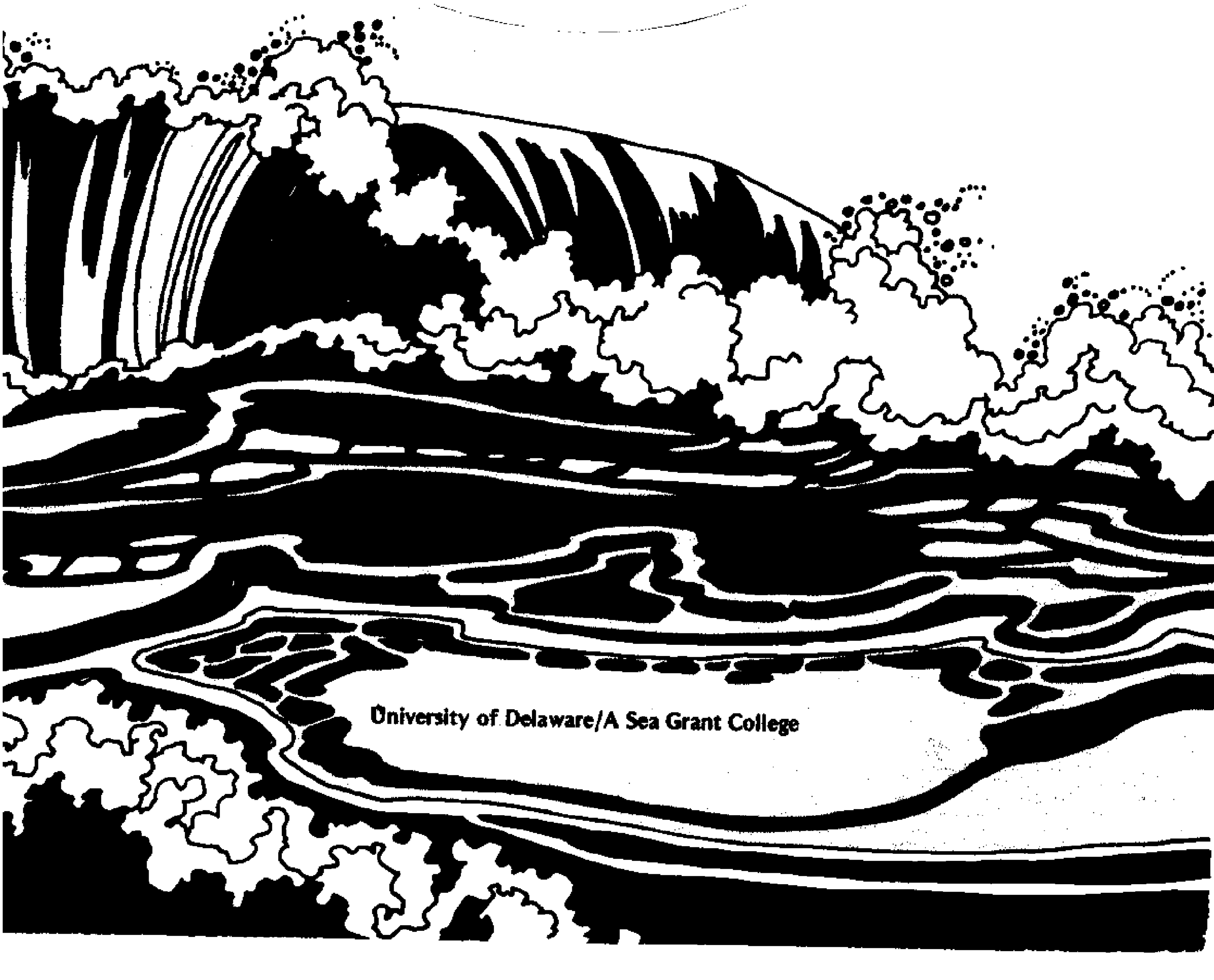
A Delaware Sea Grant Technical Report



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A COASTAL ENGINEERING STUDY OF INDIAN RIVER INLET, DELAWARE

by

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and

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DEL-SG-5-77
Ocean Engineering Technical Report 14

Published by
The Delaware Sea Grant College Program

Department of Civil Engineering
and
College of Marine Studies
University of Delaware
Newark, Delaware
19711

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ACKNOWLEDGMENTS

The authors wish to thank the Department of Civil Engineering of the University of Delaware, and the National Oceanic and Atmospheric Administration, Office of Sea Grant, Department of Commerce, for support of this research.

This study could not have been conducted without the assistance of many individuals. The field assistance given by numerous Civil Engineering students and others is greatly appreciated. Special thanks are extended to Chris Wethe and Bill Thompson for their help in conducting field work. The authors also wish to thank all the students who worked on this study as part-time employees. The members of the Civil Engineering Department staff who deserve special thanks are R. Essex, J. Coverdale and J. Mark for their technical assistance and S. Sulpizi and B. Shelton for their secretarial support.

The assistance of members of the Geology Department at the University of Delaware, Robert D. Henry of the Delaware Department of Natural Resources and Environmental Control, the Philadelphia District of the U. S. Army Corps of Engineers, and Mrs. Barbara Porter, owner of the South Shore Marina is gratefully appreciated.

Finally, considerable credit must be given to Barbara Lanan, for her assistance in conducting field work and editing this report.

ABSTRACT

A field study was conducted in order to determine the causes of and possible solutions to various coastal engineering problems at Indian River Inlet, Delaware. The stabilization of the inlet in 1938 has interfered with the natural littoral processes. The major problems present at Indian River Inlet are the erosion of the beach north of the inlet, the erosion of the inlet channel banks, and the loss of sand onto the ebb tidal shoal.

Information on sediment movement trends was obtained by computing volumes of erosion and accretion from historical charts and conducting a series of three hydrographic surveys. The sediment movement patterns were documented with a sand tracer study. The tidal hydraulics of Indian River Inlet were also studied through tidal and current measurements.

The conclusions of this study also include an approximate sand budget. The net annual littoral drift at Indian River Inlet was determined to be approximately 105,000 cubic yards northward. The study of various sand bypassing techniques indicated that sand bypassing would be a feasible solution of several of the problems presently being experienced at Indian River Inlet. The bulkheading of the unprotected portions of the channel banks is also recommended.

CHAPTER I

BACKGROUND

1.1 Introduction

Indian River Inlet, Delaware connects Indian River Bay and Rehoboth Bay with the Atlantic Ocean. The inlet is heavily used for recreational boating and by fishermen during the warmer portion of the year. There is little use of the inlet by waterborn commerce; however, there are several small commercial fishing boats which operate out of the inlet. Indian River Inlet is also important as a hydraulic connection to the ocean for the two bays. The inlet maintains the bay water quality through the exchange of water with the ocean and provides an exit for fresh water inflow into the bays.

In its natural state, Indian River Inlet was small and shallow, often shoaling, shifting and closing. In 1938, the inlet was stabilized and a generally satisfactory channel has been maintained since that time. The inlet stabilization has interfered with the natural littoral processes, however, and has caused several problems; most notably, the erosion of the beach north of the inlet and the increase of channel dimensions.

1.2 Topographic Conditions

Indian River Inlet is located on Delaware's Atlantic coast 13 miles south of the entrance to Delaware Bay. It is the only inlet through the barrier beach system separating Indian River Bay and Rehoboth Bay from the Atlantic Ocean. As shown in Figure 1, Indian River Inlet connects Indian River Bay directly with the ocean, and Rehoboth Bay is connected to Indian River Bay through the "Ditches." Figure 2 is a locality sketch of the immediate vicinity of Indian River Inlet.

Indian River Bay is about two miles wide north to south, about six miles long east to west, and has a surface area of 14.8 square miles. The bay is shallow with an average depth of about four feet at local mean low water. In the center of the bay, the water reaches a maximum depth of seven feet. The bay is bordered by low bluffs and tidal marshes.

Indian River is the main tributary to Indian River Bay. The river is tidal from the bay to the head of navigation at Millsboro, Delaware (13.5 miles west of Indian River Inlet). The navigable portion of the river is shallow and meandering and has a controlling depth of three feet (NOAA, 1975a).

Rehoboth Bay is about four miles long from north to south and three miles wide from east to west, having a surface area of 14.5 square miles. This bay is also quite shallow with a maximum

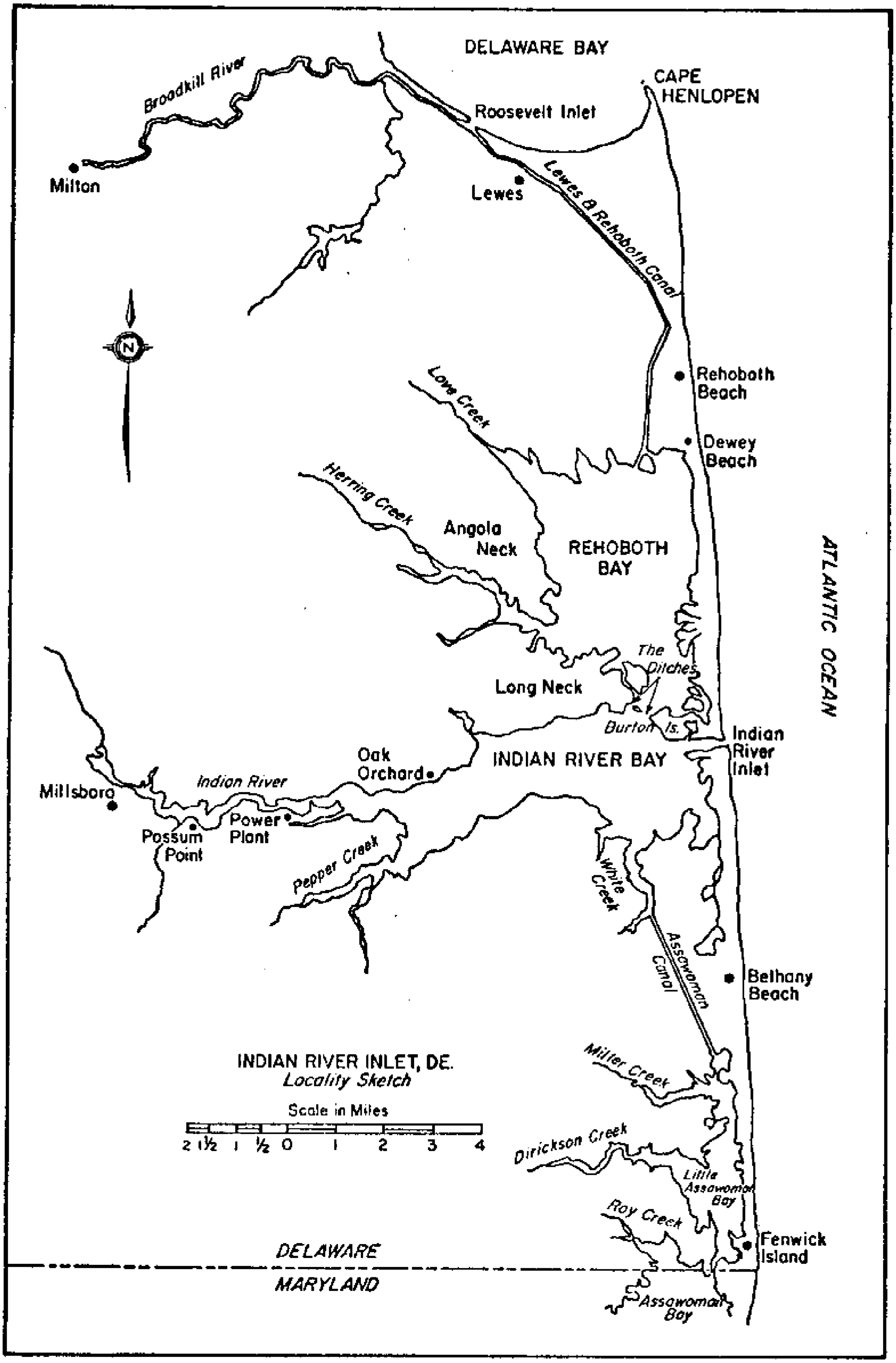


Figure 1   Locality Sketch of Indian River Inlet, Delaware

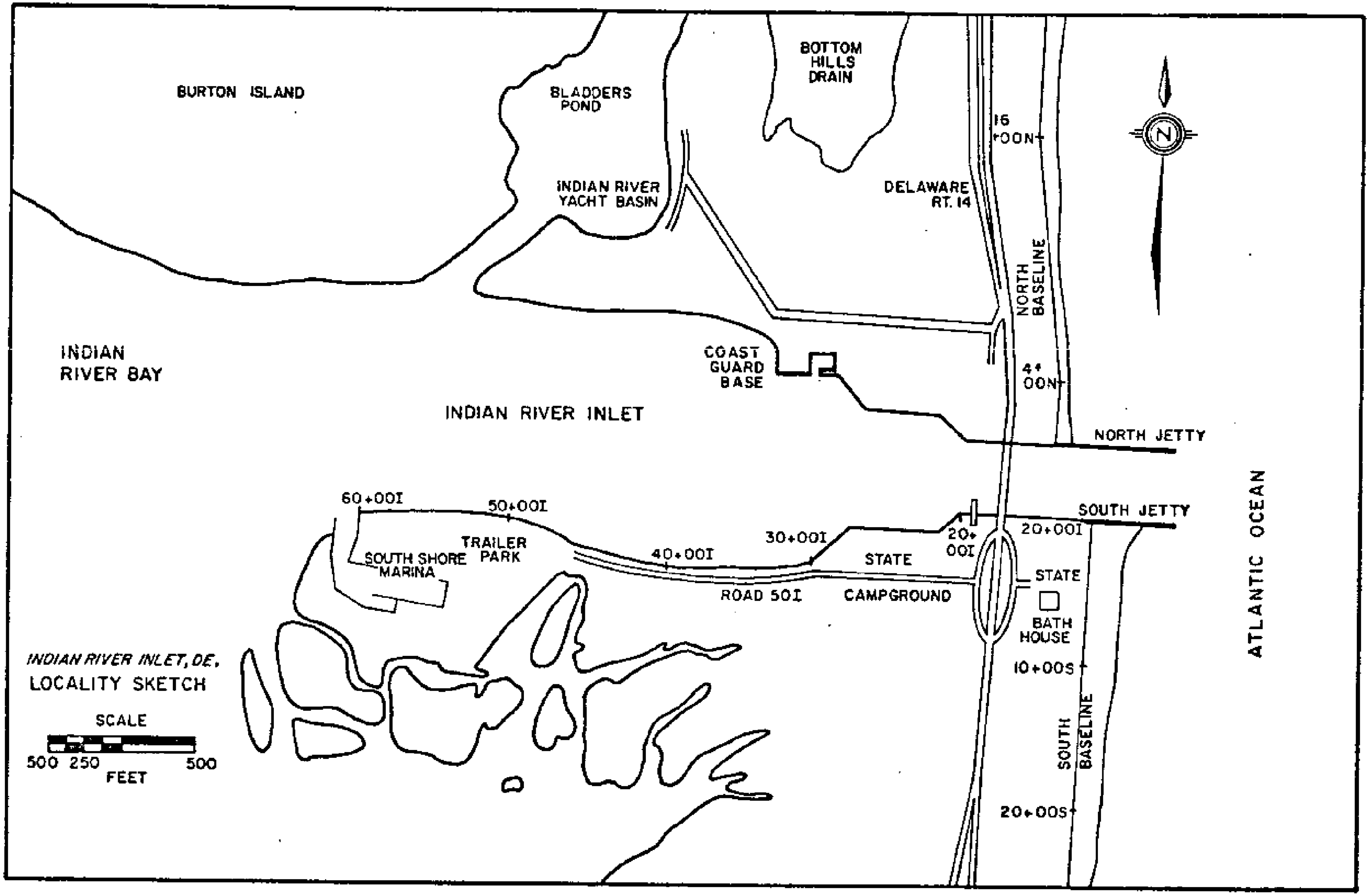


Figure 2 Indian River Inlet, Delaware. Locality Sketch of Inlet Vicinity

depth of seven feet. Rehoboth Bay is bordered mostly by tidal marsh. Its main tributaries are Herring and Love Creeks. The Ditches, which connect the Bay to Indian River Bay, have a controlling depth of three feet (NOAA, 1975a).

The Lewes and Rehoboth Canal is a tidal canal connecting the northern end of Rehoboth Bay with the Delaware Bay at Lewes, Delaware. For most of its 9.2-mile northern course, the canal passes through marsh. In July of 1974, the controlling depths were six feet for a midwidth of 100 feet in the Roosevelt Inlet channel, then five feet to the turning basin at Lewes and then two feet (reported in 1968) to Rehoboth Bay (NOAA, 1975a). The canal was cut in 1913.

Assawoman Canal connects White Creek on the south side of Indian River Bay with Little Assawoman Bay to the south. The three-mile long canal is reported to have a controlling depth of 2.5 feet. This canal was dug in 1891.

The total drainage area of both bays is 254.5 square miles (Keulegan, 1967). The Indian River Inlet Commission (in their 1931 report) estimated the fresh water inflow into the bays at 16×10^6 ft^3 per tidal cycle by measuring the inflow and outflow through Indian River Inlet and the Lewes and Rehoboth Canal. This corresponds to a unit runoff of 1.59 ft^3 per second per square mile of land area. The only portion of the drainage basin which is gaged has an area of 5.24 square miles. The 31-year average discharge is 6.98 ft^3 per second which corresponds to a unit runoff of 1.33 ft^3 per second per square mile (U.S. Geological Survey, 1974). This measured unit runoff is fairly close to the unit runoff predicted from the Indian River Inlet Commission's fresh water inflow estimate. Thus, the value of 15 x 10^6 ft^3 per tidal cycle used by Keulegan (1967) is a reasonable value for a fresh water inflow. The long term mean annual precipitation is 47 inches for the gaged basin (Johnston, 1976).

1.3 Meteorology and Wave Climate

Wind and waves are two of the major forces which influence Indian River Inlet. The effects may be either gradual, such as the monthly wind and wave conditions which influence long-term sediment movement patterns, or rapid, such as was caused by the March 1962 storm.

The prevailing winds on Delaware's Atlantic coast are generally from the southwest but vary seasonally. Winds above 30 miles per hour are most frequently from the northwest, and winds of the highest velocities are most frequently from the northeast. Figure 3, from Polis and Kupferman (1973), shows the monthly percentage frequency of wind direction by speed for Indian River Inlet, Delaware. Additional wind roses for points in the Delaware Bay area are presented in the above reference and in Figure 4 from the U.S. Army Corps of Engineers (1968).

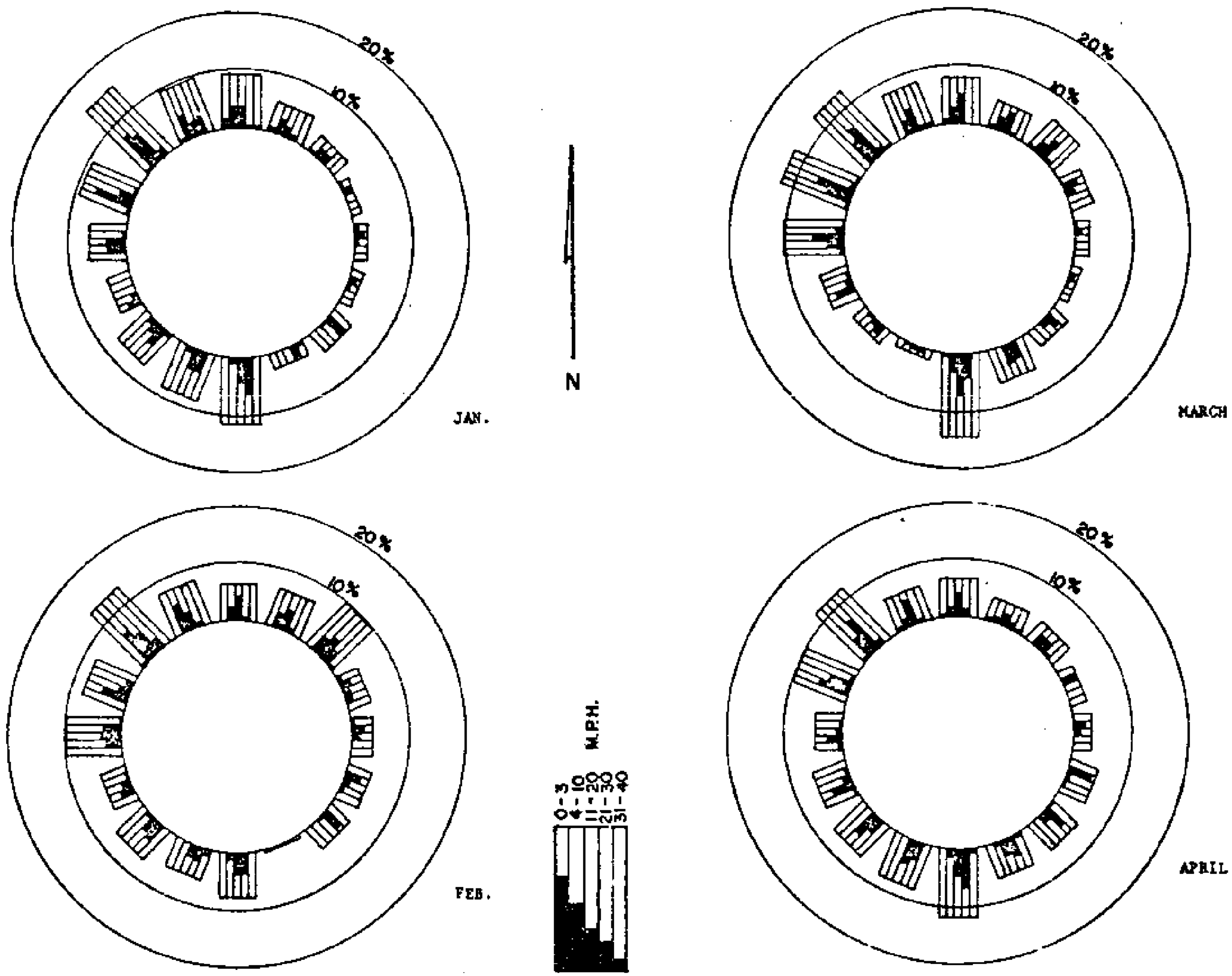


Figure 3a Percentage Frequency of Wind Direction by Speed for Indian River Inlet (Polis and Kupferman, 1973)

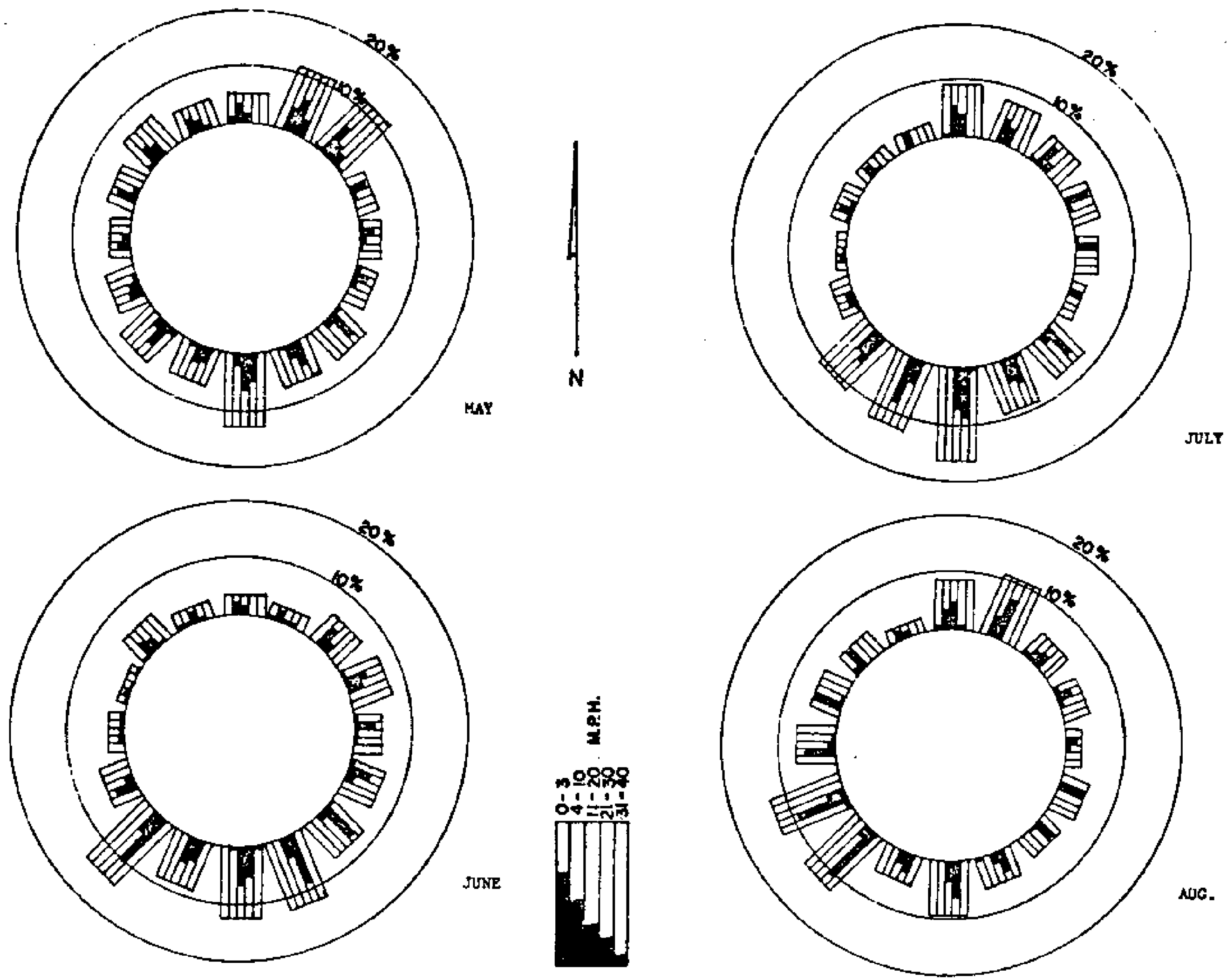


Figure 3b Percentage Frequency of Wind Direction by Speed for Indian River Inlet (Polis and Kupferman, 1973)

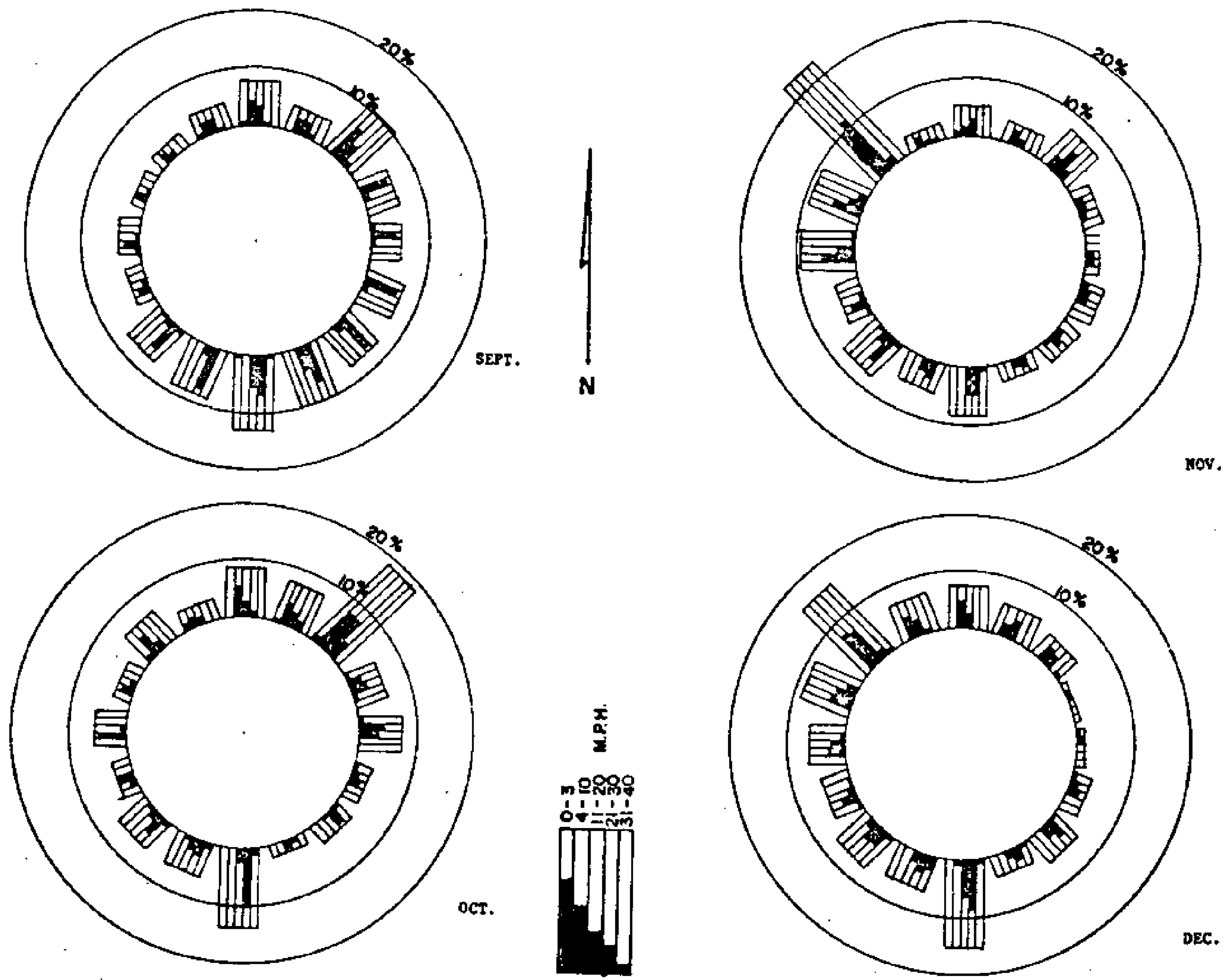


Figure 3c Percentage Frequency of Wind Direction by Speed for Indian River Inlet (Polis and Kupferman, 1973)

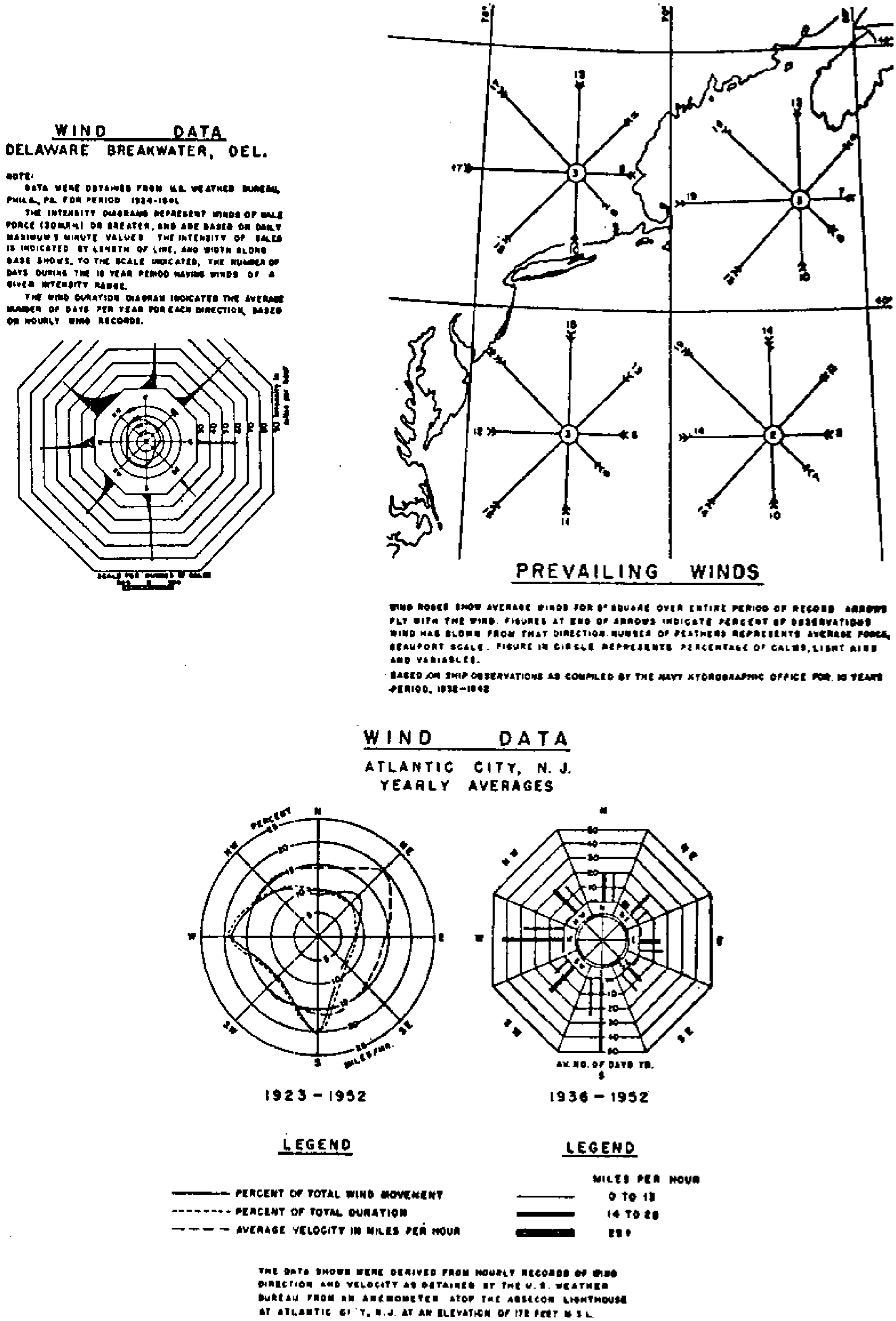


Figure 4 Wind Data in the Delaware Bay Area (U.S. Army Corps of Engineers, 1968)

There are two major types of damaging storms which affect the Delaware coast. They are tropical hurricanes and "northeasters" (characterized by their strong northeast winds). Hurricanes usually diminish in intensity by the time they reach the Delaware coast during their northward movement. In addition, Delaware has had remarkably little damage from hurricanes during this century. No hurricane storm center has passed over the Delaware coast since records have been kept. Should a major hurricane pass over the Delaware coast, it could be expected to cause a large storm surge and considerable wind and wave damage (U.S. Army Corps of Engineers, 1968). Northeasterly storms create storm surge and high waves which cause considerable beach erosion along the Delaware coast.

By far, the worst storm to effect Indian River Inlet this century was the northeaster of March 6 - 8, 1962. Two low-pressure areas joined in the ocean off the mid-Atlantic coast and remained stationary for several days. The sustained high winds over a long fetch produced large waves and a storm surge which lasted over five consecutive high tides. The storm occurred during a period of unusually high astronomical tides. The combined storm tide set a record high of 7.9 feet above mean sea level at Breakwater Harbor, Delaware.

The 20-to 30-foot high waves and high tides associated with the March 1962 storm severely eroded the beach and dunes, thus doing considerable damage to beach front properties. A total of

16.7 million dollars worth of damage was done along the Delaware coast from Cape Henlopen to Fenwick Island (U.S. Army Corps of Engineers, 1968). The storm breached the barrier beach in several places between Dewey Beach and Indian River Inlet. These new inlets were subsequently closed with sand dredged from Rehoboth Bay. Figure 5 and Figure 15 show the vicinity of Indian River Inlet

Figure 5 Indian River Inlet Viewed from the North After the March, 1962 Storm. Note the Destruction of the Dunes and the Large Amounts of Washover (Delaware State Highway Department, 1975).

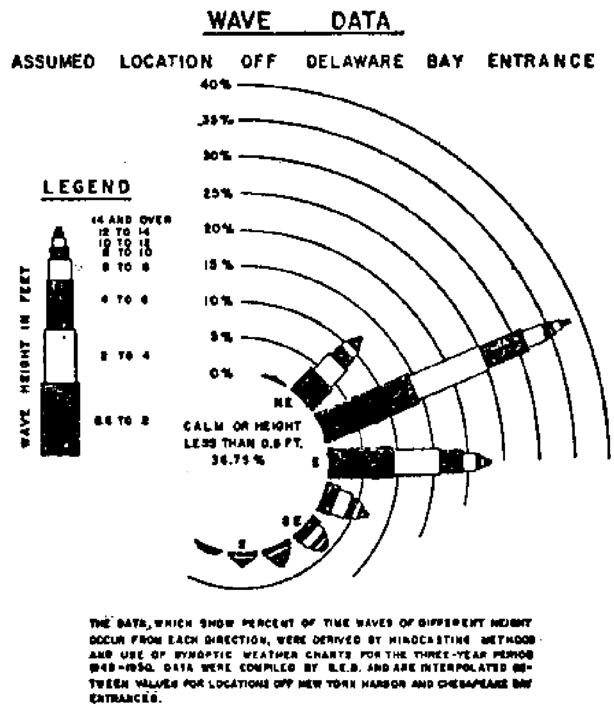
immediately after the storm. The natural sand dunes were almost completely destroyed; large quantities of sand were carried landward and deposited in washover fans, and channels were cut through the barrier.

Wave data for the Delaware coast is available from several sources. Polis and Kupferman (1973) present monthly wind wave and swell height and period distributions for Five Fathom Lightship Station, Delaware (four miles east of Cape Henlopen). Figure 6 shows sea and swell data for an assumed location off the entrance of Delaware Bay (U.S. Army Corps of Engineers, 1968). Sea and swell data is also presented for 5° squares by the U.S. Navy Oceanographic Atlas (1963). The prevailing sea and swell are from the east, the east northeast, and the northeast.

1.4 Geology of the Delaware Coast

In order for a coastal engineer to better understand the long-term processes with which he is dealing, it is important for him to look at the study area from a geologic point of view. The Delaware coast has been rather extensively studied by researchers from the University of Delaware and the Delaware Geological Survey.

Indian River Inlet is located on a transgressive barrier which extends across the seaward side of Indian River and Rehoboth Bays. These bays are drowned river valleys incised in late Pleistocene age coastal or nearshore marine sediment. These narrow



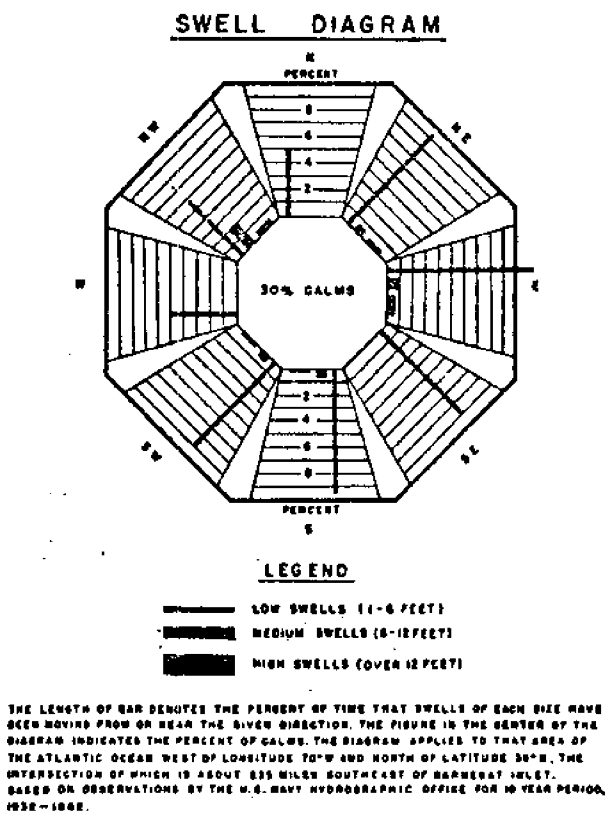


Figure 6 Wave and Swell Data for a Location Off Delaware Bay Entrance (U.S. Army Corps of Engineers, 1968)

and steep valleys were cut during the most recent glacial period (Wisconsin), when large volumes of water were contained in the polar ice caps. The local relief on the pre-Holocene (Pleistocene) surface is up to 125 feet in the Indian River area.

The present rise in sea level began during the current Holocene Epoch of geologic time. During this time, approximately 11,000 to 14,000 years ago, sea level was more than 340 feet below its present level, and the resulting shoreline was located on the outer edge of the Atlantic Continental Shelf, over 60 miles eastward of its present position (U.S. Army Corps of Engineers, 1968). All evidence indicates that this marine transgression is continuing (Kraft, 1971a).

The southern portion of Delaware is also believed to be subsiding as part of the Salisbury embayment of the Baltimore Canyon trough basin of the Atlantic continental margin geosynclines. This tectonic subsidence is on the order of one foot in 10,000 years. It has very little effect on the relative sea level for the middle-late Holocene transgression. The current sea level rise is primarily eustatic (U.S. Army Corps of Engineers, 1968).

It is instructive to look at a cross section of a retreating lagoon-barrier system, such as that found on the Delaware coast. On its landward side, the lagoon is fringed by a *Spartina* salt marsh which is adjacent to the Pleistocene highlands. The surface of the marsh keeps pace with the rising sea level by the deposition of a marsh mud which is derived from organic material and trapped suspended sediment (Godfrey and Godfrey, 1975). This marsh overrides the Pleistocene sediments as the sea level rises.

Seaward of the salt marsh there is commonly a tidal lagoon. The characteristic lagoonal sediment is silt or mud. Lagoonal mud can be differentiated from marsh mud by the lower organic content and the presence of fauna, such as *Crassostrea virginica* (oyster). Directly behind the barrier, a back barrier marsh is frequently formed. This marsh may form on top of sand carried into the lagoon and deposited in the flood tidal shoal of an old inlet. The shoal and channels of such an inlet can be clearly seen in Rehoboth Bay, about 2.5 miles north of the present inlet.

The barrier itself is situated on top of all the previously mentioned sediments. As sea level rises, the barrier retreats over the back barrier marsh and into the lagoon (Dillon, 1970). The sand is transported from the beach towards the lagoon by three major processes. The first and most obvious process is by washover. During storms, the combination of storm surge and erosive waves breach the dunes at low pointsand wash sand across the barrier. The sand is deposited on the barrier and in the lagoon in washover fans (Pierce, 1970). Secondly, sand is transported across the barrier and deposited in the flood tidal shoals of inlets. As deposition continues, the inlet becomes inefficient and cannot compete with the new inlets that are created when storms breach the barrier. The inlet may also close completely until fresh water buildup in the bay again breaches the barrier and reestablishes an inlet. Finally, sand can also be transported landward as sand dunes which move across the barrier (Kraft, 1971a).

After erosive "northeasters," the back barrier marsh can be seen outcropping in the surf zone of the beach. South of Dewey Beach, Delaware, where the back barrier had been forested, pine tree stumps are often exposed after a storm. The wood has been dated to be between 250 and 350 years old. A cross section of the bay and barrier at Indian River Inlet is shown in Figure 7.

From this geologic information, the coastal engineer can determine the long-term processes with which he is dealing. The barrier-lagoon system at Indian River Inlet and on Delaware's Atlantic coast is moving landward. Averaged over the last 150 years, this marine transgression is approximately three feet per year and has a maximum on the Delaware coast of about 10 feet per year near Cape Henlopen (U.S. Army Corps of Engineering, 1968 and Kraft, 1971a).

1.5 History Before the 1938 Stabilization

The historical behavior of Indian River Inlet in its natural state and early attempts at improving the inlet are important in obtaining an understanding of the concerns which brought about the inlet's stabilization and the subsequent changes induced by that stabilization.

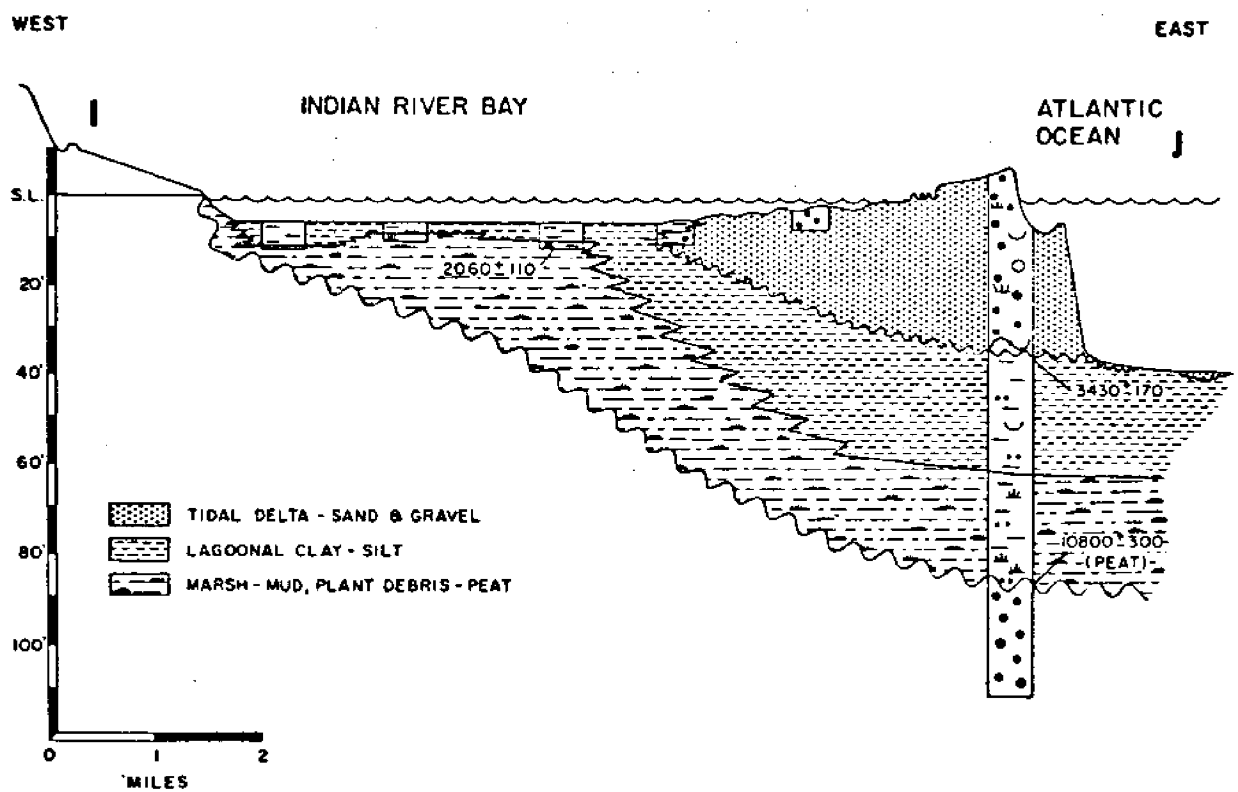


Figure 7 West to East Cross Section of Baymouth Barrier, Indian River Bay (Kraft, 1971a)

The first European settlement in southern Delaware was the Dutch settlement near Lewes, Delaware which was established in 1631. All the inhabitants of this ill-fated group were killed by Indians during that same year (Scharf, 1888). At that time, the local Indians were probably members of the Nanticoke tribe, who had settled the area after the Unalachitigo group of the Lenni Lenape Indians left the Indian River area for New Jersey. Later, about 1705, the Assateagues moved into the Indian River area from the south (Godfrey, 1953).

The English took the land over from the Indians during the late 1600's. A final cooperative effort of the Indians to drive out the white settlers ended in failure and so the Indians finally signed a peace treaty on July 24, 1742. The local Indians were then pushed westward and assimilated into the Iroquois and the Shawnee tribes. The few who remained adapted to the ways of the English and intermarried. Today, most of their decendants live on Long Neck which separates Indian River and Rehoboth Bays (Godfrey, 1953).

The name, Indian River, can be found on deeds dated as early as 1683. During the colonial days, the river was also known as South River and Baltimore River, but only for a short time (Godfrey, 1953).

The land around Indian River was initially cultivated as large plantations which raised primarily tobacco. Later, as the soil became depleted, the major crops were flour, wheat and corn. During this period, water travel was essential to commerce since roads were very poor. Many farmers had their own schooners which they used to transport crops to market (Philadelphia and New York City). After proceeding across Indian River Bay and through the natural inlet, these boats would occasionally transfer their cargo to larger ships off the coast.

These two-masted schooners were usually constructed along the banks of Indian River. The boats had from a three-to five-man crew, ranged from 30 to 90 feet long, drew from four to six feet, and displaced up to 30 tons (Godfrey, 1953).

Late in the 1800's, the Indian River area supported a large lumber industry. In 1882, there were about 50 saw mills on Indian River and Indian River Bay which produced lumber, cedar boards, and shingles. There was also an iron foundry and over 50 grist mills. Most of these products had to be transported to market via Indian River Inlet. The majority of this water-borne traffic died out about 1915 with the coming of the railroad and the building of improved roads. The importance of Indian River Inlet to commerce was also reduced by the excavation of the Assawoman Canal in 1891 and the Lewes and Rehoboth Canal in 1913 (Thompson and Dalrymple, 1976).

In its natural state, Indian River Inlet was less than ideal. Before its stabilization, the inlet had a history of constant shoaling, moving, and closing. In 1807, Joseph Scott described the inlet and the river:

> "The principal waters (of Sussex County, Delaware) are Indian river. It rises in the interior of the county, runs south, and receives the redundant waters of Rehoboth bay, by a very shallow communication, near its inlet from the Atlantic ocean. The passage of this river into the sea is very shallow and contracted, often changes, and is extremely difficult and dangerous."

In the "History of the State of Delaware" (1870), Francis Vincent states:

> "This inlet rarely ever contains more than three feet of water, and after a great easterly storm its mouth is generally stopped up by sand washed into it from the working of the ocean, after which the waters of the bays again tear themselves a passage and wash the sand which has filled up the inlet into the ocean... The small depth of water at the Indian River Inlet creates the necessity of forcing the shallops over the bar by kedging. This causes a thumping of the vessel's keel on the sand, which drives the bilge water into the hold, and spoils much of the grain which is the general cargo of these vessels."

Vessels would also lighter over the shoals; a process which could take up to three weeks during a range of low tides. At other times, boats drawing five to six feet could pass through the inlet at any stage of tide (Thompson, 1974).

The migration of the Indian River Inlet has been recorded since 1800. In 1843, the inlet (as shown in Figure 8) was located just north of its present location. From 1843 until about 1910, the inlet migrated steadily northward until its progress was checked by a deposit of mud. Its position at that time was about one mile north of the present inlet. An increase in shoaling of the inlet was also noted about 1850.

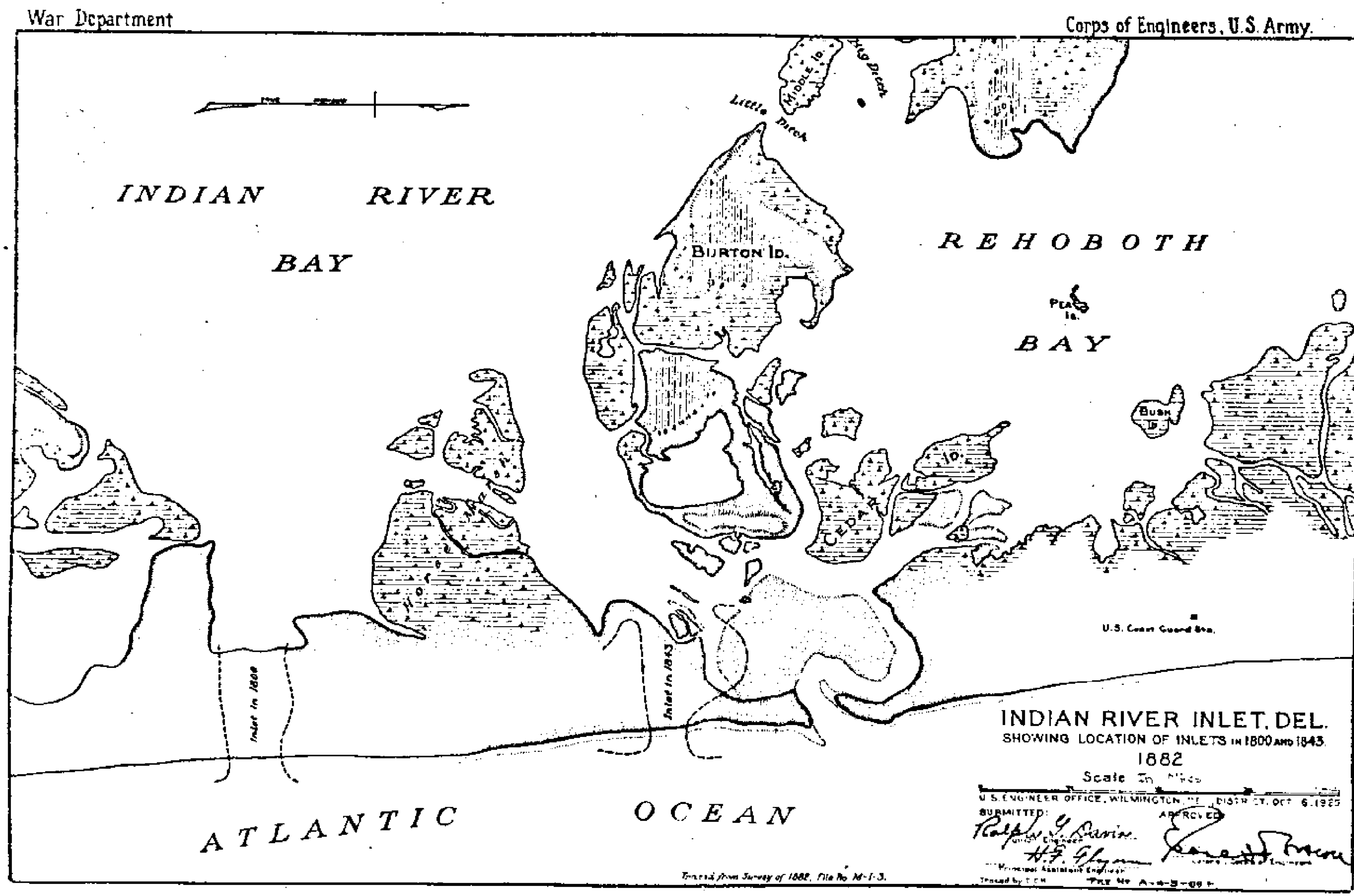


Figure 8 1882 Chart of Indian River Inlet Showing Location of Inlet in 1800 and 1843 (U.S. Army Corps of Engineers, 1975).

According to local residents, Burton Island, in the early 1800's,was connected to Long Neck by a narrow neck of marsh. At that time, the two bays were connected by a channel passing between Burton Island and the barrier beach. The owner of the peninsula is said to have cut two ditches through the neck to keep his cattle from straying (Thompson and Dalrymple, 1976 and Indian River Inlet Commission, 1931). While many view this as merely a folktale, the width of the ditches has been increasing steadily, almost doubling since 1882. This lends support to the story of the ditches being dug by hand.

The reason for the northerly migration of the inlet after 1843 was believed to be the opening of the Ditches. These ditches allowed water to pass from Indian River Bay to Rehoboth Bay and thence around the eastern end of Burton Island, rather than proceeding directly through the inlet (Godfrey, 1953). This migration can better be explained by the effects of the northward littoral drift. Sand is deposited primarily on the south side of the inlet tending to force the inlet northward (Bird, 1969). This northward migration was also occurring in the early dredged cuts at Indian River Inlet.

By 1882, the main channel had become the channel passing northward of Burton Island and into Rehoboth Bay. There, where the water velocity decreased, the sand carried in on the flood tide was deposited on a shoal known as the "Bulkhead." The Bulkhead was the major obstruction to vessels passing the inlet.

This shoal was first dredged in 1876 by private subscription and again in 1883 by the U.S. Army Corps of Engineers. One year after completion of the 1883 project, the four-foot deep channel had shoaled to its original depth of two feet. Earlier, about 1857, an attempt was made to improve Indian River Inlet by blocking all but the main channel with dikes. Some improvement was noted before the dikes were destroyed in a storm 18 months later.

In 1891, a four-foot deep, twenty-foot wide, three-mile long, pilot channel for the Assawoman Canal was excavated. Around 1905, local residents noticed a reduction in size of the inlet and claimed this was a result of the canal. This claim was further supported by the behavior of the inlet after the opening of the Lewes and Rehoboth Canal in 1913. This canal's original dimensions were six feet deep, forty to fifty feet wide, and about twelve miles long. In the summer of 1911, the inlet closed entirely and remained closed until 1912. The inlet also closed naturally in 1912, 1915, 1923, 1925, 1931 and 1933 (Thompson and Dalrymple, 1976).

Each time the pattern was the same; when the inlet closed, land runoff accumulated in the bays. Depending on the length of time the inlet remained closed, the bay water would become brackish or even fresh. The water of the bays would also become stagnant, flooding the marshes and the surrounding low lying areas. Upon gaining sufficient head, the water would breach the barrier generally at the inlet's most recent position. The ensuing out-rush of water would scour the channel, thereby reestablishing the inlet temporarily.

Due to the opening of the Lewes and Rehoboth and Assawoman Canals and the general decline in commerce, local shipping did not suffer greatly from the frequent openings and closings of the inlet. However, other local concerns were not so fortunate. When the bay waters would rise, low lying pastures were threatened with flooding, and the surrounding farmland became marshy. This caused much concern among local farmers. These flooded marshes and farmland also attracted greater numbers of mosquitoes.

The most damaging result of the inlet closings, however, was its effect on the local seafood industry. Before about 1900, the seafood industry only produced for local consumption. With the improvement of transportation, this seafood could reach Philadelphia and New York City while still fresh. By 1908, there were 300 men catching crabs on Indian River and Indian River Bay. As late as 1924 crab shipments averaged 2,500 boxes of 24 crabs each per day at a value of $5,000 per day (Godfrey, 1953). Fish, oysters and clams were also caught in large numbers.

With the inlet closings, fish could not move into the bays and the marine life already in the bays died out due to the reduced salinity. The seafood industry was at the mercy of the inlet; improving while the inlet was open, and almost being destroyed when it was closed. The closing of the inlet in 1925 caused an estimated one million dollars annual loss to local residents and the loss of hundreds of jobs.

Local and state officials made repeated requests to the federal government for aid. However, due to the expected high cost of maintaining the inlet and a general lack of commerce, the U.S. Army Corps of Engineers repeatedly recommended that no federal improvement be made to the inlet. Federal funds could legally be appropriated toward projects to improve commerce only, and not to improve the local seafood industry.

Not satisfied with the response of the federal government, the Delaware General Assembly enacted legislation creating the first of six Indian River Inlet Commissions in 1919. The primary purpose of these commissions was:

> "That their efforts should be confined to maintaining an inlet for increasing the salinity of the Bays and the restoration of marine life rather than one suitable for navigation."

The commission dredged its first channel in the summer of 1928; the inlet at this time having been closed since 1925. The channel was 60 feet wide and 4 feet deep at mean low water--beginning at the 4-foot contour in the bay and preceding to within 115 feet of the high water mark on the beach. This cut was along the centerline of the present inlet. Approximately, 155,000 cubic yards of sand were dredged from the channel and placed along its banks. The completion was to be made by removing most of the remaining sand with scrapers and the final portion by blasting. Several attempts at the completion met with failure and work was discontinued in November, 1928, due to lack of funds. One month later, the locality regained its former appearance except for the cut through the dunes.

The inlet remained closed during the winter of 1928-29. In April, 1929, heavy runoff and the resulting high bay levels once again threatened low-lying pasture land. Concerned local farmers made a small cut by hand and by teams along the axis of the dredged channel. Much to their delight, the farmers succeeded where the commission had failed. Aided by a period of westerly winds and the high level of the bay waters, estimated to be between 3.2 and 3.5 feet above the mean low water in the ocean, the impounded bay water eroded an opening. This inlet was dredged 60 feet wide and 8 feet deep from November, 1929 to January, 1930 (Indian River Inlet Commission, 1931). The reopening of the inlet increased the salinity and lowered the elevation of the bays. Salt water fish not seen in the bays for several years were caught once again. In 1932, over one million herring were taken from Indian River (Godfrey, 1953).

However, this man-made cut had problems similar to those of the natural inlet. The ocean entrance of the channel changed and migrated (see Figure 9). The channel shoaled and a flood tidal delta formed. The inlet closed again in 1931 and 1933.

In May, 1930, the Indian River Inlet Commission enlisted the aid of Lt. Col. Robert P. Howell, U.S. District Engineer, to determine the effect of the Lewes and Rehoboth and Assawoman Canals

Figure 9 Indian River Inlet, February, 1931 Viewed from the East. The Dredged Channel is Approximately 100 Feet Wide (Indian River Inlet Commission, 1931).

on the behavior of the inlet. It was the opinion of many local residents that the canals captured a sufficient portion of the tidal prism of the bays to cause the inlet to shoal and close. A dam with a flash gate, allowing only northward flow into Indian River Bay, was constructed across Assawoman Canal. Current measurements were taken at the inlet and along the Lewes and Rehoboth Canal to determine tidal flow volumes. The results are as follows:

> "During ordinary fair weather conditions, the net flow was out of the bays into the ocean through both the Inlet and the Rehoboth Canal. The average inflow at the Inlet was 17,000,000 cubic feet per tide, and the average outflow 27,500,000 cubic feet; the difference, 10,500,000 cubic feet, representing the excess of outflow over inflow. At the Rehoboth Canal, under like normal conditions, the inflow was 10,000,000 cubic feet and the outflow 15,500,000 cubic feet, the difference being 5,500,000 cubic feet. The sum of this difference or about 16 million cubic feet, represented the excess of outflow over inflow through both of these outlets, and since there were ordinarily two tides per day, the average total excess under normal conditions during the period of observation may be taken as, roughly, 30 million cubic feet per day." (Indian River Inlet Commission, 1931)

The resulting rise and fall of the bay waters was estimated by dividing the inflow by the surface area of the bays. The inflow through the Lewes and Rehoboth Canal would raise the water level only 0.14 inches and the outflow would lower the level 0.22 inches. The inflow through the inlet produced a rise in bay level of 0.25 inches and the outflow produced a drop of 0.41 inches. The Assawoman Canal was found to have an even lesser effect. This canal's flow direction was determined primarily by wind conditions. Comparing their values of bay tide range with the ocean mean tidal range (3.8 feet at the inlet), Lt. Col. Howell concluded that the canals were not the cause of the reduction in size and the closings of Indian River Inlet.

While it is true that the canals captured only a small amount of the bays' potential tidal range, it should not have been so quickly concluded that the canals had no effect on the inlet. Throughout the inlet's recorded history, the inlet had closed frequently. It appears that the major effect of the two canals was not causing the inlet to close, but reducing the ability of the inlet to reopen.

A freshwater inflow of 30 million cubic feet per day would raise the level of the bays at a daily rate of about 0.44 inches, providing the water could not escape through the canals or the inlet. The bay waters would quickly gain the head needed to breach the barrier and erode a new inlet or reestablish the old one. Flow through the canals would draw off this freshwater buildup. Only during periods of peak runoff would the bays gain the necessary head to reestablish the inlet. The salinity of the bays would be reduced much more by water flowing through the bays and out the canals for a long period of time rather than by simple dilution (caused by impoundment of water in the bays).

After the sixth Indian River Inlet Commission presented its report to the Delaware General Assembly in February, 1931, the responsibility of maintaining the inlet was delegated to the Delaware State Highway Department (Indian River Inlet Commission, 1931). The State

Highway Department used dynamite to reopen the inlet in 1931 (Godfrey, 1953). They redredged the inlet in 1933 (Delaware State Highway Department, 1933) and again in 1937. The state's total expenditure from 1919 to 1937 was $230,000 (Thompson, 1974).

In 1936, the mosquito control office at Lewes, Delaware, made a report estimating the value of a permanent inlet to mosquito control, the seafood industry, rod fishing, wild life, and water transportation. The U.S. District Engineer was authorized to make another preliminary examination and in 1937 finally approved a project to stabilize Indian River Inlet.

CHAPTER II

INLET STABILIZATION

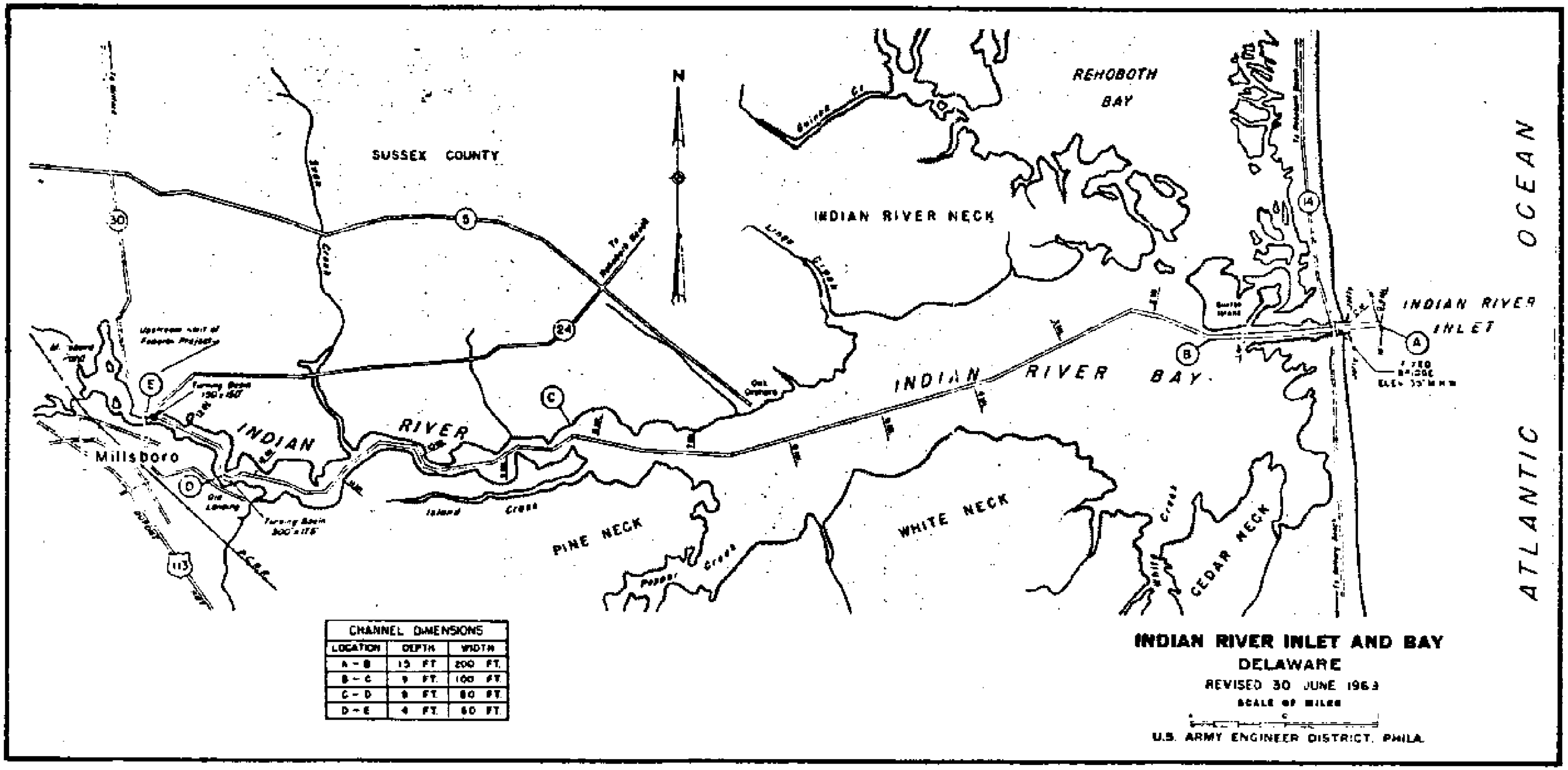
2.1 Inlet Structures

A federal project to stabilize Indian River Inlet was approved by the River and Harbor Act of August 26, 1937. The project had three goals: (1) to increase the salinity of the bays and to decrease stagnation; (2) to permit a rise and fall of the tide in the bays to increase the effectiveness of mosquito control measures; and (3) to provide a navigational waterway for commercial purposes. This section will discuss engineering structures employed to stabilize and maintain the inlet. The following section will briefly present the shoreline changes initiated by the stabilization of the inlet. Each feature will then be discussed in more detail in the chapter on sediment movement.

The inlet stabilization project provided for the construction of two jetties which paralleled the existing dredged cut. The centerlines of the jetties were spaced 500 feet apart, and they were originally 1,556 feet long. They extended from the original highway bridge seaward to the 14-foot contour in the ocean. The inner portion, 904 feet long on the north side and 890 feet long on the south side, was constructed of steel sheet piling which was driven to a depth of 18 feet below mean low water, and its base was protected by a thin layer (about 3 feet thick, 6 feet wide) of rip-rap. The outer section was of all-stone construction. The base of this section only extended to a depth of 3 feet below mean low water except on its outermost portion (U.S. Army Corps of Engineers, 1975). Construction was begun in 1938 and was completed in 1940. The state was to contribute $160,000 of the estimated $443,000 project cost (Thompson, 1974).

As part of the project, the channel was redredged west of the bridge. From the bridge to a point 7,000 ft west of the ocean shoreline, the channel was generally 200 feet wide and 15 feet deep. From this point, the depth of the channel progressively decreased to 100 feet. Dredging was completed on October 8, 1938; 1,198,409 cubic yards of material were removed by a hydraulic pipeline dredge and spoiled north of the inlet.

Channel improvements were approved March 2, 1945 and completed in 1951. The dredged channel was extended, at a depth of 9 feet and a width decreasing from 100 to 80 feet, to 1.5 miles east of Millsboro. From that point westward to Millsboro, the channel was dredged 4 feet deep and 60 feet wide. A total of 1,075,000 cubic yards were dredged during the channel extension. The current project dimensions are shown in Figure 10 (Thompson, 1974).



CHANNEL DIMENSIONS

LOCATION	DEPTH	WIDTH
A – B	15 FT.	200 FT.
B – C	9 FT.	100 FT.
C – D	8 FT.	80 FT.
D – E	4 FT.	60 FT.

Figure 10 Indian River Inlet and Bay Channel Dimensions (After U.S. Army Corps of Engineers, 1975)

Following the dredging in 1938, the inlet, where it was not confined by the jetties, began to widen rapidly. The state placed rip-rap for approximately 200 feet west of the first bridge to prevent its being flanked. In 1941, the steel sheet pile bulkheads were extended along the widened shoreline by the U.S. Army Corps of Engineers to stop bank erosion. The configuration of this dogleg extension can be seen in Figure 13. The sheet piling extended to a depth of 20 feet below mean low water in most places and 27 feet below mean low water near the bridge abutments. The stone outer section of the jetties was repaired in 1957 by the federal government. In 1963, the bulkheads were extended westward and the steel sheet pile was reinforced with rip-rap. The top portion of the sheet piling was removed, replaced with stone and then capped with concrete. This cap now serves as a walkway (U.S. Army Corps of Engineers, 1968).

The history of the bridges across Indian River Inlet is also interesting. The inlet was first bridged in 1934 with a low wooden trestle (Delaware State Highway Department, 1933). This first bridge connected Rehoboth Beach, Delaware to Ocean City, Maryland via the Ocean Highway until the completion of the Charles W. Cullen bridge in 1940. This second bridge, named after the then chairman of the State Highway Department, had a 182-foot swing span in its center which could be opened to allow large boats to pass. The remainder of the 694-foot-long bridge consisted of a concrete roadway supported on wooden pilings. Construction was begun in 1938 but delayed by the strong currents in the inlet (Delaware State Highway Department, 1938). The bridge was finally completed in the spring of 1940 (Delaware State Highway Department, 1948).

On February 10, 1948, a portion of this bridge failed with the loss of three lives. Ice, formed in the Delaware Bay during a prolonged cold spell, was carried into the inlet by a northeast wind and flood tide. The bridge supports blocked the flow of the ice and an estimated 6-to 10-foot-thick ice field formed east of the bridge. This had happened several times before during the week preceding the bridge failure with no apparent damage,and the ice had been carried out to sea by ebb tide each time.

At the time of the failure, the principal tidal flow through the inlet was on the south side of the channel. Depths on the south side ranged from 26 to 34 feet compared to 13 to 20 feet on the north side (several years earlier, the principle flow was on the north side of the channel). The northeast winds had produced swells about 6 feet high at the bridge, which in conjunction with the wind and flood tide drove the ice mass against the pilings.

About 8:45 a.m., a 117-foot-long section of the bridge deck on the southern side of the inlet failed. A car and light truck were on this portion as it fell. The fallen concrete deck was carried about 250 feet west of the bridge before it sank to the bottom. Of the two of the four vehicle occupants who made it to shore, one died enroute to the hospital (West, unpublished). The bridge's fender and draw protection were also heavily damaged. Ironically, at the time of the failure, the bridge tender was off the bridge reporting by telephone to the State Highway Department Engineer that there was no evidence of distress in the bridge (Delaware State Highway Department, 1948).

A temporary one-lane timber trestle was built by May, 1948, and final repairs were not completed until June, 1951 (Delaware State Highway Department, 1950 and 1951). The southern portion of the bridge, which was completely replaced, was supported on concrete piles. This bridge was also damaged in the March 1962 storm and was closed to traffic until repairs were completed in July, 1962 (Delaware State Highway Department, 1962). The southern end of this span was used as a fishing pier until it was removed in 1976.

The first span of the present high level (35 feet vertical clearance) bridge carrying Delaware Route 14 across the inlet was completed in 1965 (see Figure 11)(Delaware State Highway Department, 1965). The deck of the second span should be completed some time in 1976. Each new bridge across the inlet has been built seaward of the previous bridge. The past and present threatening of Route 14, north of Indian River Inlet, by erosion is partially a result of this seaward encroachment of the highway.

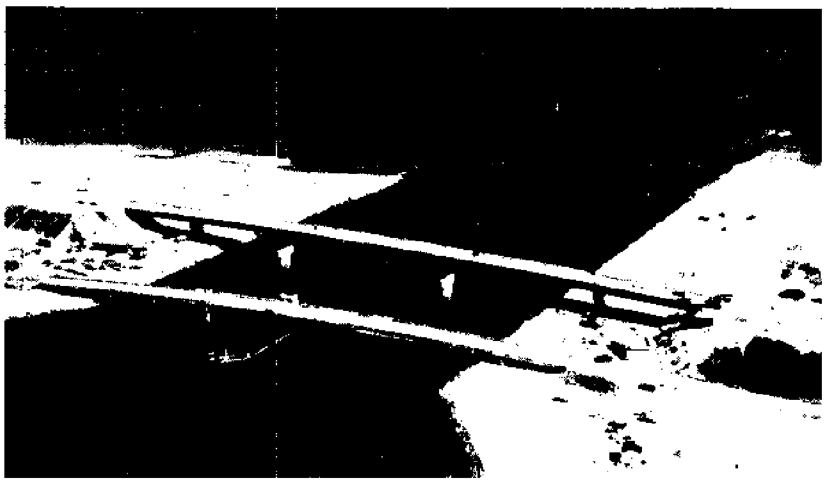


Figure 11 First Span of the Present Highway Bridge Nearing Completion and Charles W. Cullen Bridge, October, 1964 (Delaware State Highway Department, 1975)

2.2 Recent Shoreline Changes

The stabilization of Indian River Inlet initiated large changes in the local shorelines. Some of these were anticipated (Shore Protection Board, 1936); others were not. This section will discuss the general patterns of these changes. A more detailed discussion of each of these changes and the concurrent sediment volume changes and sediment movement patterns will be presented in the next chapter.

Figure 12 is an aerial photograph of Indian River Inlet taken May 7, 1938, and it shows the jetties under construction (Barwis, 1975). This photo depicts the conditions at the inlet before stabilization. Note the narrowness and shoaling of the inlet. The beaches north and south of the inlet lie on a straight line. The small body of water just north of the inlet and between the road and beach was the inlet channel around 1933 (Delaware State Highway Department, 1933). The channel migrated northward to that position before being returned to a straighter alignment by dredging in 1933.

Figure 13 shows the inlet on July 20, 1954. The pattern of accretion on the beach south of the inlet and erosion on the beach north of the inlet can be seen developing. Major erosion of the channel banks has occurred, and shoals can be seen in the inlet. The ebb tidal flow can be seen as a plume turning south upon entering the ocean. This plume, its causes and effects, will be discussed later.

Figure 12 Indian River Inlet, May 7, 1938 (National Archives, 1975)

Figure 13 Indian River Inlet, July 20, 1954 (U.S. Department of Agriculture, 1975)

Figure 14 shows the inlet and its changing shoals in 1960. Figure 15 was taken March 15, 1962 several days after a devastating northeasterly storm hit the Delaware coast. Large volumes of sand were carried west across the road and deposited in the marsh in overwash fans. Figure 16 and Figure 17 show the inlet on May 2, 1968 and June 8, 1971, respectively. The erosion of the beach north of the inlet can be seen threatening the approach to the new inlet bridge. A complex flood tidal shoal is also developing in Indian River Bay.

The relative positions of the shorelines are illustrated in Figure 18. For various years, shorelines at mean sea level were reduced to a common scale and overlayed. The erosion and accretion patterns can be visualized from this figure.

The accretion on the ocean beach south of the inlet is a maximum of about 340 feet seaward at a point 2,000 feet south of the inlet. This point corresponds to a protuberance on the south beach. By examining Figure 18, the bulge can be seen propagating southward from the inlet. The south beach is presently accreting, but at a considerable distance south of the inlet.

North of the inlet, the beach has been eroding. The beach has receded landward a maximum of about 480 feet at a point 2,000 feet north of the inlet. The erosion rate was highest from 1938 to 1954. The reduction in erosion rate is due, at least in part, to the beach nourishment projects which have been conducted since 1957.

Figure 14 Indian River Inlet, 1960 (U.S. Department of Agriculture, 1975)

Figure 15 Indian River Inlet, March 15, 1962. After the March, 1962 Storm (U.S. Geological Survey, 1975)

Figure 16 Indian River Inlet, May 2, 1968 (U.S. Department of Agriculture, 1975)

Figure 17 Indian River Inlet, June 8, 1971 (National Aeronautics and Space Administration, 1975)

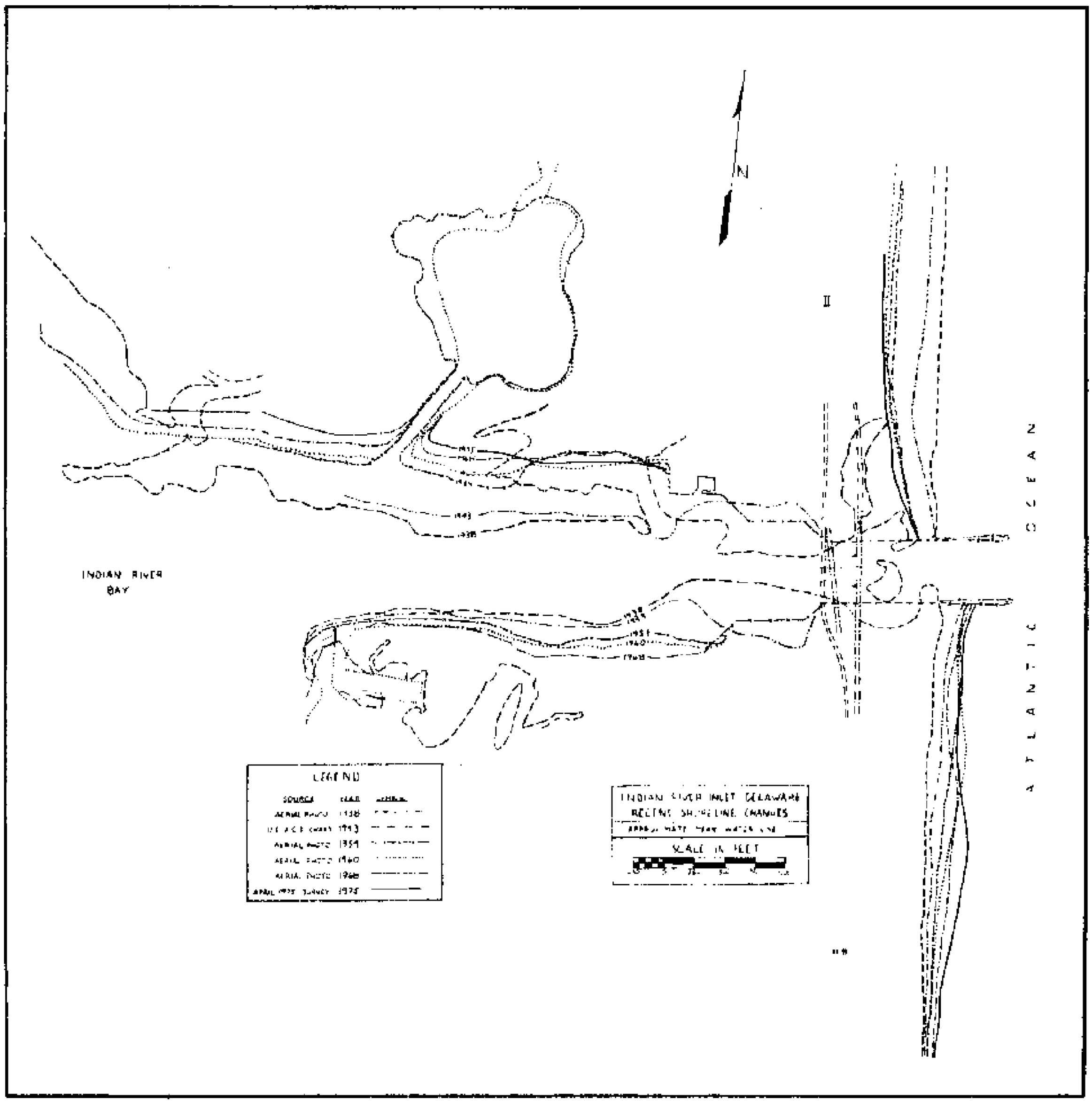


Figure 18 Recent Shoreline Changes at Indian River Inlet

The channel bank erosion can be seen west of the highway bridges. The banks have eroded a maximum of about 580 feet on the south side and 720 feet on the north side of the inlet. Bulkheads and rip-rap have almost stopped erosion of the south channel bank, but the unprotected north bank is continuing to erode.

CHAPTER III

SEDIMENT MOVEMENT

3.1 Overview of Sediment Movement Patterns

Unlike most of the U.S. Atlantic coast, the dominant littoral drift along a large portion of Delaware's ocean beaches is northward. Cape Henlopen, Delaware, a simple sand spit, is accreting and advancing northwest into the mouth of the Delaware Bay. The net littoral drift at Indian River Inlet and Rehoboth Beach, Delaware is also northward. A net southerly littoral drift is indicated at Ocean City Inlet, Maryland, 20 miles south of Indian River Inlet, by a large accretion north of the north jetty and major erosion on the northern end of Assateague Island, south of the inlet.

The nodal point, the dividing line between northerly and southerly net littoral drift, is between these two inlets. The groins in Bethany Beach show accretion on their south side in the summer and on their north side in the winter. The net accretion indicates a slightly northerly net littoral drift at this point. Thus the nodal point is believed to be in the vicinity of Bethany and South Bethany Beaches. The net littoral drift on the five miles of beach between Bethany Beach and Indian River Inlet is northward

(U.S. Army Corps of Engineers, 1968). This reversal of littoral drift on the north end of the Delmarva Peninsula is similar to the reversal on the northern portion of New Jersey's beaches.

Sand moving northward, as the net littoral drift, encountered the jetties built to stabilize Indian River Inlet and was trapped. The beach accreted seaward until the sand could pass the south jetty. Sand presently passes seaward and over the top of the south jetty and then enters the inlet channel. On flood tide, the currents in the inlet carry this sand into the inlet and deposit it in shoals in the channel and in a flood tidal delta in Indian River Bay. The sand from the beach, supplemented with sand eroded from the channel banks and bay and inlet bottom, is carried into the ocean by the ebb tidal currents. Sand deposited in the ocean has formed an ebb tidal shoal.

There is little sand transport from the ebb tidal shoal to the north beach. The lack of sand to replenish the sand carried away by littoral drift northward from the beaches north of the inlet has resulted in severe erosion north of the inlet. The state and federal governments have been forced to nourish this beach artificially in order to reduce erosion.

In order to document this net sand movement pattern as well as the impoundment of sand by the inlet, hydrographic surveys were conducted of the inlet. These surveys and their interpretation are used to develop the overall sand budget of the inlet.

3.2 Project Hydrographic Surveys

A series of three hydrographic surveys of the inside of the inlet and the ebb tidal shoal were conducted. The surveys were made in September, 1974, April, 1975, and November, 1975 to show changes in the inlet bathymetry over a one-year period. Further, they were compared to older charts made by the U.S. Army Corps of Engineers to determine long-term changes to the inlet.

The first step in the production of the charts was to establish a semi-permanent baseline along the north and south beaches and the south side of the inlet. The baselines on the north and south beaches had to be almost completely reestablished before each survey, due to removal of the baseline stakes by the heavy beach use and the erosion of the north beach. Water depths were obtained with a Raytheon recording fathometer mounted on a small outboard motor boat. The boat position on traverses of the inlet or out into the ocean was determined in one of two ways. First, the boat was directed along a straight line of known position relative to the baseline using a transit or a set of range poles. The position of the boat along this line was measured at various times (which were noted on the fathometer record) with a second transit at a known location. This method insured proper coverage of the area; however, it was time-consuming to set up each traverse and large errors in position could be introduced if the boat deviated from the line.

The second method of locating the boat was by measuring an angle to the boat with two transits of known location. This method allowed the boat to travel freely and allowed much more rapid data collection. Undesirable gaps between survey lines can result from this method, however.

The mean water surface elevation was determined with a tide staff located near the bridge during the first survey. A tide gauge at the South Shore Marina (located at the bay end of the inlet) was used during the second survey and tide gauges at both the South Shore Marina and in the ocean were used during the third survey. The elevations were determined by leveling into National Ocean Survey tidal benchmark S-27 which was set in the concrete foundation of an electric transmission tower on the south side of Indian River Inlet. The benchmark had an elevation of 6.585 feet above Sea Level Datum of 1929. (See next section for discussion of vertical datums.) The tidal corrections were applied to the sounding data from both inside and outside the inlet.

Because a small boat was used for the soundings, ocean waves of up to seven feet were sometimes superimposed on the sounding record. Difficulty in reading this record due to the waves could produce a random error of up to 0.5 feet; less inside the inlet and on calm days. Since the tide elevation during the first two surveys was not monitored in the ocean, the head drop across the inlet produced an error in the tidal correction used for the ocean soundings. This head drop across the inlet reached 1.2 feet during maximum ebb or flood tide and would produce a systematic error. This source of error was eliminated in the third survey by obtaining tidal corrections directly from the ocean tide gauge.

The beach and dunes were surveyed with an engineer's level and fiberglass rod. Profiles were run at 90° from the baseline as far into the surf as possible to overlap with the boat soundings.

Corrected depths and elevations and their locations were plotted with additional points interpolated as necessary. These points were then contoured to produce the bathymetric charts. The northernmost portion of the outer shoal was not surveyed due to equipment failure in the September, 1974 survey. The configuration of the outer edge of this shoal is also suspected to be inaccurate due to errors in locating soundings. Also, the configuration of the outer shoal as shown on the April, 1975 chart may not be exact in the places indicated. The north beach was not surveyed in April, 1975 due to extremely high winds (50 mph plus for three days).

The location of the north bank of the inlet channel on the September, 1974 chart may be inaccurate due to deviation by the boat from the traverse line close to the north bank. The vicinity of South Shore Marina was not surveyed in detail in the first survey. Hydrographic data for the central portion of the April, 1975 inlet chart was taken from the U.S. Army Corps of Engineers pre-dredge survey of December 14, 1974. Dredging was underway during the April, 1975 survey.

The inner and outer charts from the three surveys are shown in Figures 19 through 24. Many features of the inlet and shoals maintained

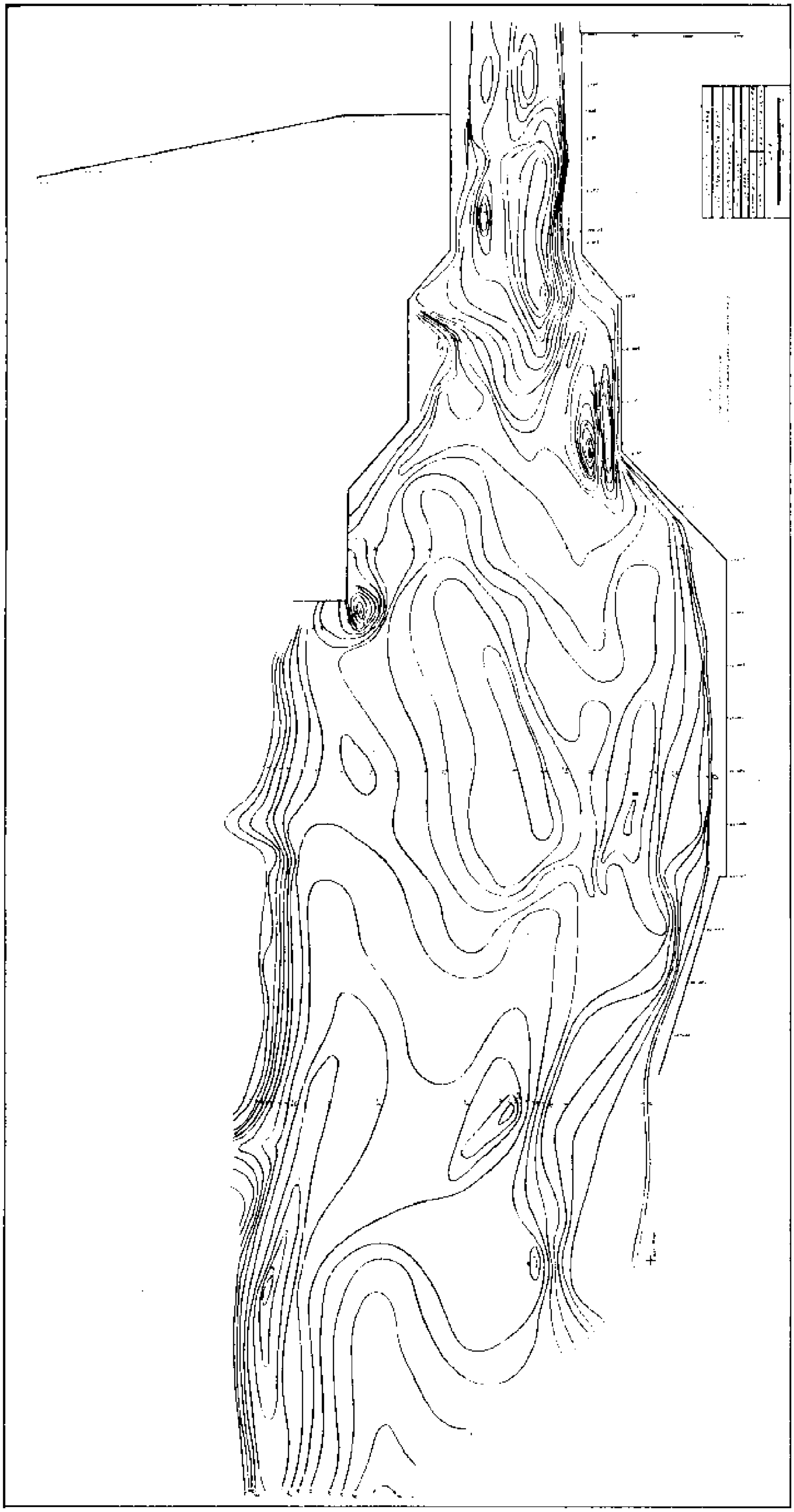


Figure 19 Hydrographic Survey of Indian River Inlet Channel, September, 1974. See Figure 2 for Baseline Orientation.

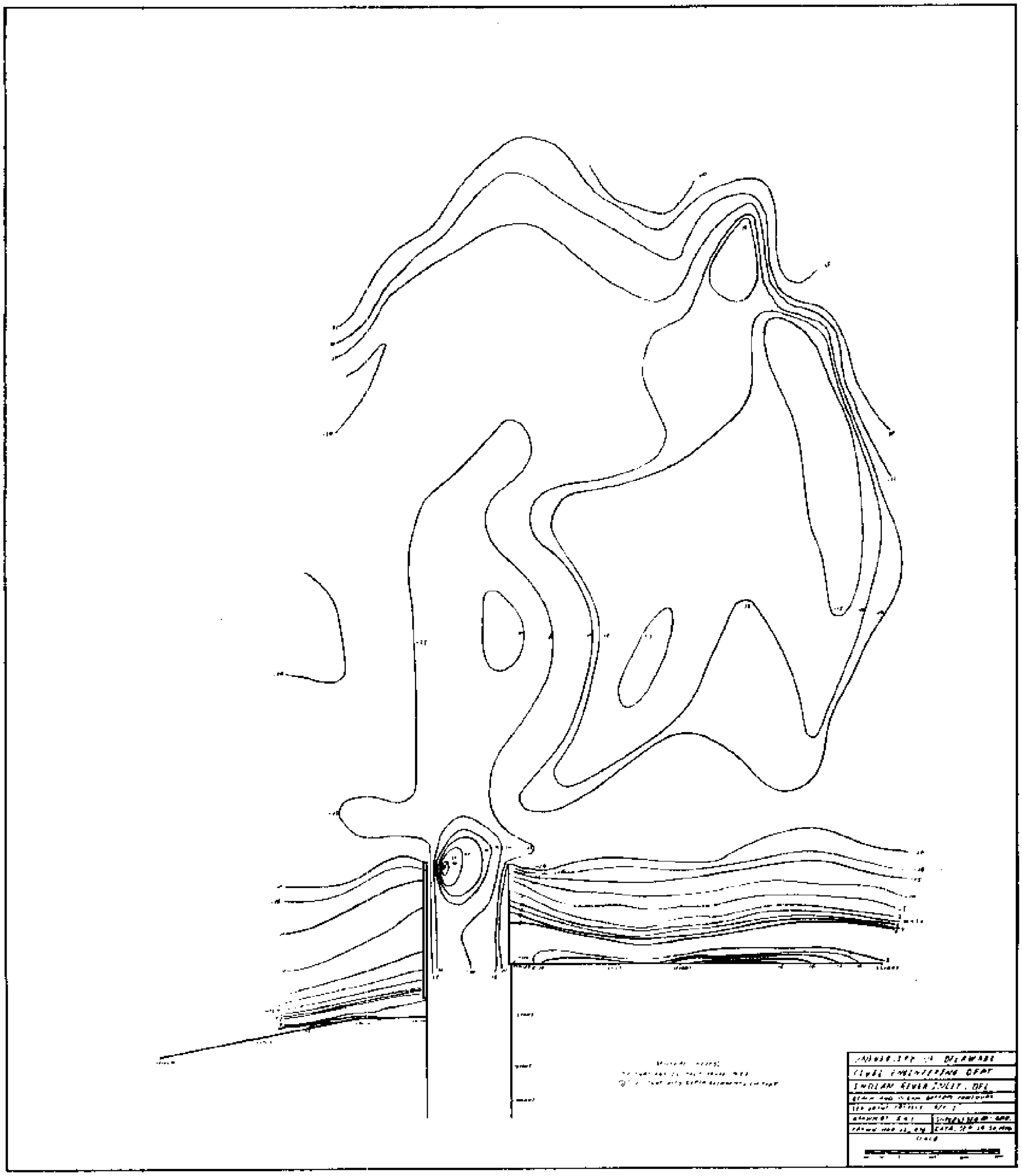


Figure 20 Survey of Beaches and Ocean Bottom in Vicinity of Indian River Inlet, September, 1974. See Figure 2 for Baseline Orientation.

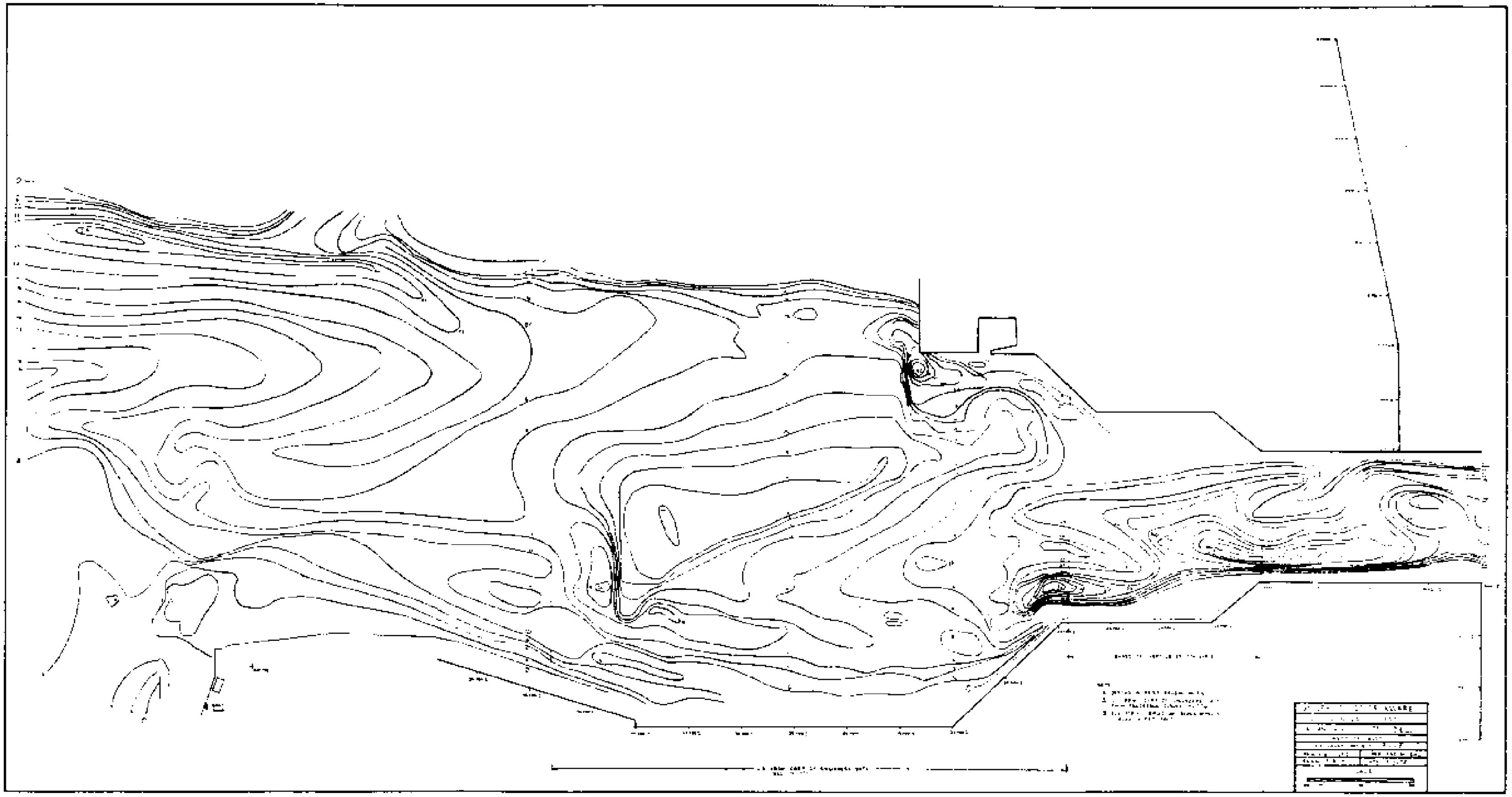


Figure 21 Hydrographic Survey of Indian River Inlet Channel, April, 1975

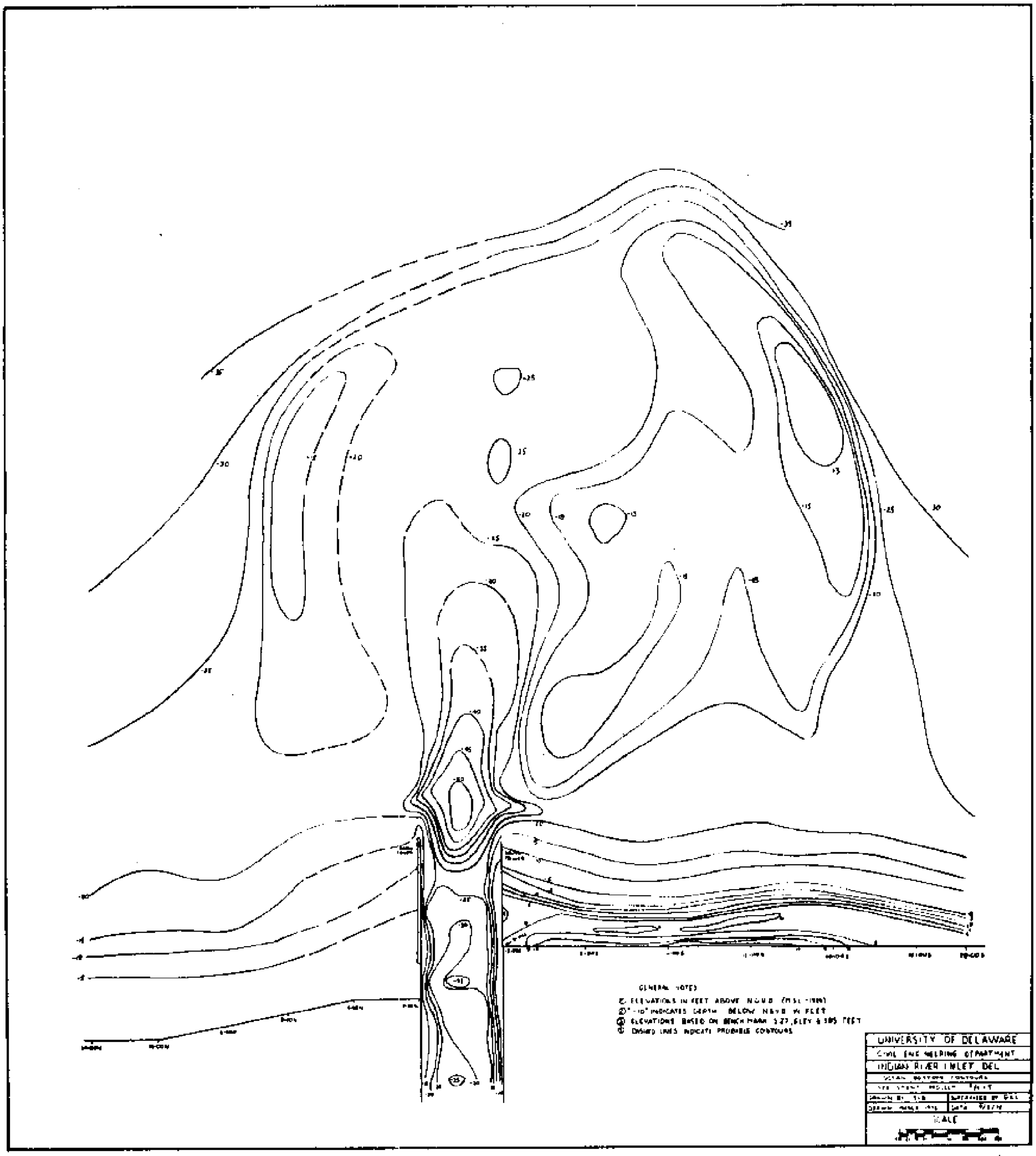


Figure 22 Survey of Beaches and Ocean Bottom in Vicinity of Indian River Inlet, April, 1975

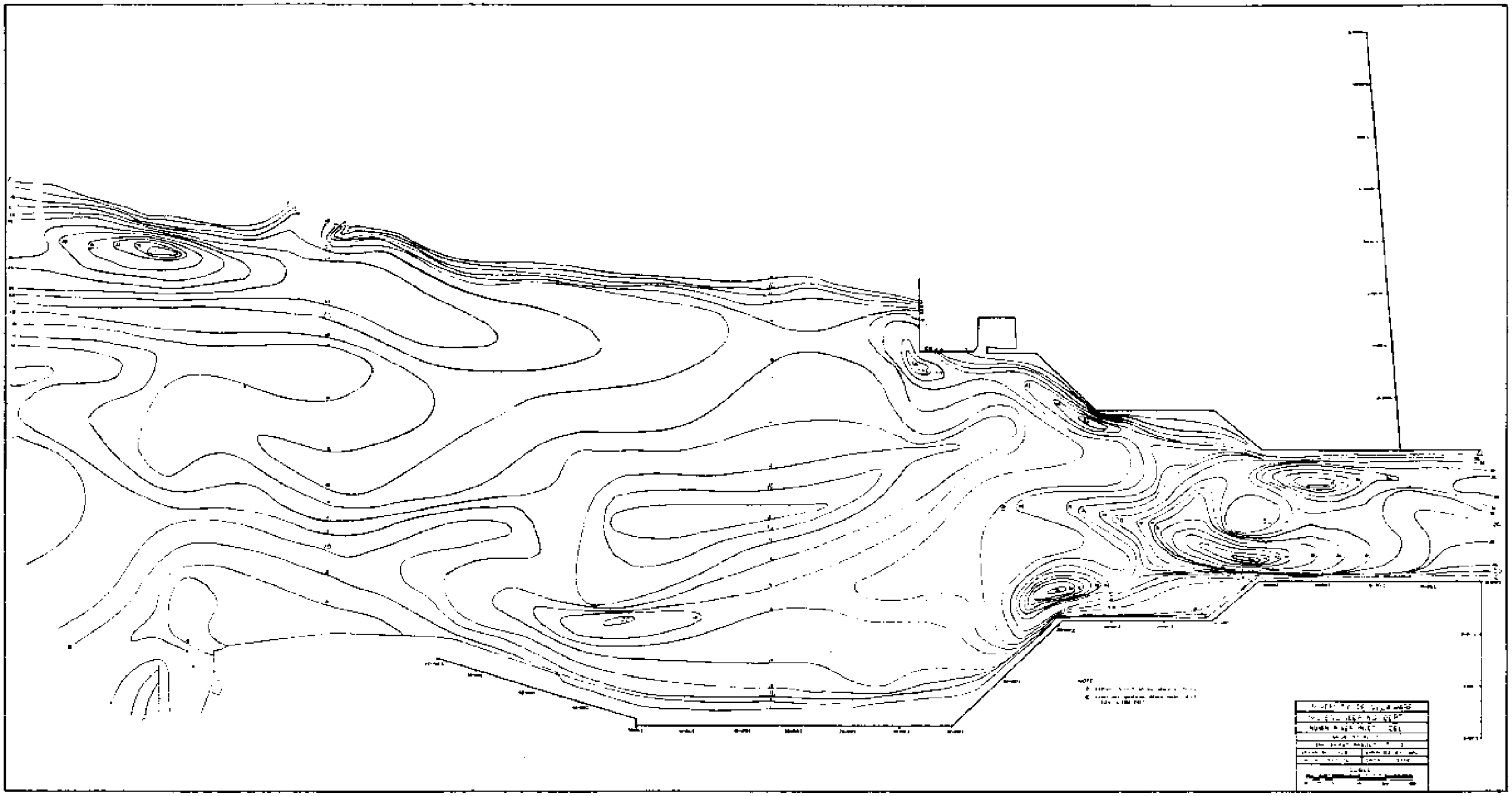


Figure 23 Hydrographic Survey of Indian River Inlet Channel, November, 1975

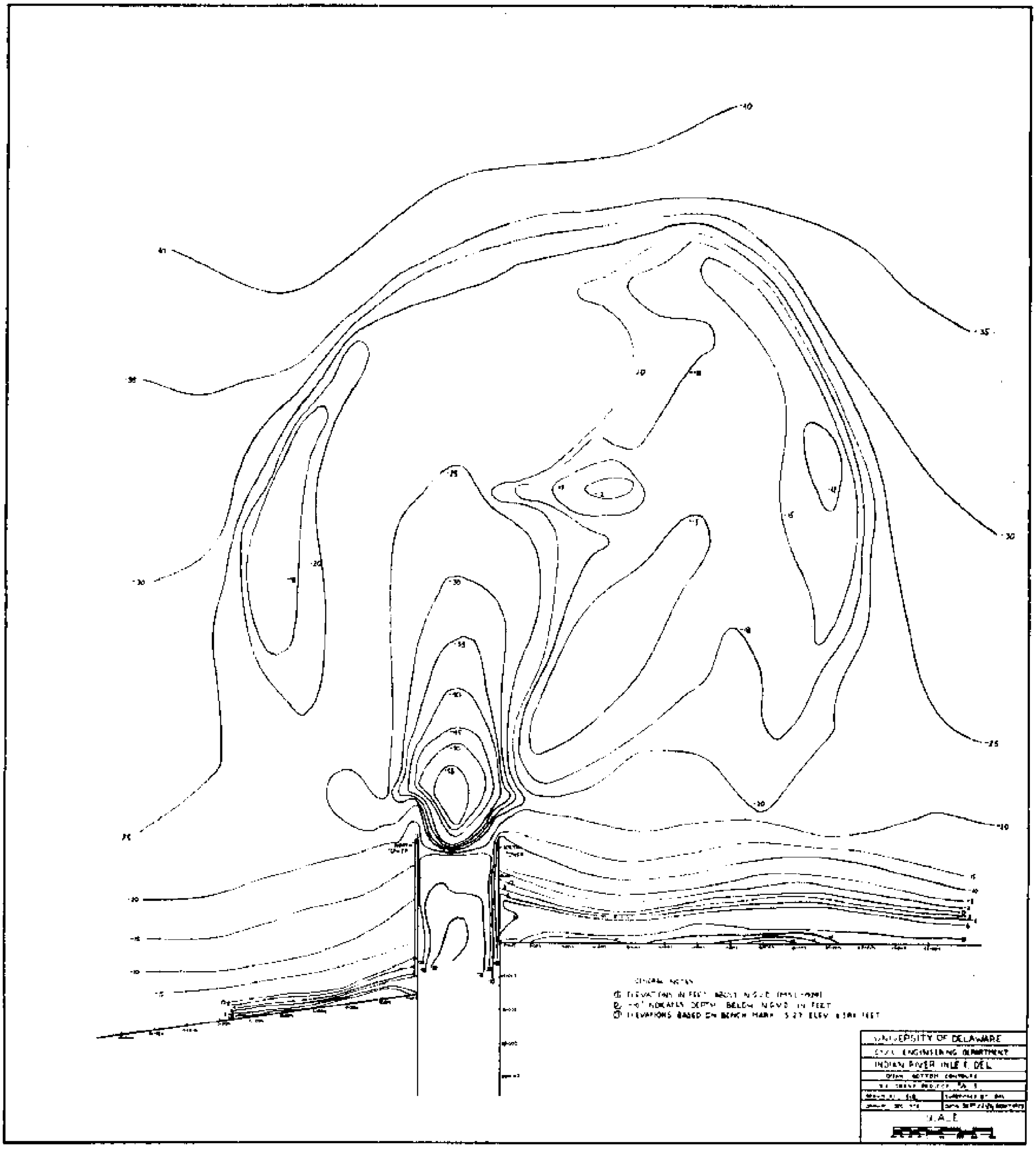


Figure 24 Survey of Beaches and Ocean Bottom in Vicinity of Indian River Inlet, November, 1975

a fixed position over the duration of the study. Inside the inlet, while the shapes and depths of water over shoals and areas of scour may vary, features such as the large shoal in the middle of the inlet (southwest of the Coast Guard Station) and various scour holes remain from survey to survey. The general features of the outer shoal show little change over the study period.

3.3 Vertical Datums

Before two different charts of the same area can be compared to calculate sediment volume changes or changes in some other factor, it is necessary to know the relationship between the datum planes of the two charts. For this reason, the various vertical datums at Indian River Inlet and how these datums are changing with time must be determined.

A popular task for coastal geologists is constructing sea level rise curves. This consists of tracing sea level elevations back through time. Unfortunately, there is very little agreement between various authors on the form of this curve.

The large disagreement between theories is partly a function of the reference plane used. A sea level change on a coast can be a result of the water level changing or the land moving vertically. Vertical land movements can be caused by sediment compaction, tectonics, or isostatic effects. If an individual is on a sinking coast, he will experience a relative sea level rise while the sea level may actually be constant.

For work on a specific coast a "relative sea level curve" is particularly useful if it was derived on that coast. Figure 25 shows

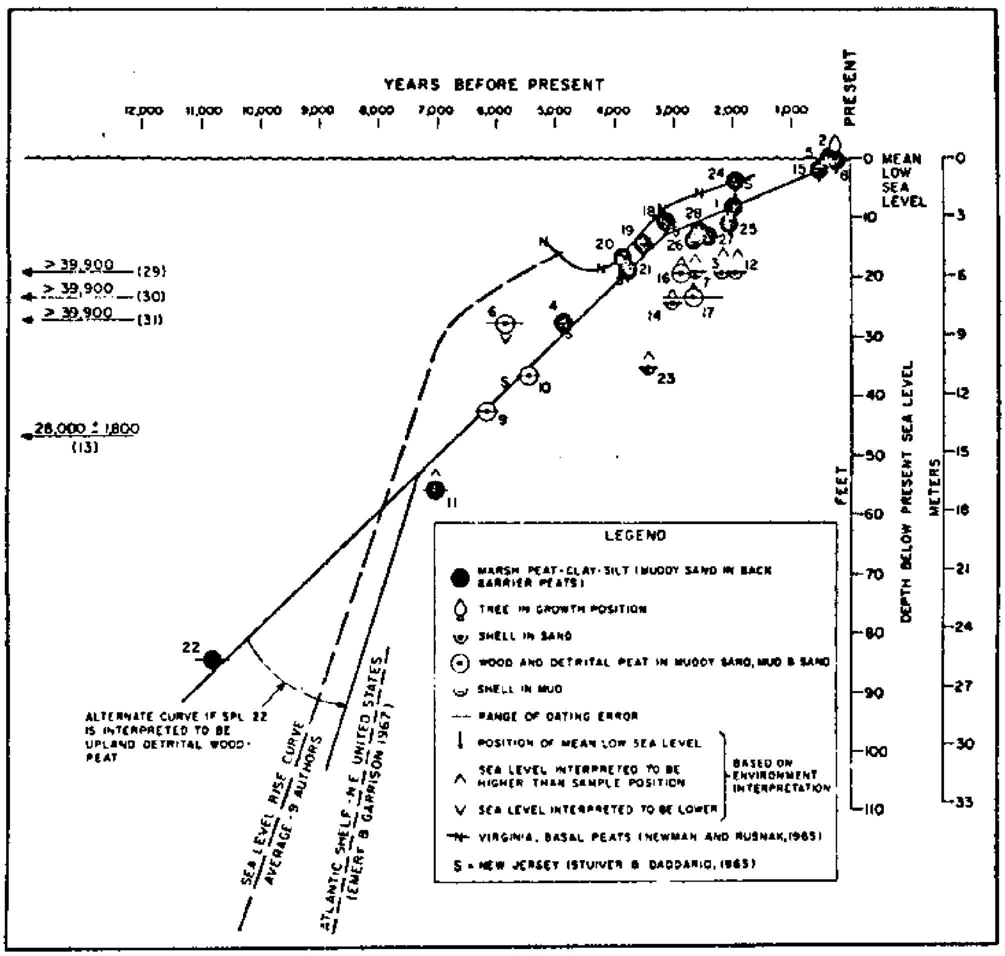


Figure 25 Relative Sea Level Rise Curve for the Delaware Coast (Kraft, 1971b)

the relative sea level rise for the Delaware coast. It indicates a sea level rise of approximately 0.5 feet per century for the past 3,000 years. From 3,000 to 7,000 years before present, the sea level rose at a rate of approximately one foot per century preceded by a period of very rapid rise. The sea level was estimated to have been 320 to 340 feet below its present level 12,000 years before present (Kraft, 1971b).

This curve varies from the curves of investigators who claim sea level has remained relatively stable for the past 5,000 years (Bruun, 1962). Other investigators claim that sea level was above the present level 4,000 to 5,000 years before present and has subsequently fallen. The earth is believed to be in an interglacial stage, but it is approaching another glacial stage in perhaps 10,000 or 15,000 years (Bruun, 1962).

Evaluation of tidal data collected by the U.S. Coast and Geodetic Survey at Cape Henlopen shows sea level rising at a rate of 0.3 meters (1 foot) per century over a recent 40-year period (Moody, 1964). Similarly, the tidal datum determined from the 1941-1959 epoch at Atlantic City, New Jersey was 0.23 feet higher than that for the 1924-1942 epoch (Hubbard, 1975). This rate, about 1 foot per century, can prove significant in the computation of sediment volumes over large areas from charts with local sea level as a vertical datum. The volume correction equals the horizontal area involved multiplied by the difference in datums. As an example, the ebb tidal shoal of Indian River Inlet covers about one square mile. Between 1938 and 1975, a period of 37 years, a sea level rise of 1 foot per century would produce an apparent erosion of 382,000 cubic yards of sediment over the one-square-mile area.

Similar complications develop when tidal ranges vary and the datum of a chart is mean low water (MLW) or local mean low water (LMLW). An increase in tidal range will lower LMLW if the mean tide level (MTL), located half way between mean high water (MHW) and mean low water (MLW), is held constant. The MTL is usually close to the mean sea level (MSL), which is the actual average water surface elevation. The tidal prism and subsequently the tidal range in Indian River Bay and Rehoboth Bay have increased greatly with the increase in cross-sectional area of the inlet. The MSL in a bay may be elevated due to freshwater inflow as has been the case with Indian River Bay.

The elevations of tide planes, computed from 21 months of record, March 1972-February, 1974, at the Coast Guard Station at Indian River Inlet, Delaware are shown in Table 1. Surveys made as part of this study and all historical charts were related to NGVD because this datum has a fixed elevation over time. National Geodetic Vertical Datum of 1929, NGVD (formerly Sea Level Datum of 1929) has been referred to as "mean sea level, MSL" by engineers and surveyors and on topographic maps, but it should not be confused with the local mean sea level described above.

TABLE 1 ELEVATIONS OF TIDE PLANES AT INDIAN RIVER INLET, DELAWARE

Datum	Feet
Mean High Water, MHW	2.20
Local Mean Sea Level, LMSL	1.16
Mean Tide Level, MTL	1.10
National Geodetic Vertical Datum of 1929, NGVD (Formerly Sea Level Datum of 1929)	0.80
Mean Low Water, MLW	0.00

The ocean tide ranges on the Delaware-Maryland outer coast, as given in the N.O.A.A. Tide Tables (1975), are shown in Table 2. Indian River Inlet, located midway between Rehoboth Beach and Fenwick Island Light, has a mean ocean tide range of 3.8 feet and a spring range of 4.6 feet. Other tide ranges in the Indian River area are given in the N.O.A.A. Tide Tables (1975b) as follows: Indian River Inlet (Bridge) 2.7 feet mean, 3.2 feet spring; 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, 1.2 feet spring; Rehoboth Bay

0.5 feet mean, 0.6 feet spring. (See Figure 1 for locations.) These values are suspected to be changing with time as the flow characteristics of the inlet change and may be inaccurate. The 0.6-foot difference in mean tide range and 0.7-foot difference in spring tide range between the Coast Guard Station and the bridge (north end of old swing bridge) seems high since the two points are only 1200 feet apart. The mean tide range at the Coast Guard Station indicated in Table 2 is 0.1 feet less than that given in Table 1. Examination of recent tidal record charts from the Delmarva Power and Light generating station suggests a mean tidal range of about 1.7 feet rather than a range of 1.0 feet or 0.9 feet (Gibbons, 1975).

TABLE 2 OCEAN TIDE RANGE (feet)

Location	Mean	Spring
Cape Henlopen	4.1	4.9
Rehoboth Beach	3.9	4.7
Fenwick Island Light	3.7	4.5
Ocean City, Maryland	3.4	4.1

The profile of tides in Indian River Inlet and Indian River Bay are shown in Figure 26 for 1948-1950 tides. Keulegan estimated the ocean tidal range as 4.1 feet (the value at Lewes, Delaware) and assumed that the local MSL was the same in the ocean and at the inlet bridge. Figure 26 illustrates the decrease in tidal range and water super-elevation from the ocean to the bay. In 1950, the variation in LMLW in the inlet vicinity is shown in Table 3.

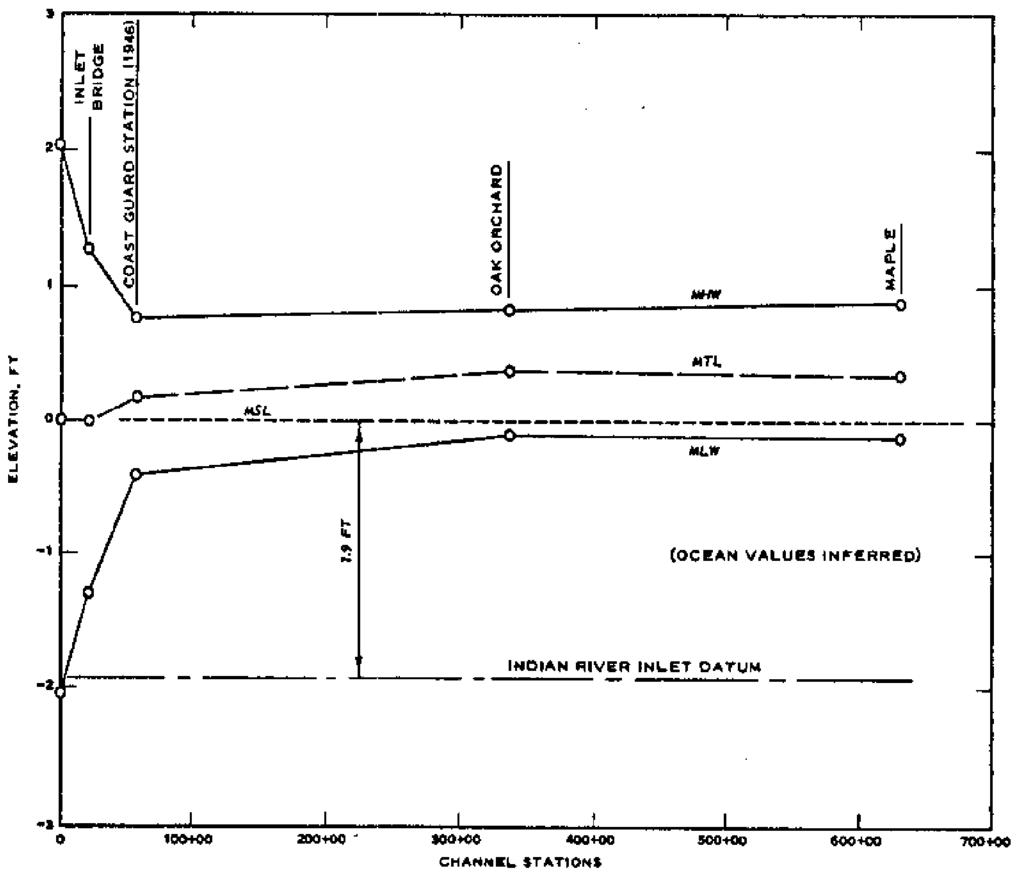


Figure 26 Profile of Tides in Indian River Inlet and Bay, 1948-1950 (Keulegan, 1967)

TABLE 3 VARIATION IN LOCAL MEAN LOW WATER AT INDIAN RIVER INLET, DELAWARE IN 1950

Location	Feet Below NGVD
LMLW Ocean (Near Inlet)	-1.89
LMLW @ Sta. 20+00 (North Side of Swing Bridge)	-1.18
LMLW @ Sta. 86+00 (Bay Entrance)	-0.14

The elevations of local mean sea level and tidal ranges have varied over the history of the inlet. These variations are a function of local relative sea level rise, freshwater runoff into the bays, and the changing hydraulic conditions of the inlet and the Ditches connecting the two bays. Further study of the tides and superelevation of the bays is necessary to update older data.

3.4 Volume Computational Technique

The erosion and accretion trends at Indian River Inlet were quantified by comparing a series of historical charts and the charts drawn as part of this study. To facilitate comparison of the study charts with the U.S. Army Corps of Engineers' charts, the inlet baseline was established coincident with the baseline used by the Corps. The inlet baseline and the baselines on the north and south beaches are shown in Figures 19 through 24.

The first step in calculating the volume of the outer shoal was to establish reference areas on various charts. The area of this shoal has increased consistently since the inlet was stabilized, and failure to consider the increased area of the shoal lends to an erroneous volume computation. For each area the volume of water from the 1936 survey minus the volume of water in the same area from a later chart yields the volume of sediment accreted between surveys after applying a correction for changes in the vertical datum. Water volumes in the offshore area were computed by planimetering the area between successive contours and multiplying by the average depth between the contours (Dean and Walton, 1975). Several volumes obtained in this manner were checked by subdividing the area into grids and computing an average depth for each grid. The volumes computed by both methods were very close.

The primary source of error in the volume computations was error in the charts. Older charts of the inlet show few data points, which made exact volume computations difficult. It is also unknown if relative differences in water surface elevation between inlet and ocean are accounted for in the tidal corrections.

Inside the inlet, volumes were calculated by comparing cross sections taken at right angles to the baseline. In addition, volume calculations were complicated by dredge spoils placed on the channel banks, which subsequently were eroded as the channel widened. This transfer of material had to be considered in the sand budget.

The lack of an accurate survey of the beach north of the inlet dictated the use of another volume computational technique. The volume of erosion (or accretion) per linear foot of beach can be determined from the recession (or advancement) of that beach. It is assumed that the beach profile is the same before and after erosion (or accretion) but simply displaced landward (or seaward). This displacement extends from the berm crest to a depth beyond which there is no erosion. At Indian River Inlet, the berm crest is about 7 feet above MSL and the beach changes to a depth of about 20 feet. This yields a conversion factor of one cubic yard (c.y.) of sediment per foot of shoreline change per linear foot of beach (Dean and Walton, 1975 and U.S. Army Corps of Engineers Coastal Engineering Research Center, 1973). Volumetric computations for individual areas at Indian River Inlet will be presented in a later section.

3.5 Accretion on the South Beach

Upon construction of the jetties, the net northerly littoral drift was obstructed. This drift was initially blocked by the south jetty, which extended to the 14-foot depth contour in the ocean (about 550 feet from the beach). This trapped sand has caused the beach south of the inlet to build seaward as much as 340 feet.

The U.S. Army Corps of Engineers has estimated the accretion rate above MLW on the south beach as shown in Table 4 (U.S. Army Corps of Engineers, 1968). Moody (1964) estimated the average accretion rate of the south beach to be 5,700 cubic meters (7,500 c.y.) per year from 1939 to 1961.

TABLE 4 ACCRETION ABOVE MLW AT INDIAN RIVER INLET

Period	No. of Years	Average Annual Accretion (cubic yards per year)
1939-1954	15	8,000
1954-1964	10	12,000
1939-1964	25	10,000

Charts drawn as part of this study were compared with old charts independently to calculate volumetric accretion on the south beach, particularly since 1964. The area over which this volume change was calculated extended from the south beach baseline at 12+00 I (station 12+00 on the baseline parallel to the inlet) offshore to 5+50 I (approximately the tips of the jetties) and southward from the south jetty a maximum of 2,750 feet (27+50 S). The volumes of accretion are shown in Figure 27. A total of 42,300 c.y. accreted on the south beach between September, 1974 and November, 1975.

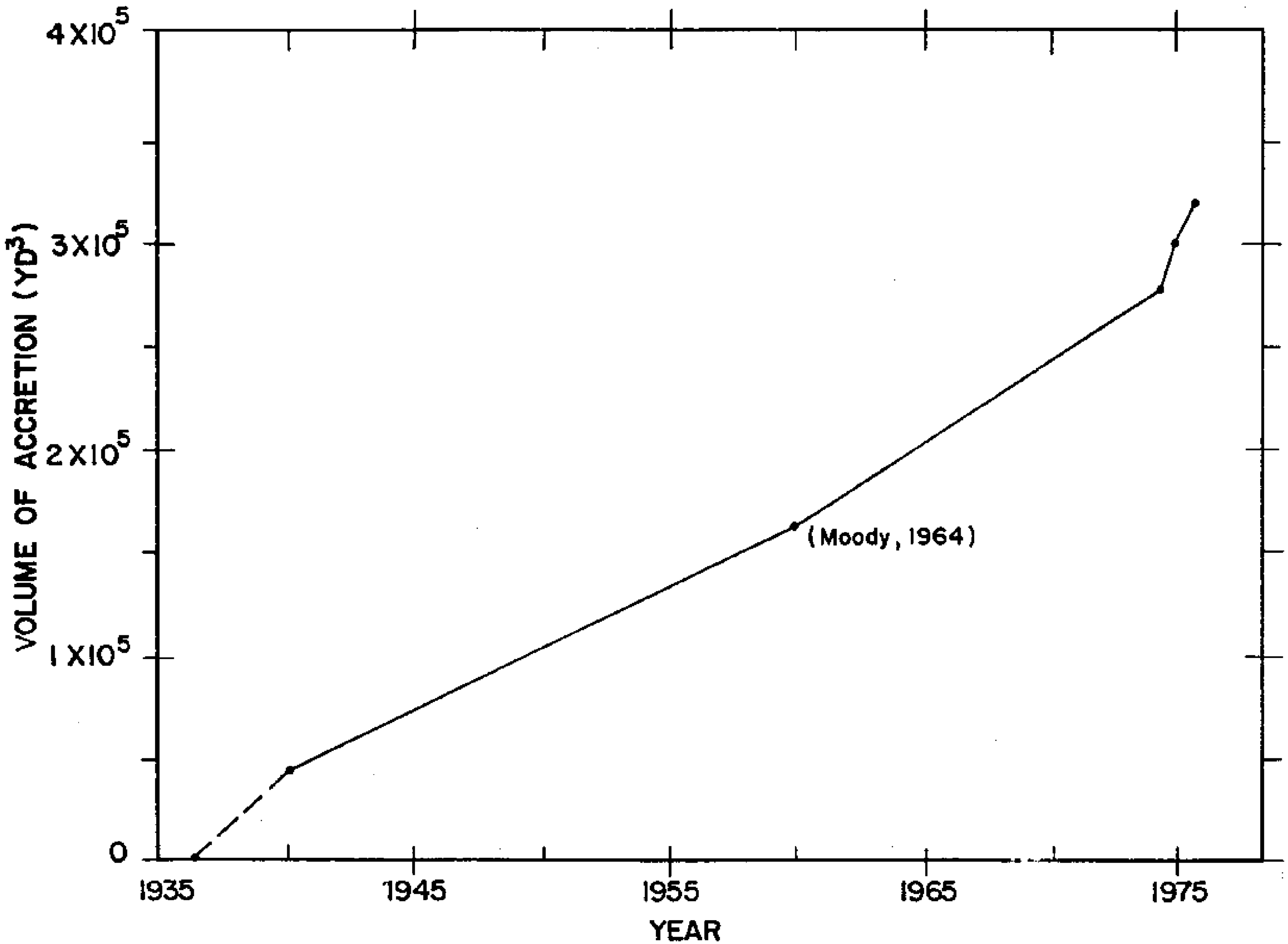


Figure 27 Volume of Accretion on the South Beach Versus Year

Figure 28 shows the approximate area of accretion on the south beach since the inlet was stabilized. The areas were obtained by planimetering Figure 18. Using the conversion factor discussed in the previous section of one cubic yard of sediment for each square foot of beach accretion, the calculations suggest a much higher volume has accreted than indicated in Figure 27. This discrepancy is due to the beach profile not remaining constant but steepening. There has also been erosion in the seaward portion of the area considered.

The configuration of the accretion on the south beach is unusual (see Figure 18). As early as 1954, there can be seen a seaward protuberance, or bulge, of the beach at a distance south of the south jetty. This pattern has continued to extend seaward and propagate southward until presently the point of maximum accretion is 2,000 feet south of the south jetty. The bulge can be seen consistently since 1954 and is even quite prominant in photographs taken immediately after the March, 1962 storm (see Figure 15). This bulge presently composes a large portion of the accreted beach area. The apex of the bulge in the south beach has moved southward, keeping pace with the southern edge of the outer shoal.

One possible cause of this bulge is that sand is being transported landward on the southern edge of the outer shoal by waves from the southeast. However, no connection is found between the

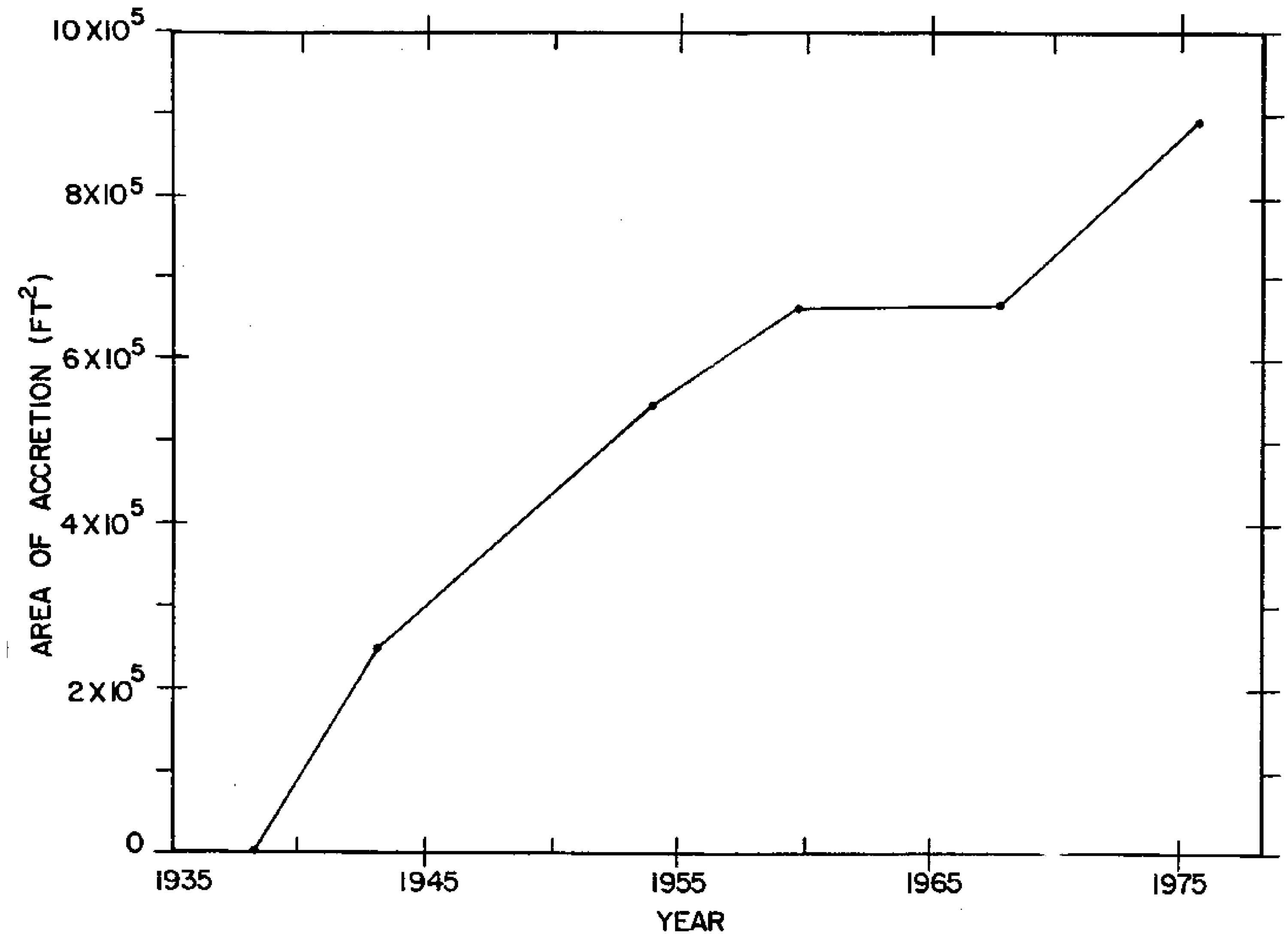


Figure 28 Area of Accretion on the South Beach Versus Year

outer shoal and the south beach as indicated from an examination of charts. The water between the two features reaches a maximum depth of 20 feet. Also, the sand tracer study, discussed later in this thesis showed no significant landward movement of sand. A more likely cause of the bulge is wave refraction patterns over the outer shoal. The shallow water over the southern edge of the outer shoal also causes large waves to break and lose energy. This tends to shelter the beach behind it. It is also possible that the bulge 2,000 feet south of the inlet, is a manifestation of an indentation in the shoreline, about 1,000 feet south of the jetty. This could be caused by a concentration of wave energy from refraction of waves coming in over the outer shoal, plus diffraction and reflection of waves from the south jetty. On several occasions, the authors have observed generally larger waves in the region 1,000 feet south of the inlet than at other locations on the beach. This could be a result of wave refraction. The irregular shoreline could also be a result of a combination of previously-mentioned processes.

3.6 Sand Passing the South Jetty

The northerly littoral drift carries more sand to the beach south of Indian River Inlet than accumulates on that beach. The beach has reached a quasi-state of equilibrium by building seaward until sand can once again pass the south jetty at approximately the same rate as it reaches the south beach.

The beach berm crest at Indian River Inlet is generally 7 to 7.5 feet above NGVD, while the crest of the south jetty is about one foot lower than that of the beach berm crest. During periods of high tides, waves sweep up the beach,across and through the south jetty,and into the channel,carrying large volumes of sand, which is deposited in the channel. A photograph of this process is shown in Figure 29.

This is believed to be the major process by which sand is carried past the south jetty. Sand carried into the channel in this manner sometimes forms a sand beach which slopes down into the channel and is exposed at low tide. At other times, the strong inlet currents carry this sand away. A portion of the sand tracer study (conducted as part of this report) was designed to follow the path of this sand which is introduced into the channel at the south jetty.

Another (less frequent) means by which sand is carried over the south jetty is the backwash of waves overtopping the jetty from inside the inlet channel. Ocean waves propagating into the inlet (primarily during flood tide) overtop the south jetty as shown in Figure 30. The backwash carries sand into the channel, causing the beach to slope towards the jetty (as seen in the photograph).

Figure 31 shows sediment-laden water being carried seaward of the south jetty and then being entrained in the ebb tidal flow

Figure 29 Sand Being Carried Over the South Jetty by Ocean Waves

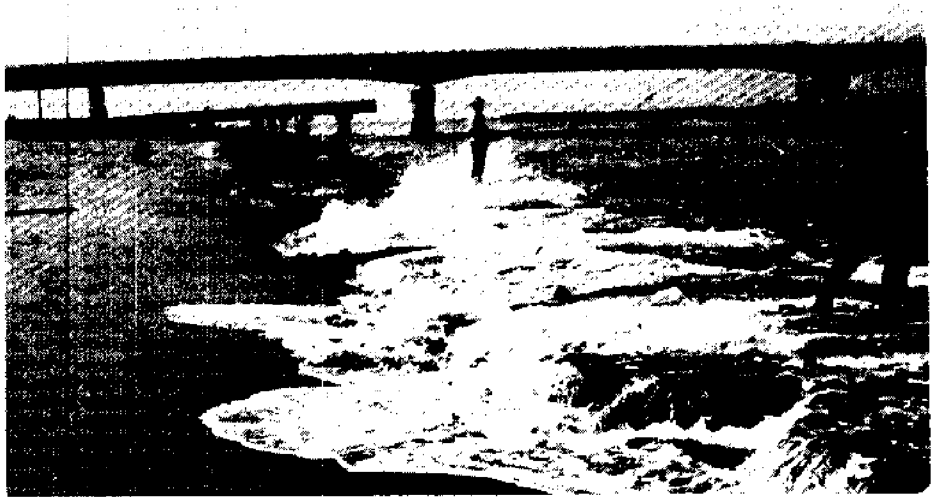


Figure 30 Wave Breaking Over the South Jetty from Inside the Channel

Figure 31 Sediment Laden Water Passing Seaward of South Jetty

to be deposited offshore. It is expected that this mode of sediment transport would be greater during flood tide due to increased northward currents in the ocean. The steep beach face near the south jetty also causes seaward sediment transport. Once seaward of the tip of the south jetty, the sand will tend to continue moving northward into the deep inlet channel. In the channel, the sand is then picked up and transported by the tidal currents.

3.7 Channel Bank Erosion

In addition to the sand which has entered the inlet at the south jetty, considerable sand has also been introduced through the erosion of the channel banks and bottom. The changing channel dimensions have also caused the inlet's hydraulic characteristics to change.

Upon completion of the dredging phase of the stabilization of Indian River Inlet, the channel banks began to erode rapidly. At station 24+00I, the channel widened about 580 feet during an 18-month period! These very high erosion rates are shown in Figure 32 for the inlet between station 22+00I (just west of the highway bridge) and station 27+00I, from October, 1938 to April, 1940. West of the highway bridge, the channel banks were composed of layers of sand, marsh mud, and dredge spoils in ascending order. The underlying sand and dredge spoils were eroded causing the marsh mud to break off and disintegrate (Wicker, 1939).

In 1939, Wicker observed waves propagating westward, up the inlet, during flood tide and the weaker parts of ebb tide. Upon entering the embayment west of the highway bridge, the waves would refract and diffract to approach approximately normal to the channel bank. The wave uprush would strike the marsh mud or scarp with considerable force and the backrush would carry away sand. He also observed

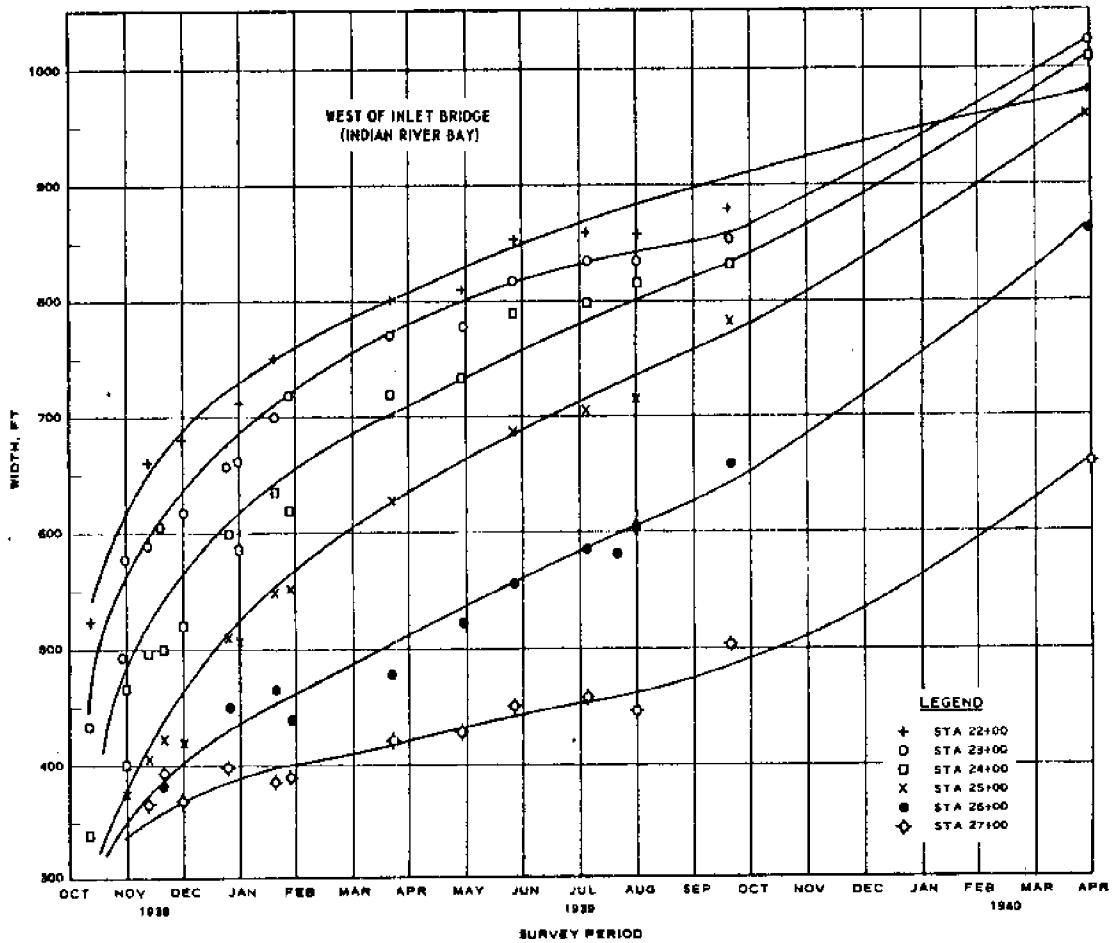


Figure 32 Early Changes in Channel Widths at High Water for Indian River Inlet (Keulegan, 1967)

eddies in the north and south sides of the embayment. These eddies, which no longer form, were not observed to influence the erosion.

In an attempt to stop the erosion and prevent the original highway bridge from being flanked, the state placed rip-rap along the channel bank west of the bridge. This did not stop the erosion, and in 1941, the U.S. Army Corps of Engineers constructed their first extension of the bulkheads, as mentioned in Chapter II. While this stopped erosion of the protected portion of the channel bank, a similar erosion pattern develpped further west on the unprotected portion of the channel bank. The south bulkhead had to be extended a second time in 1963. The changing configuration of the inlet channel banks is depicted in Figure 18.

The erosion of unprotected channel banks is common to other inlets as well as Indian River Inlet. An embayment similar to that west of the highway bridge at Indian River Inlet developed at St. George Island Channel on Florida's Gulf coast. At this inlet, waves which entered the channel from the Gulf of Mexico refracted, turning to strike the channel banks obliquely. These waves then broke, eroded sand, and transported it toward the bay. In the case of St. George Island Channel, it was recommended, in a study done by the Florida Engineering and Industrial Experiment Station (1970), that the entire length of the channel bank be protected with a bulkhead. An inlet channel cut through erodable material and stabilized with jetties will generally tend to widen if the channel banks are not protected.

The south channel bank along Road 50I (between the State Park campground and the trailer park) is presently eroding. The state has placed concrete rubble and an earthen embankment along the channel bank, but this has not stopped the erosion. Waves wash the earth out through the rubble, forming breaches as shown in Figure 33. It is felt that rip-rap and bulkhead protection similar to that on other portions of the channel bank are required to stop this erosion.

The north bank of the inlet channel is also presently eroding. The erosion rate varies from a maximum of about 20 feet per year on Burton Island to local accretion just west of the Coast Guard Station. Figure 34 shows the erosion of the north channel bank at about station 53+00I. The foundation and debris (seen in the photograph) are a remnant of one of the many shacks and trailers located on the north side of the inlet during the 1950's and 1960's. Since this area is presently undeveloped, there has been little concern expressed over this erosion. The erosion of the north bank of Indian River Inlet is expected to continue until some form of channel bank protection is constructed.

There are several causes of the present channel bank erosion at Indian River Inlet. The ocean waves moving into the channel have been cited by Wicker as the primary cause of this inlet's early bank erosion. This mechanism is believed to be still causing the erosion of the channel banks. As the inlet widens, however,

Figure 33 Breach Forming in Earthern Embankment on South Channel Bank

Figure 34 Erosion of North Channel Bank

and the wave energy is spread over a larger area, the importance of this cause of erosion would be expected to diminish.

During strong ebb tide, however, waves are generally stopped at the tips of the jetties. The interaction of the waves and the ebb current causes the waves to slow down, steepen and break. This phenomenon, known locally as the "chop," is hazardous to small boats and will be discussed in more detail in the inlet current section.

Wind-generated waves from within the inlet were observed during the study to be a major cause of bank erosion on the south side of the inlet. A strong northeast storm can generate waves over two feet high within the inlet, which, when coupled with a storm tide, can breach the earthen embankment between the inlet and road. Boats also generate wakes which break on the channel banks causing some erosion. However, little erosion appears to occur on the south side during normal tides.

The strong tidal currents are also a major cause of bank erosion. A deep channel is located immediately adjacent to the portion of the north inlet bank currently experiencing heavy erosion. The sand channel bank drops to a depth of about 15 feet, 20 feet from the shore. This steep slope permits easy erosion of material by strong currents close to shore.

As a result of the erosion, the channel dimensions have increased considerably since the inlet's stabilization. Figure 35 shows the changes in the inlet's cross section at 5 stations from 1936 until the present. The cross-sectional area versus station of the inlet for various years is shown in Figure 36. There has been a 15-fold increase in average cross-sectional area since 1938! Figure 37 shows the mean cross-sectional depth versus station for various years. The increase in average cross-sectional area for the entire channel (station 5+50I to station 60+00I) is shown in Figure 38. Likewise, the increase in average width and depth are shown in Figures 39 and 40, respectively.

The sediment eroded from the channel banks is primarily sand. The silt and clay size particles introduced into the inlet from eroding marsh mud, as seen in Figure 34, are carried out to the ocean or into the bay and then deposited in calmer waters. Some loss of material from the study area results, producing an error in the calculated sand budget (to be discussed later). However, since the proportion of silt and clay size sediments is small, it will be neglected.

The volume of sediment eroded from the channel banks and bottom was determined from historical charts. The vertical datum for each chart was related to a common fixed vertical datum (Section 3.3). The volume computation was complicated by a lack of survey data on the land surrounding the inlet. Also, some of the sand from the early

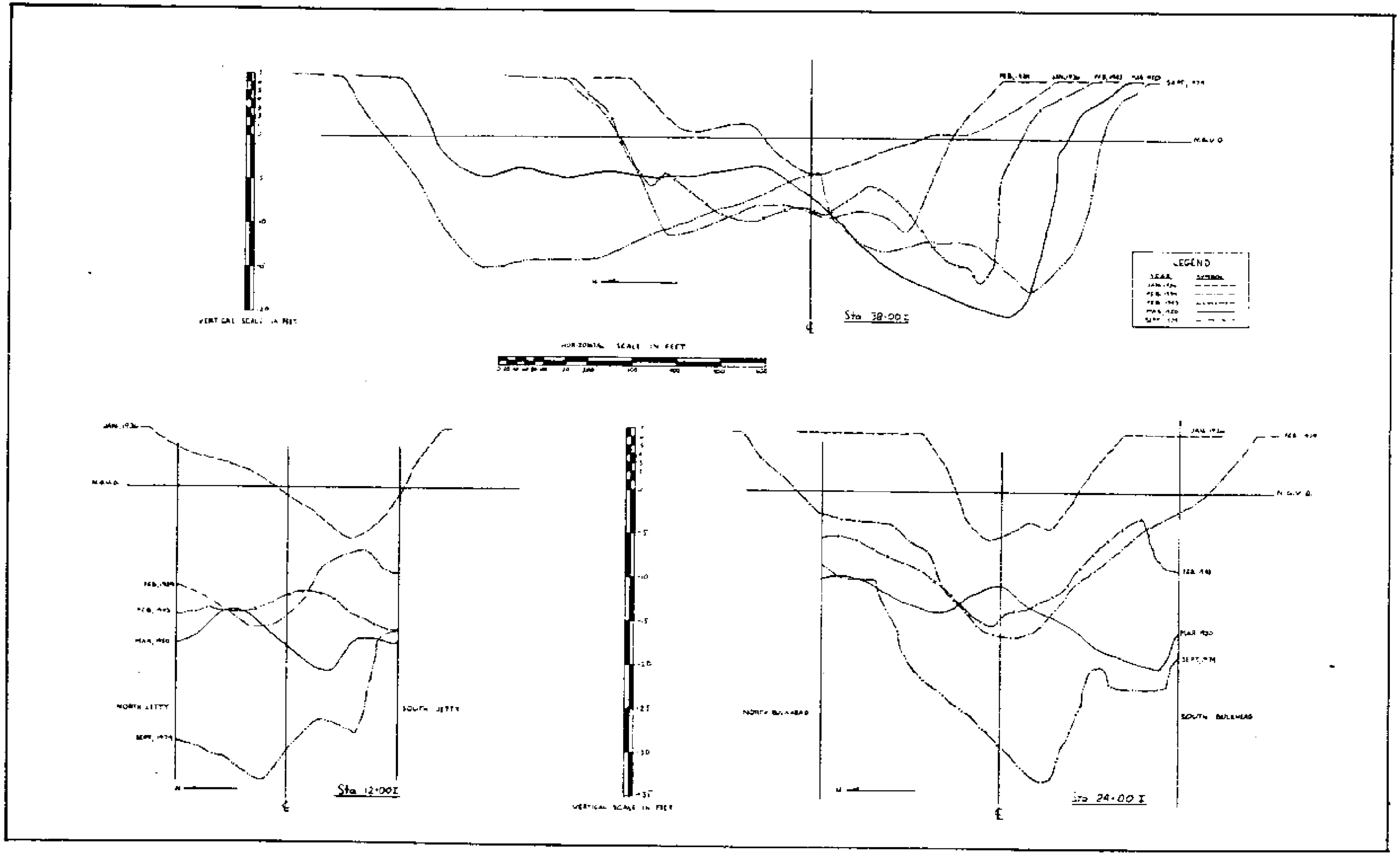


Figure 35a Changes in Channel Cross Section at Stations 12+00I, 24+00I and 38+00I

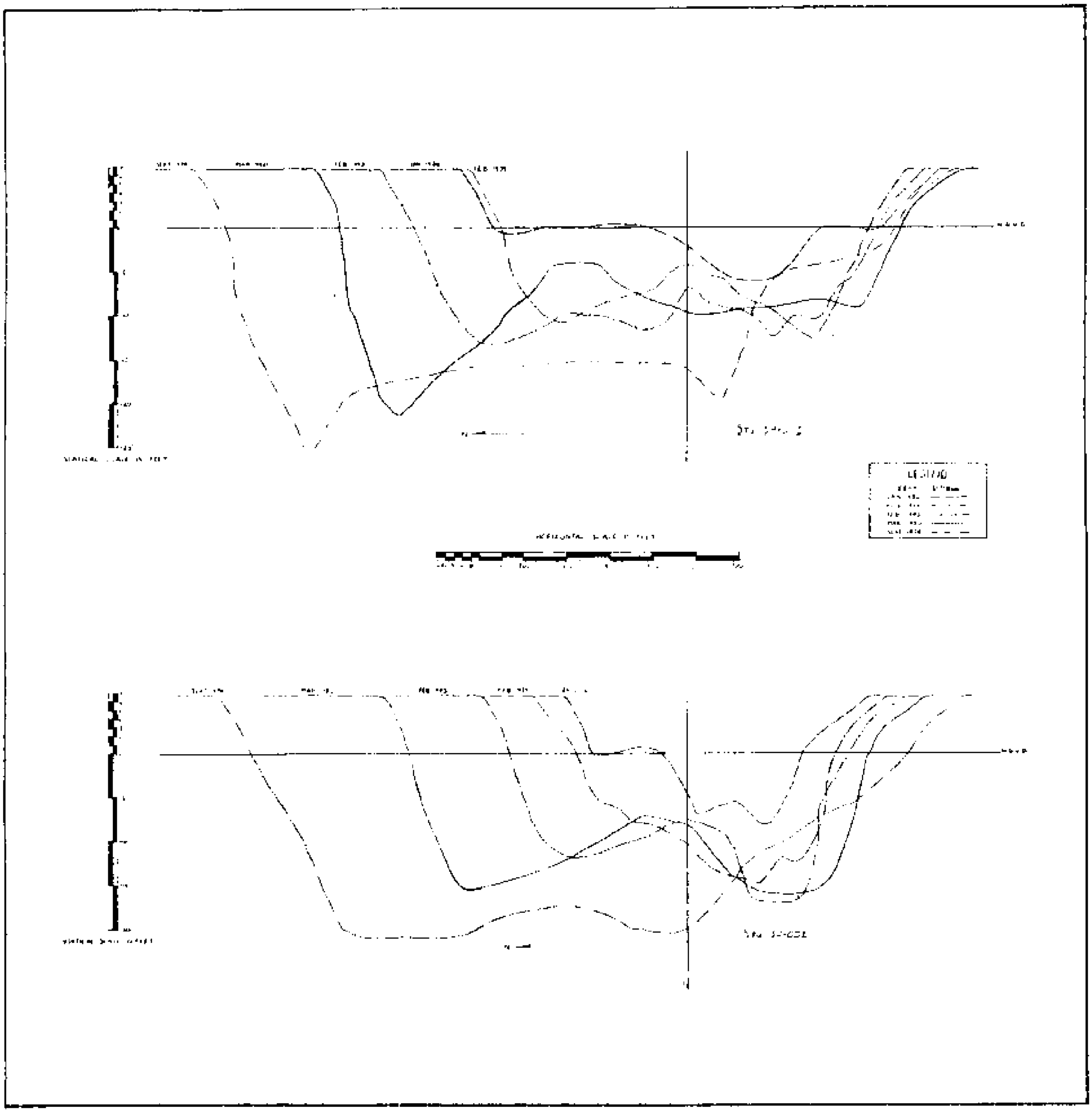


Figure 35b Changes in Channel Cross Section at Stations 50+00I and 58+00I

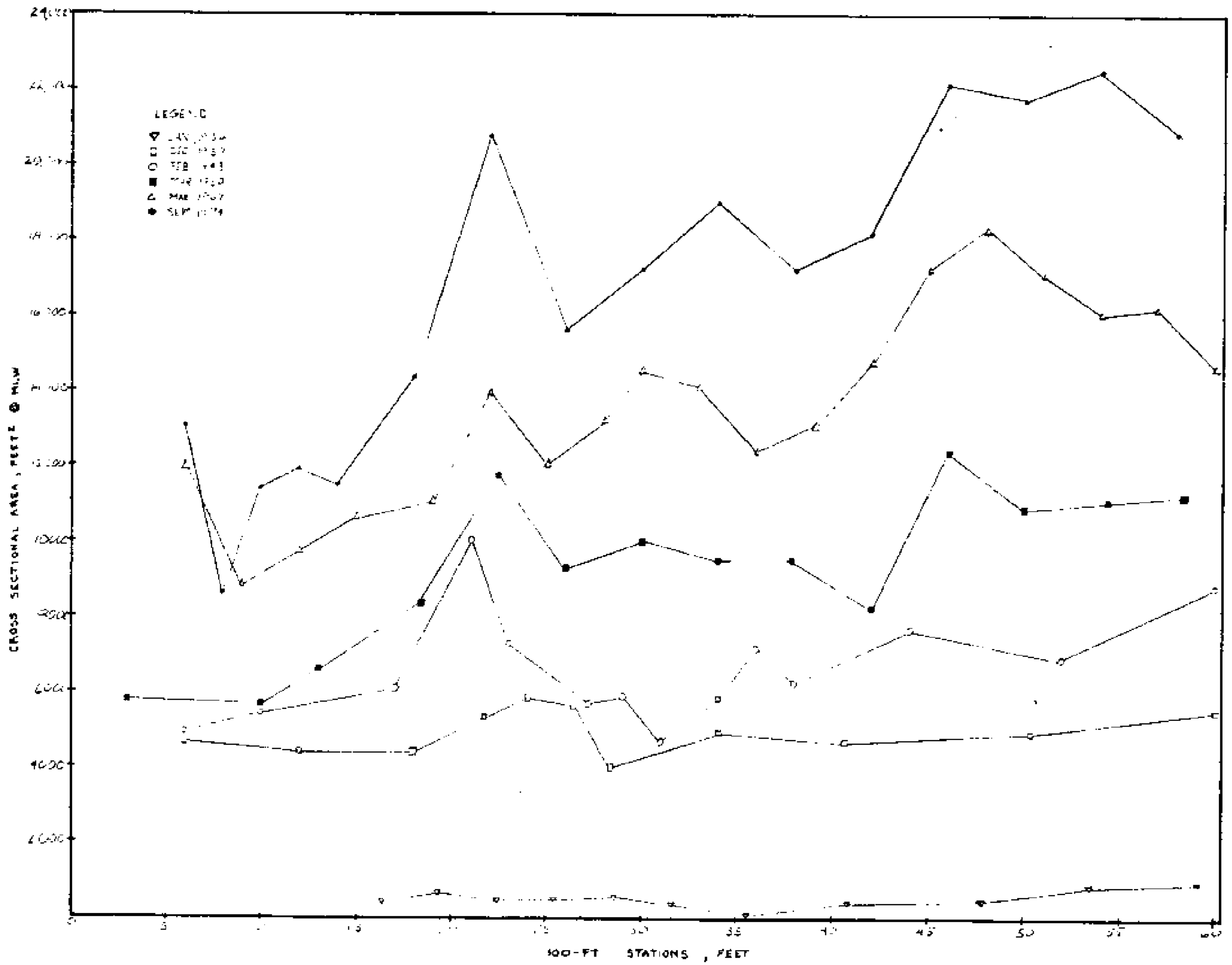


Figure 36 Channel Cross Sectional Area at MLW Versus Inlet Station for Various Years

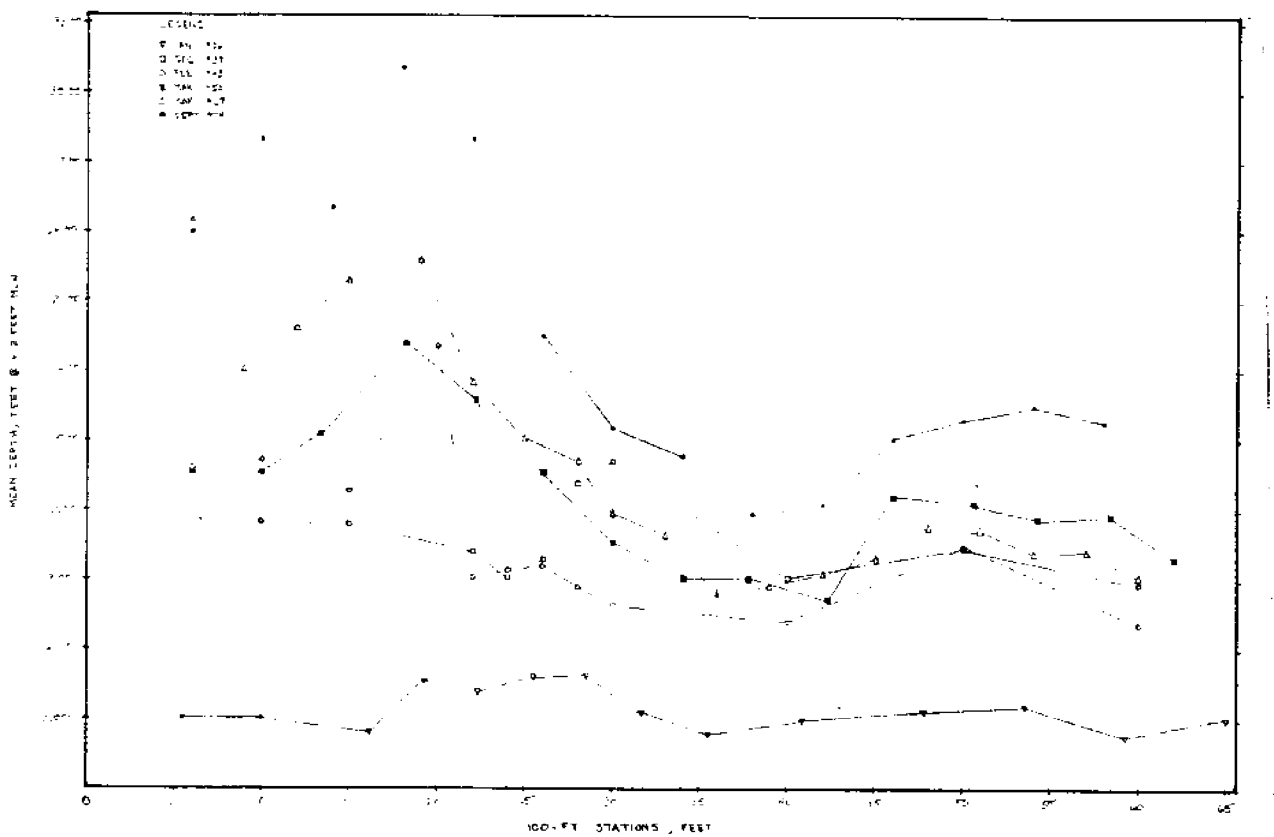


Figure 37 Channel Depth at +2 Feet MLW Versus Inlet Station for Various Years

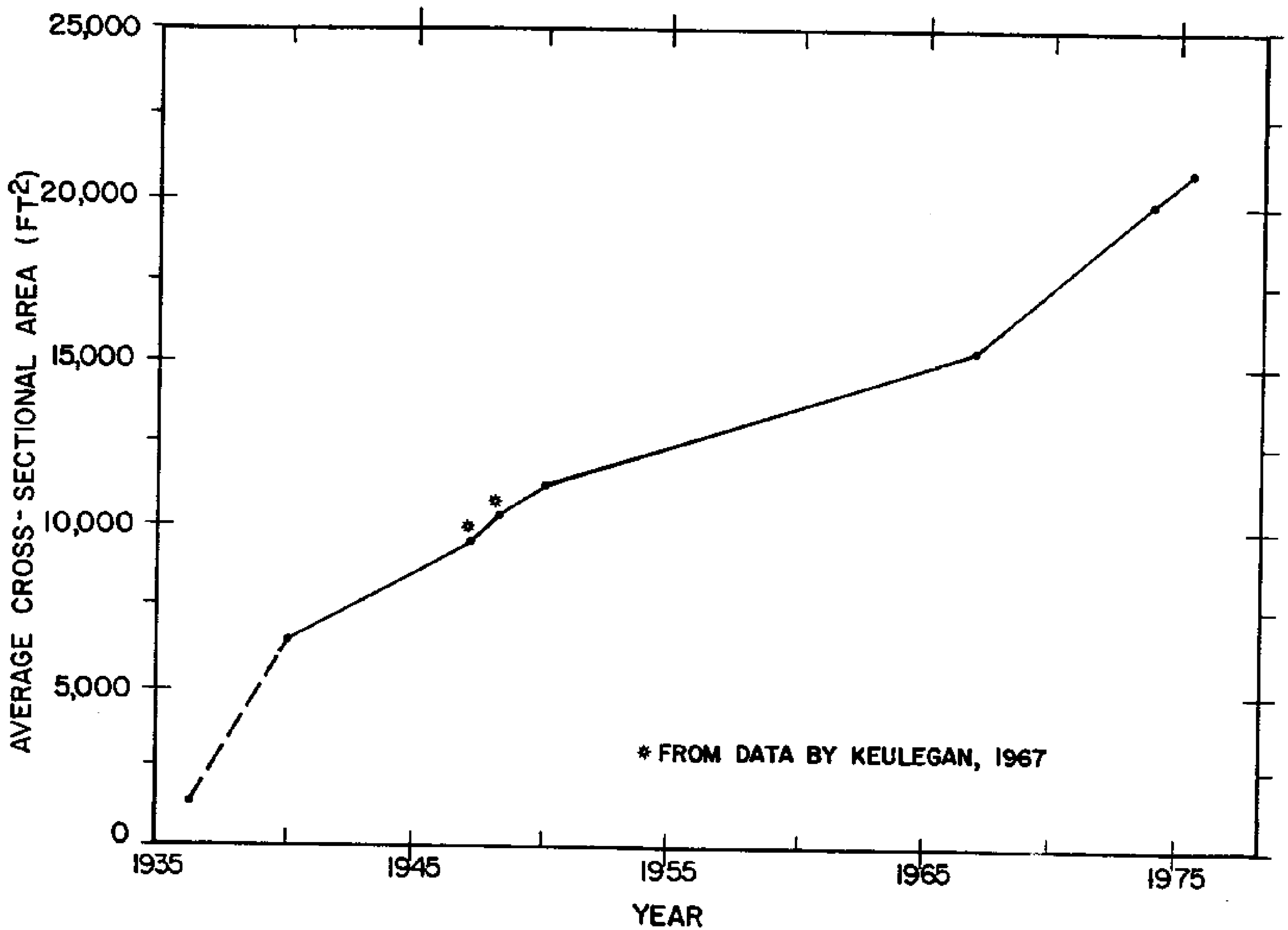


Figure 38 Average Channel Cross Sectional Area at +2 Feet MLW Versus Year

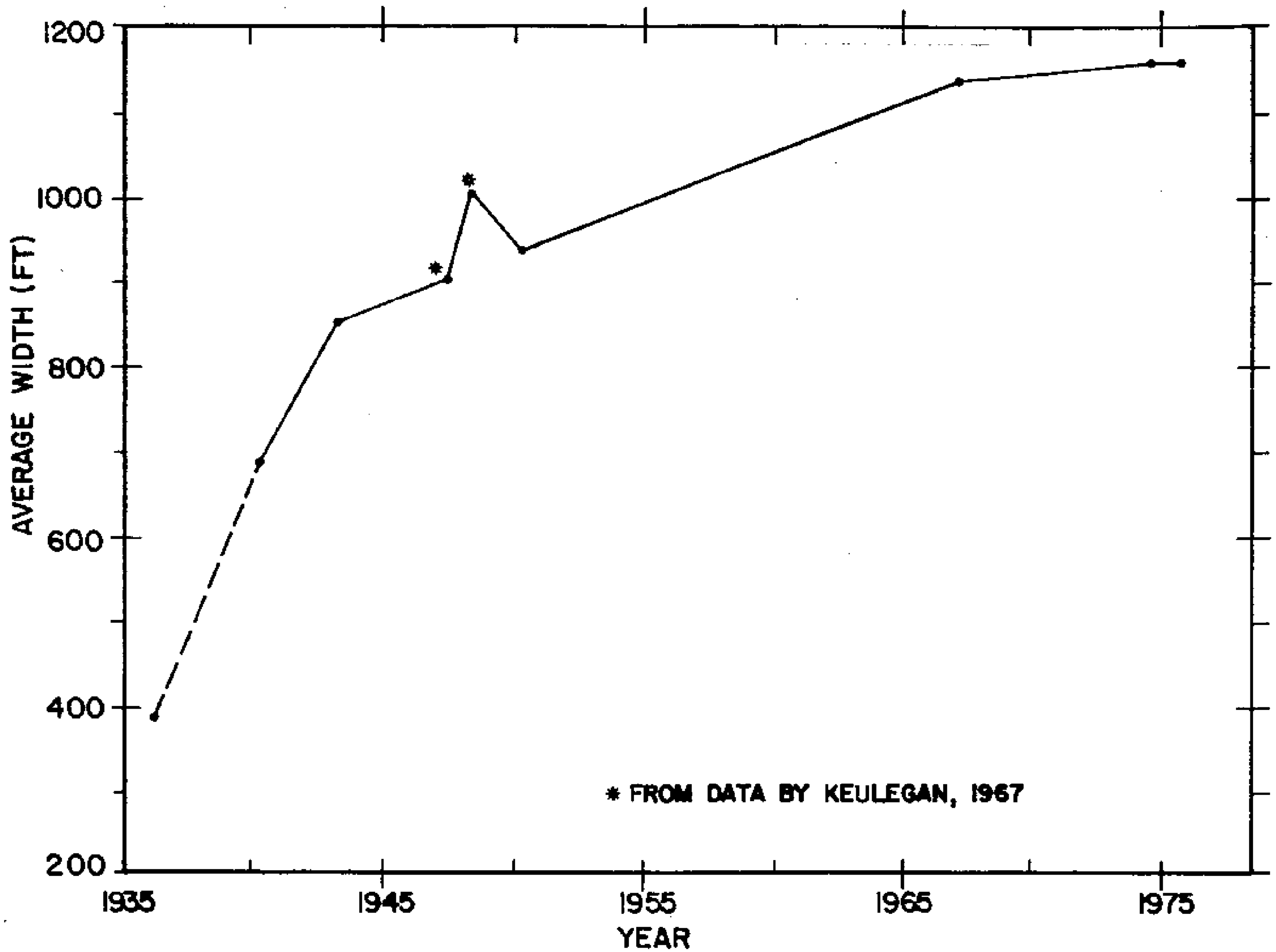


Figure 39 Average Channel Width at +2 Feet MLW Versus Year

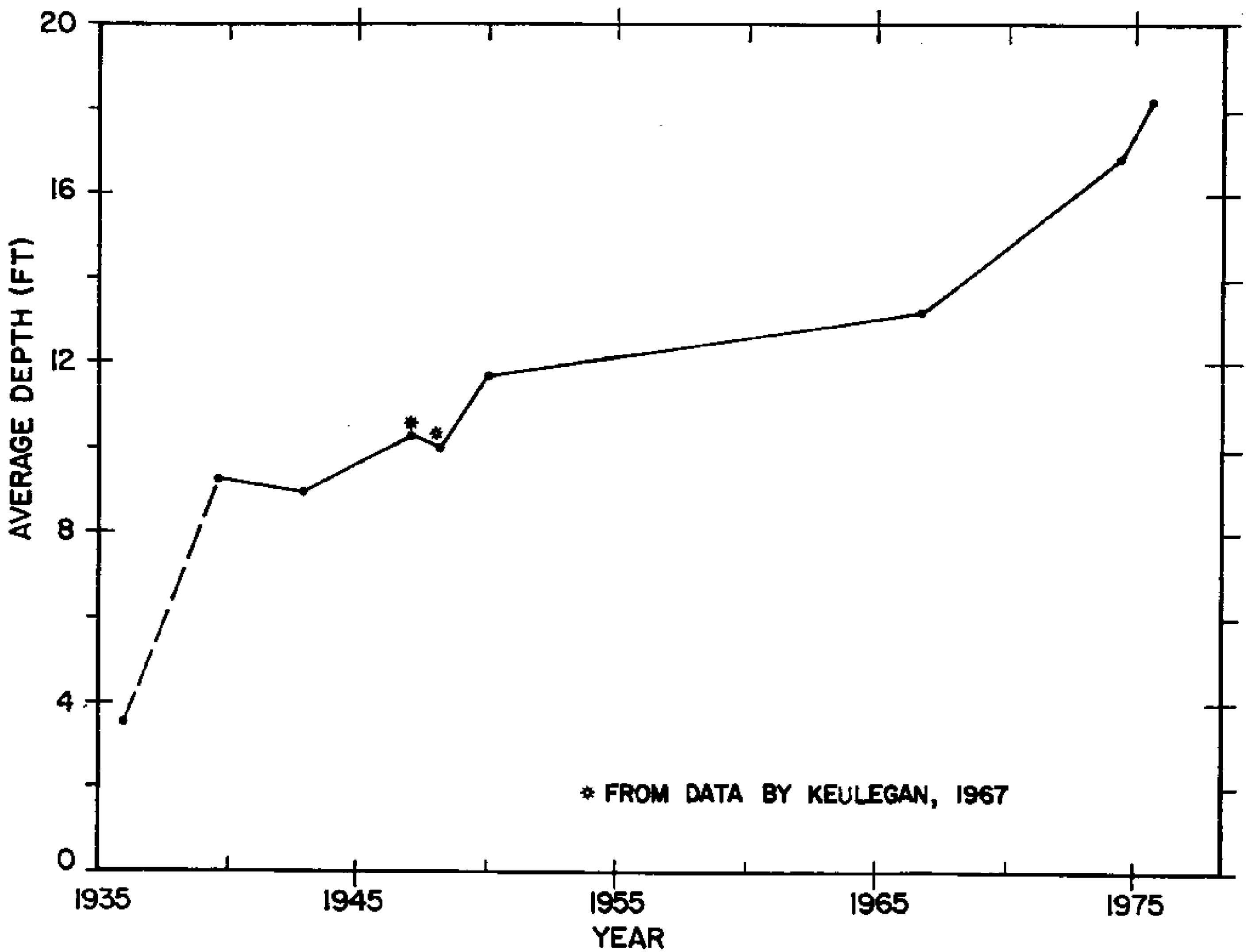


Figure 40 Average Channel Depth at +2 Feet MLW Versus Year

dredging operations was spoiled on channel banks which have subsequently been eroded, as mentioned earlier.

Erosion volumes were calculated based on the 1936 chart with a correction applied to account for the 1938 dredging operation. During this operation, 1,198,409 c.y. were removed and spoiled on the channel banks. The land elevation after 1938 was assumed to be a constant 6.7 feet above NGVD. The net erosion from the channel banks and bottom between station 5+50I and 60+00I is shown in Figure 41. The volumes indicated in Figure 41 are net sediment volume changes and include the effects of dredging. The various dredging operations from within the inlet channel will be discussed in the section on dredging history.

In addition to the general deepening of the inlet, several deep scour holes have formed on the inlet bottom. These holes appear in areas of flow constriction, where increased water velocities and turbulence erode bottom material and inhibit deposition. The deepest of these scour holes is located just off the southwest corner of the Coast Guard Station. Ebb tidal flow around this sharp contraction (expansion for flood tide) has scoured a hole as deep as 50 feet. The maximum depth and shape of this hole varies from survey to survey. There is also a distinct scour hole just off the corner of the south bulkhead at station 24+00I. Lesser scour holes have been found at the other corners in the bulkheads and in the vicinity of the bridge piers.

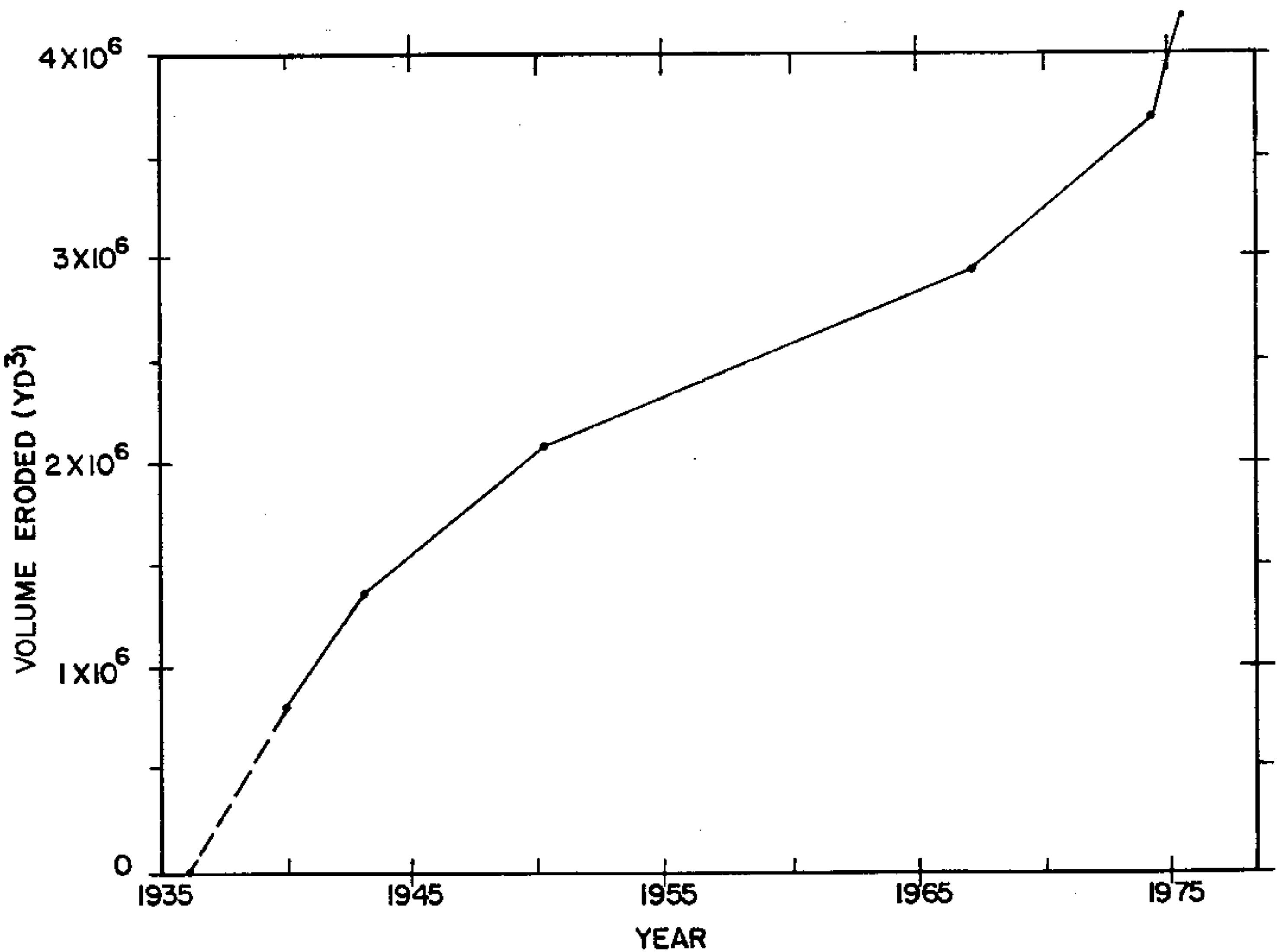


Figure 41 Volume of Erosion from Inlet Channel, Station 5+50I to 60+00I, Versus Year

A deep scour hole is also located between the tips of the jetties. Proceeding seaward from the inlet, the bottom drops sharply from about 20 or 23 feet deep to about 55 feet deep just seaward of the jetty tips. The depth is generally greater near the north jetty, perhaps due to the sand introduced over and around the south jetty. The scour hole at the tips of the jetties could be caused by the increased current velocity and turbulence associated with the flow contraction or expansion as the tidal flow enters and leaves the inlet. The high turbulence induced by the chop could also be causing the scour. In any event, it is common for an inlet to have a deep channel extending straight out from the jetties as will be discussed in the section on the outer shoal.

This deep hole off the north jetty is probably the cause of the present deterioration of the north jetty. The jetties were not designed for water of the present depth; therefore, the rip-rap would be expected to settle into the deepening hole. This would explain the present submerging of the tip of the north jetty. During this study, the light tower on the north jetty failed due to this subsidence. Figure 42 is a photograph taken in September, 1975 of a boat heading out the inlet through the chop. At this time, the north tower had just recently taken on a seawart tilt. Figure 43 was taken November 7, 1975 about two weeks after the tower fell.

The subsidence of the jetties and bulkheads into the deepening channel has made the numerous rehabilitation projects necessary. As

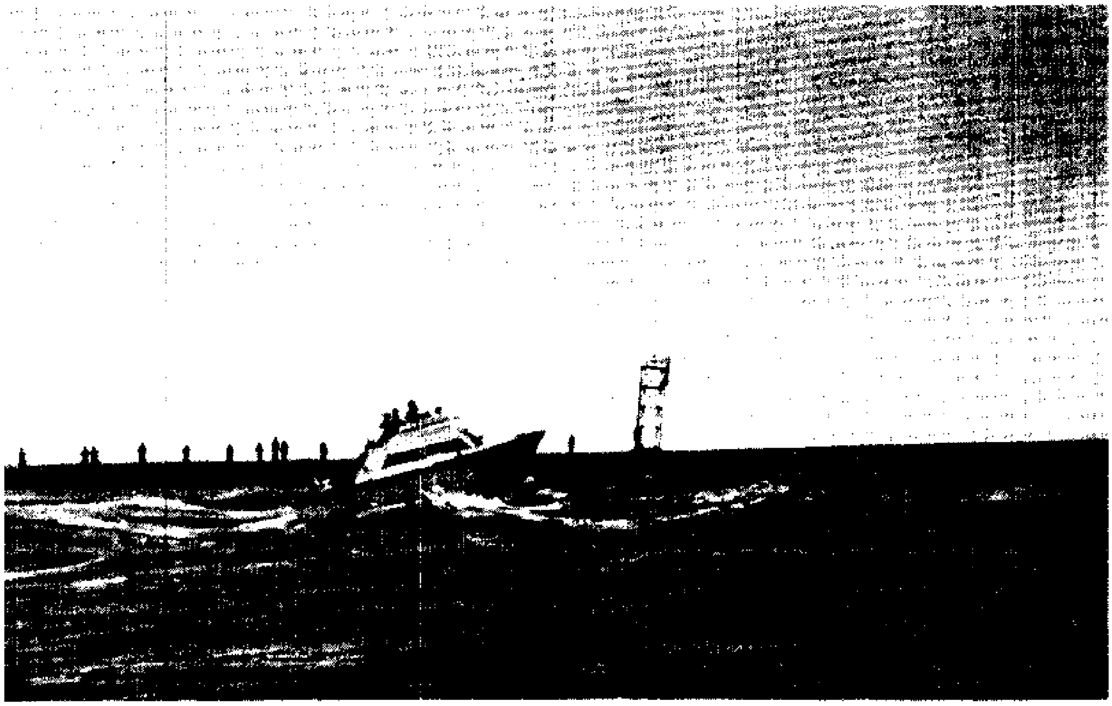


Figure 42 Boat Proceeding Out of the Inlet Through Chop. Note the Seaward Tilt of the Light Tower on the North Jetty.

Figure 43 Deterioration of Tip of North Jetty and Fallen Light Tower

previously mentioned, the original stone jetties only extended to a depth of 3 feet below MLW over the majority of their length. The original steel sheet-pile extended to a maximum depth of 18 feet below MLW. The original bulkhead and jetty design should be reevaluated in consideration of the increasing water depths. Perhaps future designs should allow for further increases in depth (see Section 5.3), particularly in areas where flow constrictions could cause deep scour holes such as the one near the Coast Guard Station.

3.8 Sediment Movement Inside the Inlet

A sand tracer study was conducted to determine the sediment movement patterns inside the inlet. Sand from the ocean beach south of the inlet was used to make the tracer sand in order to obtain a size distribution similar to that of the natural sand. The casein glue used to adhere a fluorescent pigment to the sand was non-fat dry milk. Powdered milk was chosen because it provides an adequate bond for a study lasting several days. However, it will not remain for an extended period as various resins will. Such long-lasting sand tracers could contaminate any future studies. A complete description of the sand tracer and its manufacture is given in Appendix 1.

Five hundred pounds of the red sand was dumped from the south jetty into the inlet at station 8+58I (opposite the berm crest of the south beach where sand passes over the jetty). The red tracer sand was dumped at low water slack (as shown in Figure 44) in order to allow the major portion of the sand to be carried into the inlet by flood tide.

Figure 44 Deployment of Red Tracer Sand in Channel

Bottom surface samples were then taken with a small dredge at various times after the dump. The samples were analyzed under ultra-violet light and the fluorescent sand grains were then counted.

Since the sampling program was not of sufficient density to construct concentration contours of the sand tracer, the analysis of the sediment movement patterns had to be more or less qualitative. The number of samples required to completely analyze the distribution of the sand tracer over the entire inlet after several time intervals would have been quite large. The analysis of these samples would have also taken considerable time.

A 100-gram portion of each sample was removed from the sand sample, and the number of fluorescent sand grains counted. For several selected samples with low tracer content, larger than 100 gram portions were examined. Similarly, for samples with very high tracer content, portions smaller than 100 grams were analyzed. In each case, the equivalent number of grains per 100 grams is presented. Selected samples were also sieved and the results will be discussed in a later section of this chapter.

To extend the information obtained from each sample, small particles of pigment abraided from the surface of the marked sand grains were also counted. The count of fluorescent sand grains and specks from sand samples taken from within the inlet channel is shown in Table 5. Figure 45 shows the location and tracer content of the sand samples collected on August 7, 1975, one day after the dump. Similarly, Figure 46 shows the sand tracer content of samples collected August 8 and August 9, 1975. In order to simplify Figures 45 and 46, one grain of sand tracer is considered equivalent to four specks. This

TABLE 5 CHANNEL BOTTOM SAND SAMPLE ANALYSIS

Sample No.	Time After Dump In Tidal Cycles	Red Sand Tracer Content		Green Sand Tracer* Content		Median M_d (mm)	Median $M_{d\phi}$ (ϕ)	St. Dev. σ_ϕ (ϕ)	Skewness α_ϕ (ϕ)
		Grains	Specks	Grains	Specks				
6	1.7	0	0	0	0	.45	1.15	.41	.20
7	1.7	0	0	0	0	4.77	-2.25	1.15	.07
8	1.7	0	0	0	0		G R A Y	C L A Y	
9	1.7	0	0	0	0	.61	.71	.53	.08
10	1.7	0	0	0	0	2.45	-1.29	2.20	.03
11	1.7	6	0	0	0	.49	1.03	.88	.20
12	1.7	2	5	0	0	.30	1.76	.92	.22
13	1.8	1	4	0	0	.53	.92	.72	.10
14	1.8	0	0	0	0	.71	.49	1.28	.28
15	1.8	0	0	0	0	.41	1.29	.39	-.01
16	1.8	0	0	0	0	.38	1.41	.40	-.01
17	1.8	0	1	0	0	.48	1.06	.55	.14
18	1.8	1	0	0	0	.23	2.10	.31	.02
19	1.8	0	0	0	0	.49	1.03	.68	-.14
20	1.9	1	1	0	0	.49	1.01	.89	.17
21	1.9	0	0	0	0	.69	.53	.86	.16
22	1.9	0	0	0	0	.43	1.22	.44	.13
23	1.9	0	1	0	0	.47	1.09	.63	.03
24	1.9	1	0	0	0	.38	1.42	.47	.06
101	3.6	0	0	0	0				
102	3.6	0	2	0	4				
103	3.6	0	0	0	0				
104	3.6	2	2	0	0				
105	3.6	1	0	1	0				
106	3.6	0	0	0	0				
107	3.6	1	1	0	0	.56	.84	.57	.11
108	3.6	2	0	0	0				
204	5.5	0	0	0	0				
205	5.5	0	0	0	0				
206	5.5	0	0	0	0				
207	5.5	0	0	0	0	.35	1.49	.48	-.09

* See page 114.

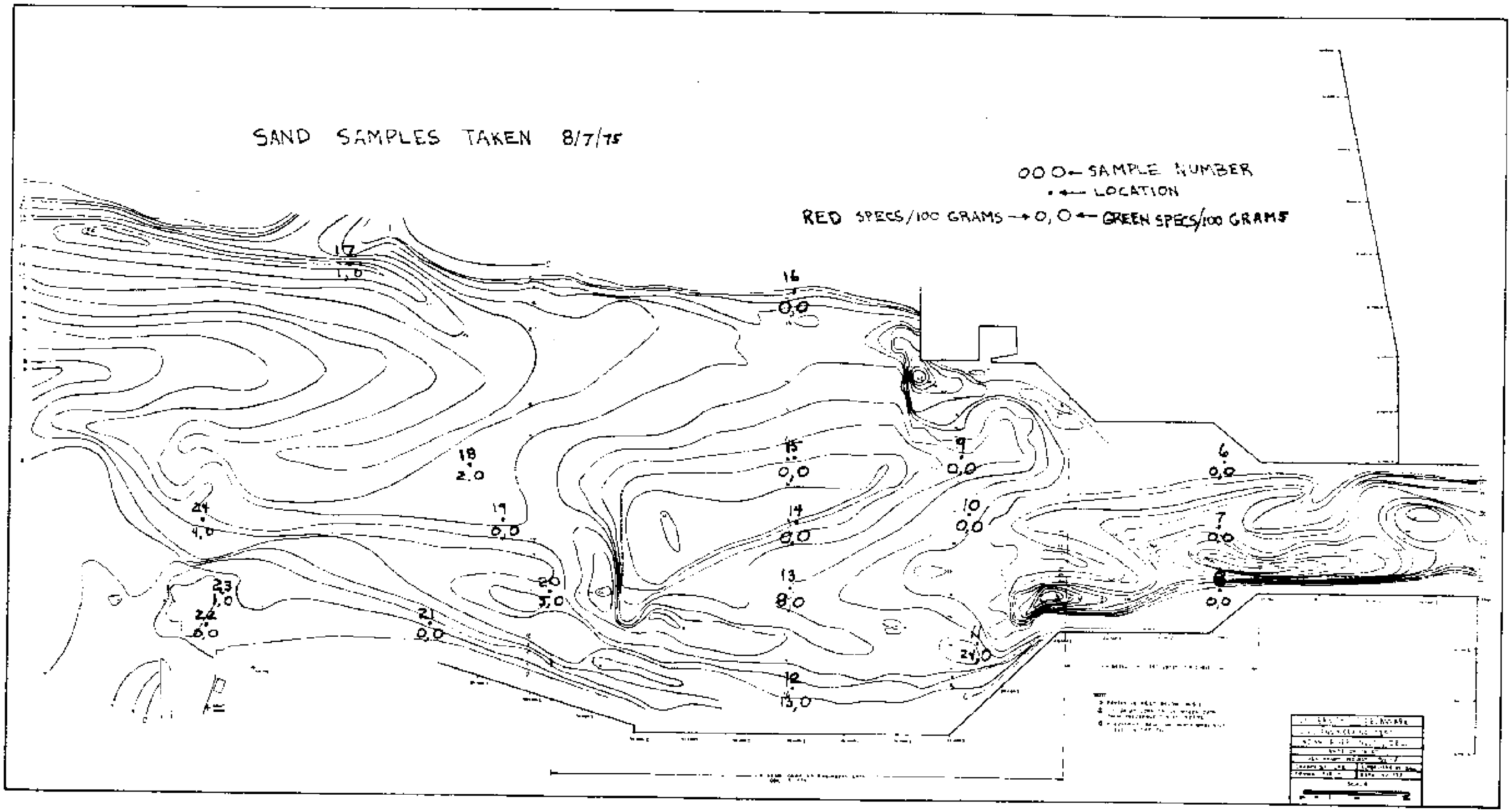


Figure 45 Sample Number, Location and Tracer Content of Sand Samples Taken in the Inlet Channel August 7, 1975

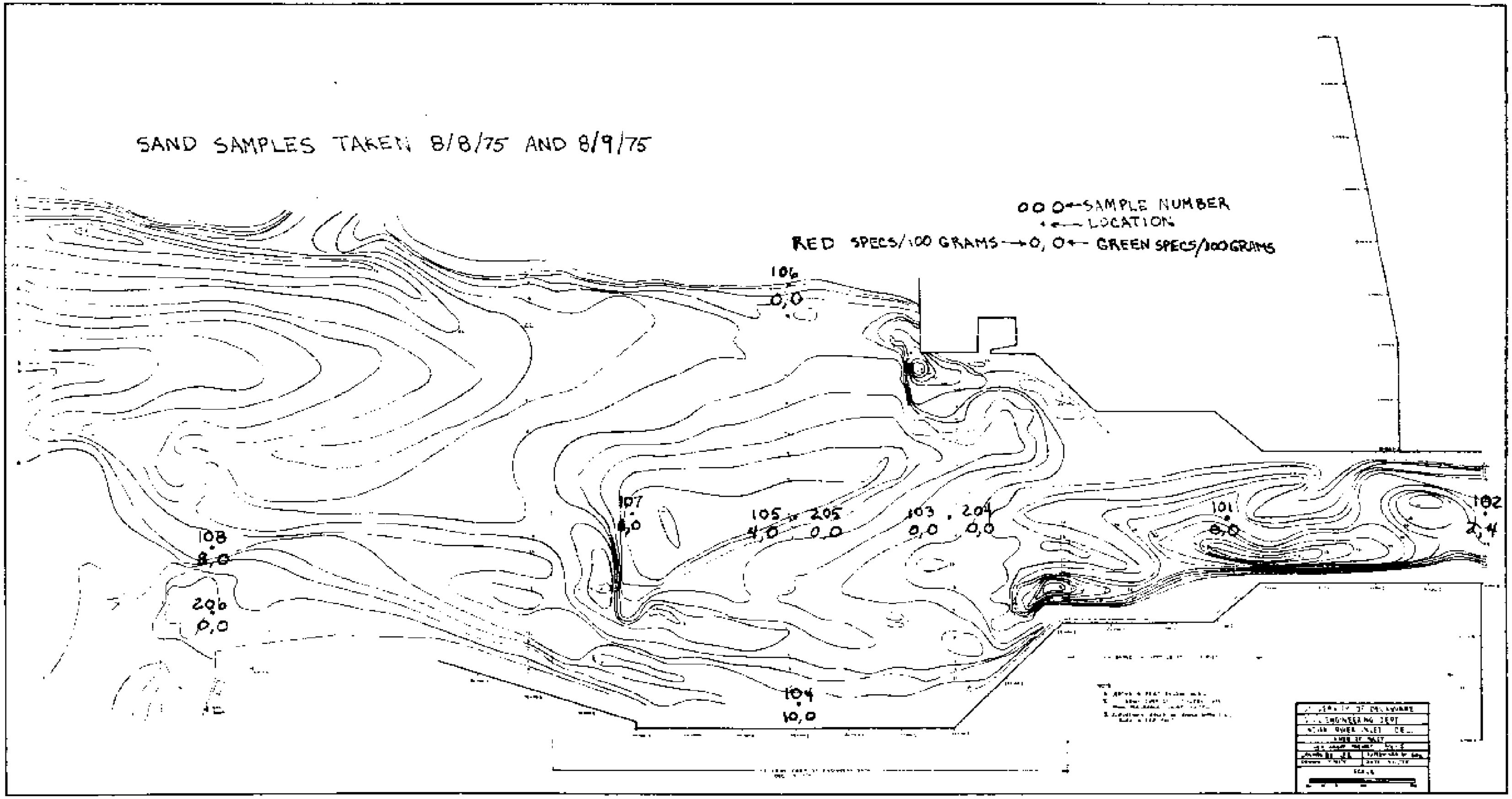


Figure 46 Sample Number, Location and Tracer Content of Sand Samples Taken in the Inlet Channel August 8, 1975

equivalent number of specks per 100-gram sample is then indicated on the figures.

Figure 45, indicating samples taken 1.7 to 1.9 tidal cycles after the dump of the red tracer, shows the red sand distributed primarily on the south side of the inlet. Tracer sand was found on the shoal near the South Shore Marina at approximately station 60+00I, a distance of almost one mile from the dump site. The tracer sand was not found in the deep, narrow portion of the inlet because high currents prohibited settlement. Also, the red sand tracer was not found on the large middle shoal at this time.

After two days, the sand had been redistributed within the channel. A relatively high concentration of red sand tracer was found on the middle shoal. No red tracer was found in a sample taken at about station 76+00I, on the shoal in the bay. Perhaps some tracer would have been found on the bay shoals with a more extensive sampling program. The tracer concentrations could be expected to decrease with time due to being spread over an increasingly larger area and being mixed and buried by the indigenous bottom sand.

Southwest of the Coast Guard Station, there is a large shoal (referred to in this report as the middle shoal). It is located along the centerline of the inlet,forcing the navigation channel to the north.(See Figure 21.) During this study, its minimum depth varied from 8 feet to 4 feet below NGVD, a hazard to even small craft during low tide. Various dredging operations conducted on this shoal will be discussed in Section 4.1.

There is also a large shoal on the south side of the inlet at about station 60+00I. This shoal has periodically threatened to block the entrance of the South Shore Marina. In April, 1975, the larger boats within the marina could not traverse this shoal during low tide. In June, 1975, the entrance of the marina was dredged by the marina owner. The high currents across the shoal rapidly filled the dredge cut. When surveyed in November, 1975, however, the shoal had reduced in size and the condition of the marina entrance was greatly improved.

The strong tidal currents move large amounts of sand around within the inlet. The shoals were seen to change significantly between surveys. This redistribution of sand in the channel and in the flood tidal shoal will probably continue despite dredging. A safe navigation channel could probably be most economically maintained by moving channel markers in accordance with the shifting of the channel.

3.9 Bay Shoal

Before the stabilization of Indian River Inlet, the eastern portion of Indian River Bay was shallow and had a flat bottom. Since that time, a large flood tidal delta complex has developed in this portion of the bay. Figure 47 shows a chart of the inlet vicinity in 1935 - 1936, and the original channel dimensions. The datum used for this chart is about 1.4 feet below NGVD.

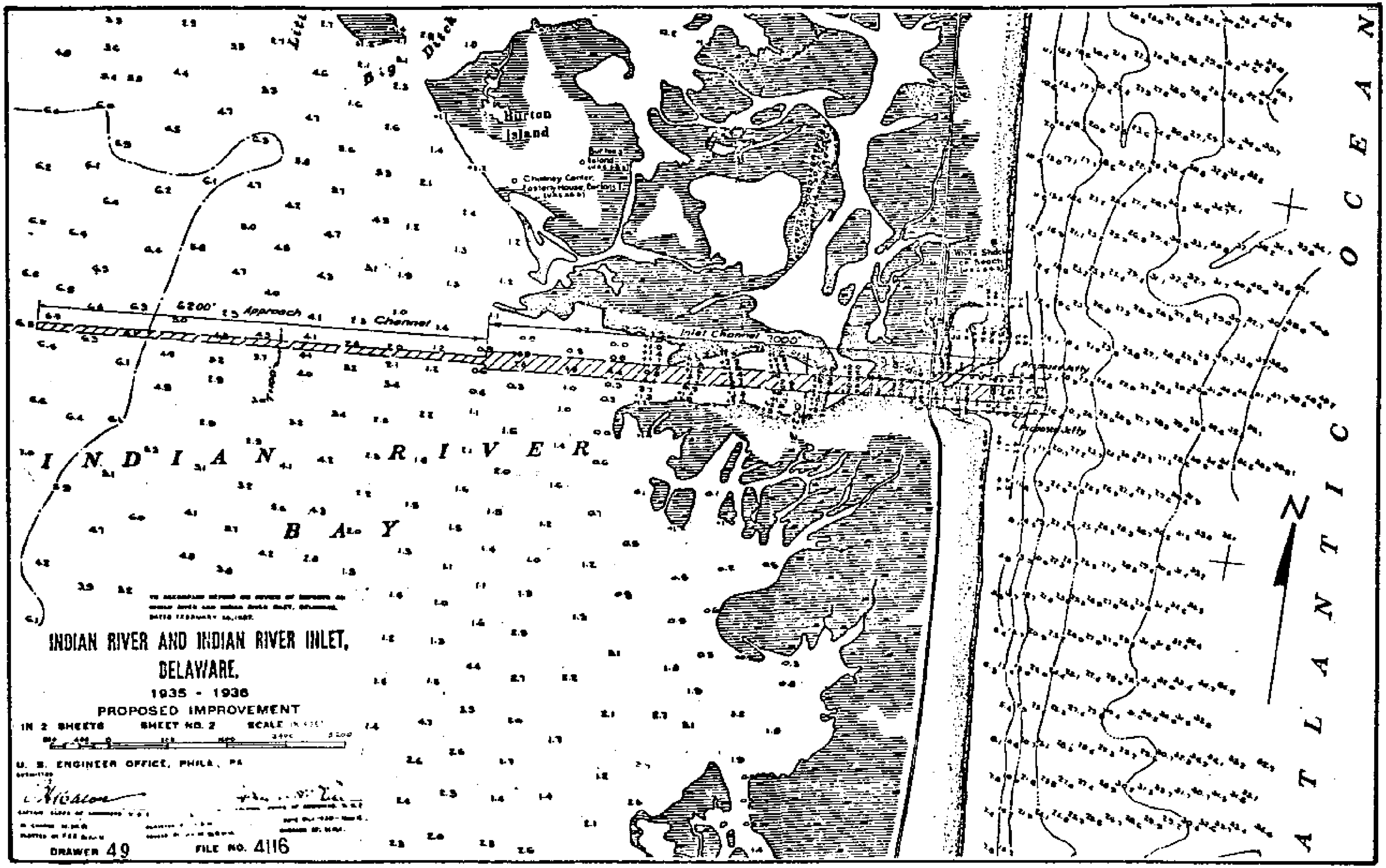


Figure 47 Indian River Inlet and Vicinity 1935-1936 With Original Channel Dimensions (After U.S. Army Corps of Engineers, 1975)

The complex shoal configuration can be seen in Figure 17. The navigation channel follows the northern channel into the bay which goes to the Ditches and westward. Two smaller channels turn to the south and southwest. The large shoal northeast of the crescent-shaped island shows bed forms characteristic of typical flood ramps (Boothroyd and Hubbard, 1975 and Hine, 1975). The smaller channels within this shoal are dispersed quite irregularly. The smaller shoal just west of Burton Island has many of the typical features of a flood tidal shoal. This smaller shoal shows a flood ramp, ebb shield and spill over lobes as shown in Hayes'(1975) model (Figure 48). Another similar shoal is seen south of the crescent-shaped island.

The configuration of the crescent-shaped island in Figure 17 is interesting to note. In 1954, this island, as shown on a topographic map, was nearly round. The sand making up the island was probably deposited as dredge spoils from the channel improvements completed in 1951. The steady change in configuration, as shown in Figure 49, indicates that it is being shaped by forces from the west. The forces shaping the island could be due to either the much greater fetch from the west than other directions, which allow the formation of larger waves from the west,or a predominance of ebb currents over flood in this vicinity.

There have been no recent surveys of the flood tidal shoal area. The complex bottom bathymetry would require an extensive survey to produce an accurate chart. The configuration of these shoals can be seen from aerial photographs to change significantly in a period of

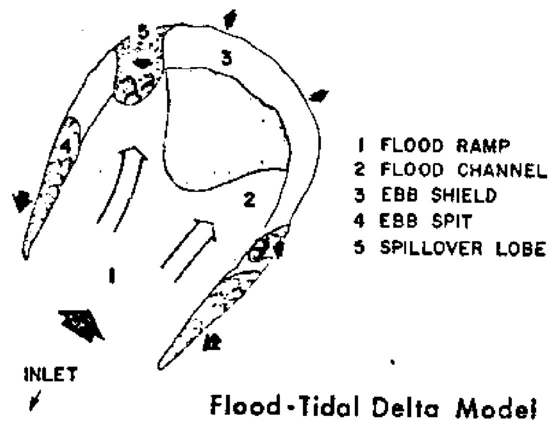


Figure 48 Flood Tidal Delta Model. Arrows Indicate Dominant Direction of Tidal Currents (Hayes, 1975).

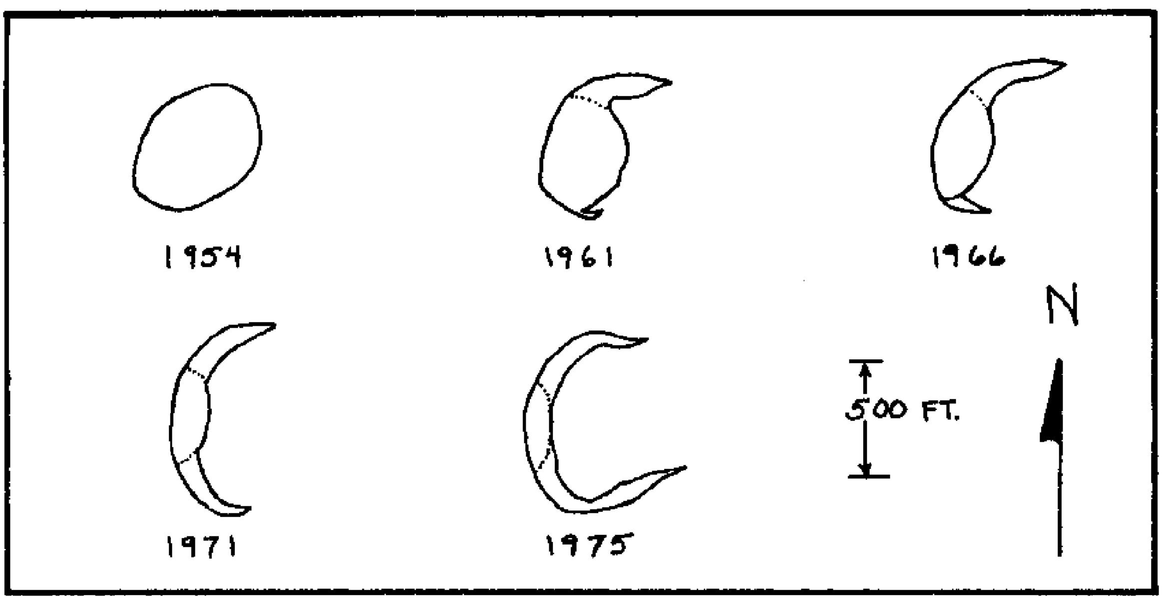


Figure 49 Changing Configuration of the Crescent-Shaped Island in Indian River Bay.

several years (see Figures 12 to 17). The shallow shoals prohibited the use of a small outboard motorboat to take soundings even during a spring high tide. Exploration on foot is also impractical due to the steep drop-offs into the numerous smaller channels.

The lack of an accurate recent survey of the bay shoal area made the calculation of a sediment volume impounded in this area very difficult. While presently some shoals are above water at low tide, it should be remembered that these areas were already quite shallow before the shoal developed. Thus, the depth of accretion is relatively small. The channels which have developed in the shoal area are over 25 feet deep in some places, with most about 10 feet deep, indicating significant local erosion.

An estimate of the volume of accretion in the flood tidal shoal westward of station 60+00I was made as follows. Areas of accretion and scour were determined from an aerial photograph. Present depths below NGVD were compared to those on the 1935-1936 chart (Figure 47), and a volume of accretion or erosion was computed for each area. The summation over the entire shoal produced an estimated net erosion of 1.1×10^6 c.y. The average error in the estimated accretion or erosion depth is believed to be less than 1 foot. However, even a 1-foot error when multiplied by the large area of the shoal could produce a volumetric error of about 1.0×10^6 c.y.

The shifting of the shoals making up the flood tidal delta creates problems in maintaining the navigation channel. Maintenance dredging of the channel through the bay shoal has been performed twice. From June to December 1968, 80,447 c.y. of material were dredged from the channel about 2.5 miles westward from the ocean at a cost of $67,429. This area was again dredged in July, 1970; 22,907 c.y. were removed and spoiled 1,000 feet south of the channel at a cost of $34,754. Maintenance dredging was again scheduled for 1974 but has been delayed due to objections by the State of Delaware and the U.S. Department of the Interior concerning dredge spoil disposal (Caccese, 1975). The dredging within the bay shoals will be discussed further in the dredging history section.

In November, 1975, the channel depths in the bay shoal area were considerably less than the project dimensions. Depths of as little as 3 feet below NGVD were found in the marked channel.

3.10 Outer Shoal

The bathymetry offshore of Indian River Inlet before the inlet was stabilized is shown in Figure 47. The offshore contours were relatively straight and parallel to the shoreline. Since 1938, however, a large shoal has formed offshore of the inlet.

Ebb tidal currents in the inlet transport sand to this outer shoal. The red sand tracer experiment discussed earlier was designed to follow the movement of sand onto the ebb tidal delta as well as inside the inlet. Sand passing the south jetty is entrained by the ebb tidal currents and carried onto the outer shoal. In addition, sand eroded from the channel banks and bottom is carried out to sea by the strong currents. As the currents decrease upon reaching open water, the sediment is deposited, forming the outer shoal.

The fluorescent red sand tracer dumped off the south jetty was found in all but one of the sand samples taken from the outer shoal the next day. The concentration of the red sand tracer declined in samples taken the following days. This was probably due to the tracer being spread over an increased area and mixing with the bottom sand. The data for the red sand tracer on the outer shoal is presented later in this section with the results of the green sand tracer study.

The ocean currents off Indian River Inlet are influenced by the tidal currents at the entrance of the Delaware Bay, 13 miles to the north. The ebb flow from Delaware Bay produces a general southerly flow along the Delaware coast which coincides with the ebb flow from Indian River Inlet. The plume of water issuing from the inlet is deflected southward by this current. (Further discussion of the ocean currents and the plume are presented in Sections 5.4 and 5.5, respectively). As a result of the deflection of the inlet water, the sand carried out of the inlet is deposited primarily to the south, causing the southern portion of the outer shoal to be much larger than the northern portion (as seen in Figure 24).

This influence of an estuary on an inlet appears to be unique. Factors such as proximity of a large estuary, being a stabilized inlet on a sandy coast, an ample supply of sand, and a location with approximately straight and parallel bottom contours before the establishment of the inlet, appear to cause this unique configuration of the outer shoal of Indian River Inlet.

The outer shoal consists of a north and south lobe separated by a channel. The north and south lobes are each in turn separated from the beach by a channel. The outer shoal has retained this same basic shape throughout its growth since about 1943. The north and south lobes are each shallowest near their outer edge. Between the surveys made in this study, the shallowest portion of the north lobe varied from 13 to 18 feet deep (below NGVD) and varied from 11 to 12 feet deep for the south lobe. The central channel also becomes progressively shallower as it progresses outward, ranging from about 55 feet deep near the tips of the jetties to about 22 feet deep near the outer edge. Except for the unique predominance of the southern portion of the outer shoal, its general shape is similar to the ebb tidal delta model in Figure 50 by Hayes (1975).

The channels separating the outer shoal from the beach on the north and south side of the inlet are about 20 feet deep. The

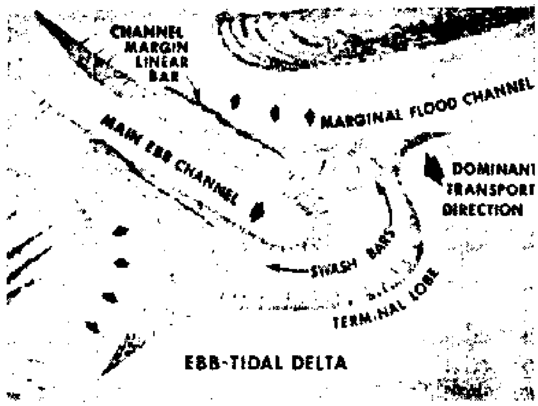


Figure 50 Ebb Tidal Delta Model. Arrows Indicate Dominant Direction of Tidal Currents (Hayes, 1975).

Figure 51 Waves Breaking on Outer Shoal May, 1964 (Delaware State Highway Department, 1975)

shape of the outer portion of the south shoal seems to suggest that sand is transported to the beach by waves from the southeast and east. Sand supplied to the south beach at this point could explain the presence of the bulge on the south beach. However, there is no shoal across the channel over which the sand can be readily transported by waves from the outer shoal to the beach. A similar situation is noted to the north of the inlet.

At present, waves break on the outer shoal only during periods of high waves. During the moderate northeast storm of December 1, 1974, waves broke on the outer portions of the north and south lobes and on the shallow tongue of the south lobe just southeast of the jetty tips. In the early 1960's, waves broke over the outer shoal much more frequently, as seen in Figure 51. This picture, taken in May, 1964, also shows the construction of the present highway bridge which replaced the old swing bridge. The water over the outer shoal was considerably shallower at that time, being only about 6 feet deep at the outer edges.

The March, 1962 storm had several effects on the outer shoal. According to Moody (1964), the outer edge of the shoal was moved seaward 30 meters (98 feet), and the depth of the outer edge increased from 1.8 meters (6 feet) to 2.7 meters (9 feet) below MLW. Figure 52 is a chart of the inlet as surveyed in April, 1962, one month after the storm. This is the only chart which shows a connection between

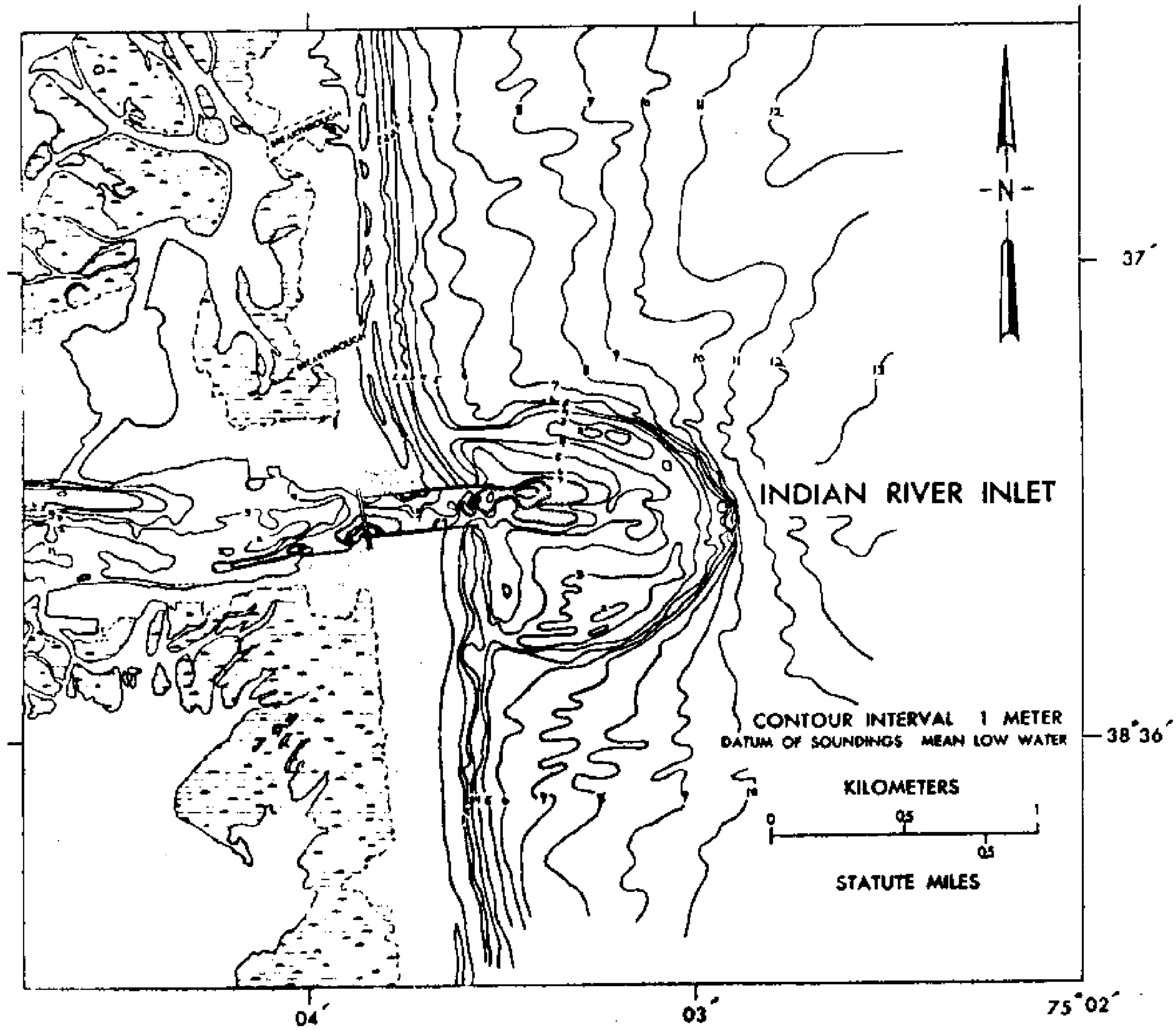


Figure 52 Indian River Inlet and Vicinity April, 1962. Note the Connection Between the Outer Shoal and the North Beach (Moody, 1964).

the outer shoal and the beach. A 3 to 4 meter (10 to 13 feet) deep shoal, not seen on a 1961 chart, connects the north and south lobes to the beach. The high waves associated with this storm presumably transported sand westward onto the beach. The volume of sand moved from the shoal to the beach by this storm is unknown.

A second sand tracer experiment was conducted to determine the sediment movement patterns on the outer shoal. Five hundred pounds of fluorescent green sand were placed on the western tip of the south lobe of the shoal. The sand was dumped during flood tide with a strong southerly breeze which produced a northward surface current of about 2 feet per second towards the inlet. Sand samples taken the following day showed the green tracer sand had been dispersed significantly. As shown in Figure 53 there was some westward movement indicated. The sample analyses are shown in Table 6. Samples taken two days after deployment and illustrated in Figure 54 showed low concentrations of green sand except on the section of the shoal extending towards the tips of the jetties.

The green sand placed on the outer shoal became dispersed very quickly and probably became buried. The green sand tracer experiment showed that the sand on the outer shoal moves about quickly. Long-term transport directions could not be determined from the data, however, as no green sand was found in any of the sand samples taken from the outer shoal three days after the initial placement (as shown in Figure 55). No red or green sand was found in any samples taken from the beach on any of the days sampled.

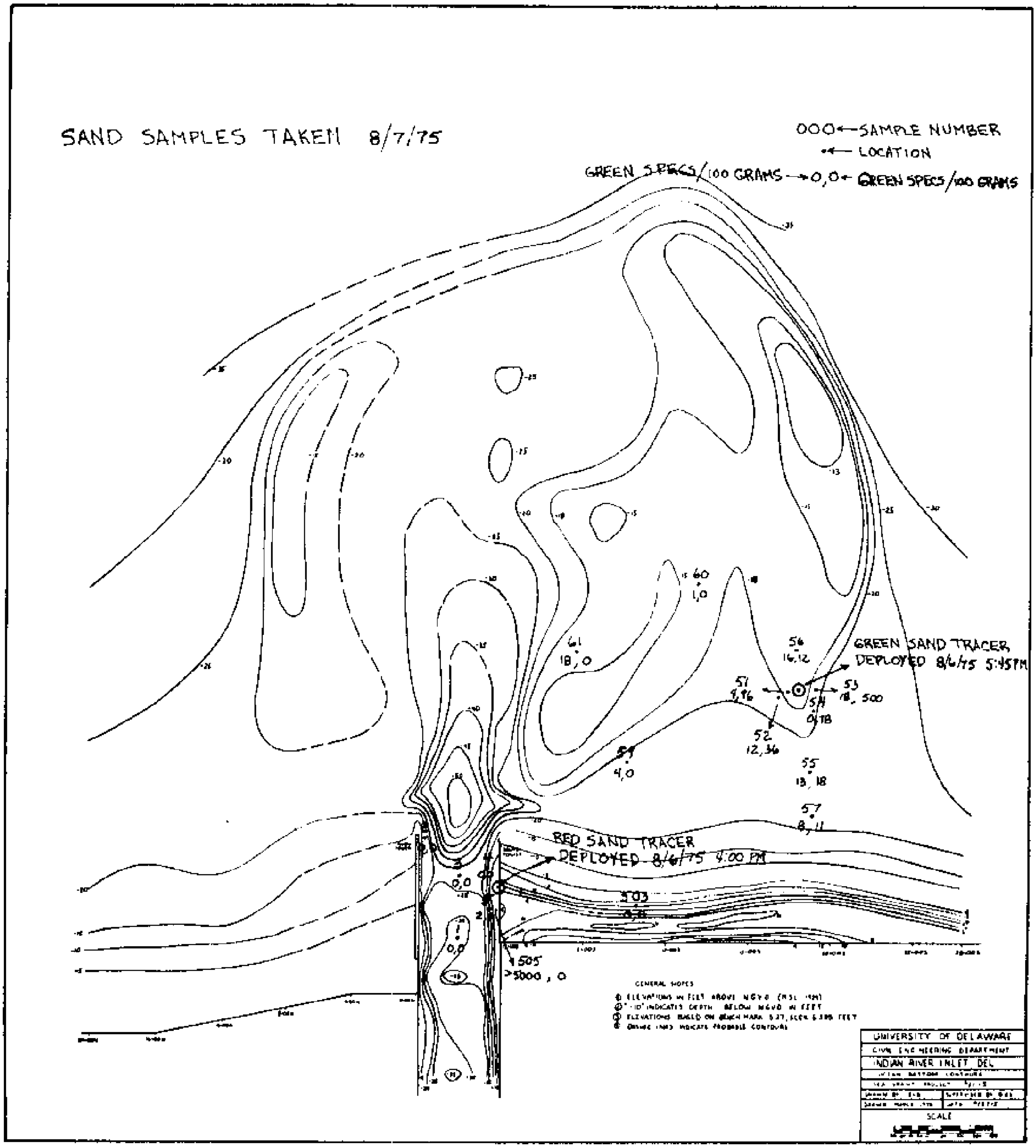


Figure 53 Sample Number, Location and Tracer Content of Sand Samples Taken on the Outer Shoal August 7, 1975

TABLE 6 OCEAN SAND SAMPLE ANALYSIS

Sample No.	Time After Dump in Tidal Cycles	Red Sand Tracer Content Grains	Red Sand Tracer Content Specks	Green Sand Tracer Content Grain	Green Sand Tracer Content Specks	Median M_d (mm)	Median $M_{d\phi}$ (ϕ)	St. Dev. σ_ϕ (ϕ)	Skewness α_ϕ (ϕ)
1	1.4	0 0	0	0	0	1.64	-.71	.65	.07
2	1.4	0	0	0	0	2.48	-1.31	.95	.03
3	1.4	0	0	0	0	1.88	-.91	.72	.09
4	1.4	0	0	0	0	1.08	-.11	.50	.29
5	1.4	0	2	0	0	.99	.01	.49	.15
51	1.7	1	0	12	0	.46	1.12	.43	.02
52	1.7	1	0	9	0				
53	1.7	4	2	>25	>100				
54	1.7	0	0	17	0	.44	1.20	.50	-.02
55	1.7	3	0	4	0				
56	1.7	4	0	3	0	.43	1.22	.52	.01
57	1.7	0	8	0	11	.43	1.23	.57	-.12
58	1.7	0	0	0	0				
59	1.7	0	4	0	0	.42	1.25	.43	.12
60	1.7	1	0	0	0	.45	1.13	1.13	.30
61	1.7	4	2	0	0	.47	1.10	.48	.04
102	3.6	0	2	0	4				
110	3.3	0	1	6	10	.43	1.24	.48	.03
111	3.3	0	1	0	3				
112	3.3	0	0	0	3				
113	3.3	0	0	5	22				
114	3.3	0	0	0	0				
201	5.4	0	1	0	0	.88	.18	.71	.01
202	5.4	1	0	0	0	.23	2.10	.45	.01
203	5.4	0	0	0	0	.45	1.15	.45	.01

(CONTINUED ON NEXT PAGE)

Table 6 (Continued)

Sample No.	Time After Dump in Tidal Cycles	Red Sand Tracer Content Grains	Red Sand Tracer Content Specks	Green Sand Tracer Content Grain	Green Sand Tracer Content Specks	Median M_d (mm)	Median $M_{d\phi}$ (ϕ)	St. Dev. σ_ϕ (ϕ)	Skewness α_ϕ (ϕ)
301	7.2	0	0	0	0	.40	1.34	1.24	.13
302	7.2	0	0	0	0				
303	7.2	0	0	0	0	.60	.75	1.31	-.11
304	7.2	0	0	0	0				
305	7.2	0	0	0	0	.30	1.74	.78	.32
306	7.2	0	0	0	0				
307	7.2	0	0	0	0				
308	7.2	0	0	0	0				
309	7.2	0	0	0	0				
310	7.2	0	0	0	0	.37	1.44	.92	.16
401	7.5	0	0	0	0				
402	7.5	0	0	0	0				
403	7.5	0	0	0	0	.35	1.54	.43	.05
404	7.5	0	0	0	0				
405	7.5	0	0	0	0	.31	1.69	.71	.11
501	7.5	>3,000		0	0				
502	7.5	20	0	0	0				
503	7.5	0	0	0	0				
504	7.5	0	0	0	0				
505	7.5	>400	>4,000	0	0				
509	7.5	0	0	0	0				

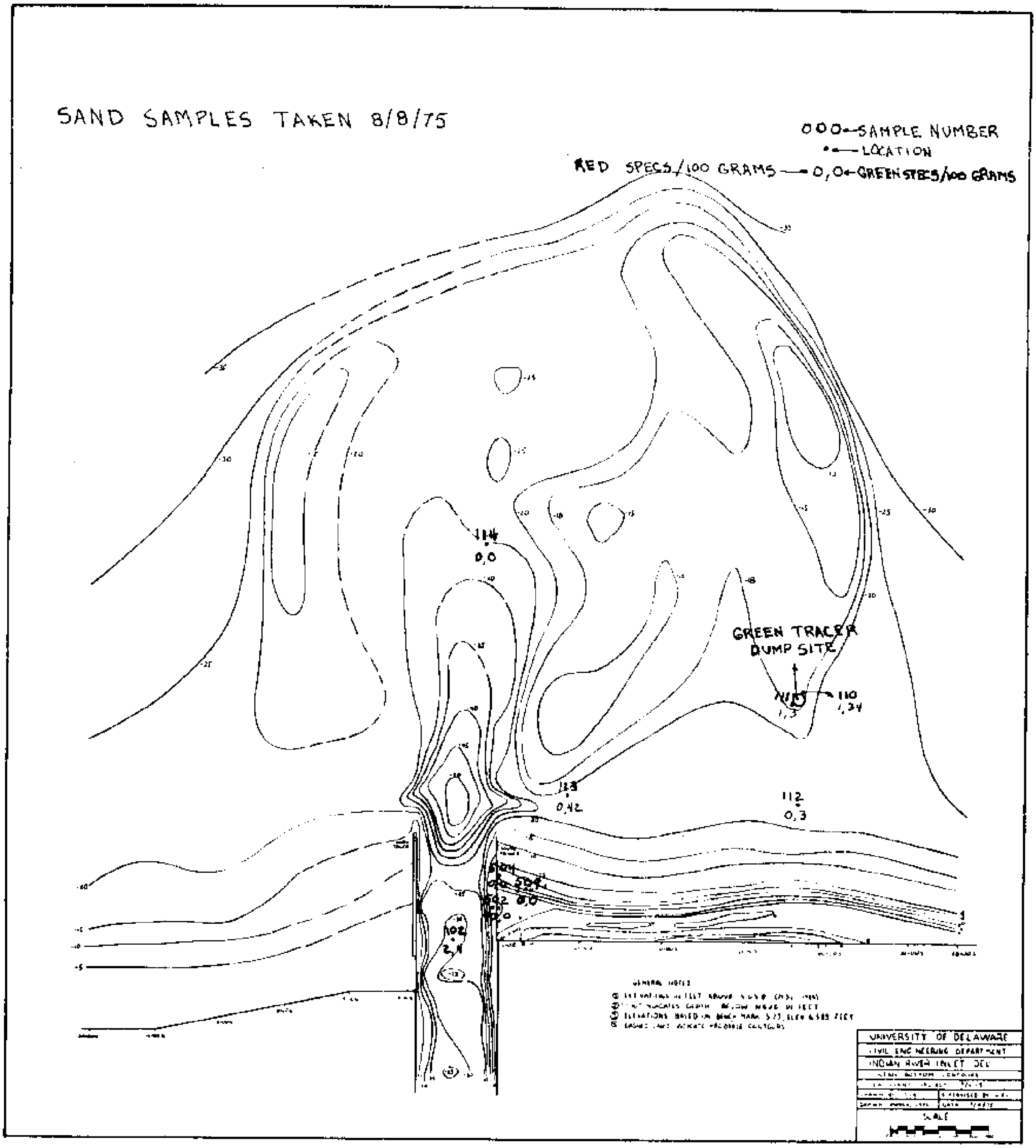


Figure 54 Sample Number, Location and Tracer Content of Sand Samples Taken on the Outer Shoal August 8, 1975

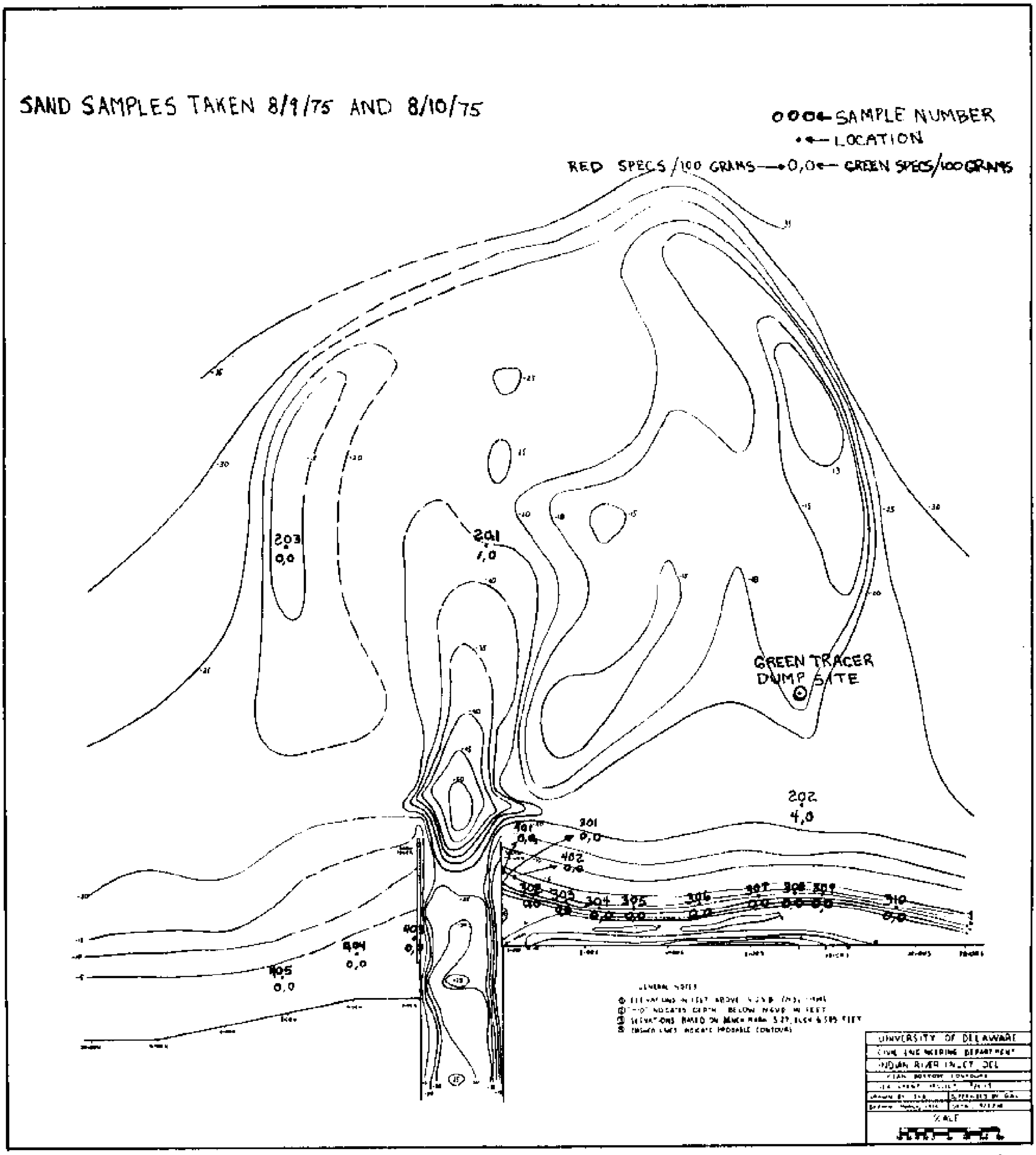


Figure 55 Sample Number, Location and Tracer Content of Sand Samples Taken on the Outer Shoal August 9 and 10, 1975

An important characteristic of the outer shoal is the volume of sand which has become impounded in it since the inlet's stabilization in 1938. The U.S. Army Corps of Engineers (1968) has estimated the volumetric change in the outer shoal of Indian River Inlet as shown in Table 7.

TABLE 7 VOLUMETRIC CHANGES IN OUTER BAR AREA OF INDIAN RIVER INLET (cubic yards)

Period	No. of Years	Net Volume Change Total	Annual
1939-1954	15	+ 1,605,000	+ 107,000
1954-1960	6	+ 57,000	+ 9,000 *
1960-1962	2	- 251,000	- 125,000 *

* Computations based on surveys covering a rectangular area (seaward of the ends of the jetties) having a width of 2,000 feet parallel to the shore, and a length of 2,800 feet normal to the shore.

The U.S. Army Corps of Engineers attributed the loss of sand between 1960 and 1962 to erosion caused by the March, 1962 storm (U.S. Army Corps of Engineers, 1968). The area considered (see footnote to Table 7), however, was too small. In 1962, the shoal extended about 3,300 feet parallel to the beach and about the same distance seaward. The storm caused the portion of the shoal surveyed by the Corps to deepen. The sand could have been moved seaward, as evidenced by Moody's observation of the seaward movement of the outer edge of the shoal, and not lost from the shoal.

Moody (1964) calculated the volume of the outer shoal as 3,433,500 c.y. in 1961. This corresponds to an annual rate of accretion of 156,000 c.y. per year. These values compare well with values determined in this study. Moody also estimated the increase in volume of the outer shoal as greater than 65,000 c.y. between 1961 and 1963, despite the 1962 storm.

In this study, the outer shoal was divided into different areas as shown in Figure 56. For the 1939 and 1943 charts, areas C and D were different than those shown,extending back to the original U.S. Army Corps of Engineers baselines on the north and south beaches. For the 1954 chart, areas A and B were extended slightly to include all of the shoal but exclude areas not surveyed on the chart. All original volumes for the standard and modified areas were determined from the 1936 chart. Changes in sediment volumes for each area for the charts examined are shown in Table 8.

The total growth of the outer shoal (seaward of the tips of the jetties) is shown in Figure 57. The growth rate is seen to be relatively constant at 132,000 c.y. per year. Dividing the shoal into a north and south portion and plotting the volume impounded versus year, Figures 58 and 59 result. Before 1943, while the outer shoal was still quite small, the north half of the shoal was slightly larger

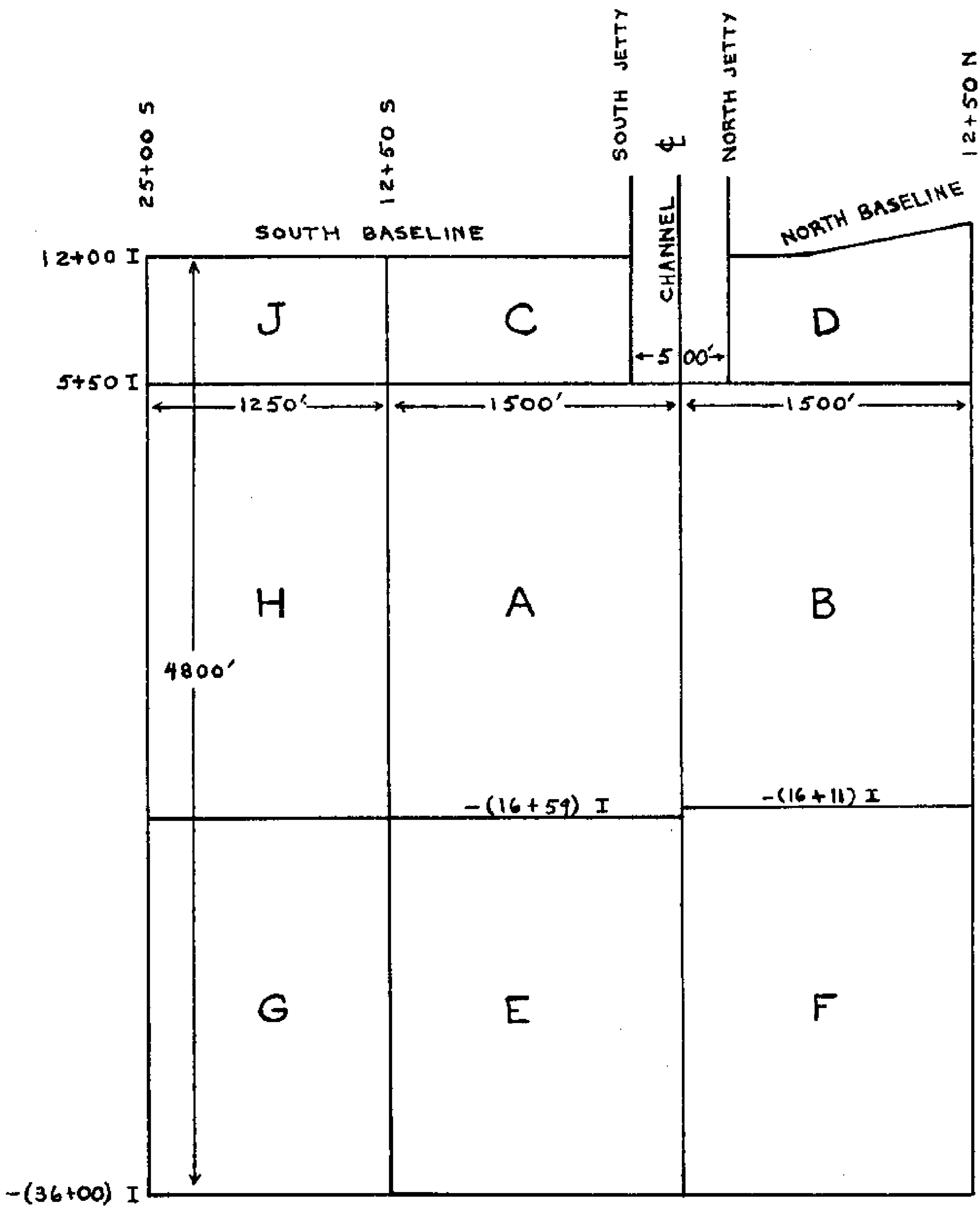


Figure 56 Division of Outer Shoal into Subareas

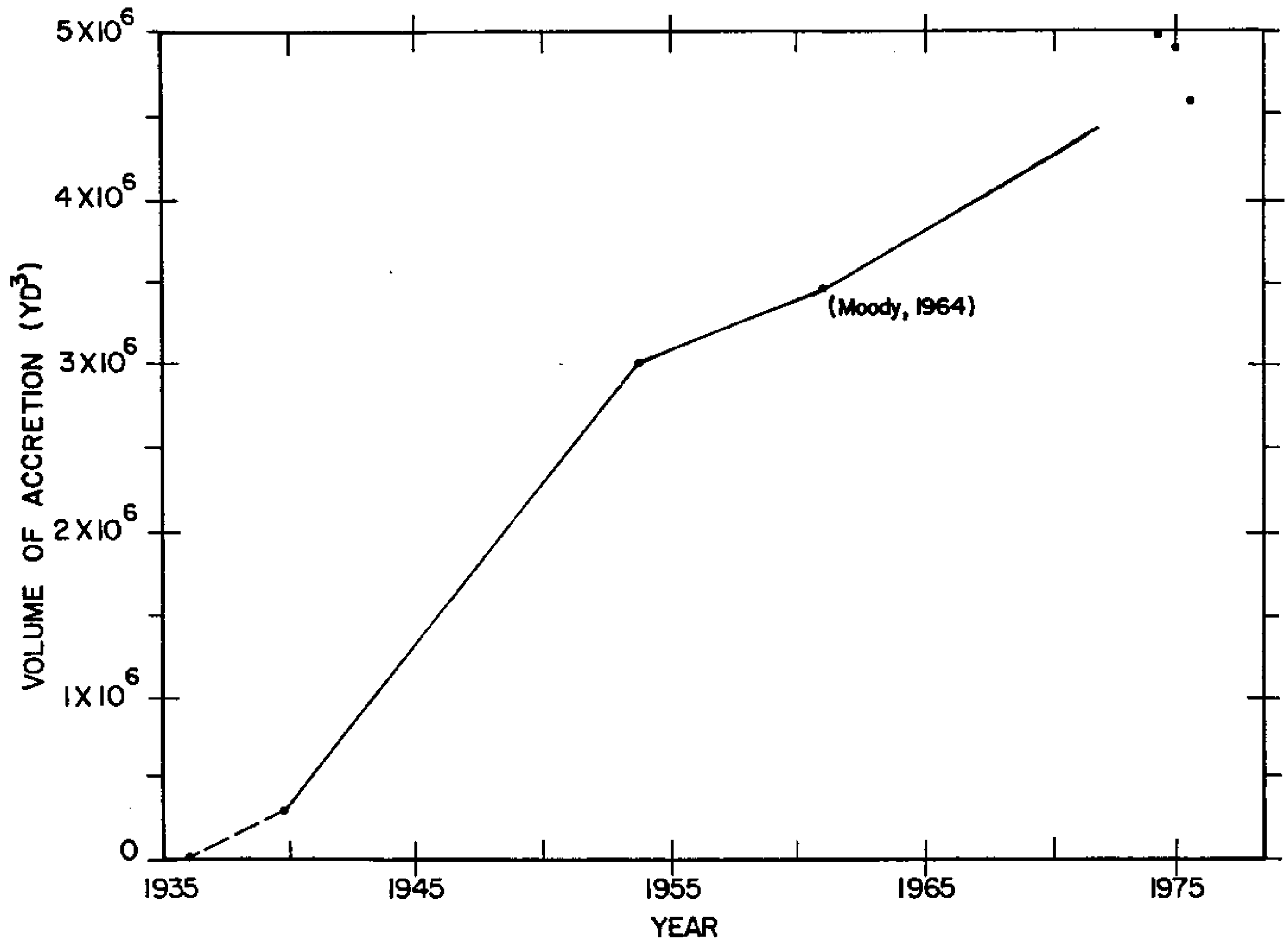


Figure 57 Volume of Accretion on Outer Shoal Versus Year

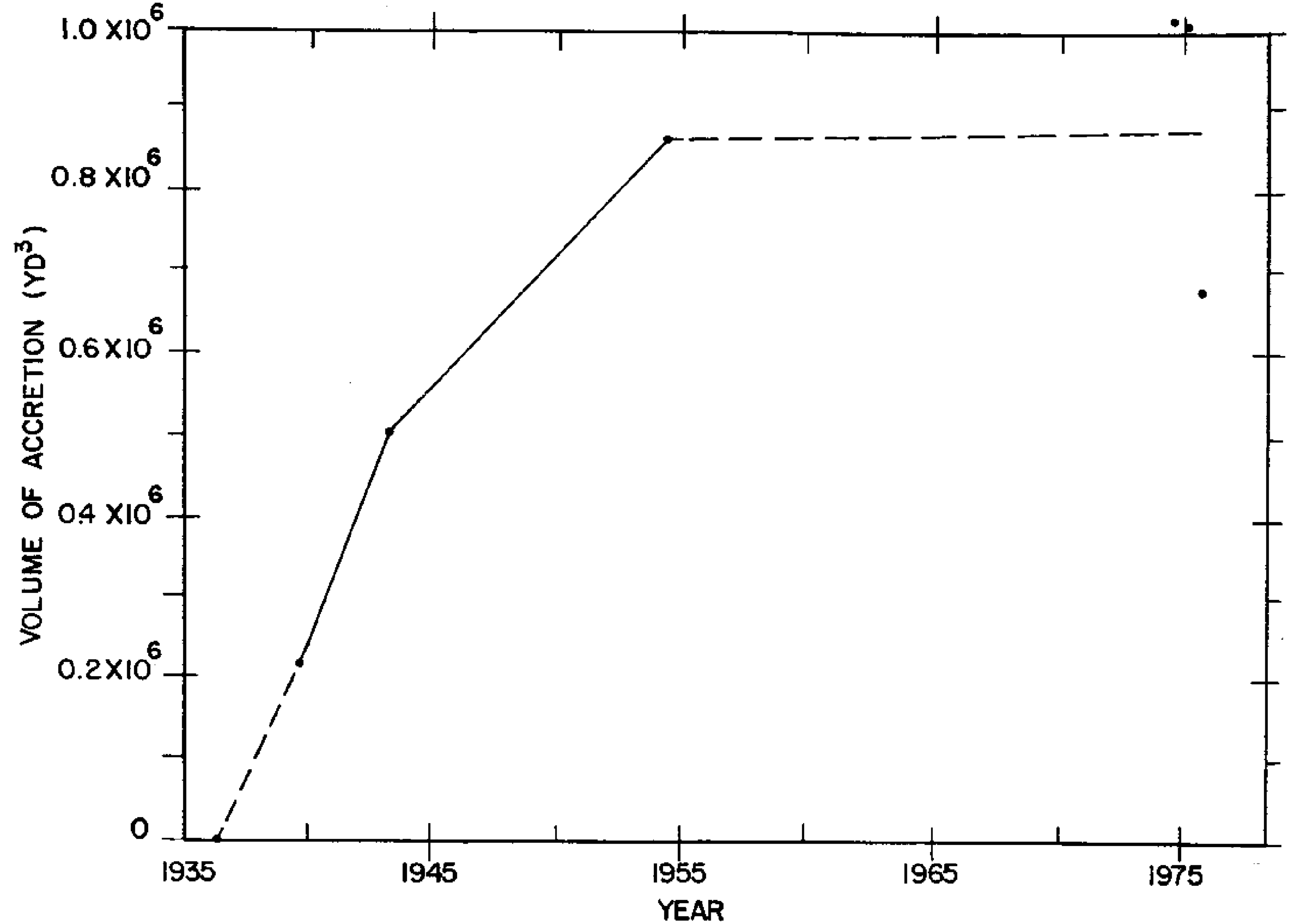


Figure 58 Volume of Accretion on North Portion of Outer Shoal Versus Year

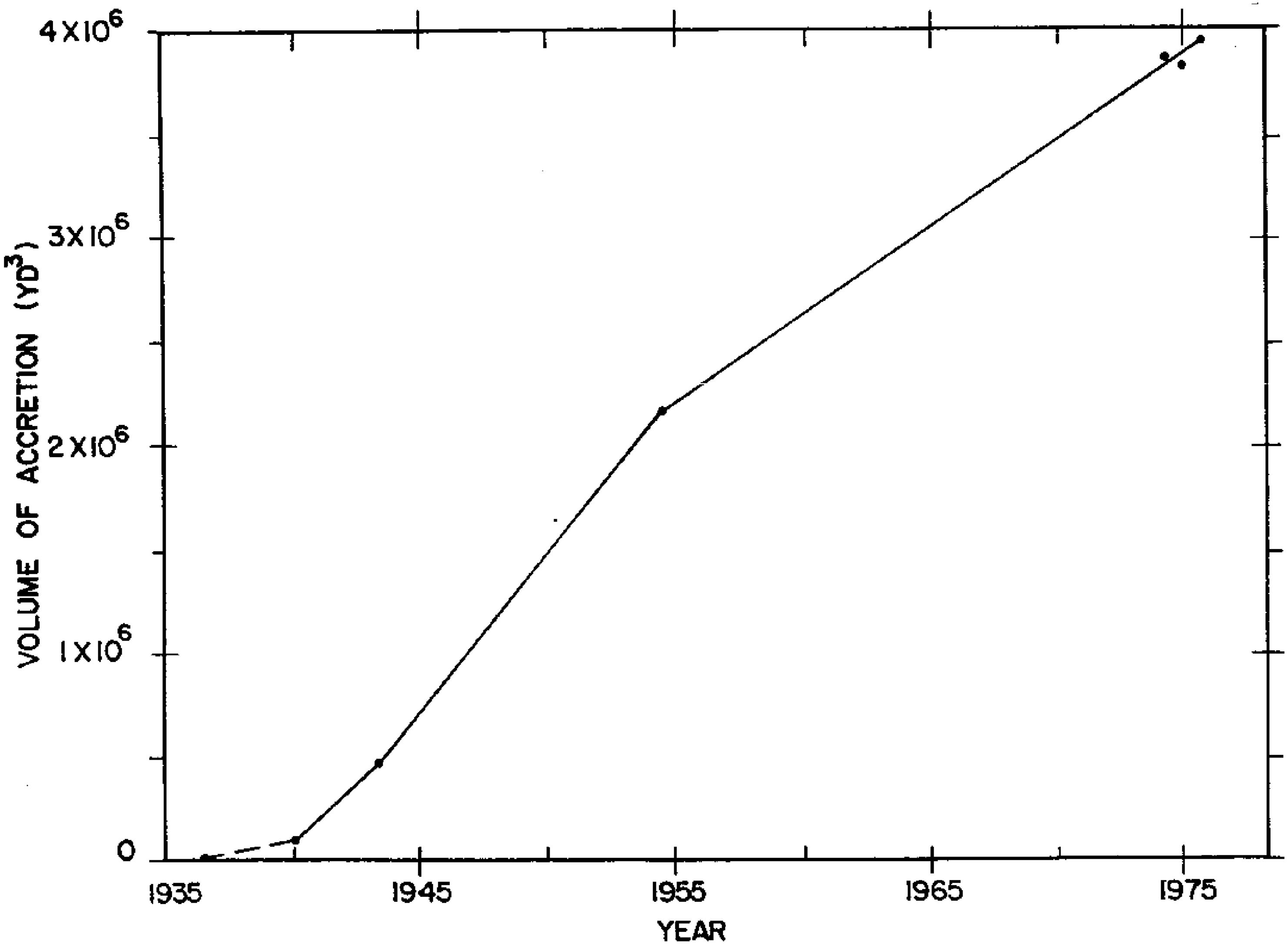


Figure 59 Volume of Accretion on South Portion of Outer Shoal Versus Year

TABLE 8 VOLUMETRIC CHANGES IN OUTER SHOAL SINCE 1936
BY QUADRANT (cubic yards)

Date / Quad.	Dec. 1939	Feb. 1943	June 1954	Sept. 1974	Mar. 1975	Nov. 1975
A	90,000	463,000	1,857,000	635,000	640,000	606,000
B	240,000	501,000	*865,000	420,000	490,000	266,000
E & G			*300,000	2,374,000	2,372,000	2,443,000
F				796,000	625,000	405,000
H				850,000	814,000	913,000
J				204,000	189,000	208,000
C	46,000			72,000	111,000	111,000
D	-6,000			-375,000		-478,000

* Quadrant Area Other Than Shown on Figure 56

than the south. Presently, however, the south shoal contains four times as much sand as the north.

The change in volume of areas J and C were discussed in the section on accretion of the south beach, and the results from area D will be discussed in the next section dealing with the erosion of the north beach.

In a recent paper by T. L. Walton and W. D. Adams (1976), the relationship between the volume of sand stored in the outer shoal of an inlet and the tidal prism of the inlet is discussed. While there is considerable scatter to the data, this relationship (volume = $10.7 \times 10^{-5} P^{1.23}$) appears to hold for inlets on sandy coasts. The values of tidal prism and shoal volume for Indian River Inlet presented in the paper are older values and not very close to the theoretical relationship. The new values presented in this report appear to be off by a similar amount, however.

3.11 Erosion of the North Beach

The stabilization of Indian River Inlet has interrupted the net northerly littoral drift. Previously, sand bypassed the natural inlet without the impoundment of large volumes in ebb or flood tidal shoals. Sand is presently being accumulated on the beach south of the inlet and on the outer shoal. It appears that little sand moves from the north lobe of the outer shoal to the north beach by natural means except in rare events such as the March, 1962 storm. The sand which has reached the north beach is primarily from beach nourishment projects. This artificial placement of sand on the north beach will be discussed in Section 4.1. If sand is transported from the south lobe of the outer shoal to the south beach, it will reenter the channel and be deposited primarily on the south lobe of the outer shoal again.

Very little sand is transported northward to the beach north of the inlet. The littoral drift, however, reestablishes itself north of the inlet and transports sand away from the north beach.

Continuity thus requires that the north beach erode, and so it has. The north beach has eroded a maximum of 480 feet at a point 2,000 feet north of the inlet since 1938.

The present configuration of the north beach is shown in Figure 60. The landward indentation in the beach extending about 1.75 miles north of Indian River Inlet is only partially caused by the inlet. In 1938, there was an indentation centered near the old

Figure 60 Indian River Inlet and Beach North of Inlet Viewed from the South

Coast Guard Station, 1.75 miles north of the inlet, as shown in Figure 61. The erosion of the north beach has progressed northward and has joined with the old indentation to produce the present beach configuration.

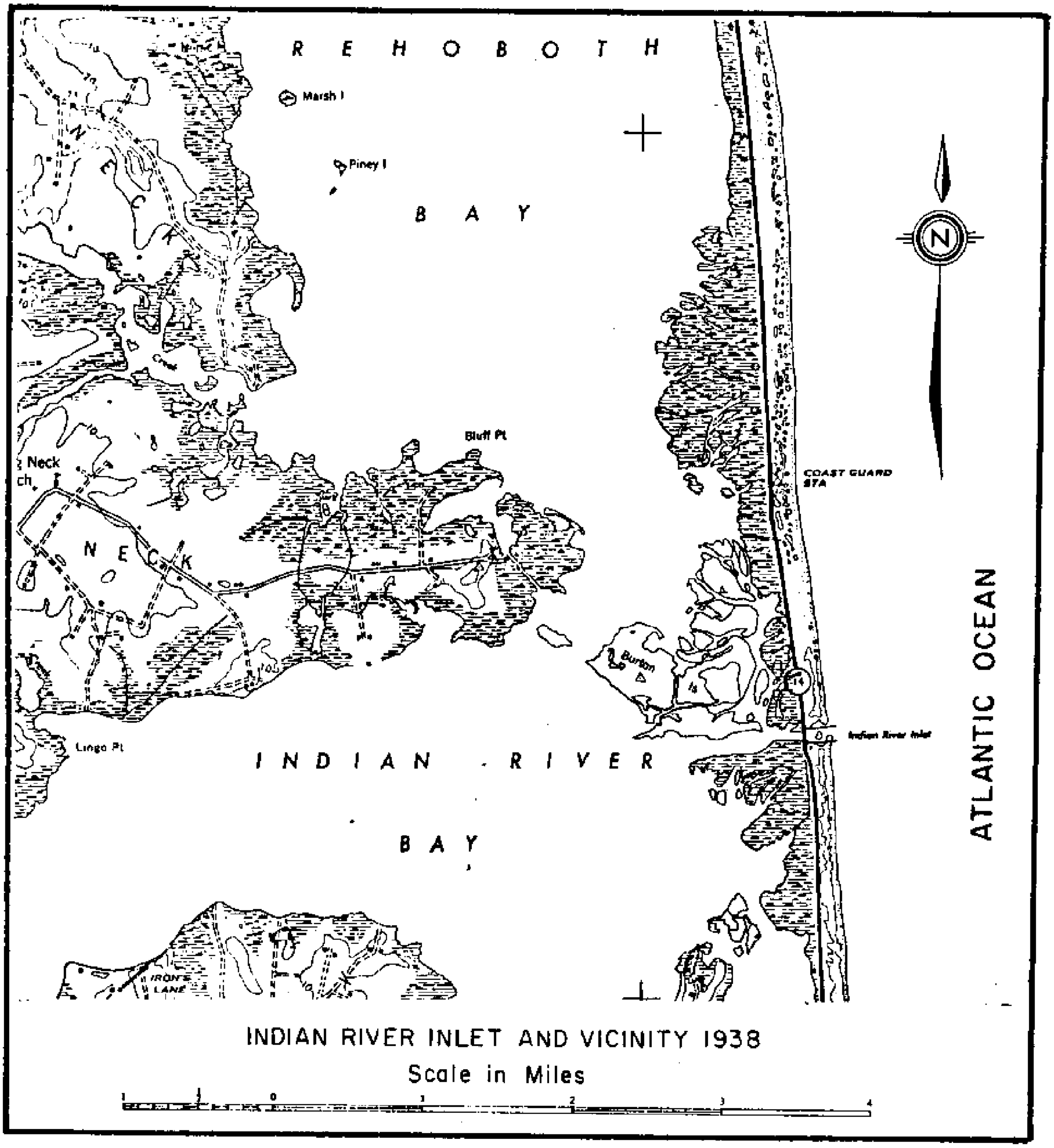


Figure 61 Map of Indian River Inlet and Vicinity, 1938. Note the Indentation in the Shoreline Near the Old Coast Guard Station (After U.S. Geological Survey, 1938).

The state and federal governments have been nourishing the beach north of the inlet since 1957 to counteract erosion. To date, a total of 2,019,549 c.y. of sand have been placed on the north beach, but the north beach continues to erode. The various nourishment projects will be discussed later.

According to the U.S. Army Corps of Engineers (1968), the one-mile section of beach northward of the inlet had an average landward recession of 7 feet per year between 1843 and 1939. Between 1939 and 1954, the recession in this area averaged 21 feet per year. Beach fills in 1957 and 1963 and sand placed during Operation FIVE-HIGH after the storm of March, 1962 resulted in an advance of the shoreline immediately north of the inlet from 1954 to 1964. The average annual advance for this 10-year period was 20 feet per year at the north jetty and 13 feet per year about 1,000 feet north of the jetty. During this same period, 4,000 feet north of the north jetty the shoreline receded landward an average of 5 feet per year and 13 feet per year about 1.75 miles north of the inlet (U.S. Army Corps of Engineers, 1968).

The U.S. Army Corps of Engineers (1968) has estimated an erosion rate of approximately 85,000 c.y. per year for the half mile section of beach north of the inlet between 1954 and 1964. Their computed values for erosion in the 1.75 mile section of beach north of the inlet are shown in Table 9.

TABLE 9 BEACH EROSION WITHIN 1.75 MILES NORTH OF INDIAN RIVER INLET (c.y.)

	1843-1929 (86 years)[1]	1929-1954 (25 years)	1954-1964 (10 years)[2]	1843-1964 (121 years)[2]
Volume	-6,200,000	-3,440,000	-1,976,000	-11,616,000
Annual Rate	-72,000	-138,000	-198,000	-96,000

[1] Computations based upon change in beach area between shorelines and upon an estimated relation between volume change and unit area.

[2] Computations based upon profile changes and include adjustments to account for beach fill and dune fill placed after 1954.

The volumes eroded from area D (shown in Figure 56) versus years are shown in Figure 62. This area extends from the tips of the jetties landward to the north beach baseline, and 1,250 feet north from the north jetty. Between September, 1974 and November, 1975, there was a net erosion of 103,400 c.y. of sand from the north beach area surveyed. This is an alarming erosion rate considering that there was a beach nourishment project carried out on the north beach during January, 1975. The erosion necessitated the relocation of the north baseline for the November, 1975 survey.

The lack of an accurate survey of this area before the inlet was stabilized and at present prohibits an accurate calculation of the volume of erosion beyond about 1,250 feet north of the inlet. An estimate of the volumes was made by first estimating the surface

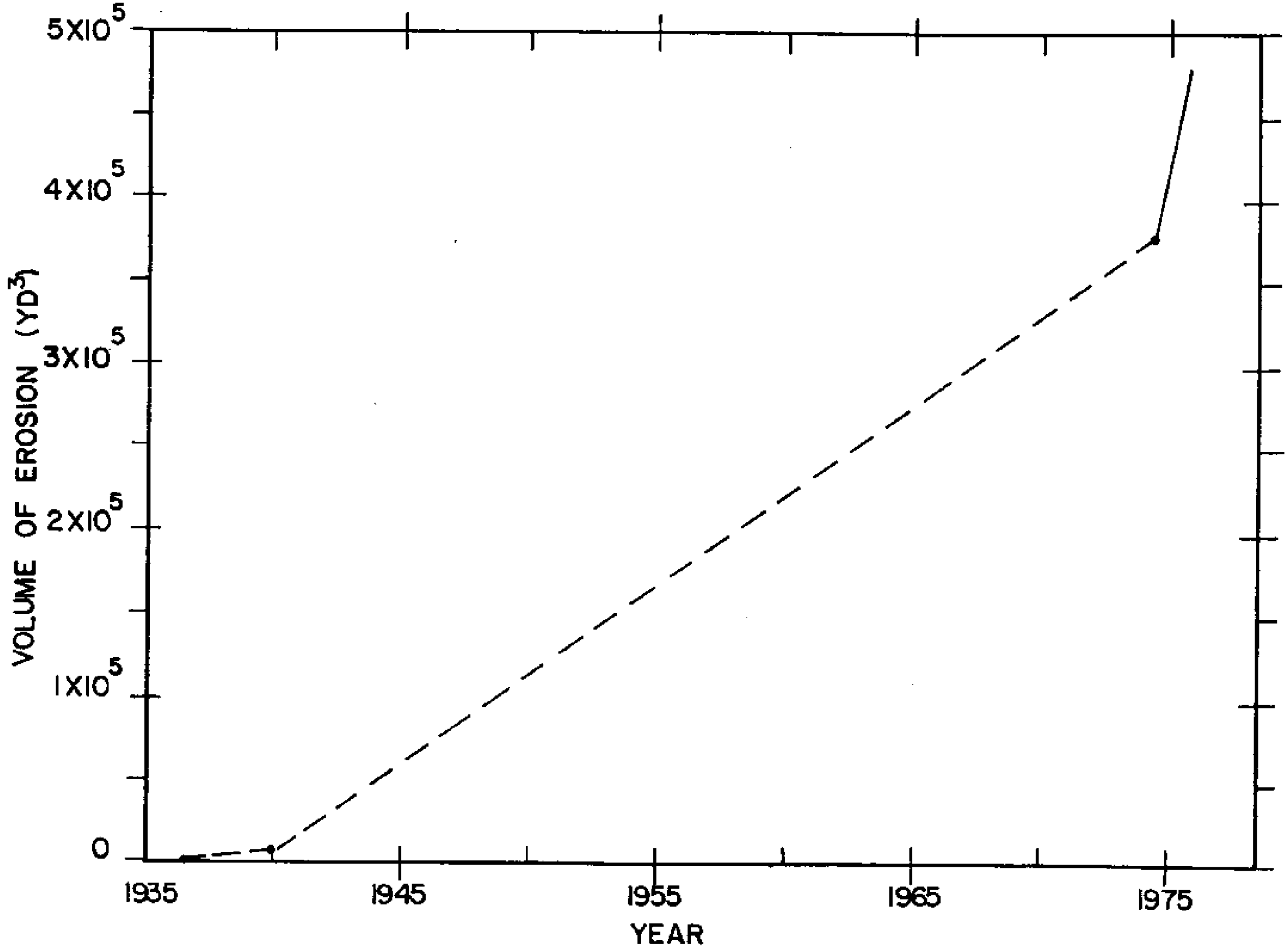


Figure 62 Volume of Erosion from the North Beach Versus Year

area of beach recession and then applying the conversion factor discussed in section 3.4. The combined erosion from the north jetty to 10,000 feet north of the north jetty was estimated to be about 3 million cubic yards. This figure is comparable to those of the U.S. Army Corps of Engineers when it is considered that their 1954 to 1964 value included the effects of the March, 1962 storm. The volumes of erosion calculated in this study were also net changes in sediment volumes and no correction was included to account for the erosion of sand placed as beach nourishment. The volumes calculated by the Corps were gross volumes and included the erosion of sand placed as beach fill.

An interesting phenomenon,which is possibly the result of the increased erosion due to the stabilization of the inlet, is found at "Coin Beach," about one mile north of Indian River Inlet. Since about 1935, many Irish halfpennies (dated between 1766 and 1782) have been found on this section of beach. In the early 1950's, the old coins could be gathered in considerable numbers from the face of the beach, particularly after a storm. It is believed the coins were from the wreck of the "Faithful Steward." On September 1, 1785, the vessel, bound from Londonderry to Philadelphia, ran aground about four leagues south of Cape Henlopen (one mile north of the present Indian River Inlet). There was a heavy surf running and although the ship was only about 100 yards from the beach, only 68 of the 249 persons on board were saved. It is believed that these coins were being sent to America to relieve a coin shortage (Zwaanendael Museum, 1976). The coins were apparently washed onto the beach and later exposed by beach erosion. The early 1950's, when the coins were found in increased numbers, corresponds approximately to the time of increased erosion of the north beach when inlet stabilization began to effect that portion of the beach.

3.12 Sediment Characteristics in the Vicinity of the Inlet

The sand on Delaware beaches has variable size characteristics. Depending upon the location on the beach from which a sample is taken and the condition of the beach at the time a sample is taken, the characteristics of the sand can vary greatly. Typical size characteristics for Delaware beach sands are given in Table 10 (U.S. Army Corps of Engineers, 1968).

Layers of pebbles are sometimes found on the beaches. In addition, marsh muds are frequently exposed on the beach face after a severe northeast storm.

The sand is composed primarily of quartz and lesser amounts of feldspar. The heavy minerals are composed of variable percentages of hornblende, zircon, staurolite, garnet, sillimanite and black opaques (Schneider, 1962).

The ocean bottom is composed primarily of sand with some gravel and silt (Moody, 1964). The sediment deposited on the outer

TABLE 10 SIZE CHARACTERISTICS OF LITTORAL MATERIALS - DELAWARE ATLANTIC COAST

Comparison of Data from Samples Taken at Mid-Tide
1936 - 1954 - 1964

Location of Sample in Miles North (N) & South (S) of Indian R. Inlet	M (mm) 1936	M (mm) 1954	M (mm) 1964	S_o 1936	S_o 1954	S_o 1964	Log S_k 1936	Log S_k 1954	Log S_k 1964	1936 Sample No.	1954 Survey Line No.	1964 Survey Line No.
10.4	.42	.94	.47	1.28	1.49	1.27	-.02	-.03	-.01	24	37	42
9.5	.41	.60		1.35	1.48		0	.01		23	38	
8.5	.40	1.15		1.23	1.48		-.02	0		22	39	
7.6	.55	.88		1.46	1.50		-.06	-.09		21	40	
6.6	.52	1.05		1.34	1.57		-.01	.02		20	41	
5.7	.68	.59		1.24	1.42		-.01	-.04		19	42	
4.7	.57			1.45			.12			18		
3.8	.48	.90		1.33	1.57		-.01	.11		17	43	
2.8	.36			1.30			.00			16		
1.9	.61	.72		1.45	1.53		-.05	0		15	44	
0.9	.49	1.00		1.32	2.02		0	.15		14	45	
N. Side of Indian R. Inlet	.52	.53	.24	1.25	1.62	1.31	.02	.10	.04	13	46	54
S. Side of Indian R. Inlet	.60			1.28			.05			12		
0.9	.56	.50	.30	1.26	1.79	1.45	.02	0	.08	11	48	56
1.9	.43			1.21			-.02			1		
2.8	.65	.50		1.35	1.45		.02	.09		2	49	
3.8	.40	.85	.29	1.22	1.29	1.25	-.01	0	-.02	3	50	59
4.2		.60			1.28			.04			51	
4.7	.41	.83		1.25	1.48		.01	-.10		4	52	
5.1		.28			1.15			-.01			53	
5.7	.50			1.31			.03			5		
6.0			.33			1.23			.01			62
6.6	.62			1.34			-.06			6		
7.6	.37			1.26			.03			10		
7.9		.36			1.21			.03			55	
8.5	.46			1.33			.01			9		
8.9			.28			1.62			-.44 *			65
9.5	.40			1.28			-.04			8		
9.9		.68			1.41			-.08			56	
10.4	.36			1.26			.01			7		
							Total Number Samples			24	18	6
Average Values	0.49	0.72	0.32	1.31	1.48	1.35	0.00	0.01	0.02			

* Not included in average value.

shoal of Indian River Inlet is a medium sand with an average median diameter of 0.46 mm (1.12 ϕ) and is well sorted. The sand size characteristics for samples taken during this study from the outer shoal were presented in Table 6. The locations of the samples are shown in Figures 53, 54 and 55.

The sediment in Indian River Bay is primarily dark gray lagoonal mud. The bay is fringed with some sand and *Spartina* and *Distichlis* marshes. The eastern part of the bay contains sands derived from tidal deltas. The sediment distribution patterns for Indian River Bay are shown in Figure 63 from Kraft and Margules (1971).

Sediment sizes within the inlet channel range from fine sand to pebbles. The samples examined were well to poorly sorted. The sediment size characteristics for these samples are shown in Table 5. The locations of the samples are indicated in Figures 45 and 46. The locations of samples taken between station 12+00I and the tips of the jetties were shown in Figures 53 and 54.

The sediment samples taken from within the 500-foot wide throat section of the channel were generally coarser than samples taken at other locations. This trend is shown in Figure 64 which plots the median grain size versus inlet station at which the sample was obtained. At some placed within the inlet throat, there is no sand on the bottom. The water velocities are too high for the deposition of sediment in these areas.

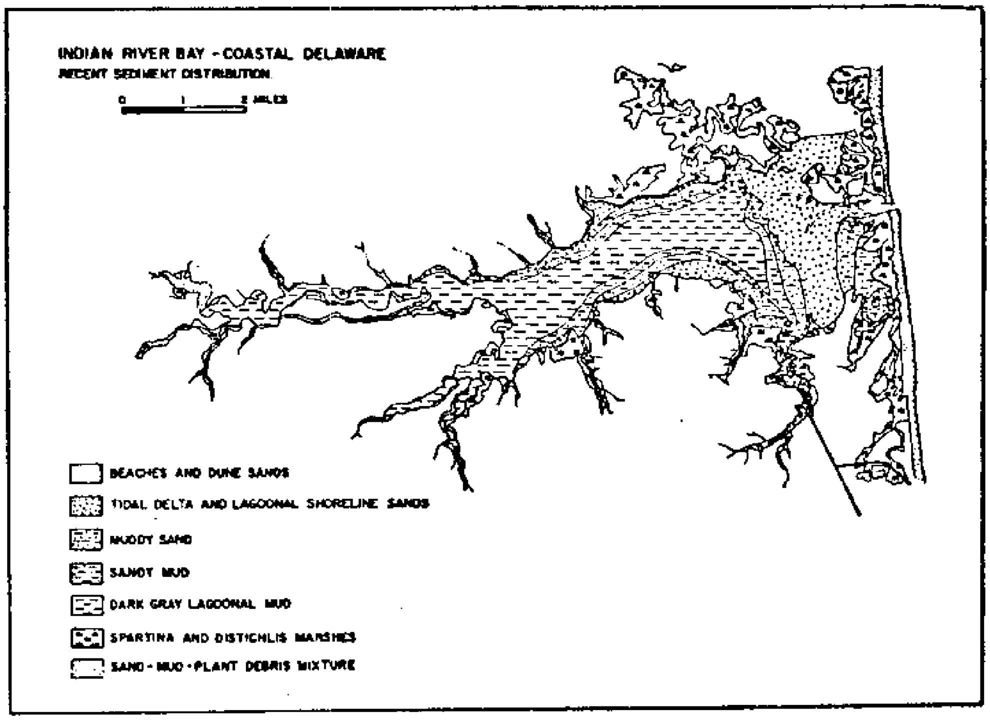


Figure 63 Sediment Distribution Patterns in Indian River Bay (Kraft and Margules, 1971)

Figure 65 is a cross section of the inlet based on soils borings for the present highway bridge (Delaware State Highway Department, 1975). The continuous clay layer is about 35 to 40 feet below MSL near the channel. Continued erosion of the channel bottom

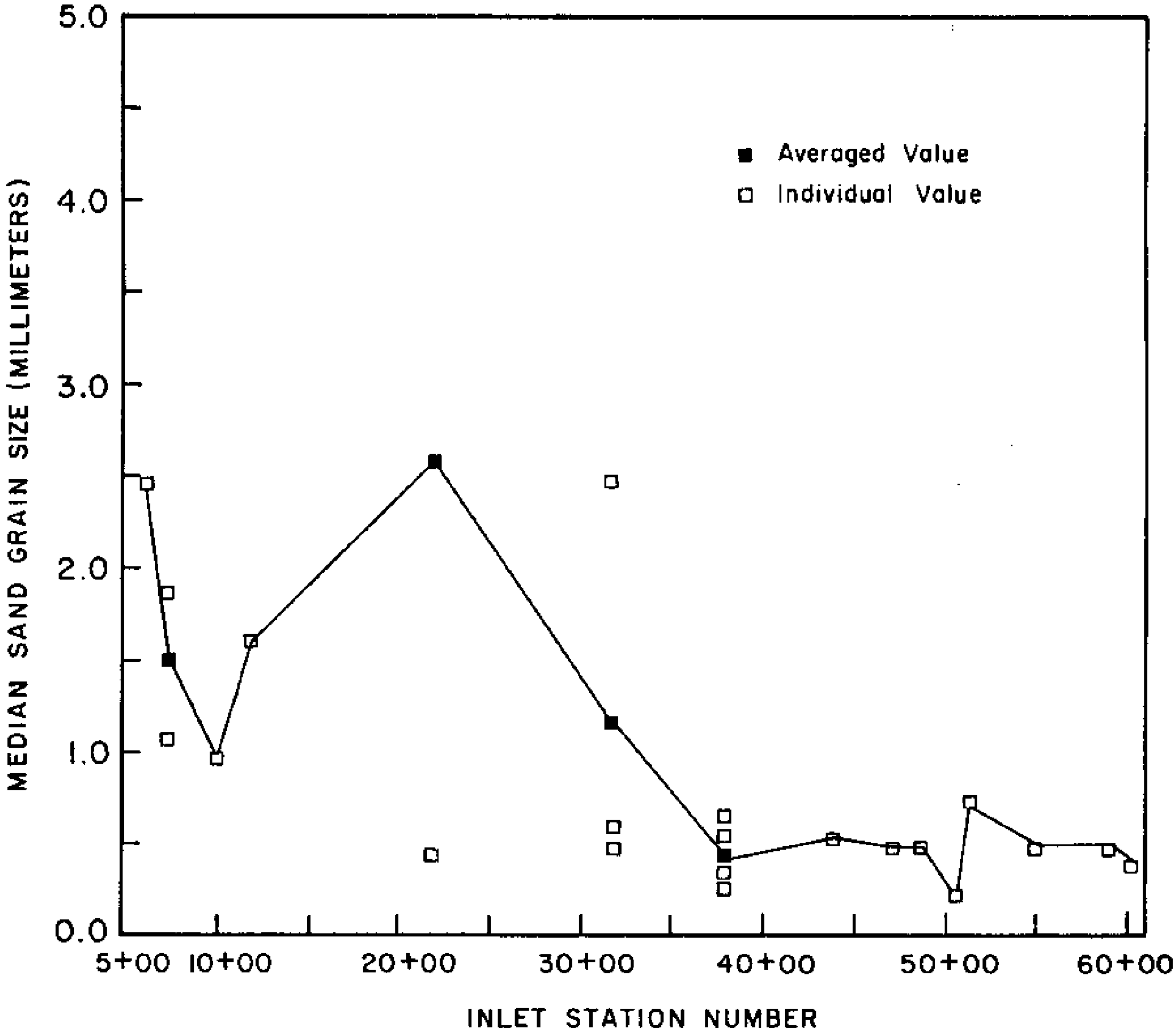


Figure 64 Median Sand Grain Size Versus Inlet Station

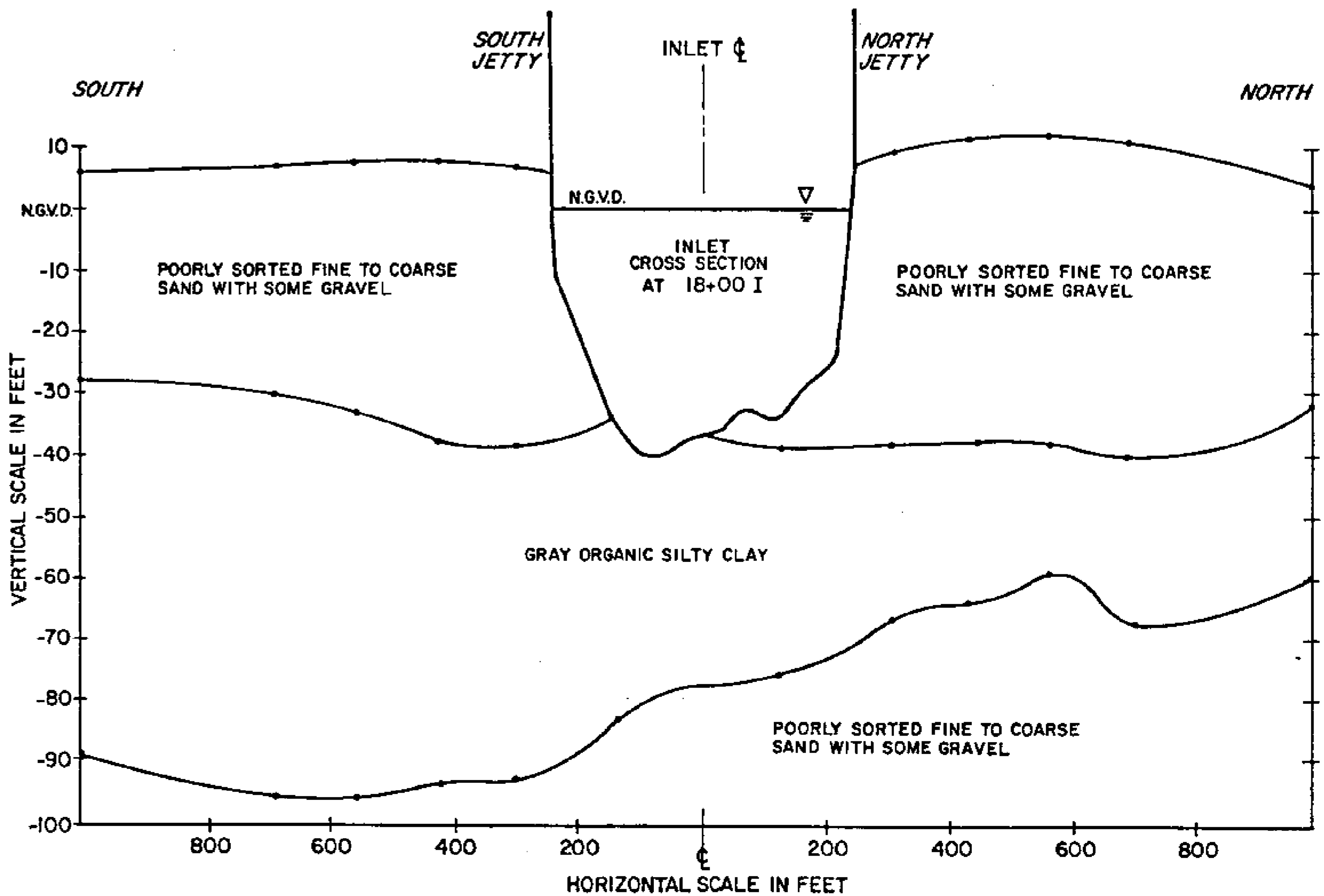


Figure 65 Inlet Cross Section Showing Location of Clay Layer

in this area requires the erosion of the clay, which is much more stable than sand due to its cohesive properties.

It is unknown if the clay exposed on the channel bottom is slowing the increase in depth of the inlet. There has been some erosion of this clay in the vicinity of the bridges since the soils borings were made. The depth to this clay layer is unknown at the locations of the deep scour holes, such as at the tips of the jetties and near the Coast Guard Station. It is likely that some clay was eroded to produce scour holes up to 55 feet deep.

CHAPTER IV

SAND BUDGET

In addition to the natural sediment movement discussed in the previous chapter, a significant amount of sand has been removed from Indian River Inlet by dredging. The various dredging and beach nourishment projects will be discussed in this chapter. This information will be combined with the information from the sediment movement chapter to yield the sediment budget for Indian River Inlet and an estimate of the net littoral drift in the vicinity of the inlet.

4.1 Dredging and Beach Nourishment

Indian River Inlet has been dredged for 3 main purposes: construction of the inlet, maintenance of the channel, and as a borrow source for beach nourishment and land fill operations. The two most recent dredging operations have obtained sand for nourishment of the north beach by dredging from shoals within the inlet. Thus, two goals have been achieved in one dredging operation. The dredging information for the construction of the inlet was presented in Chapters I and II.

The first maintenance dredging of the inlet was done in 1951. As part of the channel improvements to Millsboro, a shoal of approximately 86,000 c.y. was removed from west of the highway bridge (Thompson, 1974).

In 1957, and again in 1963, the state dredged from an area behind the dunes and north of the inlet to obtain sand for nourishing the north beach. The borrow site from the 1957 dredging can be seen in Figure 14. The 1963 dredging was conducted in the same area.

The first of these two projects was conducted from November 20, to December 11, 1957. Under this contract, 349,102 c.y. of sand were placed on a 4,200-foot length of beach at a cost of $.62 per c.y., and 163,294 c.y. of sand were placed on a 3,000-foot length of the dunes of which 154,898 c.y. were placed at a cost of $.62 per c.y. and 8,396 c.y. at a cost of $.48 per c.y. Under the River and Harbor Act of July 3, 1958, the federal government paid 33.8% of the $316,510 total cost (Henry, 1975). This federally authorized project called for the periodic nourishment of the beach north of Indian River Inlet for a 10-year period after the initial 1957 placement. The state planned to place the remainder of the periodic nourishment in 1962 but these plans were delayed by the March, 1962 storm.

A second dredging, similar to the 1957 project, was conducted from August 11 to September 30, 1963. A 5,000-foot length of the north beach and dunes was nourished with 590,342 c.y. of sand. The unit price was $.635 per c.y., for a total cost of $374,867.77.

The practice of dredging immediately behind the barrier for beach nourishment sand has two drawbacks. The barrier is more likely to be breached by a storm in the location of borrow areas, and deep holes in the bottom of the bay are also a hazard to shell fishermen. The borrow areas for beach nourishment sand in Rehoboth Bay have claimed the lives of several people who were wading in the shallow bay. These holes are presently being filled by the state.

The 1957 and 1963 operations placed 1,102,739 c.y. of fine to coarse sand on the north beach. Sand placed as dune fill has been included in this figure. The placement of this large quantity of sand was effective in temporarily stopping the landward recession of the shoreline (as seen in Figure 18).

The March, 1962 storm carried large quantities of sand from the beach landward across the highway (as seen in Figure 15). In April, 1962, under Operation FIVE-HIGH, 290,000 c.y. of sand were bulldozed onto the north beach between the Coast Guard Station (1.75 mi. north of the inlet) and Indian River Inlet. The sand was obtained from the washover fans created by the storm. It will be assumed that this operation negated the sediment loss due to washover during the 1962 storm.

Maintenance dredging of the channel in Indian River Bay has been performed twice by the U.S. Army Corps of Engineers. In both operations, the section of channel dredged was about 2.5 miles west of the ocean and the sand was spoiled approximately 1000 feet south of the channel. Between June 13 and December 3, 1968, 80,447 c.y. of sand were dredged at a cost of $67,429 ($.838 per c.y.). The second dredging operation, conducted between July 13 and July 21, 1970, removed 22,907 c.y. at a cost of $34,754 ($1.52 per c.y.). Further channel maintenance dredging was scheduled for 1974 but was cancelled due to disposal area problems (Caccese, 1975).

The two most recent dredging operations were conducted primarily to nourish the north beach. Between December 6, 1972 and January 4, 1973, 774,263 c.y. of sand were dredged from the channel west of the highway bridge and placed on the north beach. A 27-inch hydraulic pipeline dredge was used to place the sand along 5,000 feet of the north beach. The unit cost was $.823 per c.y. with a total cost of $637,218.47. The project was funded with 30% state money and 70% federal (Henry, 1975).

Between January 13 and April 8, 1975, the inlet was dredged once again. This time a 12-inch hydraulic pipeline dredge was used and a booster pump was needed to place the sand on the northern portion of the 5,000-foot-long beach fill. The unit price for this contract was $1.94 per c.y. The total volume of sand dredged was 142,547 c.y. The federal government paid all of the $276,541.18 cost. The unit price for this operation was substantially higher than that of the 1973 operation because of the smaller dredge size and the need for the booster pump. The sand was dredged from the middle shoal, about 800 feet southwest of the Coast Guard Station. A relatively small area of the shoal was dredged to a depth of about 20 feet below LMLW. The sand which was dredged contained varying amounts of gravel, making it necessary to occasionally flush the pipeline with water to prevent buildup of gravel in the pipeline.

The 1973 and 1975 beach nourishment project placed 916,810 c.y. of sand on the north beach, bringing the total to 2,019,549 c.y. By dredging from within the inlet channel, sand which otherwise could have been carried onto the inner or outer shoal and lost from the littoral drift zone is reintroduced into the northward littoral drift.

Sand obtained from within the channel is relatively coarse. The finer sand has been carried further into the bay or out onto the outer shoal. Coarse sand will produce a steeper beach slope and provide a longer-lasting beach fill than fine sand (U.S. Army Corps of Engineers, Coastal Engineering Research Center, 1973). However, the sand placed in 1975 eroded rapidly. Figure 66, taken in November, 1975, shows a 5.5 foot scarp eroding into the beach fill station 4+00N. Note the coarseness of the sand dredged from the channel.

Between September, 1974, and September, 1975, the water line at NGVD eroded landward an average of 55 feet within 1,000 feet north of the inlet. Even with the beach nourishment project of January to

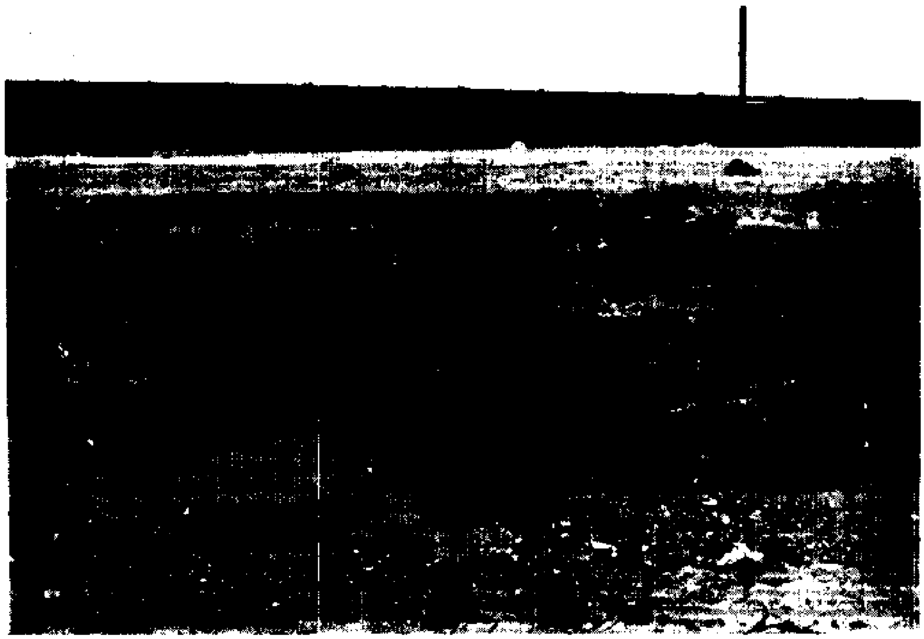


Figure 66 Scarp Eroding into Fill Placed on North Beach. The Can is 10 Inches High.

April, 1975, a total of 103,000 c.y. were eroded within 1,250 feet north of the north jetty.

During the 1975 beach nourishment project, a booster pump was necessary to pump sand up to 5,000 feet north of the inlet. When relatively small amounts of sand are placed during a single project, it should not be necessary to pump the sand that far north. The northward littoral drift could be allowed to redistribute the sand northward.

Beach fill sand should be placed sufficiently far northward of the jetties to prevent its reentering the inlet. Northeast storms create a southerly littoral drift. The longshore current can be seen turning seaward upon encountering the north jetty. It is unknown how much sand is introduced into the channel in this manner. Fill on the north beach should be placed preferably in the spring, since the littoral drift can be southerly during the winter, carrying sand into the channel from the north.

There have also been several minor dredging operations in Indian River Inlet. In early 1961, about 48,000 c.y. were dredged from the inlet and spoiled on the south channel at the present site of the State Park campground. This sand was then hauled to Bethany Beach, Delaware for beach nourishment. This project was halted by numerous difficulties such as the dredge drifting into the bridge and having to be beached to prevent its sinking (Delaware State Highway Department, 1975). During 1965, sand was dredged from the inlet for the construction of the State Park campground on the south side of the inlet. The volume dredged was approximately 90,000 c.y. The dredging of the South Shore Marina and Bladder's Pond (see Figure 2 for location) were neglected in sediment movement computations. Little or no sand was introduced or taken from the inlet during these operations.

In 1975, a large shoal threatened to completely block the entrance of the South Shore Marina. Dredging operations conducted by the marina owner had very little success. The dredged channel was quickly filled in by sand from the shoal.

Information on the major dredging operations at Indian River Inlet have been compiled in Table 11. From 1938 to the present, 2,338,724 c.y. of sand have been removed from the inlet. A total of over $1,707,320 has been spent on dredging since 1957. All dredging was done with hydraulic pipeline dredges.

4.2 Sand Budget

A sand budget for Indian River Inlet was developed by combining knowledge of sediment movement patterns, sediment volume changes, dredging and beach nourishment. The sand budget states how much sediment has moved and where. Each component of the sand budget for Indian River Inlet will now be reviewed.

Since 1938, sand from the net northward littoral drift has been deposited on the south beach. The total volume accreted is 319,000 c.y. in 37 years for an average annual rate of 8,600 c.y. per year. The excess of sand carried to the south beach over the amount of sand accreted on the south beach has been carried into the inlet channel.

The erosion of the channel banks and bottom has introduced considerable sediment into the system. The increase in the channel width and depth, between stations 5+50I and 60+00I, corresponds to a loss of 3,666,000 c.y. of sediment. This loss of sediment is a result of both erosion and dredging operations.

TABLE 11 DREDGING HISTORY

Year	Dredging Site	Spoil Site	Volume Removed (c.y.)	Unit Cost (cost/c.y.)	Total Cost
1938	Channel Construction	North Channel Bank	1,198,409		
1951	Canal Extension		1,075,000		
1951	West of Inlet Bridge		~86,000		
1957	North of Inlet	North Beach	512,396	~$0.62	$316,510.08
1961	West of Inlet Bridge	South Channel Bank	~48,000		
1963	North of Inlet	North Beach	590,343	~$0.635	$374,867.77
~1965	West of Inlet Bridge	Campground South Bank	~90,000		
1968	2.5 mi. West of Ocean	1,000' South of Channel	80,447	~$0.838	$67,429
1970	2.5 mi. West of Ocean	1,000' South of Channel	22,907	~$1.517	$34,754
1972–1973	West of Inlet Bridge	North Beach	774,263	$0.823	$637,218.47
1975	Middle Shoal	North Beach	142,547	$1.94	$276,541.18

There was an estimated loss of 1,090,000 c.y. of sediment from Indian River Bay west of station 60+00I. This sand is assumed to have been carried into the inlet channel by the tidal currents. An accurate survey of the bay shoals would improve the accuracy of the computed volume change, and it would indicate if any sand is being carried further into the bay or through the Ditches.

A total of 1,422,000 c.y. of sand has been dredged from the inlet channel and spoiled on the banks from 1938 to the present. A portion of these dredge spoils have since reentered the inlet channel through channel bank erosion. The volume of dredge spoils which has thus reentered the inlet is unknown.

A total of 916,810 c.y. of sand has been dredged from the inlet and placed on the north beach. The north beach has also been nourished with 1,102,739 c.y. of sand from a borrow site north of the inlet.

The sand eroded from the channel is carried onto the outer shoal by the ebb tide. Since 1938, approximately 4,884,000 c.y. (taking the average of the three values computed from surveys obtained in this study) have been deposited on the outer shoal. This corresponds to an average annual rate of 132,000 c.y. per year. It will be assumed that there has been no exchange of sand between the beaches and the outer shoal. The volume actually transported from the northern lobe of the shoal onto the north beach by the March, 1962, storm is unknown.

There has been a net erosion of the north beach of approximately 3,000,000 c.y. since 1938. This sand, along with the sand placed as beach nourishment, has been carried northward in the littoral drift. It will be assumed that no sediment was carried from the north beach into the inlet channel.

These components of the sediment budget have been combined in Figure 67. Solving the continuity equation for the inlet channel, node B, one can solve for the volume of sediment to pass from the south beach into the channel.

$$\underset{\text{Beach Erosion}}{-3.67 \times 10^6} = \underset{\text{Bay Erosion}}{1.09 \times 10^6} - \underset{\text{Dredging}}{1.42 \times 10^6} - \underset{\text{Dredging}}{0.42 \times 10^6} - \underset{\text{Shore Accretion}}{4.88 \times 10^6} + x$$

$$x = \text{Sand Passing South Jetty} = 2.46 \times 10^6 \text{ c.y.}$$

Next, solving at node D, the south beach, it is found that a total of approximately 2.78×10^6 c.y. of sand were carried to the south beach by littoral drift. This corresponds to an average annual value of 75,000 c.y. per year (dividing 2.78×10^6 by 37 years, 1938-1975).

Similarly, solving the continuity equation for the north beach, node C, the average annual littoral drift can be estimated at 136,000 c.y. per year.

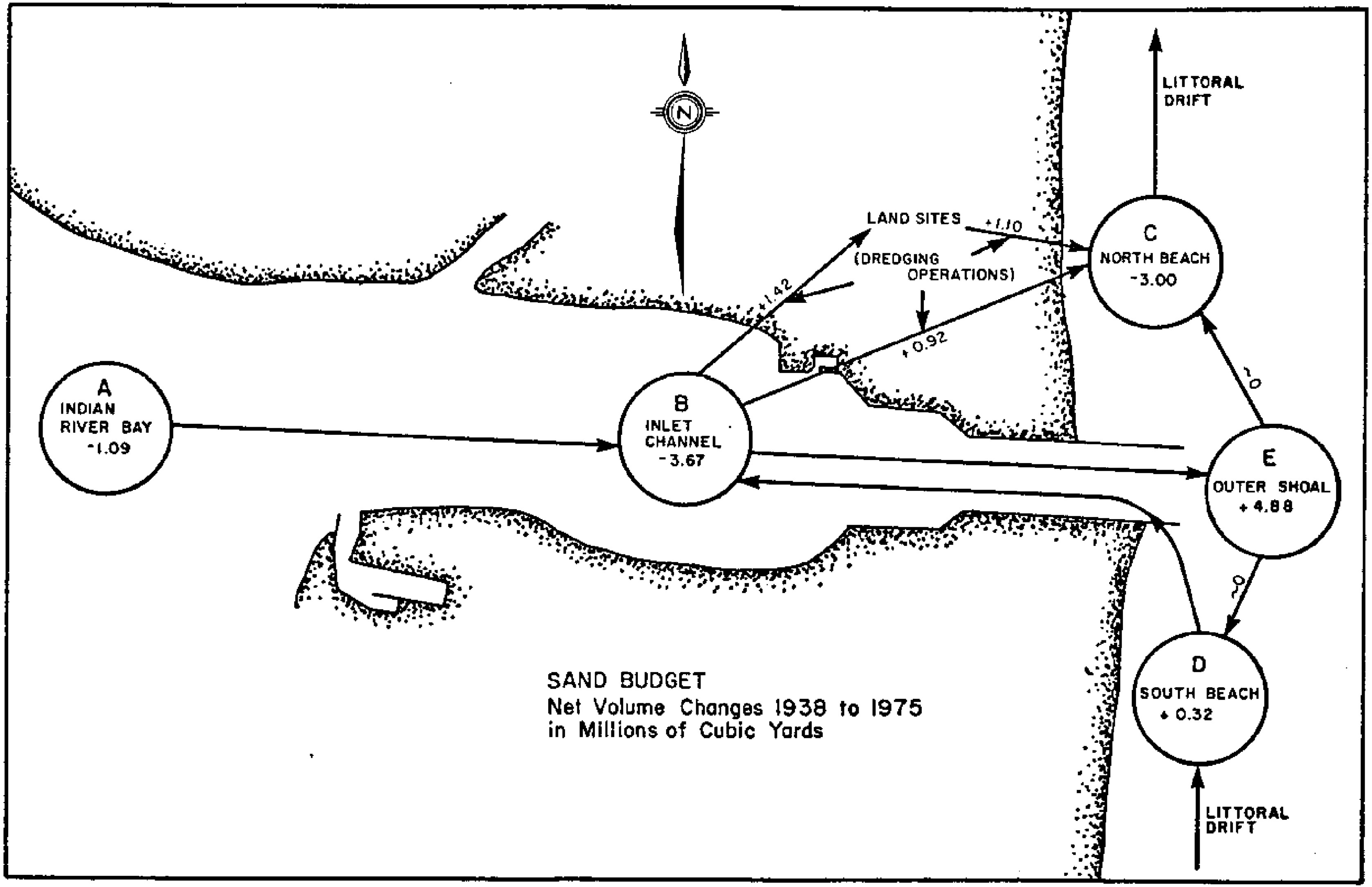


Figure 67 Sand Budget for Indian River Inlet

Beach Erosion		*Nourishment*		*Nourishment*			*Littoral Drift*

$$-3.00 \times 10^6 = 1.10 \times 10^6 + 0.92 \times 10^6 + 0 - \text{L.D.}$$

$$\text{L.D.} = 5.02 \times 10^6 \text{ c.y. or } \frac{5.02 \times 10^6 \text{ c.y.}}{37 \text{ years}} = 136{,}000 \text{ c.y./year}$$

The accuracy of the sand budget is limited by the accuracy of the volume computations. The estimated erosion from within the bay and from the north beach were not highly accurate. Errors in the sand budget would also be introduced through such things as: sand exchanged between the outer shoal and the north and south beaches, sand moving from the north beach into the inlet channel, overwash,dredge spoils reentering the channel due to channel bank erosion, errors in volume computations, errors in dredging volumes, or movement of sediment out of the study area. With this many possible sources of error, the sand budget can only be approximate. However, it is thought that the sand budget presented in this report is reasonably accurate.

If one neglects offshore sediment movement and loss of sand from the beach due to washover, the amount of sand reaching Indian River Inlet from the south should be approximately the amount of sand eroded from the beach between Indian River Inlet and the nodal area near Bethany Beach. Sand eroded from this 5-mile stretch of beach is carried northward to the inlet by littoral drift.

From 1843 to 1929, the mean high water line between Indian River Inlet and the Delaware-Maryland state line retreated at an average annual rate of nearly three feet per year. Between 1929 and 1954, the erosion was comparatively slight, with accretion on the beach south of the inlet. The recession averaged about six feet per year between 1954 and 1964 (U.S. Army Corps of Engineers, 1968). Assuming an average annual shoreline retreat of three feet per year along the five miles of beach between Indian River Inlet and Bethany Beach, and a conversion of 1 c.y. of sand per square foot of beach erosion, the littoral drift can be estimated as 79,000 c.y. per year. This compares reasonably well with the values of net littoral drift computed at Indian River Inlet from the sand budget.

Moody (1964) presented a sediment budget for the barrier and offshore areas between Indian River Inlet and Bethany Beach as shown in Table 12. Including offshore erosion and deposition, and assuming that the only sediment loss from the area south of Indian River Inlet was littoral drift carrying sand to the inlet, the littoral drift can be estimated as 92,000 cubic meters (120,000 c.y.) per year. This value compares reasonably well with the values of the littoral drift calculated in this study.

In summary, four estimates of the net littoral drift at Indian River Inlet have been presented. The values of 75,000 c.y. per year and 136,000 c.y. per year are based on the sand budget as determined

TABLE 12 SEDIMENT BUDGET FOR THE BARRIER AND OFFSHORE AREAS BETWEEN INDIAN RIVER INLET AND BETHANY BEACH, DELAWARE (cubic meters)

Sediment source	Period	Average volumetric change (cubic meters per year, "+" indicates accretion; "-" indicates erosion)
Barrier (mean low water to toe of sand barrier)	1929 - 1961	- 148,000
Sand Dunes (mean low water to top of sand dunes)	1954 - 1961	- 100,000 (estimated)
Offshore Erosion (principally on northwest side of ridges)	1919 - 1961	- 100,000
Erosion from bay inside Indian River Inlet	---	- 69,000
	TOTAL EROSION	- 417,000 cubic meters per year
Site of deposition		
Tidal Delta	1939 - 1961	+ 120,000
Barrier south of Indian River Inlet	1939 - 1961	+ 5,700
Offshore Accretion	1919 - 1961	+ 256,000
	TOTAL ACCRETION	+ 381,700 cubic meters per year

TOTAL EROSION	- 417,000
TOTAL ACCRETION	+ 318,700
NET EROSION	- 98,300 cubic meters per year

in this study. The estimated value of 79,000 c.y. per year is based on the erosion of the beach south of the inlet, and the estimated value of 120,000 c.y. per year is based on Moody's sediment budget for the onshore and offshore area south of the inlet. Averaging the two values determined from the inlet study sand budget, the net littoral drift can be estimated as 105,000 c.y. per year northward. The value of 125,000 c.y. per year, however, will be used in the computation of sand bypassing costs in the inlet maintenance chapter. This value was chosen due to the uncertainty over the exact value of the littoral drift quantity and a desire to be conservative in the estimates of costs.

CHAPTER V

INLET HYDRAULICS

During this study the tides and currents at Indian River Inlet were recorded. This information was used to determine the Manning's n and stability against closure of the inlet. The ocean currents offshore of the inlet and the interaction of these currents with the ebb tidal flow from the inlet were also investigated,as this appears to be the major mode of sediment transport to the outer shoal.

5.1 Tidal Survey

Tides were recorded during each survey. During the November, 1975 survey, a temporary tide gauge was successfully installed in the ocean near the inlet. With a tide gauge near the bay and in the ocean, tidal corrections for the hydrographic surveys, head difference between ocean and bay, difference in tide range, and phase lag from ocean to bay could be determined.

Two Stevens type F water elevation recorders were used in this study. The recorders were mounted in aluminum weather boxes having 8-inch-diameter and 10-foot-long stilling wells. The weather boxes were then chained to an appropriate vertical support, such as a wooden piling.

The tides inside the inlet were recorded at the South Shore Marina, located at about station 60+00I on the south side of the inlet. This tide gauge was leveled into bench mark S-27, mentioned previously. This bench mark provided the vertical datum for surveys done as part of this study.

The installation of the tide gauge in the ocean proved to be a fairly difficult task. The ocean tide gauge was located about 300 feet north of the north jetty and 400 feet from the beach in about 12 feet of water. The first attempt to jet a wooden pile into the bottom required considerable effort. Due to delays, only a few hours of data were collected before the conclusion of the survey. The piling was left in place to be reoccupied during the next field trip. However, the piling was lost as a result of a northeast storm between surveys.

A second installation using a steel pipe jetted into the bottom proved more successful. The weather box and stilling well were then strapped to the pipe, and the recorder was installed. This installation technique, which can be accomplished from a small boat with 3 or 4 people, is described in greater detail in Appendix 2.

In addition to the two tide gauges installed as part of this study, there were also tide gauges located at the Coast Guard Station (operated by N.O.A.A.) and at the Indian River Power Plant (operated by the Delmarva Power and Light Company). The records of these tide gauges were not continuous during the study period.

Figure 68 shows tides recorded at the South Shore Marina, the Coast Guard Station and in the ocean, between 0700 hours E.S.T., November 7 and 1700 hours November 9, 1975. The vertical datum of the ocean tide gauge was determined by assuming that the water surface elevation was the same in the ocean and in the inlet during slack waters. As expected, the water surface elevation at the Coast Guard Station is between that in the ocean and the South Shore Marina.

The elevation difference, or head drop, between ocean and bay can be seen from Figure 68. This head drop reached a maximum of 1.3 feet at 0930 hours November 7 during flood tide and had an ebb tide maximum of 1.3 feet at 1600 hours November 7 during the tidal survey. The average maximum head drop during the survey was 1.0 feet for flood tide and 0.9 feet for ebb tide. The largest portion of the head drop is between the ocean and the Coast Guard Station.

The difference in tidal range between the ocean and the bay (South Shore Marina) can also be seen from Figure 68. The average ratio of ocean tidal amplitude to bay tidal amplitude was 1.63 to 1 during the tidal survey. The tidal amplitude at the Coast Guard Station was slightly higher than that in the bay.

The tidal phase in the bay lags behind that of the ocean. The high tide at the South Shore Marina was an average of one hour

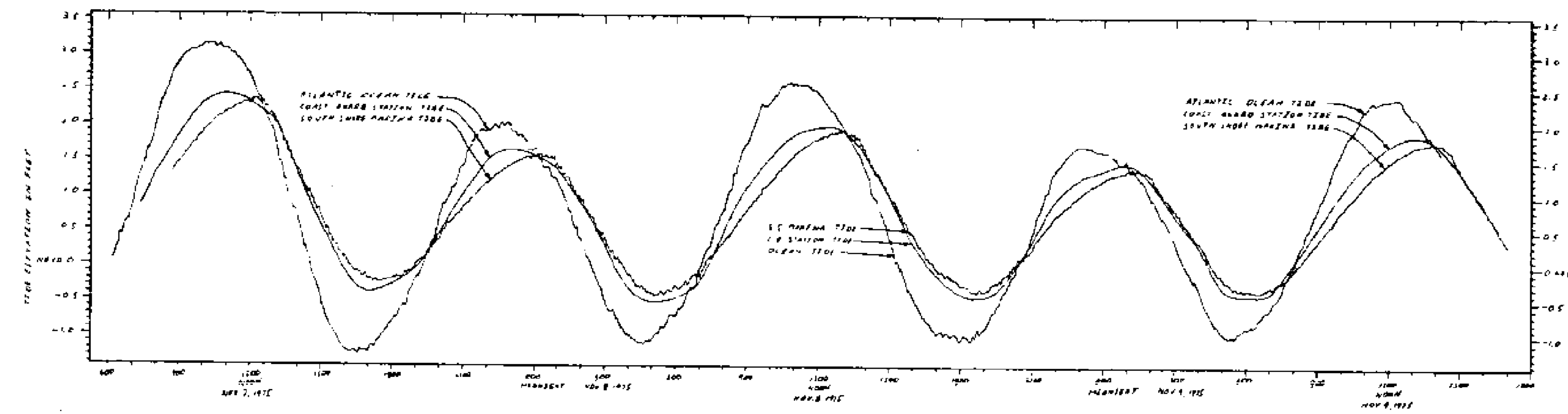


Figure 68 Tidal Record in the Ocean, Coast Guard Station and South Shore Marina

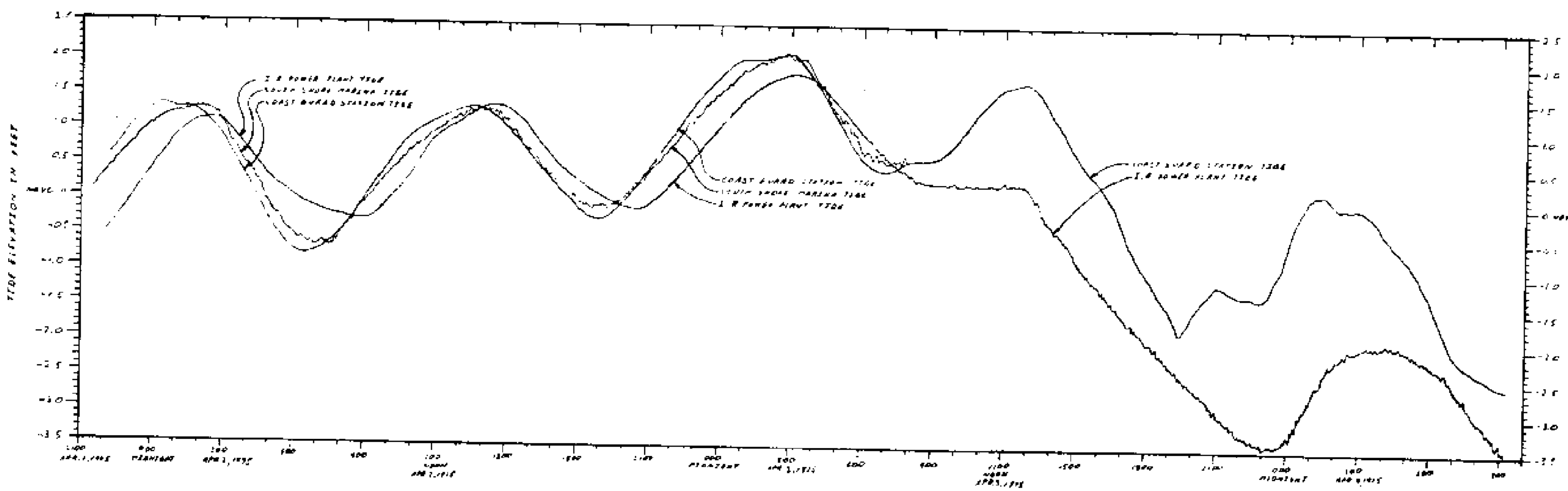


Figure 70 Tidal Record at the Coast Guard Station, South Shore Marina and Indian River Power Plant. Note Response to Strong Westerly Wind.

55 minutes later than the high in the ocean. The low tide was an average of 55 minutes after the low in the ocean. The time of slack water in the inlet, however, lagged behind the ocean high or low tide by an average of 2 hours and 55 minutes.

The tide in Indian River Bay rises significantly above its lowest value before the water surface is the same level in the ocean and bay, and the current goes slack. In other words, the tide is rising in the bay while the water in the inlet is still flowing out! Similarly, the high water slack occurs after the tide has begun to fall in the bay. In an ideal bay, with the only inflow being through its opening to the ocean and a uniform tide elevation throughout the bay, the low tide in the bay corresponds to low water slack in the inlet (Keulegan, 1967 and King, 1974). The lowest tide in the bay corresponds to the time at which the most water has left the bay, low water slack. In his 1967 report, Keulegan showed a tidal record taken in 1948 which displayed a trend similar to that observed during this study. However, he did not comment on the cause of the slack waters not coinciding with high and low tides at the bridge. Also, his ocean tide was approximated from Atlantic City, New Jersey and was not actually recorded in the ocean near Indian River Inlet.

In the case of low tide in eastern Indian River Bay, for the tide to begin to rise, more water must be entering this part of the bay than leaving. Since there is still ebb flow in the inlet, the water must be coming from somewhere else, presumably from

Rehoboth Bay through the Ditches. When Keulegan considered the effects of flow from Rehoboth Bay, he predicted tides as shown in Figure 69. Tides similar to those predicted by Keulegan for Indian River and Rehoboth Bays have been recorded in another two-bay system in Florida (Shemdin and Forney, 1970).

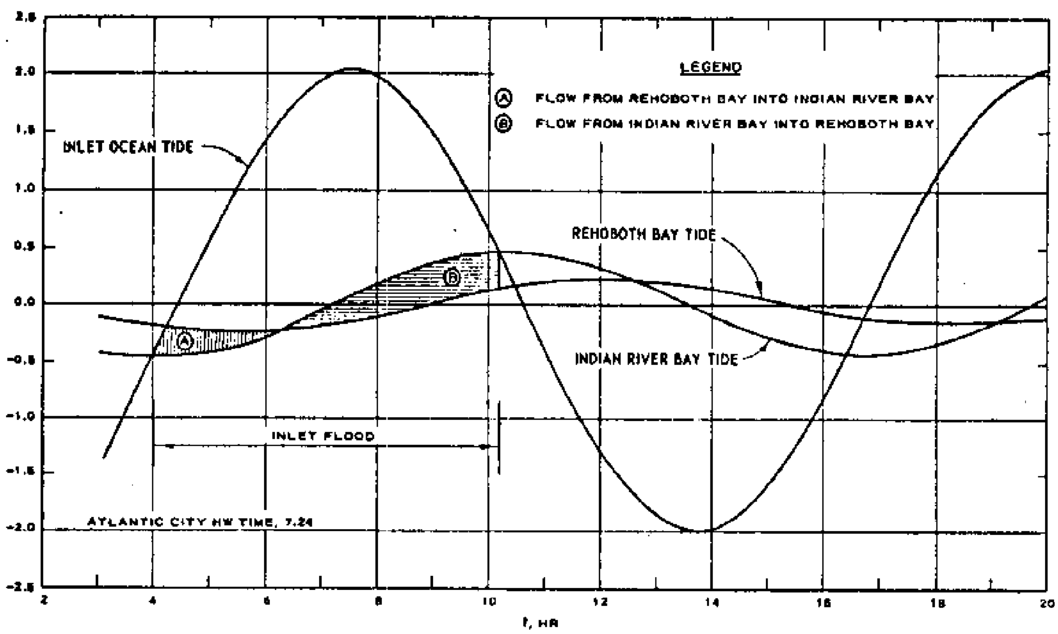


Figure 69 Predicted Tides in the Ocean, Indian River Bay and Rehoboth Bay (Keulegan, 1967)

Figure 70 shows the tidal record at the Coast Guard Station, the South Shore Marina, and the Indian River Power Plant from April 2 to April 4, 1975. The normal tidal relations are depicted in the first portion of the record. There is a phase lag between the eastern end of Indian River Bay and the western portion; also, there is some decrease in tidal height. The record for April 3 and 4 shows the response of the bay to a prolonged 50 m.p.h.-plus westerly wind. There was a large set up across the shallow bay and abnormally low tides were recorded.

5.2 Inlet Currents

The currents in Indian River Inlet were studied because of their importance in moving sediment and their effects on navigation. The ocean tides generate strong tidal currents in the inlet which frequently exceed six feet per second (3.6 knots). Probably the most interesting aspect of the tidal currents is the chop at the tips of the jetties. This phenomenon can be very hazardous to small boats.

During ebb tide, ocean waves approach the inlet channel and interact with the strong tidal current. Upon encountering the current between the tips of the jetties, the wave speed and consequently the wave length decrease greatly. The waves generally continue slowly upstream but they also steepen and break. Waves can sometimes be seen propagating westward past the highway bridge, but their heights are greatly reduced after they have broken in the chop. Such wave-current interactions are discussed by Phillips (1966) and Dalrymple and Dean (1975).

Negotiating the entrance of the inlet becomes very hazardous for small boats during ebb tide when there is a moderate surf. Many people fish in Indian River Inlet from small boats which are allowed to drift with the tidal current. Upon reaching the highway bridge (on ebb tide), they start their motor and proceed west, back toward the bay. Should the motor fail to start, however, it only takes about 2 minutes to drift from the bridge into the chop. This allows very little time to effect repairs.

Pictures of one of the numerous annual small boat swampings were taken in June, 1976. Figure 71 shows a boat (1) in such a predicament. An attempt to drop an anchor and an attempt to get a tow rope from boat number 2 both failed. On passing through the first wave, two of the three occupants were thrown from the boat, and the boat was filled about one-third full of water. Boat number 2 (which had attempted to aid the other boat) also passed through the wave but fared much better. Figure 72 shows both boats just after they have passed through the wave. Figure 73 shows the swamped boat continuing to drift through the chop. Boat number 2 summoned a Coast Guard boat which quickly responded and pumped out the swamped boat. The small boat was almost swamped again while being towed in by the Coast Guard.

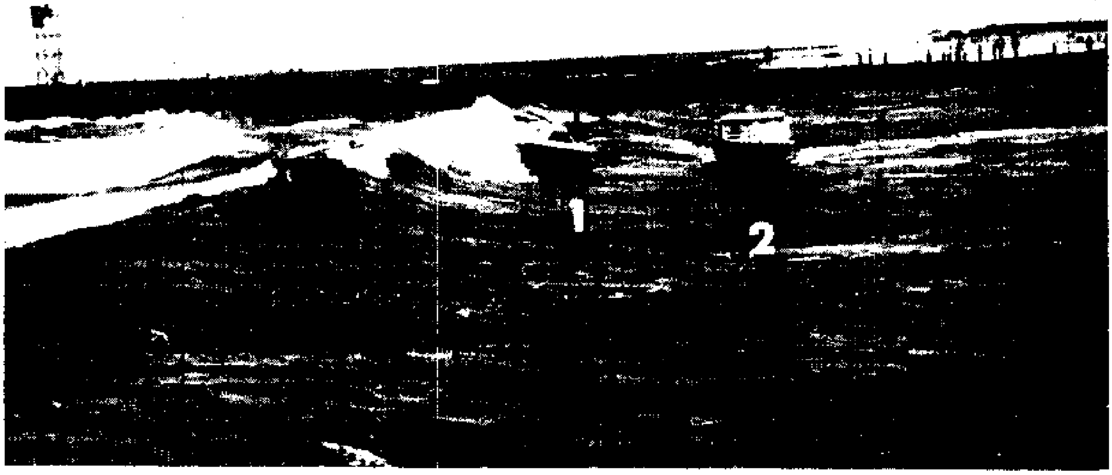


Figure 71 Boat Being Carried into the Chop Without Power. Number 2 Boat Had Attempted a Rescue.

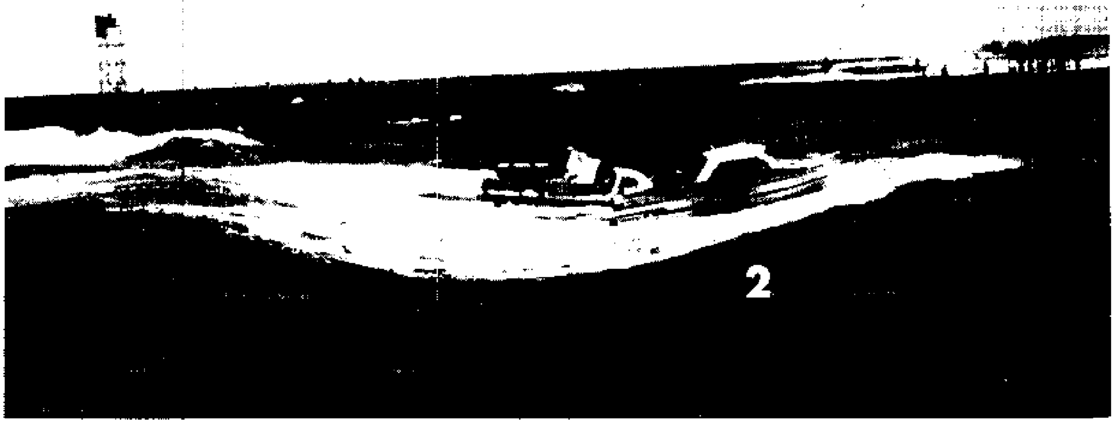


Figure 72 Both Boats After Passing Through First Wave

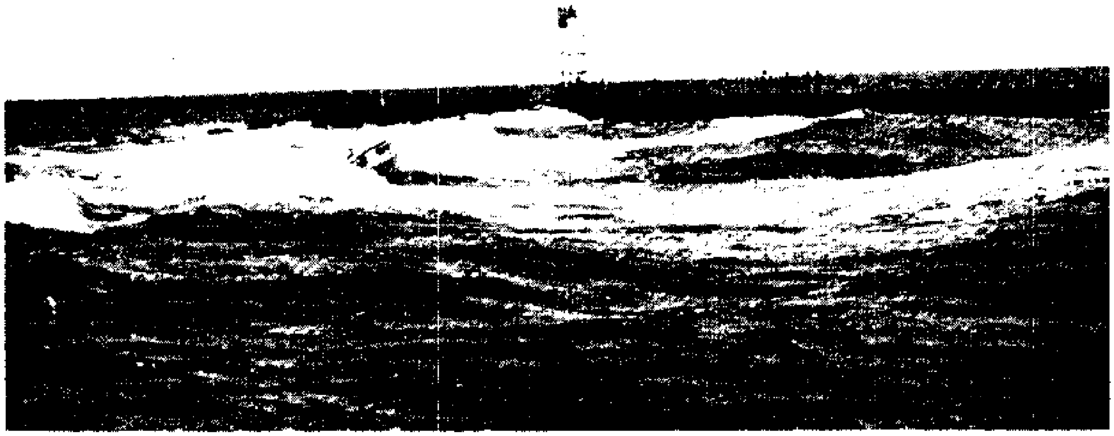


Figure 73 Swamped Boat Continuing to Drift Through the Chop

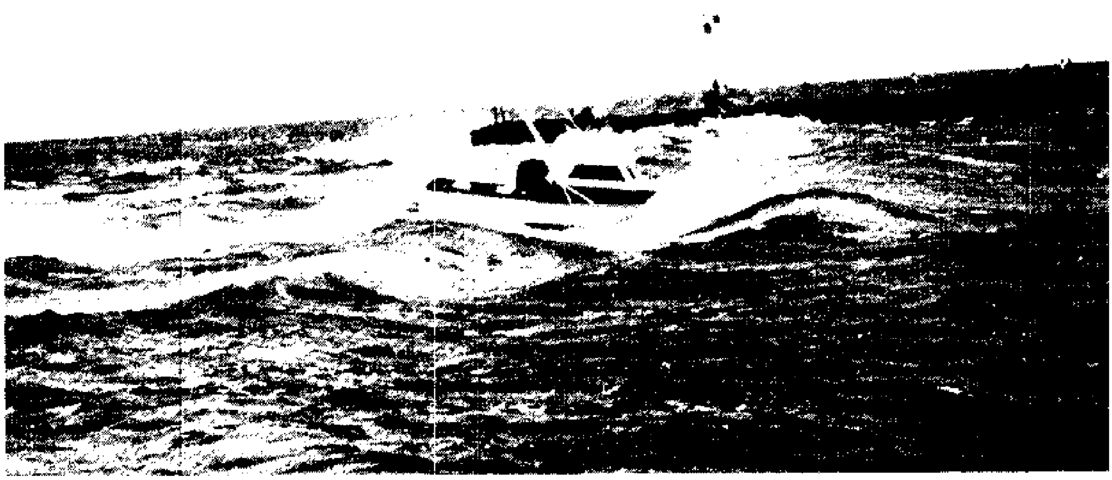


Figure 74 Boat Entering Inlet Through Chop. The Bow has Been Forced Beneath the Wave in Front of the Boat.

Entering the inlet through the chop can also be hazardous. Figure 74 shows a boat which was proceeding into the inlet too slowly. A wave approaching from behind the boat lifted the stern and forced the bow beneath the wave in front of the boat. In this position, an open boat would take on a considerable amount of water. In addition, the boat is quite susceptable to broaching and subsequently being capsized. A boat moving faster than the waves is also exposed to similar hazards. One good way to enter the inlet through the chop is to place the bow of the boat on the crest of a wave and follow that wave through the chop. The water is also usually calmer near the jetties than in the center of the channel. Boats operating near the jetties, however, should exercise caution to avoid submerged rocks.

Currents were measured in the inlet on several occasions as part of this study. Drogues and dye were deployed by boats and then monitored with aerial photography. Figure 75 shows a streak of red dye released from a small buoy. (The large patch of dark water near the shore was released from the small sewage treatment facility at the bottom of the picture.) The drogue and dye studies, however, provided little more than qualitative information on the tidal currents. This was due to problems with transferring the location of the drogues and dye to charts. Also, information could only be obtained while the airplane was overhead; this amount of time was limited by cost.

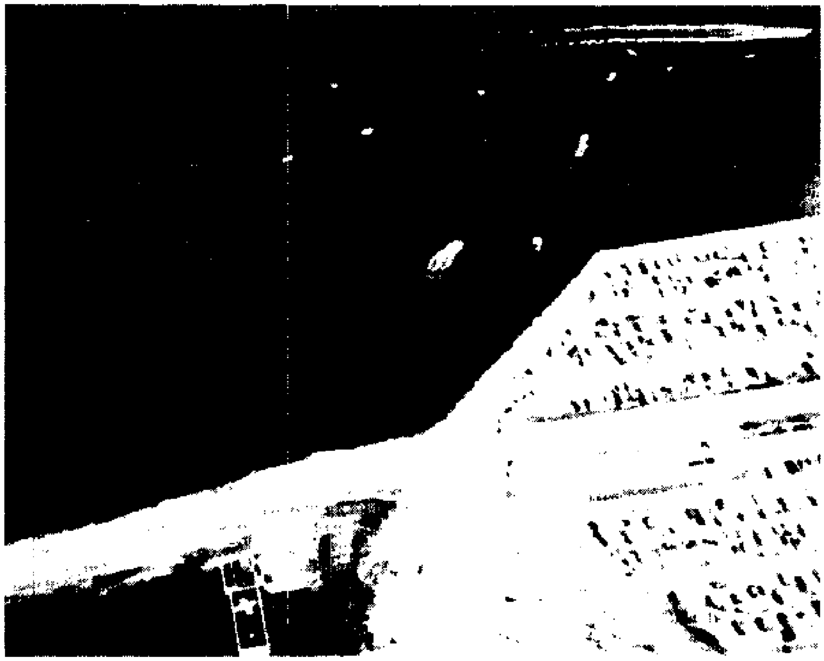


Figure 75 Red Dye Streak and Sewage Treatment Plant Effluent

Currents were also measured with a hand-held current meter. The Ott current meter was held over the side of a small boat while the boat was held at a fixed location. Currents could be determined in this manner at various locations with little difficulty.

Figure 76 shows currents measured in the inlet on November 8, 1975. The currents were measured near the surface of the water in mid-channel at station 20+00I. The head drops from the ocean to the South Shore Marina, from the ocean to the Coast Guard Station, and from the Coast Guard Station to the South Shore Marina are also included in the figure. The currents can be seen to lag behind the head differential which drives them. The slack water in the inlet is after the time at which there is zero head difference.

The maximum current recorded in the inlet during this study was 7.85 feet per second. At the time of this flood current reading, there was a head drop of 1.2 feet from the ocean to the South Shore Marina. The velocity across the inlet throat is nearly constant except near flow constrictions. The vertical velocity profile was not measured, but there is probably little variation except very near the bottom. This is due to the highly turbulent, non-stratified nature of the flow.

The tidal prism of Indian River Bay and Rehoboth Bay passing through Indian River Inlet was estimated in two ways. First, the tidal range listed in the tide tables (N.O.A.A., 1975b) was multiplied by the surface area of the bays. This method requires the assumption that the bays rise and fall uniformly and that all flow is through the inlet. Also, the accuracy of the tidal ranges presented in the tide tables is questionable. The spring tidal prism was estimated in this manner to be 8.3×10^8 ft^3.

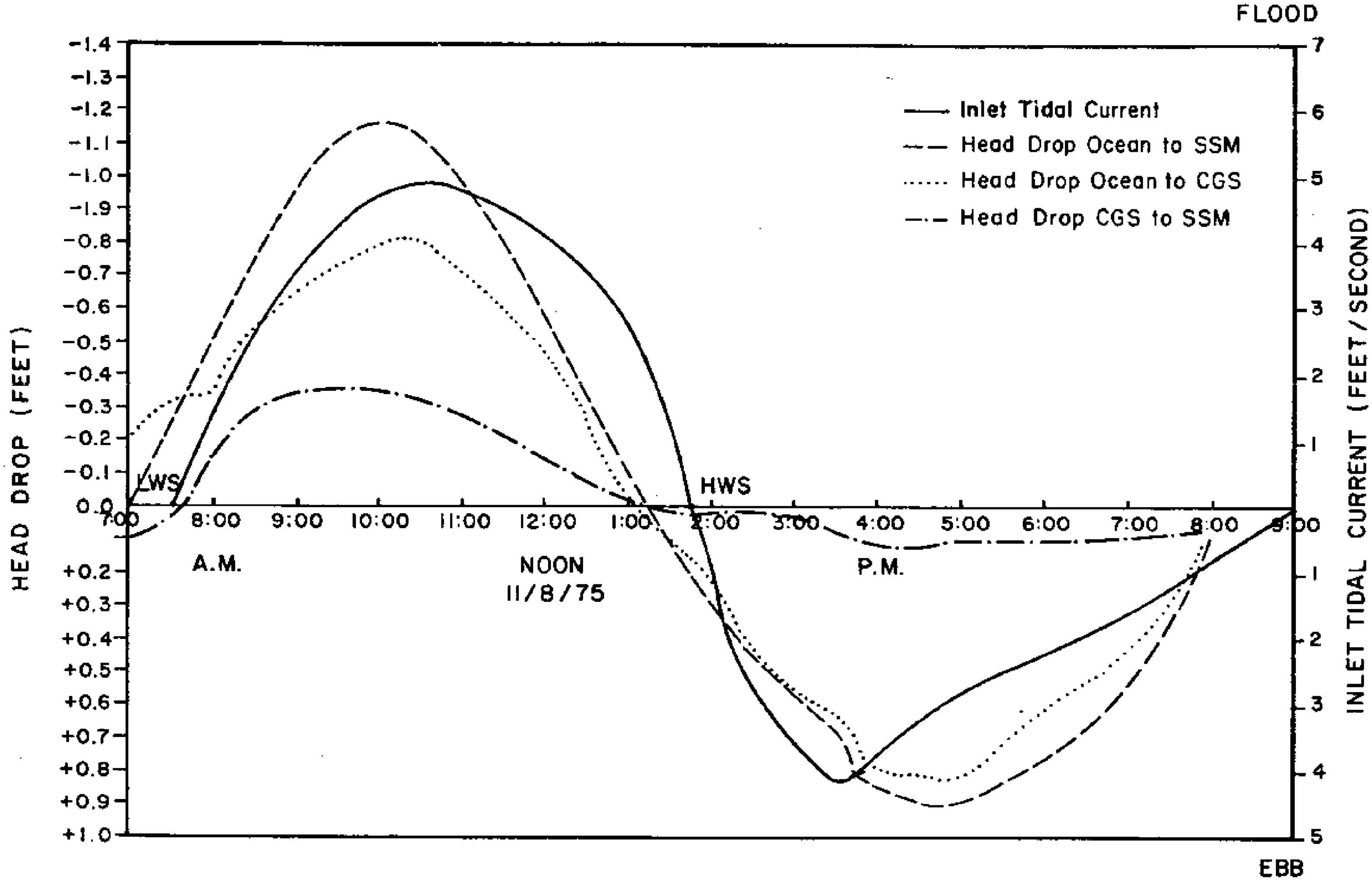


Figure 76 Inlet Tidal Current and Head Drops November 8, 1975

Even though the current readings in Figure 76 are not complete, they were used to obtain another estimate of the inlet tidal prism. The current velocities were integrated over a tidal cycle and multiplied by the cross-sectional area at station 20+00I. A value of approximately 10×10^8 ft^3 was obtained in this manner. On the basis of these two values, the tidal prism is estimated to be 9×10^8 ft^3.

The tidal prism of the bays has increased with the increase of the inlet's cross-sectional area. This greater exchange of bay water for cleaner ocean water is helpful in maintaining the water quality of the bays. The present trend of increased development and use of the land surrounding the bays could lead to the introduction of more pollutants. One possible method of improving the water quality in Rehoboth Bay would be to increase the exchange of water with the ocean by establishing a new inlet connecting the bay to the ocean. Such an inlet should not be considered, however, without extensive preliminary study. Problems similar to those found at Indian River Inlet could be expected to occur at the new inlet, and the new inlet would change some conditions at the existing inlet. Another means of reducing pollution would be the use of ocean outfalls for sewage treatment plants (Ditmars, 1974).

Closer investigation of tide and current data can yield additional information on the tidal hydraulics of Indian River Inlet. Knowing the water surface elevation in the ocean, at the Coast Guard Station, CGS, and at the South Shore Marina, SSM, and the velocity of the current between these points, the Manning's roughness, n, can be determined for the inlet. The head at one point is equal to the head at another point plus the summation of all the head losses encountered by flow between the points (Daily and Harleman, 1966 and Chow, 1959).

The ebb tidal flow at Indian River Inlet encounters only friction loss between the South Shore Marina and the Coast Guard Station. Between the Coast Guard Station and the ocean, the flow has friction losses, losses due to the contraction of the inlet, and an exit loss as the flow leaves the jetties. The ebb tidal head equation can be written approximately as follows:

SSM to Ocean $\qquad L = 5{,}450$ ft. $\qquad r_{avg} = 16.55$ ft.

$$h_{ssm} + \frac{V_{ssm}^2}{2g} = h_o + \frac{V_o^2}{2g} + \underbrace{.20\left(\frac{V_{thr}^2}{2g} - \frac{V_{ssm}^2}{2g}\right)}_{\textit{Contraction Loss}} + \underbrace{V_{avg}^2 \frac{n^2}{1.49^2} \frac{L}{r_{avg}^{4/3}}}_{\textit{Friction Loss}} + \underbrace{\frac{V_{thr}^2}{2g}}_{\textit{Exit Loss}}$$

SSM to CGS $\qquad L = 3{,}000$ ft. $\qquad r_{ssm} = 12$ ft.

$$h_{ssm} + \frac{V_{ssm}^2}{2g} = h_{cgs} + \frac{V_{ssm}^2}{2g} + V_{ssm}^2 \frac{n^2}{1.49^2} \frac{L}{r_{ssm}^{4/3}}$$

CGS to Ocean $\qquad L = 2{,}450$ ft. $\qquad r_{thr} = 22$ ft.

$$h_{cgs} + \frac{V_{ssm}^2}{2g} = h_o + \frac{V_o^2}{2g} + .20\left(\frac{V_{thr}^2}{2g} - \frac{V_{ssm}^2}{2g}\right) + V_{thr}^2 \frac{n^2}{1.49^2} \frac{L}{r_{thr}^{4/3}}$$

$$+ \frac{V_{thr}^{2}}{2g}$$

where h is the water surface elevation, V is the water velocity, L is the length of flow, r is the hydraulic radius, g is the acceleration due to gravity, and n is Manning's n. The subscript "thr" refers to the throat of the inlet, station 20+00I, the subscript "ssm" refers to the portion of the channel near the South Shore Marina, station 60+00I, the subscript "o" refers to the ocean and the subscript "avg" refers to the average value of a parameter for the entire length of the inlet channel, station 5+50I to 60+00I. The variation in velocity within the inlet is due to the larger cross-sectional area of the western portion of the channel.

The flood tidal current experiences an entrance loss, similar to that of a Borda mouthpiece, as it enters the inlet from the ocean. It also has a head loss, due to the expansion as it proceeds through the inlet, and friction losses. The flood tidal head equations can be written approximately as follows:

<u>Ocean to SSM</u> $\quad L = 5{,}450$ ft. $\quad r_{avg} = 16.55$ ft.

$$h_o + \frac{V_o^{2}}{2g} = h_{ssm} + \frac{V_{ssm}^{2}}{2g} + .4\,\frac{V_{thr}^{2}}{2g} + .50\,\left(\frac{V_{thr}^{2}}{2g} - \frac{V_{ssm}^{2}}{2g}\right) + V_{avg}^{2}$$

Entrance Loss *Expansion Loss*

$$\frac{n^2}{1.49^2}\,\frac{L}{r_{thr}^{4/3}}$$

Friction Loss

Ocean to CGS $L = 2{,}450$ ft. $r_{thr} = 22$ ft.

$$h_o + \frac{V_o^2}{2g} = h_{cgs} + \frac{V_{ssm}^2}{2g} + .4\,\frac{V_{thr}^2}{2g} + .50\left(\frac{V_{thr}^2}{2g} - \frac{V_{ssm}^2}{2g}\right)$$

$$+ V_{thr}^2\,\frac{n^2}{1.49^2}\,\frac{L}{r_{thr}^{4/3}}$$

CGS to SSM $L = 3{,}000$ ft. $r_{ssm} = 12$ ft.

$$h_{cgs} + \frac{V_{ssm}^2}{2g} = h_{ssm} + \frac{V_{ssm}^2}{2g} + V_{ssm}\,\frac{n^2}{1.49^2}\,\frac{L}{r_{ssm}^{4/3}}$$

Applying these equations at times of known head differences and current velocities, one can solve for n. Table 13 shows values of n computed for Indian River Inlet. The computed values show considerable variation. Keulegan (1967) computed the value of n as 0.046 between the ocean and the old bridge. He stated that this value was rather high for a channel and postulated that his high value was due to errors in the approximated ocean tide and resistance to flow from the bridge piers.

Keulegan described the flow characteristics of an inlet with a coefficient of repletion, K. K can be determined from the following equation:

$$K = \frac{T\sqrt{2gH}}{2\pi H}\sqrt{\frac{r}{\lambda L + r}}\;\frac{a}{A}$$

TABLE 13 VALUES OF MANNING'S n FOR INDIAN RIVER INLET

Date	Time	Tide	n Ocean to South Shore Marina	n Ocean to Coast Guard Station	n Coast Guard Station to South Shore Marina
11/8/75	1425	Ebb	.045	.057	.019
11/8/75	1555	Ebb	.042	.049	.031
11/7/75	1415	Ebb	.037	.045	.023
11/8/76	1015	Flood	.037	.038	.040
11/8/75	1115	Flood	.034	.035	.037
11/7/75	0945	Flood			.027
Average		Ebb	.041	.051	.024
Average		Flood	.036	.037	.035
Average		Total	.039	.045	.029

where a is the cross sectional area of the inlet, A is the surface area of the bay, g is the acceleration due to gravity, H is 1/2 the ocean tidal ranges, L is the length of the inlet, r is the hydraulic radius of the inlet channel, T is the tidal period and

$\sqrt{\lambda} = \dfrac{n\sqrt{2g}}{1.486\ r^{1/6}}$ where n is Manning's n for the inlet.

Using the following values for Indian River Inlet:

$a = 9{,}660\ ft^2$

$A = 4.2 \times 10^8\ ft^2$

$g = 32.2\ ft/sec^2$

$H = 2.05\ ft/sec^2$

$L = 5{,}500\ ft$

$r = 12.24\ ft$

$T = 12.42$ hr or 4.47×10^4 sec

and $n = 0.046$.

Keulegan calculated K to be 0.254.

Knowing K, the height of mean bay tidal range, H_{1m}, the phase lag of the tide, α, and the maximum current in the inlet, V_m, can be predicted from the following equations:

$$H_{1m} = 2H \sin \tau$$

$$\alpha = \pi/2 - \tau$$

$$V_m = 2\ \pi C \frac{A}{a} \frac{H}{T} \sin\tau$$

where: when K < 0.3, $\sin\tau = 1.14$ K and C = 0.812; and when K > 0.3, $\sin\tau$ and C can be determined from Figure 77. Keulegan's theory thus predicted H_{1m} to be 1.19 ft, α to be 1.276 radians or 2.52 hours, and V_m to be 2.94 ft/sec. These values compared well with measured values for 1948. Keulegan also determined a correction to include the effects of Rehoboth Bay.

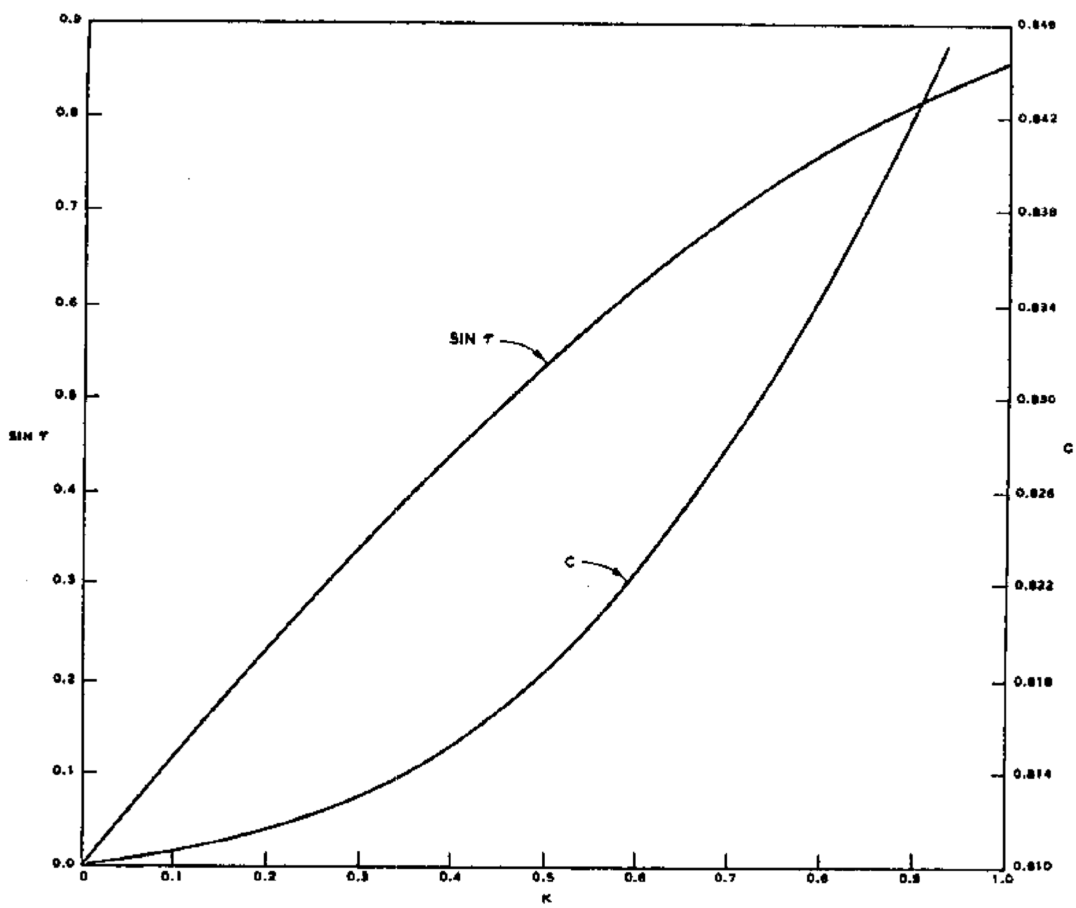


Figure 77 Sin τ and C Versus Repletion Coefficient, K (Keulegan, 1967)

As mentioned previously, however, the inlet's characteristics are changing with time. The average cross-sectional area is now (September, 1974) 19,800 ft^2 and the average hydraulic radius is 16.55 ft. Also, the importance of Rehoboth Bay to Indian River Inlet would be expected to increase if the cross-sectional area of the Ditches is increasing. The new value of K found by using the current value of cross-sectional area and hydraulic radius, an ocean half tidal range of 1.9 ft, a value of 0.039 as an average Manning's for the inlet (from Table 13), and a surface area of Indian River Bay of 4.11×10^8 ft^2, is K = 0.766. This value of K corresponds to a predicted value of H_{1m} = 2.82 ft, α = 0.738 radians or 1.46 hours, and V_m = 3.42 ft/sec. These values do not compare well with observed values.

When the area of both bays (8.14×10^8 ft^2) is used to compute the repletion coefficient, K = 0.387. The theory predicts H_{1m} = 1.63 ft, α = 1.13 radians or 2.24 hours, and V_m = 3.88 ft/sec. The bay tidal range and phase lag thus predicted are comparable to those found in Indian River Bay, but the theoretical maximum velocity in the inlet, V_m, is much lower than the maximum current values measured in the throat of the inlet. The theoretical maximum current values are also considerably lower than current values for the average channel cross section (computed as follows: $V_{max} = \frac{A_{thr} V_{thr}}{A_{avg}}$). In addition to the variation of current velocities along the length of the channel, there is also some variation in velocity across the inlet due to the channelization of the flow into the deeper areas, expecially in the inner portion of the channel. This is one of the reasons that the current readings were made at the throat of the inlet where there was very little cross-channel variation in currents.

Several researchers have made improvements upon Keulegan's work. Keulegan made the following assumptions: the bay shores are nearly vertical, the inflow from streams is negligible, the ocean tide can be approximated as a sine wave, the water level varies uniformly throughout the bay, the inlet channel is prismatic, the inlet channel depth is large with respect to tidal variation, and the inertia effect is negligible. Most of these assumptions are quite reasonable when applying the theory to Indian River Inlet. Improvements upon Keulegan's theory made by other researchers have been mostly extentions of his original work. Parameters such as variable bay area, inflow from streams, variable inlet depth, and inertial effects (for which Keulegan made simplifying assumptions) have been incorporated into the theory.

C. J. Huval and G. L. Wintergerst (1971) extended Keulegan's work to include the effects of variable bay area and applied their theory to Indian River Inlet. They assumed a bay area of 18 $mi.^2$ at high water and 12 $mi.^2$ at low water with a linear relationship between bay area and bay tidal height. In actuality, the bay area varies very little from high to low tides as Keulegan assumed. Thus, their computed bay tide range of 1.26 ft, maximum inlet current velocity of 3.12 ft/sec and 3.15 ft/sec for flood and ebb tide, respectively, and phase lag of 2.54 hours are not as accurate as the values Keulegan computed.

Keulegan computed the maximum difference in head due to inertia between the ocean and the bridge at Indian River Inlet to be 0.018 ft. in 1948. He further stated that this value was very small and neglected it in his subsequent computations. This inertia effect is a sloping of the inlet water surface which balances the acceleration of the water in the inlet due to varying tidal currents. This acceleration and resultant head difference are at maximum at the time of slack water in the inlet.

The inertia effect increases with increasing current velocities in the inlet. Using Keulegan's equation and current and tide measurements made during this study, the maximum head difference can be computed as 0.035 ft. between the ocean and the bridge. Between the ocean and the South Shore Marina, this head difference reaches a theoretical value of 0.098 ft. This computed value composes a significant portion of the observed head difference of up to 0.3 ft. at slack waters.

D. B. King (1974) included the effect of inertia in his study of tidal flow. While he found that the maximum bay tide occurred when the velocity reached zero in the inlet, this slack water occurred after the head difference became zero. This latter phenomenon is seen at Indian River Inlet. He proposed that the inertia effect changes the value of K. Using his equation, the new value of 5.41 is found for K. This value is much higher than that found by Keulegan's method and predicts $H_{1m} = 3.8$ ft, $\alpha =$ .035 radians or 0.07 hours, and $V_m = 6.14$ ft/sec. While this current is close to the observed values in the channel, the bay tidal range and phase lag are off considerably.

5.3 Inlet Stability

Knowledge of an inlet's ability to resist closure due to deposition of sand in the channel is important to the coastal engineer and has been under study for many years. Inlet stability is discussed in a recent paper by M. P. O'Brien and R. G. Dean (1972). The stability curve, as presented by F. F. Escoffier (1940) is discussed, and a stability coefficient is introduced.

Figure 78 illustrates Escoffier's stability concept. Inlets with cross-sectional areas to the right of the peak maximum velocities are stable and resist decreases in their cross-sectional areas with an increase in maximum velocity. Inlets to the left on the curve are unstable. The right portion of the curve is governed by continuity with velocities decreasing with increasing cross-sectional area, and the left portion of the curve is governed by friction with velocities decreasing with decreasing cross-sectional area.

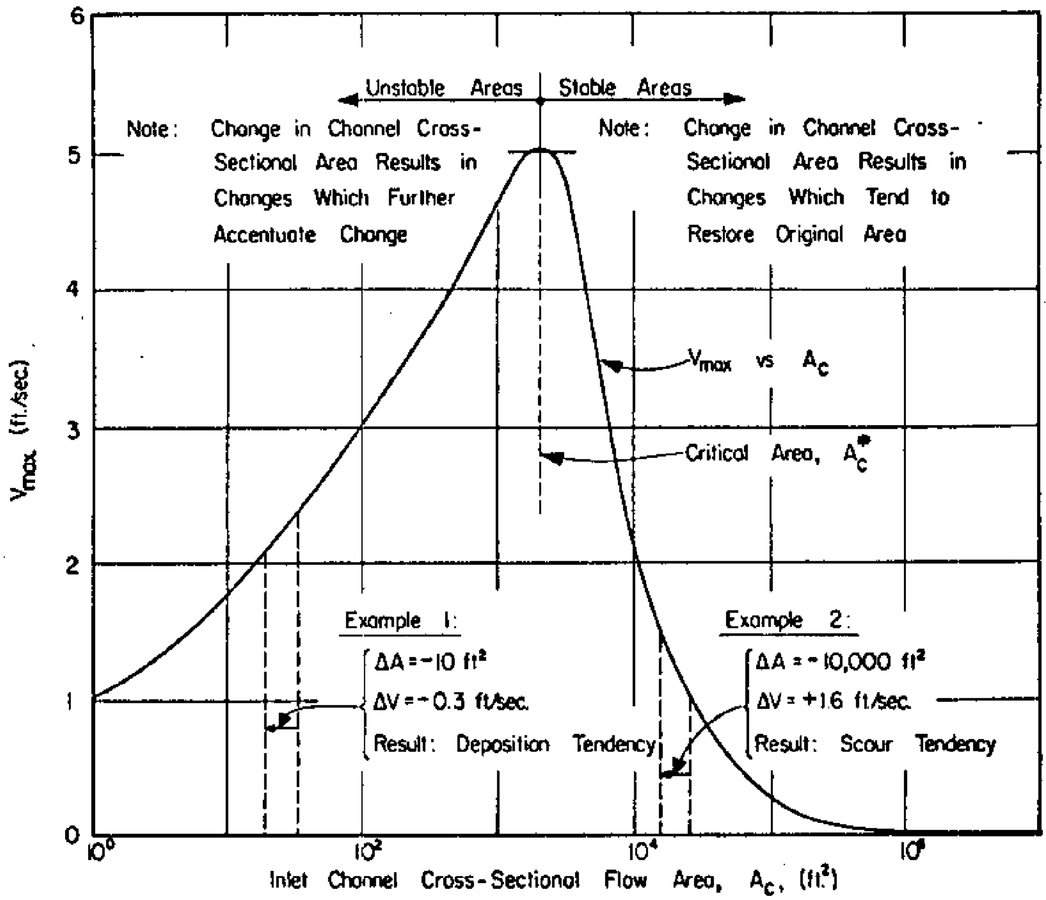


Figure 78 Illustration of Escoffier's Stability Concept (O'Brien and Dean, 1972)

The changing channel dimensions of Indian River Inlet have caused the repletion coefficient and theoretical maximum velocity of the inlet to change. Presumably, the entrance and exit loss coefficients and the friction factor have also changed with time.

To simplify the computations, the entrance and exit loss coefficients were assumed constantly equal to 1.3, ($K_{en} + K_{ex} = 1.3$), and the friction factor was assumed to equal 0.03, ($f = 0.03$). In the expression for the repletion coefficient, K, as presented by O'Brien and Dean:

$$K = \frac{T\sqrt{2gH}}{2\pi H}\frac{a}{A}\sqrt{K_{en} + K_{ex} + \frac{f\ell}{4r}} \quad .$$

O'Brien and Dean also combined the C and sin τ terms from Keulegan (1967) into a single term, V'_{max}. Thus,

$$V_{max} = V'_{max}\frac{2\pi}{T}H\frac{A}{a} \quad .$$

Table 14 shows computed values of K and V_{max} for the inlet channel for various years. V_{max} is then plotted versus average cross-sectional area in Figure 79. The inlet has progressed from the unstable portion of the curve, through the critical area and into the stable portion. The points corresponding to cross-sectional areas larger than the present cross-sectional area were computed for the inlet deepening but not widening. The solid line was computed based on a constant rectangular cross-sectional shape, the width 20 times the channel depth. This curve is smoother than the curve based on the inlet conditions. Also, the critical area and the peak maximum velocity are different from values based on historical inlet conditions.

TABLE 14 AVERAGE HYDRAULIC PARAMETERS OF INDIAN RIVER INLET FOR VARIOUS YEARS AT +2 FEET MLW

Date	Depth	Width	Cross-Sect. Area, A_c (ft^2)	Hydraulic Radius, r (ft)	Repletion Coefficient k	Predicted Max. Velocity, V_{max} (ft/sec)	Comment
July 1928	4	60	240	3.53	.0061	2.86	Dredged Channel
June 1929	4.5	100	450	4.13	.0122	3.05	
Nov. 1929	8	60	480	6.32	.0157	3.68	Dredged Channel
Jan. 1936	3.41	380	1,300	3.35	.0323	2.78	
Oct. 1938	15	200	3,000	13.04	.1297	4.86	Dredged Channel
Dec. 1939	9.26	690	6,390	9.02	.241	4.24	
Feb. 1943	8.95	860	7,660	8.77	.286	4.20	
1947	10.22	910	9,280	10.00	.364	4.32	
1948	10.04	1,010	10,130	9.85	.395	4.29	
Mar. 1950	11.74	940	11,080	11.45	.457	4.41	
Mar. 1967	13.22	1,140	15,080	12.92	.650	4.35	
Sept 1974	17.03	1,160	19,800	16.55	.928	4.34	
Nov. 1975	18.15	1,160	21,090	17.60	1.009	4.25	
	19.57	1,180	23,000	18.94	1.126	4.12	Values Predicted
	25.53	1,180	30,000	24.47	1.58	3.65	Assuming Channel
	34.04	1,180	40,000	32.18	2.27	2.97	Deepening But No
	42.55	1,180	50,000	39.68	2.98	2.43	Further Widening
	59.57	1,180	70,000	54.09	4.44	1.74	

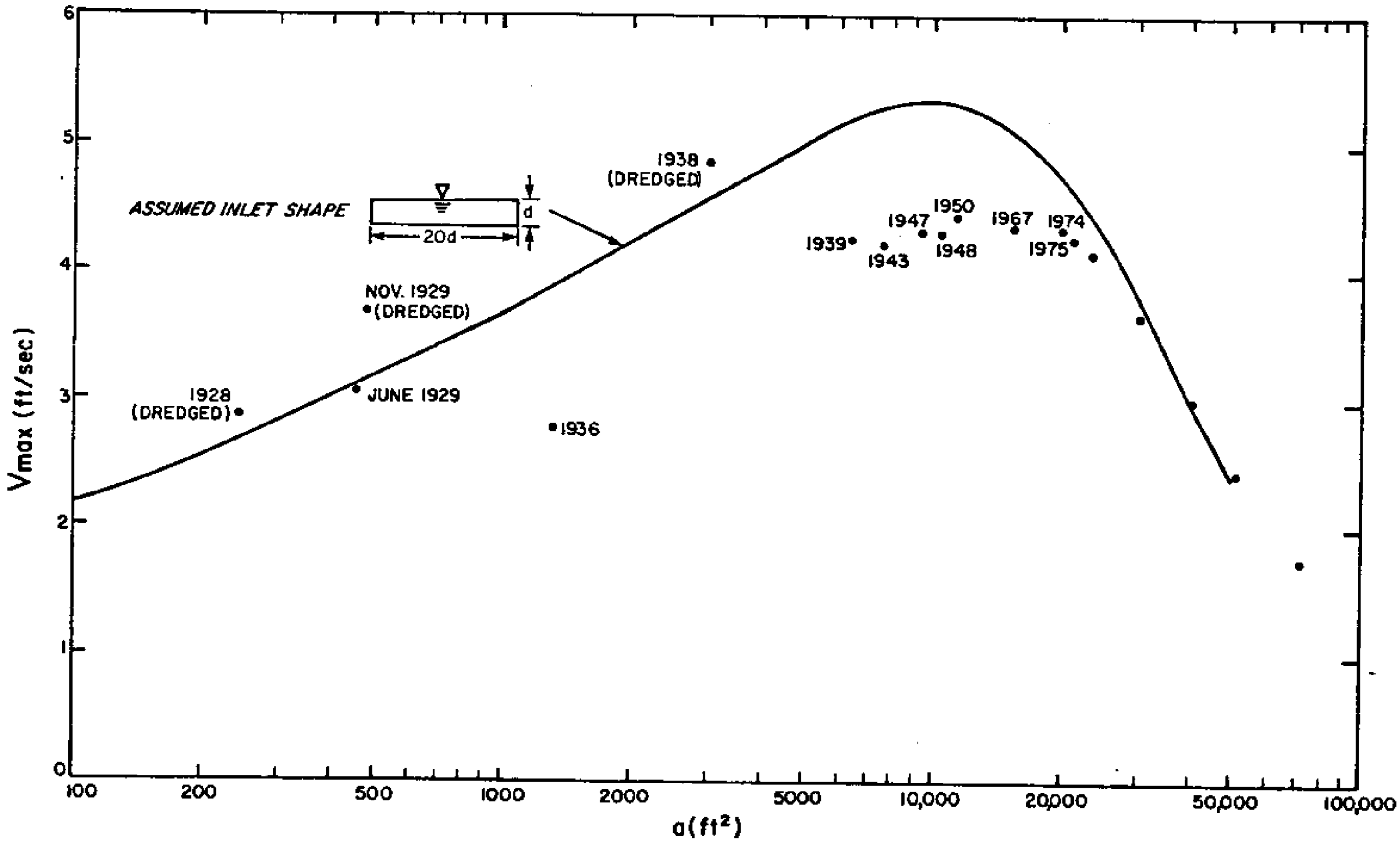


Figure 79 Stability Curve for Indian River Inlet for Historical Cross Sections and Assumed Cross Section

The early inlet channel cross sections placed them on the left-hand portion of the curve in Figure 79. The 1928, June, 1929, November, 1929 and 1936 channels are clearly on the unstable portion of the curve. The 1938 through 1975 channel dimensions place the inlet near the critical area or on the stable portion of the curve. The stability curve was able to predict the unstable condition of the early inlets (the inlet before 1938 was discussed in the history section). The role of the jetties (installed in 1938) in stabilizing the inlet must also be considered, however.

The stability coefficient proposed by O'Brien and Dean is the integral from the critical cross-sectional area to some equilibrium area of V_{max} minus the threshold velocity for sand transport. While they assumed that each inlet is in equilibrium in its present condition, it is doubtful that Indian River Inlet is actually at an equilibrium area. The channel dimensions are continuing to increase.

M. P. O'Brien (1969) developed a relationsihp between the cross-sectional area of an inlet at MSL and the spring tidal prism of the corresponding bay. This relationship, shown in Figure 80, predicts an equilibrium cross-sectional area of 16,000 ft^2 for a spring tidal prism of 9 x 10^8 ft^3. This predicted area compares quite well with the actual value of 19,800 ft^2.

The inlet cross-sectional area and tidal prism have both been increasing according to O'Brien's relationship. In 1930, the inlet's area was approximately 420 ft^2. This corresponds to a tidal prism

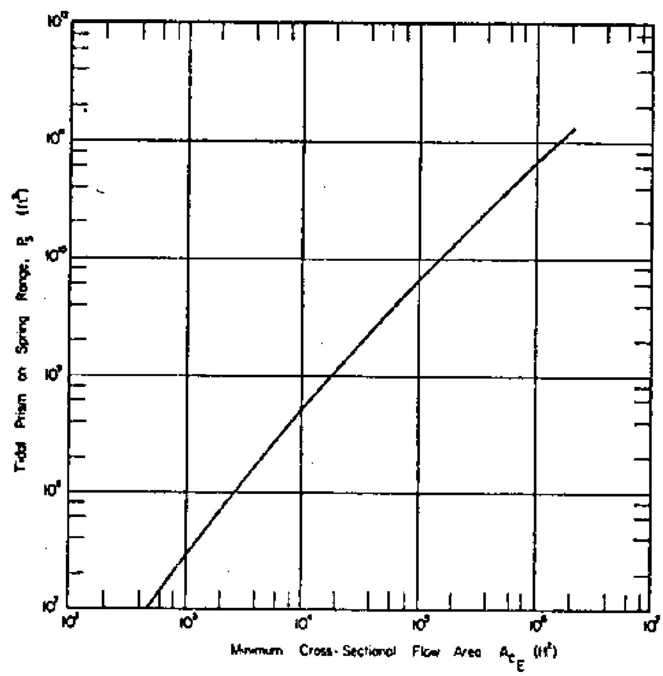


Figure 80 Equilibrium Cross Sectional Area and Tidal Prism Relationship (O'Brien and Dean, 1972)

of about 8×10^6 ft^3. The average observed inflow per tidal cycle at that time was 1.7×10^7 ft^3, and the average outflow was 2.75×10^7 ft^3 (Indian River Inlet Commission, 1931). In 1948, the tidal prism of Indian River Inlet was measured to be approximately 4×10^8 ft^3, and the average cross-sectional area was 8,400 ft^2 (Keulegan, 1967). Using O'Brien's relationship, a tidal prism of 4×10^8 ft^3 corresponds to an area of approximately 8,000 ft^2.

O'Brien's relationship (expressed as $a = 4.69 \times 10^{-4} p^{0.85}$ where a is the inlet cross-sectional area and p is the tidal prism) can be converted into a relationship between the maximum inlet velocity and the cross-sectional area by making several assumptions. It will be assumed that the tidal prism, p, can be expressed as follows:

$$P = a \int_0^{T/2} V_{max} \sin \frac{2\pi t}{T} \, dt = \frac{a \, V_{max} \, T}{\pi}$$

where T = tidal period = 44,700 sec. Introducing this into O'Brien's relationship we find that:

$$a = 4.69 \times 10^{-4} \left(\frac{a \, V_{max} \, T}{\pi}\right)^{0.85}$$

which reduces to:

$$V_{max} = \frac{a^{0.1765}}{T} \, \frac{\pi}{(4.69 \times 10^{-4})^{1.176}} = 0.577 \, a^{0.1765}$$

Figure 81 shows this relationship plotted along with the V_{max} versus cross-sectional area of a channel 20 times as wide as the depth (the stability curve discussed earlier in this section).

More extensive investigation of tidal prism-inlet area relationships has been conducted recently by J. T. Jarrett (1976). Jarrett expresses the relationship between cross-sectional area and tidal prism as: $a = 3.76 \times 10^{-4}\ p^{0.86}$ for inlets on the Atlantic, Gulf and Pacific coasts of the United States with two jetties. For inlets on the Atlantic coast with two jetties, he expresses the relationship as: $a = 5.77 \times 10^{-5}\ p^{0.95}$. These relationships can be reduced to the following forms similarly to O'Brien's relationship: $V_{max} = 0.695\ a^{0.1628}$ for all inlets with two jetties and $V_{max} = 2.035\ a^{0.0526}$ for Atlantic inlets with two jetties. These relationships have also been plotted in Figure 81. All three curves cross the stability curve at about $a = 31{,}000\ ft^2$ and $V_{max} = 3.6$ ft/sec. At this crossing point the theoretical maximum velocity computed from the inlet hydraulics (plotted as the stability curve) is equal to a maximum velocity which satisfies the tidal-prism-inlet area relationship. An inlet could be expected to reach an equilibrium state when it satisfies both relationships simultaneously. The present enlargement trend of the inlet channel may be an attempt to reach this equilibrium area.

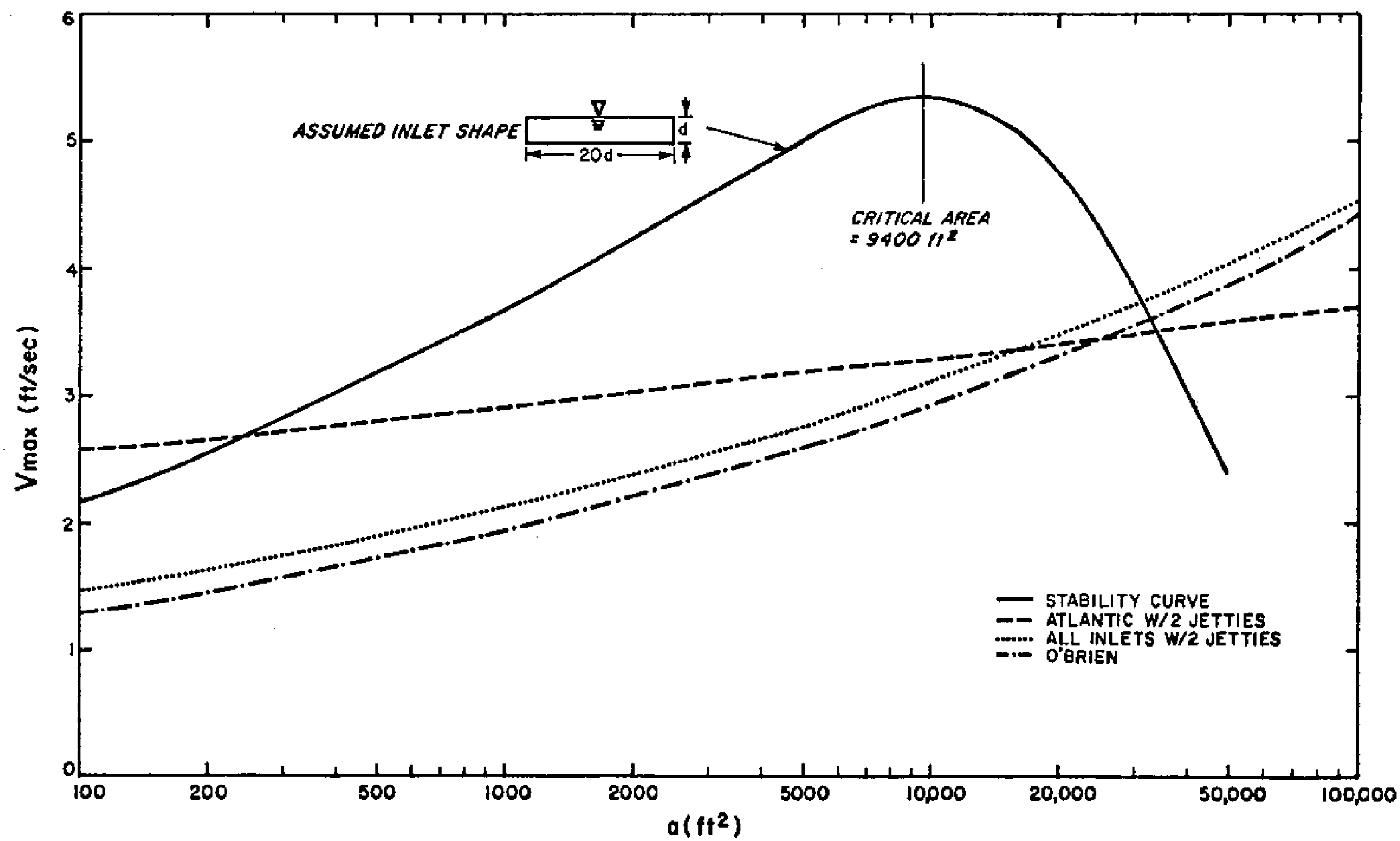


Figure 81 Stability Curve for Assumed Cross Section and Tidal Prism-Inlet Cross Sectional Area Relationships

According to Escoffier's stability concept, Indian River Inlet is stable and should resist changes in cross-sectional area. However, the predicted maximum inlet velocities are significantly lower than observed currents. This would indicate that the actual curve of V_{max} versus cross-sectional area is above the predicted curve. The actual curve cannot be determined due to the lack of historical current data and the critical area of this curve is unknown. Thus, the exact location of the inlet on this actual curve is unknown.

The stability index of Indian River Inlet would be relatively small based on Figure 79. The present area is only a little larger than the critical area. If the present area is actually in the unstable region, however, the stability index would be negative.

Overall, Indian River Inlet appears quite resistant to closure. The strong tidal currents are capable of scouring the channel bottom and removing sand deposited in the channel at the south jetty and elsewhere. Increased flow resistance (due to development of the inner and outer shoal) and decreased hydraulic radius (due to widening of the channel without an increase in cross-sectional area) will tend to make the inlet less stable. Also, should another inlet be formed between Indian River Bay and the ocean or between Rehoboth Bay and the ocean by a storm breaching the barrier or by construction, the inlet will become less stable.

5.4 Ocean Currents

The currents in the ocean outside of Indian River Inlet are influenced by the tidal currents in the Delaware Bay. The ocean currents have a large effect on the sediment movement patterns of the inlet. The interaction of the ebb tidal plume with these ocean currents is discussed in the next section.

There has been relatively little study done on the currents in the approaches to the Delaware Bay. Generally, the tidal wave approaches the bay from the northeast. Comparison of tidal ellipses determined for stations around Cape May, New Jersey with that at the Overfalls Light Vessel (located just outside the entrance of the Delaware Bay and illustrated in Figure 82), indicates that the phase of the tidal current off Cape May is significantly earlier than that at the Overfalls Light Vessel. The ebb tidal flow from the bay tends to be directed southeast along the Delaware Submarine Channel (U.S. Coast and Geodetic Survey, 1926).

The nearest ocean tidal current information to Indian River Inlet was recorded at the Fenwick Island Shoal Light Vessel, located 12 nautical miles east of Fenwick Island and 15 nautical miles southeast of Indian River Inlet. Hourly current observations were made from November, 1912 to February, 1913. It is possible to obtain a rough estimate of the currents outside the inlet from the recorded values at the Fenwick Island Shoal Light Vessel. The current

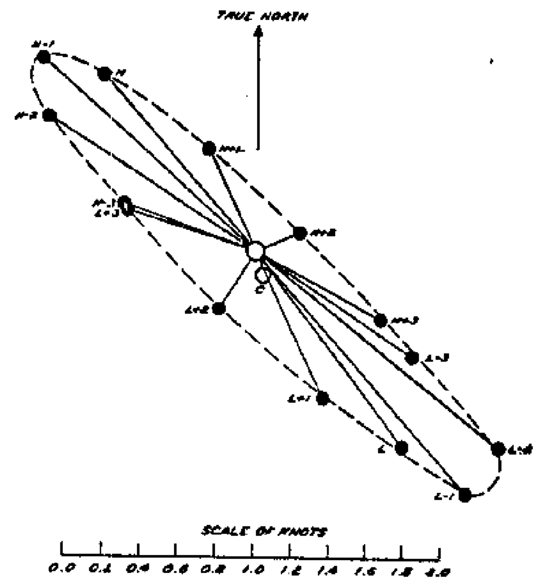


Figure 82 Tidal Current Ellipse, Overfalls Light Vessel (U.S. Coast and Geodetic Survey, 1926)

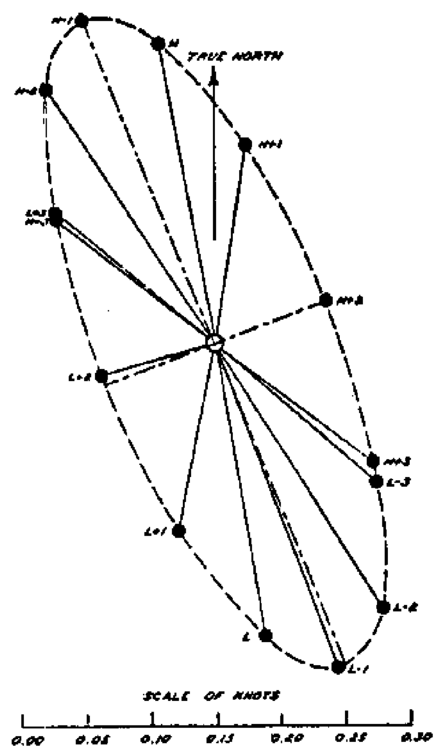


Figure 83 Tidal Current Ellipse Fenwick Island Shoal Light Vessel (U.S. Coast and Geodetic Survey, 1926)

ellipse for the M_2 component of the tide is shown in Figure 83. The phase of the current is almost the same as at the Overfalls Light Vessel, 23 nautical miles to the north northwest. In the figures, the velocities and directions of the tidal currents are shown for each hour with reference to the time of high and low tide at Sandy Hook, New Jersey (H representing the time of high water, and L representing the time of low water).

At Fenwick Island Shoal Light Vessel, the strongest flood currents (setting N. 20°W. with a velocity of 0.3 knots) occur about one hour before the time of high tide at Sandy Hook, New Jersey. The minimum before ebb occurs about two hours after the time of high water at Sandy Hook. The strongest ebb currents (setting S. 20°E. with a velocity of 0.3 knots) occur about 1.1 hours before the time of low water at Sandy Hook. The minimum before flood occurs about 1.9 hours after the time of low water at Sandy Hook. The current in shallower waters nearer to shore appears to run somewhat later than that at the Fenwick Island Shoal Light Vessel (U.S. Coast Guard and Geodetic Survey, 1926).

Comparing predicted times of slack tides at Indian River Inlet and times of high and low tides at Sandy Hook, it is found that high water slack at Indian River Inlet is about 2.2 hours after the time of high water at Sandy Hook. Similarly, the low water slack at Indian River Inlet is about 2.4 hours after low water at Sandy Hook (N.O.A.A., 1975c). Thus, the tidal currents in the ocean outside Indian River Inlet, which are induced by the tidal flow in the Delaware Bay, are quite closely in phase with the tidal currents in Indian River Inlet. The times of maximum current in ocean and inlet are also predicted to be quite close.

The magnitude of the ocean current velocities can be approximated from Figure 83. As one considers currents closer to shore, one would expect the rotary tidal currents depicted for Fenwick Island Shoal Light Vessel to become more closely aligned parallel to the coast. In Figure 83, the maximum currents are seen to be aligned approximately parallel to the shoreline. The magnitude of these maximums is about 0.3 knots. This value compares well with data from drogue and dye patch studies conducted off Bethany Beach, Delaware by the Civil Engineering Department at the University of Delaware (Ditmars, 1974).

During ebb tide, two large eddies form along Delaware's coast. The northern eddy is located between Hen and Chickens Shoal and the coast. It is formed by water becoming entrained in the Delaware Bay's ebb flow offshore of Cape Henlopen. The replacement of this water induces a northerly current along the coast (Moody, 1964). The second eddy is formed by the ebb flow from Indian River Inlet meeting the north-to-south current in the ocean.

5.5 Ebb Tidal Plume

The ebb tidal flow from Indian River Inlet encounters a north-to-south current upon entering the ocean. As mentioned in the previous section, the ocean current is generated by the tides in Delaware Bay which are very nearly in phase with the currents in the inlet. There is a southerly current in the ocean during ebb tide which reverses during flood tide. Upon encountering the current in the ocean, the flow from Indian River Inlet is deflected to the south, as can be seen in Figure 84.

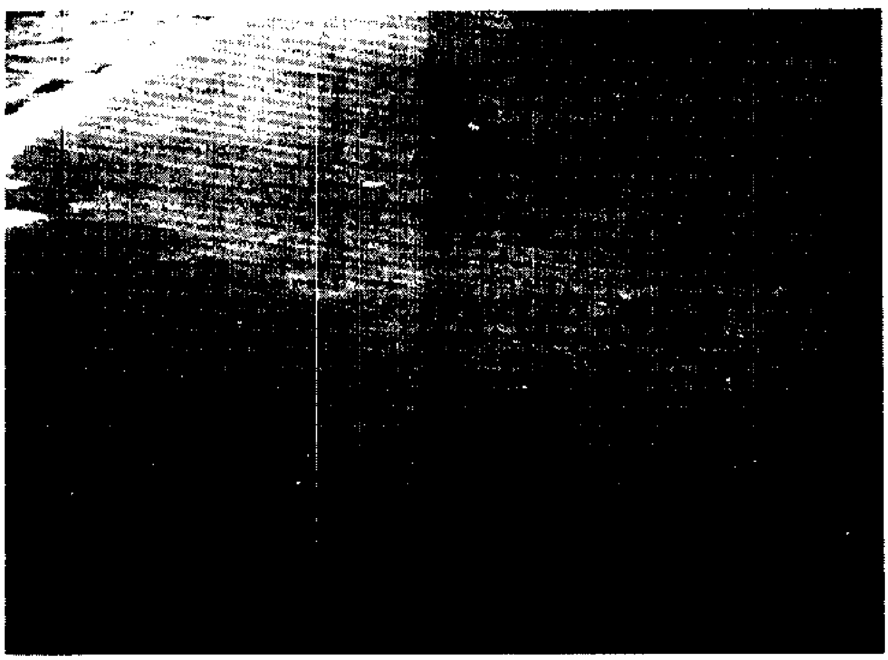


Figure 84 Ebb Tidal Plume from Indian River Inlet Turning South Viewed from the South

In recent years, there has been considerable interest in the near field behavior of such discharges. Researchers have been interested in the environmental effects of cooling water discharges and outfalls into rivers, lakes, and oceans. In studying these discharges, researchers have worked with two-dimensional and three-dimensional models. Two-dimensional models are simpler than three-dimensional models since they do not consider any vertical variation of parameters. Two-dimensional models neglect the effects of such things as density currents, stratification of the water, and bottom shear (which will effect the vertical velocity distribution and the outward momentum, altering the trajectory).

H. H. Carter (1969) developed a mathematical model to describe a two-dimensional jet discharging into a flowing body of water by using results obtained from hydraulic modeling. Carter assumed a drag force on the jet similar to the drag on a solid object. The jet is actually turned by a combination of entrainment of momentum from the crossflow and the pressure difference between the seaward (upstream) and the landward (downstream) side of the jet. His early work was done in a relatively narrow flume and failed to properly account for boundary effects.

The geometry of Indian River Inlet can be simplified to that studied by various researchers as follows: assume the ocean and channel bottoms are flat and at the same level, assume the beach is a vertical wall located at the tips of the jetties, assume the inlet channel is rectangular and at right angles to the beach, and assume a constant velocity in the ocean in the offshore direction.

Carter solved for the trajectory, dilution, horizontal distribution of excess heat, and lateral spread of the two-dimensional jet in his 1969 publication and two later publications. Centerline trajectories predicted using Carter's equation did not agree well with observed centerline trajectories for Indian River Inlet's ebb tidal plume. Predictions of lateral spreading and dilution are based on the densimetric Froude number of the jet and are not directly applicable to a neutrally buoyant jet such as Indian River Inlet.

Other researchers (Policastro and Tokar, 1972) have solved for trajectory, width, velocity and temperature distribution of a jet discharged into a flowing body of water. They develop a system of five differential equations by considering conservation of mass, momentum, and energy, and by making several assumptions, such as assuming similar velocity and temperature profiles. However, before solving for the useful information about the jet, one must first determine a value for the drag coefficient and the entrainment coefficient. There is no method to determine these coefficients a priori for each application.

A three-dimensional numerical model was developed by Stolzenbach and Harleman (Policastro and Tokar, 1972). They divide the cross section of the jet into several sections and then integrate the steady-state, time-averaged equations over the cross section. The model can be adapted to various geometries such as a jettied inlet.

There is considerable controversy among researchers about the eddy in the lee of the jet. As water from the jet moves out into the crossflow, it entrains water from both sides. To supply the water to the downstream edge of the jet, a current develops along the shore which flows in the opposite direction of the ocean current.

Some researchers claim that the pressure difference between the seaward and landward sides of the jet creates a force which returns the jet to the shore. Other researchers claim that the jet will not deflect landward after it turns parallel to the ambient flow. The return of the jet to shore could also be due to the demand for water to supply the return flow of the eddy. The researchers also disagree on the distance downstream to the point at which the jet reattaches to the shore.

In summary, the various theories on the behavior of jets discharged into a cross flow showed little more than qualitative agreement with the ebb tidal plume from Indian River Inlet. The one solution which could be applied without prior knowledge of drag and entrainment coefficients, Carter's equation, gave poor agreement with the observed behavior of the plume. The return of the plume to the shore was also not predicted accurately by the theories of these researchers. Much more work will be required to develop the kind of understanding necessary to predict the behavior of Indian River Inlet's ebb tidal plume.

CHAPTER VI

INLET MAINTENANCE

In this chapter two possible means of maintaining Indian River Inlet are considered. In the first section of this chapter, sand bypassing, the transfer of sand from the updrift beach to the downdrift beach is discussed and several bypassing techniques are presented. The second section discusses maintenance dredging as a means of maintaining the inlet and the surrounding beaches. Both techniques will be examined, and a specific recommendation regarding inlet maintenance will be presented in the recommendation section.

6.1 Sand Bypassing

Bypassing is the movement of sediment from one side of an inlet to the other. Natural inlets usually allow sand, supplied to the updrift side by littoral drift, to pass the inlet by tidal flow action or on an offshore shoal or bar (Bruun, 1965). In its natural state, Indian River Inlet had a shallow bar at its entrance across which sand could pass (see Figure 9).

Indian River Inlet was stabilized partly to reduce the outer bar and make it navigable. A new outer bar is presently forming in the ocean. This bar has not yet grown to sufficient size and elevation to allow sand to pass the inlet naturally once again. In addition, the unique interaction of the ebb tidal plume with the ocean currents also tends to inhibit bypassing.

The sand carried to one side of an inlet can be artificially transferred to the downdrift beach by several techniques. This has been done at several inlets with varying success. While many installations fail to bypass all of the net littoral drift, it is the general practice to design for transferring that amount. Thus, an installation at Indian River Inlet should be designed to handle approximately 125,000 c.y. per year.

One method of bypassing is through the use of a fixed land-based hydraulic dredge. There have been several of these facilities constructed in the past (U. S. Army Corps of Engineers Coastal Engineering Research Center, 1973). Two of the most notable plants are located at Lake Worth and South Lake Worth Inlet, Florida.

The first consideration for a fixed plant is its siting. Most are located on the updrift jetty between the tip of the jetty and the beach. There are disadvantages associated with this site for Indian River Inlet. The south beach is the swimming area for the State Park. Dredging at the south jetty would form a deep hole which would be hazardous to children and poor swimmers. The appearance and noise of the plant would also be objectionable to bathers and the large numbers of fishermen who fish from the south jetty.

There are three main advantages in locating a fixed plant on the south jetty. First, the pumping distance to the north beach would be less than for other locations. A plant located on the south jetty would be protected somewhat from waves from the northeast. Also, this location would be best for trapping the littoral drift before it enters the inlet.

An alternate site for a fixed plant would be 2,000 feet south of the south jetty. This site is at the apex of the bulge of the south beach and away from the heavily used portion near the jetty. The borrow hole could fill with littoral drift from both directions, and there would be more sand within the reach of the plant. The pumping distance would be longer than for the south jetty site, and the plant would be more exposed to storm damage. There is also less control over, and more variation of, the shoreline position on the open beach than at the jetty side.

Sand transferred by a bypassing plant should be discharged far enough northward to prevent its reentering the inlet. Such a southerly transport would be especially likely in the winter due to northeast storms. The discharge should also be moveable to prevent the building of a large bulge in the beach during periods of heavy sand bypassing. It should be adequate to discharge the sand from 1,000 feet to 1,500 feet north of the north jetty.

The discharge pipe can either be run over the bridge or under the channel. The 35-foot vertical rise and longer pumping distances involved with crossing the bridge would require more power for pumping than going under the inlet. The pipe could be suspended under the bridge and a catwalk provided to allow easy access for servicing and replacing the pipe. The load on the bridge introduced by the pipe and walkway would not be excessive. While it would take less power to pump sand under the inlet, there would be a constant danger of the pipe becoming plugged. At Lake Worth Inlet, Florida, where the discarge line is under the inlet, an automatic line-flushing system is used to prevent clogging (Senour and Bardes, 1959). The strong inlet currents at Indian River Inlet would make installation difficult and cause the bottom to shift rapidly. Should the submerged portion of the line become plugged or damaged, it would be hard to service. Thus, it would be best to run the line across the bridge.

The pumping distance, from the south jetty across the bridge to 1,500 feet north of the north jetty, is about 2,800 feet. The pumping distance would then be about 4,700 feet from a plant 2,000 feet south of the south jetty.

The sand bypassing plant at Lake Worth Inlet was constructed from 1956 to 1958 at a cost of about $569,000 (Middleton, 1959). The plant pumps about 100,000 c.y. per year through 1,750 feet of 10-inch discharge line (U. S. Army Corps of Engineers Coastal Engineering Research Center, 1973). Assuming this same plant would be adequate for Indian River Inlet, the present first cost can be estimated at approximately $1.8 million using a construction cost index (Fox, 1976). This is equivalent to an annual cost of $160,000, amortizing $1.8 million over 30 years at 8 percent interest and assuming zero salvage value. The present operating costs are about $85,000 per year (Singer, 1976). The total cost is $245,000 per year or $2.45 per c.y. Thus, the cost of transferring sand by a fixed bypassing plant at Indian River Inlet would be expected to be approximately $2.50 per c.y. or $310,000 per year.

Another means of bypassing the sand is with a mobile two-stage system. A small pump is used to pick up a sand-water slurry from the beach. The pump could be a jet pump or a pump operated by compressed air or hydraulics. This slurry is then piped to a booster pump and finally to the desired location. In this manner, the booster pump can be located behind the dunes where it would be less exposed to storm damage.

In the scheme currently under development at the U. S. Army Corps of Engineers Waterways Experimental Station, a small center-drive jet pump is suspended from a buoy in the borrow area. The pump is connected to flexible water supply and discharge hoses and can easily be moved around (McNair, 1976). With this system, there is very little equipment exposed to storm damage. The cost of this form of bypassing for Indian River Inlet is estimated to be on the order of $1.75 per c.y. or $220,000 annually (McNair, 1976).

The first stage pump could also be suspended from a crane on the beach. However, the crane could probably not reach beyond the surf zone, and the effects of waves on such a system is unknown.

A primary advantage of the two-stage system is its mobility. There is no danger of becoming land locked as with a fixed plant. The system also has a much larger area to borrow from and will not have to stand idle for long periods of time when there is low wave action and little sand being carried within reach. This is a major problem with existing fixed plants.

Sand could also be bypassed by trucking it from the south beach to the north beach. The advantages of truck bypassing are: the sand can be picked up and dumped anywhere on the beach without the limitations of a fixed plant, there need be no deep hole formed at any point on the beach, the equipment could be removed from the beach in case of a storm, and there need be no fixed structures located on the beach.

Probably the biggest disadvantage would be the heavy truck traffic over the bridge. Using 12 c.y. capacity trucks, over 10,000 loads would be required annually. This many trips could be expected to significantly increase the road wear, noise, and number of traffic accidents.

A dragline crane could be used to remove sand from the beach face and surf zone. This sand would then be loaded directly into trucks driven onto the beach. A paved roadway would be required from Route 14 through the dunes. Some form of moveable matting would then be needed from the dunes to the crane's location. On the north beach, another roadway onto the beach would be required. There the sand could be dumped and then spread with a small bulldozer. The cost of this form of bypassing including equipment rental, wages, and site setups would be about $2.80 per c.y. or $350,000 annually (R. S. Means Company, Inc., 1975).

Other variations of this scheme would be to use a four-wheeled front-end loader to load the trucks, or self-loading scrapers. Both of these schemes could not reach the sand in the surf zone as well as with a dragline. The trucks could also be loaded and dumped from a trestle as has been done at Shark River Inlet, New Jersey (U. S. Army Corps of Engineers Coastal Engineering Research Center, 1973).

All of the above sand bypassing operations would use the south beach as an impoundment area. The sand would be transferred to the north beach before it entered the inlet. To increase the efficiency of these operations, it would be desirable to make the south jetty more sand-tight than at present. With sand no longer introduced at the south jetty, the south side of the inlet channel would be expected to deepen. It is possible that this would cause the south jetty to fail in the same manner as the north jetty is presently failing.

Stopping the flow of sand onto the channel from the south beach would also be expected to reduce shoaling in the inlet. The large shoals in the bay would be expected to continue to shift, however. Sand bypassing would also be expected to greatly reduce the amount of sand lost onto the outer shoal.

Before beginning a sand bypassing operation, the effects on the beach to the south of the inlet should be considered. The inlet is presently causing the beach immediately south of the inlet to accrete. Bypassing would effectively remove the barrier to littoral drift and allow the beaches to resume their natural erosion rate. The effects on the beach immediately south of the inlet could be regulated by the amount of sand bypassed, and the effect on the undeveloped State Park land would be minimal. The shoreline is developed beyond about 1.3 miles south of the inlet and the erosion rate at this point could increase due to bypassing. The legal aspects of this possible erosion should be reviewed before bypassing is commenced.

The present practice of obtaining beach nourishment for the north beach from within this inlet is a form of sand bypassing. The shoal southwest of the Coast Guard Station is used as a deposition zone. This operation will be discussed in the next section.

Another technique for bypassing sand around Indian River

Inlet would be to dredge sand from the outer shoal. Waves passing over the outer shoal become confused because of refraction, shoaling, and wave-current interaction. These waves would be too large for a conventional pipeline dredge. A hopper dredge could be used in this wave climate but it would be expensive to discharge the sand on the north beach rather than dumping it in open water. Perhaps in the future the sand contained in the outer shoal can be used for beach fill; for the time being, it is not economically feasible (Black, 1971).

Many modern inlet designs include a low weir section in the updrift jetty. The sand is allowed to pass over the weir and deposit in a deposition basin from which it can be dredged and pumped to the downdrift beach. The addition of such a feature to Indian River Inlet would be very difficult and expensive, as it would require extensive modification to the existing inlet.

6.2 Maintenance Dredging Schedule

Sand for nourishment of the north beach is presently obtained from within the inlet. This sand is periodically dredged from the shoal southwest of the Coast Guard Station. Though the primary purpose of this dredging is to obtain beach fill, it does help to reduce the volume of the shoal. In December, 1974, the water depth over this shoal was less than four feet. The navigation channel passes northward of the shoal (which is located in the center of the inlet channel).

The middle shoal is a deposition zone. Sand which passes from the south beach into the inlet channel is deposited on this shoal. If the middle shoal is going to continue to be used as a deposition site, the south jetty should not be raised or extended. Sand should continue to be allowed to enter the channel.

The advantage of dredging from the middle shoal is that a floating pipeline dredge can be used. If the sand is placed up to 1,500 feet north of the north jetty, the pumping distance is about 4,000 feet and no booster pump is needed. Also, since the middle shoal is composed almost entirely of sand, the cutter head and spuds used on most dredges are not required. A simple suction dredge could be used. Dredging within Indian River Inlet is difficult, however. The strong tidal currents must be contended with. In addition, the dredging site is exposed to considerable wave action. Some ocean waves propagate up the channel, and there is a several-mile fetch for waves from the west. During the summer, the heavy pleasure boat traffic can also be a problem.

In taking sand only from the middle shoal, sand can still be lost to the outer shoal. Sand picked up by the ebb tidal flow would continue to be deposited on the outer shoal. In addition, it appears that the finer sand grains are winnowed out and either carried onto the bay shoals or onto the outer shoal. The larger sand sizes are generally believed to be more stable than fine sand as beach fill (Dean, 1974); but relatively coarse sand placed during the 1975 operation eroded quickly.

It is unknown how the dredging from within the inlet effects the increasing channel dimensions. Either the large amounts of sand which move through the inlet could refill any borrow area and only natural forces cause the inlet to enlarge, or the dredging could be partially responsible for the increase in the channel dimensions.

The banks of the channel can be expected to continue to erode until protected by an effective structure. Large amounts of dredging from within the inlet could cause the inlet to deepen. The loss of sediment from within the inlet in the last five years is of the same order of magnitude as the dredging volume.

Since 1972, 916,810 c.y. of sand have been dredged from the inlet west of the highway bridge and then placed on the north beach. This corresponds to an annual rate of 230,000 c.y. at an annual cost of $228,000 for a four-year period from 1972 to 1976. Since 1957, sand has been placed on the north beach at a rate of about 106,000 c.y. per year.

The relatively low unit price of $.82 per c.y. for the 1972-1973 dredging operation was due to the more economical large dredge. During the 1975 operation, a 12-inch dredge was used, and the unit price was \$1.94 per c.y. It is generally uneconomical to mobilize a large dredge for a job smaller than about 500,000 c.y., and a dredge for smaller operation will generally cost in the \$1.75 to \$2.50 per c.y. range (Hughes, 1976).

A beach nourishment rate of about 125,000 c.y. per year should be adequate to replace sand removed from the north beach by littoral drift. The fluctuations in the north beach and in the inner shoals associated with waiting long periods between large dredging operations would be undesirable. However, the unit prices would be lower using a large dredge.

The middle shoal would yield about 250,000 c.y. if dredged to a depth of 17 feet. This shoal could be removed every other year using a small dredge, and the sand could be placed on the north beach. There has been a shoal located near the Coast Guard Station since at least the early 1960's, and it is assumed that this shoal would rebuild after its removal.

CONCLUSIONS

Stabilization radically altered the littoral processes at Indian River Inlet. The littoral drift was stopped from proceeding across the inlet and onto the north beach. As a result, the south beach has accreted, and the north beach has eroded. Sand from the littoral drift and from the erosion of the channel banks has been deposited in the ocean in a large ebb tidal shoal. This sediment movement pattern was documented with a sand tracer study and quantified through study of historical charts. Without the implementation of corrective measures, these accretion and erosion patterns can be expected to continue. Another product of the formulation of the sand budget was the estimation of the net littoral drift at Indian River Inlet to be 105,000 c.y. per year to the north.

With the erosion of the channel banks, the inlet hydraulics have been changing. The tidal prism of Indian River Bay and Rehoboth Bay has increased with the cross-sectional area of the channel in accordance with O'Brien's relationship. Evidence indicates that Indian River Inlet is quite stable against closure. This study further showed that the interaction of the inlet's ebb tidal plume with the ocean current decreases the natural sand bypassing ability of Indian River Inlet.

The present undesirable trends at Indian River Inlet can be corrected. Artificial sand bypassing would stop the erosion of the north beach, reduce the shoaling within the inlet, and reduce the loss of sand onto the outer shoal. Sand bypassing is technically and economically feasible for Indian River Inlet. Recommendations based on the findings of this study are presented in the following chapter.

RECOMMENDED IMPROVEMENTS

On the basis of the findings of this study, the following improvements to Indian River Inlet are recommended. Sand should be bypassed around the inlet on a regular basis. The best results would probably be obtained by using a mobile two-stage pumping system on the beach 2,000 feet south of the south jetty, as discussed in Chapter VI. An alternate method, which would be less effective, would be to continue the practice of dredging from the middle shoal and spoiling the sand on the north beach.

Should a sand bypassing scheme be implemented which removes sand from the south beach, the south jetty should be improved to prevent sand from passing into the channel. The crest elevation of the jetty should be raised, and the jetty should be made sand-tight.

The south channel bank of the inlet should be bulkheaded to prevent further erosion. Measures should also be taken to stop the present erosion of the north channel bank.

If it is decided to repair the north jetty and reinstall the navigation light, the deep scour hole must be considered in the design of the repair work. The south jetty may also need protection against slumping if the sand presently introduced into the channel at the south jetty is stopped.

RECOMMENDATIONS FOR FURTHER STUDY

The sand bypassing section of this report was only a preliminary study. More detailed study is needed in the design and economics of individual bypassing techniques. In this manner, the best bypassing technique can be determined more precisely.

A follow-up study should be conducted within the next 5 to 10 years to evaluate the effectiveness of any sand bypassing performed. Such a study should include hydrographic surveys and computations of accretion and erosion since the previous study. Also, the volume of sand bypassed and changes in bathymetry and tidal hydraulics should be examined.

If the north channel bank is not protected by bulkheading, the erosion rate should be monitored closely. More accurate knowledge of this erosion rate would allow better evaluation of the need for shoreline protection.

The study of several additional features and phenomena at Indian River Inlet would improve the coastal engineer's knowledge of this inlet and others similar to it. A study of the chop and scour hole at the tips of the jetties could provide useful information on wave-current interactions. Better understanding of this phenomenon may lead to possible ways of eliminating it.

Other areas which would be helpful to study are: the refraction of waves over the outer shoal; further current studies in the inlet; a study of the tides in the ocean, Indian River Bay, and Rehoboth Bay; a detailed study of the flood tidal shoal in Indian River Bay; and a reevaluation of the design of the inlet structures in consideration of the deepening water.

APPENDIX 1

SAND TRACER MANUFACTURE

The sand tracer used in this study was produced by glueing a fluorescent pigment to beach sand with powdered milk. This form of sand tracer was chosen because it is relatively easy to manufacture and provides adequate marking of sand grains. Also, it does not last for extended periods of time, which could possibly contaminate future studies.

Five hundred pounds of sand tracer were manufactured, using Auramine as a pigment. This yellow sand tracer proved unsatisfactory because it was difficult to distinguish the marked sand from small pieces of shell and other sand grains. Much better results were obtained using fluorescent red and green pigment purchased from Lawter Chemicals, Inc., 990 Skokie Boulevard, Northborok, Illinois 60062. Non-fat dry milk was purchased at local grocery stores. Sand used to make the tracer was obtained from the beach at Indian River Inlet so that the sand tracer size characteristics would be similar to the natural sand.

The sand tracer ingredients are:

100 lbs. of sand

3.5 lbs. of powdered milk

1.25 lbs. of pigment

4,000 c.c. of water

Mix the sand and pigment together while dry. Next, mix the powdered milk and water and then combine with the sand-pigment mixture. Mix thoroughly. Bake at about 200° F until dry. Temperatures above boiling will burn the milk. The quantities of pigment, milk, and water required for different sands will vary. The above proportions can be varied to obtain the desired results.

When dry, the mixture is relatively hard and must be broken into individual grains. This can be done with a hammer mill, tumbling in a cement mixer with a few stones, or by hand. The tracer is then sifted to remove lumps and should be stored in burlap bags lined with plastic bags.

APPENDIX 2

TIDE GAUGE INSTALLATION

In order to study the tides and hydraulics of Indian River Inlet, it was necessary to establish a temporary tide gauge in the ocean near the inlet. The Coast Guard and the U. S. Army Corps of Engineers approved the temporary installation (north of the north jetty and about half-way between the tip of the jetty and the beach).

Figure 85 shows the installation of the weather box onto a 20-foot telephone pole which had been jetted into the bottom. The installation of the pole was accomplished by first weighting one end of the pole and holding it upright. A hole was then jetted under the tip of the pole with a 3/4-inch diameter pipe connected to a water pump. This technique proved very difficult and is not recommended in exposed areas or from a small boat.

The second installation of the tide gauge, which was mounted on a steel pipe jetted into the bottom, proved much more successful. A 21-foot length of 3-inch galvanized steel pipe was jetted into the sandy bottom. The tip of the pipe was partially flattened to increase the water velocity, and a joint was made in the middle of the pipe to facilitate handling. The top of the pipe was then connected to a 3 h.p. water pump with a 2-inch hose.

The pipe was held vertically in the water, and the pump was then turned on. It quickly penetrated to the desired depth under its own weight. The weather box was then strapped to the pipe with specially-made aluminum bands, and the instrument was installed. The completed installation is shown in Figure 86.

To remove the tide gauge, the weather box was first removed. Reconnecting the pump to the 3-inch pipe failed to free it. The pipe was freed by jetting around it with a 3/4-inch diameter pipe while pumping through the 3-inch pipe. Once free, the 3-inch pipe could then be recovered.

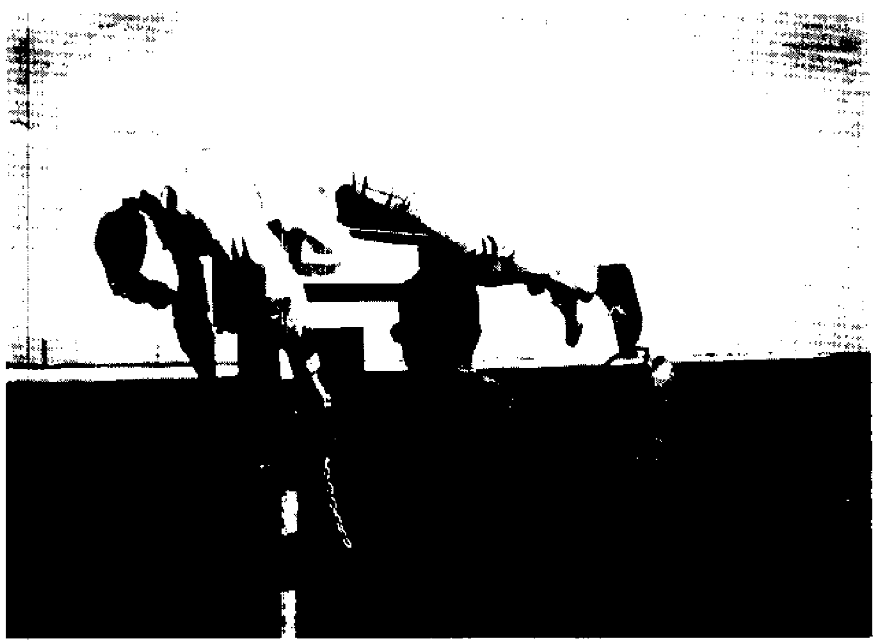


Figure 85 Installation of Weather Box onto Wooden Pole

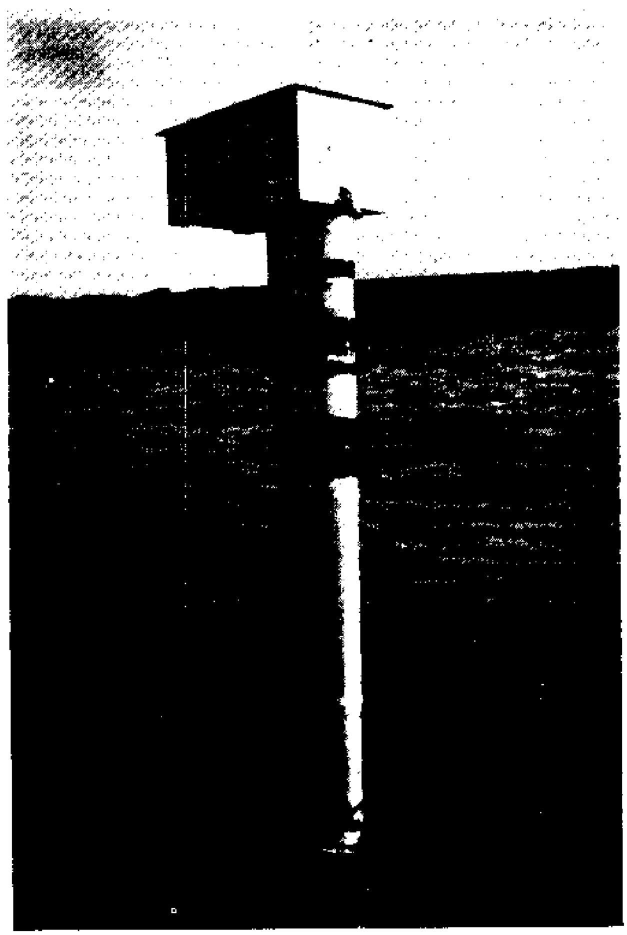


Figure 86 Tide Gauge Mounted on Pipe

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