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A COASTAL ENGINEERING ANALYSIS OF ROOSEVELT INLET, LEWES, DELAWARE

by

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This report is dedicated to Michael Peter John, deceased, October 1974.

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ABSTRACT

The presence of Roosevelt Inlet has caused significant erosion problems at Lewes Beach, Delaware. Furthermore, the deteriorated condition of the steel sheet pile jetties have allowed excessive shoaling to occur in the inlet.

Historical material was collected for the proper analysis of these problems consisting of: a complete history of the connecting waterways prior to the initial excavation of Roosevelt Inlet in 1937, an analysis of shoreline changes in the vicinity of the inlet both before and after stabilization, comparison of historic hydrographic charts resulting in estimates of shoaling rates and the identification of trends in changing inlet dimensions, and a compilation of dredging and beach nourishment histories for the inlet and Lewes Beach, respectively.

To examine the present littoral and hydraulic processes on-going in the vicinity of the inlet, four field studies were conducted. These studies resulted in complete surveys of both the offshore and inlet bathymetries, comparative beach profiles

along 1,000-foot sections of both adjacent beaches, a sand tracer study, and current and tide measurements — within the inlet throat as well as in the Lewes and Rehoboth Canal.

A numerical model was developed encompassing the connecting waterways from Roosevelt Inlet to Indian River Inlet. The model was used to investigate the hydraulic and stability characteristics of Roosevelt Inlet. A major result of the model is a prediction of a mean southerly pumping of water throughout the entire system. The effect of this mean flow through Roosevelt Inlet was found to significantly enhance its tendency to shoal.

Results of this study indicate that sand should be periodically bypassed in order to help alleviate the erosion at Lewes Beach. This may be accomplished in conjunction with the maintenance dredging of the inlet channel. The results also indicate that the inlet should be redesigned, decreasing the jetty width from 500 to approximately 350 feet, resulting in a deeper and more maintenance-free navigation channel. Furthermore, it may be advantageous to incorporate a low-sill weir section at the shoreward end of the updrift (west) jetty allowing sand to spill over the weir into a depositional area, thus becoming readily available for bypassing.

CHAPTER I

INTRODUCTION

1.1 The Problem

Between December, 1936 and May, 1937, approximately 520,000 cubic yards of sand, mud, and clay were excavated across Lewes Cape. The new inlet, connecting Delaware Bay and the Lewes and Rehoboth Canal, realized during the presidency of Franklin Delano Roosevelt, and named for him, was met with much approval from the townspeople of Lewes, most of whom depended on free access to Delaware Bay in order to make a living. In conjunction with the initial excavation, two parallel steel sheet pile jetties were constructed to stabilize the inlet and to maintain a navigable channel. During the early 1940's, the harsh salt environment began to corrode the twin steel structures, until today, the jetties have deteriorated well beyond their effectiveness (see Figure 1). The present state of the jetties allows easy passage of the littoral drift into the inlet which deposits in lobe-shaped shoals along both the east and west banks. Progressive shoaling has resulted in continuous maintenance dredging to afford a safe and unobstructed entrance. A further consequence of the trapped littoral drift is the starvation and erosion of adjacent beaches. Unfortunately, the major erosion is

FIGURE 1 Badly Deteriorated Steel Sheet Pile Jetties at Roosevelt Inlet.
Courtesy of Dr. Kent S. Price

occurring along the Lewes Beach which is downdrift of Roosevelt Inlet (see Figure 2). Numerous beach nourishment projects along with groin construction have been necessary to assure protection of upland property and provide adequate recreational beachfront.

1.2 Location

Roosevelt Inlet is located on the extreme southern edge of Delaware Bay approximately three nautical miles west of Cape Henlopen (see Figure 3). The inlet, which lies on a northeast bearing, forms the junction between the Broadkill River, the Lewes and Rehoboth Canal, College of Marine Studies Harbor (University of Delaware), and Canary Creek (see Figure 4).

To the west of the inlet is located the long, narrow undeveloped barrier known locally as Beach Plum Island. This area of beach will be termed "West Beach" throughout this report. To the east of the inlet lies the incorporated municipality of Lewes. The municipality is physically divided into two sections by the Lewes and Rehoboth Canal. The area north of the canal and adjoining Delaware Bay is commonly known as Lewes Beach, while the area to the south is Lewes. Cottages and beach houses line the beach for nearly two miles. The shoreline further east is occupied by commercial fish canneries and fish products industries, which have numerous piers projecting into Delaware Bay. Also in this general vicinity are located the terminal facilities for the Cape May-Lewes Ferry system.

FIGURE 2 Downdrift Erosion Occurring Along Lewes Beach

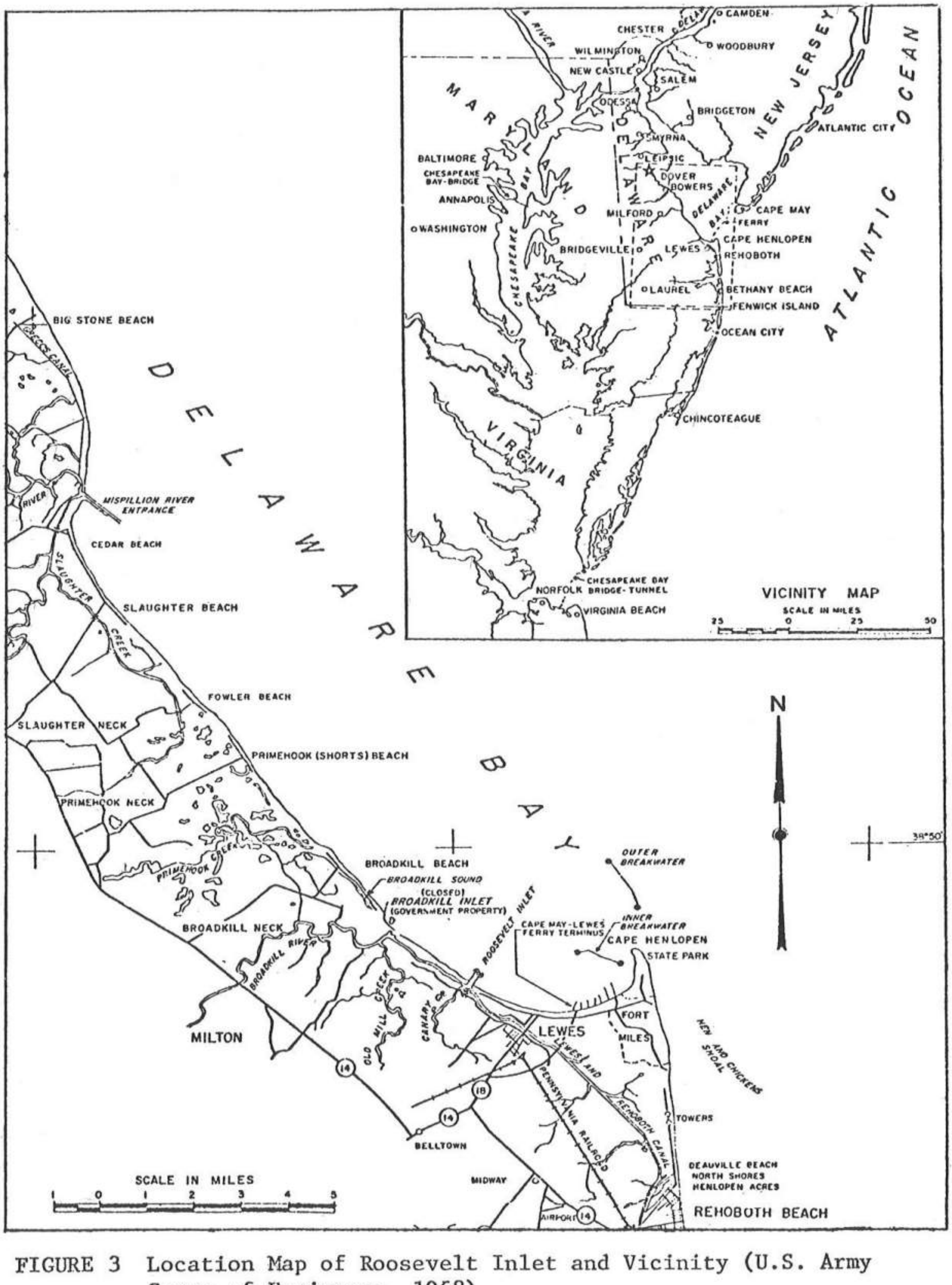

FIGURE 3 Location Map of Roosevelt Inlet and Vicinity (U.S. Army Corps of Engineers, 1968)

FIGURE 4 The Junction of Roosevelt Inlet formed by the Lewes and Rehoboth Canal on the left, the College of Marine Studies Harbor and Canary Creek in the background, and the Broadkill River on the right

Directly offshore of eastern Lewes Beach are two large hooked breakwaters. The breakwaters were constructed to offer storm protection to passing vessels along the heavily travelled shipping lanes of Delaware Bay. The outer breakwater forms the Harbor of Refuge, and the inner, Breakwater Harbor. The presence of these structures as well as Cape Henlopen considerably reduces the wave energy emanating from the east and northeast onto Lewes Beach.

1.3 General Geomorphology

The southern Delaware Bay shoreline lies within the Coastal Plain province, a physiographic region bordered along the west by the base of the eastern Appalachian Mountains (Piedmont province) and extending eastward to the edge of the continental shelf. During the Wisconsin, or latest glacial stage, it is estimated that sea level was 320 to 340 feet below its present level (Kraft, 1971). This would place the shoreline about 60 miles east of the present Delaware Coast. A general world-wide warming trend and thus the gradual melting of the glaciers marked the beginning of the Recent or Holocene epoch some 15,000 years ago. The slow return of melt water to the ocean from the receding glaciers and consequent rise in sea level caused a progressive migration of the shoreline across the continental shelf and the formation of barrier beaches, bays, and lagoons. Many lagoons eventually became marshes as they continually filled with sediment, typical of those along the Delaware Bay shoreline.

All evidence indicates that the marine transgression is continuing today. Radioactive carbon dating of core samples shows that the rate of relative sea level rise has been approximately one half a foot per century over the past 3,000 years, and twice that rate between 3,000 and 7,000 years ago (Kraft, 1971). Whether or not the sea level is actually rising or the land is subsiding is immaterial, since the end result is the same. The gradual barrier migration is accomplished by the erosion of the seaward or trailing edge of the barrier and deposition on the leading edge via the overwash process. The growth of shoals in the vicinity of inlets also contributes significantly to barrier migration. Evidence of this process such as washover fans and exposed marsh outcroppings are common features along the Delaware coast.

Beaches bordering the southern Delaware Bay shore are generally narrow being 20 to 100 feet wide at high tide. Foreshore slopes vary between 1 on 15 to 1 on 20, with offshore slopes being excessively mild, ranging between 1 on 300 to 1 on 500. The beaches are typically backed with a belt of grass-covered dunes with heights of 8 to 12 feet and widths of 50 to several hundred feet. Extensive saltwater marsh systems back the sandy barriers and may extend for as much as two miles inland. Numerous drainage ditches and meandering creeks are present throughout the marshes.

Principal waterways in the area are the Broadkill River and the Lewes and Rehoboth Canal. The Broadkill River covers a length of approximately 13 miles with its major source being Wagamons Pond near the town of Milton. The upper 11 miles follow a meandering northeasterly course deflecting eastward and paralleling the west beach barrier for the remaining two miles before exiting into Delaware Bay through Roosevelt Inlet. This estuarine system has a drainage basin of approximately 110 square miles (DeWitt, 1968). The Lewes and Rehoboth Canal (L&R Canal) is a tidal waterway that extends southeastward and southward for approximately 8 miles from Roosevelt Inlet to Rehoboth Bay. It forms an important link in Delaware's Inland Waterway allowing protected passage from Indian River Inlet to Roosevelt Inlet. The canal, most of which was man-made, was excavated through two relatively high sections of land; one being in the western end of Lewes near the site of the original settlement and the other near Rehoboth--the latter nearly 26 feet above mean low water.

At the confluence of Delaware Bay and the Atlantic Ocean a simple spit, Cape Henlopen, has formed. This spit is growing steadily northwestward as sand is continually added to the system from Delaware's eroding Atlantic coast beaches. Most of the sand reaching the spit is deposited in deep water at its terminus or else is winnowed out by tidal currents and deposited on Hen and Chickens Shoal which stretches for about 10 miles to the southeast from the tip of the Cape. Before the early 1800's the Cape was much rounder and more blunted. Sand within the littoral regime along the Atlantic coast moved easily around the blunted Cape acting as a sand supply for Lewes Beach and vicinity. As the spit progressed into deeper water, larger volumes of sand were spread along the base of its leading edge and less sand was transported around the Cape to serve as a supply for Lewes Beach and vicinity. Today, the washover beaches and barriers of lower Delaware Bay have a much reduced source of sediment supply from the east and as a result rapid transgression is occurring.

1.4 The Environment

Winds, waves and currents all significantly influence the movement of sand in the vicinity of Roosevelt Inlet. These forces vary in both intensity and duration and thus can cause rapid change in a short time span on the order of hours, or slower seasonal and yearly changes can occur.

The prevailing winds over Delaware Bay are from the west at average speeds around 10 knots. Wind data recorded by the U.S. Weather Bureau of Breakwater Harbor for a period of 18 years show that southwest is the prevailing wind direction, although winds from other directions are nearly as frequent. The data also show that gale force winds, those over 30 miles per hour, originate most often from the northwest. Furthermore, winds of the highest sustained velocity occur most frequently out of the northeast.

The mean wave height in Delaware Bay is approximately 2 feet (Brower). The largest waves reaching the Roosevelt Inlet vicinity are generated in the north to northwest sections of the Bay as a result of the dominant westerly winds usually of long duration aligning with the longest fetch distance in the bay. The existence of the numerous Cape May shoals, the Harbor of Refuge and Cape Henlopen significantly reduce the effect of the waves generated in the Atlantic out of the east and northeast. However, strong northeast winds can still generate waves of at least 2 feet at Lewes Beach within the fetch between Harbor of Refuge and the Delaware Bay shore (Personal Observation). Tables I and II are wind and wave data summaries for Delaware Bay and offshore Delaware Bay (after Brower, 1972).

Maximum flood and ebb tidal currents within Delaware Bay attain speeds of 2 - 2.5 knots. Surface tidal currents tend to be directed along the axes of the bay except in the area behind Cape May where currents tend to follow the shoreline. (Kupferman, et al., 1974). Nearshore tidal currents are generally weak (less than 1 knot) and presumably do not contribute significantly to the transport of sand size-particles. Figures 5 and 6 show the maximum flood and ebb currents at Delaware Bay entrance.

There are two major types of storms which cause severe damage along the Delaware coast; hurricanes and "northeasters."

TABLE 1 Wind and Wave Data Summary for Delaware Bay (After Brower, 1972)

ENVIRONMENTAL FACTORS	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANN
WIND SPEED (KNOTS)													
01% ≤	2	2	2	2	2	2	1	1	1	2	2	3	2
Mean	11.1	11.2	11.3	10.4	9.0	8.3	7.0	7.8	8.5	9.2	10.2	10.3	9.6
99% ≤	37	35	37	32	25	24	22	22	24	33	36	36	30
Maximum observed (1871-1971)	Winds near 90 knots have probably occurred over Delaware Bay												
≥ 34 knots (% freq.)	2.7	2.5	1.8	0.8	0.2	0.1	0.2	0.3	0.5	1.0	1.4	1.8	1.1
≥ 41 knots (% freq.)	0.7	0.7	0.5	0.3	0.1	0.1	0.1	0.1	0.1	0.2	0.4	0.5	0.3
Prevailing direction	NW	NW	NW	NW	SW	SW	SW	SW	S	N	NW	W	W
WAVES (FEET)													
01% ≤	0	0	0	0	0	0	0	0	0	0	0	0	0
Mean	3	3	3	2	2	2	1	1	2	3	3	3	2
99% ≤	11	10	10	9	9	8	7	6	8	9	10	10	9
≥ 12 feet (% freq.)	1.0	+	+	+	+	0.0	0.0	+	+	+	+	+	0.1
≥ 20 feet (% freq.)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	+	0.0	0.0	0.0	+

(+) less than 0.05%

TABLE 2 Wind and Wave Data Summary for Offshore Delaware Bay (After Brower, 1972)

ENVIRONMENTAL FACTORS	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANN
WIND SPEED (KNOTS)													
0.1% ≤	3	3	3	3	2	2	2	3	2	3	3	4	3
Mean	15.7	14.6	14.0	12.1	10.6	10.0	9.9	10.3	11.6	13.5	14.6	15.0	12.6
99% ≤	45	44	40	38	31	30	30	31	38	38	43	44	37
Maximum observed (1871-1971)	Winds in excess of 100 knots have been recorded in Hurricane and Northeasters												
≥ 34 knots (% freq.)	4.6	3.0	3.2	1.3	0.5	0.2	0.4	0.6	0.6	2.2	2.3	2.9	1.8
≥ 41 knots (% freq.)	1.1	1.1	1.0	0.1	0.1	0.1	0.2	0.3	0.1	0.8	1.1	1.2	0.6
Prevailing direction	NW	NW	NW	S	S	S	S	S	NE	N	NW	NW	SW
WAVES (FEET)													
01% ≤	0	0	0	1	0	0	0	0	0	0	1	0	<1/4
Mean	4	4	4	3	3	3	3	3	3	4	4	4	4
99% ≤	18	18	15	14	11	10	9	12	17	14	16	15	14
≥ 12 feet (% freq.)	3.7	4.9	3.9	2.0	1.0	0.8	0.2	1.1	1.7	4.0	2.7	3.4	2.5
≥ 20 feet (% freq.)	0.2	0.6	0.5	0.2	0.0	0.0	0.0	0.1	0.2	0.2	0.2	0.2	0.2

(+) less than 0.05%

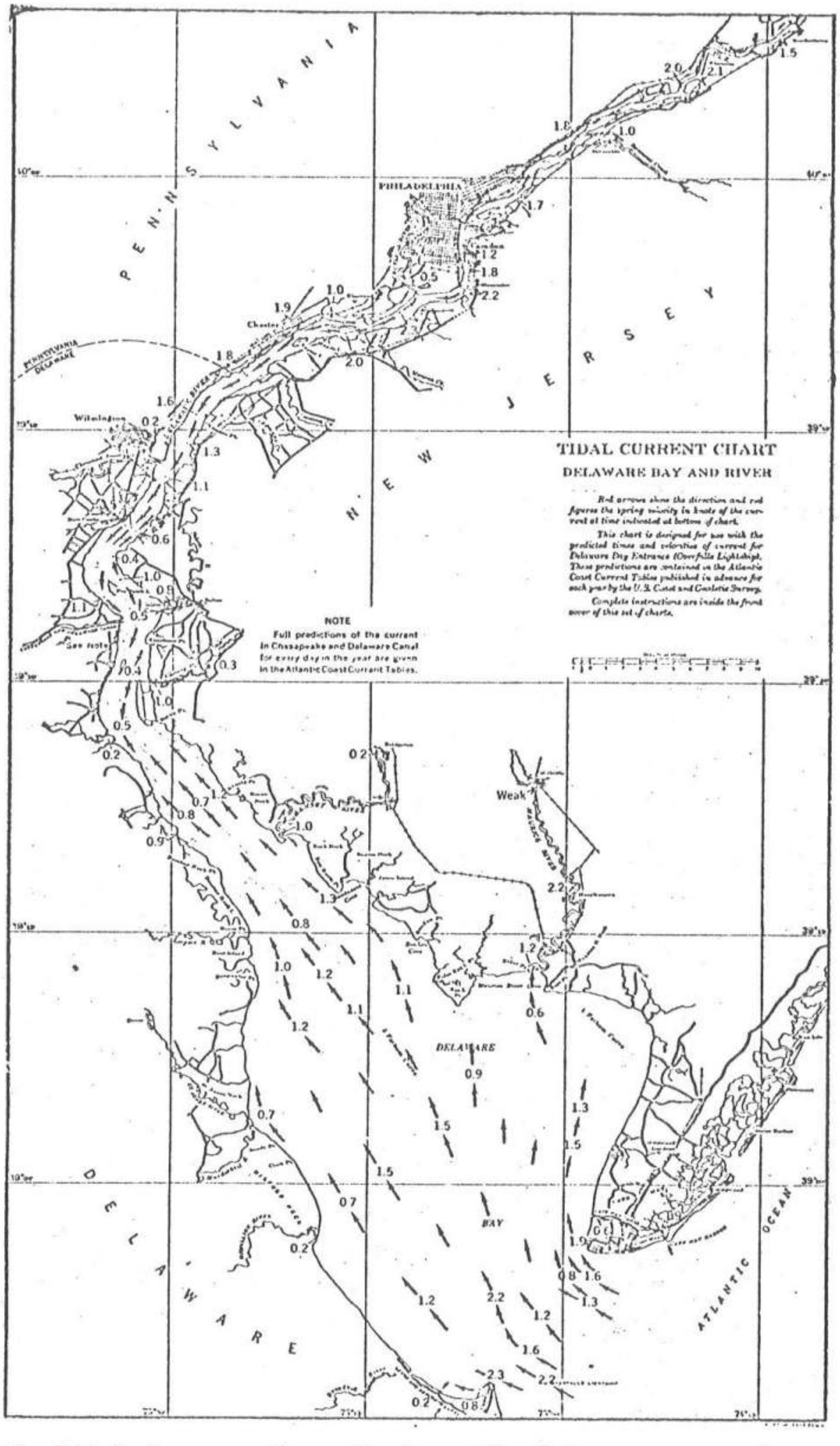

FIGURE 5 Tidal Current Chart-Maximum Flood Current at Delaware Bay Entrance (U.S. Coast and Geodetic Survey, 1960)

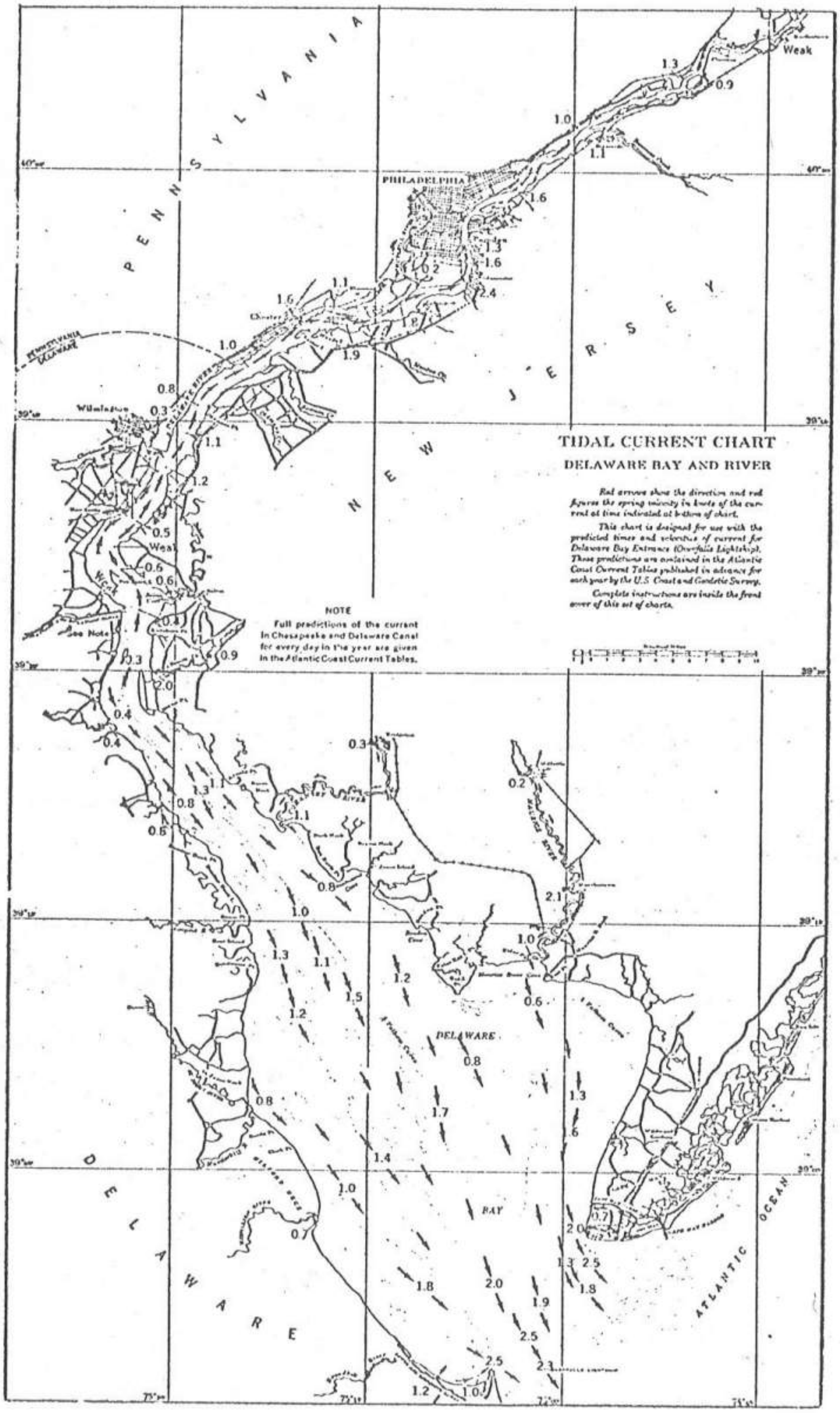

FIGURE 6 Tidal Current Chart-Maximum Ebb Current at Delaware Bay Entrance (U.S. Coast and Geodetic Survey, 1960)

Hurricanes are generated in the tropical Atlantic and generally either follow paths into the Gulf of Mexico or move along the Atlantic coast, their paths being quite erratic and unpredictable. The centers of tropical storms have passed both inland and seaward of the Delaware coast; however, no storm of hurricane intensity has directly hit the area (Figure 7, U.S. Army Corps of Engineers, 1968). Even so, the associated high winds, torrential rains, and accompanying storm surges can be very destructive in a passing hurricane.

"Northeasters" which occur when intense low pressure centers form in the mid- to north-Atlantic coast, are more frequent than hurricanes. These storms cause large storm surges and high waves causing much damage and beach erosion. The most severe "northeaster" in recorded history was the March storm of 1962. The unequaled destructiveness was caused by unabating northeast winds lasting over five consecutive high tides. High tides of Breakwater Harbor reached 7.9 feet above sea level datum or 9.9 feet above mean low water -- the highest recorded level (U.S. Army Corps of Engineers, 1970). Tremendous damage was done to barrier dunes, beaches and shore installations. Flooding, badly eroded beachfront, and washovers were commonplace along the entire coast (see Figures 8 and 9). The damage along the Delaware shore from Pickering Beach to Fenwick Island resulting from the storm totalled 21.9 million dollars (1962 prices). Damage in Lewes was estimated at $1,600,000

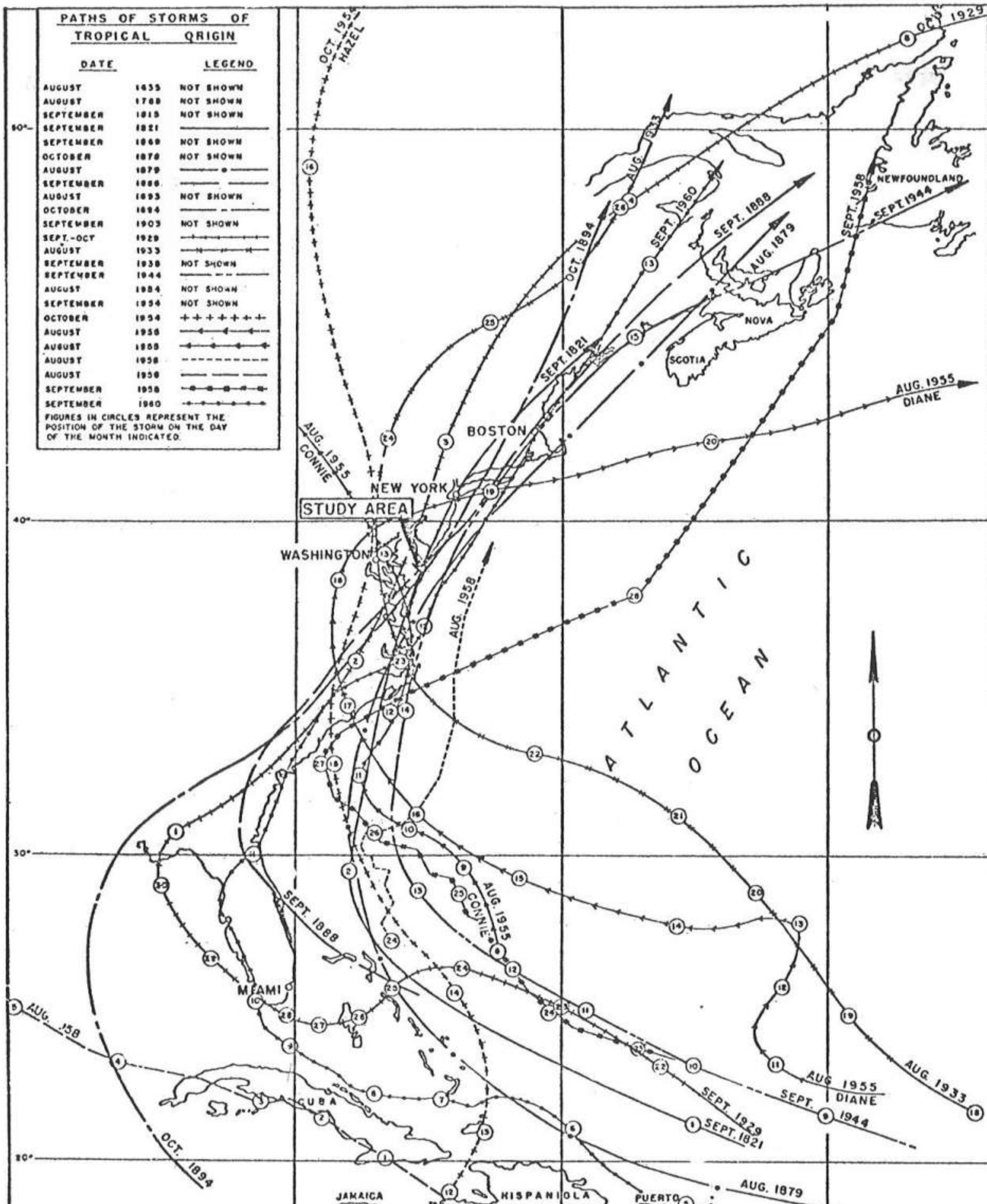

FIGURE 7 Hurricane Storm Tracks Along the Atlantic Coast (U.S. Army Corps of Engineers, 1968)

Figure 8 Flooding in Downtown Lewes During the March, 1962 Storm (Courtesy of the Delaware State Highway Department)

Figure 9 Badly Battered Lewes Beachfront Following the March, 1962 Storm (Courtesy of the Delaware State Highway Department)

CHAPTER II

Early History

2.1 The First Settlement

With the formal organization of the Dutch West India Company in June, 1621, a concentrated effort to explore and claim tracts of land in the new world was made. Two unknown explorers were sent from Holland in 1629 for this project. They found their way to the vicinity of lower Delaware Bay. A patent - registered under the name of Samuel Blommaert - was known to encompass nearly the entire coastal region of Sussex County. Across the bay a similar tract was registered under Samuel Godyn, after whom the early Delaware Bay was named.

Since the Dutch West India Company required colonization of the newly acquired acreage for the fulfillment of the contract, Bloomaert appointed David Pieterson DeVries of Hoorn to command what was later to become known as the "DeVries" expedition. During December, 1630, about 30 Dutchmen set sail from Texel, aboard the ship "Whale" and a sister ship, whose cargos included many implements for capturing whales, which were thought to be plentiful in Godyn's Bay. History notes that DeVries himself was not aboard his own expedition but instead it was captained by Peter Heyes

(Weslager, 1961). Four months later the expedition found itself sailing up the western bank of Godyn's Bay. They passed a sandy point and entered what was recorded as "a fine navigable stream filled with islands, abounding in good oysters" and flowing through a fertile region (Pusey, 1902). They selected a section of highland on the southern bank of the stream, near present-day Lewes (at the approximate site of the DeVries Monument), for their landing site which they named Hoorn, and the stream Hoornkill, in honor of DeVries' home town. The entire settlement became known as Zwaanendael or "Valley of the Swans." Near the landing site they erected a small building surrounded with palisades which they named Oplandt.

Soon thereafter these early settlers met up with the Naticoke or "Tide Water" Indians, a tribe of the great Leni-Lenape Indians whose tribes covered much of the central and eastern portions of North America. Initial friendly meetings soon turned sour, which led to the failure of the first settlement.*

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*As the legend goes, a chief of the Naticoke fancied a piece of tin bearing the coat of arms of the United Province, which was fixed to a pillar erected by the Dutch, probably as a boundary marker. The chief removed the metal with interest in making a new pipe, an act which enraged the settlers. In order to keep peace with the new residents, the Naticoke slew their chief and offered his scalp in forgiveness. Instead of forming reconciliation with the Indians, their chastisement was continued which instilled hostility in the Indians. Friends of the slain chief banded together and sought revenge. The enraged group of Naticoke attacked the settlers while at work in the field leaving no survivors and Fort Oplandt in ruins (Cullen, 1956).

By 1659 the Dutch had reestablished the settlement. Ownership volleyed between the English and the Dutch until 1673 when the English established control. Soon thereafter, under the reign of William Penn, the settlement was renamed Lewes and the county Sussex, after the English town of Lewes in the Shire of Sussex.

2.2 History of Connecting Waterways

At the time of the "DeVries" expedition the geomorphic configuration of the Lewes vicinity was very much different than at present. Figure 11 (after Kraft and Caulk, 1972) shows a geologic interpretation of a map believed to be sketched by a member of the original expedition, in 1631. The main points of interest here are the great width of Bloemaerts Kill, the predecesser of Lewes Creek (present day Lewes and Rehoboth Canal), and the broadly rounded shape of Cape Henlopen. Various early written accounts document the existence of a wide navigable channel. For instance, during the construction of Cape Henlopen Lighthouse in the 1760's, materials for construction were unloaded in Lewes Creek indicating that it was still deep enough for coasting vessels. As time went on Lewes Creek silted to a small, shallow creek. Around 1800, about the time Cape Henlopen started migrating northward, it was noted that Lewes Creek could no longer be used to transport supplies to the lighthouse; it had silted up too much for even the light draft vessels of coastal trade (Kraft and Caulk, 1972).

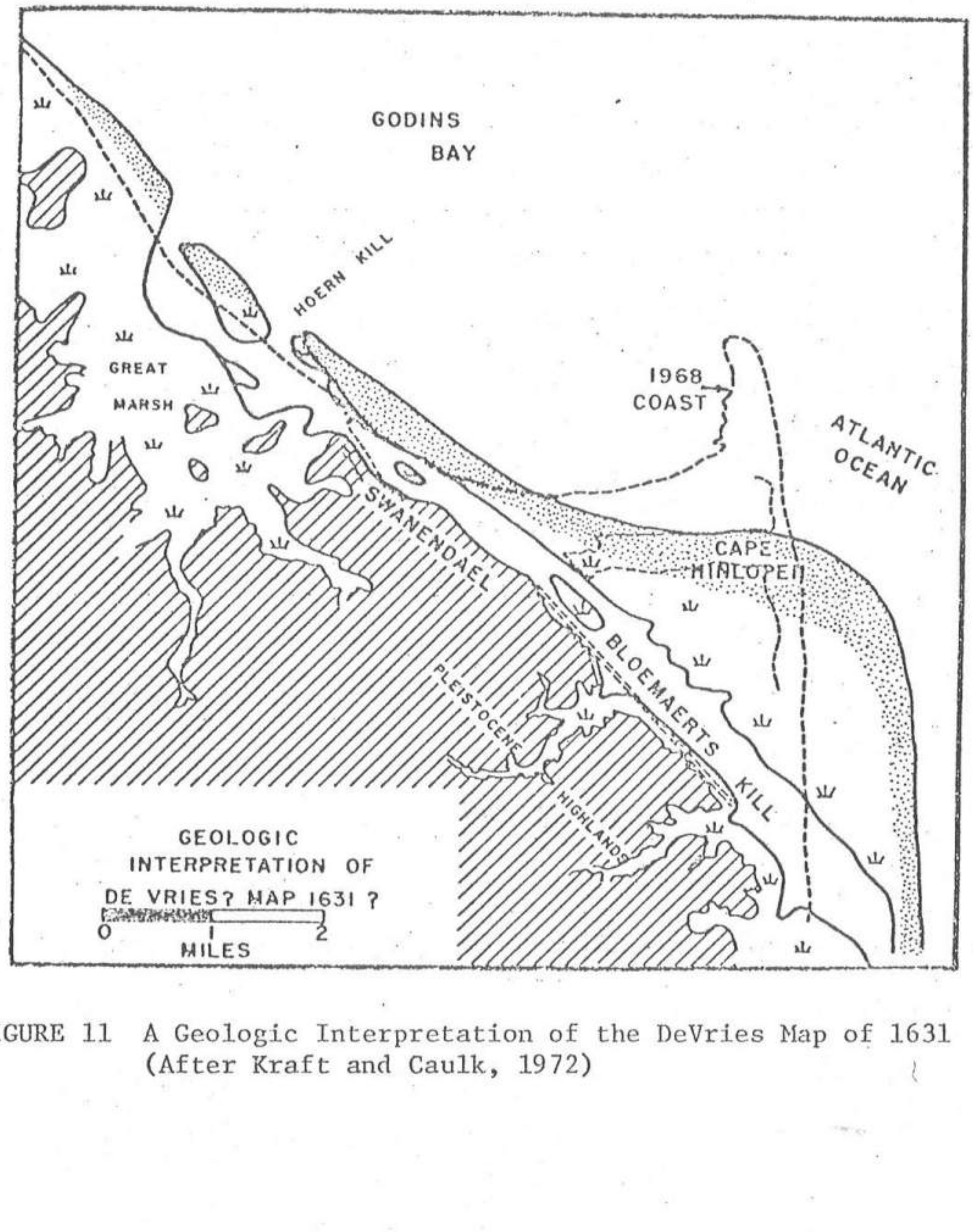

FIGURE 11 A Geologic Interpretation of the DeVries Map of 1631
(After Kraft and Caulk, 1972)

As mentioned previously, the blunted shape of Cape Henlopen allowed easy passage of sand around the Cape from the eroding Atlantic beaches, to serve as a supply for the lower Delaware Bay beaches. This sand was deposited in a long narrow spit known locally as Cape Lewes. The continued growth of Cape Lewes caused the westward deflection of the entrance of Lewes Creek and eventually Broadkill River. Figure 12 is a U.S. Coast and Geodetic Survey Map of 1882 showing this deflection (After Maurmeyer, 1974).

As a result of further deflection of the Lewes Creek and Broadkill River a once deep and navigable entrance was becoming extremely shoal. The inconvenience irritated the local residents, particularly those of Milton (at the headwaters of the Broadkill) since their thriving commercial trades, including a steamship line connecting to Philadelphia, were being hindered. Persistent complaining to Congress by Milton residents resulted in the adoption of a project in 1871 which provided for the improvement of the river channel to a six-foot depth at mean low water and a 40-foot width as well as the establishment of a navigable entrance. It was not until 1890 that actual appropriations were made for improvements. However, the funds were insufficient to afford a new entrance. Required dredging from Milton to (but not including) the mouth were completed by October, 1890. Under a separate project of 1886, it was proposed to form a continuous inland navigation system, much of which would have to be excavated, from Chincoteague Bay,

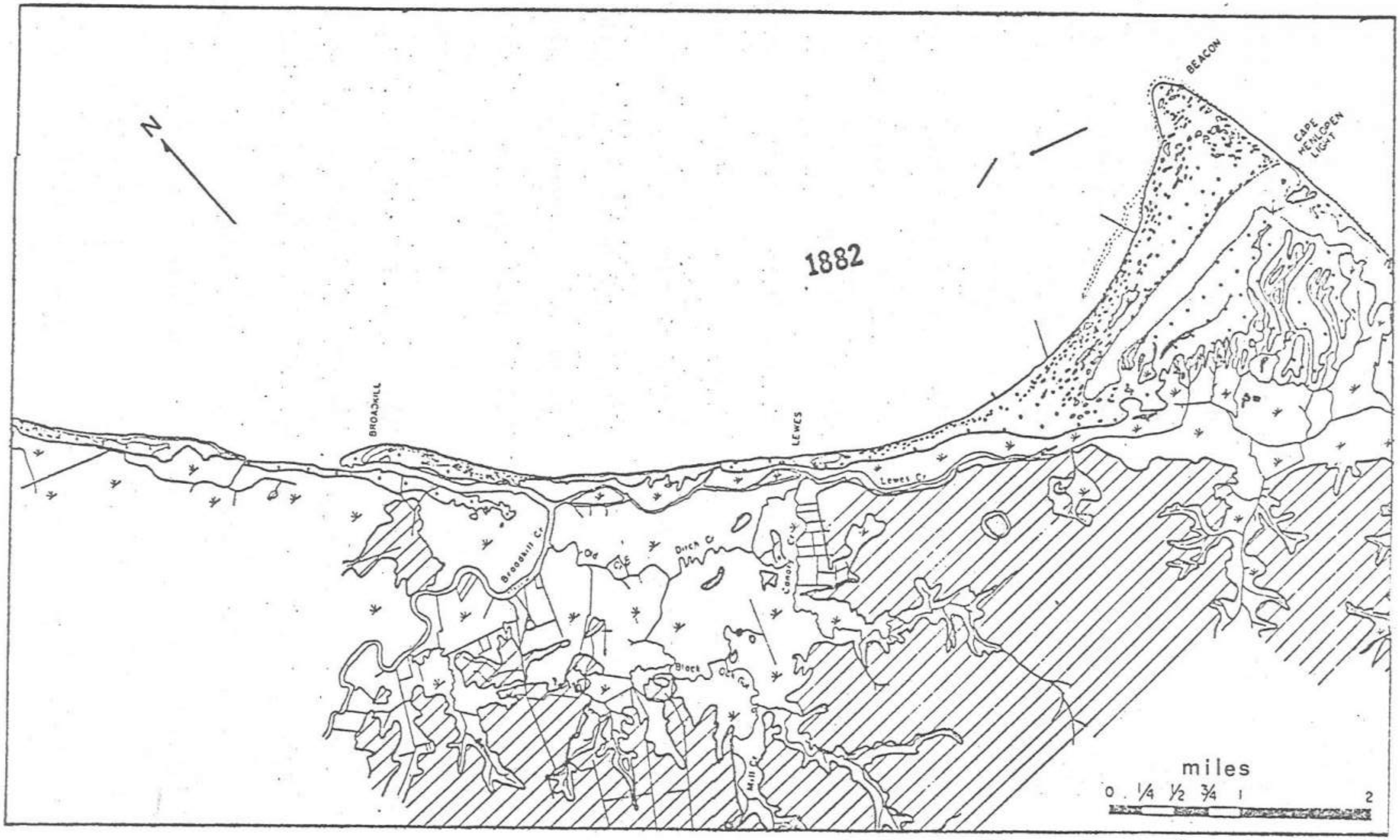

FIGURE 12 A U.S. Coast and Geodetic Map Showing the Westward Deflection of Lewes Creek and Broadkill River Caused by the Dominant Sand Supply Migrating Around Cape Henlopen (After Maurmeyer, 1974)

Virginia, to Delaware Bay, providing an entrance protected by jetties at or near Lewes. During the winter in 1891-92 a preliminary examination was made for the new mouth at the Broadkill entrance. A report following recommended the postponement of any project until the matter of an inlet for the Inland Waterway could be studied, with the idea of combining the two entrances (U.S. Congress, 1892). Again, in 1894, an examination was called for but for the same reason postponement of the project was again recommended (U.S. Congress, 1895). On August 20, 1886, the dredge "Regina" arrived and began work in Rehoboth Bay at the 6-foot contour, working towards Lewes. Meanwhile, steam shovels excavated high land through an area southwest of Rehoboth. All excavation came to a halt when the proposed path of the Inland Waterway encountered the Delaware, Maryland and Virginia (DM&V) Railroad crossing which fed into Rehoboth Beach at that time. An injunction was served upon the United States, restraining them and all others from interferring with their tracks upon the grounds that the U.S. had not acquired title to the right of way for the waterway across the land used by the railroad (U.S. Congress, 1907). Pending the settlement of the bridge question work was resumed north of the railroad crossing. Further legal battles of this type also occurred along the lower section of the Inland Waterway, namely the Assawoman Canal. Meanwhile residents of Lewes and Milton were still waiting for a fully navigable entrance into Delaware Bay. In 1900, an inlet was dug by private interests attempting to re-establish trade with the steamship Newborne from Philadelphia. The excavation took place where Lewes Creek approaches nearest the bay shore approximately one-half mile east of the confluence with Broadkill River. The experimental cut failed shortly thereafter and the Broadkill continued to follow its old course, Lewes Creek followed the new outlet, and the connecting reach shoaled, being bare at low water. This made Lewes Creek an independent stream, but did not offer any benefits since in a short time the littoral drift reduced the new entrance to a mere drain at low water rendering it useless (U.S. Congress, 1906).

Legal problems with bridge crossings continued, particularly in the southern end of the Inland Waterway. Finally, due to lack of progress, the project for the Inland Waterway between Chincoteague Bay, Va., and Delaware Bay was repealed by the act of March 3, 1905. The act also provided for a reexamination of the Broadkill River. The report that followed was favorable, since the Inland Waterway project no longer existed to be considered in conjunction with an improved mouth for the Broadkill. A survey was made and a project submitted for a 6-foot channel from Milton to Delaware Bay, including a new entrance protected by jetties (U.S. Congress, 1907). See Figure 13 for dredged and proposed inlet locations. This project was adopted in the River and Harbor Act of March 2, 1907, with an appropriation for completing the project, equal only to the amount asked for in the report as an initial appropriation

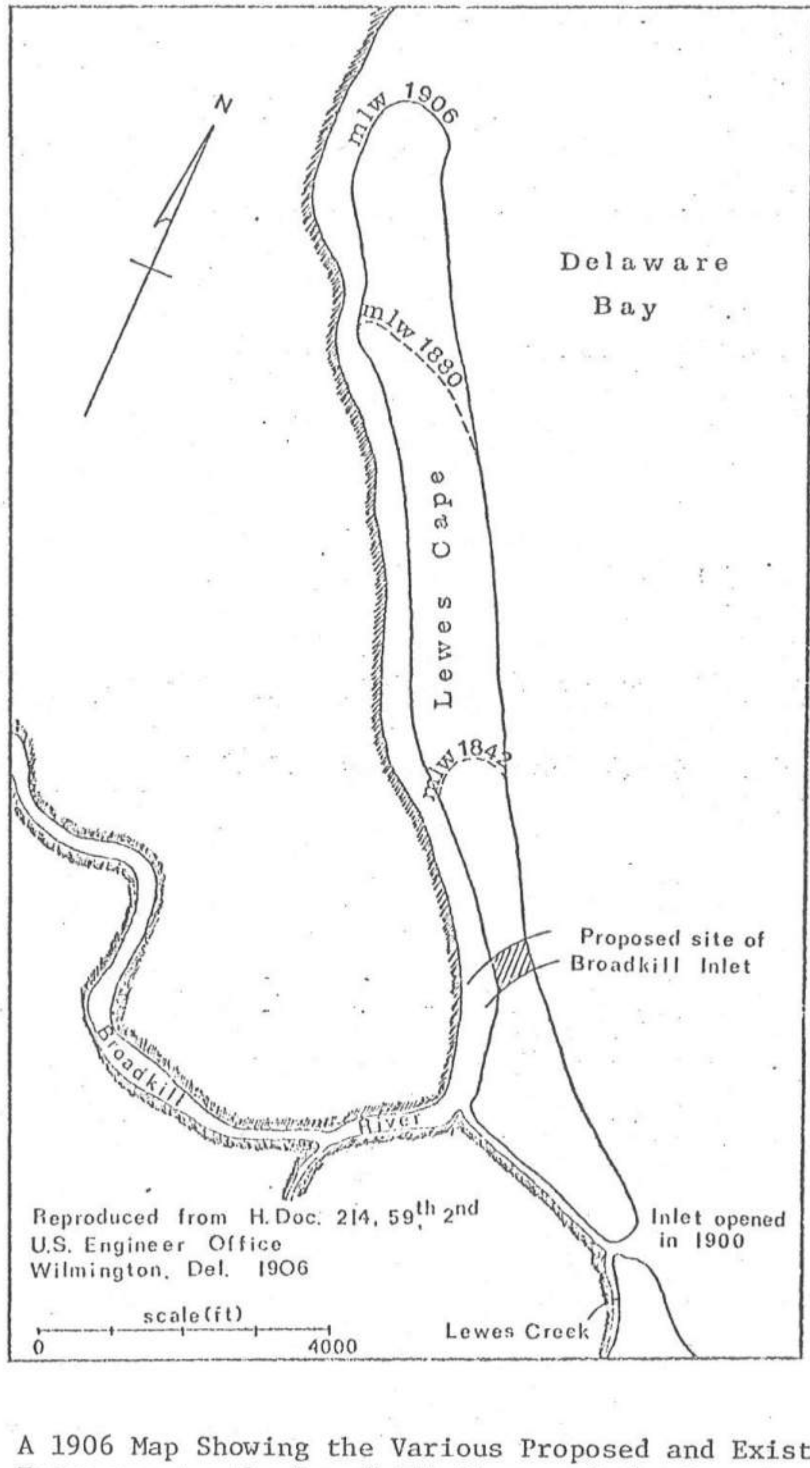

FIGURE 13 A 1906 Map Showing the Various Proposed and Existing Entrances to the Broadkill River and the Growth of Cape Lewes

for beginning the work, totalling $33,330. This covered the dredging of the new entrance across Cape Lewes and the construction of the north jetty, but excluded a south jetty and the dredging of the river to Milton; the balance of the plan for improvement submitted in the report on the survey. The land needed for the cut across Cape Lewes was deeded to the U.S. free of cost. Following the cut (150 feet wide by 6 feet deep) a 1263-foot timber crib-style jetty filled with stone was constructed along the west (north) side extending out to the seven-foot contour in Delaware Bay. The shoreward end of the jetty was extended across the old deflected entrance (Broadkill Sound). A small opening was left in the jetty to allow a small tidal flow circulation in the sound. The project was completed in November, 1908.

In light of the new entrance, and with the award of $37,343.38 to the DM&V Railroad Company and thus construction of a temporary bridge, provisions for a new Inland Waterway project were made under the River and Harbor Act of July 25, 1912. The new project called for an Inland Waterway between Rehoboth Bay and Delaware Bay, along a slightly different route from the previous project of 1886, following Lewes Creek at its northern end and using the new Broadkill Inlet as its northern entrance. An estimated cost of $356,000 included dredging a canal 50 feet wide and 6 feet deep, the purchase of land for right of way, and the construction of two bridges.

While excavation and dredging of the new Inland Waterway project (initially termed the Lewes Canal and later called the Lewes and Rehoboth Canal) was in progress, the new Broadkill Inlet began to shoal. Since the inlet lacked an east (south) jetty, sand driven by east-northeast waves and winds easily entered the mouth. Furthermore, the timber crib jetty design proved inadequate, being quite permeable to sand and easily damaged by moderate wave activity to the extent that repairs (mostly through the addition of more stone) were needed during the years of 1911-13, 1914, 1917, 1920 and 1923.

During the fiscal year 1917, the final stretch of the Inland Waterway from a point 450 feet north of the DM&V Railroad at Rehoboth to a point 700 feet below the crossing was excavated, making the final connection between Rehoboth and Delaware Bays (U.S. Congress, 1917). However, it was not until 10 years later that the Lewes and Rehoboth Canal was completed to project dimensions (U.S. Congress, 1928).

With the completion of the L&R Canal the realization of an adequate northern entrance became that much more important. Continued shoaling of Broadkill Inlet aggravated all interested parties, particularly those from Lewes, who now had an easily navigable canal passing directly through their municipality. At a public hearing Lewes residents claimed that before the mouth of the Broadkill River was shifted 2 miles to its present position, boats drawing 8 feet of water were able to pass in and out of the river at least two-thirds of the time, and that the construction of the present inlet by the Government had deprived local interests of the former adequate outlet channel. Further inconveniences were even registered by Federal Government groups. The U.S. Coast Guard, which then used the Government-owned wharf on the canal at Lewes as a base for mooring their boats, was greatly handicapped in answering distress signals from craft in the bay or ocean, by the limitation of depth in the present entrance. Similar complaints were echoed by the Lighthouse Department which depended on a free entrance to maintain its base operations in Lewes.

In 1935, the Board of Engineers for Rivers and Harbors, concurring in general with the district and division engineers, considered that the provision of a new inlet and the further improvement of the intercoastal waterway near Delaware Bay were justified. Also, the Board considered that the present entrance which was intended to provide a much needed harbor of refuge to small craft could not be maintained in its existing location at a reasonable cost (U.S. Congress, 1935).

With the concurrence of the Board of Engineers for Rivers and Harbors the present Inland Waterway project was modified by the Rivers and Harbors Act of August 30, 1935 to provide an entrance channel at Lewes. The recommended modifications were: to provide an entrance channel near Lewes six feet deep and 200 feet wide, protected by steel sheet pile jetties 500 feet apart and 1,700 feet long; and for the widening of the canal to 100 feet from the Broadkill River to the Savanna Street Bridge at Lewes; and a basin at Lewes, 6 feet deep, 1,200 feet long and up to 375 feet in width, at an estimated cost of $230,000 for new work and $12,000 for annual maintenance. The completion of the project was contingent on total local cooperation in the form of a $60,000 contribution by the State of Delaware for the new inlet, and the furnishing of rights of way and spoil disposal areas without cost as required for new work and subsequent maintenance.

On December 30, 1936 dredging was begun on the new project and 521,828 cubic yards of mud, clay and sand were removed by May 17, 1937, completing the first phase of the project. Construction of the twin steel sheet pile jetties was commenced on February 1, 1937 and completed on October 21, 1937. The stabilization of the inlet rendered the project 85% complete. The remaining portion of the project was the reinforcement of the rubblemound jetties forming the southern entrance into Rehoboth Bay which were initially built in 1903 under the former project of 1886.

The name Roosevelt Inlet was choosen by Mayor David W. Burbage of Lewes for the new entranceway, after then President Roosevelt. At first there was much local opposition to the name, and it was not until it was listed in the charts of the U.S. Lighthouse Service as such that it was accepted (Cullen, 1956). Figure 14 is a map showing all former or proposed inlet locations.

The final work done on the Broadkill Inlet was a dredging operation in 1927, when the L&R Canal was completed to its project specifications. In 1935, it was stated by the Board of Engineers for Rivers and Harbors that the closure of Broadkill Inlet might be desirable to insure an adequate tidal flow through the proposed inlet. No further maintenance work was accomplished, allowing the inlet to close by natural means. The entrance was officially abandoned by the Government by an act approved on June 26, 1953 although the inlet had shoaled closed by the early 1940's (U.S. Congress, 1953).

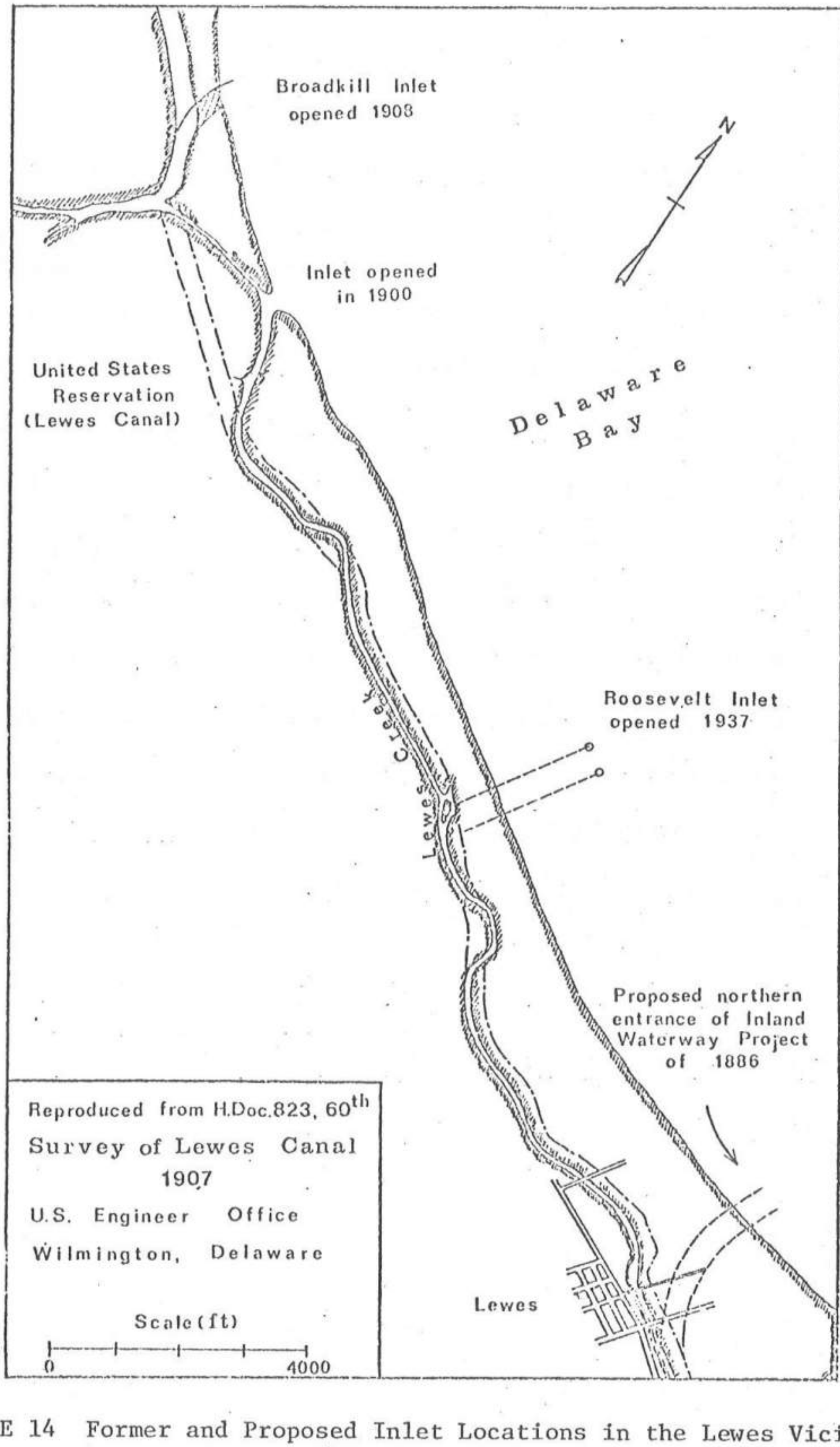

FIGURE 14 Former and Proposed Inlet Locations in the Lewes Vicinity

CHAPTER III

Shoreline Response to Man-Made Structures

3.1 Effect of Offshore Breakwaters

Between 1828 and 1898 the Federal Government constructed a rubblemound breakwater about 5,300 feet long at the entrance to Delaware Bay, forming Breakwater Harbor. In 1901 a second breakwater 8,000 feet in length, about a mile north of Breakwater Harbor, was constructed, forming the Harbor of Refuge. See Figure 3 for the locations of the inner and outer breakwaters. These structures were fabricated for the purpose of providing a shelter for vessels in time of storms. The presence of the breakwaters has significantly affected the adjacent shoreline. The sheltering effect of these structures from waves emanating from the northeast quadrant has resulted in a realignment of the shoreline along Lewes Beach as well as a reversal of the direction of littoral drift. These modifications will be discussed in the following paragraphs.

As mentioned previously, early Cape Henlopen had a broad, blunted cuspate shape allowing the littoral transport of sand to flow around the Cape, serving as a source to Lewes Beach and resulting in the growth of Cape Lewes. Between 1765 and 1831, Cape Henlopen began to grow in the form of a simple spit advancing approximately 1,100 feet to the north. The tip of the Cape is still advancing today towards the north-northwest. Over the past 200 years the average annual accretion rates for the tip of Cape Henlopen have varied between 18 and 60 feet per year (Maurmeyer, 1974). The continued supply of sand feeding the Cape from the eroding Atlantic beaches was deposited in deeper and deeper water. Increasing volumes of sand were needed to provide a base for the advancing spit and thus considerably less sand was transported around its tip. The spit growth combined with the sheltering effect of the breakwaters effectively reduced the supply of sand from the east along Lewes Beach. Sand transport around the Cape through wave refraction reaches a low energy area behind the Cape itself and within the shelter of Breakwater Harbor. At this point little energy is available to continue the transport of sand, and deposition occurs, which contributes to the shoaling of Breakwater Harbor.

With the gradual construction of the breakwaters and the growth of Cape Henlopen a significantly biased wave climate developed along Lewes Beach. With the reduction of the east-northeasterly waves, the waves generated along the axis of Delaware Bay (with the longest fetch) and driven by the prevailing northwesterly winds became the dominant environmental force. As a result, the shoreline reoriented to accommodate the change in wave climate. Figure 15 shows the mean high water shoreline along Lewes Beach from 1843 to 1943. The response to the biased wave climate can be visualized as a counterclockwise rotation of the shoreline around a fictitious axis centered approximately between lines G and H on the map, with erosion occurring in the vicinity of Roosevelt Inlet and accretion along central and eastern Lewes Beach.

Accompanying the reorientation of the shoreline was also the reversal in the direction of net littoral drift. As noted previously, the growth of Cape Lewes (Figure 13) indicated the net littoral drift was from east to west. Soon after the stabilization of Roosevelt Inlet, accretion occurred along the west beach adjacent to the jetty and increased erosion occurred along Lewes Beach. This response indicated a reversal in the dominant drift had occurred with sand depositing on the updrift (west) side of the inlet and erosion occurring on the downdrift (east) side of the inlet. It may be that this reversal was considered temporary or was not understood — borne out by the fact that during the first dredging activity at Roosevelt Inlet in 1942-43, dredged material was placed on the west beach or updrift side!

A study by the Army Corps of Engineers in 1966 reported that accretions were present on the southeast sides of groins along both Broadkill and Slaughter Beaches (U.S. Army Corps of

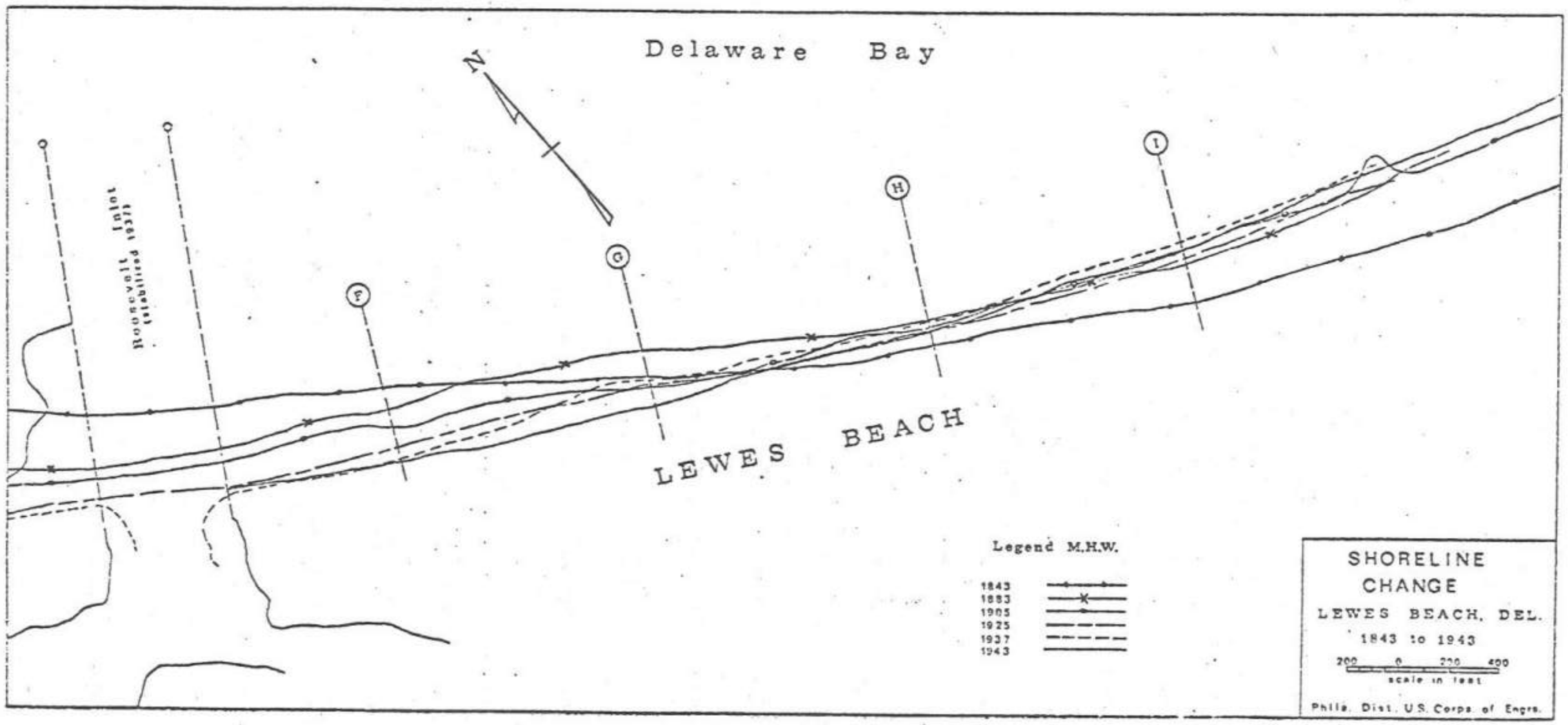

FIGURE 15 Shoreline Change at Lewes Beach (1843 to 1943)

Engineers, 1968). With this fact in mind, and the fact that accretion occurs on the western jetty of Roosevelt Inlet, it is evident that a nodal area must exist within this region, between Broadkill Beach and Roosevelt Inlet. The nodal region, which is characterized by a change in the direction of net longshore transport, is not by any means stationary but varies with changing winds and thus wave climate. For example, an earlier beach erosion control study by the Army Corps of Engineers in 1956 indicated the nodal area to be above Broadkill River, as evidenced by accretions on the northwest sides of the groins at Broadkill Beach (U.S. Army Corps of Engineers, 1956).

3.2 Inlet Stabilization and Resulting Problems

On October 21, 1937, the construction of the inlet jetties was completed. The jetties extended from the bayward end of the cut out to the six-foot contour in Delaware Bay, a total length of 1,700 feet. The tops of the jetties were eight feet above local mean low water except at the shoreward ends which were ten feet. As previously mentioned, the existence of the jetties constituted an impermeable barrier through the surf zone, causing the blockage of sand on the updrift side, and erosion on the downdrift side (Lewes Beach). This is a common problem to most stabilized inlets and it is the job of the coastal engineer to make provisions for artificially by-passing the accumulated sand alleviating the starvation of downdrift beaches. The blockage of littoral drift is only one of the problems present at Roosevelt Inlet.

A second problem was encountered shortly after stabilization; the flanking of the jetties at the shoreward ends, causing erosion of the channel banks. The flanking may be caused by the refraction of waves entering the inlet and striking the channel banks obliquely. Also, the expansion of the flood currents upon exiting from the guides of the jetties cause eddies to flow which may result in scour of the banks. Indian River Inlet suffered from the same problem only on a much larger scale. Channel banks at this location eroded a maximum of 580 feet on the south side and 720 feet on the north side. Bulkheads and rip-rap along the south channel bank were successful in stopping the erosion (Lanan and Dalrymple, 1977). To alleviate the problem at Roosevelt Inlet, rubble was placed along the east bank, resulting in a 400-foot shoreward extension of the east jetty. This work was completed in September, 1944. No work was done along the west bank since the property was not valued as highly on this side and the possibility of the jetty becoming completely flanked on the updrift side was highly remote.

A third and major problem that developed was the corrosion and deterioration of the steel sheet pile. This problem became apparent as early as 1939, just two years after construction. During that year, surveys were made to determine the extent of the problem. No action was taken until 1944 when the outer ends of both jetties were repaired. The River and Harbor Act of March 2, 1945 provided a modification to the existing inlet project. The proposal called for a 10-foot depth from Delaware Bay to the basin at Lewes and for extending the jetties to the 10-foot contour in Delaware Bay. With the anticipation of the jetty extensions no further repairs were done, with the thought that both the extensions and repairs could be combined in a single project. At present, no repairs or extensions of the jetties have been made. The steel pile has deteriorated well beyond its effectiveness, with much of it only visible during low tide. See Figure 16.

Steel sheet pile is normally used for jetty construction where the wave climate is not severe. These structures are very economical and may be constructed quickly. Of course, their major disadvantage is corrosion. Initial exposure of the steel to the salt water causes a thin corrosive coating to form which protects the inner metal. Moderate sand and wave action abrade the protective coating leaving fresh steel exposed. Continued abrasion and corrosion results in complete deterioration of the pile. The life expectency of a pile under such abrasive action is not likely to exceed 10 years compared to 35 years in a more quiescent environment (U.S. Army Corps of Engineers, 1973). It is possible that the severity of the wave climate at Roosevelt Inlet was under-estimated during the initial design. More likely the selection of construction materials was probably economically limited, steel being least expensive and more readily available than other materials.

FIGURE 16 The Remains of the Jetties at Roosevelt Inlet After 40 Years in the Harsh Salt Environment

As a consequence of the condition of the jetties, wave action easily moves sand through, around and over the severed sheet pile. Once inside the inlet channel, the sand is reworked by wave and current action and is usually deposited on lobe-shaped shoals which grow on both the east and west banks. The west lobe is usually larger being on the updrift side of the inlet and hence closer to the dominant source of sand. However, there are times when the eastern lobe is larger. These times presumably occur after periods of heavy northeast winds which remove the sand from Lewes Beach and deposit it within the confines of the inlet. Figure 17 is a photograph showing sand deposition along the east bank of the inlet. This sand had entered the inlet through overtopping of the east jetty under the action of northeast waves. A similar problem is occurring at Ocean City Inlet, Maryland. At this location, the low and permeable inshore portion of the south jetty allows easy passage of sand into the inlet and along the north shore of Assateague Island (Dean and Perlin, 1977).

Once the sand is worked within Roosevelt Inlet, there appears to be no effective mechanism to return the sand to the littoral regime other than dredging. Therefore, in its present condition, the inlet acts as an effective sink to the littoral system, trapping not only the <u>net</u> littoral drift (the difference between east and west littoral drifts) but more likely the <u>gross</u> littoral drift (the sum of the east and west littoral drifts).

FIGURE 17 The Deposition of a Sand Mound as a Result of Overtopping the East Jetty (Right-Hand Side of Photo)

3.3 Aerial Photo History

Through the use of aerial photographs an overview of the shoreline response to inlet stabilization and pertinent processes occurring in the area can readily be observed. Figure 18, taken on July 5, 1938 shows both the Broadkill (upper left) and Roosevelt Inlets open. The narrow waterway in the extreme upper left of the photo paralleling the shoreline is the Broadkill Sound. This sound was once the outlet of the Broadkill River (left center) and the Lewes Creek before the inlet was cut and partially stabilized. The wide sand buildup along the western side of the Broadkill jetty (present day, Broadkill Beach) and the exposed marsh outcrop along the tip of the eastern bank indicate that the dominant drift is from west to east. The shoreline offset around the jetty also indicates this. The offset seems quite large; however, much of it is due to the previous curvature of the shoreline at this location. Shoals are visible along the eastern side of the inlet, which eventually were responsbile for its closure. Along the west beach (Beach Plum Island) many washover fans are present, indicative of a highly erosive area. The circular-shaped sand mounds are dredge spoils deposited during the widening of the lower Broadkill. Of special interest is the bare shoal located offshore in the Delaware Bay. Features of this nature are fairly common to the vicinity and are indicative of mild offshore slopes and low-energy coastlines. Shoals of this type, however, are quite unstable and migratory in nature.

Figure 18 Aerial Photo Showing Broadkill Inlet and Roosevelt Inlet on July 5, 1938 (U.S. Deparment of Agriculture)

Figure 19 is a continuation of Figure 18. The prominent feature of this photo is the essentially straight shoreline interrupted by the presence of Roosevelt Inlet. At the time of this photo, approximately eight months after stabilization, the shoreline had not yet visibly responded to the blockage of littoral drift by the jetties. A small linear offshore shoal is visible just to the west of the inlet. Many dredge spoils are present along the north side of the Lewes and Rehoboth Canal and basin.

On July 22, 1954, Figure 20, the buildup on the updrift side and the accompanying erosion on the downdrift side of Roosevelt Inlet could be seen. A wreck approximately 2,000 feet west of the inlet has caused a bulge in the shoreline. The sand buildup to the west of the wreck and erosion to the east is further evidence of the west-to-east littoral drift in this vicinity. In the extreme upper left of the photo the complete closure of the Broadkill Inlet is evident. The somewhat irregular shoreline along Lewes Beach has resulted from the construction of six groins between 1948 and 1950 in an effort to alleviate the downdrift erosion problem.

Figure 21 shows Roosevelt Inlet on November 12, 1960. The pervasive deterioration of the jetties has resulted in lobe-like shoals forming in the throat of the inlet. Note that the eastern lobe is larger. A further point of interest is the alignment of the shoreline along the west beach. In the figure, the updrift shoreline is nearly straight and at right angles to the jetty. In Figure 20,

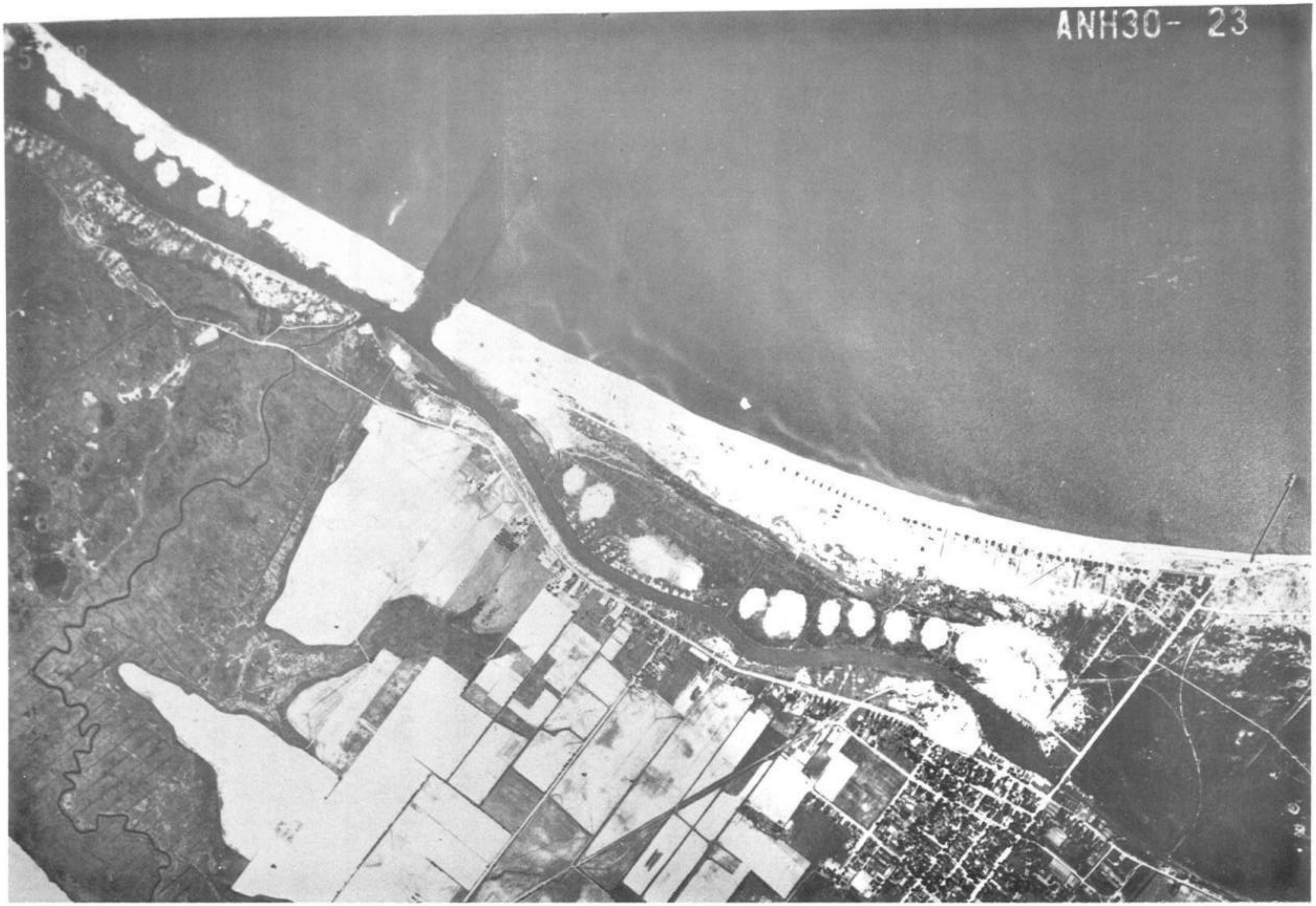

Figure 19 Aerial Photo Showing Roosevelt Inlet Shortly After Stabilization on July 5, 1938 (U.S. Department of Agriculture)

Figure 20 Aerial Photo Showing the Stabilization Effect of Roosevelt Inlet and the Closure of Broadkill Inlet, July 22, 1954 (U.S. Department of Agriculture)

Figure 21 Aerial Photo of Roosevelt Inlet and Lewes Beach, November 12, 1960 (U.S. Department of Agriculture)

the accretion is in the form of a fillet, containing more trapped sand. This indicates that the jetty capacity for retaining sand has peaked and is decreasing as the jetties continue to deteriorate.

Figure 22 was taken on May 19, 1968. The western barrier beach has suffered severe erosion. Comparison with Figure 18 shows that the barrier has become significantly thinner particularly along the western reaches near the point of eastward deflection of the Broadkill River. At one point in this reach, a washover is seen to extend completely across the barrier. This location is near the point that an inlet was dug by private interests in 1900 (see Figure 13). Shoals are again present in the inlet, this time the western lobe being larger. The size and shape of the shoals have been influenced by numerous maintenance dredging activities.

A continuation of Figure 22 is shown in Figure 23. In the right-hand corner of the photo is the Cape May-Lewes Ferry Terminal protected by a 2,550 foot hooked breakwater. Accretion along the eastside of the breakwater is further indication of the easterly littoral drift along Lewes Beach. This breakwater was constructed in 1964 with stone removed from three ice breakers located at the western end of the outer breakwater in Delaware Bay (U.S. Army Corps of Engineers, 1970). The poor condition of the jetties is also evident from this photo. Compare the distinctness of the jetties in Figure 19 with that of the present figure.

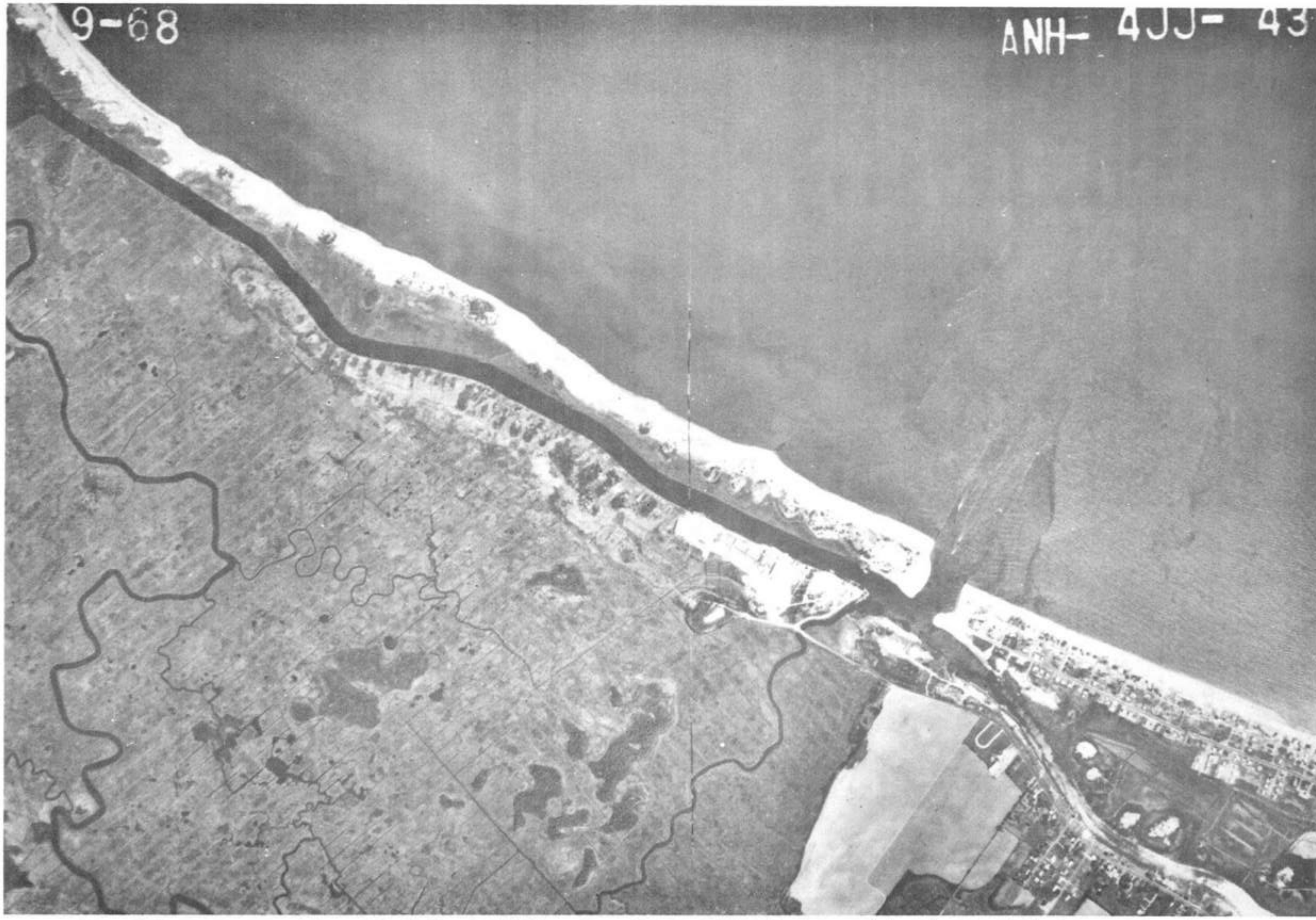

Figure 22 Aerial Photo of Roosevelt Inlet on May 19, 1968 (U.S. Department of Agriculture)

Figure 23 Aerial Photo of Roosevelt Inlet, Lewes Beach and the Ferry Breakwater on May 19, 1968
(U.S. Department of Agriculture)

Figure 24 is an enlargement of a high altitude photograph taken on April 17, 1975. This shot was taken approximately only four months after dredging had occurred in the inlet. A new lobe has already begun to form along the west bank. The College of Marine Studies Harbor is seen at the rear of the inlet entrance. This new harbor was dredged July through October, 1974.

3.4 Erosion Rates

For comparative purposes the aerial photographs presented in the preceding section were overlayed and a large-scale map was constructed, shown in Figure 25. Specific locations were selected along which shoreline changes were measured. These locations are lettered A through K on the map. Locations F through I were duplicated on Figure 15 in order to ascertain both post- and pre-stabilization erosion rates. Since no adequate map or aerial photos prior to the inlet stabilization were available for the west beach, beach profiles presented in the U.S. Army Corps of Engineers 1956 report were used. These profiles corresponded to locations A, C, and E. The measured shoreline change and erosion rates for all locations including along the east and west jetties are presented in Table 3.

The measurements indicate that high rates of erosion are present along the west beach, locations A through E. Since the stabilization of the inlet, the erosion has slowed down significantly in the vicinity of the inlet. Location E averaged a loss of 10 feet per year between 1843 and 1954; however, no net change occurred

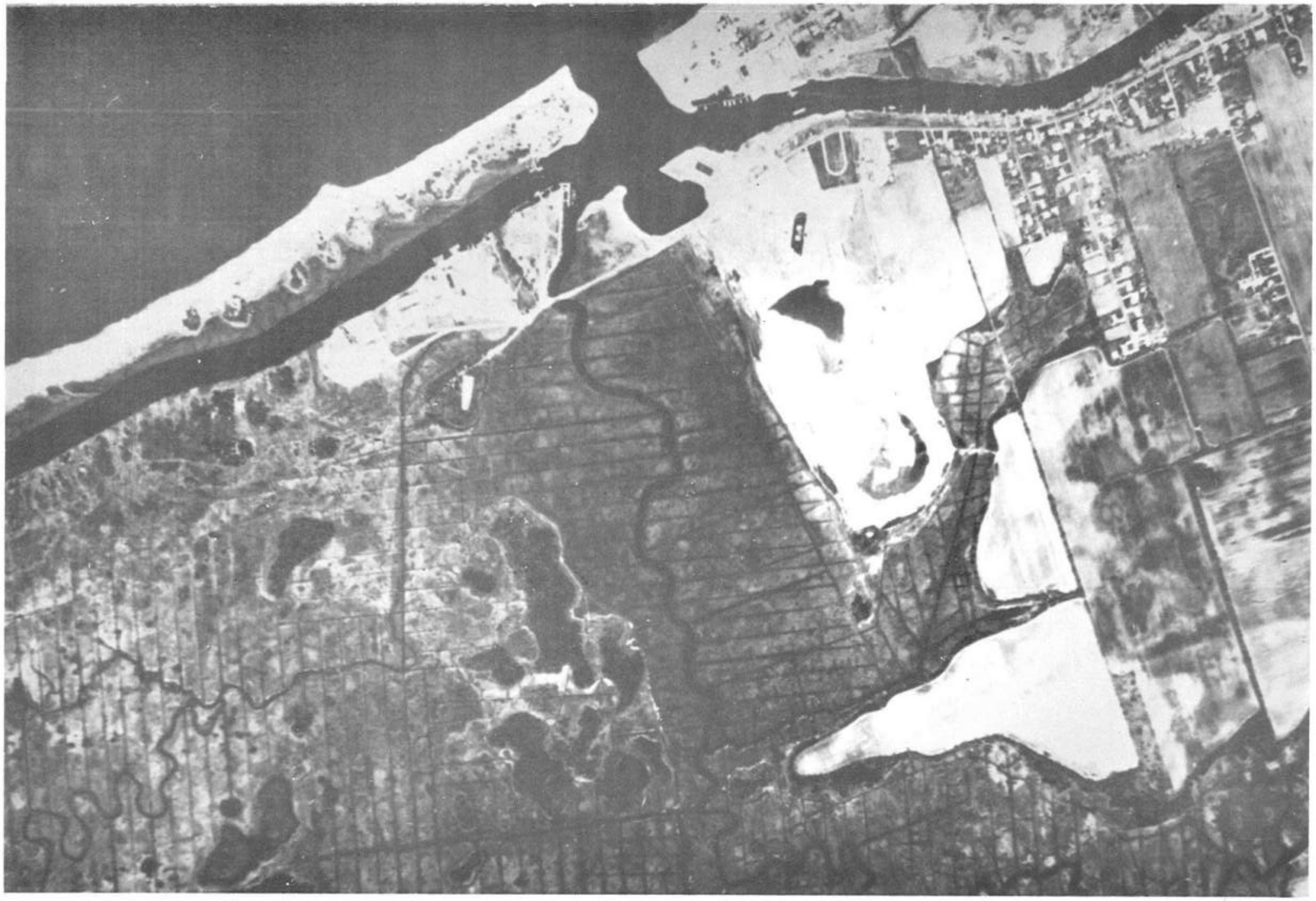

Figure 24 Aerial Photo of Roosevelt Inlet and College of Marine Studies Harbor, April 17, 1975 (National Aeronautic and Space Administration)

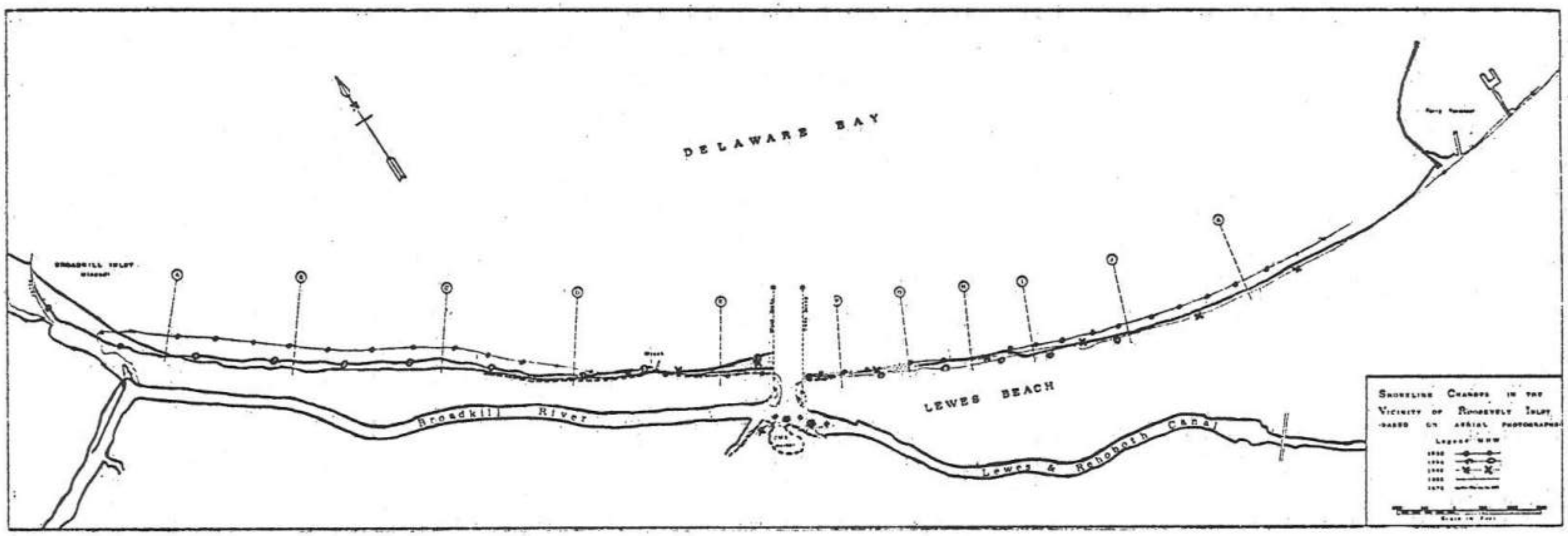

FIGURE 25 Shoreline Changes in the Vicinity of Roosevelt Inlet 1938-1975 (Based on Aerial Photographs)

TABLE 3 Rate of Shoreline Change in the Vicinity of Roosevelt Inlet

Location	Interval	Net Change (ft)	Rate (ft/yr)	Average Rate (ft/yr)	Source
A	1903 - 1914	-220	-20	-11	Army Corps Profile
	1914 - 1954	-360	- 9		
	1938 - 1954	-330	-55	-10	Figure 25
	1954 - 1968	+ 40	+ 3		
B	1938 - 1954	-260	-16	-12	Figure 25
	1954 - 1968	-110	- 8		
C	1843 - 1903	-645	-11	-9	Army Corps Profile
	1903 - 1914	-275	-25		
	1914 - 1954	-130	- 3		
	1938 - 1954	-200	-33	-12	Figure 25
	1954 - 1968	-165	-12		
D	1938 - 1954	-110	- 7	-6	Figure 25
	1954 - 1968	- 50	- 4		
	1968 - 1975	- 45	- 6		
E	1843 - 1884	-755	-18	-10	Army Corps Profile
	1884 - 1903	-160	- 8		
	1903 - 1914	-165	-15		
	1914 - 1954	- 40	- 1		
	1938 - 1954	+140	+9	0	Figure 25
	1954 - 1960	- 25	-4		
	1960 - 1968	- 60	-8		
	1968 - 1975	- 55	-7		

(CONTINUED)

TABLE 3 (CONTINUED)

Location	Interval	Net Change (ft)	Rate (ft/yr)	Average Rate (ft/yr)	Source
West Jetty	1938 - 1954	+345	+58	+1	Figure 25
	1954 - 1960	-150	-25		
	1960 - 1968	-110	-14		
	1968 - 1975	- 50	- 7		
East Jetty	1843 - 1883	-165	-4	-4*	Figure 15
	1883 - 1905	- 50	-2	-3	
	1905 - 1925	-100	-5		
	1925 - 1937	- 25	-2		
	1937 - 1943	+ 20	+3		
	1938 - 1954	- 40	-7	-3	Figure 25
	1954 - 1960	- 50	-8		
	1960 - 1968	- 40	-5		
	1968 - 1975	+ 30	+4		
F	1843 - 1883	- 90	-2	-3*	Figure 15
	1883 - 1905	- 75	-3	-3	
	1905 - 1925	- 80	-4		
	1025 - 1937	- 50	-4		
	1937 - 1943	- 15	-3		
	1938 - 1954	- 40	-7	-1	Figure 25
	1954 - 1960	+ 10	+2		
	1960 - 1968	- 60	-8		
	1968 - 1975	+ 40	+6		

(CONTINUED)

TABLE 3 (CONTINUED)

Location	Interval	Net Change (ft)	Rate (ft/yr)	Average Rate (ft/yr)	Source
G	1843 - 1883	+105	+3	+2*	Figure 15
	1883 - 1905	+ 15	+1	-1	
	1905 - 1925	+ 20	+1		
	1925 - 1937	+ 20	+2		
	1937 - 1943	-105	-18		
	1938 - 1954	-125	-21	-3	Figure 25
	1954 - 1960	+ 85	+14		
	1960 - 1968	- 35	-19		
H	1843 - 1883	+115	+3	+1*	Figure 15
	1883 - 1905	-100	-5	0	
	1905 - 1925	- 15	-1		
	1925 - 1937	+ 50	+4		
	1937 - 1943	- 55	-9		
	1938 - 1954	-120	-20	0	Figure 25
	1954 - 1960	+ 85	+14		
	1960 - 1968	+ 25	+ 3		
I	1843 - 1883	+160	+4	+3*	Figure 15
	1883 - 1905	+ 60	+3	+2	
	1905 - 1925	- 40	-2		
	1925 - 1937	+ 70	+6		
	1937 - 1943	- 35	-6		
	1938 - 1954	-155	-26	-5	Figure 25
	1954 - 1960	+ 80	+13		
	1960 - 1968	- 60	- 8		

(CONTINUED)

TABLE 3 (CONTINUED)

Location	Interval	Net Change (ft)	Rate (ft/yr)	Average Rate (ft/yr)	Source
J	1938 - 1954 1954 - 1960 1960 - 1968	-200 + 25 + 10	-33 +4 +1	-6	Figure 25
K	1938 - 1960 1960 - 1968	-235 + 65	-11 + 8	-6	Figure 25

* Prestabilization rate, 1843 - 1937.

between 1938 and 1975. Both the west jetty and line E show strong accretionary periods between 1938 and 1954, followed by periods of continued erosion. As stated previously, the reason for the accretion followed by erosion is believed to be the deterioration of the west jetty. It is not possible to pinpoint the year when the maximum buildup occurred since the shoreline data are spaced over periods of 6 years or longer.

Along Lewes Beach the shoreline change has been significantly influenced by the numerous beach nourishment projects and the construction of groins (see Section 6.1). Also, the sheltering effect of the offshore breakwaters causing a shoreline reorientation, followed by the downdrift erosion caused by the inlet, has resulted in many fluctuations of the shoreline. The east jetty line has experienced a relatively constant rate of recession averaging approximately three feet per year. Location H has shown alternating periods of erosion and accretion resulting in little net change over all years. Lines I, J and K all have experienced a phenomenally large recession between 1938 and 1954 followed by accretionary periods.

The locations of lines F, H and I were selected to correspond with actual measured profiles maintained by the U.S. Army Corps of Engineers (1968), to serve as a source for comparison. The actual field data show that profile F has experienced a relatively constant rate of recession averaging about 4 feet per year between

1843 and 1954. Profiles H and I had advanced seaward about 230 feet per year during that same time span. Between 1954 and 1964 profiles F and H advanced seaward averaging 11 and 8 feet per year respectively in response to beach fill. The total net change between 1843 and 1964 was an average of three feet per year recession of the mean high water line at profile F. A negligible net change occurred at profile H. The time intervals of the field data do not correspond exactly with those contained in Figure 25. However, the actual measured rates by the U.S. Army Corps of Engineers and those listed in Table 3 generally show good agreement for locations F, H and I.

Overall, generally higher erosion rates are experienced along the west beach than along Lewes Beach. Erosion has been present at all locations along the west beach at all times except at locations E and the west jetty which have shown periods of accretion following the inlet stabilization. Along Lewes Beach the shoreline movement has been quite variable. The construction of the offshore breakwaters, the growth of Cape Henlopen, the stabilization of Roosevelt Inlet, the construction of groins, and numerous beach nourishment projects have contributed to the shoreline fluctuations along this reach.

CHAPTER IV

Field Studies

During the course of the study, four field trips were conducted in order to help in understanding the sediment movement patterns and document the present trends in the vicinity of Roosevelt Inlet. Two hydrographic surveys were made: one of the offshore bathymetry, while the other was conducted inside the inlet. During both of the surveys beach profile measurements were made along Lewes and the west beach. A third field study was a sand tracer experiment performed to document, at least qualitatively, the sand movement in the vicinity of the inlet. During the fourth field trip current and tide measurements were made both in the inlet and the Lewes and Rehoboth Canal. These will be discussed in Chapter VII.

4.1 Offshore Hydrographic Survey

Prior to the actual surveys a semipermanent baseline was established along Lewes and the west beach as well as the west bank of the inlet. Wooden stakes were implanted every 200 feet covering a distance of 1,000 feet from the inlet, along both the east and west beach fronts. Three stakes were driven each 100 feet apart along the west bank of the inlet. The east baseline had to be partially re-established several times due to heavy erosion along this reach. In one instance two of the three-foot stakes driven two feet into the sand forming the center section of the east baseline were found washed up on the western shoal inside the inlet.

The water depth measurements were obtained using a Raytheon recording fathometer. The boat was maintained on a straight traverse perpendicular to each baseline station with the use of range poles set on the beach and careful navigation. The location of the boat was monitored by a transit at a known location. Radios were used for communication between the beach and the boat. Sounding locations within the inlet were established through triangulation, using two transits. This method provided for more freedom of movement for the boat.

The mean water surface elevation was recorded during the surveys by a tide gauge maintained in the College of Marine Studies Harbor. No effort was made to actually record the mean water level in Delaware Bay during the offshore measurements. It was assumed that the head drop through the inlet was small and within the accuracy of the measurements as the tidal velocities are quite small.

The raw sounding data were related to the local mean low water datum. This datum, which is mean low water at the mouth of the Broadkill River, has been used consistently by the Army Corps of Engineers on all past charts of the area. It was established by tide gauge recordings at the U.S. Coast Guard Facility at the junction of Canary Creek and Broadkill River. This datum plane is 2.11 feet below the Nátional Geodetic Vertical Datum (formerly Sea Level Datum of 1929) and 0.42 feet below mean low water at Breakwater Harbor.

The results of the first survey, accomplished on July 3 and August 2, 1976 are shown in Figure 26. The survey indicates a very irregular offshore bathymetry is present. This is indicative of low-energy coastlines on which normally present wave energy is insufficient to form parallel contours with the existing shoreline. Further evidence of irregular bathymetry is shown in Figures 27 and 28. Figure 27 is a photo showing several sand waves (transverse bars) in the nearshore zone. Figure 28 is a closeup of a similar feature taken almost a year later. No detailed measurements of such features have been made; however, they are not stationary and are believed to be migrating in the direction of dominant littoral drift. There is no doubt that these features cause temporary fluctuations of the adjacent shoreline.

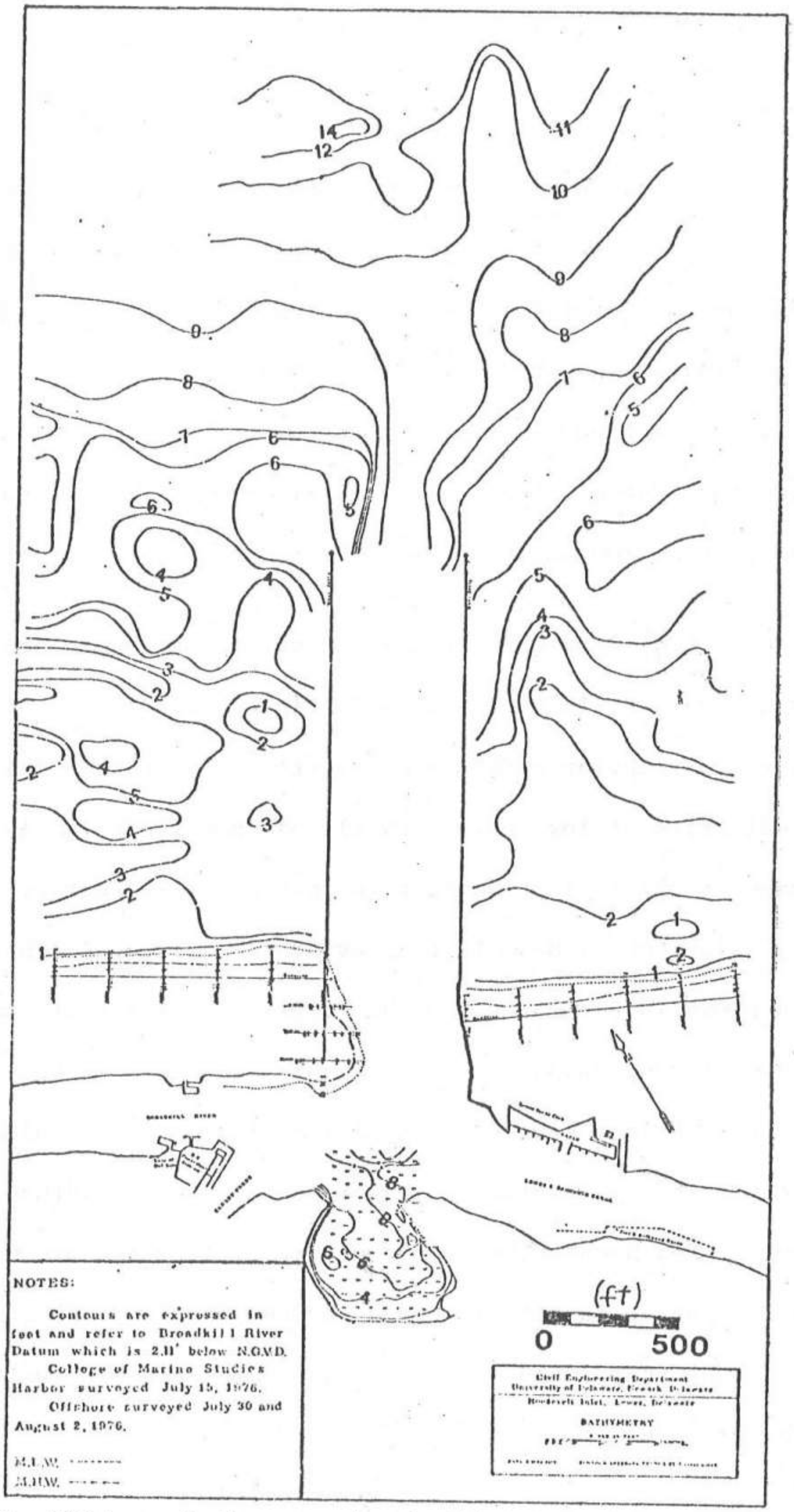

FIGURE 26 Offshore Bathymetry in the Vicinity of Roosevelt Inlet, July 30-August 2, 1976

FIGURE 27 Sand Waves in the Vicinity of Roosevelt Inlet, July, 1976

FIGURE 28 A Sand Wave Along the West Beach, April, 1977

The survey also shows a longshore bar present along the west beach approximately 1,000 feet from shore. Near the west jetty the bar is only one foot below mean low water. Waves usually break on this bar at low tide. On one occasion (during an exceptionally low tide) waves were seen breaking <u>inside</u> the inlet near the west jetty indicating that the eastern end of the bar had spilled over the jetty into the inlet.

Offshore of the inlet mouth no major ebb tidal shoal was found within the survey limits, despite the presence of this type shoal at most inlets. The general condition of the jetties may be partly responsible for this,allowing the ebb tidal flow to diffuse out rather than "jet" out. In this regard the currents present in the inlet are generally too weak to transport the bulk of the sand entering the inlet throat to form an ebb tidal shoal.

The alignment of the contours indicates that the ebb tidal currents may have some effect in shaping the bathymetry toward the north-northeast. Inspection of Figure 26 shows that contours extending from the tip of the east jetty appear to follow a north-northeasterly trend. This alignment is presumably caused by the easterly deflection of ebb tidal currents from the inlet encountering southeasterly tidal currents of Delaware Bay, also on ebb (refer to Figure 20). This phenomenon also occurs at Indian River Inlet.

The ebb flow from Delaware Bay produces a general southerly flow along the Atlantic coast of Delaware which coincides with the ebb flow from Indian River Inlet, producing a southerly deflection of the ebb tidal plume (Lanan and Dalrymple, 1977).

Also contained in Figure 26 are the results of a survey completed on July 15, 1976 of the College of Marine Studies Harbor (Wethe, 1976). The mean depth present in the harbor at that time was six to seven feet below mean low water (Broadkill River datum). Following the initial excavation of the harbor (October, 1974) the mean depth was 10 feet, indicating shoaling rates greater than 1.5 feet per year. This excessive shoaling has resulted in scheduling problems for the research vessel, RV Cape Henlopen, moored in the harbor. The RV Cape Henlopen has a maximum propeller draft of nine feet and cannot enter or depart from the harbor at low tide. Bottom sediment samples from within the harbor indicate that the generation of shoaling may be from the Broadkill River (Wethe, 1977). These sediments, mostly fine silts, clays, and organics, stay in suspension until entering the harbor on flood tide. Once within the confines of the harbor, a lower-energy regime, these fine particulates settle out of suspension. Some deposition within the harbor has also resulted from the erosion of its unprotected western banks.

4.2 Inlet Survey

The second hydrographic survey was concluded on April 7, 1977, covering the inside of Roosevelt Inlet. The results of this survey are presented in Figure 29. Again, the contours refer to local mean low water. The two most predominate features evidenced by the survey are the extensive western sand lobe and the very deep scour hole at the confluence of the Broadkill River and the L&R Canal. The deepest measurement actually recorded in surveying across the hole was 25.4 feet. Comparison with previous surveys done by the Army Corps of Engineers reveals that this scour hole is a fairly permanent feature although it varies slightly in depth and location, undoubtedly influenced by dredging activities. It is speculated that the cause of the hole is the turbulence generated by the convergence of the flow issuing from the Broadkill and the Lewes and Rehoboth Canal during ebb.

On January 22, 1975 the latest dredging operation was completed. The 200-foot inlet channel was dredged to an over-depth of 19 feet. It is easily seen from Figure 29 that much shoaling has occurred, reaching a maximum depth of only 12 feet within the outer end of the inlet. Direct comparison of this survey with a survey by the Army Corps of Engineers immediately following the dredging, reveals a total of 45,860 yd^3 of sand had been deposited within the inlet, below mean low water. This

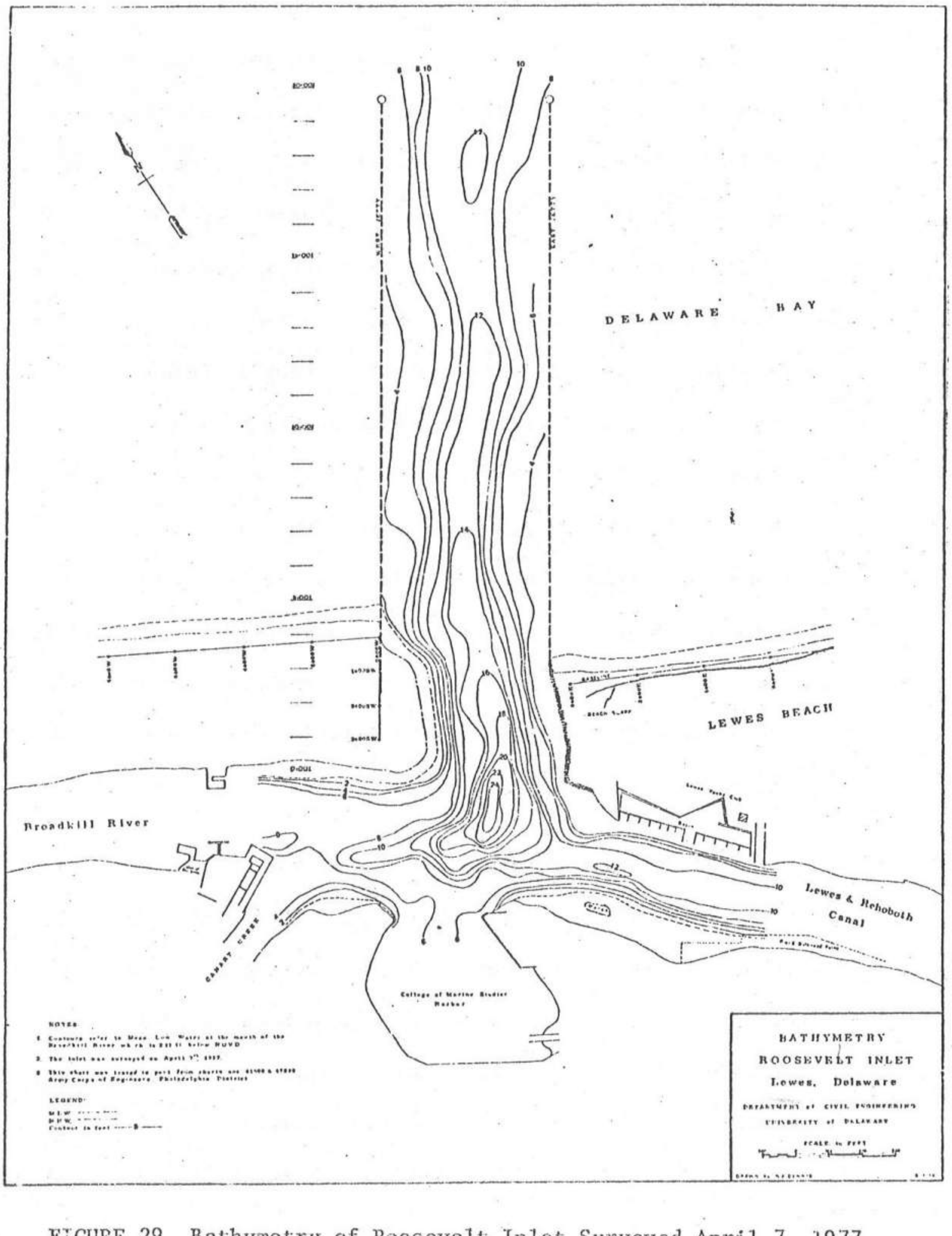

FIGURE 29 Bathymetry of Roosevelt Inlet Surveyed April 7, 1977

reduces to a yearly rate of 21,130 yd^3 which is nearly twice the average shoaling rate! (Section 5.2). The maintenance dredging of 1975 marked the first time the inlet was dredged to an overdepth of 19 feet. Previous to this (and after 1945) the channel was maintained by dredging to an overdepth of 11 feet below mean low water. The increased dredging depth and the continued deterioration of the jetties are responsible for the increased shoaling rate. As previously noted, the jetties were only constructed out to the six-foot contour in Delaware Bay (refer to Figure 26) and at this time only a six-foot depth was authorized. The authorized depth has increased over the years but the jetty length has remained the same. The sand now approaching the inlet passes rather freely by the remains of the jetties and encounters a drop from six to 19 feet in the inlet channel. This increase in the slope of the channel sides enhances the ability of the inlet to trap sand resulting in a higher shoaling rate.

Two detailed surveys of the west lobe were completed on July 30, 1976 and April 6, 1977, along with the hydrographic surveys. The results of these surveys are shown overlain in Figure 30. The contours from each survey were planimetered in order to estimate the volume of sand contained within the shoal. Approximately 6,900 yd^3 of sand were contained in the lobe following the first survey (above M.L.W.) and 8,700 yd^3

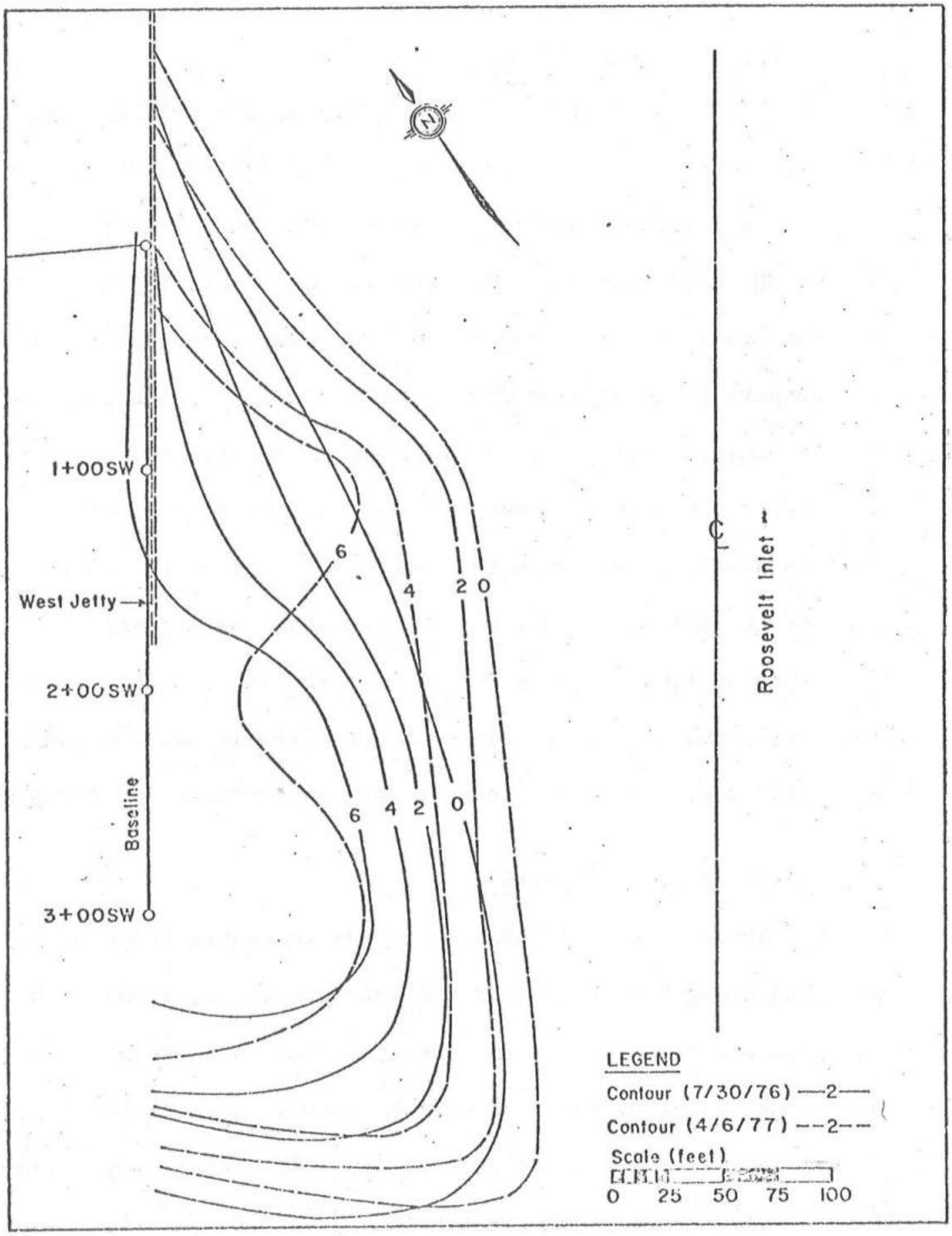

FIGURE 30 Changing Topography of the Western Sand Lobe, July 30, 1976 to April 6, 1977

following the second - the net accumulation being 1,800 yd^3 over approximately eight months or 2,400 yd^3 expressed as a yearly rate.

Waves and currents are primarily responsible for the growth, size,and shape of the sand lobe. It is of interest here to notice the shape that the lobe has taken. Referring to Figure 30 most of the accumulation seems to have taken place between stations 2+00SW and 0+00W. Also the zero contour of the latest survey almost lies on a straight line from station 1+00SW to its southernmost point. The photo in Figure 31 (taken two days after the survey) shows that this linear section of the lobe is nearly parallel to the rubble mound along the east bank of the inlet. It may be that the lobe has adjusted to form the equilibrium cross section for the inlet. This is one indication that the inlet width is considerably overdesigned. Section 7.3 offers more discussion on equilibrium considerations.

4.3 Beach Profile Comparison

During each hydrographic survey foreshore beach profiles were run along both Lewes and the west beach. Profiles were surveyed perpendicular to the baseline from each station; thus numbering six along each reach. The surveys were accomplished using an ordinary engineer's level and a fiberglass level rod. The profiles extended as far into the water as possible, usually being near the local mean low water datum.

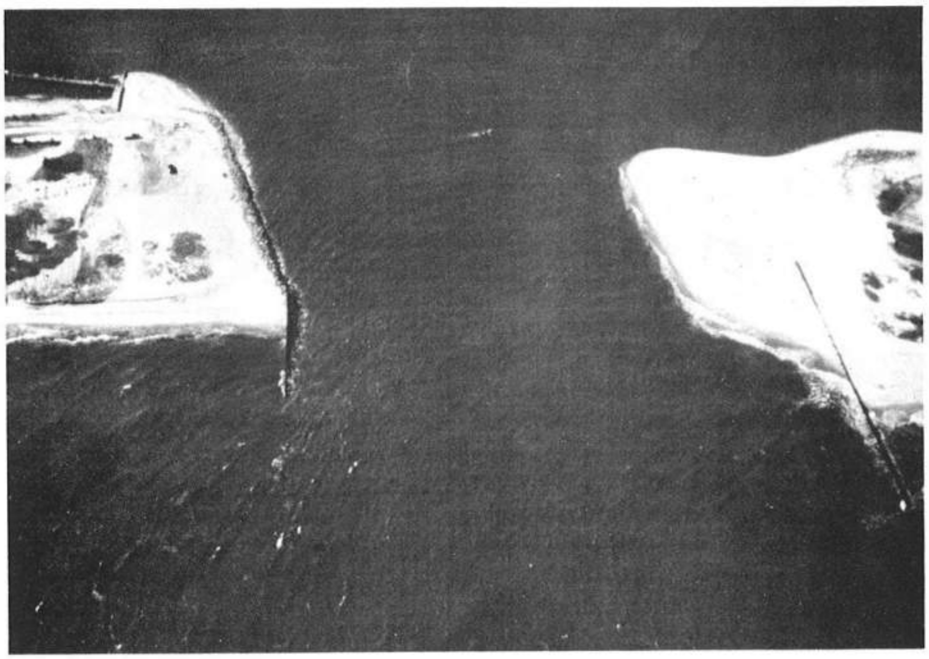

FIGURE 31 Roosevelt Inlet Throat on April 8, 1977 Showing the Parallel Sand Lobe Shape With the East Bank

Results of the profile comparison indicate that severe erosion is occurring along the 1,000-foot reach of Lewes Beach. A four-to five-foot erosional scarp had developed along much of this section making it necessary to re-establish the baseline before the second survey could be accomplished (see Figure 32). The actual volume of erosion between surveys (7/30/76 to 4/7/77) was 10,300 yd^3 within the 1,000-foot section. This reduces to an alarming rate of 15.4 cubic yards per year per linear foot of beach front. Most of the sand eroded was coarse-grain beach fill placed during the inlet dredging operation in 1975. Much of the foreshore has now eroded down to its natural base, exposing relict marsh surfaces which indicate that the beach once lay a significant distance bayward (see Figure 2). The actual comparative beach profiles on Lewes Beach are contained in Figure 33.

The bulk of the sand eroded from this reach is being transported easterly along Lewes beach. Significant buildup of the foreshore is noticeable along a section of beach approximately a mile east of the jetty. Comparison of a profile within this section (Oregon Avenue) originally established and monitored by the University of Delaware, Department of Civil Engineering and now surveyed by the Department of Natural Resources and Environmental Control of the State of Delaware, confirms that significant accretion has occurred. Between May and August of 1976

FIGURE 32 Five-Foot Erosional Scarp Along Lewes Beach

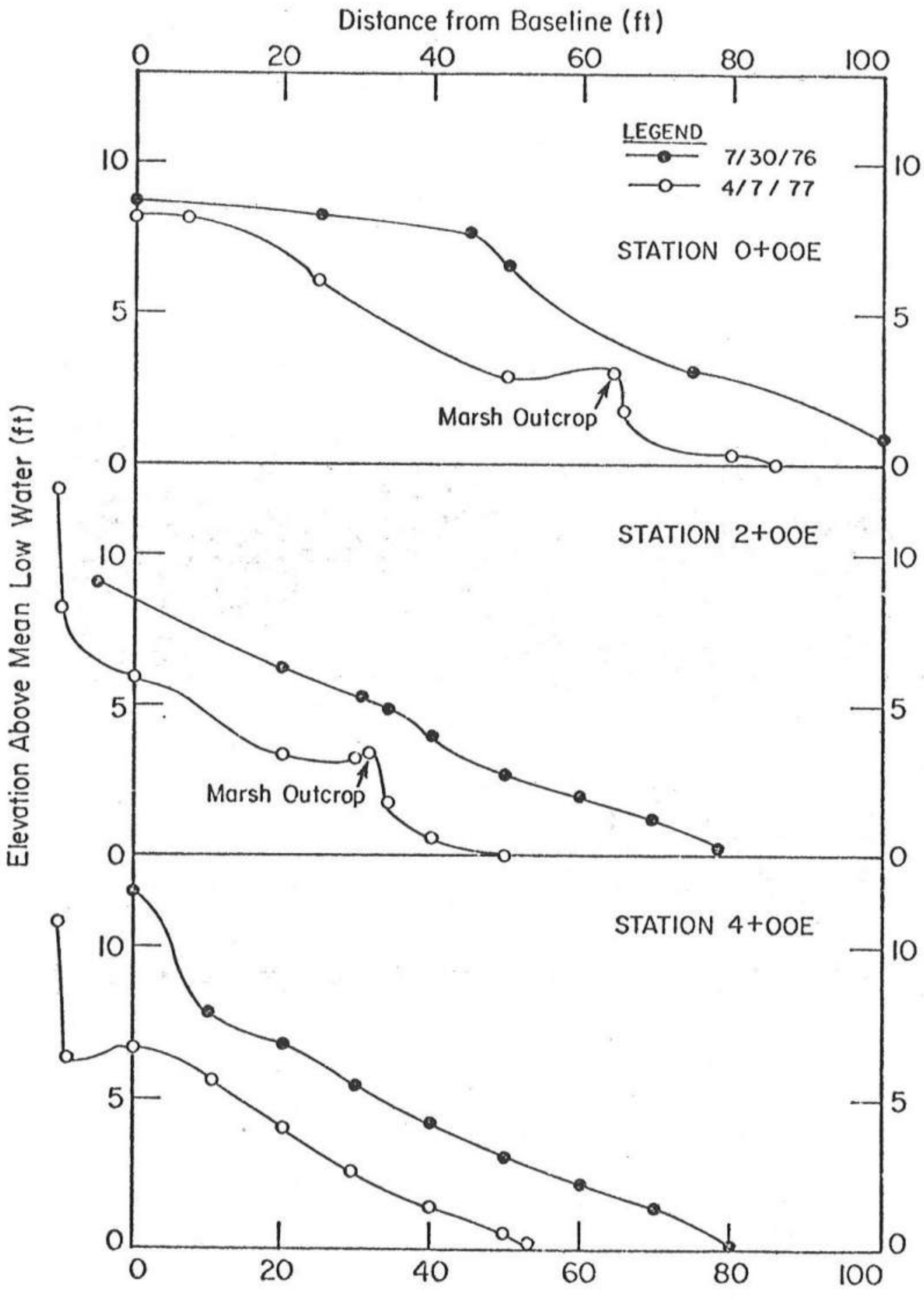

FIGURE 33 Comparative Beach Profile Along Lewes Beach (CONTINUED)

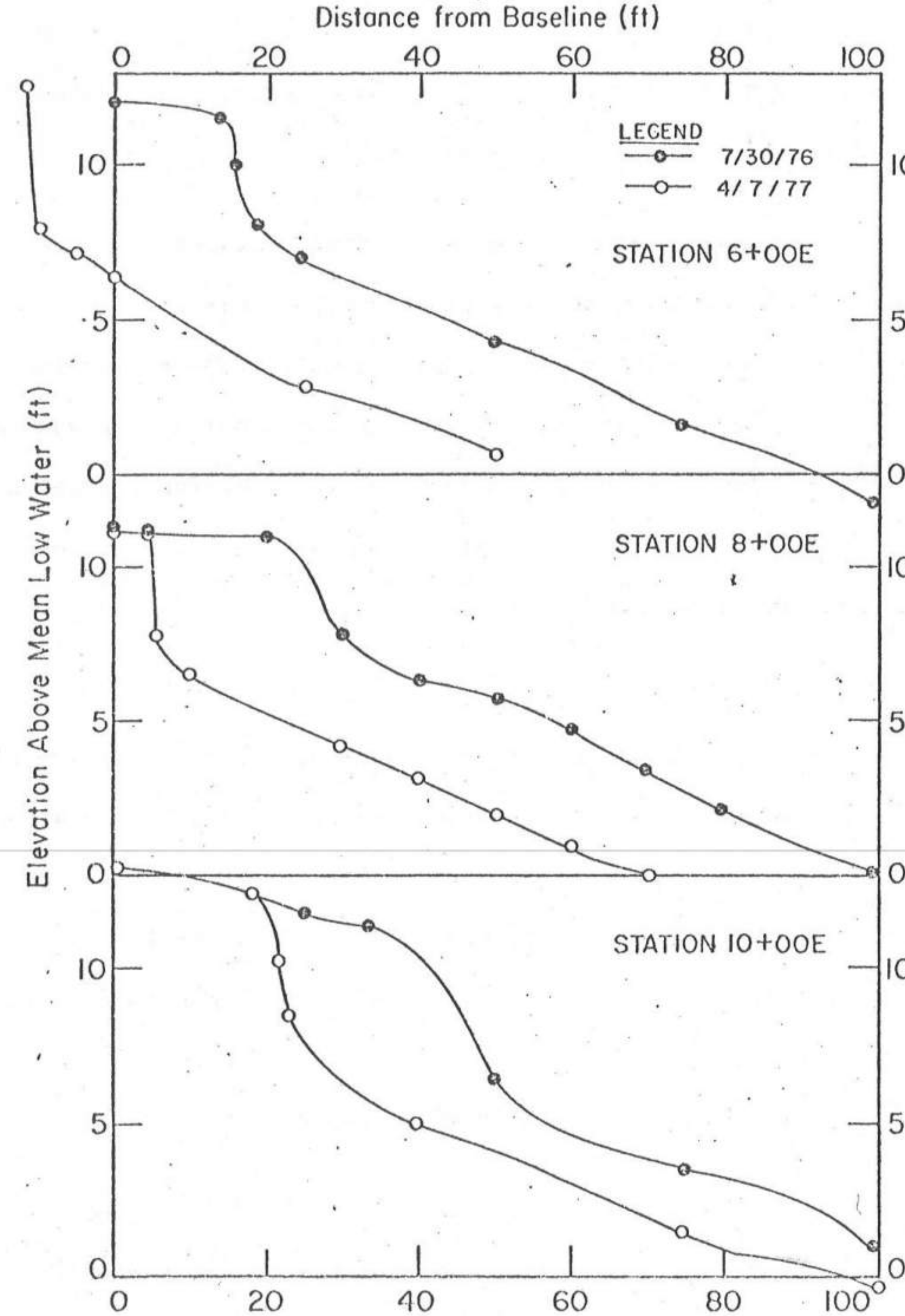

FIGURE 33 (CONTINUED)

the profile showed an accumulation rate of 1.4 cubic yards per year per foot. Between August, 1976 and April, 1977 the accretion rate had increased to 4.5 cubic yards per year per foot at this location. Figure 34 shows the newly-deposited ridge of sand along eastern Lewes Beach.

Along the west baseline erosion rates 15 times smaller than Lewes beach rates were measured. This result is not surprising, since this is the updrift side of the inlet. The important point to note is that even though the erosion rate was small (approximately one cubic yard per year per foot) erosion <u>still</u> had occurred. Most of the eroded sand is believed to have entered into the inlet.

4.4 Sand Tracer Study

A sand tracer study was conducted to determine the sediment movement patterns in the vicinity of the inlet. Both red and green fluorescent tracer was manufactured according to a recipe given by Lanan and Dalrymple (1977). This recipe uses powdered milk to provide the bond between the sand and the fluorescent pigment, which lasts several days. This being the case, no contamination of future studies could result.

Approximately 150 pounds of red and of green tracer were used for the experiment. The red tracer was spread in a thin layer along the waterline approximately 100 feet west of the

FIGURE 34 A Newly-Deposited Sand Ridge Along Eastern Lewes Beach

west jetty at station 1+00W at low tide. The green tracer was deployed in a similar fashion approximately 100 feet east of the east jetty shortly after the red tracer deployment. The experiment was conducted on November 12, 1976 during a mild northeaster accompanied by winds of 15 to 20 mph, snow and sleet. The wave conditions present were a small wind chop of approximately one foot generated by the northeasterly winds superimposed on a larger northwesterly swell of about two feet which had resulted from two days of consistent northwesterlies, previous to the experiment.

A total of 27 samples were collected between six and nine hours following deployment. Inlet samples were retrieved from a boat using a small drag sampler, while those along the beach were collected by hand into standard sample bags. The locations of these samples are shown in Figure 35 along with the tracer injection sites. Each sample was analyzed under an ultra-violet light and the individual tracer grains counted. In samples containing large amounts of tracer only one-fifth of the total sample (by weight) was analyzed. For all others the total sample was examined. Following this a sieve analysis was performed on each sample. The results of the experiment are listed in Table 4.

The aim of this experiment was to obtain qualitative results regarding the sediment movement in the vicinity of the inlet. No

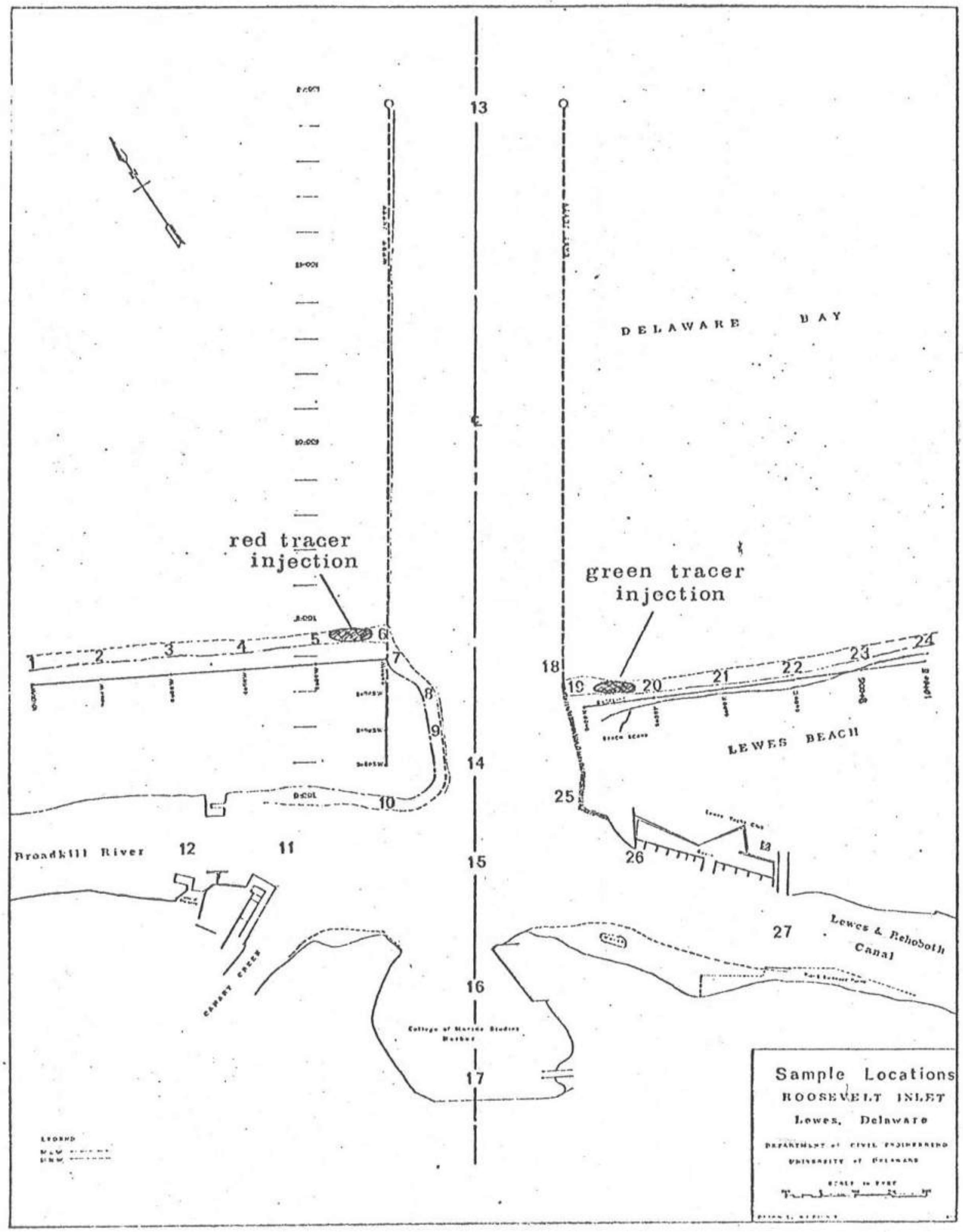

FIGURE 35 Sand Tracer Sample Locations

TABLE 4 Sand Tracer Results

Sample No.	Sample Wt. (gm)	No. of Tracer Grains		Tracer Concentration (grains/gm)	Median Diameter (mm)	Coefficient of Sorting[2]	Log Skewness[3]
		Red	Green				
1	728	0	0	0	1.40	1.37	-0.11
2	704	0	0	0	0.83	1.53	0.09
3	653	0	0	0	0.70	1.40	0.00
4	690	0	0	0	1.50	1.69	-0.05
5	594	5	0	0.008	1.60	1.77	-0.04
6	692	1280[1]	0[1]	1.85	1.80	1.89	-0.05
7	685	1435[1]	0[1]	2.10	0.72	1.60	0.17
8	653	5	0	0.008	0.52	1.20	0.03
9	557	2	0	0.004	1.20	1.57	-0.04
10	578	5	0	0.009	0.54	1.33	0.09
11	400	0	0	0	0.45	1.24	0.02
12	75	0	0	0	Small Sample Retained Gray sandy silt with some clay		
13	55	0	0	0	Small Sample Retained Gray fine sand with some gravel		
14	275	0	0	0	0.45	1.20	0.01
15	103	0	0	0	0.45	1.37	0.06
16	42	0	0	0	Black silty clay with some organics		

(CONTINUED)

(TABLE 4 CONTINUED)

Sample No.	Sample Wt. (gm)	No. of Tracer Grains		Tracer Concentration (grains/gm)	Median Diameter (mm)	Coefficient of Sorting[2]	Log Skewness[3]
		Red	Green				
17	28	0	0	0	Black silty clay with some organics		
18	149	0	0	0	0.88	1.64	0.04
19	385	0	2100	5.5	1.50	1.45	0.05
20	425	0	19	0.05	0.49	1.20	0.00
21	459	0	2	0.004	0.60	1.52	0.00
22	450	0	0	0	0.35	1.54	-0.16
23	419	0	0	0	0.51	1.53	0.11
24	505	0	0	0	0.70	1.80	0.09
25	197	0	0	0	0.70	1.22	0.04
26	119	0	0	0	0.39	1.04	0.14
27	102	0	0	0	Fine black silty sand		

1. Based on one-fifth sample by weight
2. The coefficient of sorting is defined as $\sqrt{Q1/Q3}$ where Q_1 and Q_3 are the grain size diameters at the first and third quartiles, respectively, of the sample's cumulative size distribution curve. The higher the value of the coefficient of sorting the more poorly sorted is the material.

(CONTINUED)

3. Skewness is defined as Q_1Q_3/M^2 where Q_1 and Q_3 are defined above (2) and M is the median grain diameter. The amount of skewness is conveniently expressed as a logarithm. The more the value for log of skewness diverges from unity the more unsymmetrical is the size distribution curve. A negative value indicates maximum sorting in sizes coarser then M and vice versa.

attempt was made to use these results to estimate a transport volume. Of major significance is the fact that in all sample locations on the west lobe (7, 8, 9 and 10) red tracer was found. It is surprising that sand is transported completely around the lobe to location 10, during just one flood tide. This is a transport rate on the order of one foot per minute. It is also seen that the highest concentration of green tracer occurred at location 19. This is possibly because the secondary northeast waves during the experiment were slow in moving the tracer westward from the deployment location. It is also possible that the section of jetty adjacent to the sample location was more impermeable to the sand attempting to enter the inlet than the west jetty due to the rubble mound reinforcement. If this was the case that would explain why no tracer was found along the east bank at locations 18, 25 and 26.

4.5 Sediment Distribution and Characteristics

Along with the sand tracer results Table 4 also lists the sediment characteristics of each sample. All the beach samples were taken at approximately the mid-tide level from the top two-inch layer. The median diameter of these samples range from 1.8 mm to 0.35 mm. Generally speaking much coarser sand is found along the west beach than along Lewes Beach. Samples were collected along beach profiles at Lewes by the Army Corps of Engineers in 1954 and 1964. These samples had median diameters at the mid-tide level ranging from 2.01 mm to 0.53 mm. Above the mean high water mark the range of median diameters was somewhat lower, being 0.84 mm to 0.27 mm. Offshore samples indicate a generally finer gradation as one proceeds from the mid-tide line bayward. The median diameters of samples taken 1,200 to 2,000 feet offshore have ranges of 0.38 to less than 0.08 (U.S. Army Corps of Engineers, 1972).

Sample 12 in the Broadkill River and Sample 27 in the Lewes and Rehoboth Canal both lack significant amounts of large-size sands (found along the beach and the west lobe) indicating a tendency for the sand entering the inlet throat to remain localized. Samples 16 and 17 further indicate that fine-grained sediments are responsible for the College of Marine Studies Harbor silting problem.

Prior to the dredging of Roosevelt Inlet in 1975, the Army Corps of Engineers also collected bottom samples within the inlet. These samples were taken either by a drag or harpoon sampler. All sampling was done on September 9-11, 1974. The sample locations are given in Figure 36. The descriptions of the samples are listed in Table 5. The outer part of the inlet (between station 5+00I to station 20+00I) had not been dredged since August, 1969. Samples in this region should indicate natural bottom sediments unaffected by dredging activities and be indicative of the sediments causing most of the outer shoaling.

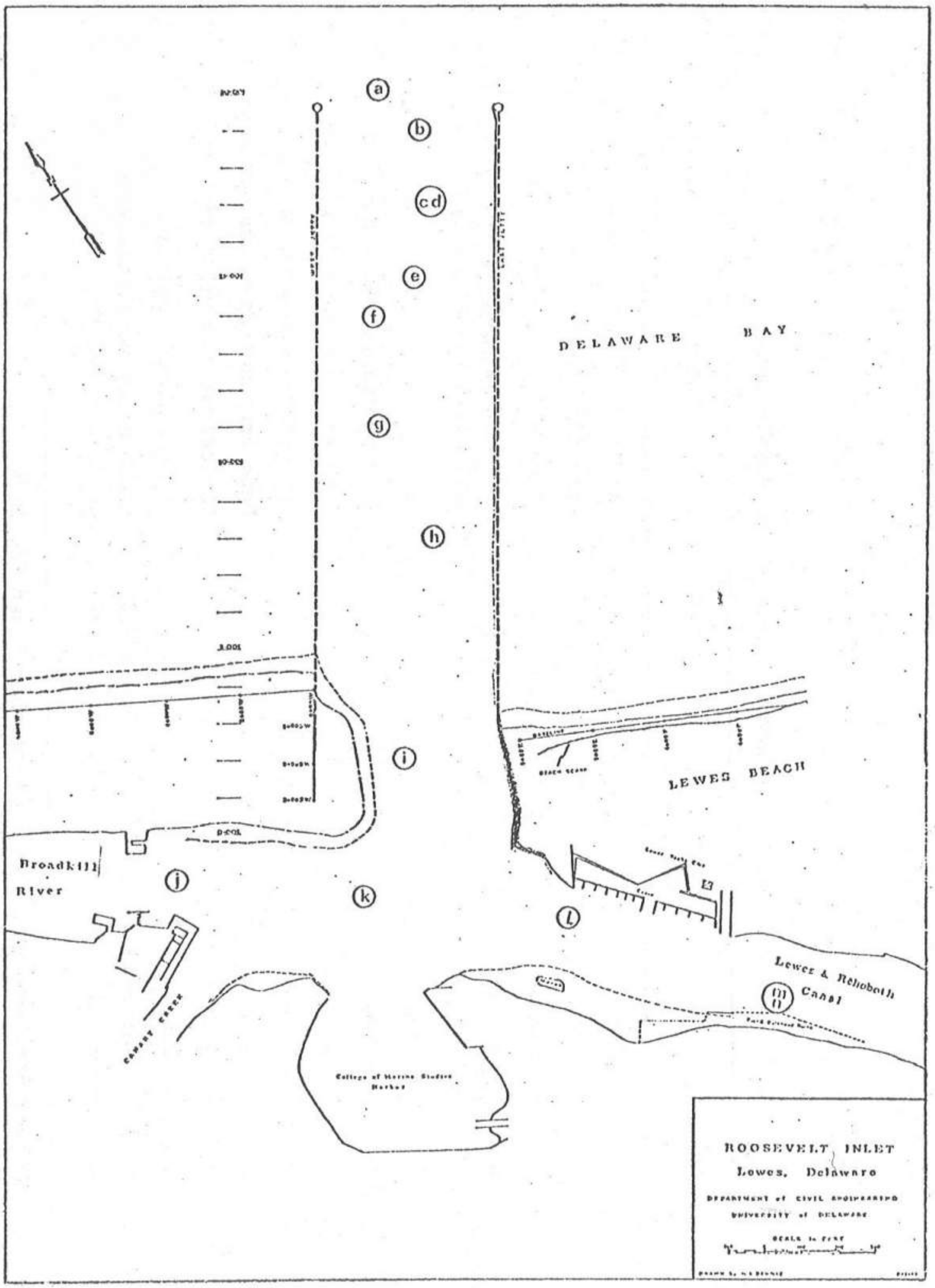

FIGURE 36 Bottom Sample Locations Taken by the Army Corps of Engineers September 1974 (U.S. Army Corps of Engineers Chart No. 42638, Philadelphia District)

TABLE 5 Bottom Sediment Samples in Roosevelt Inlet

Sample	Type	Sample Retained	Remarks	Material Description
a	Drag			Gray coarse to fine sand
b	Drag			Gray fine sand
c	Harpoon	1.5'	Upper 1.0'	Black very fine sandy silt
d	Harpoon	1.5'	Lower 0.5'	Dark gray silt with some fine sand
e	Harpoon	0.6'		Black silt with some medium-fine sand
f	Drag			Gray medium to fine sand
g	Drag			Gray medium to fine sand
h	Drag			Dark gray coarse sand with some gravel
i	Drag			Tan coarse sand with some gravel
j	Drag			Dark gray coarse to medium sand
k	Drag			Tan coarse to medium sand with trace gravel
l	Drag			Tan medium to fine sand
m	Harpoon	2.0'	Upper 1.0'	Black silty very fine sand
n	Harpoon	2.0'	Lower 1.0'	Black very fine sandy silt

NOTE: Samples were taken on September 9 and 11, 1974.
Source: U.S. Army Corps of Engineers, Philadelphia District, Chart No. 42638.

These sediments are mostly fine gray sands. Samples (m) and (n) again indicate black silty fine sands are in the Lewes and Rehoboth Canal. Sample (j) shows that coarse to median sands are at times present in the Broadkill River mouth. Whether or not this sample was affected by the dredging of the inlet throat by the state in 1973 is unknown.

Figure 37 shows the sediment distribution of southwest Delaware Bay (Strom, 1972). Offshore of Roosevelt Inlet mostly muddy sands are present. A larger percentage of mud is present in an area behind Breakwater Harbor where sheltering allows for the settlement of fine-grained sediments.

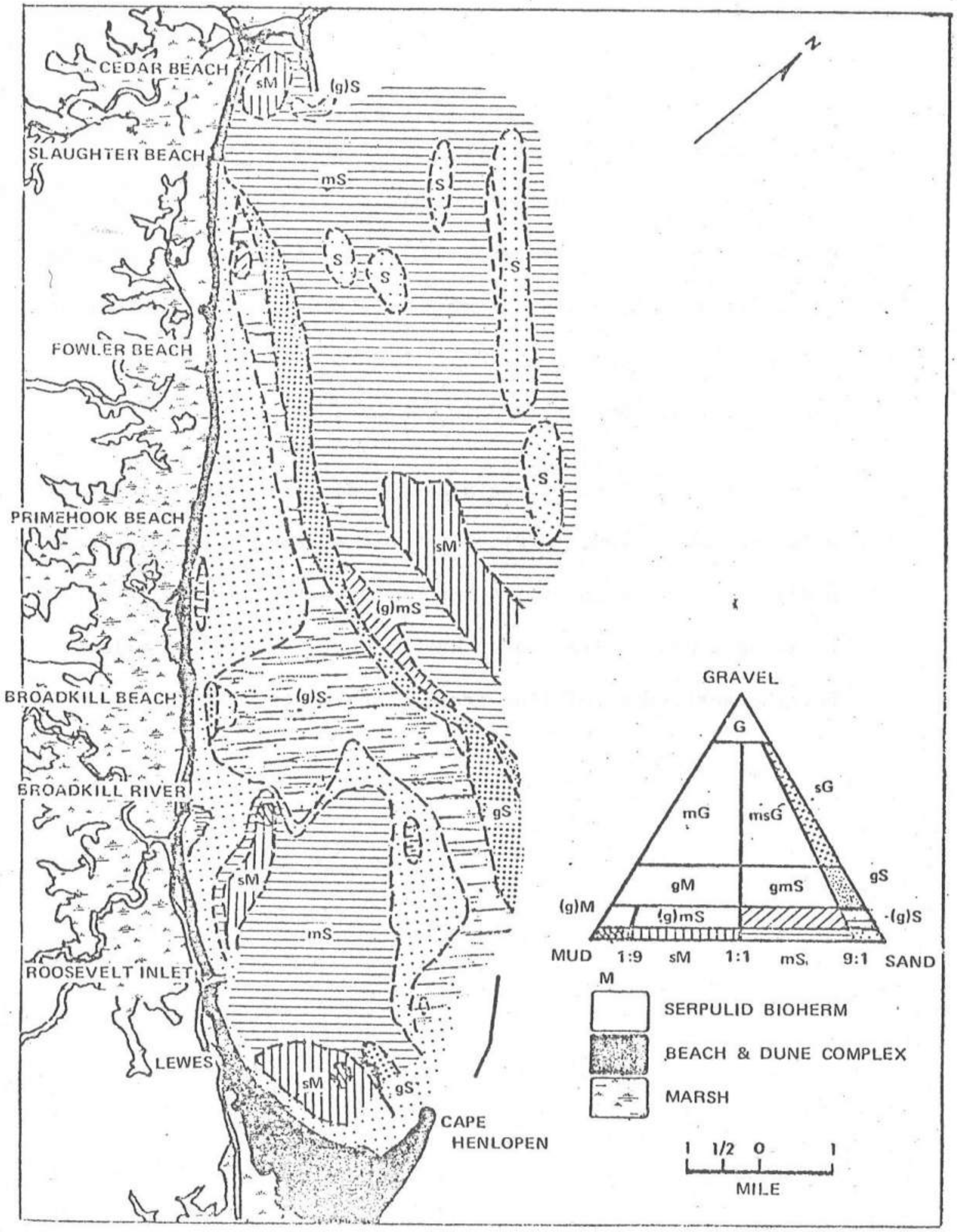

FIGURE 37 Sediment Distribution Map for Southwest Delaware Bay (Strom, 1972)

CHAPTER V

Inlet Maintenance

5.1 Dredging History

Since the initial excavation in 1937, Roosevelt Inlet has been dredged eight times. The dredging activities are usually combined with beach nourishment projects for adjacent Lewes Beach. In this manner sand is artifically by-passed onto the starved downdrift beach. The initial excavation through the barrier removed 521,828 cubic yards (place measurement) of mud, sand, and clay at the cost of $68,382.89. These materials were spoiled along the side banks.

Between November, 1942, and March, 1943, approximately 66,200 cubic yards were dredged, forming a channel of nine feet in depth and 100 feet in width from Delaware Bay to the mouth of Canary Creek. This dredging was accomplished under the Department of the Navy. The spoil area for the material dredged at the mouth of Canary Creek was behind the U.S. Coast Guard facilities. The material removed from the inlet was spoiled on the west beach indicating there might have been confusion as to the direction of net drift at this time.

In March, 1945, the existing project was modified to provide an entrance of ten feet in depth plus the extension of the jetties; but this was never realized. In 1953 maintenance dredging of the inlet was completed by the Federal Government. The removal of 26,500 cubic yards provided a ten-foot channel depth in the throat of the inlet and seven feet near the jetty entrance, at a cost of $35,000. In conjunction with maintenance dredging in the Lewes and Rehoboth Canal, also by the Federal Government, during 1957, the new project dimensions (10'x200') were finally realized. A total of 312,606 cubic yards were dredged from Lewes to Delaware Bay, of which 79,026 cubic yards were removed from the inlet itself. The cost of the total project was $108,500. The material removed from the inlet in both 1953 and 1957 was placed along Lewes Beach.

In 1962, the State of Delaware commissioned the East Coast Dredging Company to remove the sand lobes along the east and west banks of the inlet and to use this material for beach fill along Lewes. Approximately 20,735 cubic yards were placed along the beach until the dredging operation was aborted by the storm of March 6-8, 1962. It was estimated that approximately a 50% loss of fill material resulted from the storm activity (State Highway Department, 1963). The contract cost was $14,100.

Further required inlet maintenance by the Federal Government was accomplished in 1963 and 1969. During 1963, 86,997 cubic yards were dredged from the inlet and placed along Lewes Beach, the job costing $50,190. In 1969 the inlet was dredged along with the dredging of the L&R Canal to the turning basin at Lewes. A total of 346,400 cubic yards were removed, of which 135,648 cubic yards were from the inlet. The canal material was spoiled along its banks behind Lewes Beach (see Figure 23) and the inlet material was spoiled along Lewes Beach. The project ran to a total cost of $179,563.

In 1973 the state dredge removed 69,800 cubic yards from the throat of the inlet. Most of this material was contained in the two lobes and served as a source of good quality fill placed along Lewes Beach. The project cost was roughly one dollar per cubic yard.

Between December 12, 1974 and January 22, 1975 the American Dredging Company removed a total of 120,000 cubic yards from the inlet, establishing a channel 18 feet deep and 200 feet wide. The project was a 50/50 state/federal cost-share amounting to $274,550. The material was spoiled along Lewes Beach providing 4,800 feet of much needed nourishment.

A summary of the dredging history of Roosevelt Inlet is given in Table 6. A total of 633,495 cubic yards has been dredged since the inlet's opening in 1937. The yearly maintenance cost is approximately $23,600 (expressed in 1974 prices, U.S. Department of Labor, 1975). This cost is slightly overestimated

TABLE 6 Dredging History of Roosevelt Inlet

Date: Start	Date: Finish	Area Dredged	Disposal Area	Volume (yd^3): To Overdepth	Volume (yd^3): Gross	Comment	Cost ($)*
12/30/36	5/17/37	Initial Cut	Channel Banks		521,828	Mud, clay, and sand composition	68,382
11/16/42	3/12/43	Stations 0+00I to 28+00I Canary Creek entrance	West beach and behind U.S.C.G. station		66,200	U.S. Navy funds	--
2/4/53	3/14/53	Stations 0+00I to 21+00I	Lewes Beach	26,500	55,085	Federal maintenance	35,000
5/7/57	6/6/57	Stations 0+00I to 30+00I	Lewes Beach	59,960	79,030	In conjunction with dredging of Lewes and Rehoboth Canal	108,500
1/15/62	3/12/62	Inlet throat	Lewes Beach		20,735	State funds, contract dismissed by March, 1962 storm	14,100
8/15/63	9/3/63	Stations 0+00I to 29+00I	Lewes Beach	48,717	86,997	Federal maintenance	50,190
6/24/69	8/14/69	Stations 0+00I to 28+00I	Lewes Beach	66,889	135,648	In conjunction with dredging of Lewes and Rehoboth Canal	179,563

(CONTINUED)

TABLE 6 (CONTINUED)

Start	Finish	Area Dredged	Disposal Area	Volume (yd^3) To Overdepth	Volume (yd^3) Gross	Comment	Cost($)*
11/9/72	5/30/73	Inlet throat	Lewes Beach		69,800	State dredge	69,800
12/12/74	1/22/75	Station 0+00I to 25+00I	Lewes Beach	96,800	120,000	50/50 State/ Fed. Cost Share	274,550

* not adjusted to a base year

since it includes the total cost of the dredging operations in 1957 and 1969 of which part of the sum included maintenance of the L&R Canal.

5.2 Shoaling Rates

To obtain an estimate of the shoaling rates in Roosevelt Inlet since its stabilization, 24 survey charts were analyzed and compared. These charts were obtained from the U.S. Army Corps of Engineers Office, Philadelphia District. They covered a time period of 37 years or 1938 to 1975. The survey completed during the course of the project (Figure 29) was also included in the analysis. Each chart was divided into 14 sections (see Figure 38) which were further subdivided into small grids. The volumes were computed by averaging the depths of the grid points, multiplying this depth times the surface area of the grid and summing over each section. This technique was very easily applicable to the charts used. Most of the chart soundings contained within the inlet were run on straight lines perpendicular to the inlet centerline at regularly spaced intervals, hence already in grid form. Soundings within the junction of the inlet with the Broadkill River and the L&R Canal were of a more irregular nature necessitating the superposition of grid spacing. Contours were drawn to fill in gaps where necessary. All charts used mean low water at the mouth of the Broadkill as a common datum,hence no adjustment was needed for proper comparison.

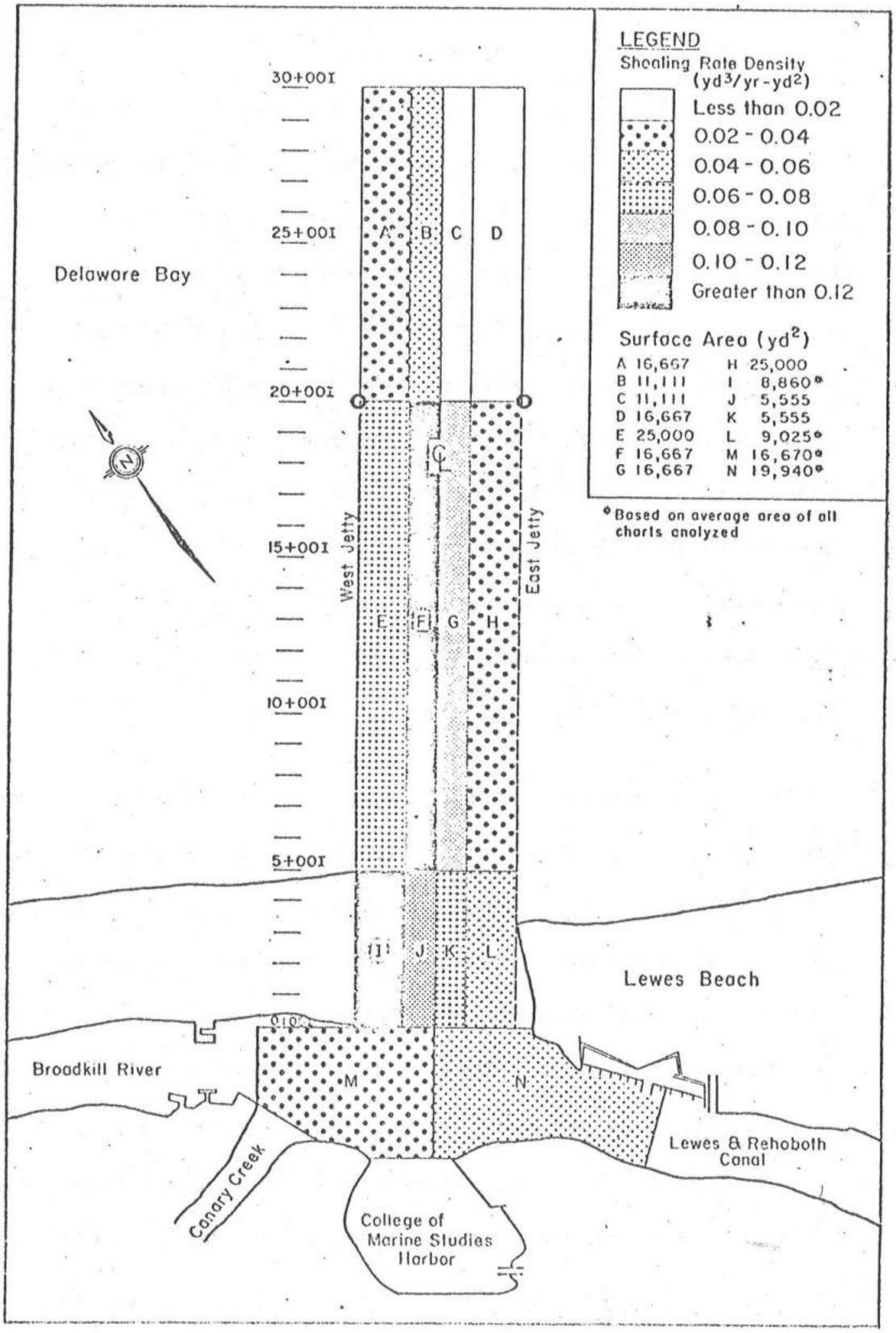

FIGURE 38 Relative Shoaling Rates for Various Sections in Roosevelt Inlet

The comparison of the calculated volumes of water below mean low water for each chart resulted in estimates of dredging volumes, shoaling volumes and shoaling rates. The dredging volumes served as an accuracy check; by comparison with estimates of dredging volumes by the Army Corps of Engineers they were found to be within an average accuracy of $\pm$ 15%. The resulting shoaling rates are shown schematically in Figure 38. To properly compare the amount shoaled in each section (A through N) which vary in size, the shoaling rate for each section was divided by its surface area, resulting in a shoaling rate density ($yd^3/yr-yd^2$). For instance to ascertain the actual average shoaling rate for Section A multiply its shoaling rate density (0.02 to 0.04) times its surface area (16,667 yd^2).

The results in Figure 38 indicate that Sections I and F have the greatest shoaling rate. This is not surprising, especially since Section I is the location of the dominant sand lobe buildup. Section F on the other hand is located on the updrift side of the maintained inlet channel. As mentioned previously (Section 4.2) the presence of this relatively deep channel has an enhanced capacity to trap sediments due to the added gradient of the channel banks. Section J shows the second highest shoaling rate. This is undoubtedly due to its proximity to the growth of the western shoal as in Section I. Sections K and L have relatively low shoaling rates, indicating that these sections may be more or less self-maintaining. The close proximity of Section I, having the highest rate of shoaling, to Section M, with a very low rate of shoaling, leads to the belief that the sand entering the inlet from the west tends to remain more or less localized (as proposed in Section 4.5). On the eastern side of the inlet Sections L and N lie within the same shoaling rate range. This helps to demonstrate that sand entering from the east is more readily transported to the mouth of the L&R Canal, the reason being that as the western lobe builds, this causes the inlet channel to migrate eastward. The presence of the main channel currents in closer proximity to the east bank results in a more efficient sand transport mechanism within Section L.

At the inlet mouth it is noticed that Sections C and D possess very low shoaling rates. This is partially because of the lower availability of sediments (due to deposition in Sections A and B), plus the eastward deflection of the ebb tidal plume which continually transports fine grain sediments from this region (Section 4.1).

The total volume shoaled since 1938 in all sections of the inlet sums up to approximately 429,000 cubic yards. This results in a yearly shoaling rate of approximately 11,000 cubic yards or a shoaling rate density of 0.054 cubic yards per year per square yard. Dividing the inlet down the centerline it is found that the western half shoals nearly twice as fast as the eastern half, the actual shoaling rates being approximately 7200 and 3800 cubic yards per year, respectively.

A further point of major significance borne out through analysis of the past survey charts is the dramatic increase in shoaling rate. Figure 39 is a graph of cumulative volume shoaled between inlet stations 0+00I and 20+00I versus year. The two dashed lines were sketched to a "best fit" representation of the data for the years 1937 to 1962 and 1962 to 1977. The slopes of the lines represent the shoaling rates for the two time intervals. Between 1937 and 1962 the shoaling rate was only 2520 cubic yards per year. Between 1962 and 1977 the shoaling rate was 20,300 cubic yards per year, an eightfold increase! This dramatic increase occurring in the early 1960's is due in part to the attempt to maintain a deeper channel (six to ten feet). Also, it is likely that the devastating storm of March 6-8, 1962 caused significant damage and fatigue to the steel sheet pile, initiating the increased shoaling trend.

5.3 Past and Present Inlet Dimensions

Along with the volumetric analysis of the past and present survey charts, the characteristic inlet dimensions were also recorded for stations 2+00I and 15+00I (see Figure 29 for locations). Although the dimensions have been influenced by

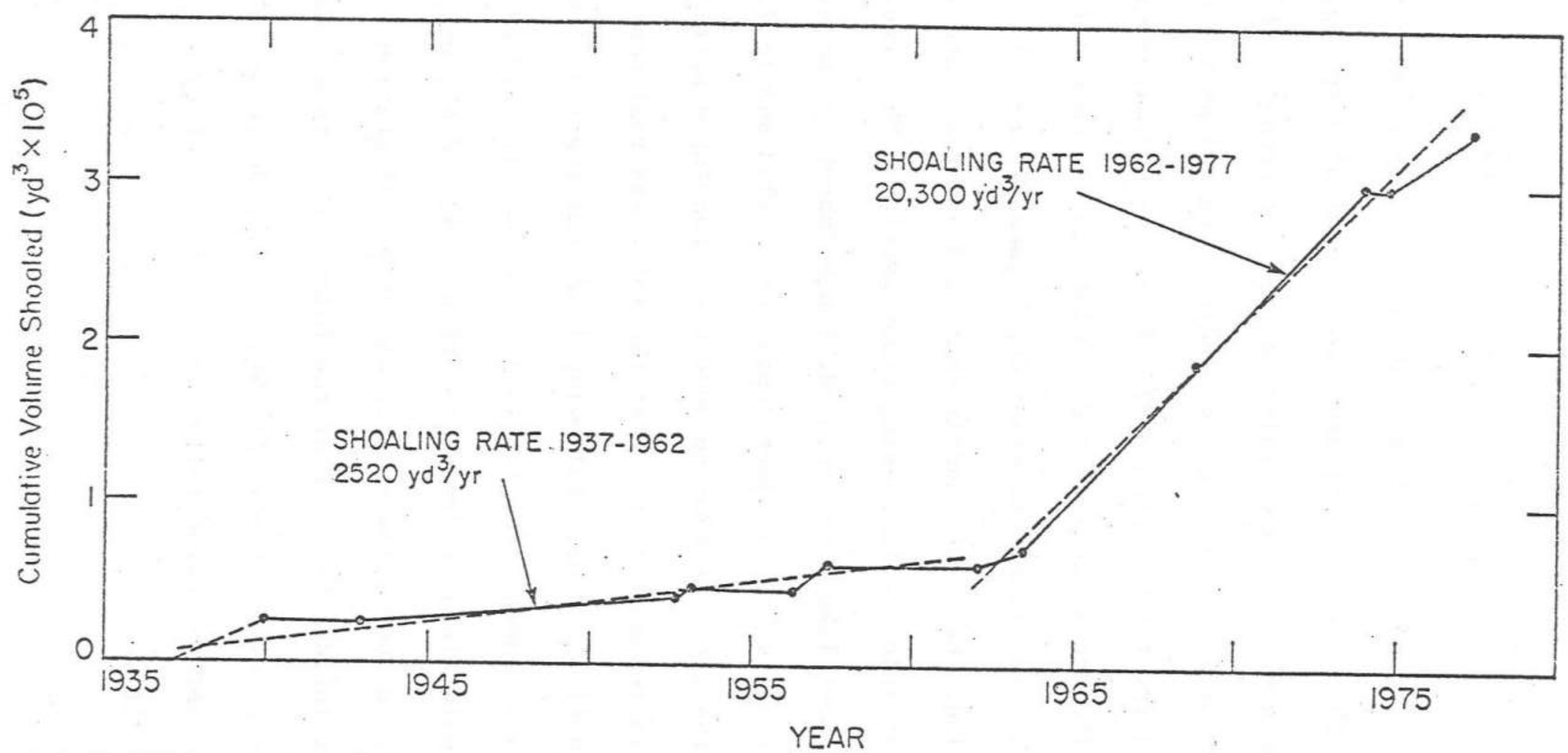

FIGURE 39 Cumulative Volume Shoaled Between Inlet Sta. 0+00I and 20+00I Versus Year (Dashed Line Indicating Trends)

dredging activity certain trends are apparent.

The characteristic dimensions versus year for station 2+00I (at the inlet throat) are shown graphically in Figures 40 and 41. Figure 40 shows a definite enlargement trend of the cross-sectional area prior to 1962, corresponding with the period of low shoaling rate in Figure 39. This cross-sectional enlargement was manifested by an increase in width until the mid-1950's along with a relatively constant depth (see Figure 41). Following this period, the width sharply decreased and the depth sharply increased maintaining an increasing cross section. This reversal in trend was most likely man-induced, brought about by the six-to-ten-foot increase in the project depth during 1953 and 1957. The ability of the cross section to adjust to changing conditions and continue to increase indicates that the inlet was resistant to closure during this time period. Following 1962, examination of Figures 40 and 41 shows a tendency for the inlet to close, its dimensions having to be maintained by dredging. Figures 42 and 43, graphically illustrating the characteristic dimensions of station 15+00I (500 feet inside the mouth of the inlet), show this closure tendency to a greater extent. At this location decrease in cross section and depth occurred following practically every dredging operation for all years.

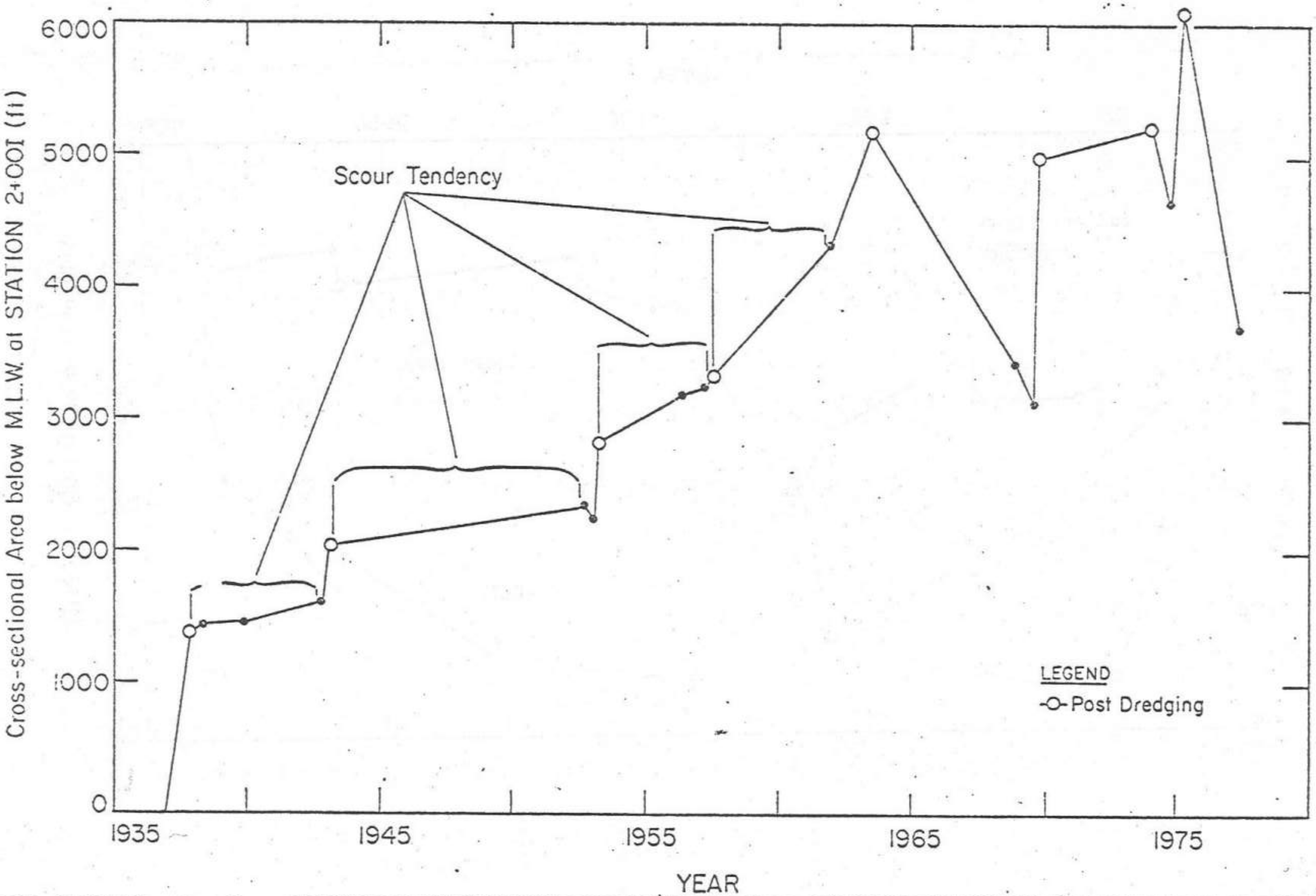

FIGURE 40 Cross-Sectional Area Below M.L.W. at Sta. 2+00I Versus Year

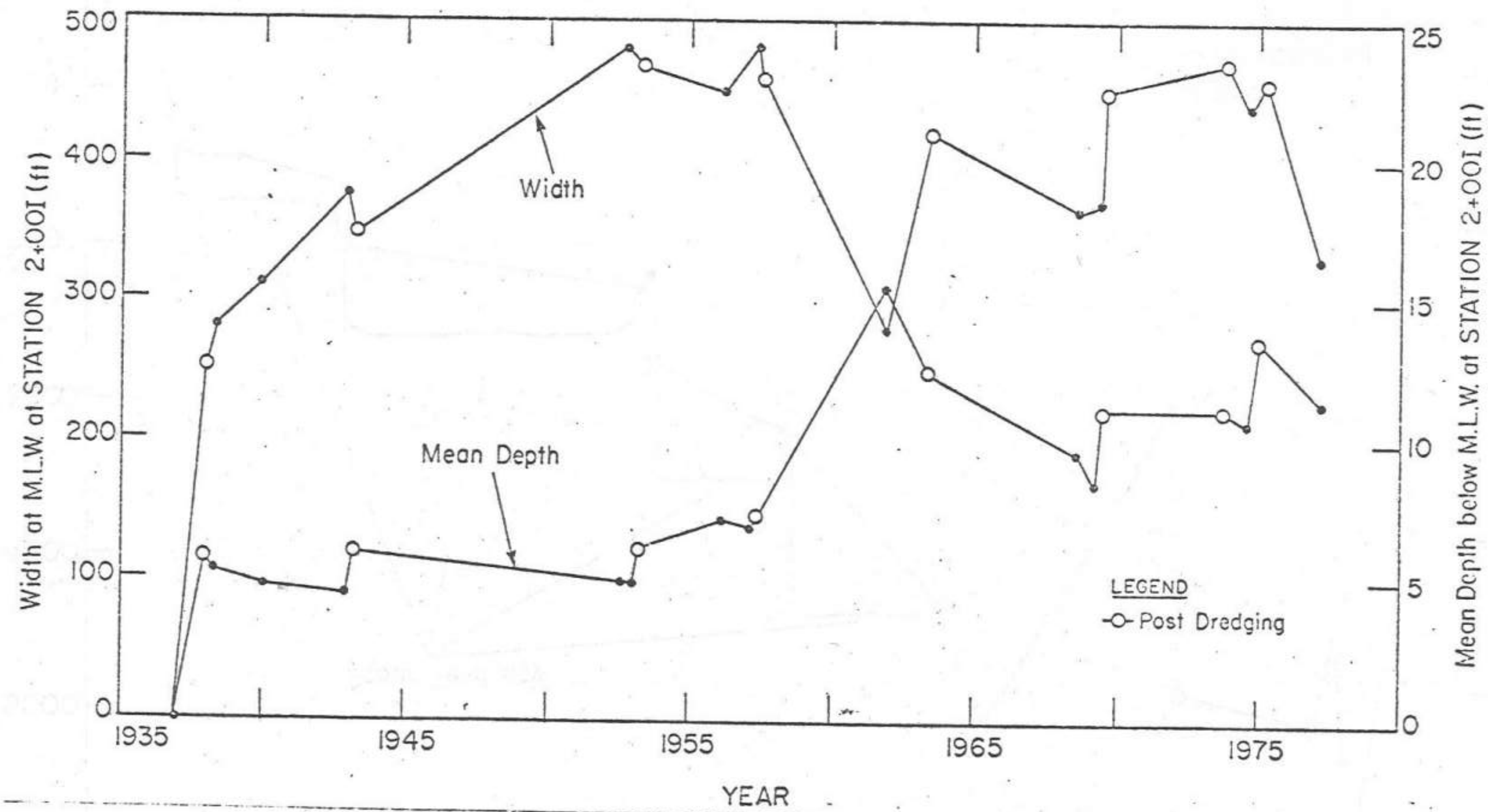

FIGURE 41 Mean Depth and Width at M.L.W. at Sta. 2+00I Versus Year

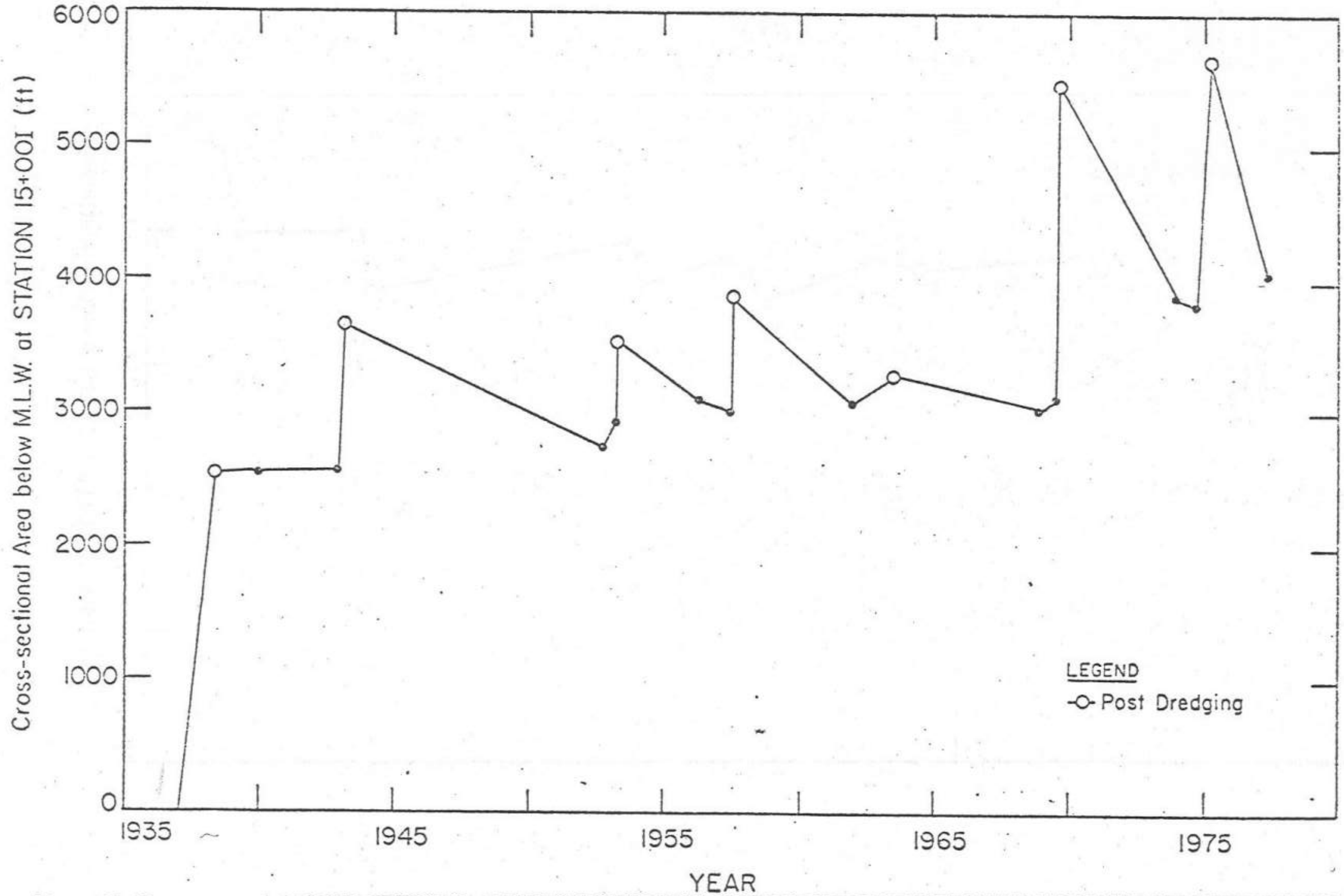

FIGURE 42 Cross-Sectional Area Below M.L.W. at Sta. 15+00I Versus Year

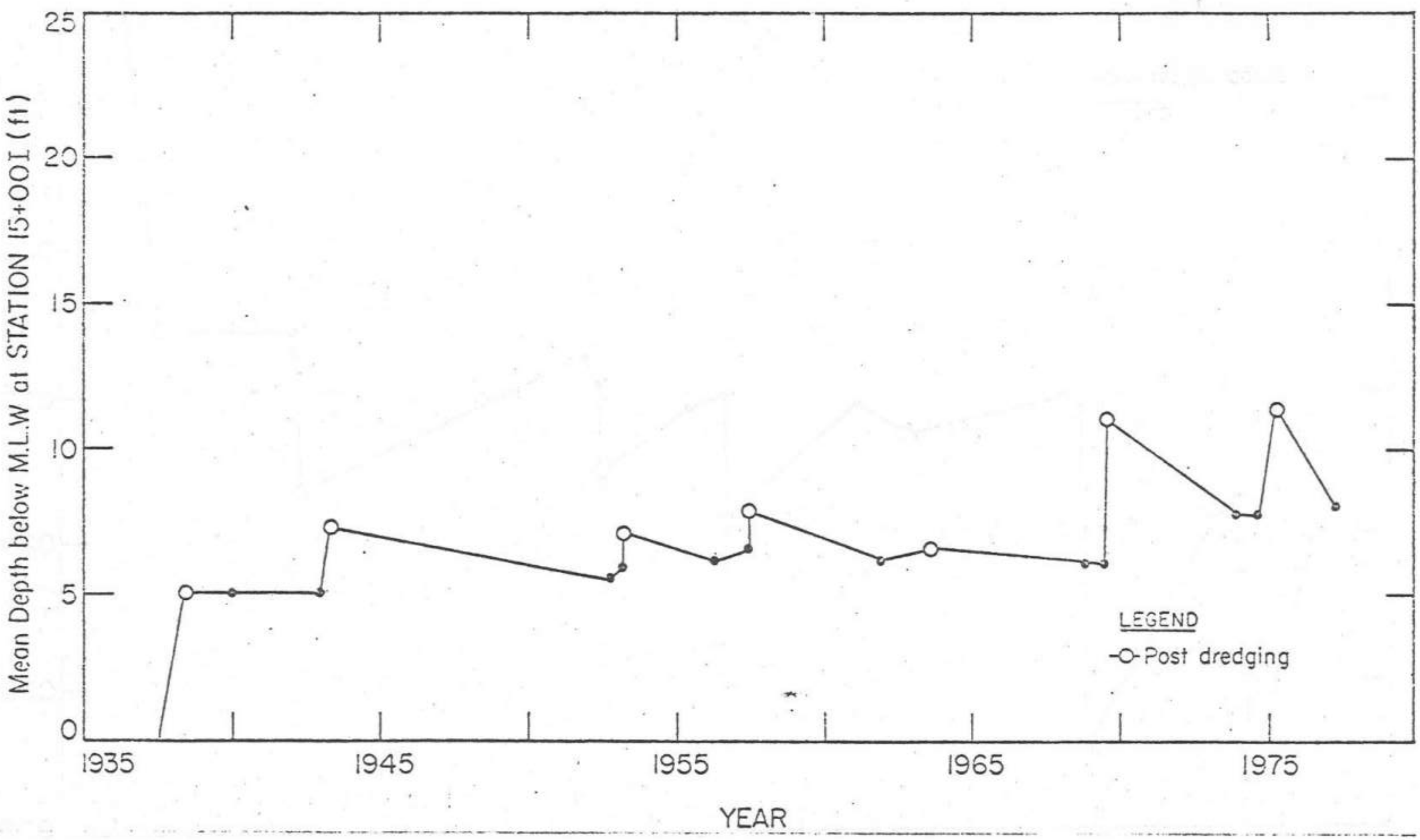

FIGURE 43 Mean Depth Below M.L.W. at Sta. 15+00I Versus Year

In conclusion it is evident that to maintain the inlet in its present condition continuous dredging is needed unless the jetties are improved both structurally and in design. Considering that the outer end of the inlet (Sta 15+00I) has never shown a significant tendency to maintain itself but the inlet throat (Sta 2+00I) has shown a tendency to deepen with decreasing width, the conclusion may be drawn that the jetties are too widely spaced. Further discussion of this topic will be resumed in Section 7.3.

CHAPTER VI

Sand Budget

The knowledge of the sediment movement patterns and the estimates of dredging and shoaling volumes can be combined with estimates of volumetric change along Lewes and the west beach to yield a sand budget for the overall Roosevelt Inlet system. A sand budget states how much sand is transported and where. The results of this sand budget are estimates of the littoral drift entering and leaving the system. The actual estimate of volumetric change along Lewes and the west beach was calculated from the aerial photo overlay contained in Figure 25. This map was planimetered to ascertain changes in beach surface area during each specific time interval. It was then assumed that these changes in surface area occurred uniformly along the length of each region (7,725 feet along Lewes Beach and 10,300 feet along the west beach). Furthermore, it was assumed that these changes were accomplished by the uniform retreat or advance of the beach profile over an active vertical distance of ten feet (depth of closure). This formulation results in a change of one square foot of surface area equaling a volumetric change of 0.34 cubic yards per foot of beach front. Normally along ocean beaches a depth of closure of 27 feet is assumed, resulting in a one-foot retreat of shoreline equaling a loss of one cubic yard per foot of beach front (U.S. Army Corps of Engineers, 1973). Considering the much reduced wave climate of Delaware Bay, this estimate seems reasonable.

6.1 Erosion Along Lewes Beach

Lewes Beach like most all communities along the coast of Delaware is experiencing erosion. The erosion is caused by the gradual rise in sea level along with the pervasive action of waves. In the case of Lewes Beach, man's interference with the system by the excavation and stabilization of Roosevelt Inlet has aggravated the erosion problem. In its present condition, the inlet is acting as an effective trap to most littoral sediments from both directions. Furthermore, the usual protection found in the lee of a jetty along the downdrift side is not fully operating at Lewes Beach. The elevation of the jetties (presently deteriorated to about the mean low water line) allows for the passage of most of the waves emanating from the north which would otherwise (if the jetties were present in their original state) have been effectively reduced.

As stated in the previous chapter, in an effort to curtail the erosion, beach fill has been placed along Lewes Beach following each inlet dredging operation. As a further measure the state has dredged offshore to provide additional nourishment on two occasions. In 1954 approximately 48,000 cubic yards were dredged providing 3200 feet of nourished beach at a cost of $0.73 per cubic yard. In 1957, a rather large operation was completed, pumping approximately 400,000 cubic yards onto Lewes Beach. This project cost nearly $0.65 per cubic yard, totalling $258,100. The borrow areas for both of these operations were located approximately 1000 feet offshore of Lewes Beach. Table 7 is a summary of beach nourishment performed along Lewes Beach. To date a total of 1,015,300 cubic yards of material have been placed along this reach.

In addition to the placement of fill, a total of nine groins have been constructed, spaced along a 4000-foot stretch of beachfront. Three groins were constructed during each of the following years; 1948, 1950, and 1956. The cost of the respective projects was $10,000, $17,023.42, and $29,056.42. Figure 44 is a photo taken on August 2, 1956 showing part of the existing groin field. The characteristics and location of each groin are contained in Table 8.

The purpose of a groin is to provide an effective barrier across the littoral zone and thus trap and retain sand from the littoral drift stream. This cannot be done, however, without the accompanying erosion on the downdrift side. This detrimental effect can be minimized through the use of a series of groins as along

TABLE 7 Nourishment Along Lewes Beach

Year	Volume (cubic yards)		Approximate Length (feet)	Source
	State	Federal		
1953		55,085	2000	Inlet
1954	48,000		3200	Offshore
1957		79,030	2000	Inlet
	400,000		5000	Offshore
1962	20,735		1600	Inlet
1963		87,000	2000	Inlet
1969		135,650	2800	Inlet
1973	69,800		3700	Inlet
1975	120,000 (50/50)		4800	Inlet
Total	1,015,300			

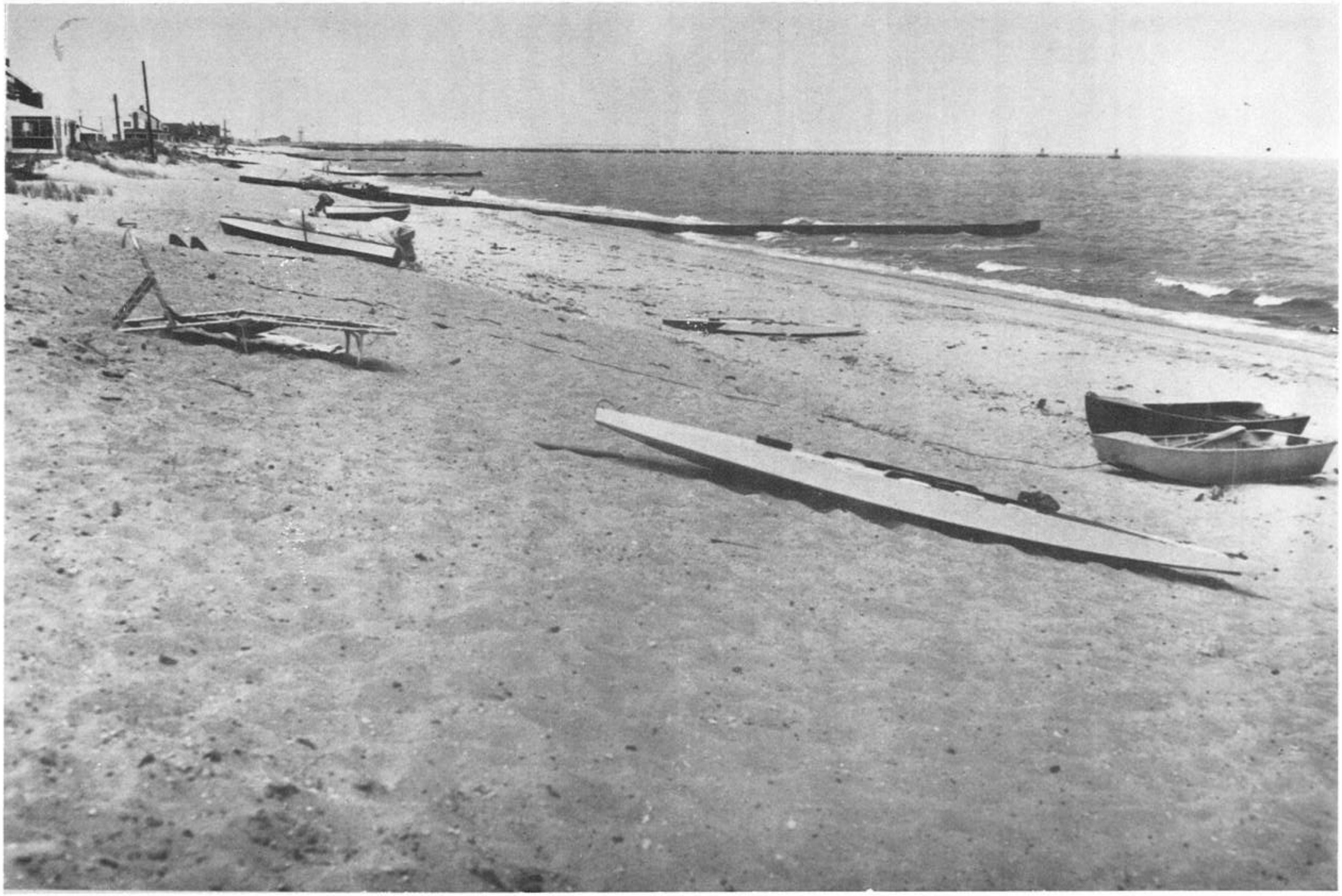

Figure 44 Part of the Groin Field Along Lewes Beach, August 2, 1956 (Courtesy of Delaware State Highway Department)

TABLE 8 Groins Along Lewes Beach

Date	Description of Work	#	Approximate Distance from East Jetty	Dimensions (ft) Length	Top Elevation Above M.L.W. Inner End	Top Elevation Above M.L.W. Outer End	Cost
1948	Construction of 3 timber groins	I	So. Illinois Ave 3600'	150	11	2	$10,000
		II	Between Penna & Michigan Ave 3000'	135	11	2	
		III	So. Calif. Ave 2000'	145	12	2	
1950	Construction of 3 timber groins	IV	Between Kentucky & Ohio Ave. 3960'	161	11	4	$17,023.42
		V	Indian Ave. 4470'	164	11	4	
		VI	Iowa Ave. 1600'	172	11	4	
1956	Construction of 3 stone-filled timber groins. Repairs to Groin IV	VII	Oregon Ave. 4820'	166	10	1	$29,956.42
		VIII	Between N.J. & N.Y. Ave. 5165'	168	10	1	
		IX	South of N.Y. Ave. 5510'	206	10	1	

Lewes Beach, passing the erosion further downdrift. An even more effective method is to supply fill to the groin field thereby providing some "sacrificial" material to alleviate the downdrift problem.

Along Lewes Beach both beachfill and the groin field are used in conjunction; however, the existing groins are too low, being completely covered most of the time by the large quantities of beach fill. In this manner they are rendered ineffective until the beach becomes very lean. Figure 45 shows the initial exposure of a groin along Lewes Beach. Figure 46 shows the same groin (distant center) less than a month later under more intense conditions. Both of the photos were taken prior to the beach nourishment project of 1972-73.

In order to quantify the amount of erosion actually occurring along Lewes Beach, Figure 25 was analyzed according to the method described in the introduction to this chapter. The results of this analysis are listed in Table 9. The term "net" volumetric change is the volume estimated from the map. The term "gross" volumetric change is the actual change that had to occur accounting for beachfill. For the computation of the "net" and "gross" rates it was assumed that both measured volume as well as the nourishment were distributed uniformly over the length of the control section.

FIGURE 45 Initial Exposure of a Groin Along Lewes Beach, September, 1972 (Courtesy of Department of Natural Resources and Environmental Control, Delaware)

FIGURE 46 Advanced Stage of Erosion Along Lewes Beach, September, 1972 (Courtesy of Department of Natural Resources and Environmental Control, Delaware)

TABLE 9 Volumetric Change Along Lewes Beach

Interval	Quantity (yd^3)			Rate (yd^3/yr-ft)	
	Net	Nourishment	Gross (Net + Nourishment)	Net	Gross
7/5/38 - 7/22/54	-533,330	55,085	-588,415	-4.3	-4.7
7/22/54 - 11/12/60	202,670	48,000 79,030 400,000 527,030	-324,360	4.1	-6.6
11/12/60 - 5/19/68	- 53,400	20,735 89,000 107,735	-161,135	-0.9	-2.8
All Years 7/5/38 - 5/19/68	-384,060	689,850	-1,073,910	-1.7	-4.7

It is seen from Table 9 that the greatest actual erosion occurred between 1954 and 1960, i.e.,- 6.6 cubic yards per year per foot, although the shoreline showed a net accretion during this period. This resulted from the large quantity of beachfill placed over this time interval, particularly from the 400,000 cubic yard offshore project in 1957. These results indicate that the beachfill eroded quite rapidly.

Surprisingly low rates of erosion were calculated for the period of 1960 to 1968. One would expect the effects of the major storm of March 1962 to produce much greater erosion. This leads to some speculation about the technique utilized. The errors inherent in the system range from the actual overlay and alignment of each photo to the instantaneous conditions at the time the photo was taken. As a comparison, a rate of -2.2 cubic yards per foot per year was estimated along Lewes Beach by the Army Corps of Engineers (1973) for a period of 10 years, 1954 to 1964.

For all years analyzed (1938-1968) a total of 1,073,910 cubic yards eroded from Lewes Beach, more than half of the total placed as beachfill. The average net rate was -1.7 cubic yards per foot per year. The average annual rate estimated by the Army Corps of Engineers over a 121-year period (1843-1964) was -1.6 cubic yards per foot.

Figure 47 is a graph of the cumulative volume eroded along Lewes Beach versus year using the data in Table 9. The slope of the line is indicative of the rate of erosion. The average annual erosion rate is approximately 36,300 cubic yards.

6.2 Erosion Along the West Beach

The initial analysis of volumetric change along the west beach concentrated on the amount of accumulation along the west jetty. The results are shown graphically in Figure 48. There are two major points illustrated by this graph. First, a dramatic decrease in the retention capacity of the jetty is evident around 1962. Before this date, the rate of accumulation was approximately 4,580 cubic yards per year since the construction of the jetty. Following 1962, loss of the volume accumulated occurred at an average rate nearly three times as great, or approximately -12,300 cubic yards per year. The date of this abrupt change in retention capacity of the west jetty closely corresponds to the time of the eightfold increase in shoaling rate revealed in Figure 39. This is further evidence that the March, 1962, storm severely damaged and weakened the already degrading steel sheet pile.

The second major point of interest contained in Figure 48 is that the net accumulation along the west jetty has nearly reached zero as of April, 1975. Extrapolation of the lower portion of the graph predicts this would occur by 1977. Comparison

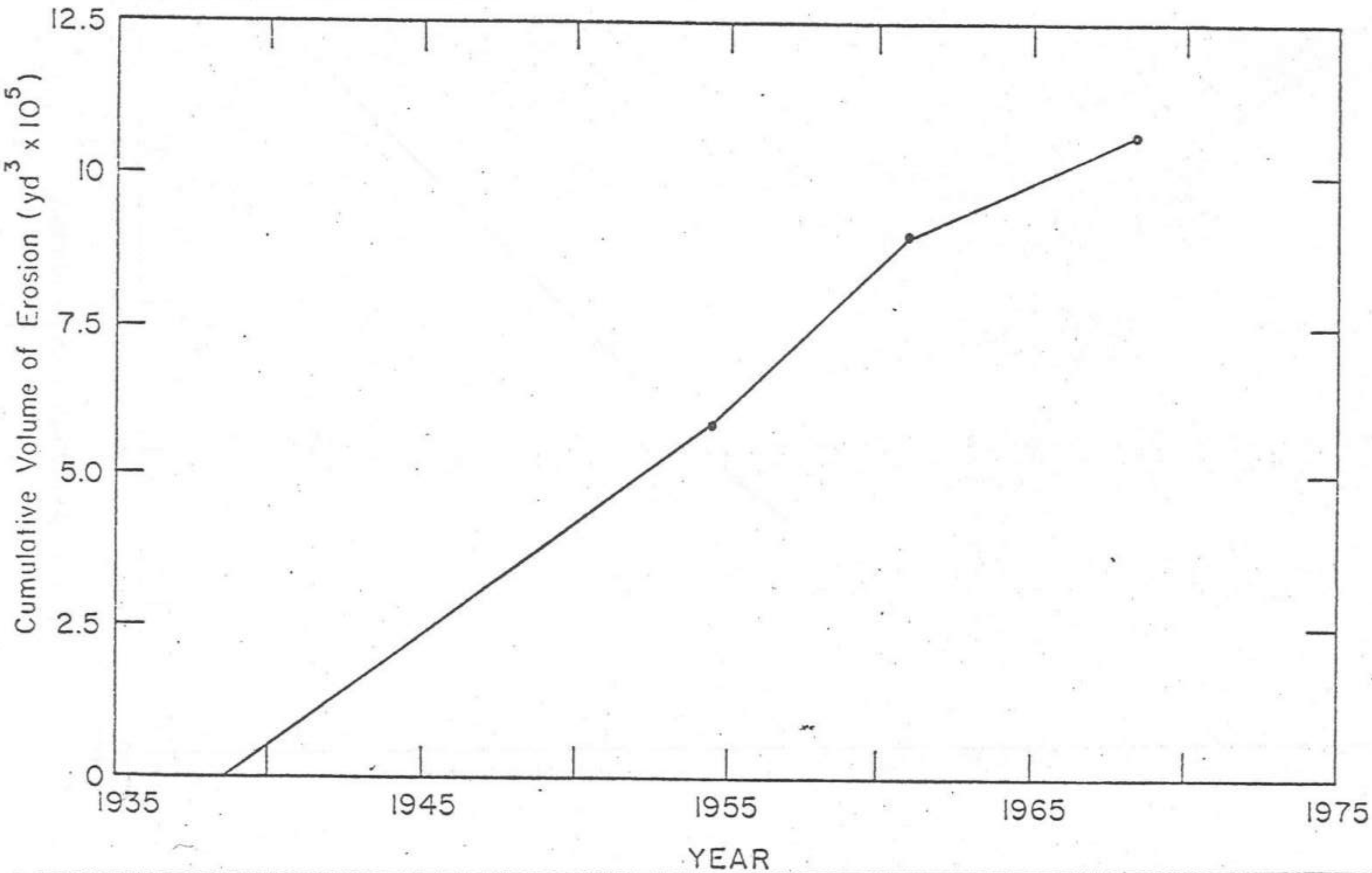

FIGURE 47 Cumulative Volume of Erosion Along Lewes Beach Versus Year (1938-1968, Adjusted for Placement of Beachfill)

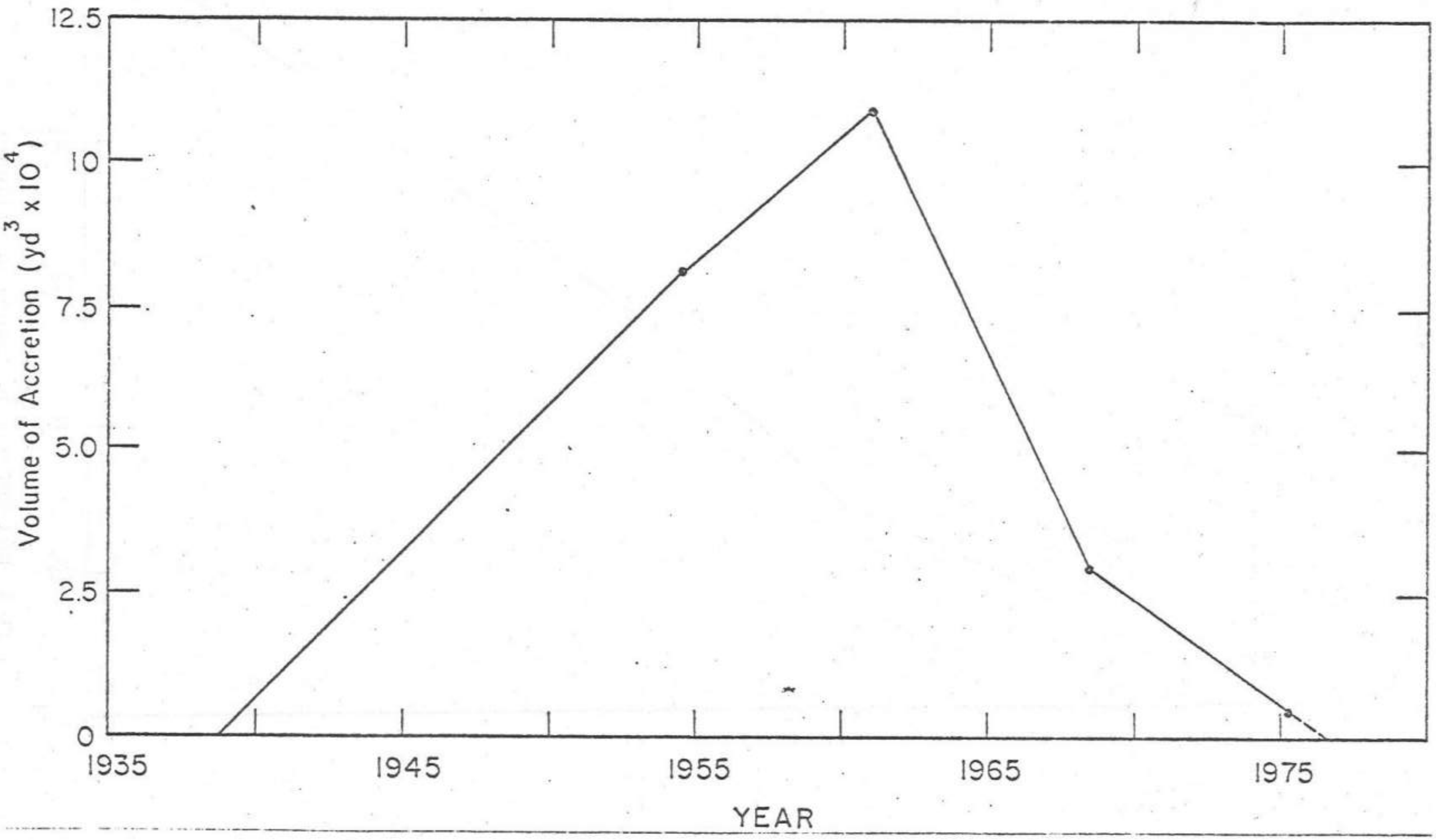

FIGURE 48 Volume of Accretion Along the West Jetty Versus Year (1938-1975, Adjusted for Placement of Material in 1943)

of the map constructed from the survey of April, 1977, from this project with an Army Corps of Engineers map of 1938 confirms that this point has in fact been reached.

The results of total volumetric analysis along the west beach from the west jetty to Line A (see Figure 25) are listed in Table 10. Only two separate time intervals were available for comparison. During the first time span, 1938 to 1954, the rates of erosion were more than two times greater than those of the second time interval, between 1954 and 1968. This is shown graphically in Figure 49; the slope of the first interval being twice that of the second. The total volume eroded along this reach was 665,700 cubic yards between 1938 and 1968, or approximately 22,300 cubic yards per year. The net annual erosion rate per linear foot of shoreline, being approximately -2.0, is slightly higher than the comparable rate along Lewes Beach. The bulk of the volume eroded within this section occurred along the western two-thirds of shoreline (west of Line D in Figure 25). This area is characterized by rapid foreshore erosion, marsh outcroppings and numerous washover features. Figure 50 is a photo showing erosion of a small headland along the west beach. The location of this photo is along the shoreline between Lines B and C of Figure 25. The Broadkill River is seen to have a southerly dip in this vicinity around the section of higher ground. Figure 51 is a photo of one of many exposed relict

TABLE 10 Volumetric Change Along the West Beach

Interval	Quantity (yd^3)			Rate (yd^3/yr-ft)	
	Net	Nourishment	Gross (Net + Nourishment)	Net	Gross
7/5/38 - 7/26/54	-430,500	51,200	-481,700	-2.6	-2.9
7/26/54 - 5/19/68	-184,000	0.0	-184,000	-1.3	-1.3
All Years 7/5/38 - 5/19/68	-614,500	51,200	-665,700	-2.0	-2.2

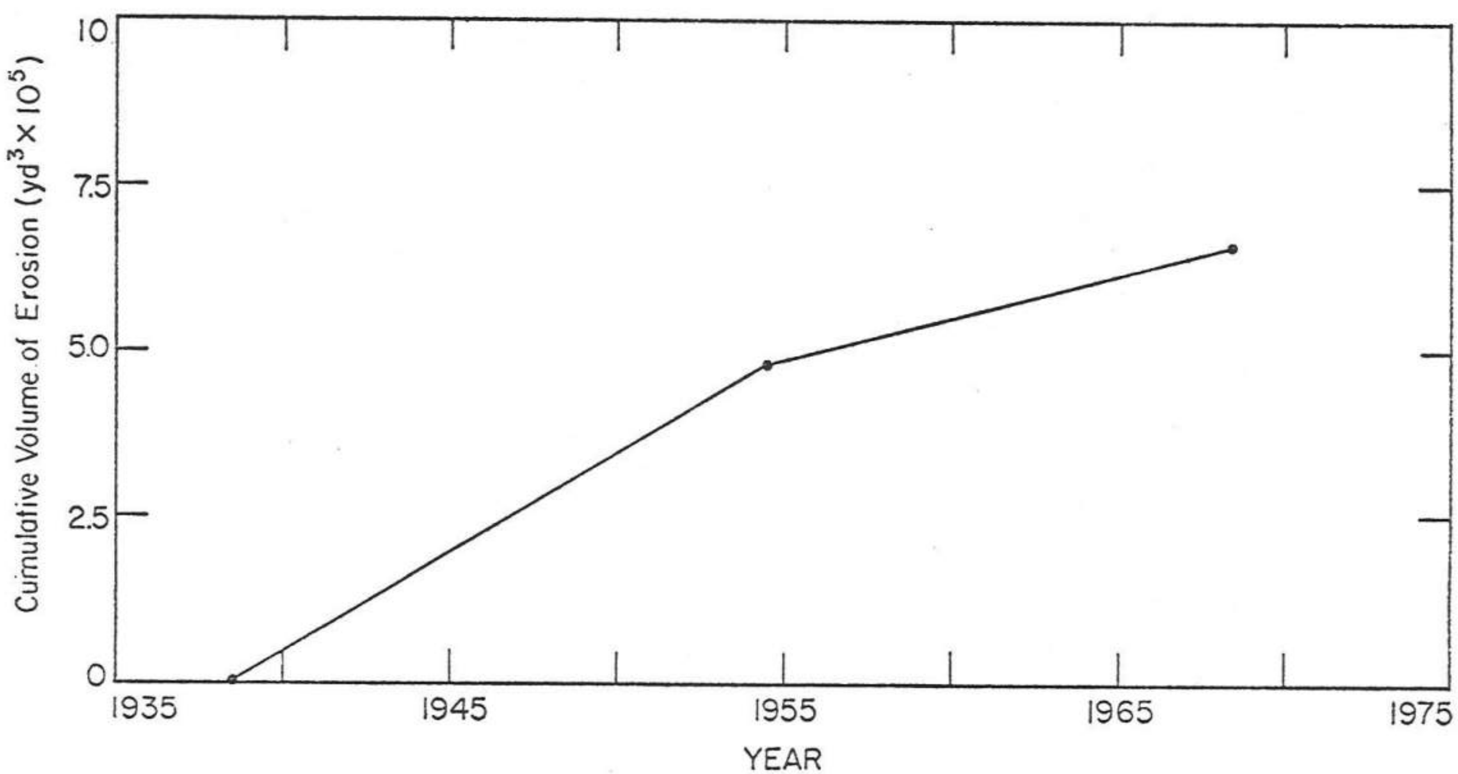

FIGURE 49 Cumulative Volume of Erosion Along the West Beach Versus Year (1938-1968, Adjusted for Placement of Material in 1943)

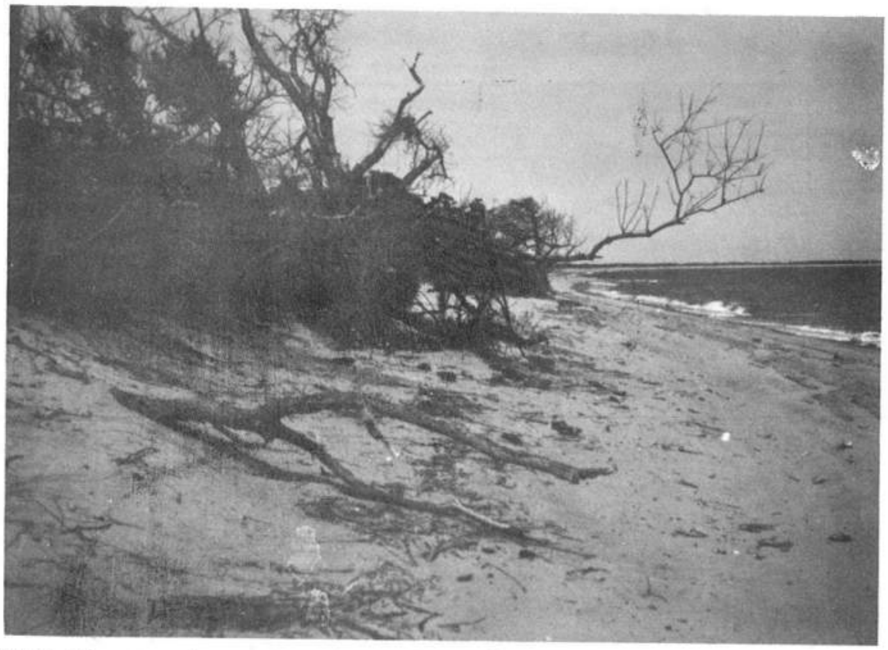

FIGURE 50 Eroding Section of Headland Along the West Beach

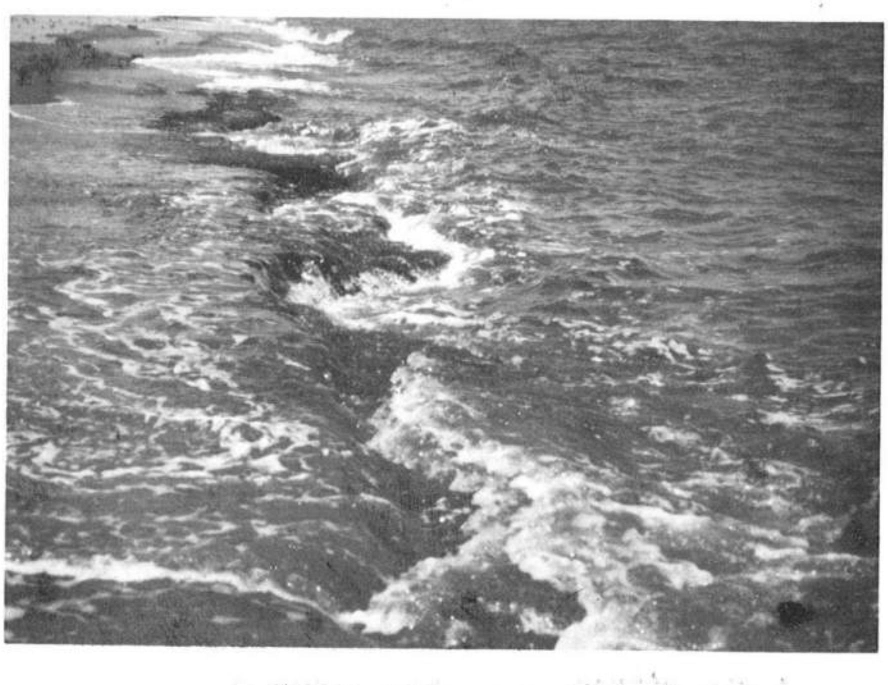

FIGURE 51 One of Many Marsh Outcroppings Along the West Beach

marshes along the region.

The eastern one-third of this section has been somewhat stabilized by the western jetty and also a wrecked barge approximately 2,000 feet from the jetty. The effect of this wreck on the adjacent shoreline is shown in Figure 52. Accretion has occurred on the updrift side and vegetation has advanced bayward behind the wreck, resulting in a more stable shoreline.

6.3 Sand Budget

A sand budget for the vicinity of Roosevelt Inlet will be performed within this section based on dredging estimates (Section 5.1), shoaling rates (Section 5.2), beach nourishment and volumetric change along Lewes and the west beach as discussed in the previous two sections. The continuity equation for sand will be applied to three separate control sections lettered A, B and C in Figure 53 for the years 1937 to 1977.

In applying the continuity equation to Control Section B, one needs to know the initial and final volumes of sand displaced by the inlet, plus all the sand that has entered or been removed between these two times. It was estimated from surveys conducted by the University of Delaware that the present inlet is composed of a total of 666,060 cubic yards of water below mean low water. Prior to the excavation of the inlet a total volume of 275,840 cubic yards of water existed below

FIGURE 52 Stabilization Effect of the Wreck on the Adjacent Shoreline Along the West Beach

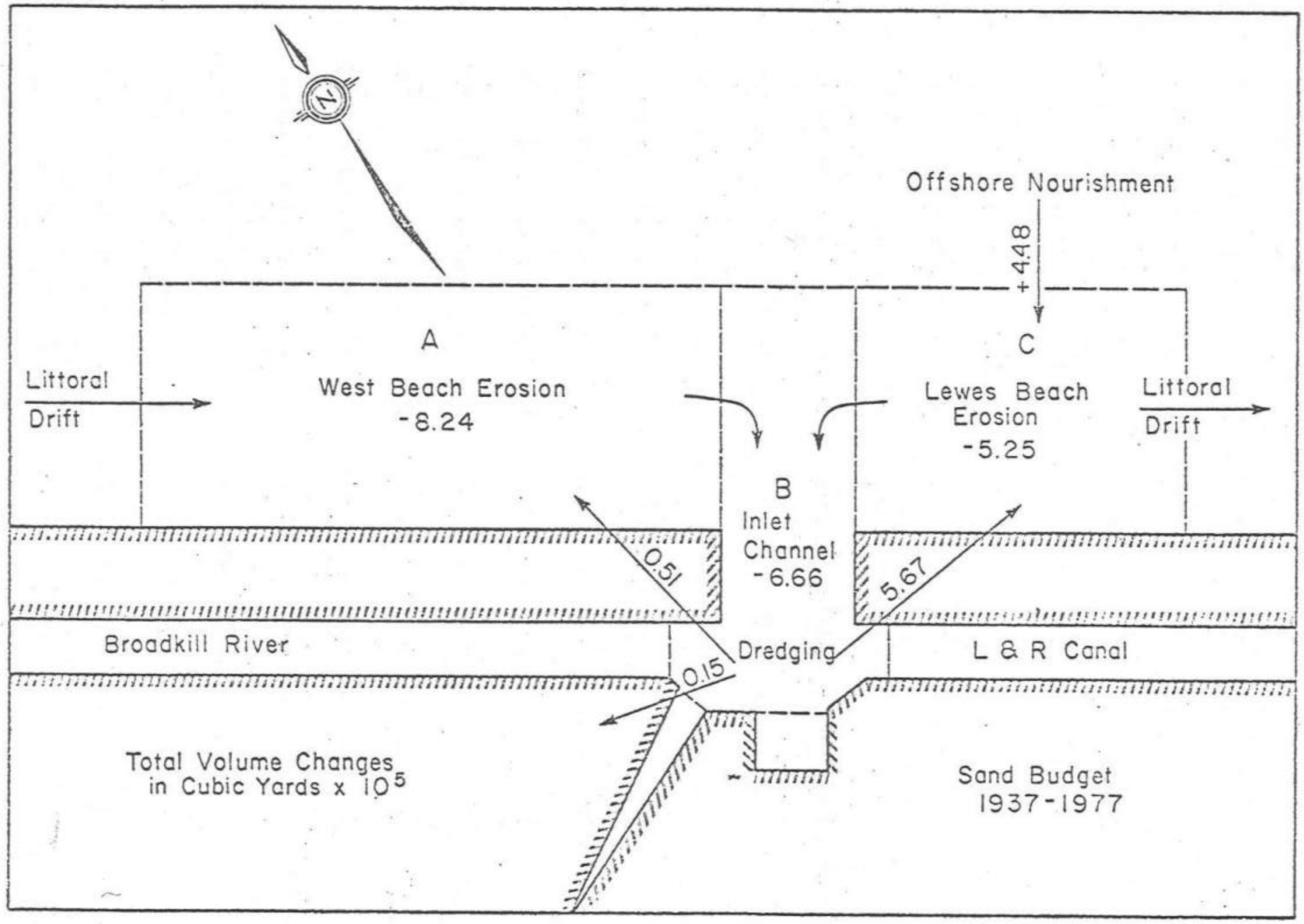

FIGURE 53 Sand Budget for the Vicinity of Roosevelt Inlet

mean low water within the present day offshore inlet boundaries. Furthermore, a total of 95,000 cubic yards of sand above mean low water were present as the beach prior to the initial excavation. Both of these volumes were estimated using an Army Corps of Engineers' chart containing the initial survey for Roosevelt Inlet prior to its excavation dated April, 1935. From Table 6, the total volume dredged to date equals 633,495 cubic yards. Also the total volume excavated during the initial cut equals 521,828 cubic yards which were spoiled along the channel banks and assumed lost to the system. Of this total 521,828 minus 95,000 or approximately 426,830 cubic yards were removed below the mean low water line. To estimate the total volume shoaled, it will be assumed that the shoaling rate calculated in Section 5.2, being 11,000 cubic yards per year, is indicative of the annual rate for the 40-year sand budget. Also it is assumed that the amount shoaled deposits in a two-to-one, west-to-east, ratio as calculated in Section 5.2. From the field study of the western lobe it was found that the lobe accumulated at a yearly rate of 2,400 cubic yards above mean low water. Applying the two-to-one ratio results in a total accumulation of 3,600 cubic yards per year above mean low water. Assuming that this rate of accumulation has occurred over the past 40 years; results — a total volume shoaled of 40 x (11,000 + 3,600) or 584,000 cubic yards between 1937 and 1977.

Organizing the above information into equation form:

Control Section B

Present Inlet Volume = Initial Offshore Volume + Initial Cut - Volume Shoaled + Volume Dredged - Any Losses

666,060 = 275,840 + 426,830 - 584,000 + 633,495 - Losses

Losses = 86,105 cubic yards or 2150 cubic yards per year

These losses could have resulted from slightly incorrect estimates (such as shoaling rate) in the above equation or else they may actually be present within the system. Part of the sand may be lost either into the mouth of the Broadkill or the Lewes and Rehoboth Canal. Also some of the sand entering the inlet, particularly the fine fraction, may be jetted offshore during ebb tide. Other sources of error which may account for some losses are small dredging operations performed by the Lewes Yacht Basin, spoiling the material on their upland property.

In applying the continuity equation to Control Section A, it is possible to solve for the littoral drift entering the system from the west. Of course, this is only possible if it is assumed that no appreciable losses occur via overwash or offshore sediment transport. This assumption will surely introduce some error to the system since this reach is one which is frequently overwashed. However, no quantitative measurements or methods for calculating such losses are available at present.

An estimate must be made for sand passing around and through the west jetty. From Section 5.2, it was estimated that 7,200 cubic yards per year shoal in the western half of the inlet below mean low water. Also, the western lobe accumulates approximately 2,400 cubic yards per year above mean low water.

It is assumed that the volume loss computed in Control Section B enters into the inlet through both the west and east jetties and also in a two-to-one ratio, respectively. This would amount to an additional volume of 28,670 cubic yards entering from the east and an additional 57,330 cubic yards from the west. Therefore, the sand by-passing the west jetty would amount to a total of 40 x (7,200 + 2,400) + 57,330 or 441,330 cubic yards. From Table 6 it is seen that with the first dredging operation approximately 51,200 cubic yards of material was spoiled along the west beach. From Section 6.2, it was estimated that the net erosion along the west beach was 2.0 cubic yards per year per foot. This would result in a total loss of -2.0 (yd^3/yr-ft) x 40 yrs x 10,300 ft or -824,000 cubic yards.

Equating these estimates in the continuity expression:

<u>Control Section A</u>

West Beach Erosion = Littoral Drift + Nourishment - Sand Passing the West Jetty

$-824,000$ = Littoral Drift + $51,200 - 441,330$

Littoral Drift = $-443,870$ yd^3 or $-10,850$ yd^3/yr

For the sand budget calculation for this control section, it was assumed that the littoral drift was entering from the west. It is seen from the above computation that a negative sign has resulted for the littoral drift indicating that the assumed direction was incorrect and that sand is leaving the control section from the western end. This being the case, the littoral drift must change direction within this section (since sand is exiting from both the east and west ends) and hence a nodal point must exist. Using the average recession rates presented in Section 3.4 and assuming that these rates vary linearly between each profile location (A,B, etc.), the nodal point is found to lie approximately 500 feet east of Line B in Figure 25 (based on a 10-foot depth of closure). This is roughly the location of the headland alluded to at the end of Section 6.2. Of course, to say that the nodal point exists at this exact location is quite fictitious. It is more important to know that the nodal point does exist although its position shifts about with time.

It is worth noting that the above littoral drift estimate is somewhat conservative since losses resulting from overwash were unknown and hence neglected. Inclusion of these losses within the sand budget (if possible) would place the nodal point to the west of the one calculated previously.

An estimate of the littoral drift to the east leaving the system can be obtained by applying the continuity equation to Control Section C. Again losses by overwash and to the offshore will be neglected as they are a priori unknown. Sand entering into the section by artificial means in the form of beach nourishment has totalled 1,015,300 cubic yards. Of this total 448,000 cubic yards was dredged from an area approximately 1,000 feet offshore and the remaining 567,300 cubic yards was dredged from the inlet. For an estimate of the amount of sand by-passing the east jetty, one component would be the annual shoaling rate for the eastern half of the inlet below mean low water or about 3,800 cubic yards. Applying the two-to-one ratio for accumulation within the inlet above mean low water results in an average annual rate of 1,200 cubic yards, along the eastern bank. Also the additional amount of 28,670 cubic yards is assumed to have entered the inlet from the east but is eventually lost to the system as mentioned previously in this section. Summing the above estimates results in a total volume of 40 x (3,800 + 1,200) + 28,670 or 228,670 cubic yards that have passed the east jetty into the inlet over the 40-year period. From Section 6.1 it was calculated that the net annual erosion rate per linear foot along Lewes Beach was -1.7 yd^3/ft. This amounts to a total loss to the control section of -1.7 (yd^3/ft-yr) x 40 yr x 7,725 ft or -525,300 cubic yards.

Again arranging the above information into equation form:

Control Section C

Lewes Beach Erosion = Nourishment - Sand Passing the East Jetty - Littoral Drift

-525,300 = 1,015,300 - 228,670 - Littoral Drift

Littoral Drift = 1,311,930 cubic yards or 32,800 cubic yards per year to the east

In an effort to check the above littoral drift estimate, the accumulation east of the hooked breakwater at the Cape May-Lewes Ferry Terminal was planimetered (see Figure 25). The breakwater was built in 1964. Between 1964 and 1968 the volume accumulated was calculated to be 55,500 cubic yards (based on a 10-foot depth of closure) or 13,875 cubic yards per year. Comparing this number with that of the littoral drift estimate indicates that 18,925 cubic yards per year or a total of 757,000 cubic yards for the entire time span are unaccounted for!

The number seems very large; however, there are many errors inherent in the system which could quite possibly total up to a number of that magnitude. First of all, it is highly likely that some of the sand reaching the ferry breakwater is actually transported around it. Recent bottom sampling in this vicinity by the Department of Geology, University of Delaware (Demerest, 1977) indicate that sand has, in fact, accumulated off the tip of the breakwater. Also, this sampling indicates that appreciable amounts of sand mixed with mud are present within the eastern half of Breakwater Harbor. The study by Demerest also indicates that Breakwater Harbor has a shoaling rate of approximately 0.1 of a foot per year. The surface area of Breakwater Harbor is about 2.5×10^7 square feet. Over a 40-year period shoaling in just the eastern half of the harbor would total 3.7×10^6 cubic yards. Assuming only 25% of this is sand results in 9.25×10^5 cubic yards of which some, but not all, surely had eroded from Lewes Beach. This estimate does not even take into account the dredging required to maintain a 300-foot wide channel from the ferry terminal to the western tip of the inner breakwater. This channel which is maintained to 12 feet has had to be dredged approximately every four years since the ferry service began. The presence of this comparatively deep channel (the mean depth in Breakwater Harbor is approximately nine feet) undoubtedly acts as an effective sediment trap much as Roosevelt Inlet does in its present condition.

A more remote candidate for losses to the system is the offshore region. In Section 4.2, it was stated that approximately 45,860 cubic yards of sand had shoaled in the inlet over a period of slightly more than two years. Much of this shoaling had occurred between station 15+00I and 20+00I. This indicates that significant movement of sand is evident this far offshore although admittedly enhanced by the presence of the deep inlet channel. With this fact in mind, it is quite possible that the borrow pit used for the source of beachfill material only 1,000 feet offshore is also refilling, thus accounting for some (although probably small) losses to the system.

A third source of error which would contribute to the volume lost would be the overestimate of volumes reported for effective beachfill. Much of the material used as fill, particularly from the offshore area, contains a high percentage of fines. These fines are easily washed away by normal wave action and do not help in significantly advancing the nourished beach bayward. If the reported estimates of fill only contain 70% sand then significant errors in the sand budget will occur. If 30% of the reported beachfill along the Lewes Beach consisted mostly of fines, an error of approximately 300,000 cubic yards would result.

Considering these three modes of loss, it is believed that sand passing around the hooked breakwater and into Breakwater Harbor is the most likely and the most significant.

CHAPTER VII

Hydraulics

7.1 Development of Numerical Model

In order to gain a better understanding of the complicated hydraulics present at Roosevelt Inlet a numerical model was developed. The model was one-dimensional and encompassed all the bays and waterways from Indian River Inlet to Roosevelt Inlet, including the effect of the Broadkill River. It provided a basis for simulating the tides and the cross-sectionally averaged currents at any location within the system. The tides and currents (discharges) predicted by the model were compared with measured field data of specific locations and gave surprisingly accurate results. No effort was made to "fine tune" or calibrate the model to exactly predict the field data since the simplicity of the model would preclude the accuracy of the measurements and also it was uncertain whether or not these data were representative of the average conditions. The model further predicted the location of the tidal division line, and more important – a mean pumping of water throughout the entire system.

The governing equations used in the model are the depth-integrated equations of motion and continuity. The effect of wind and the addition of fresh water inflow were neglected in the application and development of this model, although they are easily added. The vertically integrated differential equation of motion can be written in a semi-linearized form for flow in the x-direction as follows:

$$\frac{\partial q}{\partial t} = -g\, D\, \frac{\partial \eta}{\partial x} - \frac{\tau}{\rho} \tag{1}$$

where q = discharge per unit width in the x-direction

t = time

g = gravitational constant

D = total depth = h+η

h = depth at mean sea level

η = tide displacement above mean sea level

x = horizontal distance coordinate in flow direction

ρ = mass density of salt water

τ = frictional stress on the bottom of water column

$= \rho f \dfrac{q|q|}{8D^2}$

f = Darcy-Weisbach friction factor

The continuity equation for one dimension is expressed as:

$$\frac{\partial \eta}{\partial t} + \frac{\partial q}{\partial x} = 0 \tag{2}$$

In order for the above equations to be operable for the computer, they must be cast into finite difference form. Also, the bays and waterways of the system must be divided into finite segments. In the operation of the model a time and space staggered procedure is used in which the equation of motion is applied between midpoints of the adjacent segments (i.e., across segment boundaries) at full time steps, Δt, and the continuity equation is applied at each segment at half time step increments.

The finite difference form of Equation (1), expressed in terms of total discharge onto the n^{th} segment Q_n, follows, as:

$$Q_n' = \frac{Q_n - \overline{WD}\, g \left[\eta_n - \eta_{n-1}\right] \frac{\Delta t}{\Delta x}}{1 + \frac{\overline{W} \Delta t f |Q_n|}{8(\overline{DW})^2}} \tag{3}$$

where Δt = time step

Δx = space step

W = segment width

The primed quantities indicate unknown quantities whose values are determined at time $t + \Delta t$, from the known quantities on the right-hand side of the equation. The over-barred quantities represent averages based on the n^{th} and $(n-1)^{th}$ segments.

The continuity equation is expressed in finite difference form as:

$$\eta_n' = \eta_n + \frac{\Delta t}{\Delta x}\frac{1}{W_n}(Q_n - Q_{n+1}) \tag{4}$$

The segment characteristics used in the model are given in Table 11 and their locations in Figure 54. Where small inlet segments connect two very large bodies of water, such as Indian River Inlet and the "Ditches", a Keulegan (1967) type inlet equation is used. The equation may be expressed as an example for Indian River Inlet as:

$$Q_2 = \frac{A_c \sqrt{2g|\eta_1 - \eta_2|}\ \text{sign}\ (\eta_1 - \eta_2)}{\sqrt{K_{en} + K_{ex} + f\ell/4R}} \tag{5}$$

where A_c = cross-sectional flow area of Indian River Inlet

Q_2 = flow onto Indian River Bay from the Atlantic Ocean

η_2 = Indian River Bay tide

η_1 = Atlantic Ocean tide (specified)

K_{en} = entrance loss coefficient = 0.3

K_{ex} = exit loss coefficient = 1.0

R = hydraulic radius of the inlet

ℓ = length of the inlet

f = Darcy-Weisbach friction factor

TABLE 11 Characteristics of Hydraulic Model Segments

Bay and Waterway Segments

#	Length(ft)	Width(ft)	Depth(ft)	f	Description
1					Atlantic Ocean Tide
2	31,000	11,000	6	0.03	Indian River Bay
3	21,000	16,000	6	0.03	Rehoboth Bay
4	4,800	100	3	0.03	Lewes and Rehoboth Canal
5	4,800	100	3	0.03	
6	6.600	75	6	0.03	
7	4,500	100	6	0.03	
8	4,500	100	6	0.03	
9	4,500	100	7	0.03	
10	4,500	100	7	0.03	
11	4,500	100	7	0.03	
12	4,000	150	10	0.03	
13	4,000	150	10	0.03	
14	800	500	12	0.03	Canal-Inlet Junction
15	3,700	150	9	0.03	Lower Broadkill River Section
16	3,700	150	5	0.03	
17	3,700	150	5	0.03	
18	9,000	1,000	3	0.03	Effective Marsh System
19					Delaware Bay Tide

Inlet Characteristics

Length(ft)	Width(ft)	Depth(ft)	f	$K_{en}+K_{ex}$	Description
6,000	800	16	0.03	1.3	Indian River Inlet
4,000	1,000	5	0.03	1.3	"Ditches"
2,000	470	12	0.03	---	Roosevelt Inlet

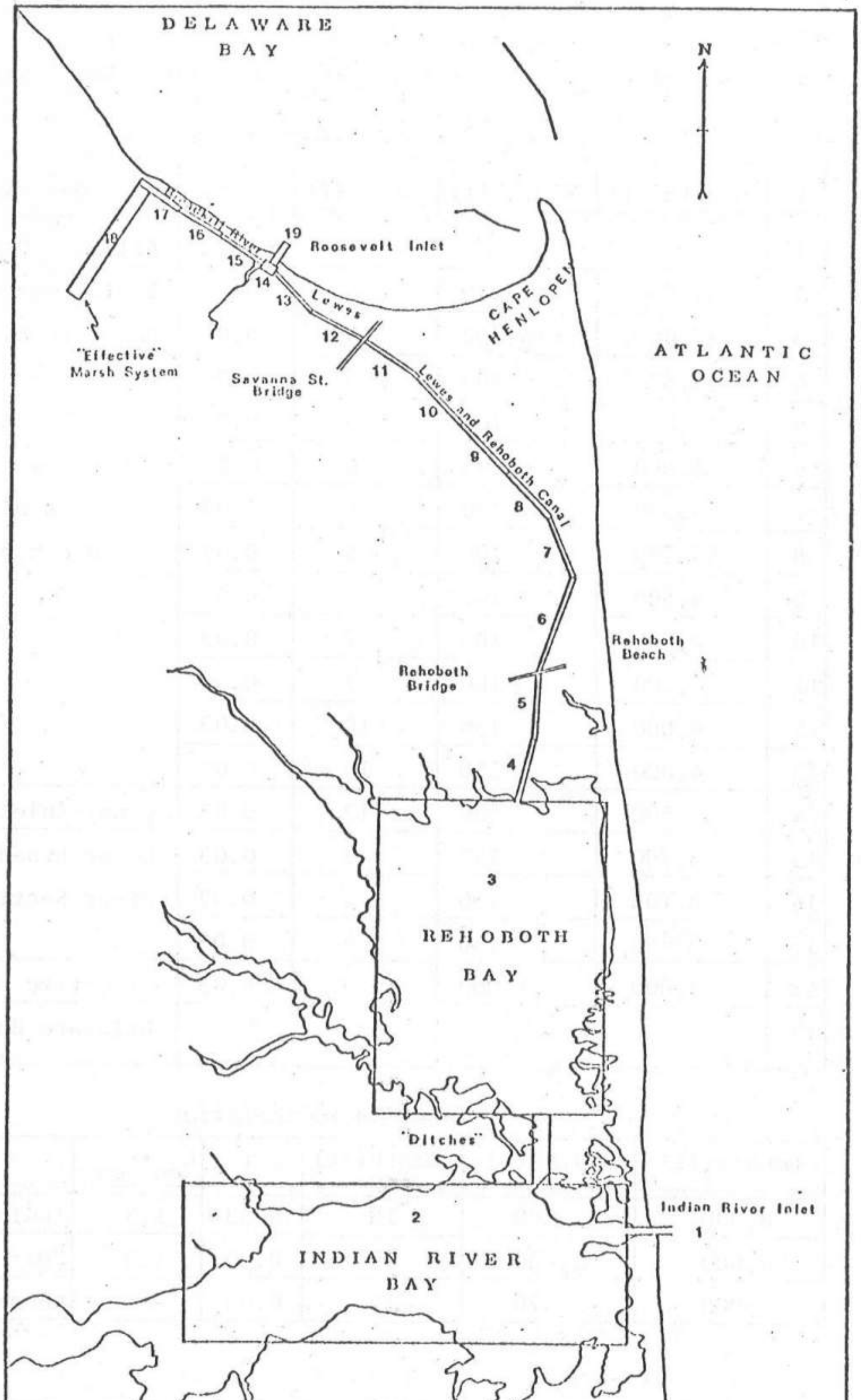

FIGURE 54 Schematization of the Bay and Waterway System, Roosevelt Inlet to Indian River Inlet

The boundary conditions to be specified are the tides outside Indian River Inlet and at Roosevelt Inlet. The tide ranges for Roosevelt Inlet are 4.4 feet mean and 5.2 feet spring (N.O.A.A., 1977). The tide ranges for Indian River Inlet are 3.8 feet mean and 4.6 feet spring (Lanan and Dalrymple, 1977). The tides are assumed to be sinusoidal and have semi-diurnal periods of 12.42 hours. No average lag time was listed in the N.O.A.A. Tide Tables between these two locations. However, the average lag time was calculated to be 0.77 hours using a month's tidal prediction for each location. The tides of Roosevelt Inlet lag behind those at Indian River Inlet.

Much of the preceding section, as well as the development of the numerical model, was based on a similar study at Navarre Pass, Florida (Coastal and Oceanographic Engineering Laboratory, University of Florida, 1973).

7.2 Results and Data Comparison

A field trip was conducted on June 13, 1977, for the purpose of measuring currents and monitoring tide heights. This work was done at two locations, one being the throat of Roosevelt Inlet (station 3+00SW, see Figure 29) and the other the Savanna Street Bridge at Lewes (see Figure 54). The goal of the field trip was to monitor the currents and tide heights at these locations over a complete tidal cycle. Prior to the current measurements, the cross-sectional area of each current station was surveyed in order that the average discharges could be computed. Non-recording Savonius rotor current meters were used for the measurements, which were taken every 15 minutes along the centerline of the cross section, approximately at a depth of one-third the water column. Currents were recorded only for a partial tidal cycle in Roosevelt Inlet, due to equipment failure. The tide heights were monitored by a recording tide gauge in College of Marine Studies Harbor and by a tide staff at Savanna Street Bridge.

The data collected during the field trip were compared with what the model predicted at these locations. Figures 55 and 56 show the measured and predicted tides for the College of Marine Studies Harbor and the Savanna Street Bridge, respectively. In both cases the curves show generally good agreement although the model consistently over-predicts the peaks. Figures 57 and 58 show the comparison of the measured and predicted discharges at these locations. Figure 57 shows that the model under-predicts the discharge at Roosevelt Inlet by as much as 40% at the peaks. The measured and predicted discharges have much better agreement at the Savanna Street Bridge location in Figure 58, especially for the northerly flow.

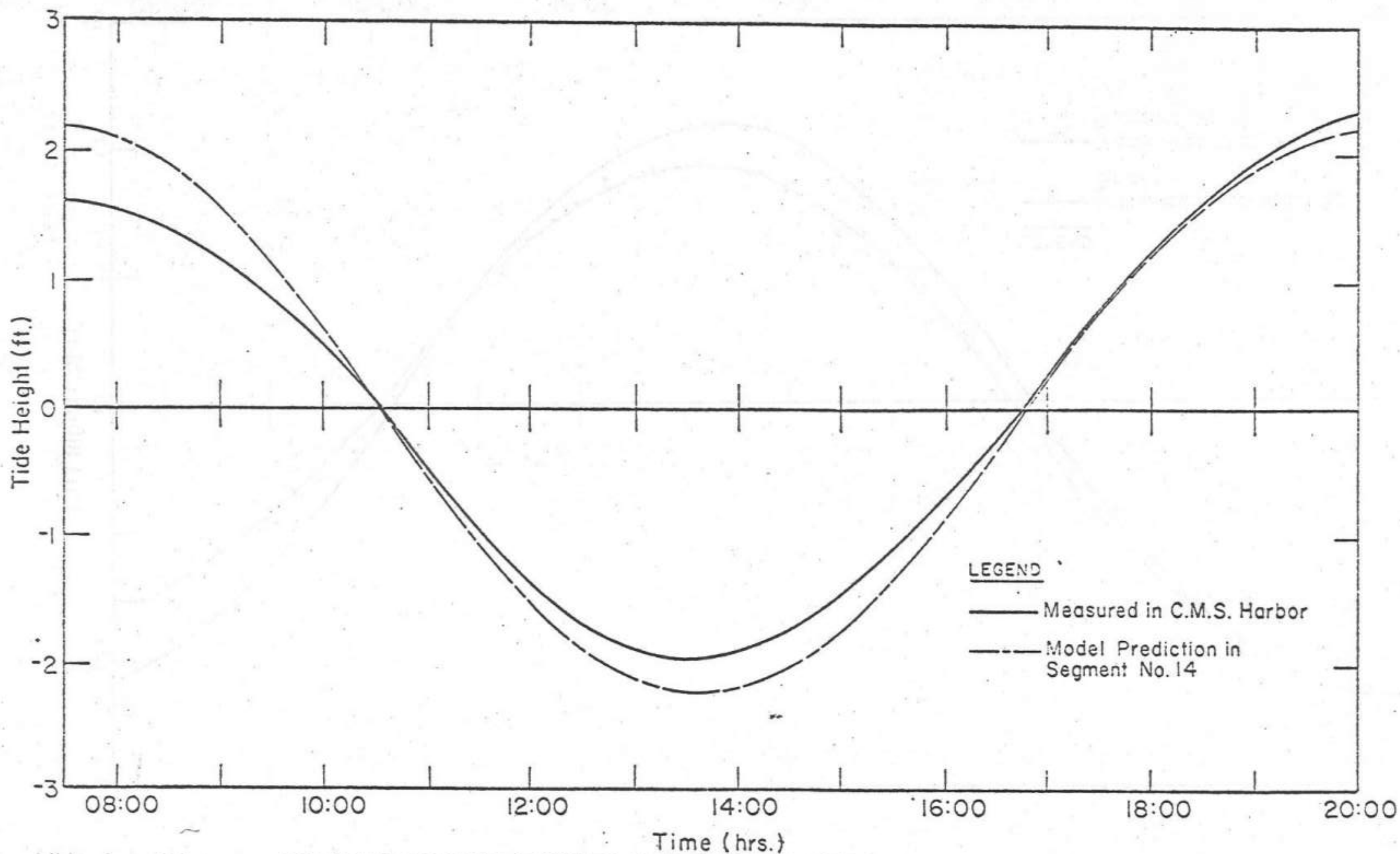

FIGURE 55 Measured and Predicted Tides in College of Marine Studies Harbor

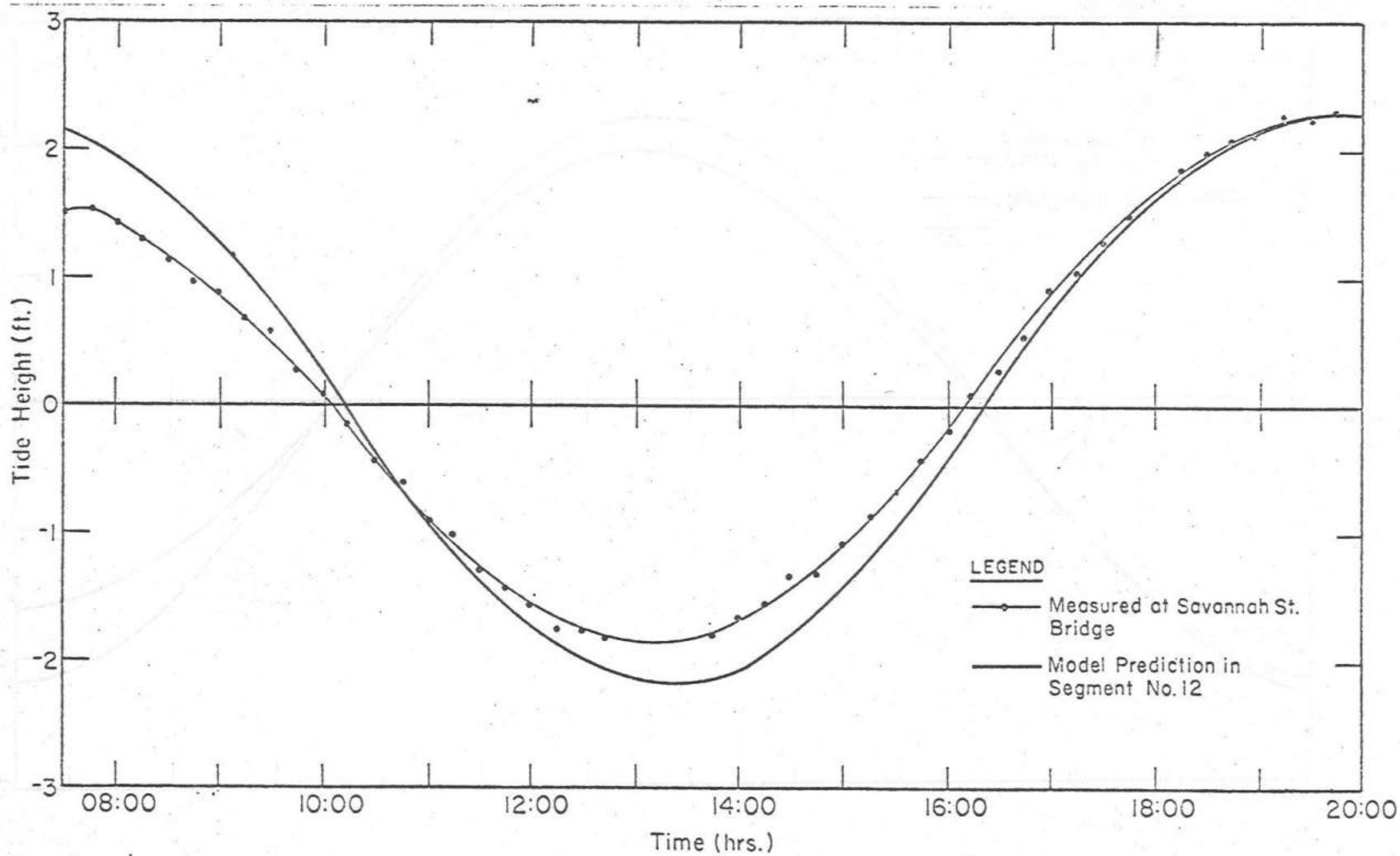

FIGURE 56 Measured and Predicted Tides at Savanna Street Bridge, Lewes

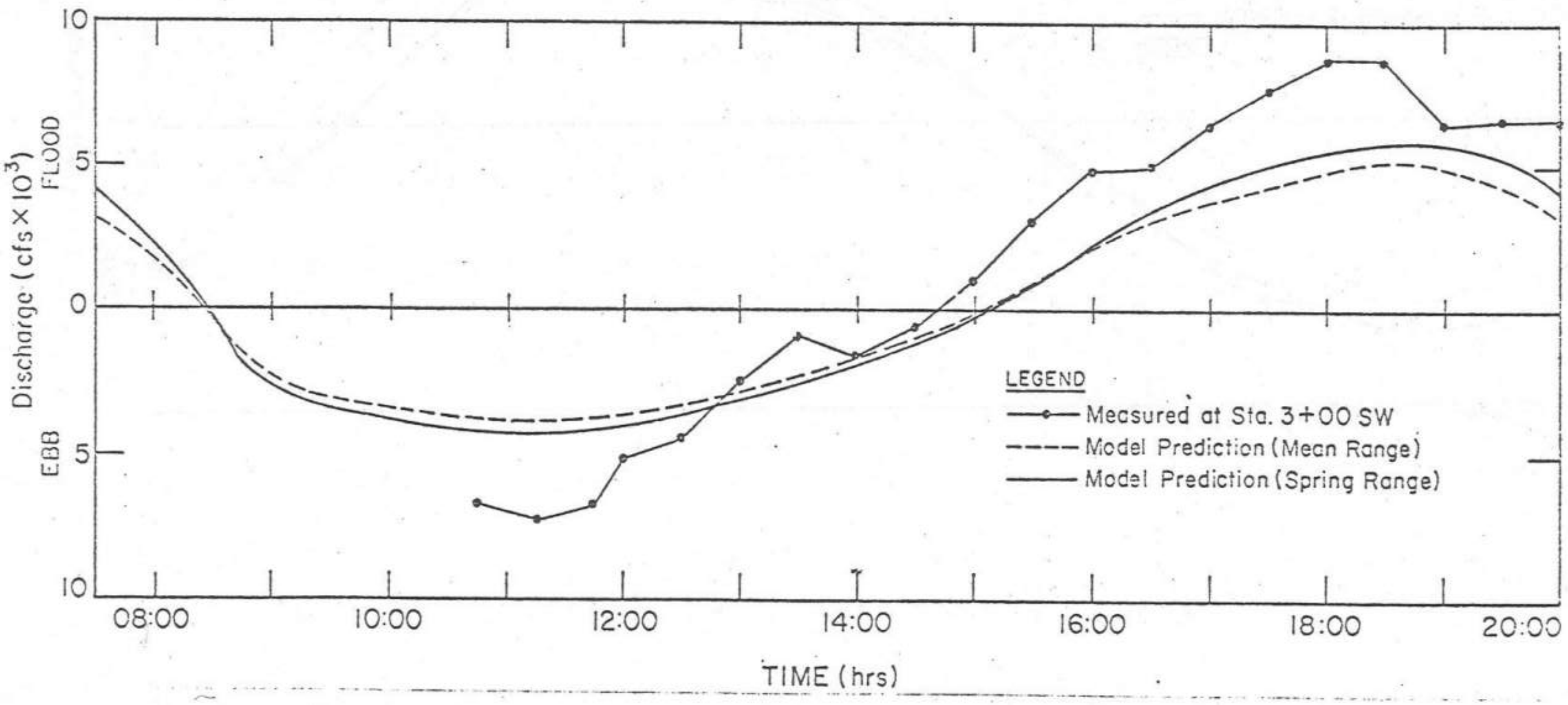

FIGURE 57 Measured and Predicted Discharge at Roosevelt Inlet

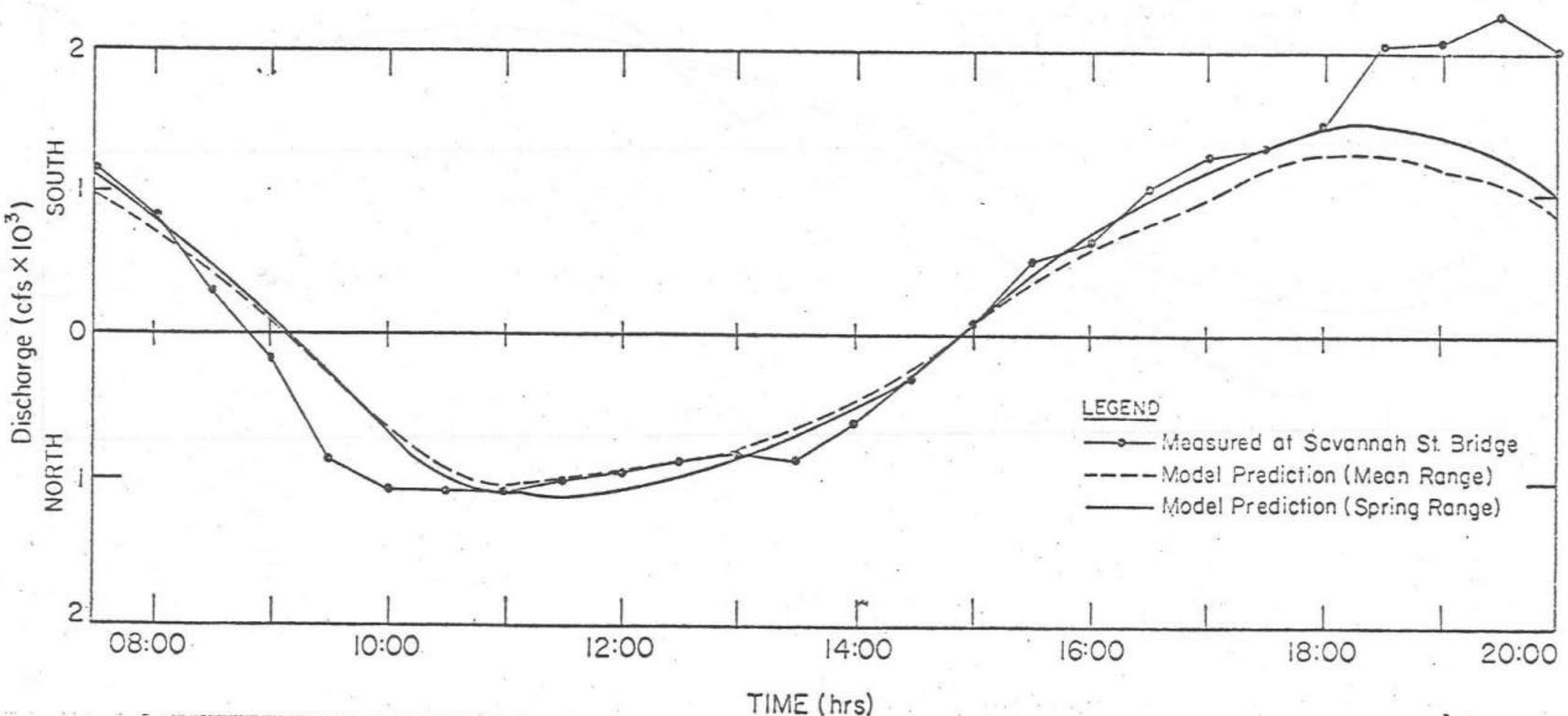

FIGURE 58 Measured and Predicted Discharge at Savanna Street Bridge, Lewes

The mean tide range for each segment as predicted by the model is illustrated graphically versus the distance from Indian River Inlet entrance in Figure 59. The graph shows the gradual dampening of the tide wave through the L&R Canal as one proceeds from the Delaware Bay entrance to its southern entrance into Rehoboth Bay (Segment No. 4). The predicted tide range in Rehoboth Bay is approximately 0.8 feet. The tide ranges listed in the N.O.A.A. Tide Tables (1977) are 0.5 feet mean and 0.6 feet spring for Rehoboth Bay. The predicted tide range for Indian River Bay is 2.4 feet. The N.O.A.A. Tide Tables list the following tide ranges for specific locations in the bay: Indian River Inlet (Coast Guard Station) 2.1 feet mean, 2.5 feet spring; Oak Orchard (5.8 miles west of Indian River Inlet in Indian River Bay) 0.9 feet mean, 1.1 feet spring; Possum Point (10.9 miles west of Indian River Inlet on Indian River) 1.0 feet mean and 1.2 feet spring. These listed values have not been updated for some time and are believed to be changing with time as the characteristics of Indian River Inlet are changing. The inlet cross-sectional area has been increasing over time allowing greater flow to pass into and out of Indian River Bay. More recent tidal records at the Delmarva Power and Light generating station on Indian River suggest a mean tidal range of about 1.7 feet (Lanan and Dalrymple, 1977). Also, measurements by Lanan in 1975 show an average ratio of 1.63 to one between the tidal range in the ocean and the bay (South Shore Marina,

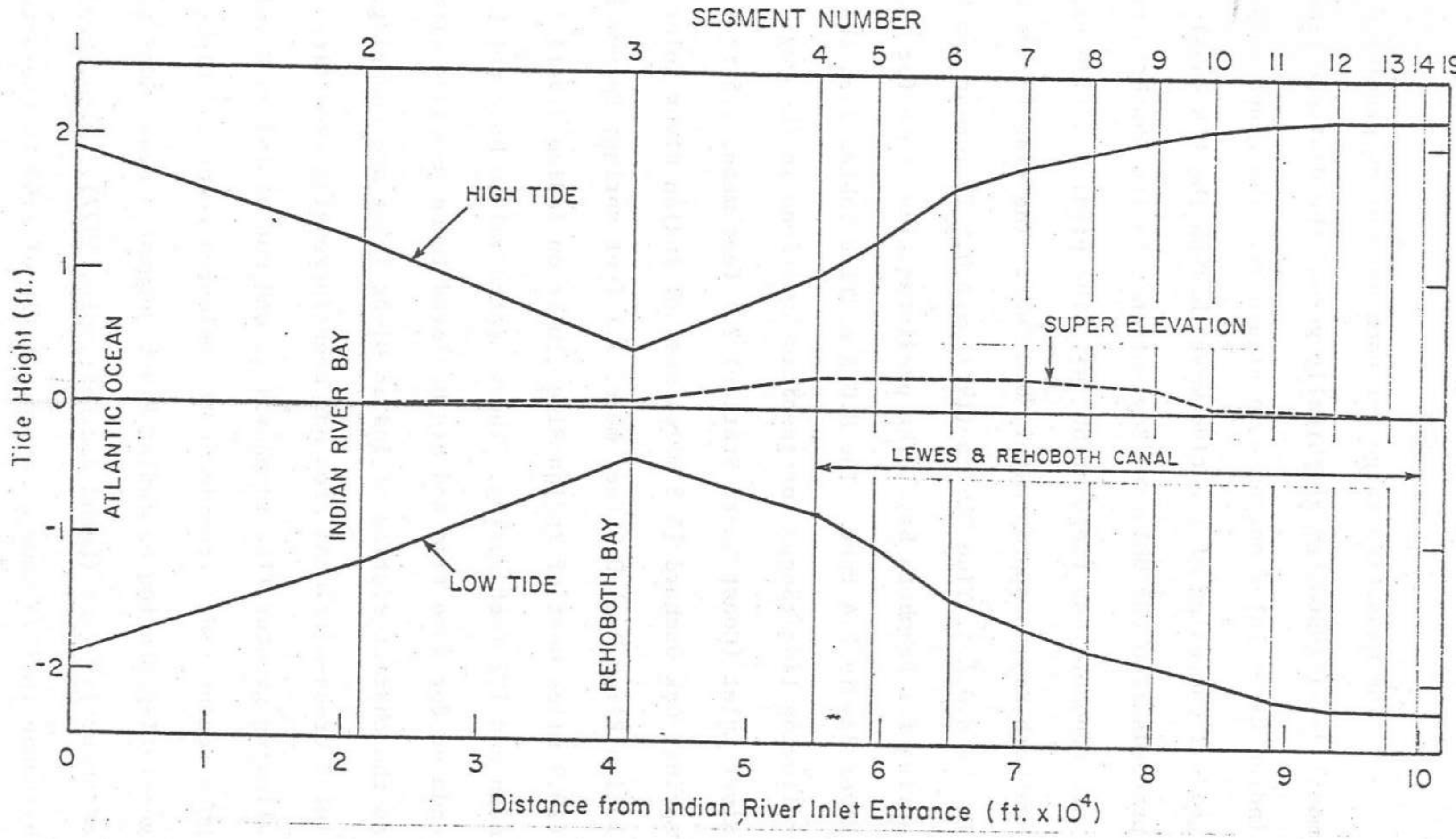

FIGURE 59 Predicted Mean Tide Range Versus Distance From Indian River Inlet Entrance for Each Model Segment

approximately 6,000 feet from the entrance along the south bank). The ratio of ocean tide range to Indian River Bay tide proposed by the model is 1.58. Other measurements by Karpas (1978) on June 9, 1976 near channel markers "1" and "20" located in the middle of the bay (see C&GS Chart No. 411 for exact locations) indicate tide ranges of about 1.9 feet and 2.2 feet, respectively. All in all, the predicted tide range for Indian River Bay may be somewhat high; however, more up-to-date and longer tidal records are needed to accurately determine by how much.

Figure 59 also indicates that within the system the bays and the canal are maintained at a superelevation above the mean water level, even without the influence of fresh water discharge.

Table 12 lists the predicted flow volumes for various locations in the system. The total volume passing the mouth of the Broadkill River is 8.36×10^7 cubic feet. This volume was adjusted in the model by varying the dimensions of the "effective marsh," Segment No. 18 (see Figure 54) to match field measurements by DeWitt (1968). These measurements indicated a total flow volume of $(8.37 \pm 18.1) \times 10^7$ cubic feet per tidal cycle. The effect of Canary Creek was considered small and its flow was assumed adequately accounted for by the "effective marsh."

TABLE 12 Predicted Flow Volumes* for Various Locations

Location	Mean Tide Conditions				Spring Tide Conditions			
	Volume Over One Tide Cycle $ft^3 \times 10^{-6}$		Net Volume $ft^3 \times 10^{-6}$	Mean Flow $ft^3/s \times 10^{-2}$	Volume Over One Tide Cycle $ft^3 \times 10^{-6}$		Net Volume $ft^3 \times 10^{-6}$	Mean Flow $ft^3/s \times 10^{-2}$
	North	South			North	South		
Roosevelt Inlet	59.1	65.4	6.3 S+	1.41	66.6	74.4	7.83 S	1.75
Broadkill River Mouth	41.8	41.8	0	0	47.1	47.1	0	0
Savanna Bridge	13.3	19.6	6.3 S	1.41	14.7	22.6	7.85 S	1.76
Rehoboth Bridge	9.08	15.4	6.3 S	1.41	9.61	17.5	7.86 S	1.76
"Ditches"	253	259	6.6 S	1.49	271	279	8.48 S	1.89
Indian River Inlet	940 (flood)	947 (ebb)	7.15 (ebb)	1.60	1063 (flood)	1072 (ebb)	9.10 (ebb)	2.04

* Numbers Rounded Off After Computation
\+ S or N indicates direction

The tidal prisms predicted for Indian River Inlet of approximately 9.4×10^8 cubic feet (mean) and 10.6×10^8 cubic feet (spring) compare rather well with estimates made by Lanan and Dalrymple (1977). Two estimates were reported by these investigators. One was computed by multiplying the tidal ranges in Indian River and Rehoboth Bays by their respective surface areas resulting in an estimate of the spring tidal prism of 8.3×10^8 cubic feet. A second estimate was computed by integrating current measurements over a tidal cycle and multiplying these by the cross-sectional area of the inlet resulting in a value of 10×10^8 cubic feet.

The results in Table 12 also indicate a net southerly flow is present throughout the entire system. The net volume pumped during each tidal cycle is approximately 6.3×10^6 cubic feet for mean tide conditions and 7.8×10^6 cubic feet during spring tide conditions. This net volume pumped is represented by a mean flow of 141 and 176 cubic feet per second per tidal cycle for mean and spring conditions, respectively (roughly 0.2 - 0.3 feet per second in the L&R Canal). The net volumes listed for all locations seem to indicate that mass is not conserved within the system (i.e., 6.3×10^6 ft^3 enter through Roosevelt Inlet and 7.15×10^6 ft^3 exit through Indian River Inlet for mean tide conditions). The error (18%) is a result of the computer accuracy in performing the integration routine over the tidal cycle. This is particularly evident with the integration of large flow volumes present at the "Ditches" and Indian River Inlet (which are an order of magnitude larger than those through the L&R Canal.

The current data collected and discussed previously in the section also indicate the presence of a net southerly flow. For instance, the graphical integration of Figure 58, of the measured data, results in a net southerly flow volume of 1.0×10^7 cubic feet at the Savanna Street Bridge station, which reduces to a mean flow of 230 cubic feet per second. This value is larger than the predicted value as expected, due to the under-prediction of the discharge by the model.

The mean pumping to the south within the system is thought to be due to the combined effects of the shape of the discharge curve and the friction present within the L&R Canal and the mass transport associated with progressive waves. Figure 60 shows the predicted tides for both the Delaware and Rehoboth Bay as well as the discharge at the midpoint of the L&R Canal. It is of significance to notice the dominance of the (see xii)

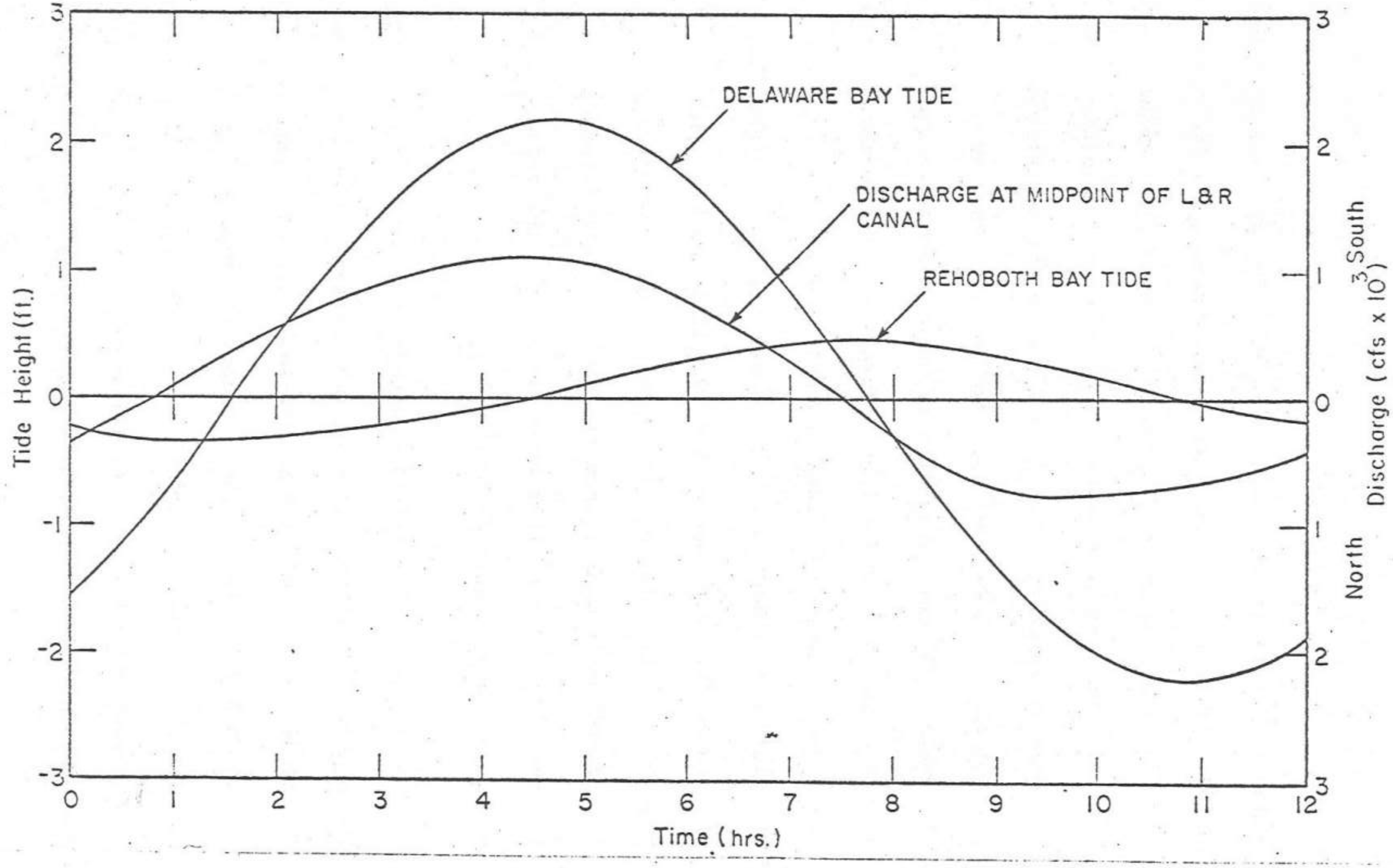

FIGURE 60 Predicted Tides at Delaware and Rehoboth Bay Plus the Discharge at the Midpoint of the Lewes and Rehoboth Canal

Delaware Bay tide range over the tide range at Rehoboth Bay, being more than four times greater. The dominant Delaware Bay tide controls the tide present throughout the canal which gradually dampens as it proceeds from Roosevelt Inlet to the southern entrance entering into Rehoboth Bay. Next take note of the shape of the discharge curve at the midpoint of the canal. The southerly flow is much more bell-shaped, with a higher peak and a longer duration. The northerly flow has a much reduced and flattened peak and shorter duration. The field data also show this general shape (see Figure 58). The reason for the shape is the fact that the high tide wave (crest) travels more freely and faster containing larger volumes of water than the low tide wave (trough) can remove. The shape of this curve is derived theoretically using long wave theory (Appendix I).

The effect of friction within the system is shown by the comparison of the following two figures. Figure 61 shows the predicted time of high tide relative to the ocean tide outside of Indian River Inlet. For instance, the time of high tide at Delaware Bay (Roosevelt Inlet) is 0.77 hours later as specified by the boundary conditions of the model. From the figure it is seen that high tide occurs at about the same time in Indian

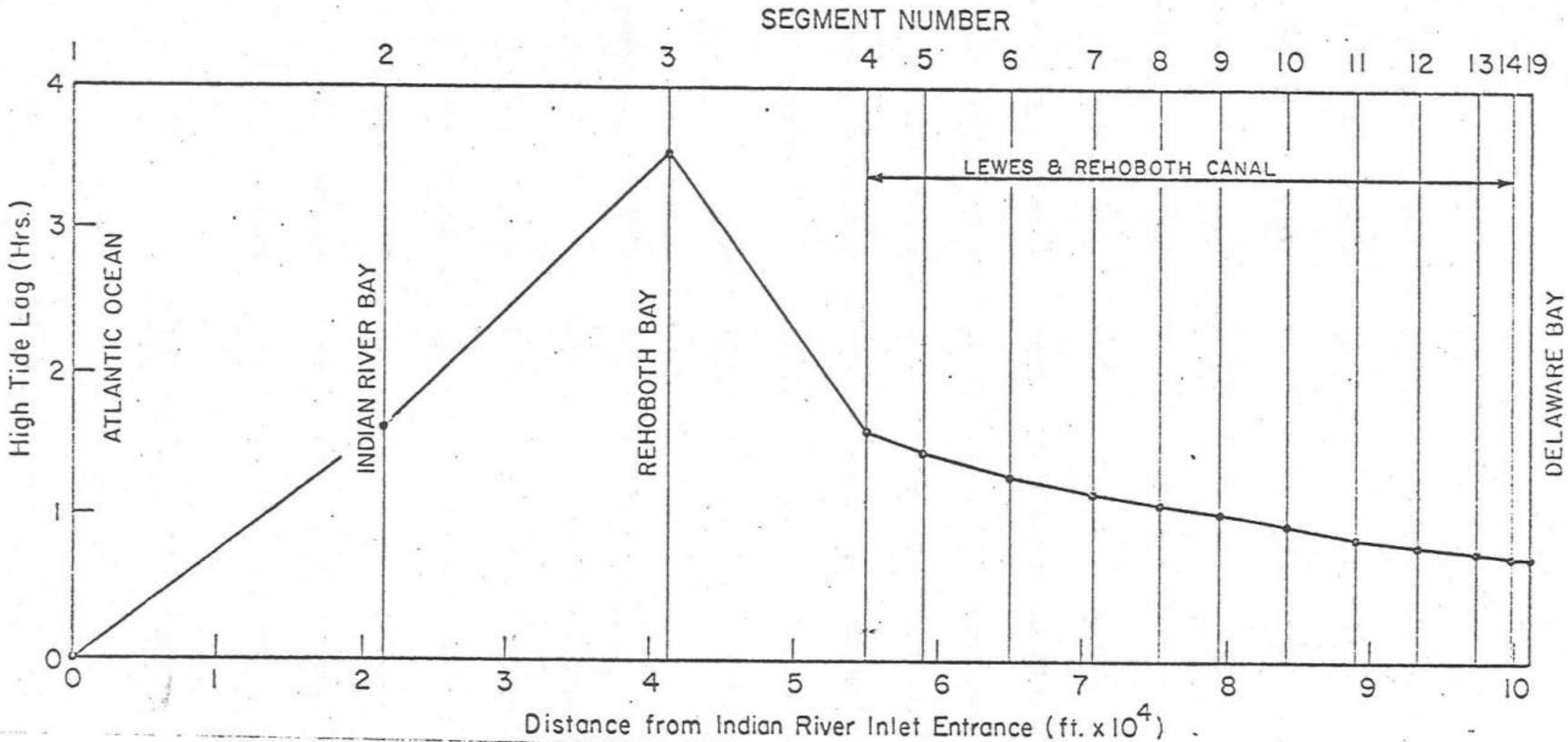

FIGURE 61 Predicted Time of High Tide Versus Distance from Indian River Inlet Entrance

River Bay and in Segment No. 4 at the southern end of the canal. High tide occurs in Rehoboth Bay approximately 1.9 hours later, hence the tidal division line lies within Rehoboth Bay. For comparison, Figure 62 contains the predicted low tide lags relative to the ocean tide. This time it is seen that low tide in Indian River Bay occurs nearly 1.4 hours before it occurs at the lower end of the canal. This indicates that Rehoboth Bay starts to drain through the "Ditches" into Indian River Bay long before it starts to drain into the L&R Canal. Looking at it from a slightly different standpoint, the water pumped into Rehoboth Bay during flood tide finds a less resistant passage towards the south into Indian River Bay than towards the north through the long and narrow canal.

In conclusion it is thought that the mean southerly pumping throughout the system is caused by the dominant discharge propagating completely through the canal into Rehoboth Bay on flood tide, whereupon during ebb tide the Bay drains more favorably towards the south through a less frictionally resistant passage. The consequences of this mean flow through Roosevelt Inlet will be discussed in the following section.

7.3 Applications of the Model

A. Effect of Mean Flow--It has been recognized for some time now that many inlets have the ability to be self-maintaining

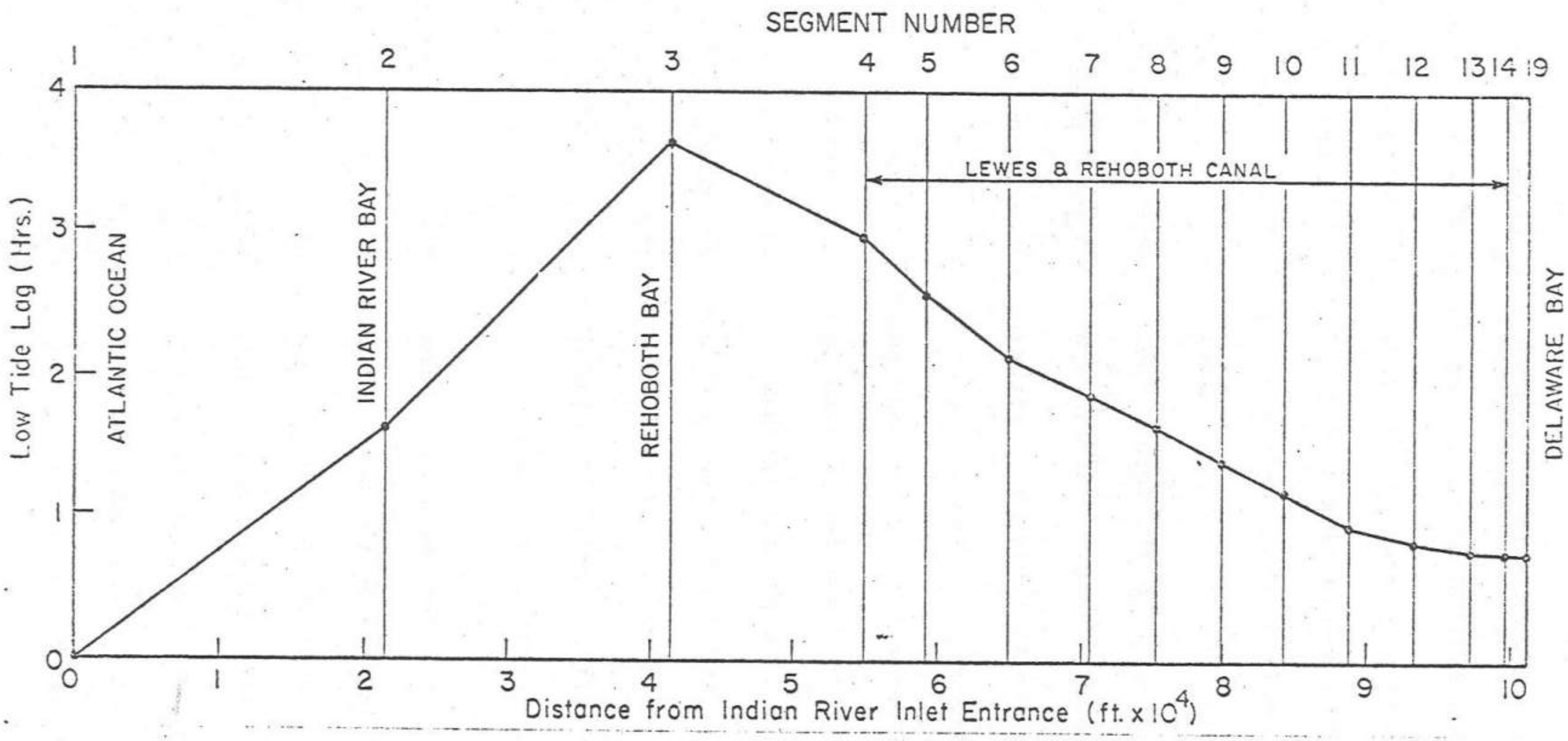

FIGURE 62 Predicted Time of Low Tide Versus Distance from Indian River Inlet Entrance

and are able to achieve an equilibrium condition. When an equilibrium is reached at the inlet mouth, sand entering the channel due to the littoral drift is removed by tidal currents, thus maintaining a unique cross-sectional area. In this condition the forces tending to close the inlet, i.e., the flood tidal currents and the waves supplying the littoral drift, must be balanced by the force supplied by the ebb tidal currents which resist closure. Should these forces be present in an unbalanced state the inlet will either increase or decrease in cross section, depending on the nature of the force bias. For the case of Roosevelt Inlet the mean pumping in the flood direction increases the tendency for closure to occur. Although the average net flow present through the inlet seems rather insignificant (only 141 cubic feet per second for mean tidal conditions as predicted by the model), it will be shown that this small flow results in a rather significant bias of the flood tidal power which is available for sediment transport.

The amount of sediment carried into or out of a channel is dependent on the power available in the ebb and flood flows to move the sediment plus by the amount of sediment supplied to the inlet by littoral transport. In a study by Costa and Isaacs (1975) the effect of an anisotropic flow through an inlet resulting in a bias of tidal power was investigated. Their study showed, using both a physical and a numerical hydraulic model, that the superposition of a small current upon an unbiased tidal flow significantly alters the deposition pattern around the inlet. In fact, the results of their physical movable bed model indicate that a secondary flow of one percent of the main flow directed in the ebb direction results in at least a twelve percent increase in sediment load being carried seaward.

Following the ideas and developments set forth by Costa and Isaacs the numerical model of Roosevelt Inlet and its connecting waterways was adapted to uncover the effect of the net flow on shoaling in the inlet. Within this development it is assumed that the work done in transporting sediment in the flood and ebb directions can be expressed as

$$I_{f,e} \sim \int_{f,e} P(t)\,dt \sim \int_{f,e} \varepsilon V^3(t)\,dt \qquad (6)$$

where $I_{f,e}$ = work accomplished in transporting sediment in the flood and ebb directions

$P(t)$ = power utilized in sediment transport

$V(t)$ = velocity in the inlet as predicted by the model

ε = transport efficiency

The transport efficiency developed empirically by Costa and Isaacs after data presented by Inman is shown to be a function of the stream power as given by

$$\varepsilon = 0.01 \left[\left(\frac{V}{V_C}\right)^3 \right]^{1.86} \quad , \quad {V_C}^3 < V^3 < 5{V_C}^3 \tag{7}$$

where V_C = velocity at which incipient motion begins.

The work done on sediment transport for both flood and ebb tide at Roosevelt Inlet was computed from the numerical integration of Equation (6). The critical velocity criteria were based on a graph developed by Hjulström (1935) contained in Graf (1971) as is shown in Figure 63. This curve was based on data using uniform bottom particles and the average flow velocity. The limiting zone at which incipient motion starts is shown as a shaded region between the sections labelled "erosion" and "transportation." It was assumed that for the range of particle sizes present in the inlet, approximately 0.4mm to 1.0mm, 20 cm/sec or 0.66 feet/sec would be representative of the critical velocity. The maximum velocities predicted by the model for Roosevelt Inlet were approximately one foot per second for mean tidal conditions.

The results of the integration of Equation (6) are given in Table 13. It is readily apparent that the mean pumping into the inlet results in a significant bias of the available tidal work to transport sediment into the inlet. In fact, the results show that the work available for transporting sediment is

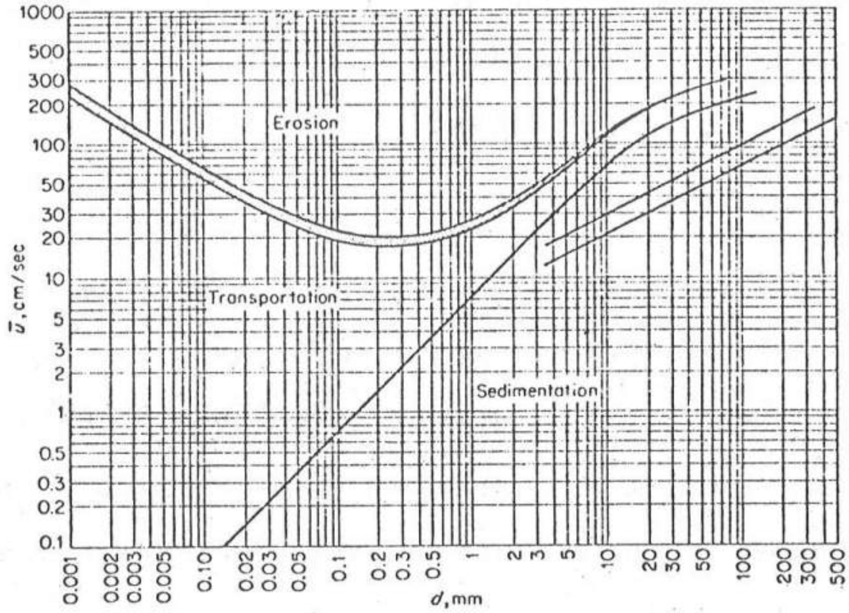

Figure 63 Erosion–Deposition Criteria for Uniform Particles (After Hjulstrom (1935)).

TABLE 13 Sediment Transport Work Per Tidal Cycle at Roosevelt Inlet

Tide Condition	Available Work Per Tidal Cycle (ft-lbs)			Work Ratio Flood/Ebb
	Flood	Ebb	Net	
Mean	148	26	122 F	5.7
Spring	470	72	394 F	6.2

approximately six times greater for flood than for ebb. Also approximately three times as much work is accomplished during spring conditions than during mean tidal conditions. The increase in transport work is expected during spring conditions since the velocities are greater than those present during mean tidal conditions. Also, the velocities are above V_C for longer durations.

The flood/ebb and spring/mean biasing both serve to show the dominance of the peak flows on the total sediment transport work. It is not surprising that such a large bias exists remembering the shape of the discharge curve shown in Figure 60, and the fact that the transport power is proportional to the third power of the velocity or perhaps greater (see Equation (6)).

The actual numbers presented in Table 13 have little meaning by themselves and are perhaps considerably under-predicted. The major point of significance lies in the comparison and resulting ratios of these numbers, which indicate a strong tendency for closure to occur at Roosevelt Inlet.

B. Inlet Stability--In this section the sedimentary stability of Roosevelt Inlet will be further investigated, combining the concepts developed by Escoffier (1940), O'Brien (1969), and Jarrett (1976). These investigations will further indicate the inlet's susceptibility to closure.

The criterion for sedimentary stability of an inlet was first investigated by Escoffier. This concept relates the maximum inlet velocity, V_{max}, with the inlet cross-sectional area,

A_c, as illustrated in Figure 64 (after O'Brien and Dean, 1972). Cross sections to the left of the peak lie within an unstable region which is frictionally dominated. Changes in inlet cross sections within this region tend to perpetuate further changes.

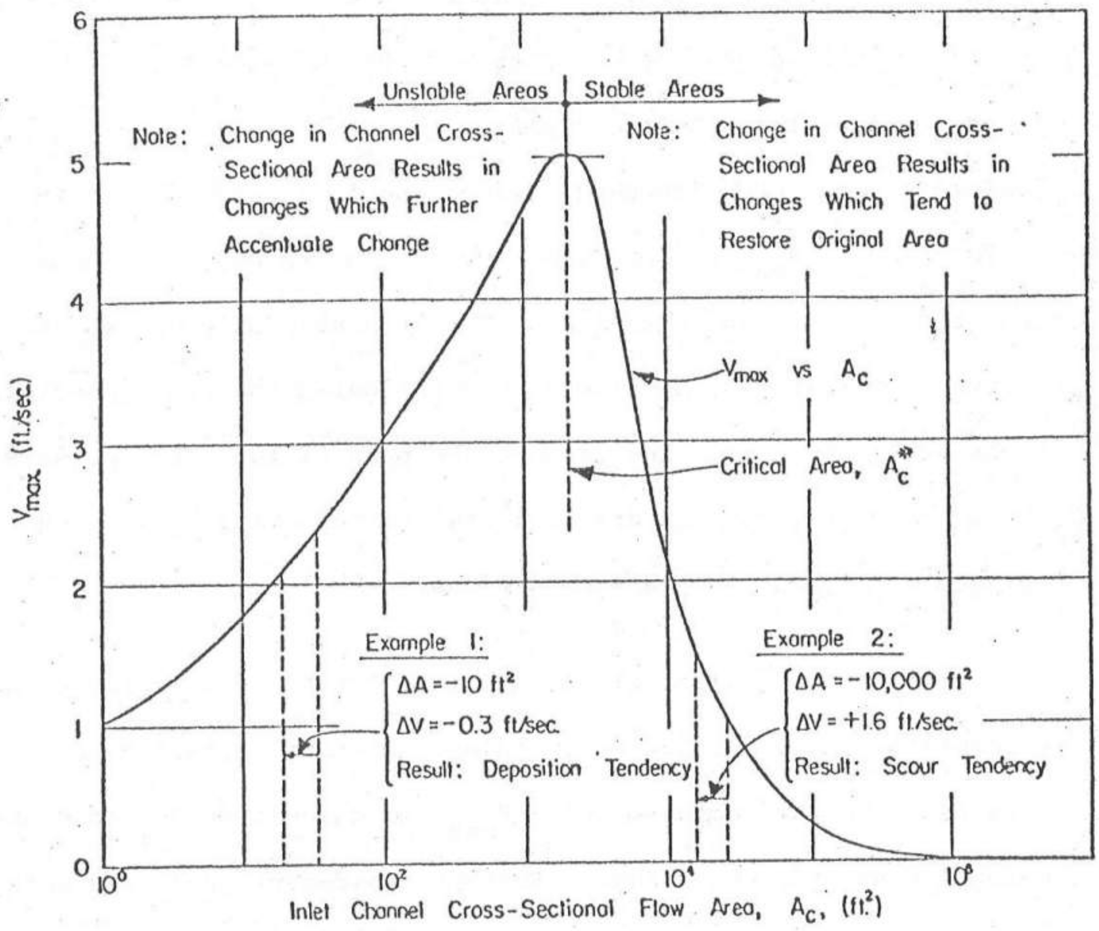

Figure 64 Illustration of Escoffier's Stability Concept (O'Brien and Dean, 1972)

For example, suppose the inlet cross section is reduced by a period of abnormally high littoral drift. From the graph it is seen that a decrease in A_c results in a reduction in V_{max} which further reduces the capability to scour the inlet and so on. A similar argument holds for an increase in A_c, within the unstable area, which results in an increase in V_{max} and thus an increased tendency for scour to result. Conversely, cross sections lying to the right of the peak are stable against change. For instance, a decrease in A_c will result in an increase in V_{max} and an increase in scouring capacity, and thus a return to the initial A_c . This resistance to change and return to the initial A_c also occurs following the enlargement in cross-sectional area. The area at the peak of the curve possessing the greatest V_{max} is the critical area representing a division between stable and unstable conditions.

The numerical model was utilized for the generation of such a stability curve for Roosevelt Inlet. Historic cross sections were fed into the computer and a V_{max} was calculated for each one using spring tide conditions. Further cross-sectional area data were generated assuming the area could continually decrease but the width could not become less than 200 feet. The resulting curve is shown in Figure 65. It is seen that the inlet has always been

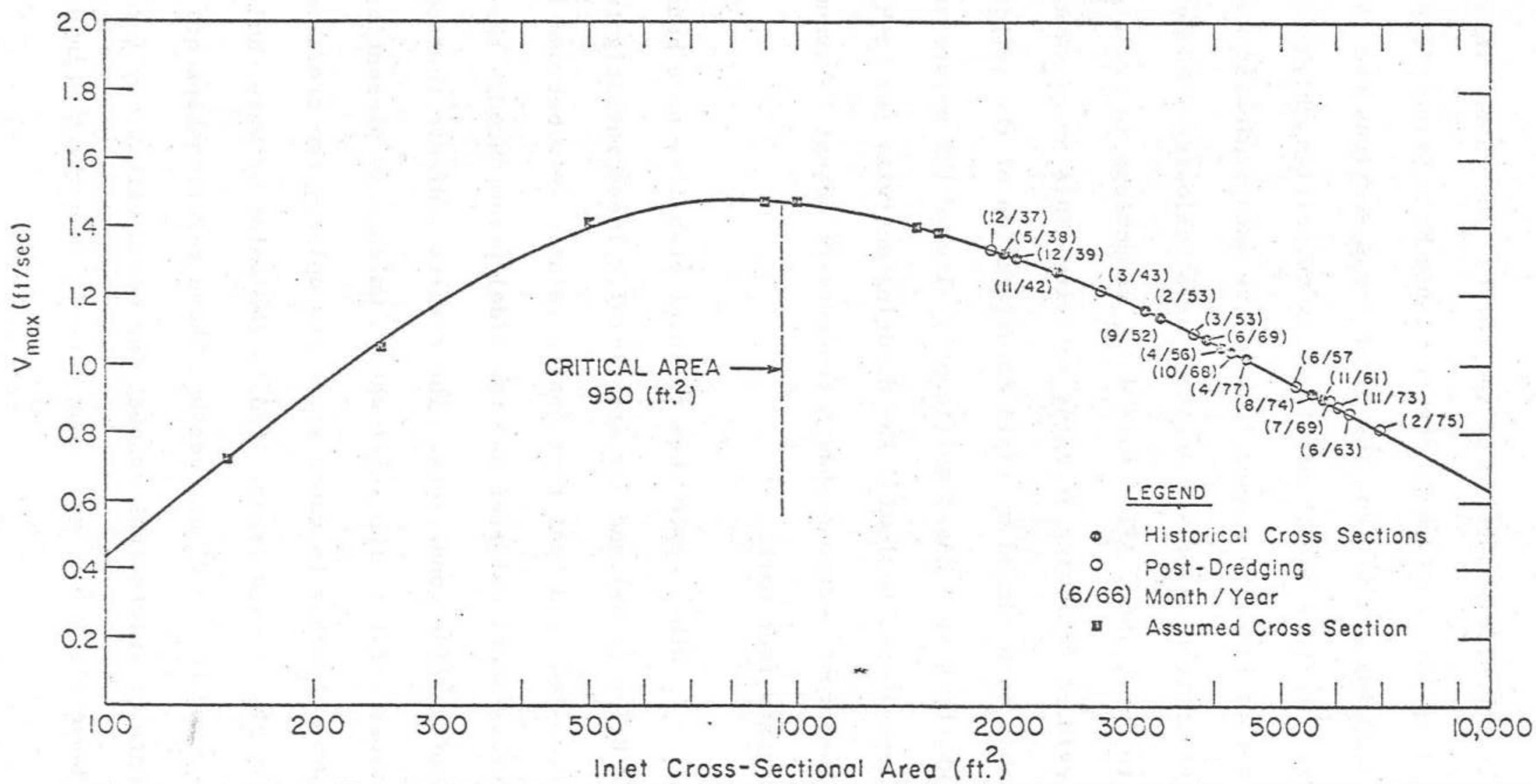

FIGURE 65 Stability Curve for Roosevelt Inlet

in the stable portion of the curve. The change in cross section along this portion of the curve has been principally dominated by dredging activities. The older cross sections are found nearer to the critical area,hence the peak velocities, higher than those of more recent time. A closer look at the data indicates a general trend of increasing cross-sectional area following dredging activity prior to June, 1963. After this date, reductions in cross section are evident following dredging activity. This trend reversal has presumably been caused by rapid deterioration of the jetties during 1962 which in turn has significantly altered the nature and rate of deposition. Following the dredging activity for both of these trends the cross sections show a tendency to adjust to around the 4,000-square-foot mark.

Other approaches to inlet stability have been presented by O'Brien (1969) and Jarrett (1976). These investigators have compiled data that represent a relationship between the inlet throat cross-sectional area and the tidal prism through the inlet during spring tide conditions. The results indicate that equilibrium conditions exist in the inlet when a balance is present between the tidal prism tending to cause scour and enlarge the cross section and the supply of sand transported to the inlet by waves and currents tending to reduce the cross section. These relationships are mostly based on data of inlets that connect the ocean with a bay (or bays); thus Roosevelt Inlet, providing connection between Delaware Bay and the

L&R Canal, is a unique case. With this in mind, the effect of the mean flow on the sedimentary stability of the inlet as discussed in the previous section somewhat supercedes the prism-area relationships at hand. However, it is worthwhile to investigate these relationships for Roosevelt Inlet as if the mean flow were not present to gain further insight in its stability against closure.

In 1969, O'Brien updated his original study of 1931 and concluded that the prism-area relationship could be expressed as $A = 4.69 \times 10^{-4} P^{0.85}$ for inlets with two jetties, where A is the minimum cross section of the entrance below mean sea level in square feet and P is the tidal prism corresponding to the spring tide range in cubic feet. This equation was developed using data mostly from Pacific coast inlets. The study also resulted in an equation for unjettied inlets: $A = 2.0 \times 10^{-5} P$.

Jarrett's study of 1976 comprised a more extensive data search, investigating 108 inlets of which 59 were located along the Atlantic coast. This analysis resulted in the formation of 11 empirical equations, two of which are pertinent to this report. The relationship of $A = 7.75 \times 10^{-6} P^{1.06}$ is representative of all inlets along the Atlantic Coast. From the data concerning unjettied or single jettied Atlantic coast inlets the equation $A = 5.37 \times 10^{-6} P^{1.07}$ was developed.

The spring tidal prism for Roosevelt Inlet as predicted by the model of approximately 7.44×10^{7} ft^3 was entered into the four preceding equations to ascertain the equilibrium cross-sectional throat area. The results are summarized in Table 14. The present cross-sectional area at the inlet throat is 4,670 square feet. According to the prism area relationships the present cross section is much larger than the predicted equilibrium cross section. This being the case deposition and reduction in the present cross section is expected.

It is quite possible that the equilibrium cross sections listed in Table 14 may be under-predicted,

TABLE 14 Predicted Equilibrium Area for Roosevelt Inlet

Equation	Equilibrium Area (ft^2)
O'Brien	
$A = 4.69 \times 10^{-4} P^{0.85}$ (2-jetties)	2300
$A = 2.0 \times 10^{-5} P$ (unjettied)	1490
Jarrett	
$A = 7.75 \times 10^{-6} P^{1.05}$ (all inlets - Atlantic coast)	1430
$A = 5.37 \times 10^{-6} P^{1.07}$ (unjettied or single jettied - Atlantic coast)	1420

one reason being that the equations developed by O'Brien and Jarrett were all based on inlets along the open ocean coasts. The wave climates at such locations would be much higher than expected along the southern shore of Delaware Bay. This being the case Roosevelt Inlet could maintain a cross-sectional area equal to that of an inlet along the ocean coast using a smaller tidal prism based on the present equations.

A second reason could be the model's under-prediction of the measured discharges in Roosevelt Inlet. For purposes of comparison assume that the tidal prism can be expressed as follows:

$$P = A_c \int_0^{T/2} V_{max} \sin \frac{2\pi t}{T} \, dt = \frac{A_c V_{max} T}{\pi} = \frac{Q_{max} T}{\pi} \qquad (8)$$

where T is the semi-diurnal tidal period of 44,700 seconds. Using the maximum discharge, Q_{max}, from the actual data (approximately 8,700 cubic feet per second in equation 8) gives a tidal prism of 1.12×10^8 ft^3. Using this estimate in the four equations of Table 14 results in equilibrium areas of 3,250, 2,230, 2,190, 2,200 square feet, respectively. These values are much closer to present cross section; however, they are still smaller.

Using Equation 8, the tidal prism-area relationships can be rewritten in a form relating V_{max} and A_c. The rearrangement results in O'Brien's equations taking the form $V_{max} = 0.579\ A_c^{0.1765}$ (two jetties) and $V_{max} = 3.5$ feet per second (for the unjettied case). For Jarrett's equations $V_{max} = 5.18\ A_c^{(-0.048)}$ (for all Atlantic coast jetties) and $V_{max} = 5.92\ A_c^{(-0.065)}$ (for unjettied or single jettied Atlantic coast inlets). In this form these equations may be plotted with the stability curve. Where intersection occurs between the stability curve and the prism-area curve the inlet is expected to reach an equilibrium condition satisfying both hydraulic and sedimentary properties. These curves are plotted in Figure 66. It is seen that the stability curve lies below the prism-area curves for all cross sections indicating a strong tendency for closure to occur.

In conclusion, the reduction of the present inlet cross-sectional area is expected to occur. The presence of the mean flow in the flood direction is considered the dominant factor in causing closure. Also the prism-area formulation indicates that reduction in the inlet throat is highly probable. Whether complete closure would result in the absence of maintenance dredging is uncertain; however, it is likely that the undredged inlet would reach a cross section which is unacceptable for present navigational purposes. In this regard, it is evident that continued maintenance dredging will be

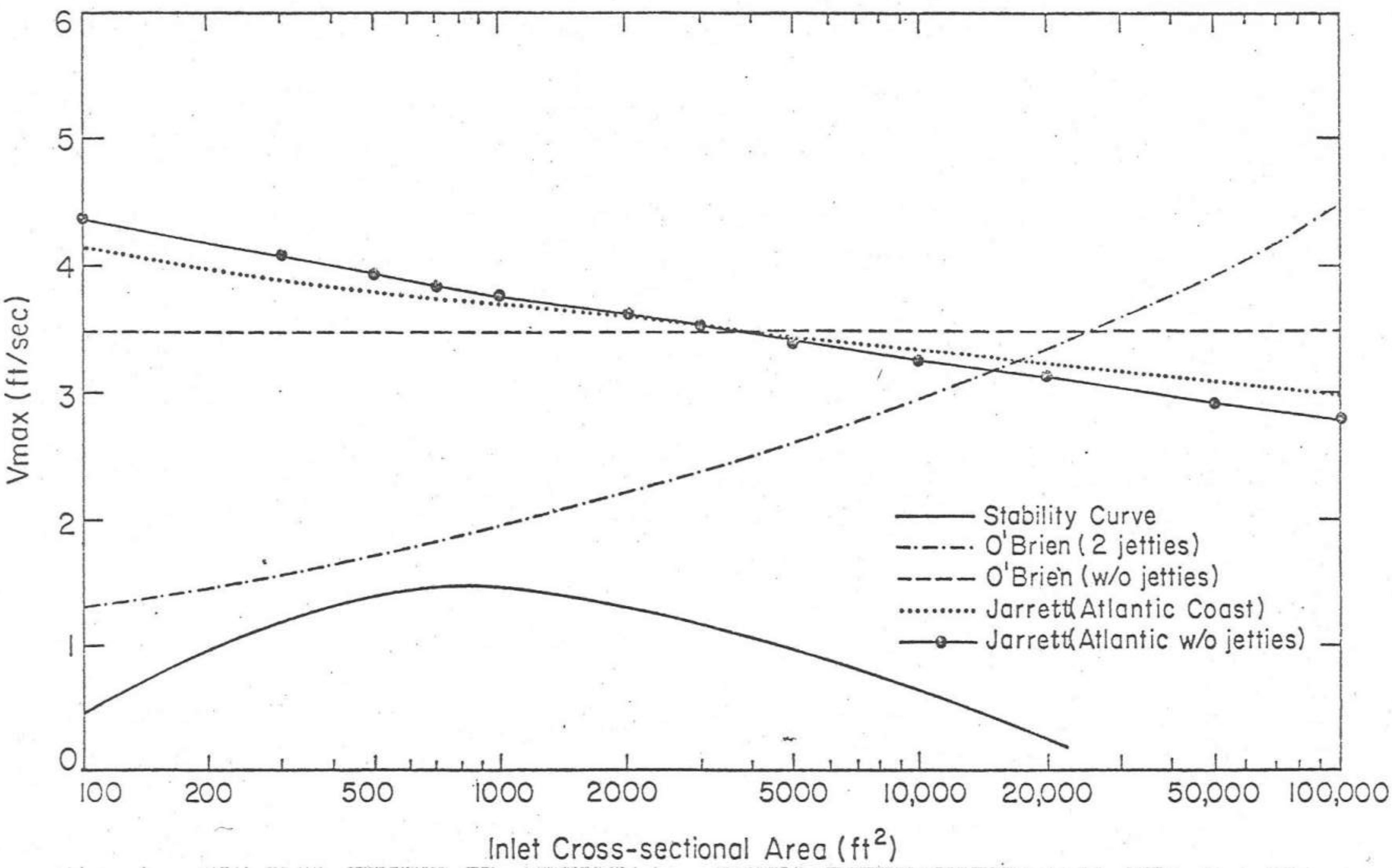

FIGURE 66 Stability Curve and Prism-Area Curves for Roosevelt Inlet

required. Should the jetties be repaired and redesigned (preferably reducing the width of the inlet) the maintenance schedule would be considerably lightened.

CONCLUSIONS

In its present condition Roosevelt Inlet acts as an effective trap for sediments entering either from the east or west. This material passes easily through and over the badly deteriorated sheet pile jetties and principally deposits in two lobe-like shoals along both banks with the western shoal normally being larger. Once inside the confines of the inlet the sediment tends to remain more or less localized as there is no efficient mechanism for removal. The presence of a mean flow in the flood direction at Roosevelt Inlet revealed by the numerical model considerably enhances the ability for the inlet to trap sediment, thus increasing the shoaling in the inlet throat and decreasing the sediment transport available to form an ebb tidal shoal.

The trapping of the littoral drift has aggravated the downdrift erosion problem at Lewes Beach. The placement of beach fill from dredging activities within the inlet as well as from offshore sources along with construction of groins has been employed to help alleviate the erosion. However, much of the beach fill finds its way back into the inlet through and over the east jetty during periods of east to northeast winds, thus short-circuiting the sand by-passing practice. Also the groins were designed too low for present beach fill operation, and are completely covered by the fill, which renders them ineffective until the beach becomes very lean. Furthermore, the present condition of the jetties, many of which are only visible at low tide, offers very little protection from the dominant northwesterly waves in the form of sheltering for Lewes Beach.

Analysis of historic cross sections of Roosevelt Inlet indicates that there was a tendency for scour to occur following each dredging operation at the inlet throat prior to 1962. Following that year, reduction in cross section occurred after each dredging operation, as both the east and west lobes have encroached on the inlet channel. Examination of shoaling rates within the inlet have revealed that this closure trend was presumably initiated by the devastating March 1962 storm, which caused considerable damage to the already weakened steel sheet pile, after which an eightfold increase in shoaling rate took place. At the outer end of the inlet (Sta. 15+00I) a scour tendency has never been shown. Its cross section has had to be maintained through dredging.

Based on equilibrium considerations, the present inlet cross section appears to be too large to be self-maintaining.

Detailed surveys and observations of the western lobe reveal that the deposition patterns are such that an equilibrium width of approximately 350 feet is being maintained at the inlet throat. As a result of these findings, it may be concluded that the jetties are too widely spaced and should be redesigned and rebuilt to provide a cross section of smaller width and greater depth.

RECOMMENDATIONS

In order to maintain a navigable channel in Roosevelt Inlet and also to alleviate the downdrift erosion problem at Lewes Beach it is recommended that the by-passing of sand be continued on a periodic basis. The present condition of the jetties allows shoaling to occur at a rate of approximately 20,000 cubic yards per year. A representative cost to by-pass this material would be about $\$1.75/yd^3$ or \$35,000 per year. Assuming that nourishment is needed every two years, 40,000 yd^3 would be available to provide replenishment of approximately 8,000 feet of beach front, realizing design dimensions of a 100-foot berm width, 10 feet above mean low water with a foreshore slope of 1 on 15 (U.S. Army Corps of Engineers, 1972).

It is not known what percentage of the total 20,000 yd^3 of material would be suitable for beach fill. Should the sand supplied to the inlet from the eroding western beaches fail to meet the demand needed to provide adequate nourishment for Lewes Beach, other sources should be investigated. Offshore areas would be the most likely and the most economical supplementary source.

Most of the sand provided as nourishment can be placed along a short section of beach adjacent to the east jetty to serve as a feeder for the reach further downdrift. In this manner natural forces can be used to spread the fill and thus a shorter pumping distance will be required. Care must be taken that the sand supplied to the feeder beach is not returned within the confines of the inlet as is presently occurring. Some sand is entering the inlet by overtopping the rubblemound protection along the east bank while still more is entering around the shoreward end of the sheet pile within the surf zone. In order to alleviate the problem it is recommended that the elevation of the stone along the east bank be increased to match the elevation of the inner end of the sheet pile which is 10 feet above mean low water. Furthermore, the structure along the bank should be made as sand-tight as possible,using a fine crushed stone or comparable material as a core. To stop the sand from passing around the shoreward end of the east jetty within the surf zone rubble should be placed along the remains of the existing jetty for approximately 500 feet offshore having a top elevation of 8 feet above mean low water as present in the original design. The section of jetty presently below mean low water is presumably still in fair condition and would serve as a useful impermeable core for the rubblemound extension. In order to estimate the cost of these improvements it will be assumed that the placement of stone costs approximately \$45/ton. At this rate the improvements along the east bank with a top elevation of 10 feet above mean low water, a crown width of 5 feet and a one-on-four side slope would cost about \$121,000 (assuming 35% voids). For the reinforcement of the jetty to 500 feet offshore the cost would be approximately \$306,000. This estimate is based on jetty cross section having a top elevation and width of eight feet and one-on-two side slopes.

Should it ever become economically justified, the inlet should be redesigned. One conceptual design would be to reduce the width of the inlet from 500 feet to 350 feet in the outer section and construct a low sill weir section in the shoreward end of the west jetty. This design is sketched in Figure 67. The weir section with an elevation at the mean water line would allow passage of the dominant westerly littoral drift into a depositional area inside the inlet. It is assumed that a similar lobe-like feature would continue to form along the western bank thus trapping and confining a large portion of the sand entering the inlet which would be periodically by-passed onto Lewes Beach as fill. This depositional area would be adequately protected from wave action with the sill at the mean water mark and, therefore, would provide the necessary sheltering for dredging operations. Due to the presence of the mean pumping of water through the inlet in the

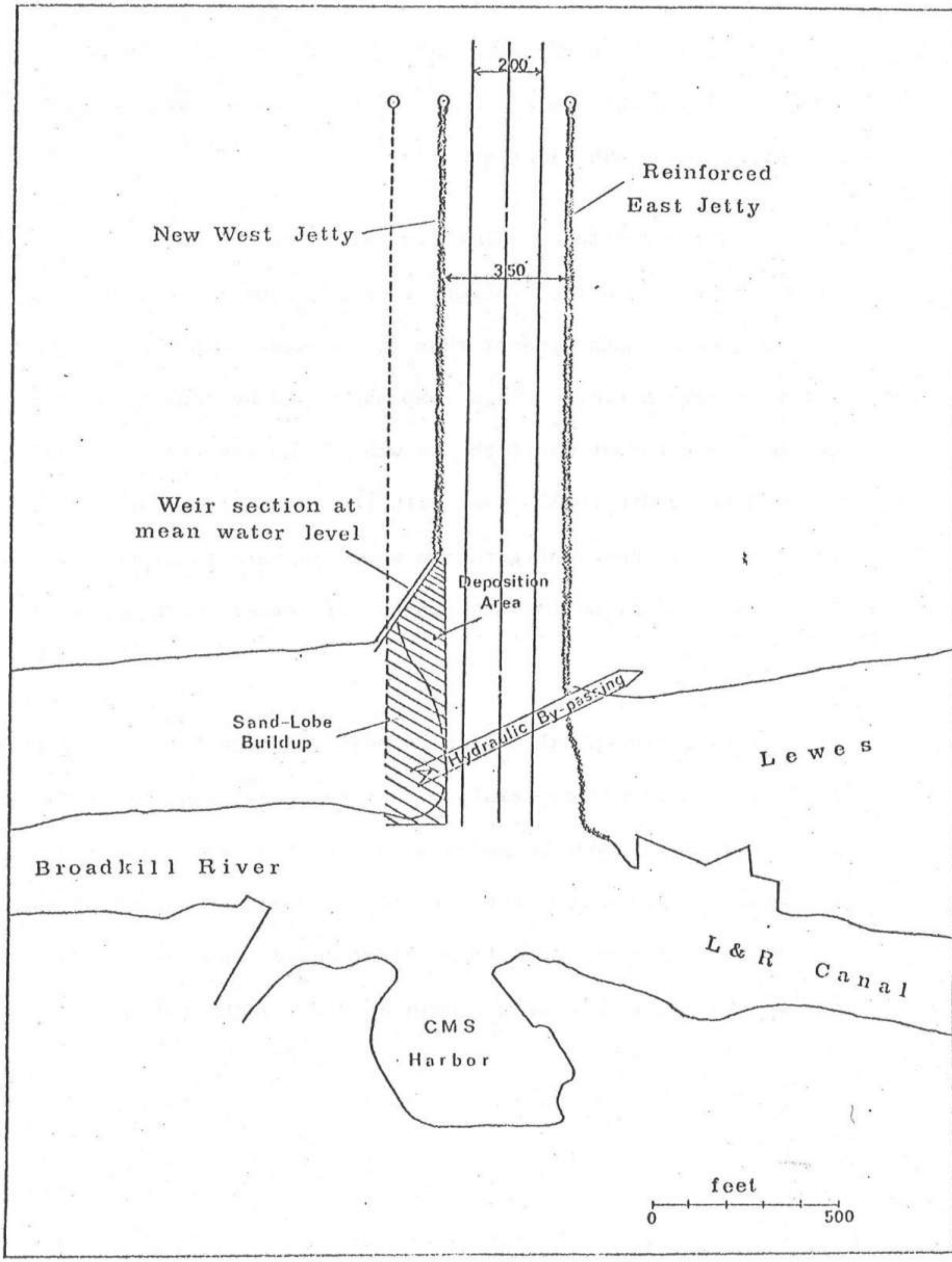

FIGURE 67 Conceptual Weir Jetty Design for Roosevelt Inlet

flood direction, the majority of the sand spilling into the inlet channel from the depositional area would be continually worked landward along the western bank and would most likely not be deposited on an ebb tidal shoal.

The reduction in width in the outer section of the inlet would presumably cause a readjustment to a new equilibrium cross section having a mean depth greater than the present mean depth of only 8 feet (above M.L.W.). This response would be analogous to that of the inlet throat where the growth of the western shoal has reduced the width to 350 feet with the mean depth adjusting to 12 feet. This new configuration would be more maintenance-free and would allow for the passage of vessels with a deeper draft.

This conceptual design is only one idea for the improvement of Roosevelt Inlet and certainly more intense and thorough exploration for a new design must be undertaken before a final plan is realized. A gross cost estimate for the reconstruction of the jetties, assuming the same trapezoidal design as mentioned previously out to the six-foot contour, would be 2.6 million dollars.

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APPENDIX I

Theoretical Shape of the Discharge Curve

From linear water wave theory and long (shallow) water approximations the theoretical shape of the discharge curve can be developed for a progressive tide wave.

The horizontal water particle velocity may be expressed as

$$U = \frac{agk}{\sigma} \cosh k(h+y) \cos(kx-\sigma t) \tag{1}$$

and the dispersion relationship as

$$\sigma^2 = gk \tanh (kh) \tag{2}$$

where a = amplitude of the free surface wave

g = gravitational constant

k = wave number, $2\pi/L$

σ = angular frequency, $2\pi/T$

h = mean water depth

y = vertical coordinate, measured positive upwards

x = horizontal coordinate, in direction of wave propagation

L = wave length

T = wave period

$C = L/T = \sigma/k$ = wave celerity

t = time

η = free surface displacement = $a \cos(kx-\sigma t)$

Applying shallow water approximations ($kh<<1$), Equations (1) and (2) become:

$$U = \frac{agk}{\sigma} \cos(kx-\sigma t) \tag{3}$$

$$\sigma^2 = gk^2 h \tag{4}$$

Combining (3) and (4) and substituting $\eta = a \cos(kx-\sigma t)$;

$$U = \frac{\eta C}{h} \tag{5}$$

where C = shallow water wave celerity = $\sqrt{gh}$ for finite amplitude waves.

The discharge per unit width q equals UA where A is the cross-sectional area and equals $(h+\eta)(1)$ or

$$q = UA = U(h+\eta) \tag{6}$$

Combining Equations (5) and (6):

$$q = \frac{\eta C}{h}(h+\eta) \tag{7}$$

Rewriting Equation (7), we obtain

$$q = \sqrt{gh}\ (1+\eta/h) \tag{8}$$

By inspection of Equation (8), it is seen that as h becomes large with respect to η, the discharge curve takes the shape of a sinusoid (as η is taken as sinusoidal) with the second bracket term in the equation tending to one. For conditions when h and η are of the same order of magnitude (i.e., in the L&R Canal), the bracketed term modifies the symmetric shape of q. When η is positive (flood tide) the second term is greater than one thus enhancing the peak. On the other hand when η is negative (ebb tide) the second term becomes less than unity thus decreasing the peak; the result being the asymmetric discharge charge as predicted by the model in Chapter 7.

For the purpose of illustration assume the following:

$\eta = a \cos(kx-\sigma t) = a \cos \Theta$

$a = 2$ ft

$h = 10$ ft

$g = 32.2$ ft/sec^2

The following table results from the equation:

$$q = 35.9 \cos \Theta\ (1 + 0.2 \cos \Theta) \tag{9}$$

TABLE I-1 Discharge Versus Various Phase Angles

Θ	0	$\pi/4$	$\pi/2$	$3\pi/4$	π
$q(ft^2/s)$	43.1	29.0	0	-21.8	-28.7
$\eta(ft)$	2	1.4	0	- 1.4	- 2

The tabulated results indicate larger values of unit discharge for positive (flood) tide than for negative (ebb) tide. To examine the mean flow per unit width, q, induced by the progressive tide wave, Equation (8) or (9) can be averaged over a tidal period.

$$\bar{q} = \frac{1}{2\pi} \int_0^{2\pi} q d\Theta = \frac{a^2}{2}\sqrt{g/h}$$

For the previous example, $\bar{q} = 3.59\ ft^2/sec$. This mean flow per unit width is about twice that predicted by the model, which includes the effects of friction, inlet geometry and the tide at the other end of the canal.