US Army Corps of Engineers
Philadelphia District

DELAWARE BAY COASTLINE
DELAWARE AND NEW JERSEY

Broadkill Beach, DE
Interim Feasibility Study

FINAL
APPENDIX A

ENGINEERING TECHNICAL APPENDIX

September 1996

APPENDIX A

ENGINEERING TECHNICAL APPENDIX

DELAWARE BAY COASTLINE
DELAWARE AND NEW JERSEY

BROADKILL BEACH, DELAWARE
INTERIM FEASIBILITY STUDY

APPENDIX A
ENGINEERING TECHNICAL APPENDIX

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GENERAL

This Engineering Technical Appendix was prepared in accordance with ER 1110-2-1150, Engineering and Design for Civil Works Projects. Information in this appendix supplements data in the Feasibility Study to satisfy criteria in ER 1110-2-1150.

SECTION 2

HYDRAULICS AND HYDROLOGY/COASTAL PROCESSES

PHASE I REPORT

BROADKILL BEACH
EROSION AND FLOODING PROTECTION

Prepared for
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Philadelphia District

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Prepared by:

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Solon, Ohio

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November 29, 1995

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BROADKILL BEACH EROSION CONTROL AND FLOODING PROTECTION
PHASE I REPORT

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CONVERSION FACTORS, NON-SI TO SI (METRIC) UNIT OF MEASUREMENT

Non-Si units of measurement used in this report can be converted to Si (metric) units as follows:

Multiply	By	to Obtain
feet	0.3048	meters
cubic feet	0.02832	cubic meters
miles (US nautical)	1.852	kilometers
miles (US statue)	1.6093	kilometers
yards	0.9144	meters
cubic yards	0.7646	cubic meters
cubic yards/foot	2.508	cubic meters/meter
knots (international)	0.5144	meters/second

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SECTION I
INTRODUCTION

The Phase I study of shoreline erosion and flood protection for Broadkill Beach, Delaware is prepared by Water Resources & Coastal Engineering, Inc. for the U.S. Army Corps of Engineers, Philadelphia District under contract number DACW61-92-D-0011, delivery order No. 3. The scope of the Phase I study includes three main tasks. The first task is to review existing documentation, studies and data for the Delaware Bay in order to understand the existing hydraulic and littoral conditions through the entire lower Delaware Bay area. Section II of this report summarizes the environmental setting, the bay, tidal and current characteristics, wind and wave climate, stage frequency, beach and nearshore sediment characteristics, sediment transport potentials, and historic shoreline change trends.

Task 2 is to review the performance of the six Delaware Bay Section 54 project sites. Section III of this report documents the project performance based on previous monitoring data and existing conditions.

Task 3 includes four subtasks that intend to establish a detailed understanding of the existing hydraulic and littoral processes at Broadkill Beach. Nine beach profiles were selected in the Broadkill Beach study area to allow a detailed examination of shoreline changes, wave climate, inundation, and longshore and cross-shore sediment transport rates. Section IV of this report documents the without-project hydraulic and littoral conditions at Broadkill Beach.

Literature reviewed during the study, and references cited in the report are listed in alphabetical order in Section V.

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SECTION II
DELAWARE BAY STUDY AREA

The study area included approximately 53 miles of New Jersey shoreline from the mouth of the Cohansey River to Cape May Point and 45 miles of Delaware shoreline from Bombay Hook Point to Cape Henlopen. Figure 1 shows the study area and limits. The east shore lies in Cumberland and Cape May Counties, New Jersey. Compared with the rest of New Jersey, this shoreline is sparsely populated along most of its length with little to no modification to the shoreline. Fortescue, the largest community in Cumberland County, has less than 1,000 residents in the summer season. Toward Cape May Point, urbanization and shoreline protection structures dominate. The western shoreline lies in Kent and Sussex Counties, Delaware and consists primarily of a narrow strip of sandy beaches separating the bay from marshes and low flat lands. The section contains the incorporated municipalities of Lewes, Bower and Slaughter Beach, the small summer resorts of Pickering Beach, Kitts Hummock, South Bower, Big Stone Beach, Cedar Beach, Fowler Beach, Primehook Beach, and Broadkill Beach. A small harbor, Port Mahon is located north of Pickering Beach.

II-1. ENVIRONMENTAL SETTING

Delaware Bay is characterized by wide, shallow, subtidal flats dissected by branching finger-like tidal channels and parallel shoals. The mean depth of the bay is 32 feet, although wide shallow (less than 6 feet below MLW) flats exist along the margins of the bay and depths over 100 feet are found in the deepest area near the mouth of the Bay.

The New Jersey shoreline in the study area is naturally divided into two rather distinct geomorphic sections. From Cape May Point to Goshen Creek, the shore is characterized by low dunes fronted by a narrow strip of eroding coarse sandy beach. From Goshen Creek to Cohansey River, the bay shore is characterized as an irregular, low, eroding saltmarsh coast with isolated small beaches occasionally backed by low dunes and wetlands. The beaches are generally narrow with an average width of 10 to 50 feet at high water. At many locations the water reaches the foot of the low dunes behind the beach during periods of storm waves or unusually high tides. Salt marshes 0.5 to 3 miles in width separate the dunes from firm ground. Bays and coves can be seen throughout this reach. Narrow bands of marsh follow the stream channels several additional miles inland. The shoreline orientation changes from northwest-southeast from the Cohansey River entrance to Egg Island Point to east-west after Egg Island Point to Moores Beach and turns to a north-northeast and south-southwest direction to Cape May Point.

The Delaware shoreline is commonly divided into three general zones. The northern zone, from the C & D Canal to Pickering Beach, consists of marsh mud made up of course-grained sand in the nearshore zone. The beaches in the second zone, from Pickering Beach to Lewes

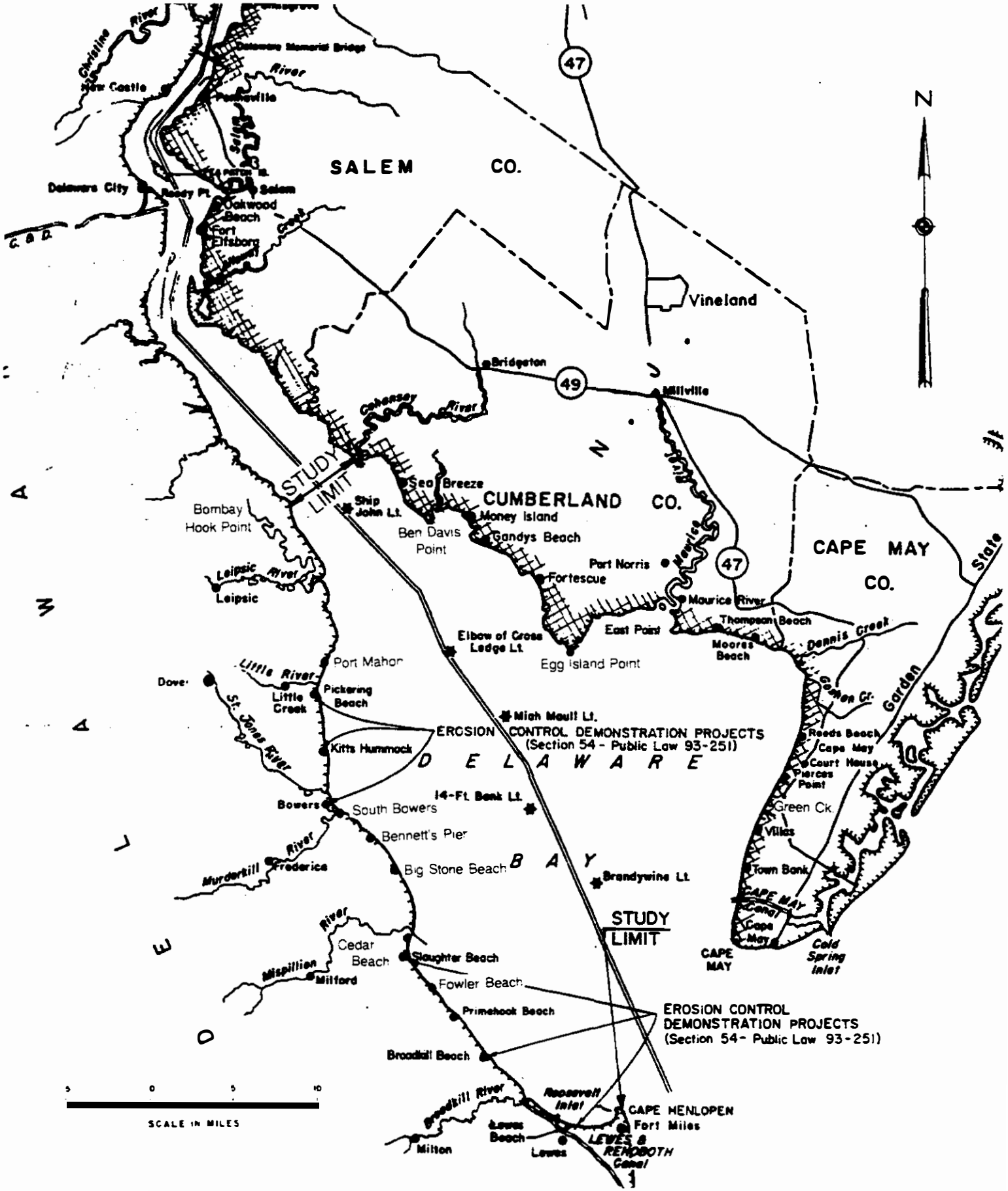

Figure 1 Delaware Bay Study Area

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Beach, are isolated in the north and progressively become continuous to the south. Cape Henlopen, the third zone, contains an abundance of sand for formation of an extensive dune field, wide gentle sloping beaches, a broad tidal flat, and numerous sand bars. From Bombay Hook Point to Bowers, the shoreline alignment is from north-south and turns to northwest-southeast from South Bowers to Roosevelt Inlet and then turns to east-west at Lewes Beach.

II-2. CLIMATE

The climate of Delaware Bay is moderated somewhat by its proximity to the Atlantic Ocean. In the winter, storms may be accompanied by strong gusty winds and rain or snow. Warm spells, sometimes with abundant rain, alternate with cool, dry weather in the spring. In the summer, weather is relatively constant with uniform warm temperatures, high humidity and low windspeeds, but there may also be frequent unstable showers and thunderstorms. The hurricane season typically extends from June to October. Precipitation is moderately heavy (45 inches per year) and well distributed, but somewhat heavier during June and July. Large amounts of ice form in Delaware Bay during severe winters. Ice pressures and impacts by floe ice often damage shore protection structures.

II-3. WINDS

Prevailing wind direction reported from different weather stations and from different time periods vary from southwest to northwest. The wind diagram (Figure 2) summarizing data for the period from 1924-1941 at the U.S. Weather Bureau Breakwater Harbor station shows that southwest is the prevailing wind direction, but winds from other directions occur nearly as often. Gale force winds, those over 30 miles per hour, come most often from the northwest and winds of more than 60 miles per hour originate from seven of the eight principal compass directions.

Wind roses summarizing annual and seasonal windspeed and direction from the Dover Air Force Base station shows that the most frequently occurring winds blow from the northwest. Monthly data show the wind regime varies from season to season with stronger winter winds prevailing from the northwest and summer winds prevailing from the southwest. The dominant winds (highest velocity) are from the northeast. Figure 3 (after Maurer and Wang 1973) shows the seasonal changes.

II-4. WAVES

Waves within the Bay may be generated by local winds or they may propagate through the mouth of the Bay. In general, the wave direction correlates with dominant wind directions. The dominant winds are from the southwest through northwest quadrants. Consequently, locally-generated waves expend much of their energy on the New Jersey shorelines most of the year.

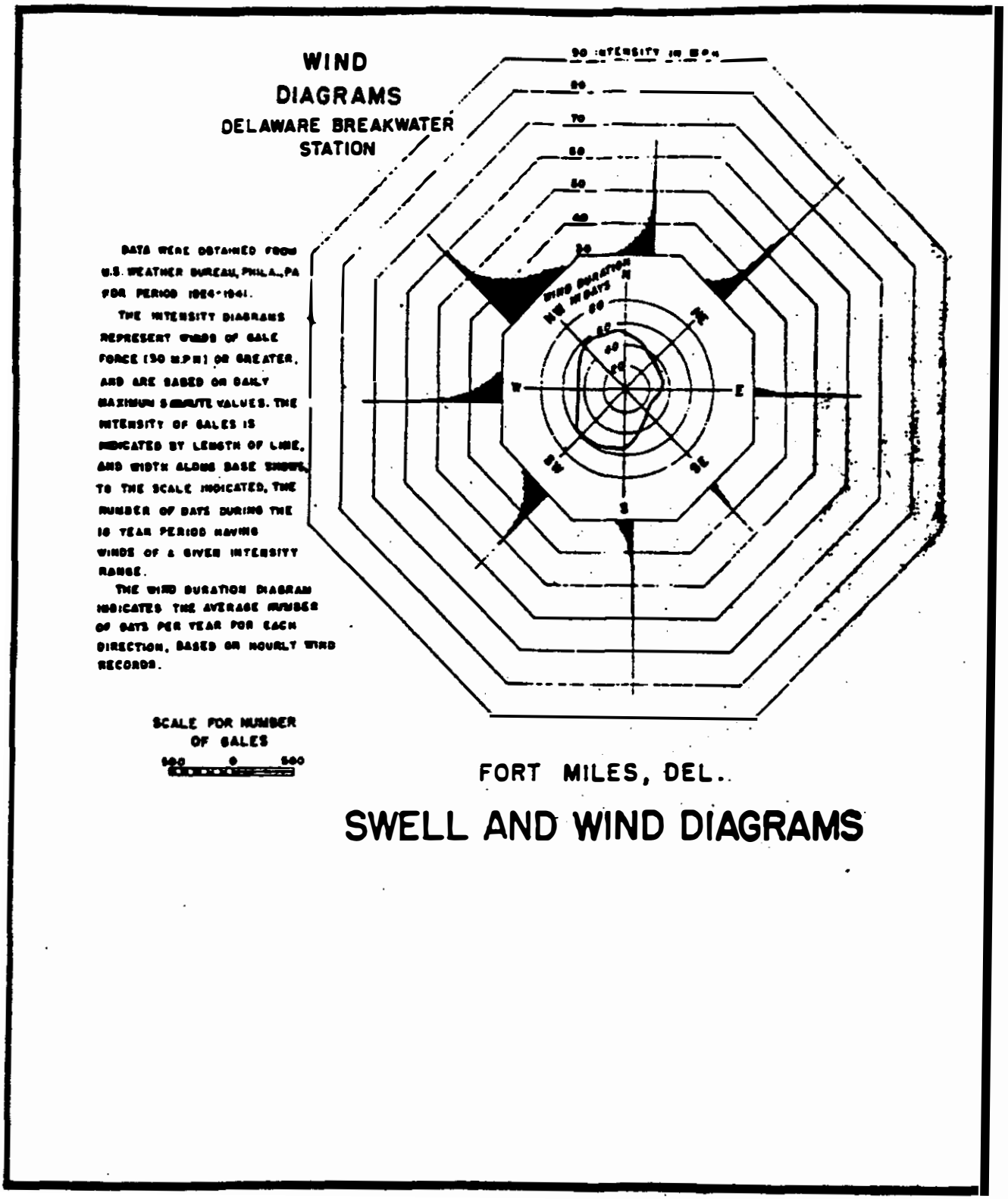

Figure 2 Wind Diagrams
Delaware Breakwater Station

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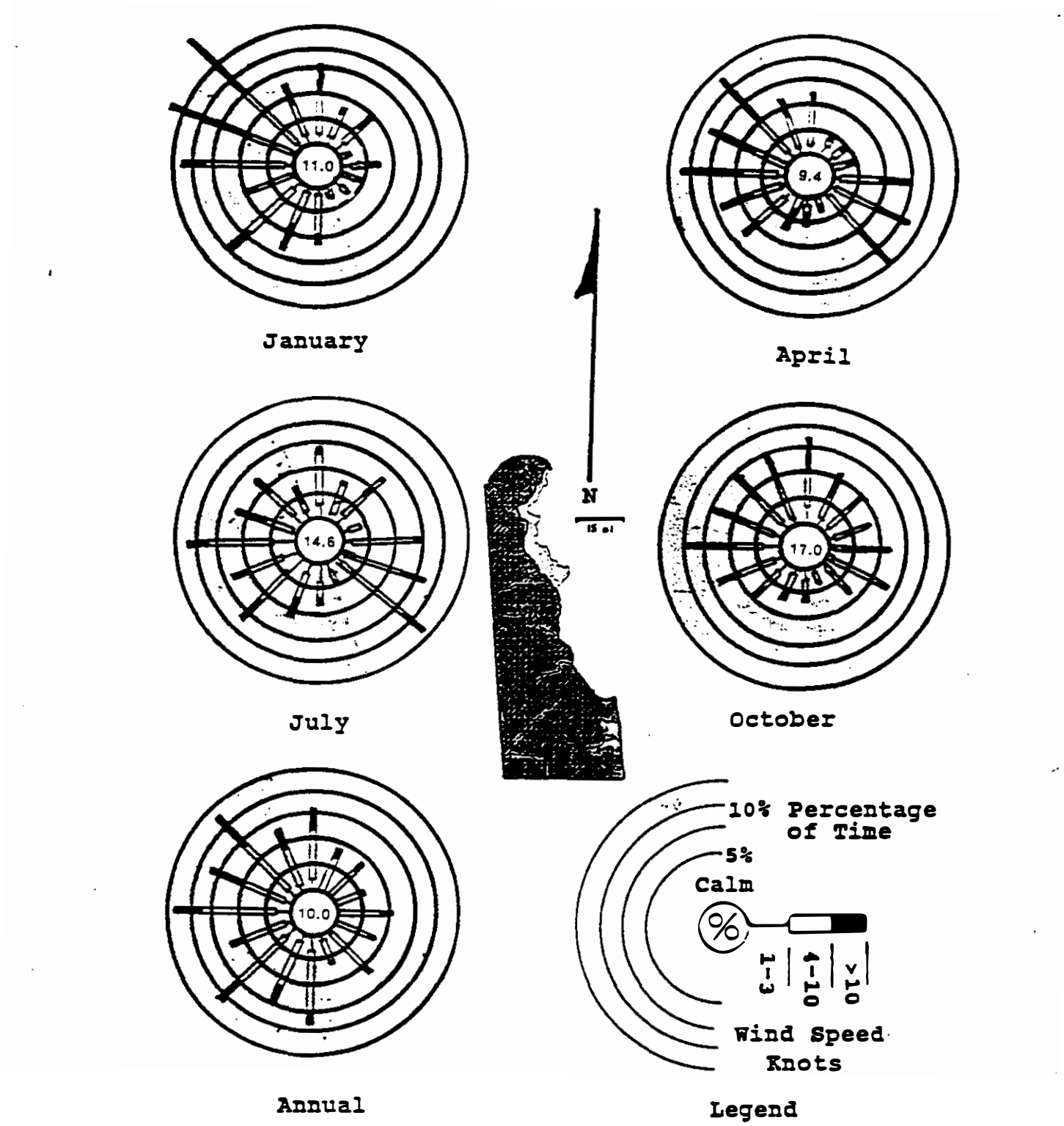

Figure 3 Annual and Seasonal Wind Rose at Dover Air Force Base, after Maurer and Wang, 1973

Significant wave heights and wave directions compiled by Maurer and Wang (1973) indicate that approximately 60% of the waves approach from the northwest in January but shift in late spring to approach dominantly from the southeast. During the fall season, waves are primarily from the north and northeast. Maurmeyer also describes waves in the bay are generally low, averaging less than 2 feet nearly 80% of the time and exceeding 6 feet only 2% of the time.

Wind and swell waves can also propagate through the opening of the bay. Significant wave height data extracted from Station 66 from the Wave Information Study (WIS) Report (Hubertz et al., 1993) indicate that 34% of the waves near the bay entrance are from the southeast and 19% are from the east. The WIS hindcast (1956-1975) reports wave height over 13 feet only 0.9% of the time between the months of September and April inclusively. Wave heights exceeding 14.8 feet only occur 0.2% of the time between October and March. Only 2% of the waves are of a height greater than 8.2 feet. About 66% of the waves are less than 3.3 feet while approximately 50% of the waves are between 1.6 and 3.3 feet.

Extreme wave statistics for the bay shore are available from the Delaware Bay Hindcast Study (USACE, in preparation):

Extreme Wave Heights in Feet
Delaware Bay, New Jersey Shore

Location	Recurrence Interval (yr)						
	5	10	20	50	100	200	500
Cohansey River	3.9	5.9	7.4	9.5	10.8	12.3	14.1
Fortescue	3.3	4.6	5.6	7.2	8.2	9.4	10.9
Maurice River Cove	3.6	4.9	6.1	7.6	8.9	10.0	11.7
Reeds Beach	4.3	5.3	6.4	7.9	8.9	9.6	10.8
Villas	5.3	6.6	7.6	8.9	9.8	11.0	12.5
Cape May Ferry-N	6.2	7.6	8.8	10.2	11.2	12.5	14.0
Cape May Ferry-S	6.9	7.9	9.0	10.5	11.5	12.4	13.7
Cape May Point	7.6	9.2	10.3	11.8	13.1	14.0	15.7

Extreme Wave Heights in Feet Delaware Bay, Delaware Shore							
Location	Recurrence Interval (yr)						
	5	10	20	50	100	200	500
Port Mahon	5.6	6.6	7.3	8.5	9.2	10.0	11.1
Kitts Hummock	6.6	7.9	9.0	10.5	11.8	12.9	14.7
Browers Beach	6.6	7.6	8.5	9.8	10.8	11.5	12.7
Big Stone Beach	6.6	7.2	7.8	8.5	9.2	10.0	10.8
Mispillion Inlet-N	6.9	7.9	8.5	9.2	9.8	10.5	11.4
Mispillion Inlet-S	6.9	7.9	8.6	9.8	10.5	11.1	12.0
Broadkill Beach	7.9	8.9	9.8	11.2	12.1	13.1	15.4
Roosevelt Inlet-N	6.9	7.9	8.3	9.2	9.5	10.4	11.0
Roosevelt Inlet-S	7.2	8.2	9.2	10.5	11.2	12.0	12.9
Lewes Breakwater	10.2	12.1	13.9	16.1	17.4	19.0	21.0

II-5. STORMS

Two types of storms can produce serious damage to the study area. They are defined in terms of their zones of origin; either tropical or extra-tropical. Storms of tropical origin in which wind velocity reaches 74 miles per hour or more are considered to be hurricanes. Hurricanes are generated in the tropical Atlantic and generally follow paths into the Gulf of Mexico or move along the Atlantic coast. Their paths are usually quite erratic and unpredictable. Storms of extra-tropical origin occur when intense low pressure centers form in the mid- to north-Atlantic coasts, and are more frequent than hurricanes. These storms are typically characterized by strong northeasterly winds and are commonly known as northeasters. Tropical storms reach these latitudes with hurricane force only in the summer and fall. Northeasters, though not excluded completely from the warm season, are far more prevalent during the fall and winter.

Hurricanes with sustained wind speeds over 74 mph typically can produce a higher peak surge than northeasters but are more limited in duration. Hurricanes can cause the highest shore erosion in the shortest period of time. The more frequently occurring northeasters, although having lower wind speeds, can cause equal or greater damage than hurricanes due to their longer duration. In the more damaging northeasters of record, including the one in 1962, many miles of beach in the Delaware shore retreated up to 75 feet.

Storms striking the Delaware Bay coast between 1923 to 1992 causing moderate to severe damage to the Delaware Bay coast are presented below (extracted and updated from Friendlander, 1977):

Year	Date	Storm Type	Reported Damage Level
1923	10/12	Northeaster	Severe
1924	9/18	Northeaster	Moderate
1927	2/19-21	Northeaster	Extremely Severe
1929	4/17	Northeaster	Moderate
1932	11/11	Northeaster	Severe
1933	1/26-27	Northeaster	Severe
	8/22-23	Hurricane	Extremely Severe
1935	11/18	Northeaster	Severe
1936	9/18-19	Hurricane	Moderate
1937	4/27	Northeaster	Moderate
1943	10/27	Northeaster	Moderate
1944	9/14-15	Hurricane	Severe
1950	11/25-29	Northeaster	Extremely Severe
1954	10/15	Hazel	Moderate
1956	9/28-29	Flossy	Severe
1957	10/07	Northeaster	Moderate
1960	9/13	Donna	Moderate
1961	10/23-24	Northeaster	Severe
1962	3/06-08	Northeaster	Extremely Severe
1962	11/05	Northeaster	Severe
	11/12	Northeaster	Moderate
	11/28-29	Northeaster	Severe
1964	1/13	Northeaster	Moderate
1968	11/11-13	Northeaster	Moderate
1974	12/01	Northeaster	Severe
1977	10/13-15	Northeaster	Moderate
1980	10/25	Northeaster	Moderate
1984	3/28-29	Northeaster	Severe
1985	9/26-28	Hurricane	Moderate
1991	10/28-31	Northeaster	Severe
1992	1/02-05	Northeaster	Extremely Severe
1992	12/10-13	Northeaster	Severe

The greatest flooding and erosion damages on record in Cumberland County were experienced in 1950. High tide stage was recorded at 7.9 feet NGVD at East Point and 8.5 ft NGVD at the mouth of the Cohansey River. The 1962 storm was the flood of record in Cape May County and west shore communities. Tides reached 7.9 feet NGVD at Lewes, 6.9 ft at East Point and 7.3 feet at the mouth of the Cohansey River.

II-6. TIDES

Tides in the study area are of the semi-diurnal type; i.e. two high waters and two low waters occur in a period of 24 hours and 50 minutes. In this area, succeeding normal high waters differ by less than a foot in elevation. The same is also true of succeeding low waters.

The National Ocean Service (NOS) operates three tide gages in the Delaware Bay and estuary. The gages are located in Breakwater Harbor, Reedy Point, and Philadelphia. The following table lists the mean high water and mean low water elevations relative to NGVD for the three tide stations. Tidal range generally increases upstream of the estuary. The mean tidal range on the New Jersey side of the lower bay may exceed that of the Delaware side by 1 foot, presumably due to the Coriolis effect. The NOS predictions indicate that it takes 7 hours for high-water phase to propagate from Breakwater Harbor to Trenton and it takes 8.5 hours for low-water phase to travel the same distance.

Location	Mean High Water (ft NGVD)	Mean Low Water (ft NGVD)
Breakwater Harbor, DE	2.6	- 1.6
Reedy Point, DE	3.1	- 2.3
Philadelphia, PA	4.1	- 2.1

Harris (1981) reports the relationship between the MLW and the MSL, MTL and NGVD for the reference tide stations as follows:

STATION	MSL	MTL	NGVD	MHW
Breakwater Harbor	2.1	2.05	1.69	4.1
Reedy Point	2.8	2.75	2.45	5.5
Philadelphia	3.2	3.10	2.14	6.19

In addition to the three tide stations, NOAA also publishes predictions of high and low tide at secondary tide stations. The following table summarizes the normal (or mean) range of tides, and the spring range for various Delaware Bay locations (Harris, 1981).

Tidal Range for Delaware Bay Locations

Location	Mean (ft)	Spring (ft)
Delaware Shore		
Cape Henlopen	4.1	4.9
Breakwater Harbor	4.1	4.9
Roosevelt Inlet	4.4	5.2
Mispillion River	4.6	5.4
Murderkill River	4.8	5.7
St Jones River	4.8	5.7
Mahon River	5.4	6.3
Leipsic River	5.5	6.4
Woodland Beach	5.9	6.8
New Jersey Shores		
Cape May Point	4.7	5.6
Bay Shore Channel	4.9	5.8
Miami Beach	5.1	6.1
Dennis Creek	5.6	6.6
East Point	5.7	6.7
Egg Island Point	5.7	6.7
Fortescue	5.9	7.0
Ben Davis Point	6.0	6.9
Cohansey River	6.0	6.9

Figure 4, reprinted from the NOAA Tidal Circulation and Water Level Forecast Atlas (NOAA, 1987), shows the mean maximum tidal height contours in the Delaware Bay. The NOAA report noted that except for persistently strong winds (20 to 40 knots) for several days, tide predictions in the Atlas are within 1.2 feet of observed water levels. Water levels at the entrance can be increased by persistent southerly or southeasterly winds and decreased by northerly or northwesterly winds.

II-7. CURRENTS

Tidal currents in the estuary are directly related to the astronomical tidal elevations and as such vary with the phase and amplitude of the various tidal constituents. Peak ebb and flood currents are largest along the axis of the bay and decrease toward either side. Using a hydrodynamic model, the National Ocean Service published a tidal circulation atlas for the Delaware River and Bay (NOAA, 1987). The charts show the speeds and directions of the tidal current in the Delaware River and Bay for each hour of an average tidal cycle. The current charts reflect the effects of channels, shoals, and other bathymetric features but do not include any meteorological effects or river flow. River runoff can considerably modify the speed and direction of currents in the bay. Strong winds can cause nontidal currents.

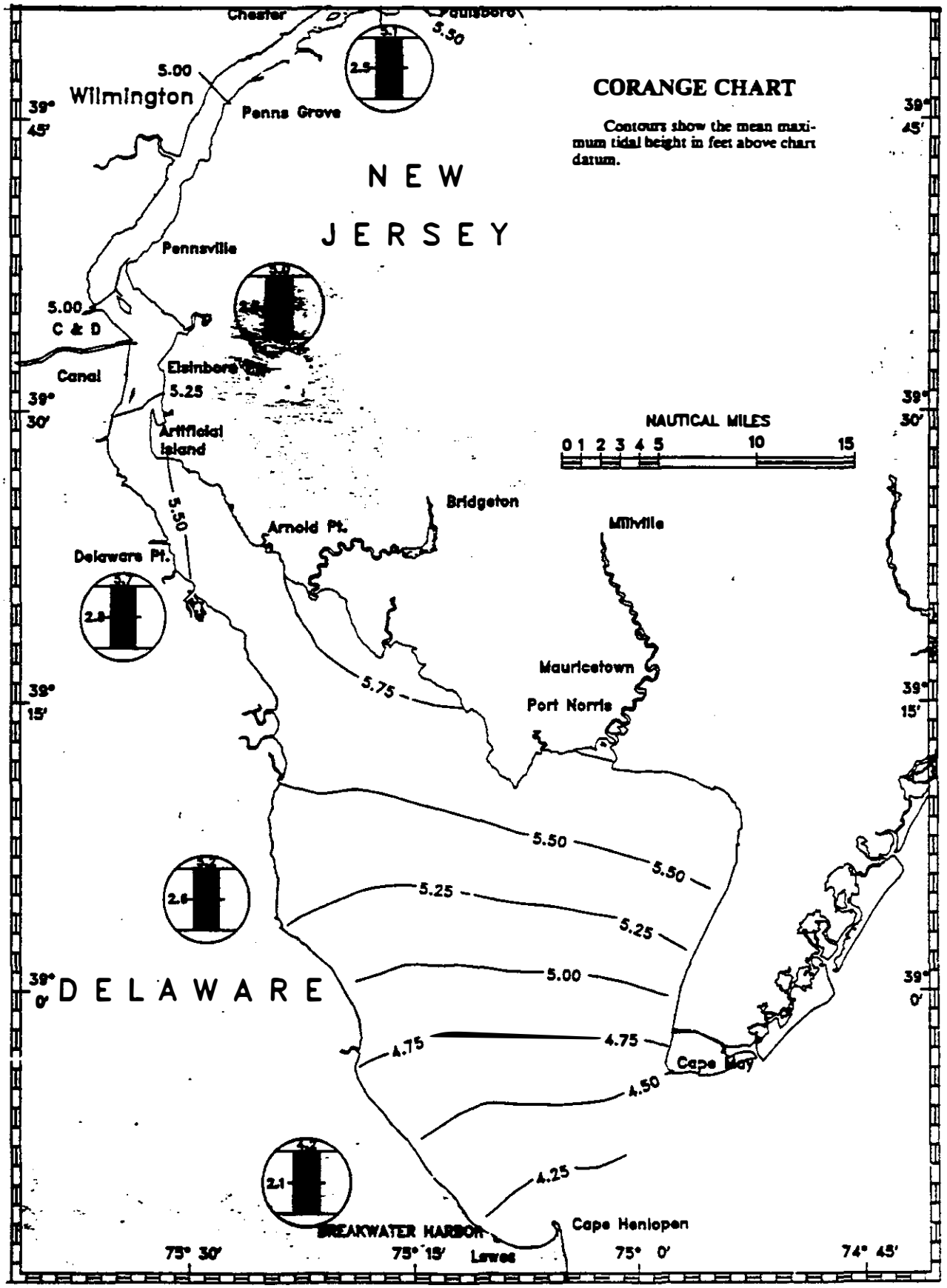

Figure 4 Maximum Tidal Height Contours
(NOAA, 1988)

Longshore current velocities landward of the breaker zone were measured by Maurmeyer (1978) along the Delaware shorelines from July 1976 to September 1977. These longshore current directions and velocities are presented below:

Longshore Current Directions and Velocities
Western Delaware Bay

(after Maurmeyer, 1978)

Location	Date	Current Speed (ft/sec)	Direction
Bennetts Pier	7/08/76	0.98	South
	10/05/76	0.66	South
	3/29/77	0.16	North
	9/28/77	0.79	South
Broadkill Beach	7/13/76	0.33	South
	3/21/77	0.46	Northwest
Fowler Beach	7/13/76	0.43	South
	12/17/76	0.33	South
	6/29/77	0.26	South
	9/28/77	0.52	South
South Browers	7/14/76	0.29	South
	10/05/76	0.43	South
	10/12/76	0.39	North
	1/10/77	0.33	South
Pickering Beach	9/28/76	0.75	South
	6/08/77	0.26	South
Kitts Hummock	9/29/76	0.78	North
	7/14/77	0.82	North
	9/28/77	0.46	South
Slaughter Beach	10/05/76	0.49	South
Primehook Beach	10/05/76	0.62	South
Big Stone Beach	10/05/76	0.85	South
	5/17/77	0.39	South
Plum Island	3/21/77	0.30	Northwest
Lewes Beach	12/20/76	0.10	Northwest
	3/21/77	0.20	Northwest
Woodland Beach	7/13/77	0.33	South
Conch Bar	7/25/77	1.31	South

From the entrance of the Delaware River and Bay to Artificial Island, the effects of wind generally dominate over river effects. However, increases in ebb currents in the lower bay have been observed when there is a large increase in the river flow. During periods of northerly or northwesterly winds, ebb currents increased and delayed times of weaker floods. Stronger ebbs were also observed after periods of increased water levels resulting from easterly or southeasterly winds. This effect can persist for up to 2 days. Northwesterly winds produced an opposite effect by temporarily lowering the water levels throughout the bay followed by a return to normal water levels.

Figures 5, 6 and 7 from the NOAA Tidal Circulation Atlas display the tidal circulation in lower Delaware Bay at the time of highwater, and 4 and 8 hours after highwater. As shown in Figure 7, at 8 hours after highwater, as the tide starts moving into the Delaware Bay entrance, the water in the upper bay is still flowing downstream. This creates a counterclockwise circulation in Maurice Cove. Figure 8 shows the contours of maximum flood current for a mean tidal range.

II-8. BEACH SEDIMENTS

The available information concerning physical characteristics of the littoral materials along the shores of Delaware Bay dates back to 1929. However, prior to 1954, only a limited number of samples were taken. During 1954 and 1964 the COE conducted beach surveys and collected beach samples at specific survey locations (USACE, 1966). The 1954 survey included 120 samples along the western Delaware Bay shore. Those samples were obtained along profiles normal to the shoreline at approximately one mile intervals and are representative of the littoral material above mean high water, in the mid-tide zone and from the nearshore bottom. The 1964 survey included 93 samples along 16 of the 52 line profiles surveyed. The beach samples obtained in 1964 were taken from a depth of 2 to 4 inches below the surface.

Analysis of both the 1954 and 1964 samples taken above mean high water shows fine to medium sand at mid-tide level. The 1954 sample data indicated medium sand at mid-tide level along the entire west bay shore, while the 1964 sample showed medium to coarse sand below Fowler Beach and fine to medium sand above Fowler Beach. Analysis of both the 1954 and 1964 nearshore samples showed fine to medium sands. A small amount of silt was found in the offshore samples.

Figure 9 shows four representative grain-size curves selected from the western bay shore as presented in the Section 54 Final Report (USACE, 1981). The shaded area of the graph indicates the range of gradations where most of the samples commonly fell.

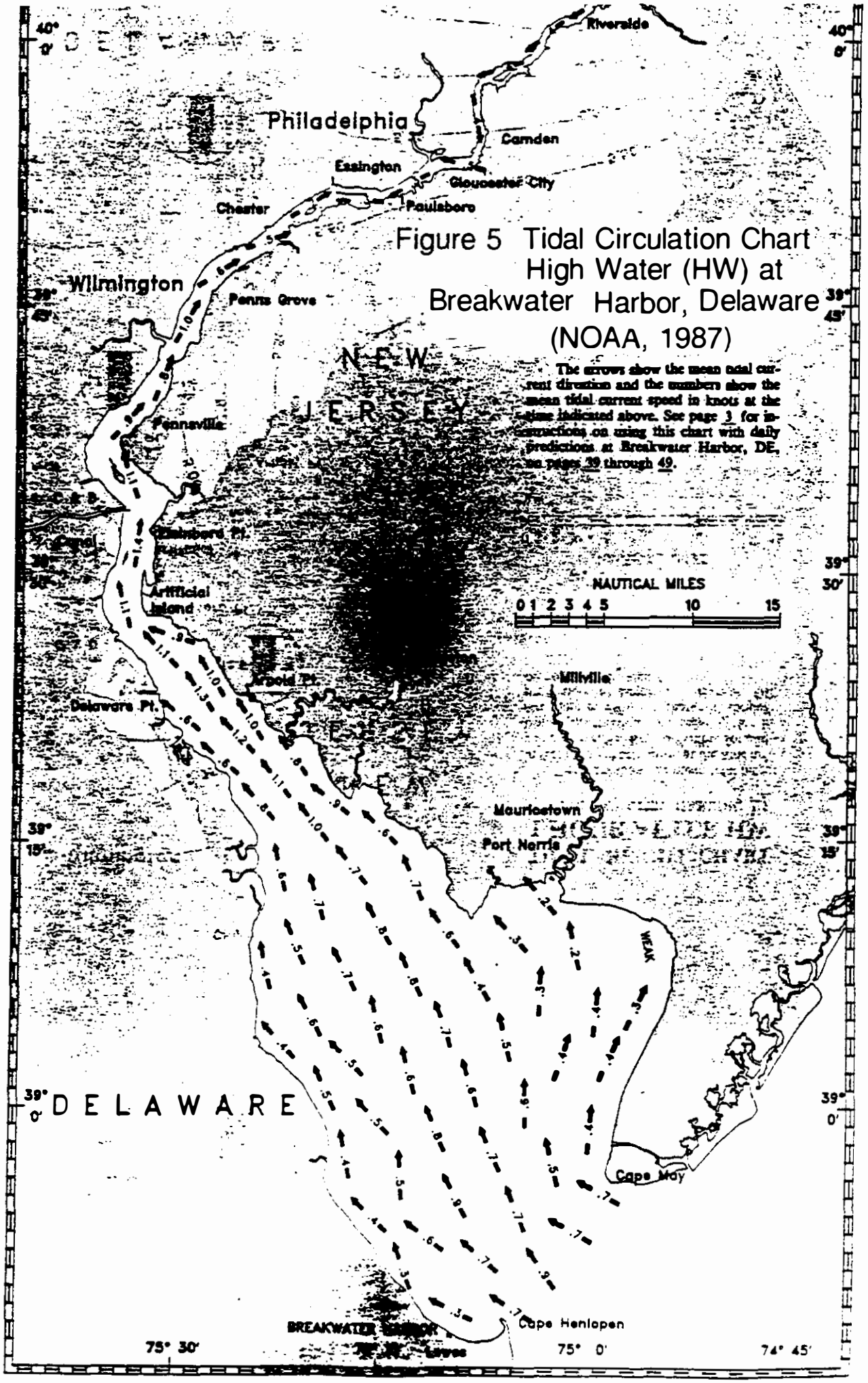

Figure 5 Tidal Circulation Chart High Water (HW) at Breakwater Harbor, Delaware (NOAA, 1987)

The arrows show the mean tidal current direction and the numbers show the mean tidal current speed in knots at the time indicated above. See page 3 for instructions on using this chart with daily predictions at Breakwater Harbor, DE, on pages 39 through 49.

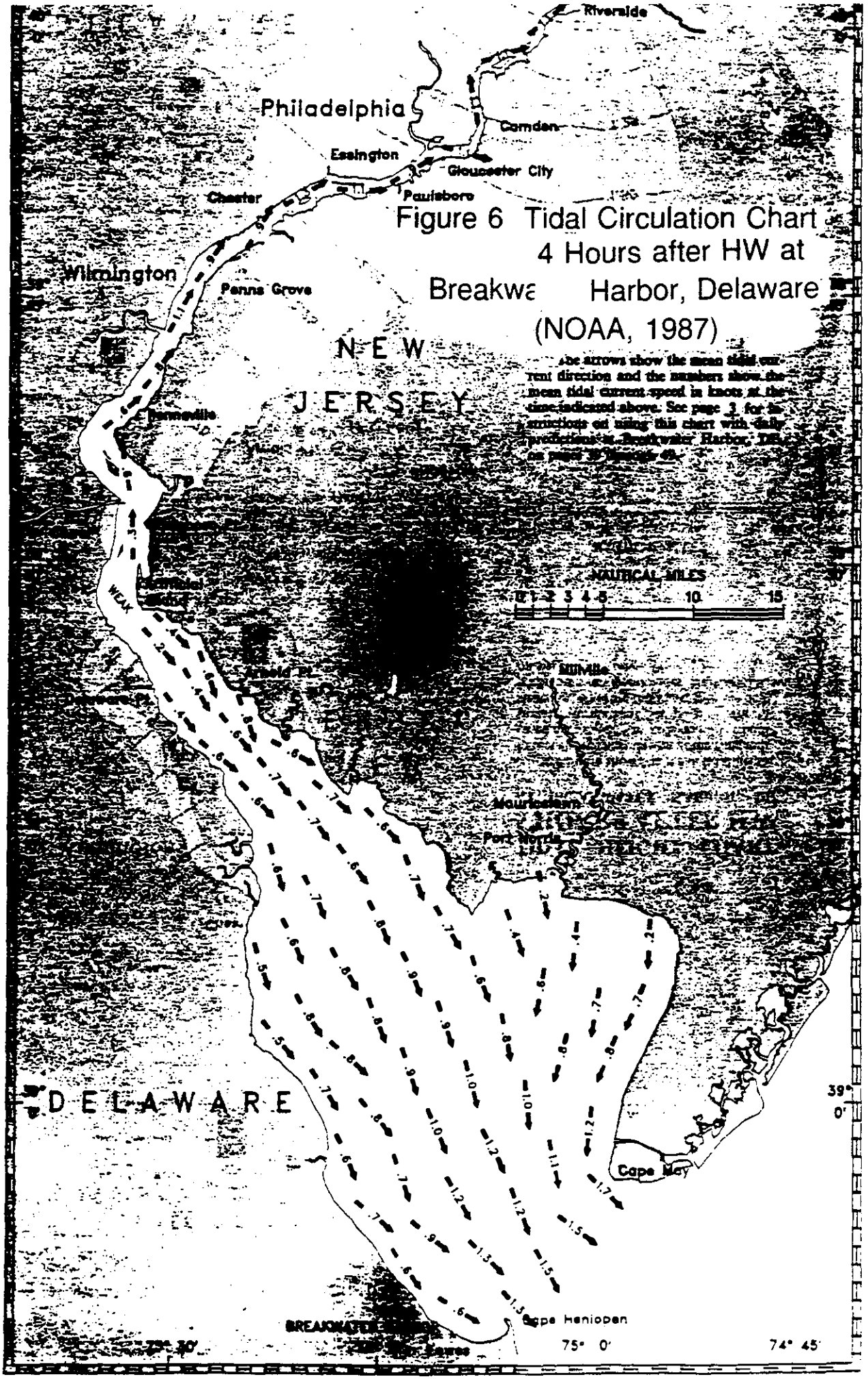

Figure 6 Tidal Circulation Chart
4 Hours after HW at
Breakwa Harbor, Delaware
(NOAA, 1987)

The arrows show the mean tidal current direction and the numbers show the mean tidal current speed in knots at the time indicated above. See page 3 for instructions on using this chart with daily predictions at Breakwater Harbor, DE, on pages 39 through 49.

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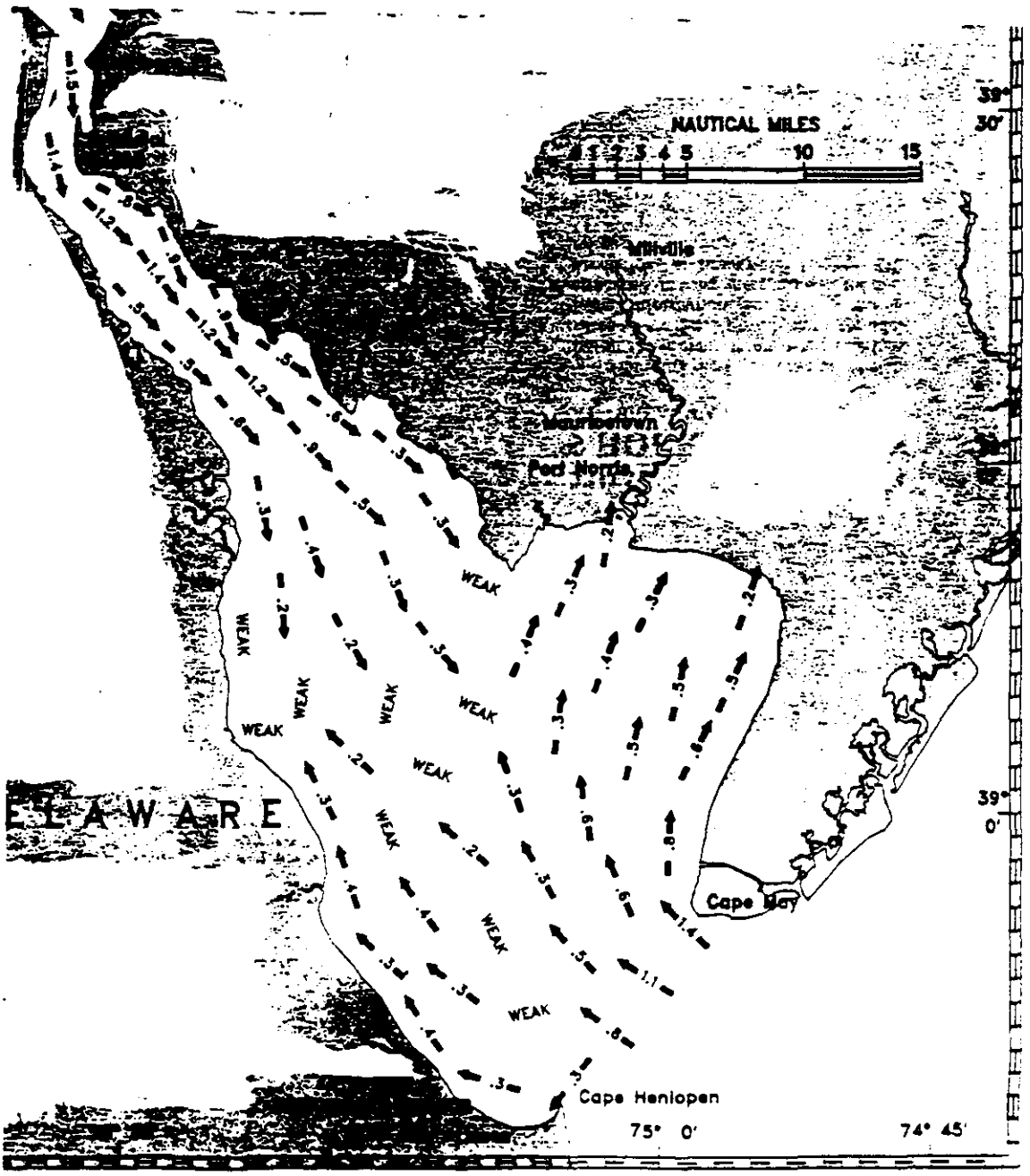
Figure 7
Tidal Circulation Chart
8 Hours after HW at
Breakwater Harbor, Delaware
(NOAA, 1987)



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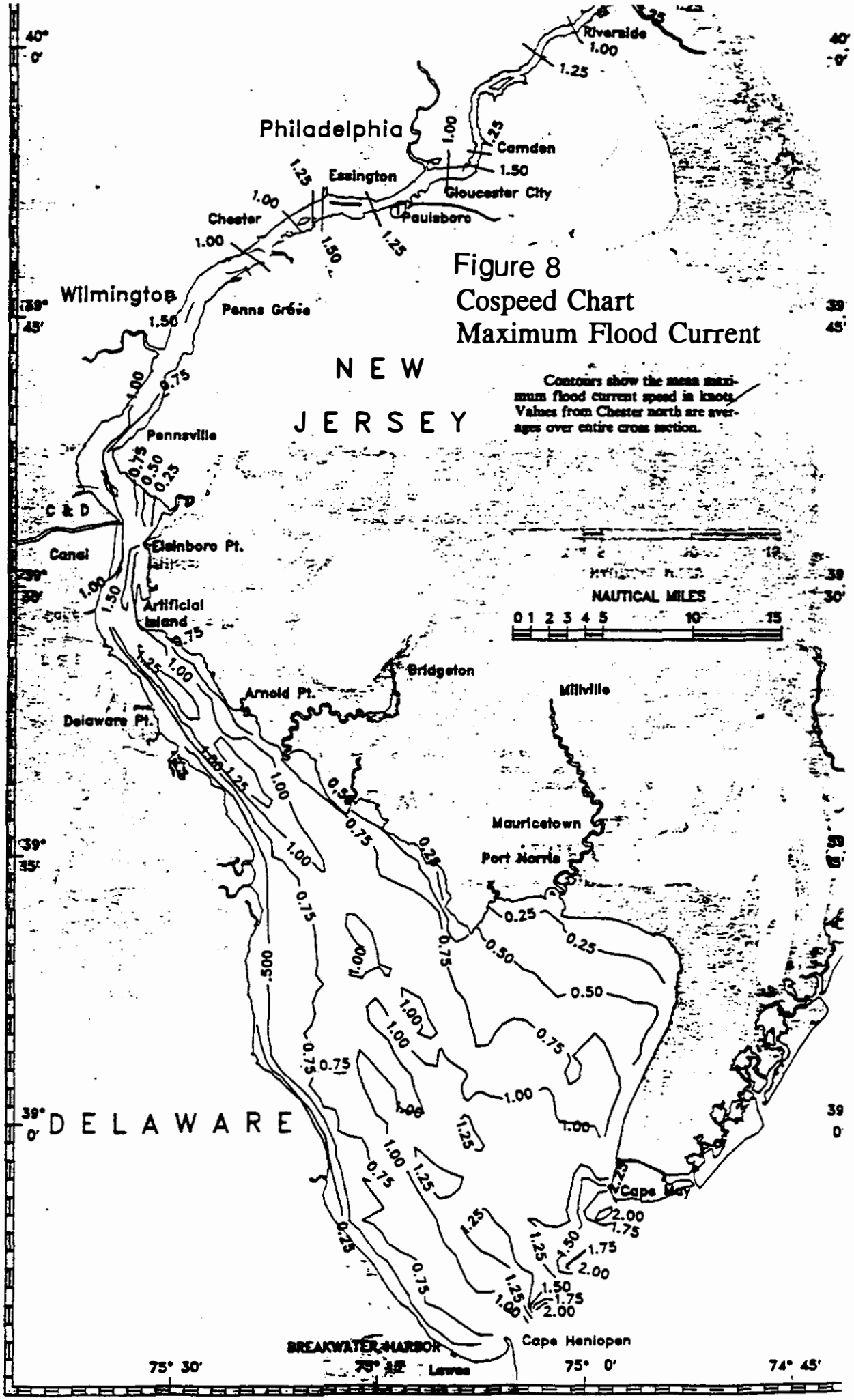
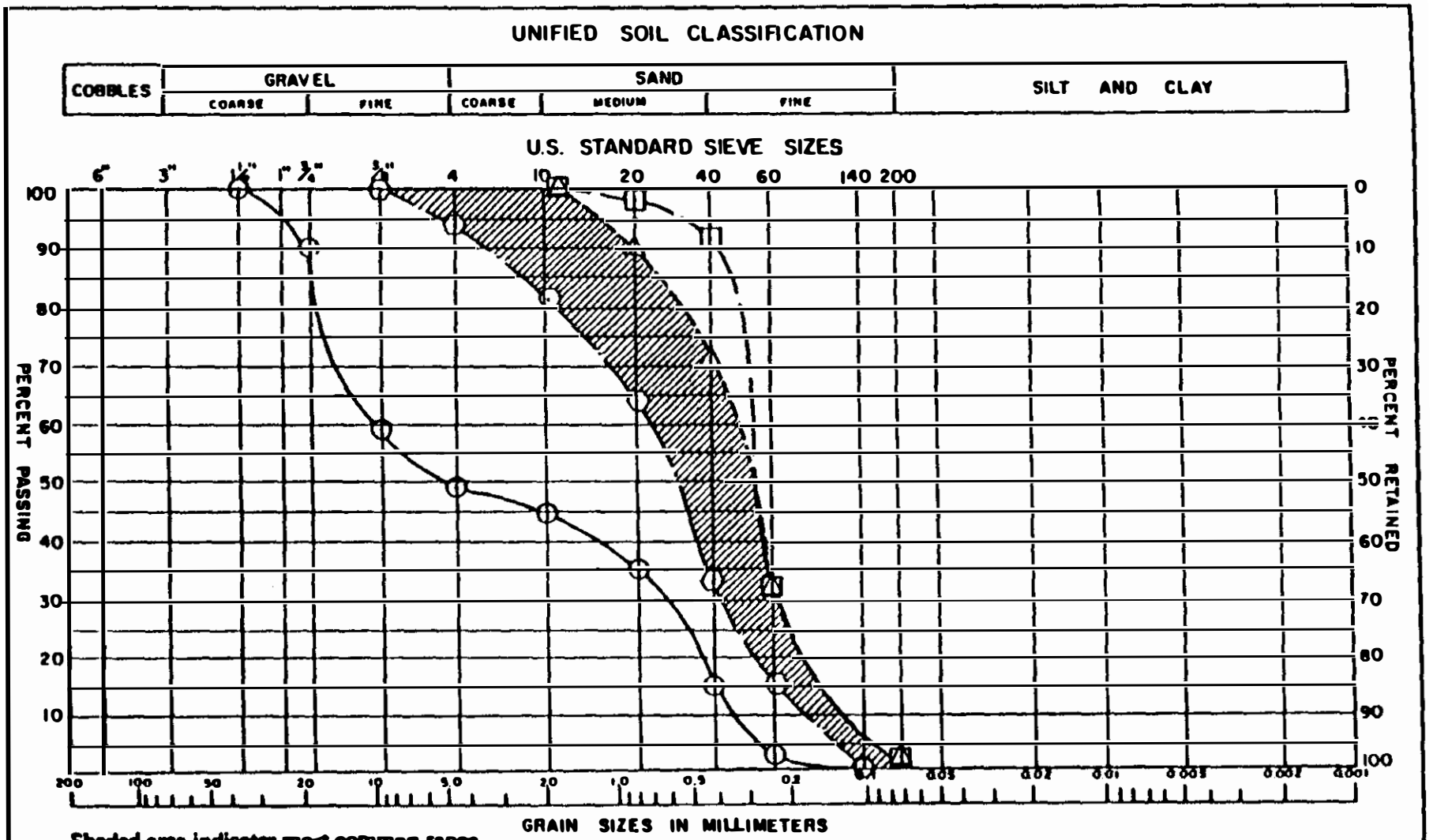


Figure 8
Cospeed Chart
Maximum Flood Current



Shaded area indicates *most common range*.
Samples collected on beach at high water line.

LOCATION	STATION	SYMBOL	LL	PI	CLASSIFICATION
Bowers	10 + 00B	⊙——⊙			Sandy gravel to gravelly SAND (GW–SW)
Broadkill Beach	4 + 35N	△- - - -△			Medium SAND (SP)
Kitts Hummock	17 + 30S	⊙—·—⊙			Medium SAND (SP)
Slaughter Beach	13 + 25S	⊡——⊡			Fine SAND (SP)

Figure 9 Representative Grain-Size Distribution Curves, Delaware Bay Sites (USACOE, 1981)

Phillips (1985) compared sand samples taken at the Cumberland County shores and concluded that the sand was generally of medium size (mean grain size 0.46 mm, standard deviation 0.8 mm) and moderately well sorted. Overall, size ranged from 0.28 to 0.8 mm.

Beach face sediment samples collected by Maurmeyer and USACE are summarized in the following table. The data indicates that the beach sediments were consistently medium to coarse sand, with some very coarse sand.

Beach Face Sediment Size Delaware Shore			
Location	Median Sediment Size (mm)		
	Maurmeyer 1978	USACE 1954	USACE 1964
Bombay Hook	0.339	-	-
Pickering Beach	0.724	-	0.53
Kitts Hummock	0.555	0.57	0.62
Bowers Beach	0.586	0.98	0.45(*)
Bennetts Pier	0.587	-	-
Big Stone Beach	1.117	-	0.58
Mispillion River	0.708	-	-
Slaughter Beach	1.125	0.45	0.46(*)
Fowler Beach	0.739	-	-
Primehook Beach	0.809	-	0.39(*)
Broadkill Beach	0.669	0.59	-
Lewes Beach	0.896	0.61	-
Breakwater	0.645	0.99	-

* these samples were taken above MHW between elevations 5 to 14 MLW

II-9. HISTORIC SHORELINE ANALYSIS

The rate of shoreline change along Delaware Bay varies considerably along shore and over time. During the past century, in general, the bay shoreline has undergone continuous erosion averaging from 1 to 25 ft/year. The small areas of accretion are localized, partially caused by man's activity.

Figure 10, prepared by the Delaware State Planning Office (1976), shows rates of shoreline erosion and accretion from 1848 to 1972. Most shoreline erosion is believed to be caused by waves generated within the Bay by local winds. The most serious erosion has occurred during northeast storms. Many miles of beach retreated 60 to 75 feet during the storm of March 1962. Noticeable accretion occurs between storm periods, but the net result is loss of beach. While Delaware Bay is partially sheltered from severe storms, its low shoreline, lack of large dunes and susceptibility to storm tidal surge contribute to severe erosion during intense storms.

The Beach Erosion Control and Hurricane Protection study (USACE, 1966) compared the beach and near shore profiles surveyed in 1954 and 1964, as well as other profiles taken between 1842-1943 and concluded that the areas offshore have undergone little gradient change since the 1842-43 survey. Offshore slopes are exceedingly flat, approximately 1 on 500. Foreshore slopes in 1964 were 1 on 15 generally compared to an average slope of 1 on 20 in 1954. In the Cape Henlopen section, the rapid northward movement of the tip of the Cape was accompanied by a steepening of the slope below the mean low water level from a slope of approximately 1 on 27 in 1884; 1 on 6 in 1929; 1 on 9 in 1954; and 1 on 8 in 1964. The volumetric changes between Kitts Hummock and Lewes during the period of 1954 to 1964 indicated that, in general, erosion occurred above mean low water and accretion occurred below mean low water. It was estimated that over this 10-year period, approximately 532,000 cubic yards of material was lost annually above mean low water and approximately 428,000 cubic yards was gained annually below mean low water, for a net loss of 10,400 cubic yards per year.

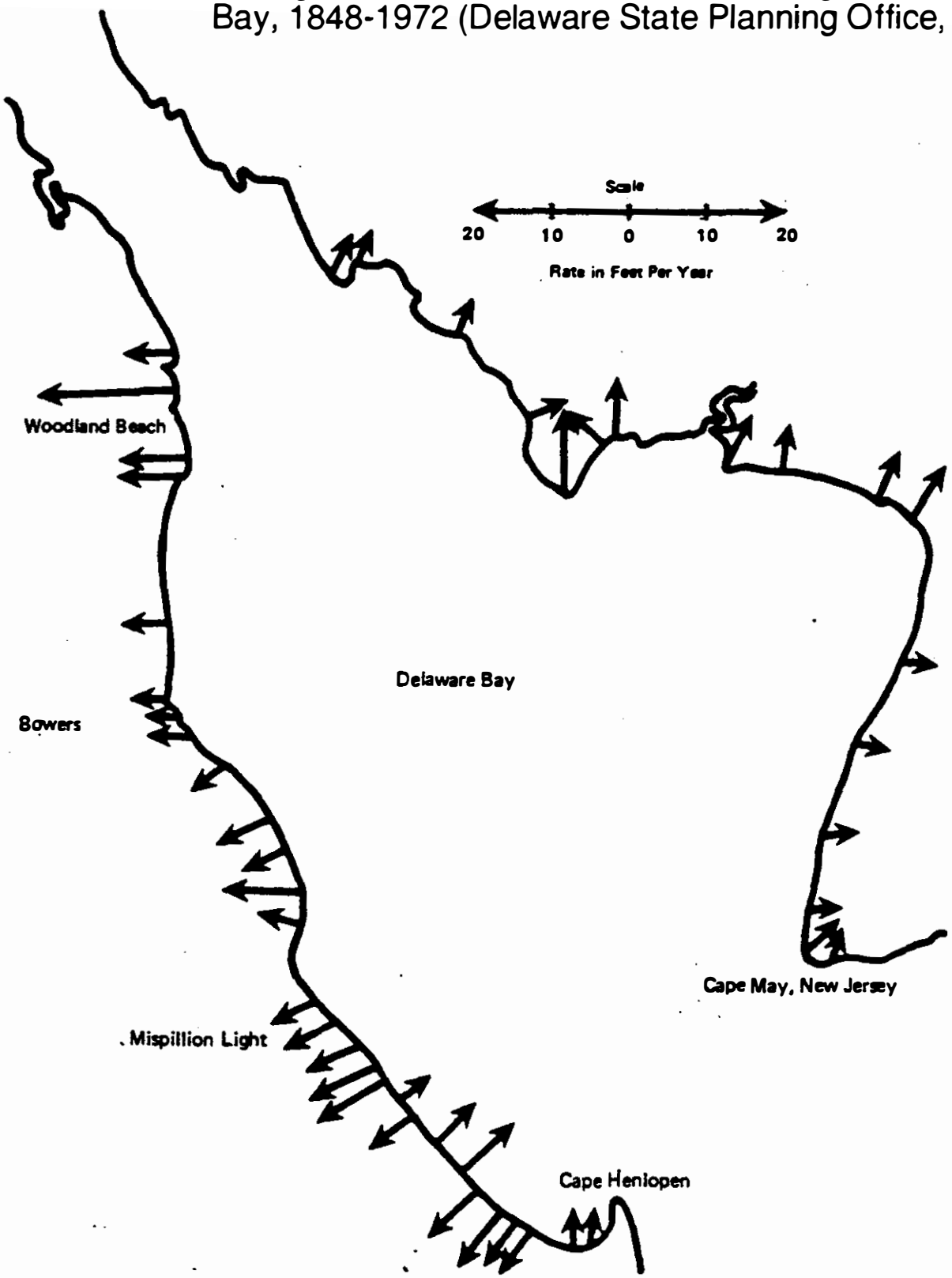
The study of historical rates of shoreline and volumetric changes around the Delaware Bay communities dates back to 1950 or earlier. The most recent and comprehensive shoreline change study on the west shore of Delaware Bay was performed by the University of Maryland (French, 1990). French compared historical maps and photographs and documented a 135 year erosion pattern. The average erosion rate reported by French is 4.5 ft/yr along the Delaware shoreline. Figure 11 displays the general pattern of shoreline changes along the Delaware shore by French. Dividing the shoreline in three sections, the following erosion rates for time spans between 1882 to 1977 were reported as:

Average Erosion Rates (ft/year)
(after French, 1990)

	1882-1977	1943-1954	1954-1969	1969-1977
Port Mahon to Pickering	-17.2	-12.3	-18.9	-25.8
Pickering to Mispillion	- 4.8	+ 2.6	- 7.2	- 2.4
Mispillion to Lewes	- 1.7	+ 1.6	- 4.7	+ 4.4

Figure 10

Average Annual Rates of Shoreline Change in Delaware Bay, 1848-1972 (Delaware State Planning Office, 1976)



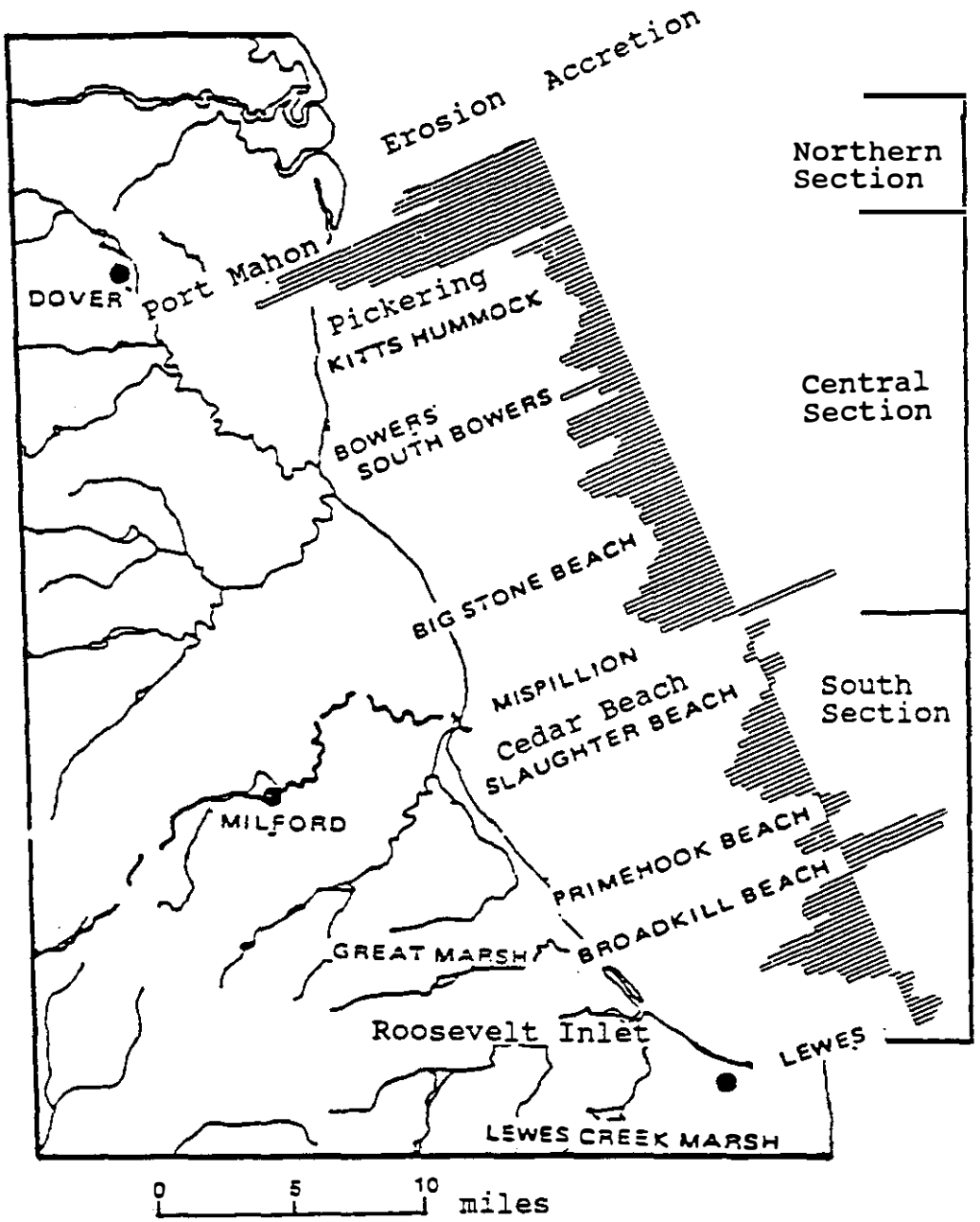


Figure 11 General Pattern of Shoreline Changes Along the West Delaware Bay Coast, 1882g-1977 (French, 1990)

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The southern section (Mispillion River to Lewes) experienced the lowest overall long-term erosion rate but has the greatest variability both temporally and spatially. Throughout the study span, erosion alternates with stability and accretion suggesting that the erosion is largely storm-driven. Because of the overall shape of the bay, this section has the longest fetch from the north to the northeast. Also, due to proximity to the bay mouth, it is influenced by ocean swell waves and experiences the highest wave energy in the bay area.

The central section extends from Mispillion River Inlet to just north of Pickering Beach. This section of shoreline is oriented in a general northwest-southeast direction, curving slightly westward from Mispillion River Inlet of Bowers where the shoreline turns north. The earliest data (1842) show this section to be entirely erosional, having experienced no areas of any long-term accretion. The irregular pattern of recession over time suggests that the shoreline retreat along this section is largely storm-driven. Because of the distance from the bay mouth, swell waves have little or no effect on local nearshore currents. This leaves locally generated waves as the primary mechanism driving the irregular and variable local longshore drift. Since the bay begins to narrow along this section, the available onshore wave energy is further reduced.

Both the south and central sections have been subjected to extensive beach nourishment and groin emplacement which has either reduced the "natural" rate of erosion, or has contributed to limited accretion. Using Slaughter Beach for example, the following table shows the erosion/accretion pattern before and after the beach fills and groin field.

Shoreline Change Rate for Slaughter Beach (ft/yr)
(French, 1990)

Transect	1882-1943	1943-1954	1954-1969	1969-1977	1882-1977
74	-0.9	20.2	-11.5	12.7	-0.1
75	0.3	16.1	- 8.5	8.7	0.6
76	-0.9	18.3	- 7.3	13.7	0.5
77	0.9	7.9	- 5.7	26.0	1.9
78	1.0	12.5	- 5.8	14.1	2.0
79	1.4	8.3	- 2.7	8.7	1.9
80	-0.6	0.9	- 0.8	4.6	-0.1
Mean	0.2	12.0	- 6.0	12.6	1.0

Figure 12 shows the historical shoreline changes of various transect locations. As shown on the above table, although the long-term shoreline change rate (1882-1977) at Slaughter Beach averaged a slight accretion of 1 ft/yr, the shoreline receded between 1954 and 1969 despite that a field of 20 timber groins was built between 1940 and 1957, and a total of 270,000 cubic yards of sand was placed between 1958 to 1962. Shoreline accreted between 1969 and 1977, perhaps due to additional 457,200 cubic yard of fill placed in 1975 and 1976. The 1946 and the 1954 shoreline positions clearly show the location and effects of the groins.

At Broadkill Beach, the erosion/accretion pattern is affected more by man's activities. Figure 13 reveals some dramatic shoreline changes from 1882 to 1977. As shown in Figure 13, the northern part of the town is built on land bayward of the 1882 shoreline. The area has received a relatively large supply of sand during the last 200 years. It is believed that much of this material came from sediment movement around Cape Henlopen. The growth of Cape Henlopen and construction of the two breakwaters near the Cape, however, interrupted the flow of sediment. In the 1950's after the present town site was established, three groins were built, but erosion continued to the north. Two groins were added in 1954, and 2 more constructed in 1964. In addition, approximately 700,000 cubic yards of sand were placed in Broadkill between 1957 and 1976. Detailed analysis of shoreline change for Broadkill Beach can be found in Section IV.

The southern section has experienced the greatest human development (approximately 63% of the coastline is developed) compared to the center section where 17% of the shoreline is occupied and the northern section with less than 1% of its shores inhabited (French, 1990). As demonstrated in the example of Slaughter Beach and Broadkill Beach, since the area became populated, every effort has been made to abate the continuous shoreline recession. Groins have been used extensively at nearly every site of human occupation. Records of beach fills and shore protection structures for the Delaware shore are summarized in Appendix A.

The northern section is an area of extensive marsh development extending from the shore several miles inland. There are only a few thin strands of sand found along the shoreline. The available fetch from the north and northeast is considerably narrower than the two other sections, and there is little or no longshore sediment transport. Yet, this is the most highly erosional section of the study area with the long-term erosion rate up to -50 ft/yr at some locations. The widely held belief that marsh grasses inhibit erosion is not supported here. The spatial pattern of erosion does not exhibit a step-wise character of retreat. This suggests that the erosion is not storm-driven, but instead retreats at a relatively regular high rate. The reason for rapid erosion of this marsh is not fully understood.

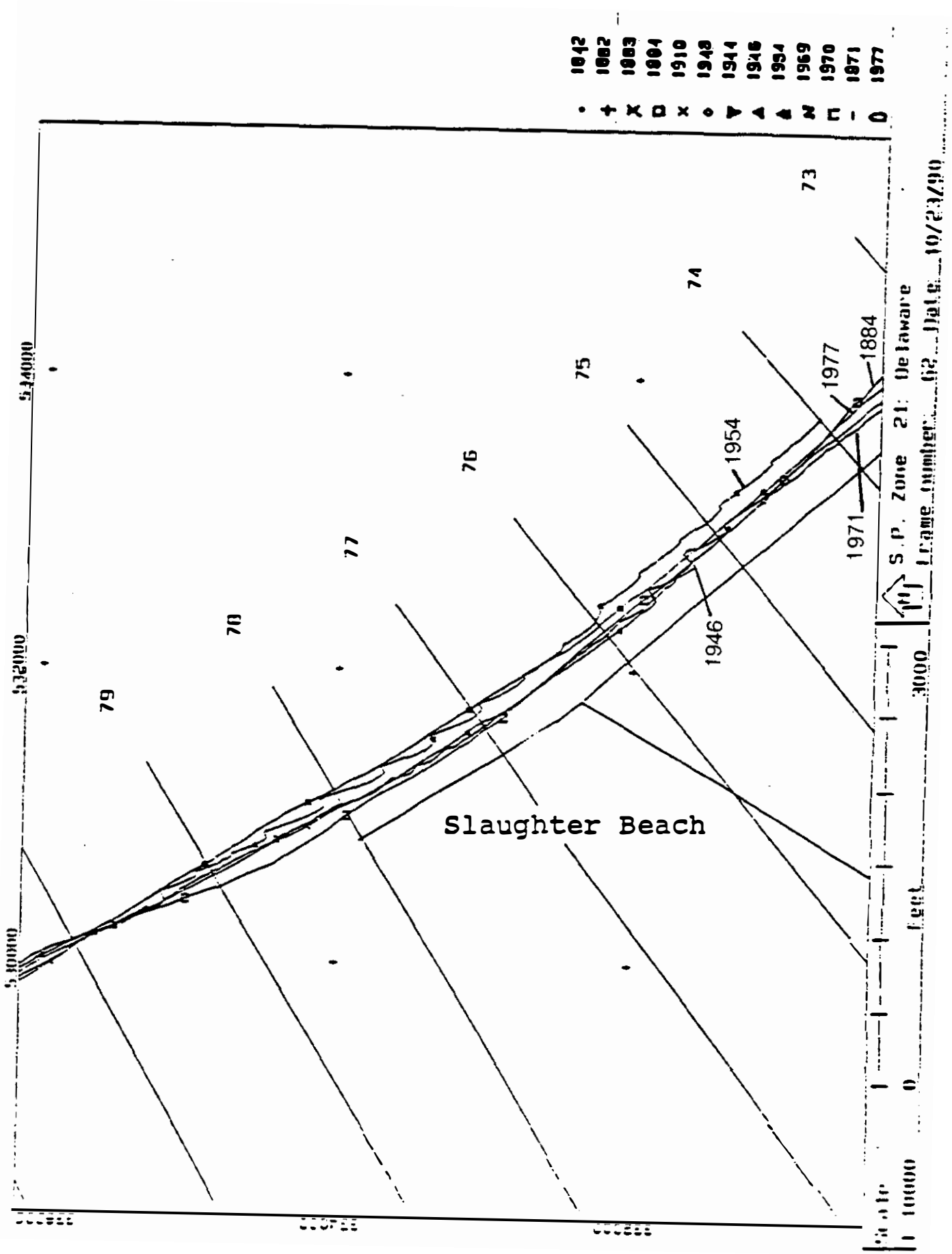

Figure 12 Index Map, Transect Location at Slaughter Beach (French,1990)

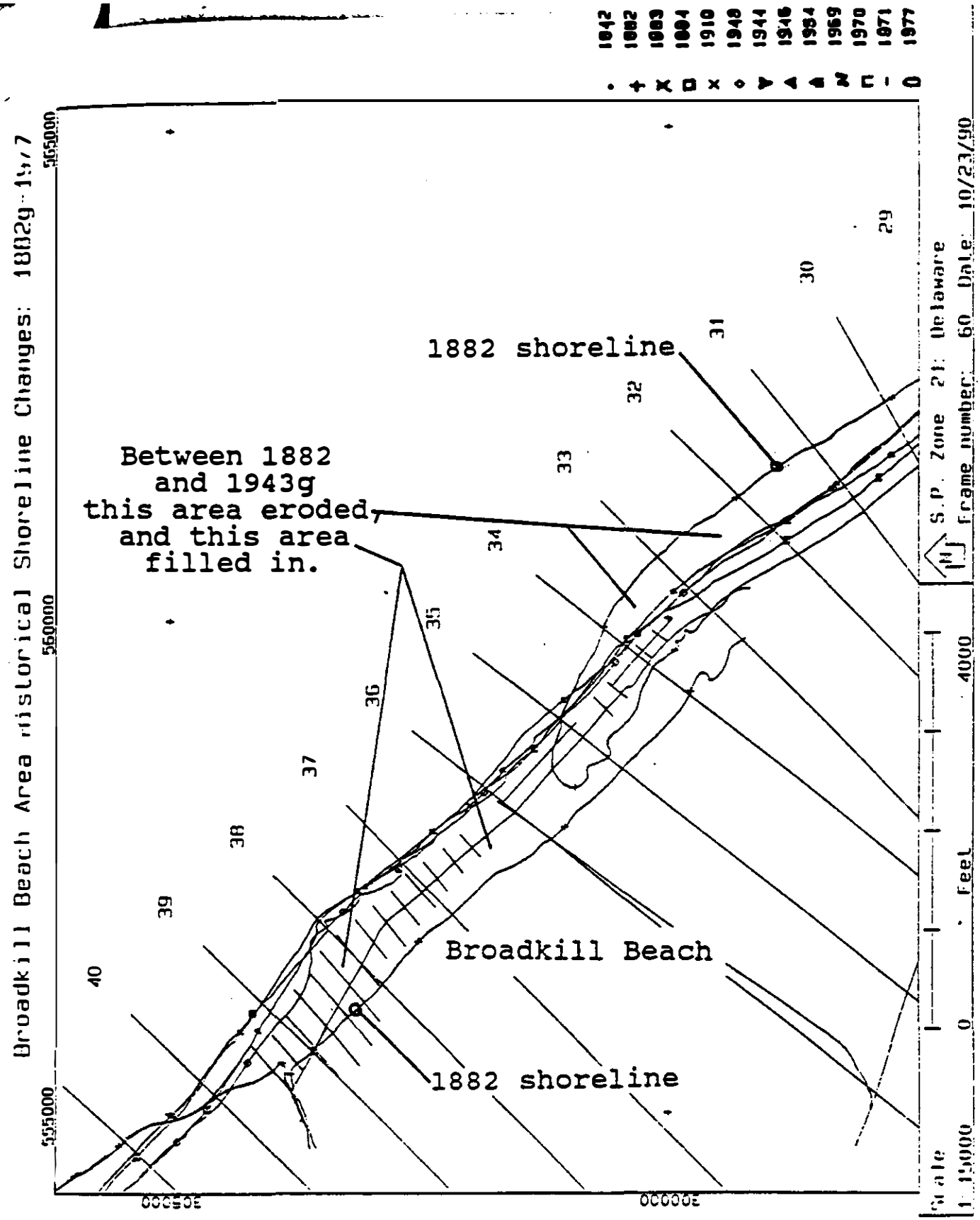

Figure 13 Index and Shoreline Change Map, Broadkill Beach for the Period of 1882-1977 (French, 1990)

Along the New Jersey shore, University of Maryland prepared a shoreline change study in 1986 for the state of New Jersey, Department of Environmental Protection. FEMA recently completed a pilot study for the historical shoreline changes for Cape May County (FEMA, 1993). In Cumberland County, a study was conducted at Rutgers University to determine the spatial variation and geomorphic controls of erosion on the New Jersey shore (Phillips, 1985). Similar to the Delaware shore, the northern marsh in the New Jersey shores experienced higher erosion than the more sandy Cape shores.

The FEMA shoreline change pilot study for Cape May County uses a technique similar to the Delaware shore study. A total of 808 transects along the bay shore were analyzed for shoreline changes. The shoreline positions were obtained from NOAA T-Sheets dated back to 1842 and 1971, 1977 and 1986 ortho-photographs. The Cape May County bayshore has been divided into 10 subsections as described below. Figure 14 shows the subsection locations. The erosion rates histograms for each subsection are presented in Appendix B.

Subsection 1A extends from West Creek to north of Bidwell Creek. This section is an unstabilized shoreline that has historically experienced erosion. The coast is largely composed of marsh, with some limited narrow stretches of sandy beach. The largest amount of historic erosion (just over 20 feet per year) occurred near Dennis Creek which corresponds to the area where the shoreline changes orientation and where waves propagates from the southwest.

Subsection 1B extends from Bidwell Creek to Pierces Point. This section includes Reeds Beach, Cooks Beach and Kimbles Beach. The shoreline in this subsection is relatively straight, oriented along a generally north-northeast by south-southwest axis. The shoreline is fronted by narrow sandy beaches, backed by extensive marshlands. Coastal development is limited to a series of small coastal villages and towns. Erosion is fairly uniform and low, averaging 2 feet per year.

Subsection 1C is similar to subsection 1B but with lower erosion rates. Average erosion rates are about 1 foot per year.

Subsection 1D extends approximately from the town of Sunray Beach to north of Town Bank. This section includes Miami Beach, Villa, Highland Beach and Cape May Beach. Development here is much more extensive than up-bay, with wider beaches backed by progressively less marshlands. Erosion rates are small with some accretional areas. At the southernmost end erosion rates again approach 2 feet per year.

Subsection 1E is a highly developed area which includes Town Bank and North Cape May. It is a highly developed area characterized by a series of groins, the first of which was placed in the 1930's. As a result, the typical erosion/accretion patterns closely reflect the presence of these groins, with updrift accretion and downdrift erosion at each of the groins. The erosion rate for pre-groin period (1883-1936) average nearly 2 feet per year as compared to

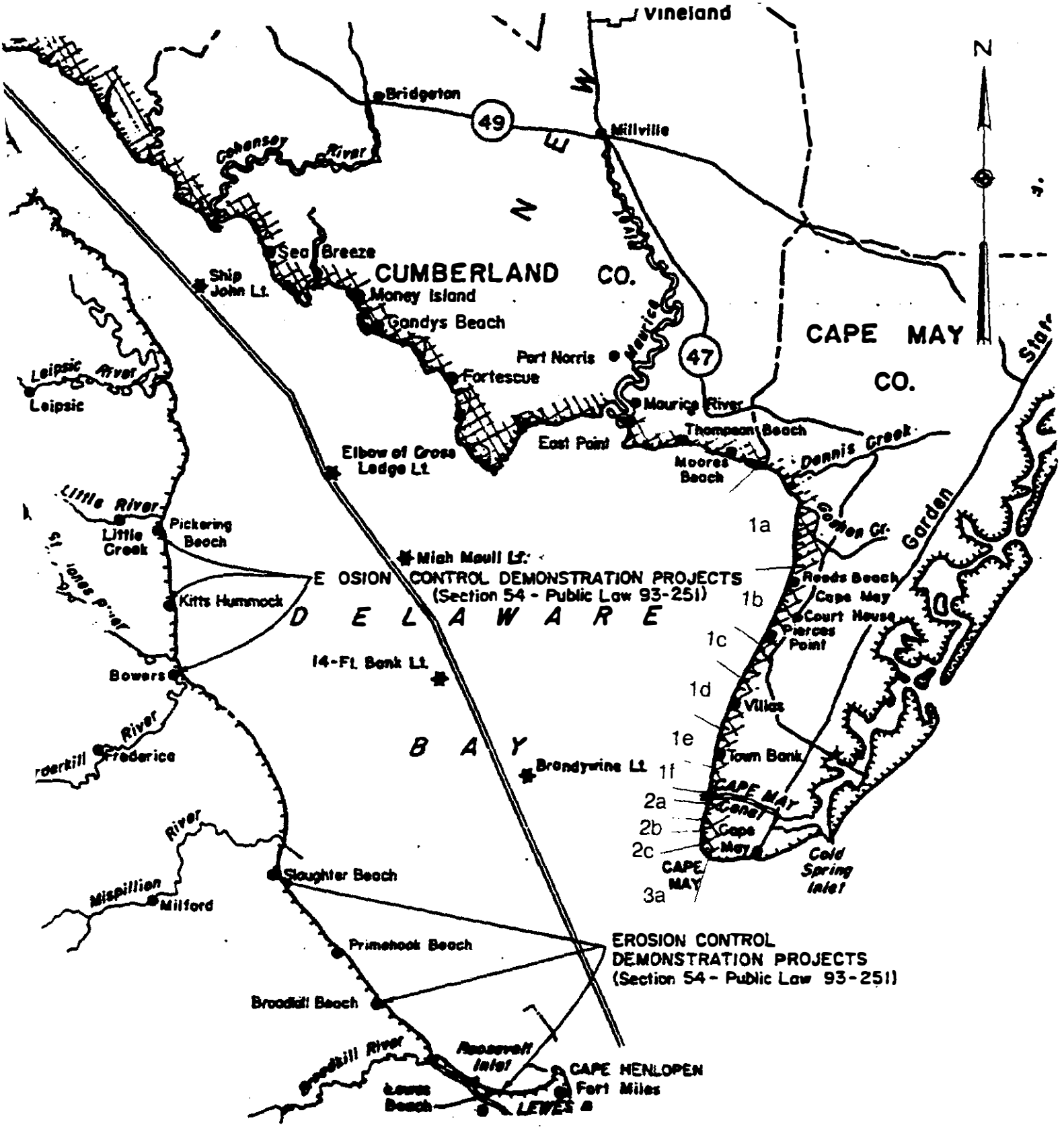

Figure 14 Subsection Locations, Shoreline Erosion Study, Cape May County, New Jersey

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the post-groin period (1936-1986) of alternating erosion/ accretion pattern.

Subsection 1F covers approximately 2,700 feet of shoreline updrift of the Cape May Canal Inlet. The Inlet was opened and stabilized in the 1930's. The period of post-stabilization from 1943 to 1986 is characterized by an accretionary trend. At the north side of the inlet, the annual growth rate is nearly 7 feet since 1943. The pre-groin period (1883-1936) shows an erosion rate of nearly 2 foot per year.

Subsection 2A extends from Cape May Canal's southern jetty to Higbee Beach. As would be expected due to the blocking effects of the jetties on the south-flowing littoral drift, this section is experiencing significantly higher rates of erosion (up to 6 ft/yr).

Subsection 2B is a 3300 foot section of undeveloped shoreline between Higbee Beach and Sunset Beach. The shoreline is composed of narrow sandy beaches backed by extensive marsh with few areas above 10 feet NGVD. Erosion rates here average approximately 1 foot per year and are relatively uniform along the length of the section.

Subsection 2C is characterized by undeveloped sandy beaches largely backed by higher land (over 10 feet NGVD). A littoral drift nodal area exists in the section where the ocean wave influence ends. North of the nodal point, the shoreline is more influenced by bay waves and currents with net littoral drift in a southerly direction. South of the nodal point, the net drift direction is to the north.

Subsection 3A is a one mile section encompassing Cape May point. The shoreline is stabilized by a series of eight large groins. The area is highly developed and occupied by a U.S. Navy reservation situated on ground elevated above the 10 foot contour. Before the placement of the groin field, the rate of erosion was approaching 10 feet per year in the middle of the section. The beach was nourished in 1962, 1967 and 1969 with a total of 400,000 cubic yards of sand. Consequently, the post-stabilization pattern of shoreline change included areas of limited erosion followed by areas of minimal accretion.

In Cumberland county, Phillips (1985) measured shoreline changes (mean high water line) between the 1940, 1963 and 1978 aerial photographs provided by the U.S. Agricultural Stabilization and Conservation Service and computed a mean rate of 10 ft/yr (1940-1978) for the study area. Phillips noted that the shoreline recession rates are highly variable. Phillips also used USGS maps to compute a second set of erosion rates between 1956 and 1972. The average erosion rate for this time period is 6.6 ft/yr. Phillips notes that some of the extreme erosion rates encountered appear to be related to the practice of salt hay farming. Another cause of extreme erosion is the rapid erosion and disappearance of low-lying points of land-peninsulas exposed to waves and currents on two or three sides. Examples include the truncating of a

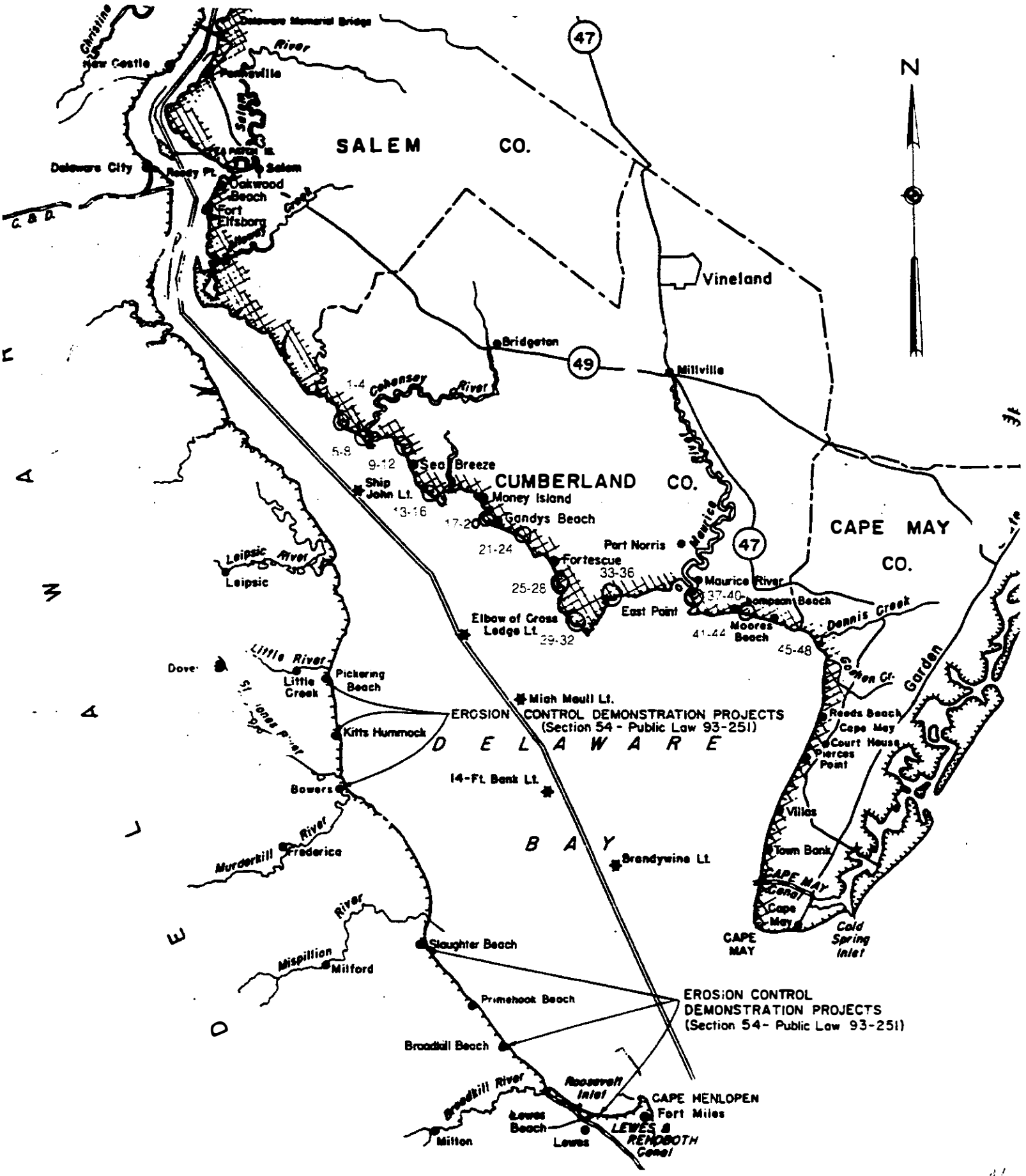
peninsula at the mouth of the Maurice River associated at least in part with a shifting of the river's mouth, and truncating of peninsular headlands such as Egg Island and Ben Davis Points.

The table below presents the point-measured erosion rates as prepared by Phillips (1985). The location of each study site can be found on Figure 15.

Shoreline Erosion Rate, Cumberland County, New Jersey

		Erosion Rate in ft/yr (+ = erosion, - = accretion)			
Location	Site #	1940-78	1940-63	1963-78	1956-72
Cohansey River	9	6.0	8.3	2.6	0.0
	10	4.9	-0.6	13.3	0.0
	11	3.0	4.5	0.6	0.0
	12	3.8	6.9	-1.0	0.0
Ben Davis Point	13	15.9	15.1	17.1	7.1
	14	1.8	12.6	-14.8	11.6
	15	36.9	9.8	78.3	3.5
	16	47.2	17.1	93.2	7.9
Money Point	17	0.3	-0.6	1.8	1.3
	18	4.6	0.1	11.5	0.0
	19	51.1	5.9	120.2	0.0
	20	4.6	5.2	3.2	0.0
Dyer Cove	21	0.2	5.4	-7.8	8.8
	22	-0.4	0.4	-1.6	11.3
	23	45.0	10.5	98.0	9.1
	24	13.6	16.4	9.2	8.5
Fortescue	25	1.0	-1.3	4.4	0.0
	26	1.3	-2.5	7.0	0.0
	27	1.3	0.5	2.5	0.0
	28	6.4	3.9	0.3	6.3
Egg Island Point	29	12.8	14.5	10.3	17.7
	30	9.9	12.8	5.5	0.0
	31	14.0	21.5	2.6	8.4
	32	11.5	18.2	0.7	7.1
Dividing Creek	33	12.9	3.9	26.8	22.5
	34	3.6	0.2	8.9	0.0
	35	9.9	-9.0	39.1	17.5
	36	12.7	5.0	24.7	28.2
Maurice River	37	16.7	5.2	34.8	23.9
	38	2.5	-2.4	0.2	11.8
East Point	39	1.5	1.7	1.3	3.7
	40	4.8	3.9	6.1	0.0
Thompson Beach	41	5.2	14.0	-8.6	8.4
	42	4.1	8.3	-2.5	7.3
	43	5.3	-1.5	15.7	3.7
	44	4.5	-3.7	17.2	3.7
Moores Beach	45	6.2	5.8	6.7	5.6
	46	3.2	-0.1	8.3	3.9
	47	6.0	8.1	3.2	7.6
West Creek	48	5.8	7.0	3.9	8.4
	AVE	10.0	5.8	16.1	6.6
	STD DEV	12.6	6.8	29.5	7.1

Figure 15 Point Erosion Sites Used in Cumberland County, New Jersey



The following two tables, extracted from the Delaware Bay Coastline Reconnaissance Study (USACE, 1991) summaries the estimated shoreline change rates along the major communities. In these tables, if beachfill projects have been implemented for a community, attempts were made to use pre-fill shoreline changes or use estimates after filtering out beach fills to eliminate the beach fill effect. Also included in these tables are the erosion rates determined by others as noted.

Shoreline Change Rates Delaware Bay, Delaware Shore		
Site	Change Rate (ft/yr) (-, erosion)	
	USACE, 1991	French, 1990 (1882-1977)
Port Mahon, Kelly Island	-15	-21
Route 89	-(20 to 26)	-
Pickering Beach	-5	-6
Kitts Hummock	-	-4
Browers Beach	-2	-3
South Browers	-8	-
Big Stone Beach	-(5 to 6)	-4
Big Stone Beach to Mispillion	-(10 to 13)	-7
Mispillion Inlet (Conch Bar)	-(9 to 11)	+3
Cedar Beach	-11	
Slaughter Beach	-2	+1
Slaughter Beach to Fowler Beach	-(1 to 5)	+1
Broadkill Beach	-3	+6
Broadkill Beach to Roosevelt Inlet:		
South of Broadkill	-(4 to 6)	-5
Central Section	-(9 to 12)	
West on inlet (precutoff)	-(0 to 1)	-3
West of inlet (postcutoff)	-(6 to 8)	
Lewes Beach	-3	+1

Shoreline Change Rates Delaware Bay New Jersey Shore	
Site	Change Rate (ft/year) (-, erosion)
Sea Breeze	-(2 to 5)
Fortescue	-1
Maurice River Cove	-(3 to 12)
East Point to Thompsons Beach	Stable
Moores Beach	-(2 to 6)
Reeds Beach	-3
Green Greek to Villas	+1
Villas to Cape May Canal	-1
Inlet to Ferry Terminal	+1

II-10. LONGSHORE SEDIMENT TRANSPORT

French (1990) reports that between Roosevelt Inlet and Broadkill Beach, there is evidence a nodal point exists where the direction of net littoral drift reverses. This reversal in the direction of drift is clearly visible in the accretion patterns observed at the jetties at Roosevelt Inlet (accumulation at north jetty) and the groins at Broadkill Beach (accumulation at south side of groins). The net rate of littoral drift is reduced at the Mispillion River Inlet as evidenced by the limited buildup of material along the southern jetty. The north jetty also has some buildup indicating that there is a convergence point between the Broadkill-Mispillion cells. North of Mispillion Inlet, the wave energies are quite low and the limited littoral drift is highly variable in direction and poorly defined. Between Broadkill and Mispillion Inlet, there are four sub-cells where there is a clear pattern of updrift erosion and downdrift deposition. North of Mispillion Inlet, drift direction is less well defined.

The FEMA study (1993) of Cape May county also reports a drift reversal zone near Sunset Beach. North of the nodal zone, the littoral drift is to the south. South of the nodal zone, the drift direction is to the north.

Potential sediment transport rates are calculated based on the recent wave hindcast study (1987-1993) in Delaware Bay (USACE, in preparation). The following tables summarize the net annualized transport volume and direction derived from the hindcast wave data.

Net Potential Sediment Transport Delaware Bay New Jersey Shore (Nov. 1987 - Oct. 1993)		
Site	Net Annual Transport (thousand cy/yr)	Net Direction
Cohansey River	26	N
Fortescue	48	N
Maurice River Cove	48	E
Reeds Beach	174	S
Villas	84	S
Cape May Ferry	314	N
Cape May Point	110	N

Source: USACE, Delaware Bay Wave Hindcast Study

Net Potential Sediment Transport Delaware Bay, Delaware Shore (Nov. 1987 - Oct. 1993)		
Site	Net Annual Transport (Thousand cy/yr)	Net Direction
Port Mahon	7	N
Kitts Hummock	99	S
Browers Beach	79	S
Big Stone Beach	223	S
Mispillion Inlet	141	S
Broadkill Beach	3	S
Roosevelt Inlet	56	S
Lewes Breakwater	89	S

Source: USACE, Delaware Bay Wave Hindcast Study

Annual net transport potential for Maurice Cove and Broadkill Beach are extracted from the Delaware Bay wave hindcast study. As shown in the following table, the littoral drift can change direction from year to year. The coastal processes at Broadkill are investigated in more detail in Section IV. At Maurice Cove, there is a pronounced pattern that the net littoral drift is to the east with the exception of November and December 1987 when the transport was to the west.

Annual Net Longshore Transport Potential
in cubic yards

Year	Maurice Cove		Broadkill Beach	
	West	East	South	North
1987 (Nov-Dec)	32,400	-	-	15,200
1988	-	6,400	202,800	-
1989	-	121,000	-	287,900
1990	-	104,800	266,900	-
1991	-	64,100	185,500	-
1992	-	22,900	-	177,900
1993 (Jan-Oct)	-	2,100	-	157,000
Net	-	288,800	16,500	-

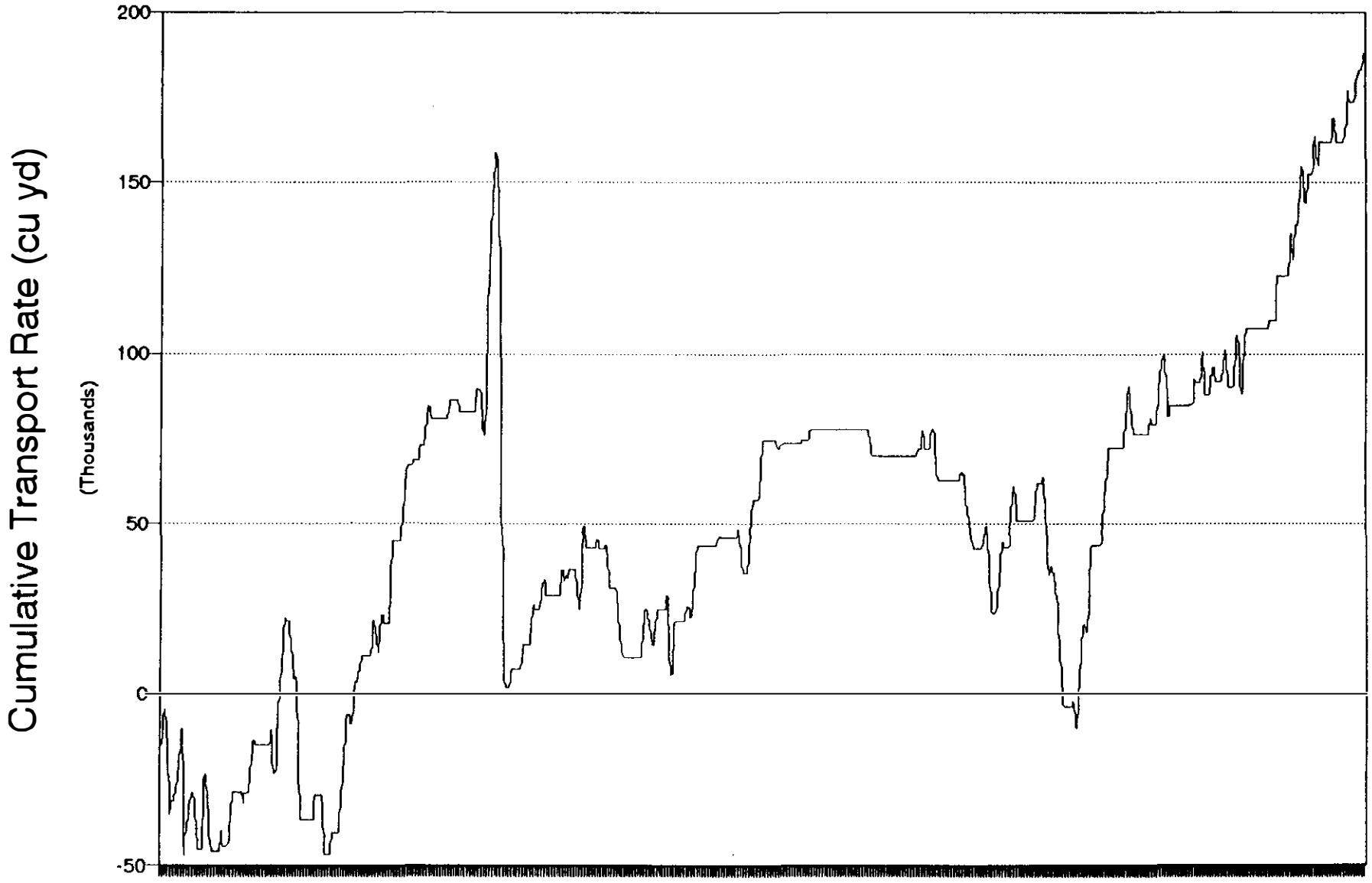
The following table summarizes the potential transport at Broadkill Beach. The hindcast station is located at 38°50'N and 75°10'W, approximately 2 miles northeast of Broadkill.

Potential Longshore Transport Rate
Broadkill Beach

	1988	1989	1990
	Volume in cubic yards per year		
Northward	800,600	1,208,800	660,600
Southward	1,003,400	920,900	927,500
Gross	1,804,000	2,129,700	1,588,100
Net	202,800(S)	287,900(N)	266,900(S)

A mass curve showing the cumulative longshore sand transport at Broadkill Beach for 1988 is shown in Figures 16. Figure 17 shows the incremental transport rate every 3 hours for 1988.

Figure 16, Broadkill Beach, Delaware
Potential Longshore Transport



Source: Delaware Bay Wave Hindcast Study

1988, (3 hour interval)

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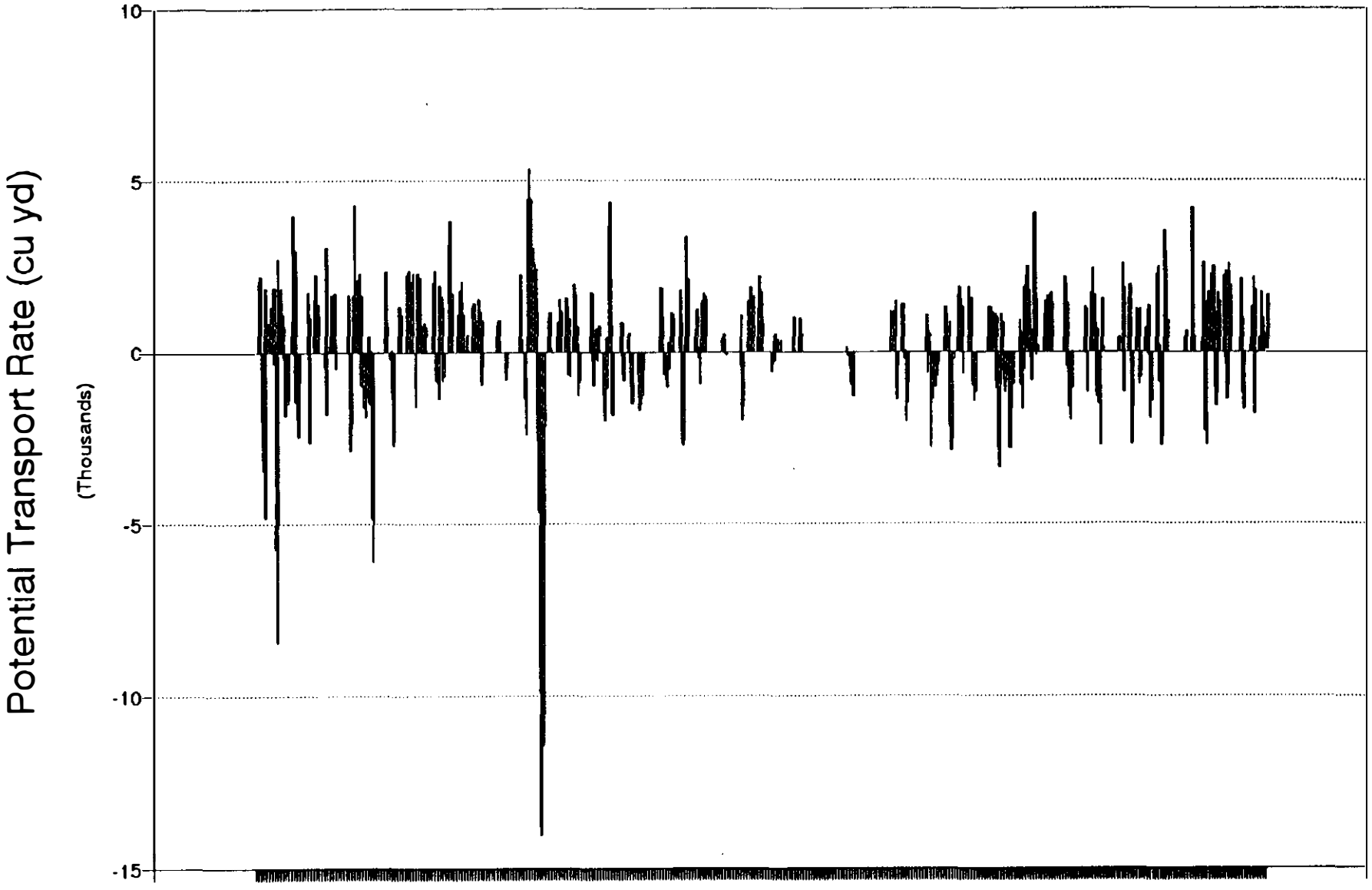
Figure 17, Broadkill Beach, Delaware
Potential Longshore Transport



Source: Delaware Bay Wave Hindcast Study

1988, (3 hour interval)

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II.11 STORM SURGE

The highest elevation of water recorded at the NOS station located in Lewes was 7.9 feet NGVD during the March 1962 northeaster. The greatest flood on record in Cumberland County, NJ occurred in November 1950, when the water level reached 9.2 feet NGVD at the mouth of the Cohansey River. The water stage reached 7.9 feet NGVD during the 1962 northeaster. The following table summarizes the significant storms and recorded high water near Lewes. The computed water levels from the Delaware Bay storm surge model (USACE, in preparation) are also presented in this table.

Northeasters that affect Delaware Bay
between 1950-1992

Date	Type	Still Water Level (feet NGVD)	
		Recorded (Lewes)	Computed (Broadkill)
November, 1950	Northeaster	7.2	7.2
October, 1953	Northeaster	6.0	8.9
November, 1953	Northeaster	5.3	-
March, 1962	Northeaster	7.9	8.9
January, 1964	Northeaster	5.9	7.2
January, 1966	Northeaster	5.9	6.2
December, 1973	Northeaster	6.2	6.6
Nov/December, 1974	Northeaster	5.6	6.9
March, 1977	Northeaster	3.3	3.0
October, 1977	Northeaster	6.6	6.9
February, 1978	Northeaster	4.6	5.6
October, 1980	Northeaster	6.6	6.2
March, 1984	Northeaster	6.6	6.6
Oct/Nov, 1991	Northeaster	6.2	6.9
January, 1992	Northeaster	7.2	8.9
December, 1992	Northeaster	6.2	7.2

Hurricanes that affected Delaware Bay (1899-1992)

Date	Type	Still Water Level (ft NGVD)	
		Recorded (Lewes)	Computed (Broadkill)
October, 1891	Hurricane	-	4.9
August, 1899	Hurricane	-	8.9
September, 1903	Hurricane	-	3.6
August, 1924	Hurricane	-	5.6
September, 1933	Hurricane	6.1	5.6
September, 1936	Hurricane	5.9	3.9
September, 1944	Hurricane	7.6	6.6
August, 1953	Hurricane	4.3	7.6
August, 1954	Hurricane Carol	3.7	6.2
September, 1954	Hurricane Edna	4.6	5.3
October, 1954	Hurricane Hazel	4.6	-
August, 1955	Hurricane Connie	4.5	-
August 1955	Hurricane Diana	4.1	-
October 1955	Hurricane	5.1	-
September, 1956	Hurricane Flossy	5.6	-
August, 1958	Hurricane Daisy	4.0	5.3
September, 1960	Hurricane Donna	5.2	5.3
September, 1967	Hurricane Doria	5.3	6.2
August, 1976	Hurricane Belle	4.9	5.6
September, 1985	Hurricane	6.6	7.2

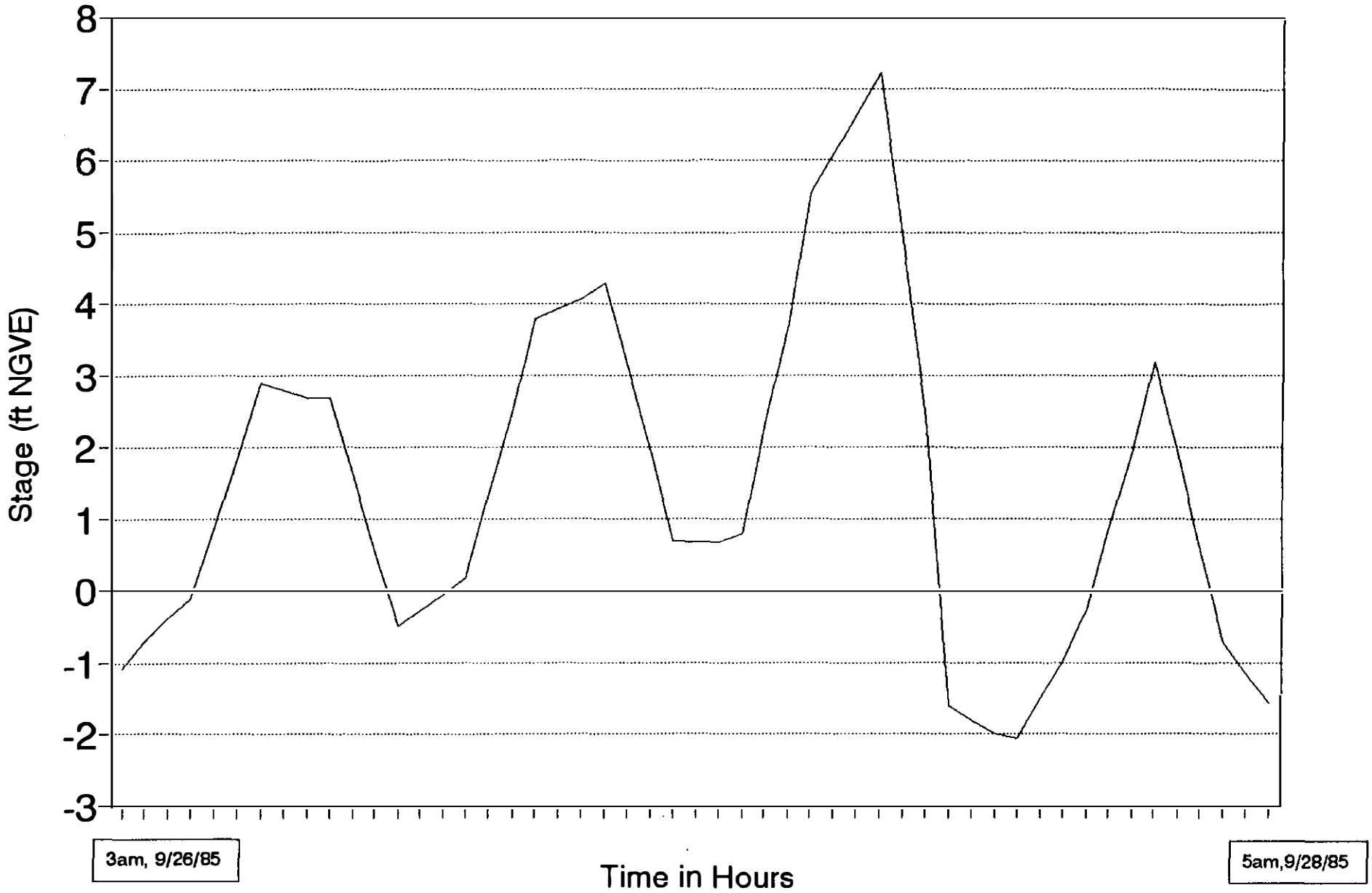
Based on the peak surge levels simulated from the historical hurricanes and northeasters, the following extreme water levels are derived (USACE, in preparation).

Stage Frequencies Delaware Bay, New Jersey Shore							
Site	Elevation in feet (NGVD)						
	5	10	20	50	100	200	500
Cohansey River	6.6	7.6	8.5	9.8	10.8	11.5	12.8
Fortescue	5.9	6.9	7.9	9.2	10.2	11.2	12.5
Maurice River Cove	5.6	6.6	7.3	8.5	9.5	10.2	11.4
Reeds Beach	4.9	5.9	6.7	7.9	8.5	9.2	10.0
Villas	5.3	6.2	7.0	8.2	8.9	9.6	10.6
Cape May Ferry	5.3	6.2	7.0	8.2	8.9	9.6	10.6
Cape May Point	4.9	5.9	6.7	7.9	8.5	9.1	10.0

Stage Frequencies Delaware Bay, Delaware Shore							
Site	Elevation in feet NGVD						
	5	10	20	50	100	200	500
Port Mahon	7.2	8.2	9.4	10.8	11.8	12.8	14.1
Kitts Hummock	7.2	8.2	9.2	10.5	11.5	12.4	13.7
Browers Beach	6.9	8.2	9.1	10.5	11.5	12.5	13.7
Big Stone Beach	6.9	7.9	8.9	10.2	11.2	12.0	13.0
Mispillion Inlet	6.6	7.6	8.6	9.8	10.5	11.5	12.7
Broadkill Beach	6.2	7.5	8.0	9.5	10.5	11.2	12.4
Roosevelt Inlet	6.2	7.2	8.2	9.5	10.2	11.2	12.5
Lewes Breakwater	6.2	7.2	8.2	9.5	10.2	11.2	12.5

Figure 18 shows the predicted stage hydrograph at Broadkill Beach during the 1985 hurricane. Figure 19 is the simulated storm stage hydrograph for the March 1962 northeaster.

Figure 18. Storm Stage Hydrograph
Broadkill Beach, September 1985



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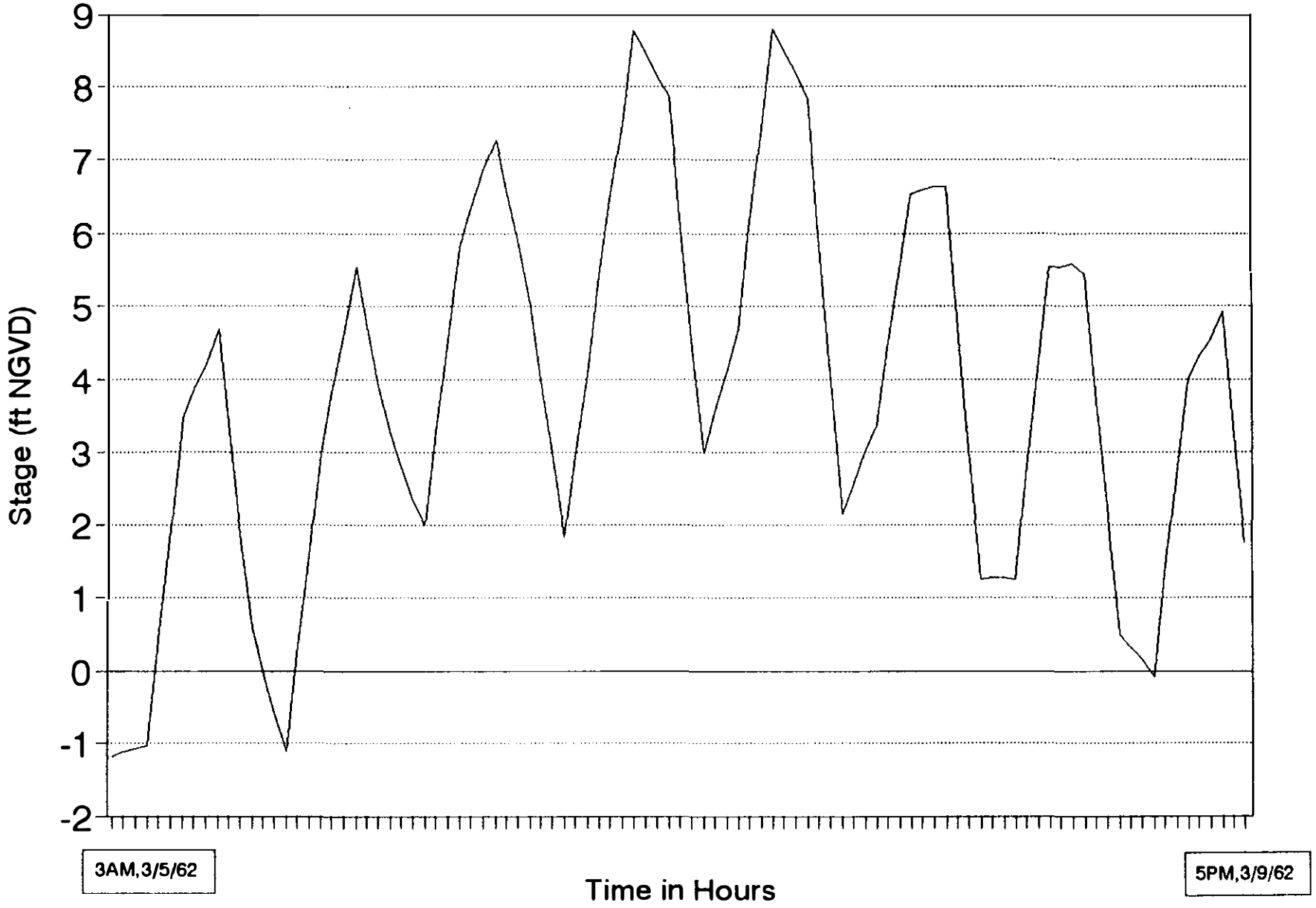
Source: Delaware Bay Storm Surge Study

Figure 19. Storm Stage Hydrograph
Broadkill Beach, March 1992



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Source: Delaware Bay Storm Surge Study

SECTION III
PERFORMANCE OF SECTION 54 PROJECT SITES

III-1. OVERVIEW

In March of 1974, Congress passed the Shoreline Erosion Control Demonstration Act, Section 54, Public Law 93-251, and authorized a five-year program to develop, demonstrate, and disseminate knowledge about low-cost methods of shore protection. This action was intended to help curb the deterioration of U.S. shorelines due to erosion. This action was taken because public and private landowners were finding it difficult to obtain the financial and technical resources necessary to eliminating shoreline erosion.

This section presents the monitoring results of the Section 54 projects at the end of the demonstration project, in 1987 and in 1994. While some of the devices may have reduced erosion, none of the methods were successful in preventing erosion.

III-2. BACKGROUND

The U.S. Army Corp of Engineers was authorized to develop and demonstrate a variety of low-cost methods to prevent or control shoreline erosion. Sites were selected mostly in sheltered, or inland waters on the Pacific Coast, the Gulf Coast, the Great Lakes and the State of Alaska. Six of the sites monitored are located on the western shore of Delaware Bay where serious erosion had occurred. The Delaware Bay sites are Pickering Beach, Kitts Hummock, Slaughter Beach, Bowers, Broadkill Beach, and Lewes. The location of Delaware Bay sites are shown in Figure 1.

Devices being tested were both structural and vegetative. The sites were monitored for about 2 years before the project ended in October of 1980. The effectiveness of these devices were evaluated based on the difference between conditions at the beginning and end of the monitoring period.

A final report on the shoreline erosion control program was published in 1981 (USACE, 1981). The final report documents the monitoring results and evaluates the performance of the devices used at each of the sites. The monitoring period was not adequate to fully appraise the structural stability and functional effectiveness of some of the devices. The Section 54 sites were revisited in 1987 to inspect the low-cost shoreline erosion control devices to learn how they withstood the environmental conditions for five years without maintenance (USACE, 1989).

III.3. DELAWARE BAY SITES

Under the Section 54 program, new protective devices were installed at Pickering Beach, Kitts Hummock and Slaughter Beach. At the other sites, corrective action had been taken before the Section 54 program was initiated. The program included monitoring at all six sites.

Climate at all six of the Delaware Bay sites is similar and wave conditions are generally mild. Waves at the sites are generally of short period, generated by local winds. Wave heights rarely exceed 6 feet due to limited fetch and shallow depth of the bay. Project sites are on public land and are located in an undeveloped and sparsely populated region. The shore areas at the project sites are typical. Their widths vary from 40 to 100 feet and are comprised of sand and gravel. The dunes have a top elevation ranging from 9 to 12 feet above mean low water with base widths from 20 to 30 feet.

A Littoral Environment Observation (LEO) station was established at each site to collect various littoral data during the duration of the monitoring. The following table summarizes the information collected. The potential longshore transports shown in this table were computed for the observation period only.

Summary of LEO data

Sites	Wave Statistics				Potential Longshore Transport Volume (thousand cy)		
	Data Base (mo)	Ave H (ft)	Max H (ft)	Ave T sec	North	South	Net
Pickering Beach	3	0-1	2.5	0-1	3	15	12(S)
Kitts Hummock	3	0-1	2.2	0-1	53	18	35(N)
Bower	3	1-2	4.0	3-4	15	50	35(S)
Slaughter Beach	10	0-1	3.0	3-4	58	57	1(N)
Broadkill Beach	4	1-2	4.6	5-6	68	46	22(N)
Lewes	3	0-1	3.0	4-5	6(W)	11(E)	5(E)

To evaluate the latest project performance at each location, the Delaware Bay sites were revisited by WRCE in 1994. The following text summarizes the Section 54 projects and provides the evaluations done at the end of the monitoring period in 1981, those done after the revisit in 1987 and those resulting from the most recent examination in 1994.

Pickering Beach

Site Description

Pickering Beach, approximately 34 miles from the bay mouth, is a small summer resort extending approximately four-tenths of a mile along the bay shore. The soil at Pickering Beach consists primarily of fine to coarse sands. The shoreline generally is oriented north to south. The beach was approximately 40 feet wide at high water and slopes at about 1 on 10. A sand dune with a base width of 30 feet and a top elevation about 10 feet above mean low water parallels the beach.

Demonstration Project

Two sections of floating scrap tires were assembled, using two different designs. One was developed by Goodyear Tire and Rubber Company and the other type was the Wave-Maze designed by Noble (1969). The structures were completed in 1978 and had two sections each. The sections were 20 feet by 202 feet and 40 feet by 202 feet and were joined together to make one 404 foot structure. They were anchored 700 feet offshore with wooden piles. Both breakwaters suffered an early failure due to dragging of the concrete anchor blocks and they were reinstalled in 1979 using a pile anchoring system. When the monitoring period ended in 1980 both breakwaters had again failed and were reinstalled by the State of Delaware in 1984 using a more robust pile anchoring system.

Vegetation was planted at two sections, the north planting and the south planting. Smooth cordgrass was planted below MLW, saltmeadow cordgrass on the beach above MLW and Cape American beach grass on the dune ridge above the beach.

Performance

End of Original Monitoring Period: The tire breakwater appeared to have little effect on the shoreline. The vegetation planting was unsuccessful. The demonstration showed that the Wave-Maze, as modified, pulled apart and therefore was unsuccessful. The Goodyear breakwater was partially successful and probably could be made fully successful with improved anchorage.

1987 Revisit: The Goodyear breakwater reinstalled by the State of Delaware was in place and appeared to be in good structural condition in 1987. However, it seemed to have little if any effect on the shoreline erosion. The dune and upper beach area have well-established stands of vegetation, but it is doubtful they were from the plantings placed during the Program.

1994 Observation: In 1990, a beachfill project placed 55,400 cubic yard of material along 2,400 feet of beach. A small tombolo was formed leeward of the floating tire breakwater. However, there is very little original structure left intact. In fact, the anchorage trapped sand and caused a small submerged sand island to form. Perhaps, this is the reason that the breakwater became more effective in wave dampening. However, the sand island becomes submerged during the high tide and is a navigational hazard. The state plans to remove the remaining structure for safety reasons.

Kitts Hummock

Site Description

Kitts Hummock, about 3 miles south of Pickering Beach, is a small fishing resort extending approximately 0.5 mile along the bay shore. Beach material at Kitts Hummock consists of granular soils ranging from medium to fine grain sands to fine gravel. The beach is oriented approximately north to south. The beach was 40 feet wide from the dune to the high water line, and slopes at approximately 1 on 10. The offshore zone has a slope of approximately 1 on 150. The dune, which parallels the beach, is about 12 feet above MLW with a base width of about 20 feet.

Demonstration Project

This demonstration project, completed in 1979, consisted of low, fixed, offshore breakwaters utilizing 3 structural devices: rubble-mound stone, nylon sandbags and precast concrete boxes. Each of the three structures were 330 feet long with a 350 foot separation between devices. They were located about 700 feet from shore and were installed at the south end of the community.

The rubble-mound stone structure consisted of two different designs, the first on a one-foot thick matstone foundation and the second on filter cloth.

The southern half of the concrete-box structure was installed on filter cloth with stone toe protection and the northern half received no foundation treatment.

Smooth cordgrass, saltmeadow cordgrass, and American beachgrass were planted at Kitts Hummock. The position of each species was the same as at Pickering Beach. With smooth cordgrass below MLW, saltmeadow cordgrass on the beach between MLW and MHW, and American beachgrass above MHW.

Performance

End of Original Monitoring Period: The rubble-mound breakwater had maintained good alignment but the north end, founded on matstone, settled a half of foot right after construction. The nylon sandbags had begun to deteriorate when inspected on March 1980. The concrete boxes were in good condition following the 1978-1979 winter storms. The boxes in the northern half were misaligned by a few feet and some were slightly skewed. Sand fill in some of the concrete boxes was lost and was replaced. It was concluded

that the monitoring period was too short for a conclusive determination of the effectiveness of the low breakwater structures. Based on comparison of profiles before and after the construction, there was some advance of the 1980 three-foot contour in the lee of the structures which gave some indication that the breakwaters were at least partially effective in preventing loss of the beach material. There was significant advance of the minus contours which shows that underwater accretion probably occurred as a result of wave-energy absorption by the breakwater.

The breakwaters were partially effective in preventing loss of the beach fill at the end of the initial monitoring period. There was a small advance of the 3 foot contour when compared the 1979 and 1980 positions. The beach grass plantings were completely gone at the end of the initial monitoring period.

1987 Revisit: Both the rubble mound and the concrete box structures seemed to have some effectiveness in refracting waves around the breakwater. Sand placed on the beach had been shaped into a salient on the shore by waves refracting around the breakwater. However, the nylon-sandbag breakwater was no longer above water and no longer had any effect on the waves in the lee of the breakwater. This device was considered to be ineffective. The southern half of the concrete-box structure that was installed on filter cloth with toe protection was at a higher elevation than the northern half.

1994 Observation: The sandbag breakwater is no longer visible. The low concrete and rubble mound breakwaters are submerged during highwater and are a navigational hazard. The state plans to place more buoy markers around the breakwater in the summer of 1994. The State of Delaware placed 15,780 cubic yards of material along 1,000 feet of beach in 1988.

Bowers

Site Description

The incorporated community of Bowers is located at the mouth of the Murderkill River, about 2 miles south of Kitts Hummock, and extends about 0.5 mile along the bay. Many cottages in the area are occupied by permanent residents. No new device was installed at Bowers as part of the demonstration project. It is protected by a groin-retained beach fill placed in 1973 and periodically renourished by the State of Delaware. The device at Bowers was observed during the monitoring period to judge how effective it was in comparison to the other devices installed at other sites as part of the project.

The beach sediments at Bowers generally consist of fine to medium sand with some fine gravel, and the dune consists primarily of fine to medium sands. The shoreline is oriented in a general northwest-southeast direction. The beach berm at Bowers had an elevation of 10 feet above MLW and was approximately 70 feet wide. The foreshore is about 40 feet wide and slopes about 1 on 8. The

fill is retained by two nylon Dura-Bag groins, on at the south end of the town, 750 feet long, and on the north end, 400 feet long. The State of Delaware restored the fill with 15,825 cubic yards in the fall of 1973 and with 28,800 cubic yards in the summer of 1974.

Performance

End of Original Monitoring Period: The groins had been somewhat effective in retaining the beach fill, although considerable erosion had occurred. Material had accreted on the north side of the south groin, and a smaller amount of accretion has occurred just south of the north groin. Comparison of profiles taken in April 1979 and March 1980 show that about 20 feet of beach fill had eroded.

1987 Revisit: The revisit team did not return to Bowers.

1994 Observation: The State of Delaware has placed sands on Bowers three times since the end of the demonstration project: 35,700 cubic yards in 1985, 13,700 and 51,700 cubic yards in 1986 and 1988 respectively.

Slaughter Beach

Site Description

Slaughter Beach is approximately 13 miles from the bay mouth. It is a small incorporated community, extending for approximately 1.5 miles along the bay shore. The beach and berm at Slaughter Beach consists primarily of fine to medium sands. The shoreline at Slaughter Beach is oriented in a general northwest-southeast direction. The foreshore has a slope of approximately 1 on 200. A 70-foot-wide beach, with a slope about 1 on 10 extends from the high water line to a 30-foot-wide berm. Bulkhead and buildings are situated on the berm. The area behind the berm is mainly marshlands.

Demonstration Project

There is a "perched beach" structure at this site which was completed in 1979. This type of beach is higher than it would be naturally because a dike or "sill" is submerged below water level to hold the sand up and thereby prevent erosion. The original concept of this type of beach is to allow the structure to fill and build a beach. Since littoral material in the area is minimal, the design was modified and sand was pumped in to create a beach and provide an area for wave energy to be dissipated.

Three types of materials were used to form the retaining sill and shore return: concrete boxes, wood sheetpiling and large nylon sandbags. The nylon sandbag section of the retaining sill and shore return was comprised of 4x12x2 feet sandbags placed 2 abreast. The bags were then placed on filter cloth to prevent uneven settlement. Each section of the sill was 330 feet long. The shore returns were continuous sections of sandbags and concrete boxes both about 300 feet in length.

Performance

End of Original Monitoring Period: The perched beach was filled with 34,000 cubic yards material in 1979. In January 1980, the dune behind and to the south of the perched beach began to erode. The final report (USACE, 1981) noted that the monitoring period was too short to determine the effects of the perched beach. Structurally, only a minor amount of settlement and damage occurred on the precast concrete-box sill section. The wood sheet-pile sill had no settlement and remained in good condition. The nylon sandbags had shifted position and had experienced some minor damage.

A case study was made of the performance of the perched beach by Douglass and Weggel (1987). The study reported a loss of approximately 20,000 cubic yards from behind the sill since it was completed. The effectiveness of the device was considered to be negligible. They also concluded that the offshore sill and return altered the beach platform and bathymetry as much as would be expected behind a submerged breakwater. The shoreline accreted at the downdrift end and receded at the updrift end. The volume of beachfill placed within the sill eroded but the presence of the structure may have slowed the rate of loss. They also observed the sand level immediately adjacent to the sill was often higher on the offshore side of sill. The increased elevation probably occurred when onshore sand transport accumulated against the sill.

1987 Revisit: The site was examined from the land and the sill was not visible. The revisit team concluded that the effectiveness of the device was negligible.

1994 Condition: Approximately 2500 feet of the beach was renourished in 1985. A total of 365,000 cubic yard of material was placed on the beach. The town of Slaughter Beach has requested that the state remove the sill. They believe that the low sill was not having a positive effect and could be a navigational hazard.

Broadkill Beach

Site Description

Broadkill Beach is about 7 miles from the mouth of the bay. Over 150 summer cottages extend along approximately 6,000 feet of bay frontage. No new devices were installed at Broadkill as part of the study. Broadkill was already protected by a beach fill placed by the Corps of Engineers as part of a Federal Small Beach Erosion Control Project.

The sediment at Broadkill Beach consists primarily of fine to medium sands. The shoreline is oriented along the bay in a general northwest-southeast direction. The onshore and offshore zones have slopes of approximately 1 on 20 and 1 on 300, respectively.

Seven groins have been placed in the study area (3 in 1950, 2 in 1954 and 2 in 1964). Before 1979, over 900,000 cubic yards of material was placed on Broadkill Beach.

Performance

End of Original Monitoring Period: The final report stated that the groin field had retained the beach fill placed since 1976 since only a minor amount of beach fill had eroded. It is believed that there is a strong littoral transport along this reach, and it is doubtful that a beach fill alone could control shoreline erosion. Therefore, it concluded that the groin field must have contributed significantly in stabilizing the beach.

1987 Revisit: The 1987 study team did not visit this site.

1994 Condition: There have been four beach nourishment projects since the end of the monitoring project. A total of 245,800 cubic yards of material was placed along the beach. In 1994, the beach width during high tide varied from 50 feet to several hundred feet. However, damage from the winter storms of 1993 was apparent. The dunes had retreated up to 40 feet. Photos taken on April 1994 reveal the dune scarp and washover (photos #2/6, #2/7, #2/25, Appendix C). Photo #1/12 shows that near the state survey baseline stake N 4+00, the dune face has retreated to less than 10 feet seaward of the baseline while the September 1993 survey profile shows the dune face to be over 30 feet from the baseline. Storm damage to residential structures can be seen on photo #1/22 and #2/6. Emergency measures taken by the local residents include placement of small piles, tree stumps, straw barriers and broken concrete. All seven groins were submerged during high tide. Above high tide, they are buried by sand.

Lewes

Site Description

The incorporated municipality of Lewes has more than 3 miles of beach front along Delaware Bay, extending from Roosevelt Inlet to Cape Henlopen State Park. This part of the city adjoining the bay is commonly known as Lewes Beach. Both the beach and dune consist of fine to medium poorly graded sand. The beach has a east-west orientation.

Lewes Beach was a monitoring site for a previous project which provided for widening 8,000 feet of beach with a berm 100 feet wide at an elevation of 8 feet NGVD. The project also provided for periodic nourishment for 10 years. The initial fill, placed in the winter and spring of 1975 comprised 101,700 cubic yards of material. In 1976 and 1978, the beach was renourished with 11,400 cubic yards and 31,000 cubic yards of sand. Between 1953 and 1975, over 900,000 cubic yards of material has been placed in Lewes Beach. Nine timber groins were constructed between 1948 to 1956.

Performance

End of Original Monitoring Period: Shoreline erosion along the beach at Lewes has been controlled only by the placement of beach fill. Aerial photos reveal no dramatic changes during the relatively short time span from March to October 1979. Beach survey profiles taken between April 1979 and April 1980 show a

retreat of the beach berm of about 50 feet near the west end and a tapering of the retreat to only a few feet near the east end of Lewes Beach. The structure of the beach was kept somewhat stable due to continuous maintenance.

1987 Revisit: The study team did not revisit Lewes.

1994 Condition: Five beach fill projects were completed since 1978 totaling over 220,000 cubic yards. In addition, a 350 foot grout-filled nylon bag groin was constructed in 1990.

III-4. SUMMARY

Throughout the county, the devices originally constructed at the Section 54 sites fall into five major shore protection systems: bulkheads and seawall, revetments, breakwaters and sills, groins, and non-structural. In general, no one type of system had a failure rate that was significantly different from the others. The structures still existing today all had good foundation preparation, adequate filtering between layers, substantial toe protection or treatment, sufficient mass to withstand wave forces and adequate strength of individual members and fasteners to maintain structural integrity. Periodic maintenance prolongs the useful life, even for those structural devices which are susceptible to differing wave conditions. Devices installed on the Delaware Bay sites include two designs of floating tire breakwaters, three types of low breakwaters, and various groins. From a standpoint of results, the devices at various Delaware Bay sites reduced erosion to varying degrees but did not prevent it.

Non-structural devices include perched beaches, beach fills and vegetation. Beach grass planting was unsuccessful in the Delaware Bay sites. The perched beach at Slaughter Beach is no longer visible. The sill is seen as ineffective.

The following conclusions were made by the 1989 revisit report:

- Low-cost shore protection is a design philosophy rather than a physical entity.
- Devices that remained in place for the first few years tend to remain in place longer than ten years.
- Low cost is very site specific.
- No panacea was found and no device solved all the problems all the time.
- Maintenance is necessary.
- Foundation design, filtration, material, and connector are all critical; structures designed by well established procedures survived best

- Vegetation normally does not work along.
- Life-cycle long-term cost is more important than low initial cost.

The 1994 condition at the Delaware Bay sites does not differ significantly from the 1984 condition. Additional beach fills have been placed at all six sites to hold the line since the end of the demonstration project. It is concluded that the devices utilized at these sites are inadequate in preventing beach erosion.

SECTION IV
BROADKILL BEACH WITHOUT-PROJECT CONDITIONS

IV-1. PREVIOUS REPORTS AND STUDIES

All available relevant reports and studies concerning Delaware Bay in general and Broadkill Beach in particular were reviewed. Table 1 presents a listing of the publications relevant to Broadkill Beach and a summary of the key findings and/or recommendations they contain.

Table 1. Summary of Publications Reviewed and Key Findings

Reconnaissance Report, Small Beach Erosion Control Project, Broadkill Beach, Delaware. (USACE 1966)

- identified central 2600 foot beach area as having an erosion problem with adjacent areas stable.
- estimated unit erosion rate at 4 cy/ft/yr based on comparison of beach profile data from 1954 and 1964.
- recommended construction of 2 groins at Georgia and Alabama Avenues, 51,000 cy of beach fill along 2600 foot problem area, and renourishment of 42,000 cy every 4 years.

Detailed Project Report, Small Beach Erosion Control Project, Broadkill Beach, Delaware. (USACE, 1972)

- found problem at Broadkill Beach to consist of erosion due to wave action during storms and determined that erosion was episodic rather than continuous.
- recommended additional 110,000 cy beach fill along central 4500 ft of beach followed by renourishment every 4 years of approximately 40,000 cy.

Broadkill Beach, An Assessment of an Erosion Problem. (Dalrymple, 1982)

- explained that localized high erosion rates in vicinity of station S23+00 as due to temporary effects of offshore borrow pits and response of beach fill materials to nearshore wave climate.

- recommended continued use of beach fill but with wider alongshore placements to reduce apparent angle of wave attack at shoreline and therefor reducing longshore transport potential.

Historical Shoreline Changes in Response to Environmental Conditions in West Delaware Bay. (French, 1990)

- compared historic shoreline positions for West Delaware Bay shorelines. Found Broadkill Beach area to be accretional but did not take beach fill quantities placed there into account.

IV-2. HISTORIC SHORELINE AND BEACH CHANGES

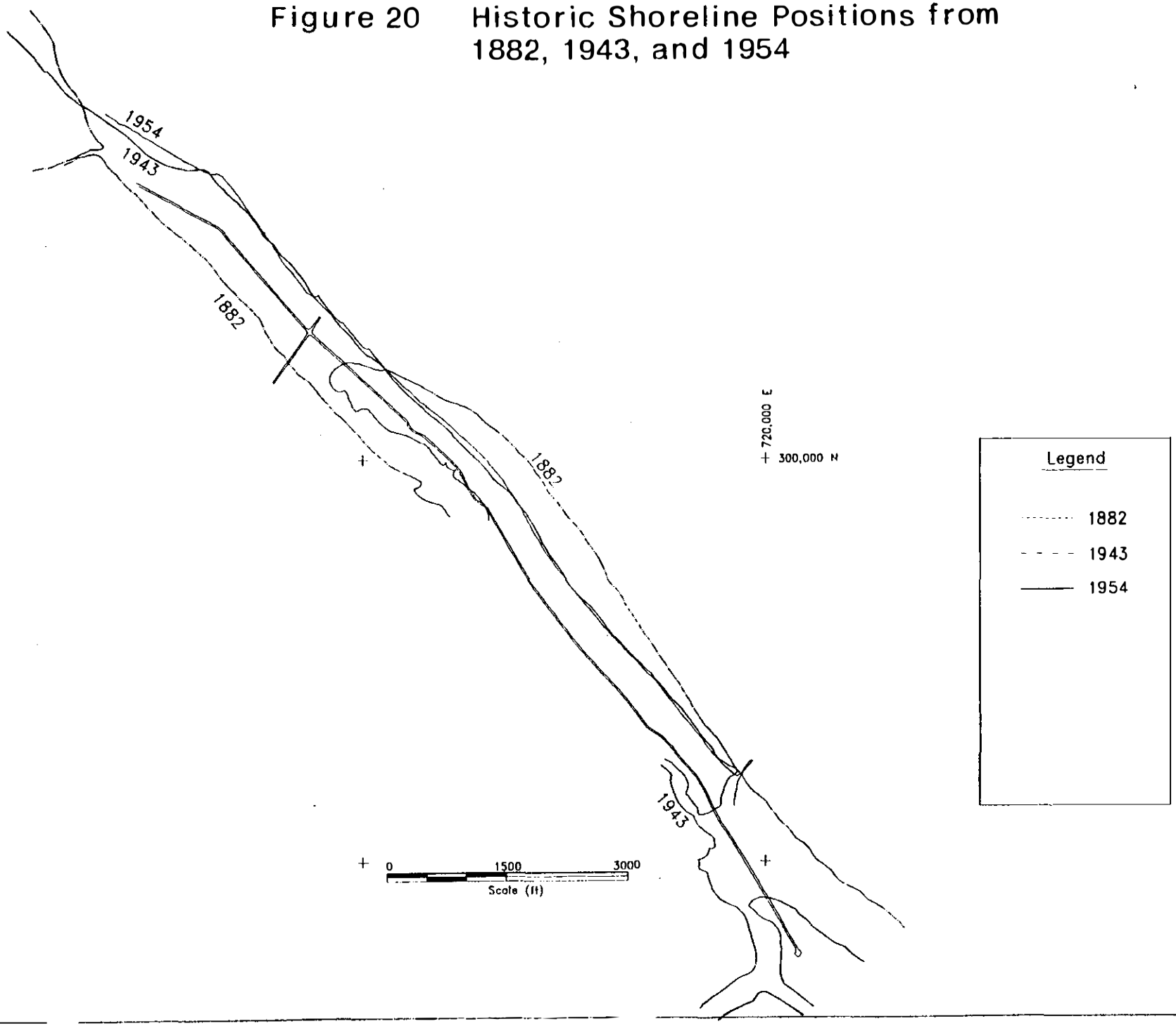
In this section, the change of historic shoreline positions and beach profiles at Broadkill Beach are first presented. Following the presentation of the beach fill and groin construction history and an analysis of the coastal processes, the change data will be re-examined so that the "background" profile change rates may be estimated. This approach takes into account both the modifying effects of the beach fills and groins which have been constructed at Broadkill Beach. Previous works have also provided estimates for shoreline and beach profile volume change rates but have either been of limited temporal and/or spatial scope, or have ignored or poorly represented the potential effects of groin and beach fill. The present study represents the most comprehensive and up to date compilation of historic shoreline and beach profile data for Broadkill Beach.

Historic shoreline position data for Broadkill dates back to the late 1800's. The chronology of events affecting the Broadkill shoreline from that period to the late 1900's has been documented in the reference reports (USACE, 1972; Dalrymple, 1982; French, 1990).

Briefly, the history of the Broadkill Beach shoreline is closely tied to the northerly advance of the Cape Lewes spit from the late 1800's up to about 1900 (Dalrymple, 1982). At this time the bayward migration of the Cape Henlopen spit, the completion of the inner breakwater at Breakwater Harbor, and the cutting of a new inlet to the Broadkill River had a significant impact to the shoreline changes in the lower western shores. The new inlet shoaled and in 1908 the Army Corps of Engineers cut another inlet about 1500 feet to the northwest and stabilized it with a timber and stone jetty. Over time the spit remnant welded onto the beach to form what is presently Broadkill Beach. The jettied inlet again experienced high shoaling and in 1953 the Congress authorized to abandon this project.

Gross shoreline change rates were markedly reduced approximately after 1954, compared to the period from the late 1880's to the early 1950's. Figure 20 shows the remarkable changes in shoreline positions for the years 1882, 1943, and 1954. Assuming that major events such as the opening or closing of an inlet or any significant changes to the breakwaters at Breakwater Harbor will not occur over the lifetime of any proposed alternatives, the more recent (1950's) temporal baseline should be adopted for the present study. Considering the availability of historic shoreline position data and the approximate onset of existing littoral conditions, 1950 was chosen as the base year for shoreline and beach profile change analysis. This is the year groins were first constructed at Broadkill. The first beach profiles were surveyed in 1954. The first beach fill was placed in 1957.

Figure 20 Historic Shoreline Positions from 1882, 1943, and 1954



IV-2.A. Historic Shoreline Changes

Much of the available shoreline position data for Broadkill Beach was assembled by French (1990). That study utilized two data sources: National Ocean Survey (NOS) "T" sheets and aerial photographs. The T-sheets are coastal maps which were produced by the U.S. Coast and Geodetic Survey (presently NOS) and date back to the 1840's. The shoreline delineated on the T-sheets is a debris line. It is therefore dependent on the wave conditions and water elevation during the high tide that preceded the survey used to develop the map. The aerial photographs used were obtained from the U.S. Department of Agriculture (USDA) and the Delaware Department of Natural Resources and Environmental Control (DNREC). The shoreline position for these dates was determined by digitizing the wet/dry sand interface which is usually discernable on the photographs. The shoreline determined in this way can be taken as an approximation of the MHW position. The details of the procedures used on these two sources are described in French (1990).

In addition to these data, additional shoreline positions were obtained by digitizing other aerial photographs. The dates of these additional photographs are April 1964, March 1979, April 1990, and April 1993. The April 1993 shoreline position was taken as the MHW line (+2.6 foot NGVD) from the Broadkill Beach topographic map (Greenhorn and O'Mara, 1994). The topographic map was prepared using an aerial photograph taken in April, 1993. This topographic map was available in a digital file which allowed additional aerial photos to be registered to it using common physical landmarks such as road intersections.

Table 2 provides a listing of the dates for which shoreline positions were compared. The University of Maryland data were prepared based on the NAD27 coordinate system. The COE's coordinate conversion program CORPSCON (USACE, 1990) was used to convert the NAD27 to NAD83 coordinates.

Table 2. Historic Shoreline Position Summary.

Date	Source	Prepared by
Jul 1954	USDA aerial	Univ. of Md.
Apr 1964	USDA aerial	WRCE
Aug 1971	NOS T-sheet	Univ. of Md.
May 1977	DNREC aerial	Univ. of Md.
Mar 1979	USACE aerial	WRCE
Nov 1980	USACE aerial	WRCE
Jun 1989	Unknown	Univ. of Md.
Apr 1990	USDA aerial	WRCE
Apr 1993	USACE survey	Greenhorn & O'Mara

The transect locations at which shoreline positions were measured were chosen to coincide with the locations of beach surveys. Both DNREC and the U.S. Army Corps of Engineers Philadelphia District (CENAP) have conducted profile surveys over the years. These beach profiles will be analyzed in the following sections. The baseline used to reference MHW positions was the survey baseline established by DNREC. Nine survey profiles were selected in the Broadkill Beach shoreline) for analysis. The baseline, transect locations, and study limits are shown in Figure 21.

Figures 22 to 29 show the shoreline positions, baseline, and transect locations for the study area, from north to south. The most southern study area is outside the limit of the DNREC surveys (Station S76+00). Five transects and an approximately shore-parallel baseline were established outside the DNREC survey area for shoreline change analysis.

Available beach profile data at the transects shown on Figures 22 to 29 can also be used to compare shoreline positions by determining the distance to the MHW elevation (+2.6 ft NGVD) from the survey baseline. Comparison of shoreline changes determined by these two methods should yield the same trends for the same time periods. Figures 30 to 37 plot the shoreline positions vs time for selected beach profiles from north to south. The top plot in each of the figures contains data from the shoreline position analysis. The bottom plots contain data obtained from the available beach profile surveys.

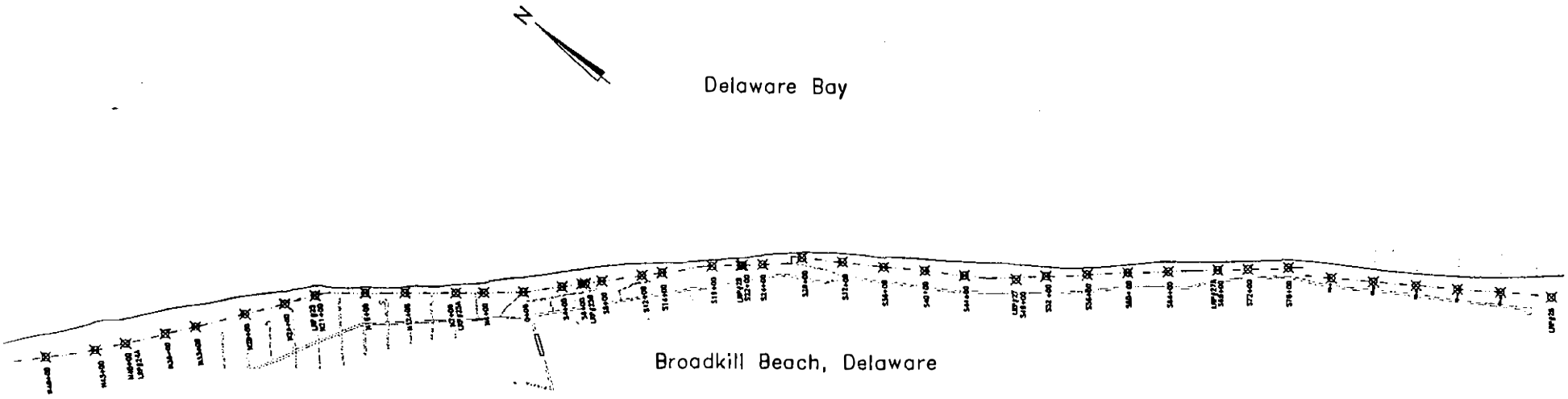
The data plotted in Figures 30 to 37 are unadjusted for the effects of beach fills which have been placed at Broadkill since 1957. As a result the trends they depict are apparent trends and must be adjusted for the effects of beach fills for shoreline planning purposes. The data in Figures 30 to 37 are presented in tabular form in the Appendix D.

IV-2.B. BEACH PROFILE VS SHORELINE CHANGE DATA

For coastal planning and design purposes, volumetric profile change rates are more meaningful than linear shoreline change rates. Linear change rates are sensitive to seasonal profile variations due to changing wave regimes. Also, for an eroding beach apparent shoreline changes are sensitive to changes in profile shape which can be significant for a sand supply deficit condition. In both cases, the MHW position change analysis may exaggerate the actual erosion considering the entire active profile from dune crest to depth of closure.

For a storm erosion analysis, the unit volume in the beach profile and the volumetric change rate are important quantities for planning purposes. During a severe storm it is the total quantity of sand eroded which results in contour movements.

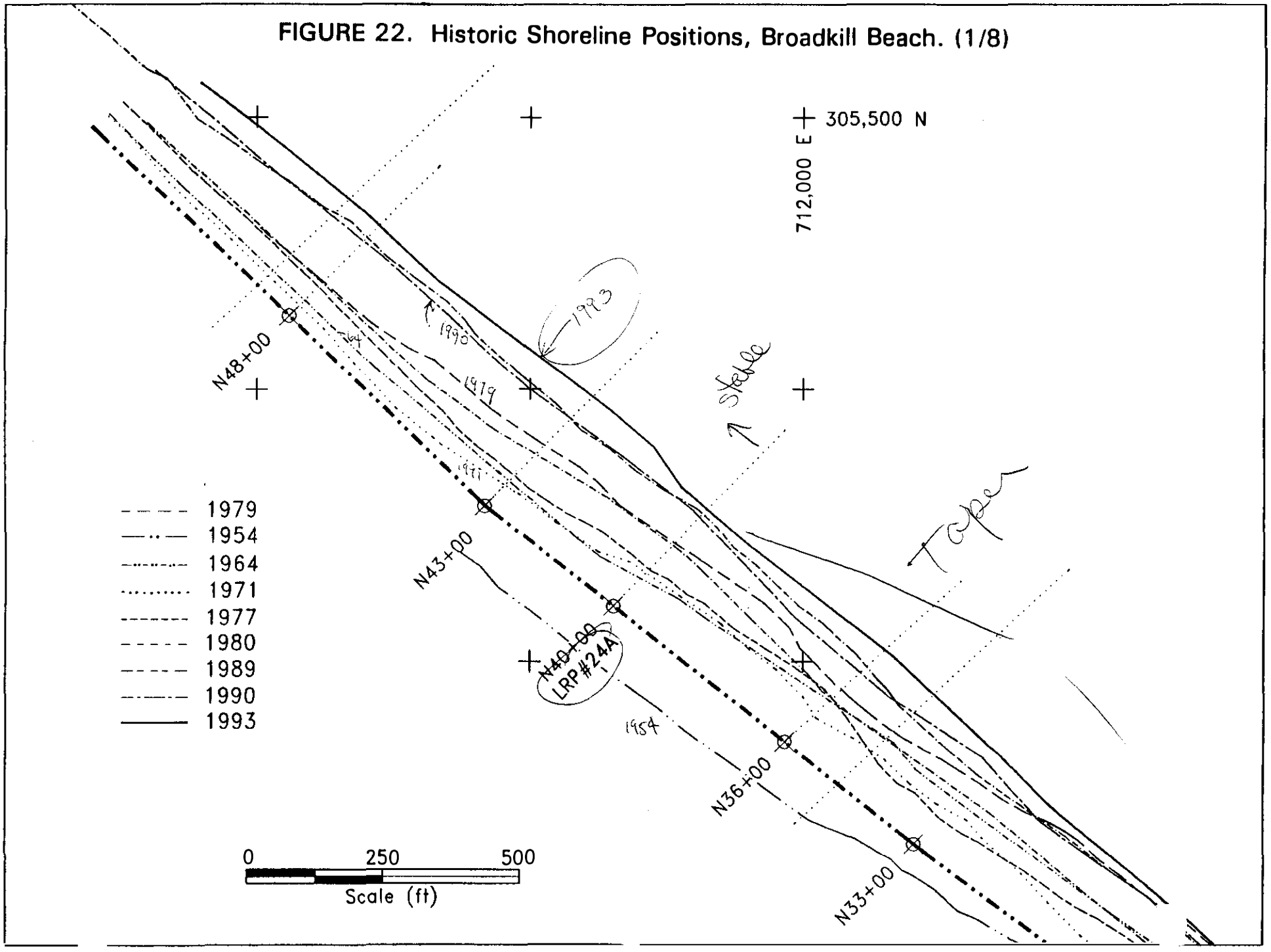
Figure 21 Project Map



Scale 1" = 1000'

D:\ACADWIN\OOB\FIG21.DWG

FIGURE 22. Historic Shoreline Positions, Broadkill Beach. (1/8)



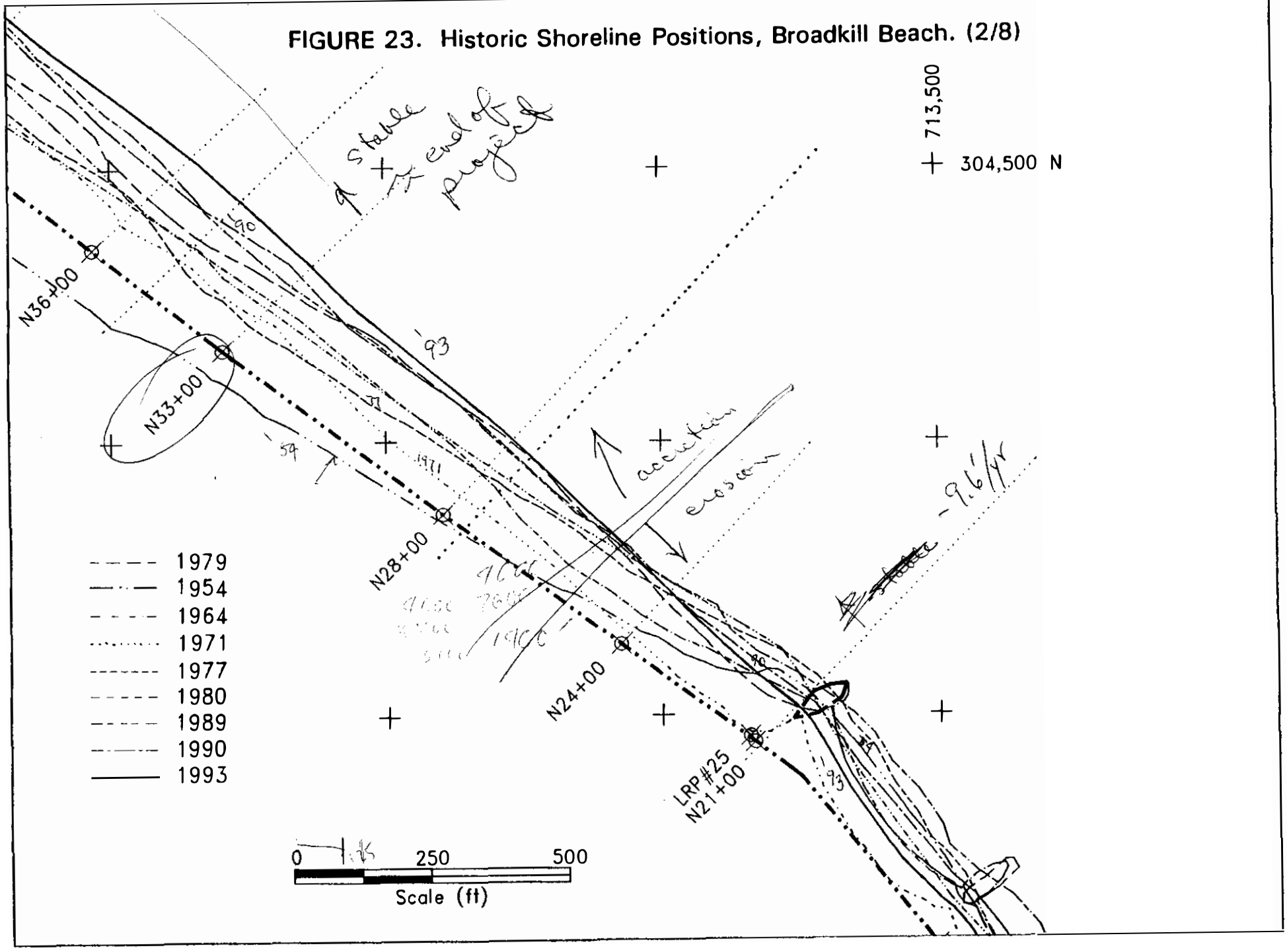


FIGURE 23. Historic Shoreline Positions, Broadkill Beach. (2/8)

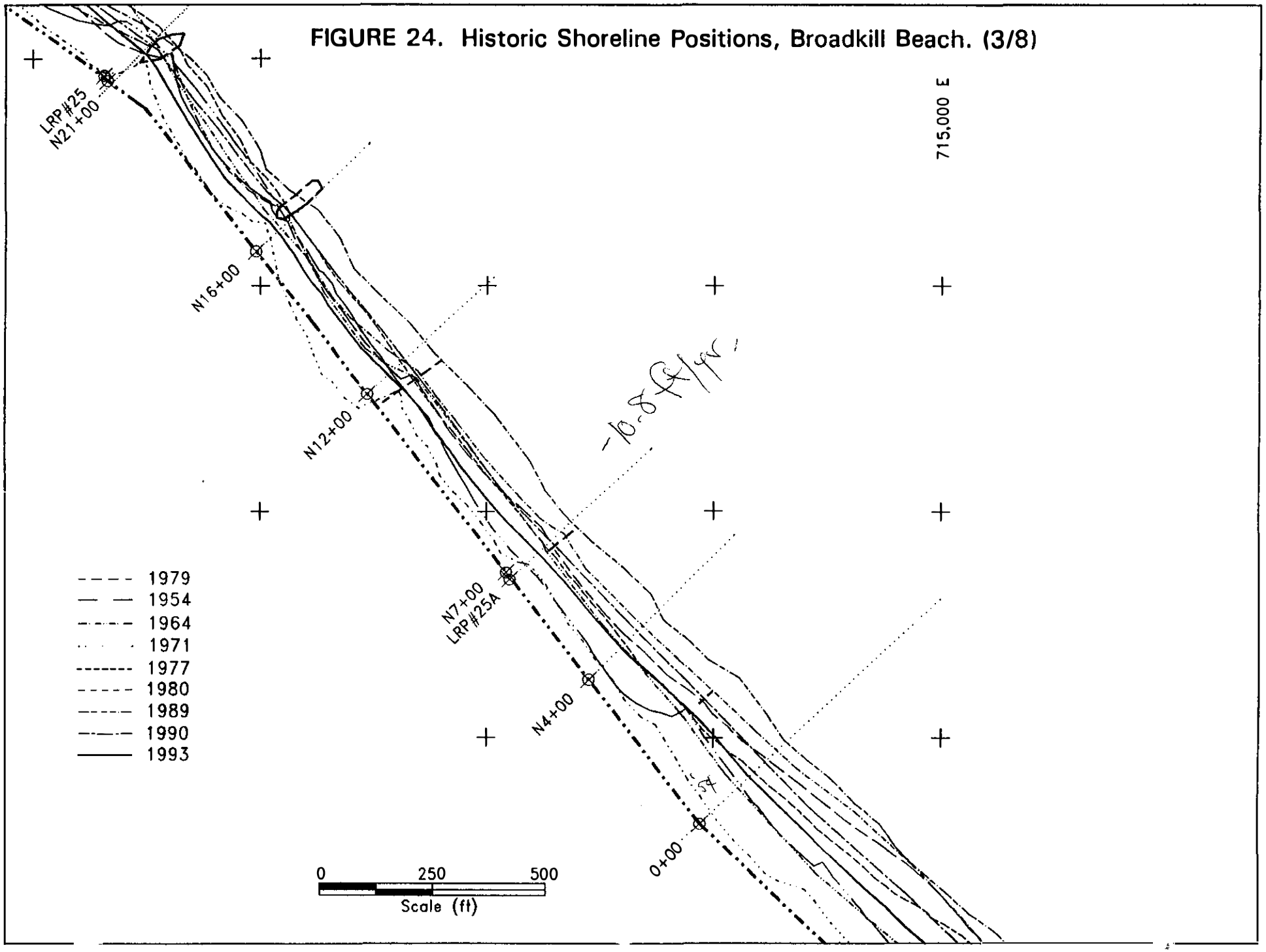


FIGURE 24. Historic Shoreline Positions, Broadkill Beach. (3/8)

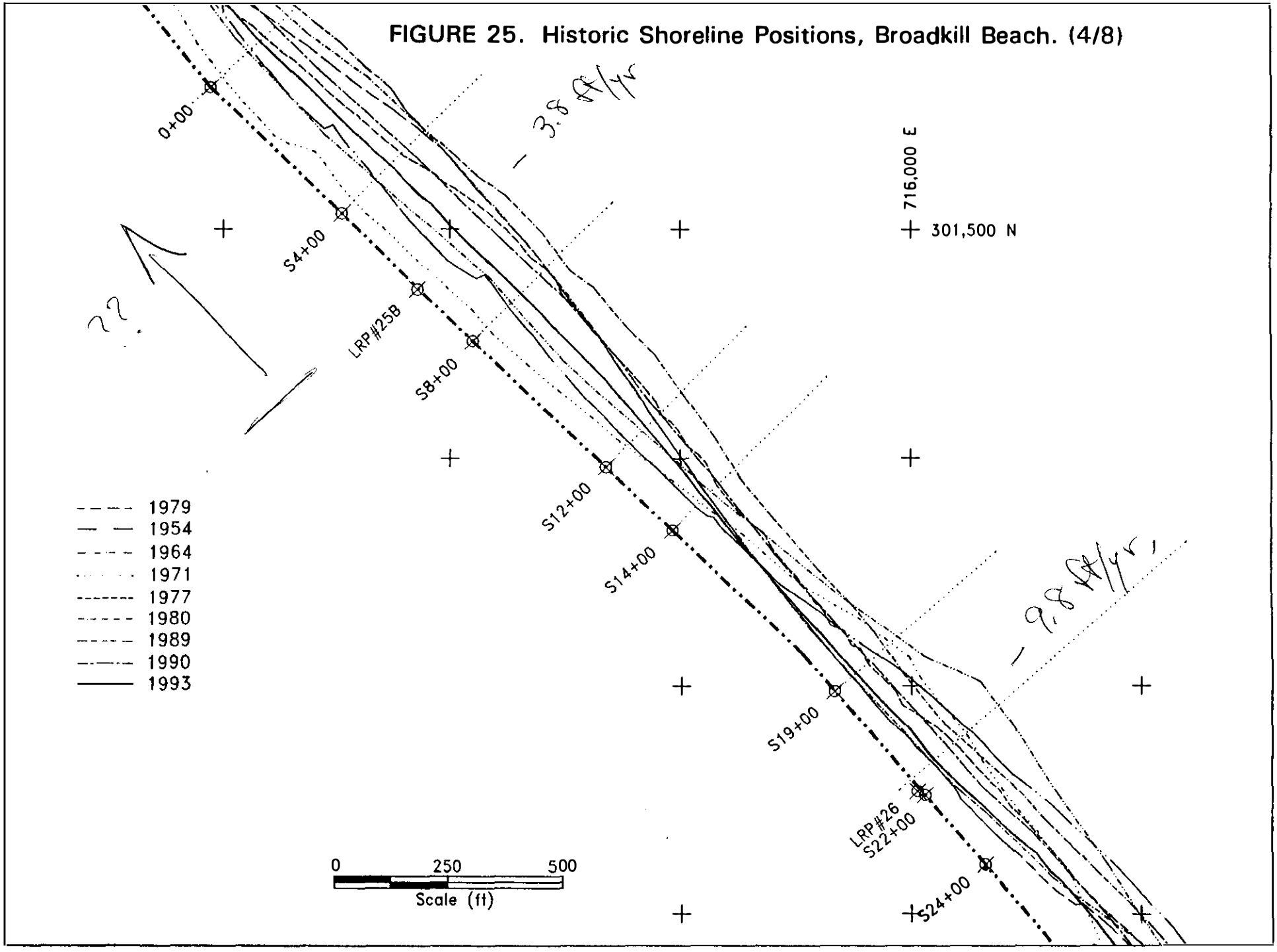


FIGURE 25. Historic Shoreline Positions, Broadkill Beach. (4/8)

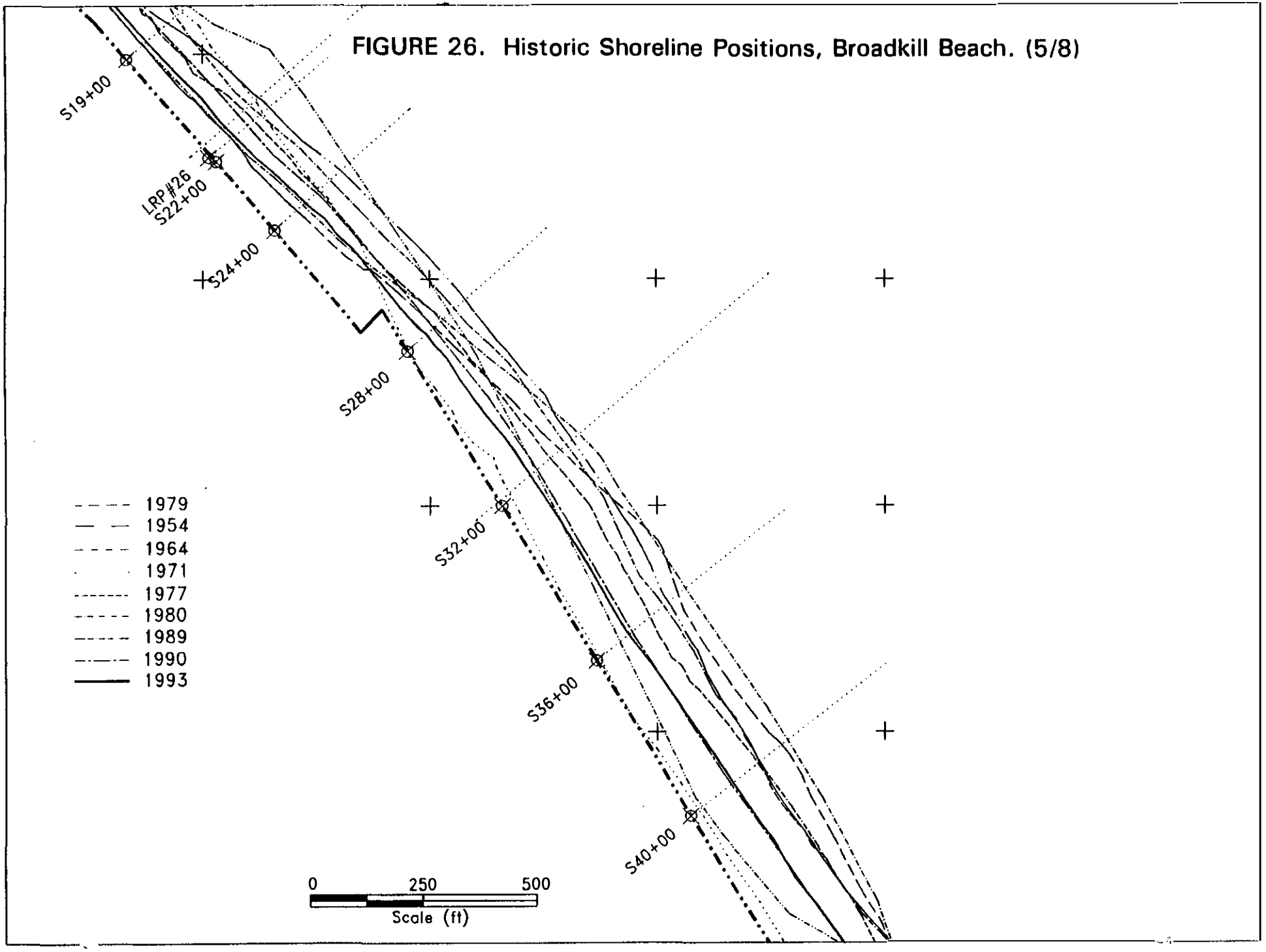


FIGURE 26. Historic Shoreline Positions, Broadkill Beach. (5/8)

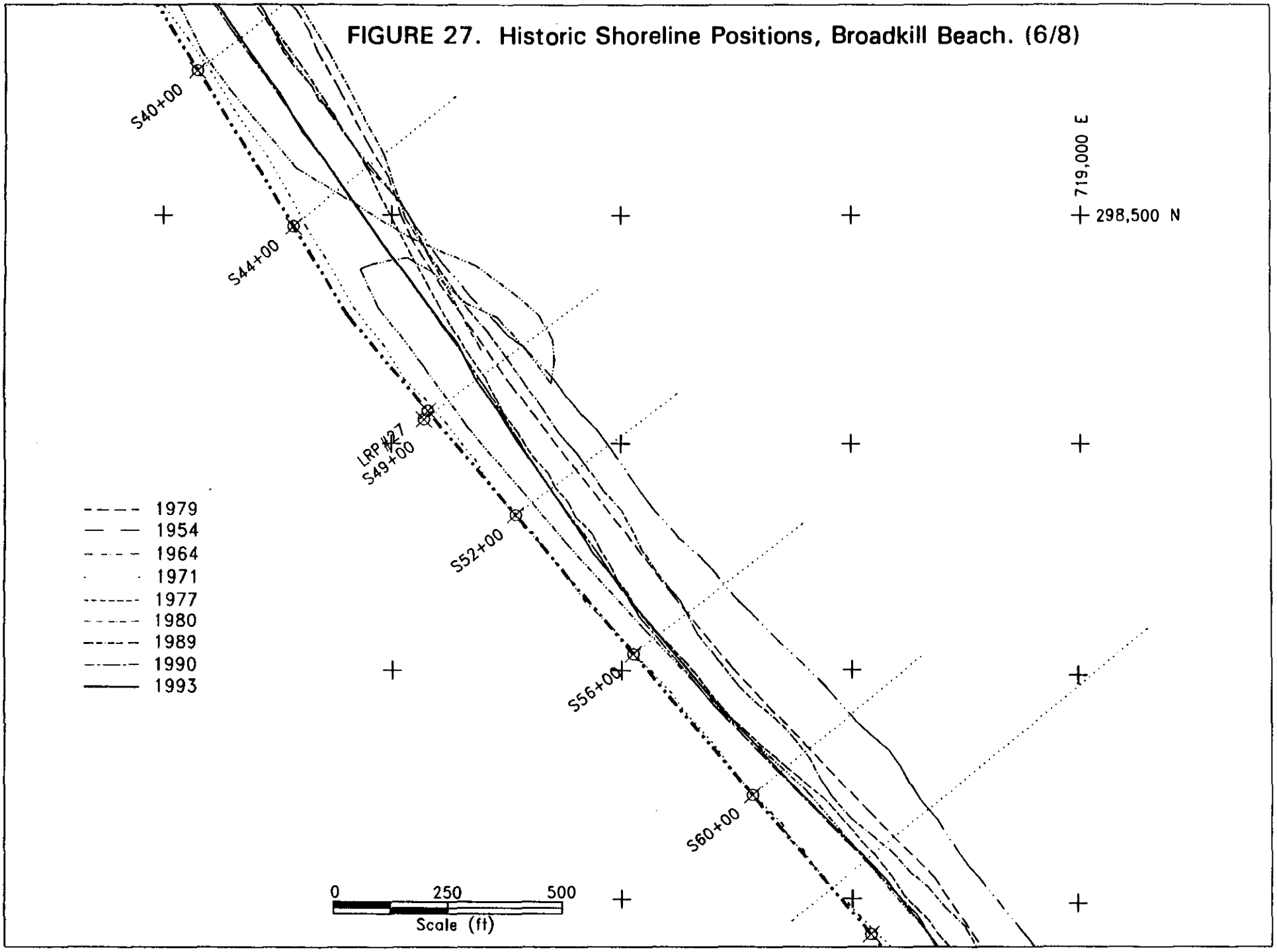


FIGURE 27. Historic Shoreline Positions, Broadkill Beach. (6/8)

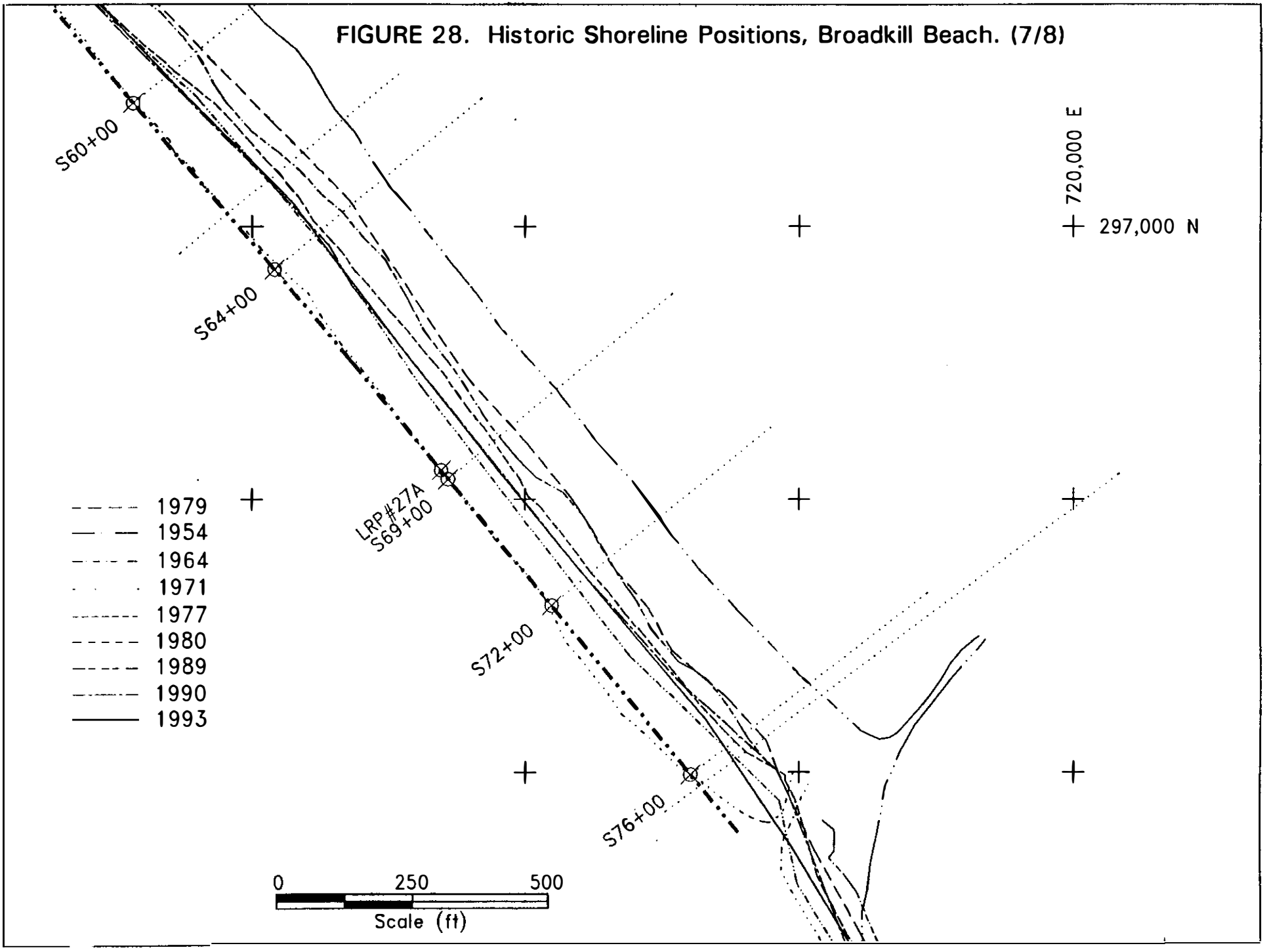


FIGURE 28. Historic Shoreline Positions, Broadkill Beach. (7/8)

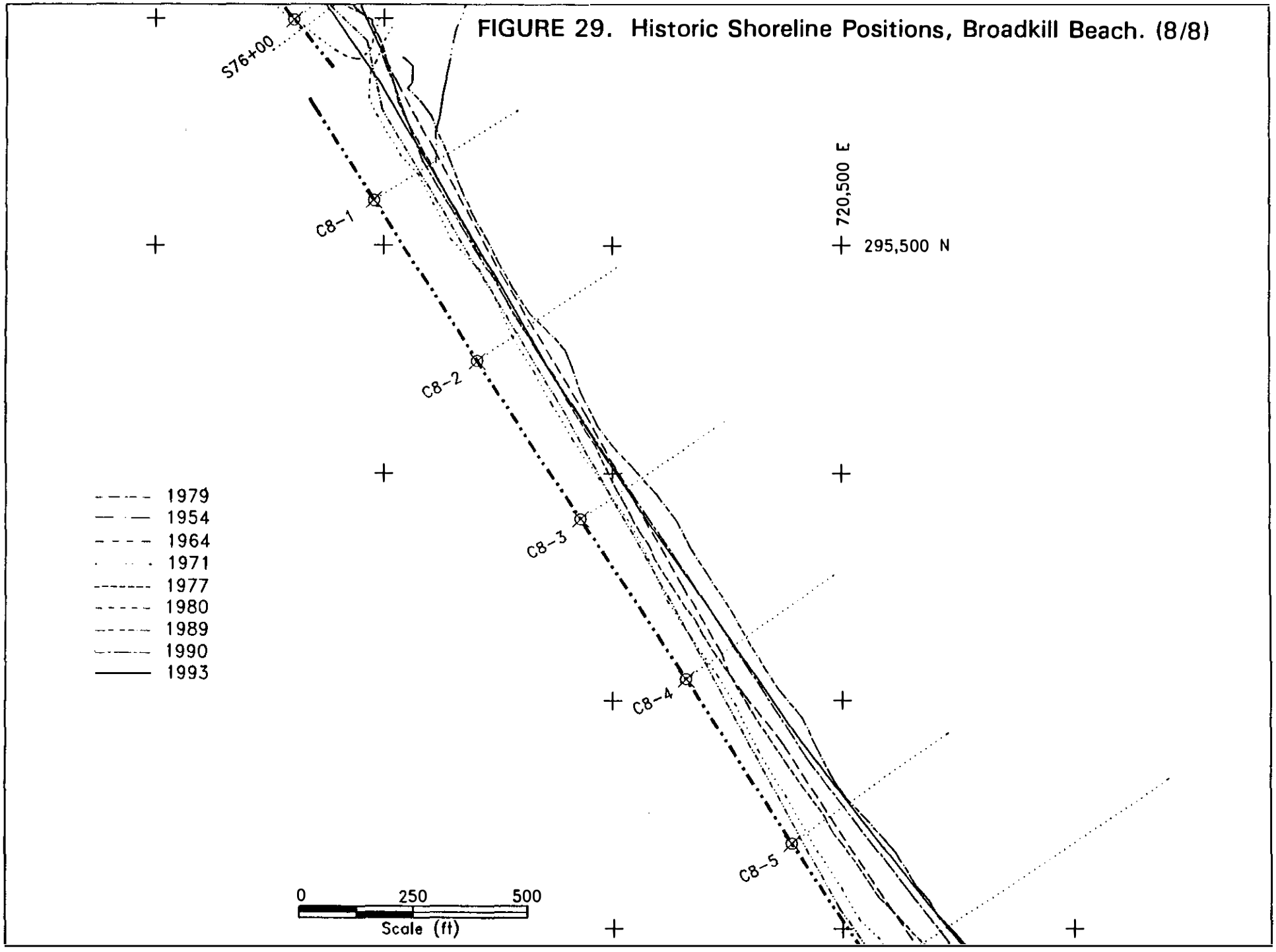


FIGURE 29. Historic Shoreline Positions, Broadkill Beach. (8/8)

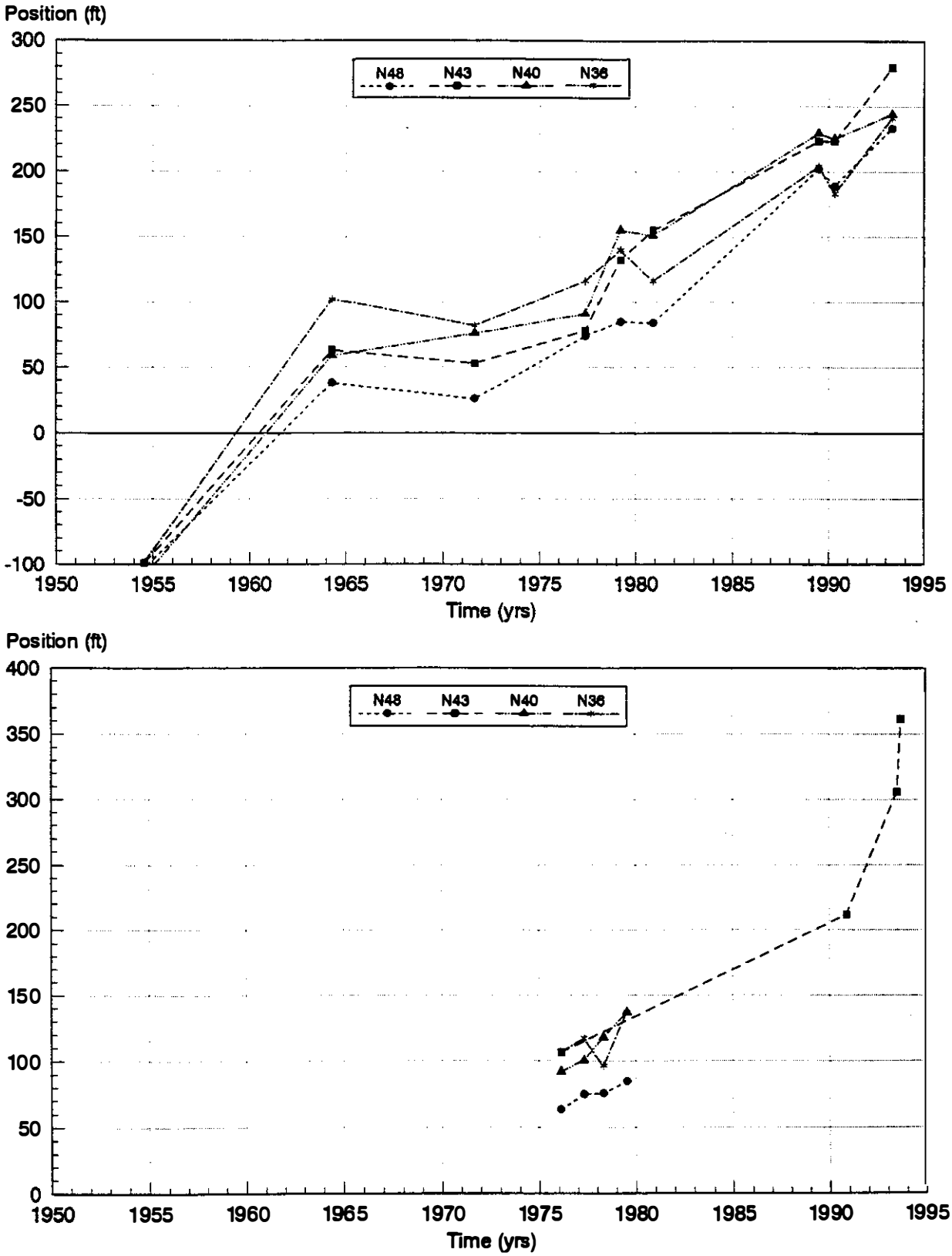
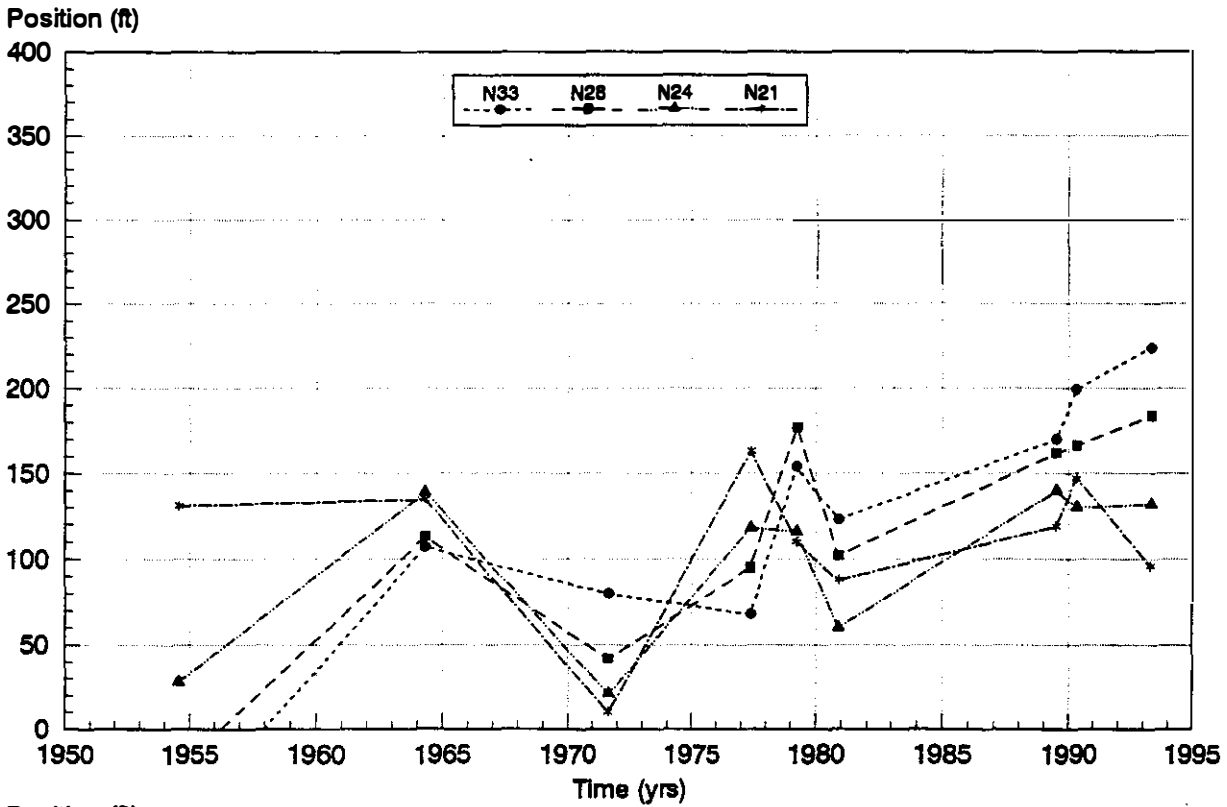


FIGURE 30. Shoreline Positions for Transects in Broadkill Beach Measured from DNREC Baseline. Top from aerial photos and T-sheets; bottom from surveyed profiles. No beach fills placed in this area. (1/8)

82



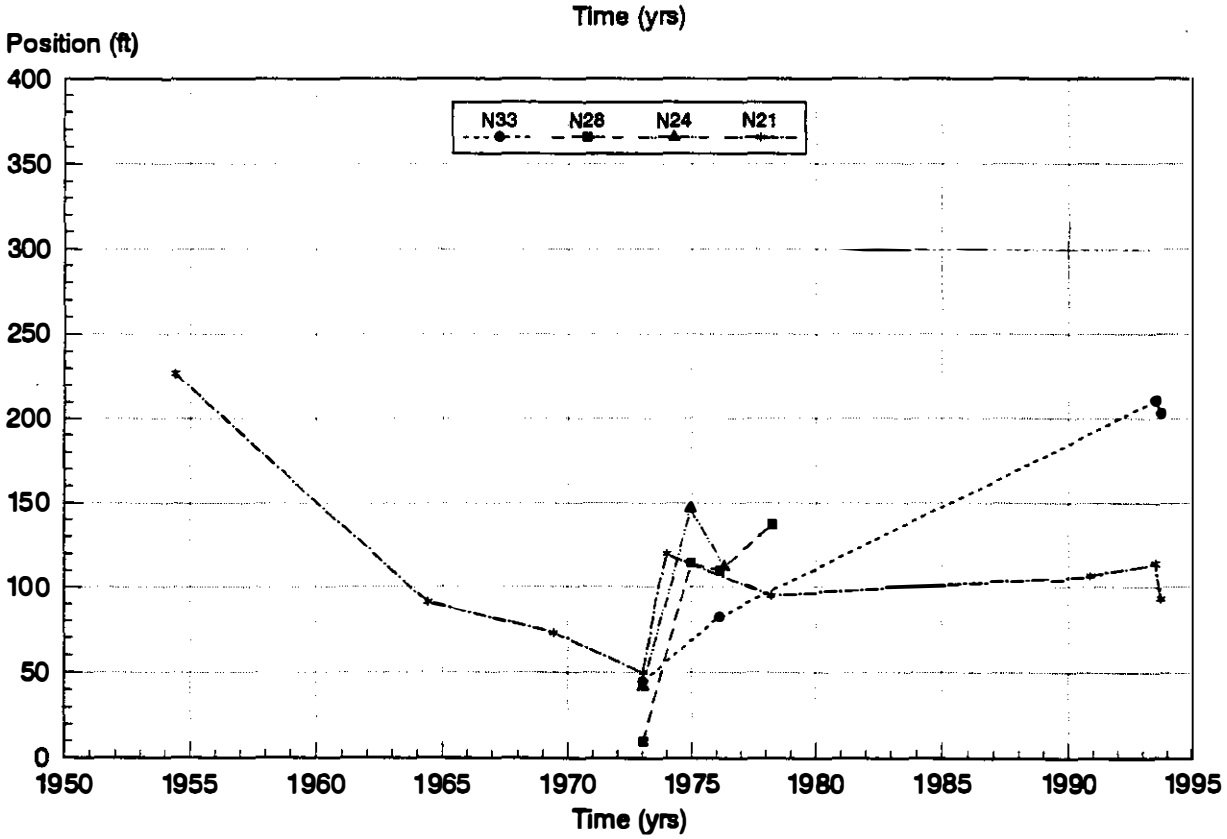


FIGURE 31. Shoreline Positions for Transects in Broadkill Beach Measured from DNREC Baseline. Top from aerial photos and T-sheets; bottom from surveyed profiles. Data unadjusted for beach fills. (2/8)

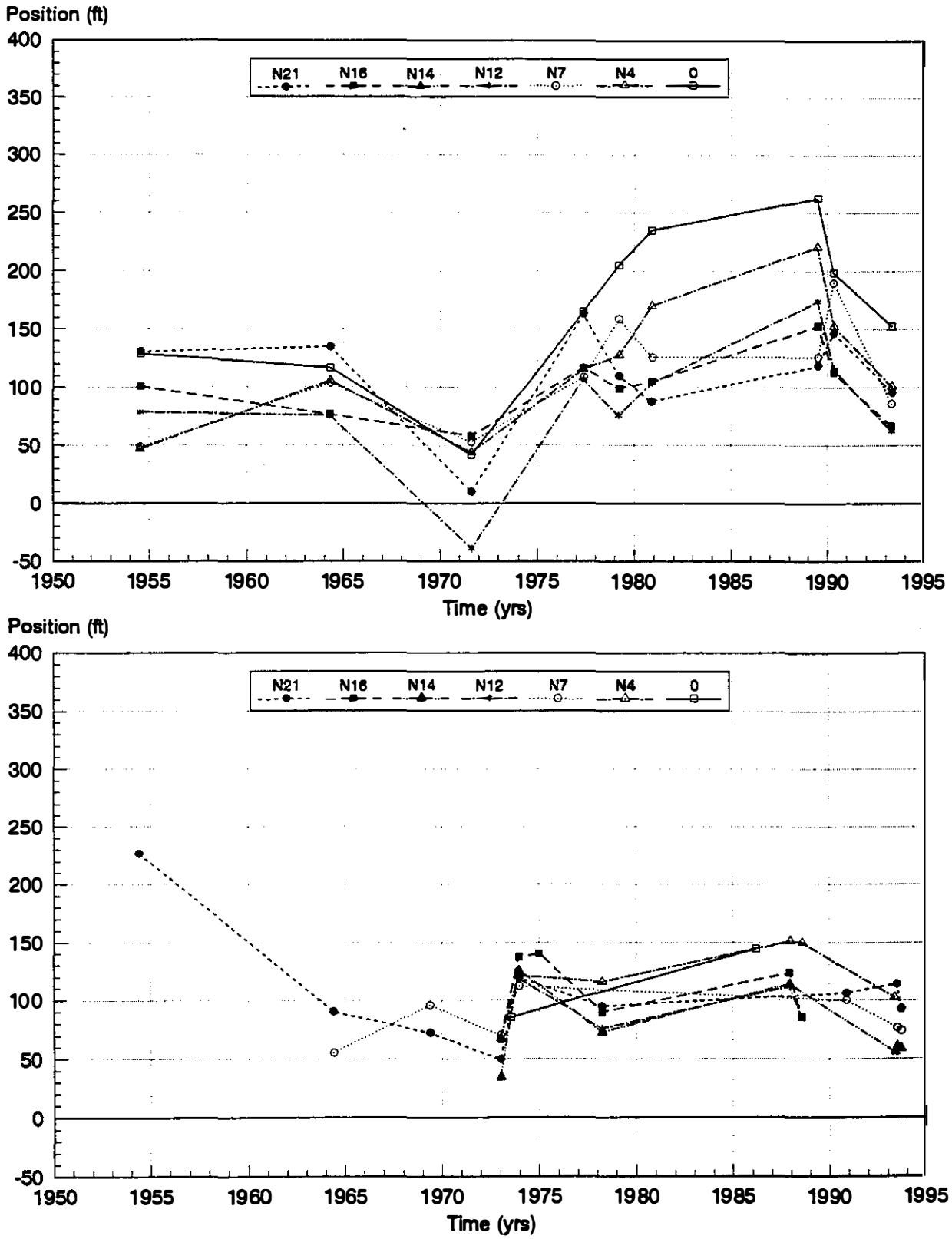


FIGURE 32. Shoreline Positions for Transects in Broadkill Beach Measured from DNREC Baseline. Top from aerial photos and T-sheets; bottom from surveyed profiles. Data unadjusted for beach fills. (3/8)

84

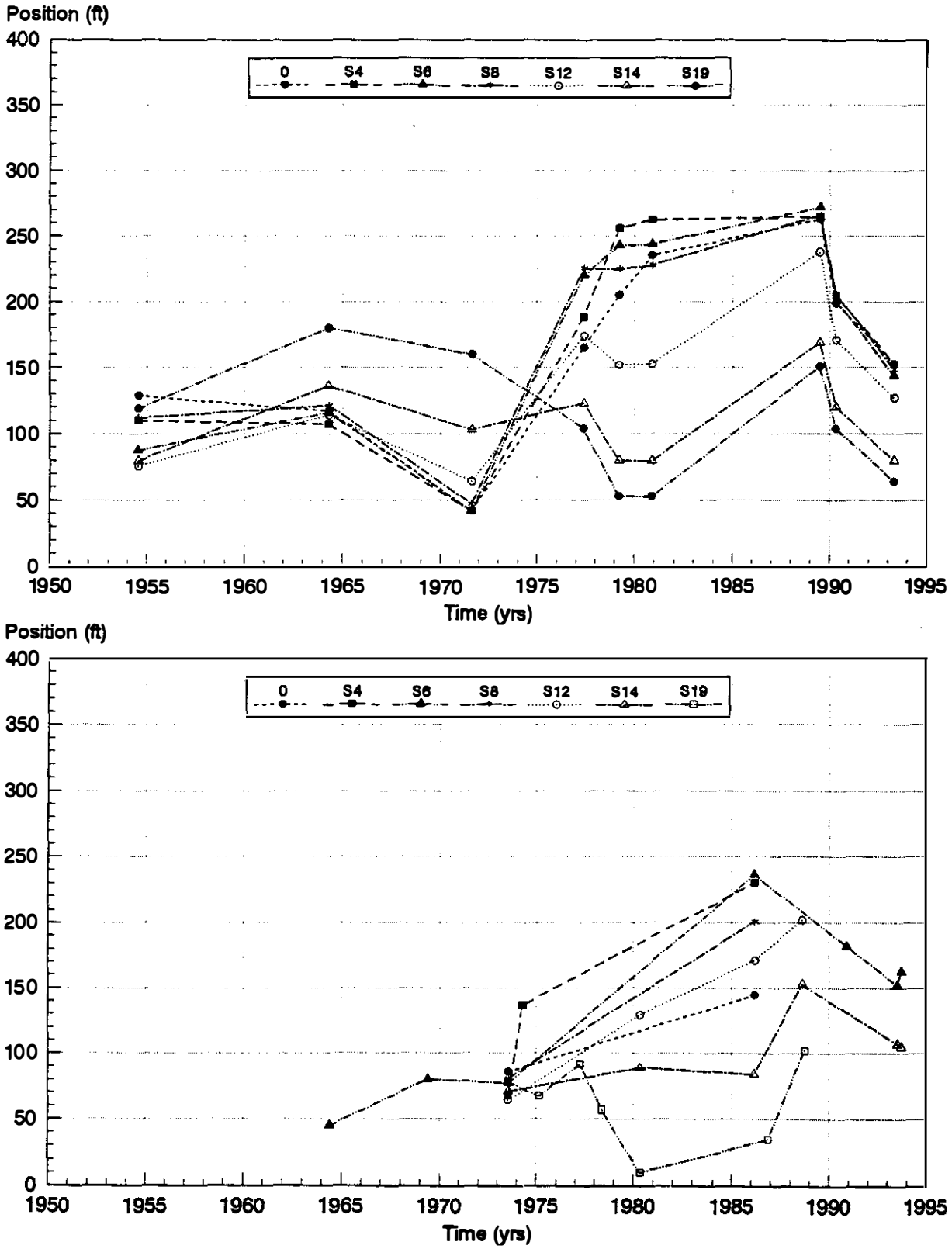


FIGURE 33. Shoreline Positions for Transects in Broadkill Beach Measured from DNREC Baseline. Top from aerial photos and T-sheets; bottom from surveyed profiles. Data unadjusted for beach fills. (4/8)

85

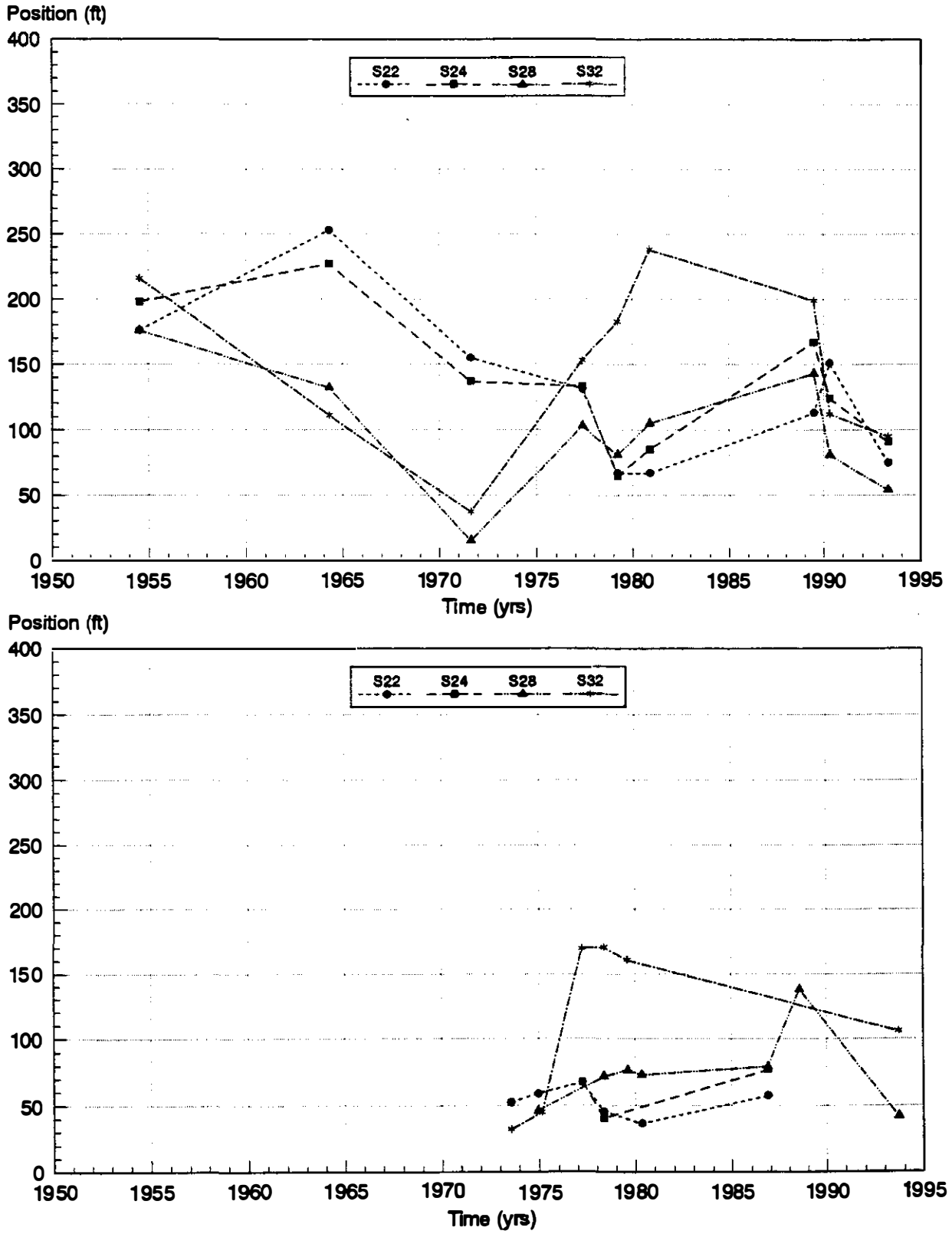


FIGURE 34. Shoreline Positions for Transects in Broadkill Beach Measured from DNREC Baseline. Top from aerial photos and T-sheets; bottom from surveyed profiles. Data unadjusted for beach fills. (5/8)

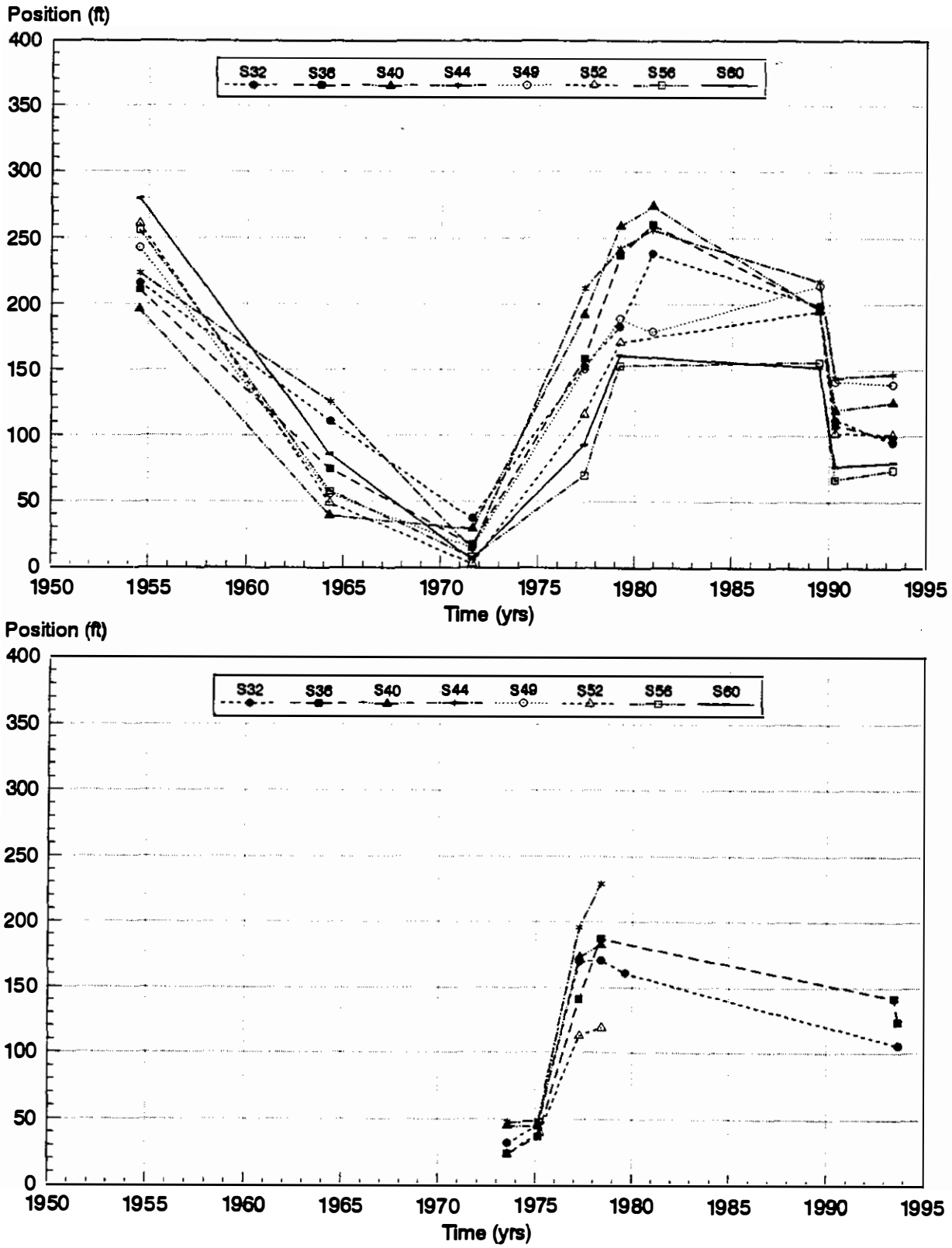


FIGURE 35. Shoreline Positions for Transects in Broadkill Beach Measured from DNREC Baseline. Top from aerial photos and T-sheets; bottom from surveyed profiles. Data unadjusted for beach fills. (6/8)

87

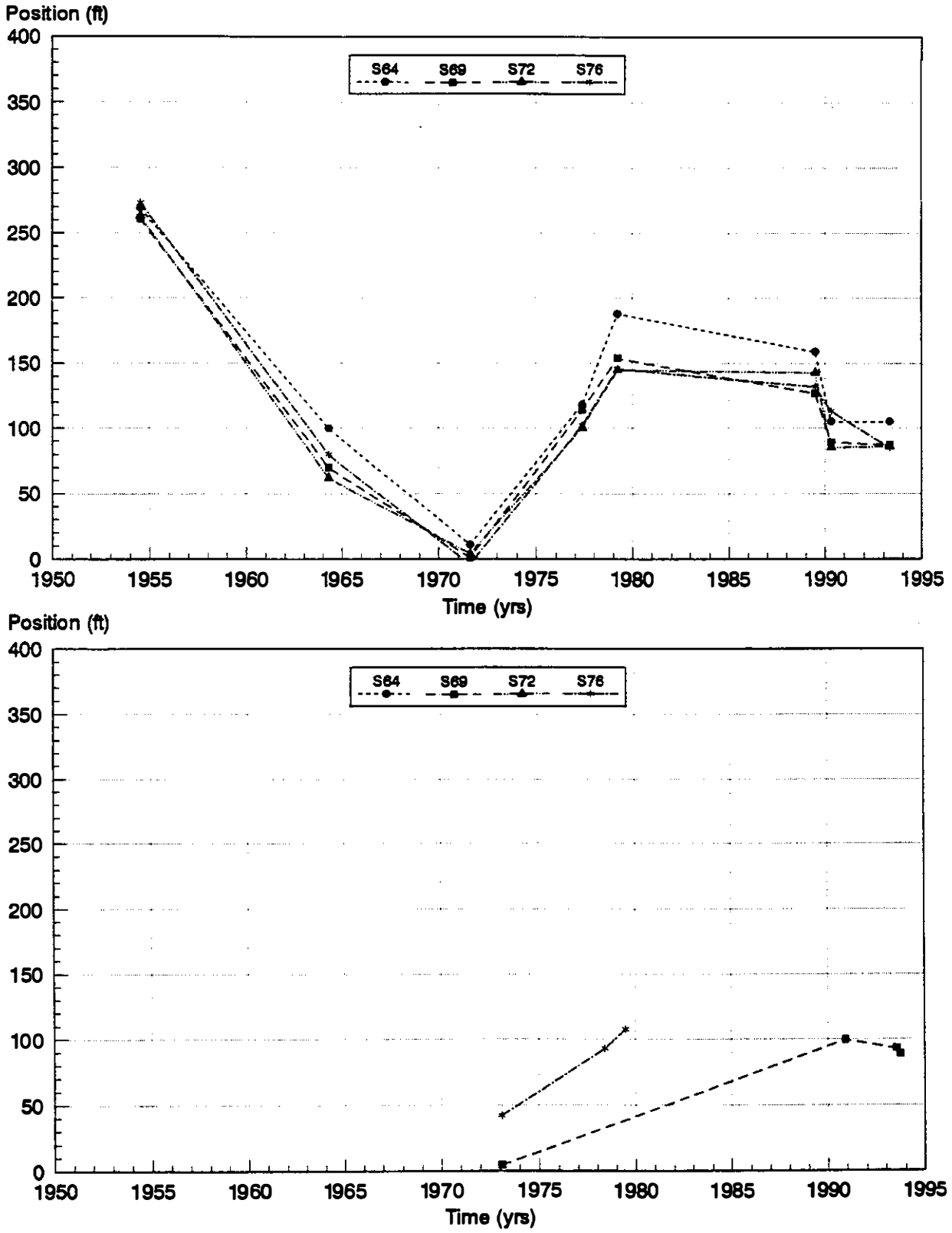


FIGURE 36. Shoreline Positions for Transects in Broadkill Beach Measured from DNREC Baseline. Top from aerial photos and T-sheets; bottom from surveyed profiles. Data unadjusted for beach fills. (7/8)

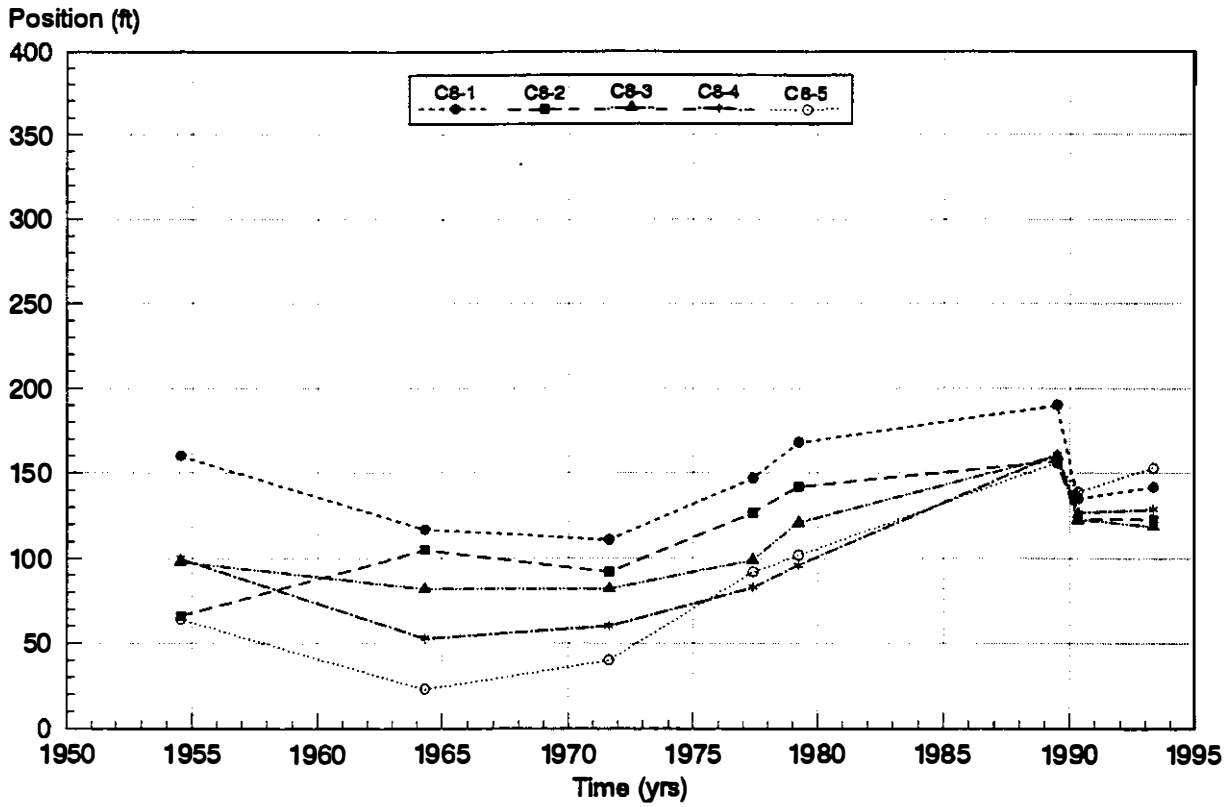


FIGURE 37. Shoreline Positions for Transects in Broadkill Beach Measured from DNREC Baseline. Data from aerial photos and T-sheets. Data unadjusted for beach fills. No profile data available in this area. (8/8)

In order to plan a beach fill project, profile shape, unit volume and volumetric erosion rates provide a basis for estimating the amount of fill required to provide a given level of storm protection or recreational high tide beach width. In addition, an estimate of the alongshore variation in volumetric erosion rates provides an important check on the results of a longshore transport analysis.

Although extensive beach profile data are best for determining historic beach change trends, sufficient quantities of such data with the necessary spatial and temporal extent are often not available. The relatively extensive amount of beach profile data at Broadkill Beach make the examination possible. In areas where the profile data are inadequate, the shoreline change data should be used instead.

Often in cases where there are shoreline position data but no profile data it would be necessary to utilize the shoreline changes as a means of estimating volumetric changes. At Broadkill where the shoreline position data is more extensive than the profile data, it would be useful to relate shoreline position changes to profile volumetric changes.

To do this, unit volume changes were plotted vs MHW shoreline changes. The profile volumes were computed from the dune crest to the apparent depth of closure as indicated on plots of the beach profiles. The changes in these quantities were plotted instead of the computed values so that the results would not be biased by the choice of starting and ending distances along the profile. The results are plotted in Figure 38 with the least squares best fit line drawn through the data. The correlation coefficient, r, has a value of 0.92 indicating a strong linear relationship between the unit volume and MHW change at Broadkill Beach. The relationship from the regression analysis is 1 cy/ft volume loss = 2.0 ft MHW recession.

IV-2.C. Volumetric Profile Analysis

Available beach profile data for Broadkill Beach were obtained from CENAP and DNREC. The data from DNREC spanned the period from the early 1970's to the late 1980's. The profiles are referenced to the survey baseline shown in Figures 22 to 29. The alongshore distances are referenced to a starting point at State Route 16 in the central portion of the beach. The DNREC profiles are spaced at 100 foot intervals. Based on WRCE's site visit in April 1994 and a review of the shoreline change history, a spacing of 400 feet is considered to be sufficient for estimating volumetric beach changes.

In addition to DNREC profiles, CENAP has eight "LRP" stations within the Broadkill Beach study area. Their locations are shown in Figures 22 to 29. In addition to one 1990 and two 1993 surveys, LRP stations 25, 25A, 25B, and 26 were surveyed in 1954 and 1964. The CENAP report on Broadkill Beach (USACE, 1972) presents plots of

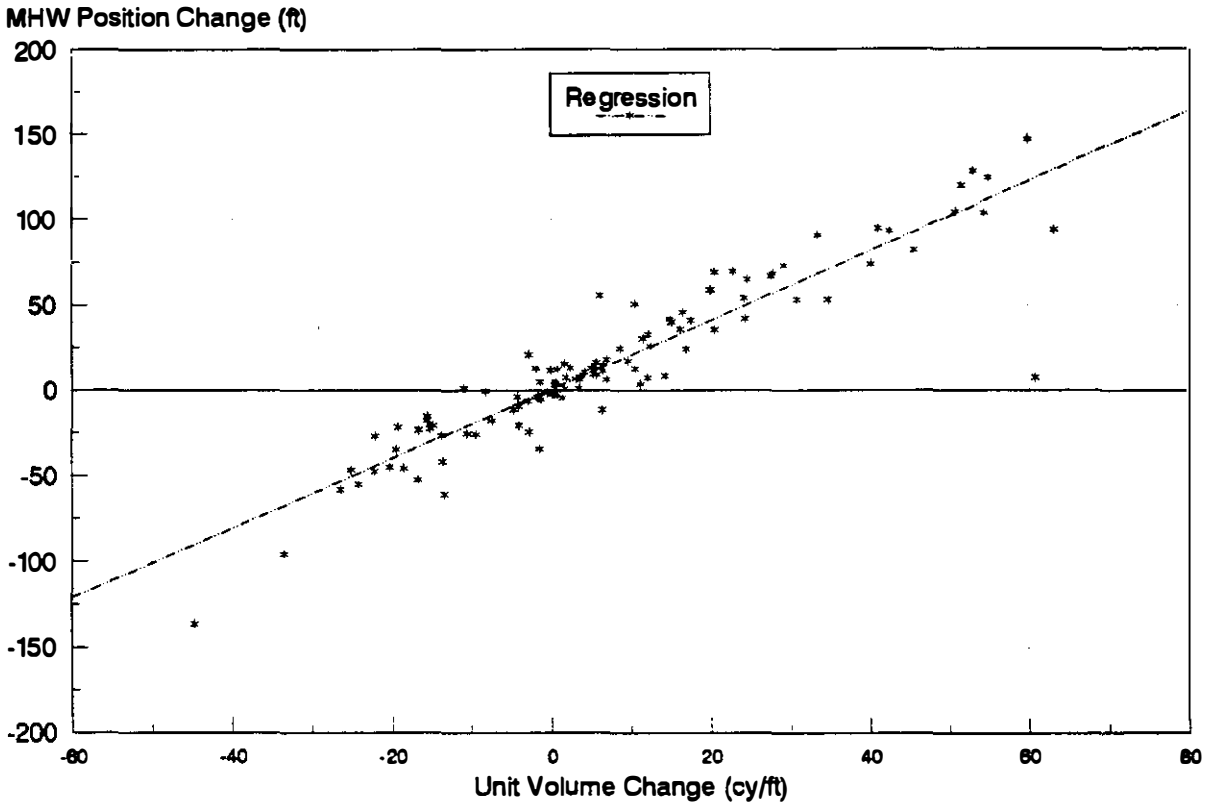


Figure 38 Profile Unit Volume Change vs. MHW Position Change for Profiles Used in Unit Volume Analysis.

91

the 1954 and 1964 profiles. These profiles were digitized from the plots to create a set of (distance, elevation) pairs for comparison with the more recent data.

For plotting and unit volume computations the DNREC baseline was used as the reference baseline. The transects used were chosen so that they would coincide as closely as possible with the LRP stations and maintain a profile spacing of 400 feet. The LRP data were referenced to the DNREC baseline so they could be compared directly with the DNREC data. The set of transects chosen for analysis are given in Table 3. Transect locations are the same as those used to determine historic shoreline changes.

For example, Figure 39 shows the available profiles for transect S6+00, which approximately coincides with transect LRP#25B. The plots for all the transects in Table 3 are included in the Appendix D. Also included in the Appendix D are plots of the available LRP data which extend bayward several thousand feet in contrast to the DNREC transect data which generally extend only several hundred feet from the baseline.

Both the CENAP data for the LRP transects and the DNREC data were in the USACE's Interactive Survey Reduction Program (ISRP) format. Each set of transect profiles were analyzed for apparent errors and inconsistencies. Unit volumes were calculated for discrete vertical profile lenses. The lenses chosen were dune crest to +2.5 ft, +2.5 to -1.5 ft, and -1.5 to -10.0 ft NGVD. These lenses approximately represent the above-tide, tidal, and below-tide elevation ranges, respectively. Starting distances were chosen to match the most landward common limit or the dune crest. Ending distances were chosen to match the apparent depth of closure for the toe of the beach face, beyond which the profiles slopes become extremely mild. In many cases the DNREC profile data terminated at relatively high

DNREC Station	LRP Station
N48+00	
N43+00	
N40+00	#24A
N36+00	
N33+00	
N28+00	
N24+00	
N21+00	#25
N16+00	
N14+00	
N12+00	
N 7+00	#25A
N 4+00	
0+00	
S 4+00	
S 6+00	#25B
S 8+00	
S12+00	
S14+00	
S19+00	
S22+00	#26
S24+00	
S28+00	
S32+00	
S36+00	
S40+00	
S44+00	
S49+00	#27
S52+00	
S69+00	#27A
S76+00	
	#28

Table 3. Transect Stations Used in Profile Volume Analysis.

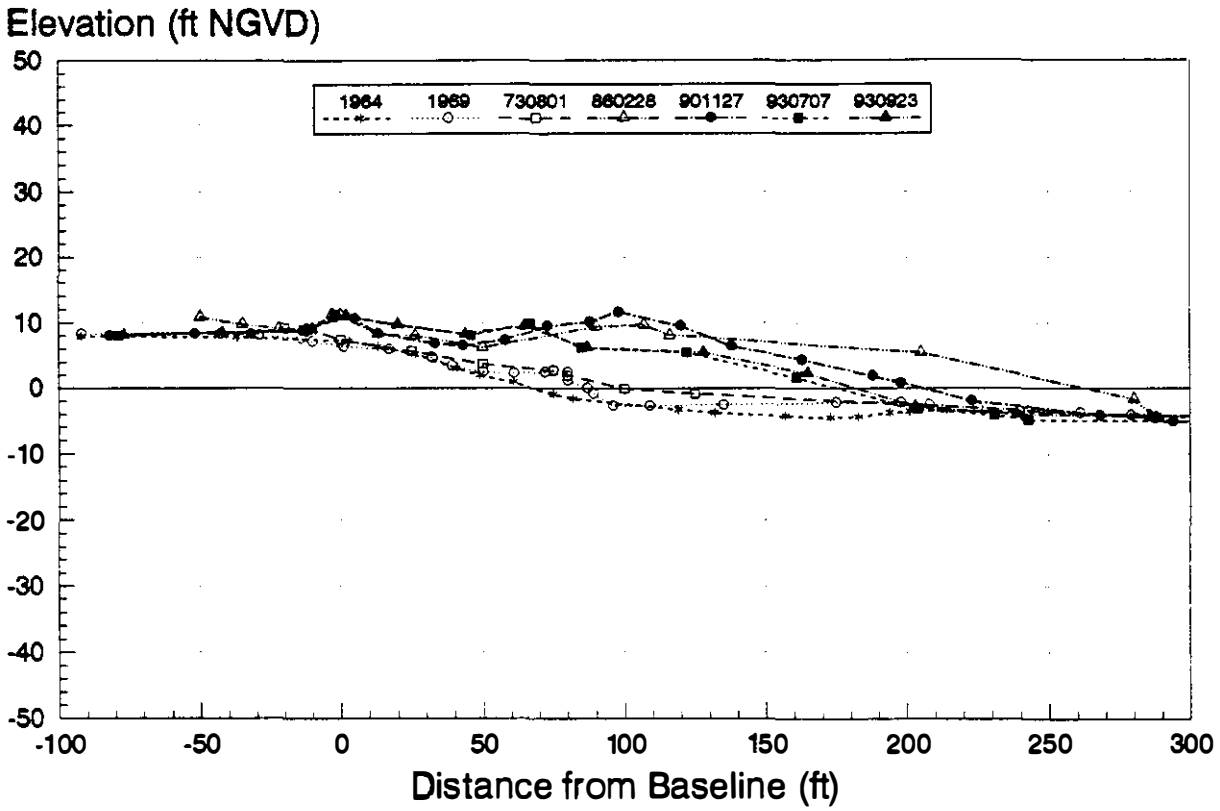


Figure 39 Beach Profiles at Transect S6+00.
Data from LRP#25B included.

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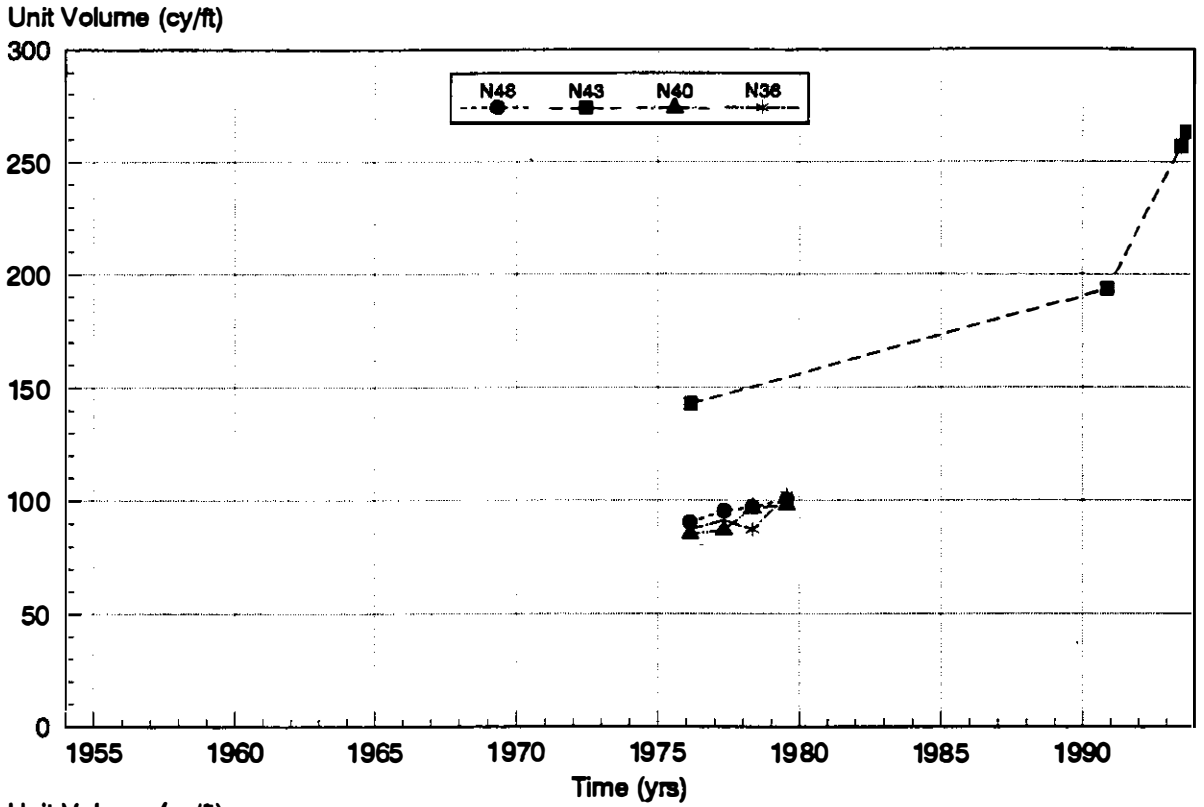
elevations on the beach face (0.0 to -2.0 ft NGVD) and had to be artificially extended to tie into with the beach face toe as determined from the other transects in the area of interest.

Figures 40 to 43 plot the unit volumes vs time for the entire profile from dune crest to -10 ft NGVD for the study area from north to south. The data in these figures have not been adjusted for the effects of beach fills at Broadkill. The unit volume themselves are sensitive to the selection of the starting and ending distances and the vertical lens limits. In most cases the ending distance was chosen to coincide with the apparent closure limit. In most open-coast settings there is a definite dune system and a well-defined depth of closure with a significant profile slope change. At Broadkill however there are areas where there is no definite dune system or depth of closure considering the very mild profile slopes adjacent to the bay slopes. As a result, alongshore unit volume variations may be dependent on the cross-shore extent of the survey data.

The unit volumes plotted in Figures 40 to 43 are presented in Table 4. The dates are in the year-month-day format. A complete set of unit volume output is included in the Appendix D.

Examination of the data in Table 4 shows that there are many coverage gaps. Of the transects considered, less than 40% have been surveyed in the 1990's. For nearly 30%, the most recent survey was conducted in the 1970's. The remainder were last surveyed sometime in the 1980's. The alongshore coverage for any given survey date is also sporadic.

It is probable that the trends indicated for most transects do not adequately describe the beach history at Broadkill Beach due to the incompleteness of the data. The most reliable volumetric trends should result from those transects having the greatest temporal coverage (i.e. transects combining the LRP and DNREC survey data). These data along with the shoreline change data and information on the history of groins and beach fills will be evaluated carefully to determine representative shoreline and beach profile change rates.



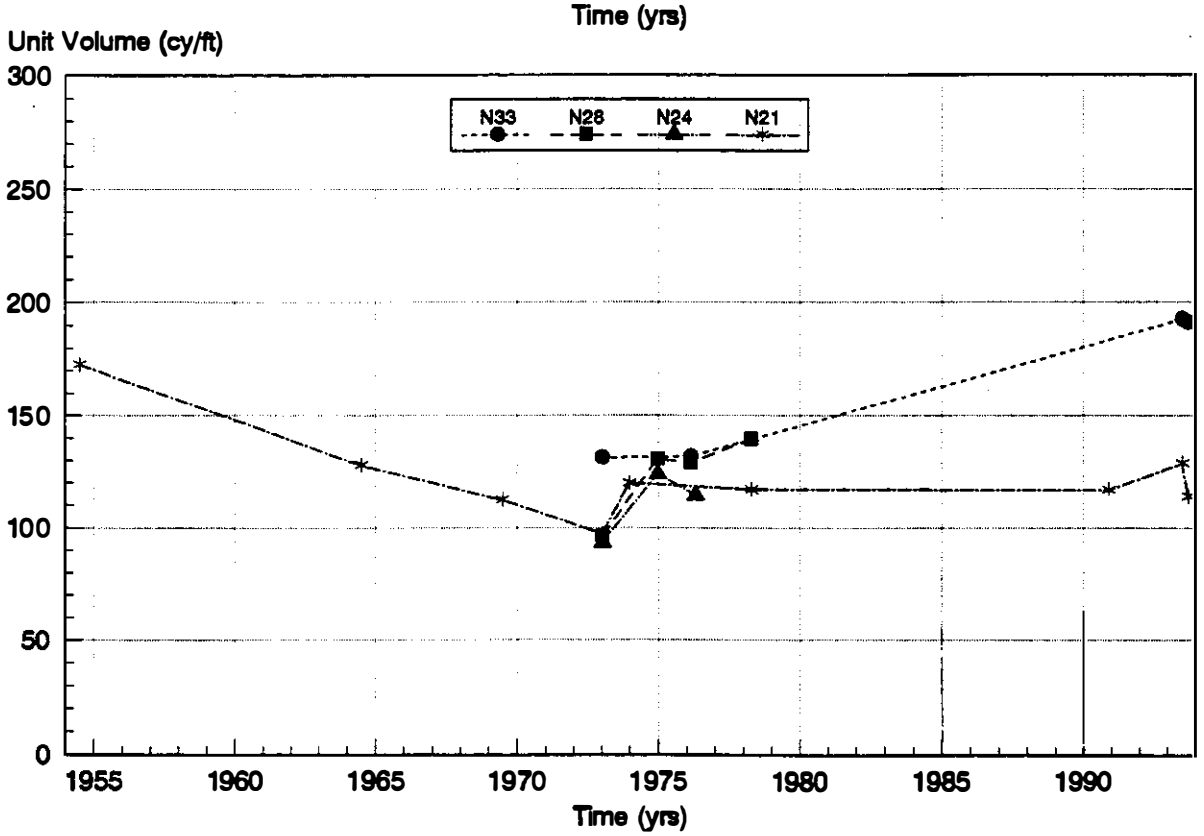


FIGURE 40. Broadkill Beach, Unit Volumes vs. Time. Unit volumes are calculated from dune crest to -10 ft. Volumes are unadjusted for beach fills. (1/4)

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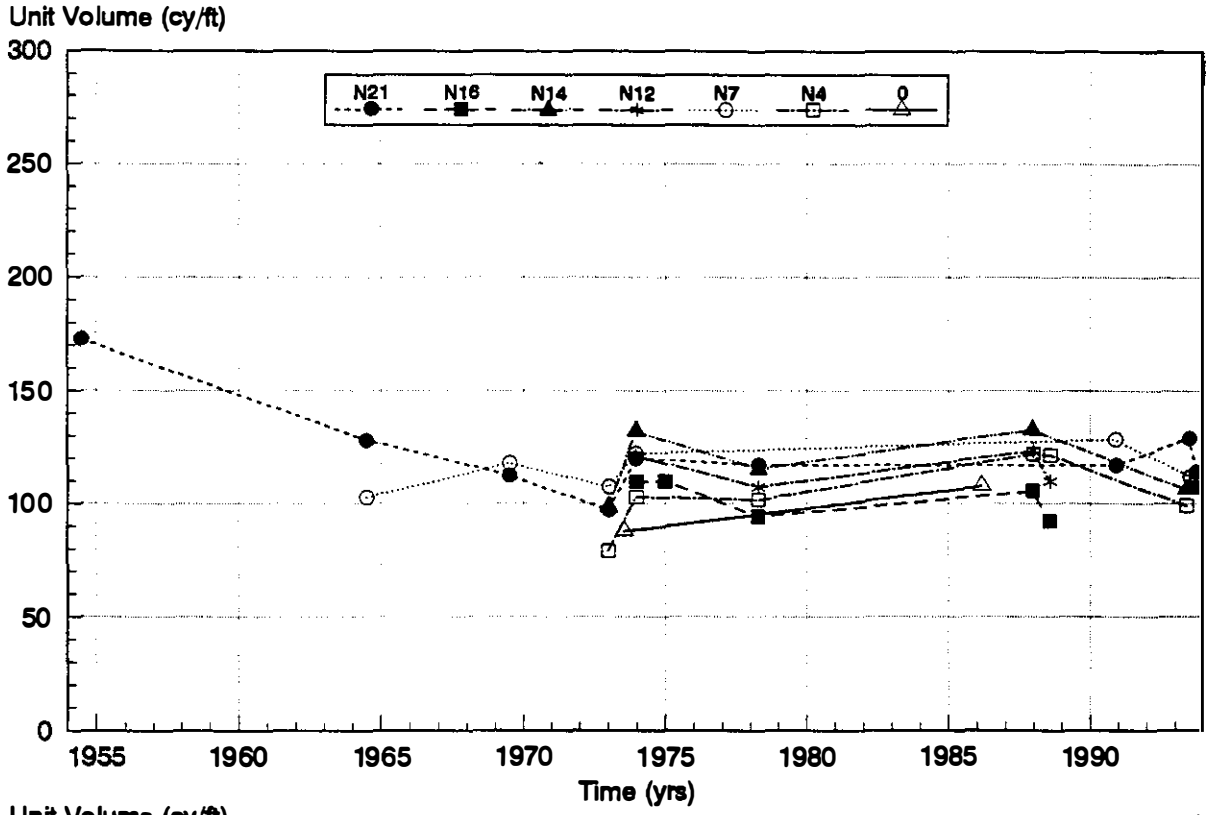


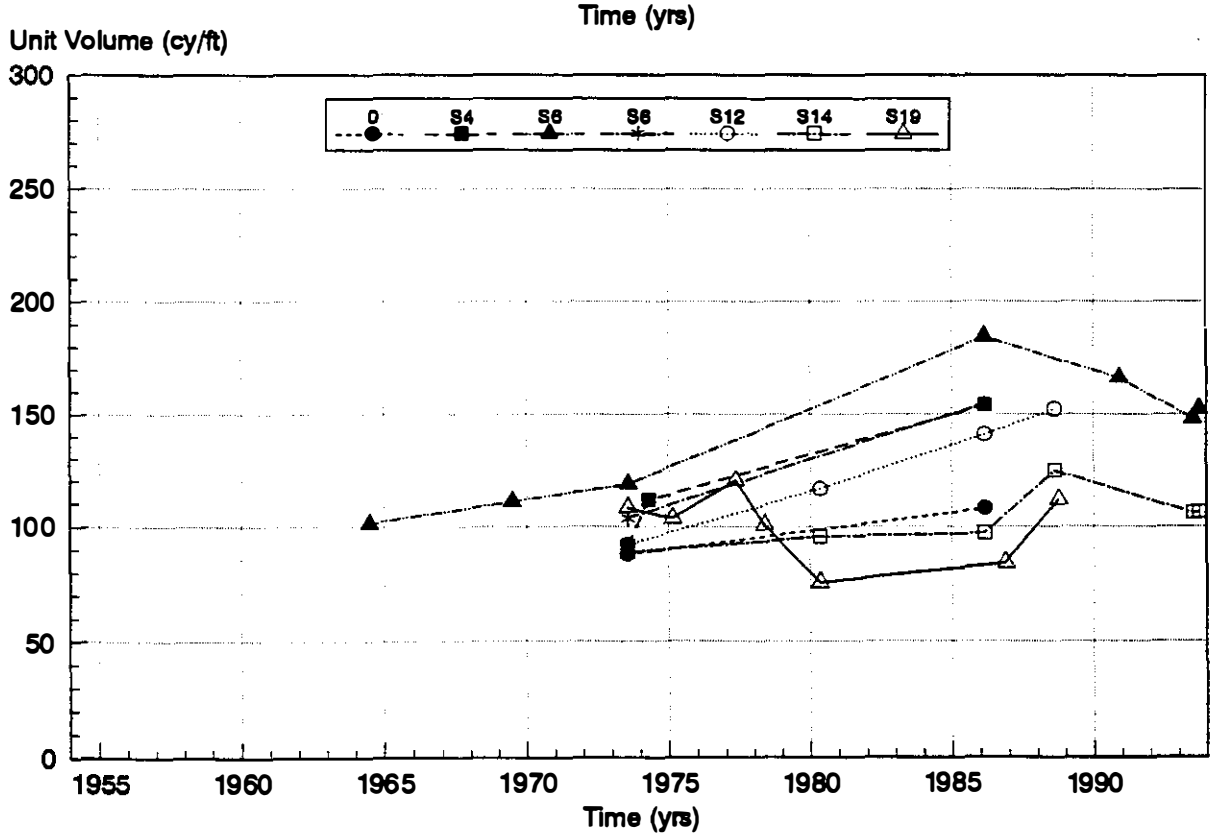
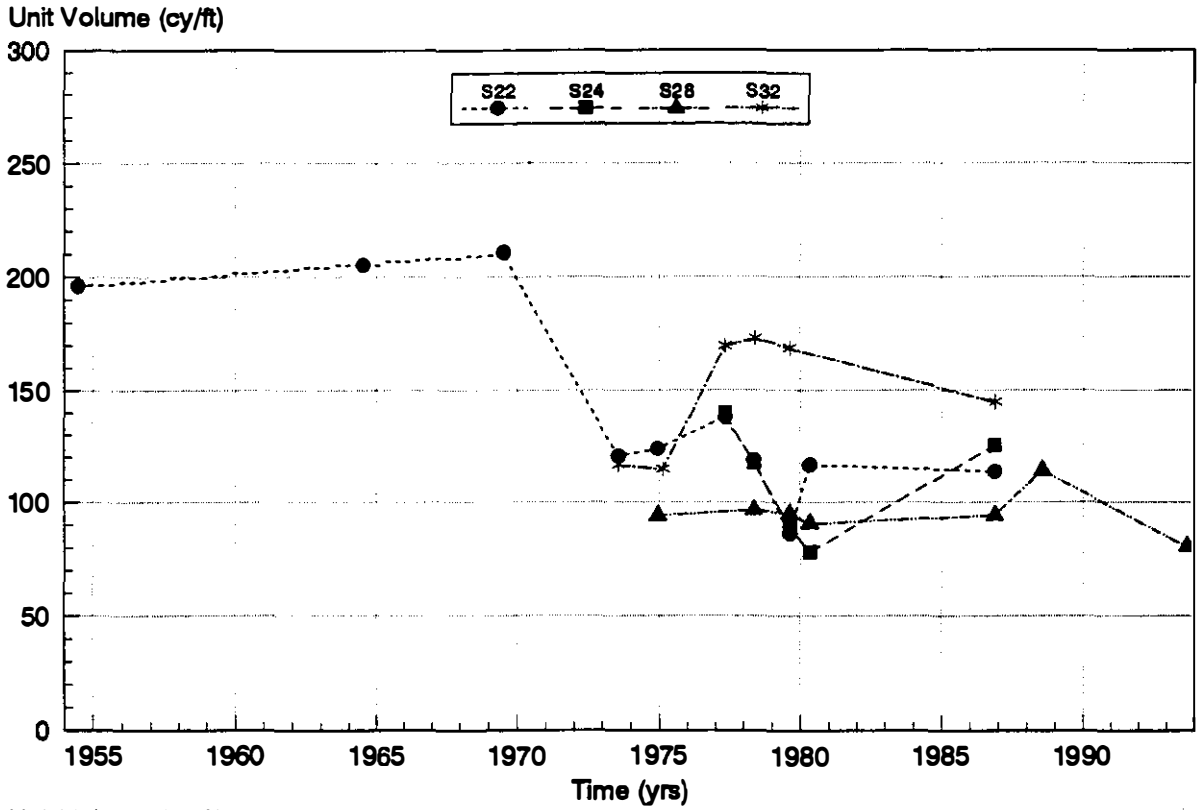


FIGURE 41. Broadkill Beach, Unit Volumes vs. Time. Unit volumes are calculated from dune crest to -10 ft. Volumes are unadjusted for beach fills. (2/4)



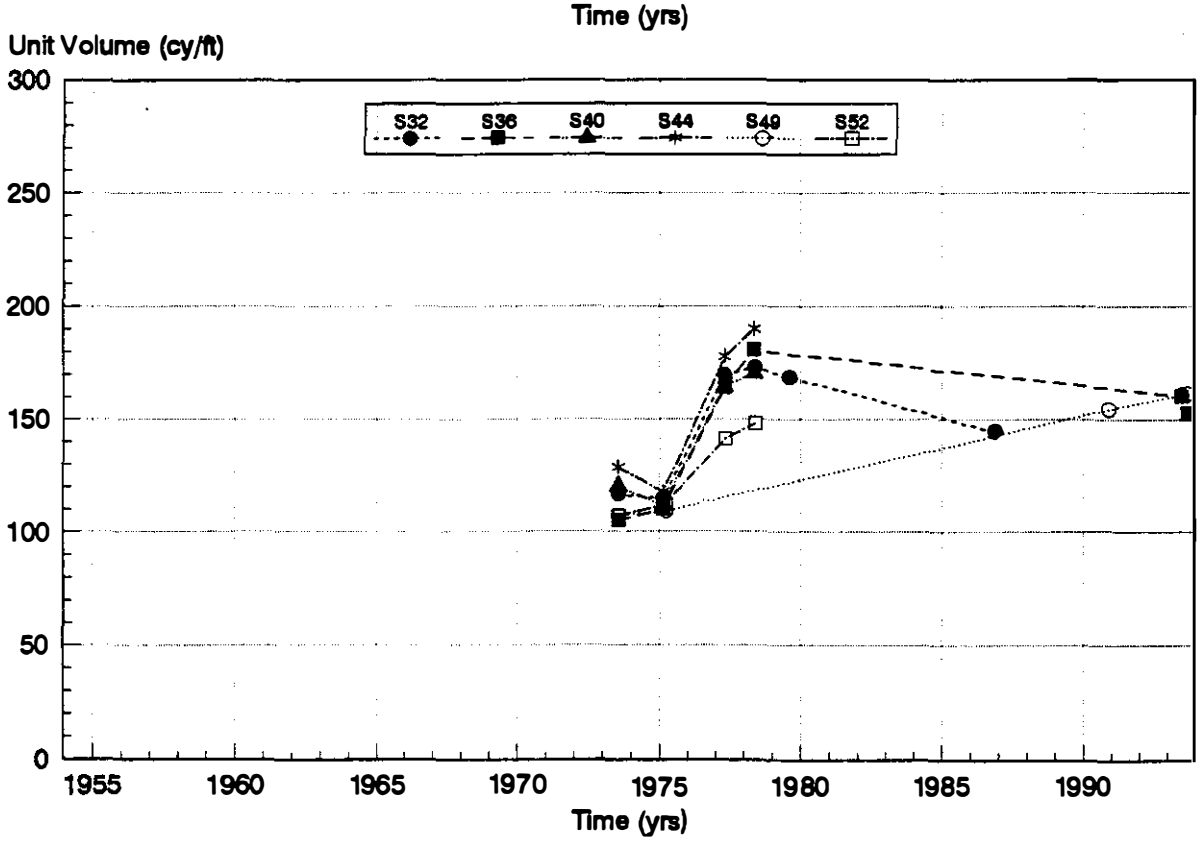


FIGURE 42. Broadkill Beach, Unit Volumes vs. Time. Unit volumes are calculated from dune crest to -10 ft. Volumes are unadjusted for beach fills. (3/4)

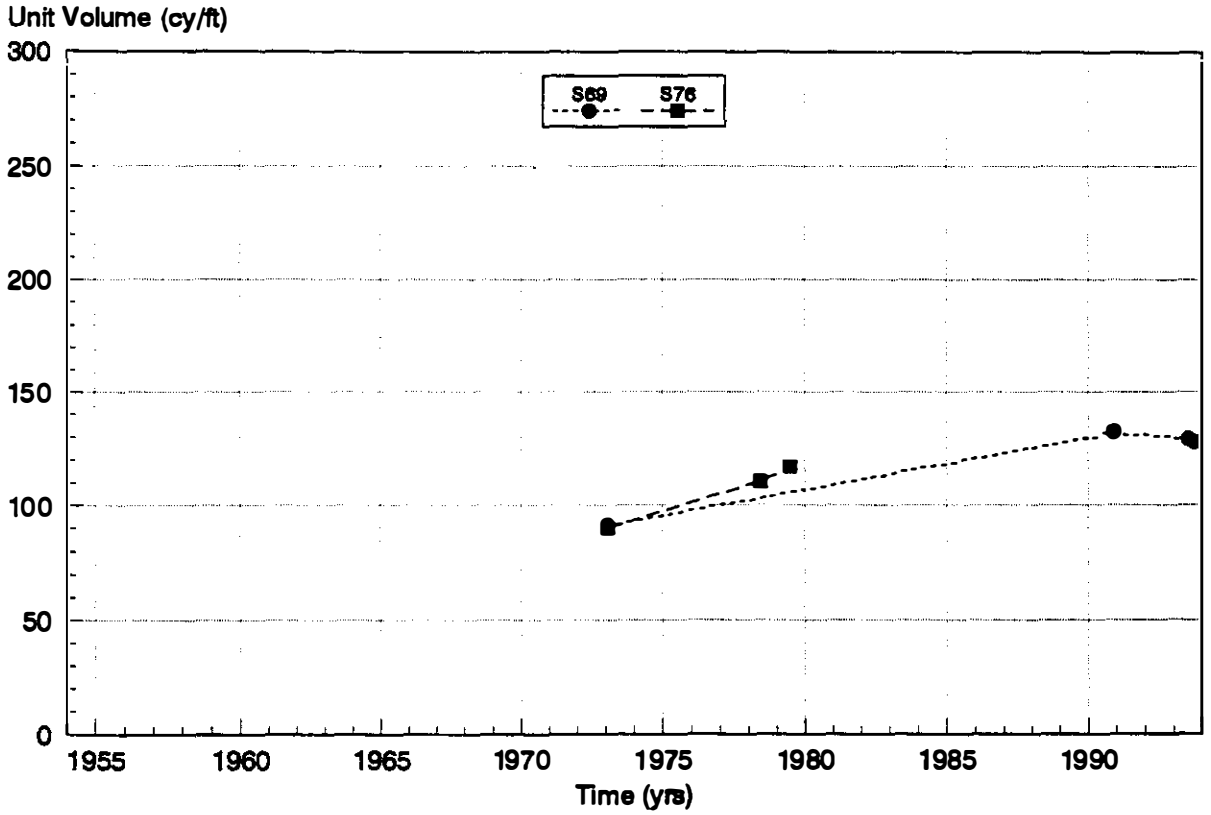


FIGURE 43. Broadkill Beach, Unit Volumes vs. Time. Unit volumes are calculated from dune crest to -10 ft. Volumes are unadjusted for beach fills. (4/4)

Table 4. Unit Profile Volumes

Figure 40
TOP GRAPH

Volume in cy/ft from dune crest to -10 ft NGVD

Date	N48+00	N43+00	N40+00	N36+00
760216	90.3			
760224		142.9	85.3	87.4
770503	95.5		87.1	91.5
780502	97.1		96.7	87.3
790718	100.6		98.3	102.3
901127		193.7		
930707		256.8		
930923		262.9		

FIGURE 40
LOWER GRAPH

Volume in cy/ft from dune crest to -10 ft NGVD

Date	N33+00	N28+00	N24+00	N21+00
540630				172.8
640630				128.1
690630				112.5
730112	131.4	96.2	93.1	97.2
731227				120.0
741226		130.9	123.9	
760224	132.1	129.1		
760426			114.5	
760411		139.6		117.2
901127				117.0
930707	192.8			129.0
930923	191.3			114.1

Table 4. Unit Profile Volumes (continued)

FIGURE 41
TOP GRAPH

Date	Volume in cy/ft from dune crest to -10 ft NGVD N21+00	N16+00	N14+00	N12+00	N7+00	N4+00	0+00
540630	172.8						
640630	128.1				102.9		
690630	112.5				117.9		
730112	97.2		98.7		107.4	78.8	
730801							87.9
731227	120.0	109.8	132.1	121.0	122.2	103.0	
741226		110.1					
760411	117.2						
780411		94.5	115.4	107.4		101.6	
860228							107.9
871211		105.6	132.8	123.5		122.0	
880726		92.2		109.8		121.4	
901127	117.0				128.5		
930524			106.4			99.2	
930707	129.0		106.9		111.9		
930923	114.1		106.9		112.5		

FIGURE 41
LOWER GRAPH

Date	Volume in cy/ft from dune crest to -10 ft NGVD 0+00	S4+00	S6+00	S8+00	S12+00	S14+00	S19+00
640630			101.4				
690630			111.0				
730801	87.9	91.1	118.7	103.4	91.9	88.6	108.2
740417		111.5					
750220							103.4
770401							120.2
780517							100.7
800504					116.5	95.5	75.6
860228	107.9	153.9	184.4	154.9	140.8	96.8	
861120							84.2
880809					152.3	124.6	
881002							111.8
901202			166.3				
930707			148.3			106.1	
930923			152.5			106.4	

Table 4. Unit Profile Volumes (continued)

FIGURE 42
TOP GRAPH

Volume in cy/ft from dune crest to -10 ft NGVD

Date	S22+00	S24+00	S28+00	S32+00
730801	120.6			116.7
741213	123.6		94.1	
750220				114.7
770401	137.8	139.5		169.6
780517	118.5	117.4	96.5	
780530				173.0
790816	86.2	89.3	95.0	168.8
800504	116.5	78.0	90.7	
861120	113.6	125.1	94.2	144.6
880726			114.0	
930923			80.5	

FIGURE 42
LOWER GRAPH

Volume in cy/ft from dune crest to -10 ft NGVD

Date	S32+00	S36+00	S40+00	S44+00	S49+00	S52+00
730801	116.7	105.0	120.1	128.8	106.9	106.8
750220	114.7	110.0	111.8	117.8		
750325					109.2	112.4
770401	169.6	164.3	164.8	177.7		141.5
780523		180.7	170.4	189.8		
780530	173.0					148.4
790816	168.8					
861120	144.6					
901127					154.6	
930707		160.4			161.0	
930923		152.9			161.5	

FIGURE 43

Volume in cy/ft from dune crest to -10 ft NGVD

Date	S69+00	S76+00
730203	91.1	90.1
780530		110.6
790625		117.1
901127	132.1	
930707	129.1	
930923	127.6	

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IV-3. HISTORY OF ENGINEERED WORKS AT BROADKILL BEACH

IV-3.A. Groin History

In 1950, three timber groins were built to stabilize the south end of Broadkill Beach. During the next several years erosion accelerated in the area immediately north of the original three. In 1954, two additional groins were constructed in the eroded area. These were also timber groins with one having a crib in its central section with quarrystone placed inside.

Erosion to the north of this expanded set of five groins apparently increased and in 1964 the Delaware Highway Department dumped broken concrete, presumably from broken road sections, on the beach to form two additional "groins". It is unlikely that these "groins" were of much value, being low and permeable, a poor combination in groin design. By sometime in the 1970's these two groins were not functioning while the other five were in "relatively good condition" (DNREC report on Broadkill Beach, undated). Also in 1964 a "revetment" of concrete rubble was placed from North Carolina Avenue to 700 feet north of Alabama Avenue (Stations N12+00 to N26+00).

Table 5 provides summary information for these seven groins and Figure 44 shows their location and the location of the revetment placed in 1964.

Table 5. Groin Construction History.

Year Built	Type	Location	Length	Crest Elev Range
1950	Timber	N2+00	195 ft	+11 to +1 ft MLW
1950	Timber	S2+65	195 ft	+11 to +1 ft MLW
1950	Timber	S7+40	200 ft	+11 to +1 ft MLW
1954	Timber*	N6+70	195 ft	+12 to +1 ft MLW
1954	Timber	N11+70	185 ft	+12 to +1 ft MLW
1964	Rubble	N20+80		
1964	Rubble	N16+40		

* with stone-filled crib

Groin lengths without reference to a recoverable baseline that tied to historic shoreline positions is not very useful. The important "length" for groin design is the length from the MSL shoreline position to the seaward end of the groin, the groin head. Much work has been done to show that an important variable in groin design is the spacing to length ratio, s/l. This parameter is defined for each groin "cell". A cell is comprised of two adjacent groins and the beach between them. The spacing is the alongshore distance between the groins. The length to use in determining the value of s/l is the average distance from the MSL shoreline to the

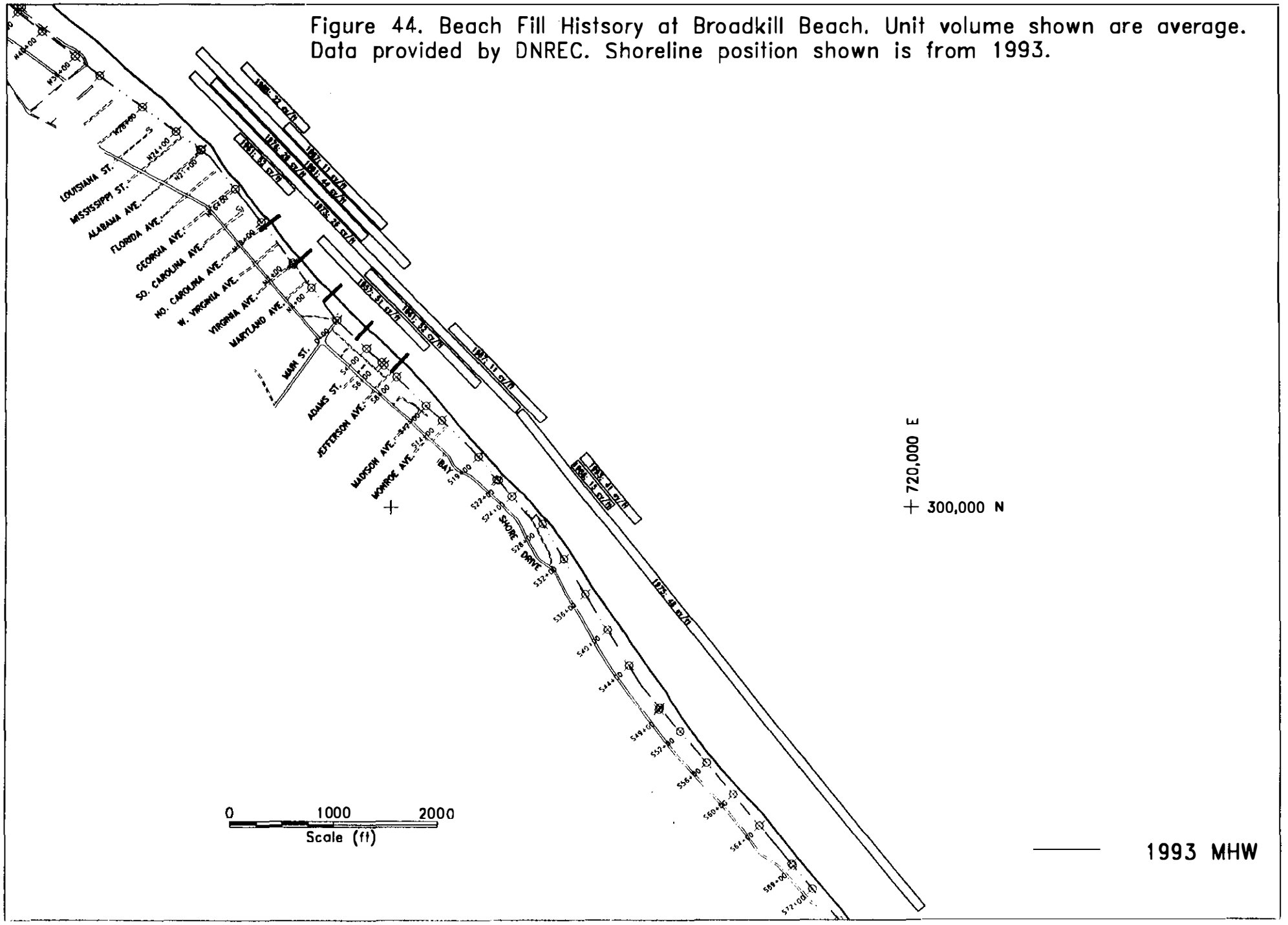

Figure 44. Beach Fill Histsory at Broadkill Beach. Unit volume shown are average. Data provided by DNREC. Shoreline position shown is from 1993.

groin head. For a variety of reasons (CSE, 1993) s/*l* ratios of 2-3 are generally recommended.

The groin spacing at Broadkill was generally 450 to 500 ft. A reasonable assumption is that the groins were constructed so that approximately 75% of their length extended from the MSL contour to their bayward ends. If so, the s/*l* ratio is about 3.

As-built groin profiles are not available. It is unknown whether the groin profiles consisted of a single slope connecting the endpoint top elevations listed or were constructed of two or more sections having different slopes. The latter case, utilizing a flat or mild upper section slope, is a common design for groins. It allows the upper portion of the groin cell to fill to a level even with the groin crest.

IV-3.B. Beach Fill History

Based on recommendations contained in a 1956 COE study, the Delaware State Highway Department placed the first beach fill at Broadkill in 1957 and again in 1961. In 1972 the state authority for beach maintenance passed from the highway department to DNREC who placed the next several beach fills. The first Federal involvement in beach maintenance at Broadkill occurred in 1976 with the cost sharing of that year's beach fill operation. According to the USACE (1981), a 50 foot wide berm was constructed at Broadkill Beach in 1976.

Table 6 presents a summary of the beach fill project history at Broadkill Beach. USACE (1972) mentions a 1962 placement of 180,000 cy. No record of this placement could be verified by a DNREC official who regards that reference as an error (personal communication, 1994).

Table 6. Beach Fill History at Broadkill Beach.

Built (yr/mo)	Volume (cy)	Limits (ft)	Unit volume (cy/ft)
1957	76,800	N 7+50 - S 7+50	51
1961	120,000	N19+00 - N12+00	52
		N 2+00 - S14+00	52
1973/11-12	118,000	N27+00 - S18+00	26
1975/2-6	295,000	S18+00 - S79+00	48
1976/8-9	59,700	N25+00 - N 4+00	28
1981/10-3('81)	127,700	N28+00 - S 1+00	44
1987/6-12	21,700	N17+00 - N 3+00	11
	30,900	S 8+00 - S21+00	11
1988/6-10	19,600	N24+00 - N16+00	22
	8,900	S25+00 - S31+00	15
1993	37,000	S25+00 - S34+00	41

The unit volumes reported in Table 6 are simple averages considering the total volume placed and the total length of placement. These data were supplied by DNREC. Figure 44 shows the fill limits (depicted as rectangular boxes), and the average unit volumes placed, with the boxes arranged chronologically. Figure 45 plots the cumulative unit beach fill volumes vs alongshore station location.

Figure 45 shows that the greatest amount of fill has been placed between Main Street and Louisiana Street, with a total unit volume in one area of approximately 180 cy/ft. No fill has been placed north of Louisiana Street although this area has certainly benefitted from the placements to the south, considering the variable directions of wave approach. Areas to the south of station S76+00 also received no direct placement of beach fill. The areas south of Main Street were affected primarily by the 1975 placement of 48 cy/ft with the exception of a small section between S25+00 and S34+00 which has received additional attention in the form of a modest fill in 1988 (15 cy/ft) and more substantially in 1993 (41 cy/ft).

Following the analysis of the coastal processes at Broadkill, the effects of beach fills can be assessed within the context of the available data on shoreline and beach profile change data. It is not possible to assess the effects from the 7 groins without design data.

IV-4. COASTAL PROCESSES AT BROADKILL BEACH

IV-4.A. Tides and Tidal Currents

The tides at Broadkill Beach are mixed, semi-diurnal with a mean tide range of 4.1 feet. Table 7 presents the mean tide statistics at nearby Breakwater Harbor.

Table 7. Tide Statistics at Breakwater Harbor

Stage	Elevation[1] (ft MLLW)	Elevation[2] (ft NGVD)
MHHW	4.7	3.0
MHW	4.3	2.6
MLW	0.2	-1.5
MLLW	0.0	-1.7

[1] Source: NOS Chart 12304
[2] NGVD is 1.69 ft above MLLW, Source: Harris (1981)

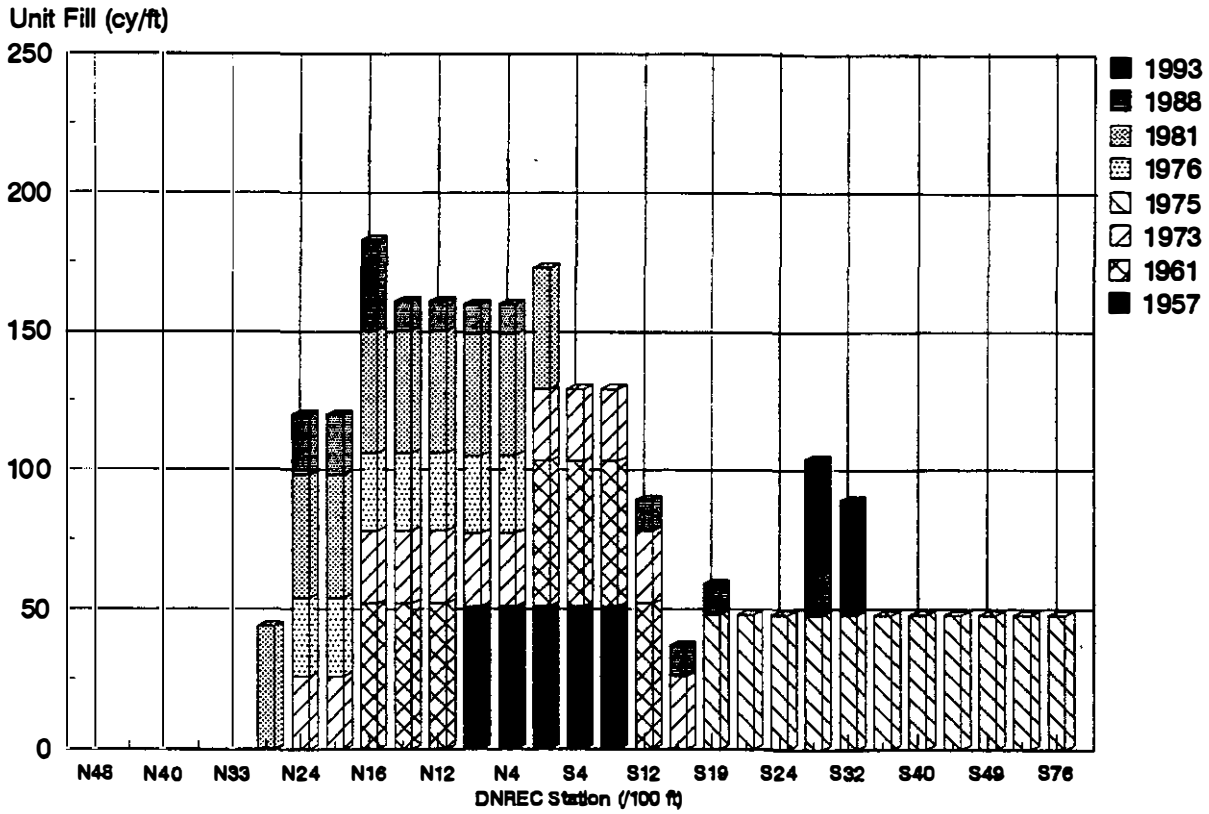

Figure 45 Cumulative Beach Fill Unit Volumes (cy/ft).
1987 fill data included in 1988 data.

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Tidal currents for stations in Delaware Bay were predicted over the course of an average tidal cycle using a hydrodynamic computer model (NOAA, 1987). The model results were verified with 14 months of field observations of tide elevations, currents, and environmental conditions. The tidal current results are presented as a series of vector plots at one hour intervals at locations throughout the bay. Some of these plots are presented in Section II (Figures 5, 6 and 7). The closest location to Broadkill Beach is approximately 3500 feet offshore. The results predict maximum flood and ebb tidal currents of approximately 0.6 ft/sec. USACE (1972) reports normal flood and ebb tides at Broadkill having velocities of 3 ft/sec. The average values from the NOAA hydrodynamic model are considered the best available estimates.

IV-4.B. Average Offshore Wave Conditions

Hindcast wave data for the Delaware Bay and entrance are available from several sources. The U.S. Army Waterways Experiment Station, Coastal Engineering Research Center has developed a set of hindcast data for stations in the Atlantic ocean including the Delaware Bay entrance area for the 1956-1975 period (Hubertz et al. 1993). Work is under way to extend these data from 1975 to the early 1990's (R. Hoban, personal communication, 1994).

A recent COE study (USACE, in preparation) includes a wave hindcast study for the Delaware Bay and vicinity for the period November 1987 to October 1993. These data were utilized in the present study for the classification of average offshore wave conditions. They were also used to develop longshore transport (Q_l) estimates, as input to the shoreline change model GENESIS, and for the storm-induced beach erosion analysis.

The Delaware Bay wave study uses a time step of 3 hours and produces one set of significant wave height (H_s), peak period (T_p), and peak mean direction (α) values per time step. The spectral energy is related to H_s according to $H_s=4\sqrt{E}$, where E is the spectral energy. T_p is the inverse of the frequency at which the peak energy occurs. The peak mean direction is an energy weighted mean of the directions associated with each of the discreet frequency bands in the spectrum.

Wave hindcast results are saved on a 5 minute grid spacing throughout Delaware Bay. Two stations for which data were applied in the present study are located at 38-55-00° latitude, 75-10-00° longitude and 38-50-00° latitude, 75-10-00° longitude. The first station is located approximately 7 miles from Broadkill Beach. The water depth at this hindcast station is approximately 60 feet (NGVD). The second station is located approximately 2 miles from Broadkill Beach with a water depth of approximately 15 feet.

The mean shoreline orientation of the shoreline including Broadkill Beach from Cape Henlopen to the Mispillion River is 319.8° Azimuth. This leaves Broadkill Beach open to waves

approaching from 319.8° to 139.8° Az. The time series was processed to determine the frequency distribution of wave characteristics for those waves traveling onshore. Offshore traveling waves occur 49.7% of the time annually. Only those waves traveling onshore were counted so the total frequency of wave events would sum to 100%.

Direction bands of 22.5° were chosen so that shore-normal would coincide with a mid-band direction. Wave period bands of 2 seconds (except for the lowest 0-3 second band), and wave height bands of 1.6 feet were chosen. Frequency distributions were prepared annually and seasonally. The results are presented in joint frequency tables in the Data Appendix. Figure 46 presents the average direction and period distribution of wave events for the 6 year hindcast period.

IV-4.C. Storm Wave Conditions and Extreme Estimates

Broadkill Beach is affected both by tropical (hurricanes) and extra-tropical (northeasters) storms. Both can cause severe beach erosion and damage to coastal structures. Hurricanes are associated with extreme low pressure systems and can result in large increases in water level. Coupled with a high tide condition and with waves superimposed on the flood profile, a hurricane can result in significant flooding and damage. Northeasters cause their damage principally through wave attack of the beach and adjacent structures. They can be as damaging or more damaging than hurricanes depending on their duration which can extend over several tidal cycles. During successive high tides, higher and steeper waves caused by the persistence of winds over a nearly unlimited fetch can cause extensive beach erosion. Since the 1950's the significant storms of record have been chronicled (USACE, 1966, 1977; Dalrymple, 1982; French, 1990, USACE, in preparation).

Table 8 presents a summary of some of the more severe storms in the last 40 years or so. The data are compiled from previous reports and the Delaware Bay hindcast study.

The extreme wave analysis conducted for CENAP in 1994 (USACE, in preparation) includes wave and water level calculations for 15 northeasters and 15 hurricanes. A probability analysis was conducted for the two sets of storm types and for the combined 30 storms. The Hs, Tp, Tm and the peak surge still water level (SWL) values for each storm were ranked and a regression analysis conducted for an assumed Gumbel type distribution. The results for the station located at 38-50-00 latitude, 75-10-00 longitude are presented in Table 9.

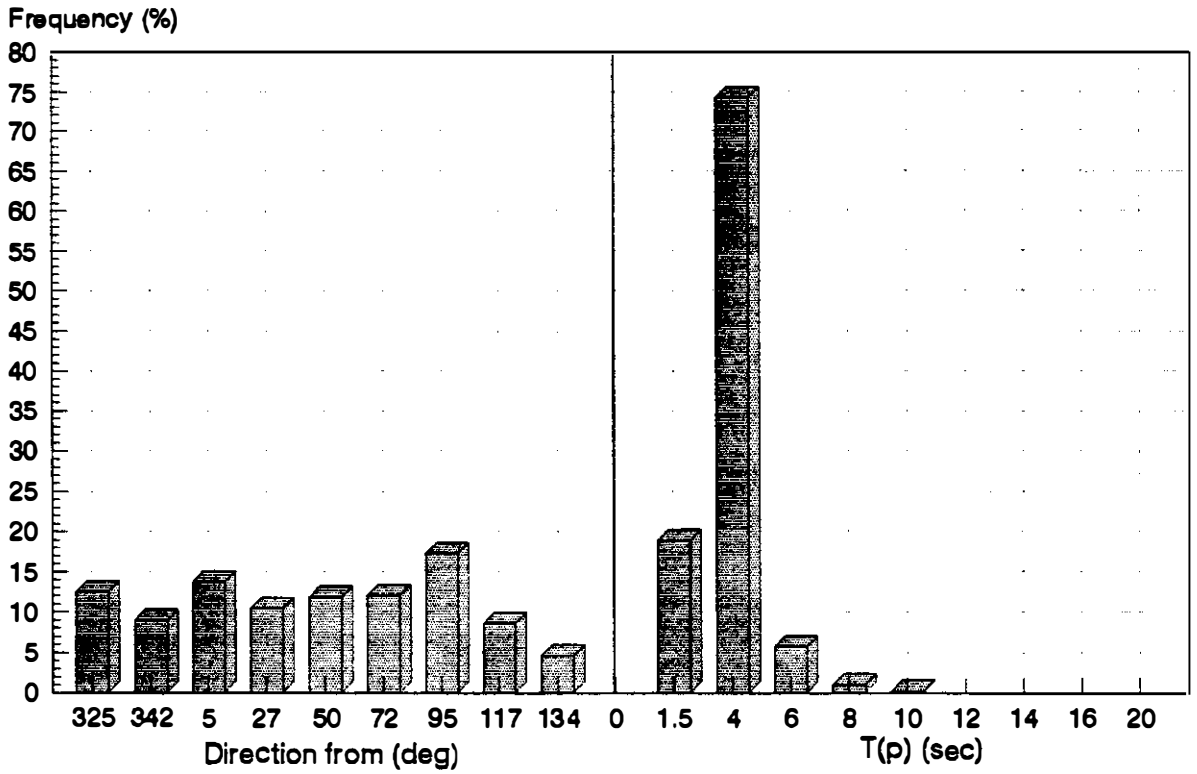

Figure 46 Annual Direction and Period Distribution from Delaware Bay Hindcast Wave Study. Directions are degree azimuth "from".

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Table 8. Summary of Significant Storms Affecting Broadkill Beach.

Date	Type	Significant features
Aug 1933	Hurricane	75 mph onshore winds; flooding; moved inland S of Broadkill; minor residential damage
Sep 1936	Hurricane	wind=46 mph; H=9.6 ft; TWL=4.0 ft
Sep 1938	Hurricane	passed 75 mi offshore at low tide; minor damage
Sep 1944	Hurricane	passed 50 mi E of DE coast; extensive damage
Nov 1950	Northeaster	strong E wind at high tide (7.2 MSL); loss of beach
Aug 1953	Hurricane	wind=37 mph; H=8.6 ft; TWL=7.6 ft
Nov 1953	Northeaster	60 mph winds; 5.4 MSL tide; minor damage
Aug 1954	Hurricane	wind=32 mph;H=5.3 ft;TWL=6.3 ft
Sep 1954	Hurricane	wind=32 mph;H=5.9 ft;TWL=5.3 ft
Jan 1956	Northeaster	high water levels and some beach loss
Aug 1958	Hurricane	wind=40mph;H=7.9 ft;TWL=5.3 ft
Sep 1960	Hurricane Donna	peak winds of 110 mph
Mar 1962	Northeaster	strong winds over 5 tide cycles; tide of 7.9 SLD; 20-30 ft wave heights; extensive beach damage; cut an inlet to sound at ~S42+50
Jan 1964	Northeaster	wind=43 mph;H=10.6 ft;TWL=7.3 ft
Oct 1965	Northeaster	significant beach erosion in central Broadkill
Jan 1966	Northeaster	wind=47 mph;H=8.9 ft;TWL=6.3 ft
Sep 1967	Hurricane	wind=43 mph;H=8.9 ft;TWL=6.3 ft
Dec 1973	Northeaster	wind=39 mph;H=7.9 ft;TWL=6.6 ft
Dec 1974	Northeaster	wind=38 mph;H=6.3 ft;TWL=6.9 ft SR 16 flooded in Broadkill
Aug 1976	Hurricane	wind=47 mph;H=7.3 ft;TWL=5.6 ft
Mar 1977	Northeaster	wind=46 mph;H=7.3 ft;TWL=3.0 ft
Oct 1977	Northeaster	wind=45 mph;H=6.6 ft;TWL=6.9 ft
Feb 1978	Northeaster	wind=43 mph;H=7.6 ft;TWL=5.6 ft
Oct 1980	Northeaster	wind=46 mph;H=8.6 ft;TWL=6.3 ft
Mar 1984	Northeaster	(NO DATA AVAILABLE)
Sep 1985	Hurricane	wind=46 mph;H=7.9 ft;TWL=7.3 ft
Oct 1991	Northeaster	wind=39 mph;H=7.3 ft;TWL=6.6 ft
Jan 1992	Northeaster	wind=53 mph;H=11.6 ft;TWL=6.9 ft
Dec 1992	Northeaster	wind=40 mph;H=9.6 ft;TWL=8.9 ft
Feb 1994	Northeaster	(NO DATA AVAILABLE)

Table 9. Extreme Wave and Water Level Estimates Broadkill Beach, Delaware				
Return Period (year)	Hs (ft)	Tp (sec)	Tm (sec)	SWL (ft) NGVD
5	7.9	9.2	5.6	6.2
10	8.9	10.3	6.5	7.5
20	9.8	11.2	7.8	8.0
50	11.2	12.2	9.7	9.5
100	12.1	12.7	11.5	10.5
200	13.1	14.0	13.0	11.2
500	15.4	16.0	16.0	12.4

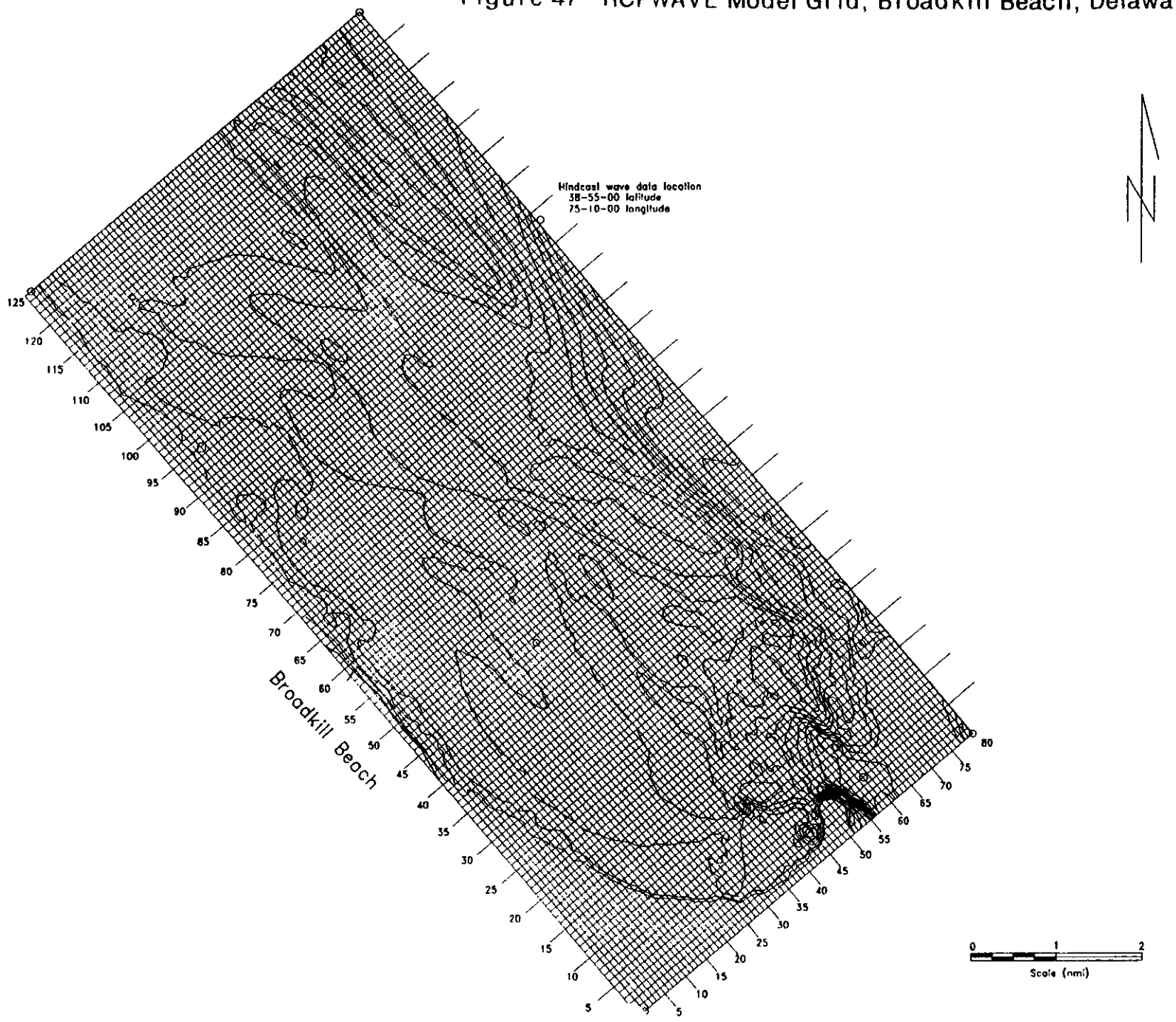
IV-4.D. Wave Transformation Model - RCPWAVE

To estimate the potential longshore transport rates, nearshore wave characteristics are required. The finite difference model, Regional Coastal Processes Wave Transformation Model (RCPWAVE), developed by CERC (Ebersole et al., 1986) was used to calculate nearshore wave characteristics as a function of deep water wave characteristics and the regional bathymetry.

The model grid limits were chosen based on the range of directions of wave approach, the location of the offshore wave data, regional bathymetry and shoreline orientations, and model capabilities. RCPWAVE uses rectangular grid elements which should be oriented so the smaller dimension is parallel with the offshore grid axis and the offshore axis is parallel to the dominant direction of wave approach. For Broadkill Beach where the directional distribution is fairly uniform (cf., Figure 46) the grid was oriented so the offshore axis was shore-perpendicular. Considering the bathymetric change and the model's element number limitations, a grid array of 125 x 80 was chosen for the alongshore and offshore directions, respectively. Grid element dimension are 543 ft x 382 ft in the alongshore and cross-shore directions, respectively.

The model grid for the study area is shown in Figure 47. The numbering along the grid axes is consistent with the numbering scheme used by RCPWAVE. The bathymetry was generated by an interpolation program using a subset of sounding data from the National Ocean Services Hydrographic Data Base. Only a subset of the available sounding data could be used due to the extremely high density of data locations. For the area within the grid boundaries there were over 47,800 soundings from the years 1970, 1971 and 1972 with 89% from 1971.

Figure 47 RCPWAVE Model Grid, Broadkill Beach, Delaware



A program was written to search the entire database on a element by element basis to find the two soundings nearest each element center and save these locations and soundings to a file. RCPWAVE uses depth data at each element center for calculating the transformation of wave characteristics. This file was then used in the interpolation program to determine representative water depths at each element center and to plot the bathymetric contours. The bathymetric contours plotted were verified using NOS nautical chart 12304.

RCPWAVE uses one set of H, T, and α values for each transformation. It is unrealistic and unnecessary to run RCPWAVE for every set of H, T, and α contained in the offshore time series. Rather, a finite set of condition combinations can be used which will adequately describe the range of offshore conditions for the purposes of estimate Q_l and shoreline modeling. Testing results from the GENESIS model (Gravens et al., 1991) show that direction bands of 22.5° and period bands of 2 seconds provide adequate resolution of an offshore time series for shoreline modeling purposes on open coasts. A unit wave height was chosen for each of the RCPWAVE runs to reduce the total number of runs required. In this way, output wave height values are actually $k_s k_r$ values and are multiplied by the input wave height to determine nearshore wave heights. Although there are some offshore shoals in the modeled area, the small frequency of wave heights justifies the decision. The mid-band direction and period combinations run in RCPWAVE are reported in Table 10. All model runs were conducted for MSL water depths.

RCPWAVE calculates output for each element in the grid. Besides $k_s k_r$, water depth, wave direction, breaker index, and wave phase function are reported at each . The breaker index indicates whether the wave has broken. Wave phase function is related to wave length and is described in greater detail in Ebersole et al. (1986).

The RCPWAVE elements along Broadkill Beach generally having water depths between 8-9 feet MSL. Although internal calculations are performed over the entire grid, only elements 1-40 along the cross-shore axis and elements 25-84 along the shore-parallel axis were output. The Broadkill Beach study area is located approximately within the RCPWAVE model elements 68-42. The results from each of the 25 input conditions were saved for calculating potential longshore transport rates and for the shoreline modeling. The results were verified for

Direction (° Az)	Period (sec)
325.4	3,6
342.4	3,6
4.8	3,6
27.4	3,6
49.8	3,6
72.4	3,6,8,10,12
94.8	3,6,8,10
117.4	3,6,8
134.2	3,6,8

Table 10. Input Wave Conditions for RCPWAVE.

physical sensibility and errors by inspecting output vector plots with a program written for that purpose.

The program displays the water depth, wave direction, and relative wave height at each element center. Figure 48 shows the RCPWAVE results for an incident wave angle of 324.6° Az and period of 6 seconds. The cross-shore element numbers shown are 1-40 and the alongshore elements shown are 40-69. The asterisks plotted indicate land elements. The direction of the arrows correspond to the output wave direction and the arrow length is proportional to the wave height. When viewed on a color monitor the arrows are colored according to their depth to reveal the bathymetric contours and aid in error checking of the input depth file.

A complete set of the graphical output is presented in the Data Appendix. RCPWAVE's internal calculations proceed best when the input wave direction is shore-perpendicular. For highly oblique cases the computations become unstable. Several instances of this instability are indicated in the output by very large wave heights. For these cases, the nearshore results from the closest T,α input combination was substituted for subsequent longshore transport rate calculations.

IV-4.E. Longshore Transport Rates

Estimates of the gross and net potential transport rates are one of the most important variables in a coastal processes analysis. Appropriate estimates of transport rates and alongshore patterns can determine whether a coastal engineering project involving a beach fill, groin field, or detached breakwater will be successful.

The gross transport rate is the sum of both upcoast and downcoast rates for the shoreline of interest. The net transport rate is the difference between the upcoast and downcoast rates.

An important consideration in the calculation of Q_l is the difference between potential transport rate and actual transport rate. The potential rate is that which is calculated based on recognized empirical relationships developed by comparing measured transport rates with a measure of longshore energy for areas where there is no deficit of sand supply. In this case, actual and potential rates will be equal. However, when the upcoast sediment supply is deficient due to a lack of sand on the beach and in the surf zone, or there is some sort of sediment sink, actual transport rates will be less than potential rates. The amount by which actual rates lag potential rates for such a case is usually difficult to estimate.

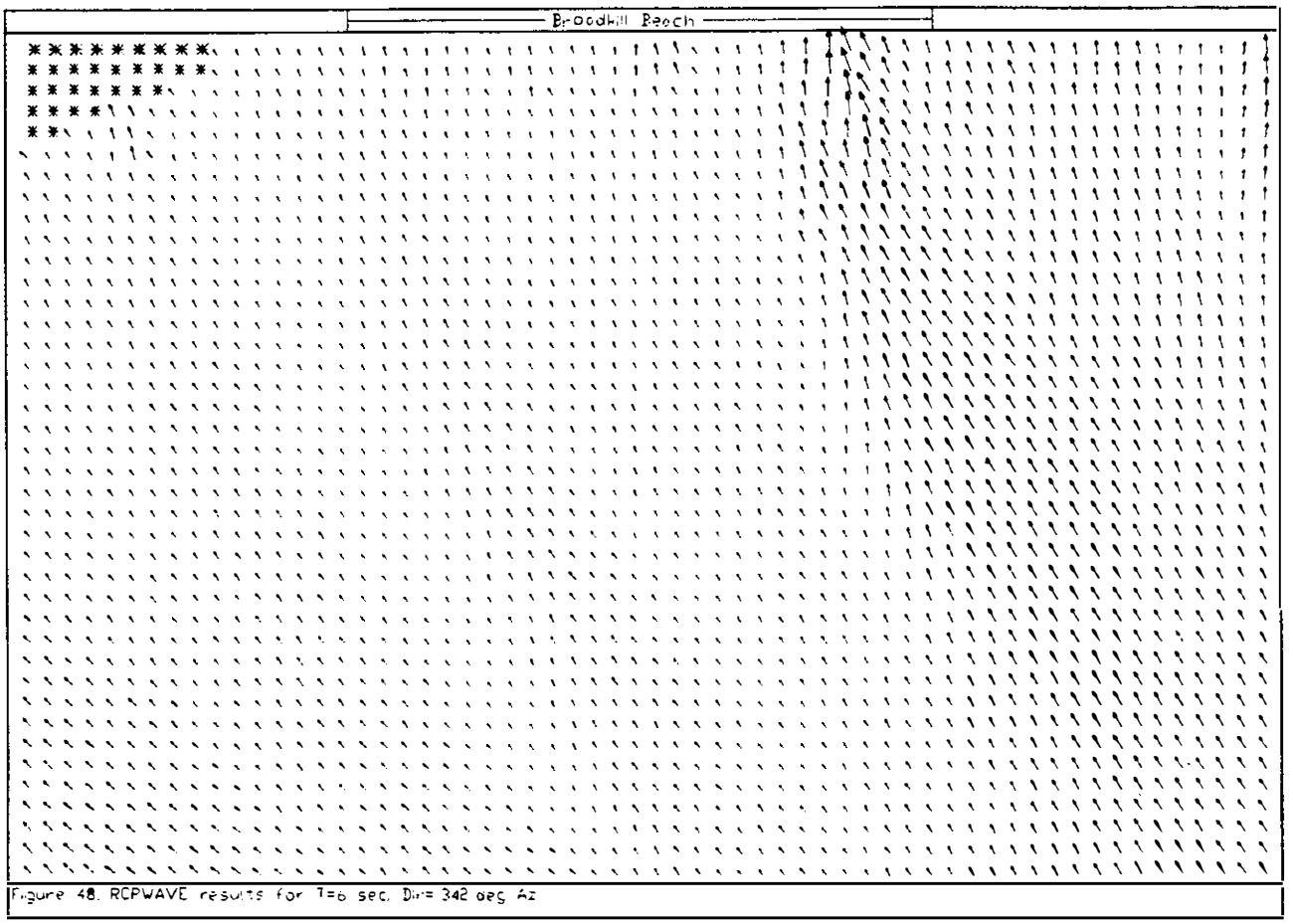


Figure 48 Sample Plot of RCPWAVE Result

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Calculation of Potential Transport Rates

Empirical relationships between measures of the longshore wave energy and transport rates have been developed from measured longshore transport rates (directly and indirectly) and nearshore wave processes. The empirical equation used in this study relates the wave-energy flux factor (P_{ls}) for incipient breaking conditions to the transport rate. The basis for the relationship is described in the Shore Protection Manual (SPM) (USACE, 1984). A program included as part of the Shoreline Modeling System (SMS) (Gravens, 1992) was used to calculate Q_l in the nearshore elements off Broadkill Beach in 8-9 feet of water (MSL). The input required were an offshore wave time series, nearshore wave conditions (RCPWAVE), and the water depths for the relevant RCPWAVE elements. The files required can subsequently be used in the GENESIS model.

The total 6 year hindcast period was used. The program determines which set of nearshore transformation conditions to use for the particular offshore event using "keyed" data in the nearshore wave file. Waves are then transformed to incipient breaking conditions using linear wave theory. The breaking conditions are used to calculate P_{ls}, Q_l, and direction for each time step in the time series. The procedure is repeated at every time step, the results are summed and reported on an annual basis in an output file.

The empirical equation used to determine Q_l is:

$$Q_l = 7500\ (\gamma/16)(H_s^2 C_g)_b \sin(2\alpha_b) \qquad (1)$$

where: Q_l = longshore transport rate (cy/yr)
γ = unit weight of water (64lbs/cf)
C_g = wave group celerity (fps)
α_b = angle between wave crest and shoreline

The "b" subscript refers to breaking conditions. Numerous investigators have provided criticism and modification to the above equation to account for the effects of longshore gradient of wave height, sediment characteristics, and bed slope (e.g.,Bailard, 1984; Hanson and Kraus, 1989; Kamphuis and Sayao, 1982), but without changing the essence of the relationship.

The gross and net transport rates are presented in Figure 49. A net transport budget can be performed on each element to indicate whether erosion or accretion is predicted for that area. Such a budget for RCPWAVE element 65 in Broadkill near LRP#25 indicates a potential transport rate of approximately 51,000 cy/yr to the northwest. The element width is 543 ft, yielding an average unit erosion rate of 94 cy/ft/yr. An examination of the unit volume change data for this area reveals that such a value is an entire order of magnitude too high.

The transport rates and transport rate gradients at the southeast portion of Broadkill are completely unrealistic. For example, at RCPWAVE element 46, the net Q_l gradient predicts accretion of over 1800 cy/ft/yr.

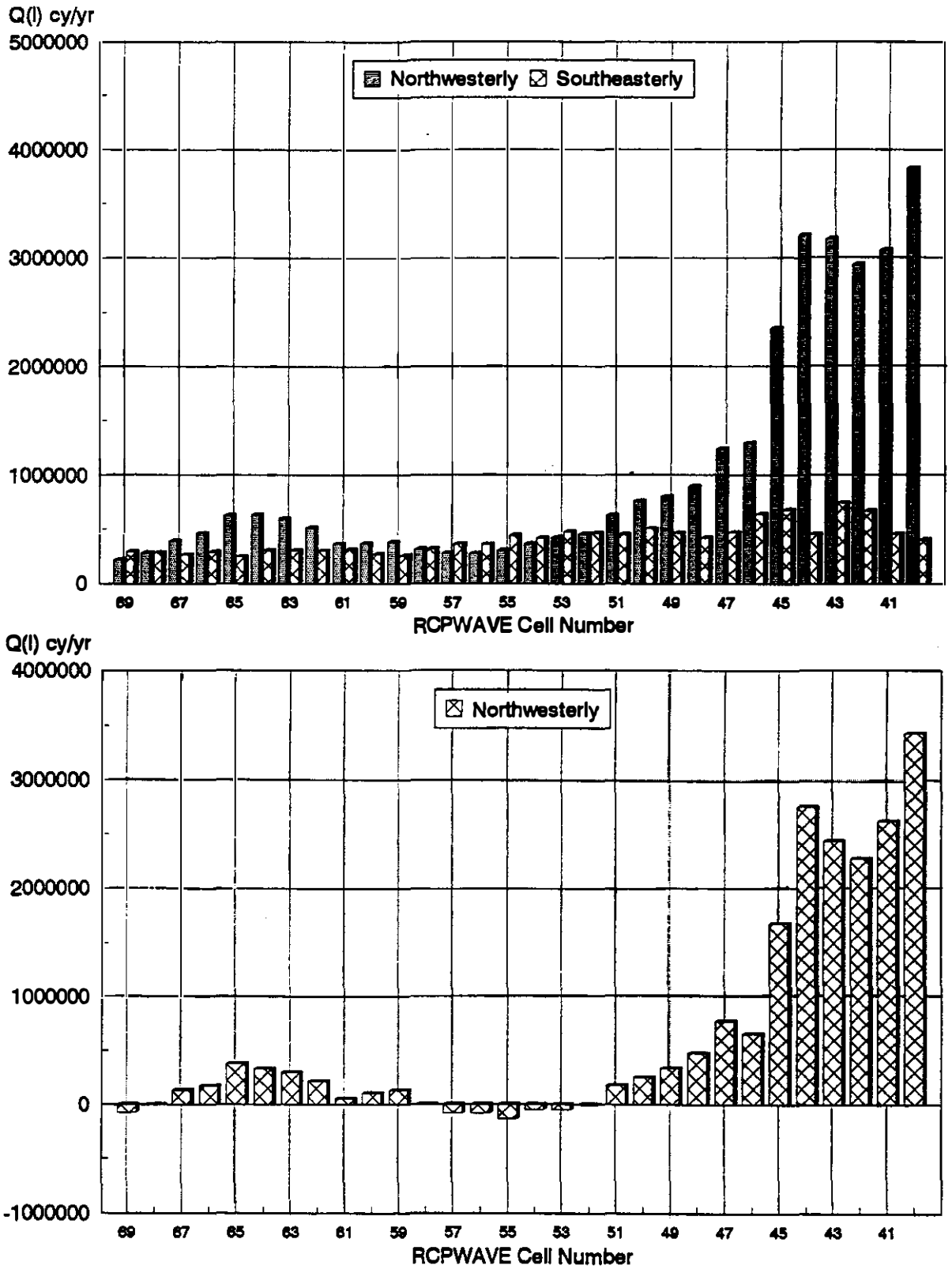


Figure 49 Gross (top) and Net (bottom) Longshore Transport Rates

There are several reasons why the predicted potential Q_l rates might be so high:

- The rates are potential values and in general are higher than actual rates.

- The model is not capable of directly accounting for the sheltering effect of the breakwaters at Breakwater Harbor. The difference in nearshore wave heights off Broadkill for incident waves from the Atlantic should be significant. The modeling of this sheltering was attempted by mixing the time series data for a station at 38-50 lat, 75-10 long, along with the data at the bayward edge of the grid. However, the sheltering may still have been only partially accounted for considering the relationship of the nearshore station to the breakwaters for waves approaching from the E-NE.

- The use of a unit wave height as input to RCPWAVE runs resulted in unnaturally high wave heights since actual wave heights would have broken. This should not effect the results significantly since in the Q_l calculating algorithm wave heights are restricted to be no larger than 78% of the water depth.

- The input wave period and direction bands recommended in the SMS guidance are too wide for use at Broadkill. For example, the program which calculates Q_l recognizes as one band period values from 0 to 5 seconds. At Broadkill nearly 75% of waves approaching shore have 3-5 second periods. When the 0-3 second period waves are included, over 90% of shore-directed waves are accounted for. With the period distribution being so tight, more low period RCPWAVE runs should be used to represent the distribution.

- RCPWAVE is an open coast model and Broadkill Beach is not an open coast location. Open coast sites typically experience a much wider range of incident period waves and as a result the period band widths recommended in the SMS guidance are acceptable.

- The orientation of the Broadkill shoreline relative to the incident waves originating in the Atlantic may result in unreasonable results which are manifest in the Q_l results. Ideally, the majority of incident wave angles should not be highly oblique. At Broadkill, the range of incident wave directions might best be handled with two or even three grids.

IV-4.F. Cross-Shore Transport and Storm-Induced Beach Erosion

In addition to moving alongshore, sediment is also moved under the action of waves and currents across the beach profile. Cross-shore transport occurs as a result of changing water level and wave

energy flux in the surf zone. The concept of an equilibrium beach profile has been used to describe beach profile changes from cross-shore transport resulting from a relative change in surf zone energy. Research has shown that beach profile slope is a function of sediment characteristics, wave energy, and wave steepness (Bascom, 1951; Bruun, 1954; Dean, 1977). When one of these environmental characteristics is changed the existing beach profile is no longer in dynamic equilibrium and cross-shore transport occurs until a new equilibrium profile is established.

Significant volumes of cross-shore transport can occur over both short (hours) and long (months) time periods. Storms can produce significant cross-shore transport which is manifest as erosion of the upper beach profile. Principally, the erosion is caused by elevated water levels allowing wave steepening and breaking on the normally above-water portion of the profile. The longer time period changes result from average seasonal variation in nearshore wave climate. These changes are on average balanced by the following season's reversal in cross-shore transport so that over the long-term this seasonal cross-shore transport has a negligible net value.

Under relatively mild storm conditions sediment eroded from the upper beach profile is later recovered under the action of longer low-steepness waves. However for severe storms, eroded upper profile sediments can be deposited in water depths at which prevailing bottom wave-generated stresses are too low to transport the sediments back up the profile. In this case the eroded sediments are permanently lost. For Broadkill Beach-type coasts characterized by low closure depths and nearly flat bottom slopes beyond the beach face toe having normal low-energy nearshore wave climates, this type of storm-induced erosion can be the dominant erosion mechanism.

Storm-induced cross-shore erosion is analyzed using the computer model SBEACH (Larson and Kraus, 1989; Larson, Kraus, and Byrnes, 1990) and presented in Section IV-6.

IV-4.G. Analysis of Shoreline and Profile History

Available historic data on shoreline and beach profile change has been presented in earlier sections. However, these data were unadjusted for the effects of beach fills and groins constructed at Broadkill. By considering the magnitude of these effects and their chronological occurrence in the known sequence of shoreline and profile changes, realistic values of background change rates might be determined. These rates can then be used to estimate likely shoreline positions for planning purposes.

Besides the effects of beach fills and groins on the apparent profile changes, the available records of storm effects indicate that they can significantly affect the beach profile in a matter of days or even hours. These extreme events can significantly alter the average rates of profile change, depending on the period of

record considered. Since storms are a natural part of coastal processes, albeit an unpredictable one, their effects should be included in an assessment of background profile change rates. To obtain the most realistic estimates possible, the longest possible period of record should be used.

Profile change indicates that historically the central portion of Broadkill Beach has experienced the greatest amount of erosion. The placement of the three groins in 1950 at stations S7+40, S2+65, and N2+00 stabilized the beach in this area (USACE, 1972). Erosion to the north prompted the construction of two more in 1954 at stations N6+70 and N11+70. By the early 1970's these groins were still in good condition and the erosion problem had proceeded again to the north. The fact that the areas immediately north of the groin group were eroding suggests that net longshore transport was directed to the north and the groins were benefitting the area they occupied at the expense of downcoast areas to the north. This provides support to the general pattern of Q_l, predicted from the RCPWAVE results indicating net Q_l to the northwest over most of the beach. However, the 1950's were a period of frequent storm activity (cf., Table 8) and the erosion north of the groin group may have been storm-induced with the area occupied by groin fields stabilized by the groins.

The analyses of shoreline linear erosion rate and volume change rates for Broadkill Beach is done by analyzing all available beach profiles. Table 11 summarizes the adjusted background shoreline and volumetric change rates for areas represented by selected survey profiles. These rates are recommended for planning purposes. Areas near LRP#24A, N33+00 and LRP#28 are conservatively considered stable rather than accreting. In addition to the adjusted rates, the apparent rates for each profile were also determined using the slopes of the linear regression lines from the shoreline positions in Figures 30-37. The rates shown in Table 11 are the average for the transects near the profile. The estimated annual unit fill rates were calculated from dividing the total unit fill by the number of years between the first fill and 1993. For areas where only a portion of a year's fill extended into the area, it's unit volume was averaged over the entire area. The adjusted rates in Table 11 are determined as follows:

LRP#24A

LRP#24A represents the northernmost area in the Broadkill Beach study area. Figure 45 shows that it is outside the area directly affected by beach fills. However, Figures 30 and 40 show that it has apparently benefitted significantly from the fills placed in the adjacent areas to the southeast. The first fill was placed in area adjacent to LRP#25 in 1973.

The shoreline positions plotted in Figure 30 show a strong trend of shoreline accretion with the exception of minor recession for the 1964 to 1971 period and a few other short periods.

TABLE 11. BROADKILL BEACH, BACKGROUND EROSION RATES (1954-1993)				
SURVEY PROFILE	APPARENT LINEAR (ft/yr)	ANNUAL FILL (cy/ft)	ADJUSTED VOLUMETRIC (cy/ft/yr)	ADJUSTED LINEAR (ft/yr)
LRP#24A	8.0	0	stable	stable
N33+00	5.3	0	stable	stable
LRP#25	-1.5	4.6	-4.0	- 9.6
LRP#25A	1.7	3.9	-5.4	-10.8
LRP#25B	2.2	2.4	-1.9	- 3.8
LRP#26	-2.0	2.3	-4.9	- 9.8
LRP#27	-3.9	2.8	-6.5	-13.0
LRP#27A	-2.0	2.7	-7.8	-15.6
LRP#28	1.7	0.4	stable	stable

The trend is approximately the same in the lower plot for profile-based shoreline positions. The available unit volumes plotted in Figure 40 also show accretion. Linear regression yields a range of +7.0 to +8.7 ft/yr for the 1954 to 1993 period with an average accretion rate of 8.0 ft/yr. This rate is about the same as prior to 1973 in which the first fill was placed.

The unit volume data yield an accretion rate of 3.4 cy/ft/yr for transect N43+00 for the 1976 to 1990 period, which is similar to the value obtained using the correlation relationship between the shoreline position change and unit volume change (1 cy/ft/yr = 2 ft/yr MHW). For planning purposes, a stable condition is assumed for LRP#24A.

Ironically, the northern section of LRP#24A eroded over 400 feet between 1882 and 1943 indicating this area may be eroding without the benefit of updrift supply.

N33+00 and LRP#25

The profile changes in this area are characterized by alternating periods of accretion and erosion (Figures 31 and 40) and likely reflect the placement of beach fill there. N33+00 are in areas where fill was not placed directly. The approximately uniform shoreline change trends indicate that the effects of the fill were approximately evenly distributed throughout the area.

The first beach fill placed in LRP#25 was in 1973. The effect of this fill is seen in the lower plot of Figure 31. Fill was also placed in 1976, 1981, and 1988 (cf. Figures 44 and 45). Since 1973, the shoreline and unit volume change data (Figure 31 and 40) indicate generally accreting or stable trends. The exception is erosion from 1979 to 1981 which may be the result of a storm in Oct 1980. The only transect having relatively long term data unaffected by beach fills is N21+00 (lower plot Figures 31 and 40). The change rates there for 1954 to 1973 from linear regression for LRP#25 are -9.6 ft/yr and -4.0 cy/ft/yr.

The shoreline trend for 1954 to 1964 is stable in the upper plot of Figure 31 and eroding in the lower plot. The data in the upper plot are shoreline position data while the lower plot data are from survey profiles. The profile data are considered more reliable since they are referenced to horizontal and vertical control points.

LRP#25A

As shown in Figure 45, area adjacent to LRP#25A has received the largest amount of beach fill at Broadkill. There are no long periods of shoreline or profile change data unaffected by beach fills. The longest intra-fill period is 1961 to 1973. The volumetric erosion rate from 1954 to 1973 was steady and the shoreline erosion rate was greater for the period before 1954.

The data from transect N21+00 are applied for LRP#25A because they provide a superior period of record as compared to the other transect's data.

There is a distinct difference in the trends before and after 1973. The effects of the 1957 and 1961 beach fills were apparently offset by the effects of the severe Mar 1962 northeaster. If it is assumed that the accretional effects of the beach fills from 1954 to 1973 were offset by the storms of that period then the 1954 to 1973 shoreline and profile change data can be taken as representative background rates (-9.6 ft/yr; -4.0 cy/ft/yr).

A comparable volumetric erosion rate can be obtained by attributing the stability of shoreline position and unit volumes since 1973 to the beach fills placed after that date. The 109 cy/ft placed over 20 years equates to 5.4 cy/ft/yr of erosion. This is probably a more representative erosion value for area near LRP#25A since it reflects the trends for all the transects near this profile. Storm-induced erosion is automatically included in this estimate. The equivalent shoreline change rate is -10.8 ft/yr, based on the correlation shown in Figure 38.

LRP#25B

The area near LRP#25B has experienced a non-uniform placement of beach fills (Figures 44 and 45). There are no profile data to

measure shoreline positions prior to 1964. Analysis of shoreline changes for LRP#24A and LRP#25 have shown that digitized shoreline position data for 1954 to 1964 may be unreliable when compared to profile-based changes.

The apparent erosion problem at LRP#25B is less severe than at LRP#25A. There has been a significantly lower amount of beach fill placed in LRP#25B than in LRP#25A (cf., Figure 45) and yet LRP#25B trends show accretion where the higher fill rates at LRP#25A yielded only stability. As shown in Figure 33 (top plot), for the period 1973 to 1979, transects S14+00 and S19+00 show net erosion while all other transects in the Figure 33 show strong accretional trends.

The profile-based shoreline and unit volume change data are used to estimate representative change rates for LRP#25B. The erosion from the late 1980's to the early 1990's is probably due to the effects of storms in the absence of any beach fill projects. These effects are included by performing linear regression for the period 1973 to 1993 (1964 to 1993 for S6+00) on transects S6+00 and S14+00. The other transect's shoreline change trends serve as validation for the two used in the calculation. The weighted average of beach fills placed in LRP#25B is subtracted from the regression-based change rates (accretional) to determine the background change rates.

The shoreline and unit volume accretion rates for S6+00 are approximately twice those of S14+00. The average values are +3.3 ft/yr and +1.6 cy/ft/yr. The weighted average of beach fills placed from 1964 to 1993 is 3.5 cy/ft/yr. Subtracting the average unit fill rate from the accretion rate yields a volumetric change rate of -1.9 cy/ft/yr. Using the results of the correlation analysis between shoreline and volumetric profile change yields an equivalent shoreline change rate of -3.8 ft/yr.

LRP#26, LRP#27 and LRP#27A

Examination of the shoreline position data for the area near these three profiles indicate very similar trends, with the exception of transects S22+00 and S24+00 near LRP#26 (Figures 34 to 36). The trend is of uniform, consistent erosion for the 1954 to 1971 period. The first beach fill placed in this area occurred in 1975 and extended from the southern portion of LRP#25B to the northern portion of LRP#28 (Figure 44). With the exception of a small local fill near LRP#26 in 1988 (15 cy/ft) and a larger one in 1993 (41 cy/ft), the 1975 fill is the only fill affecting this area. The available evidence on longshore transport suggests that net Q_l is to the north. It follows that previous fills to the north probably did not affect this area significantly.

There are no profile data available (with the exception of S22+00) prior to 1973. As a result, available volumetric trend data is affected by the 1975 beach fill. It is recommended that the

representative background change rates for these profiles be calculated from the 1954-71 shoreline position data and that volumetric values be established using the derived relationship 1 cy/ft/yr = 2 ft/yr MHW.

For areas adjacent to LRP#26, shoreline change rates of -9.2 and -10.5 ft/yr are derived for S28+00 and S32+00 for the period 1954-71. This yields an average rate of -9.8 ft/yr for the period. The equivalent volumetric change rate is -4.9 cy/ft/yr.

The shoreline change rates for areas near LRP#27 range from -10.1 ft/yr at transect S40+00 to -16.2 ft/yr at transect S60+00, with an average of -13.0 ft/yr. The equivalent volumetric rate is -6.5 cy/ft/yr.

For areas near LRP#27A, the shoreline change trends for 1954 to 1971 ranged from -15.2 ft/yr to -16.4 ft/yr with an average of -15.6 ft/yr. The equivalent volumetric rate is -7.8 cy/ft/yr.

LRP#28

LRP#28 is outside the DNREC survey. Five transects at a spacing of 400 feet were established in order to compare shoreline positions from the digitized data (Figure 29). The data are plotted in Figure 37.

Only the northern portion of the area represented by LRP#28 has experienced direct fill placement. It has however, apparently benefitted from the 1975 fill to the north as indicated by the general trend of shoreline accretion for the 1975-1989 period. The data for the 1954 to 1971 period should be unaffected by beach fill activity. The relatively large recession and accretion at transects C8-1 and C8-2, respectively, for the 1954 to 1964 period is the result of the erosion of the spit that had existed at the south side of the jetty at the old Broadkill Inlet. Moving to the south, transects C8-3 to C8-5 should be relatively unaffected by the jetty for that period. Considering these transects, the 1954 to 1971 pre-fill period shows relative stability of the shoreline position. Therefore, the background condition for LRP#28 can be considered stable. This stability should not be assumed to extend to areas south of LRP#28, as the historic shoreline change map prepared by French (1990) shows accelerating erosion immediately south of LRP#28.

IV-5. EXISTING CONDITIONS

A site visit was conducted by WRCE personnel in mid-April 1994. Spot measurements of beach width were made to compare with the most recent detailed survey data of April 1993.

A series of northeasters in the winter of 1993 resulted in an estimated 25-35 feet of dune erosion in the vicinity of State Route 16 (as per discussion with beachfront property owner). Recent scarping was evident as shown in the photographs in Appendix C. A comparison of the dune widths with those indicated on a topographic map of the April 1993 condition confirmed this estimate.

At the south end of the study area, no scarping was observed. Significant scarping was found again beginning north of LRP#27A. A detailed profile survey is recommended to update the site conditions and provide up-to-date baseline information for detailed project design.

Spot observations during the site visit indicated that the berm sand are well-graded, fairly clean medium sands. There are some grain size distribution (GSD) statistics reported in USACE (1972) for beach samples taken along the LRP survey lines. The Delaware Department of Transportation sampled and performed GSD analyses on surface samples taken along LRP#25 and LRP#27 in Oct 1993. Samples were collected at various cross-shore locations from the dune base to the -10 foot (NGVD) contour. Table 12 presents the d_{50} values for the sampled locations. The anomalous values for the MHW and MLW positions at transect LRP#25 may be due to a deposit of shell hash or other coarse relic material from previous beach fills. These two values should be neglected since the spatial extent they represent is unknown. Furthermore, grain size statistics from beach samples should not be considered true "native" samples due to the beach fill history of the area.

Location	LRP#25 (N21+00) (mm)	LRP#27 (S49+00) (mm)
dune base	0.52	0.40
mid-beach	0.37	0.62
MHW	1.80	0.27
MSL	0.80	0.37
MLW	7.80	1.00
-5' NGVD	0.20	0.18
-10' NGVD	0.90	(NO DATA)

Table 12. Median Grain Size for 1993 Beach Samples

IV-6. STORM-INDUCED EROSION AND INUNDATION POTENTIAL

IV-6.A. Storm-Induced Erosion

Based on examination of the shoreline change history over the last 40 years at Broadkill Beach, it is likely that shoreline and profile erosion is largely storm-related. From Table 8, there were 18 northeasters and 8 hurricanes between 1950 and 1994 which affected area shorelines. The average frequency for this period is one northeaster every 2.5 years and one hurricane every 5.5 years, or an average of one storm every 2 years.

Two approaches can be taken to estimate storm-induced beach erosion: the "design-storm" and the "storm-ensemble" approach. For the storm-ensemble approach, erosion rates are calculated from a large number of historical storms and then ranked statistically to yield an erosion frequency curve. In the design-storm approach, the modeled storm is either a hypothetical or historical event that produces a specific storm surge hydrograph and wave condition of the desired frequency. The design-storm approach is used to analyze storm induced erosion at Broadkill Beach. A comparison of the two approaches is presented in Appendix E.

The Storm Induced Beach Change Model (SBEACH) (Rosati et. al., 1992) was used to estimate storm-induced erosion at Broadkill Beach. SBEACH is a geomorphic-based model utilizing large wave tank profile change data and high quality field data. Breaking waves and a variable water level are the major driving forces in SBEACH that produce sediment transport and beach profile changes. It is a two-dimensional model in that only cross-shore processes are considered. SBEACH was designed to predict and analyze short-term, storm-induced erosion.

The model consists of three calculation modules that are executed consecutively at each time step in a simulation. The modules calculate wave height as a function of water depth and profile slope, cross-shore transport, and profile change. Wave heights are used to calculate transport rate which is in turn used to calculate profile change. At the end of a time step, the revised profile is used as input for the next time step. Wave height, period, and water level are specified at each time step. The process continues until the simulation period is complete.

A fundamental assumption of the model is that breaking waves are the major cause of storm-induced profile change, and thus the majority of cross-shore transport occurs in the surf zone. The volume of sand transported is determined by the wave energy dissipation per unit of water volume in the surf zone and by empirical functional relationships derived from large wave tank data.

Due to the empirical foundation of SBEACH and the natural variability that occurs alongshore during storms, the model should be tested and calibrated. Pre- and post-storm beach profiles

together with wave and water level time series data for the storm are required to calibrate the model. Since these data are not available at Broadkill, the calibration parameters were first adopted from the nearby Dewey Beach study.

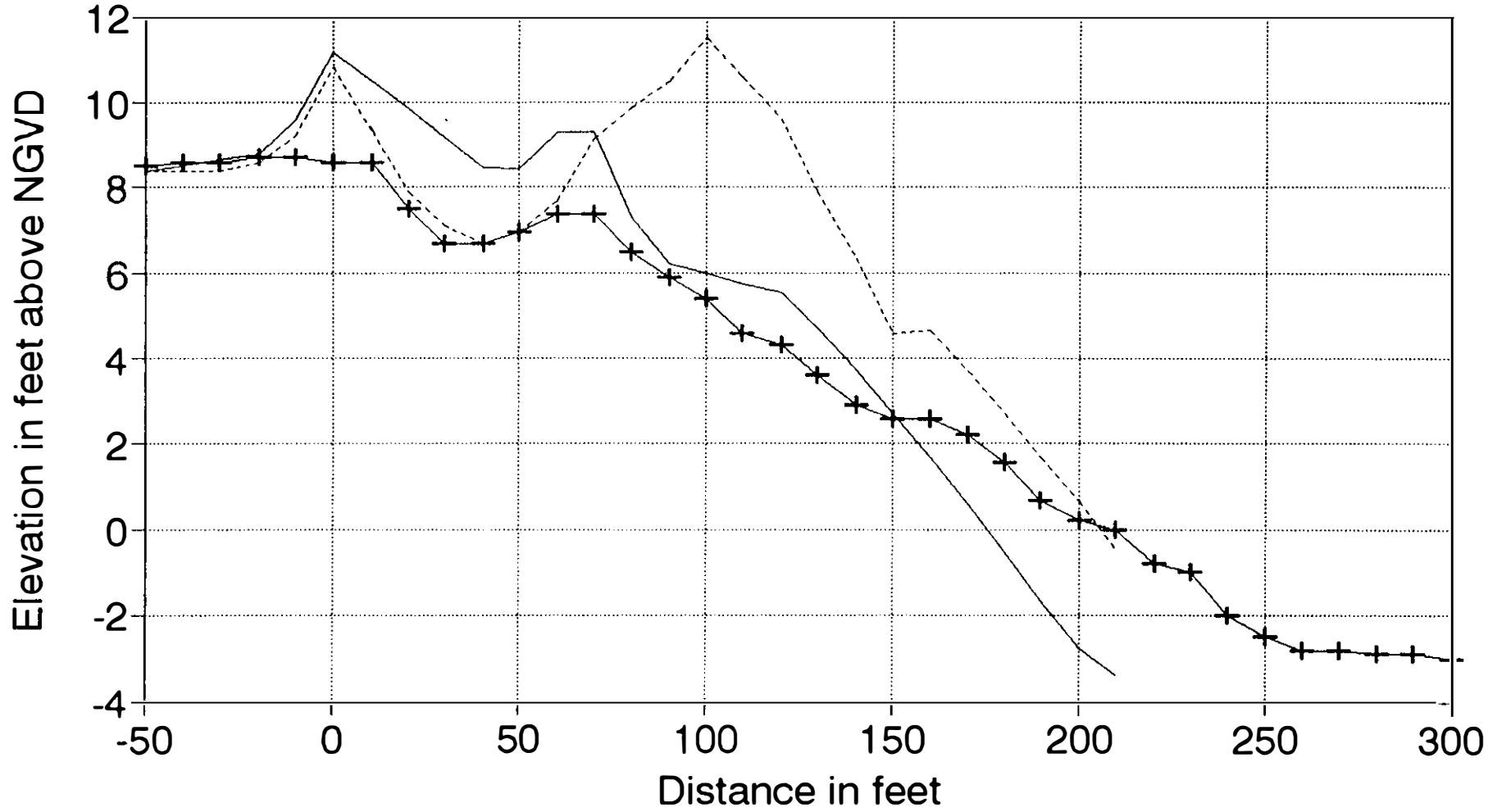
From 1991 to 1992, three northeasters attacked the study area (January 1991, January 1992, and December 1992). Beach profiles were surveyed two months prior to the January 1991 storm and seven months after the December 1992 storm. These profiles along with the computer simulated wave and water stage conditions for these storms were used to test the SBEACH model. This is not for model calibration or verification but to test the SBEACH model using available information. In order to calibrate the model, it is necessary to have accurate field wave and water level data, and pre- and post-storm beach profile surveys.

The November 1990 beach profile at LRP#25B was used as the initial profile. The stage hydrograph and wave conditions for the January 1991 storm simulated in the Delaware Bay wave and surge study were used for model input. The eroded profile was then used as the initial profile for the January 1992 northeaster. The wave and stage time series for that storm were then used to compute a second eroded profile. The procedure was then repeated for the December 1992 storm. Figure 50 shows a comparison between the initial measured profile (November 1990), the calculated profile, and the final measured profile (July 1993).

Since the SBEACH simulation indicated that the rear dune was eroded as the result of the January 1991 storm, subsequent model testing was done only for the 1991 storm. During the process, it was determined that a smaller value of the Eps parameter should be used to produce a profile similar to the July 1993 profiles. Also, reducing the profiles' landward extent behind the rear dune resulted in less erosion of the second dune and produced a receded dune profile more like the measured final profile. Still, SBEACH calculated more erosion on the dune crest and flatter foreshore slopes as compared to the measured profiles. This could be the result of input wave and water level conditions. Considering that the time series of waves and water levels used were hindcast and not field data, SBEACH still can be considered a useful tool for evaluating storm-induced erosion at Broadkill Beach and formulating project alternatives.

Twelve beach profiles were surveyed at Broadkill Beach in September 1993. These profiles are used as the project's base condition. For erosion and wave inundation analyses, a project design line was chosen as shown in Figure 51. All profile distances were adjusted relative to the design line. Nine profiles were selected for SBEACH modeling. Table 13 presents the transects selected, dune crest positions relative to the design line, and the dune crest elevations. Tables 14-22 present the SBEACH results for select return periods. Figures 51-59 present the initial beach profiles and the 100-year eroded profiles. Eroded beach profiles for 500-, 200-, 50-, 20-, 10- and 5-yr storms are presented in Appendix F.

Figure 50. Erosion Model Results
Broadkill Beach, Delaware



11/90 7/93 SBEACH

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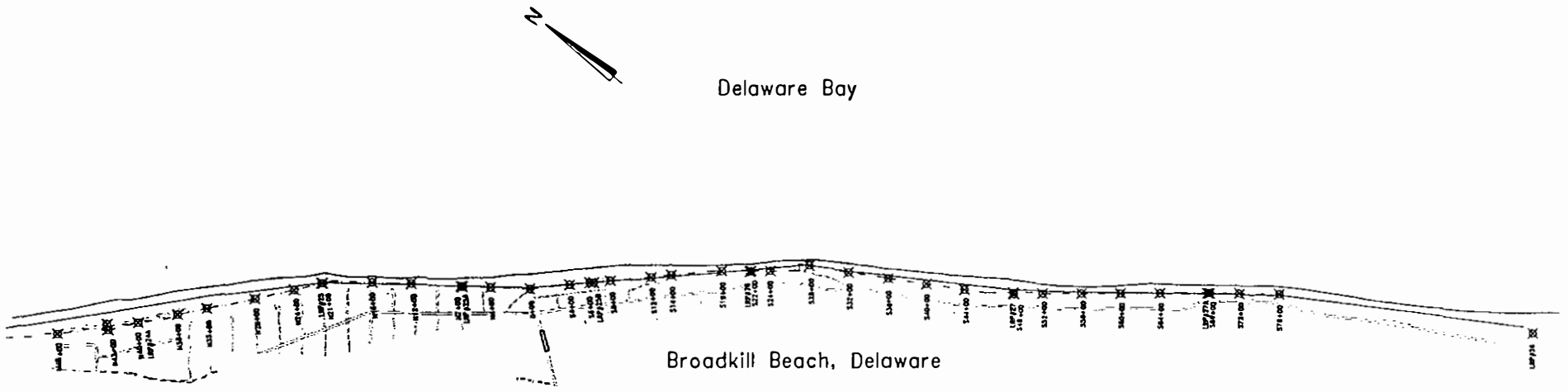
Figure 51 Location Map – Project Design Line



Scale 1" = 1000'

Project Design Line

Survey Baseline

D:\ACADWIN\008\FIG51.DWG

TABLE 13
SITE CONDITION FOR BROADKILL BEACH

PROFILE STATION	DESIGN LINE TO CREST (FT)	DUNE CREST (FT, NGVD)
LRP#24A	0	12
N33+00	90	12
LRP#25	- 3	12
LRP#25A	0	13
LRP#25B	- 4	11
LRP#26	10	11
LRP#27	25	13
LRP#27A	6	15
LRP#28	20	12

- bayward + landward

TABLE 14
EROSION MODEL RESULTS
WITHOUT PROJECT
LRP#24A

RETURN INTERVAL	DUNE CREST EROSION POSITION (feet from design line)	VOLUME ERODED ABOVE NGVD (cu yd/ft)
Original Dune Crest Location = 0 feet		
500	14	17.1
200	14	15.5
100	10	14.4
50	10	12.8
20	0	12.1
10	0	10.8
5	0	8.8

TABLE 15
EROSION MODEL RESULTS
WITHOUT PROJECT
N33+00

RETURN INTERVAL	DUNE CREST EROSION POSITION (feet from design line)	VOLUME ERODED ABOVE NGVD (cu yd/ft)
Original Dune Crest Location = 90 ft		
500	114	17.5
200	110	16.3
100	110	15.9
50	110	13.2
20	90	13.2
10	90	12.8
5	90	10.8

TABLE 16
EROSION MODEL RESULTS
WITHOUT PROJECT
LRP#25

RETURN INTERVAL	DUNE CREST EROSION POSITION (feet from design line)	VOLUME ERODED ABOVE NGVD (cu yd/ft)
Original Dune Crest Location = -3 ft		
500	112	19.1
200	88	16.9
100	88	15.5
50	82	14.4
20	75	12.4
10	42	10.8
5	10	7.6

TABLE 17
EROSION MODEL RESULTS
WITHOUT PROJECT
LRP#25A

RETURN INTERVAL	DUNE CREST EROSION POSITION (feet from design line)	VOLUME ERODED ABOVE NGVD (cu yd/ft)
Original Dune Crest Location = 0 ft		
500	82	22.3
200	72	19.4
100	72	18.7
50	59	16.7
20	42	14.4
10	32	13.2
5	6	10.4

TABLE 18
EROSION MODEL RESULTS
WITHOUT PROJECT
LRP#25B

RETURN INTERVAL	DUNE CREST EROSION POSITION (feet from design line)	VOLUME ERODED ABOVE NGVD (cu yd/ft)
Original Dune Crest Location = -4 ft		
500	53	21.5
200	23	18.7
100	23	16.7
50	16	14.8
20	7	12.8
10	0	11.2
5	-4	8.4

TABLE 19
EROSION MODEL RESULTS
WITHOUT PROJECT
LRP#26

RETURN INTERVAL	DUNE CREST EROSION POSITION (feet from design line)	VOLUME ERODED ABOVE NGVD (cu yd/ ft)
Original Dune Crest Location = 10 ft		
500	74	20.3
200	64	18.3
100	61	16.7
50	61	15.5
20	55	13.2
10	51	12.0
5	35	9.2

TABLE 20
EROSION MODEL RESULTS
WITHOUT PROJECT
LRP#27

RETURN INTERVAL	DUNE CREST EROSION POSITION (feet from design line)	VOLUME ERODED ABOVE NGVD (cu yd/ ft)
Original Dune Crest Location = 25 ft		
500	122	17.5
200	83	14.0
100	76	12.8
50	76	10.8
20	37	8.0
10	34	8.0
5	25	8.0

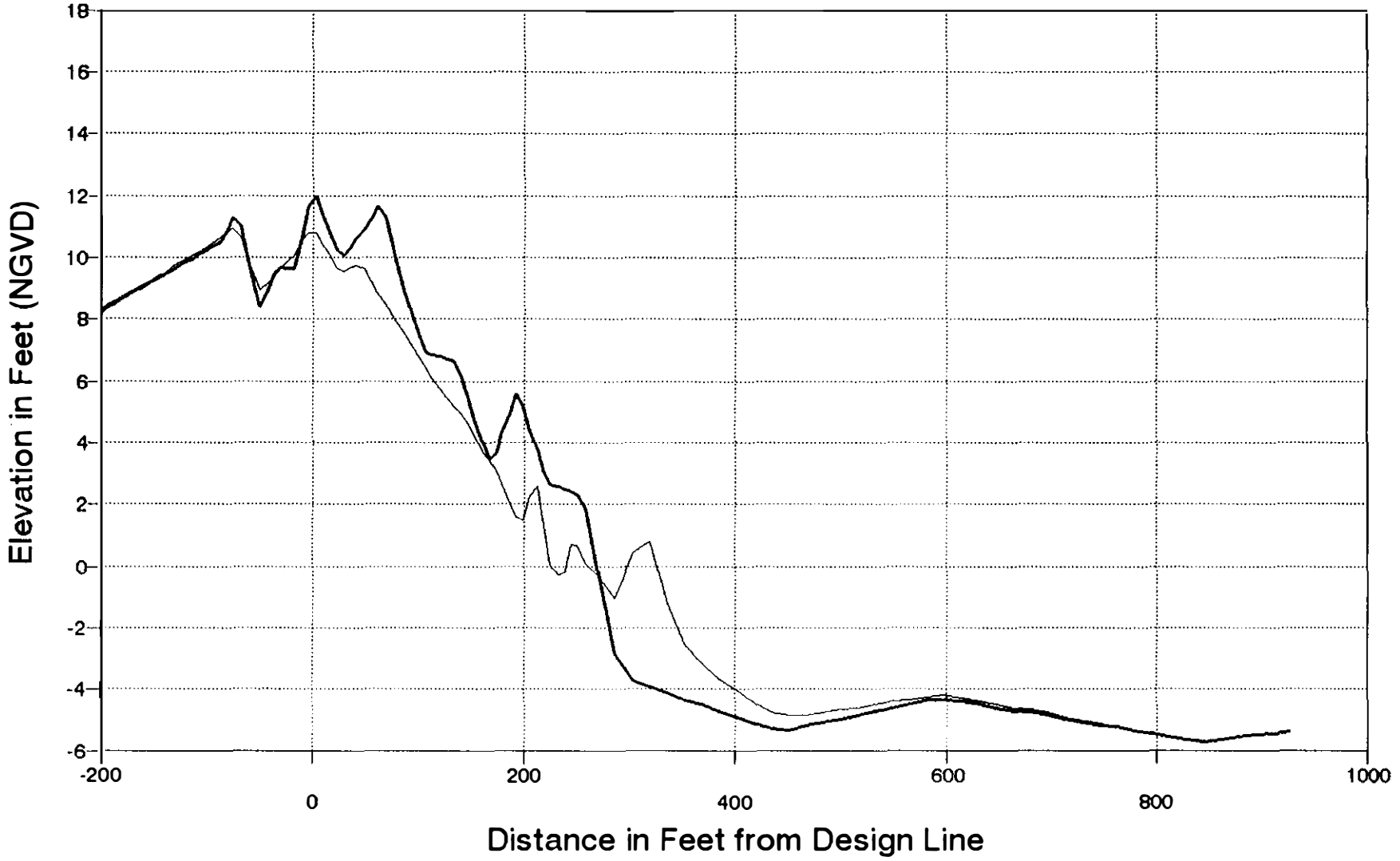
TABLE 21
EROSION MODEL RESULTS
WITHOUT PROJECT
LRP#27A

RETURN INTERVAL	DUNE CREST EROSION POSITION (feet from design line)	VOLUME ERODED ABOVE NGVD (cu yd/ ft)
Original Dune Crest Location = 6 ft		
500	102	19.1
200	98	17.9
100	92	17.1
50	92	15.9
20	79	14.4
10	36	12.8
5	20	10.4

TABLE 22
EROSION MODEL RESULTS
WITHOUT PROJECT
LRP#28

RETURN INTERVAL	DUNE CREST EROSION POSITION (feet from design line)	VOLUME ERODED ABOVE NGVD (cu yd/ft)
Original Dune Crest Location = 20 ft		
500	107	16.3
200	90	15.2
100	84	14.8
50	77	14.0
20	67	12.0
10	64	10.4
5	38	8.8

FIGURE 52. BEACH PROFILES, LRP#24A
BROADKILL BEACH, DELAWARE



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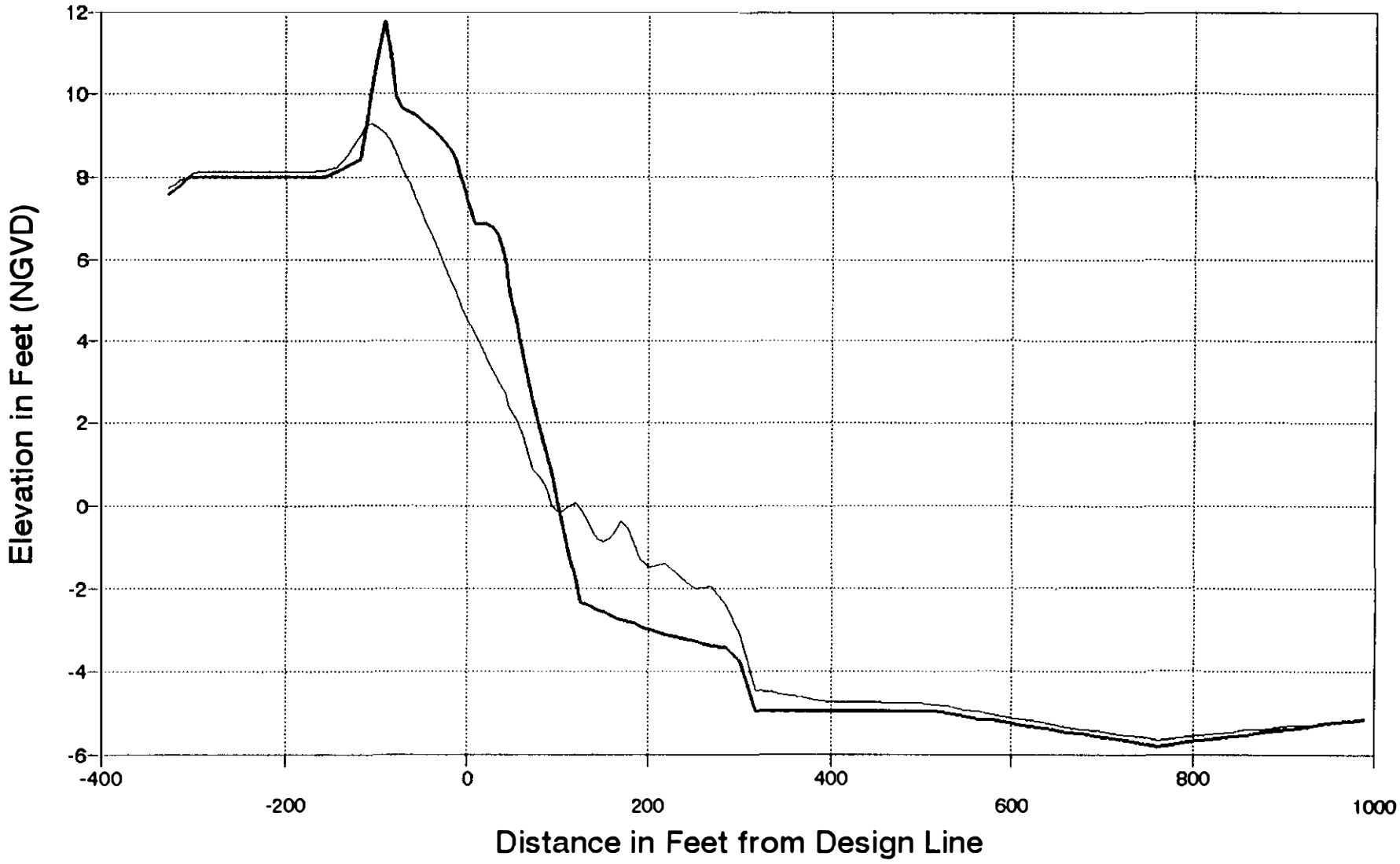
Initial — Eroded-100yr

FIGURE 53. BEACH PROFILES, N33+00
BROADKILL BEACH, DELAWARE



Initial — Eroded-100yr

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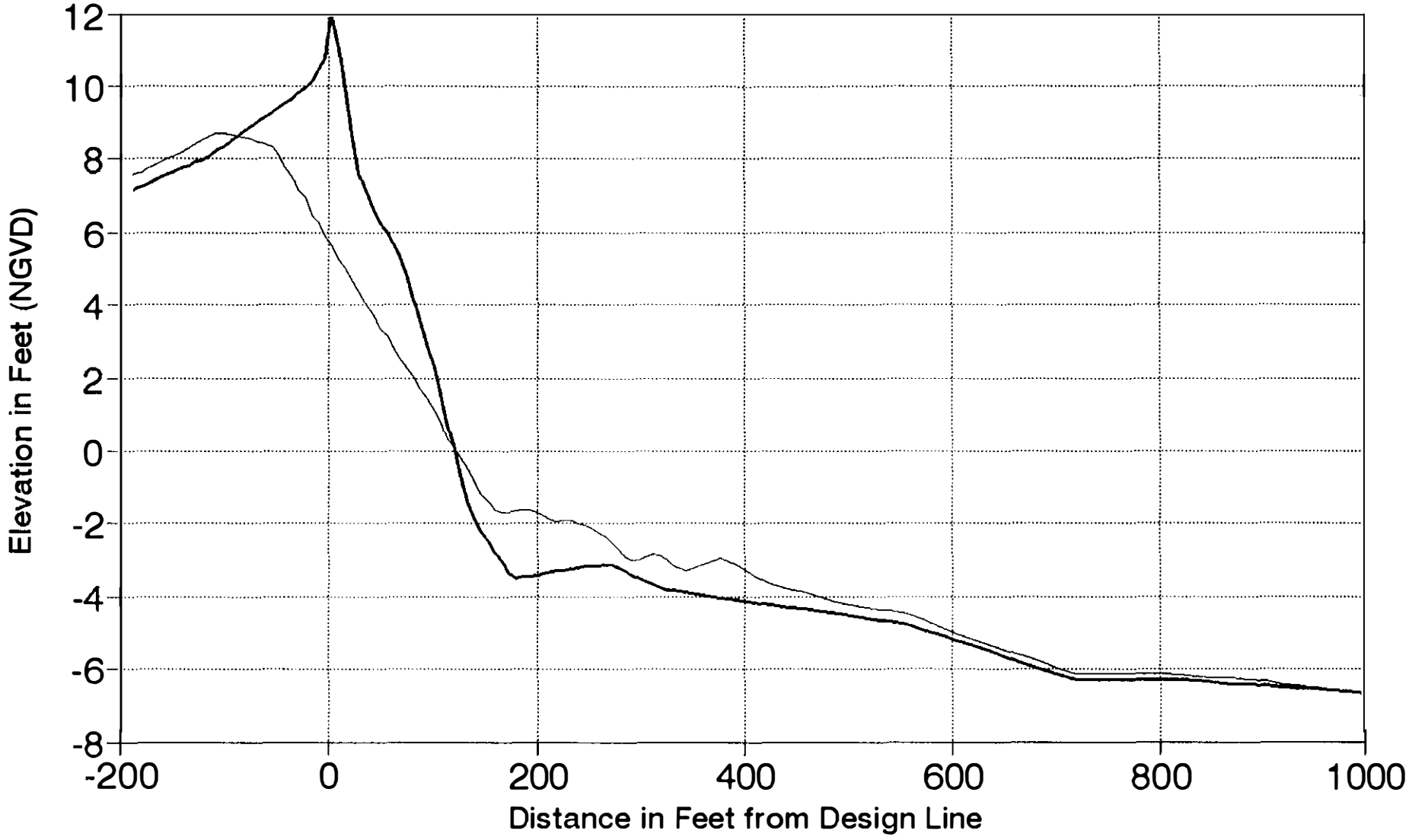
FIGURE 54. BEACH PROFILES, LRP#25
BROADKILL BEACH, DELAWARE



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Initial — Eroded-100yr

FIGURE 55. BEACH PROFILES, LRP#25A
BROADKILL BEACH, DELAWARE

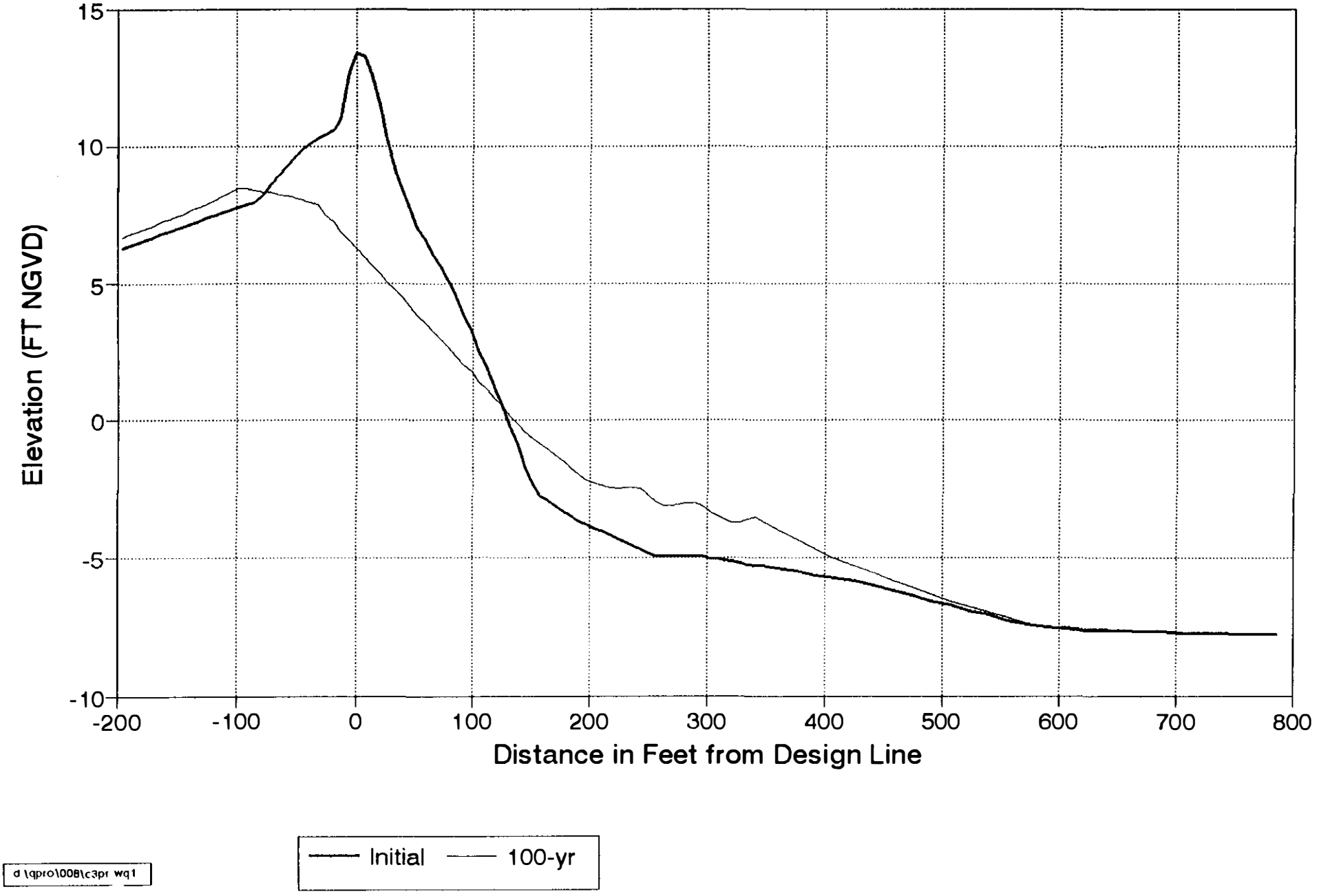

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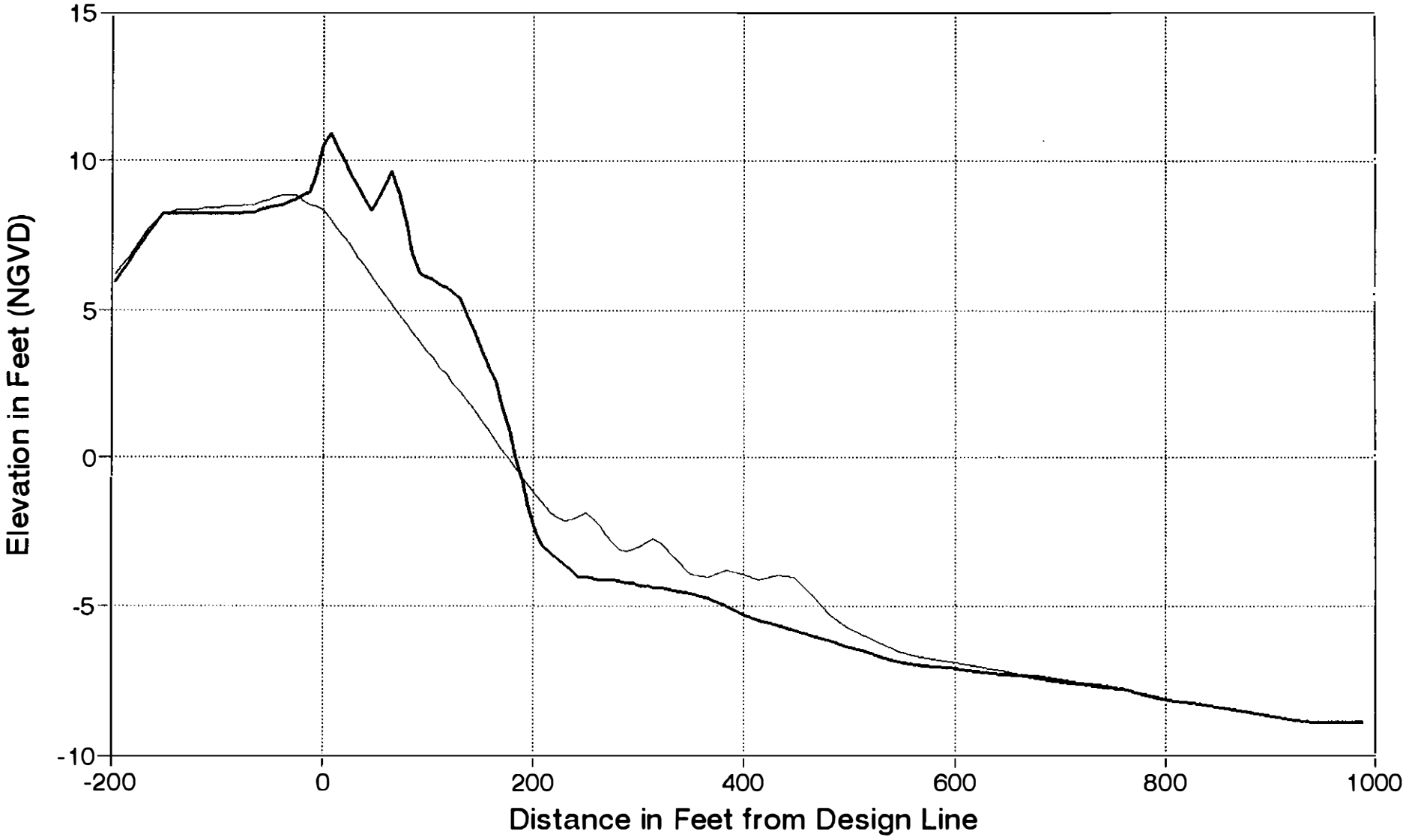
FIGURE 56. BEACH PROFILES, LRP#25B
BROADKILL BEACH, DELAWARE



Initial — Eroded-100yr

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FIGURE 57. BEACH PROFILES, LRP#26
BROADKILL BEACH, DELAWARE

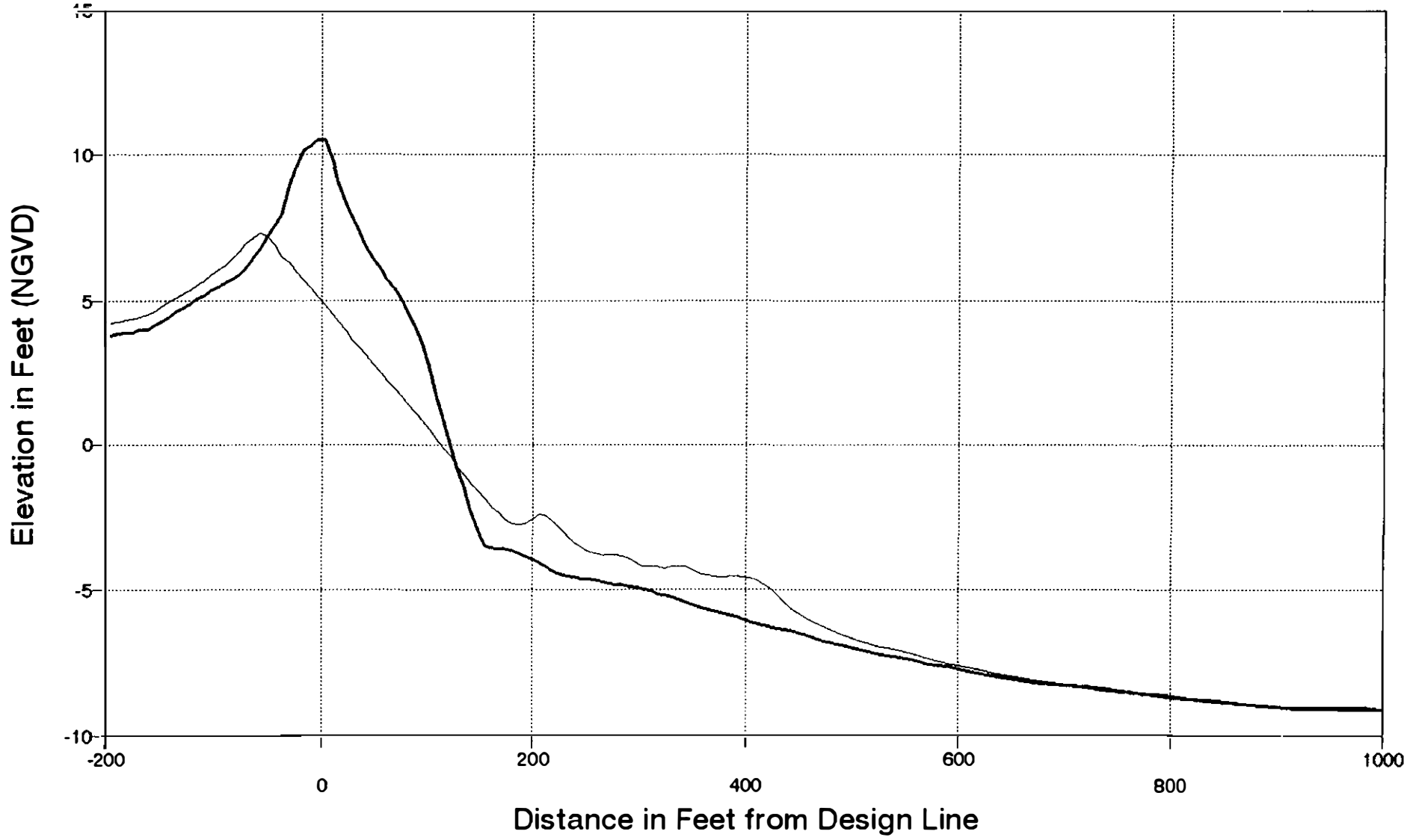

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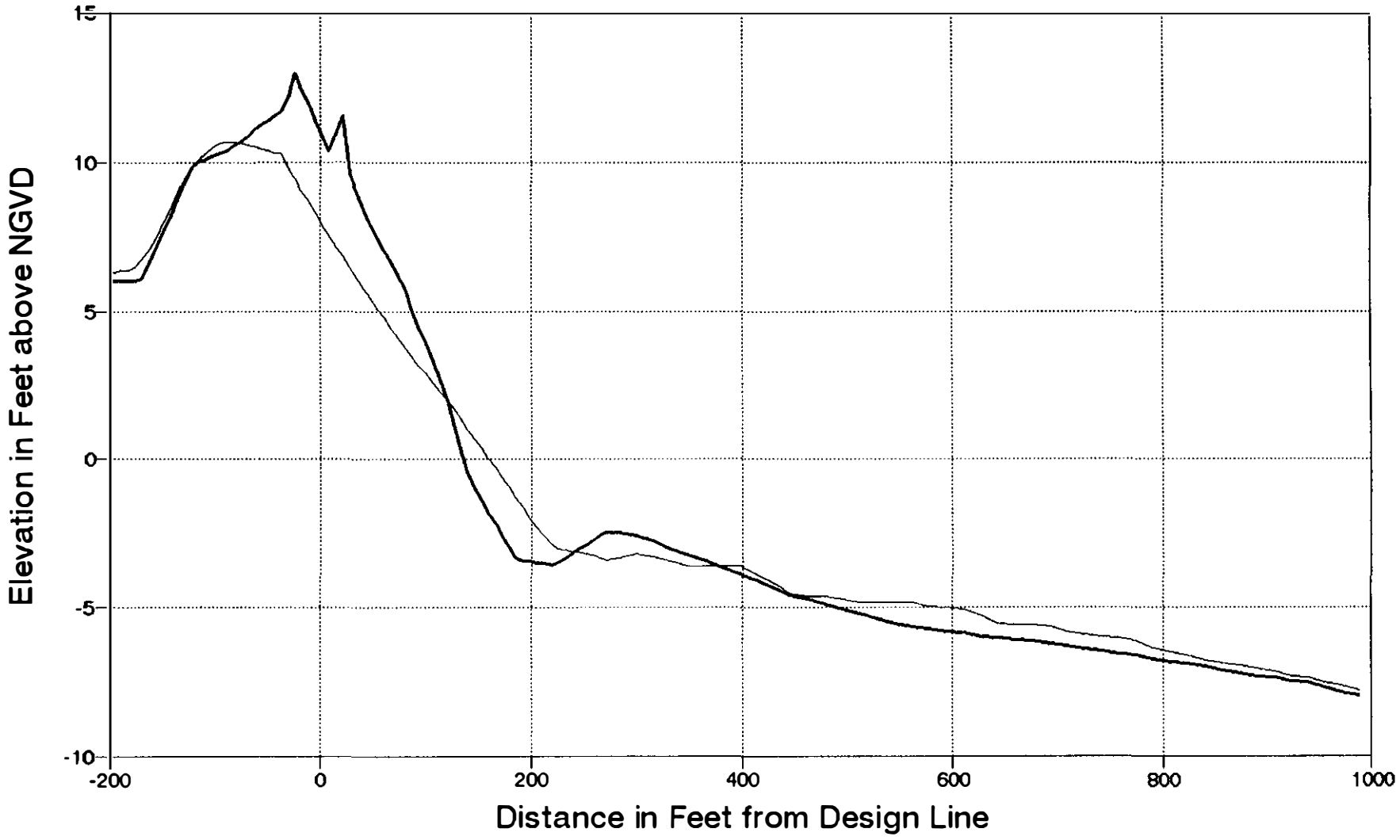
FIGURE 58. BEACH PROFILES, LRP#27
BROADKILL BEACH, DELAWARE



Initial — Eroded-100yr

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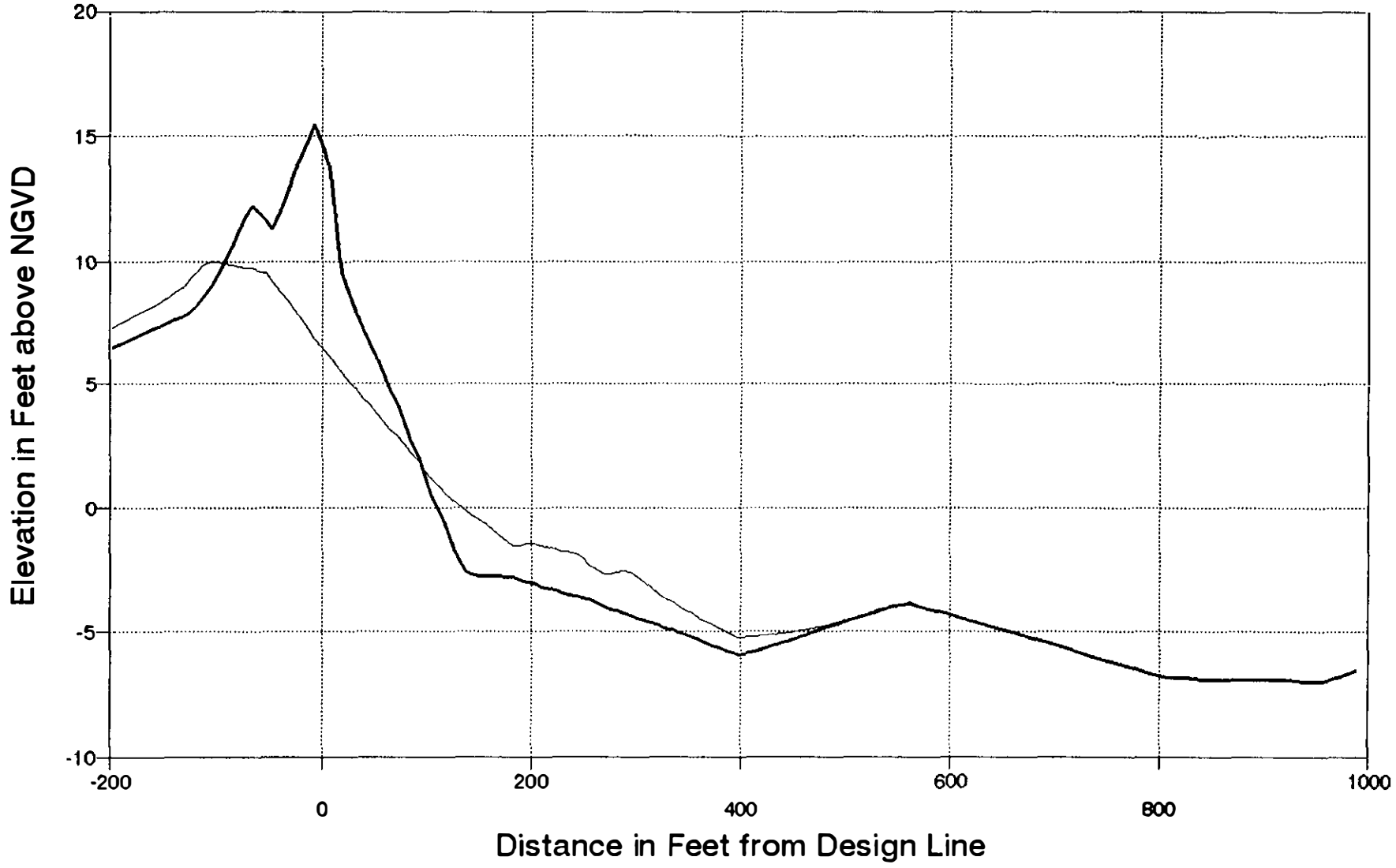
FIGURE 59. BEACH PROFILES, LRP#27A
BROADKILL BEACH, DELAWARE



Initial — Eroded-100yr

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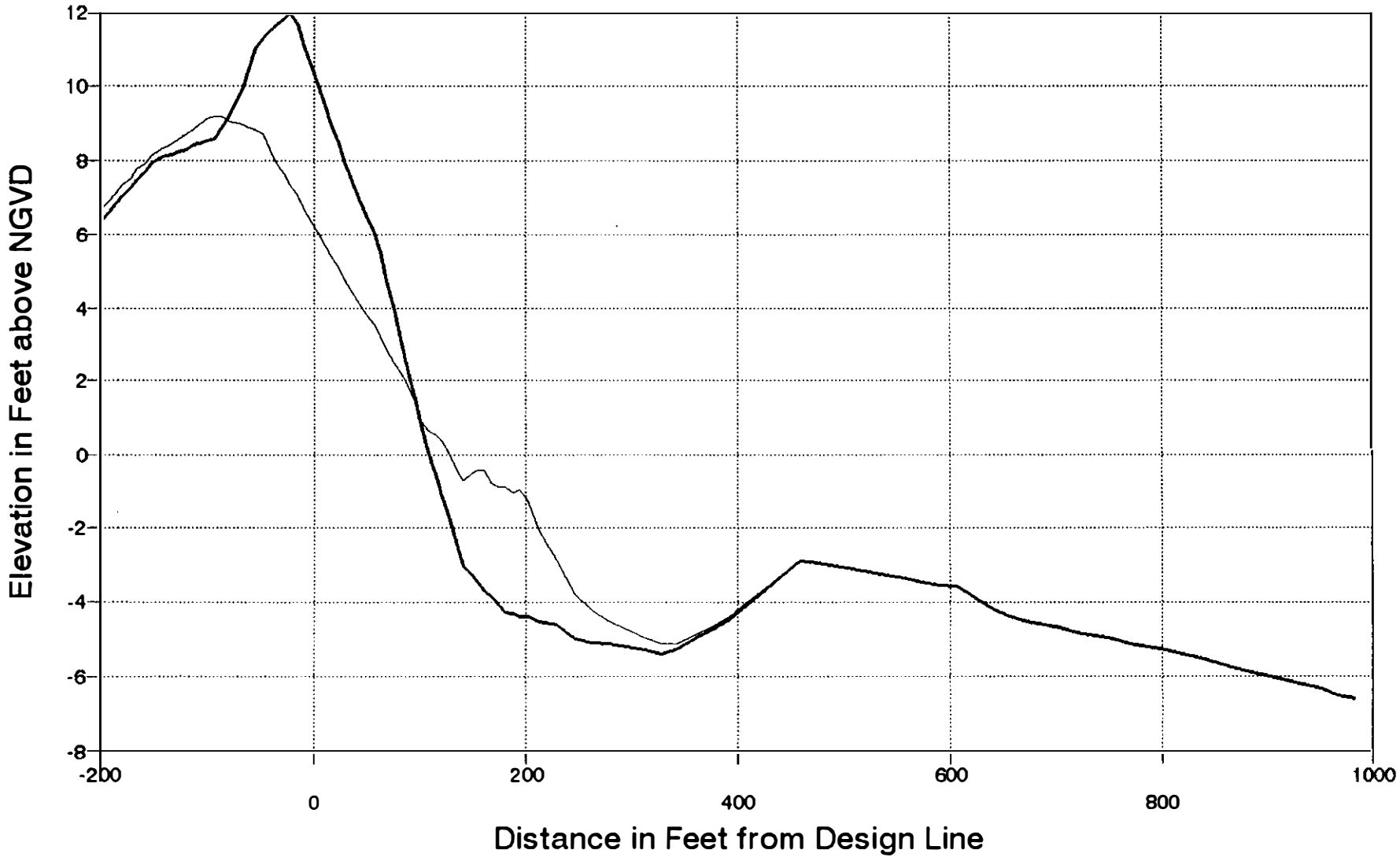
FIGURE 60. BEACH PROFILES, LRP#28
BROADKILL BEACH, DELAWARE



Initial — Eroded-100yr

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IV-6.B. Wave Attack and Inundation

The project area is subject to inundation from several sources including waves overtopping the dune and flooding from Broadkill Sound. Inundation can be categorized as two separate phenomena: (1) Static flooding due to the superelevation of the water surface (storm surge and wave setup), (2) Wave attack, the direct impact of waves and runup on the dunes.

Wave setup has long been recognized as an important component in the design of coastal projects. The U.S. Army Corps of Engineers, Shore Protection Manual (1984), defines setup as the superelevation of the mean water level caused by wave action alone. As wave trains approach the coast, non-linear wave breaking in the surf zone causes the water level at the shoreline to increase. Depending on the wave characteristics and nearshore bottom slope, this accumulation of water will continue until the slope of the water surface results in a head which balances the forces driving the water onto the shoreline.

In order to quantify the effects from flooding and wave attack, all inundation events are based on the stage frequency analysis in the recently developed Delaware Bay surge study. To estimate the setup during storm events, the techniques presented in the SPM (1984) were used. The wave heights and periods used for setup calculations at each frequency event were also obtained from the Delaware Bay wave study. The stage-frequency adopted at Broadkill Beach and the water stage with wave setup are presented in Table 23.

TABLE 23
Stage Frequencies Adopted at Broadkill Beach

	Elevation in ft. (NGVD) at given recurrence interval (yr)						
	5	10	20	50	100	200	500
Stage+setup	7.5	9.2	10.1	11.2	12.5	13.5	14.6
Stage	6.2	7.5	8.0	9.5	10.5	11.2	12.4

To evaluate the added potential for structural damage, the boundaries of the wave attack were estimated for each survey profile. The analysis estimates the location of a wave attack line and the associated zones of high energy stages. The wave attack line is the most landward position in the swash zone where the force due to waves exceeds the force required to damage typical coastal structures. A 3-foot wave height is adopted as the minimum wave that would cause damage to typical structures.

The methodology applied to estimate the wave attack is outlined in the FEMA report "Guidelines and Specifications for Wave

Elevation Determination and V-Zone Mapping" (1989). The analysis is done for each of the selected profiles. It should be noted that variability exists which cannot be modeled at this level of detail.

A potential wave is transmitted across the eroded profiles. The total water level (wave crest elevation) is computed by adding 70% of the maximum sustained wave height to the inundation stage. The first seaward estimate starts at the design line. If the wave zone location is seaward of the design line, the estimate begins at the wave zone. Seaward of the dune, the total water level is the higher of the breaking wave crest elevation or the maximum runup elevation. At the dune crest, the maximum total water level again is either from runup or from breaking waves. The maximum runup level is limited to the dune crest elevation plus 3 feet. In this case, the wave zone is at the dune crest if the still water level cannot support a three foot wave across the dune. If the still water level is over 3.8 feet higher than the dune crest, the wave zone can be further landward. Landward of the dune crest, it is assumed that the total water level drops at the rate of 1 foot every 50 feet.

Tables 24-32 present the wave inundation results for each profile for the without project condition. The total water level in these tables is the combined height (relative to NGVD) of surge, setup and wave height.

TABLE 24
WAVE INUNDATION RESULTS
LRP#24A WITHOUT PROJECT

RETURN INTERVAL (years)	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0 50 100-river	16.5 15.5 14.5	0
200	0 50 80-river	15.0 14.0 13.5	0
100	-40 10 60 110-river	14.0 13.5 12.5 10.5	-40
50	-50 0 50 200-river	13.5 12.5 11.5 9.5	-50
20	-80 -60 0 50 120-river	13.0 13.0 12.0 11.0 7.0	-80
10	-90 -70 -20-river	11.0 11.0 5.5	- 90
5	-150 - 90 -40-river	9.0 9.0 4.5	-150

TABLE 25
WAVE INUNDATION RESULTS
N33+00 WITHOUT PROJECT

RETURN INTERVAL (years)	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0 50 100 130 180-river	20.0 19.0 18.0 17.5 16.5	entire area in wave zone
200	0 50 110 160-river	18.5 17.0 16.0 15.5	entire area in wave zone
100	0 50 100 180 200-river	17.5 15.0 14.5 13.0 12.5	130
50	0 50 100 150 200 250-river	14.5 14.0 13.5 12.5 11.5 9.5	100
20	0 90 150 200-river	13.0 12.5 9.0 7.0	60
10	0 90 140-river	12.0 11.5 5.5	40
5	0 80 130-river	9.5 9.5 4.5	20

TABLE 26
WAVE INUNDATION RESULTS
LRP#25 WITHOUT PROJECT

RETURN INTERVAL (years)	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0	20.0	entire area in wave zone
	90	18.0	
	140	17.0	
	170-river	16.5	
200	0	18.0	entire area in wave zone
	40	17.0	
	120-river	15.5	
100	0	16.5	entire area in wave zone
	60	15.0	
	120	14.5	
	270-river	11.5	
50	0	14.0	70
	70	13.0	
	120	12.0	
	170	11.0	
	240-river	9.5	
20	0	12.0	40
	40	12.0	
	90	11.0	
	140	10.0	
	190	9.0	
	240-river	7.0	
10	0	11.0	10
	10	11.0	
	60	10.0	
	110	9.0	
	210	7.5	
	240-river	5.5	
5	-30	10.5	-30
	20	10.5	
	70	6.2	
	120-river	4.5	

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TABLE 27
WAVE INUNDATION RESULTS
LRP#25A WITHOUT PROJECT

RETURN INTERVAL (years)	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0 100 130 180-river	20.0 18.0 17.0 16.5	entire area in wave zone
200	0 70 120-river	18.0 16.5 15.5	entire area in wave zone
100	0 70-river	16.5 14.5	entire area in wave zone
50	0 20 60 110 160 210-river	14.0 13.0 12.5 11.5 10.5 9.5	60
20	0 70 120 170 270 370-river	12.5 12.5 11.5 10.5 8.5 7.0	20
10	0 40 90 140-river	11.0 11.0 7.5 5.5	0
5	-60 -20 30 80-river	9.5 9.5 6.2 4.5	-60

TABLE 28
WAVE INUNDATION RESULTS
LRP#25B WITHOUT PROJECT

RETURN INTERVAL (years)	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0 50 100 130-river	19.0 18.0 17.0 16.5	entire area in wave zone
200	0 30 50-river	16.5 16.0 15.5	entire area in wave zone
100	0 50-river	15.0 14.5	entire area in wave zone
50	0 30 80 130 150-river	12.5 12.0 11.0 10.5 9.5	30
20	0 50 100 150 230-river	12.5 11.5 10.5 9.5 8.0	0
10	-45 0 50-river	11.0 11.0 7.5	-45
5	-70 -20 -30-river	9.5 9.5 6.2	-70

TABLE 29
WAVE INUNDATION RESULTS
LRP#26 WITHOUT PROJECT

RETURN INTERVAL (years)	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0	20.5	entire area in wave zone
	80	19.0	
	130	18.0	
	180	17.0	
	200-river	16.5	
200	0	18.5	entire area in wave zone
	70	17.0	
	120	16.0	
	150-river	15.5	
100	0	17.0	entire area in wave zone
	70	15.5	
	120-river	14.5	
50	0	15.0	70
	70	13.5	
	120	12.5	
	170	11.5	
	220	10.5	
	270-river	9.5	
20	0	12.5	50
	50	11.5	
	100	10.5	
	150	9.5	
	180	8.0	
	280-river	7.0	
10	0	11.0	50
	50	11.0	
	100	10.0	
	150	9.0	
	200	8.0	
	280-river	5.5	
5	-20	9.5	-20
	30	9.5	
	80	6.0	
	130-river	4.5	

TABLE 30
WAVE INUNDATION RESULTS
LRP#27 WITHOUT PROJECT

RETURN INTERVAL (years)	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0	18.5	entire area in wave zone
	50	17.5	
	100-river	16.5	
200	0	16.5	80
	50	15.5	
	80	15.0	
	100	14.5	
	150-river	13.5	
100	0	15.0	70
	40	14.0	
	70	13.5	
	170	12.5	
	250-river	10.5	
50	0	13.0	30
	50	13.0	
	100	12.0	
	150	11.0	
	200	10.0	
	250-river	9.5	
20	0	13.0	10
	50	13.0	
	100	12.0	
	150	11.0	
	200	10.0	
	250	9.0	
	300-river	7.0	
10	-30	11.5	-30
	20	11.5	
	70	7.5	
	130-river	5.5	
5	-70	9.5	-50
	-30	9.5	
	20	6.0	
	100-river	4.5	

TABLE 31
WAVE INUNDATION RESULTS
LRP#27A WITHOUT PROJECT

RETURN INTERVAL (years)	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0	19.0	entire area in wave zone
	50	18.0	
	100	17.5	
	150-river	16.5	
200	0	17.5	100
	50	16.0	
	100	15.5	
	150	14.5	
	200-river	13.5	
100	0	16.0	80
	50	15.0	
	150	13.0	
	200	11.5	
	250-river	10.5	
50	0	13.5	50
	50	12.5	
	100	12.0	
	150	11.0	
	200	10.0	
	230-river	9.5	
20	0	12.5	20
	70	12.5	
	170	10.5	
	270	8.5	
	320-river	7.5	
10	0	12.0	0
	20	12.0	
	70	7.5	
	120-river	5.5	
5	-20	11.0	-20
	0	11.0	
	50	6.0	
	100-river	4.5	

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TABLE 32
WAVE INUNDATION RESULTS
LRP#28 WITHOUT PROJECT

RETURN INTERVAL (years)	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0 100 150 200-river	20.0 18.0 17.0 16.5	entire area in wave zone
200	0 50 150-river	17.5 16.5 15.5	entire area in wave zone
100	0 100 150 250 300-river	16.0 14.5 13.0 11.5 10.5	110
50	0 40 140 190 220-river	14.0 12.5 11.5 10.0 9.5	90
20	0 80 130 180 230 280-river	12.0 12.0 11.0 10.0 9.0 7.0	30
10	0 70 120 170 220 270-river	11.5 11.5 10.5 9.5 8.0 5.5	20
5	-30 20 70 120 -river	10.0 10.0 6.0 4.5	-30

Section V
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PHASE I REPORT

BROADKILL BEACH
EROSION AND FLOODING PROTECTION

APPENDIX A
SHORELINE PROTECTION HISTORY
DELAWARE BAY WESTERN SHORE

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DELAWARE BAY WEST SHORE
BEACH EROSION CONTROL PROJECTS

PROJECT LOACTION	YEAR	PROJECT TYPE	MODE/MATERIAL USED	FILL AMOUNT (CY)	LENGTH (FT)

DELAWARE BAY WEST SHORE
BEACH EROSION CONTROL PROJECTS

PROJECT LOACTION	YEAR	PROJECT TYPE	MODE/MATERIAL USED	FILL AMOUNT (CY)	LENGTH (FT)
PICKERING BEACH	1962	BEACHFILL	TRUCKFILL	39600	0
PICKERING BEACH	1969	BEACHFILL	TRUCKFILL	5000	0
PICKERING BEACH	1978	BEACHFILL	HYDRAULIC DREDGE	85200	1600
PICKERING BEACH	1990	BEACHFILL	14" HYDRAULIC DREDGE	55400	2400
KITTS HUMMOCK	1961	BEACHFILL		80000	0
KITTS HUMMOCK	1962	BEACHFILL	TRUCKFILL	30600	0
KITTS HUMMOCK	1969	BEACHFILL	TRUCKFILL	12000	0
KITTS HUMMOCK	1973	BEACHFILL	TRUCKFILL	3000	0
KITTS HUMMOCK	1974	BEACHFILL	12" HYDRAULIC DREDGE	46500	1700
KITTS HUMMOCK	1979	BEACHFILL	HYDRAULIC DREDGE	74000	0
KITTS HUMMOCK	1988	BEACHFILL	14" HYDRAULIC DREDGE	15780	1000
BOWERS	1962	BEACHFILL	TRUCKFILL	35500	0
BOWERS	1968	BEACHFILL	HYDRAULIC DREDGE	18000	0
BOWERS	1969	BEACHFILL	TRUCKFILL	6500	0
BOWERS	1972	BEACHFILL	HYDRAULIC DREDGE	21200	0
BOWERS	1973	BEACHFILL	12" HYDRAULIC DREDGE	15800	1400
BOWERS	1974	BEACHFILL	12" HYDRAULIC DREDGE	28800	1000
BOWERS	1985	BEACHFILL	TRUCKFILL	35700	0
BOWERS	1986	BEACHFILL	12" HYDRAULIC DREDGE	13700	500
BOWERS	1988	BEACHFILL	14" HYDRAULIC DREDGE	51700	0
SOUTH BOWERS	1961	BEACHFILL		20000	0
SOUTH BOWERS	1962	BEACHFILL	TRUCKFILL	10000	0
SOUTH BOWERS	1969	BEACHFILL	TRUCKFILL	4000	0

DELAWARE BAY WEST SHORE
BEACH EROSION CONTROL PROJECTS

PROJECT LOACTION	YEAR	PROJECT TYPE	MODE/MATERIAL USED	FILL AMOUNT (CY)	LENGTH (FT)
SOUTH BOWERS	1974	BEACHFILL	TRUCKFILL	4000	830
SOUTH BOWERS	1975	BEACHFILL	12" HYDRAULIC DREDGE	15000	1000
SOUTH BOWERS	1976	BEACHFILL	HYDRAULIC DREDGE	9400	0
SOUTH BOWERS	1984	BEACHFILL	12" HYDRAULIC DREDGE	17000	0
SOUTH BOWERS	1989	BEACHFILL	12" HYDRAULIC DREDGE	8000	0
BIG STONE BEACH	1962	BEACHFILL	HEAVY EQUIPMENT	26000	0
SLAUGHTER BEACH	1958	BEACHFILL		49000	0
SLAUGHTER BEACH	1961	BEACHFILL	HYDRAULIC DREDGE	165000	0
SLAUGHTER BEACH	1962	BEACHFILL	TRUCKFILL & HEAVY EQUIP.	56600	0
SLAUGHTER BEACH	1975	BEACHFILL	14" HYDRAULIC DREDGE	179500	4700
SLAUGHTER BEACH	1976	BEACHFILL	20" HYDRAULIC DREDGE	277700	9600
SLAUGHTER BEACH	1979	BEACHFILL	12" HYDRAULIC DREDGE	20000	0
SLAUGHTER BEACH	1985	BEACHFILL	12" HYDRAULIC DREDGE	26200	1720
SLAUGHTER BEACH	1985	BEACHFILL	12" HYDRAULIC DREDGE	10300	720
PRIMEHOOK BEACH	1962	BEACHFILL	HEAVY EQUIPMENT	20200	0
BROADKILL BEACH	1957	BEACHFILL		76800	1500
BROADKILL BEACH	1961	BEACHFILL	HYDRAULIC DREDGE	120000	0
BROADKILL BEACH	1962	BEACHFILL		180000	0 — ?
BROADKILL BEACH	1973	BEACHFILL	16" HYDRAULIC DREDGE	118100	4500
BROADKILL BEACH	1975	BEACHFILL	14" HYDRAULIC DREDGE	295000	6100
BROADKILL BEACH	1976	BEACHFILL	12" HYDRAULIC DREDGE	59700	2200
BROADKILL BEACH	1981	BEACHFILL	HYDRAULIC DREDGE	127700	0
BROADKILL BEACH	1987	BEACHFILL	14" HYDRAULIC DREDGE	21700	0
BROADKILL BEACH	1988	BEACHFILL	14" HYDRAULIC DREDGE	59400	0
BROADKILL BEACH	1993	BEACHFILL	14" HYDRAULIC DREDGE	37000	0
LEWES	1953	BEACHFILL		55100	0

DELAWARE BAY WEST SHORE
BEACH EROSION CONTROL PROJECTS

PROJECT LOACTION	YEAR	PROJECT TYPE	MODE/MATERIAL USED	FILL AMOUNT (CY)	LENGTH (FT)
LEWES	1954	BEACHFILL	HYDRAULIC DREDGE	44900	0
LEWES	1957	BEACHFILL	HYDRAULIC DREDGE	79000	0
LEWES	1957	BEACHFILL	HYDRAULIC DREDGE	434400	0
LEWES	1962	BEACHFILL	HYDRAULIC DREDGE	20700	0
LEWES	1963	BEACHFILL	HYDRAULIC DREDGE	87000	0
LEWES	1969	BEACHFILL	HYDRAULIC DREDGE	135600	0
LEWES	1973	BEACHFILL	12" HYDRAULIC DREDGE	69800	3700
LEWES	1975	BEACHFILL	16" HYDRAULIC DREDGE	101700	4800
LEWES	1976	BEACHFILL	12" HYDRAULIC DREDGE	11400	1000
LEWES	1978	BEACHFILL	12" HYDRAULIC DREDGE	31000	1000
LEWES	1981	BEACHFILL	HYDRAULIC DREDGE	113900	0
LEWES	1983	BEACHFILL	12" HYDRAULIC DREDGE	50500	0
LEWES	1987	BEACHFILL	12" HYDRAULIC DREDGE	12000	0
LEWES	1989	BEACHFILL	14" HYDRAULIC DREDGE	150000	0
LEWES	1990	BEACHFILL	12" HYDRAULIC DREDGE	32000	0
MISPILLION INLET	1939	JETTY	TIMBER & STONE	0	6496
MISPILLION INLET	1920	JETTY	TIMBER & STONE	0	0
MISPILLION RIV BEA	1985	DIKE	STONE	19500 TONS	970
SLAUGHTER BEACH	1957	GROIN	TIMBER	0	98
SLAUGHTER BEACH	1957	GROIN	TIMBER	0	154
SLAUGHTER BEACH	1954	GROIN	TIMBER & STONE	0	168
SLAUGHTER BEACH	1954	GROIN	TIMBER	0	172
SLAUGHTER BEACH	1950	GROIN	TIMBER	0	179
SLAUGHTER BEACH	1950	GROIN	TIMBER	0	138
SLAUGHTER BEACH	1950	GROIN	TIMBER	0	139
SLAUGHTER BEACH	1947	GROIN	TIMBER	0	110

DELAWARE BAY WEST SHORE
BEACH EROSION CONTROL PROJECTS

PROJECT LOACTION	YEAR	PROJECT TYPE	MODE/MATERIAL USED	FILL AMOUNT (CY)	LENGTH (FT)
SLAUGHTER BEACH	1947	GROIN	TIMBER	0	110
SLAUGHTER BEACH	1947	GROIN	TIMBER	0	115
SLAUGHTER BEACH	1947	GROIN	TIMBER	0	140
SLAUGHTER BEACH	1947	GROIN	TIMBER	0	150
SLAUGHTER BEACH	1947	GROIN	TIMBER	0	150
SLAUGHTER BEACH	1947	GROIN	TIMBER	0	150
SLAUGHTER BEACH	1943	GROIN	TIMBER	0	150
SLAUGHTER BEACH	1943	GROIN	TIMBER	0	150
SLAUGHTER BEACH	1940	GROIN	TIMBER	0	150
SLAUGHTER BEACH	1940	GROIN	TIMBER	0	150
SLAUGHTER BEACH	1940	GROIN	TIMBER	0	150
SLAUGHTER BEACH	1940	GROIN	TIMBER	0	150
FOWLER BEACH	1932	GROIN	TIMBER	0	140
BROADKILL BEACH	1964	GROIN	RUBBLE MOUND		
BROADKILL BEACH	1964	GROIN	RUBBLE MOUND		
BROADKILL BEACH	1954	GROIN	TIMBER & STONE	0	195
BROADKILL BEACH	1964	REVETMENT	RUBBLE MOUND		
BROADKILL BEACH	1954	GROIN	TIMBER	0	186
BROADKILL BEACH	1950	GROIN	TIMBER	0	196
BROADKILL BEACH	1950	GROIN	TIMBER	0	196
BROADKILL BEACH	1950	GROIN	TIMBER	0	199
BROADKILL BEACH	1908	JETTY	TIMBER & STONE	0	1263
LEWES	1937	JETTY	STEEL	0	1700
LEWES	1937	JETTY	STEEL	0	1700
LEWES	1898	BREAKWATER	STONE	0	5300
LEWES	1901	BREAKWATER	STONE	0	8000

DELAWARE BAY WEST SHORE
BEACH EROSION CONTROL PROJECTS

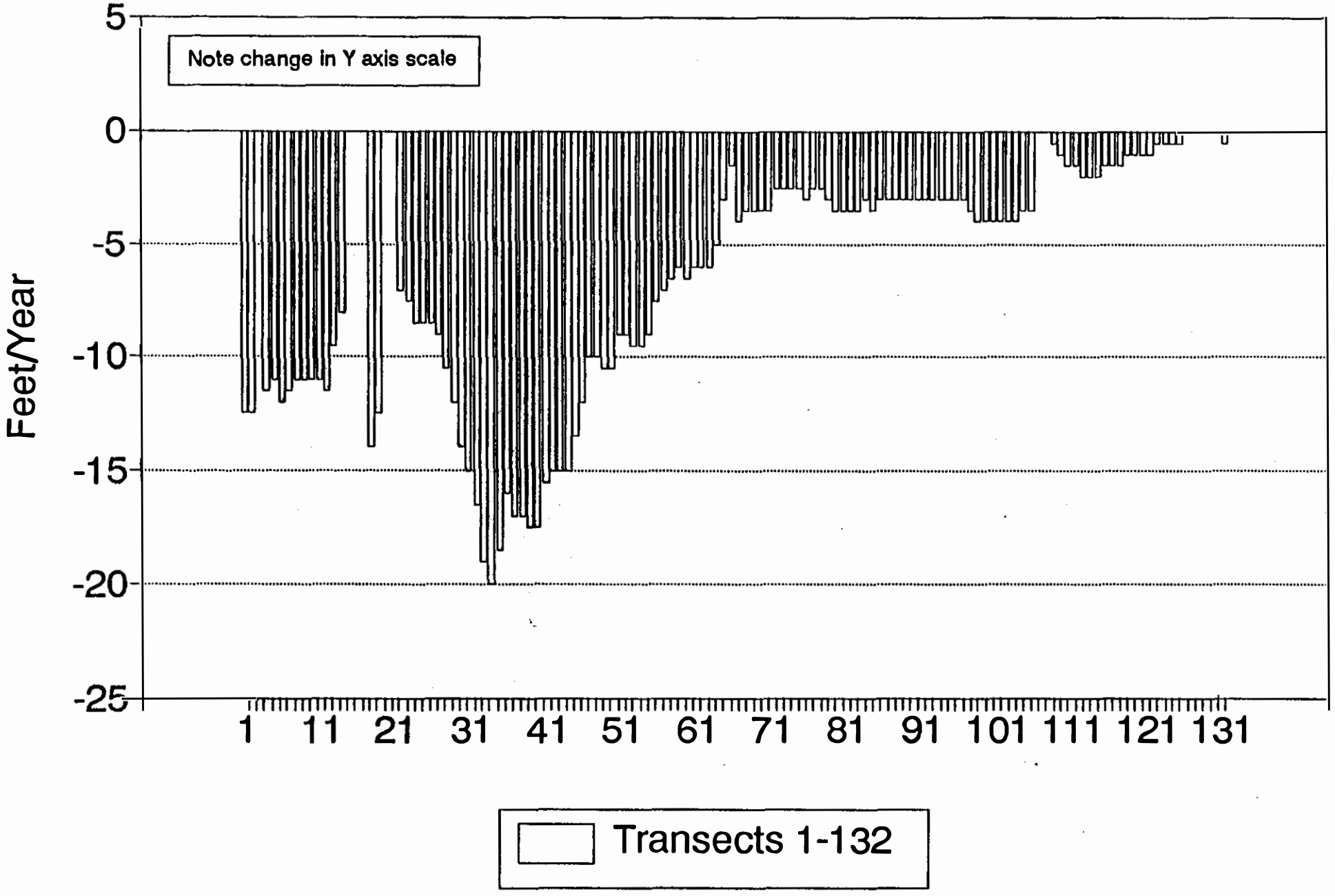
PROJECT LOACTION	YEAR	PROJECT TYPE	MODE/MATERIAL USED	FILL AMOUNT (CY)	LENGTH (FT)
LEWES	1950	GROIN	TIMBER	0	172
LEWES	1948	GROIN	TIMBER	0	145
LEWES	1948	GROIN	TIMBER	0	135
LEWES	1948	GROIN	TIMBER	0	150
LEWES	1950	GROIN	TIMBER	0	161
LEWES	1950	GROIN	TIMBER	0	164
LEWES	1956	GROIN	TIMBER	0	0
LEWES	1956	GROIN	TIMBER	0	0
LEWES	1956	GROIN	TIMBER	0	0
LEWES	1990	GROIN	GROUT FILLED NYLON BAGS	0	350
BOWERS	1976	GROIN	SAND FILLED NYLON BAGS	0	900
BOWERS	1976	GROIN	SAND FILLED NYLON BAGS	0	400
BOWERS	1986	GROIN	GROUT FILLED NYLON BAGES	0	213
BOWERS	1988	GROIN	GROUT FILLED NYLON BAGES	0	290
BOWERS	1988	GROIN	GROUT FILLED NYLON BAGES	0	320
SOUTH BOWERS	1976	GROIN	SAND FILLED NYLON BAGS	0	325
SOUTH BOWERS	1976	GROIN	SAND FILLED NYLON BAGS	0	325
SOUTH BOWERS	1988	GROIN	GROUT FILLED NYLON BAGES	0	600
PICKERING BEACH	1979	BREAKWATER	SCRAP AUTOMOBILE TIRES	0	400
PICKERING BEACH	1984	BREAKWATER	SCRAP AUTOMOBLILE TIRES	0	200
SLAUGHTER BEACH	1979	PERCHED BEACH	TIMBER, CONCRETE BOXES & NYLON BAGS	20000	0
KITTS HUMMOCK	1979	BREAKWATER	STONE	0	330
KITTS HUMMOCK	1979	BREAKWATER	SAND FILLED NYLON BAGES	0	330
KITTS HUMMOCK	1979	BREAKWATER	CONCRETE BOXES	0	330
KITTS HUMMOCK	1987	GROIN	CON. FILLED NYLON BAGS	0	180

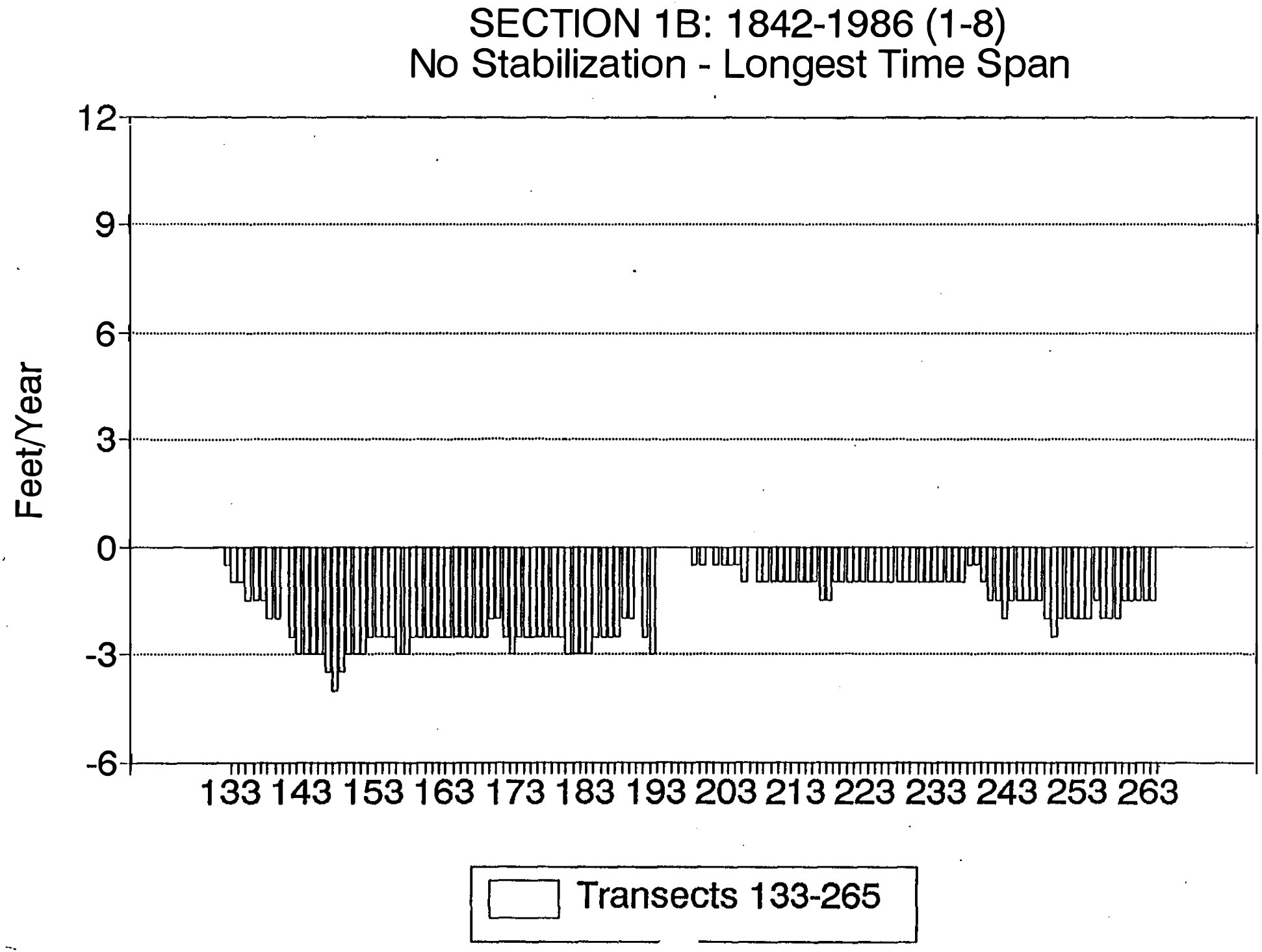
PHASE I REPORT

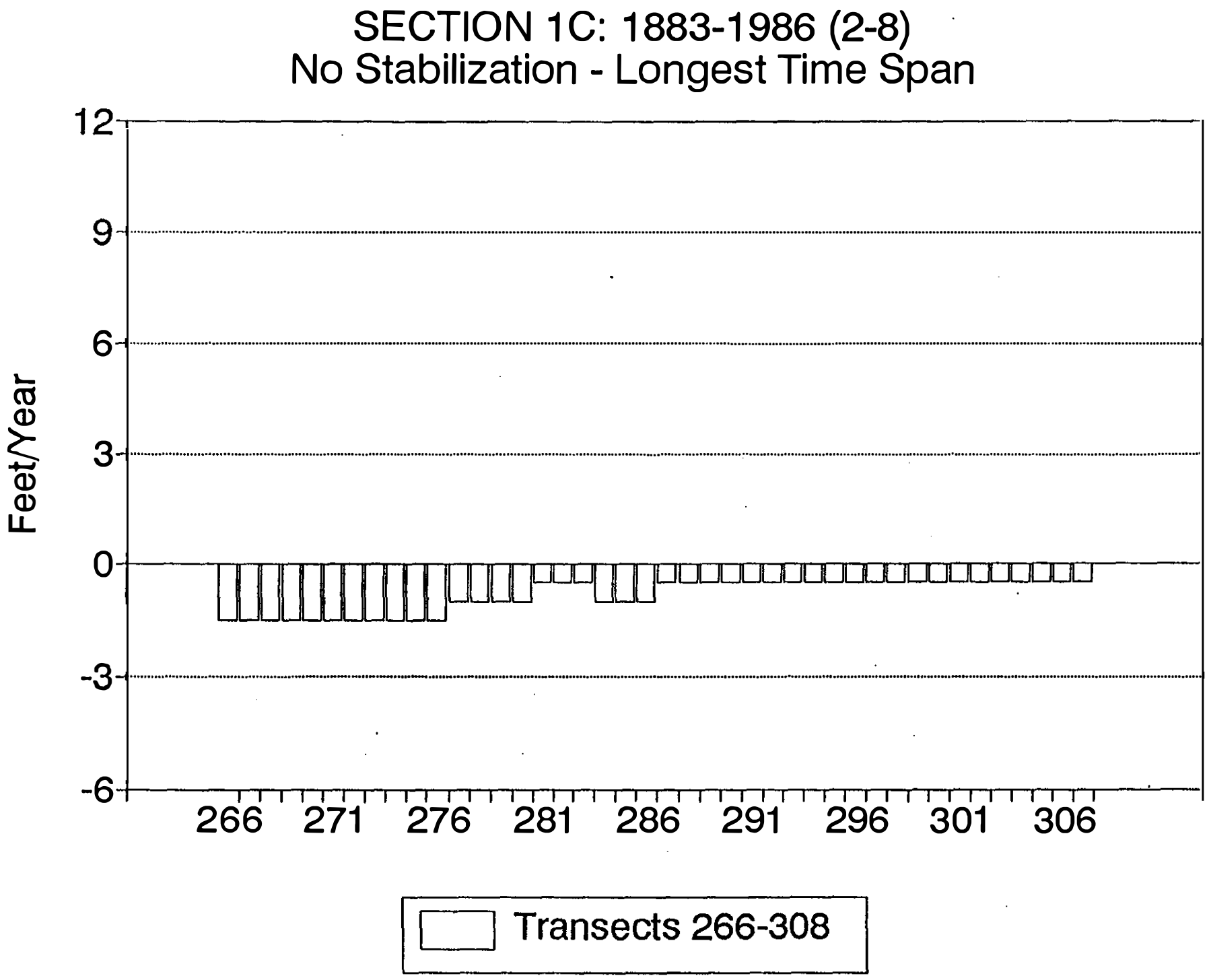
BROADKILL BEACH
EROSION AND FLOODING PROTECTION

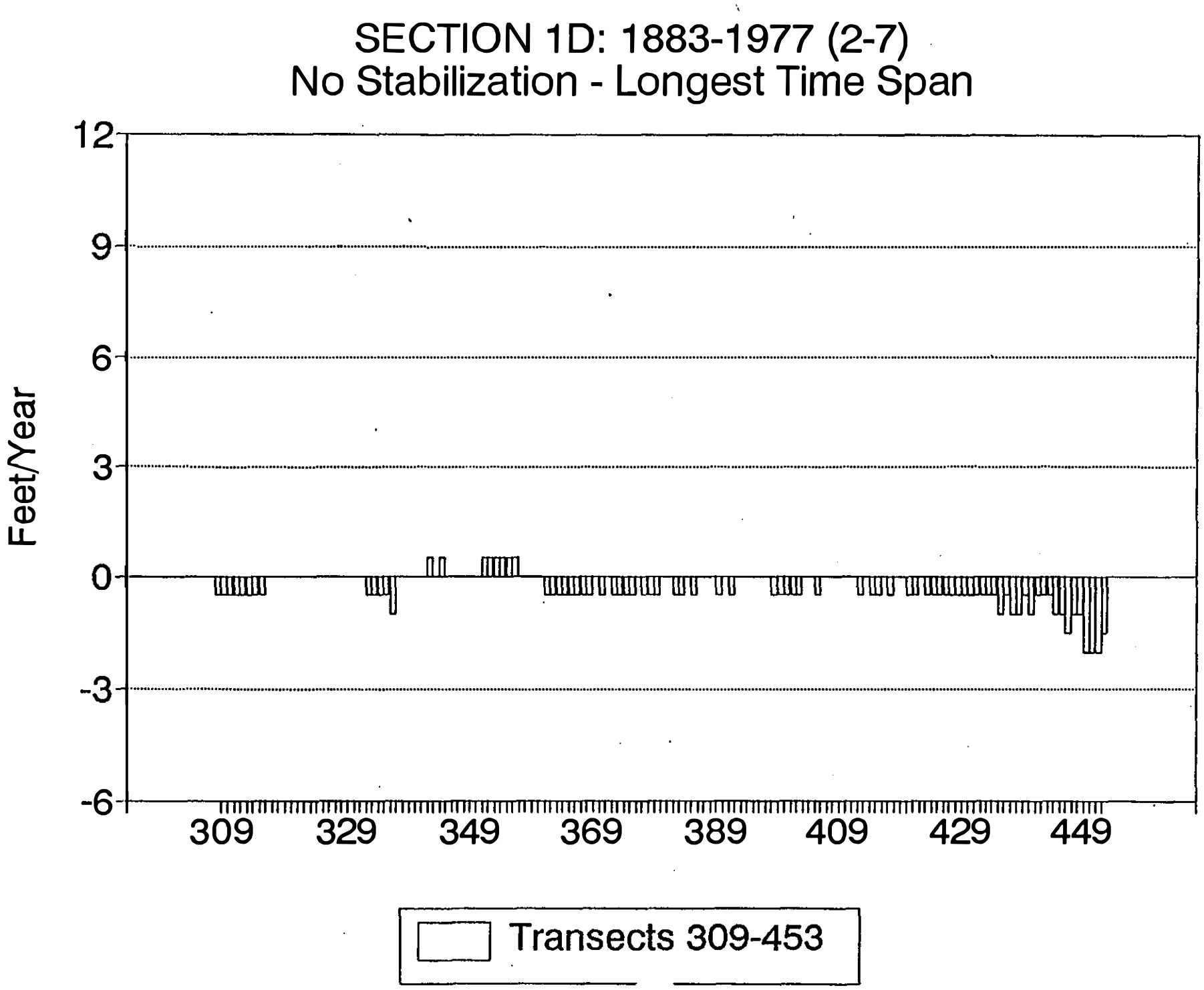
APPENDIX B
EROSION HISTOGRAMS
CAPE MAY COUNTY, NEW JERSEY
(after FEMA, 1993)

SECTION 1A: 1842-1932 (1-4)
No Stabilization - Longest Time Span




SECTION 1B: 1842-1986 (1-8)
No Stabilization - Longest Time Span
Feet/Year
12
9
6
3
0
-3
-6
133 143 153 163 173 183 193 203 213 223 233 243 253 263
Transects 133-265


SECTION 1C: 1883-1986 (2-8)
No Stabilization - Longest Time Span
Feet/Year
12
9
6
3
0
-3
-6
266
271
276
281
286
291
296
301
306
Transects 266-308


SECTION 1D: 1883-1977 (2-7)
No Stabilization - Longest Time Span
Feet/Year
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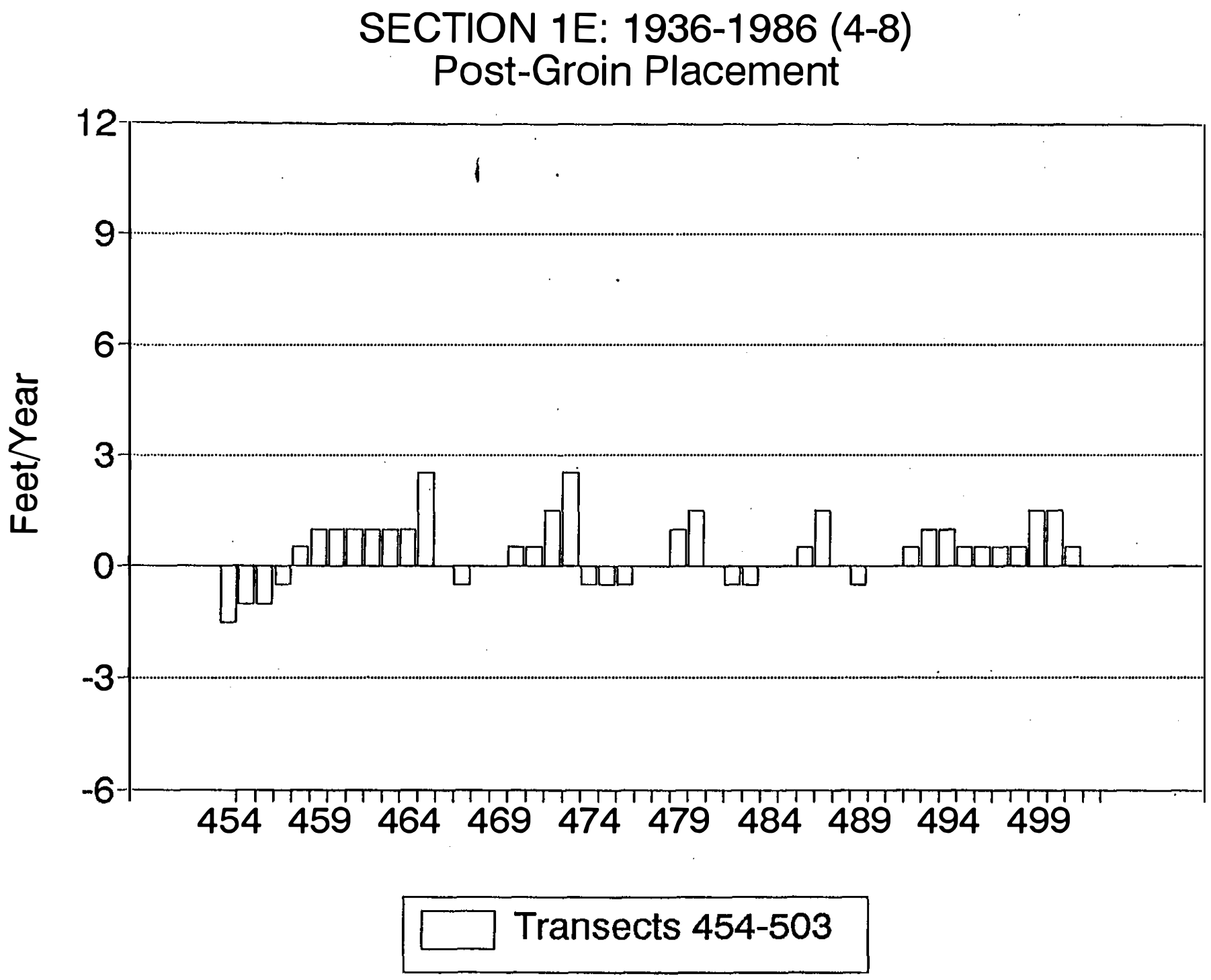
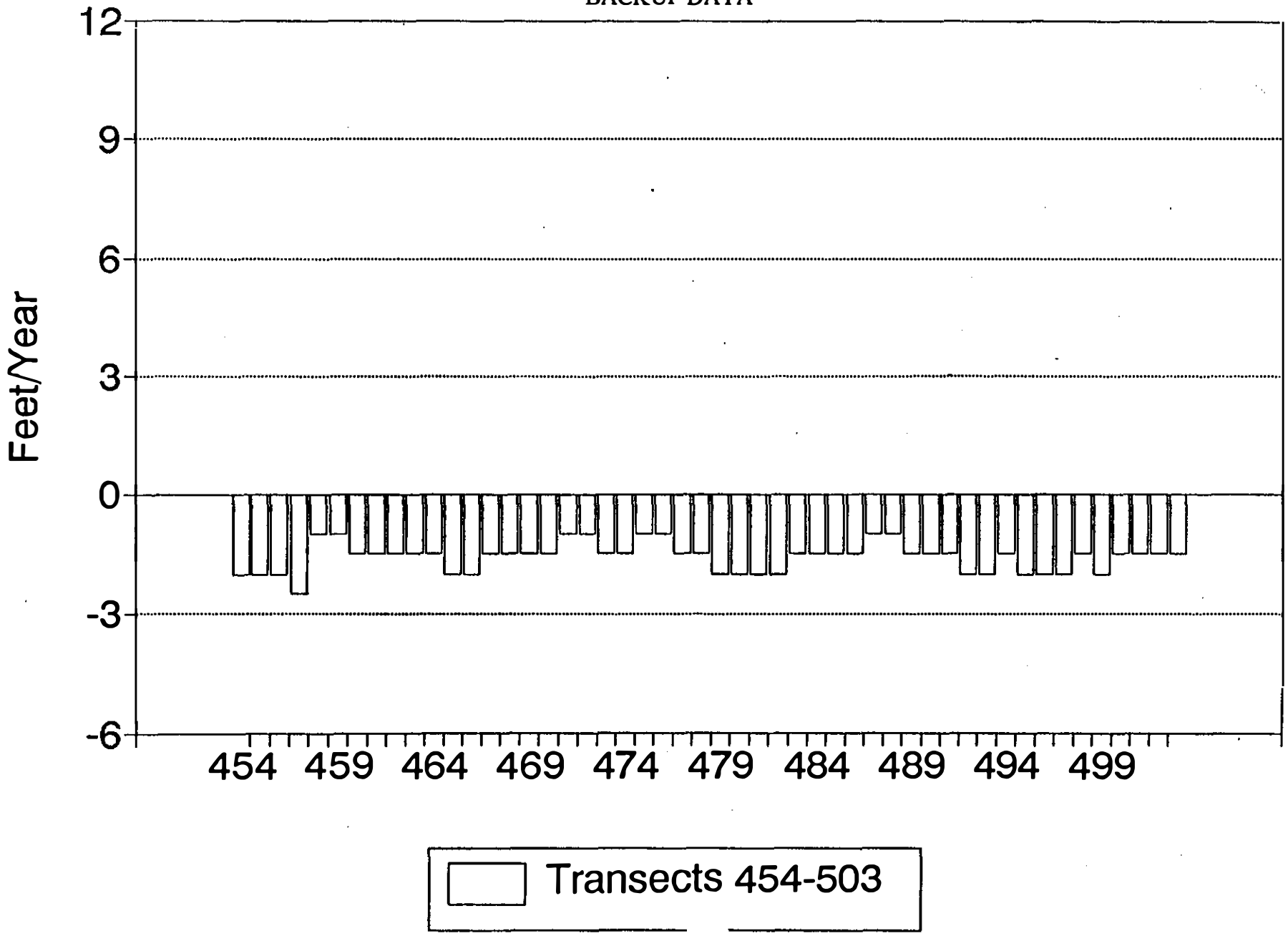
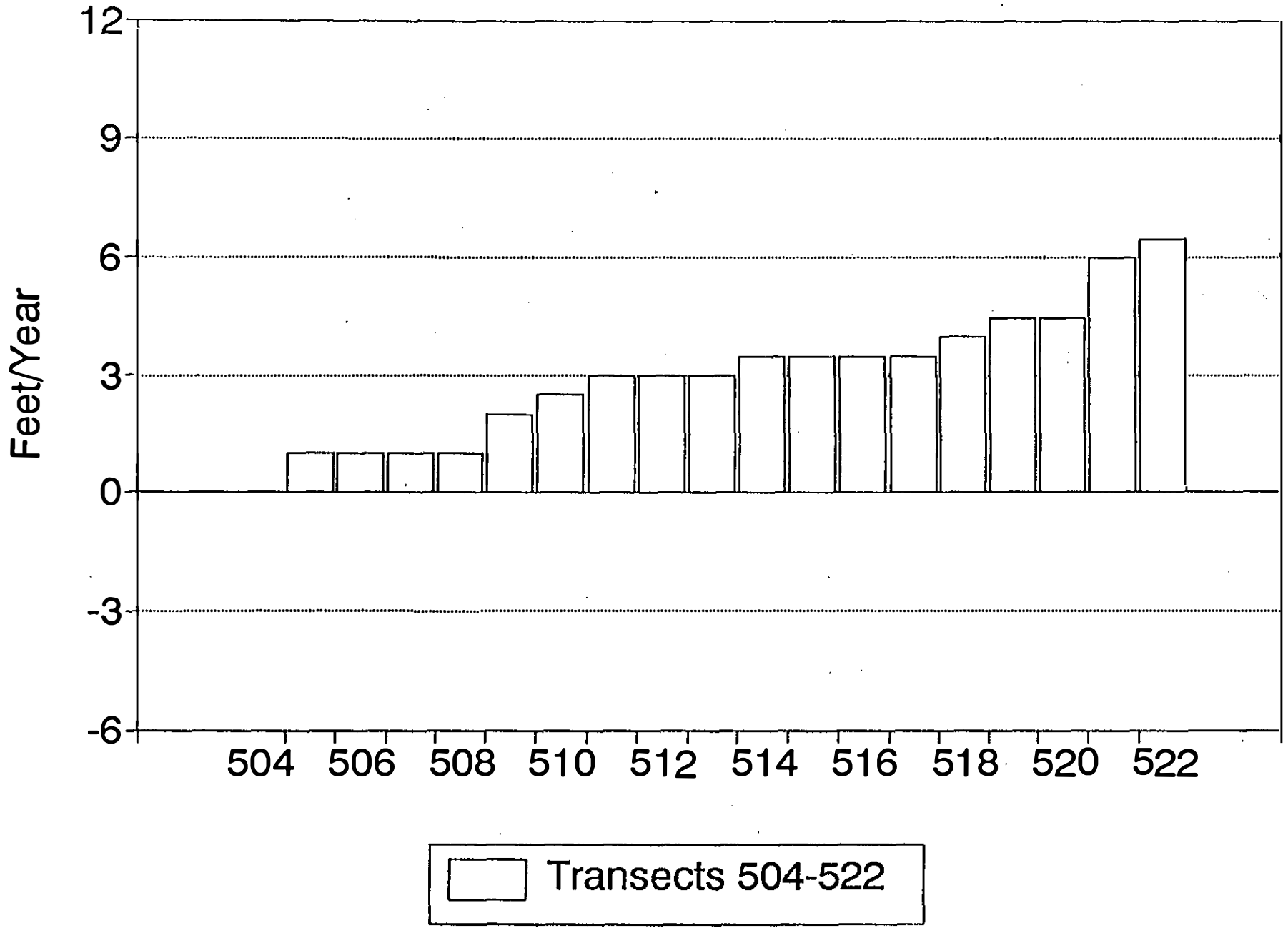
9
6
3
0
-3
-6
309
329
349
369
389
409
429
449
Transects 309-453


SECTION 1E: 1936-1986 (4-8)
Post-Groin Placement
Feet/Year
12
9
6
3
0
-3
-6
454
459
464
469
474
479
484
489
494
499
Transects 454-503

SECTION 1E: 1883-1936 (2-4)
Pre-Groin Period
BACKUP DATA

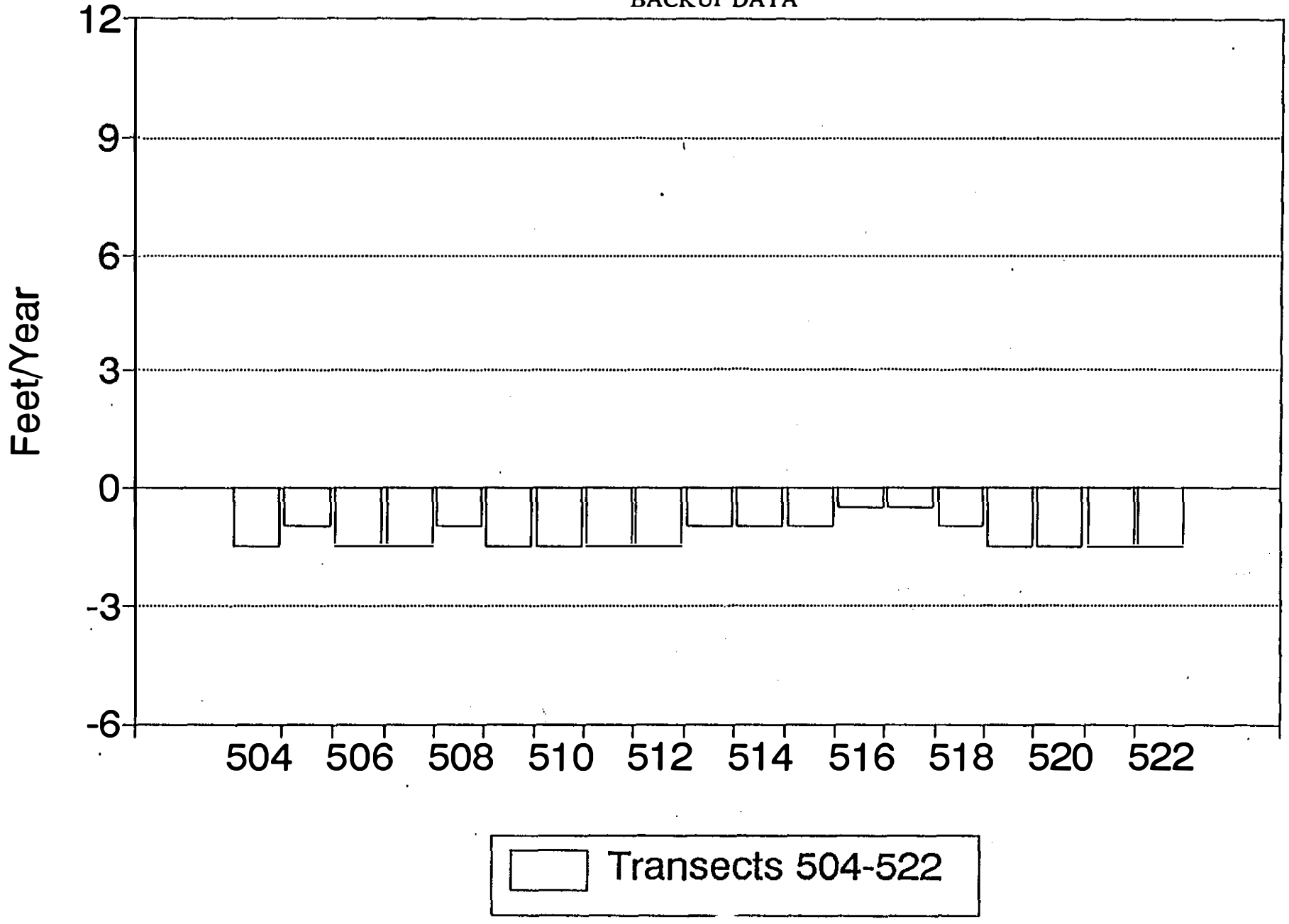


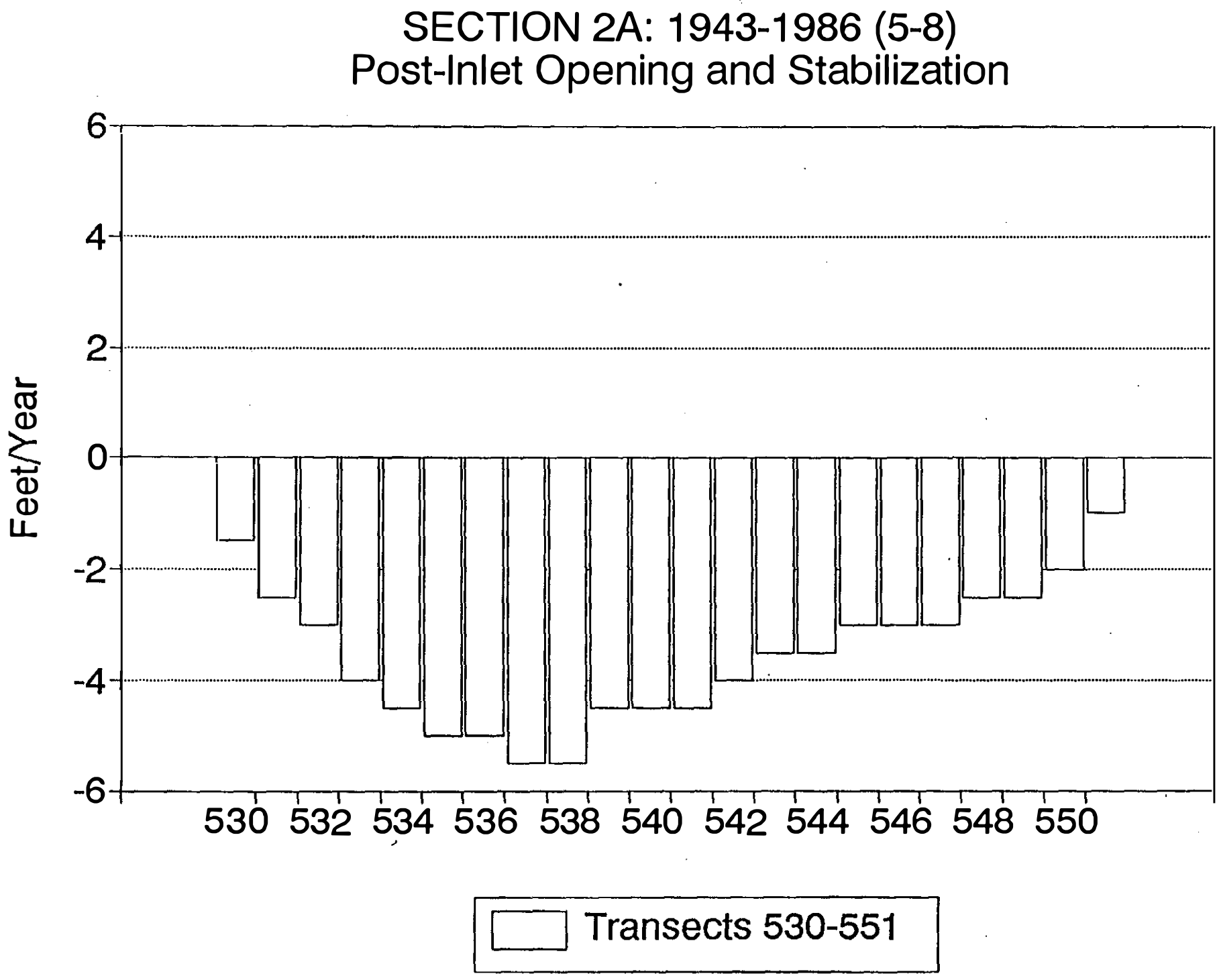
SECTION 1F: 1943-1986 (4-8)
Post-Inlet Opening and Stabilization



SECTION 1F: 1883-1936 (2-4)
Pre-Inlet Opening, No Stabilization

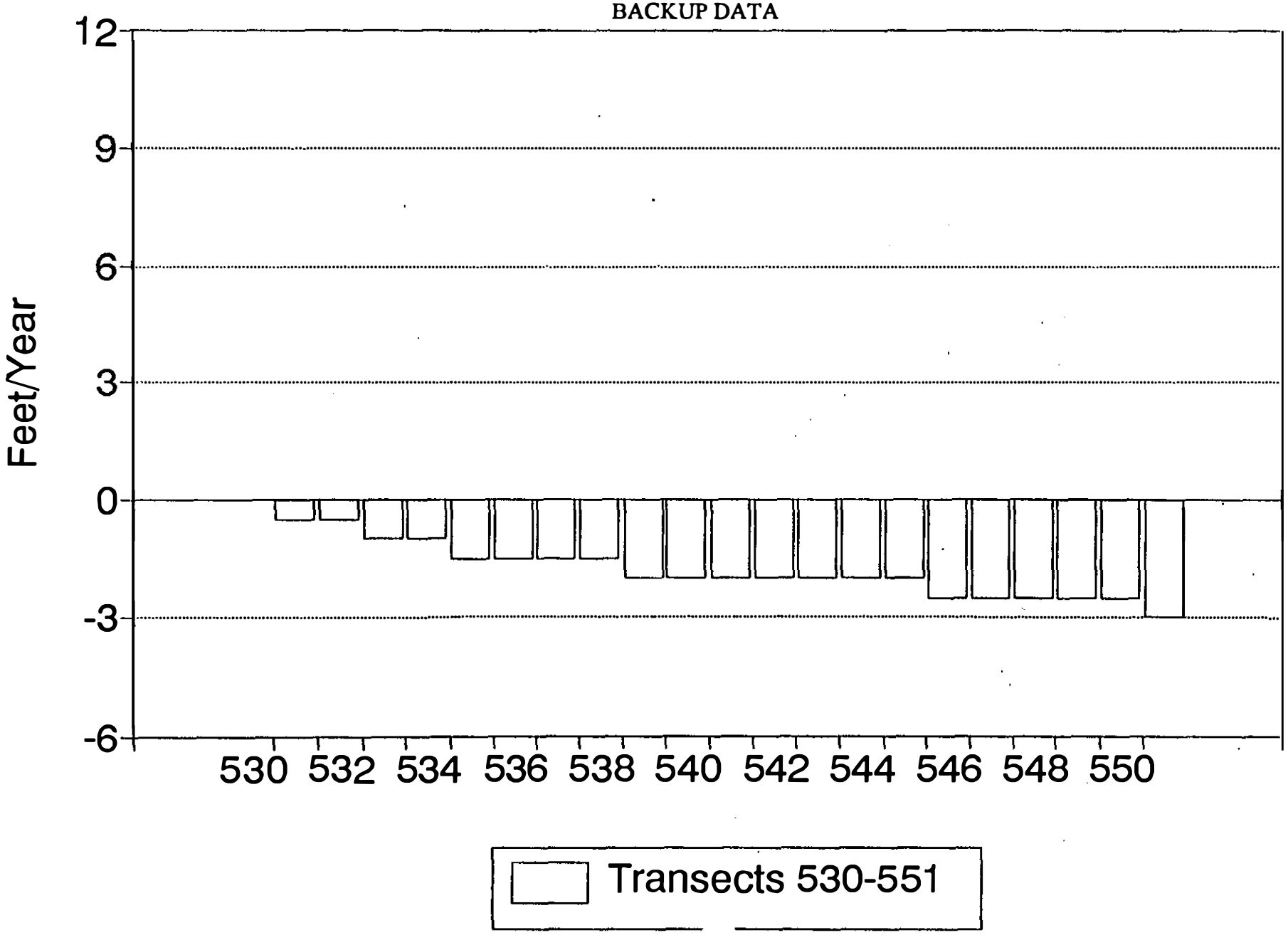
BACKUP DATA

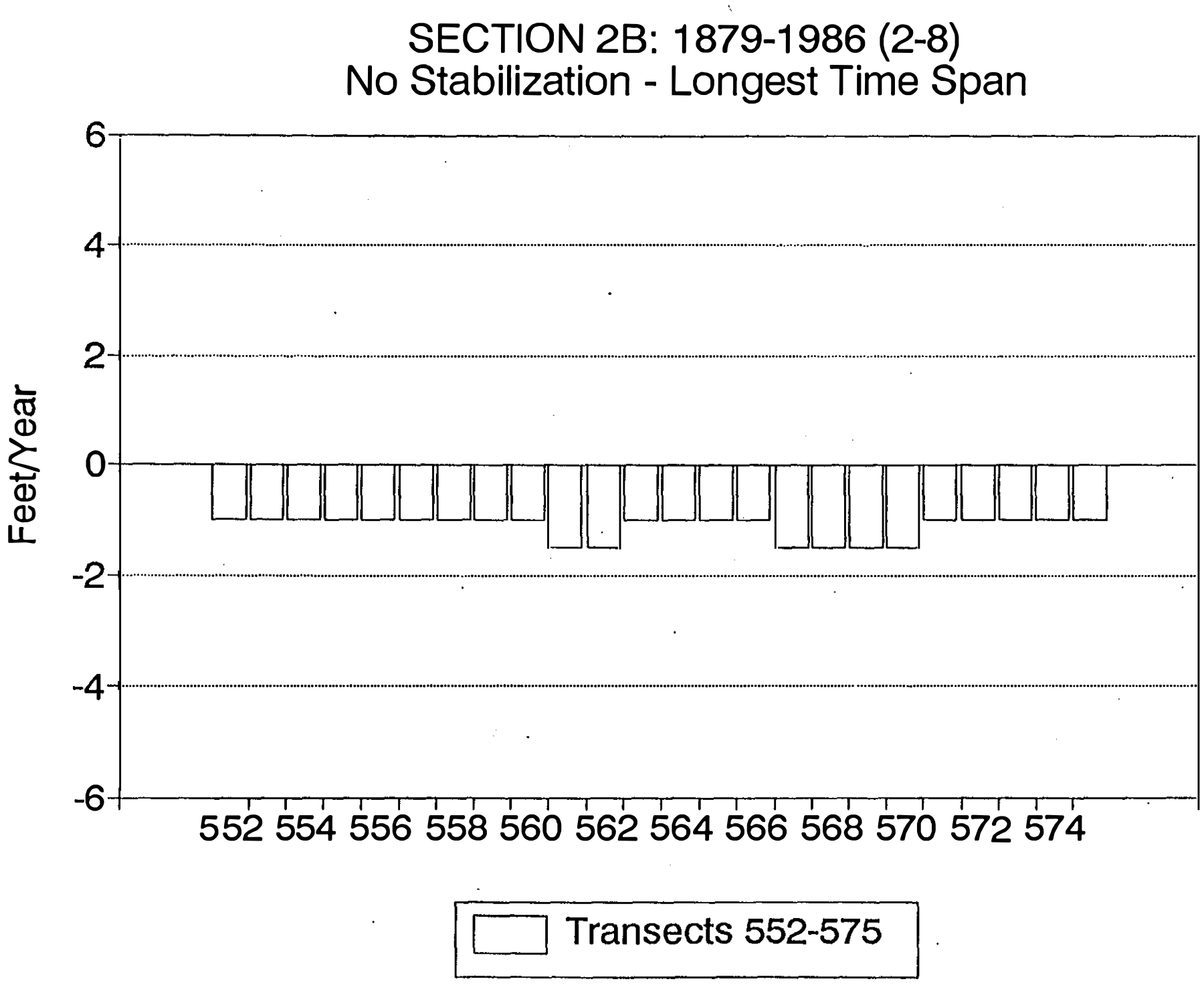



SECTION 2A: 1943-1986 (5-8)
Post-Inlet Opening and Stabilization
Feet/Year
6
4
2
0
-2
-4
-6
530
532
534
536
538
540
542
544
546
548
550
Transects 530-551

SECTION 2A: 1883-1936 (2-4)
Pre-Inlet Opening, No Stabilization

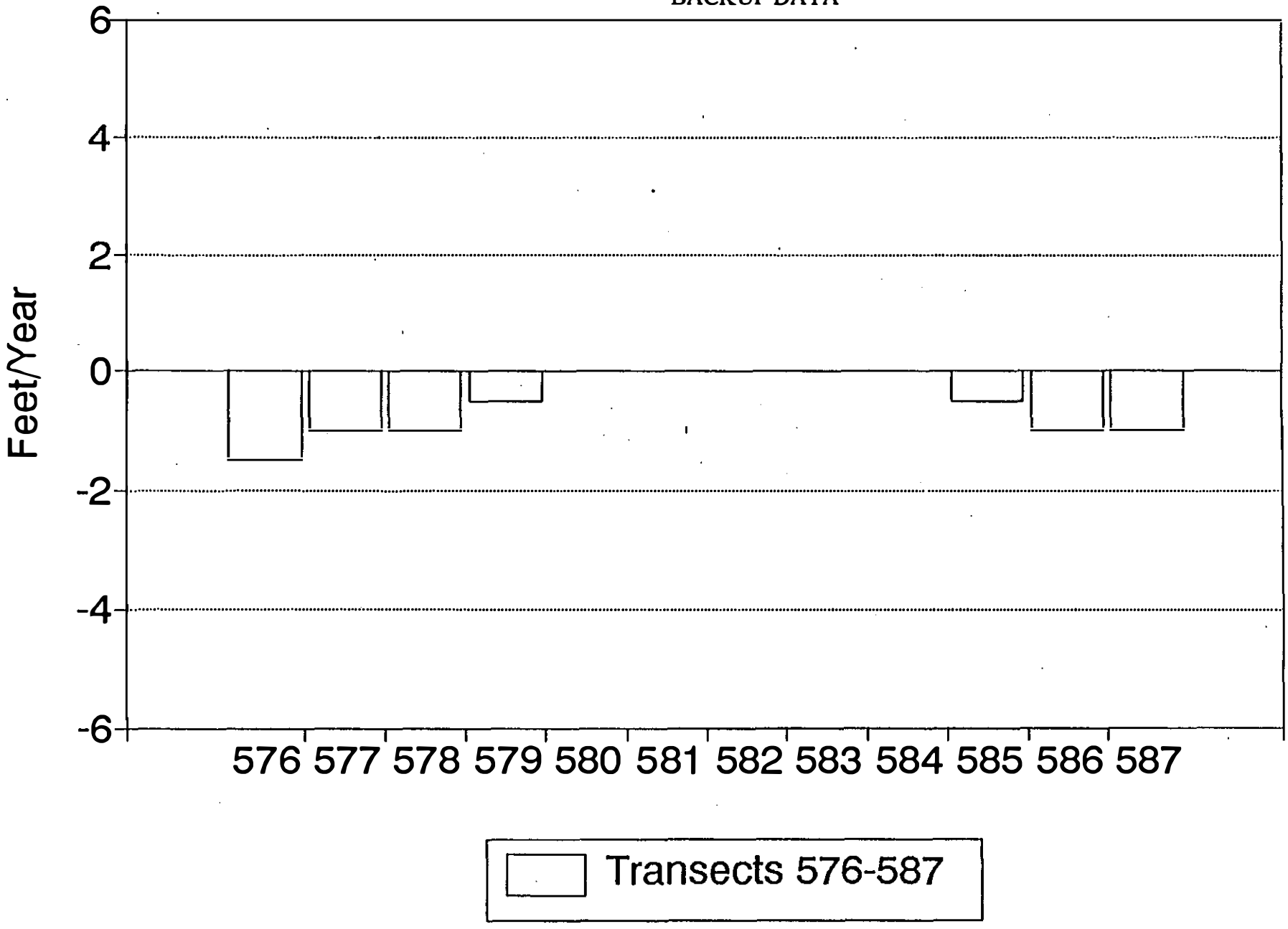
BACKUP DATA

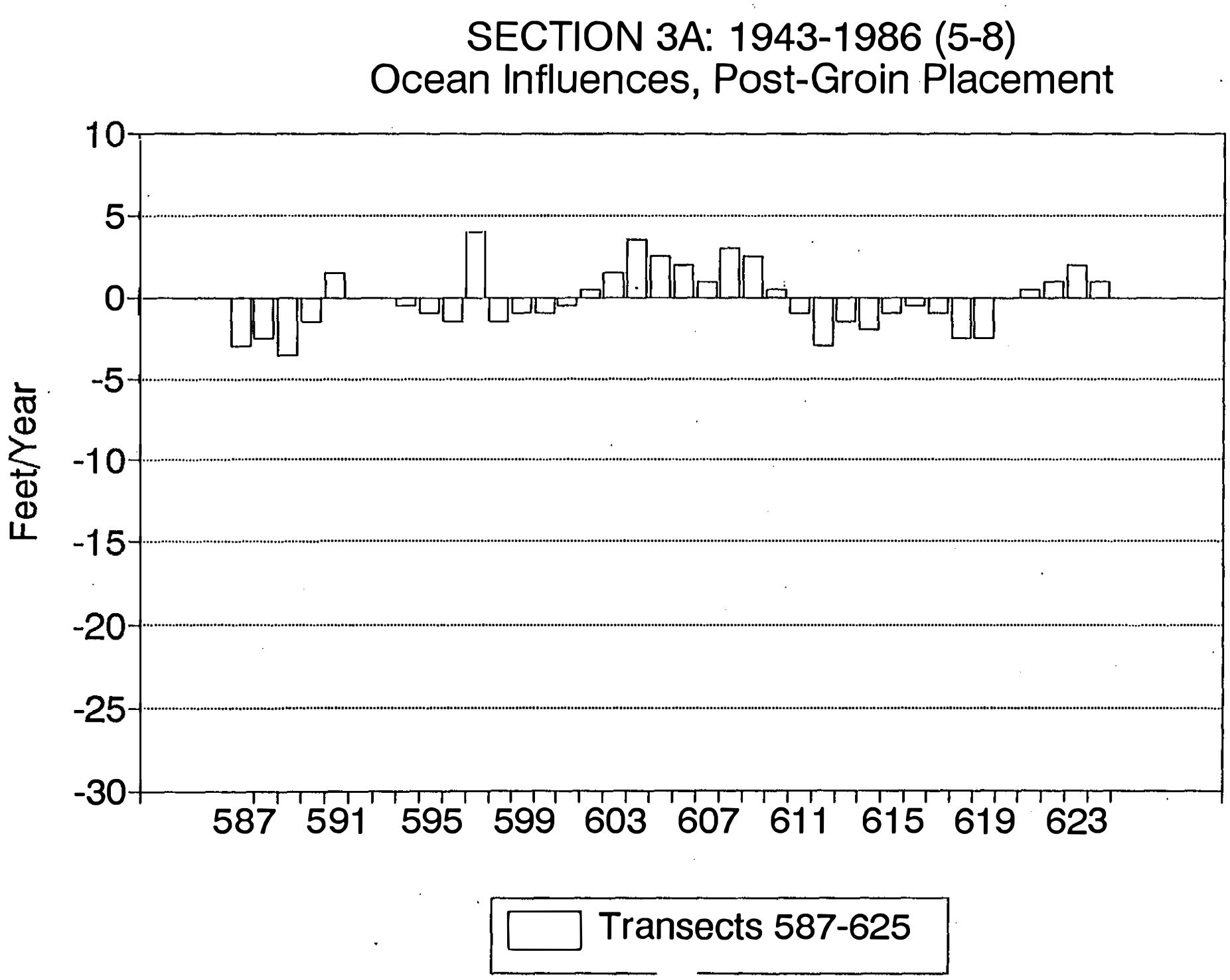



SECTION 2B: 1879-1986 (2-8)
No Stabilization - Longest Time Span
Feet/Year
6
4
2
0
-2
-4
-6
552 554 556 558 560 562 564 566 568 570 572 574
Transects 552-575

SECTION 2C: 1943-1986 (5-8)
Bay Influences, Post-Groin Placement
Feet/Year
6
4
2
0
-2
-4
-6
576
577
578
579
580
581
582
583
584
585
586
587
Transects 576-587

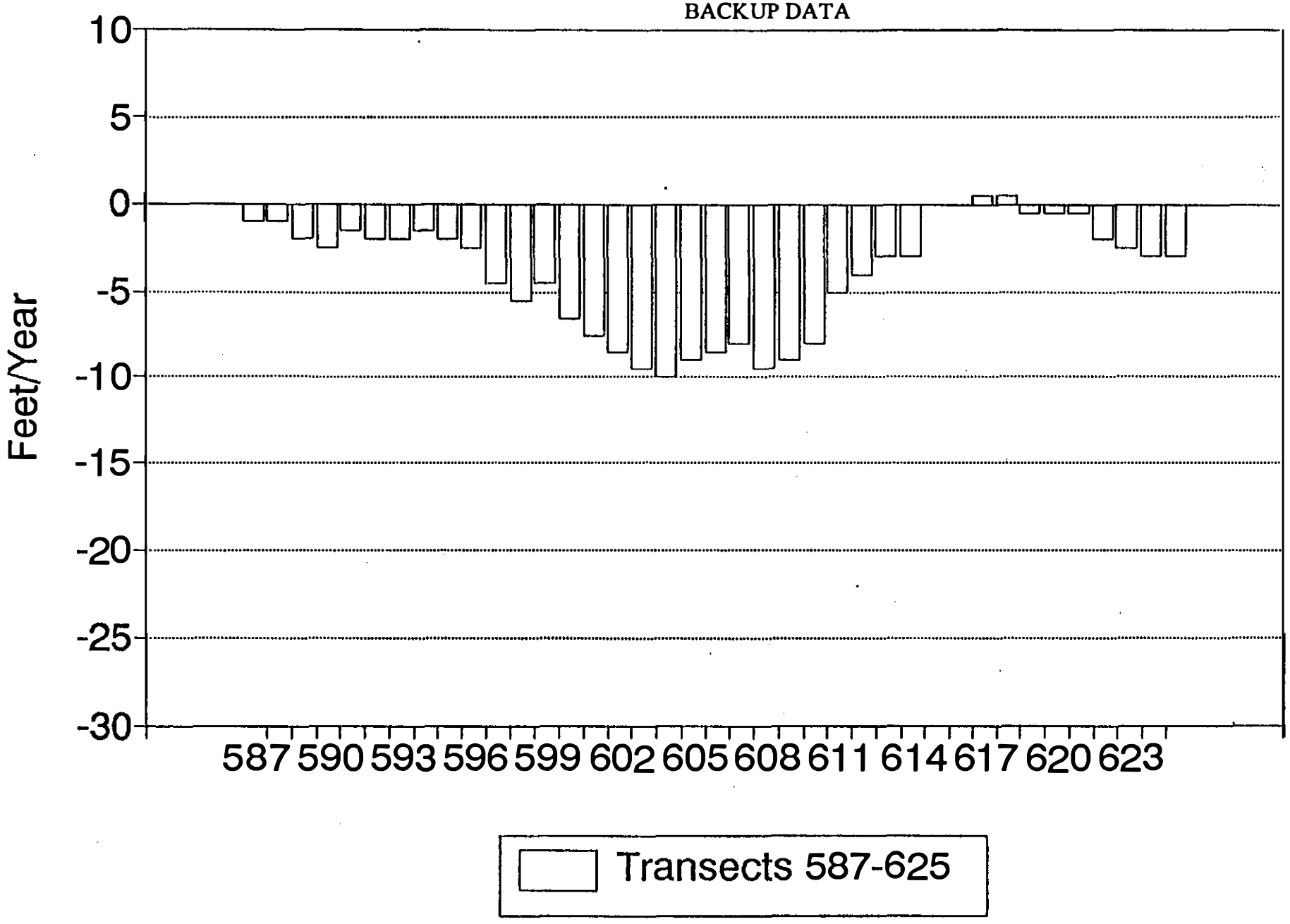
SECTION 2C: 1879-1936 (2-4)
Bay Influenced Area, Pre-Groin Period
BACKUP DATA




SECTION 3A: 1943-1986 (5-8)
Ocean Influences, Post-Groin Placement
Feet/Year
10
5
0
-5
-10
-15
-20
-25
-30
587
591
595
599
603
607
611
615
619
623
Transects 587-625

SECTION 3A: 1879-1936 (2-4)
Ocean Influences, Pre-Groin Placement

BACKUP DATA



PHASE I REPORT

BROADKILL BEACH
EROSION AND FLOODING PROTECTION

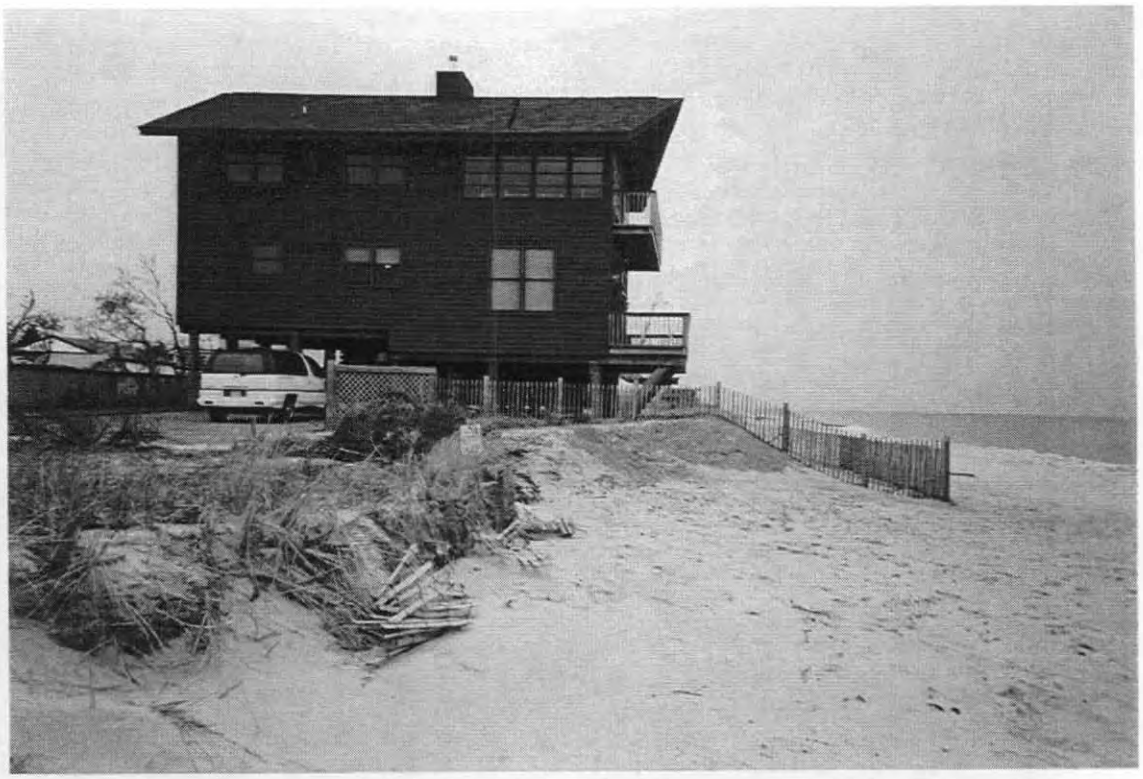
APPENDIX C
BROADKILL BEACH
FIELD PHOTOGRAPHS
APRIL, 1994

152

1/9 Cell 3, from the end of SR 16 looking northwest at the beach.

1/10 Cell 4, from the end of SR 16 looking southeast at the beach. The trailers can be seen on the far end.

1/11 Cell 3, looking at the landward end of the first timber groin north of SR 16. A dual dune system exists here with a lower front dune and a higher back dune.



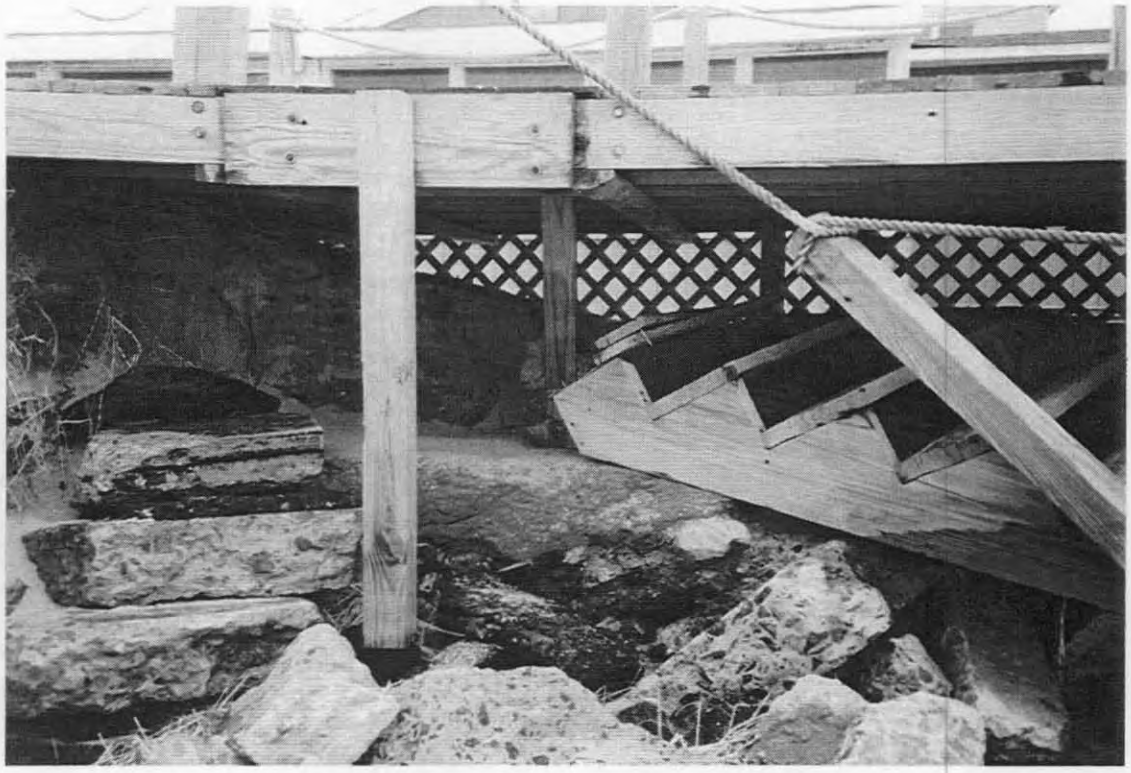
1/12 Cell 3, looking northwest at the house at the end of Maryland Ave. The new sand placed by the property owner after the winter '94 storms is visible. The resident reported that during an ice storm, the dune receded up to 75 feet. The state survey stake (Station N4+00) is visible behind the minivan.

1/14 Cell 3, looking at the landward end of the second timber groin north of SR 16.

1/17 Cell 3, newly placed sand and tree stumps in front of a house at the southeast end of W. Virginia Ave. Rubber stone protection can be seen to the left.

1/19 Cell 3, looking at the third groin north of SR 16, new sand and grass in front of the second house at northwest end of N. Carolina Ave.

1/20 Cell 3, the third house at the northwest end of N. Carolina Ave has no dune left.



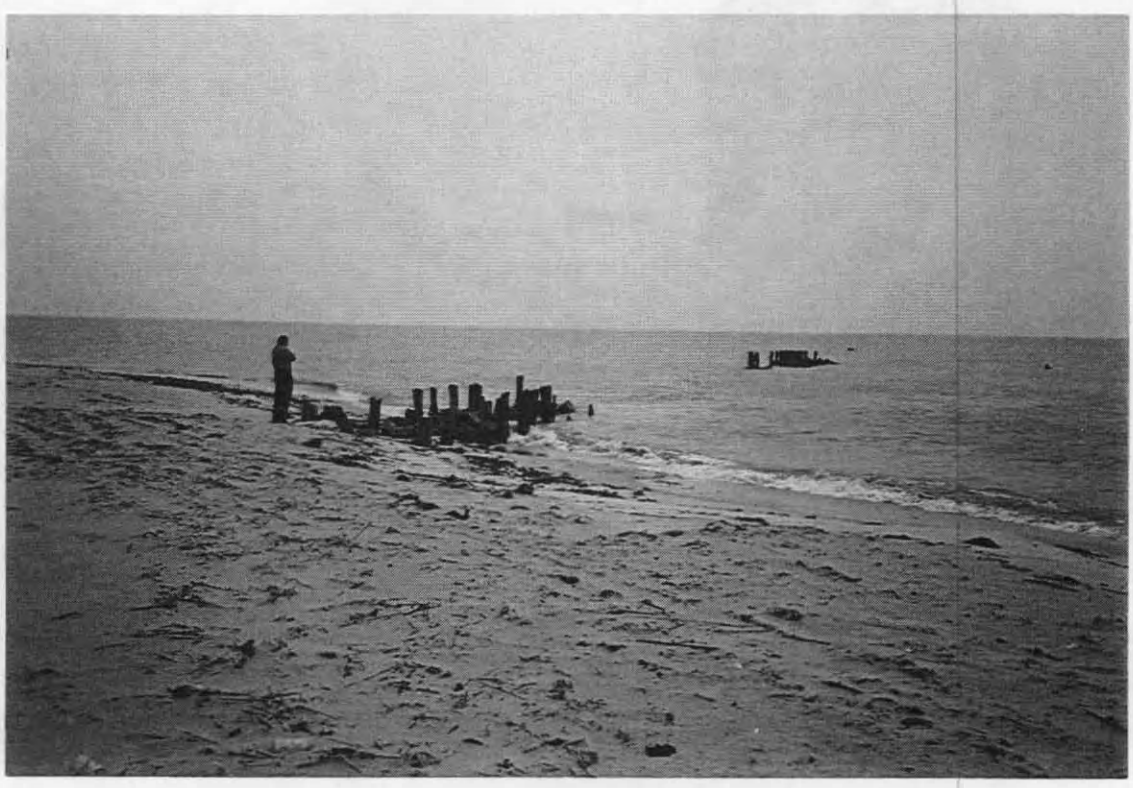
1/22 Cell 3, recent damage to the deck of the house located on the northwest side of Georgia Ave.

1/23 Cell 2, looking northwest at the beach from Cell 3.

1/24 Cell 3, between Georgia Ave. and Alabama Ave., looking southeast from fourth groin north of SR 16. Fresh tree stumps were dumped in front of dune for temporary protection.

2/1 Cell 8, looking southeast at the beach near the end of Broadkill Beach.

2/2 Cell 8, near the end of Broadkill Beach looking northwest at the beach in Cell 8.



2/3 Cell 8, looking at the old jetty during high tide.

2/4 Cell 7, from the dome shape house at the northwest end of the old jetty. Fresh tree stumps were recently placed to protect the dunes. Dune scrap can be seen on the left of the picture.

2/6 Cell 8, dune eroded under the steps southwest of the old jetty about 600 feet southeast from the south end of Cell 7.

2/7 Cell 6, in front of a hexagonal shaped house, looking northwest, storm swash debris can be seen on the dune.

2/8 Cell 6, same location as photo 2/7, looking southeast at the beach.

2/9 Cell 5, south of abandoned elevated trailer, a small dune and protective planting installed by resident in front of a house about 560' northwest of Cell 6.

2/11 Cell 5, in the center of the cell, recently placed sand dune.

2/13 Cell 4, tree stump protection in front of a house just southeast of the old trailer north of Cell 5.

2/15 Cell 4, looking at eroded dune southeast of Monroe Ave.

2/20 Cell 4, low dune in front of the houses between SR16 and Adams Ave.

2/21 Cell 4, an assortment of shore protection in front of a house 190' northwest of Adams Ave.

PHASE I REPORT

BROADKILL BEACH
EROSION AND FLOODING PROTECTION

APPENDIX D
DATA APPENDIX

Shoreline Position Data

Measured MHW shoreline positions. Positions are referenced to DNREC baseline.

			Cell 1					Cell 2				
	N48+00	N43+00	N40+00	N36+00		N33+00	N28+00	N24+00	N20+00			
1954.54	-103	-99	-109	-98		-58	-24	28	131	1954.54		
1964.30	38	63	59	102		107	113	139	135	1964.30		
1971.63	26	53	76	82		80	42	21	10	1971.63		
1977.39	74	78	91	116		68	95	118	163	1977.39		
1979.22	85	132	155	140		154	177	116	110	1979.22		
1980.89	84	155	151	116		123	102	60	88	1980.89		
1989.48	202	223	229	204		170	162	140	119	1989.48		
1990.31	189	223	225	183		199	166	130	147	1990.31		
1993.32	233	280	244	241		224	184	132	96	1993.32		
			Cell 3							Cell 4		
	N16+00	N12+00	N7+00	N4+00	0+00		S4+00	S8+00	S12+00	S16+00	S19+00	S22+00
1954.54	101	79	49	47	129		110	112	76	80	119	176
1964.30	77	76	104	106	117		107	121	114	136	180	253
1971.63	58	-39	53	44	42		42	47	64	103	160	155
1977.39	117	107	109	116	165		188	225	174	123	104	131
1979.22	99	76	159	128	205		256	225	152	80	53	67
1980.89	105	104	126	170	235		262	228	153	80	53	67
1989.48	153	174	126	221	263		265	266	238	169	151	113
1990.31	113	116	190	153	199		205	206	171	120	104	151
1993.32	67	63	86	101	153		153	149	127	80	64	75
		Cell 5					Cell 6					
	S24+00	S28+00	S32+00		S36+00	S40+00	S44+00	S49+00	S52+00	S56+00	S60+00	
1954.54	198	176	216		211	196	223	243	261	256	280	
1964.30	227	132	111		75	39	126	56	49	58	86	
1971.63	137	15	37		17	29	14	15	3	8	6	
1977.39	133	103	153		159	192	212	151	116	70	93	
1979.22	65	81	183		237	259	242	189	171	153	161	
1980.89	85	105	238		260	274	256	179				
1989.48	167	143	199		198	196	217	214	195	156	152	
1990.31	124	81	112		108	119	144	141	102	67	77	
1993.32	91	54	95		98	125	147	139	101	74	80	

			Cell 7									
	S64+00	S69+00	S72+00	S76+00								
1954.54	269	261	263	273								
1964.30	100	70	62	80								
1971.63	11	1	4	-4								
1977.39	118	114	100	102								
1979.22	188	154	145	146								
1989.48	159	127	143	132								
1990.31	105	89	85	113								
1993.32	105	87	86	85								
			Cell 8									
	C8-1	C8-2	C8-3	C8-4	C8-5							
1954.54	160	66	98	100	64							
1964.30	117	105	82	53	23							
1971.63	111	92	82	60	40							
1977.39	147	127	99	83	92							
1979.22	168	142	121	96	102							
1989.48	190	157	160	161	156							
1990.31	135	123	123	127	139							
1993.32	142	123	119	129	153							

MHW shoreline positions calculated from profile survey data. Positions referenced to DNREC baseline.

				N48						N14
76	16	2	1976.126	63.4		73	12	1	1973.033	34.5
77	3	5	1977.337	74.8		73	27	12	1973.978	125.7
78	2	5	1978.334	75.5		78	11	4	1978.277	73.1
79	18	7	1979.542	84.8		87	11	12	1987.934	114.4
				N43		93	24	5	1993.395	56.3
76	25	2	1976.151	106.6		93	7	7	1993.512	61.0
90	27	11	1990.896	211.7		93	23	9	1993.721	58.8
93	7	7	1993.512	305.9						N12
93	23	9	1993.721	361.5		73	27	12	1973.978	118.3
				N40		78	11	4	1978.277	76.1
76	25	2	1976.151	91.8		87	11	12	1987.934	112.1
77	2	5	1977.334	100.7		88	26	7	1988.564	85.8
78	2	5	1978.334	118.1						N7
79	18	7	1979.542	136.9		64	30	5	1964.411	55.5
				N36		69	30	5	1969.411	95.7
76	25	2	1976.151	106.8		73	12	1	1973.033	70.3
77	2	5	1977.334	117.2		73	27	12	1973.978	112.2
78	2	5	1978.334	96.4		90	27	11	1990.896	100.3
79	18	7	1979.542	136.9		93	7	7	1993.512	76.9
				N33		93	23	9	1993.721	74.4
73	12	1	1973.033	44.7						N4
76	24	2	1976.148	82.2		73	12	1	1973.033	67.4
93	7	7	1993.512	210.5		73	27	12	1973.978	121.4
93	23	9	1993.721	203.2		78	11	4	1978.277	115.9
				N28		87	11	12	1987.934	151.6
73	12	1	1973.033	9.8		88	26	7	1988.564	150.2
74	26	12	1974.975	114.3		93	24	5	1993.395	102.7
76	23	2	1976.145	109.8						0+00
78	11	4	1978.277	136.9		73	1	8	1973.578	85.7
				N24		86	28	2	1986.159	144.6
73	12	1	1973.033	42.0						S4
74	26	12	1974.975	146.5		73	1	8	1973.578	67.7
76	26	4	1976.318	112.1		74	17	4	1974.293	136.8
				N20		86	28	2	1986.159	230.5
54	30	5	1954.411	226.9						S8
64	30	5	1964.411	90.4		73	1	8	1973.578	80.6
69	30	5	1969.411	72.5		86	28	2	1986.159	200.9
73	12	1	1973.033	49.8						S12
73	27	12	1973.978	119.5		73	1	8	1973.578	64.1
78	11	4	1978.277	94.9		80	2	5	1980.334	129.1
90	27	11	1990.896	106.5		86	28	2	1986.159	171.3
93	7	7	1993.512	114.0		88	9	8	1988.6	201.9
93	23	9	1993.721	92.9						S14
				N16		73	1	8	1973.578	70.9
73	12	1	1973.033	66.5		80	2	5	1980.334	89.0
73	27	12	1973.978	137.3		86	28	2	1986.159	84.3
74	26	12	1974.975	140.5		88	9	8	1988.6	152.9
78	11	4	1978.277	89.7		93	9	7	1993.518	107.1
87	11	12	1987.934	123.8		93	23	9	1993.721	104.9
88	26	7	1988.564	85.8						

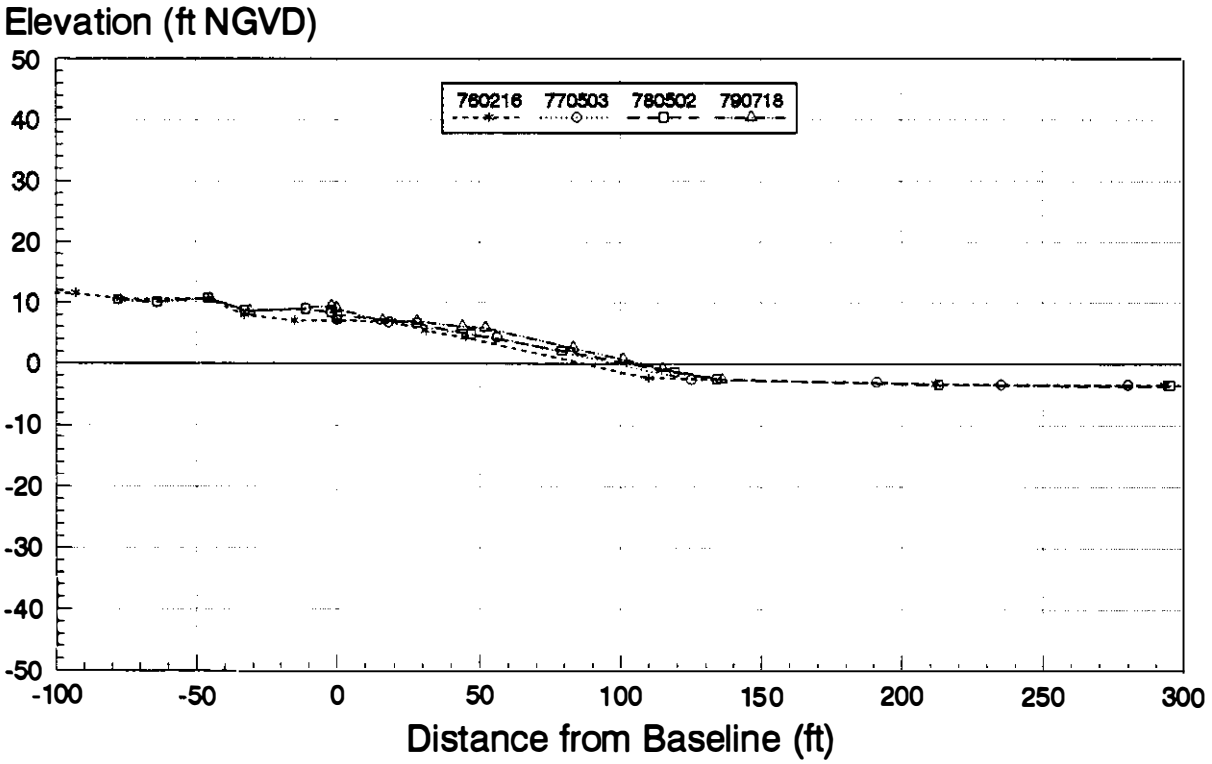
199

				S19						S49
73	1	8	1973.578	78.5	73	1	8	1973.578		41.0
75	20	2	1975.137	67.1	75	25	3	1975.233		54.1
77	1	4	1977.249	91.5	90	27	11	1990.896		136.0
78	17	5	1978.375	56.6	93	7	7	1993.512		147.0
80	2	5	1980.334	10.1	93	23	9	1993.721		147.9
86	20	11	1986.877	34.8						S52
88	2	10	1988.745	102.2	73	1	8	1973.578		23.8
				S22	75	25	3	1975.233		39.8
73	1	8	1973.578	52.3	77	1	4	1977.249		112.9
74	13	12	1974.94	58.8	78	30	5	1978.411		118.9
77	1	4	1977.249	67.4						S69
78	17	5	1978.375	45.5	73	3	2	1973.09		4.6
80	2	5	1980.334	36.6	90	27	11	1990.896		99.7
86	20	11	1986.877	57.6	93	9	7	1993.518		93.2
				S24	93	23	9	1993.721		88.9
77	1	4	1977.249	67.6						S76
78	17	5	1978.375	40.7	73	3	2	1973.09		42.1
86	20	11	1986.877	77.0	78	30	5	1978.411		92.9
				S28	79	25	6	1979.479		107.8
74	13	12	1974.94	45.9						S6
78	17	5	1978.375	72.0	64	30	5	1964.41		45.0
79	16	8	1979.619	76.7	69	30	5	1969.41		80.1
80	2	5	1980.334	72.9	73	1	8	1973.58		76.8
86	20	11	1986.877	79.3	86	28	2	1986.16		236.7
88	30	7	1988.575	137.9	90	27	11	1990.90		182.0
93	21	9	1993.715	42.0	93	7	7	1993.51		152.2
				S32	93	23	9	1993.72		162.7
73	1	8	1973.578	32.1						
75	20	2	1975.137	44.9						
77	1	4	1977.249	169.8						
78	30	5	1978.411	170.6						
79	17	8	1979.622	160.9						
93	21	9	1993.715	105.8						
				S36						
73	1	8	1973.578	23.9						
75	20	2	1975.137	36.7						
77	1	4	1977.249	141.1						
78	23	5	1978.392	187.1						
93	7	7	1993.512	142.0						
93	23	9	1993.721	123.9						
				S40						
73	1	8	1973.578	44.9						
75	20	2	1975.137	44.3						
77	1	4	1977.249	173.1						
78	25	5	1978.397	182.1						
				S44						
73	1	8	1973.578	47.5						
75	27	2	1975.156	48.7						
77	1	4	1977.249	196.0						
78	25	5	1978.397	229.1						

200

Beach Profile Data

Broadkill Beach Profile N48+00



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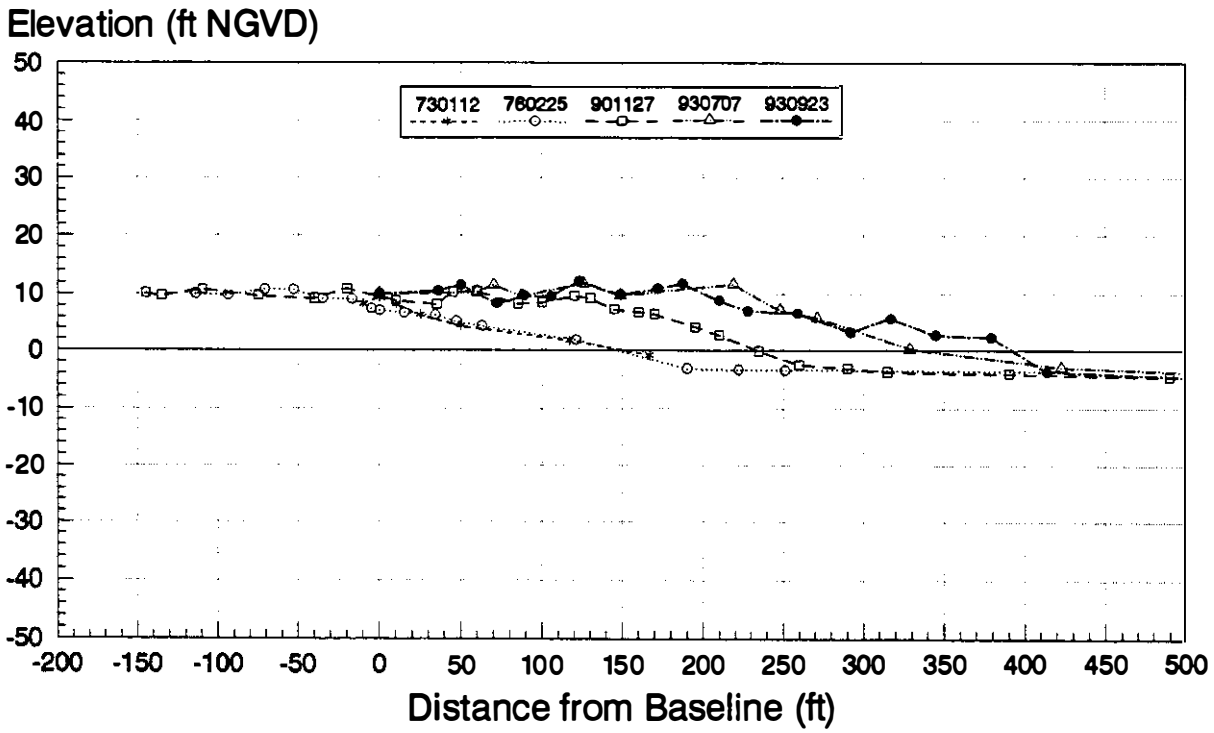

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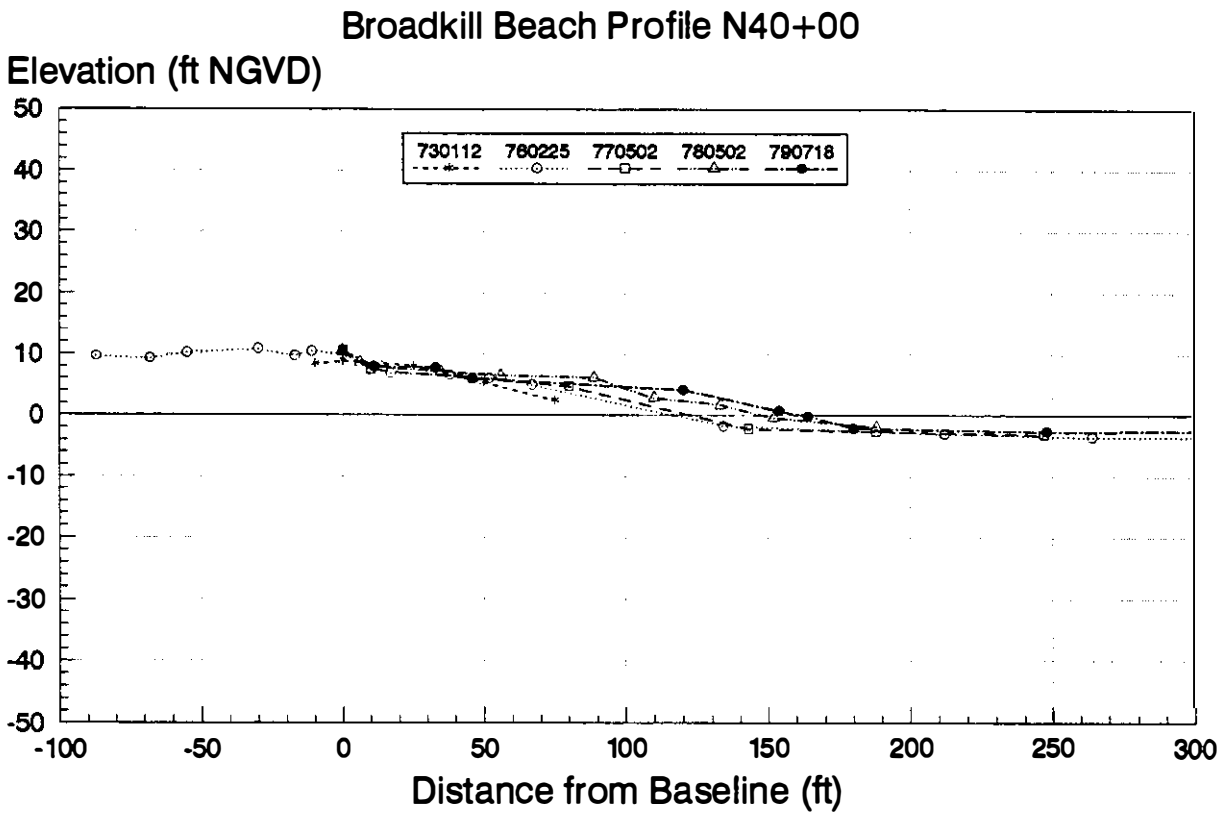
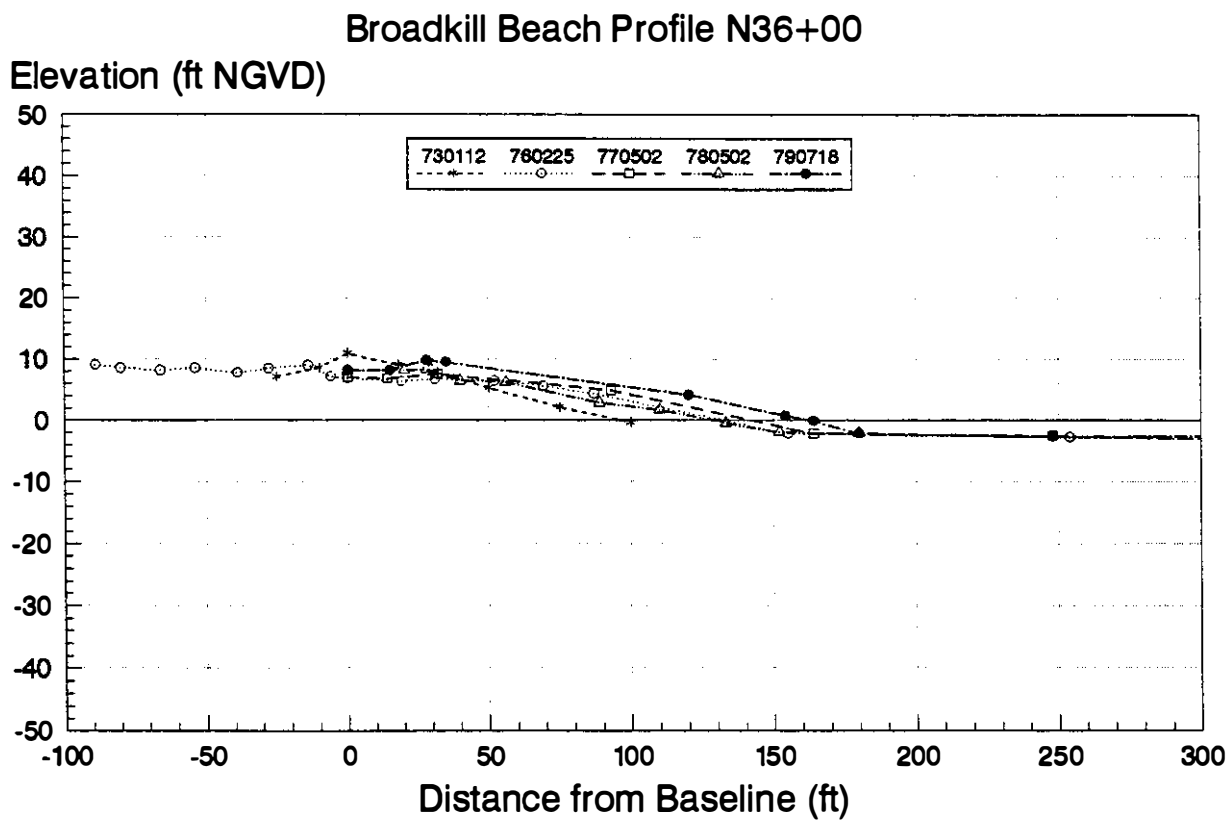
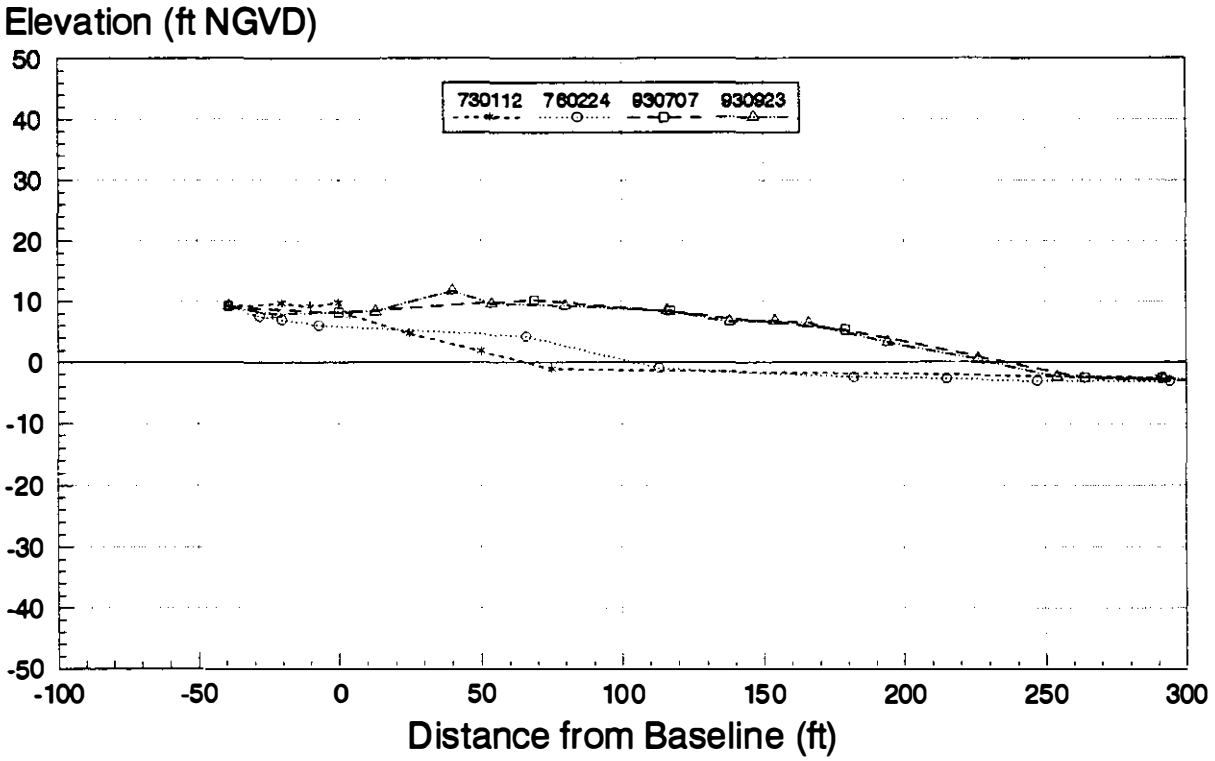
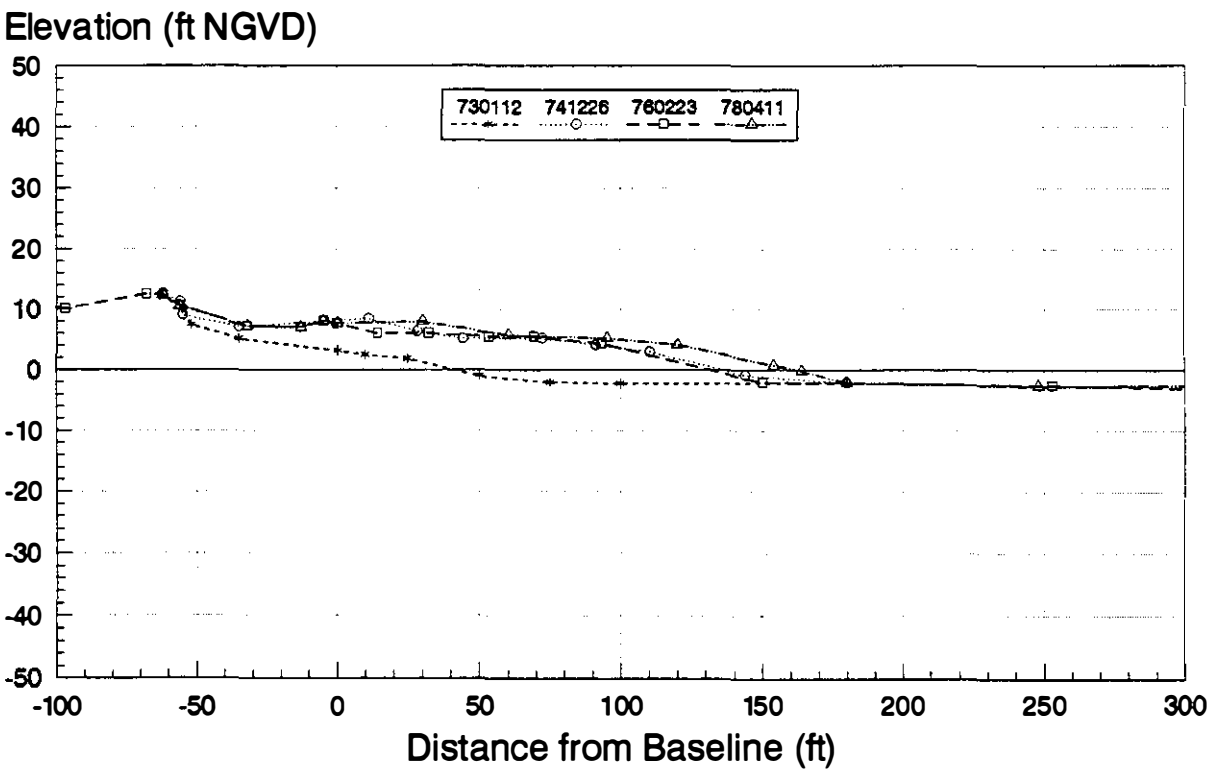

Broadkill Beach Profile N40+00
Elevation (ft NGVD)
730112
760225
770502
780502
790718
50
40
30
20
10
0
-10
-20
-30
-40
-50
-100
-50
0
50
100
150
200
250
300
Distance from Baseline (ft)


Broadkill Beach Profile N36+00
Elevation (ft NGVD)
730112
760225
770502
780502
790718
50
40
30
20
10
0
-10
-20
-30
-40
-50
-100
-50
0
50
100
150
200
250
300
Distance from Baseline (ft)

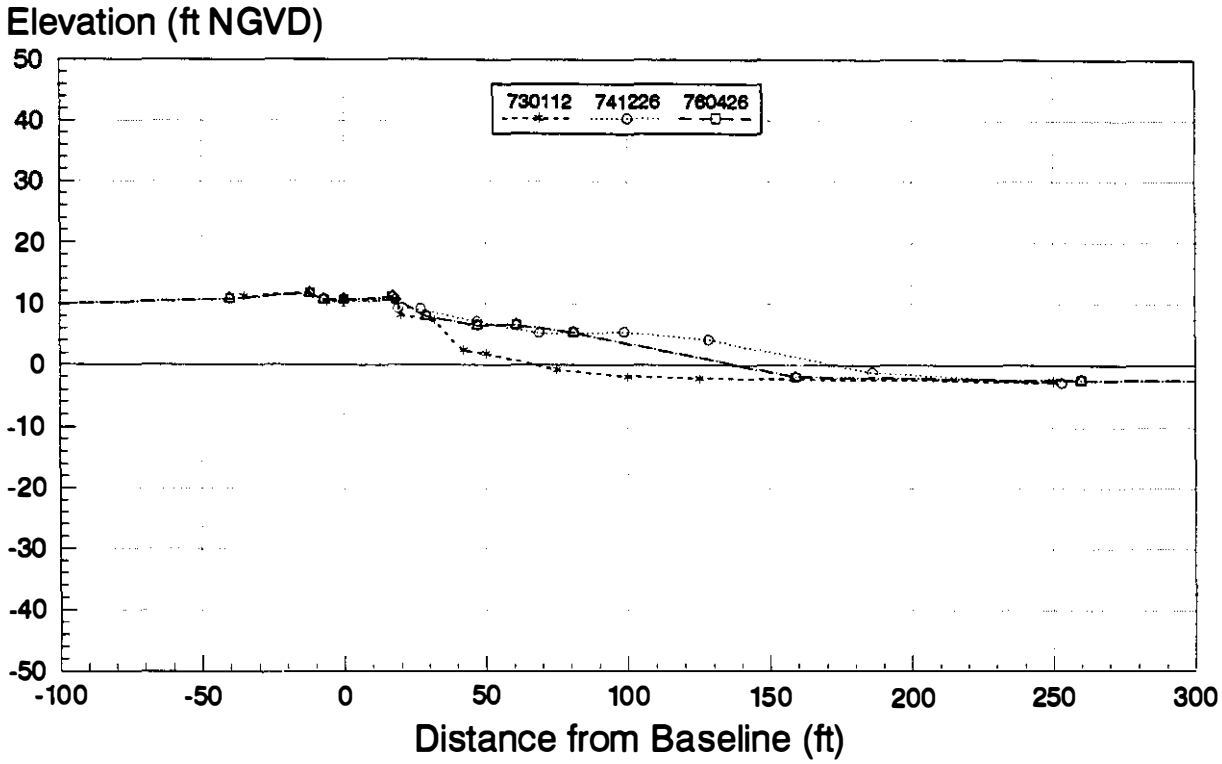
Broadkill Beach Profile N33+00



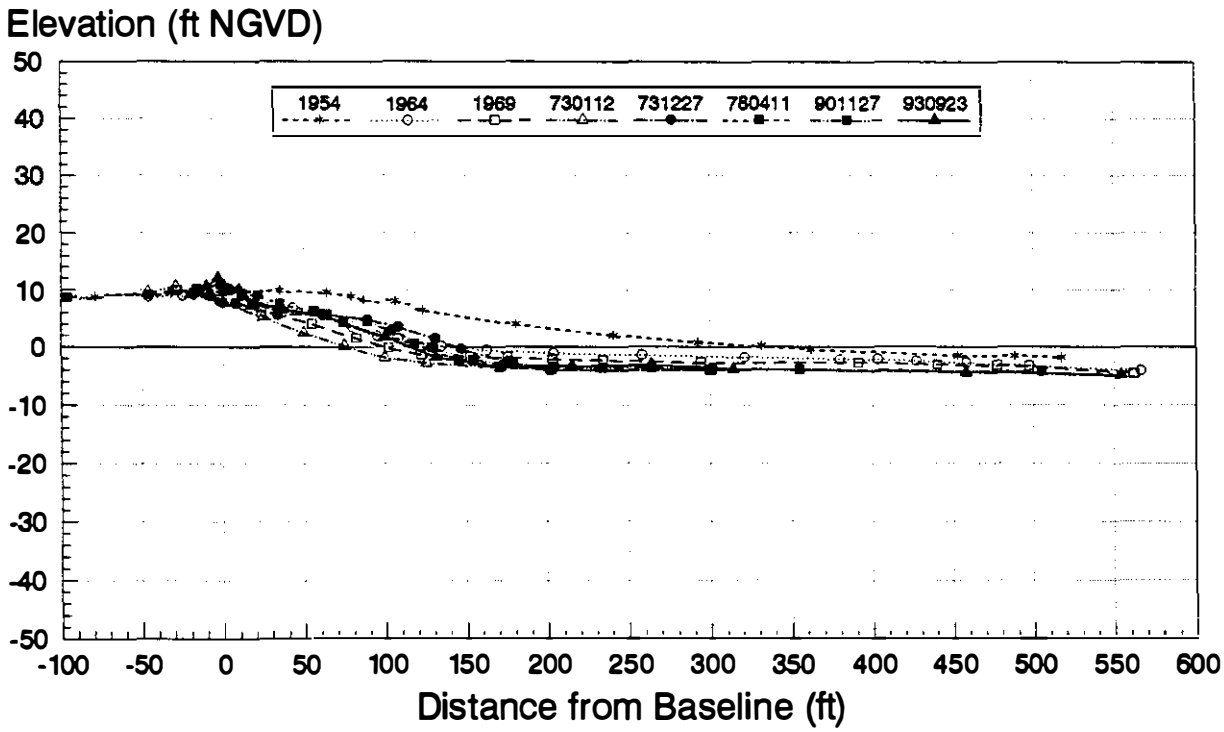
Broadkill Beach Profile N28+00



Broadkill Beach Profile N24+00

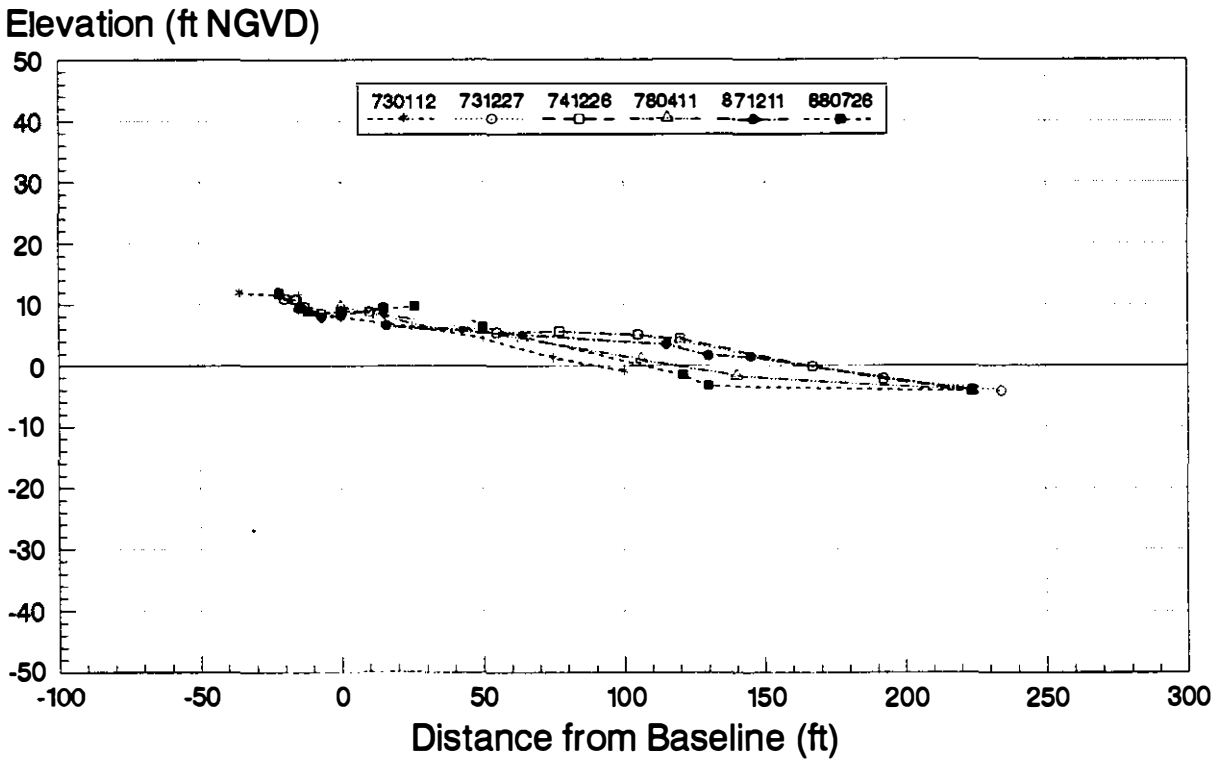


Broadkill Beach Profile N21+00

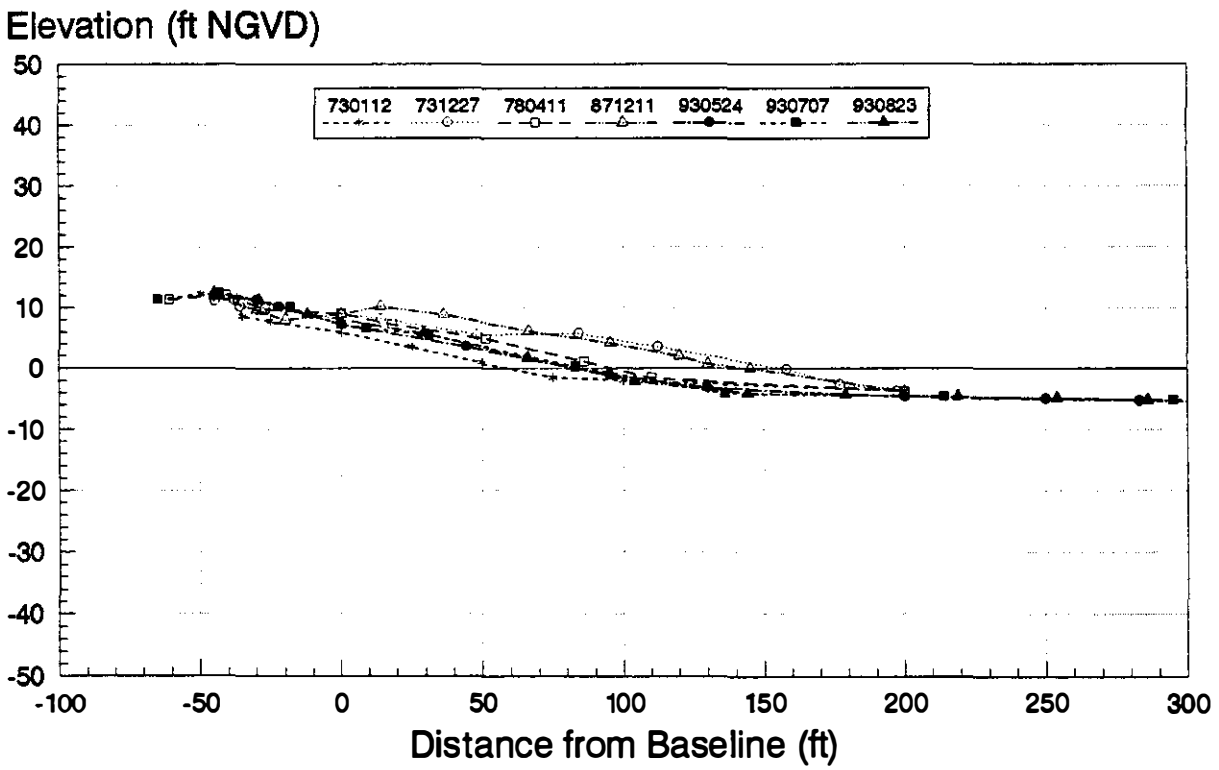


includes data from LRP#25

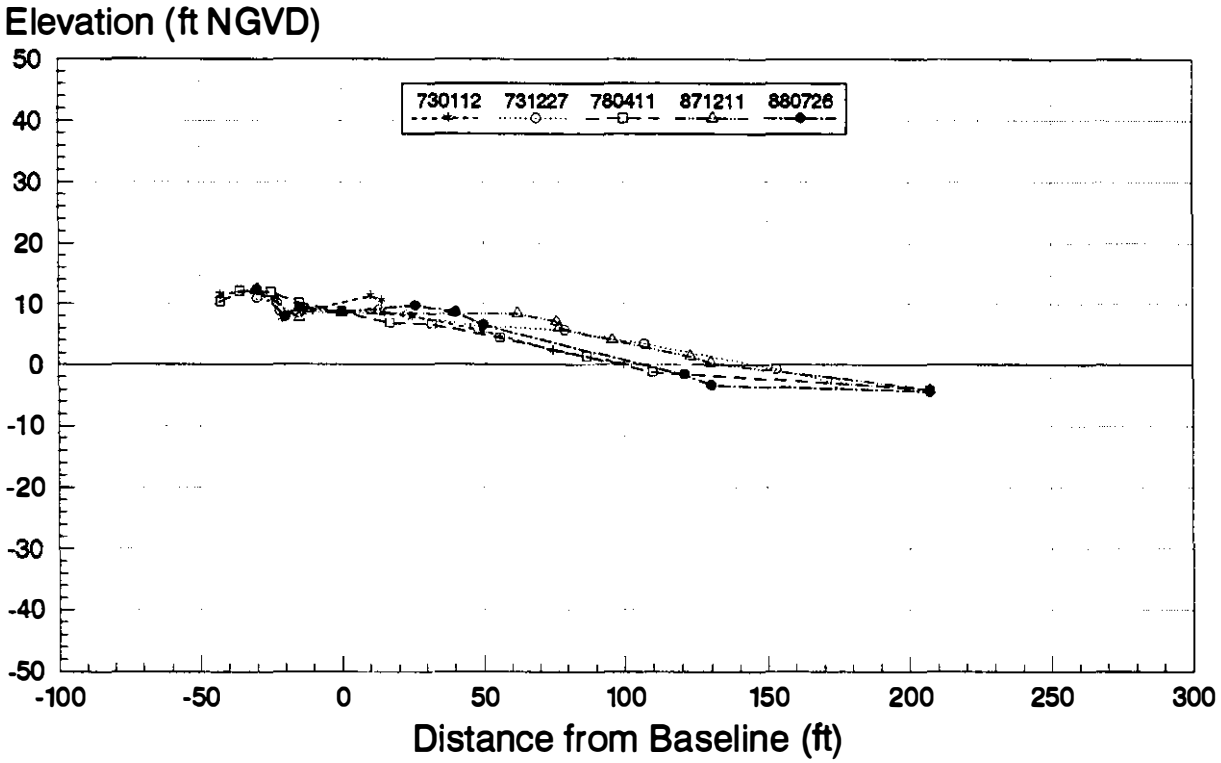
Broadkill Beach Profile N16+00



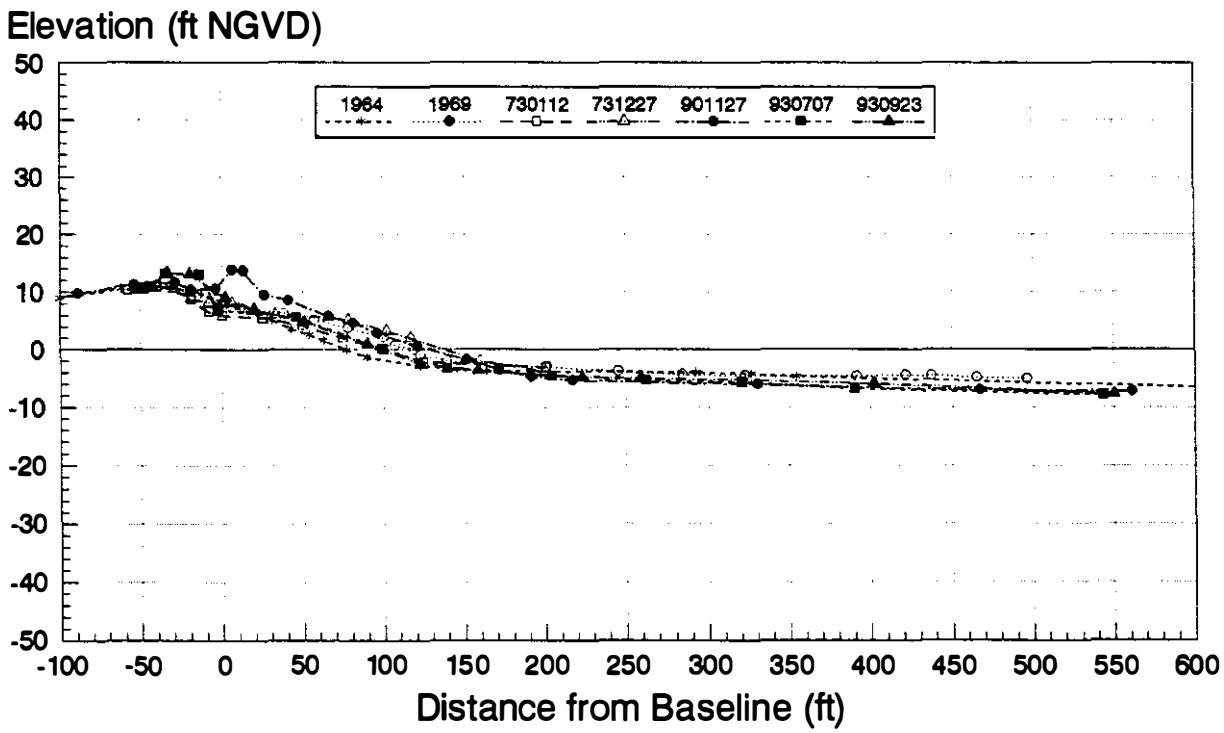
Broadkill Beach Profile N14+00



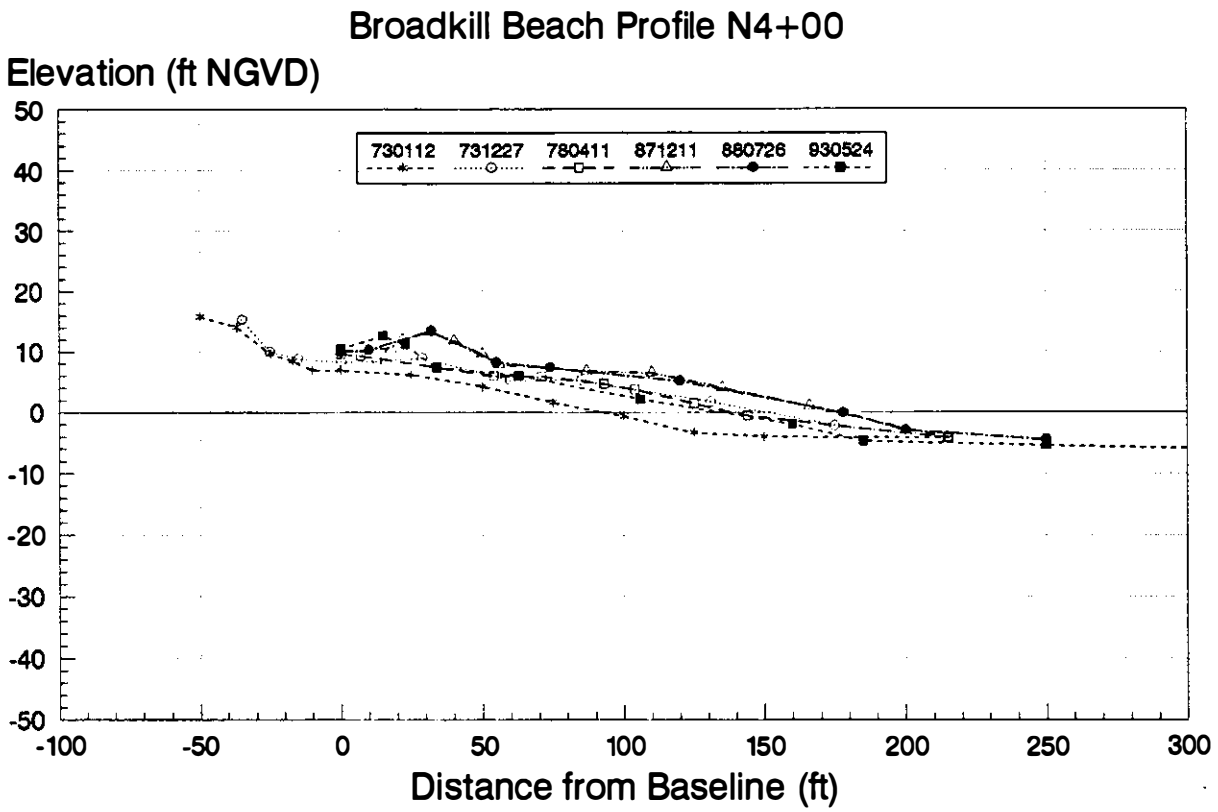
Broadkill Beach Profile N12+00

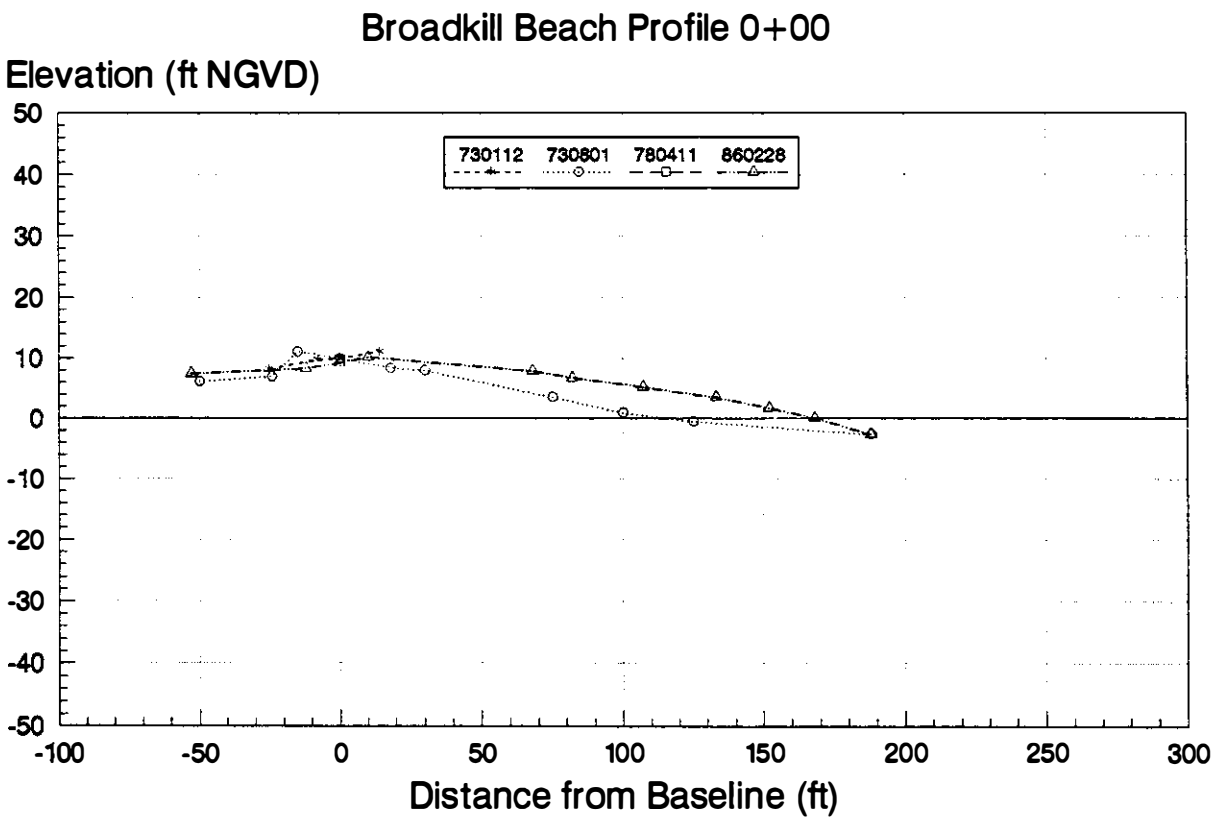


Broadkill Beach Profile N7+00

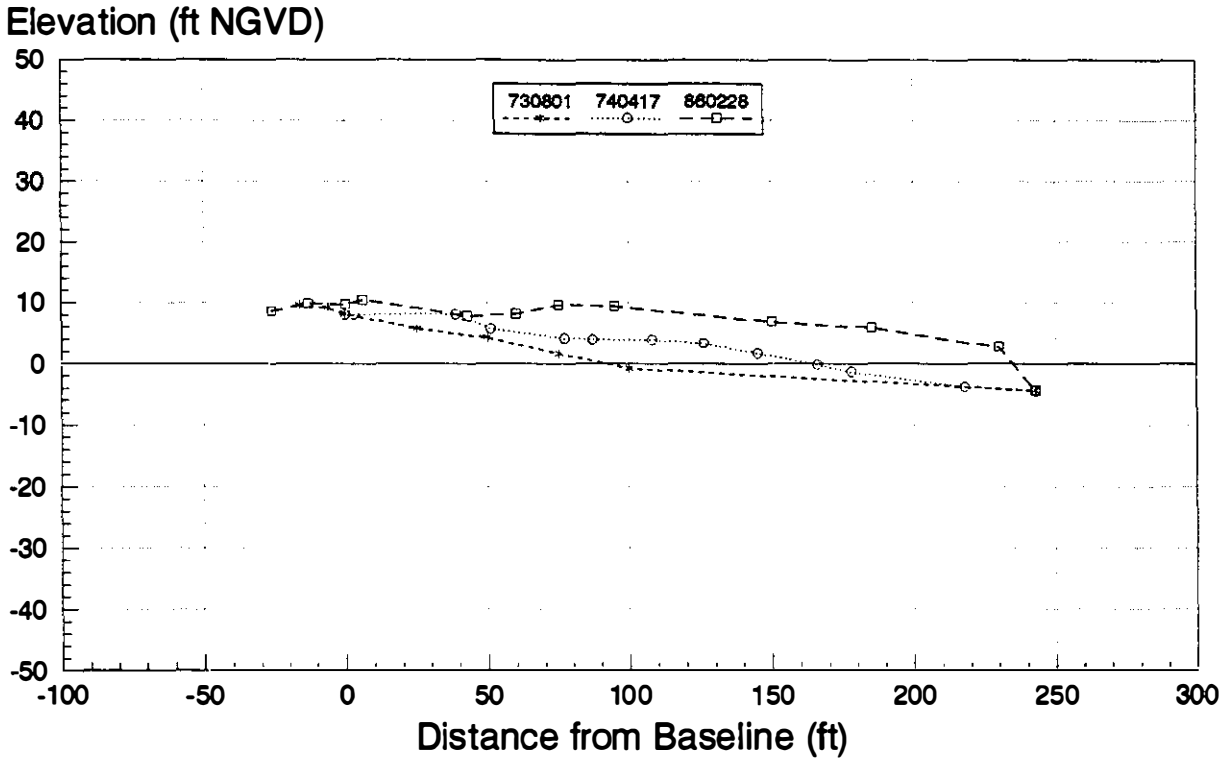


includes data from LRP#25A

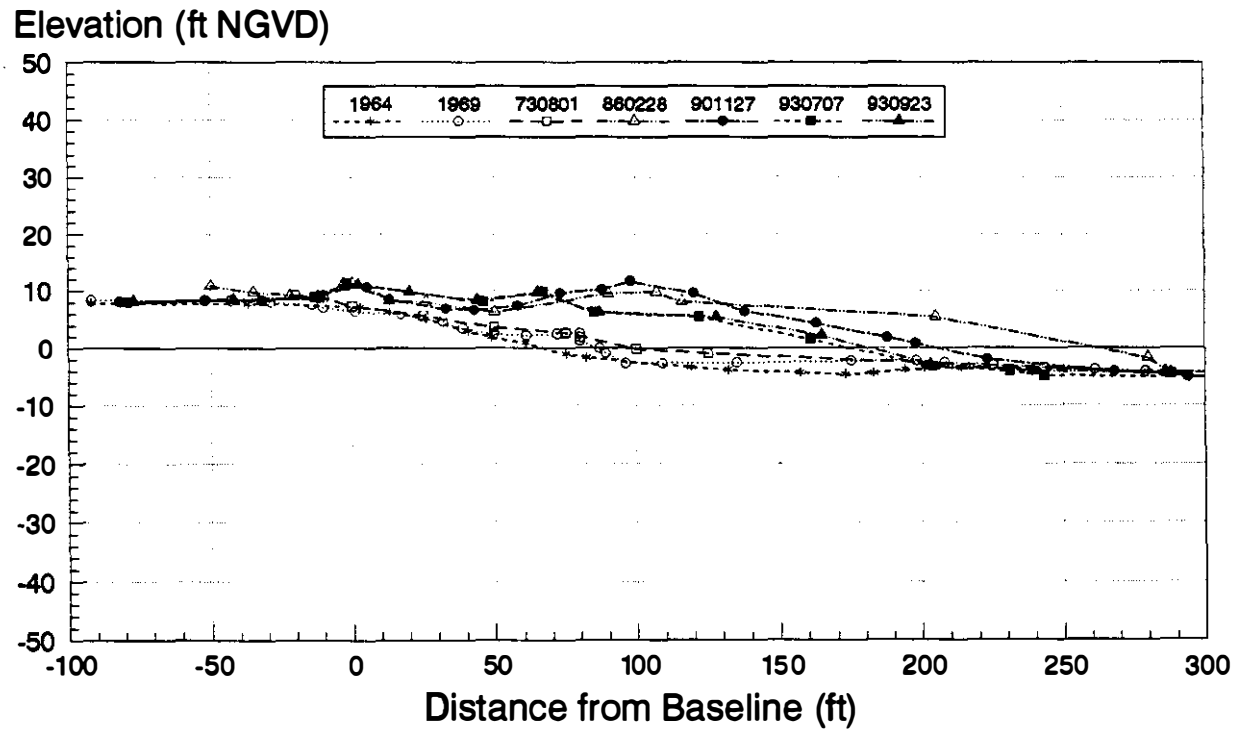

Broadkill Beach Profile N4+00
Elevation (ft NGVD)
730112
731227
780411
871211
880726
930524
50
40
30
20
10
0
-10
-20
-30
-40
-50
-100
-50
0
50
100
150
200
250
300
Distance from Baseline (ft)


Broadkill Beach Profile 0+00
Elevation (ft NGVD)
730112
730801
780411
860228
50
40
30
20
10
0
-10
-20
-30
-40
-50
-100
-50
0
50
100
150
200
250
300
Distance from Baseline (ft)

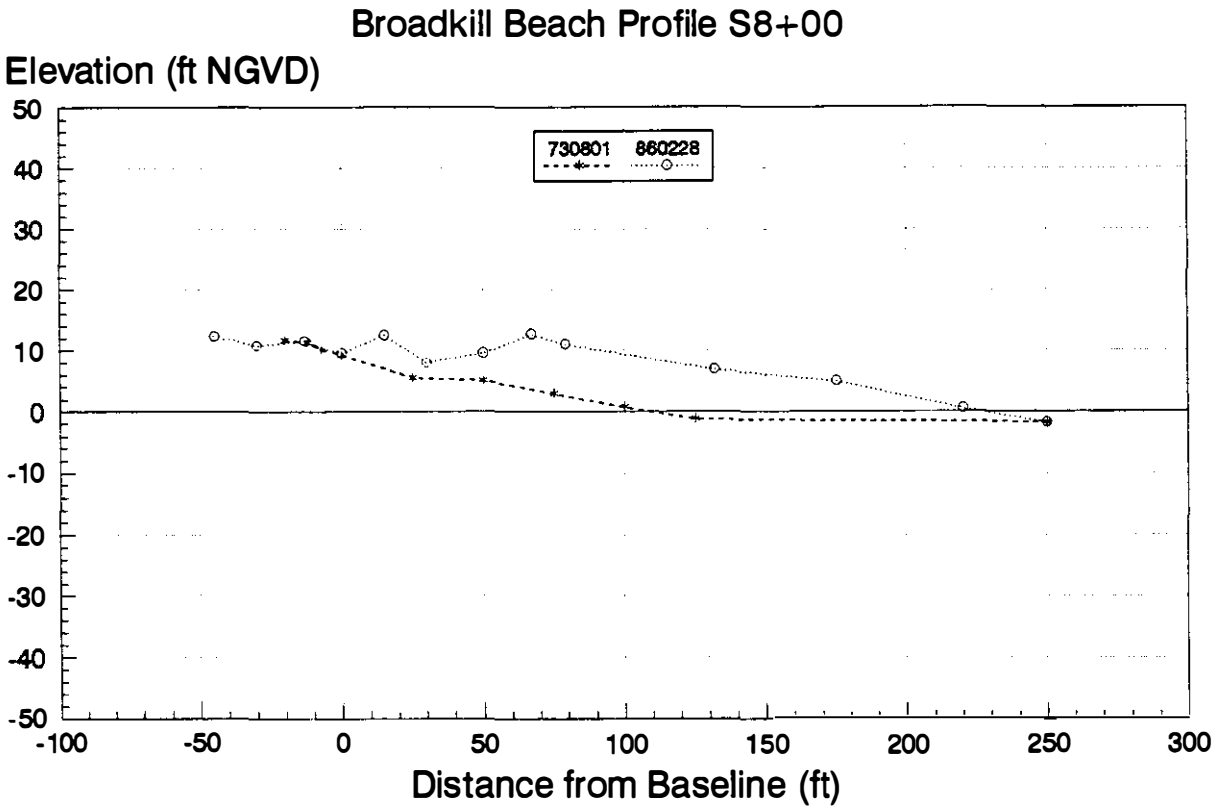
Broadkill Beach Profile S4+00

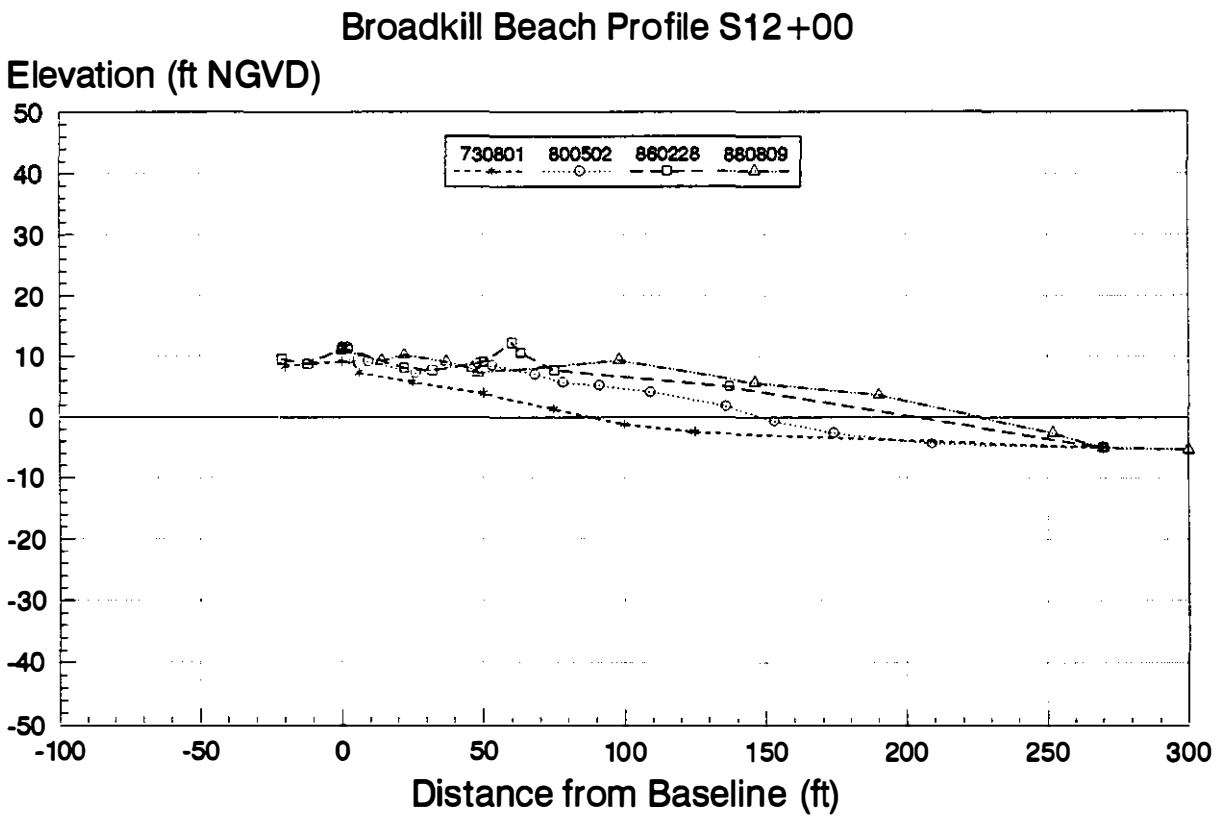


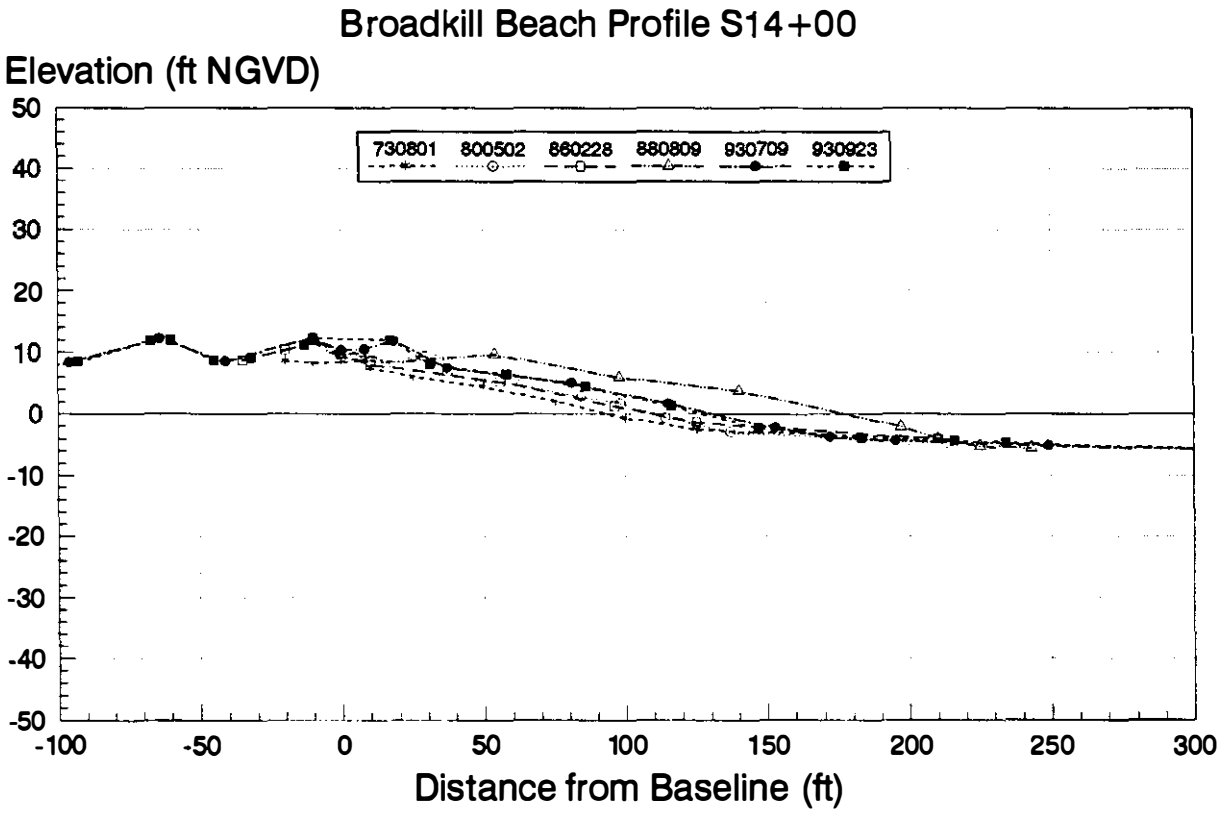
Broadkill Beach Profile S6+00

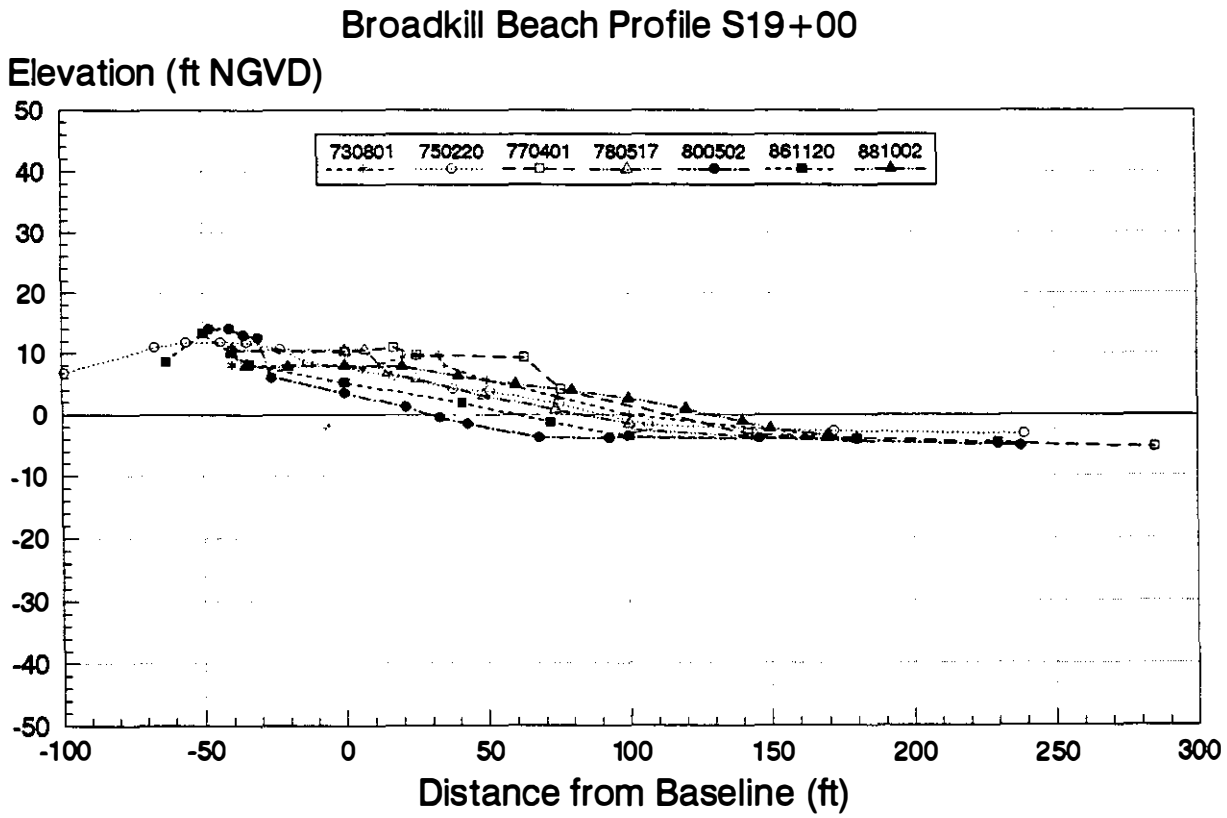


includes data from LRP #25B

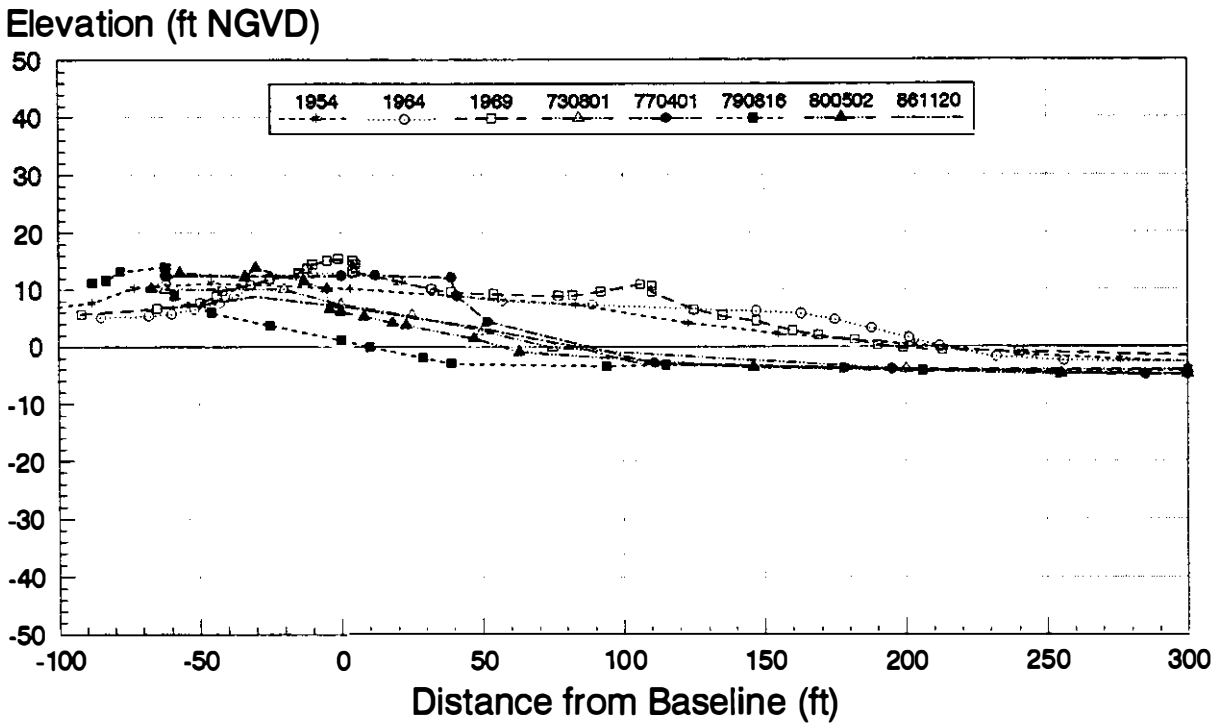

Broadkill Beach Profile S8+00
Elevation (ft NGVD)
730801
860228
50
40
30
20
10
0
-10
-20
-30
-40
-50
-100
-50
0
50
100
150
200
250
300
Distance from Baseline (ft)


Broadkill Beach Profile S12+00
Elevation (ft NGVD)
730801
800502
860228
880809
50
40
30
20
10
0
-10
-20
-30
-40
-50
-100
-50
0
50
100
150
200
250
300
Distance from Baseline (ft)


Broadkill Beach Profile S14+00
Elevation (ft NGVD)
730801
800502
860228
880809
930709
930923
50
40
30
20
10
0
-10
-20
-30
-40
-50
-100
-50
0
50
100
150
200
250
300
Distance from Baseline (ft)

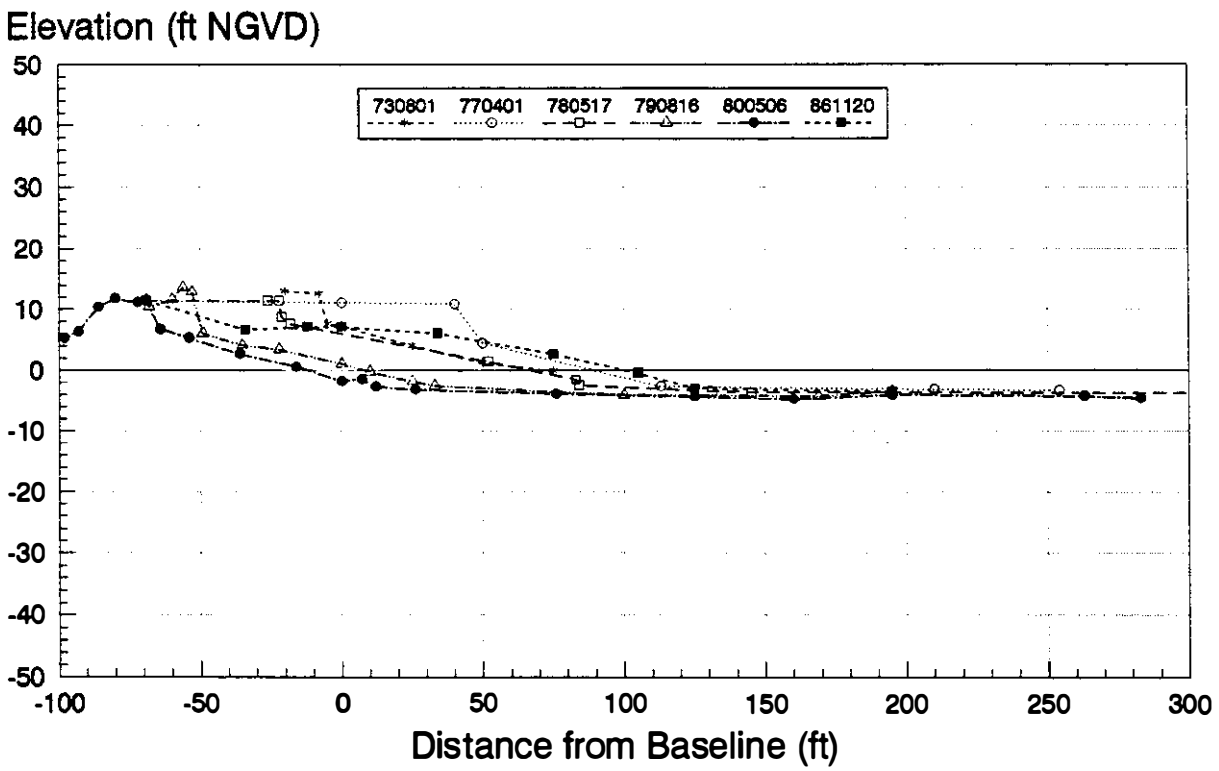

Broadkill Beach Profile S19+00
Elevation (ft NGVD)
730801
750220
770401
780517
800502
861120
881002
50
40
30
20
10
0
-10
-20
-30
-40
-50
-100
-50
0
50
100
150
200
250
300
Distance from Baseline (ft)

Broadkill Beach Profile S22+00

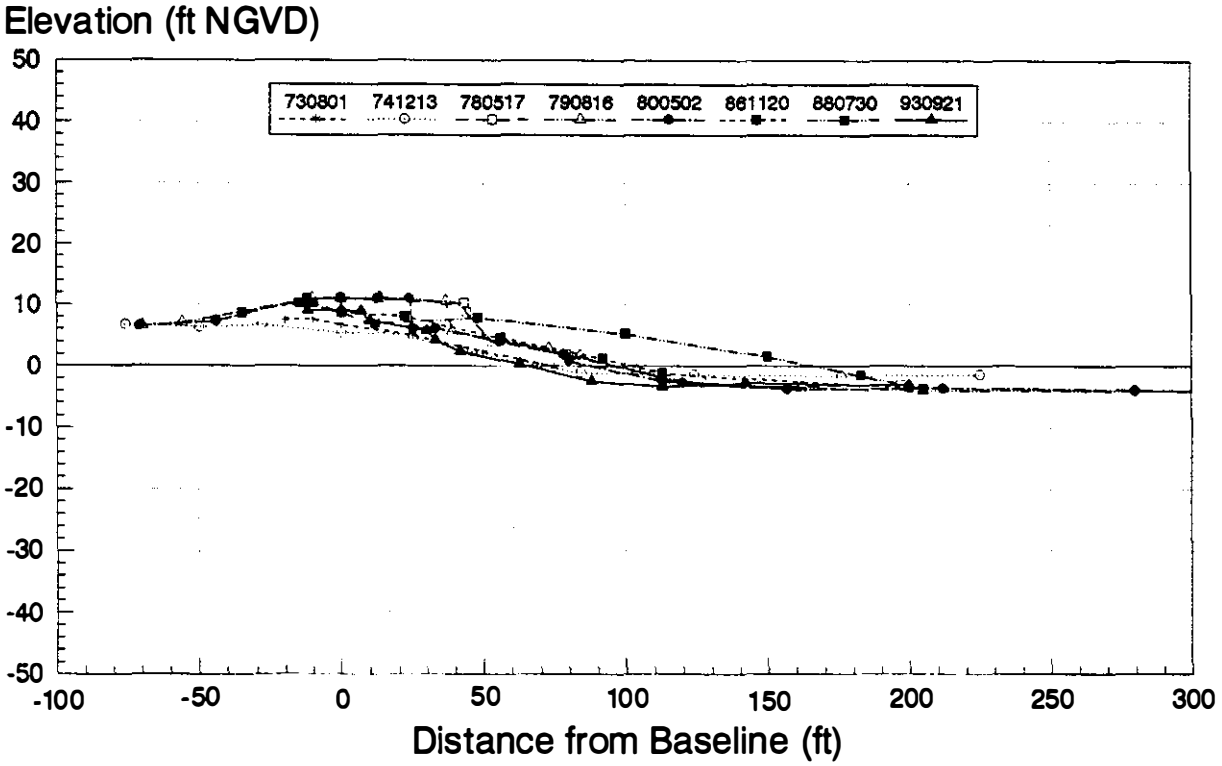


includes data from LRP#26

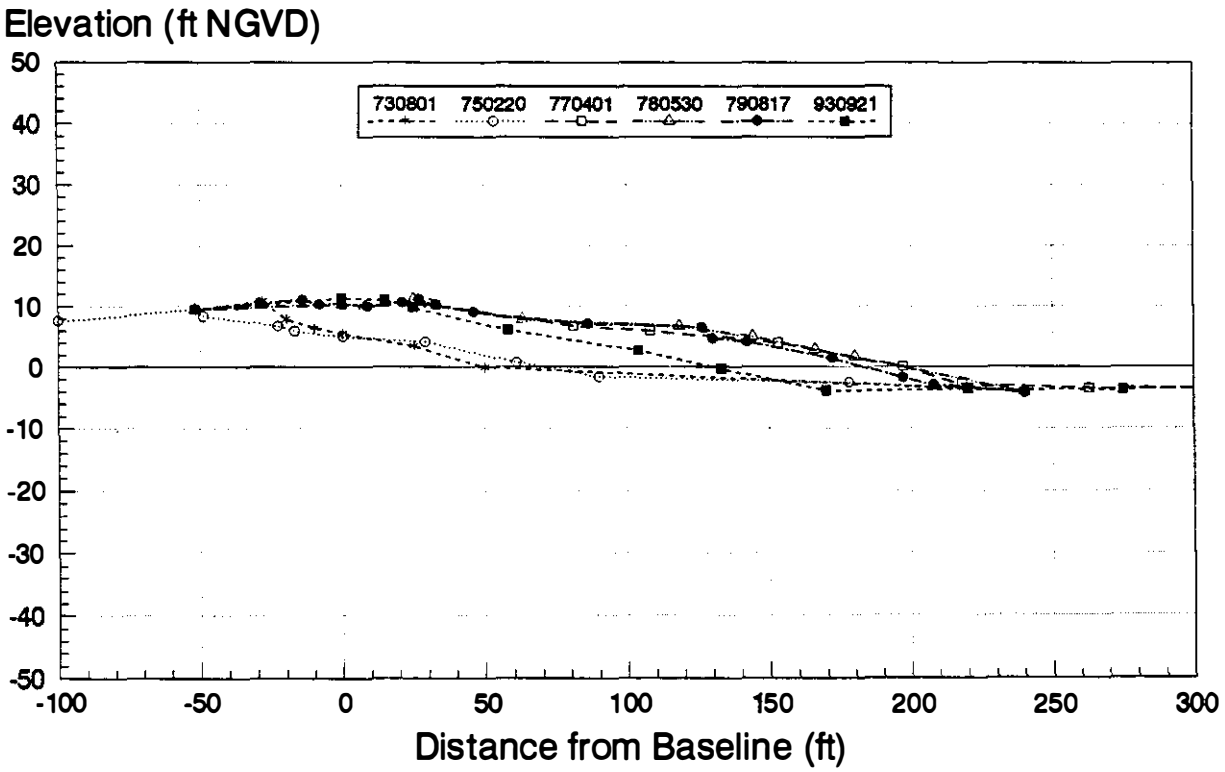
Broadkill Beach Profile S24+00



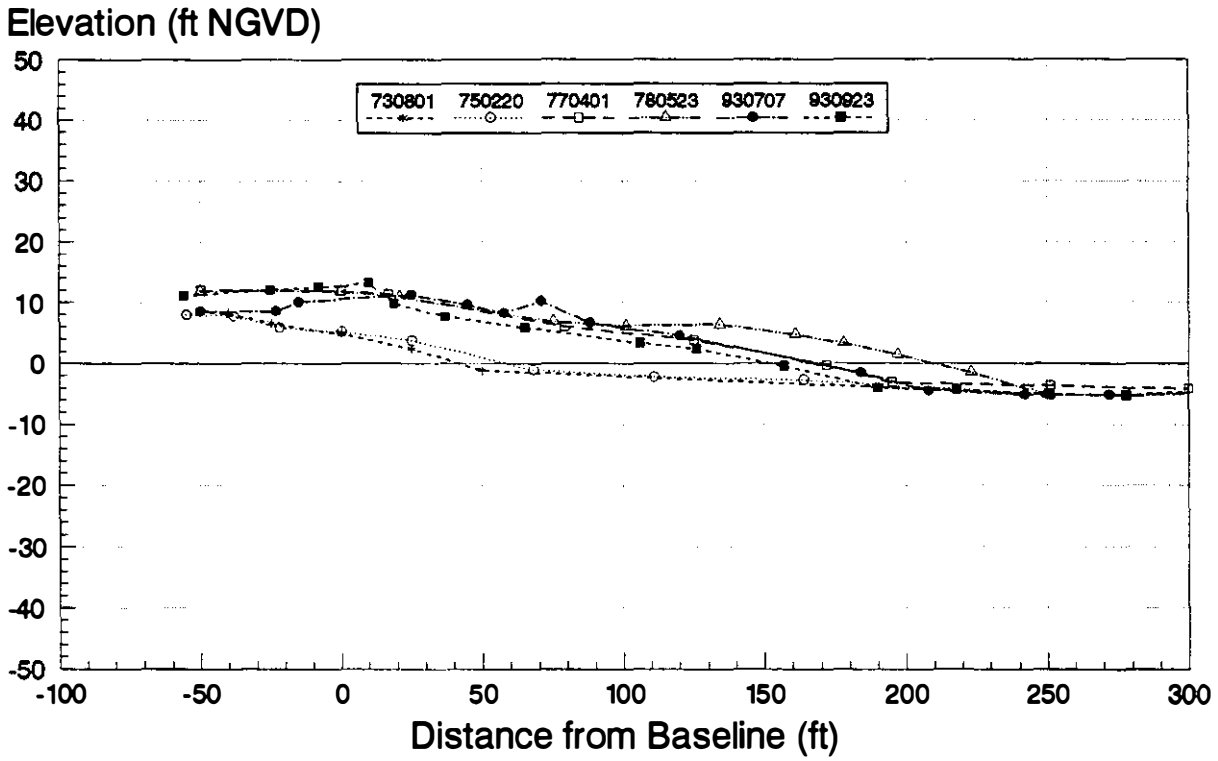
Broadkill Beach Profile S28+00



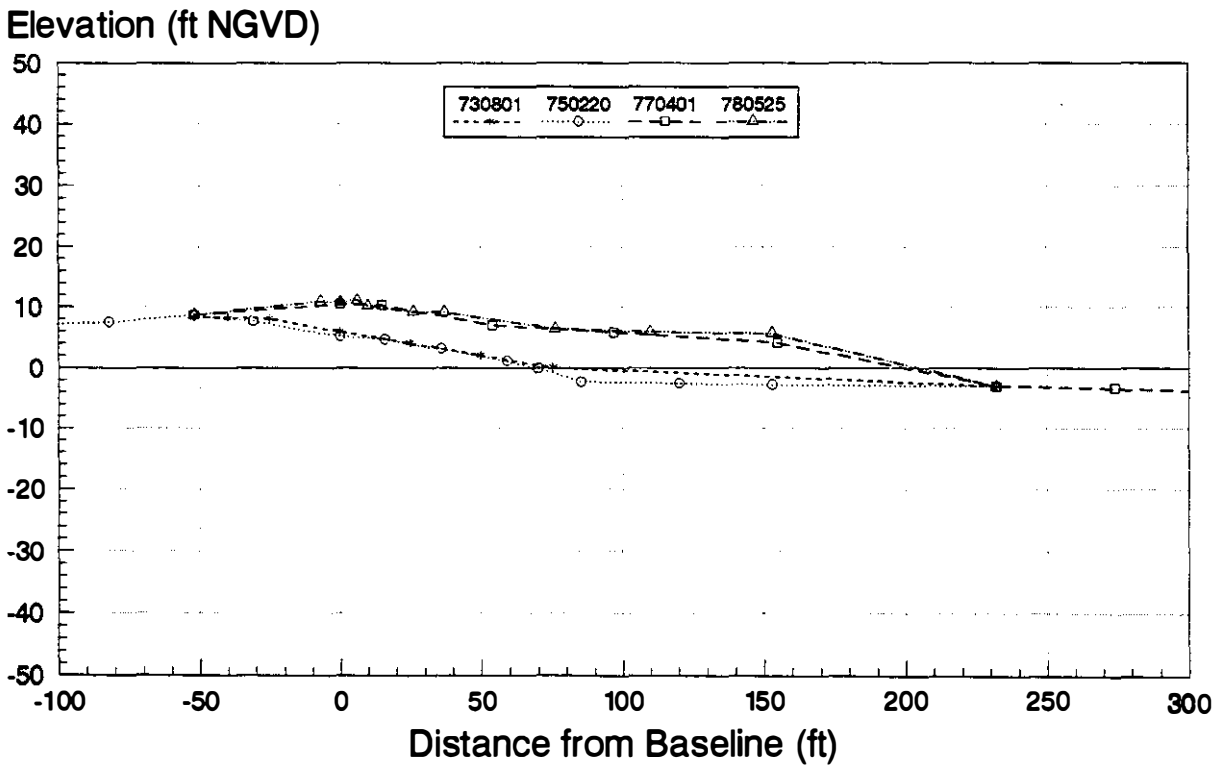
Broadkill Beach Profile S32+00



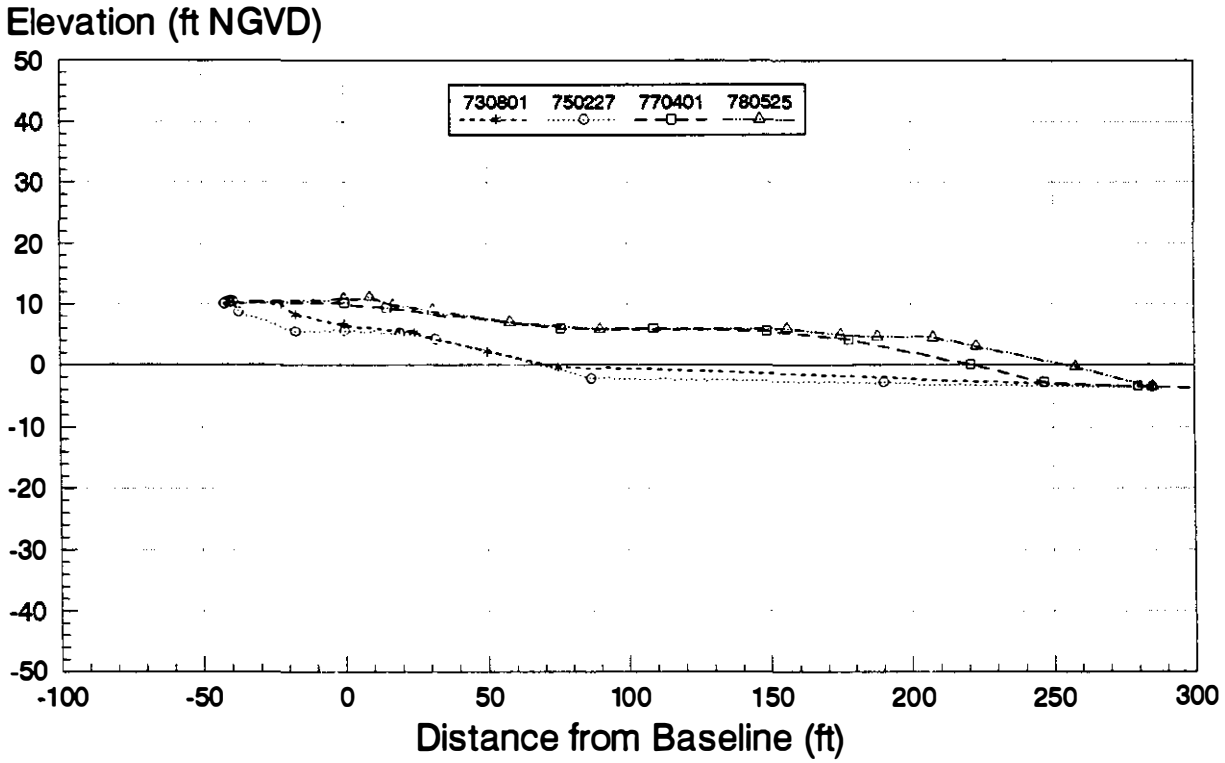
Broadkill Beach Profile S36+00



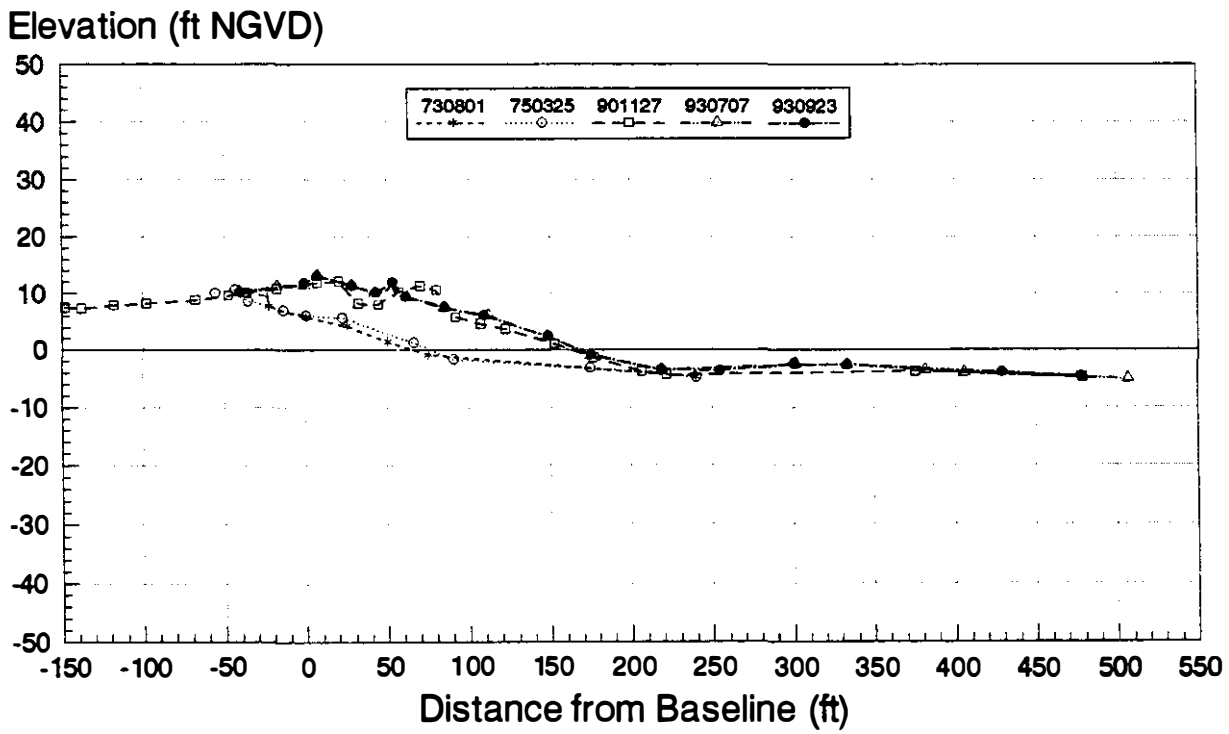
Broadkill Beach Profile S40+00



Broadkill Beach Profile S44+00

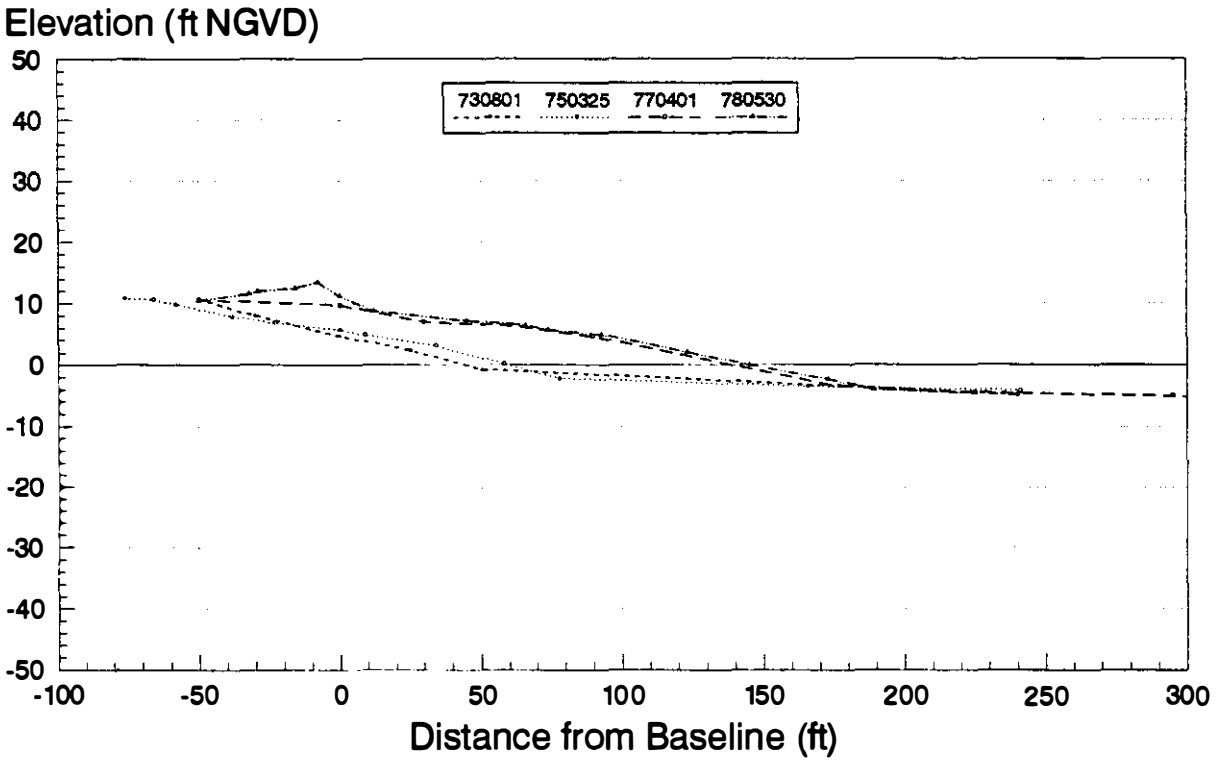


Broadkill Beach Profile S49+00

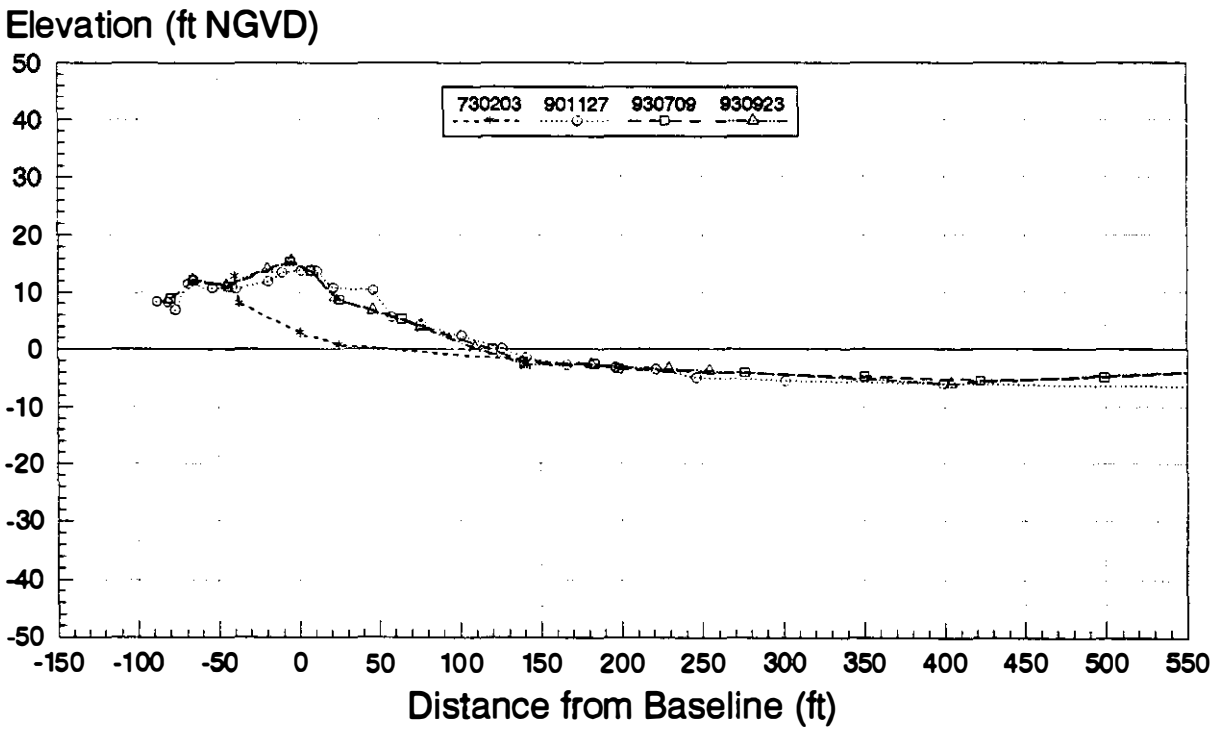


includes data from LRP#27

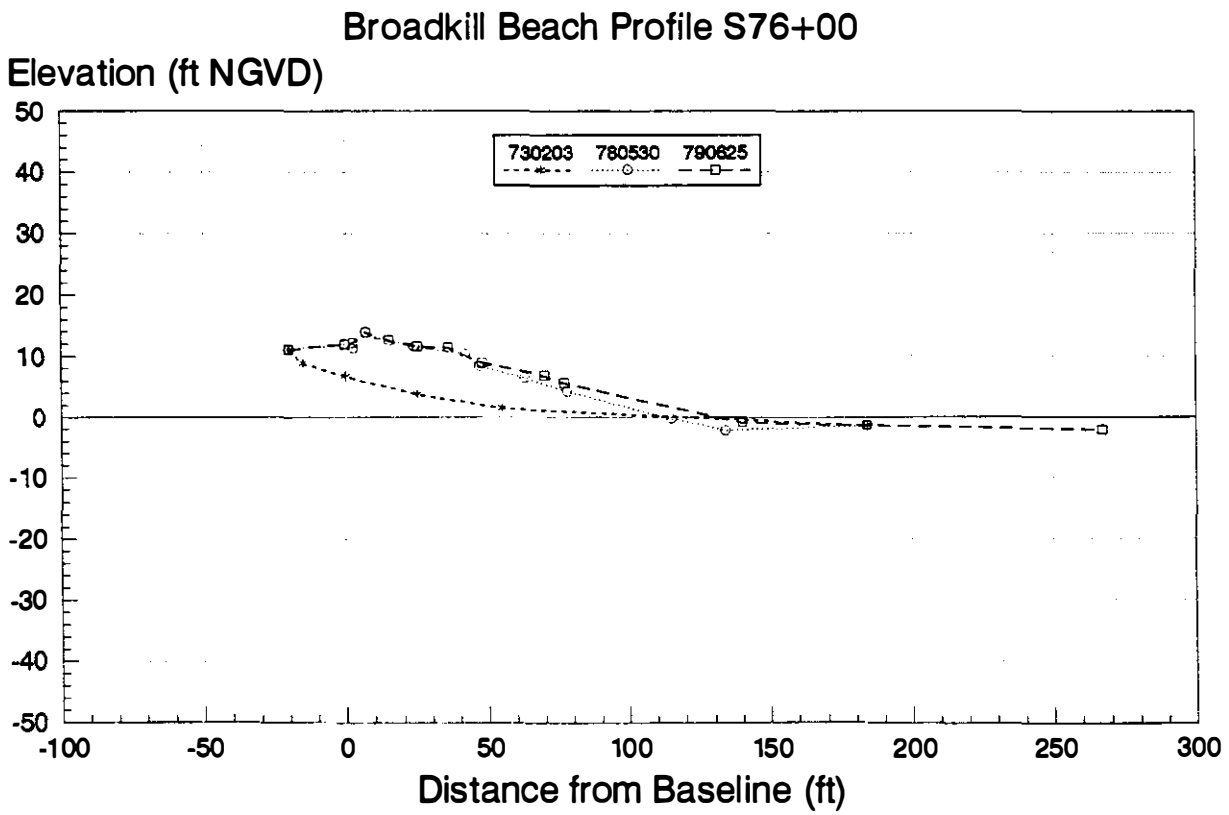
Broadkill Beach Profile S52+00



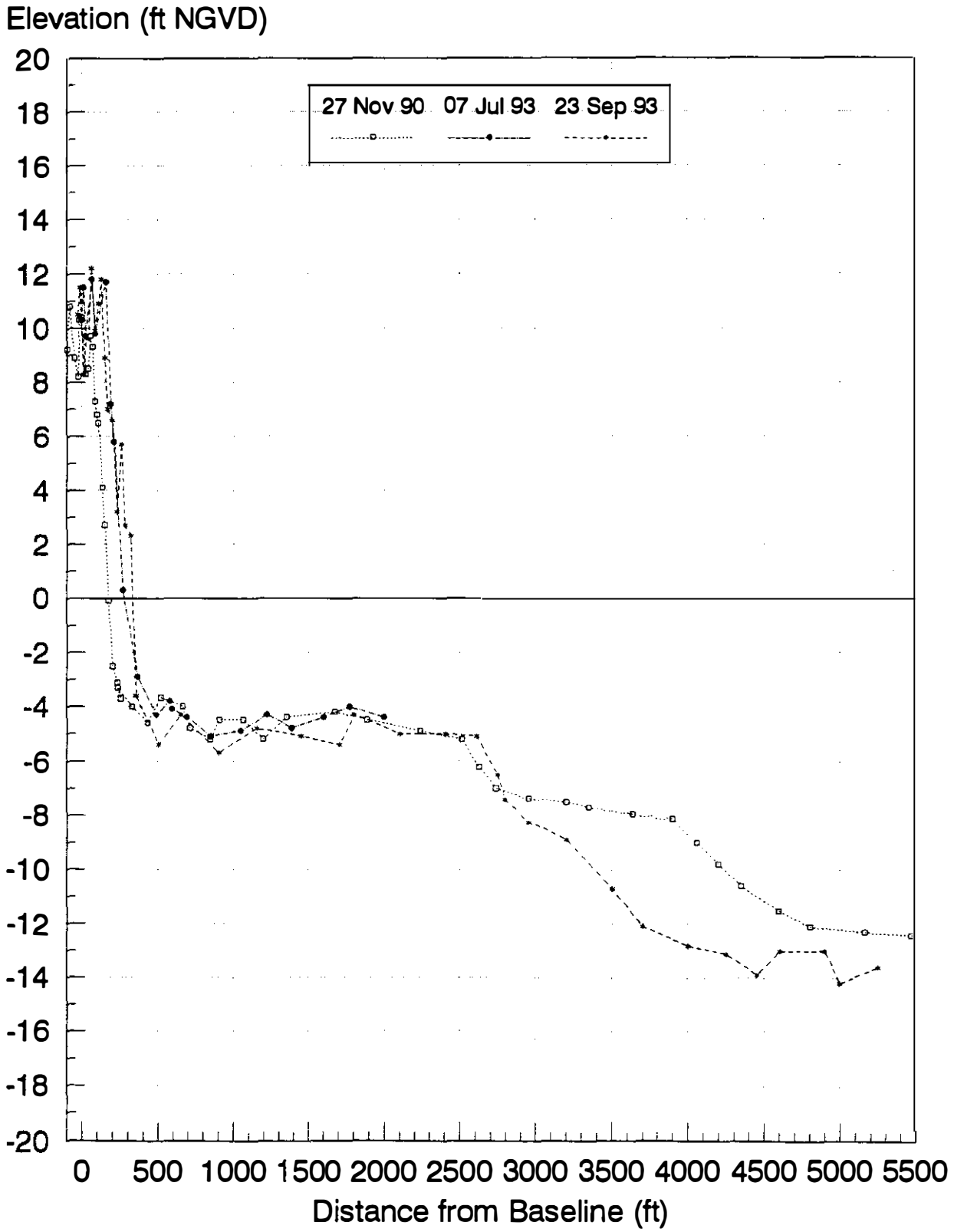
Broadkill Beach Profile S69+00

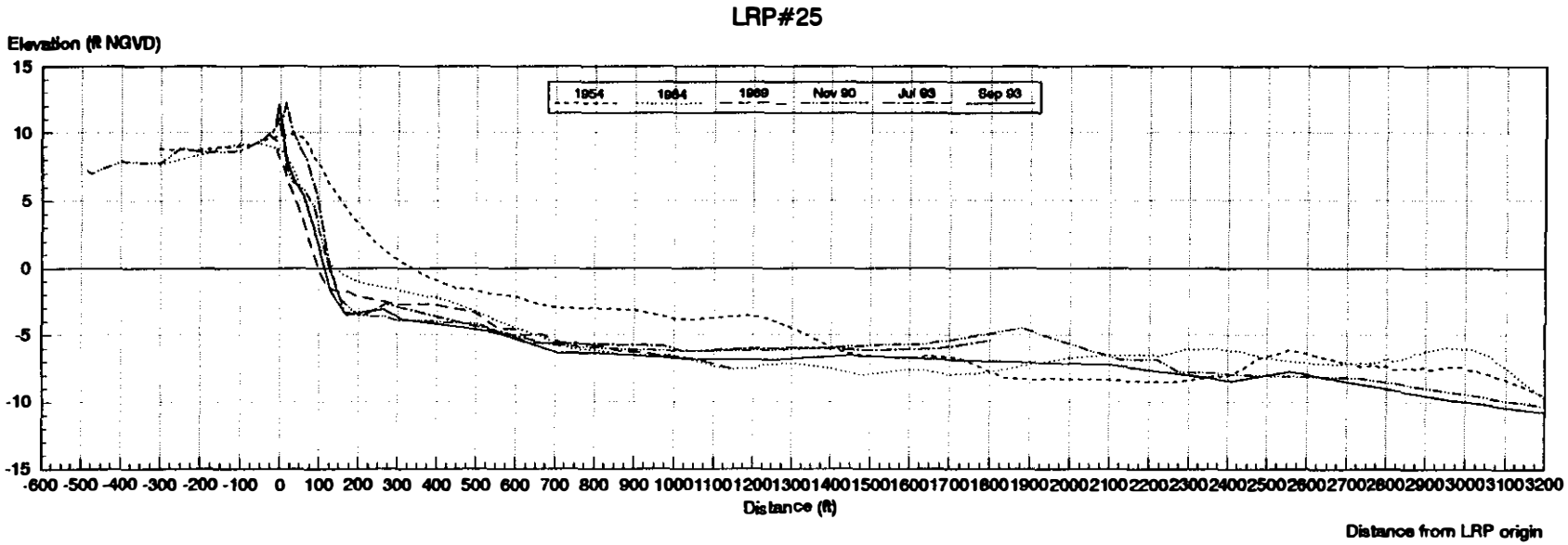


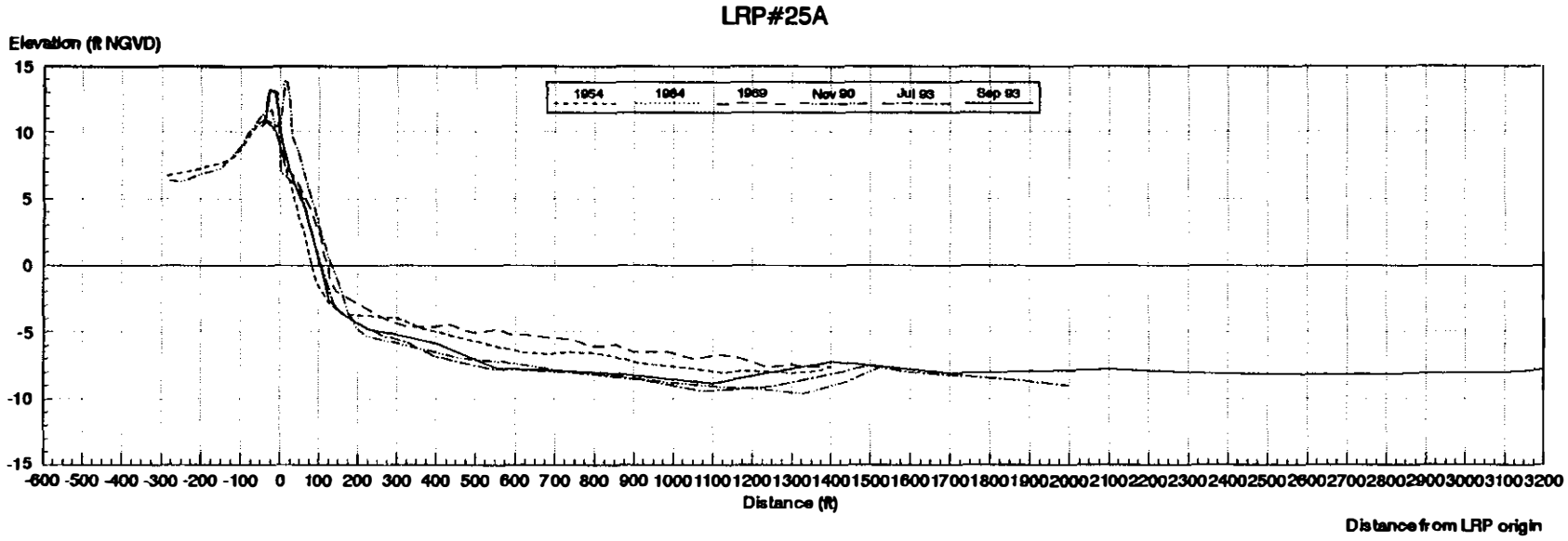
includes data from LRP#27A

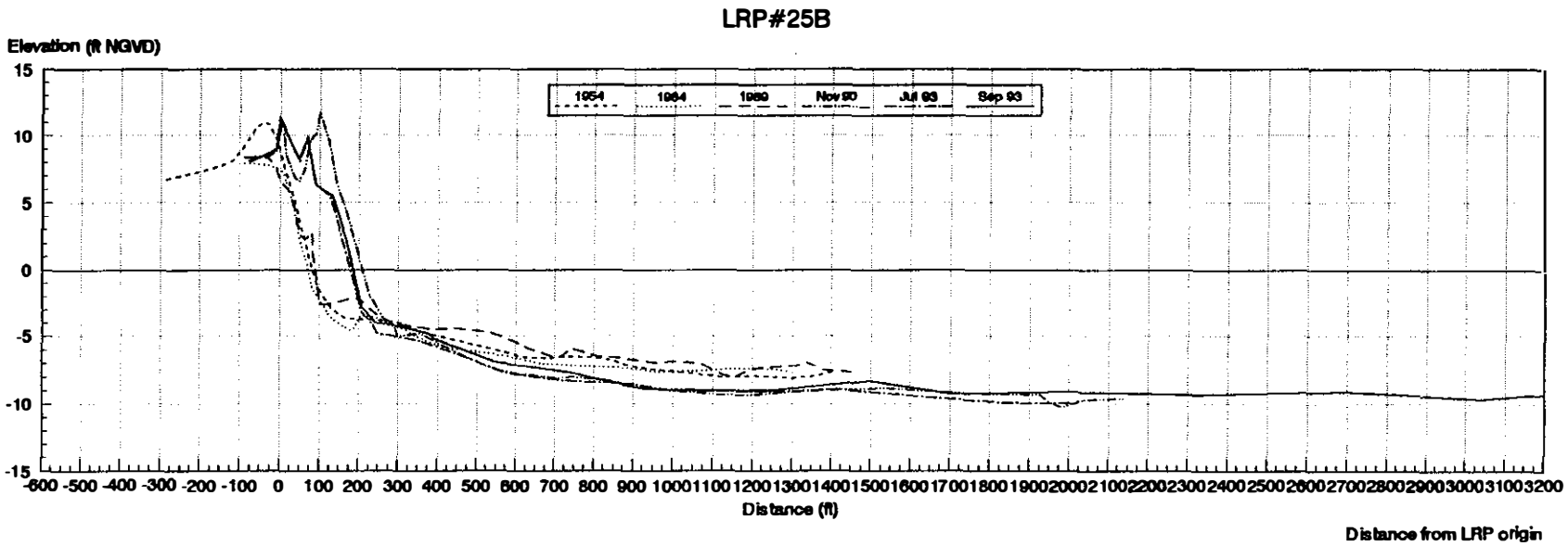

Broadkill Beach Profile S76+00
Elevation (ft NGVD)
730203
780530
790625
50
40
30
20
10
0
-10
-20
-30
-40
-50
-100
-50
0
50
100
150
200
250
300
Distance from Baseline (ft)

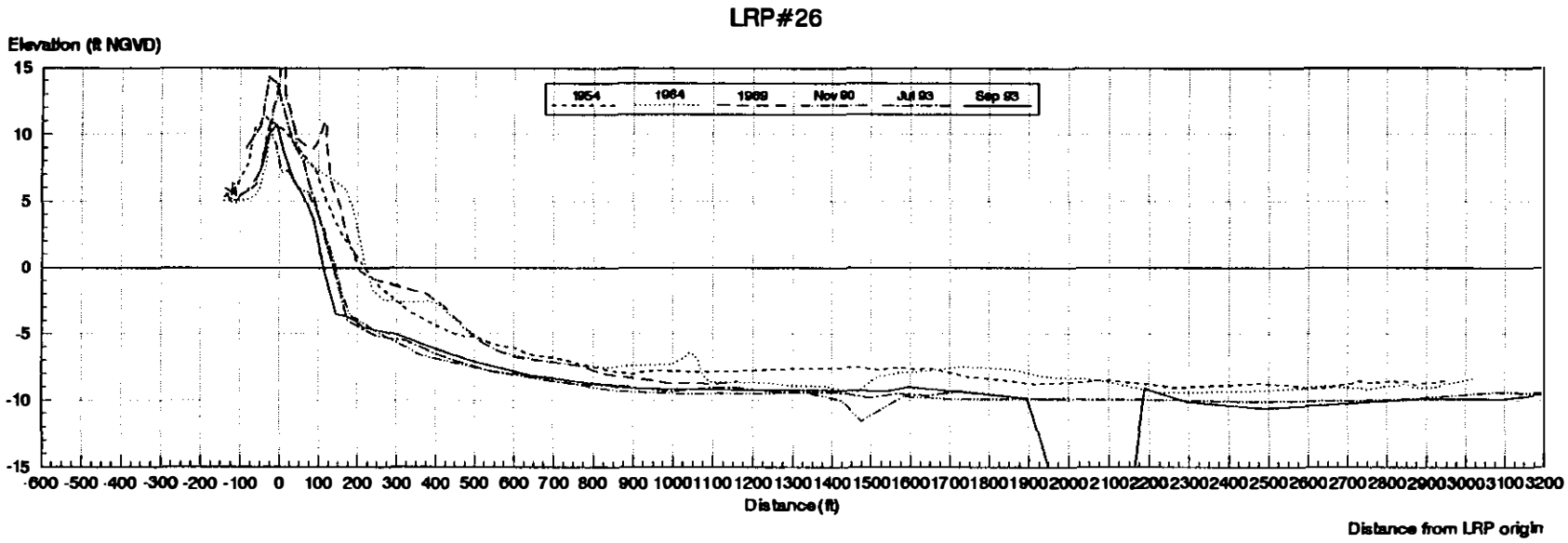
Broadkill Beach Profile LRP#24A



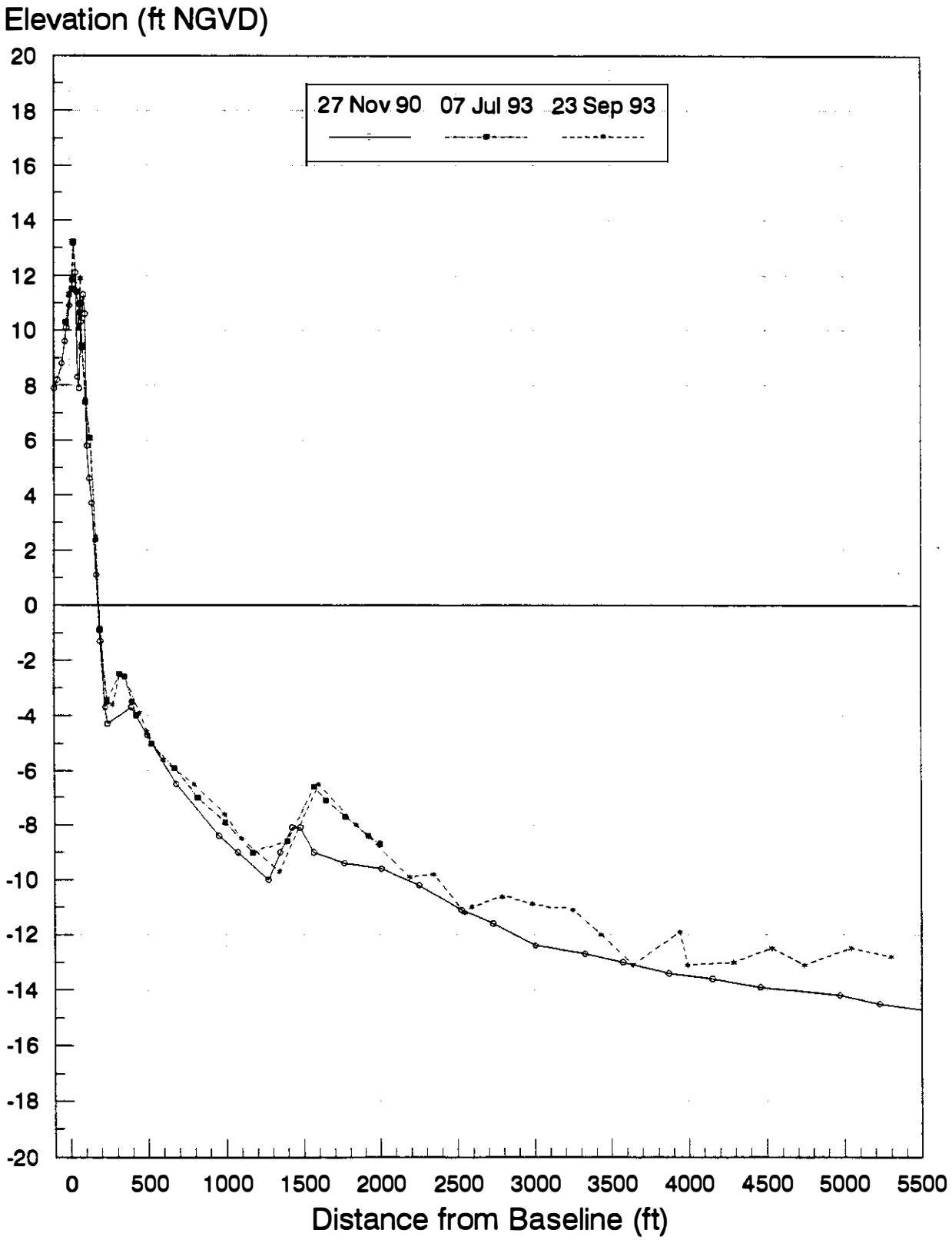

LRP#25
Elevation (ft NGVD)
15
10
5
0
-5
-10
-15
1954
1964
1989
Nov 90
Jul 93
Sep 93
-600 -500 -400 -300 -200 -100 0 100 200 300 400 500 600 700 800 900 1000 1100 1200 1300 1400 1500 1600 1700 1800 1900 2000 2100 2200 2300 2400 2500 2600 2700 2800 2900 3000 3100 3200
Distance (ft)
Distance from LRP origin


LRP#25A
Elevation (ft NGVD)
15
10
5
0
-5
-10
-15
1954
1964
1969
Nov 90
Jul 93
Sep 93
-600 -500 -400 -300 -200 -100 0 100 200 300 400 500 600 700 800 900 1000 1100 1200 1300 1400 1500 1600 1700 1800 1900 2000 2100 2200 2300 2400 2500 2600 2700 2800 2900 3000 3100 3200
Distance (ft)
Distance from LRP origin

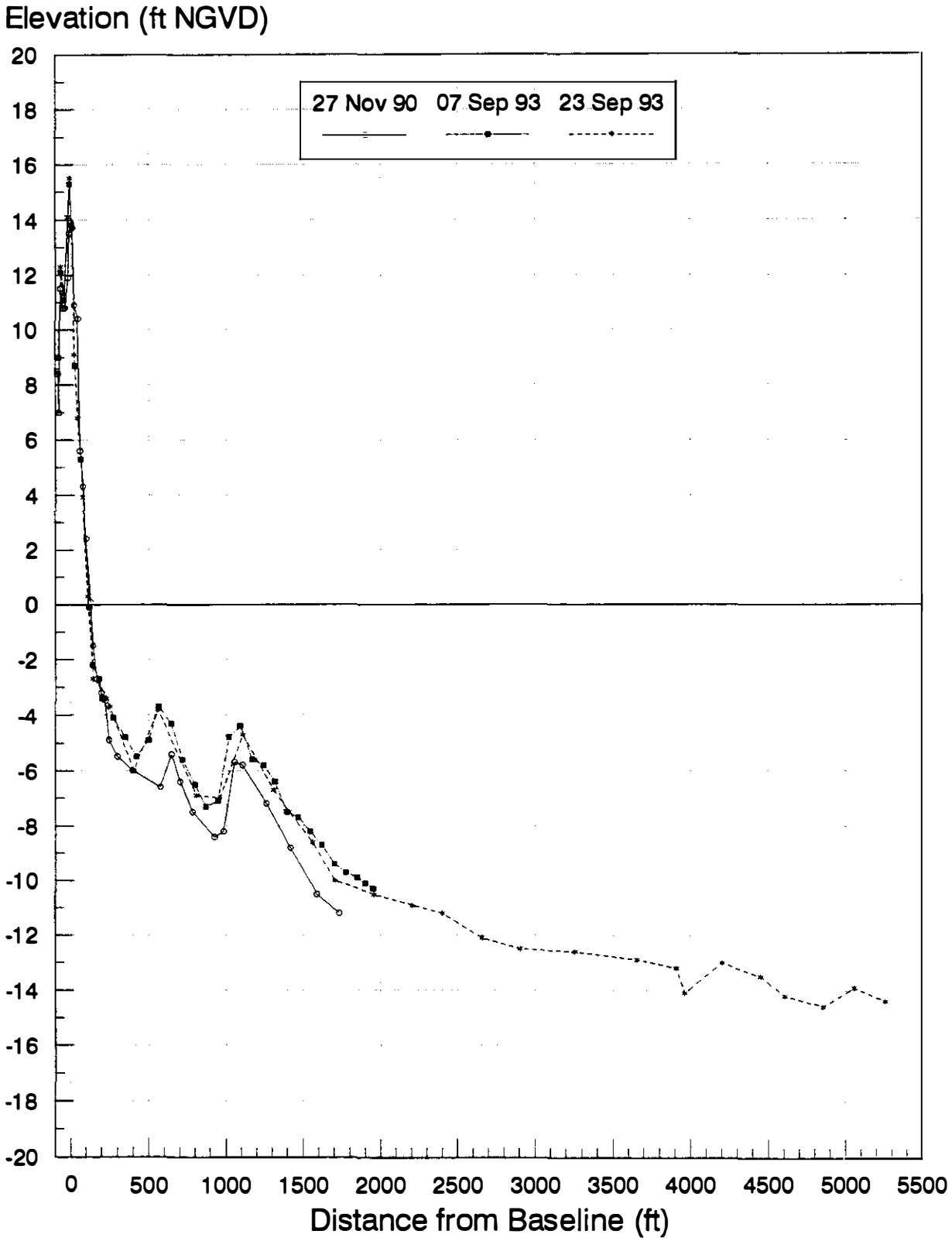

LRP#25B
Elevation (ft NGVD)
15
10
5
0
-5
-10
-15
1954
1964
1989
Nov 90
Jul 93
Sep 93
-600 -500 -400 -300 -200 -100 0 100 200 300 400 500 600 700 800 900 1000 1100 1200 1300 1400 1500 1600 1700 1800 1900 2000 2100 2200 2300 2400 2500 2600 2700 2800 2900 3000 3100 3200
Distance (ft)
Distance from LRP origin

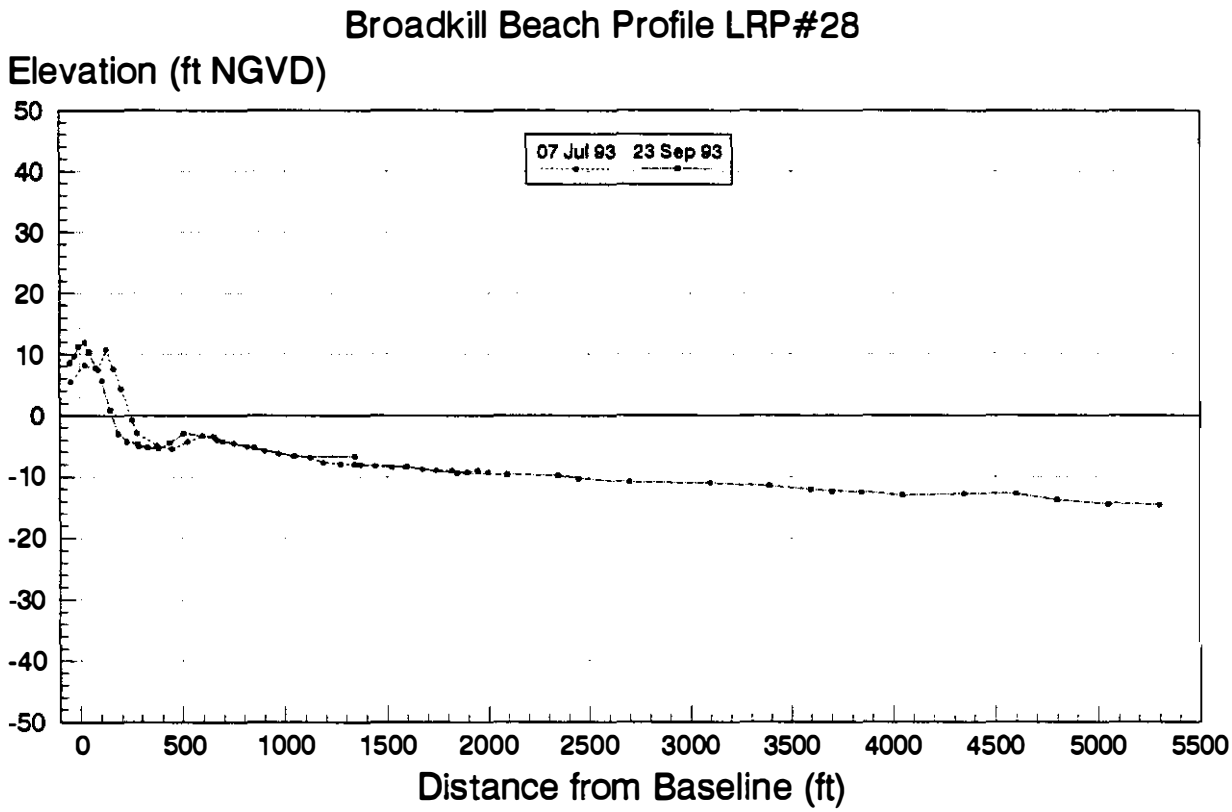

LRP#26
Elevation (ft NGVD)
15
10
5
0
-5
-10
-15
1954
1964
1969
Nov 90
Jul 93
Sep 93
-600 -500 -400 -300 -200 -100 0 100 200 300 400 500 600 700 800 900 1000 1100 1200 1300 1400 1500 1600 1700 1800 1900 2000 2100 2200 2300 2400 2500 2600 2700 2800 2900 3000 3100 3200
Distance (ft)
Distance from LRP origin

Broadkill Beach Profile LRP#27



Broadkill Beach Profile LRP#27A




Broadkill Beach Profile LRP#28
Elevation (ft NGVD)
07 Jul 93
23 Sep 93
50
40
30
20
10
0
-10
-20
-30
-40
-50
0
500
1000
1500
2000
2500
3000
3500
4000
4500
5000
5500
Distance from Baseline (ft)

Annual and Seasonal Frequency Distribution of Wave Data from Delaware Bay Wave Study

Annual Wave Frequency Distribution at Broadkill Beach (% Occurrence).
Directions are degrees from. Station location: 38-55 lat, 75-10 long.

Dir=319.8-331.1				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.25	0.11	0.00	0.00	0.00	0.00	0.00	0.36
1.0-1.5	1.53	8.50	0.00	0.00	0.00	0.00	0.00	10.03
1.5-2.0	0.02	1.55	0.41	0.00	0.00	0.00	0.00	1.98
2.0-2.5	0.00	0.00	0.10	0.00	0.00	0.00	0.00	0.10
2.5-3.0	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01
3.0-3.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.5-4.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4.0-4.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	1.80	10.18	0.51	0.00	0.00	0.00	0.00	12.49

Dir=331.1-353.6				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.17	0.11	0.00	0.00	0.00	0.00	0.00	0.28
1.0-1.5	1.03	6.14	0.00	0.00	0.00	0.00	0.00	7.18
1.5-2.0	0.01	0.90	0.51	0.00	0.00	0.00	0.00	1.42
2.0-2.5	0.00	0.07	0.06	0.00	0.00	0.00	0.00	0.12
2.5-3.0	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01
3.0-3.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.5-4.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4.0-4.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	1.21	7.23	0.57	0.00	0.00	0.00	0.00	9.01

Dir=353.6-16.1				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.31	0.15	0.00	0.00	0.00	0.00	0.00	0.45
1.0-1.5	1.77	9.58	0.00	0.00	0.00	0.00	0.00	11.35
1.5-2.0	0.01	1.05	0.42	0.00	0.00	0.00	0.00	1.49
2.0-2.5	0.01	0.09	0.26	0.00	0.00	0.00	0.00	0.36
2.5-3.0	0.00	0.03	0.03	0.00	0.00	0.00	0.00	0.07
3.0-3.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.5-4.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4.0-4.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	2.10	10.91	0.71	0.00	0.00	0.00	0.00	13.72

Dir=16.1-38.6				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.23	0.11	0.00	0.00	0.00	0.00	0.00	0.34
1.0-1.5	1.66	6.96	0.00	0.00	0.00	0.00	0.00	8.62
1.5-2.0	0.00	0.84	0.35	0.00	0.00	0.00	0.00	1.19
2.0-2.5	0.01	0.03	0.09	0.00	0.00	0.00	0.00	0.14
2.5-3.0	0.00	0.03	0.03	0.00	0.00	0.00	0.00	0.07
3.0-3.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.5-4.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4.0-4.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	1.89	7.98	0.48	0.00	0.00	0.00	0.00	10.35

Dir=38.6-61.1				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.28	0.16	0.00	0.00	0.00	0.00	0.00	0.44
1.0-1.5	1.67	7.87	0.00	0.00	0.00	0.00	0.00	9.53
1.5-2.0	0.00	0.90	0.29	0.00	0.00	0.00	0.00	1.19
2.0-2.5	0.01	0.12	0.31	0.00	0.00	0.00	0.00	0.44
2.5-3.0	0.00	0.03	0.12	0.00	0.00	0.00	0.00	0.16
3.0-3.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.5-4.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4.0-4.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	1.96	9.08	0.73	0.00	0.00	0.00	0.00	11.77
Dir=61.1-83.6				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.24	0.24	0.00	0.00	0.00	0.00	0.00	0.48
1.0-1.5	2.21	7.33	0.03	0.00	0.00	0.00	0.00	9.58
1.5-2.0	0.00	0.83	0.42	0.00	0.00	0.00	0.00	1.25
2.0-2.5	0.00	0.08	0.17	0.03	0.00	0.00	0.00	0.28
2.5-3.0	0.00	0.01	0.03	0.10	0.00	0.01	0.00	0.16
3.0-3.5	0.00	0.00	0.01	0.08	0.05	0.00	0.00	0.14
3.5-4.0	0.00	0.00	0.00	0.00	0.09	0.00	0.00	0.09
4.0-4.5	0.00	0.00	0.00	0.00	0.08	0.00	0.00	0.08
	2.45	8.49	0.67	0.22	0.22	0.01	0.00	12.05
Dir=83.6-106.1				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.32	0.42	0.00	0.00	0.00	0.00	0.00	0.74
1.0-1.5	3.55	10.69	0.06	0.00	0.00	0.00	0.00	14.30
1.5-2.0	0.01	0.37	0.88	0.00	0.00	0.00	0.00	1.27
2.0-2.5	0.00	0.05	0.39	0.15	0.00	0.00	0.00	0.58
2.5-3.0	0.00	0.01	0.01	0.36	0.01	0.00	0.00	0.40
3.0-3.5	0.00	0.00	0.00	0.03	0.00	0.00	0.00	0.03
3.5-4.0	0.00	0.00	0.00	0.01	0.02	0.00	0.00	0.03
4.0-4.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	3.88	11.54	1.34	0.56	0.03	0.00	0.00	17.34
Dir=106.1-128.6				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.33	0.23	0.00	0.00	0.00	0.00	0.00	0.56
1.0-1.5	2.06	5.19	0.08	0.00	0.00	0.00	0.00	7.33
1.5-2.0	0.01	0.29	0.22	0.00	0.00	0.00	0.00	0.52
2.0-2.5	0.00	0.01	0.09	0.06	0.00	0.00	0.00	0.16
2.5-3.0	0.00	0.00	0.00	0.08	0.00	0.00	0.00	0.08
3.0-3.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.5-4.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4.0-4.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	2.40	5.72	0.39	0.14	0.00	0.00	0.00	8.65

Dir=128.6-139.8				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.26	0.27	0.00	0.00	0.00	0.00	0.00	0.53
1.0-1.5	0.92	2.63	0.05	0.00	0.00	0.00	0.00	3.59
1.5-2.0	0.00	0.17	0.19	0.00	0.00	0.00	0.00	0.36
2.0-2.5	0.00	0.02	0.06	0.01	0.00	0.00	0.00	0.09
2.5-3.0	0.00	0.00	0.00	0.03	0.00	0.00	0.00	0.03
3.0-3.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.5-4.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4.0-4.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	1.18	3.09	0.29	0.05	0.00	0.00	0.00	4.61

Winter Wave Frequency Distribution at Broadkill Beach (% Occurrence).
Directions are degrees Az from. Station location: 38-55 lat, 75-10 long

Dir=319.8-331.1				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.045	0.102	0.000	0.000	0.000	0.000	0.000	0.15
1.0-1.5	0.091	2.086	0.000	0.000	0.000	0.000	0.000	2.18
1.5-2.0	0.000	0.805	0.170	0.000	0.000	0.000	0.000	0.97
2.0-2.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
2.5-3.0	0.000	0.011	0.000	0.000	0.000	0.000	0.000	0.01
3.0-3.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.5-4.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
4.0-4.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
	0.14	3.00	0.17	0.00	0.00	0.00	0.00	3.31

Dir=331.1-353.6				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.08	0.10	0.00	0.00	0.00	0.00	0.00	0.18
1.0-1.5	0.09	1.81	0.00	0.00	0.00	0.00	0.00	1.90
1.5-2.0	0.01	0.40	0.28	0.00	0.00	0.00	0.00	0.69
2.0-2.5	0.00	0.05	0.00	0.00	0.00	0.00	0.00	0.05
2.5-3.0	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01
3.0-3.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.5-4.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4.0-4.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	0.18	2.37	0.28	0.00	0.00	0.00	0.00	2.83

Dir=353.6-16.1				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.08	0.06	0.00	0.00	0.00	0.00	0.00	0.14
1.0-1.5	0.16	2.88	0.00	0.00	0.00	0.00	0.00	3.04
1.5-2.0	0.00	0.40	0.10	0.00	0.00	0.00	0.00	0.50
2.0-2.5	0.01	0.08	0.09	0.00	0.00	0.00	0.00	0.18
2.5-3.0	0.00	0.03	0.00	0.00	0.00	0.00	0.00	0.03
3.0-3.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.5-4.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4.0-4.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	0.25	3.45	0.19	0.00	0.00	0.00	0.00	3.89

Dir=16.1-38.6				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.06	0.09	0.00	0.00	0.00	0.00	0.00	0.15
1.0-1.5	0.17	1.80	0.00	0.00	0.00	0.00	0.00	1.97
1.5-2.0	0.00	0.31	0.18	0.00	0.00	0.00	0.00	0.49
2.0-2.5	0.01	0.02	0.05	0.00	0.00	0.00	0.00	0.08
2.5-3.0	0.00	0.03	0.01	0.00	0.00	0.00	0.00	0.05
3.0-3.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.5-4.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4.0-4.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	0.24	2.26	0.24	0.00	0.00	0.00	0.00	2.73

Dir=38.6-61.1				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.12	0.12	0.00	0.00	0.00	0.00	0.00	0.25
1.0-1.5	0.25	1.58	0.00	0.00	0.00	0.00	0.00	1.83
1.5-2.0	0.00	0.36	0.09	0.00	0.00	0.00	0.00	0.45
2.0-2.5	0.01	0.06	0.07	0.00	0.00	0.00	0.00	0.14
2.5-3.0	0.00	0.03	0.02	0.00	0.00	0.00	0.00	0.06
3.0-3.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.5-4.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4.0-4.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	0.39	2.15	0.18	0.00	0.00	0.00	0.00	2.72
Dir=61.1-83.6				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.09	0.19	0.00	0.00	0.00	0.00	0.00	0.28
1.0-1.5	0.26	2.05	0.01	0.00	0.00	0.00	0.00	2.32
1.5-2.0	0.00	0.36	0.19	0.00	0.00	0.00	0.00	0.56
2.0-2.5	0.00	0.05	0.06	0.02	0.00	0.00	0.00	0.12
2.5-3.0	0.00	0.01	0.01	0.07	0.00	0.00	0.00	0.09
3.0-3.5	0.00	0.00	0.01	0.03	0.01	0.00	0.00	0.06
3.5-4.0	0.00	0.00	0.00	0.00	0.06	0.00	0.00	0.06
4.0-4.5	0.00	0.00	0.00	0.00	0.06	0.00	0.00	0.06
	0.35	2.66	0.28	0.12	0.12	0.00	0.00	3.55
Dir=83.6-106.1				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.07	0.35	0.00	0.00	0.00	0.00	0.00	0.42
1.0-1.5	0.32	2.63	0.02	0.00	0.00	0.00	0.00	2.97
1.5-2.0	0.00	0.23	0.34	0.00	0.00	0.00	0.00	0.57
2.0-2.5	0.00	0.03	0.26	0.08	0.00	0.00	0.00	0.37
2.5-3.0	0.00	0.01	0.01	0.18	0.01	0.00	0.00	0.22
3.0-3.5	0.00	0.00	0.00	0.02	0.00	0.00	0.00	0.02
3.5-4.0	0.00	0.00	0.00	0.01	0.02	0.00	0.00	0.03
4.0-4.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	0.39	3.25	0.63	0.29	0.03	0.00	0.00	4.60
Dir=106.1-128.6				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.10	0.19	0.00	0.00	0.00	0.00	0.00	0.29
1.0-1.5	0.12	1.27	0.05	0.00	0.00	0.00	0.00	1.44
1.5-2.0	0.01	0.09	0.07	0.00	0.00	0.00	0.00	0.17
2.0-2.5	0.00	0.01	0.06	0.02	0.00	0.00	0.00	0.09
2.5-3.0	0.00	0.00	0.00	0.06	0.00	0.00	0.00	0.06
3.0-3.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.5-4.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4.0-4.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	0.24	1.56	0.17	0.08	0.00	0.00	0.00	2.05

Dir=128.6-139.8				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.15	0.27	0.00	0.00	0.00	0.00	0.00	0.42
1.0-1.5	0.08	0.57	0.02	0.00	0.00	0.00	0.00	0.67
1.5-2.0	0.00	0.02	0.14	0.00	0.00	0.00	0.00	0.16
2.0-2.5	0.00	0.00	0.03	0.01	0.00	0.00	0.00	0.05
2.5-3.0	0.00	0.00	0.00	0.02	0.00	0.00	0.00	0.02
3.0-3.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.5-4.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4.0-4.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	0.23	0.86	0.19	0.03	0.00	0.00	0.00	1.32

Spring Wave Frequency Distribution at Broadkill Beach (% Occurrence).
Directions are degrees Az from. Station location: 38-55 lat, 75-10 long

Dir=319.8-331.1				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.034	0.000	0.000	0.000	0.000	0.000	0.000	0.03
1.0-1.5	0.624	2.324	0.000	0.000	0.000	0.000	0.000	2.95
1.5-2.0	0.011	0.170	0.023	0.000	0.000	0.000	0.000	0.20
2.0-2.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
2.5-3.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.0-3.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.5-4.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
4.0-4.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
	0.67	2.49	0.02	0.00	0.00	0.00	0.00	3.19

Dir=331.1-353.6				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.011	0.000	0.000	0.000	0.000	0.000	0.000	0.01
1.0-1.5	0.476	1.610	0.000	0.000	0.000	0.000	0.000	2.09
1.5-2.0	0.000	0.238	0.045	0.000	0.000	0.000	0.000	0.28
2.0-2.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
2.5-3.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.0-3.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.5-4.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
4.0-4.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
	0.49	1.85	0.05	0.00	0.00	0.00	0.00	2.38

Dir=353.6-16.1				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.045	0.000	0.000	0.000	0.000	0.000	0.000	0.05
1.0-1.5	0.601	2.109	0.000	0.000	0.000	0.000	0.000	2.71
1.5-2.0	0.000	0.125	0.068	0.000	0.000	0.000	0.000	0.19
2.0-2.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
2.5-3.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.0-3.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.5-4.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
4.0-4.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
	0.65	2.23	0.07	0.00	0.00	0.00	0.00	2.95

Dir=16.1-38.6				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.068	0.000	0.000	0.000	0.000	0.000	0.000	0.07
1.0-1.5	0.658	1.712	0.000	0.000	0.000	0.000	0.000	2.37
1.5-2.0	0.000	0.113	0.000	0.000	0.000	0.000	0.000	0.11
2.0-2.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
2.5-3.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.0-3.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.5-4.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
4.0-4.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
	0.73	1.83	0.00	0.00	0.00	0.00	0.00	2.55

Dir=38.6-61.1				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.079	0.000	0.000	0.000	0.000	0.000	0.000	0.08
1.0-1.5	0.442	1.814	0.000	0.000	0.000	0.000	0.000	2.26
1.5-2.0	0.000	0.215	0.023	0.000	0.000	0.000	0.000	0.24
2.0-2.5	0.000	0.057	0.011	0.000	0.000	0.000	0.000	0.07
2.5-3.0	0.000	0.000	0.011	0.000	0.000	0.000	0.000	0.01
3.0-3.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.5-4.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
4.0-4.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
	0.52	2.09	0.05	0.00	0.00	0.00	0.00	2.65
Dir=61.1-83.6				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
1.0-1.5	0.703	1.394	0.000	0.000	0.000	0.000	0.000	2.10
1.5-2.0	0.000	0.181	0.068	0.000	0.000	0.000	0.000	0.25
2.0-2.5	0.000	0.023	0.045	0.000	0.000	0.000	0.000	0.07
2.5-3.0	0.000	0.000	0.000	0.023	0.000	0.011	0.000	0.03
3.0-3.5	0.000	0.000	0.000	0.045	0.034	0.000	0.000	0.08
3.5-4.0	0.000	0.000	0.000	0.000	0.034	0.000	0.000	0.03
4.0-4.5	0.000	0.000	0.000	0.000	0.023	0.000	0.000	0.02
	0.70	1.60	0.11	0.07	0.09	0.01	0.00	2.58
Dir=83.6-106.1				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.057	0.000	0.000	0.000	0.000	0.000	0.000	0.06
1.0-1.5	0.941	2.290	0.000	0.000	0.000	0.000	0.000	3.23
1.5-2.0	0.011	0.068	0.136	0.000	0.000	0.000	0.000	0.22
2.0-2.5	0.000	0.011	0.079	0.034	0.000	0.000	0.000	0.12
2.5-3.0	0.000	0.000	0.000	0.068	0.000	0.000	0.000	0.07
3.0-3.5	0.000	0.000	0.000	0.011	0.000	0.000	0.000	0.01
3.5-4.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
4.0-4.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
	1.01	2.37	0.22	0.11	0.00	0.00	0.00	3.71
Dir=106.1-128.6				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.091	0.000	0.000	0.000	0.000	0.000	0.000	0.09
1.0-1.5	0.521	1.179	0.000	0.000	0.000	0.000	0.000	1.70
1.5-2.0	0.000	0.079	0.045	0.000	0.000	0.000	0.000	0.12
2.0-2.5	0.000	0.000	0.011	0.000	0.000	0.000	0.000	0.01
2.5-3.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.0-3.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.5-4.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
4.0-4.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
	0.61	1.26	0.06	0.00	0.00	0.00	0.00	1.93

Dir=128.6-139.8				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.034	0.000	0.000	0.000	0.000	0.000	0.000	0.03
1.0-1.5	0.193	0.748	0.000	0.000	0.000	0.000	0.000	0.94
1.5-2.0	0.000	0.102	0.000	0.000	0.000	0.000	0.000	0.10
2.0-2.5	0.000	0.011	0.000	0.000	0.000	0.000	0.000	0.01
2.5-3.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.0-3.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.5-4.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
4.0-4.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
	0.23	0.86	0.00	0.00	0.00	0.00	0.00	1.09

Summer Wave Frequency Distribution at Broadkill Beach (% Occurrence).
Directions are degrees Az from. Station location: 38-55 lat, 75-10 long

Dir=319.8-331.1				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.147	0.000	0.000	0.000	0.000	0.000	0.000	0.15
1.0-1.5	0.669	1.496	0.000	0.000	0.000	0.000	0.000	2.17
1.5-2.0	0.011	0.000	0.000	0.000	0.000	0.000	0.000	0.01
2.0-2.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
2.5-3.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.0-3.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.5-4.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
4.0-4.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
	0.83	1.50	0.00	0.00	0.00	0.00	0.00	2.32

Dir=331.1-353.6				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.068	0.000	0.000	0.000	0.000	0.000	0.000	0.07
1.0-1.5	0.374	0.964	0.000	0.000	0.000	0.000	0.000	1.34
1.5-2.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
2.0-2.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
2.5-3.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.0-3.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.5-4.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
4.0-4.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
	0.44	0.96	0.00	0.00	0.00	0.00	0.00	1.41

Dir=353.6-16.1				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.159	0.000	0.000	0.000	0.000	0.000	0.000	0.16
1.0-1.5	0.726	2.018	0.000	0.000	0.000	0.000	0.000	2.74
1.5-2.0	0.011	0.068	0.000	0.000	0.000	0.000	0.000	0.08
2.0-2.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
2.5-3.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.0-3.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.5-4.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
4.0-4.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
	0.90	2.09	0.00	0.00	0.00	0.00	0.00	2.98

Dir=16.1-38.6				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.091	0.000	0.000	0.000	0.000	0.000	0.000	0.09
1.0-1.5	0.601	1.134	0.000	0.000	0.000	0.000	0.000	1.73
1.5-2.0	0.000	0.113	0.000	0.000	0.000	0.000	0.000	0.11
2.0-2.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
2.5-3.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.0-3.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.5-4.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
4.0-4.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
	0.69	1.25	0.00	0.00	0.00	0.00	0.00	1.94

Dir=38.6-61.1				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.079	0.000	0.000	0.000	0.000	0.000	0.000	0.08
1.0-1.5	0.748	1.984	0.000	0.000	0.000	0.000	0.000	2.73
1.5-2.0	0.000	0.023	0.068	0.000	0.000	0.000	0.000	0.09
2.0-2.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
2.5-3.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.0-3.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.5-4.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
4.0-4.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
	0.83	2.01	0.07	0.00	0.00	0.00	0.00	2.90
Dir=61.1-83.6				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.125	0.000	0.000	0.000	0.000	0.000	0.000	0.12
1.0-1.5	0.998	2.301	0.023	0.000	0.000	0.000	0.000	3.32
1.5-2.0	0.000	0.136	0.136	0.000	0.000	0.000	0.000	0.27
2.0-2.5	0.000	0.000	0.023	0.000	0.000	0.000	0.000	0.02
2.5-3.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.0-3.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.5-4.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
4.0-4.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
	1.12	2.44	0.18	0.00	0.00	0.00	0.00	3.74
Dir=83.6-106.1				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.170	0.000	0.000	0.000	0.000	0.000	0.000	0.17
1.0-1.5	1.689	2.993	0.023	0.000	0.000	0.000	0.000	4.70
1.5-2.0	0.000	0.023	0.057	0.000	0.000	0.000	0.000	0.08
2.0-2.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
2.5-3.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.0-3.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.5-4.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
4.0-4.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
	1.86	3.02	0.08	0.00	0.00	0.00	0.00	4.95
Dir=106.1-128.6				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.113	0.000	0.000	0.000	0.000	0.000	0.000	0.11
1.0-1.5	1.111	1.530	0.000	0.000	0.000	0.000	0.000	2.64
1.5-2.0	0.000	0.045	0.023	0.000	0.000	0.000	0.000	0.07
2.0-2.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
2.5-3.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.0-3.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.5-4.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
4.0-4.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
	1.22	1.58	0.02	0.00	0.00	0.00	0.00	2.82

Dir=128.6-139.8				T(p) sec					
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15		
0.0-1.0	0.034	0.000	0.000	0.000	0.000	0.000	0.000		0.03
1.0-1.5	0.442	0.590	0.000	0.000	0.000	0.000	0.000		1.03
1.5-2.0	0.000	0.000	0.011	0.000	0.000	0.000	0.000		0.01
2.0-2.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000		0.00
2.5-3.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000		0.00
3.0-3.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000		0.00
3.5-4.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000		0.00
4.0-4.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000		0.00
	0.000	0.000	0.000	0.000	0.000	0.000	0.000		0.00

Autumn Wave Frequency Distribution at Broadkill Beach (% Occurrence).
Directions are degrees Az from. Station location: 38-55 lat, 75-10 long

Dir=319.8-331.1				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.023	0.011	0.000	0.000	0.000	0.000	0.000	0.03
1.0-1.5	0.147	2.596	0.000	0.000	0.000	0.000	0.000	2.74
1.5-2.0	0.000	0.578	0.215	0.000	0.000	0.000	0.000	0.79
2.0-2.5	0.000	0.000	0.102	0.000	0.000	0.000	0.000	0.10
2.5-3.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.0-3.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.5-4.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
4.0-4.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
	0.17	3.19	0.32	0.00	0.00	0.00	0.00	3.67

Dir=331.1-353.6				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.011	0.011	0.000	0.000	0.000	0.000	0.000	0.02
1.0-1.5	0.091	1.757	0.000	0.000	0.000	0.000	0.000	1.85
1.5-2.0	0.000	0.261	0.181	0.000	0.000	0.000	0.000	0.44
2.0-2.5	0.000	0.023	0.057	0.000	0.000	0.000	0.000	0.08
2.5-3.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.0-3.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.5-4.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
4.0-4.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
	0.10	2.05	0.24	0.00	0.00	0.00	0.00	2.39

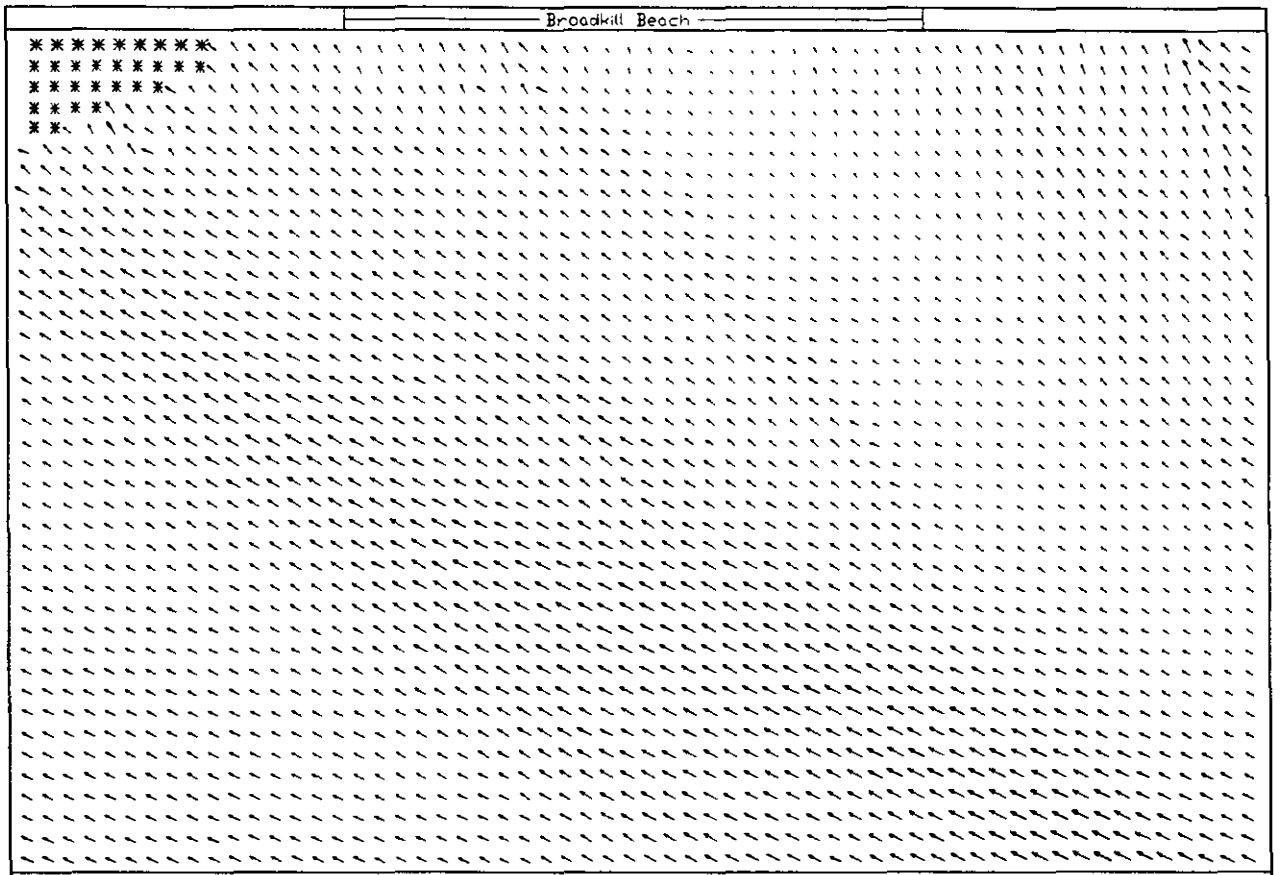
Dir=353.6-16.1				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.023	0.091	0.000	0.000	0.000	0.000	0.000	0.11
1.0-1.5	0.283	2.573	0.000	0.000	0.000	0.000	0.000	2.86
1.5-2.0	0.000	0.465	0.249	0.000	0.000	0.000	0.000	0.71
2.0-2.5	0.000	0.011	0.170	0.000	0.000	0.000	0.000	0.18
2.5-3.0	0.000	0.000	0.034	0.000	0.000	0.000	0.000	0.03
3.0-3.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.5-4.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
4.0-4.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
	0.31	3.14	0.45	0.00	0.00	0.00	0.00	3.90

Dir=16.1-38.6				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.011	0.023	0.000	0.000	0.000	0.000	0.000	0.03
1.0-1.5	0.227	2.313	0.000	0.000	0.000	0.000	0.000	2.54
1.5-2.0	0.000	0.306	0.170	0.000	0.000	0.000	0.000	0.48
2.0-2.5	0.000	0.011	0.045	0.000	0.000	0.000	0.000	0.06
2.5-3.0	0.000	0.000	0.023	0.000	0.000	0.000	0.000	0.02
3.0-3.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.5-4.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
4.0-4.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
	0.24	2.65	0.24	0.00	0.00	0.00	0.00	3.13

Dir=38.6-61.1				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.000	0.034	0.000	0.000	0.000	0.000	0.000	0.03
1.0-1.5	0.227	2.494	0.000	0.000	0.000	0.000	0.000	2.72
1.5-2.0	0.000	0.295	0.113	0.000	0.000	0.000	0.000	0.41
2.0-2.5	0.000	0.011	0.227	0.000	0.000	0.000	0.000	0.24
2.5-3.0	0.000	0.000	0.091	0.000	0.000	0.000	0.000	0.09
3.0-3.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.5-4.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
4.0-4.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
	0.23	2.83	0.43	0.00	0.00	0.00	0.00	3.49
Dir=61.1-83.6				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.023	0.045	0.000	0.000	0.000	0.000	0.000	0.07
1.0-1.5	0.249	1.598	0.000	0.000	0.000	0.000	0.000	1.85
1.5-2.0	0.000	0.147	0.023	0.000	0.000	0.000	0.000	0.17
2.0-2.5	0.000	0.011	0.045	0.011	0.000	0.000	0.000	0.07
2.5-3.0	0.000	0.000	0.023	0.011	0.000	0.000	0.000	0.03
3.0-3.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.5-4.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
4.0-4.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
	0.27	1.80	0.09	0.02	0.00	0.00	0.00	2.19
Dir=83.6-106.1				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.023	0.068	0.000	0.000	0.000	0.000	0.000	0.09
1.0-1.5	0.601	2.766	0.011	0.000	0.000	0.000	0.000	3.38
1.5-2.0	0.000	0.057	0.351	0.000	0.000	0.000	0.000	0.41
2.0-2.5	0.000	0.000	0.045	0.034	0.000	0.000	0.000	0.08
2.5-3.0	0.000	0.000	0.000	0.113	0.000	0.000	0.000	0.11
3.0-3.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.5-4.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
4.0-4.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
	0.62	2.89	0.41	0.15	0.00	0.00	0.00	4.07
Dir=106.1-128.6				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.023	0.034	0.000	0.000	0.000	0.000	0.000	0.06
1.0-1.5	0.306	1.213	0.034	0.000	0.000	0.000	0.000	1.55
1.5-2.0	0.000	0.079	0.079	0.000	0.000	0.000	0.000	0.16
2.0-2.5	0.000	0.000	0.023	0.034	0.000	0.000	0.000	0.06
2.5-3.0	0.000	0.000	0.000	0.023	0.000	0.000	0.000	0.02
3.0-3.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.5-4.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
4.0-4.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
	0.33	1.33	0.14	0.06	0.00	0.00	0.00	1.85

Dir=128.6-139.8				T(p) sec				
H(s) m	0-3	3-5	5-7	7-9	9-11	11-13	13-15	
0.0-1.0	0.045	0.000	0.000	0.000	0.000	0.000	0.000	0.05
1.0-1.5	0.204	0.726	0.023	0.000	0.000	0.000	0.000	0.95
1.5-2.0	0.000	0.045	0.045	0.000	0.000	0.000	0.000	0.09
2.0-2.5	0.000	0.011	0.023	0.000	0.000	0.000	0.000	0.03
2.5-3.0	0.000	0.000	0.000	0.011	0.000	0.000	0.000	0.01
3.0-3.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
3.5-4.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
4.0-4.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
	0.25	0.78	0.09	0.01	0.00	0.00	0.00	1.13

RCPWAVE Output Results



RCPWAVE output for T= 3 sec, Dir= 342deg Az

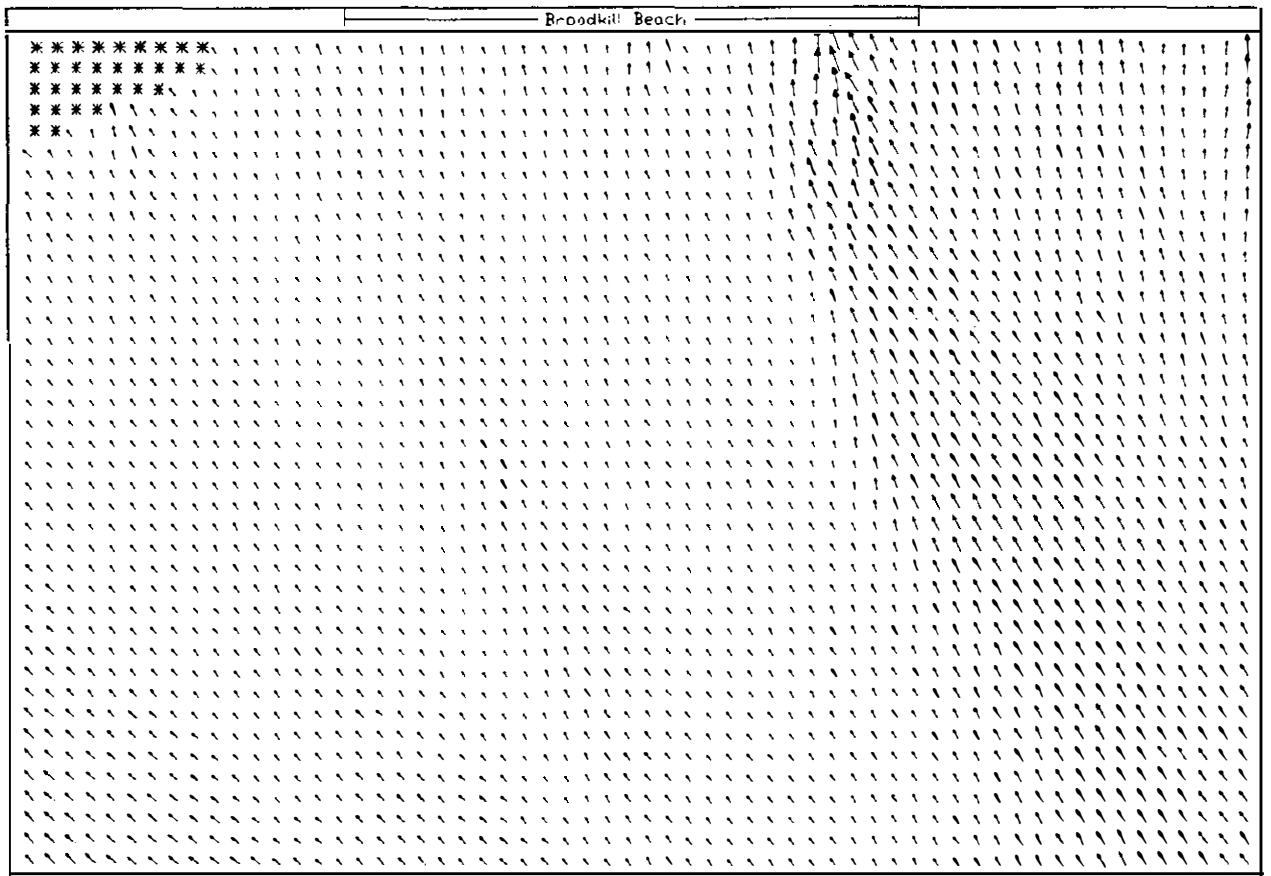
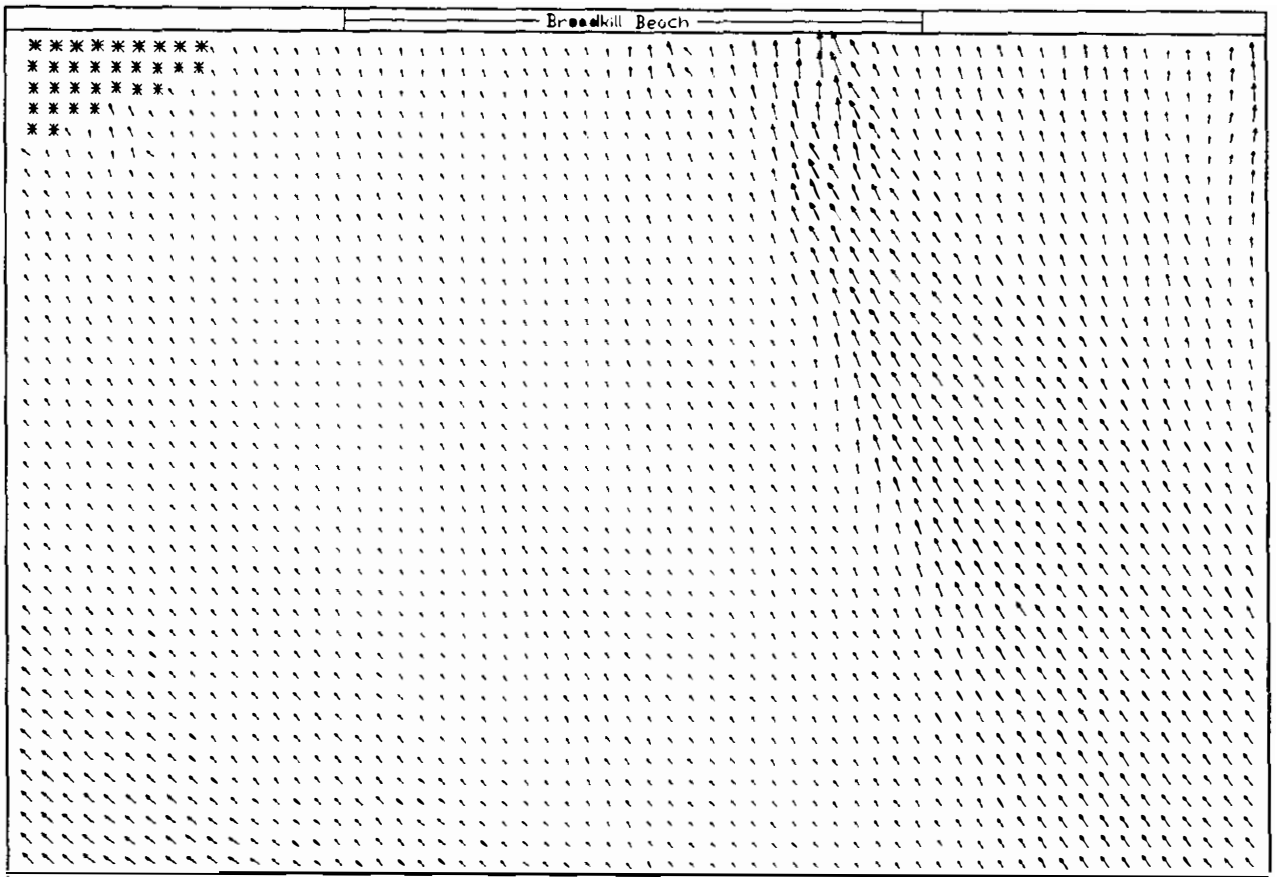
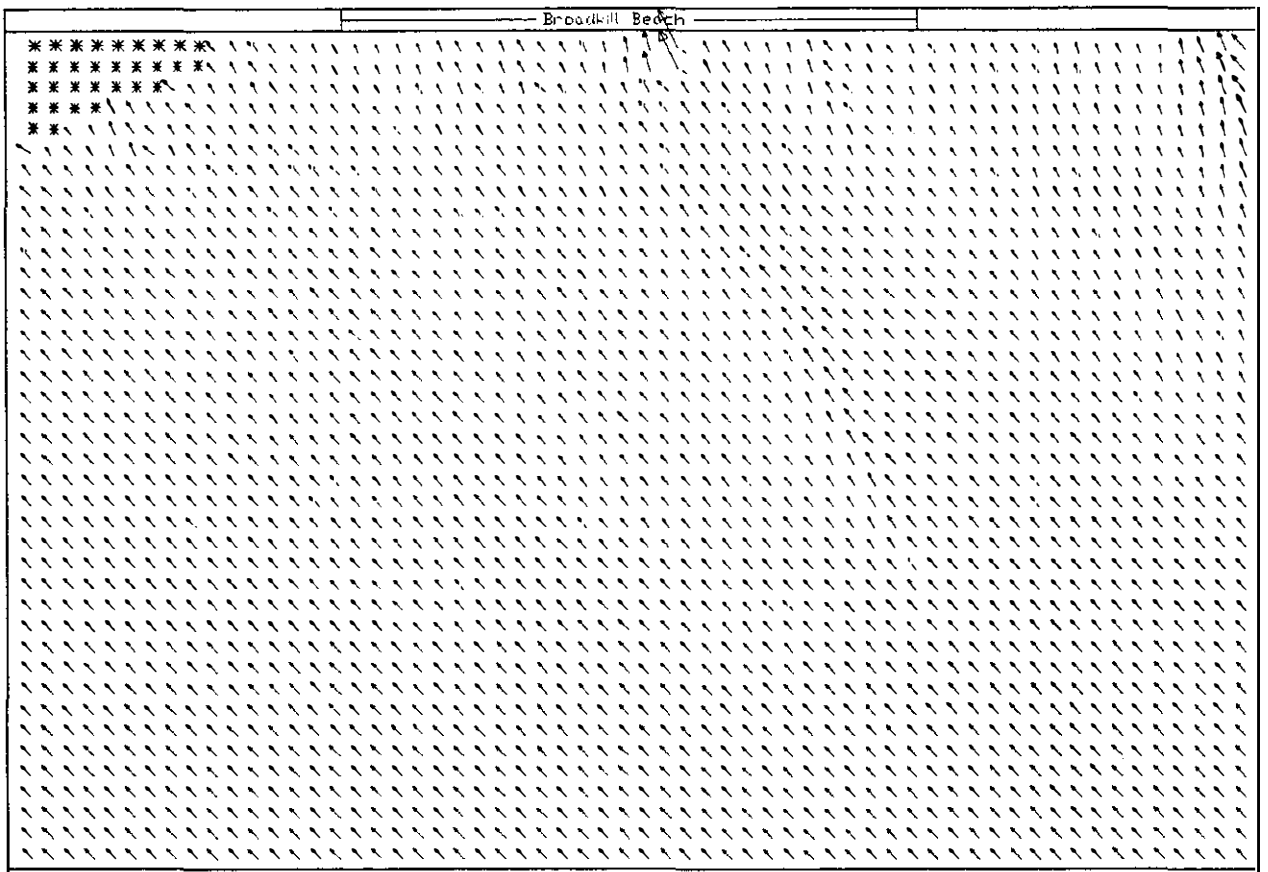


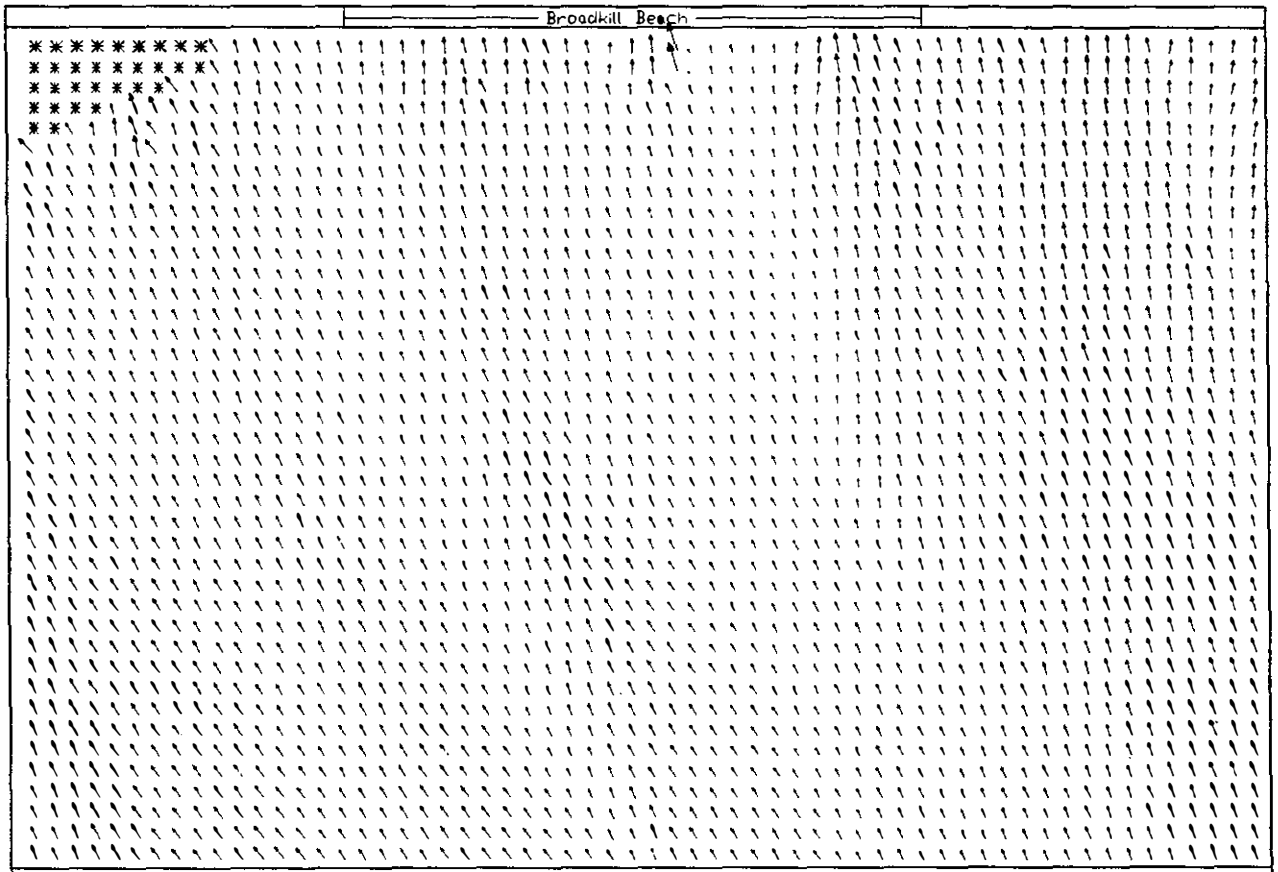
Figure 48. RCPWAVE results for T=6 sec, Dir= 342 deg Az



RCPWAVE output for T=6 sec, Dir=325 deg Az



RCPWAVE output for T=3 sec. Dir= 4 deg Az



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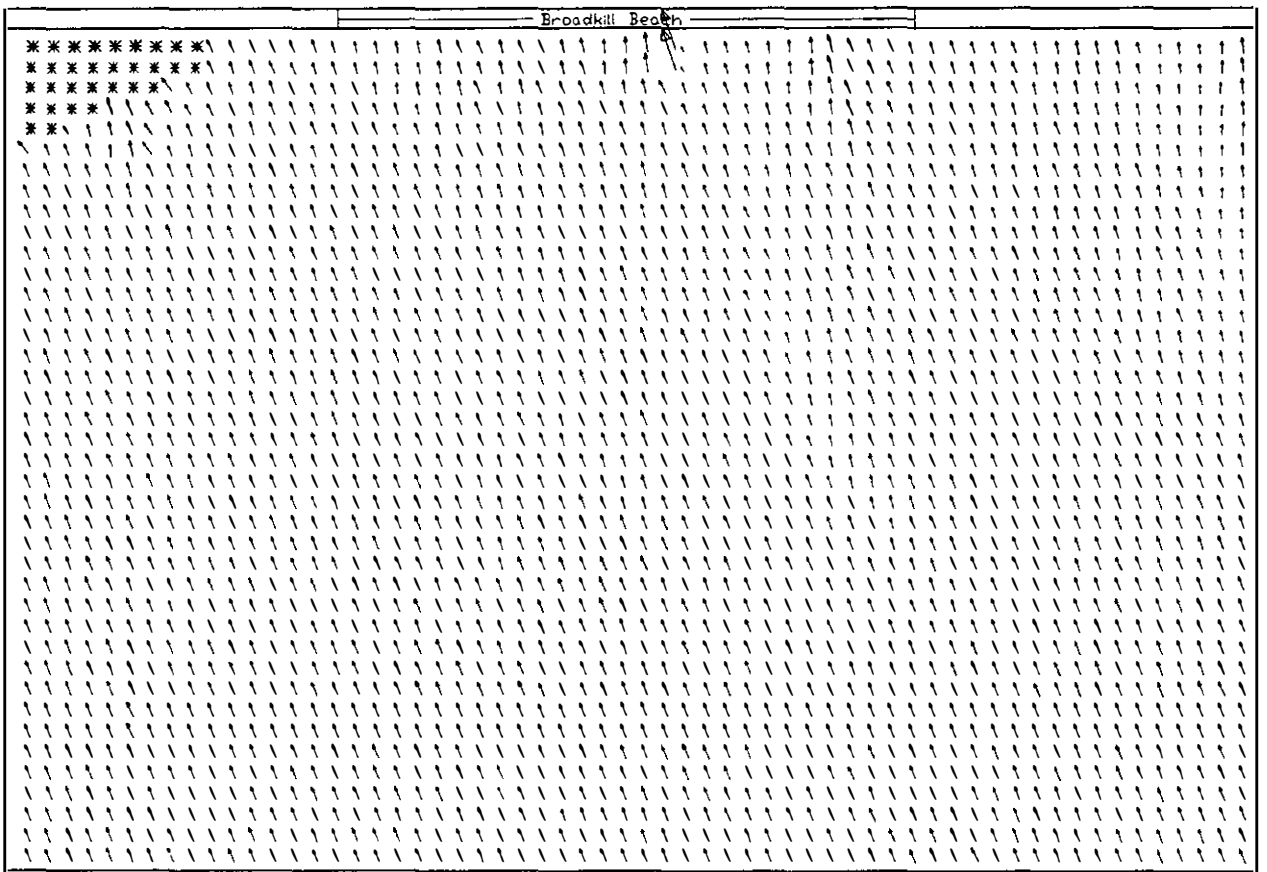
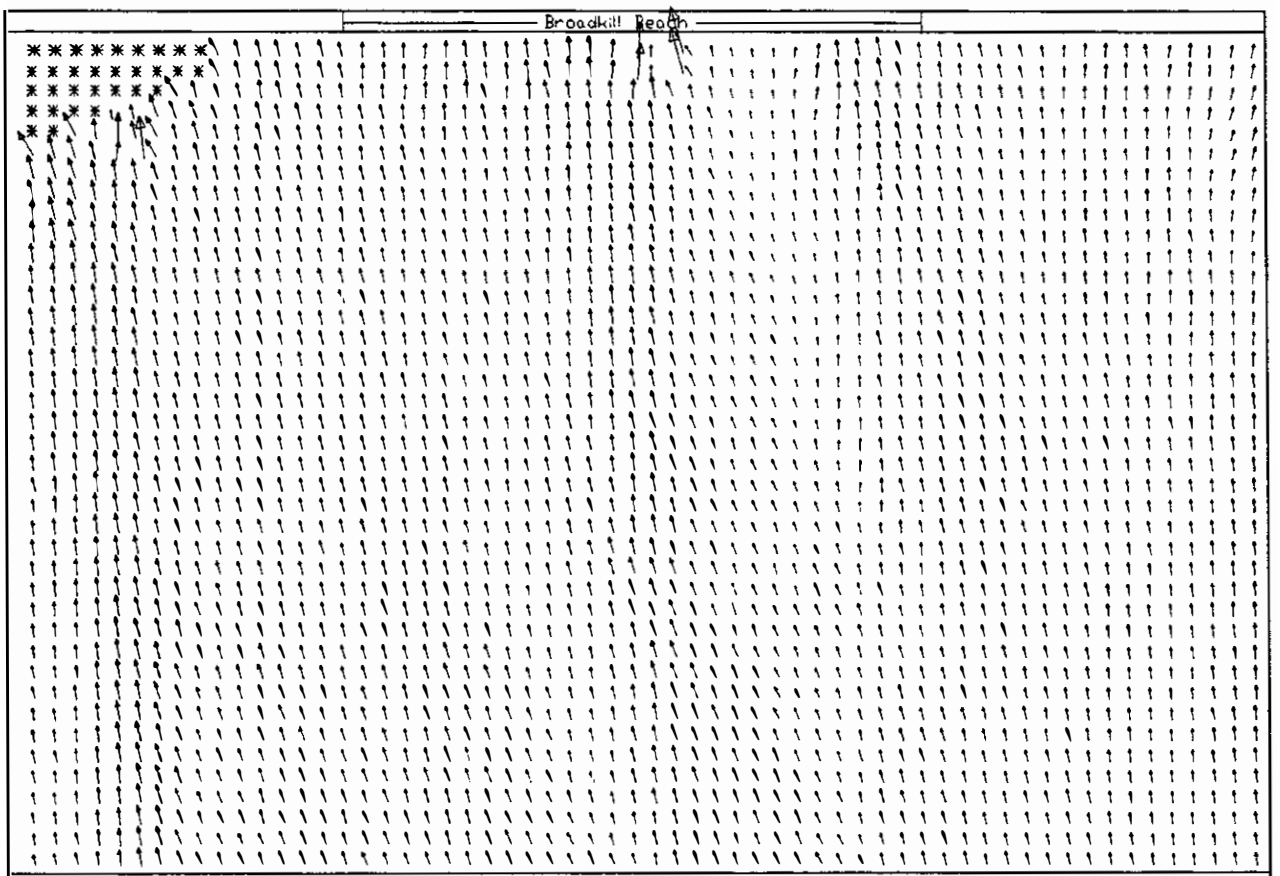


RCPWAVE output for T= 3 sec, Dir= 27 deg Az



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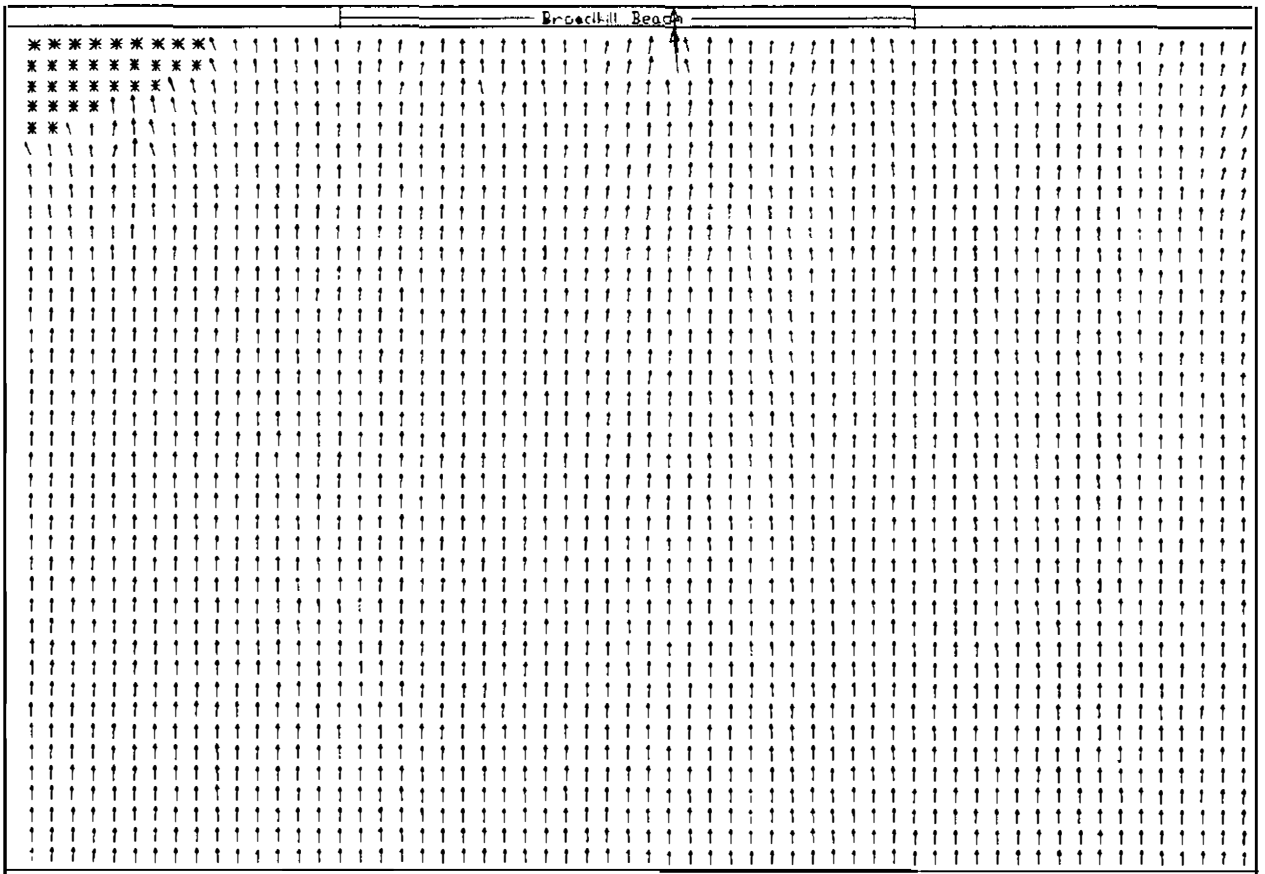
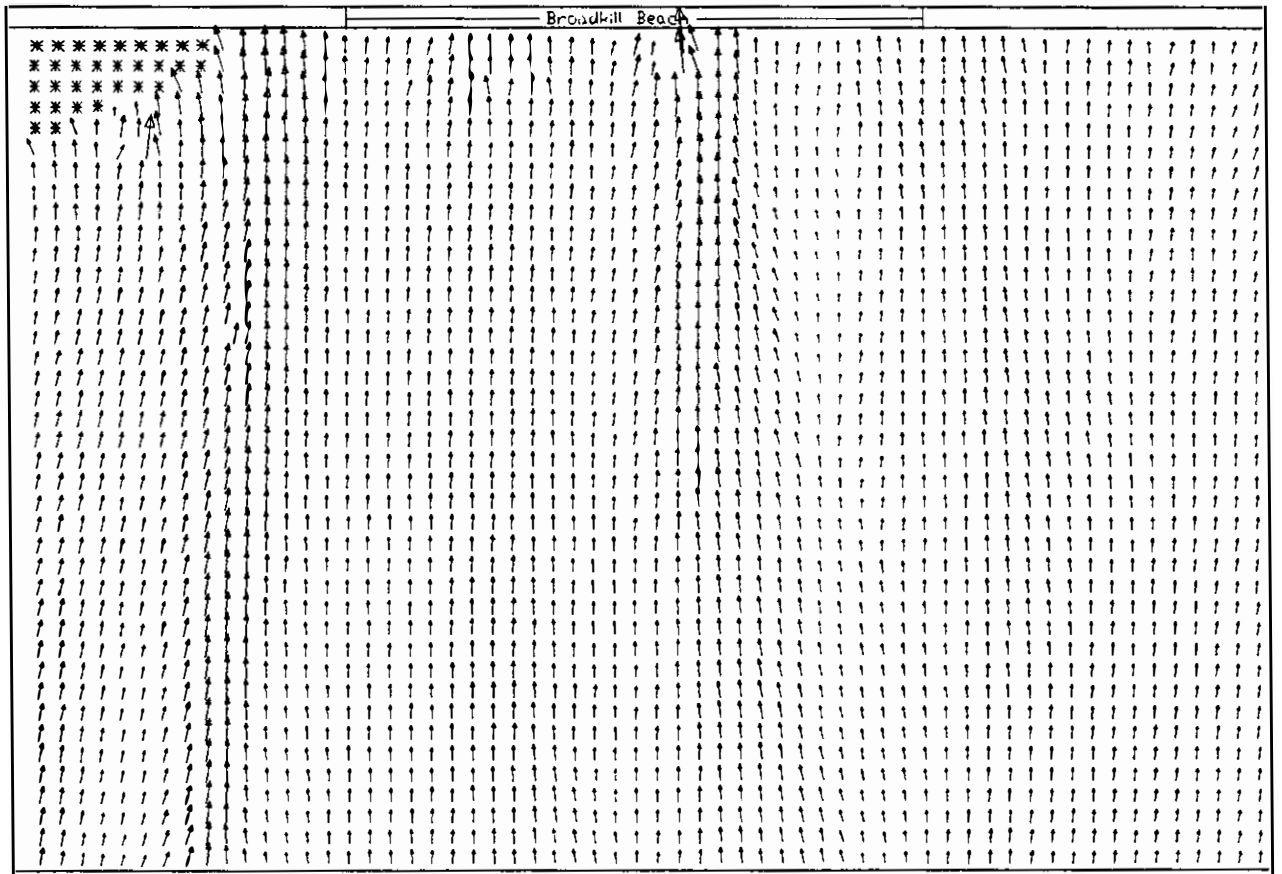
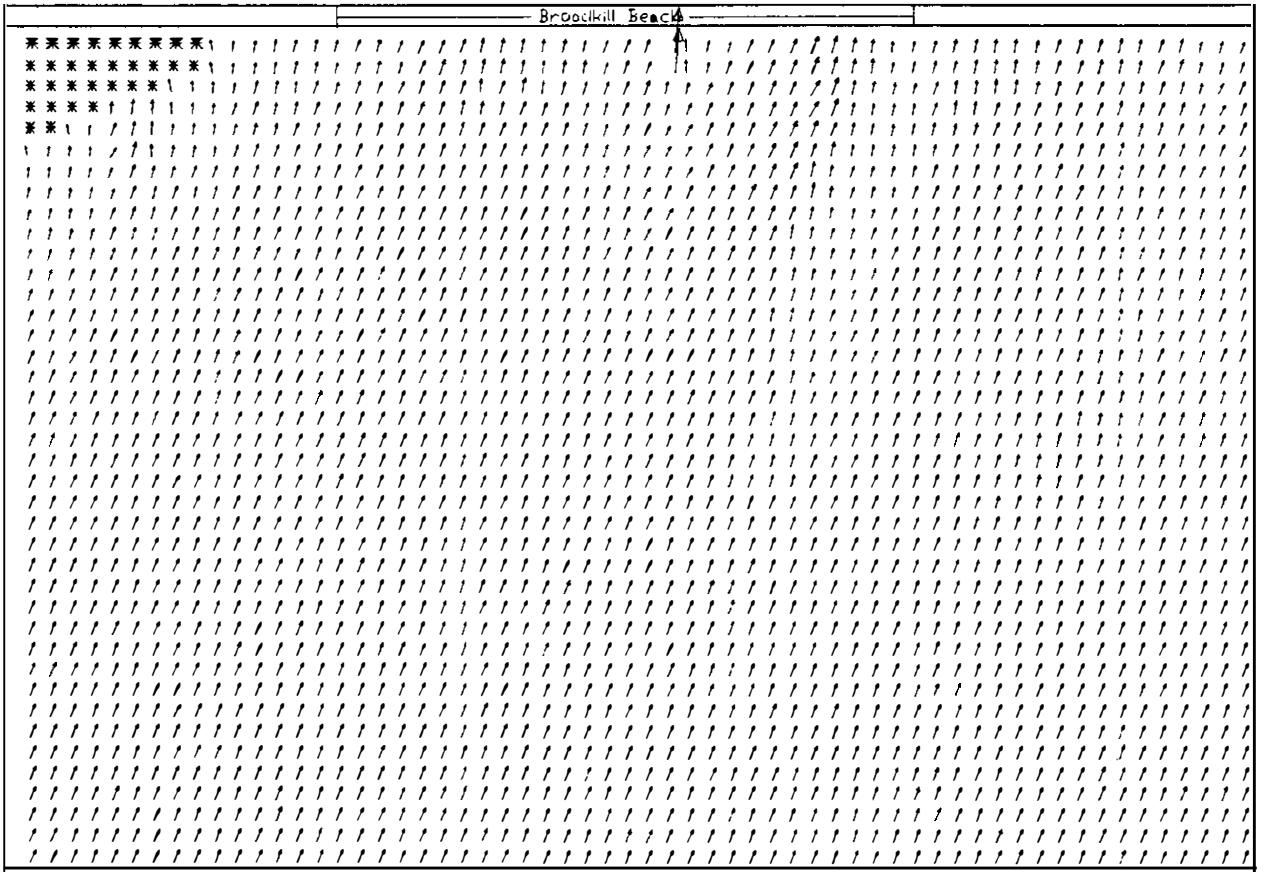


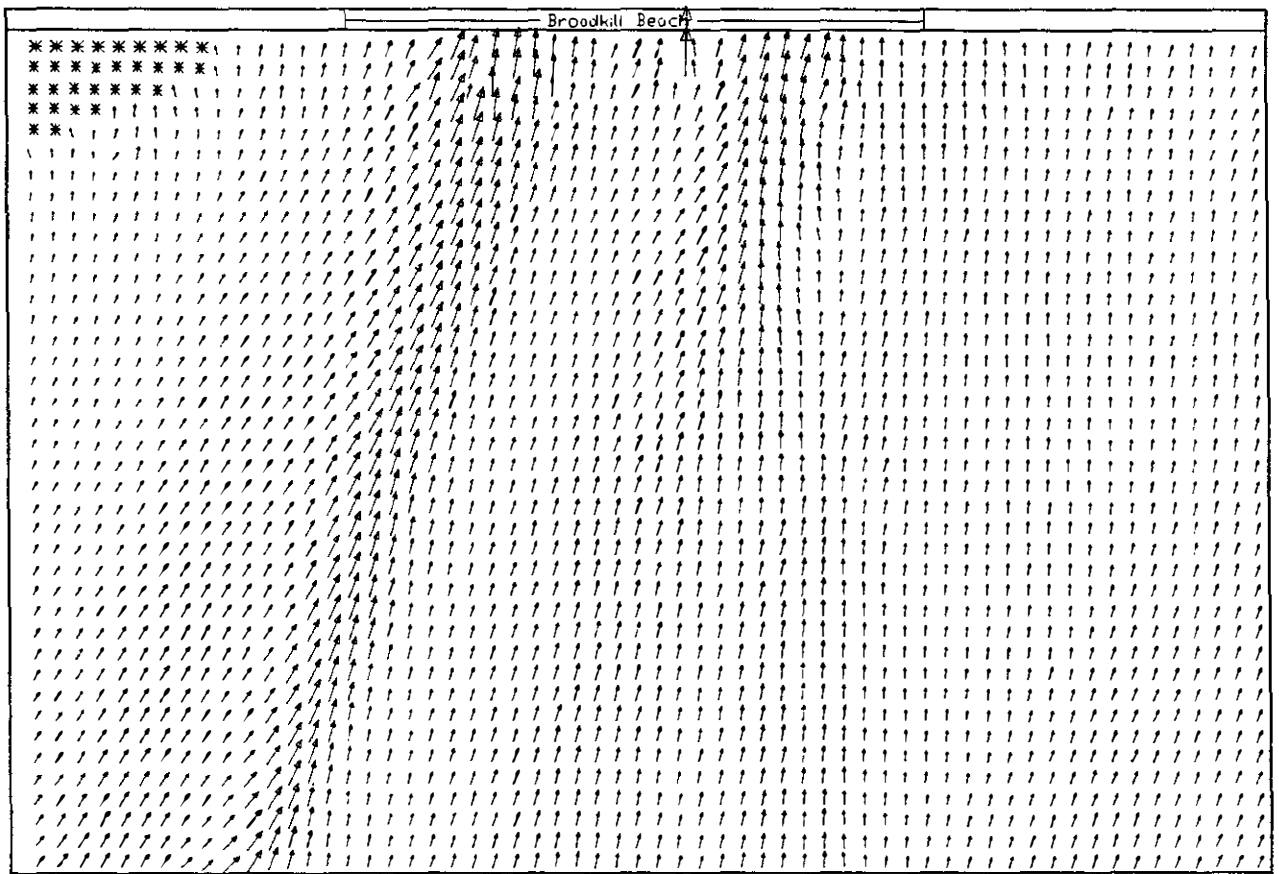
RCPWAVE output for T= 3 sec, Dir= 49 deg Az



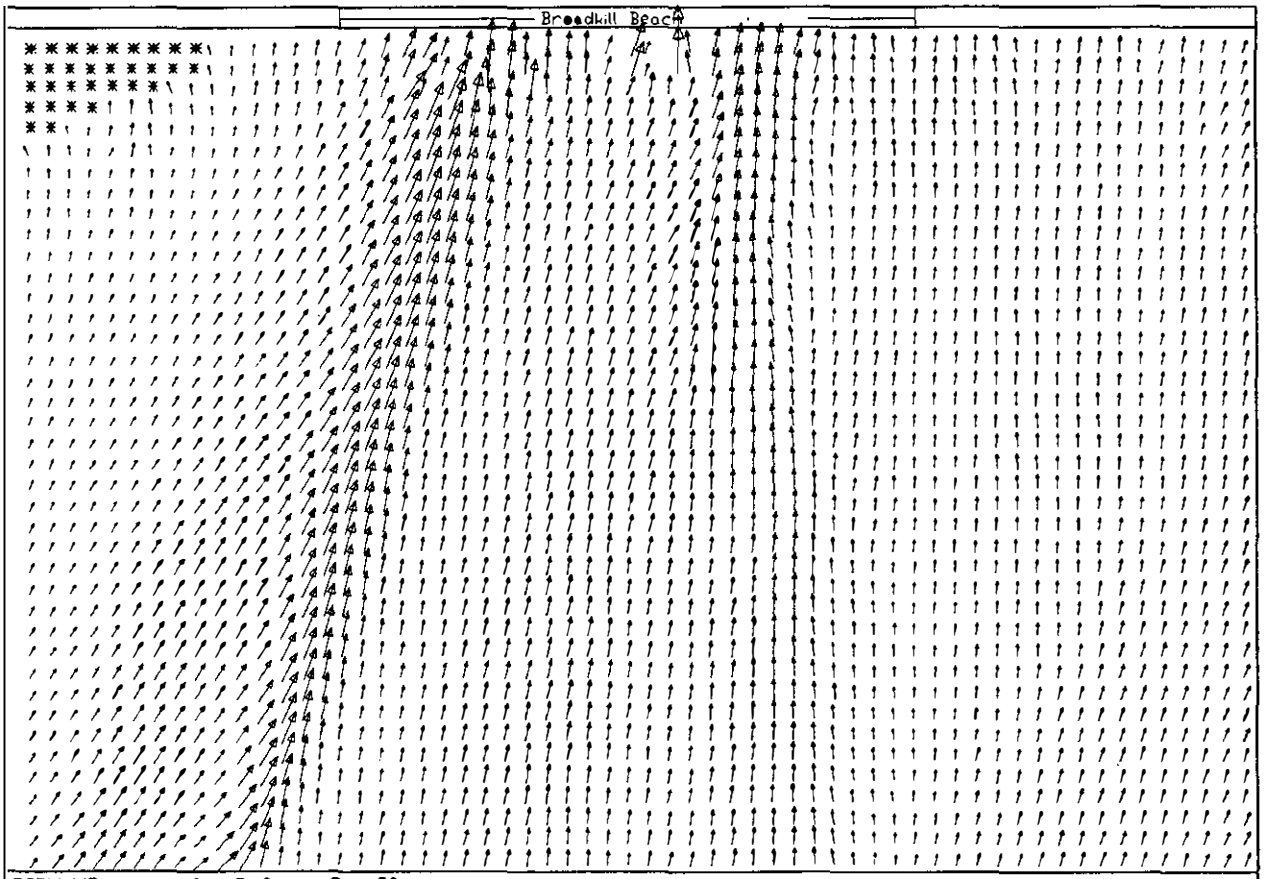
RCPWAVE output for T=6 sec, Dir= 49 deg Az



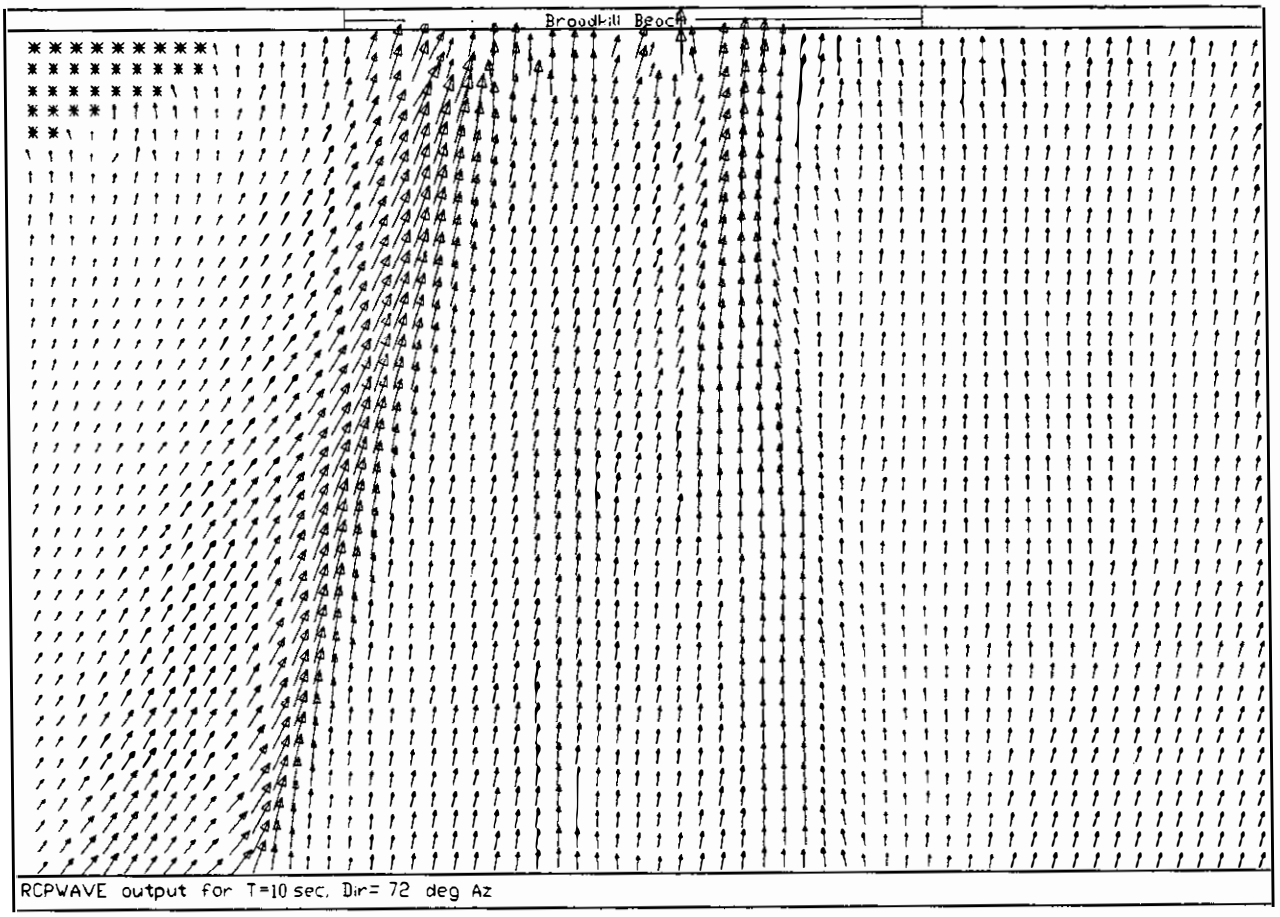
RCPWAVE output for T=3 sec, Dir= 72 deg Az

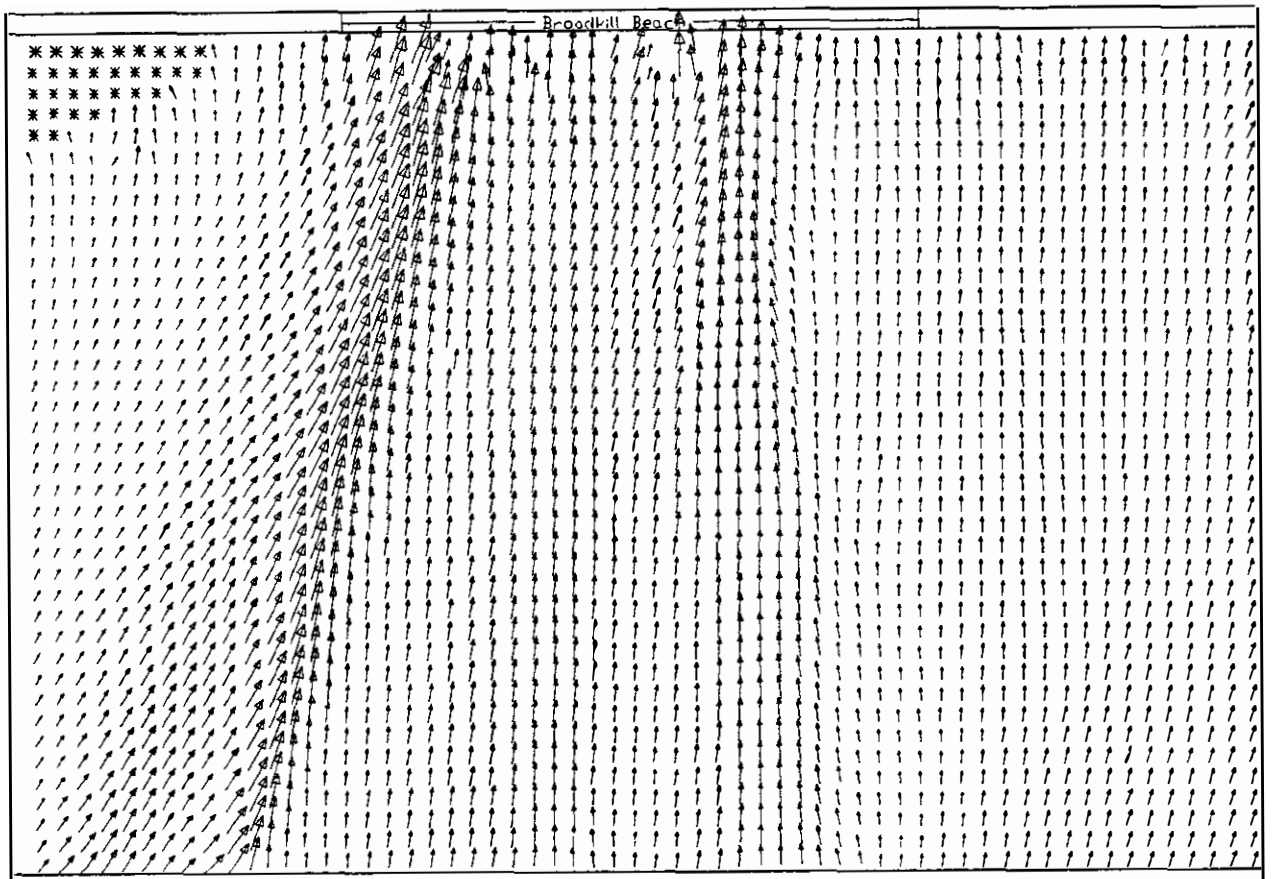


RCPWAVE output for T=6 sec, Dir= 72 deg Az

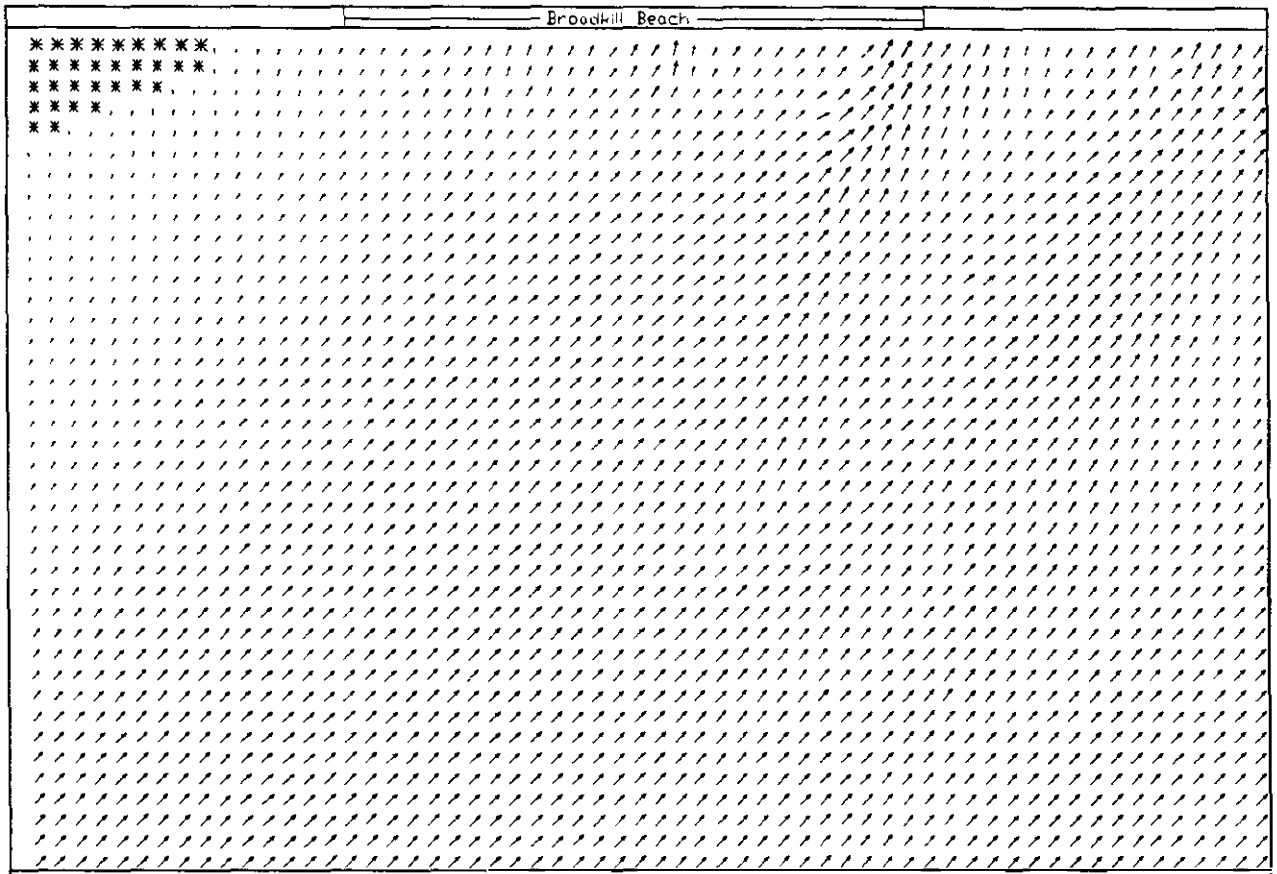


RCPWAVE output for T=8 sec, Dir= 72 deg Az

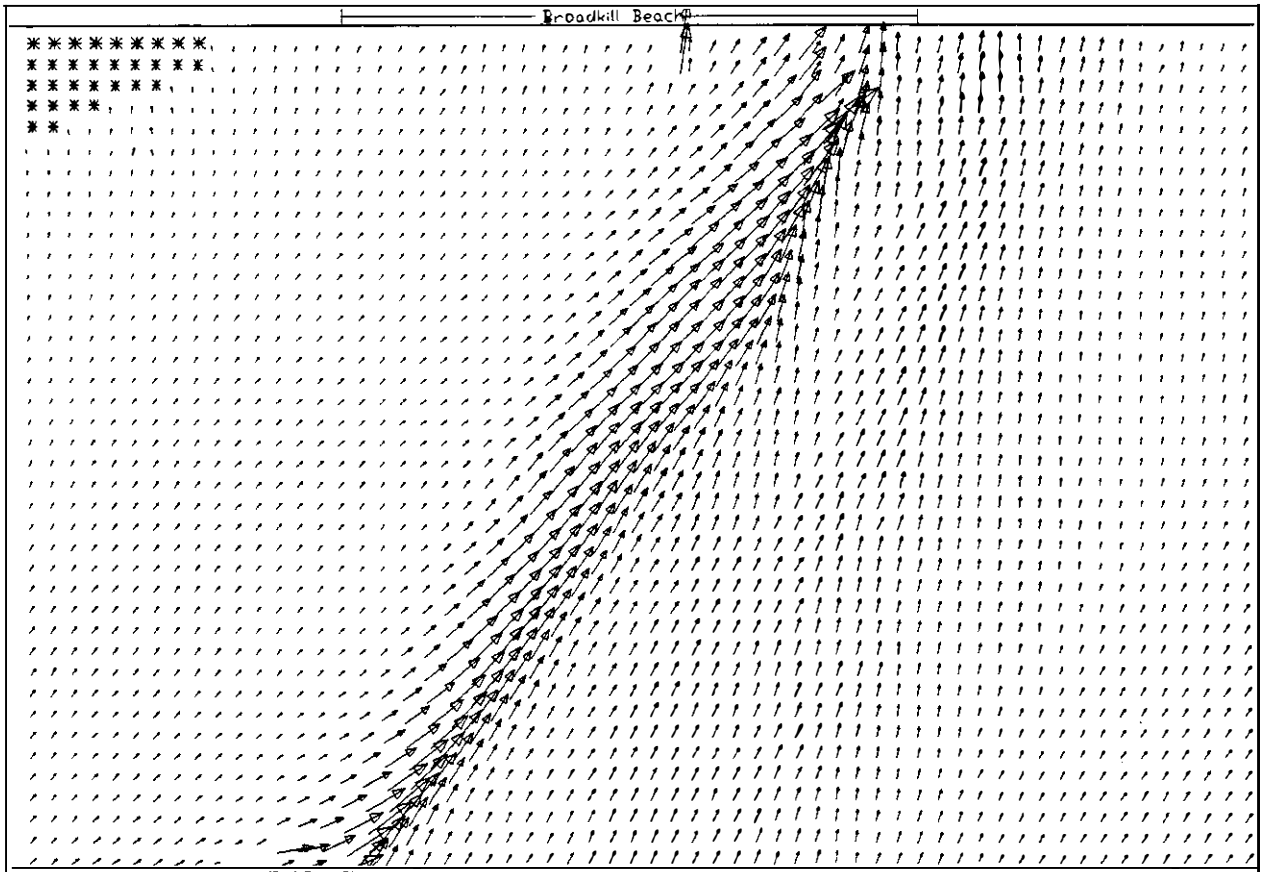

Broadhill Beach
RCPWAVE output for T=10 sec, Dir= 72 deg Az



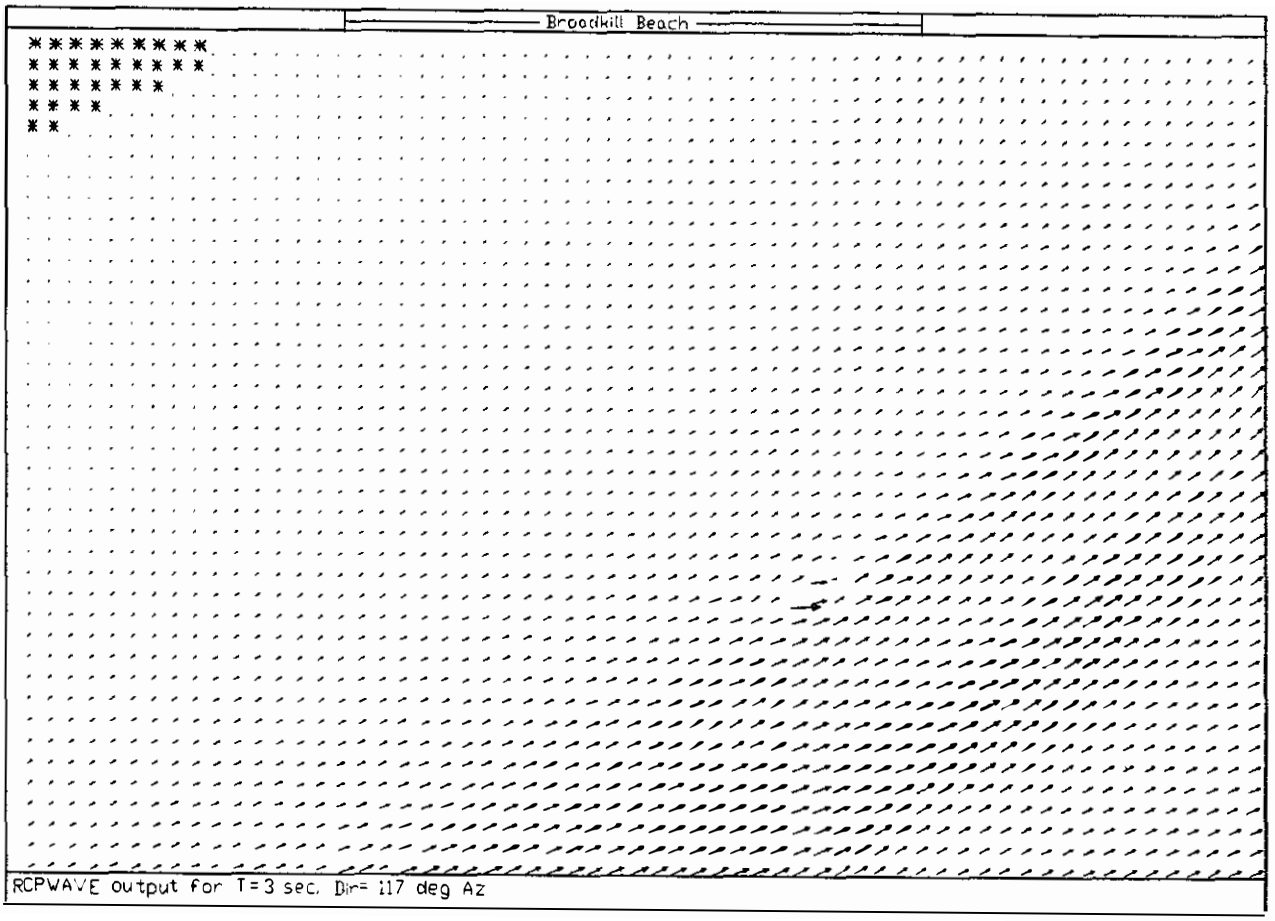
RCPWAVE output for T=12sec. Dir= 72 deg Az

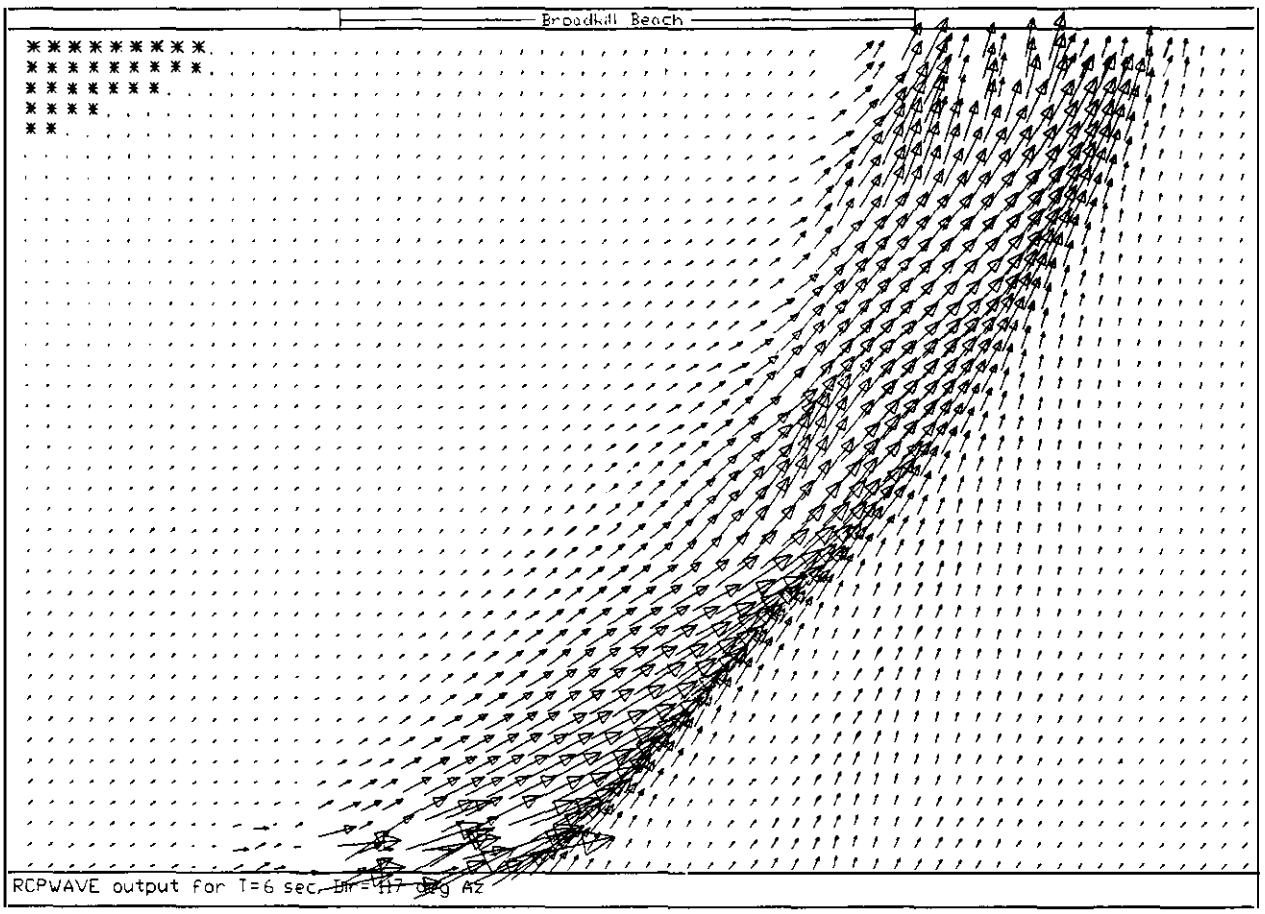


RCPWAVE output for T=3 sec, Dir= 94 deg Az



RCPWAVE output for T=8 sec. Dir= 94 deg Az


Broadkill Beach
RCPWAVE output for T=3 sec. Dir= 117 deg Az


Broadkill Beach
RCPWAVE output for T=6 sec, Dir=117 deg AZ

PHASE I REPORT

BROADKILL BEACH
EROSION AND FLOODING PROTECTION

APPENDIX E
COMPARISON OF DESIGN STORM AND STORM-ENSEMBLE APPROACHES FOR ANALYZING STORM-INDUCED EROSIONS

APPENDIX E
COMPARISON OF DESIGN STORM AND STORM-ENSEMBLE APPROACHES FOR ANALYZING STORM-INDUCED EROSIONS

A problem with the design-storm approach is that in addition to peak stage, profile change is sensitive to storm duration, shape of the surge envelope, wave height, and wave period. The maximum water level for the surge of a design storm can produce less erosion than a storm of lower surge but greater duration, or a storm of lower surge but greater wave heights. Even so, in the absence of an extensive storm data base, this approach is commonly used.

The storm-ensemble approach is preferred since it solves the many-to-one relationship between beach erosion and stage frequency. Also, the Delaware Bay wave and surge study provides the needed hurricane and northeaster time series. This approach however, would require simulation of 30 storms for each of the 8 profiles, 240 simulations total. The results for each profile would undergo a frequency analysis to determine erosion frequency curves. At this point, it would be necessary to prepare eroded beach profiles for each of the return periods of the frequency curve. If the dune volumes are large enough, relatively small storms will yield no dune crest recession and the frequency curves so determined would have too few points to be meaningful.

Alternately, the design-storm approach was used for Broadkill Beach to determine beach erosion and dune crest recession. The resulting eroded profiles were then used in the wave attack and inundation analyses. The storm-ensemble approach was also used for profiles LRP#25, LRP#26, and LRP#27A to check the design-storm results. Tables E-1 AND E-2 compare the results from the two approaches.

TABLE E-1. COMPARISON OF EROSION VOLUME ABOVE NGVD (cubic yard/feet)								
PROFILE	METHOD	RETURN FREQUENCY						
		5	10	20	50	100	200	500
LRP#25	1	162	226	269	345	388	474	538
	2	205	291	334	388	420	452	517
LRP#26	1	183	248	312	377	420	474	528
	2	248	323	355	420	452	495	549
LRP#27A	1	226	291	355	431	484	538	603
	2	280	345	388	431	463	484	517

METHOD 1: base on frequency analysis of 30 storms
METHOD 2: use synthetic storm hydrograph

TABLE E-2. COMPARISON OF DUNE CREST EROSION POSITION (feet from baseline)								
PROFILE	METHOD	RETURN FREQUENCY						
		5	10	20	50	100	200	500
LRP#25	1	36	56	72	92	108	121	144
	2	20	53	85	92	98	98	121
LRP#26	1	36	49	59	72	82	89	95
	2	33	49	53	59	62	62	72
LRP#27A	1	30	49	62	85	95	105	112
	2	20	36	79	92	92	98	102

METHOD 1: base on frequency analysis of 30 storms
METHOD 2: use synthetic storm hydrograph

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BROADKILL BEACH
EROSION AND FLOODING PROTECTION

APPENDIX F
STORM INDUCED EROSION BEACH PROFILES

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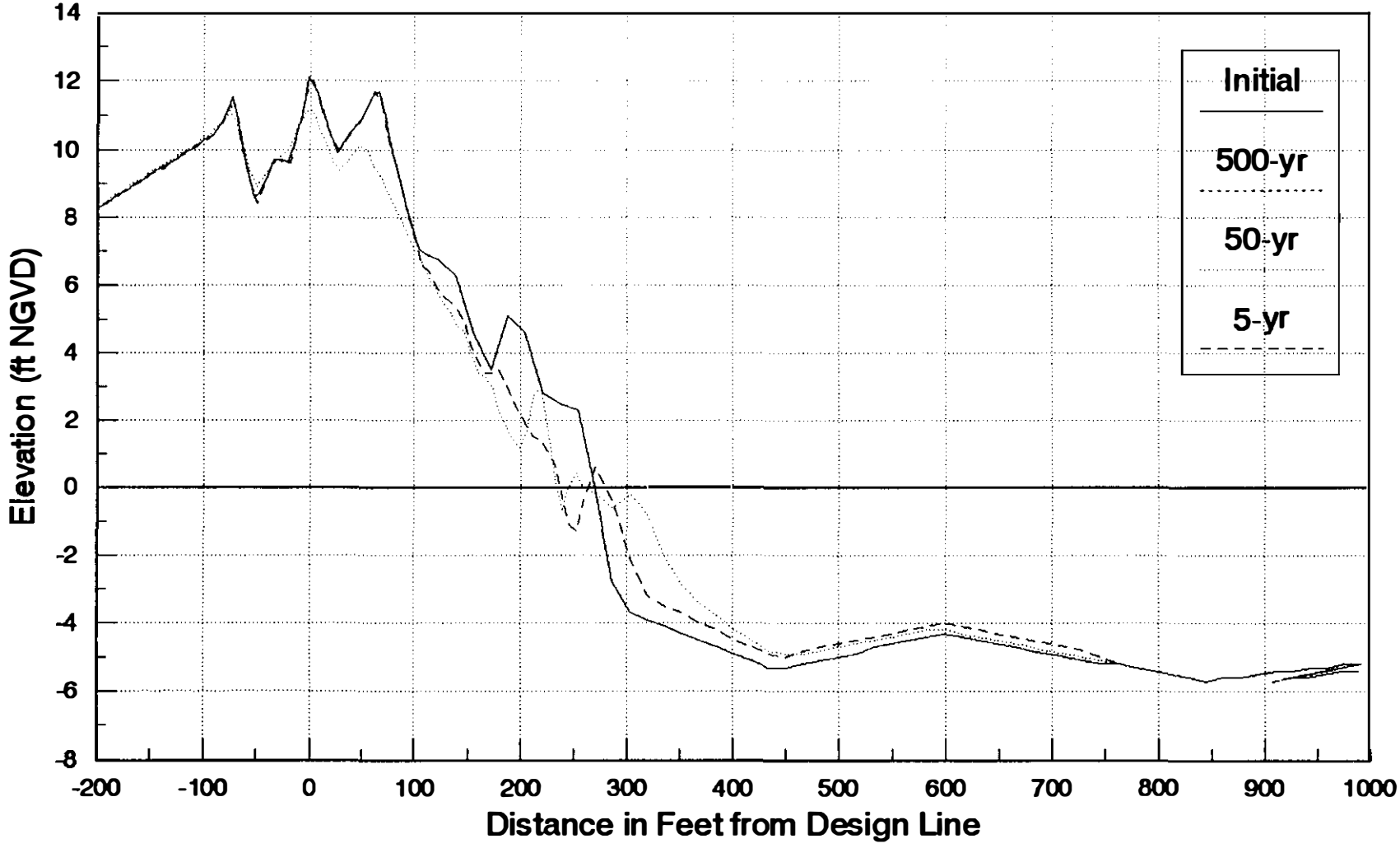
BEACH PROFILES, LRP#24A
BROADKILL BEACH, DELAWARE



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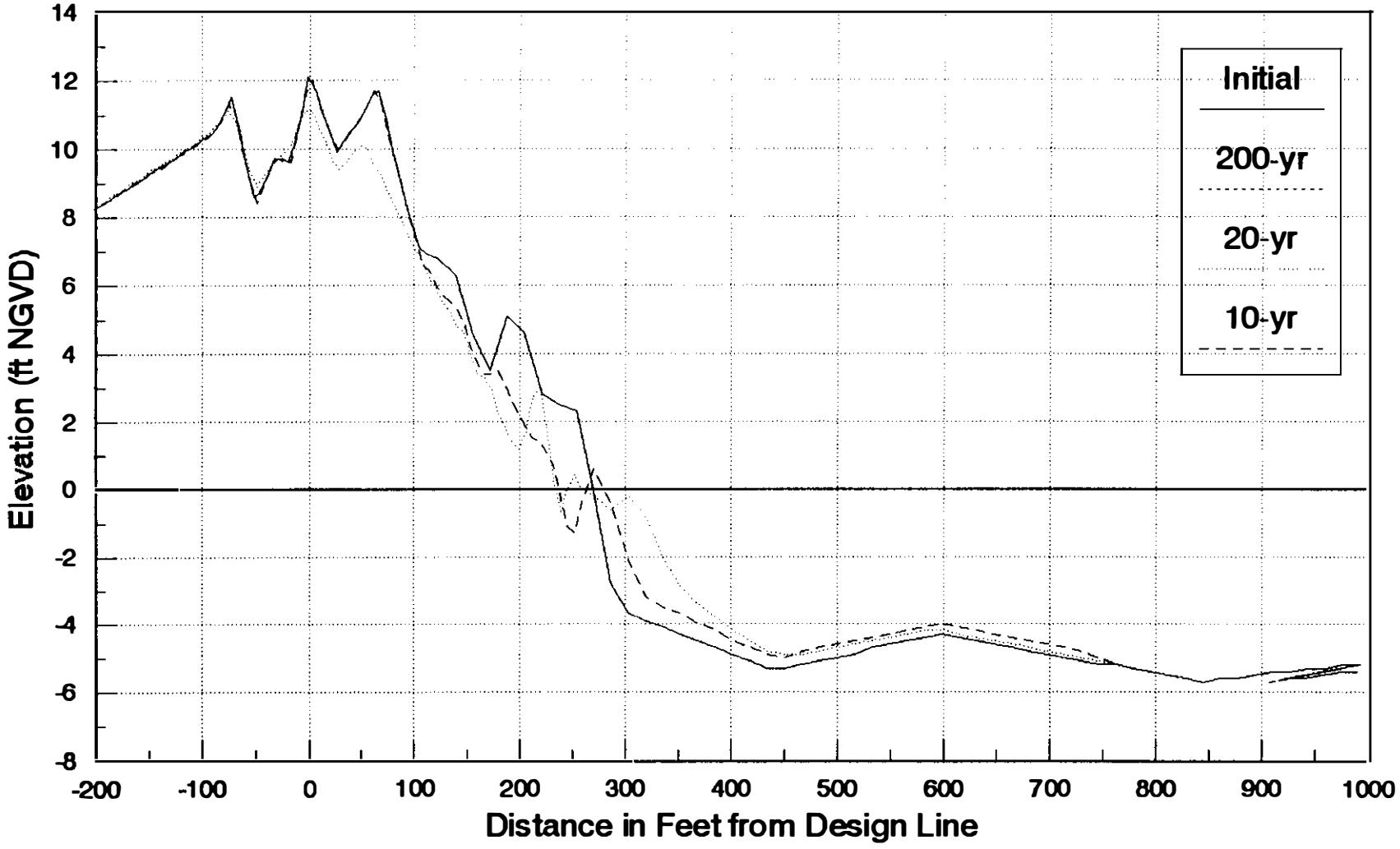
BEACH PROFILES, LRP#24A
BROADKILL BEACH, DELAWARE



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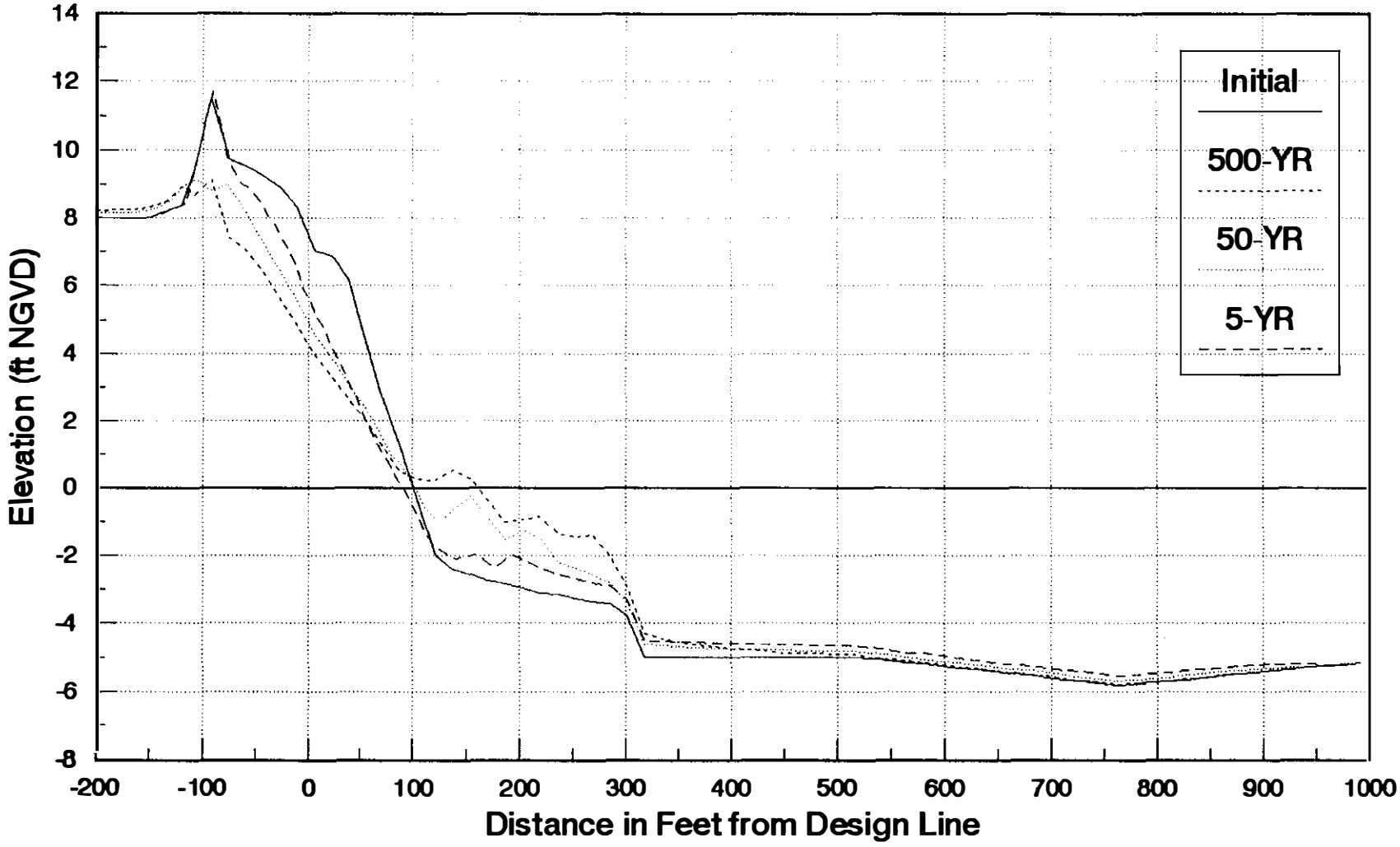
BEACH PROFILES, N33+00
BROADKILL BEACH, DELAWARE



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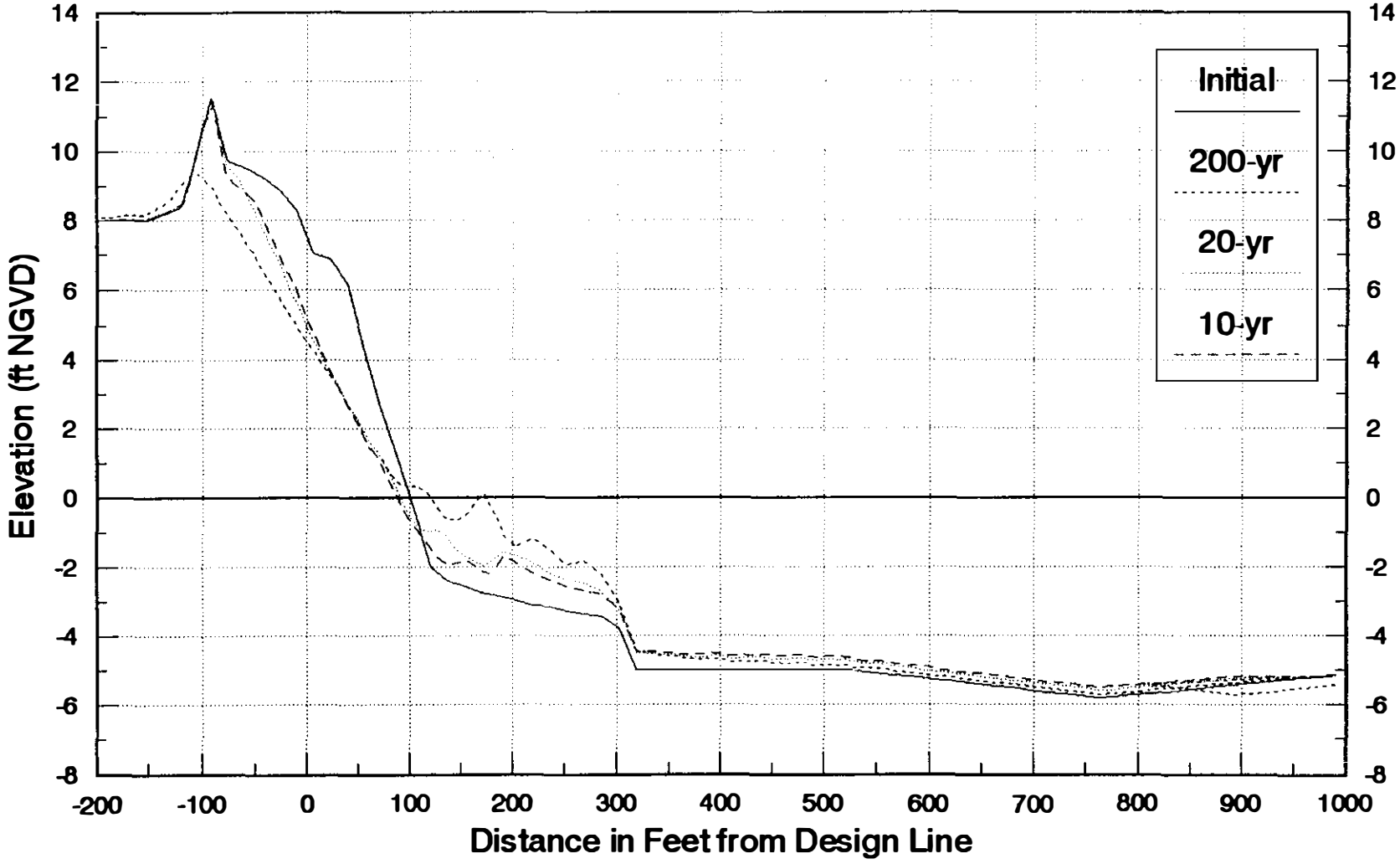
BEACH PROFILES, N33+00
BROADKILL BEACH, DELAWARE



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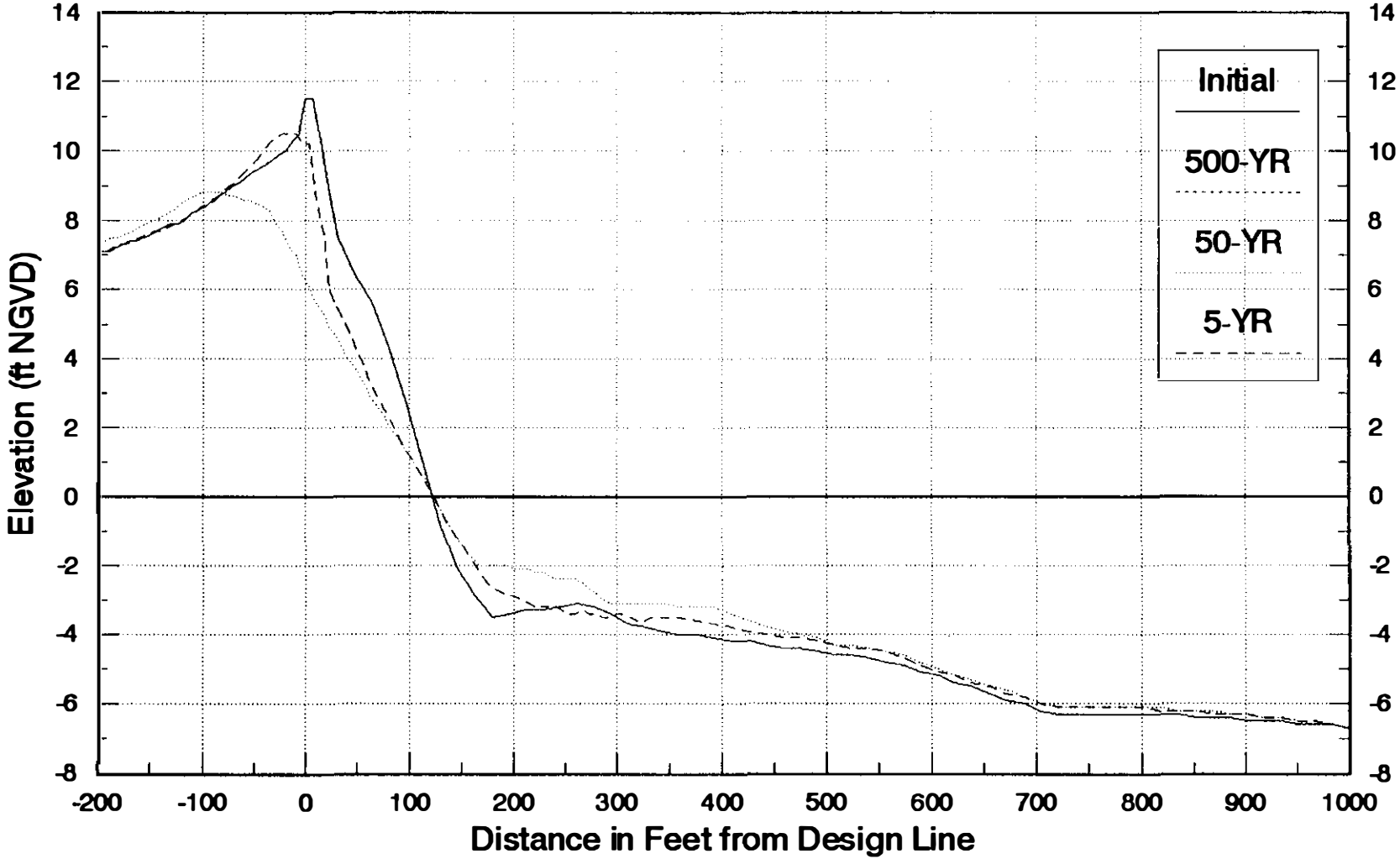
BEACH PROFILES, LRP#25
BROADKILL BEACH, DELAWARE



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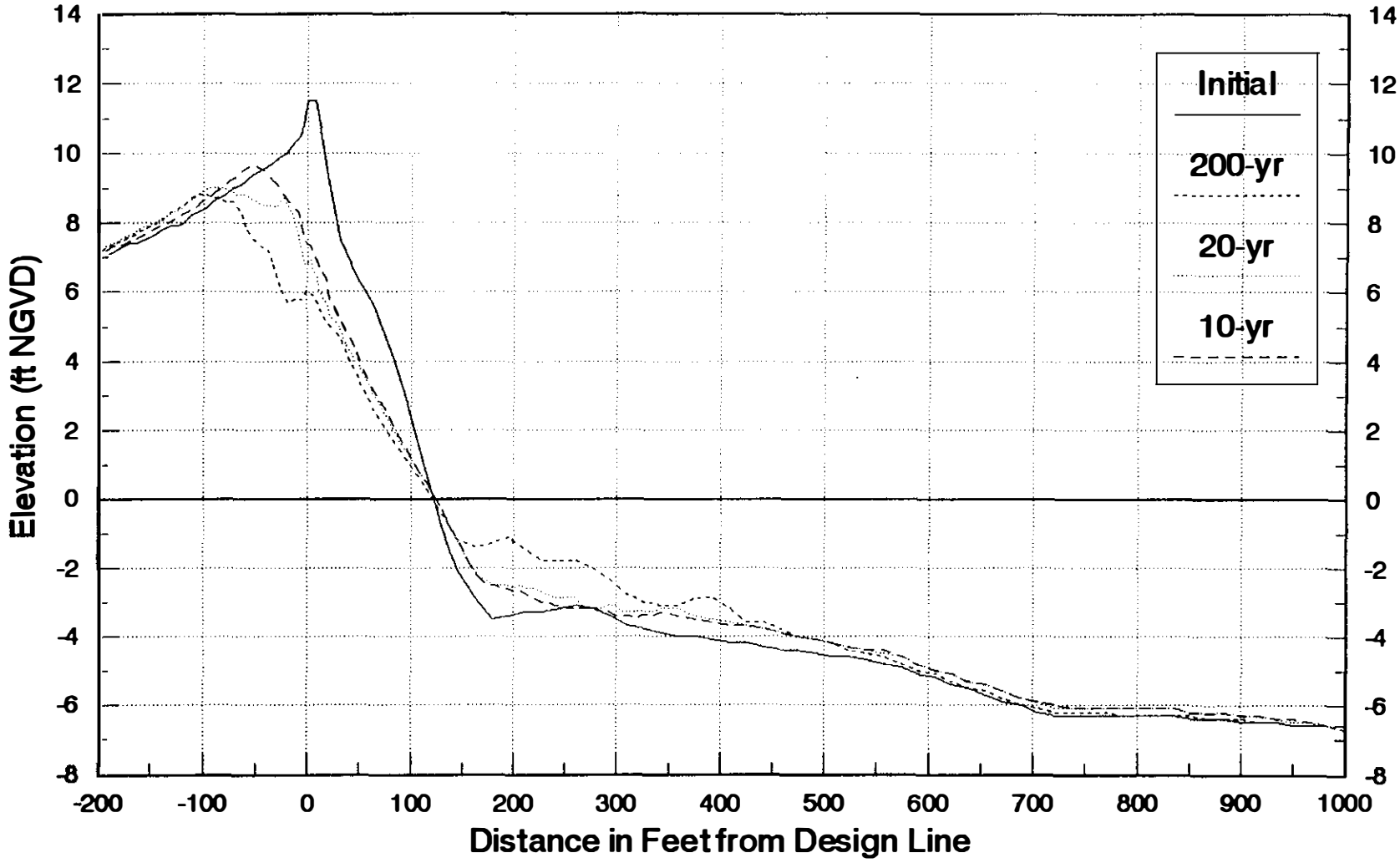
BEACH PROFILES, LRP#25
BROADKILL BEACH, DELAWARE



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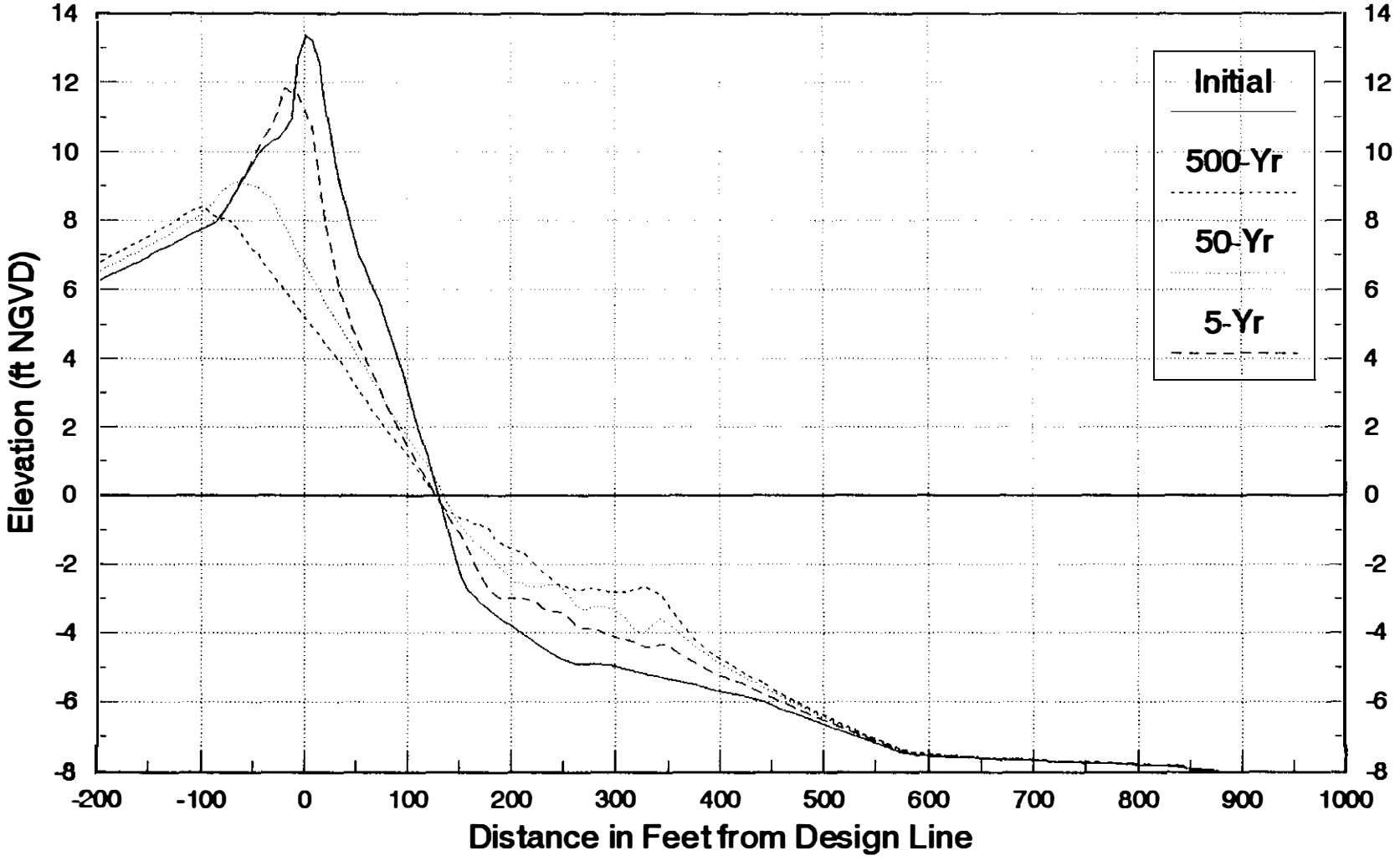
BEACH PROFILES, LRP#25A
BROADKILL BEACH, DELAWARE



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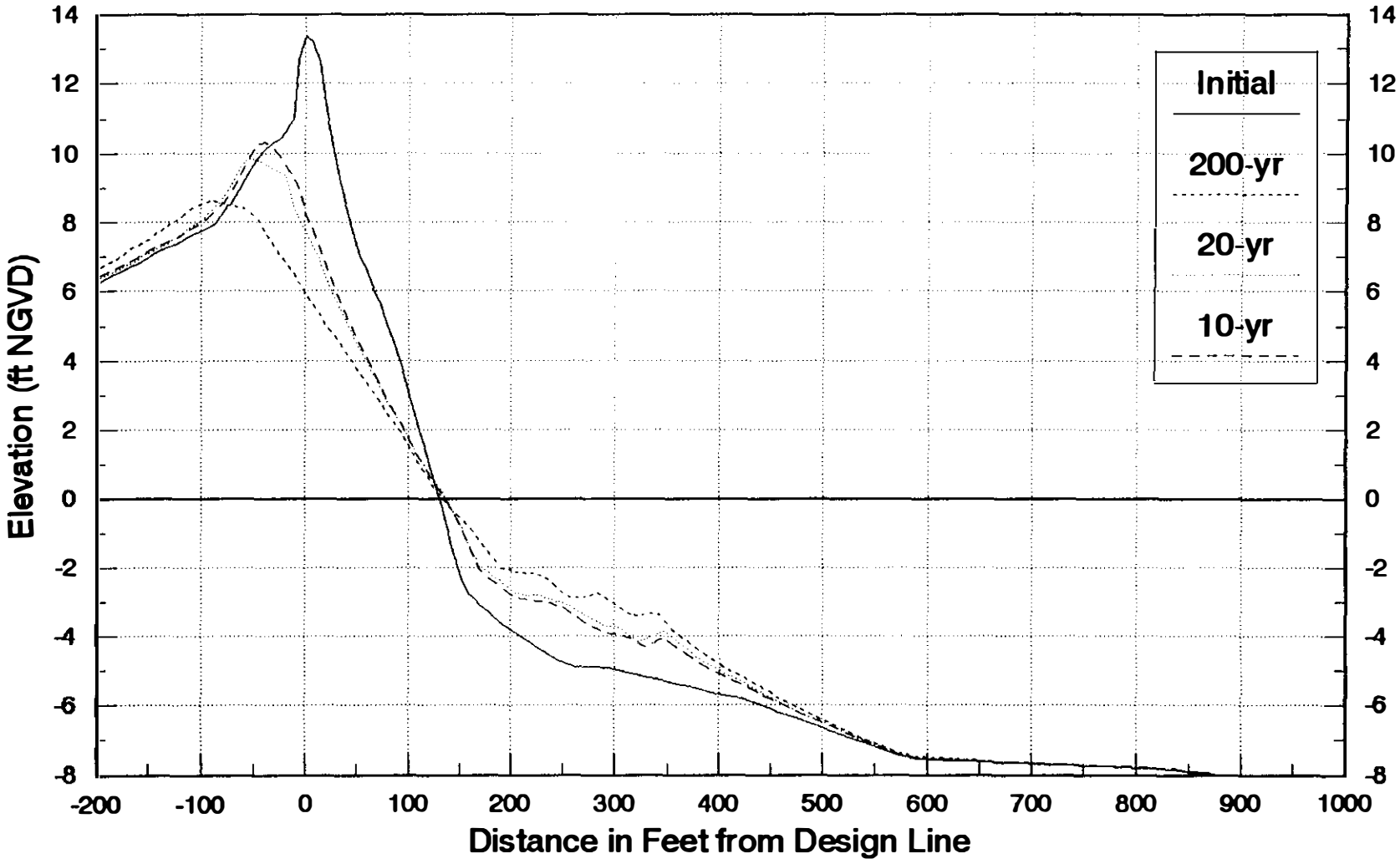
BEACH PROFILES, LRP#25A
BROADKILL BEACH, DELAWARE



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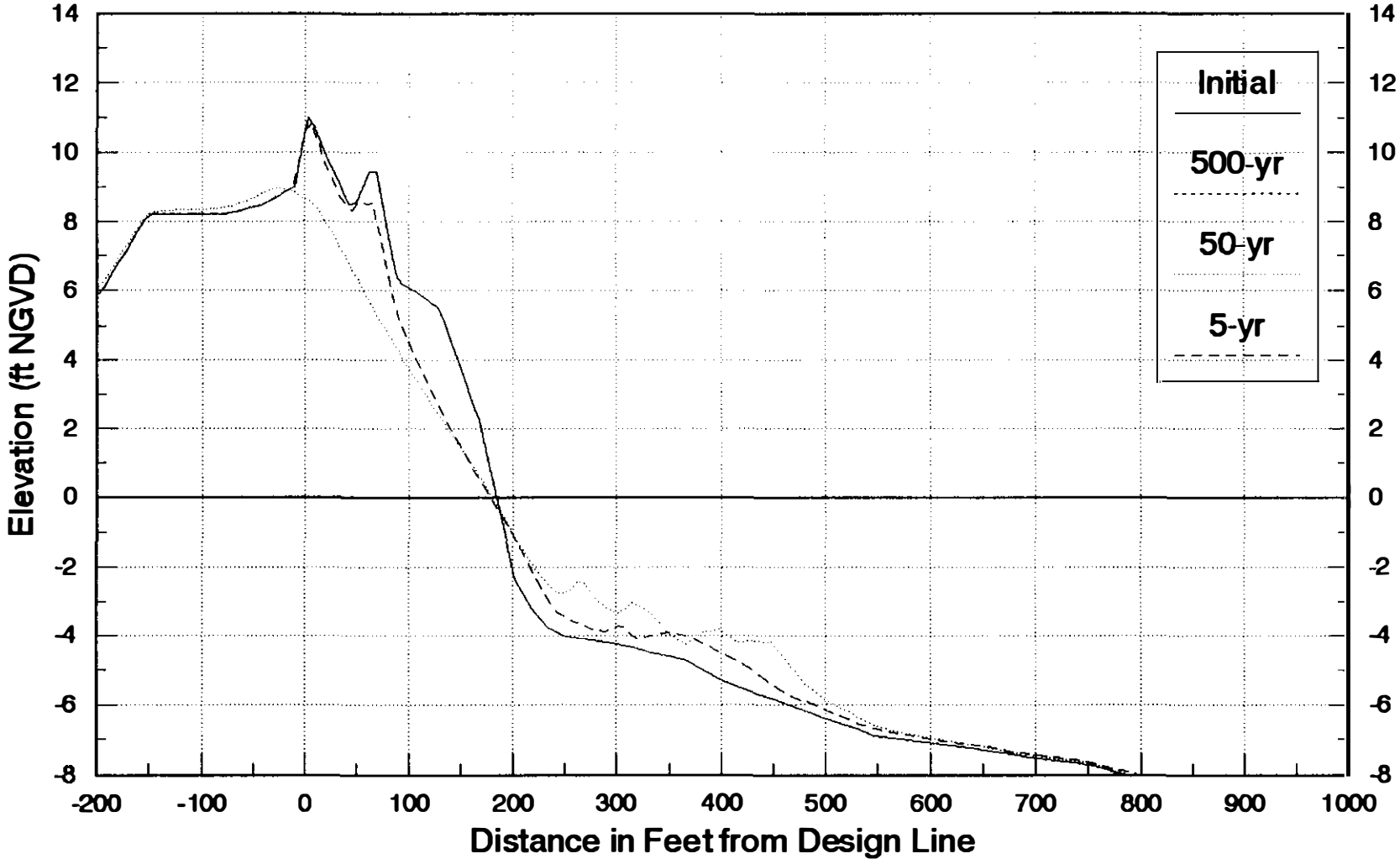
BEACH PROFILES, LPR#25B
BROADKILL BEACH, DELAWARE



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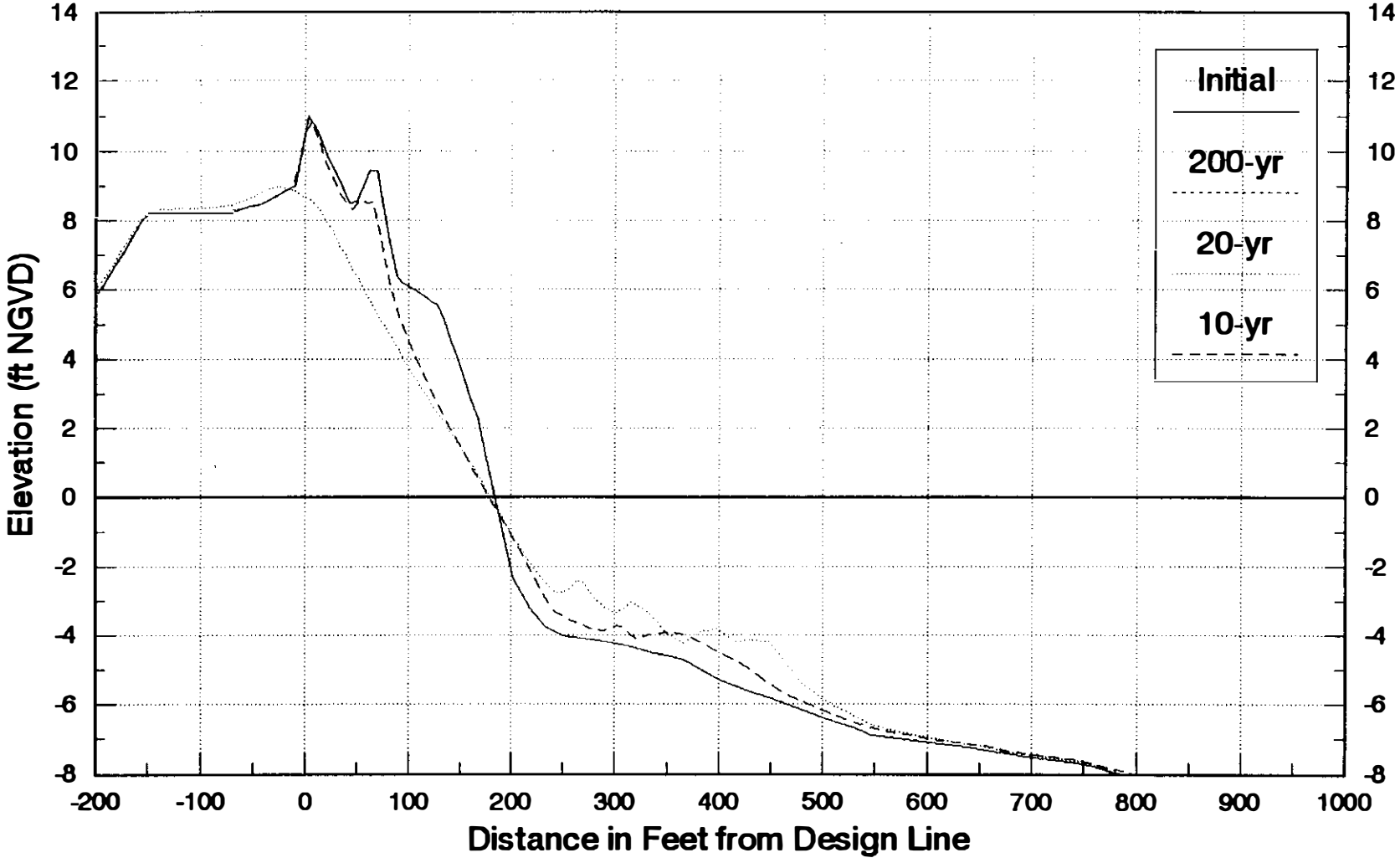
BEACH PROFILES, LRP#25B
BROADKILL BEACH, DELAWARE



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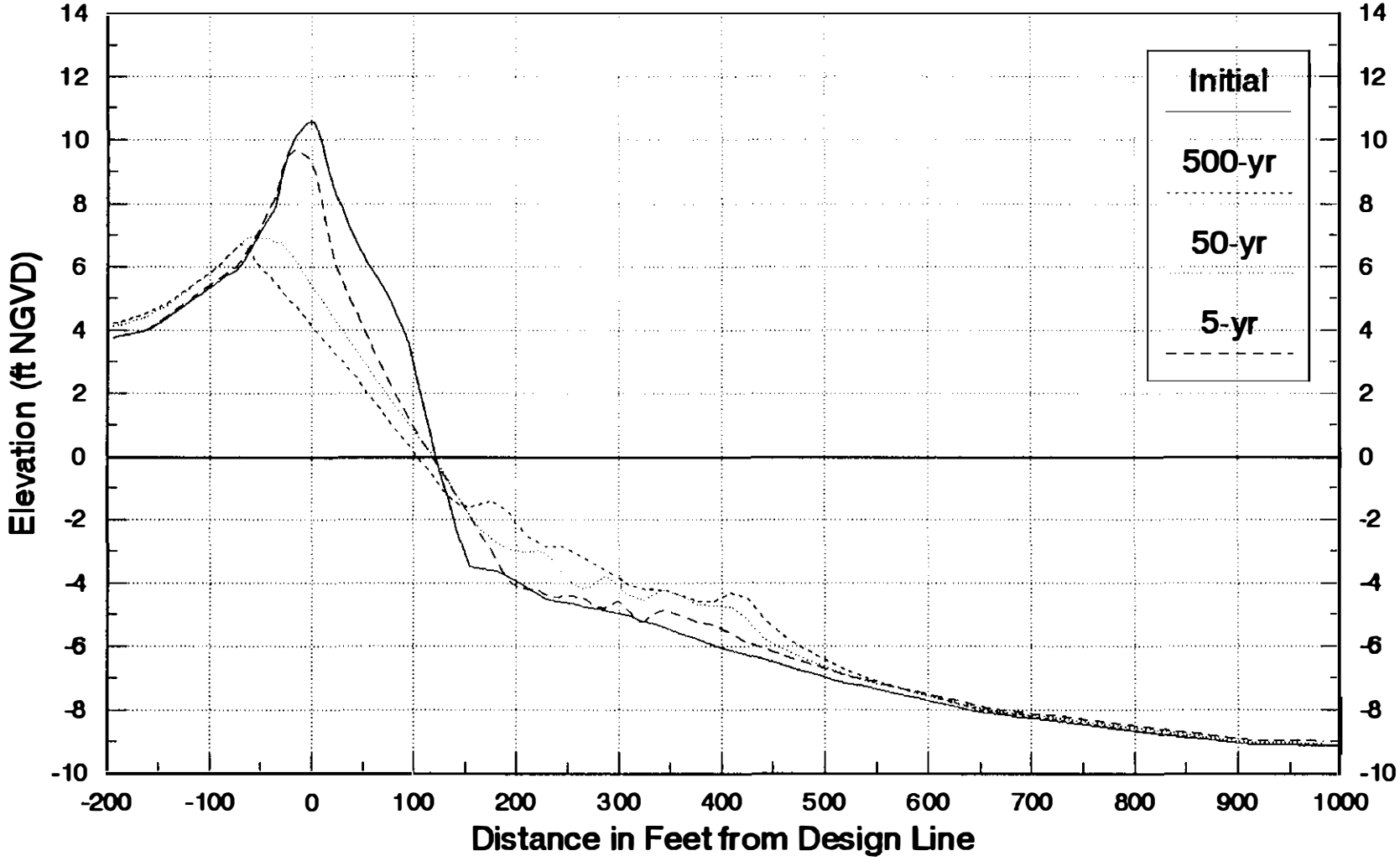
BEACH PROFILES, LPR#26
BROADKILL BEACH, DELAWARE



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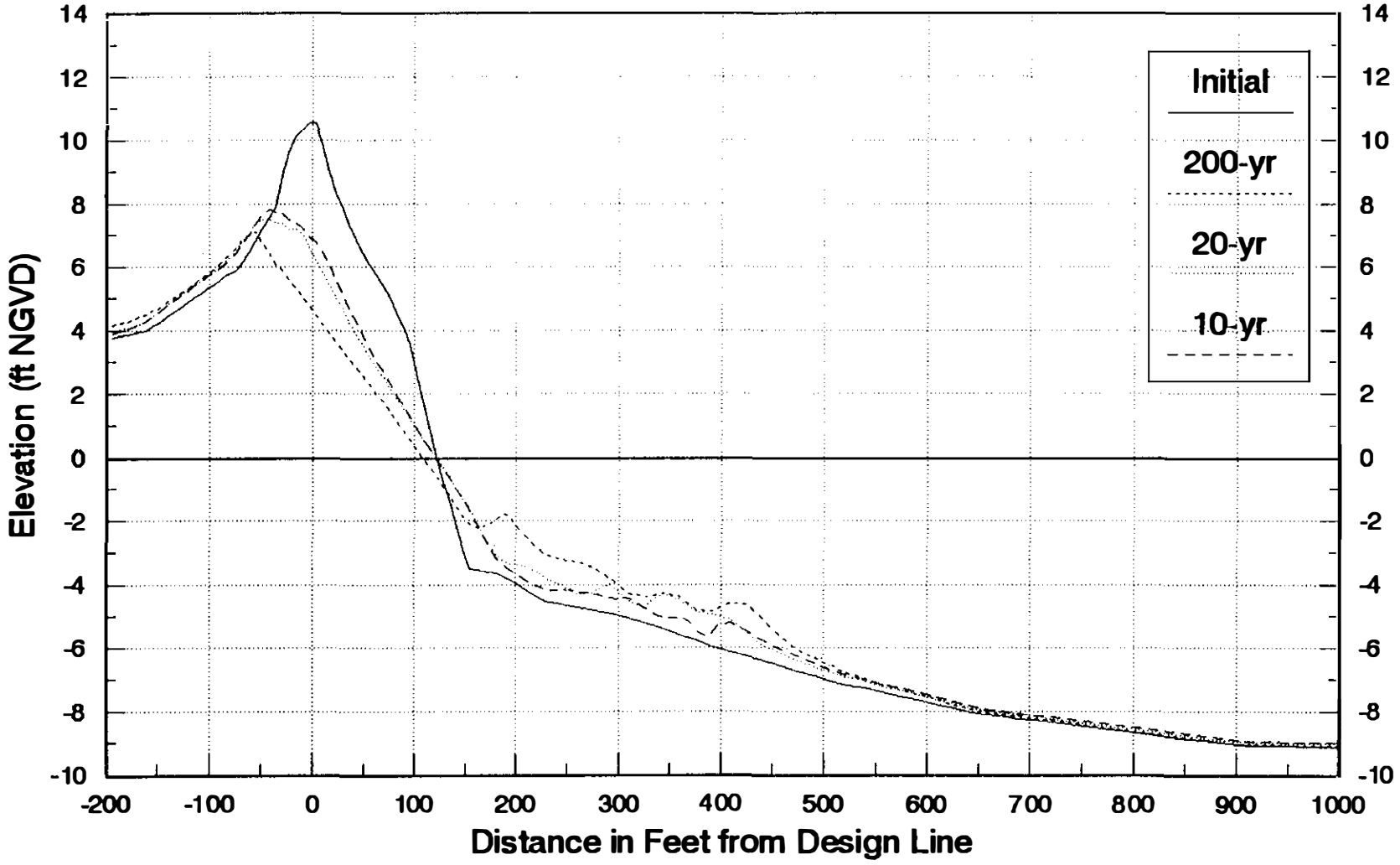
BEACH PROFILES, LPR#26
BROADKILL BEACH, DELAWARE



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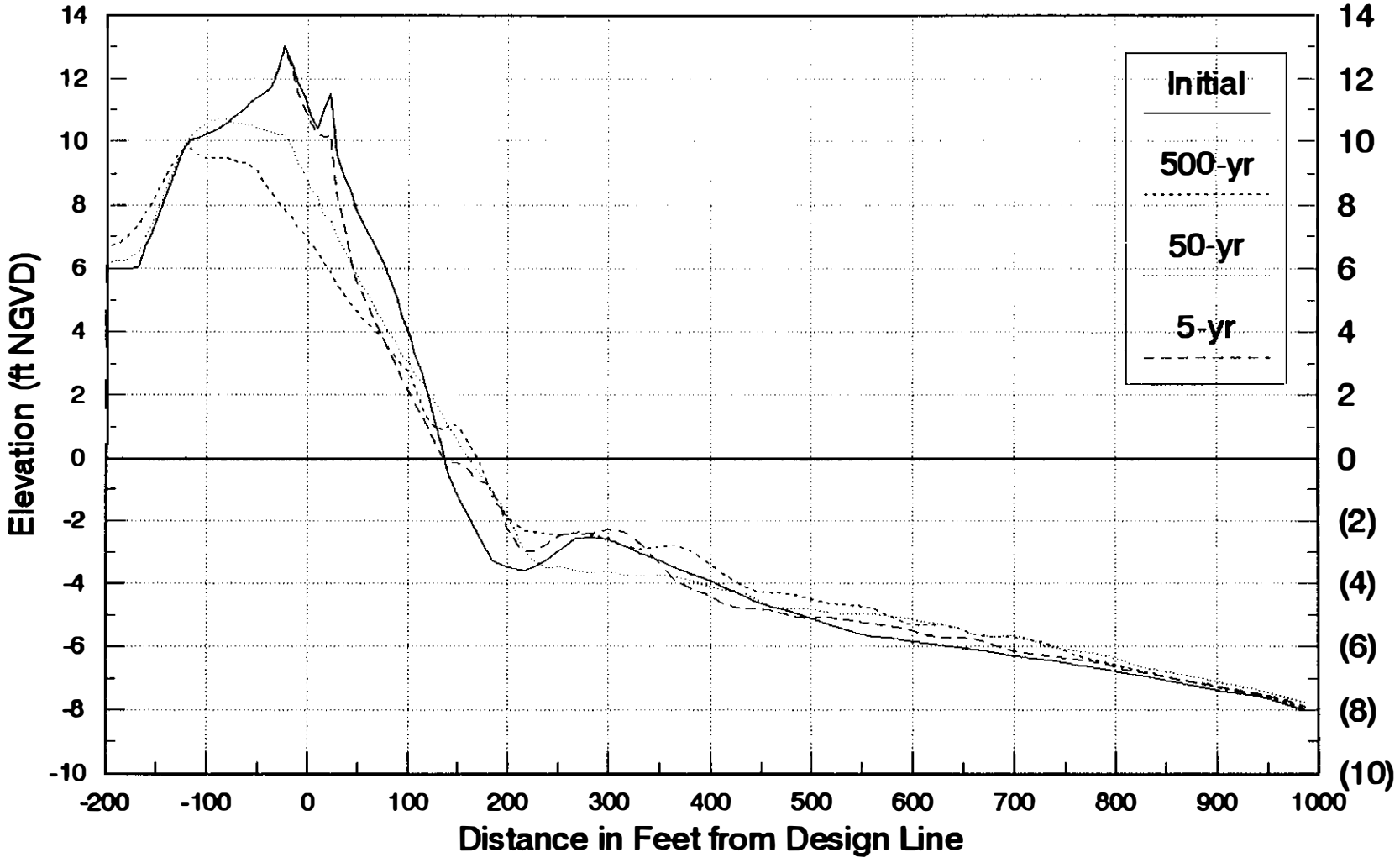
BEACH PROFILES, LRP#27
BROADKILL BEACH, DELAWARE



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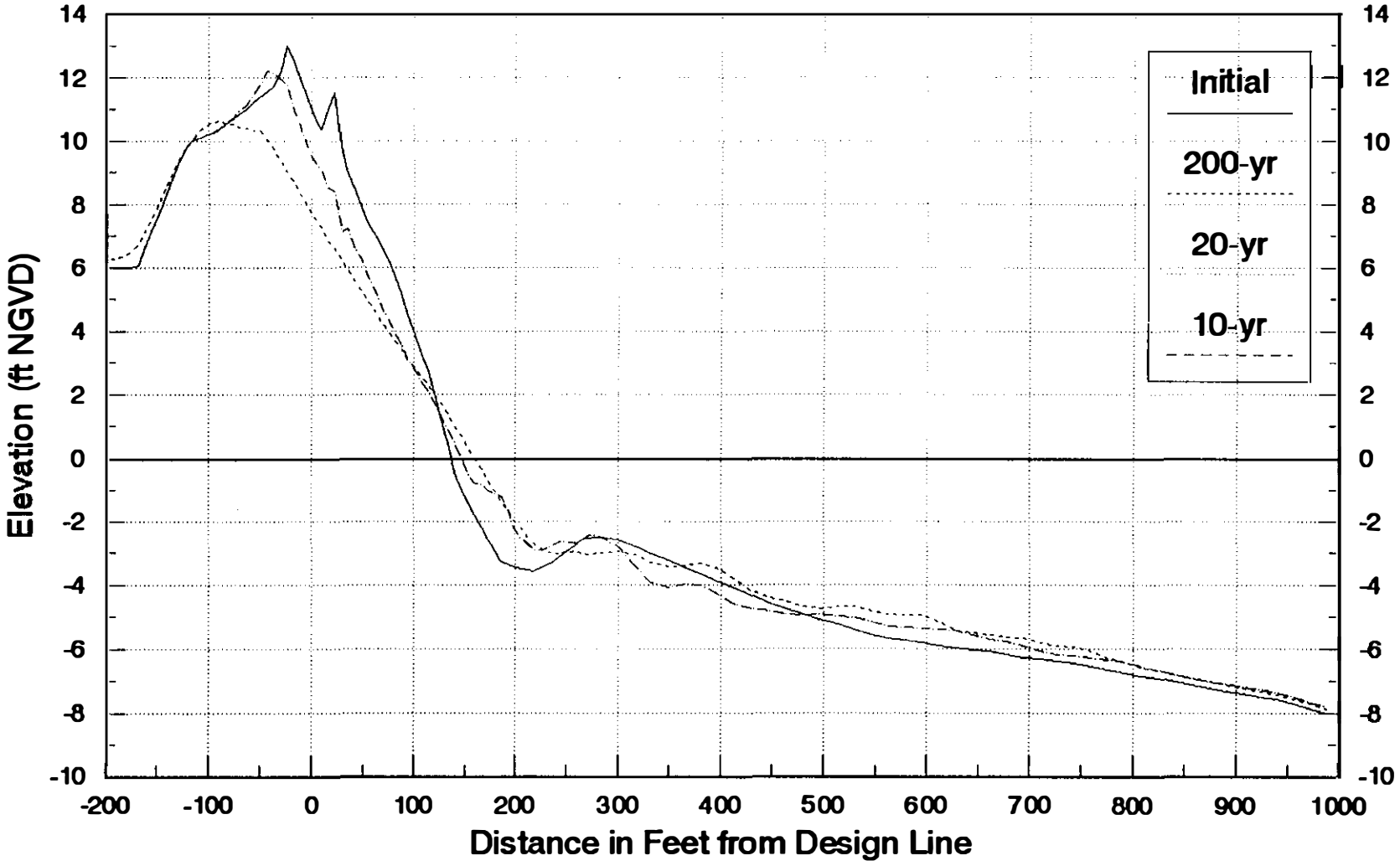
BEACH PROFILES, LRP#27
BROADKILL BEACH, DELAWARE



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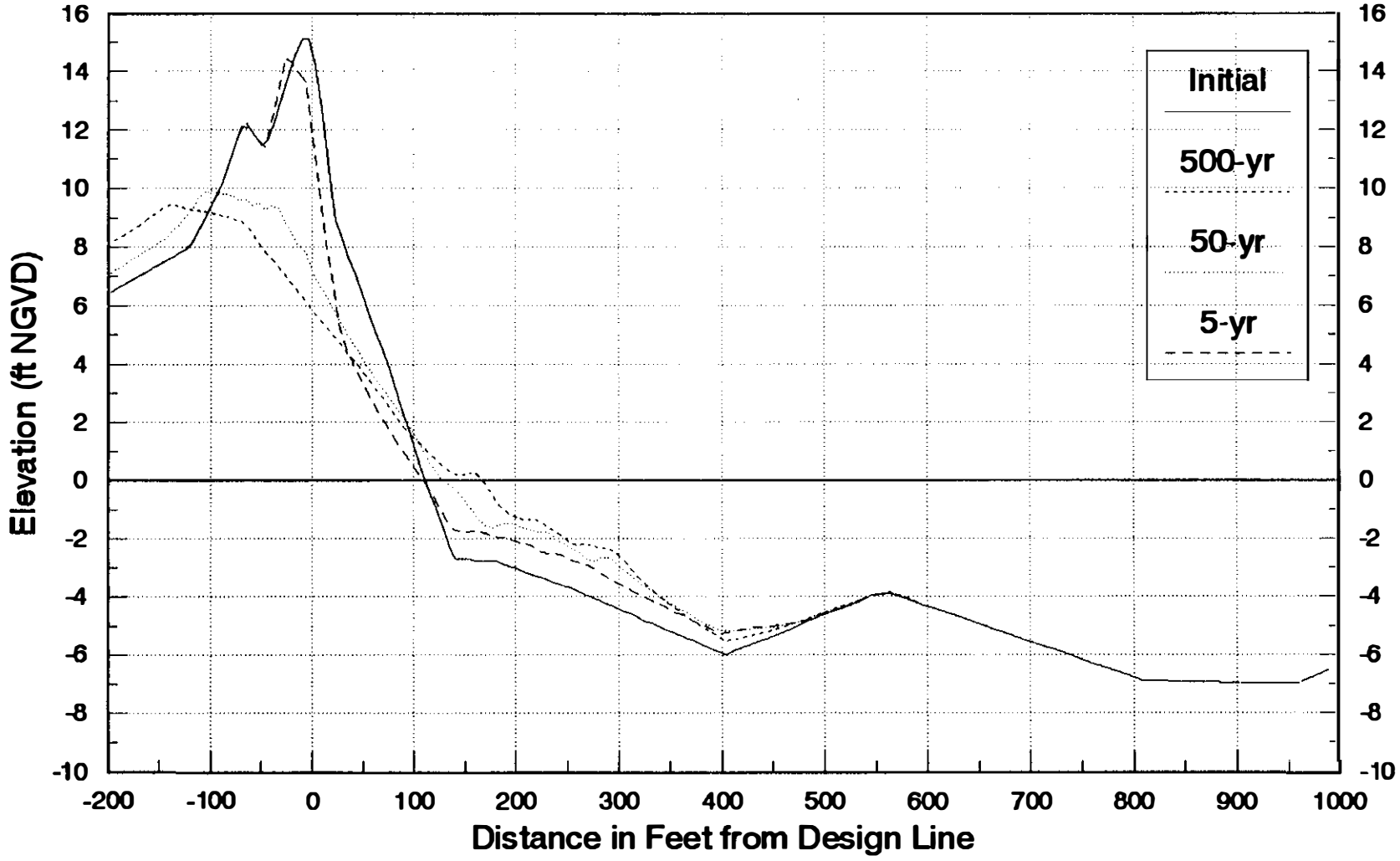
BEACH PROFILES, LRP#27A
BROADKILL BEACH, DELAWARE



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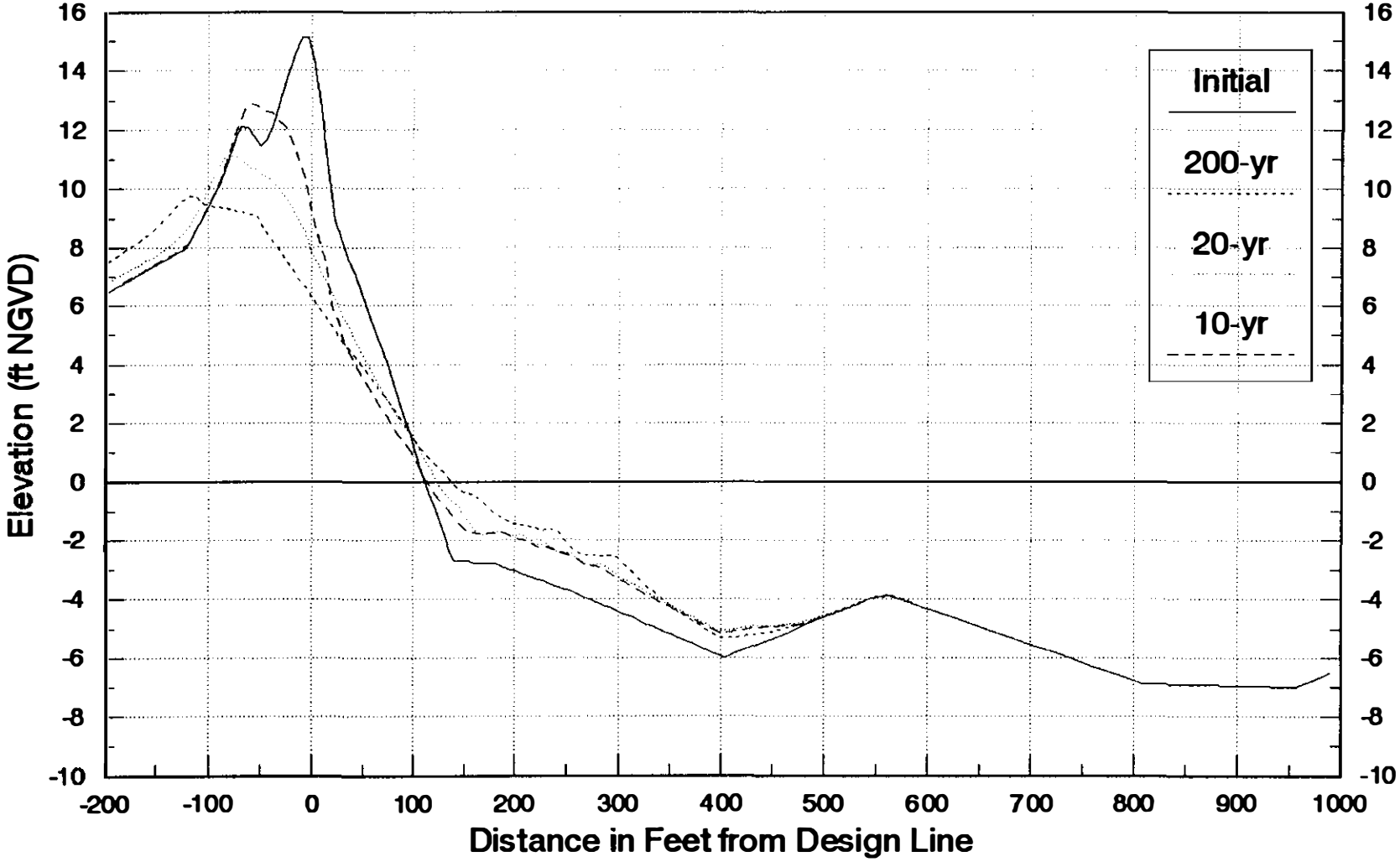
BEACH PROFILES, LPR#27A
BROADKILL BEACH, DELAWARE



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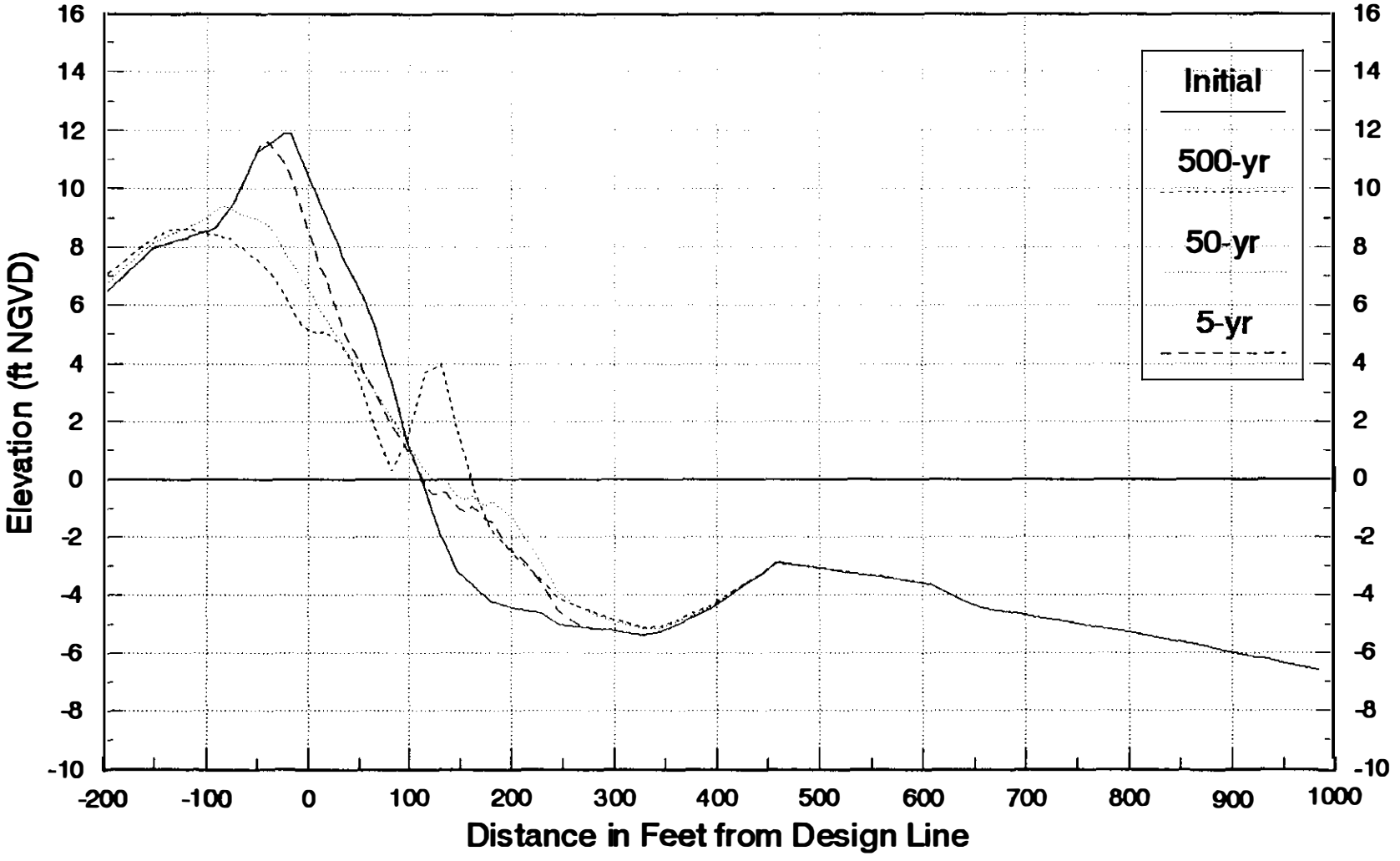
BEACH PROFILES, LRP#28
BROADKILL BEACH, DELAWARE



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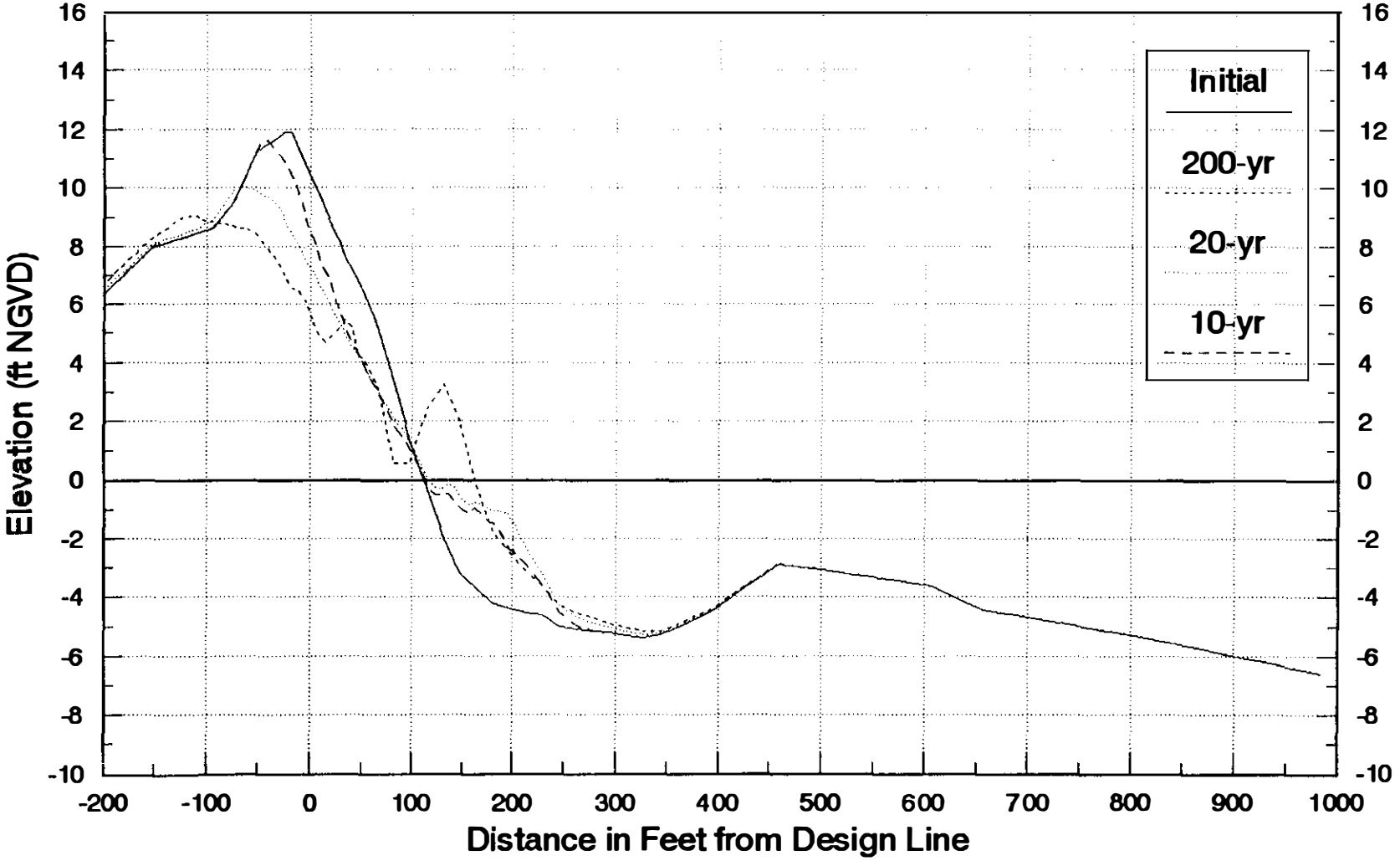
BEACH PROFILES, LRP#28
BROADKILL BEACH, DELAWARE



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PHASE I REPORT

BROADKILL BEACH
EROSION AND FLOODING PROTECTION

APPENDIX G
SHORELINE MODELING - GENESIS

APPENDIX G
SHORELINE MODELING - GENESIS

The numerical model GENESIS (Generalized Model for Simulating Shoreline Change) was applied to Broadkill Beach. The model was developed by USACE-CERC (Hanson and Kraus, 1989; Gravens et al., 1991). It is designed to simulate long-term shoreline changes as a function of nearshore wave climate, beach profile, and the presence of coastal structures such as groins, breakwaters, seawalls, and beach fills. Historical shoreline positions are used to start the calculations and determine the accuracy of calculated shoreline positions.

The model must be calibrated and verified before applying it in the evaluation of engineering alternatives. It is realized that the unrealistic net transport rates at the southeast end of Broadkill Beach (cf., Figure 49, Section IV-4.E) will be reflected in the GENESIS results. It is possible that with additional work these problems could be overcome. For this reason, the model input files were prepared and an initial test was conducted. Should CENAP decide on further effort in this direction, a significant foundation has been prepared.

The basis of the GENESIS computations are detailed in Hanson and Kraus (1989). Gravens et al. (1991) provides a detailed description of the input files necessary to run GENESIS and Gravens (1992) describes the programs available from CERC which assist in file preparation and results interpretation.

GENESIS uses a finite set of wave height and direction transformation coefficients applied to a time series of offshore wave data to determine breaking wave conditions alongshore at each time step. The nearshore wave characteristics determined using RCPWAVE are linked to the offshore time series in the same way as was used for the Q_1 estimates. GENESIS transforms the nearshore waves to breaking using linear theory and a breaking criteria. These breaking wave conditions are used at each GENESIS cell to determine the longshore transport rate at each time step. These transport rates drive the shoreline change calculations, with the assumption that the profile shape remains constant.

Figure G-1 shows the GENESIS computation cells at Broadkill Beach. The cells are sub-cells of the RCPWAVE cells. There are 7 GENESIS cells per RCPWAVE cell. The cells are numbered the opposite of the RCPWAVE cell numbering convention.

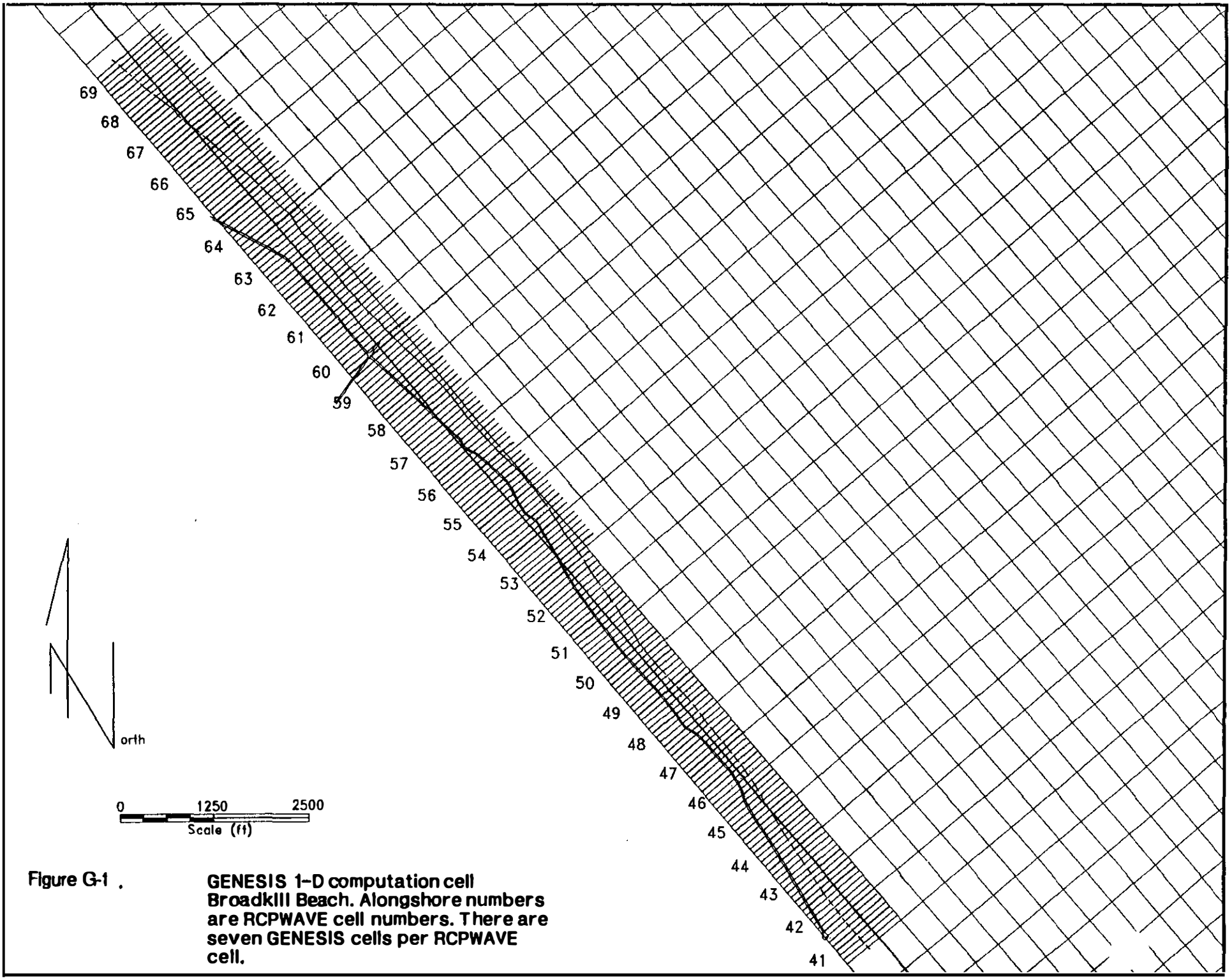

Figure G-1 . GENESIS 1-D computation cell Broadkill Beach. Alongshore numbers are RCPWAVE cell numbers. There are seven GENESIS cells per RCPWAVE cell.

The hindcast wave time series was edited to correspond to the model period. The 1989 and 1993 shoreline positions fall in the wave hindcast period (1987-1993) and were thus used in the test run. A representative profile shape was chosen based on the available profile data. A depth of closure of -5 ft NGVD and an average berm elevation of +8 ft NGVD were chosen as model variables. For boundary conditions, the "pinned-beach" condition was selected for the northwest boundary and a "gated" boundary condition with no transport restriction was chosen at the southeast model boundary. The results are plotted in Figure G-2. The initial calibration results show only marginal agreement with the measured shoreline changes. Based on the initial calibration runs, the model will be refined and furher calibrated in Phase II of this study.

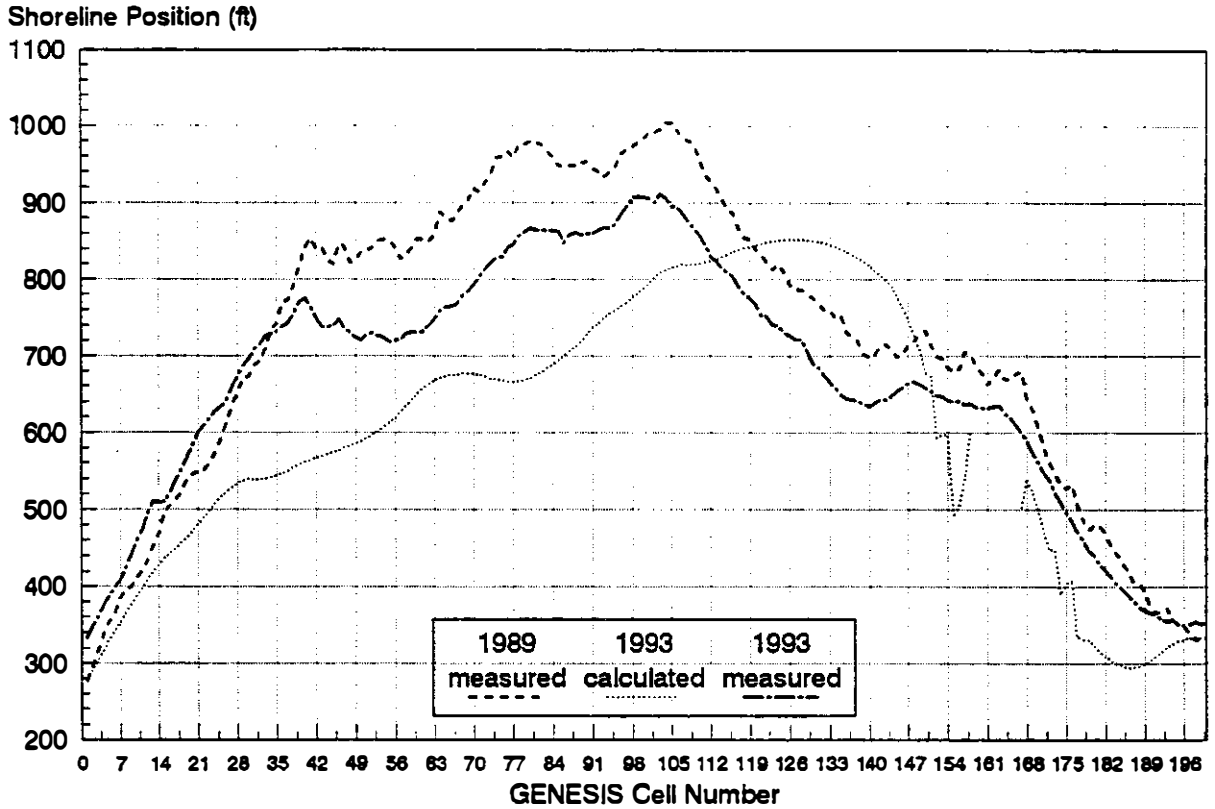

Figure G-2. Test run of GENESIS shoreline change model for period June 1989 to April 1993 using offshore time series and results of RCPWAVE at nearshore reference line. Calculated shoreline position data for cells 159-166 omitted due to extreme fluctuations.

PHASE II REPORT

BROADKILL BEACH
EROSION AND FLOODING PROTECTION

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Philadelphia District

Under Contract No.
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Prepared by:

Water Resources & Coastal Engineering, Inc.
Solon, Ohio

Final Report
November 29, 1995

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BROADKILL BEACH EROSION CONTROL AND FLOODING PROTECTION
PHASE II REPORT

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CONVERSION FACTORS, NON-SI TO SI (METRIC) UNIT OF MEASUREMENT

Non-Si units of measurement used in this report can be converted to Si (metric) units as follows:

Multiply	By	to Obtain
feet	0.3048	meters
cubic feet	0.02832	cubic meters
miles (US nautical)	1.852	kilometers
miles (US statue)	1.6093	kilometers
yards	0.9144	meters
cubic yards	0.7646	cubic meters
cubic yards/foot	2.508	cubic meters/meter
knots (international)	0.5144	meters/second

SECTION I
INTRODUCTION

The Phase II study of shoreline erosion and flood protection for Broadkill Beach, Delaware was prepared by Water Resources & Coastal Engineering, Inc. for the U.S. Army Corps of Engineers, Philadelphia District under contract number DACW61-92-D-0011, delivery order No. 3. Figure 1 presents the study area, locations of survey profiles and the project design line. The without project conditions at Broadkill Beach were established in the Phase I study (WRCE, 1995). The scope of the Phase II study is to evaluate project alternatives.

The first step was initial screening of alternatives that warranted detailed investigation. The second step involved modeling the hydraulic performance of the selected alternatives for comparison with the without project conditions. The computer program SBEACH was used to simulate storm-induced erosion. After the preferred project was selected, the computer program GENESIS was used to determine project maintenance requirements.

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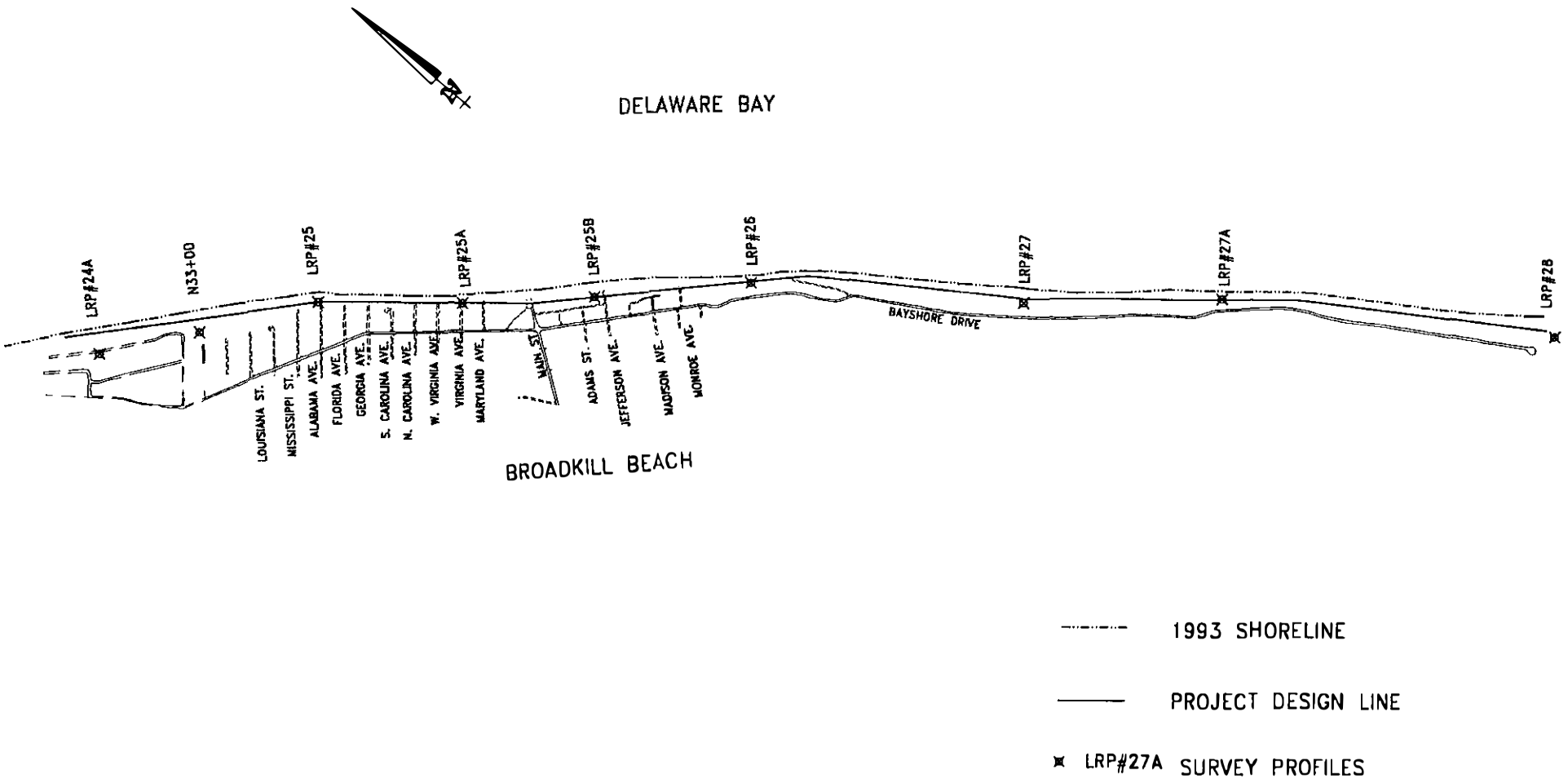

FIGURE 1. BROADKILL BEACH, STUDY AREA

SCALE 1"=1500'

SECTION II
INITIAL ALTERNATIVE SCREENING

The major considerations for selecting beach stabilization alternatives are: the objectives of the project, the physical process prevailing at the project site, and the potential for adverse impacts along adjacent beaches. The objectives for this project are to retard the beach erosion process and provide protection from storm wave attacks on the homes at Broadkill Beach.

On August 16, 1994, a Cycle 1 alternative screening meeting was conducted to discussed suitable alternatives for beach stabilization and flood protection. Project team members from the district and the study contractor attended the meeting. The following alternatives were discussed:

- Do nothing
- Perched beach
- Offshore breakwaters using filled geotextile tubes
- Repair of existing groins using filled geotextile tubes
- Repair of existing groins using quarrystone
- Beach restoration (berm and beach face)
- Beach restoration and dune enhancement

Do Nothing

Longterm background erosion rates were investigated in the Phase I study. The weighted average longterm erosion rate for the Broadkill project area is 10.5 feet of beach retreat per year. Figure 2 shows the estimated shoreline position for the year 2011 assuming there will be no project of any type. The shoreline position was estimated by projecting the longterm rate from the 1993 shoreline. The computer program GENESIS was also used to estimate future shorelines. Although the model predictions begin to deviate from the longterm rate projection at the southeast end, it is clear that without projects, many homes and roads in the Broadkill Beach area will be destroyed. Discussion of the use of the GENESIS model is presented in Appendix A.

Perched Beach

Based on the experience gained at Slaughter Beach during the Section 54 demonstration project, the perched beach alternative was rejected from further consideration. Also contributing to the decision was a study by Drexel University (Dalrymple, 1982), concluding that the effectiveness of the device is negligible.

Low Offshore Breakwater

The low offshore breakwater was used at Kitts Hummock under the Section 54 program. The demonstration project included using three different materials: rubble-mound stone, nylon sandbags and precast concrete boxes. The nylon sandbags deteriorated and were non-functional. The remaining sections were not very effective in reducing shoreline retreat. The result was placement of 15,780 cubic yards of sand along 1000 feet of beach at Kitts Hummock in 1988 by the state of Delaware. Low breakwaters will not provide the project area with protection from storm wave attack. Appendix C provides a

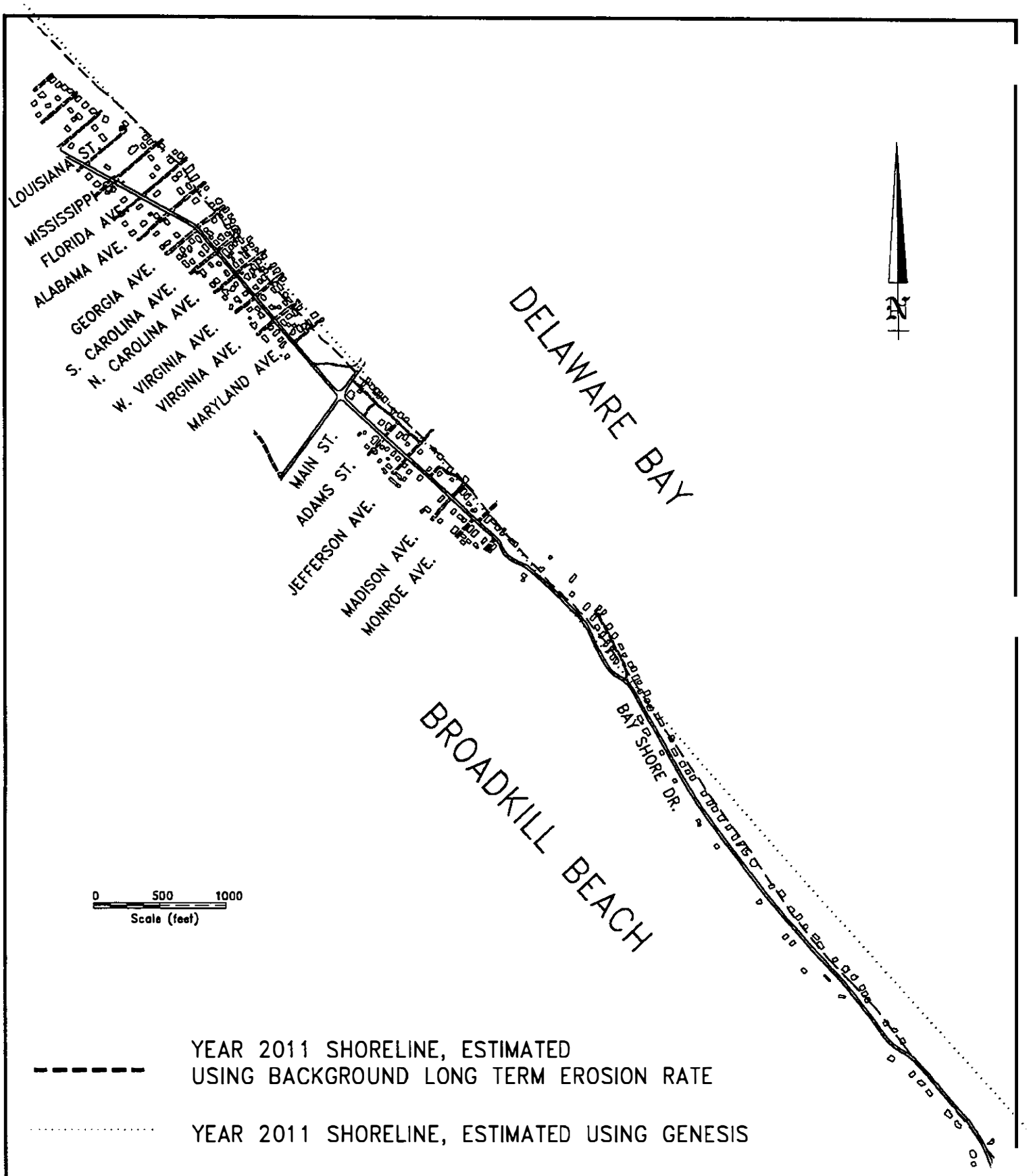

FIGURE 2. ESTIMATED SHORELINE POSITION, WITHOUT PROJECT BROADKILL BEACH, YEAR 2011

conceptual design for using offshore breakwaters at Broadkill Beach. The cost-benefit ratio for this alternative does not warrant the use of offshore breakwaters.

Groin Repair

Groin fields have been used successfully to reduce shoreline erosion at many coastal locations. A major disadvantage of a groin field is that longshore sand transport can be significantly reduced, causing erosion at downdrift beaches. In order to consider the groin alternative, effects on shoreline stability and erosional characteristics should be investigated along with the effects to the adjacent shoreline. Appendix B details an investigation of using a groin field without beach nourishment. The groin field alternative can retard shoreline erosion but does not provide protection from wave attack. Cost analysis indicated that a groin field does not warrant further investigation.

In addition, beach stabilization structures alone do not provide the sand to maintain a beach. They simply redistribute available sand. Therefore, accretion in one area is balanced by erosion elsewhere unless additional sand is introduced. Table 1 summarizes the advantages and disadvantages of beach stabilization structures considered in this study.

Beach Restoration and Dune Enhancement

Beach and dune restorations can be designed to provide protection from storm waves but are vulnerable to coastal storms. Periodic renourishment may be necessary to maintain a given level of protection. Building coastal structures in conjunction with beach nourishment may be justified if the savings realized by increasing the time between required renourishment exceeds the cost of the structures. The advantages and disadvantages of the beach restoration alternative are listed in Table 2.

Based on the initial screening by hydraulic performance, the beach restoration alternative without dune enhancement is insufficient in protecting homes from wave attack. The initial cost screening indicated that beach restoration with dune enhancement and periodic nourishment is the preferred alternative. This alternative provides the most effective protection from storm wave attacks. The beach restoration/dune enhancement alternative with groin fields will also be investigated to determine whether the groin field can increase the renourishment cycle.

Table 3 lists the beach restoration/dune enhancement plans selected by the project team for performance/cost optimization. The top width of the dune is 25 feet and the dune slope is 1V:5H. The crest of the berm is measured from the project design line. The elevation of the berm crest is at +8 feet NGVD. The slope of the beach face is 1V:15H. Figures 3-7 present the profile for each plan. Profile LRP#25A is also shown in these plots for comparison. Nine beach profiles surveyed in 1993 were selected to represent the project area. The profiles used for project evaluation are LRP#24A, N33+00, LRP#25, #25A #25B, #26, #27, #27A and #28 as shown in Figure 1. Table 4 summarizes the unit fill quantities for each profile.

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TABLE 1. ADVANTAGES AND DISADVANTAGES OF BEACH STABILIZATION STRUCTURES

GROIN FIELD	
ADVANTAGES	DISADVANTAGES
• Effective against erosion caused by longshore transport • Can be built using shore-based equipment • Can be constructed using various types of material to allow sand bypassing	• Not effective in preventing offshore sand losses • Can cause rip currents and increase offshore sand loss • May starve downdrift beaches
OFFSHORE BREAKWATER	
• Effective against erosion caused by both longshore and offshore sand losses • Submerged design is often more aesthetically acceptable • Can be built using inexpensive materials • Can allow sand bypassing	• May be expensive to construct • Can significantly alter the character of the surf zone • Navigation hazard • Tombolos may form and could seriously interrupt longshore transport and cause downdrift erosion
PERCHED BEACHES	
• Aesthetically acceptable • Reduce the level of wave action • Slow/retard offshore sand losses	• The low sill may not be high enough to retard offshore losses • The submerged sill may prevent beach recovery during beach-building wave conditions • Navigation hazard

TABLE 2. ADVANTAGES AND DISADVANTAGES OF BEACH RESTORATION

ADVANTAGES	DISADVANTAGES
• Introduce additional sand to littoral environment • Enhance eroded beaches • Provide storm wave protection	• Can be vulnerable to coastal storms • Must be periodically renourished

TABLE 3. BEACH AND DUNE RESTORATION PLANS BROADKILL BEACH		
PLAN #	DUNE HEIGHT (FT NGVD)	BERM CREST FROM DESIGN LINE (FT)
1	14	100
2	16	100
3	14	150
4	16	150
5	18	150

TABLE 4. FILL QUANTITIES FOR BEACH AND DUNE RESTORATION
BROADKILL BEACH FEASIBILITY STUDY
(IN CY/FT)

PLAN	LRP# 24A	N33 +00	LRP# 25	LRP# 25A	LRP# 25B	LRP# 26	LRP# 27	LRP# 27A	LRP# 28
1	5.2	69.3	58.9	57.5	29.3	68.1	57.0	65.7	72.4
2	11.8	76.5	67.1	64.5	36.4	76.3	64.0	72.8	79.6
3	11.3	94.5	85.5	87.2	59.5	99.6	82.0	93.2	102.5
4	17.8	101.5	93.6	94.2	67.0	107.7	85.0	100.3	109.6
5	26.4	110.0	102.2	102.7	75.0	116.3	89.0	108.8	118.0

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FIGURE 3. BEACH FILL PLAN 1
BROADKILL BEACH, DELAWARE

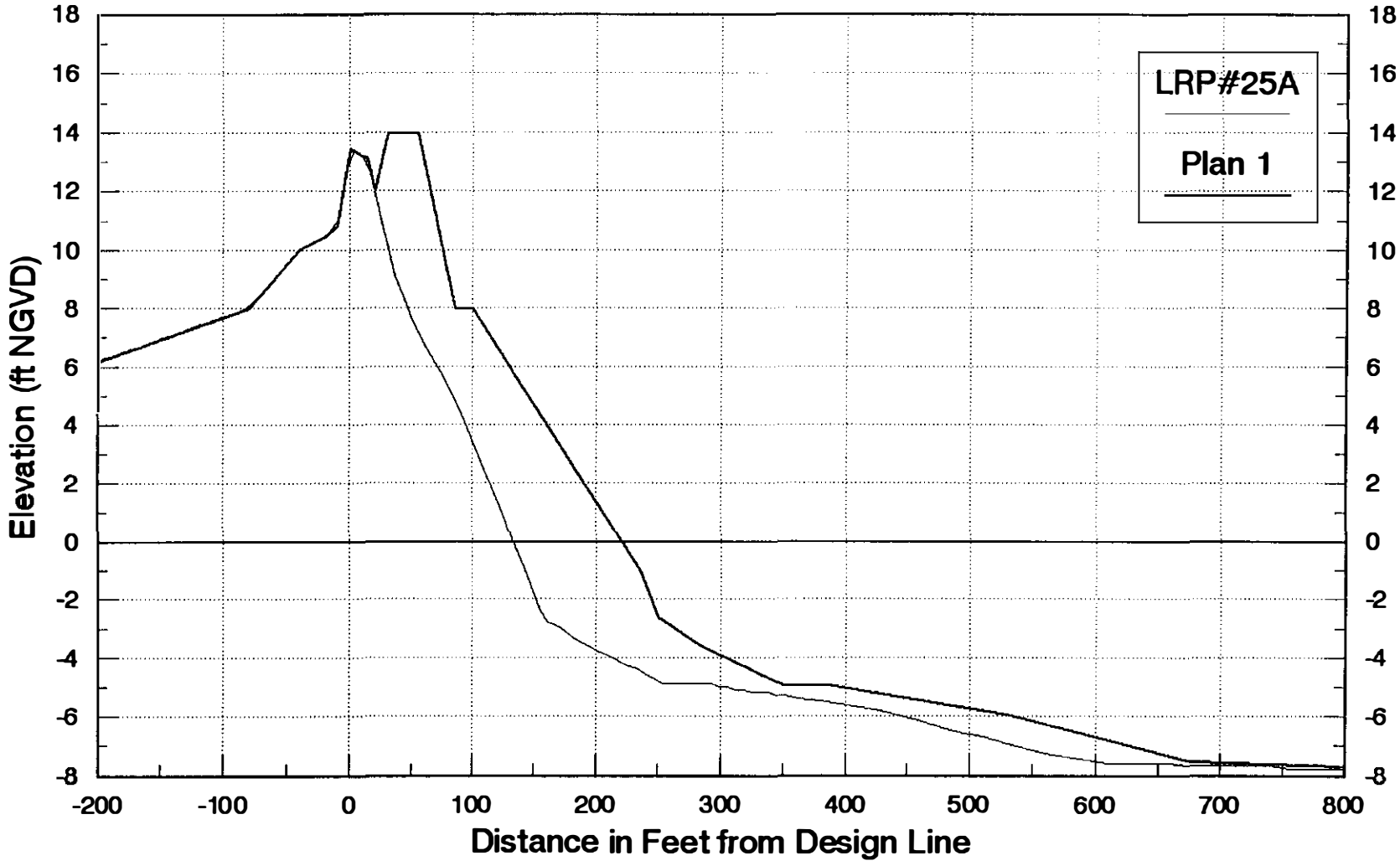

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FIGURE 4. BEACH FILL PLAN 2
BROADKILL BEACH, DELAWARE

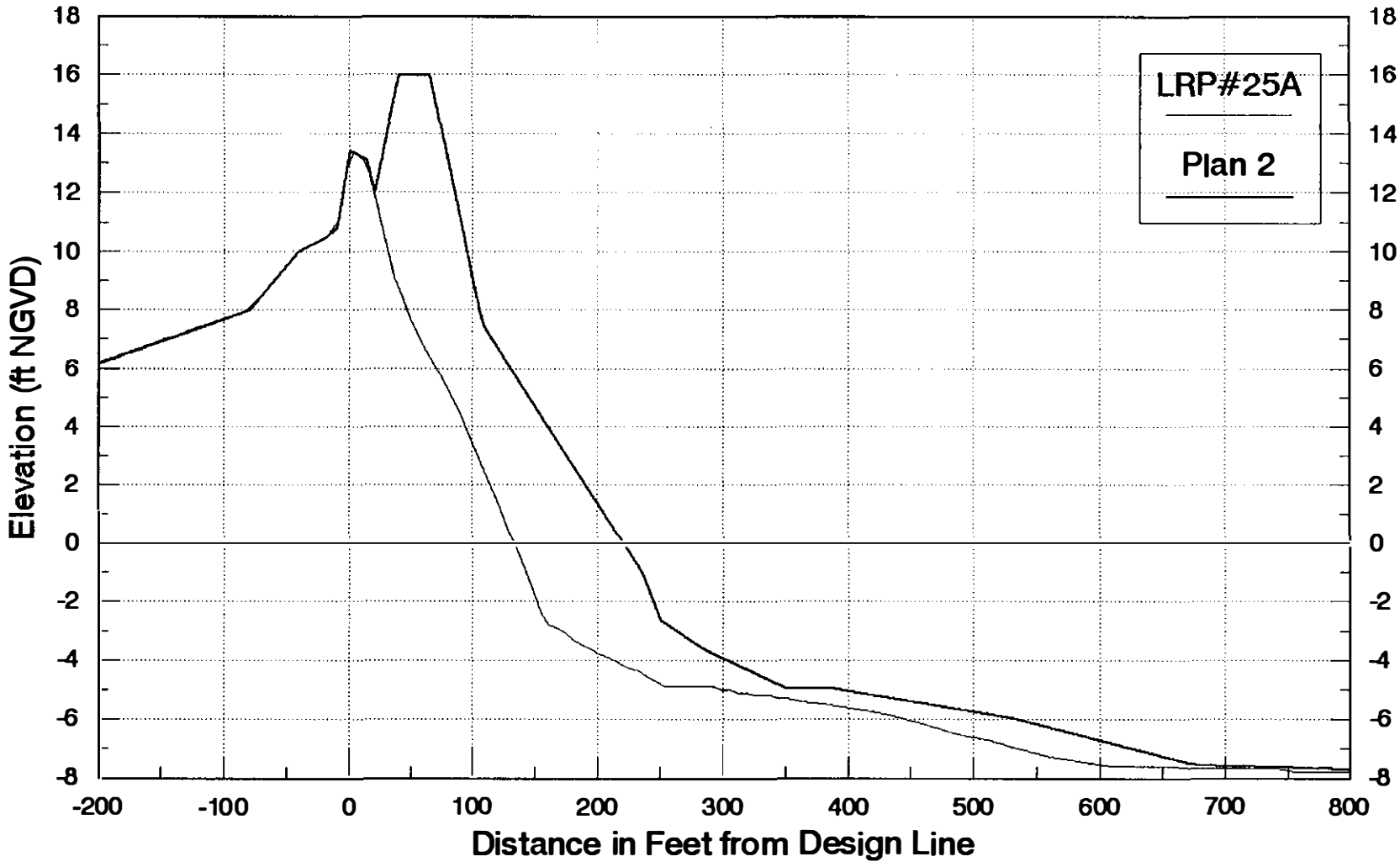

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FIGURE 5. BEACH FILL PLAN 3
BROADKILL BEACH, DELAWARE

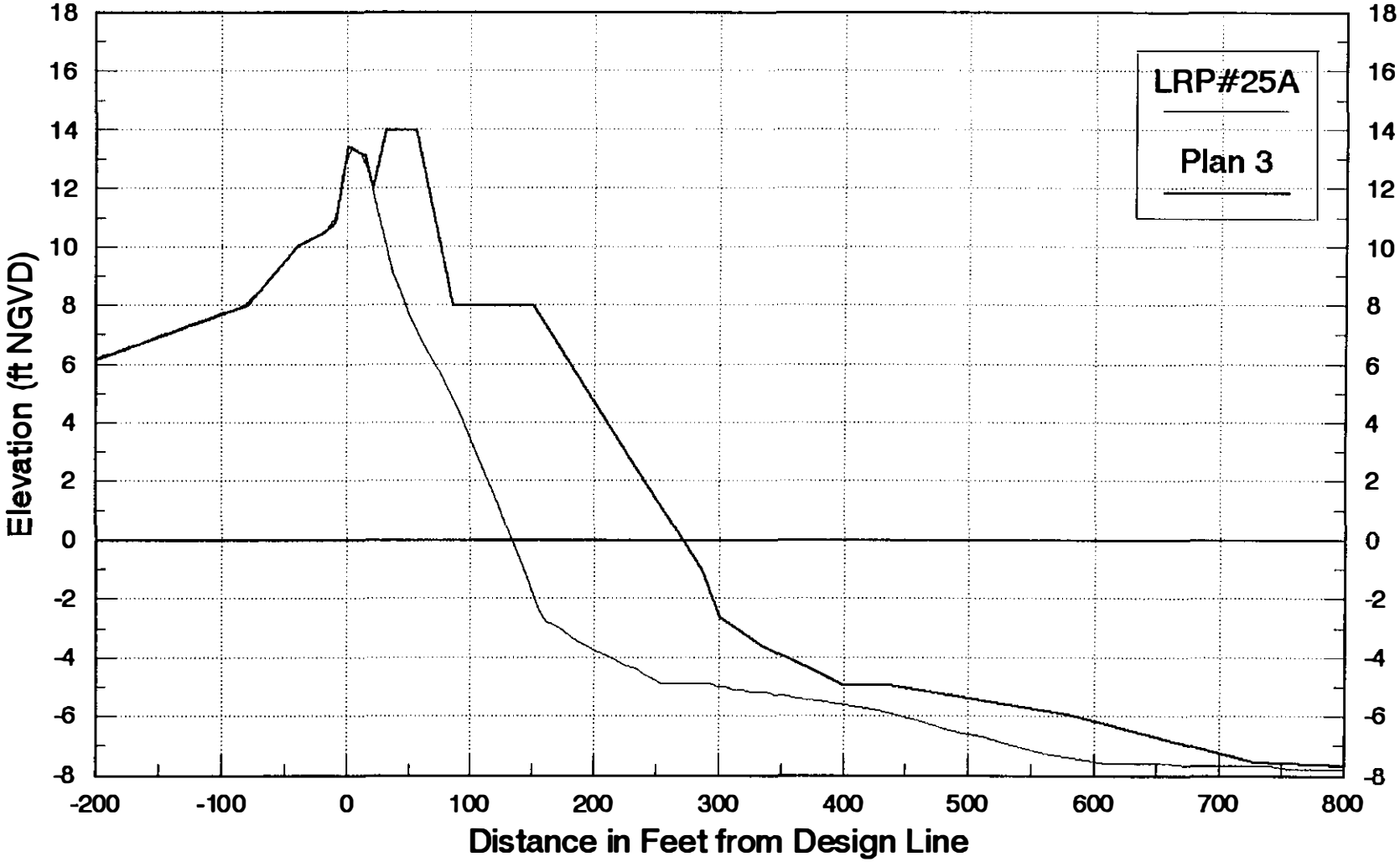

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FIGURE 6. BEACH FILL PLAN 4
BROADKILL BEACH, DELAWARE

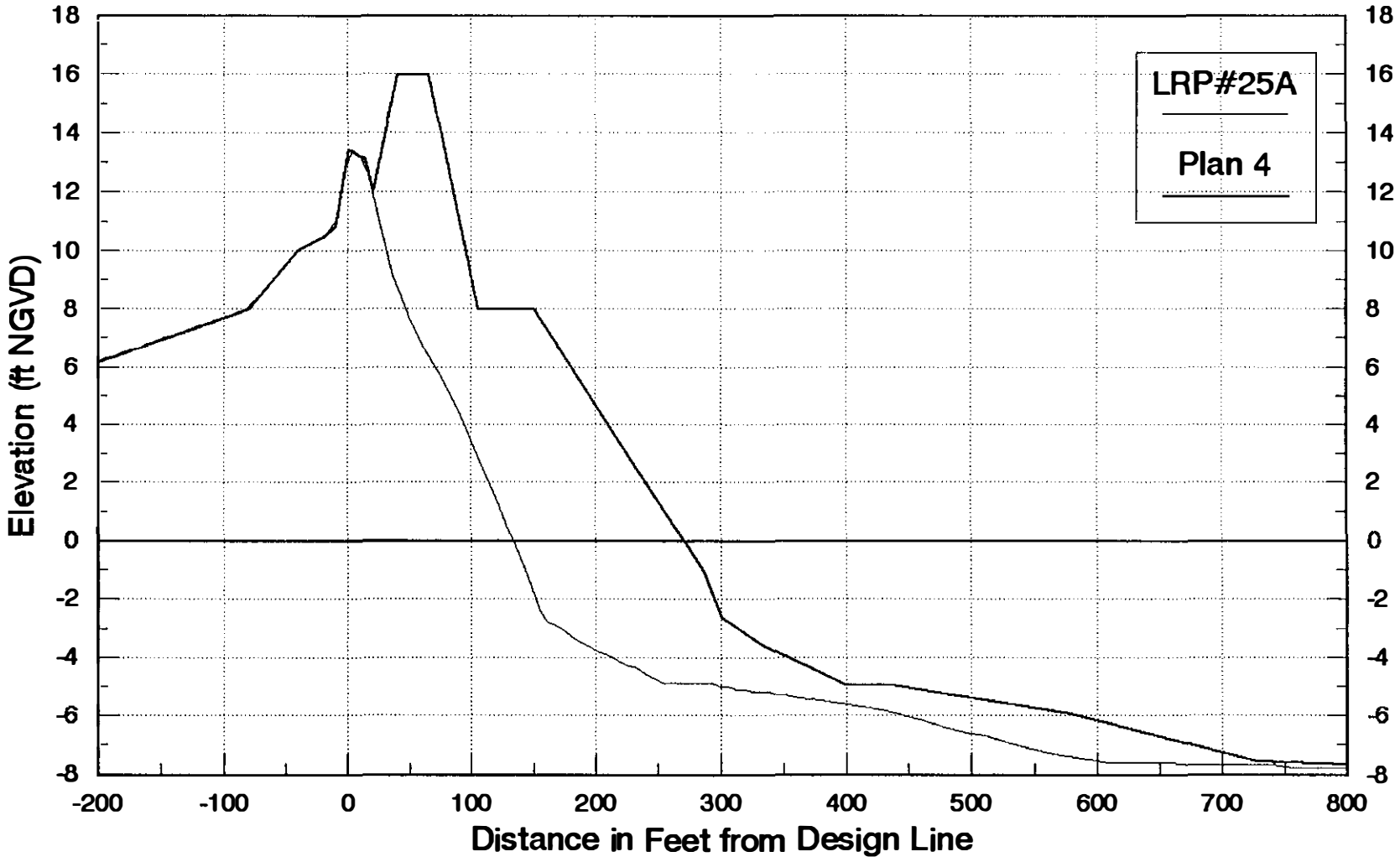

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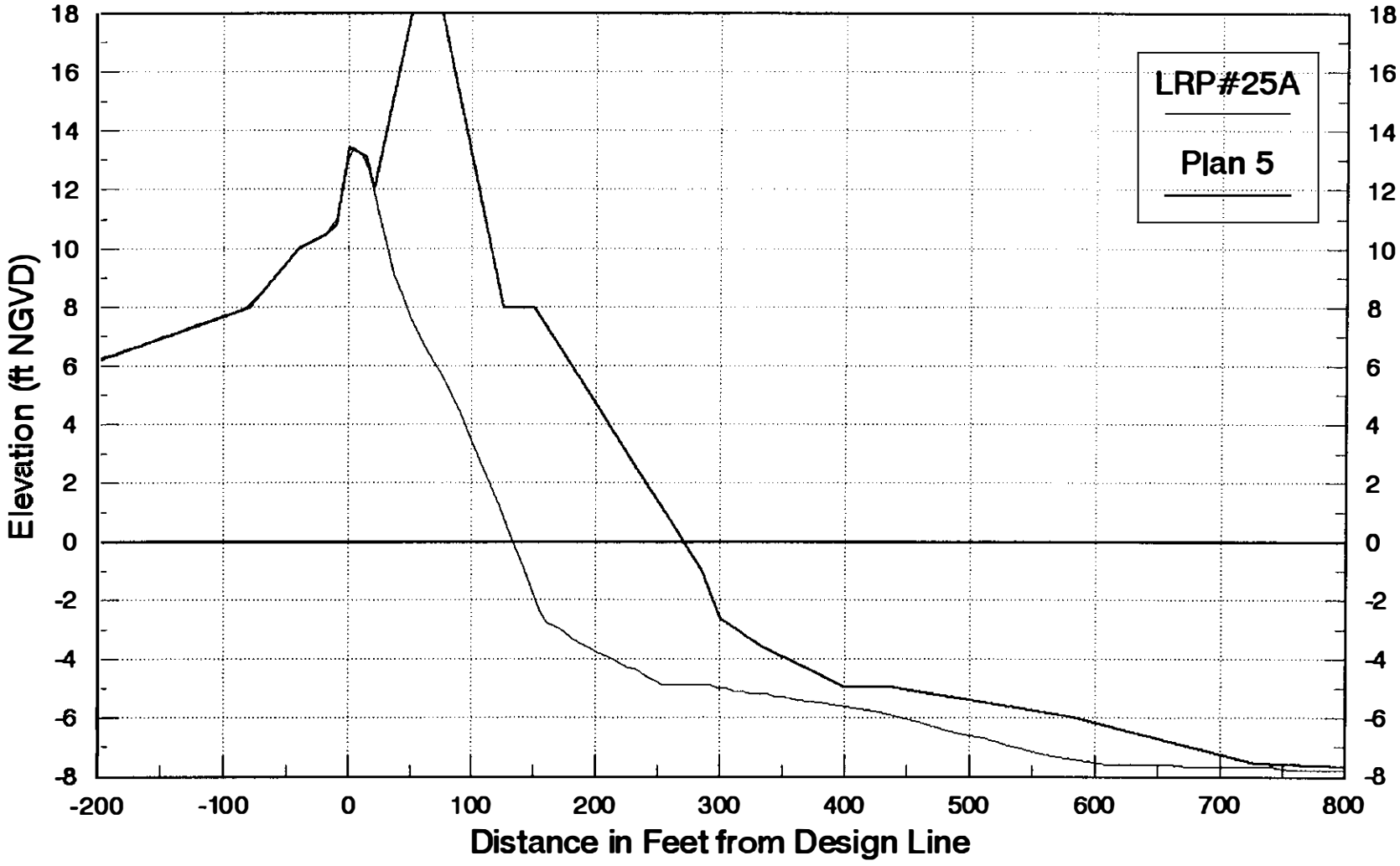
FIGURE 7. BEACH FILL PLAN 5
BROADKILL BEACH, DELAWARE



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SECTION III
DETAILED EVALUATION OF SELECTED ALTERNATIVES

The hydraulic performance of the five beach and dune restoration plans were evaluated based on erosion, wave attack and inundation under a range of design storm conditions. The Storm Induced Beach Change Model (SBEACH) (Rosati et. al., 1992) was used to estimate the dune erosion for a range of selected storms. The application of the SBEACH program to model beach profile changes at Broadkill Beach was discussed in the Phase I report. Briefly, SBEACH simulates beach profile changes under varying storm waves and water levels. It was used in this study to evaluate the performance of different beach fill plans. The program was calibrated for Broadkill Beach and has been used for several nearby locations.

For each survey profile, Table 5 presents a comparison of the distance of the eroded dune from the design line for the plans evaluated and without project conditions. Table 6 presents the volume eroded above NGVD for the plans. Table 7 presents the erosion volume from private properties (determined by eroded volume landward of the design line).

Wave attack and inundation under each plan were then evaluated considering wave setup and runup over the eroded dunes. The approach for analyzing the storm erosion and wave attack was described in detail in the Phase I report. Tables 8-12 present the wave inundation results for each plan. The total water level in the tables is the combined height (above NGVD) of storm surge, setup and wave height.

The erosion and wave inundation results were provided to the project team to determine the optimal plan. Based on economic analysis, Plan 2 with a 16-foot dune and 100-foot berm was selected. The eroded profiles modeled by SBEACH under this plan are provided in Appendix D. The project team also determined the limit of project fill area based on economic analysis. The project limit is shown in Figure 8. Also shown in Figure 8 is the shoreline position for the project base year (2001) predicted by the GENESIS model. Table 13 presents the total eroded volume above NGVD for the study area under selected storm events.

TABLE 5. BROADKILL BEACH FEASIBILITY STUDY
DUNE EROSION DISTANCE BEHIND DESIGN LINE
(FEET FROM DESIGN LINE)

PROFILE: LRP#24A

PLAN	5-YR	10-YR	20-YR	50-YR	100-YR	200-YR	500-YR
EXIST	-	-	-	10	10	14	14
1	-	-	-	10	10	10	14
2	-	-	-	-	5	10	10
3	-	-	-	5	10	10	14
4	-	-	-	-	5	5	5
5	-	-	-	-	-	5	5

PROFILE: N33+00

PLAN	5-YR	10-YR	20-YR	50-YR	100-YR	200-YR	500-YR
EXIST	90	90	90	110	110	110	114
1	-	-	-	110	110	110	110
2	-	-	-	100	100	105	110
3	-	-	-	100	100	105	110
4	-	-	-	100	105	105	105
5	-	-	-	100	100	100	105

PROFILE: LRP#25

PLAN	5-YR	10-YR	20-YR	50-YR	100-YR	200-YR	500-YR
EXIST	10	42	75	82	88	88	112
1	-	-	-	-	20	40	70
2	-	-	-	-	-	20	50
3	-	-	-	-	-	10	30
4	-	-	-	-	-	-	10
5	-	-	-	-	-	-	-

- eroded dune crest seaward of the design line

TABLE 5. BROADKILL BEACH FEASIBILITY STUDY
DUNE EROSION DISTANCE BEHIND DESIGN LINE
(FEET FROM DESIGN LINE) (Continued)

PROFILE:LRP#25A

PLAN	5-YR	10-YR	20-YR	50-YR	100-YR	200-YR	500-YR
EXIST	6	32	42	59	72	72	82
1	-	-	6	21	36	50	52
2	-	-	5	6	10	35	35
3	-	-	-	5	6	10	16
4	-	-	-	3	6	10	11
5	-	-	-	1	1	4	5

- eroded dune crest seaward of the design line

PROFILE:LRP#25B

PLAN	5-YR	10-YR	20-YR	50-YR	100-YR	200-YR	500-YR
EXIST	-	0	7	16	23	23	53
1	-	-	-	-	5	15	39
2	-	-	-	-	5	10	20
3	-	-	-	-	-	5	15
4	-	-	-	-	-	-	10
5	-	-	-	-	-	-	-

- eroded dune crest seaward of the design line

PROFILE:LRP#26

PLAN	5-YR	10-YR	20-YR	50-YR	100-YR	200-YR	500-YR
EXIST	35	51	55	61	61	64	74
1	-	-	-	25	25	30	40
2	-	-	-	-	-	20	30
3	-	-	-	-	-	10	20
4	-	-	-	-	-	-	10
5	-	-	-	-	-	-	-

- eroded dune crest seaward of the design line

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TABLE 5. DUNE EROSION DISTANCE BEHIND DESIGN LINE
(FEET FROM DESIGN LINE) (Continued)

PROFILE: LRP#27

PLAN	5-YR	10-YR	20-YR	50-YR	100-YR	200-YR	500-YR
EXIST	25	34	37	76	76	83	122
1	-	-	-	50	60	60	60
2	-	-	-	20	60	60	60
3	-	-	-	-	40	50	50
4	-	-	-	-	30	40	40
5	-	-	-	-	-	40	40

- eroded dune crest seaward of the design line

PROFILE:LRP#27A

PLAN	5-YR	10-YR	20-YR	50-YR	100-YR	200-YR	500-YR
EXIST	20	36	79	92	92	98	102
1	-	-	-	35	40	50	80
2	-	-	-	30	35	40	50
3	-	-	-	20	30	35	40
4	-	-	-	20	25	30	35
5	-	-	-	15	20	25	30

- eroded dune crest seaward of the design line

PROFILE:LRP#28

PLAN	5-YR	10-YR	20-YR	50-YR	100-YR	200-YR	500-YR
EXIST	38	64	67	77	84	90	107
1	-	-	-	50	60	60	70
2	-	-	-	-	40	50	55
3	-	-	-	-	-	35	40
4	-	-	-	-	-	-	-
5	-	-	-	-	-	-	-

- eroded dune crest seaward of the design line

TABLE 6. BROADKILL BEACH FEASIBILITY STUDY
VOLUME ERODED ABOVE NGVD

VOLUME ERODED ABOVE NGVD (CY/FT) PLAN 1				
PROFILE	20-YR	50-YR	100-YR	200-YR
LRP#24A	13.1	16.5	18.8	19.8
N33+00	8.3	18.5	18.5	18.6
LRP#25	11.0	12.2	18.3	21.2
LRP#25A	12.8	17.1	20.5	26.9
LRP#25B	12.8	16.7	20.3	24.7
LRP#26	12.5	19.9	20.4	23.3
LRP#27	5.8	9.5	12.8	13.7
LRP#27A	10.9	15.2	16.9	18.3
LRP#28	7.6	10.8	14.8	20.5

VOLUME ERODED ABOVE NGVD (CY/FT) PLAN 2				
PROFILE	20-YR	50-YR	100-YR	200-YR
LRP#24A	12.6	17.2	19.9	24.1
N33+00	10.6	16.7	20.7	22.3
LRP#25	12.3	16.8	20.2	24.2
LRP#25A	12.0	17.4	21.1	27.0
LRP#25B	14.0	18.7	23.8	26.2
LRP#26	14.0	18.8	23.1	24.9
LRP#27	7.3	11.6	14.5	20.0
LRP#27A	12.0	14.7	17.5	19.3
LRP#28	9.3	12.4	14.1	17.1

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TABLE 6. BROADKILL BEACH FEASIBILITY STUDY
VOLUME ERODED ABOVE NGVD (Continued)

VOLUME ERODED ABOVE NGVD (CY/FT) PLAN 3				
PROFILE	20-YR	50-YR	100-YR	200-YR
LRP#24A	12.5	15.3	20.4	21.9
N33+00	7.4	11.1	17.0	18.6
LRP#25	9.3	13.8	16.3	19.1
LRP#25A	11.0	15.6	17.0	22.1
LRP#25B	10.4	17.2	21.2	23.4
LRP#26	10.4	15.4	18.1	19.2
LRP#27	5.4	7.2	11.8	14.0
LRP#27A	10.9	13.5	16.7	16.5
LRP#28	5.9	8.5	10.8	15.4

VOLUME ERODED ABOVE NGVD (CY/FT) PLAN 4				
PROFILE	20-YR	50-YR	100-YR	200-YR
LRP#24A	12.3	17.6	22.8	24.6
N33+00	7.4	14.9	20.5	22.0
LRP#25	11.7	16.0	18.0	25.0
LRP#25A	11.0	17.0	19.0	27.2
LRP#25B	10.5	19.3	20.5	26.2
LRP#26	10.3	17.7	21.9	23.3
LRP#27	5.5	10.1	14.0	15.9
LRP#27A	10.9	16.3	17.8	18.0
LRP#28	5.8	8.7	12.5	14.3

TABLE 6. BROADKILL BEACH FEASIBILITY STUDY
VOLUME ERODED ABOVE NGVD (Continued)

VOLUME ERODED ABOVE NGVD (CY/FT) PLAN 5				
PROFILE	20-YR	50-YR	100-YR	200-YR
LRP#24A	12.4	18.4	23.5	26.3
N33+00	7.4	14.5	23.6	23.6
LRP#25	9.6	16.8	20.0	23.0
LRP#25A	11.5	18.1	21.0	25.0
LRP#25B	10.5	20.1	24.9	28.2
LRP#26	10.6	19.0	22.3	25.3
LRP#27	5.5	11.3	16.0	17.8
LRP#27A	11.0	16.5	18.8	18.9
LRP#28	6.0	10.9	14.6	17.2

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TABLE 7. BROADKILL BEACH FEASIBILITY STUDY
VOLUME ERODED FROM PRIVATE PROPERTY
(CY/FT)

PROFILE: LRP#24A

PLAN	5-YR	10-YR	20-YR	50-YR	100-YR	200-YR	500-YR
EXIST	0.0	0.0	0.2	0.4	0.5	0.5	0.8
1	0.0	0.0	0.0	0.4	0.5	0.5	0.8
2	0.0	0.0	0.0	0.0	0.5	0.5	0.5
3	0.0	0.0	0.0	0.2	0.5	0.5	0.8
4	0.0	0.0	0.0	0.0	0.3	0.3	0.3
5	0.0	0.0	0.0	0.0	0.0	0.2	0.2

PROFILE: N33+00

PLAN	5-YR	10-YR	20-YR	50-YR	100-YR	200-YR	500-YR
EXIST	0.0	0.1	0.1	1.6	1.3	1.3	2.1
1	0.0	0.0	0.0	0.7	0.8	0.8	0.8
2	0.0	0.0	0.0	0.5	0.6	0.6	0.6
3	0.0	0.0	0.0	0.3	0.3	0.5	0.6
4	0.0	0.0	0.0	0.4	0.5	0.5	0.5
5	0.0	0.0	0.0	0.4	0.5	0.5	0.5

PROFILE: LRP#25

PLAN	5-YR	10-YR	20-YR	50-YR	100-YR	200-YR	500-YR
EXIST	0.6	3.4	5.0	6.4	7.9	9.2	12.4
1	0.0	0.0	0.0	0.0	1.0	1.2	4.0
2	0.0	0.0	0.0	0.0	0.0	0.6	1.4
3	0.0	0.0	0.0	0.0	0.0	0.0	0.7
4	0.0	0.0	0.0	0.0	0.0	0.3	0.3
5	0.0	0.0	0.0	0.0	0.0	0.0	0.0

TABLE 7. BROADKILL BEACH FEASIBILITY STUDY
VOLUME ERODED FROM PRIVATE PROPERTY (Continued)
(CY/FT)

PROFILE: LRP#25A

PLAN	5-YR	10-YR	20-YR	50-YR	100-YR	200-YR	500-YR
EXIST	0.8	2.9	3.9	6.2	8.1	9.9	11.1
1	0.0	0.0	0.5	1.6	2.1	3.9	3.7
2	0.0	0.0	0.5	0.5	1.0	2.4	2.0
3	0.0	0.0	0.0	0.3	0.3	0.9	1.1
4	0.0	0.0	0.0	0.2	0.3	1.2	1.1
5	0.0	0.0	0.0	0.0	0.1	0.4	0.6

PROFILE: LRP#25B

PLAN	5-YR	10-YR	20-YR	50-YR	100-YR	200-YR	500-YR
EXIST	0.0	0.1	0.5	1.2	1.6	2.0	3.4
1	0.0	0.0	0.0	0.0	0.5	1.9	1.2
2	0.0	0.0	0.0	0.0	0.5	0.8	0.9
3	0.0	0.0	0.0	0.0	0.2	0.4	0.6
4	0.0	0.0	0.0	0.0	0.0	0.0	0.5
5	0.0	0.0	0.0	0.0	0.0	0.0	0.0

PROFILE: LRP#26

PLAN	5-YR	10-YR	20-YR	50-YR	100-YR	200-YR	500-YR
EXIST	0.6	3.1	3.4	5.0	5.6	6.4	7.7
1	0.0	0.0	0.0	1.7	1.0	1.6	2.6
2	0.0	0.0	0.0	0.0	0.0	0.3	2.4
3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4	0.0	0.0	0.0	0.0	0.0	0.0	0.0
5	0.0	0.0	0.0	0.0	0.0	0.0	0.0

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TABLE 7. BROADKILL BEACH FEASIBILITY STUDY
VOLUME ERODED FROM PRIVATE PROPERTY (Continued)
(CY/FT)

PROFILE:LRP#27

PLAN	5-YR	10-YR	20-YR	50-YR	100-YR	200-YR	500-YR
EXIST	0.0	0.4	0.5	2.4	2.7	3.2	6.9
1	0.0	0.0	0.0	1.0	1.3	1.0	1.0
2	0.0	0.0	0.0	0.0	1.0	1.0	1.0
3	0.0	0.0	0.0	0.0	0.1	0.4	0.5
4	0.0	0.0	0.0	0.0	0.0	0.2	0.4
5	0.0	0.0	0.0	0.0	0.0	0.0	0.0

PROFILE: LRP#27A

PLAN	5-YR	10-YR	20-YR	50-YR	100-YR	200-YR	500-YR
EXIST	0.6	3.2	7.5	10.8	11.9	13.4	15.0
1	0.0	0.0	0.0	1.8	1.8	2.0	4.4
2	0.0	0.0	0.0	1.0	1.7	2.0	2.8
3	0.0	0.0	0.0	0.4	1.4	1.8	2.6
4	0.0	0.0	0.0	0.0	1.6	1.6	2.4
5	0.0	0.0	0.0	0.0	1.0	1.0	1.2

PROFILE: LRP#28

PLAN	5-YR	10-YR	20-YR	50-YR	100-YR	200-YR	500-YR
EXIST	0.5	2.6	3.1	5.1	5.7	6.6	8.4
1	0.0	0.0	0.0	0.3	2.3	1.5	1.0
2	0.0	0.0	0.0	0.0	0.1	0.2	0.3
3	0.0	0.0	0.0	0.0	0.0	0.2	0.4
4	0.0	0.0	0.0	0.0	0.0	0.0	0.0
5	0.0	0.0	0.0	0.0	0.0	0.0	0.0

TABLE 8. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 1

PROFILE: LRP#24A

RETURN INTERVAL (years)	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0 50 100 150-river	16.5 15.5 14.5 12.4	0
200	0 50 80-river	15.0 14.0 13.5	0
100	-50 0 50 100 130-river	14.0 13.0 12.0 11.0 10.5	-50
50	-80 -30 20 70 120-river	13.0 12.0 11.0 10.0 9.0	-80
20	-80 -60 0-river	12.5 12.5 7.0	-80
10	-90 -75 -50-river	11.0 11.0 5.5	-90
5	-120 -80 -50-river	9.5 9.5 4.0	-120

TABLE 8. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 1 (Continued)

PROFILE: N33+00

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0	17.5	80
	30	17.0	
	80	16.5	
	130	15.5	
	150-river	14.6	
200	-20	15.5	
	0	15.0	0
	50	14.0	
	100	13.0	
	150	12.0	
	170-river	11.2	
100	-20	14.0	-20
	30	13.0	
	80	12.0	
	100	11.5	
	150-river	10.5	
50	-50	13.0	-50
	0	12.0	
	50	11.0	
	100	10.0	
	150-river	9.0	
20	-80	12.5	-80
	-60	12.5	
	0-river	7.0	
10	-80	11.0	-80
	-70	11.0	
	-50-river	5.5	
	-100	9.5	-100
	-80	9.5	
	-50-river	4.0	

TABLE 8. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 1 (Continued)

PROFILE: LRP#25

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0	17.0	
	50	16.0	50
	100	15.0	
	120-river	14.6	
200	-30	15.5	-30
	20	14.0	
	70	13.0	
	120	12.0	
	170-river	11.2	
100	-50	14.0	-50
	0	13.0	
	50	12.0	
	100-river	10.5	
50	-50	13.0	-50
	0	13.0	
	50-river	9.0	
20	-70	12.5	-70
	-40	12.5	
	0-river	7.0	
10	-80	11.0	-80
	-50	11.0	
	0-river	5.5	
5	-100	9.5	-100
	-80	9.5	
	-50-river	4.0	

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TABLE 8. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 1 (Continued)

PROFILE: LRP#25A

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0 50-river	17.5 16.5	entire area in wave zone
200	0 50 100 150-river	16.0 15.5 14.5 13.5	50
100	0 50 100 150 200-river	14.0 13.5 12.5 11.5 10.5	0
50	-50 0 50 100 150-river	13.0 12.0 11.0 10.0 9.0	-50
20	-80 -50 0-river	12.5 12.5 7.0	-80
10	-100 -50 -20-river	11.0 11.0 5.5	-100
5	-100 -80 -50-river	9.5 9.5 4.0	-100

TABLE 8. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 1 (Continued)

PROFILE: LRP#25B

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0 30 80-river	18.0 17.5 16.5	entire area in wave zone
200	0 30 80 130-river	16.5 15.5 14.0 13.5	50
100	0 50 100 150 200-river	14.0 13.0 12.5 11.5 10.5	0
50	-30 0 50 100 150 180-river	13.0 12.5 11.5 10.5 9.5 9.0	-30
20	-50 0 50-river	12.5 12.5 7.0	-50
10	-80 -50 0-river	11.0 11.0 5.5	-80
5	-100 -80 -50-river	9.5 9.5 4.0	-100

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TABLE 8. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 1 (Continued)

PROFILE: LRP#26

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0	18.0	
	50	17.0	
	100	16.0	100
	150	15.0	
	200-river	14.6	
200	0	16.0	
	50	15.0	50
	100	14.0	
	130-river	13.5	
100	-30	14.5	-30
	0	14.0	
	50	13.0	
	100	12.0	
	150	11.0	
	170-river	10.5	
50	-50	13.0	-50
	0	12.0	
	50	11.0	
	100	10.0	
	150-river	9.0	
20	-70	12.5	-70
	-50	12.5	
	0-river	7.0	
10	-80	11.0	-80
	-60	11.0	
	0-river	5.5	
5	-100	9.5	-100
	-80	9.5	
	-50-river	4.0	

TABLE 8. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 1 (Continued)

PROFILE: LRP#27

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0	17.0	
	20	16.5	20
	50	15.5	
	100	14.5	
	150-river	12.4	
200	-20	15.0	-20
	0	14.0	
	50	13.0	
	100-river	11.2	
100	-40	14.0	-40
	0	13.0	
	50	12.0	
	100	11.0	
	120-river	10.5	
50	-80	13.0	-80
	-50	13.0	
	0-river	9.0	
20	-80	12.5	-80
	-60	12.5	
	0-river	7.0	
10	-80	11.0	-80
	-70	11.0	
	-50-river	5.5	
5	-120	9.5	-120
	-80	9.5	
	-50-river	4.0	

TABLE 8. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 1 (Continued)

PROFILE: LRP#27A

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0 50 100 150-river	16.5 15.5 14.5 12.4	0
200	-40 0 50-river	15.5 14.5 11.2	-40
100	-60 -30 0-river	14.5 14.0 10.5	-60
50	-60 20 50-river	13.0 13.0 9.0	-60
20	-80 -70 -50-river	12.5 12.5 7.0	-80
10	-80 -70 -50-river	11.0 11.0 5.5	-80
5	-100 -80 -50-river	9.5 9.5 4.0	-100

TABLE 8. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 1 (Continued)

PROFILE: LRP#28

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0 20 50 100 150-river	17.5 16.5 16.0 15.0 12.4	20
200	0 50 100 150-river	15.5 14.5 13.5 11.2	0
100	-70 -50 0-river	14.5 13.0 10.5	-70
50	-70 -40 0-river	13.0 13.0 9.0	-70
20	-80 -50 0-river	12.5 12.5 7.0	-80
10	-80 -70 -50-river	11.0 11.0 5.5	-80
5	-110 -80 -50-river	9.5 9.5 4.0	-110

TABLE 9. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 2

PROFILE: LRP#24A

RETURN INTERVAL (years)	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0 50 100-river	16.5 15.5 14.6	0
200	-30 0 50 100 150 170-river	15.5 15.0 14.0 13.0 12.0 11.2	-30
100	-50 0 50 100 130-river	14.0 13.0 12.0 11.0 10.5	-50
50	-80 -30 0-river	13.0 13.0 9.0	-80
20	-100 -80 -50-river	12.5 12.5 7.0	-100
10	-100 -90 -50-river	11.0 11.0 5.5	-100
5	-120 -100 -50-river	9.5 9.5 4.0	-120

TABLE 9. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 2 (Continued)

PROFILE: N33+00

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0	17.5	
	50	16.5	50
	100	15.5	
	150	14.5	
	200	13.5	
	250-river	12.4	
200	-50	15.5	
	0	15.0	0
	50	14.0	
	100	13.0	
	150	12.0	
	170-river	11.2	
100	-20	14.0	-20
	30	13.0	
	80	12.0	
	100-river	10.5	
50	-50	13.0	-50
	0	12.0	
	30	11.0	
	50	10.0	
	100-river	9.0	
20	-80	12.5	-50
	-30	12.5	
	0-river	7.0	
10	-100	11.0	-100
	-80	11.0	
	-50-river	5.5	
5	-120	9.5	-120
	-100	9.5	
	-50-river	4.0	

TABLE 9. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 2 (Continued)

PROFILE: LRP#25

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0 50 100 120-river	17.0 16.0 15.0 14.6	50
200	-40 0 50 100 150-river	15.5 14.5 13.5 12.5 11.2	-40
100	-50 0 50 100-river	14.0 13.0 12.0 10.5	-50
50	-70 -20 0-river	13.0 13.0 9.0	-70
20	-80 -50 0-river	12.5 12.5 7.0	-80
10	-100 -80 -50-river	11.0 11.0 5.5	-100
5	-120 -100 -50-river	9.5 9.5 4.0	-120

TABLE 9. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 2 (Continued)

PROFILE: LRP#25A

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0 50 100 130-river	17.0 16.0 15.0 14.6	50
200	0 50 100-river	15.5 14.5 13.5	0
100	-30 0 50 100 150-river	14.0 13.5 12.5 11.5 10.5	-30
50	-80 0 50 100 130-river	13.0 12.0 11.0 10.0 9.0	-80
20	-80 -60 0-river	12.5 12.5 7.0	-80
10	-90 -50 0-river	11.0 11.0 5.5	-90
5	-100 -80 -50-river	9.5 9.5 4.0	-100

TABLE 9. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 2 (Continued)

PROFILE: LRP#25B

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0 50 100 150-river	17.5 16.5 15.5 14.6	50
200	0 30 80 130-river	16.0 15.5 14.5 13.5	30
100	0 30 80 130-river	14.0 13.0 12.5 10.5	0
50	-50 0 50 150-river	13.0 12.0 10.0 9.0	-50
20	-80 -20 0-river	12.5 12.5 7.0	-80
10	-100 -80 -50-river	11.0 11.0 5.5	-100
5	-150 -100 -50-river	9.5 9.5 4.0	-150

TABLE 9. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 2 (Continued)

PROFILE: LRP#26

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0	18.0	
	50	17.0	
	100	16.0	100
	150	15.0	
	170-river	14.6	
200	0	15.5	0
	50	14.5	
	100-river	13.5	
100	0	14.0	0
	50	13.0	
	100	12.0	
	150-river	10.5	
50	-50	13.0	-50
	0	12.0	
	50	11.0	
	100	10.0	
	150-river	9.0	
20	-80	12.5	-80
	-50	12.5	
	0-river	7.0	
10	-80	11.0	-80
	-70	11.0	
	-50-river	5.5	
5	-110	9.5	-110
	-100	9.5	
	-50-river	4.0	

TABLE 9. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 2 (Continued)

PROFILE: LRP#27

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0 50 100 150-river	16.5 15.5 14.5 12.4	0
200	-30 0 50 100-river	15.0 14.0 13.0 11.2	-30
100	-50 0 50-river	13.5 12.5 10.5	-50
50	-30 0 50-river	13.0 12.0 9.0	-30
20	-80 -60 0-river	12.5 12.5 7.0	-80
10	-80 -70 -50-river	11.0 11.0 5.5	-80
5	-130 -100 -50-river	9.5 9.5 4.0	-130

TABLE 9. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 2 (Continued)

PROFILE: LRP#27A

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	-30 0 50-river	16.5 15.5 12.4	0
200	-50 0 30-river	15.5 14.0 11.2	-50
100	-80 -50 0-river	14.5 13.0 10.5	-80
50	-80 0 30-river	13.0 13.0 9.0	-80
20	-80 -50 0-river	12.5 12.5 7.0	-80
10	-100 -80 -50-river	11.0 11.0 5.5	-100
5	-120 -100 -50-river	9.5 9.5 4.0	-120

TABLE 9. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 2 (Continued)

PROFILE: LRP#28

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0 50 100 150-river	16.5 15.5 14.5 12.4	0
200	-50 0 50-river	15.5 14.0 11.2	-50
100	-70 -50 0-river	14.5 13.0 10.5	-70
50	-70 -50 0-river	13.0 13.0 9.0	-70
20	-70 -50 0-river	12.5 12.5 7.0	-70
10	-100 -80 -50-river	11.0 11.0 5.5	-100
5	-120 -70 -50-river	9.5 9.5 4.0	-120

TABLE 10. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 3

PROFILE: LRP#24A

RETURN INTERVAL (years)	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0 50 100-river	16.5 15.5 14.6	0
200	0 50 100 150 170-river	15.0 14.0 13.0 12.0 11.2	0
100	-50 0 50 100 130-river	14.0 13.0 12.0 11.0 10.5	-50
50	-80 -30 0-river	13.0 12.0 9.0	-80
20	-80 -60 0-river	12.5 12.5 7.0	-80
10	-90 -75 -50-river	11.0 11.0 5.5	-90
5	-120 -80 -50-river	9.5 9.5 4.0	-120

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TABLE 10. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 3 (Continued)

PROFILE: N33+00

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0 50 100 150-river	16.5 15.5 14.5 12.4	0
200	-50 0 50 100 150-river	15.5 14.5 13.5 12.5 11.2	-50
100	-50 0 50 100 130-river	14.0 13.0 12.0 11.0 10.5	-50
50	-80 -30 20 70 130-river	13.0 12.0 11.0 10.0 9.0	-80
20	-80 -60 0-river	12.5 12.5 7.0	-80
10	-80 -70 -50-river	11.0 11.0 5.5	-80
5	-150 -80 -50-river	9.5 9.5 4.0	-150

TABLE 10. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 3 (Continued)

PROFILE: LRP#25

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0	17.0	
	50	16.0	50
	100	15.0	
	150	14.0	
	200-river	12.4	
200	-50	15.5	-50
	0	14.5	
	50	13.5	
	100	12.5	
	120-river	11.2	
100	-50	14.0	-50
	0	13.0	
	50-river	10.5	
50	-80	13.0	-80
	-30	13.0	
	0-river	9.0	
20	-80	12.5	-80
	-50	12.5	
	0-river	7.0	
10	-80	11.0	-80
	-50	11.0	
	0-river	5.5	
5	-100	9.5	-100
	-80	9.5	
	-50-river	4.0	

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TABLE 10. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 3 (Continued)

PROFILE: LRP#25A

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0 50 100-river	16.5 15.5 14.6	0
200	-30 0 50 100 170-river	15.0 14.5 13.5 12.5 11.2	-30
100	-50 0 50 100 150-river	14.5 13.5 12.5 11.5 10.5	-50
50	-70 -50 0-river	13.0 13.0 9.0	-70
20	-80 -65 -50-river	12.5 12.5 7.0	-80
10	-100 -60 -50-river	11.0 11.0 5.5	-100
5	-150 -75 -50-river	9.5 9.5 4.0	-150

TABLE 10. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 3 (Continued)

PROFILE: LRP#25B

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0 30 80 130-river	17.0 16.5 15.5 14.6	30
200	0 50 100-river	15.5 14.5 13.5	0
100	-80 -60 0 50 100-river	14.5 14.0 12.5 11.5 10.5	-60
50	-60 0 50 100-river	13.0 12.0 11.0 9.0	-60
20	-90 -65 -50-river	12.5 12.5 7.0	-90
10	-90 -70 -50-river	11.0 11.0 5.5	-90
5	-130 -80 -50-river	9.5 9.5 4.0	-130

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TABLE 10. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 3 (Continued)

PROFILE: LRP#26

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0	16.5	0
	50	15.5	
	100	14.5	
	150	13.5	
	200-river	12.4	
200	-50	15.5	-50
	0	14.5	
	50	13.5	
	100	12.5	
	150-river	11.2	
100	-80	14.5	-80
	-50	13.5	
	0	12.5	
	50-river	10.5	
50	-70	13.0	-70
	-50	13.0	
	0	12.0	
	50	11.0	
	100-river	9.0	
20	-70	12.5	-70
	-50	12.5	
	0-river	7.0	
10	-80	11.0	-80
	-70	11.0	
	-50-river	5.5	
5	-150	9.5	-150
	-80	9.5	
	-50-river	4.0	

TABLE 10. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 3 (Continued)

PROFILE: LRP#27

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	-50 0 50-river	16.5 15.0 12.4	-50
200	-80 -50 0 50-river	15.5 15.0 14.0 11.2	-80
100	-100 -80 0-river	14.5 14.0 10.5	-100
50	-100 -60 0-river	13.0 13.0 9.0	-100
20	-100 -80 -50-river	12.5 12.5 7.0	-100
10	-100 -90 -50-river	11.0 11.0 5.5	-100
5	-150 -100 -50-river	9.5 9.5 4.0	-150

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TABLE 10. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 3 (Continued)

PROFILE: LRP#27A

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	-50 0 50 100-river	16.5 15.5 14.5 12.4	-50
200	-50 0 30-river	15.0 14.0 11.2	-50
100	-80 -50 0-river	14.5 13.0 10.5	-80
50	-80 -50 0-river	13.0 13.0 9.0	-80
20	-80 -70 -50-river	12.5 12.5 7.0	-80
10	-100 -80 -50-river	11.0 11.0 5.5	-100
5	-150 -80 -50-river	9.5 9.5 4.0	-150

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TABLE 10. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 3 (Continued)

PROFILE: LRP#28

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0 50 100 150-river	16.5 15.5 14.5 12.4	0
200	-70 0 50-river	15.5 14.0 11.2	-70
100	-80 -30 0-river	14.5 12.0 10.5	-80
50	-80 -70 -50-river	13.0 13.0 9.0	-80
20	-80 -70 -50-river	12.5 12.5 7.0	-80
10	-90 -80 -50-river	11.0 11.0 5.5	-90
5	-120 -70 -50-river	9.5 9.5 4.0	-120

TABLE 11. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 4

PROFILE: LRP#24A

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	-30 0 50 100 150-river	16.5 16.0 15.0 14.0 12.4	-30
200	-30 0 50 100 150 170-river	15.5 15.0 14.0 13.0 12.0 11.2	-30
100	-80 -30 20 70 100-river	14.0 13.0 12.0 11.0 10.5	-80
50	-90 -40 0-river	13.0 13.0 9.0	-90
20	-100 -80 -50-river	12.5 12.5 7.0	-100
10	-100 -90 -50-river	11.0 11.0 5.5	-100
5	-120 -100 -50-river	9.5 9.5 4.0	-120

TABLE 11. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 4 (Continued)

PROFILE: N33+00

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	-20 0 50 100 150-river	17.0 16.0 15.0 14.0 12.4	-20
200	-50 0 50 100-river	15.0 14.0 12.5 11.2	-50
100	-80 -30 0 50-river	14.0 13.0 12.0 10.5	-80
50	-80 -50 0-river	13.0 13.0 9.0	-80
20	-100 -80 -50-river	12.5 12.5 7.0	-100
10	-100 -80 -50-river	11.0 11.0 5.5	-100
5	-150 -100 -50-river	9.5 9.5 4.0	-150

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TABLE 11. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 4 (Continued)

PROFILE: LRP#25

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0	17.0	
	30	16.5	30
	50	16.0	
	100	15.0	
	150	14.0	
	200-river	12.4	
200	-50	15.5	-50
	0	14.5	
	50	13.5	
	100	12.5	
	120-river	11.2	
100	-50	14.0	-50
	0	13.0	
	50-river	10.5	
50	-80	13.0	-80
	-30	13.0	
	0-river	9.0	
20	-80	12.5	-80
	-50	12.5	
	0-river	7.0	
10	-100	11.0	-100
	-80	11.0	
	-50-river	5.5	
5	-130	9.5	-130
	-100	9.5	
	-50-river	4.0	

TABLE 11. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 4 (Continued)

PROFILE: LRP#25A

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0 50 100-river	16.5 15.5 14.6	0
200	-50 0 50 100 150-river	15.5 14.5 13.5 12.5 11.2	-50
100	-50 0 50 100 150-river	14.5 13.5 12.5 11.5 10.5	-50
50	-90 -50 0-river	13.0 13.0 9.0	-90
20	-100 -80 -50-river	12.5 12.5 7.0	-100
10	-100 -80 -50-river	11.0 11.0 5.5	-100
5	-150 -100 -50-river	9.5 9.5 4.0	-150

TABLE 11. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 4 (Continued)

PROFILE: LRP#25B

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0 50 100 150-river	17.5 16.5 15.5 14.6	50
200	0 50 100 150 170-river	15.0 14.0 13.0 12.0 11.2	0
100	-50 -30 50 100-river	14.0 14.0 12.5 10.5	-50
50	-80 -30 0-river	13.0 13.0 9.0	-80
20	-100 -80 -50-river	12.5 12.5 7.0	-100
10	-100 -80 -50-river	11.0 11.0 5.5	-100
5	-150 -100 -50-river	9.5 9.5 4.0	-150

TABLE 11. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 4 (Continued)

PROFILE: LRP#26

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0	17.0	
	50	16.0	50
	100	15.0	
	150	14.0	
	200-river	12.4	
200	-50	15.5	-50
	0	14.5	
	50	13.5	
	100-river	11.2	
100	-80	14.0	-80
	-30	13.0	
	0	12.0	
	50-river	10.5	
50	-80	13.0	-80
	-30	12.0	
	20	11.0	
	70-river	9.0	
20	-100	12.5	-100
	-80	12.5	
	-50-river	7.0	
10	-110	11.0	-110
	-80	11.0	
	-50-river	5.5	
5	-150	9.5	-150
	-100	9.5	
	-50-river	4.0	

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TABLE 11. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 4 (Continued)

PROFILE: LRP#27

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	-50 0 50-river	16.5 15.0 12.4	-50
200	-80 -50 0 50-river	15.5 15.0 14.0 11.2	-80
100	-100 -80 0-river	14.5 14.0 10.5	-100
50	-100 -60 0-river	13.0 13.0 9.0	-100
20	-100 -80 -50-river	12.5 12.5 7.0	-100
10	-100 -90 -50-river	11.0 11.0 5.5	-100
5	-150 -100 -50-river	9.5 9.5 4.0	-150

TABLE 11. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 4 (Continued)

PROFILE: LRP#27A

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	-50 0 50-river	16.5 15.5 12.4	-50
200	-100 -50 0-river	15.5 14.0 11.2	-100
100	-100 -50 0-river	14.5 14.0 10.5	-100
50	-100 -80 -50-river	13.0 13.0 9.0	-100
20	-100 -80 -50-river	12.5 12.5 7.0	-100
10	-110 -90 -50-river	11.0 11.0 5.5	-110
5	-150 -100 -50-river	9.5 9.5 4.0	-150

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TABLE 11. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 4 (Continued)

PROFILE: LRP#28

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	-50 0 50 100-river	16.5 15.5 14.5 12.4	-50
200	-80 -50 0-river	15.5 14.5 11.2	-80
100	-100 -80 -50 0-river	14.5 13.5 12.5 10.5	-100
50	-100 -70 -50-river	13.0 13.0 9.0	-100
20	-100 -80 -50-river	12.5 12.5 7.0	-100
10	-120 -100 -50-river	11.0 11.0 5.5	-120
5	-150 -100 -50-river	9.5 9.5 4.0	-150

TABLE 12. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 5

PROFILE: LRP#24A

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	-50 0 50 100-river	16.5 15.5 14.5 12.4	-50
200	-90 -75 -50 0 50 70-river	15.5 15.0 14.0 13.0 12.0 11.2	-90
100	-90 -40 0-river	14.0 13.0 10.5	-90
50	-100 -80 -50-river	13.0 13.0 9.0	-100
20	-120 -100 -50-river	12.5 12.5 7.0	-120
10	-120 -110 -50-river	11.0 11.0 5.5	-120
5	-150 -120 -100-river	9.5 9.5 4.0	-150

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TABLE 12. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 5 (Continued)

PROFILE: N33+00

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	-30 -10 30 80 100-river	17.0 16.0 15.0 14.0 12.4	-30
200	-70 0 50-river	14.0 12.0 11.2	-70
100	-80 -50 0-river	14.0 13.0 10.5	-80
50	-110 -80 -50-river	13.0 13.0 9.0	-110
20	-120 -100 -50-river	12.5 12.5 7.0	-120
10	-130 -110 -50-river	11.0 11.0 5.5	-130
5	-150 -120 -100-river	9.5 9.5 4.0	-150

TABLE 12. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 5 (Continued)

PROFILE: LRP#25

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0 50 100-river	16.0 15.0 12.4	0
200	-70 0 50-river	15.0 13.5 11.2	-70
100	-80 -30 0-river	14.5 14.5 10.5	-80
50	-100 -70 -50-river	13.0 13.0 9.0	-100
20	-120 -100 -50-river	12.5 12.5 7.0	-120
10	-130 -110 -50-river	11.0 11.0 5.5	-130
5	-150 -120 -100-river	9.5 9.5 4.0	-150

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TABLE 12. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 5 (Continued)

PROFILE: LRP#25A

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	-40 0 50 100 150-river	16.5 15.5 14.5 13.5 12.4	-40
200	-50 0 50 100 170-river	15.0 14.0 13.0 12.0 11.2	-50
100	-70 -20 0 50-river	14.5 13.5 12.5 10.5	-70
50	-100 -50 -30-river	13.0 13.0 9.0	-100
20	-120 -100 -50-river	12.5 12.5 7.0	-120
10	-120 -110 -50-river	11.0 11.0 5.5	-120
5	-150 -120 -50-river	9.5 9.5 4.0	-150

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TABLE 12. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 5 (Continued)

PROFILE: LRP#25B

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0	17.0	0
	50	16.0	
	100	15.0	
	150	14.0	
	200	13.0	
	230-river	12.4	
200	-80	15.5	-80
	-30	14.5	
	20	13.5	
	70	12.5	
	100-river	11.2	
100	-80	13.5	-80
	-30	12.5	
	20	11.5	
	70-river	10.5	
50	-100	13.0	-100
	-50	13.0	
	0-river	9.0	
20	-120	12.5	-120
	-100	12.5	
	-50-river	7.0	
10	-130	11.0	-130
	-110	11.0	
	-50-river	5.5	
5	-150	9.5	-150
	-120	9.5	
	0-river	4.0	

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TABLE 12. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 5 (Continued)

PROFILE: LRP#26

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	0 50 100 150-river	16.5 15.5 14.5 12.4	0
200	-70 -20 0 50-river	15.0 14.0 13.0 11.2	-70
100	-80 -50 0-river	14.0 14.0 10.5	-80
50	-100 -50 0-river	13.0 13.0 9.0	-100
20	-120 -100 -50-river	12.5 12.5 7.0	-120
10	-120 -110 -50-river	11.0 11.0 5.5	-120
5	-150 -120 -100-river	9.5 9.5 4.0	-150

TABLE 12. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 5 (Continued)

PROFILE: LRP#27

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	-50 0 50-river	16.5 15.0 12.4	-50
200	-100 -50 -30 0-river	15.5 14.5 13.5 11.2	-70
100	-120 -50 0-river	14.5 14.0 10.5	-120
50	-120 -50 0-river	13.0 13.0 9.0	-120
20	-130 -100 -50-river	12.5 12.5 7.0	-130
10	-130 -110 -50-river	11.0 11.0 5.5	-130
5	-150 -120 -100-river	9.5 9.5 4.0	-150

TABLE 12. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 5 (Continued)

PROFILE: LRP#27A

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	-70 -50 0 50-river	16.5 15.5 14.5 12.4	-70
200	-100 -50 0-river	15.5 14.0 11.2	-100
100	-120 -50 0-river	14.5 12.5 10.5	-120
50	-120 -100 -50-river	13.0 13.0 9.0	-120
20	-120 -100 -50-river	12.5 12.5 7.0	-120
10	-130 -110 -50-river	11.0 11.0 5.5	-130
5	-150 -120 -100-river	9.5 9.5 4.0	-150

TABLE 12. WAVE INUNDATION RESULTS
BROADKILL BEACH, PLAN 5 (Continued)

PROFILE: LRP#28

RETURN INTERVAL	DISTANCE (ft landward of design line)	TOTAL WATER LEVEL (ft NGVD)	WAVE ZONE LOCATION (ft from design line)
500	-70 -50 0 50-river	16.5 15.5 14.5 12.4	-70
200	-100 -50 0-river	15.5 14.0 11.2	-100
100	-120 -100 -50 0-river	14.5 14.0 12.0 11.5	-120
50	-120 -80 -50-river	13.0 13.0 9.0	-120
20	-120 -100 -50-river	12.5 12.5 7.0	-120
10	-130 -110 -50-river	11.0 11.0 5.5	-130
5	-150 -120 -50-river	9.5 9.5 4.0	-150

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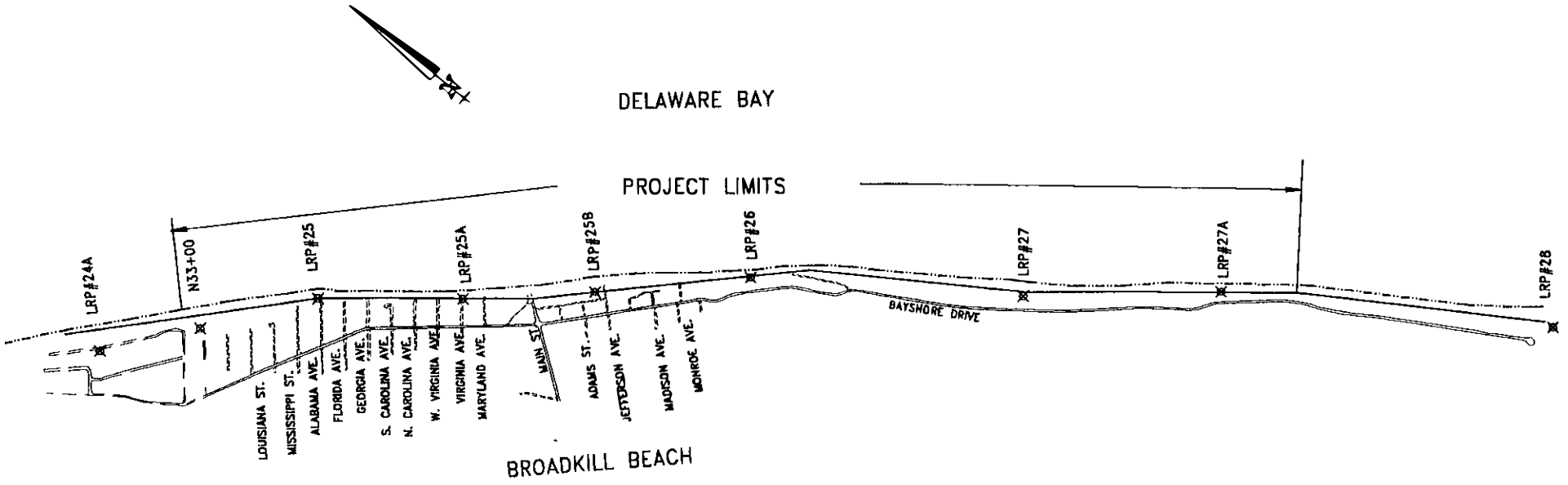

1993 SHORELINE

PROJECT DESIGN LINE

LRP#27A SURVEY PROFILES

FIGURE 8. PROJECT LIMITS, BROADKILL BEACH

SCALE 1"=1500'

TABLE 13. VOLUME ERODED ABOVE NGVD VS. STORM FREQUENCY
BROADKILL BEACH, PLAN 2

RETURN INTERVAL (yr)	5-YR	10-YR	20-YR	50-YR	100-YR	200-YR	500-YR
VOLUME (thousand cy)	85	118	153	210	254	312	341

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Section IV
Project Maintenance

In order to maintain the integrity of the design beachfill alternatives, beachfill nourishment must be included in the project design. If periodic nourishment is not performed throughout the life of the project, longshore and cross shore sediment transport mechanisms will act to erode the design beach. This erosion will reduce the protection from storm damage afforded by the project. The nourishment quantities are considered sacrificial material which acts to protect the design fill volume. Various coastal processes were analyzed to develop an estimate of the required annual nourishment fill volumes.

The nourishment rates for design were developed considering long term historic erosion losses using shoreline recession rates developed for the sediment budget, volumetric analysis of recent beach fills and profiles, beachfill losses due to the predicted rate of sea level rise, and losses due to storm induced erosion. The results of these analyses were compared and the volumetric requirements were combined to obtain the total nourishment needs for each of the alternatives.

Sea Level Rise Rate

Using the current sea level rise rate of 0.012 feet per year, as described earlier in this report, and the Bruun method, the distance of shoreline retreat over the 50 year project life was determined. It was estimated that an additional 3,000 cubic yards per year is required for each year between nourishments to offset losses due to sea level rise within the project area. This retreat rate was added to the longterm erosion losses to develop the project nourishment estimates.

Long Term Losses

The GENESIS model developed in the Phase I study was refined and calibrated to estimate long-term shoreline changes within the project area. The only time period where both shoreline positions and wave data are available for model calibration is from 1989 to 1993. During this period, the beach within the project area retreated in excess of 100 feet due to three large winter storms from 1991 to 1992. Figure 9 presents a comparison of the highwater shoreline position as photographed in April 1993, the computed shoreline using hindcast wave data, and the 1989 shoreline. The model shows good agreement in the center of Broadkill Beach but starts to deviate at both ends of the study area. It predicted more erosion to the north and more accretion to the south than actually occurred. Using the GENESIS model to estimate longterm erosion rates at the Broadkill Beach produced the same results as those derived from beach profile analysis.

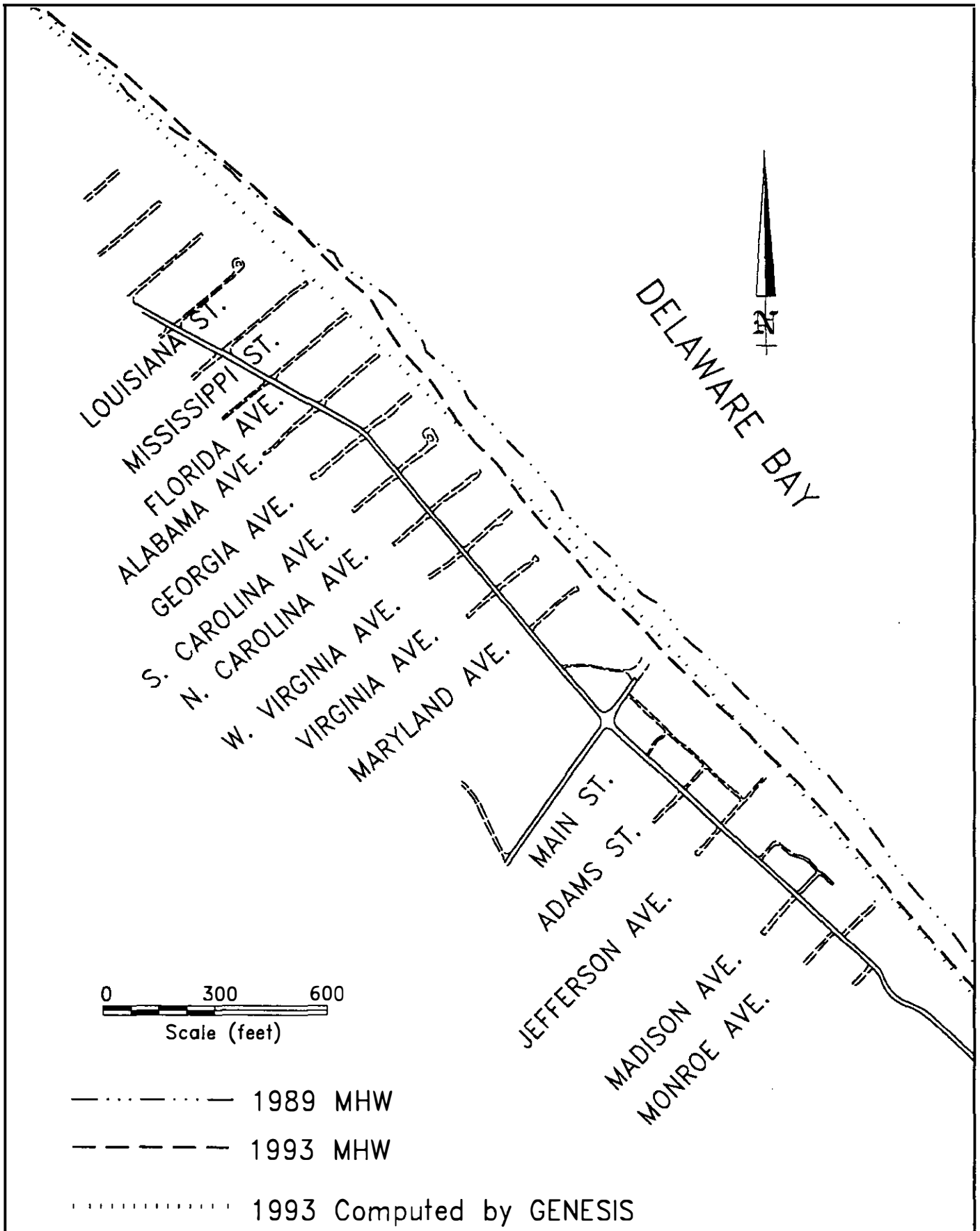


FIGURE 9. COMPARISON OF SHORELINES

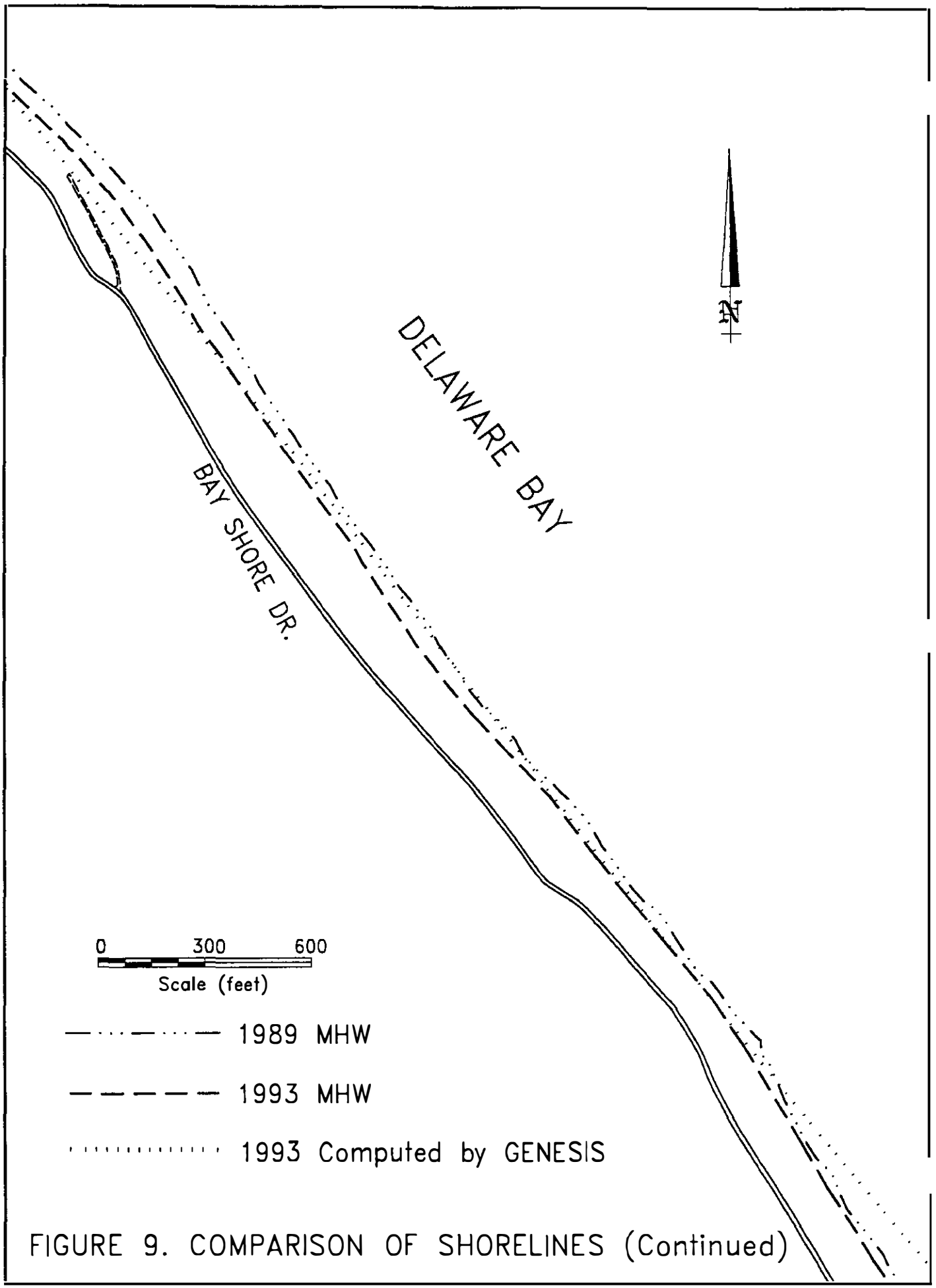
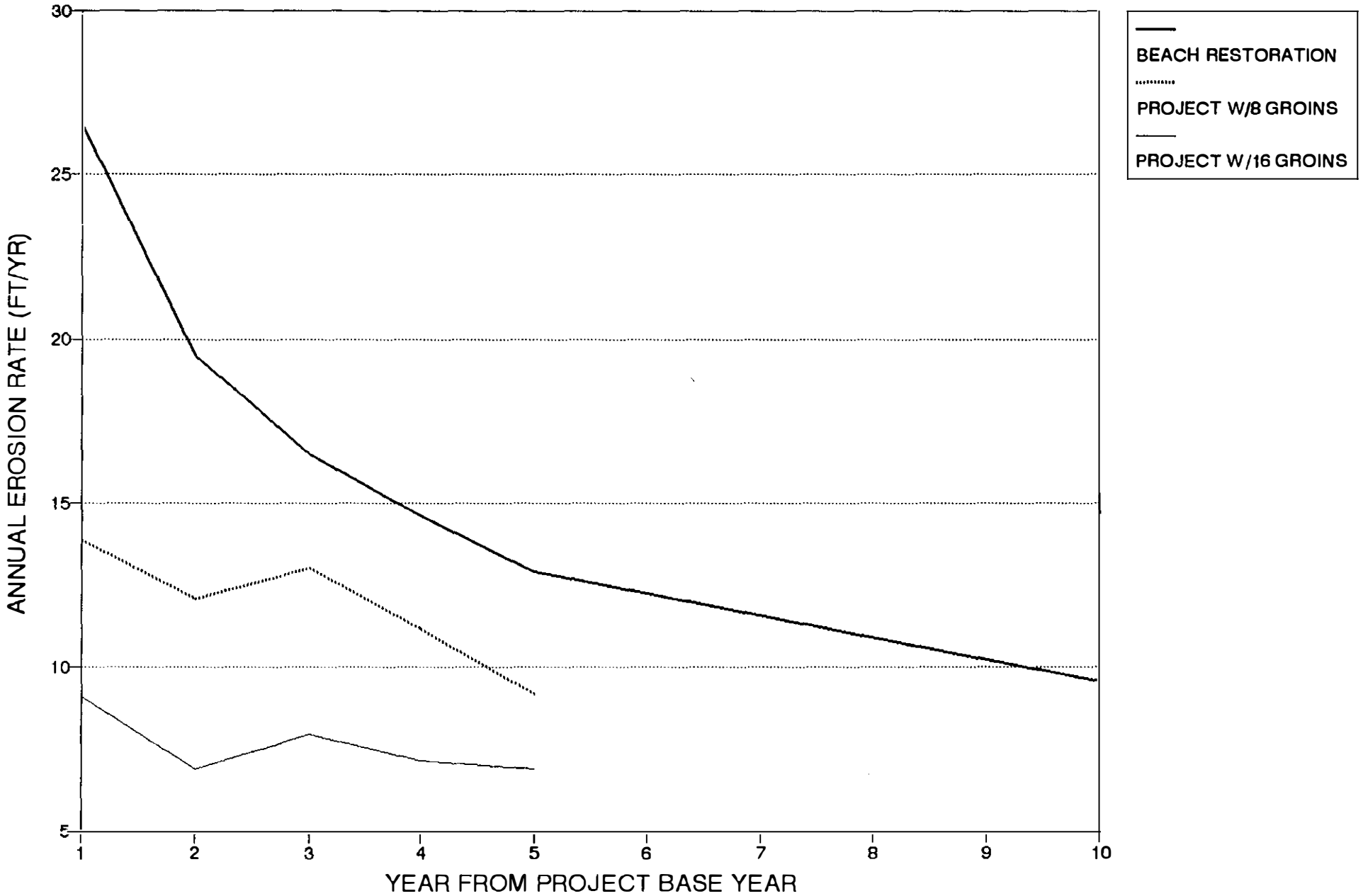


FIGURE 9. COMPARISON OF SHORELINES (Continued)

FIGURE 10. PREDICTED EROSION RATES
WITH PROJECT, BROADKILL BEACH



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It has been concluded that the GENESIS model can be a useful tool for evaluating shoreline change under various project conditions. The model calibration and simulation procedures are described in Appendix A.

To estimate the required nourishment quantities for various intervals due to long term shoreline change, the GENESIS model was used to predict the shoreline position out to 7 years from the project base year. The initial shoreline within the project limits is 100 feet from the project design line. The beach fill is tapered at both ends of the project to the predicted 2001 shoreline. As the duration of the nourishment cycle is increased, from 2 to 7 years, the incremental quantity predicted by the model plus sea level rise requirements lessens. This method of analysis will yield a result based upon the largest storm which occurred during the wave record used.

However, this analysis does not take into account the evident risk of a large storm or storms occurring between nourishments nor the risk of a higher wave energy year occurring outside of the representative envelope in the current modeling. These risks grow larger with every year the nourishment cycle is increased. At the present time, there is no generally accepted method to quantify this risk and apply it to the economic optimization of nourishment cycle, but common sense dictates that the increase in this risk will diminish returns as the nourishment interval is lengthened.

Nourishment Rates

In order to account for the inherent risk involved with extending replenishment intervals, beachfill losses due to storm-induced erosion for various nourishment cycle time periods are considered. It was assumed that the nourishment volume required for storm-induced erosion must be calculated to withstand the losses for the event which has a 50% chance of being exceeded during each nourishment cycle, from 2 to 7 years. To develop the total project nourishment volume required for each cycle, the volumetric losses from the three processes, losses due to long term erosion, losses due to the predicted rate of sea level rise, and losses due to storm induced erosion were combined. Table 14 presents the adopted nourishment estimates by interval for the selected beachfill design alternative.

Also investigated were two groin field alternatives. The first groin field alternative has 16 groins spaced at 720 feet and the second alternative has 8 groins with a spacing of 1440 feet. The groins project 200 feet from the design line.

Figure 10 presents the predicted annualized erosion rates for the beach fill and beach fill with groin alternatives. Table 14 presents the nourishment quantities for each of the alternatives. For the groin alternatives, the nourishment

volumes were smoothed and include quantities needed to restore erosion downdrift of the groin field. The GENESIS simulations for the groin alternatives were only carried out through year 5, erosion rates and nourishment quantities for year 6-7 for the groin alternatives were extrapolated.

TABLE 14. BROADKILL BEACH FEASIBILITY STUDY
TOTAL NOURISHMENT QUANTITIES (1000's CY)
PER NOURISHMENT CYCLE

ALT.	NOURISHMENT INTERVAL (years)					
	2	3	4	5	6	7
1	116	136	171	256	391	576
2	90	110	145	205	290	400
3	105	125	160	230	335	475

ALTERNATIVE 1: Beach Restoration/Dune Enhancement only
ALTERNATIVE 2: Beach Restoration/Dune Enhancement with 16 groins
ALTERNATIVE 3: Beach Restoration/Dune Enhancement with 8 groins

Major Rehabilitation

Major Rehabilitation quantities were developed in accordance with ER 1110-2-1407 to identify additional erosional losses from the project due to higher intensity (low frequency), storm events. The nourishment rates developed for the project alternatives include losses due to storms that have occurred within the analysis period; storms of an approximately 50 year return period and more frequent are encompassed in those rates. The major rehabilitation losses are computed as the losses that would occur from the 50% risk event over the project life. The annual percent frequency exceedence with a 50% risk of being exceeded during the 50 year economic project life is 1.37, which is approximately a 75 year event. The period of record of stages recorded at the study area is approximately 73 years, and the storm of record was the March 1962 northeaster. This storm was not only the stage of record, but also by far induced the greatest loss of beach material during the period. The 1962 northeaster was considered to be the 50% risk event for the purposes of the major rehabilitation analysis. SBEACH was employed to compute volumetric erosion from the selected beach for the 1962 northeaster. Waterlevels and waves were hindcasted at the study area for the storm, and all model parameters were identical to the without and with project analyses. The volumetric storm induced erosion was computed along each profile location for the design beach profile and than an average loss

quantity was computed for the entire study area. Based on methodologies and beachfill experience developed at the Philadelphia District, and throughout the Corps of Engineers, it has been estimated that between 60% and 75% of the material displaced during large storms will return to the foreshore within weeks and only the remaining 40% to 25% will require mechanical replacement. Therefore, as a conservative estimate of the necessary major rehabilitation quantity for the present study, a volume equal to 50% of the estimated volume removed from the subaerial beach will require mechanical placement onto the beach to regain the design cross-section and insure the level of protection.

It is estimated that a volume of 150,000 cubic yards within Broadkill Beach would be required to perform major rehabilitation in response to the 50% risk event over the economic project life.

Summary

The existing or without project condition within the project area provides a low level of protection against storm damage. The beachfill design alternatives will reduce storm damage by reducing profile recession, flooding incurred due to high levels of ocean storm water elevations, and wave impacts.

SECTION V
BROADKILL BEACH BEACHFILL MONITORING PLAN

The project monitoring plan will evaluate beach fill performance and determine conditions within the borrow areas. Periodic assessments will assist in determining renourishment quantities and availability of borrow material. The monitoring program will continue throughout the life of the project. The following items are to be included in the project monitoring plan:

BEACH PROFILES

PURPOSE: Quantify loss rates from project cells in order to define required renourishment quantities and determine the accuracy of predicted loss rates, and document cross-shore and longshore transport patterns of the beach fill.

FREQUENCY: Collect detailed profiles immediately before and after construction.
Construction is to mean initial placement and each nourishment.
Quarterly in March, June, September, and December during the year following construction, then Semiannually in March and September during non-construction years.

NUMBER AND LOCATION: All beach profile surveys shall be taken along the established project reference baseline established during the present feasibility study. The surveys should extend across the entire zone of active profile change. The subaerial portion of the surveys should begin landward of the primary dune at a point where no erosion is expected. The offshore portion should extend seaward beyond the depth of closure.

The before and after construction surveys within the project area will be collected as part of construction and their amount included as part of the construction costs.

The Quarterly and Semiannual, on/offshore surveys include a total of 25 lines previously occupied during the feasibility study, 21 throughout Broadkill Beach, 2 south, and 2 north of the project area.

Surveys will also be performed after major storms at the 25 lines throughout the project area. A contingency is included for the storm profiling in the monitoring costs.

Aerial Photography

PURPOSE: Document shoreline changes along Broadkill Beach as supplement to beach profile data.

FREQUENCY: The aerial photography will be collected within 30 days prior to each construction, (initial and nourishment), and within 10 days after each construction. During the first 3 years of the project photographs shall be taken annually in September. After the first three years, photographs shall be taken in September of the year following each nourishment and every other year thereafter up to the next nourishment.

Tidal Data

Lewes, Breakwater Harbor NOS gage is available; no additional gage will be placed.

Sediment Sampling

PURPOSE: Identify sediment resorting and fill behavior; identify cross-shore and longshore grain size distribution changes; evaluate fill factor method.

FREQUENCY: Collect samples semiannually, March and September, every other year in conjunction with planned beach profile surveys.

NUMBER AND LOCATION: Along 14 established lines only; sample at dune, berm, high tide, mid tide, low tide lines, then 6 ft intervals to 30 ft (total of 10 samples per profile line). Additionally, within borrow area at 25 sample locations.

Section VI Bibliographies and References

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PHASE II REPORT

BROADKILL BEACH
EROSION AND FLOODING PROTECTION

APPENDIX A
SHORELINE MODELING - GENESIS

APPENDIX A
SHORELINE MODELING - GENESIS

The numerical model GENESIS (Generalized Model for Simulating Shoreline Change) was developed by USACE-CERC (Hanson and Kraus, 1989; Gravens et al., 1991). It is designed to simulate long-term shoreline changes as a function of nearshore wave climate, beach profile, and the presence of coastal structures such as groins, breakwaters, seawalls, and beach fills.

The basis of the GENESIS computations are detailed in Hanson and Kraus (1989). Gravens et al. (1991) provides a detailed description of the input files necessary to run GENESIS. Gravens (1992) describes the programs available from CERC which assist in file preparation and results interpretation.

GENESIS uses a finite set of wave height and direction transformation coefficients applied to a time series of offshore wave data to determine breaking wave conditions alongshore at each time step. The nearshore wave characteristics determined using RCPWAVE are linked to the offshore time series in the same way as was used for the P_{ls} estimates. GENESIS transforms the nearshore waves to breaking using linear theory and a breaking criteria.

Alternatively, an offshore wave time series can be used and GENESIS will use an internal wave transformation model to determine braking characteristic. To do this the model assumes a monotonically decreasing profile slope from the breaking point to deep water with a profile related to the user-defined median sediment size.

These breaking wave conditions are used at each GENESIS cell to determine the longshore transport rate at each time step. These transport rates drive the shoreline change calculations, with the assumption that the profile shape remains constant. The empirical equation used to calculate the transport rate is similar to the equation for longshore transport rate (Q_l) in Phase I, with the addition of a term to account for the longshore variation of H_b and variable coefficients used in the calibration and verification processes.

The GENESIS model was used in connection with the RCPWAVE model in the Broadkill Beach Erosion and Flooding Protection Phase I study (WRCE, 1995). The RCPWAVE modeling study resulted in unrealistic transport rates, which were reflected in the GENESIS modeling. In the Phase II study, it was decided to use the GENESIS internal wave transformation capability.

A shore-perpendicular grid was established to reference shoreline positions and coastal structures. The grid consists of 150 cells, each cell having a width of 180 ft. The model cells extend beyond the project area and include approximately 5 miles of shoreline. The study area is centered in the GENESIS grid. Figure A-1 shows the GENESIS computation cells at Broadkill Beach.

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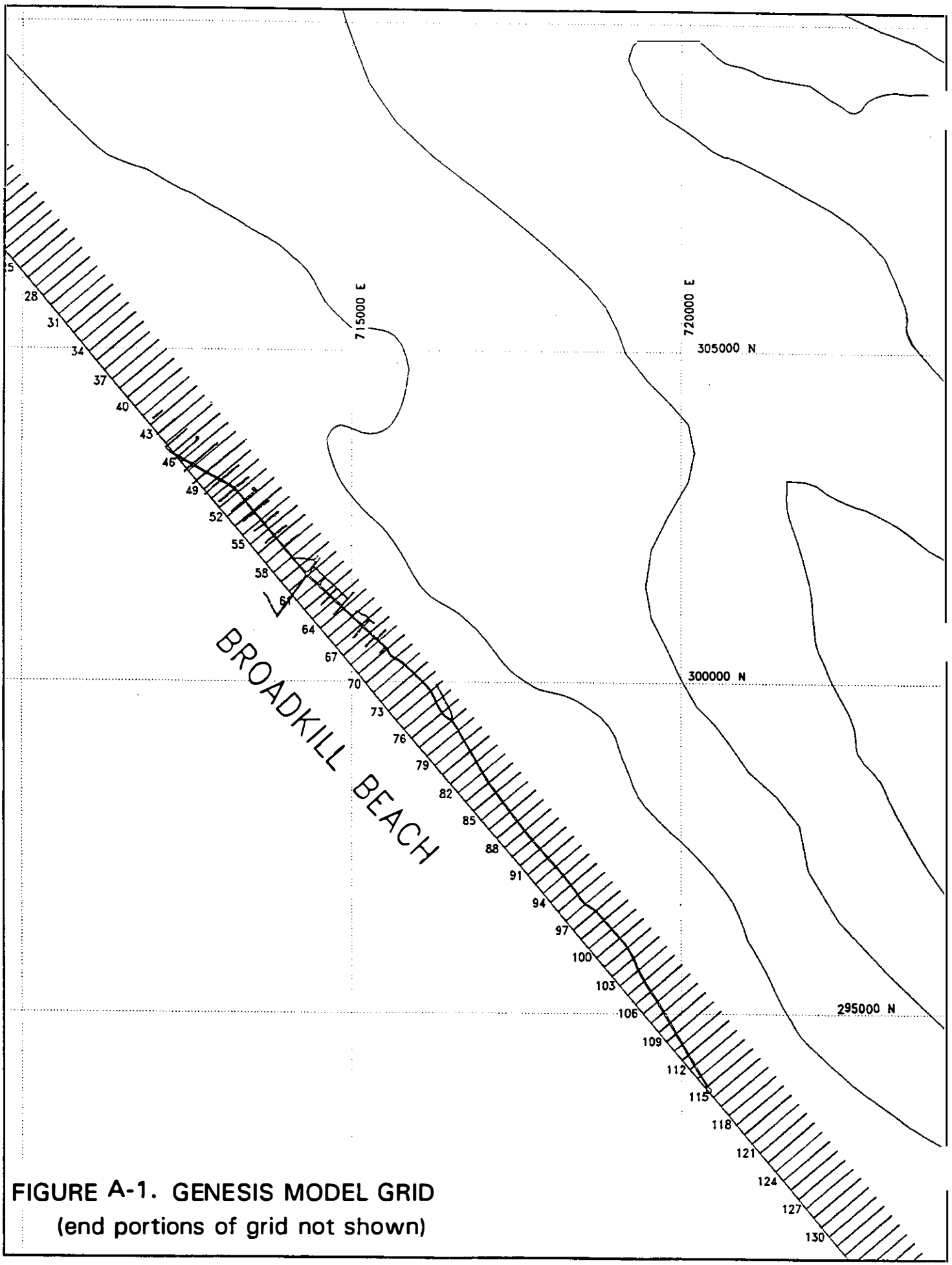

FIGURE A-1. GENESIS MODEL GRID
(end portions of grid not shown)

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Model calibration and verification are crucial for an accurate prediction of future shoreline positions. Calibration is the process of initially adjusting the transport rate coefficients so that the calculated shoreline position is in good agreement with the measured shoreline position at the end of the simulation period. The model is verified when it continues to accurately predict other known shoreline positions for different time periods. Historic shoreline positions were established in Phase I of the study for 1954, 1964, 1971, 1977, 1979, 1980, 1989, 1990, and 1993. The Delaware Bay hindcast wave data is available between 1987 and 1993. For this reason, 1989-1993 was chosen as the calibration period.

The hindcast wave time series at the station located at 38°50' N and 75°10' W from the Delaware Bay wave study was used. The water depth at this location is approximately 15 ft MLLW. The time series was edited to correspond to the calibration period. When the wave period in the hindcast data was too small, the GENESIS model would quit. Therefore the wave data file was edited to set wave periods less than 2 seconds equal to 2.

GENESIS uses a START file to adjust global input parameters governing the transport rate, boundary conditions, time step, and the inclusion of groins, seawalls, breakwaters, and beach fills. A representative profile shape was chosen based on the available profile data. A depth of closure of -5 ft NGVD and an average berm elevation of +8 ft NGVD were chosen for all model runs.

Although the use of the internal wave transformation calculations yielded better results than did the RCPWAVE-generated nearshore characteristics, most of the variation in the known alongshore erosion rates was lost. Using the internal model resulted in smoothed shoreline change. This results in the inability to closely reproduce the known shoreline (1993 for calibration) for any combination of input parameters. Therefore, several different runs were conducted with variable transport rate coefficients and boundary conditions to document this effect. The input parameters yielding the best results for the calibration period are listed in Table A-1.

Table A-1. Summary of calibration input parameters.

Parameter	Value
Period	1989-1993
K_1	0.75
K_2	0.5
Time step	3 hrs
left B.C.	pinned
right B.C.	pinned

The known 1989 and 1993 shorelines along with the calculated 1993 shoreline are plotted in Figure A-2. The model produced good results in the northern and central portion of the study area. However, it predicted unrealistic accretion at both ends of the study area. Considering the uncertainty of hindcast wave data, the use of an internal wave transformation and the use of a computer program to model highly complexed shoreline change process, it is concluded that GENESIS is applicable for the project area.

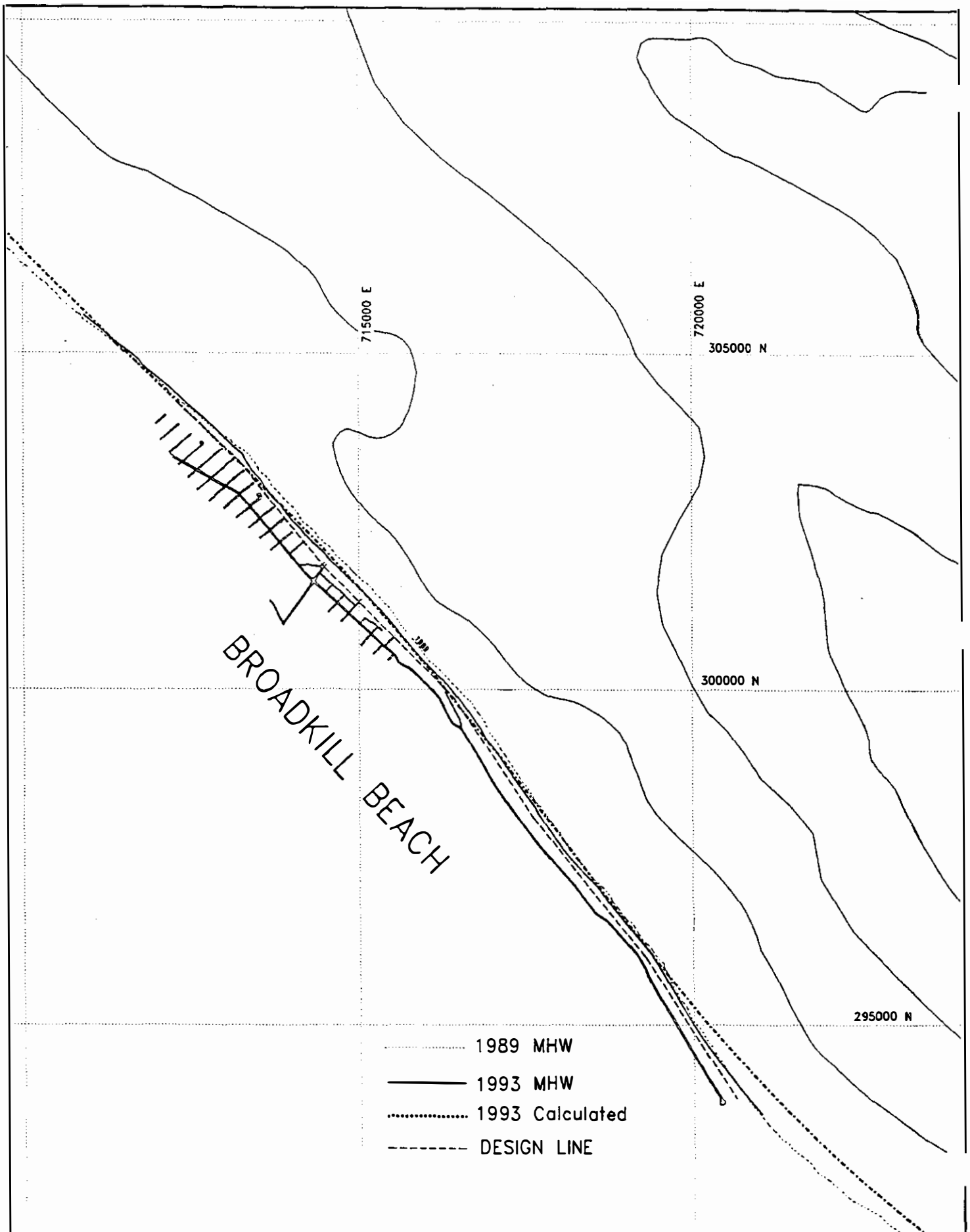

FIGURE A-2. GENESIS MODEL CALIBRATION 1989-1993

There was no other period for which both wave data and shoreline position were available for verification. An alternative solution to this problem was to generate simulated wave data for periods of interest and use the GENESIS model to predict long-term shoreline changes. The computer program HPDSIM developed at the Waterways Experiment Station (Borgman and Scheffner, 1991) was used to develop a 10-year synthetic time series file using the Delaware Bay hindcast wave data as input. The synthetic wave data then used by GENESIS to compute the shoreline for the project base year (2001). The annualized erosion computed by GENESIS for 1993-2001 is 10.5 ft/year, the same as determined by profile analysis from the Phase I study. The model was further run for another 10 years. Figure A-3 shows the annual erosion rates by year, after year 2001. The annual erosion rate computed by using the GENESIS model decreases as the shoreline retreats. The simulation assumes that there will be no beach nourishment. It should be noted that the background rate determined by profile analysis was based on the shoreline positions maintained by local interests.

Beyond 2001, GENESIS continues to predict accretion at the southern Broadkill area and beyond. This may be the result of the flattening out of the regional shoreline bulge centered on Broadkill due to the erosion during the previous 1993-2001 period. The 2001 shoreline is oriented so that the nearshore waves break at a smaller angle to the shoreline thereby reducing the rate of longshore transport. In this way the shoreline is self-stabilizing, although the extent of erosion required to reach this condition is likely to be intolerable.

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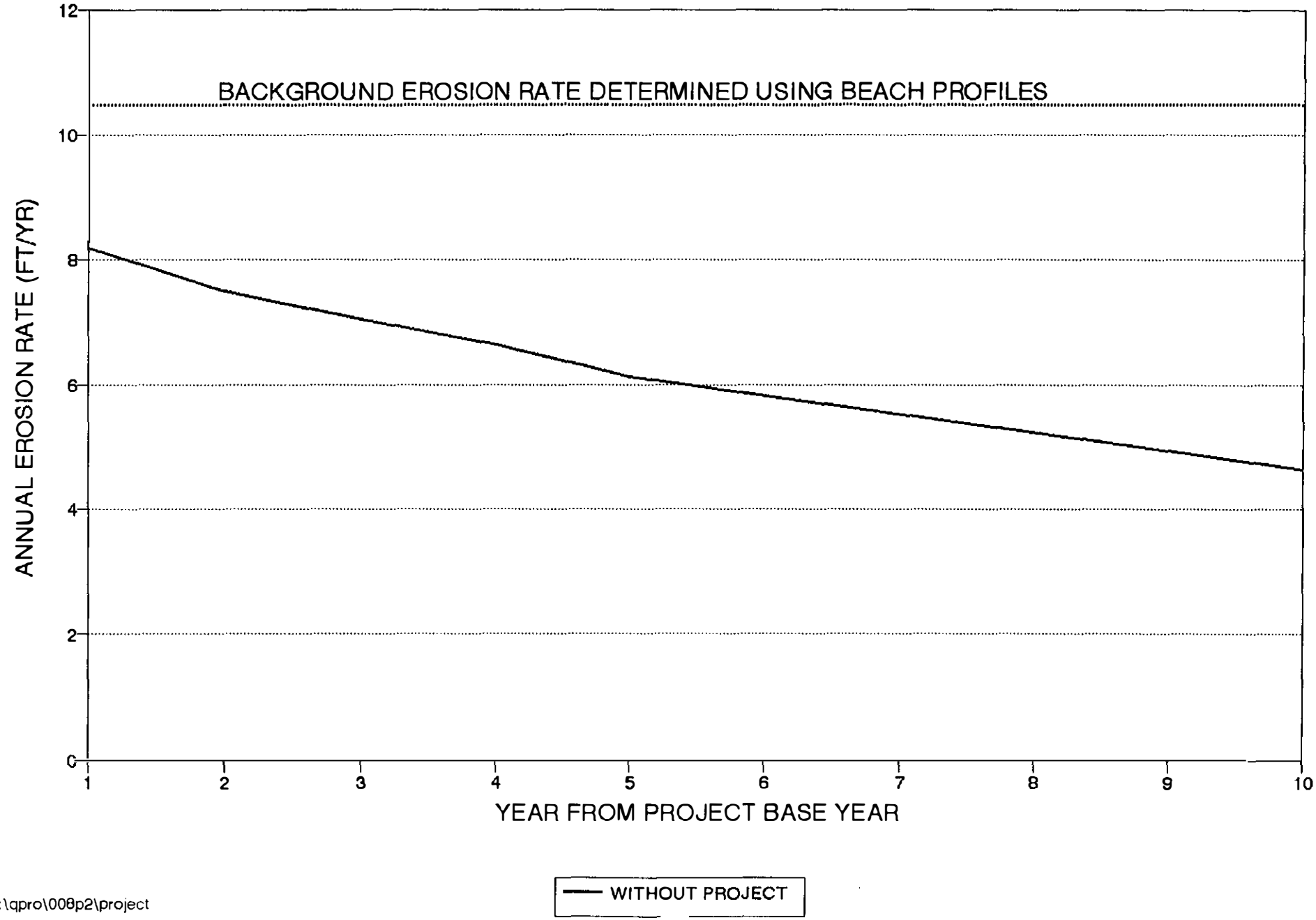
FIGURE A-3 PREDICED EROSION RATES
WITHOUT PROJECT, BROADKILL BEACH



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PHASE II REPORT

BROADKILL BEACH
EROSION AND FLOODING PROTECTION

APPENDIX B
CONCEPTUAL DESIGN
LOW-CREST OFFSHORE BREAKWATER

APPENDIX B
CONCEPTUAL DESIGN
LOW-CREST OFFSHORE BREAKWATER

Based on the long-term shoreline change trends, the shoreline reaches that require protection are between LRP#25 and LRP#27A. Due to the long project length, a segmented breakwater system is recommended. To reduce the chance of forming tombolos, a length to distance ratio near 1.0 is recommended. For a breakwater segment length of 700 feet with a spacing of 300 feet, a total of 12 breakwater segments are required to protect the designated area. The breakwaters should be located approximately 600 feet offshore where the depth varies from -5.5 to -8 feet NGVD. The exposure ratio of this system is 0.28. Figure B-1 shows the breakwater alignment. Table B-1 summarizes the breakwater design for a 50-year design storm.

Figure B-2 presents a typical cross section of the breakwater. This cross section is provide for the purpose of preliminary cost estimated only. Bottom sediments should be sampled to provide data for a detailed foundation design to minimize settlement. Armor stones should be a rough, angular granite with a minimum specific gravity of 2.6.

Table B-1
Specifications for Offshore Breakwater at Broadkill Beach

Length of Project Area	10,400 feet
Crest Elevation	+3.0 NGVD
Segment Length	700 feet
Segment Spacing	300 feet
Number of Segments	12
Distance from Shoreline	600 feet
Design Water Depth	-7.5 feet NGVD
Design Water Level	9.5 feet
Design Wave Height	11.6 feet
Type of Construction	Rubble Mound
Weight of Armor Stone/Toe Apron	8.2 tons/1.5 tons

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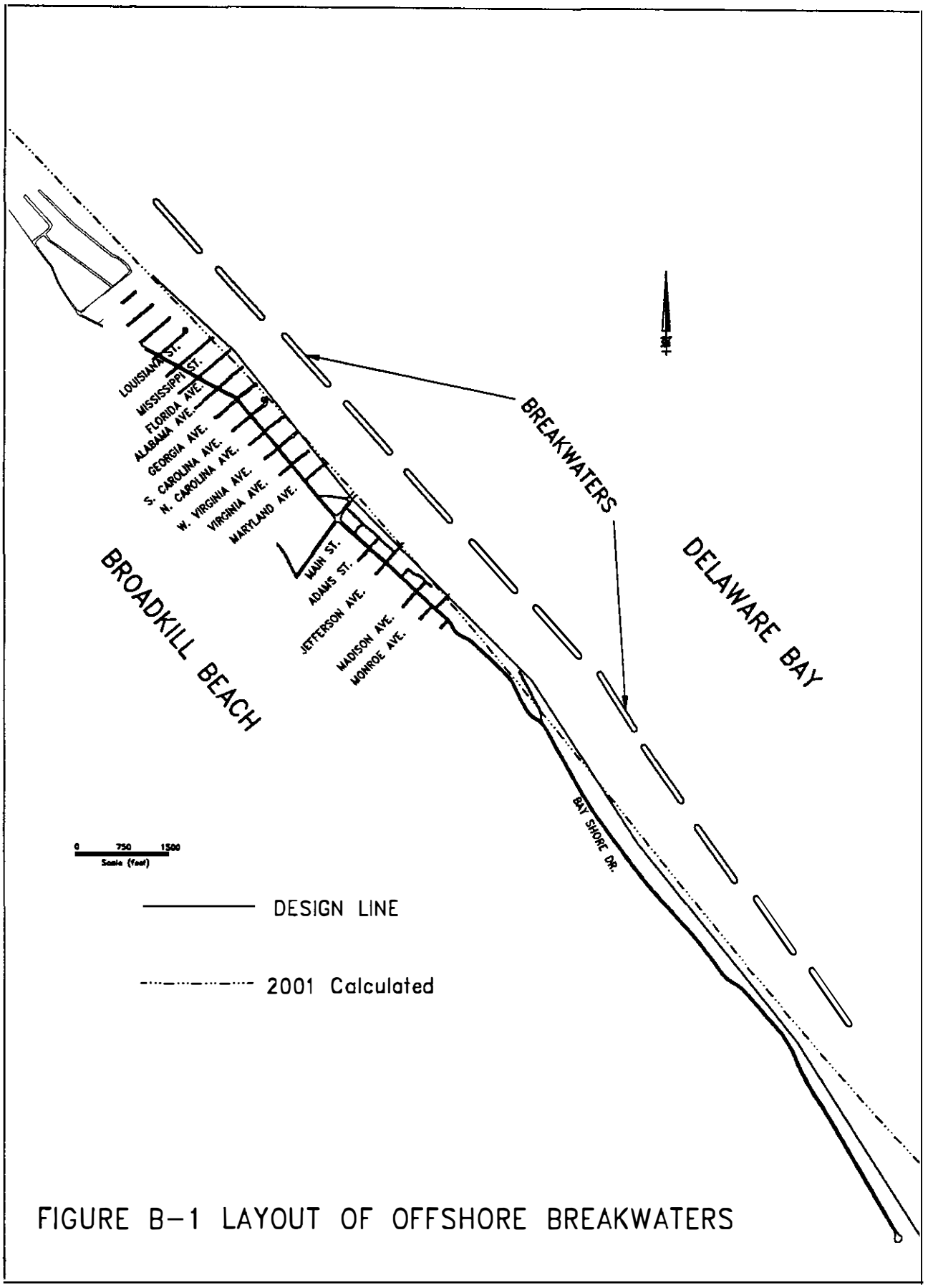
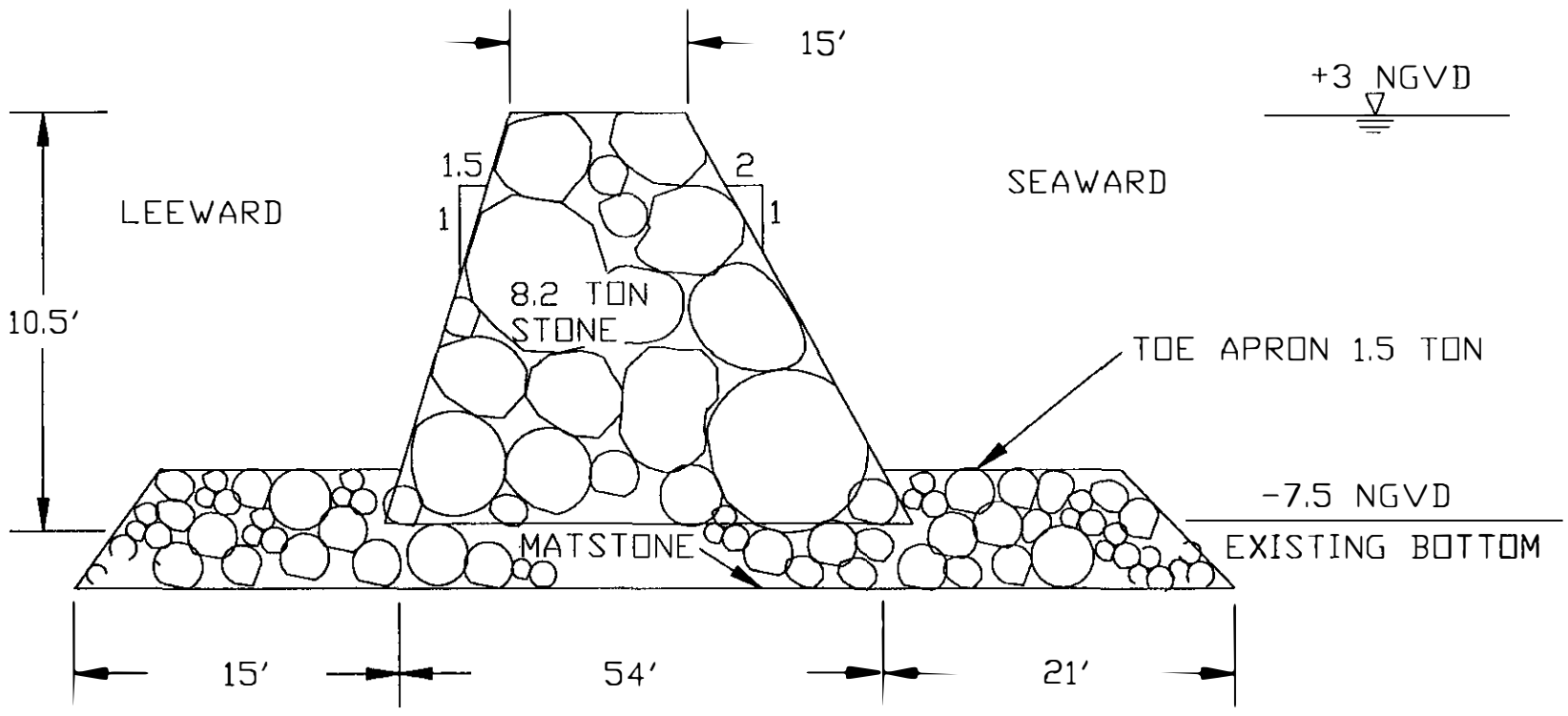


FIGURE B-1 LAYOUT OF OFFSHORE BREAKWATERS

FIGURE B-2
TYPICAL BREAKWATER SECTION



PHASE II REPORT

BROADKILL BEACH
EROSION AND FLOODING PROTECTION

APPENDIX C
CONCEPTUAL DESIGN
GROIN FILED

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APPENDIX C
CONCEPTUAL DESIGN
GROIN FIELD

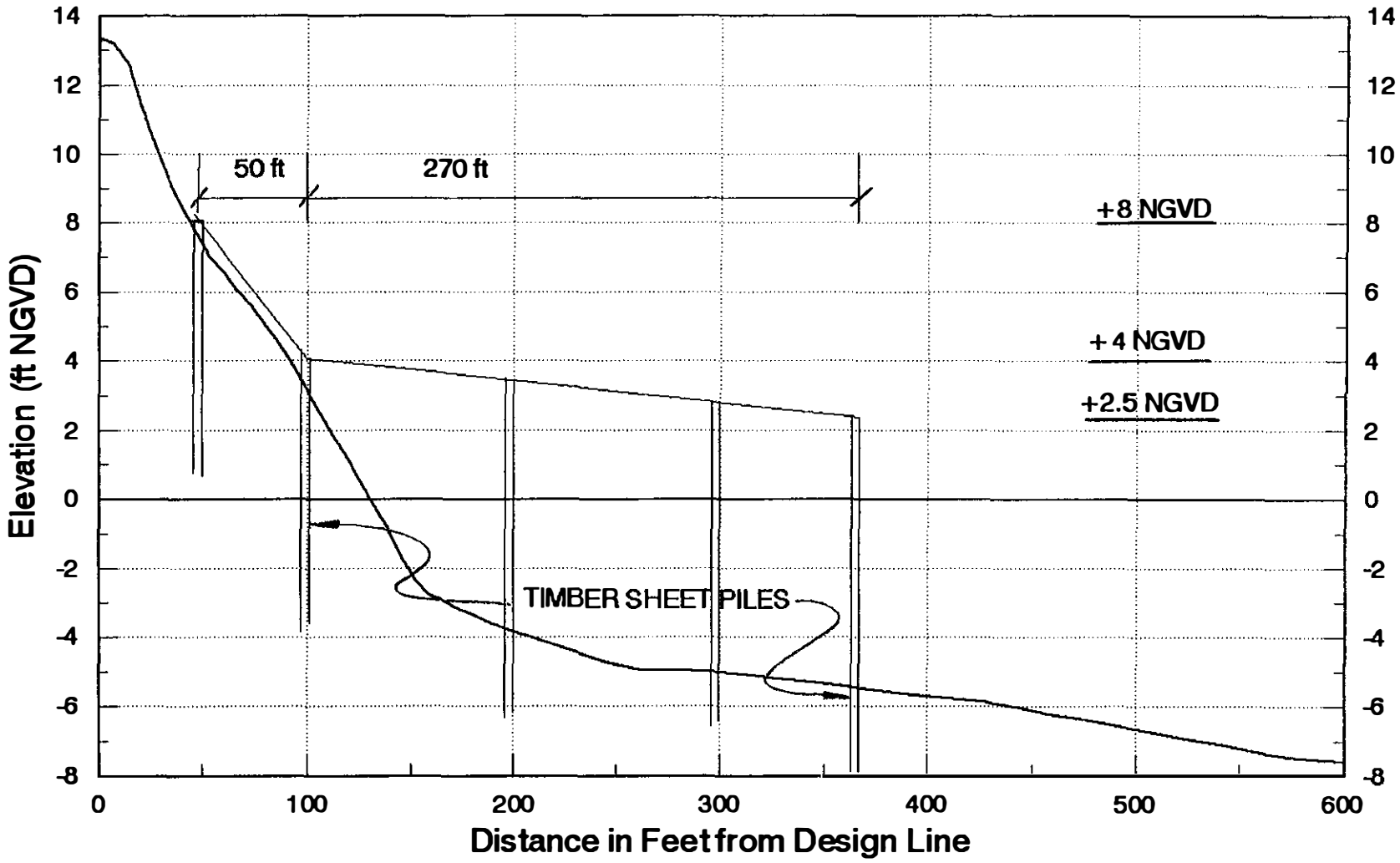
Based on available guidance for groin field performance in stabilizing shorelines, a spacing to length ratio of 2.5 is recommended. The length is the distance from MSL to the seaward end of the groin. The spacing is the alongshore distance between adjacent groins. Using a groin length from MSL of 250 feet yields a groin spacing of 625 feet. In order to place the groins properly in the GENESIS grid, the spacing is adjusted to 720 feet. The groins will tie into the beach berms at +8 NGVD and slope to +4.0 NGVD at the existing MSL location. From the MSL, the groins slope from +4 to +2.5 NGVD. Using the existing beach profile of LRP 25A, the groin profile is as shown in Figure C-1. Figure C-2 shows the layout of the groin field. Table C-1 summarizes the design data for the groin field.

Table C-1
Specifications for Groin Field at Broadkill Beach

Length of Project Area	15,000 ft
Type of Construction	Timber Sheet-Pile
Cross Shore Length for Each Groin	320 feet
Depth at End of Groin	vary from -4 to -6.5 NGVD
Groin Spacing	Alternative 1: 720 feet Alternative 2: 1440 feet
Number of Cells Required to Protect the Project Area	Alternative 1 : 21 Alternative 2: 11

The GENESIS model developed for the study area was used to evaluate the performance of the groin field. The initial shoreline used for simulation was the calculated 2001 position from the GENESIS model. Figure C-2 shows the shoreline position 10 years after the project. The severe erosion at the center of the project area is considered unrealistic. A second groin alternative was also tested with a groin spacing of 1440 feet. The model becomes unstable and stops after 5 years of simulations. Figure C-3 shows the shoreline at the end of 5 years for 1440 feet groin spacing. The model results suggest that the groin alternative can be used to promote localized stability, but at the expense of adjacent shorelines.

FIGURE C-1 TYPICAL GROIN PROFILE
BROADKILL BEACH, DELAWARE



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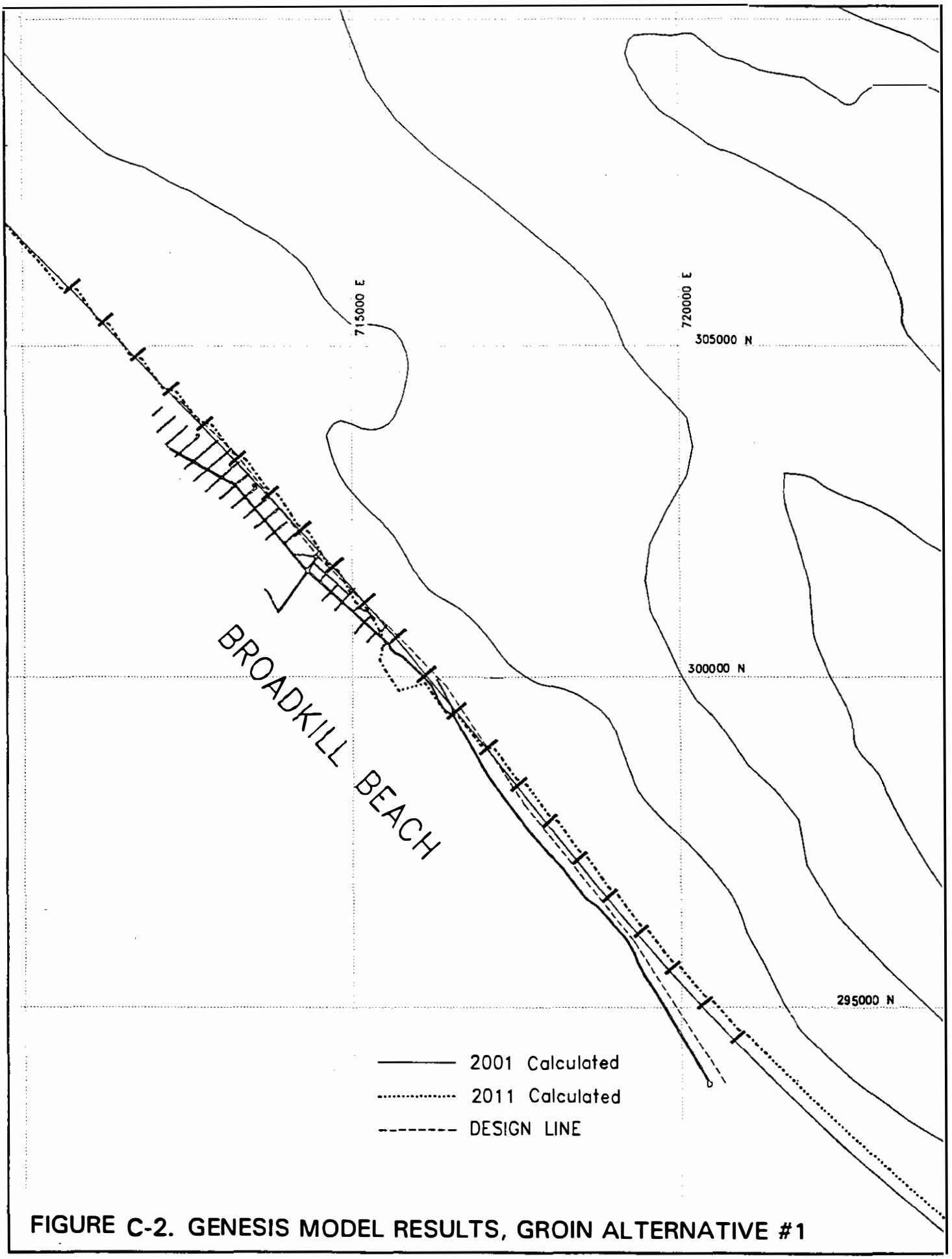

FIGURE C-2. GENESIS MODEL RESULTS, GROIN ALTERNATIVE #1

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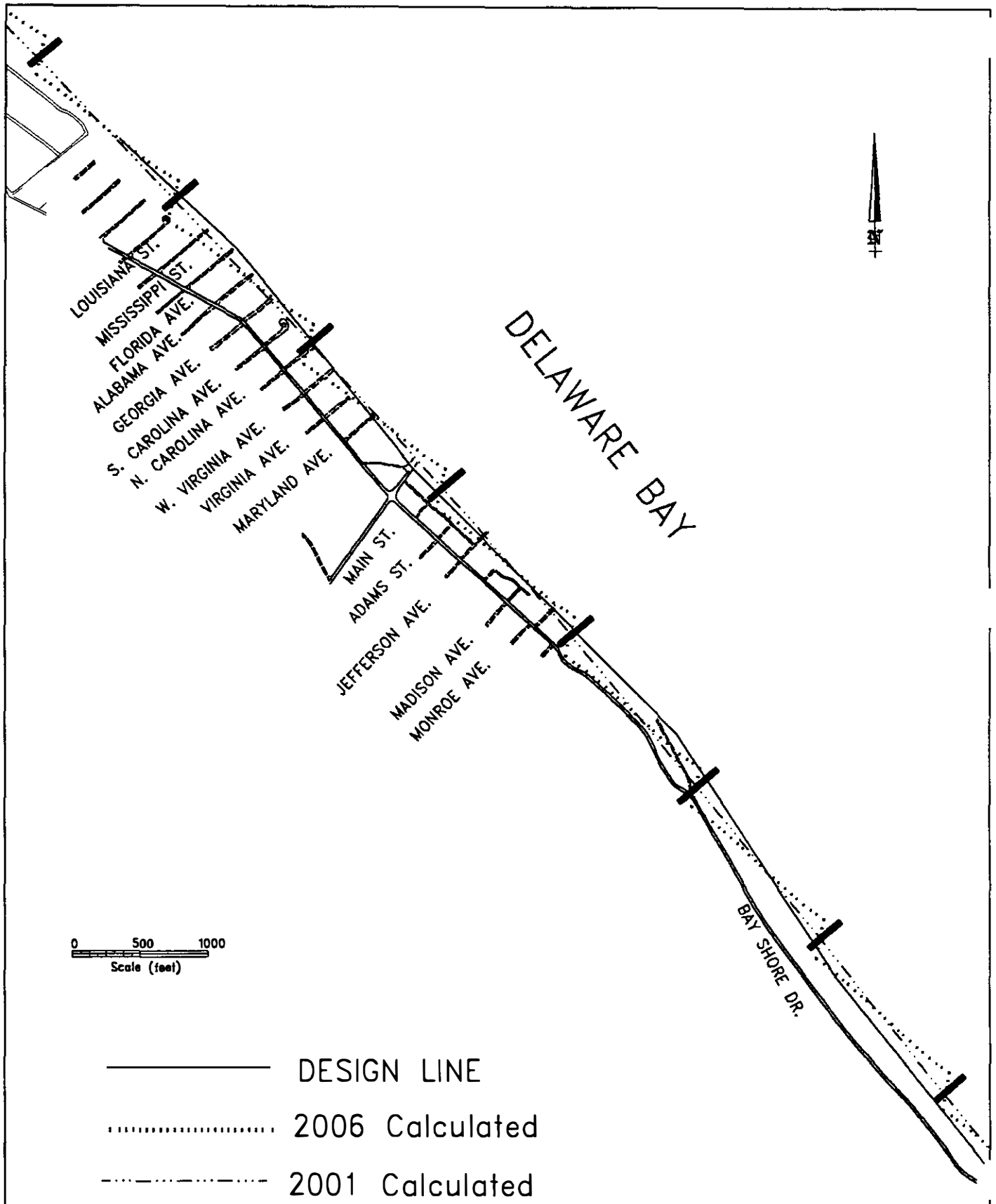

FIGURE C-3 ESTIMATED SHORELINE POSITION, GROIN ALTERNATIVE 2

PHASE II REPORT

BROADKILL BEACH
EROSION AND FLOODING PROTECTION

APPENDIX D
BEACH PROFILES WITH STORM INDUCED EROSION, WITH PROJECT

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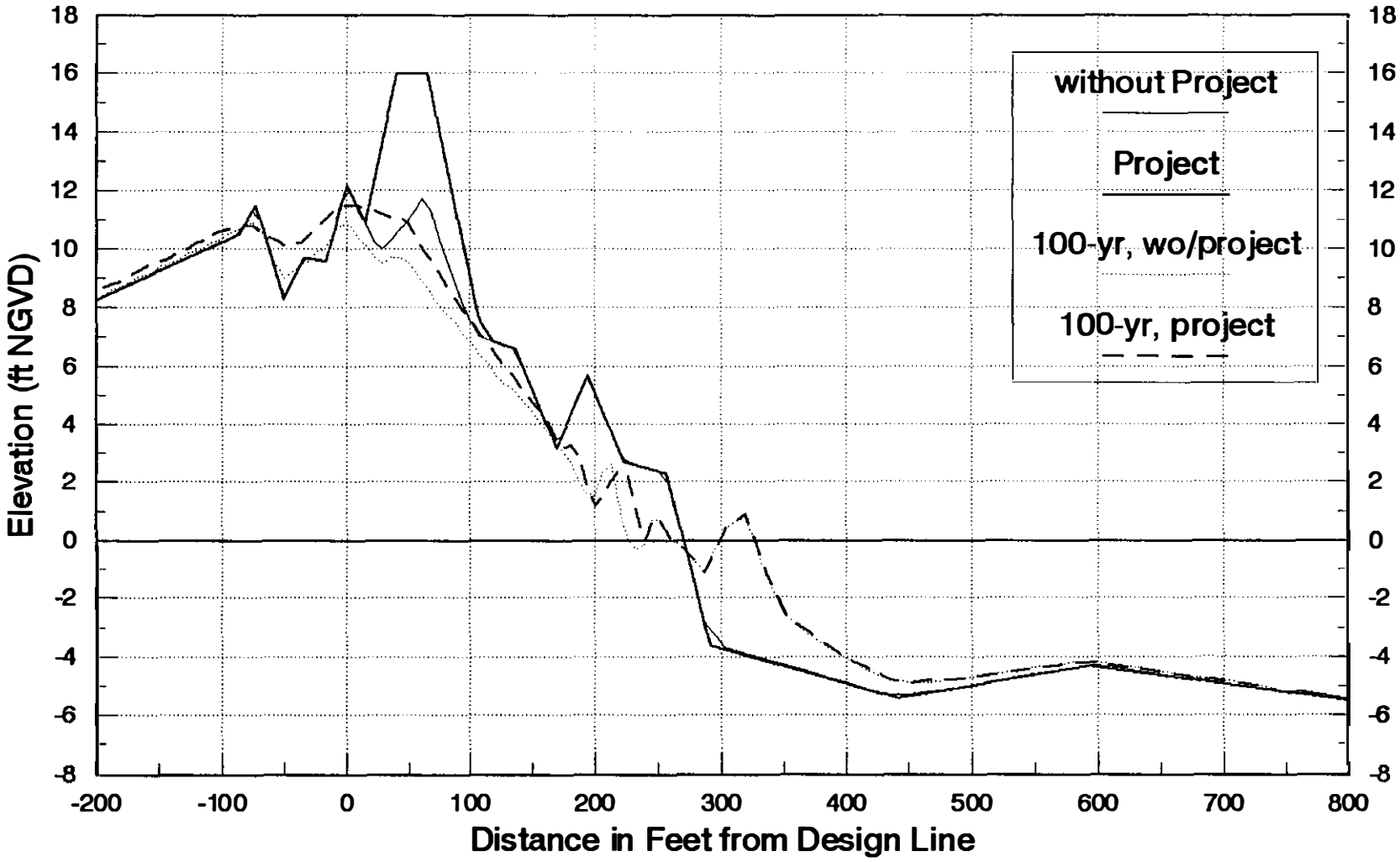
BEACH PROFILES, LRP#24A
BROADKILL BEACH, DELAWARE



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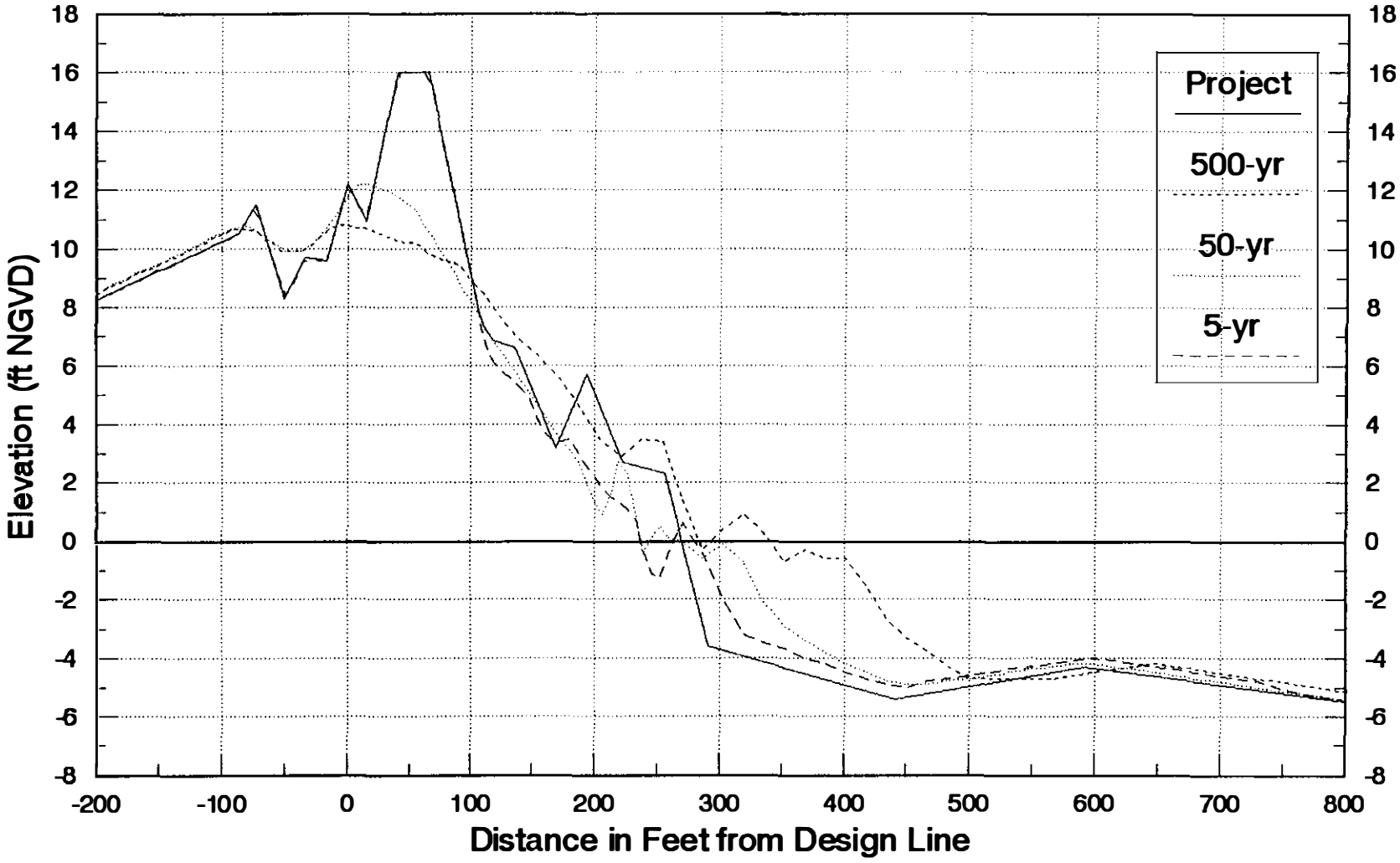
BEACH PROFILES, LRP#24A
BROADKILL BEACH, DELAWARE



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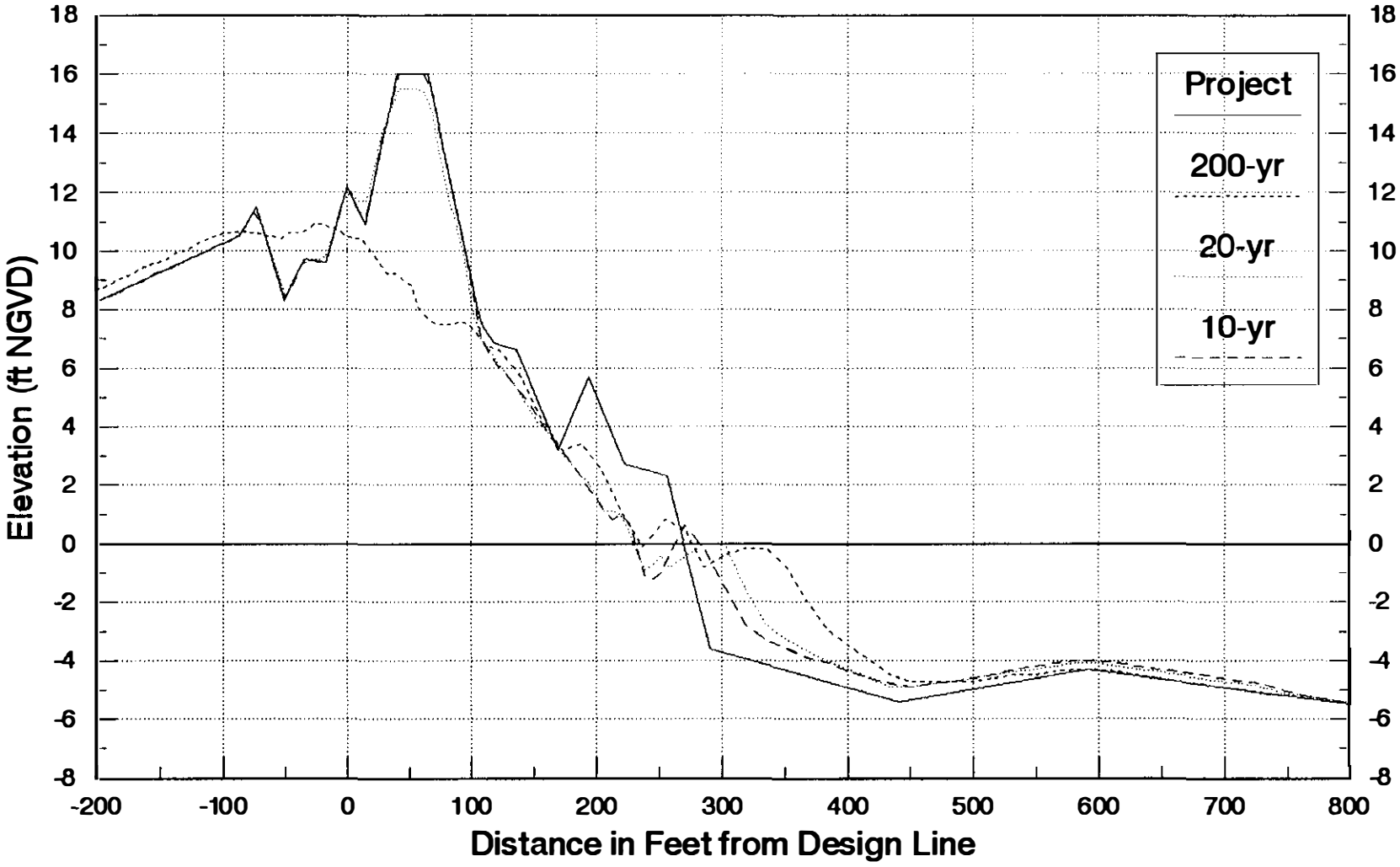
BEACH PROFILES, LRP#24A
BROADKILL BEACH, DELAWARE



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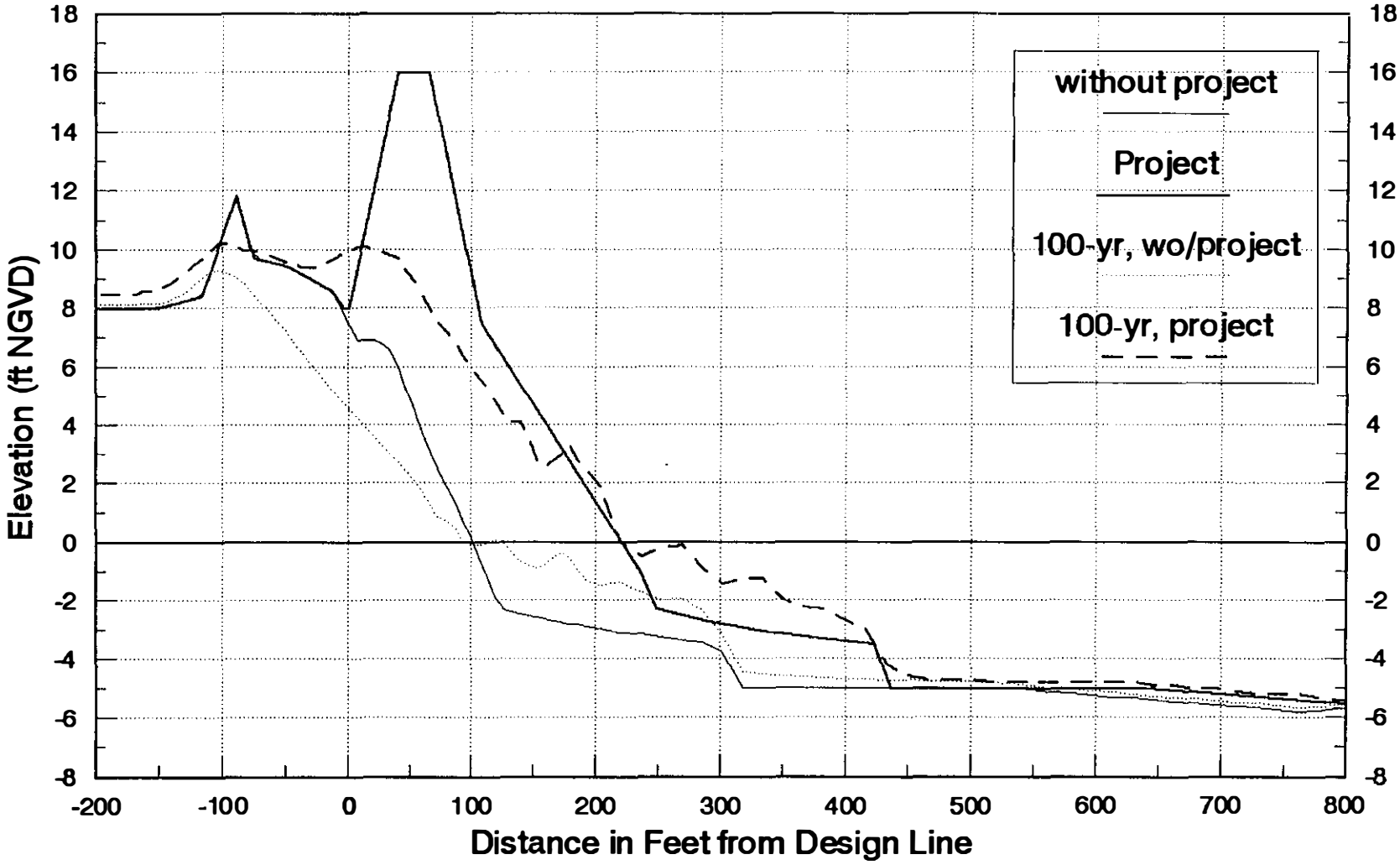
BEACH PROFILES, N33+00
BROADKILL BEACH, DELAWARE



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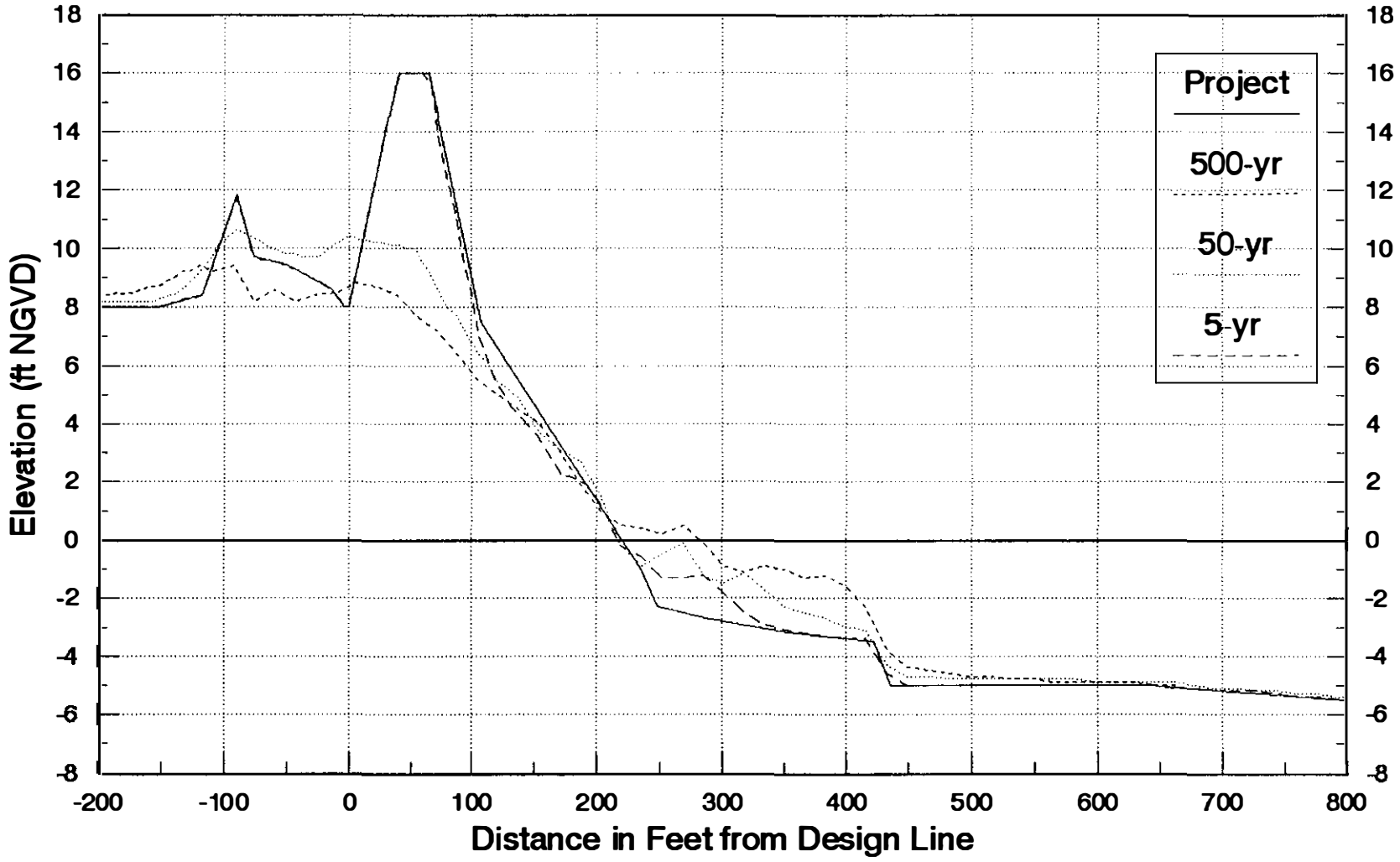
BEACH PROFILES, N33+00
BROADKILL BEACH, DELAWARE



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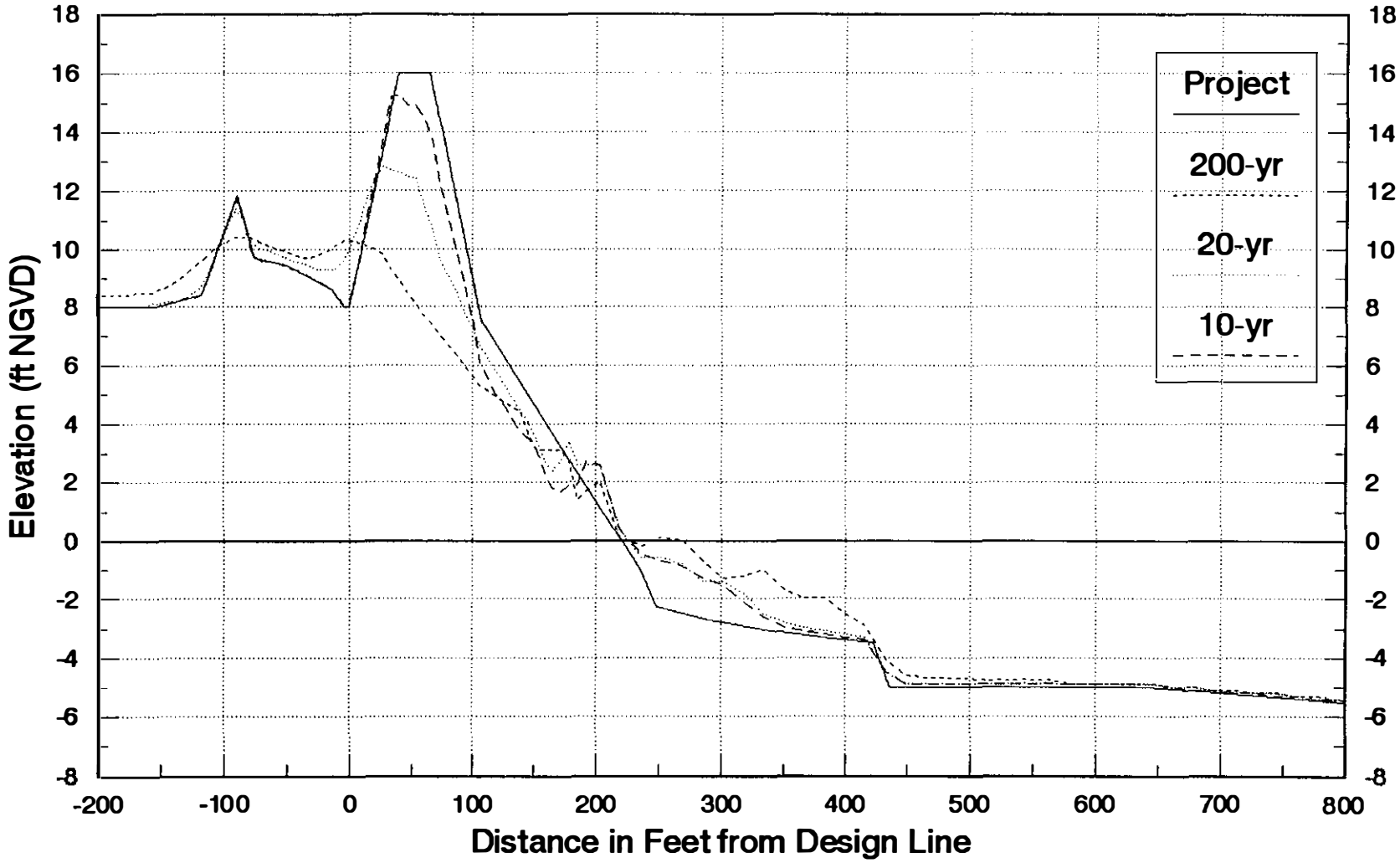
BEACH PROFILES, N33+00
BROADKILL BEACH, DELAWARE



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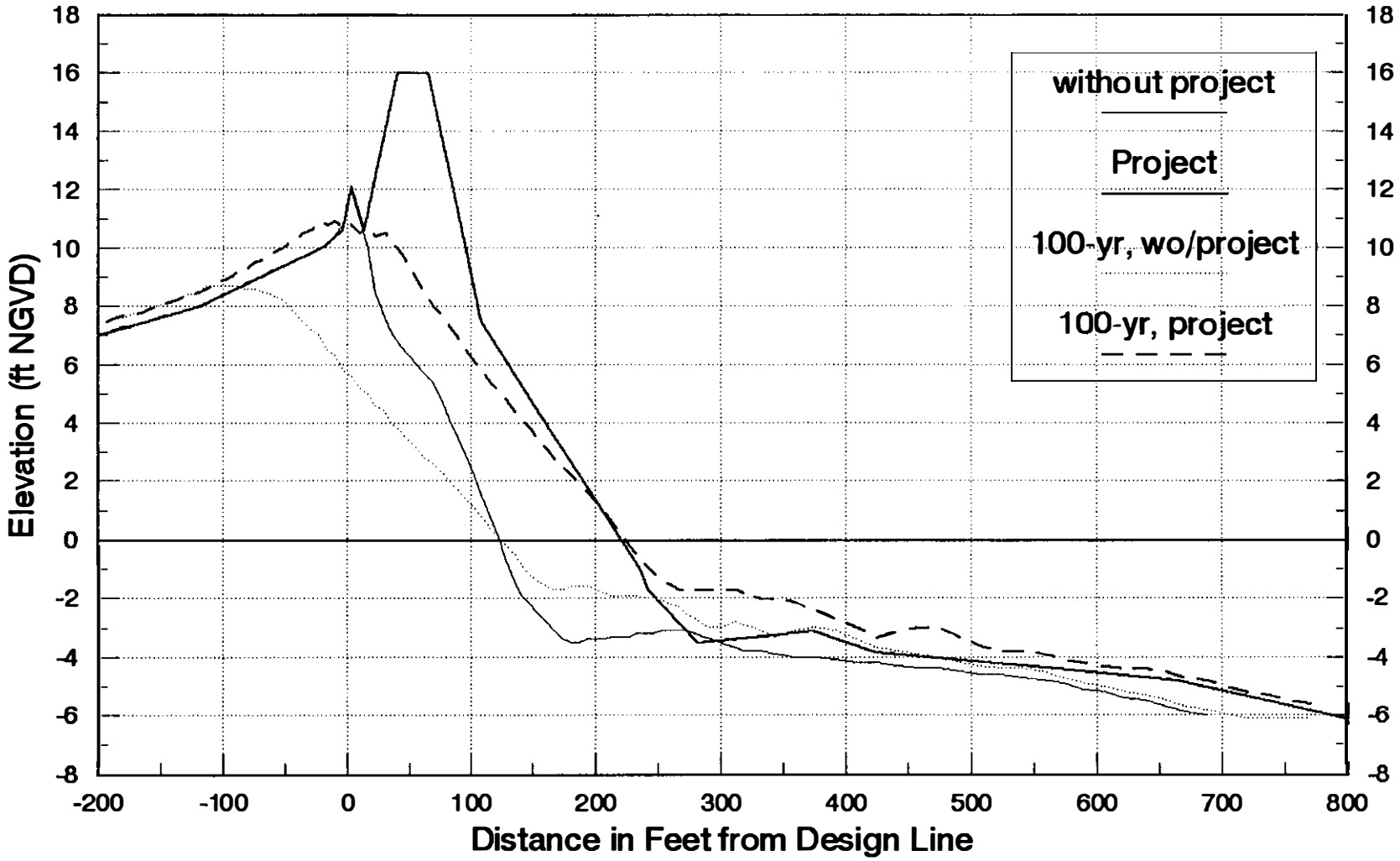
BEACH PROFILES, LRP#25
BROADKILL BEACH, DELAWARE



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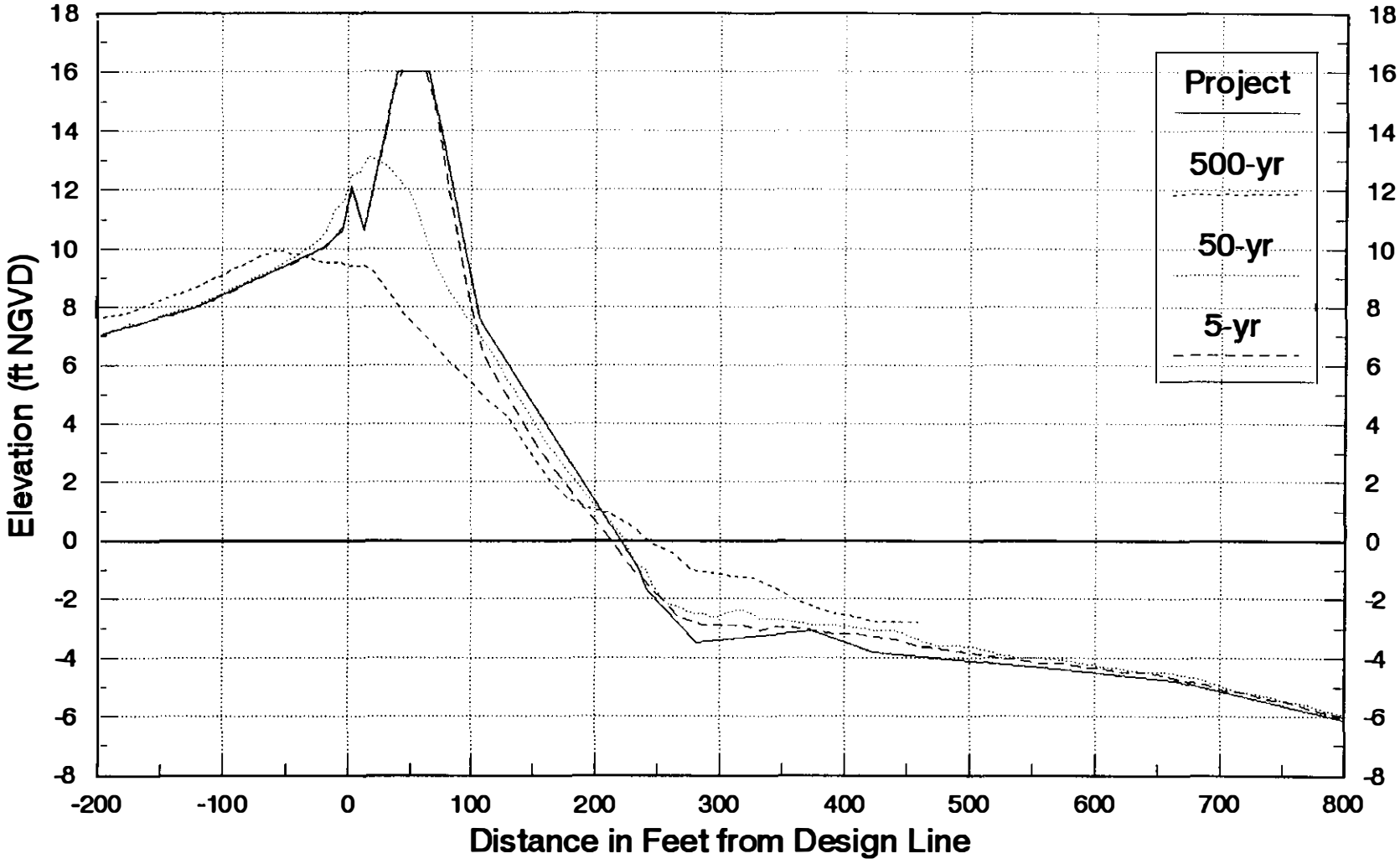
BEACH PROFILES, LRP#25
BROADKILL BEACH, DELAWARE



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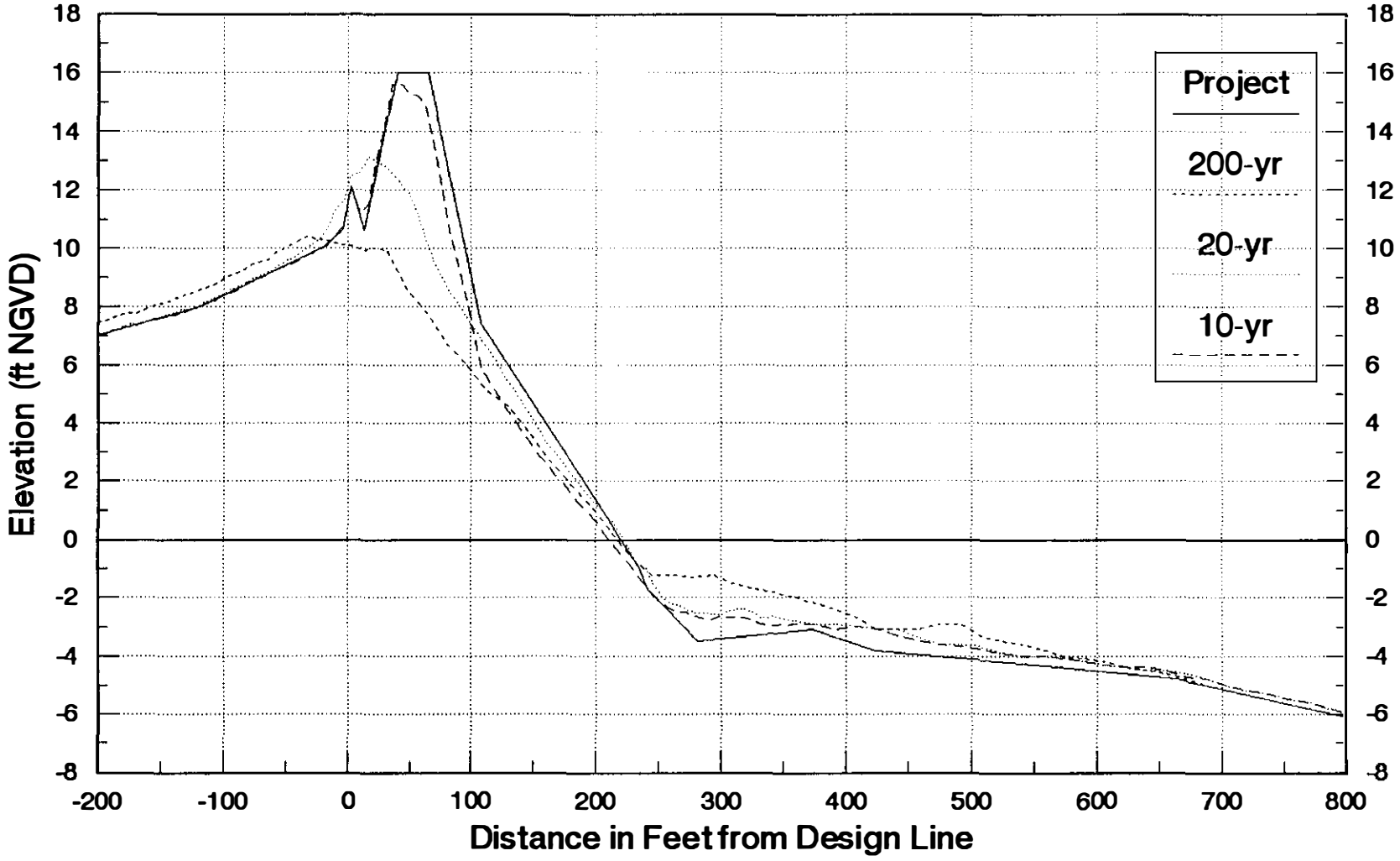
BEACH PROFILES, LRP#25
BROADKILL BEACH, DELAWARE



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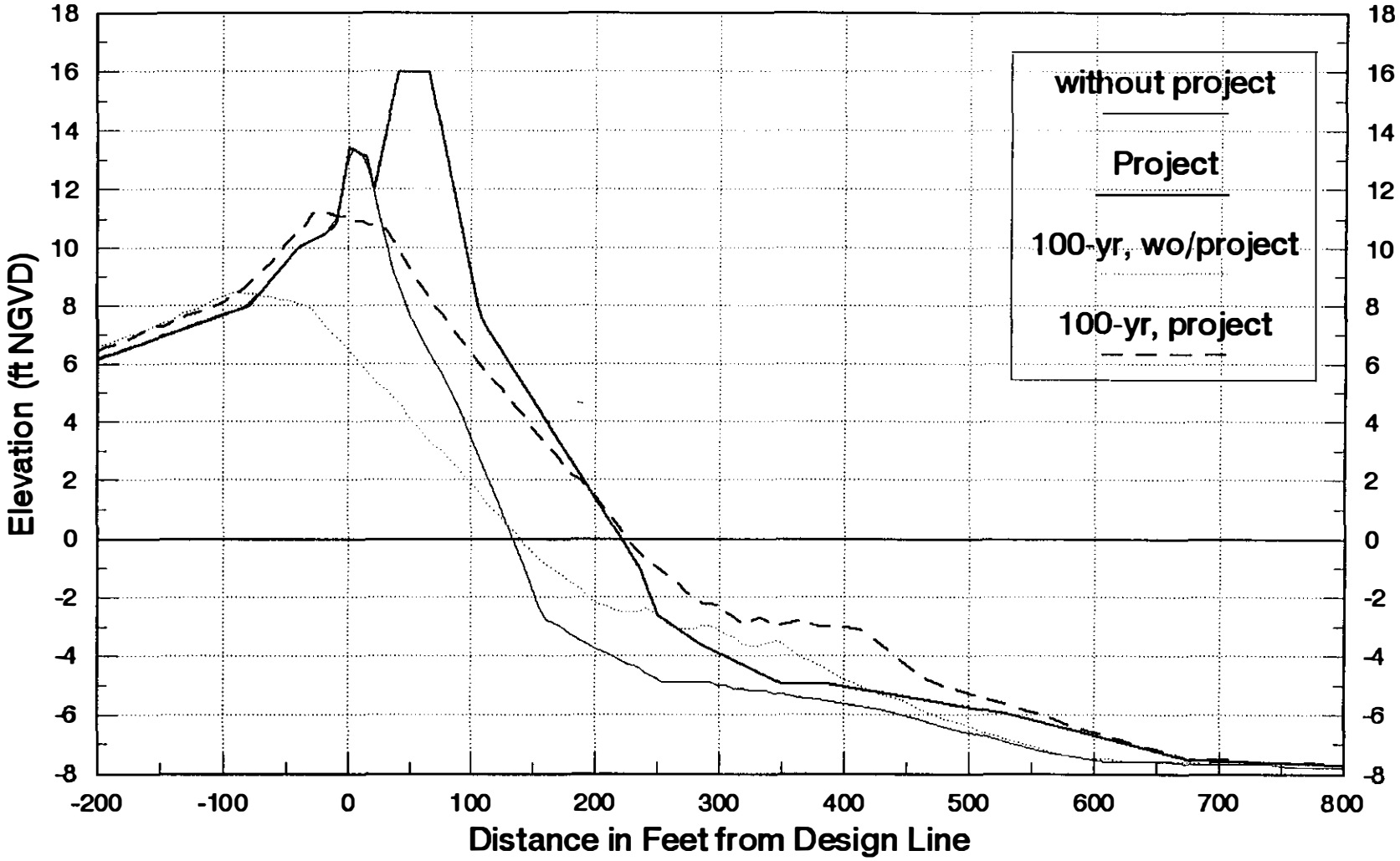
BEACH PROFILES, LRP#25A
BROADKILL BEACH, DELAWARE



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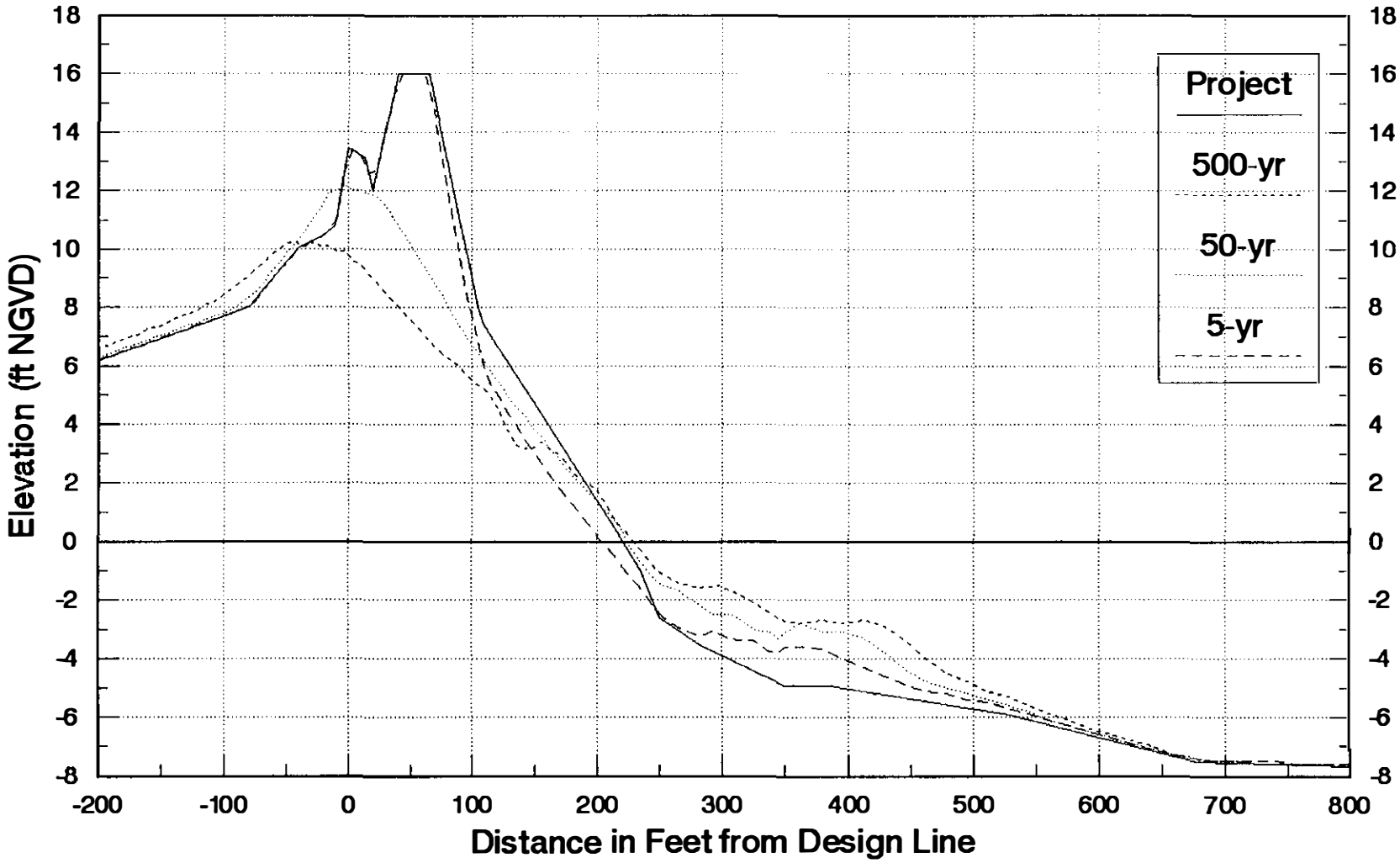
BEACH PROFILES, LRP#25A
BROADKILL BEACH, DELAWARE



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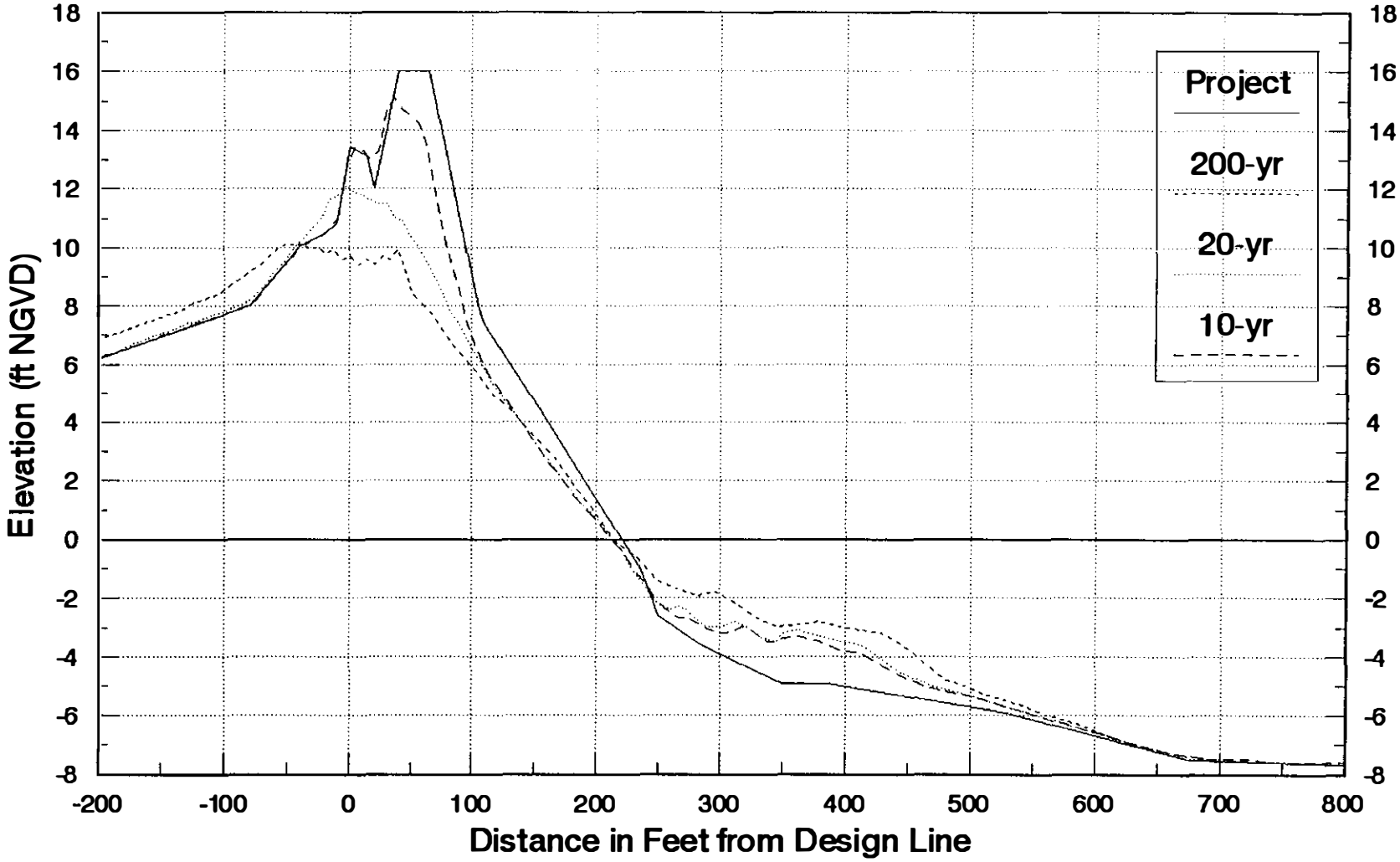

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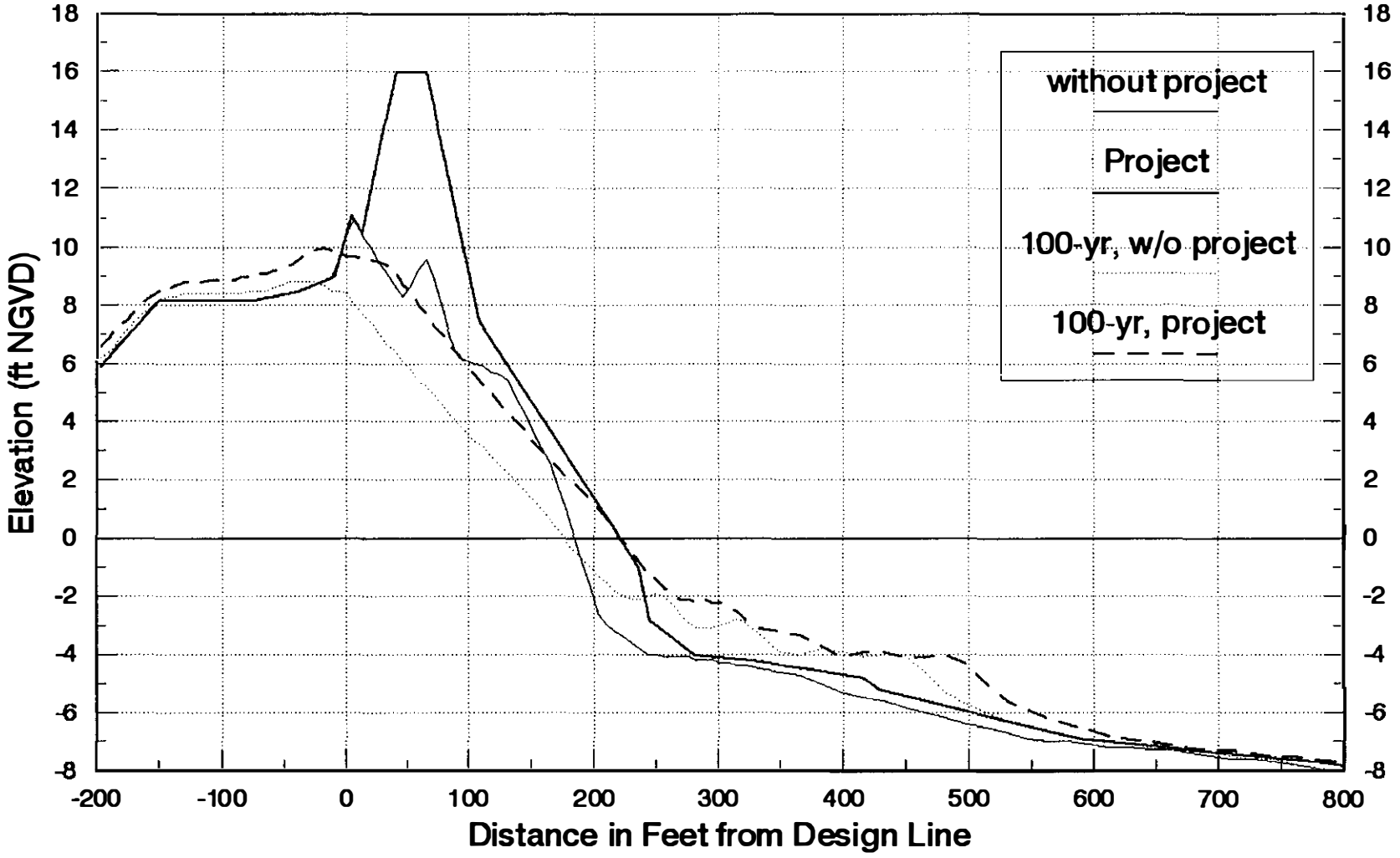

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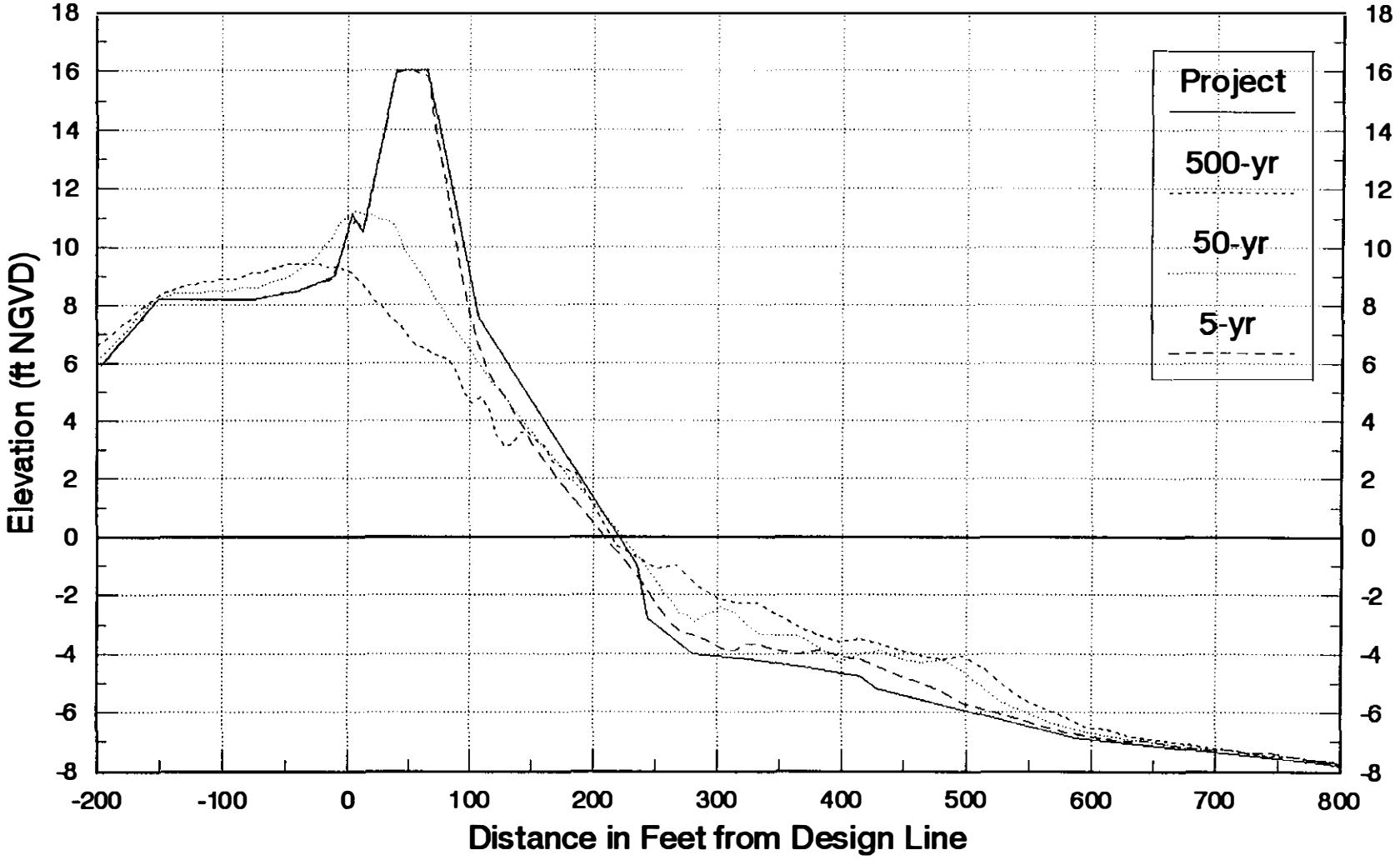

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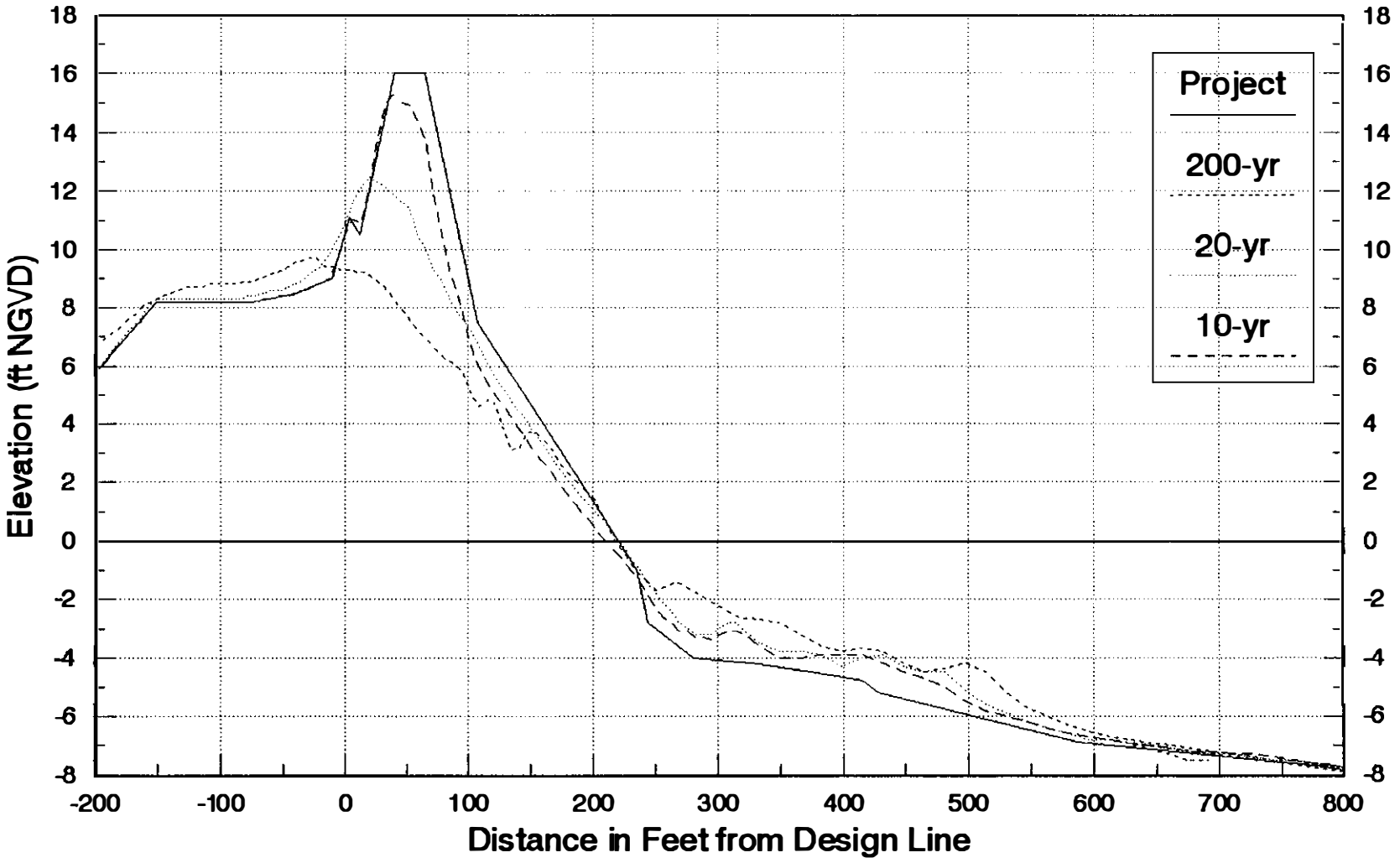

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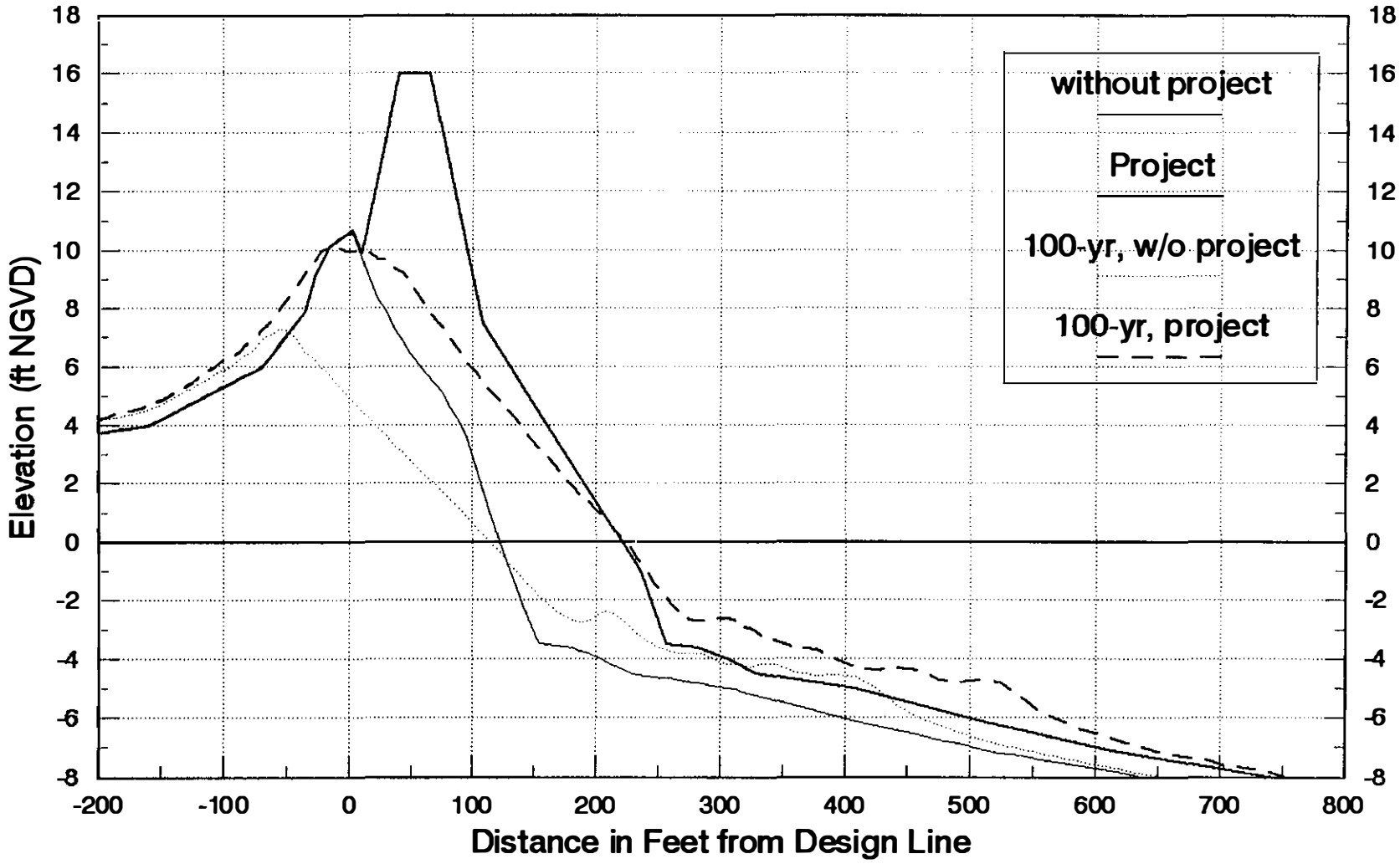

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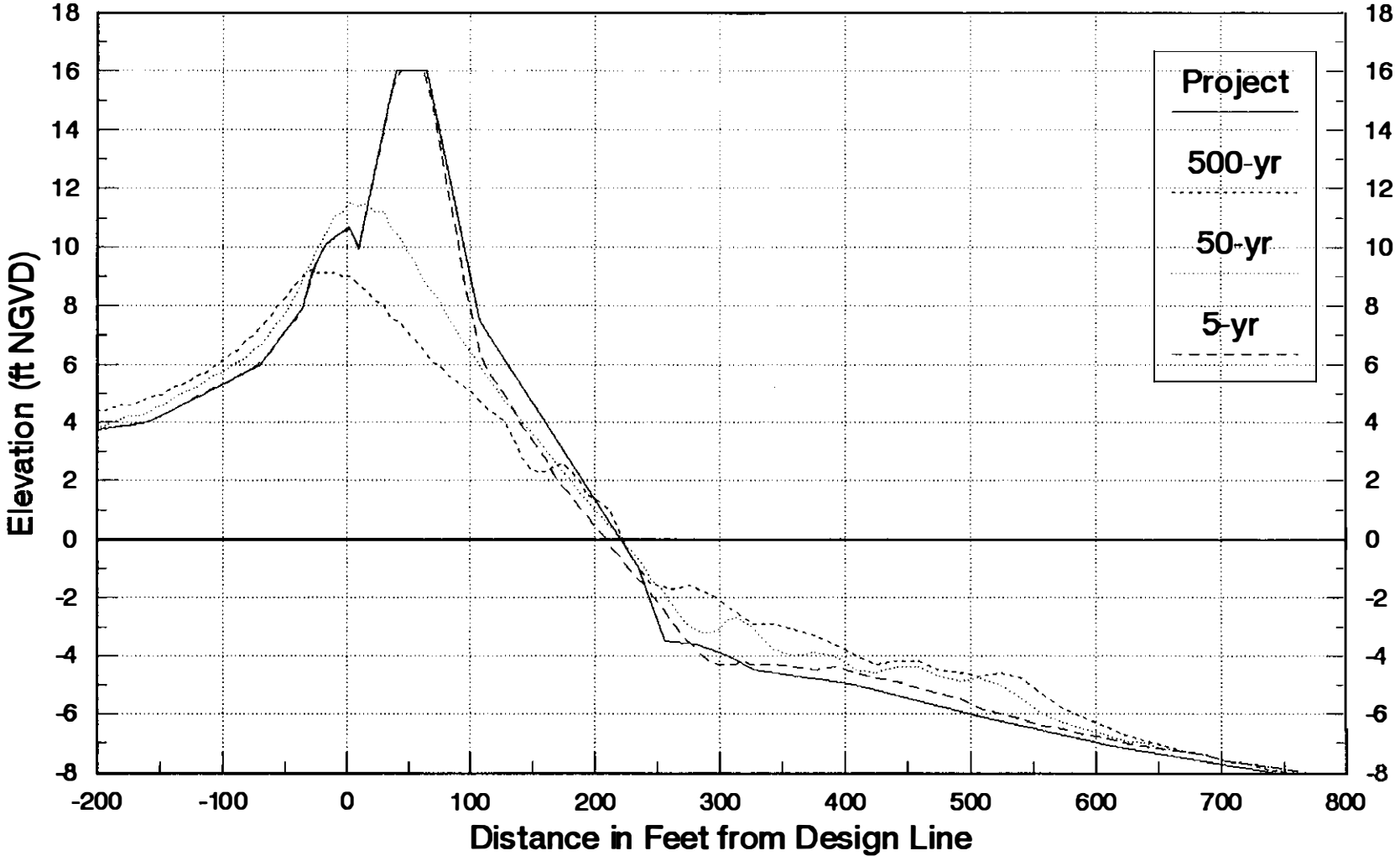

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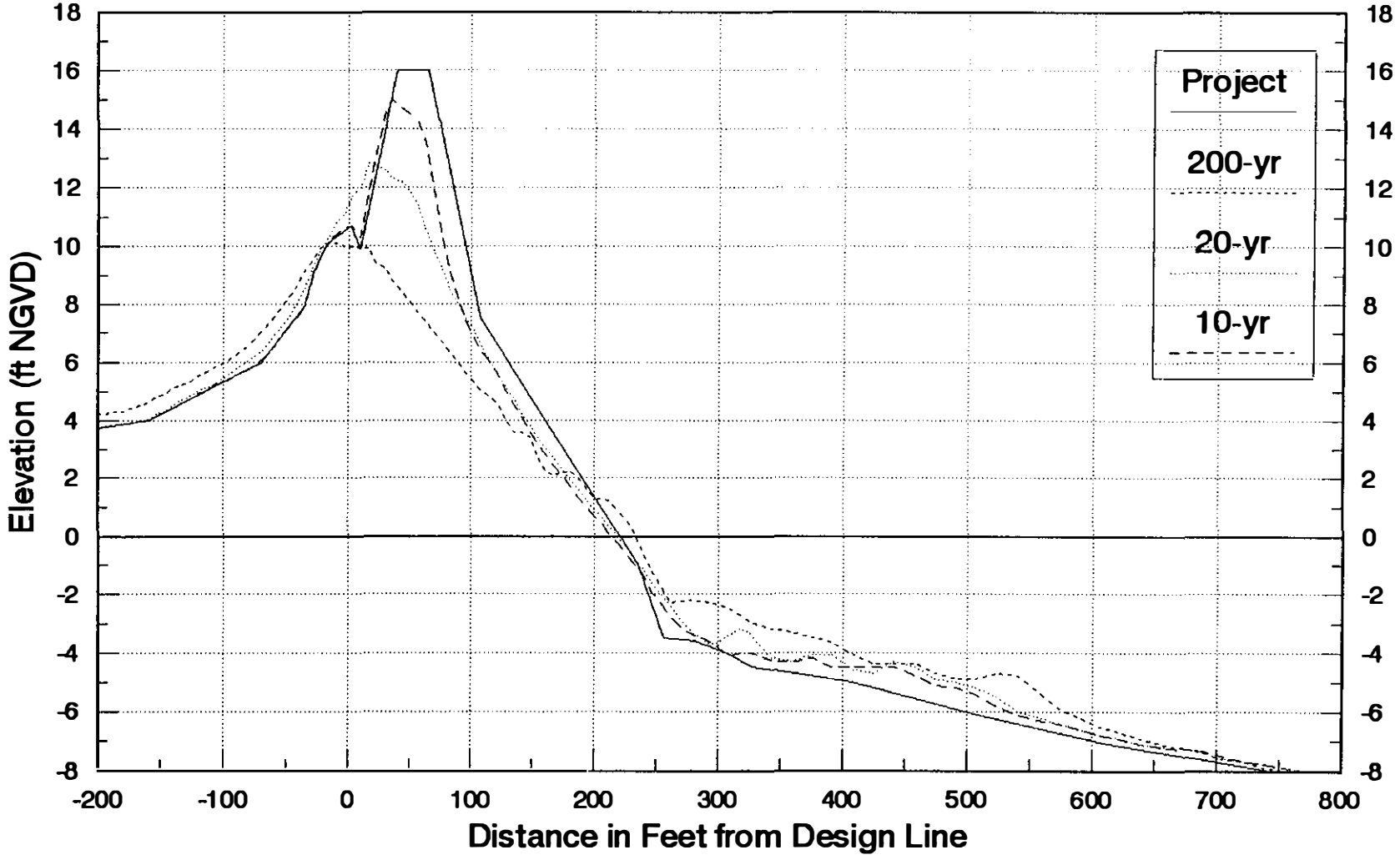

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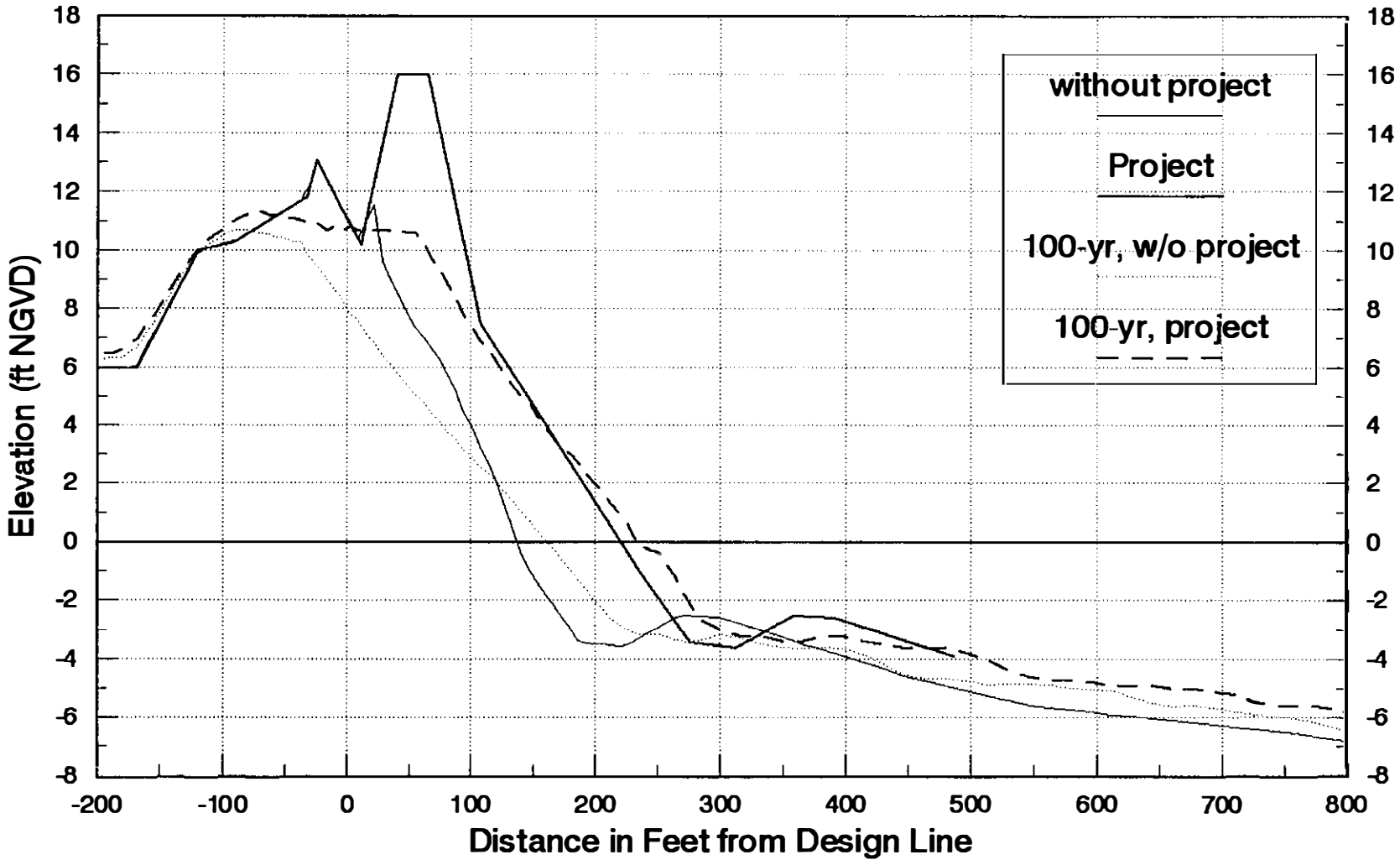

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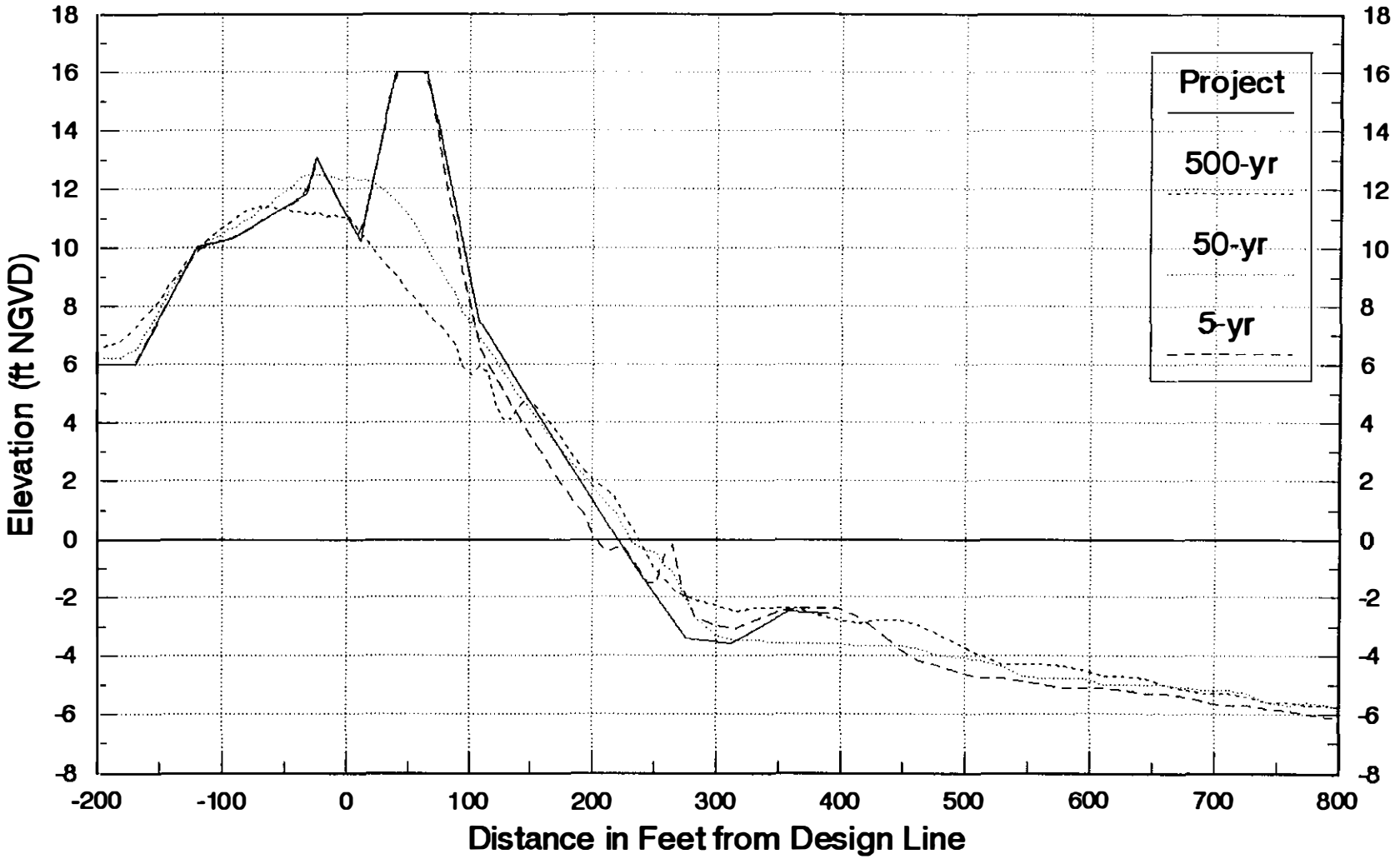

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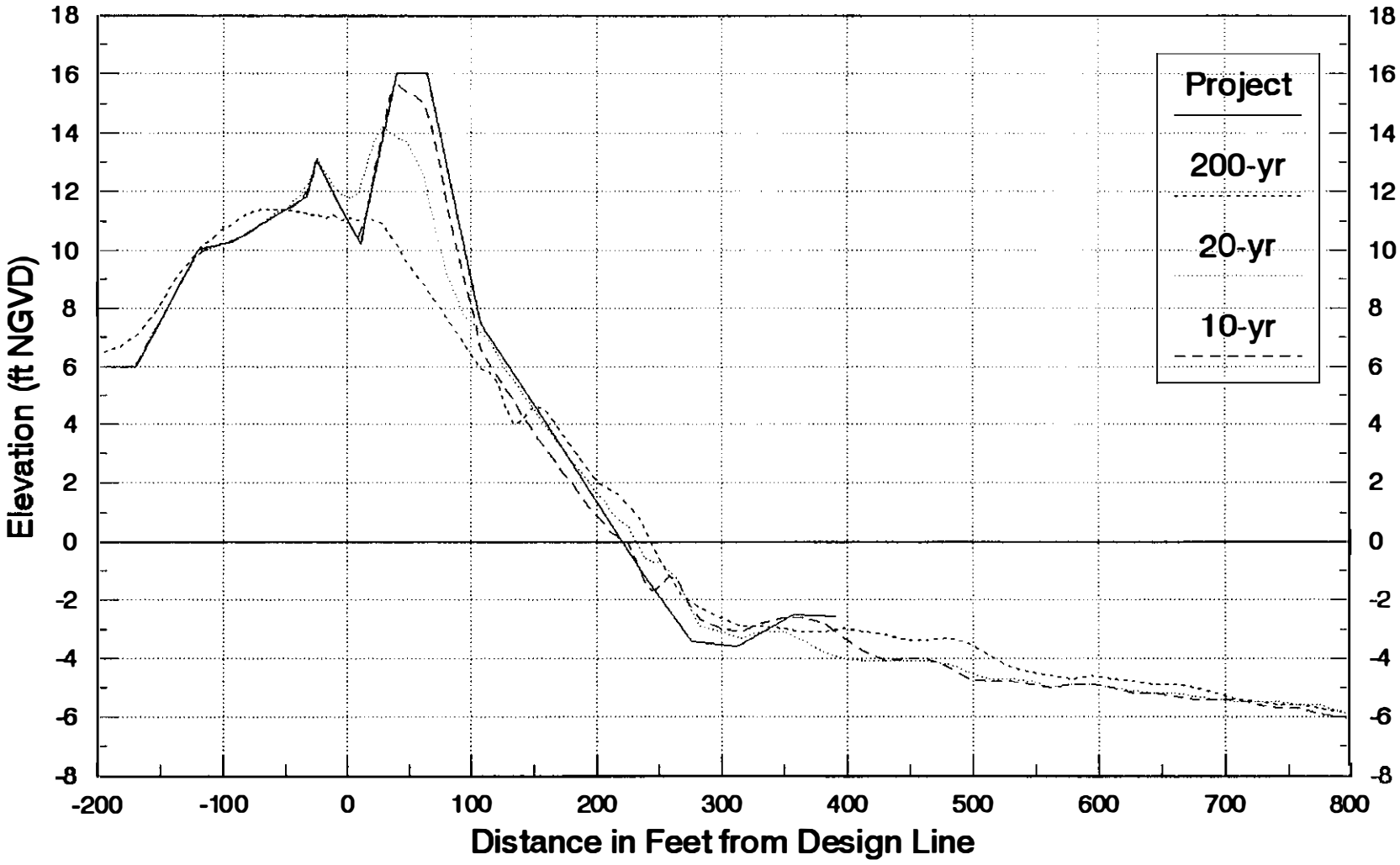

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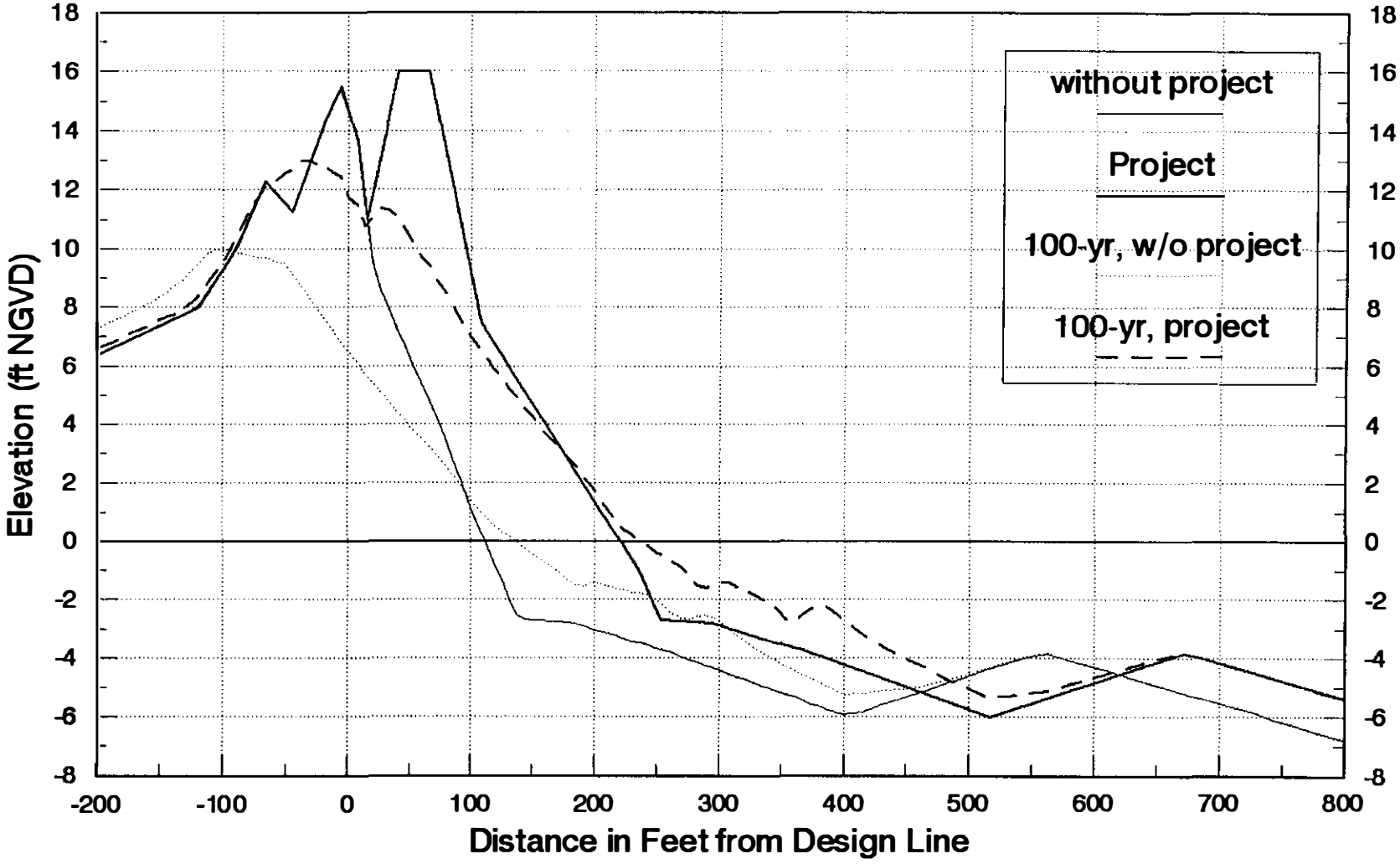
BROADKILL BEACH, DELAWARE



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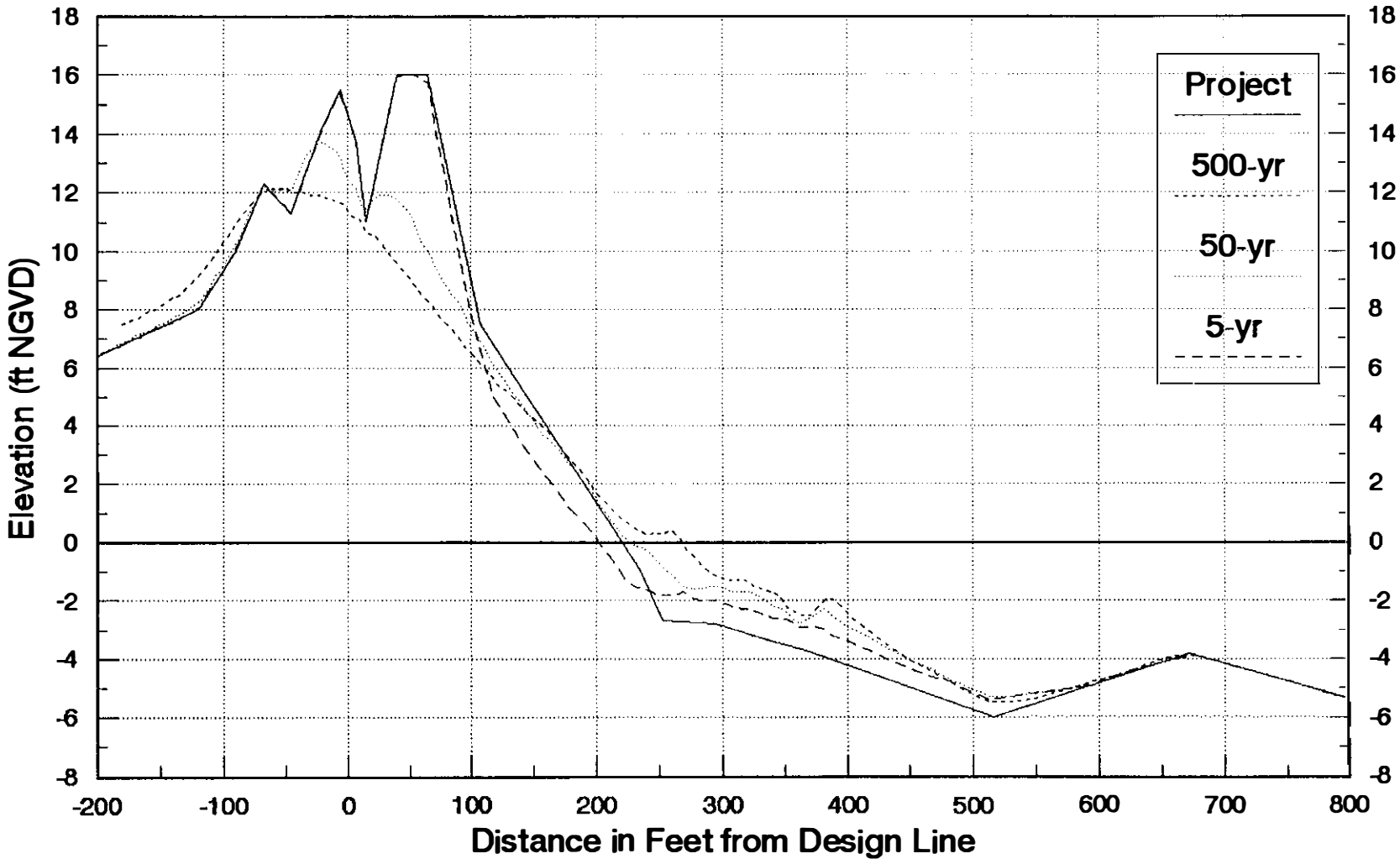

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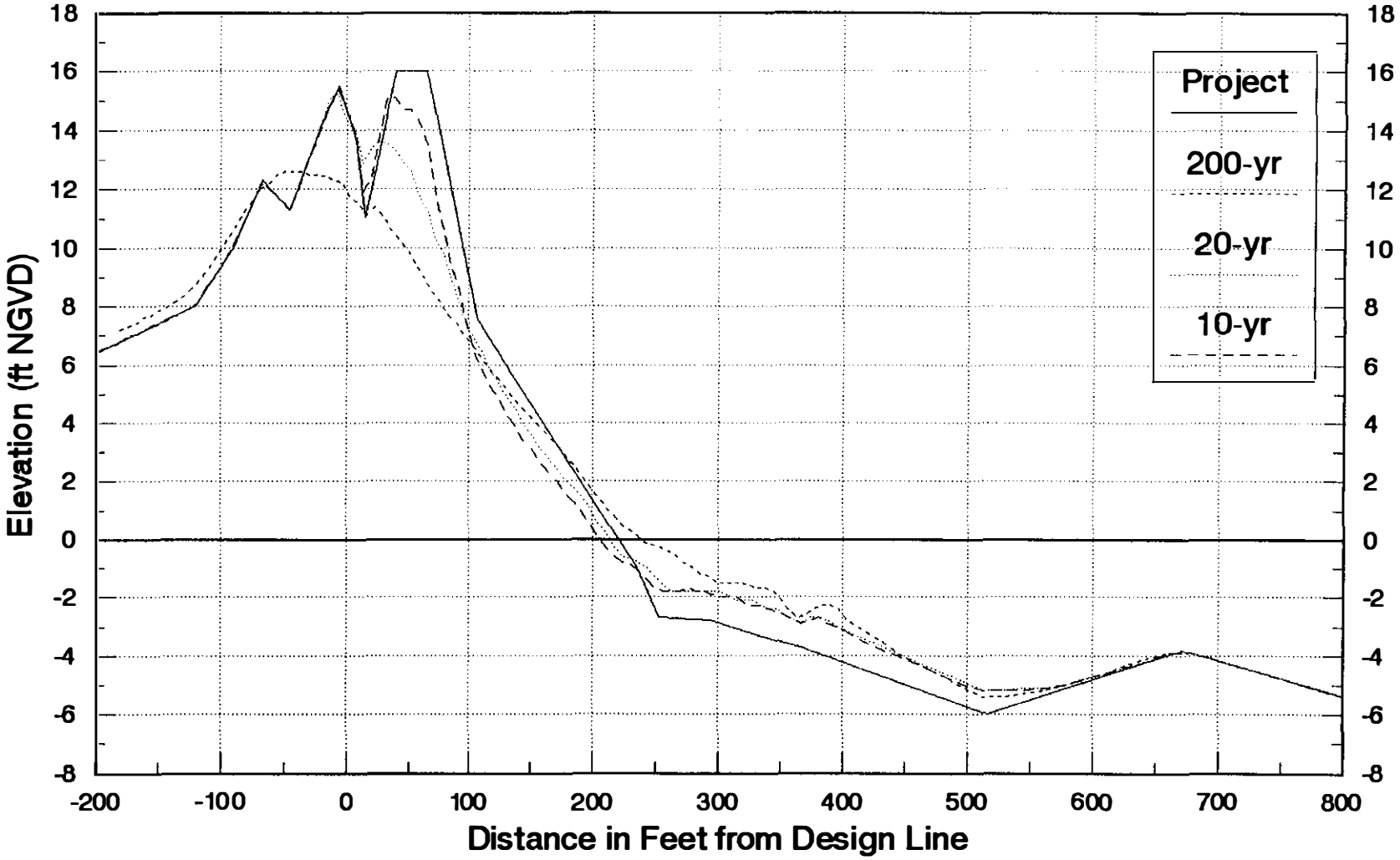

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BEACH PROFILES, LRP#28
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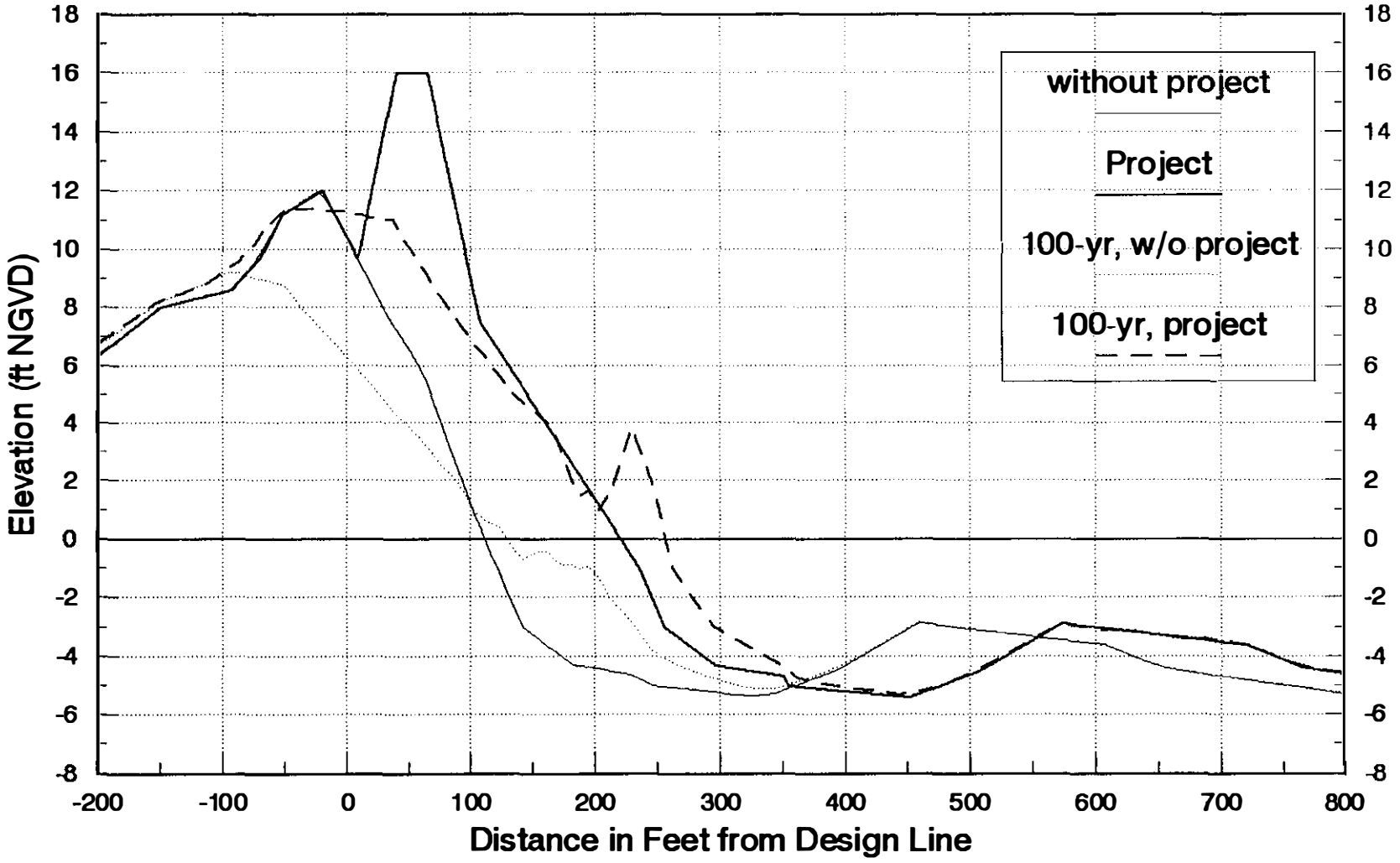

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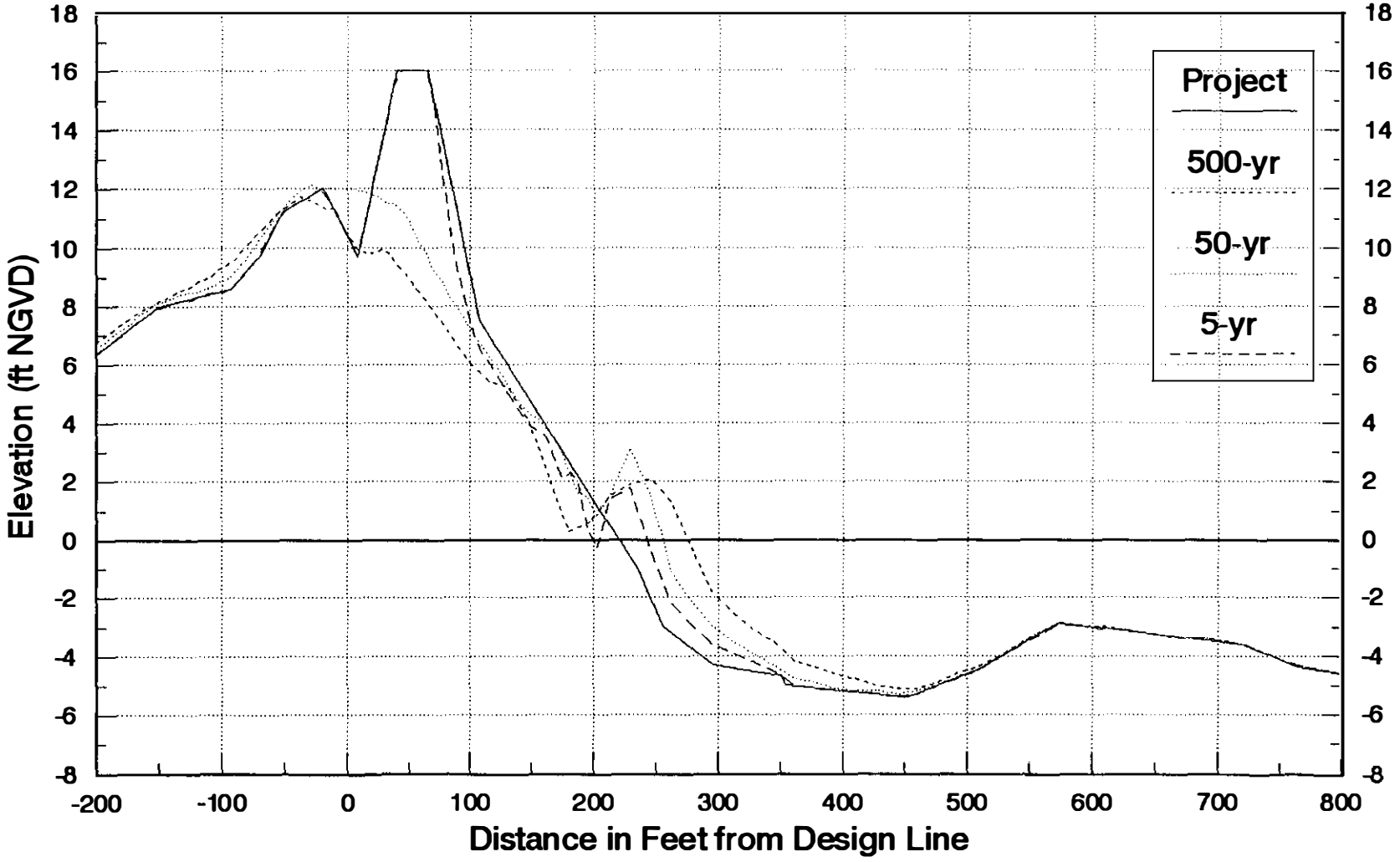

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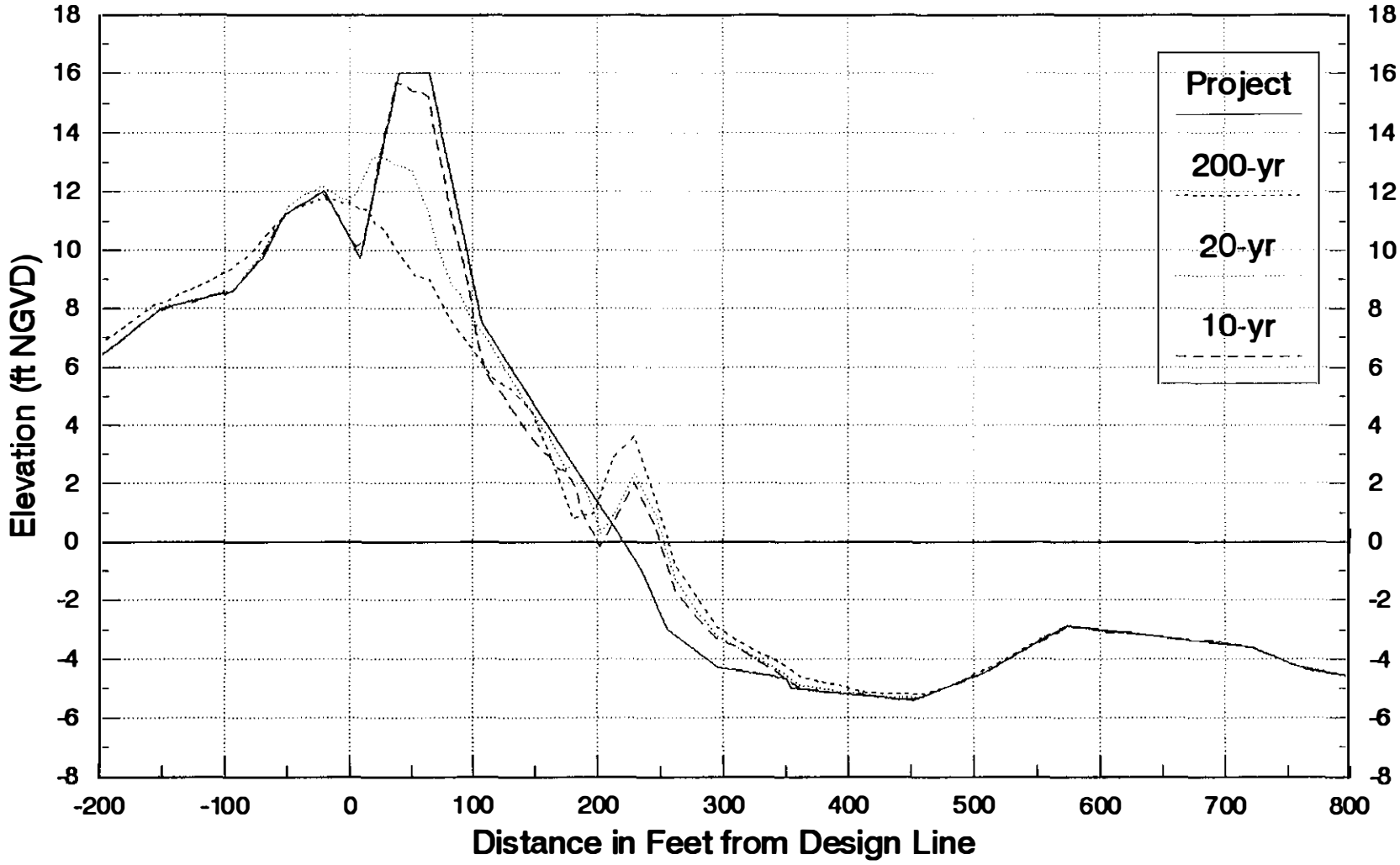
BROADKILL BEACH, DELAWARE



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SECTION 3

SURVEYING AND MAPPING REQUIREMENTS

Survey data used for the Feasibility Study was collected at Broadkill Beach in September 1993 by the State of Delaware. The spacing between survey lines was sufficient to adequately describe the Broadkill Beach shoreline. Each profile was extended landward using 1993 digital photogrammetric elevations.

Data collected for the study mapping effort include the following: aerial photography, first floor elevations of all structures in the first block fronting the ocean, street centerlines, and spot elevations throughout the study area. Computer programs used to read and store the mapping include AutoCAD and Map and Imaging Processing System (MIPS), which is a program used for Geographic Information Systems (GIS).

Mapping developed for this Feasibility Study is sufficient for the plans and specs phase, but new survey data will be required. Beach profile surveys every 100 feet from landward of the existing dune to beyond the depth of closure will be necessary to accurately determine quantities in developing the plans and specs.

SECTION 4

GEOTECHNICAL INVESTIGATION

Geotechnical Appendix
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December 1995

US Army Corps
of Engineers
Waterways Experiment
Station

Sediment Characterization and Beachfill Borrow Area Assessment of the Delaware Bay Study.

Report 1
Identification of Sediment Types Offshore of Broadkill Beach, DE Area.

by *Donald K. Stauble, PhD*
Coastal Engineering Research Center

Richard G. McGee
Hydraulics Laboratory

DRAFT

Prepared for U.S. Army Engineer District, Philadelphia

418

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3909 Halls Ferry Road,
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DRAFT

Report 1 of a series
Approved for public release; distribution is unlimited

Prepared for U.S. Army Corps of Engineer District, Philadelphia
Wannamaker Building, 100 Penn Square East
Philadelphia, PA 19107-3390

2

1 Introduction

In response to a request from the Philadelphia District (NAP) for assistance in identification of beachfill borrow areas in the offshore of the Delaware and New Jersey coasts within the Delaware Bay as part of a Feasibility Report for the Delaware Bay Study, the Coastal Engineering Research Center (CERC) in conjunction with the Hydraulics Laboratory (HL) at the Waterways Experiment Station (WES) is submitting Report 1 on the identification of sediment types offshore of Broadkill Beach, DE Area.

Eroding areas of the bay shoreline have resulted in a need to investigate the shallow offshore areas of the bay adjacent to both the Delaware and New Jersey coasts for use as a borrow area for beachfill material. The area is relatively unexplored from a geotechnical standpoint and this investigation will provide acoustical subbottom profiling, vibracore locations and interpretation of the sediment substrate of the study area. **Figure 1** shows the extent of the seismic track lines and location of the sediment cores for the entire study area. The study will be subdivided into several reports on sections of the coast. Field work was conducted to provided seismic tracks of the study area. A review of existing data was done concurrently to incorporate this data into the study. Vibracore locations were chosen based on the seismic data collected for this study and the existing data. The vibracore collection was done by the district, and provided to WES. Interpretation of the seismic and sedimentological data from these cores was made and this report characterizes the sediment of the Broadkill Beach portion of the Delaware Bay study and suggest suitable borrow areas for beachfill use.

Modern sedimentary deposits in Delaware Bay are controlled by past geologic processes. Lower Delaware Bay was thought to be a dendritic drainage pattern of gravely and muddy sands during the late Wisconsinan low stand of the sea around 15,000 to 12,000 years ago (Knebel et al., 1988). As sea level rose, the estuary became flooded as the bayside shoreline retreated northwest along the pre-transgression topography. A detailed history of the evolution of the bay shoreline on the Delaware side of the bay is presented in Fletcher et al. (1990). The sediment and bathymetry off Broadkill Beach is a result of this transgression of sea level into the drowned river valley. Coarse grained sediments were originally deposited in the river beds. Rising sea level flooded these river valleys and fine silts and clays were deposited as the turbidity maximum migrated up the estuary. As the estuary grew, narrow barrier and headland beaches were formed, from wave and tidal action and were composed of

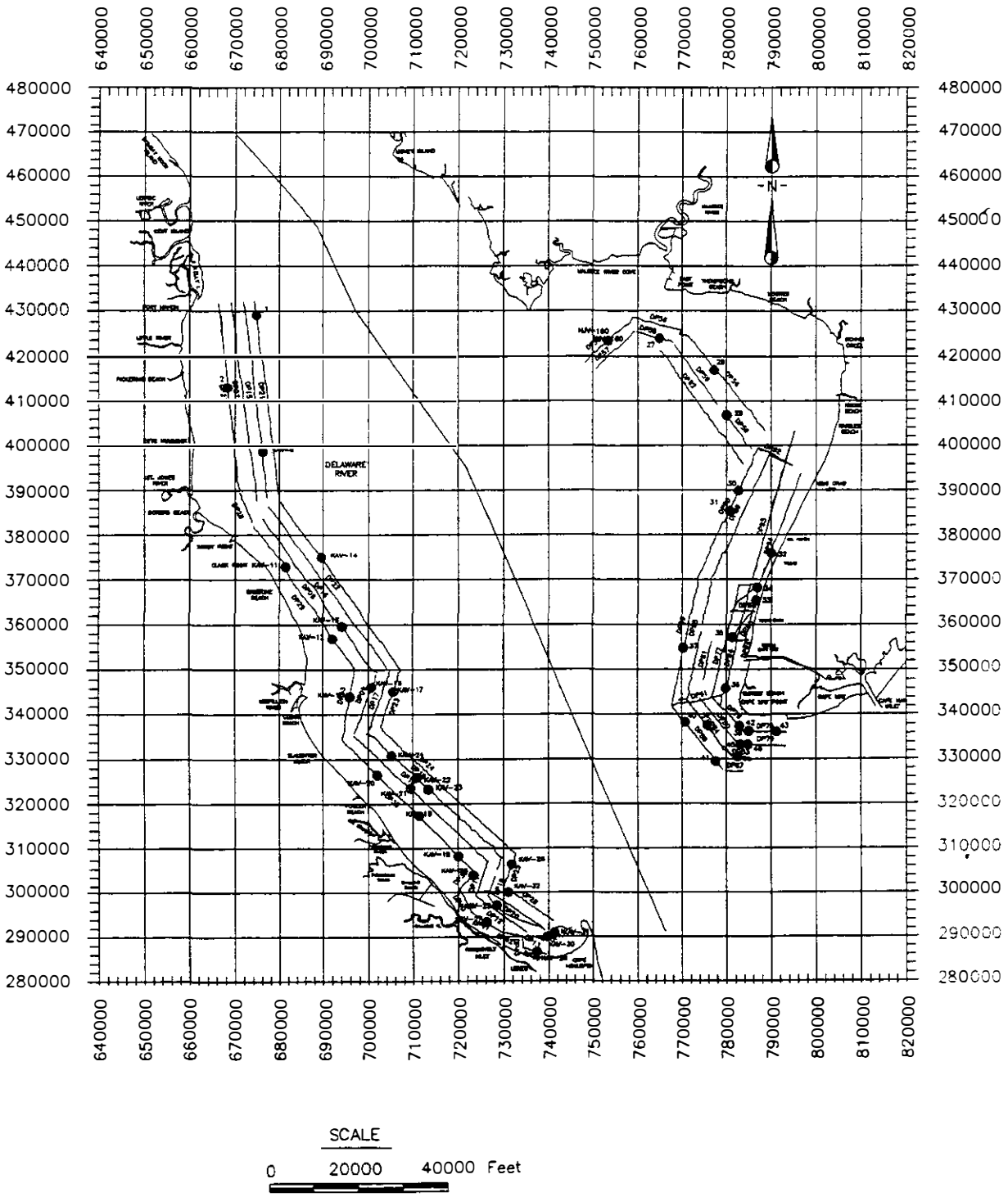


Figure 1. Location map of seismic track lines and cores collected along the Delaware and New Jersey shore of Delaware Bay

fine to coarse sands. Tidal marshes composed of silts were formed behind these barrier islands in the tributary river beds. In the present day open estuary, sediment scour, reworking and transport have become the dominant processes. Delaware Bay is a well-mixed estuary from Port Mahon to the bay mouth at Cape Henlopen.

2 Approach

The field work included planning, testing and collection of new acoustic subbottom profile data along the Delaware and New Jersey shore adjacent areas (5-20 ft depths). The area between Port Mahon and Lewes, Delaware was studied from approximately 1 to 3 miles offshore. The area approximately 1 to 3 miles offshore between the Maurice River mouth and Cape May Point was studied on the New Jersey side of the Bay. This field work was conducted on both the Delaware and New Jersey sides of the Delaware Bay in conjunction with field work on the Delaware River Main Channel Preconstruction and Engineering Design Study during the summer of 1993. The R.V. Waterways Explorer was outfitted with a fathometer, side scan sonar, 3.5 kHZ subbottom profiles, and a 600-900 Hz bubble pulser. A differential global positioning system (DGPS) was used to provide accurate positioning of the seismic lines. Details of the acoustic calibration are presented in the Delaware Ship Channel Report by McGee (in press).

An analysis of core locations was provided to NAP at the completion of the field work to allow for a contract for vibracoring to the done 1994. Sediment core logs and sediment grain-size analysis was performed by the South Atlantic Division Laboratory in February 1995. Analysis of the sediment characteristics and borrow area identification was done after receipt of the vibracore data from NAP in April 1995. The data analysis has been divided into two separate subsections that separates the analysis of the Delaware and New Jersey sides of the Bay at the request of the District. Similar analysis is being performed on both data sets and the data has been presented to the District in GIS compatible format.

A review of existing sets of subbottom and core data was combined with the new field data collected to assess the bathymetry, stratigraphic sequences and sediment composition of the upper 20 ft of sediment along the Delaware and New Jersey shore adjacent areas of Delaware Bay. This report will focus on the southern Delaware Bay shore area between the coast and 3 miles offshore, from an area just south of the Mispillion River to Cape Henlopen. This includes the area offshore of Lewes and Broadkill Beach.

After the field work was completed a review the seismic data and assessment of the best locations to take cores to identify the sediment characteristics and ground truth the seismic signature. Details of the analysis procedure are located in McGee, et al. 1995.

3 Analysis

Identification of Possible Beach Fill Borrow Areas and Sand Substrate Identification

The purpose is to assist NAP with techniques and evaluation of the suitability of borrow areas for possible beach fill projects adjacent to the Delaware and New Jersey bay shoreline. Sediment data typing, grain size analysis interpretation and suggestion of borrow areas is provided to assist the State of Delaware in a proposed beach fill project along Broadkill Beach. Broadkill Beach is a small community located on the bay shoreline, five miles north of the mouth. A map has been generated and originally sent to the district by INTERNET as requested that shows the possible bottom types that can be used to select areas for possible sand sources for beach nourishment (**Figure 2**). This map of the surface sediment types was constructed from analysis of the seismic data and the surface sediment data from the cores. The use of core information has allowed a better refinement of the seismic data. The map outlines the best areas for possible sand.

Additional information, taken as part of a State of Delaware study of the nearshore area off Broadkill Beach, DE. was supplied by the State of Delaware. Two technical reports by Wethe et al. (1982) and Wethe (1984) indicate that there may be possible sand in the nearshore vicinity of Broadkill Beach. This area was inshore of our seismic study. A copy of the pertinent core locations and possible sand sources from these reports are found in **Figure 3**. Several former borrow areas used in 1961, 1973, 1975, 1976, and 1981 are shown in the nearshore region (around 1000 ft from shore).

Former pre-transgressive river valleys appear to trend offshore in a southeast direction from the three present day rivers in the area (Maley, 1981). Sand size material was identified in shoals between these former valleys (**Figure 4**). Cross-sections from Maley (1981) in the inshore area indicate that the sand shoals are over lagoonal mud deposits, particularly in the ancient valleys (**Figure 5**). The present data agrees with the Delaware reports. The interpretation of the surface acoustic impedance and sediment data shows that sand areas appear to trend in a NW-SE direction from the Delaware shoreline off Broadkill Beach. The red, yellow, and green areas on **Figure 2** identify the sand areas, with magenta, blue and brown areas denoting a mix of mostly silt and mud. The area behind Cape Henlopen is a basin that contains very fine grain surficial sediments. Some of the cores in Breakwater

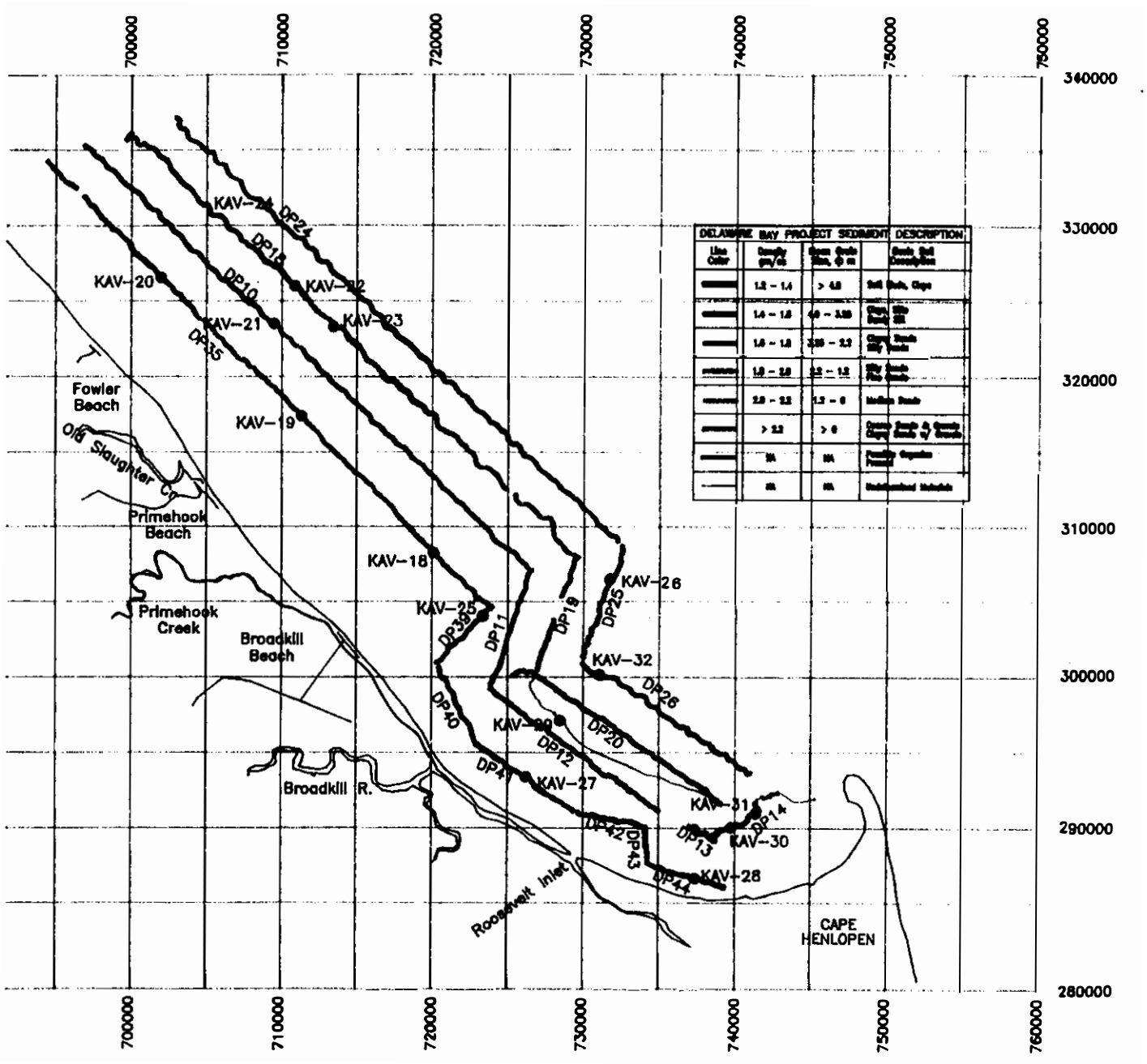


Figure 2. Detail of seismic lines and core locations from Mispillion River to Cape Henlopen, DE. showing surface sand types

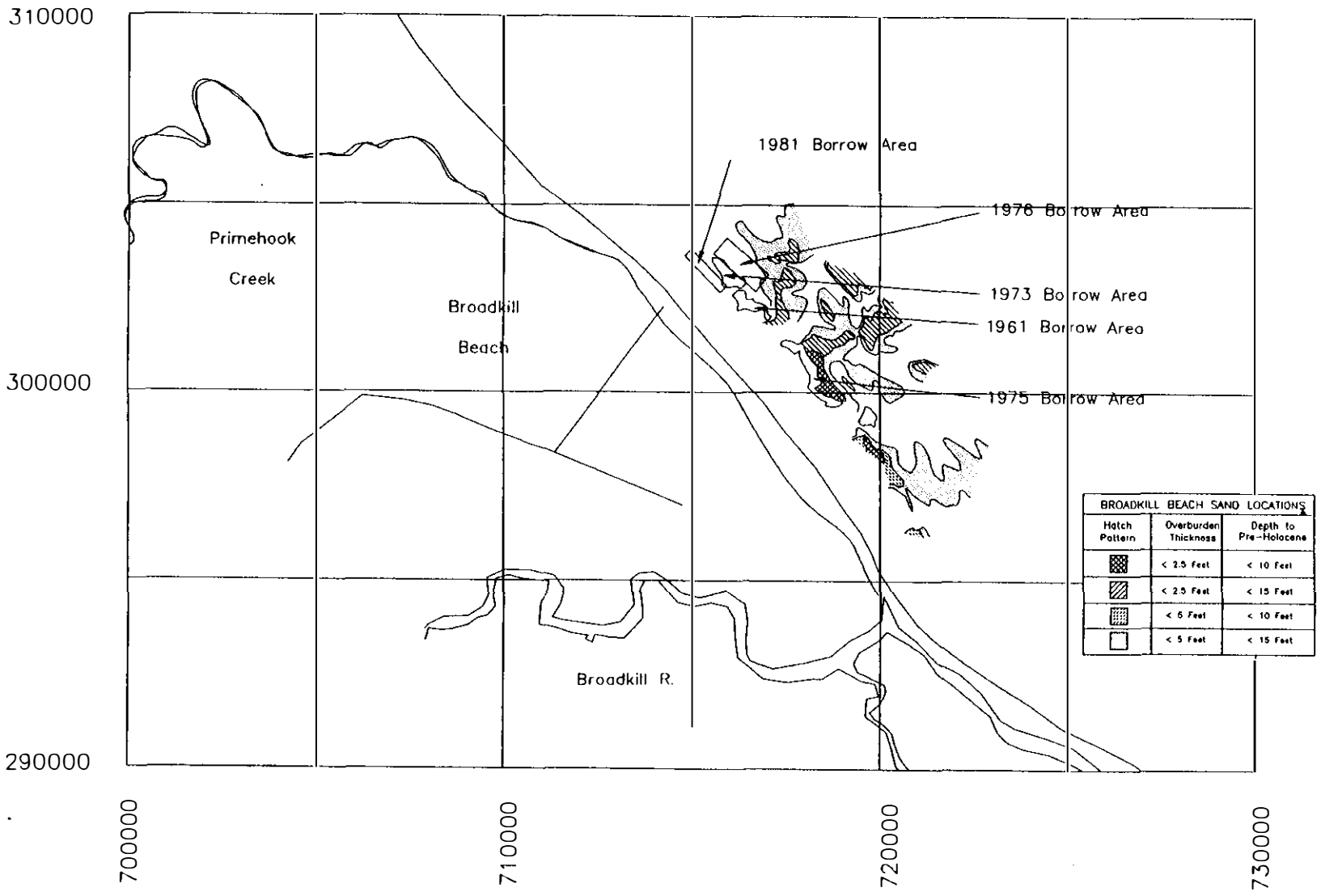


Figure 3. Location of inshore sand deposits and borrow areas (after Wethe, 1982)

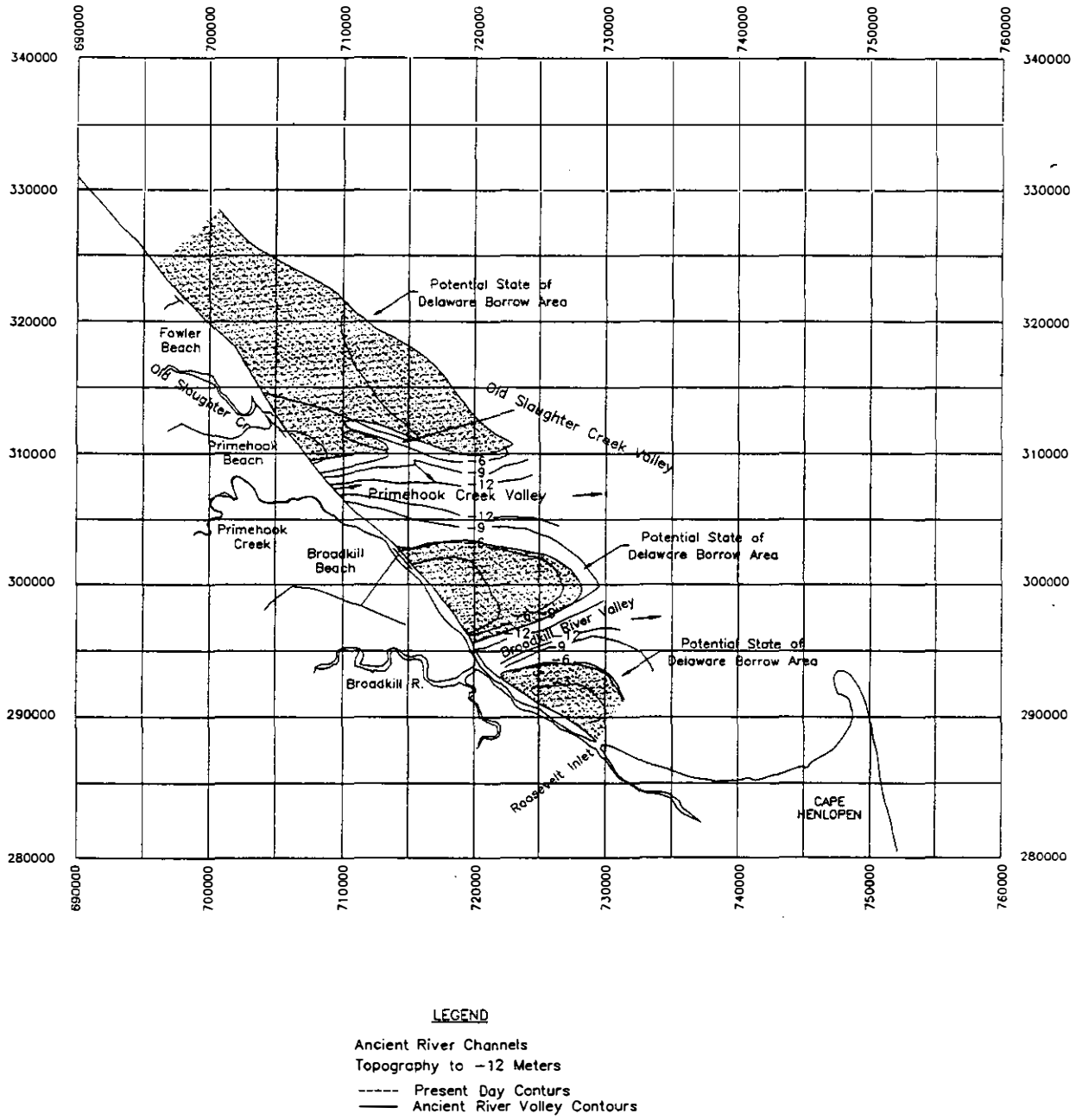


Figure 4. Location of sand shoals and ancient river valleys around Broadkill Beach (after Maley, 1981)

Figu

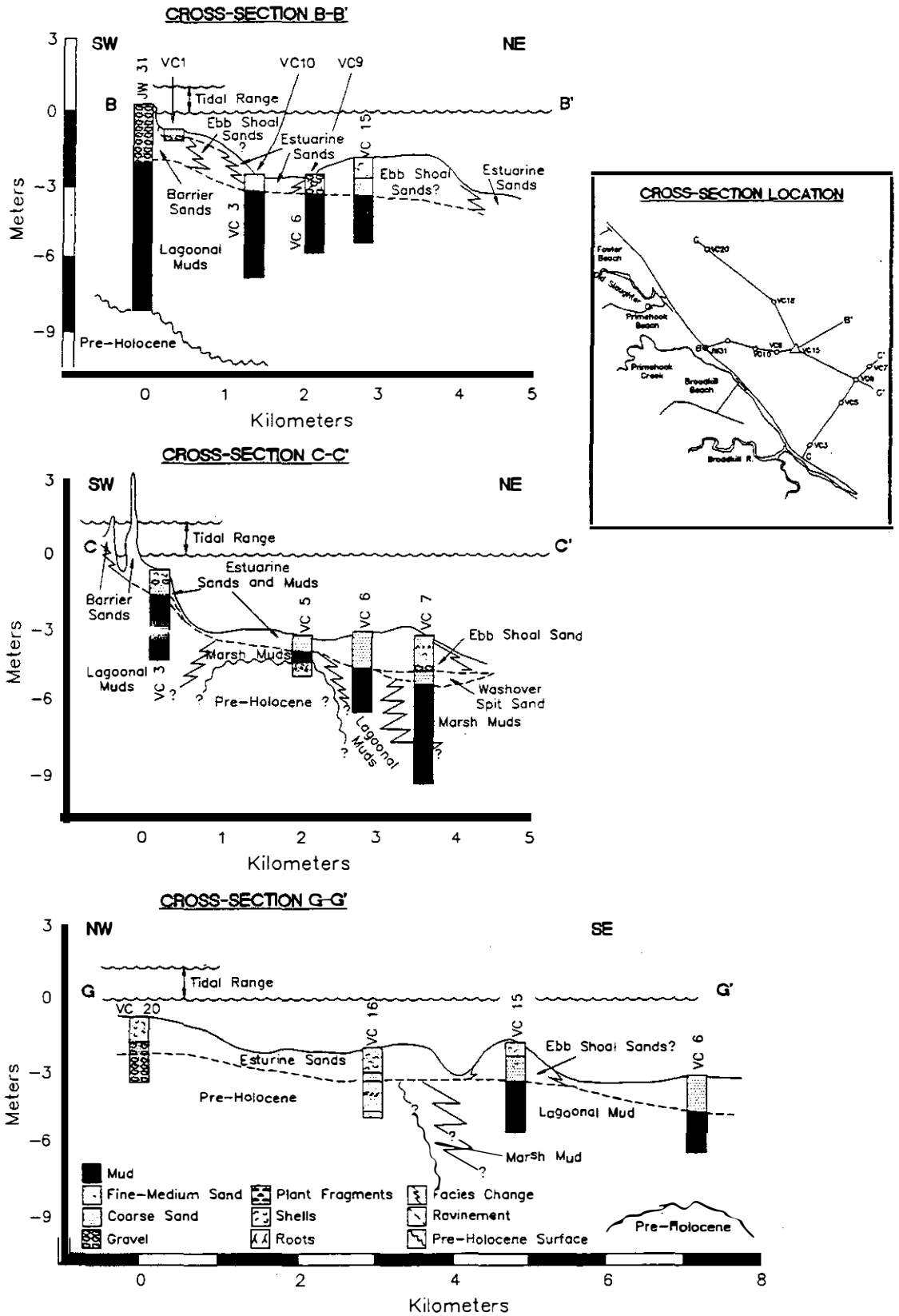


Figure 5. Cross-sections of inshore area (after Maley, 1981)

Harbor show a coarser Pleistocene sand deposit around 10 feet below the surface in this area, but for the most part the surface material is too fine for beach nourishment.

Line DP35

Acoustic impedance analysis of four seismic track lines off Broadkill Beach show the stratigraphic distribution of sediment with depth. Line DP35 was the closest to shore. **Figure 6** shows the typical pinger data collected on that line. From this data and additional information of three cores taken along this line at KAV-18, KAV-19 and KAV-20, **Figure 7** show the sequence of shoals and river valleys. The southern part of the line contains medium to fine surficial sands. The sediments grade into silts at a depth of around 12-14 ft. This area corresponds to the ancestral Broadkill River Valley as described by Maley (1981).

Core KAV-18 is located in the area of the southern tip of the shoal identified by Maley (1981). The surface layer contains a poorly sorted fine gravel down to 3 ft. Below that the majority of the sediments are poorly sorted sands with some gravel to a depth of 13 ft. Below that depth the sediments are a fine silt. Appendix A gives the core log and sediment gradation curves for each vibracore location.

The central part of seismic line DP35 has a section of fine silty sands at the surface, located in a valley between two shoals. This is the landward extent of a wedge of fine material. Core KAV-19 located in this valley indicates that this silty sand material extends to a depth of at least 17.5 ft and supports the acoustic signature of fines at depth. Below a 3 ft thick poorly sorted medium sand layer, there are well sorted fine sands that have silt contents between 10% and 35%.

The northern portion of the line contains medium to fine sandy surfacial sands. The northern part of this area is in another valley and the sediments grade into fine silts, possibly representing another ancestral river valley trending to the southeast. These fines may be outwash from an ancient more straight path of Slaughter Creek. Core KAV-20 in this northern area contains a thin gravely sand on the surface, with gravely clay lenses alternating with silts at depth. Most of the sediment has a D_{50} of around 0.05 mm (4.25 ϕ) from 1 ft down to 8 ft and gets only slightly coarser below that depth. Below 3 ft the sediment has iron staining.

Line DP10

The second seismic line seaward of the shoreline was Line DP 10 (**Figure** 8). The northern, central, and southern sections of this line have medium to fine sand surficial sediments located on shoals. Between the shoals, finer silty sediments were found in the valleys. Core KAV-21 was located on the central shoal area and contains well sorted medium to fine sands representing the shoal on the surface and to a depth of around 7 ft. Below that depth most of the sediment is composed of silty sand with some clay lenses around 9 ft. This sediment at depth is finer bay material underlying the shoal.

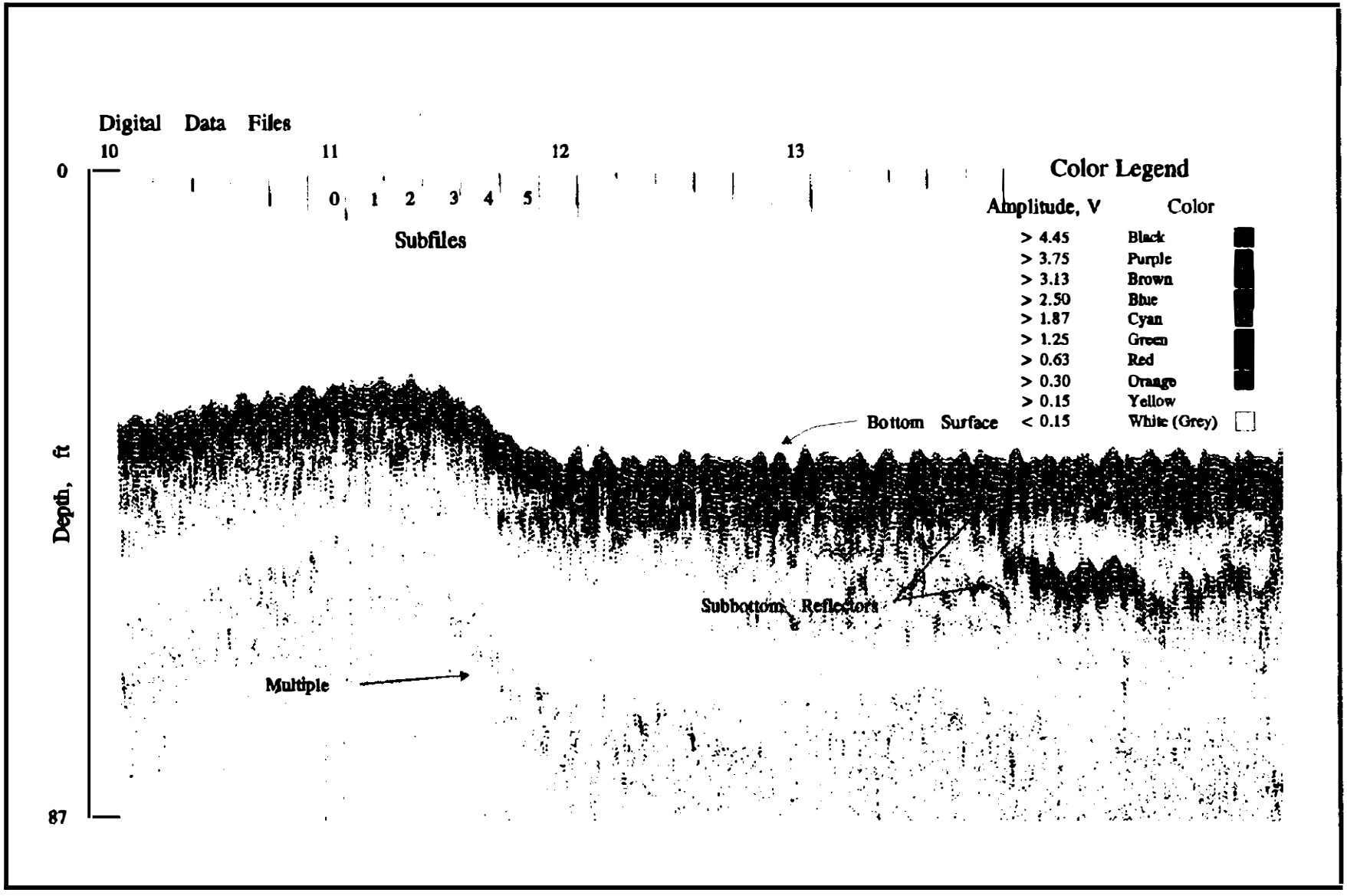
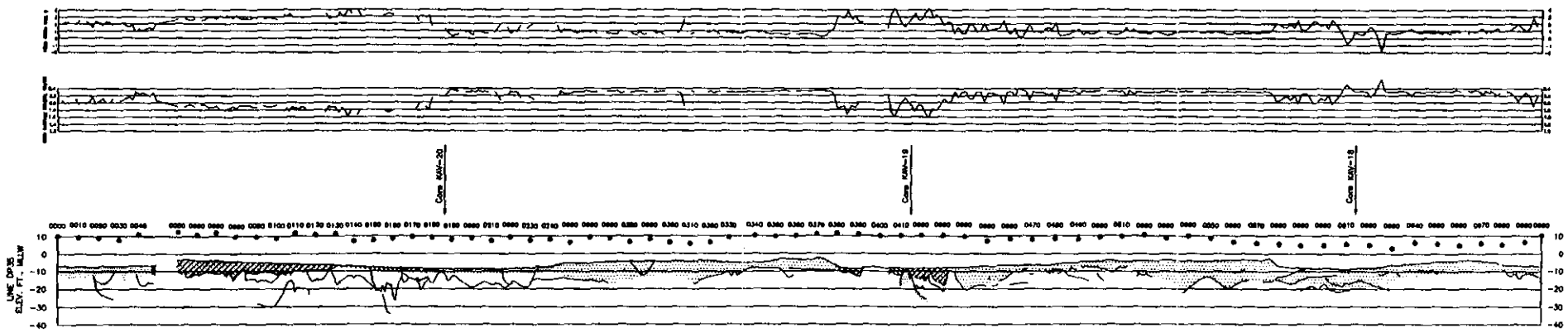


Figure 6. Example of typical pinger data record



DELAWARE BAY PROJECT SEDIMENT DESCRIPTION			
Hatch Pattern	Density gm/cc	Mean Grain Size, ϕ m	Basic Soil Description
	1.0 - 1.4	> 4	Soft Muds, Clays
	1.4 - 1.6	4 - 3.2	Clays, Silts Sandy Silt
	1.6 - 1.8	3.2 - 2.2	Clayey Sands Silty Sands
	1.8 - 2.0	2.2 - 1.2	Silty Sands Fine Sands
	2.0 - 2.2	1.2 - 0	Medium Sands
	> 2.2	> 0	Coarse Sands & Gravels Clayey Sands w/ Gravels
	> 2.4	N/A	Rock, Consolidated Clays

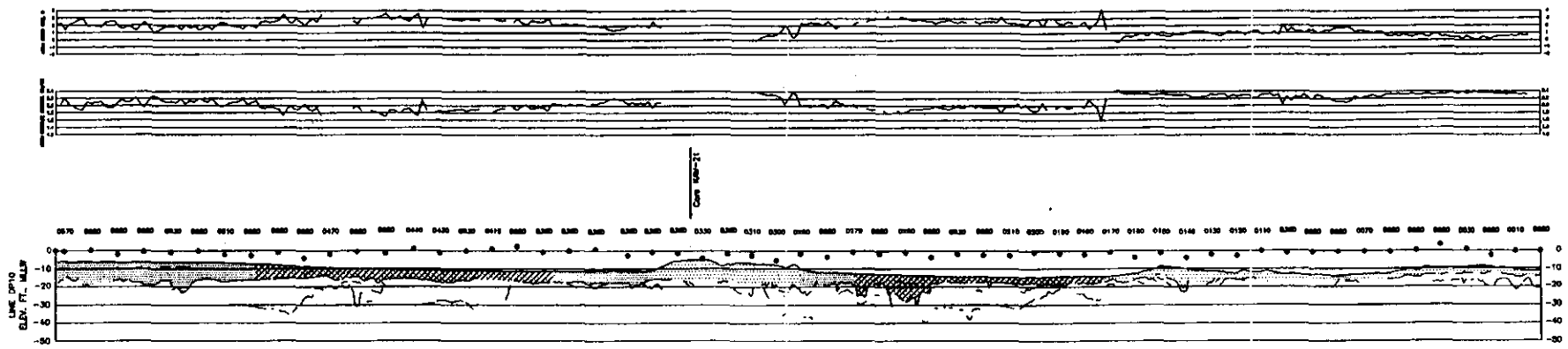
V.E. = 50

WATERWAYS EXPERIMENT STATION
CORPS OF ENGINEERS
VICKSBURG, MS 39180

DELAWARE BAY PROJECT
SEDIMENT PROFILE
LINE DP35

FILE NAME: DP35.DWG
SCALE: 1"=1000' | DATE: DECEMBER 11, 1993 | SHEET

Figure 7. Cross-section of Line DP35 with core locations



DELAWARE BAY PROJECT SEDIMENT DESCRIPTION			
Hatch Pattern	Density gm/cc	Mean Grain Size, φ m	Basic Soil Description
	1.0 - 1.4	> 4	Soft Mod. Clays
	1.4 - 1.8	4 - 3.2	Clays, Silts Sandy Silt
	1.8 - 1.8	3.2 - 2.2	Clayey Sands Silty Sands
	1.8 - 2.0	2.2 - 1.2	Silty Sands Fine Sands
	2.0 - 2.2	1.2 - 0	Medium Sands
	> 2.2	> 0	Coarse Sands & Gravels Clayey Sands w/ Gravels
	> 2.4	N/A	Rock, Consolidated Clays

V.E. = 50

Figure 8. Cross-section of Line DP10 with core location

WATERWAYS EXPERIMENT STATION
CORPS OF ENGINEERS
VICKSBURG, MS 39180

DELAWARE BAY PROJECT
SEDIMENT PROFILE
LINE DP10

FILE NAME: DP10.DWG
SCALE: 1"=1000' | DATE: DECEMBER 11, 1995 | SHEET

15

Line DP18

Further bayward Line DP18 (**Figure 9**) contains sand size material on the southern end shoal feature. Cores located along this line included KAV-22, KAV-23, and KAV 24. The central section of the line contains fine sands and silts. Core KAV-23 is the southern most core on this line and is located in a pocket of fine sand. This core is composed of mostly silty sand, with lenses of silt between 2 to 7.5 ft and below 7.5 ft a gravel component. Core KAV-22, located on the middle of the line contains silty sand with some clay material in the surface layers. Sandy silt is also present within the upper 4 ft. From 4-8 ft a well sorted sand with from 10 to 20 % silts was present.

Core KAV-24 is the northern most core and is located on the flank of a shoal which occupies the northern section of the line. The upper 5 ft of the core contains silty sand grading into well sorted medium to fine sands with between 2 and 10 % silts. Between 5 and 14 ft, silt material is present with some sand-size material. Below 14 ft the gravel and sand content increases along with a silt layer.

Line DP24

The most bayward seismic line is Line DP24 (**Figure 10**). No cores were collected along this line. The acoustic impedance analysis indicated that the southern end of the line has a sandy shoal feature. The central section is a long valley, containing finer sediments. A shoal of sandy material is at the northern end of the line. This central valley may be related to an ancestral river valley, possibly associated with Slaughter Creek, north of the study area. A deeply incised channel is present on the seismic record cutting down to around 45 ft below the bay bed.

Line DP39

Line DP39 is almost shore normal, extending bayward from just south of Broadkill Beach. **Figure 11** shows the cross-section of Line DP39 with core KAV-25 at the northeast end. The entire line is composed of medium sized sand.

This core is in the area of the ancestral Primehook River Valley. The upper 2 ft of the core contained sand material, with some shells and gravel-size material. Below 3 ft the sands graded into silt-size material, possibly the lagoonal mud deposits described in the cross-section G-G' in **Figure 5**.

Line DP11

Line DP11 runs perpendicular to the Primehook River valley axis. The cross-section is shown in **Figure 12**. No cores were taken along this line. The acoustic impedance analysis of the seismic data indicated that all of the surface sediment was medium to fine sands. A deep subbottom reflector in the center of the line may be associated with the ancestral Primehook River Valley. Line G-G' (**Figure 5**) crosses line DP11 and from that data the sub-bottom may be composed of fine lagoonal muds under the surficial sands.

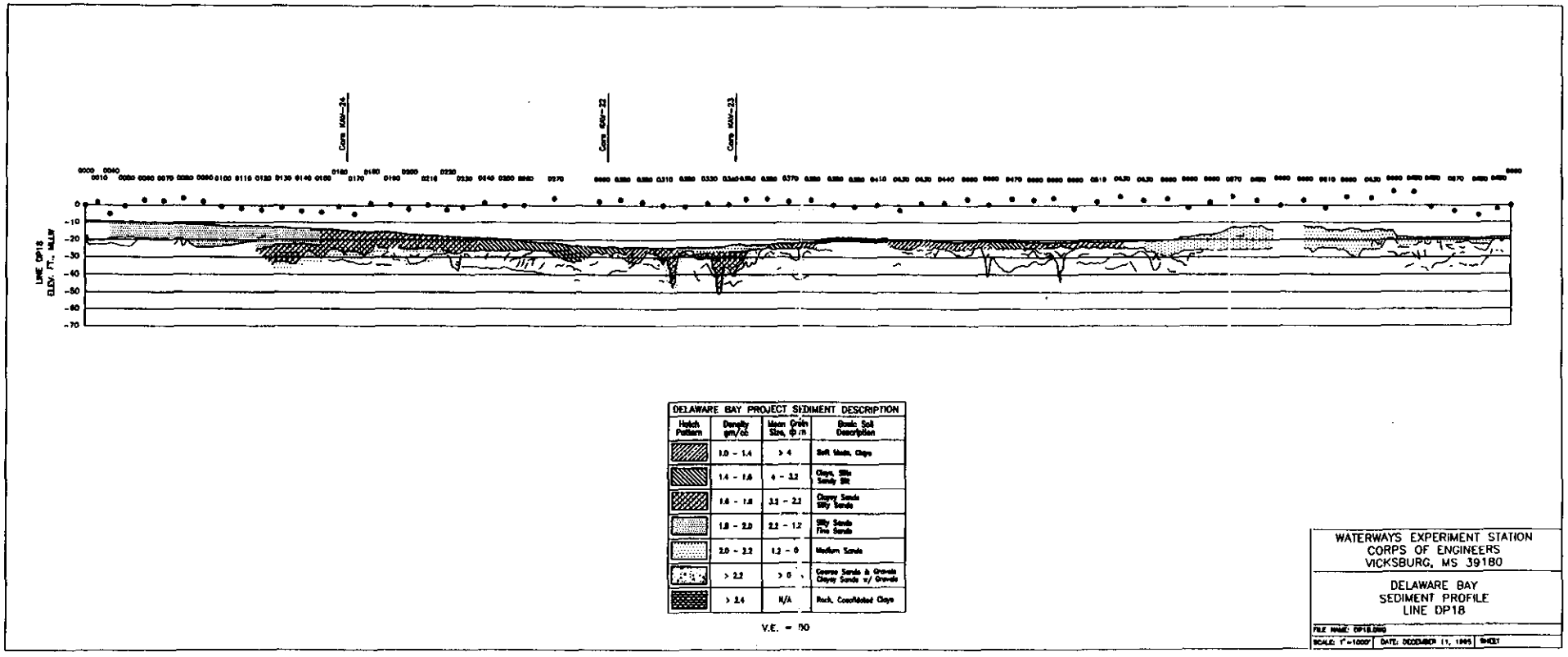
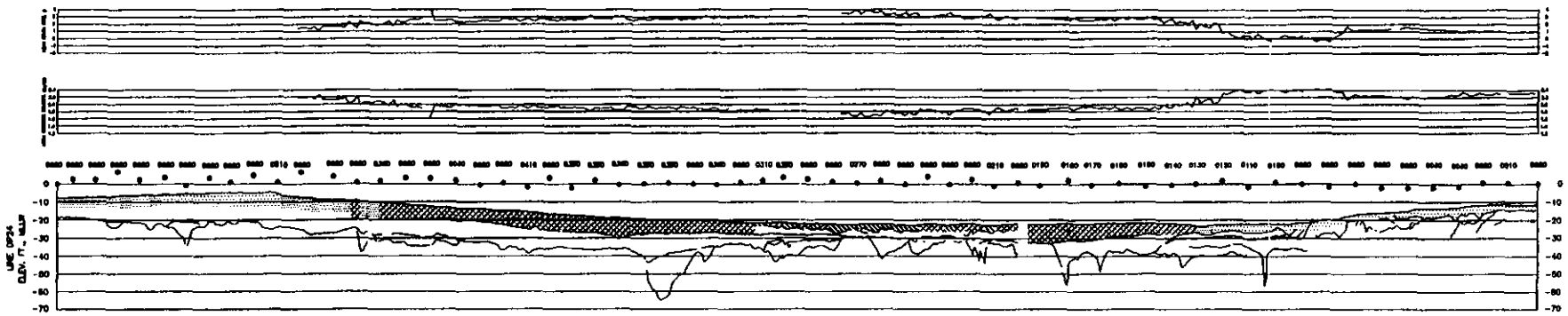


Figure 9. Cross-section of Line DP18 with core locations



DELAWARE BAY PROJECT SEDIMENT DESCRIPTION			
Hatch Pattern	Density gm/cc	Mean Grain Size, ϕ m	Basic Soil Description
	1.0 - 1.4	> 4	Soft Muds, Clays
	1.4 - 1.8	4 - 3.2	Clays, Silts Sandy Silt
	1.8 - 1.8	3.2 - 2.2	Clayey Sands Silty Sands
	1.8 - 2.0	2.2 - 1.2	Silty Sands Fine Sands
	2.0 - 2.2	1.2 - 0	Medium Sands
	> 2.2	> 0	Coarse Sands & Gravels Clayey Sands w/ Gravels
	> 2.4	N/A	Rock, Consolidated Clays

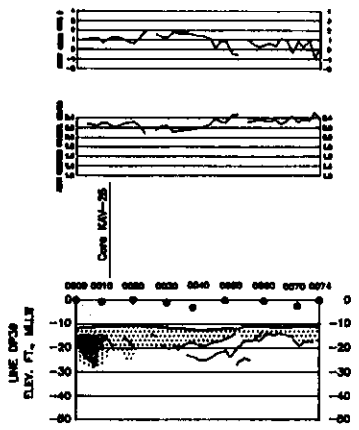
V.E. = 50

Figure 10. Cross-section of Line DP24

WATERWAYS EXPERIMENT STATION
CORPS OF ENGINEERS
VICKSBURG, MS 39180

DELAWARE BAY PROJECT
SEDIMENT PROFILE
LINE DP24

FILE NAME: DP24.DWG
SCALE: 1"=1000' DATE: DECEMBER 11, 1995 SHEET



DELAWARE BAY PROJECT SEDIMENT DESCRIPTION			
Hatch Pattern	Density gm/cc	Mean Grain Size, Φ m	Basic Soil Description
	1.0 – 1.4	> 4	Soft Muck, Clays
	1.4 – 1.8	4 – 3.2	Clays, Silts Sandy Silt
	1.8 – 1.8	3.2 – 2.2	Clayey Sands Silty Sands
	1.8 – 2.0	2.2 – 1.2	Silty Sands Fine Sands
	2.0 – 2.2	1.2 – 0	Medium Sands
	> 2.2	> 0	Coarse Sands & Gravels Clayey Sands w/ Gravels
	> 2.4	N/A	Rock, Consolidated Clays

V.E. = 50

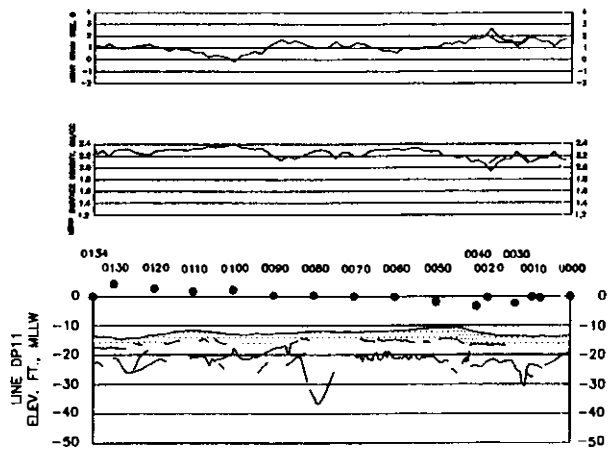
WATERWAYS EXPERIMENT STATION
CORPS OF ENGINEERS
VICKSBURG, MS 39180

DELAWARE BAY PROJECT
SEDIMENT PROFILE
LINE DP39

FILE NAME: DP39.DWG

SCALE: 1"=1000' | DATE: DECEMBER 11, 1986 | SHEET

Figure 11. Cross-section of Line DP39 with core location



DELAWARE BAY PROJECT SEDIMENT DESCRIPTION			
Hatch Pattern	Density gm/cc	Mean Grain Size, Φ m	Basic Soil Description
	1.0 ~ 1.4	> 4	Soft Muds, Clays
	1.4 - 1.6	4 - 3.2	Clays, Silts Sandy Silt
	1.6 - 1.8	3.2 - 2.2	Clayey Sands Silty Sands
	1.8 - 2.0	2.2 ~ 1.2	Silty Sands Fine Sands
	2.0 - 2.2	1.2 - 0	Medium Sands
	> 2.2	> 0	Coarse Sands & Gravels Clayey Sands w/ Gravels
	> 2.4	N/A	Rock, Consolidated Clays

V.E. = 50

WATERWAYS EXPERIMENT STATION
CORPS OF ENGINEERS
VICKSBURG, MS 39180

DELAWARE BAY PROJECT
SEDIMENT PROFILE
LINE DP11

FILE NAME: DP11.DWG
SCALE: 1" = 1000' | DATE: DECEMBER 11, 1995 | SHEET

Figure 12. Cross-section of Line DP11

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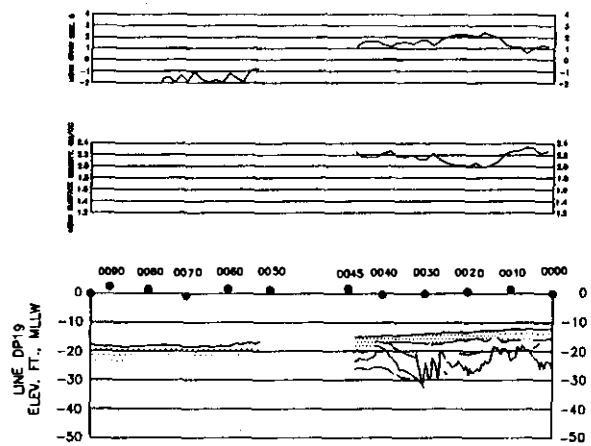
Line DP19

Parallel to line DP11 but further bayward was line DP19. **Figure 13** shows the cross-section of Line DP19. No cores were collected along this line and some data was missing from the center of the line. The acoustic signature indicates fine to medium sands in the surficial layer. Again the underlying layers are probably the lagoonal mud deposit of Maley (1981).

Line DP 25

The most bayward seismic line of this group was Line DP25. **Figure 14** shows the cross-section of Line DP25 with core KAV-26. The surficial sediments increase in grain-size from fine silty sand on the landward end to medium sand on the shoal on the bayward end of the line. Core KAV-26 was located on the northeastern end of the line, on the southern edge of the sand shoal next to the ancestral Primehook River Valley. Most of the core contains well sorted sands, with some silt. Some gravel material was found in the lower section of the core.



DELAWARE BAY PROJECT SEDIMENT DESCRIPTION			
Hatch Pattern	Density gm/cc	Mean Grain Size, ϕ m	Basic Soil Description
	1.0 - 1.4	> 4	Soft Muds, Clays
	1.4 - 1.6	4 - 3.2	Clays, Silts Sandy Silt
	1.6 - 1.8	3.2 - 2.2	Clayey Sands Silty Sands
	1.8 - 2.0	2.2 - 1.2	Silty Sands Fine Sands
	2.0 - 2.2	1.2 - 0	Medium Sands
	> 2.2	> 0	Coarse Sands & Gravels Clayey Sands w/ Gravels
	> 2.4	N/A	Rock, Consolidated Clays

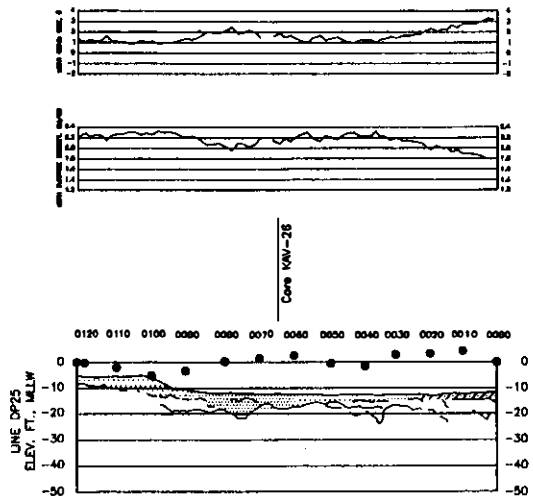
V.E. = 50

WATERWAYS EXPERIMENT STATION
CORPS OF ENGINEERS
VICKSBURG, MS 39180

DELAWARE BAY PROJECT
SEDIMENT PROFILE
LINE DP19

FILE NAME: DP19.DWG
SCALE: 1"=1000' | DATE: DECEMBER 11, 1995 | SHEET

Figure 13. Cross-section of Line DP19



DELAWARE BAY PROJECT SEDIMENT DESCRIPTION			
Hatch Pattern	Density gm/cc	Mean Grain Size, ϕ m	Basic Soil Description
	1.0 - 1.4	> 4	Soft Mude, Clays
	1.4 - 1.6	4 - 3.2	Clays, Silts Sandy Silt
	1.6 - 1.8	3.2 - 2.2	Clayey Sands Silty Sands
	1.8 - 2.0	2.2 - 1.2	Silty Sands Fine Sands
	2.0 - 2.2	1.2 - 0	Medium Sands
	> 2.2	> 0	Coarse Sands & Gravels Clayey Sands w/ Gravels
	> 2.4	N/A	Rock, Consolidated Clays

V.E. = 50

Figure 14. Cross-section of Line DP25 with core location

WATERWAYS EXPERIMENT STATION
CORPS OF ENGINEERS
VICKSBURG, MS 39180

DELAWARE BAY PROJECT
SEDIMENT PROFILE
LINE DP25

FILE NAME: DP25.DWG
SCALE: 1"=1000' | DATE: DECEMBER 11, 1995 | SHEET

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4 Summary

Sediment in the lower Delaware Bay is controlled by the geologic processes of the Pleistocene regression and transgression and is modified by the present day coastal processes. The area of potential sand resources has been identified using seismic survey profiles and vibracores. The area identified as possible borrow areas is outlined in **Figure 15**. The thickness ranges from 3 ft on the inshore end to possibly 10 ft on the shoal areas offshore. The area is characterized by two shoal areas divided by valleys that correspond to pre-Holocene river valleys. These valleys are composed of fine material not suited for beach nourishment. The shoal areas are topographic highs and contain medium to fine sands, that are suitable for consideration to nourish Broadkill Beach.

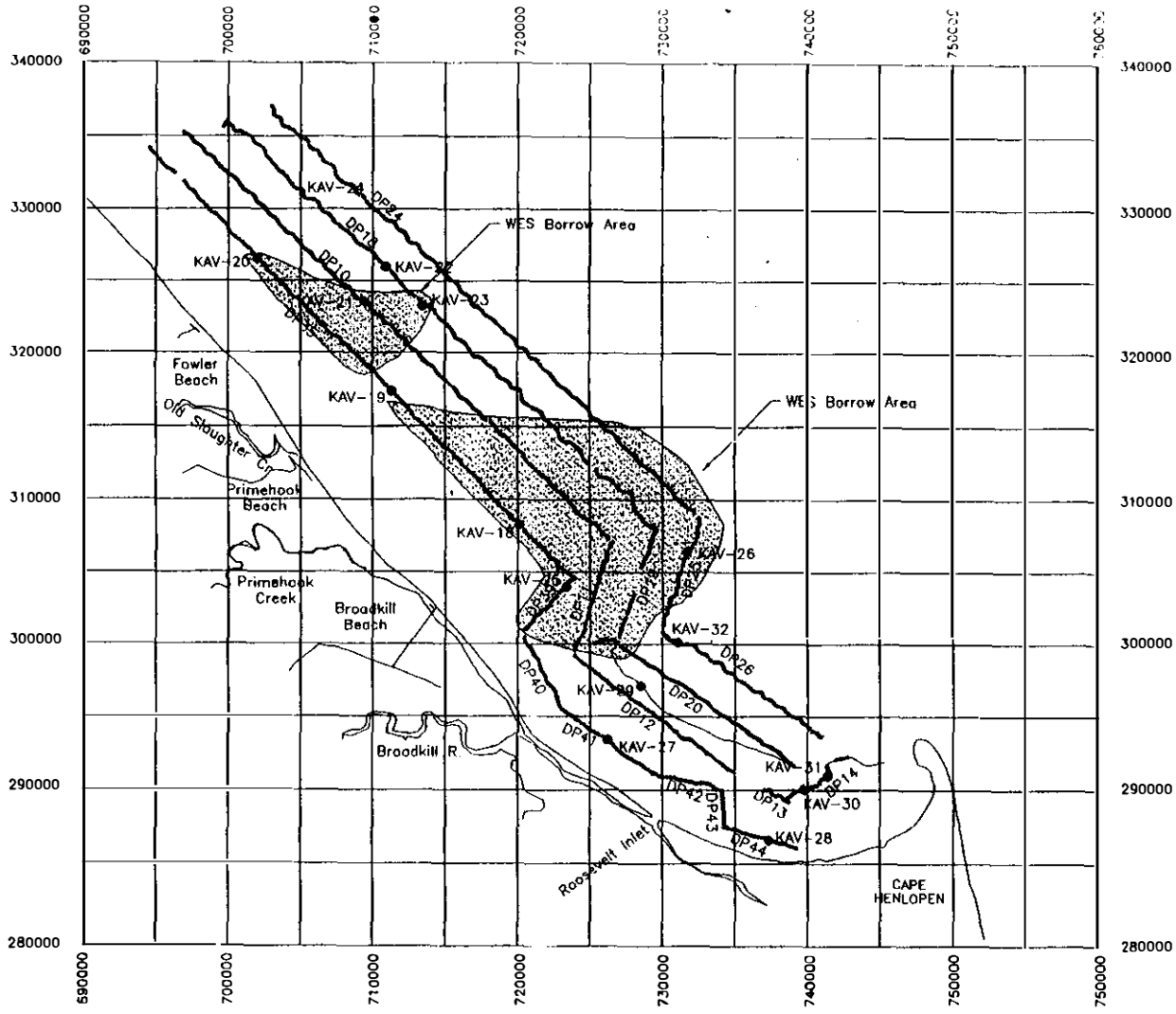

Figure 15. Area of potential sand resources.

Acknowledgements

This report was produced for the U.S. Army Engineer District, Philadelphia. Seismic field data collection was done aboard the R/V Waterways Explorer (WES). Data collection was assisted by Mr. Rodney Leist, and Mr. Tom Harmon of the Geotechnical Laboratory (WES), Mr. Sam Varnell (HL) and Mr. David Caulfield, Caulfield Engineering Group. Assistance in data reduction were done by Ms. Clare Livingston, and Ms. Darla McVan and Mr. Brian Williams provided the graphics support.

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Appendix A
Core logs and gradation curves

Appendix A contains the core logs and gradation curves supplied by the U.S. Army Corps of Engineers, South Atlantic Division Sediment Laboratory. The core loges list the core length, material description and the location of the sediment samples collected within the core for grain-size analysis and selected samples that were analyzed for density. The gradation curves are provided for each sample analyzed within each core. The sample number and depth range of the channel sample correspond to the listing on the core log comments column.

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE, MARIETTA, GA. 30060

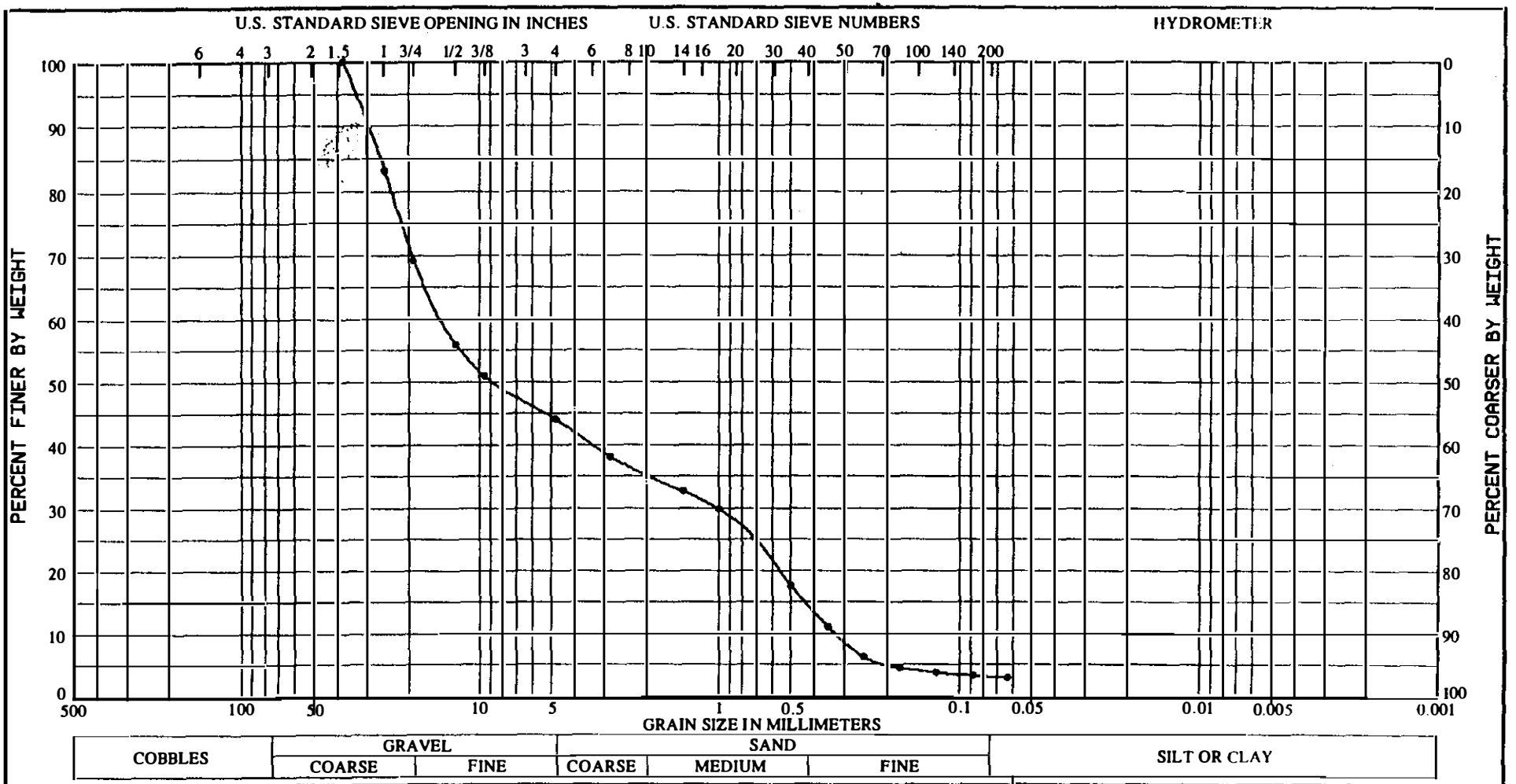
Project: *DELAWARE BAY*		Boring No. *KAV-18*	
Location: *VIBRA CORE SAMPLES*		Lab No. *184/925*	
Boring Depth (ft): *15.20*	Elevation:	Work order: *7476*	
Datum/Notes: *See grain size data on enclosed gradation curves.*		Requisition: *CENAP-95-707*	

Elev. (feet)	Depth (feet)	Leg-end	Material Description	Comments
				(Density Units = pcf)
	1		TAN, SANDY POORLY GRADED GRAVEL (GP).	SA (A) 1.2 - 2.2'
	2			
	3		----------	SA D-1 3.0 - 3.5' WET DEN. = 128.9, DRY DEN. = 119.1, MC = 8.2%
	4		TAN, POORLY GRADED SILTY SAND (SP-SM).	
	5		----------	
	6			
	7			
	8		TAN, GRAVELLY POORLY GRADED SILTY SAND (SP-SM), WITH A TRACE OF MICA.	SA D-2 7.5 - 8.0' WET DEN. = 139.5, DRY DEN. = 132.7, MC = 6.0%
	9			
	10			
	11		----------	
	12		TAN AND GRAY, SILTY SAND (SM), WITH A LITTLE GRAVEL SIZES.	SA (B) 11.9 - 12.3'
	13		----------	SA D-3 13.3 - 13.9' WET DEN. = 127.1, DRY DEN. = 102.4 MC = 24.2%
	14		TAN AND GRAY, MICACEOUS INORGANIC SILT LOW LL (ML), WITH A LITTLE SAND SIZES, SAND SIZES MORE PREDOMINANT BOTTOM 0.4 OF SAMPLE.	
	15		----------	
	16			
	17			

Date: *01/19/95* **LABORATORY LOG AND SAMPLE DATUM** Sheet No. *1* of *1*

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Sample No.	Depth	Classification	Nat w%	LL	PL	PI
A	1.2-2.2'	(VISUAL) TAN, SANDY POORLY GRADED GRAVEL (GP).				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/925A
Boring No. KAV-18
Date 01/18/95

GRADATION CURVES

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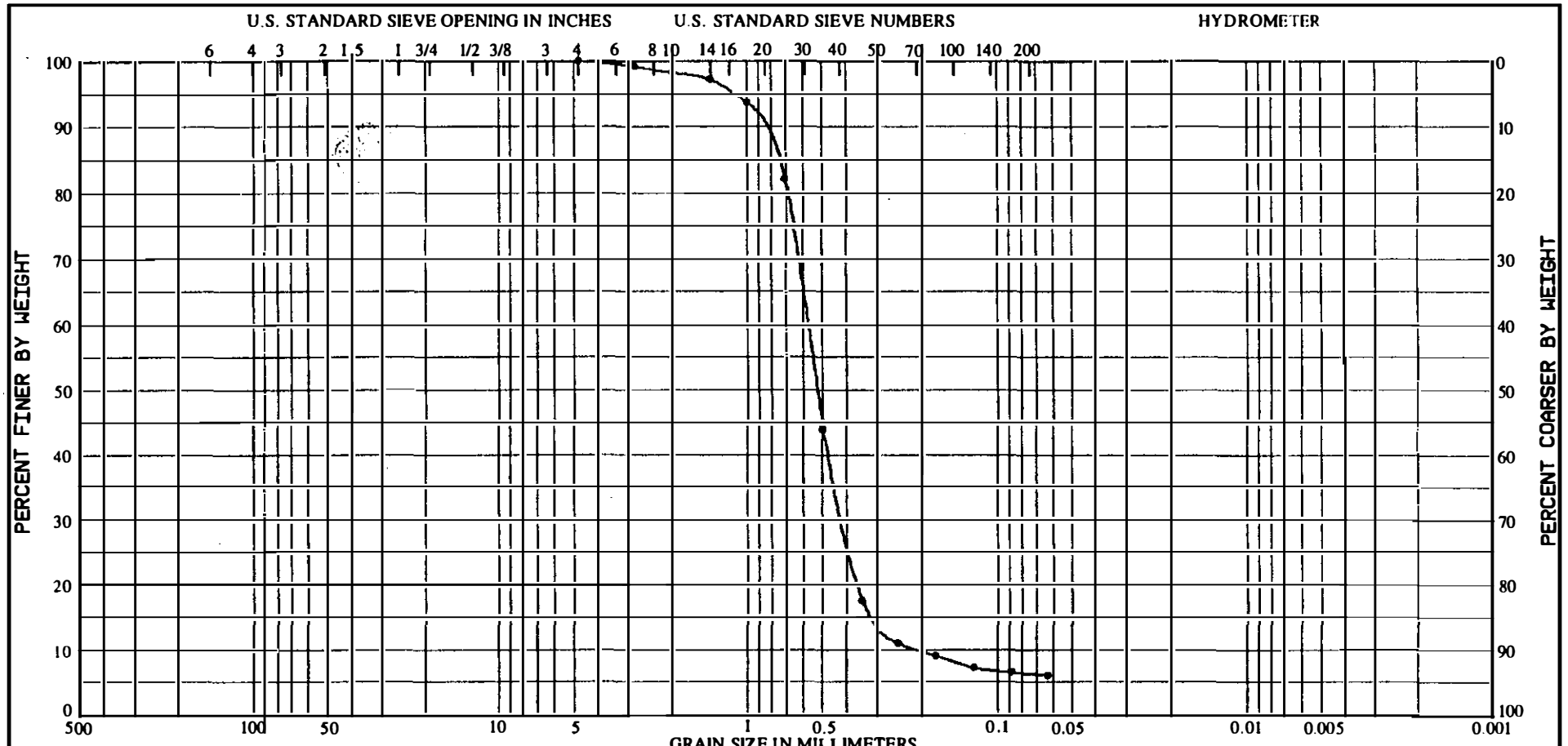
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Sample No.	Depth	Classification	Nat w%	LL	PL	PI
D-I	3.0-3.5'	(VISUAL) TAN. POORLY GRADED SILTY SAND	8.2			
		(SP-SM).				
		SP. GRAVITY = 2.64				
		WET DENSITY = 128.9 PCF.				
		DRY DENSITY = 119.1 PCF.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/925
Boring No. KAV-18
Date 01/18/95

GRADATION CURVES

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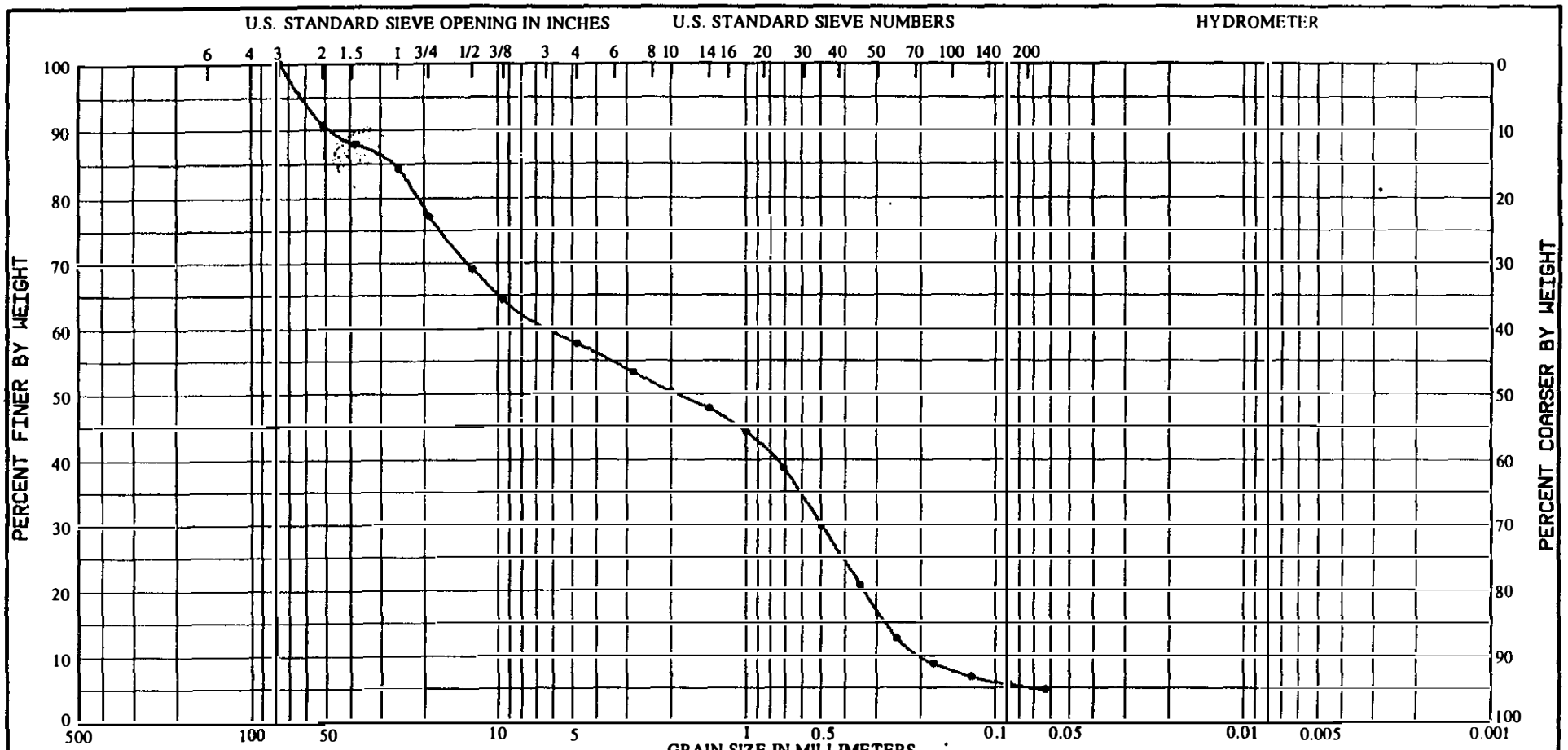
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Sample No.	Depth	Classification	Nat w%	LL	PL	PI
D-2	7.5-8.0'	(VISUAL) TAN, GRAVELLY POORLY GRADED	6.0			
		SILTY SAND (SP-SM), WITH A TRACE OF MICA				
		SP. GRAVITY = 2.66				
		WEY DENSITY = 139.5 PCF.				
		DRY DENSITY = 132.7 PCF.				

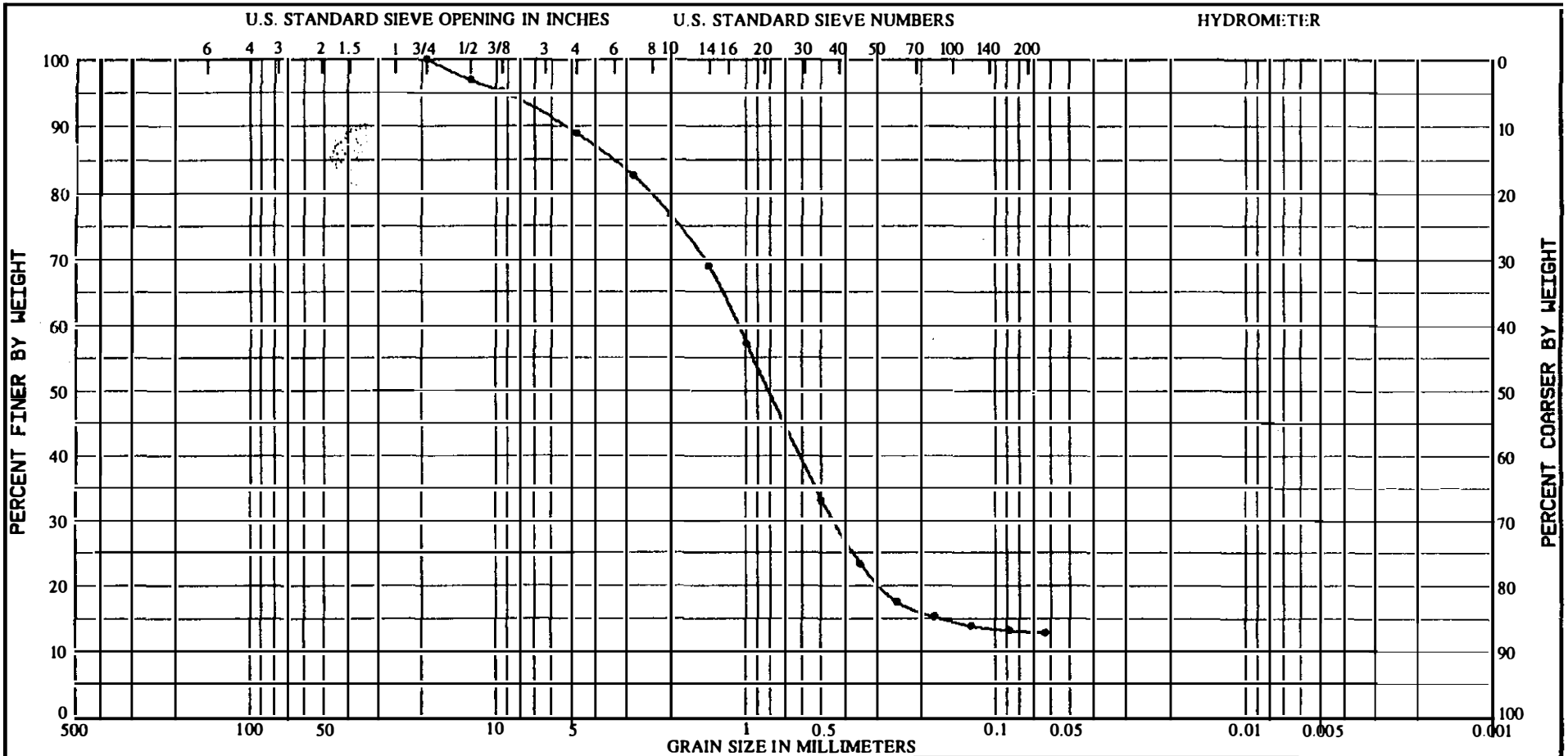
Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/925
Boring No. KAV-18
Date 01/18/95

GRADATION CURVES

32

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Sample No.	Depth	Classification	Nat w%	LL	PL	PI
B	11.9-12.3'	(VISUAL) TAN AND GRAY, SILTY SAND (SM), WITH A LITTLE GRAVEL SIZES.				

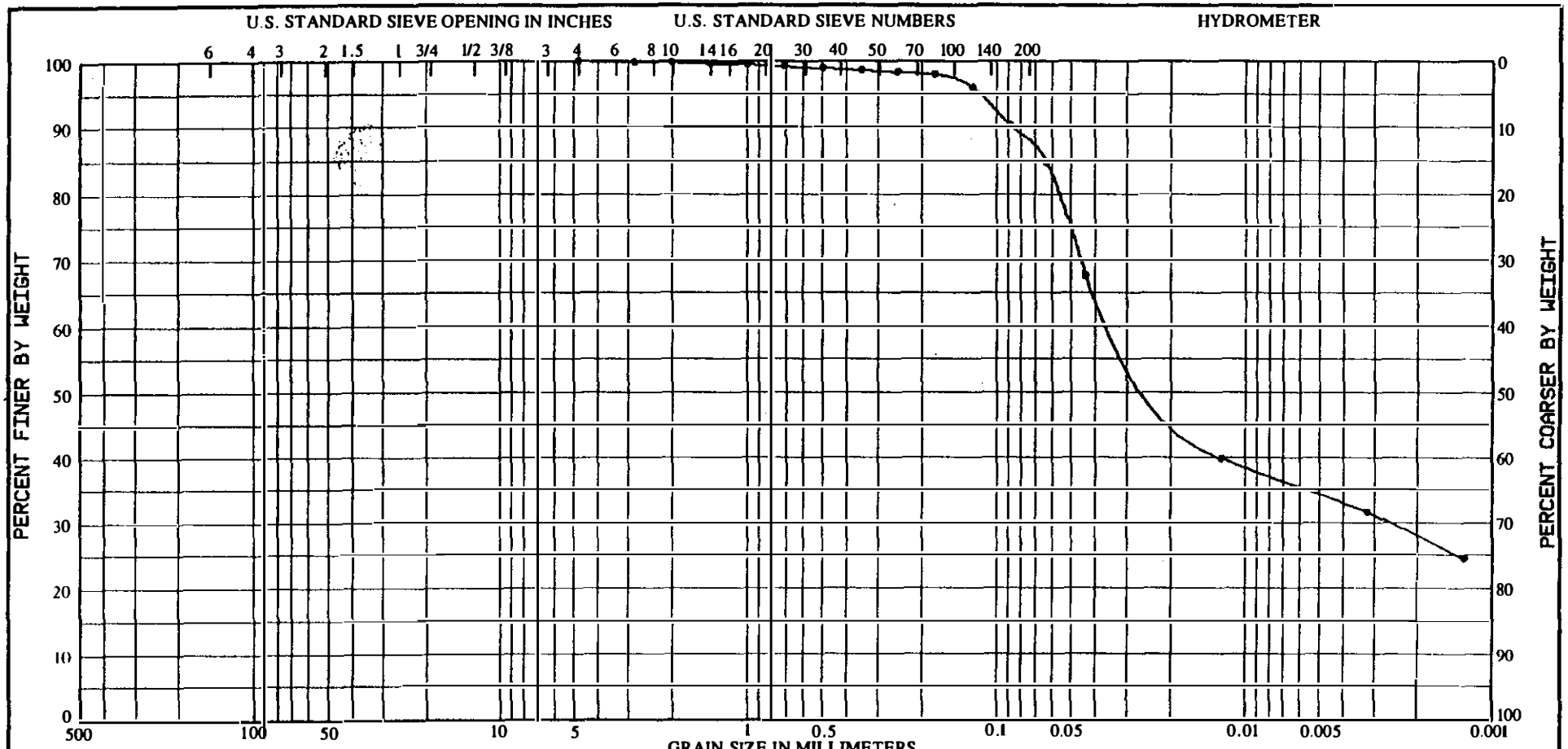
Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/925B
Boring No. KAV-18
Date 01/18/95

GRADATION CURVES

33

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Sample No.	Depth	Classification	Nat w%	LL	PL	PI
D-3	13.3-13.9'	(VISUAL) TAN & GRAY, MICACEOUS INORGANIC	24.2			
		SILT LOW LL (ML), WITH A LITLE SAND SIZES.				
		SP. GRAVITY = 2.69				
		WET DENSITY = 127.1 PCF.				
		DRY DENSITY = 102.4 PCF.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/925
Boring No. KAV-18
Date 01/19/95

GRADATION CURVES

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DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE, MARIETTA, GA. 30060

Project: *DELAWARE BAY*		Boring No. *KAV-19*	
Location: *VIBRA CORE SAMPLES*		Lab No. *184/926*	
Boring Depth (ft): *17.60*	Elevation:	Work order: *7476*	
Datum/Notes: *See grain size data on enclosed gradation curves.*		Requisition: *CENAP-95-707*	

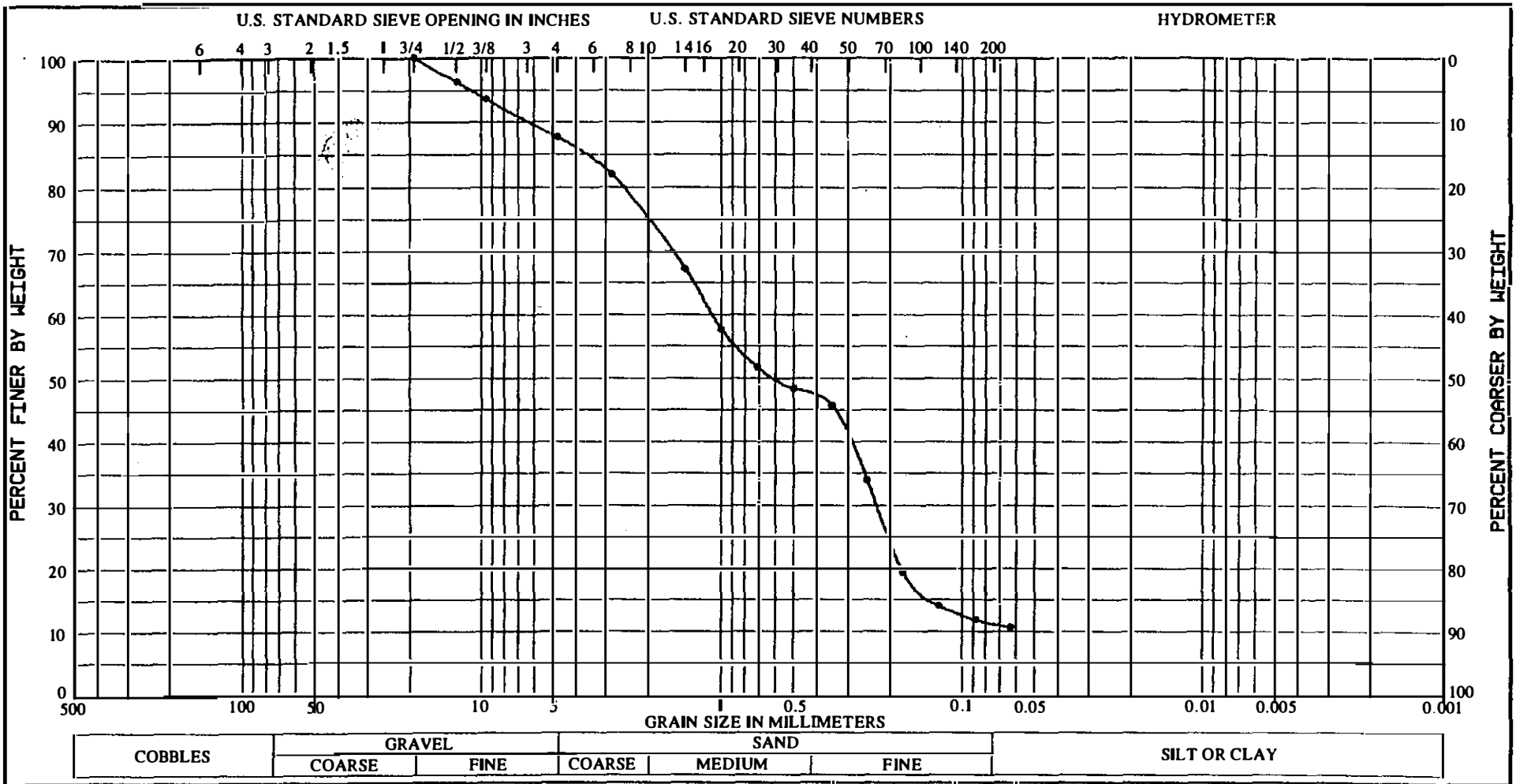
Elev. (feet)	Depth (feet)	Leg-end	Material Description	Comments
				(Density Units = pcf)
	1			
			TAN AND GRAY, POORLY GRADED SILTY SAND (SP-SM), WITH A LITTLE GRAVEL SIZES.	
	2			
				SA D-1 2.3 - 2.9 WET DEN. = 129.1, DRY DEN = 114.0, MC = 13.3 %.
	3		----------	
	4		TANNISH GRAY, SILTY SAND (SM).	SA (A) 3.7 - 4.0'
	5		----------	
	6			
			LT. GRAY AND TAN, SILTY SAND (SM), WITH A TRACE OF MICA.	
	7			SA D-2 7.1 - 7.6' WET DEN. = 111.0, DRY DEN. = 93.7, MC = 18.5 %.

	8			
	9			SA (B) 8.6 - 8.9'
	10			
	11			
	12		TAN AND TANNISH GRAY, SILTY SAND (SM), WITH A TRACE OF MICA.	
	13			SA D-3 12.5 - 13.0' WET DEN. = 120.4, DRY DEN = 99.0, MC = 21.6 %.
	14			SA (C) 13.7 - 14.0'
	15			
	16			SA D-4 16.0 - 16.5' WET DEN = 120.6, DRY DEN = 99.3, MC = 21.4 %.
	17			SA (D) 17.3 - 17.6'

Date: *01/25/95* **LABORATORY LOG AND SAMPLE DATUM** Sheet No. *1* of *1*

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



Sample No.	Depth	Classification	Nat w%	LL	PL	PI
D-1	2.3-2.9'	(VISUAL) TAN & GRAY, POORLY GRADED	13.3			
		SILTY SAND (SP-SM), WITH A LITTLE GRAVEL				
		SIZES. SP. GRAVITY = 2.63				
		WET DENSITY = 129.1 PCF.				
		DRY DENSITY = 114.0 PCF.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/926
Boring No. KAV-19
Date 01/19/95

GRADATION CURVES

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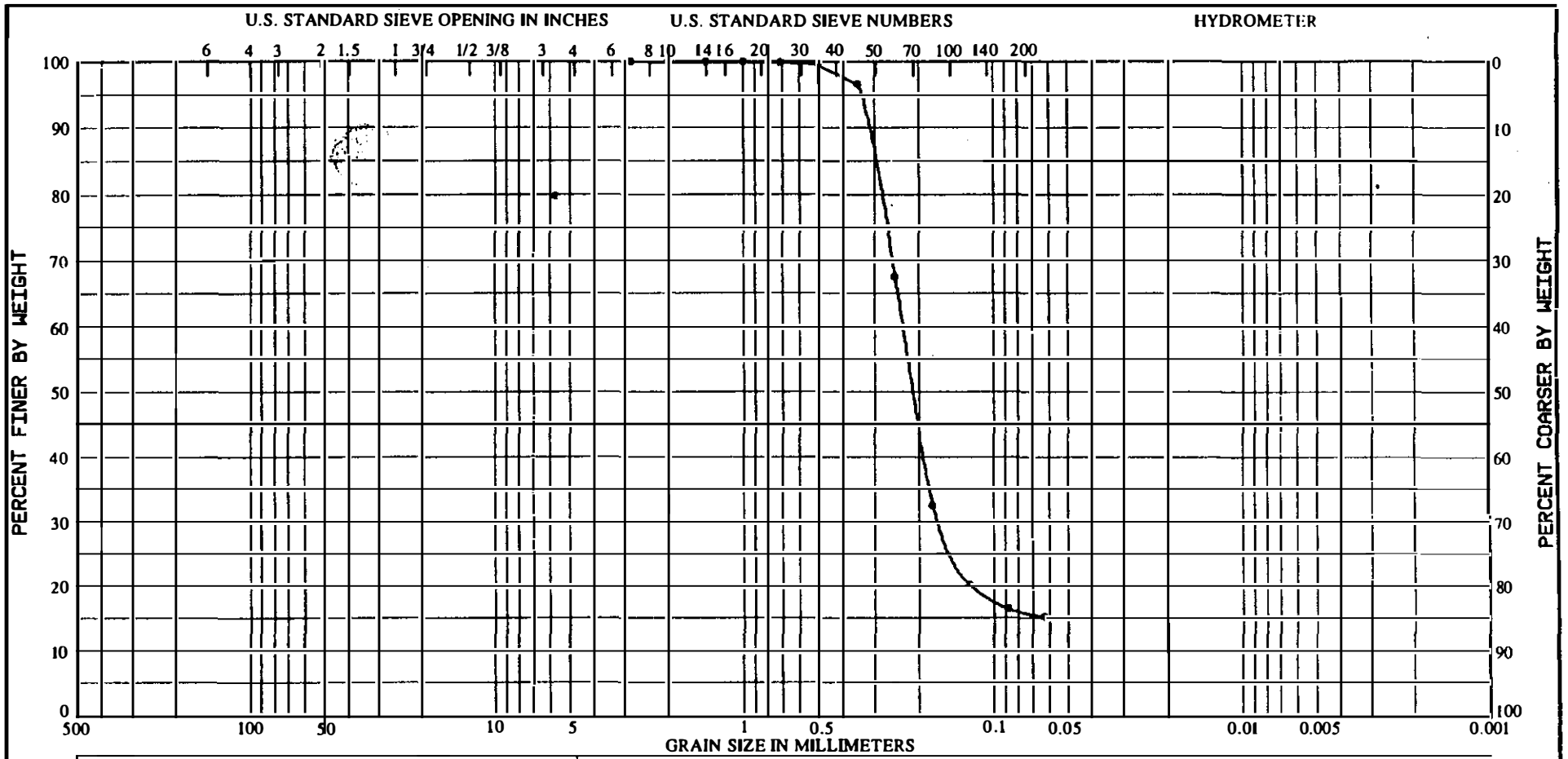
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



Sample No.	Depth	Classification	Nat w%	LL	PL	PI
A	3.7-4.0'	(VISUAL) TANNISH GRAY, SILTY SAND (SM).				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/926A
Boring No. KAV-19
Date 01/19/95

GRADATION CURVES

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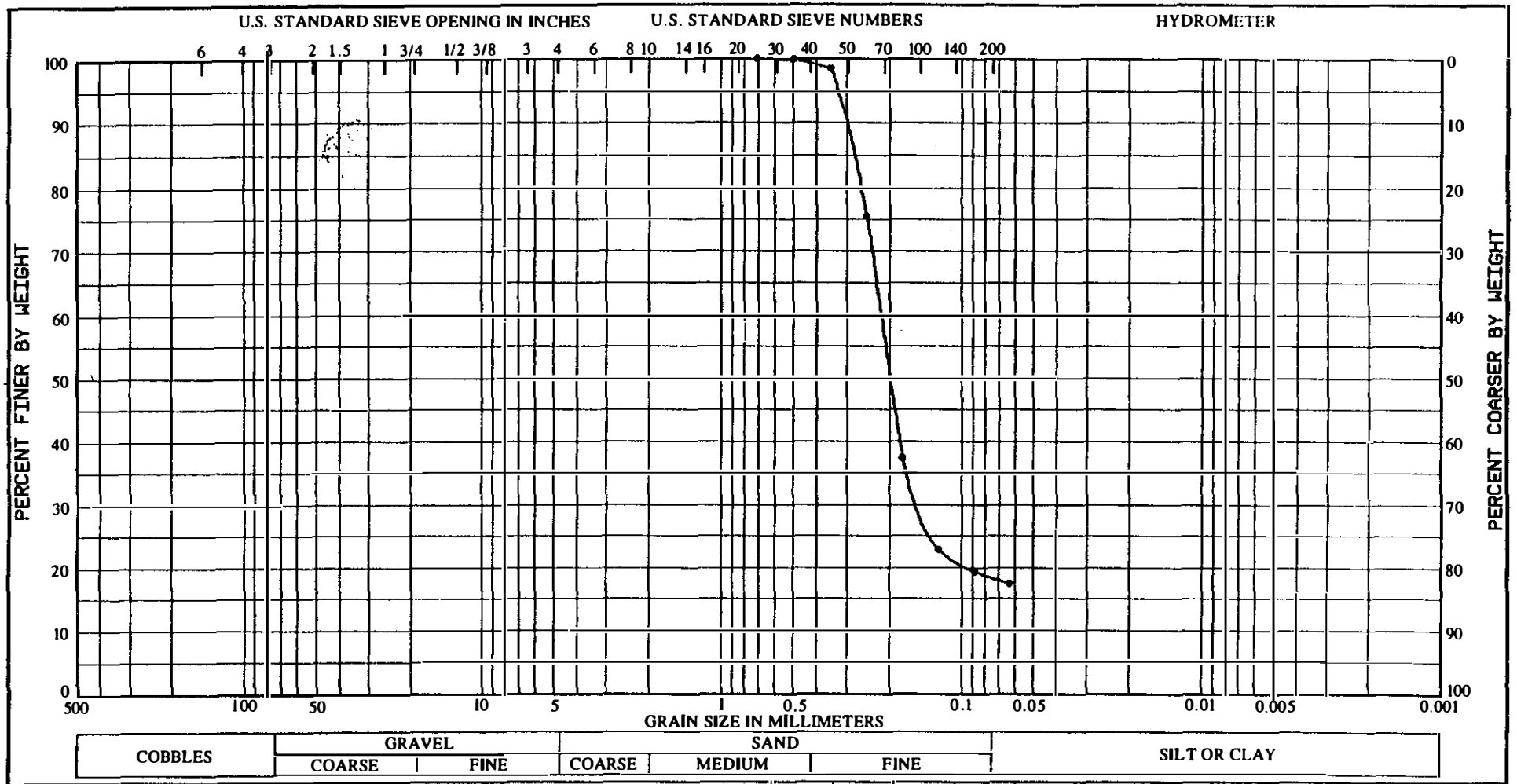
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



Sample No.	Depth	Classification	Nat w%	LL	PL	PI
D-2	7.1-7.6'	(VISUAL) LT. GRAY & TAN, SILTY SAND	18.5			
		(SM), WITH A TRACE OF MICA.				
		SP. GRAVITY = 2.64				
		WET DENSITY = 111.0 PCF.				
		DRY DENSITY = 93.7 PCF.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/926
Boring No. KAV-19
Date 01/19/95

GRADATION CURVES

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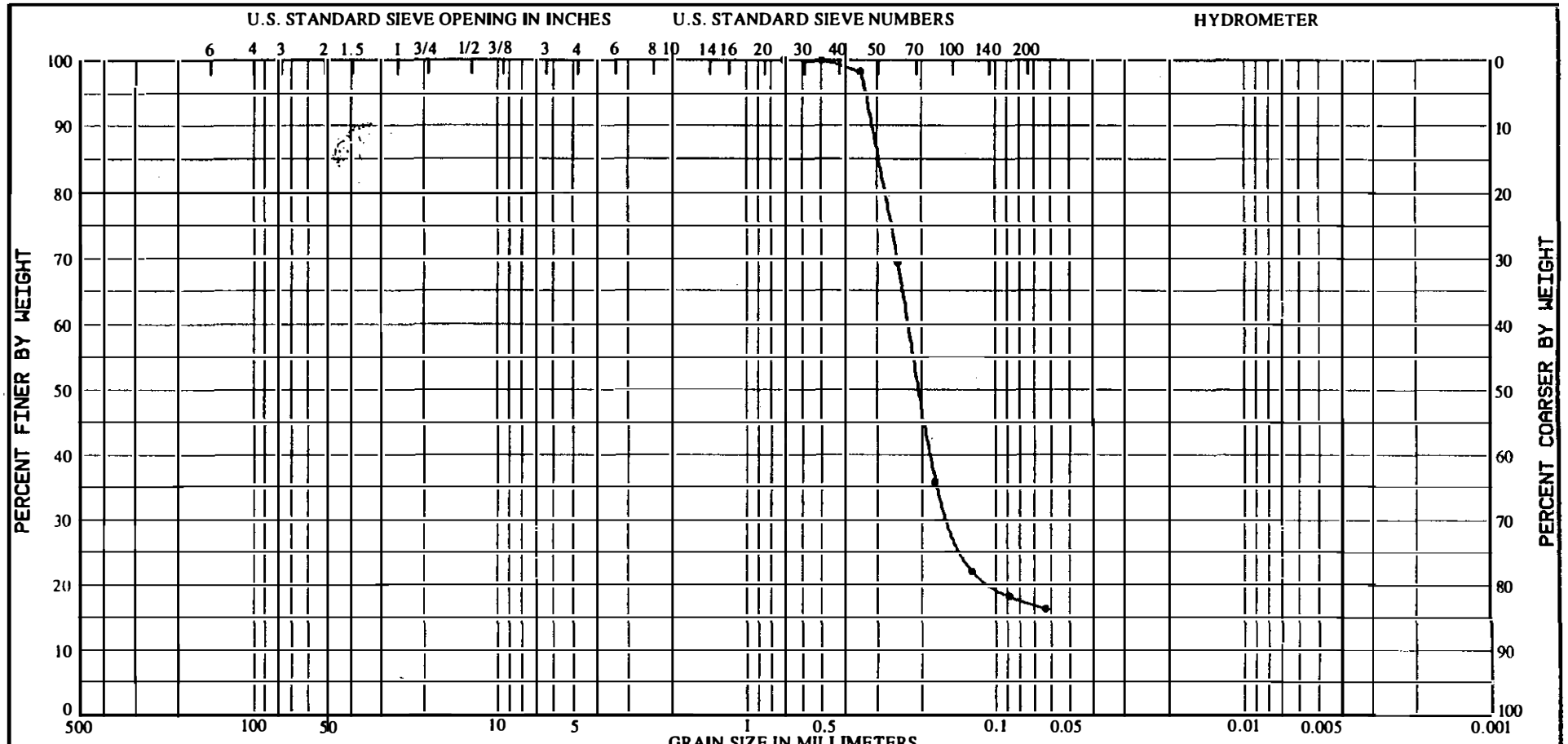
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Sample No.	Depth	Classification	Nat w%	LL	PL	PI
B	8.6-8.9'	(VISUAL) TAN & TANNISH GRAY, SILTY SAND (SM).				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/926B
Boring No. KAV-19
Date 01/19/95

GRADATION CURVES

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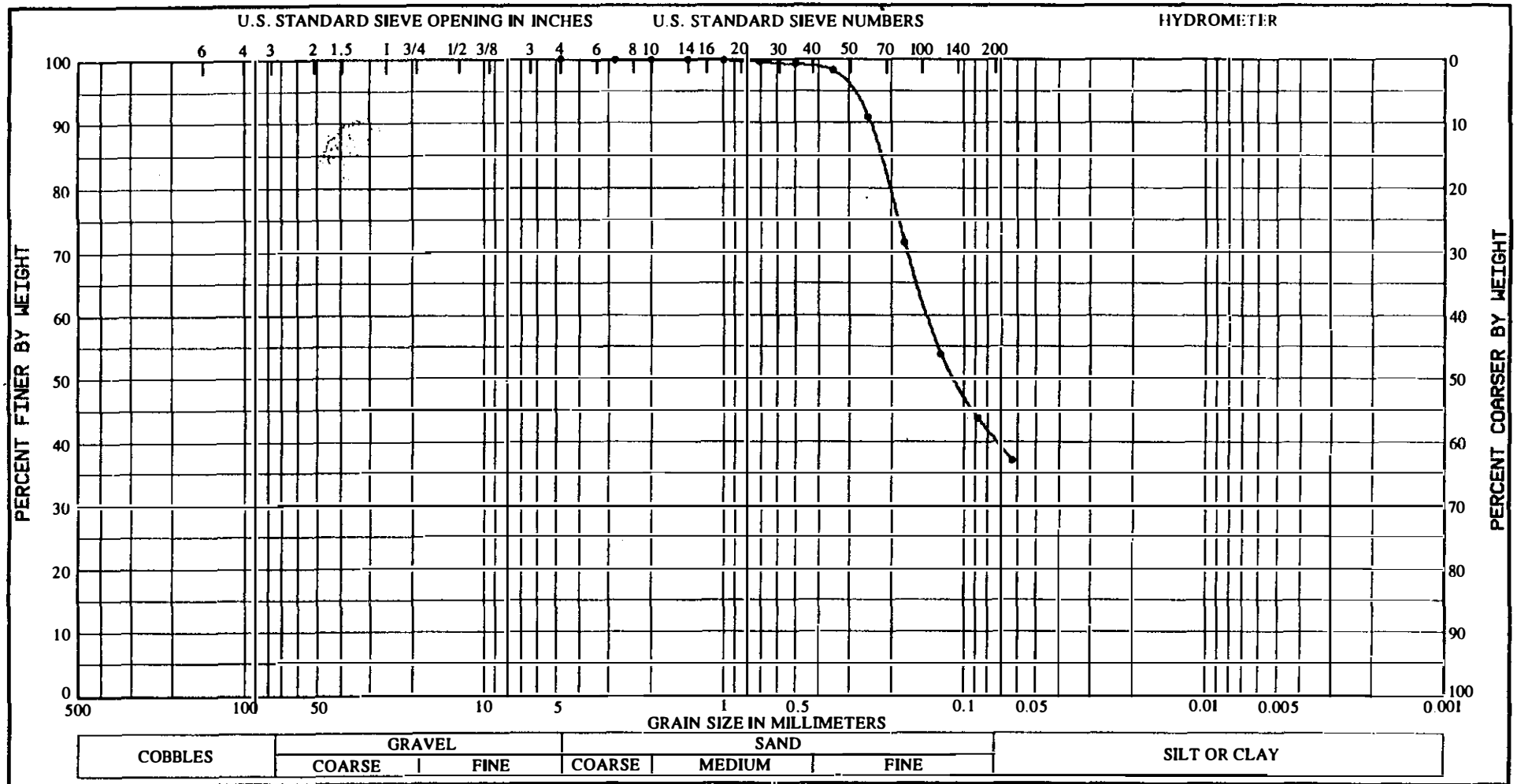
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



Sample No.	Depth	Classification	Nat w%	LL	PL	PI
D-3	12.5-13.0'	(VISUAL) TAN & TANNISH GRAY, SILTY SAND	21.6			
		SM), WITH A TRACE OF MICA.				
		SP. GRAVITY = 2.64				
		WET DENSITY = 120.4 PCF.				
		DRY DENSITY = 99.0 PCF.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/926
Boring No. KAV-19
Date 01/19/95

GRADATION CURVES

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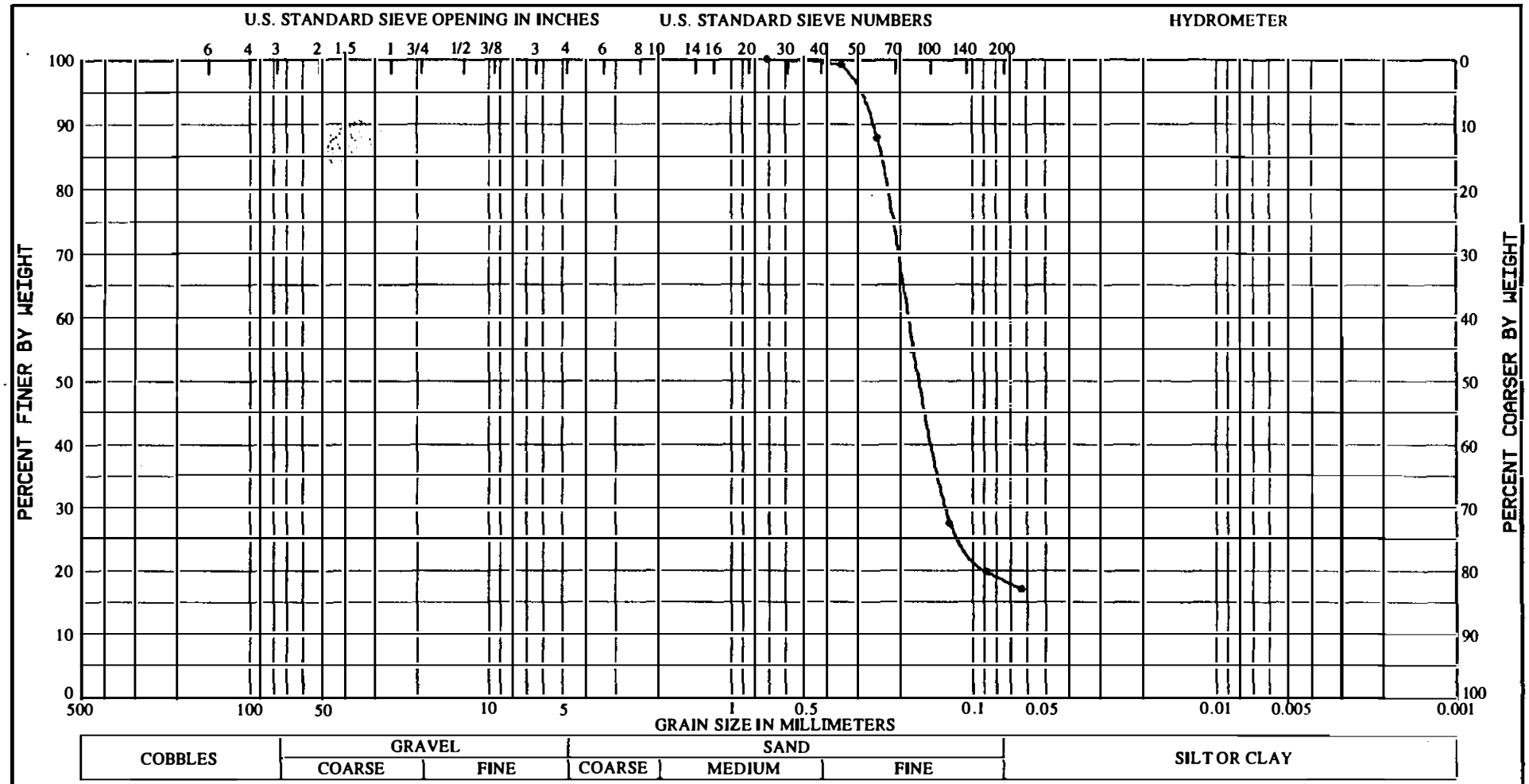
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



Sample No.	Depth	Classification	Nat w%	LL	PL	PI
C	13.7-14.0'	(VISUAL) TAN & TANNISH GRAY, SILTY SAND (SM).				

Project DELAWARE BAY
VIBRA CORE SAMPLES
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Boring No. KAV-19
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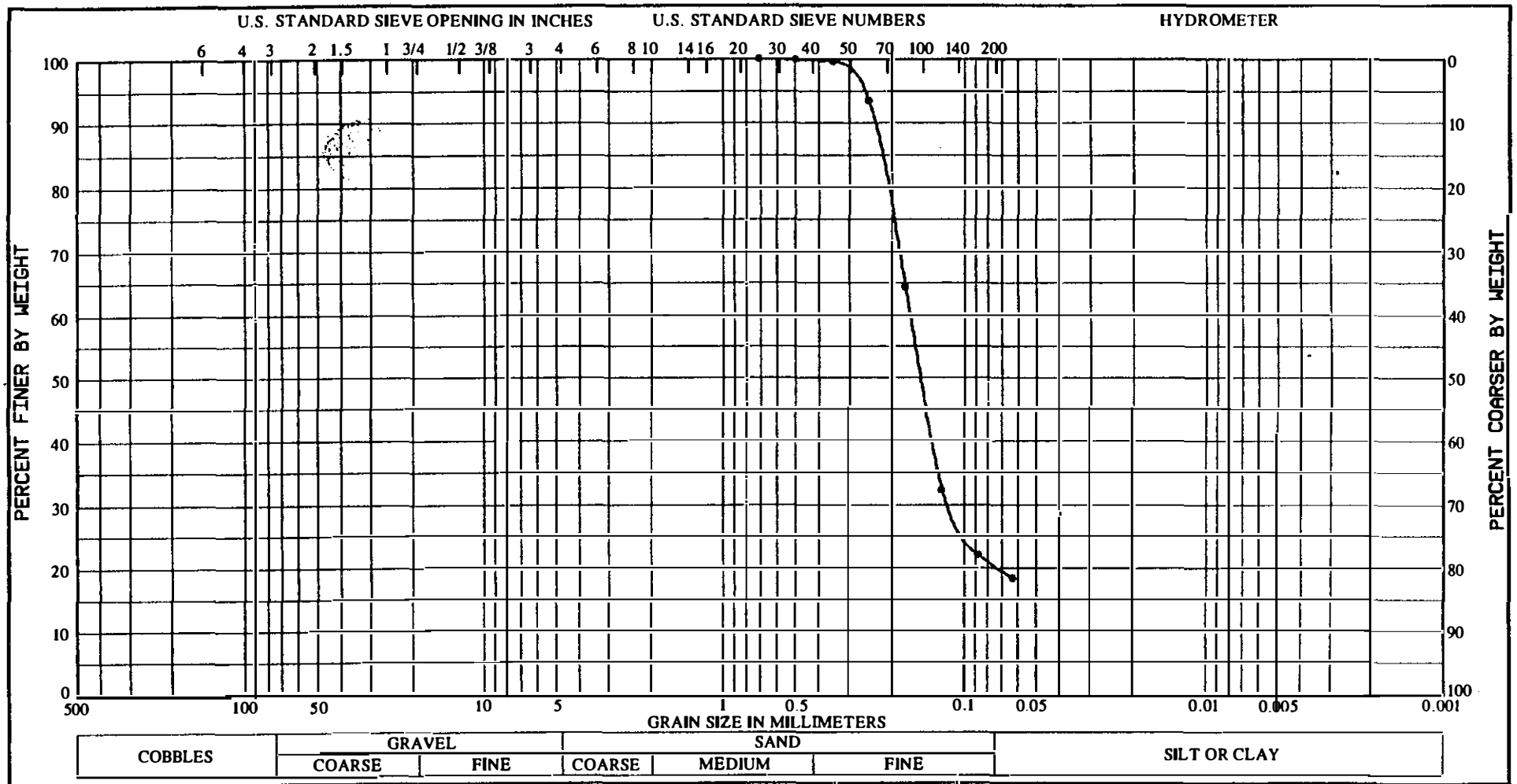
GRADATION CURVES

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



Sample No.	Depth	Classification	Nat w%	LL	PL	PI
D-4	16.0-16.5'	(VISUAL) TAN & TANNISH GRAY, SILTY SAND	21.4			
		(SM), WITH A TRACE OF MICA.				
		SP. GRAVITY = 2.64				
		WET DENSITY = 120.6 PCF.				
		DRY DENSITY = 99.3 PCF.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/926
Boring No. KAV-19
Date 01/19/95

GRADATION CURVES

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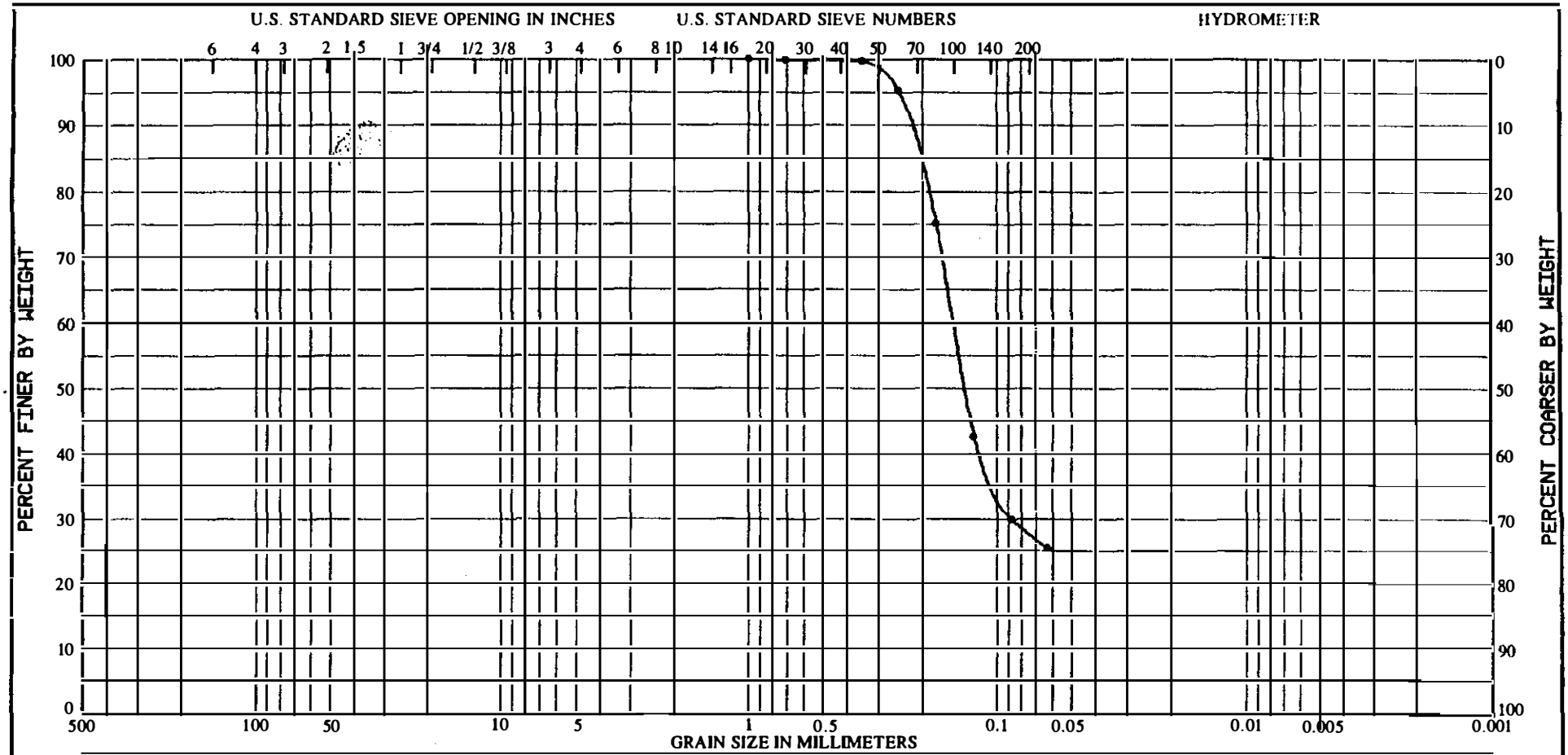
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



Sample No.	Depth	Classification	Nat w%	LL	PL	PI
D	17.3-17.6'	(VISUAL) TAN & TANNISH GRAY, SILTY SAND (SM).				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/926D
Boring No. KAV-19
Date 01/19/95

GRADATION CURVES

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DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE, MARIETTA, GA. 30060

Project: *DELAWARE BAY*		Boring No. *KAV-20*	
Location: *VIBRA CORE SAMPLES*		Lab No. *184/927*	
Boring Depth (ft): *16.00*	Elevation:	Work order: *7476*	
Datum/Notes: *See grain size data on enclosed gradation curves.*		Requisition: *CENAP-95-707*	

Elev. (feet)	Depth (feet)	Leg-end	Material Description	Comments
			GRAY, GRAVELLY SILTY SAND (SM).	
	1			
			TAN, LEAN CLAY (CL), WITH A LITTLE SAND SIZES AND A TRACE OF GRAVEL SIZES.	MA ATT (A) 1.3 - 1.6'
	2			
	3			MA D-1 2.5 - 3.0' WET DEN = 121.6, DRY DEN = 95.1, MC = 27.9 %.
	4			MA ATT (B) 4.0 - 4.3'
			TAN, LEAN CLAY (CL), WITH SOME SAND SIZES.	
	5			
	6			
			TAN, SANDY INORGANIC SILT LOW LL (ML).	
	7			MA D-2 7.3 - 7.8' WET DEN = 117.9, DRY DEN = 92.7, MC = 27.2 %.
	8			
	9			
	10			
	11			
	12			MA D-3 12.3 - 12.8' WET DEN = 119.3, DRY DEN = 96.1, MC = 24.2 %.
	13		LT. GRAY, SILTY SAND (SM).	
	14			MA (C) 14.6 - 14.9'
	15			
	16			
	17			

Date: *03/13/95* **LABORATORY LOG AND SAMPLE DATUM** Sheet No. *1* of *1*

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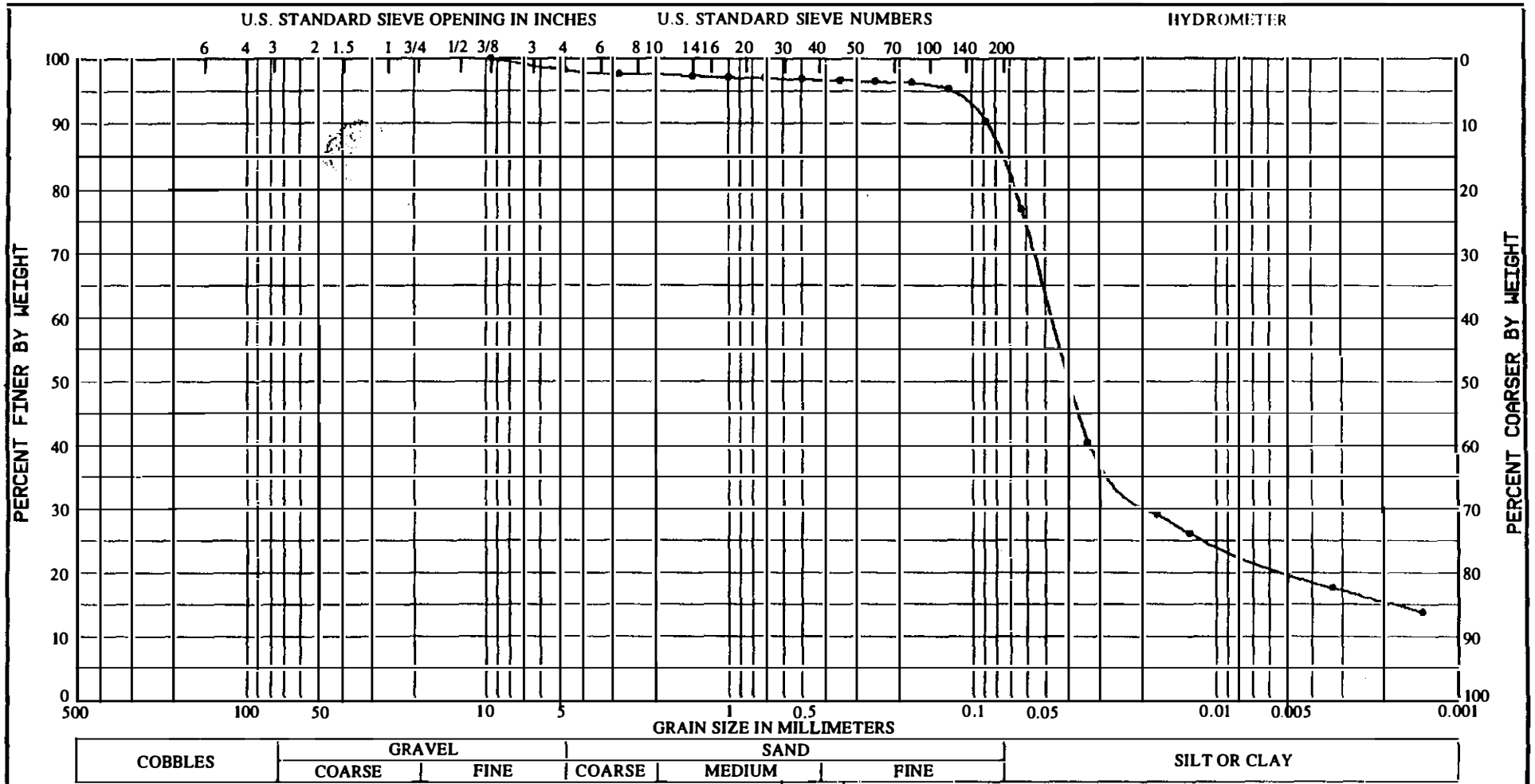
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



Sample No.	Depth	Classification	Nat w%	LL	PL	PI
A	1.3-1.6'	TAN. LEAN CLAY (CL). WITH A LITTLE SAND SIZES AND A TRACE OF GRAVEL SIZES.		32	18	14

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/927A
Boring No. KAV-20
Date 02/07/95

GRADATION CURVES

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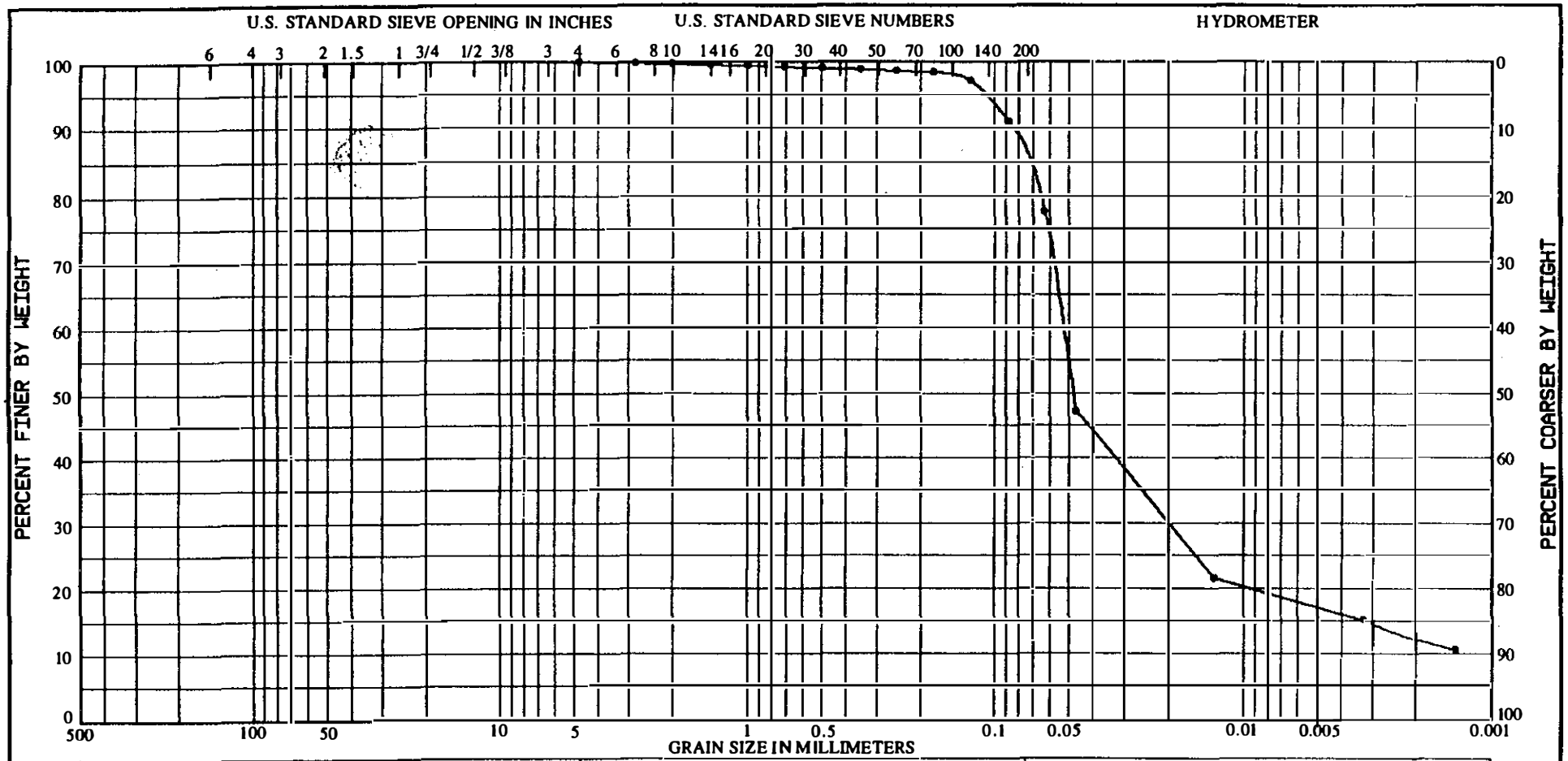
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Sample No.	Depth	Classification	Nat w %	LL	PL	PI
D-I	2.5-3.0'	(VISUAL) TAN, INORGANIC SILT LOW LL	27.9			
		(ML), MICACEOUS WITH A LITTLE SAND SIZES.				
		SP. GRAVITY = 2.66				
		WET DEN = 121.8 PCF.				
		DRY DENSITY = 95.1 PCF.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/927
Boring No. KAV-20
Date 02/07/95

GRADATION CURVES

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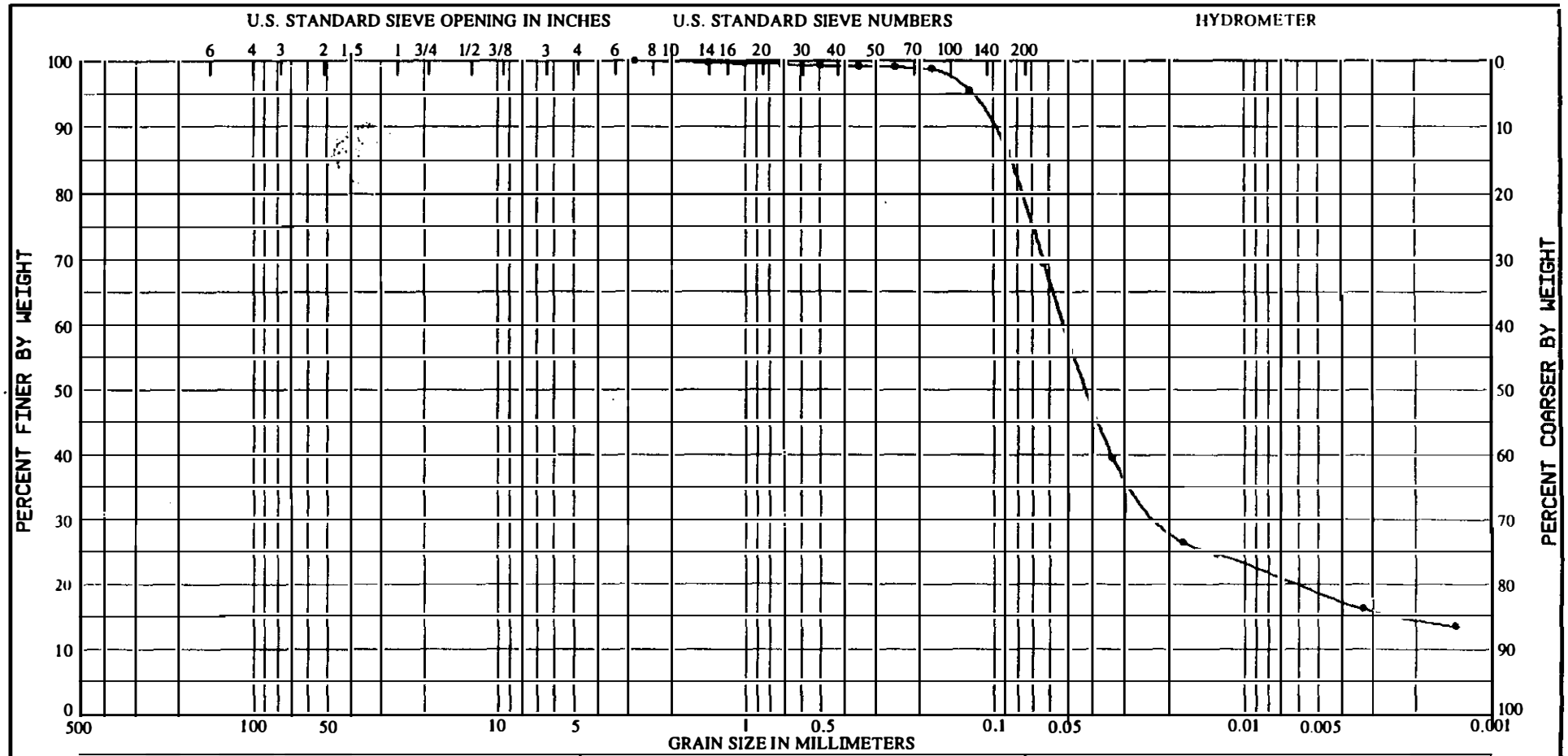
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Sample No.	Depth	Classification	Nat w%	LL	PL	PI
B	4.0-4.3'	TAN, LEAN CLAY (CL), WITH SOME SAND SIZES.		31	20	11

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/927B
Boring No. KAV-20
Date 02/07/95

GRADATION CURVES

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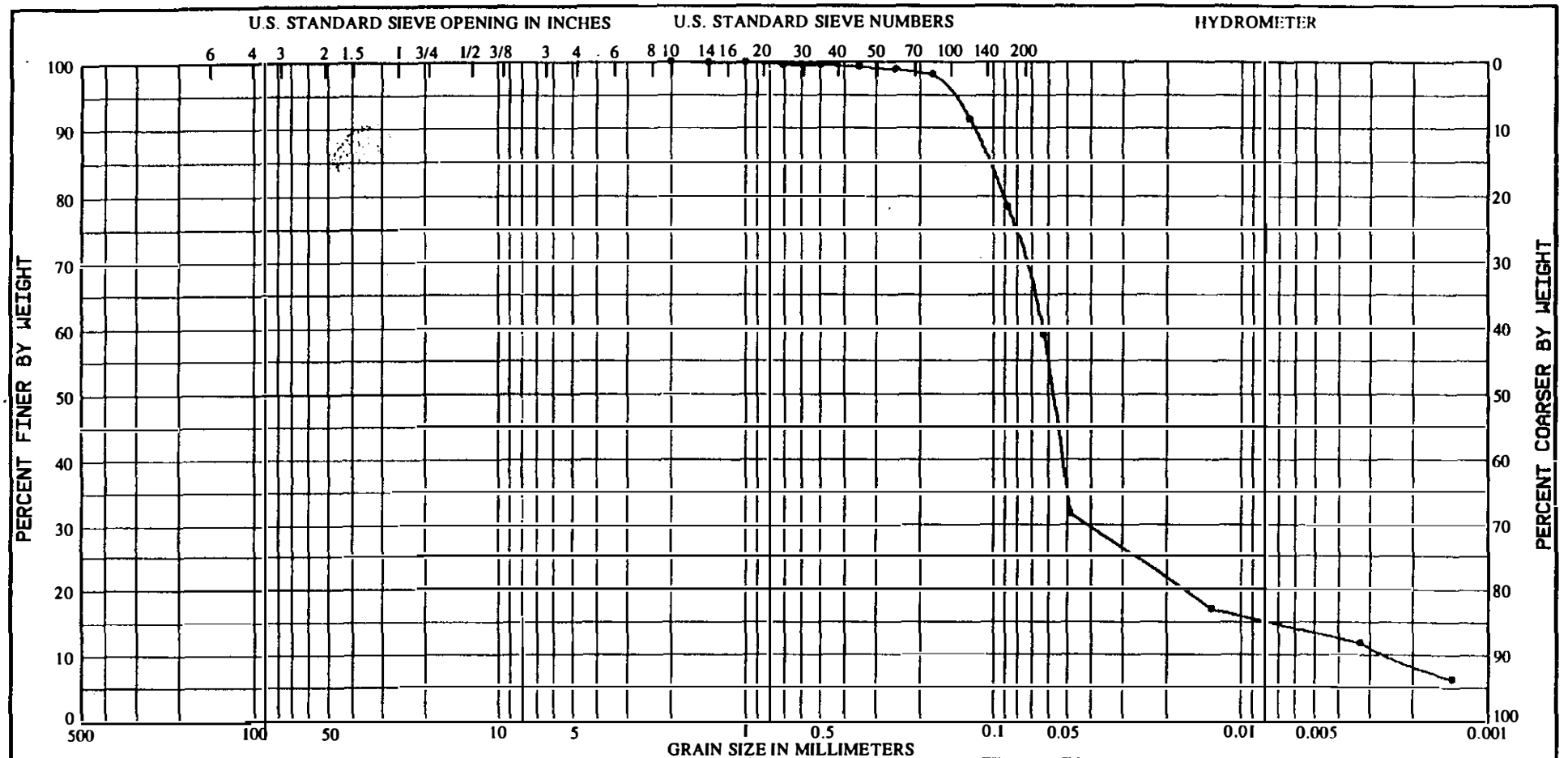
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



Sample No.	Depth	Classification	Nat w%	LL	PL	PI
D-2	7.3-7.8'	(VISUAL) TAN. MICACEOUS INORGANIC SILT	27.2			
		LOW LL (ML). WITH SOME SAND SIZES.				
		SP. GRAVITY = 2.67				
		WET DENSITY = 117.9 PCF.				
		DRY DENSITY = 92.7 PCF.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/927
Boring No. KAV-20
Date 03/13/95

GRADATION CURVES

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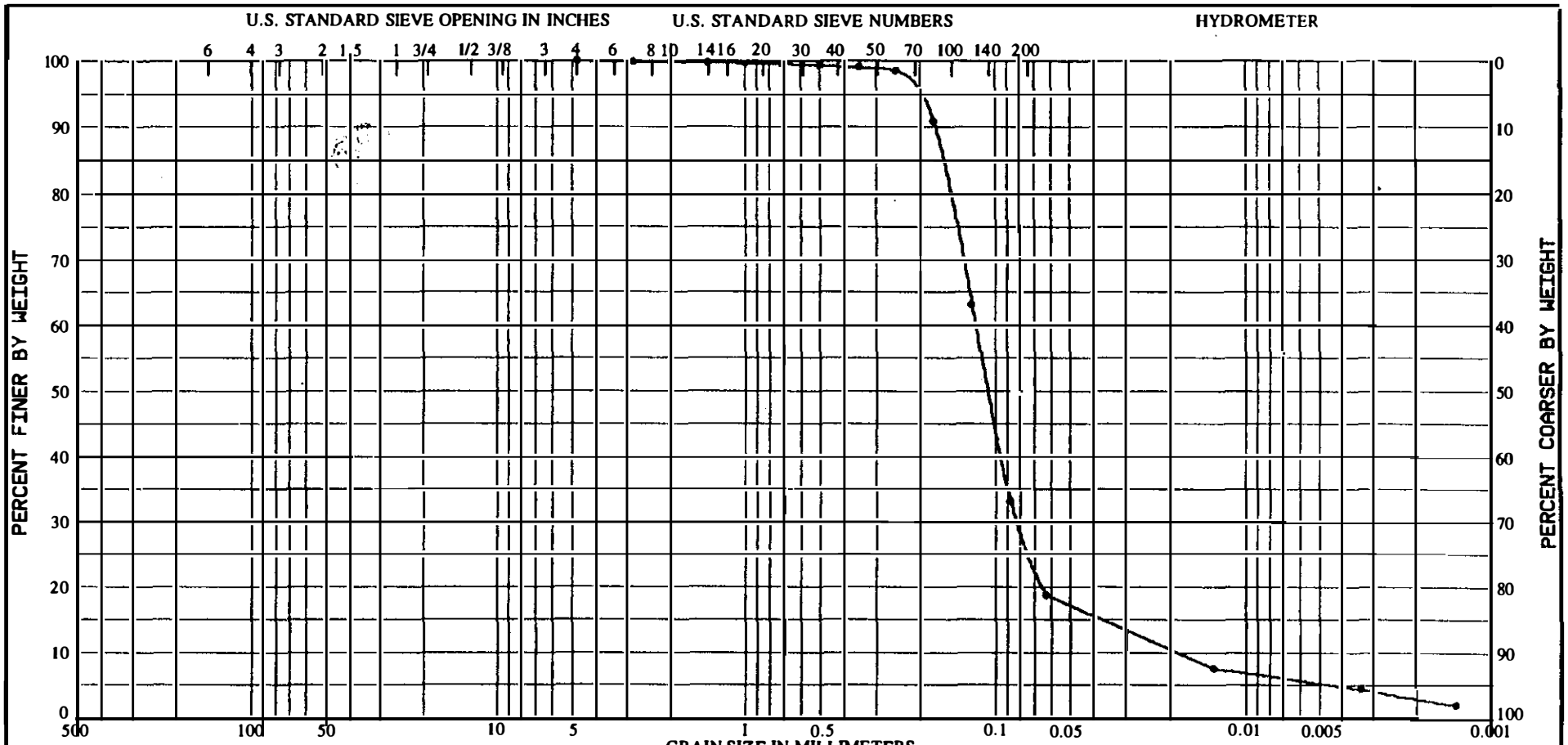
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



Sample No.	Depth	Classification	Nat w %	LL	PL	PI
D-3	12.3-12.8'	(VISUAL) LT. GRAY, SILTY SAND (SM).	24.2			
		SP. GRAVITY = 2.67				
		WET DENSITY = 119.3 PCF.				
		DRY DENSITY = 96.1 PCF.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/927
Boring No. KAV-20
Date 02/07/95

GRADATION CURVES

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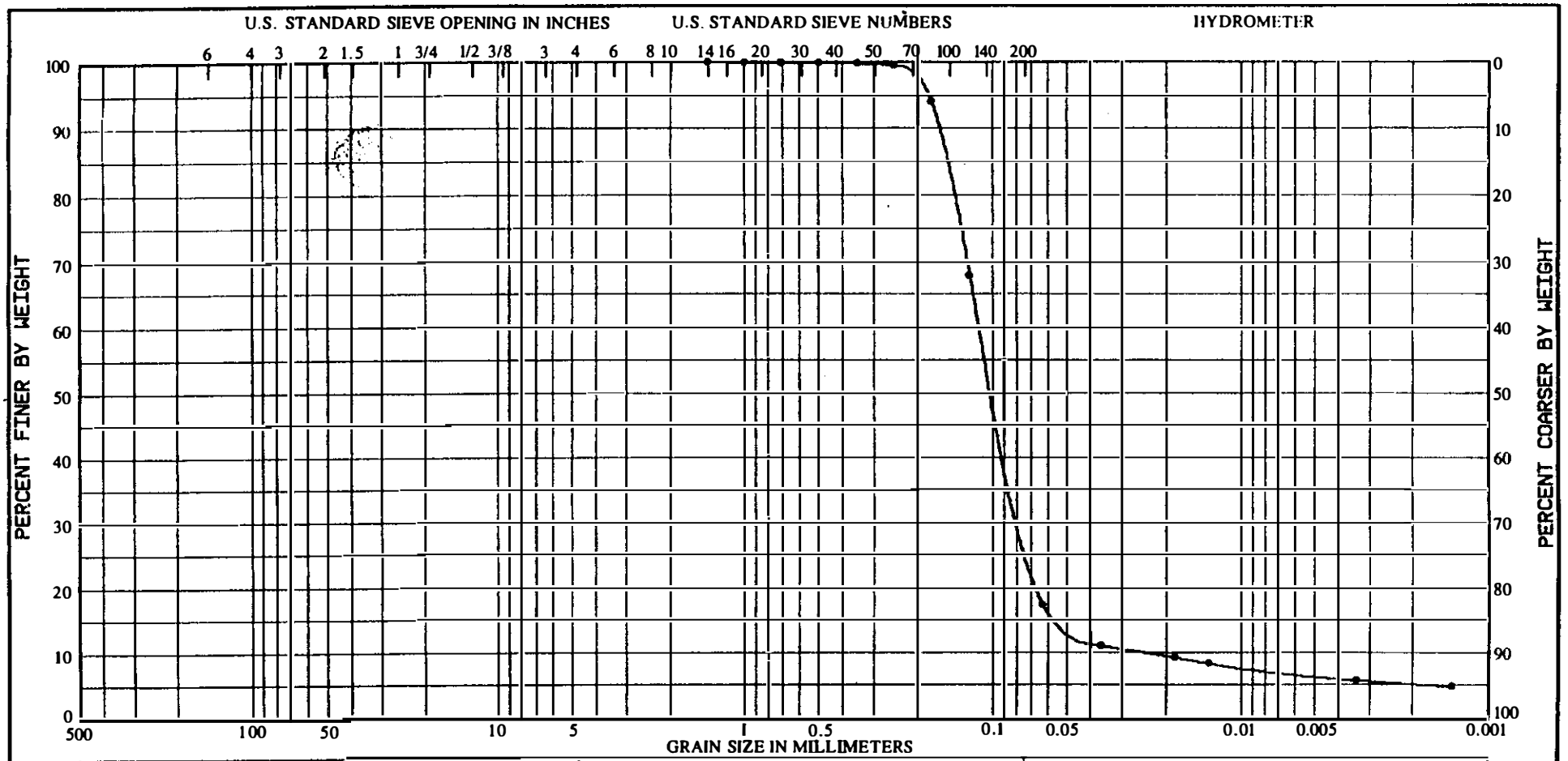
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



Sample No.	Depth	Classification	Nat w%	LL	PL	PI
C	14.6-14.9'	(VISUAL) LT. GRAY, SILTY SAND (SM).				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/927C
Boring No. KAV-20
Date 02/07/95

GRADATION CURVES

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DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE, MARIETTA, GA. 30060

Project: *DELAWARE BAY*			Boring No. *KAV-21*
Location: *VIBRA CORE SAMPLES*			Lab No. *184/928*
Boring Depth (ft): *17.30*		Elevation:	Work order: *7476*
Datum/Notes: *See grain size data on enclosed gradation curves.*			Requisition: *CENAP-95-707*

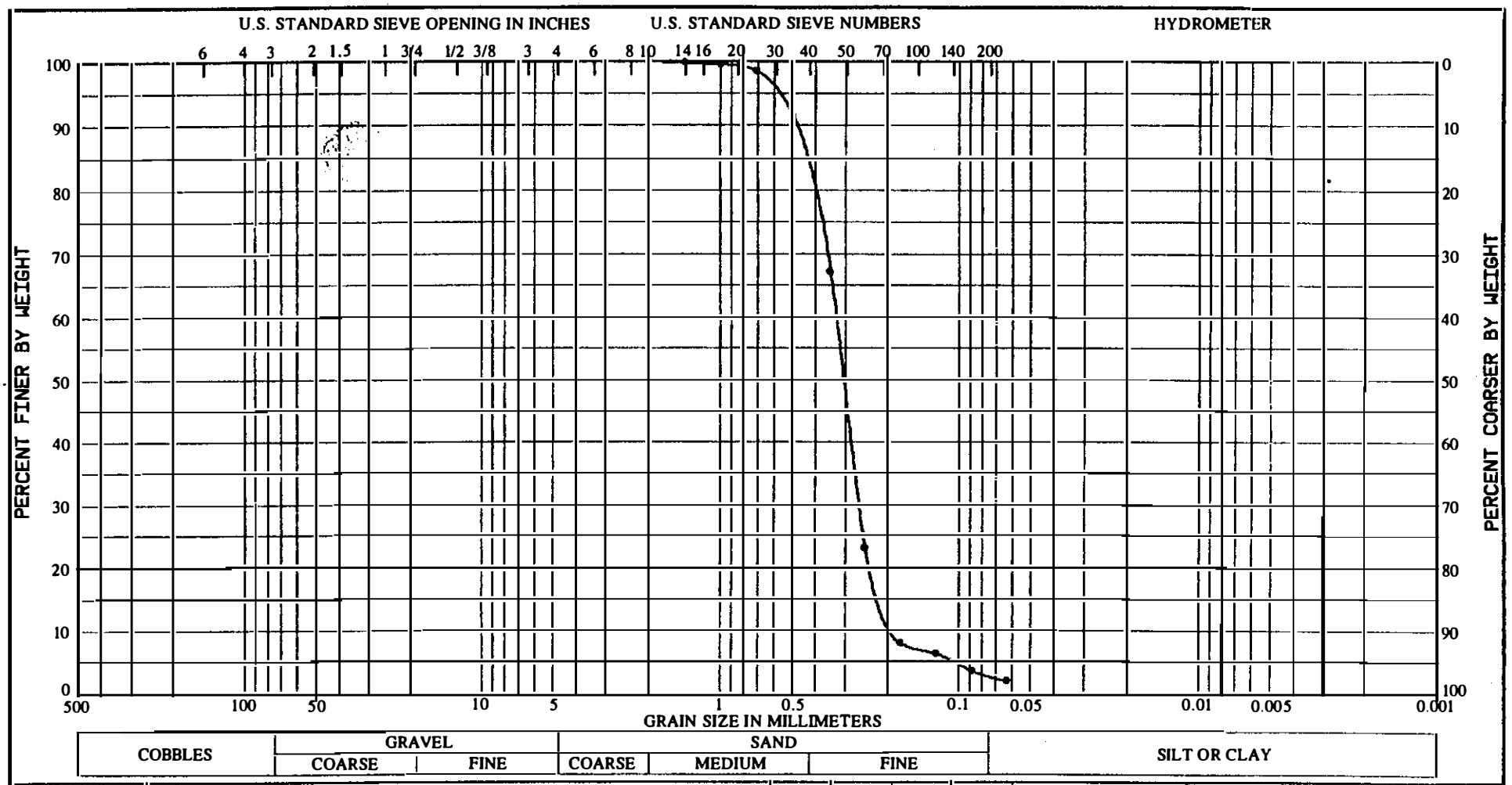
Elev. (feet)	Depth (feet)	Leg-end	Material Description	Comments
				(Density Units = pcf)
	1		TAN, POORLY GRADED SAND (SP), WITH A TRACE OF MICA AND SHELL FRAGMENTS.	
	2			SA D-1 2.1 - 2.6' WET DEN = 114.5 DRY DEN = 106.5, MC = 7.5 %.
	3			
	4			SA (A) 4.0 - 4.3'
	5		TAN AND LT. BROWN, POORLY GRADED SAND (SP).	
	6			SA (B) 5.6 - 5.9'
	7		GRAY, SILTY SAND (SM).	SA D-2 7.0 - 7.5' WET DEN = 120.0 DRY DEN = 95.1, MC = 26.2 %.
	8			MA (C) 8.2 - 8.5'
	9		GRAY, SILTY SAND (SM), WITH POCKETS AND LENSES OF POORLY GRADED SILTY SAND (SP-SM) AND OCCASIONAL POCKETS OF PLASTIC FINES.	
	10			
	11		BROWNISH GRAY, SILTY SAND (SM), WITH SOME GRAVEL SIZES AND A TRACE OF SHELL FRAGMENTS.	SA (D) 11.3 - 11.6'
	12			
	13			SA D-3 12.6 - 13.1' WET DEN = 124.1, DRY DEN = 103.1, MC = 20.4 %.
	14			
	15		TAN, POORLY GRADED SILTY SAND (SP-SM).	
	16			SA (E) 16.0 - 16.3'
	17			

Date: *01/25/95*

LABORATORY LOG AND SAMPLE DATUM

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



Sample No.	Depth	Classification	Nat w%	LL	PL	PI
D-1	2.1-2.6'	(VISUAL) TAN, POORLY GRADED SAND (SP),	7.5			
		WITH A TRACE OF MICA AND SHELL FRAGMENTS				
		SP. GRAVITY = 2.66				
		WET DENSITY = 114.5 PCF.				
		DRY DENSITY = 106.5.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/928
Boring No. KAV-21
Date 01/19/95

GRADATION CURVES

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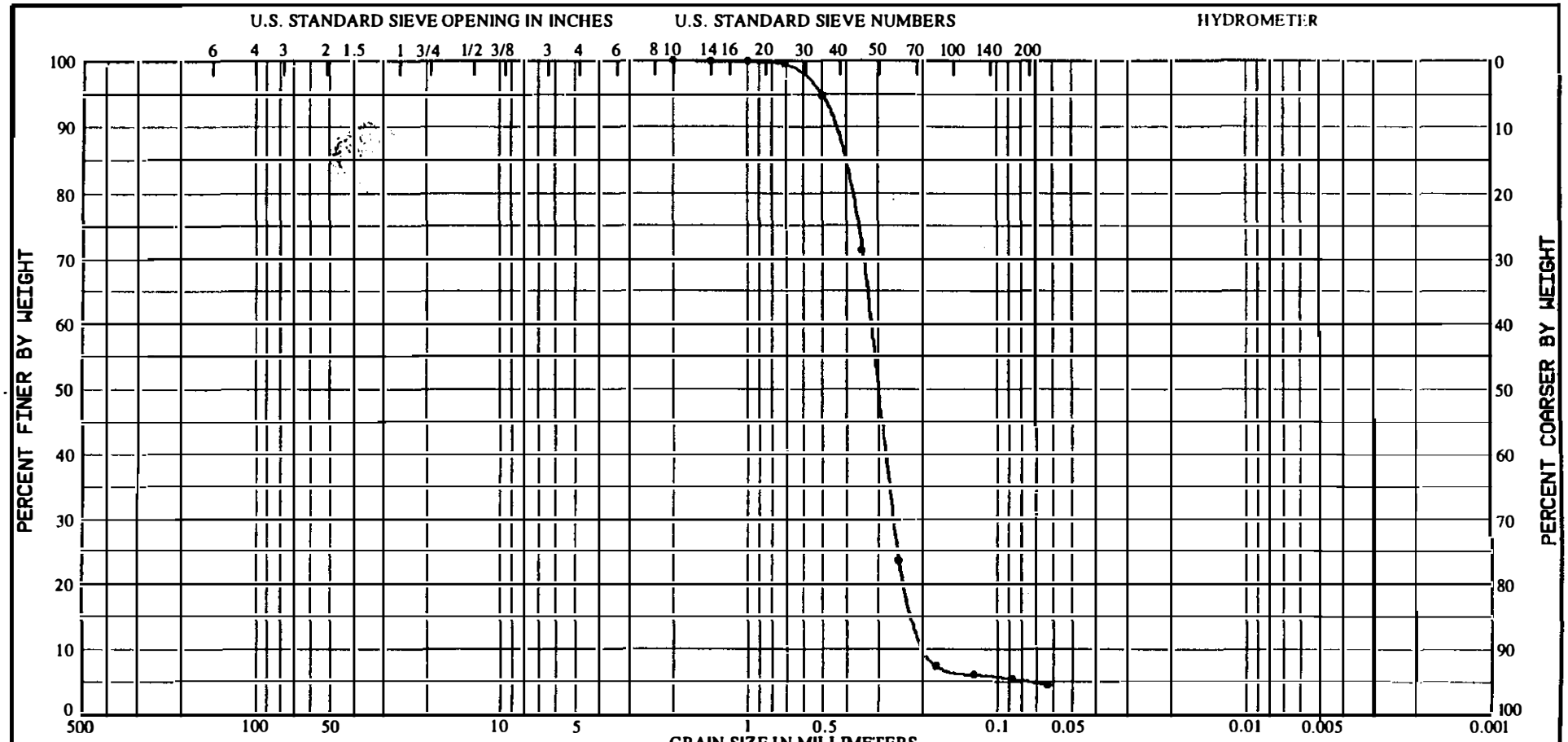
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Sample No.	Depth	Classification	Nat w%	LL	PL	PI
A	4.0-4.3'	(VISUAL) TAN AND LT. BROWN, POORLY GRADED SAND (SP).				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/928A
Boring No. KAV-21
Date 01/19/95

GRADATION CURVES

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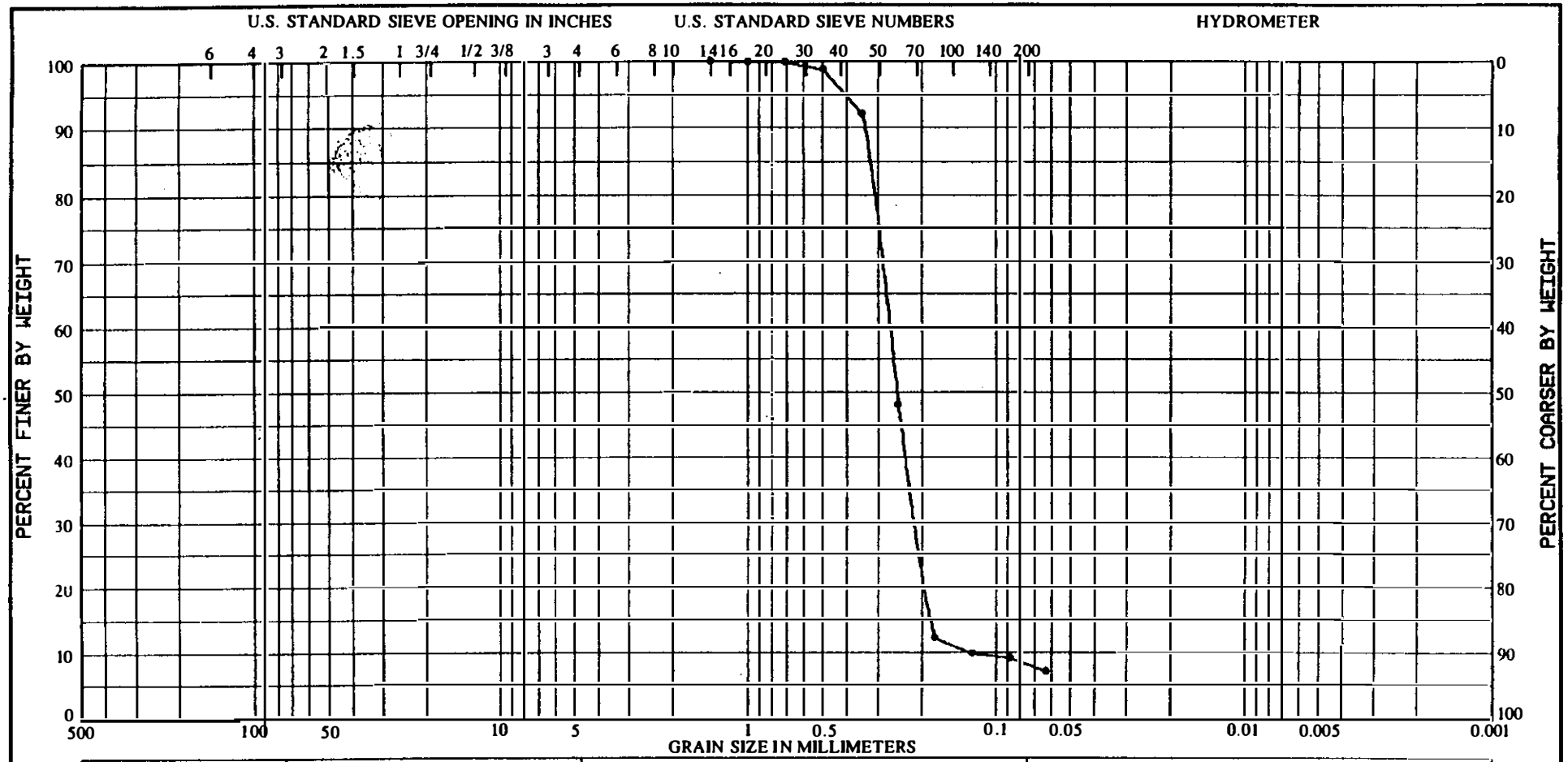
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



Sample No.	Depth	Classification	Nat w%	LL	PL	PI
B	5.6-5.9'	(VISUAL) TAN AND LT. BROWN, POORLY GRADED SILTY SAND (SP-SM).				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/928B
Boring No. KAV-21
Date 01/19/95

GRADATION CURVES

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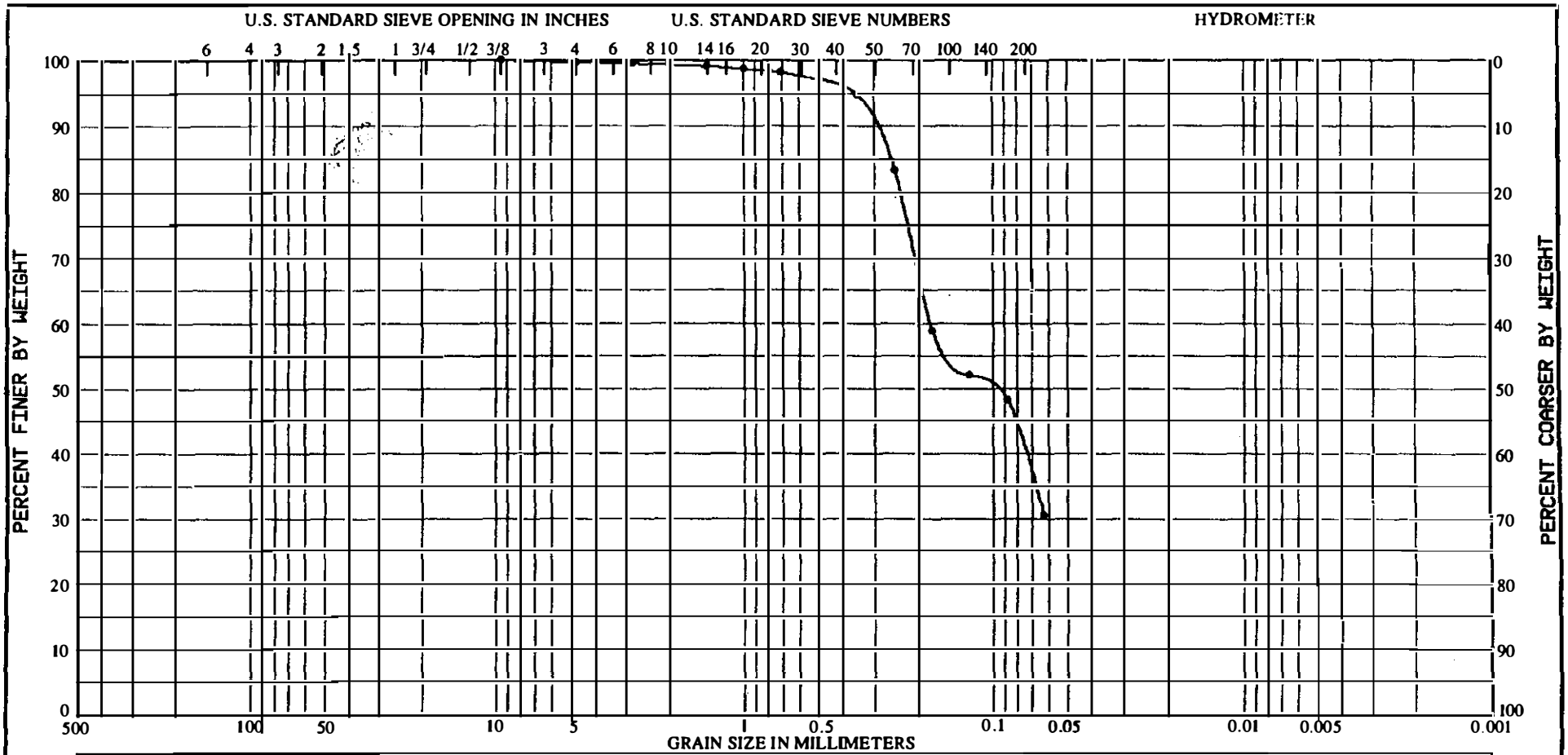
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Sample No.	Depth	Classification	Nat w%	LL	PL	PI
D-2	7.0-7.5'	(VISUAL) GRAY, SILTY SAND (SM).	26.2			
		SP. GRAVITY = 2.65				
		WET DENSITY = 120.0 PCF.				
		DRY DENSITY = 95.1 PCF.				

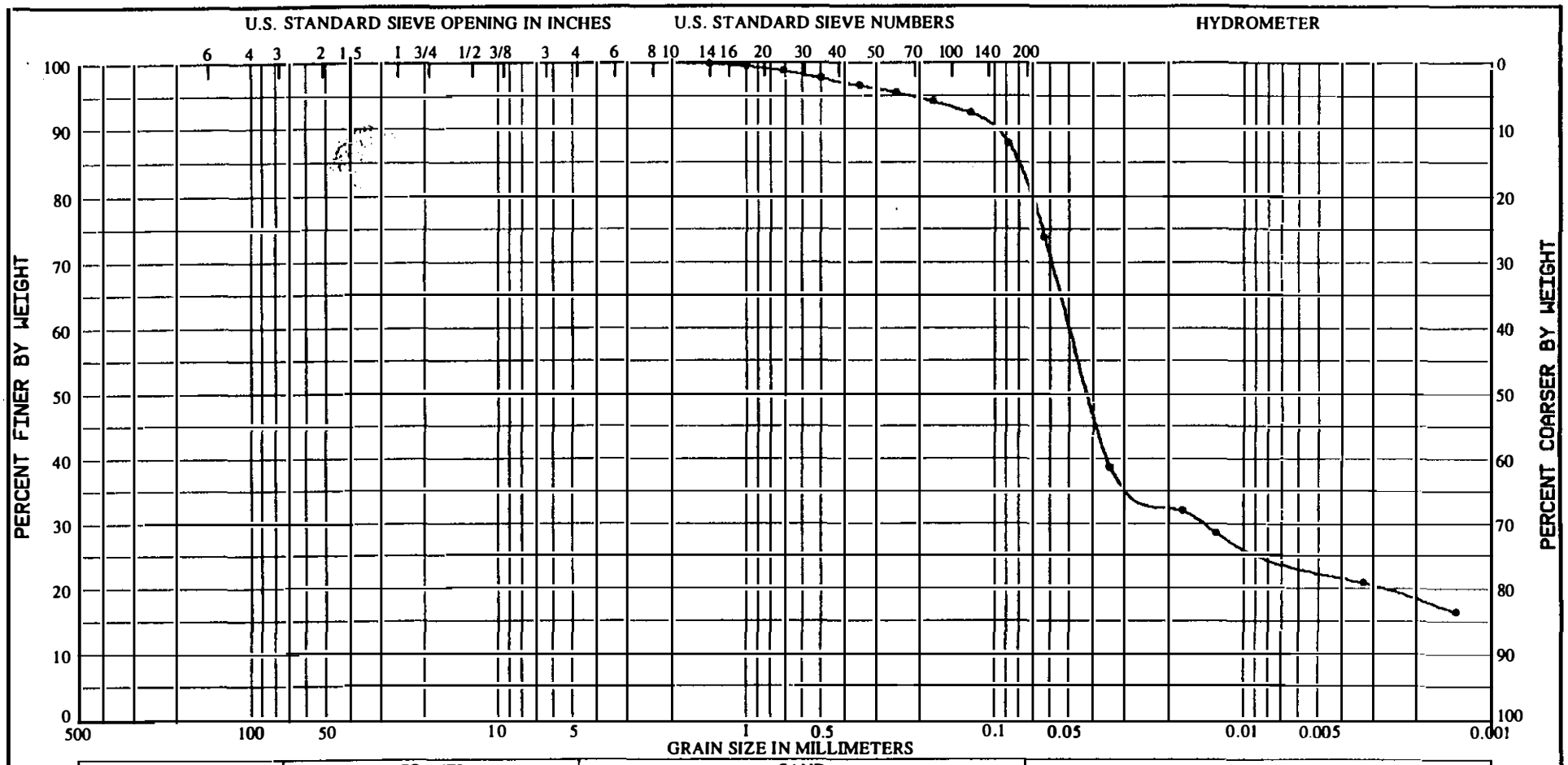
Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/928
Boring No. KAV-21
Date 01/19/95

GRADATION CURVES

55

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Sample No.	Depth	Classification	Nat w%	LL	PL	PI
C	8.2-8.5'	(VISUAL) GRAY, INORGANIC SILT LOW LL (ML), WITH A LITTLE SAND SIZES.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/928C
Boring No. KAV-21
Date 01/19/95

GRADATION CURVES

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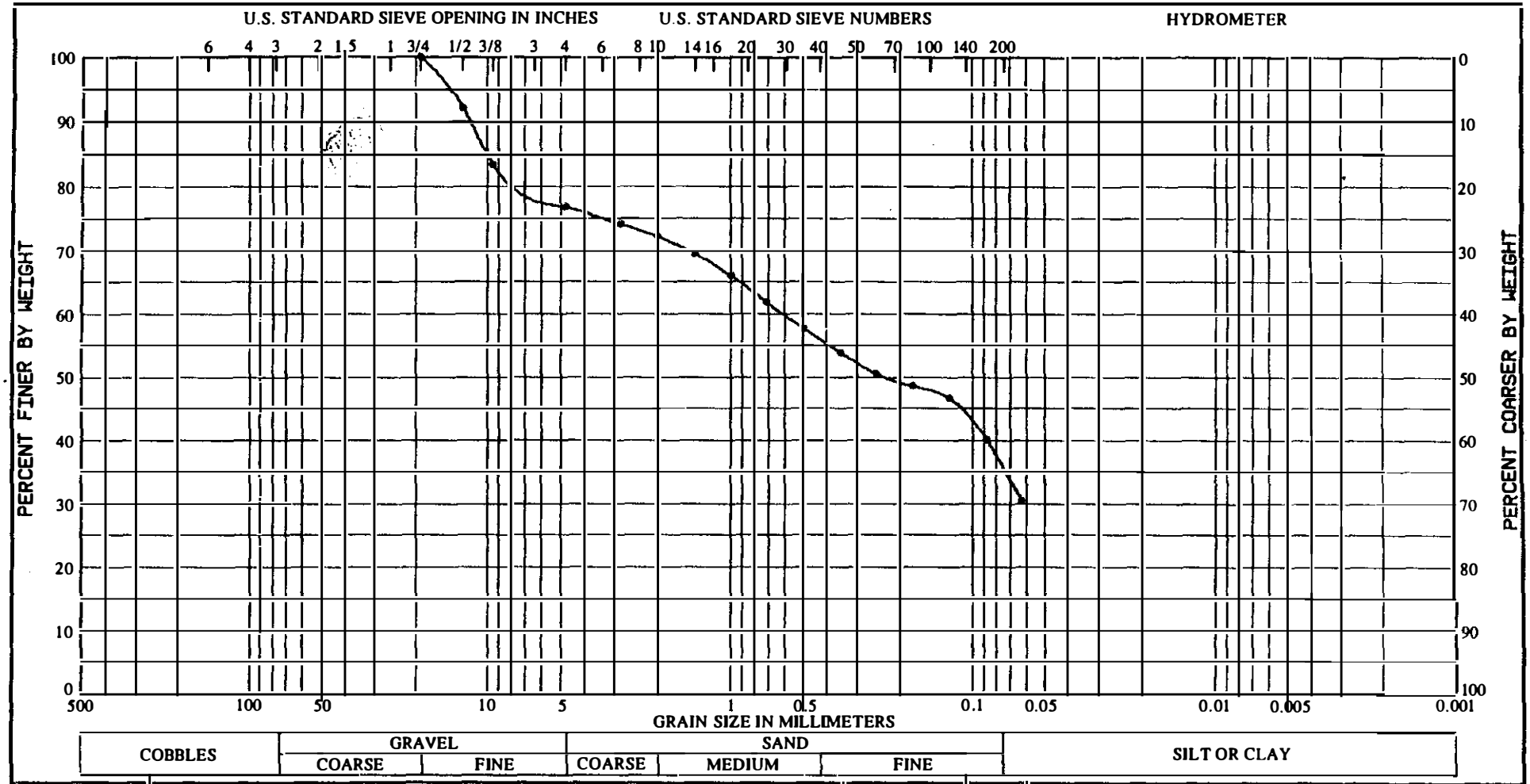
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



Sample No.	Depth	Classification	Nat w%	LL	PL	PI
D	11.3-11.6'	(VISUAL) BROWNISH GRAY, SILTY SAND (SM), WITH SOME GRAVEL SIZES AND A TRACE OF SHELL FRAGMENTS.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/9280
Boring No. KAV-21
Date 01/19/95

GRADATION CURVES

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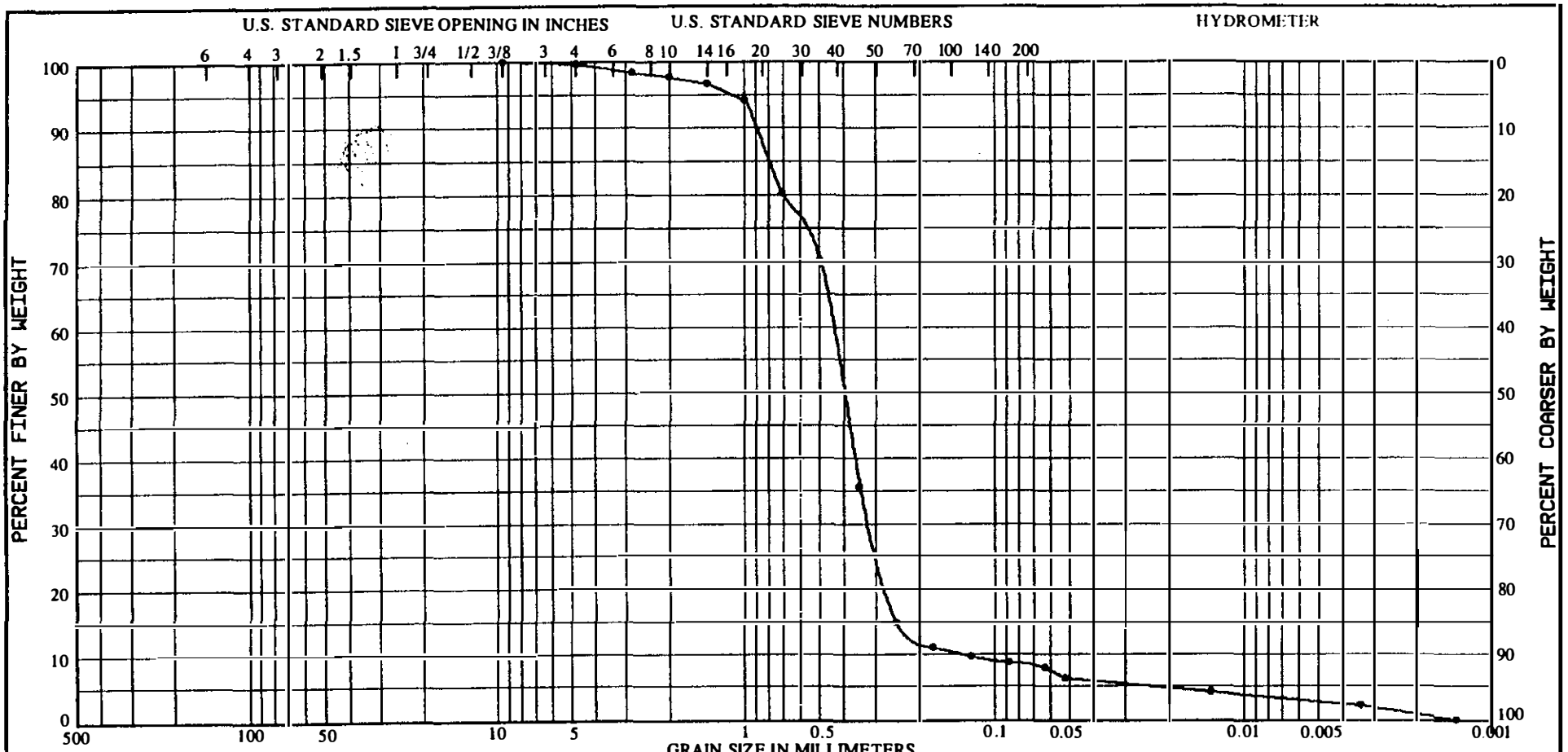
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



Sample No.	Depth	Classification	Nat w%	LL	PL	PI
D-3	12.6-13.1'	(VISUAL) TAN, POORLY GRADED SILTY	20.4			
		SAND (SP-SM).				
		SP. GRAVITY = 2.64				
		WET DENSITY = 124.1 PCF.				
		DRY DENSITY = 103.1 PCF.				

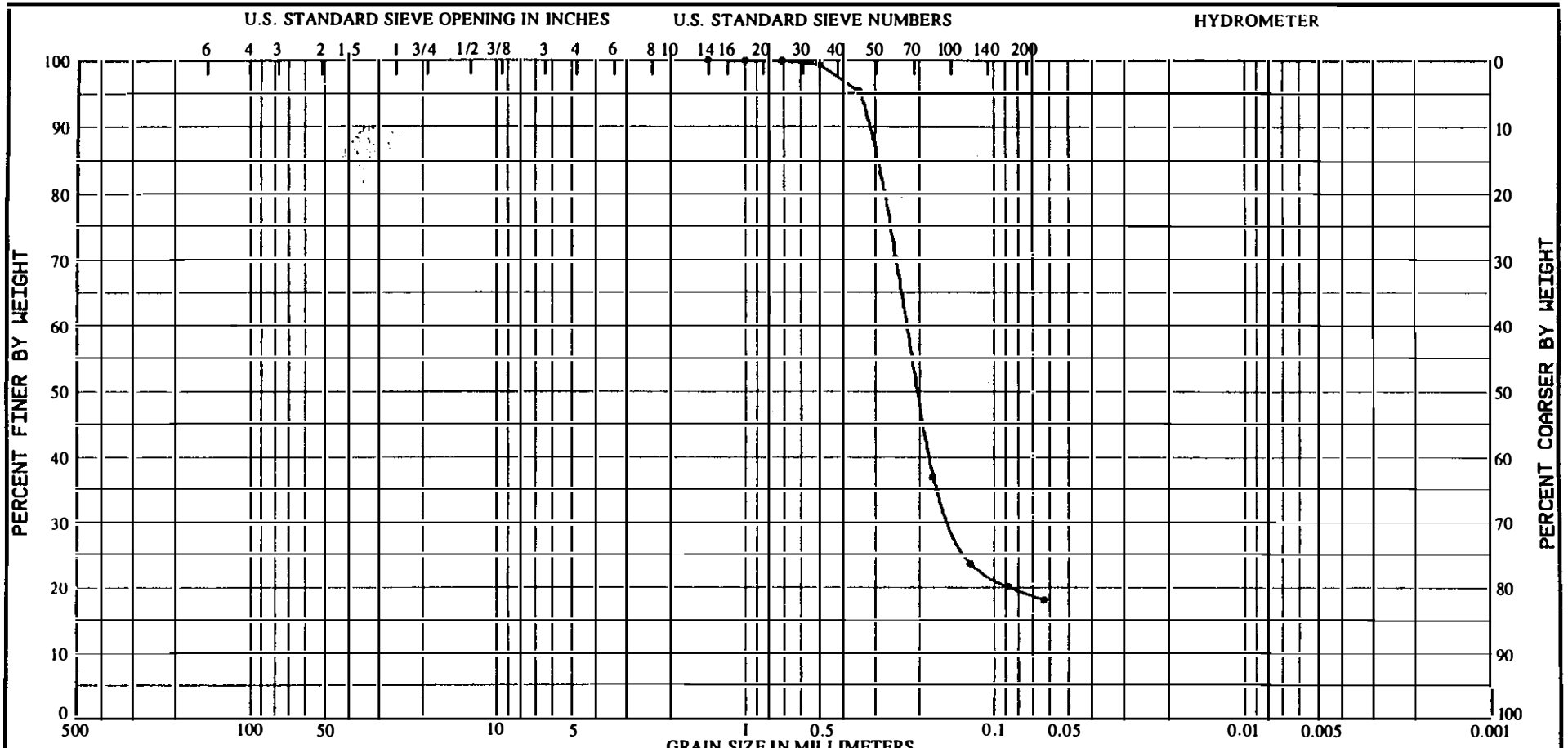
Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/928
Boring No. KAV-21
Date 01/19/95

GRADATION CURVES

58

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Sample No.	Depth	Classification	Nat w%	LL	PL	PI
E	16.0-16.3'	(VISUAL) TAN. SILTY SAND (SM).				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/928E
Boring No. KAV-21
Date 01/19/95

GRADATION CURVES

59

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE, MARIETTA, GA. 30060

Project: *DELAWARE BAY*		Boring No. *KAV-22*	
Location: *VIBRA CORE SAMPLES*		Lab No. *184/929*	
Boring Depth (ft): *10.20*	Elevation:	Work order: *7476*	
Datum: Notes: *See grain size data on enclosed gradation curves.*		Requisition: *CENAP-95-707*	

Elev (feet)	Depth (feet)	Legend	Material Description	Comments
				(Density Units = pcf)
	1		RUN # 2	MA (A) 0.9 - 1.2'
			TAN AND BROWNISH GRAY, SILTY SAND (SM), WITH PLASTIC FINES.	
	2			SA D-1 2.2 - 2.7' WET DEN = 118.2, DRY DEN = 95.1, MC = 24.6 %.
	3		----------	
			GRAY, SANDY INORGANIC SILT LOW LL (ML).	
	4		----------	MA (B) 3.8 - 4.1'
	5			
			LT. GRAY, SILTY SAND (SM).	
	6			SA (C) 6.0 - 6.3'
	7			
	8		----------	MA D-2 8.2 - 8.6' WET DEN = 106.2, DRY DEN = 67.7, MC = 56.8 %.
	9			
			GRAY, INORGANIC SILT HIGH LL (MH), WITH A LITTLE SAND SIZES AND A TRACE OF MICA.	
	10		----------	
	11			
	12			
	13			
	14			
	15			

Date: *01/20/95* **LABORATORY LOG AND SAMPLE DATUM** Sheet No. *1* of *1*

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE, MARIETTA, GA. 30060

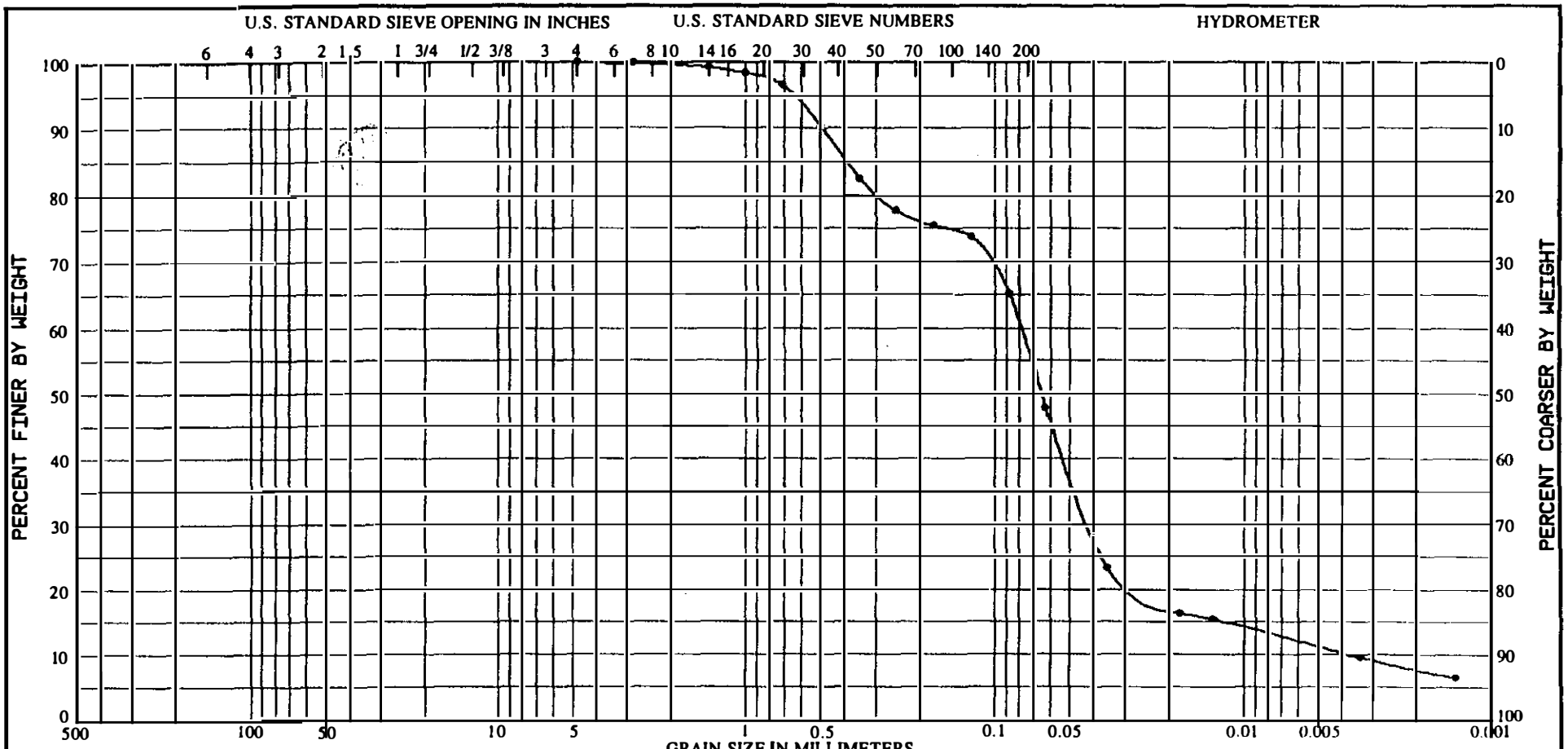
Project: *DELAWARE BAY*		Boring No. *KAV-22*
Location: *VIBRA CORE SAMPLES*		Lab No. *184/929*
Boring Depth (ft): *15.20*	Elevation:	Work order: *7476*
Datum/Notes: *See grain size data on enclosed gradation curves.*		Requisition: *CENAP-95-707*

Elev. (feet)	Depth (feet)	Leg-end	Material Description	Comments
				(Density Units = pcf)
	1		RUN # 1	
	2			
	3			
	4			
	5		----------	
	6			
	7			SA D-3 7.0 - 7.5' WET DEN = 128.3 DRY DEN = 113.2, MC = 13.3 %.
	8			
	9		TAN, POORLY GRADED SILTY SAND (SP-SM), WITH A TRACE OF GRAVEL SIZES.	SA (D) 9.0 - 9.3'
	10			
	11			
	12			
	13			
			----------	SA D-4 13.5 - 14.0' WET DEN = 119.7, DRY DEN = 112.0, MC = 6.9 %.
	14		TAN, POORLY GRADED SAND (SP), WITH SOME GRAVEL SIZES.	
	15		----------	

Date: *01/20/95* **LABORATORY LOG AND SAMPLE DATUM** Sheet No. *1* of *1*

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Sample No.	Depth	Classification	Nat w%	LL	PL	PI
A	0.9-1.2'	(VISUAL) TAN & BROWNISH GRAY, SANDY INORGANIC SILT LOW LL (ML).				

Project OELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/929A
Boring No. KAV-22
Date 01/25/95

GRADATION CURVES

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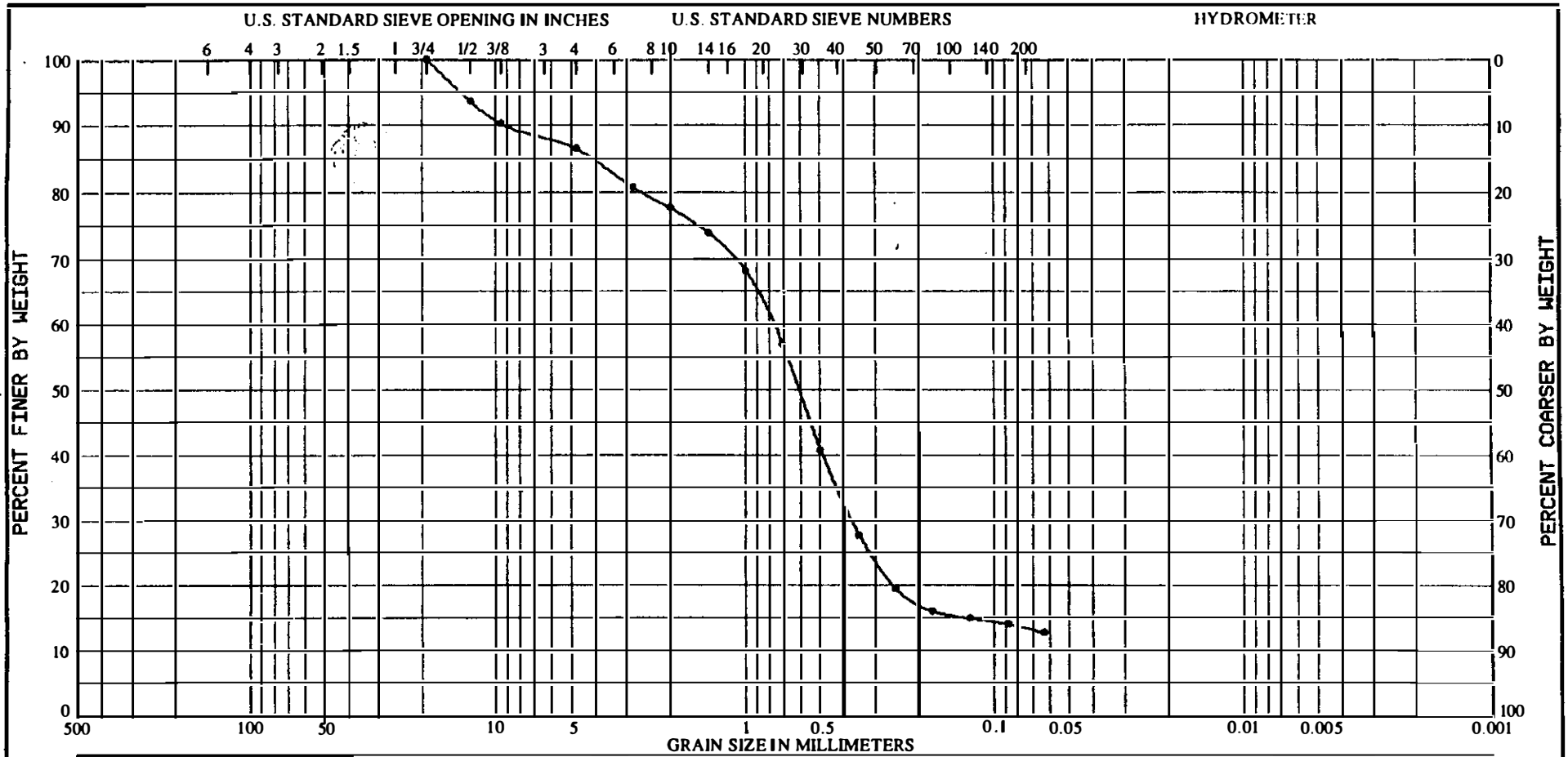
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Sample No.	Depth	Classification	Nat w%	LL	PL	PI
D-I	2.2-2.7'	(VISUAL) GRAY, SILTY SAND (SM), WITH A	24.6			
		LITTLE GRAVEL SIZES AND A TRACE OF MICA.				
		SP. GRAVITY = 2.62				
		WET DENSITY = 118.5 PCF.				
		DRY DENSITY = 95.1 PCF.				

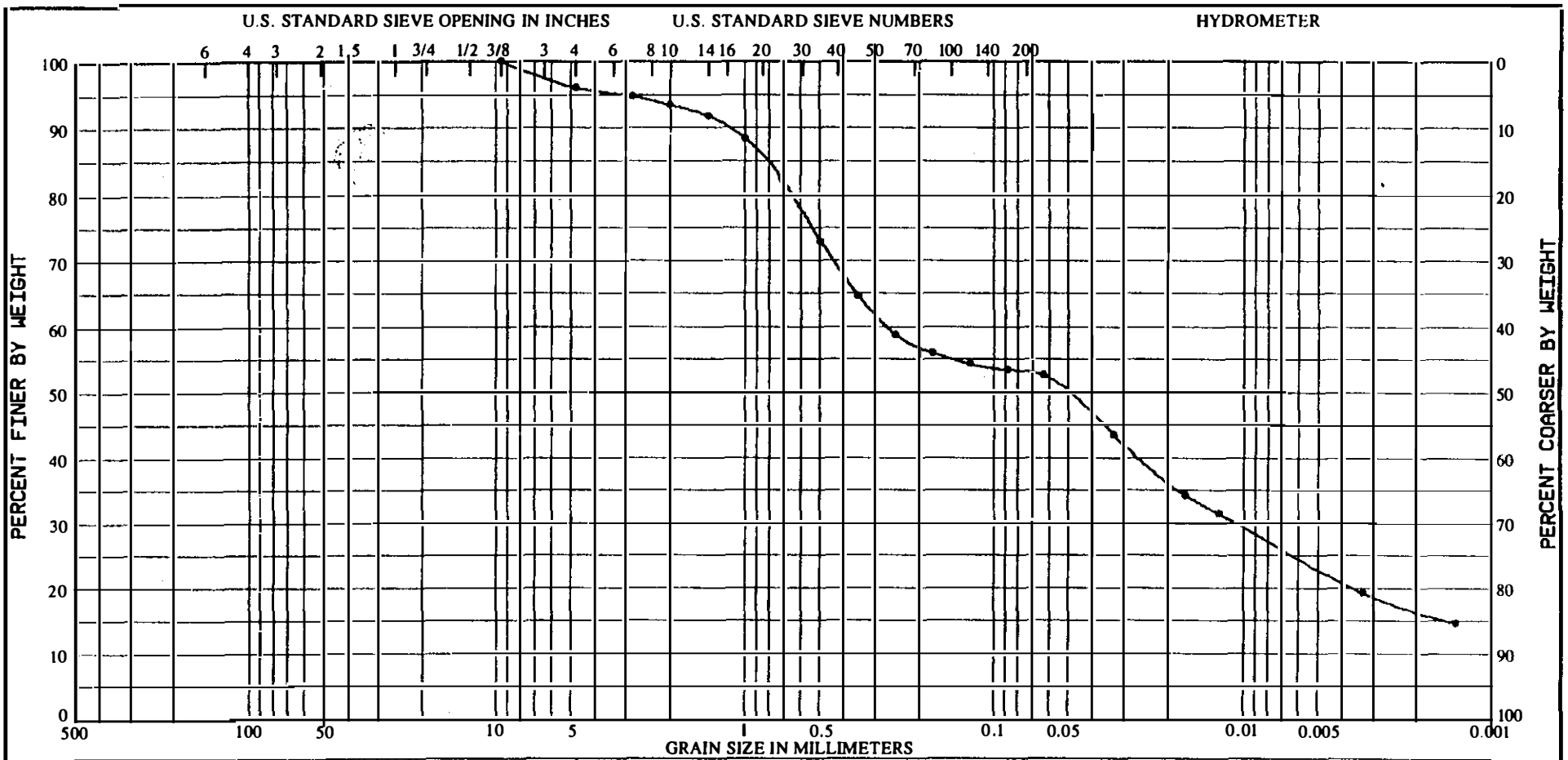
Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/929
Boring No. KAV-22
Date 01/20/95

GRADATION CURVES

63

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Sample No.	Depth	Classification	Nat w%	LL	PL	PI
B	3.8-4.1'	(VISUAL) GRAY, SANDY INORGANIC SILT				
		LOW LL (ML), WITH A TRACE OF GRAVEL SIZES.				

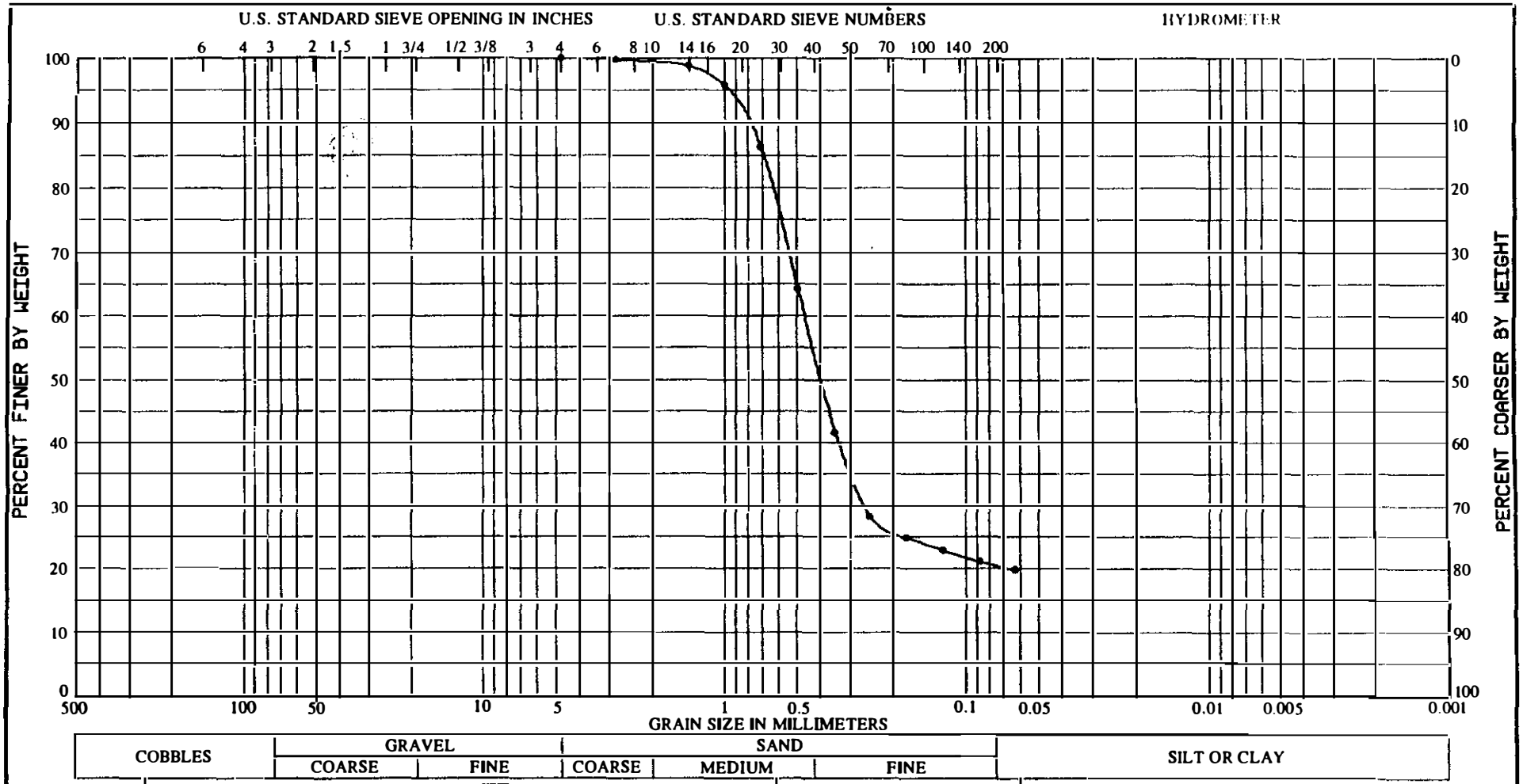
Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/929B
Boring No. KAV-22
Date 01/20/95

GRADATION CURVES

69

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



Sample No.	Depth	Classification	Nat w%	LL	PL	PI
C	6.0-6.3'	(VISUAL) LT. GRAY, SILTY SAND (SM).				

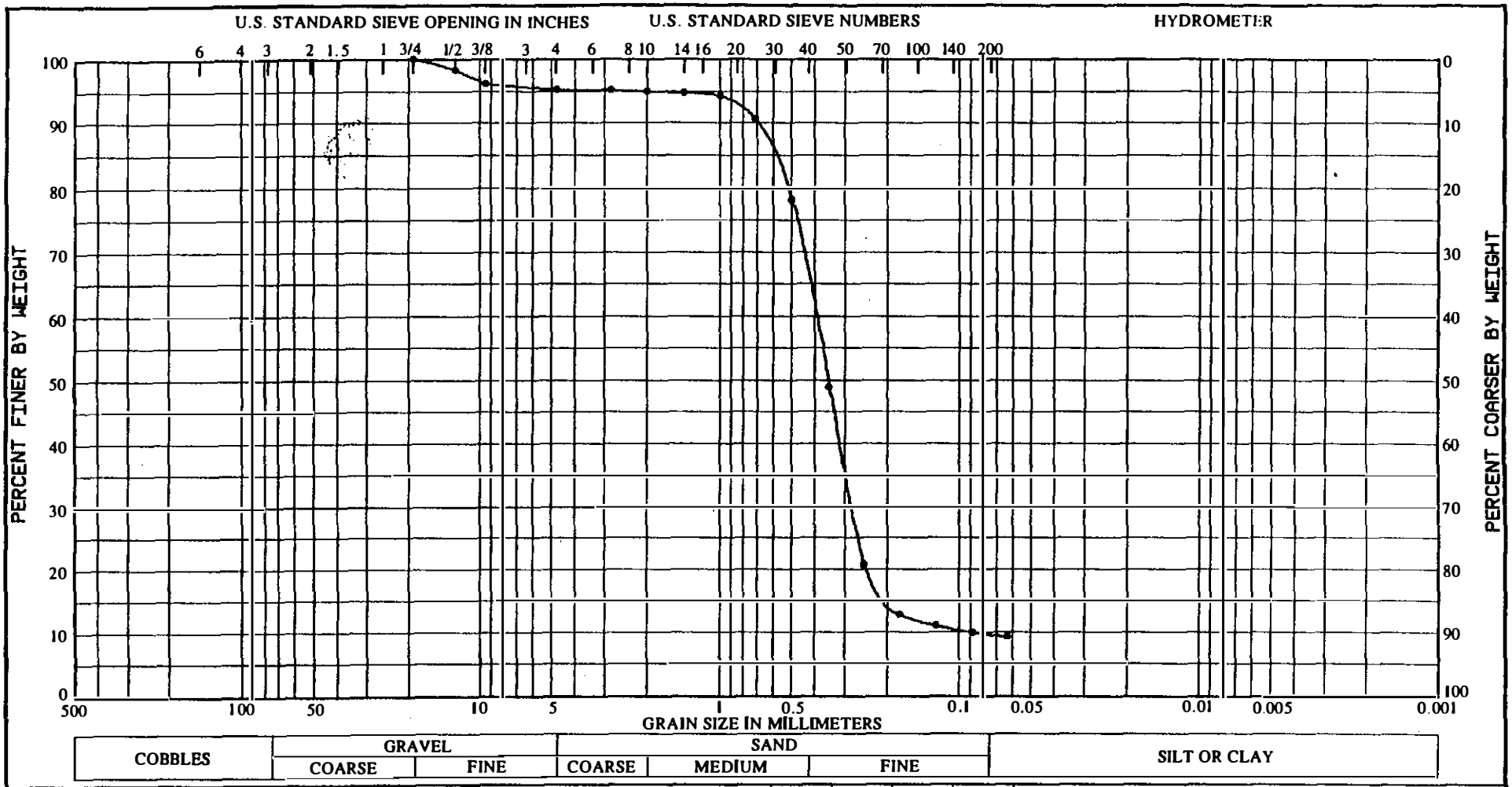
Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/929C
Boring No. KAV-22
Date 01/20/95

GRADATION CURVES

65

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



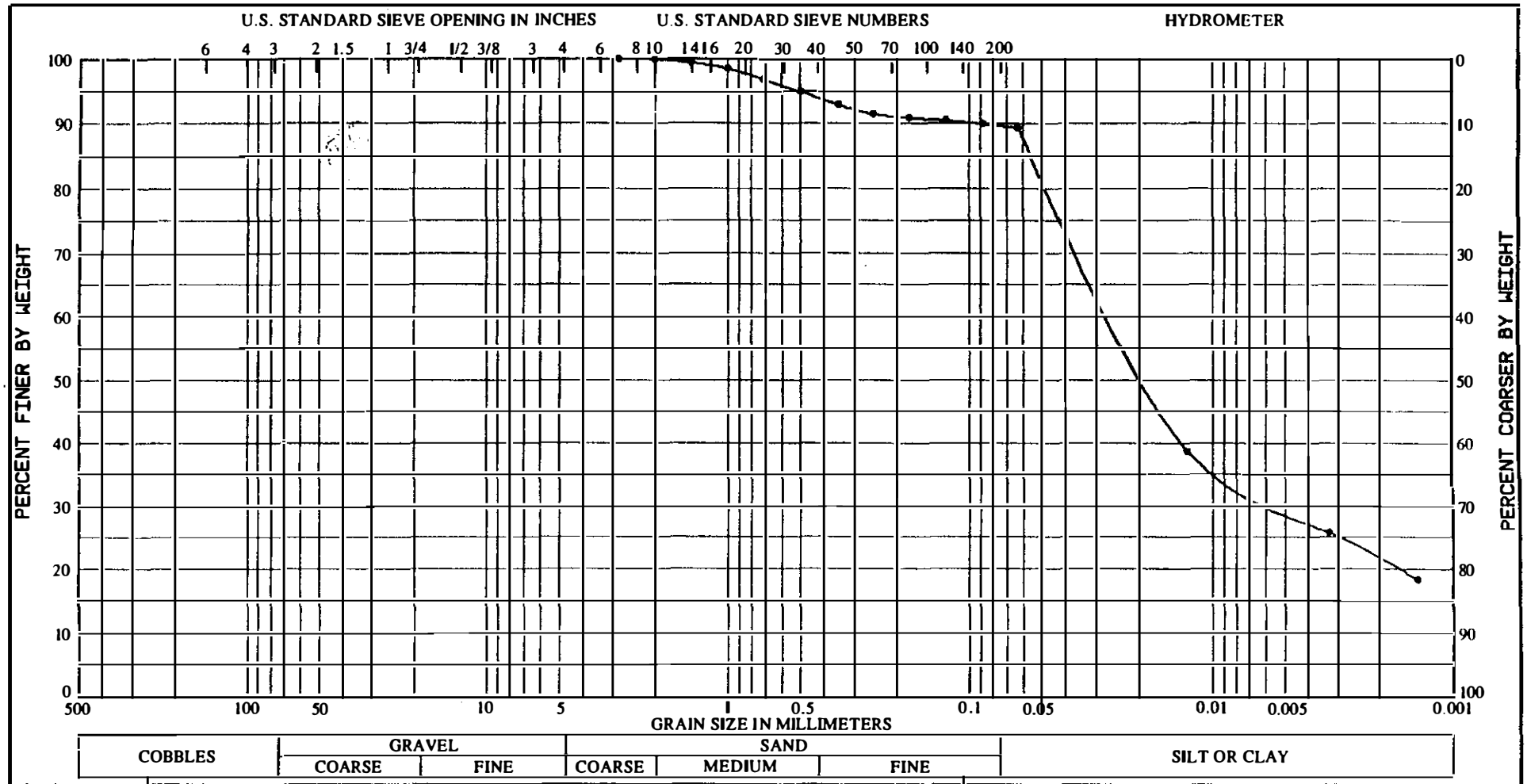
Sample No.	Depth	Classification	Nat w%	LL	PL	PI
D-3	7.0-7.5'	(VISUAL) TAN, POORLY GRADED SILTY SAND	13.3			
		(SP-SM), WITH A TRACE OF GRAVEL SIZES.				
		SP. GRAVITY = 2.64 WET DEN. = 128.3 PCF.				
		DRY DENSITY = 113.2 PCF.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/929
Boring No. KAV-22
Date 01/20/95

GRADATION CURVES

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



Sample No.	Depth	Classification	Nat w%	LL	PL	PI
D-2	8.2-8.6'	(VISUAL) GRAY, INORGANIC SILT HIGH LL (MH), WITH A LITTLE SAND SIZES AND A TRACE OF MICA. SP. GRAVITY = 2.69 WET DENSITY = 106.2 PCF. DRY DENSITY = 67.7 PCF.	56.8			

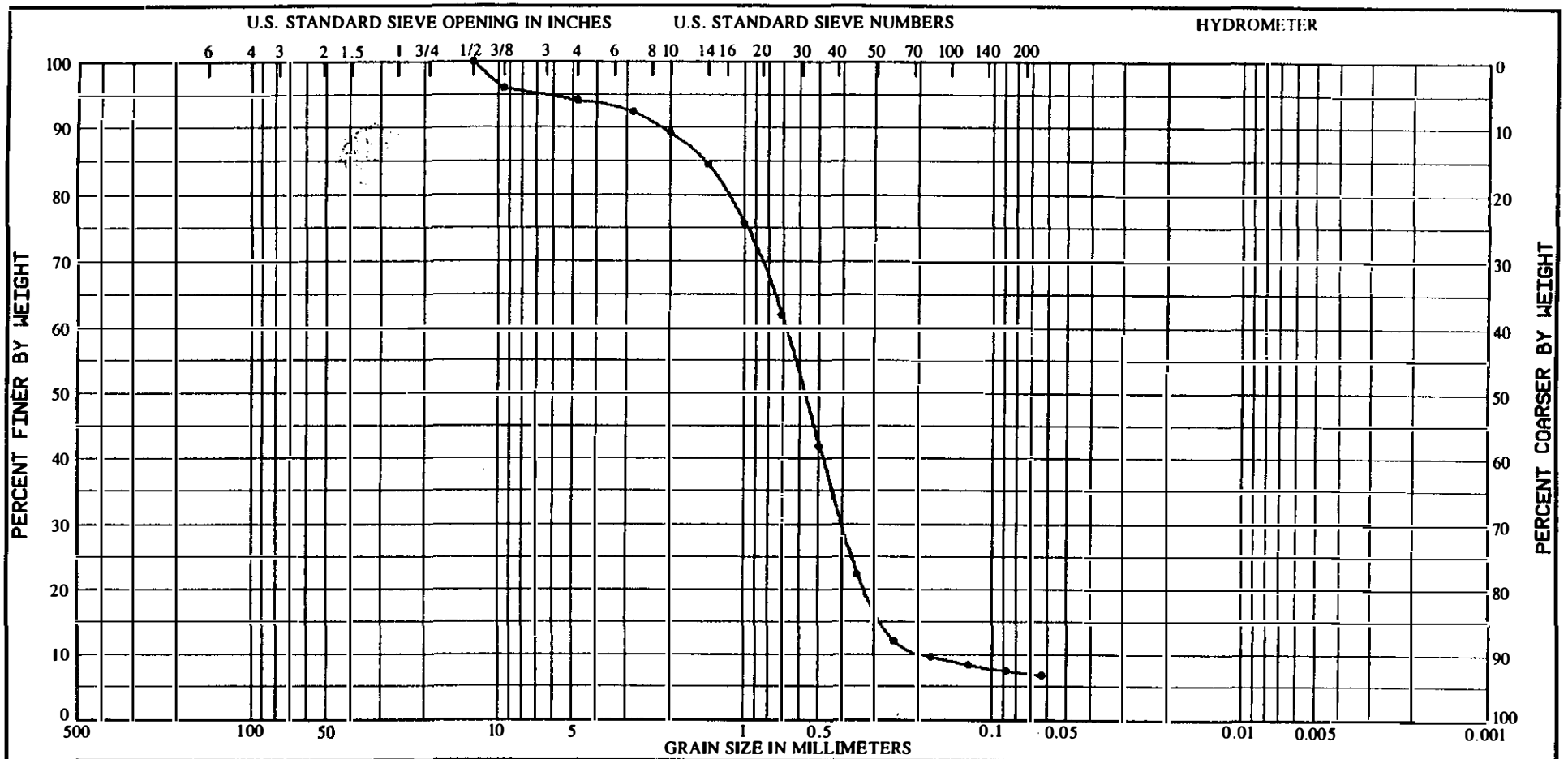
Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/929
Boring No. KAV-22
Date 01/20/95

GRADATION CURVES

67

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Sample No.	Depth	Classification	Nat w%	LL	PL	PI
D	9.0-9.3'	(VISUAL) TAN, POORLY GRADED SILTY SAND (SP-SM), WITH A TRACE OF GRAVEL SIZES.				

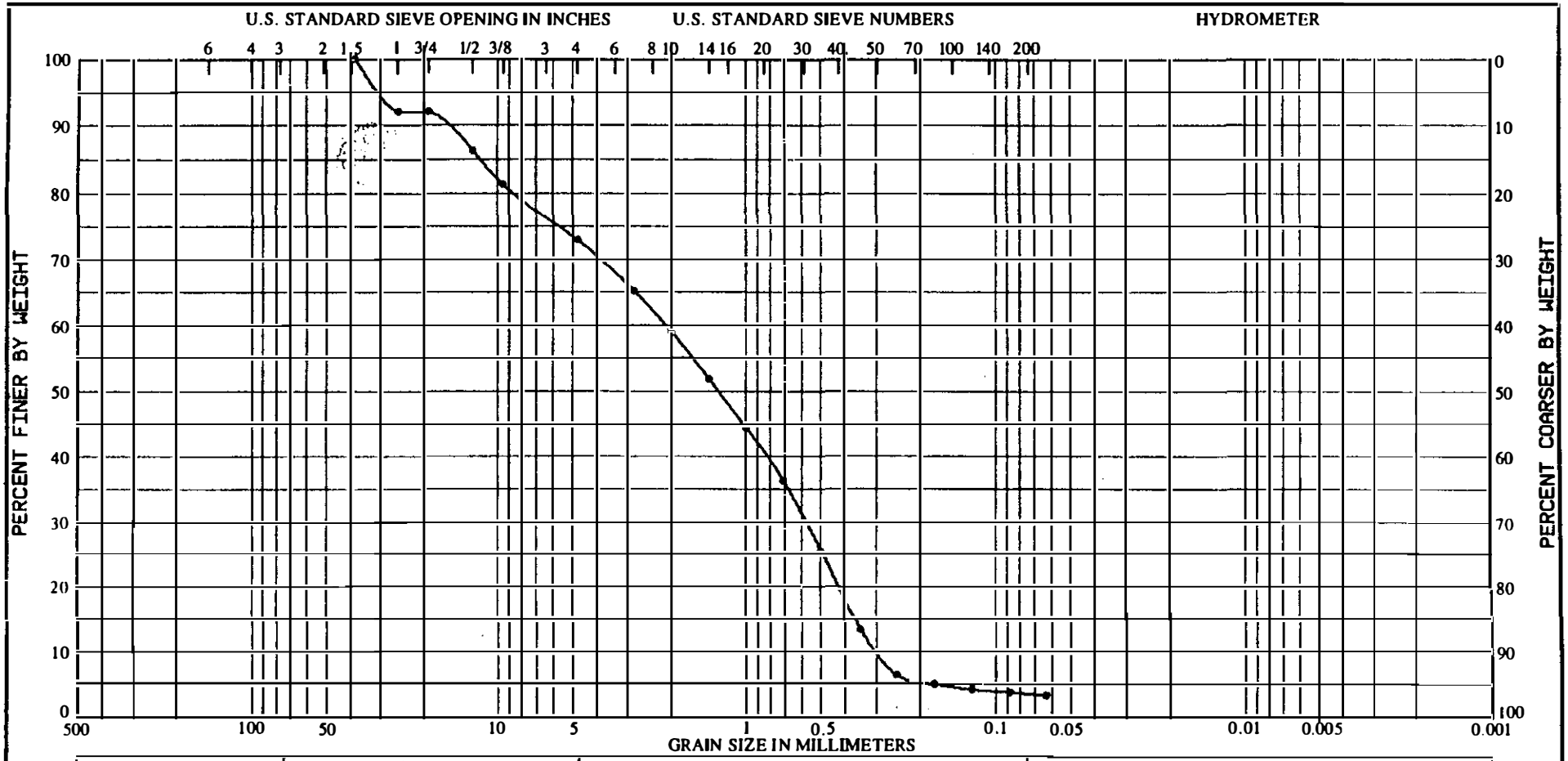
Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/929D
Boring No. KAV-22
Date 01/20/95

GRADATION CURVES

68

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Sample No.	Depth	Classification	Nat w %	LL	PL	PI
D-4	8.3-8.7'	(VISUAL) TAN, POORLY GRADED SAND (SP),	6.9			
		WITH SOME GRAVEL SIZES.				
		SP. GRAVITY = 2.63				
		WET DENSITY = 119.7 PCF.				
		DRY DENSITY = 112.0 PCF.				

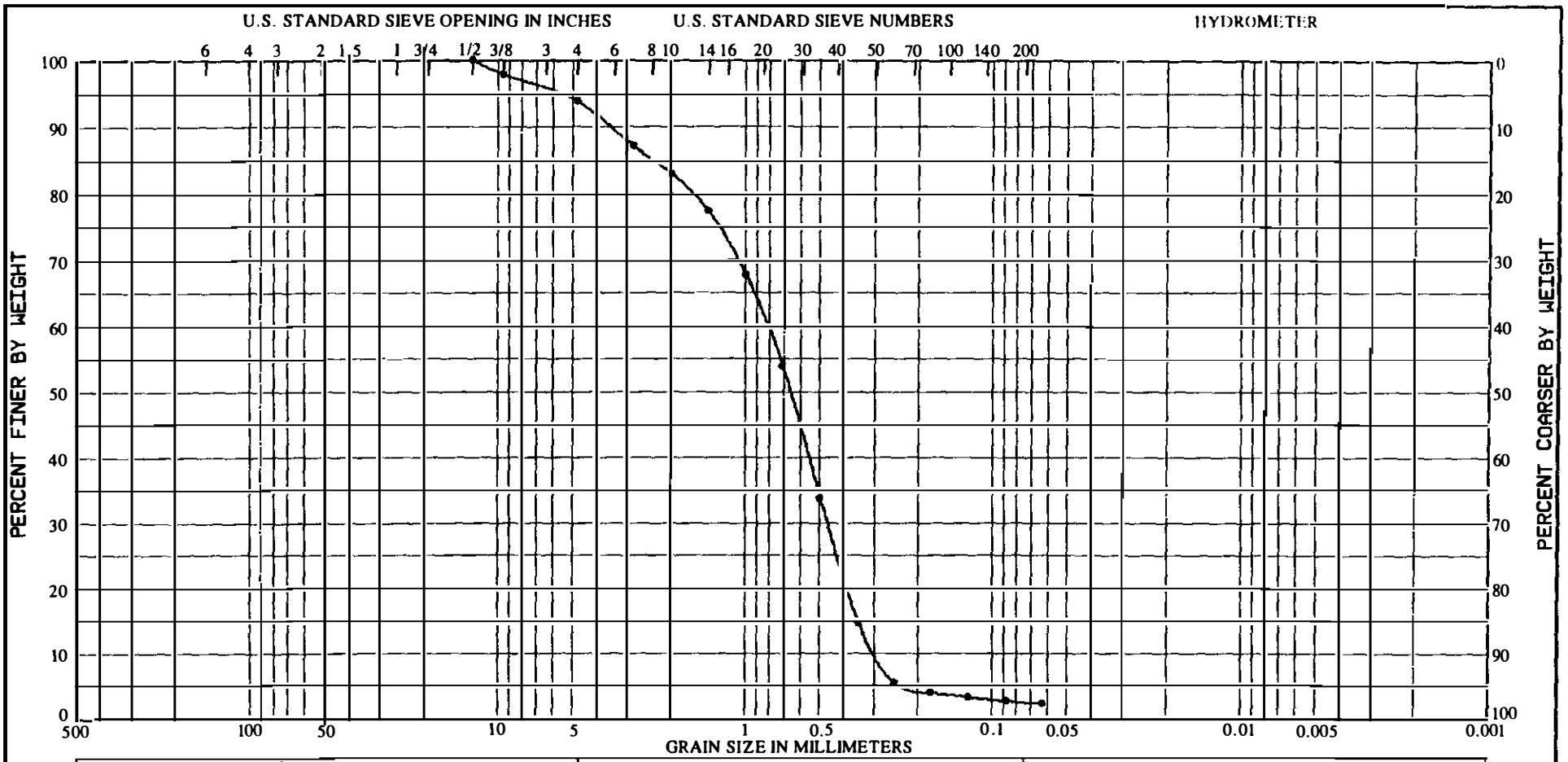
Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/929
Boring No. KAV-22
Date 01/20/95

GRADATION CURVES

69

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

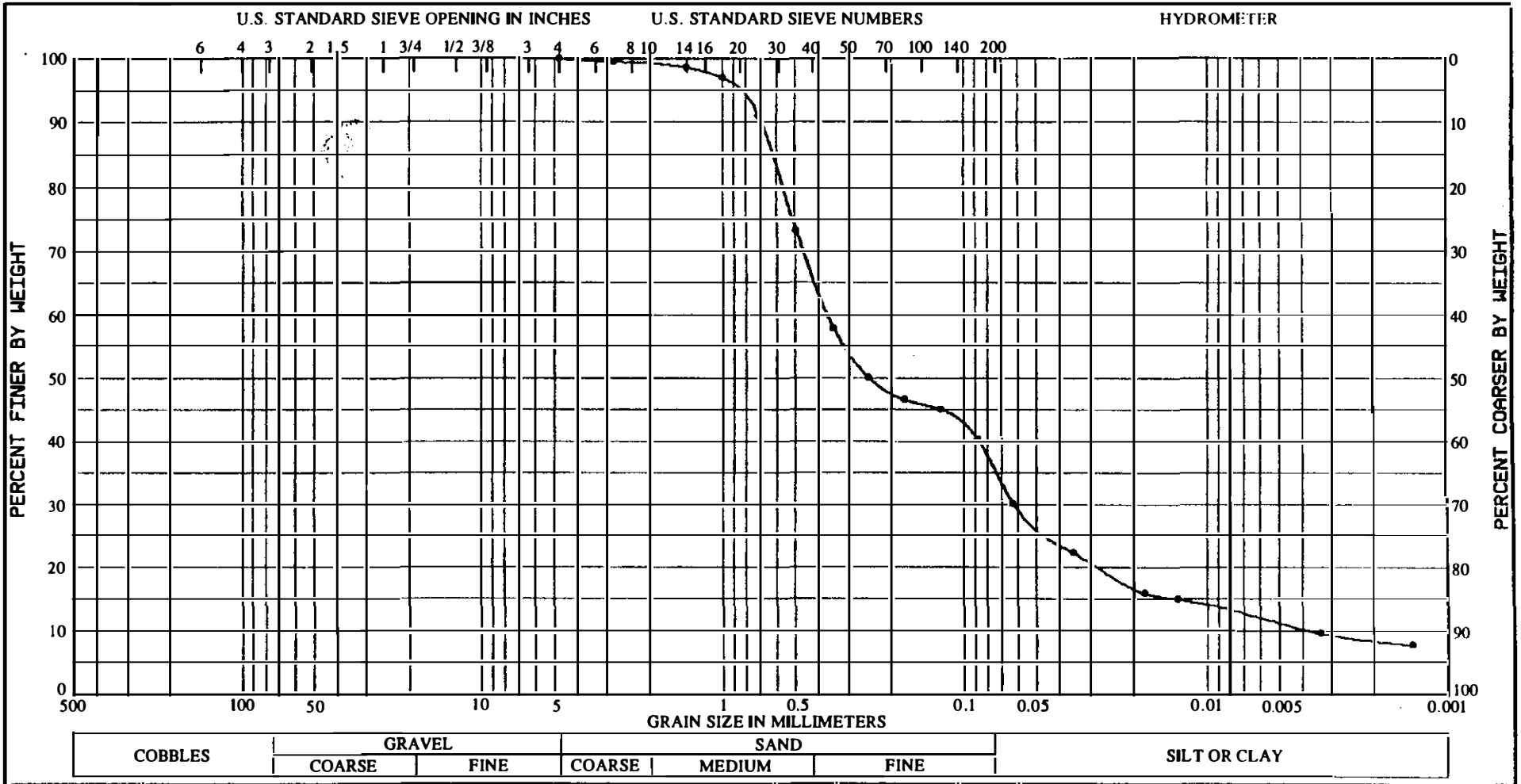
Sample No.	Depth	Classification	Nat w%	LL	PL	PI
D-3	12.0-12.5'	(VISUAL) TAN & LT. GRAY, POORLY GRADED	10.3			
		SAND (SP), WITH A TRACE OF GRAVEL SIZES.				
		SP. GRAVITY = 2.65				
		WET DENSITY = 135.4 PCF.				
		DRY DENSITY = 122.7 PCF.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/939
Boring No. KAV-32
Date 02/15/95

GRADATION CURVES

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



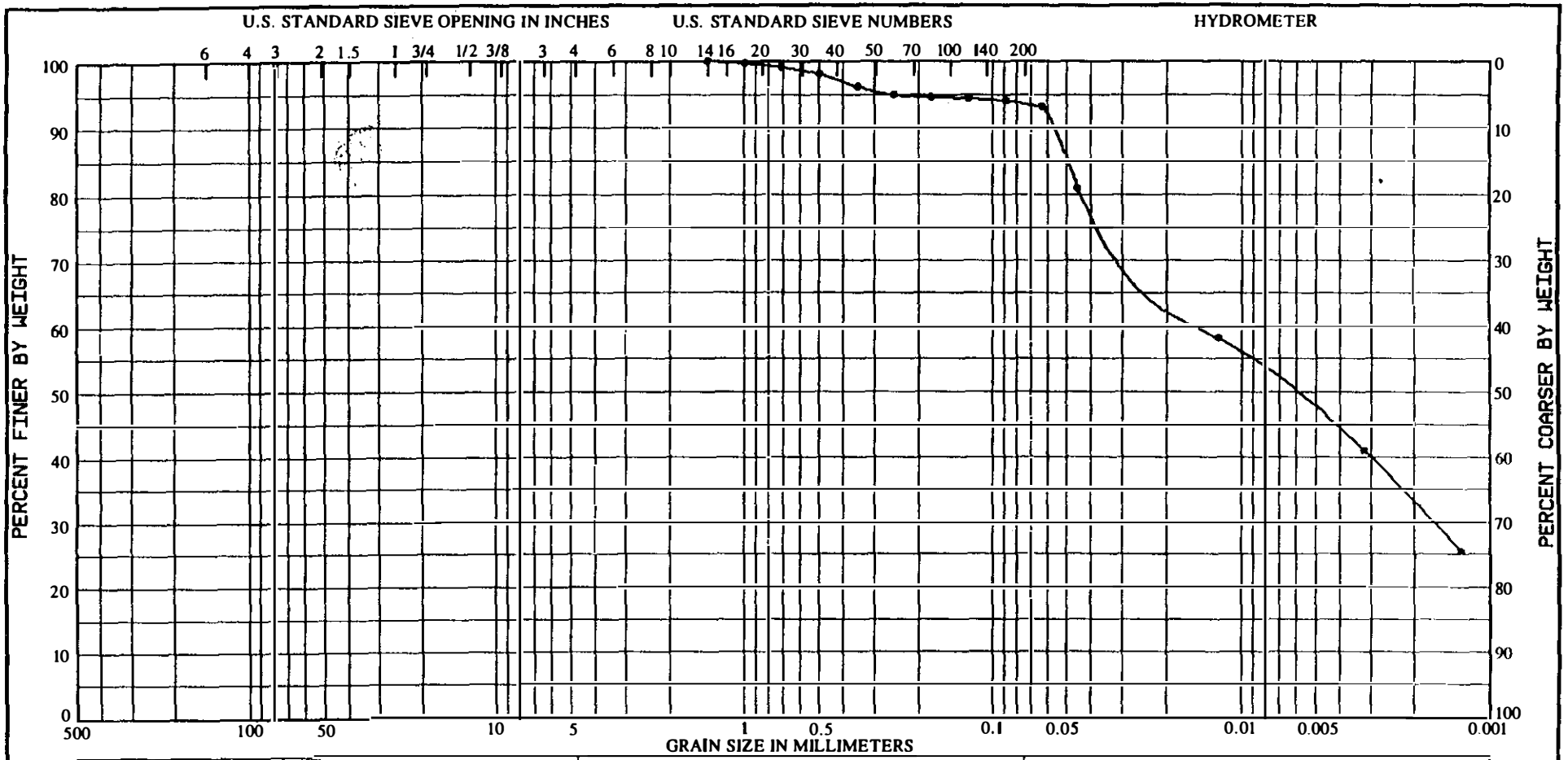
Sample No.	Depth	Classification	Nat w %	LL	PL	PI
A	1.0-1.3'	VISUAL) GRAY, SILTY SAND (SM), WITH A TRACE OF SHELL FRAGMENTS.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/930A
Boring No. KAV-23
Date 01/20/95

GRADATION CURVES

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

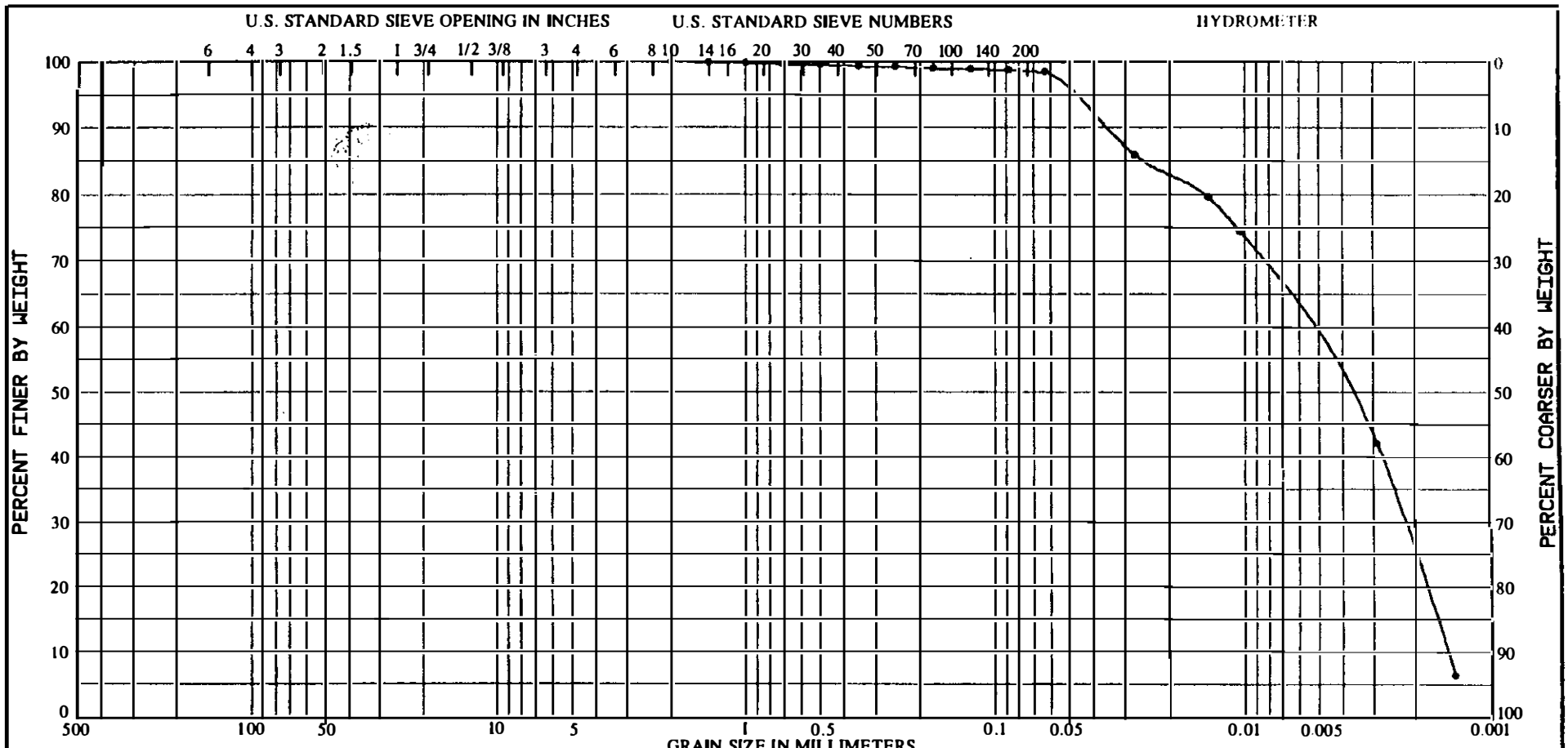
Sample No.	Depth	Classification	Nat w%	LL	PL	PI
D-I	3.1-3.6'	(VISUAL) GRAY, INORGANIC SILT HIGH LL	72.6			
		(MH), WITH A TRACE OF SAND SIZES.				
		SP. GRAVITY = 2.60				
		WET DENSITY = 96.9 PCF.				
		DRY DENSITY = 56.2 PCF.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/930
Boring No. KAV-23
Date 01/20/95

GRADATION CURVES

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

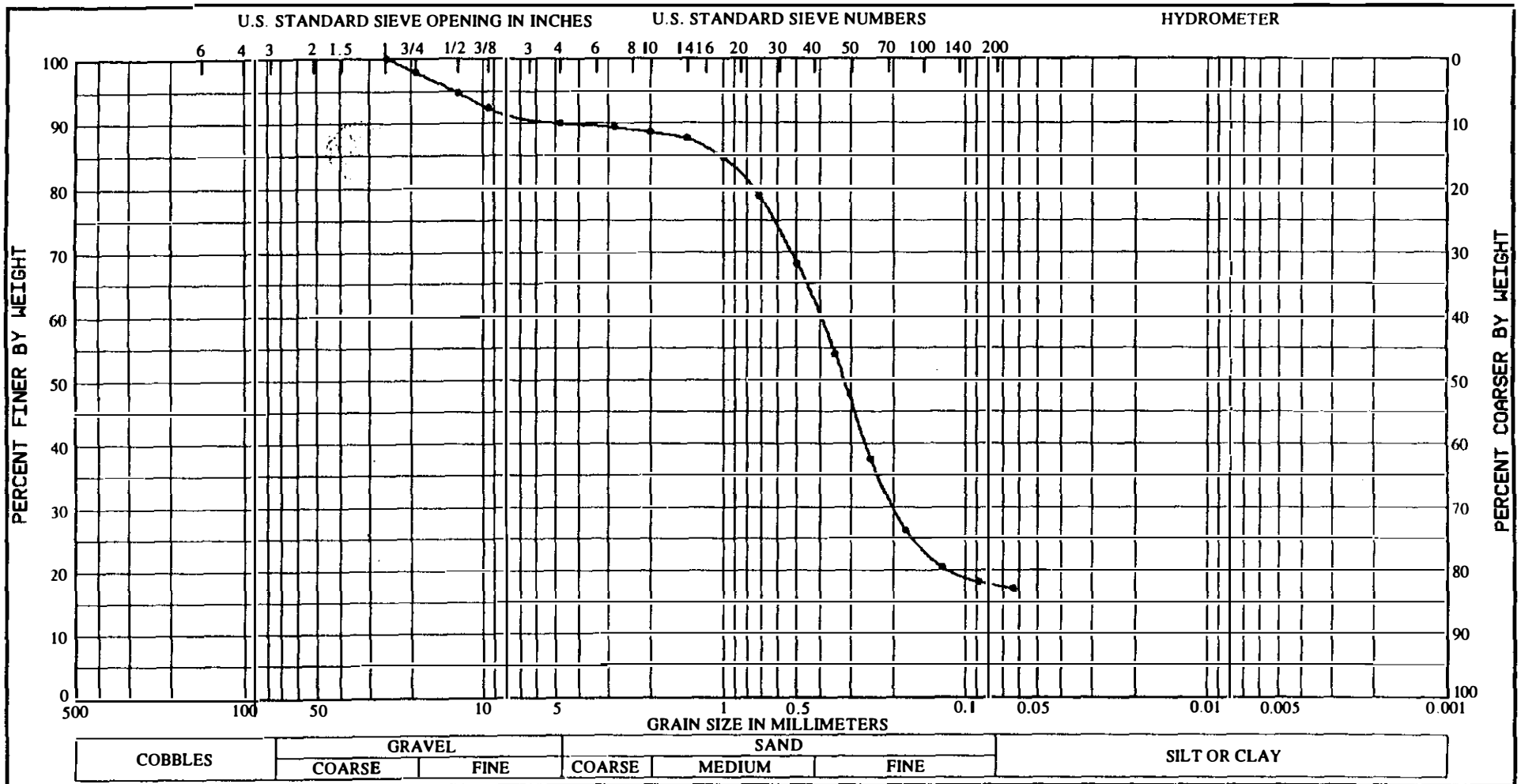
Sample No.	Depth	Classification	Nat w%	LL	PL	PI
B	6.0-6.3'	(VISUAL) GRAY, INORGANIC SILT HIGH LL (MH), WITH A TRACE OF SAND SIZES.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/930B
Boring No. KAV-23
Date 01/20/95

GRADATION CURVES

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



Sample No.	Depth	Classification	Nat w%	LL	PL	PI
D-2	7.4-7.9'	(VISUAL) GRAY, SILTY SAND (SM), WITH	12.3			
		A LITTLE GRAVEL SIZES.				
		SP. GRAVITY = 2.66				
		WET DENSITY = 138.9 PCF.				
		DRY DENSITY = 123.8 PCF.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/930
Boring No. KAV-23
Date 01/20/95

GRADATION CURVES

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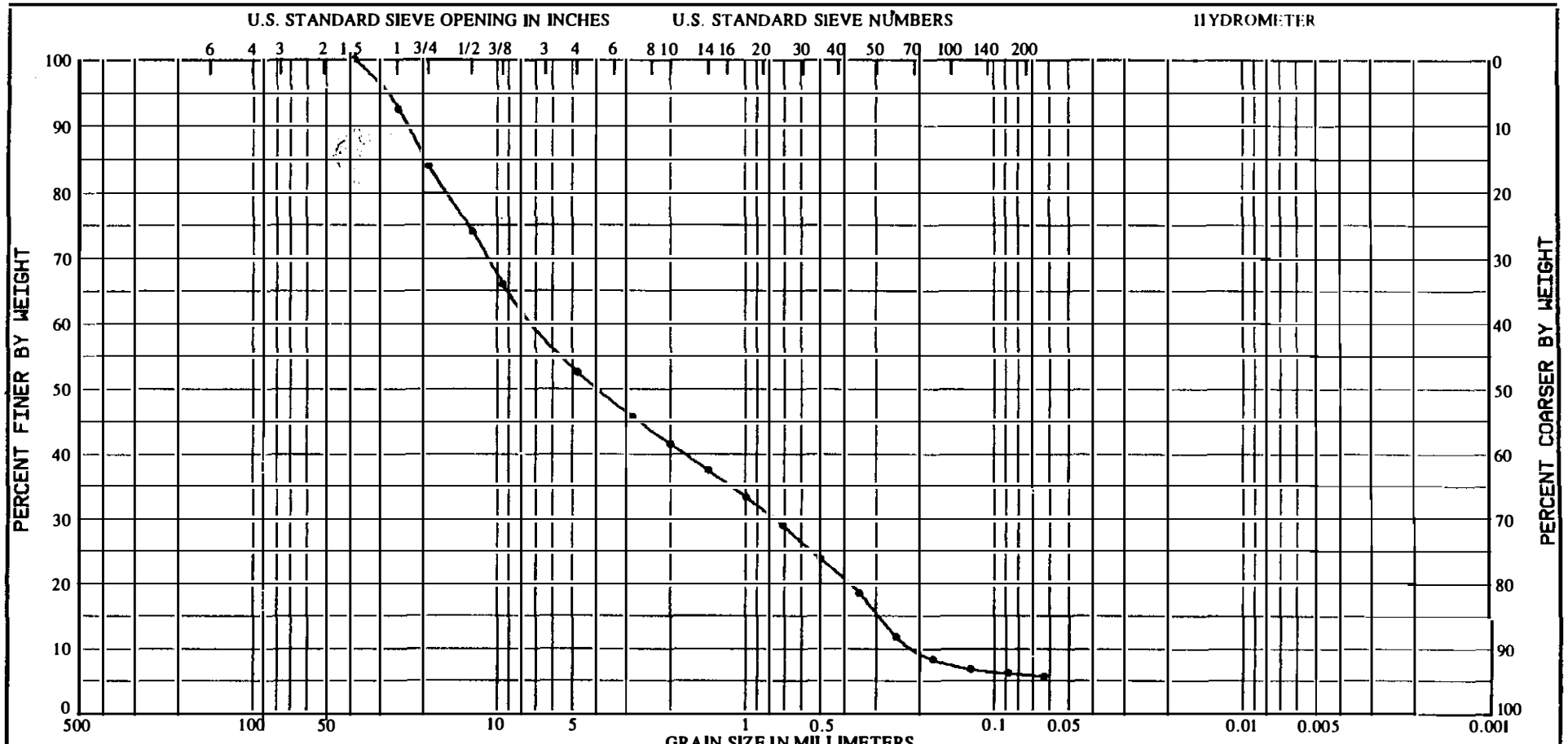
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



Sample No.	Depth	Classification	Nat w%	LL	PL	PI
C	8.4-9.2'	(VISUAL) GRAY, SANDY POORLY GRADED SILTY GRAVEL (GP-GM).				

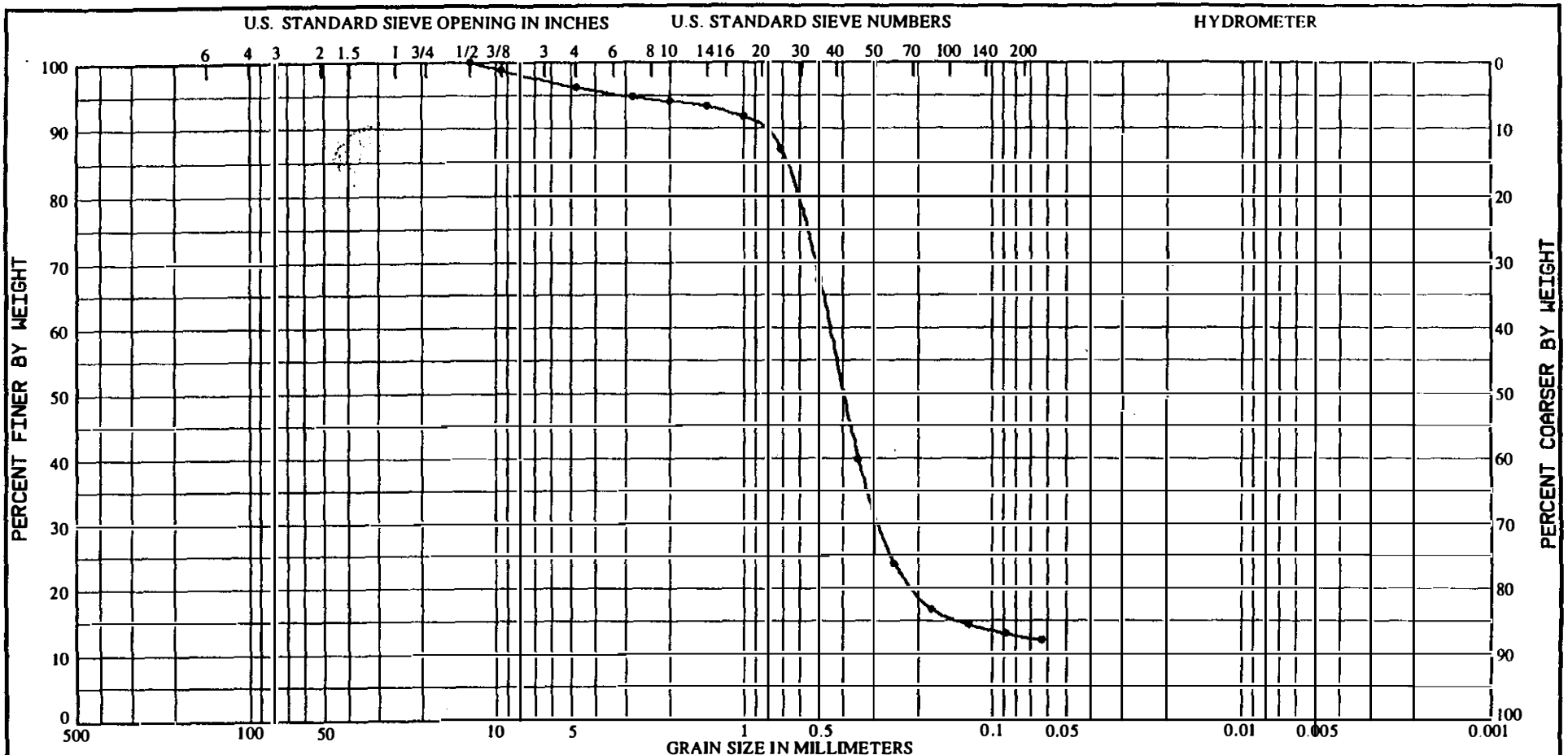
Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/930C
Boring No. KAV-23
Date 01/20/95

GRADATION CURVES

75

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Sample No.	Depth	Classification	Nat w%	LL	PL	PI
D	11.0-11.2'	(VISUAL) TAN, SILTY SAND (SM), WITH A TRACE OF GRAVEL SIZES.				

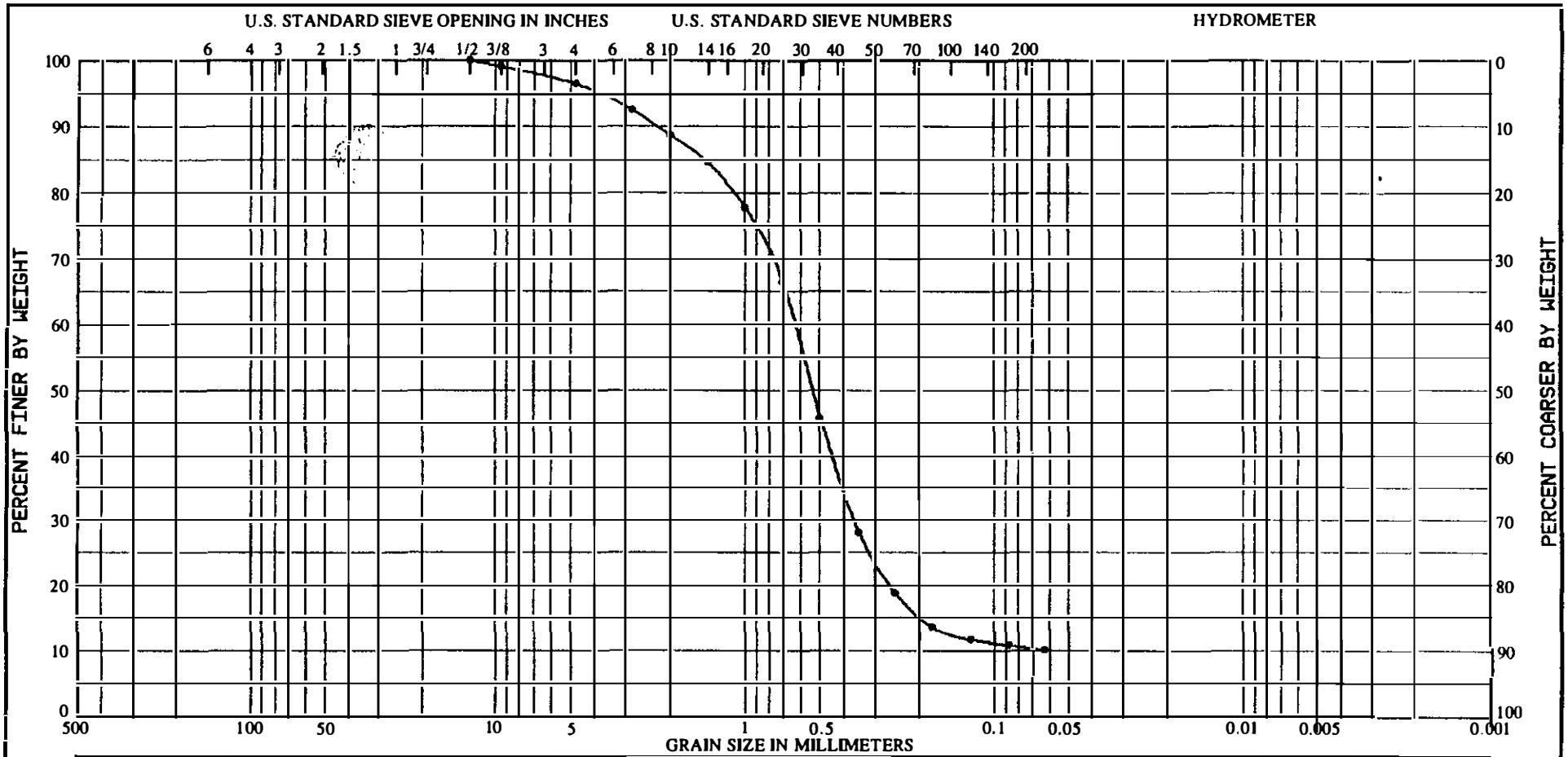
Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/930D
Boring No. KAV-23
Date 01/20/95

GRADATION CURVES

76

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Sample No.	Depth	Classification	Nat w %	LL	PL	PI
D-3	11.8-12.3'	(VISUAL) TAN, POORLY GRADED SILTY SAND	13.3			
		(SP-SM), WITH A TRACE OF GRAV EL SIZES.				
		SP. GRAVITY = 2.67				
		WET DENSITY = 137.3 PCF.				
		DRY DENSITY = 116.8 PCF.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/930
Boring No. KAV-23
Date 01/20/95

GRADATION CURVES

77

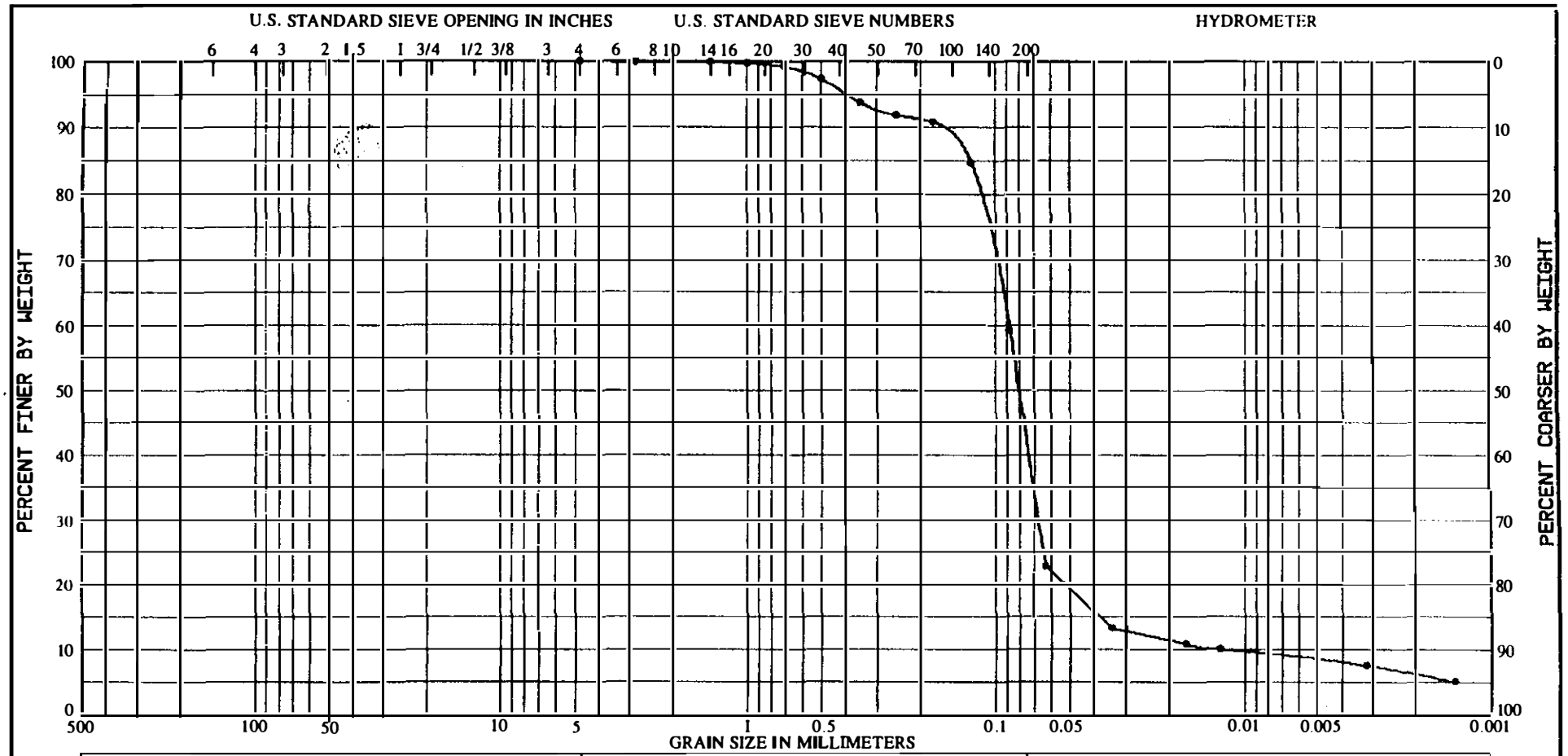
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE, MARIETTA, GA. 30060

Project: *DELAWARE BAY*		Boring No. *KAV-24*	
Location: *VIBRA CORE SAMPLES*		Lab No. *184/931*	
Boring Depth (ft): *18.50*	Elevation:	Work order: *7476*	
Datum/Notes: *See grain size data on enclosed gradation curves.*		Requisition: *CENAP-95-707*	

Elev (feet)	Depth (feet)	Leg-end	Material Description	Comments
				(Density Units = pcf)
	1			MA (A) 0.9 - 1.2'
			GRAY, POORLY GRADED SILTY SAND (SP-SM).	
	2			
	3			SA D-1 2.5 - 3.0' WET DENSITY = 127.2, DRY DENSITY = 111.4, MC = 14.2 %.
			TAN, POORLY GRADED SAND (SP).	
	4			SA (B) 3.4 - 3.8'
			GRAY, POORLY GRADED SILTY SAND (SP-SM).	
	5			
	6			
	7			
	8		GRAY, INORGANIC SILT HIGH LL (MH), WITH A LITTLE SAND SIZES AND A TRACE OF MICA.	MA D-2 7.4 - 7.9' WET DENSITY = 98.9, DRY DENSITY = 62.5, MC = 58.2 %.
	9			
	10			
	11			MA (C) 10.4 - 10.7'
	12			SA D-3 12.0 - 12.5' WET DENSITY = 126.9, DRY DENSITY = 106.3, MC = 19.4 %.
	13		TAN, POORLY GRADED SILTY SAND (SP-SM), WITH A TRACE OF GRAVEL SIZES AND POCKETS OF GRAY, INORGANIC SILT HIGH LL (MH).	
	14			SA (D) 14.2 - 14.4'
	15		TAN, POORLY GRADED SAND (SP), WITH A TRACE OF GRAVEL SIZES.	
	16			MA D-4 15.9 - 16.4' WET DENSITY = 98.1, DRY DENSITY = 76.2, MC = 28.7 %.
	17		GRAY, INORGANIC SILT HIGH LL (MH), WITH A TRACE OF SAND SIZES AND MICA.	
	18		TANNISH GRAY, POORLY GRADED SILTY SAND (SP-SM), WITH A TRACE OF GRAVEL SIZES.	SA (E) 17.5 - 17.8'
	19			

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Sample No.	Depth	Classification	Nat w %	LL	PL	PI
A	0.9-1.2'	(VISUAL) GRAY, SILTY SAND (SM).				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/931A
Boring No. KAV-24
Date 01/21/95

GRADATION CURVES

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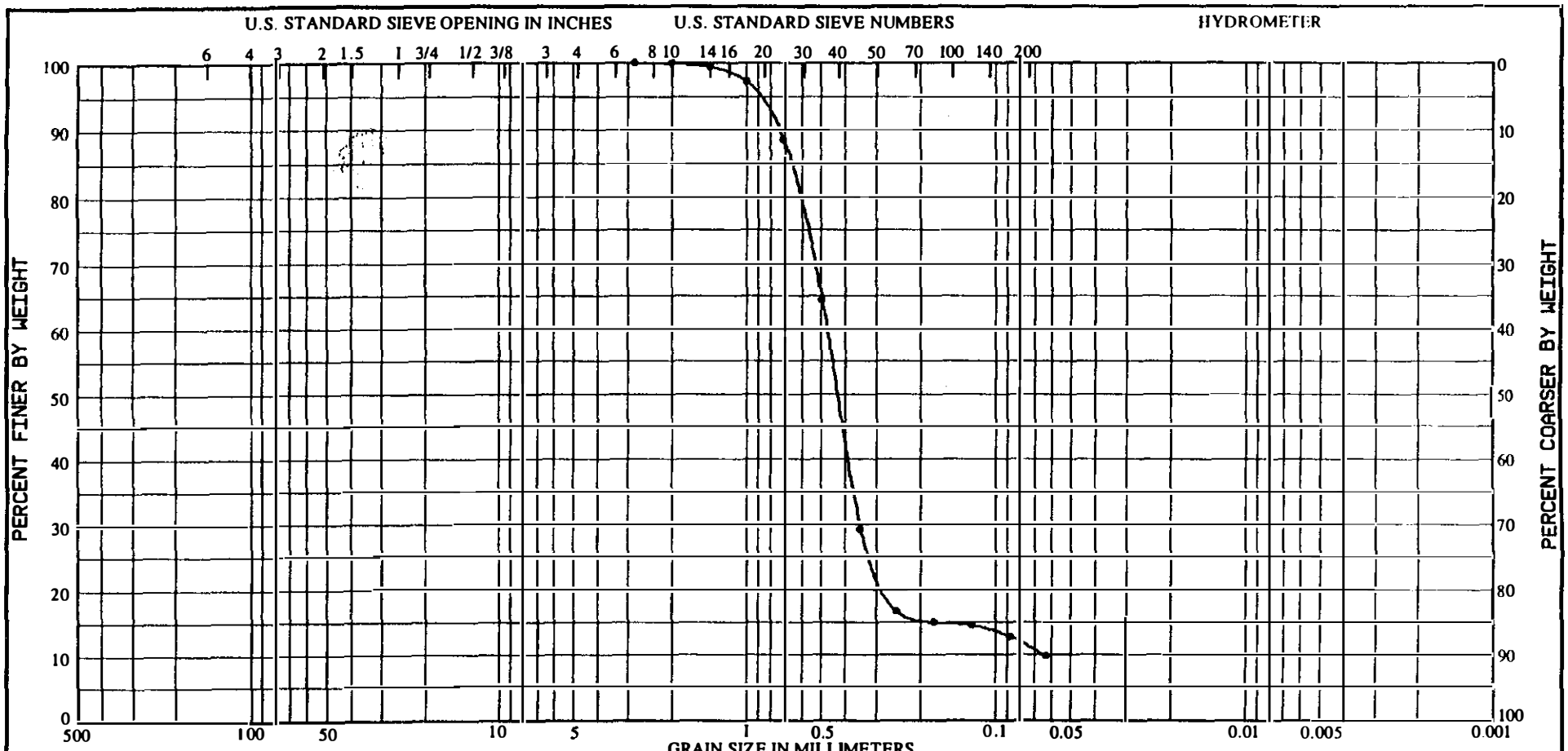
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Sample No.	Depth	Classification	Nat w%	LL	PL	PI
D-1	2.5-3.0'	(VISUAL) GRAY, POORLY GRADED SILTY SAND	14.2			
		(SP-SM).				
		SP. GRAVITY = 2.66				
		WET DENSITY = 127.2 PCF.				
		DRY DENSITY = 111.4 PCF.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/931
Boring No. KAV-24
Date 01/20/95

GRADATION CURVES

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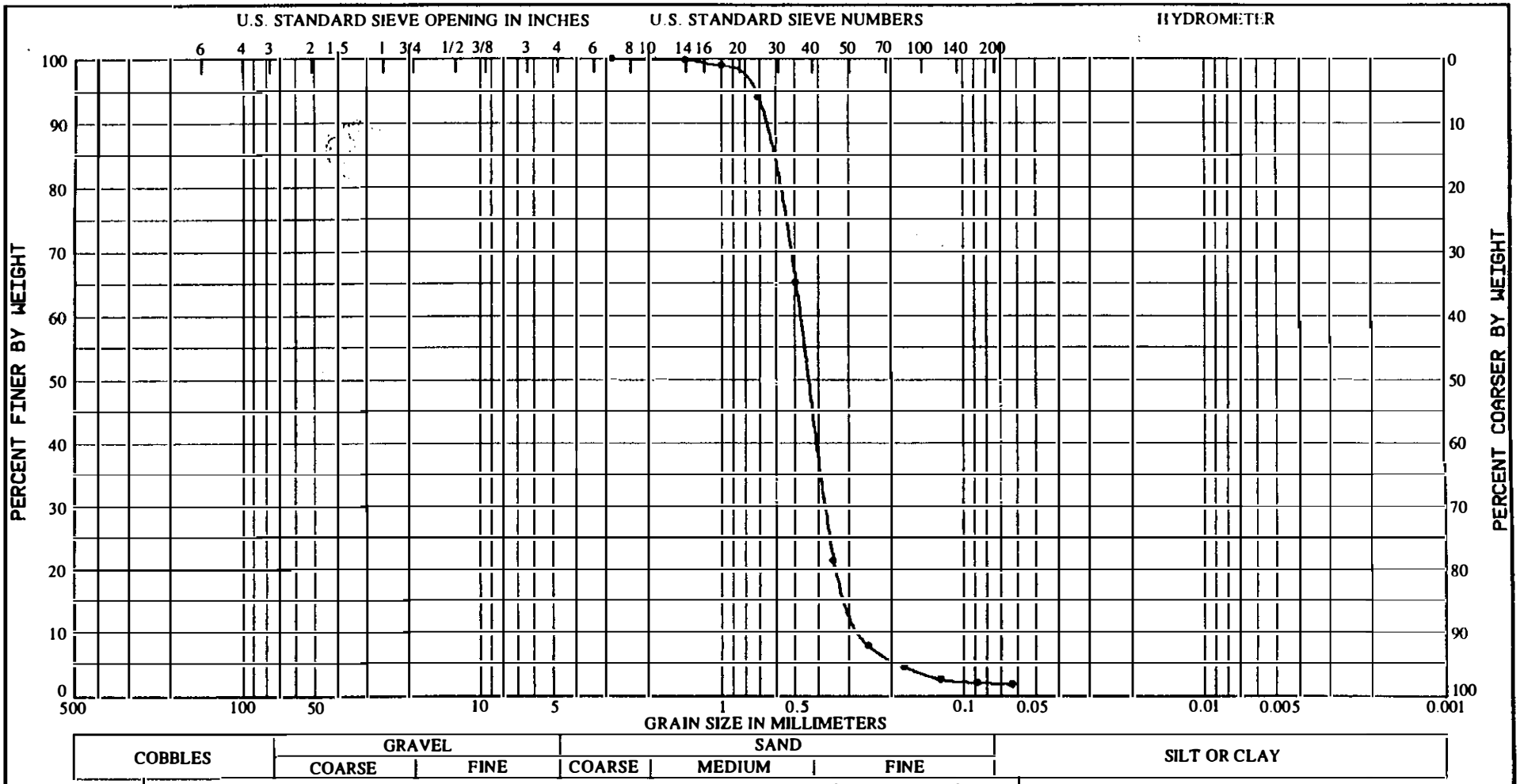
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



Sample No.	Depth	Classification	Nat w%	LL	PL	PI
B	3.4-3.8'	(VISUAL) TAN, POORLY GRADED SAND (SP).				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/931B
Boring No. KAV-24
Date 01/21/95

GRADATION CURVES

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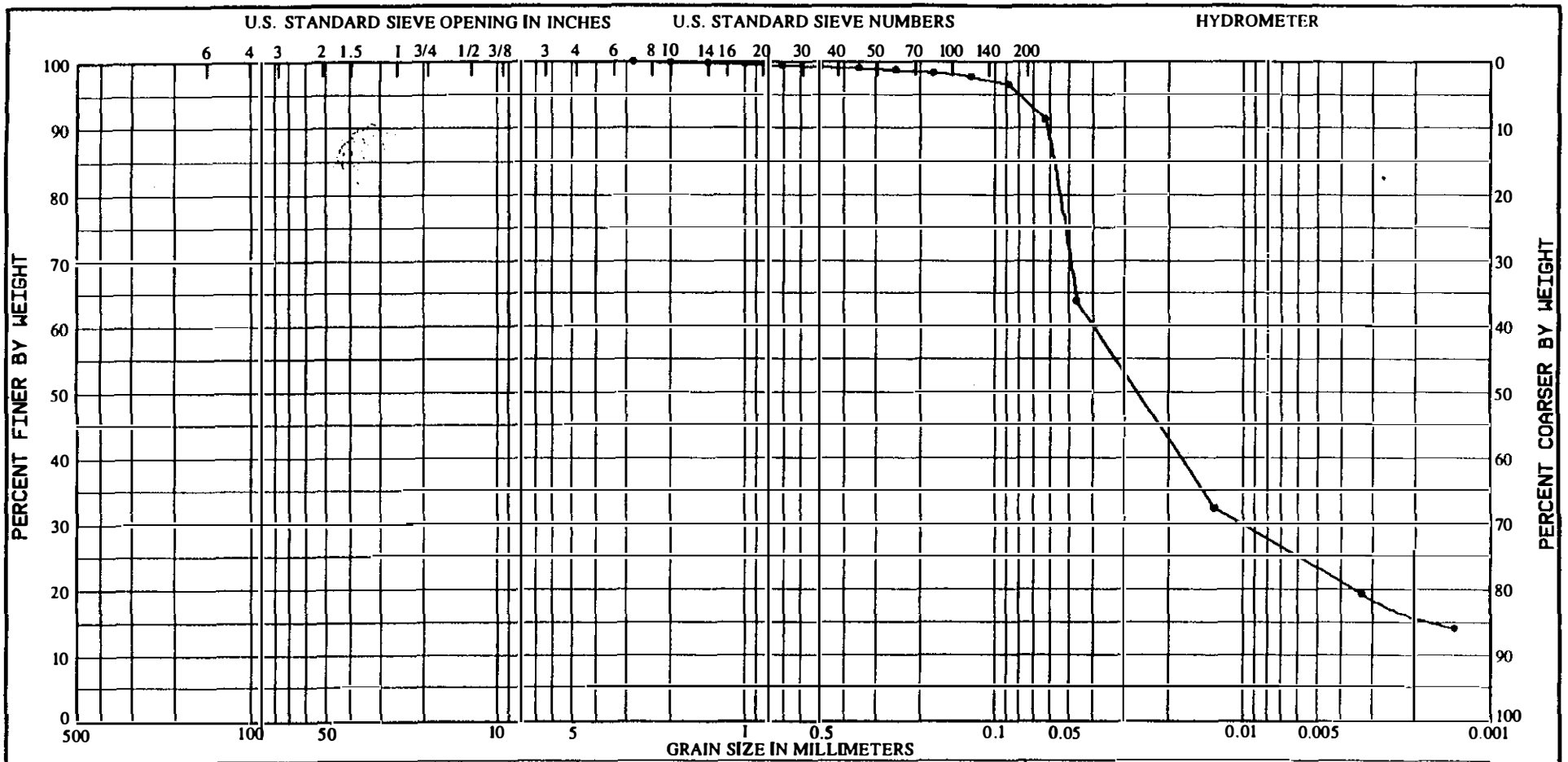
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Sample No.	Depth	Classification	Nat w%	LL	PL	PI
D-2	7.4-7.9'	(VISUAL) TAN, INORGANIC SILT HIGH LL	58.2			
		(MH), WITH A TRACCE OF SAND SIZES AND				
		MICA. SP. GRAVITY = 2.66				
		WET DENSITY = 98.9 PCF.				
		DRY DENSITY = 62.5 PCF.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/931
Boring No. KAV-24
Date 01/25/95

GRADATION CURVES

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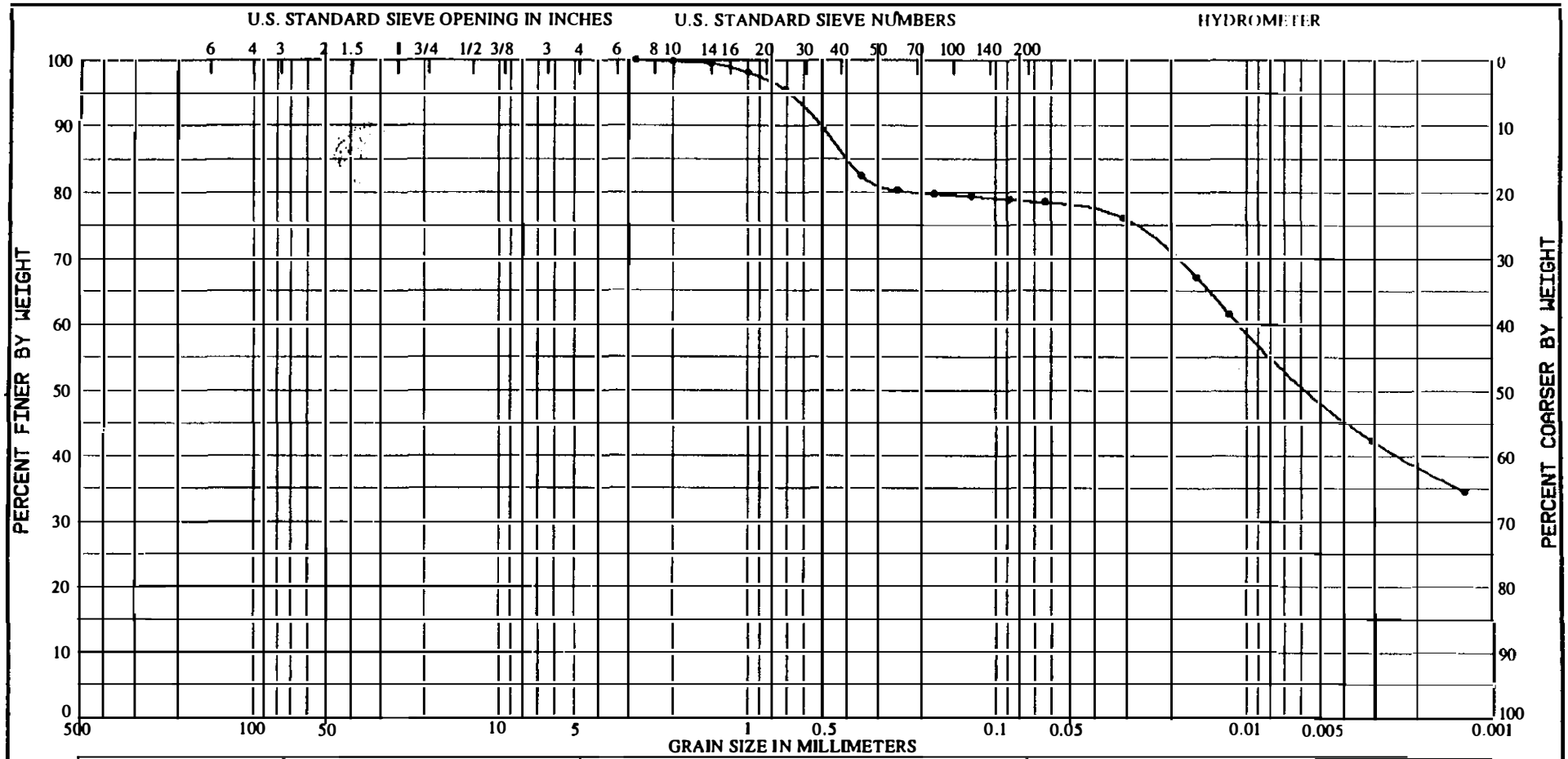
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Sample No.	Depth	Classification	Nat w%	LL	PL	PI
C	10.4-10.7'	(VISUAL) GRAY, INORGANIC SILT HIGH LL (MH), WITH SOME SAND SIZES.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/931C
Boring No. KAV-24
Date 01/21/95

GRADATION CURVES

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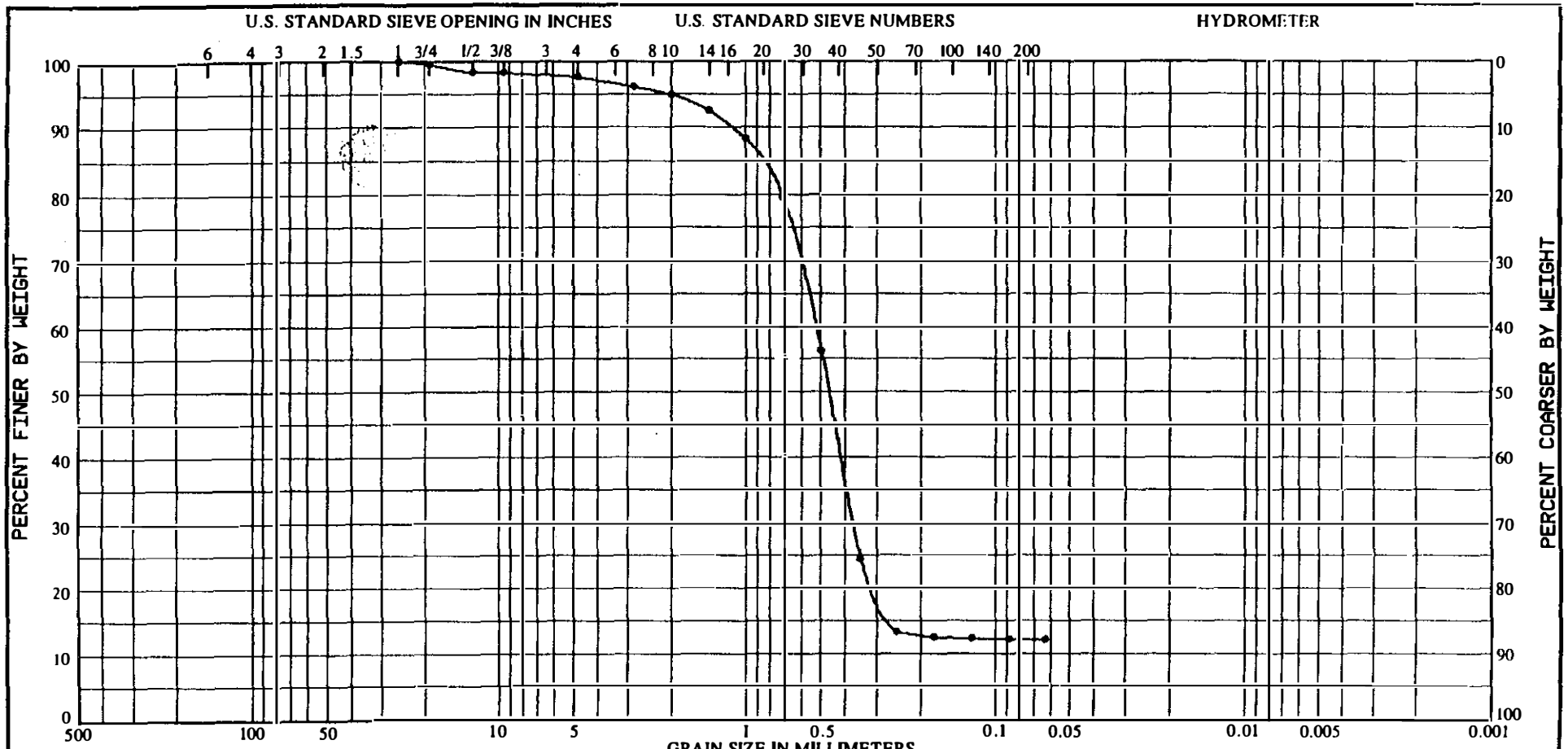
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Sample No.	Depth	Classification	Nat w%	LL	PL	PI
D-3	12.0-12.5'	(VISUAL) TAN, POORLY GRADED SILTY SAND	19.4			
		(SP-SM), WITH A TRACE OF GRAVEL SIZES				
		AND MICA. SP. GRAVITY = 2.66				
		WET DENSITY = 126.9 PCF.				
		DRY DENSITY = 106.3 PCF.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/931
Boring No. KAV-24
Date 01/20/95

GRADATION CURVES

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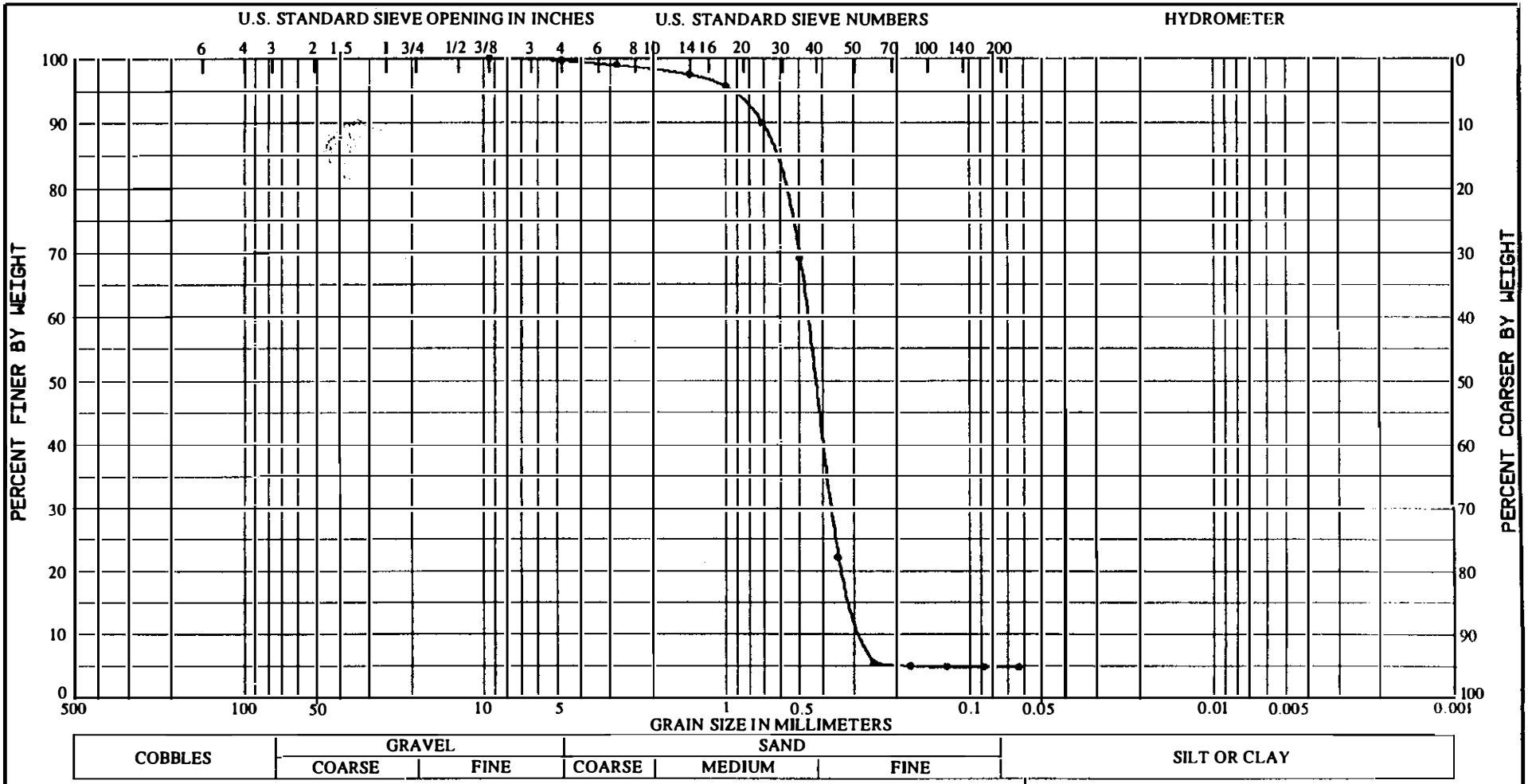
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



Sample No.	Depth	Classification	Nat w%	LL	PL	PI
D	14.2-14.4'	(VISUAL) TAN, POORLY GRADED SAND (SP).				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/931D
Boring No. KAV-24
Date 01/21/95

GRADATION CURVES

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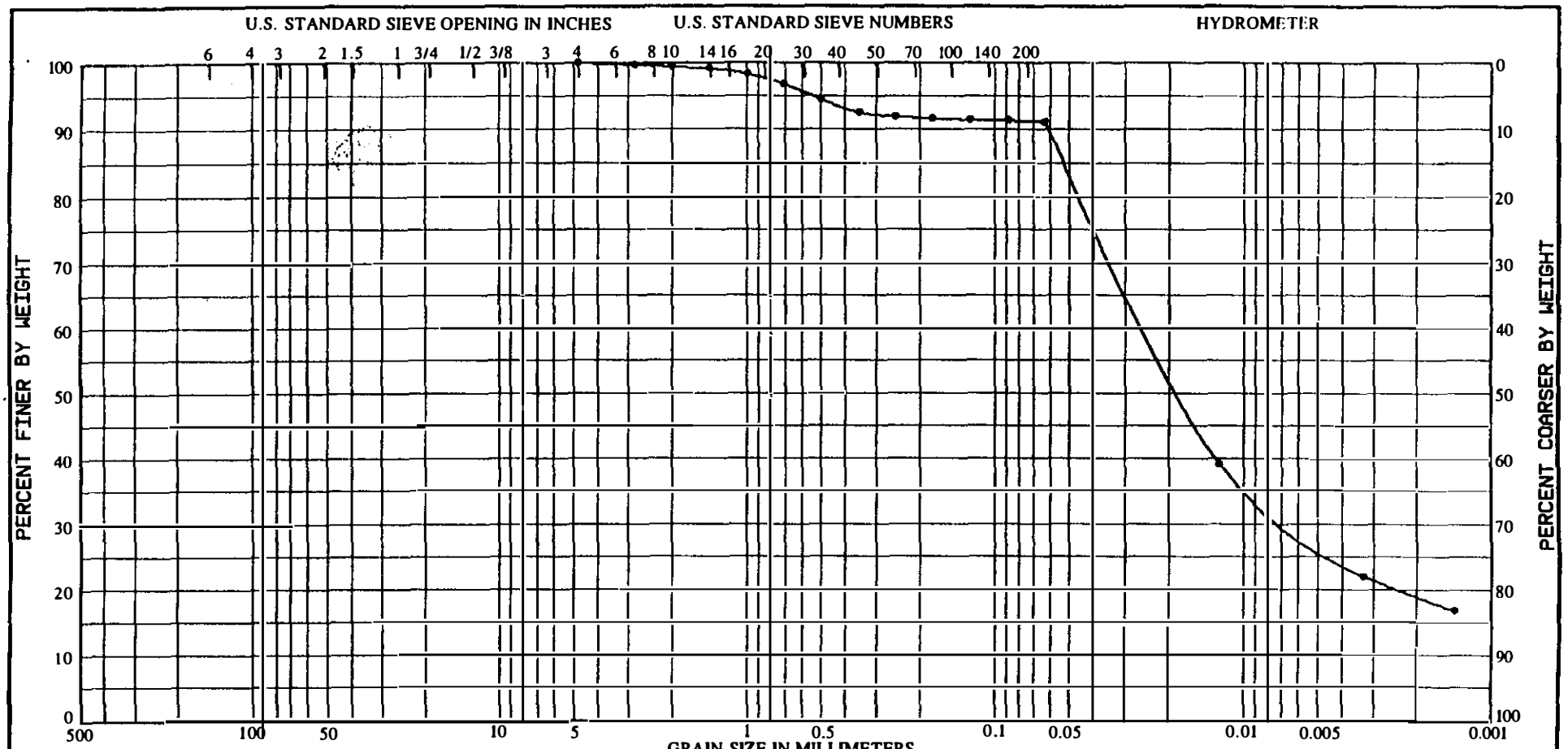
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Sample No.	Depth	Classification	Nat w%	LL	PL	PI
D-4	15.7-16.2'	(VISUAL) TAN. INORGANIC SILT HIGH LL	28.7			
		(MH). WITH A TRACE OF SAND SIZES & MICA.				
		SP. GRAVITY = 2.68				
		WET DENSITY = 98.1 PCF.				
		DRY DENSITY = 76.2 PCF.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/931
Boring No. KAV-24
Date 01/25/95

GRADATION CURVES

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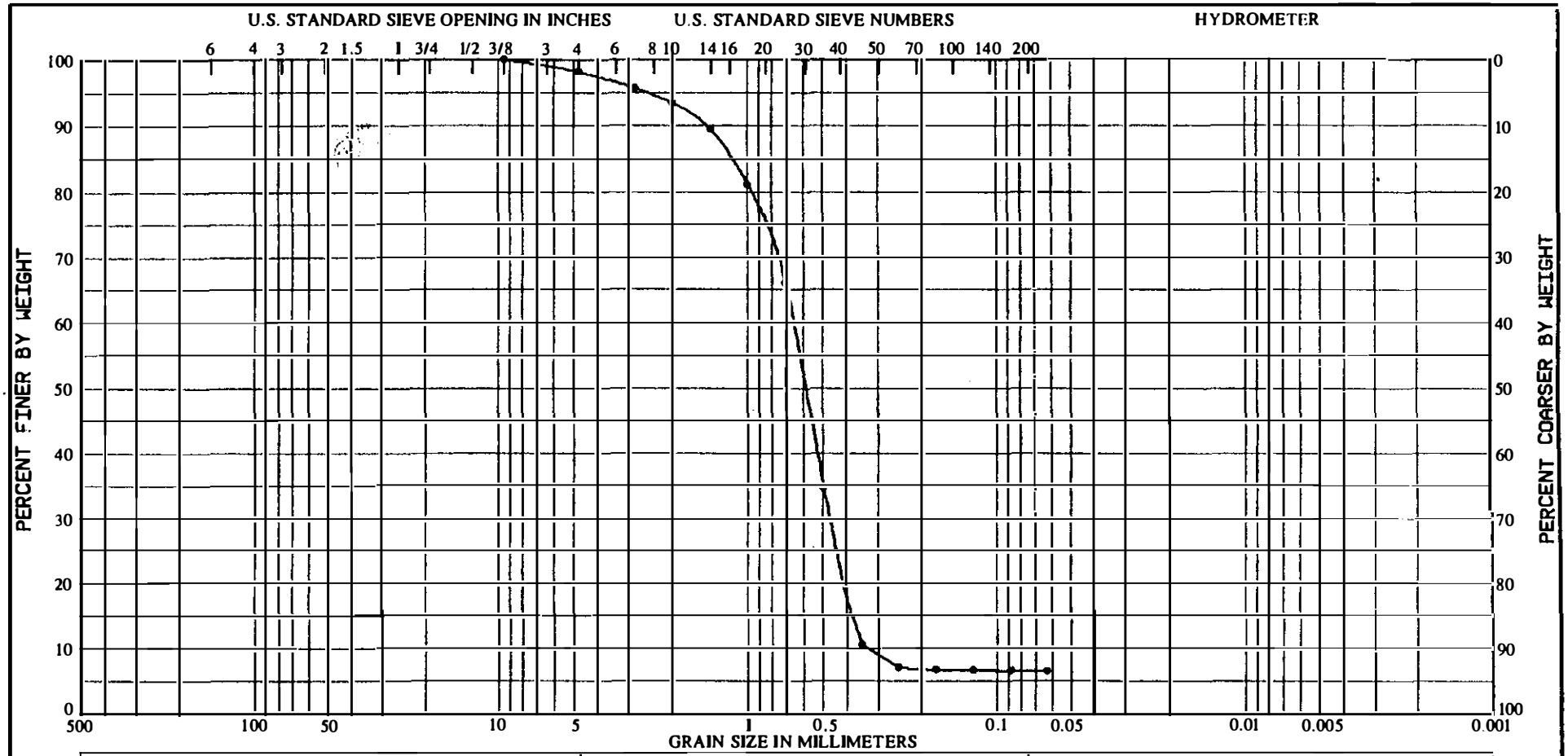
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Sample No.	Depth	Classification	Nat w%	LL	PL	PI
E	17.5-17.8'	(VISUAL) TANNISH GRAY, POORLY GRADED SILTY SAND (SP-SM), WITH A TRACE OF GRAVEL SIZES.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/931E
Boring No. KAV-24
Date 01/21/95

GRADATION CURVES

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DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE, MARIETTA, GA. 30060

Project: *DELAWARE BAY*		Boring No. *KAV-25*	
Location: *VIBRA CORE SAMPLES*		Lab No. *184/932*	
Boring Depth (ft): *17.00*	Elevation:	Work order: *7476*	
Datum/Notes: *See grain size data on enclosed gradation curves.*		Requisition: *CENAP-95-707*	

Elev (feet)	Depth (feet)	Legend	Material Description	Comments
				(Density Units = pcf)
	1		TAN AND GRAY, POORLY GRADED SAND (SP), WITH A TRACE OF GRAVEL SIZES AND GRAVEL SIZE SHELL .	SA (A) 1.2 - 1.5'
	2		----------	
	3			SA D-1 2.8 - 3.3' WET DENSITY = 137.9, DRY DENSITY = 121.1, MC = 13.9 %.
	4			
	5			
	6			MA (B) 5.6 - 5.8'
	7			MA D-2 6.9 - 7.4' WET DENSITY = 101.2, DRY DENSITY = 62.2, MC = 62.5 %.
	8			
	9		GRAY, INORGANIC SILT HIGH LL (MH), WITH OCCASIONAL LENSES OF POORLY GRADED SILTY SAND (SP-SM) AND A TRACE OF MICA.	MA (C) 8.9 - 9.2'
	10			
	11			MA (D) 11.3 - 11.5'
	12			
	13			MA D-3 13.4 - 13.9' WET DENSITY = 105.2, DRY DENSITY = 66.3, MC = 58.7 %.
	14			
	15			MA (E) 15.1 - 15.4'
	16			
	17		----------	MA D-4 16.5 - 17.0' WET DENSITY = 92.6, DRY DENSITY = 55.7, MC = 66.2 %.

Date: *01/21/95* **LABORATORY LOG AND SAMPLE DATUM** Sheet No. *1* of *1*

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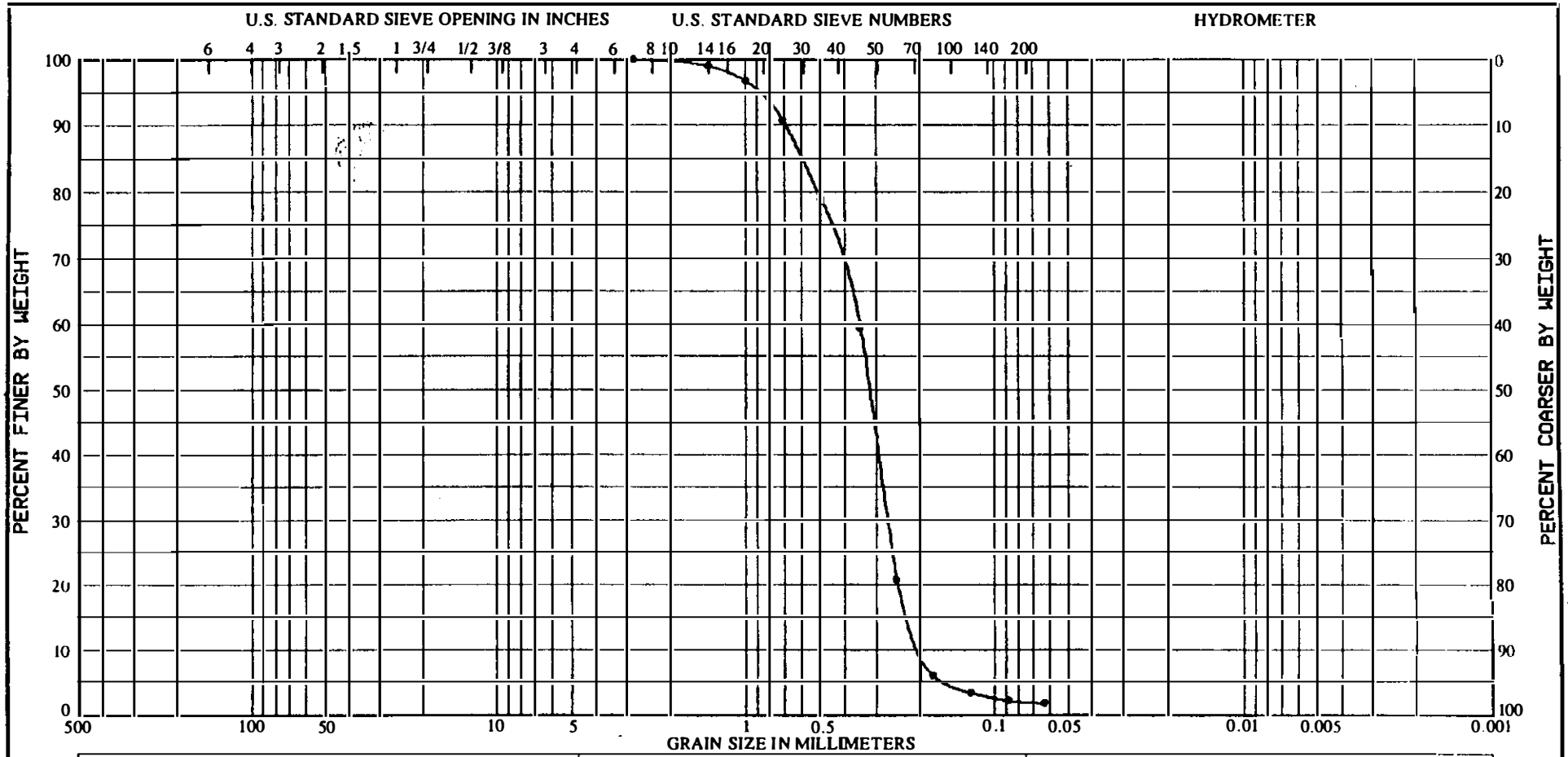
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



Sample No.	Depth	Classification	Nat w%	LL	PL	PI
A	1.2-1.5'	(VISUAL) TAN AND GRAY, POORLY GRADED SAND (SP).				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/932A
Boring No. KAV-25
Date 01/21/95

GRADATION CURVES

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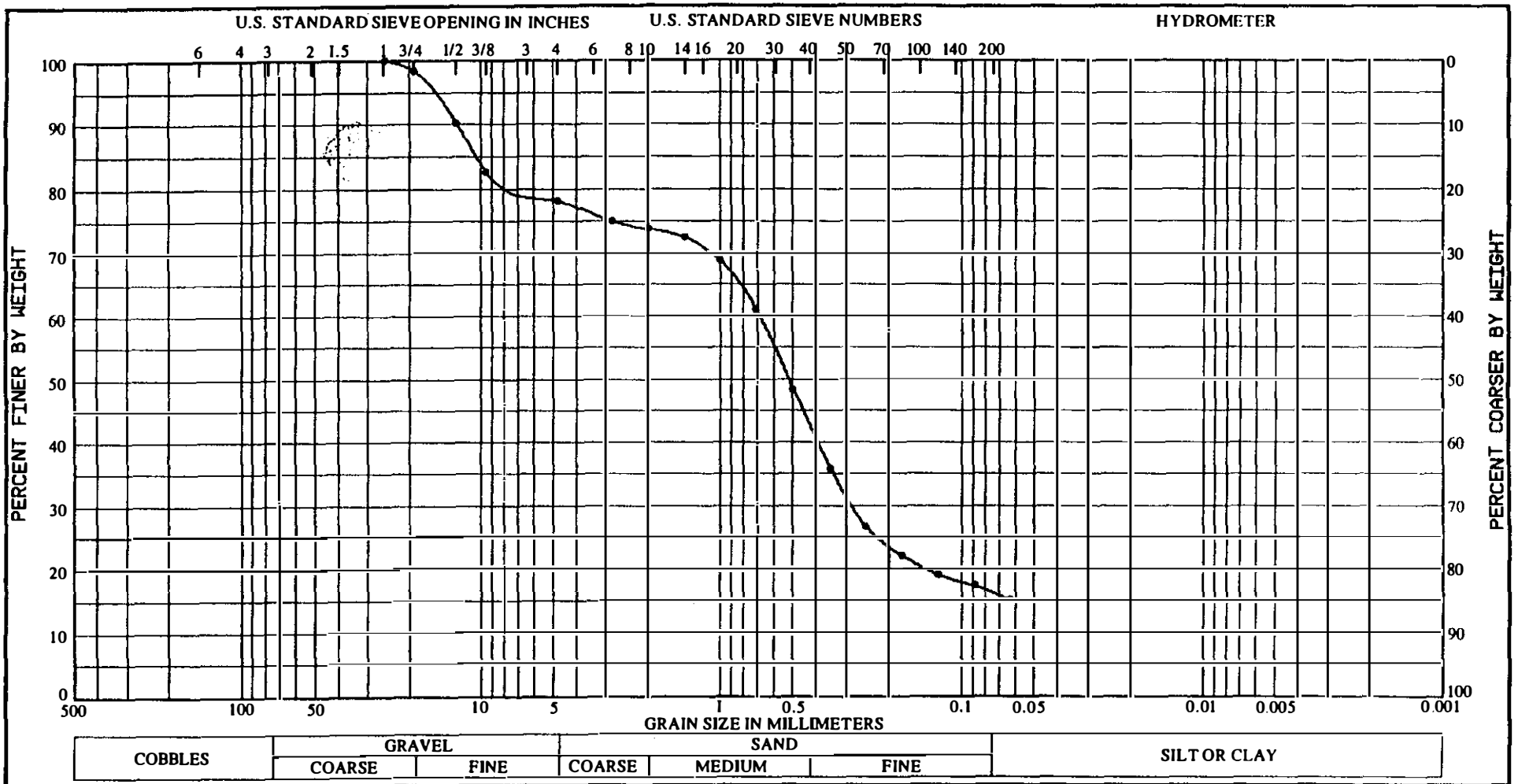
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



Sample No.	Depth	Classification	Nat w%	LL	PL	PI
D-I	2.8-3.3'	(VISUAL) GRAY, SILTY SAND (SM), WITH	13.9			
		SOME GRAVEL SIZES, A TRACE OF SHELL				
		FRAGMENTS AND MICA. SP. GR. = 2.66				
		WET DENSITY = 137.9 PCF.				
		DRY DENSITY = 121.2 PCF.				

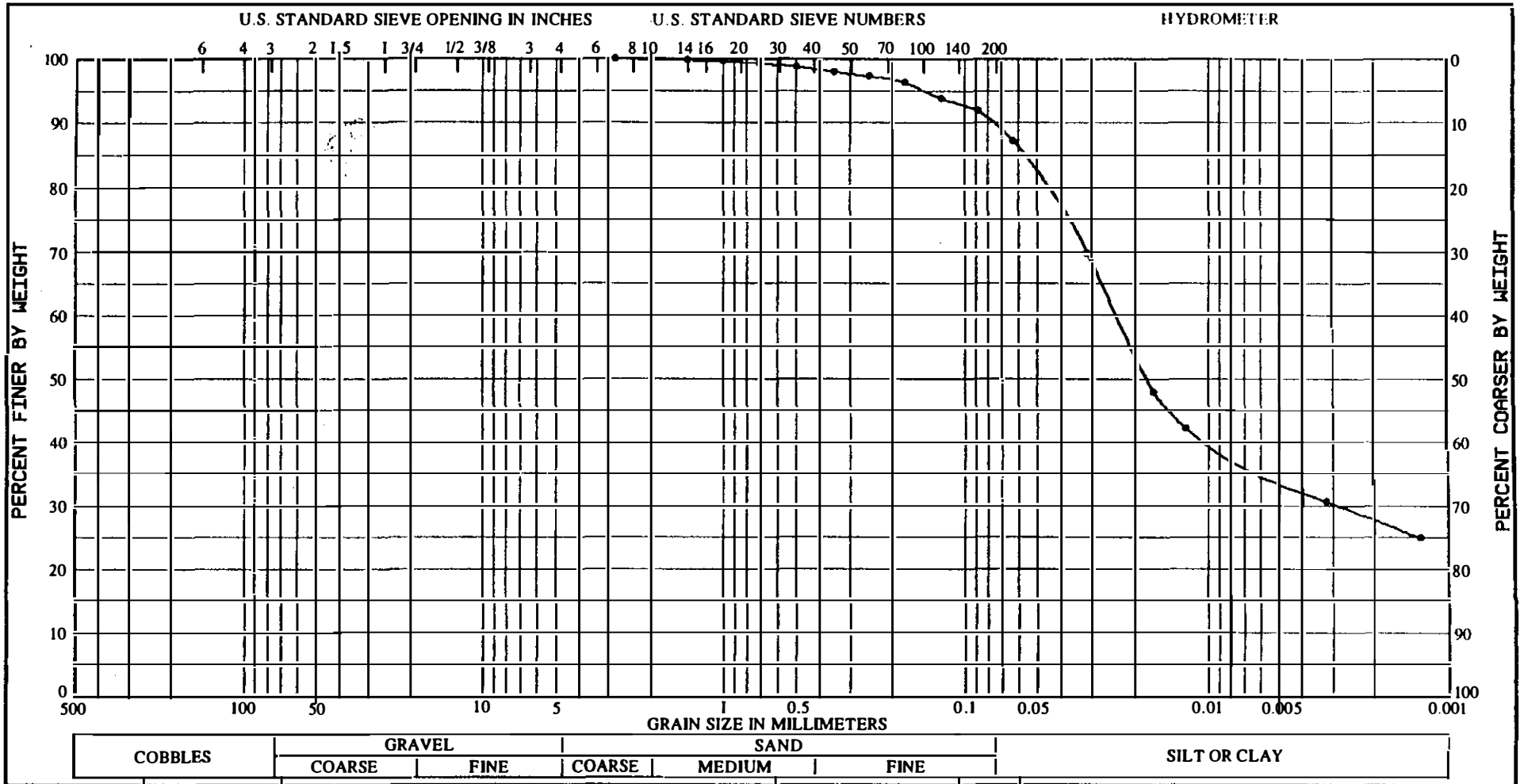
Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/932
Boring No. KAV-25
Date 01/21/95

GRADATION CURVES

06

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



Sample No.	Depth	Classification	Nat w%	LL	PL	PI
B	5.6-5.8'	(VISUAL) GRAY, INORGANIC SILT HIGH LL (MH), WITH A LITTLE SAND SIZES.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/932B
Boring No. KAV-25
Date 01/21/95

GRADATION CURVES

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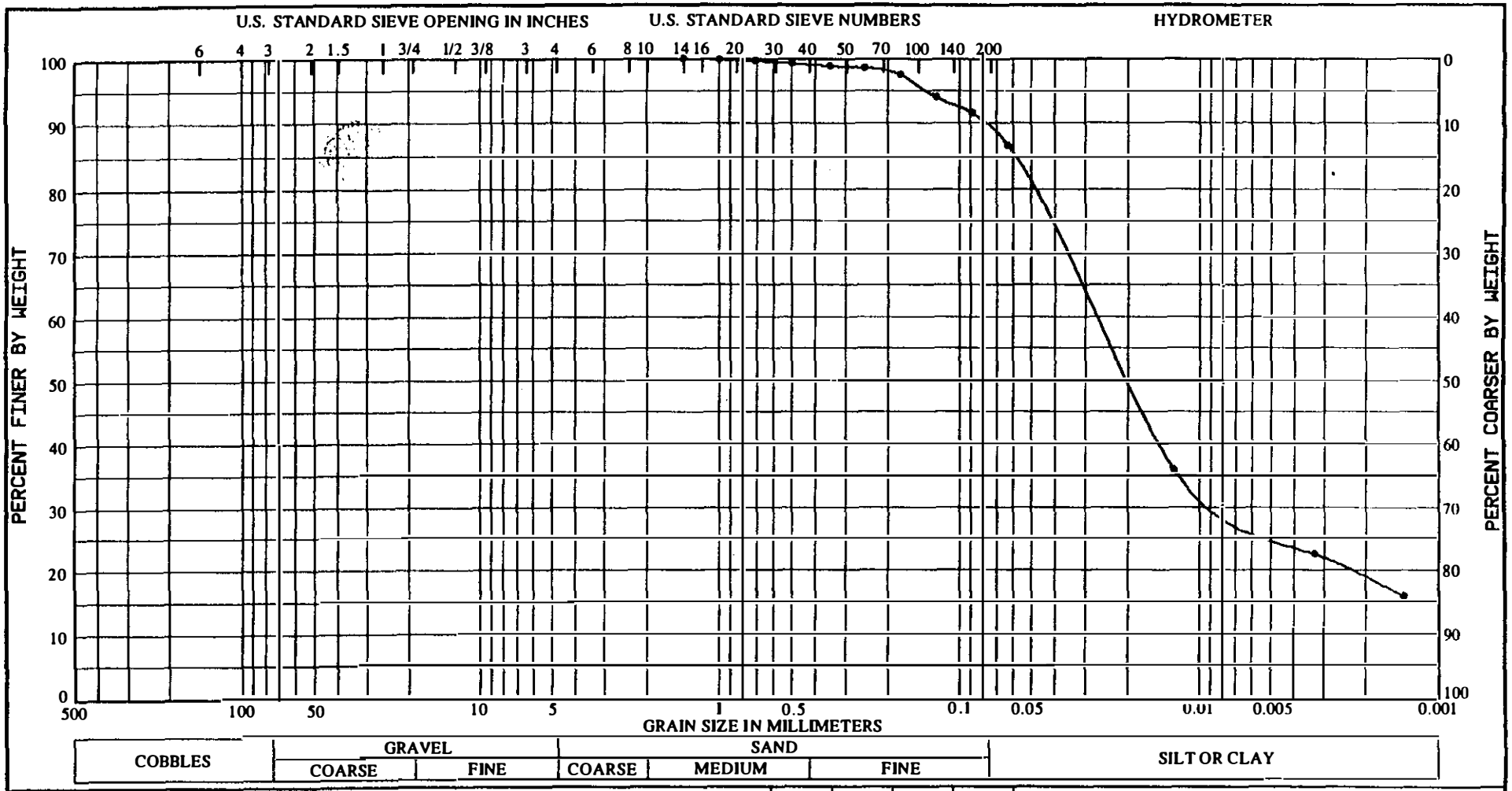
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



Sample No.	Depth	Classification	Nat w %	LL	PL	PI
D-2	6.9-7.4'	(VISUAL) GRAY, INORGANIC SILT HIGH LL	62.5			
		(MH), WITH A LITTLE SAND SIZES.				
		SP. GRAVITY = 2.68				
		WET DENSITY = 101.2 PCF.				
		DRY DENSITY = 62.2 PCF.				

Project OELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/932
Boring No. KAV-25
Date 01/21/95

GRADATION CURVES

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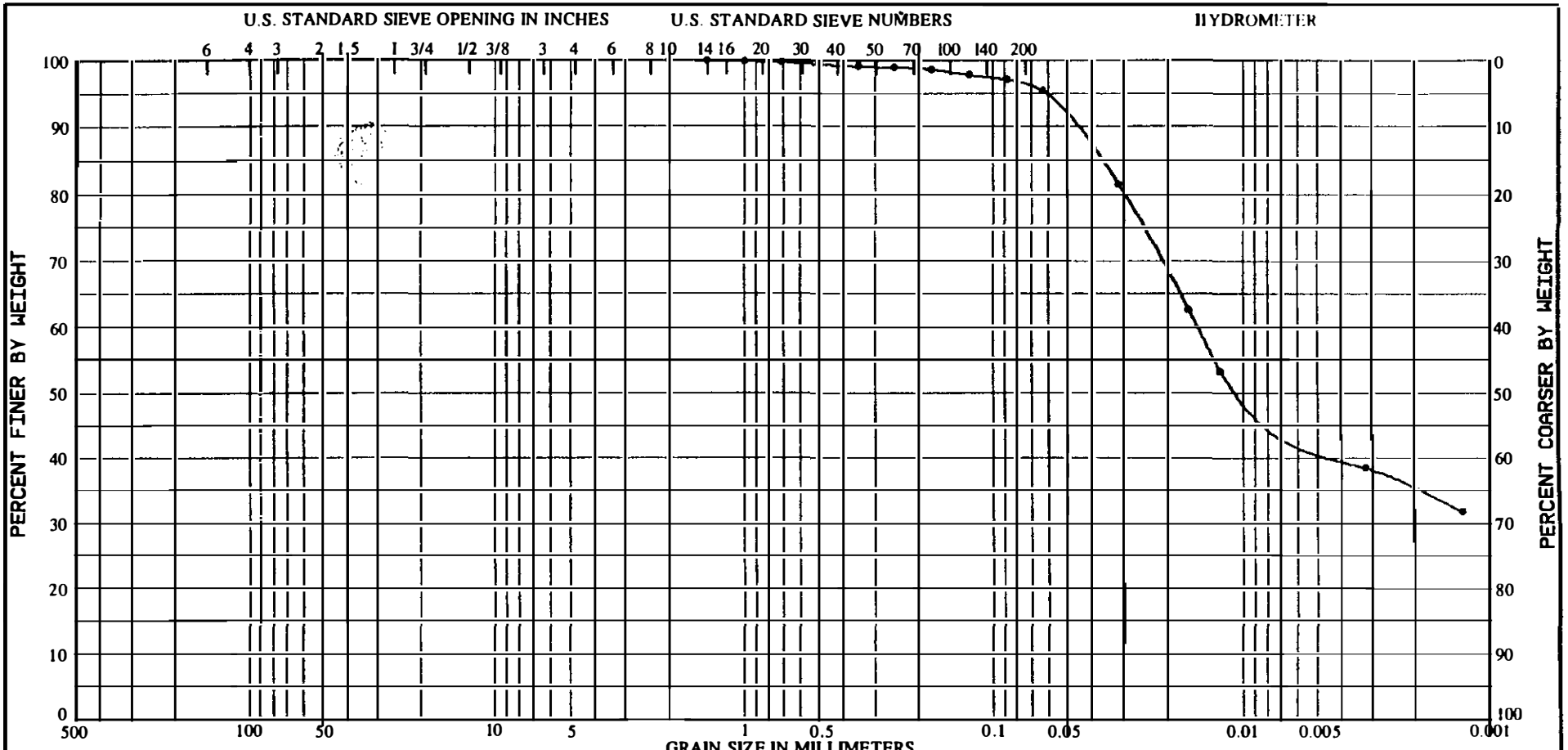
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Sample No.	Depth	Classification	Nat w %	LL	PL	PI
C	8.9-9.2'	(VISUAL) GRAY, INORGANIC SILT HIGH LL (MH). WITH A TRACE OF SAND.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/932C
Boring No. KAV-25
Date 01/21/95

GRADATION CURVES

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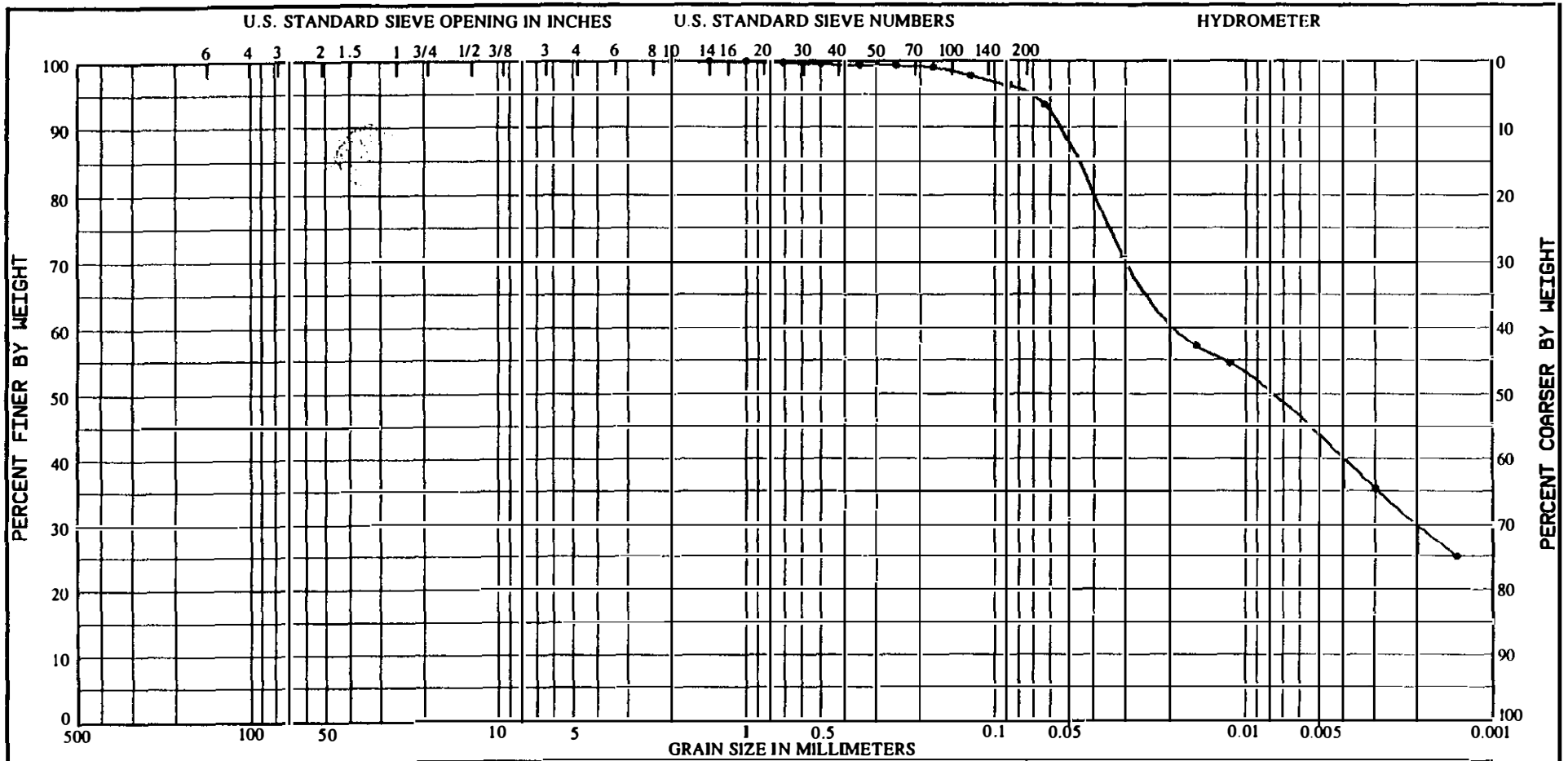
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Sample No.	Depth	Classification	Nat w%	LL	PL	PI
D	11.3-11.5'	(VISUAL) GRAY, INORGANIC SILT HIGH LL (MH), WITH A TRACE OF SAND SIZES.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/932D
Boring No. KAV-25
Date 01/21/95

GRADATION CURVES

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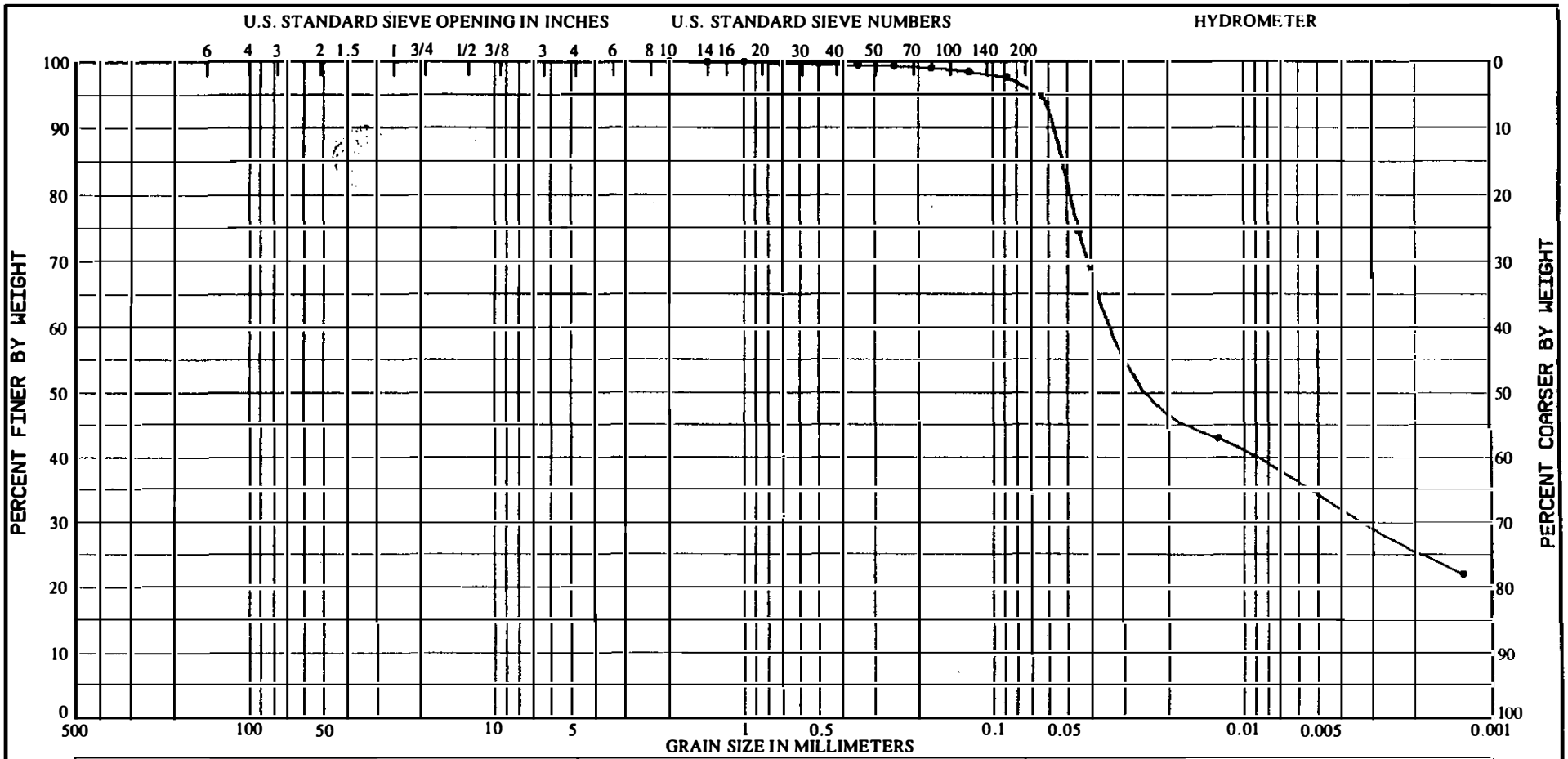
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Sample No.	Depth	Classification	Nat w%	LL	PL	PI
D-3	13.4-13.9'	(VISUAL) GRAY, INORGANIC SILT HIGH LL	58.7			
		(MH), WITH A TRACE OF SAND AND MICA.				
		SP. GRAVITY = 2.65				
		WET DENSITY = 105.2 PCF.				
		DRY DENSITY = 66.3 PCF.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/932
Boring No. KAV-25
Date 01/21/95

GRADATION CURVES

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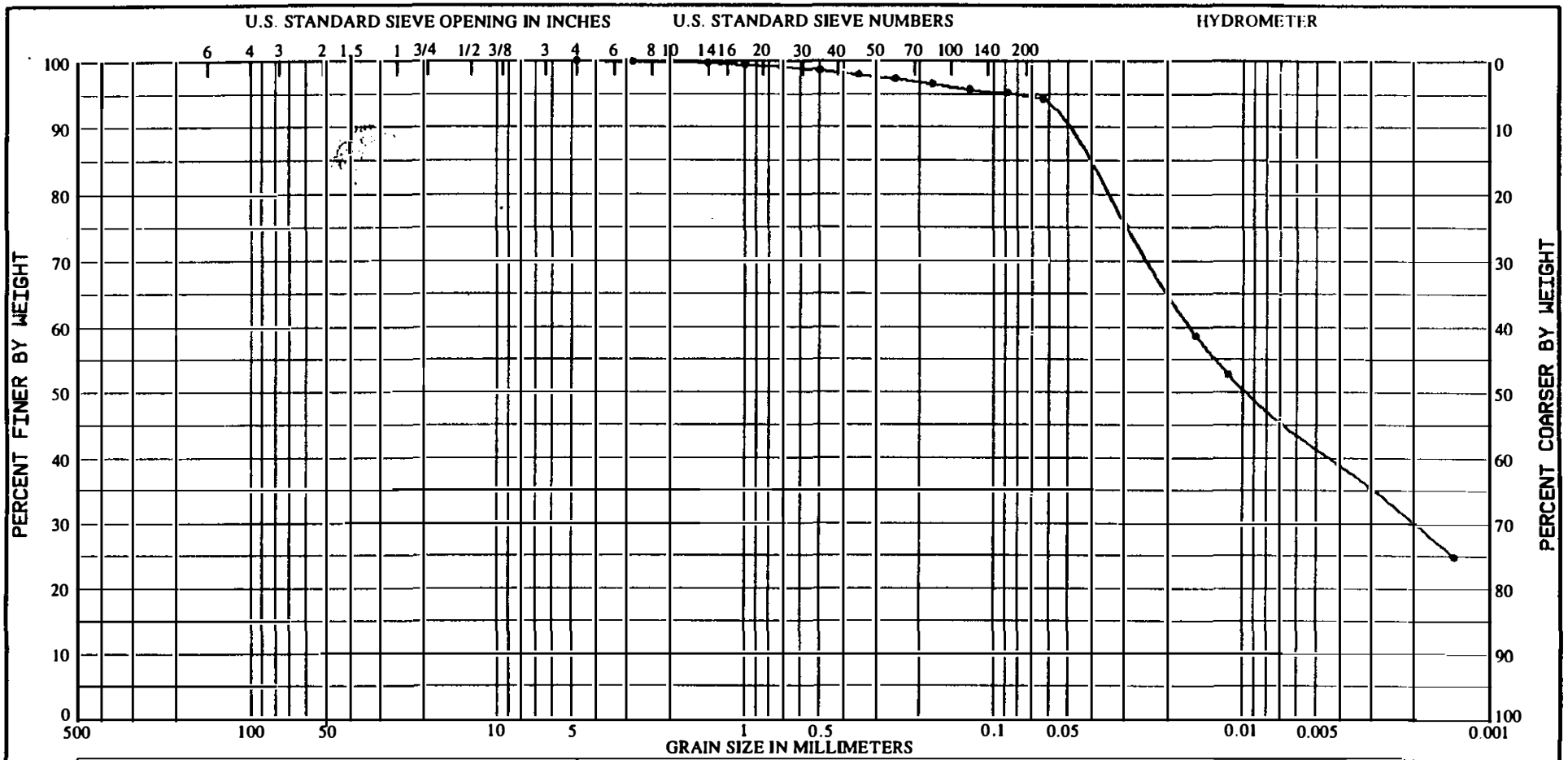
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



Sample No.	Depth	Classification	Nat w%	LL	PL	PI
E	15.1-15.4'	(VISUAL) GRAY, INORGANIC SILT HIGH LL (MH), WITH A TRACE OF SAND SIZES.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/932E
Boring No. KAV-25
Date 01/21/95

GRADATION CURVES

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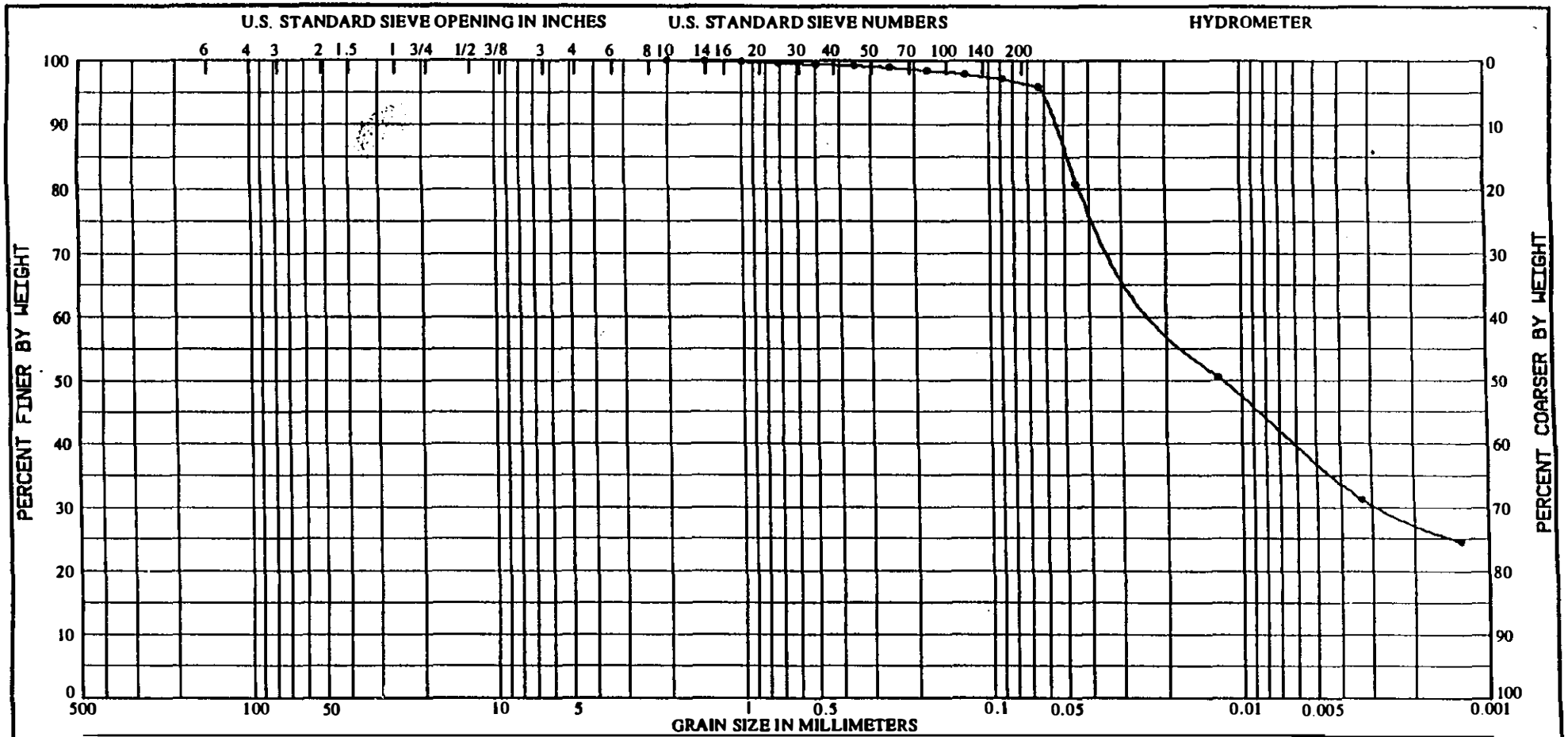
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Sample No.	Depth	Classification	Nat w%	LL	PL	PI
D-4	16.5-17.0'	(VISUAL) GRAY, INORGANIC SILT HIGH LL	66.2			
		(MH), WITH A TRACE OF SAND AND MICA.				
		SP. GRAVITY = 2.65				
		WET DENSITY = 92.6 PCF.				
		DRY DENSITY = 55.7 PCF.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/932
Boring No. KAV-25
Date 01/21/95

GRADATION CURVES

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DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE, MARIETTA, GA. 30060

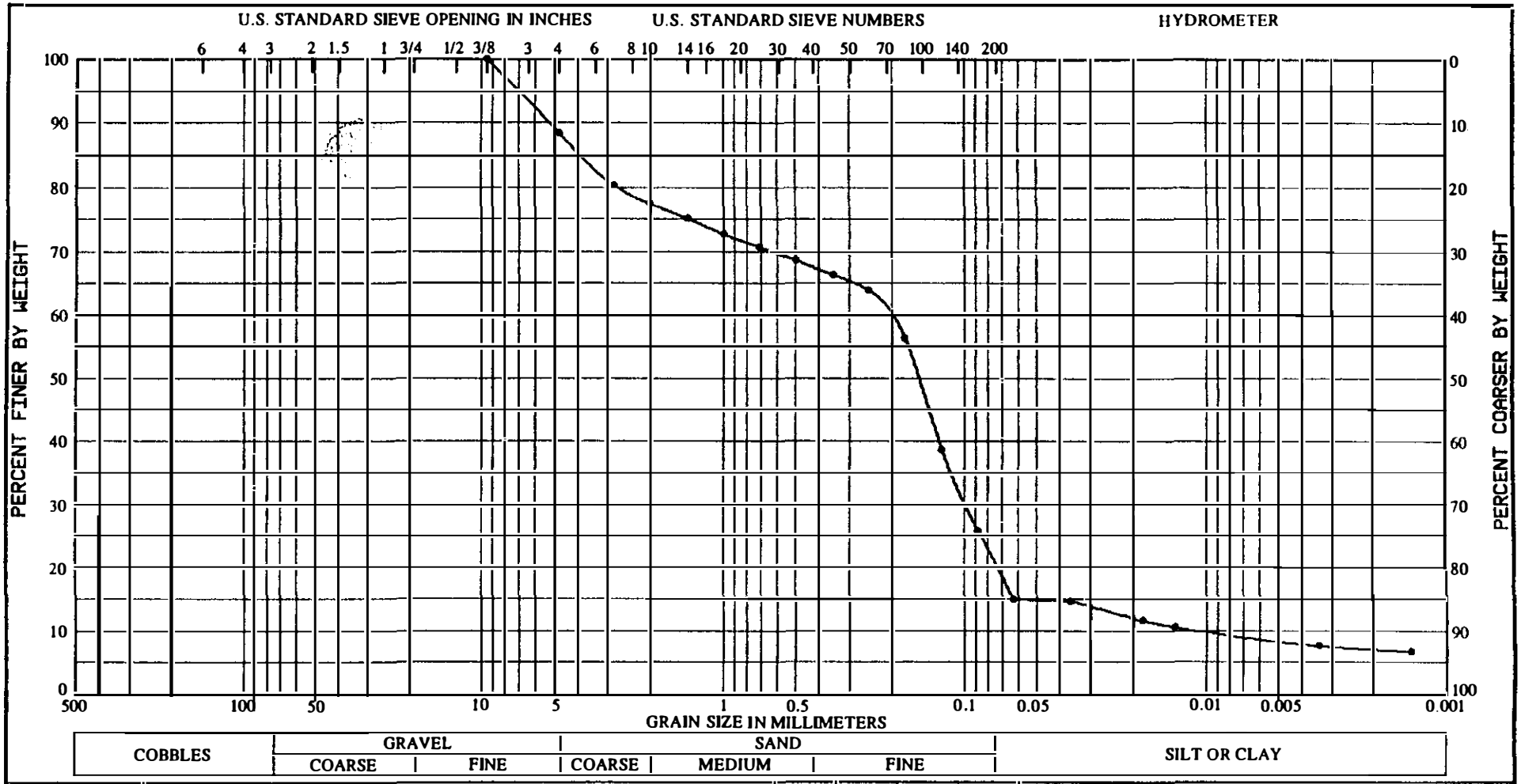
Project *DELAWARE BAY*		Boring No. *KAV-26*	
Location: *VIBRA CORE SAMPLES*		Lab No. *184/933*	
Boring Depth (ft): *15.30*	Elevation:	Work order: *7476*	
Datum/Notes: *See grain size data on enclosed gradation curves.*		Requisition: *CENAP-95-707*	

Elev (feet)	Depth (feet)	Leg-end	Material Description	Comments
			LT. BROWN, POORLY GRADED SILTY SAND (SP-SM).	(Density Units = pcf)
	1		GRAY, SILTY SAND (SM), WITH A TRACE OF GRAVEL SIZES.	MA (A) 1.1 - 1.3'
	2			
	3		GRAY, GRAVELLY POORLY GRADED SAND (SP).	SA D-1 3.0 - 3.5' WET DENSITY = 137.8, DRY DENSITY = 131.0, MC = 5.2
	4			
	5			
	6		GRAY, POORLY GRADED SILTY SAND (SP-SM).	SA (B) 6.1 - 6.3'
	7			SA D-2 7.2 - 7.7' WET DENSITY = 137.8, DRY DENSITY = 121.8, MC = 13.2 %.
	8		GRAY, POORLY GRADED SILTY SAND (SP-SM), WITH SOME GRAVEL SIZES.	
	9			SA (C) 9.0 - 9.5'
	10			
	11		GRAY, POORLY GRADED SAND (SP), WITH A TRACE OF GRAVEL SIZES.	SA (D) 11.5 - 11.8'
	12			
	13			SA D-3 12.5 - 13.0' WET DENSITY = 125.4, DRY DENSITY = 117.2, MC = 7.0 %.
	14		GRAY, POORLY GRADED SAND (SP), WITH A TRACE OF GRAVEL SIZES.	SA (E) 13.6 - 14.1'
	15			

Date: *01/21/95* **LABORATORY LOG AND SAMPLE DATUM** Sheet No. *1* of *1*

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



Sample No.	Depth	Classification	Nat w %	LL	PL	PI
A	1.1-1.3'	(VISUAL) GRAY, SILTY SAND (SM), WITH A LITTLE GRAVEL SIZES.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/933A
Boring No. KAV-26
Date 01/25/95

GRADATION CURVES

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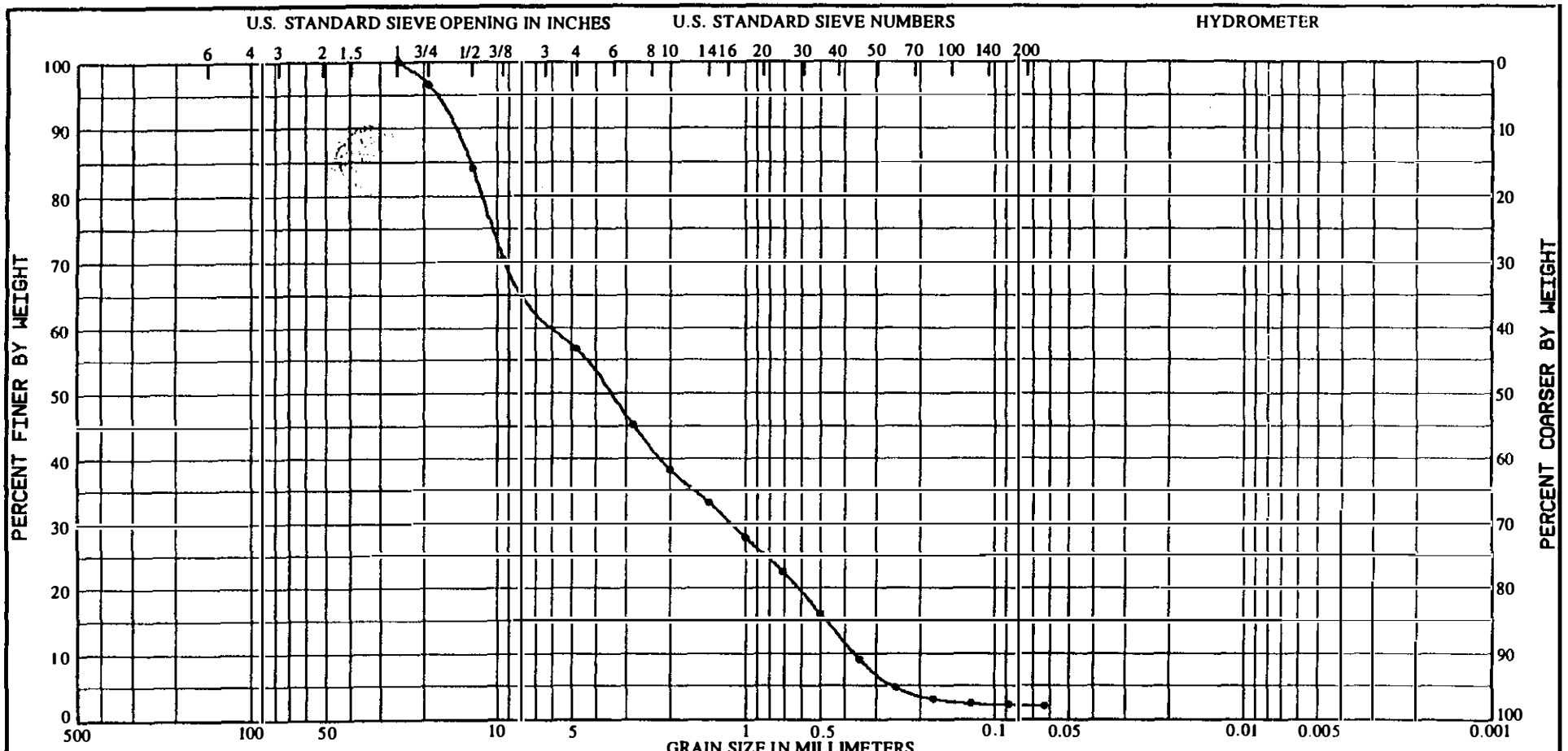
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Sample No.	Depth	Classification	Nat w%	LL	PL	PI
D-I	3.0-3.5'	(VISUAL) GRAY, GRAVELLY POORLY GRADED	5.2			
		SAND (SP).				
		SP. GRAVITY = 2.65				
		WET DENSITY = 137.8 PCF.				
		DRY DENSITY = 131.0 PCF.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/933
Boring No. KAV-26
Date 01/21/95

GRADATION CURVES

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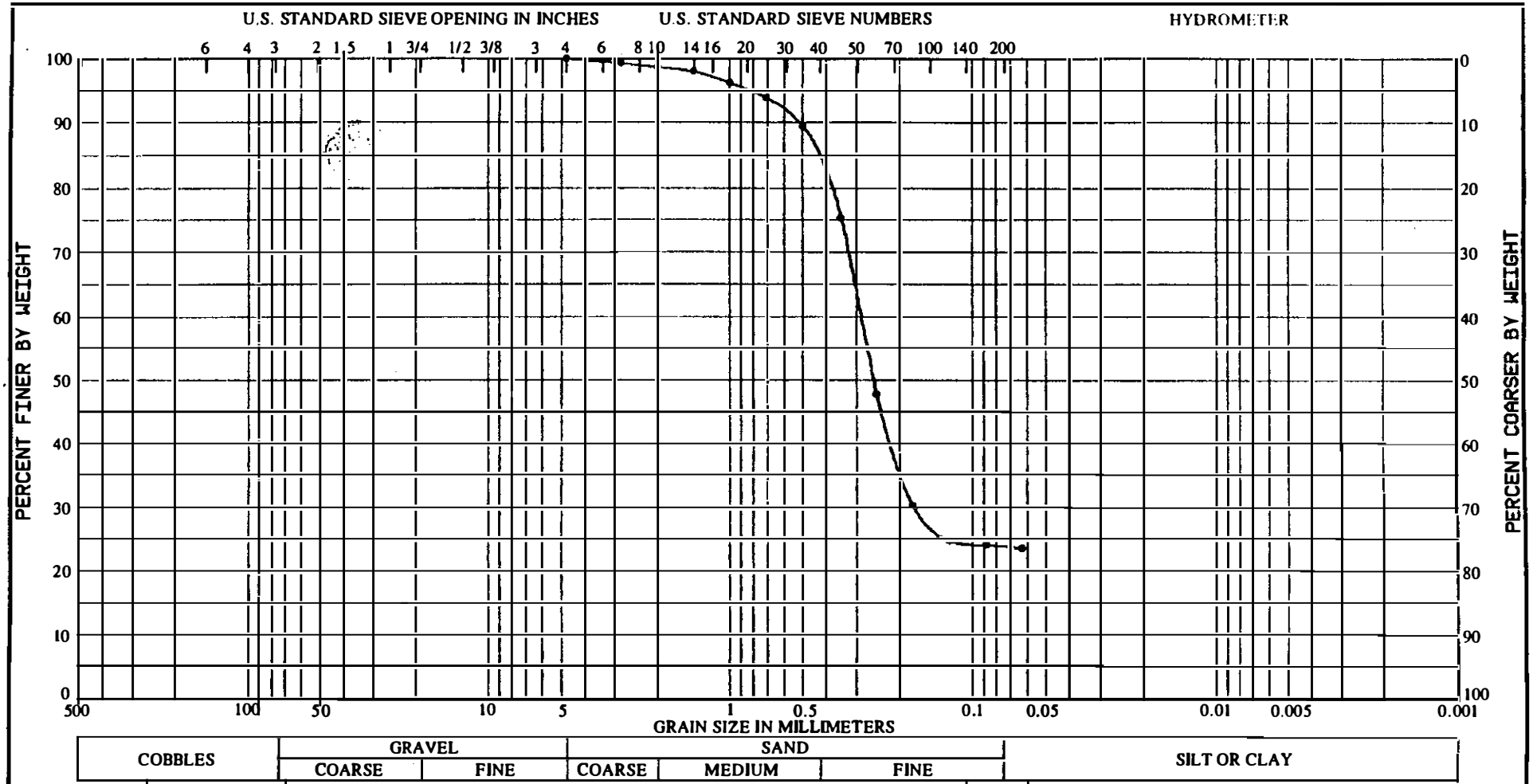
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



Sample No.	Depth	Classification	Nat w%	LL	PL	PI
B	6.1-6.3'	(VISUAL) GRAY, SILTY SAND (SM).				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/933B
Boring No. KAV-26
Date 01/21/95

GRADATION CURVES

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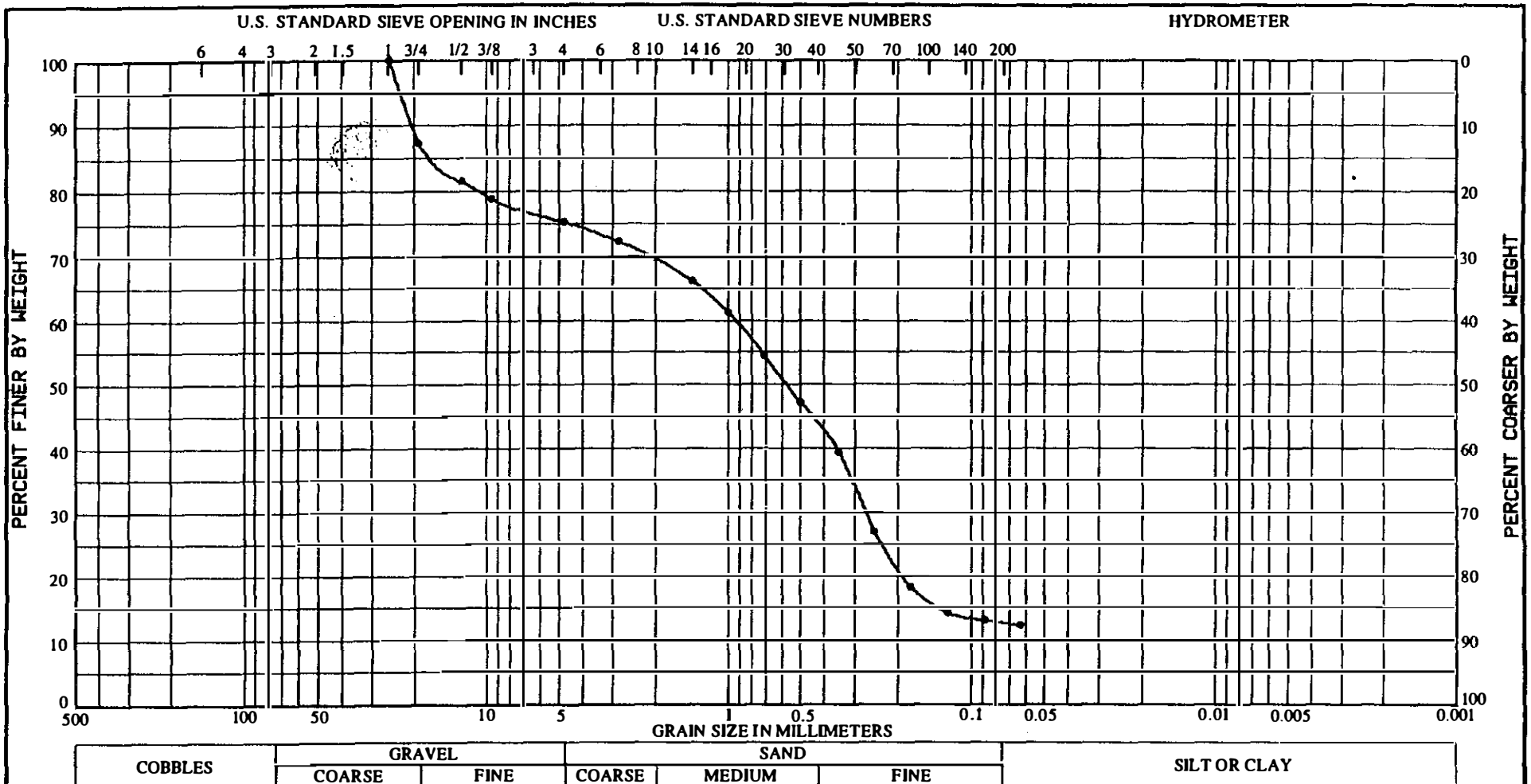
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



Sample No.	Depth	Classification	Nat w%	LL	PL	PI
D-2	7.2-7.7'	(VISUAL) GRAY, SILTY SAND (SM), WITH	13.2			
		SOME GRAVEL SIZES.				
		SP. GRAVITY = 2.66				
		WET DENSITY = 137.8 PCF.				
		DRY DENSITY = 121.8 PCF.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/933
Boring No. KAV-26
Date 01/25/95

GRADATION CURVES

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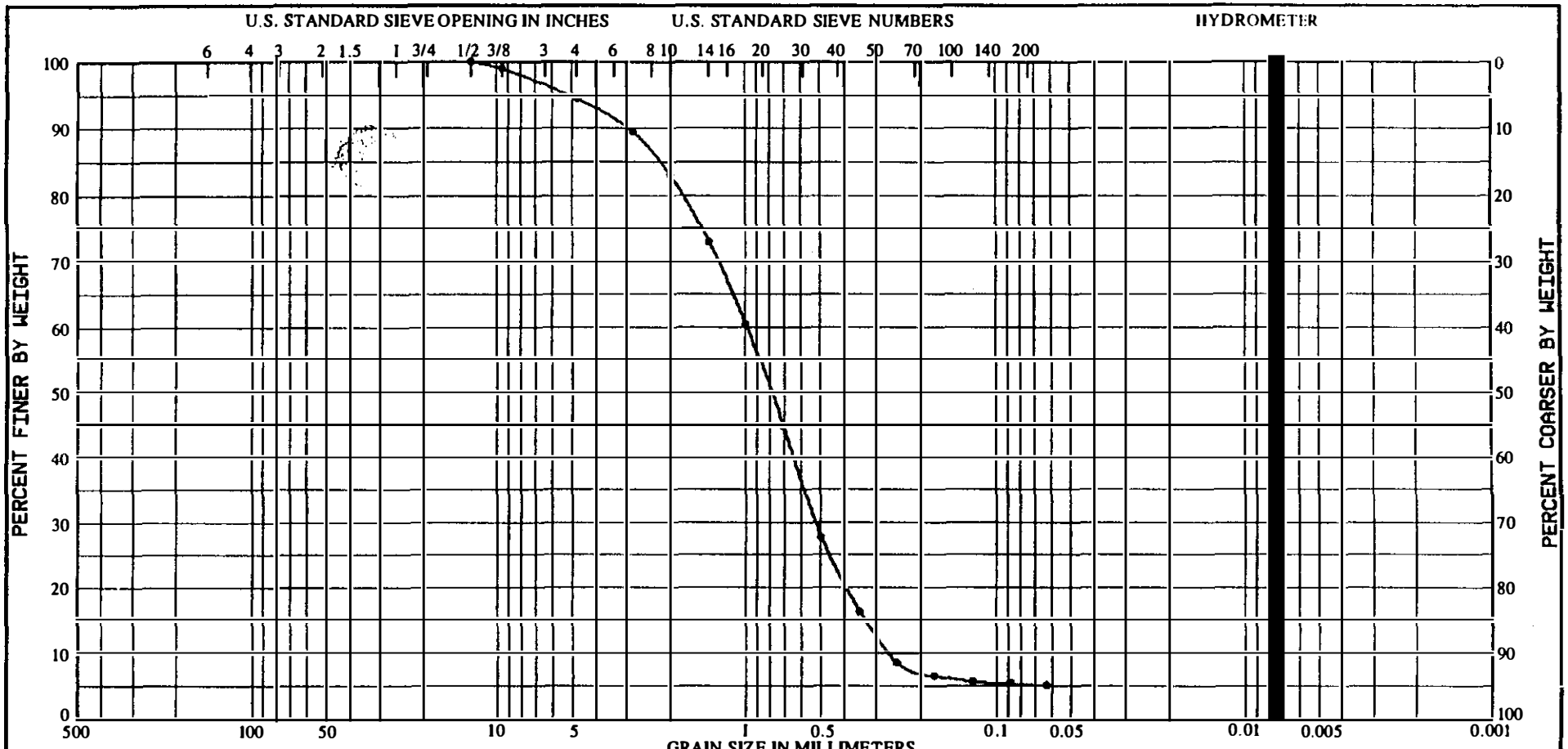
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



Sample No.	Depth	Classifica ion	Nat w%	LL	PL	PI
C	9.0-9.5'	(VISUAL) GRAY, POORLY GRADED SILTY SAND (SP-SM), WITH A TRACE OF GRAVEL SIZES.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/933C
Boring No. KAV-26
Date 01/21/95

GRADATION CURVES

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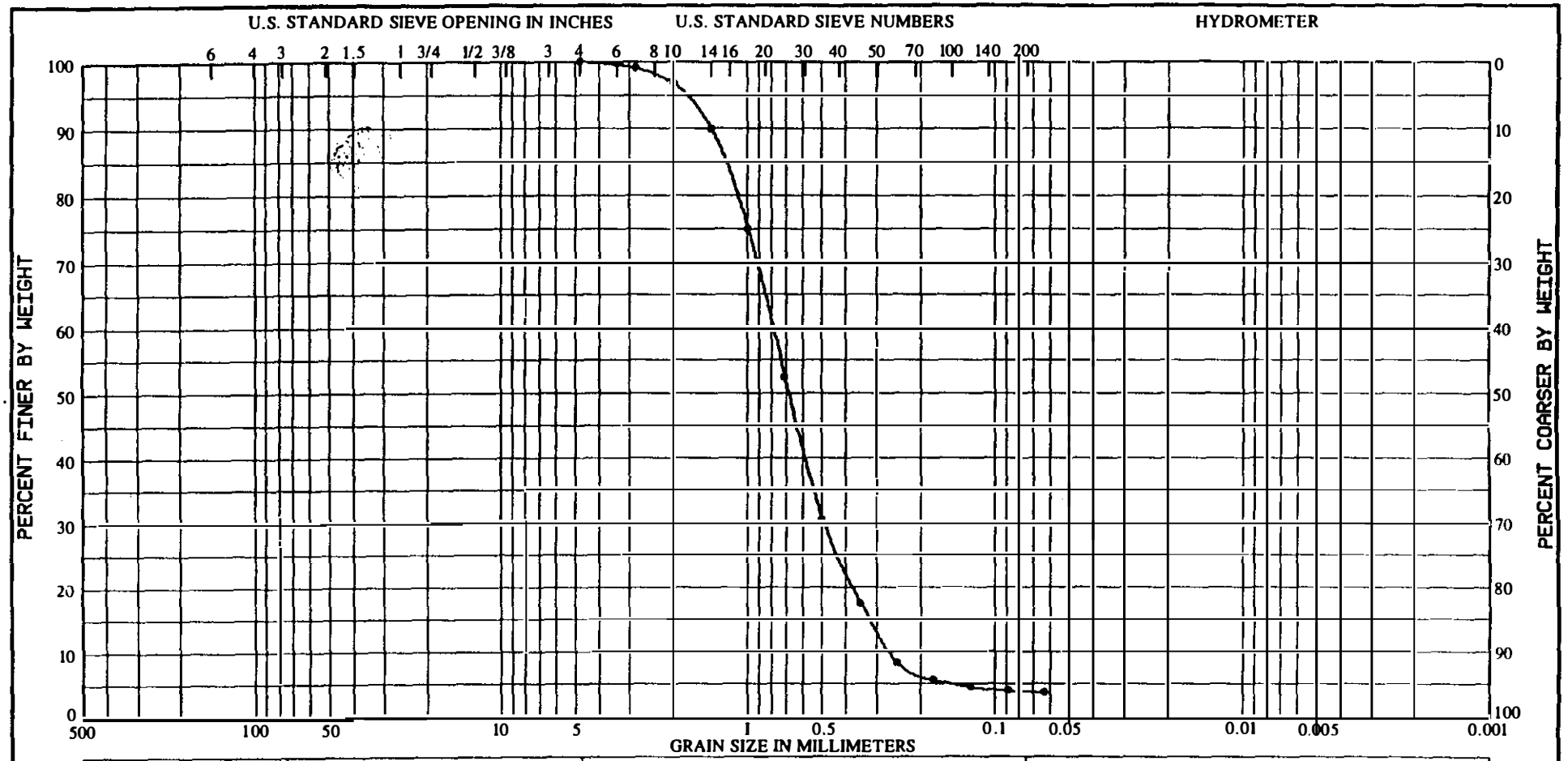
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Sample No.	Depth	Classification	Nat w%	LL	PL	PI
D	11.5-11.8'	(VISUAL) GRAY, POORLY GRADED SAND (SP).				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/9330
Boring No. KAV-26
Date 01/21/95

GRADATION CURVES

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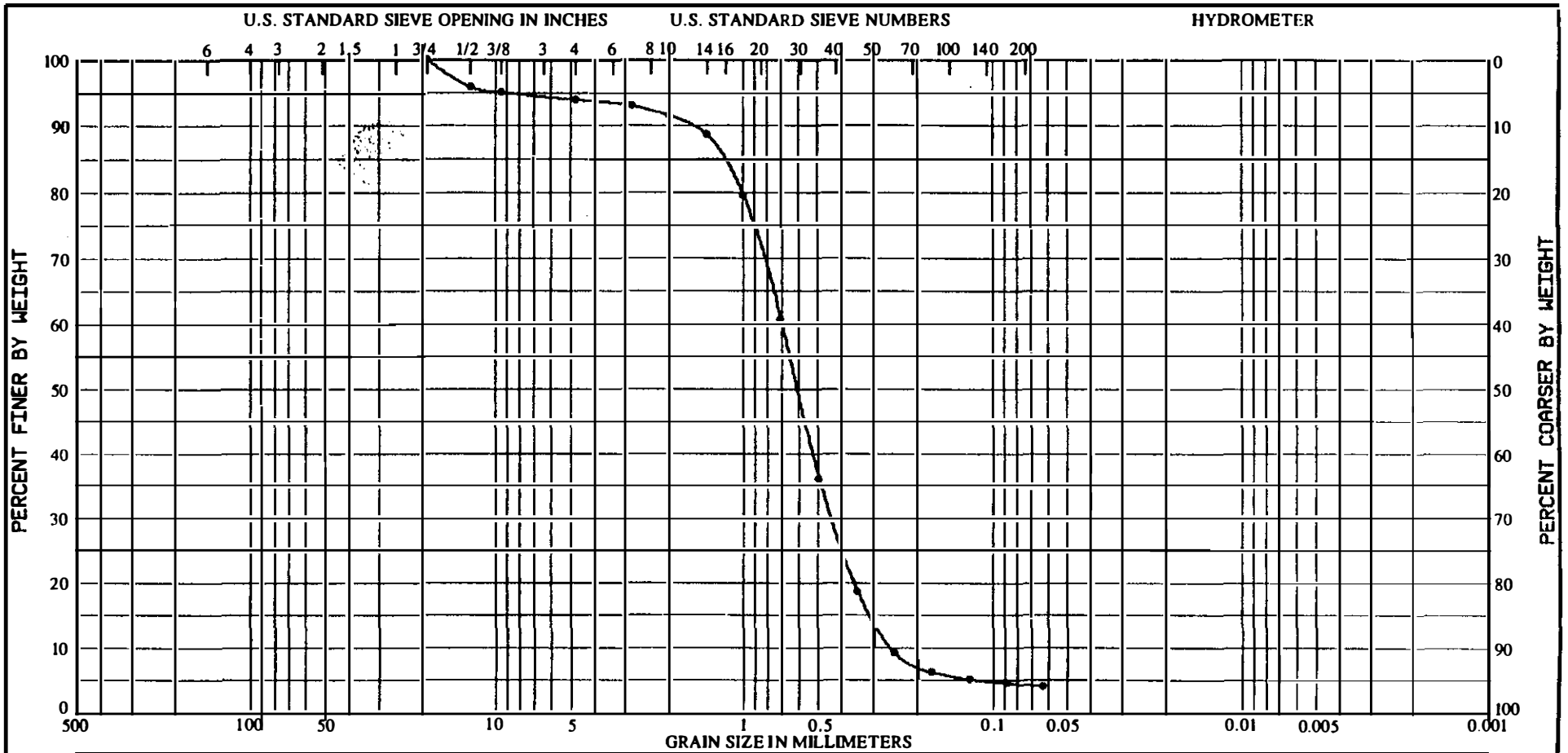
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



Sample No.	Depth	Classification	Nat w%	LL	PL	PI
D-3	12.5-13.0'	(VISUAL) GRAY, POORLY GRADED SAND (SP),	7.0			
		WITH A TRACE OF GRAVEL SIZES.				
		SP. GRAVITY = 2.63				
		WET DENSITY = 125.4 PCF.				
		DRY DENSITY = 117.2 PCF.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/933
Boring No. KAV-26
Date 01/21/95

GRADATION CURVES

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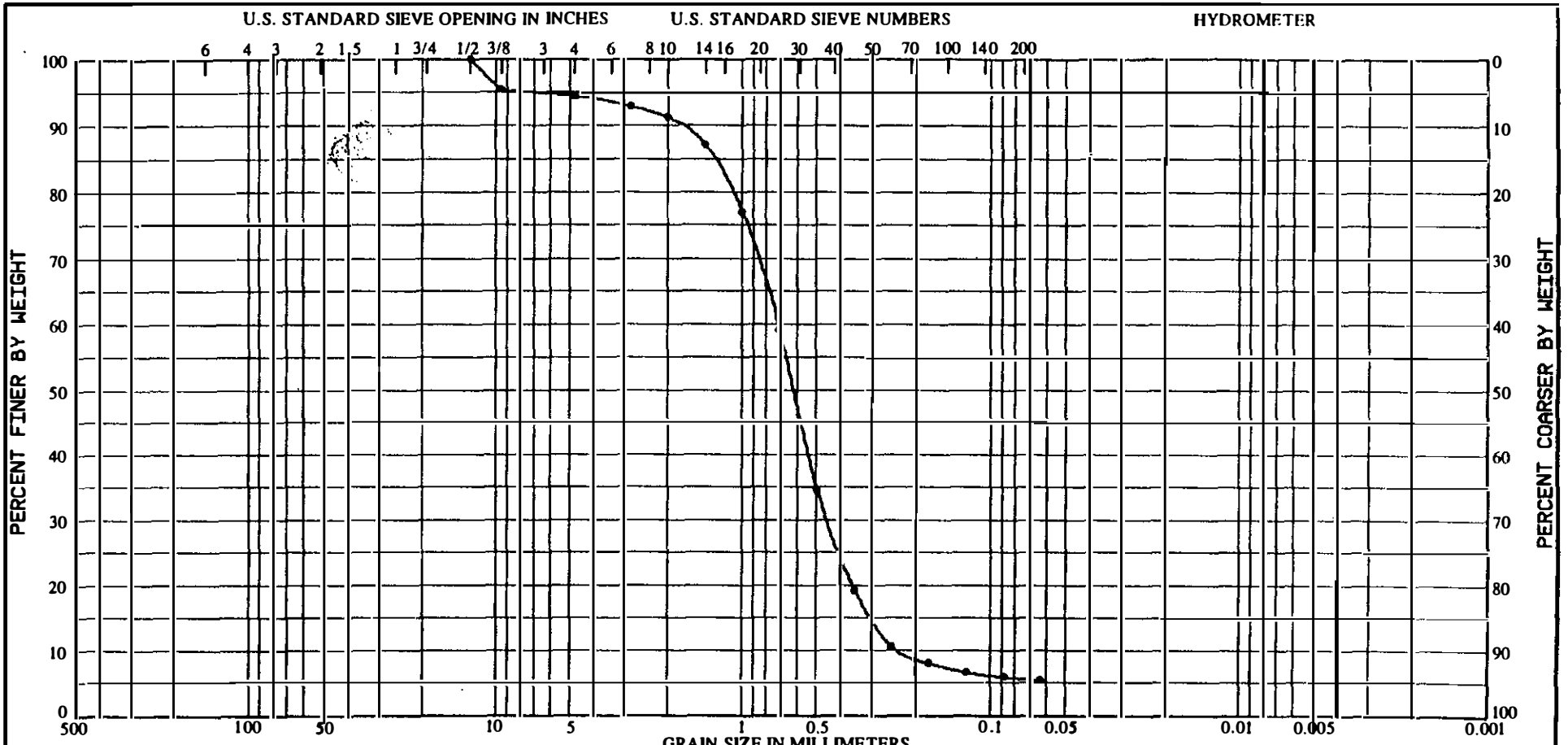
DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7476
REQUISITION: CENAP-95-707



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Sample No.	Depth	Classification	Nat w%	LL	PL	PI
E	13.6-14.1'	(VISUAL) GRAY, POORLY GRADED SILTY SAND (SP-SM), WITH A TRACE OF GRAVEL SIZES.				

Project DELAWARE BAY
VIBRA CORE SAMPLES
Lab No. 184/933E
Boring No. KAV-26
Date 01/21/95

GRADATION CURVES

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DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE, MARIETTA, GA. 30060

Project: *DELAWARE BAY*		Boring No. *KAV-29*	
Location: *VIBRA CORE SAMPLES*		Lab No. *184/936*	
Boring Depth (ft): *17.20*	Elevation:	Work order: *7476*	
Datum/Notes: *See grain size data on enclosed gradation curves.*		Requisition: *CENAP-95-707*	

Elev. (feet)	Depth (feet)	Leg-end	Material Description	Comments
			GRAY, SILTY SAND (SM), WITH A TRACE OF SHELL.	
	1		----------	
			GRAY, SILTY SAND (SM), WITH PLASTIC FINES AND A TRACE OF SHELL.	
	2		----------	MA D-1 2.0 - 2.5' WET DEN = 109.0, DRY DENSITY = 79.9, MC = 36.4 %.
	3		GRAY, SILTY SAND (SM), WITH A TRACCE OF SHELL.	
	4		----------	MA (A) 4.0 - 4.3'
			GRAY, CLAYEY SAND (SC), WITH A LITTLE GRAVEL SIZES.	
	5		----------	
	6			
	7			MA (B) 6.7 - 6.9'
	8			MA D-2 7.6 - 8.1' WET DEN = 99.6, DRY DEN = 61.0, MC = 63.5 %.
	9			
	10			
	11		GRAY, FAT CLAY (CH), WITH SAND LENSES AND LAMINA.	MA (C) 11.0 - 11.3'
	12			
	13			
	14			MA D-3 13.6 - 14.1' WET DEN = 101.3, DRY DENSITY = 65.3, MC = 55.1 %.
	15			
	16			
	17		----------	

Date: *02/08/95* **LABORATORY LOG AND SAMPLE DATUM** Sheet No. *1* of *1*

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