Cape Charles Public Beach Assessment Report

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Public Beach Assessment Report
Cape Charles Beach
Town of Cape Charles, Virginia

Virginia Institute of Marine Science
School of Marine Science
College of William and Mary

Technical Report Obtained Under
Contract with
The Virginia Department of
Conservation and Recreation
via the
Joint Commonwealth Programs Addressing
Shore Erosion in Virginia.

By
C. Scott Hardaway, Jr.
Donna A. Milligan
George R. Thomas

June 1993
CAPE CHARLES
PUBLIC BEACH ASSESSMENT REPORT

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Donna A. Milligan
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Virginia Institute of Marine Science
College of William and Mary
Gloucester Point, Virginia  23062

June 1993
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I. INTRODUCTION

A. Background and Purpose

The Town of Cape Charles is located on the Chesapeake Bay in Northampton County, Virginia (Figure 1). It has the only bayside commercial harbor and the only public beach on Virginia's Eastern Shore. Cape Charles' Chesapeake Bay waterfront has been a major recreational area since 1900. In fact, the town's symbol is the Gazebo that sits on the boardwalk. Until recent construction and beach nourishment efforts, the beach had steadily diminished over the past 30 years.

The involvement between the Town of Cape Charles and the Virginia Board for the Development and Conservation of Public Beaches began on February 10, 1982 when Brown and Root, Inc. transferred the title of the public beach to the Town of Cape Charles. The transfer carried the stipulations that the property be renovated, that erosion control improvements be installed, and that the beaches be dedicated for perpetual use as a public park, maintained by the Town for the use and benefit of its citizens and the general public. The existing groin field and bulkhead were in need of repair and erosion had severely reduced the beach.

The Cape Charles Critical Area Treatment Project began in the summer of 1982 and involved groin construction and beach nourishment. The groins were constructed 150 feet long and 300 feet apart. The project was planned and coordinated by the Soil Conservation Service as a Resource Conservation and Development Project. The cost of the project totalled $213,200 (Resource Conservation and Development = $106,600; Cape Charles = $53,300; Public Beach Board = $53,300).

As a result of damage to the bulkhead from the severe storm of November, 1985, Cape Charles received $136,000 from the State's Special Emergency Assistance Fund in the spring of 1986. The funds were used to repair the bulkhead and boardwalk and for beach nourishment of about 4,000 cubic yards (cy). An additional $25,000 was directly appropriated to Cape Charles by the 1986 Virginia General Assembly for additional bulkhead repairs.

In the winter of 1987, the public beach received approximately 87,000 cy of clean, beach quality sand from the Cape Charles Harbor Maintenance Project sponsored by the Norfolk District of the U.S. Army Corps of Engineers. The beach was surveyed before and after construction and again in 1992 and 1993. Since the widened beach would impede the drainage of storm water, the Virginia Department of Transportation relocated the storm sewer outfalls to either end of the beach at a cost of $229,000. The Town's portion was $95,000 which was obtained from the General Assembly. The Department of Transportation paid $134,000.

The beach nourishment significantly increased the width of the beach. The following winter the finer sand fraction began to blow inland onto the adjacent road and yards as a result of strong northwest winds. In the spring of 1988, the town initiated a project to install sand fencing and dune grasses to control the blowing sand. The project involved planting 30,000 dune plants, using a mixture of American beach grass and "Atlantic" coastal panicgrass. Materials for the project were purchased using $3,500 of town funds and $3,500 of Public Beach Board funds. The Department of Transportation supplied and installed one of the sand fences and the Youth Conservation Corps installed two rows of sand fence and fence islands. Extensive dunes have developed as a result of these efforts.

The Virginia Institute of Marine Science (VIMS) of the College of William and Mary, under contract with the Virginia Department of Conservation and Recreation, established a baseline and performed beach surveys in connection with the beach nourishment project in 1988.
Figure 1. Site location: Cape Charles, VA; Virginia Power Station at Yorktown; Nandua Creek and Kiptopeke.
photography was obtained and sediment samples were collected. The objectives of this report are to present the rates and patterns of beach change and to relate those changes to the hydrodynamic forces acting upon the shore zone.

B. Limits of Study Area

The Cape Charles public beach shoreline lies at the south end of the reach defined by the jetty and Cape Charles harbor channel on the south and the mouth of Kings Creek on the north. The public beach area extends from the large north harbor jetty northward for about 2400 feet toward King’s Creek (Figure 2). The detailed hydrodynamic modelling efforts were confined to the offshore region to -25 feet mean low water (MLW).

C. Approach and Methodology

Field data and computer models were used to address the aforementioned objectives. Field data analyzed for this report include beach profiles taken on November 2, 1987 (pre-fill), March 15, 1988 (post-fill), April 16, 1992 and April 14, 1993. The datum for vertical control is MLW as established by the U.S. Army Corps of Engineers (1982). Thirteen beach profiles were established along the public beach shoreline at 200 foot (61 m) intervals (Figure 2). The profile origins or benchmarks were established along the top edge of the bulkhead protecting the boardwalk. Data were summarized in terms of the relative position of mean high water (MHW). Profile data were also used to calculate beach and nearshore volume changes over time.

Figure 3 gives a pictorial definition of the profile terminology used in this report. All the nearshore data were calculated by taking into account all sand below MLW to the end of each profile. The subaerial beach occurs above MLW and is divided into the beach face and backshore region and the dune region above +5.0 feet. The mean tide range at Cape Charles is 2.4 feet.

The hydrodynamic forces acting along the Cape Charles shoreline were evaluated using RCPWAVE, a computer model developed by the U.S. Army Corps of Engineers (Ebersole et al., 1986). This program was modified to run on the VIMS prime 9955 mainframe. RCPWAVE is a linear wave propagation model designed for engineering applications. The model computes changes in wave characteristics that result naturally from refraction, shoaling, and diffraction over complex shoreface topography. To this fundamental linear-theory-based model, VIMS has added routines which employ recently developed understandings of wave bottom boundary layers to estimate wave energy dissipation due to bottom friction. The VIMS revision also estimates wave-induced, longshore, surf zone currents and littoral drift by means of three different theoretical models, two of which incorporate the effects of longshore gradients in breaker height. The reader is referred to Ebersole et al. (1986) and Wright et al. (1987) for a thorough discussion of RCPWAVE, its use and theory.

The model was run using four incident wave conditions (wave height, period, and direction) which were determined from wind/wave hindcast methods across fetch-limited water bodies as developed by Sverdrup, Monk and Breitsnieder (SMB) and modified by Camfield (1977) and then by Kiley (1982). Wind data, obtained from Virginia Power’s Yorktown Station, were used to develop the incident wave conditions for input into the RCPWAVE program.

II. COASTAL SETTING

A. Shore Morphology and Sediment Transport

The historic erosion rate for the shore reach from King’s Creek to Cape Charles Harbor is about 1.5 ft/yr (Byrne and Anderson, 1978). The accumulation of a large sand fillet on the north side of the harbor channel jetty indicates
Figure 2. Base map of Cape Charles Beach with profile and cell locations.
Figure 3. Typical beach profile demonstrating terminology used in the report.
a net southerly transport of sand. The channel jetty is a significant barrier to littoral transport and protrudes about 1200 feet into the Chesapeake Bay. The Cape Charles shoreline is oriented almost north-south with an average fetch to the west of about 23 nautical miles.

The public beach currently is bordered on the north by a large storm water outfall pipe that extends about 300 feet from the bulkhead into the bay. The pipe was installed as part of the 1988 beach nourishment project and subsequently has been reinforced with gabions including gabion spurs on either side. The outfall has a local effect on the public beach by partially blocking sand moving south along the shoreline from King's Creek. Also, the outfall and associated spur are causing an alteration in the beach planform to the immediate south.

The nearshore region extends from MLW to a distance between 1000 feet and 1200 feet to a depth of -3.0 feet MLW where it goes to -15.0 feet to -20.0 feet over a distance of 1000 feet. These depths are associated with a north-south running channel that enters Cherrystone Inlet to the north. Beyond the channel there is a broad shoal averaging about -3.0 feet that extends bayward another 4500 feet. This shoal is part of a much larger bar and shoal complex that runs along most of the bay shoreline of the Eastern Shore from Kiptopeke to Nandua Creek.

B. Beach, Dune, and Nearshore Sediments

Sediment samples were collected after the beach nourishment project (March 1988) and again in April, 1992 and April, 1993 along profiles 3, 7, and 11. The samples were collected at particular morphologic points along the profile rather than the same distance from the baseline (Table 1). The position of the shore features, such as the beach berm and beach toe, changed significantly between sample dates. The sediment samples were analyzed using the VIMS Rapid Sediment Analyzer (RSA) that determines the grain size distribution of the sand fraction.

Figures 4A and 4B (Folk, 1980) are the plots for mean grain size (in phi units and mm) and sorting (in phi units) of the sand fraction. The samples taken at the toe of the beach (TOE) are consistently the coarsest material along the beach. The TOE zone usually is a very narrow step along the shore and is not very representative of the entire profile. Another noticeable trend is the relatively finer and more uniform sand sizes along profile 11 in April, 1993 compared to profiles 3 and 7. This may be due to the lack of beach width in that area as a result of wave reflection from the exposed bulkhead. Except for that, the overall sand size distribution is finer at the base of the dune and nearshore and coarser along the beach berm, high water line and TOE. This is typical of estuarine beaches in the Chesapeake Bay (Hardaway et al., 1991)

The sorting of sediments can be defined as the Inclusive Graphic Standard Deviation (Folk, 1980). The spread of the grain size distribution about the mean defines the concept of sorting. Well sorted sands will have a frequency distribution curve that is sharp peaked and narrow; this means only a few size classes are present (Friedman and Sanders, 1978). Poorly sorted sediments are represented by most size classes in the sample.

Analyses of the Cape Charles sediments generally show better sorting with time at profile 11 and poorer sorting at profile 3 with profile 7 being in transition. This coincides with the fining of the sediment at profile 11 and coarsening at profile 3. This trend may support a general southward transport of material with the coarse sands mixing with the fine sands at the south end of the public beach to produce a poorly sorted sediment profile.
Figure 4. Results of sediment sample analysis for A) mean size (phi) and B) sorting.
Table 1. Morphologic features at which sediment samples are located.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Mar 1988 Feature</th>
<th>Apr 1992 Feature</th>
<th>Apr 1993 Feature</th>
</tr>
</thead>
<tbody>
<tr>
<td>3-1</td>
<td>Base of Dune</td>
<td>Base of Dune</td>
<td>Base of Dune</td>
</tr>
<tr>
<td>3-2</td>
<td>On Beach</td>
<td>On Beach</td>
<td>On Beach</td>
</tr>
<tr>
<td>3-3</td>
<td>High Water on Beach</td>
<td>Last High Tide</td>
<td>Last High Tide</td>
</tr>
<tr>
<td>3-4</td>
<td>Toe of Beach</td>
<td>Toe of Beach</td>
<td>Toe of Beach</td>
</tr>
<tr>
<td>3-5</td>
<td>Offshore</td>
<td>Offshore</td>
<td>Offshore</td>
</tr>
<tr>
<td>7-1</td>
<td>Base of Dune</td>
<td>Base of Dune</td>
<td>Base of Dune</td>
</tr>
<tr>
<td>7-2</td>
<td>On Beach&amp;Last High Tide</td>
<td>On Beach&amp;Last High Tide</td>
<td>On Beach&amp;Last High Tide</td>
</tr>
<tr>
<td>7-3</td>
<td>Toe of Beach</td>
<td>Toe of Beach</td>
<td>Toe of Beach</td>
</tr>
<tr>
<td>7-4</td>
<td>Nearshore Sandbar</td>
<td>Nearshore Sandbar</td>
<td>Nearshore Sandbar</td>
</tr>
<tr>
<td>7-5</td>
<td>Sandbar</td>
<td>Sandbar</td>
<td>Sandbar</td>
</tr>
<tr>
<td>11-1</td>
<td>Base of Dune</td>
<td>Base of Dune</td>
<td>Base of Dune</td>
</tr>
<tr>
<td>11-2</td>
<td>Last High Tide Swash</td>
<td>Last High Tide</td>
<td>Last High Tide</td>
</tr>
<tr>
<td>11-3</td>
<td>Toe of Beach</td>
<td>Toe of Beach</td>
<td>Toe of Beach</td>
</tr>
<tr>
<td>11-4</td>
<td>Offshore</td>
<td>Offshore</td>
<td>Offshore</td>
</tr>
<tr>
<td>11-5</td>
<td>Offshore</td>
<td>Offshore</td>
<td>Offshore</td>
</tr>
</tbody>
</table>

C. Wave Climate

The wave climate acting upon the Cape Charles shoreline is created by winds blowing across, up and down the Chesapeake Bay. Typically, this reach is affected by northwest winds which occur during the late fall to early spring as well as southwest and westerly winds that are most frequent during the early spring to late fall (Rosen, 1978). Waves created by northeast storms do not impact the Cape Charles shoreline directly but usually produce significant storm surge. As the post-storm winds often shift to the northwest, the water level is elevated for a short period of time. This scenario can produce high waves acting on the Cape Charles shoreline.

Large waves greater than 3.0 feet entering the reach from the bay are affected initially by the broad shoal. They may briefly reform across the Cherrystone Inlet channel before again shoaling in the nearshore region. The smaller portions of the wave regime (< 3.0 ft) are not significantly attenuated until they enter the very nearshore region in front of the Cape Charles Public Beach. The relative water elevation and significant wave height will determine what portion of the public beach shore is most impacted in terms of sediment transport. The effects of tidal currents on wave height and direction may be significant at times but wave/current interaction is a complex relationship and is beyond the scope of this report.

In order to develop a wave-climate evaluation, it is necessary to provide RCPWAVE with reasonable incident wave conditions. The best wave input data come from a wave gage placed offshore of the subject site. VIMS has maintained two wave gages in the bay for the past several years.
Unfortunately, both deployments are on the west side of the bay and are partially shielded from the northwest, west and southwest components of the wind/wave field to which Cape Charles is exposed. Therefore, it is necessary to estimate the wave climate using available wind data. The nearest applicable wind station is at Yorktown. Although it is on the opposite side of the bay, the wind record is applicable to Cape Charles, especially for westerly winds of long duration (> 9 hrs).

The wave prediction model was initially developed by Sverdrup and Munk (1947) and revised by Bretschneider (1952, 1958). The current model (known as SMB) used in this study was further modified by Camfield (1977) and Kiley (1982). It is essentially a shallow water, estuarine, wind-wave prediction model.

Preliminary results from Hardaway and Milligan (in prep) show a close correlation between measured and predicted waves at the VIMS' Wolf Trap wave gage (Boon et al., 1992). This same wave prediction procedure, developed during a previous project (Hardaway et al., 1991), was used to produce a set of wave conditions for input into RCPWAVE. The procedure involves the following steps:

1. Determine effective fetch for three directions. This was accomplished using procedures outlined in the U.S. Army Corps of Engineers Shore Protection Manual (1977) for northwest, west and southwest directions from a point 10,500 feet due west of the Cape Charles public beach at -25 feet MLW. This also involves measuring a bathymetric transect across the bay in each of the three subject directions.

2. Use the above data as input into the SMB program which provides wave height, period and length for a suite of wind speeds. In this case, wind speeds of 4 to 48 mph were used at 4 mph increments. The results of this step are used to create a data file of wind speeds and associated wave heights and periods for each subject direction.

3. Wind data for four years, 1987 to 1990, were set up along with the data file from step 2, as input requirements for running the program WINDOWS (Suh, 1990). WINDOWS takes the data file as input parameters from step 2 and matches them with wind speed and direction from each of the subject directions for each year to produce another data file of wave heights, periods and directions through a series of vector-averaging steps. The limiting criterion is that the wind must be blowing from within the assigned sextant window for at least 9 hours. In other words, winds recorded by the wind station must be within, for example, 300 and 360° for 9 or more hours to qualify for this analysis.

4. The result of step 3 is a file for each year giving date, hour beginning, wave height, wave period, local wave direction and duration of each qualifying wind event. These data then are mean weighted to provide a weighted mean for wave height, period and direction with duration as the independent variable for each year (Table 2).

5. Finally, the results of step 4 were mean weighted for each year to produce a weighted mean of wave parameters for the northwest, west and southwest directions (Table 3). The duration of each year was averaged for each direction. These results were used as input into RCPWAVE for annual conditions. One severe northwest storm condition was also modelled. The RCPWAVE analysis will be discussed further in section IV.
Table 2. Cape Charles hindcasted wave data by year and direction $\bar{X}$ = weighted mean.

<table>
<thead>
<tr>
<th>Year</th>
<th>Height ($\bar{X}$) (ft)</th>
<th>Period ($\bar{X}$) (sec)</th>
<th>Angle ($\bar{X}$) (deg TN)</th>
<th>Duration (avg) (hrs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Northwest</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1987</td>
<td>2.61</td>
<td>3.17</td>
<td>315.41</td>
<td>16.06</td>
</tr>
<tr>
<td>1988</td>
<td>2.18</td>
<td>2.97</td>
<td>321.61</td>
<td>15.54</td>
</tr>
<tr>
<td>1989</td>
<td>2.31</td>
<td>2.95</td>
<td>319.36</td>
<td>14.62</td>
</tr>
<tr>
<td>1990</td>
<td>2.48</td>
<td>3.17</td>
<td>312.18</td>
<td>15.00</td>
</tr>
<tr>
<td>West</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1987</td>
<td>1.35</td>
<td>2.45</td>
<td>268.36</td>
<td>17.53</td>
</tr>
<tr>
<td>1988</td>
<td>1.29</td>
<td>2.42</td>
<td>272.80</td>
<td>18.00</td>
</tr>
<tr>
<td>1989</td>
<td>1.16</td>
<td>2.28</td>
<td>269.80</td>
<td>13.75</td>
</tr>
<tr>
<td>1990</td>
<td>1.68</td>
<td>2.74</td>
<td>271.40</td>
<td>13.50</td>
</tr>
<tr>
<td>Southwest</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1987</td>
<td>1.35</td>
<td>2.46</td>
<td>223.04</td>
<td>21.06</td>
</tr>
<tr>
<td>1988</td>
<td>1.45</td>
<td>2.55</td>
<td>219.32</td>
<td>19.51</td>
</tr>
<tr>
<td>1989</td>
<td>1.35</td>
<td>2.46</td>
<td>220.11</td>
<td>22.05</td>
</tr>
<tr>
<td>1990</td>
<td>1.55</td>
<td>2.61</td>
<td>221.88</td>
<td>18.50</td>
</tr>
</tbody>
</table>

Table 3. RCPWave input wave conditions.

<table>
<thead>
<tr>
<th></th>
<th>Northwest</th>
<th>West</th>
<th>Southwest</th>
</tr>
</thead>
<tbody>
<tr>
<td>$H$ ($\bar{X}$) (ft)</td>
<td>2.38</td>
<td>1.35</td>
<td>1.42</td>
</tr>
<tr>
<td>$T$ ($\bar{X}$) (sec)</td>
<td>3.1</td>
<td>2.5</td>
<td>2.5</td>
</tr>
<tr>
<td>Wave Dir ($\bar{X}$) (deg TN)</td>
<td>152</td>
<td>91</td>
<td>26</td>
</tr>
<tr>
<td>Duration(avg) (hrs)</td>
<td>15.3</td>
<td>15.7</td>
<td>20.3</td>
</tr>
</tbody>
</table>

Weighted mean $\bar{X}$ by duration.

III. BEACH CHARACTERISTICS AND BEHAVIOR

A. Beach and Nearshore Profiles and Their Variability

Thirteen profiles were established prior to the beach fill project in 1988. These are shown on Figure 2, the base map. The profiles are 200 feet apart except for profile 12 to profile 13 which is 138 feet. The nearshore shoal region is wider on the south end near the harbor jetty and narrows to the north; the distance the profiles extend reflect this narrowing. The baseline turns at profile 10.
Figures 5 to 18 are the 13 plots of each profile for the four surveys. Profile numbers, survey numbers and date are found in figure legends. Profile 12 is shown with and without the outfall. Most of the beach fill was placed between profiles 3 and 13. Profiles 1 to 3 were the area of the existing sand fill which was adjacent to the harbor jetty and did not need additional fill material. Due to the nature of the emplacement (i.e., by hydraulic dredging), the fill was distributed unevenly along the shore. The bulk of the 87,000 cy was initially placed above MLW.

Due to the width of the created beach and backshore, a considerable amount of sand was blown landward across Bay Avenue and onto residential lawns for several months after the beach fill project. Sand fences were installed by the Virginia Department of Transportation in March, 1988 and the backshore area was vegetated with American Beachgrass (*Ammophila breviligulata*). Within a year, a well vegetated, low dune had developed. The dune system has grown in width and elevation by trapping wind blown sand from the beach area with the combination of beach grasses and fencing. By April, 1992, the dune system had reached elevations between 4 and 5 feet above the initial beach fill. This can be seen in profiles 2 to 9.

The dune system is less developed between profiles 10 and 13 where a significant loss of sand has caused the beach to erode to near pre-fill positions, particularly at profiles 10 and 11. In fact, the beach from MLW to about +5.0 MLW has decreased in width along Cape Charles Beach since fill installation. The consequence is that much of the material, the finer sand fraction, has been trapped in the dune system and the remaining beach losses have gone offshore. The nearshore profile data indicate an increase in elevation from 0.5 to 1.5 feet above the pre-fill conditions.

As the dune system grew in width and elevation after the fill emplacement, the subaerial beach evolved into two morphologic units, the vegetated dune and the non-vegetated beach. The entire beach profile as of April, 1992 can thus be divided into three sections; the dune, the subaerial beach, and the nearshore. The dune area of each profile extends from the bulkhead (baseline) to a point where there is a significant break in slope. The channelward break in slope is referred to as the base of the dune (BOD) and is often accompanied by the channelward edge of the dune vegetation. The subaerial beach is between MLW and the BOD and the nearshore is the region beyond MLW.

### B. Variability in Shoreline Position and Sand Volume

#### 1. Shoreline Position Variability

The movement of the active beach through time can be depicted by plotting the position of MHW. Figure 19 shows the distance of MHW from the baseline for each profile date. The beach fill is readily identified in the plot of MHW for March, 1988 as it extends bayward well beyond pre-fill conditions. The most obvious trend is the adjustment of the beach fill after installation. Significant losses or landward movement are seen from the post-fill position (March, 1988) to April, 1992 between profiles 3 and 13. The position of MHW at profiles 1 and 2 show little change since March, 1988.

From April, 1992 to April, 1993, the MHW shoreline continued to adjust landward. This occurs mostly between profiles 7 and 11. The shoreline at profile 11 has eroded to the bulkhead, essentially to the pre-fill position.

The elevation of the BOD and its channelward limit vary both temporally and spatially (Figures 20 and 21). From April, 1992 to April, 1993, the BOD became higher in elevation but its position receded slightly at profile 1, between profiles 3 and 7 and between profiles 11 and 13. The BOD was lower at profiles 9 and 10; however, the position of profile 10 receded while the position of profile 9 stayed nearly the same. Profiles 2 and 8 both
Figure 5. Profile 1 plot depicting changes at Cape Charles Beach.

Figure 6. Profile 2 plot depicting changes at Cape Charles Beach.
Figure 7. Profile 3 plot depicting changes at Cape Charles Beach.

Cape Charles Beach

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Figure 8. Profile 4 plot depicting changes at Cape Charles Beach.

Cape Charles Beach

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Figure 9. Profile 5 plot depicting changes at Cape Charles Beach.

Figure 10. Profile 6 plot depicting changes at Cape Charles Beach.
Figure 11. Profile 7 plot depicting changes at Cape Charles Beach.

Figure 12. Profile 8 plot depicting changes at Cape Charles Beach.
Figure 13. Profile 9 plot depicting changes at Cape Charles Beach.

Figure 14. Profile 10 plot depicting changes at Cape Charles Beach.
Cape Charles Beach

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Figure 15. Profile 11 plot depicting changes at Cape Charles Beach.

Cape Charles Beach

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Figure 16. Profile 12 plot depicting changes at Cape Charles Beach including the sewage outfall pipe.
Figure 17. Profile 12 plot depicting changes at Cape Charles Beach excluding the sewage outfall pipe.

Figure 18. Profile 13 plot depicting changes at Cape Charles Beach.
Figure 19. Distance of MHW from the baseline (feet).
Figure 20. Elevation above MLW of the base of dune.

Figure 21. Distance to base of dune from the baseline.
showed a slight increase in elevation. The BOD position of profile 2 grew significantly bayward whereas the BOD position of profile 8 moved only several feet. The average elevation of the BOD for both April, 1992 and April, 1993 is about +5.0 feet MLW. This elevation was used to determine changes in dune volume that will be discussed in the next section.

The peak elevation of the dune system has increased with time except for the dune area between profiles 10 and 11, the area of chronic erosion. Between profiles 3 and 10, the peak dune elevations averaged +10 feet MLW by April, 1993.

2. Beach, Dune, and Nearshore Volume Changes

The amount of fill material either lost or gained along the shore zone can be measured by changes in cubic yards (cy) per shore cell. The rate of change in fill volume is expressed in terms of cubic yards per linear foot along the shore per year (cy/ft/yr). The subaerial portion of the beach profile which includes the subaerial beach and dune has shown a marked loss of material since the beach nourishment project, from March, 1988 to April, 1993, in shore cells 6 through 12 (Figure 22). There has been an increase in subaerial volume in shore cells 1 to 3. There was an initial loss of material between shore cells 3 and 6 from March, 1988 to April, 1992 but then a subsequent increase was seen from April, 1992 to April, 1993.

The rate of change of the subaerial portion of the profiles for the four years following the beach fill project show significant losses in cells 4 to 12 (Figure 23). Slight gains in subaerial volume rates change occur in cells 1 and 2 with no net gain at cell 3. From April, 1992 to April, 1993, loss rates continue for cells 8 to 11 with what would appear to be significant corresponding gains in cells 1 to 6.

The dune portion of the subaerial region has shown a marked increase in volume from March, 1988 to April, 1992 and to April, 1993 between shore cells 1 and 8 (Figure 24). There was an increase in dune volume from March, 1988 to April, 1992 from shore cells 8 to 13 but then the dune eroded significantly in that area from April, 1992 to April, 1993. While small dunes still exist at profiles 10 and 11, a portion of the beach between them has eroded such that the bulkhead is now exposed.

The rate of change of dune volume shows an increased rate of gain from cells 2 to 5 and an increased rate of loss from cells 8 to 12 between the periods March, 1988 to April, 1992 and April, 1992 to April, 1993, respectively (Figure 25). These patterns of rates of volume change correspond to the patterns of volume change for the subaerial region.

Generally, the nearshore region has experienced an increase in volume from March, 1988 to April, 1992 for shore cells 2 to 9 (Figure 26). Slight losses in the nearshore have occurred in shore cells 10 to 13 from March, 1988 to April, 1993 and cells 2 to 4 from April, 1992 to April, 1993. The rate of volume change in the nearshore for the first four years after the fill (March, 1988 to April, 1992) reflect the volume change discussed above (Figure 27).

An increase in nearshore sediment volume creates a corresponding decrease in nearshore water depth off the southern half of the public beach creating a large low tide terrace. In fact, MLW moved an average of 95 feet bayward of its post-fill position by April, 1993 (profiles 1 to 7). A corresponding landward movement of MLW was measured along the northern half of the public beach (profiles 8 to 13).

The overall assessment of the sediment volume changes at Cape Charles is that the beach fill placed in March, 1988 has significantly eroded on the northern half of the project. The transport of the eroded material appears to be southward alongshore as well as offshore. Also, a significant portion of material has been trapped in the dune area. A summary of volume changes is
Figure 22. Subaerial beach volume calculations by cell.

Figure 23. Subaerial beach rate of change.
Figure 24. Volume of sand in the dune area relative to the 1987 fill.

Figure 25. Volume rate of change in the dune area.
Figure 26. Nearshore volume change relative to the 1987 beach nourishment project.

Figure 27. Volume rate of change in the nearshore region.
depicted in Figure 28 by shore cell. Once again, the major losses were to the subaerial region from March, 1988 to April, 1992 from cells 7 to 12 indicating that erosive wave forces are concentrated in that region.

Of the original 87,000 cy placed along the Cape Charles Public Beach in 1988, approximately 70,500 cy remained in 1992 (Table 4). By 1993, an additional 200 cy was present in the system; however, this net gain was exclusively in the dune area. In all, there was a net loss of 19% of the total fill volume over five years. There was no doubt some fill losses beyond the limits of the survey as well as sediment input from native sources. Also, some fill material was probably lost around the end as well as through the channel jetty given the net southward littoral drift.

C. Anthropogenic Impacts to Shoreline Processes

Obviously, the major impact to the Cape Charles Public Beach has been the 1988 beach fill project. The channel harbor jetty is the main structural feature controlling the littoral processes and the fate of the nourished beach material but the storm drain outfall has also had a local impact to the very north end of the beach. The channel jetty is relatively low at the shoreward end.

The storm drain outfall exits the bulkhead at about profile 13, heads due west for 285 feet and crosses the line of profile 12 (Figure 29). The current beach planform shows that there is an embayment on the south side of the structure. Wave diffraction around the end of the outfall, especially from the northwest alters the direction of wave approach along the shoreline. This causes sand to be transported both north into the lee of the gabion spurs as well as southward, creating the high erosion zone seen at profiles 10 and 11. However, without the storm drain and spurs, the entire north end of the beach most likely would have eroded back to the bulkhead by this time.

Table 4. Cape Charles volume changes relative to the 1987 beach nourishment project.

<table>
<thead>
<tr>
<th>Date</th>
<th>Subaerial* (cy)</th>
<th>Nearshore (cy)</th>
<th>Dune (cy)</th>
<th>Total (cy)</th>
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<tr>
<td>Mar 1988</td>
<td>62,371</td>
<td>21,821</td>
<td>2,605</td>
<td>86,797</td>
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<td>33,173</td>
<td>27,158</td>
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<td>Apr 1993</td>
<td>32,750</td>
<td>26,475</td>
<td>11,452</td>
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* Excludes dune volume.

IV. WAVE MODELLING AT CAPE CHARLES

A. RCPWAVE Setup

A detailed discussion of wave processes, sediment transport and numerical modelling are beyond the scope of this report, the interested reader can refer to Appendix I for a listing of pertinent references. The use of RCPWAVE to model the hydrodynamics at Cape Charles assumes that wave transformation is affected only by the offshore bathymetry (Figure 30). The purpose here is to present a general view of the impinging wave climate. Figure 30 was constructed from digital bathymetric data obtained from the National Oceanic Survey (NOS, 1983). The position of the shoreline is MLW prior to the beach nourishment project.

The local wave climate input for RCPWAVE was derived by wind/wave hindcasting by using wind data from the Yorktown Power Plant owned and operated by Virginia Power. The procedure described in section II.B. of this
Figure 28. Summary of volume changes.
Figure 29. Base map depicting distance to MHW from the baseline.
Figure 30. Shoreline and offshore bathymetry grid at Cape Charles Beach used in the RCPWAVE evaluation.
The report produces a significant wave height and period for three fetch exposures, Northwest (NW), West (W) and Southwest (SW), for the wind record years 1987 to 1990 (Table 3). We have defined these parameters as the seasonal modal wave conditions and they were modelled at a still water level of +2.5 feet MLW or about MHW. A severe winter storm condition was also run using an incident wave condition of 4.9 feet from the northwest at a still water level of +4.9 feet MLW which has a return frequency of three years (Boon et al., 1978) associated with an extratropical storm event.

B. Wave Height Distribution and Wave Refraction

RCPWAVE takes an incident wave condition at the seaward boundary of the grid and allows it to propagate shoreward across the nearshore bathymetry. Frictional dissipation due to bottom roughness is accounted for in this analysis and is relative in part to the mean sand size (0.25 mm). Waves also tend to become smaller over shallower bathymetry and remain larger over deeper bathymetry. In general, waves break when the ratio of wave height to water depth equals 0.78 (Komar, 1976).

Upon entering shallow water, waves are subject to refraction, in which the direction of wave travel changes with decreasing depth of water in such a way that wave crests tend to parallel the depth contours. Irregular bottom topography can cause waves to be refracted in a complex way and produce variations in the wave height and energy along the coast (Komar, 1976).

From the perspective of beach stability and behavior, it is the energy and momentum flux entering the surf zone that are important. Both quantities are proportional to the square of the wave height; the height of the setup at the shore is directly proportional to the breaker wave height (Wright et al., 1987). However, in the case of the three annual modal wave conditions, RCPWAVE did not define a breaking wave condition across the grid’s nearshore region due to the extreme dissipative nature of the shallow bathymetry. That is to say, the incident waves simply became gradually smaller over a long shallow nearshore region and only broke at the immediate shore cell. The wave height and direction at the shore cell were then used to compute longshore transport rates. Figures 31A and 31B show wave height distribution along the Cape Charles shoreline for the southwest and west annual modal wave conditions. Figures 32A and 32B show the annual modal and storm wave for the northwest condition which defined breaking waves 1 to 3 cells from the shoreline.

The distribution of wave heights along the Cape Charles shoreline is predicted to be rather uniform under the west modal condition. There are larger waves predicted in the central portion of Cape Charles relative to the north and south ends under the southwest modal wave. Under the northwest modal wave condition, slightly larger waves are predicted at each end of the public beach. The northwest storm condition shows a significant increase in breaker wave height overall, especially at either end of the public beach.

In order to compare the model runs for the modal wave conditions, wave refraction patterns were plotted from the RCPWAVE output. Figures 33, 34 and 35 show the wave refraction vectors for the regional grid and the local grid for each annual modal wave condition. Figure 36 depicts wave refraction vectors for the northwest storm condition. The waves break just beyond the shoreline in the nearshore region under the storm scenario.

The wave vector plot of the west modal condition shows little refraction or alteration in the wave patterns on the regional and local grid scales (Figures 33A and 33B). The angle to the shoreline is slight or almost shore normal. The west condition has a relatively low incident wave energy with about the same event average duration as the northwest modal condition (i.e. about 15 hours).

The southwest modal wave condition shows perhaps the most oblique wave approach to the Cape Charles shoreline (Figures 34A and 34B). This would be very conducive to alongshore transport of beach material northward. However,
Figure 31. Breaking wave height ($H_b$) (ft) distribution for A) southwest and B) west annual modal wave conditions.
Figure 32. Breaking wave height ($H_b$) (ft) distribution for A) northwest annual modal and B) northwest storm wave conditions.
Figure 33. West modal conditions wave vector plot for A) regional and B) local grid scales.
Figure 34. Southwest modal wave vector plot for A) regional and B) local grid scales.
Figure 35. Northwest modal wave vector plots for A) regional and B) local grid scales.
Figure 36. Northwest storm wave vector plot for A) regional and B) local grid scales.
the southwester usually occurs during periods of normal water levels and has a low incident wave condition close to that of the west modal wave condition (i.e. about 1.3 feet). Also, RCPWAVE does not account for the effects of the harbor jetty that no doubt alters wave approach from the southwest by diffraction. The general effect would be to cause southwest approaching waves to bend and become more shore normal, thus reducing the potential for alongshore transport of beach material. The shallow nearshore created by the channel jetty is considered in the RCPWAVE runs.

The northwest modal wave condition appears to most affected by the Cherrystone Inlet Channel as seen in Figure 35A where the wave vectors become more southerly in direction and slightly larger in wave height. The northwest modal wave begins more oblique to the grid shore than the southwest modal wave but after crossing the channel and nearshore the resultant shore vectors are more shore normal. The wave heights at the shoreline (Figure 35B) are only slightly higher than the southwest and west conditions indicating that bottom friction may affect the larger waves before they reach the shore cell.

The effect of one storm can translate into a major sediment transport event that may be greater than several years of modal wave activity. The severe storm that is modelled in Figures 36A and 36B occurred in February, 1987, just before the beach fill project. Several other slightly smaller northwest wind events are recorded in the winters of 1988 and 1989. The storm condition was run at an elevated water level which allows waves in the shore zone to reach almost a meter in height before breaking. This will have a major impact on the subaerial beach and dunes.

The movement of sand along a beach zone is dependent on breaking wave height and angle of wave approach. Applications of littoral drift formulae are subject to large errors; hence, the absolute magnitudes predicted must be considered suspect or, at best, accepted with caution (Wright et al., 1987). However, the relative magnitudes as they vary along the coast under different wave scenarios is probably more meaningful as are predicted directions of transport. Estimates obtained using the selected method in this report include the moderating effects of breaker height variations.

The methods of calculating littoral drift used here are Gourlay's (1982) as discussed in Wright et al. (1987). The reader is referred once again to Wright et al. (1987) for a complete discussion of these formulae and their applications. Erosional or accretionary changes in the volume of sand stored in a beach are determined by the gradients in alongshore flux \((dQ/dy)\). Specifically, when the rate of littoral drift entering a given coastal sector exceeds the rate exiting the sector, accretion results. Erosion results when output exceeds input; there is no change when input and output are equal (Wright et al., 1987). Onshore-offshore sediment fluxes are not accounted for in the estimates of \((dQ/dy)\) here.

For the west and southwest modal wave conditions, the calculation of \((dQ/dy)\) for the Cape Charles public beach are shown in Figures 37A and 37B respectively. The calculation of \((dQ/dy)\) for the west modal condition shows that a significant area of deposition arises only near the channel jetty. The rest of the beach shows no net gain or loss (Figure 37A). For the southwest modal wave condition, the plot of \((dQ/dy)\) shows deposition along much of the Cape Charles shore except for the area between profiles 4 and 7 and between profile 11 to the northern boundary of the plot (Figure 37B).

For the northwest modal wave condition, the plotted patterns of \((dQ/dy)\) (Figure 38A) display alternating areas of erosion and deposition with a significant depositional spike near the channel jetty. Predicted areas of erosion occur just north of that spike at profile 1 as well as near profiles 4 and 9, and north of profile 13. According to the northwest storm condition, the patterns of erosion and deposition show erosion at profiles 4 and 9 and significant deposition near the channel jetty (Figure 38B).
Figure 37A. Gradient of alongshore energy flux \( \frac{dQ}{dy} \) (cy/hr) for the west modal condition.

Figure 37B. Gradient of alongshore energy flux \( \frac{dQ}{dy} \) (cy/hr) for the southwest modal condition.
Figure 38A. Gradient of alongshore energy flux (dQ/dy)(cy/hr) for the northwest modal condition.

Figure 38B. Gradient of alongshore energy flux (dQ/dy)(cy/hr) for the northwest storm condition.
The southwest modal wave condition has a greater per event duration but the northwest modal wave condition has a slightly higher wave energy input along the Cape Charles shore. The one area of predicted deposition is near the channel jetty. The rest of the public beach shore fluctuates under the impinging modal wave conditions with no other dominant erosion or depositional areas. The southwest and northwest appear to modify each other in that respect. However, the northwest storm condition forces the transport patterns in such a way that the areas of erosion and deposition closely align with the shore changes determined from the field survey data. The best correlation is the erosional area predicted between profiles 7 and 11 and the accretional area at the channel jetty.

What RCPWAVE fails to do is account for the beach fill, the effects of tidal currents and predict onshore-offshore sediment movement. Losses due to eolian action is also beyond its capability. The model did predict significant areas of deposition near the channel jetty as well as provide a somewhat balanced sediment transport scenario under modal wave conditions. The predicted area of significant erosion under the storm condition was also fairly accurate. However, without field surveys or other shoreline change data, it would be unwise to rely on the model output alone for shoreline management plans. With the field data and a close model correlation, one can begin to develop an accurate picture of a given shoreline situation.

V. CONCLUSIONS

Cape Charles’ public beach has been reduced in volume approximately 19% since the beach nourishment project of 1988. Added fill at the north end is needed to maintain a wide recreational beach. However, some type of sand retaining device should be used to keep the sand from eroding. Additional projects consisting simply of beach fill will only serve to increase the beach and nearshore along the southern half of the public beach.

The pattern of sediment movement has been well documented by a series of 13 beach profiles taken by VIMS personnel before (November, 1987) and after (March, 1988) beach nourishment and subsequently in April, 1992 and April, 1993. The beach, dune and nearshore regions have been significantly reduced in size and volume in an area corresponding to profiles 9, 10 and 11 to a point that the bulkhead is exposed.

Except for profiles 1 and 2, the entire post-beach fill shoreline has receded. Most of the sand losses from the subaerial beach have shown up in the nearshore and newly created dune system. A relatively wide usable subaerial non-vegetated beach zone occurs only along the southernmost half of the public beach (profiles 1 to 6).

The increase in sediment volume in the nearshore also decreases the water depth, particularly along the south end of the public beach shore. This may tend to impede swimming at low water. The best option is to use the north end of the beach at those times where the nearshore is somewhat deeper.

Model runs using RCPWAVE show predicted deposition just north of the channel jetty. Significant erosion is predicted during periods of high water and northwest storms at profiles 9, 10 and 11, the area of measured significant erosion.

Part of this erosional pattern may be attributed to the storm water outfall, which redirects northwest impinging waves by diffraction. However, without the outfall even more of the northern part of the public beach may have eroded back to the bulkhead.
VI. RECOMMENDATIONS

In order to correct the erosional areas and provide a more usable subaerial beach between the BOD and MLW at Cape Charles Public Beach, the following is recommended:

1. Place an offshore breakwater(s) at the north end of the public beach so that it works in conjunction with the existing storm water outfall. Breakwater(s) specifications and position are subject to further analysis.

2. Place approximately 15,000 cy of select beach sand along the mid to northern half of the public beach in the area of severe erosion. The placement and position of the breakwater(s) will be designed to accommodate the fill. The bulkhead will be protected as well.

3. Raise the level of the channel jetty to above MHW at the shoreward end and place a small spur on the north side to prevent sand losses to the south around and through the jetty.

4. The dune grasses will continue to colonize the backshore and migrate bayward. A limit should be established along the backshore so the grasses will not reduce usable subaerial beach but still maintain the protection and integrity of the dune system during storms. Each fall, grasses that grow beyond the established line can be transplanted into bare areas of the dunes.

ACKNOWLEDGEMENTS

The authors thank Lee Hill, Woody Hobbs and Jerome Maa for their editorial reviews. A special thanks to Beth Marshall for the report preparation and compilation.
VII. REFERENCES


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APPENDIX I

Additional References about Littoral Processes and Hydrodynamic Modeling


