



**US Army Corps
of Engineers®**

Mispillion Inlet, Milford, DE

Mispillion Inlet Section 111 Shore Damage Mitigation Project

Appendix A - Engineering

April 2026

NAP

**North Atlantic Division
Philadelphia District**



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1 Existing Conditions

The existing conditions for the study area are used to inform the Future Without Project Conditions, and the design of the alternatives.

1.1 Vertical Datum

In accordance with ER 1110-2-8160 the Mispillion Inlet Project is designed to North American Vertical Datum of 1988 (NAVD88), the current orthometric vertical reference datum within the National Spatial Reference System (NSRS) in CONUS. The study area is subject to tidal influence and is directly referenced to National Water Level Observation Network (NWLON) tidal gages and coastal hydrodynamic tidal models established and maintained by the U.S. Department of Commerce (NOAA). The current NWLON National Tidal Datum Epoch (NTDE) is 1983-2001.

The Brandywine Shoal Light NWLON tidal gage is used to reference tidal water levels to NAVD88. Daily tidal fluctuations at the project site are semi-diurnal, with two highs and two lows per 24-hour day.

Table 1-1. Vertical Datum

Datum¹	Brandywine Shoal Light (feet, NAVD88)	Lewes (feet, NAVD88)
MHHW	2.60	2.02
MHW	2.16	1.60
NAVD88	0.00 ²	0.00
MSL	-0.29	-0.40
MLW	-2.70	-2.47
MLLW	-2.90	-2.63
MN	5.34	4.07

Notes: ¹Tidal datums based on 1983-2001 Tidal Epoch, ²NAVD88 based on NOAA's VDATUM Software

1.2 Sea Level Change

In accordance with ER 1100-2-8162, potential effects of relative sea level change (RSLC) were analyzed over a 50-yr economic analysis period and a 100-yr planning horizon. Research by climate science experts predict continued or accelerated climate change for the 21st century and possibly beyond, which would cause a continued or accelerated rise in global mean sea level. ER 1100-2-8162 states that planning studies will formulate

alternatives over a range of possible future rates of SLC and consider how sensitive and adaptable the alternatives are to SLC.

ER 1100-2-8162 requires planning studies and engineering designs consider three future sea level change scenarios: low, intermediate, and high. The historic rate of SLC represents the “low” rate. The “intermediate” rate of SLC is estimated using the modified National Research Council (NRC) Curve I. The “high” rate of SLC is estimated using the modified NRC Curve III. The “high” rate exceeds the upper bounds of IPCC estimates from both 2001 and 2007 to accommodate the potential rapid loss of ice from Antarctica and Greenland, but it is within the range of values published in peer-reviewed articles since that time.

Historical RSLC for this study, 1.24 ft over 100 years (3.77 mm/yr) is based on NOAA tidal records from 1919-2026 at Lewes, DE. Figure 1-1 shows historical RSLC at Lewes, DE. Several metrics for sea level are presented, the monthly mean sea level, 5-year moving average, and 19-year moving average. It is apparent that over long-time scales (19 years) mean sea level is steadily increasing. However, over shorter time scales mean sea level may increase or decrease. The monthly mean sea level goes up and down every year capturing the seasonal cycle in mean sea level. The 5-year moving average, orange line in Figure 1-1 captures the interannual variation (2 or more years).

USACE low, intermediate, and high SLC scenarios over the 100-yr planning horizon at Lewes, DE are presented in Table 1-2. Water level elevations at year 2030 are expected to be between 0.5 and 1.0 feet higher than the current NTDE. Water elevations at year 2080 are expected to be between 1.1 and 4.0 feet higher than the current NTDE.

Table 1-2. Sea Level Change Projections (Derived from Lewes, DE)

Year	Feet above Mean Sea Level Datum (1983-2001 epoch)		
	Low	Intermediate	High
1992	0	0	0
2000	0.1	0.1	0.12
2010	0.22	0.25	0.34
2020	0.35	0.42	0.64
2030	0.47	0.6	1.01
2040	0.59	0.8	1.45
2050	0.72	1.02	1.96
2060	0.84	1.25	2.56
2070	0.96	1.51	3.22
2080	1.09	1.78	3.96
2090	1.21	2.07	4.77
2100	1.34	2.37	5.66

2110	1.46	2.7	6.62
2120	1.58	3.04	7.66
2130	1.71	3.4	8.77

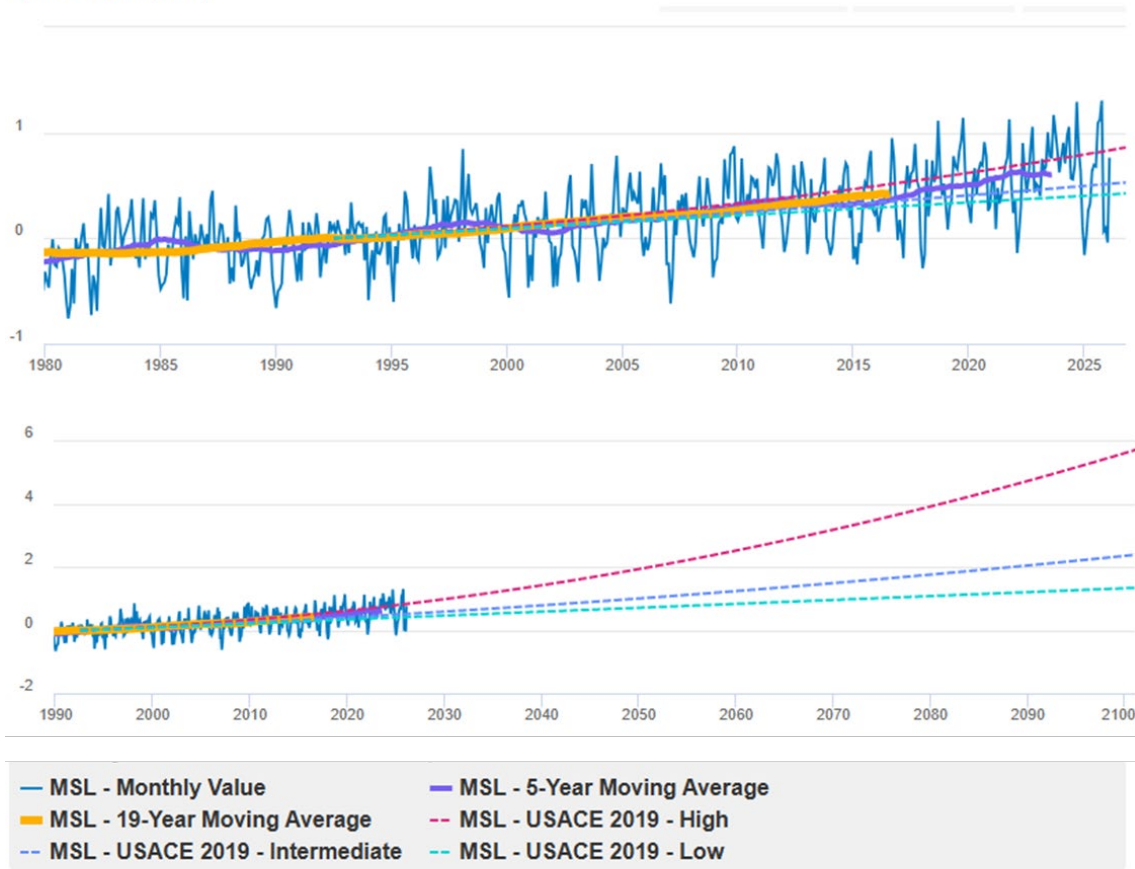
Notes: SLC rate used in equation based on 3.77 mm/yr (1.24 ft/100 yrs)

Sea Level Data and Projections: Lewes, DE (8557380)



NOAA Tide Gauge

Feet above Mean Sea Level Datum
(1983-2001 epoch)



SLC rate used in equation based projections: 3.77 mm/yr (1.24 ft/100 yrs)
SLC source: NOAA-NOS Tides & Currents Trend (Feb 1919 - Dec 2024)
MSL record span: 1919 to 2026 (107 years)
Missing data: The MSL record for this gauge has a gap of 5 or more years

Figure 1-1. Sea Level Change at Lewes, DE

The NACCS modeling performed for Delaware Bay was completed in the current NTDE. Therefore, the modeled water levels represent MSL in 1992. Future water levels are determined by adding the SLC values in Table 1-2. For example, a water level elevation of 10 feet NAVD88 based on the current National Tidal Datum Epoch (1983-2001), will have

an elevation in the year 2080 of 11.1, 12.1, and 14.0 feet NAVD88 under the USACE low, intermediate, and high SLC scenario respectively.

1.3 Seasonal and Interannual Fluctuations in Sea Level

The average seasonal cycle of mean sea level, shown in Figure 2.2, is caused by regular fluctuations in coastal temperatures, salinities, winds, atmospheric pressures, and ocean currents and on average causes a 0.6 foot (0.17 m) difference in sea level from September (highest) to January (lowest).

Interannual (2 or more years) variations in sea level, shown in Figure 2.3, are caused by irregular fluctuations in coastal ocean temperatures, salinities, winds, atmospheric pressures, and ocean currents (El Niño).

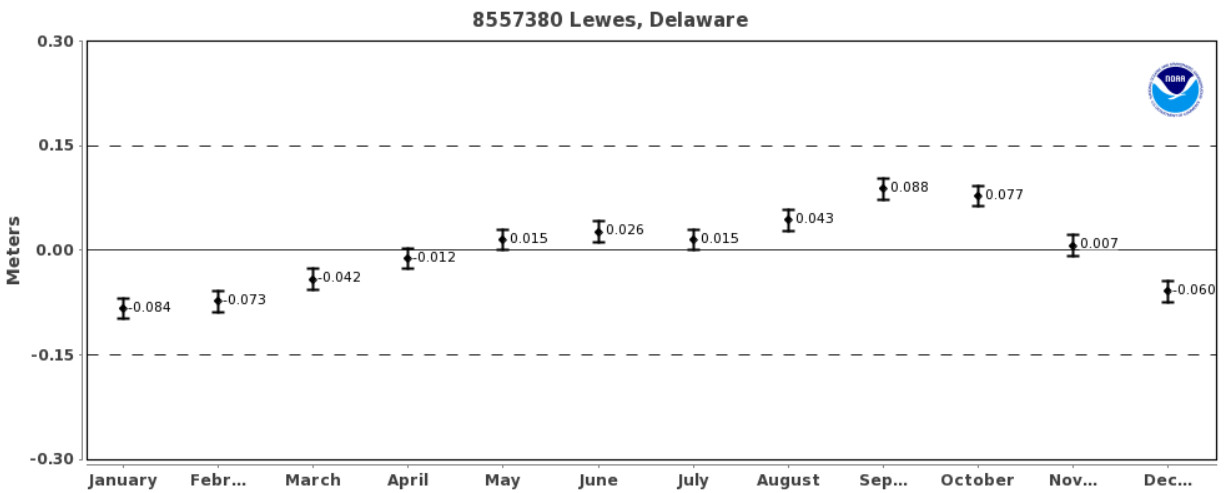


Figure 1-2. Average Seasonal Cycle in Sea Level at Lewes, DE

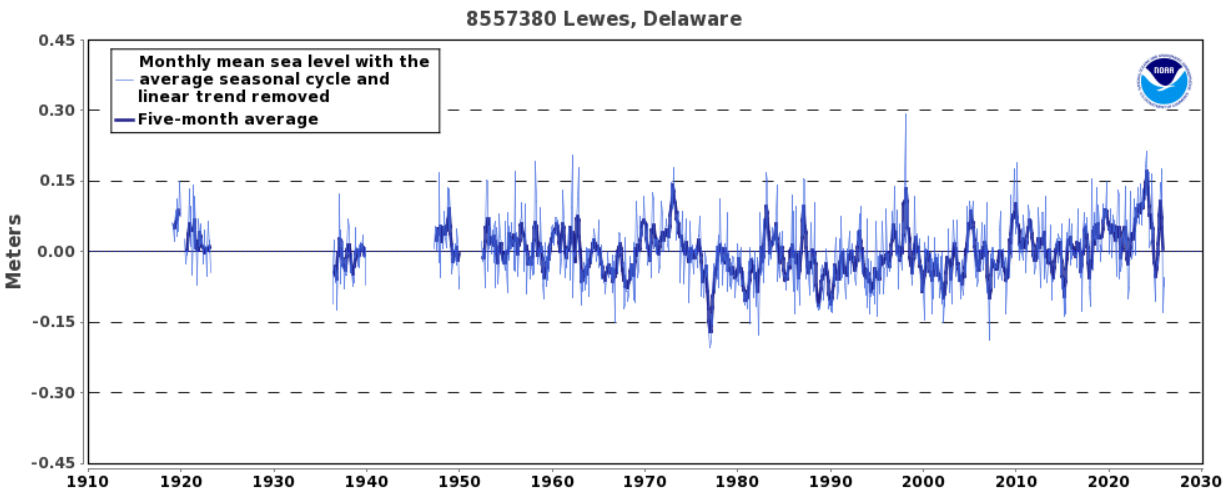


Figure 1-3. Interannual Variation in Sea Level at Lewes, DE

1.4 Astronomical Tide

Daily tidal fluctuations in the study area are semi-diurnal, with a full tidal period that averages 12 hours and 25 minutes; hence there are nearly two full tidal cycles per day. The mean tidal range in the Delaware Bay at Brandywine Light Shoal is 5.3 feet (Table 1-1). The tidal range in Delaware Bay varies as shown in Figure 1-4.

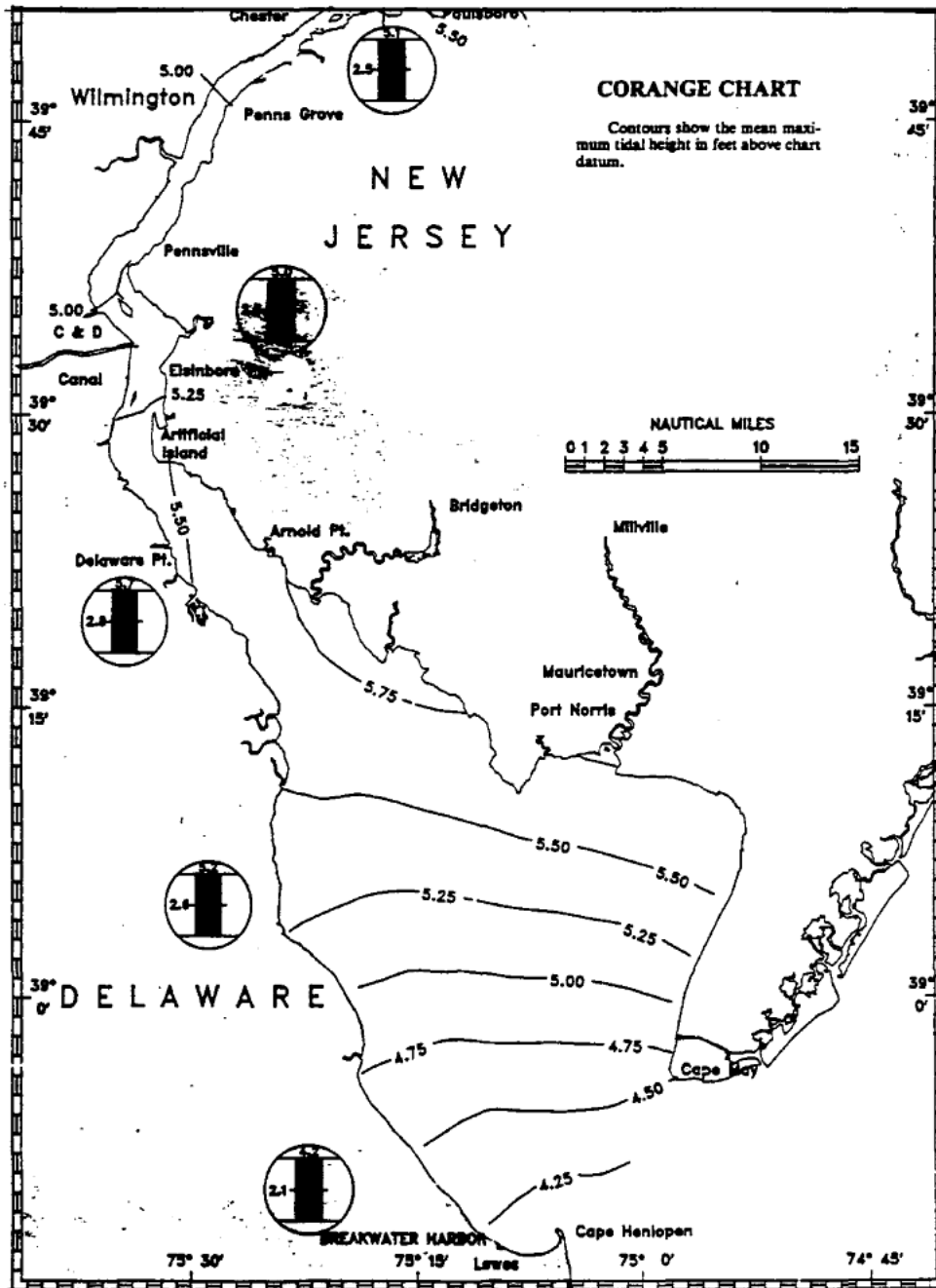


Figure 1-4. Maximum Tidal Range Contours (NOAA, 1988)

1.5 Storm Surge

Storm surge is the increased water level above the predicted astronomical tide due to storm winds over the ocean and the resultant wind stress on the ocean surface. The principal factor that creates flood risk for the study area is storm surge. The magnitude of the storm surge is calculated as the difference between the predicted astronomic tidal elevation and the actual water surface elevation at any time. Wind blowing over the ocean surface can generate storm surge. However, the largest and most damaging storm surges develop as a result of either tropical cyclones (hurricanes and tropical storms) or extra-tropical cyclones (“nor’easters”).

NACCS modeling results are used to define wave and baseline water level Annual Exceedance Probabilities (AEP) and in the development of the alternatives. The NACCS modeling study provides nearshore wind, wave, and water level estimates and the associated marginal and joint probabilities critical for effective coastal storm risk management. This modeling effort involved the application of a suite of high-fidelity numerical models within the Coastal Storm Modeling System (CSTORM-MS) to 1050 synthetic tropical storms and 100 historical extra-tropical storms. Documentation of the numerical modeling effort is provided in Cialone et al. 2015 and documentation of the statistical evaluation is proved in Nadal-Caraballo et al. 2015. Products of the study are available for viewing and download on the Coastal Hazards System (CHS) website: <https://chs.erdc.dren.mil/>.

NACCS Save Point #15247, Figure 1-5, is used to characterize the water levels and AEP for the project (Table 1-3).

Table 1-3. NACCS Water Level Annual Exceedance Probability

Return Period (years)	Average Annual Exceedance Probability	#15247 (ft, NAVD88)
1	100.0%	4.5
2	50.0%	5.2
5	20.0%	6.1
10	10.0%	6.6
20	5.0%	7.2
50	2.0%	8.7
100	1.0%	10.1
200	0.5%	11.4
500	0.2%	13.1

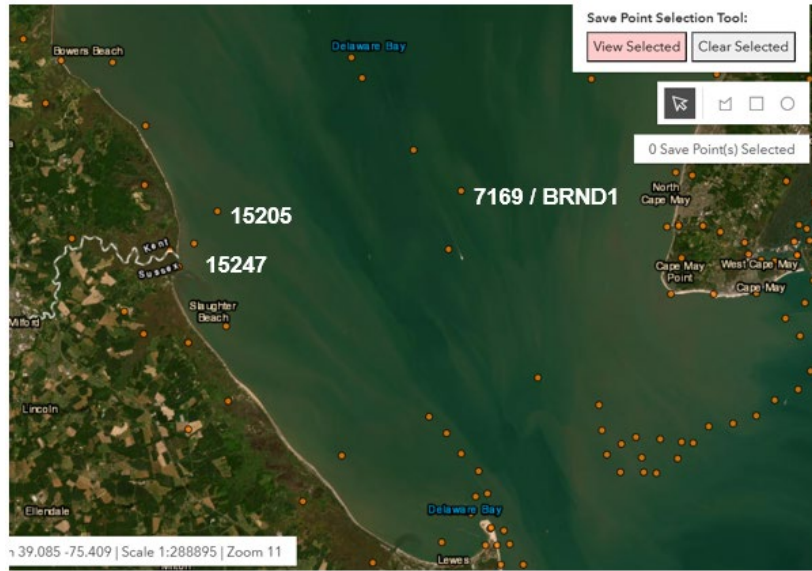


Figure 1-5. NACCS Save Point 15247

1.6 Wind

The prevailing wind direction reported at the Brandywine Shoal Light in Lower Delaware Bay is from the northwest. The annual wind rose diagram in Figure 1-6 shows that the most frequent and strongest wind directions (greater than 26 knots) are from the northwest. However, relatively strong winds (greater than 18 knots) occur from all directions. Seasonal wind roses, as seen in Figure 11, show that the wind regime varies from season to season, with the stronger winter winds prevailing from the northwest and the majority of the summer winds prevailing from the south. However, some of the strongest winds (highest velocity) observed throughout the year are from the northeast.

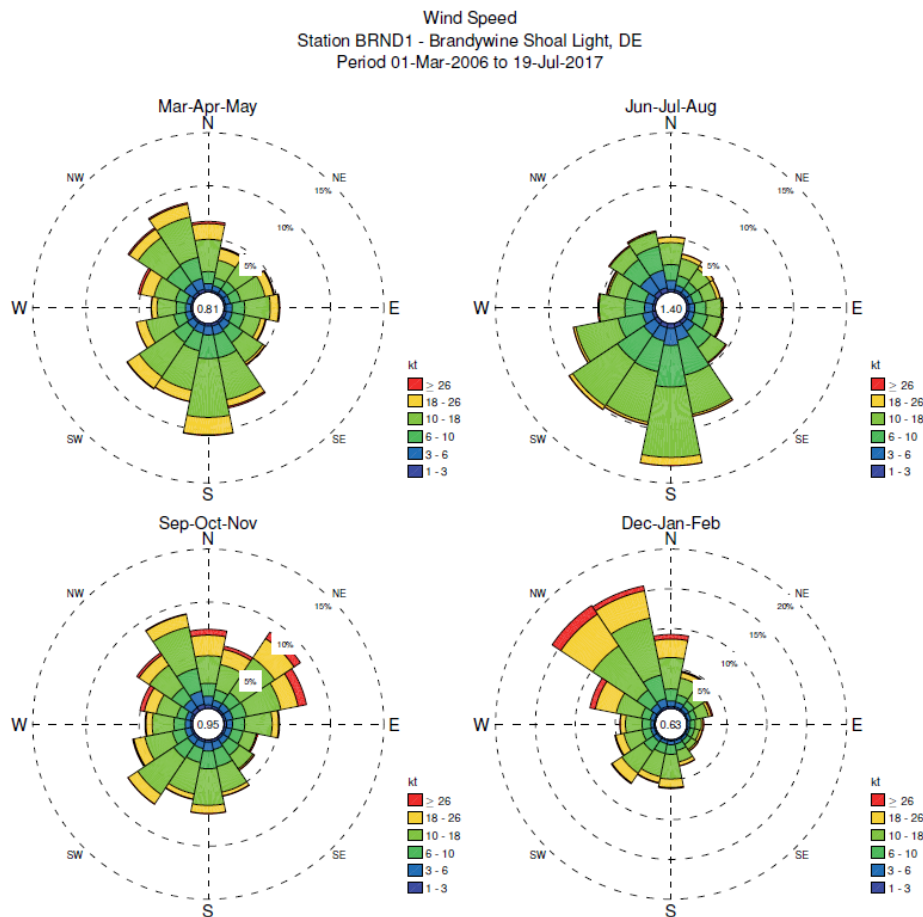


Figure 1-6. Seasonal Wind Roses at the Brandywine Shoal Light

1.7 Waves

Waves within Delaware Bay may be generated by local winds or propagate from the ocean through the mouth of the Bay. Further away from the mouth of the Bay the wave direction is associated with the wind direction and prevailing fetch. Two NOAA National Data Buoy Center (NDBC) stations are available inside Delaware Bay, 44054 and 44055. Station

44054 is located near the mouth of the Bay and is exposed to a combination of local winds and waves that propagate through the mouth of the Bay. Station 44055 is located farther up the Bay and is primarily exposed to locally generated waves. Figure 1-7 shows the annual wave rose Station 44054.

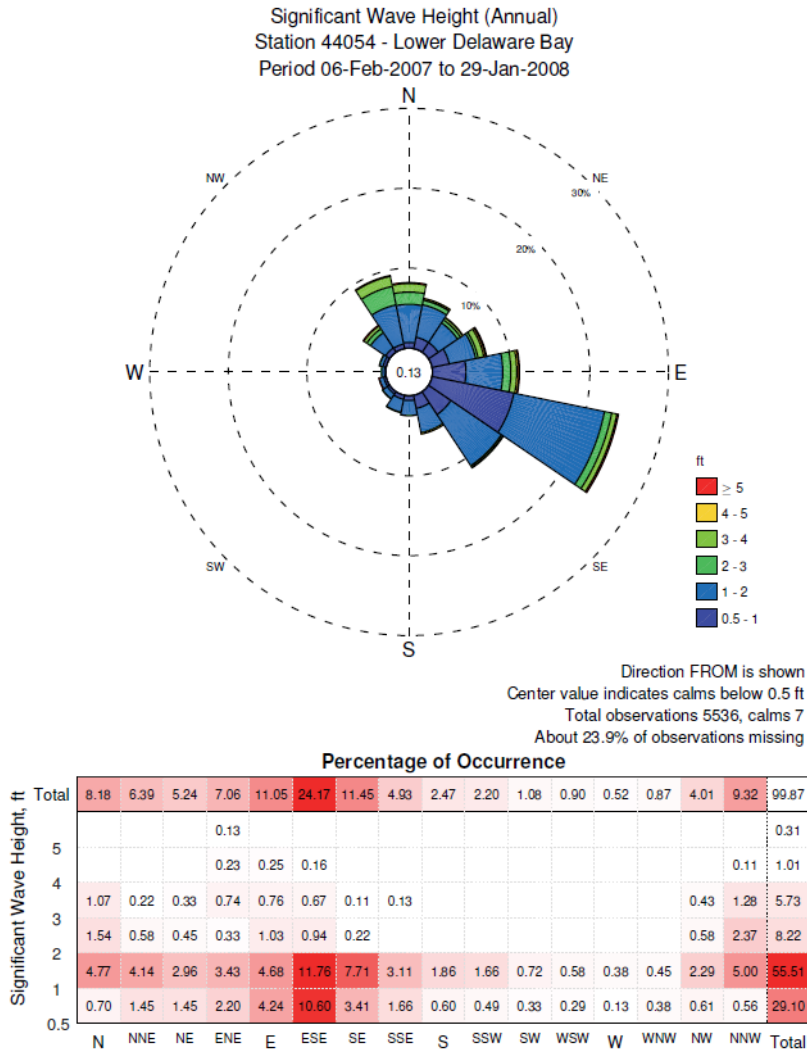


Figure 1-7. NOAA NDBC Buoy 44054 Annual Wave Rose

Nearshore wave conditions are obtained from NACCS save point 15247 (Table 1-4) at -6 ft NAVD88. These nearshore wave heights are transformed to the toe of the breakwater structure following Goda’s formulas for wave transformation across the surfzone (Goda, 2010). The design wave height (H_{10}) for the armor stone size in the breakwaters was 9 ft with a peak period of 6 seconds based on the 2% AEP.

Table 1-4. NACCS Wave Height Annual Exceedance Probability

Return Period (years)	Average Annual Exceedance Probability	#15247 Hs (ft)
1	100.0%	4.8
2	50.0%	6.2
5	20.0%	6.9
10	10.0%	7.2
20	5.0%	7.5
50	2.0%	7.8
100	1.0%	8.1
200	0.5%	8.5
500	0.2%	9.1

1.8 Sediment Transport

The shorelines adjacent to Mispillion Inlet are classified as “estuarine washover barriers” (Maurmeyer, 1978). These barriers (i.e., “beaches”) consist of a relatively narrow and vertically thin veneer of sand and gravel that is derived from older (pre-Holocene) headlands that exist along portions of the Delaware Bay shoreline. Historically the estuarine washover barriers on Delaware Bay have migrated landward and upward over older sediments in response to sea level rise. When compared to nearby open-ocean beaches in Delaware, the nearshore zone of Delaware Bay in the vicinity of Mispillion Inlet is a relatively low-energy environment in terms of wave energy, with corresponding lower rates of longshore sand transport under “normal” conditions. Potentially larger transport rates can occur in the vicinity of Mispillion Inlet during storm events. It appears that net littoral transport is from south to north at this location, based on the relatively small change in shoreline location south of the jetty along Slaughter Beach.

Longshore sediment transport (LST) is the process by which incident waves, and to a lesser degree, tidal currents, mobilize sandy sediment in the swash zone and transport it in the alongshore direction. The project area experience LST in either alongshore direction at different times, depending on the incident wave direction at any given time. Over long periods, transport in one direction will usually dominate transport in the other direction, with the dominant direction referred to as the direction of “net” transport.

1.9 Subsurface Conditions

A geotechnical investigation was conducted in 2015 by Geo-Technology Associates for DNREC prior to construction of the rock sill revetment at the North Mispillion Inlet. The exploration consisted of performing Standard Penetration Test (SPT) borings at 5

locations and 14 hand augers along the existing shoreline to collect subsurface data for engineering analysis of the revetment.

In general agreement with the published geology, the borings encountered approximately 7 to 12 feet of beach sediments composed of poorly graded sands and gravel. The beach sediments were generally loose to medium dense. Below surficial sediments, the borings encountered organic and elastic silts and clays. The very soft to stiff organic and elastic silts and clays were encountered to the termination depths of 25 feet.

Based on the results of the subsurface investigation, there is a greater than 10-foot-thick layer of very soft to soft compressible silts and clays below the surficial beach sediments. Assuming the compressible stratum is normally consolidated, any new load from sand placement and/or hardened structures such as revetments or breakwaters would settle from consolidation of the underlying soft strata. For “hardened” structures, such as breakwaters, the breakwater crest should be “over built” to allow for future settlement to maintain the minimum elevation for wave energy dissipation.

2 Erosion Caused by Federal Navigation Project

The study is authorized under Section 111, Rivers and Harbors Act (RHA) of 1968 as amended – Shore Damage Prevention or Mitigation of Damages Caused by Federal Navigation Projects (FNPs), P.L. 90-483, as amended by Section 940 of the Water Resources Development Act (WRDA) 1986 (P.L. 99-662). Section 111 provides authority to USACE to investigate, study, plan, and implement measures (structural or nonstructural) to prevent or mitigate damage to shorelines attributable to Federal Navigation Projects. Section 111 studies do not require a traditional cost benefit economic analysis but the least cost plan to restore the shoreline. Section 111 authorizes a justified level of work for prevention or mitigation of damages to both non-Federal public and privately owned shores to the extent that such damages can be directly identified and attributed to Federal navigation works.

Historical shoreline mapping of the study area, Figure 2-1 and Figure 2-2, demonstrates that there has been continuous shoreline erosion north of the inlet whereas the shoreline south of the inlet demonstrates relative stability. The shoreline change data indicate that during periods that predate the construction of the Mispillion Inlet navigation works (i.e., 1893 to 1939), the beach north of the inlet retreated at rates between 6 and 7 feet per year prior to the construction of the navigation works. Since construction of the Federal Navigation Project the shoreline north of the inlet has experienced an increase in rate of the shoreline erosion, as much as 10 to 14 feet per year. In contrast the shoreline south of Mispillion Inlet (Slaughter Beach) experienced accretion initially and has been relatively stable over the last 30 years.

A sand-starved shoreline on Delaware Bay might not respond like a sand-rich ocean shoreline. There may not be sufficient sand in Delaware Bay nearshore to create the same, typical impact seen at many ocean coast inlets with jetties, i.e., with updrift accretion and downdrift erosion. Clearly there was a pre-project erosion trend along the beach north of Mispillion Inlet that was exacerbated by construction of the navigation works. The apparently “stable” beach (no appreciable gain or loss of shoreline) south of the inlet, since the jetties have been in place, might represent an accretion. Without the jetties, the Slaughter Beach and downdrift sides of the inlet might have retreated a similar rate, e.g., 6 to 7 feet per year.

Moffatt & Nichol (2007) estimated the historical shoreline change rate as 9.1 feet per year from 1969-2002, yielding approximately 1,200 feet of shoreline erosion since 1893, with approximately one third of these losses (400 feet) attributable to construction of the Federal Navigation Project.

Historic shoreline mapping indicates that the Federal navigation works at Mispillion Inlet (i.e., the jetties) have interrupted littoral drift patterns in the study area. These impacts, which are most obvious as erosion along the zone north of the inlet, have led to several breaches of the barrier over the past four decades. The initial breach in the barrier occurred during a storm in 1977. This breach was closed by construction of a rock dike in 1985. However, continued shoreline erosion and storm conditions led to a further breach in 1987, followed by extension of the dike southward to close the new breach in 1993. Most recently (2016-2018), DNREC collaborated with U.S. Fish and Wildlife Service (USFWS), National Fish & Wildlife Foundation (NFWF), The Nature Conservancy, and Delaware Wild Lands to raise and lengthen the rock dike, add several groins on the west side of the dike, and place sandy sediment to serve as habitat for horseshoe crab spawning and migratory bird feeding. A project that cost approximately \$7.8 million.

Breaches in the barrier north of the inlet were closed because they allowed the tidal flow between Delaware Bay and the Mispillion River to take a “short-cut” through the breach and hence avoid the jettied portion of the inlet. This increased shoaling in the portion of the Mispillion River between the breach, the inner end of the jetties, and in the Cedar Creek navigation project. It also transported some of this material offshore to form ebb tidal shoals that were exposed at low tide affecting safe navigation of the inlet; allowed damage to docks and shorefront property along the Mispillion River by waves that propagated through the breach; and potentially increased the flood risk to developed areas upstream of the breach. Secondary impacts of the breaches have led to deterioration of important horseshoe crab spawning habitat and feeding and resting habitat for the threatened red knot and other migratory birds. Furthermore, the extensive work in 2016 by the US Fish and Wildlife Service to restore normal tidal circulation and wetland restoration in the Prime Hook National Wildlife Refuge south of Mispillion Inlet on Cedar Creek, may be jeopardized by breach-induced shoaling in Cedar Creek.

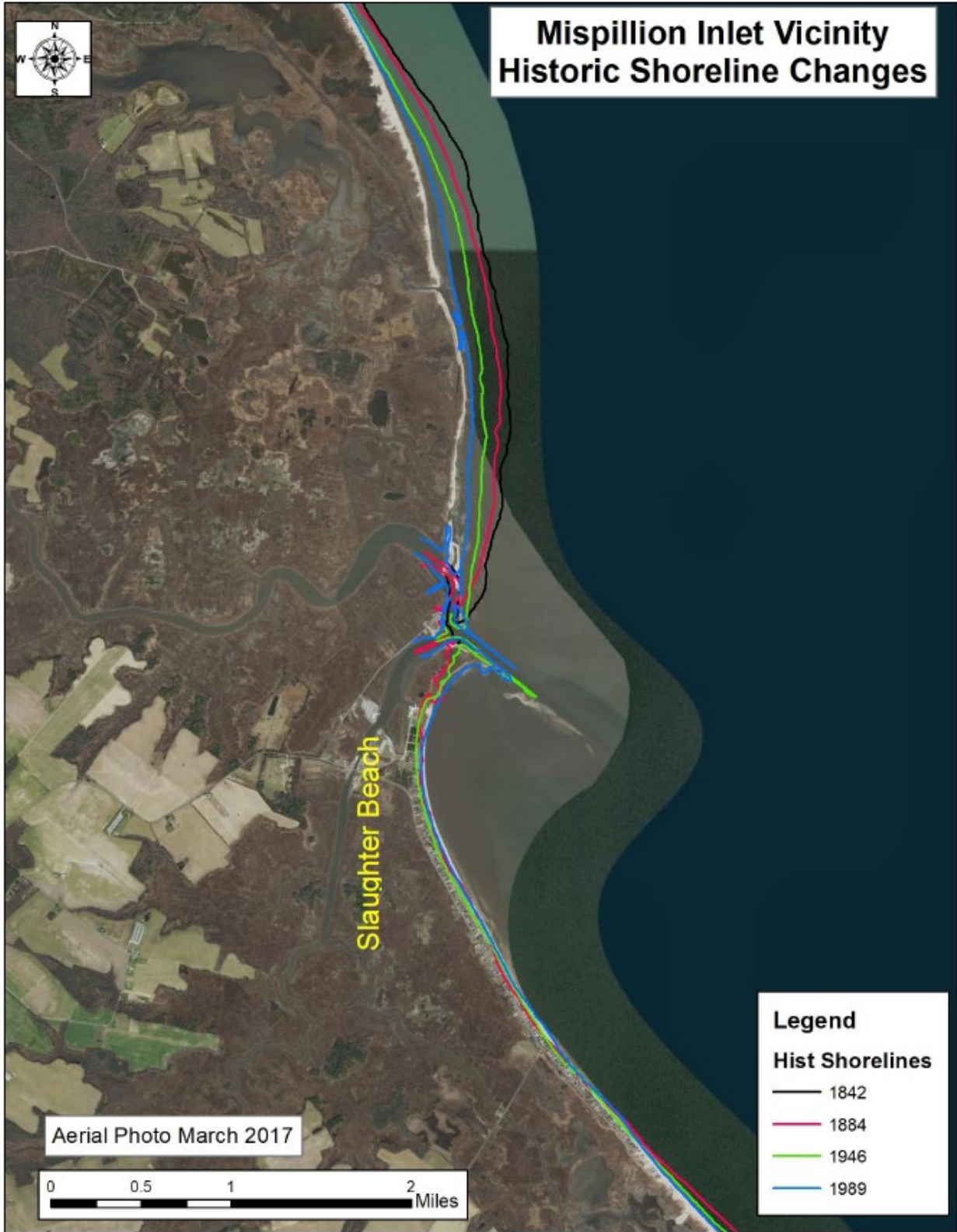


Figure 2-1. Historical Shoreline Changes (1842-1989)

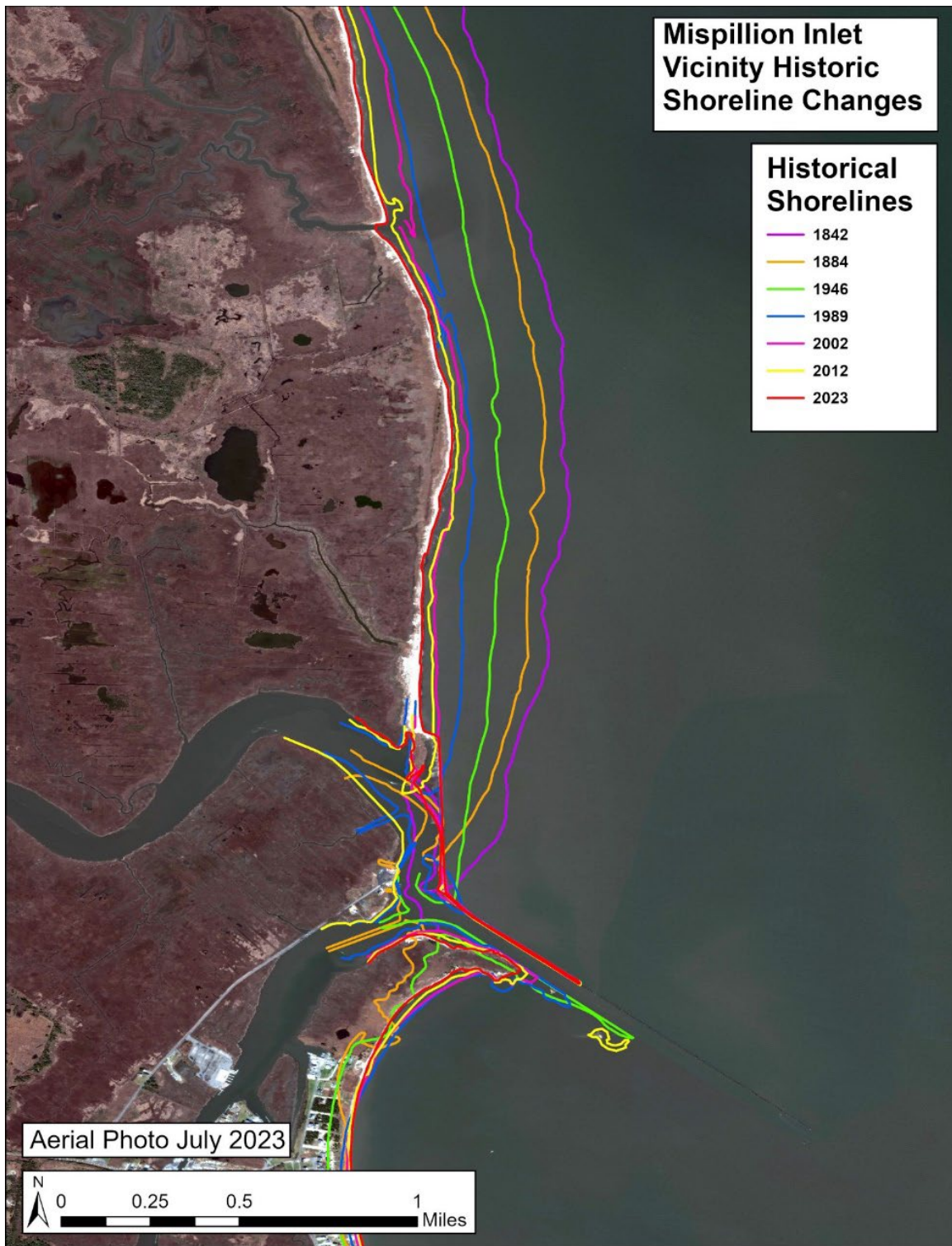


Figure 2-2. Historical Shoreline Changes (1842-2023)

3 Future Without Project Condition

In the future without the project, the shoreline is expected to continue to erode at 9.1 feet per year, and by the year 2055 could past the landward tip of the dike and significantly increase the risk of a breach (Figure 3-1). This would result in the loss of beach, dune, and salt marsh habitat north of the inlet. In addition, a breach would cause water flowing in and out of Mispillion to be diverted to the new breach resulting in shoaling in Mispillion Inlet. In the past breaches have been closed with costly dike construction projects.

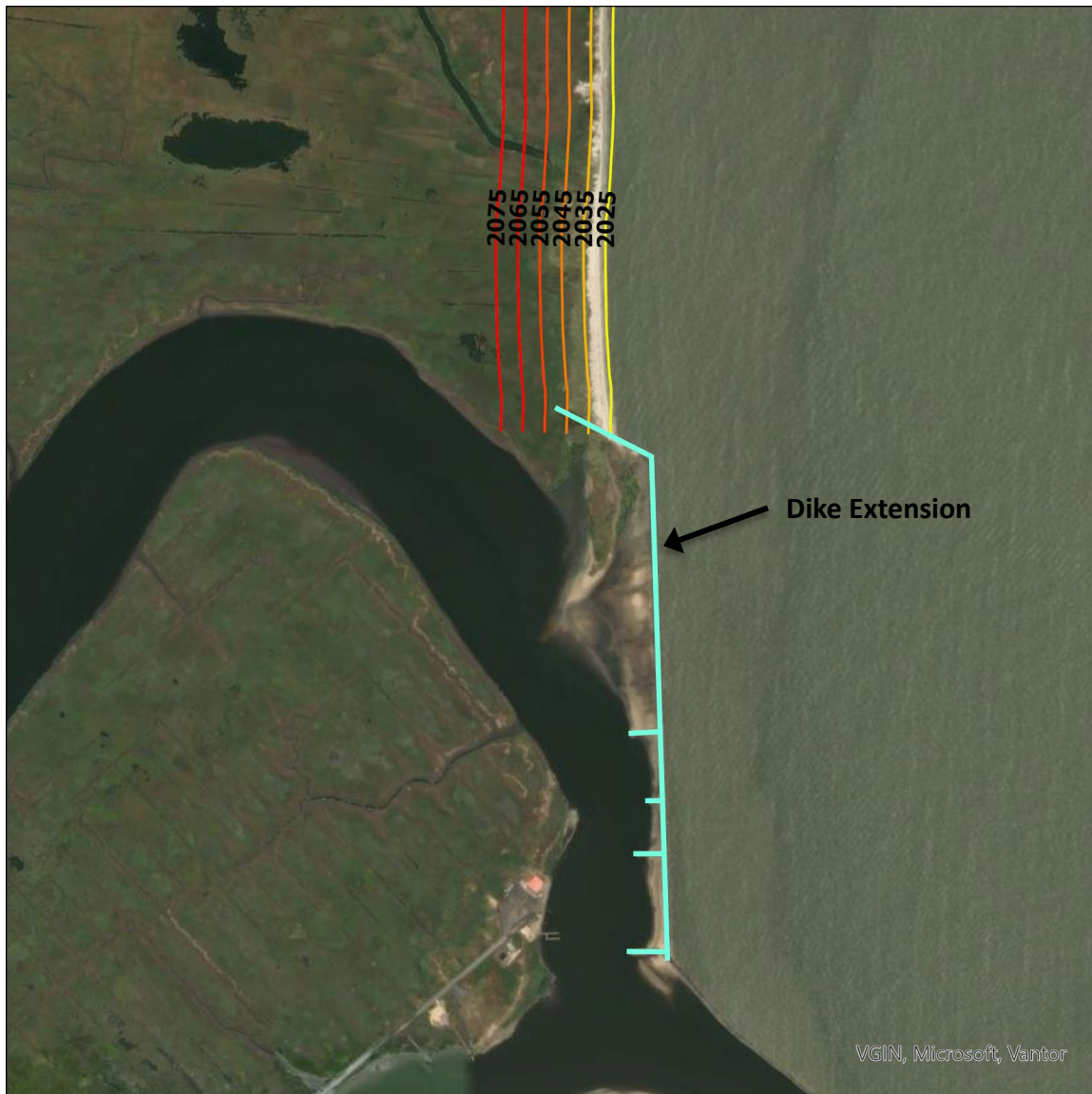


Figure 3-1. Projected Shoreline Changes over 50-years (2025–2075)

4 Alternative Development

4.1 Overview

The final array of alternatives included three plans:

- No Action
- Beach Nourishment (Mechanical or Hydraulic Placement)
- Beach Nourishment with Nearshore Breakwaters

In addition to the final array of alternatives, Engineering developed designs and quantities for a dike extension. Engineering designs and quantities were developed for a new dike extension before being screened out and eliminated from further consideration.

The objective of the beach nourishment alternatives is to restore and maintain the remaining beach north of the existing dike at the Conch Bar (Figure 4-1). Restoring the beach to mitigate damage to shorelines attributable to Federal Navigation Project will decrease the likelihood of another breach occurring and restore critical habitat for horseshoe crabs and the threatened and endangered species that rely on the horseshoe crabs.



Figure 4-1. Photograph of the 2017 DNREC Dike Extension and Conch Bar (North of the Dike Extension)

Initially beach nourishment alternatives considered periodic nourishment, but it became evident that it was not feasible to include several periodic nourishment operations within the Section 111 CAP Authority. Therefore, several beach alternatives were developed to maximize the longevity of a beach nourishment alternative as a one-time placement with advance nourishment and nearshore breakwaters.

4.2 Beach Nourishment Longevity

The longevity of a beach nourishment project is typically controlled by the background erosion rate and “spreading out” losses associated with the nourishment. Storm events are inherently included in background erosion rate, but an above average number of storm events could cause a nourishment project to erode faster, and likewise a calmer period of storms could increase the longevity of a project.

4.2.1 Background Erosion Rate

The historical shoreline change mapping has shown that the background erosion rate is -9.1 feet per year (Moffatt & Nichol, 2007).

4.2.2 Beachfill Diffusion (Spreading Out)

Beach nourishment projects constructed on a long beach represent a perturbation or planform anomaly, which under wave action, will spread out along the shoreline (Dean, 2002). This process is illustrated in Figure 4-2, which shows waves interacting with the beach nourishment causing sediment transport away from the anomaly and smoothing or spreading out of the sediment (Dean & Grant 1989). The term “spreading out” losses actually refers to a redistribution of the sediment and not a total loss to the system but rather a loss from the region in which the sediment is placed (Dean & Grant 1989). This process is referred to as “beachfill diffusion” since the process is modeled analytically using the one-dimensional diffusion equation, first utilized by Pelnard Consideré (1956). Diffusion losses within the study area could be significant at many of sites and greater than the background erosion rates, thus having an outsized effect on periodic nourishment quantities.

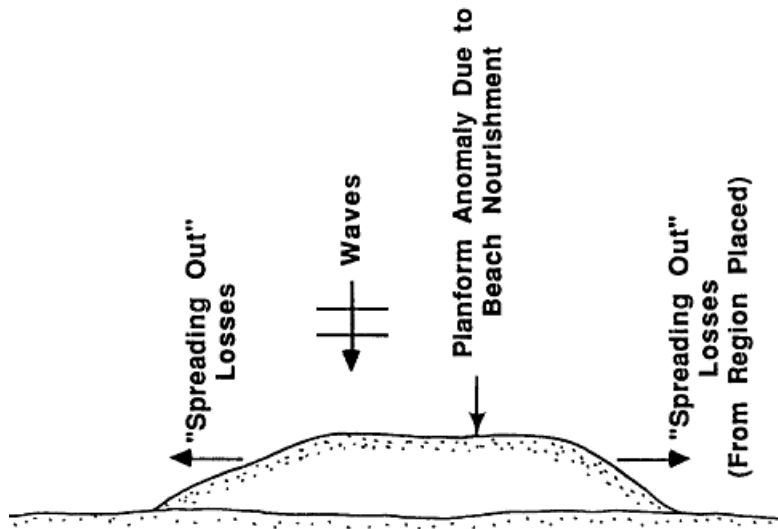


Figure 4-2. "Spreading Out" losses occurring from beachfill diffusion

Beachfill diffusion is modeled analytically in this study using solutions to the Pelnard-Consideré equation for a rectangular planform anomaly on an infinitely long shoreline. Losses are primarily a function of the wave energy, alongshore length of beach nourishment, and cross-shore width of planform anomaly. The non-dimensional solution to the equation is shown in Figure 4-3, where t' is a non-dimensional representation of time based on the ratio of the alongshore length (l) of beach nourishment anomaly, time (t) after construction, and longshore diffusivity (G). The longshore diffusivity is a function of how energetic the wave environment.

Figure 4-3 shows how the planform anomaly spreads out over time. The non-dimensional form of time indicates that rate at which diffusion occurs is a function of the diffusivity and alongshore length. Locations with more wave energy will have a larger longshore diffusivity and t' will increase. Similarly, as the alongshore length decreases, t' increases. An example solution to the Pelnard-Consideré equation for a 4,000 foot-long beach nourishment project is shown in Figure 4-4. The bottom panel of Figure 4-4 shows the fraction of sand volume remaining and the impact of background erosion, which is linearly added to diffusion losses.

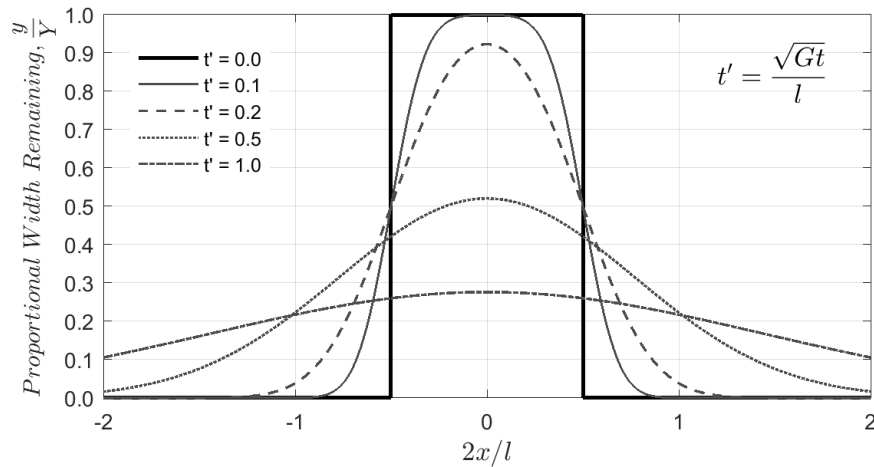


Figure 4-3. Non-dimensional Shoreline Evolution

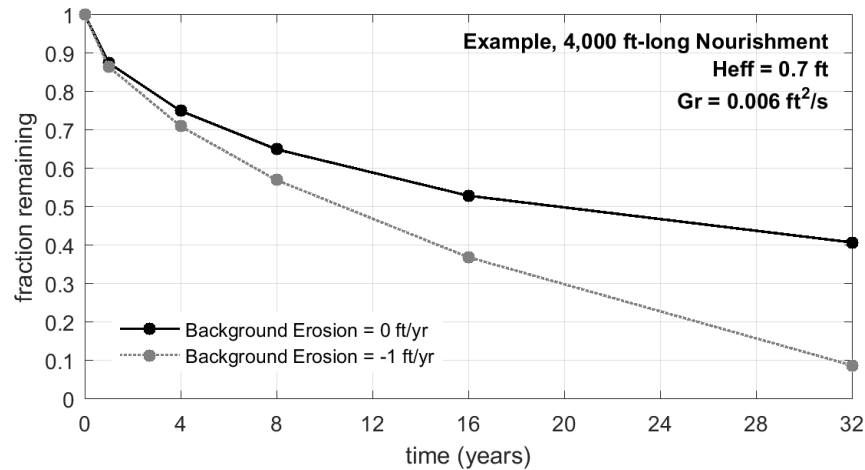
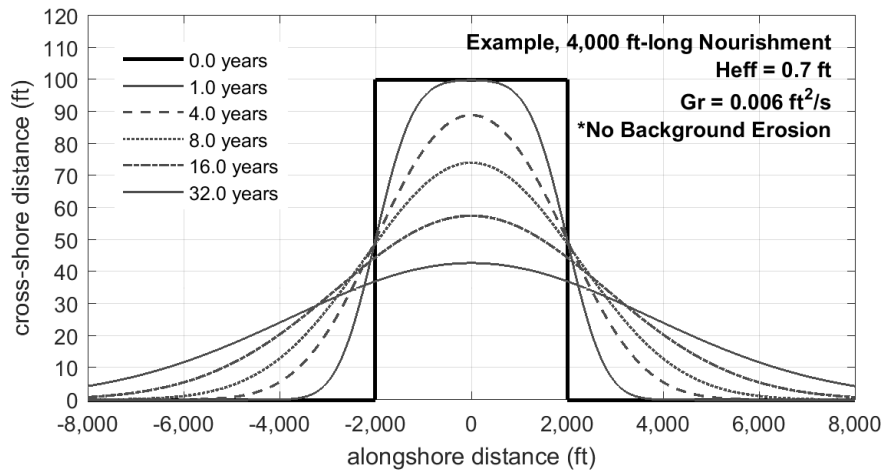


Figure 4-4. Example of Diffusion Losses at 4,000 foot-long Nourishment Project

The longshore diffusivity for the analysis is 0.0143 ft²/s, based on previous analyses of the effective wave height at NOAA Buoy 44055 conducted for the Delaware Dredge Material Utilization (DE DMU) Coastal Storm Risk Management Feasibility Study (USACE, 2018).

The existing dike just south of the proposed beach nourishment site is functioning as a terminal groin by reducing sediment transport to the south and will significantly reduce diffusion losses. Dean & Grant (1989) describe a simple approach to incorporate terminal groins in the diffusion analysis. The recommended approach for a single terminal groin is to increase the effective length of the nourishment to twice the physical length of the project and apply background erosion rates that account for the influence of the terminal groin. By doubling the effective length of the nourishment, diffusion losses are cut in half.

The results of the beachfill diffusion analysis for a 60 foot berm and 150 feet berm width are shown below in Table 4-1.

Table 4-1. Diffusion Results

Berm Width	Diffusion (ft/yr)	Background Erosion (ft/yr)	Longevity
60 feet	-3.6	-9.1	5 years
150 feet	-6.2	-9.1	10 years

4.3 Nearshore Breakwaters

Nearshore breakwaters are an alternative beach stabilization structure to groins and are designed to reduce wave energy and longshore sediment transport along the shoreline. The length and gaps between the nearshore breakwaters may be designed to try and encourage a salient (shoreline bulge) or tombolo (shoreline attached to breakwater) to form behind the breakwater. EM 1110-2-1617 provides a description of nearshore breakwaters:

Shore-parallel, detached (not shore-connected) breakwaters may be built singly or in series spaced along the shoreline. Detached breakwaters are constructed close to shore to protect a stretch of shoreline from low to moderate wave action and to reduce severe wave action and beach erosion. Sand transported along the beach is carried into the sheltered area behind the breakwater where it is deposited in the lower wave energy region. Protection afforded by the breakwater will limit erosion of the salient during significant storms and promote growth during periods of low to moderate wave activity. The effectiveness of a nearshore breakwater or breakwater system depends on the level of wave protection and the length of the shoreline it protects; thus, the breakwater's height, length, wave transmission characteristics, and distance from shore contribute to its effectiveness. For a system of breakwaters, the width

of the gap between adjacent breakwaters and the length of the individual breakwater segments are also important.

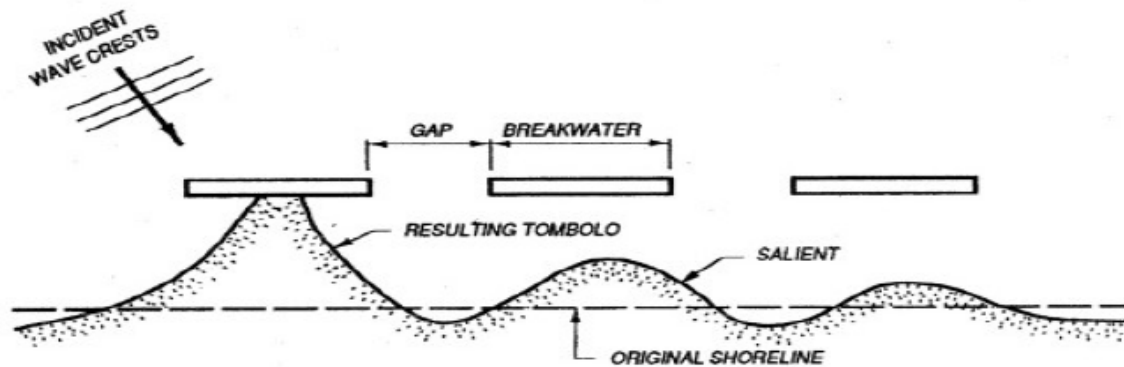


Figure 4-5. Nearshore Breakwaters and Features

Breakwaters are added to increase life of beach nourishment project by reducing wave energy. The design layout of breakwaters considers the length of the breakwaters, gap, distance offshore, and wave transmission characteristics. The Mispillion layout is designed to promote salient formation behind breakwaters and avoid tombolos. Tombolos are not desired for this project because they can function like a groin and severely reduce sediment transport downdrift. The breakwaters are estimated to increase the longevity of the 60-foot berm nourishment from 5 years (without breakwaters) to 10 years.

The following parameters are used in the breakwater layout to promote salients:

- Breakwater Length, $L_s = 200$ ft
- Gap = 100 ft
- Distance offshore = 130 ft
- Crest Elevation = 5 ft NAVD88
- Wave Energy Transmitted = $\sim 50\%$



Figure 4-6. Example of Nearshore Breakwaters at Norfolk, VA

4.4 Sediment Source & Staging Area

A sufficient quantity of suitable beach nourishment material can be sourced from local quarries nearby from approved quarry by the NFS. It is assumed that at least two approved quarries would be available within 30 miles of the project area. The contractor can use the same construction staging area on Lighthouse Road by the Mispillion River to give the contractor water access to the project site as previously used by DNREC project in 2018. This staging area would allow the contractor to conduct the two expected placement options (mechanical and hydraulic) for the beach nourishment. Water access allows the contractor to barge the sand to the project site and can determine their best placement option.

Other potential sources of sediment were considered but determined to be infeasible. Beneficially reusing dredge material from Mispillion Inlet was not considered feasible due to the relatively low quantities of dredge material (20,000 cy once every 4 to 5 years). Beneficially reusing dredge material from the main stem of the Philadelphia to Sea Federal Navigation Project or Buoy 10 (open water disposal area for the project) was not explored further because several existing projects (Delaware Bayshore) are already planning to use this material. In addition, previous planning studies and value

engineering studies have shown that the cost of placing dredge material from the main stem or Buoy 10 is expected to be very expensive due to the long distances the hopper dredge would have to carry the sediment before unloading and pumping the sediment the remaining distance over the shallow bathymetry to the site, which would require multiple booster pumps.



Figure 4-7. Photo of Dike Construction in 2018 (Moffatt & Nichol, 2018)



Figure 4-8. Photo of Staging Area in 2018 (Moffatt & Nichol, 2018)

4.5 Alternatives

4.5.1 Alternative 1 - Beach Nourishment

The beach nourishment for alternative 1 begins at the dog leg portion of the existing stone dike at Station 25+00 and extends for 1,700 feet to Station 45+00, with a 300' taper beginning at Station 42+00. The beach berm consists of a 120-foot-wide design berm, extending approximately 30 feet landward of the survey and construction baseline, for a total design beach width of approximately 150 feet (Figure 4). The construction berm width is 175 feet from the baseline. See Figure SK-001 for plan view and Figure SK-004, Detail 1 for a typical section.

Initial construction quantities were performed by calculating cross-sectional areas at each survey profile and then computing volumes between adjacent profiles using the average-end-area method. The average-end-area method was applied by averaging adjacent profile areas and multiplying by the orthogonal distance between profile azimuths. Wedge volumes, where the adjacent profile azimuths differed, were not included in the volume. Taper volumes are included in the initial construction quantities and computed as a pyramid.

Alternative 1 is estimated to need 80,500 cubic yards for initial construction and to have a longevity of approximately 10 years.

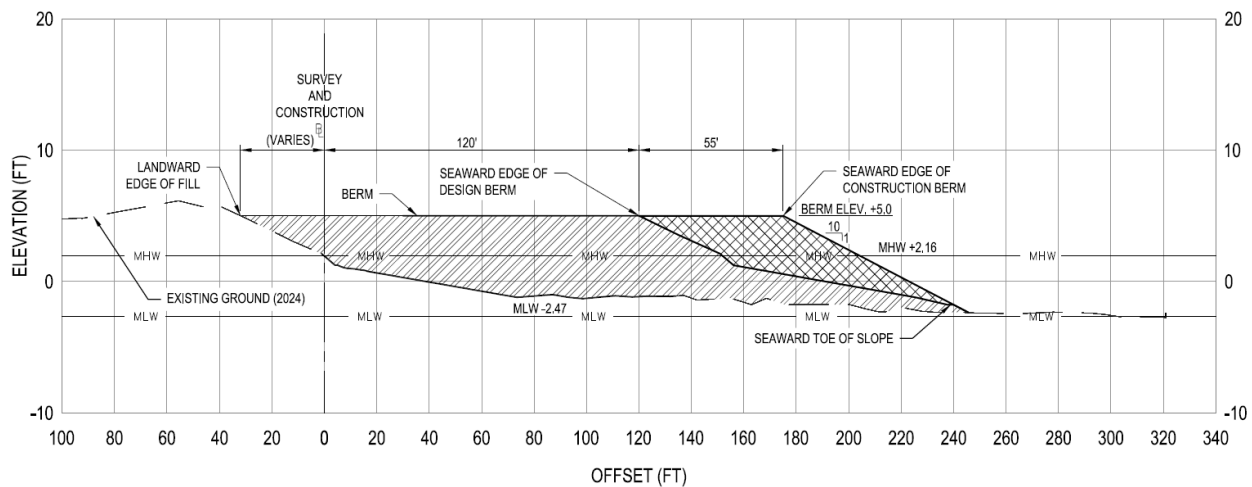


Figure 4-9. Alternative 1 – Beach Nourishment Typical Section

4.5.2 Alternative 2 - Beach Nourishment and Breakwaters

The beach nourishment for the breakwater alternative is like the design for Alternative 1 beach nourishment, extending over the same 1,700 feet of shoreline plus a 300-foot taper. However, the beach berm consists of a 30-foot-wide design berm, extending approximately 30 feet landward of the survey and construction baseline, for a total design beach width of approximately 60 feet.

Alternative 2 includes five rubble mound breakwaters to reduce wave energy, sediment transport, and increase the longevity of the nourishment project. Each breakwater is 200 feet long with an opening gap of 100-feet from the adjacent breakwater. Each breakwater consists of a 4-foot layer of Armor Stone with $W_{50}=1360\text{lb.}$, and a core stone layer consisting of R-4 riprap (Figure 4-10). The crest of the breakwater is 6-feet wide with a top elevation of +6 feet. The side slopes extend at 2H:1V to a 4-foot bench at elevation +4 feet. The breakwater foundation layer is a 12-inch-thick marine mattress that extends 5 feet beyond the stone on each side to prevent scour and undermining of the breakwater. The existing seafloor elevations where the mats would be placed are approximately +2 feet. Mats were chosen due to the durability against wave induced scour, and protection of geotextile during placement of rock.

See Figure SK-002 for Plan view and Figure SK-004 Detail 2 below of a typical section.

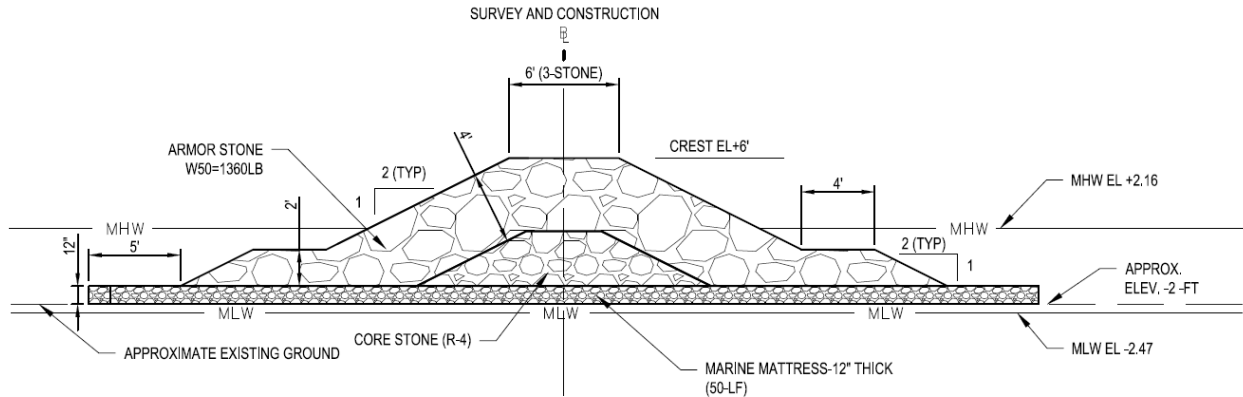


Figure 4-10. Alternative 2 - Breakwater Typical Section

Alternative 2 is estimated to need 30,200 cubic yards for initial construction and to have longevity of 10 years.

Breakwater Quantities:

- Marine Mattress: 52 SF x 200 LF = 10,400 CF
- Armor Stone: 125 SF x 200 LF = 25,000 CF
- R-4 Stone: 30 SF x 200 LF = 6,000 CF

4.5.3 Alternative 3 - Dike Extension

In addition to the final array of alternatives, Engineering developed designs and quantities for a dike extension. Engineering designs and quantities were developed for a new dike extension before being screened out and eliminated from further consideration.

In 2018 DNREC constructed a stone dike to fix the previous breach north of the inlet. The dike cross section designed by DNREC was utilized in the dike extension design for this report. The dike extension begins at the end of the existing DNREC dike approximately STA 26+00 and extends 1500 ft north, parallel to the shoreline. See Figure SK-003 for plan view and SK-004 Detail 3 for Alternative 3 typical section.

The stone dike was designed to be consistent with the DNREC dike design. The dike consists of a 3' thick layer R-7 riprap, and a corestone layer of R-4 riprap. The crest is 3-foot wide at elevation +6 feet with a side slope of 2H:1V. The toe of the dike extension where it meets existing ground is approximately elevation -2 feet. Geotextile forms the separation layer between the stone and sandy existing ground, to prevent the stone from settling into the sand. To construct the toe of the dike extension, excavation is required at

a 2H:1V slope. The primary reason why a geotextile layer, rather than a marine mattress, was selected for the dike design is to remain consistent with the existing DNREC design from 2017

Dike quantities:

- R-4 Core Stone 60 SF x 1500LF = 90,000 CY
- R-7: Riprap: 95 SF x 1500LF = 142500 CY
- Geotextile: 50 LF x 1500 LF = 75,000 SF
- Excavation: 350 SF x 1500LF = 525,000 CY
- Excavated Backfill: 120 SF x 1500 LF = 180,000 CY

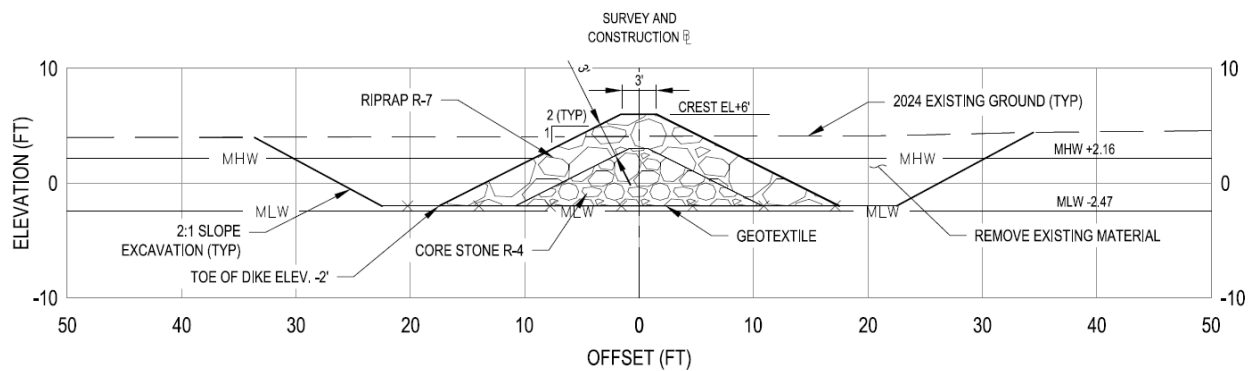


Figure 4-11. Alternative 3 - Dike Typical Section

5 Cost Estimate

5.1 Initial Project Charges

5.1.1 General

This section presents a detailed cost estimate for initial construction for the subject project. The selected plan consists of a beach nourishment placed along the shoreline north of the dike. The beach fill will be a one-time placement of 80,500 cubic yards of sand on the existing beach. Cost estimates are prepared separately for Mechanical Beach Replenishment and Hydraulic Beach Replenishment.

5.1.2 Basis of Cost

Cost estimates presented herein for the NED plan are based on November 2025 price levels including labor, equipment, materials, crews, unit prices, quotes, sub-contractor markups and prime contractor markups. The estimate was developed at a Class 4 level effort utilizing unit prices from sources such as 2025 RSMeans Cost Data Books and RSMeans Online for 2026 Quarter 1. The estimate is subdivided by feature and contains U.S. Army Corps of Engineers (USACE) feature Work Breakdown Structure (WBS) codes. Each WBS cost is subdivided into base cost, contingency and total cost. It is assumed that bidding competition will be present. Labor rates were developed from the local Davis-Bacon Wage Determination in Sussex County, Delaware.

5.1.3 Total First Cost

Initial costs assume work done by an established contractor placing the compatible sand onto the existing beach. NED real estate acquisition costs and pertinent contingency, engineering and design and construction management costs are also included. For more information, refer to the Main Report describing the NED Plan. Initial construction costs are shown in Table 5-1 for Mechanical Beach Replenishment and Table 5-2 for Hydraulic Beach Replenishment.

Table 5-1. Total First Cost – Mechanical Beachfill
 MISPELLION INLET
 MILFORD, SUSSEX COUNTY, DELAWARE
 SECTION 111- SHORE DAMAGE MITIGATION PROJECT

CWBS

18-Nov-2025

CIVIL WORKS
 WORK BREAKDOWN STRUCTURE

Number	Product Description	Quantity	UOM	Estimated Amount	Contingency Amount	Total Cost
01 -- -- -- --	LANDS AND DAMAGES	1	Job	\$207,000.00	\$41,400.00	\$248,400.00
17 -- -- -- --	BEACH REPLENISHMENT	1	Job	\$8,404,500.00	\$2,521,350.00	\$10,925,850.00
30 -- -- -- --	PLANNING, ENGINEERING, AND DESIGN	1	Job	\$1,136,700.00	\$341,010.00	\$1,477,710.00
31 -- -- -- --	CONSTRUCTION MANAGEMENT (S&A)	1	Job	\$779,368.00	\$233,810.40	\$1,013,178.40
TOTAL PROJECT COST				\$10,527,568.00	\$3,137,570.40	\$13,665,138.40

Prepared by:
 CENAP-ECT-E

Table 5-2. Total First Cost – Hydraulic Beachfill
 MISPELLION INLET
 MILFORD, SUSSEX COUNTY, DELAWARE
 SECTION 111- SHORE DAMAGE MITIGATION PROJECT

CWBS

18-Nov-2025

CIVIL WORKS
 WORK BREAKDOWN STRUCTURE

Number	Product Description	Quantity	UOM	Estimated Amount	Contingency Amount	Total Cost
01 -- -- -- --	LANDS AND DAMAGES	1	Job	\$207,000.00	\$41,400.00	\$248,400.00
17 -- -- -- --	BEACH REPLENISHMENT	1	Job	\$7,440,500.00	\$2,157,745.00	\$9,598,245.00
30 -- -- -- --	PLANNING, ENGINEERING, AND DESIGN	1	Job	\$1,136,700.00	\$329,643.00	\$1,466,343.00
31 -- -- -- --	CONSTRUCTION MANAGEMENT (S&A)	1	Job	\$779,368.00	\$226,016.72	\$1,005,384.72
TOTAL PROJECT COST				\$9,563,568.00	\$2,754,804.72	\$12,318,372.72

Prepared by:
 CENAP-ECT-E

5.2 Contingencies, (P, E & D), and Construction Management

5.2.1 Contingencies

The estimated cost for each major subdivision or feature of the recommended project includes an item for "contingencies". The item for "contingencies" is an allowance against some adverse or unanticipated condition not susceptible to exact evaluation from the data at hand but which must be expressed or represented in the cost estimate. The contingency allowances used in the development of the cost estimate for the selected project were estimated as an appropriate percentage using the abbreviated method for preparing risk analysis. A contingency factor of 30% was included in the Mechanical Beach Replenishment section of the estimate and a contingency factor of 29% in the Hydraulic Beach Replenishment section. A provided contingency factor of 20% was included in the Lands and Damages costs.

5.2.2 Preconstruction Engineering & Design (P, E & D)

Preconstruction Engineering and Design costs include local cooperative agreements, environmental and regulatory activities, general design memorandum, preparation of plans and specifications, engineering during construction, A/E liability actions, cost engineering, construction and supply contract award activities, project management, and the development of the PCA. P, E & D costs were estimated as lump sums (including contingency) for the Beach Replenishment. A contingency factor of 30% was included in the P, E & D costs for Mechanical Beach Replenishment and 29% for Hydraulic Beach Replenishment.

5.2.3 Construction Management (S&A)

Construction Management costs include contract administration, review of shop drawings, inspection and quality assurance, project office operation, contractor-initiated claims and litigations, and government-initiated claims and litigations. S&A related costs were estimated as lump sums for the Beach Replenishment. A contingency factor of 30% is included in all S&A costs for Mechanical Beach Replenishment and 29% for Hydraulic Beach Replenishment.

5.3 Construction and Funding Schedule for the NED Plan

The projected construction schedule for the NED plan is provided in Table 5-3. The schedule is based on the timeliness of the report's approval and allocation of funds by OMB, the foregoing construction procedures, and the ability of local interests to implement the necessary items of local cooperation.

Table 5-3 Construction Schedule - Mechanical or Hydraulic Beachfill

	Duration (Mo)	2027				2028			
		Q1	Q2	Q3	Q4	Q1	Q2	Q3	Q4
Initial Construction	8						X	X	X

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