50‰ S and -2.00 to 33.00°C



APPENDIX E

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