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Beach Forms Induced by Coastal Structures Forming Embayments in Fetch-Limited Environments

Jian Hua Li
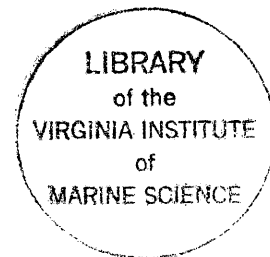*College of William and Mary - Virginia Institute of Marine Science*

BEACH FORMS INDUCED BY COASTAL STRUCTURES
FORMING EMBAYMENTS IN FETCH-LIMITED ENVIRONMENTS

A Thesis

Presented to

The Faculty of the School of Marine Science

The College of William and Mary in Virginia

In Partial Fulfillment

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by

Jian Hua Li

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APPROVAL SHEET

This thesis is submitted in partial fulfillment

of

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DEDICATION

This thesis is dedicated to my wife

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ACKNOWLEDGEMENTS

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LIST OF SYMBOLS

A — Scale factor in Bruun-Dean's model.

a_i — Incident wave amplitude.

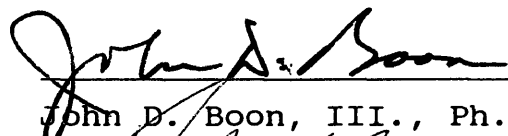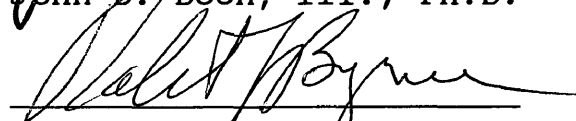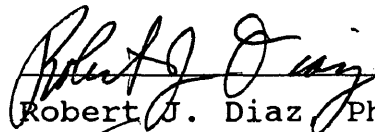α — (1).The angle between a radius vector and tangent for a given logspiral, a constant; (2).Wind direction in effective fetch calculation; (3). The angle between wave crest and respective depth contour (wave refraction).

B — The vertical increase in water depth.

β_b — Nearshore slope angle seaward from breaking waves.

C — Wave phase speed ($C=L/T$).

C_o — Wave phase speed in deep water.

ϵ — Wave reflectivity or surf-scaling parameter.

E — Wave energy density.

e — The base of Naperian logarithms.

e_r — Root-Mean-Square (RMS) error.

D — Diameter of a sediment particle.

D_{50} — Median grain size.

D_* — The average wave energy dissipation rate.

F_g — Gravitational force parameter.

F_f — Fluid force.

g — Gravitational acceleration.

H_b — Wave height at breaking.

h — Water depth.

h_{bo} — Breaking depth before water level change.

h_b — Breaking depth after water level change.

h_* — The depth of effective motion of active profile.

h_{Do} — The original dune height above mean sea level.

h_{bo} — The original breaking depth before water level rise.

H_d — Wave height in the region affected by wave diffraction.

H_i — Incident wave height unaffected by diffraction.

H_o — Wave height in deep water.

H_r — Reflected wave height.

h_s — Water depth at the tip of a spur (wave diffraction).

Symbol List Continued:

H_t — Transmitted wave height.

k — A constant relating wave height to water depth within fully developed surf zone (H_b/h_b=0.78).

k' — Wave diffraction coefficient.

L — The active beach width; Wave length.

L_o — Wave length in deep water.

L_s — Width between wave rays.

M_1 — Beach face slope ($M1$=$\tan\beta$).

M_2 — Nearshore profile slope.

m — Shape factor in Bruun-Dean's model.

P — Percentage of compatible beach material; Wave energy flux.

Q — The quantity of total water transport.

Q_{sx} — The quantity of longshore sediment transport.

R — Shore retreat distance after water level rise.

r — The length of a radius vector from log spiral center.

d — Water mass density.

S — Local sea level.

T — Wave period.

θ — The angle of a log-spiral radius.

ω — Radial frequency ($2\pi/T$).

U — Horizontal rate of change of profile.

μ — A sand fraction (refer to LeBlond 1972).

V — The rate of change of profile vertically.

X_f — Projected distance in effective fetch calculation.

X — Power of the spiral equation.

x — Distance waterward from beach.

k_r — Reflection coefficient.

ABSTRACT

The present study is focused on the descriptions of beach forms in plan and profile around breakwaters and headlands in fetch-limited environments.

Profiles in the study area show a two-segment characteristic: The upper segment is the beach face, being best predicted by a linear model, $h=M_1 x$ or $h=x\tan\beta_1$, where h is water depth; M_1 ($\tan\beta_1$) is the beach face slope with an average value of 0.12; x is the distance waterward from upper beach berm. The lower segment is from beach toe waterward, being best predicted either by the formula: $h=Ax^m+0.64$, with an average of A of 0.56, and m of 0.15, or by a linear model, $h=M_2 x+0.11$ or $h=x\tan\beta_2+0.11$, with $M_2=0.01$, where M_2 ($\tan\beta_2$) is the lower profile slope.

Plan beach forms are generally recognized as four groups: 1). Circular bay beach forms; 2). Spiral bay beach forms; 3). Straight beach forms; 4).Irregular beach forms. Circular bay beach forms are commonly found at those sites of closely spaced breakwaters without significant seasonal storm waves incident obliquely from a particular direction. Spiral bay forms are related to headland construction and storm waves from particular oblique directions. Straight beach form is insignificant at the study sites, only a small section with straight form is found at one of the seven study sites.

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BEACH FORMS INDUCED BY COASTAL STRUCTURES
FORMING EMBAYMENTS IN FETCH-LIMITED ENVIRONMENTS

1. INTRODUCTION

Shoreline erosion widely occurs either due to natural forces or human intervention such as mining of beach materials, modification of inlets, and channel dredging. In order to reduce the shoreline recession and property losses, various methods of shoreline protection have been adopted like the construction of attached or detached breakwaters, headlands, beach nourishment, and combinations of structure and nourishment. In response to the construction of various structures in shoreline protection, a variety of beach forms are evolved: spiral bay beaches, circular bay beaches and tomboloes behind breakwaters. All of these beach forms and structures combine to protect the shoreline. Therefore, knowledge of beach forms around coastal structures would be helpful not only in the design of structure construction along beaches, but also in understanding beach processes in the vicinity of structures (structures refer to breakwaters and manmade headlands in all of the text below).

With the increasing utilization of shoreline protection measures described above, more and more private and public breakwaters or headlands have been constructed in recent years to stabilize shorelines. For example, more than 30 detached and attached breakwaters and headlands were installed in a section less than 20 kilometers, from Hog Island to Waltrip, in the James River within past five years. This

shoreline stabilization is achieved through the combination of structures and stabilized bay beaches between them. However, most of the stabilized bay beaches are not really "stable", i.e. not in a static equilibrium due to the time-varying situations of the overall dynamic state in the littoral zone. The equilibrium association between beaches, structures and natural forces is dynamically shifting from one state to another. Beach forms are always changing with time, especially with seasonal wave climate.

The study areas are located in the tributary systems of the lower Chesapeake Bay: the James, York, and Potomac Rivers. They are typical shallow, fetch limited, low energy environments with terraces of different widths. These river systems together with the main body of Chesapeake Bay are the products of late Pleistocene sea level rise that started approximately 15,000 - 18,000 years ago. The estuaries developed as drowned valleys of the old Susquehanna River valley systems as sea level rose into the bay basin systems nearly 10,000 years ago (Schubel et al., 1972). The topographic features of the youthful river valley systems are still retained such as the meandering outline, triangular cross-section, and the gradual widening to the mouth areas.

Shoreline erosion has been severe around Chesapeake Bay including the study areas. Byrne and Anderson (1977) indicated that over 2.1 x 10^8 m^3 material was eroded from only the Virginia portion of Chesapeake Bay between 1850 and 1950. Summarized results from various studies of erosion rates within Chesapeake Bay estimate a net loss of nearly 27,900 acres (1.1×10^8 m^2) over the same period. The sand derived from this erosion is the primary source for the estuarine beaches. The flank

areas and channels receive most of the silt and clay fractions (Hobbs, Byrne, et. al., 1981). Table 1.1, based on information from Byrne and Anderson (1978), shows the average shore erosion rate in the reaches containing the study sites.

Table 1.1. The average erosion rate at study sites

SITE	HIH	HI2	CHP	DMF	WAL	NPS	SUM
RATE (ft/year)	2.5	1.7	1.2	1.6	1.6	1.5	5.0

Eight sites with breakwaters and headlands were selected as the study sites (Figure 1.1), which represent different fetch exposures and shore orientations. The following abbreviations are used for each study site throughout the text:

DMF - Drumonds field, James River, James City County.

CHP - Chippokes State Park, James River, Surry County.

HIH - Hog Island Headlands, James River, Surry County.

HI2 - Hog Island Breakwaters, James River, Surry County.

NPS - National Park Service beach site, York river, York County.

YTB - York Town Bays, York River, York County.

SUM - Summerille Beach site, Potomac River, Northumberland County. (This site was not formally selected as a study site as other seven locations due to its overall different physical characteristics. Its selection is only for some comparisons of plan spiral beach forms).

WAL - Waltrip, James River, James City county.

At each site, there is at least one breakwater or headland present. All breakwaters are manmade, and installed between 1985 and 1987 (detailed descriptions in Chapter 4). Three headlands are manmade at

Figure 1-1. Virginia Chesapeake Bay and its river system with study sites arrow-pointed.

sites SUM and HIH, four headlands are the combination of artificial and nature at YTB.

The objective of this research is to investigate the three dimensional beach forms around manmade coastal breakwaters and headlands: plan beach forms and profiles. Although some basic physical mechanisms behind the beach forms are to be discussed in this study, these discussions are limited. The focus of this study is the beach "forms" in plan and profile.

2. GENERAL BACKGROUND

There have been a number of field and laboratory studies on the general characteristics of equilibrium beach profiles and plan forms. It is useful to review briefly these studies as a general background of this study.

2.1. Major Previous Studies On Profiles

An early qualitative analysis of a number of mechanisms which shape the loose granular materials into an equilibrium beach profile was presented by Fenneman (1902). He observed that the resulting profile is concave upwards near to shore and concave downward in the seaward portion of the subaqueous terrace. He concluded that the waves tend to transport material onshore and the return flow ("undertow") currents are regarded as the primary agents in shaping the equilibrium profile. He also concluded that due to the increase of flow cross-section (depth) offshore, the offshore slope will decrease until the transported material is deposited on the platform on which the wave-built terrace is constructed.

Bruun (1954) developed an empirical equation between water depth, h, and distance, X, from the shoreline and scale factor A,

$$h = A \ X^{2/3} \qquad\qquad (2-1)$$

based on the analysis of beach profiles measured at different places, two mechanisms were proposed: (1). the onshore component of shear stress is uniform, and the onshore component of the gradient of wave energy

flux is constant, which lead to an empirical formula (2-1). (2). the loss of wave energy is due only to bottom friction and the loss per unit area is constant. Using a nonlinear wave theory incorporated friction factor results determined in laboratory studies, Bruun found a somewhat more detailed equation for beach profiles

$$h = (A'/T^{4/9})(X^{2/3}) \qquad (2-2)$$

where A is a scale factor, and T the wave period.

Bruun (1985) also briefly summarized the mechanisms of the development of storm (winter) and swell (summer) profiles: during the storm the point of incipient motion will be located far offshore while the point of equilibrium will be found closer to shore. Consequently, materials will move offshore. Conversely, during a (low) swell condition the point of the incipient motion will be close to shore while the oscillating equilibrium will be located further out. Consequently, materials will move onshore. This was supported by Bratteland and Bruun (1975) and Cart, Liu and Mei (1973).

Eagleson et al.(1963) developed a model for the equilibrium profile seaward of the break zone based on considerations of a balance between gravitational force, $F_g \sin(\alpha)$, and the fluid force on sediment particles, F_f , in a plane parallel to the beach. He concluded that in the case of equilibrium beach profile under the action of a given deep-water incident wave, F_g $-F_f$ will be obtained at all points on the sand beach. He developed the theory based on the earlier considerations of the mechanics of wave induced bed motion of single spherical particles on plane, roughened, impermeable beaches. A reasonably good agreement was found by comparison of model predictions with experimental

results.

A beach response model was developed by Edelman(1968) based on actual surveys of pre- and poststorm beach profiles along the coast of Holland. He found that the equilibrium profile relative to the instantaneous sea level was approximately constant and can be expressed by a uniform slope depending on the sediment characteristics. He defined the depth of effective motion to be given by the breaking depth. Then a balance is established between material deposited out to the breaking depth on the uniform slope and the materials eroded to a landward limit of the instantaneous water level on the eroded profile. The slope was found to be approximately 1:40 for Edelman's measurements.

Dean (1977) proposed a more comprehensive form of the Bruun (1954) model based on over 500 beach profiles along the Atlantic and Gulf of Mexico shorelines. He considered that there are both "constructive" and "destructive" forces physically presented in the surf zone which tend to transport sand onshore and offshore, respectively. Three destructive forces were recognized: (1).uniform longshore shear stress; (2).turbulence resulting from uniform wave energy dissipation per unit plane area; (3). turbulence resulting from uniform wave energy dissipation per unit water volume. The model has the form:

$$h = A \ X^m \qquad (2-3)$$

in which A is a scale factor depending on the stability characteristics of the bed materials, m is a shape factor, found to be 2/3.

Swart (1974) developed numerical models to describe beach profiles. He divided the beach profile into three zones:

1. The backshore above the limit of wave runup. If wind-blown

sediment transport is neglected, there is no transport in this zone.

2. Developing zone (D-profile), where a combination of bed-load and suspended load transport takes place. The dividing point of these two zones is the highest point waves can reach. The position of this point was determined empirically as

$$h_o = 7650 \; D_{50} \; [1-exp(-1.000143 \; H_{max}^{0.488}T^{0.93}/D_{50}^{0.786})] \qquad (2-4)$$

where H_{max} is the maximum wave height in the spectrum equal to twice the significant wave height, T is the wave period, D_{50} is the median particle diameter.

3. Transition area, seaward of D-profile and landward of a point where sediment is initiated by wave action. Bed-load is normally the only transport in this zone. The point dividing these two low zones was determined empirically by

$$h_m = 0.0063L_o exp(4.347 \; H_o^{0.473}/ \; T^{0.894}D_{50}^{0.093}) \qquad (2-5)$$

where h_m is the depth of the D-profile; H_o, L_o and T are deep water wave parameters; D_{50} is the median grain size.

Swain (1989) proposed a series of detailed steps/methods to calculate beach profiles based almost completely on Swart's theory stated above. These calculations were focused on the finding of the equilibrium curve termed W-curve, which was supported by his model testing.

Dean (1990) theoretically modified his previous work on equilibrium beach profile modeling. One of the most obvious improvements is the addition of a gravity effect to the turbulence effect near the shore. When the distance offshore and water depth are very small, $x=h/m(1/A^{3/2})h^{3/2} \; \rightarrow \; x=h/m$ or $h=M_1x$; when h is large,

$x \sim A^{-3/2}h^{3/2}$ or $h \sim Ax^{2/3}$, which is the original model form.

Byrne and Anderson (1978) carried out a systematical survey of shoreline erosion of Virginia Chesapeake Bay and its tributaries. In this study, they not only made detailed statistics on 2,366 miles of shoreline erosion, but also proposed major factors that control the beach processes in FETCH—LIMITED environment. These major factors are:

1). The intensity of wave action and the exposure to strong tidal currents.

2). The character of sediments at the site and the degree of protection offered by vegetative cover, specifically marsh grass, at the shoreline.

3). The supply of sand moving along the shoreline from other eroding areas or from streams along the shoreline.

4). The gradient or slope of the fastland adjacent to the shoreline and the slope of the nearshore bottom.

They further pointed out that the dominant erosion agent is the waves generated by LOCAL WIND ACTION. The growth and height of the waves is controlled by: fetch size; wind speed; the duration of wind and the depth of water. It is obvious that these are the most important factors when discussing wave conditions in fetch limited environment.

They particularly noticed another phenomenon in fetch—limited environment: the water elevation raised by local wind set—up and storm surge within the estuary during storm time. These storm effects can be further enhanced in conjunction with the spring tides, resulting in severe beach erosion. Therefore, the shoreline changes are greatly influenced by the local weather patterns. The northeast storms during

the fall, winter and early spring generate local waves that attack the western shore of the bay and tributary shorelines with a fetch to the northeast and east. The winds and low pressure along the coast line have an additional, indirect effect on the bay system. The storms force additional water into the bay. This local "wind tide" or storm surge may be one or two feet above the normal tide level.

Boon and Green (1988) performed field measurements on two Caribbean islands. They found that the two dimensional nearshore profiles can be well represented by Dean's (1977) model, $h=Ax^m$. Based on the least squares curve fitting of observed profile data, they found an average value for the curvature, $m=1/2$, yielding a more concave and therefore steeper profile in inshore than Bruun and Dean's average value, $m=2/3$. They also suggested an interesting surrogate for the beach face slope, tan(Beta), on HIGHLY concave beaches to be $A^{1/m}$, where A and m have the same definitions as in Dean's model. An important result of their work is the correlation analysis, which connected the beach slopes closely with the most important physical and environmental factors such as wave height and period, sediment size and gravitational acceleration, g.

According to Wright et al. (1979), the linear, steep beach face and the linear gentler nearshore profile are one of the major morphological characteristics of reflective beaches. The reflectivity is determined mainly by the incident wave amplitude near the breaker point, wave period, the acceleration of gravity and the beach slope, this can be expressed:

$$\epsilon = a_1 \omega^2 / g \; \tan^2 \beta_1 \qquad (2-7)$$

ϵ is referred to as the reflectivity or surf-scaling parameter. When

$\epsilon<1.0$, waves completely reflect with negligible dissipation; when $\epsilon<2.0-$ 2.5, low dissipation and strong reflection occur; when $\epsilon>2.5$, then waves begin to plunge, dissipating energy; when $\epsilon>20$, then spilling breakers occur; when ϵ ranges from 30 to over 100, spilling breakers and approximately saturated surf zones occur across which bores decay progressively to become very small by the time they reach the subaerial beaches.

They identified a number of distinct morphological "states" or "stages" associated with various wave and tide regimes (1879; 1982a,b,c; 1983; 1984). They have been able to integrate many seemingly disparate hydrodynamic and morphological factors into coherent models, emphasizing the role of antecedent conditions in determining morphological states. In their summary papers, Wright and Short (1984) have replaced the original, somewhat confusing terminology with dissipative and reflective domains separated by four intermediate domains:

(1). Dissipative domain extreme. This is characterized by flat shoaling slope and wide surf zone. The surf scaling parameter has values ranging from 20 to 200. Nearshore topography was characterized by multiple parallel bars.

(2). Reflective domain extremes. The beach gradients are invariably steep (Beta > 6°), with no nearshore bars. The values of surf scaling parameter are less than 2.5. Leaky or trapped mode gravity edge waves dominate the secondary fluid motions in the narrow breaker zone, leading to the formation of subharmonic or synchronous beach cusps.

(3). Intermediate domains. These incorporate elements of both the

reflective and dissipative domains, coupling of waves and morphology becomes important. Intermediate states may arise as a consequence of tidal changes (Wright et al. 1982b), especially across beach profiles with separate sand and gravel elements. Four intermediate types in this domain are recognized: Longshore bar-trough; rhythmic bars; transverse (welded) bars and low tide terrace.

Sunamura (1984) summarized the previous studies on beach slopes and proposed two formulas to try to give a quantitative predictions of beach slopes:

(1). For laboratory beach slope

$$Tan\beta_1 = 0.013/(H /D^{0.5}g^{0.5}T)^2 + 0.15 \qquad (2-8)$$

(2). For field beach slope

$$Tan\beta_1 = 0.12/(H /D^{0.5}g^{0.5}T)^{0.5} \qquad (2-9)$$

He pointed out that the beach slopes predicted based on laboratory data are always higher than those based on field data for a given value of the dimensionless parameter, which indicates the laboratory beaches are high energy beaches, why it is high in energy is at present unknown (Sunamura, 1984, p244).

2.2. A Brief Review on Plan Beach Models

Yasso (1965), proposed the "headland-bay" beach conception which was defined as a beach lying in the lee of a headland subjected to a predominant direction of wave attack. He concluded that the erosion caused by refraction, diffraction and reflection of waves into the shadow zone behind the headland were the mechanisms leading to the formation of a seaward concave plan shape. The tidally induced currents have no direct effect on the formation of the plan headland bay beaches.

He suggested the formula,

$$r = e^{\theta \cot \alpha} \qquad (2\text{--}10)$$

as an approximation to the plan form of the headland-bay beach in considering the fact that the radius of the curvature from the headland increases with distance (see Fig.2.1 and 2.2).

Silvester (1972) pointed out that the crenelate shaped bays are the rule rather than the exception on shorelines where sedimentary beaches exist between headlands. There exists a particular relationship between the advance direction of swell or resultant energy vector and the orientation, such that the straight tangent section is down coast and the curved portion is upcoast. This curved portion at upcoast is a logarithmic spiral (Figs.2.3,4) when the bay is fully stable. The constant, α, in the equation

$$r_2/r_1 = e^{\theta \cot \alpha} \qquad (2\text{--}11)$$

has a specific relationship to the approach angle of waves, to the headland alignment. Wave refraction and diffraction are involved when waves sculpture the curved beach in the lee of the upcoast headland. He also proposed a particular ratio, a/b, to identify stable bays, where a is the distance of the bay indentation, b is the distance between structures at the open side of a bay.

LeBlond (1972) studied spiral bay beaches from a somewhat different aspect. This study is a mathematical simulation of the development of spiral bay beaches using the theory of wave induced longshore currents. In order to relate sand transport to longshore current, he assumed that the volume rate of sand transport, T, is proportional to the total water transport, Q, in the surf zone times a "sand fraction", μ,

Fig.2.1. Plan of spiral beach, Sandy Hook, New Jersey, and fitted logarithmic spiral (after Yasso, 1965).

Fig.2.2. Logarithmic spiral nomenclature (after Yasso 1965).

Fig.2.3. Definition sketch of logarithmic spiral (after Yasso, 1965).

Fig.2.4. Typical prototype bay showing refracted
orthogonals (after Silvester, 1972).

Fig.2.5. The initial linear beach configuration
and qualitative estimates of some consecutive
shapes (after LeBlond, 1972).

$$T - Q(\mu) \qquad\qquad (2\text{-}12)$$

Therefore, sand transport is an integral property of the surf zone. With the erosion occurring from the initial linear shoreline, the sand transport divergence is large and positive at the origin (see Fig 2.5) because of the sand fraction discontinuity. The beach will gradually deform as shown in Figure 2.5.

Hsu and Silvester (1989) argued that the previous methods for determining the bay-beach planimetric shape using the logarithmic spiral principle apply mainly to the shadow zone behind the upcoast headland, not to the entire bay periphery, and do not have a fixed center. In order to solve this problem, they proposed a new method utilizing arcs from the point of wave diffraction to the shoreline, whose lengths and angles to the wave crest line are related to those of the control line.

In summary, the theoretical and empirical relationships presented in the previous studies are valuable to help explain some of the phenomena found in our fetch-limited study areas. It is seen that almost all relationships rely heavily on the empirical parameters and constants, which implies the difficulty in explaining the complex natural phenomena with theory. It can be said that a successful explanation on the natural phenomena is not at all easier than a precise calculation of the orbits of a sophisticated satellite or missile. It is just like what Komar (1976) said on wave prediction that the "state of the art" is largely a matter of accumulation or collecting of data. This is also very true in developing models on beach forms.

On beach profiles at present level, the new Bruun-Dean (1990) model, $h = M^1 x$ (beach face) and $h = A x^m$ (lower profile), is of great

reference in describing beach profiles. This model is actually a new version or improvement of the earlier Bruun-Dean (1977) single curve model, h=Ax^m. The old model has proven to work reasonably well with m=2/3 (Bruun, 1954; Dean, 1977), and m=1/2 (Boon and Green,1988) in open ocean conditions. Boon's data were collected from beaches with carbonate dominated materials of a different environment. It would be expected that the new model would also work in fetch-limited, low energy level areas though some improvements are needed in explanation on the mechanisms of the nearshore profile development. This is because the origination of Bruun-Dean's model was based on profiles formed under fully dissipative wave conditions, the turbulence scale and nearshore circulation patterns are somewhat different from those in the fetch-limited environment. However, a rigorous improvement on the mechanisms is beyond the ability of the author for the purpose of this study.

At present, the quantitative predictions of beach slope are still remote based on the controlling physical parameters in the nearshore region (Komar, 1976; Sunamura, 1984) though the qualification of relationships between the controlling parameters and the slope has been attempted since early 70's. This is because of the complexity of the nearshore processes. However, the most important factors affecting beach slopes have been identified by many researchers (e.g. Byrne, 1978, Sunamura, 1984, Boon, 1988 and Wright et al., 1979, 1982, 1983, 1984). These factors are: (1). Sediment characteristics; (2). Wave properties such as wave height, period and length; (3). Tide range and stages; (4). Longshore current velocities; (5). Water table (ground water level).

The equation 2-9 proposed by Sunamura would represent a useful model in predicting beach slopes based on a number of important environmental parameters.

On plan beach forms, less modeling has been done than profiles, especially for beach forms between structures. Most modeling work has been concentrated on spiral bay beaches, and there are still severe arguing on the spiral bay beach predictions. This can be seen from Hsu and Silvester (1989) paper. On regular or irregular circular bay beach modeling, there has been few studies in the literature.

Yasso's spiral bay beach model gave a bright preliminary idea in simulating spiral bay beaches though some weak points exist as pointed by Hsu and Silvester (1989). Hsu and Silvester did not use a particular logarithmic equation to simulate the plan bay shape, but use the empirical parameters based on field and laboratory data, or hind-forecast analysis. The advantage of their method is that it provides a better fit to the natural headland bay beaches. The disadvantage is that the diffraction point and the critical point down beach where the wave orthogonal is determined are often difficult to determine. Any small error in determining the locations of these two points can lead to large variations in predicted bay shape. And they did not propose a formula to generalize their method which would result in a difficulty in simulating by computers.

3. DATA AND METHODOLOGY

3.1. Surveys and Data Sets of Beach Profiles and Photography

Field data used in this study are part of an ongoing shoreline study of Chesapeake Bay. It is a cooperative project between the Commonwealth of Virginia and the Norfolk District of the U.S. Army Corps of Engineers. The study area and sites in this thesis are generally the same as those in the project (Fig.1.1).

The field data collection involved quarterly and after—storm shore profiling and low level aerial photography as well as sediment sampling and laboratory grain size analysis (Tables 3.1 and 3.2).

The detailed arrangements of structures and profiles at each site are presented in Figures 3.2 through 3.8. The baselines in these figures were established using a transit. The spacings between profiles along baselines are determined based on site situations. The more concerned areas have smaller spacings so that beach changes can be more accurately measured. From the baselines, profile lines were determined and surveys were performed using a rod and level. The feature profiling method is used (Fig.3.1): at any perceived break along a profile, a point is set up for measuring. The square symbols in figure 3.1 are such points. In addition to these perceived breaking points, some more points are usually added in certain critical section such as the one connecting the beach face and offshore profile. When a beach face looks

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Table 3.1 Profiling and Aerial Photography Schedule by Month and Day

Site	Phase I			Phase II			Phase III		
	Jul (1987)	Aug	Sep	Oct (1987)	Nov	Dec	Jan (1988)	Feb	Mar
Chippokes State Park 33 profiles baseline = 839 ft									
profiles	28				12			23	
aerial photos			2			9			9
Parkway Breakwaters 27 profiles baseline = 748 ft									
profiles	29				16			26	
aerial photos			2			9			9
Hog Island Breakwaters 61 profiles baseline = 1275 ft									
profiles			1					9	
aerial photos			2			9			9
Drummonds Field 35 profiles baseline = 1674 ft									
profiles			9			1		24	
aerial photos			2			9			9
Hog Island Headlands 30 profiles baseline = 2400 ft									
profiles		12		20				10	
aerial photos						9			9
Yorktown Bays 21 profiles baseline = 357 ft									
profiles		7			17				28
aerial photos			2			9			9
Summerille 12 profiles baseline = 863 ft									
profiles			18			11			8
aerial photos	2								11

Table 3.2. Profiling and Aerial Photography Schedule by Month and Day

	1988						1989					
	Phase I			Phase II			Phase III			Phase IV		
Site	Jul	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun
CHP												
33 profiles												
baseline — 839 ft												
profiles			21			14			28			8
aerial photos			15			19			29		25	
NPS												
27 profiles												
baseline — 748 ft												
profiles			14			6			17			2
aerial photos			15			19			29		25	
HI2												
61 profiles												
baseline — 1275 ft												
profiles			7		22				16			6
aerial photos			15			19			29		25	
DMF												
35 profiles												
baseline — 1674 ft												
profiles			13			8			30			15
aerial photos			15			19			29		25	
WAL												
17 profiles												
baseline — 447 ft												
profiles			27			20			20		26	
aerial photos			15			19			29		25	
HIH												
30 profiles												
baseline — 2400 ft												
profiles			6	20		6			17			8
aerial photos			15			19			29		25	
YB												
21 profiles												
baseline — 357 ft												
profiles			28		17	19			15		23	
aerial photos			15			19			29		25	
SUM												
12 profiles												
baseline — 863 ft												
profiles			2			5			27			5
aerial photos						19			29		25	

FEATURE PROFILLING METHOD

DISTANCE FROM BASE LINE IN FEET

ELEVATION IN FEET

Fig.3.1. A diagram showing the feature profilling method used in this study. The data used in this diagram was taken on Dec. 1988, at National Park Service beach, York River.
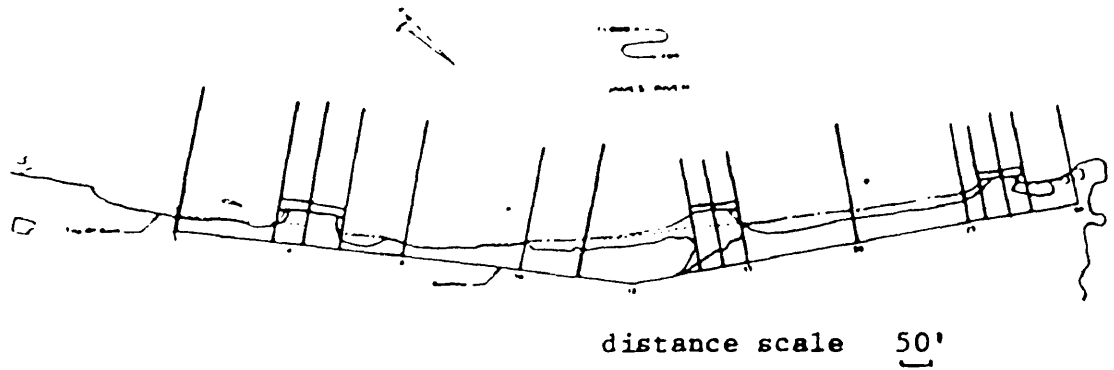
distance scale   50'

Fig.3.2. Hog Island Headlands, Base map with some profiles numbered.

distance scale   50'

Gig.3.3. Hog Island Breakwaters, Base map with numbered profiles.

distance scale   50'

Fig.3.4. Drummonds Field, Base map with numbered profiles.

distance scale __50'__

Fig.3.5. Chippokes State Park, Base map with numbered profiles.

GRAPHIC SCALE ___50'___

Fig.3.6. Parkway Breakwaters, Base map with numbered profiles.

distance scale <u>50'</u>

Fig.3.7. York Town Bays, Base map with some numbered profiles.

POTOMAC RIVER

FLOOD

EBB

—··—	Aug	86	MHW	
————	Aug	86	Base of Bank	
—·—·	Mar	88	MHW	
————	Mar	88	Base of Bank	

Baseline

1   2   3   4   5   6   7   8   9   10   11   12
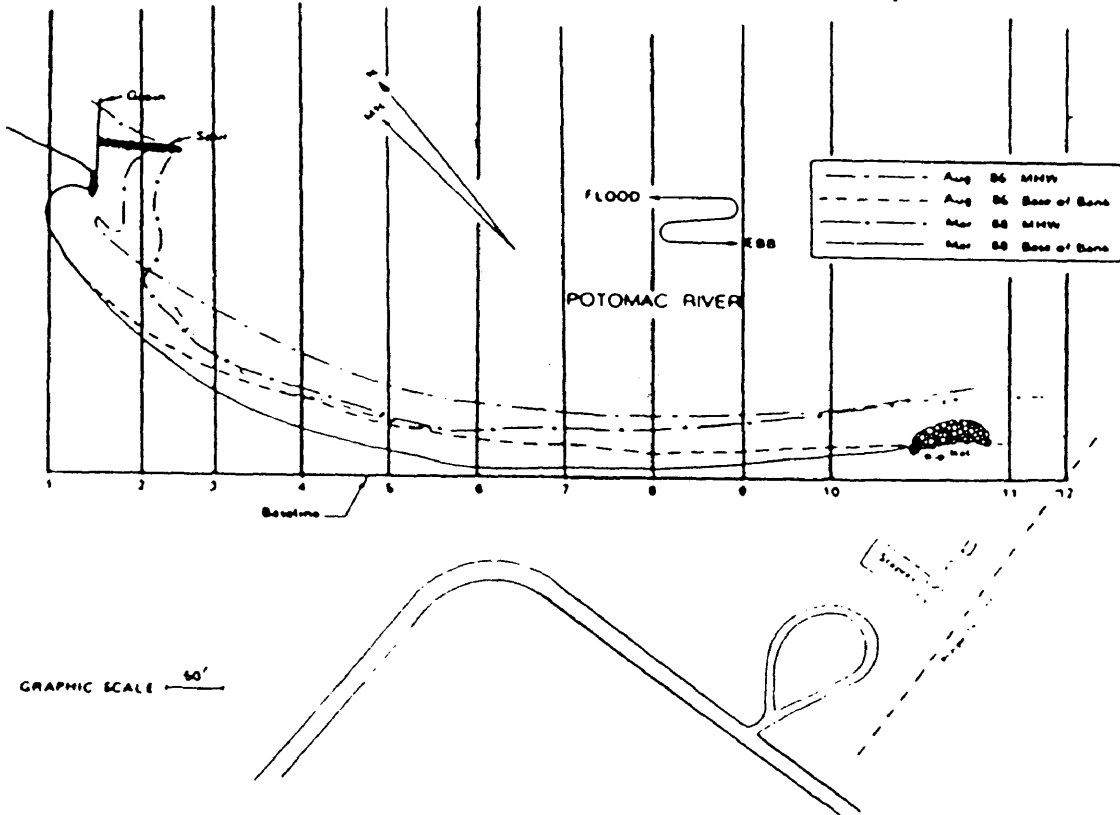
GRAPHIC SCALE 60'

Fig.3.8. Summerille, Base map with numbered profiles.

straight across it, we did not just measure two points at both ends of the beach face, but a series points for the purpose of statistical analysis. Tables 3.1 and 3.2 give the detailed schedule of profiling and aerial photography.

All coastal structures are manmade except the headlands at Yorktown Bays which is a combination of manmade and natural structures. Five sites have breakwaters: HIH, CHP, DMF, WAL and NPS. The breakwaters at CHP, HI2 and NPS were initially set up at or near mean low water (MLW). CHP and NPS had no beach fill added, whereas HI2, DMF and WAL did. Three sites have headland structures: HIH, YTB and SUM. Only the headlands at HIH were constructed for the project. The headlands, their crenelate bays and pocket beaches at both sites SUM and YTB have existed for over ten years.

All profile data are plotted in Appendix B,C AND D.

3.2. Data Sets on Plan—view Configurations

The plan—view configurations of shores at each study site were determined by 1). low aerial photography; 2). comparisons of adjacent profiles along with the field pen records on beach features; 3). detailed measurements of plan beach forms on selected bays between breakwaters.

From Table 3.1 and 3.2, it is seen that frequent photography was done according to schedule. The photographs were taken at 500, 1,000 and 2,000 feet, and were used along with the profile data to create a base map for each site against which the general trend of sediment movement could be determined by carefully examining photographs. The variation of beach forms relative to certain fixed targets can be more

easily determined. The comparisons of adjacent profiles were very useful in determining plan-beach forms. For instance, there generally exists a clear slope break at the beach toe for most sites and an upper beach point defining the approximate mean high high water level. These features were hand-recorded while the profile was measured in the field. After these points were connected from adjacent profiles, a clear plan-view of the distribution of the beach face zone would be fairly accurate. Some representative bays were selected to be measured to determine the general pattern of beach slope distribution within a whole bay between structures. Five bays were selected for this purpose: Bay 11 at HI2; Bay 3/C at CHP; Bay A at DMF; Bay 1/A at WAL and Bay 2/B at NPS. For detailed results of analysis on these bay measurements please see Section 5.3.

3.3. Sediment Data and Analysis

Surface sediment samples were taken from back shore to offshore along selected profiles at each site. Sediment sampling was done at least three times along the same profiles for each site. Borings on the beach and nearshore were also done to measure the thickness of recently eroded bank materials overlying the older, in situ strata. All sediment samples were analyzed at VIMS sediment laboratory for percent of gravel, sand, clay/silt and other characteristics of sediment such as moment measure mean, standard deviation. The detailed results of sediment analysis are presented in section 5.

3.4. Methods of Wave Hindcasting

As pointed out by Komar(1976), in the absence of a satisfactory understanding of the mechanisms of wave generation, semiempirical

approaches should be developed for wave prediction. Development of the "state of the art" is largely a matter of collecting data on winds and waves. As new data accumulates, the relationships must be modified and, so, continue to evolve.

In general, the prediction methods can be classified into two categories: 1). the significant wave method; 2). wave spectrum method. The first method, significant wave method, was proposed by Sverdrup and Munk(1947) and revised later by Bretschneider(1952, 1958). A name of S-M-B method has been used after their names. The second category, the wave spectrum method, was first introduced by Pierson, Neumann and James(1955). This method is commonly called P-N-J method which describes the generated storm waves in terms of the complete spectrum of periods and energies, representing a considerable improvement in wave forecasting techniques. However, the S-M-B method is chosen in this thesis because: (a) only significant wave parameters are concerned in this study; (b) predicted results can be used to compare with field observations of significant wave parameters; (c). this method is easier to use but give a useful reference results. Table 3.3 shows comparisons of some predicted values by various methods.

Based on the known storm conditions such as wind velocity U, fetch distance F, and storm duration D, the significant wave parameters like $H_{1/3}$, $T_{1/2}$ and $L_{1/3}$ can be predicted. In practice there are basically two types of procedures to use S-M-B method: 1). computerized S-M-B procedure which is used in this thesis (This procedure gives a faster and more accurate predicted results than the traditional dealing by graphical method). 2). graphical procedure which has been more

traditional (This second procedure is based on the empirical graphs

first presented by Bretchneider(1959) and Wiegel(1961). The graphs were

formed based on all available data at that time in terms of the

dimensionless ratios gF/U^2, gD/U, $gH_{1/3}$ and $gT_{1/3}/U$. The graphs have

been updated and widely used in coastal engineering researches).

Table 3-3.Wave predictions by various methods

WIND SPEED (m/s)	FETCH DISTANCE (km)	SIGNIFICANT WAVE HEIGHT(m)				SIGNIFIC. WAVE PERIOD			
		S-M-B	P-N-J	LIU	DARBYSHIRE	S-M-B	P-N-J	LIU	D.
10	200	2.1	2.4	2.1	1.2-1.4	7	8	7	7
10	1,000	2.7	2.4	3.4	1.2-1.5	11	8	10	7
20	200	5.2	4.3	5.8	4.3-5.2	10	8.5	10	10
20	1,000	8.9	11	9.2	4.6-5.6	15	16	15	11
30	200	8.2	7.9	11	9.7-12	12	10	12	15
30	1,000	15	15	17	11-13	19	15	18	16

Steps of S-M-B method used in this thesis:

1) Determine the size of the effective fetch. This step is

particularly important when S-M-B method is to be used in small water

bodies such as the tributary estuaries to lower Chesapeake Bay. If the

S-M-B method is used in open waters like the ocean, there is not too

much concern about the size of effective fetch because the fetch

distance is usually "unlimited". However, the small water bodies are

often very fetch-limited (Wright, 1985), any error in determining the

effective fetch distance can lead to errors in predicting the

significant wave parameters. In determining the effective fetches, the

distances within 90° are measured symmetrically around the direction of

main storm winds. A formula

$$F_{eff} = \sum X_i \cos(\alpha) / \sum \cos(\alpha) \qquad (3-1)$$

is used to determine the effective fetch based on the values
measured, where X_i is distance, α is the angle between wind
direction and any other lines.

2) Divide the whole length of water surface along the wind
direction concerned into segments. The water depth within each segment
should be relatively evenly distributed. This can be done on a detailed
topography map based on the well established depth contours by map
surveyor.

3) Take the average of water depths for each segment using a simple
formula

$$d = (d_i + d_{i+1})/2 \qquad (3\text{-}2)$$

4) Input the effective fetch, wind direction, wind speed, surge
levels and the average depth for each segment into the computer, or
alternatively, look up directly the predicted values of wave parameters
on the empirical graphs.

3.5. Methods on Wave Refraction and Diffraction

Wave refraction:

Waves are subject to refraction as they enter shallow water. Wave
orthogonals keep change directions with decreasing water depth in such a
way that the crests tend to parallel the depth contours. The wave
refraction is similar to that of light rays. Therefore, the wave
refraction can also be described by Snell's law

$$\text{Sin}(\alpha 1)/C1 = \text{Sin}(\alpha 2)/C2 = \text{constant} \qquad (3\text{-}4)$$

where a is the angle between wave crest and the respective depth
contour. C1 and C2 are the successive phase velocities at the two
depths. Based on the Airy wave theory, C can be obtained from:

$$L/L_o = C/C_o = \tanh (2\pi \, h/L) \qquad (3-5)$$

From the simple relationship in equation 3-4, it is easily seen that to keep the constant, the two factors, a and C, have to change in a similar manner, decreasing as the shore is approached. That is, when a decreases, C can not increase.

The method for generating refraction diagrams in the Shore Protection Manual was employed for considering wave refraction patterns. This method is easy to use and able to generate refraction diagrams with reasonable accuracy. It is actually a orthogonal or template method. It uses bottom topography charts of any scale. Underwater contours are drawn on the charts or a tracing paper overlay. The depth intervals chosen depend on the degree of accuracy desired. In tracing contours, small irregularities must be smoothed out, because the bottom features comparably small in respect to the wave length do not affect waves appreciably. For each wave period and direction selected, a separate diagram must be prepared. C1/C2 values for each contour interval are then marked between contours. C1 and C2 are phase velocities. For further detailed procedures of this method, please refer to S.P.M., volume I, pages 265 - 279.

Wave diffraction:

The phenomenon of wave diffraction is very common along coasts with various manmade structures and natural headlands. In this process, wave energy is transferred laterally along wave crests from where the wave height is large to where it is small.

The result of wave diffraction is the re-distribution of wave energy, which is indicated by the variation of wave height. The wave

diffraction is best expressed by the patterns of wave diffraction coefficient, K′, which is defined as the ratio of a wave height, H_d, in the area affected by diffraction to the incident wave height, H_i, in the area unaffected by diffraction:

$$K' = H_d/H_i \qquad\qquad (3-6)$$

The diagrams for wave diffraction were first systematically presented by Wiegel (1962). These diagrams have been widely used or referenced by coastal engineers and geologists(e.g. Shoreline Protection Manual, Vol.1). Figure 3-9 is only one example of these diagrams. This diagram was constructed in polar coordinate form with arcs and rays centered at the tip of the structure. The unit used for rays is radius-wavelength. One wave length between arcs in this diagram. The predicted wave parameters are only to be used as a very general reference to understand the general wave conditions in the further offshore areas near the study site.

Fig.3.9. Wave diffraction diagram – 135° wave angle.

4. FIELD SITE DESCRIPTIONS

In this section, the following characteristics for each site will be described: a). site exposure direction; b). effective fetch; c). predicted wave characteristics; d). sediment characteristics; e). Morphology and substrate characteristics.

Site 1. Hog Island Headlands (HIH)

The site is located within a long shallow embayment between Hog point and Walnut point (Fig.1.1 and Fig.3.2). The shoreline along the embayment generally faces northeast. The effective fetches are 2.1 nautical miles (northeast), and 3.0 nautical miles (southeast). The historical erosion rate was 2.5 feet per year (Byrne and Anderson, 1978). The tide range is 2.1 feet. The bank along the northern section is approximately 10 feet high and is composed of dredged material (mixture of clay/silt, sands and gravels). Proceeding southward the bank's elevation decreases and becomes a low (about 2 feet) clayey fastland. On the northern end of the site, large concrete blocks were placed along 150 feet of shoreline at MLW in the early 1960's. Midway between the blocks and the southern end of the site is a small erosion resistant bank. These features segmented the shore into two segments.

The beach is composed of medium to coarse sand mixed with gravel and pebbles overlying stiff brown clay. The sand was only about 1 foot thick at MHW before construction. Three rock headlands were built at

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this site in Oct. 1987. Breakwater 1 has a 150 ft crest length, a 4 ft crest width and is 3.5 ft above MHW. Breakwaters 2 and 3 have the same crest lengths of 100 feet, 4 feet crest width and 2 feet above MHW. 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 HIH. The mean diameter of the fill material was 0.4 mm (1.23 Phi). The bank behind breakwater 1 was graded and about 1,200 cubic yards of fill was emplaced. About 600 cubic yards of fill was placed behind each of breakwaters 2 and 3. The new beach fill was placed to an elevation of about 3.0 feet above MHW at the backshore.

The shore normal long term wave climate can be implied by the general shoreline orientation and the aerial photographs. Only very slight changes were shown by historical aerial photographs. There are no significant offsets. The predicted wave height for this site can be seen from Table 4.1.

Site 2. Hog Island Breakwaters (HI2)

This site is located on the western shore of Hog Island, along the shoreline between Virginia Power's Surry Nuclear Power Plant outfall to a marshland at the north end. The baseline is 1,475 feet long. The shore faces west northwest (Fig.1.1 and 3.2) with effective fetches of 2.4 nautical miles (southwest), and 2.5 nautical miles (northeast). The fast land bank is 3 to 11 feet above MSL and consists of mixed clayey sands and gravels in an earthen dike composed of dredged materials from nearby channels. The historical erosion rate along this reach is approximately 1.7 feet per year (Byrne and Anderson, 1978). There is a gabon revetment in the middle of the site that was built in 1962. This

Table 4.1. Predicted Wave Height for each site

SITE	WAVE HEIGHT (2 ft ABOVE MLW, WIND 20 MPH)						WAVE HEIGHT (4 ft ABOVE MLW, WIND 40 MPH)					
	NEARSHORE (m)			OFFSHORE (m)			NEARSHORE (m)			OFFSHORE (m)		
HIH	0.27 (SE)	0.27 (NE)		0.43 (SE)	0.40 (NE)		0.55 (SE)	0.55 (NE)		0.76 (SE)	0.73 (NE)	
HI2	0.24 (N)	0.24 (NW)	0.24 (SW)	0.34 (N)	0.36 (NW)	0.34 (SW)	0.56 (N)	0.56 (NW)	0.56 (SW)	0.64 (N)	0.73 (NW)	0.62 (SW)
CHP	0.27 (N)	0.27 (NW)	0.27 (NE)	0.38 (N)	0.37 (NW)	0.37 (NE)	0.43 (N)	0.43 (NW)	0.43 (NE)	0.73 (N)	0.70 (NW)	0.72 (NE)
DMF	0.27 (S)	0.27 (SW)	0.27 (W)	0.34 (S)	0.38 (SW)	0.38 (W)	0.55 (S)	0.55 (SW)	0.55 (W)	0.72 (S)	0.73 (SW)	0.76 (W)
NPS	0.28 (N)	0.28 (NE)		0.30 (N)	0.36 (NE)		0.55 (N)	0.55 (NE)		0.75 (N)	0.66 (NE)	
YTB	0.31 (N)	0.31 (NE)	0.31 (E)	0.33 (N)	0.49 (NE)	0.61 (E)	0.64 (N)	0.64 (NE)	0.64 (E)	0.66 (N)	1.04 (NE)	1.37 (E)
SUM	0.31 (N)	0.31 (NE)		0.76 (N)	0.73 (NE)		0.61 (N)	0.55 (NE)		1.62 (N)	1.46 (NE)	

Note: N—north, NW—northwest, NE—northeast, S—south, SE—southeast, SW—southwest, W—west.

gabon headland resulted in two types of shoreline configuration at the site. To the south of the headland, there was a narrow beach (3-5) feet from base of bank to MHW) fronting the steep, wave cut bank. The northern shoreline segment had a narrow beach fronting a low, wooded terrace. Net littoral sediment transport to the south is indicated by the impoundment of sand, wider beaches along the northern segment and the faster erosion at the south end of the middle headland. The beaches are basically composed of sand and gravels.

Twelve breakwaters were installed in June, 1987. Six of them were placed south of the gabon headland, and six placed north. Approximately

1,000 cubic yards of beach fill of same particle size as those used at HIH was placed along the northern section, and 500 cubic yards on the southern section.

Site 3. Chippokes State Park (CHP)

Six gaped breakwaters were installed in June, 1987 at this site (Fig.1.1 and 3.4). The site is located between College Run and lower Chippokes Creek. The reach is characterized by high bank (40 feet) and very wide (over 7,000 feet) shallow terrace. The high banked shore faces almost due north and have effective fetches of 3.2 nautical miles (northwest), 3.6 nautical miles (north), and 3.0 nautical miles (northeast). 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. The beach itself consists of a fine to coarse, well sorted, shelly sand derived from the eroding bluff.

This six breakwater system has a length to gap ratio of 1:1.5. The crest lengths are 50 feet and gaps are 75 feet. The center line of the breakwaters is about 30 feet from the initial MHW line. The crest width of each breakwater is 4 feet. Net transport here is eastward but with seasonal fluctuations and onshore offshore movement.

Site 4. Drummonds Field (DMF)

Drummonds Field is a private development located on the north shore of the James River just west of the Jamestown Ferry pier (Fig.1.1 and 3.3). The site faces south southwest. The effective fetches are 2.2 nautical miles (south southeast), 2.8 nautical miles (south west) and 3.8 nautical miles (west). The baseline is about 1,300 feet long. The historical erosion rate is 1.6 feet per year (Byrne and Anderson, 1978).

The 25 ft high bank is composed of a blue grey, very stiff clay overlain by a layer of fine to medium sand. The clay layer is an aquiclude 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 net sediment transport was judged to be down river (to the southeast).

The breakwater system was installed in Sep.,1985. The purpose of the system is to provide a stable, protective and recreational beach for this waterfront development. About 10,000 cubic yards of material was placed with the construction of these breakwaters. An additional 3,000 cubic yards of material were added in Apr.,1986. This was placed mostly behind breakwaters 1, 2 and 3. The sediment characteristics can be seen from table 6.1. and 6.2. A Field observation was carried out in Sep., 1985 when Hurricane Gloria hit. The northwest winds of 50-60 mph generated waves of heights 1.5 to 2.0 feet.

Site 5. Parkway Breakwaters (NPS)

This site is located along the Colonial Parkway in the Colonial National Historical Park (Fig.1.1 and 3.5), between Sandy Point and the piers at Yorktown Navy Weapons Station. The shore faces northeast with an effective fetches of 2.7 nautical miles (north), and 1.6 nautical miles (northeast). The historical erosion rate along this reach is approximately 1.5 ft/year (Byrne and Anderson, 1978).

Five breakwaters were placed in May 1985 along the crenelate embayment between two low headlands. The upriver headland is the terminal end of a rock revetment with a salt marsh fringe in its lee. The downriver headland is actually made up by a narrow salt marsh

fringe. The distance between the two headlands is about 800 feet along which breakwaters were built. The bank and back shore are characterized by shelly, medium sand. The beach is composed of medium to coarse sand and gravel with abundant shell fragments, which was derived from the low bank. Silty fine sands reside offshore. The general trend of littoral drift is downstream, which is indicated by the offset at the downstream end of the revetment and the northerly facing tangential shore between breakwaters.

Site 6. Yorktwon Bays (YTB)

This site has three pocket bay beaches located about one mile downriver from the George P. Coleman Bridge at Yorktown, Virginia. They face east northeast with effective fetches of 4.7 nautical miles (northeast), and 7.8 nautical miles (east). However, there is a long distance of 23 nautical miles to the east through the mouth of York River and across Chesapeake Bay. This site represents estuarine beaches which have been relatively stable over a long period of time. Yorktown bays have evolved over the past 50 years. The headlands separating each bay 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 been developed in the adjacent drainage. 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 (Fig.1.1 and 3.6). The three bays at this site are designated by YB1, YB2 and YB3. YB1 is the largest with approximately 400 feet long from the MHW. It has a slightly crenelate shape and the tangential section of the beach faces

east northeast, showing the influences of both the NE winds in winter
and the long fetch from the east. The sediments on these beaches are
characterized by generally well sorted medium coarse, shelly sand and
gravel. The sand has been derived from historical and continued erosion
of the adjacent headlands.

Site 7. Summerille (SUM)

This site is located on the Potomac River near Smith Point. The
shore faces northeast with effective fetches of 15.5 nautical miles
(north), and 14.3 nautical miles (northeast). Fig.3.7 depicts the
general shape of this crenelate bay. The segment in front of the
Summerille House evolved into headlands after the installation of a
groin field in 1967. A low sill installed in front of Staples house
about 1975 had the effect of slowing the erosion. A bay has evolved
between the Staples house and Summerille house. The Staples' was 50 feet
offshore in 1978. A short rock revetment was placed in front of
Staples. This acts as a small headland. The beach along the embayment
averages about 30 feet wide from the base of bank (BOB) to MHW. It is
characterized by well sorted, medium sands. This material is derived
from erosion of the bay banks and littoral transport which brings sand
into the embayment from the north. The general configuration of the bay
indicates a northeast wave climate and a littoral transport southward
toward Smith Point.

The wind climate can be seen from figure 4.1. The general trends
are that during summer seasons, south winds are dominant; while during
winter seasons north winds are prevail.

RICHMOND
1948-1967

PATUXENT
1945-1970

LANGLEY
1938-1970

NORFOLK
1946-1972

Fig.4.1. Long term wind roses for Richmond, Patuxent,
          Langley, and Norfolk.
          •——• Frequency of occurrence(%)
          •——• Average velocity(kts)
          (Hardaway et al., 1984)

5. RESULTS OF PROFILE AND SEDIMENT ANALYSIS

5.1. The Analytical Results of Field Profile Measurements

The field profile measurements for more than three years show a general two segment profile character from the beach berm offshore: the upper segment is the beach face, the lower segment is the nearshore segment from beach toe offshore. The analysis below shows the general profile characteristics in the fetch-limited environment around coastal breakwaters and headlands.

The profile characteristics can be seen by plotting the water depth, h, against the distance offshore, x, and the statistical analysis. These two parameters are the same as used in Bruun-Dean's 1977 model. Figures 5.1 through 5.17 in this chapter and the figures in Appendixes B, C and D are the results of such analysis.

The selected bays for bay beach face profile analysis are representative of the study area. Once again, the selection of the five bays presented here are the same as those in the Chesapeake Bay Shoreline Study Project selected in advance. More detailed discussions on profiles will be presented in chapter 6.

The results of beach face and nearshore slopes and sediment size analysis are presented in Table 5.1.a and b.

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Table 5.1.a. Results of Slope and Sediment Analysis

SITE	BEACH FACE SLOPE	BEACHFACE SEDIMENT SIZE (mm)	(ϕ)	SORTING	NEARSHORE PROFILE SLOPE	NEARSHORE SEDIMENT SIZE (mm)	(ϕ)	SORTING
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
YTB	0.13	0.67	0.55	0.63	0.015	0.32	1.63	0.82
GRAND MEAN	0.116	0.35	1.52	0.72	0.009	0.17	2.46	0.72

NOTE: The grand mean values for beach face and offshore slopes are the average for all seven sites, the mean grain size did not include these from YTB that are almost 50% larger than the average of others.

Table 5.1.b. Bay Beach Slope and Sediment Size

SITE	POSITION	BEACHFACE SLOPE FALL'89	SEDIMENT FALL'89 SIZE (ϕ)	SORT.	BEACHFACE SLOPE SPRING'90	SEDIMENT SPRING'90 SIZE (ϕ)	SORT.
	Left bay	0.11	1.30	0.52	0.13	1.20	0.79
HI2	Mid bay	0.12	0.95	0.64	0.15	1.07	1.06
	Right Bay	0.13	1.40	0.88	0.14	1.09	1.00
	Left bay	0.11	1.61	0.81	0.12	1.06	0.87
CHP	Mid bay	0.10	1.57	0.59	0.10	1.98	0.53
	Right bay	0.11	1.84	0.42	0.09	1.69	0.52
	Left bay	0.10	1.19	0.60	0.07	1.43	0.93
DMF	Mid bay	0.13	0.93	0.58	0.12	1.50	0.67
	Right bay	0.12	1.26	0.52	0.06	0.89	0.75
	Left bay	0.12	1.88	0.77	0.12	1.87	0.83
WAL	Mid bay	0.13	1.47	1.12	0.14	1.39	1.09
	Right bay	0.11	1.86	0.81	0.12	0.76	0.68
	Spiral	0.16	1.88	0.43	0.16	1.76	0.41
NPS	Mid bay	0.11	1.92	0.44	0.11	1.74	0.56
	Tangential	0.07	1.89	0.55	0.09	1.33	0.77

Note: For the convenience to get a reference on sediment diameter in mm, refer to the following: 0.000 ϕ - 1.000 mm; 1.000 ϕ - 0.5000 mm; 1.5000 ϕ - 0.353 mm; 2.000ϕ - 0.250 mm.

Fig.5.1 NEARSHORE PROFILES AT DMF

Fig.5.2. NEARSHORE PROFILES AT WAL

PROFILE 7.

Fig.5.3. **NEARSHORE PROFILES AT NPS**

PROFILE12

High bank

MAR89
SEP88

MSL

ELEVATION IN FEET

DISTANCE FROM BASELINE (feet)

Fig.5.4. **NEARSHORE PROFILES AT NPS**

PROFILE 12

High bank

OCT89
MAY89

MSL

ELEVATION IN FEET

DISTANCE FROM BASELINE (feet)

Fig.5.5. **NEARSHORE PROFILES AT YTB 1.**

PROFILE 6

MAY88
NOV87

Beach face profile

MSL

ELEVATION IN FEET

DISTANCE FROM BASELINE (feet)

Fig.5.6. **NEARSHORE PROFILES AT YTB 1.**

PROFILE 6

OCT89
MAR89

MSL

Lower profile

ELEVATION IN FEET

DISTANCE FROM BASELINE (feet)

Fig.5.7. A schematic digram showing the measurements on Bay 11 at HI2, Nov.,1989. Profiles No.1 through No.7 corresponse to the slope numbers in Figures 5.8 and 5.9.

Fig. 5.8.

BEACH FACE PROFILE

HI2.

LEGEND

- ▽ NO.4
- △ NO.3
- ◇ NO.2
- □ NO.1

MSL

ELEVATION IN FEET

DISTANCE IN FEET

MEASURED IN OCT.1980

Fig. 5.9.

BEACH FACE PROFILE

HI2, NO.5–8.

LEGEND

△ NO.7
◇ NO.6
□ NO.5

MSL

DISTANCE IN FEET

MEASURED IN OCT.1980

ELEVATION IN FEET

Fig.5.10 A schematic digram showing the measurements on Bay C, CHP, Oct., 1989. Profiles No.1 through No.5 along bay beaches corresponse to the numbers in Fig.5.11.

Fig. 5.11

BEACH FACE PROFILE

CHP, BAY B,

DISTANCE IN FEET

MEASURED IN OCT.1986

ELEVATION IN FEET

Fig. 5.12. A schematic digram showing the measurements on Bay A at DMP, Oct.,1989. Profiles No.1 througfh No.5 along bay beaches corresponse to the numbers in Fig. 5.13.

Fig. 5.13.

BEACH FACE PROFILE

DMF, BAY A, NO1-5

DISTANCE IN FEET

MEASURED IN OCT.1989

ELEVATION IN FEET

LEGEND

$\vdots$ NO5
$\triangledown$ NO4
$\blacktriangle$ NO3
$\diamondsuit$ NO2
$\square$ NO1

Fig.5.14. A schematic digram showing the measurements on
bay A at WAL, Oct. 1989. No.1 through No.5 represent profiles
measured, which corresponse to the numbers in figures 5.15.

Fig. 5.15.

BEACH FACE PROFILES

WALTRIP, NO.1-5

DISTANCE IN FEET

MEASURED IN OCT. 1988

ELEVATION IN FEET

LEGEND

NO.5
NO.4
NO.3
NO.2
NO.1

MSL

Fig.5.16. A schematic digram showing the measurements on bay B at NPS, Oct. 1989. No.1 through No.5 represent profiles measured, which corresponse to the numbers in figure 5.17.

Fig.5.17.

BEACH FACE PROFILES

NPS, BAY B, NO.1-5

MSL

LEGEND

······ NO.5
▽ NO.4
▲ NO.3
◇ NO.2
□ NO.1

ELEVATION IN FEET

5.0 4.0 3.0 2.0 1.0 0.0 -1.0 -2.0 -3.0 -4.0 -5.0

-10 0 10 20 30 40 50 60 70 80 90 100

DISTANCE IN FEET

MEASURED IN OCT.1986

5.2. Sediment Characteristics in the Study Area

Sediments in the study area are basically quartz based materials though sediments of some sites are more shelly (NPS, YTB and CHP). There are no great difference in the average density among the sites, being around 2.65. Beach fills/nourishments were made at HIH, HI2, DMF, WAL and YTB. The beach fill quantity is limited, this can be seen from table 6.1. Most beach fills were already completed at the beginning of this study. The average grain size of the fills are 0.40mm for HIH and HI2, 0.45 for DMF and WAL.

Sediment samples are taken along selected profiles. As pointed out by Boon (1988, p1626) when considering sediment diameter, D, as one of the environment factors the more representative sediment samples are those taken from the beach face and offshore. The sediment samples taken near the wave step were too coarse and extent limited. The representative grain diameters were taken as the average of a few sediment samples taken on different dates but from the same beach face for upper segment and nearshore for the lower profile segment. The final results of sediment analysis are also presented in Table 5.1. In this table, the Moment Measure Mean, is used when expressing average grain size resulting from a computational method (not graphical method). It gives a truer picture than the graphic method. This is because it is supposed that every grain in the sediment affects the measure in the computation. However, the median obtained graphically is merely the reading of the diameter at the 50% mark of the cumulative curve. It is not affected by the character of the rest of the curve. Table 5.2 shows the horizontal sediment size distributions along bay beaches between

breakwaters and headlands sampled in Oct.1989.

Table 5.2. Sediment Size Distribution Along Bay Beaches

SITE No.

LEFT BAY		MIDDLE BAY (ϕ)	RIGHT BAY (ϕ)	(ϕ)
HI2	1	1.425	1.539	1.395
	2	0.281	1.479	1.036
	3	0.328	1.310	1.226
	4	1.181	1.123	1.683
	5	2.005	1.923	2.487
	MEAN	1.044	1.475	1.565
	TOTAL MEAN = (1.044+1.475+1.565)/3 = 1.361			
CHP	1	1.513	1.441	1.267
	2	1.900	1.371	1.847
	3	1.724	1.074	1.665
	4	2.063	1.845	1.709
	5.	2.570	2.474	2.207
	MEAN	1.952	1.641	1.739
	TOTAL MEAN = (1.952+1.641+1.739)/3 = 1.777			
DMF	1	1.570	1.331	1.755
	2	1.412	0.871	1.480
	3	0.601	0.612	0.571
	4	2.424	1.991	1.106
	5	2.469	2.913	2.203
	MEAN	1.695	1.544	1.423
	TOTAL MEAN = (1.695+1.544+1.423)/3 = 1.554			
WAL	1	1.182	0.949	0.820
	2	1.553	1.485	1.791
	3	1.075	0.908	1.261
	4	2.786	2.020	2.633
	5	2.985	3.072	3.192
	MEAN	1.916	1.687	1.939
	TOTAL MEAN = (1.916+1.687+1.939)/3 = 1.847			
NPS	1	1.662	1.787	1.819
	2	1.714	1.708	1.541
	3	1.749	1.651	1.683
	4	2.116	2.354	2.346
	5	2.222	2.273	2.268
	MEAN	1.893	1.955	1.931
	(spiral part)		(tangential part)	
	TOTAL MEAN = (1.893+1.955+1.931)/3 = 1.908			

Table continued:

SUM				
	1	0.114	1.003	0.391
	2	1.103	0.814	0.766
	3	0.480	-0.034	-0.252
	4	0.220	1.151	1.196
	5	0.902	1.519	1.750
	MEAN	0.564	0.891	0.770
		(spiral part)		(tangential part)

TOTAL MEAN = (0.564+0.891+0.770)/3 = 0.474

YTB			
	1	0.696	0.786
	2	0.944	0.551
	3	0.555	0.264
	4	-0.133	0.447
	5	0.991	2.104
	MEAN	0.611	0.831

TOTAL MEAN = (0.611+0.831)/2 = 0.721

were collected. 1 — backshore; 2 — berm; 3 — beachface; 4 — beach toe; 5 — offshore.

6. DISCUSSIONS AND CONCLUSIONS - PROFILES

6.1. General Characteristics of Nearshore Profiles

From the data analysis in the previous chapters, a general picture of beach and nearshore profiles can be obtained: the whole profile is two segmented. The upper segment is the beach face, from the beach berm to beach toe or wave step (Dean,1990, and refer to Komar, 1976 for the definition of beach face). This segment is more or less linear, and best represented by

$$h = M_1 x \quad \text{or} \quad h = x\tan\beta_1 \quad (6-1)$$

where M_1 is beach face slope, x the distance offshore from beach berm (Dean used h=Ay in his 1990 theoretical work). The lower segment is the offshore section, from beach toe waterward. This segment can be well described by either

$$h = Ax^m \quad (6-2)$$

or

$$h = M_2 x \quad \text{or} \quad h = x\tan\beta_2 \quad (6-3)$$

where M_2 is the nearshore profile slope. Equation 6-2 has the same mathematical form as Bruun-Dean's model, but the mechanism leading to such a profile is not totally the same. This is because that this formula was originally derived based on the assumption of uniform average wave energy dissipation rate per unit water volume (D.) under FULLY dissipative surf zone condition of open ocean environments. The

areas of this study are located in very fetch—limited environments. The wave climate is incomparable with those of open ocean. The surf zone width is also incomparable with that of open ocean. It is very narrow in this study area and sometimes disappears. Therefore, the mechanisms of development of profiles can not be totally the same (see section 6.2 for further discussions).

The variation of the profiles from a straight line was low, being only 0 — 4% of the total variance for beach face; and 3.5% of the total variance for lower profile. The average value of M_1, the beach face slope, is 0.12 in this study (Fig.6.1 and 6.2). The average value of m, shape factor, is 0.15, and A, the scale factor, is 0.56 (See Appendix A for the detailed calculation of these two factors). Dean's (1977) m and A ranges are [0.5,0.8] and [0.0025,6.31], respectively. The average value of M_2, the nearshore profile slope, is 0.01, which is a little bit smaller in comparison with a range of the value of this parameter [0.012, 0.020] in the study by Wright et al.(1979). The average beach width in this study is 31 feet with a range [21, 43]. The profiled distance of the lower profile waterward is variable depending on the tidal stage when profiling was made.

When talking about the fitness of a certain mathematical model to the real data, one thing that should be noticed is the "unrealistic" properties that represented by the model. For example, one of the main improvements Dean made in his 1990 work is that on the unrealistic property of the form of the equilibrium beach profile which has an infinite predicted slope at the shoreline. Other examples can be seen from the fitness of Yasso's spiral bay model, LeBlond's mathematical

Fig.6.1.

Beachface
in this study
h=xtanβ₁
(tanβ₁=0.12)

Lower profile
in this study
h=Ax*+0.64
(A=0.56, m=0.15)

Boon (1988): h=Ax*
(A=0.34, m=0.58)

Dean (1977): h=Ax*
(A=0.36, m=0.67)

DISTANCE (METER)

DEPTH (METER)

Fig.6.2.

Beachface
in this study
h=xtanß₁
(tanß₁=0.12)

Lower profile
in this study
h=xtanß₂+1.10
(tanß₂=0.01)

The linear
Beachface
(Wright,1979)
tanß₁=0.12-0.20

The linear
lower profile
(Wright.,1979)
tanß₂=0.012-0.020)

Boon (1988)
h=Ax^m (A=0.34,m=0.58)

Dean (1977)
h=Ax^m (A=0.36,m=0.67)

DISTANCE (METER)

DEPTH (METER)

4.00

1.00

6.00

11.00

10.00

40.00

90.00

140.00

shoreline model (1972). The same thing is true in this study. When one connects the two linear profile segments together, the joining point/angle is unrealistic in terms of the real world profiles. There is no profiles that have a perfect mathematically described sharp angle between the two segments. However, this unrealistic property represented by the two segment model is much more reasonable than that represented by the single curve model (at least in this study).

The two segment profiles appeared in figures 6.1 and 6.2 from this study are established within the same coordinate. The starting point of the lower profile is the lower end of the beach face profile. Because of the unrealistic properties of the very beginning part of the profile represented by the lower profile model, this part does not appear in the figures of 6.1 and 6.2. Although there is no reference or method in the literature dealing with this kind of jointing of model curves, from the good fitness represented by these curves to the real profiles in the study area it can be said that this kind of method is reasonable.

Boon (1988)'s profile is different from Dean (1977)'s, being more concave upward near the beach face with a average shape factor value equal to 1/2. It is probably because Boon's data was from a different environment with different beach state and carbonate based materials which response differently to the local wave climate than Dean's mostly quartz based beach materials.

According to Wright et al (1979), the steep, linear beach face and the linear gentler nearshore profile are one of the major morphological characteristics of reflective beaches (Fig.6.3). The beach profile in figure 6.3 is very similar to those in this study. The surf scaling

Fig.6.3. Typical cross-sectional profile of a reflective beach (after Wright 1979).

parameter, ϵ,is one of the main standards of beach state classification (Guza and Brown, 1975; Guza and Inman, 1975, Wright 1979 and 1983). It has been shown theoretically and in the laboratory that wave breaking characteristics, runup amplitude, and the degree of inshore resonance are dependent on this parameter. Table 6.1 shows its variation in this study (refer to Table 6.3 for the parameters that are used to calculate the surf-scaling parameter).

Table 6.1. Surf scaling parameters in the study area

SITE	HIH	HI2	CHP	DMF	WAL	NPS	YTB	AVERAGE
ϵ	4.03	3.64	4.23	4.76	4.14	4.03	3.47	4.04

The average ϵ value of 4.04 should represent a situation of partial wave reflection environment, more detailed discussions on this will be presented a little bit later.

It is rather obvious that Dean (1990) made a great improvement theoretically on his previous model in his new work (Fig.6.4). It is believed that such an improvement was resulted from the fact that the mathematical formula $h=Ax^m$ can not predict a non-monotonous curve or profile. A certain form has to be introduced to predict the two-segment characteristics of some beach profiles in nature. The early Bruun-Dean's model is only a very general theoretical one. When one goes further to apply this model to real situations, problems arise: where should be the starting modeling point on a beach? from beach berm? mean high high water? mean sea level? or mean low low water (beach toe in this study)? The beach face profile can not be excluded from the discussions and predictions of beach and nearshore profiles. It is these problems at

Fig.6.4. Comparisons of Dean's new work (1990) and the work in this study of beach profiles.

least partly that made Dean improve his previous work greatly. This
shows again the fact that like wave prediction the state of the art of a
certain model on beach processes largely depends on the increase of data
collection, when data accumulates model evolves.

6.2. The Mechanisms of Profile Development

At present stage, there is still not a totally satisfactory
explanation on the mechanisms of beach and nearshore profile
development. This can be seen from various points of view in the
literature and the ever changing or improvement on the previous models
with the increasing accumulation of data and analysis.

The following discussions will be mainly around two groups of
ideas: The first one is represented by the Bruun-Dean's model being
started in 1954. The focus has been around the model form, $h=Ax^M$, and
its origination or the mechanisms of profile development, being more
heavily relied on the traditional descriptive methods. The second one is
represented by the work done by Inman and Guza (1975), Wright et. al.
(1979, 1982a, b, c; Short,1979; Wright and Short 1983, 1984). In this
group, the focus has been on the beach morphology and dynamics, or
"morphodynamics" which is a term frequently used in the studies by
Wright et al. One of the obvious characteristics of their work is the
more rigorous analysis on the nearshore dynamics such as the wave
behavior in surf zone. They classify beaches into dissipative and
reflective extremes and the intermediate states between the extremes.

Like solving a complicated mathematical problem, there may exist a
couple of ways to get the same correct answer. The co-existence of
different ideas is a good way to get the final goal. The following

discussions will focus on these two groups.

(1). As briefly mentioned earlier the Bruun-Dean's model was derived theoretically from the mechanism of uniform wave energy dissipation rate per unit water volume in fully dissipative surf zone. The wave dissipation is due to breaking of waves in the surf zone in which the ratio of H_b/h is basically constant ($k=H_b/h=0.78$). The bottom friction is insignificant which only occurs in the bottom boundary layer. Because of the peculiar characteristics such as the limited variable fetch size, not fully dissipative surf zone, complex shoreline configuration and bottom topography in this study, it is obvious that the mechanism of the profile development proposed by Dean can not be simply moved to this study without any change. Here some efforts are made to try to explain the mechanism of profile development in fetch limited environment.

Because there are still some similarities in the process of profile development in the environments between open ocean and fetch limited river systems, it would be helpful to give a short brief summary here on some relatively new work in this group on the mechanisms of beach profile development.

As implied in Dean's new work (1990) that the establishment of a rational physics-based prediction of equilibrium beach profiles is still difficult due to the complex force system acting on sediment particles. However, in a broad sense, it is rather obvious that sediment particles are acted on by a complex system of constructive and destructive generic "forces" acting to displace the sediment particles landward and vice versa. Constructive forces include landward directed bottom shear

stress due to the nonlinear character of shallow water waves, landward directed "streaming" velocities in the bottom boundary layer (Bagnold, 1946; Phillips, 1966), the phasing associated with intermittent suspended sediment motion, etc. The most obvious destructive force is that of gravity coupled with the destablizing effects of turbulence induced by wave breaking. Others include the effect of seaward directed bottom undertow currents and forces due to wave set—up within the surf zone (e.g. Svendsen, 1984; Stive and Wind, 1986).

When explaining beach profile development in Caribbean area, Boon (1988) pointed out that the main processes driving nearshore sediment movements are: asymmetric waves, rips, combined wave and steady currents, downwelling and upwelling, gravity flows and "groupy" waves (Wright, 1987).

The most important factors controlling the beach processes in the fetch limited environments of Chesapeake Bay were summarized by Byrne and Anderson (1978). These factors are: the intensity of wave action, the exposure to strong tidal currents, the orientation of shorelines and sediment characteristics. The raised water level by wind set—up during storms in conjunction with the higher tides during the lunar month can enhance the beach erosion. The orientation of the tributary estuaries is also important. The most frequently happening northeast storms attack western shorelines of tributaries with fetches to the northeast and east. All these factors contribute to the development of profiles.

In Dean's new theoretical work, he consider the gravity force as one of the important destructive forces, especially in the development of beach face profiles,

$$\frac{D_*}{M} \frac{\partial h}{\partial x} + \frac{1}{h} \frac{\partial}{\partial x} (EC_G) = D_* \qquad\qquad (6-4)$$

where the two terms on the left hand side consist of the destablizing

forces due to gravity (first term) and turbulent fluctuations (second

term) due to wave energy dissipation. M is beach face slope, D_* is the

average wave energy dissipation rate per unit water volume, x is the

distance offshore from beach. Equation 6-4 can be integrated to

$$x = (h/M) + h^{3/2} / A^{3/2} \qquad\qquad (6-5)$$

where A is related to D_* by equation 6-6.

$$A = [24D_*(D)/5dg^{3/2} k^2]^{2/3} \qquad\qquad (6-6)$$

where g is gravity, D sediment particle size. $k = H_b/h = 0.78$ under fully

developed wave condition. In shallow water or beach face, the first

term in equation 6-4 dominates, simplifying to

$$h = Mx \qquad\qquad (6-7)$$

In deeper water, the second term dominates with the simplification

$$h = Ax^m \qquad\qquad (6-8)$$

As stated earlier, Dean's model was derived based on the uniform

wave energy dissipation rate per unit water volume under fully

dissipative surf zone condition. However, in fetch-limited environment,

e.g. the area in this study, the wave climate is incomparable with those

of open ocean. Therefore, the mechanism of profile development differ

in some aspects. In the case of open ocean, sediment particles are

always actively under the influence of wave actions such as the

reflection of wave energy, the turbulence from the dissipation of wave

energy and the gravity effects. Sediment particles are constantly

moving around toward the best equilibrium position with the ever

changing wave conditions. In fetch-limited environment with complex

shoreline configurations, however, the influences on sediment particles

from waves are smaller. The wave period and length can be as smaller as

five to ten times than those of open ocean (wave periods are 2-3 seconds

in this study comparing with the common wave period in ocean of more

than 10 seconds). The scale of turbulence from waves is much small and

the wave energies dissipated on the sediment particles on the bed are

much less. The turbulence is only large enough to rigorously disturb

sediment during severe weather condition like storm waves. The 2-3

second waves in the study areas are not strong enough to move sediment

particles in a rigorous manner. This basic difference in the general

dynamical background has to be considered in the application of

Bruun-Dean's model when applied in fetch limited environment. The lower

profiles are relict from a time when forces were actively at work, being

a settling basin. Large quantities of fine sand/silt can stably stay on

the bed, no much is moved to the further offshore region. The result of

such a deposition of fine materials due to the low wave energy and

weaker turbulence is the formation of a gentler bed topography with near

linear slope.

(2). Another explanation on the mechanisms of two segment beach

profile development is based on the reflective beach characteristics

(Inman and Guza, 1975; Wright et al. 1979, 1982, 1983).

According to Wright et al., the steep, linear beach face and the

linear gentler nearshore profile are one of the major morphological

characteristics in reflective environments. The beach reflectivity is

determined mainly by the incident wave amplitude near the breaker

height, wave period, acceleration of gravity and the beach slope.

$$\epsilon - a_1\omega^2/g \ \tan^2\beta \qquad (6-9)$$

Based on the value of this parameter, beaches can be classified into

dissipative, reflective and intermediate domains of beach states (refer

to chapter 2 for a review on this).

The average ϵ value in this study area is 4.04 (Table 6.1) with the

range [3.47, 4.76], which indicating a situation of partial reflection

environment. The average beach face slope is 0.116, which means the

beach angle, β, being greater than 6° , which is one of the two major

factors described by Wright et al: e < 2.5 and β > 6° for strong

reflective beaches. Therefore, the profiles in this study have the

similar appearances as that in figure 6.4 by Wright et al (1979). One

thing that should be pointed out here is that some important

morphological and dynamical features commonly found in Bowen, Inman

(1975) and Wright's studies are not common in this study of

fetch-limited environment such as the much larger wave parameters, some

well evolved rhythmic beach forms, "unlimited fetches" etc. With all

these differences in the general morphodynamic background between open

ocean and river water environments and the poorness of study references

in the literature on nearshore processes in fetch-limited river systems,

a rigorous establishment of the mechanisms on profile development in

fetch-limited environment would be an interesting topic large enough for

future study. Such a study would need more detailed wave data and

analysis like what Wright did (1979, 1982, 1983).

Two major limitations in using the two segment model proposed in

this study should be considered: (1). The two segment model is only

valid within the nearshore region. Special cautions should be exercised when using the model to predict profiles in the region beyond 200 meters from beach. This is because the model was established based only on the measured profile data within the region of 150 meters from beach. (2). The two segment model should be used only in fetch limited tributary systems.

6.3. Beach Slope and Sediments

Sediments around breakwaters and headlands are mostly derived from local bank erosion, either from the bank behind structures or from adjacent bank erosion. Some fine sediments such as silt and clay are moved by longshore currents, mostly tidal currents as stated by Byrne et al (1978). The coarser sediments like sands and gravel can only be moved alongshore by obliquely incident waves generated by strong winds or wind "set-up" during storm time (Byrne, 1978). It is observed in field that there are often no waves at all during calm weather conditions or weak winds (below 10 MPH) blowing in a direction from land to water. Under such conditions, longshore movement of coarser materials dose not occur.

From table 5.1 in chapter 5 and table 6.2, one can get a general picture of sediment characteristics and bank erosion rate in the study area at which the bank has been supplying materials to the water in the past century. One thing should be pointed out that the erosion rate in the table represents the average of a larger section of shore line including the study site. It dose not necessarily accurately represent the erosion rate at the site in this study in the past few years. It only serves as a general reference. For more detailed information,

refer to Byrne et al (1978).

Table 6.2 Bank Erosion History And Beach Fill

SITE	REACH #	EROSION RATE (ft/100yr)	BEACH FILL (yds^3)	BEACH FILL GRAIN SIZE (mm)
HIH	197	2.8	1,000	0.40
HI2	196	1.7	1,500	0.40
CHP	189	——	——	——
DMF	305	2.4	10,000	0.45
WAL	305	2.4	——	0.45
NPS	22	0.0	——	——
YTB	25	0.7	——	0.69

Note: —— means no sure data available. For detailed information on erosion rate refer to Byrne et al (1978).

Study shows that after approximately one or two years of the construction of breakwaters and/or headlands, bank erosion generally stops for most sites like those: NPS, DMF, WAL, YTB and CHP. Only two sites are still experiencing bank erosion almost three years after the construction of structures, HIH and HI2. HI2 is a site with which experiment of breakwater design and construction is ongoing. Six pairs of breakwaters were set up in June, 1987. Five pairs of those are proven not very successful due to either being constructed too close to the high bank or being undersized. High banks in between breakwaters still face erosion whenever storm surges come into the area, especially during high tide stage of the lunar month which was already stated by Byrne et al (1978). The bank erosion in between headlands at HIH is within expectation as designed in advance. The final plan geometry of this section will be two crenelate shaped embayments in between three headlands. However, the development of such a final plan shape is a slow process due to the small size of fetch, weak wave climate and hard

clayey erosion resistant bed.

Sunamura (1984) suggested the best fit equation for beach slope prediction in field based on extensive field data (refer to chapter 2 for more detailed review) to be:

$$\tan\beta_1 = 0.12/[(H_b/g^{0.5}D^{0.5}T)]^{0.5} \qquad (6-10)$$

The results of our data analysis (Fig.6.5) are reasonably fit the model suggested by Sunamura (refer to chapter 2 for Sunamura's figure). This means that the selection of the four major parameters in his model is reasonable. These parameters are also included in many other researchers' papers as important factors in the nearshore processes. The empirical constant in his model may be changed to a certain degree to predict beach slopes in different environments. However, such a serious change should be based on much more accurate data in future study. At present stage, the most difficult thing in this kind of beach slope prediction is still the collection of accurate wave data. This is mostly because of the complexity of the nearshore nonlinear wave actions coupled with many other time varying factors. However, it is expected that with the increasing of field wave data collection and the development of various wave predicting models, wave prediction in the nearshore zone will get to the point very close to the nature reality. Table 6.3 presents the field measured data and Sunamura's dimensionless parameter.

Figure 6.5.a-f. Diagrames showing the relationship among waves, sediment diameter and accelerational gravity based on Sunamura's (1984) model.

a. HI2

b. CHP

c. DMF

d. WAL

e. NPS

f. YTB

Table 6.3. Field Measured Data and Sunamura's Parameter

SITE	DATE MEASURED	WAVE HEIGHT (m)	PERIOD (sec)	GRAIN SIZE $(X10^{-4}m)$	BEACH SLOPE MEASURED	$h_b/g^{0.5}D^{0.5}T$
HI2	OCT87	0.145	2.5	4.4	0.118	0.92
	MAY88	0.140	2.0	2.5	0.119	1.12
	DEC88	0.156	2.1	6.1	0.120	1.19
	JUN89	0.152	2.4	6.6	0.121	1.01
	MAY90	0.141	2.3	6.6	0.123	0.98
CHP	OCT87	0.142	2.0	5.4	0.107	0.99
	MAY88	0.140	2.1	3.9	0.109	1.07
	DEC88	0.146	2.2	4.3	0.112	1.00
	JUN89	0.153	2.0	4.4	0.108	1.17
	MAY90	0.139	2.1	2.6	0.111	1.32
DMF	OCT87	0.156	2.1	3.5	0.112	1.20
	DEC88	0.137	2.1	5.1	0.113	0.91
	APR89	0.132	2.3	4.0	0.109	0.92
	MAY90	0.158	2.3	7.7	0.108	0.81
WAL	DEC88	0.154	2.2	5.0	0.119	0.93
	MAY90	0.158	2.0	5.4	0.122	1.10
NPS	OCT87	0.155	2.2	3.1	0.109	1.26
	MAY88	0.130	2.1	3.2	0.105	1.10
	DEC88	0.131	2.4	3.5	0.111	0.92
	MAY90	0.147	2.4	3.2	0.113	1.09
YTB	OCT87	0.23	2.6	7.0	0.127	1.08
	MAY88	0.31	2.7	7.7	0.129	1.21
	DEC88	0.32	2.7	8.4	0.131	1.31
	MAY90	0.28	2.5	7.9	0.130	1.27

The bay beaches are very similar in slope for most bays in between breakwaters with smaller gaps such as these at HI2, CHP and WAL. All these bays have a near half-circle plan bay shape. However, there are variations along bay beaches for some bays with larger gaps between breakwaters with a elongated bay plan shape. The bays at DMF are the examples. Along the bay beaches, the general tendency is that slopes increases toward the central bay. The sediment size on the central

beach is coarser (0.85 ϕ), it is finer on both sides (1.45 ϕ), which is consistent with some of the traditional explanation.

6.4. Profile Variations With Seasons/wave Climate

From frequently field profile measurements at fixed profile locations, it is noticed that not only short-term profile variation occurs all the time, but also systematic long-term variations. Here short-term refers to days and weeks, and long-term to seasons and years. The most significant variation is seasonal changes. A series of names have been given to profiles formed in different seasons such as summer profile (swell profile, normal profile) and winter profile (storm profile). Traditionally, summer profile is described as a profile with a wide berm and a relatively smooth offshore profile, and winter profile is described as one having almost no berm, the sand having been shifted offshore to form a series of bars parallel to the shore. In our very fetch-limited environments with breakwaters and headlands, these traditional descriptions seem to be not sufficiently inclusive, because all data from the seven study sites do not fit the traditional descriptions of the mechanisms for winter and summer profile development. There were no significant bars forming during winter due to the sand shifting from beaches at all seven sites (see all profile plots for the seven sites in Appendixes) and therefore there were no bars to disappear in summer time. However, sand does shift from beaches to offshore insignificantly during winter though no bars were formed. This kind of sand shifting or on/offshore motion can be seen commonly from the data plots in Appendixes.

6.5. Patterns of Sediment Distribution Along Profiles

The patterns of grain size distribution along profiles are believed to be one of the reflectors of dynamical strength exerting on nearshore region. The average grain size at a certain point on a profile represents the particle size that is in a relatively dynamical equilibrium with the forces in fluids and its own gravity. If the grain size is too big to be moved by fluid forces, it simply stays there. If the grain size is too small than what it should be for an equilibrium state it will be suspended in the water column or moved to other places where the environment is suitable for it to settle out. Sediments with different sizes are always being sorted by mainly fluid forces and gravity. The general tendency is that every particle on the very surface of a profile moves around to a location where it could be settle down more stably, which depends on its own size, density, local slopes, the strength of fluid forces and hence its incipient motion velocity. The final consequence of such a process is the zonation of sediment sizes, or sediment sorting.

Based on many field observations and the results of sediment analysis in VIMS' sediment laboratory, a general pattern of sediment distribution was obtained along profiles (Fig.6.6). This pattern divides the nearshore region into five general zones:

1). Bank zone — from the line between backshore and bank landward, which provides most of the beach and nearshore material.

2). Backshore zone — This is the zone where both water and winds act. It usually consists of well sorted sands.

3). Beach face zone — This is the zone where sediments most actively adjust themselves to the tidal and wave actions. A narrow

Fig.6.6. A schematic diagram showing a general pattern of
sediment distributions along a profile.

strip of coarse material (mostly gravel) was found to move up and down with the instantaneously changing water level, being basically consistent with the colliding zone between backwash and incident waves.

4). Beach toe zone — this is a zone where particles most poorly sorted (The particles great than 2 mm are excluded in the results presented in Chapter 5). Particles larger than the gravel size are very common in this zone. These large particles are mostly from the bank material and sorted by mainly gravity and turbulence. As mentioned earlier the nearshore profile is broken here. According to the field observations during storms, this is also the zone where waves break during severe weather conditions.

5). Offshore zone — from the beach toe offshore. This is the zone where sediments are the finest and best sorted. Fine sands/silt dominate. They are mostly from the fine part of the bank material and adjacent shore sections.

7. RESULTS OF PLAN-VIEW BEACH FORM MEASUREMENTS

Four basic plan-beach forms can be generally recognized based on:
(1) low aerial photographs; (2) frequent profiling; (3) records from
field observations; (4) detailed measurements of selected bays. The
four types are as follows:

7.1.Circular Bay Beach Forms

This type of bay beaches generally evolves in regions well
sheltered by breakwaters. This type of bay beaches is found at four of
the study sites: HI2, CHP, DMF and WAL. They represent the most stable
beach forms along shorelines sheltered by breakwaters, but they also are
the most expensive to install.

Breakwater systems with circular bays generally have smaller gaps
between breakwaters and are larger in relative sizes than other
breakwater systems with irregular bays. Fig. 7.1 shows the details of
the arrangement of breakwaters at CHP. The closely spaced breakwaters
provide good shelter to the bay beaches. In Fig. 7.2, we can see a
variety of beach forms. The original purpose of HI2 is to examine the
effects of breakwaters of varying lengths, heights, and offshore
distances. From this figure, we can see that the best evolved circular
bay is formed between the two largest breakwaters, 11 and 12. Two other
circular bays are also formed in relatively well sheltered areas, bays
between break-waters 2 and 3, 10 and 11. Figure 7.3, 7.4 and 7.5 exhibit

Fig.7.1. Breakwater system and its embayments at CHP.

Fig.7.2. Breakwater system and its embayments at HI2.

JAMES RIVER

Fig.7.3. Breakwater system and its embayments at DMF.

WALTRIP
JAMES RIVER

EBB
FLOOD

F 1 A 2 B 3

1 2 4 7 11 14 16

POOL

————— SEPT 86 MHW
— — — MAR 89 TOE
··········· MAR 89 MHW

60'
GRAPHIC SCALE

Fig.7.4. Breakwater system and its embayments at WAL.

Fig.7.5. A photo showing the Breakwaters and embayments.

some other circular bay beaches at DMF and WAL. Because of the smaller

ratio of breakwater crest length to gap, the bay beaches at DMF are

formed not as regularly as those at CHP, WAL and some at HI2.

7.2. Spiral Bay Beaches

Spiral bay beaches are well evolved only at these sites with higher

energy levels and obliquely incident waves. The only well evolved

spiral bay is found at site SUM (Fig.7.6 and 7.7), the site with the

highest energy in the study area. This bay retains its spiral plan form

all the year round. A couple of other spiral bays are not well evolved

at site NPS and YTB. Especially at NPS (Fig.7.8), the spiral bays are

sensitive to the variations in seasonal wave climate. They can not

retain their spiral form well year around. Spiral bays can only be

formed during winter storm season. During summer, the tangential

sections of the bays are likely destroyed by the southeastly wave

climate. The YTB site (Fig.7.9 and 7.10) is the most dynamically

complicated site in the study area. The beach forms are not only

dependent as heavily on the seasonal wave climate as other sites, but

also on the general characteristics of the geological setting. The east

wave climate is not as important to beach forms at other sites, but at

YTB with a horn-shape shorelines at the lower York River and a more than

20 nautical mile fetch from the east all the way to the eastern shore of

Chesapeake Bay, east wave climate plays a more important role than waves

from other directions in influencing beach forms at YTB. The nearly

east facing of the tangential sections at this site clearly indicates

this.

Fig.7.6. The spiral bay at SUM.

Fig.7.7. A photo showing the general spiral bay shape at SUM.

Fig.7.8. Breakwater system and its embayments at NPS.

YORK RIVER

Fig.7.9. Headlands and their bays at YTB.

York River

Beach

Yorktown cliff

Fig.7.10. Photo showing the Headlands at YTB.

7.3. Irregular Bay Beach forms

The word "irregular" may be improperly used here. It only means these beach forms between circular and spiral. The beach forms of this type are mostly transitional. They may finally evolve into either circular or spiral bay beaches. Some of the bay beaches which appears to be irregular may just due to erosion-resistant spots within the bay such as the stiff hard clayey bed or tree roots.

Part of the plan beach forms at HIH belongs to this type (Fig.7.11). It is located within a fetch-limited environment. The very stiff, hard clayey bed and bank material is erosion resistant. The progress of the beach adjustment to the local wave climate is a fairly slow and long term process. Other irregular beach forms can also be found at HI2 (Figure 7.2). More detailed discussions will be addressed in chapter 8.

The straight beach form is an insignificant characteristic for all study sites. Only a small section at HIH between headland 2 and 3 is found to be somewhat straight. This kind of straight beach is similar to other regular straight beaches out of the structure systems, because the two headlands on each end were originally installed on a basically straight section of shore, and far apart, the middle straight section needs more time to adjust itself to the installation of headland structures. An embayment will be expected in future though it is a slow process.

Fig.7.11. Three headlands and the plan beach forms at HIH.

8. DISCUSSIONS AND CONCLUSIONS – PLAN BEACH FORMS

8.1. General characteristics

As mentioned in section 7, the plan beach forms can be generally classified into four broad groups in the study area: 1). Circular bay beaches such as those at CHP, DMF, WAL and some at HI2; 2). Spiral bay beaches like these at SUM and some at NPS and YTB; 3). Straight beach form like the small section between the headlands 2 and 3 at HIH; 4). Irregular bay beaches such as those at HIH and some at HI2 (Table 8.1).

The general rule is that whenever there is a breakwater or headland system installed along a sand beach, bay beaches will be formed immediately, either spiral or circular, depending on the local wave climate, sediment characteristics and the geological setting of the site. Circular bay beach forms are usually formed at these sites with closely spaced breakwaters; the main incident wave direction is nearly normal, especially during storm seasons there are no significant obliquely incident waves from a particular direction. In contrast, spiral bay beaches are usually formed at those sites with wider spaced breakwaters or headlands; the overall energy level is relatively higher (e.g. the sites SUM, NPS and YTB all have higher energy level); the incident wave direction during severe storms is oblique relative to the general site orientation. Sediment characteristics are not crucial factors in determining the plan beach forms.

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Table 8.1 Summary Table of Beach Forms

FORMS	TYPE	DESCRIPTIONS	GENERAL SHAPE/EXAMPLE
P L A N B E A C H F O R M S	Circular Bay Beaches	This type of bay beaches are generally formed due to one or more of the following reasons: (1). low energy level; (2). small gap between BWs. (3). normally incident waves; (4). sediment supply aplenty.	
	Spiral Bay Beaches	Spiral bay beaches may be evolved due to the following reasons: (1). higher energy levels;(2). obliquely incident waves; (3).headlands; (4).sandy beaches.	
	Irregular Bay Beaches	This type of bay beaches are mostly in a transional phase.The irregular shapes may be eventually evolved into circular or spiral bay beaches, depending on the local environmental factors.	
	Straight Beach	This is a very insignificant beach form in the study area. It is only found in between headlands at HIH.	
P R O F I L E		Profiles in the study area are two-segmented: Beach face profile: h=Mx (M=0.11); The lower profile: h=Ax^m, (A=0.56, m=0.15).	Beach face MSL

The mechanisms of the formation of various plan beach forms are not only related to the general geological and physical characteristics as briefly mentioned above, but also are closely related to other subsequent wave related dynamics such as wave refraction, diffraction and reflection in the vicinity of structures. Wave refraction, diffraction and reflection are three of the most basic mechanisms in shaping beaches into various forms around coastal structures. The following are discussions on these mechanisms based on field observation data.

8.2. Plan Forms and Wave Refractions

Waves are subject to refraction as they enter shallow water. Wave orthogonals change direction with decreasing water depth in such a way that the crests tend to parallel the depth contours.

A spreading or convergence of wave orthogonals/wave rays can be caused by wave refraction, which also means the spreading or convergence of wave energy. That is, when the wave rays spread out as the shore is approached, the wave energy per unit wave length is reduced. The wave energy flux (or wave power), P, between the rays is constant. The spreading due to the refraction causes the same amount of P to be distributed over a longer wave crest as shown in figures 8.1 a and b. The conservation of wave energy flux can be expressed by a relationship (Komar, 1976)

$$P - E \ Cn \ L_a - E \ Cn \ L_{ao} \qquad (8-1)$$

where E — wave energy (or energy density); C — phase velocity; n — $\{[1+2kh/\sinh(2kh)]\}/2 - C/C \to 1$ (in nearshore shallow water); C here is

Fig.8.1. Wave refractions in waters with curved shorelines(a) and straight shoreline(b). Big arrows represent schematically the decreasing of energy flux, ECn, per unit wave length as waves approaching the beach.

the group velocity; L is the width between wave rays, the subscript o represents the deep water condition. In Fig.8.1a, the shoreline is not straight. As waves enter into the depression, wave rays are refracted so that the rays could be as perpendicular to the depth contours as possible. Figure 8.1b is an example of straight shoreline with almost parallel depth contours. In this case, as waves enter into the shallow water, wave crests tend to parallel to the depth contour. Wave energy is reduced on approaching the beach due to the widening/ spreading of wave rays. Figures 8.2 and 8.3 are examples showing wave refractions around coastal breakwaters.

In the case of circular bays formed between breakwaters (figure 8.2), wave energy is reduced as waves approach the bay periphery due to wave refraction, while wave energy is increased at the ends of breakwaters due to the converging of wave rays. This kind of method of making wave rays spreading and converging at desired points is actually one of the principles that is used for shoreline protection using structures like breakwaters. Figure 8.2 is a schematic diagram that represent the bays formed in between breakwaters at the study sites such as those of Chippokes State Park, Drummonds Field, Waltrip and Hog Island West.

In the case of spiral bays, refraction patterns are more complex, as can be seen from figure 8.3. This complexity is basically due to the more complex bottom topography, which is, in turn, closely related to wave diffraction as will be discussed later. Figure 8.3 is schematically drawn based on field observations during a northeast storm in Feb. 1989 at national park service beach site.

Fig.8.2. A schematic diagram showing wave refractions in a semi-elliptical/
round bays in between breakwaters, which represents the situations of some of
our study sites such as Chippoke State Park, Drummonds Field, And Hog Island
West. The wave rays are drawn based on field observations.

Fig.8.3. A schematic diagram showing wave refractions in a half-heart or
spiral bay formed in between breakwaters at National Park Service beach site.
The construction of this diagram is based on field observations in Feb.,1989
during a storm. The unit is arbitrary.

The situation is likely that waves from offshore arriving in the vicinity of structures at most of the river sites have been subjected to refraction while travelling over the very shallow and wide terraces (often upto thousands of feet in distance). Wave energy is also greatly reduced due to friction of the shallow bed. These waves will be refracted more severely in the bays and other regions near structures in a much shorter distance and time (Figs.8.2 and 8.3).

8.3. Plan Forms and Wave Diffraction

As mentioned earlier, the phenomenon of wave diffraction is very common along coasts with various manmade or natural structures. In this process, wave energy is transfered lateraly along wave crests from where the wave height is large to where it is small. The result of wave diffraction is the re-distribution of wave energy, which is manifested in the variation of wave height. The wave diffraction is best expressed by the patterns of wave diffraction coefficient, K', which is defined as the ratio of a wave height, H_d, in the area affected by diffraction to the incident wave height, H_i, in the area unaffected by diffraction:

$$K' = H_d/H_i \qquad\qquad (8-2)$$

There is a whole set of steps in applying the diffraction diagrams to actual problems. The following example shows the steps in calculating and constructing a diffraction diagram.

1). Location: SUM (Fig. 8.4), one of the study sites located on the west shore of Potomac River near its mouth.

2). Wave condition: Significant wave parameters were observed during a spring north east storm in April, 1988. Wave period(T) = 2.9 sec, wave length (L) = 33 feet. Incident wave height, H_i= 3 feet.

Fig.8.4. Wave diffraction diagram for Summerial, Potomac River, showing the patterns of distributions of diffraction coefficient, K', by which wave height can be estimated more accurately. This diagram is constructed based on field observation data.

3). Wind direction – NE, approximately 135 in Wiegel and SPM's system.

4). The water depth at the tip of the spur, $d_s = 4$ feet.

5). Purpose: construct diffraction diagram, understand the patterns of wave height distribution and get a general picture about the interrelationship between the beach equilibrium shape and the process of wave diffraction.

The first step is to find the wave length in the area affected by wave diffraction, because the wave length will be used as the unit for the space/span between arcs. The diagram can not be constructed without this wave parameter and the accuracy of the diagram is also dependent on this parameter. In our case, the wave length was measured directly at site during the storm as stated in 2) above. However, if wave length was not given, it must be calculated based on a simple relation

$$d_s/L_0 = d_s/5.12 \ T \qquad (8-3)$$

and use a previously prepared table C-1 in SPM to find the ratio d_s/L, then deviding ds by d_s/L will give the shallow water wave length, L. The value of the ratio in this case is

$$d_s/L = 4/33 = 0.1212$$

$$L = d_s/(d_s/L) = 4/0.1212 = 33 \ feet$$

The second step is to make the scale of working chart as the same as the standard diffraction diagram or vice versa.

The third step is to transfer the contours of K' onto the working chart.

In the fourth step, the wave height at any point in the lee of the structure can be found very easily from the relationship

$$H_d - K'H_i \qquad (8\text{-}4)$$

For instance, if the wave height at point A is concerned, simply measure A in units of wave length in the rectangular coordinate system. Find the value of K' at that point. Then use equation 8-3 to find the wave height at that point.

8.4. Wave Reflection

Waves impinging on natural or manmade barriers will reflect either partially or totally depending on the incident direction and wave parameters such as wave height, steepness etc. Obviously, this process can not be negligible when considering equilibrium beach forms around structures. This is because beach forms are closely related to the patterns of wave energy distribution, while wave reflection implies reflection of wave energy. Therefore, various processes in the nearshore region are not isolated, but closely related each other.

The wave reflection can be expressed by a ratio of the reflected wave height, H_r, to the incident wave height, H . This ratio is called reflection coefficient, K_r, having the form

$$K_r - H_r/H_i \qquad (8\text{-}5)$$

K_r changes from 1.0 for complete reflection to zero for no reflection.

When the concern is focused on permeable structures like most of the breakwaters in the study sites (Fig.8.5), another ratio termed transmission coefficient, K_t, is used which is defined as the ratio of transmitted wave height,H_t, to incident wave height, H_i, i.e.

$$K_t - H_t/H_i \qquad (8\text{-}6)$$

The magnitude of transmitted wave height, H_t, is variable depending on the permeability of structures. The higher the permeability, the

Breaking Wave

ruble

profile of transmitted wave

instantaneous water level

H_t

Fig.8.5. A schematic diagram of the transsection of a breakwater which represents the very permeable type of structures like those at all of our study sites. water level fluctuates within the structure in response to incident waves that are partially transmitted through the structures. Wave energy is dissipated by: 1). partial reflection; 2). breaking; 3).transmitting through the structure.

larger the H_t value, and the more wave energy that can be transferred through. The space volume within rubble—mound structures in our study sites is estimated approximately 20% of the whole gross volume of structures. This large volume of space within structures helps dissipate wave energy by transmission. Figure 8.5 shows a typical transect through rubble—mound structures in our study sites.

As shown conceptually in figure 8.5, breakwaters can also dissipate wave energy by breaking waves. This kind of breaking as shown in the figure is not common in the study areas during average weather conditions according to the field observation data. This is because almost all of our study sites are located in the low energy river systems. It is, however, a common phenomenon during severe weather conditions.

Wave reflection from breakwaters almost always occurs when the incident wind speed is over 30 MPH based on field observation. In addition to wave breaking and transmission through the breakwaters, reflection can account for most of the remaining energy redistribution from breakwaters. The short waves are usually dissipated or "absorbed" by breakwaters, long waves are partially reflected.

9. FUTURE STUDY CONSIDERATIONS

The followings are considered as the future study problems:

1. The more detailed morphodynamic characteristics of fetch-limited environments in the tributary systems as a general topic need to be further studied. The fetch-limited environment should be treated separately as one of the major domains in nature in contrast to the open water environment that has its own characteristics in morphology and dynamics. The differences in some primary aspects such as in morphology, waves and current patterns between open and fetch-limited environments need to be more clearly defined.

2. The relationship between longshore drift/transport and profile development need to be further studied, though Bruun (1962) pointed out that the gradient in longshore sediment transport is not important in influencing the profile development.

3. Studies on the physics-based mechanism of profile variation and development around structures and bays in between structures should be further addressed in future, which is involved many factors. One of which is the wave characteristic that needs more accurate measurement and analysis.

APPENDIX A

Calculations

of

the scale parameter, A, Shape parameter, m,

and the Root-Mean-Square error, e_r

1. Procedures in Calculating Shape and Scale Factors, m and A:

The calculation of "best fit" values of scale and shape factors, A and m, in Bruun-Dean's model $h = Ax$ can be carried out by using the least square procedures which can be found in Dean's report (1977). The two equations resulted from the usual method of least squares are:

$$m = [S_{1,n}S_{4,n} - I\,S_{3,n}] / [(S_{1,n})^2 - I\,S_{2,n}] \qquad \text{(A-1)}$$

$$A = e^{Vn} \qquad \text{(A-1)}$$

where

$$Vn = [S_{1,n}S_{3,n} - S_{2,n}S_{4,n}] / [(S_{1,n})^2 - I\,S_{2,n}] \qquad \text{(A-3)}$$

and

$$S_{1,n} = \sum_{i=1}^{I} \ln x_{i,n} \qquad \text{(A-4)}$$

$$S_{2,n} = \sum_{i=1}^{I} (\ln x_{i,n})^2 \qquad \text{(A-5)}$$

$$S_{3,n} = \sum_{i=1}^{I} (\ln x_{i,n})(\ln h_m) \qquad \text{(A-6)}$$

$$S_{4,n} = \sum_{i=1}^{I} \ln h_m \qquad \text{(A-7)}$$

where the subscripts n and i denote the nth profile and distance index across the profile, respectively, I represents the total data points on the profile. h_m is the water depth measured in field. Two examples below show how these equations work.

Example 1.

Site: NPS, Profile 12, Dec.6, 1988

x	ln x	(ln x)	h_m	lnh_m	$(lnx)(lnh_m)$
0.00			0.00		
23.0	3.14	9.86	0.32	-1.14	-3.58
66.0	4.19	17.56	0.34	-1.08	-4.53
146.0	4.98	24.80	0.80	-0.22	-1.10
216.0	5.38	28.94	1.01	0.01	0.05
299.0	5.70	32.49	1.19	0.17	0.97
total	23.39	113.65		-3.30	-8.19
	(S_1)	(S_2)		(S_4)	(S_3)

hence
m = 0.21
Vn = -1.36
$A = e^{Vn} = 2.718^{-1.36} = 0.26$

Example 2. Site: WAL, profile 7, May 26, 1989

x	ln x	ln(x)	h_m	lnh_m	$(lnx)(lnh_m)$
0.00			0.00		
19.0	2.94	8.64	0.40	-0.92	-0.18
55.0	4.01	16.08	0.80	-0.22	-0.09
131.0	4.88	23.81	1.08	0.08	0.05
total	11.83	48.53		-1.06	-0.23

hence
m = 0.21
Vn = -0.90
$A = e^{Vn} = e^{-0.90} = 0.41$

Example 3:
    Site: CHP, profile 13, Mar.,1989

x	ln x	$(lnx)^2$	h_m	ln h_m	$(lnx)(lnh_m)$
0.00			0.0		
16.0	2.77	7.67	0.80	-0.22	-0.61
38.0	3.64	13.25	1.13	0.12	0.44
77.0	4.34	18.84	1.11	0.10	0.43
144.0	4.97	24.70	1.07	0.07	0.35
317.0	5.76	33.18	1.50	0.41	2.36
total	21.48	97.64		0.48	2.97

hence
    m = 0.06
    Vn = -0.16
    A = e^{Vn} = 0.85

Example 4.
    Site HI2, Mar.16, 1989, Oct.,1989

x	ln x	$(ln x)^2$	h_m	ln h_m	$(lnx)(lnh_m)$
0.0					
6.0	1.79	3.20	0.54	-0.62	-1.11
18.0	2.89	8.35	1.19	0.17	0.49
24.0	3.18	10.11	1.28	0.25	0.80
39.0	3.66	13.40	1.57	0.45	1.65
78.0	4.36	19.01	1.92	0.65	2.83
132.0	4.88	23.81	2.02	0.70	3.12
total	20.76	77.88		1.60	7.78

hence
    m = 0.19
    V_n = -0.32
    A = e^{Vn} = 0.7

2. Calculation of Root-Mean-Square (RMS) Error, e_r:

When evaluating the Goodness-of-Fit of the data points RSQ (R Squared) is a good method (see section 3 for the explanation of the method) like the results as presented in Section 5 and Appendix C. Another method evaluating the goodness-of-fit of data points to a certain model line is the calculation of RMS e_r (Root Mean Square Error). The Table below lists the average e_r value distribution for this study.

Root-Mean-Square Error in this study

RMS e_r (ft)	DECIMAL PERCENTAGE (BEACHFACE)	DECIMAL PERCENTAGE (LOWER PROFILE)
0.0-0.1	0.00	0.00
0.1-0.2	0.00	0.10
0.2-0.3	0.50	0.20
0.3-0.4	0.30	0.40
0.4-0.5	0.10	0.10
0.5-0.6	0.10	0.10
0.6-0.7	0.10	0.10
0.7-0.8	0.00	0.00
0.8-0.9	0.00	0.00
0.9-1.0	0.00	0.00

The followings are the procedures showing how these e_r values are obtained. It is not possible to list all lalculations for all profiles due to the space limitation, here only two examples are shown. In the following tables, h_m represents field measured water depth, h_p represents the predicted water depth by the model: $h=0.56x^{0.15} + 0.64$.

1. Site: DMF;

Jun.1989, Profile 8, beach face profile:

x	h_m	h_p	$(h_m-h_p)^2$
0	0.00	0.00	0.00
5	-1.00	-0.55	0.20
15	-2.06	-1.65	0.17
23	-2.98	-2.53	0.20
33	-4.46	-3.63	0.69
46	-6.31	-5.06	1.56
Total			2.82

Using equation 3-10:
    e_r= 0.69 feet

2. DMF, Mar.1989, Profile 8, Lower profile:

x	h_m	h_p	$(h_m-h_p)^2$
0	0.00	0.00	0.00
25	-0.85	-1.55	0.49
51	-2.04	-1.65	0.15
73	-1.20	-1.71	0.26
100	-1.35	-1.76	0.17
Total			1.07

Using equation 3-10:
    e_r= 0.46 feet

APPENDIX B

Time varying and enlarged beach profiles

Profile variation with time at NPS.

Date profile measured:

JUL18,1985
OCT16,1985
JUN02,1985
JUL29,1987
NOV16,1987
SEP14,1988
APR03,1990

MSL

FEET

FEET

30

20

10

0

-10

-20

0   50   100   150   200   250   300

CHP PROFILES
PROFILE NO. 13

Profile variation with time.

Date profile measured:

NOV10,1987
FEB23,1988
APR16,1988
MAY10,1988
DEC14,1988
MAR28,1989
JUN08,1989
OCT03,1989

Profile variation at DMF.

Date profile measured:

SEP09,1987
APR21,1988
SEP13,1988
DEC08,1988
JUN15,1989
OCT10,1989
MAY11,1990

BEACH PROFILES AT CHP

PROFILE 13

LEGEND

◇ MAR89
□ DEC88

MSL

beach face

DISTANCE IN FEET

ELEVATION IN FEET

DEC14/88, MAR28/89

PROFILES AT HI2, PROFILES 11 AND 21

OCT.4,1969

MSL

beach face

LEGEND

◇ RP21
□ RP11

ELEVATION IN FEET

DISTANCE IN FEET

PROFILES AT CHP
APR.16,1988 AND SEP.21,1988

LEGEND

SEP88

APR88

beach face

MSL

DISTANCE FROM BASE LINE IN FEET

ELEVATION IN FEET

BEACH PROFILES AT HI2

PROFILE 28

ELEVATION IN FEET

DISTANCE IN FEET

LEGEND

∇ NOV88
△ MAY88
◇ OCT89
□ MAR89

MSL

APPENDIX C

Bay beach profiles

Measured in Spring of 1990

BAY BEACH SLOPES
HI2 BAY L, MAY 1990.

MSL

LEGEND

$\vdots$ NO5

$\triangledown$ NO4

$\triangle$ NO3

$\circ$ NO2

$\square$ NO1

ELEVATION IN FEET

50 40 30 20 10 00 -10 -20 -30 -40 -50

DISTANCE IN FEET

-10 0 10 20 30 40 50 60 70

BAY BEACH SLOPES
CHP, BAY B, 8 MAY, 1990

ELEVATION IN FEET

DISTANCE IN FEET

LEGEND

∎ NO5	
▽ NO4	
▲ NO3	
◇ NO2	
▫ NO1	

MSL

BAY BEACH SLOPES
DMF BAY A, MAY 11, 1990

LEGEND

⋮	NO.5
▽	NO.4
△	NO.3
◇	NO.2
□	NO.1

MSL

ELEVATION IN FEET

DISTANCE IN FEET

BAY BEACH SLOPES
DMF BAY A, MAY 11, 1990

MSL

LEGEND

▽	No. 9
▲	No. 8
◇	No. 7
◻	No. 6

DISTANCE IN FEET

ELEVATION IN FEET

BAY BEACH SLOPES
DMF BAY D, PROFILES 5-8.

MAY 11,1990

LEGEND

∇ NO.8
△ NO.7
◇ NO.6
□ NO.5

MSL

DISTANCE IN FEET

ELEVATION IN FEET

BAY BEACH SLOPES
WALTRIP BAY B, PROFILES 1-5

MAY 13,1990

MSL

LEGEND

NO.5
NO.4
NO.3
NO.2
NO.1

ELEVATION IN FEET

DISTANCE IN FEET

BAY BEACH SLOPES
NPS BAY B, PROFILES 1-4.

APR 6,1990

LEGEND

▽ NO.4
△ NO.3
◇ NO.2
□ NO.1

MSL

DISTANCE IN FEET

ELEVATION IN FEET

BAY BEACH SLOPES
NPS BAY B, PROFILES 5-8

APR 6,1990

LEGEND

△ NO.7
◇ NO.6
□ NO.5

MSL

ELEVATION IN FEET

DISTANCE IN FEET

APPENDIX D

Plots of beach profiles

HOG ISLAND HEADLANDS
PROFILE NO 01

HOG ISLAND HEADLANDS
PROFILE NO. 02

HOG ISLAND HEADLANDS
PROFILE NO. 09

HOG ISLAND HEADLANDS
PROFILE NO 10

HOG ISLAND HEADLANDS
PROFILE NO. 20

HOG ISLAND HEADLANDS
PROFILE NO. 21

HOG ISLAND BREAKWATERS
PROFILE NO. 01

Legend:
- —·—·— MAR1689
- ·········· NOV2288
- ————— SEP0788

HOG ISLAND BREAKWATERS
PROFILE NO 02

Legend:
- —·—·— MAR1689
- ·········· NOV2288
- ————— SEP0788

HOG ISLAND BREAKWATERS
PROFILE   NO. 05

```
20 ─
              ─ ─ ─     MAR1689
              ........  NOV2288
15 ─          ───────   SEP0788

10 ─

FEET
(MSL)

5 ─

0 ─                                                           MHW
                                                              MSL
                                                              MLW
-5 ─
    0      25      50      75      100      125      150
                        FEET
```

HOG ISLAND BREAKWATERS
PROFILE   NO. 06

```
20 ─
              ─ ─ ─     MAR1689
              ........  NOV2288
15 ─          ───────   SEP0788

10 ─

FEET
(MSL)

5 ─

0 ─                                                           MHW
                                                              MSL
                                                              MLW
-5 ─
    0      25      50      75      100      125      150
                        FEET
```

HOG ISLAND BREAKWATERS
PROFILE   NO. 09

HOG ISLAND BREAKWATERS
PROFILE   NO. 12

HOG ISLAND BREAKWATERS
PROFILE NO 17

HOG ISLAND BREAKWATERS
PROFILE NO 18

HOG ISLAND BREAKWATERS
PROFILE NO. 21

HOG ISLAND BREAKWATERS
PROFILE NO 23

HOG ISLAND BREAKWATERS
PROFILE NO. 24

Legend:
- MAR1689
- NOV2288
- SEP0788

FEET (MSL) vs FEET

MHW
MSL
MLW

HOG ISLAND BREAKWATERS
PROFILE NO. 26

Legend:
- MAR1689
- NOV2288
- SEP0788

FEET (MSL) vs FEET

MHW
MSL
MLW

HOG ISLAND BREAKWATERS
PROFILE   NO. 31

HOG ISLAND BREAKWATERS
PROFILE   NO. 32

HOG ISLAND BREAKWATERS
PROFILE NO 34

HOG ISLAND BREAKWATERS
PROFILE NO. 36

HOG ISLAND BREAKWATERS
PROFILE NO. 37

------- MAR1689
...... NOV2288
——— SEP0788

HOG ISLAND BREAKWATERS
PROFILE NO. 38

------- MAR1689
...... NOV2288
——— SEP0788

HOG ISLAND BREAKWATERS
PROFILE NO 41

HOG ISLAND BREAKWATERS
PROFILE NO. 42

HOG ISLAND BREAKWATERS
PROFILE NO. 47

HOG ISLAND BREAKWATERS
PROFILE NO. 48

HOG ISLAND BREAKWATERS
PROFILE   NO. 51

```
FEET
(MSL)

20 ─
        ─·─·─   MAR1689
15 ─    ······  NOV2288
        ─────   SEP0788

10 ─

 5 ─

                                                    MHW
 0 ─                                                MSL
                                                    MLW

-5 ─
    └─────┬─────┬─────┬─────┬─────┬─────┬─────┬
    0     25    50    75    100   125   150
                    FEET
```

HOG ISLAND BREAKWATERS
PROFILE   NO. 52

```
FEET
(MSL)

20 ─
        ─·─·─   MAR1689
15 ─    ······  NOV2288
        ─────   SEP0788

10 ─

 5 ─

                                                    MHW
 0 ─                                                MSL
                                                    MLW

-5 ─
    └─────┬─────┬─────┬─────┬─────┬─────┬─────┬
    0     25    50    75    100   125   150
                    FEET
```

HOG ISLAND BREAKWATERS
PROFILE NO 57

MAR1689
NOV2288
SEP0788

20

15

10

FEET
(MSL)

5

0   MHW
                                                                     MSL
                                                                     MLW

-5

0   25   50   75   100   125   150
                    FEET

HOG ISLAND BREAKWATERS
PROFILE NO 58

MAR1689
NOV2288
SEP0788

20

15

10

FEET
(MSL)

5

0   MHW
                                                                     MSL
                                                                     MLW

-5

0   25   50   75   100   125   150
                    FEET

Chippokes state park
Profile No.01

MAR2889
DEC1488
SEP2188

MHW
ML W

CHIPPOKES STATE PARK
PROFILE NO.02

MAR2889
DEC1488
SEP2188

MHW
ML W

CHIPPOKES STATE PARK
PROFILE   NO. 05

CHIPPOKES STATE PARK
PROFILE   NO. 06

CHIPPOKES STATE PARK
PROFILE NO. 13

MAR2889
DEC1488
SEP2188

FEET
(MSL)

MHW
MLW

FEET

CHIPPOKES STATE PARK
PROFILE NO. 14

MAR2889
DEC1488
SEP2188

FEET
(MSL)

MHW
MLW

FEET

CHIPPOKES STATE PARK
PROFILE   NO. 17

40

MAR2889
DEC1488
SEP2188

30

20

FEET
(MSL)

10

MHW
0
ML W

-10

0   25   50   75   100   125   150

FEET

CHIPPOKES STATE PARK
PROFILE   NO. 18

40

MAR2889
DEC1488
SEP2188

30

20

FEET
(MSL)

10

MHW
0
ML W

-10

0   25   50   75   100   125   150

FEET

CHIPPOKES STATE PARK
PROFILE   NO 21

CHIPPOKES STATE PARK
PROFILE   NO 22

CHIPPOKES STATE PARK
PROFILE   NO. 29

CHIPPOKES STATE PARK
PROFILE   NO. 30

CHIPPOKES STATE PARK
PROFILE   NO 31

CHIPPOKES STATE PARK
PROFILE   NO. 33

DRUMMONDS FIELD
PROFILE NO 01

Legend:
- MAR3089 (dash-dot line)
- DEC0888 (dotted line)
- SEP1388 (solid line)

Y-axis: FEET (MSL), range -5 to 20
X-axis: FEET, range 0 to 150

Horizontal reference lines: MHW, MSL, MLW

DRUMMONDS FIELD
PROFILE NO. 02

Legend:
- MAR3089 (dash-dot line)
- DEC0888 (dotted line)
- SEP1388 (solid line)

Y-axis: FEET (MSL), range -5 to 20
X-axis: FEET, range 0 to 150

Horizontal reference lines: MHW, MSL, MLW

DRUMMONDS FIELD
PROFILE NO 04

DRUMMONDS FIELD
PROFILE NO 08

DRUMMONDS FIELD
PROFILE NO 13

DRUMMONDS FIELD
PROFILE NO. 14

DRUMMONDS FIELD
PROFILE   NO 23

DRUMMONDS FIELD
PROFILE   NO 25

DRUMMONDS FIELD
PROFILE   NO 26

DRUMMONDS FIELD
PROFILE   NO 27

DRUMMONDS FIELD
PROFILE NO 30

DRUMMONDS FIELD
PROFILE NO 31

WALTRIP
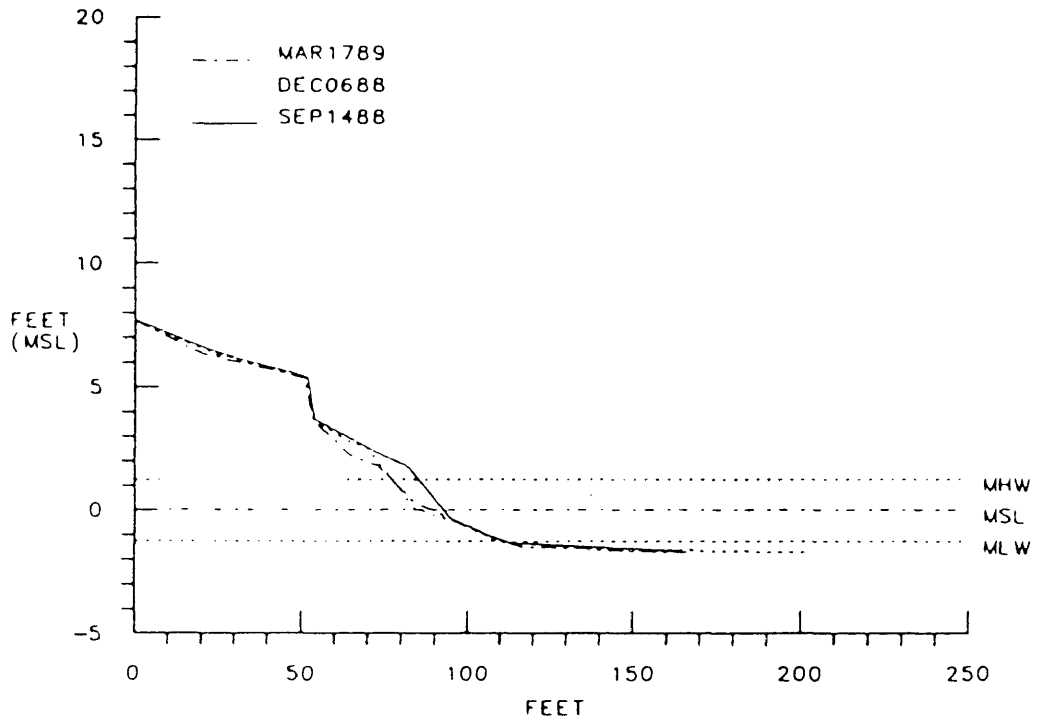PROFILE   NO 01

WALTRIP
PROFILE   NO 02

WALTRIP
PROFILE   NO 05

MAY2689
MAR2089
DEC2088
SEP2788

FEET
(MSL)

MHW
MSL
MLW

FEET

WALTRIP
PROFILE   NO. 06

MAY2689
MAR2089
DEC2088
SEP2788

FEET
(MSL)

MHW
MSL
MLW

FEET

WALTRIP
PROFILE   NO 07

WALTRIP
PROFILE   NO 08

WALTRIP
PROFILE   NO 13

WALTRIP
PROFILE   NO 14 .

PARKWAY BREAKWATERS
PROFILE NO. 05

MAR1789
DEC0688
SEP1488

FEET
(MSL)

MHW
MSL
MLW

FEET

PARKWAY BREAKWATERS
PROFILE NO 06

MAR1789
DEC0688
SEP1488

FEET
(MSL)

MHW
MSL
MLW

FEET

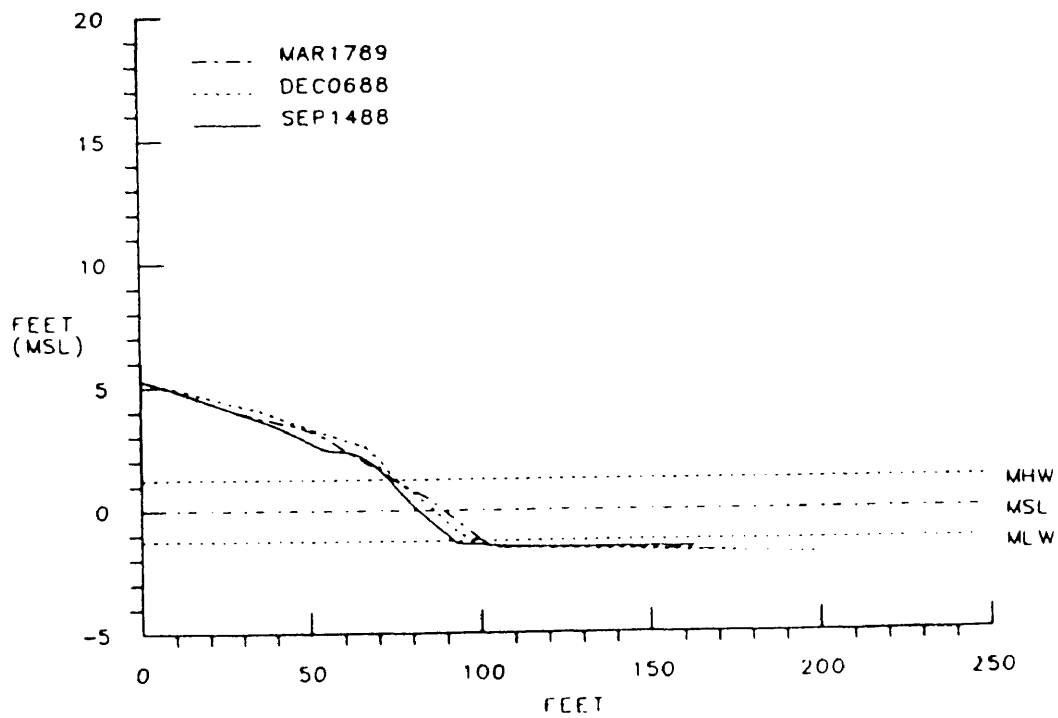PARKWAY BREAKWATERS
PROFILE   NO 11

PARKWAY BREAKWATERS
PROFILE   NO 12

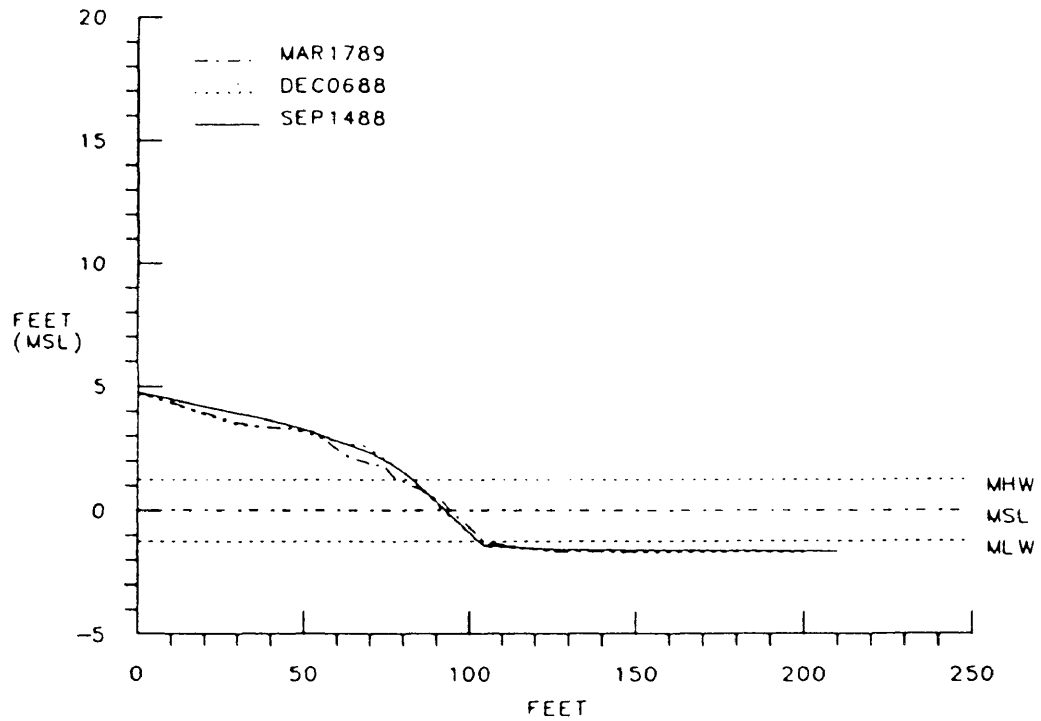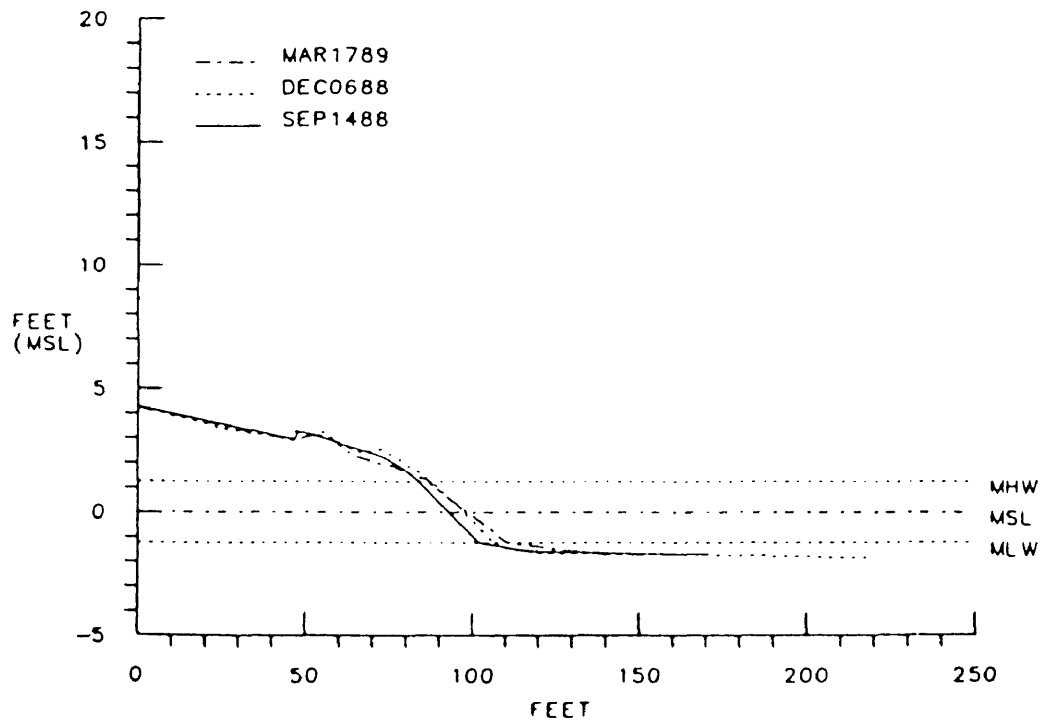PARKWAY BREAKWATERS
PROFILE NO 13

PARKWAY BREAKWATERS
PROFILE NO 14

PARKWAY BREAKWATERS
PROFILE   NO 17

PARKWAY BREAKWATERS
PROFILE   NO. 18

PARKWAY BREAKWATERS
PROFILE   NO 19

MAR1789
DEC0688
SEP1488

FEET
(MSL)

MHW
MSL
MLW

FEET

PARKWAY BREAKWATERS
PROFILE   NO 20

MAR1789
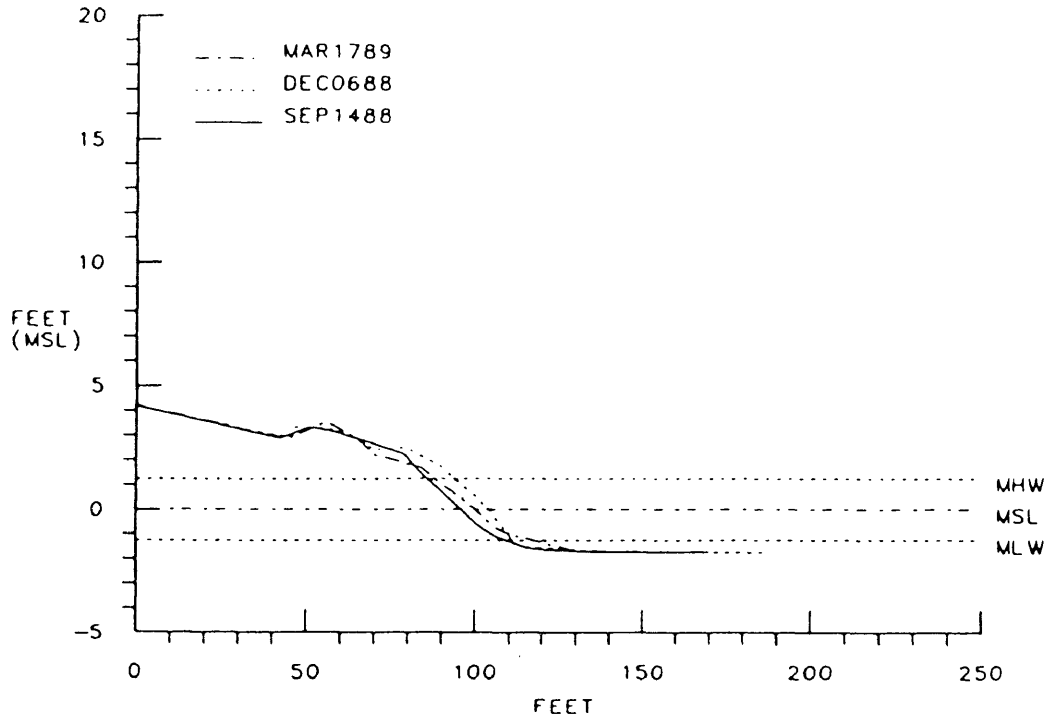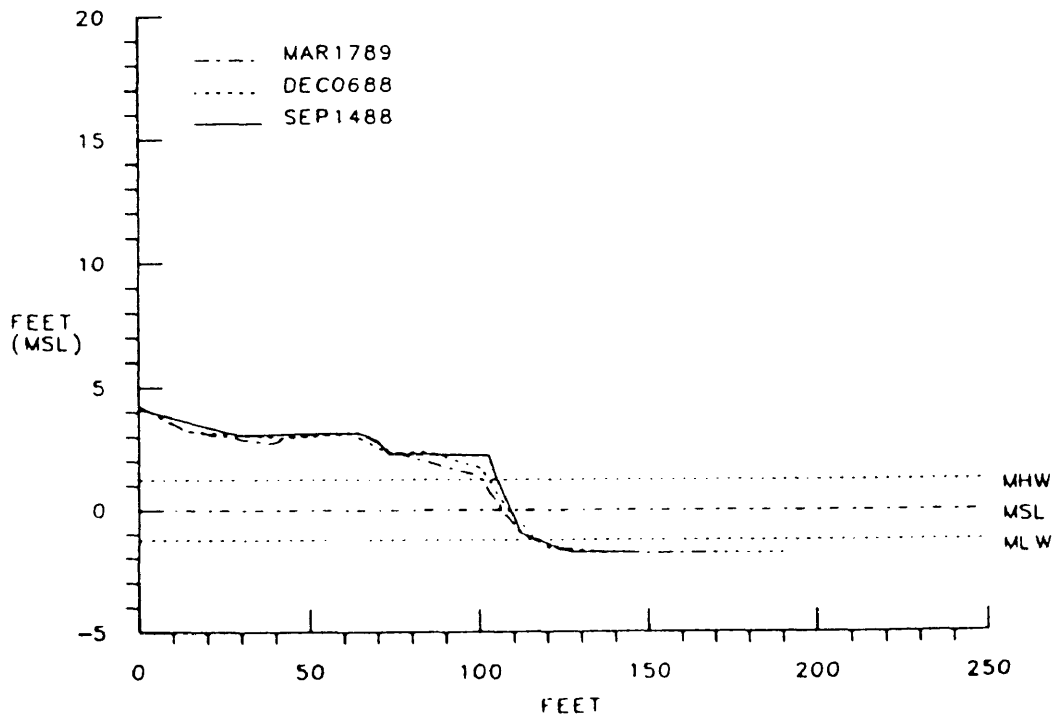DEC0688
SEP1488

FEET
(MSL)
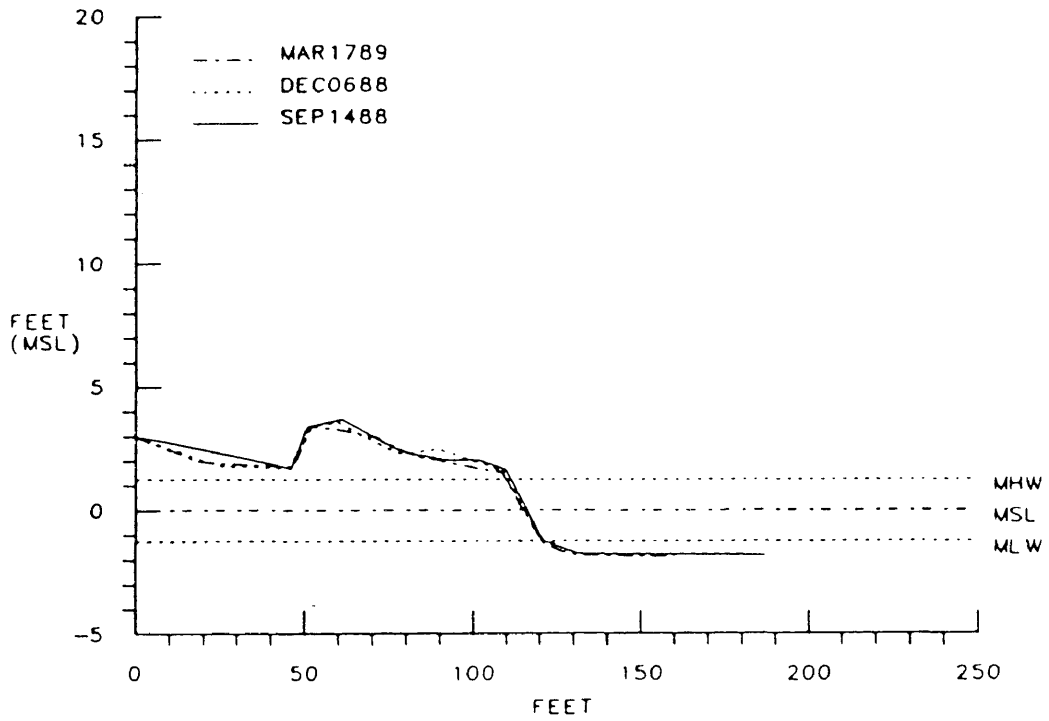
MHW
MSL
MLW

FEET

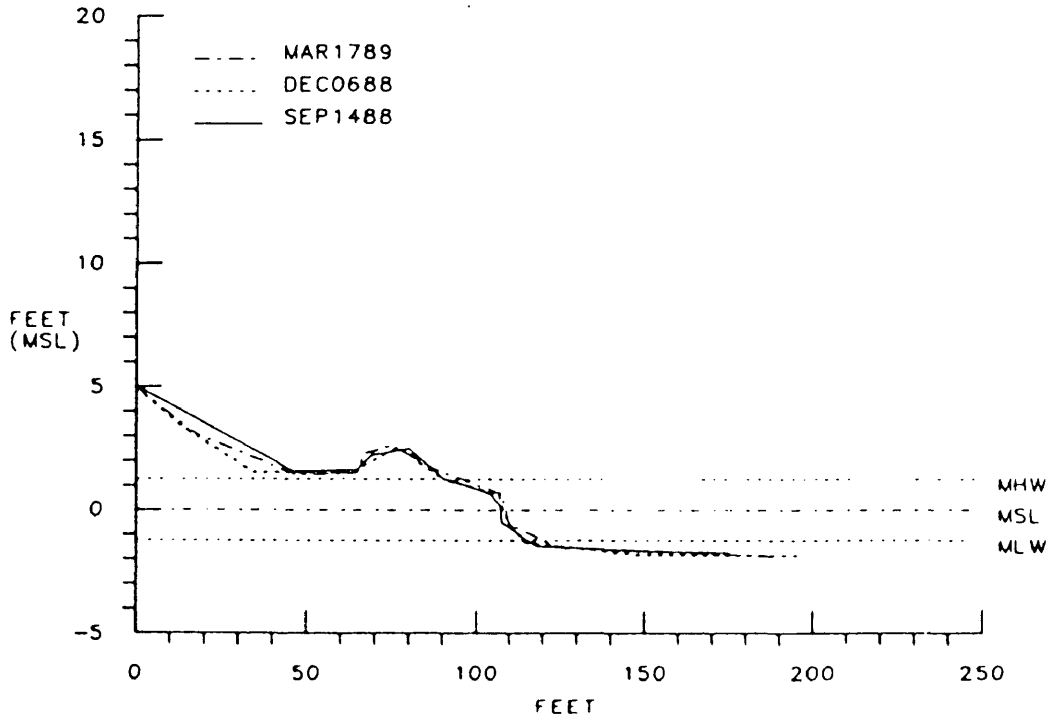PARKWAY BREAKWATERS
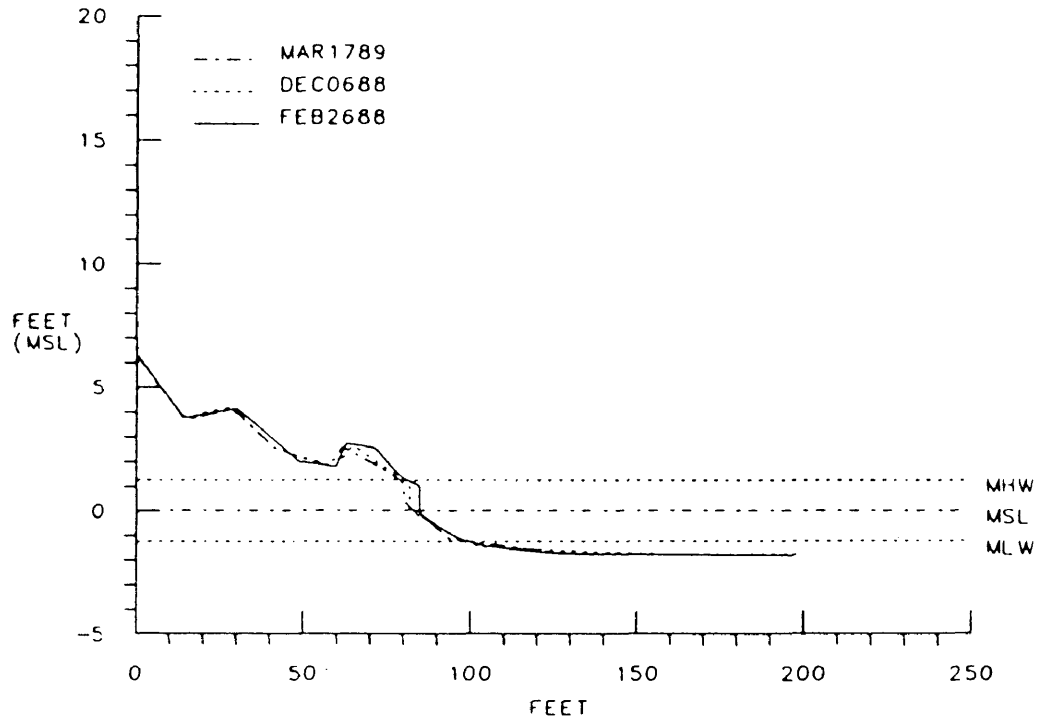PROFILE   NO 21

PARKWAY BREAKWATERS
PROFILE   NO 22

PARKWAY BREAKWATERS
PROFILE   NO 25

PARKWAY BREAKWATERS
PROFILE   NO 26

PARKWAY BREAKWATERS
PROFILE NO 27

YORKTOWN BAY 1
PROFILE NO 01

YORKTOWN BAY 1
PROFILE NO. 02

YORKTOWN BAY 1
PROFILE NO 06

YORKTOWN BAY 1
PROFILE NO 07

YORKTOWN BAY 2
PROFILE NO 01

MAR1589
NOV3088
SEP2888

YORKTOWN BAY 2
PROFILE NO 03

MAR1589
NOV3088
SEP2888

YORKTOWN BAY 2
PROFILE   NO 07

MAR1589
NOV3088
SEP2888

FEET
(MSL)

MHW
MSL
MLW

FEET

YORKTOWN BAY 3
PROFILE   NO 01

MAR1589
NOV3088
SEP2888

FEET
(MSL)

MHW
MSL
MLW

FEET

YORKTOWN BAY 3
PROFILE NO 02

MAR1589
NOV3088
SEP2888

FEET
(MSL)

MHW
MSL
MLW

FEET

YORKTOWN BAY 3
PROFILE NO 03

MAR1589
NOV3088
SEP2888

EET
(SL)

MHW
MSL
MLW

FEET

YORKTOWN BAY 3
PROFILE   NO 04

FEET
(MSL)

MAR1589
NOV3088
SEP2888

MHW
MSL
MLW

FEET

YORKTOWN BAY 3
PROFILE   NO 05

FEET
(MSL)

MAR1589
NOV3088
SEP2888

MHW
MSL
MLW

FEET

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