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Chesapeake Bay Shoreline Study: Headland Breakwaters and Pocket Beaches for Shoreline Erosion Control Final Report

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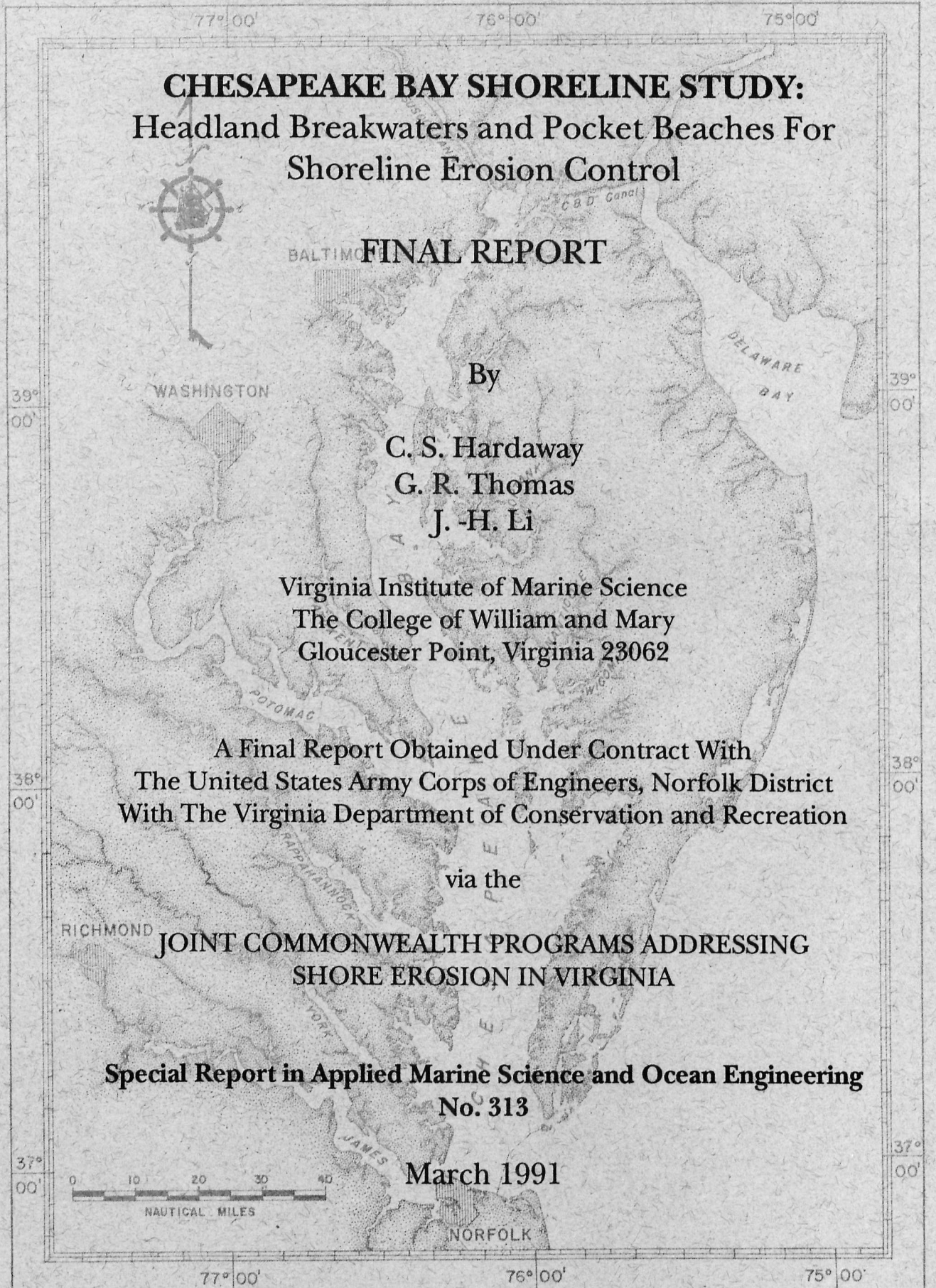
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CHESAPEAKE BAY SHORELINE STUDY:
Headland Breakwaters and Pocket Beaches For Shoreline Erosion Control

FINAL REPORT

By

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J. -H. Li

Virginia Institute of Marine Science
The College of William and Mary
Gloucester Point, Virginia 23062

A Final Report Obtained Under Contract With
The United States Army Corps of Engineers, Norfolk District
With The Virginia Department of Conservation and Recreation

via the

JOINT COMMONWEALTH PROGRAMS ADDRESSING
SHORE EROSION IN VIRGINIA

Special Report in Applied Marine Science and Ocean Engineering
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CHESAPEAKE BAY SHORELINE STUDY - FINAL REPORT

Executive Summary
March 1991

The Chesapeake Bay Shoreline Study is a cooperative project of the Commonwealth of Virginia and the Norfolk District of the U.S. Army Corps of Engineers. The project consists of three modeling and five monitoring sites located on the tributary estuarine shores of the Virginia portion of Chesapeake Bay. The purpose of the study is to examine headland breakwaters and the headland - pocket beach concept for the abatement of estuarine shoreline erosion. These structures may represent a lower cost approach to shoreline erosion control as well as provide an "environmental edge" or buffer between what we perceive as land and marine resources.

The world-wide use of segmented or gapped breakwaters, both attached and detached, has spurred interest within the Commonwealth of Virginia. The applications of both concepts may represent an _effective_ low cost approach to the abatement of shore erosion along hundreds of miles of estuarine shoreline. Long stretches of agricultural, wooded, and unmanaged shorelines are appropriate areas for such applications.

Eight sites were selected for analysis in this study. Three sites involved the construction of offshore breakwaters which are designated modeling sites. Five sites were selected for monitoring, two of which have previously installed breakwater systems and two which exhibited crenulate-bay morphology. These sites are representative of 215 miles of estuarine shoreline in Virginia.

Analysis of the sites involved quarterly shore profiles and low level aerial photography, as well as selected sediment sampling and analysis. A shore response computer model was created to evaluate the effect of breakwaters on pocket beach planforms.

The definitive protective beach/breakwater system must be designed to withstand given storm conditions and the consequent surge. The main objective is the design of breakwaters made high enough and placed far enough offshore to permit sufficient input of fill material to provide a stable protective beach and backshore.

The installation of widely spaced breakwaters to create a headland/bay situation must be done after a proper site analysis along appropriate reaches. The geomorphic expression of a shoreline, especially the fastland configuration, shows the long term response to the impinging seasonal wave climate. The forcing by waves onto the various "natural" and anthropogenically altered shorelines must be evaluated in terms of what level of storm surge protection a given site will require to meet shoreline management goals.

I. Introduction

A. Project Scope and Purpose

The Chesapeake Bay Shoreline Study is a cooperative project of the Commonwealth of Virginia, the Norfolk District of the U.S. Army Corps of Engineers, and the Virginia Institute of Marine Science. The project consists of three modeling and five monitoring sites located on the tributary estuarine shores of the Virginia portion of Chesapeake Bay. The purpose of the study is to examine more closely gapped-offshore-headland breakwaters and the headland concept for the abatement of estuarine shoreline erosion. Headland breakwaters provide fixed points along a shore between which a series of stable pocket beaches can develop. These structures may represent a lower cost approach to control shoreline erosion as well as provide an "environmental edge" between what we perceive as land and marine resources.

The world-wide use of segmented or gapped breakwaters, both attached and detached, has spurred interest within the Commonwealth of Virginia. The applications of this concept may represent an _effective_ low cost approach to the abatement of shore erosion along hundreds of miles of estuarine shoreline. Long stretches of agricultural, wooded, and unmanaged shorelines may be appropriate areas for such applications. In general, the cost per linear foot of shore decreases as the spacing between breakwaters increases.

B. Previous Research Summary

Previous research on detached and headland (attached) breakwaters and their effects on shore morphology has been extensive (Lesnik, 1979). Much of this research has been conducted on ocean shores. Headland breakwater models were developed from physical scale models and observations of naturally occurring headlands with their adjacent crenulate, log-spiral or hook-shaped bay beaches as reported by Yasso (1965), Silvester (1970, 1974, 1976,), Silvester and Ho (1972), LeBlond (1972), Rea and Komar (1975), Finkelstein (1982), the U.S. Army Corps of Engineers (1984),

Berenguer and Fernandez (1988), Quevauviller (1988) and Hsu et al. (1989a, 1989b). The generally stable planform, geometry or morphology of the log-spiral bay-beaches is a function of the prevailing direction of wave incidence combined with refraction and diffraction. Everts (1983) emphasized that for an equilibrium bay to form, there must be a fixed downdrift boundary.

Detached breakwaters have been examined by Toyoshima (1974), Shinohara and Tsubaki (1966), Perlin (1979) and the U.S. Army Corps of Engineers (1984). According to the Corps of Engineers, the formation of tombolos usually can be prevented if the structure length is less than the distance offshore. (A tombolo is a sandbar or spit that connects or ties a breakwater or island to the mainland or another breakwater or island.) If a detached breakwater system becomes fully attached by a consequent tombolo, the breakwater units should function more as headland breakwaters because longshore drift is considered essentially stopped. Unattached tombolos (cuspate spits) allow for more continuous longshore transport with less deleterious downdrift effects (U.S. Army Corps of Engineers, 1984).

c. Study Site Summary

In 1987, seven sites that represent different fetch exposures and shore orientations were selected for analysis. In 1988, the second year of the study, another site adjacent to an existing monitoring site was added.

Analyses of these sites involves quarterly shore profiles and low level aerial photography as well as sediment sampling and grain-size analysis. Procedures developed by Sverdrup and Munk (1947), and Bretscheider (1966) were performed to estimate the wave climate at each site. Also a shoreline response computer model was developed for this project. The purpose of this report is to provide results on the field monitoring and laboratory analysis of the Chesapeake Bay Shoreline Study from July 1987 to June 1990. Also, general design guidelines were

developed using empirical data to compare the study sites to other breakwater installations in Virginia and Maryland.

II. Previous Research

A. Gapped Breakwaters

Shinohara and Tsubaki (1966) performed physical model tests propogating shore normal waves onto single breakwaters. They concluded that the main cause of shore change and sand movement on a beach is the diffraction of the incoming wave around the breakwater. The diffraction in turn depends on the ratio of offshore position of the breakwater to its length. The amount of sand deposition per unit area in the sheltered region behind the breakwater rapidly decreases with the increase of distance offshore.

In 1969 Toyoshima did a statistical study on 217 breakwaters in 86 locations worldwide (Toyoshima, 1974). These included single and multiple breakwaters. He stated that for gapped breakwaters, no clear factor for sand deposition and tombolo formation could be found. He did not indicate whether any of the installations he studied involved beach fill. Sites with an identical ratio of breakwater length to distance offshore sometimes exhibited tombolos and sometimes did not.

Perlin (1979) used a numerical model after the physical model of Shinohara and Tsubaki, 1966 (Figure 1). Qualitatively, the two models agree. For his model, Perlin normalized distances using linear deepwater wave length $L_o = \frac{gT^2}{2\pi}$, where g is acceleration due to gravity and T is wave period. According to Perlin, the following list of variables completely describe the problem of shore response to a single breakwater:

1) relative breakwater length,

2) relative distance of breakwater from shore,

3) relative depth of profile closure,

4) wave steepness,

5) wave angle,
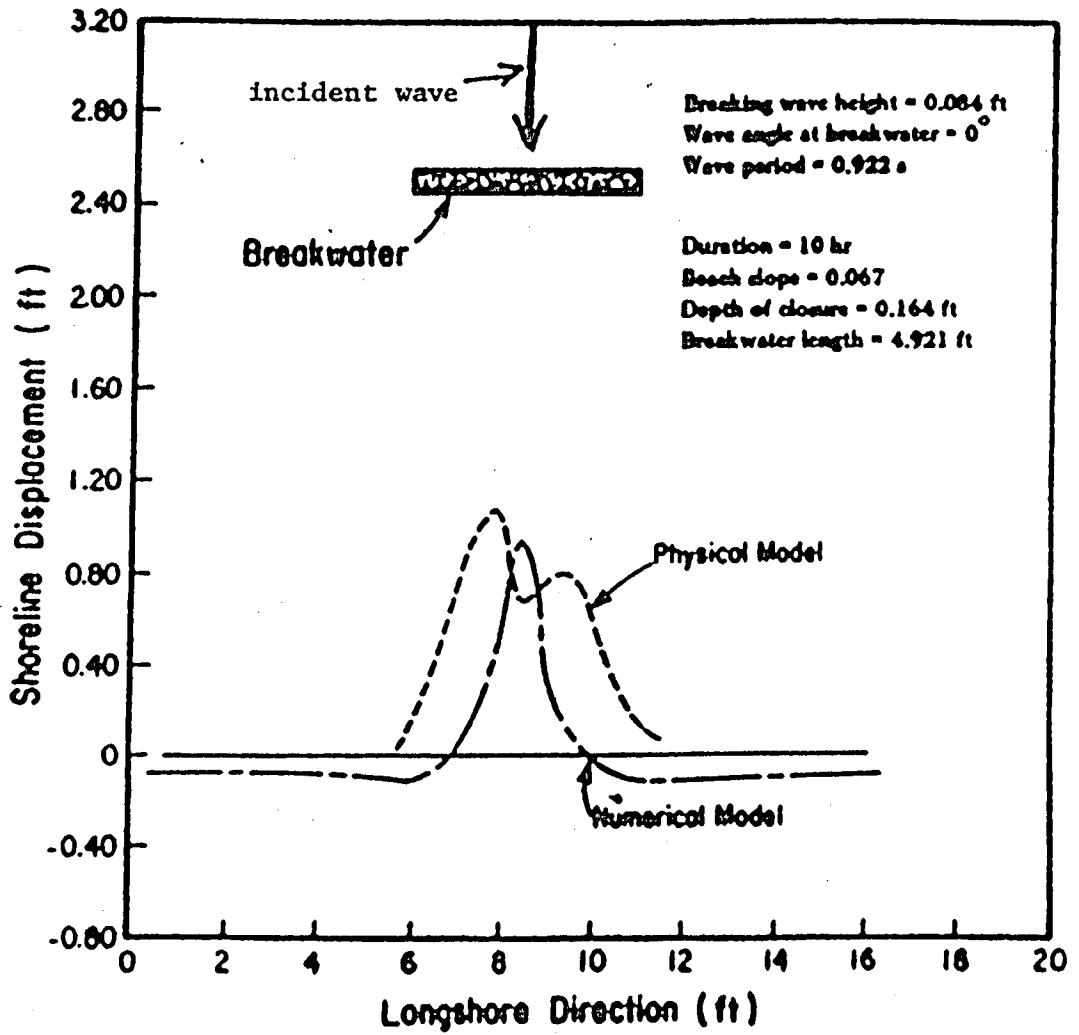
Figure 1. Simulation of the physical breakwater model by Shinohara and Tsubaki (after Perlin, 1979).

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6) beach slope, and

7) wave period.

In all the analyses, as the tombolo forms, adjacent shorelines erode to provide the sediment. However, as the tombolo approaches an equilibrium planform, the amount of sand it requires is reduced, and the adjacent shorelines begin to fill because the shoreline is not aligned with the wave angle (Perlin, 1979).

Perlin's analysis demonstrated some intuitively obvious ideas. As the structure is moved further offshore, it has less effect on the shoreline. Also, as wave steepness increases, the shoreline responds more quickly. It was also shown by this numerical model that the initial double tombolo can be a permanent feature or it can evolve into a single tombolo.

From field and model observations, Gourlay (1974) recognized the existence of wave generated currents in the lee of breakwaters and headlands. The basic mechanism producing the current was shown to be an alongshore gradient of wave set-up within the surf zone. For a given shore geometry, the alongshore current velocity primarily is determined by the deepwater wave height (Gourlay, 1976).

Rosen and Vajda (1982) concluded that a morphological and sedimentologic equilibrium is reached when the shape of the nearshore bottom and beach contour lines is such that along the sheltered beach the diffracted waves have a component of momentum flux opposed to the gradient of the mean sea level induced by radiation stress due to non-uniform wave heights along the wave fronts. This varies from what Silvester (1974) explained. According to Silvester, a state of morphologic shore equilibrium is reached when the bottom contour lines become parallel to the diffracted wave fronts. Silvester's model thus ignores the change in breaker height alongshore and only considers the Sxy component of radiation stress (which vanishes when wave crests parallel bottom contours).

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Dally and Pope (1986) recognized the natural parameters most important to the design of a detached breakwater system to be those that affect wave diffraction (wave length, height, direction, and the gap width-to-wave length ratio for segmented breakwaters), natural beach slope, water-level range, native sediment-size, and available supply of sediment. They analyzed numerous in-place breakwater systems and physical model tests and found tombolo formation for single and segmented detached breakwaters generally is assured when the ratio of breakwater length (ℓ) to distance offshore (x) approaches 1.0. Conversely, to prevent tombolo formation (i.e. only spit or salient formation), breakwater length should be equal to or less than one-half the distance offshore, $\ell \leq \frac{1}{2}$ x. Tombolo formation may also be reduced by allowing waves to overtop the breakwater(s) and/or increasing breakwater permeability (Dally and Pope, 1986).

Suh and Dalrymple (1987) performed small scale model tests in a spiral wave basin for single and multiple offshore breakwaters to examine the effects of geometric parameters on the morphological change in the shore. They compared the model tests with studies reported by others and with offshore breakwaters in the field. All horizontal lengths were non-dimensionalized with respect to the offshore distance of the breakwaters from the original shoreline, X_B. Three dimensionless variables (denoted *), x_b^* (= x_b/X_B), L_B^* (= L_B/X_B), and $G_B^* = (G_B/X_B)$, were found to be important to shore morphology, in which x_b, L_B, and G_B are the surf zone width, the breakwater length, and the gap spacing between adjacent breakwaters, respectively. They concluded that for multiple offshore breakwaters, tombolos form when G_B^*/L_B^{*2} is about 0.5.

Silvester (1974) considered at least two fixed breakwaters or headlands in his definition of equilibrium shore for the embayed pocket beach. From numerous investigations of natural crenulate or log-spiral bays and physical scale models, Silvester (1974) developed a model to determine maximum bay indentation given the incident wave angle, starting

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from the center line between two headland breakwaters. Suh and Dalrymple (1987) demonstrated that when the gap between two diffraction points (i.e. the ends of adjacent breakwaters) becomes approximately twice the incident wave length or more, the shoreline behind each breakwater responds independently as if there were no interaction among the breakwaters. According to the U.S. Army Corps of Engineers (1984), for normal wave incidence, the diffraction effects of gapped breakwater-ends act independently when the breakwater gap is greater than five wavelengths.

Recently the U.S. Army Corps of Engineers developed a shoreline change model called GENESIS: Generalized Model for Simulating Shoreline Change (Hanson and Kraus, 1989). This model is capable of predicting shoreline change behind detached breakwaters, wave transmission through detached breakwaters and diffraction at detached breakwaters, jetties and groins among other options. At this time, however, GENESIS cannot predict shoreline change involving tombolos development behind offshore breakwaters or headland breakwaters.

B. Headlands and Pocket Beaches

Oblique incident waves approaching widely spaced breakwaters may cause an effect in the adjacent embayment. Natural headlands and their embayments (i.e. pocket beaches) have been studied by Yasso (1965), Silvester (1974), and others. The planform of the headland-bay beaches is dependent on the predominant direction of wave attack (Yasso, 1965; Silvester, 1974). Headland-bay beaches often are referred to as crenulate, pocket or log-spiral bay beaches.

Because of the decreasing radius of plan curvature that characteristically occurs toward the headland and because the rate of decrease in radius curvature appears to be non-linear, Yasso (1965) tested the equiangular (logarithmic) spiral,

$$\frac{R_2}{R_1} = e^{\theta \cot \alpha}$$

for goodness of fit to the plan shape of headland-bay beaches. In the equation above, $\frac{R_2}{R_1}$ is the ratio of 2 radius vectors from a log-spiral

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center; α is the angle between a radius vector and tangent to the wave at that point and is a constant for a given log-spiral; θ = the angle between radius vectors; and the constant e is the base of Naperian logarithms. A diagram of log-spiral nomenclature is shown in Figure 2.

Silvester (1976) recognized the difficulty in defining the equilibrium beach to the log-spiral formula. Extensive research on crenulate bays resulted in relating the equilibrium beach planform to maximum bay indentation and incident wave angle (Figure 3). Silvester divided the bay into the updrift shadow reach or logarithmic spiral and the tangential reach. The logarithmic spiral reach is affected most by wave diffraction. The tangential reach, which is slightly convex seaward or straight, is affected mostly by wave refraction.

Rea and Komar (1975), in studying log-spiral bays through numerical modeling, indicated that the shoreline will always attempt to achieve an equilibrium configuration which is governed by the patterns of offshore wave refraction and diffraction and by the distribution of wave energy flux. If the system is closed, then a true equilibrium is achieved wherein the shoreline everywhere takes on the shape of the wave crests (i.e. breaker angles are everywhere zero). If the system is not closed and sediment continues to be transported to the downdrift end of the model and further, then equilibrium occurs where the breaker angles are precisely those required to transport the sediment eroded from the updrift section of beach. Under this definition of equilibrium the shoreline continues to erode but retains its overall shape (Rea and Komar, 1975).

Everts (1983) recognized the difficulty in using a logarithmic spiral shape, i.e. the trial and error establishment of the center location of the spiral. He noted for an equilibrium, crenulate-shaped bay to form, there must be a fixed downdrift boundary. Without one, the rate of sediment loss will not decrease progressively with time after headland or breakwater construction. Only with a fixed boundary will the alongshore length of the bay be controlled and the total volume loss be fixed.

$$\frac{R_2}{R_1} = e^{\theta \cot \alpha}$$
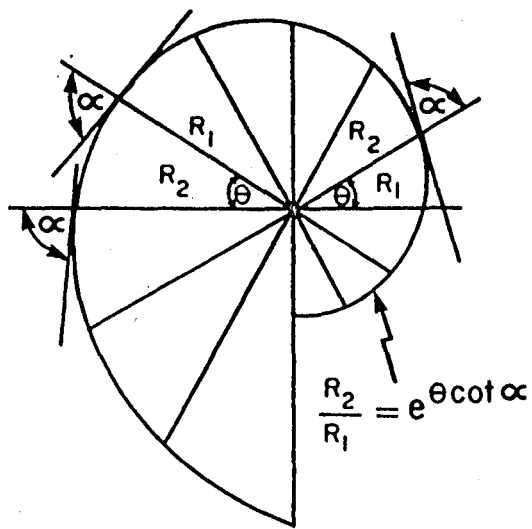
Figure 2. Definition sketch of logarithmic
spiral (after Yasso, 1965).
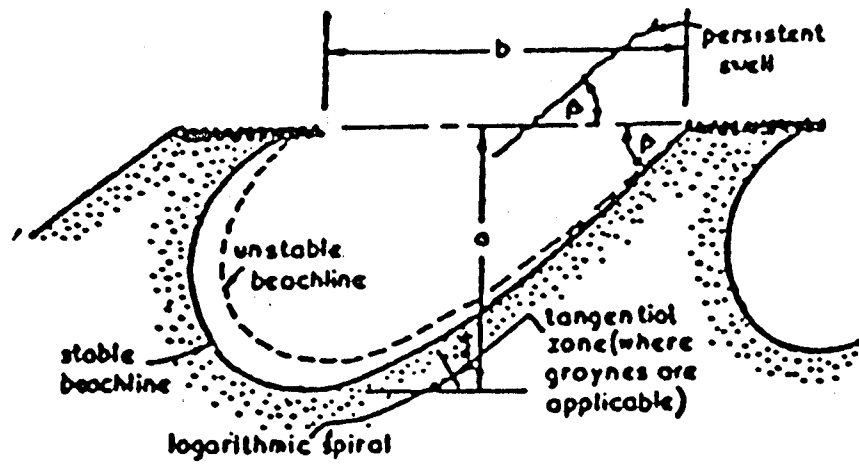
Figure 3.   Crenulate shaped bay in stable and unstable
            conditions (after Silvester, 1976).

However, the downdrift boundary does not have to be a littoral barrier. It must, though, provide a fixed limit for the bay such that the angle between the equilibrium-tangent-sector-alignment and the pre-construction shoreline becomes constant at the downdrift boundary until equilibrium conditions are reached (Everts, 1983).

Berenguer and Enriquez (1988) found that the log-spiral equilibrium formula is applicable to over 30 pocket beach locations along the Mediterranean coast of Spain. Spanish pocket beaches closely fit the following equation:

$$S = 25 + 0.85 \ A$$

where S is the gap between headland breakwaters and A is the depth of the pocket beach.

Hsu et al. (1989a) determined that defining bay curvature through the log-spiral method was not precise and should be replaced by some new relationships. These new relationships revolve around what they call a static equilibrium bay (Figure 4). The line joining the point of diffraction to the downcoast limit of the bay (R_o) is termed the "control line" and its angle to the incident wave crests is the obliquity of the waves (θ), which is the only input variable that determines the bay shape. This angle is the same as that between R_o and the downcoast tangent to the beach when the bay is in static equilibrium. From the definition sketch in Figure 4, it is seen that the variables (R and θ) involved in this new presentation are an arc of length R angled θ to the wave crest line, which is assumed parallel to the tangent at the downcoast limit of the beach (Hsu et al., 1989b). Hsu et al. (1989a) admitted that the log-spiral formulation of Silvester (1974) is still useful as a secondary check.
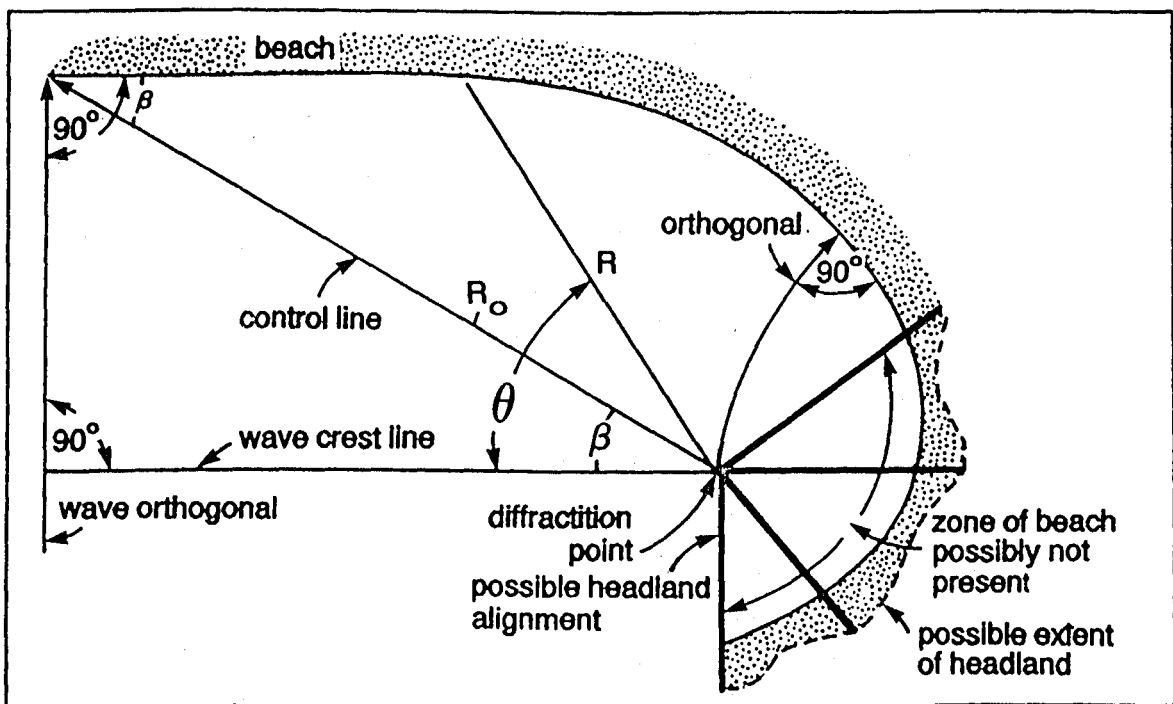
Figure 4.   Parameters of the static equilibrium bay (after Hsu et al., 1989).

III. Data Collection and Site Analysis

    A. Field Methods

    The eight project sites for the Chesapeake Bay Shoreline Study are:

Modeling Sites

Breakwaters

1. Chippokes State Park, James River, Surry County (CHP)

2. Hog Island Breakwaters, James River, Surry County (HI2)

Headland

1. Hog Island Headlands, James River, Surry County (HIH)

Monitoring Sites

Breakwaters

1. Drummonds Field, James River, James City County (DMF)

2. Parkway Breakwaters, York River, York County (NPS)

3. Waltrip, James River, James City County (WAL)

Headlands

1. Summerille, Potomac River, Northumberland County (SUM)

2. Yorktown Bays, York River, York County (YB)

Figure 5 shows the locations of the sites.

    Shore-parallel baselines were established for each site with profile distances and elevations determined using stadia and level. The position and spacing of the profiles were site specific. Additional profiles were established at the breakwater sites in order to measure more accurately the changes in shore position. The long curvilinear shores at two sites, Summerille and the Yorktown Bays, had less closely spaced profiles. Tidal datums were established using the nearest local tide stations (NOAA). Table 1 is the schedule of profiling and aerial photography.

    Initially, aerial photography was done during each phase of the project at 500, 1,000 and 2,000 feet. Later, in March 1989, this was changed to 750 and 1,500 feet so that an even scale of 1 inch = 100 feet and 1 inch = 200 feet respectively could be used directly on each photo print. The photographs were used along with the profile data to create a
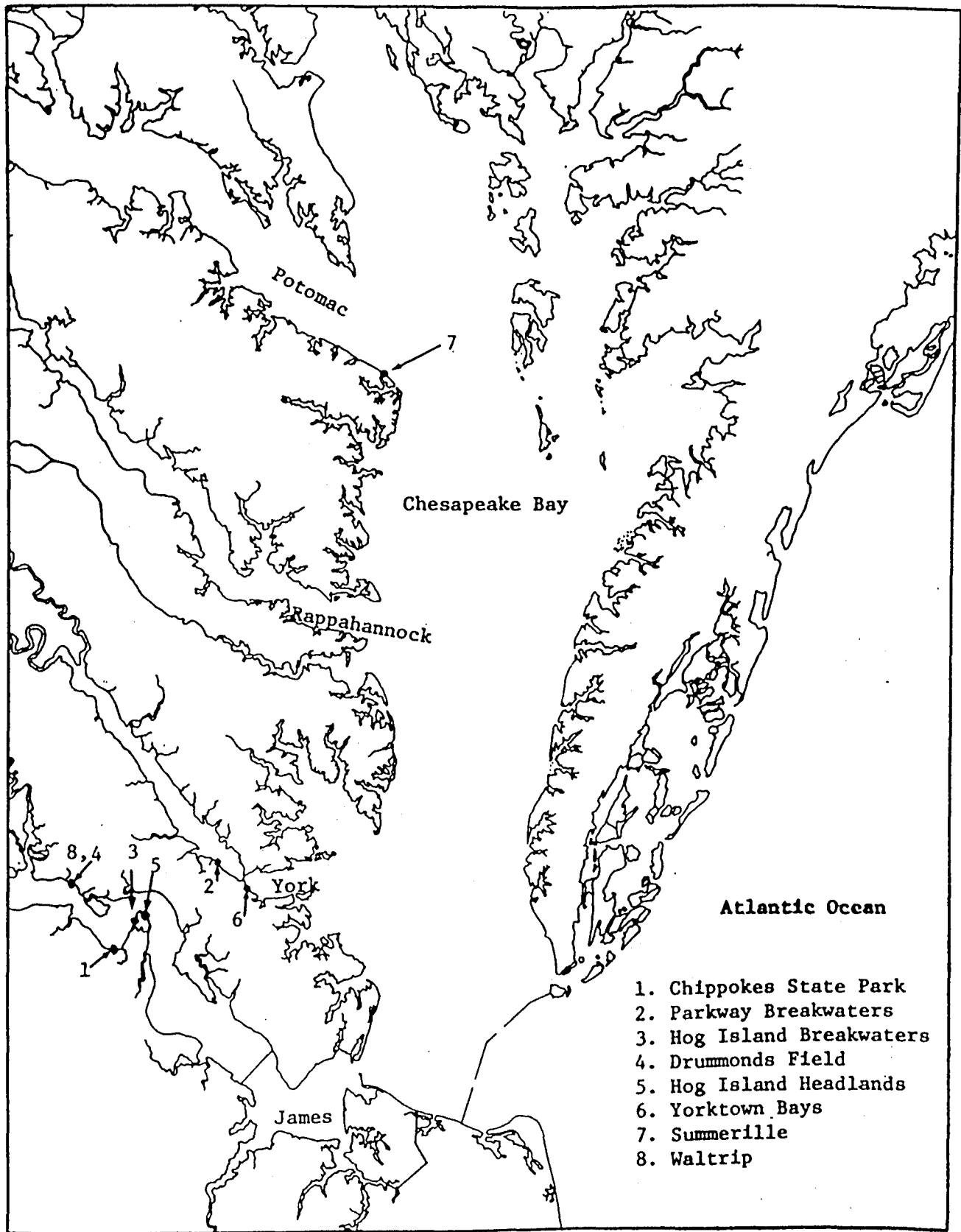
Figure 5.   Virginia Chesapeake Bay and its tributaries with project
            site locations.

Legend on map:

1. Chippokes State Park
2. Parkway Breakwaters
3. Hog Island Breakwaters
4. Drummonds Field
5. Hog Island Headlands
6. Yorktown Bays
7. Summerille
8. Waltrip

Map labels: Potomac, Chesapeake Bay, Rappahannock, York, James, Atlantic Ocean

Table 1. Profiling and Aerial Photography Schedule by Year, Month and Day

Site	1988 Summer (Jul Aug Sep)	1988 Fall (Oct Nov Dec)	1989 Winter (Jan Feb Mar)	1989 Spring (Apr May Jun)	1989 Summer (Jul Aug Sep)	1989 Fall (Oct Nov Dec)	1990 Winter (Jan Feb Mar)	1990 Spring (Apr May Jun)
CHP — 33 profiles, baseline = 839 ft								
profiles	21	14	28	8		3		17
aerial photos	15	19	29	25				25
NPS — 27 profiles, baseline = 748 ft								
profiles	14	6	17	2		12		3
aerial photos	15	19	29	25				25
HI2 — 61 profiles, baseline = 1475 ft								
profiles	7	22	16	6		4		1
aerial photos	15	19	29	25				25
DMF — 35 profiles, baseline = 1674 ft								
profiles	13	8	30	15		10		11
aerial photos	15	19	29	25				25
WAL — 17 profiles, baseline = 447 ft								
profiles	27	20	20	26		11	13	
aerial photos	15	19	29	25				25
HIH — 30 profiles, baseline = 2400 ft								
profiles	6 20	6	17	8		5		2
aerial photos	15	19	29	25				25
YB — 21 profiles, baseline = 652 ft								
profiles	28	17 19	15	23		13	4	
aerial photos	15	19	29	25				25
SUM — 12 profiles, baseline = 863 ft								
profiles	2	5	22	5		6		30
aerial photos		19	29	25				24

base map for each site upon which the baseline, profile locations, the shoreline and banks, the breakwaters and/or headlands could be drawn to scale. Some profile data and aerial photography that had been acquired before the Chesapeake Bay Shoreline Study were also incorporated in the analysis.

Surface sediment samples were collected from the beach and nearshore areas along selected profiles at each site. Selected surface samples were analyzed for percent of gravel, sand, silt, and clay utilizing the Rapid Sand Analyzer (RSA) at VIMS. The graphic mean and standard deviation were the statistical parameters used to evaluate the sediment samples. The standard deviation is used to determine the sorting of each sample. Sediment regimes at each site are classified as backshore, beach face and nearshore.

Of the five breakwater sites, Chippokes, Hog Island Breakwaters and Parkway Breakwaters, are similar in that the breakwaters were initially located at or near mean low water (MLW). All of the breakwater sites involved structures with some degree of tombolo formation. Chippokes and Parkway Breakwaters had no beach fill added, whereas Hog Island Breakwaters and Drummonds Field did. Waltrip was constructed in 1988 with beach fill added and is located adjacent to Drummonds Field.

Of the three headland sites, only Hog Island Headlands was constructed for this project. Both Summerille and Yorktown Bays have existed as a log-spiral bay and pocket beach for over 10 years. The designation of breakwater and headland is somewhat arbitrary since all the study sites function to a degree as headland breakwaters. The main difference is that the designated headland sites have much wider gaps or bays relative to breakwater (headland) length.

The parameters that will be disscussed are depicted in Figure 6. Some parameters were obtained from Suh and Dalrymple (1987) and are L_B, G_B, h_B, X_B and X_s. The other parameters were developed by this study and include M_b, B_m, B_I, T_W, T_A, F_B, S_e, and T_e.
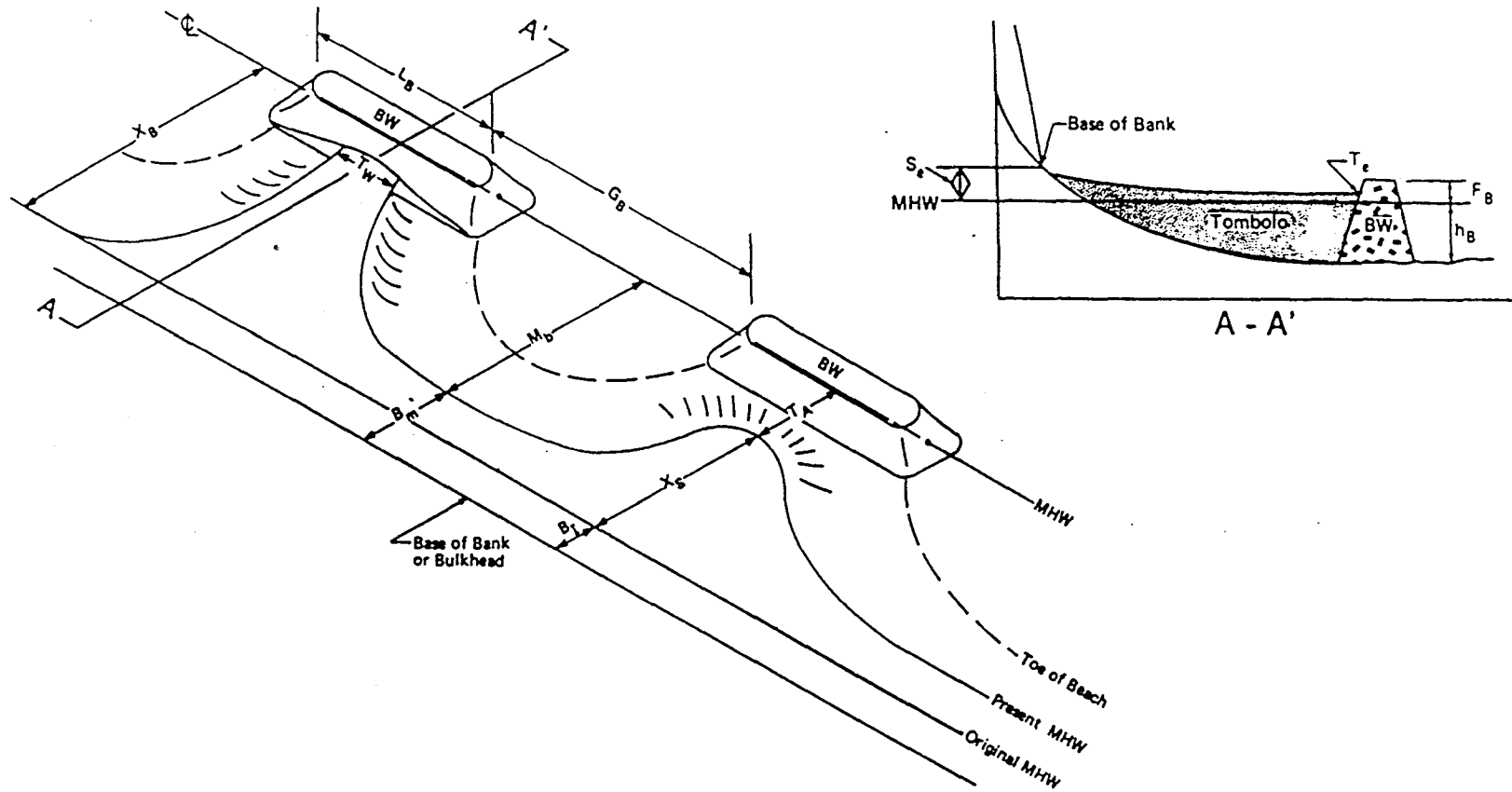
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Figure 6. Perspective sketch of breakwater parameters.

L_B – Breakwater crest length

G_B – Breakwater gap

X_B – Distance offshore CL breakwater to original MHW

h_B – Height of breakwater from bottom at CL to MHW

F_B – Breakwater freeboard, MHW to crest

M_b – Maximum bay indentation, CL breakwater to MHW

T_e – Tombolo elevation in lee of breakwater $\pm$ MHW

S_e – Backshore elevation at base of bank

B_I – Initial beach width, base of bank to MHW

B_m – Present beach width, base of bank to MHW

T_A – For unattached tombolo, MHW to CL of breakwater

T_W – For attached tombolo, tombolo width at MHW

X_s – Salient length

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B. Wave Climate

1. Fetch Limited Wave Regimes

The wave climate along the tributary estuaries of the Virginia Chesapeake Bay Estuarine System is fetch limited. Six of the eight sites in this study have average fetches of 2.0 to 3.5 nautical miles. Two sites, the Yorktown Bays and Summerille, have average fetches of approximately 10 nautical miles. The seasonal wave climate favors northerly winds in the winter and southwesterly winds in the summer (Figure 7). Mean seasonal winds generate limited waves across the rivers.

Perhaps the least understood relationship is the wave climate that ultimately drives sediment transport around fixed offshore structures. The response of the beach planform to the impinging wave climate is evident in the formation of salients and tombolos. As is often the case in wave climate assessment one tries to simplify this complex procesess for ease of understanding and comparison.

The relative wave climate regimes within the Chesapeake Bay estuarine system maybe measured in terms of average fetch exposure. According to Hardaway et al. (1984) a low wave energy shoreline would be exposed to less than 1.0 nautical mile average fetch, a medium energy shore 1.0 to 5.0 nautical miles and high energy shore greater than 5.0 nautical miles. Thus, all sites in this study are within the medium wave energy exposure except for Summerille and Yorktown which are high energy sites.

Further sub-categories within each wave energy regimes can be defined in terms of low, medium and high wave conditions. For instance, in the medium energy regime, low wave conditions are defined under normal tidal elevations where the breaking wave heights are less than 0.5 feet. Medium wave conditions are defined under normal tidal elevations with breaking wave heights of 0.5 to 1.0 feet. Generally a high wave condition will be associated with storm events (i.e. northeaster) and an elevated water level or storm surge. Breaking waves across a broader surf zone would then be 1.0 to 3.0 feet. An extreme wave condition might occur during a

RICHMOND
1948-1967
N

PATUXENT
1945-1970
N

LANGLEY
1938-1970
N
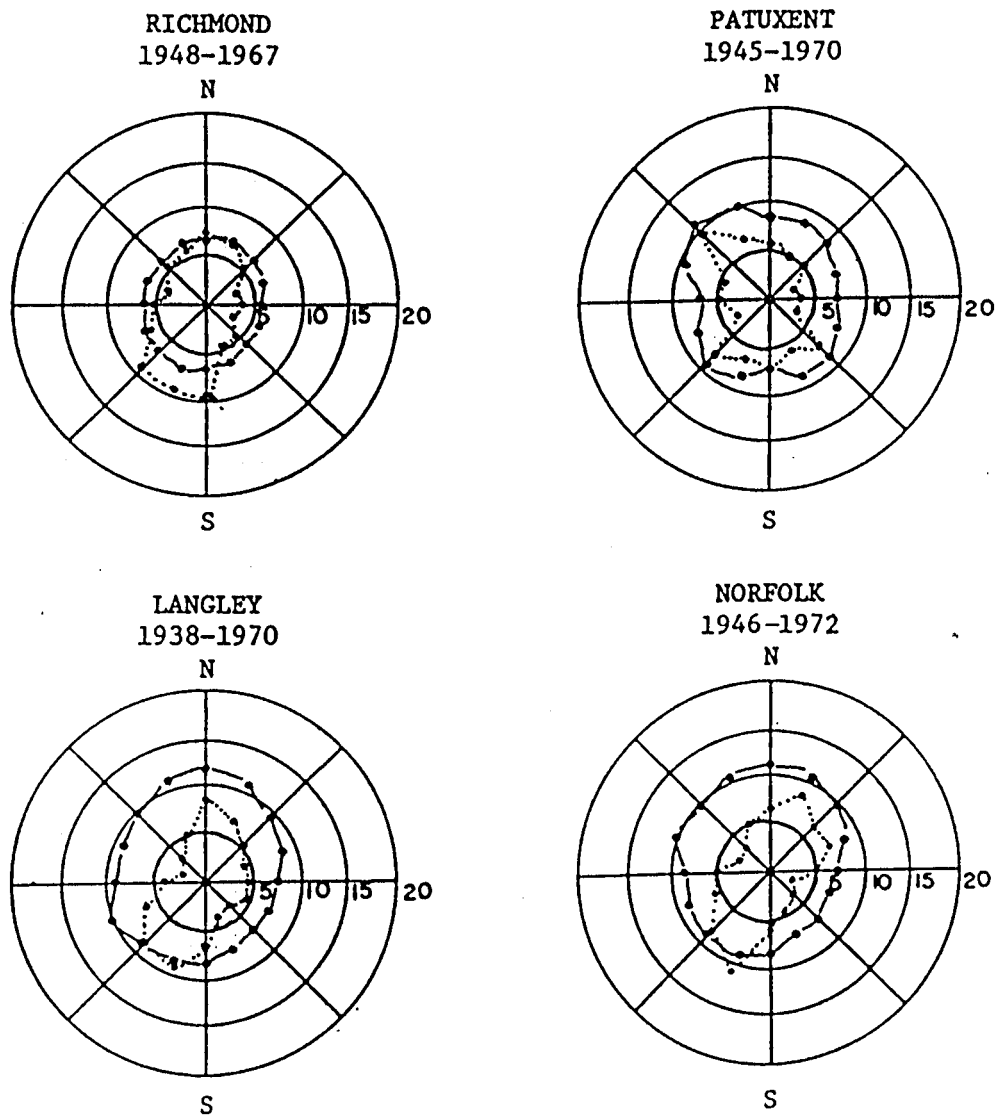
NORFOLK
1946-1972
N

Figure 7. Long term wind roses for Richmond, Patuxent,
Langley, and Norfolk.
•———• Frequency of occurrence(%)
•———• Average velocity(kts)
(Hardaway et al., 1984)

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severe storm, northeaster or hurricane with very high storm surge (greater than 4.0 feet) and breaking waves greater than 3.0 feet.

In the high energy regime the relative wave conditions would increase. The low and medium wave conditions are slightly higher than those in the medium wave energy regime. However, the high energy breaking wave condition in the high wave energy regime might be 2.0 to 4.0 feet and even greater under extreme conditions.

2. Wind - Wave Modeling

Modeling the aforementioned wave conditions at a given site is a difficult task because of the lack of long term real wave data. Therefore, indirect methods using wind hindcasting to numerically generate waves must be used.

Kiley (in press) developed a wave prediction model for shallow water estuarine conditions. This model is a quasi-empirical -- quasi-theoretical wind wave predition model developed by Sverdrup and Munk (1947) and revised by Bretschneider (1952, 1958). Wave energy losses due to bottom friction and percolation are incorporated in the model based on the relationships developed by Putnam and Johnson (1949), Putnam (1949), and revised by Bretschneider and Reid (1954). The resulting numerical model is based on successive approximations in which wave energy is added due to wind stress and subtracted due to bottom friction and percolation. The model is presented in the Shore Protection Manual (U.S. Army Corps of Engineers, 1975). The nondimensional equations for significant wave height and period as functions of wind speed, fetch and water depth are:

$$\frac{g\,H}{u^2}=0.283\ \tanh\left[0.530\left(\frac{gd}{u^2}\right)^{0.75}\right]\ \tanh\left[\frac{0.0125\left(\frac{gF}{u^2}\right)^{0.42}}{\tanh\left[0.530\left(\frac{gd}{u^2}\right)^{0.75}\right]}\right]\ , \qquad (1)$$

and

$$\frac{g\,T}{2\pi u} = 1.20 \tanh \left[0.833 \left(\frac{gd}{u^2} \right)^{0.375} \right] \tanh \left[\frac{0.077 \left(\frac{gF}{u^2} \right)^{0.25}}{\tanh \left[0.833 \left(\frac{gd}{u^2} \right)^{0.375} \right]} \right] , \qquad (2)$$

where:

  d = water depth in feet.

  F = effective fetch distance in feet.

  g = gravitational acceleration in feet/sec^2.

  H = significant wave height in feet.

  T = significant wave period in seconds.

  u = wind speed in feet/sec.

Significant wave height and period given by these by these equations increase hyperbolically with wind speed, fetch, and depth to asymptotic values referred to as "fully arisen " sea states (Kiley, in press).

In inland waters, such as estuaries, fetches are limited by the land forms surrounding the water. The general effect is to limit the radial transfer of wind energy to the water, thus resulting in wave generation significantly lower than that found in open waters. A method proposed by Saville (1954) and presented in the Shore Protection Manual (U.S. Army Corps of Engineers, 1975) is employed to estimate the effect of the surrounding land forms on the fetch and determine the effective fetch:

$$F = \frac{\Sigma x_i \cos\alpha}{\Sigma \cos\alpha} , \qquad (3)$$

where:

  F = effective fetch.

  xi = distance along radial i from point of interest to shoreline.

  α = angle between the radial and the wind direction.

This method is based on the following assumptions:

  1. Wind transfers energy to the water surface in all directions 45 degrees either side of the wind direction.

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2.   The energy transferred by the wind to the water varies as the
     cosine of the angle between the radial and the wind direction.

3.   Waves are completely absorbed at the shoreline.

The morphology of coastal plain estuaries, such as the James River
York River, and Potomac River, results in significant changes in water
depth frequently occurring along fetches that traverse the side terraces
and deeper channel areas.   A method, developed by Bretschneider (1966) and
modified by Camfield (1977), in which the variation in water depth is
incorporated into the wave prediction model by segmenting the effective
fetch into regions of locally similar depths, has been employed in Kiley's
program.   In equations (1) and (2), significant wave height and period are
functions of wind speed, effective fetch, and water depth.   Given these
equations, effective fetch can also be determined as a function of
signicant wave height (or period), wind speed, and water depth as follows:

$$
F = \frac{u^2}{g} \left[\frac{1}{0.0125} \tanh \left[0.533 \left(\frac{gd}{u^2} \right)^{0.75} \right] \tanh^{-1} \left[\frac{\left(\frac{gh}{u^2} \right)}{0.283 \tanh \left[0.533 \left(\frac{gd}{u^2} \right)^{0.75} \right]} \right] \right]^{\frac{1}{0.42}} , \tag{4}
$$

and

$$
F = \frac{u^2}{g} \left[\frac{1}{0.077} \tanh \left[0.833 \left(\frac{gd}{u^2} \right)^{0.375} \right] \tanh^{-1} \left[\frac{gT}{(1.22) 2\pi u \tanh \left[0.833 \left(\frac{gd}{u^2} \right)^{0.375} \right]} \right] \right]^{\frac{1}{0.25}} , \tag{5}
$$

The effective fetch at the beginning of each effective fetch segment
(assuming present segment depth constant to windward shore) is then
calculated from the significant wave height (or period) entering the
segment, the wind speed, and the water depth of the segment.   The result
is to reduce the effective fetch when the previous effective fetch
segments are mostly shallower than the present segment and wave growth has
been less than expected at the present water depth, and to increase the

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effective fetch when the previous effective fetch segments are mostly

deeper and wave growth has been greater than expected (Kiley, in press).

The wave climate at each site in this study was estimated using the

above procedures. For the SMB analyses, bathymetric transects for each

wave direction were created by segmenting the bottom contours into average

depths. Nearshore depths were segmented more closely to better

approximate the actual bottom slope.

Wind data were obtained from weather stations owned by Virginia Power

at the Surry Power Plant on Hog Island in the James River and the Yorktown

Power Plant on the York River. These wind data were used as input for the

Kiley modified SMB model to determine wave conditions at the study sites.

The resulting hindcast wave parameters include wave height, period and

direction.

Wave parameters from the SMB analysis may then be used as incident

wave conditions for Model Tombolos (Suh, this report). Model Tombolos is

a one-line computer program developed for this project which simulates

shoreline change. Its purpose is to predict the shoreline response to

offshore breakwaters. The breakwater system at Chippokes State Park was

chosen as the test site for this model. Model Tombolos will be discussed

further in Section IV.A.1, Section V.A.4. and in Appendix B.

C. Shore Morphology

Evaluating the effects of wave/current interaction is a complex

procedure. At this point our best estimate of long term effects of wave

climate (especially angle of wave approach) on a given site can be

determined from an evaluation of a shoreline's evolution to its present

state. Shorelines in the Chesapeake Bay Estuarine System, being composed

of varying lithologies, erode at different rates. Headland-bay situations

often evolve when one section of shore is artifically stabilized and the

adjacent shoreline is not. The long-term direction of wave aproach is

reflected in the orientation of the tangential section of the the eroded

embayment (Figure 3). Headland-bay shorelines have evolved over the past

15 to 50 years at the Yorktown Bays, Summerille, Drummonds Field and at the Hog Island Headlands. Historical aerial photography is used to determine the evolution of these sites and thus net long term direction of wave approach by recognizing the tangential or wave parallel section of the embayments.

D. Storm Events

The frequency of storm surges in the Chesapeake Bay was reported by Boon et al. (1978). At Hampton Roads, the storm surge for extratropical (i.e. northeaster) storms for a 10-year and a 50-year storm are +3.2 feet and +3.8 feet above MHW respectively. Extratropical and tropical storms with the associated storm surges and increased wave energy are the main forces causing movement of beach sand and shoreline erosion. Table 2 lists the major storm events experienced in southeastern Virginia between 1956 and 1978.

A period of sustained northeasterly winds was experienced in southeastern Virginia between April 11 and April 13, 1988. According to the National Oceanic and Atmospheric Administration (NOAA), the average peak wind speeds and directions at Norfolk International Airport were as follows:

Date	Average Speed (mph)	Direction	Peak Speed (mph)	Direction
11 April	10.6	NE	23.0	E
12 April	23.4	NE	47.0	NE
13 April	28.3	NNE	51.0	NE

Storm surges measured at VIMS ranged from about +1.0 foot MHW on April 11, 1988 to about +3.0 feet MHW on April 13, 1988. Field observations were made at Yorktown Bay 1, Parkway Breakwaters, Drummonds Field, Chippokes, Hog Island Breakwaters and Hog Island Headlands on April 13, 1988, during the peak of the storm. The results of these observations are shown in Table 3. The observed wave parameters were measured just outside the line of breakwaters or just before breaking.

Table 2. Occurrence of Major Storms in Southeastern Virginia From 1956-
1978. Data from Sewells Point and Norfolk International
Airport.

Storm	Date	Storm Surge (ft)	Wind Speed (kn)	Direction
	11 Jan 1956	3.4	33	NE
	11 Apr 1956	4.3	62	N
	03 Nov 1956	2.0	29	NE
	28 Feb 1957	2.4	33	NE
	08 Mar 1957	2.2	27	NE
	01 Nov 1957	2.7	28	NE
	25 Jan 1958	2.3	44	E
	01 Feb 1958	2.2	30	W
	19 Mar 1958	2.2	21	NE
	27 Mar 1958	2.6	20	N
	11 Dec 1958	2.1	27	NE
	29 Dec 1958	2.3	38	E
	12 Apr 1959	2.5	45	NE
	19 Dec 1959	2.1	29	N
	31 Jan 1960	3.0	42	NE
	13 Feb 1960	2.3	49	NE
	03 Mar 1960	2.4	52	E
	12 Dec 1960	2.0	40	W
	16 Jan 1961	2.0	13	W
	08 Feb 1961	2.4	27	NE
	22 Mar 1961	2.2	33	E
	28 Nov 1961	2.0	23	NW
	28 Jan 1962	2.2	37	NE
Ash Wed	07 Mar 1962	5.6	41	NE
	22 Mar 1962	2.4	20	N
	03 Nov 1962	2.5	33	N
	26 Nov 1962	3.3	41	N
	08 Feb 1963	2.3	30	NE
	06 Nov 1963	2.4	38	E
	04 Jan 1964	2.0	28	W
	12 Jan 1964	2.6	42	E
	12 Feb 1964	2.0	32	E
Cleo	01 Sep 1964	1.0	42	ESE
Dora	18 Sep 1964	0.3	61	NE
Gladys	23 Sep 1964	2.3	44	N
Isabell	16 Oct 1964	2.6	50	NE

Table 2 (continued)

Storm	Date	Storm Surge (ft)	Wind Speed (kn)	Direction
	16 Jan 1965	3.9	35	NE
	22 Jan 1965	3.0	36	E
	29 Jan 1966	3.6	37	E
Alma	13 Jun 1966	1.0	40	N
	24 Dec 1966	2.3	31	NE
	07 Feb 1967	2.6	33	NE
Doria	16 Sep 1967	3.4	55	N
	12 Dec 1967	2.0	30	E
	29 Dec 1967	2.0	31	W
	14 Jan 1968	2.3	33	E
	08 Feb 1968	2.6	30	NE
Gladys	20 Oct 1968	1.3	46	NE
	10 Nov 1968	4.3	34	N
	12 Nov 1968	2.6	47	NE
	02 Mar 1969	5.9	40	N
	02 Nov 1969	2.6	36	NE
	10 Nov 1970	2.6	22	SE
	16 Dec 1970	2.0	31	E
	27 Mar 1971	2.8	45	NE
	06 Apr 1971	4.0	44	NE
	19 Oct 1972	-	34	N
	11 Feb 1973	3.5	44	N
	21 Mar 1973	3.1	28	N
	02 Mar 1975	2.2	22	SSE
	14 Oct 1977	2.6	29	NE
	30 Oct 1977	2.3	24	NE
	20 Dec 1977	-	-	-
	28 Apr 1978	4.6	39	NE

(W.S. Richardson, U.S. Weather Service, personal communication, 1979)

Revised from Senate Document No. 4, Report of the Coastal Erosion Abatement Commission, 1979.

Table 3. Wave Observations, Northeaster of 13 April 1988

Site	Wave Angle (TN) (degrees)	Wave Height (feet)	Wave Period (seconds)
YB1	65-70	2.0-2.5	3.5-4.0
NPS	40	1.0-1.5	2.0
HI2	335	1.0-1.5	2.0-2.5
HIH	25	1.0-1.5	2.0-2.5
CHP	15	1.0-2.0	2.5-3.0

In 1989, two coastal storms passed through southeastern Virginia. The first storm occurred on February 24 in the form of a blizzard with wind gusts to 50 mph from the northwest to the north northeast. Winds at Norfolk International Airport averaged 26.2 mph (U.S. Dept. of Commerce) and a storm surge of only 1.5 feet above MHW was observed. The second storm occurred on March 6 through 9 as a moderate northeaster with average winds of about 24 mph and gusts of 40 to 45 mph (U.S. Dept. of Commerce, 1988). This storm mostly affected the ocean coast of Virginia where most of the property damage occurred. There was about a 2-foot storm surge in the Chesapeake Bay but very little wind and wave action was experienced as compared to the April 1988 storm. There was essentially no storm event observed through the fall and winter of 1990.

IV. Project Sites

    A. Breakwater Sites

        1. Chippokes State Park, James River, Surry County

           a. <u>Setting</u>

The gapped breakwater system at Chippokes State Park is located on Cobham Bay (Figure 8). Chippokes is a recreational and historic state park as well as a "model" farm. The site lies within an estuarine reach of the James River between College Run and Lower Chippokes Creek. The reach is characterized by high (40 ft), eroding, fastland banks which give way to low fastland banks toward each bounding drainage. The high banked shore at Chippokes faces almost due north.

Cobham Bay appears to be the geomorphological remnant of the outside bank of a meander of the ancestral James River. Erosion of the bank is driven by wind and waves from the northeast and northwest. The high banks are composed of a lower unit of shelly, fossiliferous, fine to coarse sand overlain by an upper layer of slightly muddy, fine to medium sand. Net transport here is eastward but with seasonal fluctuations and onshore-offshore movement.

The preconstruction beach at Chippokes was a curvilinear strand of sand about 25 feet wide from MHW to the base of the bank. The beach itself consists of a fine to coarse, well sorted, shelly sand derived from the eroding bluff.

           b. <u>Wave Climate</u>

The Chippokes breakwater system faces almost due north with an average fetch of 2.4 nautical miles. Long fetches of 5.0 and 8.0 nautical miles occur to the north northeast and northwest, respectively. Strong seasonal winds from the north and northwest tend to force beach sediments to the east. During northeast storm events (i.e. April 1988), waves approach from the north to north northeast with breaking wave heights of 1.4 to 1.8 feet. Wave conditions at the breakwaters are seen in Figure 9 for the period January 1987 to August 1990. The mean wave height for this

period is 0.72 feet, 0.60 feet and 0.54 feet for waves from the northwest, north, and northeast respectively.

c. Design and Construction

The goal at Chippokes was to design a system which would permit a tombolo to form utilizing the existing volume of sand on the beach, such that, with time a stable backshore would develop and protect the base of the high banks. A system of six breakwater units with a length to gap ratio of 1:1.5 was designed (Figures 10A and 10B). The crest lengths are 50 feet and gaps are 75 feet. The centerline of the breakwaters is approximately 30 feet from the initial MHW line and the crest width of each breakwater is 4 feet.

Construction of the breakwater system took place during June 1987. A road had to be cut down the bank to provide access for the equipment. Subsequent rains washed out the road several times, thus providing additional material to the beach system. Rock for construction of the rubble mound breakwaters, as depicted in the SPM (U.S. Army Corps of Engineers, 1984), was trucked in and dumped over the bank behind the site for each breakwater unit. The rock was then placed with a large, tracked backhoe to form each unit.

d. Shore Changes

Figure 11 shows shoreline changes from June 1987 to April 1990. The position of the MHW line was used to track beach changes at each site. Sand began accumulating and migrating toward each breakwater unit as cuspate spits formed almost immediately after construction. The shore behind breakwater number 1 showed the quickest response. The characteristic double spits (Perlin, 1979, Rosen and Vajda, 1982) evolved behind each structure by September 1987. By February 1988 the double salients or saddles had coalesced into a single, attached tombolo with a swale between the saddles. Sediment for the tombolos was derived from the adjacent embayments. This is most evident in Bays A, B, and C. Tombolos eventually attached symmetrically to the lee of each breakwater.

By February 1988 all the bays showed signs of filling. Accretion of sand on the west end of the system and a marked loss of sand on the east end was apparent. From this one would infer a net west to east movement of sand along this portion of the reach.

On April 11, 12 and 13, 1988, northeast winds blew continuously across the James River and Cobham Bay. Post-storm surveys showed a general decrease in the intertidal beach slope in the center of each embayment and erosion along the base of the bank (Hardaway et al. 1989). There was a corresponding increase in tombolo elevation (T_e) behind each breakwater but the overall tombolo widths (T_w) were reduced. Material contributing to the increased tombolo elevation came partially from the eroded embayed beaches and partially from runoff down the upland banks. There appeared to be no significant offshore movement of beach material. The April 1990 shoreline shows an equilibrium condition that is reflected in the average bay and breakwater parameters (Table 4).

The overall shape of each embayment has remained constant. Slight shifts are observed in beach postion in response to more oblique northwest winds but a general symmetrically curvilinear planform persists.

To date the tombolos at Chippokes have been colonized by various upland species of vegetation including cypress seedlings (Figure 10C). This vegetative cover has had the opportunity to spread and grow due to the lack of effective storm events since April 1988. This "erosion resistant turf" should help in attenuating wave action during future storm events.

e. Sediments

Sediment samples taken along profile 13 in Bay B show that the sorting of all samples remains good through the study period (Figure 12B). However the mean grain size varies for each sediment regime with time, the coarser material occurs along the backshore and beach face and the finer sediments in the nearshore (Figure 12A).

The net rate of volumetric change behind each breakwater and along each embayed shoreline in the current state of equilibrium is seen in Table 4. These volumes are computed from the pre-construction shoreline and the last survey. V_f is the current amount of sediment in the lee of each structure and contained along the shore of each bay.

Sand accretion on the updrift end and behind breakwater 1 is observed as well as sand bypassing into bay A and behind breakwater 2. Breakwater 3, in the center of the system has accumulated less material than the other structures. Bays B, C, and D have lost sand while Bay E has gained a significant amount. This increase is mostly from the continued washout of the contruction access road at that point. For the most part the base of the bank continues to erode. Most of the losses occured durng the April 1988 storm (Hardaway et al., 1989). Shore profile changes for the project period are found in Appendix A.

f. Model Tombolos

Model Tombolos is a one-line numerical computer model developed for this project. The model was applied to the simulation of the shoreline response during the first eight months after the construction of the breakwaters at Chippokes. The Chippokes site was selected because this model requires essentially a straight initial shoreline behind the breakwater system. As the model runs, the shoreline is adjusted at specific time intervals in response to the hindcasted wave input. Results of the model runs show a very close shoreline correlation to what was measured in the field. Further discussion of Model Tombolos is reserved for the results section (V.A.1.).

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J A M E S   R I V E R

C O B H A M   B A Y

Bayse Point

SITE

Chestnut

Tidal Flat

Broders

Cobham

Tidal Flat

Chippokes

College Run

Run

Cem

Highgate

Lower

Chippokes

Castle Mill Run
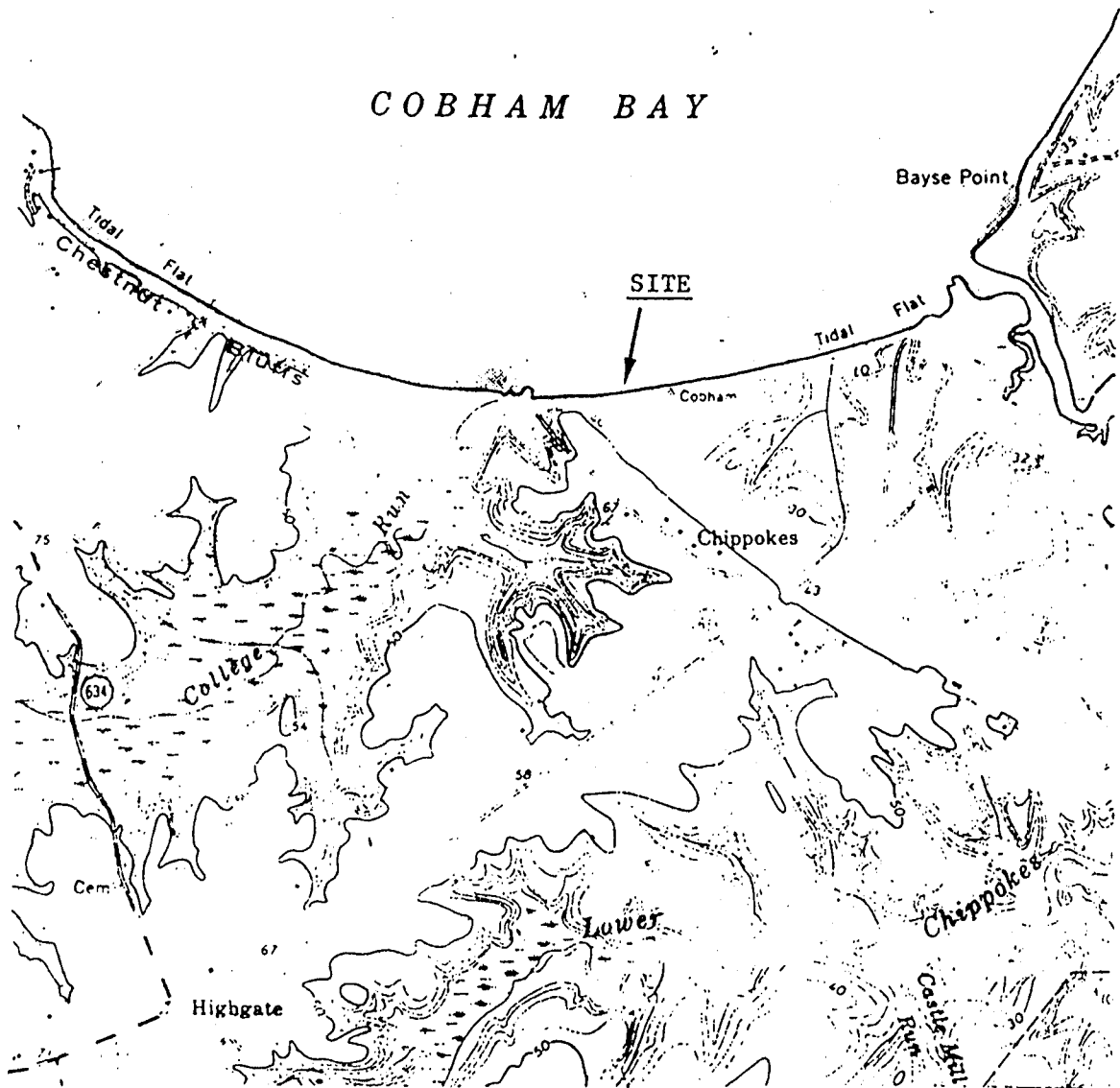
Figure 8. Chippokes State Park, James River, Surry County.
From Hog Island 7.5 minute quadrangle.
Scale: 1 inch = 2,000 feet.

AVERAGE WAVE HEIGHT AT CHP
DURING THE PERIOD 1987 - 1990
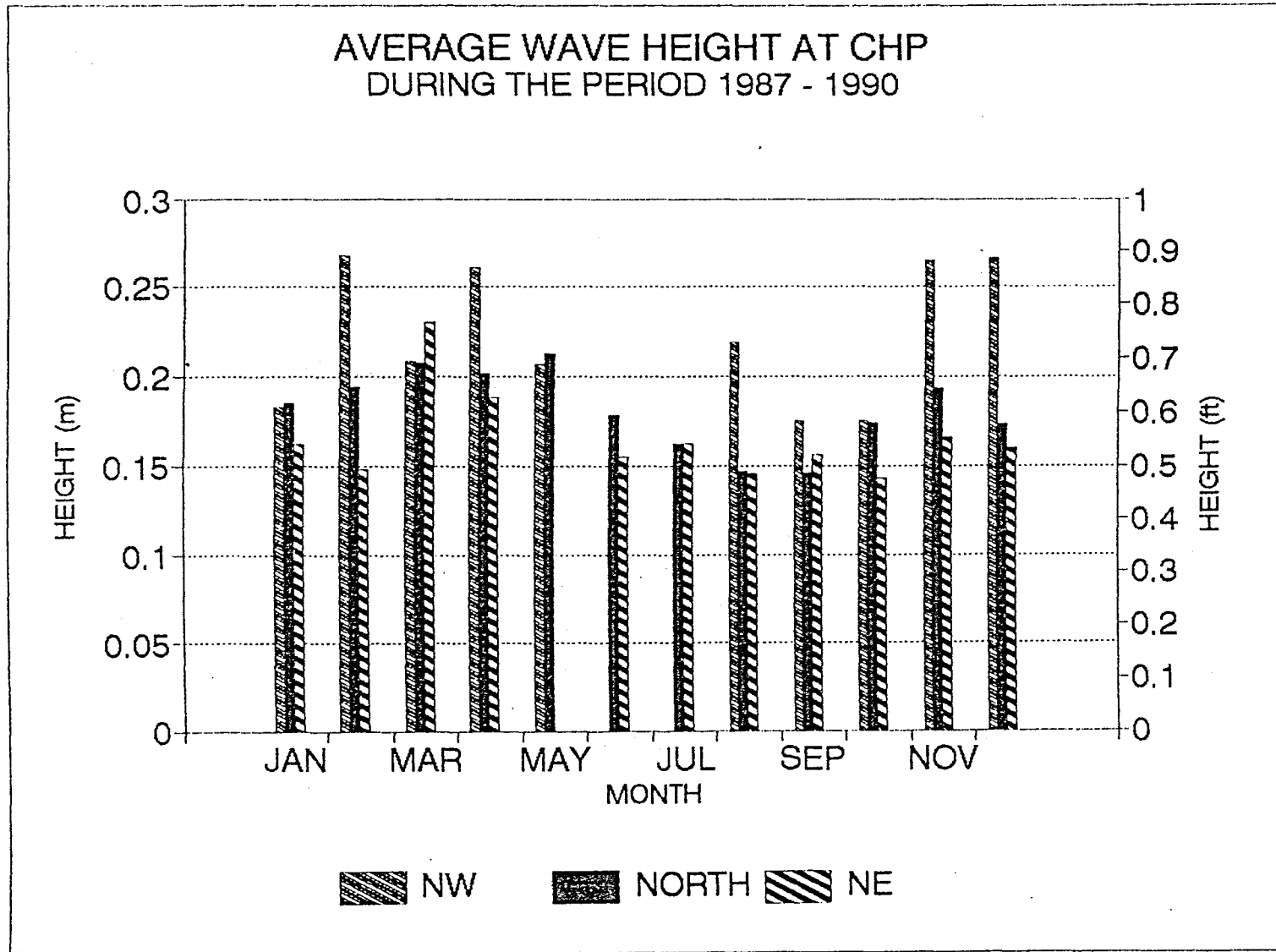
Figure 9. Chippokes State Park wave heights.

Figure 10A.   Chippokes State Park – vertical aerial, 25 September 1987.

Figure 10B.   Chippokes State Park – vertical aerial, 25 May 1990.
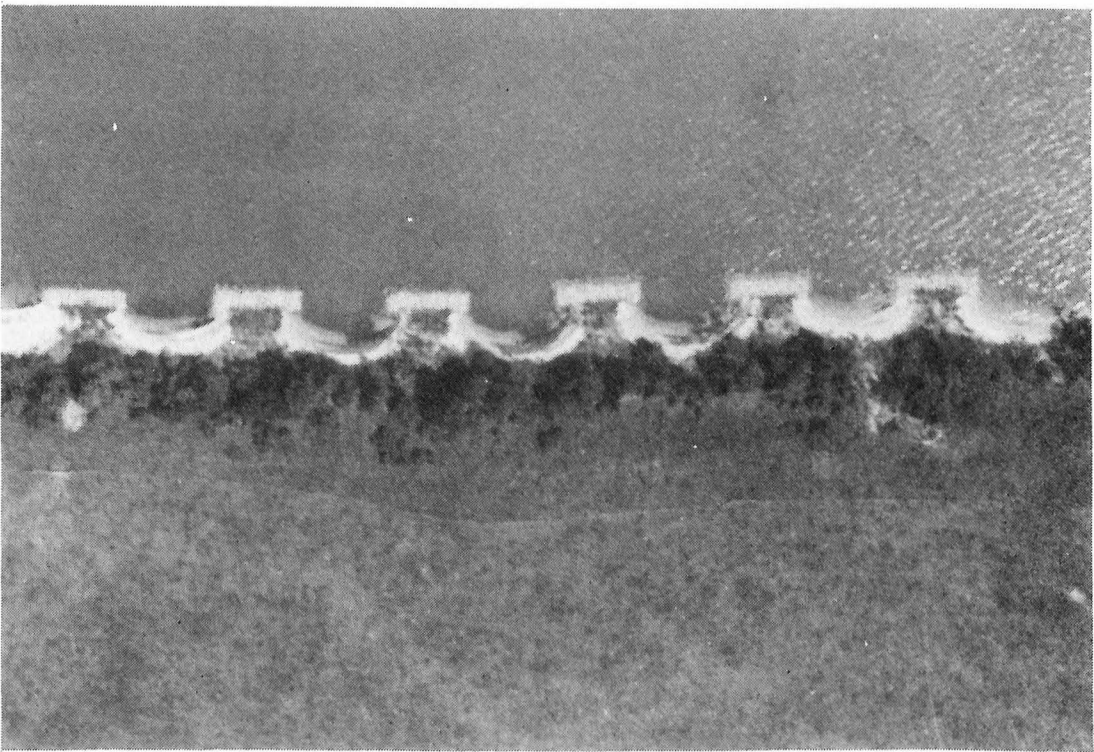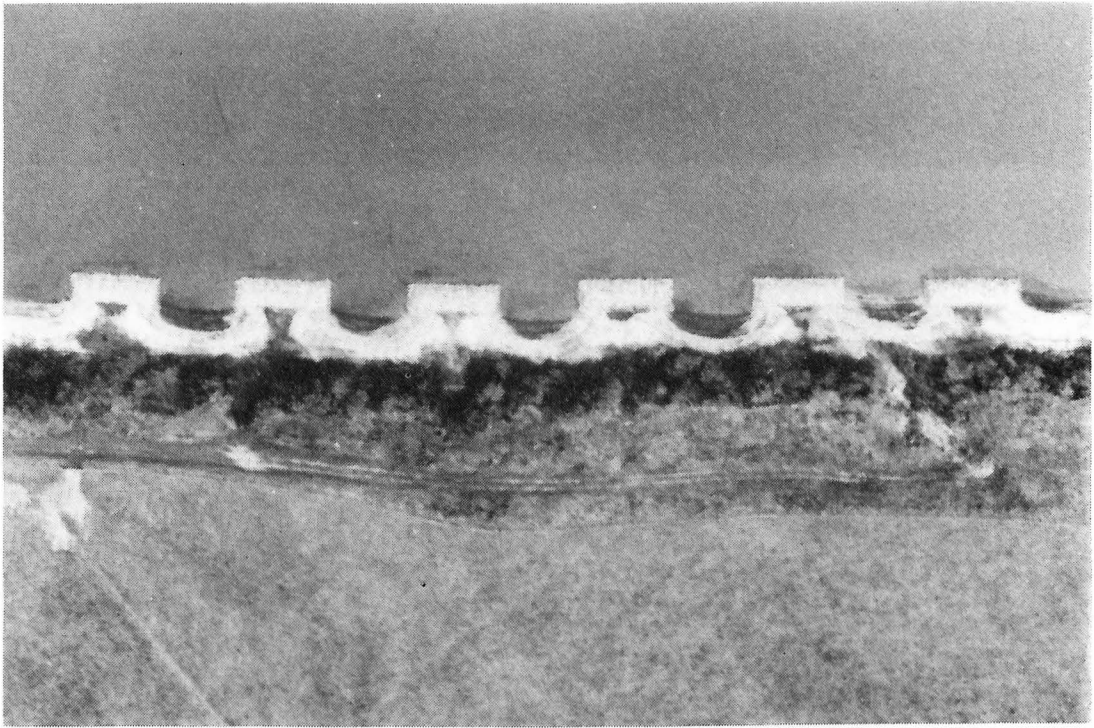
Figure 10C.   Chippokes State Park — ground view looking east from
breakwater number 1.

MN ↑ N

JUN 87 MHW
SEP 87 MHW
APR 90 MHW
APR 90 TOE

FLOOD ◄────► EBB

JAMES RIVER

A   B   C   D   E

1   2   3   4   5   6

1   3   6   10   13   15   17   19   22   26   29   31   33   ─ BASELINE

◄─ TOP OF BANK

FENCE LINE

50'
GRAPHIC SCALE

Figure 11. Chippokes State Park base map.

Table 4. Parameters for Chippokes State Park*
May 1990 (See Figure 6 for Definition Sketch)

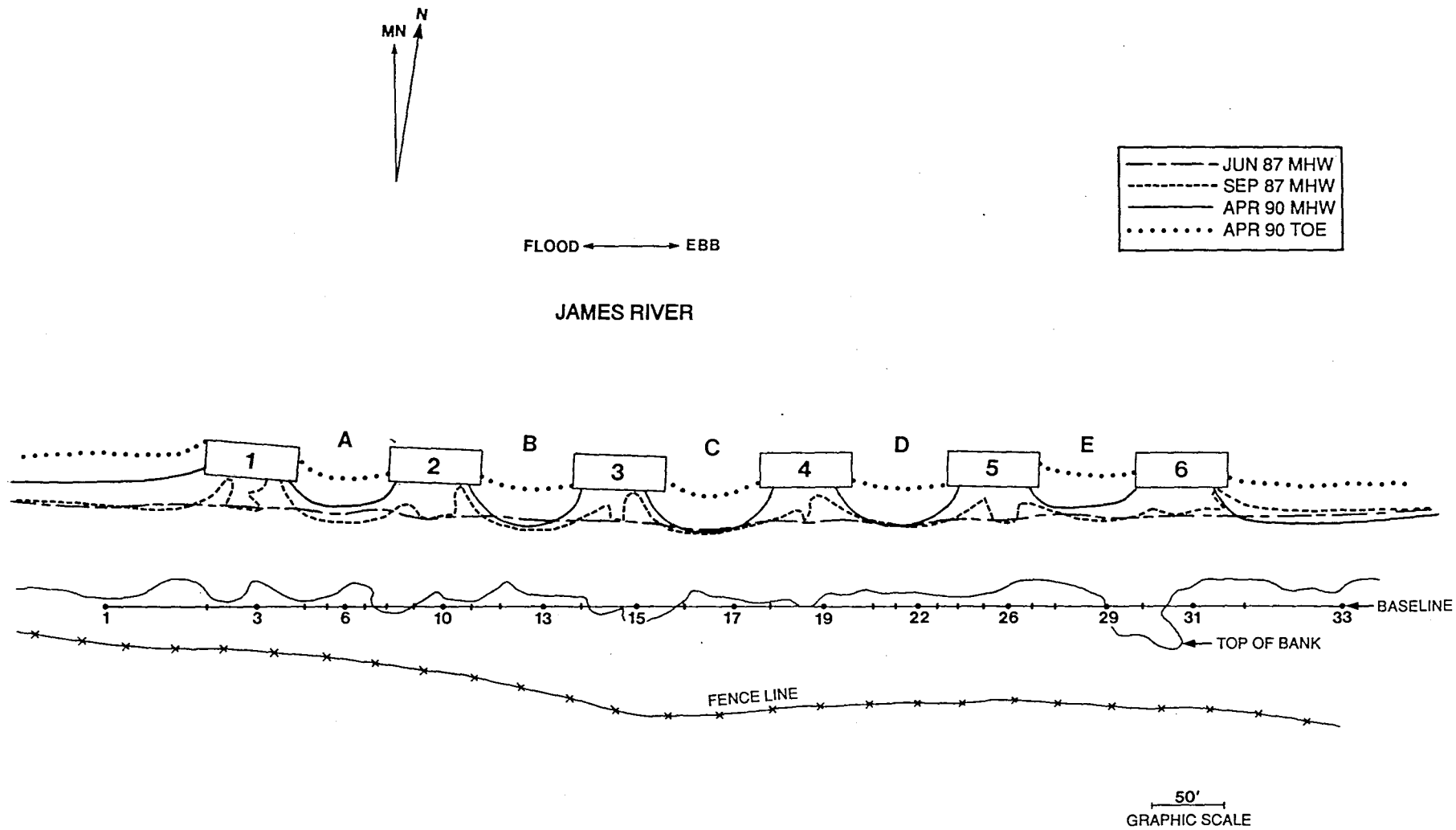| Breakwater/Bay	L_B	G_B	X_B	h_B	F_B	M_b	T_e	S_e	B_I	B_M	X_s	T_A	T_W	E_R	V_f
Updrift								3.0	23	40		-		0.0	1.7
Breakwater 1	50		30	2.5	2.2		0.9	4.5	20		22	-	45	-0.3	2.9
Bay A		75				28		3.0	20	29		-		-0.6	1.1
Breakwater 2	50		30	2.7	2.3		0.8	2.8	20		24	-	45	-1.0	2.6
Bay B		75				36		2.2	21	19		-		-0.3	0.1
Breakwater 3	50		33	2.9	2.1		0.9	3.0	16		22	-	39	0.6	1.6
Bay C		75				37		1.0	13	11		-		0.5	-0.1
Breakwater 4	50		35	2.6	2.1		0.8	2.7	15		25	-	40	-0.4	2.0
Bay D		75				37		1.2	10	13		-		-2.0	-0.1
Breakwater 5	50		35	2.8	2.0		1.2	3.0	12		23	-	45	-0.9	2.1
Bay E		75				26		1.0	10	16		-		-1.7	1.3
Breakwater 6	50		33	2.6	2.1		1.0	4.0	10		21	-	46	-1.1	2.0
Downdrift								0.9	14	14		-		-0.5	-0.4

*   All dimensions in feet except E_R and V_f.

L_B - Breakwater crest length

G_B - Breakwater gap

X_B - Distance offshore CL breakwater to original MHW

h_B - Height of breakwater from bottom at CL to MHW

F_B - Breakwater freeboard, MHW to crest

M_b - Maximum bay indentation, CL breakwater to MHW

T_e - Tombolo elevation in lee of breakwater $\pm$ MHW

S_e - Backshore elevation at base of bank

B_I - Initial beach width, base of bank to MHW

B_m - Present beach width, base of bank to MHW

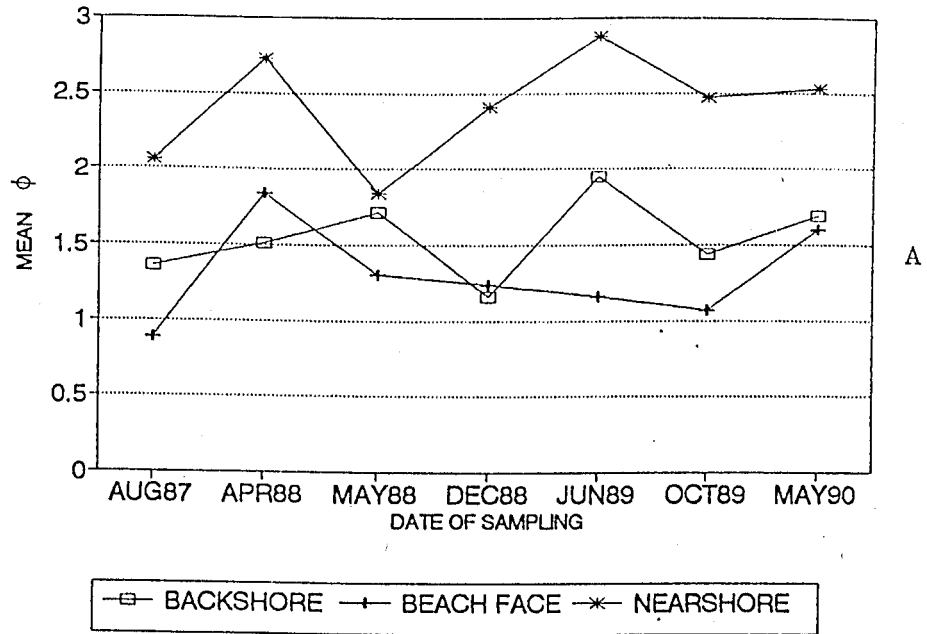X_s - Salient or tombolo length

T_A - For unattached tombolo, MHW to CL of breakwater

T_W - For attached tombolo, tombolo width at MHW

E_R - Erosion rate of BOB (ft/yr)

V_f - Net beach volume from 1988 - 1990 (cy/ft)

SEDIMENT ANALYSIS
CHIPPOKES STATE PARK, PROFILE 13

A

SEDIMENT ANALYSIS
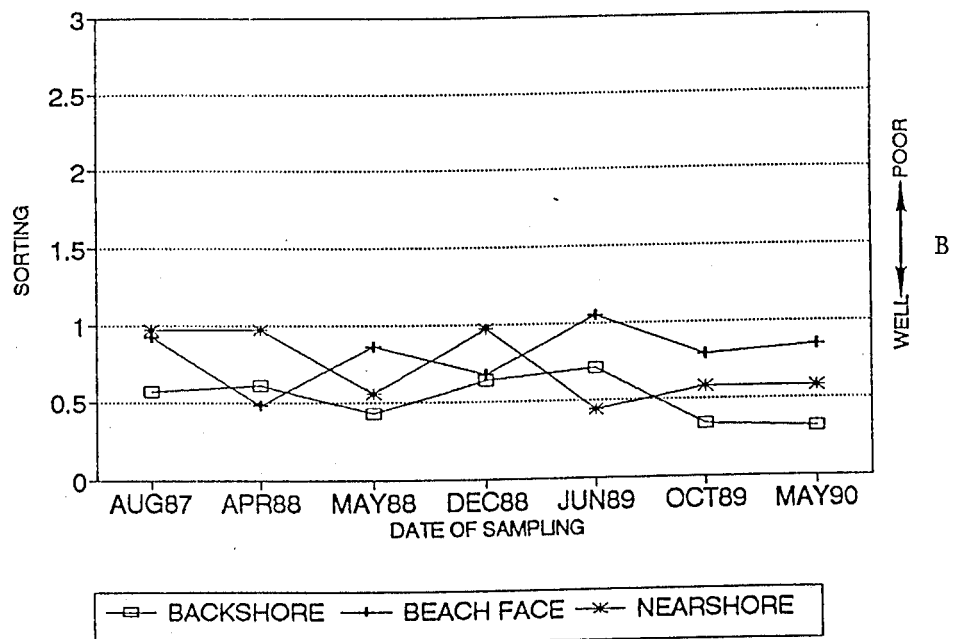CHIPPOKES STATE PARK, PROFILE 13

B

Figure 12.   Chippokes State Park sediment analysis.

2.   Parkway Breakwaters, York River, York County

a.   Setting

The Parkway breakwater system is located along the Colonial Parkway
in the Colonial National Historical Park.   The site is located between
Sandy Point and the piers at Yorktown Naval Weapons Station (Figure 13).
The historical erosion rate along this reach is approximately 1.5 ft/yr
(Byrne and Anderson, 1978).

The Parkway breakwater system is situated within a shallow crenulate
shaped embayment between two low headlands.   The upriver headland is the
terminal end of a rock revetment with a salt marsh fringe in its lee.
There is a narrow salt marsh fringe approximately 800 feet downriver
comprising the next headland.   Prior to construction, a narrow curvilinear
beach connected the two headlands.   The beach is medium to coarse sand and
gravel with abundant shell fragments.   This material is derived from
erosion of the adjacent low bank which is dredged material composed of
shelly coarse sand and gravel.   The northerly facing tangential shore
indicated a net downstream littoral drift.

b.   Wave Climate

The shore at Parkway breakwaters faces approximately northeast and
has an average fetch of 1.78 nautical miles.   The mean wave height for
each month of the study period are shown in Figure 14 for the wind window
at the site. Predicted mean wave heights for north, northeast and easterly
winds are 0.63, 0.57 and 0.34 feet respectively.

c.   Design and Construction

The Parkway breakwater system was designed around the available
material and construction force.   The original design called for a
trapazoidal cross-section similar to the rubble mound structures at
Chippokes.

The Parkway breakwaters were built in May 1985.   Four hundred pound
concrete blocks were placed with a crane in a rectangular crib
configuration.   Then, concrete slabs were broken up and placed inside the

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crib. The cross section of each unit resembled a rectangle rather than a trapazoid.

Five units were placed at approximately the MLW line. Limitations in the equipment prevented the breakwater system from being constructed as designed (which was five equal length and equally spaced units placed at -0.5 ft MLW). As finally constructed there were five breakwater units with decreasing gap from upriver to downriver (Figures 15 and 16A). The breakwater systems' parameters are listed in Table 5.

d. Shore Changes

The Parkway breakwaters were exposed to the "no-name storm" of November 4, 1985. The main direction of wave attack during the storm was observed to be east northeast with a storm surge of over 2 feet MHW. An average of 10 feet per linear foot of fastland bank was eroded. Bank erosion provided additional sand that widened the backshore. The storm mostly affected the shore between breakwaters 1, 2, and 3 and was responsible for high annual erosion rates as of 1988 (Table 6).

The shore at Parkway breakwaters is typically beset by frequent northwest and northerly winter winds. The relatively wide gaps and oblique incident waves have resulted in the formation of shallow crenulate bays. These gaps are most pronounced in Bays A and B. The orientation of the tangential shore indicates onshore wave approach to be approximately N 25° E. Bays C and D are more narrowly spaced and are more symmetrical than Bays A and B (Figure 16B).

The April 1988 northeaster caused additional bank erosion at Parkway breakwaters which also widened the backshore along most of the site (Figure 16C). Storm waves approached the site during high tide from between 35 and 40 degrees (TN). This caused a shift in the beach planform to a more symmetrical shape with flattened embayments. This general symmetrical planform persisted until the late fall of 1988 when the return of northwesterly winds reshaped the beaches into a log-spiral configuration. The late winter storms of 1989 once again shifted the

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embayed beach sands into a symmetrical planform under the influence of more northeasterly winds. Bank erosion rates have decreased as the backshore beach width has increased (Table 5).

e. Sediments

The bank and backshore are characterized by shelly, medium sand. Silty fine sands reside offshore and gravelly medium sands dominate the beach face. Beach samples show little change through time (Figure 17). The shells are from the Yorktown Formation (Pliocene) which occurs just below the bottom and they comprise a large percentage of the beach material.

Beach volume changes show net losses in Bays A and B (Table 5). This material most likely was shifted behind adjacent breakwaters as well as downriver. Bank erosion has supplied additional sediments and accounts for the increase in beach volume behind breakwaters 3, 4 and 5.
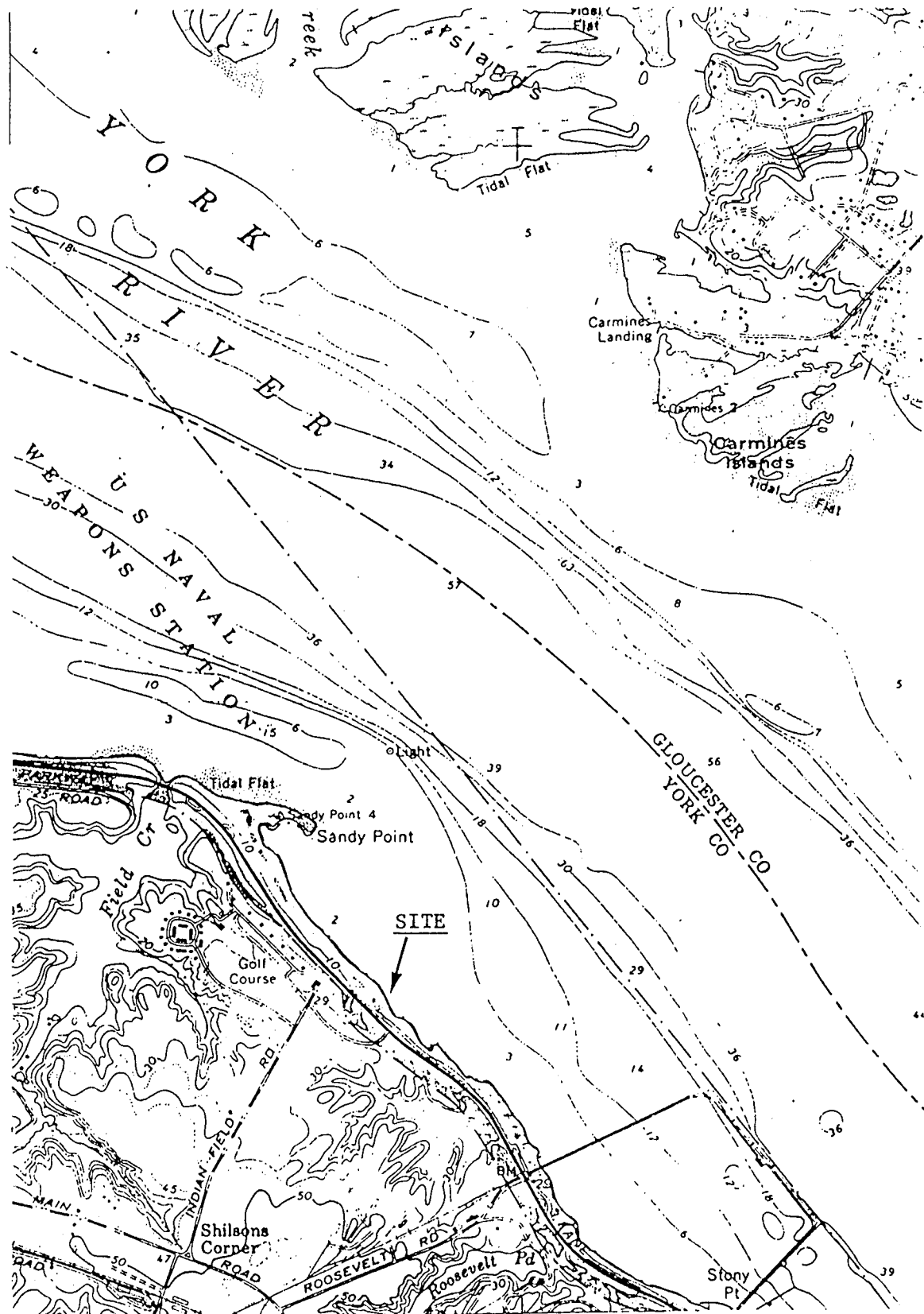
Figure 13. Parkway Breakwaters, York River, York County.
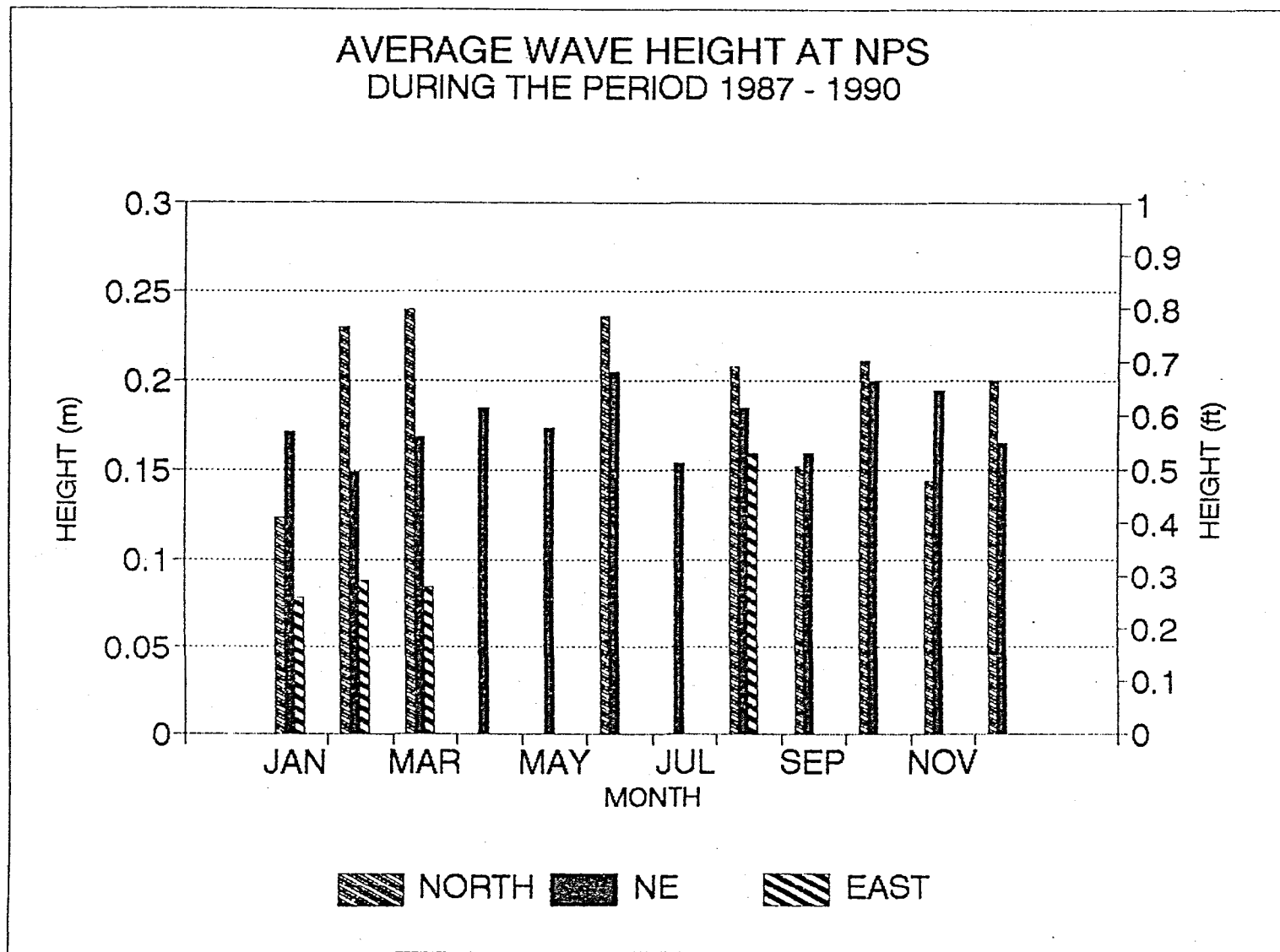From Clay Bank 7.5 minute quadrangle.
Scale: 1 inch = 2,000 feet.

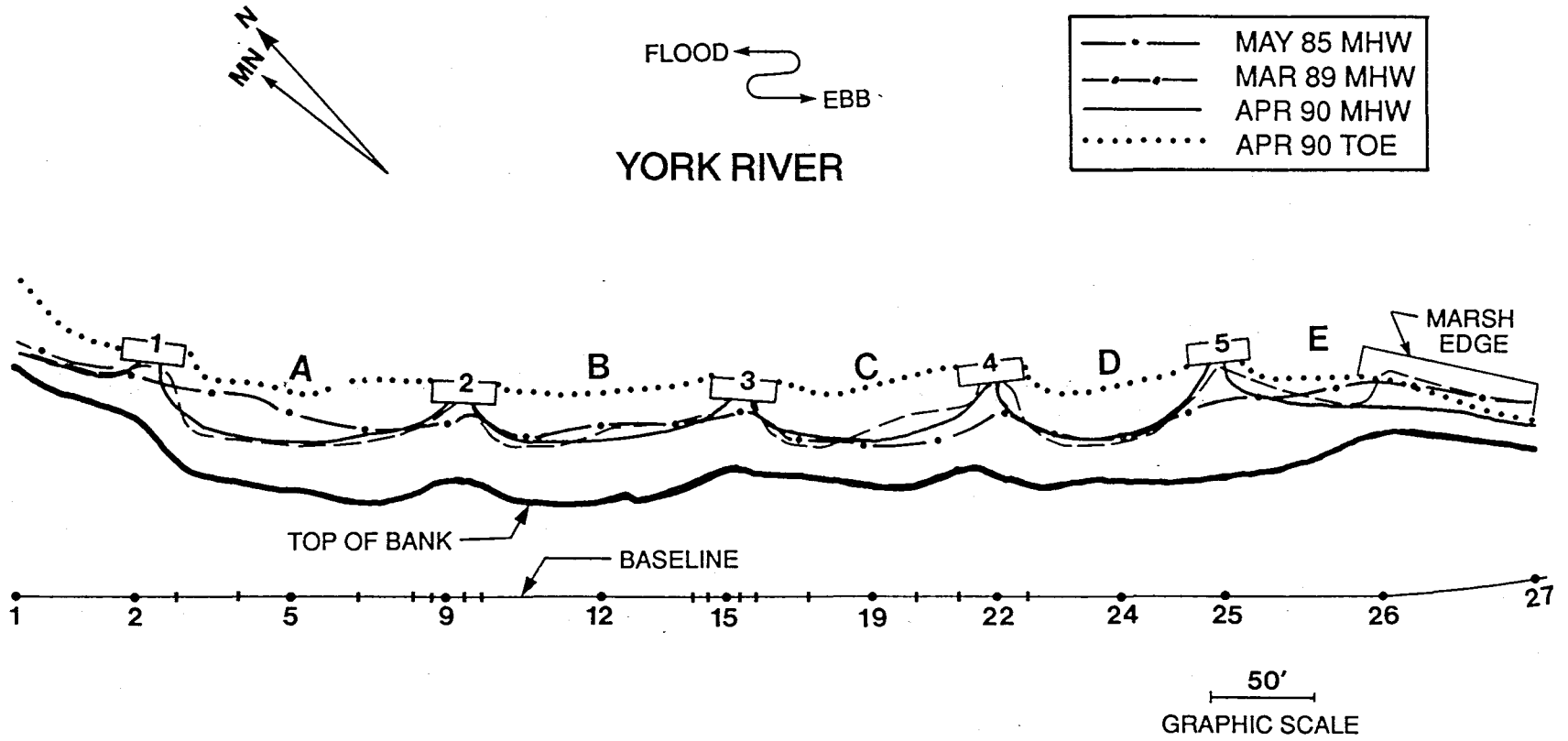Figure 14.   Parkway Breakwaters wave heights.

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Figure 15.   Parkway Breakwaters base map.

Table 5. Parameters for Parkway Breakwaters*
May 1990 (See Figure 6 for Definition Sketch)

Breakwater/Bay	L_B	G_B	X_B	h_B	F_B	M_b	T_e	S_e	B_I	B_M**	X_s	T_A	T_w	E_R	V_f
Breakwater 1	36		14	1.9	1.3		1.0	1.6	8		7	-	15	-2.2	0.3
Bay A		118				33		2.2	8	24		-		-2.4	-1.7
Breakwater 2	38		10	1.9	1.3		1.2	2.4	10		10	-	13	-1.5	0.4
Bay B		100				34		2.1	7	28		-		-2.8	-0.9
Breakwater 3	34		10	2.3	1.3		1.0	2.2	14		5	-	16	0.3	1.5
Bay C		85				28		2.0	4	21		-		-0.1	0.8
Breakwater 4	33		20	2.6	0.5		0.4	1.6	20		16	-	10	-0.1	2.0
Bay D		80				34		1.8	15	16		-		0.4	0.2
Breakwater 5	22		20	2.5	0.5		0.5	1.7	20		18	-	8	0.0	0.7

*   All dimensions in feet except E_R and V_f.

**   Distance between MHW + base of bank does not reflect change in position of the base of bank, refer to Fig. 14.

L_B - Breakwater crest length

G_B - Breakwater gap

X_B - Distance offshore CL breakwater to original MHW

h_B - Height of breakwater from bottom at CL to MHW

F_B - Breakwater freeboard, MHW to crest

M_b - Maximum bay indentation, CL breakwater to MHW

T_e - Tombolo elevation in lee of breakwater $\pm$ MHW

S_e - Backshore elevation at base of bank

B_I - Initial beach width, base of bank to MHW

B_m - Present beach width, base of bank to MHW

X_s - Salient or tombolo length

T_A - For unattached tombolo, MHW to CL of breakwater

T_w - For attached tombolo, tombolo width at MHW

E_R - Erosion rate of BOB (ft/yr)

V_f - Net beach volume from 1988 - 1990 (cy/ft)

Table 6. Bank Erosion Rates, Parkway Breakwaters
March 1988

Breakwater 1	7 ft =	− 2.5 ft/yr
Bay A	23 ft =	− 8.1 ft/yr
Breakwater 2	35 ft =	−12.3 ft/yr
Bay B	20 ft =	− 7.0 ft/yr
Breakwater 3	25 ft =	− 8.8 ft/yr
Bay C	15 ft =	− 5.3 ft/yr
Breakwater 4	6 ft =	− 2.1 ft/yr
Bay D	5 ft =	− 1.8 ft/yr
Breakwater 5	10 ft =	− 3.5 ft/yr
Downdrift	20 ft =	+ 7.0 ft/yr

Figure 16A. Parkway Breakwaters - ground view looking east. Breakwater
number 1 is in the foreground.

Figure 16B.   Parkway Breakwaters - vertical aerial, 9 March 1988.

Figure 16C.   Parkway Breakwaters - vertical aerial, 20 April 1988.
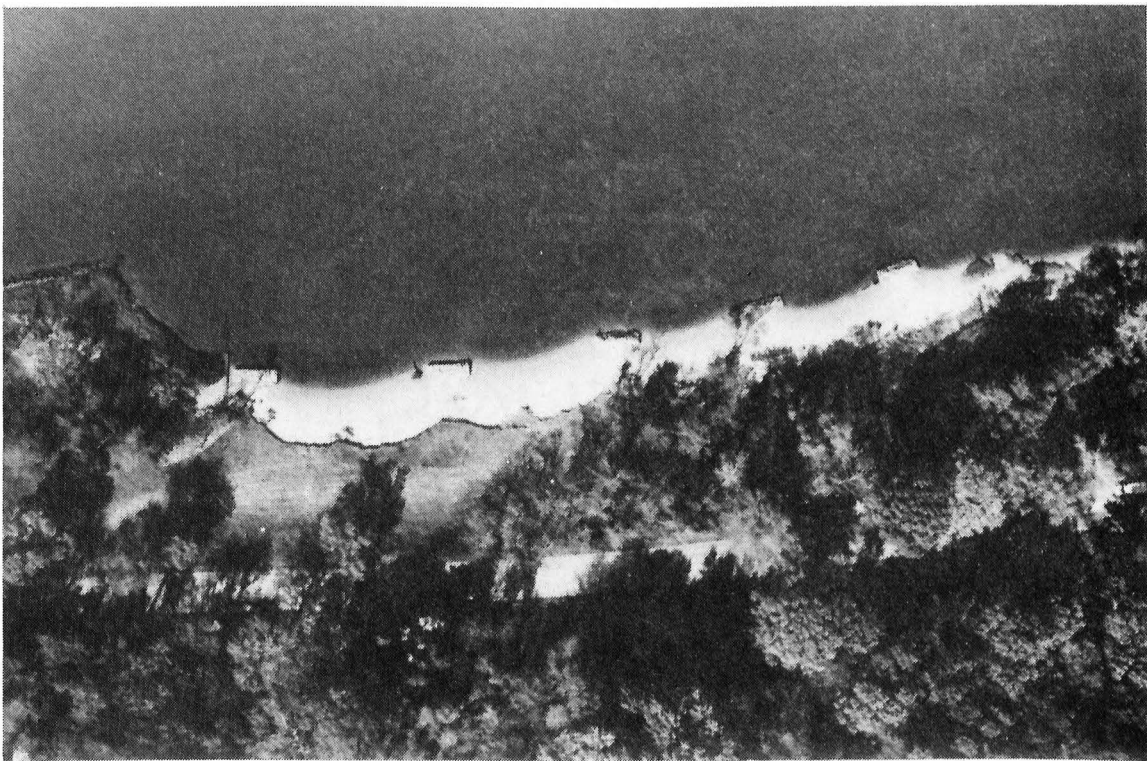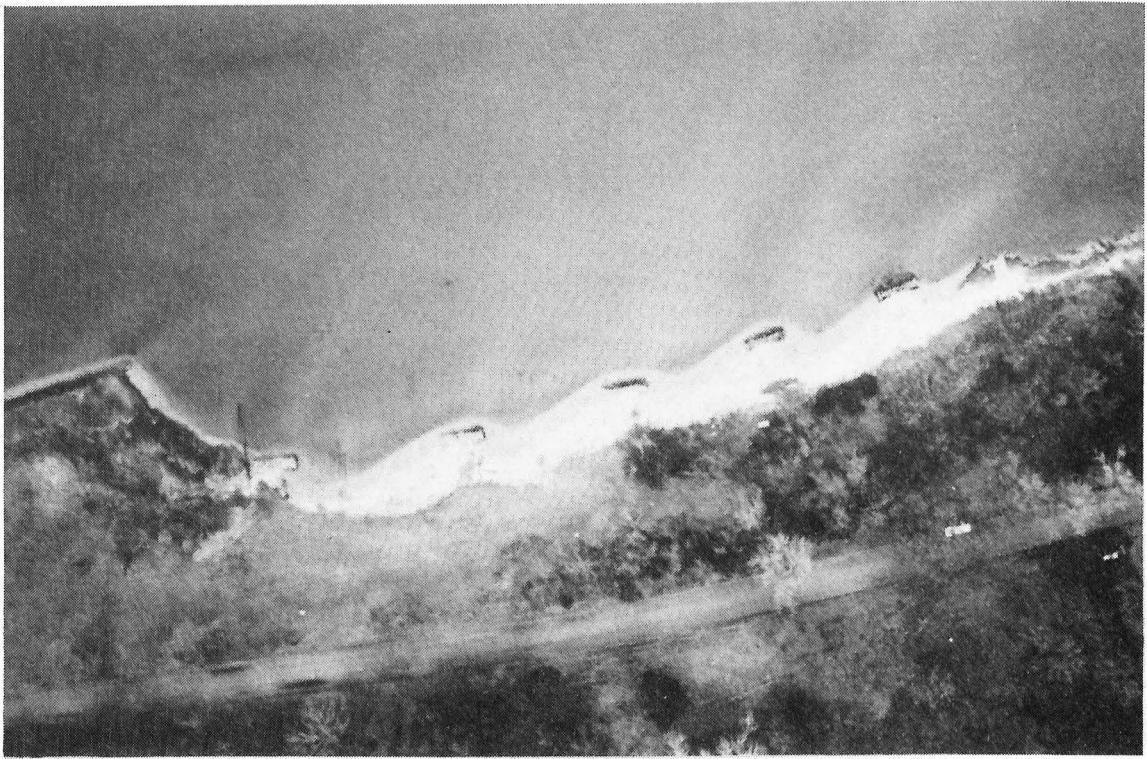
Figure 17. Parkway Breakwaters sediment analysis.
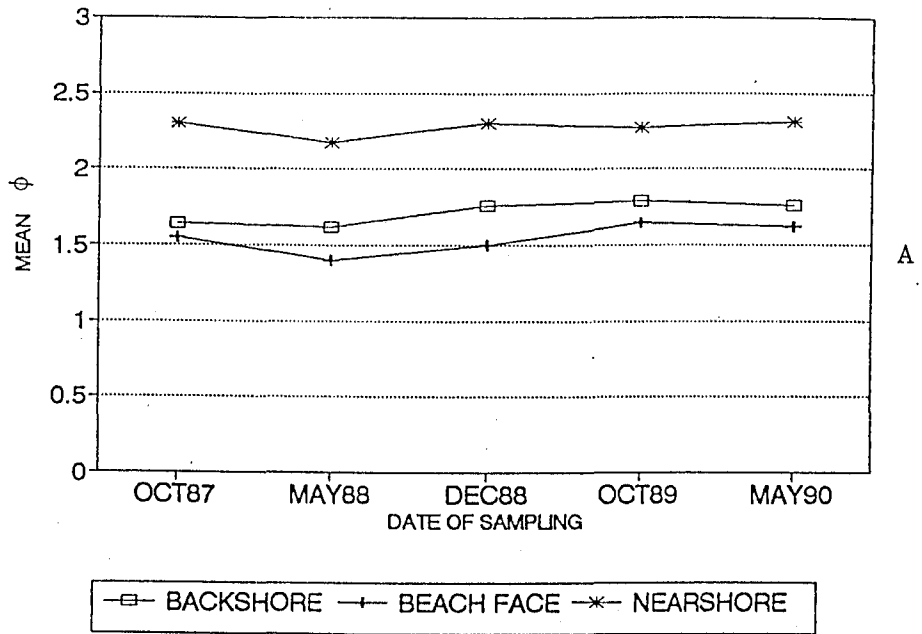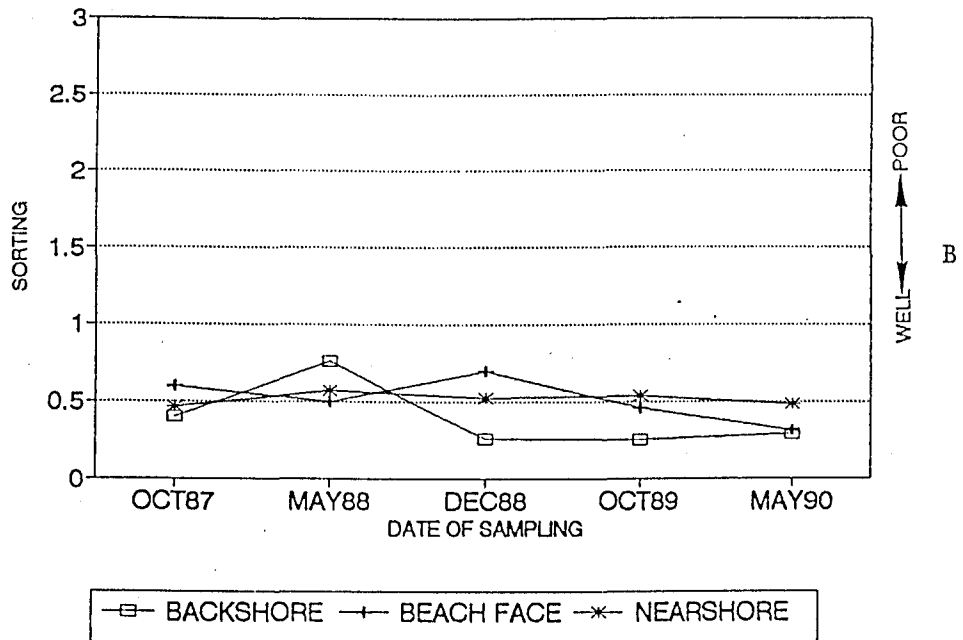
3. Hog Island Breakwaters, James River, Surry County

    a. <u>Setting</u>

The Hog Island breakwater system is located on the western shore of Hog Island on the James River (Figure 18). The site is located on Virginia's Department of Game and Inland Fisheries's Hog Island State Wildlife Management Area. The reach encompassing the Hog Island breakwaters extends from a small headland approximately 0.5 nautical miles north of Virginia Power's Surry Nuclear Power Plant's outfall northward approximately 0.7 nautical miles to a marsh headland. The baseline at the site is 1,275 feet long. The shore at the site faces west northwest with an average fetch of 2.7 nautical miles. The fastland bank is 3 to 11 feet above MSL and consists of mixed clayey sands and gravels in an earthen dike composed of dredged material from nearby channels. The access road around Hog Island was built on this dike. Management of the dike allows occasional flooding of the interior of the Hog Island Wildlife Management Area.

The historical erosion rate along this reach is approximately 1.7 feet per year (Byrne and Anderson, 1978). A gabion revetment and a single groin were built along this shore in 1962. Those structures have abated the erosion of the fastland bank at this site. However, the adjacent banks have continued to recede. Thus, the gabion revetment had become a small headland by 1982.

The gabion headland resulted in two types of shoreline configuration. To the south of the structure, there was a narrow beach (3 to 5 ft from the base of the bank to the MHW line) fronting the steep, wave-cut bank of the dredged disposal dike. The northern shoreline segment had a narrow beach fronting a low, wooded terrace which runs along the dike for 700 feet before the dike turns close to the river again. Net littoral sediment transport to the south is indicated by the impoundment of sand and wider beaches along the northern segment. An associated decrease in

beach width is observed for several hundred feet south of the gabion headland.

b.   Wave Climate

The average fetch at Hog Island breakwaters is approximately 2.7 nautical miles westward up the James River. After a northeast storm, the winds often shift quickly to the northwest and the storm surge will remain for a few hours. It is during this post-storm period that wave action will significantly affect westerly-facing shores such as Hog Island breakwaters. The average seasonal wave heights are 0.48 feet from the southwest, 0.46 feet from the west and 0.50 feet from the northwest (Figure 19).

c.   Design and Construction

The Hog Island breakwaters were installed in June 1987. The purpose of this project is to examine the effects of breakwaters of varying lengths, heights, and offshore distances. The use of salt marsh grass implantation for shore erosion control had been tested here in 1982 and 1983 without success. The grasses washed out during the winter storm seasons in both 1983 and 1984 (Hardaway et al., 1984).

Six pairs of breakwaters were built using two construction methods. The first method was installation of rubble mound breakwater units using a backhoe. The second method employed the use of gabion baskets, placed by hand, to hold rocks ranging from 50 to 150 pounds, each loaded by front-end loader and backhoe.

Six breakwater units were placed south of the gabion headland and six breakwater units were placed north (Figure 20). Breakwater units 1, 2, 5, 6, 9 and 10 were constructed of gabions. The remaining breakwaters (3, 4, 7, 8, 11 and 12) were constructed as rubble mounds.

Approximately 1,000 cubic yards of beach fill was placed along the northern section and 500 cubic yards on the southern section. The mean diameter of the fill was 0.4 mm (1.23 phi). The fill was truck hauled

from a borrow pit near Smithfield, Virginia. After installation of the breakwaters, the fastland banks were graded.

d. Shore Changes

The sand fill increased the beach width several feet. Immediately after installation, a cuspate spit began to form behind each breakwater unit (Figure 21A). In general, this occurred by wave action taking sand fill from the embayments and shifting it behind the breakwaters.

By March 1988, the position of the MHW line behind breakwater units 1, 2, 3, 4, 7 and 8 had receded and the MHW line behind breakwater units 5, 6, 9 and 10 had stabilized. The cuspate spits which fully attached (i.e. beach elevation above MHW) to become tombolos were behind breakwater units 11 and 12 (Figure 21B). Breakwater units 11 and 12 are higher and wider than the other structures and are more capable of holding and maintaining a tombolo. The general trend is for the cuspate spits and/or tombolo to become wider and higher as the breakwater becomes wider, higher and longer (Table 7).

The April 1988 northeaster produced little wave action at Hog Island breakwaters, mostly high water from the associated storm surge. The effect on the beach was to deflate in the embayments along the entire site. The low crested breakwater tombolos were also reduced (units 1, 2, 3, 4, 5, 6, 7, 8, 9 and 10), but the high crested breakwaters gained tombolo elevation (units 11 and 12). The principal wave action from the strong northwesters during the fall and winter usually affects only the intertidal beach. As previously mentioned, the upland banks erode during post-storm conditions when winds shift from northeast to northwest on top of the storm surge.

Base of bank erosion has occurred along the entire shoreline (Table 7). Higher rates are on the south section and along the the higher banks of the north section. The high banks are closer to MHW at these areas.

Some marsh plants were planted in the spring of 1989. The work was contracted out by the Virginia Division of Soil and Water. The plants

(Spartina alterniflora and Spartina patens) have taken hold behind each breakwater and appear to be thriving (Figure 21 C and D). The breakwater system has stabilized the beach and allowed the marsh grasses time to establish in an environment that proved to hostile before. Installing wave damping devices was a recommendation from the previous project (Hardaway et al. 1984).

d. Sediments

The beach fill material at the Hog Island breakwaters, shortly after being emplaced, was characterized as slightly silty, gravelly sands. Figures 22A, B, C, and D show the change in beach material size and sorting through time for profiles 26 and 57 (i.e. Bay E and Bay L). Increases in grain size along the beach face the fall of 1988 and 1989 may reflect a response to the dominant northwesters during those months.

Beach widths (B_m) are restricted in the most bay types due to the initial close proximity to the upland bank. There has been a general sand loss from the entire system but locally net gains in beach sand are noted at breakwaters 3, 4, 5, 7, 11 and 12 (Table 7). The value V_f indicates the net gain or loss after the initial beach fill. There is a general gain behind each breakwater unit and a corresponding loss from the embayments. Beach profiles for the duration of the project are found in Appendix A.
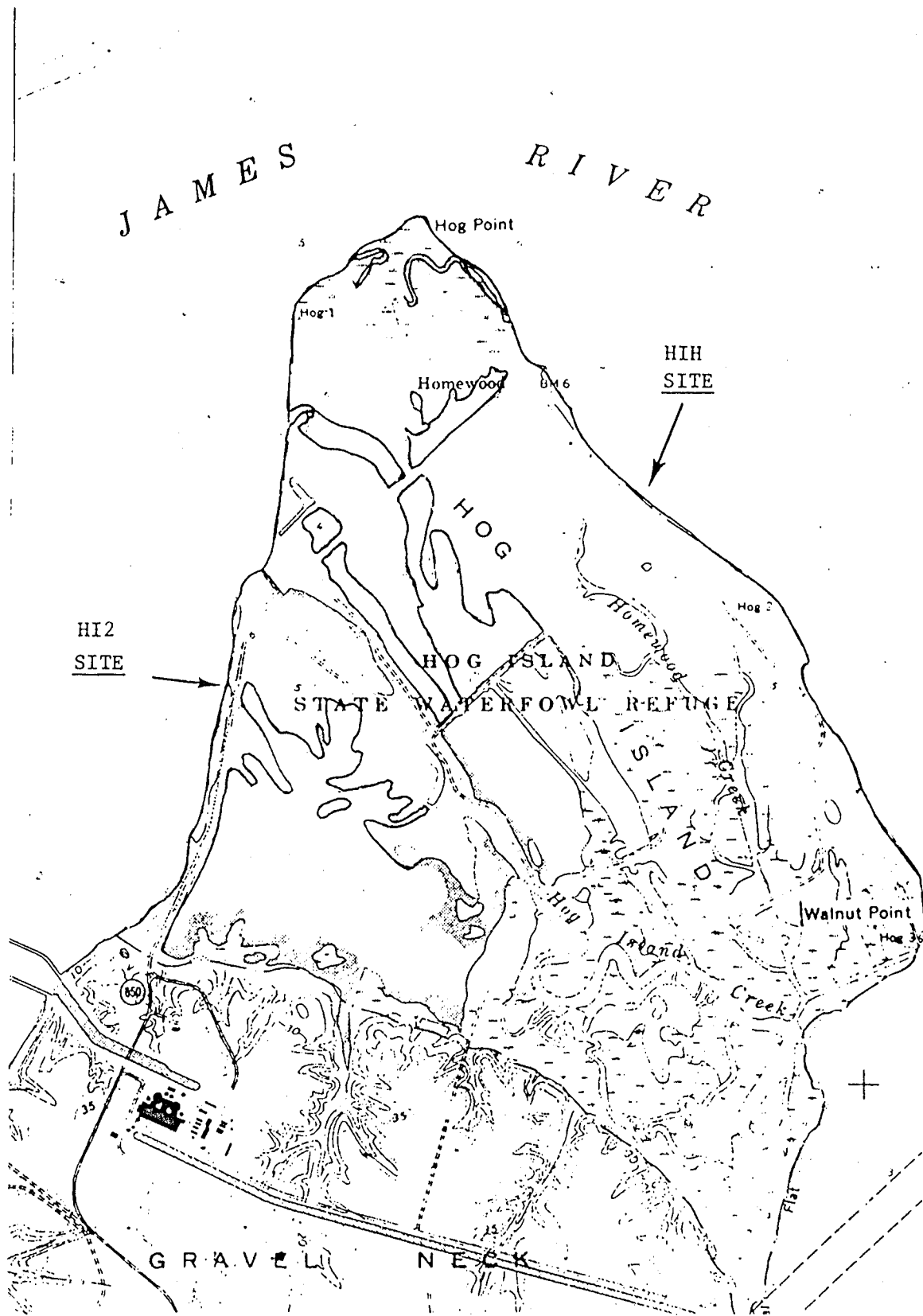
Figure 18. Hog Island Breakwaters and Headlands, James River, Surry County.
From Hog Island 7.5 minute quadrangle.
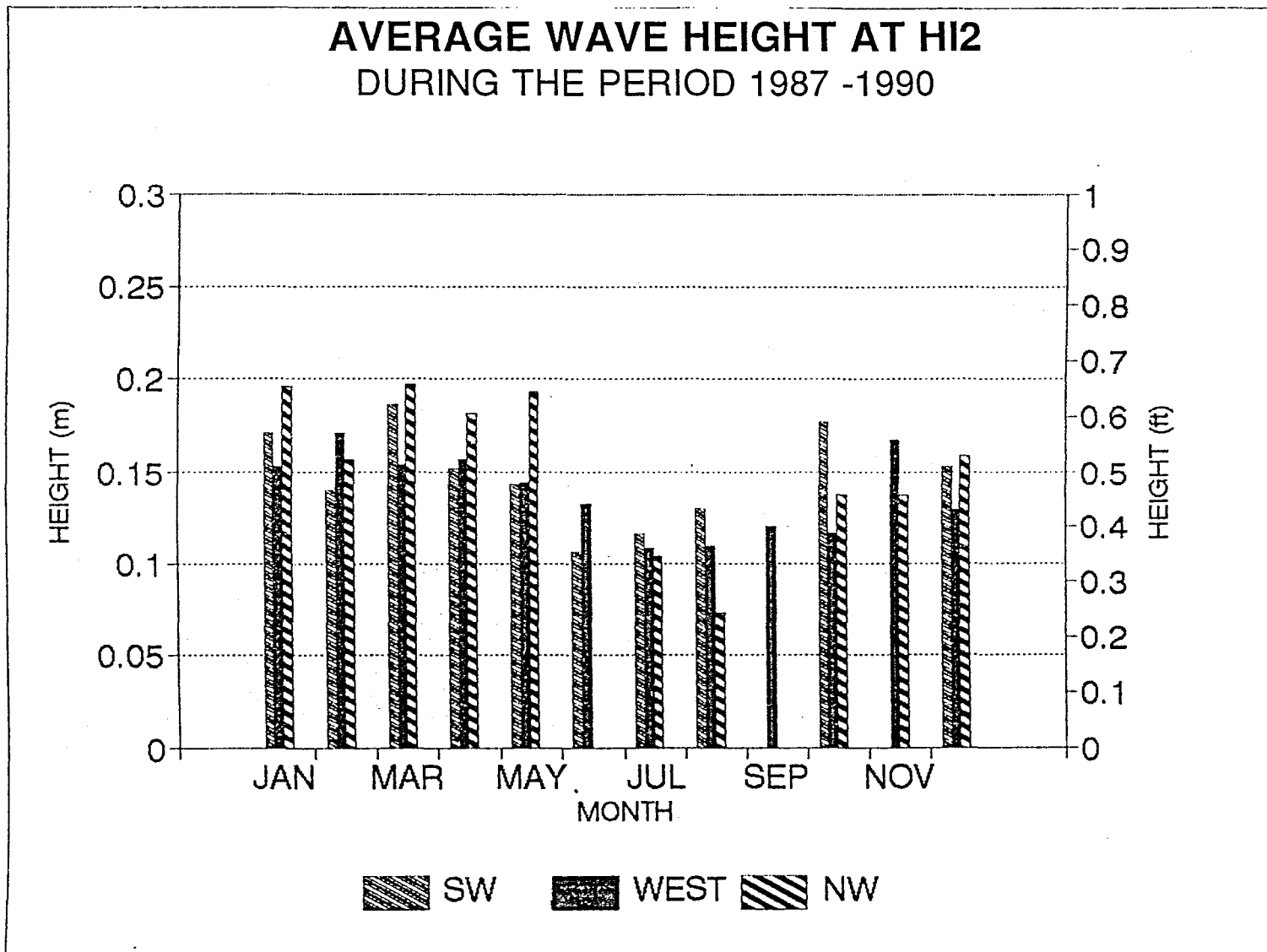Scale: 1 inch = 2,000 feet.

Figure 19. Hog Island Breakwaters wave heights.

FLOOD
EBB

MAY 87 MHW
MAY 90 MHW
MAY 90 TOE

JAMES RIVER

REFUGE ROAD

TREES

BASELINE

TREES

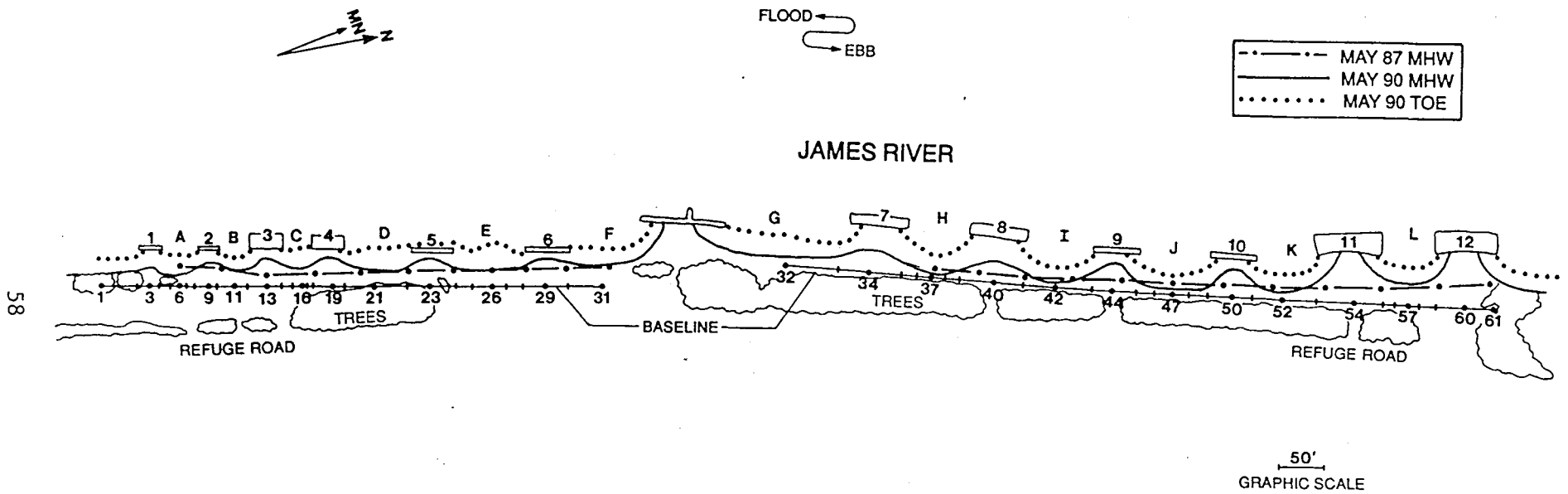REFUGE ROAD

50'
GRAPHIC SCALE

Figure 20.   Hog Island Breakwaters base map.

Figure 21A. Hog Island Breakwaters - vertical aerial, 10 September 1987.

Figure 21B. Hog Island Breakwaters - vertical aerial, 25 May 1990.

Figure 21C. Hog Island Breakwaters - ground view looking west from
breakwater number 5.

Figure 21D. Hog Island Breakwaters - ground view looking west from
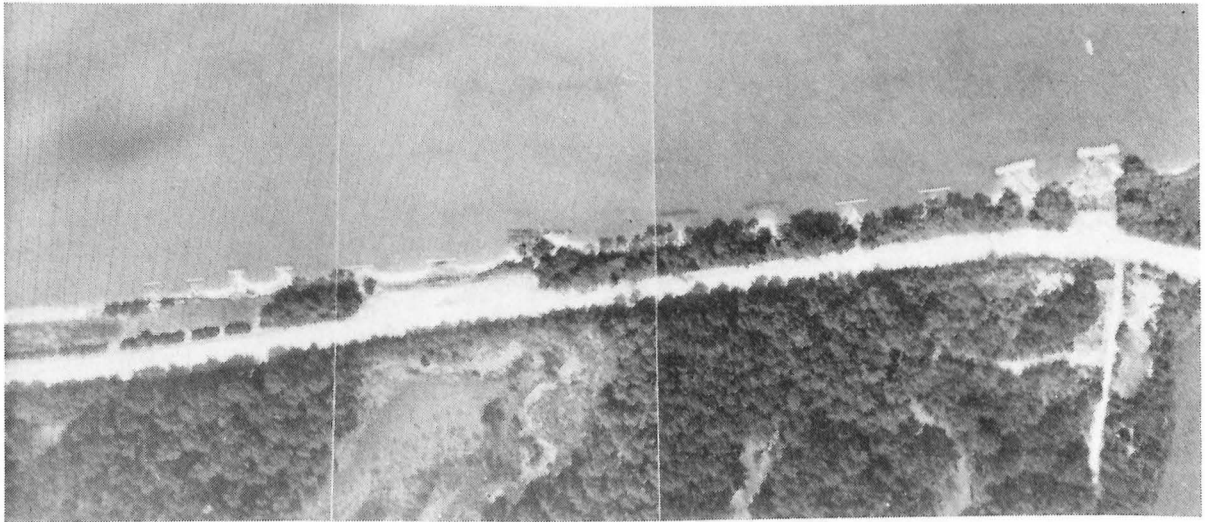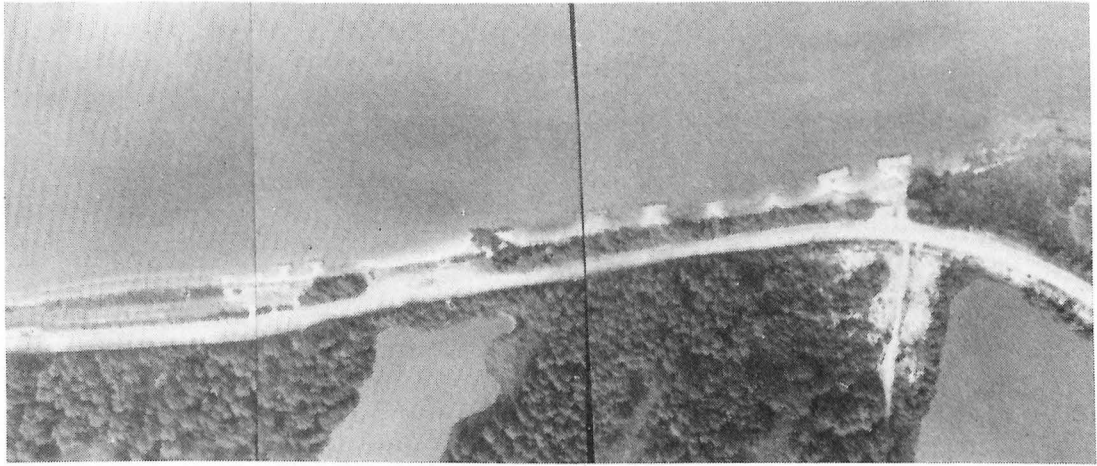breakwater number 12.

Table 7. Parameters for Hog Island Breakwaters*
May 1990 (See Figure 6 for Definition Sketch)

Breakwater/Bay	L_B	G_B	X_B	h_B	F_B	M_b	T_e	S_e	B_I	B_M	X_s	T_A	T_W	E_R	V_f
Downdrift								0.2	5	2				-3.3	-
Breakwater 1	24		17	2.0	1.0		-0.8	1.0	4		10	18		-2.2	0.1
Bay A		36				22		1.0	5	6				-1.9	-0.3
Breakwater 2	24		21	2.4	0.6		-1.5	1.3	4		5	13		-1.4	0.2
Bay B		36				28		1.1	5	8				-2.7	-0.1
Breakwater 3	25		32	3.5	0.5		-0.6	1.7	1		16	11		-2.2	0.9
Bay C		42				26		1.4	2	14				-1.5	0.7
Breakwater 4	25		32	3.5	0.5		-0.9	0.3	2		17	11		-1.6	0.9
Bay D		72				29		1.0	7	9				-0.2	0.5
Breakwater 5	48		20	2.2	0.8		-0.8	1.2	2		15	7		-1.3	0.5
Bay E		76				19		0.6	3	9				-1.9	-0.5
Breakwater 6	48		19	1.5	1.5		-0.4	2.1	3		10	7		-0.8	0.3
Bay F		70				29		0.6	4	5				-2.3	-0.6
Gabion Revetment	92												55		
Bay G		136				32		1.0	5	12				-1.3	0.5
Breakwater 7	50		45	3.5	-0.9		-2.9	1.2	10		18	26		-1.0	1.8
Bay H		77				48		1.2	7	6				-2.4	-0.9
Breakwater 8	50		43	3.8	-0.4		-1.4	1.4	10		20	23		-1.7	1.6
Bay I		75				42		0.8	8	4				-0.7	-0.2
Breakwater 9	48		30	3.3	1.2		-1.4	1.7	7		20	12		-0.5	0.8
Bay J		75				42		0.3	15	2				-1.4	-1.3
Breakwater 10	48		29	3.0	1.5		-1.3	1.5	12		20	13		-0.3	0.2
Bay K		70				43		0.6	5	7				-1.2	-0.4
Breakwater 11	50		46	3.4	1.8		0.2	2.2	17		42		25	-2.0	2.3
Bay L		75				43		1.0	3	8				-3.7	-0.4
Breakwater 12	50		45	3.6	1.8		0.3	2.6	5		?		30	-0.8	1.9
Updrift								1.6	5	16				-	-

*   All dimensions in feet except E_R and V_f.

L_B - Breakwater crest length

G_B - Breakwater gap

X_B - Distance offshore CL breakwater to original MHW

h_B - Height of breakwater from bottom at CL to MHW

F_B - Breakwater freeboard, MHW to crest

M_b - Maximum bay indentation, CL breakwater to MHW

T_e - Tombolo elevation in lee of breakwater $\pm$ MHW

S_e - Backshore elevation at base of bank

B_I - Initial beach width, base of bank to MHW

B_m - Present beach width, base of bank to MHW

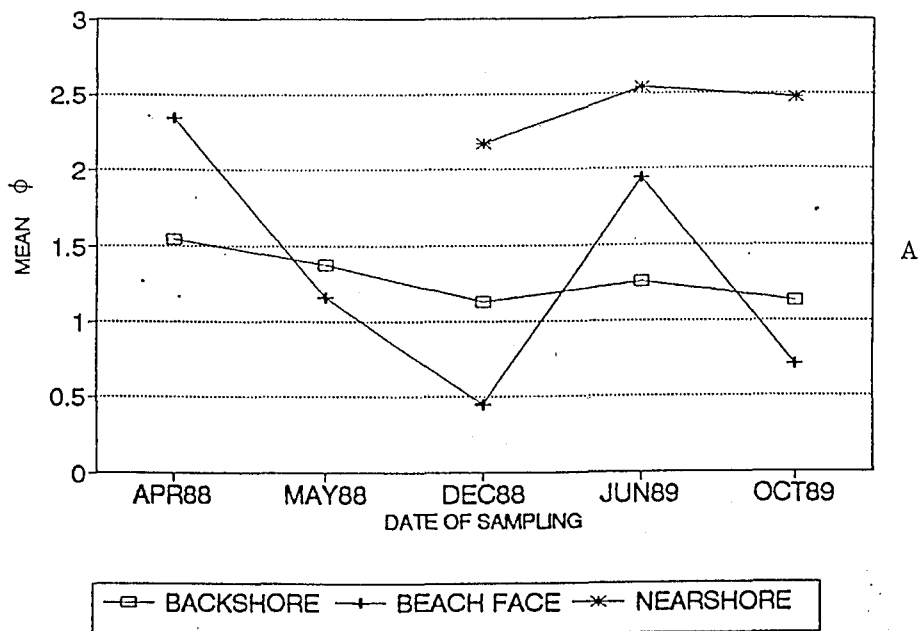X_s - Salient or tombolo length

T_A - For unattached tombolo, MHW to CL of breakwater

T_W - For attached tombolo, tombolo width at MHW

E_R - Erosion rate of BOB (ft/yr)

V_f - Net beach volume from 1988 - 1990 (cy/ft)

SEDIMENT ANALYSIS
HOG ISLAND BREAKWATERS, PROFILE 26

A

SEDIMENT ANALYSIS
HOG ISLAND BREAKWATERS, PROFILE 26
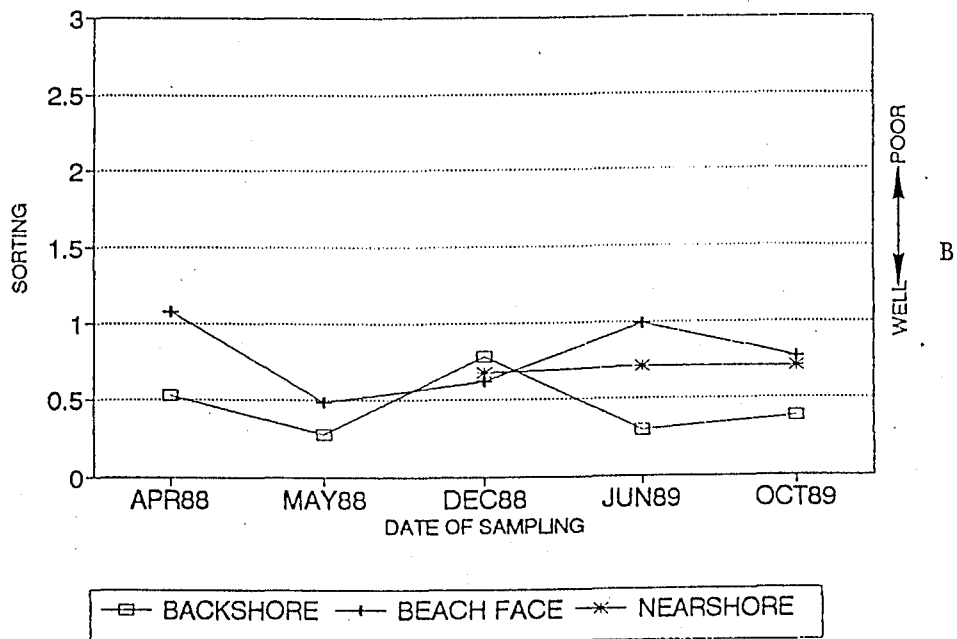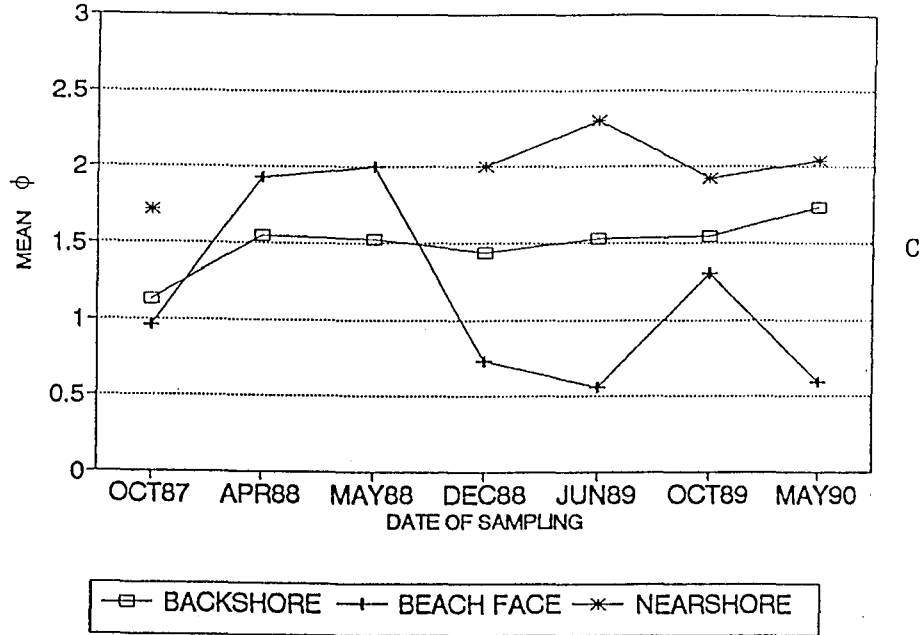
B

Figure 22. Hog Island Breakwaters sediment analysis.

SEDIMENT ANALYSIS
HOG ISLAND BREAKWATERS, PROFILE 57

C

SEDIMENT ANALYSIS
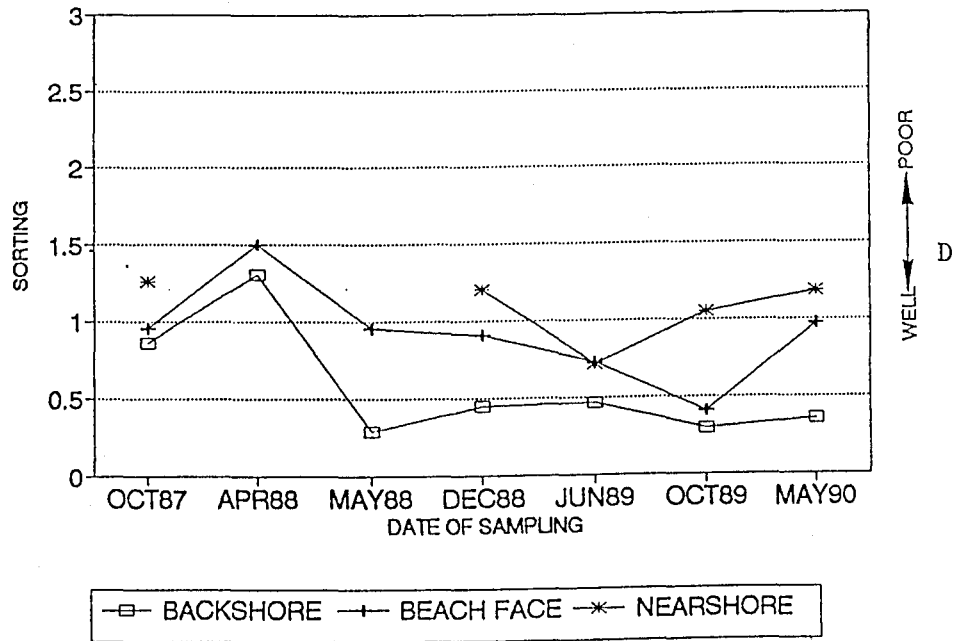### HOG ISLAND BREAKWATERS, PROFILE 57

D

Figure 22.   Hog Island Breakwaters sediment analysis.

4. Drummonds Field, James River, James City County

   a. Setting

Drummonds Field is a private development located on the north shore
of the James River just west of the Jamestown Ferry's pier (Figure 23).
The Drummonds Field development which was established in 1985, has
approximately 1,300 feet of river frontage and is situated in a reach
whose boundaries are defined by the boat basin at Lake Pasbehegh to the
northwest and the pier at the Jamestown Ferry to the southeast. The
historic shoreline erosion rate is approximately 1.6 feet per year (Byrne
and Anderson, 1978).

The 25-foot high bank at Drummonds Field is composed of a blue-grey,
very stiff clay overlain by a fine to medium sand. The clay layer is an
aquaclude causing intermittent springs to occur along the bank. Bank
erosion provided sand to the narrow beach which, in turn, provided little
or no buffer to wave action under storm conditions.

The Drummonds Field breakwater system was installed in September
1985. The purpose of the system was to provide a stable, protective and
recreational beach for this waterfront development.

   b. Wave Climate

There is an average fetch to the southwest of approximately 3.5
nautical miles. Frequent westerly winds dominate the Drummonds Field
shore (Figure 24). The mean wave height for the study period is 0.60,
0.59 and 0.54 feet for the wind directions from the south, southwest and
west.

   c. Design and Construction

In the initial design phases, the developers wanted to protect the
eroding banks and considered a bulkhead or revetment. However, such a
structure would preclude any natural accumulation of beach material since
it would cut off a major source of sand (i.e. the eroding fastland banks).
Because of a strong desire for a recreational beach, artificial
nourishment to form a defensive shoreline structure was considered. The

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retention of the fill was problematic and the annual renourishment cost estimates were high.

To accomplish the goals of a recreational beach and a protected bank, a gapped breakwater system was recommended. At the time in the Chesapeake Bay estuarine system, there were examples of breakwater systems at VIMS in Gloucester Point, Colonial Beach in Westmoreland County, and the aforementioned Parkway breakwaters in the Colonial National Historical Park in York County. These were used to compare cost and effectiveness.

A field and aerial photo investigation of Drummonds Field revealed a naturally occurring crenulate bay (Figure 25A). The two headlands which defined the bay were groups of cypress trees. The trees had reduced the erosion of the fastland and allowed the adjacent banks to evolve into a rough crenulate bay with a log-spiral section and a tangential section. From the tangential section, a net angle of wave approach of approximately 225° was determined.

An offshore breakwater was placed in front of each natural headland in order to reinforce the cypress tree headlands and, thus, stabilize the natural bay. Three more breakwaters were placed downriver at approximately the same spacing as the natural headlands and approximately 100 feet from MHW (Figure 25B). One small offshore breakwater was placed upriver. Along with these structures, over 10,000 cubic yards of beach fill was emplaced (Figure 26).

d. Shore Changes

The backshore elevation of the newly created beach was designed for protection from a storm surge of +4.5 MSL. Construction of Drummonds Field was still in progress on September 27, 1985, when Hurricane Gloria passed offshore. The effect on Drummonds Field was a 2-foot storm surge accompanied by northwest winds of 50 to 60 mph. Wave heights were measured at 1.5 to 2.0 feet. The breakwaters were in place, and enough beach fill was present, to prevent major damage to the fastland bank.

The post beach-fill planform left the MHW line approximately 50 feet behind breakwaters 1, 2 and 3. Breakwaters 4, 5 and 6 were semi-attached. Tombolo development proceeded with partial attachment by January 1986. An additional 3,000 cubic yards were added in April of 1986. This was placed mostly behind breakwaters 1, 2 and 3 and enhanced the tombolos. Because the upriver bank continued to erode, it was evident that breakwater 6 was too short. It was extended with a dog-leg addition approximately 80 feet long.

By March 1988, the tombolos at breakwaters 1, 2 and 3 were firmly attached at MSL but the MHW line was just landward of breakwaters 2 and 3 (Figure 26). Breakwaters 4, 5 and 6 were also attached at MSL with the MHW line several feet away. Also, it became necessary to abate bank erosion between breakwaters 5 and 6. Another adjustment to the system was made by adding an extension to breakwater 5 and placing several hundred cubic yards of beach fill in Bay E.

Due to the southwest exposure of Drummonds Field, the April 1988 northeaster had little effect except for high water from the storm surge. However, subsequent northwesters appeared to shift material from bay D to breakwater 4 (Hardaway et al., 1989).

Following an initial period of adjustment, the embayed beaches have remained fairly stable in terms of backshore elevation (S_e), beach width (B_m) and bay depth (M_b). The status of site parameters is shown in Table 8. Net volume at the site is difficult to interpret due to the intermittent additions of beach fill by the residents and frequent beach grading to remove vegetation. Thus, the general shore planforms show accretionary patterns. The shapes of Bays A, B, C and D are generally symmetrical, flattened and curvilinear with a slight log-spiral pocket on the upstream side.

   e. Sediments

Sediment samples analysis shows a sharp decrease in beach face sand size in April 1989 and a tendency to poorer sorting for Bay A (Figure 27A

and B). A sharp rise is beach grain size is seen on subsequent dates. Beach face sands in Bay D indicate a similar trend (Figure 27C and D). However, the affects of beach grading and beach filling negate the ability to properly evalutate those trends.

The bay beaches appear to fluctuate locally but maintain a generally stable planform through time (Figure 26C). Most of the beach fill is contained behind the breakwater units. The only measureable bank ersoion occure downdrift and at breakwater 6 and Bay D where backshore beach widths were less thn 20 feet until fill was added. Profiles for the project period are found in Appendix A.
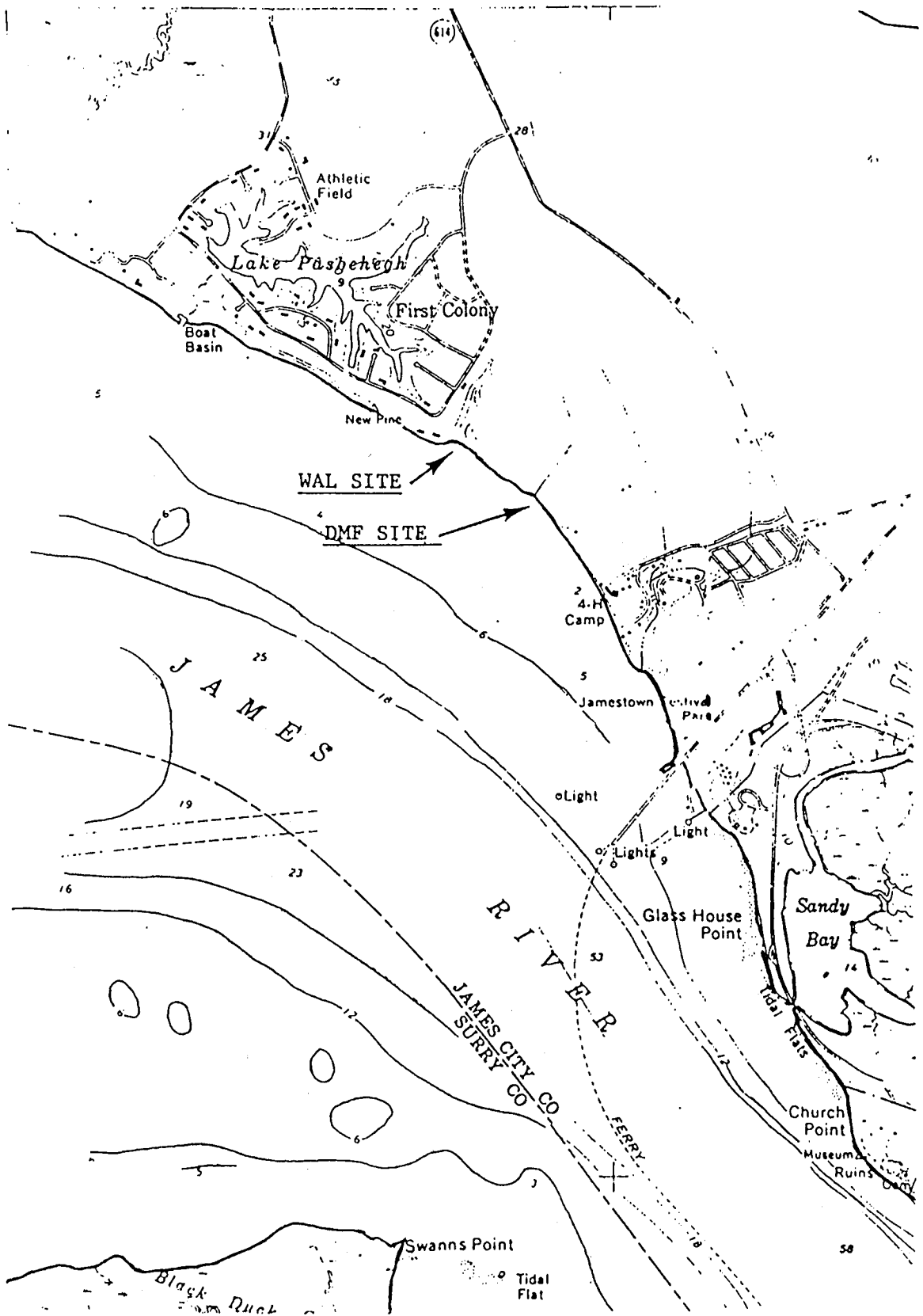
Figure 23. Drummonds Field and Waltrip, James River, James City County.
From Surry 7.5 minute quadrangle.
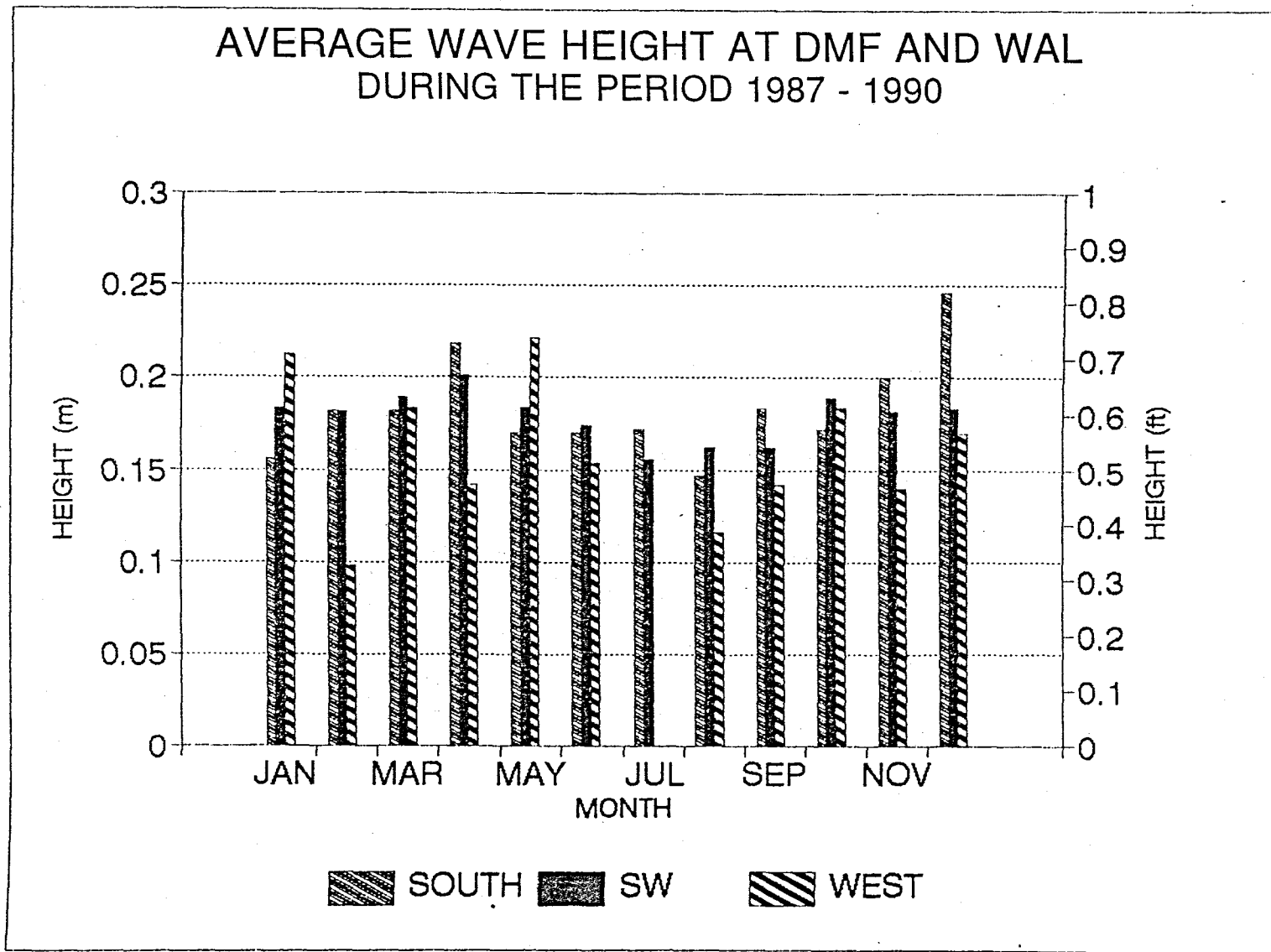Scale: 1 inch = 2,000 feet.
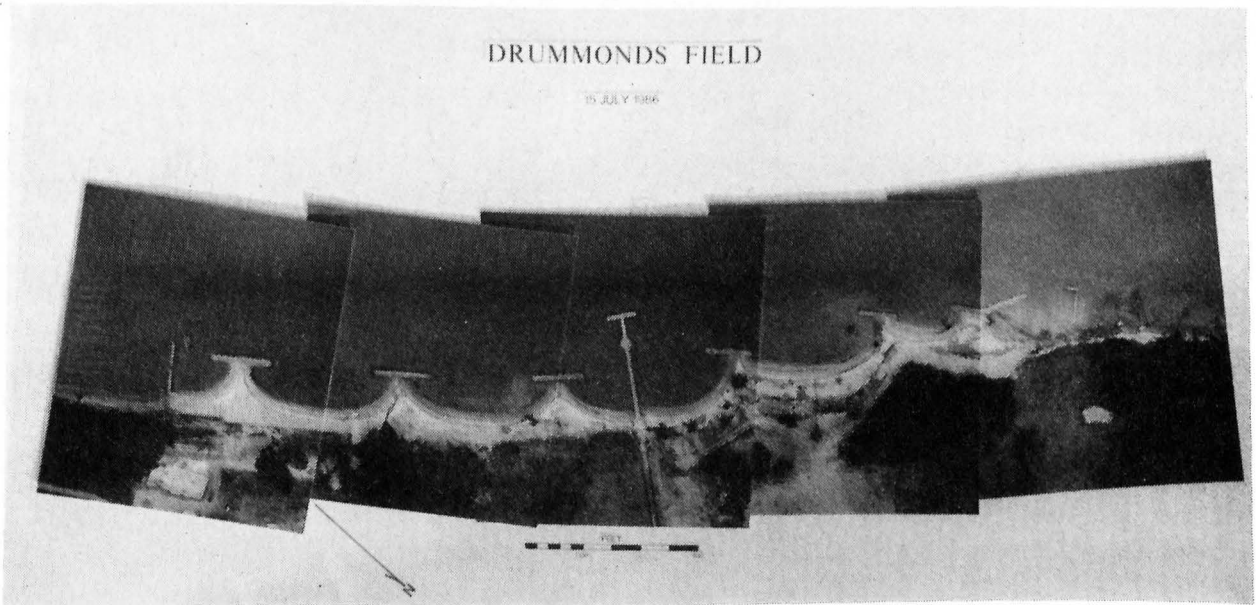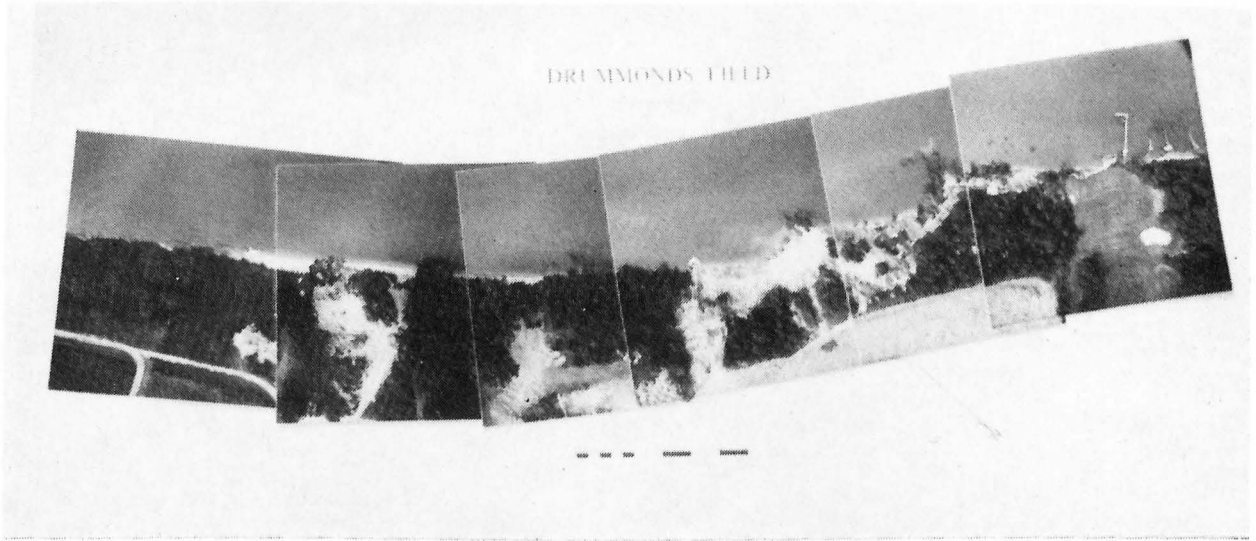
Figure 24. Drummonds Field and Waltrip wave heights.

69

Figure 25A. Drummonds Field - vertical aerial, 11 September 1985.

Figure 25B. Drummonds Field - vertical aerial, 15 July 1986.

DRUMMONDS FIELD

DRUMMONDS FIELD

15 JULY 1966

EBB

FLOOD

JAMES RIVER

MN

N

	SEPT 85 MHW
	MAR 88 MHW
	MAY 90 MHW
	MAY 90 TOE

1

A

2

B

3

C

4

D

5

E

6

F

BASELINE

1   4   8   11   14   18   22   25   27   29/30 31   35
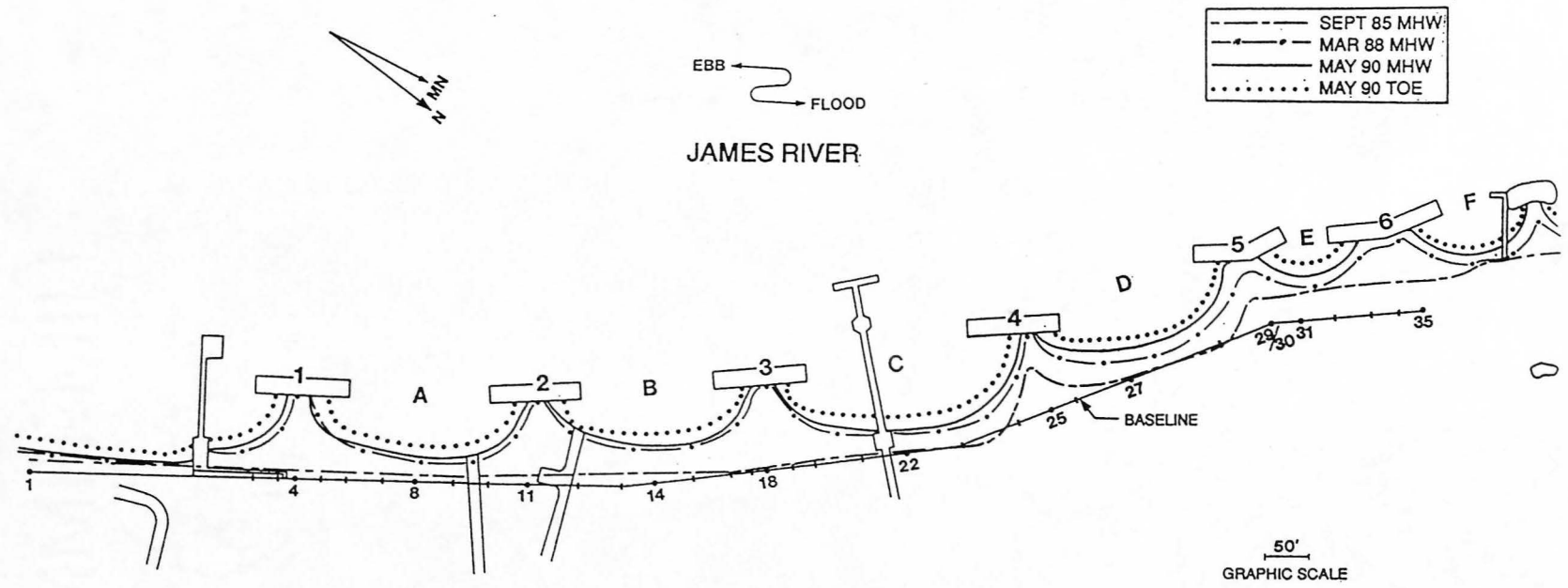
50'
GRAPHIC SCALE

Figure 26.   Drummonds Field base map.

Table 8. Parameters for Drummonds Field*
May 1990 (See Figure 6 for Definition Sketch)

Breakwater/Bay	L_B	G_B	X_B	h_B	F_B	M_b	T_e	S_e	B_I	B_M	X_s	T_A	T_W	E_R	V_f
Downdrift								1.8	10	5				-0.8	-0.1
Breakwater 1	90		100	2.0	2.3		0.5	4.5	5		85		20	0.0	8.8
Bay A		180				74		4.0	5	29				0.0	3.0
Breakwater 2	90		95	2.7	2.0		-0.4	3.3	10		90	8		0.0	10.2
Bay B		180				65		4.2	5	43				0.0	9.5
Breakwater 3	90		105	2.4	2.0		-0.2	4.3	5		102		10	0.0	9.7
Bay C		220				97		2.4	10	28				0.0	-
Breakwater 4	80		45	2.2	2.0		0.2	2.3	0		40		16	-0.7	-
Bay D		200				61		3.6	10	34				-0.9	-
Breakwater 5	100		50	2.2	1.7		-0.4	2.9	0		45		11	0.0	-
Bay E		60				44		3.8	5	20				0.0	-
Breakwater 6	120		55	2.1	2.2		0.5	3.5	10		50		50	0.0	-

* All dimensions in feet except E_R and V_f.

L_B - Breakwater crest length

G_B - Breakwater gap

X_B - Distance offshore CL breakwater to original MHW

h_B - Height of breakwater from bottom at CL to MHW

F_B - Breakwater freeboard, MHW to crest

M_b - Maximum bay indentation, CL breakwater to MHW

T_e - Tombolo elevation in lee of breakwater ± MHW

S_e - Backshore elevation at base of bank

B_I - Initial beach width, base of bank to MHW

B_m - Present beach width, base of bank to MHW
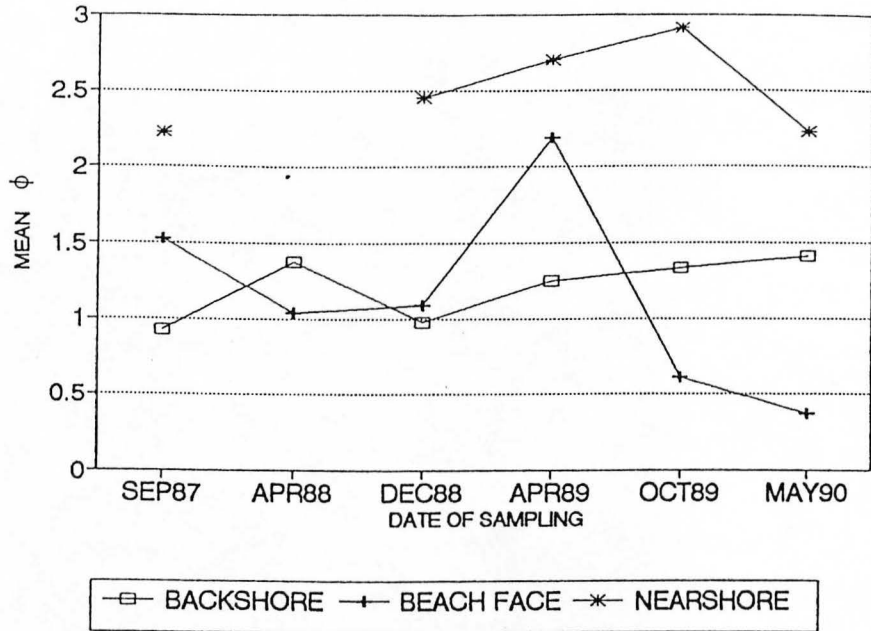
X_s - Salient or tombolo length

T_A - For unattached tombolo, MHW to CL of breakwater

T_W - For attached tombolo, tombolo width at MHW
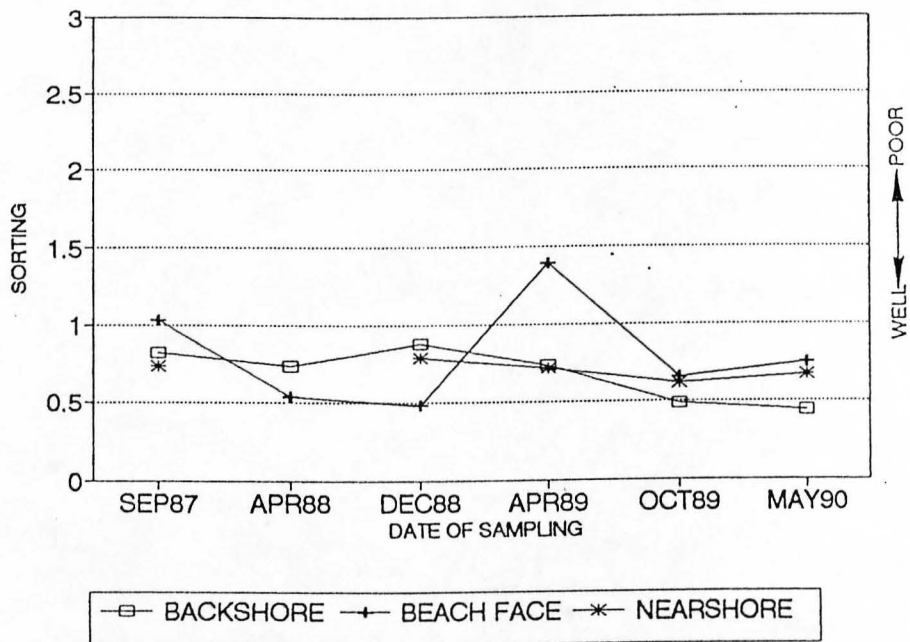
E_R - Erosion rate of BOB (ft/yr)

V_f - Net beach volume from 1985 - 1990 (cy/ft)

Figure 27.   Drummonds Field sediment analysis.
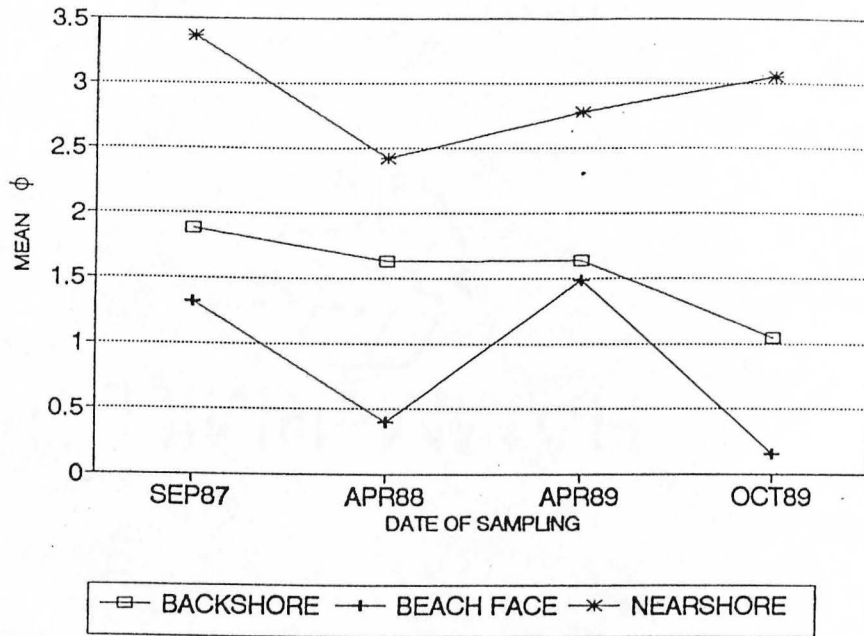
SEDIMENT ANALYSIS
DRUMMONDS FIELD, PROFILE 27

C

BACKSHORE   BEACH FACE   NEARSHORE

SEDIMENT ANALYSIS
DRUMMONDS FIELD, PROFILE 27

D

BACKSHORE   BEACH FACE   NEARSHORE

Figure 27.   Drummonds Field sediment analysis.

5. Waltrip, James River, James City County

   a. Setting

The Waltrip breakwaters essentially are an upriver extension of the Drummonds Field breakwater system (Figure 23). Waltrip has the same bank type, fetch and shore orientation as Drummonds Field. Three rock breakwaters were installed in October 1987 along with about 3,000 cubic yards of beach fill. At the same time a rock extension was built onto breakwater number 6 at Drummonds Field. The upland bank was then graded. Also, a low rock groin was built on the upriver breakwater unit to keep the beach fill out of the small adjacent wetland area.

   b. Wave Climate

The Waltrip site is exposed to the same general wave climate as Drummonds Field (Figure 24). The shore normal wave approach is reflected in the symmetrical planforms of the embayments.

   c. Design and Construction

The Waltrip breakwater system was designed to protect the remainder of the eroding high banks along the reach upriver of the Drummond Field project. It was evident from earlier installations in this project that higher breakwaters and a higher, broader backshore would offer greater protection of the fastland banks during storm events. The result was a breakwater system composed of three rubble mound units which have crest lengths of 50 feet, crest elevations of 3.0 feet above MHW and gaps of 75 feet (Figure 28). These are similar in design to the breakwaters at Chippokes in terms of breakwater length and gaps. However, the Waltrip breakwaters were placed about 100 feet from the original MHW line which provided enough area to develop deep pocket beaches and a broader backshore (Figure 29A). The backshore elevation was set at 4.5 feet above MHW. The construction roads which were built to each breakwater were left as attached tombolos.

d. <u>Shore Changes</u>

There were no pre-construction profiles established at Waltrip. The site was included in this project in August 1988 and monitoring began in September 1988. After one year, a decrease in tomoblo elevation was noted behind breakwater 2. This corresponds to a net sand volume loss in the lee of the same structure. A slight narrowing of the tombolos behind each breakwater unit was noted, as well as a landward shift of MHW in Bay B and Bay F (in the adjacent Drummonds Field breakwater system) (Figure 28).

Initial adjustments to the beach planforms occurred between the fall of 1987 and the fall of 1988. Since that time, slight changes have occurred through time to bay and breakwater parameters. Table 9 shows the site parameters for the Waltrip breakwater system. There has been no bank erosion since the system was installed.

e. <u>Sediments</u>

Sediments are similar in size and sorting across the beach and nearshore through time (Figure 30). A noted decrease in backshore grain size appears to be related to the addition of finer wind blown material. Beach planforms have remained symmetrical and semi-circular through time. Relatively narrow gaps and a shore normal wave climate have accounted for this. Profiles for Waltrip are found in Appendix A.

JAMES RIVER

EBB

FLOOD

SEPT 88 MHW
MAR 89 MHW
APR 90 MHW
APR 90 TO E

F   1   A   2   B   3

BASELINE

1   2   4   7   11   14   16   17

BASE OF
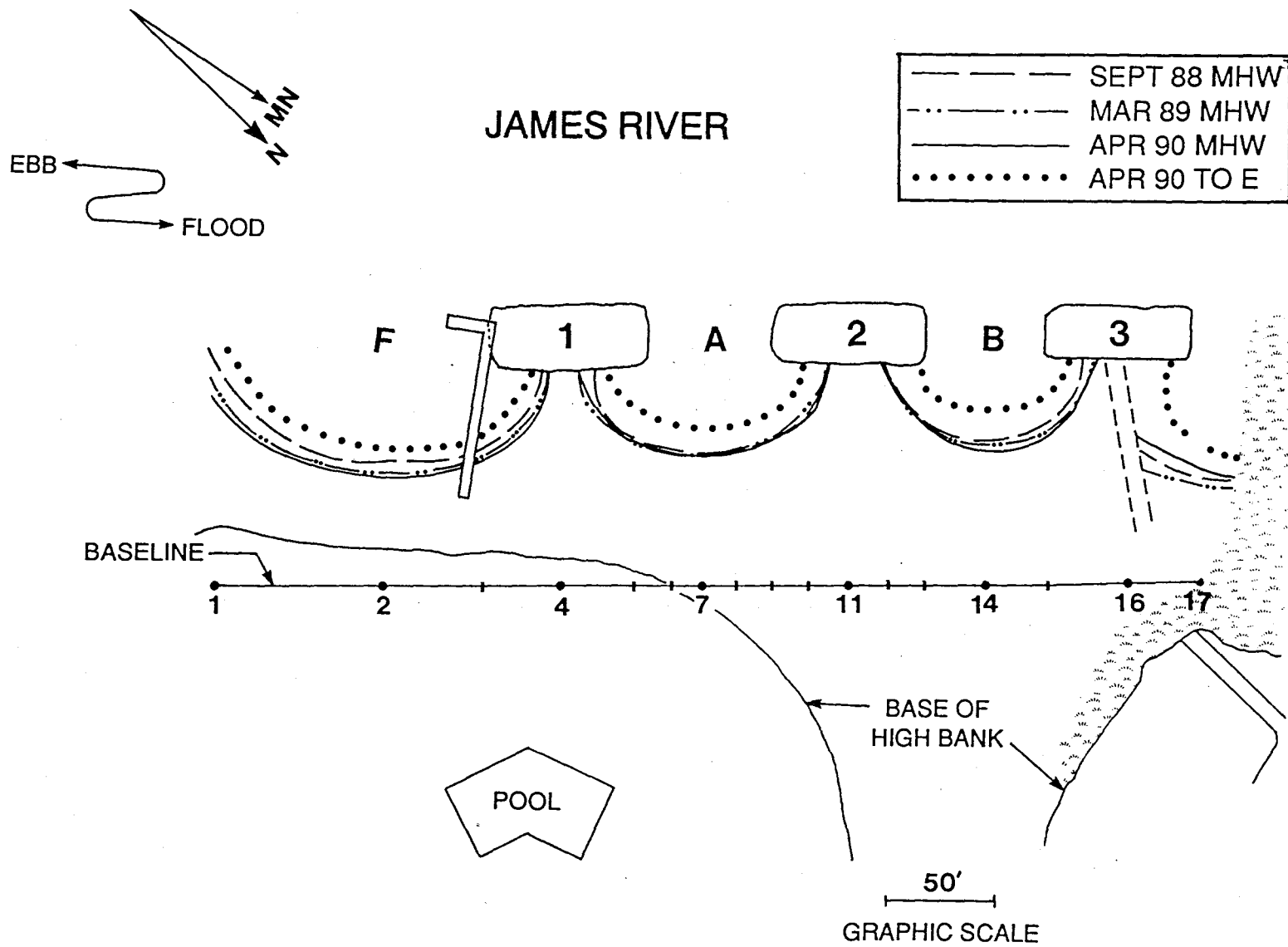HIGH BANK

POOL

50'

GRAPHIC SCALE

Figure 28.   Waltrip base map.

Figure 29A. Waltrip - vertical aerial, 25 May 1990.
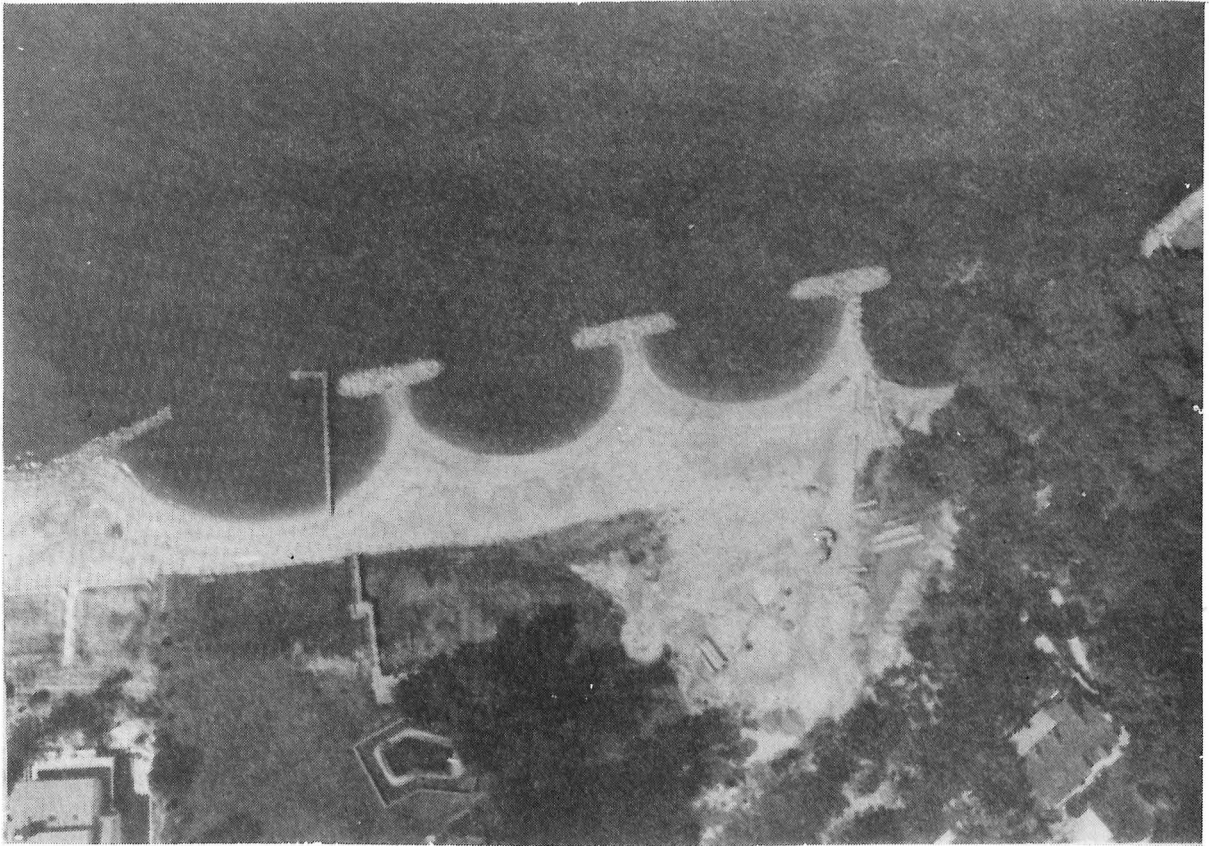
Figure 29B. Waltrip - ground view, 27 September 1988.

Table 9. Parameters for Waltrip*
May 1990 (See Figure 6 for Definition Sketch)

Breakwater/Bay	L_B	G_B	X_B	h_B	F_B	M_b	T_e	S_e	B_I	B_M	X_s	T_A	T_W	E_R	V_f
Updrift								4.6	30	34		-		0.0	2.7
Breakwater 1	50		84	2.4	3.1		1.4	5.1	32		65	-	16	0.0	6.2
Bay A		75				53		5.3	31	50		-		0.0	2.5
Breakwater 2	50		92	2.4	3.6		1.3	4.9	29		68	-	26	0.0	6.1
Bay B		70				47		5.1	30	54		-		0.0	2.4
Breakwater 3	50		95	2.3	3.7		1.0	2.8	27		70	-	15	0.0	6.1

* All dimensions in feet except E_R and V_f.

L_B - Breakwater crest length

G_B - Breakwater gap

X_B - Distance offshore CL breakwater to original MHW

h_B - Height of breakwater from bottom at CL to MHW

F_B - Breakwater freeboard, MHW to crest

M_b - Maximum bay indentation, CL breakwater to MHW

T_e - Tombolo elevation in lee of breakwater ± MHW

S_e - Backshore elevation at base of bank

B_I - Initial beach width, base of bank to MHW

B_m - Present beach width, base of bank to MHW
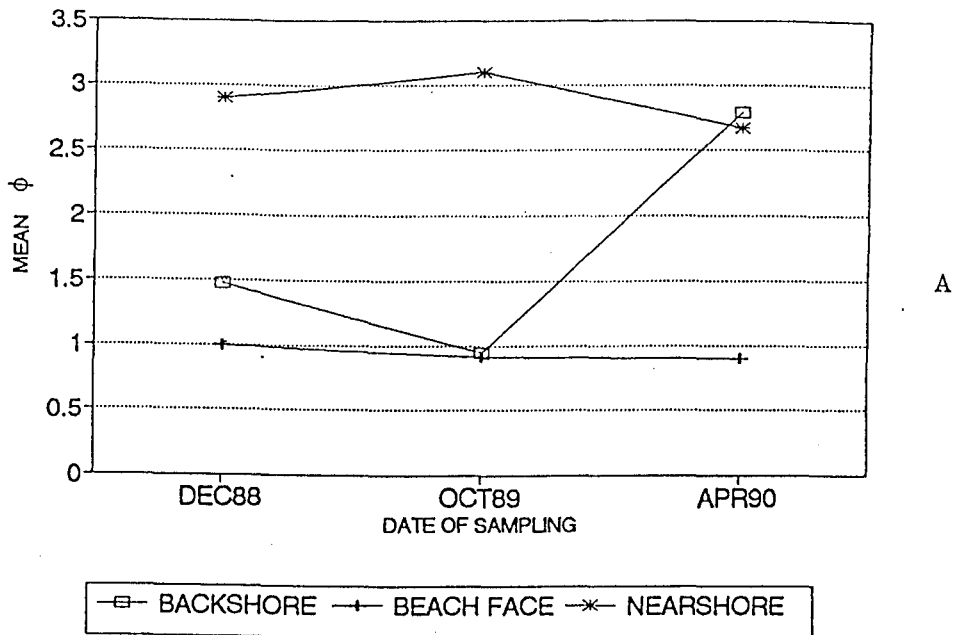
X_s - Salient or tombolo length

T_A - For unattached tombolo, MHW to CL of breakwater

T_W - For attached tombolo, tombolo width at MHW

E_R - Erosion rate of BOB (ft/yr)

V_f - Net beach volume from 1988 - 1990 (cy/ft)

SEDIMENT ANALYSIS
WALTRIP, PROFILE 7

A

SEDIMENT ANALYSIS
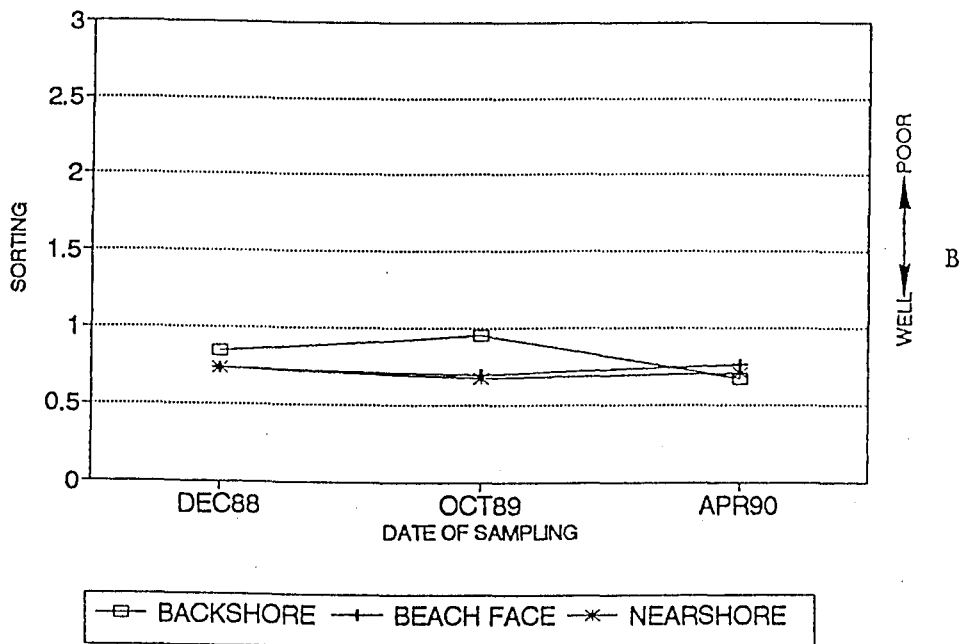## WALTRIP, PROFILE 7

B

Figure 30. Waltrip sediment analysis.

80

B.   Headland Sites

6.   Hog Island Headlands, James River, Surry County

a.   Setting

Three rock headlands (breakwaters) were built on the northeast shore of Hog Island in October 1987. The Hog Island headlands site is situated within a long, shallow embayment between Hog Point and Walnut Point (Figure 18). The shoreline along the embayment is curvilinear and generally faces northeast. The historical erosion rate is 2.5 feet per year (Byrne and Anderson, 1978). The bank along the northern section is approximately 10 feet high and is composed of dredged material. As one proceeds southward, the bank's elevation decreases and becomes a low (3 ft) clayey fastland. At the southern end of the site there is a marsh fringe which acts as a low, erosion-resistant headland.

In the early 1960s, large concrete blocks were placed along 150 feet of the shoreline at MLW on the north end of the site (Figure 31A). There was a small, erosion-resistant bank midway between the blocks and the marsh headland. These features segmented the shore into two shallow embayments. Historical aerial photography shows very slight changes in shore orientation and there are no significant offsets often caused by oblique angles of wave approach. Thus a general, shore normal, long term wave approach was indicated from the shore morphology.

Before construction, the initial beach width from the MHW line to the base of the bank varied from 0 to 20 feet and the sand layer at MHW was approximately 1 foot thick. The beach is composed of medium to coarse sand overlying stiff brown clay. Another erosion resistant, clayey bank occurs just before the downriver marsh headland. This clay bank is a small headland and marks the downriver end of the second bay.

b.   Wave Climate

The average fetch along the Hog Island headlands shore is 2.5 nautical miles. The site is exposed to winds from the north northwest to the southeast. The shoreline is oriented normal to the northeast, which

81

supports the theory of a dominant shore normal wave approach. The average

wave heights for the study period are 0.49, 0.53 and 0.51 feet

respectively for winds from the north, norhteast and east (Figure 32).

c.   Design and Construction

The design of Hog Island headlands was based on the geomorphic

expression of the shore. The existing protuberances caused by the

concrete blocks and erosion resistant banks were designated points to

construct rock headlands. The system consists of three rubble-mound

breakwaters (Figure 33). Breakwater 1 has a 150-foot crest length, a 4-

foot crest width and is 3.5 feet above MHW. Breakwaters 2 and 3 have 100-

foot crest lengths, 4-foot crest widths and are 2.0 feet above MHW.

Breakwater 1 was designed with larger dimensions to provide greater

protection for the high bank and nearby service road. Breakwater 1 was

placed at -1.0 MLW, while breakwaters 2 and 3 were placed at 0.0 MLW.

Approximately 2,400 cubic yards of fill was placed on Hog Island

headlands. This came from the same pit in Smithfield where fill was

obtained for the Hog Island breakwaters. The bank behind breakwater 1 was

graded and approximately 1,200 cubic yards of fill was emplaced.

Approximately 600 cubic yards were placed behind each of breakwaters 2 and

3. The fill was put behind each structure as a fully attached tombolo

(Figure 31B). The new beach fill was placed at an elevation of about 3.0

feet above MHW at the backshore. Lawn grasses (Kentucky 31 and rye) were

then planted on the tombolos.

d.   Shore Changes

The most noticeable changes to the shore planform at Hog Island

headlands were to the sides of the tombolos where some beach fill has been

lost. The area in the lee of each structure was almost filled to

capacity. However, there has been an net increase in beach material to

the system since the initial installation (Table 10). This has most

likely been from onshore transport and some bank erosion around breakwater

1.

The April 1988 storm completely flooded the tombolos behind breakwaters 2 and 3, but only partially covered the river side of the tombolo behind breakwater 1 (Figure 31D). The low bank downriver of profile 10 was flooded back to the high bank by the service road. The intertidal beach was flattened along the embayed shorelines but subsequently recovered. Little sand appeared to be lost offshore, but as of March 1989, sand began shifting downriver around the outside of each headland. This is probably due to the lack of strong northeast winds during the winter of 1989. The beach was more under the influence of north and northwest wind conditions (U.S. Dept. of Commerce, 1989). The generally very flattened, shallow, symmetrical bays attained a slight log-spiral component against the downriver sides of each tombolo. The status of site parameters for Hog Island headlands is shown in Table 10.

e. Sediments

The sediments in Bay A show general coarsening through the project period until May 90 when they became finer (Figure 34A). Bay B showed a similar trend but the beach sands became coarser by May 90 (Figure 34C). Profile changes for the project period are found in Appendix A.

Figure 31A.   Hog Island Headlands – vertical aerial, 18 June 1987.

Figure 31B.   Hog Island Headlands – vertical aerial, 19 December 1988.
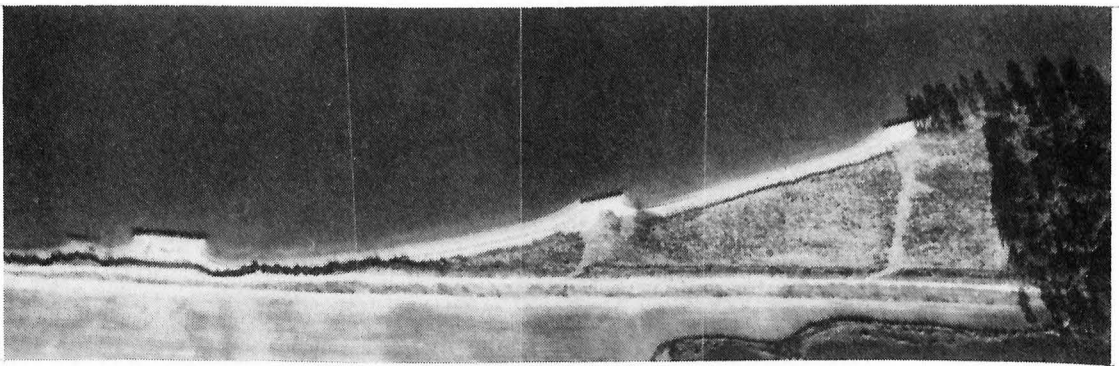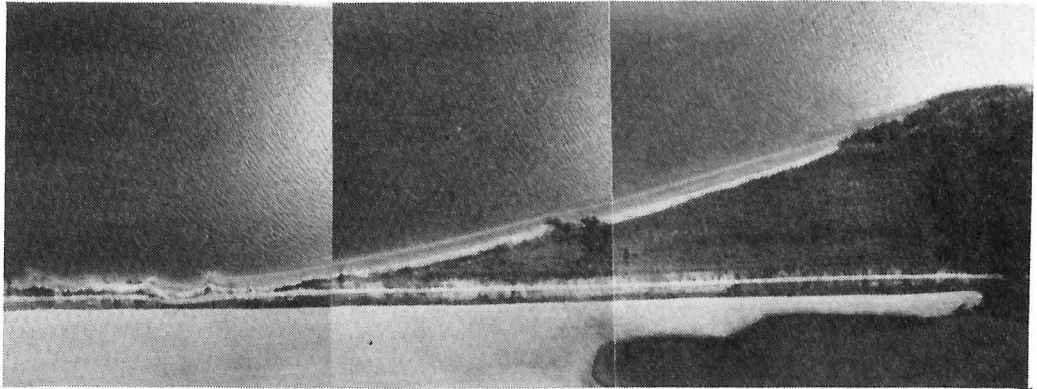
Figure 31C.   Hog Island Headlands - ground view looking southeast, 14 June
              1990.

Figure 31D.   Hog Island Headlands - ground view looking southeast during
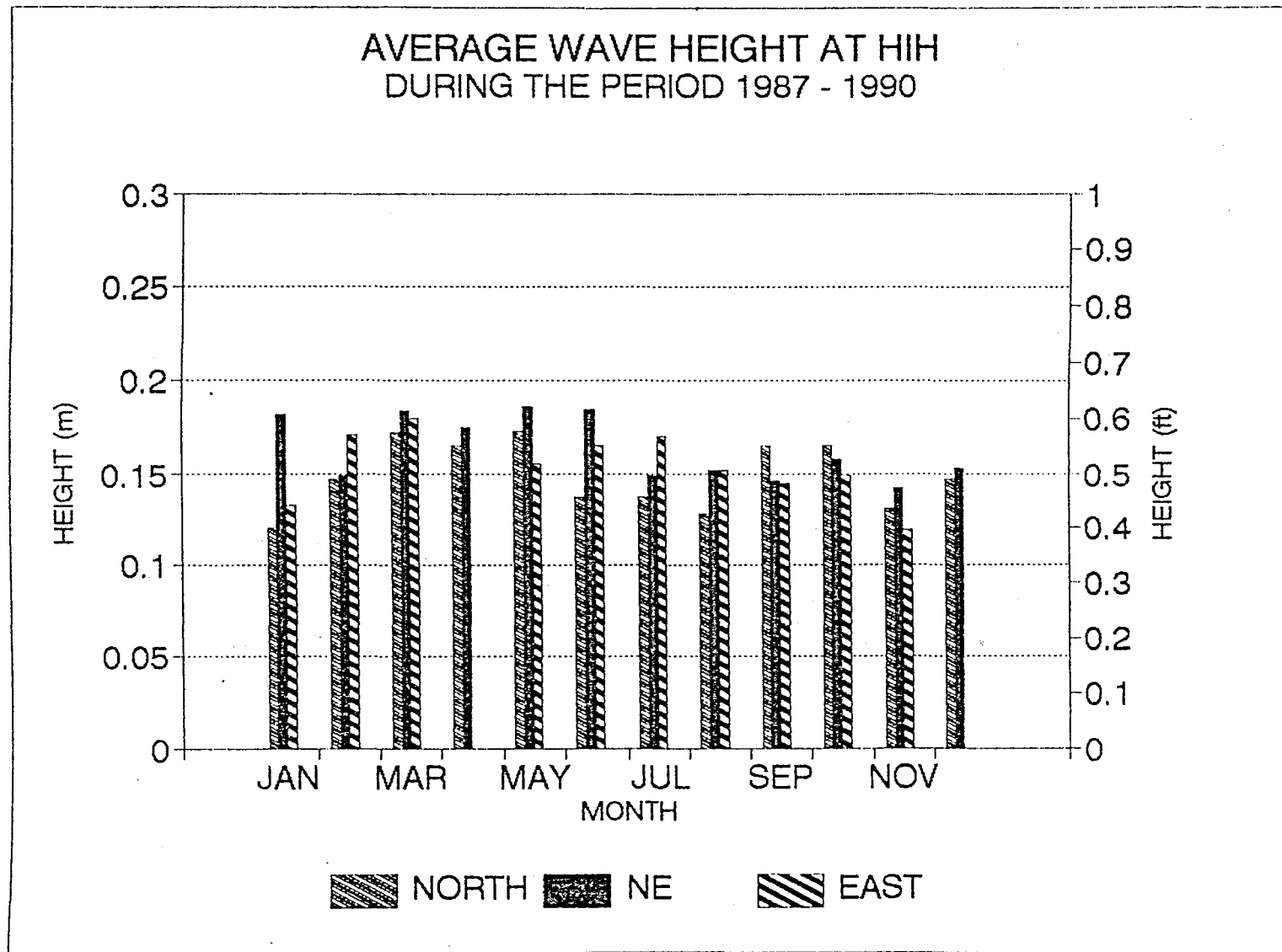   13 April 1988 northeaster.
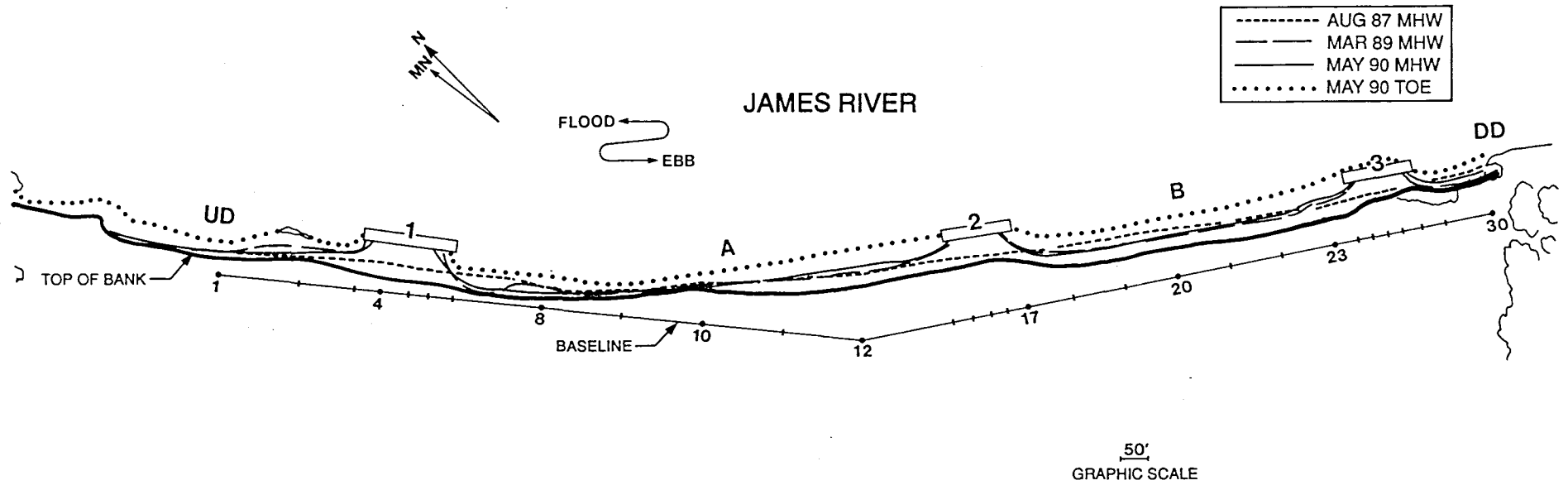
Figure 32. Hog Island Headlands wave heights.

Figure 33. Hog Island Headlands base map.

Table 10. Parameters for Hog Island Headlands*
May 1990 (See Figure 6 for Definition Sketch)

Breakwater/Bay	L_B	G_B	X_B	h_B	F_B	M_b	T_e	S_e	B_I	B_M	X_s	T_A	T_W	E_R	V_f
Updrift						?		0.5	6	3					
Breakwater 1	150		45	3.4	3.6		1.7	3.2	19	57	150	145			
Bay A		910				77.5		0.0	10	0					
Breakwater 2	100		40	2.8	2.7		1.0	2.9	12	46	100	95			
Bay B		600				54.5		0.8	17	12					
Breakwater 3	100		38	2.8	2.6		1.0	2.9	15	50	90	96			
Downdrift						?		1.0	5	16					

*   All dimensions in feet except E_R and V_f.

L_B - Breakwater crest length

G_B - Breakwater gap

X_B - Distance offshore CL breakwater to original MHW

h_B - Height of breakwater from bottom at CL to MHW

F_B - Breakwater freeboard, MHW to crest

M_b - Maximum bay indentation, CL breakwater to MHW

T_e - Tombolo elevation in lee of breakwater $\pm$ MHW

S_e - Backshore elevation at base of bank

B_I - Initial beach width, base of bank to MHW

B_m - Present beach width, base of bank to MHW

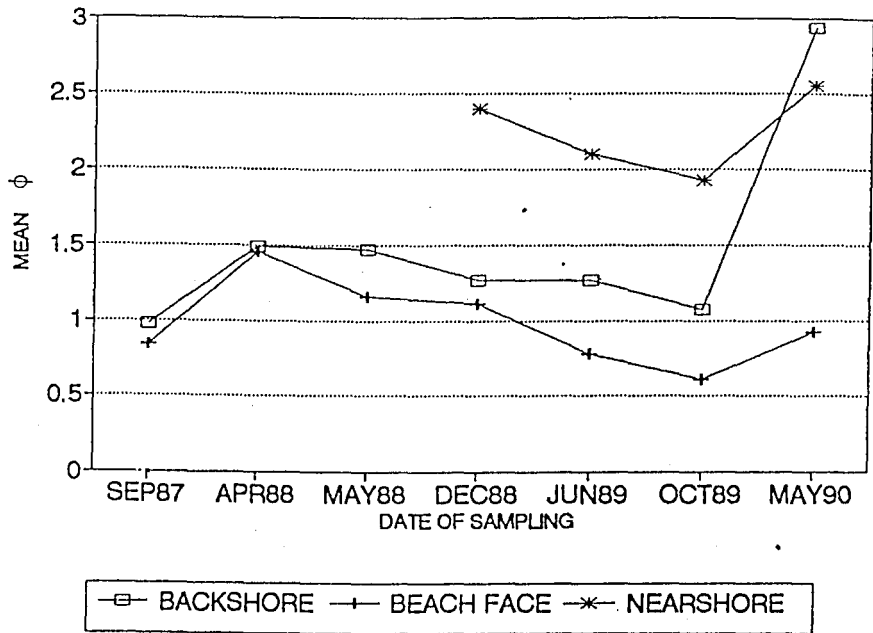X_s - Salient or tombolo length

T_A - For unattached tombolo, MHW to CL of breakwater

T_W - For attached tombolo, tombolo width at MHW
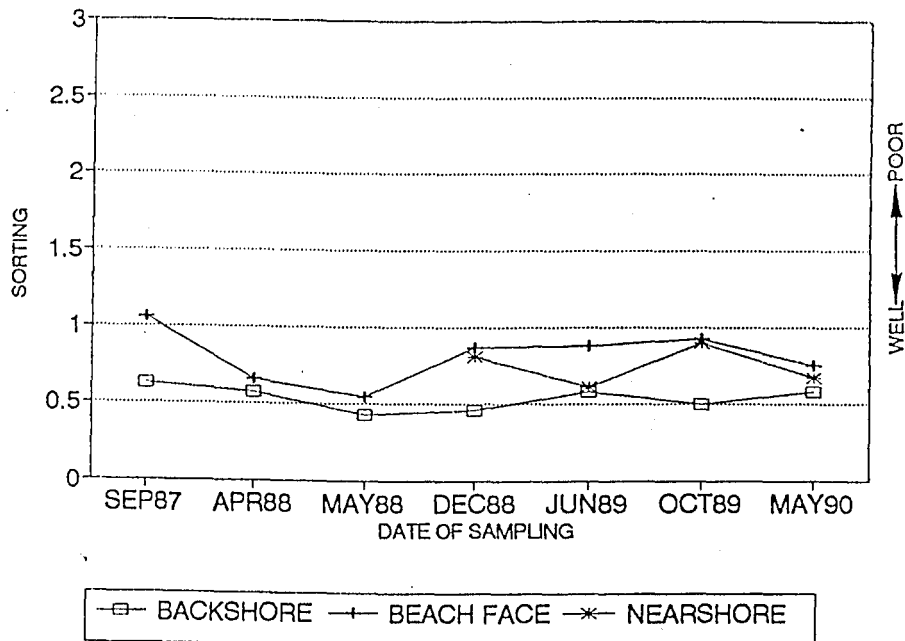
E_R - Erosion rate of BOB (ft/yr)

V_f - Net beach volume from 1985 - 1990 (cy/ft)

88

Figure 34.   Hog Island Headland sediment analysis.

SEDIMENT ANALYSIS
HOG ISLAND HEADLAND, PROFILE 20

C

--□-- BACKSHORE ---+--- BEACH FACE ---*--- NEARSHORE

SEDIMENT ANALYSIS
HOG ISLAND HEADLAND, PROFILE 20

D

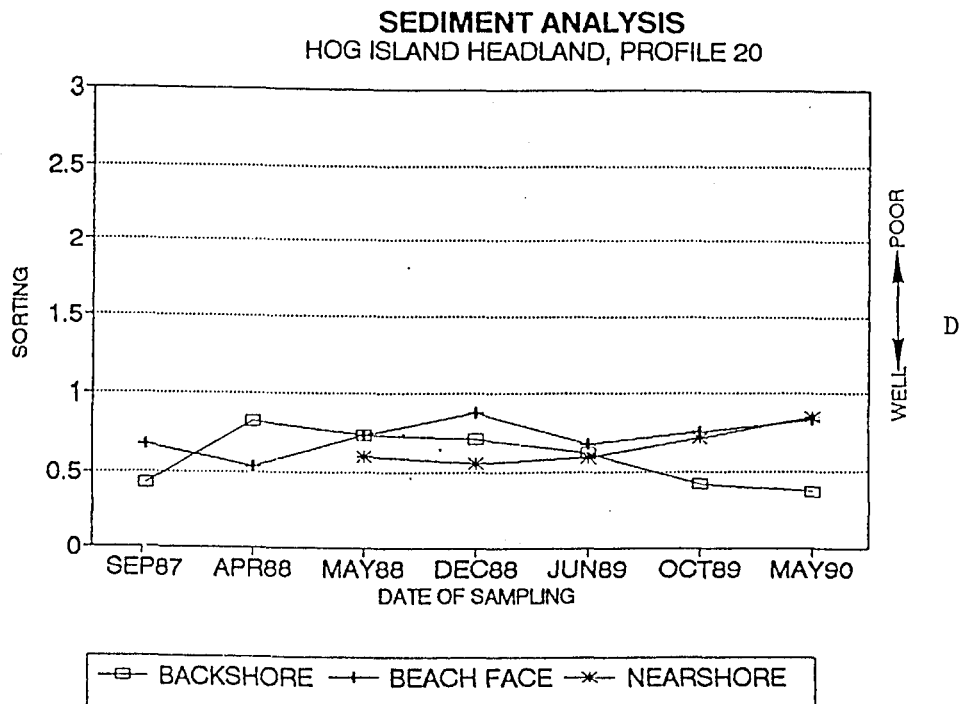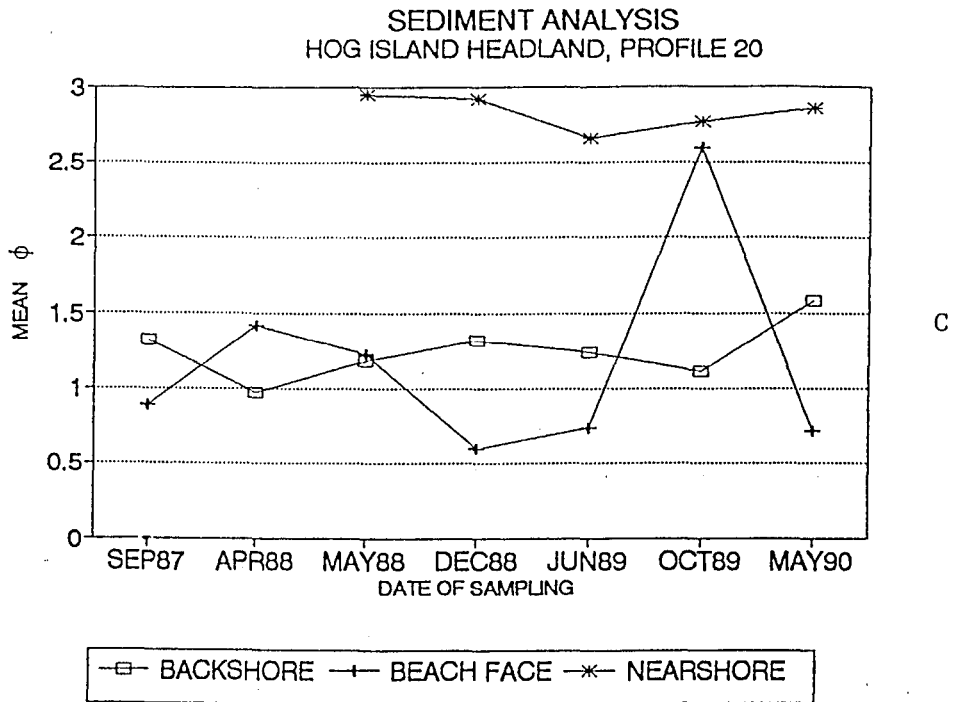--□-- BACKSHORE ---+--- BEACH FACE ---*--- NEARSHORE

Figure 34.   Hog Island Headlands sediment analysis.

90

7. Yorktown Bays, York River, York County

    a. Setting

The Yorktown Bays consist of three pocket beaches located about one mile downriver from the George P. Coleman Bridge at Yorktown, Virginia (Figure 35). The Yorktown Bays are on the property of the National Park Service's Colonial National Historical Park. This site is an example of estuarine beaches which have been stable over a relatively long period of time.

The Yorktown Bays have evolved over the past 50 years into three pocket beaches. The headlands separating each bay beach are composed of a highly indurated, shelly marl of the Yorktown Formation. The headlands are interfluves with banks approximately 80 feet above MSL. The bay beaches have developed in the adjacent drainages. The headlands were hardened with rock revetments in the early 1960s and reinforced in 1979. This created stable headlands and the beaches evolved into their present configuration (Figure 36A).

The Yorktown Bays are treated as three separate sites with three separate base lines. The bays are designated YB1, YB2 and YB3 (Figure 37). YB1, the largest Yorktown Bay, is approximately 400 feet long from the MHW line on each headland (Figure 36B). It is slightly crenulate shaped. The tangential section of the beach faces approximately 065°. YB2 and YB3 are smaller bays (150 ft and 190 ft, respectively) and have similar orientations but are more symmetrical in shape. All three bays are most influenced by northeasterly winds.

    b. Wave Climate

The Yorktown Bays face east northeast and have an average fetch across the York River of 2.2 nautical miles. However, there is a long fetch of 23 nautical miles to the east out the mouth of the York River and across Chesapeake Bay. The nearshore bathymetry moderates incoming storm waves. The predicted storm wave height (slightly greater than 2 feet) compares favorably to waves observed during the April 1988 northeaster.

Seasonal waves over the study period average about 0.66, 0.82 and 0.52 feet with the highest wave heights coming from the northeast (Figure 38).

c. Shore Changes

During the period August 1987 to March 1988, northwest winds were most dominant. The result was a shift of sand in each bay from the northwest to the southeast. Representative profiles of YB1 reflect this shift (Hardaway et al., 1988). This is an ephemeral situation. The general orientation of each bay aligns to the northeast over the long term as seen in historical aerial photography.

The Yorktown Bays have been observed during several northeast storms including the severe storm on November 4, 1985 and the April 1988 northeaster. YB1, with backshore elevations of 3.5 to 4.5 feet above MSL, became slightly deflated along the beach face. Beach sand was shifted back to the northwest end of the bay. However, no major beach cut or erosion to the backshore has been seen through profile surveys (Appendix A). The Yorktown Bays represent a unique shoreline situation in Virginia where stable, pocket beaches have evolved by a combination of geologic setting and the landowner's response to shore erosion (i.e. with the installation of riprap revetments).

d. Sediments

The beach on YB1 is characterized by generally well sorted medium coarse, shelly sand and gravel. The sands have been derived from historic and continued erosion of the adjacent headlands. Although riprap revetments protect the lower fourth of the headlands, the upper three quarters are exposed and actively eroding by surface runoff and pedistrian traffic.

The April 1988 storm produced the most noticeable changes in beach sand characteristics along the upper beach of YB1 in the tangential section of the bay and along the lower beach or step in the log-spiral section of the bay. Here, there was a general trend toward coarser sands

92

after the storm, followed by a return of finer sized material (Hardaway et al. 1988). Sediment change through time are seen in Figure 39.
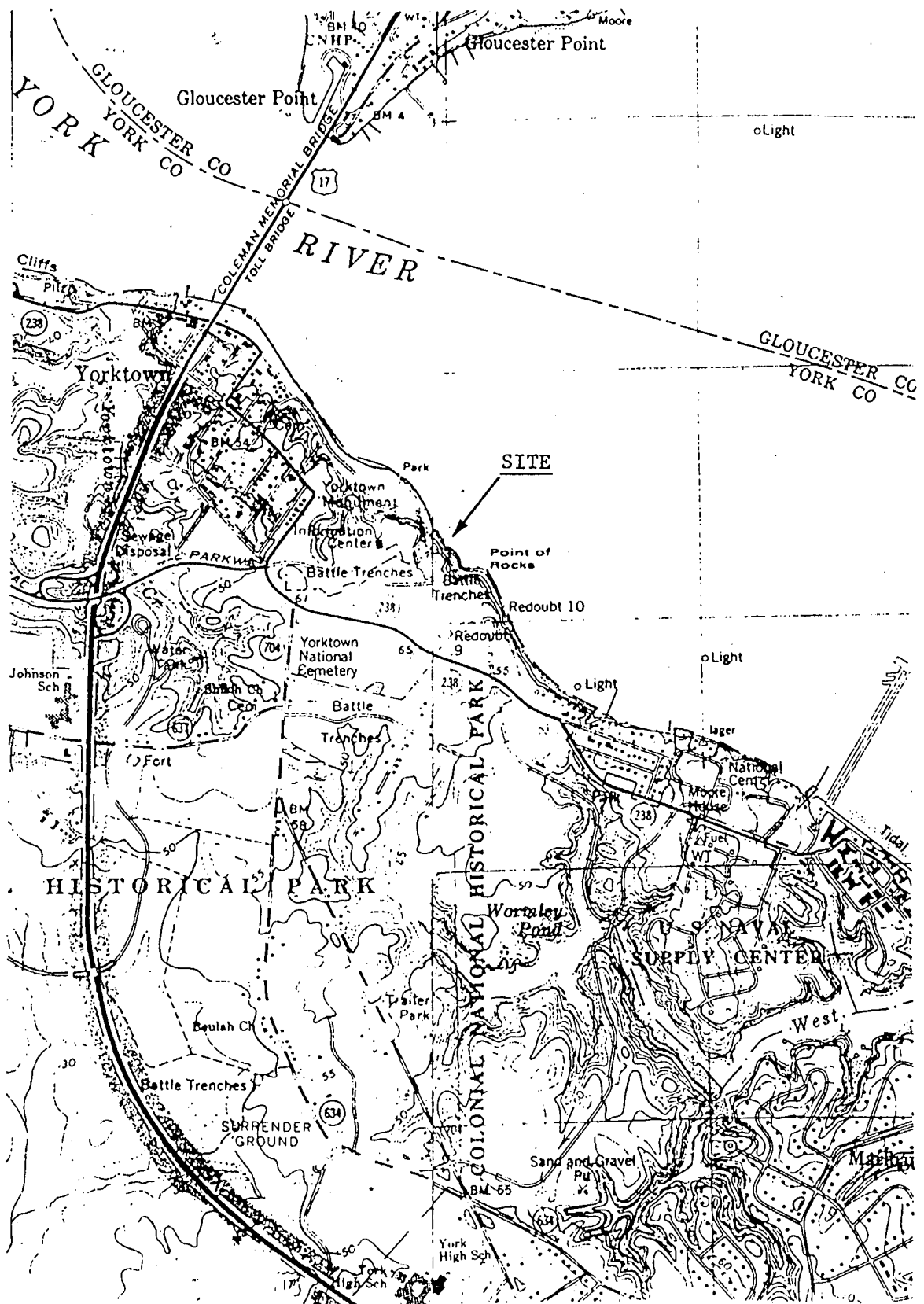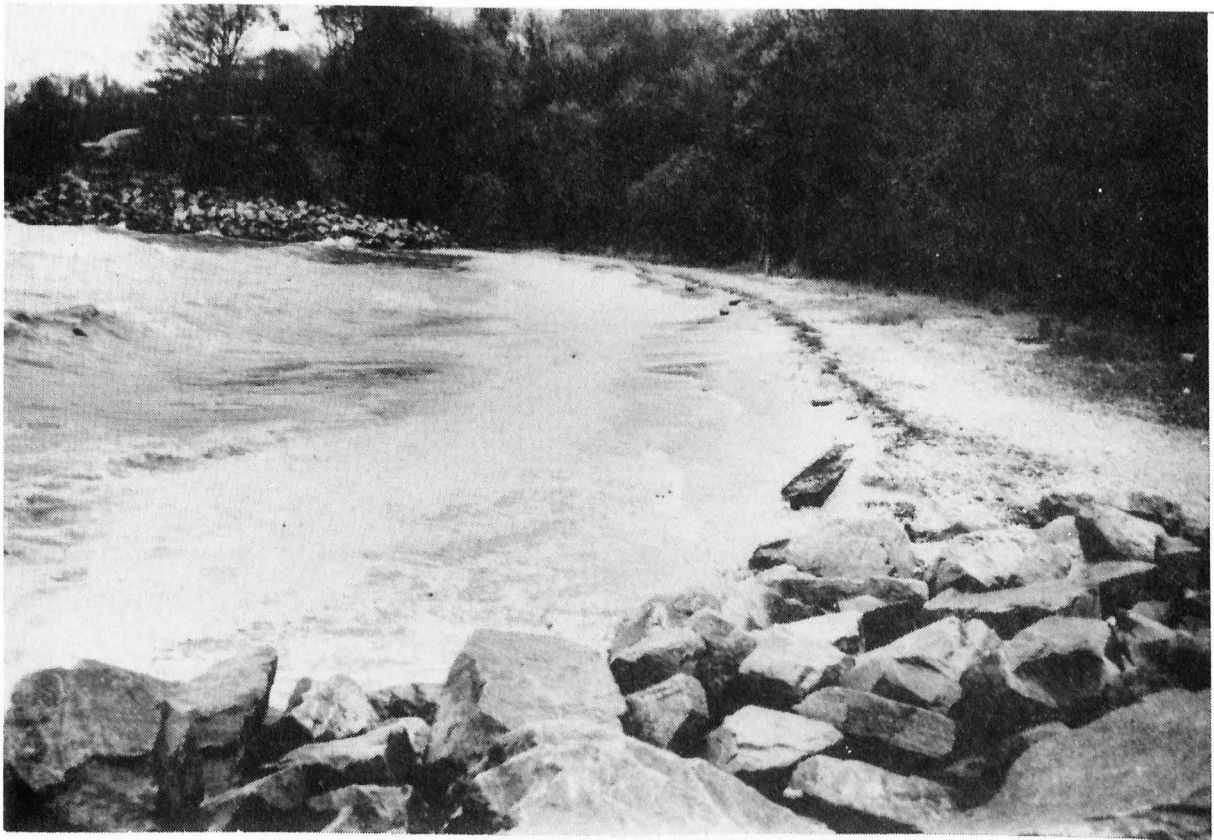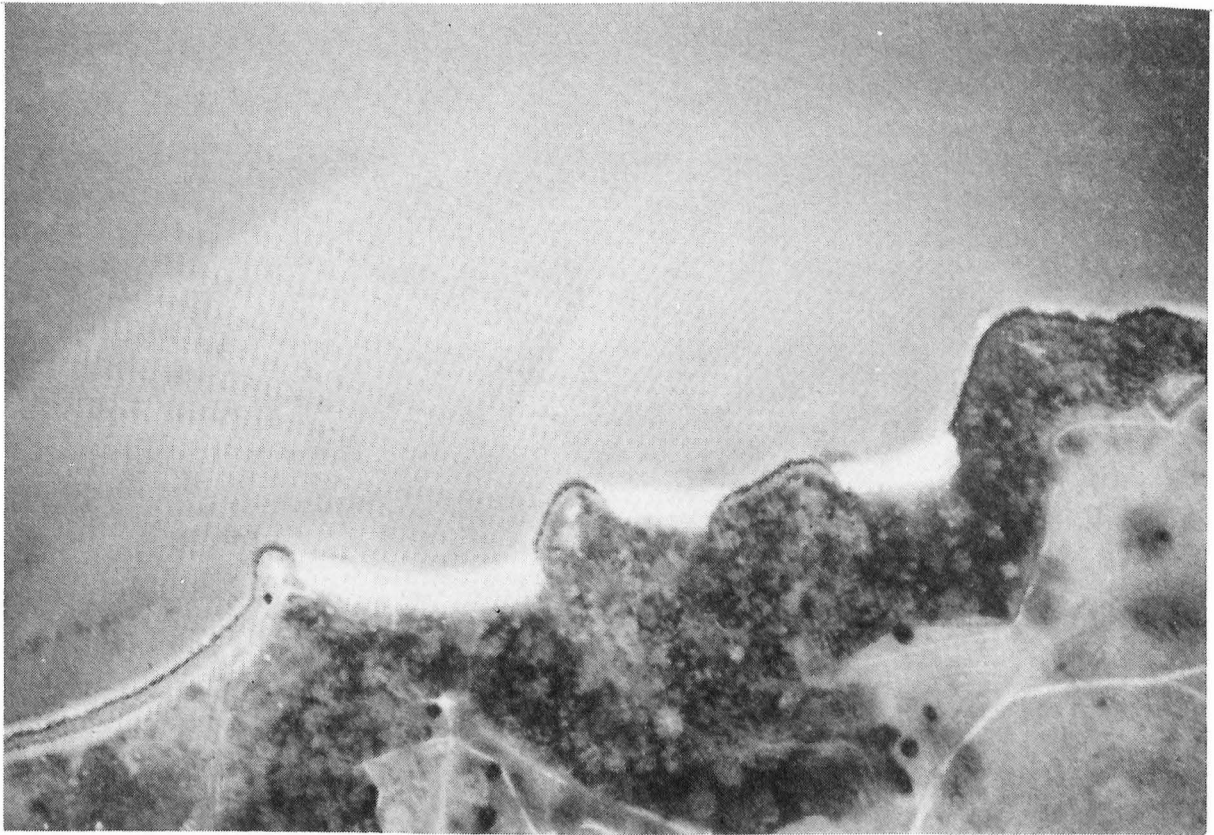
Figure 35. Yorktown Bays, York River, York County.
From Yorktown and Poquoson West 7.5 minute quadrangles.
Scale: 1 inch = 2,000 feet.

Figure 36A. Yorktown Bays - vertical aerial, 9 March 1988.

Figure 36B. Yorktown Bay number 1 - ground view looking east during 13
April 1988 northeaster.

YORK RIVER

FLOOD

EBB

AUG 87 MHW
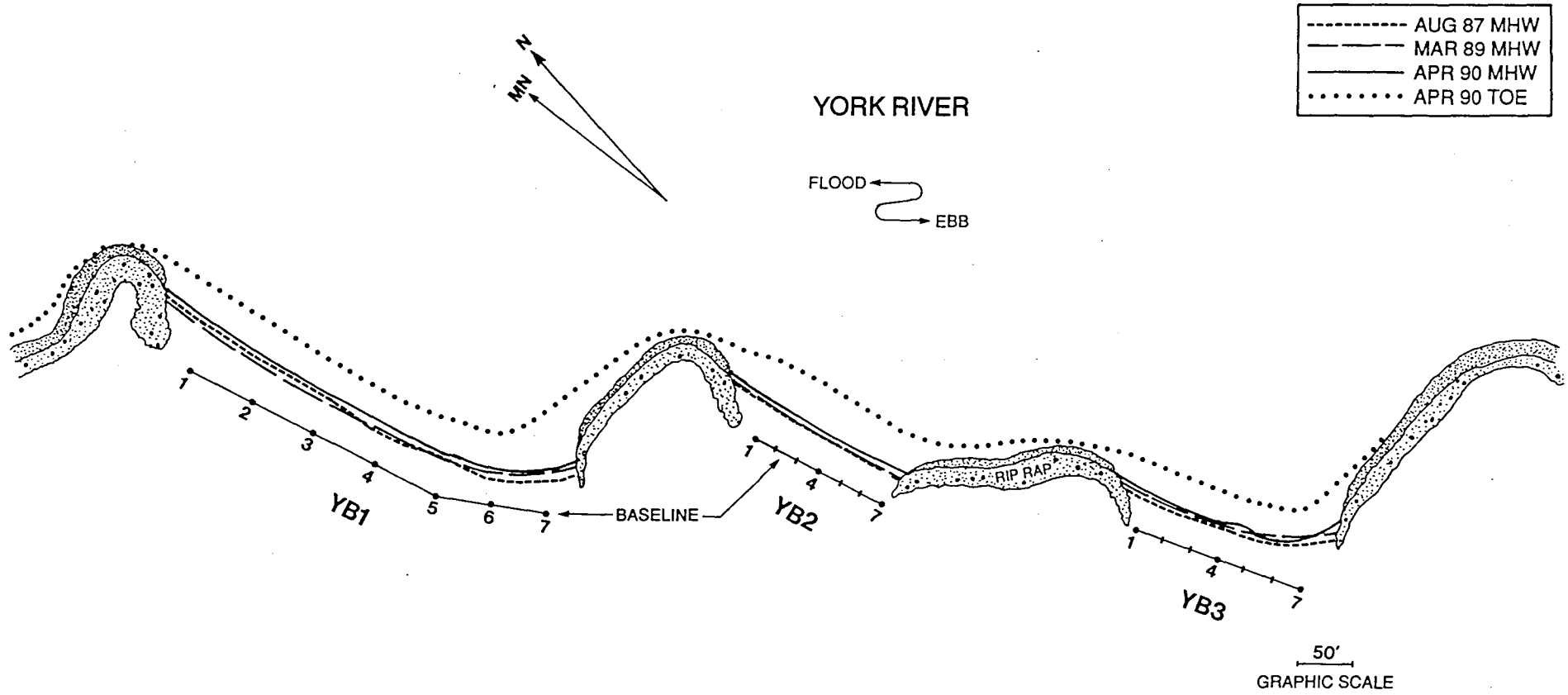MAR 89 MHW
APR 90 MHW
APR 90 TOE

N

MN

YB1

1
2
3
4
5
6
7

BASELINE

YB2

1
4
7

RIP RAP

YB3

1
4
7

50'
GRAPHIC SCALE

Figure 37.   Yorktown Bays base map.
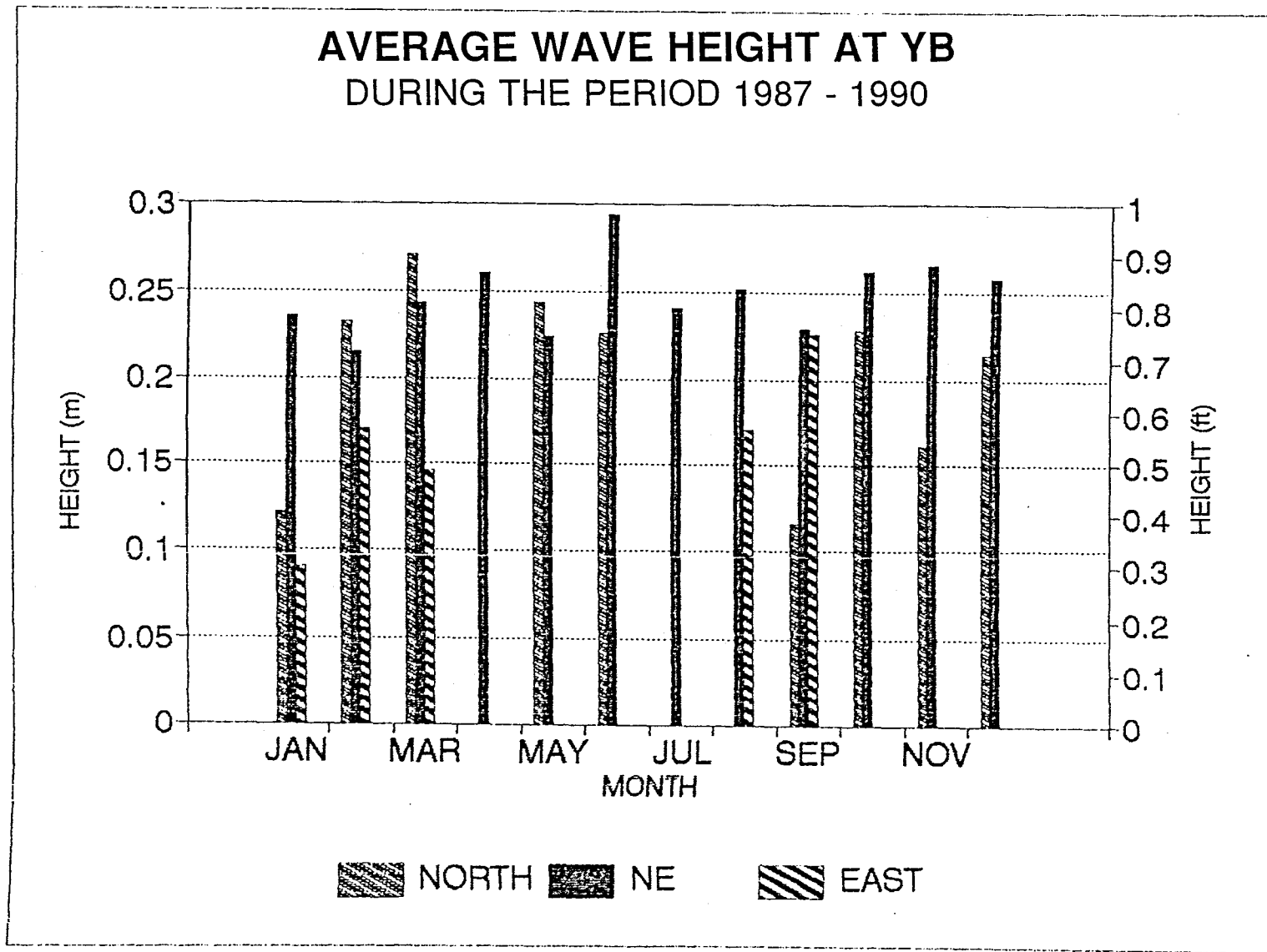
Figure 38. Yorktown Bays wave heights.
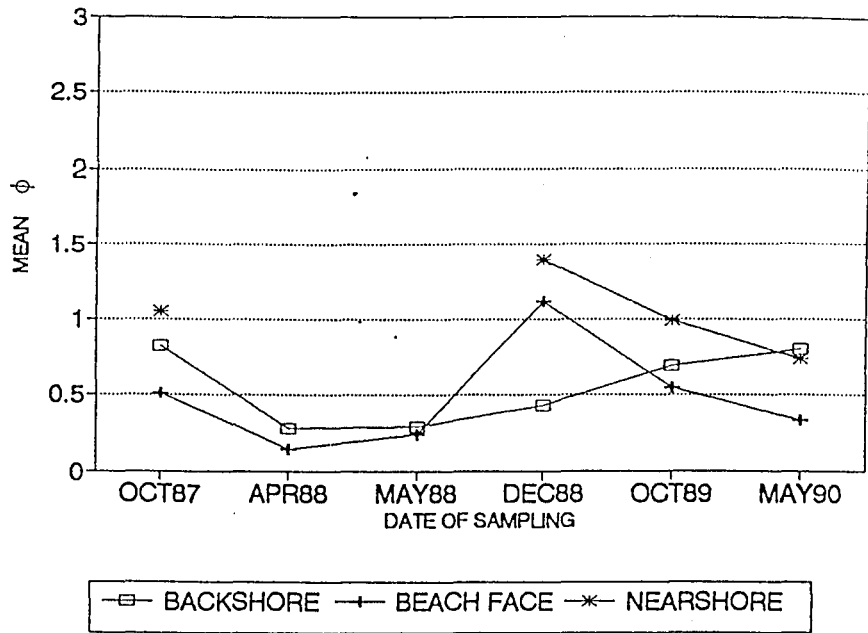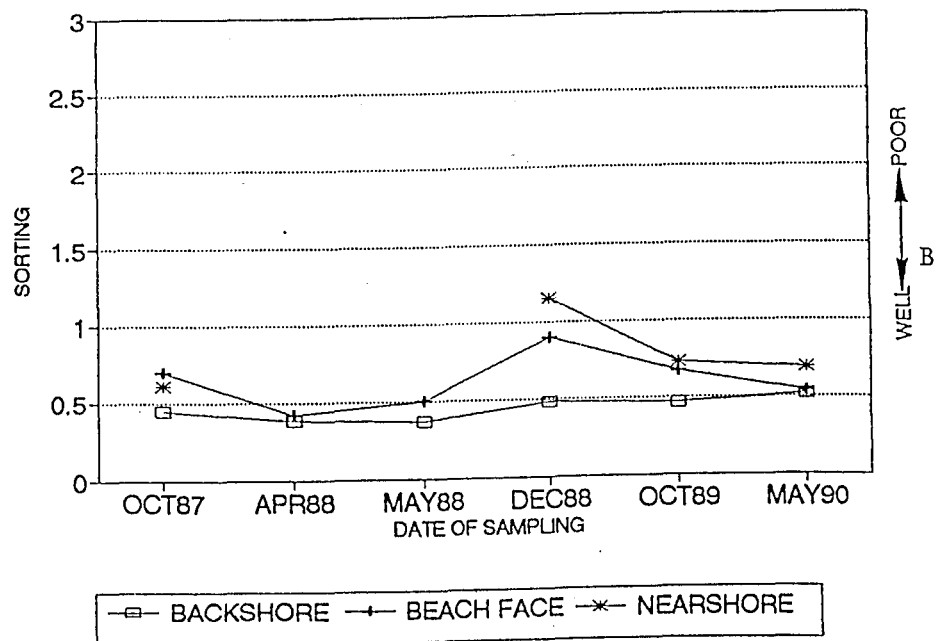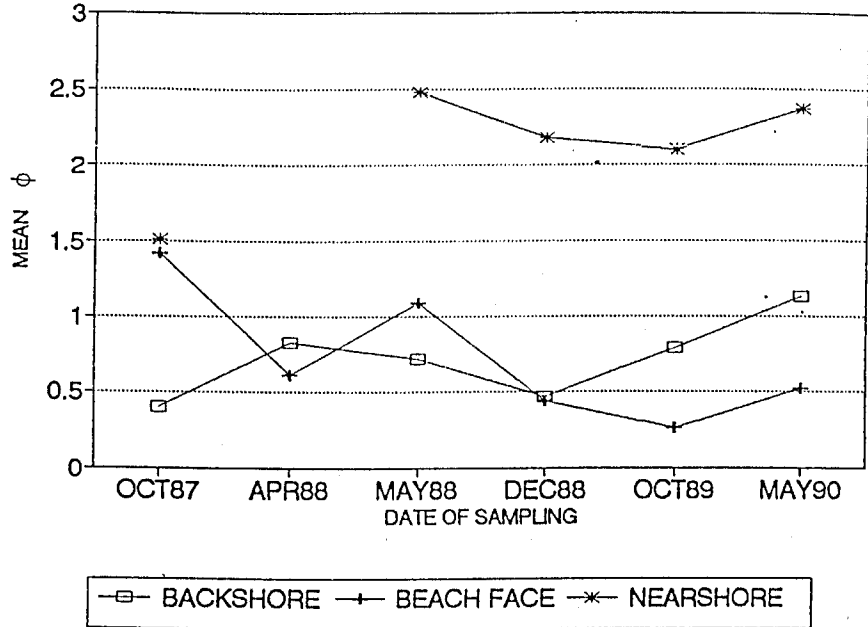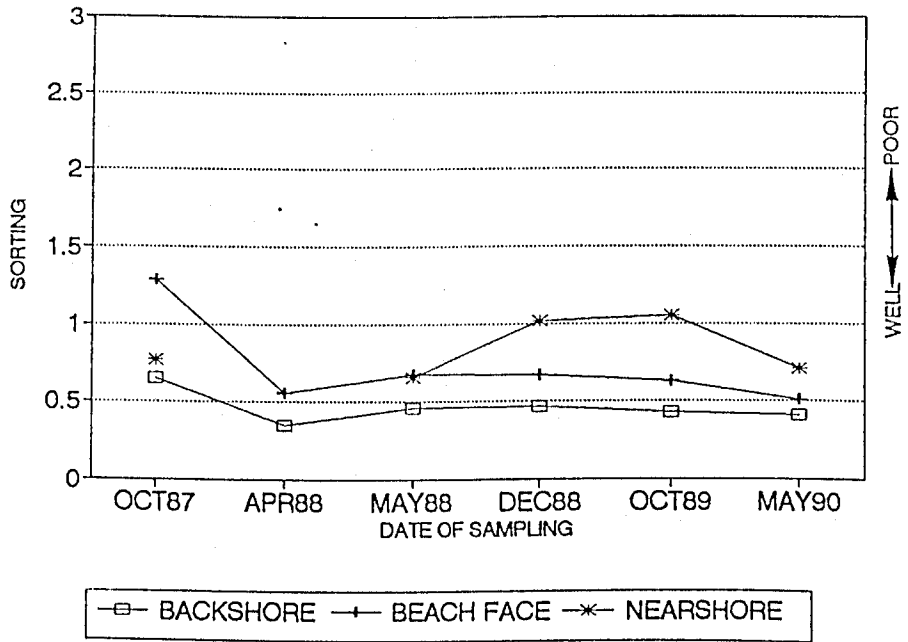
Figure 39. Yorktown Bay 1 sediment analysis.

SEDIMENT ANALYSIS
YORKTOWN BAY 1, PROFILE 6

C

SEDIMENT ANALYSIS
YORKTOWN BAY 1, PROFILE 6

D

Figure 39. Yorktown Bay 1 sediment analysis.

8. Summerille, Potomac River, Northumberland County

    a. <u>Setting</u>

The Summerille site is located on the Potomac River near Smith Point (Figure 40). The purpose of including the Summerille site in this project was to document the evolution of a crenulate-shaped embayment along an estuarine shore. Figure 41 depicts the positions of the 10-foot high fastland bank from 1937 to 1973. The segment in front of the Summerille house evolved into headlands after the installation of a groin field in 1967. The fastland here has an historic erosion rate of approximately 5 feet per year (Byrne and Anderson, 1978). A low sandbag sill, which was installed in front of the Staples house (downdrift neighbor) in 1975, had the effect of slowing the erosion. Consequently, a bay has evolved between the Staples house and Summerille house.

    b. <u>Wave Climate</u>

The Summerille shoreline faces northeast and has an average fetch of 9.5 nautical miles. The mean wave heights for different directions affecting the site are shown in Figure 42. There have been few wave observations during storm events. Consequently, the effect of the nearshore bar system on wave height and angle of approach is not known. It is felt that these bars play a significant role in the littoral processes acting upon Summerille.

    c. <u>Shore Changes</u>

By 1973, the Summerille groin field had created a 40-foot offset to the southeast. In 1978, a northeast storm caused an additional 10 feet of bank loss (Anderson et al., 1983). A gabion spur, which reduced erosion immediately downdrift of the groin field, was constructed. However, the banks continued to erode further downriver. The Staples' sill was 50 feet offshore in 1978. In 1987 a short rock revetment was placed in front of the Staples house (Figure 43A). This will eventually act as a small headland within the larger embayment.

The evolution of the Summerille/Staples bay sparked interest in controlling shore erosion by headland emplacement and allowing the unprotected banks to evolve into what would eventually become a stable shore planform. How far this might be is addressed in the following section on the results of this project. The headland breakwaters built at Hog Island were meant to evaluate this philosophy.

The tangential section of the bay runs from the Staples revetment northwestward along, and roughly parallel to, the surveyed baseline to profile 4, where the log-spiral curves toward the Summerille spur (Figure 44). Its orientation is generally to the north northeast which indicates seasonal wave climate at this point from a northern direction and net littoral transport southeastward toward Smith Point. Bank losses have been greatest in the center of the embayment since August 1986. The April 1988 northeaster caused further erosion of the fastland bank, especially at profile 4. No erosion of the fastland occurred in the lee of the spur. Profiles for the project period are found in Appendix A.

d.   Sediments

The beach along the embayment averages about 30 feet wide from the base of the bank to MHW. It is characterized by well-sorted, medium sands (Figure 45). This material is derived from erosion of the bay banks and littoral transport which brings sand into the embayment from eroding shorelines to the north.
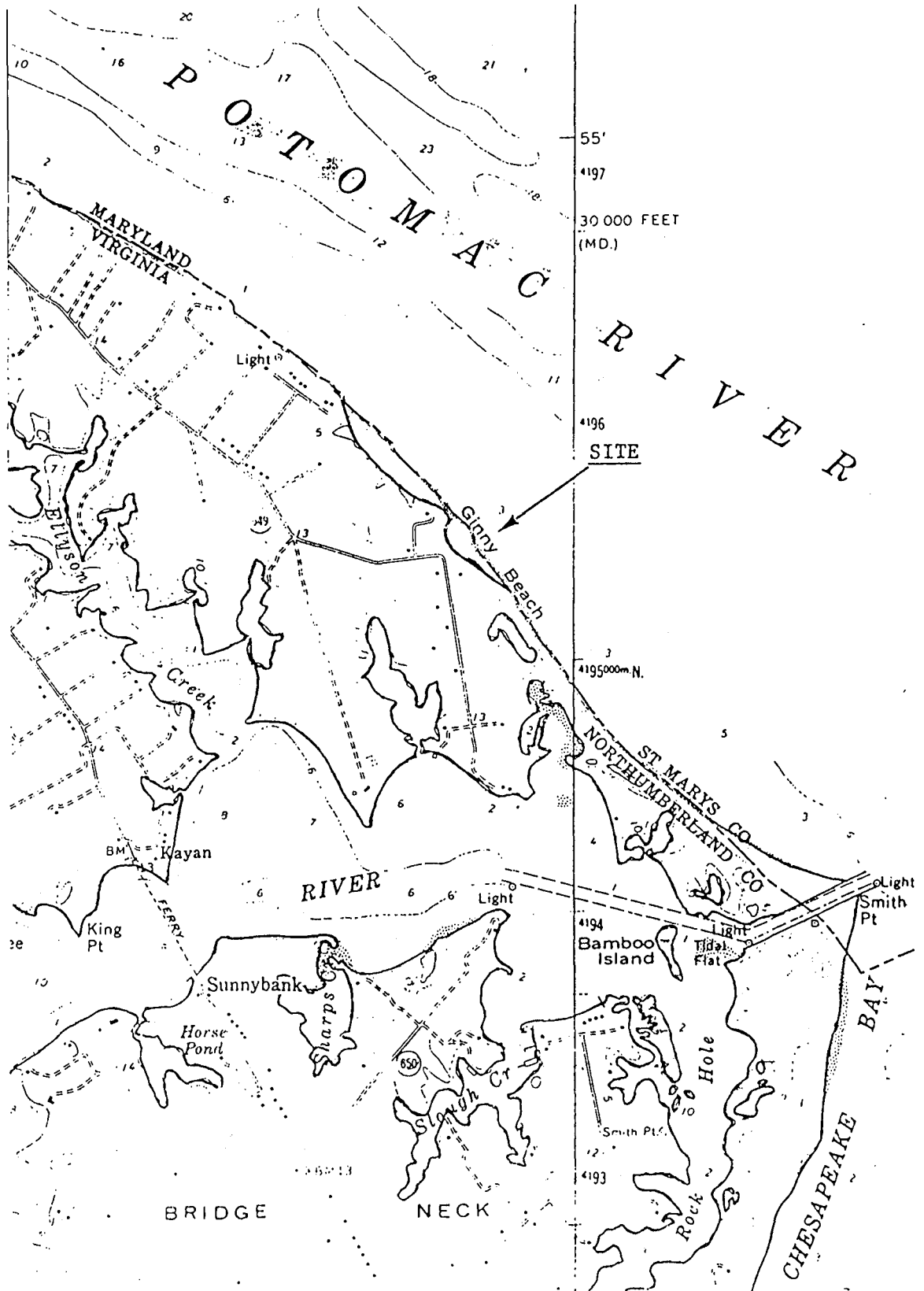
Figure 40. Summerille, Potomac River, Northumberland County.
From Burgess 7.5 minute quadrangle.
Scale: 1 inch = 2,000 feet.
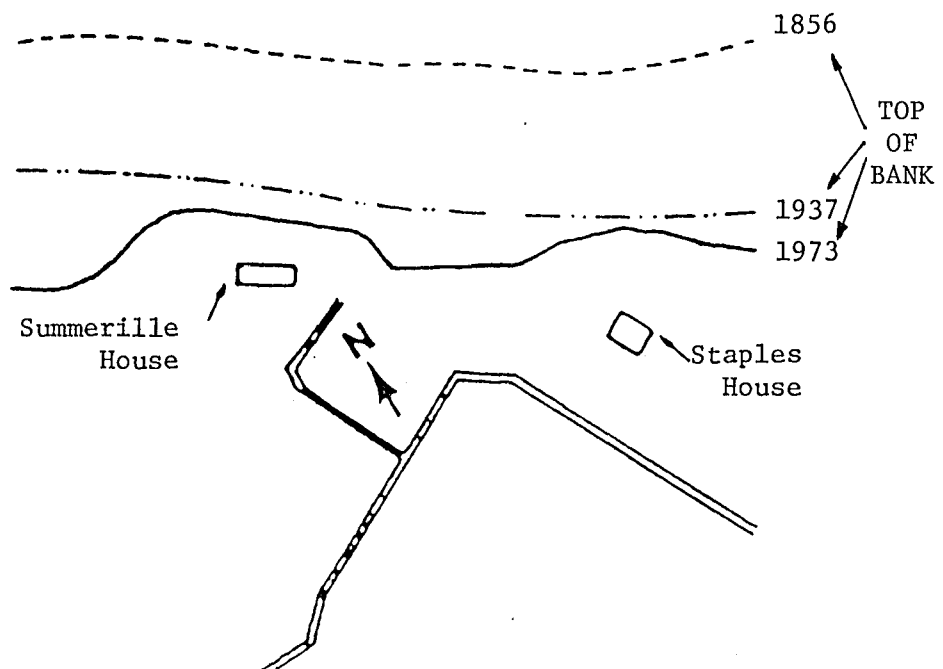
Figure 41. Summerille spur site, historical shoreline changes (after Anderson et al., 1983).
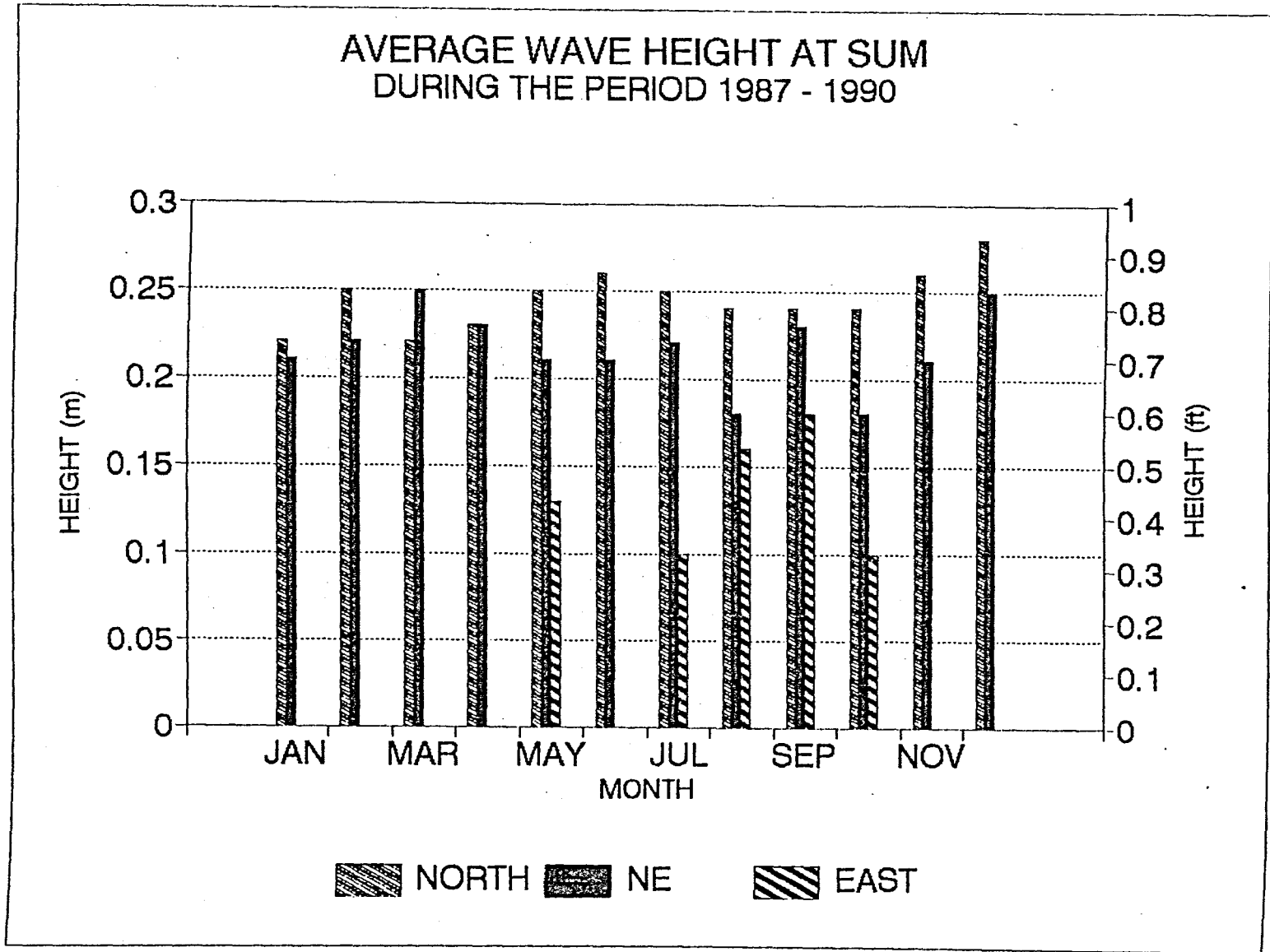
Figure 42.   Summerille wave heights.

Figure 43A. Summerille - vertical aerial, 29 March 1988.

Figure 43B. Summerille - ground view looking west from Staples'
revetment, 13 May 1988.
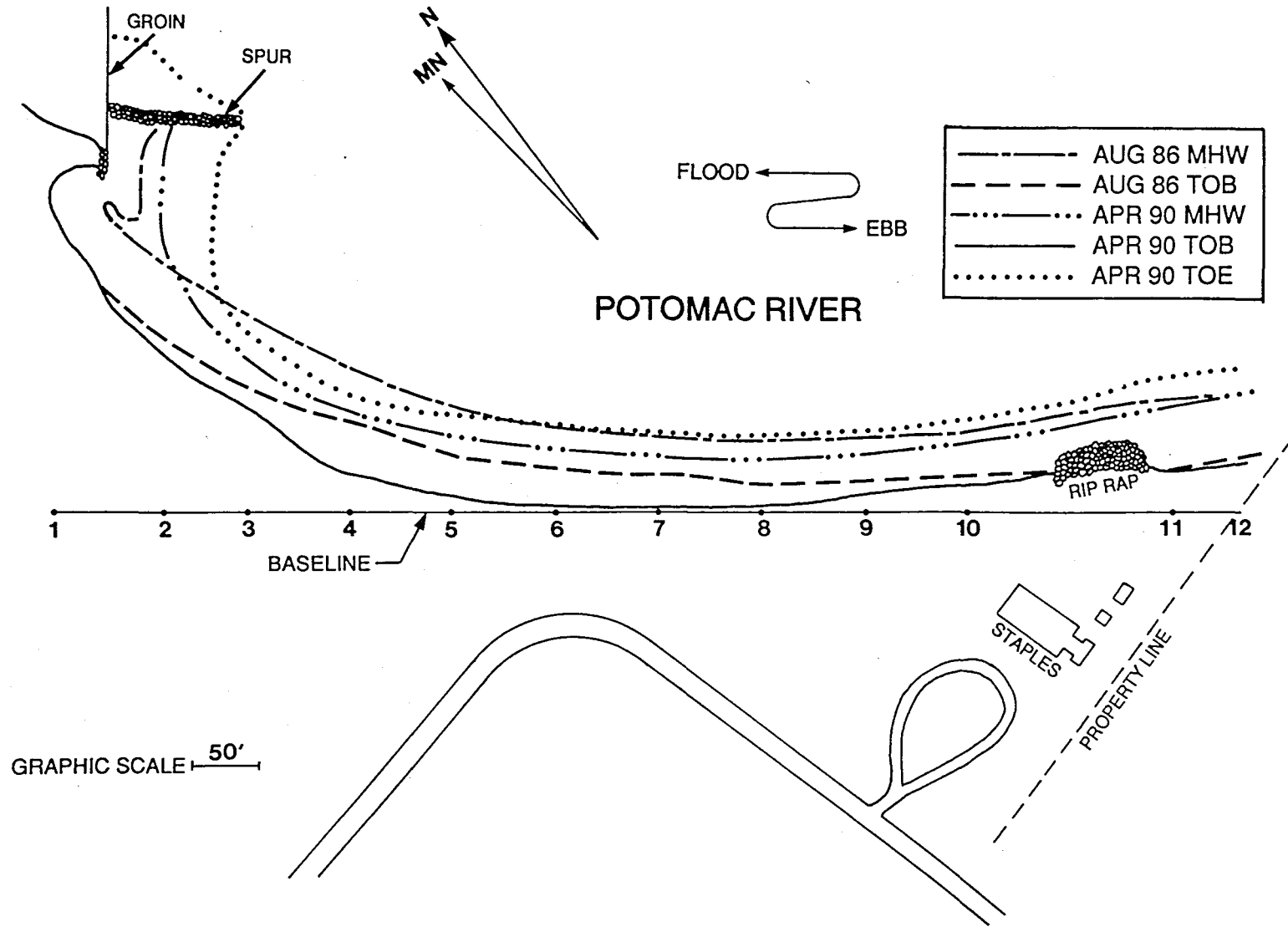
Figure 44.   Summerille base map.

SEDIMENT ANALYSIS
SUMMERILLE,PROFILE 4
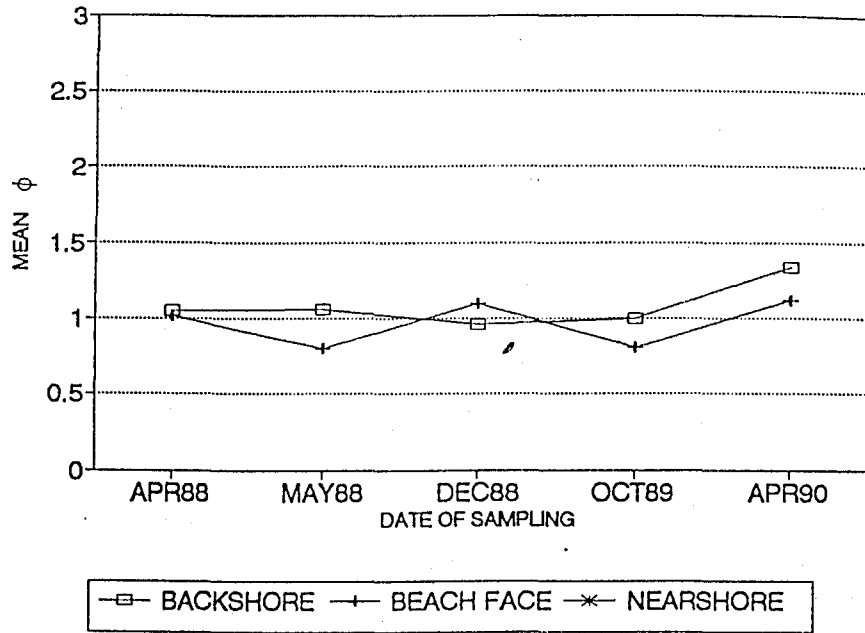
A
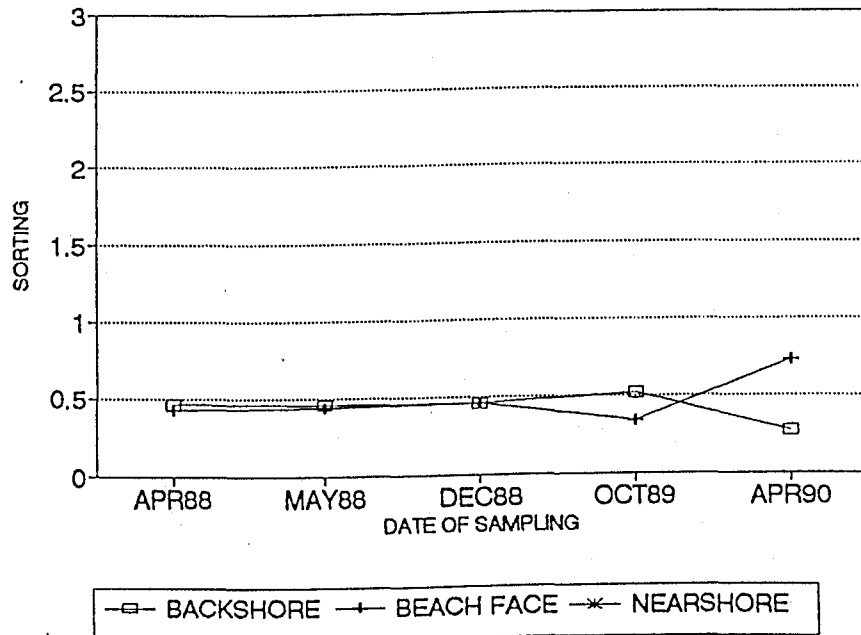
—□— BACKSHORE —+— BEACH FACE —*— NEARSHORE

SEDIMENT ANALYSIS
SUMMERILLE,PROFILE 4

B

—□— BACKSHORE —+— BEACH FACE —*— NEARSHORE

Figure 45.   Summerille sediment analysis.

SEDIMENT ANALYSIS
SUMMERILLE,PROFILE 6

C

-□- BACKSHORE -+- BEACH FACE -*- NEARSHORE

SEDIMENT ANALYSIS
SUMMERILLE,PROFILE 6

D

-□- BACKSHORE -+- BEACH FACE -*- NEARSHORE
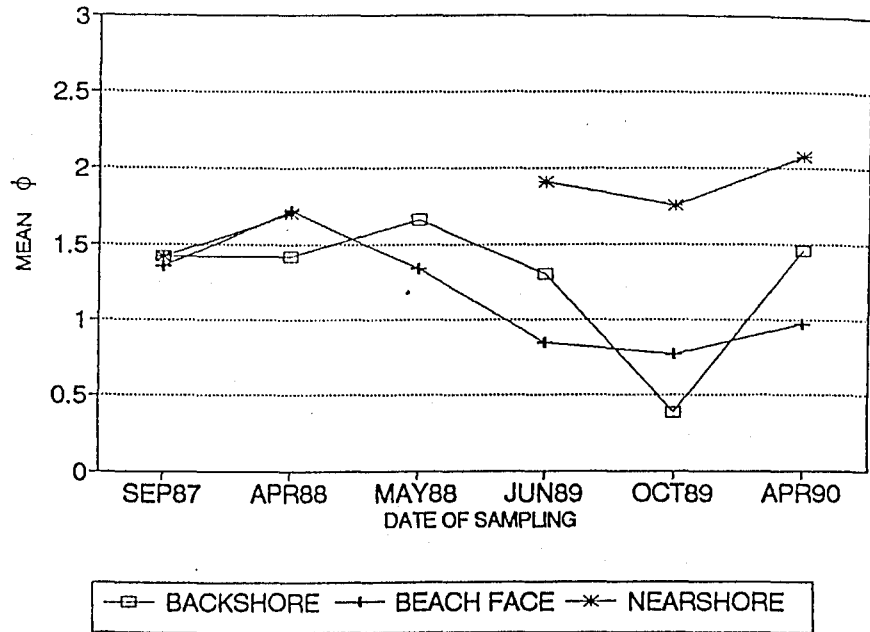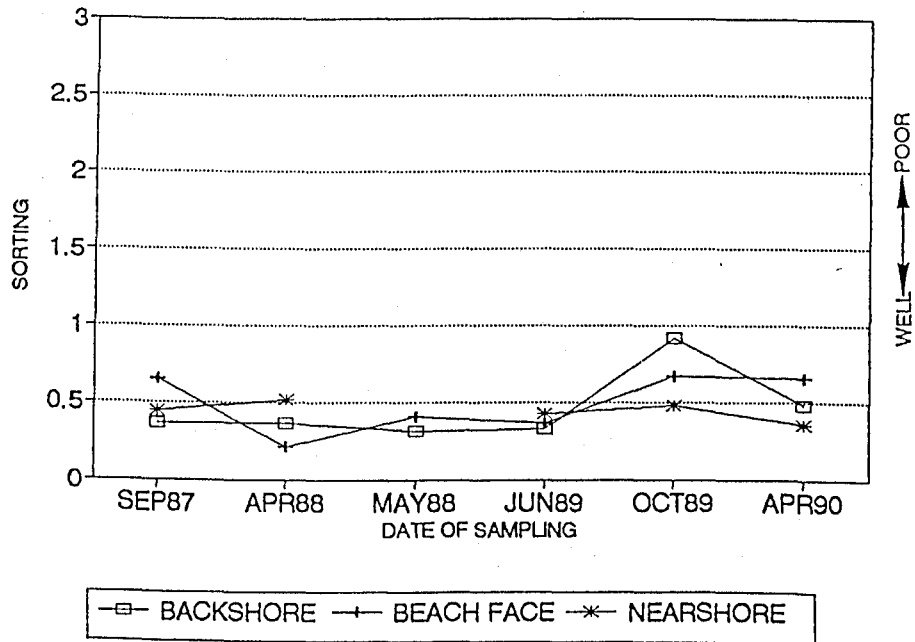
Figure 45.   Summerille sediment analysis.

V. Results and Discussion

A. Sediments

Sediments for the project sites were analyzed for grain size distribution and sorting. Beach fill from upland sources (borrow pits) were placed at HI2, DMF, WAL and HIH. The backshore regions were subjected to further additions of beach fill material (DMF) and bank slumping (CHP, HI2 and SUM). Beach face and nearshore samples were also evaluated in terms of grain size and slope for each profile period. No reasonable correlation could be drawn from linear regression analysis although there is a very slight increase in beach slope with increasing mean grain size (Figure 46). The correlation coeficient is 0.235. Thus, a simple summary table is presented for mean values of beach and nearshore slopes, sediment size and sorting means (Table 11).

The beach sands at the project sites are medium to coarse grained and moderately to well sorted. Though there is no significant comparsion to beach slopes, the site with the coarsest beach sand (YB) does have the steepest beach slope. Overall, nearshore sediments are fine to medium sands and are also moderately to well sorted.

B. Breakwater Sites

1. Beach Volumes and Erosion Rates

Beaches along the Chesapeake Bay shorelines have planar beach faces and would be considered highly reflective. There is generally a measurable sharp break at or just below MLW where the offshore portion of the profile flattens out. This has been called the beach step or toe. There has been very little research done on the dynamics of these fetch-limited beaches. However, Skrabel (1987) found that beach profiles along a nourished beach on the York River tended to follow an equilibrium state as defined by Dean (1977) where coarser sands are associated with steeper beach profiles and lesser beach profiles are relative to finer sands.
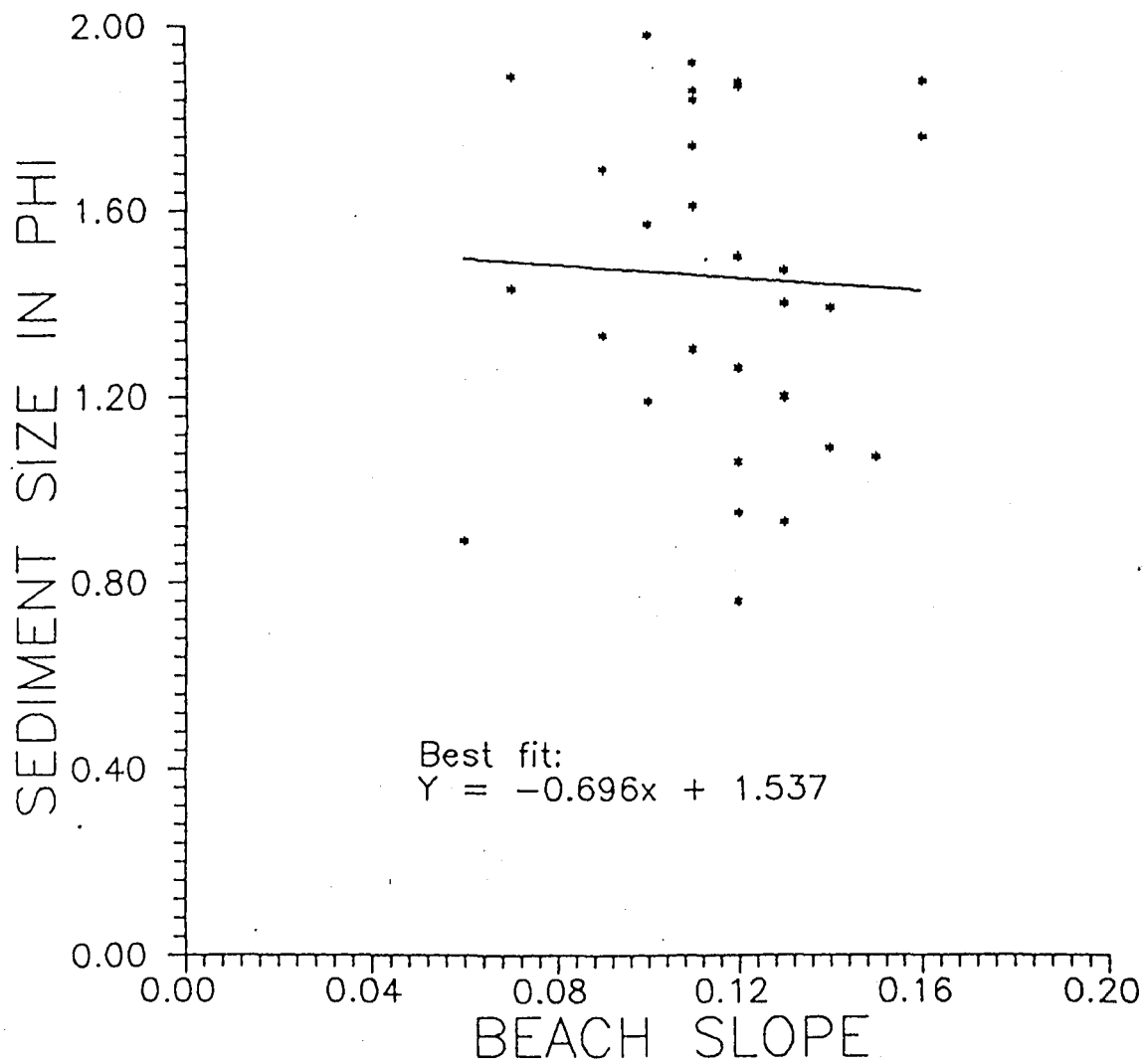
Figure 46. A linear regression analysis for beach slope and sediment size. The beach slope increases slightly with sediment size.

Table 11. Results of Slope and Sediment Analyses

| Site | Beachface | | | | Nearshore | | | |
| | Slope | Sediment Size | | Sorting (phi) | Slope | Sediment Size | | Sorting (phi) |
		(mm)	(phi)			(mm)	(phi)	
HIH	0.11	0.40	1.32	0.68	0.013	0.19	2.39	0.69
HI2	0.12	0.40	1.32	0.73	0.010	0.21	2.19	0.85
CHP	0.11	0.30	1.73	0.93	0.009	0.18	2.47	0.77
DMF	0.11	0.43	1.29	0.59	0.007	0.22	2.26	0.87
WAL	0.12	0.32	1.62	0.93	0.012	0.12	3.09	0.60
NPS	0.11	0.27	1.88	0.47	0.005	0.20	2.34	0.56
YB	0.13	0.67	0.55	0.63	0.015	0.32	1.63	0.82

A linear regression analysis was performed for the amount of beach material, cy/ft, and the rate of bank erosion, ft/yr. The r^2 value yielded 0.236 which indicates very little correlation amongst the data. There were no trends between bay and breakwater (beach volumes versus erosion rates) for CHP, NPS, and HI2. However, there was essentially zero bank erosion for DMF and WAL. This situation occurs when the alongshore beach volume is equal to or greater than 3 cy/ft for the bays. Volumes of up to 6 and 10 cy/ft occur behind breakwater units at WAL and DMF respectively.

2. Site Parameters

Storm wave observations and wave hindcasting (using modified SMB method, Section III, B.2) were used to evaluate the fetch-limited wave climate in the area of the study sites. These methods were used to estimate average seasonal and storm breaking wave conditions for medium energy estuarine shorelines in the Chesapeake Bay (Table 12).

Wave climate assessment has been the most difficult task in this study. However, it is the most important factor because the wave climate is the dominant process that drives beach and nearshore sediment transport. The response to the process is seen in the beach planforms and profiles at each site. By comparing profiles and site parameters among the breakwater sites over 3 years in this study, some general

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Table 12. Estimated and Observed Seasonal and Storm Wave Climate for Medium Energy Shorelines in Chesapeake Bay

Water Level	Wind Speed Range (mph)	Breaking Wave Height (ft)	Wave Period (sec)	Wave Length (ft)
Between MHW and MLW	10-20	0.4-0.8	1.2-1.8	12-20
1 foot above MHW	15-25	0.8-1.4	1.8-2.2	20-35.
2 feet above MHW	20-40	1.5-2.0	2.0-3.0	35-45

relationships can be drawn. Also, a comparison of the study sites with other breakwater installations in the Chesapeake Bay provides useful empirical data for further evaluation of this type of shoreline management strategy.

The long term performance of each breakwater site should be assessed by observing how stable beach planforms attenuate wave action and reduce erosion along the base of the upland bank (BOB). At this point, relative stability of the BOB can be further evaluated in terms of bay beach backshore width (B_m) and backshore elevation (S_e) (Figure 47). Breakwater sites which have a stable BOB to date are Drummonds Field (DMF) and Waltrip (WAL) where B_m and S_e are > 30 feet and 3 feet respectively. Individual breakwater units in this study with a higher freeboard (F_B) also tend to maintain a higher backshore elevation (Figure 48).

Unstable and eroding BOB occurs at CHP, NPS, and HI2. The breakwater units at these sites were placed at or just beyond MLW. Presently, they do not allow enough offshore distance for the development of stable backshore beach width, even at HI2 where beach fill was placed. Further bank erosion may create the necessary distance. This is especially true of NPS where erosion of the low bank continues to increase the backshore beach width and erosion rates have decreased over the past two years.
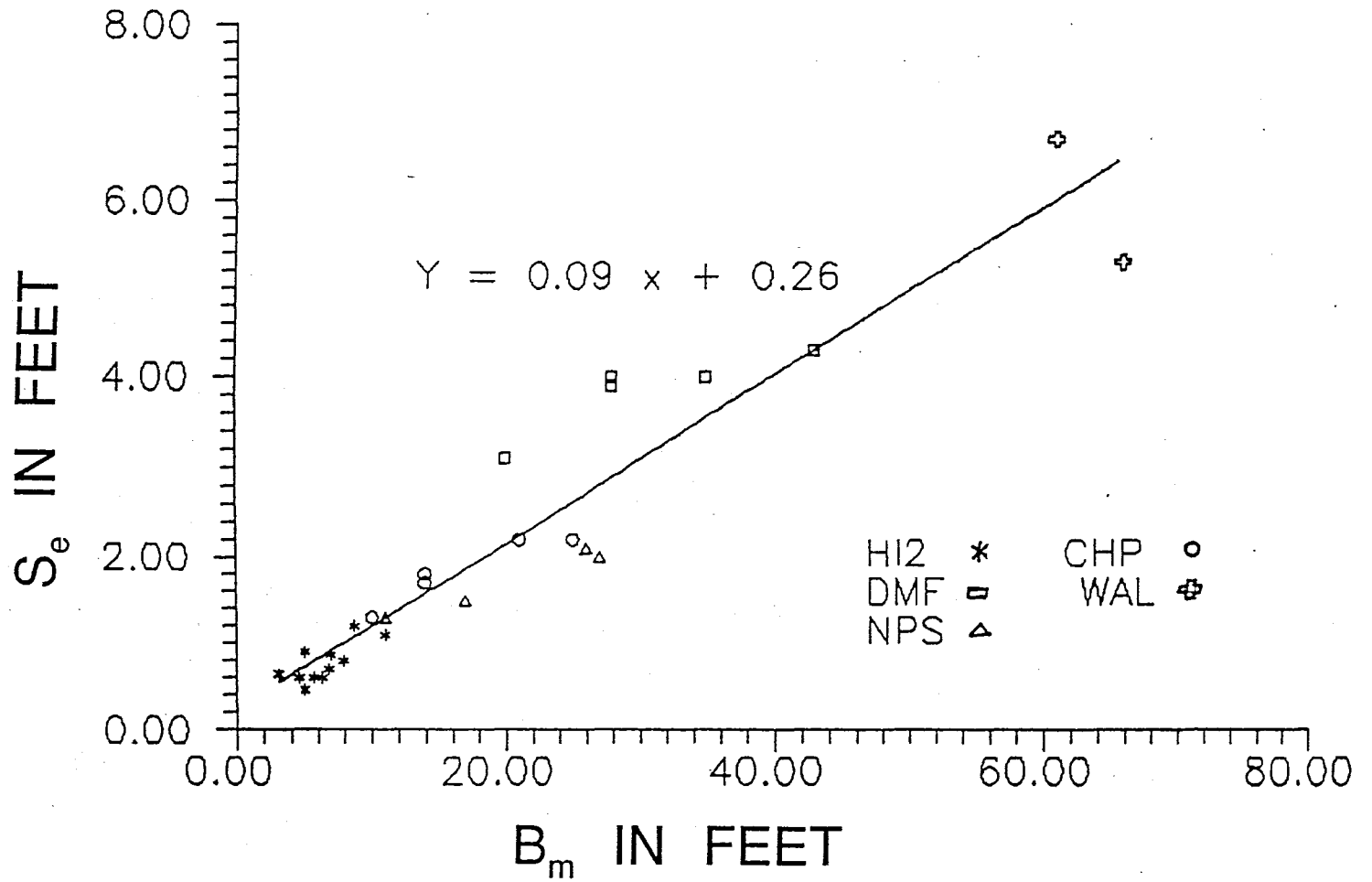
$$Y = 0.09 x + 0.26$$

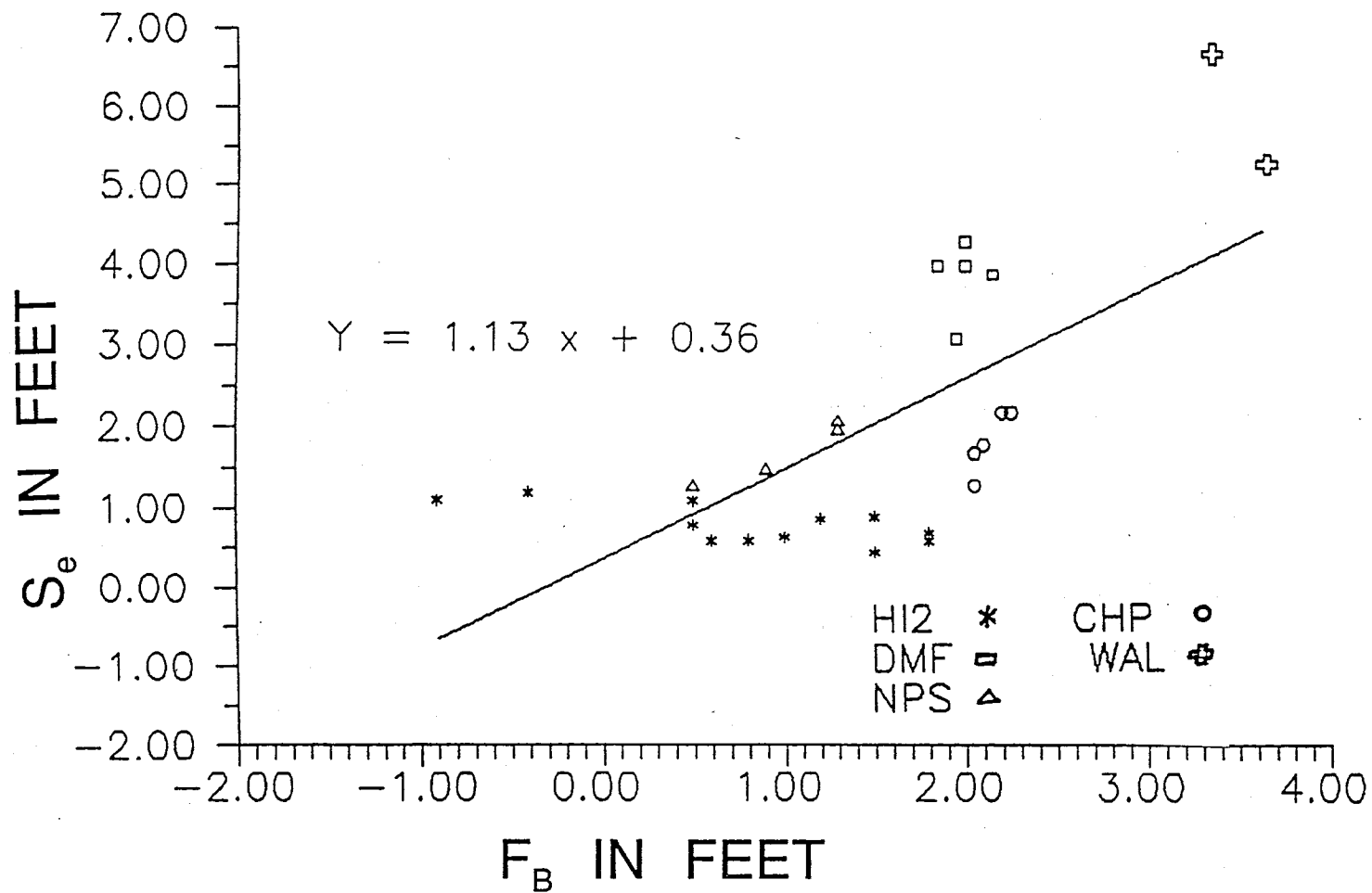Figure 47. The relationship of B_m to S_e for breakwater/bays at HI2, CHP, DMF, WAL and NPS.

Figure 48. The relationship between F_B and S_e for breakwater/bays at HI2, CHP, DMF, WAL and NPS.

The April 13, 1988 storm was a moderate northeaster with a storm surge high (at Gloucester Point) of approximately 2.2 feet MHW (VIMS VBASE). Statistically this water level occurs about once per year (Boon et al., 1978). This particular storm had sustained winds of over 30 mph (Virginia Power wind data-Yorktown Station). The mean high water level during the peak of the storm was closer to 2.0 feet MHW and is used as a benchmark to assess B_m and S_e at each breakwater site. This level of storm surge is seen on the representative profiles for each breakwater site.

Figures 49 to 53 are typical mid-bay and breakwater profiles for the breakwater sites showing MLW, MHW and a storm surge of +2.0 feet MHW. The beginning and ending profiles for the project show the net change. It can be seen that B_m and S_e are indequate for BOB protection for this water level at CHP, NPS, and HI2, (Figures 49 to 51). As previously mentioned, continued bank erosion may eventually allow the backshore region at these sites to increase to a protective width. It must also be noted that a storm surge is most erosive when the shore is generally facing the direction of wave approach.

DMF and WAL had sufficient B_m and S_e to provide BOB protection (Figures 52 to 53). However, during the April 1988 storm, these two sites were not subjected to direct storm wave attack because of their southwesterly exposure and no significant beach changes were measured. Storm conditions during November 4, 1985 storm were at a similar water elevation but with 1.5 to 2.0 foot observed onshore waves from the southeast occurring at DMF. This site (Waltrip was not built then) had breakwaters which were built far enough offshore (X_B) so that a wide backshore could be emplaced with beach fill and no beach losses or bank erosion was observed.

A linear regression analysis was performed using selected bay and breakwater parameters for all the breakwater sites or a total of 27 bays

CHIPPOKES STATE PARK
PROFILE NO. 06

A
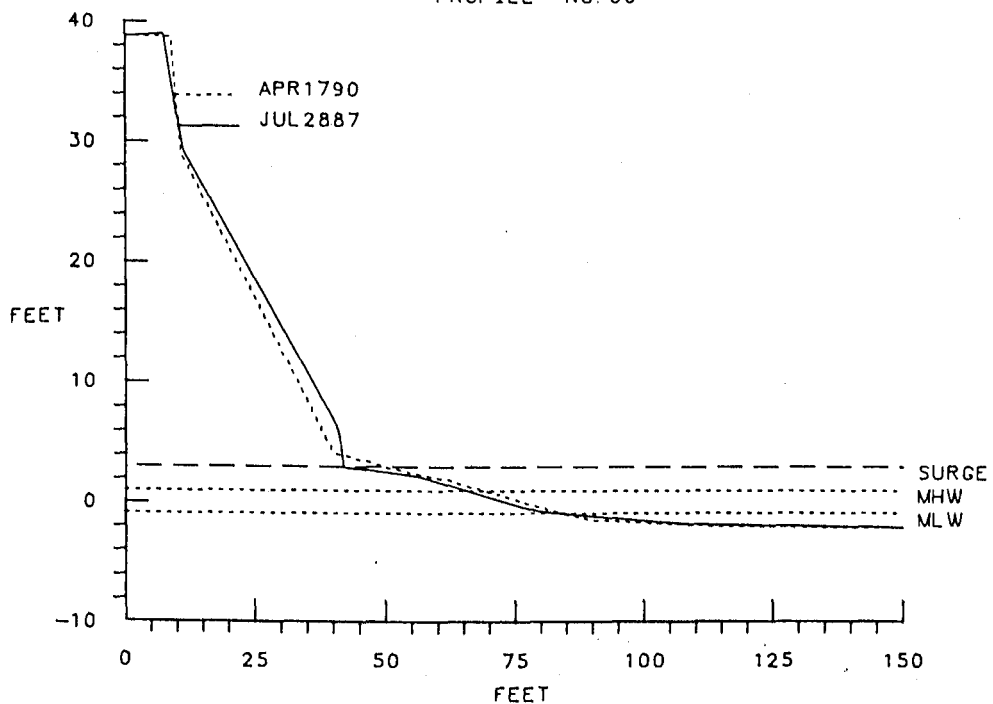
CHIPPOKES STATE PARK
PROFILE NO. 10
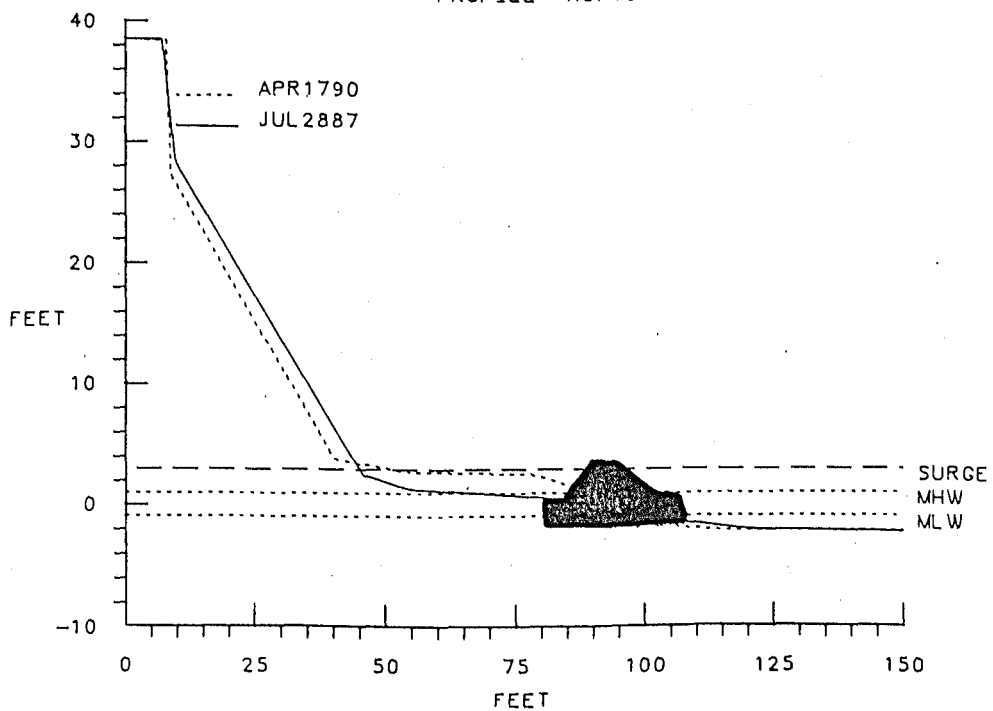
B

Figure 49. Chippokes State Park representative profiles.

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PARKWAY BREAKWATERS
PROFILE NO. 09

A

FEET

20

15

10

5

0

-5

0   50   100   150   200   250
FEET

...... APR0390
——— MAY1385

SURGE
MHW
MLW
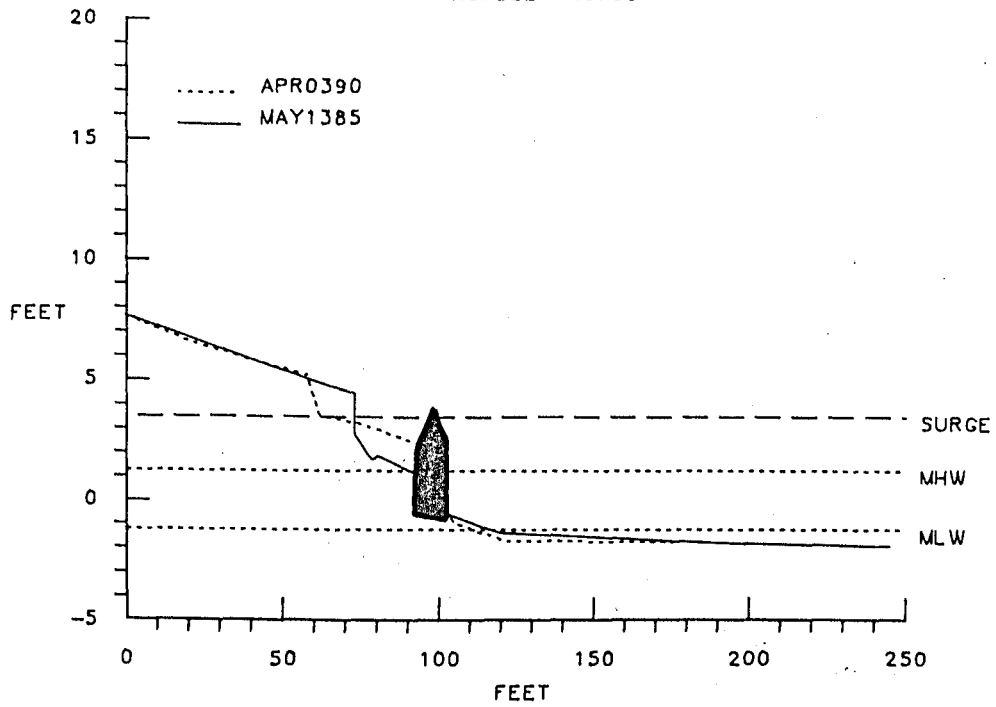
PARKWAY BREAKWATERS
PROFILE NO. 12

B

FEET

20

15

10

5

0

-5

0   50   100   150   200   250
FEET

...... APR0390
——— MAY1385

SURGE
MHW
MLW
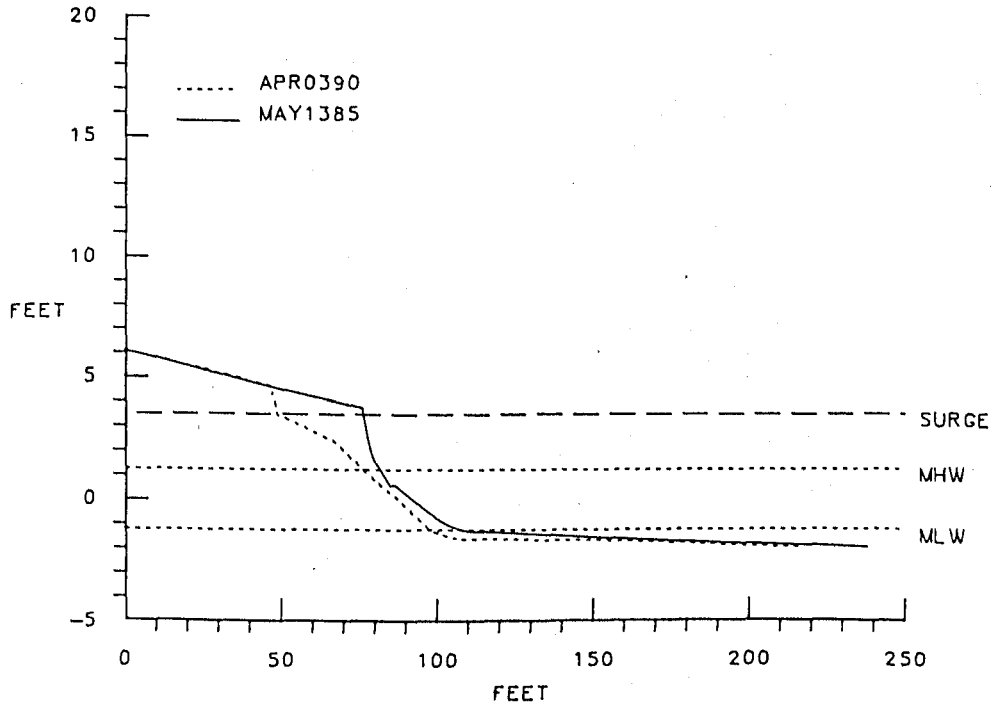
Figure 50.   Parkway Breakwaters representative profiles.
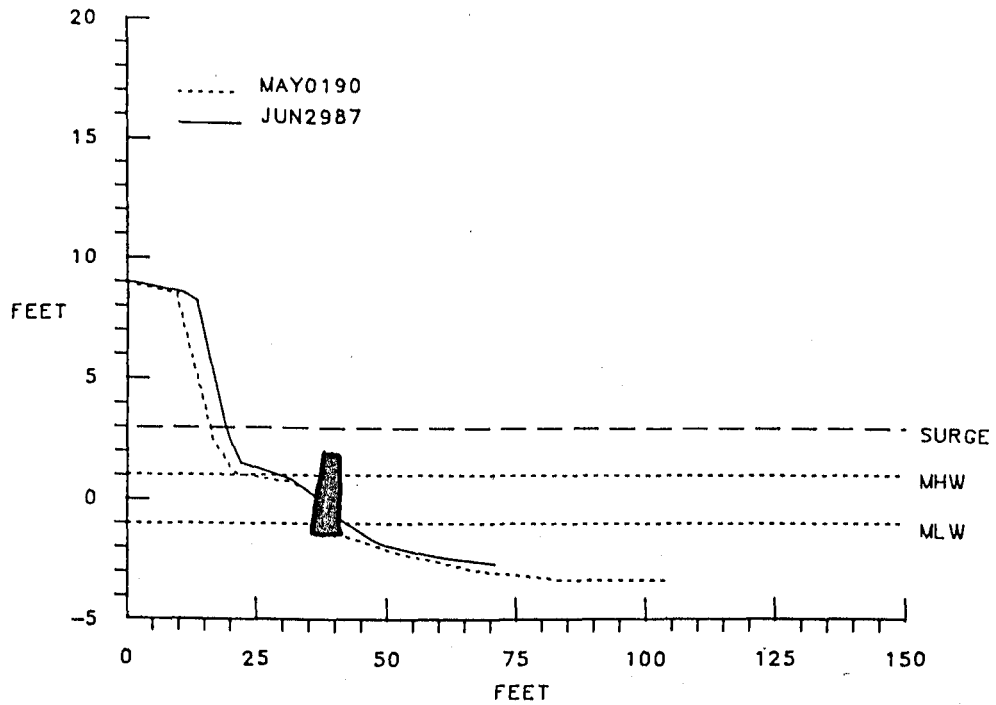
117
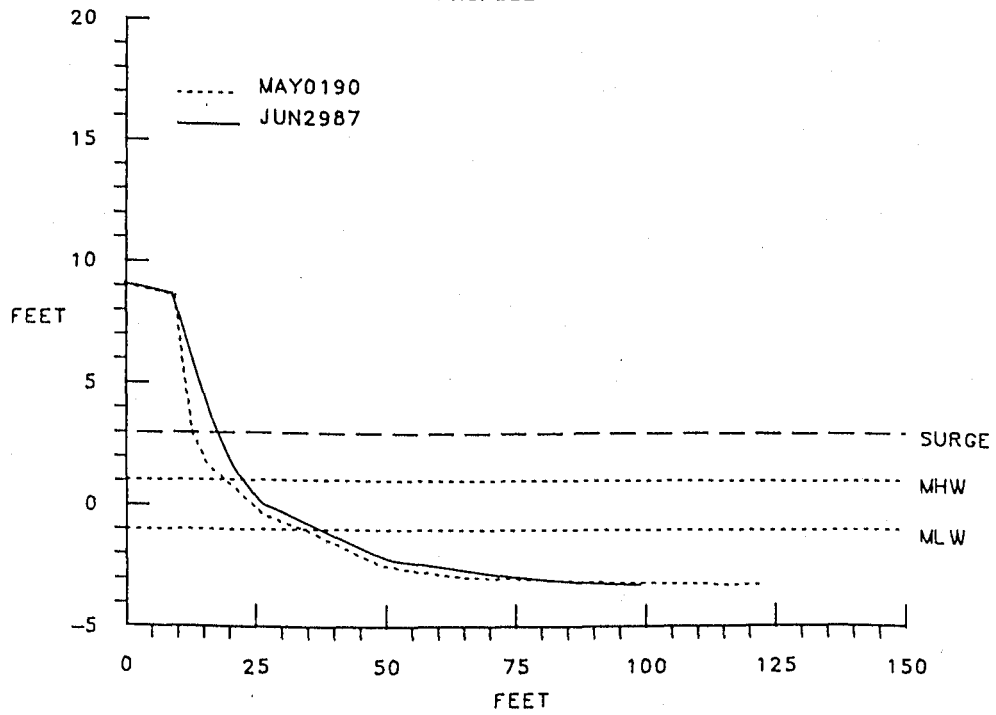
Figure 51. Hog Island Breakwaters representative profiles.

HOG ISLAND BREAKWATERS
PROFILE   NO. 13

C
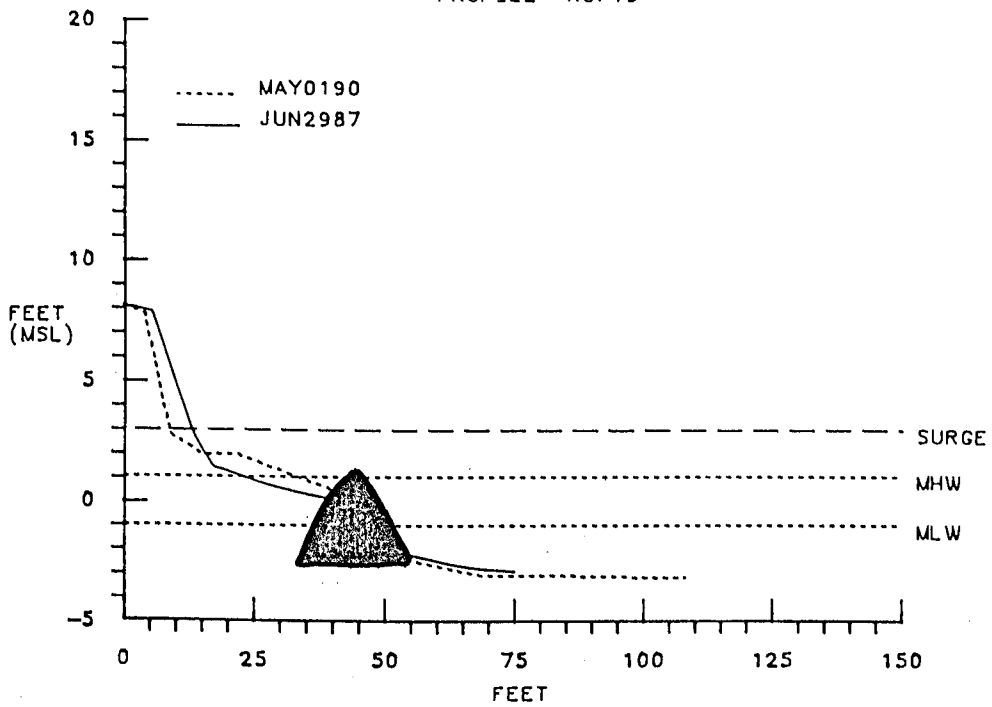
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PROFILE   NO. 16
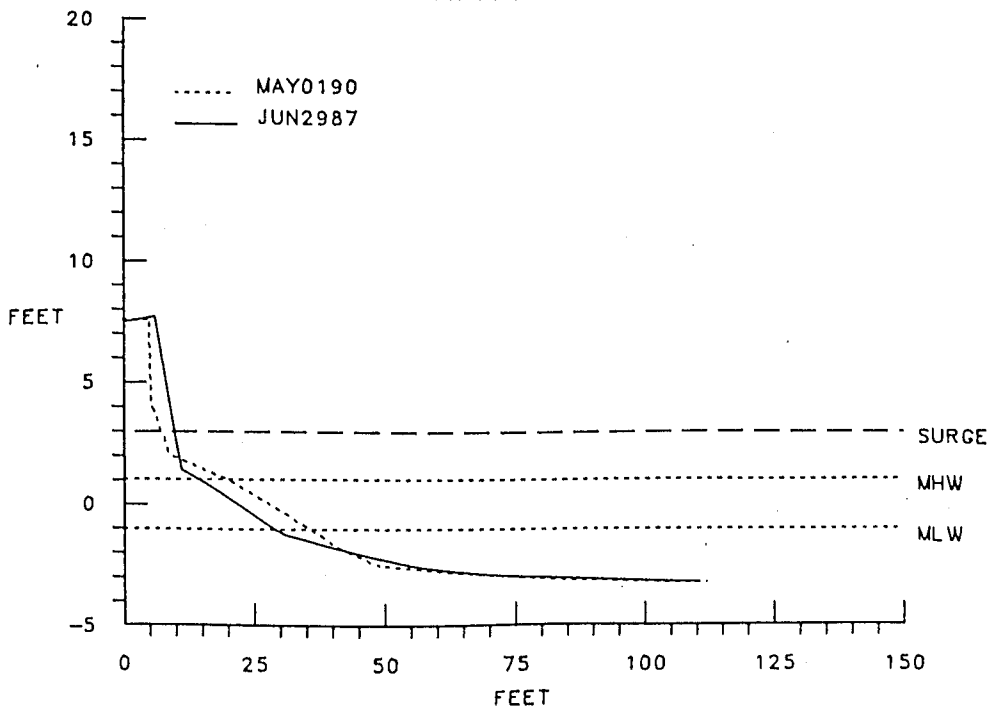
D

Figure 51.   Hog Island Breakwaters representative profiles.

HOG ISLAND BREAKWATERS
PROFILE NO. 23

E
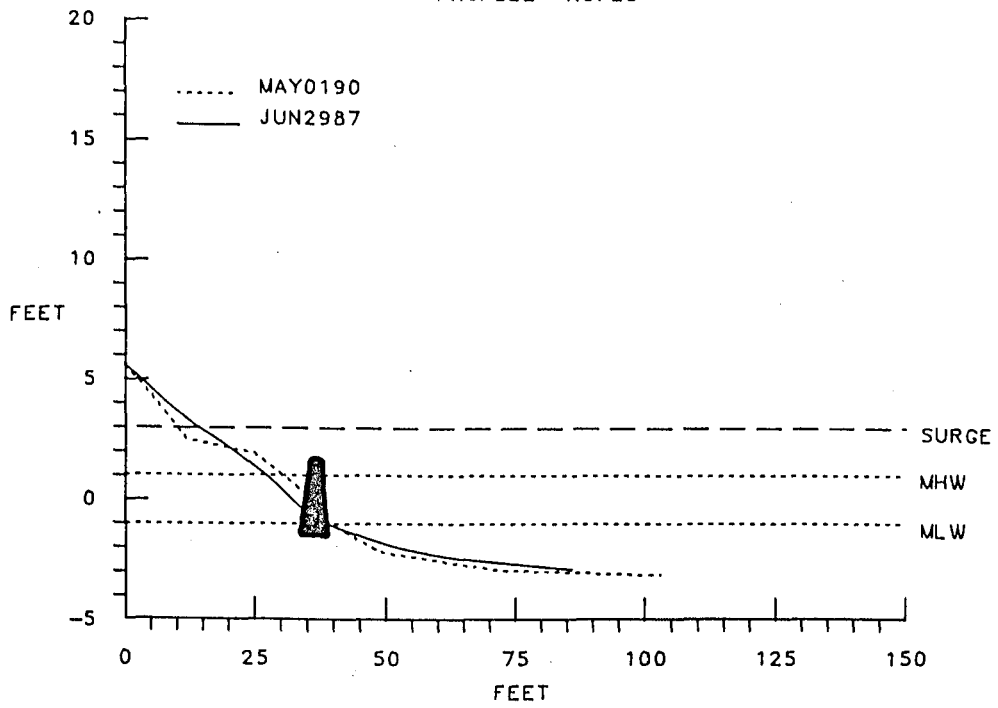
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PROFILE NO. 26
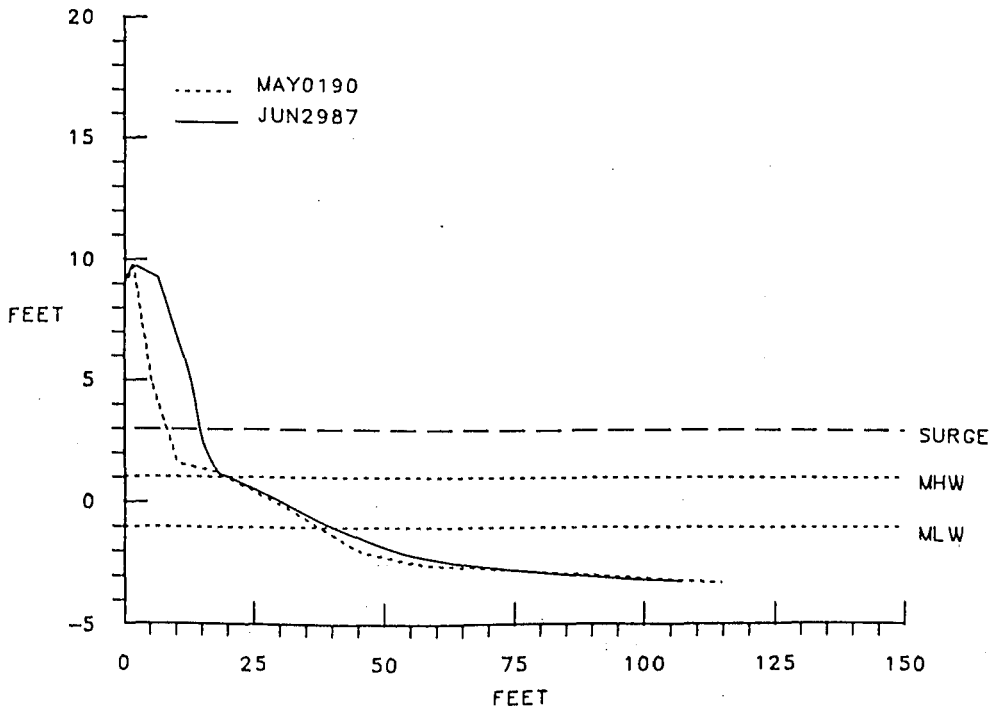
F

Figure 51. Hog Island Breakwaters representative profiles.

120

HOG ISLAND BREAKWATERS
PROFILE NO. 34
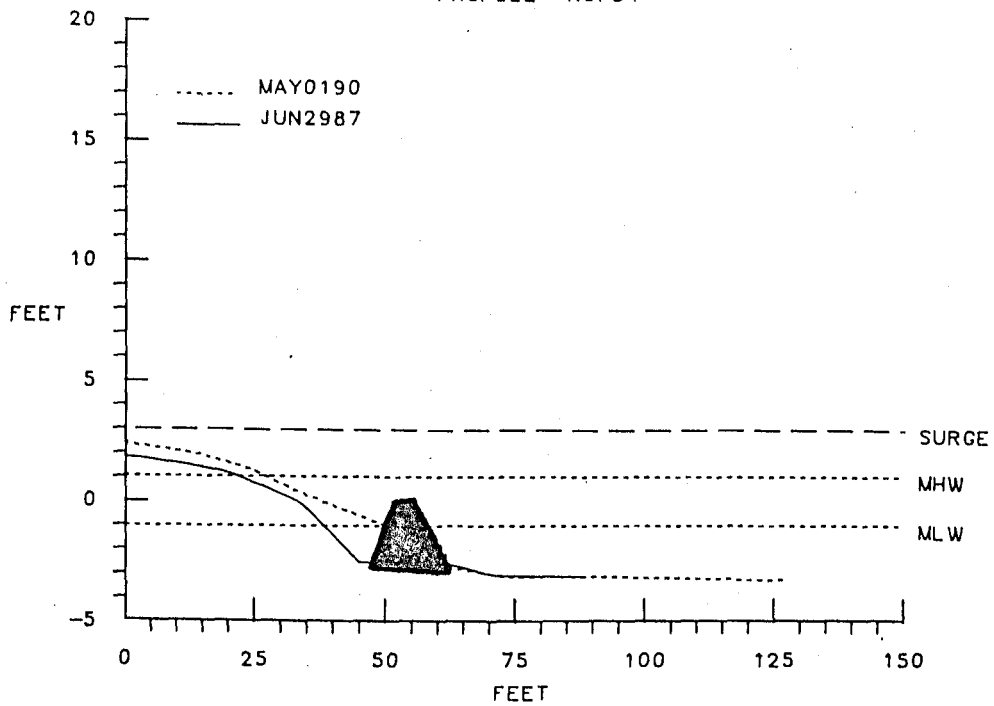
G

HOG ISLAND BREAKWATERS
PROFILE NO. 37
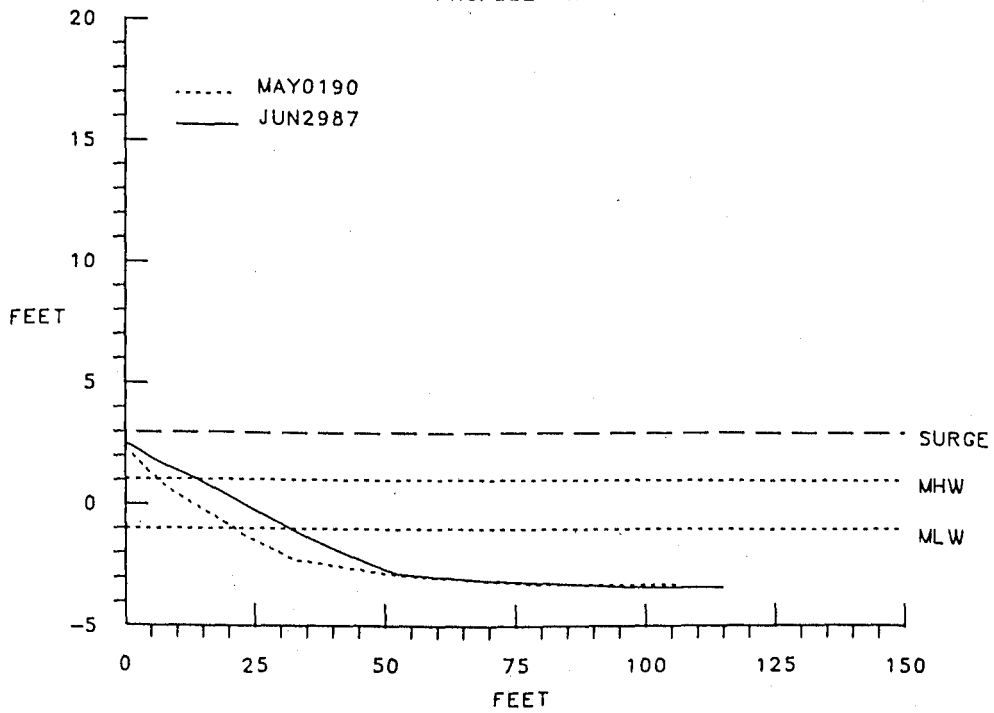
H

Figure 51. Hog Island Breakwaters representative profiles.

121

HOG ISLAND BREAKWATERS
PROFILE NO. 44
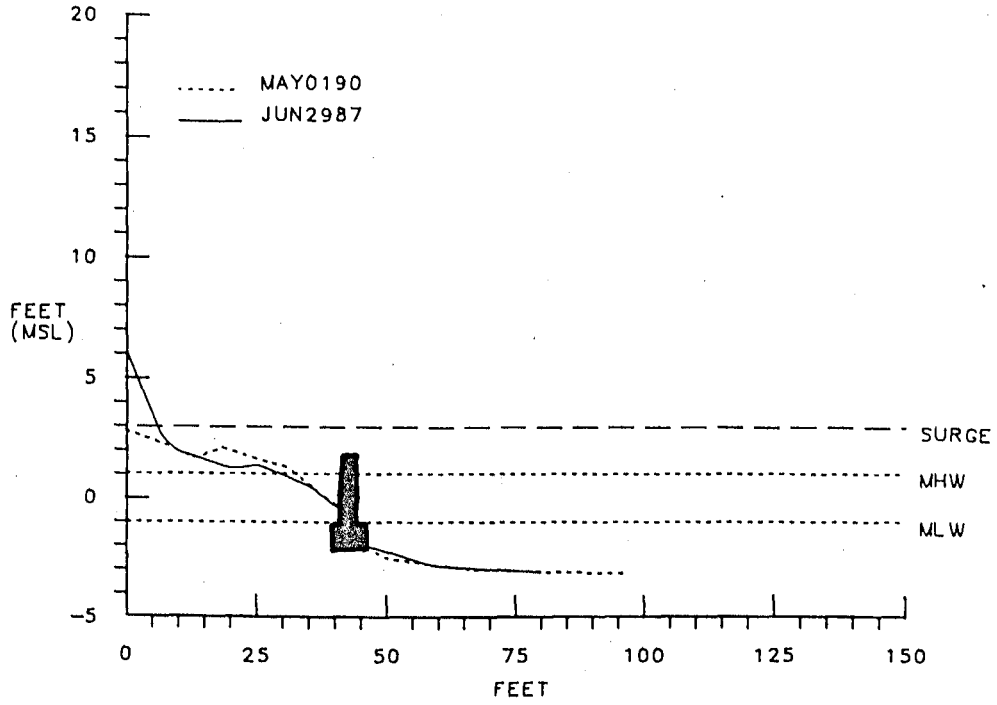
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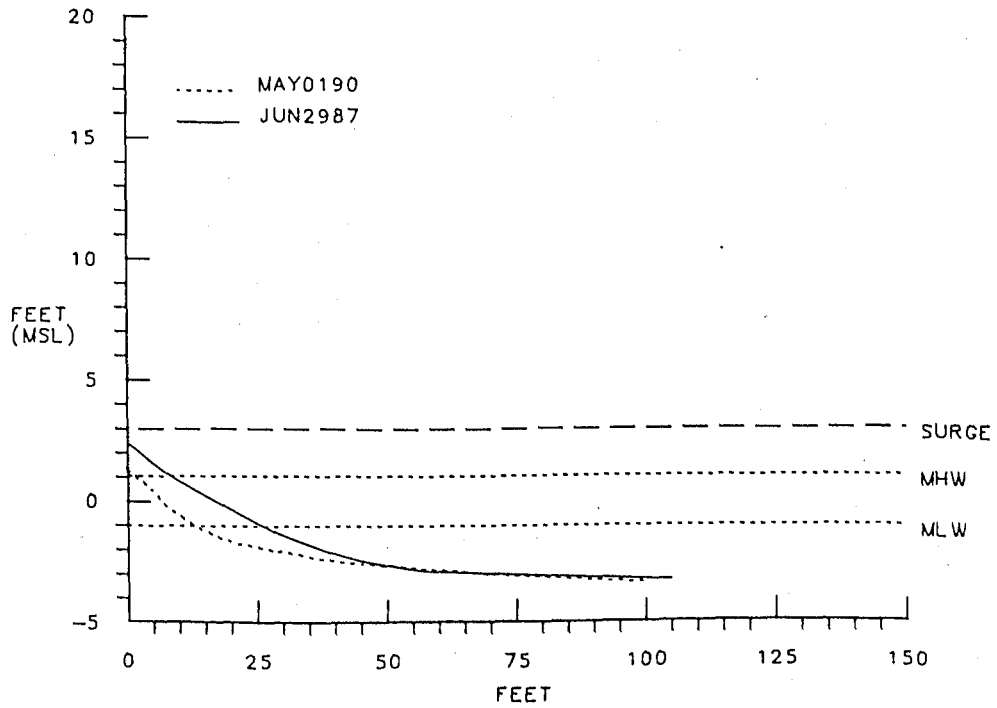
Figure 51. Hog Island Breakwaters representative profiles.

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HOG ISLAND BREAKWATERS
PROFILE NO. 54
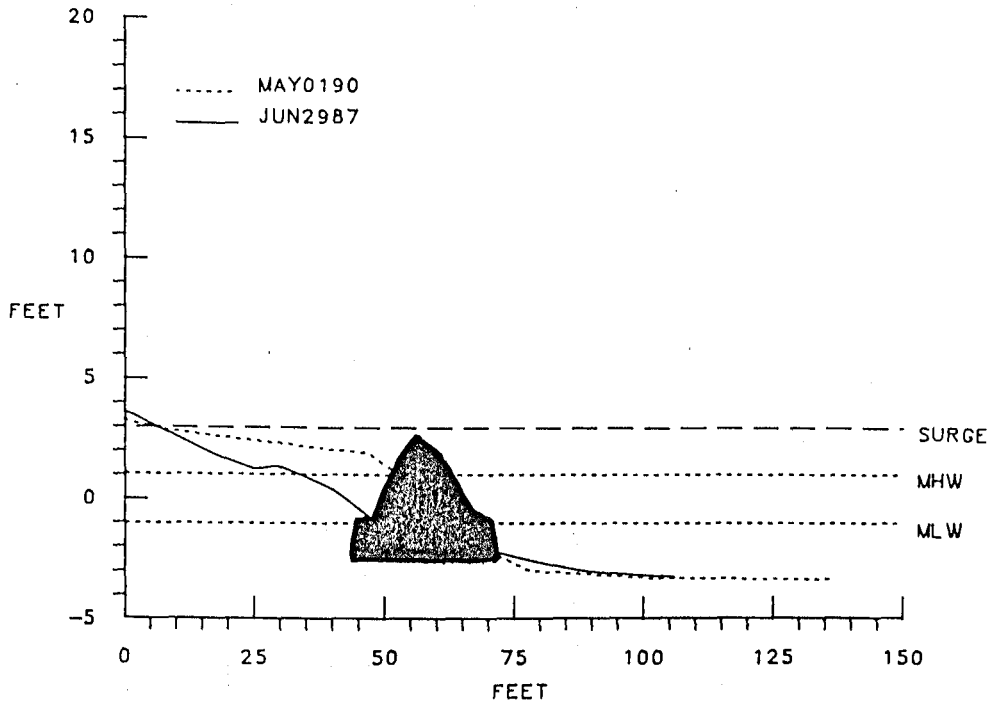
K

SURGE

MHW

MLW

HOG ISLAND BREAKWATERS
PROFILE NO. 57
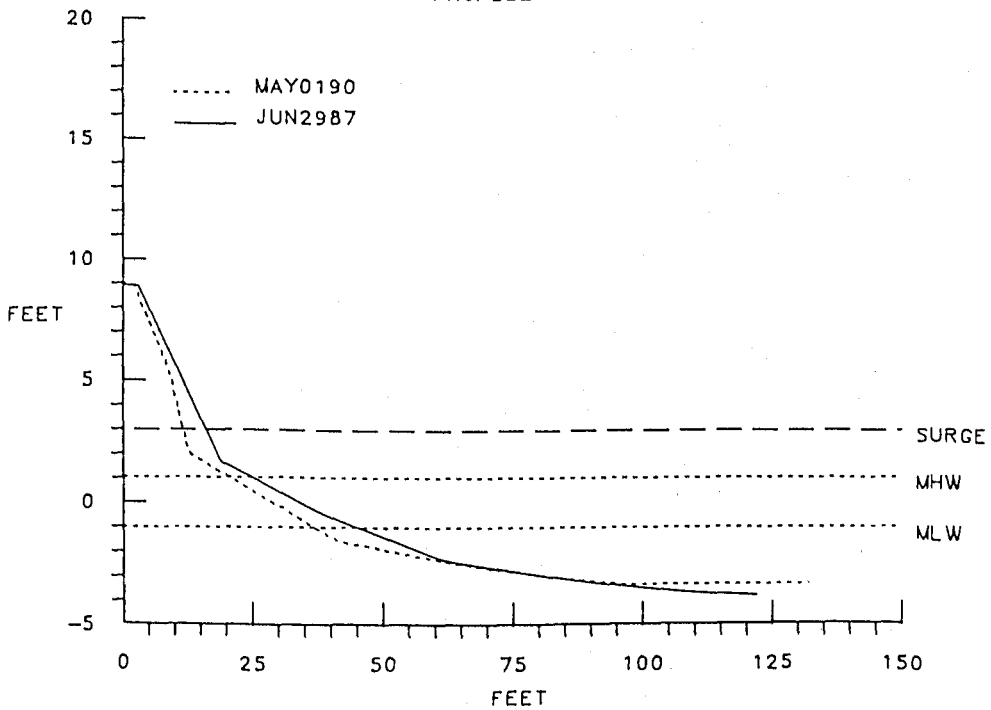
L

SURGE

MHW

MLW

Figure 51.   Hog Island Breakwaters representative profiles.

123
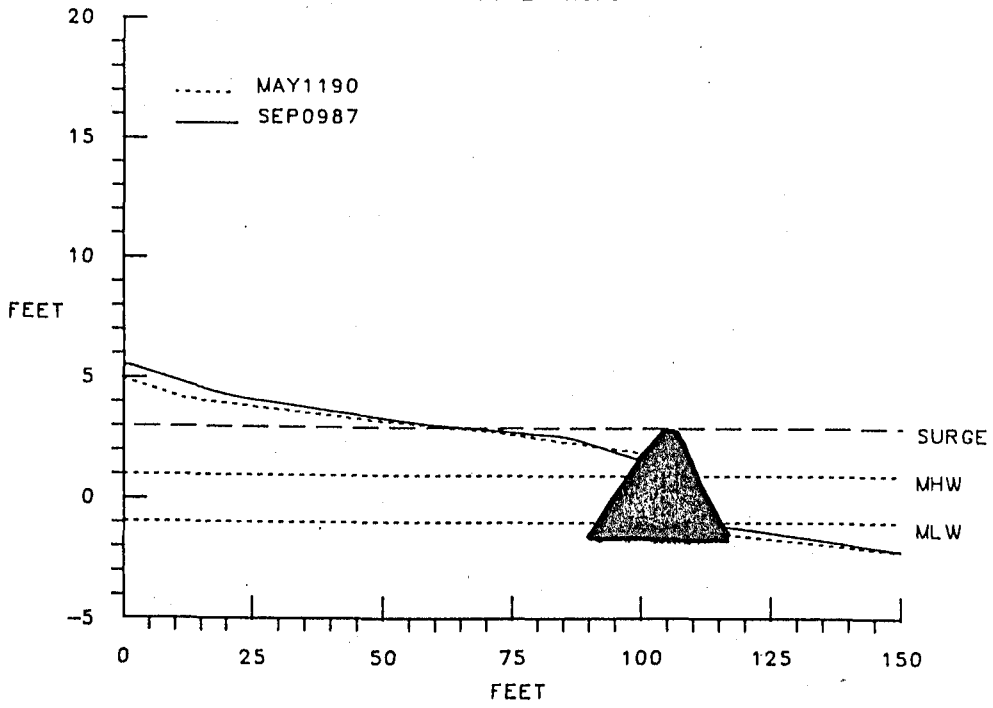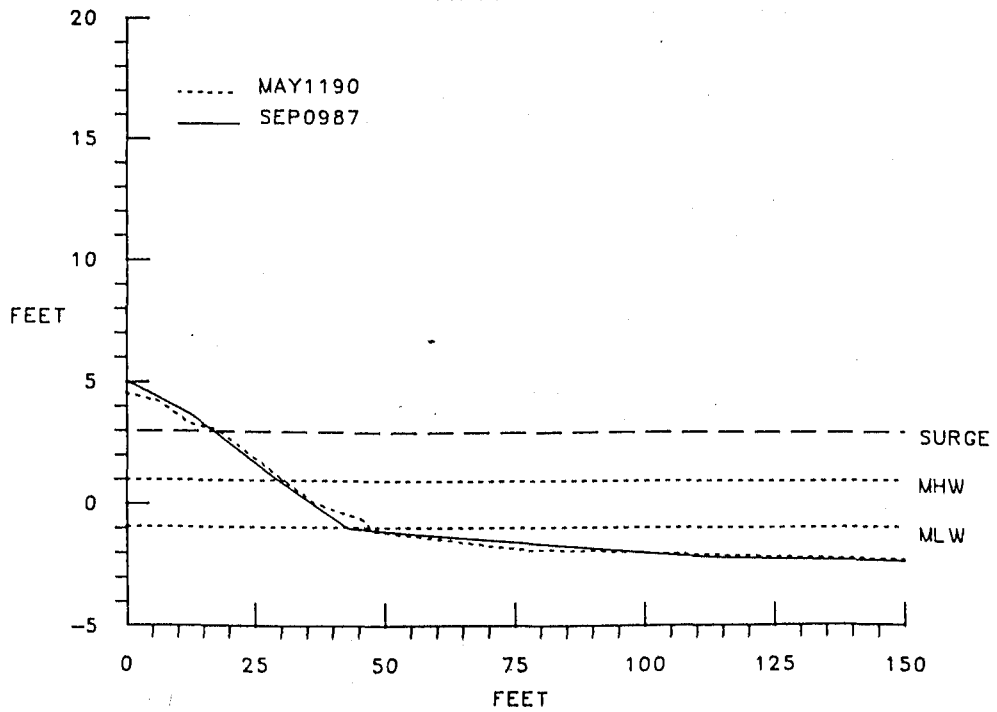
Figure 52.   Drummonds Field representative profiles.

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WALTRIP
PROFILE NO. 04
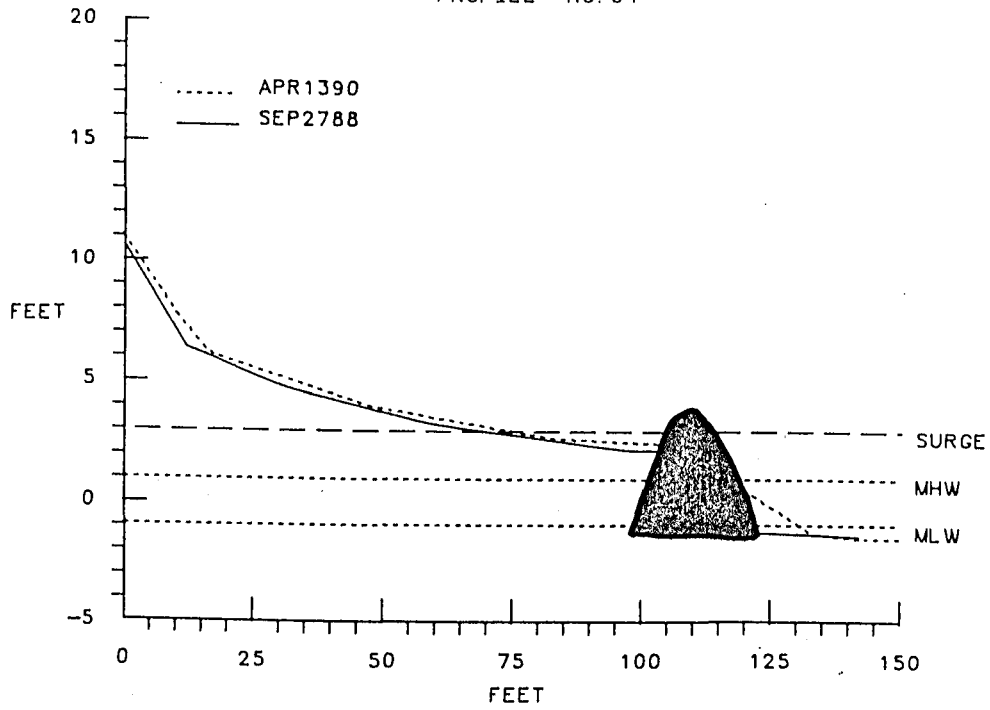
A

WALTRIP
PROFILE NO. 07
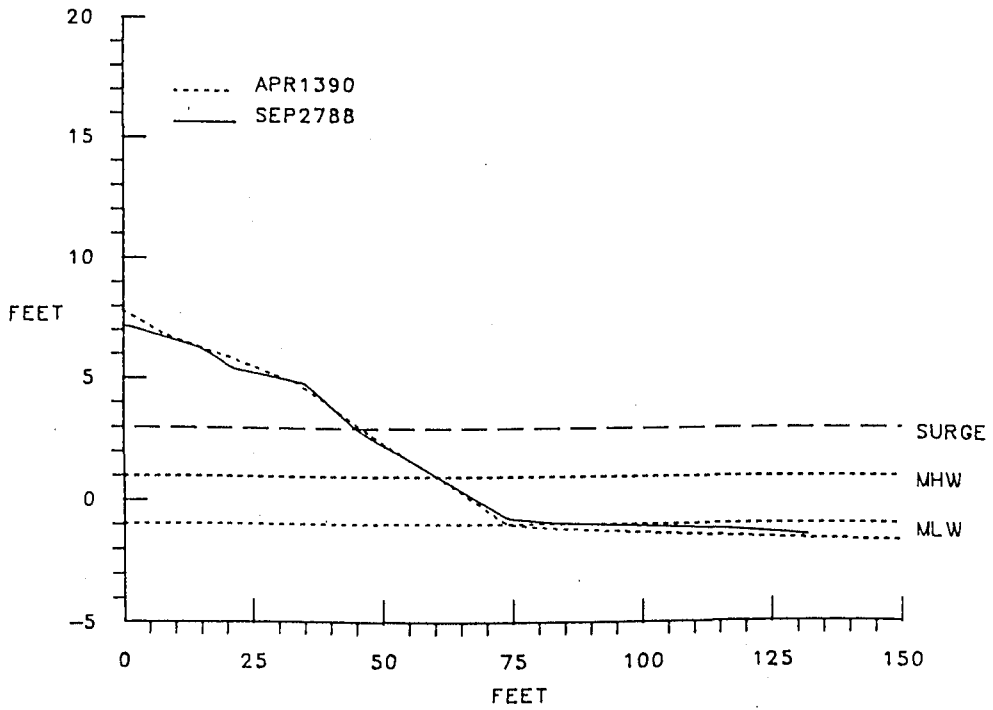
B

Figure 53. Waltrip representative profiles.

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and 28 breakwaters (Figures 54 to 57). Bay C and D at DMF were omitted because they are offset geomorphic features that are inherently stable and comparison to a straight line of offshore structures was deemed inappropriate here. Best fit plots were obtained for Figure 55 (M_b:X_B, r^2 = 0.85) and Figure 56 (M_b:G_B, r^2 = 0.80). Figure 57 (L_B:G_B, r^2 = 0.70) had the third best fit while Figure 54 (B_m:X_B, r^2 = 0.60) had the worst. The low correlation coefficient for Figure 54 results from the virtual lack of backshore width for the embayed beaches at CHP and HI2.

Through a review of storm wave observations and wave analysis through the application of the SMB model, a minimum design table was devised for specific parameters (Table 13). It is meant as a preliminary guide to establish what minimum B_m, F_b and S_e are required for a protective beach under projected water levels and breaking wave heights based on regression results from the breakwater sites. A backshore beach width slope of 1:10 is assumed as an average. Values for wave height and the breakwater parameters are estimated for various storm surges.

In order to further test these design guidelines it was necessary to look at other breakwater intallations around the Chesapeake Bay with adequate backshore areas to allow full bay indentation to acquire additional data points (Table 14). Thus, DMF and WAL were used for comparison. It became evident that directly comparing the B_m and S_e of the sites was not appropriate due the variability of these parameters. This variability stems from different project objectives which range from shore protection to marsh creation to providing recreational beaches. However, site parameters, X_b, M_b, G_B and L_B allows for a comparison of the stable bay beach parameters between the headland breakwater sites.

A series of comparsions among parameters was done using linear regression (Figures 58, 59 and 60). The breakwater offshore position (X_B) correlates fairly well (r^2 = 0.764) with the bay indentation (M_b) (Figure 58). Figure 59 shows the best realtionship, M_b and G_B, with a correlation coefficient of 0.892. A similar "good fit" for M_b and G_B was obtained by
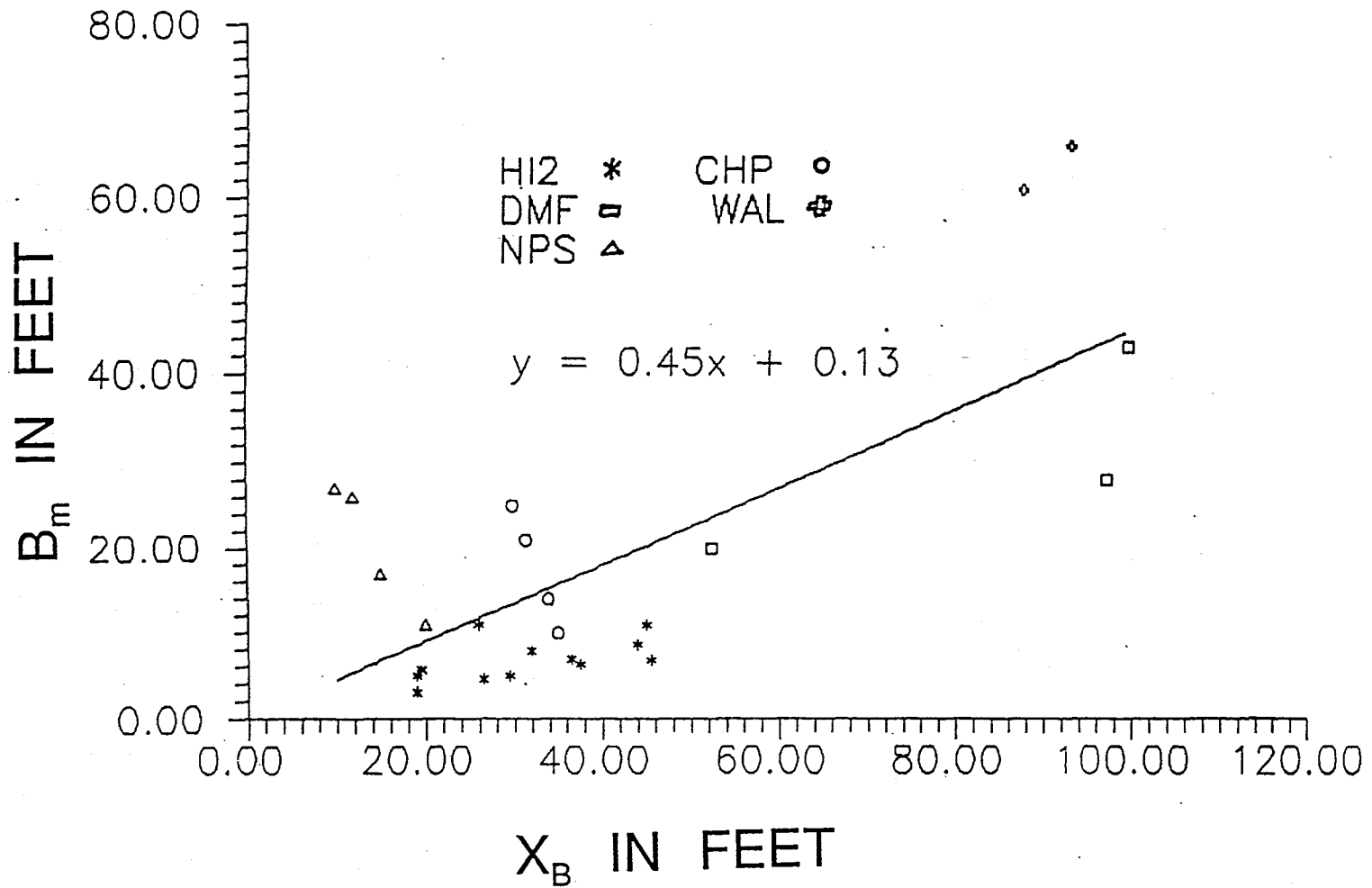
Figure 54. The relationship between X_B and B_m for breakwater/bays at HI2, CHP, DMF, NPS and WAL.
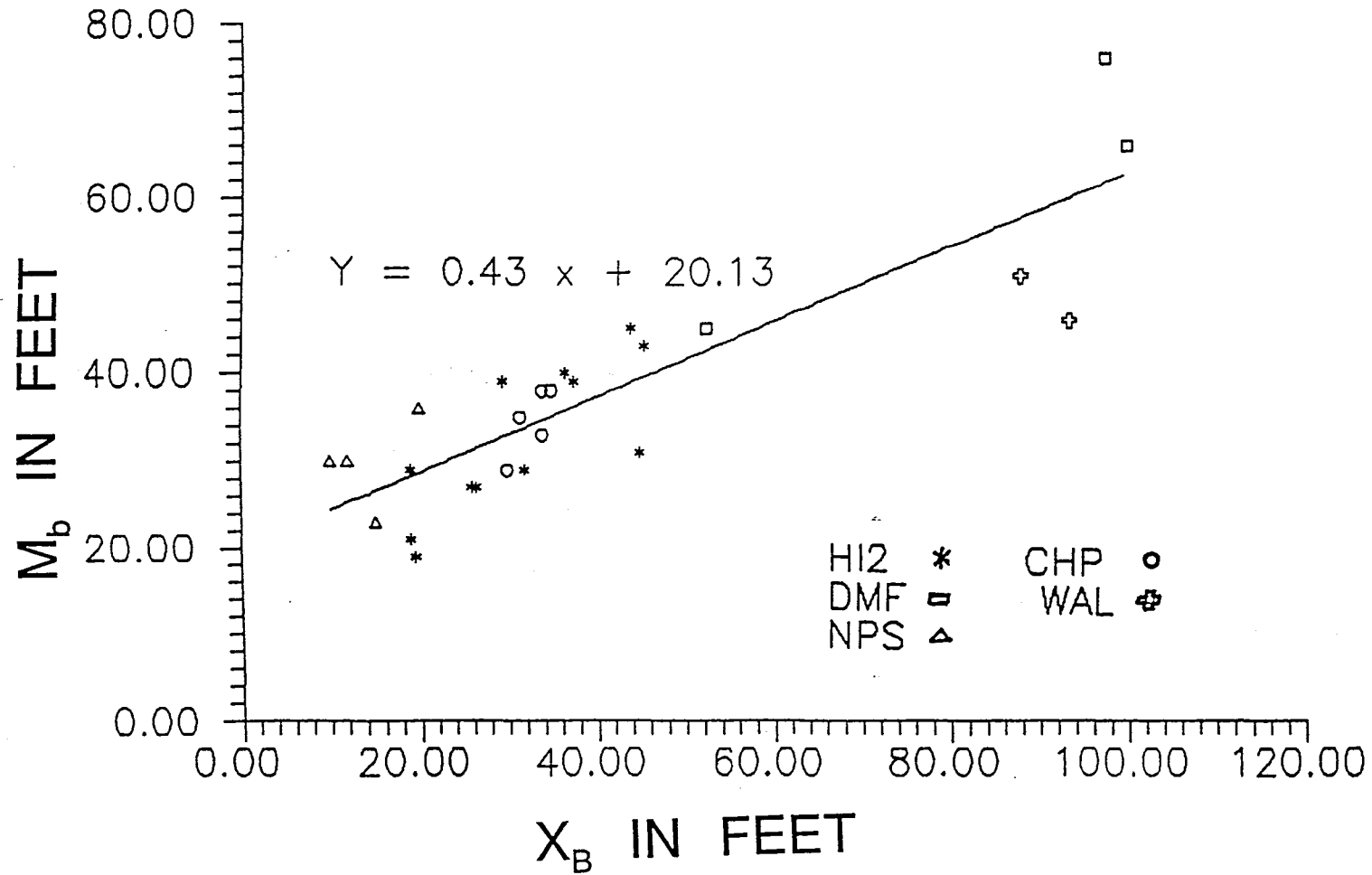
Figure 55. The relationship between X_B and M_b for breakwater/bays at HI2, CHP, DMF, WAL and NPS.
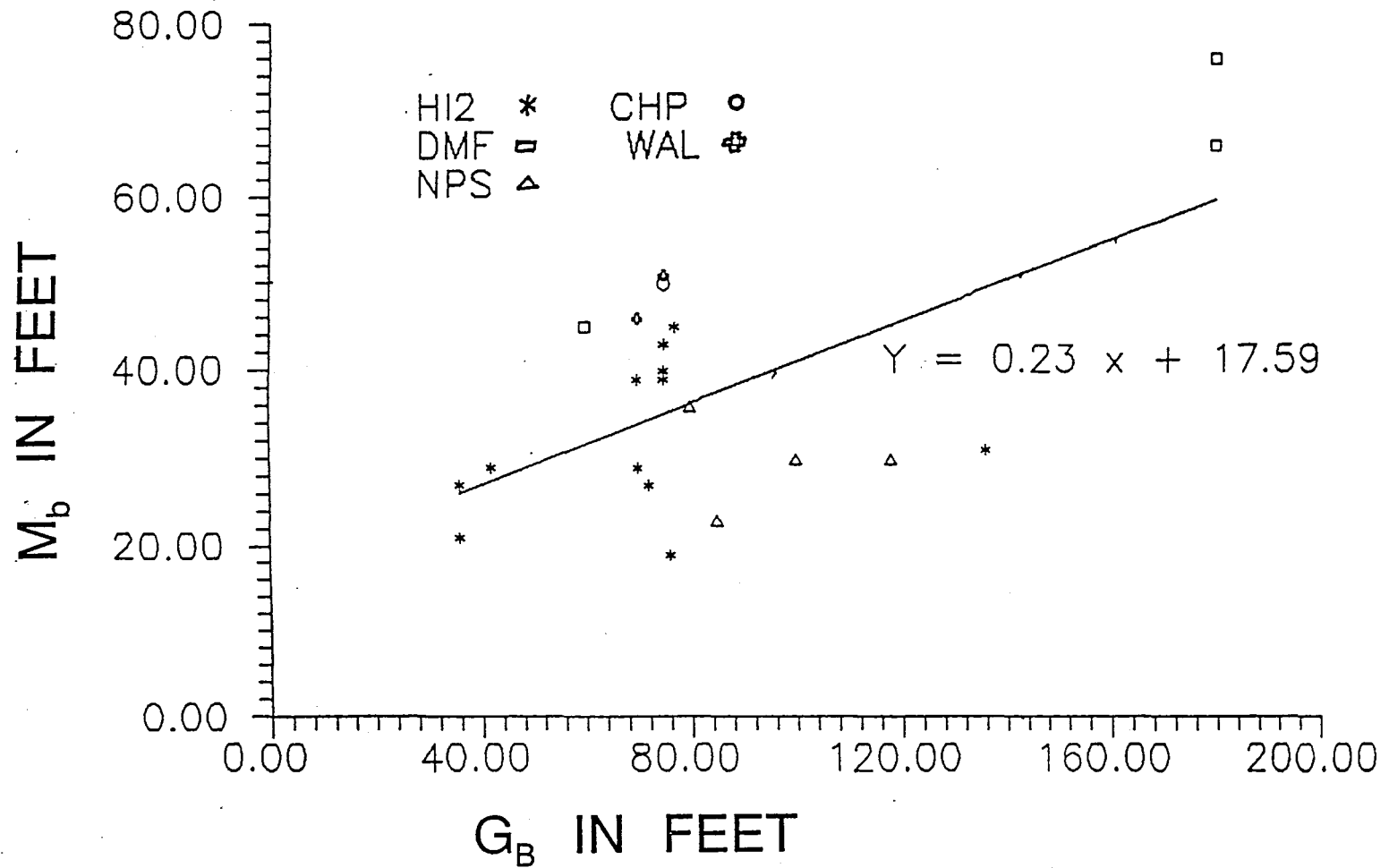
Figure 56. The relationship between G_B and M_b for breakwater/bays at HI2, CHP, DMF, WAL and NPS.
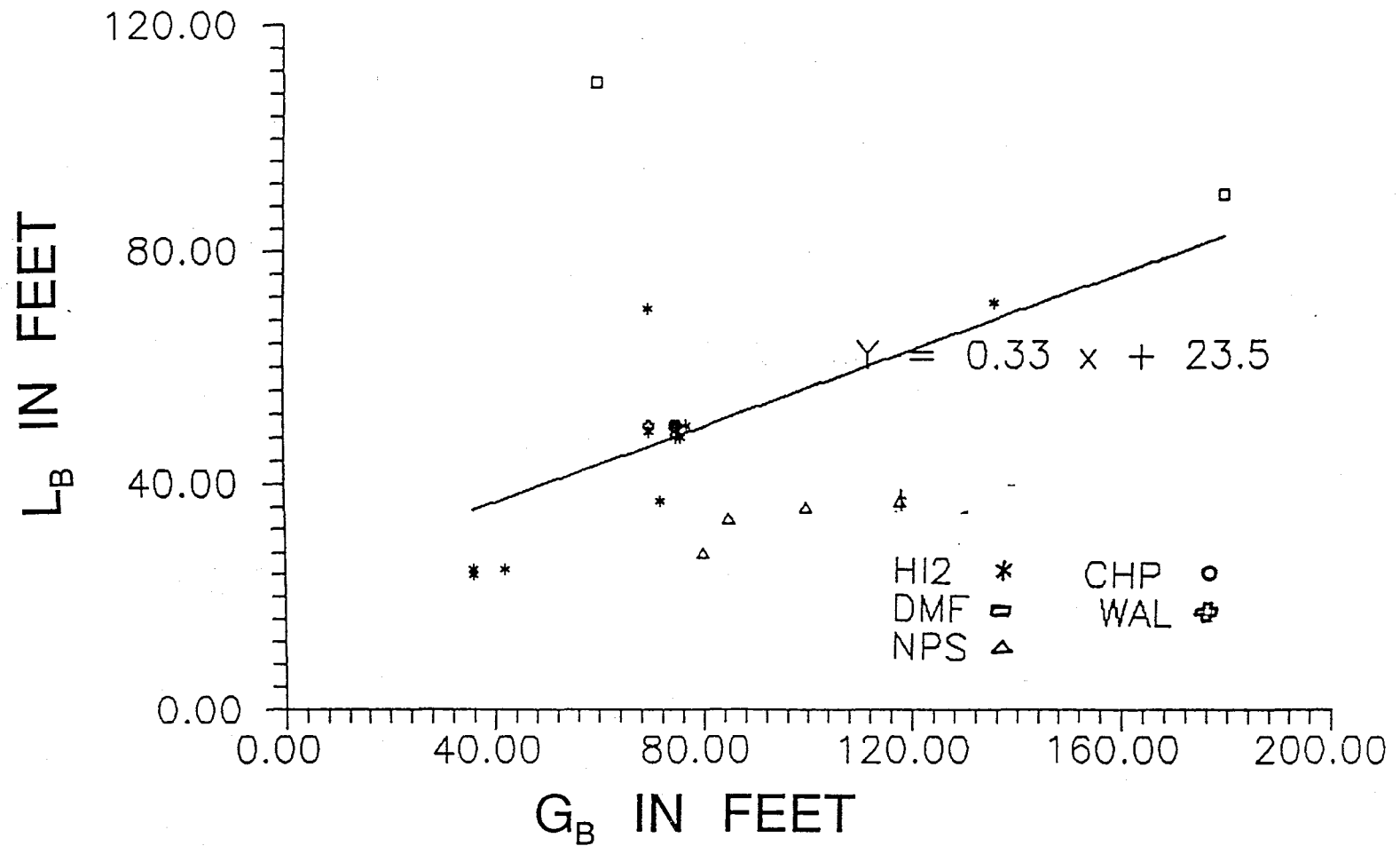
Figure 57. The relationship between G_B and L_B for breakwater/bays at HI2, CHP, DMF, WAL and NPS.

Table 13. Minimum Design Parameters for Medium Wave Energy Shorelines (Average Fetch 1 nm to 5 nm)

Wave Conditions			Breakwater Parameters		
Estimated Wave Height (ft)	Surge (MHW)* (ft)	Frequency* (times/yr)	F_B (MHW) (ft)	B_m+ (ft)	S_e (MHW) (ft)
1.0	1.0	40	1.0	20	2.0
1.5	2.0	2	2.0	30	3.0
2.0	2.5	1/4	2.5	35	3.5
2.5	3.0	1/9	3.0	40	4.0
3.0	3.5	1/25	3.5	45	4.5

* For Hampton Roads area (Boon et al., 1978).

+ Slope 1:10.

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Table 14. Chesapeake Bay Breakwater Sites and Significant Parameters

Site	Breakwater Units (BW) Bay Beaches	L_B ft	G_B ft	X_B ft	M_h ft	B_B ft	Remarks
1. Colonial Beach	Updrift						Designed by U.S.
Potomac River	BW1	200		200			Army Corps of
Westmoreland, VA	Bay A		150		85	130	Engineers using
Installed-1983	BW2	200		200			wave hindcasting
Central Beach	Bay B		150		85	100	and their
Average Fetch =	BW3	200		200			experience at
6.8 nm to E	Bay C		150		85	65	sites in the
	BW4	200		200			Great Lakes.
	Downdrift						Purpose-recreation
2. Drummond Field	Groin						Designed by
James River	BW1	90		100			evaluation of shore
James City, VA	Bay A		180		75	29	morphology through
Installed-Sep 1985	BW2	90		95			historical aerial
Average Fetch =	Bay B		180		65	43	photography.
3.5 nm to SW	BW3	90		105			
	Bay C		220		97	28	Purpose-erosion
	Updrift						control/recreation
3. Aqua-Po Beach	Spur						Designed by
Potomac River	Bay A		105		40	60	evaluation of shore
Stafford, VA	BW1	110		100			morphology on
Installed-Mar 1987	Bay B		160		60	60	existing groin
Average Fetch =	BW2	110		105			field in
3.9 nm to E	Bay C		160		65	65	combination with
	BW3	110		105			log spiral
	Bay D		170		65	60	formulation of
	BW4	110		105			Silvester, 1976.
	Bay E		115		55	55	Purpose-recreation/
	Spur						erosion control
4. Waltrip	Downdrift						Adjacent Drummond
James River	BW1	50		84			Field but with more
James City, VA	Bay A		75		53	49	conservative design
Installed-Oct 1987	BW2	50		92			(i.e. smaller gaps)
Average Fetch =	Bay B		70		46	54	and more beach fill
3.5 nm to SW	BW3	50		95			per foot.
	Groin						Purpose-erosion
							control/recreation
5. Claiborne	Downdrift						Designed by
Eastern Bay	BW1	65		70			evaluation of shore
Talbot, MD	Bay A		95		45	15	morphology which
Installed-Apr 1988	BW2	75		60			was eroding marsh
Average Fetch =	Bay B-Sill	-	-	-	-	-	peats; included
10 nm to SW	BW3	80		75			beach and marsh
	Bay C		90		60	15	implantation.
	BW4	115		65			Purpose-erosion
	Bay D		120		75	15	control/creek inlet
	Spur						stabilization/
							create marsh habitat
6. Christ	Spur						Designed by
Choptank River	Bay A		80		50	15	evaluation of shore
Talbot, MD	BW1	55		90			morphology to
Installed-Jul 1988	Bay B		75		55	20	determine net wave
Average Fetch =	BW2	70		90			approach. Beach
1.8 nm to NE	Bay C		85		55	20	fill from graded 45

Site	Breakwater Units (BW) Bay Beaches	L_B ft	G_B ft	X_B ft	M_h ft	B_B ft	Remarks
Christ (cont'd.)	BW3	75		90			ft bank. Purpose-
	Bay D		60		35	20	erosion control/
	Spur						create marsh habitat
7. Elm's Beach	Updrift						Designed by wave
Chesapeake Bay	BW1	155		165			refraction and bay
St. Mary's, MD	Bay A		175		150	45	shape analyses
Installed-Oct 1988	BW2	155		165			from Silvester,
Average Fetch =	Bay B		175		150	45	1976; and previous
16 nm to NNE	BW3	155		165			sites.
	Revetment						Purpose-recreation/
							erosion control/
							create marsh habitat
8. St. Catherines	Spur						Designed by wave
Island	Bay A		95		80	30	refraction analysis
Potomac River	BW1	100		75			procedures from
St. Mary's, MD	Bay B		95		70	70	Hsu et al., 1989.
Installed-Mar 1989	BW2	100		80			Purpose-to retain
Average Fetch =	Bay C		95		60	70	dredge material
3.6 nm to NW	BW3	100		60			from nearby channel
	Bay D		80		60	80	project/erosion
	Spur						control
9. Ware	Spur						Designed after
Choptank River	Bay A		65		45	35	Christ project.
Talbot, MD	BW1	60		70			
Installed-Sep 1989	Bay B		65		45	40	
Average Fetch =	BW2	60		70			
1.7 nm to E	Bay C		65		45	35	Purpose-erosion
	BW3	60		70			control/create
	Groin						marsh habitat
10. Dietrick	Spur						Design procedures
Choptank River	Bay A		75		50	40	after Ware and
Talbot, MD	BW1	65		70			Christ
Installed-Oct 1989	Bay B		95		60	40	
Average Fetch =	BW2	65		75			
1.8 nm to SE	Bay C		95		65	35	
	BW3	65		80			
	Bay D		120		50	25	Purpose-erosion
	BW4	55		-			control/create
	Downdrift						marsh habitat
11. Rock Hall	Spur						Designed by wave
Chesapeake Bay	Bay A		83		65	80	hindcasting, wave
Kent, MD	BW1	60		85			refraction analysis
Installed-Jun 1990	Bay B		83		65	80	and application of
Average Fetch =	Spur						procedures from Hsu
15 nm to SW							et al., 1989.
							Purpose-recreation/
							create marsh habitat
12. Willoughby Spit	Groin						Beach designed using
Chesapeake Bay	Bay A		450		240	60	VIMS wave gauge data
Norfolk, VA	BW1	200		200			and procedures from
Installed-Aug 1990	Bay B		310		230	55	Hsu et al., 1989.
Average Fetch =	BW2	150		120			Purpose-recreation/
33 nm to N	Updrift						erosion control

Figure 58. The relationship between X_B and M_b for breakwater/bays at D-W (DMF and WAL), ELB, WCD (WAR, CHR and DRK), AQP, STC, CLA, RXH, CLB and WIL.

Figure 59. The relationship between G_B and M_b for breakwater/bays at D-W (DMF and WAL), WCD (WAR, CHR and DRK), AQP, CLA, STC, RXH, CLB, WIL and ELB.

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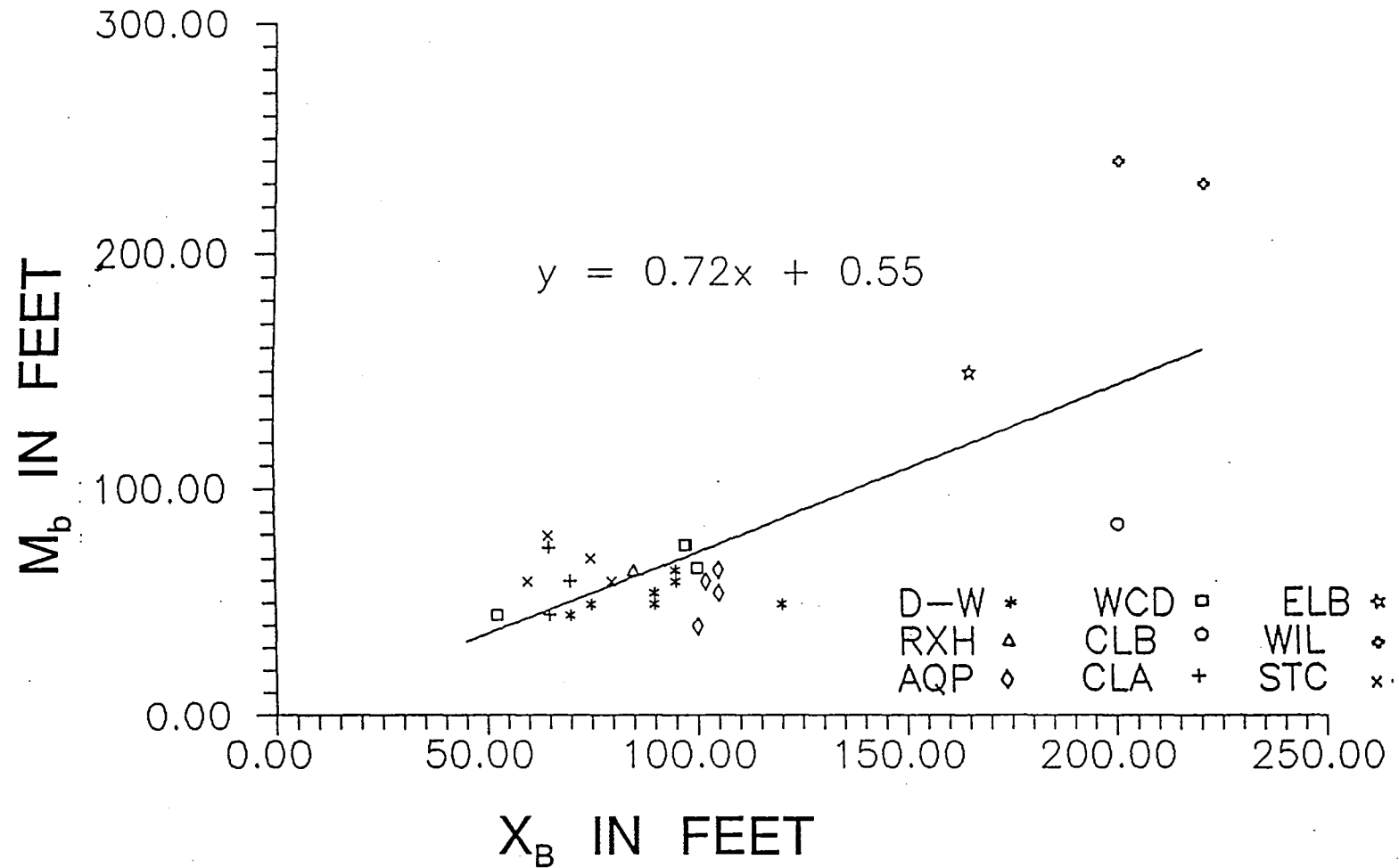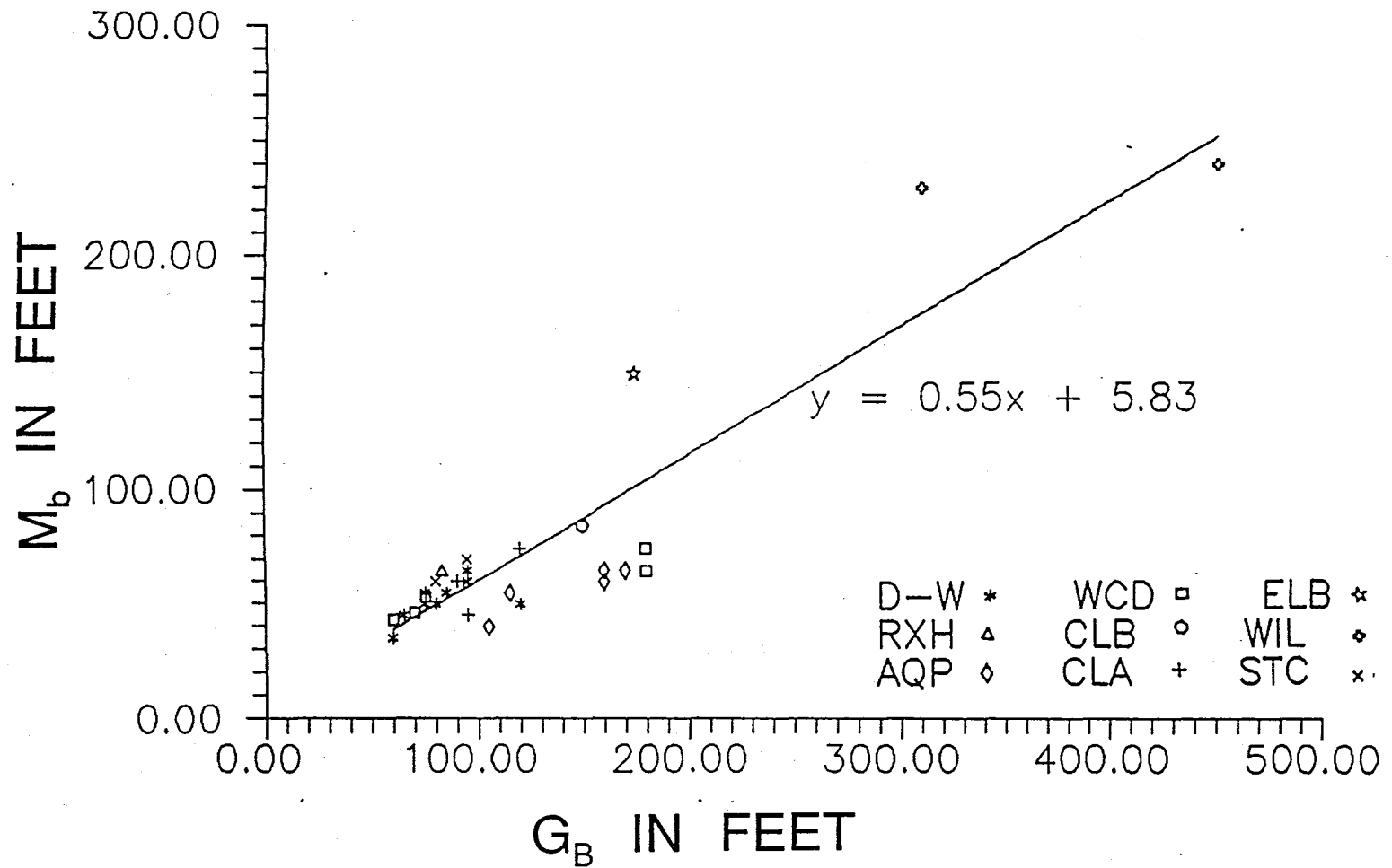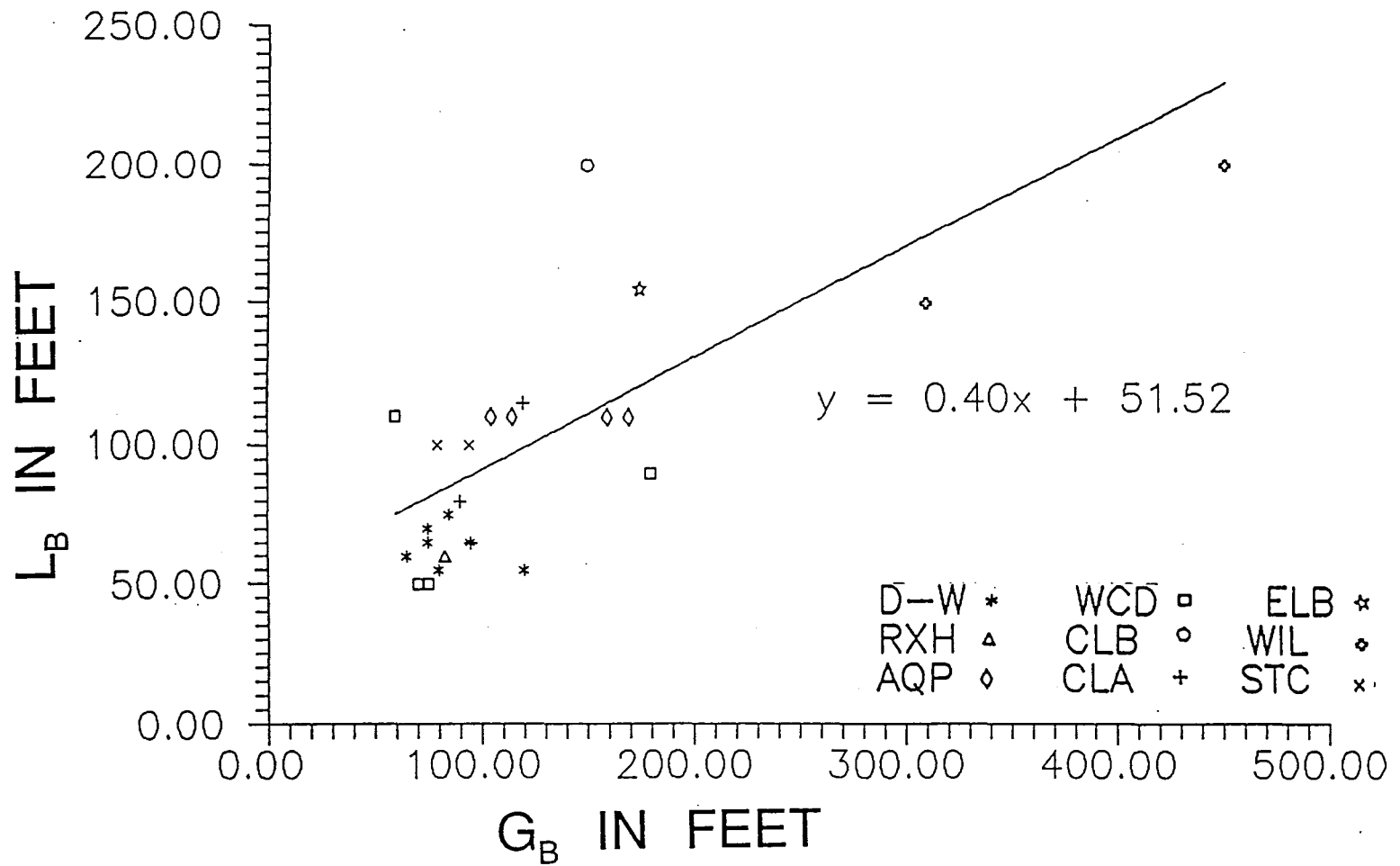Figure 60.   The relationship between G_B and L_B for breakwater/bays at D-W (DMF and WAL), WCD (WAR, CHR and DRK), AQP, CLA, STC, RXH, CLB, WIL and ELB.

Berenguer and Enriquez (1988) for the Spanish pocket beaches on the Mediterranean Sea. However, the average ratio of $M_b:G_B$ for the Chesapeake Bay sites is 1:1.65 whereas the average ratio of $M_b:G_B$ for the Spanish beaches is about 1:0.75. This shows the Spanish pocket beaches require a more conservative design (i.e. smaller gap vs. bay depth).

The breakwater length relationship is less clear (Figure 60), the correlation coefficient is only 0.634. The L_B can vary depending on the project goals. Longer, higher breakwaters store more sand and may provide more wave attenutation. These conditions are perhaps most needed in higher wave climates and/or recreational beaches. Shorter L_B can be used where beach fill and tombolos requirements are less. This condition may occur on sites with lesser wave climates or with the need for less shore protection (i.e. farm land).

By using Table 13 as guide, one can address the dimensions of X_B to M_b and G_B to M_b and then G_B to L_B. For example, a B_m of 35 feet is desired for protecting a potential shore site against a 1 in 4 year storm using breakwaters. By trial and error the relationship of X_B to M_b would be investigated. By arbitrarily selecting X_B = 100 feet and using the best fit equation in Figure 58, M_b = 72 feet. A value of 107 feet is obtained when $B_m + M_b$. Since B_m is defined as the distance from post-construction MHW and the base of the graded bank (BOB), the position of BOB can be adjusted landward during construction if necessary.

Further values for the above scenario are obtained from Figure 59 where the G_B = 121 feet and Figure 60 where L_B would be 99 feet. The ratio of $M_b:G_B$ = 1:1.7. This procedure is only a guide for the empirical evalutation used in breakwater design for the Chesapeake Bay.

3. Beach Planforms

Beach planforms of the bays at the breakwater sites surveyed for this study are generally symmetrical and semi-circular. This is a function of breakwater gap and wave direction (Silvester, 1974). Onshore waves (shore normal) and narrow gaps will create circular bays such as those at CHP,

WAL HI2, and NPS Bays C and D. Wider gaps will allow greater wave energy into the bay shore. If the net wave climate is shore normal a flattened but symmetrical bay is formed as at DMF Bays A and B. If waves are oblique to a wider gap situation a spiral bay is formed (NPS Bays A and B).

At NPS the net seasonal wave climate is oblique to the breakwater system. Gaps of 118 and 100 feet for Bays A and B allow sprial bay formation whereas gaps of 85 and 80 feet restrict the wave passage and form circular emabayments. When the wave length exceeds a certain gap width the wave diffracted at two points by opposing breakwater tip acts independently and a spiral bay will form if the wave is oblique. At this point the minimum gap that will "filter" incoming oblique wave trains appears to be about 80 feet for medium energy sites. The result of this relationship means that bay beach sands will tend to shift less in a more closed embayment.

4. Model Tombolos

There are several numerical models which have attempted to predict shore planform changes in the lee of offshore breakwaters (refer to section IIA). These models appear to fall short of being able to predict shore change with attached beach salients or tombolos. A numerical model was specifically developed during this study for that purpose by Dr. K.D. Suh called Model Tombolos (Suh, in press). This is a one-line model that predicts shore planforms behind gapped offshore breakwaters and includes tombolo formation. Details of Model Tombolos are discussed in Appendix B.

Model Tombolos was applied to the simulation of the shoreline change near the Chippokes State Park breakwaters for the first eight months after construction. The initial shoreline response to the breakwaters was reported during June 1987 (Hardaway et al., 1988). The shoreline changes were for September 1987 and February 1988.

There was no available wave data near the project site for the simulation period nor was the digital wind data available from the Surry

Power plant. So, the input wave data at the location of the breakwaters were hindcasted from the wind data at Norfolk, Virginia, which is located about 35 miles southeast from the project site. The wind data includes speed and direction at three hour intervals. Assuming that the wave direction corresponds to the wind direction in the middle of the James River, the Chippokes State Park breakwaters are affected by the wind blowing within the directional window fanning from 320° to 20° of direction measured from true north. It is also assumed that in order to generate the wave field that affects the shoreline change, wind should blow for more than nine hours within the directional window. The average wind speed and direction for the period are calculated by vector-averaging the observations given every three hours. A constant wave field corresponding to the averaged wind speed and direction is assumed for the period.

The significant wave height and period at the location of the breakwaters area computed using the method of Kiley (in press), which is essentially a shallow water estuarine version of the quasi-empirical/ quasi-theoretical wind wave prediction model developed by Bretschneider (1966) and modified by Camfield (1977). As disscussed in Section IIIB, this method includes the variation in water depth, the effect of the surrounding land forms on the computation of effective fetch, wave growth due to wind stress and wave decay due to bottom friction and percolation. The effect of refraction is not included in the computation of wave height since the method assumes shore-normal wind direction. The wave angle at the location of the breakwaters is determined by Snell's Law assuming that the offshore bottom contours are straight and parallel to the x-axis and the deep water wave direction (at the center of the river) is the same as the wind direction. The x-axis of the shoreline prediction model that corresponds to the baseline of the beach profile measurement (Hardaway et al., 1988) is tilted by 9° counterclockwise from the E-W axis. The computed data and the numerical model are found in Appendix B.

Figure 61 shows the measured (Figure 61a) and the computed (Figures 61b and 61c) shoreline changes. Figure 61b is the result with varying longshore sediment transport coefficients, whereas Figure 61c is that with constant coefficients. In Figures 61b and 61c, the shoreline positions for June 1987, September 1987 and February 1988 are indicated by a solid line, a dashed line, and a long-dashed line, respectively.

In September 1987 the field measurement shows the formation of double salients behind each breakwater and in February 1988 the double salients coalesced into a single tombolo. As can be seen in Figure 61b, the formation of double salients in September 1987 is predicted by the model using varying longshore transport coefficients, even though it is not so clear as in the field measurement. This feature is hard to observe in Figure 61c which is the result using constant coefficients.

The final shoreline positions in February 1988 are almost identical between the two model results. The model predicted smaller tombolos and more prominent erosion behind the gaps compared with the field measurements. This may be due to the addition of sediment to the system by runoff and bank erosion as reported in Hardaway et al. (1988). It should be noted that the onshore-offshore sediment transport was assumed to be zero in the present model.

C.   Headlands

1. Bay Planforms

Shore response within the embayments between headlands on open coasts is, in part, a function of incident wave direction (Silvester, 1974). In the case of the three designated headland sites of the Chesapeake Bay Shoreline Study, this would be the net incident wave direction because of the seasonality of the fetch-limited wave climate. In the first year of this study, Silvester's model of stability criteria for equilibrium shaped bays for comparison in the Virginia estuaries was investigated (Hardaway et al., 1988). It did not allow the entire bay shape to be evaluated but instead only supplied the maximum bay indentation distance for bay beach
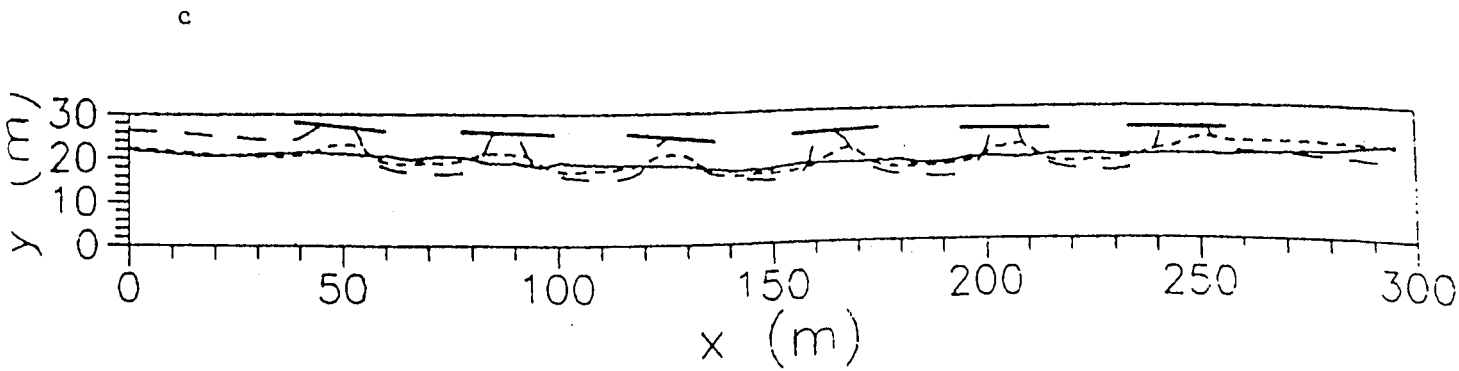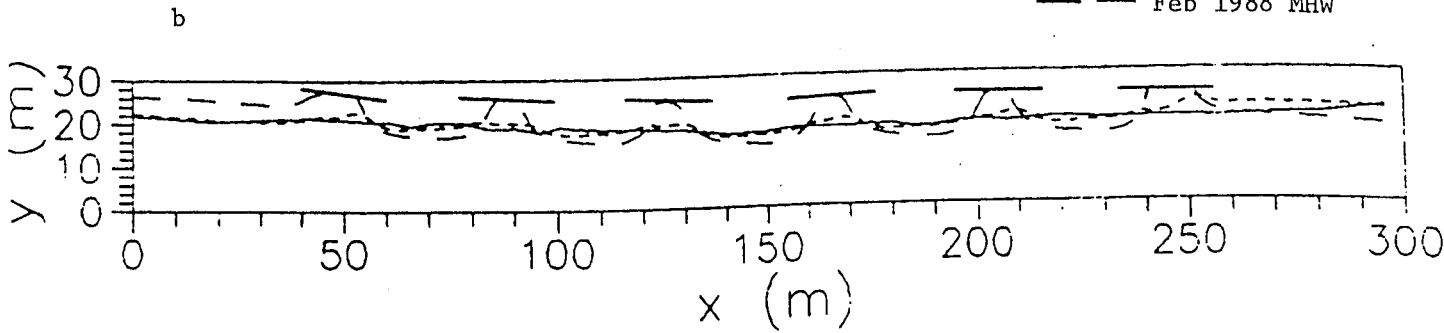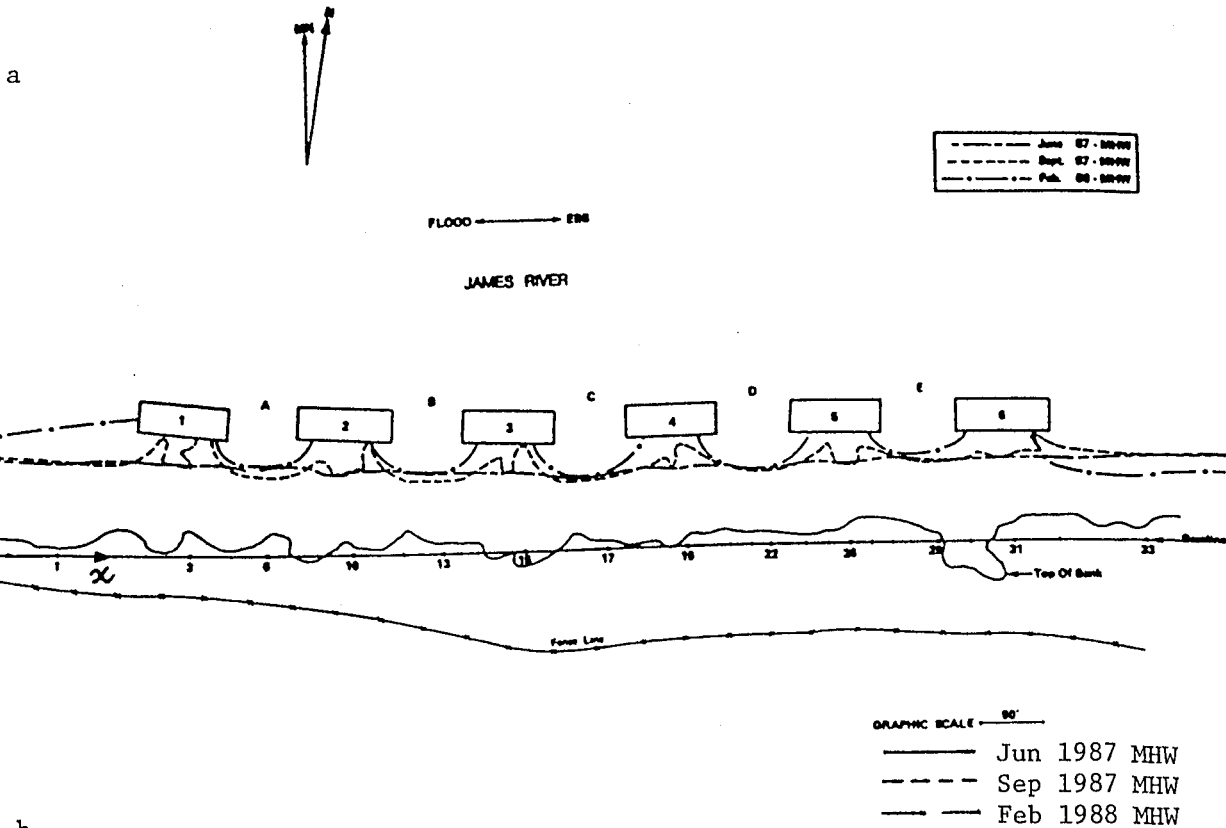
Figure 61. Chippokes State Park measured and computed shoreline changes.

equilibrium. Hsu et al. (1989a, 1989b) used something akin to a modified log-spiral approach and empirical data to come up with another model for analyzing pocket beaches. This model is termed Static Equilibrium Bays (SEB). This method appears to be appropriate in evaluating pocket beaches between major headlands in the Chesapeake Bay.

In general, the difference between a series of headland breakwaters and the more widely spaced major headlands and broader embayments is the distance between adjacent wave diffraction points or gap (G_B). For the so called headland sites (YB, HIH and SUM) this distiction begins at about 200 feet (YB2 and YB3) and goes to over 1000 feet (SUM).

For the headland sites, the SEB model utilizes two new parameters, an arc of length R angled theta to the wave crest line, which is assumed parallel to the tangent at the downcoast limit of the beach (Figure 62). The point on the upcoast headland where diffraction takes place is generally easy to define.

The downcoast control point may not be so easy to recognize, especially if a headland protrudes into the bay such as the end of a breakwater (Figure 63). There it is seen that wave diffraction in the shadow zone could cause an almost circular beach form which joins the main bay shape at some transition point. It is the tangent at this point that dictates the orthogonal used for computing the stable bay shape. The length of R then can be computed at given angles from the wave crest (theta) around the bay (Figure 64).

The SEB model was applied to the headland sites on the bay at Summerille, Bay B at the Hog Island Headlands and Bay 1 at the Yorktown Bays (Figure 65). The direction of wave approach was determined from the orientation of the tangential section of each embayment. The predicted planform for each bay appears to correspond to the top of the bank at Summerille and Hog Island Headlands Bay B and to about 3.0 feet above MHW at Yorktown Bay 1. The SEB model shows that Yorktown Bay 1 is in near equilibrium. At Summerille further bank erosion is predicted from profile
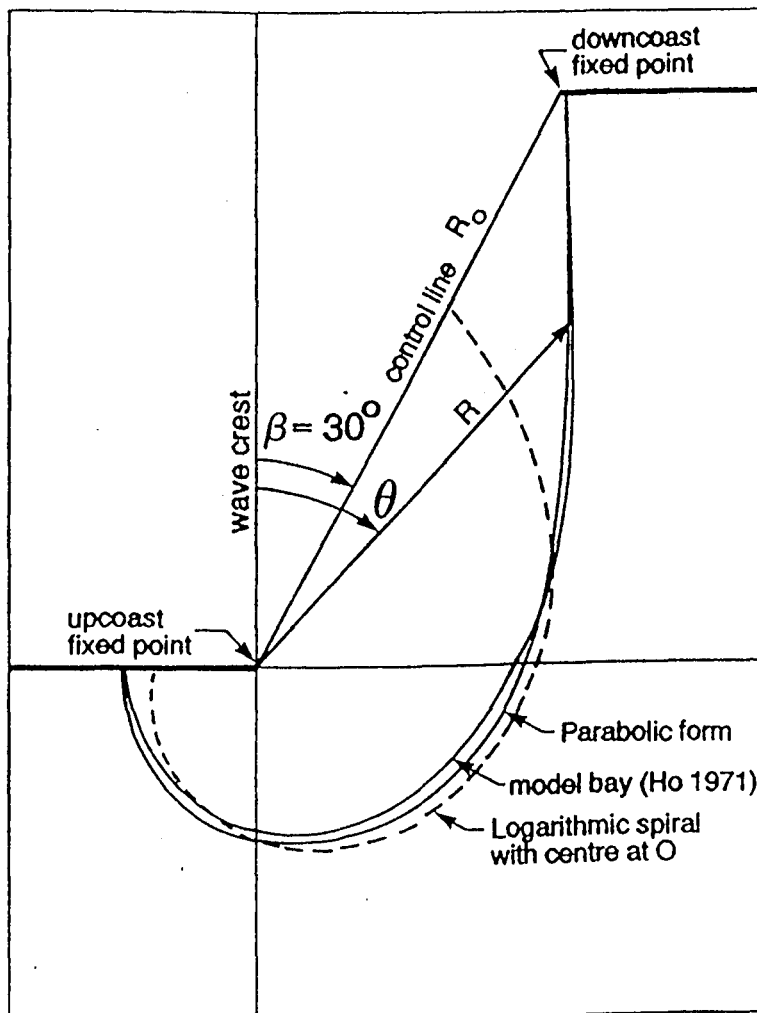
Figure 62.   Hsu's static equilibrium bay (SEB) model.
            (Hsu et al., 1989a).
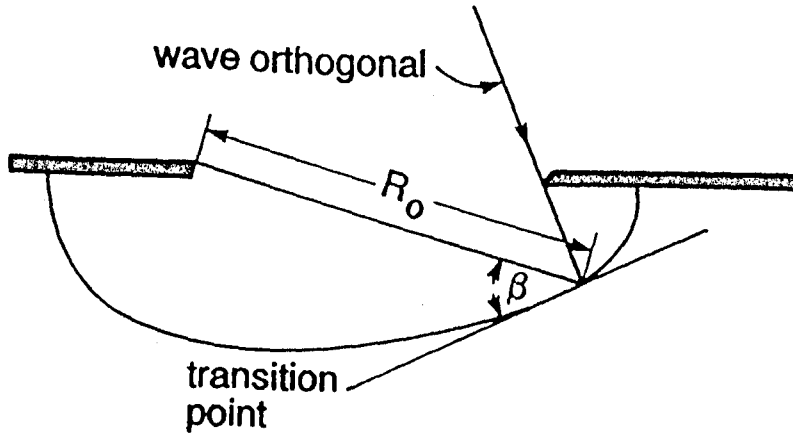
Figure 63.   Static equilibrium bay determination of R_O + B
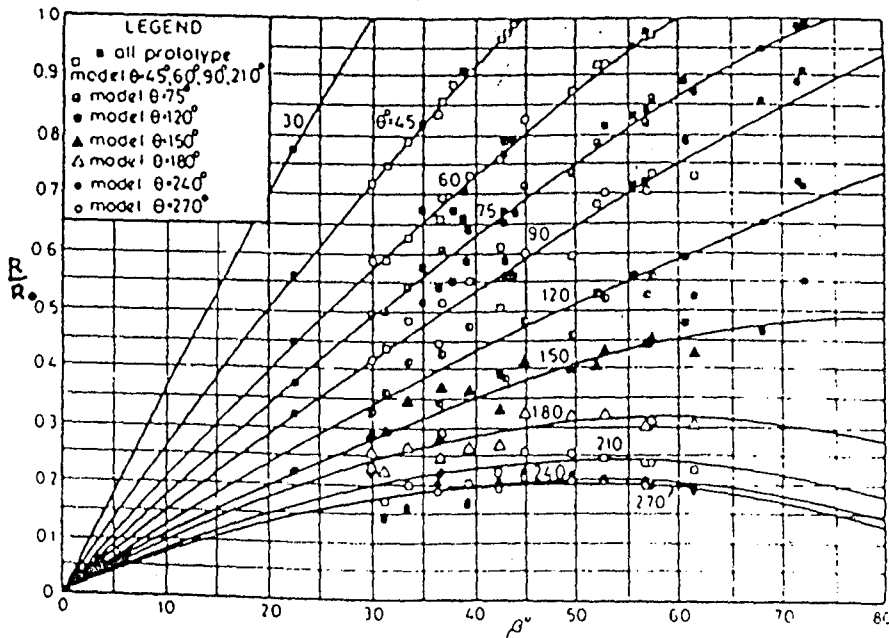(after Hsu et al., 1989a).

Figure 64.   Empirical relationships for static equilibrium bays
(after Hsu et al., 1989a).
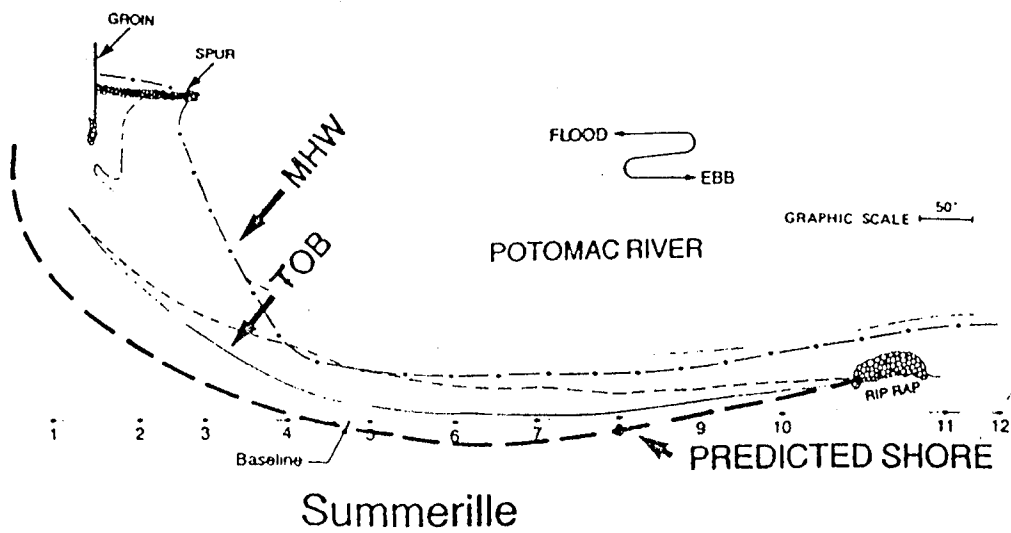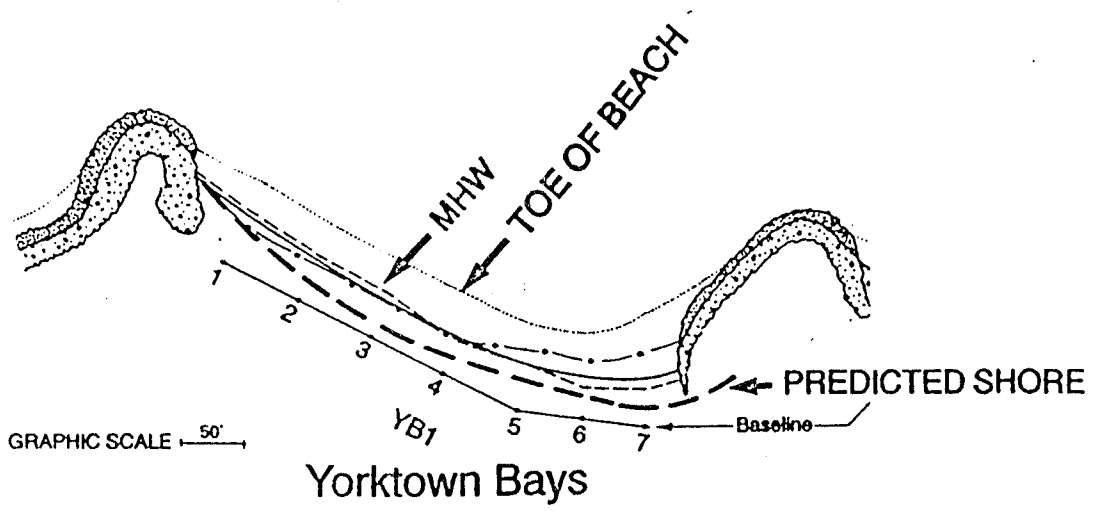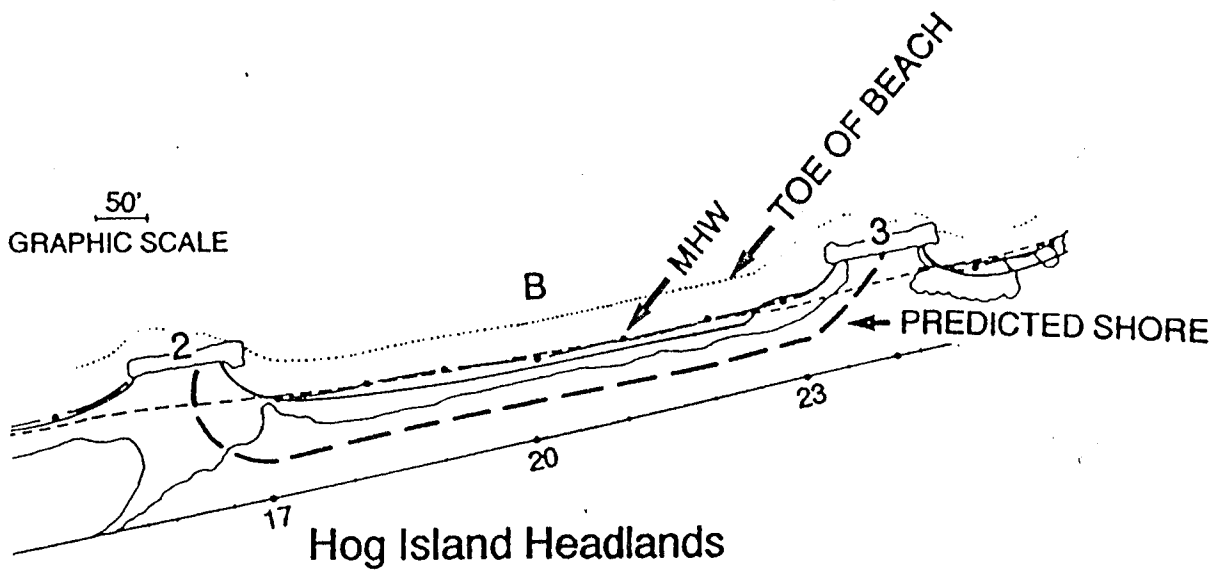
Figure 65. SEB model applied headland sites.

3 to profile 6 before equilibrium is reached. Also, Hog Island Headland require further bay shore recession to approach the predicted shoreline.

The situation at Summerille is difficult to ascertain due the low diffraction point which is the end of the spur. Under storm surge conditions, the spur is overtopped and the diffraction occurs at the base of the bank. If the predicted planform is MHW, the bank must erode back an additonal 40 to 50 feet in order to provide a protective beach. At the Hog Island Headlands the same situation applies where if the predicted planform is along MHW then the bank must continue to erode to achieve equilibrium.

The stability of the base of the bank at the headland sites at this point only occurs at the Yorktown Bays. The fastland banks at Summerille and Hog Island Headlands are and will continue to erode, in theory, to a stable planform.

This model should be considered another tool in the assessment of pocket beaches for shoreline erosion control. The previous log-spiral model of Silvester is still useful as a secondary check for stability (Hsu et al., 1989).

2. Storm Response

The April 1988 northeaster reduced beach slopes in the bays between the headlands. Also, significant bank erosion was documented at Summerille. Generally, a return of the beach sands was observed to a steeper beach slope. Maximum offshore movement was less than 20 feet at any one beach. This must be taken into account when when designing the placment of headland units. Transport of beach sands beyond the limit of the headlands may mean the permanent loss of beach material.

D. Shoreline Management

The objective of this study is to evaluate the use of headland breakwaters and the headland pocket-beach concept for the abatement of estuarine shoreline erosion as it pertains to the Chesapeake Bay. The use of breakwater systems for this purpose has been increasing over the past

decade with still less than 30 "major" installations in Virginia and Maryland. The design of breakwater systems requires a more detailed knowledge of littoral processes than the more traditional shoreline mitigation strategies such as wood bulkheads, stone revetments and groins.

Bulkheads and revetments must also be designed with the impinging wave climate in mind but there is generally less effect on the littoral processes due to the close proximity of the structures to the upland banks. Groins do affect the transport of beach and nearshore sediments in order to trap sand and create a beach. Since beach fill is generally not used in groin field construction, there is often a "downdrift" effect where native sands are kept from adjacent shores, beach widths decrease and bank erosion may increase with time. The knowledge needed to properly install these structures has been gained by numerous contractors over the past several decades.

Breakwaters are still a fairly new shoreline management strategy as far as usage in the Chesapeake Bay estuarine system. World wide, breakwaters have been used for shoreline management for some time, especially in Japan. It is the documentation and research done abroad, coupled with analysis of numerous Bay installations, that have enabled the authors to gain a perspective on how and where breakwaters can be used properly along the Bay shorelines.

Shoreline management is a term used here to address how erosion (or accretion) along a given shore reach should be assessed. This will most importantly depend of how the shoreland is to be used. Whether the shoreland is unmanaged, agicultural, residential or recreational will determine the shoreline management strategy and the money required to accomplish shoreline management objectives. In light of the current efforts to clean up the Chespeake Bay, improved water quality should be one of the more important shoreline management goals. These goals might include the following:

1. Reduce sedimentation and nutrient input from shoreline erosion,

2. address water quality by addressing upland runoff and groundwater input,

3. provide access to the Bay waters by maintaining beaches,

4. provide wetlands habitat and in so doing, preserve it, and

5. reduce the loss of taxable lands within localities.

Other management goals may be stated. However, these goals (individually or in combination) must be viewed as the principal choices for guiding shoreline management within the reach. Not all goals will have equal weight for any given reach. In fact, satisfaction of all of the goals for any reach is not likely, as some may be mutually exclusive (Byrne et al., 1979).

Bulkheads and revetments reduce sedimentation and loss of taxable lands but generally do not directly address water quality or create habitat. Properly designed breakwater systems which include beach fill and marsh grass plantings, bank grading and vegetation address four of the five goals. A recreational beach area would be the fifth goal but may restrict the marsh plantings to the supratidal species or dune grasses. The dimensions of a given project (i.e. structure size) would depend on land use, wave climate and construction access.

For instance, a long stretch of farmland in a medium energy setting might be conducive to widely spaced headlands such as at (HIH) or taking advantage of existing shore morphology (SUM). If the embayed shoreline is allowed to erode, then the eventual predicted planform can be calculated so that upland best management practices (BMPs - such as filter strips, graded and vegetated banks, grass waterways, drop outlets etc.) can be designed in concert with the shoreline system.

On medium and high energy shorelines in the Chesapeake Bay, it may be difficult to design breakwater systems for single lots of 100 feet. Breakwater systems are more appropriate for residential communities, either existing or planned, such as the case of (DMF) and (WAL). A stable

community beach is created for water access and recreation as well as erosion control. The upland banks are graded and heavily vegetated to help address nutrient laden surface runoff from lawns.

No doubt the shoreline management issue will continue to develop in response to environmental concerns about water quality and shoreline erosion and the right of individual property owners to protect their land in the way they desire. Only through a thorough understanding of the numerous components of littoral processes and shoreline management measures can the proper "environmental edge" be created and maintained.

VI. Conclusions

The definitive protective beach/breakwater system must be designed to withstand given storm conditions including the consequent surge. The Yorktown Bays, although a unique situation, offer a long term, stable series of pocket beaches with exposure to a relatively high wave climate for comparison to other sites. The problem may be the cost required to simulate the same situation on other shorelines.

The test of site success would appear to be the long term stability of the base of the bank. For the breakwater sites, this would include Drummonds Field and Waltrip. Drummonds Field and Waltrip were artificially nourished and both sites have similar fetches. Both sites are protected from the north and northeast wind directions but have a long fetch to the southeast and west. Chippokes, Parkway Breakwaters and Hog Island Breakwaters will continue to adjust mainly by fastland erosion.

Evaluation of breakwater parameters of the eight sites selected for detailed study in the Chesapeake Bay Shoreline Study and 10 other breakwater installations around the bay provide a reasonable empirical data base for comparsion. The best relationship among parameters was found to be between the bay indentation (M_b) and breakwater gap (G_B) where the ratio is about 1:1.65.

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A site analysis along project reaches must be done pursuant to the installation of widely spaced breakwaters to create a major headland/bay situation. Shore morphology evaluation has shown to be valuble in this aspect. The Yorktown Bays are geomorphically isolated, relatively stable, pocket beaches. The 1000 foot long Summerille embayment evolved into its present log-spiral configuration since 1967. At Hog Island Headlands, the shoreline had evolved into a planform which suggested the feasibility of using the existing shore points or major headlands for erosion control.

The use of models to predict shoreline change in the lee of offshore breakwaters or headlands is based on beach response to the impinging wave climate. The results of this study utilized two basic types of models; the numerical computer model and the empirical model as presented by Suh, in press and Hsu et al., 1989a and b, respectively. The computer model was performed on one breakwater site and the empirical model on the three headland sites. The computer model can analyze a large number of wave conditions for a given design, but is time consuming to set up, and accurate wave data is often unavailable. In this situation wind hindcasting has to be done, thus adding to the complexity. Suh's Model Tombolos does not work well with a single embayment with two headlands as boundaries but works very well for beach planform evolution behind a series of gapped offshore breakwaters.

The empirical model of Hsu et al. (1989a and b) was developed for large headland and pocket beach situations. The empirical model utilizes the net wave direction which is the main parameter and and can usually be determined from local shore morphology and/or the shorelines exposure to dominant wind directions. Calculations using several "type" wave conditions can be done quickly.

These models seem to be appropriate for evaluating breakwater and headland sites in fetch-limited environments. How a given breakwater or headland system withstands storm conditions and provides adequate shore protection will be the final test of success.

Finally, several points for Chesapeake Bay breakwater systems:

1. For storm surges of +2.0 feet MHW bank protection is provided when the backshore beach width and backshore beach elevation are 30 feet and 3.0 feet respectively and the Bay beach volume is greater than 3 cy/ft.

2. An optimum relationship for bay parameters in breakwater systems appears to be 1.65 for the ratio $M_b:G_B$.

3. The shoreline response model (Model Tombolos) developed by K.D. Suh will be a valuble tool in evaluating bay beach response to real time wind wave conditions. Further testing is neccessary to determine model limitations.

4. Predicting shore planforms between major headlands can be done using models as developed by Silvester and Hsu in combination with analysis of shore morphology and wave climate.

5. Further work is needed in evaluating wave climates in the Chesapeake Bay, especially beach responses to storm conditions.

6. The use of headland breakwaters as a shoreline management strategy is attractive because they address most of the shoreline mangement goals stated in this report and are economically competitive with other shoreline management measures such as bulkheads and rock revetments.

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