CECW-EH-D Engineer Manual 1110-2-1618	Department of the Army U.S. Army Corps of Engineers Washington, DC 20314-1000	EM 1110-2-1618 10 July 1989
	Engineering and Design COASTAL INLET HYDRAULICS AND SEDIMENTATION	
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EM 1110-2-1618
28 April 1995

US Army Corps
of Engineers

ENGINEERING AND DESIGN

Coastal Inlet Hydraulics and Sedimentation

ENGINEER MANUAL

	DEPARTMENT OF THE ARMY	EM 1110-2-1618
	U.S. Army Corps of Engineers	
CECW-EH-D	**Washington, DC 20314-1000**	

Manual
No. 1110-2-1618 — 28 April 1995

Engineering and Design
COASTAL INLET HYDRAULICS AND SEDIMENTATION

1. Purpose. This manual describes methods for evaluating the hydraulic and associated sediment transport problems encountered at coastal inlets. Guidance is provided in selecting and applying such methods to support the various investigations required for U.S. Army Corps of Engineers (USACE) civil works activities. The manual references publications that contain the theoretical basis of the methods and detailed information on their use.

2. Applicability. This manual applies to all HQUSACE elements, major subordinate commands, districts, laboratories, and field operating activities having civil works project design responsibilities.

FOR THE COMMANDER:

R. C. JOHNS
Colonel, Corps of Engineers
Chief of Staff

DEPARTMENT OF THE ARMY EM 1110-2-1618
U.S. Army Corps of Engineers
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Chapter 1
Introduction

1-1. Purpose

This manual provides guidance for the development, improvement, and maintenance of navigation and flood control projects at entrances to tidal inlets. Factors are presented that should be considered in providing safe and efficient navigation facilities with the least construction and maintenance costs. The design engineer is expected to adapt general guidance presented in this manual to site-specific projects; deviations from this guidance are acceptable if adequately substantiated. As the state of the art advances, this manual will undergo periodic revision.

1-2. Applicability

This manual applies to all HQUSACE elements, major subordinate commands, districts, laboratories, and field operating activities having civil works responsibilities.

1-3. References

The references listed below are required to carry out the design effort described in this manual.

a. U.S. Army Corps of Engineers Publications.[1]

(1) ER 1110-2-1404, Deep-Draft Navigation Project Design.

(2) ER 1110-2-1458, Shallow-Draft Navigation Project Design.

(3) EM 1110-2-1202, Environmental Engineering for Deep-Draft Navigation.

(4) EM 1110-2-1412, Storm Surge Analysis.

(5) EM 1110-2-1414, Water Levels and Wave Heights for Coastal Engineering Design.

(6) EM 1110-2-1502, Coastal Littoral Transport.

(7) EM 1110-2-1607, Tidal Hydraulics.

(8) EM 1110-2-1613, Hydraulic Design of Deep-Draft Navigation Projects.

(9) EM 1110-2-1614, Design of Coastal Revetments, Seawalls, and Bulkheads.

(10) EM 1110-2-1615, Hydraulic Design of Small Boat Harbors.

(11) EM 1110-2-1616, Sand Bypass System Design.

(12) EM 1110-2-2904, Design of Breakwaters and Jetties.

(13) EM 1110-2-5025, Dredging and Dredged Material Disposal.

b. U.S. Government Publications.

Shore Protection Manual (SPM), 4th ed., Vols I and II, U.S. Army Engineer Waterways Experiment Station (WES), Coastal Engineering Research Center. Available from Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402.

1-4. Bibliography

Bibliographic information throughout this manual is denoted by author and date corresponding to the listing in Appendix A. These documents are available for loan upon request to the WES Technical Information Library, 3909 Halls Ferry Road, Vicksburg, MS 39180-6199.

1-5. Background and Scope

Providing systematic design guidance for tidal inlets is a most difficult task. This is principally due to the inherent complexities in the morphology, migration patterns, and hydrodynamics of tidal inlets. A thorough understanding of the processes which control the inlet/back-bay system, however, will help assure that the design of engineering structures, or the modification of inlet hydraulics, will result in the most efficient project design. Optimally, such designs will have minimal impact on the tidal inlet system.

1-6. Overview of Manual

a. Coastal engineering projects at tidal inlets often require estimation of sediment transport, channel stability, structure stability, and a critical assessment of project requirements. The design engineer can gain insight about a tidal inlet project by dividing the system into

[1] U.S. Army Corps of Engineers publications available from: USACE Publications Depot, 2803 52nd Avenue, Hyattsville, MD 20781.

components and accurately assessing or classifying each component. A thorough understanding of the complex tidal inlet system is essential for proper planning and design of coastal structures at tidal inlets. Basic principles for making these decisions are presented in this manual.

b. This manual is general in nature and therefore requires that engineering judgment be exercised when applying methods and procedures presented herein to actual tidal inlets. Although a thorough understanding of the underlying concepts is not essential in performing tidal inlet design analysis, a basic understanding of the hydrodynamic processes and their interaction with the structural and geomorphic features present in the inlet system are required to ensure proper application.

c. This manual contains eight chapters. Chapter 1 provides an introduction and overview of the remaining chapters. Chapter 2 describes the geomorphology and morphodynamics of tidal inlets. Because inlets interrupt the continuity of coastal processes, they exert a dramatic influence on shoreline erosional and depositional trends, sediment transport patterns, and sediment budgets. Successful design and implementation of an inlet project require an ability to predict the morphologic behavior of an inlet; this chapter provides the necessary background information for making such predictive determinations. Various inlet classification schemes are presented and examples of types of information that can be gained through geomorphic and geologic analysis are demonstrated.

d. Hydrodynamic aspects of tidal inlets are described in Chapter 3. In addition to a presentation of the governing equations and general hydrodynamic parameters, techniques for evaluating inlet stability are discussed. Classic work by O'Brien, Bruun, Bruun and Gerritsen, Keulegan, and Jarrett regarding relationships between inlet cross-sectional area, tidal prism, maximum throat velocity, and littoral transport rate are reviewed.

e. Chapter 4 focuses on sediment budget analyses of tidal inlets. Included are discussions of factors to be considered in an inlet sediment budget analysis and a detailed summary of a sediment budget study performed for Beaufort Inlet, North Carolina by the Wilmington District. The sediment budget analysis was used to help evaluate shoaling patterns in the inlet area, bypassing mechanisms, effects of earlier dredging on the adjacent barriers, and future impacts of proposed channel deepening on the entire inlet system.

f. Engineering design of tidal inlets involves either improvements to an existing inlet or development of a new inlet. Structural improvements at inlets may include construction or rehabilitation of jetties, breakwaters, or sand bypassing plants. The ability to anticipate project impacts and implement appropriate measures to alleviate adverse effects are the keys to successful design practice. It is also important that the designed features perform their intended functions with minimum maintenance requirements. Chapter 5 discusses design aspects of inlet projects, including navigation channel design, jetty design theory and principles, types of construction material, stability considerations, and studies of estimated costs and benefits.

g. Chapter 6 describes the physical modeling of tidal inlets. Physical model studies of inlets are typically designed to investigate various methods of maintaining an effective navigation channel through an inlet. Additional inlet-related problems that can be addressed by physical models include optimizing structure dimensions and locations, shoaling and scouring trends, tidal prism changes, and salinity effects. Model theory, including assumptions and limitations, is discussed. Fixed-bed and movable-bed models are described and examples of each are provided. Considerations of scale, distortion, historical applications, utility of physical models, and combined numerical and physical models are discussed.

h. Numerical models and their application to tidal inlet analysis are discussed in Chapter 7. Various types of numerical models and modeling systems that have been applied in Corps inlet studies are presented. A brief description of each model is given, followed by model input and output requirements, example model applications, and additional references.

i. Chapter 8 presents guidance related to monitoring existing inlet projects. Criteria needed to evaluate structure performance, recommended equipment, instrumentation, and surveying techniques are outlined.

j. Appendices A and B respectively provide lists of references and notation appearing in the text. Appendix C presents an annotated bibliography of publications from the General Investigations of Tidal Inlets (GITI) program.

Chapter 2
Inlet Geomorphology and Geology

2-1. Introduction

a. Purpose and scope.

(1) Tidal inlets are prevalent morphologic features along the coastlines of the United States. They are most commonly associated with the barrier island shorelines that typify the geomorphology of the east and Gulf of Mexico coasts. Recent surveys of the coast from Long Island to Florida suggest that there are over 144 presently active tidal inlets; another 164 inlets in this same area are inferred from historical maps, navigation charts, or aerial photographs (McBride and Moslow 1991). Many of these inlets provide the primary navigation link between the ocean and inland waterways, harbors, and ports. Along the coasts of Maine and the western United States, tidal inlets are much less frequent but play equally as important a role in coastal processes, sedimentation, and erosion.

(2) Tidal inlets serve as extremely important conduits for the exchange of water and sediments between bays, lagoons, or estuaries and the continental shelf. Because they interrupt the uniformity and continuity of coastal processes and sediment transport, tidal inlets exert a tremendous influence on shoreline erosion/deposition trends, sediment budgets, and migration history. Many tidal inlets are either ephemeral in nature or are associated with rapid large-scale morphologic changes. Thus, the behavior of inlets can have extremely significant environmental, social, and economic impacts.

(3) For the coastal engineer, a thorough understanding of the geomorphology and sedimentation of tidal inlets is a critical prerequisite to successful design analysis. For example, any project to create or maintain an inlet channel cannot be assured of success without first acquiring a knowledge of the sediment hydrodynamics and migration characteristics of that inlet. In general, successful design and implementation requires an ability to predict the behavior and performance of a tidal inlet. The best source of information for making such an assessment comes from long-term frequent observations and site monitoring. For those inlets undergoing long-term maintenance programs, this may not present a serious problem. Alternatively, in areas where new projects are being developed, historical navigation charts and maps may provide the best available information. However, in most instances, such historical information is either not available or is reliable only as a baseline indicator of major long-term morphologic trends. Thus, the best source of information for predicting the behavior of tidal inlets comes from a thorough understanding of their geomorphology and sedimentation. It is the purpose of this chapter to provide the necessary background information for making such predictive determinations. In addition, this chapter will review the general stratigraphy of tidal inlets to provide an appreciation of their three-dimensional variability, ultimate sediment dispersal patterns, and utility as a source of sediment for beach nourishment or replenishment projects.

b. Objectives. The major objectives of this chapter are to describe and evaluate the following:

(1) Specific definitions of the geomorphic features that comprise tidal inlets and the different types of inlet systems.

(2) Standard classification schemes for tidal inlets and the types of information gained from such classifications.

(3) Morphology and process of deposition at tidal inlets including the geologic controls on inlet distribution, position and migration, and inlet sedimentation patterns.

(4) Sedimentology and stratigraphy of tidal inlet deposits.

(5) Sand resource potential of inlet deposits.

2-2. Definitions

a. Inlets.

(1) In the broadest sense of the term, an *"inlet"* is generally recognized as a relatively small-scale waterway that connects an inland body of water with the ocean. The term inlet is generally associated with those channels that serve as conduits for the exchange of waters during the tidal cycle between a lagoon, bay, or estuary and a larger tidal body. These inlet channels are referred to as *tidal inlets*, and are by far the most common type of inlet found on the coastlines of the United States. It is important to realize, however, that inlets do exist in nontidal environments such as lake basins. There are numerous examples of such inlets along the shorelines of the Great Lakes.

(2) On a geological level, tidal inlets can be categorized as one of the following three types: (a) marine, (b) fluvial, or (c) bedrock controlled. Marine tidal inlets

are associated with barrier island coastlines and thus are common to the U.S. east and gulf coasts. Fluvial tidal inlets are found in coastal areas influenced by a combination of marine and fluvial processes, such as abandoned river deltas. Tidal inlets along the Louisiana coastline are examples of this inlet type. As a function of the active tectonic uplift common to the western continental margin of North America, tidal inlets along the Pacific coast of the United States are often bedrock-controlled. They are also relatively rare in a coastal setting of this nature.

b. Morphologic features.

(1) General. Variations in tidal inlet geomorphology are a function of the hydrographic and hydrodynamic regime, specifically, tidal range, tidal prism, and wave energy. The balance and interaction between these parameters in any open system dictate the relative size, distribution, and abundance of inlet-affiliated morphologic features. The principal morphologic units associated with tidal inlets are tidal deltas, inlet channel(s), and recurved spits. For the sake of clarity and consistency in use, the spatial relationships of these main morphologic elements are illustrated in Figure 2-1 for a tidal inlet situated along a barrier island coastline. This figure depicts a hydrodynamic setting in which there exists a relative balance between tidal and wave energy resulting in tidal deltas of approximately equal size and shape. Such situations are rare; typically one of the two deltas is dominant in all dimensions.

(2) Tidal deltas. Hayes (1969, 1980) proposed the following terminology and definitions for tidal deltas: (a) *ebb-tidal delta* - sediment accumulation seaward of a tidal inlet, deposited primarily by ebb-tidal currents and modified by waves and (b) *flood-tidal delta* - sediment accumulation formed on the landward side of an inlet by flood-tidal currents. Engineers often refer to these sand deposits as the interior shoal (flood-tidal delta) and the outer shoal (ebb-tidal delta) (Dean and Walton 1975). Components of typical ebb- and flood-tidal deltas have been identified by geomorphologists through numerous field investigations and aerial reconnaissance.

(a) Ebb-tidal deltas. Hayes (1980) developed a model for ebb-tidal delta morphology that displays all its major components (Figure 2-2). By way of providing a definition for these morphologic features, the following is extracted from Hayes (1980):

- Main ebb channel - a channel which usually shows a slight-to-strong dominance of ebb-tidal currents over flood-tidal currents.

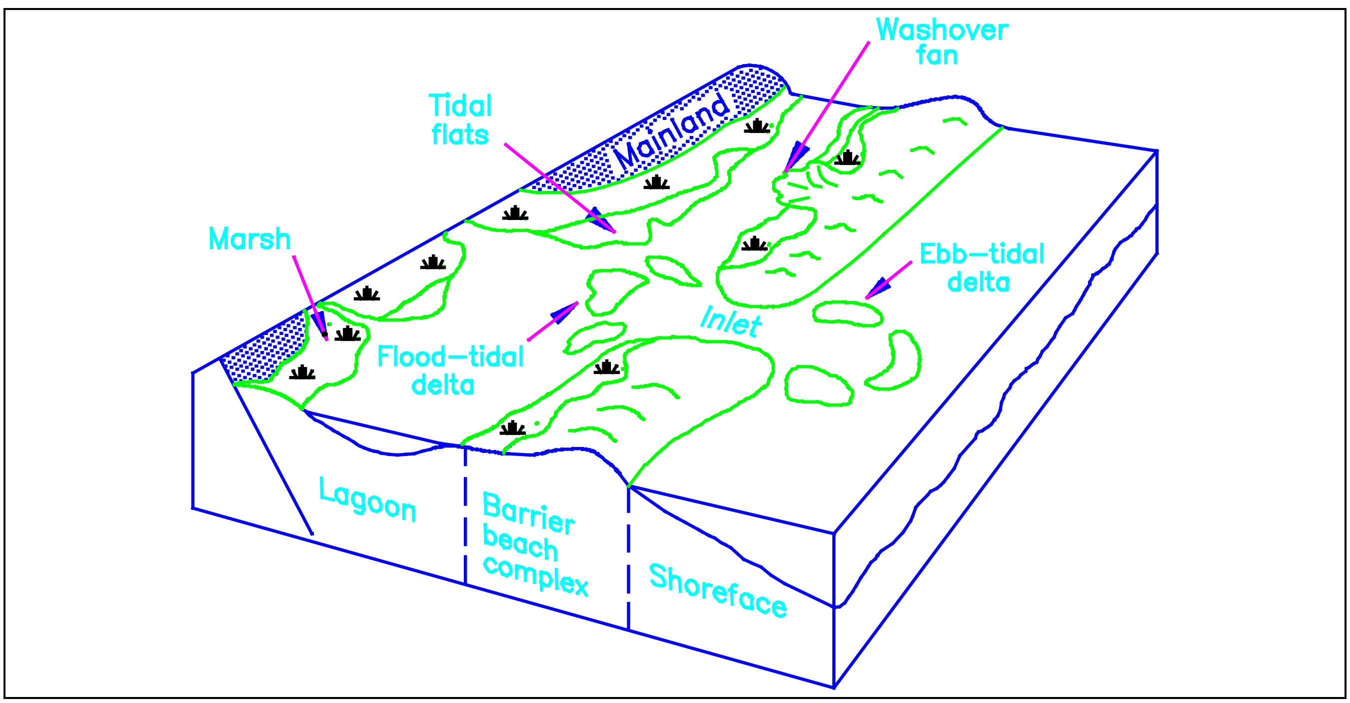


Figure 2-1. Block diagram displaying the depositional environments associated with a barrier island shoreline (after Reinson (1984))

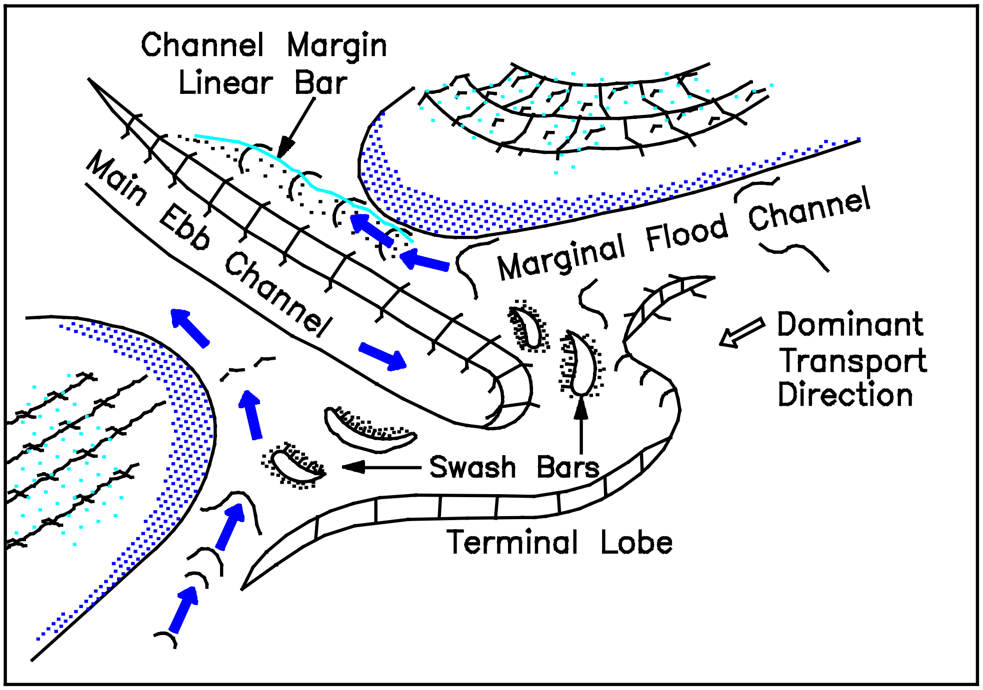


Figure 2-2. Model of the ebb-tidal delta morphology. Arrows indicate dominant direction of tidal currents (from Hayes (1980))

- Channel-margin linear bars - flanking the main ebb channel on either side, built by the interaction of ebb- and flood-tidal currents with wave-generated currents.

- Terminal lobe - a relatively steep, seaward-sloping lobe of sand at the seaward end of the main channel.

- Swash platforms - broad sheets of sand that flank both sides of the main channel.

- Swash bars - built by swash action of waves on the swash platforms.

- Marginal flood channels - marginal tidal channels dominated by flood-tidal currents; usually occur between the swash platform and the adjacent updrift and downdrift beaches.

(b) Flood-tidal deltas. A model for flood-tidal delta morphology, as developed by Hayes (1980) (Figure 2-3), consists of the following morphologic features:

- Flood ramp - seaward-facing slope on the tidal delta over which the main force of the flood current is directed.

- Flood channels - channels dominated by flood currents that bifurcate off the flood ramp.

- Ebb shields - topographically high rims or margins around the tidal delta that protect portions of it from modification by ebb currents.

- Ebb spits - spits formed by ebb-tidal currents.

- Spillover lobes - lobate bodies of sand formed by unidirectional currents.

(3) Inlet channel(s). The following definitions are provided for the various morphologic features of inlet channels (Figure 2-2):

(a) Gorge - main section of the inlet channel where primary tidal flows occur.

(b) Flood (marginal) channels - secondary channels of flow during flood tide conditions, typically located on either side of the inlet between the barrier island beach and the ebb-tidal delta.

(c) Throat - the narrowest part of the inlet channel cross section.

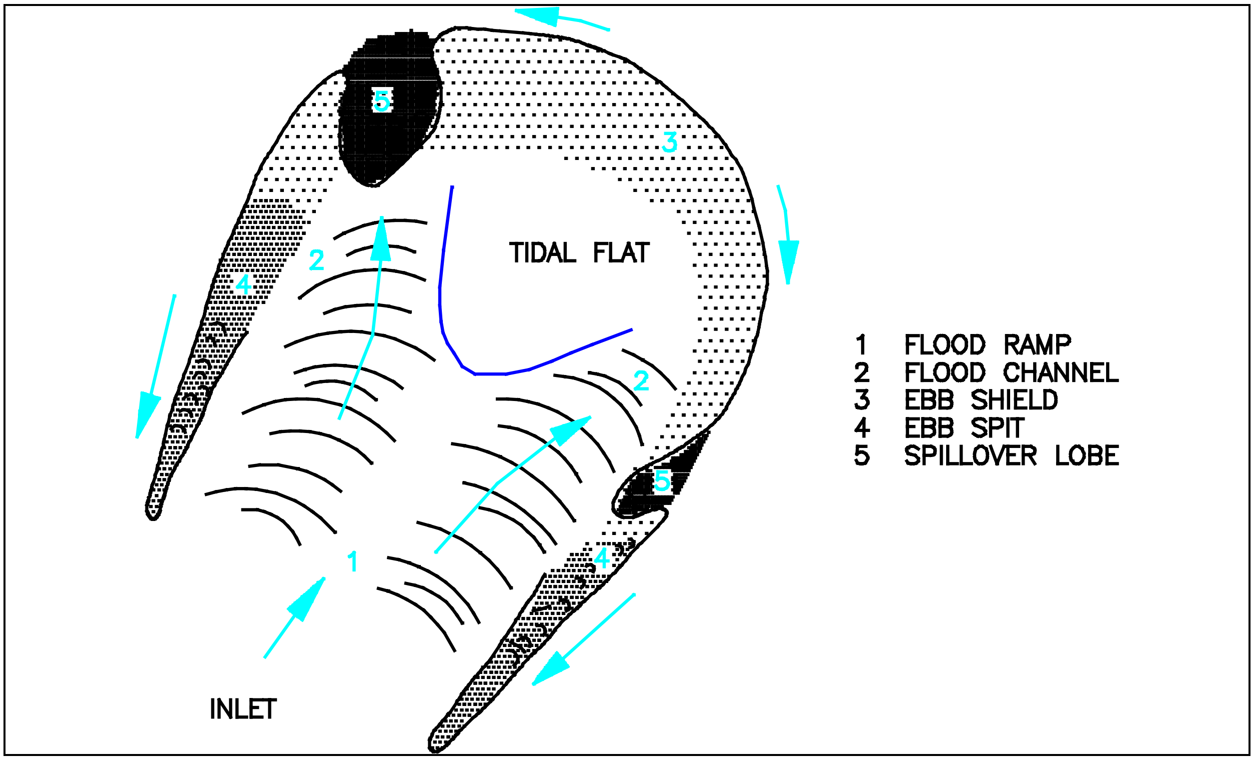


Figure 2-3. Model of flood-tidal delta morphology. Arrows indicate dominant direction of tidal currents (from Hayes (1980))

(d) Main ebb channel(s) - secondary channels of flow during ebb tide.

(4) Recurved spits. The barrier beach on the updrift side of a laterally migrating tidal inlet is a *recurved spit* (Figure 2-4). The spit is characterized by a series of curving beach ridges and an intertidal ridge and runnel system that is welded onto the channel margin of the spit and fronted by foredunes.

2-3. Classification Schemes

All commonly accepted tidal inlet classification schemes are based on a recognition of the controls on morphology, sedimentation, and stratigraphy exerted by natural environmental factors and depositional processes. The most important of these are tidal range, wave energy, tidal prism, and sediment supply. Varying classification schemes are presented in this section, all of which underscore the inherent relationship between tidal inlets and the associated barrier island shoreline.

a. Geologic classification.

(1) The fundamental geologic classification distinguishes between primary features, which are the result of geologic (tectonic) activity, and secondary features, which are the result of modifications of primary landforms by natural processes. Primary inlets exist where tectonic activity, such as fault block movement, or glacial activity resulting in the breaching of a sill or other feature, creates a previously nonexistent egress to the sea or other large body of water. These features are generally restricted to fluvial outlets.

(2) The more common inlet is a secondary feature that is the result of marine or tidal processes that have created a breach in an island or spit and maintain the connection through tidal flow. Potentially, a tidal inlet may develop on any depositional shoreline where antecedent topography and sea level fluctuations permit such a geometry to develop. The maintenance of large tidal shoals is dependent on available sediment supply.

Figure 2-4. Oblique aerial photograph of recurved spit on the updrift margin of a laterally accreting tidal inlet, Kiawah River Inlet, SC

b. Morphologic classification.

(1) A morphological classification of inlets was developed by Galvin (1971) that relates inlet geometry relative to the island segments it separates and net sediment transport. Four classes of natural inlets are defined, three of which are shown in Figure 2-5.

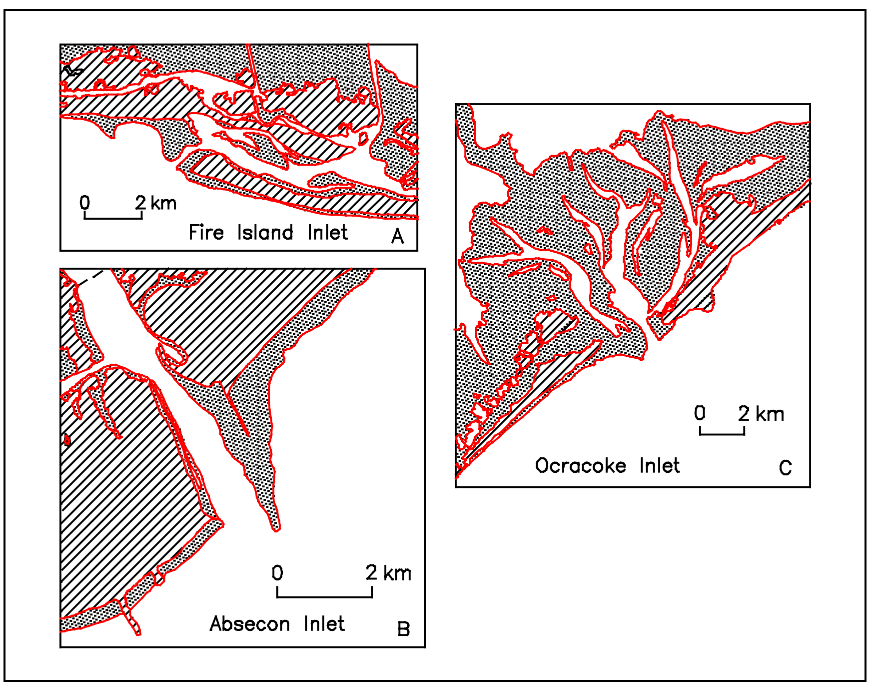
(a) Overlapping offset. Exists where there is an adequate sediment supply and unidirectional or strong net transport, e.g., Fire Island Inlet, New York (Figure 2-5a).

(b) Updrift offset. Exists where there is an adequate updrift sediment supply and moderate to weak net transport.

(c) Downdrift offset. Exists where there is an inadequate supply of sediment and weak net transport. When there is an insufficient supply of sediment, the updrift beaches become the sediment source and, thus, recede at a faster rate than the downdrift beaches. Southern New Jersey beaches provide a classic example of the downdrift offset, e.g., Absecon Inlet, New Jersey (Figure 2-5b).

(d) Negligible offset. Exists where wave directions are equally distributed through the year and there is little or no net transport. Consequently, wave energy is focused equally on the updrift and downdrift island ends, e.g., Ocracoke Inlet, North Carolina (Figure 2-5c).

(2) Each of these configurations can be created artificially through the placement of engineering structures, or by altering the sediment supply along the reach of coastline. Updrift offsets are particularly sensitive to structural



2-5. Examples of inlet types: (a) overlapping offset (Fire Island Inlet, New York), (b) downdrift offset (Absecon Inlet, New Jersey), and (c) negligible offset (Ocracoke Inlet, North Carolina) (after Swift (1976))

modifications. An example of a structurally imposed updrift offset inlet is Ocean City Inlet, Maryland. Classic examples of overlapping offset inlets occur at Fire Island, New York, and at Chincoteague Inlet, Virginia, where the distal end of Assateague Island has accreted past Chincoteague Island. Table 2-1 shows the relationship between each of the inlet geometries and sediment transport conditions (Galvin 1971).

Table 2-1
Inlet Geometry and Sediment Transport

	Overlapping Offset	Updrift Offset	Downdrift Offset	Negligible Offset
Availability of Sediment	Adequate updrift	Adequate updrift	Reach is only updrift source	?
Transport Ratio Tidal/ Longshore	Relatively equal	Relatively equal	Relatively less longshore	?
Transport Ratio Net/ Gross	1.0	< 1.0	< 1.0	Near 0
Wave Direction	1 direction only	1 direction dominant	1 direction dominant	2 directions equal

c. Hydrographic classification.

(1) Tidal inlets are intrinsic parts of the barrier shoreline in which they exist. Thus, they are subject to classification based on the hydrographic processes (wave energy and tidal range) affecting the coastal system as a whole. Davies (1964) delineated three types of shorelines based on tidal characteristics, shown in Table 2-2.

Table 2-2
Shoreline Classification Based on Tidal Range

Class	Tidal Range
Microtidal coast	< 2 m
Mesotidal coast	2-4 m
Macrotidal coast	> 4 m

(2) Based on regional observations of coastlines from around the world, Hayes (1975) identified consistent morphologic trends for each of the three classes of coast delineated by Davies (1964). In so doing, Hayes (1975) and Hayes and Kana (1976) developed the classification scheme for non-deltaic coastlines which is currently the most popular with sedimentologists and coastal geomorphologists. This coastal classification scheme is summarized in Figure 2-6, which illustrates the distribution of

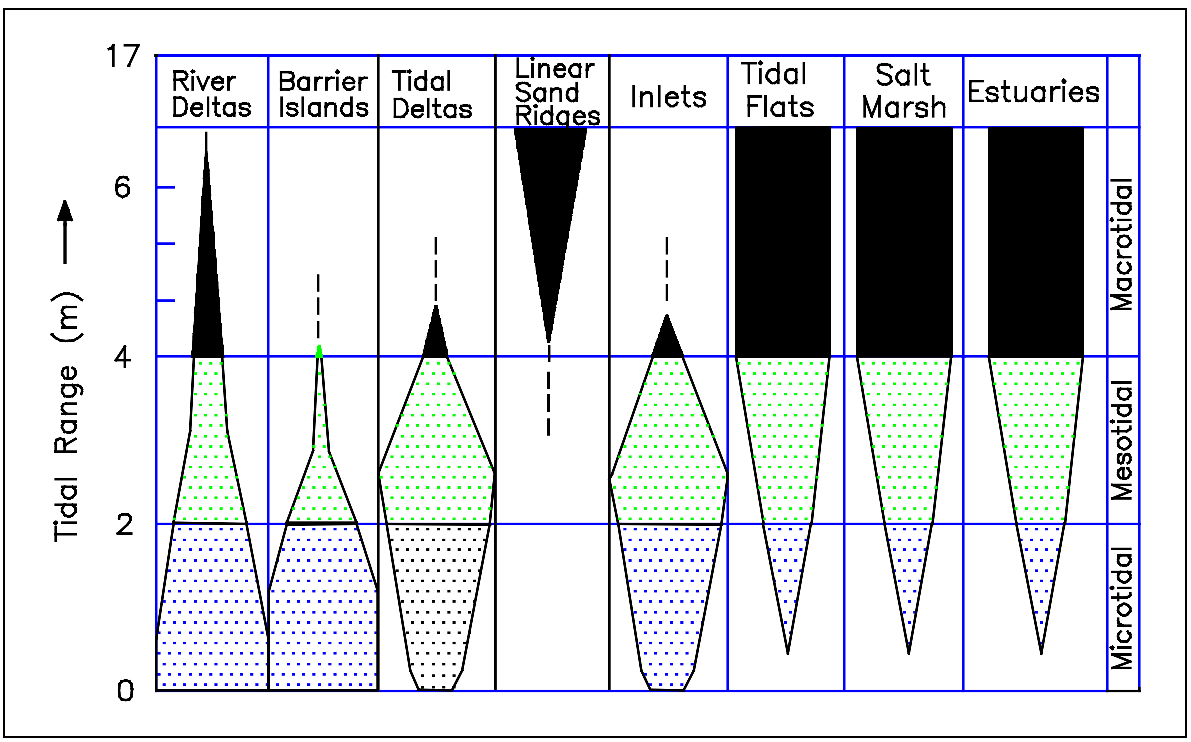


Figure 2-6. Distribution of shoreline types with respect to tidal range (after Hayes (1975, 1979))

major shoreline morphologies, including barrier islands and tidal inlets, relative to tidal range. From this association, it can be seen that tidal inlets and tidal deltas are almost exclusively restricted to the microtidal and mesotidal ranges, and are inversely proportional to the density of barrier islands within these ranges. In microtidal areas, barrier islands are laterally extensive and tidal inlets are infrequent (Figure 2-7a); however, in mesotidal areas, barrier islands are shorter and occur in association with more numerous tidal inlets and deltas (Figure 2-7b). A summary of the shoreline morphologic characteristics that could be expected for each of the three coastal types as classified by tidal range are presented below:

(a) Microtidal.

- Long, linear barrier islands.
- Frequent storm washover terraces.
- Infrequent tidal inlets.
- Poorly developed ebb-tidal deltas.
- Well-developed flood-tidal deltas.

(b) Mesotidal.

- Short, stunted (drumstick) barrier islands.
- Numerous tidal inlets.
- Well-developed ebb-tidal deltas.
- Poorly developed or absent flood-tidal deltas.
- Downdrift offset configuration related to wave refraction around large ebb-tidal deltas.

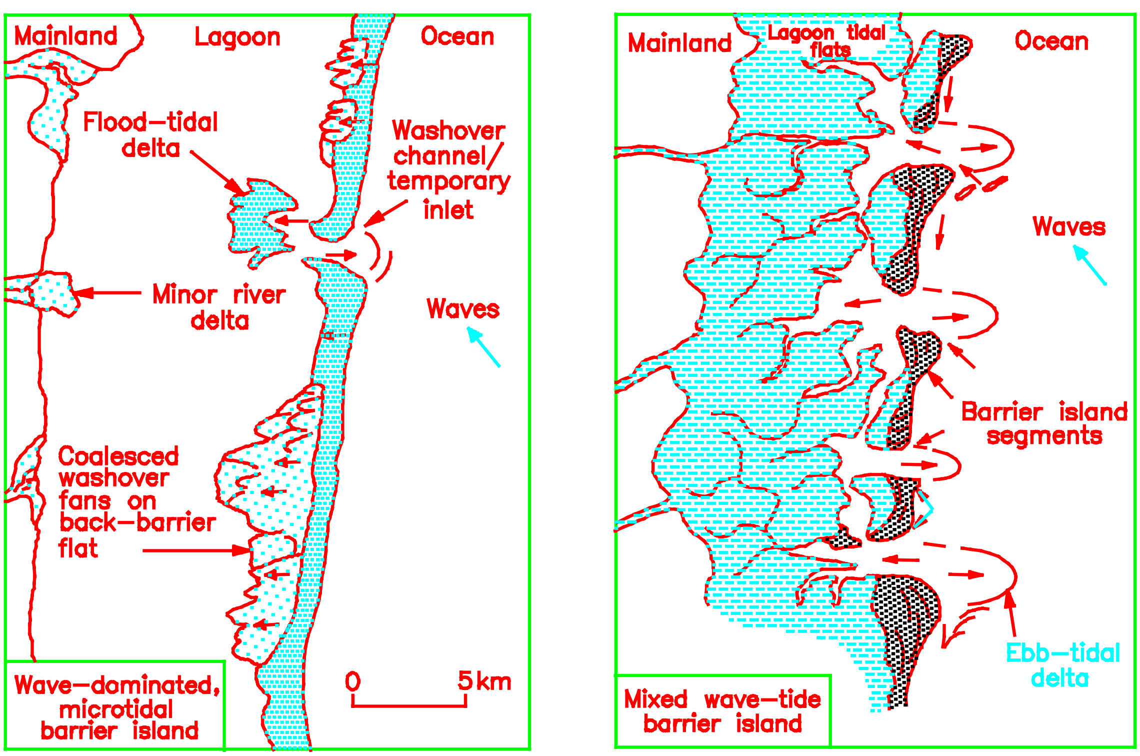
(c) Macrotidal.

- Barrier islands absent.
- Well-developed tidal flats and salt marshes.
- Depositional features in the form of linear sand shoals or tidal current ridges.
- Funnel-shaped embayments.

Through a comparison of mean wave height and mean tidal range, Hayes (1979) modified the Davies classification as indicated in Table 2-3.

Table 2-3
Coastal Types - Medium Wave Energy (H = 60-150 cm)

Class	Tidal Range (m)	Example
Microtidal	0 - 1	Gulf of St. Lawrence
Low-mesotidal	1 - 2	New Jersey
High-mesotidal	2 - 3.5	Plum Island, Mass.
Low-macrotidal	3.5 - 5	German Bight
Macrotidal	>5	Bristol Bay, Alaska



(a) Microtidal **(b) Mixed-tide, mesotidal**

Figure 2-7. A wave-dominated barrier island model (after Hayes (1979))

(3) Each of these classifications carries connotations of a range of shoreline morphologies that could be formed given an adequate sediment supply. Most of the tidal inlets in the United States fall into the microtidal and mesotidal categories. Hubbard, Barwis, and Nummedal (1977) cataloged 27 inlet-lagoon systems between North Carolina and Florida to determine the relative geometries of inlet and lagoon features. The conclusions of this study, based on the southeast coast of the United States, were that the wave-dominated (microtidal) inlets of North Carolina and Florida have small ebb-tidal deltas close to shore, wide throats with multiple sand bodies, and significant inner shoals (Figure 2-8a). Tide-dominated inlets (mesotidal) typically have large ebb-tidal deltas extending offshore, well-defined deep main channels and throats, and few inner shoals (Figure 2-8b). Georgia and South Carolina have an increased tidal range due to shoaling of the tide over the broad shallow offshore shelf. Tide-dominated inlets are common to these coasts of Georgia and South Carolina.

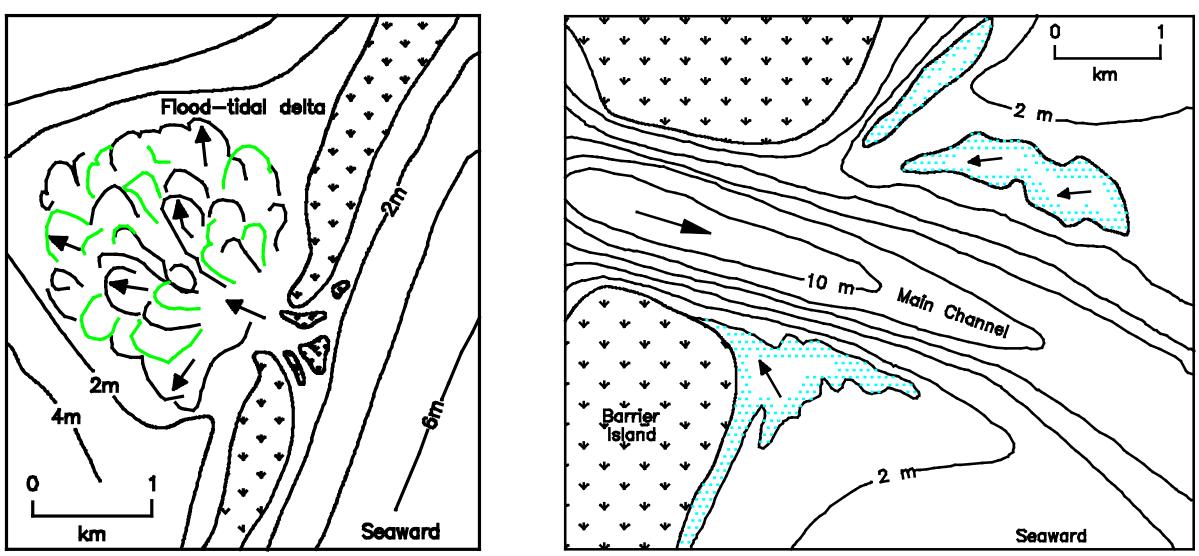
(4) The classification developed by Hayes (1979) has been accepted as a standard. The morphology characteristic of each of the tidal regimes is of a scale sufficient to be well-represented on historical maps and charts. Consequently, semi-quantitative estimates of the tidal regime controlling a region can be made for discrete times in the past, and the development of an inlet system can be traced and predicted.

2-4. Morphology and Processes

Many studies in coastal geomorphology and engineering have focused on the modes of tidal inlet formation, morphology, and migration. This section presents an overview of geomorphic models for tidal inlets, their processes of formation, and relevance to migration and behavior. Hayes (1967) and Pierce (1970) documented inlet formation by the seaward return of storm-surge breaching narrow areas along a barrier island. In this manner, narrow, shallow, ephemeral inlets form during hurricanes and migrate in a downdrift direction. If the tidal prism is unable to maintain these hurricane-generated inlets, landward swash-bar migration and overwash seal the inlet mouth. Inlets whose origin can be attributed to storm processes generally occur in microtidal (wave-dominated) settings. Price and Parker (1979) and Tye (1984) allude to paleotopographic control in the formation of tidal inlets and suggest that, during the Holocene transgression, tidal inlets were concentrated in, and confined to, Pleistocene estuaries and fluvial channels. Although documented examples are few, those inlets whose formation was strongly controlled by paleotopography most commonly occur on mesotidal (tide-dominated) shorelines.

a. Geomorphic models.

(1) The interaction of wave regime and tidal range has a profound effect, not only on the morphology, but also on the migration and behavior of tidal inlets and barrier islands. In microtidal settings, long, narrow, wave-dominated barriers extend for tens of kilometers and are separated by ephemeral, rapidly migrating tidal inlets. The associated flood-tidal deltas, deposited by waves and tidal currents, form large, lobate sand bodies in the lagoon. Wave energy and flood-tidal currents exert more influence on sedimentation than ebb currents; therefore, ebb-tidal delta development is poor. Wave-dominated



(a) Wave-dominated **(b) Tide-dominated**

Figure 2-8. Tidal inlets (after Hubbard, Barwis, and Nummedal (1977))

inlets migrate laterally along the shoreline in a downdrift direction for many kilometers and at relatively rapid rates. As the hydraulic efficiency of the inlet decreases, wave-reworked ebb-tidal delta sands accumulate in the inlet mouth, resulting in closure of the inlet channel and abandonment of the flood-tidal delta. Fisher (1962) identified positions of former tidal inlets in the Cape Lookout area of North Carolina by locating vegetated relict flood-tidal deltas attached to the landward side of the barriers (Figure 2-9).

(2) Unlike wave-dominated coasts, tidally influenced mesotidal barriers often assume a stunted, drumstick-shaped configuration. These barriers are wider, extend for several kilometers, and are separated by numerous, more stable tidal inlets. The backbarrier lagoon and flood-tidal delta of the wave-dominated shoreline are replaced by salt marsh and tidal creeks. Tidal current dominance over wave energy helps to confine these inlets, restricting downdrift migration to less than 2 km (1.2 miles). Tidally influenced inlet channels are deflected downdrift by preferential addition of sand to the updrift lobe of the ebb-tidal delta. These inlet channels lose hydraulic efficiency and breach the barrier to form a shorter updrift channel. Large sediment lobes are reworked from the former ebb-tidal delta and eventually weld onto the barrier, closing the earlier inlet channel (Figure 2-10). Landward, out of the influence of wave transport, silt and clay accumulate in the former channel due to the absence of strong tidal currents.

b. Inlet migration processes. Many tidal inlets naturally migrate alongshore in the direction of net longshore drift. The rate and mechanisms of inlet migration vary depending on several factors including wave climate, tidal range, depth of the main channel, nature of the substrate into which the channel is incised, sediment supply, and rate of longshore sediment transport. Rates of migration can be highly variable. Measured examples include 90 m/year (300 ft/year) for Nauset Inlet, Massachusetts; 60 m/year (200 ft/year) for Fire Island Inlet, New York; 40 m/year (130 ft/year) for Captain Sam's Inlet, South Carolina; and 2 m/year (7 ft/year) for Sandy Neck spit, Massachusetts (Hayes 1980).

c. Natural sediment bypassing.

(1) Inlet sediment bypassing is the transport of sediment (sand) from the updrift to the downdrift margin of the tidal inlet. This process is fundamental to understanding and predicting shoreline erosion and deposition trends

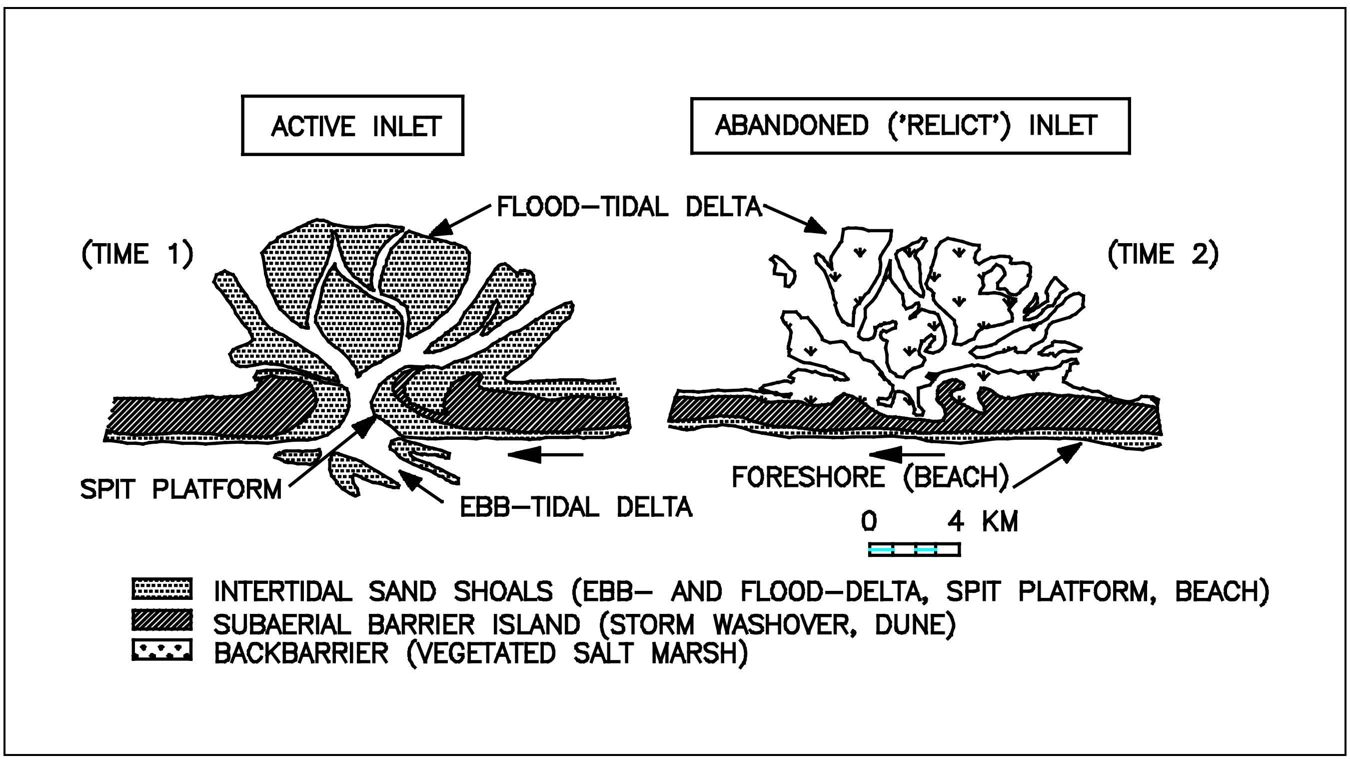


Figure 2-9. Morphologic evolution of wave-dominated inlet-related sand bodies (modified from Fisher (1962) in Moslow and Tye (1985))

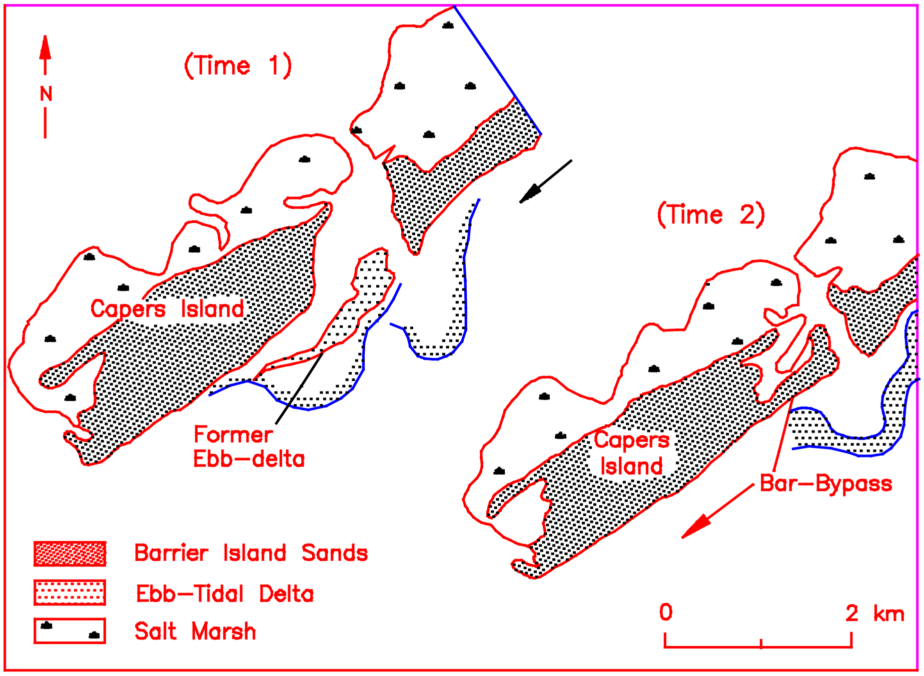


Figure 2-10. Diagram illustrating the bar-bypass mechanism for inlet channel abandonment (from Moslow and Tye (1985))

in areas adjacent to the tidal inlet. Bruun and Gerritsen (1959) first described the natural mechanisms of inlet sediment bypassing and related the variables involved in this process using the equation:

$$r = M_{mean} / Q_{max}$$

where r equals the ratio between the average rate of longshore sediment transport to the inlet M_{mean} and the maximum discharge to the inlet during spring tidal conditions Q_{max}. In so doing, Bruun and Gerritsen defined three methods by which sand "bypasses" tidal inlets: (a) by wave-induced sand transport along the outer margin of the ebb-delta (terminal lobe), (b) through transport of sand by tidal currents in channels, and (c) by the migration and accretion of sandbars and tidal channels. They concluded that inlets with high ratios (r = 200 to 300) bypass sand along the terminal lobe, while inlets with low ratios (r = 10 to 20) bypass sand through methods (b) and (c). Subsequent field studies by geologists refined the original concepts of Bruun and Gerritsen. FitzGerald (1988) proposed three models to summarize the mechanisms of tidal inlet migration through sediment bypassing on microtidal and mesotidal coasts. These models are shown in Figure 2-11 and are summarized below.

(2) Extensive and/or rapid channel migration is generally associated with relatively shallow tidal inlets (Model 1, Figure 2-11) while deeper tidal inlets are less likely to migrate as they have a greater probability of incising into semi-consolidated sediments (Models 2 and 3, Figure 2-11). In an historical analysis of tidal inlets along the South Carolina coast, FitzGerald, Hubbard, and Nummedal (1978) documented that inlets deeper than 8 m (26 ft) had been stable for the previous 100 years while those inlets shallower than 3 to 4 m (10 to 13 ft) were associated with extensive inlet migration and spit breaching. Migration and spit breaching are more common to wave-dominated tidal inlets. However, if the tidal prism is small enough, or where the backbarrier is infilled with salt marsh and tidal flats, as is characteristic of mesotidal shorelines, migration of the inlet often results in a shore-parallel elongation of the inlet channel (Model 1, Figure 2-11). In such cases, a storm or hurricane will breach the updrift part of the spit and establish a more hydraulically efficient inlet. A classic example of an inlet that has experienced several episodes of migration and breaching is the Kiawah River Inlet in South Carolina.

(3) Stable inlets have an inlet throat and main ebb channel that do not migrate. Migration in these inlets can

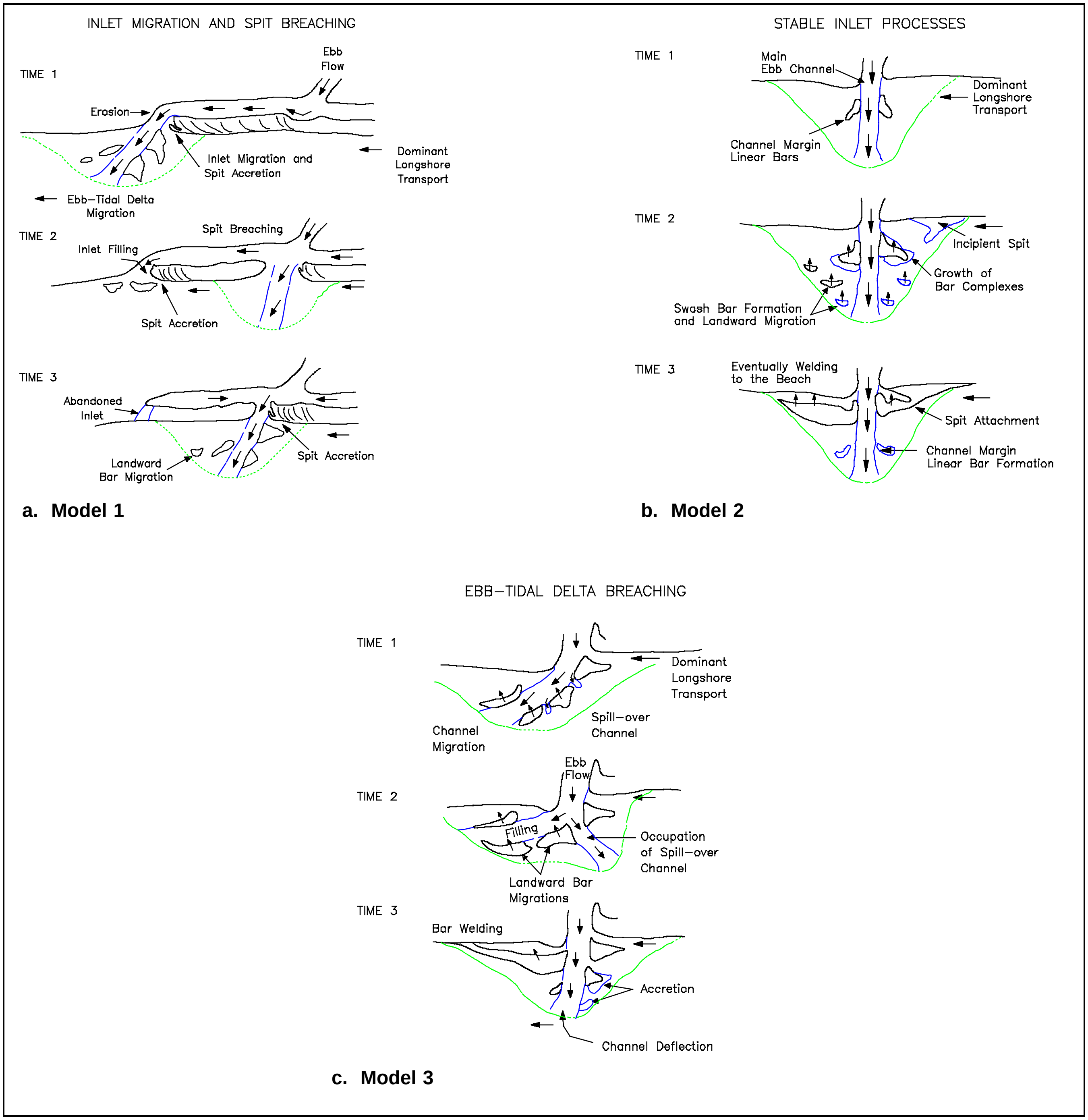


Figure 2-11. Models of inlet sediment bypassing for mixed energy mesotidal coasts (after FitzGerald, Hubbard, and Nummedal (1978))

be restricted by antecedent topography or incision into semi-consolidated material. Inlets of this type can occur in any hydrographic setting but are more commonly associated with bedrock-controlled inlets or those that can attribute their origin to ancestral distributary channels. Sand bypassing at these inlets occurs through the landward migration and accretion of large bar complexes to the downdrift margin (Hine 1975) (Model 2, Figure 2-11).

(4) Ebb-tidal delta breaching through bar-bypassing is the major process of channel migration and

abandonment at tide-dominated inlets (Figure 2-10). Tidal inlets associated with this process have a stable inlet throat, but the main ebb channel migrates or "pivots" with time (Model 3, Figure 2-11). Migration occurs through downdrift over-extension of the main ebb channel and subsequent breaching of a shorter updrift channel. Ebb-delta breaching and bar bypassing occur rapidly at smaller inlets (Sexton and Hayes 1982), but one cycle of inlet channel migration and abandonment took over 100 years at Capers, Price, and Stono Inlets in South Carolina (Tye 1984).

d. Tidal deltas.

(1) The overall morphology of ebb- and flood-tidal deltas is a function of the interaction of tidal currents and waves. Especially important is the phenomenon of time-velocity asymmetry of tidal currents. As described by Postma (1967), maximum ebb- and flood-tidal current velocities do not occur at mid-tide. Of critical significance is that maximum ebb currents typically occur late in the tidal cycle, near low water. This means that at low water, as the tide turns, strong currents are still flowing seaward out of the main ebb channel. As water level rises, flood currents seek the paths of least resistance around the margin of the delta. This process creates the horizontal segregation of flood and ebb currents in the tidal channels that ultimately molds and shapes the tidal deltas. The segregation of tidal flow around and through flood- and ebb-tidal deltas is shown in Figures 2-2 and 2-3. A typical tidal current time-velocity curve for an ebb-tidal delta is shown in Figure 2-12.

(2) Variations in flood- and ebb-tidal delta morphology are a function of tidal prism, backbarrier morphology, and relative wave energy. Figure 2-13 illustrates variations for three areas: Texas, South Carolina, and New England. South Carolina ebb deltas are generally more elongate and more ebb-dominated than those in New England because of less wave energy and a larger tidal prism. In Georgia, the increased size of ebb-tidal deltas and the near absence of flood-tidal deltas can be attributed to two factors: large tidal range and small waves, which enhance the tide dominance of the inlets; and the ratio of open water to marsh in the estuaries is such that inlet flow is ebb-dominant (i.e., peak and mean ebb velocities exceed those for flood (Nummedal et al. 1977).

(3) Patterns of sand transport on tidal deltas are quite complex and very difficult to measure in the field. However, an excellent documentation of sand transport patterns for the ebb- and flood-tidal deltas of Chatham Harbor, Massachusetts, was performed by Hine (1975) (Figure 2-14) and by Imperato, Sexton, and Hayes (1988) for Edisto Inlet, South Carolina (Figure 2-15). Note that Chatham Harbor presently does not have the configuration shown in Figure 2-14, because of major hydrodynamic and morphological evolution caused by creation of a new inlet through Nauset Beach (northern barrier island in Figure 2-14) which was breached on January 2, 1987 (Liu et al. 1993). Both Hine and Imperato, Sexton, and Hayes mapped tide- and wave-generated current transport pathways and concluded that proximal parts of the deltas (main ebb and flood marginal channels) are dominated by

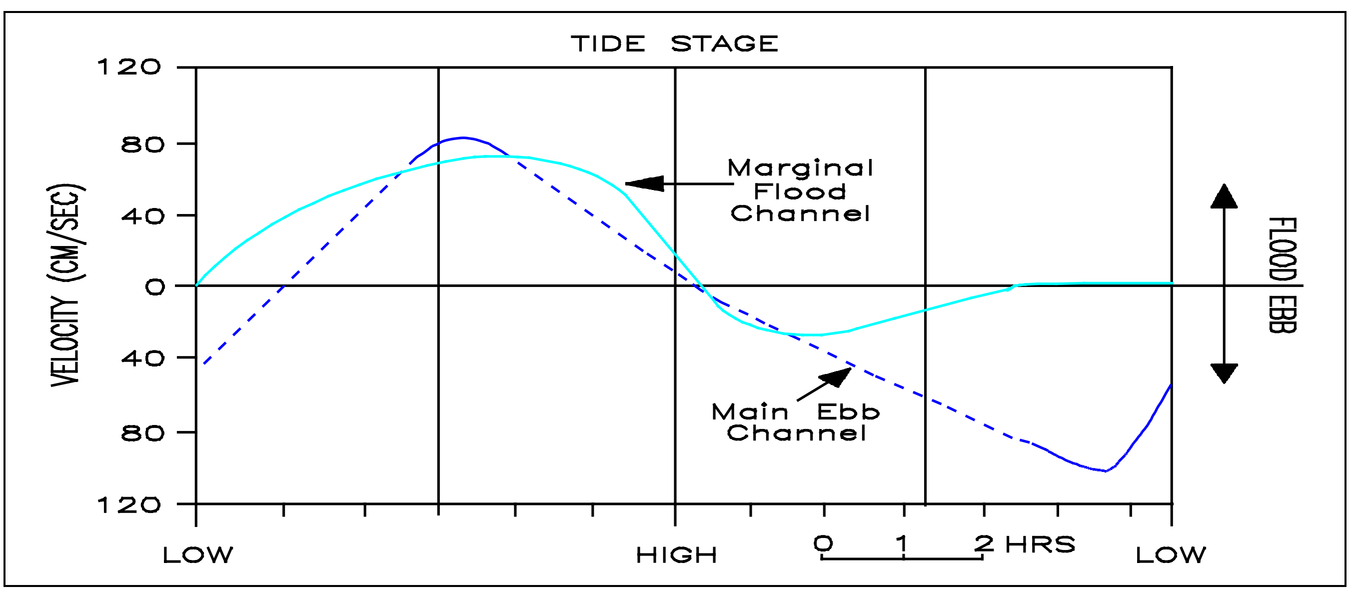


Figure 2-12. Typical tidal current time-velocity curves for an ebb-tidal delta (from Hayes (1980))

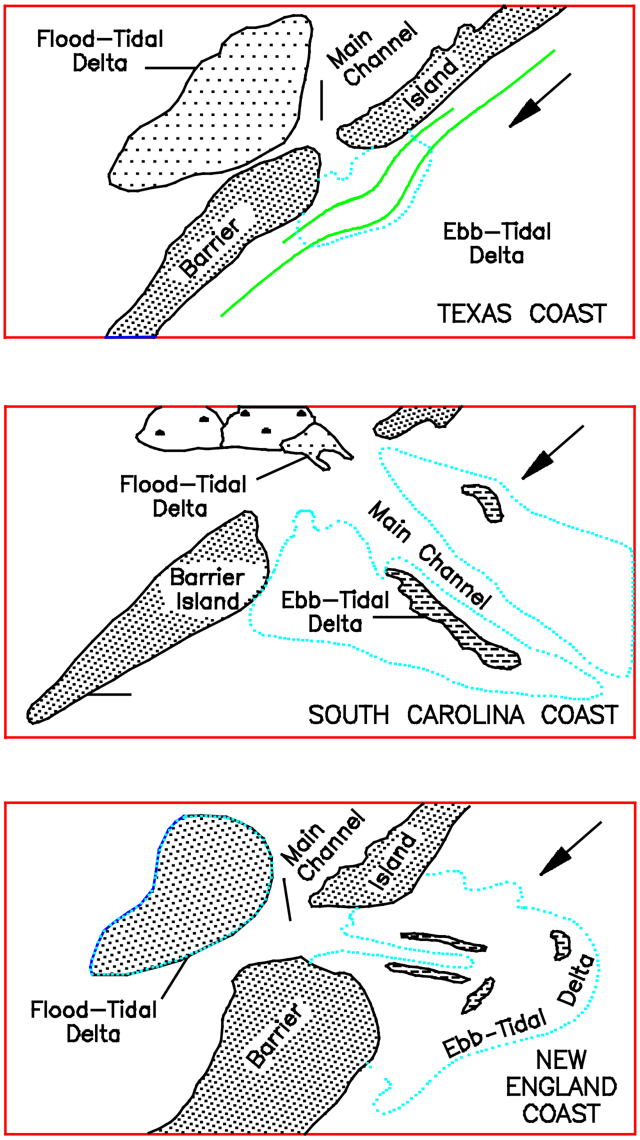


Figure 2-13. Tidal delta variations for the shorelines of Texas, South Carolina, and New England (after Hayes (1980))

tidal currents, whereas distal portions (periphery) are dominated by wave-(swash) generated currents.

2-5. Sedimentation and Stratigraphy

a. General. Sediments associated with relict (abandoned) tidal inlets can serve as large reservoirs of sand for beach nourishment projects. Thus, an understanding of the geometry and three-dimensional variability of inlet deposits can be extremely beneficial in identifying proper sources and volumes of nourishment material. From their analyses of Holocene sediments in vibracore, wash bore, and auger drill holes from coastal North and South Carolina, Moslow and Tye (1985) demonstrated sharply contrasting sedimentary sequences and stratigraphy between wave- and tide-dominated inlet-fill deposits. This variation in inlet sequences is primarily a function of the antipathetic relationship between wave height and tidal range. In addition to hydrographic regime, antecedent topography and sediment supply are important factors in determining the sedimentologic nature of tidal inlet sequences.

b. Sedimentation. Shore-parallel lateral migration of tidal inlet channels erodes and redeposits significant portions of the adjacent shoreline. In addition, tidal inlet channels and tidal deltas serve as natural sinks for sediment and can through time, acquire very large dimensions. Numerous coring investigations in microtidal and mesotidal settings on the U.S. east and gulf coasts have documented that an average of 30 to 50 percent of barrier shoreline deposits can be attributed to tidal inlet sedimentation.

c. Vertical sedimentary sequences.

(1) Wave-dominated tidal inlets.

(a) Sediments deposited in wave-dominated tidal inlets form distinct fining-upward channel deposits of fine- to coarse-grained, moderately sorted, quartzose sand and shell. One depositional cycle of wave-dominated inlet fill as observed in cores consists of coarse-grained inlet floor lag deposits overlain by the active inlet channel and capped by spit platform or overwash sands. Vertical sequences at Johnson Creek (Figure 2-16) and Beaufort Inlet (Figure 2-17), North Carolina illustrate this gradually fining upward sand deposit and are presented here as being characteristic of most wave-dominated inlet sequences. Differing vertical sequences evident at these inlets are a function of paleotopographic control, tidal prism, and sediment supply.

(b) Structureless coarse shell and gravel lag deposits of the inlet floor are found at the base of Johnson Creek and Beaufort Inlet sequences. Inlet floor deposits consist of relatively clean, coarse sand with thick, highly fragmented, and abraded shells. Cored inlet floor deposits range from 0.3 to 0.6 m in thickness. Coarse-grained shell material, sand, and pebbles were deposited as a lag in the deeper portions of the active inlet channel where current velocities are greatest during tidal exchange. Similar deposits have been sampled on the bottom of modern tidal inlet channels in the Cape Lookout area.

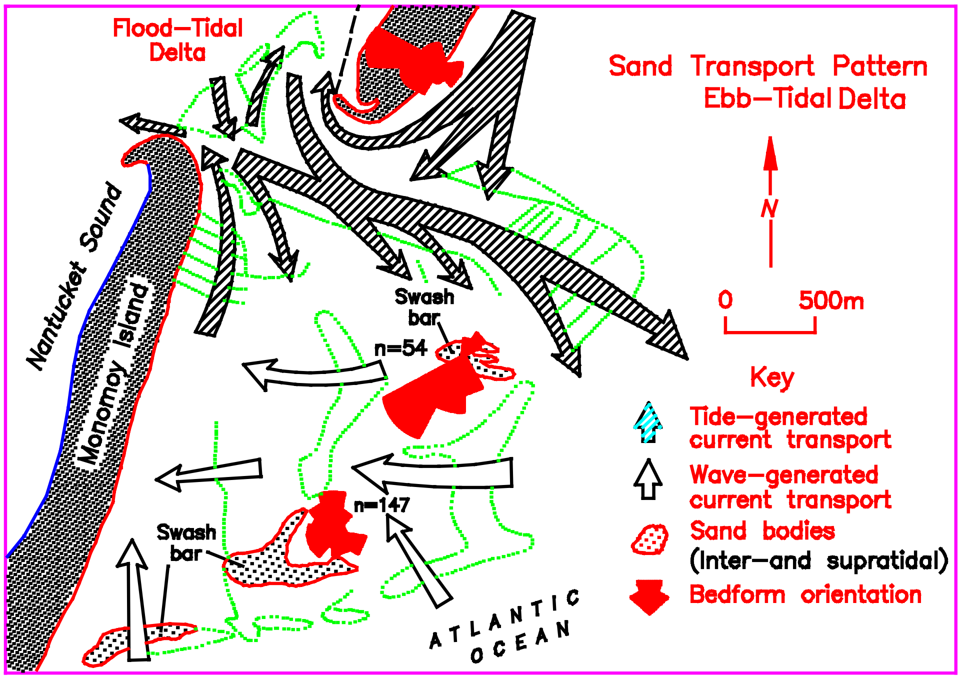


Figure 2-14. Net sand transport pattern on the ebb-tidal delta of Chatham Harbor, Massachusetts (from Hine (1975))

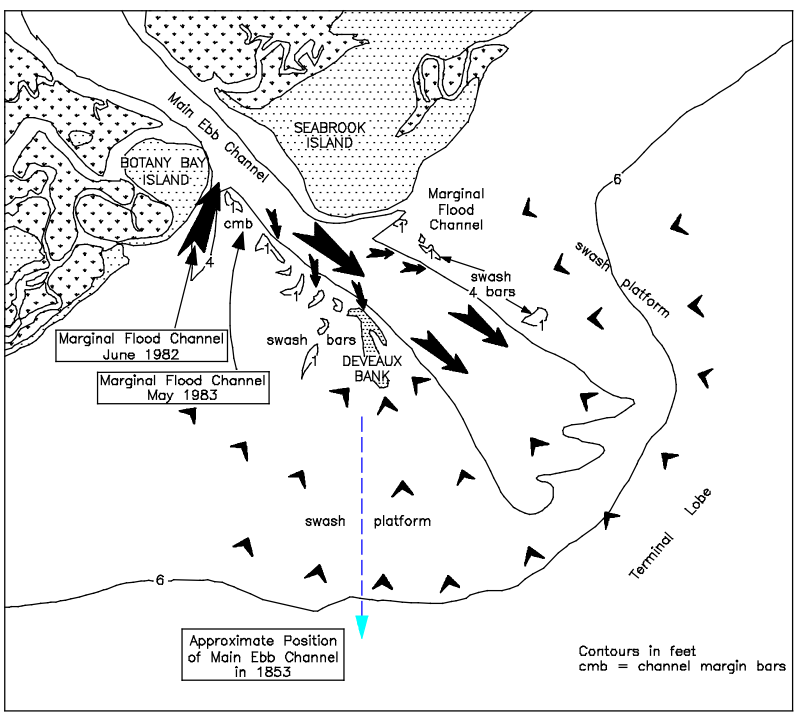


Figure 2-15. Orientation of bed forms and patterns of sand transport at North Edisto Inlet, South Carolina (from Imperato, Sexton, and Hayes (1988))

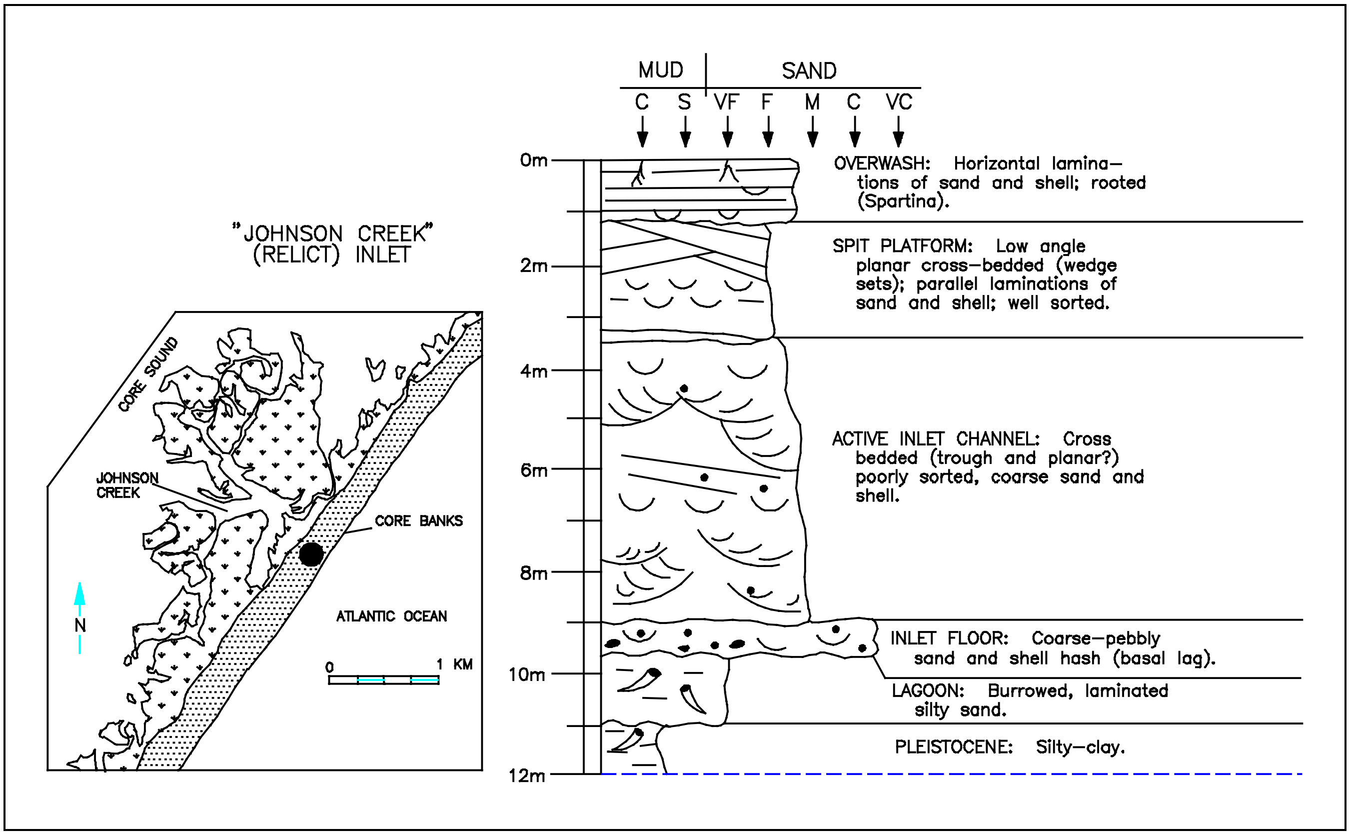


Figure 2-16. Fining-upward wave-dominated inlet sequence in the vicinity of Johnson Creek, Core Banks, North Carolina (after Moslow and Tye (1985))

(c) The Johnson Creek and Beaufort Inlet sequences are in sharp, erosional contact with underlying dense, compact Holocene and Pleistocene muds. The pebbly gravel and shell lag of the inlet floor facies grades upward into coarse- to fine-grained active inlet channel sand. The active inlet channel is the thickest sedimentary unit within wave-dominated inlet sequences and is up to 6.5 m (21 ft) at Johnson Creek (Figure 2-16). The depth of inlet scour and reworking of earlier channel deposits by a later inlet scour event accounts for the large range in channel thicknesses. Active inlet channel sand is quartzitic, often pebbly, and can contain an abundance of broken and abraded shells. Size analyses of the active inlet channel sediments indicate a generally fining-upward trend in grain size.

(d) Fine- to medium-grained overwash (Johnson Creek, Figure 2-16), and very fine- to fine-grained spit platform and dune sands (Beaufort Inlet, Figure 2-17) cap the wave-dominated inlet sequences.

(2) Tide-dominated tidal inlets.

(a) Two distinct and predictable sedimentary sequences are deposited by tide-dominated inlet channels. The first of these is formed by the landward migration of large intertidal/supratidal sand ridges (bar-bypass mechanism; Figure 2-10) that partially closed the inlet channel mouths at Price, Capers, and Stono Inlets. Inlet bar-bypass created 8- to 12-m-thick (26- to 39-ft-thick), sand-rich, fining-upward deposits of inlet floor, active inlet channel, and swash platform (ebb-tidal delta) overlain by foreshore and dune (Figure 2-18). Landward of the welded swash bar, cored inlet sediments on Capers Island reveal a fining upward mud-rich sequence of active inlet channel and abandoned inlet channel overlain by tidal creek and salt marsh deposits (Figure 2-19). Maximum sand thicknesses (combined inlet floor, inlet channel, and spit platform deposits) occur at the seaward extent of the abandoned channel and interfinger landward with mud-dominated, abandoned inlet channel deposits.

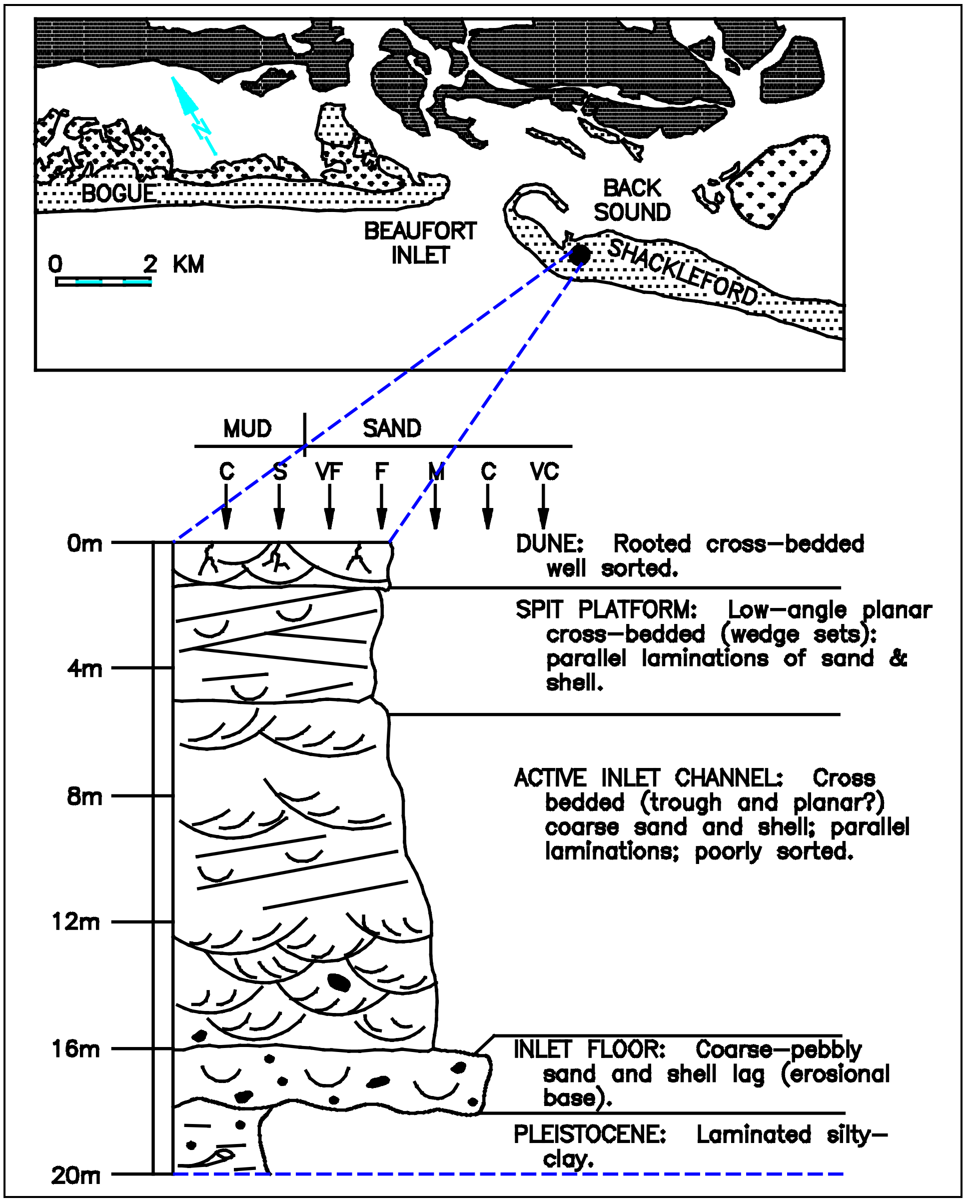


Figure 2-17. Tidal inlet sequence deposited by the lateral migration of Beaufort Inlet (after Heron et al. (1984))

(b) Sedimentary characteristics (lithology and bedding) of the sand-rich, tide-dominated deposits resemble wave-dominated inlet deposits discussed earlier. A basal shell and pebble lag (inlet floor) is scoured into Pleistocene sediments. Fine- to medium-grained inlet channel sand gradationally overlies the inlet floor. Active channel deposits range from 2 to 4 m (6 to 13 ft) thick, and fine upwards into fine-grained inlet margin and swash platform sands (Figure 2-18).

(c) Figure 2-19 illustrates the second (mud-dominated) sequence of inlet deposits from abandoned channels at tide-dominated inlets. An updip lithologic change from sand to mud occurs because swash bars closed the inlet mouths, decreasing and ultimately terminating tidal flow through the channels. After inlet closure, silt, clay, and rafted organic debris formed a dense clay plug above the active inlet sand. This clay plug

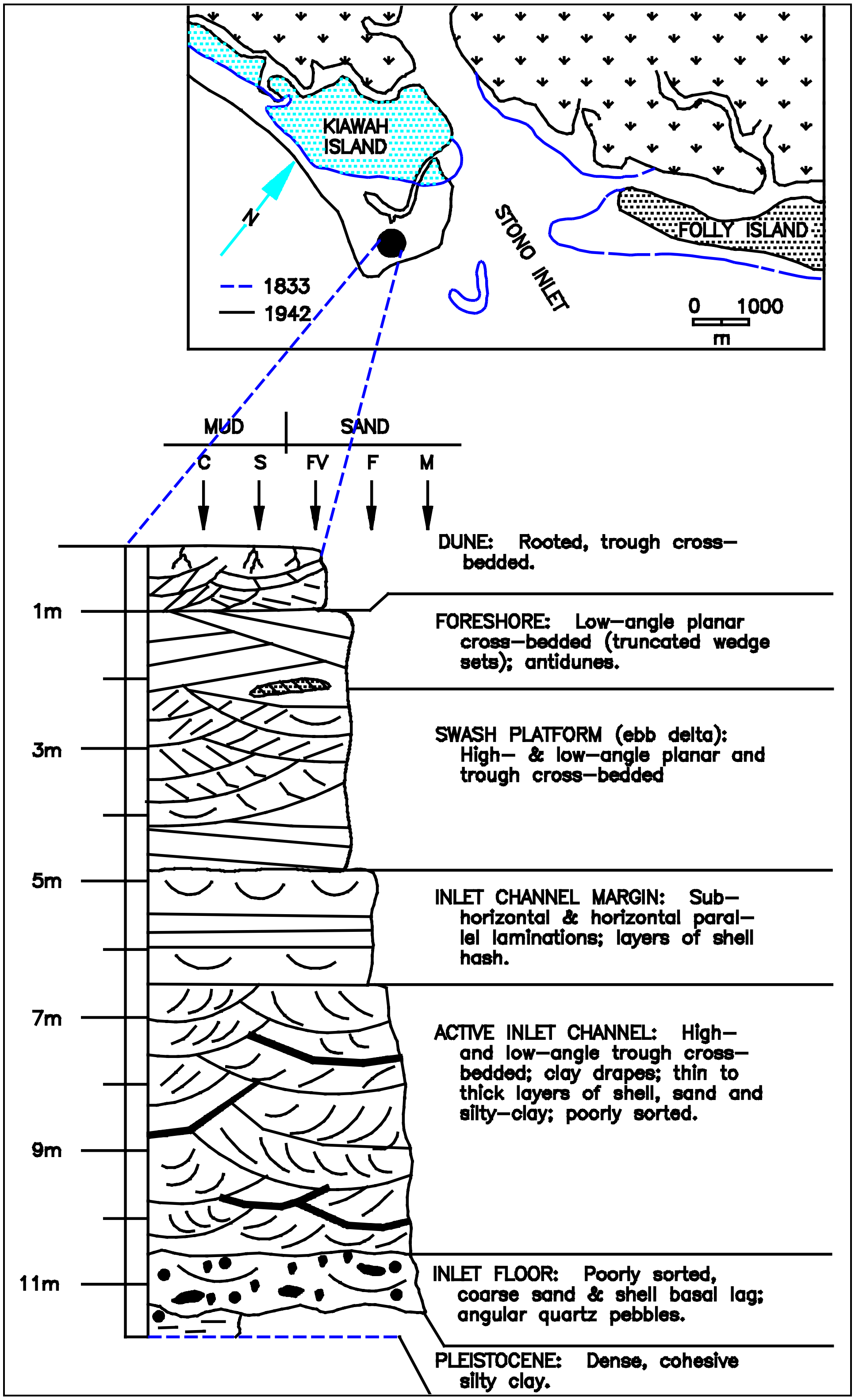


Figure 2-18. Tide-dominated inlet sequence cored beneath the updrift end of Kiawah Island (after Moslow and Tye (1985))

thickens from 1.0 m (3 ft) at its seawardmost extent to 4.0 m (13 ft) at the inlet throat.

(d) Small tidal creeks scoured into and reworked portions of the abandoned inlet fill while depositing thin, discontinuous, poorly sorted tidal creek sand lenses. Subsequent tidal creek abandonment resulted in deposition of a small clay plug capping the tidal creek deposits. Figure 2-19 illustrates this vertical sequence of active inlet channel, abandoned inlet channel, and active and inactive tidal creek capped by salt marsh.

d. Tidal inlet fill stratigraphy.

(1) Wave-dominated.

(a) Stratigraphic cross sections of abandoned wave-dominated inlet channels are lenticular to wedge-shaped when viewed parallel to the shoreline. Active and relict channels display obvious cutbank (erosional) and accretional margins, revealing the direction of migration. An associated recurved spit comprised the accreting margin and fills the inlet channel as it migrates.

(b) Once abandoned, the channel fill deposited in shallow wave-dominated inlets is lenticular in cross section. Rapid channel migration and high sediment supply result in thin, laterally continuous sequences of inlet deposited sediment. Deeper tidal inlets, entrenched in the Pleistocene "basement" are generally less laterally extensive and display wedge-shaped shore-parallel geometries. Thickness-to-width ratios reflecting maximum scour depth and lateral migration range from 1:150 for deep channels to 1:500 for shallower channels. The high ratio for shallow tidal inlets is due to the absence of paleotopographic control and rapid downdrift migration.

(c) Captain Sam's Inlet is a shallow, rapidly migrating inlet at the southern terminus of Kiawah Island, South Carolina (Figure 2-20). It illustrates the way in which the balance between waves and tides influences inlet geometry. Although it is located on a mixed-energy tide-dominated shoreline, the combination of a very small tidal prism (4.0 to 6.0 × 10^6 m^3 (140 to 210 × 10^6 ft^3); Sexton and Hayes 1982), constant wave energy, and intermittent storm processes produces a wave-dominated inlet sequence. A 3.5-m-thick (11.5-ft-thick) fining-upward sequence of an active inlet channel, a spit platform, and a dune (Figure 2-20) is scoured into easily eroded shoreface sand. As a result of rapid channel migration and recurved spit growth, this lenticular inlet deposit extends 3.0 km (1.9 miles) downdrift.

(d) Greater channel scour and Pleistocene control at Johnson Creek limited the channel's migration and produced a V-shaped inlet-channel deposit (Figure 2-21). Channel confinement by Pleistocene sediments resulted in a 9.5-m-thick (31.2-ft-thick) wedge of fining-upward deposits preserved within Core Banks. Herbert (1978) described an inlet sequence of similar geometry on Portsmouth Island, North Carolina; however, he observed four separate fining-upward cycles of inlet deposition. Inlet sequences may be stacked or vertically exaggerated by sea level rise, barrier island subsidence, or by the successive filling of the thalweg of an old fluvial channel.

(2) Tide-dominated. The depth of scouring at tide-dominated channels along the South Carolina coast is confined by the Pleistocene substrate. Channels exhibit symmetrical U-shaped shore-parallel geometries. Inlet throat stability and bar bypassing at the channel mouth inhibit extensive lateral migration and thus tidal inlet deposits accumulate in the updrift position of the barrier islands. The strike-oriented cross section at Price Inlet (Capers Island, Figure 2-22) illustrates the U-shaped inlet and the preservation of a concave-upward wedge of inlet-channel sand overlain by fine-grained abandoned-channel deposits. Compared to wave-dominated inlets, more time is required to completely close and fill an abandoned tide-dominated inlet channel. Inlet closure by a landward-migrating swash bar restricts current energy in the former channel and initiates the deposition of a fine-grained abandoned channel-fill plug.

e. Tidal delta stratigraphy.

(1) General. Stratigraphic studies of tidal deltas have been relatively rare. This has been due primarily to the severe logistical constraints imposed by the strong currents and breaking waves inherent to these environments. These daily processes make the positioning and operation of coring equipment difficult to impossible. A few examples of tidal-delta stratigraphic studies do exist, however, and can serve as models for predicting the three-dimensional distribution of sediment textures and lithologies.

(2) Flood-tidal deltas.

(a) The sedimentary deposits of the Back Sound, North Carolina, microtidal estuary/lagoon system have been studied in detail by Berelson and Heron (1985). A number of cores were taken in the intertidal sand flats and active and relict flood-tidal deposits of Beaufort Inlet,

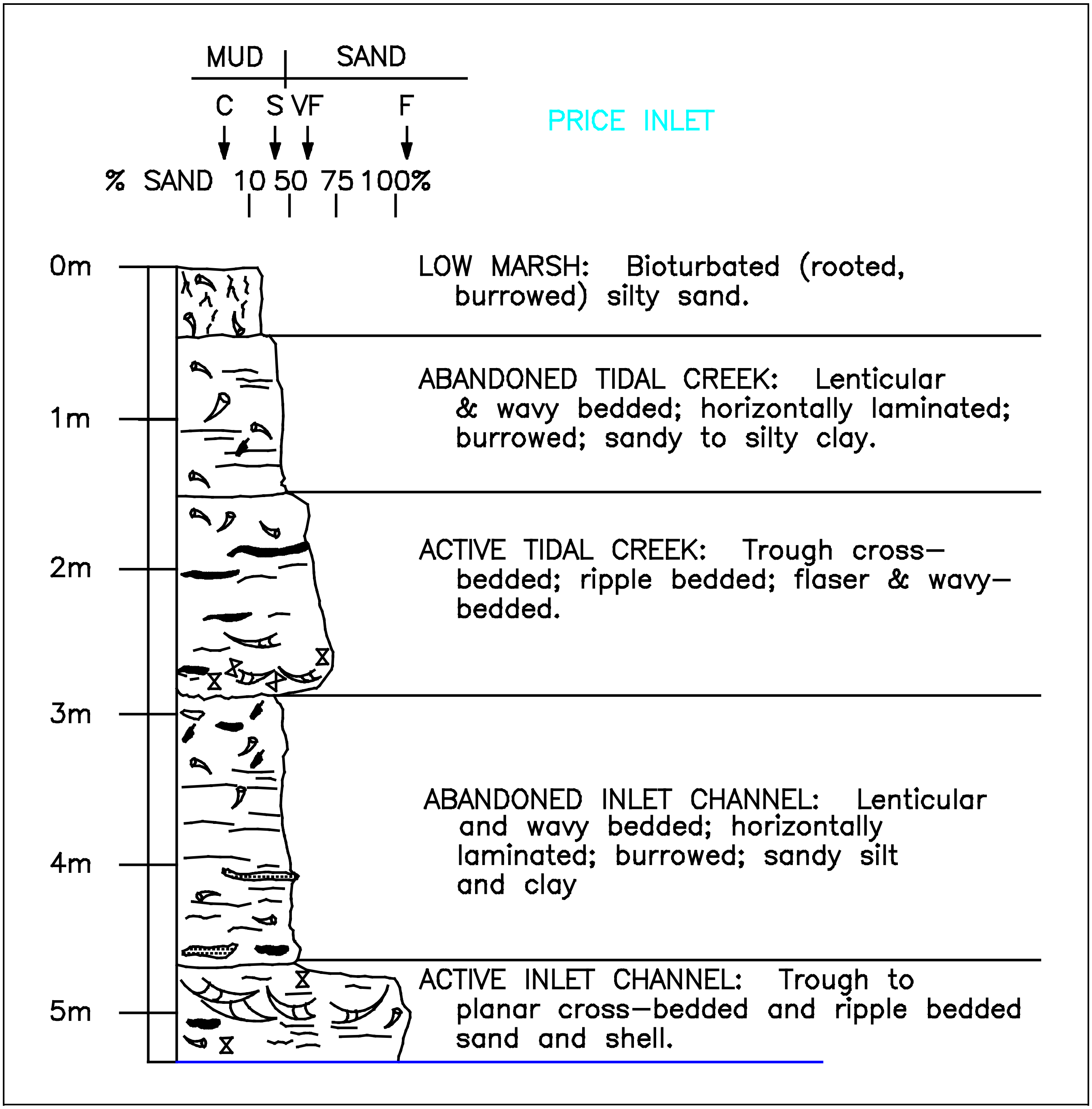


Figure 2-19. Vertical sequence of sediment deposited by the channel abandonment of Price Inlet (after Moslow and Tye (1985))

landward of Shackleford Banks (Figure 2-17). The composite Back Sound flood-tidal delta sequence is comprised of two stacked fining-upward units interbedded and overlain by thin layers of salt-marsh muds (Figure 2-23). Proximal flood-tidal delta sediments are fine- to coarse-grained sands. Distal flood-tidal delta sediments were deposited further from the inlet or active tidal channels and are a fine- to medium-grained silty sand.

(b) Cored sequences in the relict flood-tidal deltas of Back Sound can be correlated to the inlet deposits beneath

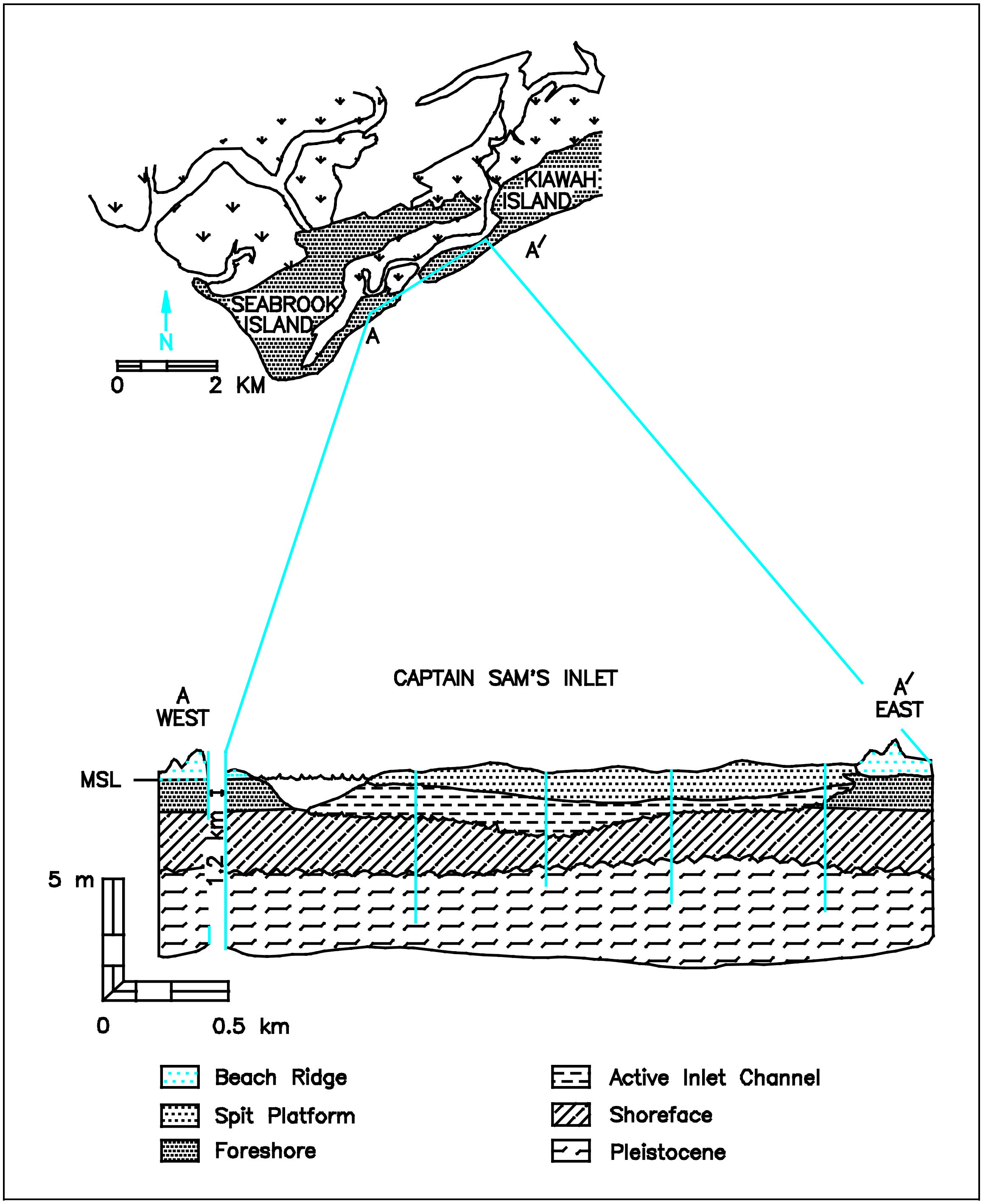


Figure 2-20. Vibracore transect and cross section for inlet-fill deposits at Captain Sam's Inlet, South Carolina (after Tye and Moslow (in preparation))

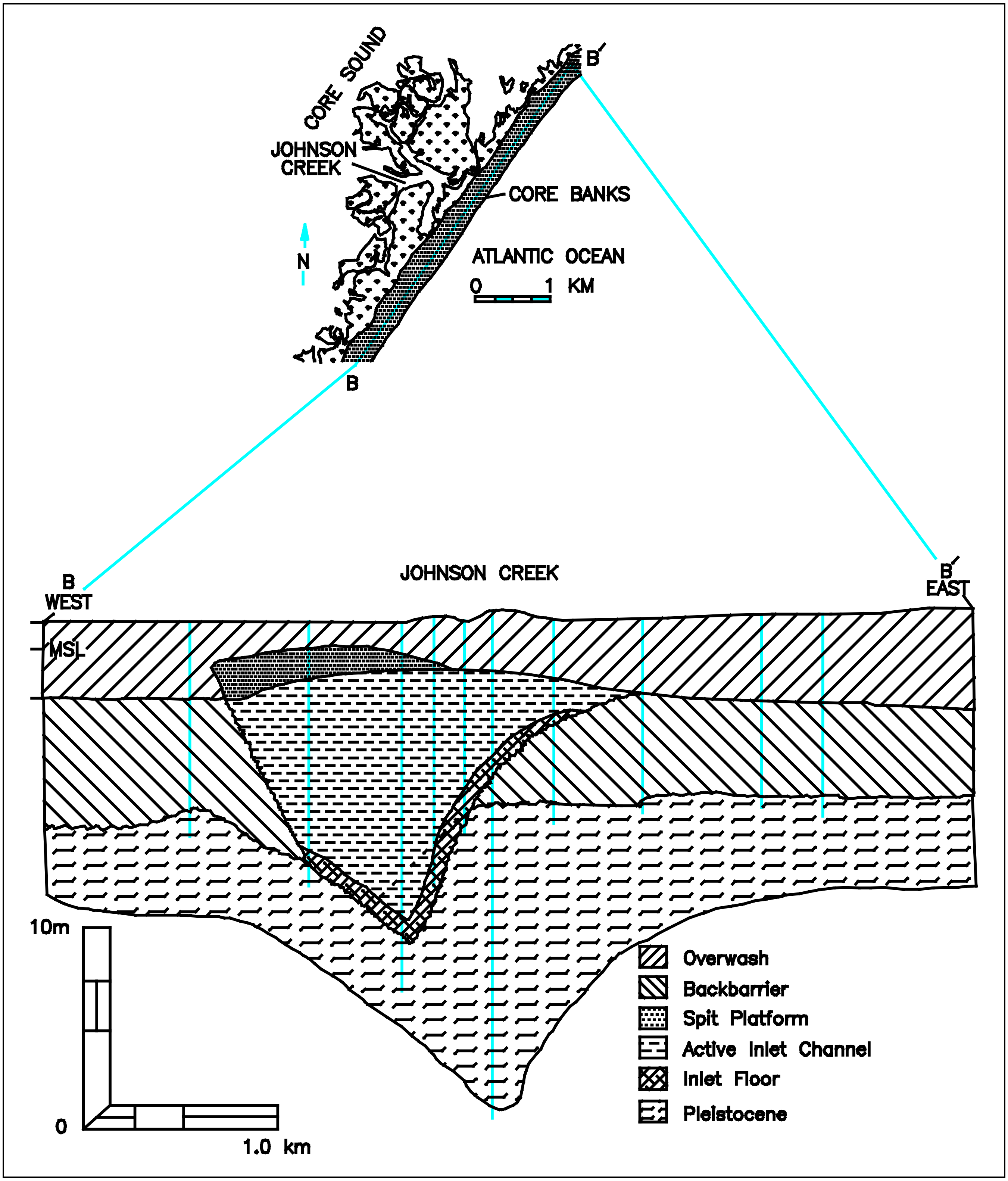


Figure 2-21. Vibracore transect and cross section for inlet-fill deposits at wave-dominated Johnson Creek, Core Banks, North Carolina (after Moslow and Heron (1978))

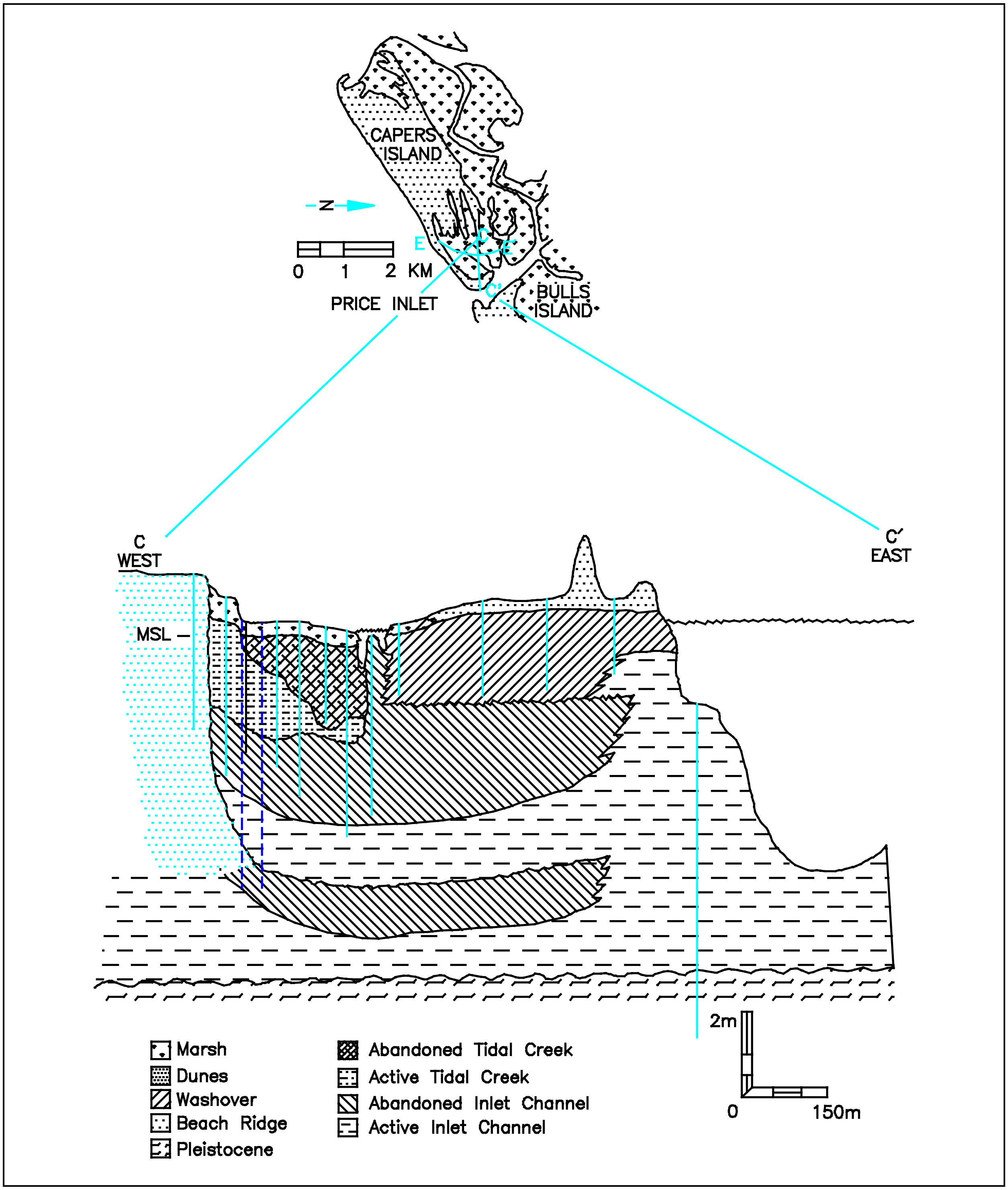


Figure 2-22. Strike-oriented vibracore transect across the abandoned tide-dominated inlet channel at Price Inlet (after Tye and Moslow (in preparation))

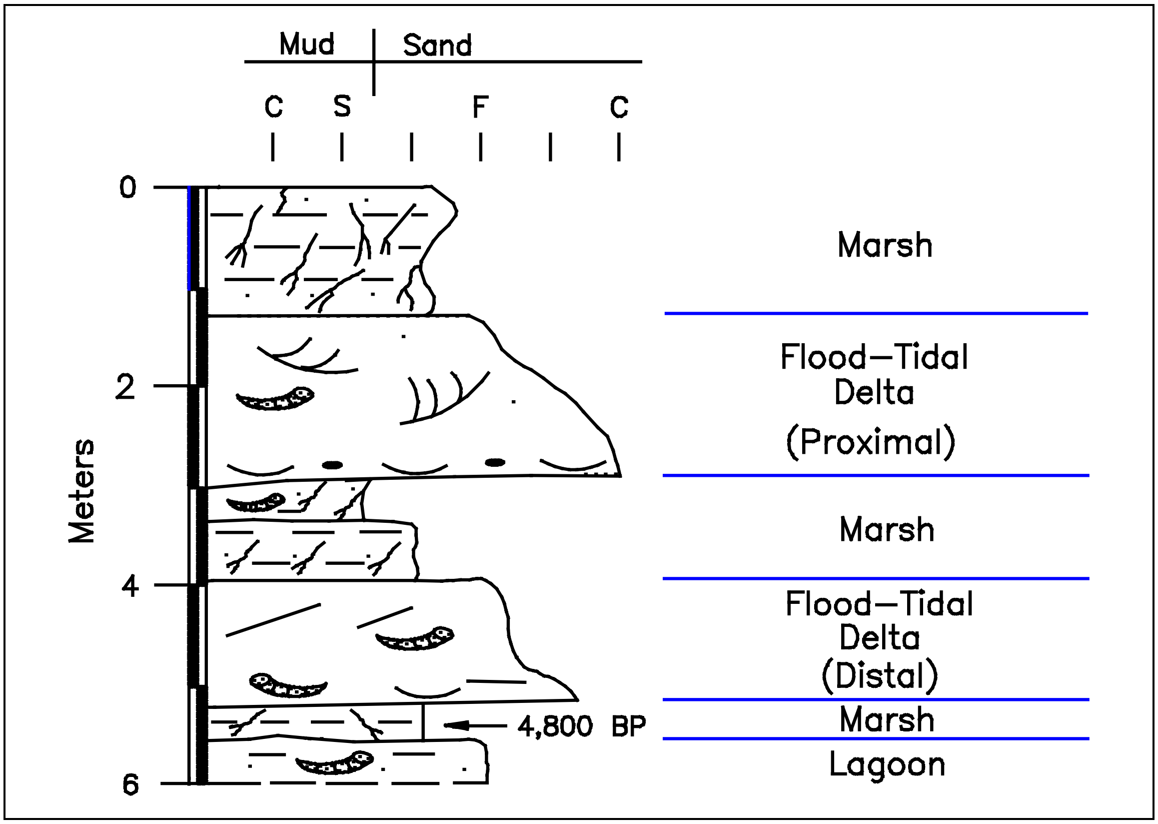


Figure 2-23. Composite vertical sedimentary sequence from an abandoned flood-tidal delta in Back Sound, North Carolina (after Berelson and Heron (1985))

Shackleford Banks. Flood-tidal sand deposits thin in a landward direction and are interbedded with lagoonal and tidal flat muds.

(3) Ebb-tidal deltas. A coring study by Imperato, Sexton, and Hayes (1988) serves as a stratigraphic model for the ebb-tidal delta of a tide-dominated inlet. The ebb-tidal delta sediments at North Edisto Inlet, South Carolina (Figure 2-15) comprise a 5- to 15-m-thick (16- to 49-ft-thick), lobate-shaped body of fine-grained, well-sorted sand with an estimated volume of 1.28×10^8 m^3 (45.2×10^8 ft^3). Ebb delta sedimentary sequences proximal to the shoreline are a fine-grained sand and mud overlying a lag of shell fragments. Distal ebb-delta sedimentary sequences are relatively thin and composed of interbedded well-sorted, fine-grained and coarse-grained sands.

2-6. Sand Resource Potential of Inlet Deposits

a. Inlet channel fill.

(1) Suitability.

(a) In almost all instances, relict tidal inlet deposits represent the most suitable and viable source of sand-sized sediment for beach nourishment projects. As shown in Section 2-5 of this chapter, the lateral and/or vertical accretionary fill within inlet channels (inlet fill) is a generally thick, voluminous, and concentrated source of coarser grained sand. The textural properties of inlet fill also make it ideal as a source of beach nourishment material. Inlet fill sediments almost always have a high sand to mud ratio, are moderate to moderately well-sorted, and typically are coarser grained than sandy environments adjacent to the inlet. These characteristics hold especially true for wave-dominated inlet deposits. Tide-dominated inlets, on the other hand, are associated with 1.0- to 3.0-m-thick (3- to 10-ft-thick) accumulations of fine-grained sediments (silts and clays) within the upper portions of vertical sedimentary profiles (Figure 2-19). These fine-grained accumulations, or "mud plugs," are difficult to predict in the subsurface without prior knowledge of the historical evolution of the inlet system and/or drill hole data. Thus, tide-dominated inlet fill sequences are less attractive sources of beach nourishment material than the wave-dominated counterparts.

(b) The extensive lateral migration and depth of scour of tidal inlet channels provide for a deeply incised, laterally extensive sand body with a very high potential for preservation. As noted earlier, inlet deposits are estimated to account for 30 to 50 percent of the sediments found within and along barrier island shorelines. There is, therefore, no shortage of potential sites for sources of relict inlet fill sediment. The large number of documented examples of inlet fill sediments found beneath the seafloor on the shoreface and inner continental shelf of the east and gulf coasts are proof of their high potential for preservation. In many instances, inlet deposits represent the only vestiges of a former barrier island shoreline that has been completely eroded or reworked during the Holocene transgression.

(2) Site selection. The dredging and utilization of inlet fill sediments for beach nourishment reintroduces sediment to the longshore transport system that would otherwise have been permanently and completely removed from the nearshore sediment budget. In fact, on some sand-deficient coastlines, such as Louisiana, sediment trapped in relict tidal inlet channels provides the only viable concentrated source of sand for beach nourishment.

b. Ebb-tidal deltas.

(1) Suitability. Several sedimentologic and stratigraphic aspects of ebb-tidal deltas make them highly attractive as sources of sediment (sand) for beach nourishment projects (see Section 2-5). In most instances, and especially along mesotidal barrier shorelines, ebb-tidal deltas are the largest, most voluminous, surface-exposed deposit of sand in the coastal system. Ebb-tidal deltas are comprised principally of clean, well-sorted sands, are several meters thick, and several hundred meters to a few kilometers in length and width. For 19 tidal inlets along the east coast of Florida, Marino and Mehta (1988) calculated a total of 420×10^6 m^3 (150×10^8 ft^3) of sandy sediment residing in the ebb deltas. This represents a tremendous volume of suitable sand resources that lie in immediate proximity to potential nourishment sites.

(2) Site selection. There are inherent dangers in dredging ebb-tidal deltas for sand resources that in most instances outweigh their positive attributes of sediment suitability. Any alteration of the morphology, bathymetric relief, or sediment dispersal of an ebb-tidal delta may result in marked alterations in local wave refraction/reflection patterns. Such changes are almost always associated with rapid and severe erosion of the shoreline immediately updrift or downdrift of the inlet. In addition, dredging and/or removal of the ebb-tidal delta may result in increased wave heights and less wave energy dissipation in immediate proximity to the shoreline. Thus, ebb deltas are tempting as a sand resource for nourishment projects, but should only be seriously considered after detailed design analysis testing and modeling have been performed.

c. Flood-tidal deltas.

(1) Suitability. The sedimentologic and stratigraphic characteristics of flood-tidal deltas make them relatively unattractive for sand resource potential. As documented in Section 2-5, flood deltas are predominantly interbedded sands and organic-rich muds that are generally no more than a few meters (2 to 5 m) thick. The sand deposits are moderately sorted and contain a high percentage of silt, clay, and shell material. Although flood deltas may cover a large surface area, they often thin appreciably with distance from the inlet throat.

(2) Site selection. Dredging of relict flood-tidal deltas, that is, flood deltas associated with relict or abandoned tidal inlets, presents no immediate natural hazards. Since they are located in semi-protected lagoons or estuaries, relict flood-tidal deltas are out of most zones of wave or current reworking. Removal by dredging of a large portion of a relict flood-tidal delta should have only a minimal impact on the adjacent back-barrier shoreline.

Chapter 3
Hydrodynamic Analysis of Tidal Inlets

3-1. Purpose and Scope

Inlets have been the focus of intense study by hydraulic engineers for many years (Watt 1905; Brown 1928; O'Brien 1931; Escoffier 1940, 1977; Bruun and Gerritsen 1960; Keulegan 1967; King 1974; Ozsoy 1977; Bruun 1978; van de Kreeke 1988). Hydraulic characteristics of interest to the practicing engineer consist of temporal and spatial variations of currents and water level in the inlet channel and vicinity. Depending on the degree of accuracy of the type of information needed, several predictive approaches are available. Although only approximate, relatively simple analytical procedures are commonly employed and yield quick answers. This chapter provides a brief description of inlet hydrodynamics and presents methods for performing an initial analysis of inlet stability and hydraulics. For more detailed descriptions of inlet hydraulics, physical and numerical modeling techniques are widely used (see Chapters 6 and 7, respectively).

3-2. Governing Equations

a. An idealized inlet system as shown in Figure 3-1 is considered to consist of a relatively short and narrow, but hydraulically wide, channel with mean depth h_c, cross-sectional area A_c, and length L_c. The sea tide represents the boundary condition, or forcing function, at one end of the channel and the bay at the other. The one-dimensional depth- and width-averaged shallow-water (long-wave) equation for the channel is

$$\frac{\partial u}{\partial t} + u\,\frac{\partial u}{\partial x} = -g\,\frac{\partial \eta}{\partial x} - g\,\frac{n^2 u|u|}{h_c^{4/3}} \tag{3-1}$$

where

$u(x,t)$ = cross-section averaged flow velocity along the channel length

t = time

$\eta(x,t)$ = tidal elevation with respect to mean water level

n = Manning's bed resistance coefficient

g = acceleration due to gravity

$n^2u|u|/h_c^{4/3}$ = slope of the energy grade line in the channel

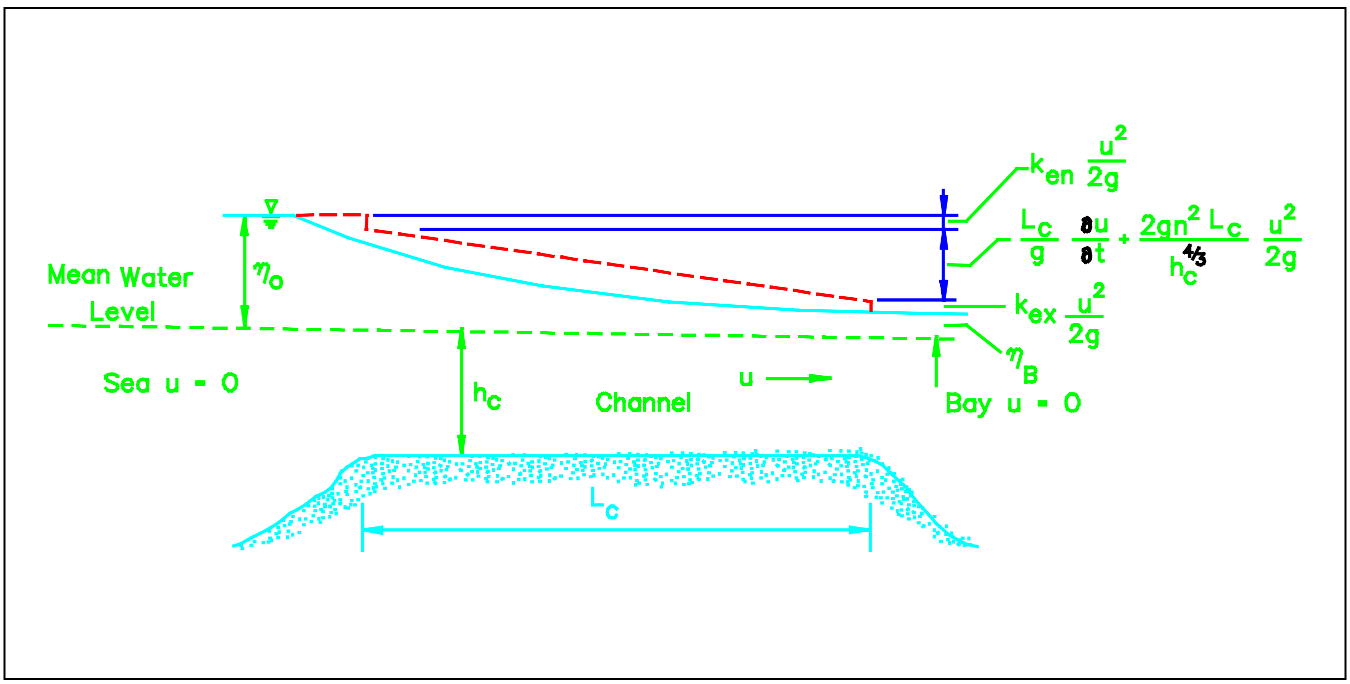


Figure 3-1. Idealized inlet channel showing contributions to head loss

With $\eta_o(t)$ and $\eta_B(t)$ representing tidal elevations in the sea and bay, respectively, integrating Equation 3-1 over length L_c gives

$$\eta_o - \eta_B = \frac{L_c}{g}\frac{\partial u}{\partial t} + \left(k_{en} + k_{ex} + \frac{2gn^2 L_c}{h_c^{4/3}}\right)\frac{u|u|}{2g} \tag{3-2}$$

where η and u are functions of time only. The quantities k_{en} and k_{ex} are the head loss coefficients associated with channel entrance and exit flows, respectively. The total head η_o - η_B is the sum of four contributions: entrance loss, $k_{en}u^2/2g$; head loss due to bed friction, $2gn^2L_c/h_c^{4/3}$; head due to inertia, $(L_c/g)\partial u/\partial t$; and exit loss, $k_{ex}u^2/2g$ (Figure 3-1).

b. Assumptions associated with Equation 3-2 include a) bay and ocean current velocities are negligible compared to those in the channel, b) tidal amplitude is small compared to mean depth, and c) change in water volume in the channel due to tidal variation is negligible compared to mean volume in the channel.

c. To apply the momentum equation (Equation 3-2), a continuity expression for the bay storage volume S is needed. The discharge Q through an inlet is related to the rate of change of S and the rate of freshwater discharge Q_f from any upstream sources by $Q = Q_f + dS/dt$ where $Q = uA_c$, $S = \eta_B A_B$, and A_B is the surface area of the bay. Assuming that the tide propagates rapidly through the bay (i.e. the bay is relatively small and deep) so that spatial gradients in the water surface at any instant may be ignored, continuity may be described in terms of the velocity u as

$$u = \frac{A_B}{A_c}\frac{d\eta_B}{d_t} + \frac{Q_f}{A_c} \tag{3-3}$$

d. Additional simplifying assumptions are needed to solve Equations 3-2 and 3-3 analytically. First, it is assumed that the bay surface area and freshwater discharge are independent of time. Also, the ocean tide is considered to be sinusoidal, $\eta_o = a_o \sin(\sigma t - \tau)$ where a_o is the tidal amplitude, σ is tidal frequency, and τ is the angular measure of the lag of slack water in the channel after midtide in the ocean. Combining Equations 3-2 and 3-3 by eliminating u and substituting for η_o yields

$$\frac{d^2\eta_B}{dt^2} + \frac{FA_B}{2A_cL_c}\left(\frac{d\eta_B}{dt} + \frac{Q_f}{A_B}\right)\left|\frac{d\eta_B}{dt} + \frac{Q_f}{A_B}\right| + \frac{gA_c}{L_cA_B}\eta_B = \frac{gA_ca_o}{L_cA_B}\sin(\sigma t - \tau) \tag{3-4}$$

where $F = k_{en} + k_{ex} + 2gn^2L_c/h_c^{4/3}$. Since the quantity F represents the effect of all influences restricting flow, O'Brien and Clark (1974) referred to it as the overall impedance of the inlet.

e. Analytical solutions to Equations 3-3 and 3-4 that have appeared in the literature can be divided into two general groups: those in which both the freshwater inflow and the inertia term have been ignored and those in which the middle term on the left side of Equation 3-4 has been simplified (Brown 1928; Escoffier 1940; Keulegan 1951, 1967; van de Kreeke 1967; Mota Oliveira 1970; Dean 1971; Mehta and Ozsoy 1978). Although these solutions are of limited accuracy, they provide insight into the response of inlet-bay systems to tidal forcing and may be used as an order of magnitude check on more rigorous numerical solutions.

f. Keulegan's (1967) solutions are attractive because of their relative simplicity and are frequently incorporated in the derivation of inlet stability criteria. Assumptions include a) sinusoidal ocean tide, b) vertical inlet and bank walls, so that the water surface area remains constant, c) small tidal range compared to water depth, d) small time variation of water volume in the channel compared to mean channel volume, e) horizontal water surface of the bay, f) mean water level in the bay equal to that of the ocean, g) negligible flow acceleration in the channel, and h) no freshwater discharge. The head difference, therefore, is due to bed frictional dissipation, and entrance and exit losses and Equations 3-3 and 3-4 can then be solved for the channel current velocity and bay tide. Keulegan's results include the phase lag between bay and ocean tides and dimensionless values of bay amplitude. Both of these can be related to the dimensionless parameter K introduced by Keulegan as an expression for the hydraulic and geometric characteristics of an inlet and referred to as the coefficient of filling or repletion (Equation 3-5, Figure 3-2).

$$K = \left(\frac{T}{2\pi a_o}\right)\left(\frac{A_C}{A_B}\right)\left(2\frac{ga_o}{F}\right)^{1/2} \tag{3-5}$$

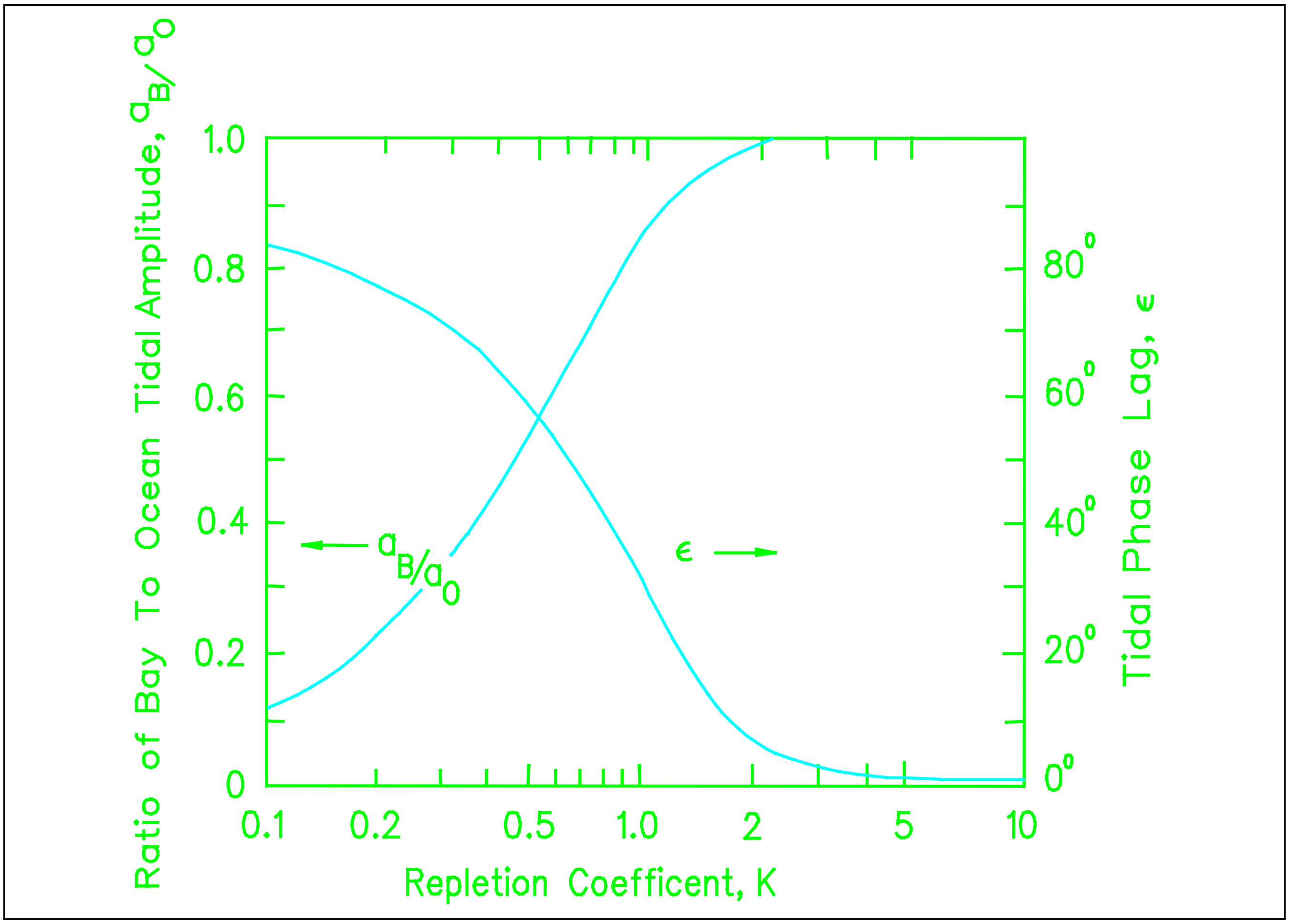


Figure 3-2. Relationship between the repletion coefficient and tidal phase lag and the ratio of bay to ocean tidal amplitude (after O'Brien and Dean (1972))

g. Keulegan also presented the relationship between K and dimensionless maximum velocity in the inlet V'_{max}[1] as shown in Figure 3-3. The maximum velocity V_{max} through a specific inlet is given by

$$V_{\max} = V'_{\max} \frac{2\pi}{T} a_o \frac{A_B}{A_c} \tag{3-6}$$

h. A set of tidal curves obtained by Keulegan's method is shown in Figure 3-4. Inherent in the result is that slack water corresponds to the time of maximum (and minimum) elevation in the bay. Maximum velocity occurs at midtide in the bay when η_o - η_B is a maximum. Amplitudes increase with increasing values of the repletion coefficient. This is expected since K increases with increasing values of cross-sectional area, hydraulic radius, and decreasing values of energy loss and friction. Because of the absence of inertia, the bay tidal amplitude is never larger than the ocean tidal amplitude.

i. Another approach to solving Equations 3-3 and 3-4 has been presented by Mehta and Ozsoy (1978) and Walton and Escoffier (1981) where the inertia term is not dropped. In Mehta and Ozsoy's (1978) method, the system of equations themselves is not linearized; however, the generation of higher harmonics is neglected in obtaining a first-order solution. Assuming a sinusoidal variation in the flow velocity, results are obtained as shown in Figures 3-5 and 3-6. Dimensionless parameters incorporated in the solution are: bay amplitude $\hat{\alpha}_B = a_B/a_o$, channel velocity $\hat{\alpha}_m = u_m A_C/a_o \sigma A_B$, tidal frequency

[1] In presenting Keulegan's work, the symbol V is used to denote channel velocity because the V is carried over in the derivation of various stability criteria.

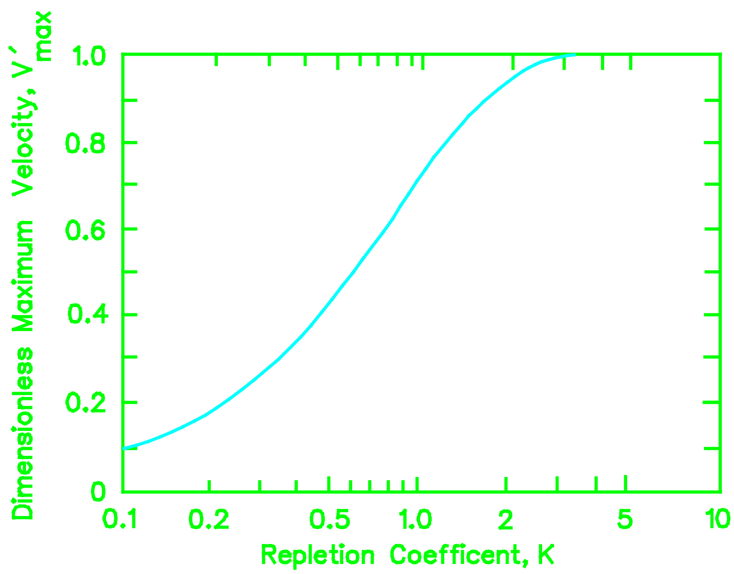


Figure 3-3. Relationship between repletion coefficient and dimensionless maximum velocity (after O'Brien and Dean (1972))

$\alpha = \sigma(L_C A_B/gA_C)^{1/2}$, and bed dissipation coefficient $\beta = a_o F A_B / 2 L_C A_C$. Bay water level amplification is predicted under a certain range of α and β conditions and lag greater than 90 deg. Also, the time of slack water does not necessarily coincide with high or low tide in the bay; at slack, the bay and ocean tide elevations differ by an amount equal to the head from flow inertia. Results compare well with those obtained by King (1974).

3-3. Hydraulic Parameters

a. Ocean tidal amplitude. The ocean tidal amplitude a_o may be obtained from published National Ocean Service (NOS) tide tables or field measurements. To minimize influence from the inlet and any associated structures, gauges should be positioned away from areas directly affected by inlet currents and values obtained from tables should be interpolated from outer coast values on either side of the inlet (Mehta and Ozsoy 1978).

b. Equivalent length. The length of the idealized equivalent inlet channel L_c used in the preceding development is related to and may be obtained from the real length of the channel by requiring that the head loss due to bed friction be equal in the two cases. Escoffier (1977) introduced the hypothetical quantity L_c, the equivalent length of a channel, as

$$L_c = A_c^2 \, h_c^{4/3} \sum_{i=1}^{i=m} \frac{\Delta x_i}{h_i^{4/3} A_i^2} \qquad (3\text{-}7)$$

In using the equation, the real inlet channel is divided into m sections of lengths Δx_i. Each section is chosen of such a length that over this length, the cross-sectional area A_i and mean depth h_i may be assumed constant. A basic assumption in deriving Equation 3-7 is that Manning's n is assumed to be independent of depth and is considered to characterize the channel bed roughness. O'Brien and Clark (1974) obtained a similar representation for L_c assuming the Darcy friction factor f to be constant rather than Manning's n. An equivalent channel cross-sectional area, rather than length, was used by Keulegan (1967).

c. Equivalent bay area. The condition of hydraulic bay filling is reasonably met only in relatively small bays

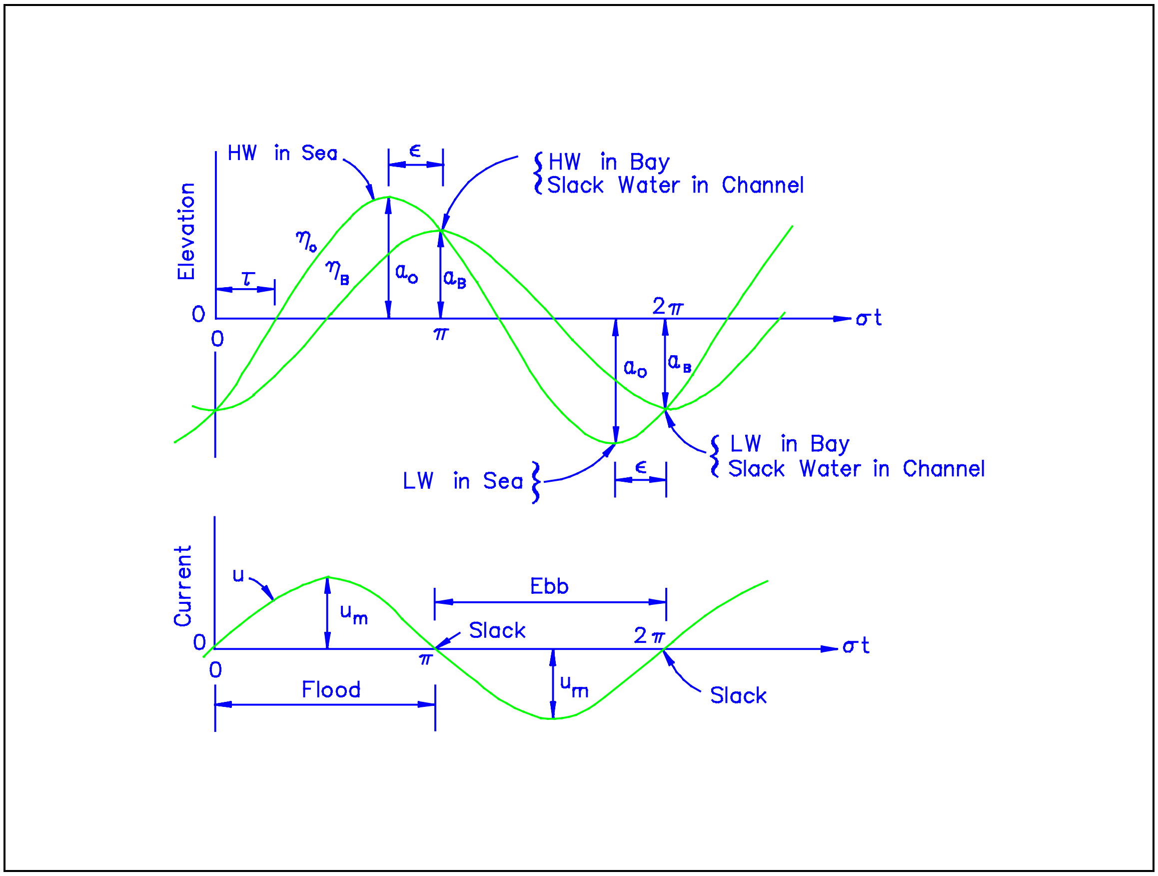


Figure 3-4. Ocean tide, bay tide, and current velocity through a channel as functions of dimensionless time (radians) (from Mehta and Joshi (1988))

(O'Brien and Clark 1974). Spatial water surface gradients due to inertia and bed friction in larger bays can be estimated using a simple approach involving the continuity principle (Escoffier 1977). If these gradients are not small compared to the bay tidal amplitude, Equation 3-3 is not applicable unless η_B is considered to be the tide at the bayward end of the inlet and A_B is redefined as an equivalent bay area corresponding to this tide. Equivalent bay area can be derived by dividing the tidal prism by the tidal range or by solving for it using Figure 3-5 and appropriate measurements of bay tidal amplitude a_B and a_o.

d. Bed resistance and loss coefficients.

(1) Bed resistance in an inlet channel varies with fluctuations in depth and bed form type that occur with changing tidal stage. For many engineering purposes, it is sufficient to estimate the bed resistance coefficient on a tide-averaged basis. Using the Chezy coefficient C (which is related to Manning's n according to $C = h_c^{1/6}/n$), Bruun and Gerritsen (1960) introduced an approximate empirical relationship: $C = \alpha_1 + \alpha_2 \log A_c$, based on measurements at sandy inlets with maximum velocities on the order of 1 m/sec (3.3 ft/sec). Proposed representative values of α_1 and α_2 were 30 $m^{1/2}$/sec and 5 $m^{1/2}$/sec, respectively, when A_c is in square meters and C is in $m^{1/2}$/sec (or α_1 = 44.3 $ft^{1/2}$/sec and α_2 = 9.4 $ft^{1/2}$/sec when A_c is in square feet and C is in $ft^{1/2}$/sec). In terms of Manning's n, the relationship between C and A_c can be written as

$$n = \frac{h_c^{1/6}}{\alpha_1 + \alpha_2 \log A_c} \tag{3-8}$$

************************************ EXAMPLE PROBLEM 3-1 **************************************

GIVEN: A bay with a surface area, A_B = 1.86 x 10^7 m^2 (2 x 10^8 ft^2) and an average depth of 6.1 m (20 ft) is located on the Atlantic coast. The tide is semidiurnal (T = 12.4 hr), with a spring range of 1.34 m (4.4 ft), as given by the National Ocean Survey Tide Tables (available from the National Oceanic and Atmospheric Administration). An inlet channel, which will be the only entrance to the bay, is to be constructed across the barrier beach which separates the bay from the ocean. The inlet is to provide a navigation passage for small vessels, dilution water to control bay salinity and pollution levels, and a channel for fish migration. The channel is to have a design length, L_c = 1,097 m (3,600 ft) with a pair of vertical sheet-pile jetties that will extend the full length of the channel. The channel has a depth below mean sea level (msl), h_c = 3.66 m (12 ft), and a width W_c = 183 m (600 ft).

FIND: The bay tidal range, maximum flow velocity V_{max}, and volume of water flowing into and out of the bay on a tidal cycle (tidal prism) for a tide having the spring range.

SOLUTION: Assume entrance and exit loss coefficients, k_{en} = 0.1, k_{ex} = 1.0, respectively, and n = 0.027.

$A_c = W_c\, h_c$ = (183 m)(3.66 m) = 669 m^2 (7,200 ft^2)

$$F = k_{en} + k_{ex} + 2gn^2/(h_c^{4/3})$$
$$= 0.1 + 1.0 + (2)(9.81\ \text{m/sec}^2)(0.027)^2/(3.66\ \text{m}^{4/3})$$
$$= 3.88$$

a_o = 1.34 m/2 = 0.67 m (2.2 ft)

Then by Equation 3-5,

$$K = [(12.4\ \text{hr})(3600\ \text{sec/hr})/(2)(3.14)(0.67\ \text{m})]\ [669\ \text{m}^2/(1.86 \times 10^7)]$$
$$[(2)(9.81\ \text{m/sec}^2)(0.67\ \text{m})]^{1/2}\ /(3.88^{1/2})$$
$$= 0.7$$

From Figures 3-2 and 3-3, with K = 0.7

V'_{max} = 0.58

and

a_b/a_o = 0.69, therefore

a_b = (0.69)(0.67 m) = 0.46 m (1.5 ft), and the bay tidal range is 2(0.46 m) = 0.92 m (3.0 ft)

From Equation 3-6, the maximum flow velocity is

$$V_{max} = 0.58\ [(2)(3.14)/(12.4\ \text{hr})(3600\ \text{sec/hr})]\ (0.46\ \text{m})$$
$$(1.86 \times 10^7\ \text{m}^2)/669\ \text{m}^2)$$

$$= 1.04\ \text{m/sec}\ (3.41\ \text{ft/sec})$$

The tidal prism is

$(2)(a_b)(A_b)$ = (2)(0.46 m)(1.86 x 10^7 m^2) = 1.7 x 10^7 m^3 (6.0 x 10^8 ft^3)

If the average depth of the bay is 6.1 m (20.0 ft) and the distance to the farthest point in the bay is 6.4 km (4.0 miles), the time t_* it will take for the tide wave to propagate to that point is

$$t_* = L_b/[(g)(d_b)]^{1/2} = 6400\ \text{m}/[(9.81\ \text{m/sec}^2)\ (6.1\ \text{m})]^{1/2}$$
$$= 827\ \text{sec, or } 0.23\ \text{hr}$$

Since this time is significantly less than 12.4 hr, the assumption that the bay surface remains horizontal is quite satisfactory.

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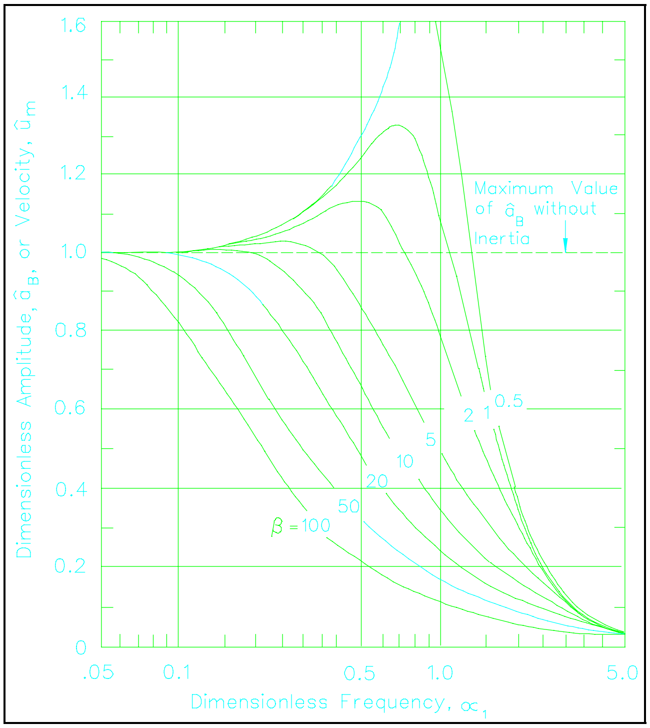

Figure 3-5. Dimensionless bay tidal amplitude or channel velocity as functions of dimensionless frequency (from Mehta and Ozsoy (1978))

Manning's n can be approximated using $A_c = h_c W_c$ and empirical relationships between throat depth and width in the form of $h_c = pW_c^q$, where p and q are coefficients, and W_c is the width at the inlet throat (Graham and Mehta 1981). For structured inlets, n ranges from 0.026 to 0.029, and from 0.025 to 0.027 for those without jetties (Mehta and Joshi 1988).

(2) Flow coming from a channel can be compared to that of a separated jet expanding from a narrow channel into a very large basin. Most of the energy dissipation occurs in the expanding part of the flow due to turbulence. Since the kinetic head is usually lost as the flow enters the basin, $k_{ex} = 1$. For flow entering a channel, energy loss is not very significant and $k_{en} \leq 0.05$.

e. Tidal current and prism.

(1) The tidal prism is the volume of water that is drawn into the bay, from the ocean and through the inlet, during flood tide. Aperiodicity of the tide, freshwater discharge, and the presence of other openings in the bay are some of the reasons why the prism in not always equal to the volume of water that leaves during the ebb. In the case of a single inlet-bay system with sinusoidal ocean tide, the tidal prism can be approximated by $2\, Q_m/\sigma C_K$ where $Q_m = u_m A_c$ is the maximum discharge and C_K is a parameter that varies with the repletion coefficient. The term C_K essentially accounts for the nonlinearity in the variation of discharge Q with time as a result of the quadratic head loss. At $K = 1$, $C_K = 0.81$ and at $K = 4$, $C_K = .999$ (Keulegan 1967). For simple calculations, an average value of 0.86 has been recommended by Keulegan and Hall (1950) and O'Brien and Clark (1974).

(2) For sandy inlets, the cross-sectional average velocity at the throat is on the order of 1 m/sec (3.3 ft/sec). For very small inlets, the velocity may be lower and for those with rocky bottoms, the velocity may be higher. To accurately determine the flow field, in situ measurements at several elevations across the flow section using current meters are recommended. It is important to keep in mind that typically well-defined ebb-and flood-dominated channels are present and flood flow is usually dominant near the bottom, while ebb flow is dominant near the surface.

3-4. Inlet Stability Criteria

a. Some inlets are permanent and remain open with relatively small changes in location, cross-sectional area, and shape; others are ephemeral or subject to intermittent openings and closings. The ability of an inlet to maintain itself in a state of stable equilibrium against wave activity and associated littoral transport depends on the availability of littoral material and the tidal prism. Many attempts at describing inlet stability have concentrated on empirical relationships between the tidal prism and inlet throat cross-sectional area (LeConte 1905; O'Brien 1931, 1969; Nayak 1971; Johnson 1973). Jarrett (1976) reviewed the previously established relationships and performed a regression analysis on data from 108 Pacific, Atlantic, and gulf coast inlets in various combinations in an attempt to determine best-fit equations. Results of his analysis indicated that the tidal prism-inlet (P) cross-sectional area (A) relationship is not a unique function for all inlets, but varies depending on inlet location and the presence or absence of jetties. Jarrett confirmed the original relationship established by O'Brien (1969) for inlets with two jetties

$$A = 4.69 \bullet 10^{-4}\, P^{0.85} \tag{3-9}$$

and concluded that natural inlets and single-jettied inlets on the three coasts exhibit slightly different P versus A

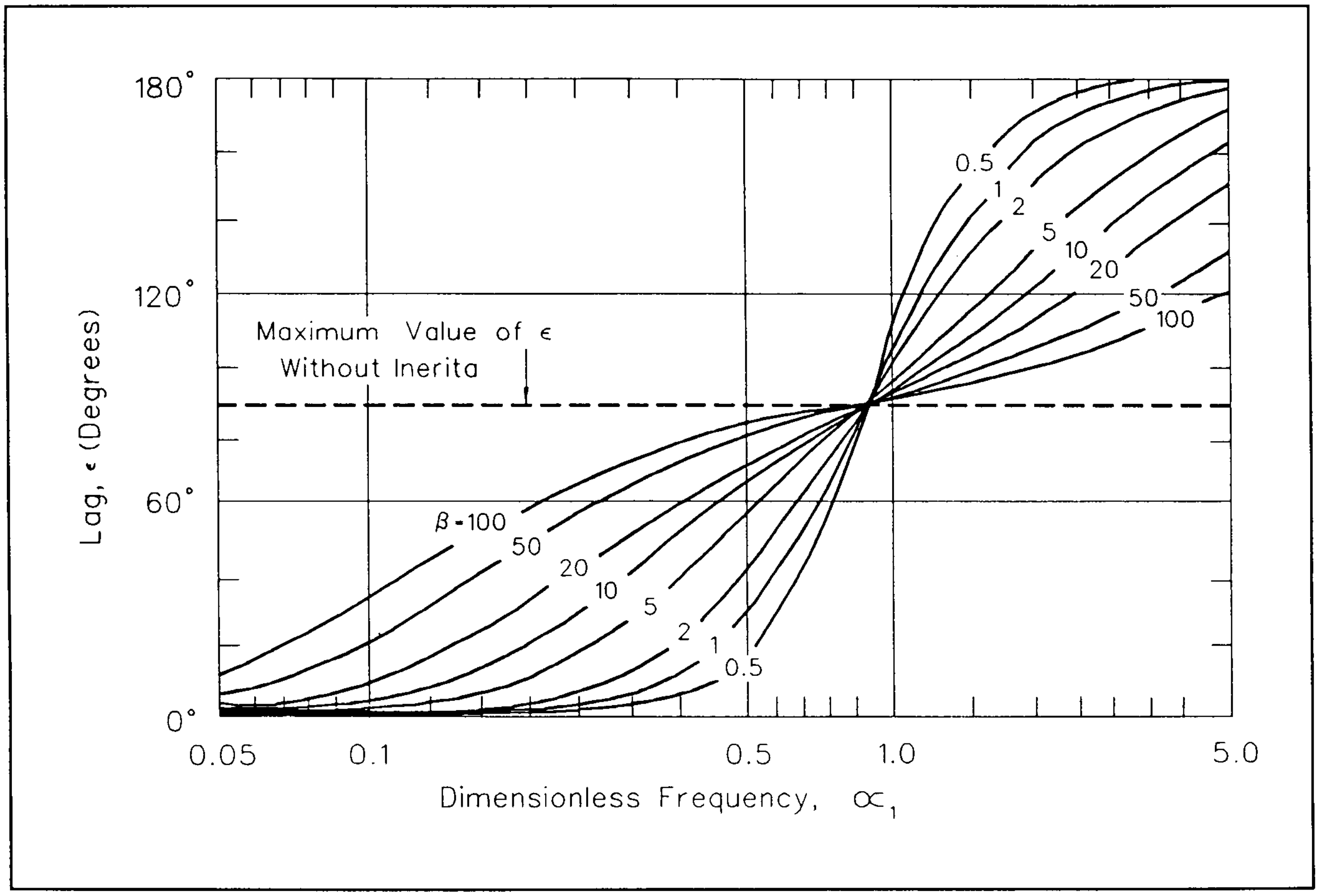

Figure 3-6. Lag as a function of dimensionless frequency (from Mehta and Ozsoy (1978))

relationships due to differences in tidal and wave characteristics (Figure 3-8).

b. The Ω/M criteria for inlet stability, where M is the total annual littoral drift, and Ω is the tidal prism, were introduced by Bruun and Gerritsen (1960) and elaborated on by Bruun (1978). The stability of an inlet is rated as good, fair, or poor according to the following:

$\Omega/M > 150$	Good
$100 \geq \Omega/M \leq 150$	Fair
$50 \geq \Omega/M \leq 100$	Fair to poor
$\Omega/M < 50$	Poor

c. Escoffier (1940) introduced a hydraulic stability curve, referred to as the Escoffier diagram, on which maximum velocity is plotted against cross-sectional flow area (Figure 3-9). A single hydraulic stability curve represents changing inlet conditions, when ocean tide parameters, and bay and inlet plan geometry conditions remain relatively fixed. If these conditions are drastically altered, a new stability curve is established. Each position on the curve represents a different value of Keulegan's repletion coefficient K, the ratio of bay to ocean tidal amplitude, and tidal phase lag between ocean high or low tide and slack water in the inlet.

************************************** EXAMPLE PROBLEM 3-2 **************************************

GIVEN: Information provided in Example 3-1.

FIND: Potential stability of the proposed channel cross section. Remember the channel has vertical sheet-pile walls, so its cross section can only change in the vertical.

SOLUTION: By varying the cross-sectional area of the channel A assuming that the channel width W_c remains constant and varying the channel depth h_c and recalculating the tidal prism as described in Example 3-1, the effect of channel area on the bay tidal prism can be evaluated and compared with Equation 3-9.

h_c (m)	0.91	1.8	2.4	4.9	7.6	10.7
K	0.11	0.29	0.44	1.1	2.0	2.7
V'_{max}	0.12	0.35	0.50	0.81	0.95	0.98
a_b/a_o	0.12	0.38	0.59	0.97	1.0	1.0
a_b	0.08	0.25	0.40	0.65	0.68	0.67
V_{max} (m/sec)	0.15	2.5	1.4	1.9	1.3	0.9
P (x10^6)(m^3)	3.0	9.3	14.9	24.2	25.3	24.9
A_c (m^2) (=$2a_bA_b$)	167.0	329.0	439.0	897.0	1,391.0	1,958.0
A_c (m^2) (Eqn 3-9)	150.0	393.0	587.0	886.0	920.0	908.0

Graphical results are presented in Figure 3-7. The common point on the two curves is the solution to the problem. In this case, however, two common points occur, indicating that the channel may either close at the lower point (approximately 220-m^2 (2,370-ft^2) cross-sectional area, and 1.2-m (4-ft^2) depth), or scour to the upper stability point (approximately 890-m^2 (9,600-ft^2) cross-sectional area, and 4.9-m (16-ft) depth). This indicates that the 183- by 3.7-m (600- by 12-ft) design channel would be unstable.

Where the hydraulic response curve lies above the stability curve, the tidal prism is too large for the inlet channel area and erosion will likely occur until a stable channel develops. If the hydraulic response curve crosses the stability curve twice, as in this example, the lower point is an unstable equilibrium point from which the channel can either close or scour to the upper stability point. If the hydraulic response curve is substantially below the stability curve at all points, a stable inlet channel is unlikely to develop and the channel should eventually close.

The stable inlet cross-sectional area depends on other factors (e.g., wave climate, monthly tidal range variations, surface runoff) besides the spring or diurnal tidal prism. As a result, the tidal prism - inlet area relationships (Figure 3-8) serve only as indications of the approximate stable cross-sectional area. The analysis performed in the example demonstrates that the design channel will most likely shoal or erode; however, the actual equilibrium depth will fluctuate with time, and can vary substantially from the indicated depths of 1.2 m (4 ft) or 4.9 m (16 ft).

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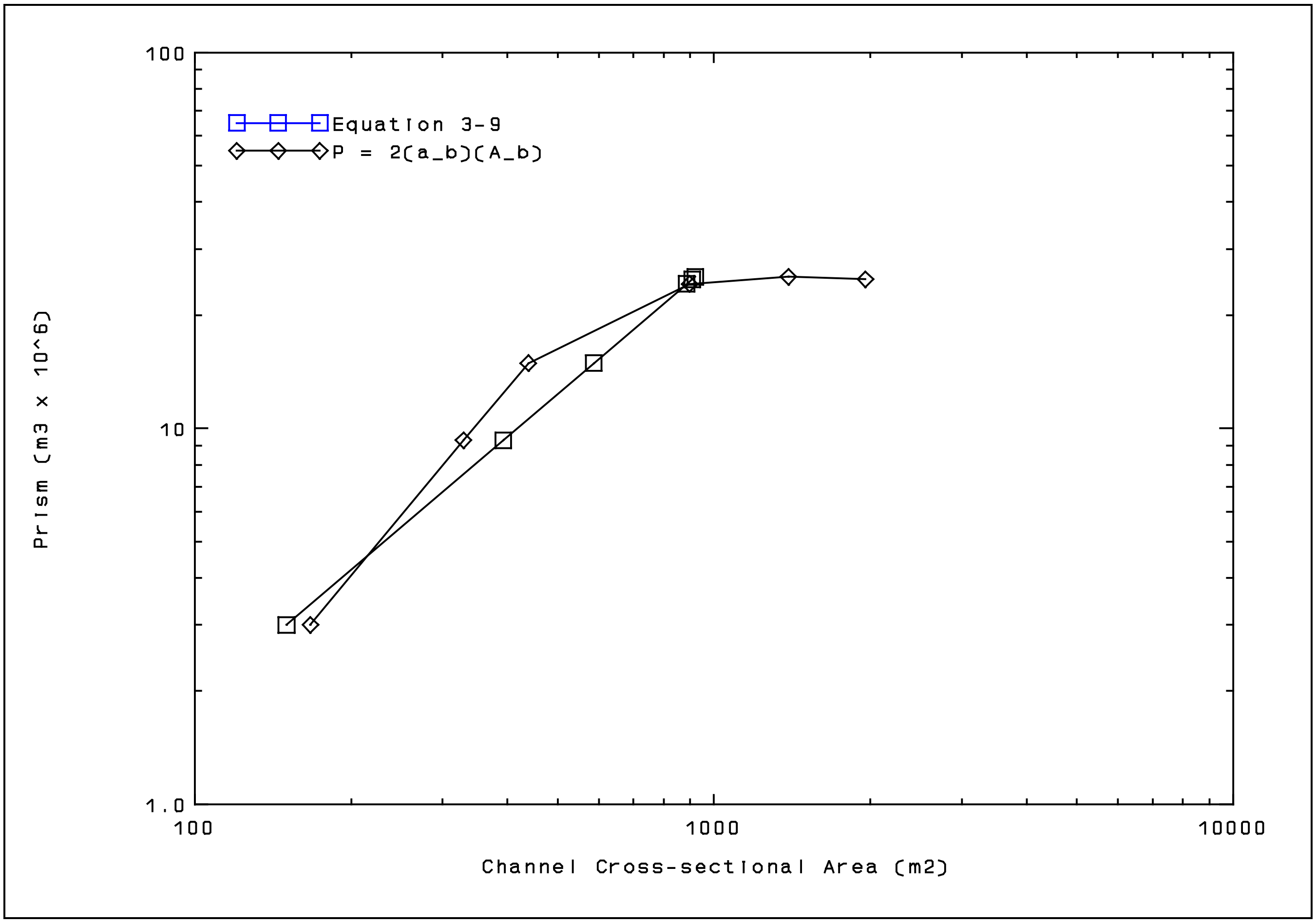

Figure 3-7. Graphical results for example problem 3-2

d. According to the Escoffier diagram, an inlet is hydraulically stable if its cross-sectional area is larger than the critical flow area A_{c*}. An induced change in the cross-sectional area of a stable inlet will result in a change in inlet velocity that attempts to return the inlet to its equilibrium size by appropriate deposition or scour. An inlet having a cross-sectional area smaller than the critical flow area is termed hydraulically unstable. The Escoffier diagram illustrates that any change in flow area is accompanied by a change in flow velocity that will perpetuate the induced change. Since any initial change in flow area is accentuated, the hydraulically unstable inlet will either continuously scour until the critical flow area is attained, or continuously shoal until inlet closure. Escoffier's hydraulic stability model has been applied in describing the behavior of "hydraulically stable" inlets by O'Brien and Dean (1972); Defenr and Sorensen (1973); and Mehta and Jones (1976).

e. In a later paper, Escoffier (1977), using Keulegan and O'Brien formulations, presented a variation of his original diagram that allows for the equilibrium value of V_{max} to vary with the repletion coefficient *K*. Dimensionless velocity values ν and ν_E (where $\nu = V_{max}/(2ga_o)^{1/2}$ and ν_E is the equilibrium value of ν), are plotted as functions of *K* (Figure 3-10). The letter *A* represents an unstable equilibrium point and *B* represents a stable equilibrium point. A small deviation from conditions represented by point *A* sets into operation forces that tend to reinforce that deviation. A similar deviation at *B* results in changes that tend to restore the inlet to its equilibrium point. Figure 3-11 shows several possible relative positions for the two curves. The first ν curve plots high enough to intersect the ν_E curve in two places, one stable and one unstable. The second ν curve has only one point of tangency with the ν_E curve, an unstable point. The third curve fails to reach the ν_E curve and therefore,

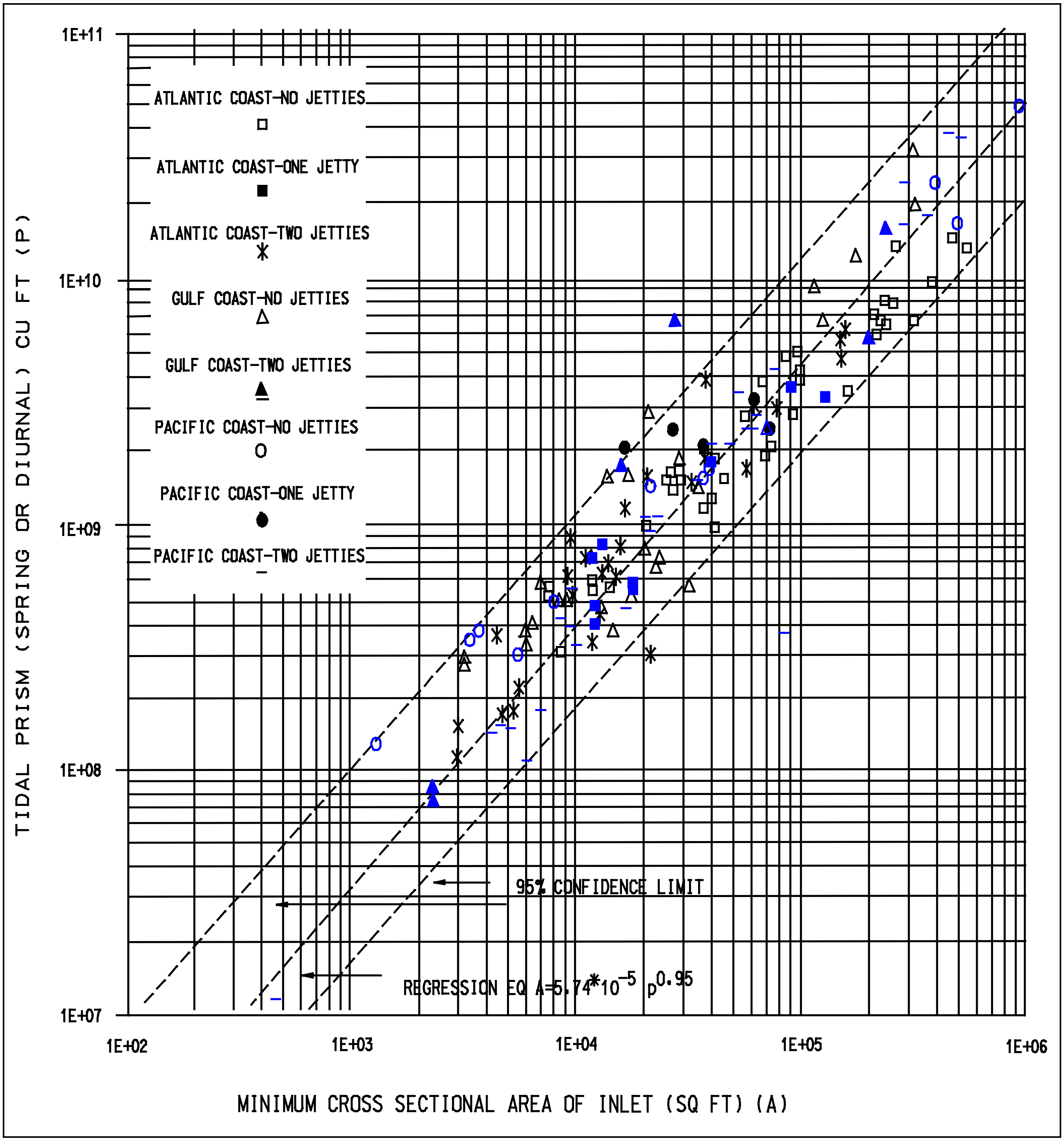

Figure 3-8. Tidal prism versus cross-sectional area for Pacific, Atlantic, and Gulf coast inlets (after Jarrett (1976))

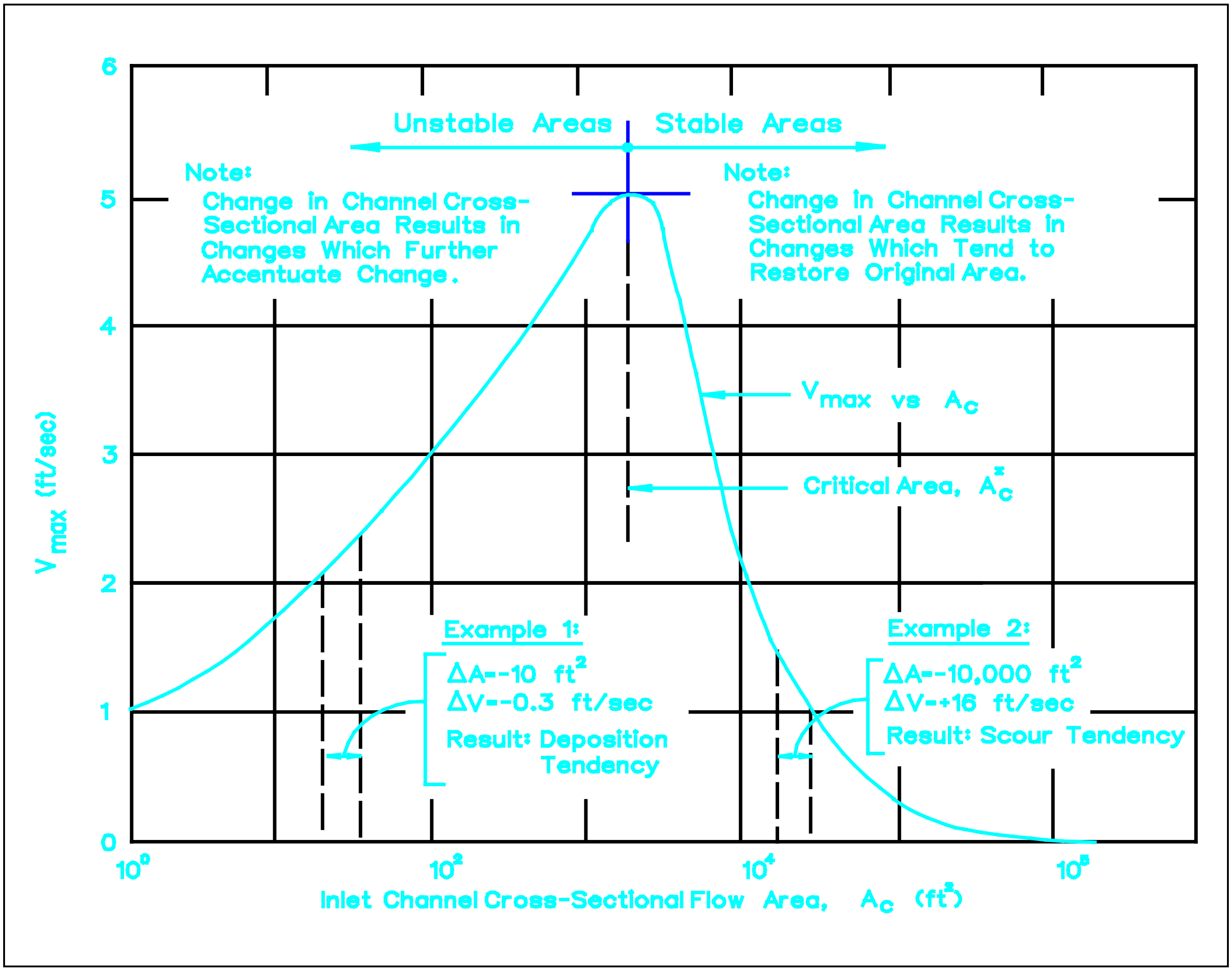

Figure 3-9. Generalization of Escoffier's hydraulic stability curve (after O'Brien and Dean (1972))

stability is not possible. Escoffier presented the idea of translating the height of the ν curve into a measure of stability, represented by the dimensionless parameter λ (Figure 3-12). The value of λ is equal to the ratio of ν to ν_E for the value of *K* that makes ν a maximum. The value of λ can be taken as a measure of the degree of stability; λ greater than 1 indicates stability.

f. O'Brien and Dean (1972) proposed a method of calculating the effect of deposition on stability of an inlet. Their method is based on earlier contributions by O'Brien (1931), Escoffier (1940), and Keulegan (1967), and assumes that a critical cross-sectional throat area A_{c*} exists with a corresponding critical velocity V_{max}. A stability index β represents the capacity of an inlet to resist closure under conditions of deposition. It incorporates the buffer storage area available in the inlet cross section, prior to deposition, and also includes the capability of the inlet to transport excess sand from its throat.

$$\beta = \int_{A_{C*}}^{A_{CE}} (V_{max} - V_T)^3 \, dA_C \qquad (3\text{-}10)$$

where

β = stability index (units of length^5/time^3)

A_{CE} = cross-sectional area of throat

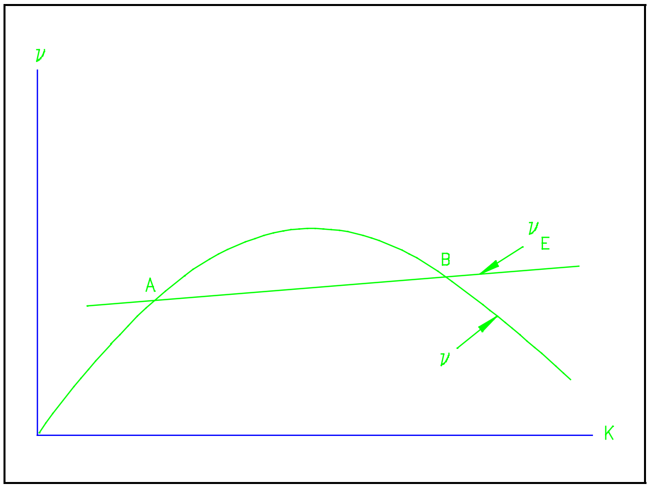

Figure 3-10. ν_E and ν versus repletion coefficient (from Escoffier (1977))

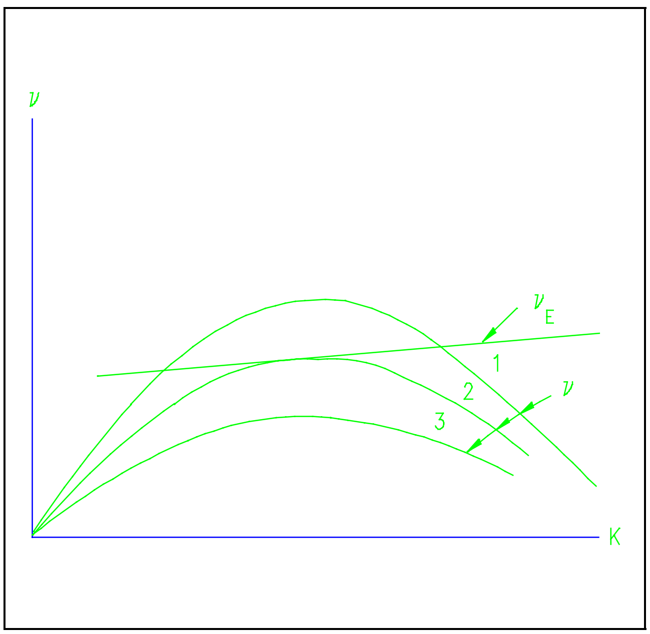

Figure 3-11. ν_E and various values of ν versus repletion coefficient (after Escoffier (1977))

$V_{\max}$ = maximum velocity in the throat

V_T = threshold velocity for sand transport

A_{C*} = critical cross-sectional area (value of A_C at peak of $V_{\max}$ curve)

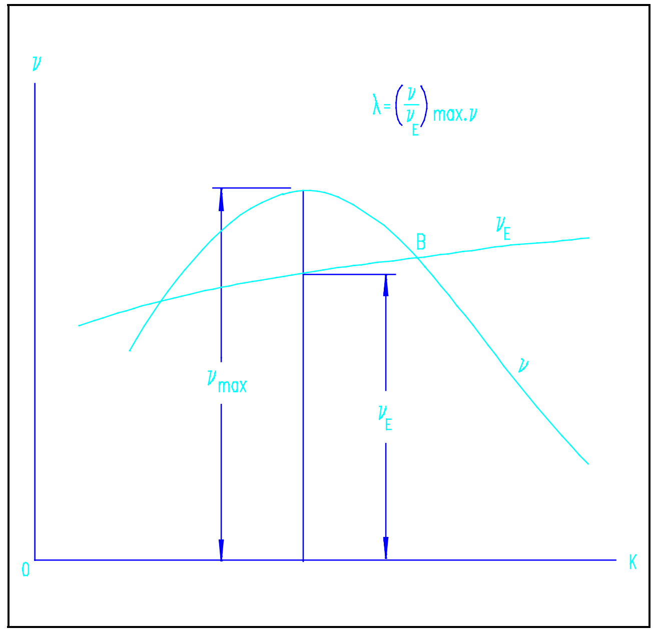

Figure 3-12. Definition diagram for the stability parameter λ (from Escoffier (1977))

Inlets with an equilibrium area much larger than the critical area have more storage area, and therefore, will be more resistant to change.

g. This method requires knowledge of existing inlet conditions, assumed to be equilibrium conditions. Minimum data needed include a survey of the inlet throat cross section and the lag between high water in the ocean and the following slack water in the inlet. Czerniak (1977) found that the O'Brien and Dean stability theory was quite successful in explaining observed behavior at Moriches Inlet, New York. Interpretation of inlet history provided qualitative verification of the hydraulically unstable portion of the inlet hydraulic curve (Escoffier diagram) in both the scour and shoaling modes as well as semi-quantitative verification of the unstable scour mode. Results suggested that the theory could be applied to a broad range of inlet-related problems including those dealing with hydraulic design, sand bypassing design, control of bay tidal conditions, and effects of jetties on inlet stability.

Chapter 4
Sediment Budget and Shoaling Rates

4-1. Introduction

a. Inlet systems represent a primary natural boundary or sink for transport of littoral and nearshore sediment. Geomorphic features associated with inlets separate adjacent beach and backshore environments and act as conduits for exchange of water and sediment between lagoon or estuarine environments and the nearshore. As such, characteristic shoal deposits form in response to wave and current interaction as water and sediment ebb and flood through primary and secondary inlet channels (see Chapter 2). Depending on the dominance of wave processes versus tidal currents, sediment deposition from cross-shore and longshore sources varies spatially, from within the lagoon or estuary (flood shoal) to seaward of the entrance (ebb shoal), and temporally as shoal migration in response to seasonal shifts in wave height and direction, and storm events. Regardless of feature characteristics within the inlet system, in most cases, this environment is a natural sink for coastal sediment. Consequently, the application of a sediment budget to inlets and adjacent environments is an effective approach for evaluating the relative significance of various sediment sources contributing to shoal growth and the relative importance of sediment bypassing from the shoals to adjacent beaches.

b. Assessing the sediment budget is particularly important where engineering activities, such as jetty construction and dredging, have fixed the position of the channel. This analysis assists scientists and engineers with quantifying the dynamic response of inlet systems by identifying relevant coastal processes and estimating volume rates of littoral transport. Engineering design, construction decisions, and management plans affect, and are affected by, sediment budget considerations. Predicting downdrift shoreline response and channel shoaling rates is crucial to efficient system maintenance efforts. This chapter reviews the components of a coastal sediment budget and presents an example of a sediment budget for engineering application. Shoaling rate prediction methods will also be discussed. Channel shoaling is an important component of the inlet system, and its prediction is critical to effective maintenance of the navigation channel.

c. Primary references on coastal sediment budgets include *Beach Processes and Sedimentation* (Komar 1976), the *Shore Protection Manual* (SPM 1984), Engineer Manual (EM) 1110-2-1502 entitled *Coastal Littoral Transport*, and Instruction Report CERC-93-1 entitled *Review of Geologic Data Sources for Coastal Sediment Budgets* (Meisburger 1993). Although none of these specifically focuses on inlet systems (i.e., the inlet system is usually the unknown portion of the sediment budget), most of the information presented in these documents is applicable to any coastal setting.

4-2. Components of a Coastal Sediment Budget

a. Sources and sinks.

(1) A sediment budget reflects an application of the principle of continuity or conservation of mass to coastal sediment. The time rate of change of sediment within a system is dependent upon the rate at which material is brought into a control volume versus the rate at which sediment leaves the same volume (Komar 1976). The budget involves assessing the sedimentary contributions and losses and equating these to the net balance of sediment in a coastal compartment. Any process that results in a net increase in sediment in a control volume is called a *source*. Alternately, any process that results in a net loss of sediment from a control volume is considered a *sink*. Some processes can function as sources and sinks for the same control volume (e.g., longshore sediment transport).

(2) The balance of sediment between losses and gains is reflected in localized erosion and deposition. Table 4-1 summarizes possible sources and sinks of sediment for a coastal sedimentary budget. In general, longshore movement of sediment into a coastal compartment, onshore transport of sediment, additions from fluvial transport, and dune/bluff/cliff erosion provide the major sources of sediment. Longshore movement of sediment out of a coastal compartment, offshore transport of sediment, and aeolian transport and washover that increase beach/island elevation produce losses from a control volume. Further discussion regarding the type and importance of sources and sinks for evaluating a coastal sediment budget are discussed in detail in Komar (1976), the *Shore Protection Manual* (SPM 1984), and Meisburger (1993).

(3) All elements of sediment budgets do not necessarily have the same spatial characteristics. For instance, tidal inlets often function as *point* sinks or features that decrease the transport of sediment across a *limited* portion of a control volume boundary. Conversely, a *line* sink causes a decrease in sediment transport across an *extended* portion of a control volume. Net transport of sediment offshore and out of the control volume along the entire

Table 4-1
Sources and Sinks for a Coastal Sediment Budget (after Bowen and Inman (1966))

Sources	Sinks
· Longshore transport of sediment into a control area	· Longshore transport of sediment out of a control area
· Onshore transport	· Offshore transport
· Fluvial transport	· Washover deposition
· Dune/bluff/cliff erosion	· Aeolian transport out of control area
· Aeolian transport onto beach	· Sediment storage in offshore shoals
· Biogenous and hydrogenous deposition	· Deposition in submarine canyons
· Beach replenishment	· Solution and abrasion
	· Dredging

offshore boundary is an example of a line sink. Unlike *point* sources or sinks that are quantified in units of volume per year, *line* elements of a sediment budget are calculated relative to the total length of shoreline over which the source or sink operates. Table 4-2 provides a classification of elements in a coastal sediment budget in terms of point and line sources or sinks. In a complete sediment budget, the difference between the addition of all source components and sediment removed from the control volume must total zero. However, in general applications, a sediment budget calculation is made to estimate an unknown erosion or deposition rate; the difference resulting from equating known sources and sinks. Detailed discussions on how gains and losses can be evaluated are given in SPM (1984) and EM 1110-2-1502, and an example of a sediment budget for engineering application is presented in Section 4-3.

b. Sediment budget boundaries. Boundaries for coastal sediment budgets are determined by the area under study (control volume), the time scale of interest, and the purpose of the study. For a given area, adjacent sediment compartments may be needed, with shore-perpendicular boundaries at significant longshore changes in the coastal system. At inlet systems, compartment boundaries are needed regardless of the magnitude and direction of shoreline response in adjacent compartments due to significant differences in processes affecting sediment transport. Although inlet systems can exchange sediment between updrift and downdrift beaches via shoal bypassing, most

Table 4-2
Classification of Elements in a Coastal Sediment Budget (after SPM (1984))

Location of Source or Sink	Offshore Side of Control Volume	Onshore Side of Control Volume	Within Control Volume	Longshore Ends of Control Volume
Point Source (volume/unit time)	Offshore shoal or island	Rivers, streams	Shoal erosion	Longshore transport into control volume
Point Sink (volume/unit time)	Offshore shoal; submarine canyon	Inlets	Dredging	Longshore transport out of control volume
Line Source (volume/unit time/unit length of coast)	Onshore transport	Coastal erosion of dunes, bluffs, and cliffs	Beach erosion; calcium carbonate production	NA
Line Sink (volume/unit time/unit length of coast)	Offshore transport	Washover; coastal land and dune storage	Beach accretion; beach nourishment; calcium carbonate losses	NA

NA - not applicable.

of the time this environment responds as a point sink for sediment, resulting in well-defined natural boundaries for a control volume. Shore-parallel boundaries also are needed on the seaward and landward sides of the control volume. The landward boundary is generally defined as a position representing the landwardmost extent of shoreline position for the temporal extent of the study, whereas the seaward boundary is established at or beyond the limit of sediment movement initiation (seaward edge of nearshore zone) or the limit of significant sediment movement due to steady wave action (closure depth) (Hallermeier 1981). Boundary criteria vary depending on study objectives. Therefore, it is critical that factors used to determine compartment boundaries be explicitly defined, such that the selection may be evaluated and compared with previously established sediment budgets.

c. Convection of littoral material. The magnitude and direction of coastal processes affect the classification of gains or losses to or from a control volume. For example, the net rate of sediment deposition or erosion in the littoral zone is controlled by differences in the rate of longshore transport into and out of a control volume. If sediment export is greater than import, erosion results and the compartment is a net source of material to adjacent compartments. Some processes may subtract at the same rate they add sediment to a control volume, resulting in no net change in material volume. The most important convecting process is longshore sediment transport. Along most coasts, gross longshore transport rates exceed net rates, and it is possible to have gross sediment transport rates in excess of 500,000 m^3 (650,000 yd^3) annually with no apparent beach changes. In other words, the same net rate of longshore sediment transport can be produced by widely varying rates of gross transport in and out of a control volume. Other convecting processes that may produce large rates of sediment transport with little noticeable change include tidal flows, especially around inlets, wind transport in the longshore direction, and wave-induced currents in the offshore zone. Because any structure that interrupts longshore sediment transport will normally result in erosion or accretion, it is important that the sediment budget quantitatively identify all processes convecting sediment through the study area.

d. Relative sea level change. Relative changes in sea level are the result of fluctuations in eustatic sea level (global water level adjustments) and regional or local changes in land level. Although eustatic sea level is rising worldwide, land levels are rising and falling due to tectonic forces, compactional subsidence, and human activities (i.e., subsurface fluid withdrawal). The importance of relative change in sea level on coastal engineering design depends on the time scale and the locality involved; impacts should be evaluated on a project-by-project basis. In terms of its impact on a coastal sediment budget, relative sea level change *does not* directly enter the evaluation procedure; however, the net effect of elevation changes may be landward (rising water level) or seaward (falling water level) displacement of the shoreline. Thus, relative changes in sea level can result in the appearance of a gain or loss of sediment volume. However, any changes in sediment volume would be balanced within the control volume because the seaward boundary of the compartment generally is defined by the seaward limit of significant sediment transport.

e. Summary. The range of significance for sinks, sources, and convective processes in a coastal sediment budget is described in Table 4-3. The relative importance of elements in the sediment budget varies with locality and with the boundaries of a particular control volume. For most beach environments, gross longshore transport rates significantly exceed other volumetric rates in the sediment budget, but if the beach is approximately in equilibrium, this may not be noticeable. Erosion of beaches, dunes, bluffs, and cliffs, as well as river contributions, are the principal natural sources of sediment in most locations. Human influences, such as beach nourishment, may provide major sources in local areas. Inlets, lagoons, and environments seaward of the depth of initiation of sediment motion comprise the principal natural sinks for coastal sediment. However, sediment transport or shoal migration from ebb-tidal deposits at inlets to the beach (Fitzgerald 1984), and erosion and offshore transport of sediment from estuaries and lagoons during major storm events (Isphording and Ismand 1991) illustrate the varying importance of sources and sinks for specific study areas. Of potential importance as either a sink or source is the offshore zone between closure depth and the point of initiation of sediment movement. Detailed analyses of historical bathymetric change on this portion of the continental shelf indicate significant sediment movement (Knowles and Gorman 1991; List, Jaffe, and Sallenger 1991; Byrnes and Hiland 1994), suggesting greater importance to the coastal sediment budget than originally anticipated.

4-3. Example Application

a. General.

(1) Coastal sediment budgets are particularly useful in assessing the possible impacts of engineering activities. For example, once a budget has been established for natural conditions at a study site, one can assess the

Table 4-3
Importance of Contributions to a Coastal Sediment Budget Relative to the Gross Longshore Sediment Transport Rate (after SPM (1984))

Sources	
Fluvial input	· Major source in limited areas where rivers carry sediment to the littoral zone; may contribute several times the gross longshore sediment transport rate during floods.
Dune, bluff, and cliff erosion	· Generally the major sources where river contributions are insignificant. Approximately 3 to 10 m^3/year (4 to 13 yd^3/year) per meter of beach.
Onshore transport	· Quantities uncertain. Net contributions can be estimated from historical bathymetric change data.
Aeolian transport	· Relatively unimportant as a source.
Beach replenishment	· Varies from 0 to greater than the gross longshore transport rate.
Calcium carbonate production	· A significant source in tropical climates. Approximately 0.5 m^3/year (0.7 yd^3/year) per meter of beach in temperate climates.
Sinks	
Inlets and lagoons	· May remove from 5 to 25 percent of the gross longshore transport rate per inlet. Depends on inlet size, tidal flow characteristics, and engineering influences.
Washover	· Less that 2.5 m^3/year (3.2 yd^3/year) per meter of beach, and limited to low-profile beach environments.
Offshore transport	· Quantity uncertain. Net contributions can be estimated from historical bathymetric change data.
Submarine canyons	· Where present, may intercept up to 80 percent of gross longshore sediment transport.
Aeolian transport	· Usually less than 5 m^3/year (6.5 yd^3/year) per meter of beach.
Dredging	· May equal or exceed gross longshore transport in some localities.
Convective Processes	
Longshore transport (waves)	· May result in accretion of gross longshore sediment transport, erosion of net longshore sediment transport, or no change depending on conditions of equilibrium.
Tidal currents	· May be important at mouth of inlet and vicinity, and on irregular coasts with a high tidal range.
Wind	· Longshore wind transport is important only in limited regions.

impact of nearshore sand mining on beach response, seawall placement on adjacent shoreline change, or jetty construction, which interrupts the longshore transport of sediment, on downdrift reaches of coast. Many examples of coastal sediment budget analyses exist (e.g., Bowen and Inman 1966; Caldwell 1966; Pierce 1969; Stapor 1973; Jarrett 1977; Headland, Vallianos, and Sheldon 1987; Jarrett 1991; Simpson, Kadib, and Kraus 1991; and others), however, the sediment budget presented below is of particular significance because an inlet system is a critical component of the analysis in the study area.

(2) As part of a feasibility and environmental assessment report for evaluating the impacts of harbor improvements at Morehead City, North Carolina, on regional coastal response, the U.S. Army Engineer District, Wilmington, summarized shoreline processes in the study area and performed a coastal sediment budget analysis to quantify the volumes of material moved by coastal processes (U.S. Army Engineer District, Wilmington 1990). The time period covered by the analysis was 1980 to 1988, and overall results of the budget were compared with previous studies to determine the impacts of a channel deepening project (1978) on adjacent shoreline response. The study area was divided into three reaches (Bogue Banks, Beaufort Inlet, and Shackleford Banks) (Figure 4-1). For each reach, average annual volume change rates due to coastal processes and dredging procedures were quantified. Longshore transport rates were then calculated using volume change rates in combination with relative energy flux values determined at the boundaries of the reaches through a wave refraction analysis. Although significant effort goes into developing a sediment budget, it must be remembered that it is an estimate that can be in error by a factor of two or more depending on the detail of knowledge of coastal processes in the study area and historical rates of shoreline and bathymetric response. In addition, sediment budgets are determined for varying periods of time and represent average rates of change for those time intervals. They

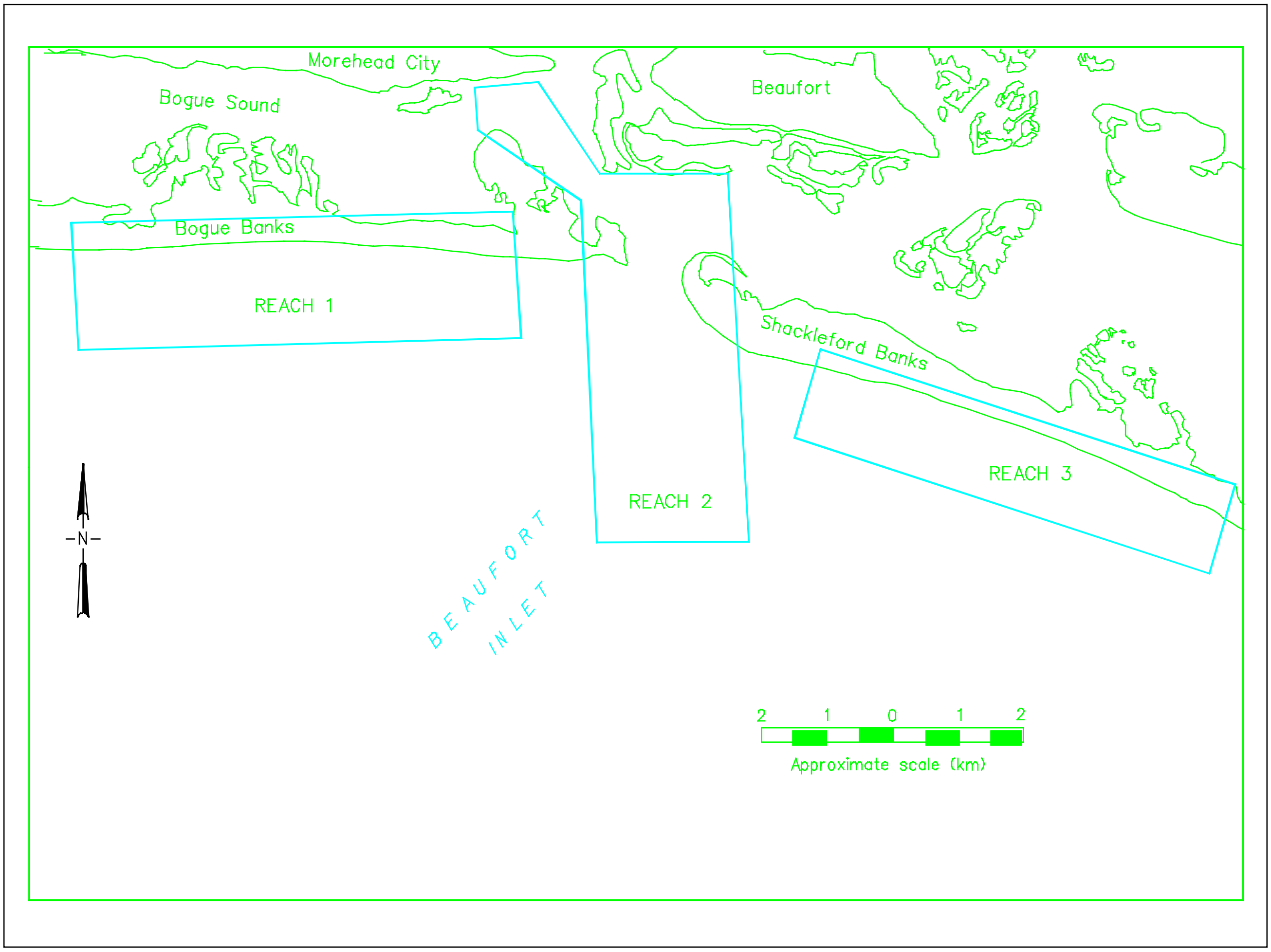

Figure 4-1. Study area showing the three sediment budget reaches

may not be indicative of changes in any one year. The following discussion is a summary of a revised sediment budget for the Beaufort Inlet area performed as part of a feasibility study for the Morehead Harbor Improvements project (U.S. Army Engineer District, Wilmington 1990), illustrating the practical use of this technique for assessing the potential impacts of engineering activities.

b. Environmental conditions. Wind-generated waves and currents, as well as tidal currents, are the primary processes affecting change in the study area. The study area is oriented east-west and predominant winds come from the southwest to south-southwest direction. Approximately 35 percent of the time, wind is blowing onshore with a mean speed of approximately 12 km/hr (7.5 mph). As such, the predominant direction of wave approach is from the southwest. Wave data used in the study were derived from Atlantic Coast Hindcast, Phase II Wave Information compiled by the Coastal Engineering Research Center for the period 1956 through 1975 (Station 42). Average significant wave height for this station is 1.3 m (4.3 ft); however, maximum wave heights of 4.7 m (15.5 ft) were predicted for the 20-year record. Mean tide level at Beaufort Inlet is 0.5 m (1.7 ft) with a mean tide range of 0.9 m (3.1 ft). The mean maximum flood current speed at the inlet channel entrance near Fort Macon (Figure 4-1) is 1 m/sec (2 knots), whereas the mean maximum ebb speed is 0.9 m/sec (1.8 knots). Note that these tide values are only averages; storm tidal heights and velocities can be four to five times higher.

c. Study site. The study area is located along the northeast margin of Onslow Bay (an open-ocean embayment between Cape Lookout and Cape Fear, NC), seaward of Morehead City, NC, and west of Cape Lookout. The three study reaches are included within an area approximately 21 km (13 miles) long (Figure 4-1) and contain fine sand barrier island beaches. Control volumes extend approximately 1,370 m (4,500 ft) offshore from the shoreline to an average depth of -10.7 m (-35 ft) MSL. The Bogue Banks reach extends 7,380 m (24,200 ft) in an east-west orientation, whereas the Shackleford Banks reach extends 7,100 m (23,300 ft) in a northwest-southeast direction. A groin constructed along eastern Bogue Banks at Fort Macon in the early 1850s is the only coastal structure present along the outer coast. The Beaufort Inlet reach is the largest control volume in the sediment budget study, encompassing the inlet channel, the Morehead City Harbor area, the ebb-tidal shoal, the Fort Macon beach area, and Shackleford Point.

d. Shoreline position and beach profile volume changes.

(1) The first component of a strategy for quantifying the sediment budget is to determine average annual volume change rates for each part of the study area. For the study at Beaufort Inlet and vicinity, volume changes were divided into two categories based on changes along the barrier island shorelines and changes associated with inlet and harbor areas.

(2) For the analysis period (1980-1988), profile data were available for quantifying volume changes associated with shoreline position change along Bogue and Shackleford Banks. Onshore and offshore sediment volume differences were calculated separately from the shoreline to an average depth of 10.7 m (35 ft) msl (≈1,370 m (4,500 ft) from baseline). The offshore length of the profile included the active littoral zone, such that differences calculated would indicate total volume changes. Rates of shoreline position change also were calculated from the beach profile data and compared favorably with existing change rates (see U.S. Army Engineer District, Wilmington (1990)). However, volume change information compiled prior to the interval 1980-1988 relied on comparisons of historical shoreline position for estimating volume rates of change. Consequently, sediment budget calculations performed for earlier time intervals may yield different results relative to variations in technique, regardless of natural changes.

(3) The onshore and offshore portions of the active beach profile on Bogue Banks showed accretion for the period 1980-1988. Shoreline movement averaged 22.0 ft/year, while onshore and offshore volume change averaged 132,000 and 255,000 m^3/year (172,000 and 334,000 yd^3/year), respectively (Figure 4-2). Overall, approximately 387,000 m^3/year (506,000 yd^3/year) of sediment accumulated in Reach 1 for the study period. Most of this increase in sediment volume was related to a beach replenishment project at Atlantic Beach in 1986 totaling

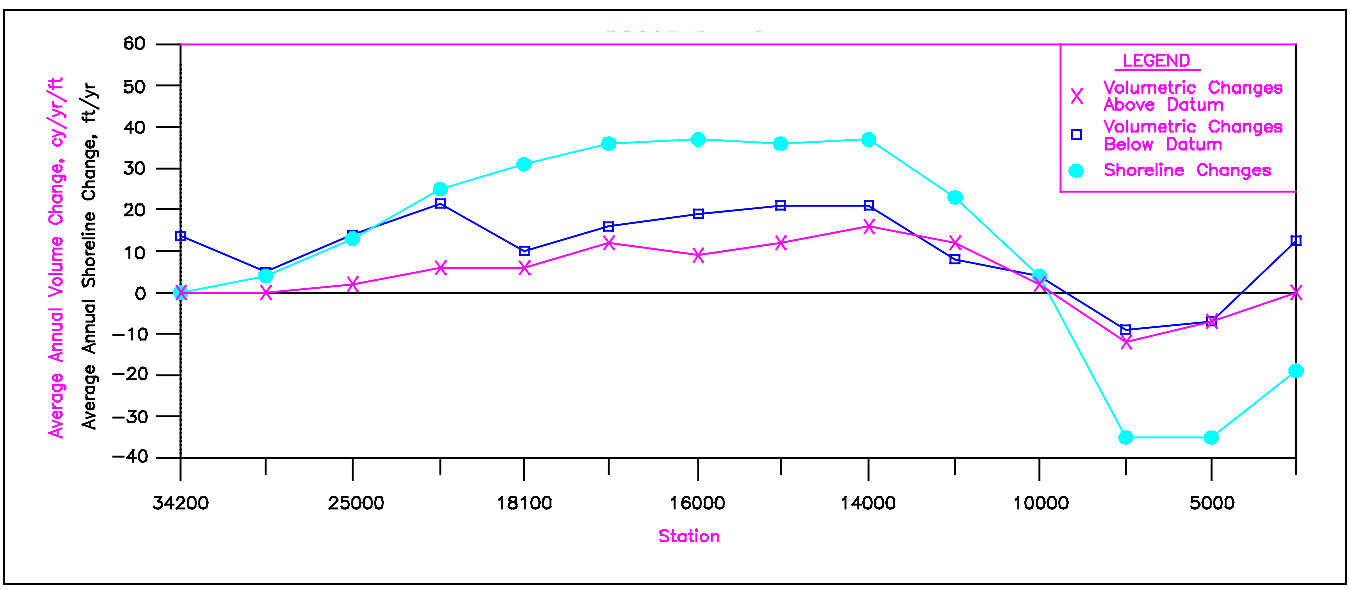

Figure 4-2. Shoreline position and volume change, Bogue Banks, NC

3.0 million m^3 (3.9 million yd^3). Annualized for the study time period, this volume of material amounts to a 373,000 m^3/year (488,000 yd^3/year) addition to the area.

(4) For Shackleford Banks, the magnitude of change was quite different. For the same time period, average shoreline movement showed net retreat (-0.70 m/year; -2.3 ft/year), and onshore volume change reflected this change (-23,000 m^3/year; -30,000 yd^3/year) (Figure 4-3). However, offshore profile volume change illustrated net accretion (95,500 m^3/year; 121,000 yd^3/year), resulting in a net addition of sediment to Reach 3 of 69,600 m^3/year (91,000 yd^3/year). Overall, the barrier island littoral zone compartments in the study area are stable for the time period of analysis.

e. Sediment volume changes near Beaufort Inlet.

(1) Shoreline changes within 610 m (2,000 ft) of Beaufort Inlet were included in the analysis of sediment volume changes in Reach 2 because shoreline movement in this area is influenced by inlet processes and responds differently than open-coast shorelines in Reaches 1 and 3. Beach profile data were supplemented using aerial photography digitized to determine area changes for Fort Macon and Shackleford Point.

(2) For the period 1978 to 1988, the Fort Macon region accreted at an average rate of 9,900 m^3/year (13,000 yd^3/year). Because only area and shoreline position change can be quantified using photography, volume change associated with shoreline adjustments had to be estimated based on change rates multiplied by the vertical distance between the shoreline and closure depth times the longshore distance covered by the control volume (SPM 1984). The estimated amount of change for the area was partially the result of deposition of 920,000 m^3 (1.2 million yd^3) of material in 1978.

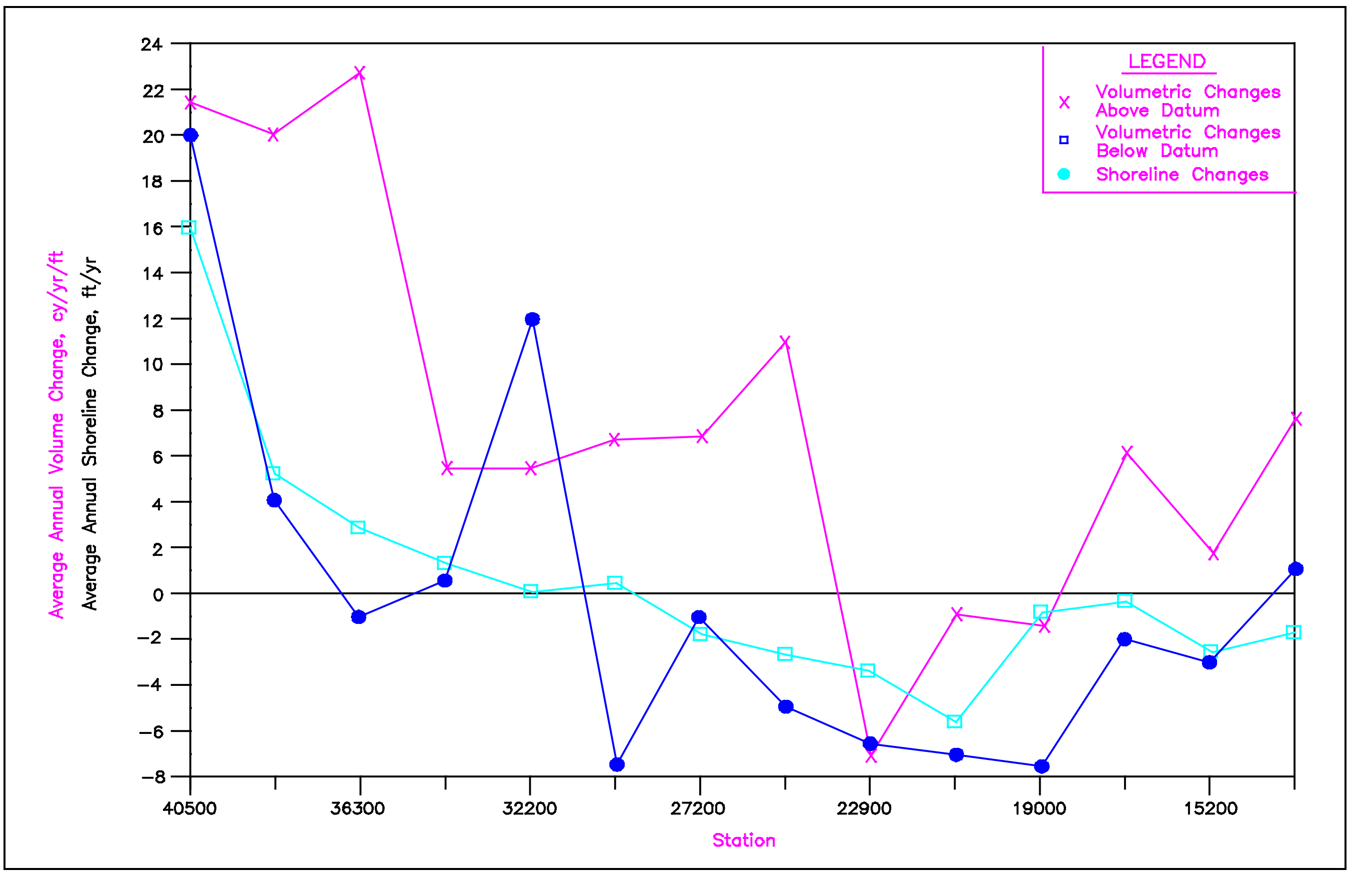

Figure 4-3. Shoreline position and volume change, Shackleford Banks, NC

(3) The western end of Shackleford Island (Shackleford Point) was analyzed for area change using aerial photography for the same time period as the Fort Macon shoreline. Using the same conversion procedures listed above, sediment volume change was estimated at -14,500 m^3/year (-19,000 yd^3/year), the opposite trend shown for the beach at Fort Macon.

(4) Sediment volume change on the offshore bar (ebb-tidal shoal) was considered one of the most critical components of the sediment budget because previous analyses indicated that the shoal was deflating at a rapid rate. Comparisons using digitized bathymetric data were made for the period 1974 to 1988 for an approximate 3.2-square-km (1.25-square-mile) area limited by the extent of the 1988 survey. After making adjustments for overlap with dredging activities and prorating net volume change to cover the same area included in the 1976 General Design Memorandum (GDM) sediment budget, it was determined that the net annual volume loss from the ebb-tidal shoal was 210,000 m^3 (274,000 yd^3). This value is slightly less than but consistent with that from the 1936 to 1974 sediment budget analysis. It was stated that if shoal deflation continued at its then current rate, it was possible that the wave climate impinging on the shoreline might change, causing increased wave energy and erosion. Volume changes for deposits in Back Sound were taken from the 1976 GDM and assumed representative for the period 1980 to 1988. This is supported by the fact that dredging volume in the inner harbor had not increased substantially since the harbor was deepened in 1978.

(5) Channel dredging is a large component of sediment movement in the inlet reach. Annual pipeline and hopper dredging volumes for this area are provided in U.S. Army Engineer District, Wilmington (1980) for the shoal, channel, and back-barrier navigation channel. From these data, the average annual dredge volume from the ebb-tidal shoal was determined to be approximately 550,000 m^3 (716,000 yd^3). Pipeline dredging volumes from interior channels behind the islands averaged 137,000 m^3/year (179,000 yd^3/year) for the period 1980 to 1988.

f. Wave energy flux analysis.

(1) Estimating the distribution of wave energy, particularly at the boundaries of coastal compartments, is an important component of any sediment budget analysis. To encompass the impacts of variable nearshore bathymetry on wave transformation along the coast, the finite-difference numerical model RCPWAVE (Ebersole, Cialone, and Prater 1986) was used to generate information on breaker wave height, breaker angle, and wave number. This information was used to predict wave energy flux at the break point so that potential sediment transport rates in and out of a sediment budget compartment, as well as at discrete longshore positions within a reach, could be calculated.

(2) Results from the analysis for the reaches along Bogue and Shackleford Banks indicated a relatively even distribution of wave energy. The eastern side of Bogue Banks is most influenced by waves out of the southwest, whereas the western portion of the island is more influenced by waves out of the east-southeast. Conversely, the shoreline response along Shackleford Banks primarily is controlled by waves from the south-southeast. Results obtained for areas near the margin of Beaufort Inlet show greater wave variability than those found along open-ocean beaches, likely the result of rapidly changing bathymetric contours that influence wave transformation and energy flux.

(3) Numerical model results also suggest that wave energy entering the inlet from the west is three times that coming from the east. For Bogue Banks, the energy flux from the west is relatively constant near the central portion of Reach 1 and then increases significantly towards the inlet. Along Shackleford Banks, very little energy is propagated from the east, due in part to sheltering by Cape Lookout. These trends are supported by inlet shoaling patterns which indicate that approximately 70 percent of sediment dredged from the Beaufort Inlet channel comes from the west. Total wave energy flux values at reach boundaries are used in the sediment budget equations presented in the next section to determine longshore sediment transport rates into and out of the inlet reach.

g. Sediment budget. After determining all the average annual volumetric change rates and the relative energy flux at reach boundaries, the parameters were combined by reaches to calculate three unknown annual volumetric rates: longshore transport rate (QE), volume rate bypassing to the east (BE), and volume rate bypassing to the west (BW). Table 4-4 provides a summary of known sediment budget volume change rates for each reach for the 1980 to 1988 time period. One sediment budget equation was established for each reach based on the information provided above. Coefficients for the longshore sediment transport (QE) values represent the relative energy flux values at reach boundaries. Figure 4-4 shows the volume relationships between the

Table 4-4
Sediment Budget Volume Change Rates (after U.S. Army Engineer District, Wilmington (1990))

	Volume Change	
Parameters	m^3/year	yd^3/year
Reach 1 - Bogue Banks		
Beach Replenishment (REPL)	+373,000	+488,000
Total Volume Change (VC1)	+387,000	+506,000
Reach 2 - Beaufort Inlet		
Channel Dredging Near Ebb-Shoal (DRED)	-548,000	-716,000
Dredging in Back Sound (BSND)	-137,000	-179,000
Back Sound Loss (from 1976 GDM) (BSND)	-44,000	-58,000
Fort Macon Volume Change (FMVC)	+9,900	+13,000
Shackleford Point Volume Change (SPVC)	-14,500	-19,000
Volume Change on the Ebb-Tidal Shoal (VC2)	-210,000	-274,000
Reach 3 - Shackleford Banks		
Total Volume Change (VC3)	+70,000	+91,000

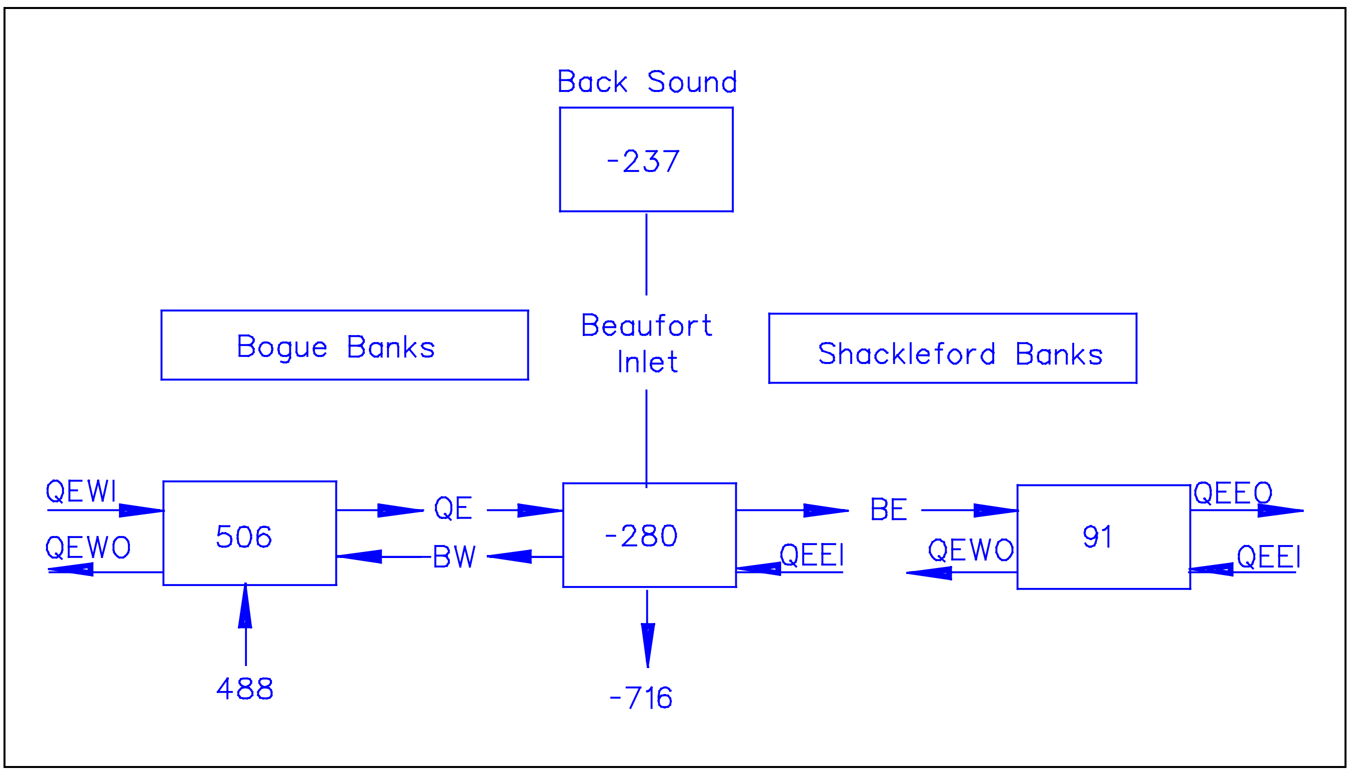

Figure 4-4. Sediment budget reaches and volumes (numbers × 765 m^3/year (1000 yd^3/year))

reaches. The following equations can be solved simultaneously to determine the three unknowns. They are:

Reach 1 - Bogue Banks

$$1.43\ QEWI - 0.29\ QEWO + REPL + BW - 1.0\ QEEO = VC1$$

$$0.14\ QE - 17 + BW = 0 \tag{4-1}$$

Reach 2 - Beaufort Inlet

$$0.29\ QEEI + 1.0\ QEWI - BSND - DRED - BE - BW = TVC2 \tag{4-2}$$

where

$$TVC2 = VC2 + FMVC + SPVC$$

$$1.29\ QE - 673 - BE - BW = 0$$

Reach 3 - Shackleford Banks

$$-0.65\ QEEO + 0.25\ QEEI + BE - 0.29\ QEWO = VC3$$

$$-0.69\ QE - 91 + BE = 0 \tag{4-3}$$

where

QEWI - transport into the reach, west side.

QEWO - transport out of the reach, west side.

QEEI - transport into the reach, east side.

QEEO - transport out of the reach, east side.

After inserting the values in Table 4-4 into the above equations and solving simultaneously, longshore transport (QE), transport bypassing to the east (BE), and transport bypassing to the west (BW) were determined as:

QE = 806,700 m^3/year (1,055,000 yd^3/year)

BE = 610,200 m^3/year (798,000 yd^3/year)

BW = -100,200 m^3/year (-131,000 yd^3/year)

The negative value for bypassing to the west indicates that the transport direction assumed in Figure 4-4 was opposite of the actual direction, suggesting that no sand is bypassing Beaufort Inlet from east to west.

h. Results of the analysis. Using measured volume change rates in combination with wave energy flux estimates provided a means of assessing the magnitude of longshore sediment transport and sediment bypassing at Beaufort Inlet. Two critical findings evolved from this analysis: 1) sediment was only bypassing the inlet from west to east, potentially providing material to beaches on Shackleford Banks, and 2) the ebb-tidal shoal area was deflating at a fairly rapid rate. Both of these observations were consistent with conclusions from previous studies. From these results, one can infer that certain human-induced processes may be adversely impacting the evolution of this coastal system relative to natural conditions. With this information, appropriate actions can be taken to alleviate future problems. Without performing a sediment budget analysis, pertinent findings may have been inadvertently missed.

i. Alternate approach. Increased capabilities in the areas of shoreline position change simulations (Grosskopf and Kraus 1994) and surface modeling software for analyzing temporal trends in cut and fill for integrated shoreline and bathymetry data sets (Byrnes and Hiland 1994) provide an automated approach for assessing coastal sediment budgets. In the analysis performed by the U.S. Army Engineer District, Wilmington (1990), volume change data for the ebb-tidal shoal were estimated by calculating differences among discrete areas represented by an average of a number of bathymetric data points rather than using the entire data set and subtracting surfaces. Analysis of change associated with entire data sets using recently developed surface modeling software provides a more accurate estimation of change, particularly in an area as critical as a navigation entrance. Of course beach profile data, integrated with shoreline position data, could be analyzed in a similar manner. Probably the most critical estimated parameter in a sediment budget analysis is the longshore transport rate. For the study at Beaufort Inlet and others (e.g., Headland, Vallianos, and Sheldon 1987), wave energy flux is calculated at the boundaries of sediment budget compartments for determining the potential rate of longshore sediment transport. Shoreline change numerical models provide a more realistic assessment of these rates because model calibration is dependent upon historical shoreline position data. In other words, potential sediment transport rates must be consistent with shoreline change data to produce reliable model output. Consequently, if model calibration is successful, longshore sediment transport rates at sediment budget reach boundaries would be more reliable than calculated potential transport rates from wave energy flux measurements that cannot be tested for accuracy.

4-4. Shoaling Rates

As noted in the previous discussion, two primary components of the sediment budget analysis were channel dredging and maintenance associated with Back Sound and the ebb-tidal shoal at Beaufort Inlet. Because sediment from these types of areas often represents large annual volume changes within the budget, measurement and prediction of shoaling is critical to planning and design of navigation improvements. Economic feasibility of any navigation project depends to a large extent on future channel dredging needs, and accurate prediction of sedimentation rates is a critical part of project planning. Due to the significance of this parameter related to sediment budget determinations and operation and maintenance procedures, a brief discussion is presented below regarding techniques used for predicting shoaling rates. Portions of the following discussions are taken directly from Sorensen (1992).

a. Prediction techniques.

(1) There are many analytical and empirical methods for shoaling rate prediction (Sorenson 1992), but there are no widely accepted techniques. Many of the empirical methods are site-specific, and the theoretical methods often contain simplifying assumptions which limit their applicability. Calculation of shoaling rates depends on assumptions in the method applied and coastal processes in the region of interest. For the purpose of classifying sedimentation processes, a navigation channel from offshore into the back bay or harbor region may be subdivided into four sections. The first is the offshore section located seaward of the surf zone; and the second is the offshore section in the surf zone, but seaward of the region in which significant inlet-induced ebb/flood tidal currents control sediment movement. Depending on inlet entrance geometry and wave climate, the second section may not exist. The portion of the inlet in which sediment transport and resulting channel conditions are dominated by flow through the entrance is the third section. The fourth section is in the harbor interior in which turbulence levels and current velocities are reduced and net deposition of sediment transported into the back bay or harbor takes place.

(2) In the sediment budget example presented herein, the U.S. Army Corps of Engineers, Wilmington District (1990) used historical data to assess the magnitude and rate of shoaling for the Beaufort Inlet entrance channel by evaluating dredging records and bathymetric surveys. This procedure works well; however, its applicability is limited to one area with an excellent record of historical information. If a study were being undertaken in an area with sparse data coverage, the prediction technique would decrease in reliability with proportion to data availability. Clearly, a universal prediction technique based on dynamic processes influencing sedimentation at entrances would be most useful for any inlet system. However, the complexity of sediment-flow interaction at inlet channels has limited the effectiveness of analytical techniques.

(3) The offshore and surf zone sections of the harbor will be discussed herein. For additional discussion of wave and tidal flow-controlled stability conditions at inlets, the reader is directed to Bruun (1978), Escoffier (1977), Jarrett (1976), and Sorensen (1977). Gole, Taraport, and Gadre (1973); Lin and Mehta (1989); Marine Board (1983); and McDougal and Slotta (1986) discuss sedimentation in interior channels and docking slips.

b. Example application - offshore (nonbreaking conditions).

(1) Kadib (1970, 1976, 1991) developed a simple and rational method based on theory and laboratory studies for describing shoaling in dredged channels given nonbreaking wave and current data in the vicinity of the channel. The method was field verified by monitoring the sedimentation rates at a test trench at Morro Bay Harbor entrance in California (Kadib 1993). Kadib's method first assumes that the basic flow field near a channel may be described with two primary processes: 1) a steady current with an average velocity u_1 at water depth d_1 (by continuity, this current will have a velocity u_2 at d_2), and 2) a maximum oscillatory current at the bed due to wave action. These processes were considered the most important factors contributing to sediment movement near a channel. Kadib took this basic premise and, given wave height, wave period, and wave length, calculated bed load and suspended load transport rates using transport relationships developed by Einstein (1950, 1972) and Abou-Seida (1965). The bed-load transport rate Q_b was determined as a function of the sediment concentration in the bed layer C_a and the local current velocity near the bed u_c. Assuming that bed load takes place within a certain bed layer, the concentration of suspended sediment C_h at a distance h above the bed can be determined (Einstein 1950). Once this value is determined, the total suspended load Q_s on the updrift side of the channel and inside the channel can be estimated.

(2) To calculate the rate of sediment deposition per unit width of channel (Q_d), Kadib assumed two primary processes would take place as sediment transported in the direction of a channel encounters the channel. First, the

channel will act as a sand trap for bed load, and second, the current carrying suspended load on the updrift side of the channel will reduce its capacity across the channel due to a decrease in the steady flow velocity, depositing sediment in the channel. Thus, the channel shoaling rate can be represented as the difference between the rate of transport of suspended load reaching the channel (Q_{s1}) and the transport rate across the channel (Q_{s2}), plus the rate of bed-load transport at the channel edge (Q_b), or

$$Q_d = (Q_{s1} - Q_{s2}) + Q_b \tag{4-4}$$

Although rather simplified in context, this approach provides a reasonable analytical technique for estimating channel shoaling rates for noncohesive sediment. In addition, it is not specific to a given inlet environment and thus has greater utility towards understanding and predicting rates of shoaling in channels.

c. Example applications - surf zone (breaking conditions).

(1) The SPM (1984) summarizes procedures for predicting longshore sediment transport rates in the surf zone. Given representative wave conditions for a period of time, the longshore transport rate can be calculated as a volumetric transport rate, or as an immersed weight rate. The SPM energy flux method empirically relates the wave power and longshore transport; however, the mechanics of sediment transport are not considered. Komar (1977) uses a relationship that considers both wave action to suspend sediment and wave-induced longshore current to transport sediment. Although equivalent to the SPM approach, Komar's method does have an advantage since it separates out wave and current effects which may be individually evaluated at points adjacent to and in the channel to calculate respective transport rates and resulting net channel deposition rate. The Komar and SPM methods require similar information: knowledge of the incident wave height, period, and angle with respect to the shoreline at the breaker point; water depths in and adjacent to the channel; sediment density; and an estimate of the in-place porosity of the sediment.

(2) Galvin (1979) developed a simple procedure to examine shoaling at Moriches Inlet, New York. The method estimates the portion of the approaching longshore surf zone sediment transport that will deposit in a channel cut across the bar. The channel cut is assumed to be around an inlet entrance whose ebb tidal flow affects the deposition rate in the bar channel. The U.S. Army Engineer District, Wilmington (1980) also developed a method for predicting the shoaling rate in a channel dredged across the bar offshore of a tidal inlet. A regression analysis of field data from four North Carolina inlets was conducted to relate the bar cut siltation rate with three influencing factors; ebb tidal flow energy, incident wave energy, and sediment entrapment potential, which depends on channel depth.

d. Empirical methods.

(1) Purely empirical methods available for sedimentation prediction do not consider wave and current conditions or local sediment characteristics, but make projections based on historic dredging records for the existing channel. These methods relate previously dredged volumes to time elapsed and pertinent channel geometry features.

(2) Vincente and Uva (1984) present a method that assumes the siltation rate is proportional to the difference between the existing bottom elevation in the channel section and the equilibrium bottom elevation in the channel section for which no deposition will occur. Trawle and Herbich (1980) applied the "volume of cut" procedure to six Atlantic, gulf, and Pacific coast harbor entrance channels where adequate historic dredging records were available. The analysis related percent increase in the volume of cut from the previous channel dimension to the new channel dimension, which therefore indicated a subsequent increase in the dredging requirement.

(3) The U.S. Army Engineer District, Portland (Hartman 1977) developed a method from historical surveys and dredging records. The empirical method predicts controlling dimensions in a navigation channel which result from dredging activities at different times and depths. It assumes that a structurally controlled entrance will have infill or scour rates for a specific depth under similar ocean and river conditions, and that ocean and river conditions are constant during any one month, year to year. A table of shoaling rates is developed, and a "typical" natural channel control dimension curve is generated.

e. Concluding remarks.

(1) Accurate analytical predictions of channel shoaling rates are difficult. This difficulty arises primarily for two reasons: sedimentation processes in navigation channels are complex, and thus development of accurate analytical techniques is difficult; and reliable estimations require a significant amount and variety of input data. An alternate approach to analytical predictions is to employ purely empirical techniques using historic data on

deposition at the project channel or a nearby channel with similar characteristics.

(2) Several methods were discussed herein for the purpose of giving the reader a brief overview of shoaling rate prediction techniques. The broad applicability of these and other methods is presently under investigation at the Coastal Engineering Research Center. The reader is cautioned that, although a given method may be very accurate at one inlet, its application to other locations may result in unreliable predictions. For a more detailed description of the methods, their development and assumptions, the reader is directed to Sorenson (1992).

Chapter 5
Design Analysis of Tidal Inlets

5-1. Introduction

a. Design considerations.

(1) Engineering design of coastal inlets typically involves either improving an existing inlet or developing a new inlet. In either case, engineers must realize that the design project is in a dynamic environment where natural processes are not completely understood. Extreme care should be exercised with any alteration to existing shoreline and bathymetric configurations. Not necessarily all physical changes will upset the natural environment; the ability to anticipate project impacts and implement appropriate measures to alleviate adverse effects is the key to successful design practice. Mathematical and physical models are important tools to be applied in inlet design analysis.

(2) It is equally important that the designed features perform their intended functions with minimum maintenance requirements. Design criteria should be established to guide the design of each feature for both functional performance and structural integrity under adverse environmental conditions such that project benefits will be maximized. A net-benefit optimization analysis is required to determine the economic optimum design.

b. Design features. Most designs for tidal inlets are navigation or navigation-related projects but consideration is also given to water quality improvement, sediment control, and recreation. Structural improvements for navigation projects may include the construction of jetties, breakwaters, or bulkheads and revetments. Jetties and breakwaters have similar structural configurations but differ in performance functions. Jetties are designed mainly to prevent navigation channels from shifting and shoaling while breakwaters are built to reduce wave energy in sheltered areas. With these objectives in mind, the alignment, layout, and length of the structure elements must be analyzed for optimum performance. Bulkheads and revetments are shoreline erosion protection structures. Nonstructural navigation improvements include channel dredging and sand bypassing.

c. Physical environmental data. Physical environmental data are required for better understanding of natural transport processes at inlet systems, development of design criteria, assessment of functional performance of designed structures, and determination of impacts on natural littoral processes within the tidal inlet system. These data include: tidal elevations and currents, freshwater inflows, winds and waves, water quality parameters, bathymetry, and geological information. In addition, weather data related to visibility and ice information may also be needed for the design analysis.

d. Sources of information. A substantial quantity of environmental data is available in the public domain and can be obtained from public or university libraries, Government agencies, and data retrieval and referral centers. Tidal data are readily available from the publications of Tide Tables, Tidal Current Tables, and Tidal Bench Marks by the NOS. Field measurements of tidal currents should be planned for engineering analysis. River flows into the tidal basin, sediment loads, and other water quality parameters are provided in the *Water Resources Data* published annually by the U.S. Geological Survey. Nautical charts and bathymetric maps published by NOS are usually adequate for preliminary engineering design and analysis. Baseline bathymetric surveys should be scheduled for planning and design purposes. Coastal geologic data, and wind and wave data generally are scarce, but Chu, Lund, and Camfield (1987) provide a listing of useful data sources for design analysis.

5-2. Navigation Channel Design

a. General. Engineering analysis of navigation channels involves identifying appropriate design criteria, determining the most economical channel dimensions, analyzing of dredging requirements, and determining dredging effects on overall inlet stability. Only entrance channel design analysis is discussed in this manual. EM 1110- 2-1613 and EM 1110-2-1615 should be consulted in the formulation of channel features. The following factors influencing channel design need careful evaluation: design vessel; tides and design water levels; winds, waves, currents, and sedimentation.

b. Design vessel. The design vessel or vessels are selected from comprehensive studies of the various types and sizes of vessels expected to use the project during its design life. Channel dimensions should be selected to safely and efficiently accommodate the amount and type of traffic anticipated. The design vessel is selected by evaluating trade-offs of the delay cost incurred by larger vessels and cost of increased channel dimensions. The maximum size vessel and least maneuverable vessel in the fleet must be able to make a safe transit; however, the following special conditions may be important considerations:

(1) Suitable wind, wave, and current conditions and visibility.

(2) Use of high tide for additional water depth.

(3) Speed restrictions to reduce squat, ship-generated wave heights, and shore damage.

(4) One-way traffic.

(5) Tugboat assistance.

(6) Provision of anchorage area.

c. Tides and design water levels. The NOS publishes tide height predictions and ranges. Figure 5-1 shows spring tide ranges for the continental United States. Historical records on tidal elevations, including extreme high water, mean higher high water, mean high water, mean tide level, mean low water, mean lower low water, and extreme low water, may be found from Tidal Bench Marks published for each NOS tide station. Cumulative probability of tidal elevations prepared by Harris (1981) can be useful in the analysis of frequency and duration of ship delays. In addition to ocean tides, water level is also affected by storm surges, seiches, and river discharges. Design water level may vary with design functions of specific project features. Lower low water levels are normally used to determine available and needed depths for various size vessels and designs for structure toes. High-water levels are used to determine wave penetration, structure height, and armor layer design.

d. Winds, waves, and currents.

(1) Estimates of winds, waves, and currents are needed to determine their effects on vessel motions and controllability, and to estimate sediment movement in the project area. Wind data are available from the National Climatic Data Center (Federal Building, Asheville, NC 28801). Estimates of wind waves and vessel-generated waves are needed for various elements of project design. Predictions of wind-generated waves can be made by using the techniques presented in EM 1110-2-1414. Vessel-generated waves can be estimated with methods presented in EM 1110-2-1615. Coastal currents are affected by tides, river discharges, seiche motions, wind waves, and coastal structures. Tidal currents published by NOS may be adequate for preliminary project analysis. Improvements such as dredging and jetty or breakwater construction will affect current conditions in the project area. Mathematical or physical model simulations of current as well as wave distributions may be necessary for detailed design analysis. Simulations of ship transits may be required to ensure that the channel design is in compliance with functional design criteria.

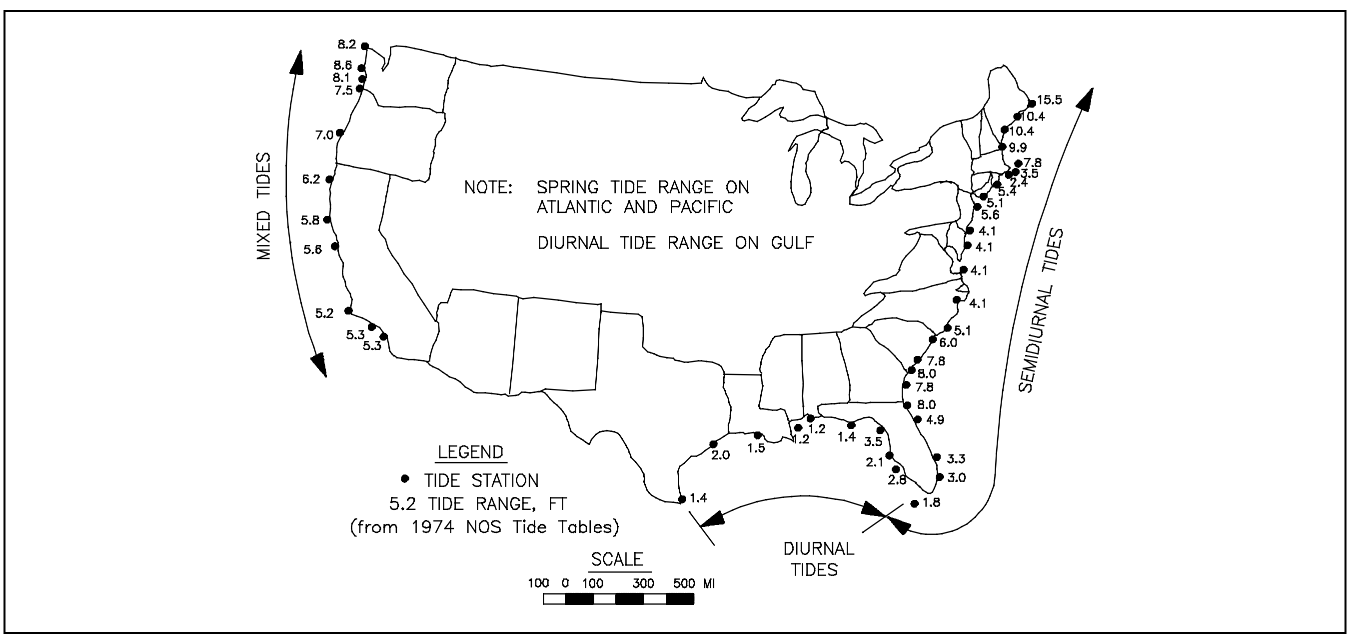

Figure 5-1. Ocean spring tide ranges

(2) One design problem of note, particularly of concern to small boat harbors, is the breaking of waves in the entrance channel during higher wave energy and/or higher flow discharge events. The following guidance is summarized by the Members of the Task Committee on Marinas 2000 (American Society of Civil Engineers (ASCE) 1992). Linear wave theory can be used to calculate a minimum breaking wave depth (SPM 1984) based on the design wave (or waves) to establish a minimum safe channel depth. Analytical methods are available to calculate the effects of ebb tidal flow and/or river flow given incident wave conditions (SPM 1984). By then adding allowances for design vessel motions such as roll, pitch, and heave (EM-1110-2-1615) a minimum depth for safe navigation can be specified. Adverse entrance conditions, caused by waves, can be minimized through the following: greater entrance depths, which will allow higher storm waves before breaking; channel widths widened to allow more maneuvering room during higher sea conditions (EM 1110-2-1615); structures such as jetties can be extended offshore to reach deeper depths thus allowing higher unbroken waves across the ebb bar; and offshore structures such as breakwaters can be constructed to provide shelter to the entrance (EM 1110-2-2904). Many small boat harbor designs have been evaluated and improved based on physical model tests (Bottin 1992).

e. Sedimentation. Aspects of sedimentation that must be considered include the characteristics and transport of native sediment as well as that of sediment introduced into the project area by littoral drift and river flow. Sediment budget and shoaling analyses should be performed before and after construction. These studies provide the basis for maintenance dredging requirements, and shoreline erosion and inlet stability control measures. Detailed discussions of sediment budget methodology are presented in Chapter 4, EM 1110-2-1502, and the SPM (1984).

f. Channel depth.

(1) Wave conditions. Allowance for wave action is required for design depth determination. For small craft, one half of the design wave height is generally adequate. Pitch, roll, and heave should be evaluated for larger vessels that use the channel. Large vessel motions can be determined by physical or mathematical model simulations or data from prototype observations. The effect on wave heights and directions in the channel due to depth change (shoaling and refraction over the ebb tidal delta) and the effect on wave period due to currents should be analyzed in the study of vessel motions and determining a safe entrance channel depth.

(2) Dredging tolerance. Dredging tolerance is taking into account the inaccuracies of dredging operations in marine environments in relation to the theoretical design channel cross section. Usually a value ranging from 0.3 to 0.9 m (1 to 3 ft) is used in contract specification.

(3) Advanced maintenance. Channel maintenance usually consists of removing sediment deposits from the channel bed. In channels where shoaling is continuous, overdredging is a means of reducing the frequency of dredging while providing reliable channel depth over longer periods of time. Advance maintenance consists of dredging deeper than the safe channel design depth to provide for accumulation and storage of sediment. Justification for advance maintenance is based on channel depth reliability and economy of less frequent dredging. Estimates of channel shoaling rates are used in the justification for advance maintenance dredging. Several depths should be considered to optimize the advanced maintenance allowance; however, deeper channels will tend to be more efficient sediment traps and could shoal more rapidly. Overdepth advanced maintenance eliminates the need for a dredging tolerance allowance.

g. Channel width. Factors to be considered in channel width design are discussed in EM 1110-2-1613. Certain shoaling patterns may warrant the consideration of advanced maintenance in the form of a channel widener. Such sediment traps are also justified based on the reduction of dredging frequency and increase in channel reliability. Navigation in the entrance channel is often affected by strong and variable tidal currents, rough seas, breaking waves, wind, fog, and other difficulties. Channel width, including advanced maintenance channel wideners in the entrance, should be judiciously selected based on an analysis and evaluation of conditions at each project. A review of methods for determining channel widths as presented in Corps of Engineers reports is included as Appendix B of EM 1110-2-1613.

h. Channel side slope.

(1) Significant factors in the design of side slopes for navigation channels include bottom soil type, slope location, seismic activity, and ease of construction. Most noncohesive soils will not stand at a slope angle greater than 45 deg. Cohesive soils will stand initially at much higher angles, but over a period of time, they tend to degrade. Table 5-1 shows various side slopes for underwater channels.

Table 5-1
Typical Side Slopes for Various Soil Types (Bray 1979)

Soil Type	Side Slope (V:H)
Rock	Nearly vertical
Stiff clay	1:1
Firm clay	1:1.5
Sandy clay	1:2
Coarse sand	1:3
Fine sand	1:5
Mud and silt	1:8 to 1:60

(2) In practice, it usually is found that characteristics other than inherent slope stability are the controlling factors. Consideration should be given to the slope location and whether the slope is totally or partially submerged. A partially submerged slope acts as a beach and therefore, is liable to assume a beach slope. Side slopes must be constructed by dredges in a manner which suits the dredging operation. In certain cases, very steep slopes are difficult and expensive to construct. In these circumstances, savings in dredging quantities may be completely offset by increase in unit cost of dredging. Generally slopes of 1:3 or less do not cause major dredging problems.

i. Channel dredging. Channel dredging involves initial construction to provide the design depth, with provisions for advance maintenance dredging, dredging tolerance, and periodic maintenance. Cost estimation for both construction and maintenance dredging should be made for various channel alignments and dimensions. Deep-draft channels usually are dredged by hopper dredges in areas exposed to wave action or where disposal is in exposed offshore or estuarine areas. Pipeline dredges are usually more economical with greater production with soft material but are restricted to protected or semi-protected areas. Dredged material can be disposed of in open water or behind confined dikes. Contaminated material is generally disposed of behind containment dikes with careful monitoring of return water quality. If the dredged material is of reasonably good quality, it should be considered for beach nourishment or landfill purposes. EM 1110-2-5025 provides guidance on dredging, disposal, and beneficial uses of dredged material.

5-3. Jetties

a. Design principles. A jetty system helps to deepen an inlet channel and reduce required dredging by concentrating and directing tidal currents to optimize scouring action. This is accomplished by confining discharge areas and making flow channels more hydraulically efficient, thereby promoting higher channel velocities. Jetties stabilize an inlet entrance by intercepting the littoral drift and preventing or minimizing deposition in the inlet channel. Jetties also minimize the effect of wave action and cross-currents on vessels transiting an inlet. As a permanent coastal structure protruding into the active littoral zone, jetties alter natural sediment transport processes. Construction of a jetty system includes features or provisions to mitigate any significant adverse effects, such as downdrift beach erosion or removal of valuable sand from the littoral system. EM 1110-2-2904 provides the structural design aspects of jetty systems.

b. Design theories.

(1) General. Existing jetty systems can be grouped into two basic designs, single jetties and twin jetties. The following discussion addresses the design theory and functional design criteria of each. (Note: In most cases, two jetties are needed to keep littoral drift from entering the channel. Because single-jetty systems have been found to be unsatisfactory, single-jetty construction is no longer recommended; however, the design theory of such systems will be presented as an aid in evaluating those already in existence.)

(2) Single jetties.

(a) A single straight jetty or curved jetty may be oriented perpendicular to the shoreline or may be placed at an angle with the shoreline depending on predominant wave direction, channel alignment of the natural inlet, and desired alignment of the improved inlet. A single updrift jetty is attached to shore on the updrift side of the channel entrance to act as a barrier to the movement of littoral drift alongshore from the net transport direction, as shown in Figure 5-2.

(b) Two variations to the basic single updrift jetty are the addition of a weir section and the Haupt jetty. A weir section is a low section with a crest elevation near mean sea level at the shore end. Sediment is transported over the weir by waves and currents into a deposition basin that is periodically dredged. With this design, the littoral drift from the updrift direction is trapped and localized in the basin before it reaches the navigation channel. Methods of deposition basin storage analysis, weir section design, and updrift beach profile design are provided by Weggel (1981). Figure 5-3 is a schematic of such a jetty system constructed at Masonboro Inlet, North Carolina. (Note: Due to unsatisfactory performance of the single-jetty system, Masonboro Inlet now has two jetties.)

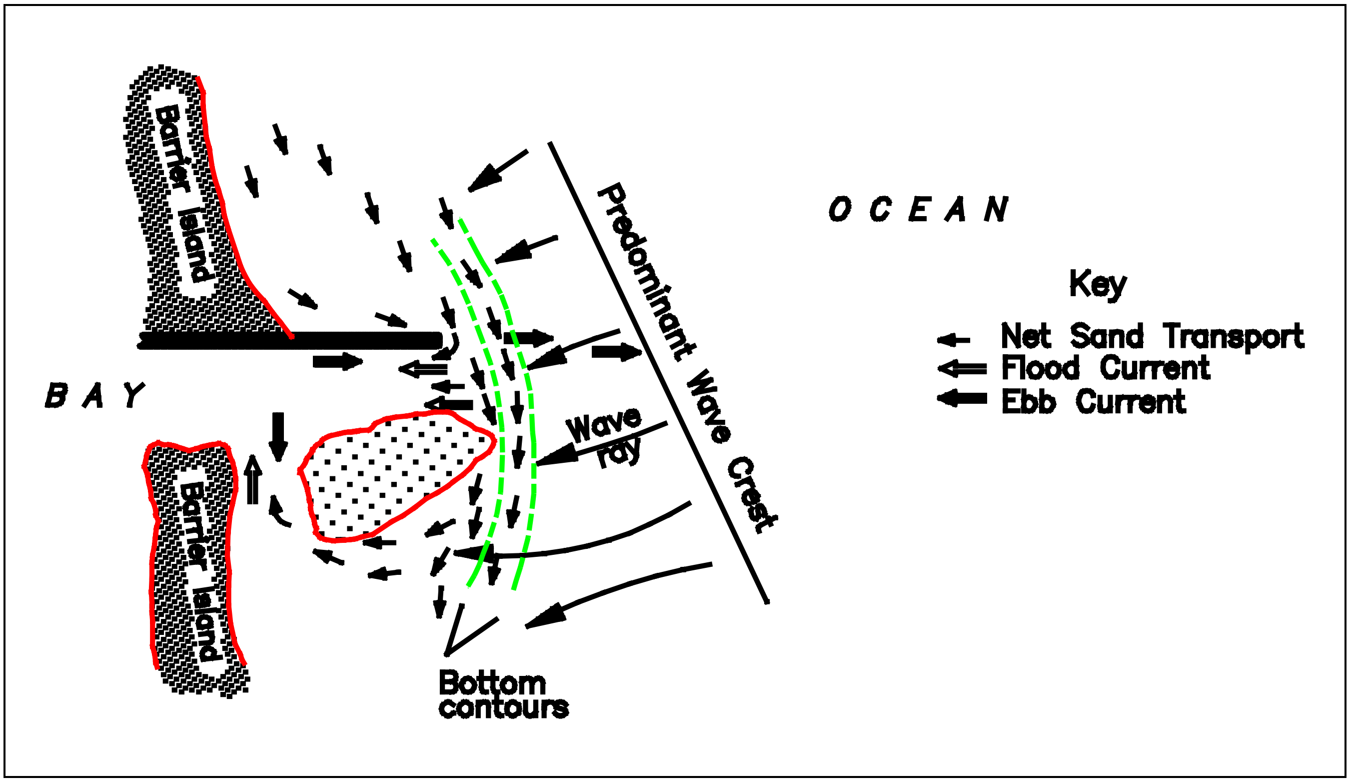

Figure 5-2. Schematic of single updrift jetty

(c) The Haupt jetty is a single curved jetty which is detached from the shoreline and located on the updrift side of an entrance channel. The jetty is concave to the main ebb-tidal currents to force the ebb current against the jetty and scour a well-defined channel. Being detached from shore, the system, as shown in Figure 5-4, readily admits flood currents to increase the tidal prism, thus permitting greater discharge through the channel during ebb tide.

(d) Single jetties located on the downdrift side of an inlet entrance permit the net longshore transport of sand from the updrift direction to force the channel against the jetty as shown by Figure 5-5. In this case, the ebb current controls channel scour activities. Kieslich (1981) discusses the response of entrance channel behavior following the construction of 13 tidal inlets in the United States. The study concluded that the construction of single jetties resulted in migration of the channel thalweg towards the jetty regardless of the inlet-bay orientation, angle of the jetty to the shoreline, position of the jetty relative to the direction of net longshore transport, the ratio of net-to-gross transport, or the gross transport.

(3) Twin jetties.

(a) The two jetties of a double-jetty system may be placed perpendicular to or at an angle with the shoreline; may be curved or straight and converging, diverging, or parallel; and may be equal or unequal in length, depending on the local conditions at the entrance. Figure 5-6 shows a typical twin-jetty system. A double-jetty system may be the original design or the later addition of a second jetty to a single-jettied entrance. Twin jetties are normally aligned parallel with the selected channel alignment; this design most effectively controls channel flow velocities. Converging alignments (arrowhead type) are generally not satisfactory since they are more costly to construct due to greater length, they do not reduce wave action more than parallel jetties, they trap more sediment, and they often allow channel meandering.

(b) The distance between jetties should be designed by considering channel width, maximum current speed within the inlet entrance, and stability of bottom material, as well as overall inlet entrance stability. Jetty lengths are determined by the channel project depth and

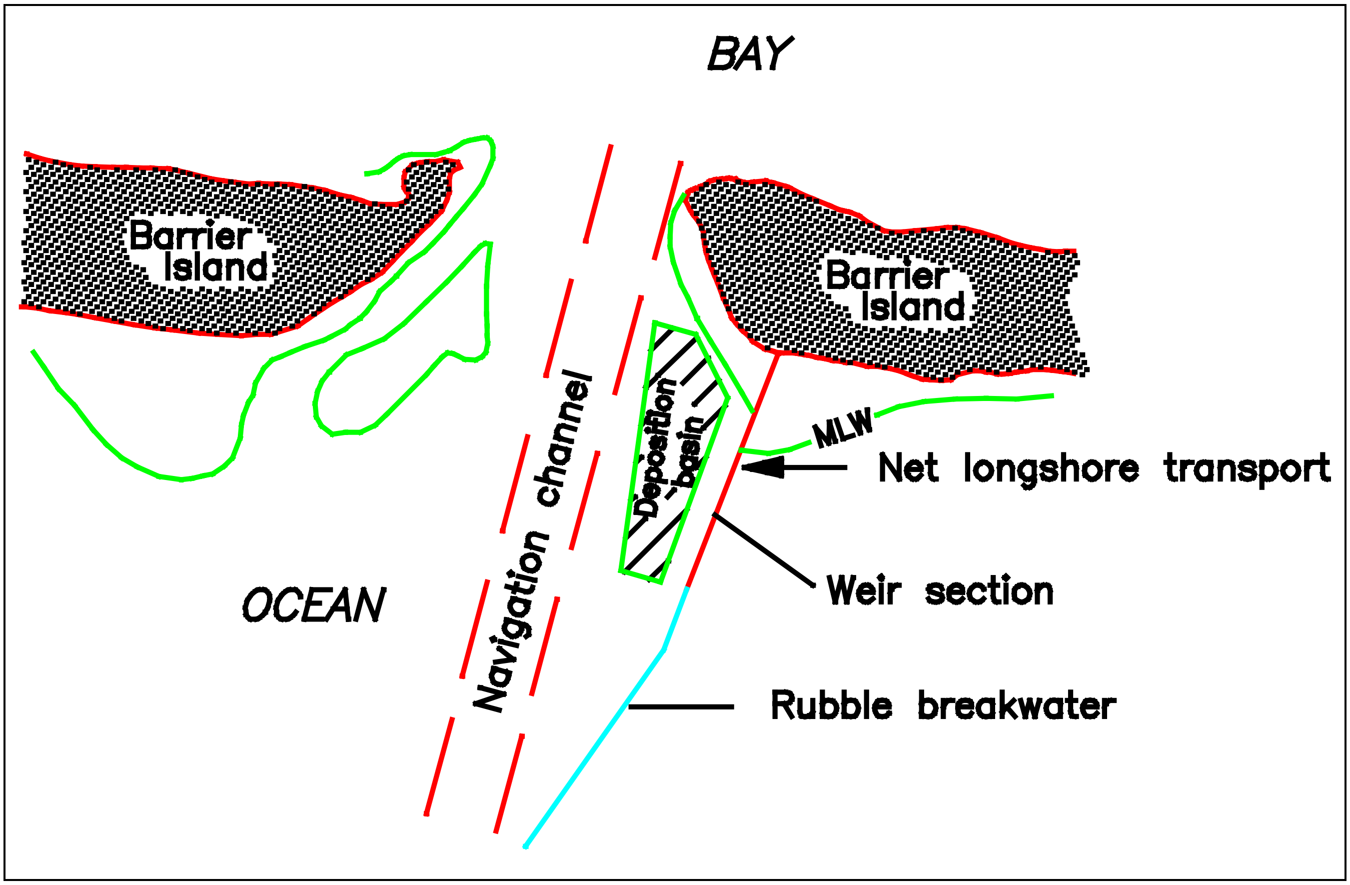

Figure 5-3. Schematic of a weir section in a jetty system

characteristics of the local littoral system. Although each project must be analyzed independently, a general rule suggests that jetties extend to the ocean contour equivalent to the dredged channel depth. Hydraulic model tests are generally advisable for jetty layout to optimize alignment and lengths. Additional information on jetty and channel layout can be obtained from EM 1110-2-2904, EM 1110-2-1613, and Committee on Tidal Hydraulics (CTH) Report 3 (CTH 1965).

c. Types of material.

(1) The principal materials for jetty construction are stone, concrete, steel, and timber. Asphalt has occasionally been used as a binder. Various jetty structure types are presented in EM 1110-2-2904.

(2) Rubble-mound. The rubble-mound structure is a mound of stone of different sizes and shapes, either dumped at random or placed in courses. Side slopes and armor unit sizes are designed so that the structure will resist expected wave conditions. Methods of stability analysis for rubble-mound structures are presented in EM 1110-2-2904. Rubble-mound jetties are adaptable to any water depth and to most foundation conditions. Chief advantages are: structure settling readjusts component stones that increase stability, damage is repairable, and the rubble absorbs rather than reflects much of the wave energy.

(3) Sheet-pile. Steel, timber, or concrete sheet piles are often used for jetty construction in areas where wave conditions are not severe. Various formations of steel sheet-pile jetties include a single row of piling with or without pile buttresses; a single row of piling arranged to function as a buttressed wall; double walls of sheet piles held together with tie rods, with the space between the wall filled with stone or sand; and cellular-steel sheet-pile structures, which are modifications of the double-wall type. Cellular-steel sheet-pile jetties require little maintenance and are suitable for construction in depths to 12 m (40 ft) on all types of foundations. Corrosion is the principal disadvantage of steel in water.

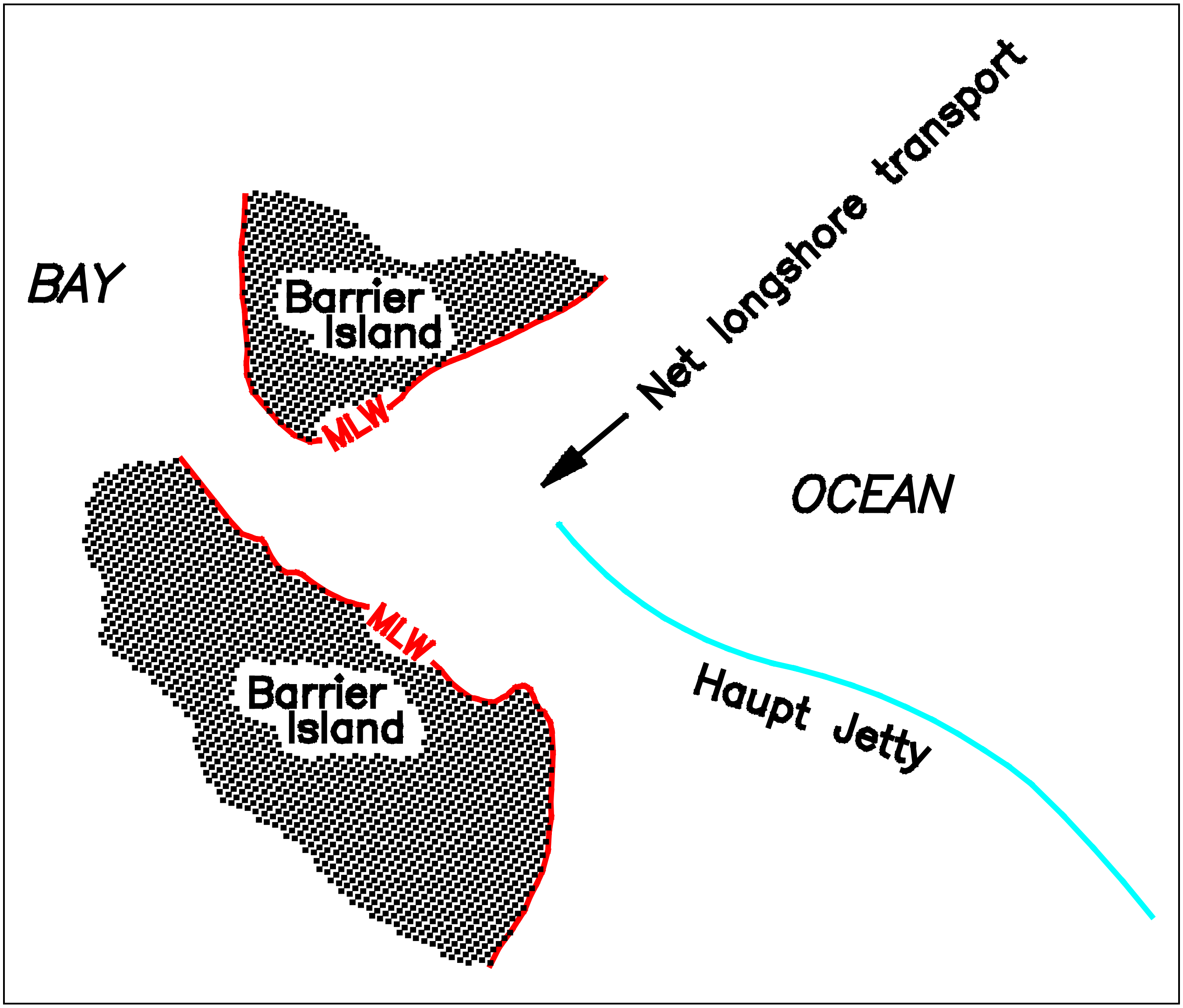

Figure 5-4. Schematic of a Haupt jetty system

d. Stability analyses.

(1) General. Jetty construction at inlet entrances will restrict the movement of tidal flows and nearshore sediment. Prediction of effects on structure stability and the nearby littoral environment, due to the changes in hydrodynamic processes, becomes an important step in the process of coastal structure design.

(2) Structure stability. Stability analysis of structure elements for rubble-mound jetties follows the Hudson formula outlined in EM 1110-2-2904. Design wave conditions of wave height, period, and direction should be selected based on long-term data, measured or hindcast. For sheet-pile jetties, appropriate wave forces under design wave conditions should be calculated according to procedures outlined in EM 1110-2-2904 and EM 1110-2-1614 to assure structure stability. Protection of structure toes, particularly at the channel side, is extremely important. Current scour at the toe area is a common cause of failure to jetty structures. The procedures of design and analysis of structure toe protection also are outlined in EM 1110-2-2904.

(3) Inlet stability. Overall stability of the inlet entrance channel should be analyzed using the methodology in Chapter 3 and for final design the modeling techniques presented in Chapters 6 and 7 may be

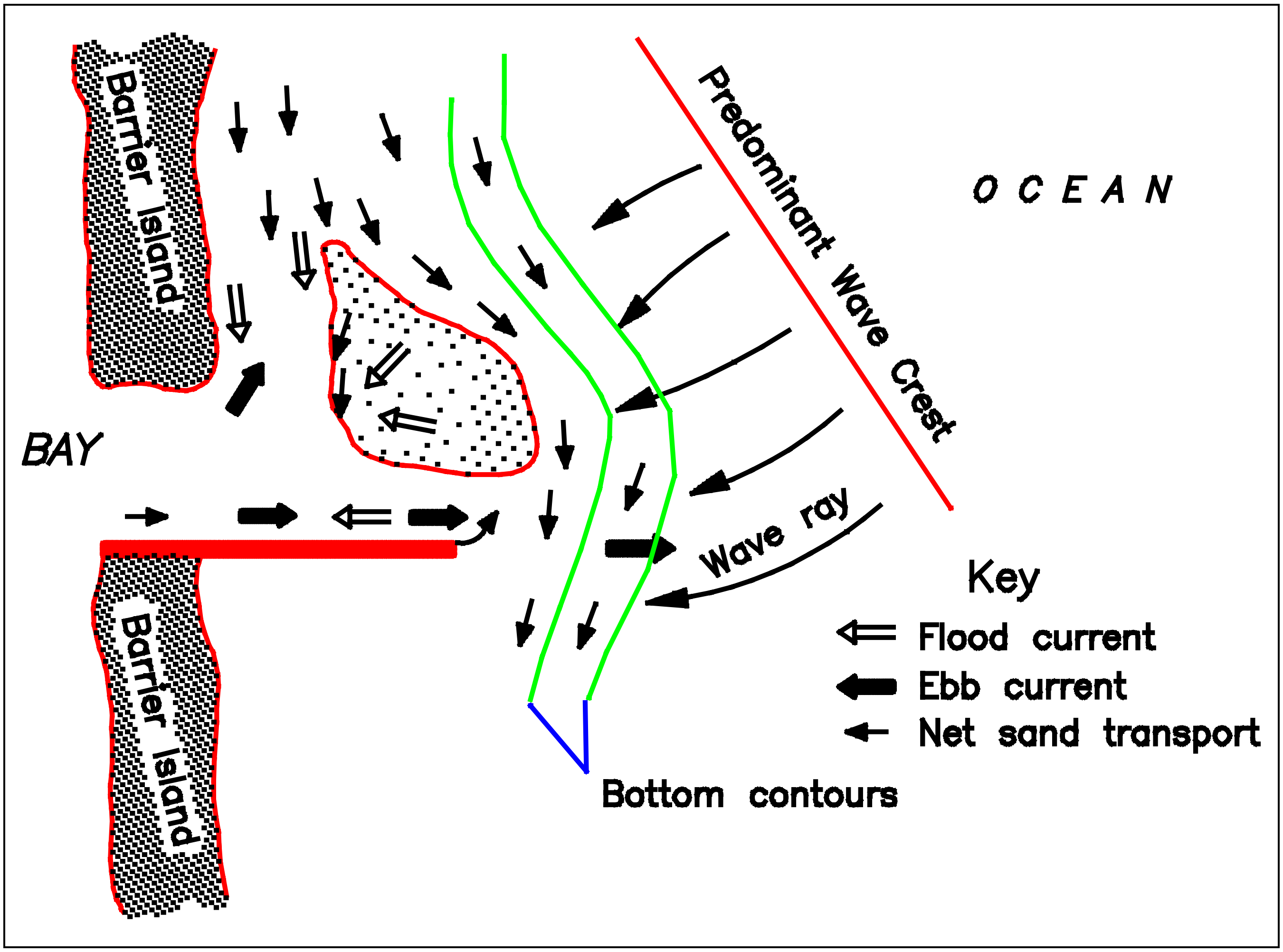

Figure 5-5. Schematic of a single downdrift jetty

required. The analysis should incorporate alterations designed for the inlet including jetty structures. If the entrance is unstable, bottom erosion may be expected and extreme caution is needed in the design of toe protection. If the entrance becomes unstable due to dredging, then excessive siltation may occur at the channel and, possibly, the bay area as well. Redesigning jetty alignment, increasing or decreasing the distance between twin jetties, or changing channel dimensions may be necessary for optimum tidal current patterns and magnitudes to improve inlet stability and to reduce maintenance requirements after the channel is improved.

(4) Shoreline changes. Effects on updrift and downdrift shorelines due to the presence of coastal structures should be thoroughly studied. Erosion control measures of sand bypassing should be considered if adverse impacts are expected. Chapters 6 and 7 discuss modeling techniques. EM 1110-2-1616 presents sand bypassing theory and technologies.

e. Other considerations.

(1) Jetty length. Jetty length should be determined by economic analysis of alternative plans. The maximum length will be the longer of either that producing a year-round design channel depth considering jetty and maintenance dredging cost on an annual basis, or that which extends the jetties beyond the breaker line of waves likely to be encountered by the design vessel. The two benefits will be evaluated on an average annual basis and will be compared with annual cost of the jetties required. Shorter jetties then should be considered at the expense of year-round navigation if the maximum length jetties result in an uneconomical project. Provisions for jetty extension at a later time should be included in a short jetty plan.

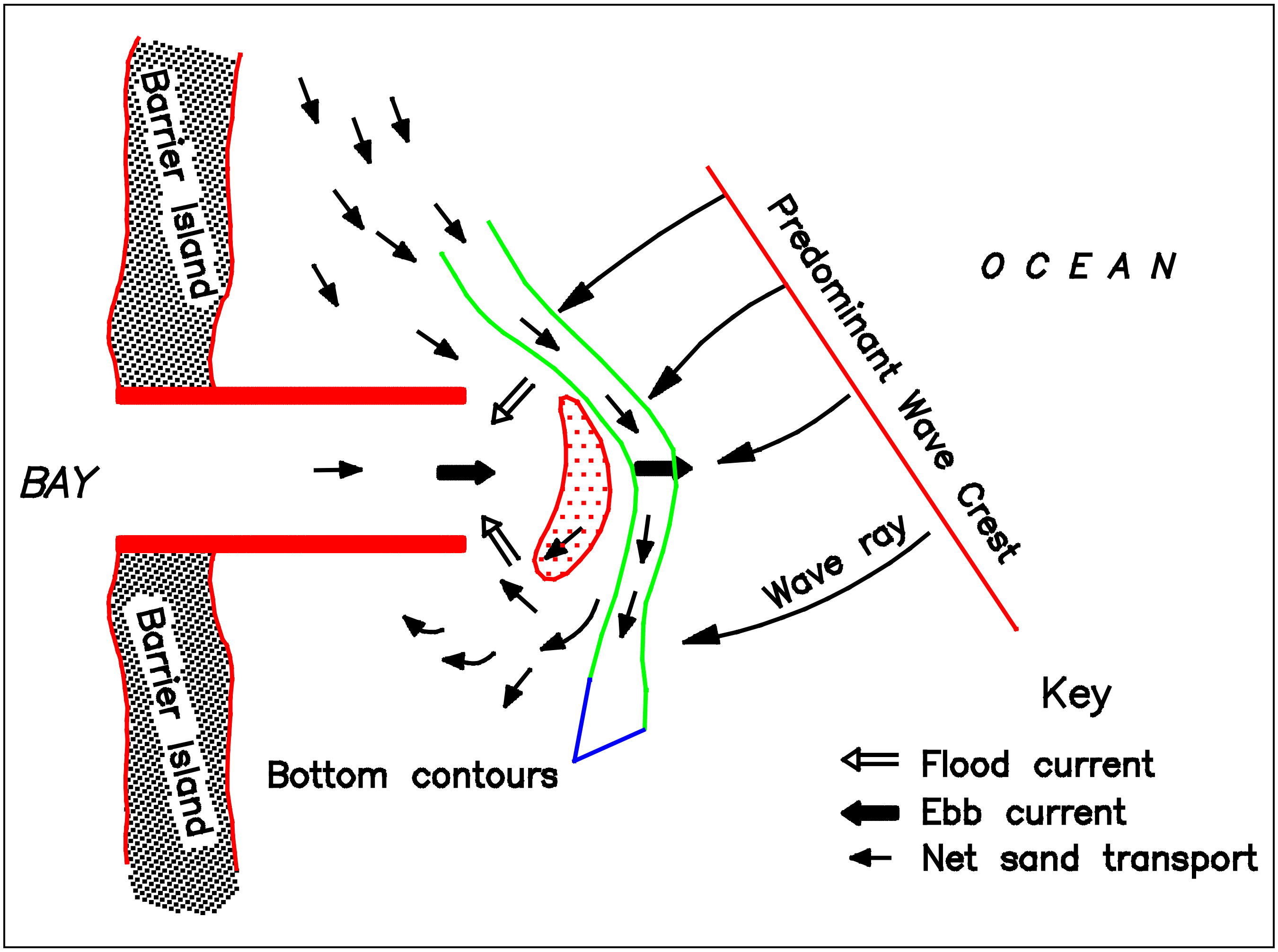

Figure 5-6. Schematic of a twin jetty system

(2) Permeability and overtopping. Rubble-mound jetties usually are not sand tight. The jetty function in littoral drift control can be reduced significantly due to structure permeability. Sand-tightening measures by constructing an impervious core layer, using geotextile fabric for leakage control, or asphalt to seal the pores, can be considered in the cost analysis. Leakage of littoral drift into the navigation channel also can occur through water overtopping low crested jetty structures. The design crest elevation should consider structure settlement and wave overtopping. A step-down type jetty may be designed when the offshore section is located in deeper water where littoral transport is at a minimum.

5-4. Sand Bypassing

The construction of jetties or breakwaters for navigation improvement at tidal inlets creates littoral barriers that interrupt the natural sand bypassing at the unimproved inlets. The resulting starvation of downdrift beaches may cause serious erosion unless measures are taken to bypass sand from the updrift side of inlets to downdrift beaches. Mechanical techniques may be used for sand bypassing, not only to minimize the shoreline erosion problem but also to reduce the potential of shoaling at the navigation channel. EM 1110-2-1616 discusses basic sand bypassing concepts and principles, presents advantages and disadvantages of various techniques and equipment, and provides guidance on developing technically feasible bypassing systems.

5-5. Economic Analysis

a. General. Optimum design of a coastal inlet improvement project requires studies of estimated costs and benefits of various plans and alternatives considering safety, efficiency, and environmental impacts. These studies are used to determine the most economical and functional channel alignment and design considering construction, maintenance, and replacement costs for various design levels. Economical optimization analysis should consider various elements involved in the development and maintenance of the project.

b. Channels. The economic optimization of a channel requires selection of several alignments and channel dimensions. Costs, which include initial construction, replacement, and annual maintenance, are developed for the various alignments and a series of dimensions are developed for each alignment. Benefits are developed from transportation savings with consideration of vessel trip time and tonnage, delays for tides, weather conditions, and effects of reduced depths in channels that have rapid shoaling tendencies. The optimum economic channel is selected from a comparison of annual benefits and annual costs for initial construction, maintenance, and replacement.

c. Structures.

(1) Optimization of structures such as jetties is accomplished by estimating the annual initial construction, replacement, and maintenance costs, and annual benefits for various design levels. Elements to be considered are: project economic life; construction cost for various design levels; maintenance and repair cost for various design levels; replacement cost for various design levels; benefits for various design levels; and probability of exceedance for various design levels.

(2) The project economic evaluation period for most coastal projects is 50 years. The design level or level of protection can be related to wave heights and water levels. The severity of these events may be represented by the probability of exceedance. Figure 5-7 shows the general relationship of exceedance probability versus the design level. Initial construction costs are estimated and annualized. Annual maintenance and repair costs can be estimated by multiplying the construction cost by the probability of exceedance of the design level. This cost estimate should compare with the actual cost of existing structures in similar coastal environments. Replacement cost should be annualized with the present worth of replacement cost considering appropriate interest rates and project life. Total cost as a function of design level is illustrated by Figure 5-8. A comparison of total costs and benefits as a function of design level is shown in Figure 5-9. Normally, the design level associated with the maximum net benefit will be selected for project design. If the net benefit point is not well-defined, it may be prudent to select a higher design level.

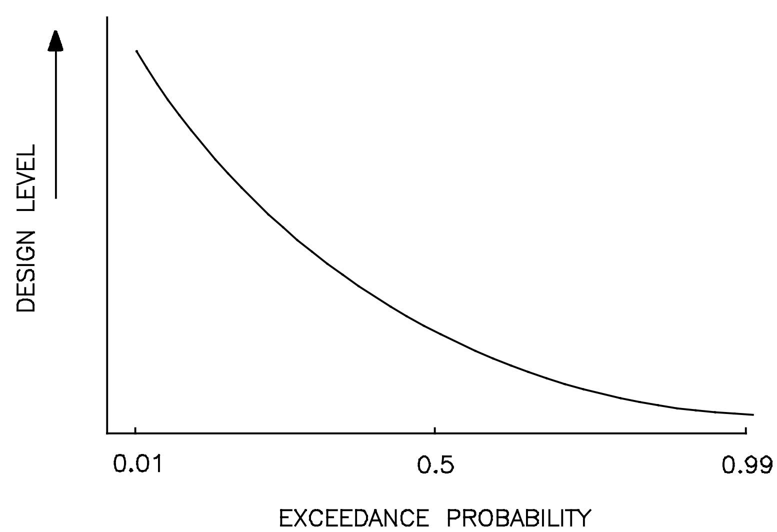

Figure 5-7. Exceedance probability versus design level

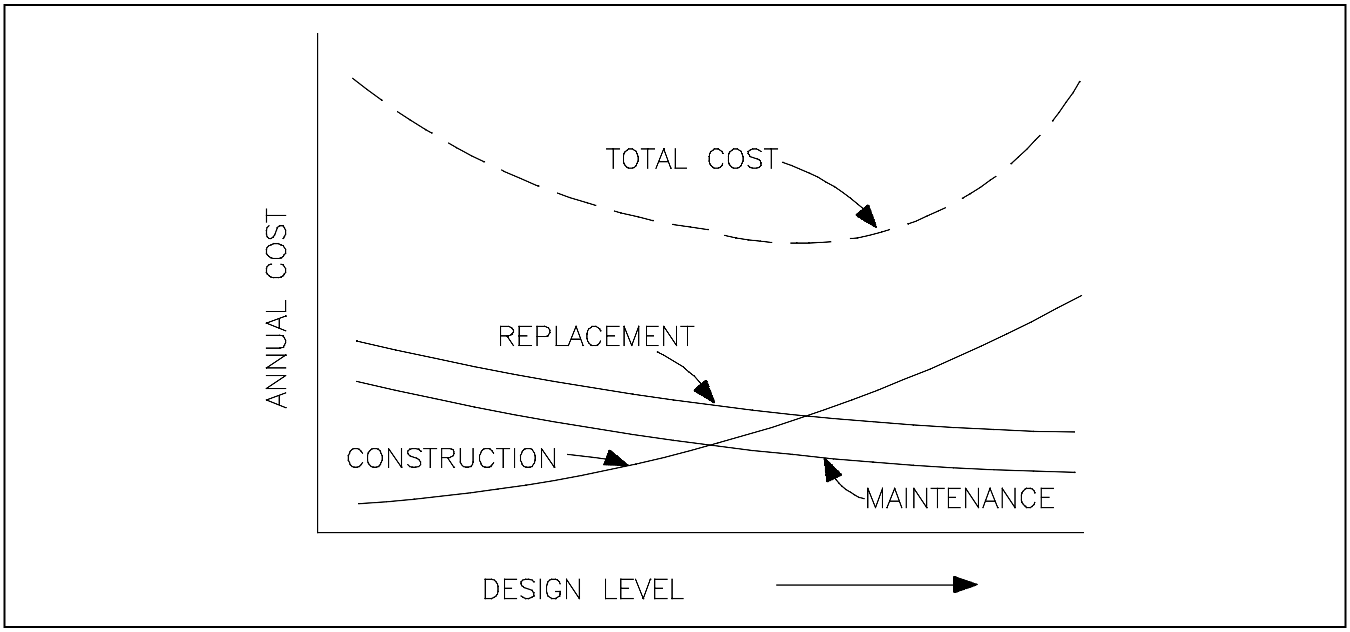

Figure 5-8. Project cost curves

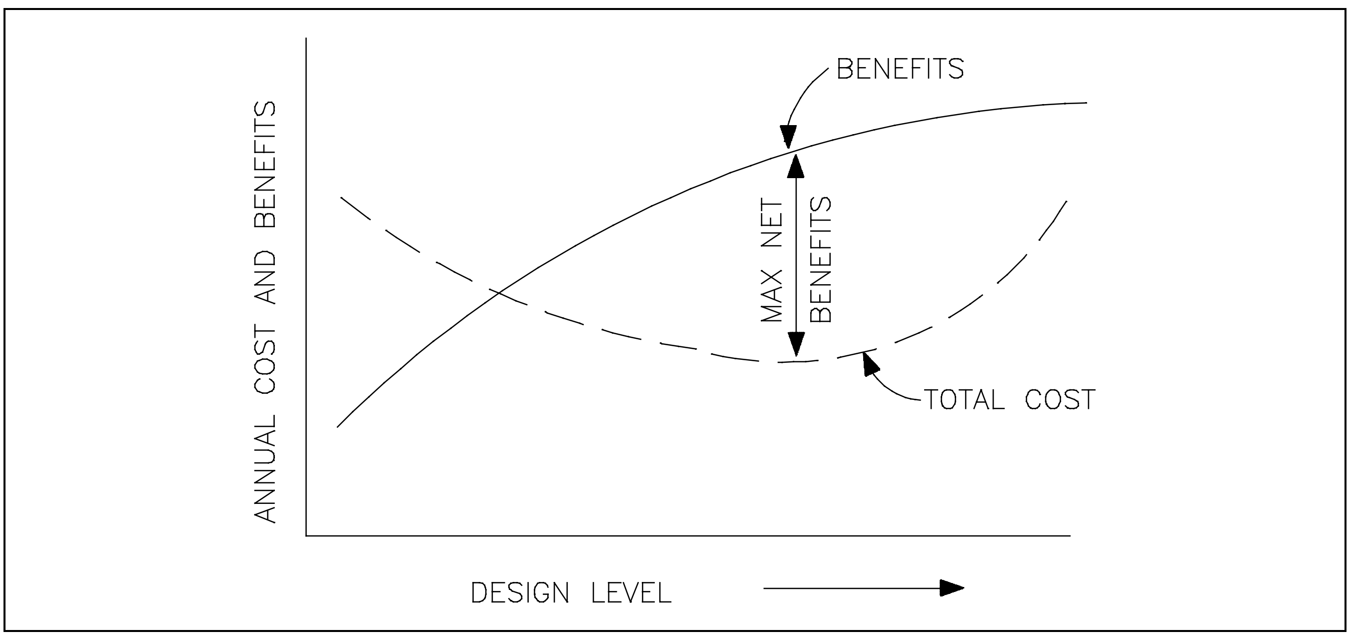

Figure 5-9. Benefits and cost versus design level

Chapter 6
Physical Modeling of Tidal Inlets

6-1. Introduction

Fluid-flow problems associated with tidal inlet studies generally involve a large number of variables, and therefore are not readily solved by simple mathematical approaches. As a result, physical hydraulic models are often used to determine the significant kinematic and dynamic features of the prototype inlet system. In any hydraulic model study, the physical phenomena observed in the model should represent those phenomena occurring in the prototype, so that prototype behavior can be predicted by operating the model. A model is then, by definition, a device which is so related to a physical system that observations of the model may be used to accurately predict the behavior of the physical system. A true physical model requires the accurate simulation of all phenomena active at a particular inlet. Such a simulation is not only beyond the capability of present physical modeling, but beyond the capabilities of any known simulation technique. However, the physical model does provide a means of investigating the effects of a significant number of dominant phenomena and in many cases allows an effective understanding of physical processes occurring at a tidal inlet.

6-2. Terminology

The following is a list of definitions of terms that will be used throughout this chapter:

a. Fixed-bed model. Bathymetry molded in concrete; a thin layer of concrete is molded to templates over a sand base.

b. Movable-bed model. Model molded to a given bathymetry in sand (or some other suitable granular modeling material) which is fully movable; scaling relationships for bed movement are not fully defined and sometimes lengthy testing is necessary to adequately verify the model for predictive use.

c. Undistorted-scale model. Model is scaled by one length scale ratio for the three dimensions; normally the maximum vertical scale used to obtain accurate results should not exceed a scale ratio of 1:100.

d. Distorted-scale model. Used when area to be modeled is extensive, horizontal scales are changed to make the size of the model more manageable while maintaining a vertical scale in the range of 1:50 to 1:100; typical horizontal scales used range from 1:200 to 1:500.

e. Section model. Rather than distorting the model, a smaller section of the inlet is selected for modeling so that larger scales of 1:40 to 1:75 may be used; boundary conditions of the section model must be accurately reproduced.

6-3. Model Preparation

a. Type of study. Physical model studies of inlets typically are designed to investigate various methods of maintaining an effective navigation channel through the inlet. Additional inlet-related problems that can be addressed by physical model studies include:

(1) Stabilization of navigation channel dimensions and location.

(2) Optimizing structural dimensions, location, and configuration of jetties and other engineering structures.

(3) Sand-bypassing techniques.

(4) Shoaling and scouring trends on adjacent beach, inlet, and bay.

(5) Tidal prism changes.

(6) Navigation conditions due to waves and currents.

(7) Salinity effects.

(8) Effects of storm surge.

b. Model design.

(1) After the purpose of the model study has been defined, the actual design of the model can proceed. The significant steps are: acquisition of prototype data to assure model accuracy, establishment of model limits, and definition and acquisition of model appurtenances.

(2) The importance of accurate prototype data cannot be overemphasized in model operation. The accuracy of the model is dependent on the use of proper field data. Although the similitude of fixed-bed, undistorted-scale models indicates that good approximation of bed form losses can be derived in the model, assurance of accurate model results can be achieved only through a comparison of model and prototype results. To assure that the model is an accurate geometric reproduction of the prototype,

hydrographic and topographic surveys must include the inlet and pertinent ocean and bay approaches that influence the inlet. The complete model requires a detailed definition of the entire bay, whereas the section model only requires definition of that part of the bay of importance to the study. A critical need is topographic information for land flooding by the highest expected water levels, particularly when investigating storm surge conditions.

(3) Because bed form plays an important role in boundary losses through an inlet, attention must be given to this feature. Although more research is needed on this subject, existing knowledge can guide the successful design of a physical model. With the physical characteristics of the prototype known and similitude as a guide, the required bed form of the model can be estimated.

(4) The final proof of model effectiveness in reproducing system hydrodynamics is a comparison of current velocities and water surface elevations in both the model and prototype. Requirements for a particular inlet model can vary extensively; however, a limited number of critically placed tide gauges and wave gauges, along with carefully located velocity stations, can provide enough information for confidence in the model operation. The appurtenances required for an effective model study include:

(a) A tide-reproducing system for the ocean.

(b) A tide-reproducing system for the bay if the bay is not completely modeled.

(c) Spectral wave generator or generators.

(d) Tidal height measuring and recording system.

(e) Velocity measuring and recording system.

(f) Wave measuring and recording system.

(g) Photographic capabilities.

Each of these systems requires proper planning in designing the model as construction of the model depends on advanced knowledge of the specific requirements of each system.

c. Model construction.

(1) Initial steps involved in model construction include:

(a) Basic site preparation.

(b) Installation of buried features (i.e., pipelines, required bases for instrumentation support systems).

(c) Installation of control templates.

(d) Installation of base material.

(e) Placement of material (normally concrete) forming the model.

(f) Finishing the model for the desired surface texture.

(g) Fabrication and installation of tide-generating capabilities.

(h) Installation of wave generators, velocity recording systems, tide recording systems, wave recording systems, and photographic capabilities.

(2) Among the details that must be planned in model construction are the various inlet changes to be evaluated during the model study. If the effects of dredging a feature are evaluated, the construction of the model should be based on this information. Templates prepared from detailed hydrographic and topographic maps to assure the model is a true representation of the prototype should be modified to include the deepest possible navigation channel, deposition basin, turning basin, etc. This would allow the study of these features in later stages of the model testing program. A second set of templates can then be installed in the molded model to allow features of lesser depth to be incorporated into the model. Tests can then be conducted with the conditions of a lesser depth in the model. When tests are completed, conversion of the model to evaluate a proposed change to the inlet can be easily accomplished.

6-4. Model Theory

a. The general theory of model design is based on the fundamental principle that a functional relationship (similitude) exists among all variables associated with the system. Further, the number of variables can be significantly reduced by forming a complete set of dimensionless variables for which a new function expressing the relationship between the dimensionless terms exists. If the model is designed so that each of the dimensionless terms of the complete set is the same in the model as in the prototype, the nature of the unknown function is identical for the model and the prototype. If all these conditions are satisfied, the model is considered a "true" model which provides accurate information concerning the behavior of the prototype.

b. Although space limitations for the construction of the model may sometimes dictate that the model be distorted, a physical model can usually be operated with the same linear scale in all three dimensions (i.e., an undistorted-scale model). This dictates that geometric similarity exists, as the ratios of all homologous dimensions on the model and prototype are equal.

c. In addition to geometric similarity, a true undistorted-scale model requires that kinematic similarity and dynamic similarity also exist. Kinematic similarity exists when the ratios of all homologous velocities and accelerations are equal in the model and prototype. Dynamic similarity requires that the ratios of all homologous forces be the same in the model and prototype. Since force is related to the product of mass and acceleration, dynamic similarity implies the existence of kinematic similarity which, in turn, implies the existence of geometric similarity.

d. For an inlet model, the forces influencing the physical phenomena include pressure, gravity, viscosity, surface tension, and Coriolis (to a lesser extent). On a regional scale, the Coriolis force has a significant effect on wind-driven and tidal circulations, and water surface elevations in large tidal estuaries, bays, and lakes. However, for a localized system such as a tidal inlet, Coriolis force is considered insignificant. Elasticity is negligible in either case.

e. Each force is related to the geometry and motion of the flow. In Newton's second law of motion, the inertial force F_i equivalent to the product of mass and acceleration, is equal to the sum of all external forces applied to a body. This inertial force can be considered as the vector sum of all the others, or

$$F_i = F_{pr} + F_g + F_\mu + F_{st} + F_e \tag{6-1}$$

where F_{pr}, F_g, F_μ, F_{st}, and F_e are the forces due to pressure, gravity, viscosity, surface tension, and elasticity, respectively. These forces usually suffice to describe hydraulic phenomena.

f. For dynamic similarity, the ratio of the inertial force between model and prototype must be the same as the ratio of the individual force components between the model and prototype. The ratios of the inertial force to the other component forces must also be the same between model and prototype. Therefore:

$$\begin{aligned} F_r &= \frac{(F_i)_p}{(F_i)_m} = \frac{(F_{pr})_p}{(F_{pr})_m} = \frac{(F_g)_p}{(F_g)_m} \\ &= \frac{(F_\mu)_p}{(F_\mu)_m} = \frac{(F_{st})_p}{(F_{st})_m} = \frac{(F_e)_p}{(F_e)_m} \end{aligned} \tag{6-2}$$

where r is ratio, p is prototype, and m is model. Equation 6-2 can be considered as indicating five independent conditions that must be satisfied for complete dynamic similitude. This can be reduced to four by using Equation 6-1 and letting one of the forces be a function of the others. Taking the pressure force as the dependent variable:

$$\begin{aligned} \frac{(F_{pr})_p}{(F_{pr})_m} = \frac{(F_i)_p}{(F_i)_m} - \Bigg[& \frac{(F_g)_p}{(F_g)_m} + \frac{(F_\mu)_p}{(F_\mu)_m} \\ & + \frac{(F_{st})_p}{(F_{st})_m} + \frac{(F_e)_p}{(F_e)_m} \Bigg] \end{aligned} \tag{6-3}$$

Therefore the pressure force will be scaled correctly if the other four forces are scaled correctly. Using Equation 6-2 and equating the inertial force as $F = Ma = p(\text{volume})a$, or as prototype to model ratio,

$$(F_i)_r = M_r a_r = \rho_r L_r^3 \frac{L_r}{T_r^2} = \frac{\rho_r L_r^4}{T_r^2} \tag{6-4}$$

where M_r = ratio of prototype to model mass and a_r = ratio of prototype to model acceleration and equating the inertial force ratio to the other force ratios individually, the following equations result:

$$\frac{\rho_r V_r^2}{p_r} = 1 \qquad (\textit{ratio of Euler numbers} = 1) \tag{6-5}$$

$$\frac{V_r^2}{g_r L_r} = 1 \qquad (\textit{ratio of Froude numbers} = 1) \tag{6-6}$$

$$\frac{\rho_r V_r L_r}{\mu_r} = 1 \quad \left(\textit{ratio of Reynolds numbers} = 1\right) \tag{6-7}$$

$$\frac{\rho_r V_r^2 L_r}{\sigma_r} = 1 \quad \left(\textit{ratio of Weber numbers} = 1\right) \tag{6-8}$$

$$\frac{\rho_r V_r^2}{E_r} = 1 \quad \left(\textit{ratio of Mach numbers} = 1\right) \tag{6-9}$$

where ρ is mass density, *V* is velocity, *p* is pressure, *g* is the acceleration due to gravity, *L* is a representative length, μ is dynamic viscosity, σ is surface tension, and *E* is elasticity.

g. It can be demonstrated that no single model fluid will permit all of these equations to be satisfied at once; therefore, true dynamic and kinematic similarity apparently cannot be achieved between a model and the prototype. However, one or more of the specific forces is often found to be negligible, and the number of equations to be satisfied can be reduced accordingly. In fact, the phenomena in a particular instance often involve the effect of only one force ratio, and the others are negligible.

h. The use of water as a model fluid is usually necessary in coastal engineering models. Surface tension, the least important term if the depths of the fluid are not excessively small, will have a negligible effect on the flow of water more than 8 cm (0.25 ft) deep, or on waves with lengths exceeding about 0.3 m (1.0 ft) in the same water depth. By ensuring that the flow and waves exceed these limiting values, the effect of surface tension can be neglected.

i. When both viscous and gravity forces are important, as in open channel flow on mild slopes, the Froude and Reynolds numbers should both be satisfied simultaneously. This requirement can only be met by choosing a special model fluid. Since water is the only practical model fluid, an approximate similarity requirement may be used, based on empirical relationships which include the major effects of frictional forces (such as Manning's equation). This approach is used in studying inlet problems.

j. Since fairly high Reynolds numbers are usually associated with tidal flow through an inlet, the shear stresses are primarily determined by form drag. When Manning's formula is used in an undistorted-scale model, and assuming similarity for velocity,

$$V_r = \frac{L_r^{2/3}}{n_r} \tag{6-10}$$

where $n_r = L_r^{1/6}$ and *n* is Manning's roughness coefficient.

k. The use of Manning's formula as a similarity criterion requires that the flow be fully rough turbulent in both the model and the prototype. When a bulk Reynolds number defined as Vd/γ is greater than about 1,400 (where *d* is the depth of flow and γ is the kinematic viscosity), fully rough turbulence will normally exist.

l. A surface gravity wave is essentially a gravitational phenomenon; therefore, the controlling criterion of similitude is the Froude number, and waves may be represented correctly in undistorted-scale models.

m. Based on the Froude criterion of scaling, and considering an undistorted-scale, fixed-bed model, the geometric, kinematic, and dynamic scaling ratios may be expressed in terms of the model-prototype length ratio used for scaling L_r when the same fluid is used in the model and the prototype (Table 6-1).

n. Several physical interpretations may be given of the Froude number. Fundamentally, it is the ratio of inertial to gravitational forces acting on a particle of fluid. It can be shown that this ratio reduces to $V/(gL)^{1/2}$, where *V* is a characteristic velocity, and *L* is a representative length. Here the velocity is taken to be a horizontal length divided by the time parameter. However, any representative velocity and any representative length can be used in the Froude number as long as dynamic similarity is maintained and corresponding regions are considered in the model and prototype. For an undistorted-scale model, the scaling ratios in Table 6-1 are appropriate; here the time and velocity ratios are equal to the square root of the linear scale ratios, where the horizontal and vertical linear scale ratios are identical. The Froude number, defined as $V/(gd)^{1/2}$, is related to the vertical scale (depth) so that the velocity ratios are equal to the square root of the depth ratios; consequently, for a distorted-scale model, the time ratios are equal to the horizontal length ratios divided by the vertical length ratio. Symbolically, for a distorted-scale model

Table 6-1
Froude Criteria Scaling Relationships (for same fluid in model and prototype)

	Undistorted Scale	Distorted Scale
Geometric Similarity		
Length	L_r	
(horizontal)		$(L_h)_r$
(vertical)		$(L_v)_r$
Area	L_r^2	
(horizontal)		$(L_h)_r^2$
(vertical)		$(L_h)_r(L_v)_r$
Volume	L_r^3	$(L_h)_r^2(L_v)_r$
Kinematic Similarity		
Time	$L_r^{1/2}$	$(L_h)_r/(L_v)_r^{1/2}$
Velocity	$L_r^{1/2}$	$(L_v)_r^{1/2}$
Acceleration	1	1
Discharge	$L_r^{5/2}$	$(L_h)_r(L_v)_r^{3/2}$
Kinematic Viscosity	$L_r^{3/2}$	$(L_v)_r^{3/2}$
Dynamic Similarity		
Mass	L_r^3	$(L_h)_r^2(L_v)_r$
Force	L_r^3	
(horizontal)		$(L_h)_r^3$
(vertical)		$(L_h)_r^2(L_v)_r$
Dynamic Viscosity	$L_r^{3/2}$	$(L_v)_r^{3/2}$
Surface Tension	L_r^2	$(L_h)_r^2$
Pressure Intensity	L_r	$(L_v)_r$
Impulse and Momentum	$L_r^{7/2}$	$(L_h)_r^2(L_v)_r^{3/2}$
Energy and Work	L_r^4	$(L_h)_r^2(L_v)_r^2$
Power	$L_r^{7/2}$	$(L_h)_r/(L_v)_r^{5/2}$

$$V_r = (L_v)_r^{1/2} \quad \text{and} \quad T_r = \frac{(L_h)_r}{(L_v)_r^{1/2}} \tag{6-11}$$

where L_v = vertical length ratio and L_h = horizontal length ratio, which shows the significance of distortion. These and other pertinent ratios required for geometric, kinematic, and dynamic similarity are easily developed.

o. Scaling of refraction and diffraction of dispersive waves cannot be correctly achieved simultaneously in a distorted scale model. Refraction phenomena are governed by a change in depth (vertical scale) while diffraction relates to wave energy spreading in the plane of the water surface (horizontal scale). Refraction requires the wavelength to be scaled by the vertical scale while diffraction requires scaling the wavelength by the horizontal scale. If the model is distorted, these phenomena cannot be simultaneously scaled correctly. However, if the influence of one of the phenomena is considered to be insignificant relative to the other, the scale effect can be determined for a distorted scale model.

6-5. Types of Models

a. Fixed-bed, undistorted-scale models.

(1) General. Fixed-bed models often can be easily developed to provide kinematic and dynamic responses indicative of the prototype conditions. Specifically, fixed-bed models reveal information regarding velocities, discharges, flow patterns, water surface elevations, energy losses between points in the prototype, reflection and

transmission by structures, and the transformation of wave spectra. In the superposition of surface gravity waves on fixed-bed flow conditions, an undistorted-scale model provides greater insight on refraction and diffraction phenomena than does a distorted-scale model. Accordingly, the fixed-bed, undistorted-scale model can be effectively used for the analysis of kinematic and dynamic conditions associated with waves, current intensities and patterns, discharges, and forces existing along coasts and in inlets.

(2) Additional uses. A fixed-bed model (although not its primary purpose) may also be useful in studying shoaling of entrance and interior inlet channels. Saltwater intrusion and the effects thereon of proposed changes in the physical or hydraulic regimes of the system can be effectively studied by fixed-bed models. The diffusion, dispersion, and the flushing of wastes discharged into inlets and the hydraulics of the inlet as related to the location and design of channels suitable for navigation can be expediently studied. Tidal flooding by hurricane surges or other unusual tidal phenomena can also be readily analyzed.

(3) Model verification. Verification of a fixed-bed, undistorted-scale model involves conducting tests in the model using realistic boundary conditions (i.e.; ocean tides, ocean waves, bay tides, and current velocities). Model data are then compared with prototype data for duplicate locations in the model and prototype to define the accuracy with which the model reproduces the prototype. If reproduction of the prototype is not achieved, the differences are evaluated for possible sources of error. Frequently, the differences can be attributed to either incorrect location of roughness in the model or improper magnitude of model roughness. If the comparison shows isolated stations to differ, the differences are usually related to incorrect model results or erroneous prototype data. Repeating the model test will indicate whether model data were in error; if so, new model data can be obtained. Model verification can also include definition of the model operating characteristics required to achieve reproduction of shoaling patterns throughout the inlet. This consists of a trial-and-error operation until the model operating conditions required to reproduce known changes in prototype shoaling are developed.

(4) Model tests. Tests in undistorted-scale, fixed-bed models can provide useful information on not only the hydrodynamics of an inlet, but the expected variations due to changes in inlet configuration. An effective model test program should initially include a complete set of tests to define the conditions that exist in the model for hydrographic, topographic, and hydraulic conditions for which the model was verified. These data then form the base conditions to which all future tests are compared to evaluate the effects of changes to the inlet. Data obtained from the model for the base conditions should include: (a) detailed current velocities at critical locations throughout the model for a complete tidal cycle, (b) detailed surface current patterns of the entire area of interest at incremental times throughout the tidal cycle, and (c) detailed wave characteristics throughout the inlet for an array of expected prototype conditions. Complete documentation of a particular proposed change to an inlet can then be accomplished by installing the proposed change in the model, duplicating the procedure followed in obtaining a base set of data, and comparing the results of each set of data.

b. Fixed-bed, distorted-scale models.

(1) Tidal inlet physical models can be distorted. Models of large inlets with shallow flood- and ebb-tidal deltas experience problems related to significant model energy attenuation and viscous friction scale effects on waves. Use of a distorted scale can minimize these effects and at the same time decrease model costs. In some cases, it is advantageous to design a distorted-scale, complete model rather than an undistorted-scale, sectional model to allow reproduction of the entire estuary. Incorporation of the tidal estuary results in the flexibility to study the effects of proposed improvements on the tidal prism, tidal circulation, tidal flushing, and salinity of the estuary. Inclusion also results in the correct nonlinear energy transfer from various tidal constituents to higher order harmonics. Deletion of a major part of the estuary leaves reproduction of this phenomenon more uncertain.

(2) Distorted-scale models for use in the study of inlets have been widely accepted. The horizontal scale ratio is often dictated by the size of the facility in which the model is placed or the construction cost. The vertical scale ratio need not be larger than the ratio of model measurement accuracy to prototype measurement accuracy. Accuracy of laboratory measurements of the water surface is generally within 0.03 cm (0.001 ft). Thus, a vertical scale ratio, model-to-prototype, of 1:100 will fully utilize the capabilities of the model in simulating the prototype. Models of larger vertical scale are often used to simplify operational techniques and to assure model depths large enough so that surface tension does not affect flow.

(3) A second factor to be considered in the selection of scales is "distortion." Distortion is the ratio of the horizontal scale to the vertical scale, and its value relates the order all slopes of the prototype are steepened in the

model. In the study of tidal inlets, particularly with movable-bed models, efforts are made to design models with distortion values of five or less. Otherwise, the slopes required in the movable-bed model for accurate reproduction of the prototype may be steeper than the angle of repose of the model material, thus creating a difficult scale effect to overcome. This point is introduced in this section because inlets often have been modeled with both a fixed bed and a movable bed and with a distorted scale. Vertical scale ratios, model-to-prototype, are generally on the order of 1:40 to 1:100; horizontal scale ratios are generally on the order of 1:100 to 1:500.

(4) Distorted-scale inlet models have been constructed for multiple purposes; for example, an investigation of an inlet may be necessary where a jetty is to be installed. A prediction will be required of the effects of the jetty on tidal currents and water levels near the inlet and also the degree to which the jetty interrupts the littoral drift and affects deposition patterns near the inlet. In this case, a multipurpose model is needed. This model would first be built with a distorted-scale, fixed-bed design and then adjusted and tested to determine the effects of the jetty on tidal heights and currents. A segment of the fixed part of the model surface would then be carefully removed and replaced with movable material to evaluate effects of the jetty on the littoral drift.

(5) Model verification and testing in a distorted-scale, fixed-bed model follow essentially the same procedures discussed for an undistorted-scale, fixed-bed model. However, because of distortion effects, the transference equations from the model to a prototype situation are, in general, completely different. A major limitation of distorted-scale models is that in cases where both wave refraction and diffraction are important, true similitude cannot be achieved simultaneously.

c. Movable-bed models. Movable-bed models are potentially the most effective type of model to investigate shoaling and scouring trends within the inlet. Limitations include the requirement that extensive prototype data be obtained for purposes of verification. Accepted practice at many hydraulic laboratories is to construct the model to a manageable size, based on space limitations and instrumentation ability, and to use a readily available material for construction (usually sand) which constitutes a model scale distortion. The empirical process of verifying the model to reproduce prototype phenomena such as scour and deposition results in distortion of a second parameter. This is usually accomplished by altering the wave climate, increasing or decreasing tidal flow, or by changing the time scale from that resulting from the hydrodynamic scaling relations to an empirically selected one which reproduces the sedimentology (referred to as the sedimentological time scale). These are empirical solutions based on the clever application of scale modeling and the experience of the researcher; however, the mechanism of most sedimentation phenomena is still not well understood. Several investigators have attempted to derive formal scaling laws resulting in a variety of modeling formulas. The reader is referred to “Coastal Hydraulic Models,” (Hudson et al. 1979, pp. 473-480) and Hughes (1993) for more detailed information on movable-bed models.

6-6. Example Model Studies

Inlet model studies that have been performed by the U.S. Army Corps of Engineers, Waterways Experiment Station fall into one of four categories (fixed-bed, undistorted-scale; fixed-bed, distorted-scale; movable-bed, distorted-scale; or a combination of fixed- and movable-bed, distorted-scale). To demonstrate the applicability of each, four inlet studies have been selected as examples. Frequently, the inlet to be investigated is a component of a much larger bay or estuary model and, therefore, is probably distorted in scale. In other cases, the inlet may not presently exist as a prototype or may not be allied with an existing model; the opportunity then occurs for construction of an undistorted-scale model. The scales of inlet movable-bed models are typically distorted, primarily for economy and due to the fact that the main forces driving sediment movement at inlets (unidirectional currents and long waves) can be properly scaled (see Hughes (1993)).

a. Shrewsbury Inlet, New Jersey.

(1) Project description and background. The Shrewsbury Inlet project involved construction of a channel connecting Sandy Hook Bay with the ocean. A new small boat channel across the peninsula was needed to shorten the distance boats had to travel from the Shrewsbury and Navesink River region to the Atlantic Ocean. Serious questions were raised concerning the effect of this new entrance on water surface elevations, current velocities, and flow patterns, and transmission of wave energy.

(2) Purpose of model study. The model study was conducted to determine the effects of the inlet on (a) water quality in Sandy Hook Bay and the Shrewsbury and Navesink Rivers from the viewpoints of public health, recreation, and fish and wildlife; (b) flooding within the areas as a result of normal tides and hurricane surges; (c) recreational boating and commercial navigation;

(d) general shoaling characteristics and maintenance requirements in the inlet channel; (e) the optimum location and length of jetties at the ocean end of the proposed inlet; and (f) transmission of wave energy through the inlet into Sandy Hook Bay. Because of the complicated phenomena to be investigated, an existing comprehensive model of the New York Harbor area was used to study the effects of the inlet on water quality. Only parts of the study concerning the new inlet model are presented here.

(3) The model. Figure 6-1 is a location map illustrating the region reproduced by the New York Harbor model. The area reconstructed for tests of the proposed inlet (Figure 6-2) was a 1:100 scale, undistorted, fixed-bed model including the inlet and adjacent portions of the ocean and Sandy Hook Bay. This model was used to provide calibration data for various inlet plans constructed in the New York Harbor model, to study flow patterns and velocity distributions for various channel alignments and jetty locations, and to define the amount of wave energy reaching the Highlands Marina area from storm waves generated in the Atlantic Ocean and propagated through the inlet. Details of the new inlet model are shown in Figure 6-3.

(4) Plan configurations. Four plans were tested in the model. Plan 1 involved a channel with a bottom width of 61 m (200 ft), beginning at the -5.2-m (-17.2-ft) depth msl in the Atlantic Ocean and continuing at that depth to the approximate center line of Sandy Hook Peninsula; a 1 on 20 transition slope of the bottom to a depth of -3.4 m (-11.2 ft) msl; and a bottom elevation of -3.4 m (-11.2 ft) msl until the inlet channel intersected the existing Federal navigation channel from Sandy Hook Bay up Shrewsbury River. The channel sides had transition slopes of 1 on 3. The ocean end of the channel was flanked on each side by protection jetties, each about 183 m (600 ft) long. The width and depth of the Plan 2 inlet were the same as in Plan 1; however, the alignment of the bay part of the Plan 2 channel was straight from the ocean to the existing Shrewsbury River channel. Alignment of the Plan 3 inlet was identical to that of

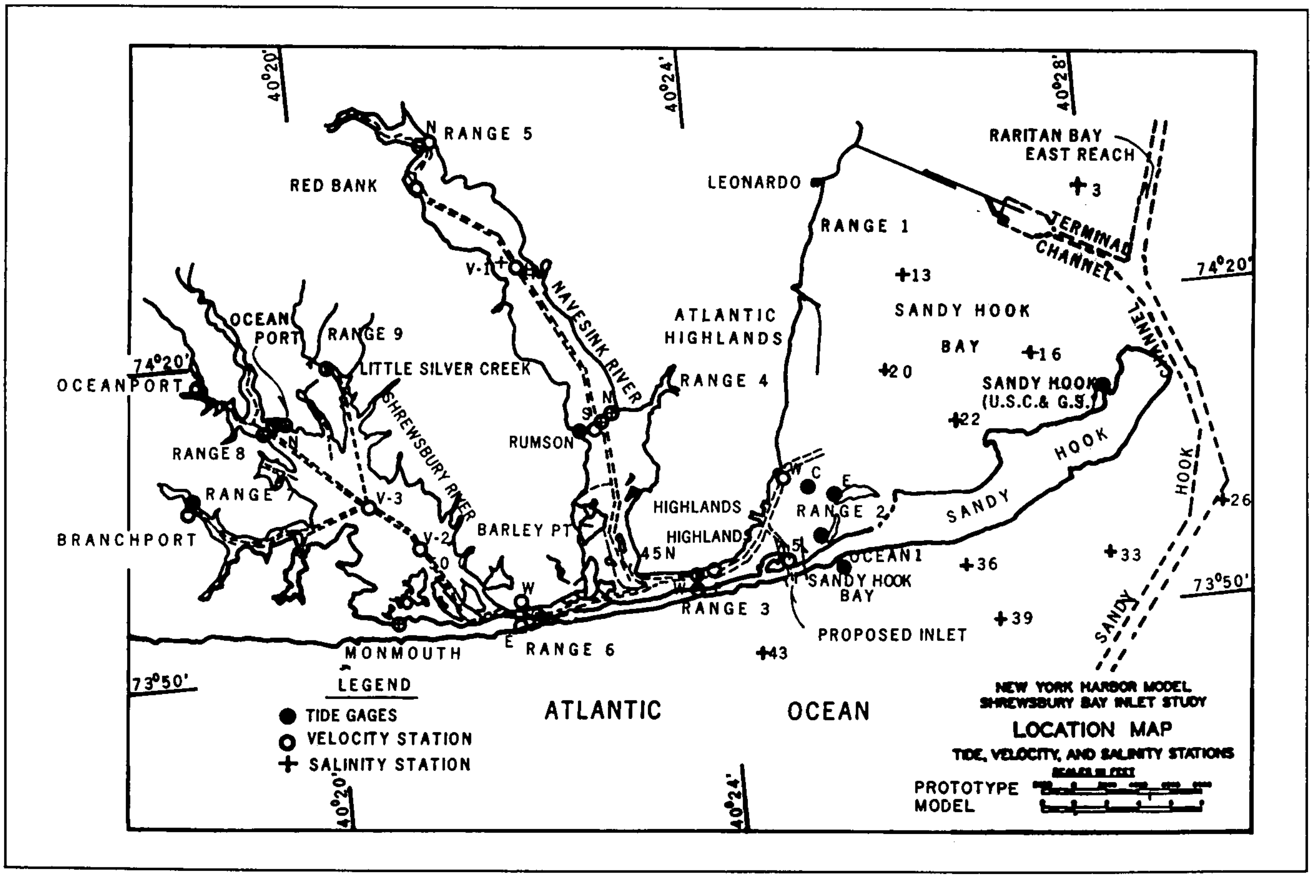

Figure 6-1. New York Harbor model limits

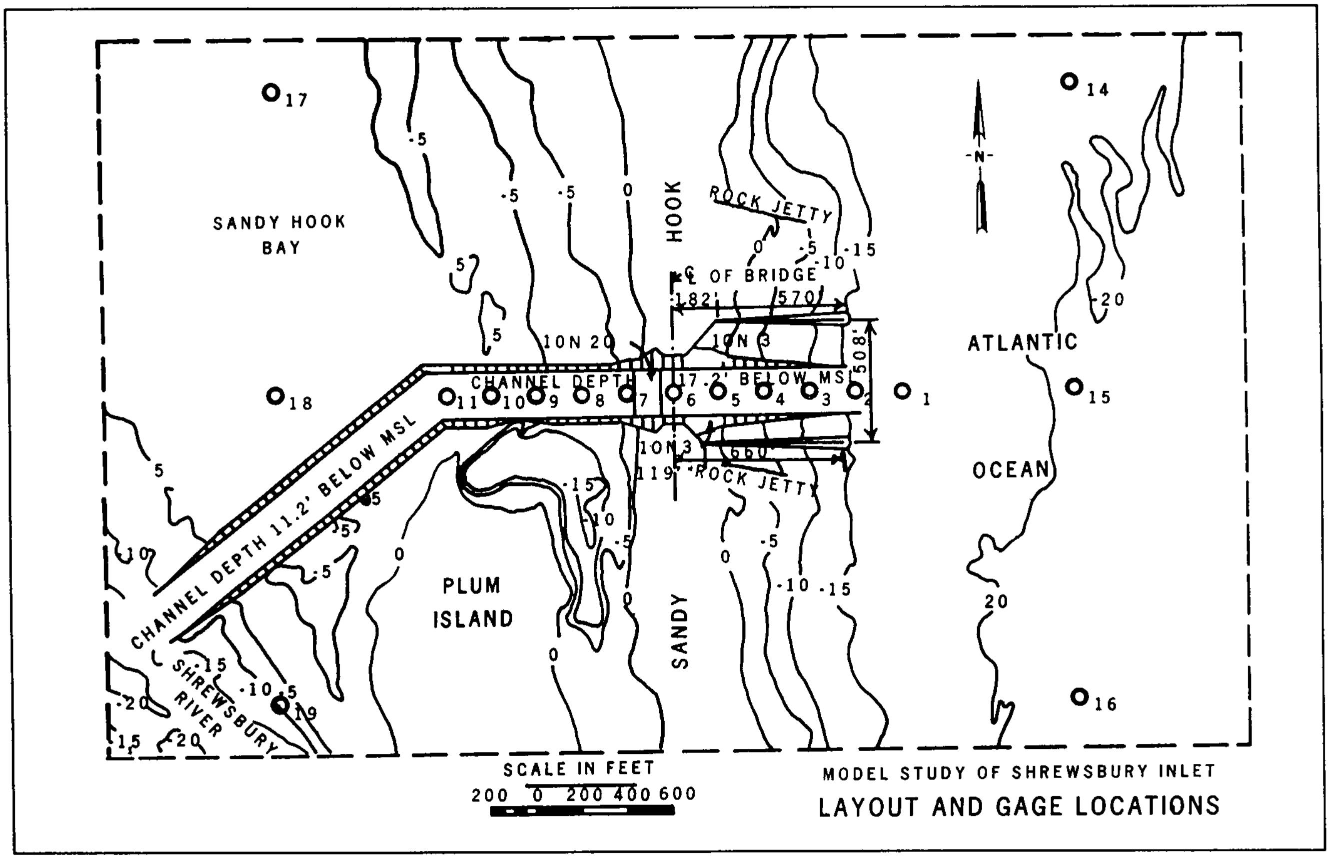

Figure 6-2. Shrewsbury Inlet and adjacent parts of Sandy Hook Bay

Plan 1; however, the depth of the Plan 3 channel was -5.2 m (-17.2 ft) msl for its entire length. Depth of the Plan 4 inlet was also -5.2 m (-17.2 ft) msl for its entire length, and the alignment was identical to that of Plan 2.

(5) Model tests.

(a) The New York Harbor model is an estuarine model operated with salt water. The new model of the proposed inlet was operated with fresh water because the required information was independent of salinity effects.

(b) The Plan 1 inlet (Figure 6-3) described in the authorizing document, was first modeled to an undistorted scale of 1:100, and steady-state tests were made for both flood and ebb flows over the full range of head differentials and water surface elevations to be expected during later tests. From these test results, discharge through the inlet as a function of head differential and water surface elevation was determined, and these data provided a basis for calibration of the model. The Plan 1 inlet was then constructed in the distorted-scale New York Harbor model. Calibration data described above were used to adjust the hydraulic resistance of the inlet so that both flood and ebb discharges were reproduced accurately over the full range of head differentials and water surface elevations to be encountered during actual tests.

(c) Tests were conducted in the New York Harbor model to determine the effects of the Plan 1 inlet on normal tides, tidal current directions, and velocities, salinities, and the dispersion of dye tracers from simulated pollution sources in Raritan Bay, Upper New York Bay, and the Shrewsbury and Navesink Rivers (Figure 6-3). Test results suggested that Plan 1 should be modified in the interest of improving flow conditions at the bay end of the inlet, and possibly improving shoaling characteristics. Consequently, Plans 2, 3, and 4 were devised. Each plan was calibrated in the undistorted-scale model of the new inlet and then reproduced in the New York Harbor model. All plans were subjected to the above-mentioned series of tests; as a result, Plan 3 was selected as the optimum design.

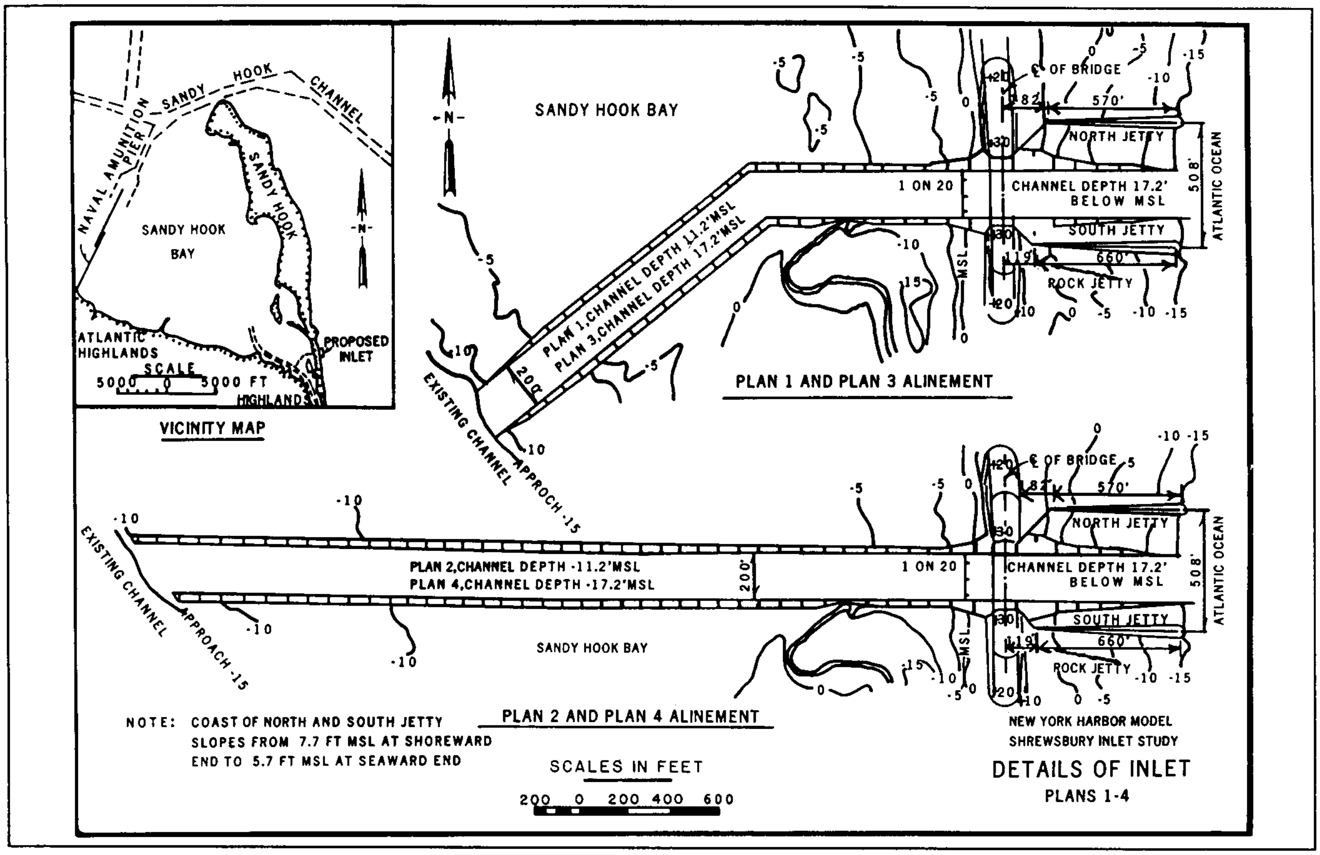

Figure 6-3. Details of plans of Shrewsbury Inlet physical model

(d) Plan 3 was tested in the New York Harbor model to determine effects on the temperature regime of Sandy Hook Bay and the Shrewsbury and Navesink Rivers. Tests were also conducted to determine the effects on flooding for conditions of the November 1950 hurricane storm surge. Finally, tests were made in the undistorted-scale model to determine the optimum location of the jetties, the wave climate between and adjacent to the jetties during storm wave action in the ocean, and wave transmission through the inlet and its effects on the Highlands Marina shoreline.

(6) Results.

(a) Three jetty plan locations were tested in the model of the new inlet (Figure 6-4). Comparison of test results showed that jetty Plan C produced the most desirable tidal flow conditions. High currents which attacked the end of the south jetty in Plan A were minimized in Plan C; eddies which developed between the navigation channel and both jetties for Plan B did not occur with Plan C. Therefore, it appears that Plan C jetty alignments and spacings would result in a satisfactory distribution of flow through the inlet.

(b) Results of tests to define wave conditions in the Highlands Marina area showed that waves generated in the Atlantic Ocean and propagated through the inlet are dissipated in Sandy Hook Bay to such an extent that essentially no wave energy reaches the Highlands Marina area. Maximum wave height recorded during these tests was 9 cm (0.3 ft), well below wave heights resulting from waves generated within Sandy Hook Bay. Thus, it appears certain that energy passing through the inlet from waves generated in the Atlantic Ocean will not significantly affect wave conditions in the Highlands Marina area.

(c) Results of tests to define wave conditions in and near the inlet showed that the maximum increase in wave height occurred with an ebb flow approximately 40 percent of the maximum spring tide ebb flow. Increases in wave heights of 60 to 70 percent, as compared to wave heights measured about 183 m (600 ft) seaward of the

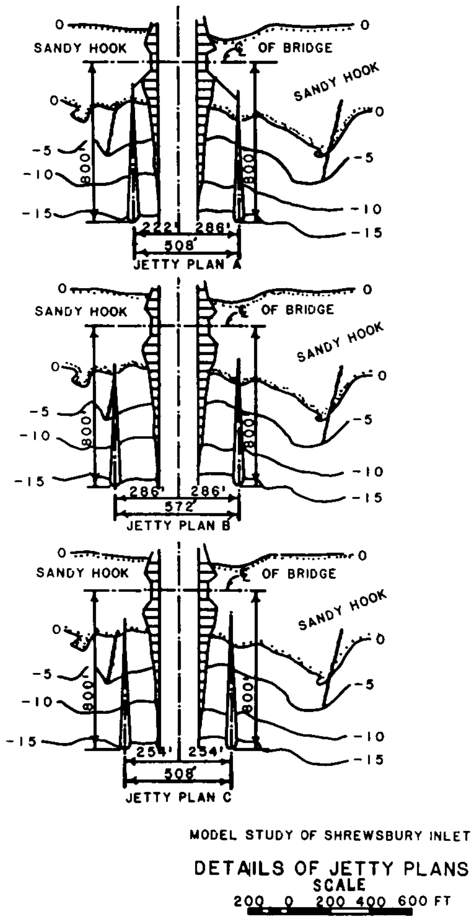

Figure 6-4. Details of jetty plans, Shrewsbury Inlet

jetties, were observed just inside the ends of the jetties. Significant increases in wave heights between the jetties were not observed for tests with flood currents or for high-water slack conditions. The alignment of the primary wave was generally perpendicular to the center line of the navigation channel during all tests. Wave conditions within the inlet apparently would not cause significant navigation problems except possibly for certain ocean wave conditions combined with a critical ebb discharge in the inlet.

(d) Conclusions based on test results in the undistorted-scale model and in the New York Harbor model, relative to Shrewsbury Inlet were:

- None of the plans tested would have significant effects on water surface elevations during normal tides or during hurricane surges.

- Current velocities and flow patterns would not change appreciably except in the immediate vicinity of the inlet.

- Current velocities in the new inlet for normal tides should not be excessive for safe navigation.

- None of the plans tested offered a unique advantage over the other plans in relation to the cross-currents during certain periods of the tide phase; however, the alignment of Plans 1 and 3 appeared to be better than that of Plans 2 and 4.

- For pollution sources in Raritan Bay, the influx of pollution into Sandy Hook Bay, Shrewsbury River, and Navesink River would be reduced slightly.

- For pollution sources in Upper New York Bay, the influx of pollutants to Sandy Hook Bay and the Shrewsbury and Navesink Rivers would be increased.

- For pollution sources in Shrewsbury and Navesink Rivers, the flushing rate would be improved by construction of the inlet.

- Plan 3 would be less expensive to maintain than the other plans tested.

- Wave energy originating in the ocean and passing through the new inlet would have insignificant effects on wave heights along the Highlands Marina shoreline.

- The wave climate between the jetties should not be difficult to navigate, except possibly under certain combinations of ocean wave conditions and critical ebb discharges in the inlet.

b. Fire Island Inlet, New York.

(1) Project description and background.

(a) The Fire Island Inlet project consisted of stabilization of the navigation channel and construction of a littoral drift trap and rehandling basin in the inlet, a connecting channel for a loaded hopper dredge, and a 305-m (1,000-ft) extension to the existing Federal jetty.

(b) Fire Island Inlet is located on the south shore of Long Island, and is the primary waterway for boat traffic between the Atlantic Ocean and Great South Bay (Figure 6-5). The inlet is about 1,067 m (3,500 ft) wide with depths to about 7.6 m (25 ft) mlw. Between 1825 and 1940, the western end of Fire Island migrated westward a distance of over 6.4 km (4 miles). A 1,524-m (5,000-ft) Federal jetty, constructed in 1940, trapped the littoral drift for about 10 years; sand then began bypassing the structure and filling the navigation channel. Corrective measures (completed in December 1959) designed to alleviate channel deposition problems and supply sand for down-beach nourishment consisted of: (a) dredging an extensive area to -5.5 m (-18.0 ft) through the mouth of the inlet; (b) using a portion of the material to construct a sand dike across the deep channel adjacent to Oak Beach; and (c) depositing an ample supply for down-beach nourishment in a feeder beach area. The sand dike was effective in diverting maximum currents from Oak Beach toward the center of the inlet; however, the entrance channel was not stabilized and continued to migrate as a result of accretion west of the Federal jetty.

(2) Purpose of model study. The Fire Island Inlet physical model study was conducted to:

(a) Investigate the proposed design of a combination sand bypassing and channel maintenance procedure, consisting of a littoral trap, a rehandling basin, an entrance channel connecting the two, and a training dike, as recommended by the U.S. Army Engineer District, New York.

(b) Investigate effects of changes in dimensions and depth of the channel, trap, basin, and dikes.

(c) Determine the need for extending the Federal jetty (estimated cost about $2,650,000 for 305 m (1,000 ft) in 1963).

(d) Determine the need for additional dikes.

(e) Establish locations and dimensions of any additional improvements needed to increase the effectiveness of the plan and maintain a stable channel through the inlet.

(3) The model.

(a) The model reproduced a 155 sq-km (60-square-mile) area including Fire Island Inlet and ocean beaches from Fire Island light on the east to beyond Gilgo on the west, along with a part of the Atlantic Ocean (Figure 6-5). The Atlantic Ocean portion of the model extended 8.1 km (5 miles) to the east and 12.1 km (7.5 miles) to the west of the Federal jetty and offshore to about the 18.3-m (60-ft) depth. The Fire Island Inlet problem was caused primarily by littoral drift being trapped in the inlet, thus starving the downdrift beaches and shoaling the navigation channel. Therefore, it was necessary that sand movement along the beaches be simulated in the model. The Fire Island Inlet model was first constructed as a fixed-bed model and all proposed improvement alternatives were tested quickly and economically to determine their effects on hydraulic conditions in the inlet. After completion of hydraulic tests, the problem area of the model was converted to a movable bed, and the most promising alternatives from the results of the fixed-bed studies were investigated further. In the fixed-bed studies, the entire model bed was molded of concrete; in the movable-bed studies, the section of the model outlined by a dashed line in Figure 6-5 was molded of sand with a mean grain diameter of 0.25 mm and a specific gravity of about 2.65.

(b) The model was constructed to linear scale relations, model-to-prototype, of 1:500 horizontally and 1:100 vertically with a resultant slope scale of 5:1. One prototype semidiurnal tidal cycle of 12 hr 25 min was reproduced in the model in 14.9 min. The computed time scale of 1:50 was applied only to the reproduction of prototype hydraulic forces in the fixed-bed model and had no relation to time required in the movable-bed model to reproduce observed changes in prototype hydrographic conditions. The results of the movable-bed verification indicated that, using the operation schedule derived empirically, the time scale for bed movement in the model was approximately 36 tidal cycles (or 9.0 hr of model operation) to 1 year in the prototype. In the final verification test, a total of 0.77×10^6 m^3 (1.0×10^6 yd^3) of movable-bed material was introduced at the extreme east end of the model beach to replace the quantity of sand moved in a westerly direction by the model waves. This rate of movement, applied to the empirically determined time scale for bed movement, is in close agreement with the computed rate of prototype littoral drift along this reach of the south shore of Long Island.

(4) Fixed-bed model tests. Eleven different plans were investigated during the fixed-bed phase of the model study to determine plan effects on tides, current velocities, tidal discharges, current patterns, and flow distribution in the problem area. These plans involved: (a) development

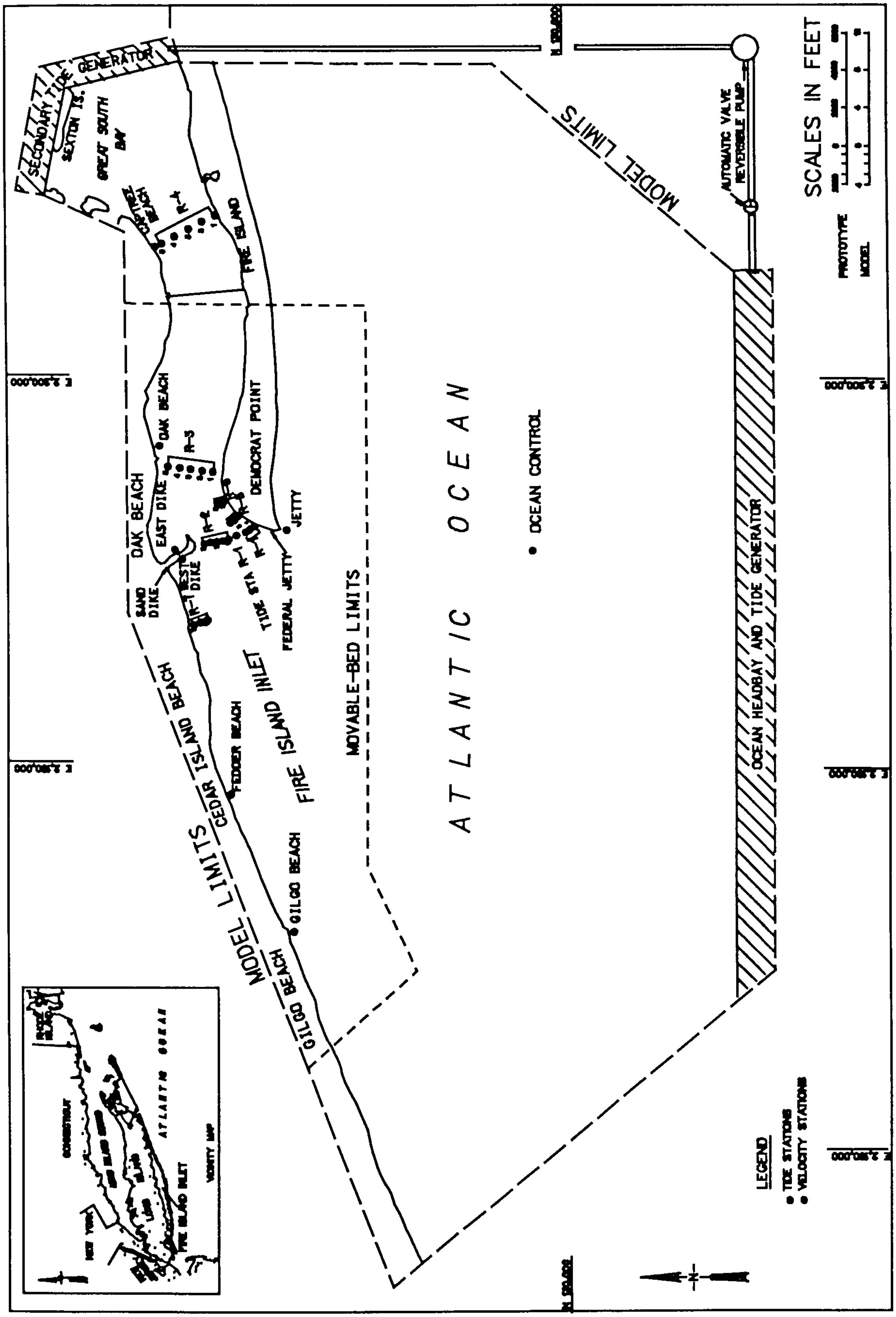

Figure 6-5. Fire Island Inlet model limits

of the best location and dimensions for the littoral drift reservoir and the rehandling basin (Plans 2, 3, and 3A); (b) groins along Oak Beach to divert the stronger ebb currents away from the beach and into the navigation channel (Plans 4 and 4B); (c) extension of the Federal jetty (Plans 6 and 7); (d) dikes to partially or completely close the secondary channel off the end of the existing sand dike and thus divert more flow into the navigation channel (Plans 8, 9, and 10); and (e) a deflector dike located on the west side of the navigation channel to deflect some ebb flow from the secondary channel into the navigation channel (Plan 5).

(5) Movable-bed model tests. In the movable-bed tests, the effects of plans on movement and deposition of bed material were determined by direct comparison of test results for existing conditions with those incorporating proposed improvements. Base movable-bed tests were started with known model-bed configurations, the model tide and wave generators were operated through a predetermined schedule, and the model bed was surveyed periodically to record developments during the test. The movable-bed test for any plan was an exact duplicate of the base test except that the alternative plan was installed at the beginning of the test. Effects of the plan were determined by comparing the configuration of the model bed at the end of the base test.

(6) Results. The verification period selected for the Fire Island Inlet model was the 2 years immediately following construction of the navigation channel in November 1964. A careful study determined the following significant changes in prototype bed configurations during this period: (a) a scour area developed just north of the inner part of the navigation channel, (b) a shoal about 1.8 to 3.7 m (6 to 12 ft) high developed in the inner part of the inlet channel, and (c) a shoal about 1.8 to 3.7 m (6 to 12 ft) high developed along the eastern edge of the outer part of the navigation channel. After operating the model for 72 tidal cycles, it was found that gross changes in bed configuration in the model were very similar to those indicated by comparison of the prototype surveys made 2 years apart. Bed movement during verification of a movable-bed model is not expected to duplicate exactly all changes in prototype bed configuration during the selected verification period because (a) trends in bed movement in the prototype are not constant with time, although the model verification procedure constitutes an attempt to reproduce such movement on an average basis, and (b) certain observed changes in the prototype are the result of storms or other extreme conditions, which the model verification (necessarily based on average conditions) cannot be expected to reproduce. Instead, the model-bed movement verification is an attempt to reproduce the gross changes that occurred in the prototype between the beginning and end of the verification period. Minor discrepancies of a local nature, and those attributable to unusual prototype conditions, may be neglected. Deviations of the Fire Island model from prototype conditions during the 2-year period are shown in Figure 6-6.

(7) Conclusions. Conclusions based on the results of hydraulic and movable-bed model tests concerning proposed improvement plans for Fire Island Inlet were:

(a) Plan 3A, which required a littoral drift reservoir dredged to -10.4 m (-34 ft), a deposition or rehandling basin to -8.5 m (-28 ft) when filled, and an 8.5-m-deep (28-ft-deep) connecting channel, will result in a safe and stabilized navigation channel with enough sand for bypassing to the downdrift beaches. Maintenance dredging of the littoral trap and connecting channel will be required about every 2 years; most of this dredging can be accomplished with conventional dredging plants. Since deposition in the littoral trap and connecting channel occurs in the form of a high bar, which builds from east to west, a sidecast dredge or similar equipment will probably be required to lower the bar crest enough for a hopper dredge to restore the basin depths.

(b) The removal of large deposits of sand from Fire Island Inlet for beach restoration projects will not adversely affect the functioning of Plan 3A.

(c) Extension to the existing sand dike will not improve the functioning of Plan 3A in channel shoaling or deposition patterns. Likewise, construction of a dike at Cedar Island Beach would not improve the functioning of Plan 3A.

(d) A deflector dike parallel to the entrance channel would not improve the functioning of Plan 3A from either shoaling of the navigation channel or deposition patterns; further, the dike would be extremely difficult to maintain.

(e) Extension of the Federal jetty would not reduce channel shoaling or improve deposition patterns observed for Plan 3A. An extension to the jetty would reduce channel shoaling on a temporary basis while the impounding capacity of the extended jetty was being filled; shoaling rates and deposition patterns would then be the same as for Plan 3A. Material impounded by a jetty extension

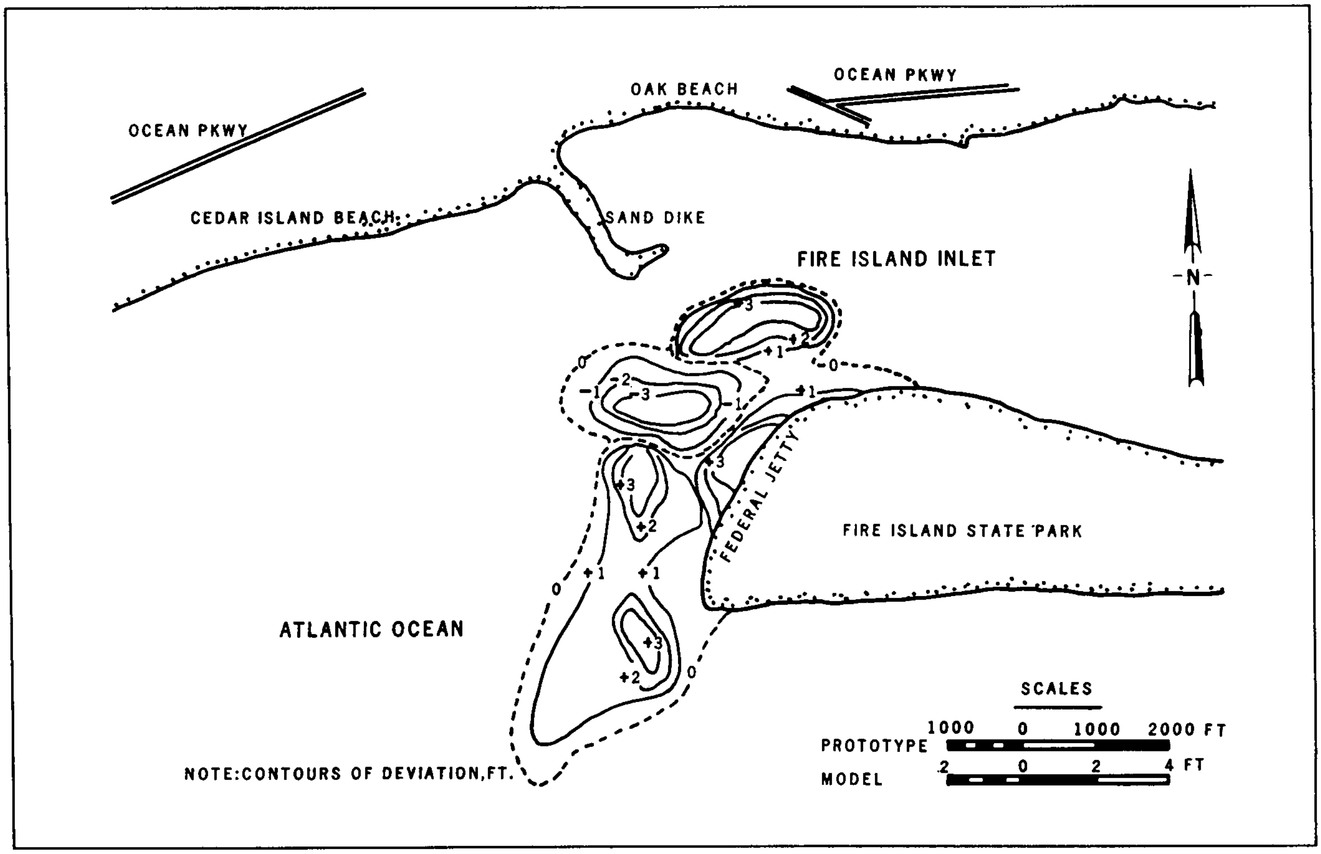

Figure 6-6. Fire Island Inlet prototype scour and fill during a 2-year period

would also be permanently lost as a source of beach nourishment material.

(f) Extension of the Federal jetty, as a combination low weir inner section and high outer section, did not function as intended. Instead of moving readily over the low weir section, a large percentage of the littoral drift was trapped to the east of the jetty extension; it appeared that the weir section would be blocked and thus would be rendered completely inoperative.

(g) An offshore breakwater and littoral trap would also satisfy all necessary requirements of a plan for channel stabilization and sand bypassing at Fire Island Inlet.

c. Murrells Inlet, South Carolina.

(1) Project description and background. Murrells Inlet is located along the South Carolina coast between Georgetown and Myrtle Beach. The inlet provides access to a well-mixed tidal lagoon of ocean salinity that has no source of freshwater inflow other than local surface runoff. The mean ocean tide range is 1.5 m (4.8 ft) and ebb and flood currents transport a tidal prism of 7.2 million m^3 (253 million ft^3) during a tidal cycle of 12.42 hr. During the last 100 years, Murrells Inlet has migrated more than 2.1 km (7,000 ft). Pre-project conditions at the inlet consisted of a difficult and hazardous navigational environment as the main channel exhibited rapid changes in location and depth, and waves frequently broke on the shallow shoals. The Murrells Inlet project consisted of constructing two jetties, a navigation channel, and a deposition basin.

(2) Purpose of model study. The primary purpose of jetties was to prevent longshore sediment from shoaling the channel and offer protection from waves to incoming and departing vessels. More recently, jetty design has taken the problem of littoral drift into consideration by providing weir sections in the jetties and sediment traps adjacent to the weir to capture the longshore transport of sand. Thus, sediment is kept out of the channel and placed in a location where it can be utilized for future beach nourishment efforts. This type of jetty construction was authorized for Murrells Inlet. One aspect of the study included construction of a physical model to

determine: (a) optimum alignment and spacing of the jetties; (b) channel alignment and current patterns at the entrance; (c) effects on the tidal prism and bay tidal elevations and velocities; and (d) wave heights in the entrance channel and deposition basin.

(3) The model. A distorted scale model of 1:200 horizontal and 1:60 vertical scales was selected (Figure 6-7). The entire lagoon was modeled to permit study of tidal elevations, currents, and tidal prism. A distorted-scale model must be verified for tidal currents and elevations, therefore, prototype measurements of these parameters were required. Data were collected from tide gauges at locations shown in Figure 6-7 and reproduced in the model by the adjustment of roughness elements usually required in distorted models.

(4) Model tests. After verification of tidal parameters, various jetty plans were installed in the model for testing. Preliminary testing consisted of measuring wave heights in the entrance channel and inner channels for a number of test waves at various stages of the tidal cycle, measuring tidal elevations at the verified locations for the entire tidal cycle, and taking surface current photographs at the entrance throughout the tidal cycle. Examination of these preliminary data permitted a reduction in the number of plans subjected to further testing (detailed current measurement and wave height measurements). Further refinements could then be made in the design. For example, Plan 1B (Figure 6-8) was selected for further testing. This plan gradually evolved into Plan 1H (Figure 6-9) as changes were made in the widths and depths of the inner auxiliary channels connecting the main navigation channel to the bay to improve flow patterns and flow admittance.

(5) Results. Results from physical model tests indicated an optimal jetty spacing of 183 m (600 ft) to provide adequate scouring currents in the channel but still maintain a tidal prism similar to that of pre-jetty conditions. The access channel to the deposition basin was relocated; and a training dike was added to prevent ebb currents from entering the region of the deposition basin. Figure 6-10 shows the completed project in January 1981. The only element of the plan not constructed was the training dike, which may be added at a later date if required. The deposition basin is filling and, to date, the navigation channel has naturally maintained depths greater than the project depth.

d. Rogue River Harbor, Oregon (Bottin 1982).

(1) Project description. The Rogue River originates in the Cascade Mountain Range and flows generally westward entering the Pacific Ocean along the Oregon coast, approximately 48.3 km (30 miles) north of the California border (Figure 6-11). The river is about 290 km (180 miles) long and drains an area of approximately 13,200 km^2 (5,100 square miles) (CTH 1970). The principal communities at the river mouth are Gold Beach and Wedderburn, located on the south and north banks, respectively. These areas have been developed for resort and recreational use. Prior to improvements, the river channel at the mouth meandered between two sand spits and was seldom less than 61 m (200 ft) wide at low water. Controlling depths over the entrance bar ranged from 0.61 m (2 ft) in late summer to 2.7 m (9 ft) in winter. The River and Harbor Act of 1954 provided for the construction of parallel jetties spaced approximately 31 m (100 ft) apart at the mouth of the river. In 1971 and 1972, the Port of Gold Beach constructed a breakwater that extended from a point on the south bank (about 305 m (1,000 ft) above the U.S. 101 highway bridge) downstream to the south jetty. A gap was left in the breakwater to provide access to harbor facilities.

(2) Project background. Every year a persistent shoaling problem exists between the Rogue River jetties. The shoal extends upstream along the inside of the south jetty and across the harbor access channel (Figure 6-12). This condition makes maintenance dredging difficult and blocks navigation channels, thus restricting vessel traffic between the ocean and port facilities. Rapid shoaling occurs in the summer (when river flows are normally low) during peak boating and salmon fishing seasons, causing unpredictable and hazardous entrance conditions. Authorized channel dimensions could not be maintained by dredging due to the rapid shoaling rate; therefore, a series of improvement plans were devised and subjected to testing.

(3) Purpose of model study. A physical model investigation was conducted to study shoaling, wave, current, and riverflow conditions in the lower reaches of the Rogue River for existing conditions and proposed improvements.

(4) The model. The Rogue River Harbor model (Figure 6-13) was constructed to an undistorted linear scale of 1:100, model to prototype. Test waves used in the model study ranged in period from 5 to 17 sec and in height from 2.1 to 8.8 m (7 to 29 ft). A water circulating system was used to reproduce steady-state flows corresponding to maximum flood and ebb tidal flows or various river discharges. River discharges ranging from 1,420 to 9,900 m^3/sec (50,000 to 350,000 cfs) were reproduced in the model. A coal tracer material was used in

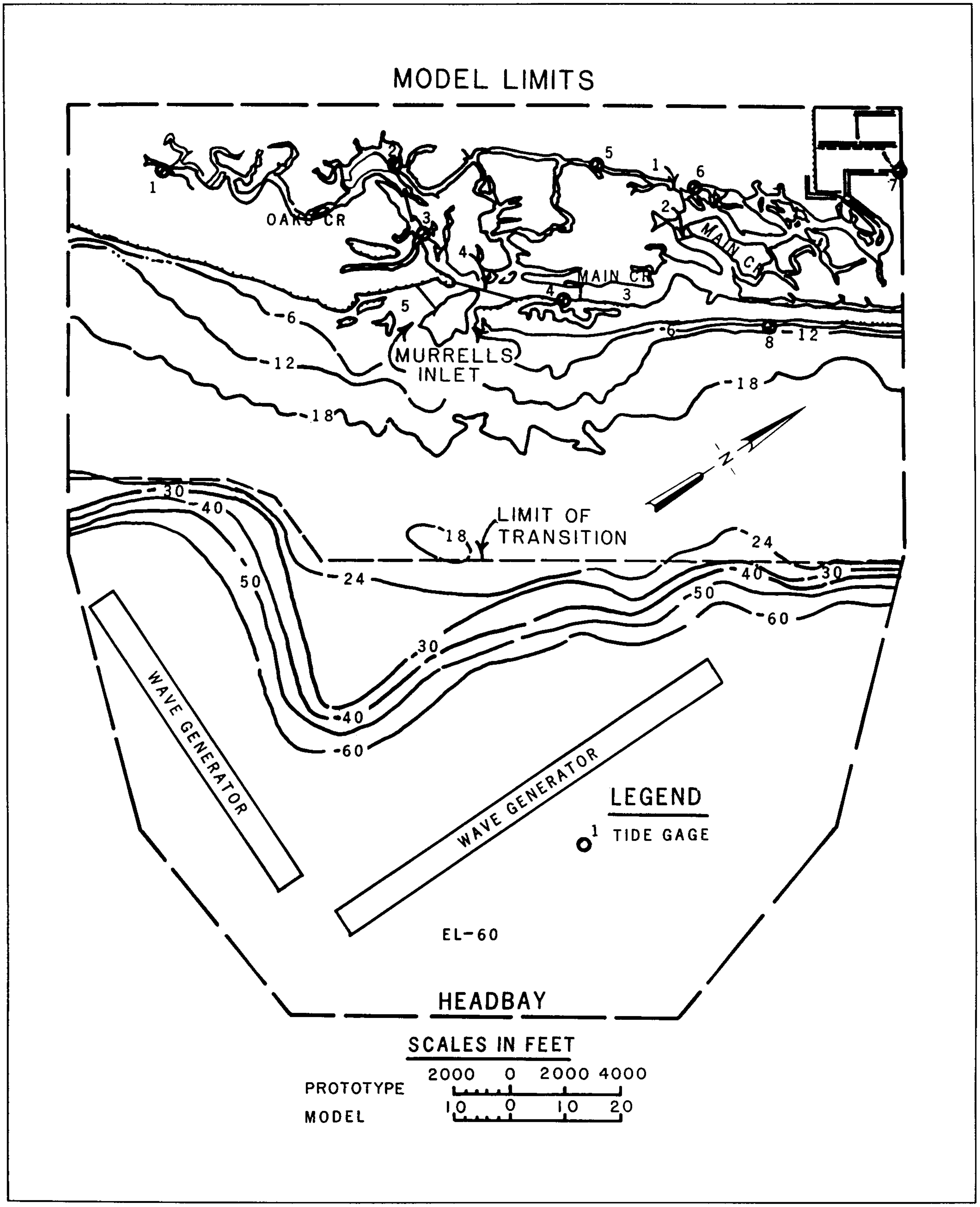

Figure 6-7. Murrells Inlet physical model layout

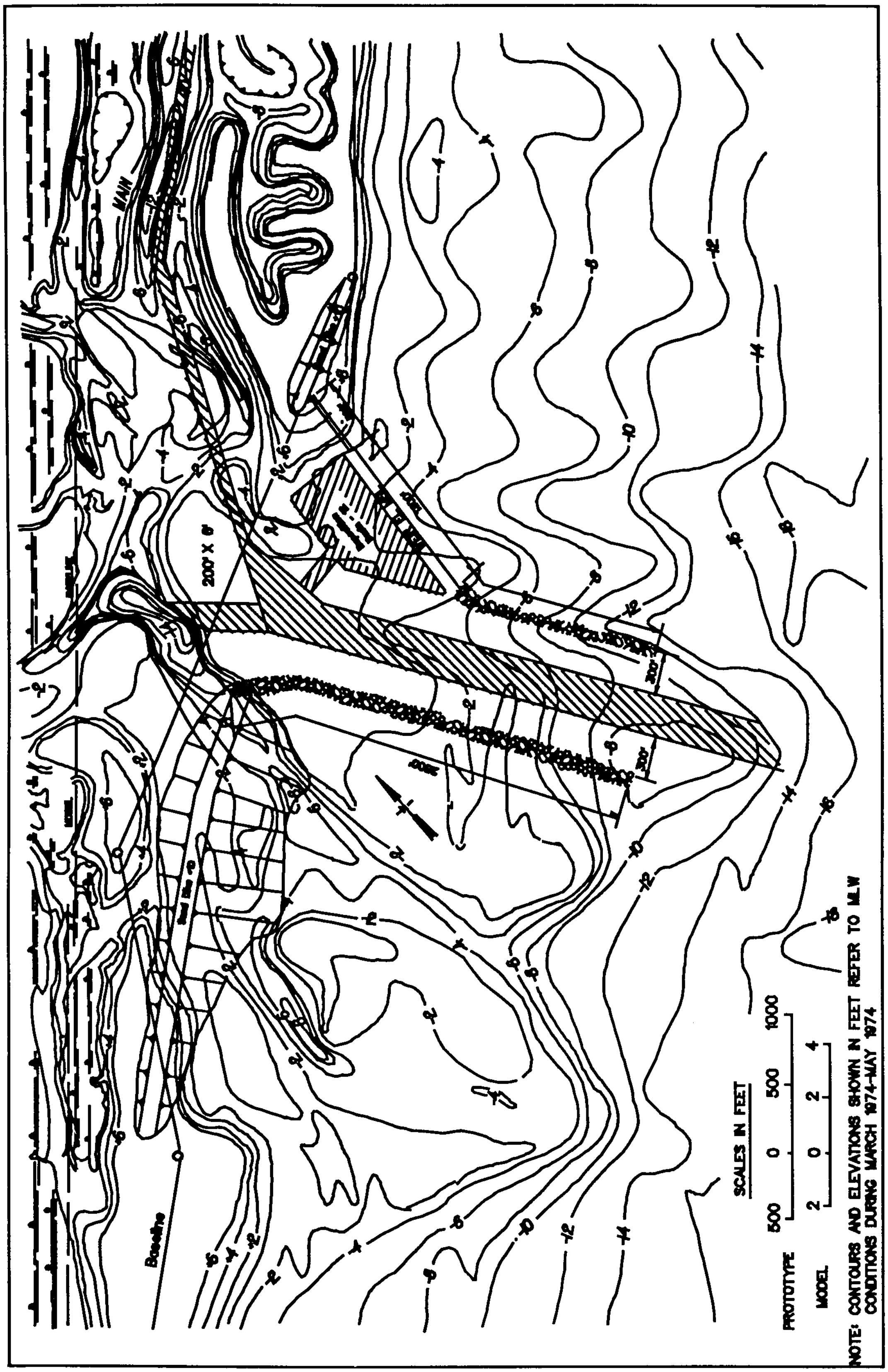

Figure 6-8. Typical plan for improvement of Murrells Inlet

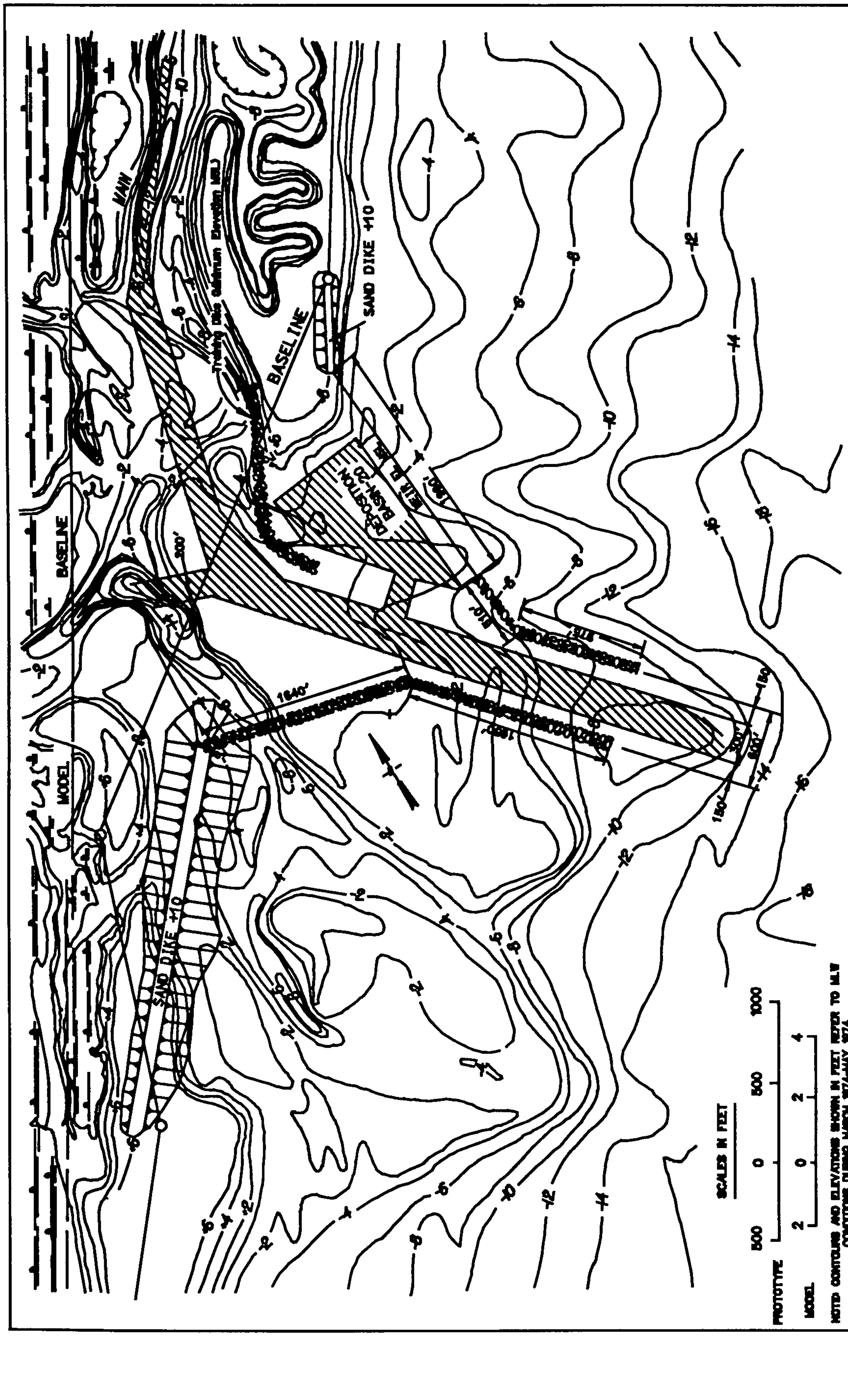

Figure 6-9. Optimum improvement plan for Murrells Inlet

Figure 6-10. View of Murrells Inlet project as constructed in 1981

the model to qualitatively determine the degree of shoaling at the river mouth. Still-water levels of 0.0 m (0.0 ft) (mean lower low water (mllw)) +0.46 m (+1.5 ft) (maximum ebb), +1.5 m (+4.3 ft) (maximum flood), and +2.0 m (+6.7 ft) (mean higher high water (mhhw)) were used during model testing. An automated data acquisition and control system was used to obtain wave height data, and water-surface profiles for various river discharges were determined by recording elevation changes on point gauges located at various stations in the river. A general view of the model is shown in Figure 6-14.

(5) Model tests and results: Existing conditions. Prior to tests of the various improvement plans, comprehensive tests were conducted for existing conditions. Wave-height data, wave-induced current patterns and magnitudes, shoaling patterns, and wave pattern photographs were obtained for representative test waves from four selected test directions. Water-surface elevations and river current velocities also were obtained for the various river discharges. During shoaling tests, tracer material was introduced into the model south of the south jetty and north of the north jetty to represent sediment from those shorelines, respectively. In addition, tracer was introduced seaward of the river mouth to represent sediment washed out of the river and deposited by various discharges. Shoaling tests conducted for existing conditions

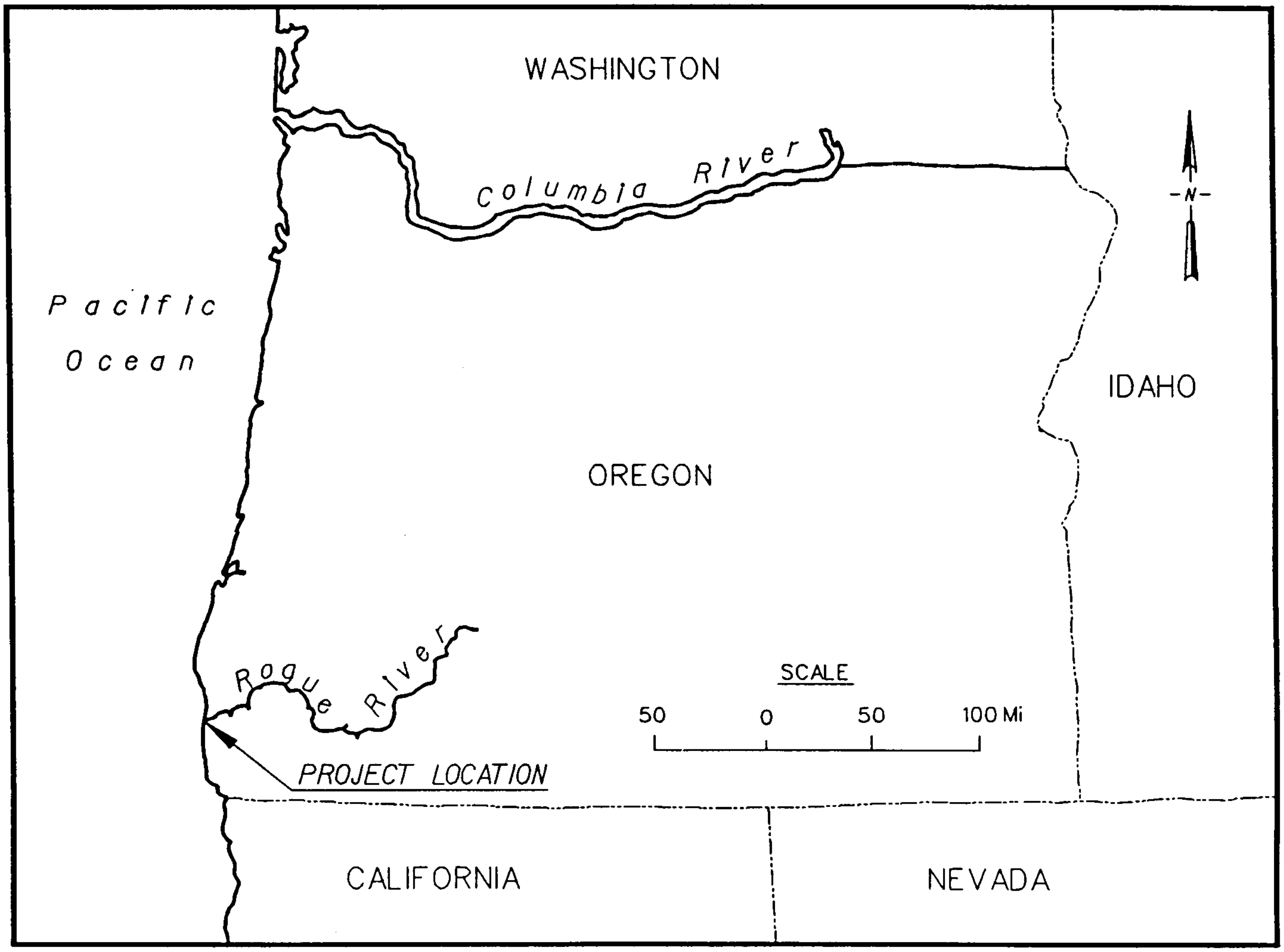

Figure 6-11. Rogue River project location

indicated that shoaling would occur in the lower reaches of the river for various test waves for each wave direction. Generally, material was deposited in the southern portion of the river adjacent to the south jetty. Under constant wave attack, this material accumulated against the south jetty and migrated upstream across the entrance to the small-boat harbor (Figure 6-15) forming a shoal similar to that of the prototype. It was also noted that, when the shoal was present, rough and turbulent wave conditions existed in the entrance (due to waves breaking on the shoal) and higher than normal river stages and river-current velocities may result for various discharges (since the shoal interferes with the passage of flood flows). When the shoal was not present, increased wave heights would be expected upstream of the small boat harbor entrance.

(6) Model tests and results: Improvement plans. Model tests were conducted for 58 variations in the design elements of three basic remedial improvement plans. Dikes installed within the existing entrance, extensions of the existing jetties, and an alternate harbor entrance were tested. Wave-height tests, wave-induced current patterns and magnitudes, wave patterns, water-surface elevations, river current velocities, and/or shoaling tests were conducted for the various improvement plans. The first series of test plans included the installation of dikes within the existing entrance. Both timber-pile and rubble-mound dikes were tested. Test results indicated shoaling of the small-boat harbor entrance would occur with the timber-pile dikes installed. The rubble-mound dike configuration, however, intercepted the movement of tracer material and prevented it

Figure 6-12. Aerial photograph of Rogue River mouth

from shoaling the harbor entrance. Water-surface elevations obtained for the dike plans indicated that river stages would increase compared to those for existing conditions, potentially contributing to flooding problems. The installation of a weir section in the existing north jetty and a conveyance channel on the north overbank reduced river stages upstream by less than 0.3 m (1 ft) and therefore, was not successful in decreasing water-surface profiles to desired levels.

(7) Additional tests. The next series of test plans involved extensions of the existing jetties. One plan consisted of extending the jetties on their original alignment, another involved orienting the extensions toward the west (on azimuths of S81°41'30"W and S16°23'22"W). Test results, with extensions along existing alignments, indicated that sediment from the river would form a shoal in the entrance adjacent to the south jetty and then extend upstream across the small-boat harbor entrance similar to existing conditions. With tests involving jetty extensions toward the west, sediment from the river would form shoals in the river entrance but would not extend upstream to the small-boat basin entrance. With tests involving jetty extensions to the south, sediment from the river would result in a shoal along the south jetty extension, extending north into the entrance. The shoals that formed in the entrance with all three jetty extension plans consisted of sediment introduced to the system by the river, since each plan provided shoaling protection from sediment from the north and south shorelines.

(8) Final test series. The last series of test plans involved a new entrance south of the existing river mouth. Test results indicated that this new jetty configuration would provide shoaling protection for the new entrance

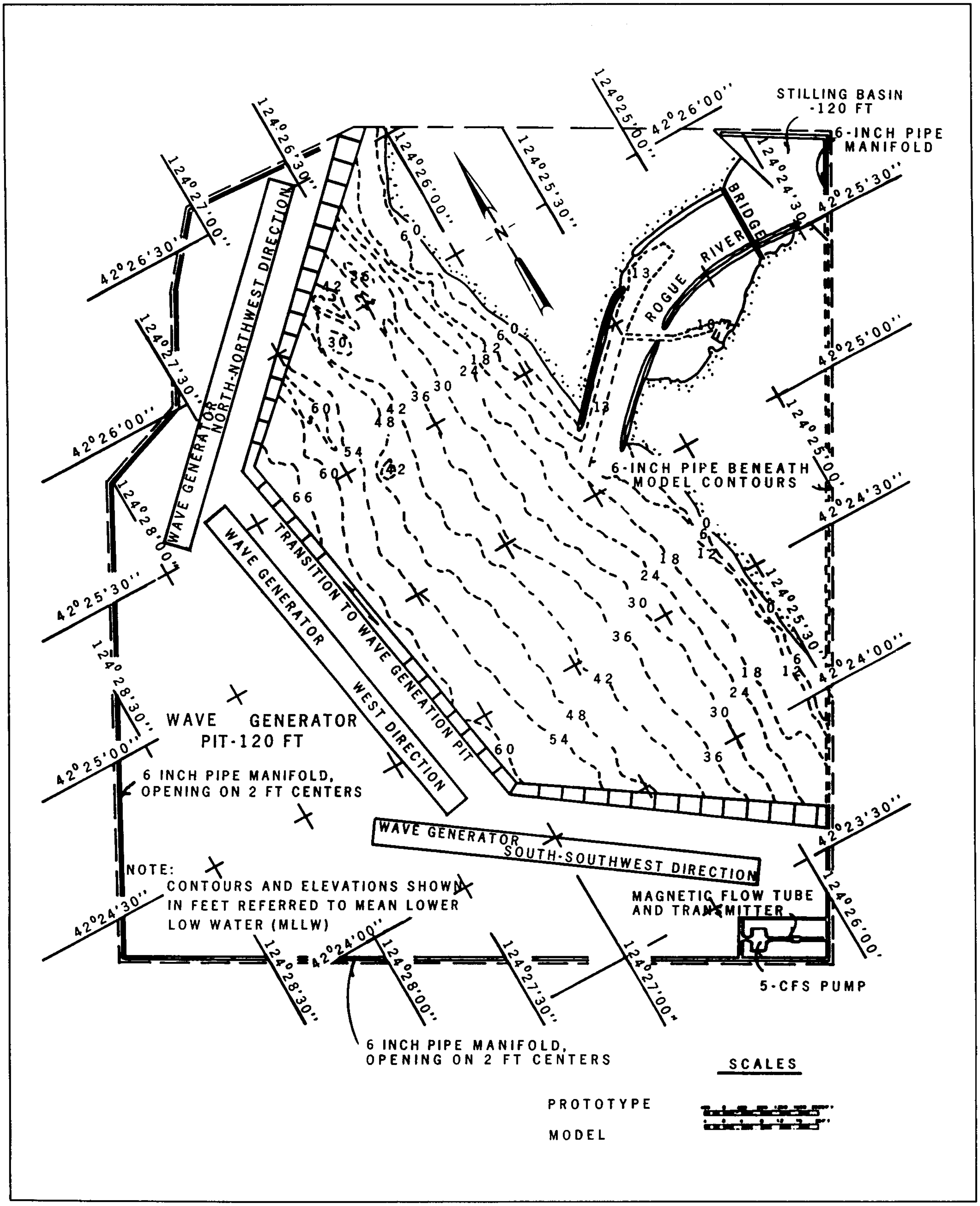

Figure 6-13. Rogue River physical model layout

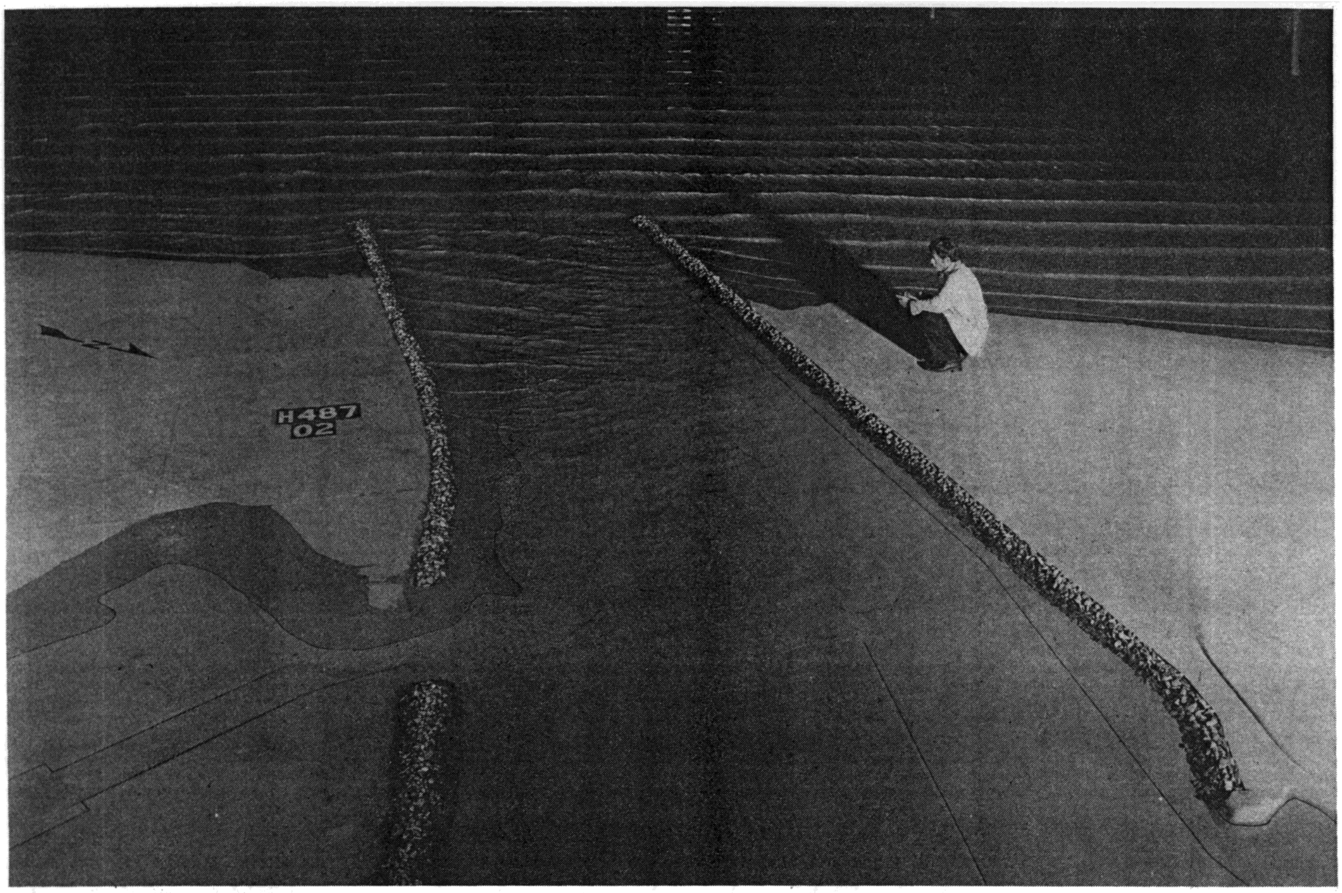

Figure 6-14. General view of Rogue River physical model

from sediment on the north and south shorelines and sediment deposited seaward of the river entrance by various discharges. In addition, this plan would provide wave protection to the small craft harbor with maximum wave heights less than 0.3 m (1 ft).

6-7. Listing of Physical Model Studies

A listing of inlet physical model studies shows the variety of specific problems for which model investigations have been conducted.

a. Wave action studies (short-period waves in entrance).

(1) Oregon Inlet, North Carolina (Hollyfield, McCoy, and Seabergh 1983).

(2) Newburyport Harbor, Massachusetts (Curren and Chatham 1979).

(3) Murrells Inlet, South Carolina (Perry, Seabergh, and Lane 1978).

(4) Wells Harbor, Maine (Bottin 1978).

(5) Little River Inlet, South Carolina (Seabergh and Lane 1977).

(6) Masonboro Inlet, North Carolina (Seabergh 1976).

(7) Barnegat Inlet, New Jersey (Sager and Hollyfield 1974).

(8) Nassau Harbor, Bahamas (Brasfield 1965).

b. Shoaling studies (entrance shoaling protection).

(1) Oregon Inlet, North Carolina (Hollyfield, McCoy, and Seabergh 1983).

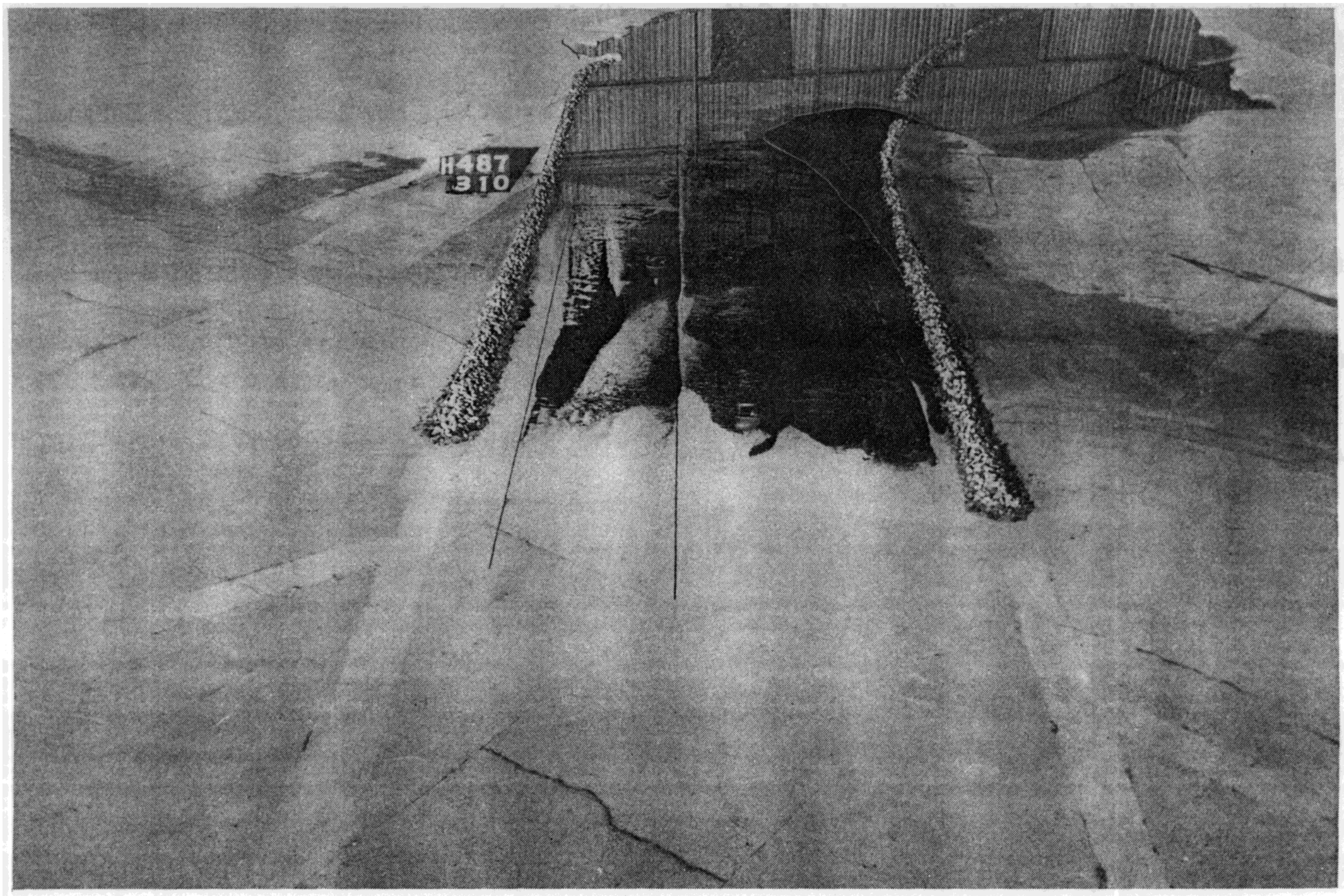

Figure 6-15. Shoal formed in the Rogue River entrance (existing conditions)

(2) Newburyport Harbor, Massachusetts (Curren and Chatham 1979).

(3) Little River Inlet, South Carolina (Seabergh and Lane 1977).

(4) Masonboro Inlet, North Carolina (Seabergh 1976).

(5) Barnegat Inlet, New Jersey (Sager and Hollyfield 1974).

c. Wave-induced circulation/current studies.

(1) Newburyport Harbor, Massachusetts (Curren and Chatham 1979).

(2) Wells Harbor, Maine (Bottin 1978).

d. Tidal circulation/flood and ebb currents.

(1) Oregon Inlet, North Carolina (Hollyfield, McCoy, and Seabergh 1983).

(2) Newburyport Harbor, Massachusetts (Curren and Chatham 1979).

(3) Murrells Inlet, South Carolina (Perry, Seabergh, and Lane 1978).

(4) Wells Harbor, Maine (Bottin 1978).

(5) Little River Inlet, South Carolina (Seabergh and Lane 1977).

(6) Masonboro Inlet, North Carolina (Seabergh 1976).

(7) Barnegat Inlet, New Jersey (Sager and Hollyfield 1974).

(8) Nassau Harbor, Bahamas (Brasfield 1965).

e. Tidal elevation studies (water-surface).

(1) Oregon Inlet, North Carolina (Hollyfield, McCoy, and Seabergh 1983).

(2) Murrells Inlet, South Carolina (Perry, Seabergh, and Lane 1978).

(3) Little River Inlet, South Carolina (Seabergh and Lane 1977).

(4) Masonboro Inlet, North Carolina (Seabergh 1976).

(5) Barnegat Inlet, New Jersey (Sager and Hollyfield 1974).

f. Salinity studies.

(1) Little River Inlet, South Carolina (Seabergh and Lane 1977).

Chapter 7
Numerical Modeling of Tidal Inlets

7-1. Purpose and Scope

Coastal phenomena such as waves, currents, water levels, flow discharge, water quality, and sediment transport can be numerically simulated at inlets to predict impacts of existing or proposed design alternatives. For example, it may be necessary to maintain or improve inlet characteristics such as water quality, channel navigability, structural integrity, channel shoaling rates, and sediment bypassing strategies for a particular inlet configuration or maintenance plan. By comparing existing coastal processes to those simulated, effects of design plans and operation and maintenance (O&M) practices can be assessed and optimized. The purpose of this chapter is to describe numerical models that have been used to predict these various coastal phenomena at inlets. Section 7-2 presents an overview of various physical processes normally considered in numerical models of tidal inlets. Sections 7-3 through 7-7 discuss different types of numerical models and modeling systems that have been applied in Corps studies and are available to Corps field offices. A brief description of each model is followed by a list of model input and output requirements, example model applications, and an additional bibliography. In Section 7-8, the implementation of numerical models is discussed. This section deals with numerical grid characteristics, grid generation, and calibration and verification of the models. Finally, Section 7-9 discusses engineering use of model results.

7-2. Overview of Physical Processes Considered

The following physical processes are usually considered in numerical modeling of tidal inlets under nonstorm conditions: astronomical tides, winds, short period waves, freshwater flows, and sediment transport. Under hurricane and storm conditions, the effects of storm surge also have to be accounted for (refer to EM 1110-2-1412).

a. Astronomical tides. Tides can be a major forcing mechanism at inlets. Tides are long-period waves, which can be predicted accurately along the open coast using results of harmonic analysis of measured water level fluctuations. Near inlets and in the interior, numerical models must be used for tidal prediction because of the complex interactions between bathymetry, inlet and back-bay geometry, proximity of structures, and interconnection with other inlets. Tides change currents and water levels, which are important for circulation and sediment transport.

b. Winds. Winds induce a change in water level (wind setup) and currents, the magnitude of which depends on wind speed and direction. Water level increases in the direction of the wind. Currents are in the direction of the wind at the surface, but direction and magnitude may vary in the vertical. Wind effects are usually accounted for in either a tidal or a wave-induced current model.

c. Short-period waves. Short-period ocean waves are represented near inlets either by a monochromatic wave (e.g., significant wave) or a wave spectrum. In the first approach, individual waves are characterized by wave height, period, and direction. In the second approach, a wave with a specified height is characterized by the distribution of energy in different frequency (period) and direction bands. Short waves result in changes in water level (wave setup) and wave-induced currents (longshore and rip currents) near inlets which cause not only changes in flow pattern, but also sediment transport. Wave orbital velocities at the bed cause increased shear stresses, resulting in greater sediment transport. Because of the complex transformation processes which take place in the nearshore, short waves are predicted near inlets using numerical models of the monochromatic or spectral type. In either case, the wave characteristics in deeper water are either measured in the field, or obtained from forecast or hindcast performed using a spectral model (e.g., Phase II of WES Wave Information Study (WIS)).

d. Freshwater flows. Freshwater flows into the back-bay system from rivers and creeks influence both flow patterns and salinities. Data on such flows are obtained from agencies such as the U.S. Geological Survey (USGS), state and local water resources agencies, and/or from special gauges installed for the project. These flows have to be specified at the boundaries of the numerical model grid.

e. Sediment transport. Magnitude and direction of inlet sediment transport depend on the processes described in *a* through *d*. Sediment transport at inlets is of major concern to coastal engineers and planners, because its rate and distribution through the inlet affect many processes of engineering concern (e.g., channel shoaling rates, erosion/ accretion of interior (bay) and ocean (adjacent) inlet shorelines, stability of structure foundations (jetties, bridge pilings), etc.). Modifications which change the existing transport rates and patterns can disrupt the integrity and viability of a stabilized, navigable inlet. Typically, sediment transport in the back bay is characterized by cohesive materials such as clays, silts, and fine sands, whereas transport in the region offshore of the inlet throat is characterized by noncohesive materials such as sand and shell. Usually, sediment transport models use the results of hydrodynamic models for input.

7-3. Long-Wave Models

a. Lumped parameter models. This type of model gets its name from "lumping" several important variables together, such as discharge or back-bay storage capacity. An example of a lumped parameter model is the Spatially Integrated Numerical Model of Inlet Hydraulics, an inlet-bay hydraulic model (Figure 7-1) developed by Seelig (1977) and Seelig, Harris, and Herchenroder (1977), and available through the U.S. Army Corps of Engineers' (USACE's) Automated Coastal Engineering System (ACES) (Leenknecht, Szuwalski, and Sherlock 1992a, 1992b).

(1) Description. This model can be used to calculate coastal inlet velocities, discharge, and bay surface level as a function of time for a given time-dependent sea level fluctuation. It is applicable to one or two inlets connected to a bay with two sea boundary conditions, although only the one-inlet, one-bay system has been tested in the ACES (Leenknecht, Szuwalski, and Sherlock 1992a). The one-dimensional equation of motion is integrated over the area of the inlet. The resulting momentum equation and continuity equation are solved by marching in time. The model idealizes the inlet-bay geometry and makes several simplifying assumptions, detailed by Seelig, Harris, and Herchenroder (1977) and Leenknecht, Szuwalski, and Sherlock (1992a, 1992b). The model can be used to evaluate inlet velocities, bay water level fluctuations, and discharges caused by tides, storm surge, seiches, and tsunamis. This type of model can be used to take a quick "first look" at several alternatives. The model should be calibrated and preferably verified for a given project before it is used for prediction.

(2) Model requirements. Five general types of information are required for input: general data describing system configuration and temporal data; inlet geometries characterized with cross-section tables and locations; seaward boundary conditions (tabulated records or predicted tides using harmonic constituents); bayside boundary conditions (bay area and shape factor, and other freshwater inflows distinct from inlet contributions); and locations where velocity hydrographs are to be reported from the simulation (Leenknecht, Szuwalski, and Sherlock 1992a). The model should be calibrated (and verified, if possible) using known bay surface elevations and inlet velocity measurements. Three output data files are written, consisting of tabular summaries of grid characteristics, velocity hydrographs at selected cell locations, and elevation and discharge hydrographs for the sea boundary conditions, bay, and inlet(s). Samples of model runs (input and output) are given by Leenknecht, Szuwalski, and Sherlock (1992a).

(3) Example applications. The model has been applied to Pentwater Inlet, Michigan, to study the response of a nontidal Great Lakes inlet to forcing due to seiching of Lake Michigan; a hypothetical harbor to predict tsunami-induced hydraulics; Masonboro Inlet, North Carolina, to determine inlet response to tides (Harris and Bodine 1977); Indian River Inlet, Delaware, to predict the effect of storm surge at a tidal inlet; and Cabin Point Creek, Virginia, to illustrate the effect of adding a second inlet to a one-inlet tidal system.

(4) Bibliography. Seelig (1977) and Seelig, Harris, and Herchenroder (1977) discuss development of the original model. Leenknecht, Szuwalski, and Sherlock (1992a, 1992b) discuss limitations and application of the model within the ACES.

b. One-dimensional models. An example of a one-dimensional (1-D) model is the Dynamic Implicit Numerical Model of One-Dimensional Tidal Flow through Inlets (DYNLET1) (Amein and Kraus 1991). The discussion presented herein has been abbreviated from Amein and Kraus (1991).

(1) Description. DYNLET1 is based on the full one-dimensional shallow-water equations, employing an implicit finite-difference technique. The model is suited for reconnaissance-level studies for most inlets, providing reliable and accurate answers with minimal data entry and grid generation. DYNLET1 predicts flow conditions in channels with varied geometry, and will accept varying friction factors across an inlet channel, geometric data at variable distances across and along an inlet channel, and a variety of boundary conditions. The inlet to be modeled may consist of a single channel connecting the sea to the bay, or it can be a system of interconnected channels, with or without bays. Values of water surface elevation and average velocity are computed at locations across and along inlet channels.

(2) Model requirements. DYNLET1 uses four input files and generates five output files. The inlet is represented by a series of channels, junctions, and nodes. If

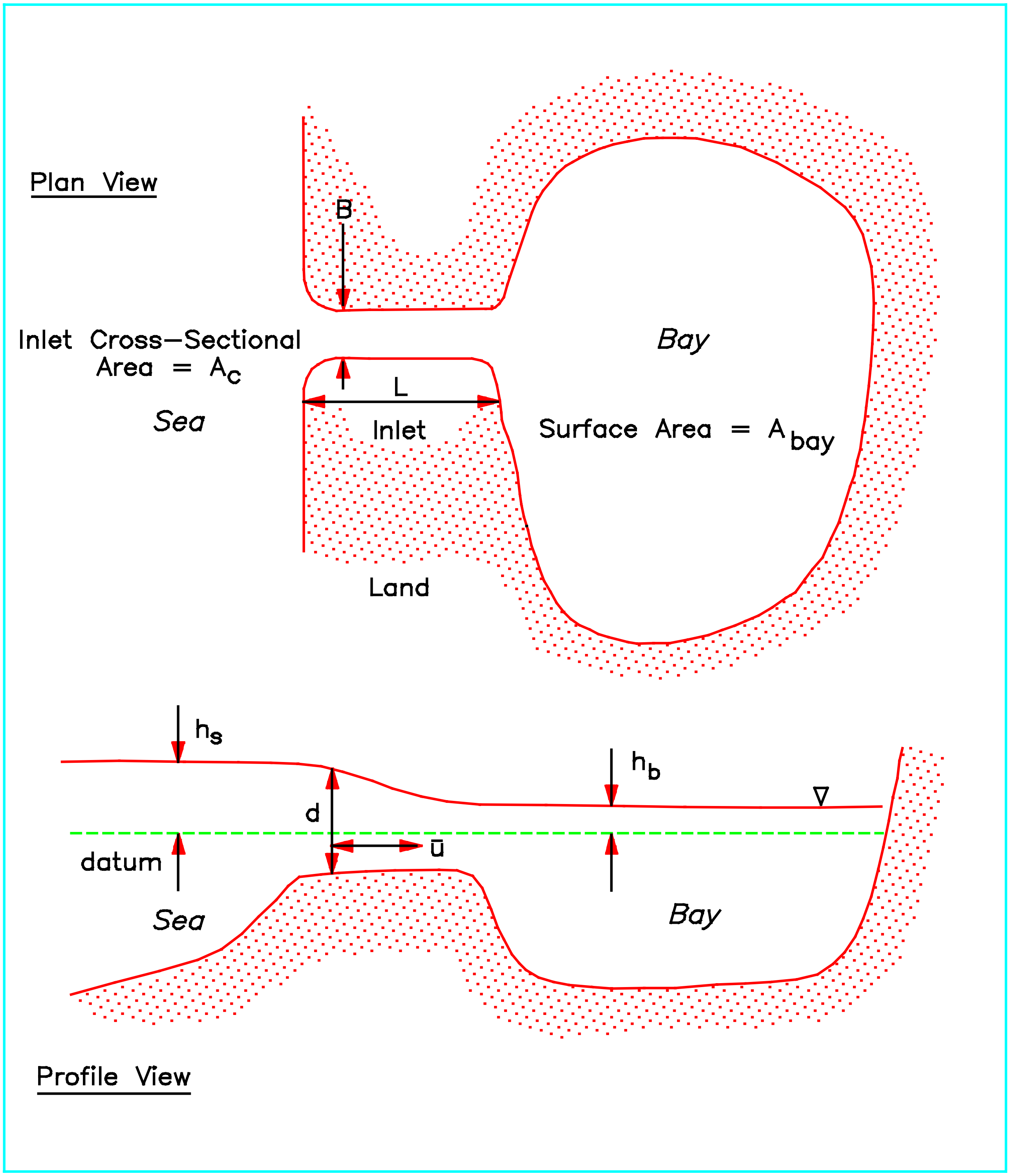

Figure 7-1. Inlet-bay system (Seelig, Harris, and Herchenroder 1977)

two channels meet or a channel branches into two forks, a junction is formed. Each representative cross section in a channel is identified by a node. Input data required include general setup parameters, detailed information on cross-section geometry and boundary resistance, time-dependent boundary data for each external boundary node, tabulated as a function of time, and desired output nodes and parameters. The primary output file includes an echo of the input data, computed values of the volume flow rate, water surface elevations, and average velocity at designated nodes at the specified times. Three other output files contain information that may be used with auxiliary programs, to facilitate evaluation and plotting of the results. The fifth output file contains nodal values of the convective acceleration, temporal acceleration, and pressure gradient normalized by the bottom stress, so that the strengths of these terms relative to that of the friction term can be evaluated.

(3) Example applications. Amein and Kraus (1991) present application and verification of DYNLET1 with two case studies: an inlet which has a system of interconnected channels without a well-defined bay, Masonboro Inlet, North Carolina; and Indian River Inlet, Delaware, an inlet with two well-defined bays that is protected by two jetties at its entrance.

(4) Bibliography. The theory and procedures for operation, application, and verification of DYNLET1 are presented by Amein and Kraus (1991).

c. Link-node model. An example of this type is the DYNTRAN (Dynamic Transport) link-node model of Camp, Dresser, & McKee (Moore and Walton 1984).

(1) Description. In DYNTRAN, the prototype system is represented by a network of nodes and links (Figure 7-2). A node corresponds to a particular reach of the prototype, having a certain volume and surface area. A link represents a channel or other pathway along which water flows from one node to an adjacent node. Each link is characterized by a length, cross-sectional area, and velocity (flow). The DYNTRAN model combines a hydrodynamic model and a transport model. The former solves the one-dimensional momentum equation along the links and applies the continuity equation at the nodes, thereby determining the velocity (flow) in the links, and the elevation at the nodes. The latter solves for mass transport of salt and a nonconservative constituent. DYNTRAN simulates hydrodynamics under the action of tides, freshwater flows, winds, and density gradients. It is simpler than the two- and three-dimensional long-wave models that will be described in the sections that follow. The model may be applied to a project either at the feasibility or design stage. In setting up the link-node system for a particular project, bathymetry, coastline, tidal and other boundary locations, desired degree of resolution, and locations of field gauges should be carefully considered. Because the model assumes the flow in a link is along the direction of the link, links should be oriented to represent known or logically expected flow directions. As a result, the model is well-suited to applications where the flows are well-channeled. It is less applicable to projects where the flows are two-dimensional and the flow directions are not known.

(2) Model requirements. Required model input consists of accurate bathymetric data, information to characterize the links and nodes (Figure 7-2), tidal elevations, freshwater flows, salinity and constituent mass flows as functions of time at the model boundaries, information on wind speed and direction, and initial concentrations. Field measurements of surface elevation, velocity, salinity, and concentration in the interior of the system are required for calibration and verification of the model. As a part of its output, the model “echo prints” the input.

(3) Example applications. DYNTRAN has been applied in several studies for the U.S. Navy (GKY and Associates 1988a, 1988b) to furnish the hydrodynamics to drive a water quality model to determine the fate and transport of organotin from Navy ships in Navy harbors. These harbors include Charleston, Mayport, Pearl Harbor, Everett, and Bremerton. The model was applied in a reconnaissance study performed for the U.S. Army Engineer District, New York. The objective of the study was to investigate the impact of a potential storm surge barrier on the Hackensack River, New Jersey, just upstream of its confluence with the Passaic River, on flood control in the two river basins. The model has been applied by WES to Bolsa Chica Bay, California, (Hales et al. 1989) to study the impacts of opening a new entrance and flooding wetlands on hydrodynamics and water quality. An improved fully two-dimensional version of this model has recently been used in storm surge analysis of the Passaic River Flood Protection Project (Demirbilek and Walton 1992).

(4) Bibliography. For additional information, refer to Moore and Walton (1984), GKY and Associates (1988a, 1988b), and Demirbilek and Walton (1992).

d. Two-dimensional vertically averaged models. These models neglect vertical accelerations and velocities and integrate the 3-D equations of motion in the vertical

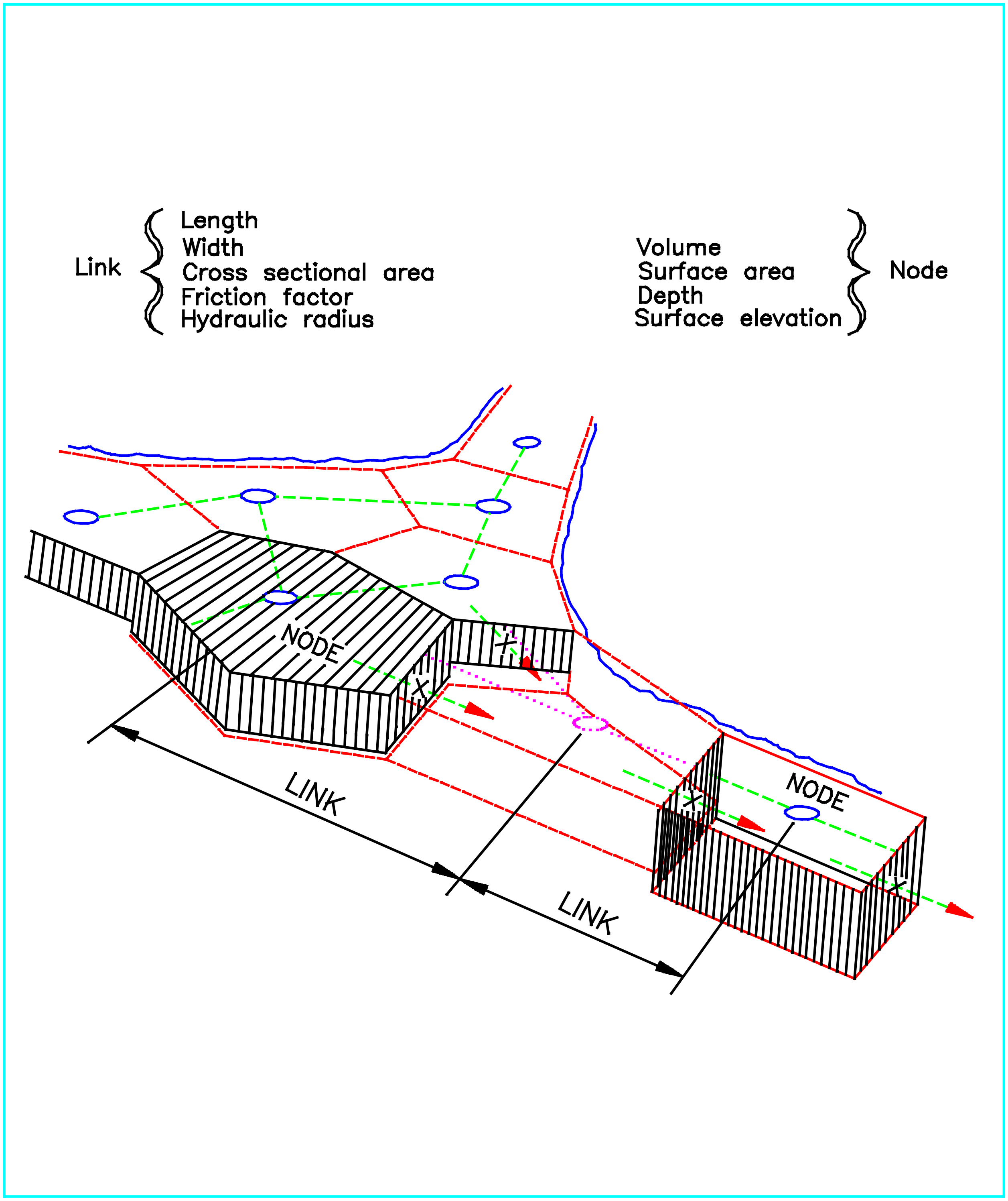

Figure 7-2. Pseudo-two-dimensional geometric representation for inlet systems

direction. They predict surface elevations and vertically averaged velocities. An example of this type is the WES Implicit Flooding Model (WIFM).

(1) Description. The WIFM is a finite difference model that uses an alternating-direction-implicit (ADI) solution scheme. It has been applied successfully to a variety of Corps shallow-water wave studies involving response to tides, winds, storm surge, and tsunamis. The model can be applied on a variable rectilinear grid, and accounts for advection and Coriolis terms. It can simulate flooding and drying of low-lying areas, and represent the effects of sub-grid scale barriers such as jetties and breakwaters. The WIFM has a “hot-start” feature, which enables simulations to be continued from results of a previous computer run. The WIFM can be accessed through the Coastal Modeling System (CMS) (Cialone et al. 1991).

(2) Model requirements. Essential input required to run WIFM consists of the following: bathymetric and geometric information for the ocean-inlet-bay system, numerical grid characteristics, information on structures (length, height, overtopping or not, discharge coefficients), friction factors for the region, time histories of surface elevations and discharges at the grid boundaries, and time history of winds (spatial and time variation of speed and direction). In the case of tides, tidal constituent information can be used instead of measured tidal levels at the boundaries. To calibrate and verify the model, measured velocities and elevations at select locations in the interior of the grid are required. Typical output consists of snapshots of elevations and velocities over the whole grid at selected instants of time during the simulation, time histories of velocity and elevation at selected cell locations ("gauges") throughout the simulation, and variation with time of discharge across selected ranges (flow openings). This information can be used during post-processing to obtain snapshot vector plots and time series plots.

(3) Example applications. The WIFM has been applied to the following inlets: Oregon Inlet, North Carolina, to study the effect of jetty spacing and channel stabilization on tides and storm surge; St. Marys Inlet, Florida, to determine the effects of channel modification and jetty sealing on tidal circulation near a jettied inlet; Yaquina Bay, Oregon, to determine tidal currents near the jetties and provide guidance for jetty rehabilitation; and Los Angeles-Long Beach Harbors, California, to determine the effects of proposed plans on tidal circulation and flushing.

(4) Bibliography. Additional information is provided in Butler (1978a,b,c; 1980); Leenknecht, Earickson, and Butler (1984); Seabergh (1985); Cialone (1986); Vemulakonda et al. (1988); and Cialone et al. (1991). WIFM is available to Corps personnel via the CMS on the CRAY Y-MP supercomputer that resides at the Information Technology Laboratory (ITL), U.S. Army Engineer Waterways Experiment Station (WES) (Cialone et al. 1991).

e. Three-dimensional models. These models solve the full three-dimensional equations of motion to obtain surface elevation and the three components of velocity, as well as the vertical variation of velocity. An example of this type is the CH3D model.

(1) Description. The CH3D determines the response of coastal currents and surface elevation to the action of tides, wind, and density gradients. The model includes Coriolis effects, advection, and horizontal and vertical turbulent mixing. In addition to hydrodynamics, the model permits computation of temperature and salinity. A second-order closure model is available to characterize turbulent transport. The CH3D is a finite difference model and permits a variable rectilinear grid in the horizontal, just like WIFM. In the vertical, a sigma stretching transformation, which allows the same number of grid layers in the shallow and deep waters, is used. As a result, the bottom is represented smoothly and the order of vertical resolution is kept constant throughout the grid. The model uses an efficient mode-splitting technique of solution. The model also permits use of a nonorthogonal boundary-fitted coordinate grid in the horizontal and/or layered (z-plane) vertical treatment. In this procedure, the external mode, which is represented by the vertically integrated equations of motion, is first solved by an alternating direction, implicit method, similar to WIFM. As a result, values of the free surface elevation and vertically averaged velocities throughout the grid are known. The model then solves the internal mode, which represents the deviation of the three-dimensional velocity field from the external mode. When the results for the two modes are combined, the full 3-D solution is obtained.

(2) Model requirements. In addition to the type of input required for WIFM, the model input consists of choices for computing bottom friction, advection terms, and lateral turbulence. Because the model solves the equations of motion in a dimensionless form, certain reference values needed to make variables dimensionless must be furnished. In addition, information on initial

values of temperature and salinity throughout the grid as well as boundary information throughout the simulation period must be provided, if these variables are to be modeled. Boundary information is needed not only at the lateral and bottom boundaries but also at the free surface. Model output consists of printouts at selected times of surface elevation, velocity fluxes in x and y directions, the three velocity components, temperature and salinity at different vertical levels over the whole grid, as well as time series of the same variables at selected gauge locations. Tide-induced residual currents also can be computed. Instead of numerical values being printed, printer plots of contours of the variables at selected instants of time over the whole grid may be obtained. Results may be stored in computer files and used for post-processing, such as for vector and time series plots and plots of velocity fields within a vertical transect across entrances.

(3) Example applications. The CH3D has been applied to determine tide and wind-driven circulation over Mississippi Sound (Sheng 1983) and, in a modified form, to compute tidal and wind-induced circulation over Los Angeles-Long Beach Harbors under existing and planned conditions (Coastal Engineering Research Center 1990; Vemulakonda, Chou, and Hall 1991).

(4) Bibliography. Additional information can be found in Sheng (1983, 1984) and Johnson et al. (1991a, 1991b).

7-4. Short-Period Wave Models

a. RCPWAVE. The Regional Coastal Process Wave (RCPWAVE) model is a monochromatic short-period wave model that employs a significant wave approach and linear wave theory.

(1) Description. The RCPWAVE (Ebersole, Cialone, and Prater 1986) is a finite difference model and allows the use of a variable rectilinear grid. It takes wave conditions in deeper water (typically 18.3-m (60-ft) depth or so) where the bottom contours are reasonably shore-parallel and where the waves have been subject only to shoaling and refraction, and propagates the waves towards the shore where most of the engineering applications are. It is assumed that Snell's law is valid from the offshore boundary of the model grid to deep water. The model computes the effects of refraction and depth diffraction. Structure diffraction may be taken into account approximately by a separate program which employs the Penney and Price (1952) solution near structures. The RCPWAVE solves a form of the "mild slope equation." It assumes that bottom slopes are small, wave reflections are negligible, and energy losses outside the surf zone are negligible. Wave breaking and subsequent wave transformation in the surf zone are modeled using the empirical method of Dally, Dean, and Dalrymple (1984). The RCPWAVE is accessible to Corps personnel via the CMS (Cialone et al. 1991).

(2) Model requirements. In addition to information on the grid characteristics and bathymetry for the region, RCPWAVE requires wave characteristics in deep water (wave height, direction, and period). These may be obtained either from WIS or field data. The model computes wave conditions at the offshore boundary from this information. Model output consists of wave height, direction, and wave number at the centers of grid cells. Also available is information on breaker location.

(3) Example applications. The RCPWAVE has been applied to numerous Corps projects by CERC and Corps Districts. These include Oregon Inlet, North Carolina; St. Marys Inlet, Florida; and Yaquina Bay, Oregon. Model simulations have been used for a variety of purposes, including design of structures, determination of wave-induced currents, and sediment transport calculations.

(4) Bibliography. For additional information, refer to Ebersole (1985), Cialone (1986); Ebersole, Cialone, and Prater (1986); Vemulakonda et al. (1988); and Cialone et al. (1991). RCPWAVE is available to Corps personnel via the CMS (Cialone et al. 1991).

b. HARBD. This model determines oscillations in harbors and water wave scattering in a region consisting of arbitrary boundaries, having variable bathymetry, and forced by ocean waves at an arbitrary depth (shallow, intermediate, or deep) (Chen and Houston 1987).

(1) Description. HARBD is a finite element model applicable to linear water waves. It permits fixed floating platforms in the region considered. The model takes into account bottom friction and boundary absorption (energy loss due to wave reflection). HARBD uses a hybrid finite element method based on a variational principle for numerical solution. The model does not account for wave breaking and transformation in the surf zone and in its present form does not predict wave direction.

(2) Model requirements. In addition to information on the finite element grid (such as identification of nodes, their coordinates, and elements and their correspondence with nodes) and bathymetry, input required consists of

friction coefficients of elements, reflection coefficients of boundary elements, and angle and period of incident waves. Because the models are linear, the input (forcing) wave amplitude is assumed to be unity for convenience. Model output consists of the absolute value and phase of the normalized nodal potential at the nodes of the grid. The absolute value represents the wave amplification factor (ratio of local wave height at that particular node to the incident wave) so the local wave height can be computed. Model output averaged over a basin (a basin is defined as an area consisting of one or more elements) can be printed for several basins specified in the input. For additional information on input and output, refer to Chen and Houston (1987).

(3) Example applications. The HARBD model has been applied to harbor response studies for Fisherman's Wharf, San Francisco, California; Indiana Harbor, Indiana; Green Harbor, Massachusetts; and Agat, Guam.

(4) Bibliography. Additional information may be found in Bottin, Sargent, and Mize (1985); Weishar (1988); Chen and Houston (1987); Clausner and Abel (1987); and Crawford and Chen (1988). HARBD is available within the CMS (Cialone et al. 1991).

c. REF/DIF. The numerical model REF/DIF is a monochromatic combined refraction/diffraction model that can account for wave-current interactions. The program calculates the forward scattered wave field in regions with slowly varying depth and current, including the effects of refraction and diffraction.

(1) Description. REF/DIF is based on Booij's (1981) parabolic approximation for Berkoff's (1972) mild slope equation, where reflected waves are neglected. The model is valid for waves propagating within 60 deg of the input direction. REF/DIF is based on Stokes perturbation expansion. In order to have a model that is valid in shallow water outside the Stokes range of validity, a dispersion relationship which accounts for the nonlinear effects of amplitude (Hedges 1976) is provided. The model may be operated in three different modes (1) linear, (2) Stokes-to-Hedges nonlinear model, and (3) Stokes weakly nonlinear. Wave breaking is based on Kirby and Dalrymple's (1986) dissipation scheme, which is initiated when the wave breaker index is exceeded. Land boundaries such as coastlines and islands are modeled using the thin film approximation where the surface piercing feature is replaced by shoals with very shallow depth.

(2) Model requirements. REF/DIF requires a depth grid representing the region of interest, as well as information about the wave (height, period, and direction), and water level (surge, tide) time history at the offshore boundary of the model grid.

(3) Example applications. REF/DIF has been applied to Indian River Inlet, Delaware, and Kings Bay, Georgia.

(4) Bibliography. Kirby and Dalrymple (1986) and Dalrymple, Kirby, and Hwang (1984) describe REF/DIF and its application.

7-5. Wave-Induced Current Models

a. WICM. The Wave-Induced Current Model (WICM) is a two-dimensional, depth-averaged model for computing wave-induced circulation and water surface setup. The following summary was abbreviated from Chapter 14 of the CMS Manual (Cialone et al. 1991).

(1) Description. WICM solves finite difference approximations of the Navier-Stokes (continuity and horizontal momentum) equations for the water surface displacement and the unit flow rate components. Because WICM is two-dimensional, velocities are treated as depth-averaged quantities (i.e., velocities are constant in magnitude and direction over depth). WICM can simulate flow fields induced by wave fields, wind fields, river inflows/outflows, and tidal forcing. This finite difference model is developed in boundary-fitted (curvilinear) coordinates.

(2) Model requirements. The types of data processed by WICM are extensive and encompass a wide range of possible applications. Since each application is unique, the type of input data required for each study will vary. In general, there are seven categories of data requirements: model control specifications (e.g., run title, units); grid description (rectilinear or curvilinear cells); physical characteristics (topography/bathymetry, bottom friction coefficients, and barriers influencing tidal circulation and storm surge levels); boundary conditions (tidal elevation, discharge, and uniform flux condition); wind field specification (steady or nonsteady); wave field specification (steady, nonuniform, monochromatic, or spectral); and output specifications.

(3) Example applications. Three illustrative examples of WICM are presented in Chapter 14 of the CMS Manual (Cialone et al. 1991). The first simulates wave breaking on a plane beach, and the other two examples discuss application of WICM to Leadbetter Beach, CA.

(4) Bibliography. WICM is available via the CMS, and model documentation is provided in Chapter 14 of the CMS manual (Cialone et al. 1991).

7-6. Sediment Transport

Numerical models for coastal sediment transport may be classified into those for noncohesive sediment and those for cohesive sediments.

a. Noncohesive sediments. An example of a model for noncohesive sediment is the sediment transport model component of the Coastal and Inlet Processes (CIP) Modeling System used in some Corps studies (Vemulakonda et al. 1985, 1988). For convenience, it will be called CIPSED hereafter.

(1) Description. The CIPSED model solves for sediment transport on a variable rectilinear grid. It is a finite difference model, that computes sediment transport using the results of a wave model, tide model, and a wave-induced current model. In the past, results of RCPWAVE, WIFM, and CURRENT were used for input and CIPSED was used for long-term simulation ("average year"), excluding severe storms such as hurricanes. A time-marching approach was used. Since the details of sediment transport are not well-understood, the model takes an empirical approach. For computing sediment transport, the area of interest is divided into two regions -- the open coast region away from the inlet, and the region near the inlet. In the open coast region, for non-storm conditions, cross-shore transport due to factors other than wave-induced and tidal currents may be neglected in comparison to longshore transport. This region may be further divided into two zones, the area within the surf zone and the area outside the surf zone. Within the surf zone, wave breaking plays a dominant role. Therefore, the total longshore transport is computed from the CERC formula (*Shore Protection Manual* (SPM) 1984) and distributed across the surf zone, using a procedure suggested by Komar (1977). Beyond the surf zone, because waves are not breaking, it is the tractive force of currents that produces sediment transport. Therefore, in this zone, the method of Ackers and White (1973) is followed after appropriate modification for the presence of waves. Finally, in the region near the inlet, where tidal and wave-induced currents play a major role in transport, the modified Ackers and White method is used. Output from CIPSED consists of transport rates in the two coordinate directions at each grid cell. These are used with a continuity equation to determine the net erosion or deposition at each grid cell for the period of simulation. In general, the model is suitable for predicting sediment transport and inlet channel shoaling under long-term average wave conditions (excluding the effect of severe storms; e.g., hurricanes and northeasters) such as those given by WIS. It can predict areas of accretion and erosion in the region under consideration. Because of grid size limitations, the model cannot accurately resolve shoreline changes, which are on the order of a meter. It can qualitatively predict changes near barrier islands. It is advisable to calibrate and verify the model with field data for the project site before applying it to new project conditions.

(2) Model requirements. Apart from grid characteristics and bathymetry, model input consists of sediment diameter, density, porosity, Manning's roughness, time-step for running the model, and parameters to control the sequencing of waves, wave-induced currents and tide during time marching. Additional input consists of output files from runs of the wave, wave-induced current, and tide models consisting of wave height, direction, wave number, tidal elevation, setup, tidal and wave-induced velocity components at the centers of grid cells, and breaker location. Model output consists of sediment transport rates in two coordinate directions, and net erosion or deposition at the end of the simulation for each grid cell. Intermediate results and a mass conservation check are also printed at desired time intervals.

(3) Example applications. An earlier version of CIPSED was used for the Oregon Inlet, North Carolina, project (Vemulakonda et al. 1985) to evaluate erosion and accretion in the ocean bar entrance channel and the lateral movement of the channel when just the south jetty was in place. This single jetty case simulated a construction sequence in which the south jetty was built before the beginning of construction of the north jetty. The model was used for St. Marys Inlet (Kings Bay Study) (Vemulakonda et al. 1988) to study channel shoaling under existing and plan conditions, and recommend advance maintenance dredging for different reaches of the channel for plan conditions.

(4) Bibliography. For additional information, refer to Vemulakonda and Scheffner (1987) and Vemulakonda, Houston, and Swain (1989).

b. Models for cohesive sediments. An example of an algorithm that has been applied to predict cohesive sediment transport resuspension is documented by Cialone et al. (1991). Note that this module does not predict sediment transport rates or directions, only the potential for sediment to suspend in the water column.

(1) Description. For the Green Bay, Wisconsin, study, the potential for cohesive sediment resuspension in the Bay was evaluated using output from the three-dimensional velocity model, CH3D, to drive a cohesive sediment resuspension module. The module was developed for shallow water based on sediment resuspension being a function of the orbital velocities associated with short wave fields, together with the shear stress imparted by the depth-averaged flow (output from CH3D). To account for this coupled process, an effective increase in the bed shear stress was used in the algorithm. A relationship developed by Bijker (1967) was used, which states that an effective shear stress reflecting both waves and currents can be written as a function of a wave-induced increase in the bed shear stress produced by currents only.

(2) Module requirements. The cohesive sediment resuspension module requires output from CH3D (root-mean-square velocities at each grid cell), as well as wave orbital velocities. Significant wave heights and periods were estimated using fetch-limited shallow-water hindcasting procedures as discussed in the *Shore Protection Manual* (SPM 1984), using measured maximum sustained wind speeds.

(3) Example applications. To evaluate potential sediment resuspension patterns over a wide range of hydrodynamic conditions, the module was applied to ten scenarios at Green Bay, Wisconsin (Mark et al. 1993).

(4) Bibliography. For a discussion of the module's development and application to Green Bay, Wisconsin, see Mark et al. (1993).

7-7. Numerical Modeling Systems

Apart from individual numerical models, numerical modeling systems containing a suite of numerical models (for example, models for tides, waves, wave-induced currents, and sediment transport) may be employed for studies on coastal hydraulics and sedimentation. There are several advantages to such systems. For instance, all the models are of the same type (finite-difference or finite element), they use the same type of grid, information can be readily transferred from one model to another, and the individual component models of the system can be applied in different combinations, depending on the specific application. An example of a numerical modeling system for the inlet environment was the Coastal and Inlet Processes (CIP) System, applied to several Corps studies.

a. Description. The CIP System consisted of models for tide, wave, wave-induced current, and noncohesive sediment transport. All the models of the system were of the finite-difference type and were applied on a variable rectilinear grid.

b. Model requirements. Input requirements for the types of models within the CIP System have been described.

c. Example applications. The modeling system was originally developed for the Oregon Inlet project (Vemulakonda et al. 1985), and was also applied at Kings Bay (Vemulakonda et al. 1988) and Yaquina Bay (Cialone 1986).

d. Bibliography. Additional information may be found in Houston et al. (1986); Cialone and Simpson (1987); Vemulakonda and Scheffner (1987); and Vemulakonda, Houston, and Swain (1989).

7-8. Numerical Model Implementation

The following paragraphs deal with aspects of model implementation that must be considered when applying numerical models at inlets.

a. Grid characteristics. The grid typically should include the inlet, the barrier islands adjacent to the inlet, the back bay, and a portion of the ocean area in front of the inlet. The grid boundaries must be located sufficiently far away from the inlet so the boundary conditions are not affected by planned changes near the inlet. Boundary locations must be chosen carefully so the flow satisfies the boundary conditions. Depending on the process modeled, the grid cells must be sufficiently fine near the inlet, the navigation channels, the back bay, and the surf zone for proper representation. Grid cells can be coarse near the lateral and offshore boundaries. In tidal and wave-induced current models, the grid cell size, depth, and expected maximum local velocity dictate the maximum time-step that can be used for simulation. Computational time and storage typically depend on some power (greater than one) of the number of grid cells. Therefore, it is desirable to minimize the total number of cells, consistent with accuracy and resolution desired. This objective is achieved by using a variable rectilinear grid.

b. Grid generation.

(1) Finite difference. A variable or uniform rectilinear grid for the region of interest can be obtained, plotted,

and listed to file by using a special interactive program called CMSGRID, which is part of the CMS (Cialone et al. 1991). Output from the program consists of grid coefficients, for different regions of the grid, in the two coordinate directions. Other preprocessing programs of the CMS can use this information to determine coordinates of grid cells as well as plot the grid to any desired scale. The grid plotted to an appropriate scale is overlaid on a bathymetric chart and the depths for different grid cells are determined.

(2) Finite element. The finite element grid for the HARBD model is created manually by selecting the nodes and elements of the grid as desired. The nodes and elements are numbered in some convenient fashion and the correspondence between the nodes and elements is established. The grid is overlaid on a bathymetric chart and the coordinates of the nodes and depths are digitized. It may be necessary to modify the grid on the basis of preliminary testing.

(3) Boundary-fitted. Nonorthogonal curvilinear (boundary-fitted) grids can be made to conform to bathymetric features and provide an accurate means of representing a study area. These grids can be generated using a numerical grid generator such as program EAGLE (Thompson 1985), which has the flexibility to concentrate grid lines in shallow-deep areas or in areas where the bathymetric gradients are great.

c. Calibration/verification. Before most numerical models are applied to determine the impact of some new plan conditions, it is necessary to ensure that they reproduce the prototype behavior corresponding to some known conditions. This is the objective of the calibration/verification process. For this purpose, ideally, two complete and independent sets of prototype data are necessary. The data should include all the information necessary to run the model and check model results. Thus information on boundary conditions, forcing mechanisms, and measurements in the interior of the model grid are needed. During calibration, the model is run to correspond to the first set of conditions. Model parameters such as friction and eddy coefficients are varied until the model reproduces the prototype measurements in the interior satisfactorily. Next, the model is run in the verification mode, using the second set of conditions. During this phase, model parameters are not changed but kept at their values corresponding to calibration. Model results are compared to prototype measurements. There should be good agreement. If measured and predicted data significantly differ, the model should be re-calibrated and verified with the new calibration parameters. In practice, prototype data of the quality needed for calibration and verification are not available unless they are collected as a part of the numerical modeling project. There may be only one data set, in which case calibration/verification is done as a one-step process. Another problem encountered is that the prototype data may not be complete and accurate. In such situations, the modeler looks for qualitative agreement between model and prototype in terms of overall behavior patterns and for reasonable explanations as to why the two might differ. Once calibration and verification are successful, the model is ready for application to plan conditions. Finally, it should be noted that model calibration and verification are essential for models of tidal hydrodynamics and sediment transport. For models of waves and wave-induced currents, model calibration and verification are desirable but not essential, because understanding of the hydrodynamics of the latter phenomenon is more complete and the models have fewer site-dependent calibration parameters.

7-9. Design Use of Model Results

Typically, models may be employed to improve our understanding of various phenomena at prototype locations and to furnish explanations for observed behavior or failure. They are often used to compare existing (base) conditions to future plan conditions and thereby predict the impact of plans on hydrodynamics (velocities, discharges, water levels), water quality, and sediment transport at key locations. By testing alternate plans in the numerical models, it is possible to assess the advantages and disadvantages of each and choose from among the alternatives the best for project implementation. It may be necessary to ensure that for the design selected, velocities in the interior are adequate for mixing and flushing, velocities and waves near the navigation channel do not adversely impact navigation, and velocities near structures are sufficiently low to prevent scour. Examples of such projects are navigation channel modifications and jetty construction for channel stabilization. Often, the designer is faced with conflicting requirements. For example, by increasing jetty spacing, velocities in the navigation channel may be reduced, thereby improving navigation but worsening channel shoaling, and vice versa. Model results enable the designer to strike a balance. In light of model testing, improved designs and modifications to original designs can result. One area where significant cost reductions may be possible is in estimating maintenance dredging required after channel modifications. Using model predictions of advance maintenance dredging required for different reaches of channel, it is possible to modify the channel design and reduce the overall dredging required. Even though they are approximate, numerical models are the only tools available to predict sediment transport quantitatively in such cases.

Chapter 8
Guidelines for Planning Tidal Inlet Monitoring

8-1. Introduction

a. The purpose of this chapter is to provide guidance to the field engineer in planning prototype monitoring of physical processes at tidal inlets. Processes in and near tidal inlets may be monitored to evaluate the feasibility of proposed inlet modifications; to investigate the impacts of existing modifications, such as a deepened or stabilized channel; to ascertain inlet safety issues relating to navigation and pollution control; and/or to provide information for subsequent numerical or physical modeling of the inlet. Tidal inlets are dynamic coastal features that migrate, shoal, and change their shape in response to various physical processes. These processes can be studied through physical or numerical modeling, or prototype (field) measurements. This chapter provides guidelines for planning and conducting prototype measurements at field sites. These guidelines are intended to be generally applicable throughout the United States, but must be adapted for project requirements.

b. Many monitoring programs have been implemented by the USACE at inlets in the United States. Under the USACE Monitoring Completed Coastal Projects Program (MCCP), the following inlets have been monitored for various time periods since inception of the program in 1981: East Pass, Florida (Morang 1992); Yaquina Bay, Oregon (in planning stages); Siuslaw, Oregon; Colorado River, Texas (White 1994); Carolina Beach Inlet, North Carolina (Jarrett and Hemsley 1988); Ocean City Inlet, Maryland; and Manasquan Inlet, North Carolina. Other inlets monitored by the Corps include: Indian River Inlet, Delaware (Anders, Lillycrop, and Gebert 1990); Oregon Inlet, North Carolina (monitoring under way); Port Everglades, Florida (Rosati and Denes 1990); Panama City, Florida (Lillycrop, Rosati, and McGehee 1989); Murrells Inlet, South Carolina (Douglass 1987); and Little River Inlet, South Carolina (Chasten and Seabergh 1992). For a detailed discussion of the specific monitoring programs at each of these inlets, the reader is directed to the references cited. This chapter illustrates various levels of a monitoring program through discussions of a few of these monitoring projects.

c. Instructions for using and analyzing data from individual instruments is beyond the scope of this chapter. This chapter is directed towards broad guidance on planning a monitoring project, listing the types of equipment available, and describing data that can be collected at an inlet. Some types of measurements may require contracting to outside organizations, as the facilities, equipment, and expertise may not be available in-house.

8-2. Overview

a. Ideally, an inlet monitoring project is divided into three phases: (1) reconnaissance; (2) preliminary measurements; and (3) detailed field study. Each phase level includes the process critical to a successful monitoring program: proper planning. During the planning process, data needs, measurement devices, and data analysis tools are identified to ensure that the type, duration, and frequency of required information will be obtained. Analysis of observations and data should be conducted during and/or after each phase.

b. Various types of data collection methods and instrumentation can be used to obtain field data at inlets, as indicated in Table 8-1. The methods listed are popular, proven methods for field use; other measurement techniques and instrumentation are available for other applications. Each measurement method/device has inherent limitations, e.g., required deployment location, length of deployment, frequency of data sampling, how the data are stored and retrieved, environmental conditions under which the technique/instrument will properly perform, theoretical assumptions in analysis of raw data, etc. Choosing the method/device to measure a particular type of data will depend on its limitations, availability, and cost, perhaps requiring outside expertise.

c. Monitoring programs are usually initiated to (1) evaluate pre-construction site processes so that a project can be properly designed with analytical, physical, or numerical models; (2) evaluate post-construction success of an inlet modification; or (3) assess existing conditions and trouble-shoot processes that may be causing a particular problem. Every field study requires an initial reconnaissance, primarily to assess what is known about the site. The reconnaissance phase may simply be a site visit supplemented with information gathered through a literature search, or may extend to preliminary field observations with low-cost measurement techniques. The reconnaissance can suggest a hypothesis to test and a scheme for field data collection. Based on the reconnaissance objective, the project engineer can then decide if a preliminary, relatively inexpensive study is to be conducted or if a more thorough, detailed study is in order. Sometimes, the preliminary field study identifies additional conditions that need to be monitored in detail, ultimately resulting in a study that has included all three phases.

Table 8-1
Data Needs and Associated Instrumentation for Inlet Monitoring Projects

Data Desired	Data Collection Method/ Instrumentation Types
Circulation patterns	Surface and/or subsurface drogues, dye
Current speed and direction	Acoustic Doppler Current Profiler (ADCP) Electromagnetic Current Meter (EMCM) Ducted impeller (self-aligning)
Wave height, period, and direction	Pressure sensor (nondirectional) Pressure sensor with EMCM Pressure sensor array Accelerometer-based buoy (deep water)
Wind speed and direction	Vane-mounted anemometer Propeller-driven anemometer Cup-type anemometer
Water/tide level	Absolute water pressure sensor and barometer Tide gauge (stilling well and acoustic level detector)
Suspended sediment concentration/rate	Sediment traps Optical Backscatter Sensors (OBS) Fluid/sediment sample jars
Total sediment transport rate	Sediment traps Sediment tracer
Bathymetry	Rod and level Boat with fathometer Sled SHOALS (Scanning Hydro-graphic Operational Airborne Laser System) SEABAT (multi-beam acoustic sounding system)
Topography	Rod and level GPS-tracked (Global Positioning System) vehicle Stereoscopic aerial photography
Information about bed forms or structure conditions	Diver inspection Aerial photography (with clear water) Side-scan sonar SEABAT (see above)
Surface/subsurface exploration	Sediment grab samples Sediment cores Subbottom profiler

d. Occasionally, unexpected field conditions (due to weather patterns, wave climate, navigational traffic, dredging activities, etc.) may create a “target of opportunity” to gain some insightful information about inlet processes. Keeping in mind the intent of the monitoring program and required data, the field work plan should accommodate flexibility so that unique field conditions can be captured.

e. The purpose of a monitoring program may be to provide information for numerical model calibration and verification, and/or to provide input data for physical models. In these cases, specific types of data and sampling site locations may be called for in the numerical/ physical model, and should be addressed in the monitoring program.

8-3. Phase I: Reconnaissance

The reconnaissance phase of the study is a vital prelude to the later field measurements. Much of it can be conducted at the home office, although at least one field visit by the project engineer is required.

a. Planning. Prior to visiting the project site, the project engineer should be familiar with the site as it is discussed in the literature, including previous studies concerned with the site (laboratory, numerical, and field) which may give insight into inlet processes. Measurement techniques which have been successful at locations with similar processes and/or navigation traffic should be considered for the project site monitoring. A preliminary monitoring plan should be developed, including data requirements, types of instrumentation, time scale for the measurements, and proposed sampling locations. The National Oceanic and Atmospheric Administration (NOAA) tidal current and height tables published by NOS present predicted current and tidal height information for inlets along the Atlantic, gulf, and Pacific coasts. These tables are published approximately 6 months prior to the referenced year; therefore, the project engineer can plan a reconnaissance visit to coincide with predicted conditions of interest (i.e., peak currents, spring tide, etc.).

b. Literature search. An extensive literature base exists concerning tidal inlets, and technical information may be available for a study area. If documentation of processes or previous studies at the project site is scarce, reports discussing inlets with similar histories and processes may be useful. Sources for such information include:

(1) Reports prepared by U.S. Government agencies, such as the USGS and the USACE. The GITI program conducted by USACE produced many site-specific and comprehensive reports about inlets. USACE District and Division offices may have reconnaissance and feasibility studies for the inlet of interest.

(2) Congressional documents.

(3) Reports by academia, such as those in the libraries of Louisiana State University's Coastal Studies Institute, and the University of Florida's Coastal and Ocean Engineering Laboratory.

(4) Conference proceedings often have several case studies describing inlet research, including discussions of processes at inlets, monitoring programs, and applications to numerical and/or physical modeling.

(5) Scientific journals such as the *Journal of Sedimentary Petrology, Journal of Geology, Marine Geology, Journal of Waterways, Ports, Coastal, and Ocean Engineering*, and *Journal of Coastal Research*. Scientific publishers such as Elsevier or the Society of Sedimentary Geologists (SEPM) have printed excellent books containing papers which describe the results of coastal research and engineering (examples include Elsevier's Lecture Notes on Coastal Engineering, SEPM Special Publications, and some of the Geological Society of America Memoirs).

c. Data search. Data (current, wave, and water level measurements, core logs, bathymetric, topographic, subbottom/seismic data, surface sediment samples, tidal/river stage data, aerial photographs, and/or dredging records) may be available from previous field studies. Sources for such data include:

(1) District offices of USACE. Historic maps, hydrographic surveys, and topographic sheets may be available.

(2) Other Federal agencies. The NOAA archives tide data, and limited hydrographic surveys dating back to the 1800s. The National Climatic Data Center has weather data from around the country. Offshore wave data may be available from the U.S. Navy for certain areas. The USGS produces topographic sheets for the United States.

(3) State agencies. Departments of natural resources and environmental regulation often have sediment samples, beach profiles, coring records, and geophysical data.

(4) Universities. Schools with oceanography, geology, or coastal engineering departments may have inlet process data.

(5) City Governments. Cities with active engineering departments.

d. Field visit.

(1) A site visit allows the project engineer to observe inlet processes, process interaction with structures, and inlet effects on adjacent beaches. The preliminary monitoring plan developed in planning stage (*a*) can be evaluated for its feasibility, and revised if necessary. Observations of dye movement through the inlet/structures, measurements of currents with hand-held current meters, and Littoral Environment Observations (LEO) (Schneider 1981) of wave conditions are simple, inexpensive methods for quantifying site processes. Discussions with local citizens, harbor masters, and city/county engineers can provide useful information about inlet conditions during normal and storm conditions, navigation/recreational hazards to instrument deployment, and public perception of inlet effects.

(2) It is usually cost-effective for the project engineer to charter an airplane to fly over the site. This overview helps fix the inlet within the broader geologic and geographic framework. Features which may be obscure from the water surface or the ground may be clear from the air, e.g., sediment plumes within the inlet or sea, bed forms, ebb/flood tidal shoals, beach ridges, and ponds which may mark former inlets. An airplane with the wing over the cabin and windows that open is recommended for the best quality photographs/video. An altitude between 300 and 600 m (1,000 and 2,000 ft) is ideal, although in some areas aircraft are not permitted to fly this low. A helicopter, which can hover over a site, can be an attractive alternative to a plane. However, vibration from the helicopter may degrade photographs/video, and the rental cost for helicopters is an order of magnitude greater than that for airplanes.

e. Controlled aerial photographs. Sources for aerial photography include:

(1) USACE.

(2) U.S. Air Force.

(3) National Atmospheric and Space Administration (NASA).

(4) U.S. Department of Agriculture.

(5) State agencies, as discussed in *c*3 above.

(6) City engineering departments, as listed in *c*5 above.

(7) Private aerial photography companies in vicinity of the inlet.

It is worth obtaining as many historical photographs and/or hydrographic surveys as possible because they often reveal the natural behavior of an inlet and demonstrate how it migrated over time. Older photographs taken before structures affected natural processes in the vicinity of the inlet give great insight into structure impacts and natural inlet processes.

f. Example of a reconnaissance level study: Port Everglades, Florida.

(1) The purpose of the Port Everglades, Florida, monitoring program was to evaluate the effectiveness of a structure sealing project at the south jetty, a rubble stone structure with large "man-sized" voids (Figure 8-1). Beach fills placed south of the inlet eroded at an extremely high rate, indicating to county and state personnel that sediment moved through the south jetty into the navigation channel. The structure was sealed with sodium silicate-cement for void cavities and with sodium silicate-diacetin for sand-filled voids during the period September-November 1988. Four site visits were conducted as part of the monitoring program: (a) reconnaissance study, (b) preconstruction experiment, (c) during-construction inspection and observation, and (d) post-construction experiment. The Port Everglades study is described by Rosati and Denes (1990).

(2) The purpose of the reconnaissance study, conducted 27-29 June 1988, was to obtain detailed information about the south jetty infrastructure, current patterns, and surrounding beach and bathymetry conditions to plan later phases of the monitoring program. Using the NOAA tidal current tables, the trip was scheduled such that extreme conditions (peak flood and ebb currents) occurred during daylight hours, and could easily be evaluated. A literature review revealed that the county had conducted a dye study at the site in February 1985 by placing dye on one side of the jetty and making visual observations of dye movement through the structure as an indication of structure permeability. Permission to access the site and operate from a staging area was obtained prior to the reconnaissance study period. Proposed plans for assessing pre- and post-construction structure permeability included: dye movement through/around structure; current speed and direction through/around structure; and sediment transport through structure (using sediment traps and/or a bed-load sampler). The feasibility of making each type of measurement during the pre- and post-construction experiments was evaluated during the reconnaissance study.

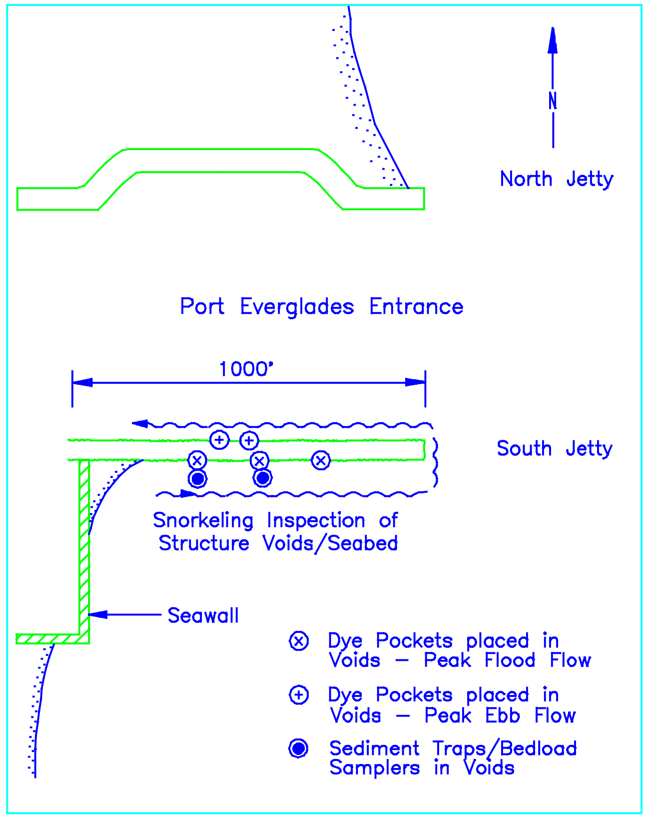

Figure 8-1. Data collected during the Port Everglades, Florida, reconnaissance field study

(3) A snorkeling inspection of structure voids, recording their location and dimensions, was initially conducted. Several structure voids that extended deep into the structure were identified and photographed for possible placement of current meters and sediment traps during future experiments. Characteristics of the seabed were also noted during the snorkeling inspection. No shoals or large sediment deposits were noted along the structure, indicating that if sediment passed through the structure, it was carried away from the sides of the jetty.

(4) A hand-held current meter was brought to the site to evaluate currents at locations along the structure; however, the equipment failed and a replacement current meter could not be obtained in a timely manner.

(5) Dye placement using a pressure sprayer did not provide the continuous, concentrated quantity of dye required. Instead, powdered dye placed in sediment sample bags weighted with rocks and placed in structure voids provided an observable dye pattern. Observations of dye dispersal were made over the experiment period

for three peak flow conditions, both from the ground and from a rented airplane.

(6) A sediment trap and bed-load sampler were placed in structure voids over several hours, and removed to measure the accumulated sediment. Both types of sediment measurements collected very little sediment. It was decided that the sediment traps and bed-load samplers would not be used to measure sediment transport through the structure in later phases of the monitoring program.

(7) It was concluded that dye dispersal through the structure provided the best measure of structure permeability. A fluorometer, an instrument that quantifies fluid florescence, was used in later phases of the monitoring program to determine the rate of dye dispersal through the structure. Using dimensions of the structure voids, three current meter mounts were designed, and the mounts were used to position 2.5-cm (1-in.) electromagnetic current meters in voids for pre- and post-construction experiments. These two types of measurements were used to quantify pre- and post-construction structure permeability at the Port Everglades south jetty.

8-4. Phase II: Preliminary Measurements

a. General. This phase of an inlet study is intended to either answer a specific question with a limited amount of field data or provide general information which can identify problem areas and be used to plan a more detailed field survey. For projects with limited scope or funding, this effort may be the only field study performed. In some cases, the collection can be designed to compliment similar data being obtained in the vicinity of the inlet by other agencies. An example is the measurement of water levels. NOAA might have a tide gauge within an inlet or harbor. In this case, a single additional tide gauge could be deployed along the open coastline so that the tidal phase difference between the bay and sea can be measured. Another example is the use of side-scan sonar to examine an inlet structure. Once the vessel and side-scan equipment have been mobilized, the equipment can be used to image bed forms within the inlet for a relatively small additional cost. Examples of the types of data that might be collected in a preliminary site survey include:

b. Controlled aerial photographs.

(1) Aerial photographs taken under controlled conditions can be used for mapping, identifying landforms, and sometimes identifying relic channels. If inlet features change shape significantly during the year, a winter flight and a summer flight are recommended. If other aerial photographs already exist for the study area, it is recommended that the new photographs be taken at the same altitude and with the same lens focal length to produce images that are the same scale as the original photos. Otherwise, two scale factors are recommended, 1:24000 to provide broad coverage of the study area, and 1:4800 to produce detailed images. If the water is clear, the 1:4800 photographs will have enough resolution to show inlet bed form features.

(2) Daylight quality should be considered when planning aerial photography. If seafloor features are of primary interest, then the photographs should be taken at midday when the sun is high and has greatest penetration through the water. If land features are of primary interest, then low-angle sunlight is preferred because long shadows help reveal features.

(3) Tidal stage is also an important consideration. At most inlets, the flood tide carries clear water into the inlet, which may facilitate photographing bed forms. Photographs during ebb flow water may be undesirable due to turbid river inflow or sediment suspension from a back bay area. Another (possibly conflicting) consideration is adjacent beach shoreline position as it varies with tidal stage. It is convenient to take aerial photographs at a known phase of the tide, i.e., mean low water (mlw), mean high water (mhw), etc., which facilitates comparison with beach surveys and/or previous aerial photographs.

c. Beach profile/inlet shoreline surveys. Beach profiles can be obtained with simple equipment (rod and level) at low cost. Many sites have previously surveyed reference locations; resurveying these locations allows direct comparison with earlier surveys. Sometimes the most difficult part of beach surveys is obtaining permission from local residents to use their property as a right of way to gain access to the beach.

d. Sediment sampling. Surface sediment samples can be collected by the field workers who perform the profile surveys. Ideally, samples should be taken within the inlet, from adjacent beaches, and from the bay behind the barrier beaches. These sampling locations can help identify the source of the sediment and suggest whether there is a net amount of sediment entering the bay or flushing out to sea. The samples should be taken from various parts of the beach profile since grain size can vary significantly across the beach. In addition, it is important that sample locations be recorded since it may be necessary to resample the same locations in the future.

e. Currents. The speed of water flowing through an inlet is basic information which is often unavailable. Measurements can be made using either in situ current meters (discussed in the next section) or surface or subsurface drifters. Drifters can be prepared inexpensively and provide basic information about current speed. The easiest method is to position painted blocks of wood or oranges/grapefruit in the inlet and time how long they take to travel a known distance.

f. Water flow patterns.

(1) Drifting floats and dye can be used to show how water flows through the complex inlet system. Drifters can provide only limited quantitative information about the volume of water in the system, but can demonstrate overall patterns such as whether certain channels are primarily ebb or flood dominated, if gyres occur around structures, and how different bodies of water interact. A drifter or dye study can be performed as part of the reconnaissance phase of monitoring or can be done in conjunction with more detailed current meter measurements in the Detailed Field Study phase. Dye is useful in indicating the relative permeability of structures during various phases of tidal flow. An experiment with drifters or dye can be performed relatively easily since material costs are modest and observations can be made from ground or a rented airplane. The main disadvantage of these inexpensive devices is that they must be used in relatively good weather so that they can be accurately tracked. Drifters with radar reflectors are available and are an alternative to consider if the weather is often poor at the study site, but the complexity of the radar and navigation equipment adds significantly to the cost.

(2) Drifters used on the water surface can simply be plywood shapes painted with fluorescent paint and numbered for identification. To trace the flow of water below the surface, a drifter can be made with vanes suspended below the surface float at the desired depth. These drifters can be difficult to use because the vanes can get caught on underwater obstructions or a shallow bottom. The surface float also produces some drag, so the resultant velocity vector may not accurately describe either the surface or subsurface speed and direction.

(3) Dye can be injected from a fixed point over a period of time, producing streak lines that can reveal areas of turbulence or mixing. Wright, Sonu, and Kielhorn (1972) used dye at East Pass, Florida, to demonstrate how sea water entering the inlet with the flood tide was subducted underneath a plume of fresh water flowing south out of the Bay. Rosati and Denes (1990) used dye at Port Everglades, Florida, to evaluate the permeability of the inlet jetty before and after structure sealing.

(4) Dye is available as a powder in bulk form, in pre-formed blocks or rings, and as a concentrated liquid. Dye rings are the most convenient to use, but tend to dissolve slowly. Powder and liquid are quickly dispersed, but can be messy to use. Two commonly used dyes are rhodamine, which is pink/red, and uranine, which is fluorescent green. Food colorings are available that have been tested for purity, and may be preferred for environmental considerations. Material Data Safety Sheets are available from the manufacturer for these food colorings, certifying that they are nontoxic. In areas where local residents are especially sensitive about environmental pollution, food colorings are recommended.

(5) In turbid conditions, dye is only visible at the surface. Formulabs, Inc. recommends that yellow/green dye be used in water bearing heavy sediment loads because red will be partially obscured by suspended clay particles. For turbid seawater conditions, it is advisable to use concentrated dye that has been mixed with fresh water, since this solution will float. If the water is clear, it may be best to mix powder with sea water at the site since this mixture will tend to remain at the depth of its injection. In inlets with rapid flow, dye may disperse too quickly to be visible. Before finalizing a monitoring program at the inlet, the feasibility of using dye and drifters at the site should be evaluated with testing.

g. Tide measurements. If there is a harbor near the project site, a tide gauge may already be located there. To determine the phase difference between tidal stage in the harbor and along the open coastline, another tide gauge will have to be installed, preferably near the inlet's mouth. Thus, short-term deployment of an inlet mouth gauge and comparison of these measurements with a harbor gauge will facilitate proper conversion of the long-term harbor tidal record.

h. Side-scan sonar.

(1) Side-scan sonar uses phased transducer arrays mounted on a towfish to emit acoustic pulses in narrow beams to each side. Timing of the return echoes permits computation of the slant range, perpendicular to the direction of towfish travel, to targets in the plane of the beam. Repeatedly pulsing the signal as the towfish is pulled forward generates a picture of the seafloor as a series of scan lines on a moving chart recorder. Stronger returns show darker images, and a lack of a signal appears white. The result is an acoustic image of the bottom as seen

from the position of the towfish (Clausner and Pope 1988). Side-scan sonar is a versatile tool that can be used to assess the condition of breakwaters or other structures and can image bed forms and other bottom features in inlets and channels (Lillycrop, Rosati, and McGehee 1989).

(2) The advantage of side-scan sonar is that it can be operated in turbid water, where aerial photography or diver inspection are ineffective. However, shallow water may limit its use. It usually is not effective in water depths less than about 3 m (10 ft), but if a shallow-draft vessel is used in calm seas, the side-scan towfish can be suspended just below the water surface. To reduce turbulence and optimize quality of the records, the side-scan surveys should be made at slack tide. Bubbles in the water column during ebb or flood tides may completely obscure the record. Turbulence caused by wave-current interaction and wave breaking near the mouth of an inlet may make this area difficult to image except on calm days.

(3) Because of the many difficulties in using side-scan sonar within an inlet, the likelihood of its success at the project site must be evaluated by comparing conditions under which it has been successful to the project inlet processes. The main cost of most side-scan projects is mobilizing the equipment, transportation of equipment and personnel to the project site, and leasing a vessel.

i. Example of a preliminary field study: Panama City, Florida.

(1) The Panama City, Florida, study was initiated in an effort to reduce dredging requirements in the inlet. Sand waves with heights as great as 15 ft in the entrance channel reduced the authorized channel depths, requiring frequent overdepth dredging. The purpose of the study was to evaluate potential changes to the inlet system that would reduce dredging requirements. Flow characteristics in the Panama City channel were such that sand waves formed; if these flow characteristics could be modified, the tendency for sand wave formation could be reduced. A limited amount of field data was obtained to (a) qualitatively monitor sand wave formation through time, (b) determine hydraulic characteristics of the inlet for numerical model calibration and verification, and (c) measure the velocity distribution associated with a fully developed bed form. A detailed discussion of the Panama City, Florida, project is given by Lillycrop, Rosati, and McGehee (1989).

(2) Bathymetric surveys were used to identify and locate individual bed forms within the inlet (Figure 8-2). Five parallel survey lines spaced approximately 30 m (100 ft) apart were used to monitor the sand waves, with surveys made in October and November 1986, and July 1987. Side-scan sonar was used to obtain a continuous picture of the bed features in November 1986.

(3) In April 1987, currents were measured with a hand-held ducted impeller meter (see short-term current measurements, next section) to determine the maximum tidal induced flow and the variation of near-bottom velocities near sand wave crests and troughs. In situ meters (see remote current meters, next section) with internal recording capability were deployed over several tidal cycles in July 1987. For ease of installation and protection from significant fishing boat traffic, gauges were mounted from taut moor buoys anchored to existing navigational buoy sinkers. Currents were recorded at 15- or 30-min intervals, depending on the gauge. Gulf and bay water level differences were measured by manual recording of tide levels on staffs placed at three locations during the same time period.

(4) Prototype data from Panama City were used to define existing conditions that create sand waves, and to obtain data to use in model calibration and verification.

8-5. Phase III: Detailed Field Study

The detailed field study is often complex and costly, and therefore must be carefully planned and coordinated, incorporating information gained from the earlier phases of the study. Examples of the types of data that may be collected during a detailed field study include:

a. Short-term current measurements.

(1) Short-term measurements of the current can be made intensively at several inlet cross sections over a tidal cycle. The measurements are typically made by field workers operating from small boats, although sometimes instruments can be deployed from a bridge if the span is not too high. Currents are usually hourly at three or four stations at each cross section, at depths of 0.8, 0.5, and 0.2 times the total water depth at each station. The resulting three-dimensional grid of current measurements gives an indication of current speed and direction, providing a detailed snapshot of water flow within the inlet. This information can help identify processes that may be contributing to problems in the inlet.

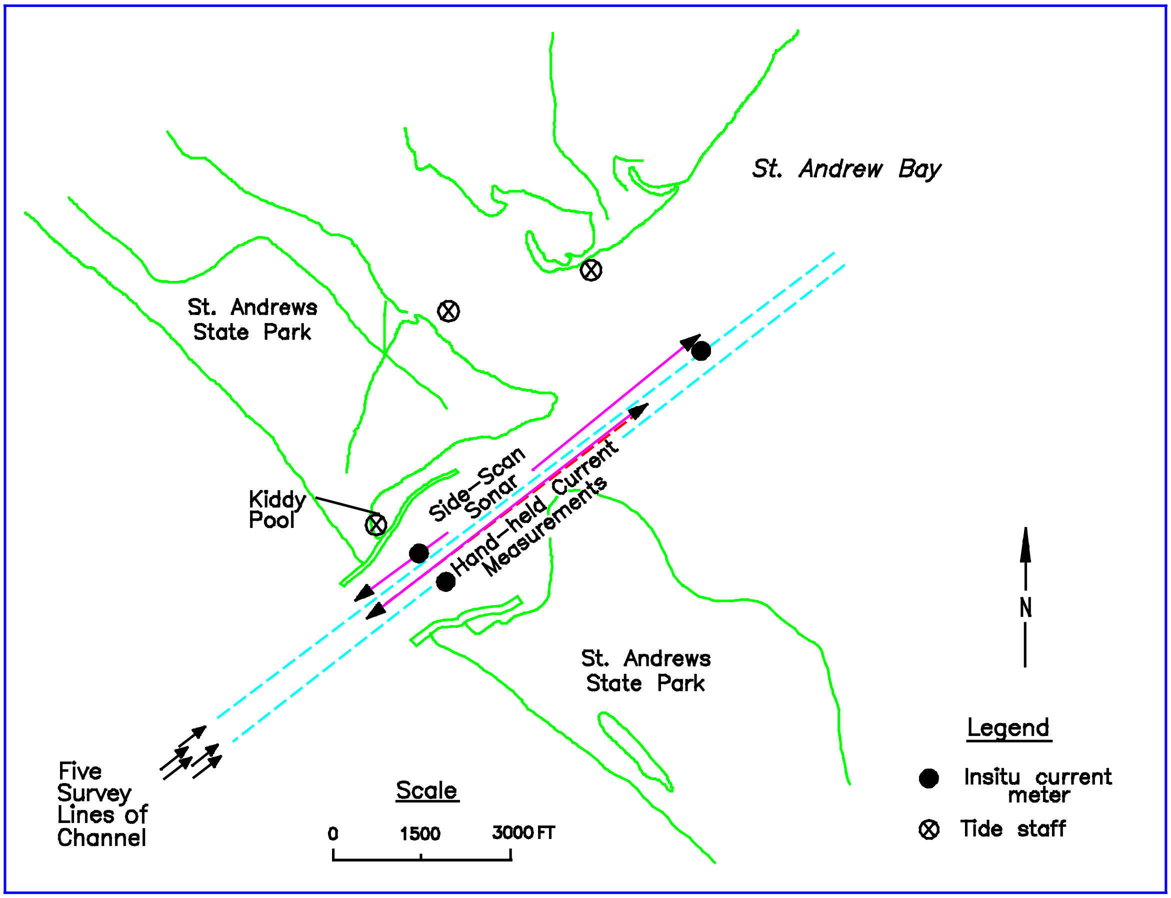

Figure 8-2. Data collected during the Panama City, Florida, preliminary field study (adapted from Lillycrop, Rosati, and McGehee (1989))

(2) However, a short-term measurement program is labor-intensive, and usually expensive. Often only one cross section at the inlet throat is monitored, where the most intense currents occur. Data collection is typically designed to coincide with spring or neap tide on the assumption that the currents will be the strongest. Other factors may also influence current speed and direction, such as runoff from rivers or complex interactions with other bay openings. It is recommended that the measuring period extend for at least 48 hr, and that some in situ current meters (discussed below) be deployed for an extended period. If the budget allows, measurements should be made several times during the year to learn more about seasonal effects on current speed and direction. If only one measurement period is possible, the study should be scheduled to coincide with the conditions under which problems at the site have been reported, or are likely to occur. For example, if structures have been damaged in spring, possibly as a result of increased river flow, then the study should be performed at that time.

(3) Significant changes in current patterns can occur while measurements are being taken, simply because all measurements cannot be made simultaneously without a large number of current meters and field workers. Significant current changes are most likely to be missed when the tides are turning. A way to reduce the likelihood of missing significant events is to perform the measurements at half-hour intervals during the tide change and at hourly intervals thereafter. Measurements during severe weather may indicate the most dynamic inlet processes, but field workers may not be able to stay safely at their stations.

b. Remote current meters.

(1) Many of the disadvantages of a short-term current measurement program can be alleviated by using remote current meters. Remote current meters can record data internally, or allow real-time reporting by sending data to shore via telemetry or cable. Data from internally recording (self-contained) meters are analyzed when the

meter is retrieved or the data are downloaded. Remote current meters are deployed by divers on a mooring during calm weather, left on station for a period of time, and recovered. If possible, they should be deployed for at least one complete lunar tidal cycle. Most internally recording meters can record flow speed and direction at 10- to 15-min intervals for a multi-week deployment; cabled or telemetered meters can stay on station indefinitely. This information can reveal subtle changes in the flow field as the tide turns, and can also show variations in maximum velocities over time. The greatest advantage of remote current meters is that they can record over conditions too severe for field workers, such as during the passage of storms or floods.

(2) Remote current meters are expensive, thereby limiting the number that can be deployed in an inlet. They must be located where they will not interfere with boat traffic, which can restrict their spatial coverage. If the meters are inadvertently in the path of trawlers or boat anchors, they can be damaged or lost. Frequent inspection of the moorings by divers can reduce the likelihood of loss, but adds expense to the project. Because most remote current meters record internally, the quality of data is unknown until the gauge is retrieved. If the gauge has malfunctioned, the data from that particular location may be lost. To help prevent equipment failure, the gauges should be thoroughly checked and calibrated prior to deployment.

(3) An ideal practice for a thorough field study would be a combination of both an intensive, manual current measurement effort accompanied with the deployment of remote current meters. The intensive field effort would provide spatial coverage, while the in situ meters would provide long-term temporal coverage.

c. Hydrographic (bathymetric) surveys.

(1) Large-area hydrographic surveys of a tidal inlet and the adjoining area can provide valuable information. Ideally, the surveys should include the inlet, the ebb tidal shoal and surrounding region, the flood tidal shoal, and back-bay channels that feed the inlet. The inlet and the offshore can usually be surveyed from a small boat, but a shallow flood tidal shoal may require rod and transit surveys. The surveys must be referenced to a standard datum.

(2) Although precision hydrographic surveys are labor-intensive and expensive, one should be conducted at the beginning of the field study, and another at the end if the study is of such a duration that significant bathymetric changes have occurred. These data can show changes in the inlet shape and orientation, and whether it is scouring or shoaling. If major construction, rehabilitation, or dredging is to be performed, the region should be surveyed before and after the work. Survey lines across the inlet can show the effect of the dredging on the navigation channels and on subsequent infilling or erosion. If current speed is obtained at various inlet cross sections, accurate survey information will allow the inlet's volumetric flow to be calculated.

d. Water level. Water level information should be obtained, either from an existing gauge, or a gauge specifically deployed for the monitoring period. The tide gauge should be deployed so that the measured water level can be referenced with respect to a standard datum. Water level information can be used in conjunction with the volumetric flow data to determine inlet tidal prism.

e. Wave information. Data on wave height, direction, and period are necessary for many inlet studies because wave-induced longshore currents can carry sediment to and from adjacent shorelines, damage structures, and be a significant process in forming ebb tidal shoals. Understanding these processes can help verify hypotheses about long-term trends at the study site. Offshore wave statistics are available for the Atlantic, gulf, Pacific, and Great Lakes coastal areas from the USACE Wave Information Study (WIS), which is based on hindcasting waves from meteorological data (Jensen 1983). Wave data are available from the National Data Buoy Center (NDBC) for 3- and 12-m (10- and 40-ft) discus buoys, which are in operation in the Great Lakes, Pacific, and Atlantic (Steele, Lau, and Hsu 1985; Steele et al. 1990). WIS statistics or NDBC data can be used to determine general trends for the project area, but complexities in local bathymetry and shoreline orientation at the study area can produce a local wave climate that is different from that projected using offshore data. Estimates of the nearshore wave climate can be obtained by using a numerical wave transformation model with local bathymetry and offshore wave data. To measure local waves, a directional wave gauge should be deployed within a few miles of the study area. If possible, the gauge should be in operation for at least 1 year so that a complete winter and summer cycle can be sampled. An 18-month deployment which covers two winters is preferable since the most severe wave climate occurs in winter for most of the United States. Exceptions would be those sites where there is significant ice cover during the winter. For these sites, the gauges should be recovered before winter so that they will not be lost during the spring thaw when drifting ice can gouge the seafloor.

f. Subsurface exploration. Inlet location and scouring/shoaling patterns may be controlled to some extent by underlying geologic structure. Clues that there might be structural control at a site are a stable inlet that has not migrated and rock outcrops on land, within the inlet, or offshore. Information about outcrop or regional structure may be available from the geological literature, but detailed exploration may be needed at some sites to plan construction or provide more information about long-term stability. Details on subsurface geology can be obtained from high-resolution geophysical surveys or from sediment cores. A combination of both is ideal: the cores provide control for the geophysics, and the geophysics provide a more regional image of the subsurface.

g. Detailed surface sampling. A comprehensive sampling program can be performed to learn more about source areas and transport patterns. In addition, sediment samples can be collected periodically if it is suspected that changes in sediment type occur during the year.

h. Sand tracer studies. Sand can be dyed and injected into the inlet system to trace sediment dispersion patterns. These studies would complement the drifter experiments described previously. Usually sand from the site is obtained, dyed, and washed with dish soap (to reduce clumping) prior to placement. The main disadvantages with the tracer experiments are that the sand may be dispersed too much to be traced, and counting sand grains is tedious.

i. Repetitive aerial photographs. At a site where the morphology changes throughout the year, periodic aerial photographs can be a valuable tool for mapping shoreline changes. At least two flights per year are recommended, with a "storm" flight reserved for severe northeasters or hurricanes that impact the site.

j. Meteorological data. Data on wind speed and direction should be collected during the hydraulic field studies. Weather records from nearby airports or military bases may be available. If not, a portable weather station can be established on a tower or pole near the project site. These data can reveal if wind setup contributed to unusual water levels in inlet back-bay areas.

k. Example of a detailed field study: Siuslaw River, Oregon.

(1) Under the MCCP, the Siuslaw River, Oregon, was monitored from 1987 to 1990 to determine the effectiveness of jetty "spurs." In 1985, the existing rubble-mound jetties were extended approximately 610 m (2,000 ft) seaward and 122-m-long (400-ft-long) spurs oriented at a 45-deg angle to the jetty trunk were constructed (Figure 8-3). Physical model studies conducted prior to spur additions indicated that the spurs would deflect material away from the structure, significantly reducing shoaling in the navigation channel. The objectives of the monitoring project were to (a) determine the effectiveness of the spurs in deflecting sediment, (b) identify shoaling patterns near the jetties, (c) compare existing prototype conditions to those predicted in the physical model study, (d) evaluate the effectiveness of the system in reducing maintenance dredging requirements, and (e) evaluate impacts of the jetties on the surrounding beaches.

(2) Bathymetric data extending alongshore for 10 km (6 miles) south of Siuslaw River and 8 km (5 miles) north, and offshore to an approximate depth of 7.6 m (25 ft), including some profiles perpendicular to the jetty in the vicinity of the spurs, were collected twice a year for 4 years prior and 5 years after spur construction (1981-1990).

(3) Dye dispersal, documented with video and aerial photographs, was conducted twice a year to indicate current patterns in the inlet and near the spur jetties. Seabed drifters were used in conjunction with the dye studies to indicate bottom current patterns. Bottom currents were also measured in the summer of 1990 by suspending a current meter with a 91-kg (200-lb) subsurface buoy from a helicopter. Current speed and direction at 22 locations in the vicinity of the inlet created a snapshot mosaic of current patterns for three different wave and current conditions during the field test. However, due to 21-m/sec (40-knot) winds, current patterns were primarily wind-dominated, and inlet-related currents were subdued. Under the MCCP, the current portion of the Siuslaw monitoring program was extended, and a similar helicopter current study was conducted during 1992 which successfully documented inlet circulation in the vicinity of the Siuslaw jetties (Pollock, in preparation).

(4) Side-scan sonar investigations of inlet and jetty conditions were conducted during a fall 1987 field test; however, wave conditions were too rough for boat maneuvering and the measurements were inconclusive.

(5) A directional wave gauge was deployed from September 1988 to September 1989 southwest of the entrance in 12-m-deep (40-ft-deep) water. Wave data during that year of deployment are being correlated with a

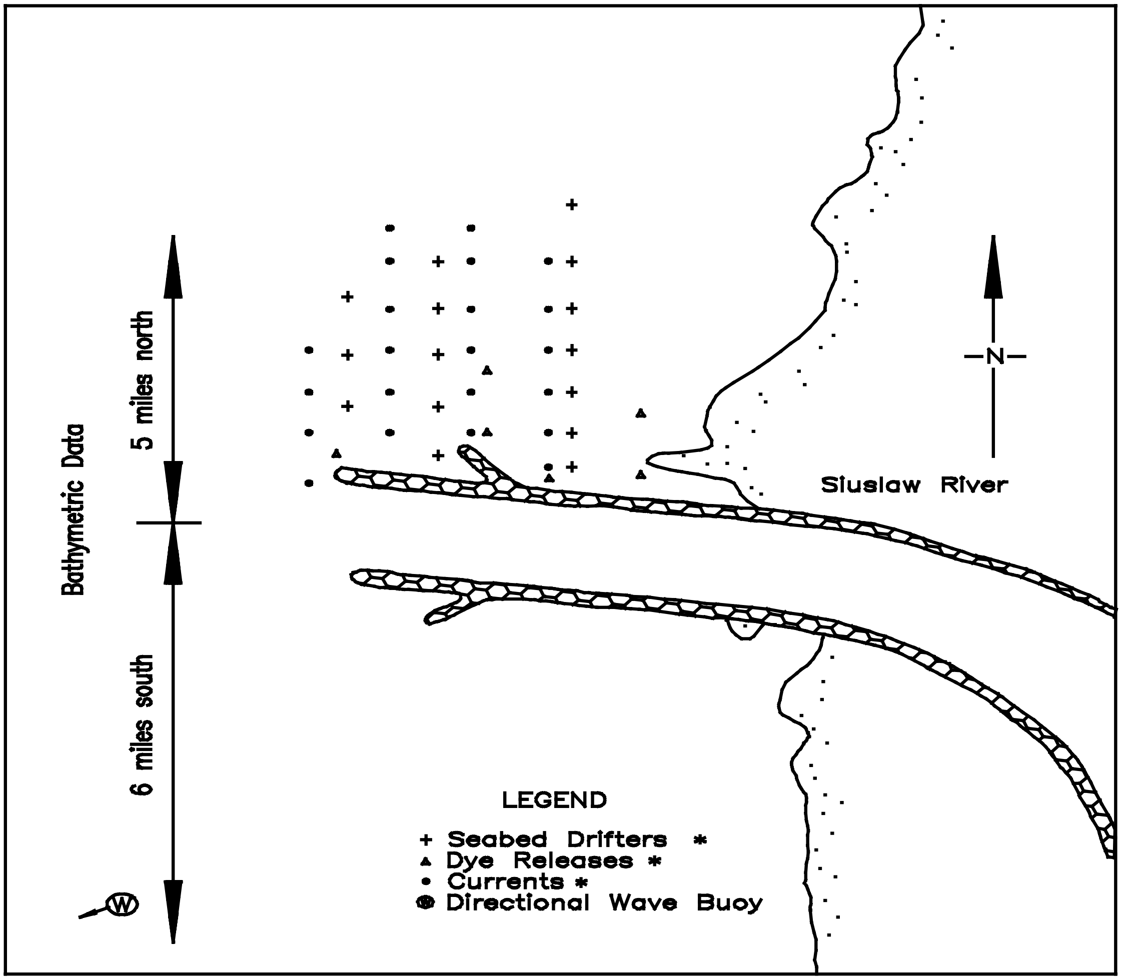

Figure 8-3. Data collected at Siuslaw, Oregon, detailed field study (*note that seabed drifter, dye, and current data were also measured south of the project)

permanent directional gauge located at Coquille, Oregon, approximately 97 km (60 miles) south. Once a correlation between the two gauges is known, data from the Coquille gauge can be adjusted for use at Siuslaw.

(6) Pre- and post-construction dredging data are being compiled and correlated with bathymetric changes, current speeds and directions, and wave information to determine impacts of the spur jetties on coastal processes.

8-6. Summary

a. Tidal inlets are dynamic coastal features that are fascinating to observe because of the rapid changes that can occur, driven by waves, tides, winds, sediment supply, structure design, and channel cross section. For engineering works to be successful, they must be in harmony with the physical processes and geographical constraints that exist at the inlet. Data necessary for a proper engineering design come from a monitoring project that has been designed to answer the critical questions.

b. It must be emphasized that data analyses should be performed during or immediately after the field work at each phase of a monitoring program. If critical measurements have been lost, there still may be time to deploy another instrument and try again. Since many new instruments perform data conversion and analysis internally or in the field by means of portable computers, quality control has improved. It has become easier to decide onsite if the instruments are performing properly, or whether a modification of the experiment is in order. In addition, field notes are available and memories of the participants are fresh during or immediately following the data collection effort.

c. Three phases to a field study have been described: reconnaissance, preliminary measurements, and detailed field study. Some level of reconnaissance is necessary for every monitoring project, although one or both of the latter phases may be omitted, depending on the purpose of the monitoring program. However, the critical process of any monitoring program, at every level, is proper planning.

Appendix A
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Appendix B
Notation

a_B Bay tidal amplitude

a_o Ocean tidal amplitude

$\hat{a}_B$ Dimensionless bay tidal amplitude

A Cross-sectional area of the channel

A_B Bay surface area

A_c Channel cross-sectional area

A_{c*} Critical cross-sectional area

A_{CE} Cross-sectional area of throat

A_i Cross-sectional area of ith channel segment

C Chezy bed resistance coefficient

C_a Sediment concentration in the bed layer

C_h Concentration of suspended sediment at a distance h above the bed

C_K Coefficient accounting for nonsinusoidal variation of current

d Depth

E Elasticity

F Impedance

F_e Force due to elasticity

F_g Force due to gravity

F_i Inertial force

F_{pr} Force due to pressure

F_{st} Force due to surface tension

F_μ Force due to viscosity

g Acceleration due to gravity

h_c Mean channel depth

h_i Depth of ith channel segment

i Inlet channel segment number (from 1 to m)

k_{en} Entrance loss coefficient

k_{ex} Exit loss coefficient

K Keulegan's repletion coefficient

L Representative length

L_c Channel length

L_h Horizontal length ratio

L_r Model-prototype length ratio used for scaling

L_v Vertical length ratio

m Total number of channel segments

M Total annual littoral drift

M_{mean} Average rate of longshore transport

n Manning's bed resistance coefficient

p Pressure

p Coefficient in channel width-depth relationship

P Tidal prism

q Exponent in channel width-depth relationship

Q Discharge through channel

Q_b Bed-load transport rate

Q_d Rate of sediment deposition per unit width of channel

Q_f Freshwater discharge from upstream sources

Q_m Maximum discharge through channel

Q_{max} Maximum discharge to inlet

Q_s Total suspended load on the updrift side of the channel

Q_{s1} Rate of transport of suspended load reaching the channel

Q_{s2} Transport rate across the channel

r Ratio of transport rate to inlet discharge used to characterize bypassing

S Bay storage volume

t Time

T_r Time ratio

u Current velocity in channel

u_m Maximum current velocity in channel

$\hat{u}_m$ Dimensionless maximum channel velocity

V Current velocity in channel

V_{max} Maximum channel velocity

V'_{max} Dimensionless maximum channel velocity

V_T Threshold velocity for sand transport

W_c Width at inlet throat

α Dimensionless tidal frequency

α_1 Coefficient in relationship between C and A_c

α_2 Coefficient in relationship between C and A_c

β Dimensionless dissipation coefficient

β Stability index

γ Kinematic viscosity

ε Lag of slack water after high or low water in the ocean

η Instantaneous water surface elevation relative to mean water level

η_B Instantaneous water surface elevation in the bay

η_o Instantaneous water surface elevation in the ocean

λ Stability index

μ Dynamic viscosity

ν Dimensionless maximum velocity

ν_E Equilibrium value of ν

ρ Density

σ Surface tension

σ Tidal frequency

τ Angular measure of the lag of slack water in the channel after midtide in the ocean

Ω Tidal prism

Appendix C
Summary of General Investigation of Tidal Inlets (GITI) Program Reports

C-1. Purpose and Scope

In 1969, the General Investigation of Tidal Inlets (GITI) Program was initiated under the technical surveillance of the U.S. Army Coastal Engineering Research Center (CERC). The GITI Program was established to conduct research into the behavior and characteristics of tidal inlets and to provide quantitative data for use in design of inlets and inlet improvements. Research was conducted by CERC, the U.S. Army Engineer Waterways Experiment Station's Hydraulics Laboratory, other Government agencies, and private organizations. This appendix describes the GITI research program and presents a brief summary of each report published under the program.

C-2. Research Objectives and Design

The GITI Program was divided into three major study areas; inlet classification, inlet hydraulics, and inlet dynamics. A total of 22 reports have been published as part of the GITI series; five related to classification, nine on hydraulic studies, and eight on dynamic studies.

a. Inlet classification. The objective of the inlet classification study was to group inlets according to their geometry, hydraulics, and stability. Early efforts were designed to produce three independent classifications. Plans for future research involved efforts to interrelate the three separate classifications and investigate reasons for correspondence between well-defined classifications. This aspect of the study involved collection of large data sets on the physical characteristics of numerous tidal inlets.

b. Inlet hydraulics. Objectives of the inlet hydraulics study were to define tide-generated flow regime and water level fluctuations in the vicinity of coastal inlets and to develop techniques for predicting these phenomena. The inlet hydraulics study was subdivided into three areas; idealized inlet modeling, evaluation of state-of-the-art physical and numerical models, and prototype inlet hydraulics.

c. Inlet dynamics. Basic objectives of the inlet dynamics study were to investigate the interactions of tidal flow, inlet configuration, and wave action at inlets as a guide to improvement of inlet channels and nearby shore protection works. The study was subdivided into four specific areas; model materials evaluation, movable-bed modeling evaluation, reanalysis of a previous inlet model study, and prototype inlet studies.

C-3. Report Summaries

The following are short summaries of each GITI report in numerical order. Names of authors and date of publication are noted at the end of the summary. Portions of the summaries have been reproduced from abstracts published with the original reports.

GITI Report 1:[1] *Reanalysis of Beach Erosion Board Technical Memorandum No. 94*

In the 1950's, the U.S. Army Beach Erosion Board (BEB)[2] and the U.S. Army Engineer Waterways Experiment Station conducted a series of small-scale model tests to evaluate the impact of an unimproved inlet on adjacent beaches. Test results from six of the eight tests were reported in BEB Technical Memorandum No. 94. A reanalysis of the data, performed under the GITI Program, was originally intended to be the first publication in the GITI series; however, publication was delayed and the report eventually was distributed as a miscellaneous paper by the Hydraulics Laboratory. Reanalysis indicated that the area of the model inlet channel was related to the tidal prism, but the relationship was different from that previously determined by O'Brien for prototype inlets. The reanalysis also showed that the minimum channel area was approximately 80 percent of the average channel area. The Keulegan method was found to be an effective tool for predicting model inlet behavior. TM 94 is included in its entirety as Appendix A for the benefit of those without access to the original report. All data, including those from tests not described in the original report, are also provided in appendices. [E. C. McNair, 1987]

GITI Report 2: Catalog of Tidal Inlet Aerial Photography

This catalog of inlet aerial photography covers inlets on the Pacific, Atlantic, and Gulf coasts and was designed to be an information source for studies of inlet geomorphology and stability. Data from approximately 6,000 inlet overflights dating from 1938 to 1974 (including date,

[1] Published as Miscellaneous Paper HL-87-1.

[2] The Beach Erosion Board is now the Coastal Engineering Research Center (CERC), U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

source agency, project name, exposure numbers, scale, and film type) are presented in tables and indexed according to Corps of Engineer District Office. Exposures for individual date listings are the minimum number necessary to cover the inlet throat and in most cases, are sufficient to identify all associated tidal delta complexes. [J. H. Barwis, 1975]

GITI Report 3: Tidal Prism-Inlet Area Relationships

This report was a secondary result of research conducted as part of the inlet classification study. During an investigation of the variation in the Keulegan repletion coefficient with variations in inlet geomorphology, new tidal prism data were generated. The opportunity was then taken to reexamine relationships originally developed by O'Brien between tidal prism (P) and inlet cross-sectional area (A). A total of 162 data points, representing 108 inlets along the Atlantic, Pacific, and Gulf coasts, were included in the analysis. Data were grouped into three categories according to number of jetties and then further divided based on location (Atlantic, Pacific, or Gulf coast). Regression analysis was performed on each data set to determine best fit relationships. Results yielded equations of the form $A = CP^n$, where C and n are constants. It was determined that unjettied and single-jettied inlets on the three coasts do exhibit different P versus A relationships as a result of the different tidal and wave characteristics of the three coasts. For jettied inlets, it was concluded that no modification of O'Brien's P versus A relationship was warranted by the additional data. [J. T. Jarrett, 1976]

GITI Report 4: Annotated Bibliography on the Geologic, Hydrologic, and Engineering Aspects of Tidal Inlets

This report contains citations for approximately 1,000 published and unpublished documents on geologic and engineering aspects of tidal inlets. References were collected during a literature survey made as background for the inlet classification aspect of the investigation and include reports on tidal hydraulics, inlet structures, littoral processes, inlet stratigraphy and geologic history, coastal aerial photography, and inlet case studies. References are listed alphabetically by last name of the senior author and a cross-referenced subject index is provided. [J. H. Barwis, 1976]

GITI Report 5: Notes on Tidal Inlets on Sandy Shores

This report presents an edited collection of observations and theories on several aspects of tidal inlets prepared by M. P. O'Brien during 40 years of inlet-related work. These notes were originally intended only to help a graduate student formulate a research project; however, it was believed that they could serve as a valuable addition to the GITI publication series and would stimulate further inlet research. [M. P. O'Brien, 1976]

GITI Report 6: Comparison of Numerical and Physical Models, Masonboro Inlet, North Carolina

GITI Report 6 presents an evaluation of existing inlet modeling techniques performed by calibrating a physical model and three numerical models with prototype data from Masonboro Inlet. Model verification was not conducted since no additional prototype data were available. A distorted scale fixed-bed physical model, a lumped parameter numerical model, and two two-dimensional numerical models were included in the study. To extend the model comparison, the two-dimensional, shallow-water hydrodynamic equations were derived from the Navier-Stokes equations and the physical interpretation and significance of each term were discussed. Study results indicated that the four models more accurately simulated tidal height than tidal current. It was concluded that physical models provide more reliable predictions than numerical models of the effect of small-scale phenomena. On the other hand, numerical models can provide better predictions of the effects of the earth's rotation, wind stress, and pressure gradients. Reports on each of the four models investigated were published as separate appendixes to the main report. [D. L. Harris and B. R. Bodine, 1977, Main text and Appendices 1-4]

GITI Report 7: Model Materials Evaluation; Sand Tests; Hydraulic Laboratory Investigation

A recognized need in making movable-bed inlet modeling a precise science was a better understanding of the model relationships between the fluid motions, sediment, and resulting inlet characteristics. This report summarizes a series of 21 laboratory tests conducted in a partitioned flume with a 40-ft-long test section of beach including an inlet. Since stable and definable flow conditions were considered essential for the tests, a series of steady unidirectional flows was substituted for cyclic tidally induced flows. Tests consisted of various steady discharges through the inlet with and without waves acting on the seaward end of the channel. Surveys of the inlet were taken periodically to evaluate changes in channel geometry. Tests indicated that minimum channel area, channel width, and the hydraulic radius were strongly related to rate of flow through the channel. The ability to scale channel geometry was demonstrated; however, it was determined that scaling relations for material transport required further investigation. The report also presents specific recommendations for improved test procedures for future model material studies. [E. C. McNair, 1976]

GITI Report 8: Hydraulics and Dynamics of New Corpus Christi Pass, Texas: A Case History, 1972 - 1973

Corpus Christi Water Exchange Pass extends from Corpus Christi Bay to the Gulf of Mexico through Mustang Island, Texas. Studies of sedimentation and hydraulics of the pass began with its opening in 1972 and continued through 1975. This report presents data on the initial adjustment (1 year) of the pass to tides, waves, and other forces. It was estimated that approximately 1 million yd^3 of sand accumulated at the pass during construction of the two jetties. Downdrift beaches exhibited considerable sand loss and extensive shoal deposits formed near the bay end of the pass. Average discharge through the pass was only about 3 percent of the total tidal prism, indicating that Aransas Pass to the north was the primary bay-gulf connection and that the creation of Corpus Christi Pass had no significant effect on flushing of the bay. Work on this research was conducted by the University of Texas Marine Science Institute. [E. W. Behrens, R. L. Watson, and C. Mason, 1977]

GITI Report 9: Hydraulics and Dynamics of New Corpus Christi Pass, Texas: A Case History, 1973 - 1975

Data obtained during the second phase (1973-1975) of a field study of Corpus Christi Water Exchange Pass are presented in this report. Qualitative and quantitative data on longshore sediment transport, tidal differentials, flood and ebb tidal discharge, wind waves, and local winds provided information on both the long- and short-term stability of the pass and on the processes affecting the dynamics of the pass and adjacent beaches. It was determined that the flood dominant nature of the system, together with a long channel, required that most sediment entering the channel be carried through its entire length to be deposited on the flood-tidal delta if the pass was to remain open. Results of stability analyses suggested that the pass was of marginal stability with a tendency toward closure. [R. L. Watson and E. W. Behrens, 1976]

GITI Report 10: Hydraulics and Dynamics of North Inlet, South Carolina, 1974 - 1975

This report presents results of the first phase of a field study to define the hydraulics and dynamics of North Inlet, South Carolina. Field work was conducted quarterly to characterize seasonal variations and included a general reconnaissance of the inlet area, beach profile surveys, bathymetric mapping, tidal elevation and current velocity measurements, and aerial photography. The importance of seasonal variation in processes was emphasized by significant differences in wave parameters, short-term morphologic response, and tidal parameters. [R. J. Finley, 1977]

GITI Report 11: Laboratory Investigation of Tidal Inlets on Sandy Coasts

This report is based on a series of 36 experiments conducted at the Hydraulic Engineering Laboratory at the University of California, Berkeley. Experiments were performed in an idealized movable-bed tidal inlet model for a variety of geometric characteristics. Sinusoidal tides and model waves were run until a periodic tide was established in the bay. Measured parameters included cross-sectional area; water surface elevations of the ocean, bay, and inlet; and inlet velocities. Results indicated that two techniques accurately predicted idealized inlet hydraulics; Keulegan's repletion coefficient technique, and the lumped parameter method which extends the Keulegan method by considering inertia and variable inlet geometry. Experimental data are presented in tables, photographs, and plots. Comparisons of tidal prisms and minimum flow areas between laboratory results and field data are also presented. [R. E. Mayor-Mora, 1977]

GITI Report 12: A Case History of Port Mansfield Channel, Texas

This report documents the hydraulic and sedimentary characteristics of Port Mansfield Channel and presents an evaluation of its behavior from construction in 1962 to 1975. The channel is an artificial, jettied inlet on the Texas coast connecting the Gulf of Mexico with Laguna Madre. Seaward migration of the updrift shoreline and shoaling in the entrance channel suggested that material was bypassing the jetty. Significant annual dredging was necessary to maintain design channel dimensions. Predictions of stability using relationships developed by Escoffier, O'Brien, and Bruun and Gerritsen were found to predict the unstable nature of the inlet. Keulegan's repletion coefficient was calculated to investigate the hydraulic capacity of the channel; a value of 0.57 indicated that infilling of the bay would be incomplete. It was also determined that channel velocities were not sufficient to cause natural scour and maintain the design cross-sectional area. The instability of the inlet was attributed to the large head loss due to friction in the extremely long channel. [J. M. Kieslich, 1977]

GITI Report 13: Hydraulics and Stability of Tidal Inlets

Classic inlet hydraulic and stability work by Brown, O'Brien, Escoffier, Bruun, Keulegan, O'Brien and Dean, Johnson, and Jarrett is summarized in this report. The original stability concept is extended and functional design requirements are discussed. In addition, case studies of Masonboro Inlet, North Carolina, Rollover Fish Pass, Texas, and Mission Bay Inlet, California, are presented. [F. F. Escoffier, 1977]

GITI Report 14: A Spatially Integrated Numerical Model of Inlet Hydraulics

This report discusses development of a simple numerical model for the prediction of inlet channel velocities and discharge as well as resulting bay surface level fluctuations for inlets responding to the tide and other long wave oscillations. The model simultaneously solves the area-averaged momentum equation for the inlet and the continuity equation for the bay. The bay surface elevation is assumed to remain horizontal during rise and fall. At each time-step, the geometric and hydraulic factors describing the inlet-bay system are calculated by evaluating flow conditions throughout the inlet and by spatially integrating this information to determine coefficients of the first-order differential equations. The model was determined to be flexible and to give realistic estimates of inlet-bay hydraulics. [W. N. Seelig, D. L. Harris, and B. E. Herchenroder, 1977]

GITI Report 15: Physical Model Simulation of the Hydraulics of Masonboro Inlet, North Carolina

This report is the first of two publications presenting detailed results of the Masonboro Inlet fixed-bed model study. The model study was conducted to determine the ability of existing physical modeling techniques to simulate hydraulic characteristics of an inlet-bay system and to determine whether simple tests could be useful in predicting the effects of proposed inlet improvements. This report presents model verification and prediction data as well as analyses concerning the effects of waves on model hydraulics. Five velocity ranges with three stations at each range were verified in the model; seven tidal elevation gauges in the ocean and bay were also verified. Model predictions of filling of the dredged navigation channel and deposition basin, and a tendency for the channel to shift toward the north jetty were substantiated by comparison with prototype data. [R. A. Sager and W. C. Seabergh, 1977]

GITI Report 16: Hydraulics and Dynamics of North Inlet, South Carolina

This report presents results of the second phase of a field study to define the hydraulics and dynamics of North Inlet, South Carolina. Detailed bathymetric profiling of the inlet throat and channels and topographic mapping of subaerial, intertidal, and shallow subtidal zones were used to define the seasonal morphologic variability of the inlet and adjacent beaches. Wave and tidal data provided basic information on wave conditions and inlet hydraulic characteristics and were correlated with observed bathymetric changes. Results presented in this report suggested that North Inlet was hydraulically ebb dominated; peak ebb velocity exceeded peak flood by a factor of 1.22. In a multiple stepwise regression analysis, the longshore component of wind velocity was found to explain more of the variance in the observed longshore current velocity than any other measured parameter. [D. Nummedal and S. M. Humphries, 1978]

GITI Report 17: An Evaluation of Movable-Bed Tidal Inlet Models

The objectives of this study were to evaluate the effectiveness of movable-bed tidal inlet hydraulic models in predicting prototype behavior and to examine the scaling requirements for such models. Seven model studies, conducted at WES between 1939 and 1969, were evaluated. Calibrations of five of the models, as measured by bed topography changes, were evaluated using correlation coefficients and root-mean-square (rms) error. Due in part to inadequate prototype data, acceptable model performance was not always achieved; values of correlation coefficients were typically low and those of rms error high. A literature review was also performed to determine the present understanding of and practice concerning similitude requirements for movable-bed coastal models. Major capabilities and limitations of movable-bed inlet models are discussed and an assessment of the general conditions and similitude requirements under which inlet models may be expected to yield reliable results are outlined. The study was conducted at the Iowa Institute of Hydraulic Research, The University of Iowa. [S. J. Jain and J. F. Kennedy, 1979]

GITI Report 18: Supplementary Tests of Masonboro Inlet Fixed-Bed Model

This report is the second concerned with testing in the Masonboro Inlet fixed-bed physical model and describes three separate supplemental test series. The first study examined the effects of closing various bay channels on inlet hydraulics. The second investigated the effects of adding a south jetty to the existing project, which had a single north jetty, and included an evaluation of the resulting hydraulics for various weir configurations on both jetties. The third study involved sediment tracer testing under the action of tides and wind waves and was designed to evaluate the effectiveness of using tracer materials in inlet model studies. Results indicated that the closure of any of the three interior channels in Masonboro Inlet produced a significant change in inlet hydraulics and morphology. Weir jetty testing indicated the effect of a south jetty was to centralize flood flow through the inlet gorge, and the presence of a weir on the south jetty did not alter the basic flow pattern. Tracer experiments showed that short-term fill and scour trends could be predicted qualitatively; however, major changes in bathymetry preclude quantitative long-range predictions. [W. C. Seabergh and R. A. Sager, 1980]

GITI Report 19: Tidal Inlet Response to Jetty Construction

During an evaluation of inlet models, a similarity in channel and beach response to jetty construction at Tillamook Bay, Oregon, and Masonboro Inlet, North Carolina, was noted. An effort was then undertaken to determine if the response pattern exhibited by these two inlets was typical of other inlets on U.S. coasts. This report presents results of that investigation. Thirteen tidal inlets located on the Atlantic, Gulf, and Pacific coasts of the continental United States were selected for study. Inlet entrance behavior following jetty construction was evaluated, and guidelines for the functional design of inlet entrance improvements are suggested. Inlets considered in the study were those where a single updrift or downdrift jetty was built first. The construction of single jetties resulted in migration of the channel thalweg toward the jetty regardless of the inlet-bay orientation, angle of the jetty to the shoreline, position of the jetty relative to the direction of net longshore transport, the ratio of net-to-gross transport, or the gross transport. Accretion of the updrift shoreline, erosion of the downdrift shoreline, and a decrease in channel cross-sectional area typically followed construction of an updrift jetty. Sufficient data were not available to generalize response following construction of a downdrift jetty. [J. M. Kieslich, 1981]

GITI Report 20: Geometry of Selected U.S. Tidal Inlets

This report presents a classification of inlets based on objective analysis of similarities between inlet geometric (morphologic) characteristics. Characteristics of the inlet throat and ebb delta of 67 U.S. tidal inlets were investigated. Thirteen parameters indicative of tidal geometry were defined and measured with correlations developed. The parameters are shown to vary in a consistent fashion that appears to be scaled according to the relative magnitude of tidal processes. Results of both cluster analysis and discriminate analysis indicate the presence of at least six well-defined clusters or types of inlets based on geometry. The classification provides a systematic organization of inlet morphology that is related to deviations from the basic scaling relationship probably due to the influence of wave action. The report contains substantial amounts of inlet morphologic data obtained from aerial photographs and boat sheets that may be applicable to site-specific studies. [C. L. Vincent and W. D. Corson, 1980]

GITI Report 21: Stability of Selected U.S. Tidal Inlets

This report presents a study of tidal inlet stability based on changes in geomorphic parameters measured from aerial photographs. A total of 51 inlets were selected for study, representing the range of inlets along U.S. coasts. Years of photographic coverage include 1938 through 1976. Hydraulic and geographic (positional) stability parameters were defined, measured, and used to create four stability indices describing the relative variation of principal aspects in which inlets can be expected to change in time. A single parameter was devised to indicate hydraulic stability and another for positional stability. Arbitrary stability limits were then selected and all inlets classified as stable or unstable. Regional patterns of inlet stability were also investigated; however, no strong correlation was found. Data presented in this report will be valuable in future studies of relationships between inlet geomorphic changes and appropriate hydraulic parameters. [C. L. Vincent, W. D. Corson, and K. J. Gingerich, 1991]

GITI Report 22: Evaluation of Physical and Numerical Hydraulic Models, Masonboro Inlet, North Carolina

GITI Report 22 is the last in a series of documents describing the calibration and verification of the physical model and several numerical models of Masonboro Inlet. This report presents a comparison of the predictions of a fixed-bed, distorted-scale physical model, a two-dimensional, vertically integrated numerical model, and a spatially integrated numerical model with a set of July 1974 prototype data. Both the physical model and the two-dimensional model reproduced measured tidal records and vertically averaged velocities equally well.

Predictions from the two models and the prototype data were averaged for comparison with the spatially integrated model. The spatially integrated model predicted mean inlet velocities significantly better than the other two models; however, it did not predict the average bay levels as well. In addition, the author presents possible ways to improve results of each of the three models. [J. E. McTamany, 1982]

Appendix D
Acknowledgement

Table D-1
Authorship and Review of EM 1110-2-1618, "Coastal Inlet Hydraulics and Sedimentation"

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1	Lee L. Weishar[1] Kathryn J. Gingerich[2]	Thomas Bender[7] Leslie M. Fields[1] John H. Lockhart, Jr.[8] Thomas F. Moslow[3]
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