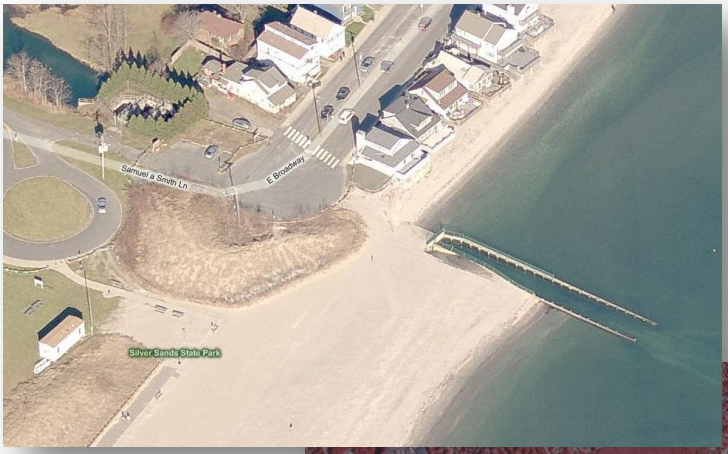
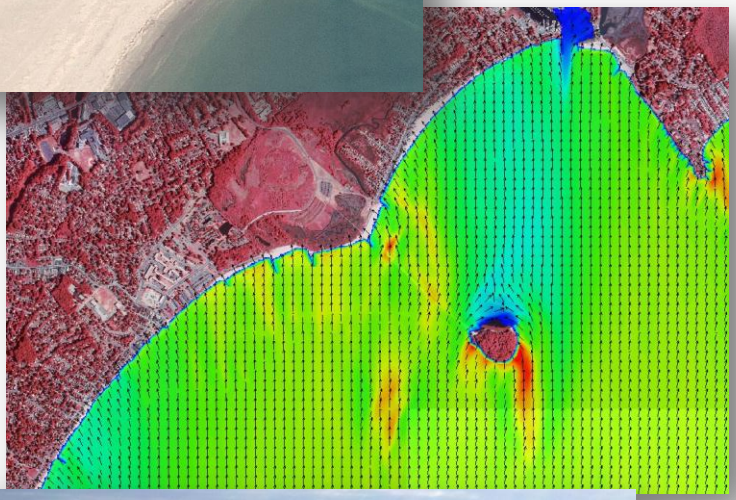
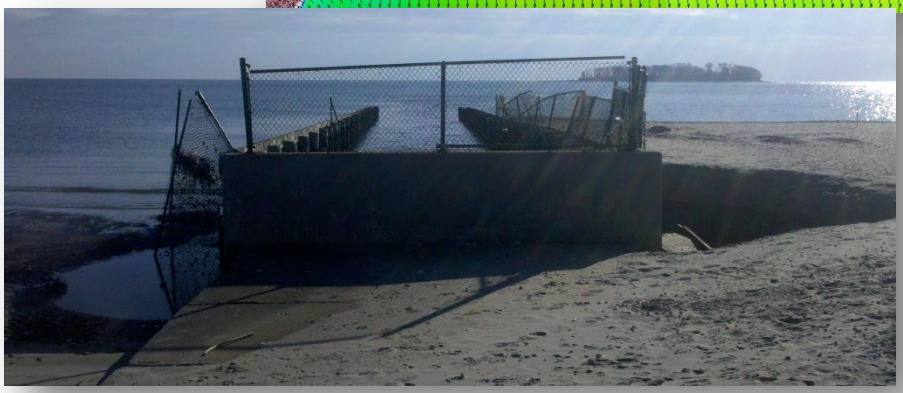
Final Report

Engineering Investigation Services: Coastal Processes Evaluation Great Creek Outlet, Silver Sands State Park







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1.0 INTRODUCTION

This report presents the results of a coastal engineering assessment of Great Creek outlet at Silver Sands State Park in Milford, Connecticut. Specifically, the engineering analysis provides a detailed assessment of site-specific coastal processes, including data observations, wave transformation modeling, and sediment transport modeling, and inlet stability analysis. These results are then used to evaluate and assess potential alternatives for enhancing the Great Creek outlet structure, which is currently in poor condition and requires replacement (Fuss and O'Neill, 2012). Alternatives are evaluated that (1) maintain the ability of the tidal waterway to remain stable (i.e., adequately flush sediment) and (2) optimize the sediment bypassed to the down drift beach, which has experiences erosion partially due to the impact of the outlet structure training walls. Woods Hole Group, Inc. (WHG) performed the analysis on behalf of the Connecticut Department of Energy and Environmental Protection (CT DEEP) under subcontract to Fuss and O'Neill, Inc. (F&O).

Since the integrity of the seaward portion of the Great Creek outlet structure has been significantly compromised and will likely need to be replaced, the CT DEEP realized that there was an opportunity to potentially improve the design of the structure to reduce the impact of the structure on the littoral transport of sediment along the shoreline. Therefore, the primary purpose of this engineering assessment is to determine an improved replacement design for the failing Great Creek outlet structure that maintains the outlet's ability to drain freshwater discharge, flush sediment from the channel to minimize potential shoaling, and provide a robust design that can contend with the dynamic coastal processes and require minimal maintenance. In addition, the replacement outlet structure design is evaluated to optimize the amount of sediment that can naturally bypass the outlet to feed the downdrift eroding beach resource. As a secondary purpose, this engineering evaluation is used to assess supplementary alternatives that could be considered to improve the condition of the downdrift beach, which has experienced historic erosion.

Engineering alternatives were evaluated by determining the site-specific coastal processes (waves, tidal currents, tides, sediment transport) and then determining the impact of the Great Creek outlet structure on sediment transport processes in the vicinity of the structure, as well as on neighboring beaches along The Gulf (e.g., Silver and Fort Trumbull Beach). The feasibility evaluation consisted of historical data analysis, numerical modeling of coastal processes, an alternative analysis, and provides recommendations for a conceptual design based on the physical processes in the vicinity of the outlet structure.

The assessment consisted of evaluating the site-specific environment to develop an understanding of the ongoing coastal processes that shape the coastal region. The numerical modeling component of the study consists of accurately simulating the existing conditions within the vicinity of Silver Beach and subsequently utilizing the modeling results to assess and simulate various alternatives for shoreline protection. The numerical modeling portion of the study ultimately evaluates the performance of selected feasible alternatives. All elements of the project are geared towards determining a technically

feasible, cost-effective, and long-term solution for replacement of the Great Creek outlet structure, while attempting to limit the impact of the structure on the natural littoral processes and downdrift beach.

The report follows a logical step-by-step process that presents the analysis and numerical modeling components, as well as the alternatives considered and evaluated in the engineering investigation. The coastal processes report is organized and divided into the following subsequent chapters:

- Chapter 2.0 provides a brief summary of the geology of the region, recent site observations, and some of the contemporary studies of the region. In essence, Chapter 2.0 sets the backdrop for the study.
- Chapter 3.0 presents the results of the data observations from this current study. This includes water surface elevation data to assess tidal dampening of the culvert, beach sediment samples and grain size analysis, beach profile data, and current data in the outlet.
- Chapter 4.0 presents the results of a site-specific wave transformation modeling effort. The wave transformation modeling is utilized to propagate waves from Long Island Sound to Silver Sands. This chapter presents the development, verification, and results of the transformation modeling effort.
- Chapter 5.0 presents the results of the sediment transport modeling, including sediment transport modeling for both average annual conditions and larger storm events. These results are used to assess the impact of the Great Creek outlet structure on the shoreline and determine viable replacement alternatives for the structure.
- Chapter 6.0 provides the results of an inlet stability analysis for Great Creek outlet. This analysis uses the current and tide observations to assess the scouring ability of the inlet relative to the sediment load.
- Chapter 7.0 briefly summarizes the alternatives considered for the outlet structure replacement, as well as secondary alternatives to assist in reducing beach erosion. Results of the performance of the alternatives relative to inlet stability and sediment bypassing are presented, and a matrix that weighs each alternative against various evaluation criteria (e.g., cost, permit requirements, maintenance, service life, etc.). Specific alternatives are eliminated due to various criteria and recommendations of preferred alternatives are provided.
- Finally, Chapter 8.0 presents a summary and conclusions of the study.

2.0 BACKGROUND GEOLOGY AND HISTORY

A brief overview of the Silver Sands region, the geologic setting, and a summary of some contemporary existing studies is presented in this chapter to provide background. Included in this chapter is:

- A brief regional overview
- A summary of the geomorphology at the site
- A summary of the history of Great Creek and recent site observations, and
- A summary of contemporary studies and existing literature evaluating the Silver Sands region.

2.1 REGIONAL OVERVIEW

The Great Creek Marsh system is located along the Connecticut shoreline within the Town of Milford, Connecticut (Figure 2-1). The beach area is located 3 miles east of the mouth of the Housatonic River in the northwest portion of Long Island Sound. The shoreline in this area is oriented in approximately a southwest/northeast direction and is exposed to the waters of Long Island Sound to the south/southeast. The local water basin, The Gulf, provides access to Milford Harbor and Gulf Pond. The northwest shoreline of The Gulf, comprised of Silver (or Silver Sands) and Fort Trumbull Beach (including the outlet structure), represents a portion of this complex coastal region. The Bar (a naturally forming tombolo landward of Charles Island), Charles Island, and the crenulate-shaped bay extending from The Bar to Welches Point, significantly influence waves, currents, and sediment transport patterns within the region (Figure 2-2). In order to assess the existing conditions, the potential impacts of the existing outlet structure, develop appropriate engineering alternatives to replace the existing failing structure, and mitigate a portion of the downdrift erosion caused by the structure; the physical processes within this complex area must be well understood.

2.2 GEOLOGY OF LONG ISLAND SOUND

The surficial geology of the Silver Sands region is glacial in origin, and has since been reworked by littoral and fluvial processes. Across much of the north shore of Long Island Sound, the geology is consistent with that of this region. The geological framework of region was established during the Wisconsin glaciation (CTDEP, 2008). Regionally, the basement rock is mainly comprised of gneissic metamorphic bedrock (Poppe et. al, 1998). The topographic depression, in what is now the location of the Hammonasset River, was scoured out of the basement rock by the advancing glaciers and melt water streams approximately 24,000 years ago, and the area was blanketed with moraine till upon retreat approximately 15-17,000 years ago (LISRC, 2008; Poppe et. al, 1998). As time passed, sea level continued to rise and Long Island Sound was formed from glacial Lake Connecticut after breaches formed in the terminal moraines that separated the lake from the Atlantic Ocean. Transgressive coastal processes began to rework the glacial till and create a littoral system (LISRC, 2008). As sea level continued to rise, the beach and barrier dune system retreated to the north over the marsh system

that was in place at the Hammonasset River delta. Presently, the surface of the region is covered by modern littoral and marsh deposits, and glacial outwash deposits and recessional moraines.

Figure 2-1. Silver Sands regional overview.

Figure 2-2. Silver Sands project area.

The seafloor geology of Long Island Sound was investigated by a team of scientists from Federal, State, and academic institutions (Poppe et. al, 1998). The comprehensive seafloor mapping investigation involved the use of bathymetry data, side-scan sonar mosaics, and physical sampling with sediment grabs and photography to create interpretative maps of bottom sediment textures and physical environments (Figure 2-3). Immediately offshore Silver Sands, the sediments are classified as sandy, becoming a mixture of silt and sand heading to the east. At Milford Harbor the seafloor texture is primarily sand.

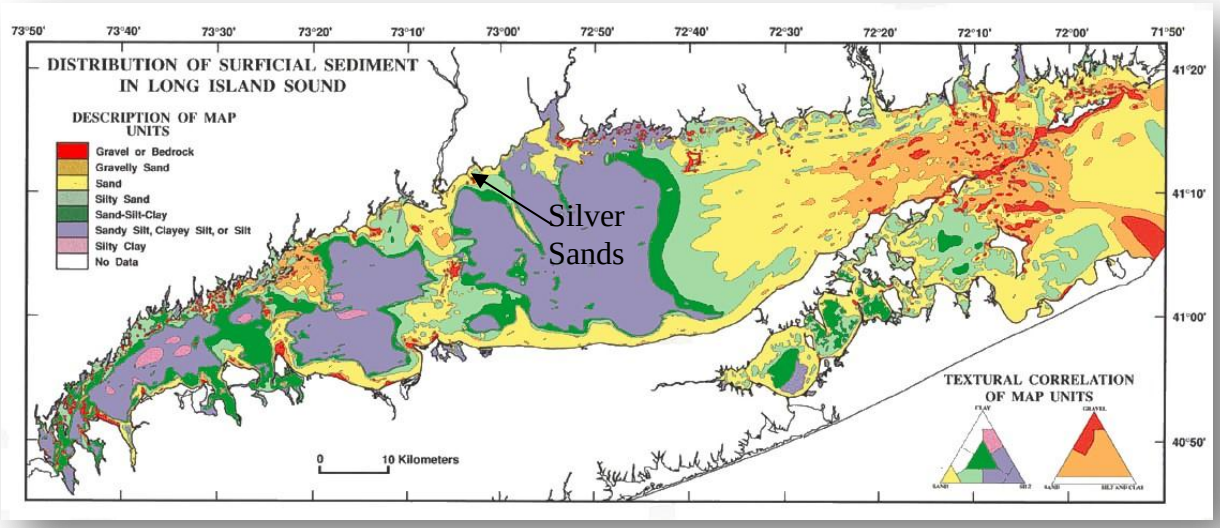

Figure 2-3. Distribution of surficial sediment in Long Island Sound (figure from Poppe et. al, 1998).

2.3 SITE HISTORY AND OBSERVATIONS

The Great Creek Marsh system is a valuable natural resource within the Town of Milford, Connecticut. The original inlet to the marsh, consisting of a low elevation outlet pipe, was located 1,250 feet northeast of the tombolo (The Bar) formed behind Charles Island (Figure 2-4), and approximately 750 feet northeast of the current outlet structure. The original outlet frequently became sediment laden due to sedimentation along this portion of the coastline, thus preventing drainage and causing flooding of streets north of East Broadway (Jacobson, 1981). Therefore, the Connecticut DEEP (or DEP at that time) constructed the current outlet structure as part of a larger restoration project for the region. The present Great Creek Outlet structure is located in the Silver Beach region, located approximately 500 feet northeast of where the tombolo (Bar) reaches the shoreline (Figure 2-5). The structure was constructed in the mid-to-late 1980's and consists of box culvert that is approximately 300 feet in length, extending from Great Creek (west of East Broadway Street) to Silver Beach. Where the culvert enters into The Gulf, two wooden retaining walls were constructed, extending approximately 180 feet into The Gulf, as shown in Figure 2-6. The retaining walls are intended to prevent the

gate vault chamber from filling in with sand, as well as act as impermeable groins and impeded the natural movement of sediment along the shoreline.

Figure 2-4. Location of the original Great Creek outlet. (Aerial photograph taken in 1986 and courtesy of State of Connecticut DEEP).

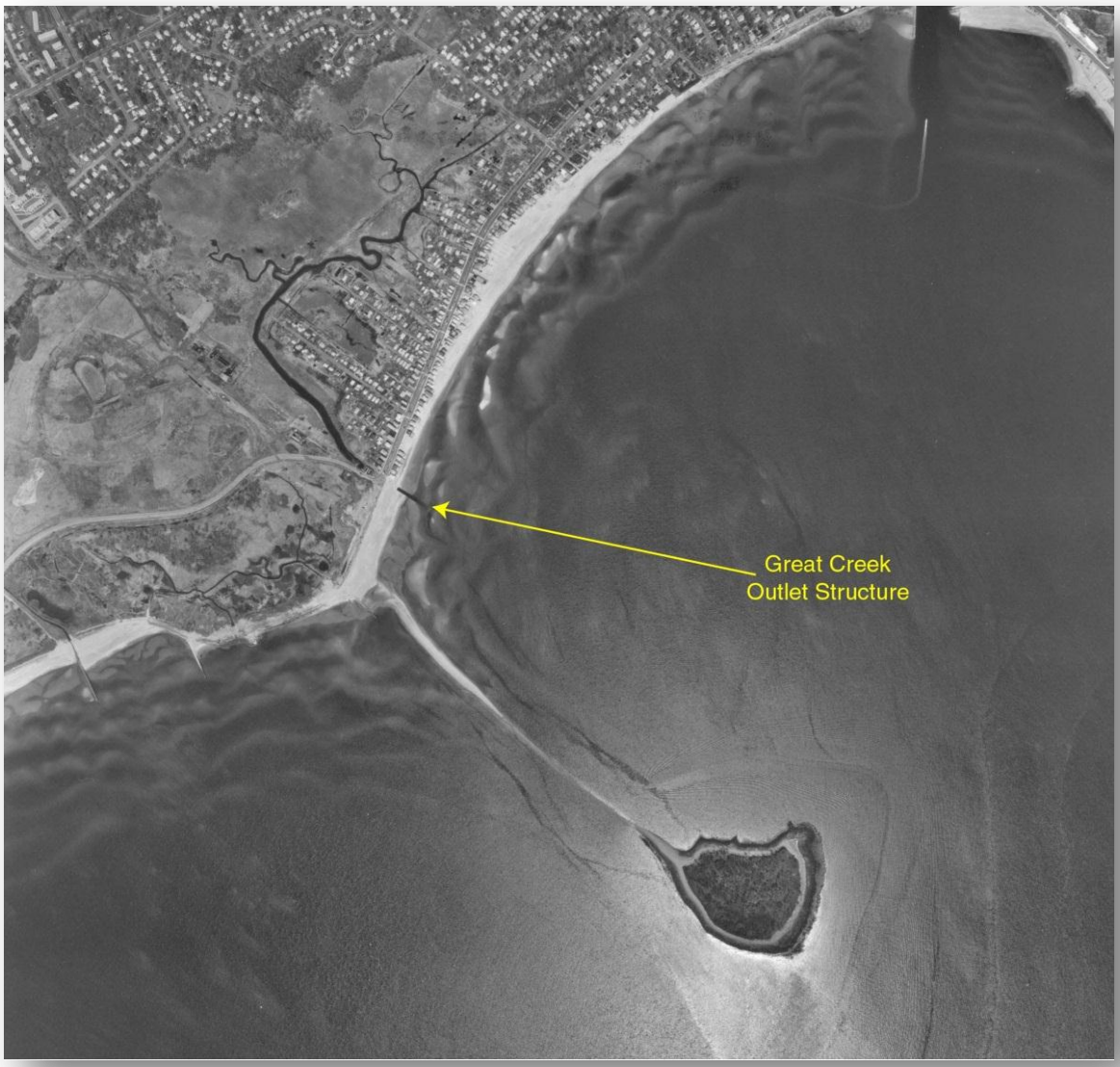

Figure 2-5. Location of the Great Creek Outlet Structure, constructed in the mid-to-late 1980's. Charles Island and the attached tombolo (The Bar) are also shown in the photograph. (Aerial photograph taken in 1995 and courtesy of State of Connecticut DEEP, 1" = 1000').

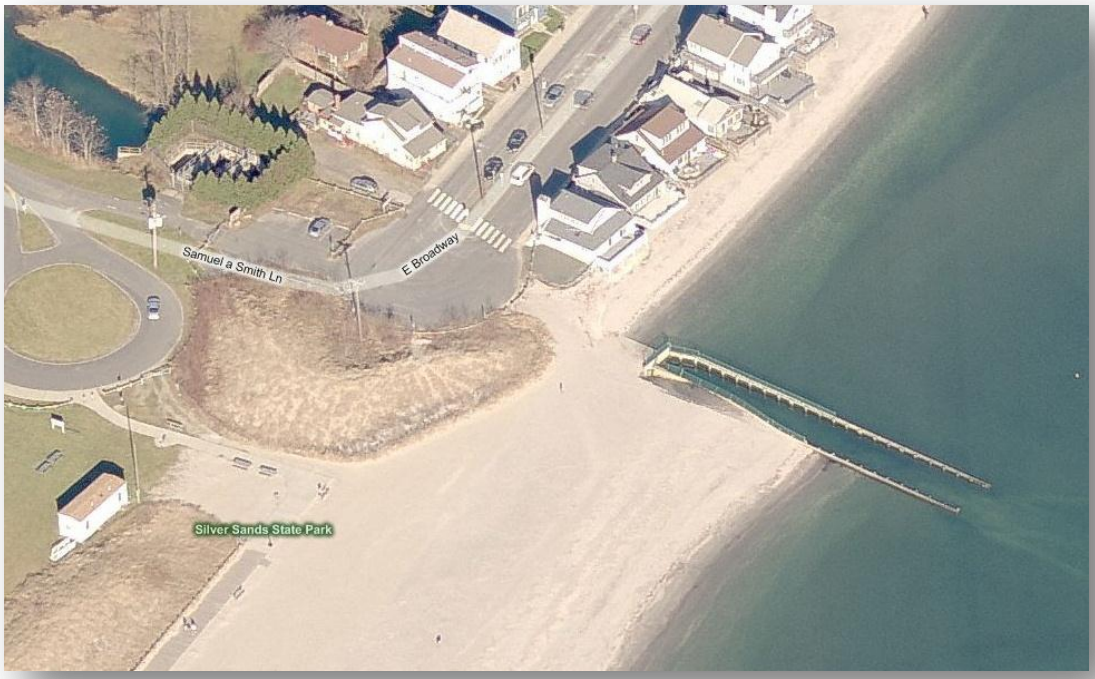

Figure 2-6. Great Creek outlet structure at Silver Sands State Park.

The impact of the retaining walls is apparent in the pre- and post-construction aerials presented in Figures 2-4 and 2-5, respectively. Figure 2-6 indicates a significant updrift fillet southwest of the structure, and significant downdrift erosion to the northeast of the structure. The influence of the structure on sediment transport capability was rapid. Since the construction, the outlet structure has created a shoreline offset of approximately 100 feet. Figure 2-7 shows the significant offset that exists between the updrift (right) and downdrift (left) shorelines in terms of both elevation and cross-shore width. The influence of the structure extends approximately 1000 feet downdrift along Silver Beach (i.e., the shoreline experiences erosion for approximately 1000 feet northeast of the structure). Due to the impedance of sediment transport, the downdrift beach is extremely narrow at high tide, and houses along this stretch of beach are vulnerable to foundation and structural damage during moderate storm events (Figure 2-8). In addition, the structure itself may be susceptible to continued downdrift flanking, resulting in structural failure of the outlet (Figure 2-9). Presently, during high tide, seawater is advancing behind the structure (Figure 2-6). The retaining walls and structural members have also failed on the updrift side of the outlet structure (Figure 2-6), and lack of structural integrity is discussed in detail in F&O (2012).

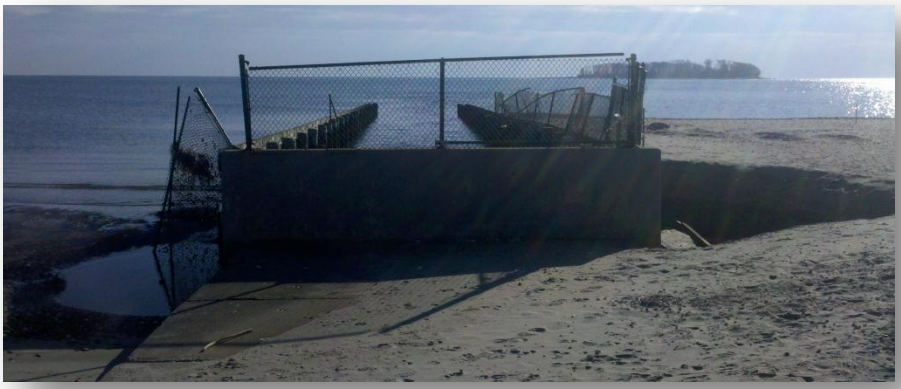

Figure 2-7. Current shoreline offset at the Great Creek Outlet Structure. The left hand side of the photograph shows the updrift (southwest) fillet, while the right-hand side of the photograph shows the downdrift (northeast) erosion. The photo also shows the failure of the training walls on the updrift side.

Figure 2-8. Downdrift erosion at Great Creek Outlet Structure. The influence of the erosion extends approximately 1000 feet along the downdrift shoreline. Mean High Water levels are indicated by the predominant rack line located along the shoreline.

Figure 2-9. View of downdrift retaining wall of the Great Creek Outlet Structure. Continued erosion may leave structure susceptible to flanking and potential failure of the downdrift side of the structure.

2.4 PREVIOUS STUDIES

Although there have been a variety of historical studies related to the Connecticut coastline and Long Island Sound, Woods Hole Group focused this brief summary on several recent studies directly related to the Silver Beach region and the Great Creek outlet structure. In 1981, Nathan L. Jacobson and Associates performed a study investigating shoreline erosion and protection measures along the coast of Milford, from Milford Point to Merwin Point. The report focused on providing a rating of likely beach erosion zones within the project area. Jacobson (1981) found that the Silver Sands Beach region has maintained its general shape since the early 1800's and was given the lowest rate of erosion potential. However, this assessment was prior to the relocation of the Great Creek outlet and construction of the outlet structure in the mid-to-late 1980s, which has resulted in significant retreat of the shoreline northeast of the outlet.

In 1997, Cascio Bechir Engineers (CBE) performed an engineering study to examine alternative shore protection strategies to mitigate the impact of the Great Creek outlet structure. The study suggested three potential alternatives focused on modification of the retaining walls bordering the outlet. The recommended solution was to extend the box culvert an additional 90 feet into the Gulf, while removing and reconstructing the retaining walls with lower profile, shorter (50 feet shorter) structures. Components of these recommended alternatives were evaluated within this report and are presented in more detail in Chapter 7.0.

Woods Hole Group (2001) also provided a preliminary assessment of potential mitigation options for the Great Creek outlet structure, prior to the initial failure of the updrift training wall. The focus of the analysis was on actions that could be taken to improve the condition of the downdrift beaches. Potential alternatives considered included removal of the retaining walls, reducing the height of the retaining walls, physical bypassing of sand, retaining wall shortening, culvert extension, CBEs recommendation, and an elevated sluiceway. These alternatives were initially evaluated in the assessment from a qualitative basis and narrowed the potential alternatives. The results of the 2001 Woods Hole Group report included:

- Evaluation of equilibrium shoreline position that indicated the qualitative erosion and accretion zones along the shoreline. The analysis indicated the shoreline will continue to erode just downdrift of the structure and accretion will continue approximately 1000-1250 feet to the northeast of the structure.

- Evaluation of the equilibrium beach profile that indicated the excessive height, and length, of the existing retaining walls, which inhibit sediment movement in the alongshore direction. The updrift beach is generally flatter than equilibrium due to the sand accumulation caused by the structure, whereas the downdrift beach is over steepened due to the starvation of sediment caused by the structure.

- Preliminary (without data) inlet stability analysis provided a general sense that the inlet flux alone would not be able to flush the sediment out the inlet on a consistent basis. This was demonstrated by the shoaling of the old inlet, and the subsequent need for the outlet structure. As such, complete removal of the training walls was not recommended.

- Determination of alongshore currents that during a high-tide scenario produce maximum currents within the surf zone between 20-50 feet offshore. Due to the flat beach, the mid-tide scenario produces maximum current further offshore (approximately 100 feet).

- Preliminary determination of the cross-shore distribution of alongshore sediment transport indicated the location of the maximum sediment transport rate is variable based on wave direction approach. The zone of maximum transport seems to lie between 25 to 100 feet offshore, which would correspond to the location of the existing retaining wall structures. For individual wave approach directions predicted, rates are dominated by the less frequent, but highly indirect wave approach directions, thereby skewing the net transport distribution. There was not a high level of confidence in the results because the simplistic wave and sediment transport theory was not valid to characterize these processes at this complex site. In addition, the influence of the Bar on sediment transport patterns was imprecisely accounted for.

- A preliminary evaluation of alternatives that indicated if manual bypassing was selected as the preferred option, no further analysis was required. For more

complex recommended solutions and changes to the outlet structure, a more detailed numerical modeling study was recommended to provide quantitative design parameters.

Therefore, it was recommended that a more detailed alternatives analysis be conducted, along with a more quantitative understanding of the site-specific coastal processes, using numerical modeling to assess potential solutions for structural modifications to the outlet structure.

GEO/Plan Associates (2007) performed a rudimentary analysis of coastal process the Silver Sands and Walnut Beach region in conjunction with a boardwalk project at the park. The assessment only evaluated shorelines to the west of the Great Creek outlet structure. Using primarily historical studies and aerial photography, GEO/PLAN indicated that Silver Sands eroding landward at a historical rate of more than 1 ft/yr, and that the erosion dominated by infrequent storm events. The study also indicated that future more detailed studies, including interpretation of historical aerial photos, should be considered to understand coastal processes.

3.0 DATA OBSERVATIONS

3.1 BEACH PROFILE DATA

Fuss and O'Neill conducted two topographic surveys along the beach at Silver Sands (a set in June 2001 and a set in January 2011). The profiles were conducted along fixed transects so that certain locations could be reoccupied and monitored for change. Figure 3-1 shows the location of the profiles collected in both 2001 and 2011 at Silver Sands. Additional data were collected in both 2001 and 2011; however, only the profiles shown in Figure 3-1 were are the profiles that were observed in both 2001 and 2011. The most recent profile data (2011), including all point data collected by F&O (2012), were used in the development of the numerical models of the site (as detailed in Chapters 4 and 5).

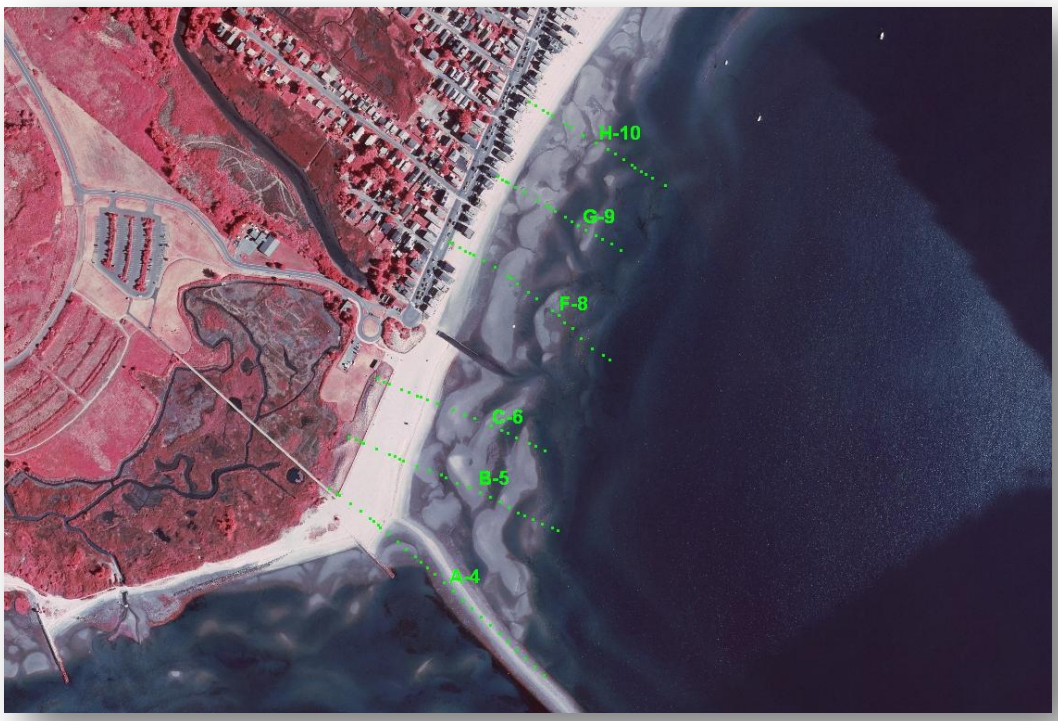

Figure 3-1. Profile locations and nomenclature in the vicinity of the Great Creek outlet structure at Silver Sands.

Figure 3-2 presents a visual comparison of three profiles (A-4, B-5, and C-6) south of the outlet structure measured in 2001 (blue line) and 2011 (red line). The vertical axis presents the elevation (in feet relative to the NGVD29 vertical datum), while the horizontal axis shows the cross-shore distance in feet. No baseline was established in the surveying effort; however, all surveyed data points were geo-referenced that were collected along each transect line. As such, the data are plotted relative to the approximate water line to provide a cross-shore reference distance, but do not necessarily cover the same distance from a landward baseline location. For example, the 2011 survey lines stop at the top of the beach foreshore slope, while the 2001 profiles included

the upper portion of the beach and dune system. For the design phase, a fixed baseline and stations will be established. The elevation of Mean High Water (MHW) and Mean Low Water (MLW) are also presented as the broken horizontal lines in each plot panel. The data indicate that the southern profiles remain relatively consistent between 2001 and 2011. The intertidal area generally shows minor changes, while the foreshore beach slope (between mean tide level and MHW) shows accretion of the shoreline of approximately 1.5-2.4 feet/year at the MHW elevation, while at the zero NGVD29 elevation the profiles have shown insignificant changes.

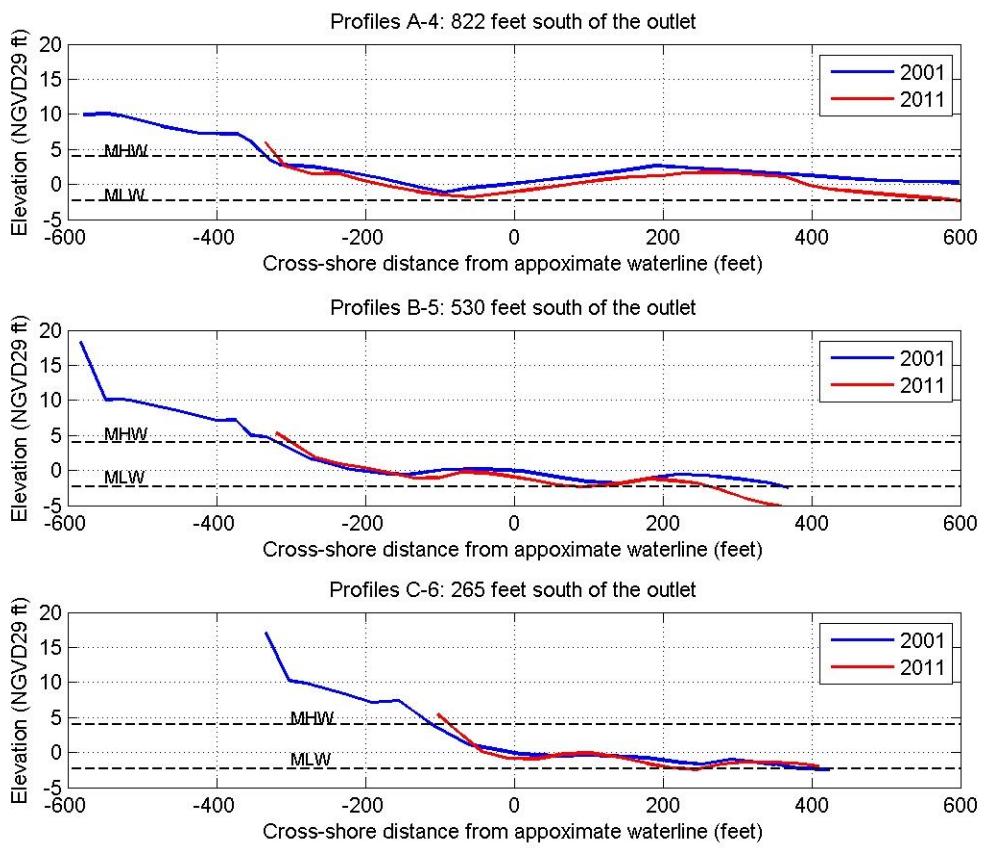

Figure 3-2. Profile comparison south of the Great Creek outlet structure at Silver Sands.

Figure 3-3 presents a visual comparison of three profiles (F-8, G-9, and H-10) north of the outlet structure measured in 2001 (blue line) and 2011 (red line). The vertical axis presents the elevation (in feet relative to the NGVD29 vertical datum), while the horizontal axis shows the cross-shore distance in feet. No baseline was established in the surveying effort; however, all surveyed data points were geo-referenced that were collected along each transect line. As such, the data are plotted relative to the approximate water line to provide a cross-shore reference distance, but do not necessarily cover the same distance from a landward baseline location. For example, the 2011 survey lines stop at the top of the beach foreshore slope, while the 2001 profiles included the upper portion of the beach and dune system. The elevation of Mean High Water (MHW) and Mean Low Water (MLW) are also presented as the broken horizontal lines

in each plot panel. Despite the significant historical erosion that has occurred (approximately 100 feet of shoreline between the mid-1980s and early 2000), the more contemporary time frame (2001-2011) shows a slower rate of erosion. Figure 3-3 indicates that the intertidal profile generally remains consistent with some movement of the offshore sandbar at the seaward most extent of the profile (below MLW). There is net erosion in the profiles north of the structure, as the shoreline has eroded at a rate of approximately 2.0 feet/year at the elevation of 0 NDVG29, while at the MHW elevation the profile has shown minimal change.

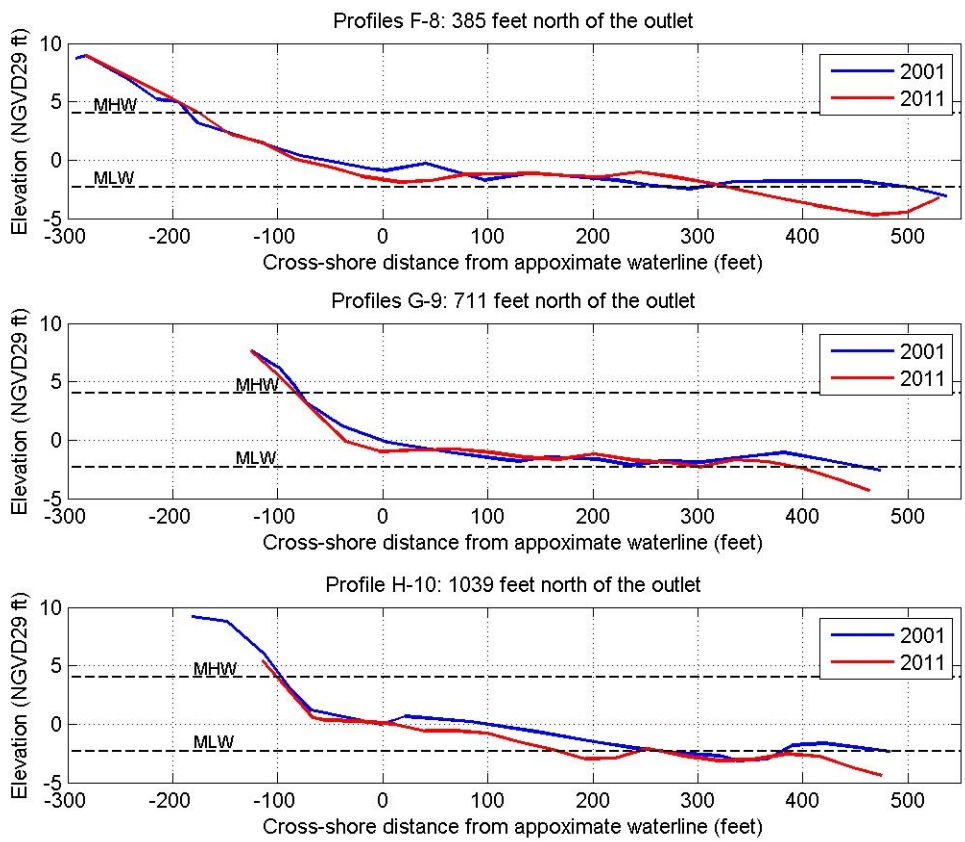

Figure 3-3. Profile comparison north of the Great Creek outlet structure at Silver Sands.

Figure 3-4 presents results from the 2001 survey at two profiles surveyed directly adjacent to the Great Creek outlet structure. The upper panel presents the data collected on the north side of the structure, while the lower panel presents the data collected on the south side of the structure. Again, the vertical axis presents the elevation (in feet relative to the NGVD29 vertical datum), while the horizontal axis shows the cross-shore distance in feet. No baseline was established in the surveying effort; however, all surveyed data points were geo-referenced that were collected along each transect line. As such, the data are plotted relative to the approximate water line to provide a cross-shore reference distance, but do not necessarily cover the same distance from a landward baseline location. The elevation of Mean High Water (MHW) and Mean Low Water (MLW) are also presented as the broken horizontal lines in each plot panel. In addition, a

representation of the training walls crest elevation in the cross-shore direction is also shown in each panel (red line). It is apparent from a comparison of the profiles that there is an accumulation of sand on the south side of the erosion and a deficit of sand on the north side of the structure, specifically in the upper portion of the intertidal profile (i.e., between NGVD29 and MHW elevations). All the most recent survey data provided by F&O (2012) was utilized in the development of the nearshore portion of the modeling domains, as detailed in section 4.3.

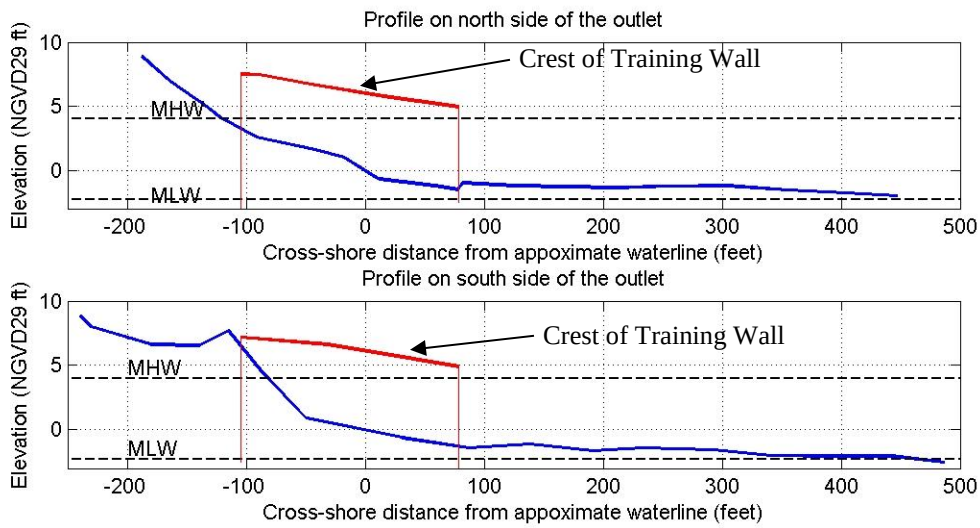

Figure 3-4. Profiles at the Great Creek outlet structure (blue) and crest of the training wall (dark red line).

3.2 WATER SURFACE ELEVATION OBSERVATIONS

In order to better understand the local hydrodynamic regime, as well as the influence of the outlet structure, tide gates, culvert, and associated anthropogenic infrastructure on the tidal exchange with the marsh system, local tidal data were collected. These data not only help understand the hydraulics of the system, but also were used to evaluate potential alternatives (Chapter 7.0). The data were observed upstream and downstream of the Great Creek outlet structure, as shown in Figure 3-5.

The instruments deployed included two In-Situ Aqua Troll 200 series gauges that recorded temperature, conductivity, and pressure. The gauges were deployed from June 11 to July 12, 2012. Prior to deployment, the instruments were synchronized with a universal atomic clock and programmed to autonomously record a time-stamped data point every 6 minutes. The first gauge was deployed roughly 100 feet upstream of the Great Creek outlet structure at a location in the main channel that ensured the gauge remained submerged during the entire deployment period (Figure 3-6). This gauge was deployed on a pipe anchor secured to the channel bed. The second gauge was deployed at the Milford Yacht club in Milford Harbor. The gauge was attached to steel sheeting of the pier bulkhead (Figure 3-7) at a depth that also remained submerged throughout the entire tide cycle. Upon deployment, the elevations of the tide gauges were surveyed to a local datum by Fuss & O'Neill. This allows the depths measured by the instruments to be translated to water surface elevation measurements.

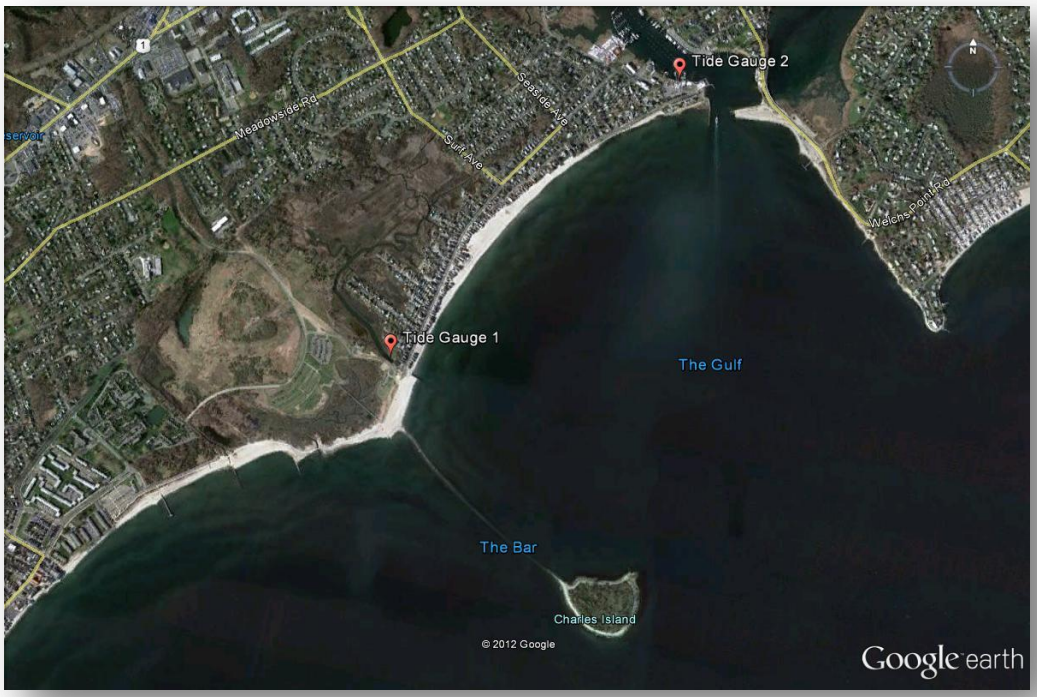

Figure 3-5. Tide gauge deployment locations at Silver Sands.

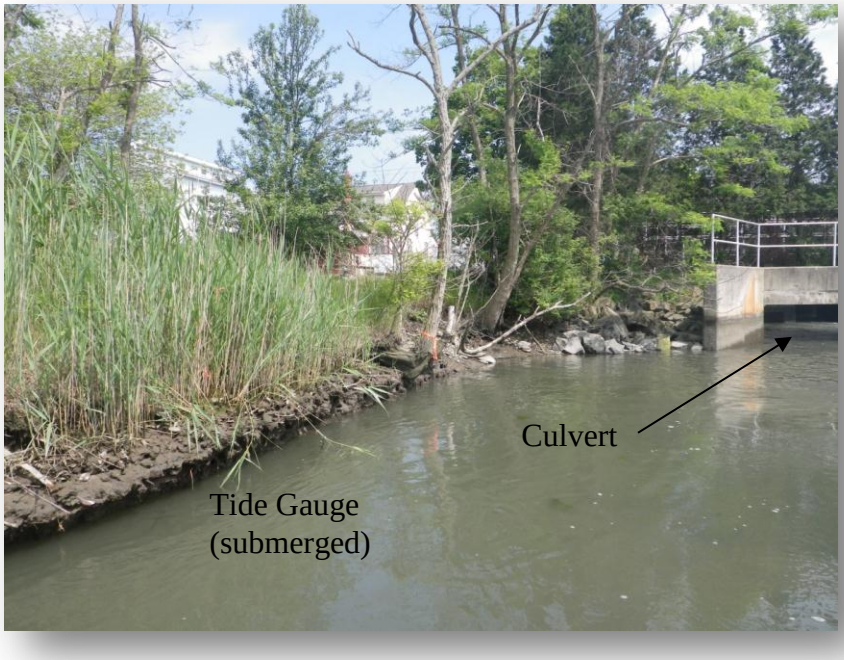

Figure 3-6. Tide gauge location upstream of the Great Creek culvert and tide gate system.

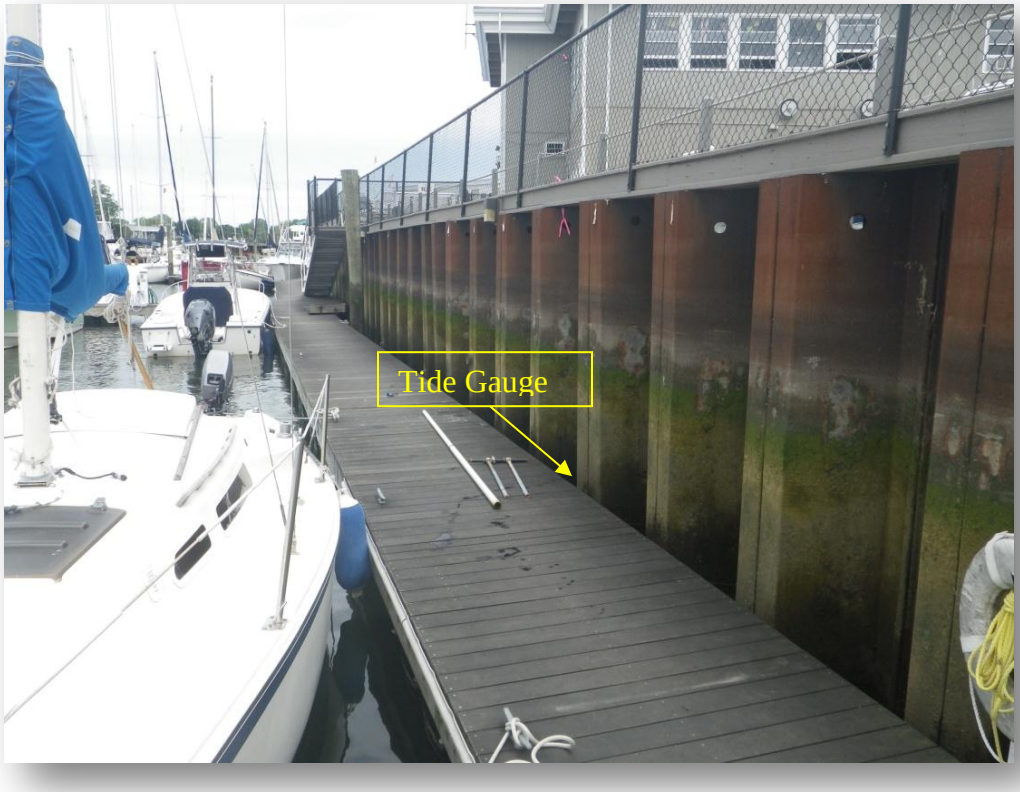

Figure 3-7. Tide Gauge location at Milford Yacht Club.

After the tide gauges were recovered, the data were processed to account for changes in atmospheric pressure during deployment and to adjust the data for survey elevations. The tide gauges deployed for this study recorded conductivity, temperature and pressure during their deployment. Because they are non-vented instruments, they record the absolute pressure; a combination of the atmospheric pressure and the gauge pressure (i.e., the pressure from the weight of water above the instrument). Changes in atmospheric pressure during deployment may lead to significant error in the computed depth if the atmospheric pressure is not accounted for; this is particularly true in shallow water. Atmospheric pressure records from the Sirkorsky Memorial Airport were used to correct the pressure data. Depth of water may be determined from the density and corrected pressure data. However, the density of water depends on both the temperature and salinity. Using methods from Fofonoff et al. (1983), salinity was determined from the conductivity and temperature, and density was calculated from the salinity and temperature. Then assuming the gauge pressure is hydrostatic, the depth was determined from the density and pressure. Finally, depth is translated to water surface elevation using the surveyed tide gauge elevations. Figure 3-8 shows the time series of water surface elevation at Milford Harbor (blue line) and at the Great Creek marsh (green line) for the deployment period. The vertical axis indicates the elevation of the water in feet NGVD29, while the horizontal axis shows the date in 2012. Figure 3-9 presents a shorter time period of the water surface elevation data (approximately July 3 through July 6, 2012 during a spring tide) in order to assess some details of the tidal signal.

Table 3-1 presents some basic tidal statistics from the 30 day observation of tides at Great Creek and in Long Island Sound (Milford Harbor). The mean water surface elevation in the marsh is approximately a foot lower than in Long Island Sound, primarily due to the tide gates in the outlet structure, which cut off the high tide in the system. The tide range is attenuated by 55% as it propagates through the inlet and control structures, and the water level in the marsh does not drain as far as possible due to the somewhat restrictive nature of the culvert size, inverts, and opening.

Table 3-1. Tide datum calculations for Great Creek marsh and Long Island Sound.

Tidal Benchmark	Sound (ft, NGVD29)	Marsh (ft, NGVD29)
Mean Tide Level	2.44	1.41
Mean Tide Range	6.67	3.03
Mean High Water	5.78	2.92
Mean Low Water	-0.89	-0.11

Figure 3-9 illustrates the influence of the outlet structure, and associated tide gates, on the tidal exchange. During the flood tide, the water surface elevation in the marsh increases at a much slower rate than in Long Island Sound. The tide gates close when the water surface elevation reaches approximately 2.5 to 2.7 feet, and the exact gate closure varies with each specific tide. While the gates remain closed, the water surface elevation continues to rise slowly in the marsh due to freshwater input into the system. During rainfall events this could lead to some short-term impoundment and upland flooding. Once the water surface elevation in Long Island Sound drops below the target gate closure elevation, the tide gates re-open and the marsh begins to drain.

Figure 3-9 also shows the water surface elevation associated with MLW in Long Island Sound (broken black line) and the approximate elevation of the lowest road in the developed marsh area (solid black line). Using these two benchmarks, the data indicate that there may be the potential to increase the tidal range and prism by approximately 35% by redesigning the entrance to the system (e.g., larger culvert, modified inverts, etc.) and maintain flood control ability. These data and evaluation will be considered and discussed further in the alternative evaluation process presented in Chapter 7.0 and are also used in the wave transformation modeling (Chapter 4), sediment transport modeling (Chapter 5), and inlet stability analysis (Chapter 6).

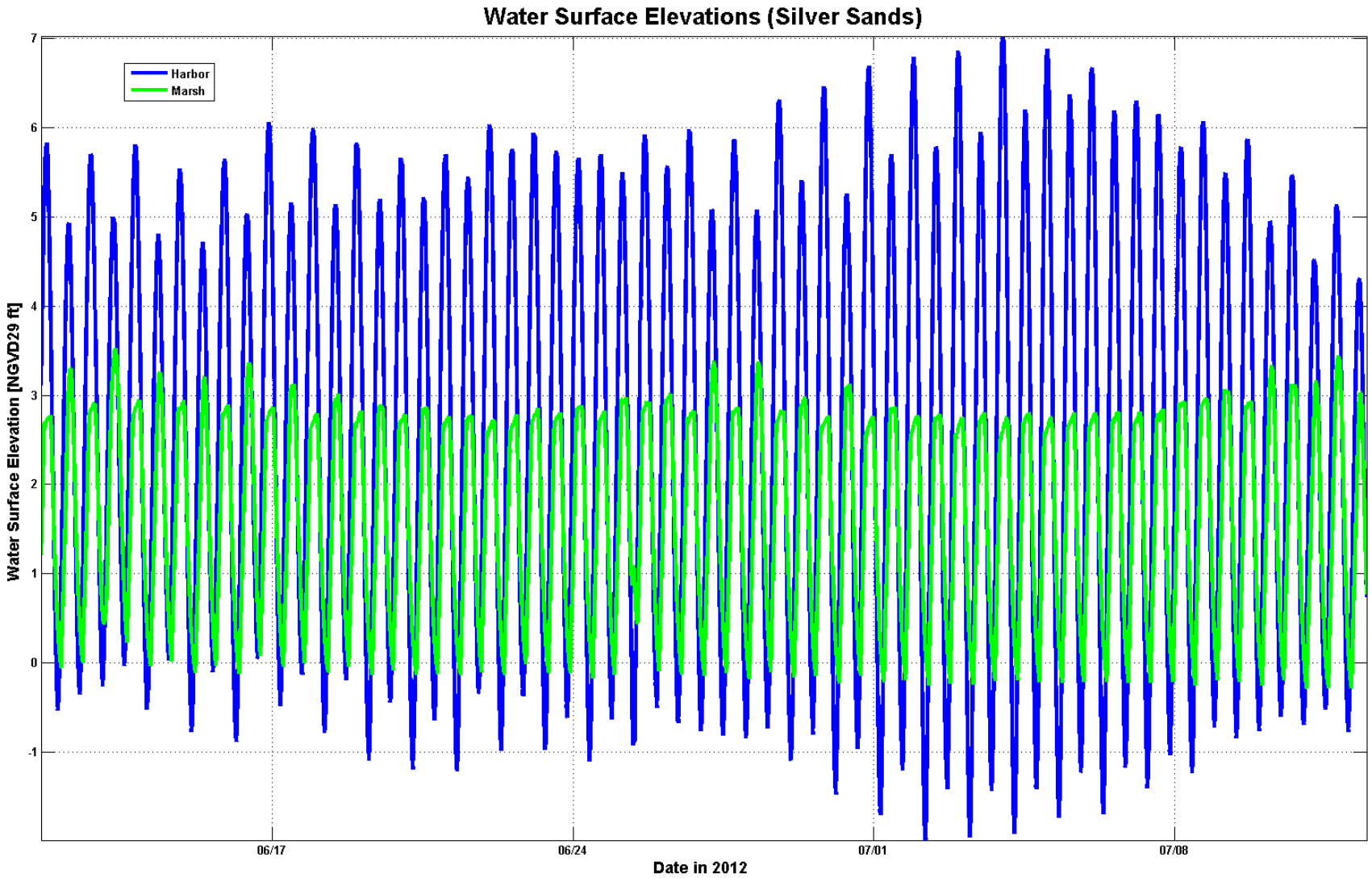

Figure 3-8. Water surface elevation comparison in the Great Creek marsh (green) and Milford Harbor (blue).

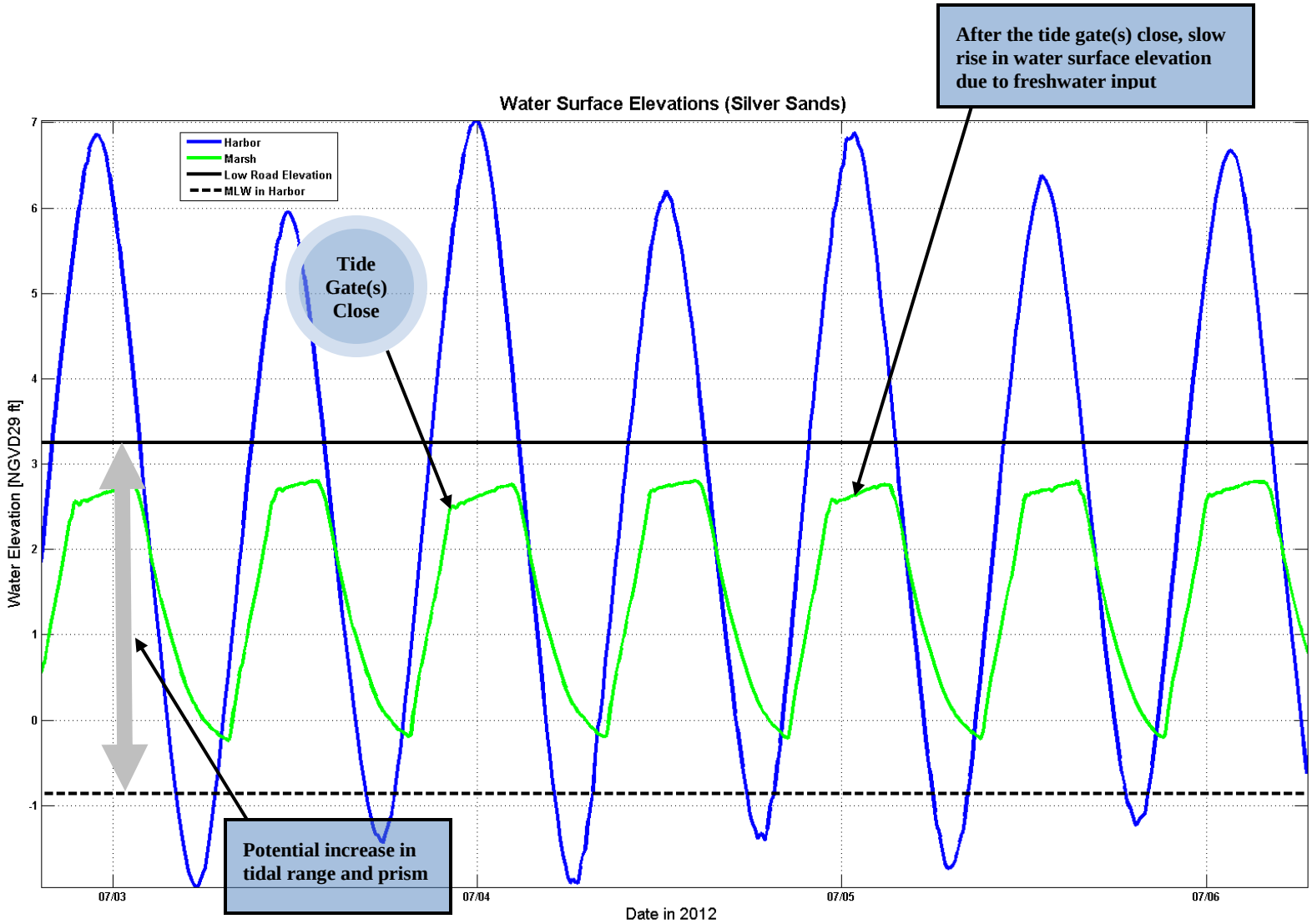

Figure 3-9. Water surface elevation at Silver Sands in the Great Creek marsh (green) and Milford Harbor (blue) between July 3 and July 6, 2012.

3.3 NATIVE GRAIN SIZE

Surface sediment samples were collected from six (6) locations at the Silver Sands project site as shown in Figure 3-10. Exact coordinates of the samples are presented in Table 3-2. One sample, SS-CN, was collected from the area between the training walls at the Great Creek outlet structure. Two additional samples were collected on either side of the structure: SS-NW was collected on the beach updrift of the structure while SS-NE was collected on the eroding downdrift portion of the beach. Another set of three (3) samples were collected just offshore of the Great Creek outlet structure.

Figure 3-10. Grain size sample locations.

A grain size analysis was performed on each of the six samples. Figure 3-11 presents the grain size distribution of the sample collected at SS-NW and indicates that over 95% of the sample is comprised of sand. The analysis of the samples collected on the beach were comprised of primarily medium to fine grain sands, while the samples collected offshore of the structure were slightly coarser with approximately 10% cobbles and gravel. Complete grain size results for all samples are provided in Appendix A. Results from the grain size analysis were used in the sediment transport modeling detailed in Chapter 5.0.

Table 3-2. Coordinates of sediment samples.

Sediment Sample ID	Latitude	Longitude
SS-NW	N 41° 11.997	W 73° 03.894
SS-NE	N 41° 12.031	W 73° 03.877
SS-SE	N 41° 12.015	W 73° 03.795
SS-SW	N 41° 11.956	W 73° 03.844
SS-CN	N 41° 12.005	W 73° 03.861
SS-CS	N 41° 11.983	W 73° 03.813

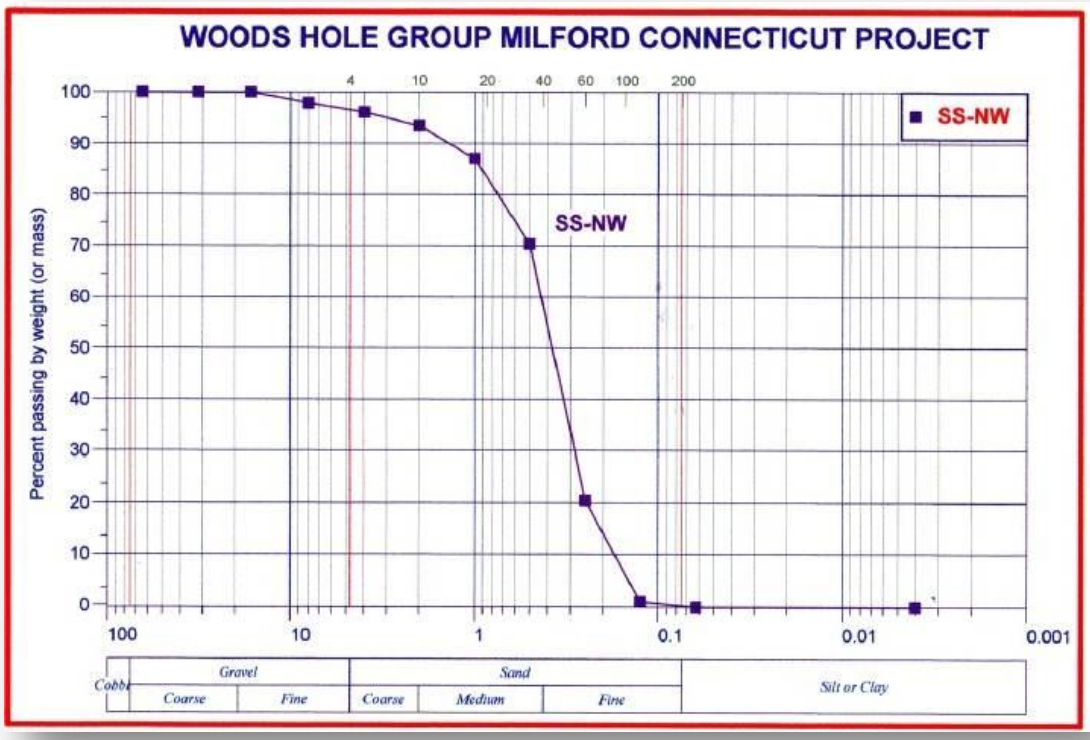

Figure 3-11. Grain size analysis for sample SS-NW collected on the beach updrift of the Great Creek outlet structure.

3.4 OUTLET STRUCTURE CURRENTS AND FLUX

Data observation of tidal velocities and flux within the outlet structure were collected between January 6 and February 1, 2012. Two Sigma Hach 910 instruments were deployed in the outlet structure at the same location for redundancy in case an instrument was compromised (Figure 3-12). Data were collected at 6 minute intervals and measured the velocity (feet/second) and flow rate (cubic feet per second) through the culvert. There was with a 100% data return for both instruments. Figure 3-13 shows a time series of the flow rate (cfs) for instrument A (upper panel) and instrument B (lower panel). The data collected from both instruments were essentially the same (within the error levels of

the instrument itself for more than 99% of the observations). Peak tidal currents in the culvert were approximately 2 feet/second on the flood tide and approximately 5 feet/second on the ebb tide. These data were used extensively in the inlet stability analysis (Chapter 6), for assessing the amount of sediment that could bypass the structure (Chapter 7), and in the alternatives analysis (Chapter 7).

Figure 3-12. The pair of Hach Sigma 910s deployed to measure velocity and flow rate in the Great Creek outlet structure.

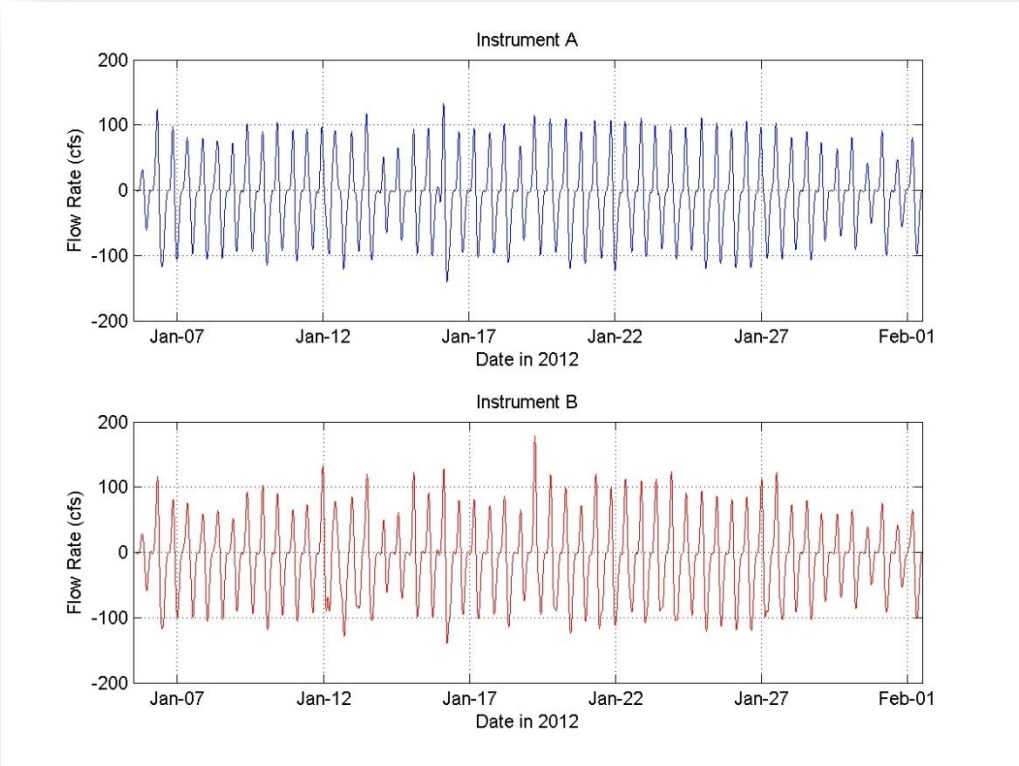

Figure 3-13. Time series results of flow rate (cfs) observed in the Great Creek outlet structure. Data from instrument A is shown in the upper panel and data from instrument B is shown in the lower panel.

4.0 WAVE TRANSFORMATION MODELING

In order to evaluate local sediment transport pathways, as well as assess impacts of, and identify potential alternatives to improve the outlet structure at Great Creek, an understanding of the regional wave climate is required. Before an effective solution can be determined, wave modeling is required to simulate the transformation of waves (e.g., refraction, diffraction, shoaling and breaking) at the regional and site-specific level. Wave modeling allows for quantitative predictions of these processes.

The wave transformation modeling for the Silver Sands area does include some specific limitations that may reduce the accuracy of the modeling. These include:

- Limited recent, local bathymetric data - Typically, a bathymetric survey of the local, nearshore region would be conducted to improve model results and increase accuracy; however, a bathymetric survey was not conducted specifically for this project.

- Lack of local nearshore wave data – Although recent regional and long-term Long Island Sound wave data are available and were used in this study. Woods Hole Group also typically collects nearshore wave data for validation of the wave transformation modeling to improve accuracy.

In most cases, Woods Hole Group includes these scope items in the wave transformation modeling to improve model results and accuracy. However, for the conceptual design level of analysis completed for Silver Sands, these limitations likely do not significantly impact the results of the study, the alternative assessment, or the recommendations presented throughout this document.

A detailed description of the procedures used to compute the wave conditions near Silver Sands is presented in this chapter. This includes:

- the analysis approach and description of the wave model applied to simulate waves at this scale,
- the generation of the bathymetric grid,
- the development of all the wave input conditions, and;
- a discussion of the transformation-scale wave modeling results.

4.1 ANALYSIS APPROACH

A quantitative understanding of wave characteristics is key to the evaluation of nearshore coastal processes and sediment transport. Wave energy is comprised of a large variety of waves moving in different directions and with different frequencies, phases, and heights. These waves undergo significant modifications as they advance into the coastal region, interact with the sea floor, and eventually reach land. The ocean climate also changes temporally with seasonal modulations. The variability in offshore wave climate, the

transformations occurring as waves propagate landward, and the temporal modulations, all result in significant fluctuations in the quantity and direction of sediment transport in the coastal zone. Therefore, in many cases, using a single representative wave height, frequency, and/or direction is not the most accurate technique for assessing the wave climate and, subsequently the sediment transport at the coastline.

This section evaluates the wave climate offshore of Silver Sands and the transformations waves experience as they propagate towards the coastline. To quantify the wave impact along the shoreline, site-specific wave conditions were determined using wind data, wave data, and a numerical wave transformation model. Wave transformation models provide predictive tools for evaluating various forces governing wave climate and sediment transport processes. For example, wave refraction and diffraction may have a significant effect on the impacts waves have on a shoreline. Wave refraction and diffraction generally result in an uneven distribution of wave energy along the coast that affects sediment transport in the region. Wave modeling results provide information on wave propagation to the shoreline, revealing areas of increased erosion (i.e., hot spots) or areas of increased energy. The refraction and diffraction mechanisms also result in changes in the offshore wave direction that may significantly influence the rate and direction of sand movement. Therefore, the quantitative information provided from the numerical model can be used to explain the physical processes that dominate a region and to furnish appropriate recommendations/solutions for each stretch of coast.

4.2 WAVE MODEL DESCRIPTION

A spectral wave model was used to propagate random waves from an offshore location to the nearshore region and to investigate potential changes to the wave field caused by the bathymetry. Both the offshore data and the nearshore wave transformation results indicate the potential limitations of representing all waves using a single set of wave parameters (one wave height, one frequency, and one direction).

The spectral wave model STWAVE version 4.0 (Smith, Sherlock, and Resio, 2001), developed by the U.S. Army Corps of Engineers Waterways Experiment Station, was employed to evaluate changes in wave propagation across the nearshore region fronting Silver Sands beach. STWAVE is regularly used and widely accepted in coastal design and studies. STWAVE is a steady state, spectral wave transformation model, based on a form of the wave action balance equation of Jonsson (1990):

$$(C_{ga})_i \frac{\partial}{\partial x_i} \frac{C_a C_{ga} \cos(\mu - \alpha) E(\omega_a, \alpha)}{\omega_r} = \sum \frac{S}{\omega_r} \tag{4-1}$$

where

i = x,y spatial coordinates
C_a = absolute wave celerity
C_{ga} = absolute wave group celerity
μ = current direction

α = propagation direction of spectral component
E = spectral energy density
ω_r = relative angular frequency (frequency relative to current)
S = energy source/sink terms

Source and sink terms include wind input, non-linear wave-wave interactions, dissipation in the wind field, and surf-zone breaking. The model simulates wave refraction and shoaling induced by changes in bathymetry and by wave interactions with currents. The model also includes wave breaking, wave growth, and influences of wave white capping on the distribution and dissipation of energy in the wave spectrum. Model outputs include zero-moment wave height, peak wave period, and mean wave direction at all grid points and two-dimensional spectra at selected grid points.

STWAVE simulates the behavior of a random sea surface by describing wave energy density as a function of direction (directional spectrum) and frequency (frequency spectrum). The two-dimensional wave spectrum is discretized into separate wave components, which constitute an essential part of the input for STWAVE. Through a combination of the various wave directions and frequencies, STWAVE is able to simulate the behavior of a natural, random sea. In addition, detailed analysis and selection of input spectra allows the model to assess the impact of different seasonal conditions, varying wave approach pathways, and storms. By simulating numerous wave components that propagate towards the Silver Sands shoreline, a spectral wave model is superior to a monochromatic wave model, which would include only one specific wave. A comprehensive discussion of the theoretical background of STWAVE can be found in Smith, Sherlock, and Resio (2001).

The STWAVE model also allows for grid nesting (Smith and Smith, 2002). Grid nesting involves using multiple grids to transform waves from an offshore location to nearshore and coastal regions. A coarse (lower-resolution) offshore STWAVE grid is used to transform the waves to the boundary of a nearshore STWAVE grid with a higher spatial resolution. The nearshore grid is considered the “nested” grid. The output wave spectra from the coarse grid are saved at several locations and interpolated onto the nearshore grid boundary. Grid nesting is a useful technique for larger regional applications where a coarse grid is sufficient offshore while complex bathymetry and current fields in the nearshore require a finer resolution grid to give a more accurate simulation of the wave field.

Using wave data observed offshore of the Silver Sands region, appropriate offshore wave conditions were developed and used as input data to specify the wave boundary conditions for the STWAVE model (discussed in Section 4.4). Then, using the best available local bathymetry, a numerical grid was created to propagate waves to the Silver Sands region at the coastline.

4.3 BATHYMETRY AND GRID GENERATION

Bathymetry is the underwater equivalent to topography, and is the measure of the ocean (or water body) depths. In order to determine the wave conditions that impact the shoreline at Silver Sands, the bathymetric features offshore of the beach need to be defined. As such, a significant amount of bathymetric information is required in order to accurately predict wave transformations offshore a coastal environment. Existing data sources were used extensively for the nearshore region of Silver Sands.

Existing bathymetric and topographic data were acquired from several sources including:

- National Oceanographic and Atmospheric Administration (NOAA) Hydrographic Survey Data,
- United States Army Corps Engineers (USACE) Hydrographic Survey data,
- University of Connecticut (UCONN) Center for Land Use Education and Research (CLEAR) Light Detection and Ranging (LIDAR) data, and;
- Topographic survey data collected by Fuss and O'Neill in 2011.

National Oceanographic and Atmospheric Administration (NOAA) data were obtained from the National Ocean Service (NOS) Office of Coast Survey Hydrographic Survey Geophysical Data System (GEODAS). The GEODAS data can readily be obtained online at http://www.ngdc.noaa.gov/mgg/bathymetry/hydro.html. The bathymetric survey performed in 2001 from Oyster Point to Stratford Shoal and Middle Ground (#H11044) was used to define the seafloor topography offshore of Silver Sands.

The US Army Corps of Engineers (USACE) data were obtained from the New England District (NAE) office that maintains the harbors and channels for New England. NAE performs surveys before and after dredging operations and makes this data available for download at: http://www.nae.usace.army.mil/navigation/navigation2.asp?mystate=ct. Hydrographic survey data was obtained for Milford Harbor and its associated navigational channel extending to 2000 ft offshore of the inlet.

The University of Connecticut (UCONN) was obtained from the Center for Land Use Education and Research (CLEAR) that maintains a database of imagery, survey data, and land use data sets for the state of Connecticut. CLEAR obtained a LIDAR data set for the State in 2000 and has made this data available for download from their website at: http://clear2.uconn.edu/ct_lidar/ct_lidar_processed-001/index.html. The LIDAR data sets 0110031, 0110032, and 0110041 were combined into one data set to define the coastline in the region including the shoreline at Silver Sands and Charles Island.

Fuss and O'Neill performed two topographic surveys in 2001 and 2011 at the Silver Sands Project Site as a part of their continued involvement with the Silver Sands project. These surveys included a series transects extending from the dune to a wading depth of approximately -5 feet NGVD29 and spanning from west of the bar (tombolo) to east of the Great Creek outlet structure. The survey data was used to define in more detail the beach at Silver Sand, the Great Creek outlet structure, and the inshore portion of the bar attached to the beach.

Nearshore data between Charles Island and Silver Sands Beach were somewhat limited in definition; however, the available data were sufficient for a first level assessment of wave transformation. Typically, a bathymetric survey of the nearshore region would be conducted to improve model results and increase accuracy; however, a bathymetric survey was not conducted specifically for this project. Therefore, the modeling results are limited by the available bathymetric data used in the modeling, and the wave and sediment transport modeling results may be improved through more detailed bathymetric data collection offshore of Silver Sands.

The data sources were combined with water depths referenced to mean tidal level (MTL) to create grids that consist of a mesh of points. For each grid, water depths interpolated from the bathymetry data are associated with points in the model domain.

In STWAVE, the reference grid consists of a mesh of points with dimensions NI and NJ, as shown in Figure 4-1. At each point within the model domain, water depth, as well as ambient current data, is specified. Reference points are separated by spacing DX (x-direction) and DY (y-direction). The east to west boundary of the model domain encompasses the shoreline from Welches Point to the Housatonic River. The north to south boundary of the model domain extended from the coastline to approximately 7,900 meters (25,919 feet) offshore. The orientation of the grid, especially the offshore boundary, was selected to closely represent a shore parallel contour line at a water depth deep enough that waves would not sense the sea floor.

The STWAVE grid is comprised of 799 cells in the cross-shore direction and 982 cells in the alongshore direction. All cells are 10 meters by 10 meters. The grid spans a distance of 9,820 meters (32,218 feet) in the cross-shore (x) direction and 7,990 meters (26,214 feet) in the alongshore (y) direction. Water depths at each grid location were interpolated from the bathymetry sources in the gridding process. Figure 4-2 shows the location and geometry of the bathymetric grid.

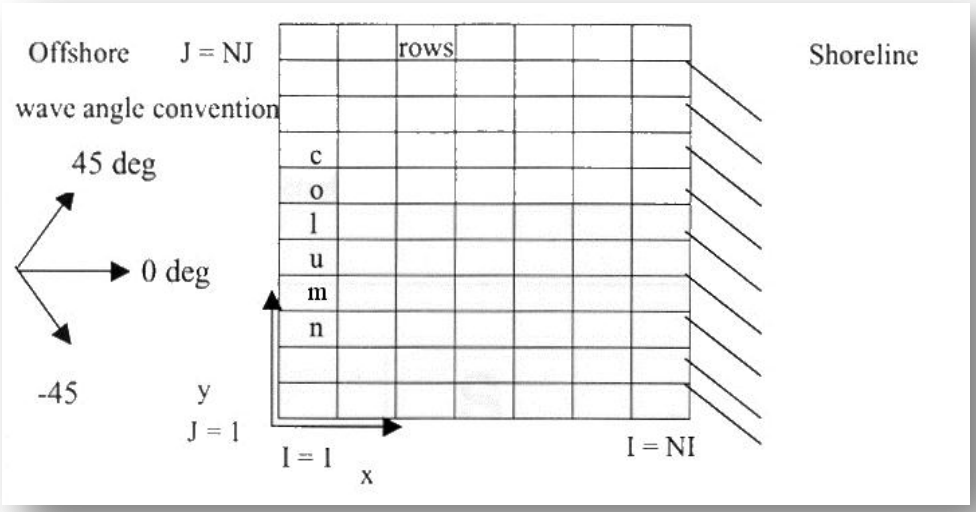

Figure 4-1. Illustration of reference grid notation (Smith, Sherlock, and Resio, 2001).

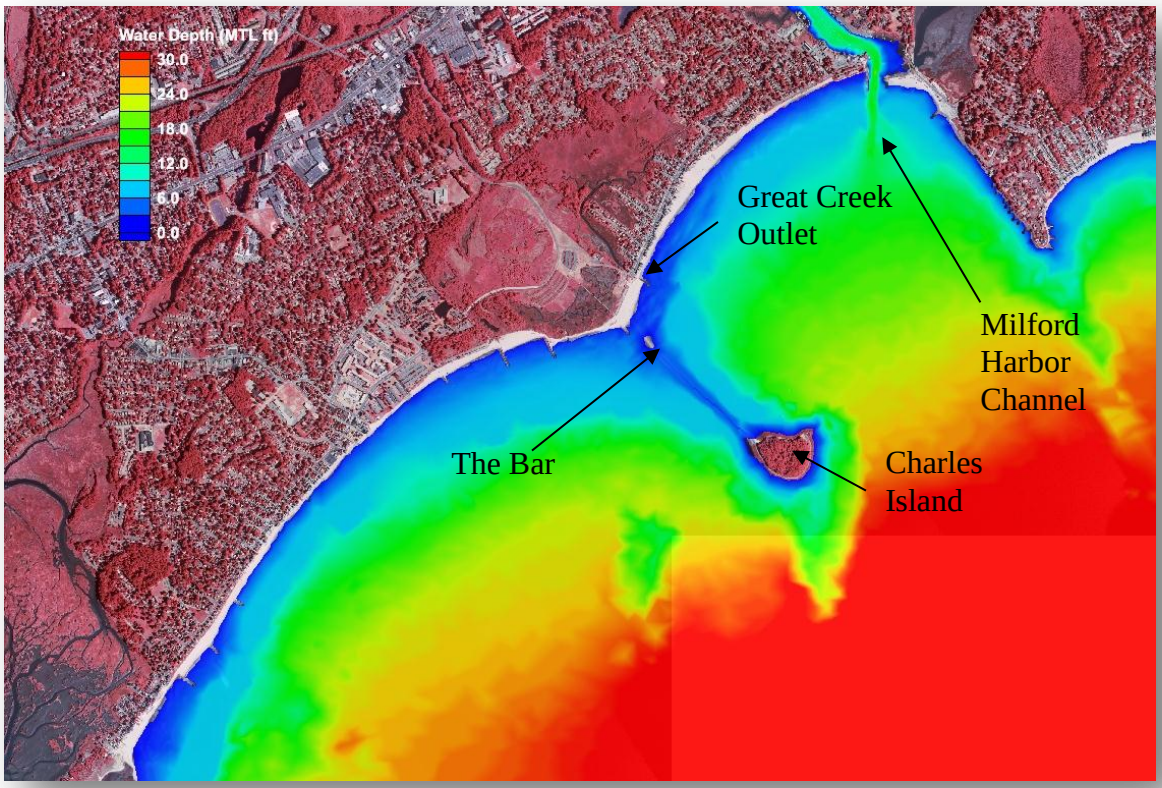

Figure 4-2. Bathymetric grid used for Great Creek outlet structure assessment. Depths are in meters relative to MTL (not to be used for navigational purposes).

4.4 WAVE CHARACTERISTICS AND INPUT SPECTRA

Transformation wave modeling can only be as accurate as the input data; therefore, a key component of accurate wave modeling is the analysis and selection of input wave data. The results derived from numerical wave transformation modeling, as well as the subsequent movement of sediment in the coastal zone, are controlled by the selected wave input conditions. This section describes the offshore wave climate and selection of input wave parameters for the wave transformation modeling. This includes the assessment of average annual conditions, specific historic storm events, and return-period storms. The data and methodology utilized to develop the wave climate is presented herein.

STWAVE simulates the behavior of a random sea surface by describing wave energy density as a function of direction (directional spectrum) and frequency (frequency spectrum). The two-dimensional wave spectrum is discretized into separate wave components, which constitute an essential part of the input for STWAVE. The two-dimensional wave spectrum is given as the product of the energy and directional spectra. Through a combination of the various wave directions and frequencies, STWAVE is able to simulate the behavior of a natural, random sea. In addition, detailed analysis and selection of input spectrum allows the model to assess the impact of different seasonal conditions, varying wave approach pathways, and storms. For the Silver Sands modeling, two distinct types of input conditions were supplied for the transformation-scale wave modeling:

- Energy-conserving annual average directional spectrum developed from the longer-term MYSOUND buoy data (section 4.4.1) and verified against Woods Hole Group data collected offshore of Hammonasset Beach.
- Storm spectra for site-specific and return period storm events.

4.4.1 Existing Wave Data

Wave data within Long Island Sound are relatively scarce, and sources that do exist (e.g., the University of Connecticut's MYSound buoy) are typically non-directional. In order to develop average annual wave conditions for use in this study, a data source containing a long-term record of wave data was required. Currently, the only long-term, site-specific wave data available within Long Island Sound are recorded by wave buoys that are deployed by the University of Connecticut. The University of Connecticut program is known as Monitoring Your Sound, or MYSound. The central Long Island Sound wave buoy has recorded waves since 2002. The MYSound buoy is located approximately 14 miles southeast of New Haven Harbor, CT (41° 8.25' N, 72° 39.30' W) and 10 miles southeast of Silver Sands in approximately 90 feet of water. The 2002 and 2003 wave data were obtained from the MYSound website (http://sounddata.uconn.edu/), while the record from 2004 onward is available for NOAA's National Buoy Data Center (http://seaboard.ndbc.noaa.gov/) under the NOAA Station 44039. This long-term data set can be used to develop average annual conditions. Development of the average annual conditions is detailed in Section 4.4.2.

In addition, Woods Hole Group collected directional wave observations in 2008 within the nearshore region closely adjacent to Silver Sands (Woods Hole Group, 2008). The Woods Hole Group collection program used two bottom-mounted Acoustic Doppler Current Profilers (ADCPs). The ADCPs recorded directional wave information once every hour over a nine-week period. The ADCPs also collected directional current information and water level elevation every 10 minutes. The data collected by the two ADCPs were statistically evaluated and utilized to provide directional information for the longer term wave data set.

4.4.2 Wind and Wave Correlation

Wave directionality is crucial to developing wave spectra that accurately characterize the wave climate. However, no long-term time series of directional wave observations are available within Long Island Sound. As such, average annual wave conditions needed to be developed from existing non-directional wave observation (e.g., MYSOUND buoy) and historic wind observations. Due to the orientation of Silver Sands within Long Island Sound, a majority of the waves impacting the shoreline will be local (within Long Island Sound) wind-generated waves. Therefore, directional wind observations, in concert with the short-term directional wave observations, can be used to determine the correlation that exists between wind and wave directions. This section describes the development of a correlation function that was applied to determine wave directions, based on wind direction, for the longer term non-directional wave data set.

The University of Connecticut's MYSOUND research project has been located in central Long Island Sound since 2002 (the National Data Buoy Center (NDBC) refers to this as station 44039). The MYSOUND buoy has been sampling waves and winds from 2004. Prior to 2004, the buoy only sampled waves. The wave sampling occurs every hour for 20 minutes and the reported statistics include significant wave height and significant wave period. The wind sampling occurs for 8 minutes and wind speed and wind direction are reported hourly.

To obtain wave direction at the central Long Island Sound location, the wave information recorded by Woods Hole Group (2008) during the field collection time period was compared to the wave and wind information record by the MYSOUND buoy. Figure 4-3 presents the directional distribution of significant wave height (m) data (illustrated using a wave rose plot) measured by Woods Hole Group (2008) offshore Hammonasset Beach. The gray scale sidebar indicates the magnitude of the wave height, the circular axis represents the direction of wave approach (coming from) relative to true north (0º), and the extending radial lines indicate percent occurrence within each magnitude and directional band. Likewise, Figure 4-4 presents the directional distribution of wind speed (ft/s) data (illustrated using a wave rose plot) observed at the MYSOUND buoy station. There does not appear to be an obvious correlation between wave and wind direction based on a direct visual comparison. However, as is seen in Figure 4-5, a strong correlation is evident for times when wave energy does exist.

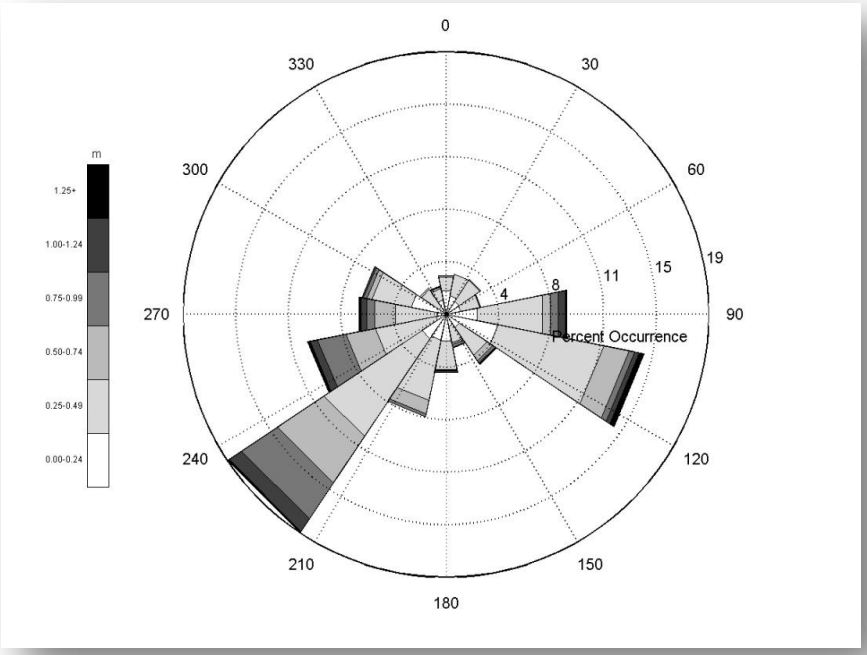

Figure 4-3. Rose plot of observed wave height at the wave station offshore Hammonasset Beach (Woods Hole Group, 2008).

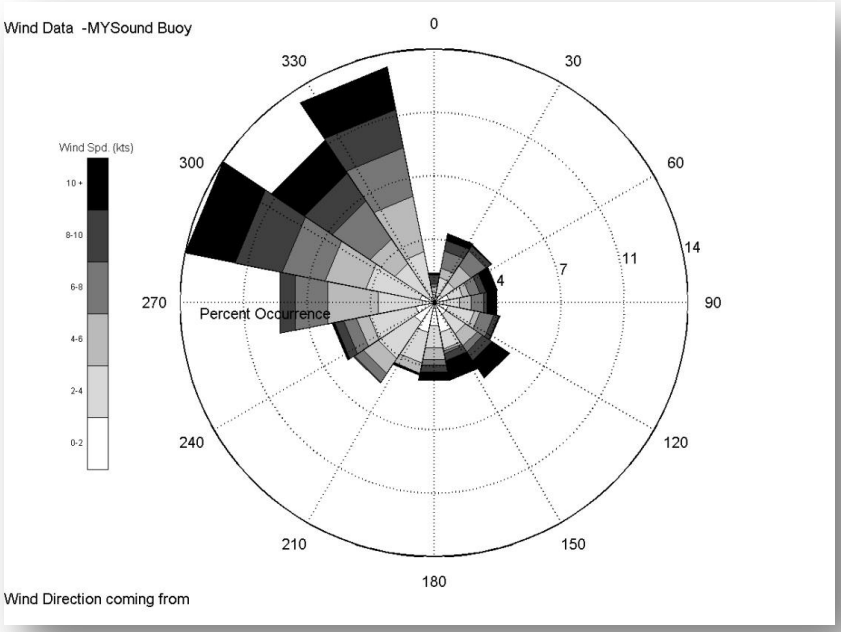

Figure 4-4. Rose plot of observed wind speed at the MYSOUND buoy over the same time period as waves were collected by Woods Hole Group (February to April, 2007).

Figure 4-5 shows the scatter plot of buoy wind direction versus ADCP wave direction when wave heights exceeded 1 foot (or only during the times when there is significant wave energy). Wind direction is shown on the bottom (horizontal) axis, while wave direction is shown on the left hand (vertical) axis. Each green dot represents an individual observation time, and the line on the figure represents a linear fit of the all the data points. The points are clustered reasonably well along the line, and primarily lie in two distinct zones, which correspond to the two dominant wave directions observed during the deployment period. Therefore, wind direction and wave direction are reasonably correlated. Strong winds coming from the west and west-northwest produce waves approaching from the west and southwest. This corresponds to the most common winds and waves impacting Silver Sands. A second cluster of points corresponds to winds from the northeast to southeast, producing waves from the east and southeast.

Wave directions can be assigned to the longer-term wave record by using the observed wind direction and applying a correlation function. The correlation function was developed from the equation representing the linear fit line presented in Figure 4-5. Figure 4-6 shows the rose plot for the observed wave heights and wind-correlated wave directions for the MYSOUND buoy during the long-term observations, and is representative of the longer-term wave climate offshore of Silver Sands. The distribution of wave approach is reasonable when compared to the shorter-term observations in the nearshore (Woods Hole Group, 2008).

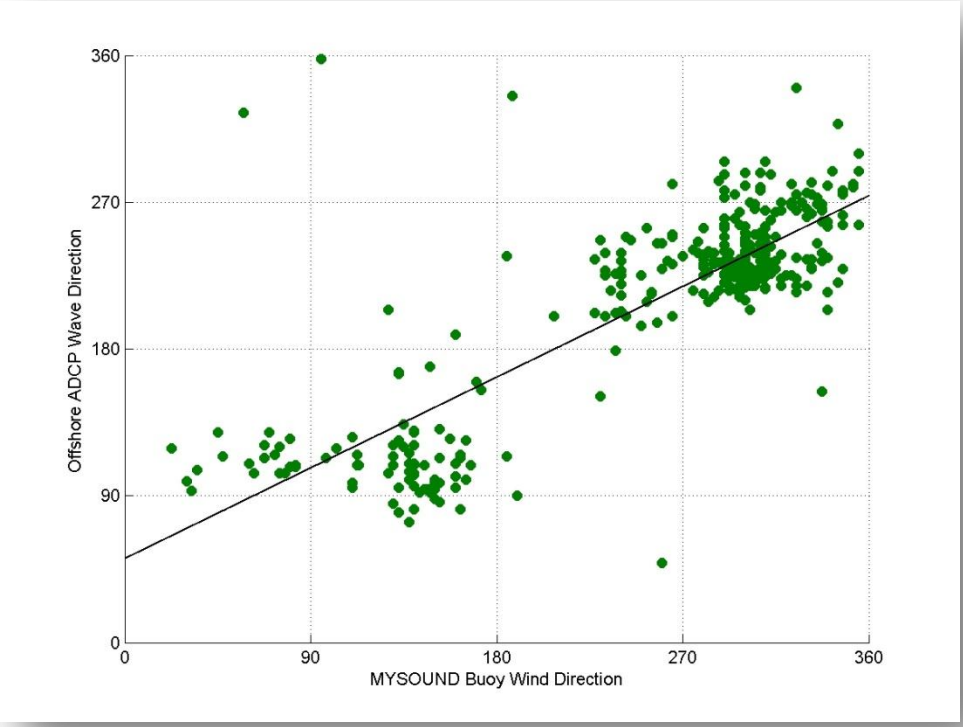

Figure 4-5. Sub-sample of buoy wind direction versus nearshore (ADCP) wave direction when waves were over 1 foot.

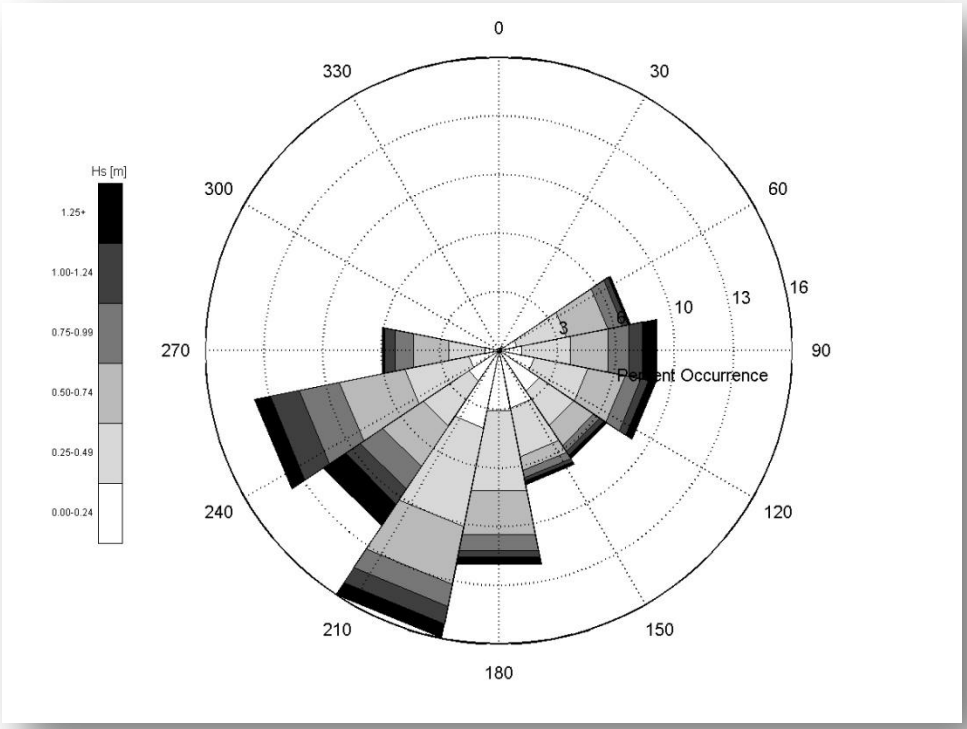

Figure 4-6. Wave rose using the observed wave heights and wind-correlated wave directions offshore of Silver Sands.

4.4.3 Average Annual Directional Approaches

In order to determine long-term wave conditions and wave statistics at the coastline, as well as for use in sediment transport modeling, the correlated waves from the MYSOUND buoy were used to derive energy-conserving annual average directional spectrum. Data were segregated by direction of approach, and an energy distribution, as a function of frequency, was generated from all the waves in each directional bin. The energy associated with each frequency was then summed to create an energy distribution for each approach direction. In essence, a representative two-dimensional spectrum was generated for each approach directional bin based on the sum of all the MYSOUND waves approaching from that mean direction. This can then be combined with the percentage of occurrence to create a longer-term evaluation of wave impacts at the shoreline. This energetic directional bin approach has been successfully utilized in transformation modeling (Byrnes et al., 2000) and identifies all potential approach directions, including those that may occur only a small percentage of time during a typical year, but potentially have significant impacts on the shoreline and sediment transport.

Table 4-1 presents the directional bin scenarios that were simulated in STWAVE to represent the complete wave climate offshore of Silver Sands. The table also presents the directional bin scenario, including the percent occurrence, the significant wave height,

peak period, and peak direction within that bin. The frequency and directional energy spectra were tailored to match the energy distribution of each approach bin that occurred in the MYSOUND data. Therefore, the directional and frequency distributions matched the data directly. Each of the directional bins presented in Table 4-1 were simulated in the wave transformation model.

In addition, STWAVE is a half-plane model, and therefore, only represents waves propagating towards the coast. Waves that may be reflected from the coastline or structures and waves that are generated by winds blowing offshore are not included. Waves headed offshore would represent a calm period along the coastline; therefore, waves propagating offshore were not modeled in the system, and were assimilated into the analysis as calm periods.

Table 4-1. Input conditions and scenarios for the wave transformation numerical modeling.

Directional Bin (0°=N)	Approach Direction	Percent Occurrence	Sig. Wave Height (ft)	Peak Period (sec)	Peak Direction (0°=N)
90 to 112.5	E	8.44	1.85	3.6	101.5
112.5 to 135	ESE	8.68	1.50	3.3	122.5
135 to 157.5	SE	6.74	1.21	3.2	146.5
157.5 to 180	SSE	9.04	1.27	3.1	169.0
180 to 202.5	S	15.09	1.57	3.3	191.5
202.5 to 225	SSW	15.06	1.84	3.5	213.9
225 to 247.5	SW	11.68	2.14	3.7	242.3
247.5 to 270	WSW	11.84	1.91	3.5	257.9
Calm	--	13.43	--	--	--

4.4.4 High Energy Events

Since high-energy events have a significant impact on many physical processes (and in most cases, dominate sediment transport), it is crucial to include storm simulations in wave modeling to assess the potential impact of a storm on the shoreline and the potential sediment transport along Silver Sands. High energy events were evaluated by reviewing existing literature on hurricanes and northeast storms that affected the southern coast of Connecticut, as well as a Woods Hole Group historical database of modeled storm events in Long Island Sound. Return period storms and two specific historic Hurricane events were simulated for the Silver Sands area.

Woods Hole Group has a set of 20 historical high energy events that were simulated as part of a coastal flooding analysis for Milford, CT. This database of storm waves, which does not include all storms of record in Long Island Sound, was evaluated to find storms that would impact Silver Sands. Of these 20 high energy events, most storms had a wave approach direction that would not have significantly influenced Silver Sands. This does not mean that other storm events would not have significantly influenced Silver Sands Beach or resulted in significant erosion, rather that the storms that were simulated for Milford, CT may not have produced as much erosion at Silver Sands. Of the historic storms simulated at Greenwich, Hurricane Gloria (September 1985) had the most potential for erosion at Silver Sands.

In addition, return-period storm events (10-year, 50-year and 100-year) were developed and simulated in STWAVE to provide varying levels of storm events expected to occur at this location. The return-period storm wave height was developed using the Generalized Extreme Value (GEV) method. This method provides reliable estimates of extremes without assuming the distribution type is known (Resio, 1989). The GEV method uses asymptotic methods to fit sampled maxima to the tail of a parent distribution, whose characteristics are estimated from the original sample. The original sample was taken from the MYSOUND data set. Table 4-2 presents the wave heights estimated by GEV, as well as the information for the simulated hurricanes. The return period storms peak wave periods were derived using the following relationship (CEM, 2002) for extreme wave parameters:

$$T_p = \sqrt{10.25 H_{mo}} \qquad (4\text{-}2)$$

where H_{mo} is the extreme wave height. Since the wave direction of extreme events is unknown for return-period storms, a wave direction of 220° (relative to north) was assumed. This direction was chosen to represent wave direction during a typical storm event observed in Long Island Sound and found within the MYSOUND data.

Storm surge values were also included in the wave modeling simulation to represent the increased water level experienced during the passage of a large storm event. Storm surge is especially critical for a site like Silver Sands, where direct wave approach from storms is not as common. Elevated water levels, even with moderate wave heights, can result in significant erosion along the shoreline. Surge values reported by a variety of sources were used to determine the water level associated with these storm events. For return-period storms, storm surge data were taken from Tidal Flood Profiles of the New England Coastline (USACE, 1988). For the September 1985 (Hurricane Gloria) specific storm event, storm surge was determined from historical observations and published storm surge values.

Table 4-2 presents the storm scenarios simulated. Storm spectra were developed from these storm parameters using standard parametric methods (e.g., TMA spectra, $\cos^n$ directional distribution), since the observed spectra during these events are unknown.

Table 4-2. Bulk wave parameters used to define the high energy events input spectra.

Event	Storm Surge [MTL, ft]	Wave Height [ft]	Wave Period [sec]	Wave Direction [deg, 0=N]
September 1985 (Hurricane Gloria)	6.6	6.6	5.1	16
10-Year	7.0	8.9	6.0	220
50-Year	8.8	15.8	8.0	220
100-Year	9.3	18.9	10.0	220

4.5 MODEL RESULTS

The Steady-State Spectral Wave (STWAVE) model was used to simulate a wide range of annual average directional conditions, return-period storm events, and specific historical storm events for the Grove Beach Point area.

4.5.1 Average Annual Directional Approaches

Model simulations were performed for the typical wave conditions represented by the directional bin spectra presented in Table 4-1. Wave focusing and divergence occur at several locations throughout the modeling domain, which results in variations in the wave energy propagating towards the coastline of Silver Sands for each directional bin.

Figure 4-7 illustrates STWAVE results within the modeling domain, for waves approaching from the southwest (202.5 to 225 bin), the highest significant wave height approach direction of the typical condition cases. The color map corresponds to the distribution of significant wave height (ft) throughout the modeling domain. Reds indicate higher wave heights, while blues indicate smaller wave heights as presented in the inset color scale legend on the figure. The model simulation was conducted at depths and shoreline positions corresponding to mean tide level (MTL). Arrows on the figure represent the modeled wave direction as they propagate and approach the shoreline.

As was shown in Table 4-1, the southwestern directional bin is the most energetic, having the largest waves out of all the directional bins. There is significant wave refraction that occurs as the waves react to the bathymetry and align themselves with the southwest to northeast portion of the Silver Sands shoreline. As waves travel across shallow water, friction from the sea floor will reduce the energy in a wave, a process known as wave shoaling. Therefore, a significant amount of energy is lost as the waves traverse the shallow bar (tombolo) connecting Charles Island to the mainland. Charles Island also has a significant impact on the wave transformation in the region, as waves diffract around the island, break against the island, and create a shadow zone of low wave energy in the lee of the island. This provides significant wave sheltering for Silver Sands under certain wave approach directions. Bands of increased wave energy are apparent throughout the

region (red, orange, and yellow areas within the grid), including an increase that is directed at Welches Point. In general however, for this particular approach direction, lower wave energy is influencing a majority of the shoreline west of the outlet structure, due to the sheltering caused by Charles Island and wave breaking over the bar. However, although difficult to visualize at this scale, there is a focused increase in wave energy where the bar meets the beach. This increased wave energy, indicated by the red arrow on Figure 4-7, occurs in most of the wave approach directions (also see Figure 4-9).

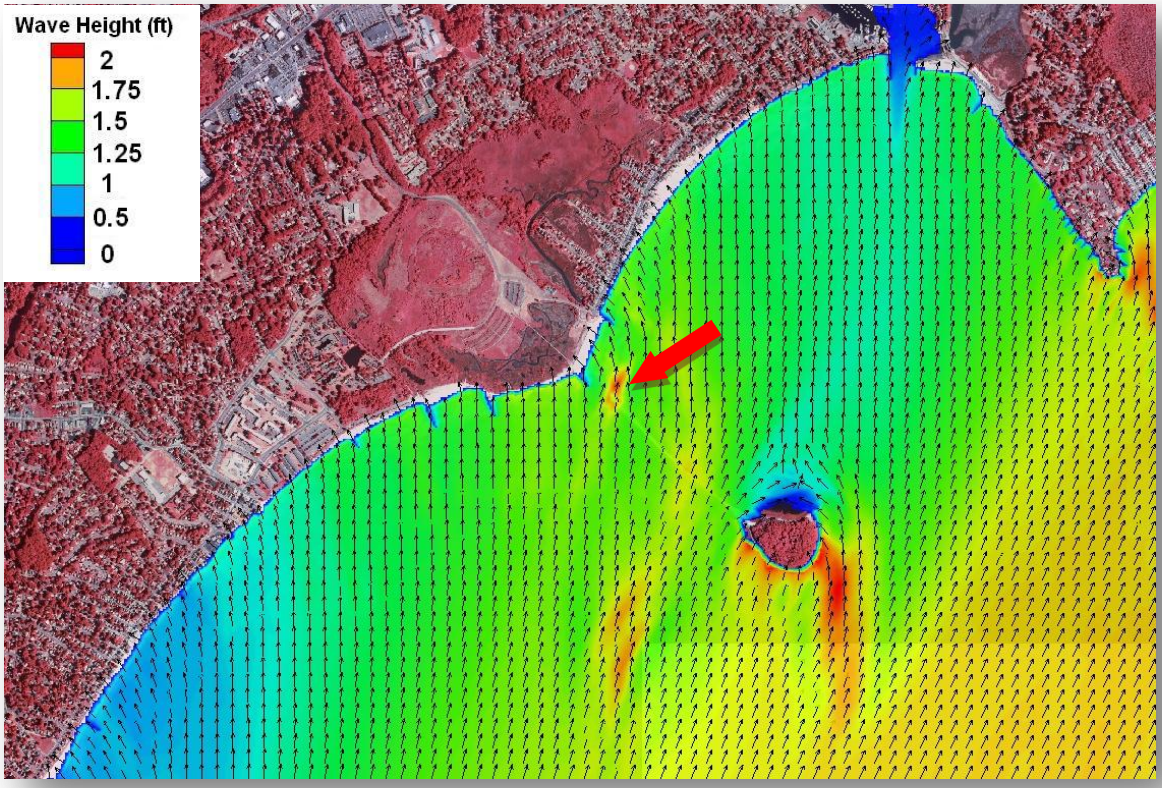

Figure 4-7. Spectral wave height (meters) modeling results for a southwestern approach direction (202.5-225 degree bin) for the Silver Sands region. Wave height and direction are represented by the color map and black arrows, respectively.

Figures 4-8 presents the model results from the eastern wave direction (90 to 112.5 degree bin) and represents a clearly different sea state than presented Figure 4-7. The color scale is consistent with Figure 4-7, such that the wave heights can be easily compared between cases. For the east approach direction, the wave shadows created by Charles Island, the Bar, and Welches Point are evidenced by the large area of reduced wave heights (blue) offshore of Silver Sands (specifically west of Welches Point). For this approach direction, wave energy along the shoreline is significantly reduced in this area, primarily due to the sheltering effects of these natural features. The overall wave energy and sediment transport, which would be driven from east to west under this condition.

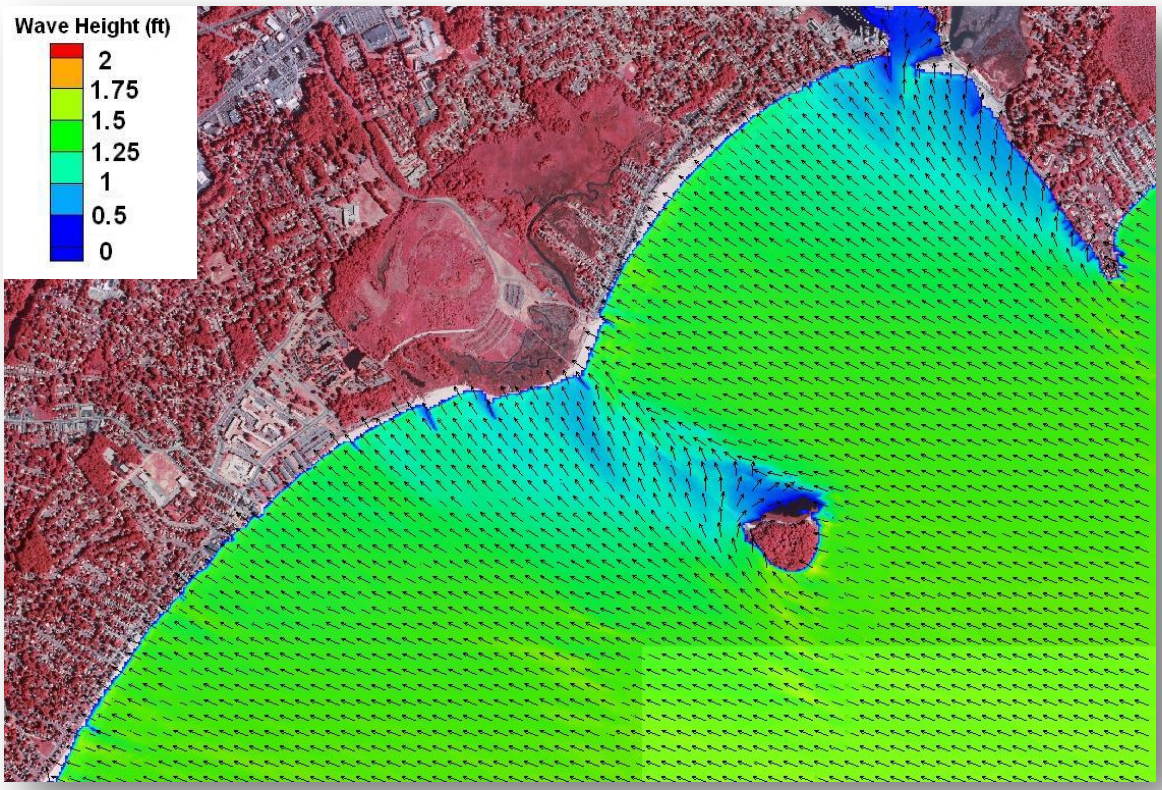

Figure 4-8. Spectral wave height (meters) modeling results for an east approach direction (90-112.5 degree bin) in the Silver Sands region. Wave height and direction are represented by the color map and black arrows, respectively.

Figure 4-9 presents the model results from the south directional approach bin. In this approach direction, there is a high energy wave area that is able to propagate through the area between Charles Island and Welches Point and subsequently is refracted directly northwest of the Great Creek Outlet Structure. So while sheltering by the Charles Island occurs, there is a smaller area (corresponding to this location downdrift of the outlet structure) that does experience higher wave energy during even normal conditions.

The variability in the wave climate is indicated by the differences in nearshore wave patterns arising from the various input spectra approach directions. Figures for the remaining approach directions for the Silver Sands region are presented in Appendix B.

In order to arrive at an accurate estimation of the sediment transport in the region, including the reversals in sediment transport direction that occur, results from the wave model can be used to generate the sediment transport flux. This includes waves coming from all directions and having various wave heights and periods. The combination of all the directional approach cases (Appendix B) allows for an assessment of the average annual wave climate. As such, the STWAVE results were used to generate wave-induced currents (from radiation stresses) and regional sediment transport. The results of all the

approach directions are used, in concert with the percent occurrence, to compute the annual sediment transport in the region (Chapter 5.0).

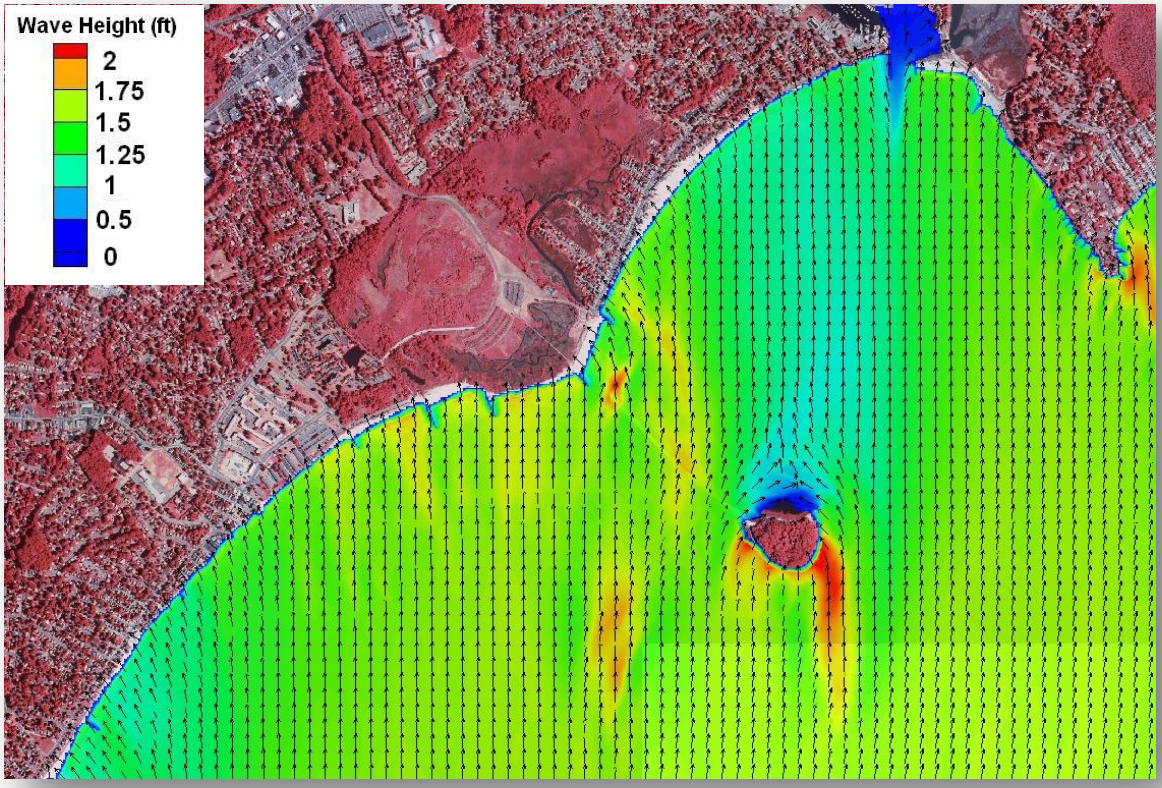

Figure 4-9. Spectral wave height (meters) modeling results for a south approach direction (180-202.5 degree bin) in the Silver Sands region. Wave height and direction are represented by the color map and black arrows, respectively.

4.5.2 High Energy Events

The wave transformation model was also used to simulate high energy events, as discussed in Section 4.4.4. The simulation of specific storm events was important to quantify the short-term impacts that occur during these energetic scenarios. Sediment transport along the coastline in many cases can be dominated by these short episodic events, and is likely the case for Silver Sands, especially considering the sheltering effects caused by Charles Island and its associated tombolo. Figures 4-10, 4-11 and 4-12 indicate the spectral wave model results for the 10-, 50-, and 100-year return period storm events, respectively. Wave heights are significantly higher than during the annual average directional cases, as the offshore heights are in excess of 5.5 m (18 feet) in some locations. All model results are plotted on the same color scale for wave height for inter-comparison of the storm waves on the Silver Sands region. The storm event spectral results, as were the annual average directional bin cases, were passed forward to the sediment transport modeling effort to assess direct impacts on the Silver Sands region.

Interior flooding of the marsh occurs at Silver Sands during these storms with flooding along the beach at East Broadway as the low lying barrier beach is overtopped during these storm events.

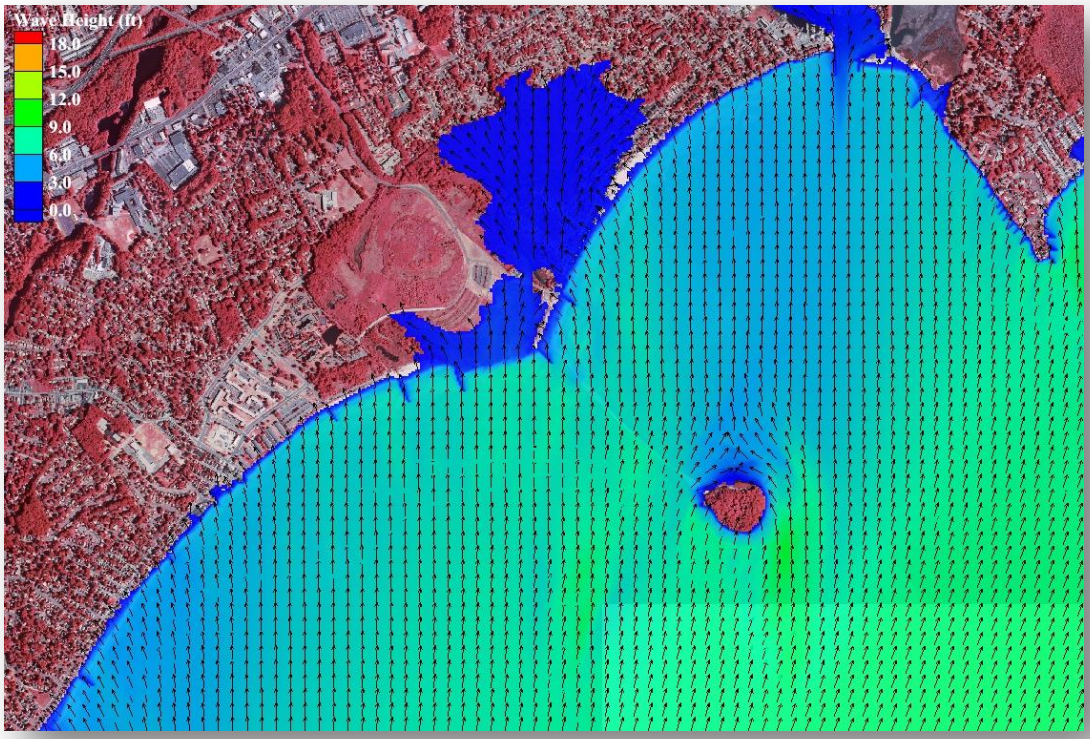

Figure 4-10. Spectral wave modeling results for a 10-year return period storm in the Silver Sands region.

4.6 SEA LEVEL RISE

Another important consideration in the wave transformation simulations, as well as the long-term planning for Silver Sands, is potential sea-level rise. The potential impacts of sea-level rise present an additional natural hazard risk for developed areas within the coastal zone. The impacts are similar to those caused by shoreline erosion, and include increased flooding and wave activity in areas previously not affected, as the shoreline moves increasingly further inland.

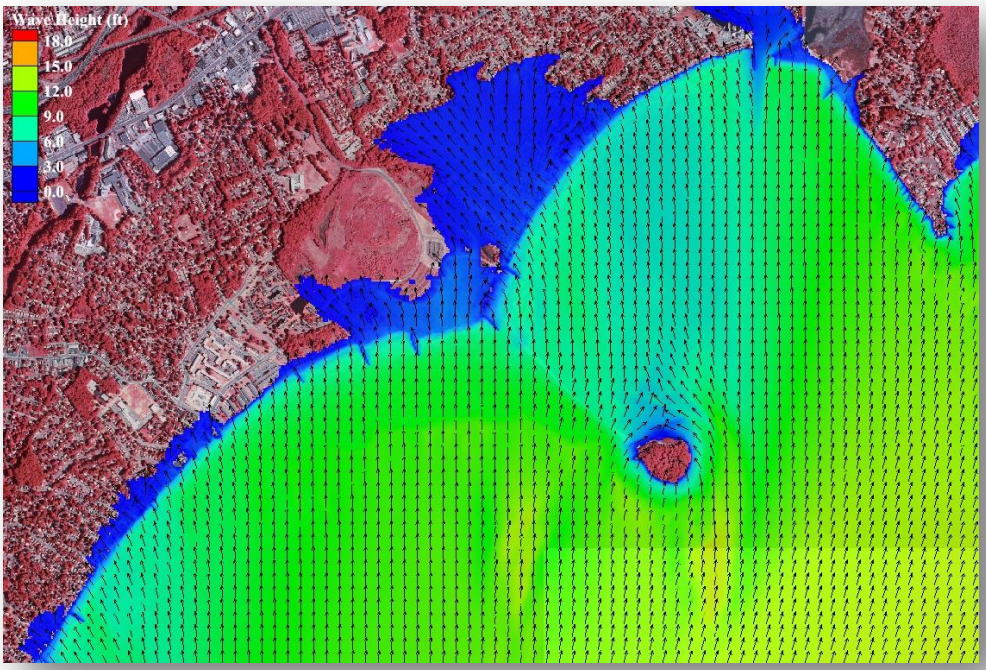

Figure 4-11. Spectral wave modeling results for a 50-year return period storm in the Silver Sands region.

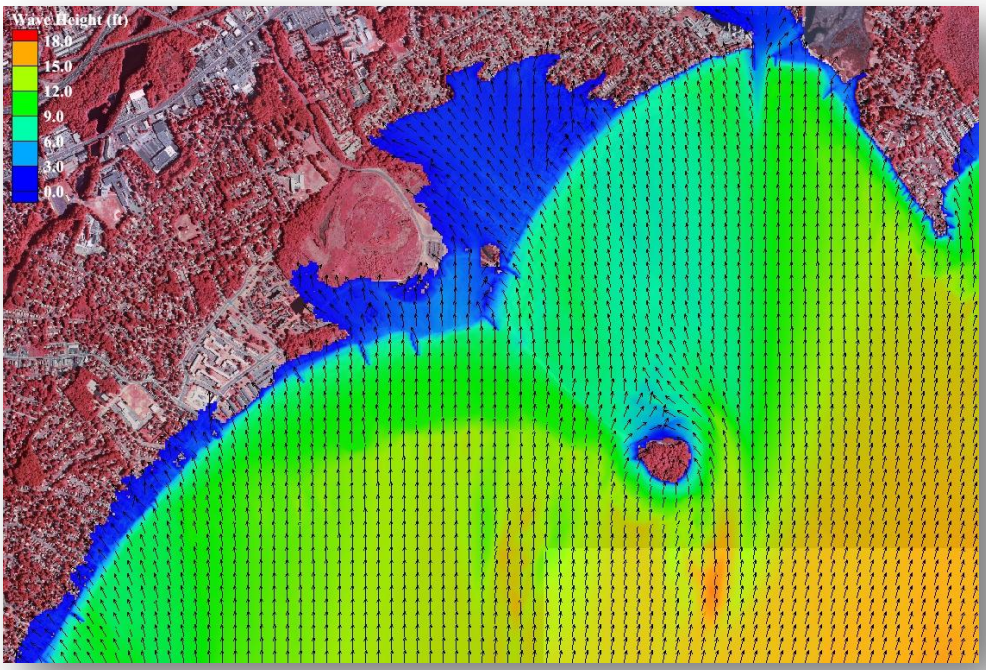

Figure 4-12. Spectral wave modeling results for a 100-year return period storm in the Silver Sands region.

The topic of sea level rise and accelerated sea level rise in the 21st century and beyond has been the subject of much debate. The Intergovernmental Panel on Climate Change (IPCC) has spent considerable time and energy reviewing and analyzing the current state of knowledge of past and future changes in sea level in relation to climate change. Taking this information, the United States Army Corp of Engineers (USACE) has developed guidance for incorporating sea-level change considerations in civil works programs (USACE, 2009). Under this design guidance, the USACE provides three (3) predicted rates of sea level rise (high, intermediate, and low) to use in the design of civil works projects. The Corps guidance uses current local sea level rise rates based on historic tide data to form projections for the "low" scenario, and a combination of these local rates with IPCC (2007) scenarios and the National Research Council's (1987) equations for the "intermediate" and "high" scenarios for accelerated sea level rise. This method derives locally specific estimates for sea level rise that span a broader range of scenarios than the IPCC estimates alone. For this study, as a preliminary basis, the existing historic sea level rise rates were evaluated.

Scientific research indicates that global (eustatic) sea level has risen approximately 6 to 8 inches over the last century (EPA, 2000). This eustatic rise in sea level has occurred in part due to glacial isostasy, warming of the world oceans, and melting of continental glaciers. Along most of the US coast, tide gage data show that local sea levels have been rising 2.5 to 3.0 mm/yr, or 10 to 12 inches over the past century. Because the tide gage stations measure sea level relative to the land, which includes changes in the elevations of both water levels and the land, tide gages measure relative sea level rise, and not the absolute change in sea level. Therefore, the rates of relative sea level rise have greater relevance to the evaluation of coastal hazards from sea level rise, than do changes in eustatic sea level.

Long-term tide gage data collected at the NOS (National Ocean Service) station in Bridgeport, CT provides the closest measurements to Silver Sands (NOAA, 2008). Tide gage data from the Bridgeport station for the period 1964 to 1999 indicate a rise in sea level of 2.56 mm/yr, or 10.1 inches over the past century (Figure 4-13). This rate was used to calculate the values for the low, intermediate, and high rates of sea level rise over a 50 year time horizon (between 2013 to 2063) at shown in Table 4-3. In 50 years, sea level rise is expected to range from 0.42 feet on the low end and 2.13 feet on the high end. Both these high and low rates are considerable increases that could have significant impacts. For this study, low and intermediate sea level rise scenarios over a 50 year time horizon (corresponding to the expected service life of a replacement outlet structure) were input into the wave transformation model to assess the influence of sea level rise on wave and sediment transport conditions.

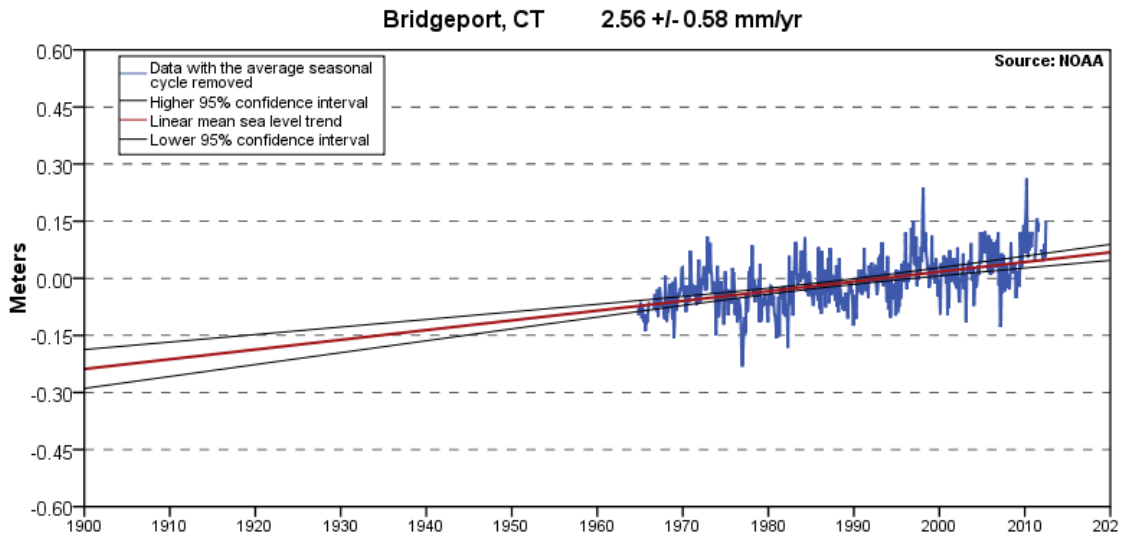

Figure 4-13. Long-term tide data from NOS gages at Bridgeport showing relative rise in sea level (NOAA, 2008).

Table 4-3. Projected sea level rise for the Silver Sands over a 50 year time horizon between 2013 and 2063

Sea Level Rise Rate	Sea Level Increase (ft)
Low	0.42
Intermediate	0.82
High	2.13

4.7 SUMMARY

A transformation wave model (STWAVE) was used to propagate the offshore wave climate into the Silver Sands region. The transformation scale model was used to simulate average annual directional cases (developed from MYSOUND buoy data), specific historic storms events, and return-period storms. Results of the transformation scale model are used to develop regional sediment transport fluxes and divergence (Chapter 5.0).

5.0 ALONGSHORE SEDIMENT TRANSPORT

Understanding the wave transformations (Chapter 4.0) is a critical step in the determination of shoreline processes and changes, and this wave information is required in order to provide an estimate on how sediment moves in the nearshore region. The wave modeling system results were the key input into the sediment transport modeling and beach performance evaluation. The specific objective is to obtain estimates of the alongshore sediment flux integrated across the surf zone. This chapter evaluates the regional sediment transport for Silver Sands in the alongshore direction.

Coastal sediment transport is comprised of sediment moving in both the alongshore and cross-shore directions. While the alongshore sediment transport is mostly due to the long-term average wave conditions, the cross-shore sediment transport is most often a result of infrequent storm events. For this engineering assessment, which focuses on potential enhancements to the Great Creek outlet structure, only the alongshore sediment transport was determined to evaluate potential modifications to the structure and subsequently the influence on the sediment being transported along the beach. Specifically, the distribution of the total alongshore sediment flux across the surf zone is determined.

5.1 ANALYSIS APPROACH

Sediment movement in the coastal zone, as well as the effects of coastal structures on shoreline processes, can be estimated by using various types of sediment transport models. These models may differ in their detail, in their degree of representation of the physics, in their complexity, and in other manners. Process-based sediment transport models (those that address directly the fundamental physics of waves and sediment transport) may focus on those essential physics that capture the variable wave field. Such sediment transport models may not represent all aspects of physical processes accurately, but they can be used to demonstrate the regional sediment transport trends and spatial influence of coastal structures on adjacent shorelines. The sediment transport model presented herein is a process-based model of the regional sediment transport trends in the presence of time-variable (in direction and height) waves.

The goal of this model is to provide a physically-based representation of alongshore currents and sediment transport driven by breaking waves in the surf zone. The specific objective is to obtain physically-based estimates of the alongshore sediment flux integrated across the surf zone. To achieve this physically-based representation, it is important to understand what alongshore sediment processes may cause erosion or accretion. Typically, a section of shoreline can be represented as a cell, having finite length along the shore. A certain amount of sediment enters this cell from the updrift side (direction from which the waves advance), and a certain amount leaves the cell from the downdrift side. This sediment balance may vary depending on the height of the wave, the direction of the wave, and the period of the wave. If the effects of a particular wave passing a cell are examined, there are three possibilities that may be observed for that wave condition:

- The same amount of sediment enters a cell as leaves the cell.

- More sediment enters a cell than leaves the cell.
- More sediment leaves a cell than enters the cell.

The first possibility leads to a stable shoreline. The shoreline neither erodes nor accretes. The second possibility leads to an accumulation of sand in the cell, which is a situation causing accretion (building out of the shoreline) to occur. This possibility is referred to as sediment convergence, as sediment converges in the cell. The final possibility leads to a net loss of sediment in the cell, which is a situation causing erosion. This possibility is referred to as sediment divergence, as sediment diverges from that cell. Thus, shoreline erosion or accretion can be thought of as a simple divergence or convergence of sediment moving alongshore. Of course, storms also can move sand offshore, and other waves may move sand onshore; however, as discussed, this onshore-offshore process is not directly modeled, and the focus is on the alongshore movement of sand in the region.

The regional sediment transport model requires the results of the wave field presented in Chapter 4.0. The sediment transport model itself consists of a hydrodynamic component to determine the wave-induced currents and a sediment transport component to quantify the amount of sediment moved by the wave-induced currents. The hydrodynamic component is based on a standard set of equations that are widely accepted and generally used, more specifically known as the steady-state depth-averaged mass and momentum equations for a fluid of constant density. These equations are standard in many surf zone applications (e.g., Mei, 1983) and provide a state-of-the-art representation of the alongshore current. The sediment transport component is based on a recent peer-reviewed and published formulation by Haas & Hanes (2004), which has been shown to be consistent with recent complex formulae for wave-driven sediment transport and with the Coastal Engineering Research Center (CERC) formula for the total (laterally-integrated) alongshore sediment flux.

For Silver Sands specifically, the same a high-resolution bathymetric grid used for the wave transformation modeling was also used for the sediment transport modeling (Figure 4-2). The wave transformation model was executed for the average annual conditions and the high-energy events on this high-resolution grid. The results from the STWAVE simulations are then supplied as input into the sediment transport model.

5.2 MODEL DESCRIPTION

5.2.1 Hydrodynamic Component

Governing Equations

The wave-averaged, depth-integrated, mass-conservation equation for a constant-density fluid with a rigid lid is

$$\frac{\partial(Hu)}{\partial x} + \frac{\partial(Hv)}{\partial y} = 0, \tag{5-1}$$

and the wave-averaged, depth-averaged momentum equations for a non-rotating system are

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} = -g\frac{\partial \eta}{\partial x} - \frac{ru}{H} + \frac{\tau_x}{(\rho H)} \tag{5-2}$$

and

$$\frac{\partial v}{\partial t} + u\frac{\partial v}{\partial x} + v\frac{\partial v}{\partial y} = -g\frac{\partial \eta}{\partial y} - \frac{rv}{H} + \frac{\tau_y}{(\rho H)} \tag{5-3}$$

Here x and y are the horizontal coordinates, t is time, u and v are the x and y components of the wave-averaged and depth-averaged horizontal velocity, g is the gravitational acceleration, η is the surface displacement, r is the bottom resistance coefficient, H is the water depth, ρ is the fluid density, and τ_x and τ_y are $-(1/H)\partial S_{xx}/\partial x - (1/H)\partial S_{xy}/\partial y$ and $-(1/H)\partial S_{xy}/\partial x - (1/H)\partial S_{yy}/\partial y$, respectively, where S_{xx}, S_{xy}, and S_{yy} are the components of the wave-induced radiation stress tensor (e.g., Mei 1989).

A stream function (ψ), which defines the two-dimensional flow, can be defined by

$$(u,v) = H^{-1}\left(\frac{\partial \psi}{\partial y}, -\frac{\partial \psi}{\partial x}\right), \tag{5-4}$$

which satisfies (5-1) identically, and an equation for the wave-averaged potential vorticity ξ, defined by

$$\xi = \frac{1}{H}\left(\frac{\partial v}{\partial x} - \frac{\partial u}{\partial y}\right) = -\frac{1}{H}\frac{\partial}{\partial x}\left(\frac{1}{H}\frac{\partial \psi}{\partial x}\right) - \frac{1}{H}\frac{\partial}{\partial y}\left(\frac{1}{H}\frac{\partial \psi}{\partial y}\right), \tag{5-5}$$

is obtained by taking the curl of (5-3) and (5-4) and dividing the result by H, which yields

$$\frac{\partial \xi}{\partial t} + u\frac{\partial \xi}{\partial x} + v\frac{\partial \xi}{\partial y} + \lambda\xi = \lambda\xi_0 + \frac{v_0 - v}{H}\frac{\partial \lambda}{\partial x} - \frac{u_0 - u}{H}\frac{\partial \lambda}{\partial y} \tag{5-6}$$

where $\lambda = r/H$, $u0 = \tau_x/(\rho_r)$, $v_0 = \tau_y/(\rho_r)$, and $\xi 0 = H\text{-}1(\partial v_0/\partial x - \partial u_0/\partial y)$.

In the present application, H is known, r is assumed to be given in the linear long wave approximation by $c_d[H_s/(4H)](gH)^{1/2}$ (e.g., Mei 1983), and τ_x and τ_y are output from the wave transformation model. Here c_d = 0.003 is the drag coefficient for the surf zone under breaking waves (Feddersen et al., 1998) and H is the significant wave height, defined to be four times the standard deviation of the wave-induced oscillatory surface displacements, which is also given by the wave model. With this information, (5-4), (5-5) and (5-6) determine the coupled evolution of ξ, ψ, u and v.

Boundary Conditions

The coordinate system is defined so that x is positive onshore, x = 0 defines the offshore boundary of the computational domain, y = 0 and y = L_y denote the alongshore boundaries of the computational domain, and the shoreline is a potentially irregular boundary in x > 0. In the present application, there can be only one shoreline, and H is restricted to be positive and nonzero everywhere in the domain. Boundary conditions are required for ψ on all boundaries and for ξ on inflow boundaries. The following boundary conditions are intended for applications in which the offshore boundary is well seaward of the surf zone and the shoreline at the alongshore boundaries is approximately straight and parallel to the y axis.

At the offshore boundary, the forcing and velocity fields are assumed to weak, so that the alongshore velocity and potential vorticity are negligibly small and the offshore boundary conditions become

$$\frac{\partial \psi}{\partial x} = 0 \text{ and } \xi = 0 \text{ at } x = 0 \qquad (5\text{-}7)$$

At the alongshore boundaries, the velocity field is assumed to be approximately confined to the y direction and approximately independent of y, so that the alongshore boundary conditions become

$$\frac{\partial \psi}{\partial y} = 0 \text{ and } \frac{\partial \xi}{\partial y} = 0 \text{ at } y = 0, L_y. \qquad (5\text{-}8)$$

The shoreline is a streamline, so that ψ on the shoreline must be a constant, which may without loss of generality be set to zero:

$$\psi = 0 \text{ on the shoreline.} \qquad (5\text{-}9)$$

The shoreline is not an inflow boundary, so that the shoreline potential vorticity does not affect the solution.

Numerical Solution

Equations (5-4), (5-5) and (5-6) are solved by means of a standard numerical procedure described, for example, by Roache (1998). Spatial derivatives are represented using finite differences on a rectangular grid with equal spacing dx in the x and y directions.

The representation of the spatial derivatives is second-order-accurate except that the advective terms in (5-6) are represented by a first-order upwind scheme. The time derivative in (5-6) is represented by an explicit first-order scheme with time step dt. The solution for each application begins from rest and advances in time until it reaches an asymptotic steady state. At each time step, the potential vorticity ξ is advanced according to (5-6), the elliptic equation (5-5) is then solved for the stream function ψ using Jacobi iteration (e.g., Lynch 2004), and finally the velocities u and v are calculated according to (5-4). Attainment of an approximate steady state requires that the solution advance until t is approximately equal to 3 times the maximum value of λ. Stability requires that the Courant number $(u_2+v_2)^{1/2}dt/dx$ based on the maximum flow speed be less than approximately unity.

5.2.2 Sediment Transport Component

Haas & Hanes (2004) proposed a simple formula for the alongshore sediment flux, which is, in the present notation,

$$q_s = \left(\frac{2c_1 c_d}{g} \right) < |\boldsymbol{u}|^2 > u_s \tag{5-10}$$

where q_s is the alongshore component of the sediment flux, c1 is an empirical constant approximately equal to 1.3, brackets denote an average over many wave periods, u is the instantaneous velocity vector (including both the wave-induced oscillatory velocity and the current), and us is the alongshore component of the current velocity.

In the present application, u is assumed to be dominated by wave-induced oscillatory velocities and to be related to wave-induced surface displacement by linear long wave theory, so that <|u|2> approximates [Hs/(4H)]2gH. In addition, a right-handed coordinate system (s,n,z) is defined so that s is locally alongshore, n is locally shore-normal, and z is vertical and positive upward. In this coordinate system, Hus = $\partial\psi/\partial$n. Equation (5-10) can therefore be written

$$q_s = 2c_1 c_d \left[\frac{H_s}{4H} \right]^2 \frac{\partial \psi}{\partial n} \tag{5-11}$$

In the surf zone, Hs/H is approximately constant (Hs/H < 0.63 is explicitly assumed by STWAVE), so that (5-11) can be integrated with respect to n across the surf zone to yield

$$Q = \frac{c_1 c_d}{8} \left[\frac{H_{sb}}{H_b} \right]^2 \psi_b \tag{5-12}$$

where Q is the alongshore sediment flux integrated across the surf zone and subscript b denotes evaluation at the break point, (i.e. at the seaward edge of the surf zone). In the present application, (5-12) is used to determine the sediment flux integrated across the

surf zone after the stream function has been computed from the hydrodynamic component.

5.3 SEDIMENT TRANSPORT RESULTS

5.3.1 Average Annual Conditions

In order to understand the nature of the transport in the vicinity of the Great Creek outlet structure, the regional sediment transport patterns were evaluated to ascertain the overall sediment transport fluxes. This section uses the results of the regional wave model, as presented in Chapter 4.0, to determine the nearshore hydrodynamics, and subsequently, the sediment flux (representing the rate of sediment moving along the coast) and divergence (indicating potential areas of erosion/deposition) along Silver Sands, with specific focus on the quantitative rate of sediment transport near the Great Creek outlet structure.

The regional wave modeling results (Chapter 4.0) were used as input into the non-linear sediment transport model. Wave results from each of the average annual directional spectra bin simulations were used to develop the complete summary of sediment movement for various wave conditions. Sediment transport results were also combined to define the average annual sediment transport regime throughout the region.

Model simulations were performed for the wave conditions represented by the directional bin spectra presented in Table 4-1. Figure 5-1 illustrates the sediment transport results for waves approaching from the southwest (202.5 to 225 degree) approach bin. Figure 5-1 presents the location map (top panel) where sediment transport was evaluated and the associated sediment flux (bottom panel). The top panel shows the bathymetry in the area surrounding Silver Sands and presents alongshore distances (in yards) for reference. The sediment flux (bottom panel) represents the rate of sediment moving along the coast. Negative sediment flux values indicate movement towards the east (from left to right) and positive values indicate movement towards the west (from right to left). This rate is presented in units of cubic yards per year and can be used to quantify the annual sediment transport at Silver Sands. These calculations assume that sediment is available on the beach for transport (e.g., potential transport). If the shoreline is armored (e.g., revetment), or doesn't have a sediment source readily available, then the sediment transport rates may vary compared to the values presented herein.

For this approach scenario, the sediment flux in Figure 5-1 shows sediment transport primarily to the east as shown by the black arrows in the lower panel (i.e., a negative sediment flux corresponds to sediment transport from the left of the plot towards the right of the plot). Due to the direction of wave approach (south-southwest), it is expected that sediment should primarily be transported to the east in this case. The rate (or strength) of the eastward transport varies along the coastline; however, averages approximately 7,000-8,000 cyd/yr for this particular approach direction. In other words, if waves solely came out of this approach direction throughout the year, the transport would follow the rates presented in Figure 5-1. In addition, there are some reversals in sediment transport direction under this approach direction. From approximately alongshore mark 4,500 (upper panel) to 4,750 in the vicinity of the groins along Silver Sands, the rate of

transport is slightly towards the west (positive on the plot). This reversal in direction is likely due to the shadow zone caused by these structures, as well as some of the wave transformations caused by the predominant shoal offshore. Sediment transport is also directed to the northwest for the area east of Milford Harbor due the significant change in shoreline orientation. As presented in Table 4-1, this approach direction (202.5 to 225 degrees, SSW) contains the highest wave energy, and therefore likely produces a significant portion of the sediment transport along Silver Sands for normal wave conditions. However, this also still only represents 15% of the total occurrence time, is only a single approach direction, and is only representative of times when waves are arriving from the south-southwest. All approach directions must be evaluated and aggregated to determine the net sediment transport movement at Silver Sands.

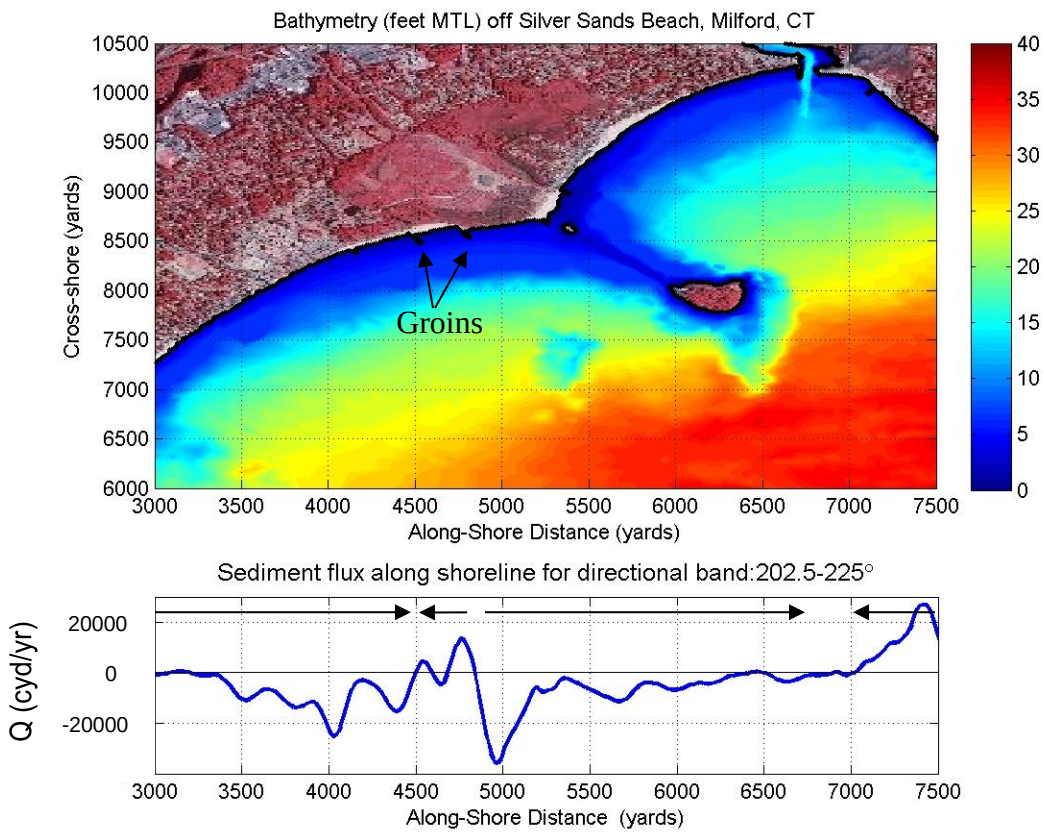

Figure 5-1. Sediment transport model results for a southwest wave approach (202.5 to 225 degrees, SSW).

Figures 5-2 and 5-3 are the sediment transport model results from two additional wave approach directions, from the east (90 to 112.5 degree bin) and south (180 to 202.5 degree bin), respectively. As in Figure 5-1, the top panel presents a location map and the bottom panel presents the associated sediment flux. The results presented in Figures 5-2 and 5-3 correspond to the wave model results presented in Figures 4-8 and 4-9, respectively. Figure 5-2 exemplifies the wave shadowing caused by Charles Island as waves approach from the east. Wave energy is reduced for a significant portion of the shoreline to the west of the Great Creek outlet structure due to this sheltering.

Additionally, the peninsula to the east of Milford Harbor provides wave sheltering for this approach direction. Therefore, there is minimal sediment transport under these conditions. Due to the wave approach direction, this energy generally transports sediment to the west (positive sediment flux), with an average rate of approximately 5,000 cyd/yr.

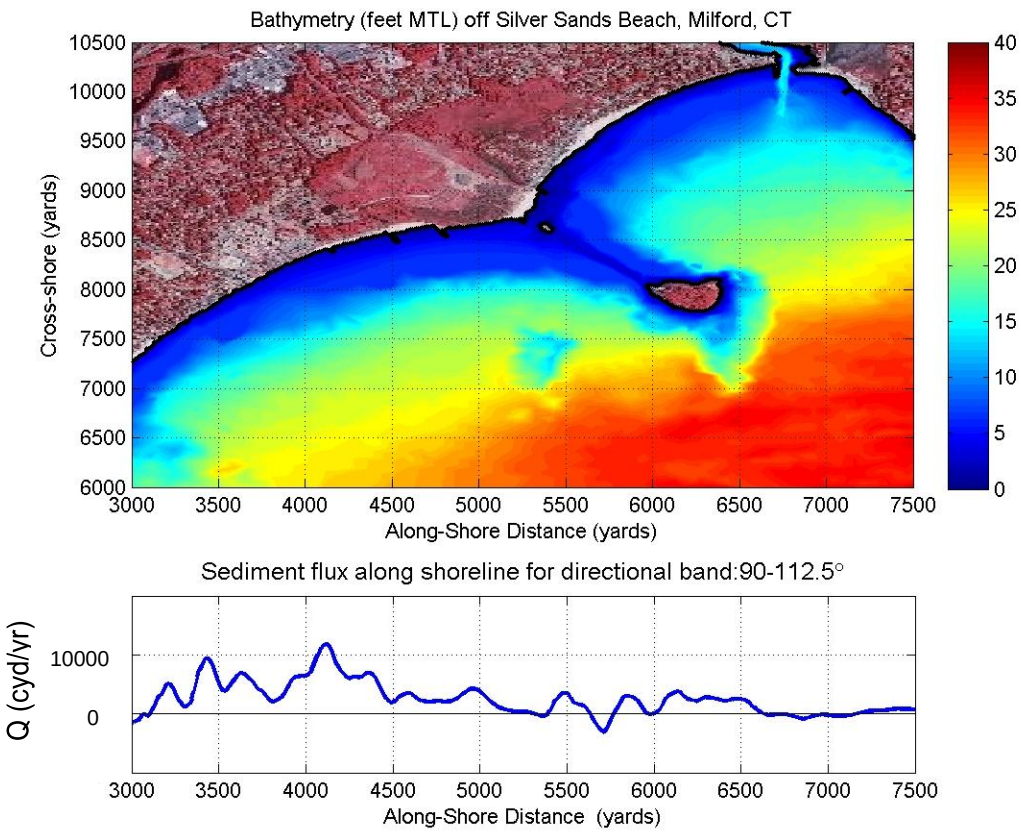

Figure 5-2. Sediment transport model results for an east wave approach (90 to 112.5 degrees).

Figure 5-3 represents a more shore perpendicular wave approach, and produces similar sediment transport results to the south-southwest case presented in Figure 5-1. The sediment transport rate is predominantly from west to east for this approach direction (except near the groins); however, the rate and direction fluctuates significantly along the coastline. In general, waves approaching from the south have a sufficient angle to drive the hydrodynamics necessary for sediment transport, but at a reduced average rate behind the sheltering of Charles Island and the bar.

Appendix C contains results for the remaining average annual wave approach directions. The figures in Appendix C indicate similar sediment transport regimes with southeast transport being dominant for wave spectra centered about a west or southwest approach and northwest transport being dominant for wave spectra centered about a south or southeast approach.

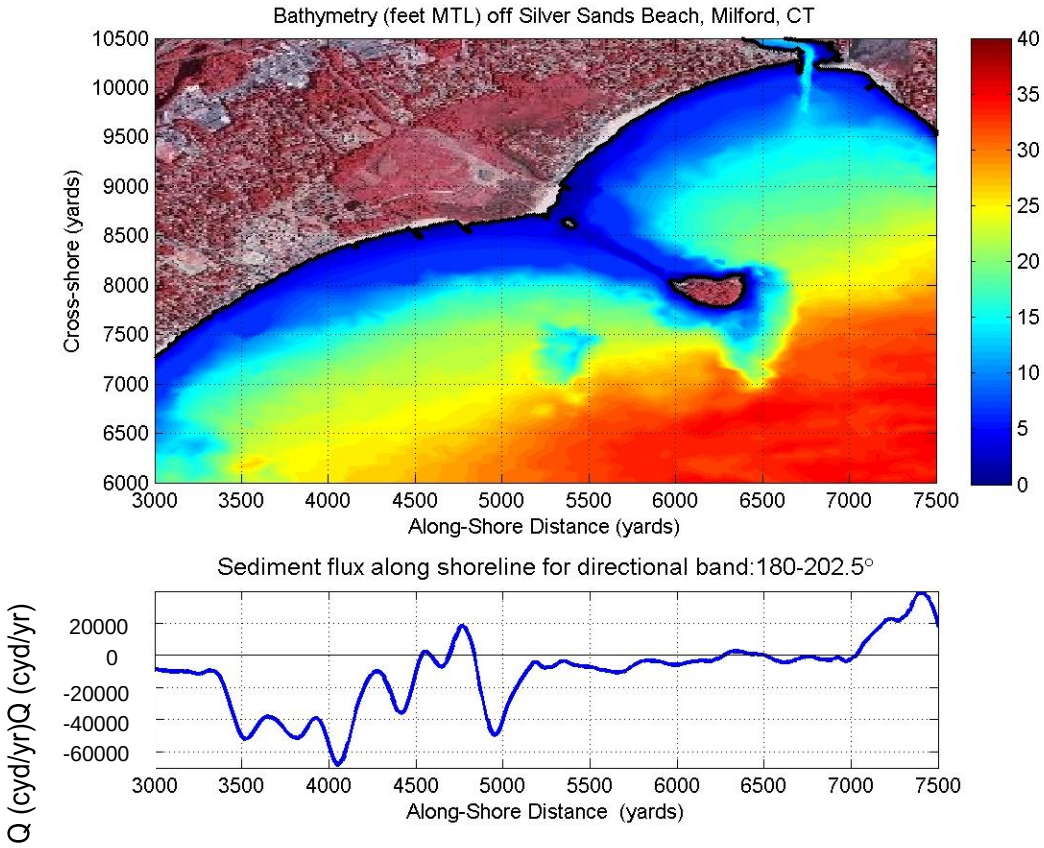

Figure 5-3. Sediment transport model results for a south wave approach (180 to 202.5 degrees).

Gross sediment transport rates vary significantly for the various average annual approach directions, and reach instantaneous maximums of approximately 60,000 to 70,000 cyd/yr. The magnitudes of the gross sediment transport rates provide an indicator of the wave energy associated within each wave approach direction, but must be accurately combined to determine the net transport rate. Table 5-1 summarizes the longshore transport for each of the annual approach directions. The first and second columns present the wave approach direction, the third column presents the percent occurrence of the spectral wave approach, and the fourth column presents the gross longshore sediment transport rate (i.e., if that wave approach occurred over the entire year). Positive values in Table 5-1 indicate transport to the west, while negative values indicate transport to the east. The magnitude of the gross longshore transport rates provides an indication of the wave energy associated within each wave approach direction.

The unique coastal features (e.g., islands) and anthropogenic structures in the vicinity of Silver Sands limit the amount of spectral wave energy, and therefore the associated sediment transport rates. The amount of gross sediment transport from the east to the west is limited by 1) the wave sheltering from Charles Island and the bar, and 2) the smaller percent occurrence over an average year. The waves from the west (causing eastward sediment transport) are dominant in both wave energy and percent occurrence.

Therefore, it is not surprising that the net sediment transport rate over an average year is from the west to the east.

Table 5-1. Gross longshore sediment transport (yards3/year) for each of the average annual approach bins.

Directional Bin (0º=N)	Wave Approach Direction	% Occurrence	Gross alongshore sediment transport rate [yards^3/year]
90 to 112.5	E	8.44	5,268
112.5 to 135	ESE	8.68	4,441
135 to 157.5	SE	6.74	-2,606
157.5 to 180	SSE	9.04	-6,706
180 to 202.5	S	15.09	-10,387
202.5 to 225	SSW	15.06	-3,820
225 to 247.5	SW	11.68	-773
247.5 to 270	WSW	11.84	-55
Total	**-**	**100**	**-14,638**

The various wave scenarios (Appendix C) can also be combined to represent an average annual year of wave climate. Using the percent occurrence of wave approach (Chapter 4.0, Table 4-1), the average annual approach directions were normalized and combined to determine the net longshore transport rate. Figure 5-4 presents the nearshore bathymetry (upper panel) and the average yearly sediment flux (lower panel). The arrows on the plots indicate the direction of sediment transport. The plus signs indicate areas where sediment is likely to accrete given the potential sediment transport processes, while the negative sign indicates area where sediment is likely to erode due to a divergence in the sediment flux rate. The sediment flux indicates an average annual longshore transport rate to the east of approximately 15,000 yd^3/yr. However, the magnitude of the transport varies throughout the domain. For example, near the western edge of the Silver Sands region, the annual average rate of transport is higher (20,000 yd^3/yr to east) than adjacent areas and sediment is transported out of the area leading to erosion. In the vicinity of the groins directly west of the bar, the sediment flux changes from east to west (positive). The two groins just west of the bar both show accretion on their eastern (updrift) sides, which supports the model results for this area. This results in a convergence of the sediment transport rate, as the sand enters from the west and east. Near the area where the bar connects to the shoreline, the sediment transport rate quickly reverses from westward to eastward, and east of the bar the sediment flux continues to be east (negative), resulting in a more erosional area as sand leaves to the east. At the center of the clenulate shaped bay of the Gulf, the sediment transport rate drops to near zero, indicating that sediment is generally deposited in this area. This region corresponds to the depositional area near the center of the Gulf shoreline, which is a more stable to accretionary shoreline, and is the former location of the Great Creek outlet that experienced continual shoaling. These results of the sediment transport model concur with the historical sedimentation problems that occurred at the former inlet location. At

the far eastern end of the model domain, the sediment transport rate is westward towards Milford Harbor.

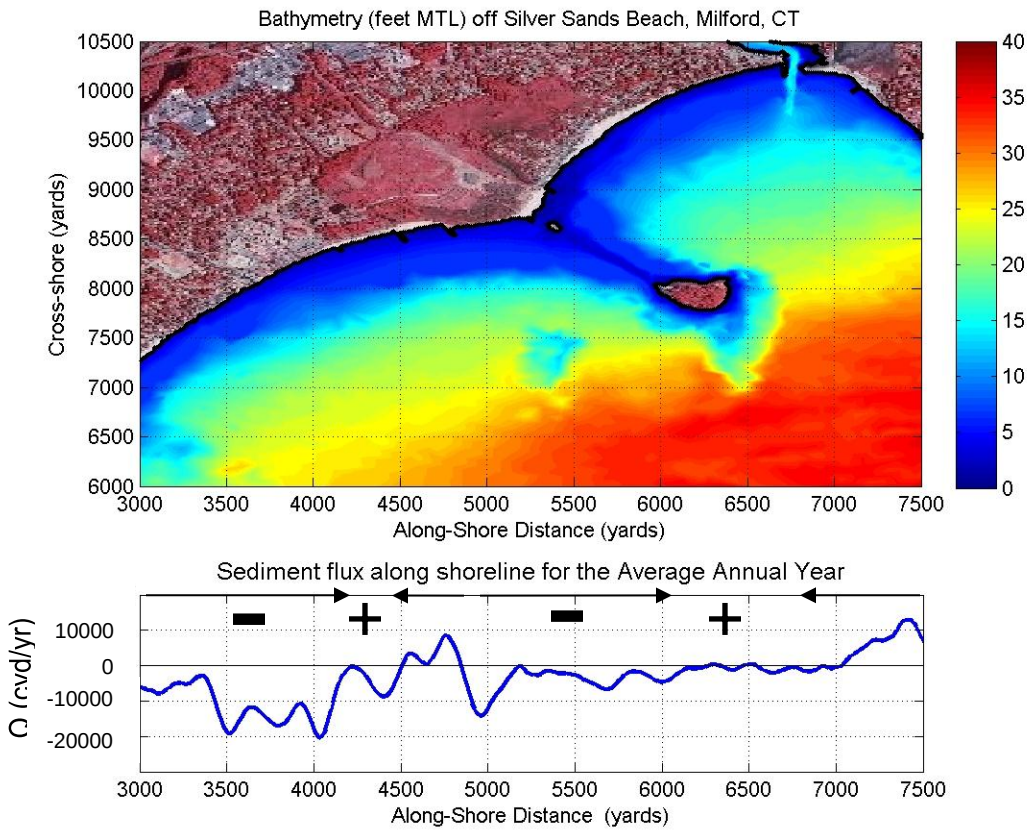

Figure 5-4. Annualized sediment flux for Grove Beach Point.

6.0 INLET STABILITY

6.1 INLET STABILITY ANALYSIS

A tidal inlet is stable if its cross-sectional area and planform location do not change significantly as a function of time. Stability occurs when the scouring ability of the tidal currents in the cross-section of the inlet is balanced by the sediment load carried into the inlet. Sand can be brought into an inlet through a variety of sources and means. On flood tides, flows into the inlet carry in sediment-laden water from the littoral system. If this sediment is more than can be flushed from the system by the tidal flux, then the inlet will be prone to shoaling. For Great Creek inlet, which is confined to a fixed cross-sectional area by the outlet structure, an analysis was performed to determine the inlet hydraulics and stability of the current inlet using the observed velocities, tidal flux, and tides (Chapter 3), as well as the results of the sediment transport modeling (Chapter 5).

Escoffier (1940) developed a method for evaluating the stability of an inlet based on the relationship between maximum velocity through the inlet and the cross-sectional area of the inlet. When a cross-section of an inlet is small, then the maximum velocity is small. This velocity will increase with increasing cross-sectional area, and then decreases. As the inlet cross-sectional area increases, the influence of friction becomes less, until the inlet cross-section becomes large enough that the velocity decreases since the entire bay or marsh fills completely during a tidal cycle. The variation in this velocity, as a function of cross-sectional area, for Great Creek is shown in Figure 6-1 as the light blue line and is based on the observed velocities and water levels in the system. Escoffier (1940) determined that there was a stable maximum tidal velocity for every inlet that would scour out any excess sand carried into it by wind and waves, given by the solid black line in Figure 6-1 for Great Creek. The two intersecting points of these lines yield an unstable equilibrium cross-section (AC-1), and a stable one (AC-2), and every natural inlet migrates to one of these equilibrium cross-sections. Escoffier (1940) contended that the unstable equilibrium cross-section (AC-1) would have the required velocity for stability, but any perturbation would cause the inlet cross-section to grow smaller or larger.

Great Creek inlet is confined to a specific cross-sectional area by the outlet structure. This cross-sectional area is shown in green on Figure 6-1 and indicates that the cross-sectional area is slightly undersized compared to the stable intersection (AC-2). However, given that the training walls inhibit a significant portion of the littoral sediment load to the inlet, the current inlet scouring ability is more than adequate to keep the channel from shoaling. Therefore, there is little maintenance (clogging) of the inlet; however, this comes at a cost of increased erosion downdrift (to the east) of the outlet structure due to the interception of the natural littoral drift. The stability of the inlet is further discussed in the next section (6.2) and evaluated for potential alternative in Chapter 7.

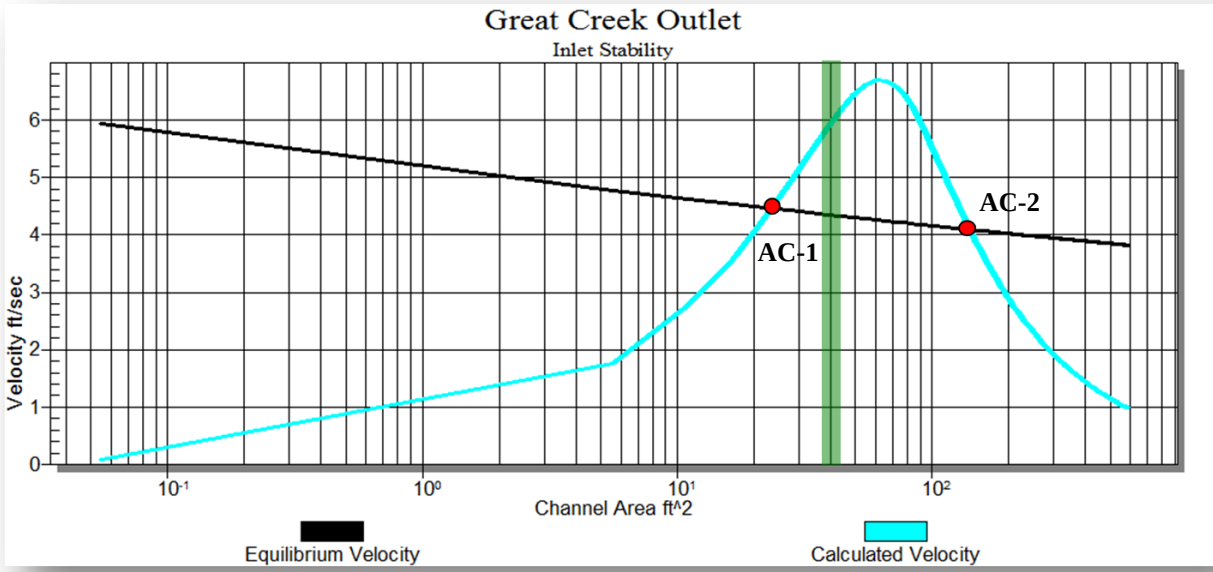

Figure 6-1. Maximum velocity in the Great Creek inlet as a function of cross - sectional area.

6.2 CROSS-SHORE DISTRIBUTION OF SEDIMENT TRANSPORT

In order to determine the stability of the Great Creek outlet, as well as the amount of sediment that was estimated to be bypassing the outlet structures to the downdrift beaches, the cross-shore distribution of the longshore transport was evaluated using relationships proposed by Longuet-Higgins (1970, 1970a). Using the cross-shore distribution, the effect of a shore-perpendicular structure (e.g., the outlet structure training walls) on reducing or increasing the longshore sediment transport can be estimated.

The cross-shore distribution of longshore transport can be determined using a theoretical radiation stress approach (Longuet-Higgins and Stewart, 1962). This momentum based theory describes the energy imparted on the bottom of a nearshore breaking zone by shallow water waves. When shallow water waves break at an angle that is not perpendicular to the coastline, the result is a net force that pushes a parcel of water in the direction of the oblique angle. In the case of a series of multiple waves breaking at a similar angle; a net current results that continually forces water along the shore (or alongshore). The total volume flow rate, Q, is given as a function of velocity, v_o, as

$$Q = \frac{1}{3s} {h_B}^2 v_0 = \frac{1}{3} h_B |x_B| v_0 \qquad (6\text{-}1)$$

where h_B is the depth of water at the breaker line, s is the slope of bottom, and x_B is the normalized distance to the breaker line.

Horizontal mixing is the result of waves breaking at different locations and wave-induced eddies varying the profile of the cross-shore velocity distribution. To account for this variability due to mixing, a quadratic equation is used to create a typical cross-shore flow profile. The shape of this new function is dependent the known variability of the wave conditions and a horizontal eddy parameter.

This distribution is calculated based on site-specific physical processes data and modeling results, and is presented in Figure 6-2 for the Silver Sands area. The vertical axis presents the water flux in cubic feet per second (used as a proxy for the sediment flux distribution), while the horizontal axis presents the normalized cross-shore distance. The distribution can then be applied to assess the impact of the existing outlet structure, as well as alternative outlet structure configurations, by determining the amount of littoral transport that may be intercepted by the structures.

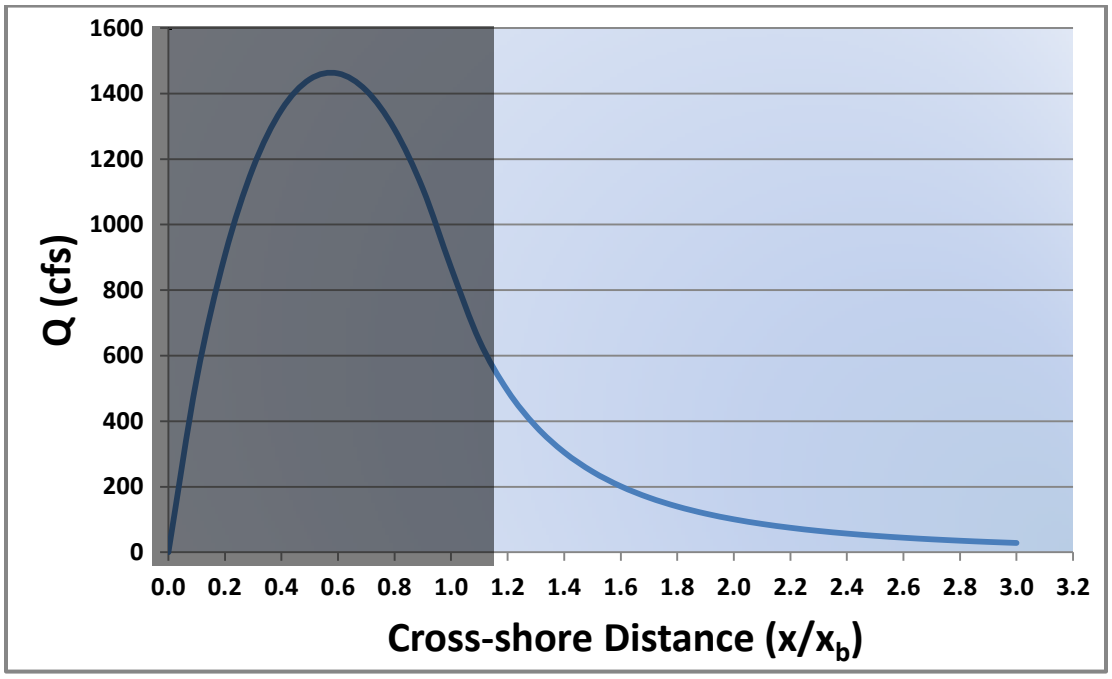

Figure 6-2. Cross-shore distribution of along shore flux for Silver Sands area. The dark gray area shows the cross-shore length of the existing training walls at the Great Creek outlet.

Assuming that the training walls are non-permeable, for existing conditions, only 17% of the existing littoral transport (approximately 2,600 cubic yards per year) is able to bypass the existing outlet structure.

In order to evaluate the tendency of sediment deposition in front of the Great Creek outlet structure, a stability analysis was performed to relate the sediment that is allowed to pass in front of the outlet structure (2,600 cyd/yr) to the tidal prism (a measure of the amount of tidal flushing in a marsh or bay). As discussed, stability is met when the sediment load arriving at the inlet balances the scouring ability of the inlet. The stability of the entrance to the marsh (whether the entrance has a tendency to close or remain open) can be

evaluated by relating these two parameters. Bruun and Gerritsen (1966) introduced a parameter Σ/Q for examining inlet stability, relating the tidal prism (Σ) to the annual rate of littoral drift (Q). They presumed that littoral drift must play a role in the stability of an inlet since coasts with higher transport rates should have different inlet sizes than those serving similar sized bays on a low drift coast. Bruun and Gerritsen (1966) found that inlets with values of $\Sigma/Q > 300$ appear to be stable, while those with $\Sigma/Q < 100$ are more likely to be unstable. The tidal prism was determined from the site specific data observations (Chapter 3). At this location, with the training walls in place, the Σ/Q ratio was calculated to be approximately 450, indicating adequate flushing ability. The stability assessment for existing conditions will be compared to other structural alternatives in Chapter 7.

7.0 ALTERNATIVE ANALYSIS

Since the current Great Creek outlet structure is in need of repair (Figure 2-7), the primary purpose of this engineering alternatives analysis is to assess options for an improved replacement design for the failing Great Creek outlet structure that will maintain the outlet's ability to drain freshwater discharge, flush sediment from the channel to minimize potential shoaling, and provide a robust design that can contend with the dynamic coastal processes and require minimal maintenance. In addition, the replacement outlet structure design is evaluated to optimize the amount of sediment that can naturally bypass the outlet to feed the downdrift eroding beach resource. A secondary set of supplementary alternatives evaluates options that could be considered to improve the condition of the downdrift beach, which has experienced historic erosion.

An alternatives analysis is the basis for determining the optimal solution and assessing potential impacts, both physical and environmental. A variety of factors are considered when evaluating the various alternatives (e.g., cost, feasibility, performance, environmental impacts, constructability, etc.), with the overall objective focused on selecting the optimal solution. As such, the goal of the assessment is to evaluate reasonable, practicable, and feasible alternatives that will achieve the goals and objectives of the project, while minimizing the short and long-term adverse effects, if any.

Decisions regarding management of the Great Creek outlet structure, as well as the shoreline at Silver Sands, can only be made after a thorough evaluation of both the coastal processes and available erosion mitigation alternatives. The following chapter presents and evaluates a variety of outlet structure alternatives, and also established coastal engineering methods for erosion mitigation, as well as several less traditional approaches that were considered in the assessment of erosion.

7.1 OUTLET STRUCTURE ALTERNATIVES

Seven (7) alternatives, in addition to the no action alternative, were considered in the evaluation of potential outlet structure alternatives. The alternatives that were considered included:

- No action
- Replacement in kind
- Removal of the training walls
- Shortening of the training walls
- Tapering the elevation of the training walls
- Culvert extension
- Culvert extension with training wall shortening
- New outlet structure for enhanced tidal restoration

7.1.1 No Action

The no action alternative implies there would be no change to the present conditions at Silver Sands. Under this action the inlet would remain stable due to reduced littoral

sediment flux; however, it would continue to be a significant littoral barrier to sediment transport along the beach. As a result, only 17% of the total sand would bypass the structure and downdrift beach erosion would continue. In addition, this alternative does not address the primary requirement, which is repair of the failing outlet structure training walls. The training wall structures may have exceeded their service life and are a safety liability in their current state. This alternative is considered unacceptable as it does not address the failing infrastructure, consists of an ongoing maintenance concern, does not reduce downdrift impacts, does nothing to mitigate beach erosion, and does not address the safety liability. Therefore, the "no action" alternative is not recommended for further consideration.

7.1.2 Replacement in Kind

Replacement in kind would involve replacing the existing training wall structures with new ones of the same dimensions and likely improved design and components. Inlet stability would be maintained, but there would be no improvement to sediment bypassing so downdrift impacts would also remain the same. The permitting requirements for this option would be minimal, and the engineering service life would be expected to be 25 -50 years.

New components could be used in the culvert replacement to allow for adaptive management. For example, adjustable panels for the training walls could be utilized such that the height and length of the structure could be adaptively managed based on the performance of the inlet and/or beach. Panels could be removed to allow more sediment to bypass the inlet or added to reduce inlet shoaling and promote inlet stability as sea level rises or other changes occur.

This alternative is not recommended since it does not reduce the impact of the structure on the downdrift beach and simply maintains the status quo at the site.

7.1.3 Removal of Training Walls

This alternative would completely remove the training walls from the outlet structure, leaving just the existing box culvert on the beach. This would allow a significant amount of sediment to bypass Great Creek outlet. Approximately 90-100% of the littoral drift, or 13,000 to 14,500 cyd/yr, would bypass the outlet and mitigate downdrift erosion (increase downdrift accretion). However, this also means that the Great Creek inlet/outlet would become unstable, shoal, and be prone to sedimentation and closure. The stability parameter, Σ/Q, would be well under 100 and indicate an unstable situation. Significant and consistent maintenance would be required of the inlet (sediment removal) and there would be increased potential for upland flooding. Therefore, this alternative is not recommended.

7.1.4 Shorten Training Walls

Instead of completely removing the training walls, this alternative evaluates shortening the training walls to allow for increased sediment bypassing while still maintaining inlet stability. Using the inlet stability and the cross-shore distribution of alongshore sediment transport analyses (Chapter 6), the training walls could be shortened by 30 feet (Figure 7-

1) and still maintain a stable inlet, and possibly as much as 100 feet, which would put the inlet into potential instability given its current flushing ability. This would increase the sediment bypassing the outlet structure by approximately 28% (for a 30 foot reduction) to 73% (for a 100 foot reduction) and allow an additional 1,500 to 8,000 cy/yr to be transported to the downdrift beach. The new, shortened training wall could be replaced with adjustable panels over the seaward 70 feet to allow for adaptive management of sand bypassing and inlet stability. Permits should be relatively easy to obtain due to reduced impact to beach and reduced structural footprint. Figure 7-1 shows a rendering of the Great Creek outlet for the training wall shortening option. The rendition also includes a beach nourishment project on the downdrift side of the structure. This alternative is recommended as one of the final alternatives for consideration.

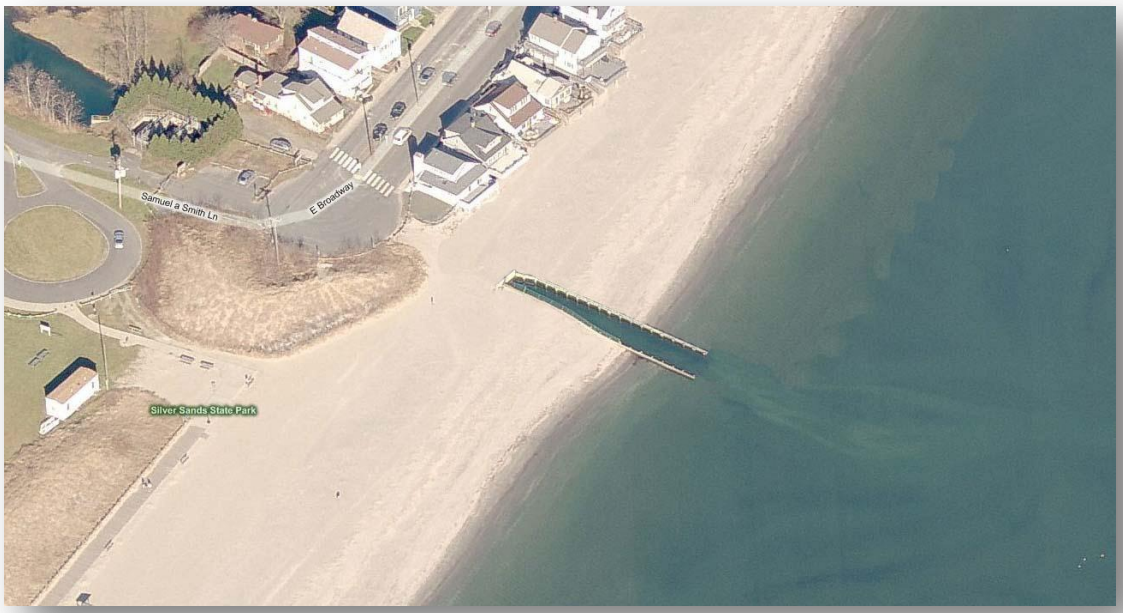

Figure 7-1. Rendering of potential Great Creek outlet structure for training wall shortening alternative with length of the training wall reduced by 30 feet. The rendition also shows the outlet structure alternative coupled with a downdrift beach nourishment project.

7.1.5 Tapering Elevation of Training Walls

In addition to shortening the training walls, they could be rebuilt to be tapered towards the ends to allow more sand bypassing during high tides. The tapering option is considered to allow slightly more sediment to bypass the structure during high tides, and also represents stages that could be implemented in an adaptive management approach for removing flash panels from the outer portions of the training walls. Figure 7-2 shows a rendering of this alternative, which includes a shortening of 30 feet and a tapering over the seaward 70 feet. The rendition also includes a beach nourishment project on the downdrift side of the structure. This would increase the sediment bypassing the outlet

structure by approximately 30% to 73% (for a 100 foot reduction and no tapering) and allow an additional 1,800 to 8,000 cy/yr to be transported to the downdrift beach. Permits should be relatively easy to obtain due to reduced impact to beach and reduced structural footprint. This alternative is recommended as one of the final alternatives for consideration.

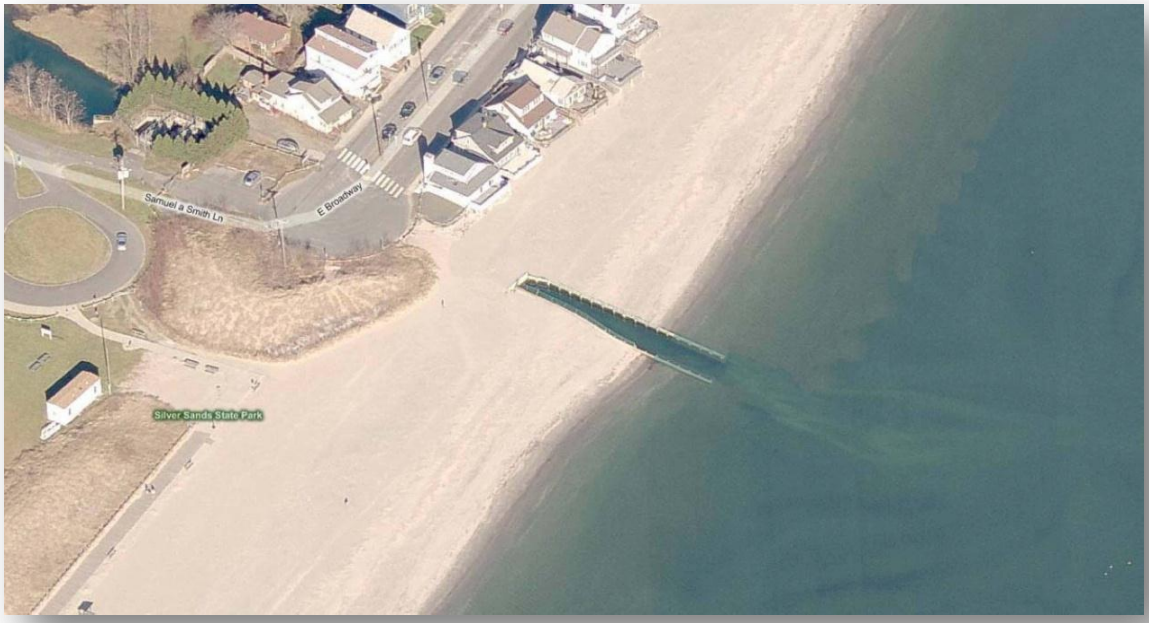

Figure 7-2. Rendering of potential Great Creek outlet structure for training wall shortening and tapering alternative with length of the training wall reduced by 30 feet and tapering over the seaward most 70 feet. The rendition also shows the outlet structure alternative coupled with a downdrift beach nourishment project.

7.1.6 Culvert Extension

Rather than modifying the training walls, another approach for modification of the outlet structure is extension of the culvert and elimination of the training walls entirely. Based on the results presented in Chapter 6, this would require an extension of the culvert by 120 feet in order to maintain inlet stability. The culvert would be the same dimensions and would maintain the same approximate invert elevation. Figure 7-3 shows a rendering of this alternative. The rendition also includes a beach nourishment project on the downdrift side of the structure. This would increase the sediment bypassing the outlet structure by approximately 45% and allow an additional 3,800 cy/yr to be transported to the downdrift beach. This alternative is not recommended due to:

- Permits for this alternative may be more difficult to obtain due to the increased footprint of the structural components and the additional hardscape added to the natural beach system.

- More difficult maintenance associated with a closed conduit if sedimentation occurs in the intertidal zone of the culvert.

- Increased safety concerns that arise from having an enclosed structure on a beach. A closed conduit in a tidal setting, especially one that resides on a recreational beach, can be a potential increased risk due to human interaction with the structure. Recently, there have been a number of cases where individuals have gotten trapped inside a conduit during periods of the tidal cycle that have strong currents and high water resulting in injury or death. Appropriate engineering safety measures would need to be incorporated into the design to improve the safety of this alternative

- Loss of flexibility to modify the design after installation. Due to the dynamic nature of the coastal environment, it may be desired to have the ability to easily modify the design in the future. For example, the unpredictability of storm events, sea level rise, changes in offshore bathymetry that effect wave transformations, etc. all create variations in the dynamics that change the design conditions. Therefore, in coastal designs, the ability to adapt to future changing conditions is sometimes a desired component.

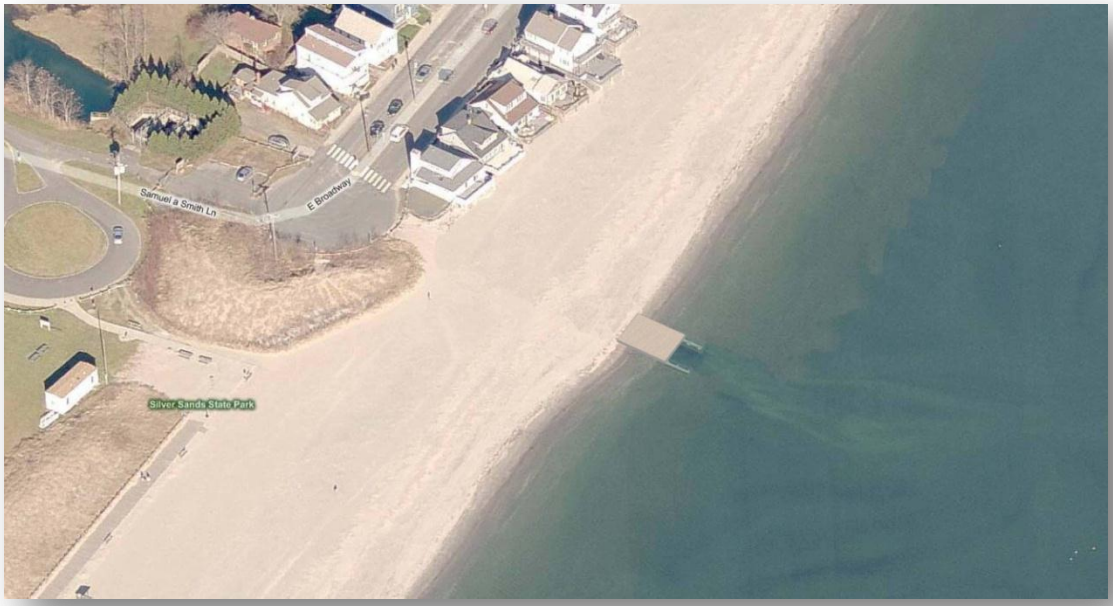

Figure 7-3. Rendering of potential Great Creek outlet structure for culvert extension alternative. The rendition also shows the outlet structure alternative coupled with a downdrift beach nourishment project.

7.1.7 Culvert Extension with Training Wall Shortening

This alternative combines the culvert extension with the training wall shortening alternative and utilizes the results from the Chapter 6 analysis to determine the optimal lengths of each structural component. This includes:

- Culvert extension of 80 feet seaward from the current culvert end. This covers a portion of the littoral zone that would cause inlet instability and shoaling.
- Training walls extending from the end of the new culvert extension for an additional 70 feet. The training walls should be modular, consisting of adjustable components (e.g., flash panels) that could be removed or modified in an adaptive management approach. The training walls could then be further shortened or tapered as part of an adaptive management approach.
- A shore parallel wall positioned at the crest of the seaward end of the culvert for safety considerations and to eliminate overtopping of sediment into the channel. . The culvert extension will require a barrier at the crest of the culvert entrance such that during storm events, sediment is not directly deposited into the channel resulting in potential blockage of the channel. This barrier could also serve as a safety/vandal measure so individuals cannot directly access the culvert from the beach. There is a similar feature at the seaward end of the existing culvert (see Figure 2-7, 2-8, and 2-9).
- The net distance of the outlet structure would be 30 feet shorter than the existing structure and could be made to be 100 feet shorter than the existing structure as needed.

This would increase the sediment bypassing the outlet structure by approximately 28% initial, and up to 73% through adaptive management of the structure. This alternative is recommended as one of the final alternatives for consideration.

7.1.8 New Outlet Structure for Enhanced Tidal Restoration

This alternative considered reconfiguration of the entire outlet structure, including the culvert and tidal control devices, to allow for increased tidal exchange and subsequently increased flushing ability. Based on the water surface level observations presented in Chapter 3.0, full tidal exchange would allow the inlet to naturally flush itself at this geographical location, while providing full sediment bypassing to the downdrift beaches. However, returning the inlet to its natural condition (pre-development) would also result in significant upland flooding of existing infrastructure (homes and roadways).

The water surface elevation data (Figure 3-9) do indicate that tidal range and prism could be increased by approximately 35% by redesigning the entrance to the system (e.g., larger culvert, etc.) and still maintaining flood control (maximum elevation would be lower than the lowest street elevation). However, this increase alone would not eliminate the need for training walls required to ensure inlet stability. With the 35% increase in tidal prism, training walls could be shortened by 100 feet, the inlet would remain at the same level of stability with minimal maintenance as it has historically, and an additional 5,300 cyd/yr of material could be naturally bypassed to the downdrift beach. This option would

require significant permits to redesign the entire entrance and likely cost well over $1,000,000 for design and construction of a new entrance.

Although complete restoration of the tidal range would increase the tidal prism such that no structures or culverts would be required and a natural channel could be utilized. However, due to the existing upland infrastructure this is not warranted. Although the potential available increase in tidal prism would allow for a reduction in training wall length and improved sediment bypassing, the limiting upland infrastructure elevations do not allow the increase to be large enough to warrant the cost and expense of a complete redesign for the additional 35% tidal prism increase. Therefore, this alternative is not recommended.

7.2 BEACH ALTERNATIVES

Some preliminary alternatives were also considered for mitigating the erosion on the downdrift beach that would be supplementary to the outlet structure modification. These alternatives are not exhaustive of a beach restoration assessment; rather provide some basic options that could be considered for beach enhancement. These alternatives are secondary to the primary goal of restoring the outlet structure, and therefore are not required and are provided as optional, supplementary alternatives, if selected.

7.2.1 Beach Nourishment

One of the primary causes of coastal erosion is a deficit of sediment within the coastal littoral cell. This is a key factor in the erosion downdrift of the Great Creek outlet structure, which has impeded the natural littoral drift of sediment. To offset this deficit, nourishing the beach with compatible sediment placement is a logical means for improving the longevity of the shoreline, where such a project is economically feasible. Beach nourishment does not stop erosion. Rather, the damage to landward properties is postponed by extending the shoreline toward the ocean. As such, periodic renourishment must be anticipated. At a site like Silver Sands, the beach can provide both a damage protection and a recreational benefit.

Beach nourishment is typically the most non-intrusive technique for coastal protection and involves placing sand, from an offshore or upland source, in a designed template on an eroding beach. Figures 7-4 and 7-5 present examples of beach nourishment projects being constructed. Beach nourishment is intended to widen the beach, as well as provide added storm protection, increased recreational area, and in some cases, added habitat area. Although nourished sand is eventually displaced alongshore or transported offshore, the nourished sand that is eroded takes the place of the upland area that would normally have been lost or eroded during a storm event. Therefore, beach nourishment serves a significant role in storm protection. In addition, beach nourishment is the only alternative that introduces additional sand into the system. For coastlines with a dwindling sediment supply, such as Silver Sands, this is critical for long-term success. Solutions that do not involve beach nourishment typically involve rearranging the existing sand in a manner that will only benefit a portion of the beach.

Figure 7-4. Beach nourishment project under construction.

Figure 7-5. Beach nourishment project under construction in Virginia Beach, VA (photo courtesy of Virginia Beach).

Environmental concerns with beach nourishment projects include the potential for decreased water quality when sediments are dredged and deposited, and disturbing

natural habitat when removing or depositing the dredged material. These concerns can be addressed by adhering to dredging time windows that avoid periods of shellfish, finfish, and shorebird activity. Grain size compatibility between the borrowed and native beach sediments should be maximized in order to avoid disturbance of offshore resources such as shellfish and submerged aquatic vegetation, as well as to increase the lifespan of the nourished beach. For example, large differences in grain size between the native and borrow material may lead to changes in beach slope through natural adjustment of the new grain size introduced to the beach. This change in beach slope, as well as the change in grain size directly, may negatively influence the offshore resources.

The many benefits of beach nourishment, and the ability to control potential negative environmental impacts with careful design and planning, make beach nourishment a viable alternative for the Silver Sands shoreline. A beach fill project for this area would mitigate the on-going erosion, improve storm damage prevention and flood protection to infrastructure, and improve the recreational resource of the beach.

A successful beach nourishment project consists of more than simply placing sediment on a beach. Beach nourishment projects are engineered. A beach nourishment template, which consists of numerous design parameters, is based on the characteristics of the site and the needs of a project and needs to be optimized. Every beach nourishment design is unique, since different beaches in different areas have different physical, geologic, environmental, and economic characteristics, as well as different levels of required protection. The design must consider climatology, the shape of the beach, type of native sand, volume and rates of sediment transport, erosion patterns and causes, waves and water levels, historical data and previous storms, probability of certain beach behaviors at the site, existing structures and infrastructure, and past engineering activities in the area. As such, beach nourishment design must identify the coastal processes at the site. Typically, computer models (Chapters 4 and 5) are used to help design the nourishment template.

The structure of a nourishment template is designed to yield a protective barrier that also provides material to the beach. A higher and wider beach berm is designed to absorb wave energy. Dunes may need to be constructed, or existing dunes improved, to reduce damage, including potential upland flooding, from storms. Figure 7-6 depicts a beach berm and dune on a typical beach profile to illustrate the components of a beach profile. Nourishment length, berm height and width, dune height, and offshore slope are critical elements of a beach nourishment design. Periodic renourishment intervals are also usually a part of the nourishment design. As a rule of thumb, if renourishment is required in less than 5 years, then the nourishment is probably not cost-effective. If renourishment is required between 7-10 years, then a nourishment project is likely cost-effective. The renourishment interval will vary based on the initial design, wave climate, sand used, number and types of storms, and project age. In addition, beach nourishment is not an exact science; variables and uncertainties exist. Actual periodic renourishment intervals may differ from planned intervals based on conditions at the nourished beach and the frequency and intensity of storms from year to year. A wide variety of beach nourishment designs were tested in the numerical models, to determine the optimal, yet reasonable size and cost, design template. The following parameters were determined:

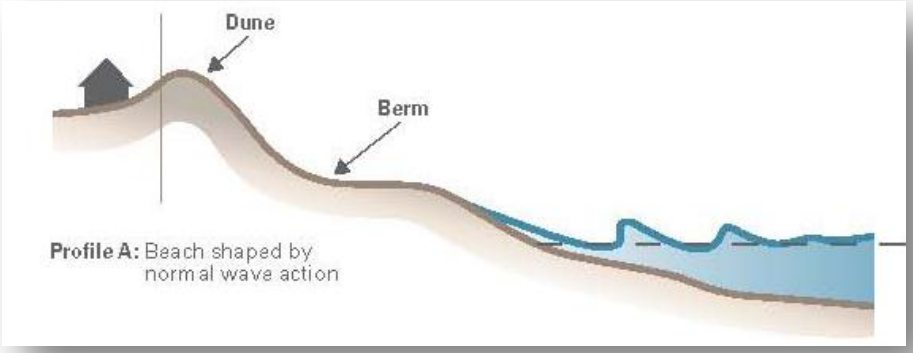

Figure 7-6. Typical beach profile and features to illustrate components of a beach profile (Coastal Engineering Manual, 2003). Not representative of Silver Sands Beach.

- Nourishment Length – A length of approximately 1,000 feet, and spanning north from the Great Creek outlet structure, as shown in Figure 7-7.

Figure 7-7. Section of downdrift beach for proposed nourishment.

- Berm Width and Berm Height - Increasing the beach berm is defined by extending the natural berm seaward at a constant elevation. An example of

extending the berm width is shown in Figure 7-8. In reality, the beach width at MHW will typically be wider than the berm width due to the sloping nature of the beach profile and nourishment template design. The berm height simply refers to the elevation of the berm. The template consists of a berm with a height of 7 to 9 feet NGVD29 (depending on exact location along the beach). The result gives approximately 30 to 100 feet of total berm width and 75 to 150 feet of beach width.

- Offshore Slope - For all berm templates and length scenarios, an offshore slope of 1V:25H was incorporated. This offshore slope is steeper than the existing estimated offshore profile, but is approximately the same as the intertidal zone slope. Distribution of fill material over the beach face is most effective when the fill forms a profile slightly steeper than the expected equilibrium profile, and the planform limits of the fill are tapered (Dean and Dalrymple, 2002). Initial erosion of the exposed recreational beach occurs as the equilibrium cross-shore profile develops. Some material is moved offshore, but is not lost, as it serves to dissipate wave energy naturally during winter months. A 1:25 slope allows for intersection of the existing profile at a reasonable distance offshore, provides a mild beach slope for wave dissipation, and can be reasonably constructed.

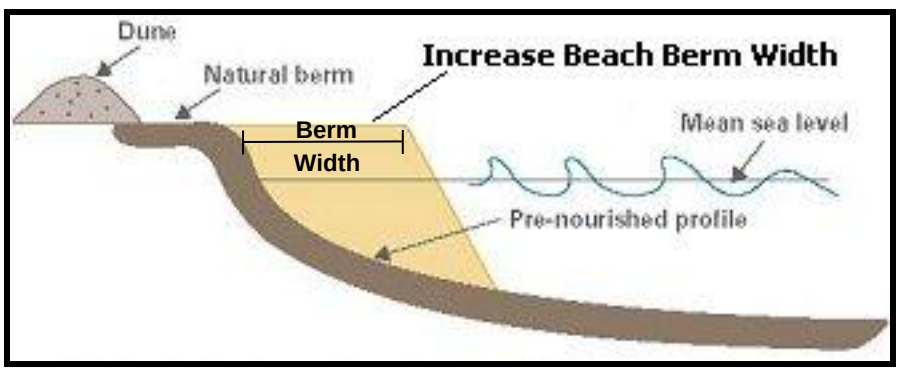

Figure 7-8. Example of increasing the berm width on a beach through beach nourishment (not to scale – vertical scale is exaggerated). This figure is intended to be illustration and not representative of Silver Sands Beach specifically.

- Grain Size –The native grain size material was presented in Chapter 3. A similar grain size to the native material was assumed for the fill material. Typically, the desired grain size for beach nourishment is the same mean grain size (and approximate gradation) as the native material, or slightly coarser. In fact, in most cases it is required (regulated) to place "beach compatible" material, which means the replenishment material needs to closely match the gradation of the native material.

  From strictly an engineering perspective, coarser material may be more desirable since it may disperse slightly slower; however coarser material may also be

undesirable from both a recreational and environmental habitat standpoint. Changes to the mean grain size of the nourishment material will also result in changes to the beach slope. For example, a coarser grain material will typically produce a steeper offshore slope, which can lead to other impacts. As such, the best nourishment material is usually considered to be the same gradation and mean grain size, or slightly coarser, than the native material. Ultimately, the sediment size, color, and distribution will be a function of the borrow site sediment (e.g. dredged material from an offshore borrow site, upland quarry sediment, etc.).

- Nourishment Volume – a total nourishment volume was estimated based on the profiles and surveys performed at Silver Sands (Fuss and O'Neill, 2011). A total volume of approximately 35,000 cubic yards is required to fill the recommended beach nourishment template.

Figure 7-9 presents representative cross-sections at Silver Sands (approximately in the erosional area north of the outlet structure) showing the proposed nourishment template (green line) and the existing profile (blue line) for the profiles northeast of the outlet structure. Figure 7-10 presents a visualization of the Silver Sands shoreline displaying a pre- and post-nourishment aerial. For the purposes of this report, the baseline used was feet north and south of the inlet for the alongshore, and a cross-shore baseline relative to the Mean Tide Level. For the design phase, a baseline and stations will be established. This is explained in detail in Chapter 3.

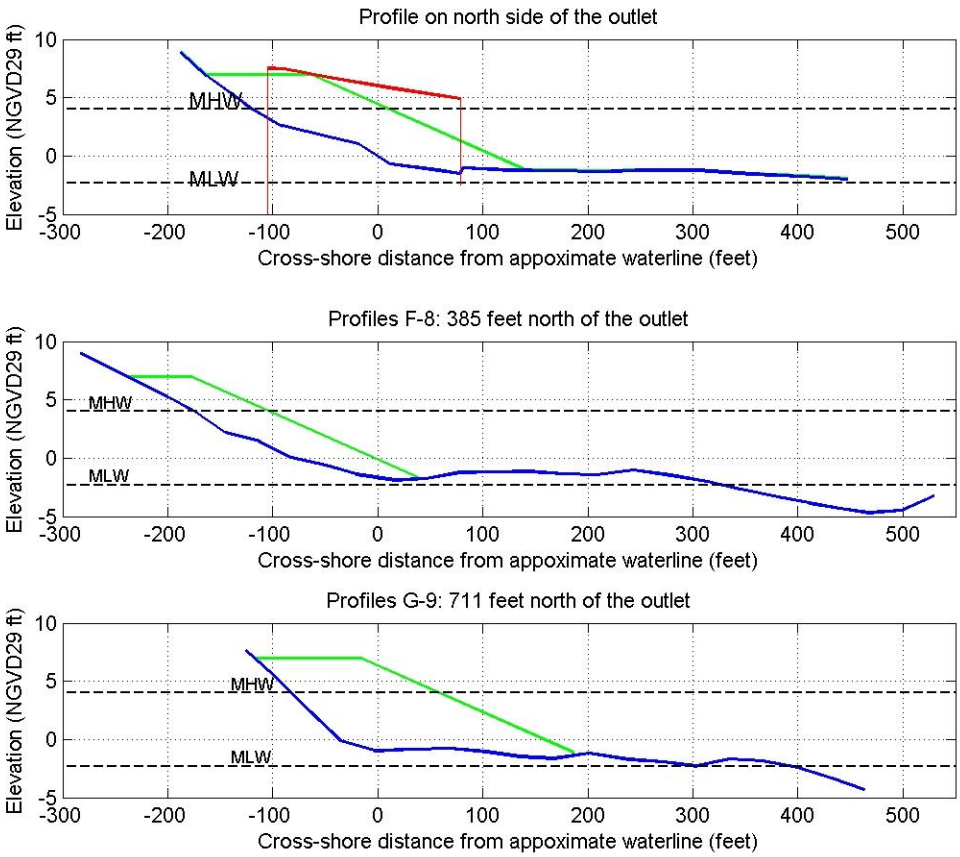

Figure 7-9. Existing (blue line) and proposed (green line) nourished cross-shore profile at Silver Sands down-drift (north) of the outlet structure.

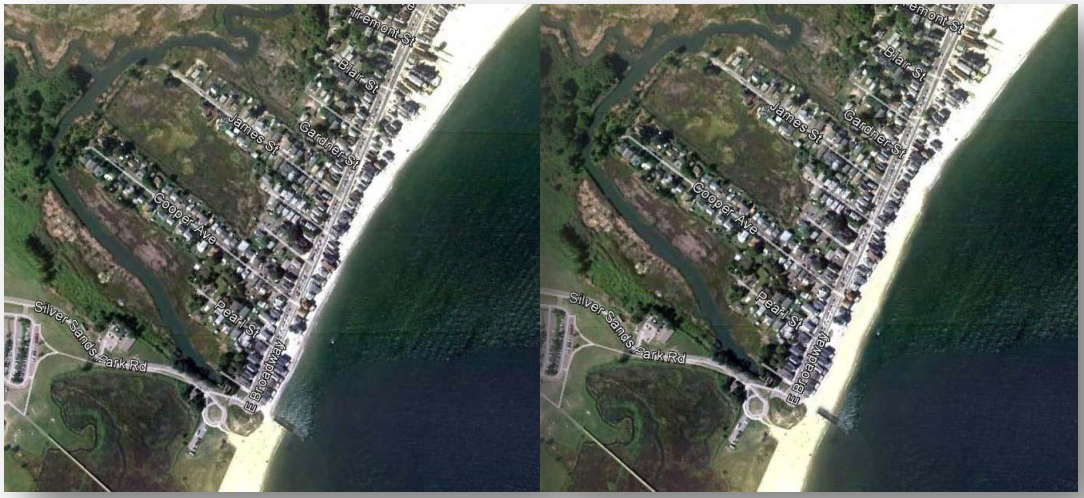

Figure 3-10. Existing conditions (left panel) and rendition of proposed nourishment (right panel).

The performance (service life) of the proposed nourishment was assessed using an analysis that combines the conservation of sediment equation with the linearized transport equation. This formulation, called the Pelnard-Considére (1956) equation (Equation 7-1), is used to obtain theoretical results to establish design and performance standards for the nourishments. A more detailed description of the derivation of the equations and their applications can be found in Dean (2002).

$$M(t) = \frac{2\sqrt{Gt}}{l\sqrt{\pi}}\left(e^{-\left(\frac{l}{2\sqrt{Gt}}\right)^2 - 1}\right) + erf\left(\frac{l}{2\sqrt{Gt}}\right) \tag{7-1}$$

where *M(t)* is the proportion of sand remaining in the placed location, *G* is the alongshore diffusivity parameter, *t* is time, and *l* is the project (nourishment) length. The alongshore diffusivity (Equation 7-2) is presented by *Pelnard-Considére* (1956).

$$G = \frac{KH_b^{5/2}\sqrt{g/\kappa}}{8(s-1)(1-p)(h_* + B)} \tag{7-2}$$

where *K* is the sediment transport coefficient, which is a function of sediment size, *B* is the berm elevation, H_b is the breaking wave height, h_* is the depth of closure, *p* is the *in-situ* sediment porosity (approximately 0.35 to 0.40), *s* is the sediment specific gravity (approximately 2.65), and κ is the ratio of wave height to water depth within the surf zone (approximately 0.78).

Since the material spreads over time, it is possible to evaluate the longevity of the nourishment by looking at the amount of material left in the project area. Subsequently, nourishment alternatives can be compared to one another based on their longevity. The service life of the beach nourishment can be based upon the percent of the initial beach nourishment left within the boundary of the initial fill. The percentage remaining will decrease with time, but that material is not necessarily lost from the system, it has just spread to regions outside of the original nourishment template. For example, sediment may have been transported offshore or along the beach. Therefore, although the sediment no longer falls within the initial nourishment template, it has not completely disappeared from the system.

Figure 7-11 presents the performance of the proposed nourishment at Silver Sands. The performance is expressed in terms of amount of material remaining in the initial template region, as a function of time. Results are presented for the nourishment alone (blue line), nourishment coupled with outlet structure modifications to allow bypassing at stable (red lines), and potentially unstable (green lines) levels (as detailed in Figure 7-12). All results were adjusted to include a background erosion rate. That is, in addition to the dispersion that is occurring, an additional amount is eroded due to the natural erosion of the beach. The percent of initial material remaining is presented along the left hand axis,

while the time in years is presented along the bottom axis. For example, after 2 years, approximately 50% of the initial fill volume is remaining for the nourishment only scenario. These performance results were used in the alternatives analysis.

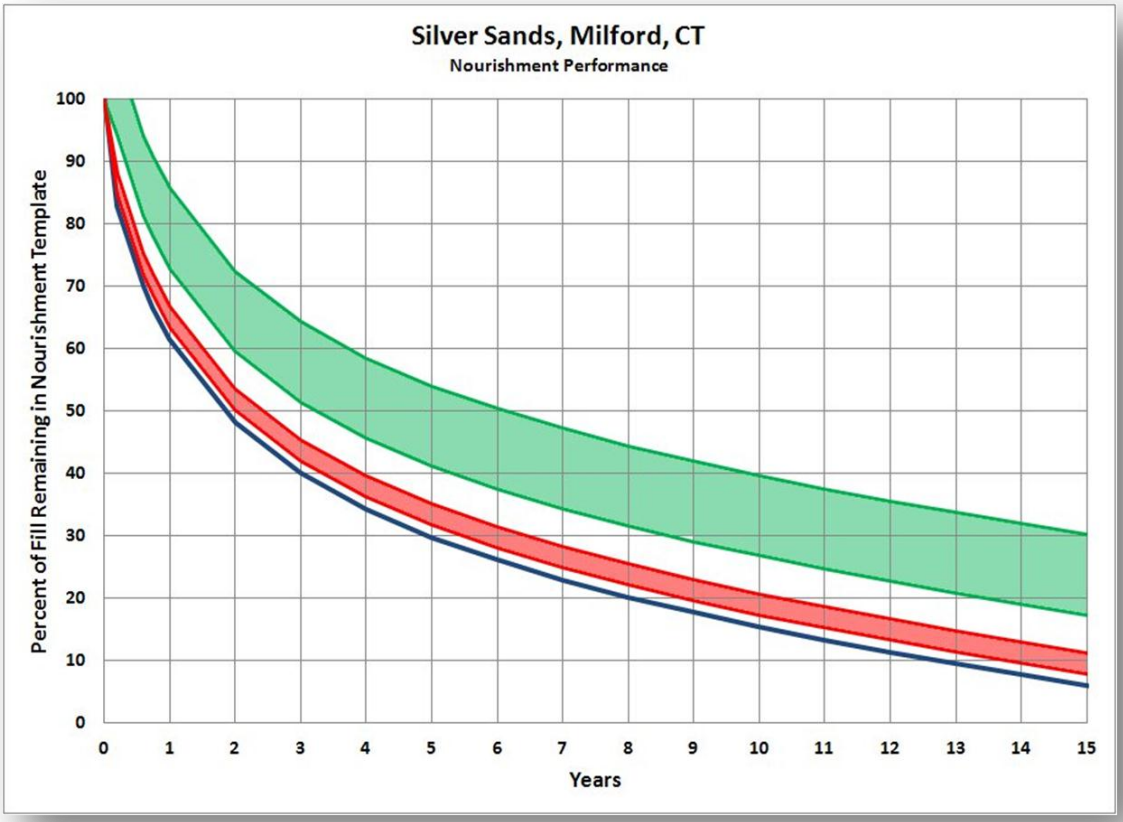

Figure 7-11. Estimated beach nourishment performance for the proposed nourishment at Silver Sands.

7.2.2 Manual Bypassing

This alternative would consist of physical transport of the sediment from the updrift side of the structure to the downdrift side of the structure on an incremental basis. The physical transport would likely consist of minimal construction equipment (e.g., bulldozer) to move the material. The interval of re-occurrence would be dependent on the amount of material moved each episode. This alternative is recommended as a feasible and easily implemented solution that has low initial cost. However, this solution would require at least annual maintenance and have re-occurring cost. Although this method can provide some additional material to the beach, manual bypassing alone would only provide a stop gap measure for the downdrift beaches. However, manual bypassing as a supplemental source may be warranted in the future.

7.2.3 Downdrift Groin or Groin Field

The addition of a groin or groin(s) on the downdrift (north) side of the Great Creek outlet was analyzed. Groins alone will not provide additional sediment to the shoreline, nor will they provide improved storm damage protection. However, appropriate placed and designed groin structures may help improve beach nourishment service life and performance and thus reduce the potential maintenance requirements of the beach nourishment. In many cases groins are included in a nourishment project to reduce the forces that cause rapid sediment losses and extend the time between renourishment events. In order for this alternative to be effective, it is likely that a groin field consisting of multiple groins may need to be built and span as much as 1,850 feet of shoreline north of the outlet. This alternative would have a lifetime of 50-75 years however it could cost $750,000 – $1,000,000 per groin and be extremely difficult to permit requiring the impacts on the down-drift shorelines to be assessed. This alternative is not recommended.

7.3 ALTERNATIVES SUMMARY

Ultimately, the goal of this evaluation is to identify the potential solution that can create a stable inlet and improved sediment bypassing at Great Creek outlet, while minimizing environmental impacts. Table 7-1 presents a summary of all the alternatives considered for both the primary purpose (outlet structure alternatives) and secondary goal (beach alternatives). The table presents the alternatives relative to a number of criteria, including inlets stability, impacts to downdrift beach, approximate cost, service life, maintenance, safety, and modular (or adaptive) design.

One key criterion in the evaluation of the outlet structure alternatives is the relationship between the inlet stability and the sediment bypassing. Figure 7-12 shows this relationship for the various outlet structure alternatives. Since the existing culvert and flow control system limits the tidal flux and flushing ability of the Great Creek system, as the sediment bypassing is increased, the potential for inlet shoaling and maintenance also increases. For example, completely removing the training walls would make the inlet unstable, while also allowing for maximum inlet bypassing. Extending the culvert would keep the inlet stable and allow a limited increase in the sediment bypassed. Shortening the training walls by 30 feet keeps the inlet stable and bypasses some additional sediment, but by using a modular design (removable jetty sections) the structure could be adaptively managed to allow increased sediment bypassing (and reduce stability going through the potentially unstable zone) based on the performance of the system. This is indicated by the arrows on the alternatives that include modular design ability.

Alternatives were evaluated using a number of factors (Table 7-1); however, from the coastal processes perspective it was most critical to:

- Maintain the stability of the inlet and reduce potential maintenance requirements related to inlet shoaling that could lead to increased upland flooding potential
- Enhance sediment bypassing as much as possible to reduce erosion on the down drift beaches and allow a portion of the sediment that would naturally be transported to the northeast beaches around the inlet.

- Flexibility in the design at the seaward end of the outlet structure. Since inlet stability is not an exact science in the dynamic coastal environment, and inlet stability spans a range of the Σ/Q parameter, the ability to adaptively manage the structure elevation and height is an important component of the preferred option. Flexibility is also warranted to address future sea level rise and beach profile changes that may modify the cross-shore distribution of the alongshore sediment transport.

Therefore, from all the outlet alternatives that were evaluated for the outlet structure, the following alternatives/components, and potential combinations of alternatives, were considered reasonable, technically feasible, and most environmentally appropriate for the Silver Sands shoreline and are therefore recommended for consideration. Woods Hole Group also highly recommends the design of adjustable training walls, such that the structure can be adjusted (in length and height) to allow flexibility in the management of the inlet and downdrift beach.

- Shorten training wall – reducing the length of the training wall to allow more sediment transport to occur around the structure. New components could consist of adjustable panels for adaptive management.

- Culvert extension and shortening training walls combination – increasing the culvert length to maintain inlet stabilization and reducing training walls in the intertidal zone that could be used as adaptive management component.

Beach alternatives were evaluated as secondary items to the reconstruction of the outlet structure and are considered optional components that could be implemented to help stabilize the downdrift beach if desired.

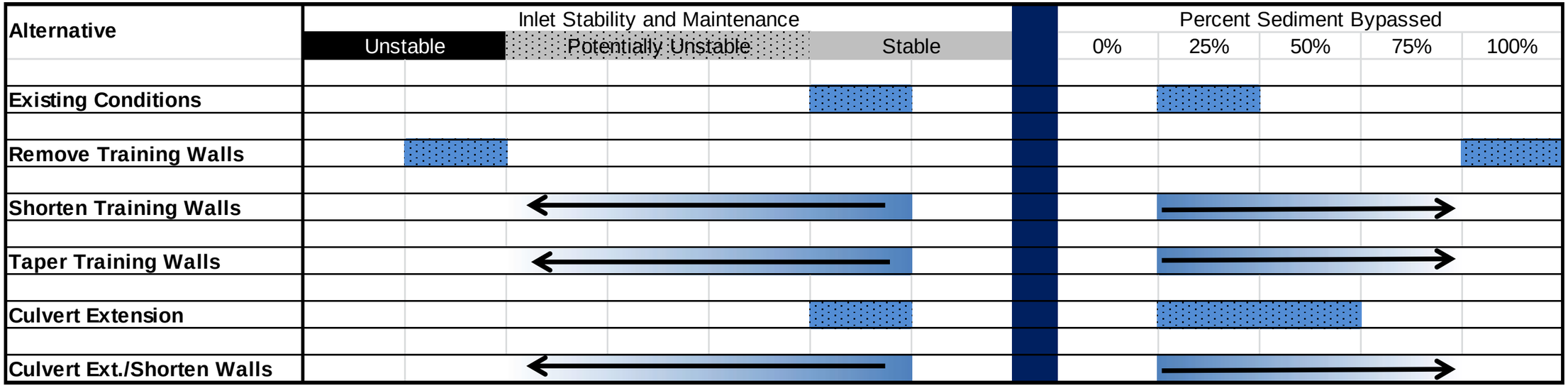

Figure 7-12. Relationship between inlet stability and sediment bypassing at Great Creek outlet for the outlet structure alternatives.

Table 7-1. Silver Sands Alternative Evaluation Summary Matrix.

	Alternative	Inlet Stability	Downdrift Beach	Approximate Cost	Service Life	Maintenance	Safety	Modular Design
	No Action	Inlet stable due to reduced littoral sediment flux	Exacerbated erosion on beach due to littoral barrier (17% of total bypassed)	N/A	Current training wall structures have exceeded service life	Inlet: None required Training walls: Currently required Beach: Erosion will continue	Current situation is a safety liability	N/A
Beach Alternatives	**Beach Nourishment**	No change to inlet stability	Replenished beach with increased width of 100 feet	$500,000-$700,000	12-15 yrs with no change to outlet structure, 20-25 yrs with structure modification	No change to inlet or training wall maintenance.	No safety concerns associated with beach nourishment	N/A
Beach Alternatives	**Manual Bypassing**	No change to inlet stability	Maintain downdrift beach with some enhancement	$25,000-$35,000/yr	N/A	No change to inlet or training wall maintenance, annual beach maintenance requirements	No safety concerns associated with manual bypassing	N/A
Beach Alternatives	**Downdrift Groin or Groin Field**	No change to inlet stability	Improvement to beach, may require multiple grains for effective erosion mitigation over 1,850 ft.	~$750,000 – $1,000,000 for single structure	50-75 years	No change to inlet or training wall maintenance, Potential improved beach stability	No safety concerns associated with downdrift groin	N/A
Outlet Structure Alternatives	**Replacement (Same Dimensions, Improved Components)**	Inlet stable due to reduced littoral sediment flux	Exacerbated erosion on beach due to littoral barrier (17% bypassed)	*Not Recommended*	25-50 years	Inlet: None required Training walls: Minor Beach: Erosion will continue	Concerns associated with open inlet access, same as previous design	New components could consist of adjustable panels for adaptive management
Outlet Structure Alternatives	**Removal of Training Walls**	Inlet would become unstable and require annual maintenance; shoaling expected	Reduced erosion on downdrift beaches (90-100% bypassed)	*Not Recommended*	N/A	Inlet: Significant maintenance expected Beach: Reduced erosion	Existing culvert entrance would pose a safety concern	N/A
Outlet Structure Alternatives	**Shorten Training Walls**	Reducing by 30' maintains stability, may be able to reduce by 100'	Increases amount of bypassed sediment to downdrift beach (28-73% bypassed)	*$650,000*	50 years	Inlet: None required at 30' Beach: Maintenance required at reduced amount compared to existing conditions	Added safety considerations integrated into design	New components could consist of adjustable panels for adaptive management
Outlet Structure Alternatives	**Taper Training Walls**	Reducing by 30' with taper in height maintains stability, may be able to reduce by 100'	Increases amount of bypassed sediment to downdrift beach (30-73% bypassed)	*Not Recommended*	50 years	Inlet: None required at 30' Beach: Maintenance required at reduced amount compared to existing conditions	Added safety considerations integrated into design	New components could consist of adjustable panels for adaptive management
Outlet Structure Alternatives	**Culvert Extension**	Eliminate training walls. Extending culvert 120' to maintain inlet stability	Increases amount of bypassed sediment to downdrift beach (45% bypassed)	*Not Recommended*	50 years	Inlet: None required at 120' Beach: Maintenance required at reduced amount compared to existing conditions	Longer culvert could add safety concerns	No adaptive management ability
Outlet Structure Alternatives	**Culvert Extension with Shorten Training Walls**	Extend culvert 80', training walls reduced by 30'	Increases amount of bypassed sediment to downdrift beach (28-73% bypassed)	*600,000*	50 years	Inlet: None required at 30' Beach: Maintenance required at reduced amount compared to existing conditions	Added safety considerations integrated into design	New components could consist of adjustable panels for adaptive management
Outlet Structure Alternatives	**Tidal Restoration**	Inlet would become unstable unless full restoration was available or training walls utilized	Increases amount of bypassed sediment to downdrift beach (50-80% bypassed)	> $1,000,000	50 years	Inlet: None required with training walls Beach: Maintenance required at reduced amount compared to existing conditions	No change to safety	Potential utilization of new flow control structures for adaptive management

8.0 SUMMARY AND CONCLUSIONS

The purpose of this coastal engineering investigation presented herein is to understand the physical processes that have been shaping the Silver Sands region and evaluate potential alternatives that may be viable solutions for replacement of the Great Creek outlet structure, which is currently in disrepair. This study focuses on evaluating the physical processes (concentrating on the wave, current, and sediment transport environment) occurring within the vicinity of Silver Sands in order to assess potential alternatives that may be used to maintain inlet stability and enhance sediment bypassing to the downdrift beach. All elements of the project were geared towards arriving at a technically feasible, cost-effective, and long-term solution.

The existing environmental conditions within the region were studied extensively in preparation for evaluating the potential alternatives. Where possible, information from existing studies was used to describe the system. In other cases, site-specific analyses were undertaken to evaluate the physical conditions within the project area. A summary of the major components of the analyses is given below.

- ***Data Collection***
  - Water surface elevation data was collected both in the marsh upstream of the culvert and in Milford Harbor to evaluate tidal attenuation through the culvert and assess how much tidal prism and range could be gained through culvert improvements.
  - Velocity measurements were taken in the culvert to analyze the water flow through the culvert, determine flux, perform inlet stability analysis, and assess tidal flushing ability.
  - Sediment grab samples were collected and analyzed for grain size for use in the sediment transport modeling.

- ***Transformation Scale Wave Modeling*** - A nearshore, transformation scale wave model was used to propagate the offshore wave climate into the Silver Sands region and evaluate the transformations waves experience as they propagate towards the coastline. The model was verified using the observed wave data and the error estimates were within acceptable bounds. Once verified, the wave transformation model was used to simulate average annual directional cases (developed from MYSOUND buoy data), specific historic storms events, and return-period storms. The wave model results were used to evaluate forces governing wave climate and sediment transport processes. The transformation scale model identified transformation effects that produced an uneven distribution of wave energy along the coast that affect sediment transport in the region and significantly influence the rate and direction of sand movement. Results of the transformation scale model are used to develop quantitative sediment transport fluxes and divergence, while evaluating the performance of various proposed alternatives.

- ***Sediment Transport*** **-** Understanding the wave transformations is a critical step in determination of shoreline processes and changes, and this wave information is required in order to provide an estimate of how sediment moves in the nearshore region. The wave modeling system results were the key input into the sediment transport modeling, alternatives assessment, and beach nourishment performance evaluations. Gross sediment transport rates vary significantly for the various average annual approach directions. The various wave scenarios were also combined to represent an average annual year of wave climate. The sediment flux indicates an average annual longshore transport rate to the east of approximately 14,500 yd^3/yr. The waves from the west (causing eastward sediment transport) are dominant in both wave energy and percent occurrence. Therefore, it is not surprising that the net sediment transport rate over an average year is from the west to the east at the Great Creek outlet structure. There are local reversals in net sediment transport in the vicinity of the groins to the west of the outlet structure.

A variety of outlet structure alternatives were considered for replacing the currently damaged outlet structure, while potentially reducing the impact of the structure on the coastal resources and natural littoral drift in the area. Outlet structure alternatives were evaluated against a number of criteria, with the most critical technical performance factors including:

- Maintain the stability of the inlet and reduce potential maintenance requirements related to inlet shoaling that could lead to increased upland flooding potential
- Enhance sediment bypassing as much as possible to reduce erosion on the down drift beaches and allow a portion of the sediment that would naturally be transported to the northeast beaches around the inlet.
- Flexibility in the design at the seaward end of the outlet structure. Since inlet stability is not an exact science in the dynamic coastal environment, and inlet stability spans a range of the Σ/Q parameter, the ability to adaptively manage the structure elevation and height is an important component of the preferred option. Flexibility is also warranted to address future sea level rise and beach profile changes that may modify the cross-shore distribution of the alongshore sediment transport.

Supplementary alternatives that evaluated enhancements to the downdrift beach were also considered as secondary components of the restoration. These included beach nourishment, manual bypassing, and groin installation. A beach nourishment project or target manual bypassing could be considered in the future following reconstruction of the outlet structure.

The recommended solutions for replacement of the Great Creek outlet structure are discussed below. In all cases, the existing, damaged training walls and associated fence needs to be removed and disposed. The alternatives presented below are therefore recommended for replacement of the training walls and to repair the existing state of the Great Creek outlet structure.

- Shorten training walls – This alternative would remove the existing, damaged training walls, associated fencing, and other components and replacing them with improved training walls (new design). The new training walls (both the updrift and downdrift structures) would be approximately 150 feet in length. A reduction in total length by approximately 30 feet when compared to the existing training walls. This would allow more sediment to naturally bypass the structure. Due to the dynamic nature of the coastal environment, as well as the unpredictability of storm events, sea level rise, changes in offshore bathymetry that effect wave transformations, design conditions can be dynamic. Therefore, in coastal designs, the ability to adapt to future changing conditions is sometimes a desired component. As such, consideration should also be given to designing the seaward most 70 feet of the new training walls with modular components (e.g., panels) so the length of the structure could be adjusted. The proposed modular components would not be intended for modification on a regular basis; rather something that would be able to be modified if conditions changed significantly on a longer term basis (e.g., 2-5 years). The new training walls could be constructed out of a wide variety of materials (e.g., concrete, steel, wood, vinyl, etc.). In general, the more durable the material type, the more expensive the material cost. For example, concrete would likely be the most durable, but will also likely be the most expensive. All material types will be considered and evaluated in detail in the design phase of this project, and a specific material type will be recommended during the design phase. It is expected that with proper and improved design, a number of material types would adequately serve the durability requirement.

or

- Culvert extension and shortening training walls combination – This alternative would remove the existing, damaged training walls, associated fencing, and other components and replacing them with an extended culvert and improved training walls (new design). The culvert length would be extended by 80 feet seaward, using the same culvert dimensions. It is recommended that the culvert extension remain a four-sided box culvert (as opposed to a three-sided box culvert) to eliminate undesirable scour effects that may impact the structural integrity of the structure. New training walls would be designed and constructed at the seaward end of the culvert extending an additional 70 feet. The total combined structure would be 150 feet, approximately 30 feet shorter than the existing training walls to allow more natural bypassing of the structure. Due to the dynamic nature of the coastal environment, as well as the unpredictability of storm events, sea level rise, changes in offshore bathymetry that effect wave transformations, design conditions can be dynamic. Therefore, in coastal designs, the ability to adapt to future changing conditions is sometimes a desired component. As such, consideration should also be given to designing the seaward most 70 feet of the new training walls with modular components (e.g., panels) so the length of the structure could be adjusted. The proposed modular components would not be intended for

modification on a regular basis; rather something that would be able to be modified if conditions changed significantly on a longer term basis (e.g., 2-5 years). The new training walls could be constructed out of a wide variety of materials (e.g., concrete, steel, wood, vinyl, etc.). All material types will be considered and evaluated in detail in the design phase of this project, and a specific material type will be recommended during the design phase.

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Appendix A.

Grain Size

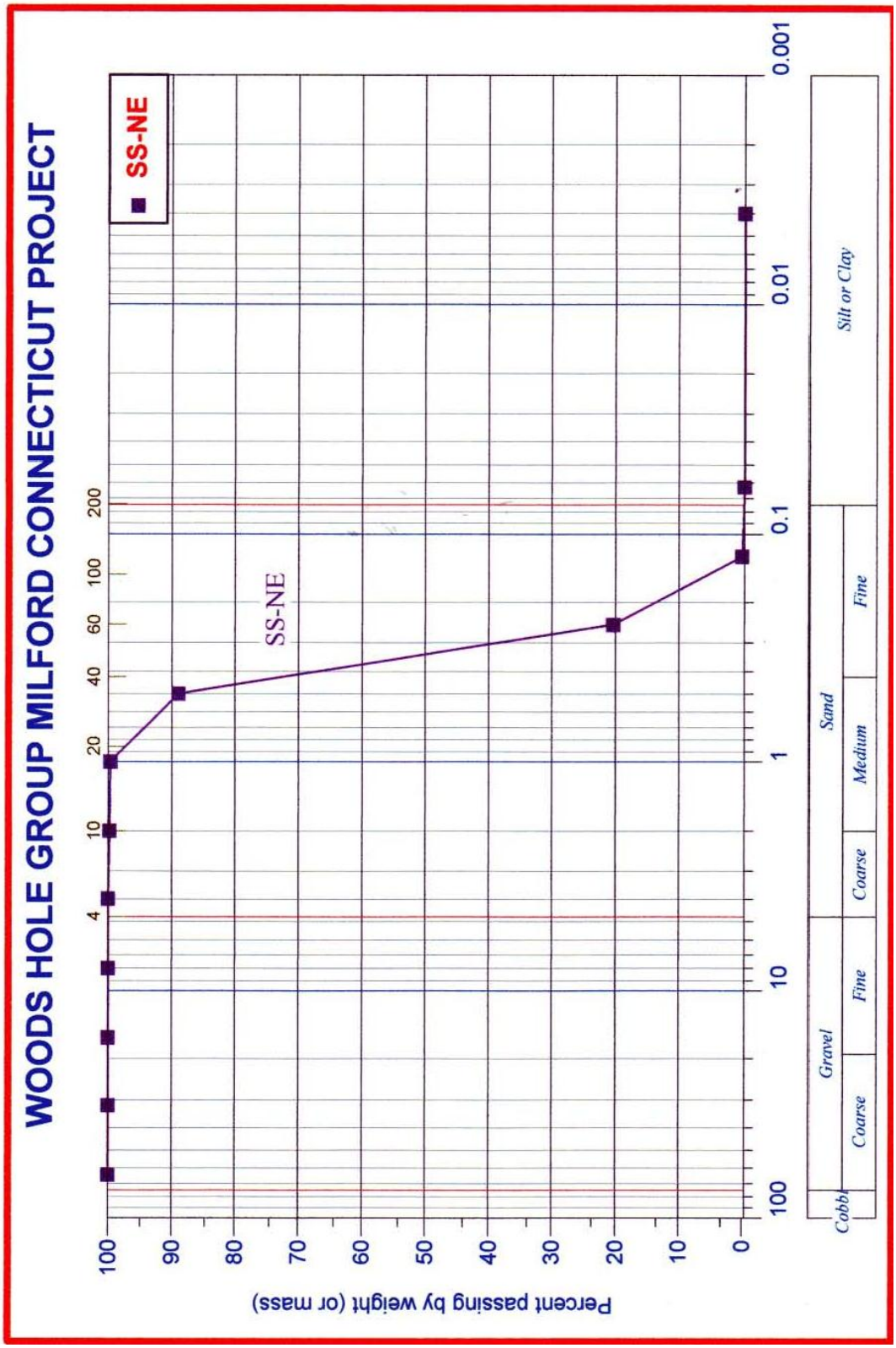

Figure A-1. Grain size analysis for sample SS-NE

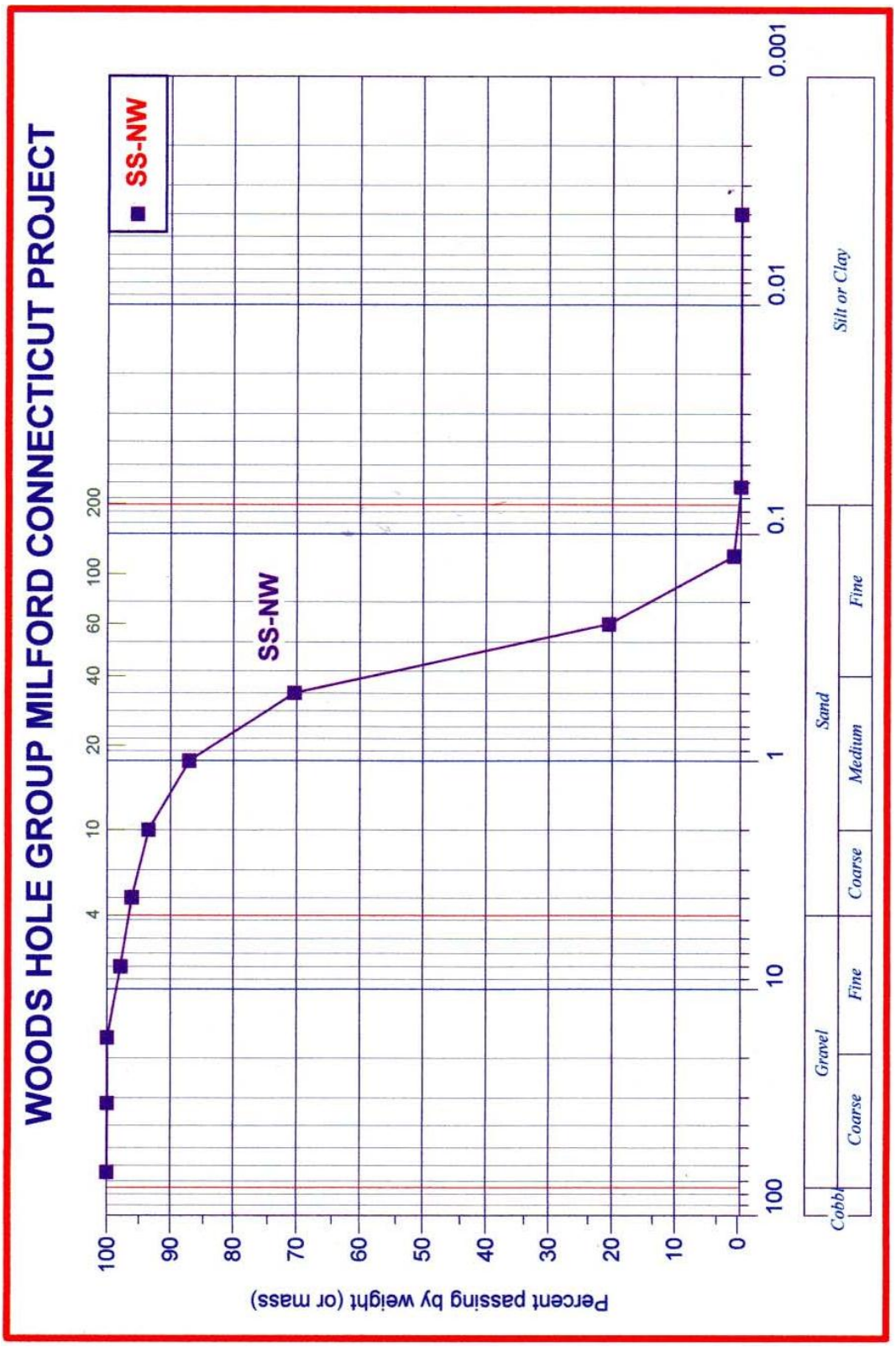

Figure A-2. Grain size analysis for sample SS-NW

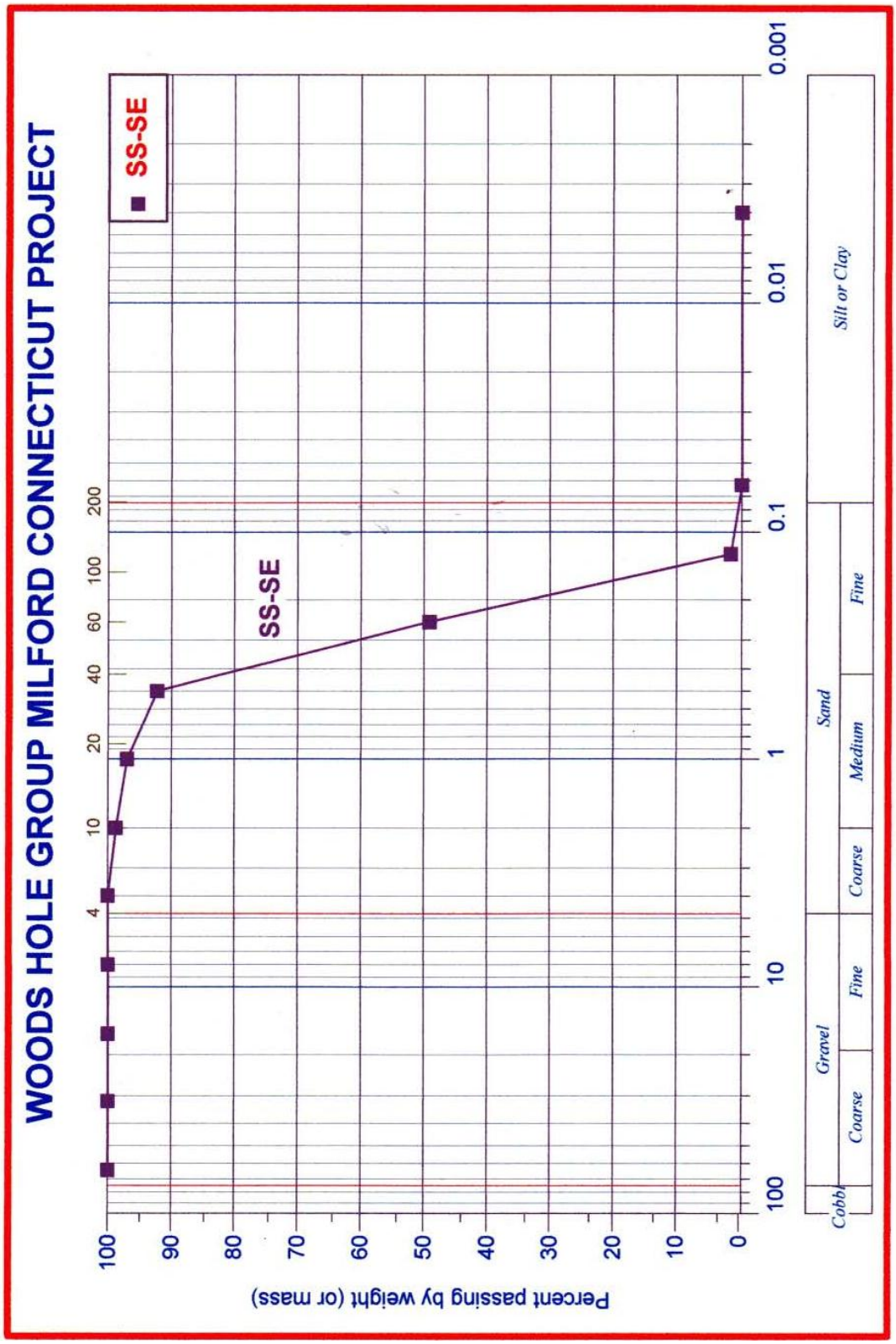

Figure A-3. Grain size analysis for sample SS-SE

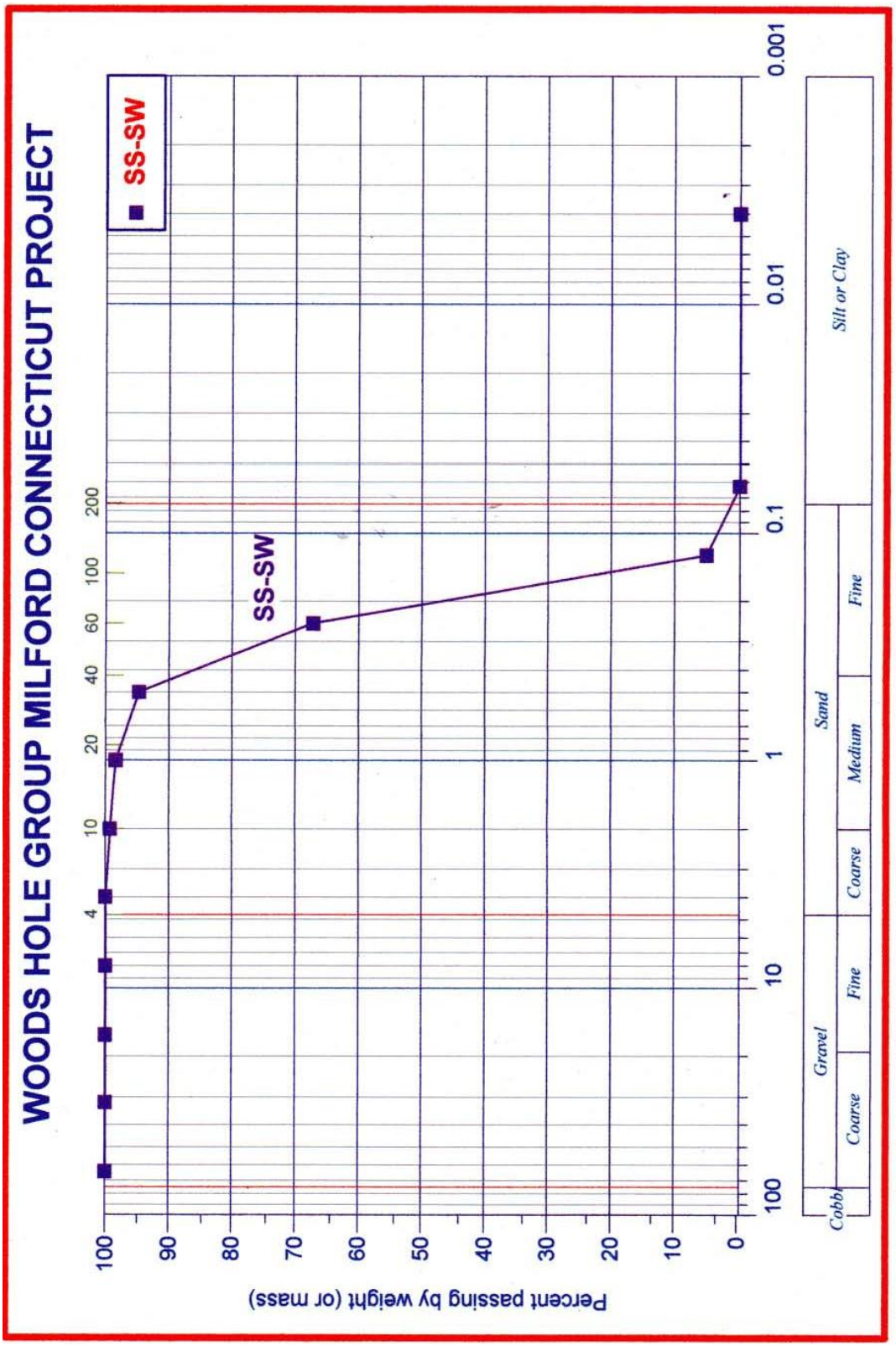

Figure A-4. Grain size analysis for sample SS-SW

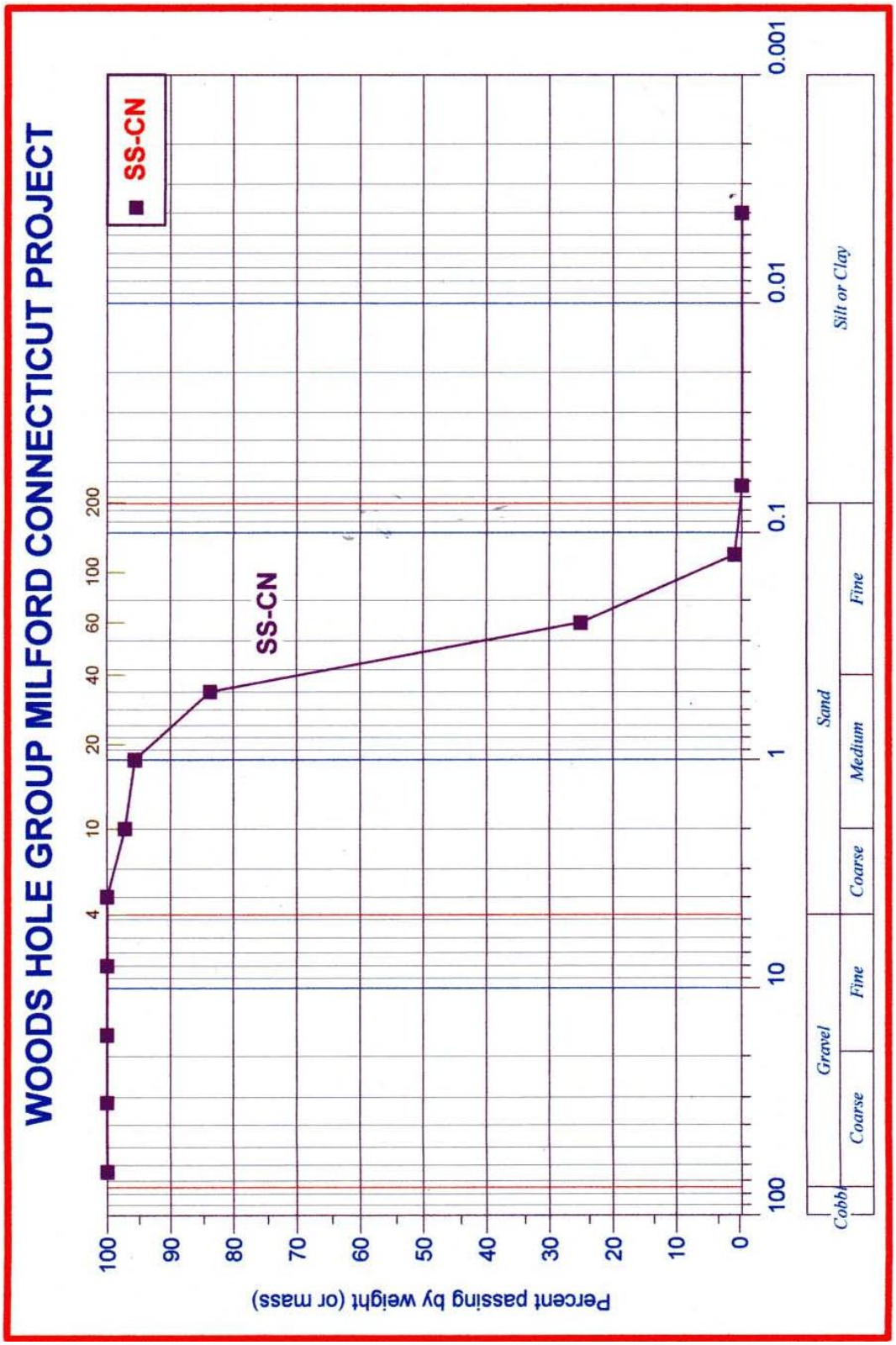

Figure A-5. Grain size analysis for sample SS-CN

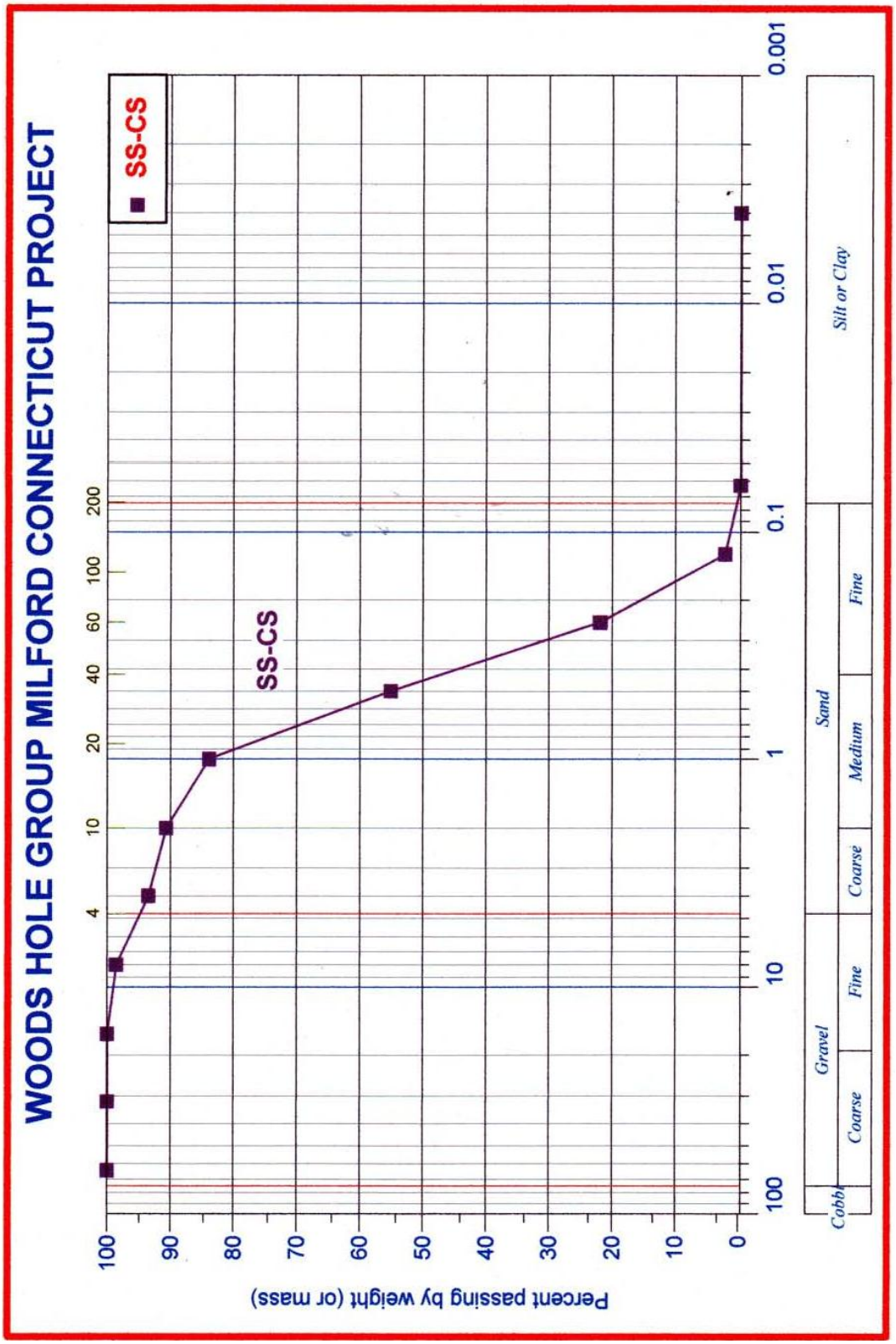

Figure A-6. Grain size analysis for sample SS-CS

Appendix B.

Waves Direction and Waves Storm Cases

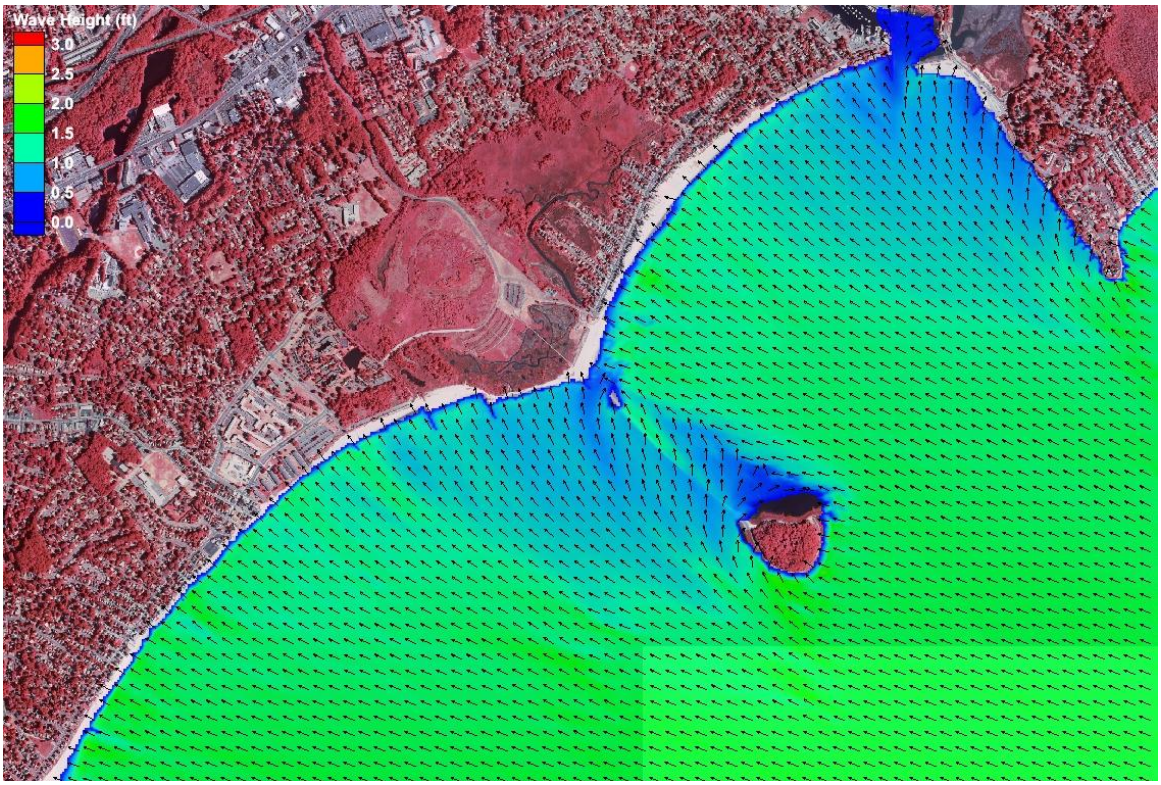

Figure B-1. Wave heights (ft) at Silver Sands for directional bin 90° to 112.5°

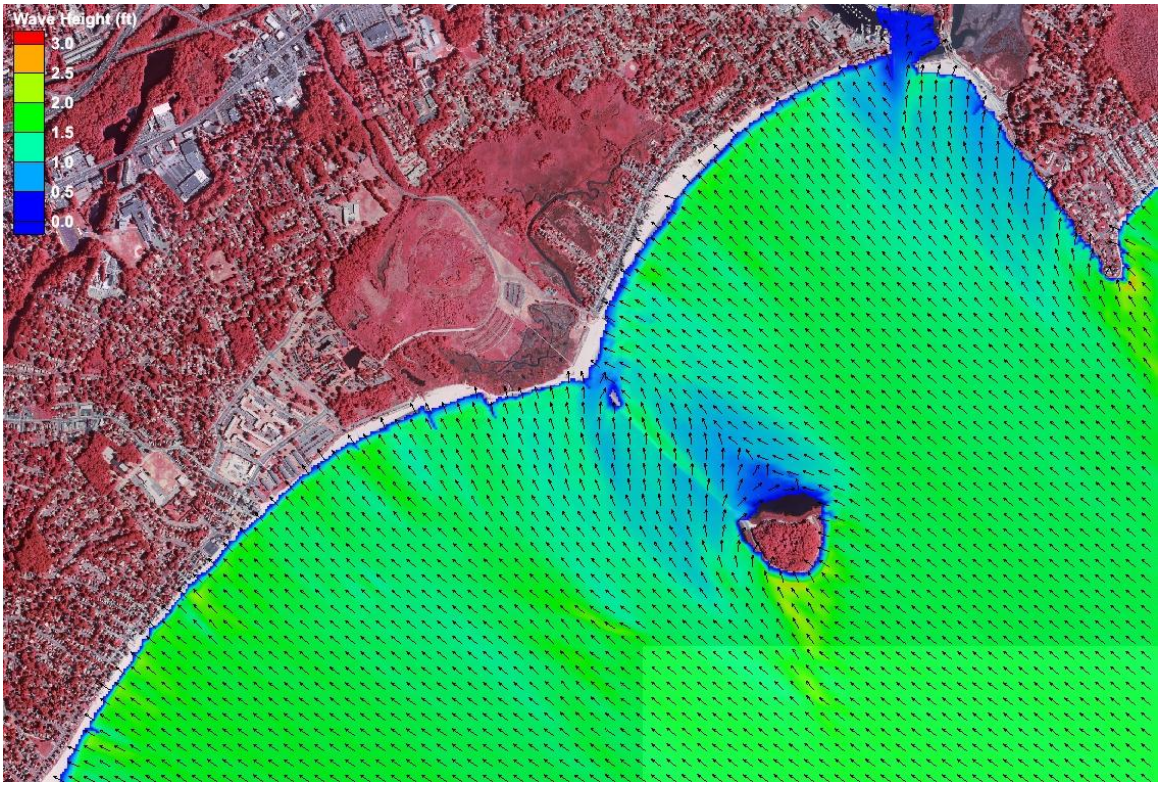

Figure B-2. Wave heights (ft) at Silver Sands for directional bin 112.5° to 135°

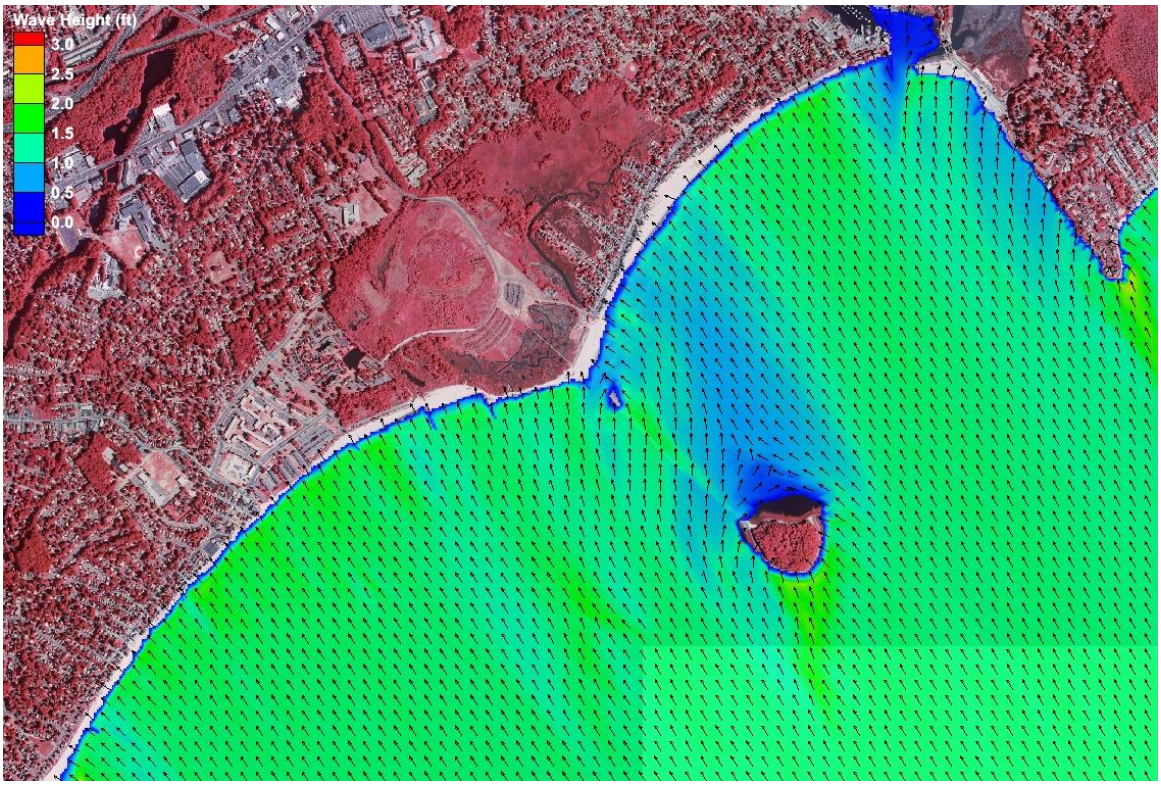

Figure B-3. Wave heights (ft) at Silver Sands for directional bin 135° to 157.5°

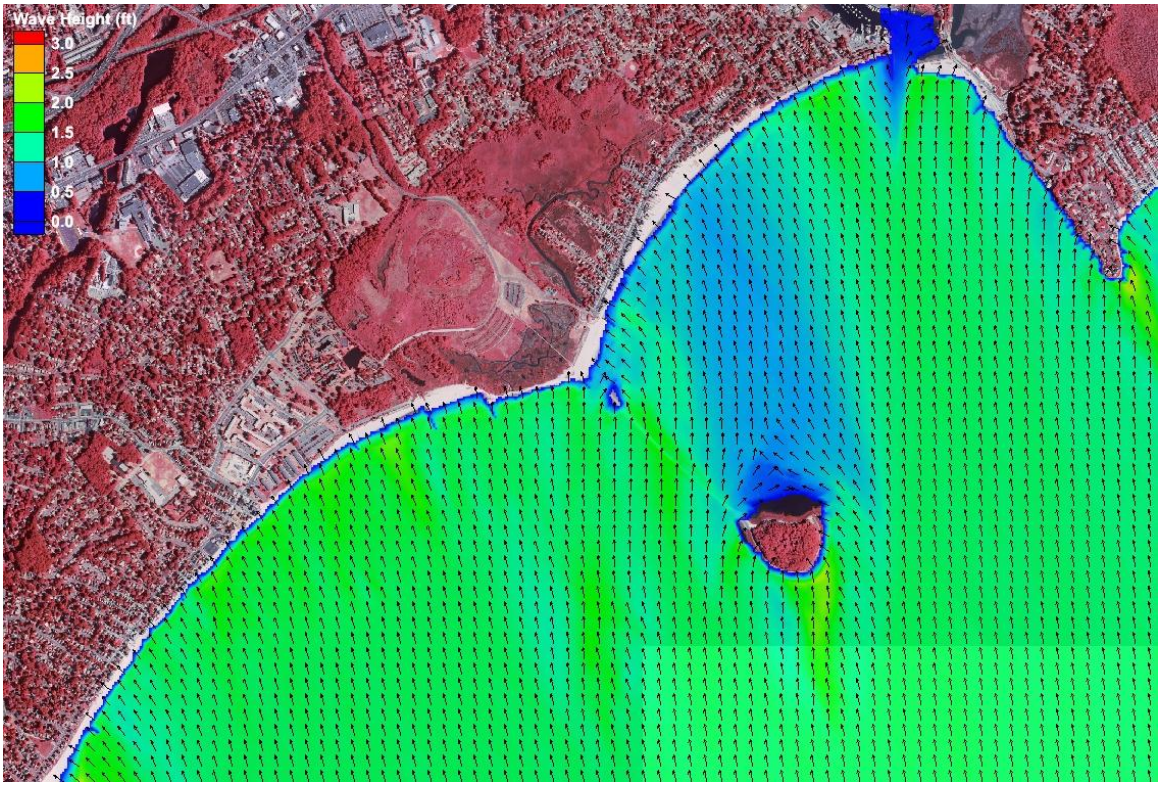

Figure B-4. Wave heights (ft) at Silver Sands for directional bin 157.5° to 180°

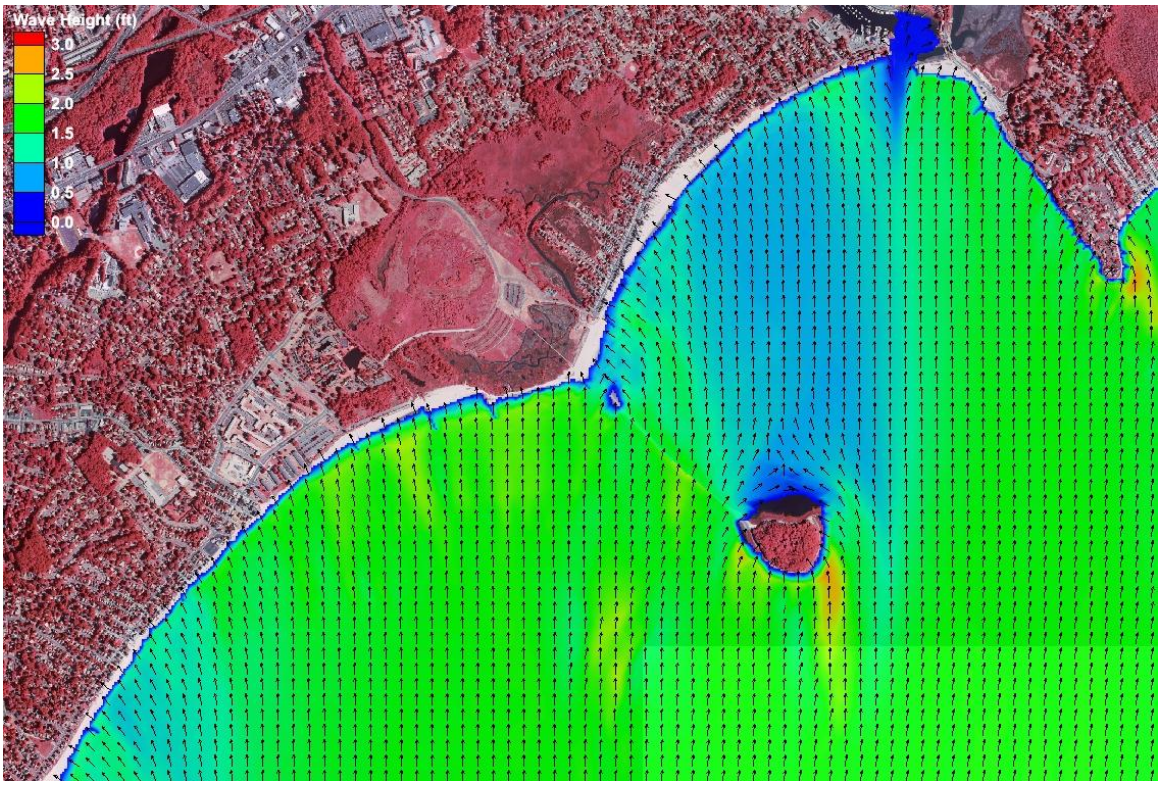

Figure B-5. Wave heights (ft) at Silver Sands for directional bin 180° to 202.5°

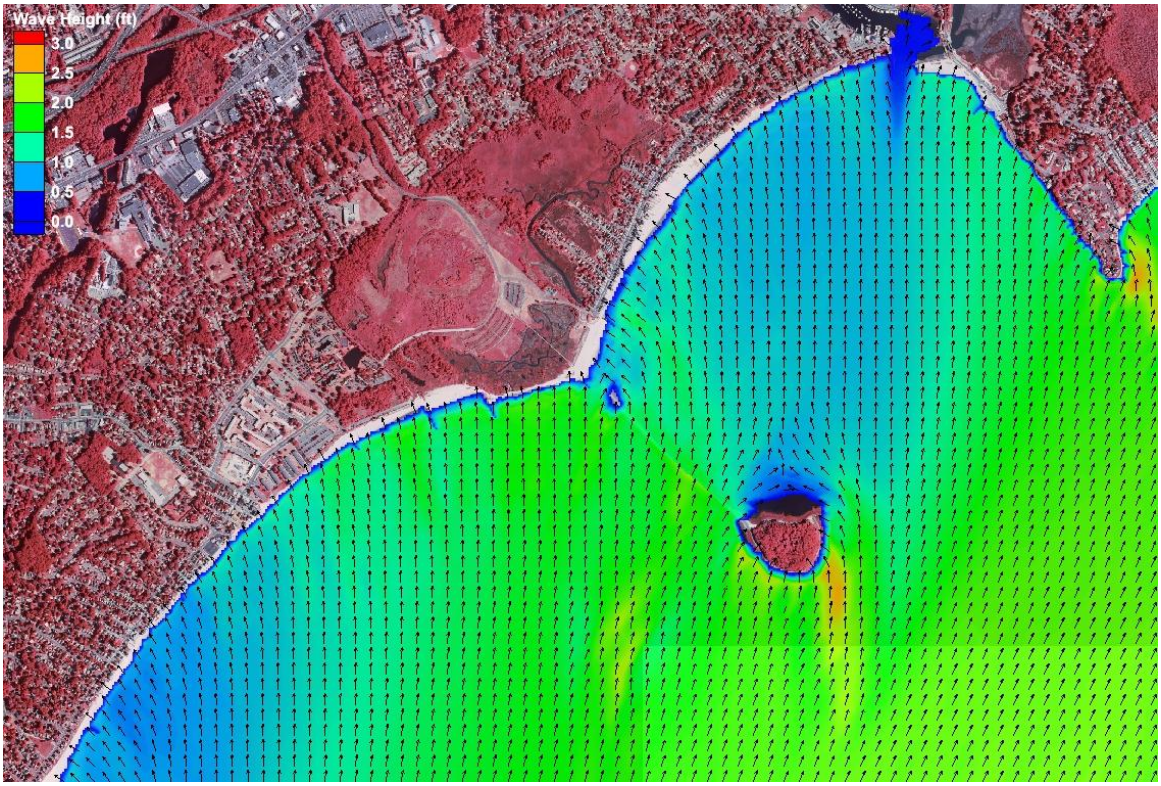

Figure B-6. Wave heights (ft) at Silver Sands for directional bin 202.5° to 225°

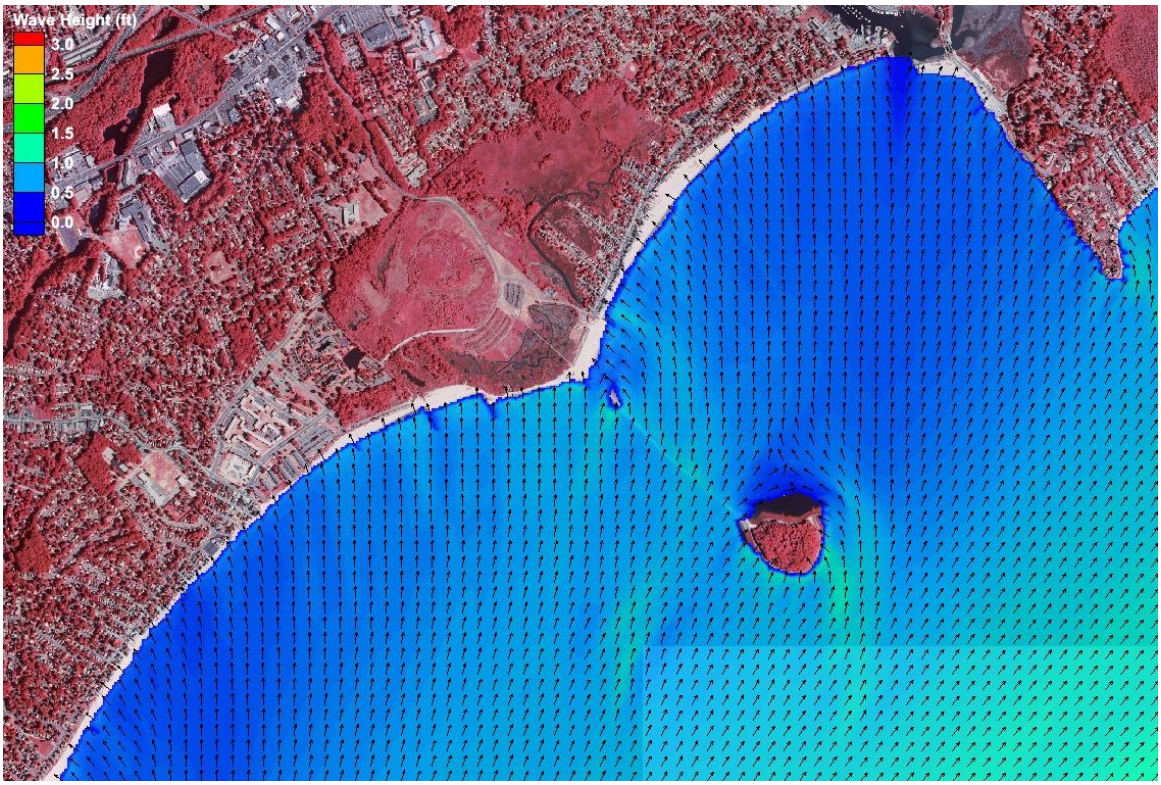

Figure B-7. Wave heights (ft) at Silver Sands for directional bin 225° to 247.5°

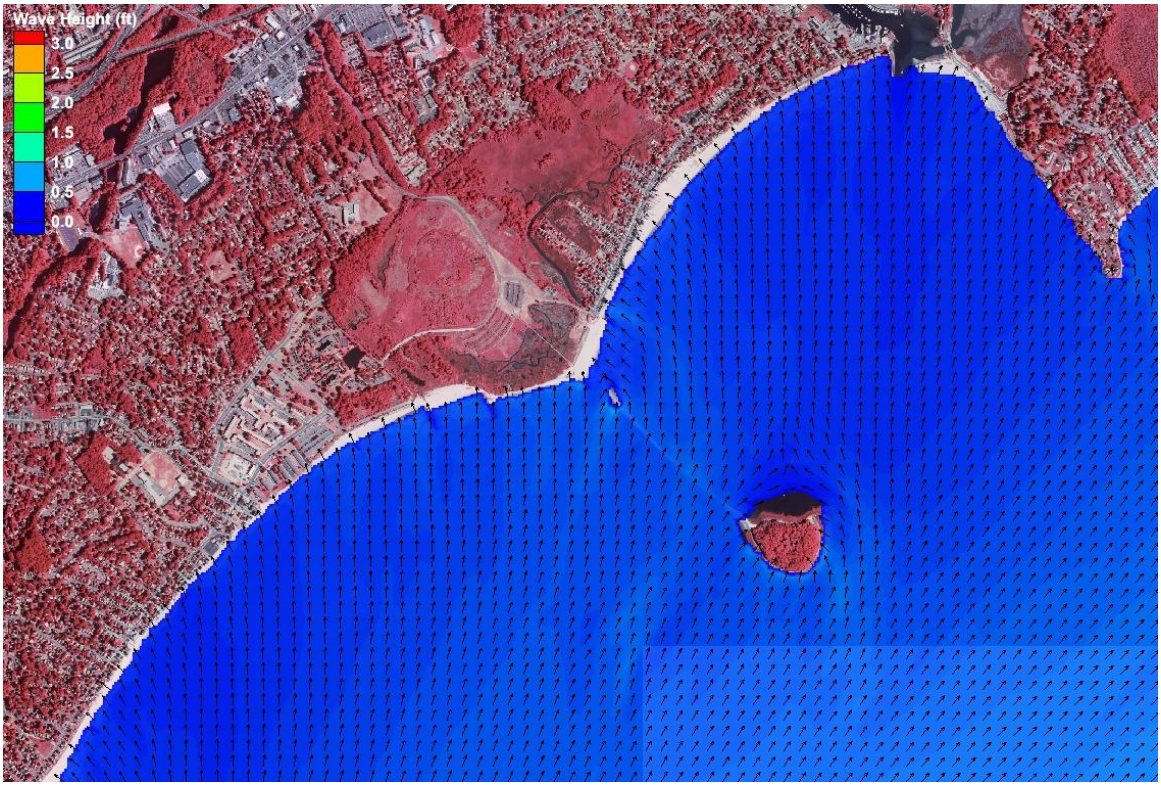

Figure B-8. Wave heights (ft) at Silver Sands for directional bin 225° to 247.5°

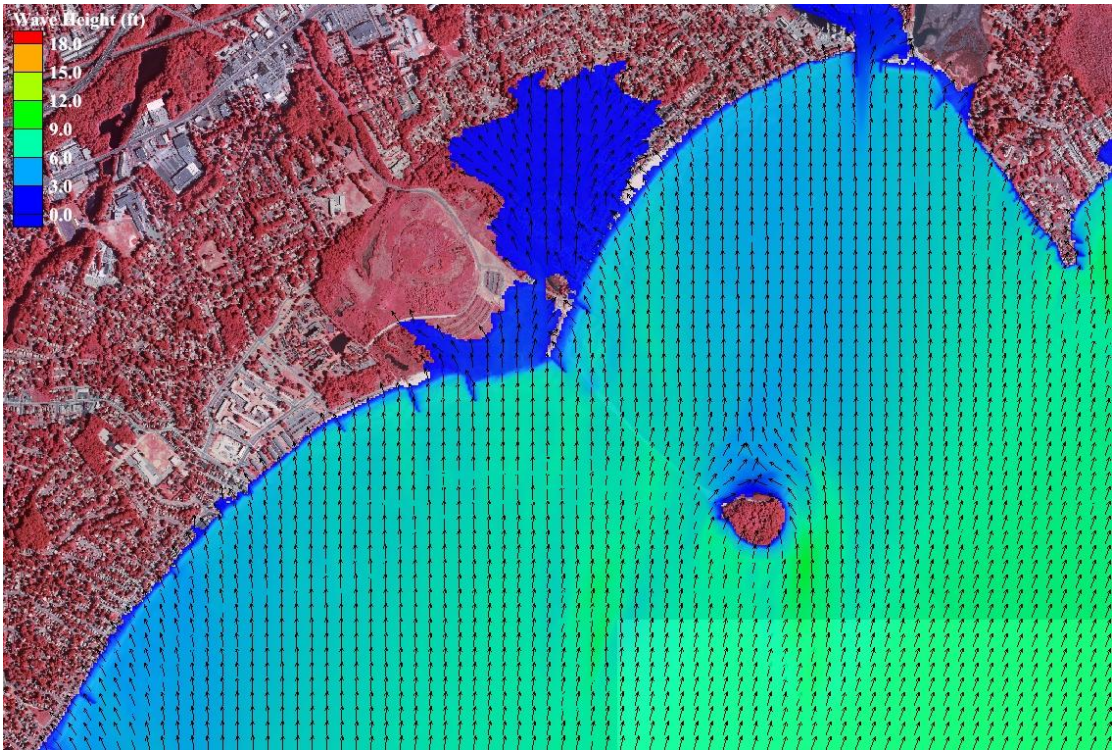

Figure B-9. Silver Sands wave height (ft) during a 10-year return period storm event

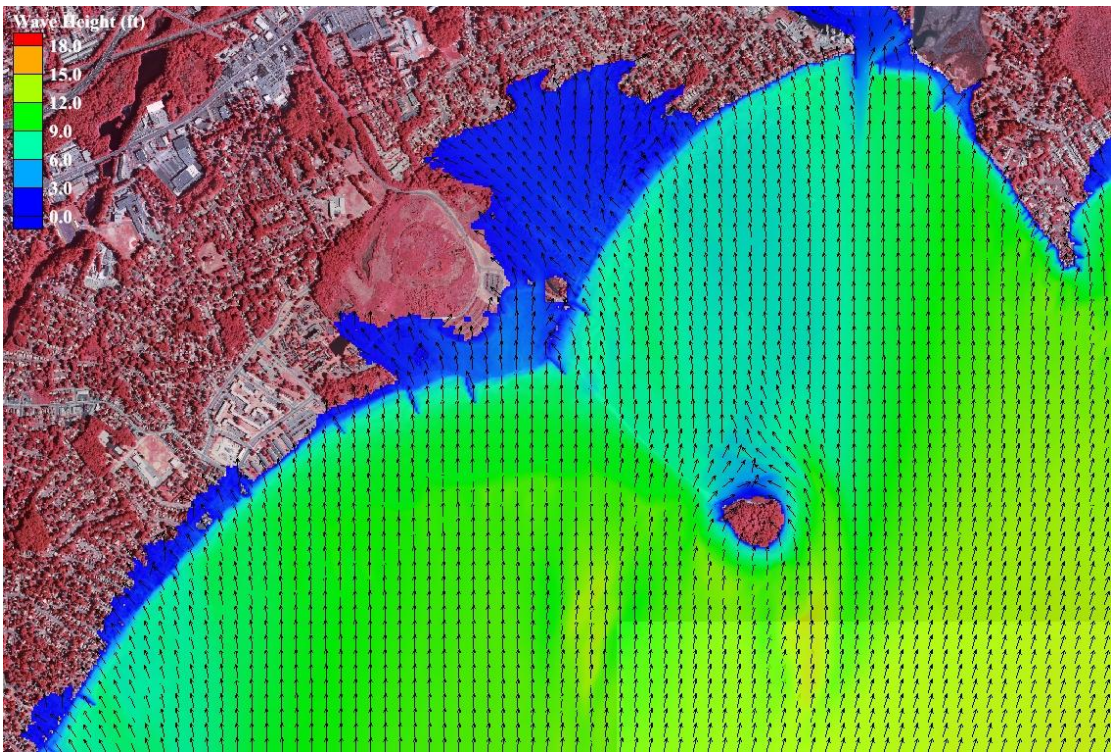

Figure B-10. Silver Sands wave height (ft) during a 50-year return period storm event

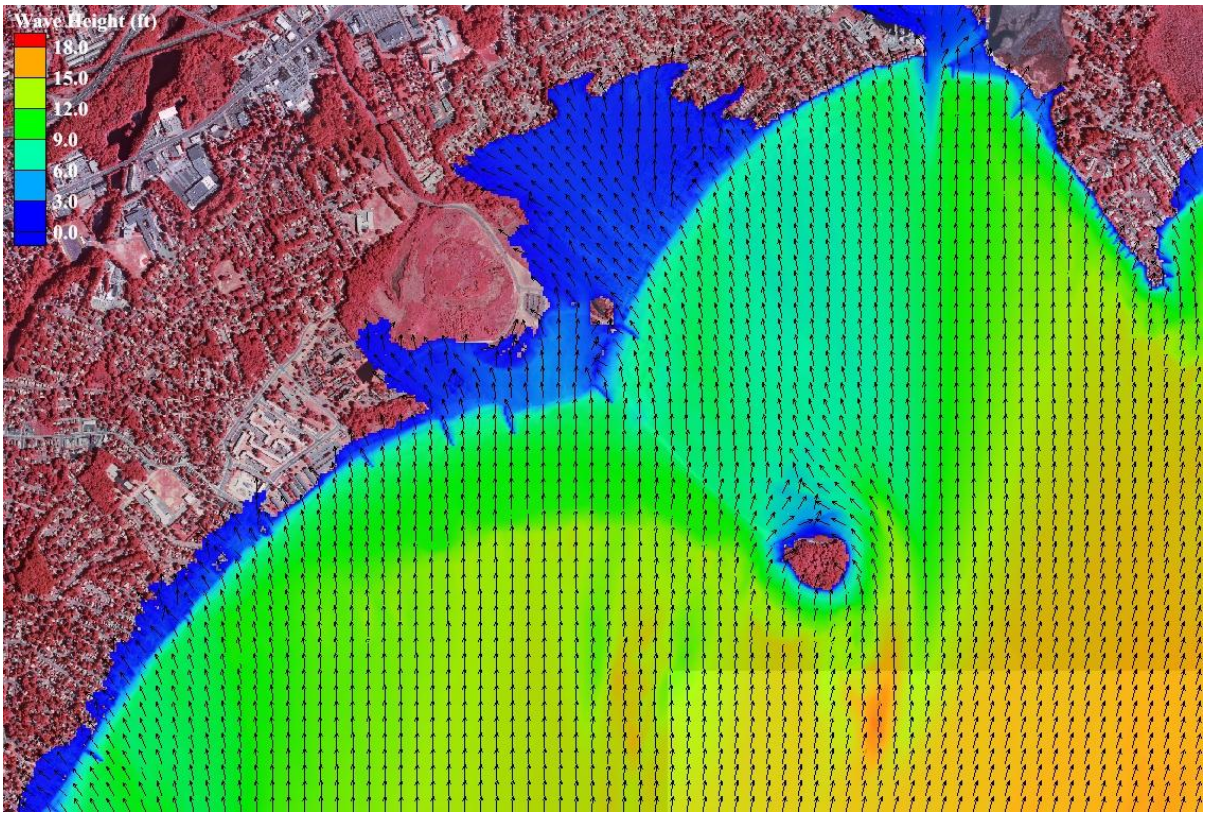

Figure B-11. Silver Sands wave height (ft) during a 100-year return period storm event

Appendix C.

Directional Cases

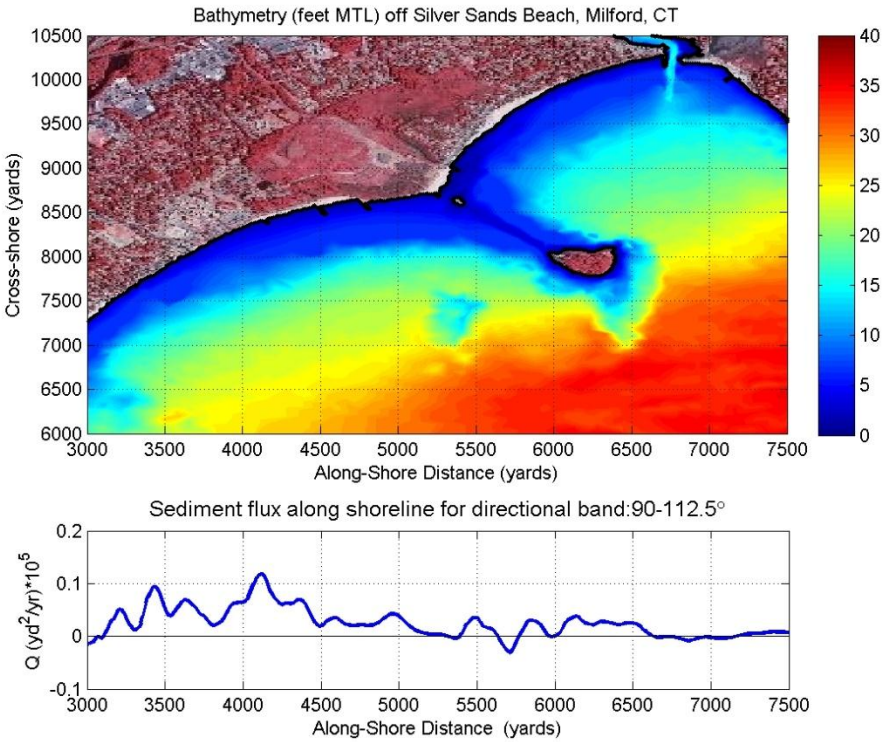

Figure C-1. Longshore distribution of cross shore sediment transport for directional bin 90° to 112.5°

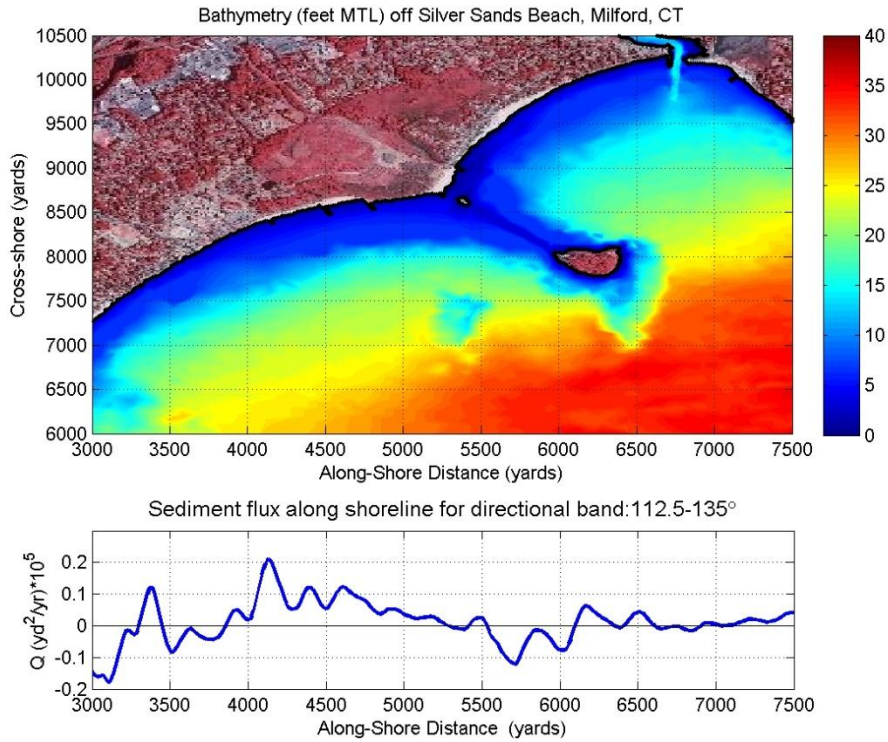

Figure C-2. Longshore distribution of cross shore sediment transport for directional bin 112.5° to 135°.

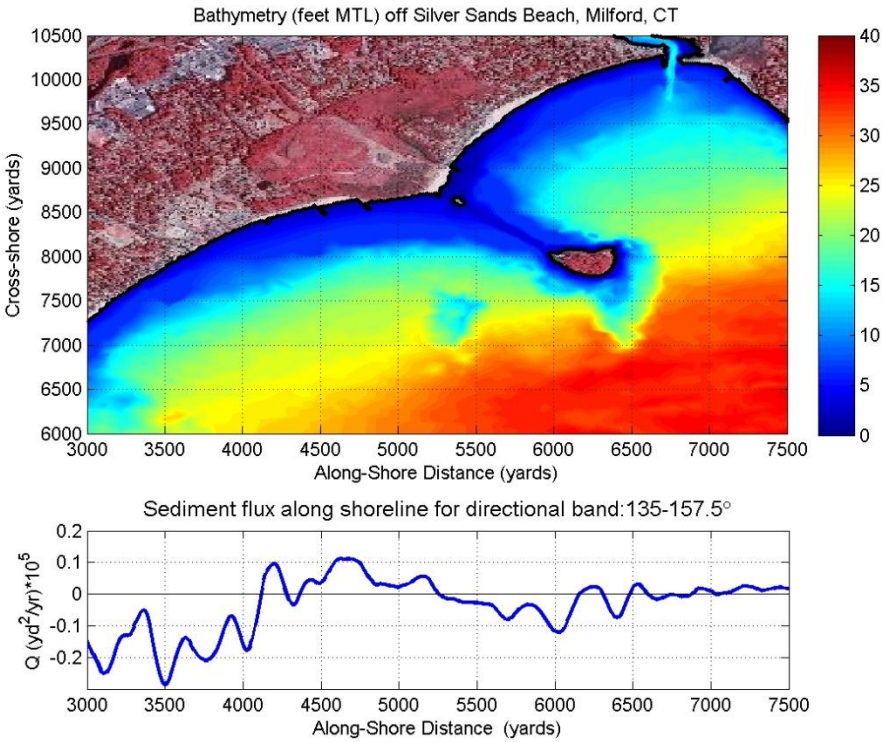

Figure C-3. Longshore distribution of cross-shore sediment transport for direction bin 135° to 157.5°

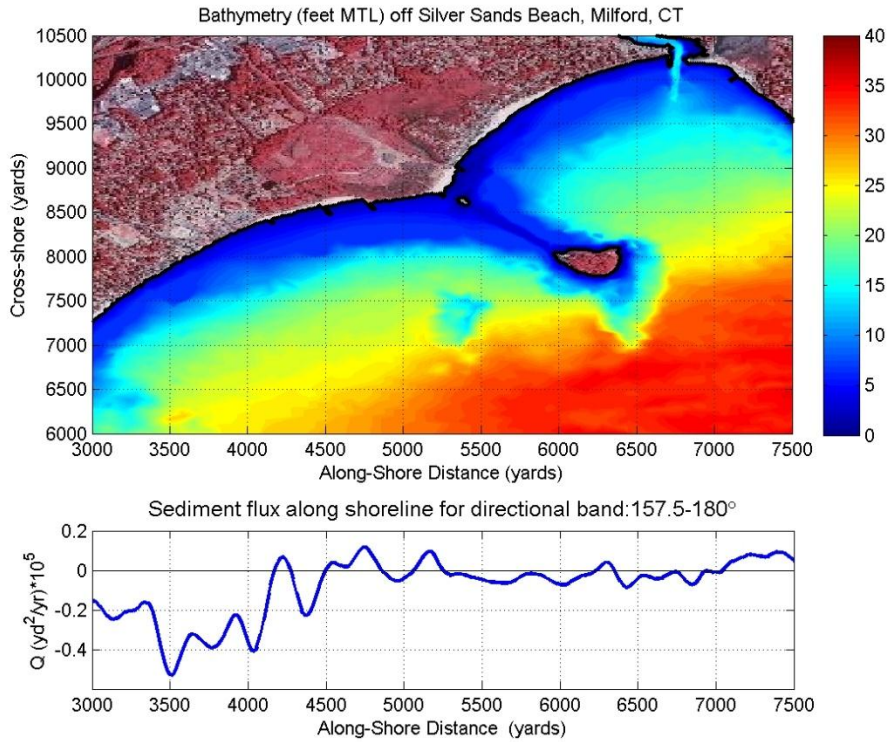

Figure C-4 Longshore distribution of cross-shore sediment transport for direction bin 157.5° to 180°

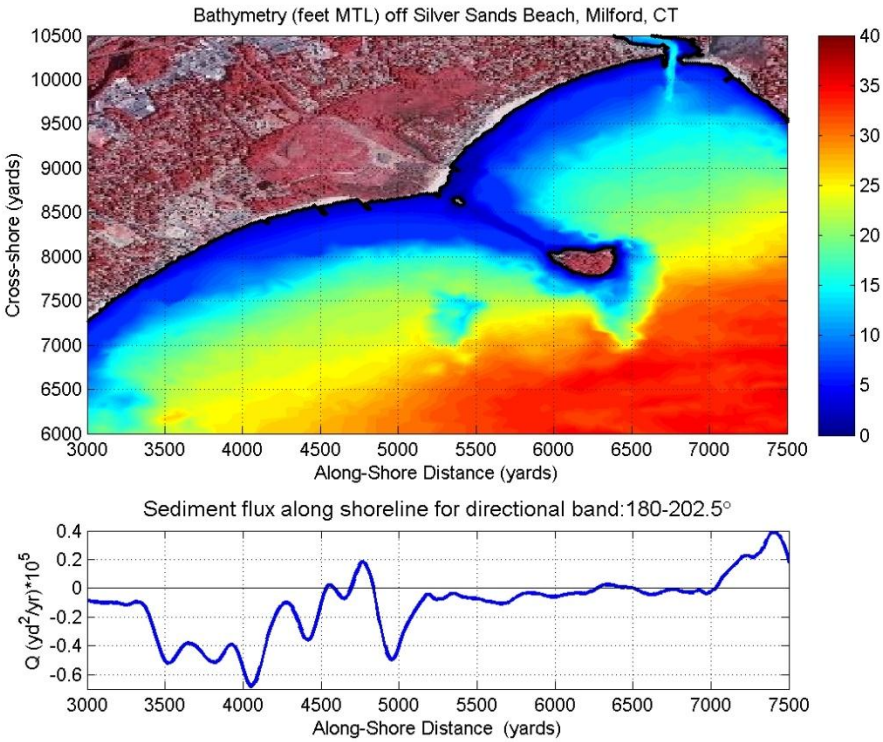

Figure C-5 Longshore distribution of cross-shore sediment transport for direction bin 180° to 202.5°

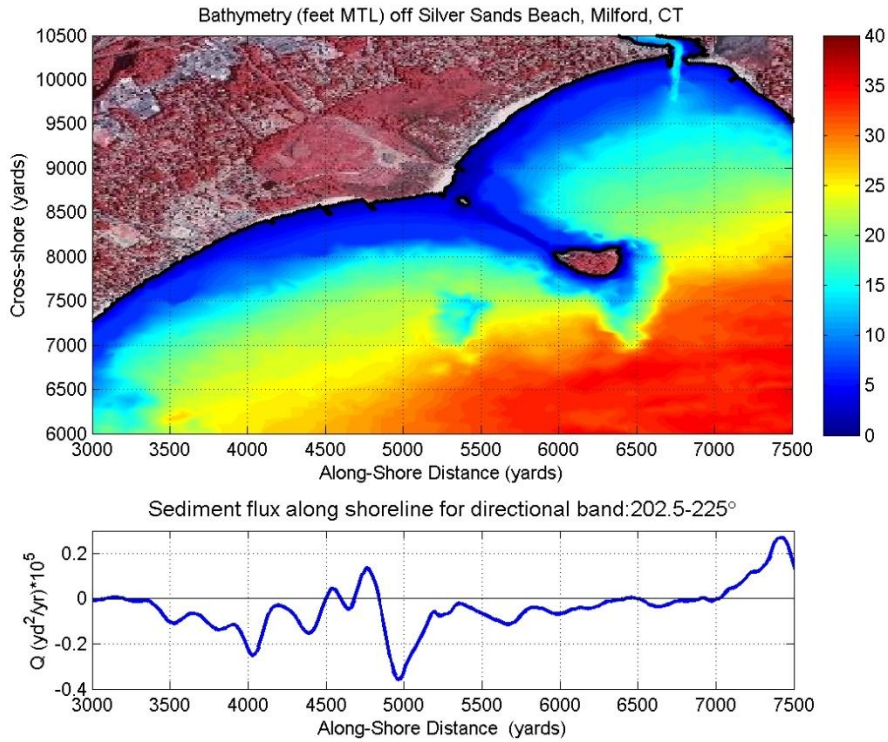

Figure C-6. Longshore distribution of cross-shore sediment transport for direction bin 202.5° to 225°

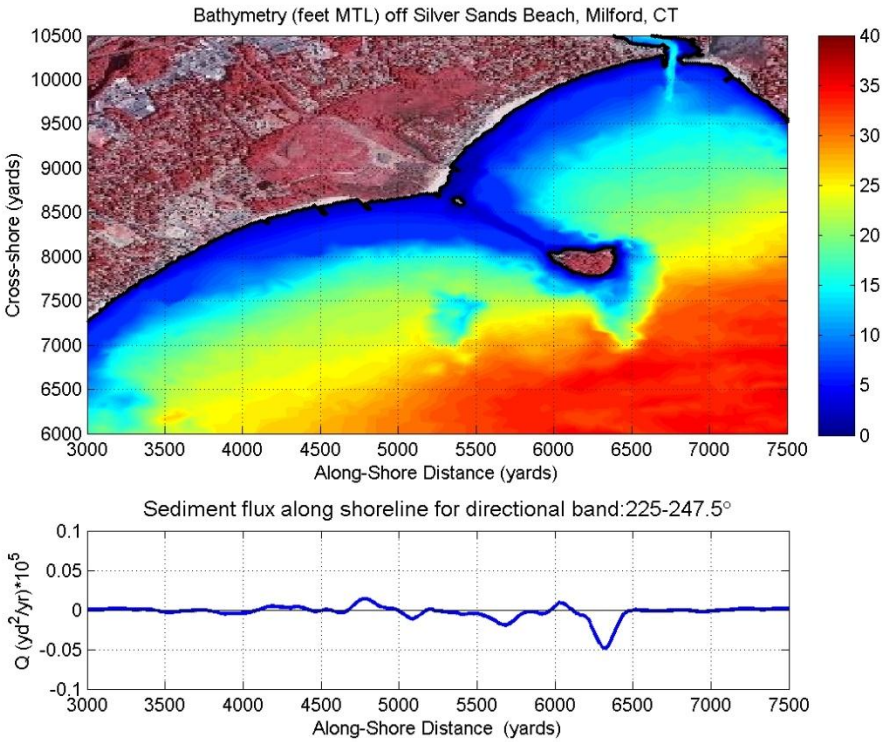

Figure C-7. Longshore distribution of cross-shore sediment transport for direction bin 225° to 247.5°.

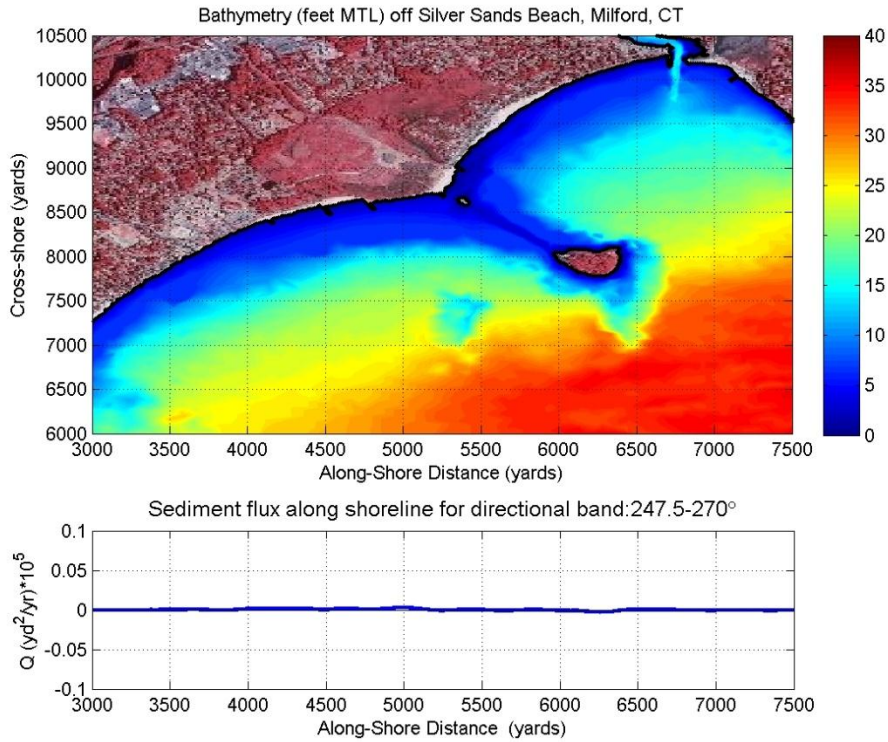

Figure C-8. Longshore distribution of cross-shore sediment transport for direction bin 247.5° to 270°

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