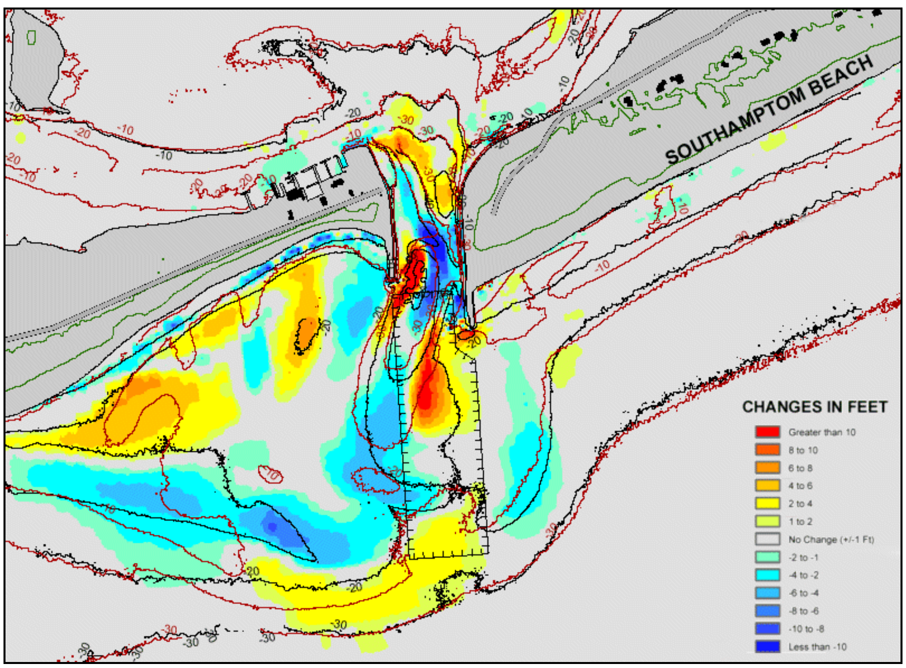
DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
NEW YORK DISTRICT

Atlantic Coast of Long Island, Fire Island Inlet to Montauk Point, New York

Storm Damage Reduction Reformulation Study



Work Order 28
Inlet Modifications

Draft Final Report

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EXECUTIVE SUMMARY

The US Army Corps of Engineers, New York District (CENAN), is conducting a comprehensive feasibility-level reformulation of the shore protection and storm damage reduction project for the south shore of Long Island, New York, from Fire Island Inlet to Montauk Point (FIMP). The Reformulation Study is a multi-year and multi-task effort, involving project planning and engineering, economic analyses and environmental studies. Numerous study tasks are involved in the planning of storm damage reduction projects for the approximately 83-mile study area length. This project area includes sections of mainland (58 km) and barrier island beaches (75 km) and three stabilized inlets.

The goal of the Inlet Modifications Study is to develop long-term solutions for Shinnecock Inlet, Moriches Inlet, and Fire Island Inlet that provide reliable navigation through the Federal navigation channels, maximize sand bypassing in order to restore, to the extent possible, natural sediment pathways, and reduce adjacent and downdrift shoreline erosion.

Section 1 presents an introduction to the Reformulation Study in general and the Inlet Modifications Study in particular. Also included in this section is a summary of the scope of this study, including an outline of the specific tasks that were completed. Section 2 presents an overview of Shinnecock, Moriches and Fire Island Inlet. The report describes the history of each inlet since its formation including all the engineering activities that have taken place since its opening. A description of coastal processes and the existing navigation conditions are also presented.

Section 3 presents the preliminary list of alternatives developed at the Coastal Technical Management Group (CTMG) meeting held on 15 November 2001. A fatal flaw analysis was applied to the preliminary list to limit the number of alternatives selected for further screening. The remaining alternatives were further evaluated with a multiple criteria screening analysis. As a result of the fatal flaw and multiple criteria screening analyses, a reduced number of alternatives were selected for detailed design and analysis, the results of which are presented in Section 8.

Combined hydrodynamic, wave, sediment transport and morphological models for each inlet are presented in Section 4. Existing data were used to calibrate the models to measured water levels, currents and bathymetric changes. After successful calibration, the models were used to simulate the selected alternatives from Section 3, as part of the detailed analysis of inlet alternatives presented in Section 8.

Section 5 presents an analysis of inlet dynamics, which provides a framework for understanding the processes at work at each inlet and along adjacent shorelines. This section addresses each inlet in terms of hydrodynamics, morphology, and sediment budget. The detailed analysis for each inlet includes: analysis of bathymetric records, a volumetric change analysis, a hydraulic analysis including an inlet stability analysis, wave climate and longshore sediment transport estimate and influence of relative sea level rise, and a sediment budget analysis. At each inlet, the extent of natural bypassing that occurs under existing conditions is an important conclusion used to develop the inlet modification alternatives.

Section 6 presents the development of a regional sediment budget for the shoreline from Fire Island Inlet to Montauk Point. The sediment budget is of recent morphological changes and beach/inlet management practices; results from the detailed inlet sediment budgets computed in Section 5 are also included. In addition, the analysis is expanded to incorporate medium- to long-term (10-30 years) historic trends and ongoing management practices and engineering activities to develop a regional sediment budget representative of existing conditions.

Section 7 presents an analysis of the navigation conditions of the inlets. A vessel transit analysis was conducted for selected design vessels at each inlet.

Section 8 presents the detailed analysis of short-listed inlet alternatives and the final recommendation for each inlet. The analysis of each alternative includes modeling results, an alternative sediment budget, costs and pros and cons based on identified project needs, risk and uncertainty and environmental, economic, recreational and engineering criteria. In the end, each recommended alternative effectively addresses each of three prominent identified issues:

- Navigation reliability through the Federal navigation channels
- Erosion of the beach immediately downdrift of the inlets
- Erosion of the beach west of the ebb-shoal reattachment

In addition, the recommended plans are as flexible as possible to allow for future changes as new data is collected and conditions change or prove to be different than presently understood.

Recommended Plans

At Shinnecock Inlet, the recommendation is for a navigation channel as described under the Authorized Project (200 feet wide, -10 feet MLW) and an optimized deposition basin, relative to the Authorized Project (800 feet wide, -16 feet MLW). The channel and deposition basin should be maintained by dredging on a two year interval. Erosion of the beach immediately downdrift (west) of the inlet will be mitigated through the placement of the sand dredged from the deposition basin. In addition, sand will be dredged from the ebb shoal and placed downdrift of the ebb shoal reattachment to restore natural sediment pathways and offset downdrift erosion. Dredging of the ebb shoal would also be on a two year cycle, to coincide and be contracted with navigation dredging. This alternative is slightly less costly than Existing Practice and provides a greater level of flood protection downdrift of the inlet.

At Moriches Inlet, it is recommended that navigation conditions be maintained as authorized with annual dredging in the navigation channel (200 feet wide, -10 feet MLW) and deposition basin (800 feet wide, -14 feet MLW). Maintaining the Authorized Project dimensions will improve navigation conditions at the inlet by providing reliable access through the inlet that it is not presently available because of infrequent dredging. Sand dredged from the deposition basin will be placed downdrift of the ebb shoal attachment point to offset the longshore sediment transport deficit. Erosion of the beach immediately downdrift of Moriches Inlet is not an issue currently; however, should monitoring surveys indicate the need for beach fill, dredged material may be placed there too. It is further recommended that the ebb shoal be dredged and the material be placed farther downdrift than the ebb shoal reattachment point in order to restore natural sediment pathways, offset erosion to the west and to reduce flooding risk. This alternative is more costly than Existing Practice because of the increased dredging frequency (under Existing Practice the channel is not regularly maintained), but provides fore more reliable navigation, and a greater level of flood protection downdrift.

At Fire Island Inlet, the recommendation is for a navigation channel as described under the Authorized Project (450 feet, -10 feet MLW) and for an optimized deposition basin (variable width, -14 feet MLW) that extends farther east in the vicinity of the sand spit west of Democratic Point. The purpose of the optimized deposition basin is to provide a buffer from the rapid shoaling that encroaches on the navigation channel from the east. The material dredged from the deposition basin and navigation channel (on a 2 year cycle as current practice is) will continue to be placed both downdrift (Gilgo Beach) and updrift (Robert Moses State Park) beaches. In order to prevent continued growth of the ebb shoal and offset the resulting sediment deficit it is also recommended that sand be dredged from the ebb shoal coinciding with navigation dredging and placed downdrift along Gilgo Beach. In the long run this alternative will be less costly than Existing Practice and will provide a greater level of flood protection downdrift.

Since the recommended alternative at all three inlets involves dredging the ebb shoal, monitoring of ebb shoal recovery and evolution of downdrift beaches will be a critical element of this plan.

Finally, as the recommended plan at each inlet occurs on a two year (or shorter) dredging cycle, it is further recommended that maintenance dredging activities be contracted together so as to reduce mobilization and demobilization costs and provide a further cost savings.

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1. INTRODUCTION

The US Army Corps of Engineers, New York District (CENAN), is conducting a comprehensive feasibility-level reformulation of the shore protection and storm damage reduction project for the south shore of Long Island, New York, from Fire Island Inlet to Montauk Point (FIMP). The Reformulation Study is a multi-year and multi-task effort, involving project planning and engineering, economic analyses and environmental studies. Numerous study tasks are involved in the planning of storm damage reduction projects for the approximately 83-mile (133 kilometer) study area length.

The project area is located entirely in Suffolk County, Long Island, New York, along the Atlantic and bay shores of the towns of Babylon, Islip, Brookhaven, Southampton and East Hampton. The study area includes three estuarial bays, including, from west to east, Great South, Moriches and Shinnecock Bays. These estuaries are respectively connected to the Atlantic Ocean through Fire Island, Moriches and Shinnecock Inlets, all of which are Federal navigation projects (Figure 1-1). The project area includes the ocean and bay shorelines, Fire Island, Moriches and Shinnecock Inlets, barrier island beaches, the mainland, as well as suitable borrow areas for beach construction and replenishment.

1.1 Purpose

The purpose of this Reformulation Study is to determine and evaluate long term solutions for storm damage reduction along the south shore of Long Island from Fire Island Inlet to Montauk Point. Formulating a long-term solution to this problem will identify alternatives that can optimize benefits by reducing economic loss to the mainland and barrier beaches, while preserving important human and ecological habitats. Furthermore, the Reformulation Study will reevaluate the Authorized Plan (House Document 1960) based on existing study area conditions and in accordance with current Corps of Engineers' policies and study criteria.

It is unlikely, however, that the objectives of the study can be met by providing a classical coastal engineering solution involving the placement of a conventional beach fill template over the length of the study area. Such an approach is costly, requires a large amount of sand over the life of the project, and is not desired by all stakeholders. On the other hand, there are opportunities to: (1) provide economical flood damage protection that would be of great benefit to a large number of residents, (2) provide benefits to the environment by restoring elements of the coastal ecosystem, and (3) preserve/improve investments in navigation infrastructure.

An alternative approach with considerable merit would be to implement a Regional Sediment Management (RSM) program including long-term sediment bypassing (and possibly back-passing) at each of the inlets which would provide economical flood damage protection while restoring a key element of the coastal ecosystem. Sand-bypassing is nearly always a positive step inasmuch as it serves to facilitate and/or restore the natural movement of sand through the littoral system. To the extent that anthropogenic actions have retarded the movement through an inlet, sand-bypassing can compensate. Bypassed sand is normally placed along the receiving shoreline at one or more locations, but, it is difficult to fully emulate the natural sediment pathways. Inefficiencies can result in some loss of material. Furthermore, sand bypassing is a somewhat passive form of coastal protection compared to beachfills and coastal structures. It acts to reduce and/or eliminate long-term shoreline recession, but it seldom provides a sufficient volume of sand to prevent dune erosion and overwash during severe storms. On the other hand, it is clear that sand bypassing will significantly improve the health of the littoral system on a decadal scale and will act to reduce shoreline maintenance.

Aside from the need for improved regional sediment management, three prominent issues have been identified at for Fire Island, Moriches and Shinnecock Inlets:

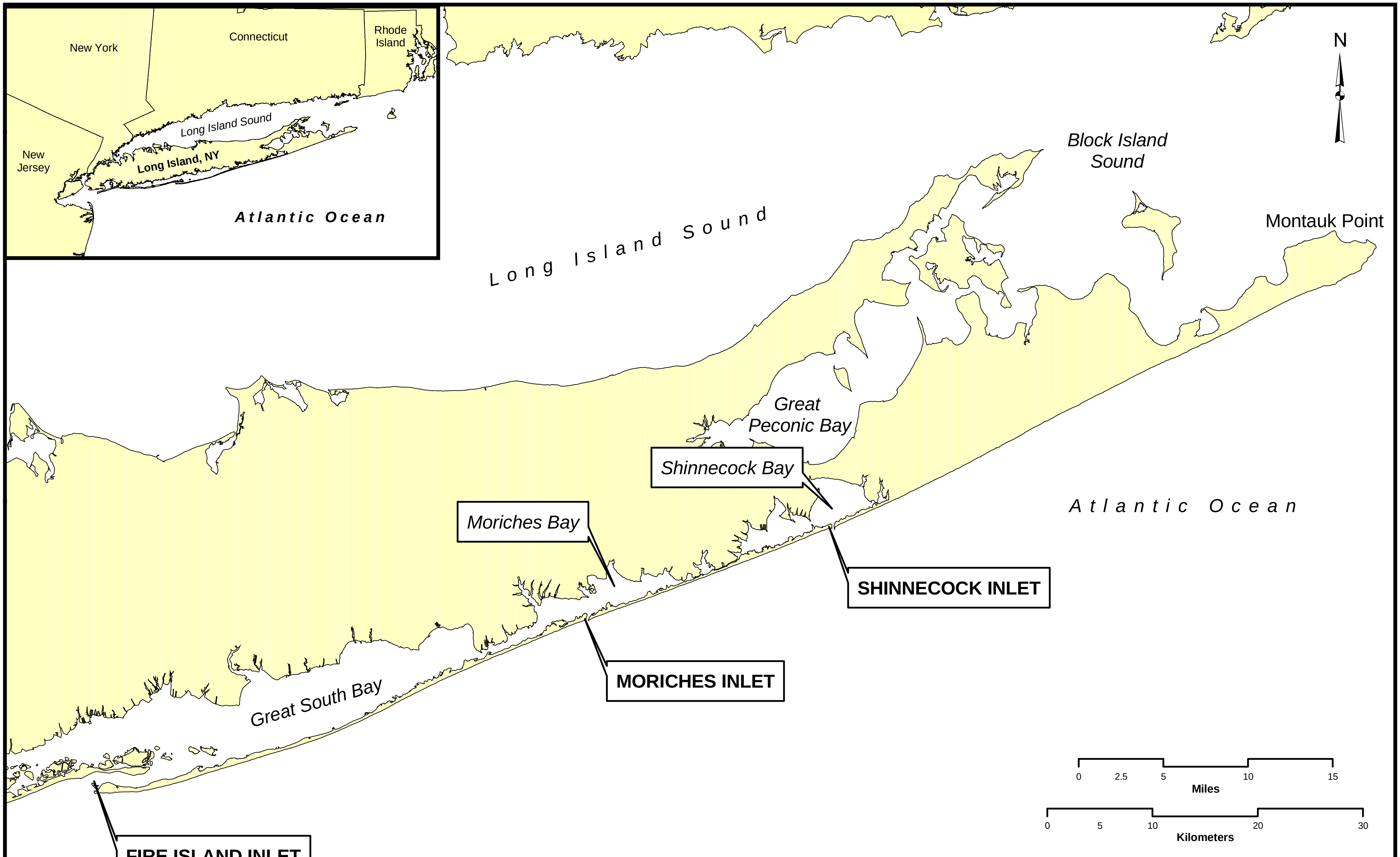
- Navigation reliability through the Federal navigation channels
- Erosion of the beach immediately downdrift of the inlets
- Erosion of the beach west of the ebb-shoal reattachment

Note that sand bypassing at the inlets and overall regional sediment management improvements will reduce erosion, particularly west of the ebb-shoal reattachment. The effort in this Inlet Modifications Study is to develop alternatives that will satisfactorily address each of the issues in the context of FIMP.

1.2 Scope

Due to the complexity of the inlets, the sediment transport paths, the inlets' relationship with the adjacent shorelines, and the great range of possible solutions, a phased approached was used to efficiently analyze the range of alternatives. The range of alternatives includes structural modifications of the jetties, changes in the navigation channel design, and changes in channel maintenance practices. A concept range of alternatives was developed and screened based on engineering, economic, institutional, and environmental criteria. Full-scale development (including numerical modeling) was conducted for selected alternatives. Specific scope tasks included:

1. Sediment Budget Updates and Improved Conditions
2. Update Inlet Dynamics Study
3. Evaluation of Navigation Issues
4. Morphological Modeling of Inlets
5. Inlet Modification Alternatives
6. Final Optimization and Recommendations for Inlet Modifications
7. Report Preparation and Coordination.



LOCATION MAP

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 1-1

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2. OVERVIEW OF EACH INLET

The following sections present a summary of the history of each inlet and the most relevant inlet-related coastal processes that affect the shoreline along the FIMP study area.

2.1 Shinnecock Inlet

Shinnecock Inlet is located along the Atlantic Coast, Long Island, New York, approximately 96 miles, by water, east of The Battery in New York City and 16 miles east of Moriches Inlet (see Figure 1-1). Shinnecock Inlet connects waters of the Atlantic Ocean with Shinnecock Bay. The inlet is generally aligned along a north-south direction as depicted in Figure 2-1 and Figure 2-2.

2.1.1 History

The present Shinnecock Inlet was formed during a hurricane on 21 September 1938. Prior to the break-through of the Inlet, the barrier beach between the bay and the ocean was continuous and a paved road crossed the present Inlet site. A shoal area in Shinnecock Bay 1 to 2 feet deep and about 3,000 feet wide extended parallel to the beach except for a narrow channel, which connected deep water in the Bay with an indentation in the barrier beach. The indentation possibly indicated the previous existence of a temporary inlet at this location, and, perhaps expectedly, the 1938 break-through occurred at this point.

Shinnecock Inlet was about 700-feet wide in 1939 and local interests constructed a 1,470-foot long jetty-type structure on the west side of the inlet to prevent its westward migration. The original structure was comprised of a timber piling bulkhead, 20 spur dikes normal to the bulkhead and a revetment fronting the bulkhead. By 1941, the inlet had widened to about 1,000 feet, and an inner and outer sand bar had formed. Maximum depths in the channel exceeded 20 feet, however, the navigable depth was merely 4 feet. The western jetty structure was repaired and a 130-foot long stone groin was added to its northerly end in 1947 due to prior storm damages.

New stone jetties were constructed on both sides of the inlet by local interests during the period from 1952 to 1953 and the west jetty was extended in 1954. After completion of the jetties, the width of the inlet was fixed at 800 feet. The inlet opening also rotated to conform to the jetties, which were constructed in a north-south direction. Inlet dimensions just prior to jetty construction were a depth from 3 to 6 feet and a width of about 500 feet. Two channels connected the inlet with deeper waters in the Bay. One of these channels, dredged at the request of the US Navy in 1943 to a depth of 6 feet and width of 100 feet, connected Shinnecock Inlet to the Intracoastal Waterway (ICW) near Ponquogue Bridge and had shoaled to a controlling depth of about 5 feet. The second channel, located on the east side of the inner sand bar had natural depths of about 8 to 9 feet.

Local interests constructed a bay channel 10 feet deep and 200 feet wide from a point inside the inlet to the ICW in 1958, and widened it to 300 feet in 1963, with maintenance dredging operations during 1973 and 1978. In 1966, local interests dredged through a shoal area northwest of the inlet and again in 1969 between the inlet and the ICW. The storm of 6 to 8 March 1962 caused widespread damages along the south shore of Long Island. At Shinnecock Inlet, the 1962 storm caused bank erosion and jetty damage (stone displacement) on the east jetty. The south end of the east jetty suffered damage and a jetty light was destroyed. The pile crib revetment at the north end of the west jetty was replaced by a rubble mound jetty in 1982.

Emergency dredging of Shinnecock Inlet was performed by the Corps of Engineers in April 1984, despite the absence of a Federal navigation project. Dredging was performed as a result of increased shoaling of the inlet, and included removal of approximately 162,000 cubic yards (cy) of material within and seaward of the Inlet.

USACE-NAN (1988) describes a plan for improvements to Shinnecock Inlet which consist of (1) an inner channel within Shinnecock Bay with a width of 100 feet and a low water depth of 6 feet, (2) an outer channel with a width of 200 feet and low water depth of 10 feet accompanied by an 800 foot wide by 20 foot deep deposition basin, (3) rehabilitation of the east and west jetties, and construction of a 1000 foot revetment facing the Bay on the eastern shoulder of the inlet. Construction of these improvements was initiated in late 1990 and completed in mid-1993. Initial construction of the navigation channel was performed from 1 October to 23 October 1990 with dredging of a total of 668,000 cy. Of this dredge volume, 138,000 cy was placed immediately west of the west jetty, 77,000 cy was used to fill a scour hole which had formed near the southern end of the west jetty, 193,000 cy was stockpiled on the east side of Shinnecock Inlet for use as fill behind the bayside revetment, and 260,000 cy was placed at Ponquogue Beach. Subsequent dredging of the deposition basin was conducted from 29 January to 14 May 1993 with removal of 475,000 cy. This material was placed in the scour hole (104,000 cy) and west of the west jetty (371,000 cy from 30 April to 14 May 1993). The last time the deposition basin was dredged was in March 2004, when 303,000 cy were removed.

The history of Shinnecock Inlet is presented in Table 2-1, which shows engineering and dredging records. It is evident that Shinnecock Inlet has been subject to less dredging prior to initiation of Federal involvement in the early 1990's than have either Moriches or Fire Island Inlets (see sections below). Throughout its history the Inlet itself has been subject to minor maintenance dredging, whereas the channels connecting to the ICW have required frequent excavation.

Table 2-1: Shinnecock Inlet Engineering Activities

Date	Activity	Description
Sep 1938	Inlet opens	Storm opens Inlet at its present location
1939	Jetty (western bank)	Inlet stabilization
1943	Channel dredging	Inlet to ICW (west channel)
1947	Jetty repair	Storm damage
1952 to 1953	Stone jetties (east & west)	Inlet stabilization
1954	West jetty extension	
1958	Channel dredging	Inlet to ICW (west channel deepened)
1963	Channel dredging	Inlet to ICW (west channel widened)
1966	Maintenance dredging	Inlet to ICW
1969	Maintenance dredging	Inlet to ICW
1973	Maintenance dredging	Inlet to ICW
1978	Maintenance dredging	Inlet to ICW
1984	Inlet dredging	Maintenance dredging 162,000 cubic yards
Oct 1990	Inlet dredging	Dredging of 668,000 cubic yards
Jan-May 1993	Deposition basin	Dredging of 475,000 cubic yards
1990 to 1993	Jetty rehabilitation	
Feb-Mar 1997	Maintenance dredging	East Cut dredging 250,000 cubic yards
Sep 1998	Channel & deposition basin	Maintenance dredging 440,000 cubic yards
Mar 2004	Channel & deposition basin	Maintenance dredging 302,590 cubic yards
Apr 2004	Western jetty rehabilitation	
Sources: USACE-NAN (1988), USACE-NAN (1998), CENAN Records (1998-2002)		

2.1.2 Coastal Processes Overview

The presence and continued evolution of Shinnecock Inlet has strongly influenced adjacent shoreline conditions, particularly west of the inlet. Historic interruption of westerly-directed sediment transport has created a large offset in the shoreline position across the inlet from east to west. Beach material is distributed throughout the inlet and is generally confined to three primary locations: (1) east of the east jetty in a large accretional fillet, (2) ebb-tidal shoal, including updrift and downdrift lobes or bars, (3) flood-tidal shoal. Nevertheless, Shinnecock Inlet has, albeit intermittently, permitted natural bypassing that serves to re-establish littoral transport to the downdrift shoreline. This effect is apparent in the shoreline near Ponquogue where a bulge in the shoreline points to the location where ebb shoal materials are bypassed to shore.

Previous Studies

Previous sediment budget estimates (Gravens et al., 1999) suggest a long-shore sediment transport rate of approximately 130,000 m^3/yr (170,000 cy/yr) entering the inlet from the east under *Existing* (circa 1999) conditions. In addition, USACE-NAN (1998) estimated *Existing* bypassing efficiency at approximately 56%, i.e., approximately 73,000 m^3/yr (95,000 cy/yr) are transported past the inlet and immediately adjacent beaches. The rest of this material, approximately 57,000 m^3/yr (75,000 cy/yr), is accumulated in the ebb shoal. According to *Existing* sediment budget results presented in Gravens et al. (1999), this inefficiency, an increase in sediment transport potential, and sea level rise result in erosion of the shoreline along Hampton and Tiana Beaches at a rate of 72,000 m^3/yr (94,000 cy/yr).

Updated Results

Recent (1995 to date) inlet maintenance practices (deposition basin dredging) yield approximately 65,000 m^3/yr (85,000 cy/yr), which is placed on the beach immediately west of the west jetty and east of the Ponquogue Point ebb-shoal re-attachment (approximately 3,000 feet). In addition, the east flood shoal channel has been dredged once in recent years to renourish the west beach resulting in an annualized placement rate of 37,000 m^3/yr (48,000 cy/yr). Volumetric and shoreline changes computed as part of this study (see Section 5) suggest that this beach has eroded at a rate higher than the combined placement rate (130,000 m^3/yr or 170,000 cy/yr). This situation, however, is not likely to be sustainable and a relative balance between placement and erosion is likely to be the case under *Existing* conditions. Studies to date and model results as part of this study suggest that most of this sand flows east and back into the channel and deposition basin where it is most likely redirected to the deposition basin and adjacent ebb shoal lobes.

Finally, analysis of recent bathymetric changes presented in Section 5 suggests that the ebb shoal may be accumulating material at a smaller rate than previously assumed (18,000 m^3/yr vs. 57,000 m^3/yr). The revised *Existing (c. 2001)* condition sediment budget for Shinnnecock Inlet suggest that approximately 79% of the net updrift westerly transport bypasses the inlet system, which includes the channels, deposition basin, shoals and west beach. The remaining 21% accumulates within the inlet shoals.

2.1.3 Navigation Conditions Overview

Shinnecock Inlet serves a fleet of 30-35 commercial fishing vessels docked in the maritime center immediately to the west of the inlet. The average annual catch exceeds ten million pounds with a value to the local community of $7-10 million (Steadman, 1999). The US Coast Guard (USCG) reports that navigation conditions at Shinnecock Inlet are generally good. While the outer channel authorized depth of -10 feet MLW appears to be too shallow for safe navigation of the typical commercial fishing boat, thanks to the presence of the 800-foot wide, 20-foot deep deposition basin, commercial vessels have little difficulty navigating the existing channel. Maintaining the depth of the entrance channel is important to the local fishing industry. The maintenance of the beach immediately west of the inlet is also important to the maritime center since the beach protects the road connecting the docks to the mainland. This road has experienced flooding during several recent storms (Steadman, 1999).

Additional details on navigation channel requirements and details on the vessels utilizing the inlet can be found in Section 7.

2.1.4 Needs and Opportunities

In recent years Shinnecock Inlet appears to have evolved to a new, more stable, morphological condition dictated by ongoing channel and deposition basin maintenance practices. Recent bathymetric surveys, as well as a thorough reanalysis of previously available information, suggests that the ebb shoal is not growing at the rates that have been hypothesized in recent studies although it may still be accumulating sediment. Although this finding is subject to some uncertainty due to the lack of comprehensive, 100% reliable, survey coverage, it certainly alters the previous perception of needs at this inlet, particularly as regards bypassing, and in the end will affect the development of modification alternatives at this inlet.

In any case, there is still a need for more effective bypassing at the inlet, and, more importantly, shoreline stabilization along the beach immediately west of the inlet which fronts the maritime center and access road. Note, however, that erosion of the shoreline west of the west jetty is not necessarily related inefficient inlet bypassing and that a shoreline stabilization feature appears to be a required component of any Shinnecock Inlet modification plan regardless of changes (if any) to bypassing practices.

Inlet cross-sectional area has remained fairly constant and continued stability is anticipated with slight increases and decreases as a result of dredging operations and sand transport into the inlet, respectively. The inlet (and Shinnecock Canal to the north) provides for significant water exchange between Shinnecock Bay and the Ocean, which results in acceptable water quality conditions in Bay and supports significant estuarine natural resources. Preservation and enhancement of these resources are important opportunities that should be addressed in any considered inlet modifications plans.

2.2 Moriches Inlet

Moriches Inlet is located along the Atlantic Coast in the Town of Brookhaven, Long Island, New York, approximately 80 miles, by water, east of The Battery in New York City and 30 miles east of Fire Island Inlet (see Figure 1-1). Moriches Inlet connects the Atlantic Ocean with Moriches Bay through a narrow barrier island opening generally characterized by a northeasterly-southwesterly orientation (Figure 2-3).

2.2.1 History

Available maps and records indicate that numerous inlets to Moriches Bay have existed during the last several centuries. There is no record of any inlets to Moriches Bay during the period 1839 to 1931. The present Moriches Inlet was opened during a storm on 4 March 1931. The inlet migrated about 3,500 feet west from 1931 to 1947 at which time its migration was halted when local interests constructed a long stone revetment on its western bank in an effort to stabilize the Inlet.

Initial surveys of Moriches Inlet in 1931 indicate an 800-foot wide channel with water depths reaching 18 feet. Inlet length at this time was approximately 1,500 feet. During inlet formation and its subsequent migration, large quantities of sand were deposited in the bay in the form of a flood tide delta. Initial inlet migration was characterized by an increasing inlet width, as deposition along the inlet eastern bank lagged behind erosion of the west bank. A hurricane in September 1938 caused extensive deposition of material in Moriches Bay adjacent to the inlet. Erosion of the banks of Moriches Inlet during this event was severe (i.e., approximately 250 and 1,000 feet of the western and eastern banks, respectively). In addition two breaches formed immediately west of the inlet. These breaches were closed artificially in May 1939. To preclude further westerly migration of the inlet, the rubble-mound revetment was constructed on the western inlet bank from 1947 to early-1948.

While the revetment was somewhat successful in maintaining the inlet, the continued growth of the Cupsogue Spit (to the east) caused the inlet channel to narrow. In November 1950, a storm caused large quantities of sand to wash over the barrier to the east depositing in the bay-connected inlet channel. This resulted in a reduction in the hydraulic efficiency of the inlet with associated shoaling. This condition led to the eventual closure of Moriches Inlet during a storm on 15 May 1951. Local interests constructed jetties on both sides of the inlet from 1952 to 1953 and the inlet was reopened during construction by a storm on 18 September 1953. Following stabilization of the inlet, its length (approximately 2,000 feet) and width (approximately 800 feet) were essentially fixed. The original channel was oriented slightly east of north entering the inlet. From a point approximately 800 feet from the inlet entrance, the main channel bifurcated to connect the east and west basins of Moriches Bay with the inlet. Local interests extended the jetties in 1954. A channel connecting Moriches Inlet to the ICW was dredged to a depth of 10 feet and width of 200 feet in 1958. This channel was widened to 300 feet in 1963.

Table 2-2 presents a summary of the known dredging and construction activities at Moriches Inlet since opening in March 1931. Total dredging quantities are near 3.5 million cubic yards, although the excavation quantities are unknown for several operations. Dredged material has typically been placed on the beaches or within nearshore areas east and west of Moriches.

In January 1980 a breach formed about 1000 feet east of the eastern jetty at Moriches Inlet as a result of a northeaster from 14 to 16 January 1980. Breach formation stemmed from the scouring of the eastern barrier's bay shoreline. The pre-breach vulnerability of the barrier is evident in available aerial photographs from December 1979 which preceded the 1980 breach by about 1.5 months. During the January 1980 northeaster, a breach occurred at the narrowest barrier section east of Moriches Inlet.

When first observed, the breach was estimated to be about 300 feet wide and 2 feet deep, however, by 20 January, the inlet had grown to a width of 700 feet with a depth of approximately 4.5 feet NGVD. By June 1980, the breach had a width of about 2,500 feet. Construction of the breach closure began in October 1980 and was completed in March 1981.

Improvements to Moriches Inlet since 1982 are described in USACE (1983) and consist of: (1) a 100-foot wide by 6-foot deep inner channel extending from the ICW to Moriches Inlet, (2) an outer channel extending from the ocean to the inner channel with a width of 200 feet, a low water depth of 10 feet and an advanced maintenance deposition basin. Construction activities were completed by 1986.

Table 2-2: Moriches Inlet Engineering Activities		
Date	**Activity**	**Description**
March 1931	Inlet opens	Storm opens Inlet 3,500 feet east of its location
1943	Channel dredging	Dredging of channel from Inlet to ICW
1947	West revetment construction	Inlet stabilization
1951	Storm closure	
1952 to 1953	Jetty construction	
1953	Storm opening	Storm opens Inlet at present location
1953	Channel dredging	Dredging of 747,000 cubic yards
1954	Jetties extended	
1957	Channel dredging	Maintenance dredging 37,000 cubic yards
1958	Channel dredging	Dredging of 366,000 cubic yards from Inlet to ICW
1959	Channel dredging	Maintenance dredging 100,000 cubic yards
1963	Channel dredging	Channel widened
1964	Channel dredging	Maintenance dredging 59,000 cubic yards
1966	Channel dredging	Dredging of 678,000 cubic yards
1969	Channel dredging	Maintenance dredging 151,000 cubic yards
1973	Dredging	Maintenance dredging 138,000 cubic yards
1977	Dredging	Maintenance dredging 59,000 cubic yards
1978	Dredging	Maintenance dredging 218,000 cubic yards
1985	Dredging	Dredging of 355,000 cubic yards
1986	Dredging	Maintenance dredging 41,000 cubic yards
1996	Dredging	Maintenance dredging 256,600 cubic yards
Oct 1998	Dredging	Maintenance dredging 186,200 cubic yards
Feb 2004	Dredging	Maintenance dredging 250,250 cubic yards
Sources: USACE-NAN (1998), USACE-NAN (1983), CENAN Records (1998-2002)		

2.2.2 Coastal Processes Overview

A notable offset in the shoreline progressing east to west across Moriches Inlet reflects shoreline impacts associated with the westerly-directed littoral drift. Nonetheless, shoreline conditions west of Moriches Inlet are generally characterized by a relatively robust barrier system with wide beaches and high dunes. Beach widths increase notably approximately 4,000 feet west of inlet, and reflect dredged material placement and natural bypassing of Moriches Inlet. It should also be noted that historic updrift sediment accumulation (fillet) east of Moriches Inlet appears to be less than at Shinnecock Inlet.

East of Moriches Inlet the barrier landmass is narrow. This condition persists despite the formation of an accretional fillet immediately adjacent to the east Moriches jetty. It is further noted that apparent sand volumes within this fillet are notably less than present at Fire Island and Shinnecock Inlets. This condition is judged to have arisen due to four primary factors, namely: (1) the Westhampton groin field reduces transport reaching Moriches Inlet, (2) historical migration of Moriches Inlet left a narrow barrier segment, (3) tidal currents have scoured the bayside shoreline, (4) a shorter updrift (east) jetty.

Moriches Inlet differs from other inlets in the study area given its proximity to the Westhampton groin field. Inasmuch as tidal inlet behavior is related to littoral drift conditions, the groin field may modify the impact of Moriches Inlet on barrier shorelines.

Previous Studies

Previous sediment budget estimates (Gravens et al., 1999) suggest a long-shore sediment transport rate of approximately 184,000 m^3/yr (240,000 cy/yr) entering the inlet from the east under *Existing* (circa 1999) conditions. USACE-NAN (1998) estimated *Existing* bypassing efficiency at approximately 29%, i.e., approximately 69,000 m^3/yr (90,000 cy /yr) are transported past the inlet and immediately adjacent beaches. The rest of this material, accumulates in the deposition basin (approximately 46,000 m^3/yr or 60,000 cy /yr) and in the ebb shoal (approximately 69,000 m^3/yr or 90,000 cy/yr). Note that this budget is based on underlying historical changes from 1979 to 1995 and assumed *Existing* coastal processes that take into consideration the effects of the Westhampton Interim Project and the present conditions of the inlet and shoals. According to sediment budget results presented in Gravens et al. (1999), this inefficiency, an increase in sediment transport potential, and sea level rise, result in erosion of the shoreline along Smith Point County Park and the Wilderness Area at a rate of 145,000 m^3/yr (190,000 cy/yr).

Updated Results

Recent (1995 to date) inlet maintenance practices (deposition basin dredging) yield approximately 56,000 m^3/yr (73,000 cy/yr), which is placed on the downdrift beach. Volumetric and shoreline changes computed as part of this study (see Section 5) as well as a recent analysis by Allen et al. (2002) suggest that the ebb shoal is relatively stable feature and that inlet bypassing is occurring more naturally and efficiently than it had in the past. Approximately 89% of the net westerly transport under *Existing (c. 2001) Conditions* bypasses the inlet system, which includes the channel, deposition basin, shoals and west beach. The other 11% accumulates within the ebb and flood shoals.

2.2.3 Navigation Conditions

Navigational charts (NOS 12352) list Moriches Inlet as officially closed to navigation "*due to rapidly changing shoaling conditions.*" Notes advise mariners, "*it is considered unsafe for Mariners to attempt to navigate this inlet at any time.*" Nonetheless, recreational boaters use the inlet on a regular basis. The Coast Guard reports an average of 200 boats use the inlet on a typical weekend. Since no navigation buoys are maintained at the inlet, boats navigate through the ebb shoal by aiming for the gaps in the breaking waves. Depths over the ebb shoal can be as shallow as -2 feet MLW. In addition, Coast Guard personnel consider the navigation conditions of the inlet and the inner channels to be poor.

2.2.4 Needs and Opportunities

Under existing conditions, Moriches Inlet appears to effectively bypass a large percentage of the westerly-directed net long shore transport. In addition, the inlet cross-section has remained fairly stable and continued stability of the inlet is anticipated. Moreover, the inlet provides for adequate water exchange between Moriches Bay and the Ocean, which results in acceptable water quality conditions in Bay and supports significant estuarine natural resources.

Navigation, however, is extremely dangerous through the inlet. Although navigational charts and the Coast Guard consider Moriches Inlet unsafe for navigation boaters continue using it. Therefore, the principal need (and opportunity) with regards to modifications at Moriches Inlet is for improved navigation. Other needs (opportunities) include continued sand bypassing, maintenance of stable shorelines and associated storm protection in areas adjacent to the inlet, preservation and enhancement of water quality and existing natural resources.

2.3 Fire Island Inlet

Fire Island Inlet is located along the Atlantic Coast of Long Island, New York, approximately 50 miles, by water, east of The Battery in New York City (Figure 1-1). Fire Island Inlet connects the Atlantic

Ocean with Great South Bay through a generally east-west aligned channel between Oak Beach on the north and the western end of Fire Island on the south (see Figure 2-5). Total inlet length is about 3.5 miles with a width of approximately 3,500 feet. Recent conditions at Fire Island Inlet are depicted in Figure 2-6.

2.3.1 History

Available records indicate that only Fire Island Inlet has existed continuously since the early 1700's. The position of the inlet, however, has varied significantly over time and has migrated a total distance of about 5 miles from a point east of its present position between 1825 and 1940. Federal jetty construction at Democrat Point in 1941 halted this westward migration. Continued dredging of the inlet has been performed to maintain a navigable channel. Sand dredged from Fire Island Inlet has been placed to the west and north of the inlet to offset the marked downdrift erosion in those areas arising from the interruption of the predominate mode of westerly-directed littoral transport.

Data cited in Saville (1960) indicate that erosion at Oak Beach prior to stabilization measures at Fire Island Inlet was a problem from 1930 to 1960. Construction activities began in earnest adjacent to Fire Island Inlet in 1927 with the placement of 40 million cubic yards of sand to create eighteen miles of beach from Jones Inlet to Captree State Park. Material was excavated from adjacent bays and lagoons. Fire Island Inlet migration was halted in 1941 by the construction of the Federal jetty on the eastern shoulder of the inlet. While the jetty was successful in checking inlet drift for the next decade or so, it did not provide an adequately stabilized navigation channel.

Modification of the Federal project to provide a channel through the inlet 10 feet deep and 250 feet wide and possible extension of the jetty was recommended in 1948. The modification authorized in 1950 provided only for initial and maintenance dredging of the navigation channel with extension of the jetty pending experience that maintenance dredging proved too costly. As maintenance of the channel was anticipated as difficult and based on experience at East Rockaway Inlet, no definite channel alignment was selected. A storm in 1953 created a channel approximating authorized dimensions along an alignment extending southwest from a gorge fronting Oak Beach and generally followed the alignment of the existing jetty. Maintenance dredging procedures were initiated in 1954 to preclude shifting of the channel to the north. Nearly annual maintenance dredging of the channel was performed following 1954 to the authorized channel dimensions, as shown in Table 2-3.

Although continued maintenance dredging provided an adequate navigation channel, shoaling of the inlet and erosion of the Oak Beach shore were of concern. This shoaling resulted in a tendency for the inlet channel to relocate north; causing marked erosion of the Oak Beach area to the north and west of the inlet. This behavior also resulted in the reduction of the inlet width to approximately 1,200 feet in 1956 relative to a width of nearly 2,000 feet in 1941.

The conditions described led to authorization and subsequent modification of the Federal project at Fire Island Inlet to include restoration and shore protection of the shore from Fire Island Inlet to Jones Inlet. Project construction took place throughout the 1950's. Measures undertaken included: (1) a navigation channel 250 feet wide with a mean low water depth of 10 feet from that depth in the ocean to that depth in the bay through the dredged area adjacent to the Democrat Point jetty; (2) dredging of an area within the inlet to affect a southward shift of the navigation channel and to provide fill material for the feeder beach located about two miles west of Democrat Point and Oak Beach, including dredging to a mean low water depth of 18 feet for a length of 6,800 feet and width varying from 600 to 1,200 feet; and (3) construction of a one-half mile sand dike across the inlet gorge extending southeast from Oak Beach. These measures, completed by 1959, attempted to stabilize the navigation channel and to minimize erosion on the north and west shores of the inlet.

While the sand dike was moderately successful in maintaining channel position and limiting erosion on the western shore, the inlet continued to experience shoaling in amounts sufficient to require frequent dredging. Continued growth of the shoal west and north of the jetty shifted the channel requiring further dredging of the channel within Fire Island Inlet. Based on this experience, modification of the Federal project was authorized in 1971 to provide for a sand bypassing system at Fire Island Inlet. The proposed plan consisted primarily of a littoral reservoir (akin to a deposition basin) in the inlet entrance and sediment-rehandling basin in the inlet interior. Provision was made for additional structures pending future needs, including jetty extension and a revetted sand dike. In summary, the final authorized project, as modified, included: (1) littoral reservoir, (2) navigation channel, (3) rehandling basin, (4) feeder beach, (5) land reclamation, (6) revetted sand dike, (7) jetty extension and (8) deflector dike. Actions since 1971 have included maintenance of the navigation channel, sporadic dredging of the littoral reservoir immediately adjacent to the jetty, land reclamation and feeder beach sand placement. The rehandling basin, revetted sand dike, jetty extension and deflector dike were not constructed.

Historic engineering activities of significance at Fire Island Inlet, including jetty construction, maintenance dredging, adjacent shore protection operations, and other pertinent activities are summarized in Table 2-3. This table summarizes Fire Island Inlet historic dredging activities that show that from 1954 to 1994 nearly 21 million cubic yards (cy) of sediment was dredged from the inlet channel and shoal areas. Records of these dredging operations indicate that much of these materials were placed west of the Inlet along the barrier island shore from Fire Island to Jones Inlet. Placement records for numerous operations are not available. A single operation placed approximately 200,000 cy at Democrat Point in 1994, and another consisted of 600,000 cy dredged from the flood shoal in 1993 with placement in front of the traffic circle at Robert Moses State Park. Other recent dredging operations are listed in Table 2-3.

Table 2-3: Fire Island Inlet Engineering Activities

Date	Location	Activity	Description
1927	Inlet to Jones Inlet	Ocean Parkway	40 million cubic yards of embankment fill
1941	Eastern Bank	Jetty	5,000-ft stone jetty to halt Inlet migration
1946	Inlet and Oak Beach	Dredging & Beachfill	Channel dredged to 15 feet deep and 200 feet wide; 400,000 cubic yards from channel placed on 4,000-ft segment of Oak Beach
1946 to 1955	Oak Beach	Beachfill	Nourishment averaged 150,000 cy/year
1950	Inlet	Design	Authorized channel dimensions modified to 10 feet deep & 250 feet wide
1953	Inlet	Storm	Storm results in new channel dimensions as modified in 1950
1954	Inlet	Dredging	Maintenance dredging 75,000 cubic yards
1954	Inlet Channel	Dredging	Maintenance dredging 43,000 cubic yards
1955	Inlet	Dredging	Advanced dredging to preclude northward channel shift
1955 to 1959	Gilgo & Tobay Beaches	Beachfill	1,000,000 cubic yards along eastern segment of Fire Island to Jones Inlet reach
1956	Inlet Channel	Dredging	Maintenance dredging 40,000 cubic yards
1957	Inlet Channel	Dredging	Maintenance dredging 40,000 cubic yards
1958	Inlet Channel	Dredging	Maintenance dredging 127,000 cubic yards
1959	Inlet Channel	Dredging	Maintenance dredging 83,000 cubic yards
1959	Feeder Beach	Dredging &	2,000,000 cubic yards placed on feeder beach

Table 2-3: Fire Island Inlet Engineering Activities			
		Beachfill	west of Inlet as obtained from dredging of Inlet ebb shoal
1959	Oak Beach	Dredging & Dike	1,100,000 cubic yards dredged from ebb shoal to construct one-half mile closure dike across channel along Oak Beach; referred to as the Thumb; later fortified with riprap
1960	Inlet Channel	Dredging	Maintenance dredging 54,000 cubic yards
1961	Inlet Channel	Dredging	Maintenance dredging 238,000 cubic yards
1961	Gilgo and Tobay Beaches	Dredging & Beachfill	2,200,000 cubic yards dredged from Bay to protect Ocean Parkway
1963	Inlet Channel	Dredging	Maintenance dredging 103,000 cubic yards
1964	Inlet Channel	Dredging	Maintenance dredging 11,000 cubic yards
1964	Feeder Beach	Dredging & Beachfill	1,925,000 cubic yards dredged from ebb shoal
1965	Inlet Channel	Dredging	Maintenance dredging 477,000 cubic yards
1966	Inlet Channel	Dredging	Maintenance dredging 109,000 cubic yards
1967	Inlet Channel	Dredging	Maintenance dredging 47,000 cubic yards
1967	Inlet & Thumb	Dredging	Material excavated to bolster revetted sand dike
1968	Inlet Channel	Dredging	Maintenance dredging 194,000 cubic yards placed at Cedar Beach
1968 to 1969	Gilgo	Dredging & Beachfill	Dredged from north of Gilgo Beach and placed on beach east of Gilgo Pavilion
1969	Inlet Channel	Dredging	Maintenance dredging 727,000 cubic yards placed at Gilgo Beach
1969	Inlet Channel	Dredging	Maintenance dredging 147,000 cubic yards
1970	Inlet Channel	Dredging	Maintenance dredging 284,000 cubic yards
1971	Inlet Channel	Dredging	Maintenance dredging 79,000 cubic yards
1972	Inlet Channel	Dredging	Maintenance dredging 51,000 cubic yards
1972	Inlet Channel	Dredging	Maintenance dredging 150,000 cubic yards
1972	Inlet Channel	Dredging	Maintenance dredging 221,000 cubic yards
1973	Inlet Channel	Dredging	Maintenance dredging 64,000 cubic yards
1973	Inlet Channel	Dredging	Maintenance dredging 247,000 cubic yards
1974	Inlet Channel	Dredging	Maintenance dredging 1,069,000 cubic yards
1975	Inlet Channel	Dredging	Maintenance dredging 931,000 cubic yards
1976	Inlet Channel	Dredging	Maintenance dredging 2,271,000 cubic yards
1985	Inlet Channel	Dredging	Maintenance dredging 374,000 cubic yards
1986	Inlet Channel	Dredging	Maintenance dredging 355,000 cubic yards
1987	Inlet Channel	Dredging	Maintenance dredging 422,000 cubic yards
1988	Inlet Channel	Dredging	Maintenance dredging 1,000,000 cubic yards placed at Gilgo Beach
1990	Inlet Channel	Dredging	Maintenance dredging 798,000 cubic yards placed at Gilgo Beach
1992	Inlet Channel	Dredging	Maintenance dredging 1,200,000 cubic yards placed at Gilgo Beach
1993	Inlet Channel	Dredging	Maintenance dredging 1,400,000 cubic yards placed at Gilgo Beach

Table 2-3: Fire Island Inlet Engineering Activities			
1993	Flood Shoal	Dredging & Beachfill	Placement fronting traffic circle at Robert Moses
1994	Inlet Channel	Dredging	Maintenance dredging 1,300,000 cubic yards placed at Gilgo Beach
1994	Inlet Channel	Dredging	Dredging 200,000 cubic yards placed Democrat Point
1997	Inlet Channel	Dredging	Dredging 1,081,861 cubic yards: 718,923 placed at Gilgo Beach and 362,938 at Robert Moses SP
1999 to 2000	Inlet Channel	Dredging	Dredging 1,107,718 cubic yards: 972,337 placed at Gilgo Beach and 135,381 at Robert Moses SP
Dec 2001 to Mar 2002	Inlet Channel	Dredging	Dredging 1,490,784 cubic yards: 1,325,990 placed at Gilgo Beach and 164,794 at Robert Moses SP
2003 to 2004	Inlet Channel	Dredging	Dredging 1,082,246 cubic yards: 953,263 placed at Gilgo Beach and 135,983 at Robert Moses SP
Sources: USACE-NAN (1998), USACE-NAN (1985), CENAN Records (1998-2002)			

2.3.2 Coastal Processes Overview

Prior to stabilization, Fire Island Inlet constituted a significant barrier to westerly-directed transport. This trapping resulted in westerly migration of the inlet, which was later arrested by construction of the jetty at Democrat Point in 1941. Erosion of the shorelines west of and in the lee of the inlet generally paralleled the migratory path of the inlet throat. Following jetty construction, increased shoreline erosion occurred at Oak and Gilgo Beaches. In concert with sand bypassing, construction of the "Thumb" has been relatively successful in stabilizing shoreline positions within and immediately west of the inlet entrance. Currently, significant annual dredging operations are required to maintain the channel.

Previous Studies

Previous sediment budget estimates (Gravens et al., 1999) suggest a long-shore sediment transport rate of approximately 188,000 m^3/yr (245,000 cy/yr) entering the inlet from the east and under existing (circa 1999) conditions. USACE-NAN (1998) suggest a that the inlet entrance at Fire Island Inlet experiences a net accumulation of 535,000 m^3/yr (700,000 cy/yr), despite the fact that approximately 306,000 m^3/yr (400,000 cy /yr) are dredged every year. This material is placed within the beaches adjacent to the inlet: Gilgo State Park (most of it) and Robert Moses State Park. Finally, USACE-NAN (1998) suggest an additional 466,000 m^3/yr (609,000 cy /yr) entering the inlet from the west (Gilgo Beach) in order to balance the inlet budget.

Updated Results

Recent (1995 to date) maintenance dredging operations have yielded approximately 279,000 m^3/yr (365,000 cy/yr), which were placed at Gilgo Beach (80%) and Robert Moses State Park (20%). Volumetric and shoreline analyses performed as part of this study (see Section 5) suggest that the ebb shoal is accumulating sediment at significantly smaller rate than previously assumed: 68,000 m^3/yr (89,000 cy/yr). Moreover, the revised sediment budget does not require easterly-directed transport entering the inlet from Gilgo Beach in order to be balanced.

2.3.3 Navigation Conditions

Fire Island Inlet is used by a combination of commercial charter fishing boats, head boats, and recreational traffic. Most of the inlet's commercial fleet operates out of Captree Basin on the east end of Jones Island (Steadman, 1999). The inlet is subject to rapid shoaling conditions. As a result, the navigation channel has been realigned several times in the last 30 years. Navigation buoys placed immediately after maintenance dredging in approximately 17 feet of water will shoal to six feet of water within two years. The Coast Guard moves the buoys about once a year to mark the deepest part of the channel and avoid the shoals.

While the number of navigation aids is considered sufficient, navigation during the night can be challenging, as the aids are not illuminated. Due to shoaling, the light at the end of the Federal jetty has been discontinued. The light is now in the middle of Democrat Point shoal and is more of a hazard than an aid to navigation.

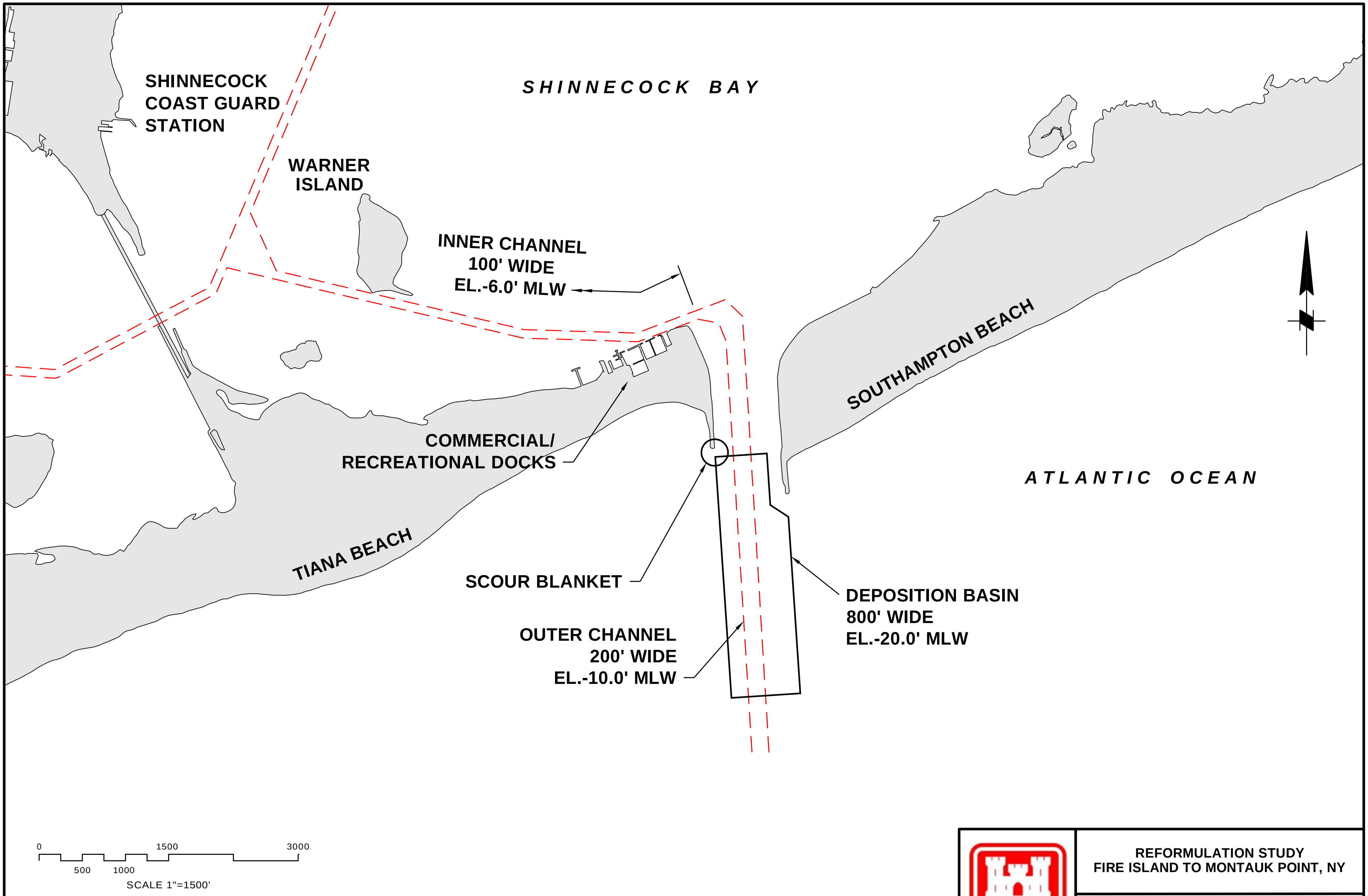
Breaking waves have been observed up to the edge of the marked channel. Typical currents in the inlet are strong: on the order of 3 knots. Overall, navigation conditions of Fire Island Inlet are considered fair.

2.3.4 Needs and Opportunities

Although an *Existing* sediment budget has been formulated as part of this study (see Section 5), its reliability is fairly limited due to the lack of comprehensive surveys of the inlet, particularly the large ebb shoal features located southwest of the outer channel. Nonetheless, recent data suggests that the ebb shoal may not be accumulating as much material as previously estimated. In addition, the beaches west of the inlet along Jones Island appear to be in fairly good condition as a result of historic bypassing and perhaps a wave climate and shoreline alignment more conducive to shoreline stability.

An inlet stability analysis (see Section 5.3), however, suggests that the inlet is only marginally stable with a tendency to shoal. In other words, the inlet may close if events cause significant accumulation of sediment and decreased cross-sectional area. In fact, in spite of numerous studies over the last 60 years leading to various recommendations and implementation of an optimum channel alignment and a deposition basin, the channel continues to shoal at significant rates requiring frequent dredging and relocation of navigation aids. Moreover, although the Coast Guard considers navigation conditions fair, they can be challenging at times due to rapidly changing morphology along the channel, particularly in the vicinity of the sand spit. Therefore, improved inlet stability and navigation conditions (i.e., reduced channel maintenance) are principal needs at Fire Island Inlet.

As in Shinnecock and Moriches Inlets other needs (opportunities) include continued sand bypassing, maintenance of stable shorelines and associated storm protection in areas adjacent to the inlet preservation and enhancement of water quality and existing natural resources.



SHINNECOCK INLET - Federal Navigation Channel

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 2-1

Filename: CAD/3982-28/DWG/NAV_CHANNEL.dwg (Shinnecock)

AERIAL PHOTO OF SHINNECOCK INLET (FALL 1998)

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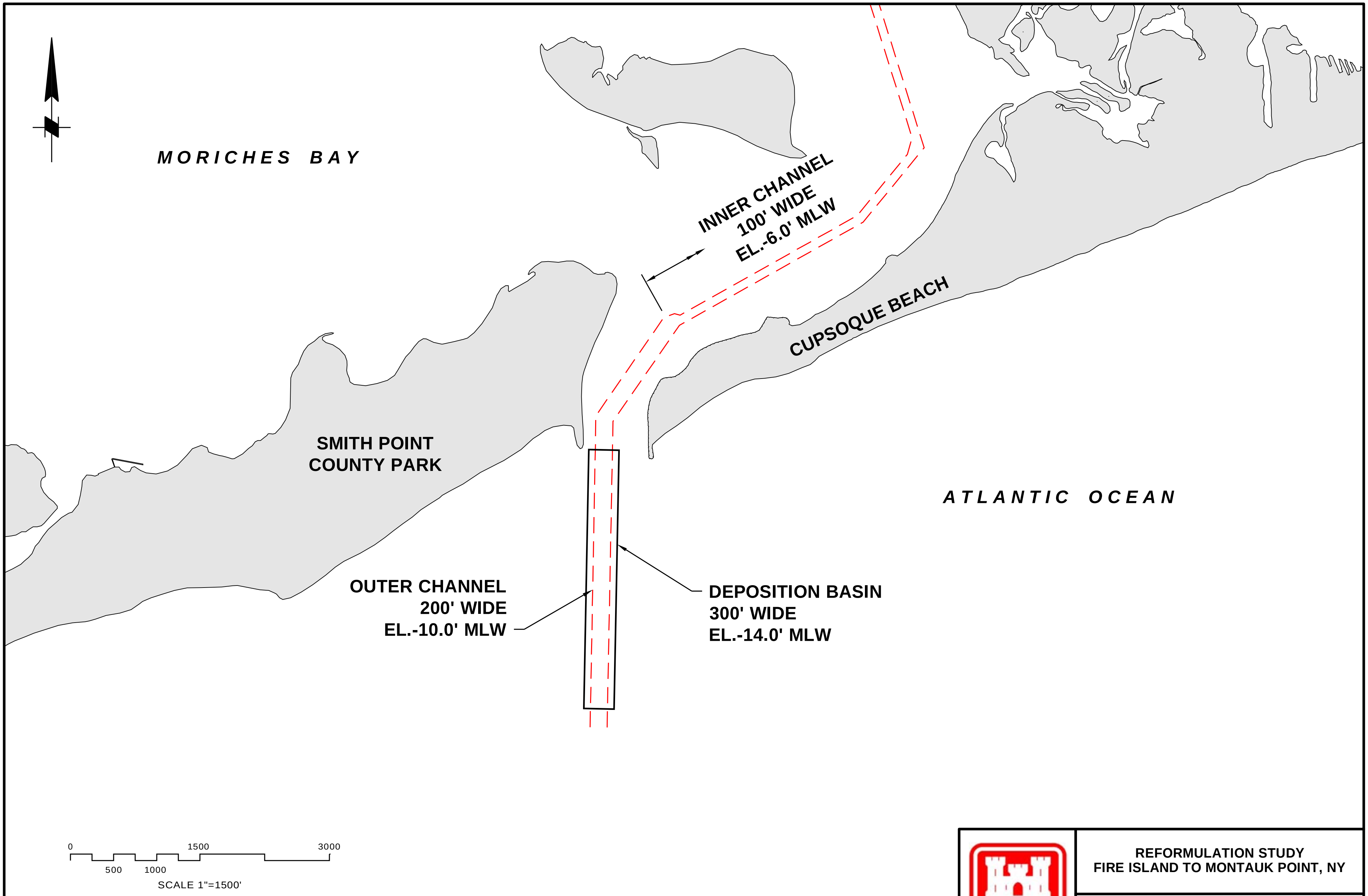
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 2-2

CAD/3982-23/Report2002/Aerial photos



MORICHES INLET - Federal Navigation Channel

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NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 2-3

Filename: CAD/3982-28/DWG/NAV_CHANNEL.dwg (Moriches)

AERIAL PHOTO OF MORICHES INLET (Spring 1998)

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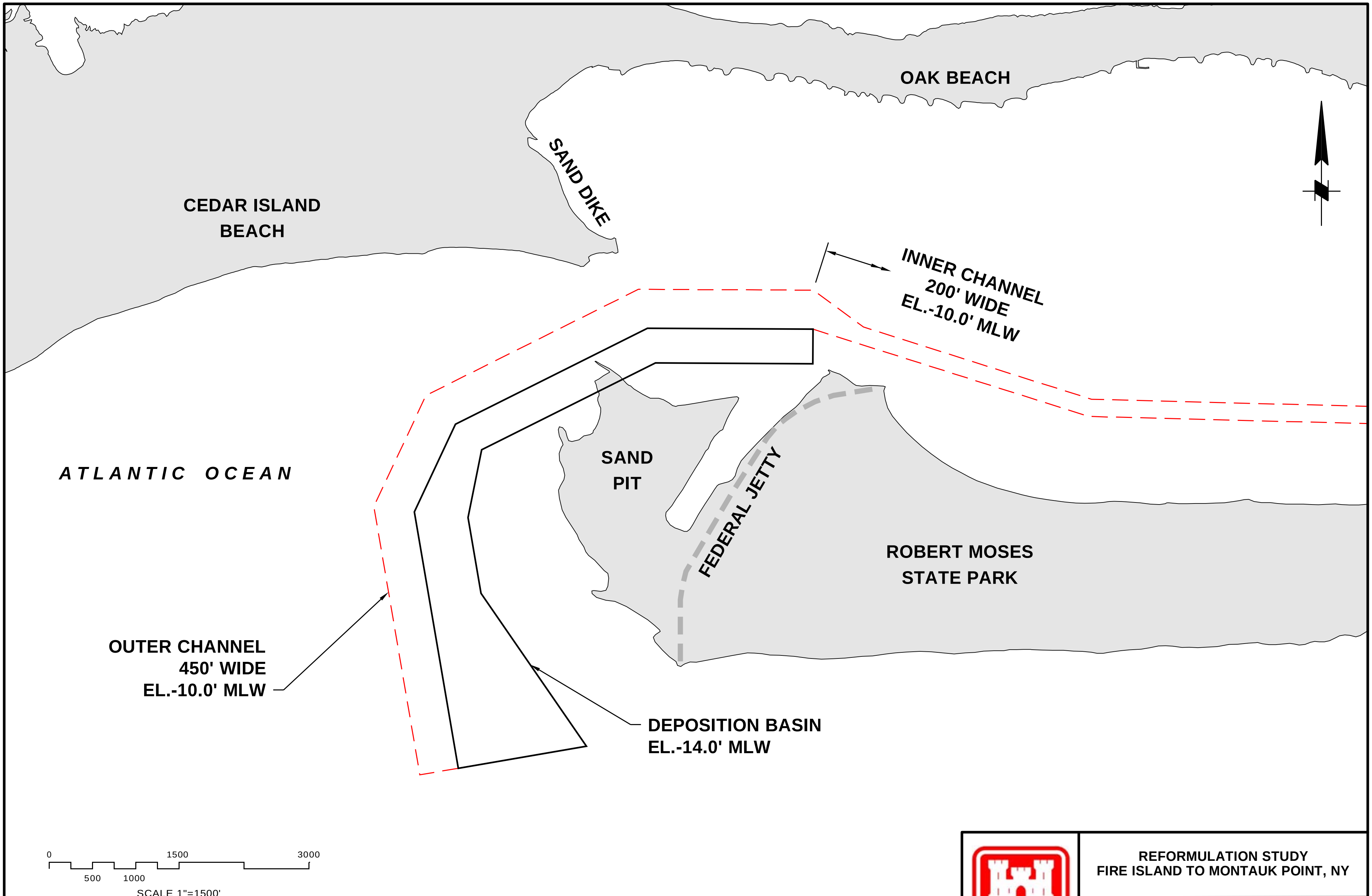
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 2-4



FIRE ISLAND INLET - Federal Navigation Channel

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REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 2-5

Filename: CAD/3982-28/DWG/NAV_CHANNEL.dwg (Fire Island)

AERIAL PHOTO OF FIRE ISLAND INLET (Fall 1998)

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REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 2-6

CAD/3982-23/Report2002/Aerial photos

3. INLET MODIFICATION ALTERNATIVES AND INITIAL SCREENING

3.1 Approach

The purpose of this study is to develop inlet modification alternatives that provide reliable navigation through the Federal navigation channels and maximize sand bypassing in order to restore, to the extent possible, natural sediment pathways and reduce adjacent shoreline erosion. Alternatives may consist of engineering modifications (e.g., navigation channel, inlet structures, etc.) and/or management modifications (e.g., optimize dredging, etc.). The selected alternatives will be combined with other optimized structural and non-structural features into a comprehensive plan for Fire Island to Montauk Point.

The design of inlet modification alternatives is being conducted in a phased approach which includes development of a concept list of alternatives, preliminary screening analysis of a range of alternatives selected based on a fatal-flaw analysis, detailed level design of a selected group of alternatives, and final screening and recommendations.

A screening methodology was specifically developed for this study. Given the relatively large number of alternative plans originally developed, a phased screening approach was proposed. First, a *fatal flaw analysis* was conducted to eliminate any alternatives that are clearly inadvisable or include negative effects that cannot be offset by any degree of benefits from other factors. Remaining alternatives were further evaluated based on several criteria initially developed by the CTMG and USACE-NAN. The results of this analysis were used to select a smaller group of alternative inlet modification plans that will be developed to a feasibility level of detail, to include detailed numerical modeling. Finally, a single optimized alternative per inlet will be developed and recommended. The screening process is summarized in Table 3-1. The idea behind the first two phases of the screening task is to develop a reduced list of plans that address the project needs, are generally feasible, and by and large meet the requirements of the sponsors as well as other stakeholders.

Table 3-1: Alternative Development and Screening Methodology
1. Develop Concept List of Alternative Plans
2. Fatal Flaw Analysis
3. Initial Multicriteria Screening Analysis
4. Feasibility Level Design
5. Detailed Modeling
6. Final Screening Optimization and Recommendation

3.2 Concept List of Alternatives

A Coastal Technical Management Group (CTMG) meeting was held on November 15, 2001 to, among other things, brainstorm an initial concept list of inlet modifications alternatives and screening criteria (USACE/NAN, 2002). Alternatives presented at the meeting were recorded regardless of consistency with USACE policies/authorization or the policies of any of the other agencies/sponsors represented in the meeting. More importantly, some of the alternatives discussed at this meeting do not qualify as complete inlet modification plans to the extent that they do not necessarily address all of the project needs as listed in Section 1.1. Therefore, the following discussion differentiates between alternative inlet modification *plans* and *components*. Inlet modification *components* are measures intended to address one or more specific issues such as bypassing, localized erosion, inlet stability, navigation reliability, regional sediment management, recreation, etc. Note that some of these issues may not directly correspond to the stated project needs (e.g., recreation). Inlet modification *plans* are combinations of *components* that together address (to varying degrees) the stated needs for the inlets (e.g., mechanical bypassing with a semi-fixed plant plus maintenance of navigation channel). Subsequent to the meeting, all of the presented alternative *components* were reorganized according to their intended function (i.e., improved bypassing, direct shoreline stabilization, or reduced channel maintenance/improved efficiency) and combined into set of complete alternative *plans*. Plans for Shinnecock, Moriches and Fire Island inlets are presented in Tables, 3-1, 3-2, and 3-3, respectively.

A subsequent CTMG meeting was held on January 30, 2003 to present preliminary study findings with regards to the sediment budget work and a preliminary alternative screening analysis (USACE/NAN, 2003). During this meeting CTMG members made suggestions with regards to the initial concept list of alternative inlet modification plans, the screening criteria, and screening methodology. The following tables include alternatives presented at this meeting. In addition, the screening process summarized in sections below incorporates comments and suggestions made at the meeting and in subsequent correspondence.

Table 3-2: Conceptual Alternatives – Shinnecock Inlet
Existing Practice (Dredging of Channel & Deposition Basin as Required)
Do Nothing
Channel & Deposition Basin Realignment
Deposition Basin (Not Channel) Realignment
Reduced Dimensions of Deposition Basin
Dredging the Ebb Shoal
Dredging the Flood Shoal
Offshore Dredging (for West Beach)
Spur Structure at West Jetty
Nearshore Structures along West Beach
Shortening the East Jetty
Sand Trapping and Bypassing System Updrift
Jetty Opening and Nearshore Breakwater
Weir Jetty and Sediment Trap
Offshore Breakwater
Modification of the Ponquogue Ebb Shoal Attachment
Relocation of the Maritime Center within Shinnecock Bay
Relocation of the Maritime Center to Smith Point
Reduce Authorized Channel Depth
Ebb Shoal Nourishment
Non-Floating Dredging & Bypassing Systems
Backpassing
Realignment of Complete Inlet System
Removal of the Jetties
Inlet Closure
Move the Inlet to a New Location
Closing the Inlet to Navigation
Extend the Westhampton Groin Field

Table 3-3: Conceptual Alternatives – Moriches Inlet
Existing Practice (Infrequent Dredging of Channel & Deposition Basin)
Do Nothing
Authorized Project
Channel & Deposition Basin Realignment
Deposition Basin (Not Channel) Realignment
Increased Dimensions of Deposition Basin
Dredging the Ebb Shoal
Dredging the Flood Shoal
Non-Floating Dredging & Bypassing Systems
Extension of the East Jetty
Sand Trapping and Bypassing System Updrift
Jetty Opening and Nearshore Breakwater
Weir Jetty and Sediment Trap
Offshore Breakwater
Limit Navigation
Backpassing
Realignment of Complete Inlet System
Removal of the Jetties
Inlet Closure
Move the Inlet to a new Location
Closing the Inlet to Navigation

Table 3-4: Conceptual Alternatives – Fire Island Inlet
Existing Practice (Dredging of Channel & Deposition Basin as Required)
Do Nothing
Bypass farther West
Optimize Existing Channel & Deposition Basin Configuration
Eastern Realignment of Channel and Deposition Basin
Western Realignment of Channel and Deposition Basin
Dredging the Ebb Shoal
Dredging the Flood Shoal
Non-Floating Dredging & Bypassing Systems
Reconfiguration of the Sore Thumb and Channel Realignment
Flow Training Structure(s)
Sand Trapping and Bypassing System Updrift
Jetty Opening and Nearshore Breakwater
Weir Jetty and Sediment Trap
Offshore Breakwater
Limit Navigation
Backpassing
Realignment of Complete Inlet System
Removal of the East Jetty
Move the Inlet to the Old Inlet Area
Closing the Inlet to Navigation

3.3 Fatal Flaw Analysis

A fatal flaw analysis was conducted to eliminate any alternatives that are clearly inadvisable or include negative effects that cannot be offset by any degree of benefits from other factors. This analysis also reduces the number of alternatives that were subject to detailed screening. Fatal flaws include:

- Not meeting all of the stated needs
- Exacerbating shoreline erosion
- Increasing barrier island breaching potential
- Significant uncertainty at a high cost
- Jeopardizing endangered species
- Significant inconsistency with applicable laws and regulations
- A similar, more effective option, is available

The following paragraphs identify alternatives that were dropped out from the concept list presented in Section 3.2 and provide a rationale for doing so.

Existing Practice

Although existing inlet management practices do not appear to fully address both navigation and bypassing needs, it may be that all other alternatives are less desirable overall than existing practices. This may be the case, for example, at Fire Island Inlet, where although improvements are required, the associated costs and impacts may be too high. On the other hand, although navigation conditions are adequate at Shinnecock Inlet, bypassing, at least on a relatively frequent basis, may be a problem, and certainly the beach immediately west of the inlet suffers chronic erosion. Navigation reliability at Moriches inlet also needs to be improved. Therefore, continuation of existing inlet management practices is only considered an alternative worth additional consideration at Fire Island Inlet.

Do Nothing

To do nothing, although always an option, would not be consistent with the stated needs, particularly navigation. Specifically, if nothing (no channel maintenance dredging in particular) was done at the three inlets, it is very likely that within a few months to a couple of years, depending on storm activity, all three inlets would be closed to navigation due to hazardous conditions. Existing conditions at Moriches Inlet provide good insight into the impacts to navigation that could be expected under a do nothing scenario. Therefore, it is recommended that this alternative be eliminated from further consideration.

Ebb Shoal Nourishment to Force the Inlet to Reach Maturity

This alternative is based on the concept of forcing the ebb shoal to reach "maturity". In other words, to place sand on the ebb shoal in hopes of reaching some estimated equilibrium value. The hypothetical benefit associated with this option is that a "mature" ebb shoal would no longer capture sediment that would otherwise be available for natural and/or mechanical bypassing.

Ebb shoal capacity is typically estimated using of an empirical relationship developed by Walton and Adams (1976). Table 3-5 summarizes existing and estimated equilibrium ebb shoal volumes using this relationship. The table also presets an estimate of High and Low equilibrium volume estimates based on 5% and 95% percentile error ranges obtained from a cursory analysis of the original data set used by Walton and Adams. The original dataset and equation fit are also shown in Figure 3-1 in log-log and "normal" scales.

Table 3-5: Existing and Theoretical Equilibrium Ebb Shoal Volumes ($m^3 \times 10^6$)				
Inlet	**Existing Volume**	**Equilibrium Volume**	**Uncertainty**	
			Low Eq. Vol.	**High Eq. Vol.**
Shinnecock	6.7	11.5	3.8	46
Moriches	3.1	6.9	2.3	19
Fire Island	31.3	37.4	10.7	151

The values presented in Table 3-5 and the lack of strong correlation shown in Figure 3-1 suggest that even though all three inlet ebb shoal volumes seem to be smaller than the theoretical value, uncertainty with regards to the final equilibrium volume is too large. In other words, even if 5 million m^3 of sand were placed at the Shinnecock Inlet ebb shoal, there is a significant probability that the ebb shoal would either continue to grow and trap sediment, albeit at perhaps a smaller rate, or that the resulting total ebb shoal volume would in fact be larger than the real "equilibrium" value. Moreover, recent surveys suggests that the ebb shoal at Shinnecock Inlet may in fact be growing at rates significantly lower than previously reported, or not growing at all (see Section 5.1).

Ebb shoal nourishment seems even less suitable at Moriches and Fire Island Inlet, where existing volumes appear to be closer to the estimated "equilibrium" values.

Finally, it should also be noted that ebb shoal nourishment would involve using relatively large quantities (up to 5 million m^3 at Shinnecock Inlet) from sand resources that would otherwise be available for conventional beach fill. Therefore, ebb shoal renourishment is not recommended for further consideration as a bypassing alternative at any of the three inlets.

<u>"Non-floating" Dredging and Bypassing Systems</u>

Williams et al (1998) evaluated several "floating" and "non-floating" dredging and sand bypassing systems at Shinnecock Inlet in a study contracted by the US Army Corps of Engineers, New York District. Alternatives considered in that study included:

Floating plant (dredge):

- Trailing suction hopper dredge (the study notes that other floating plants may be feasible)

Non-floating plants:

- Semi-fixed Plant (crawler crane and jet pump)
- Land-based Mobile Plant (*Crawldog*)
- Submerged Mobile Plant (*Puninase*)
- Mechanical Fillet Mining (scraper/dragline or bucket system)

Williams et al compared the alternatives on the basis of experience, navigation impacts, environmental impacts, aesthetics, sand source flexibility, placement flexibility, bypassing continuity, mechanical reliability, and ownership/ operational responsibility. The study concluded that a floating plant (hopper dredge), on either a 2- or 3- year cycle, would be the most desirable bypassing system for Shinnecock Inlet. Alternatively, if a more continuous bypassing system is preferred, Williams et al concluded that a semi-fixed system would be most desirable. It is important to note that deciding factors in that study were flexibility and experience with the system in United States and not only cost. In fact, a floating plant is more costly on an annual basis than a semi-fixed plant, according to the study.

Other systems ranked lower mainly because of lack of experience in the United States and the relatively large costs. This is particularly true of the *Crawldog* system, which has not been used in a similar project yet, and is relatively expensive. The submerged mobile plant system (*Puninase*), although in concept a very flexible system and with some experience in Europe, may still have some of the typical drawbacks of submerged pump systems, namely: (1) unknown performance in actual bypassing operations, (2) difficult deployment during storms, (3) relatively complicated system, and (4) pump seals susceptible to damage (USACE, 1991).

One additional advantage associated with floating plants, which Williams et al did not directly account for, is that the source of material for bypassing is generally along the navigation channel (deposition basins in Moriches and Shinnecock inlets) or immediately adjacent to it (Fire Island Inlet). Therefore, plant operation (dredging) assists both in bypassing and channel maintenance. On the other hand, a semi-fixed plant or any other alternative that does not directly dredge along the channel is likely to also require periodic channel maintenance with floating plant, albeit on a longer cycle and at reduced cost as compared to the Existing Practice.

Nonetheless, based on the results of the study by Williams et al, only the floating plant, and semi-fixed plant options will be further considered in the screening process. Other non-floating plants will be eliminated. In addition, a truck or trailer mounted jet pump system will also be considered at Shinnecock and Moriches inlets, an alternative not considered by Williams et al. Although this type of system is typically a feasible choice only for projects with small transfer rates and relatively short discharge distances (USACE, 1991) it offers the advantage of being able to use it at more than one location (e.g., Moriches and Shinnecock Inlet), and it might be effective in combination with other measures (e.g., channel maintenance with floating plant plus bypassing with truck system).

Backpassing

Backpassing at the inlets would entail transferring sediment from somewhere in the inlet system (e.g., ebb shoal, flood shoal, deposition basin, updrift fillet, etc.) to updrift beaches. Backpassing has in fact been a common practice historically at Fire Island Inlet where in recent years approximately 20% of the material dredged within the channel and deposition basin has been placed along Robert Moses State Park east of the inlet. Backpassing has also taken place on number of dredging events at Moriches Inlet to alleviate chronic erosion of Cupsogue Beach to the east associated with the Westhampton groin field. Clearly, however, backpassing cannot be the “only” selected sand management alternative at these two inlets, given that the net longshore sediment transport direction is from east to west. Nevertheless, backpassing may continue to be part of the optimal plan at both inlets, and in particular at Fire Island Inlet, where shoreline erosion appears to be a larger problem East than West of the Inlet.

Backpassing has not been a common practice at Shinnecock Inlet, mainly because beaches east of the Inlet are relatively stable and existing beach conditions provide for sufficient storm protection. Moreover, chronic erosion has typically required that most of the available sand be placed along the beaches west of the inlet. Therefore, backpassing is eliminated as a stand-alone alternative, although it may be implemented as a component of an overall bypassing plan at Fire Island Inlet.

Realignment of Complete Inlet System at its Existing Location

The basis for this alternative is that a realigned inlet system may result in improved navigation and perhaps even more efficient bypassing. Note that this discussion assumes that the realignment would include not only the jetties but also the inlet throat and associated revetments and possibly the seaward end of the inner bay channels (i.e., the “complete” inlet system).

At Shinnecock, the inlet throat is approximately 800 feet wide and 2,250 feet long from the inner bay channel bend to the seaward tip of the east jetty (Figure 2-1). The jetties and navigation channel are aligned slightly west of a north-south alignment. The channel alignment tends to migrate to a more northeast to southwest alignment between dredging events. The shift is due to the effects of the net westerly longshore sediment transport, the offset in the existing jetties (the west jetty is approximately 500 feet shorter than the east jetty), and coastal tidal currents during ebb flow conditions. Moriches is approximately 800 feet wide at the throat and approximately 2,000 feet long from the bayside bend to the seaward tip of the east jetty (Figure 2-3). The authorized channel alignment bends at the seaward end of the west jetty; changing from a northeast to southwest alignment along the inlet throat, to a north-south alignment seaward of the jetties. Similarly to Shinnecock Inlet, however, the seaward alignment tends to shift naturally to a northeast to southwest alignment. In recent surveys, this tendency is more evident at Moriches than at Shinnecock because of reduced deposition basin dimensions and less frequent dredging.

Theoretically, complete inlet realignment at Shinnecock and Moriches inlets could involve reconfiguring the inlet throat, jetties and channels to a more northeast to southwest orientation. As stated above, a realigned inlet may result in more efficient bypassing and improved navigation. One additional benefit may be reduced scour at the seaward tip of the west jetty. This alternative, however, is likely to incur significant construction costs with limited and very uncertain benefits in return, particularly with regards to sand bypassing. For example, a realigned inlet may be more hydraulically efficient, thus increasing the potential for ebb shoal sand accumulation and requiring an alternative means for providing bypassing. Another possible unintended consequence, which would be difficult to predict and account for in the design of the project, may be exacerbated west beach erosion. More importantly, similar benefits as those presumably provided by this alternative may be obtained through realignment of only the seaward end of the jetties and/or the navigation channel.

Fire Island Inlet is very different than Shinnecock or Moriches inlets. The inlet throat is significantly longer as result of unconstrained natural migration for years. The existing channel follows an east to west alignment along the inlet throat and then wraps around the western Fire Inland, which ends in a large sand spit that has grown as a result of sediment being transported past the existing east jetty (Figure 2-5). The existing channel alignment follows the shortest path to deep water from the inlet throat. Complete realignment at Fire Island Inlet was eliminated as an alternative because it was considered that realignment would in essence require relocation. Therefore, a different inlet alignment and overall characteristics will be considered in the context of a relocation alternative.

Removal of the Jetties

Clearly, completely removing the jetties would not meet all of the stated needs. Navigation reliability would be significantly compromised because of the highly variable and unpredictable conditions associated with a "natural" inlet. In addition, the lack of jetties would allow for unconstrained alongshore inlet migration, with the associated impacts on existing roads, buildings and other infrastructure. A migrating inlet also has the potential of accumulating increased volumes of sand in the form of relict flood shoal deposits, this sand would lost from the active littoral system. For all these reasons, it is recommended that this alternative be eliminated from further consideration.

Closure of Moriches or Shinnecock Inlet

Closing one of the inlets would only meet one of the stated project needs, sand bypassing. Clearly, it does not meet the navigation needs. Additionally, closing either Moriches or Shinnecock would increase the potential for breaching at random locations along the barrier island fronting Moriches and Shinnecock Bay during future storm events. Finally, closing one of the inlets may significantly impact water quality in the corresponding bay and have unintended impacts on the existing environment. Therefore it is not recommended that this alternative be further considered.

Moving the Inlets to a New Location

Moving the inlet only appears to have some benefits at Fire Island Inlet (see *Complete Inlet System Realignment* above). At Moriches and Shinnecock Inlet, however, there are no apparent reasons to consider this option. Negative impacts, on the other hand, would be numerous and hard to predict and/or quantify. Therefore, it is recommended that only moving Fire Island Inlet be further investigated.

Closing Inlets to Navigation

Clearly this option would not fulfill the stated navigation needs. In addition, closing the inlet to navigation and discontinuing all channel dredging operations may improve bypassing and average shoreline conditions along the project shoreline, but it may not be sufficient to stabilize the beach immediately west of Shinnecock. Finally, closing the inlets to navigation and eliminating channel maintenance will make Fire Island Inlet even less hydraulically efficient, increasing the possibility of breaching and permanent inlet formation during severe storms at other locations along Fire Island. Therefore, it is also recommended that this alternative be eliminated at the three inlets.

Close and Reopen Fire Island Inlet at Old Inlet

Opening a new inlet at the historic lighthouse location would accomplish similar objectives (i.e., improved efficiency) with less risk and uncertainty regarding inlet stability and shoreline impacts. In addition, impacts on existing natural resources and existing infrastructure are likely to be much greater at the Old Inlet location. Therefore, it is recommended that only the option of opening a new inlet in the vicinity of the lighthouse be given further consideration.

Open one Additional Inlet to Great South Bay

Previous analyses (USACE-NAN, 1996) have shown that if one additional inlet were to open (either naturally or artificially) at a new location in Great South Bay, Fire Island Inlet would shoal very rapidly and tend to close. It would be very difficult to maintain both inlets through dredging, and therefore this alternative is also dropped.

Relocation of Shinnecock Inlet Marine Center to Smith Point

Relocation of the Shinnecock Inlet maritime facilities and fleet to Smith Point would have a significant economic impact on the fleet as well as significant impacts on the social and historic character of the existing area at Shinnecock Inlet. In addition, other relocation options to an area much closer to the existing facilities may be available. Finally, the relocation to Smith Point would require that the Intracoastal Waterway be reauthorized and maintained a greater depth for the existing fleet to access these facilities from either Moriches or Shinnecock Inlet. Therefore, this alternative is also dropped from further analysis.

Reduce Authorized Channel Depth at Shinnecock and Fire Island Inlets

At Shinnecock Inlet this alternative is intended to increase natural bypassing around the inlet by reducing the authorized channel depth (currently authorized at -10 feet MLW) and/or eliminating the authorized deposition basin. This plan would result in a situation very similar to that prior to 1990, when the federally authorized project was constructed. Specifically, before dredging of the new channel and deposition basin in 1990 the inlet was extremely dangerous to navigate because of the location and depth of the ebb shoal delta and bypassing bar. This bar was as shallow as 6 feet below MLW and aligned approximately northeast to southwest directly across the inlet mouth. This condition is very similar to the present situation at Moriches Inlet. Vessels had the option (as in Moriches today) of navigating along a meandering natural channel that placed vessels at a dangerous angle to incoming waves, or attempt to navigate over the shallow sand bar. Arguably, however, this ebb shoal and bypassing bar configuration allowed for increased natural bypassing, as westerly longshore sediment transport was carried along the bar, over the shallow ebb shoal formation on the west side of the inlet, and onto the west shoreline. It is unlikely that this alternative will significantly alleviate the chronic erosion problem along the shoreline immediately west of the inlet.

A similar alternative at Fire Island Inlet would also theoretically increase natural bypassing around the inlet by reducing the authorized channel depth (currently authorized at -10 feet MLW) and/or eliminating the authorized deposition basin. However, there is large fleet that uses Fire Island Inlet, and a reduction in the authorized channel depth would have a very significant negative effect on this fleet. More importantly, a shallower channel might not deter vessels from trying to navigate the inlet, which will result in increased life and property losses.

In addition, unchecked growth of the sand spit will have a significant impact on the already marginal hydraulic stability of the inlet, possibly accelerating shoaling at the inlet mouth and providing an opportunity for any breaches that might occur on Fire Island during an extreme storm to evolve into new inlets.

Extend Westhampton Groin Field

This alternative does not meet the stated needs of improved navigation reliability and bypassing at Moriches inlet. Arguably, it would provide for increased shoreline stability between the inlet and the western end of the existing groin field at Pikes Beach. Therefore, although this alternative could be considered as part of overall FIMP Reformulation Study, it will not be further considered as an inlet modification alternative.

3.4 Description of Alternatives Selected for Further Screening

The following paragraphs describe each alternative selected for further screening in detail. Note that the descriptions focus on the extent to which the alternatives meet the stated project needs of (1) navigation reliability, (2) stability of the West Beach, and (3) sediment bypassing (see Section 1.1). Additional discussion on each selected alternative with regards to other impacts and screening criteria is presented in Sections 8.2, 8.3 and 8.4.

In addition to the alternative plans described below, other components that have been proposed and could be added to any of these alternatives are:

- Surfing reefs
- Inlet tolls (inlet taxing districts)
- Optimized dredging

These additional components will be analyzed in more detail during final optimization of the selected alternatives.

Note that because of the similitude among the alternatives presented for Moriches and Fire Island Inlets and those for Shinnecock Inlet, only conceptual sketches for the latest are presented. Conceptual sketches for the selected alternatives for all three inlets are presented in Section 8.

3.4.1 Shinnecock Inlet

Existing Conditions and Design Considerations

A regional sediment budget prepared by Gravens et al. (1999), principally based on shoreline changes from 1979 to 1995, suggests a net westerly directed sediment transport entering the inlet from the east under *Existing* (circa 1999) conditions of approximately 130,000 m^3/yr (170,000 cy/yr). Analysis of recent surveys (see Section 5.1) suggest that this value might be slightly higher, 168,000 m^3/yr (220,000 cy/yr). Williams et al. (1998), based on a previous inlet sediment budget prepared by Moffatt & Nichol Engineers (USACE-NAN, DRAFT, 1998), suggested a target bypass rate of around 100,000 m^3/yr (131,000 cy/yr) with capabilities to transport up to 115,000-134,000 m^3/yr (150,000-175,000 cy/yr). The recommendation from Williams et al. seems slightly low as compared to recent findings. Nonetheless, it is considered an adequate estimate for screening purposes.

The design capacity of the existing deposition basin (Figure 2-1) is approximately 350,000 m^3, and the anticipated dredging interval is 1.5 years (USACE-NAN, 1988). These design characteristics were based on an anticipated gross sediment transport rate of approximately 300,000 m^3/yr. Since 1990, however, the deposition basin has only been dredged three times, in 1993, 1998, and 2004 and as of January 2006 the deposition basin appeared to have accumulated enough material to be dredged again. This equates to a dredging interval of just over 4 years, instead of the design interval of 1.5 years. Rapid shoaling between March 2004 and January 2006 may owe to less dredging in 2004[1], significant storms in the fall of 2003, and/or the fact that approximately 555,000 $m^3$of fill were placed immediately west of the inlet as part of the WOSI project in 2004/05.

In addition, these recent dredging events amount to approximately 80,000 m^3/yr since 1990, although recent conditions (since 1995) suggest a slightly lower rate of approximately 65,000 m^3/yr (see Figure 5-20). To date, most of the material excavated from the channel and deposition basin has been placed along the shoreline immediately west of the inlet (West Beach), between the west jetty and the Ponquogue ebb shoal reattachment point, a shoreline reach that suffers from chronic erosion. Only during the initial construction project in 1990, was sand placed farther west at Ponquogue beach (approximately 200,000 m^3). Surveys and numerical model results suggest that a large percentage of this material returns to the inlet instead being transported farther west. As such, Existing Practice, although successful in maintaining a minimum level of protection at West Beach, is not very efficient in terms of preventing large shoreline fluctuations or providing hydraulic bypassing farther downdrift, although recent surveys arguably suggest that this system results in relatively efficient natural bypassing of the net longshore sediment transport balance (see Section 5.1). In other words, the chronic erosion at West Beach is not necessarily related to a lack of overall bypassing but instead may be due to the geometric configuration of the inlet system and local wave and current conditions.

It is also evident that the Existing Practice has been very successful in providing for continuous reliable navigation (see Section 7.2) without a need for the emergency dredging projects which were typical of historic channel maintenance practices prior to project implementation in 1990. This success is due to the straight channel alignment and greater than required depths, both maintained because of the deposition basin.

Existing Practice dredging operations to date have taken place in the fall or spring. Arguably, spring would be the preferred time to bypass if using an intermittent system (i.e., floating plant) such as Existing Practice for two reasons: (1) the material would not be initially exposed to strong westerly transport typical of the winter months, and (2) the channel would be ready for the busy summer and fall boating seasons. On the other hand, environmental window constraints may prevent material placement from April 1 to August 31 (Piping Plover, Least Tern and Black Skimmer nesting season).

Semi-fixed and mobile (truck mounted) systems capable of continuous bypassing may be operated from September 1 to April 1. This schedule avoids both the nesting season and the recreational summer season and includes the months during which most of the westerly transport takes place.

[1] The dredge volume in 2004 was somewhat smaller than in 1993 and 1998 because 2/3 of the deposition basin was dredged to -19 ft. MLW vs. -20 ft.

With these conditions and constraints in mind, the following paragraphs present a detailed description of alternatives considered for further screening at Shinnecock Inlet.

1. *Existing Practice plus Offshore Dredging (for West Beach)*

During the most recent channel maintenance project, March 2004, material dredged from the deposition basin was placed west of the Ponquogue reattachment point. In addition, the West Beach was restored using sand from an offshore sand source instead of from the inlet as part of the *West of Shinnecock Inlet Interim Storm Damage Reduction Project* (USACE-NAN, 1999). This approach could be implemented as a long term inlet management alternative. In concept, it provides for improved bypassing since all of the material dredged from the channel and deposition basin would be placed west of the Ponquogue attachment point where it would be transported westward. In addition, it would maintain a minimum level of protection along the West Beach through periodic offshore dredging and renourishment. It would also continue to provide for reliable navigation conditions.

The influx of offshore sediments into the inlet system would presumably increase shoaling rates within the deposition basin, perhaps increasing the frequency of required maintenance. This would also represent an “improvement” in terms bypassing, as more frequent dredging would better mimic a continuous natural bypassing system. However, this alternative, unlike any other investigated in this study, meets the stated project needs by using an source of sediment arguably exterior to the natural littoral system (i.e., the offshore borrow area), not by directly improving the bypassing efficiency of the inlet system or by reducing erosion along West Beach. As such, this alternative might not be sustainable in the long term.

2. *Existing Practice plus Dredging the Flood Shoal*

Alternatively, the flood shoal could be used as a source of sediment for periodic renourishment of the West Beach (Figure 3-2). There is already a precedent for this type action at Shinnecock Inlet; in 1997, NY State dredged approximately 190,000 m^3 from the East Cut and placed it on the West Beach. In general, material could also be removed from a relatively large area of compatible material along the south edge of the flood shoal. The amount compatible material within this area is between 700,000 and 1,450,000 m^3, depending on the depth and area of the cut (OCTI, 1999; Militello and Kraus, 2001). Bypassing and channel reliability conditions would be similar to Existing Practice (see above), although depending on the amount dredged from the flood shoal, some limited impacts to navigation might occur due to slight increases in current velocities (Militello and Kraus, 2001). More importantly, the volumes dredged from the flood shoal would be restored over time, possibly at the expense of natural and maintenance related inlet bypassing volumes. Dredging the flood shoal would reduce encroachment of this morphological feature into the East and West Cut, thereby improving navigation reliability and reducing maintenance requirements along those two inner channels.

3. *Channel & Deposition Basin Realignment along the “Natural” Channel Thalweg*

This alternative continues the Existing Practice of channel and deposition basin maintenance but on an alignment closer to the “natural” channel thalweg alignment (northeast to southwest) (see Figure 3-3). This alignment corresponds to the location of the ebb jet, which in turn is dictated by ambient nearshore tidal currents, the geometry of the inlet and the jetties, and the net westerly longshore transport. It is unclear whether or not the modified alignment would reduce existing sediment losses (if any) to the ebb shoal during ebb tidal flow conditions by slightly reducing hydraulic efficiency. Hydrodynamic model simulations performed by Millitelo and Kraus (2001) show areas of current speed reduction and increase of very similar extent, with no apparent net impact on ebb shoal growth. If hydraulic efficiency and current velocities are reduced, the deposition basin could accrete at faster rate which would increase maintenance dredging and bypassing frequency. Additional sediment transport and morphological modeling would be required to further explore this alternative.

A northeast to southwest alignment may also possibly reduce storm impacts on the West Beach by allowing the updrift ebb shoal lobe to extend farther west thus providing some of the protection that was lost after construction of a straight north to south channel and deposition basin.

Navigation would likely continue to be reliable if the deposition basin dimensions do not change, although some minor impacts should be expected given that vessels would be slightly more exposed to side waves.

4. *Relocation of the Deposition Basin (Not Channel)*

This alternative continues the existing practice of channel and deposition basin maintenance but the deposition basin is relocated. One possibility would be to place the deposition basin adjacent to the navigation channel on the east (updrift) side of the channel (Figure 3-4). The intent of this alternative would be to reduce the hydraulic efficiency of the inlet thus reducing the amount of sediment currently lost to the deeper reaches of the ebb shoal (if any) and more importantly, to increase sediment interception within the deposition basin. As with the previous alternative, this may also increase dredging frequency and net hydraulic bypassing. Although a deposition basin on the east side of the channel would prevent sediments from encroaching on the channel from the east, easterly transport around the west jetty during transport reversals, albeit smaller, may result in significant channel shoaling possibly requiring emergency dredging to allow for continued navigation. Therefore, a variant of this alternative might be to also construct a smaller deposition basin (e.g., 300 feet wide at -12 feet MLW instead 800 feet at -20 feet MLW) over the existing channel alignment.

5. *Reduced Dimensions of Deposition Basin*

This alternative continues existing practice but reduces the dimensions of the deposition basin (Figure 3-5). As described above, the design capacity of the existing basin is approximately 350,000 m^3, and the anticipated dredging interval was 1.5 years. These design characteristics were based on a gross sediment transport rate of approximately 300,000 m^3/yr (USACE-NAN, 1988). Since 1990, however, the deposition basin has only been dredged three times, in 1993, 1998, and 2004. This equates to dredging interval of just over 4 years, instead of the design interval of 1.5 years. A smaller deposition basin would possibly reduce the dredging interval and perhaps even increase natural bypassing, since tidal velocities along the channel thalweg would likely be smaller. More frequent bypassing, albeit at a similar annualized rate, may lead to a more stable shoreline (i.e., less shoreline fluctuation) downdrift. It is uncertain, however, whether a smaller basin would provide enough material to stabilize the West Beach and meet the overall bypassing requirements.

6. *Existing Practice plus Dredging the Ebb Shoal (outside limits of Deposition Basin) with a Floating Plant*

One alternative to dredging within the deposition basin is to dredge the required amount of sediment to meet the design bypass rate from other areas of the ebb shoal such as the updrift lobe or around the seaward slope of the shoal (Figure 3-6). This action may be easily implemented using a traditional floating plant on contract. In addition, dredging from these relatively large areas will provide a guaranteed source of sediment that conforms to a design bypass schedule and quantity, as opposed to dredging from a deposition basin, which, as already experienced, does not accumulate sediment at the same rate every year. A more frequent bypassing operation will reduce shoreline fluctuations along West Beach. In addition, sufficient material may be available in these areas of the ebb shoal to provide a source of sediment to both West Beach and areas farther downdrift. On the other hand this alternative may have some adverse effects on ebb shoal morphology, natural bypassing, and adjacent shorelines if the amount of material dredged from the ebb shoal is too large.

Ebb shoal dredging might also have to be supplemented with Existing Practice or a modified version of it (e.g., reduced deposition basin dimensions) to provide adequate navigation reliability. The dredging contract could be structured to include ebb shoal dredging on a regular cycle (e.g., every 2 years), while the channel and deposition basin would be dredged on as needed but coinciding with dredging of the ebb shoal.

7. *Semi-fixed Bypass System*

Williams et al (1998) considered a semi-fixed bypass plant for Shinnecock Inlet based on the Indian River Inlet system. Specifically, the system includes the use of crawler crane to position a jet pump (eductor) near MLW on the updrift fillet (Figure 3-7). The pump is supplied with clear water from the inlet and it discharges slurry that is piped across the inlet and discharged along West Beach with the help of two booster pumps. Alternatively, a boom on a trestle type of operation could be used to excavate the updrift fillet. The crane-based system at Indian River Inlet has been fairly successful, bypassing 76,500 m^3/yr (100,000 cy/yr) since its commission in 1990. The eductor pump in this system has a rated capacity of 200 cy/hr, and it typically operates from Labor Day to Memorial Day, although nesting piping plovers may impact operation between April 1 and August 31.

Increased bypassing rates can be achieved with additional pumps mounted on a trestle. For example, the Nerang system in Australia, which includes jet 10 pumps, has bypassed approximately 500,000 m^3/yr (650,000 cy/yr) since 1986. A similar system went into operation at Tweeds River, Australia in March 2001. This system has a design capacity of approximately 400,000 m^3/yr (524,000 cy/yr) using 11 jet pumps (only 5 working at the same time) mounted along a shore perpendicular trestle. However, since operations started in March 2001, the system has bypassed 800,000 m^3/yr (1,005,000 cy/yr), a significantly larger rate.

At Shinnecock Inlet, a crane mounted pump would provide for greater flexibility in reaching different areas of the updrift fillet. In addition, the crane and jet pump can be moved and protected during storms. System components could be serviced and repaired as needed off-site. Nonetheless, the system might have to be supplemented with periodic channel (and deposition basin) dredging in order to meet the required bypass rate and to provide for adequate navigation conditions. Note that increasing the capacity of the system with additional pumps or a bigger one would not necessarily eliminate the need for channel maintenance. Longshore sediment transport reversals and westerly transport beyond the reach of the semi-fixed system will continue to shoal the channel, albeit at a smaller rate. For example, at Tweeds River, even though the system has bypassed double the design rate since construction, dredging along the navigation channel is still required; 240,000 m^3/yr (314,000 cy/yr) were dredged between June 2002 to June 2003.

Material dredged with this system could be bypassed to the West Beach versus farther west (thus reducing power needs), while material dredged with a conventional floating plant from the channel and deposition basin could be placed west of the ebb shoal attachment area. Similarly to Indian River Inlet, a flexible pipeline could be shortened or extended to provide discharge at any point along the West Beach (approximately 3,000 feet).

8. *Truck/Trailer Mounted System*

In concept, a truck/trailer mounted jet eductor pump may be able to provide some of the benefits of semi-fixed and mobile systems without some of the disadvantages. Specifically, this type of system could be mobilized quickly and inexpensively, the system would be easily transported so that it could be used at more than one location (e.g., Shinnecock and Moriches inlets), it would not require permanent facilities, and unlike floating plants it could be used continuously to better mimic natural bypassing processes. Similarly to the semi-fixed crawler crane system, the truck mounted pump is very flexible in terms of reaching available sediment sources. Finally, the system could be quickly demobilized during extreme storms. Road access to the source (i.e., updrift fillet), a key requirement of this type of system, is also available at Shinnecock Inlet through Dune Road and Shinnecock East (Suffolk) County Park.

On the other hand, this type system is only feasible for projects that require relatively small bypassing rates and discharge distances. Effective production rates for reasonably sized system are on the order of 60 to 100 cy/hr (Roberge, 2003), which is less than half of the rated capacity of the Indian River System (approximately 200 cy/hr). Therefore, this system might be more appropriate in combination with other actions, such as Existing Practice. Similarly to the crawler crane system, booster pumps would also be required to achieve the required discharge distances.

9. *Existing Practice plus Spur Jetty (West)*

An alternative to continued renourishment of the West Beach is to provide for increased shoreline stability by means of structural modification to the west jetty. Field observations, surveys and numerical model results suggest that a significant amount of the material placed along the West Beach may be transported back into the channel and deposition basin, particularly when waves originate for the south and southwest sectors. Moreover, it does not appear that the West Beach receives a significant supply of sediment from the west during these periods of potential easterly transport. It has also been argued (Baird & Associates, 1999) that this chronic shoreline instability is directly related to Existing Practice in that the relatively deep deposition basin allows SE and SSE waves to propagate over this opening in the ebb shoal unimpeded and focus on the West Beach. The result is that this shoreline is not in equilibrium with the waves and thus the ongoing erosion. One possible approach to alleviating this condition is to extend the existing west jetty with a “spur” that would induce a new stable shoreline that would provide for a wider beach in this area. The configuration that Baird proposed consists of a 135 m (440 feet) long spur extending from the seaward tip of the existing jetty structure (Figure 3-8). The structure is oriented 132 degrees counterclockwise from the west jetty.

According Baird’s study, the spur would result in an acceptable (i.e., sufficiently wide) stable beach planform for the design scenario they tested (large storm waves from the SE). In addition, the study concludes that the spur will also reduce the amount of sediment that is lost to the channel and deposition basin during periods of south waves. One added benefit also suggested in the study is the reduction in scour potential at the new tip of the jetty, which would be located in deeper water, and presumably subject to lower tidal current velocities. With regards to navigation, Baird concludes that the wave climate over the existing channel would be very similar with and without the spur, thus no impacts on navigation are expected. Note, however, that Baird’s report does not present a detailed analysis of changes in tidal velocities along the channel, which preliminary modeling results conducted as part of this study suggest would increase. More importantly, the effects of the spur on inlet morphology and bypassing were not investigated in detail either. Increased training of the ebb jet as a result of spur construction could possibly lead to seaward growth of the ebb shoal and/or reduced bypassing.

If this alternative is carried forward to additional detailed design, other alternative spur configurations (e.g., a spur perpendicular to the existing west jetty), or simply extending the west jetty, may also be investigated with the idea of reducing the effects on ebb tidal velocities and ebb shoal growth.

10. *Existing Practice plus Shortening the East Jetty*

This alternative would continue the Existing Practice (channel and deposition basin maintenance) and it would also include shortening of the east jetty (Figure 3-9). This structure extends approximately 1,500 feet in an approximate North to South alignment that connects with a bay-side revetment that runs along the North side of the barrier island for an additional 1,500 feet, approximately. The seaward most 200 to 500 feet (depending on the position of the eastern shoreline) is typically exposed on both sides. The concept behind this alternative is that by shortening the structure, additional sediment would travel into the inlet and deposition basin during periods of westerly transport. Therefore, this plan may result in improved bypassing, as this additional volume of sediment would either by bypassed naturally, or collected in the deposition basin and available for dredging and bypassing. Therefore, it is likely that this alternative would increase the maintenance dredging requirements, perhaps to a level closer to the design volume of approximately 230,000 m^3/yr (USACE-NAN, 1988), and reduce the interval between dredging projects. Ideally, shortening the jetty would not affect navigation reliability, although the issue would need to be carefully investigated before implementing such an alternative.

11. *Change Distance between Inlet Jetties*

This alternative is intended to improve natural bypassing and sedimentation within the deposition basin by modifying the existing inlet geometry and specifically the distance between the two jetties. Hypothetically, the distance between the two structures (and perhaps the length and orientation) could be optimized to reduce or reverse ebb shoal growth and increase natural bypassing. The modified jetties could also increase sedimentation within the deposition basin, therefore allowing for more frequent hydraulic bypassing. This alternative, however, may have significant unintended adverse effects on existing sediment pathways and inlet morphology. Attendant impacts on navigation would also be very hard to predict.

12. *Existing Practice plus Nearshore Structures along West Beach*

An alternative West Beach stabilization approach, also considered in the *West of Shinnecock Inlet Interim Storm Damage Reduction Project* (USACE-NAN, 1999), is to stabilize the West Beach with a combination of beach restoration and a series of nearshore structures such as shore-parallel breakwaters or T-Groins (Figure 3-10). The effect of these structures would be to directly protect the shoreline from wave impacts and to induce the accumulation of sediment behind the structures. In addition, the T-Groins would prevent westerly transport of sediments into the channel and deposition basin. The Interim study concluded that although these structures would provide the necessary reduction in storm damages and long-term erosion, construction would not be reversible and therefore it was not further considered for interim protection. The report, however, recommended the alternative be considered further in the Reformulation Study.

This type of alternative, if constructed sufficiently nearshore between the west jetty and the Ponquogue ebb shoal attachment point, would have no impacts on navigation and possibly only a minimal impact on natural bypassing conditions. Maintenance dredging frequency may also be reduced since sediment would no longer be transported from the West Beach and into the channel and deposition basin. On the other hand, this alternative will greatly reduce or even eliminate the need for further renourishment. Therefore, all the sediment dredged from the deposition basin would be available for placement farther downdrift.

13. *Sand Trapping and Bypassing System Updrift*

Existing management practice at Shinnecock Inlet relies on the interception and storage of incoming sediment within the deposition basin, which is periodically dredged and placed on the West Beach. For reasons possibly related to both wave climate conditions and increased hydraulic efficiency of the inlet, the deposition basin has not captured sediment at the rate it was originally designed for and expected. At this point it is uncertain whether or not a larger amount of sediment is actually being transported westerly from the east shoreline and around the tip of the east jetty and whether this additional sediment, which is not being captured in the deposition basin, is being lost to the ebb shoal (i.e., the continued growth hypothesis) or is being naturally bypassed to the west. Although recent bathymetric surveys do not suggest continued ebb shoal growth, and thus the latter hypothesis appears more accurate (see Section 5).

An alternative to Existing Practice of interception and storage within the deposition basin is to intercept, collect, and bypass sediment from the subaerial and subaqueous littoral material accumulated at the updrift fillet adjacent to the east jetty. This could be accomplished by means of a conventional cutter-suction pipeline dredge, although protection for the dredge, which would have to work in the highly turbulent nearshore zone, would need to be provided.

Dredging of the fillet and protection could be accomplished in one of three ways:

A. *Jetty Opening and Nearshore Breakwater:* A small opening or "door" approximately 150 feet wide would be built through the east jetty (Figure 3-11). The opening would allow a small pipeline dredge to gain access to remove a portion of the subaerial accretion fillet which accumulates next to the jetty. The opening would be protected against scour by steel sheet pile cutoff walls. In addition, ocean side protection for the opening would be afforded by a rubblemound breakwater section attached to the jetty truck which would curve and open toward the existing beach. The opening between the breakwater and the jetty trunk would be sealed with steel sheet piles. During bypassing operations this steel sheet pile section would be removed to allow the dredge to excavate a channel to the fillet. The dredge would then excavate an area contained between the jetty structures and a protective nearshore breakwater. This breakwater would serve a double purpose, since it would also induce increased accumulation of sediment in the lee of the structure which would be available for dredging and bypassing. Material would be pumped through a fixed pipeline across the inlet and along the west beach past the Ponquogue reattachment point, a booster pump station on the west side of the inlet would also be required. After removal of a predetermined amount of material, the dredge would retreat to the jetty opening. The dredge would fill in a portion of the access channel in the vicinity of the jetty door so that the steel sheet pile cutoff wall could be replaced. A relatively small dredge could be locally owned and operated. However, this system does not completely eliminate the need for continued channel dredging, perhaps with a different, large oceangoing dredge. In fact, a deposition basin, albeit probably smaller, would still be required in order to maintain adequate navigation reliability between maintenance dredging projects.

   Note that to remove approximately 230,000 m^3/yr (300,000 cy/yr), which is the design capacity of the existing deposition basin, and assuming the fillet is dredged once a year, the area available for dredging would have to be approximately 12 acres and it would have to be excavated to an approximate depth of -10 feet NGVD. On the other hand, to remove 134,000 m^3/yr (175,000 cy/yr), which is the upper range of design rates considered by Williams et al., 7 acres (a 500 x 500-foot square area) would be sufficient.

B. *Weir Jetty and Sediment Trap*: A weir jetty structure and sediment trap could also be built at the east jetty and adjacent fillet area instead (Figure 3-12). Weir jetties are in operation at several inlets on the East coast (e.g., Rudee Inlet, VA and Hillsboro Inlet, FL), with varying degrees of success. The design of these systems, however, is difficult and full of uncertainty, as the amount of sediment flowing over the weir and into the sediment trap is very sensitive to the dimensions of the weir. Nonetheless, this type of bypassing alternative allows for a very flexible operation. As in the case of the jetty opening approach, a relatively small dredge could be locally owned and operated. Also, as with the jetty opening alternative, channel and deposition basin dredging would still be required, perhaps with a separate, large oceangoing dredge. This has historically been the case at Rudee Inlet, VA.

C. *Offshore Breakwater:* Another approach to excavating the updrift fillet with a floating plant would be to dredge it from the nearshore with a large oceangoing dredge (Figure 3-13). An offshore breakwater would be required to provide the necessary protection to this dredge during operations. This breakwater would be significantly larger and more expensive to build than the nearshore structure for the jetty opening scheme described above because the structure would be sitting in deeper water. This would allow, however, a relatively deep draft oceangoing dredge to navigate into the lee of the breakwater and dredge from there. The breakwater also needs to be sufficiently far offshore so as to prevent excessive shoaling in its lee, which would prevent the dredge from sailing behind it. The advantage of this alternative is that the same plant could be used to dredge the updrift fillet and the channel/deposition basin.

14. *Existing Practice plus Dredging the Ponquogue Ebb Shoal Attachment*

As stated above, it does not appear that the West Beach receives a significant supply of sediment from the west during periods of S and SW waves and related easterly transport. This lack of sediment supply from the east might be associated with the local morphology, and more specifically the area where the ebb shoal bypassing bar attaches to the west shoreline at Ponquogue. This attachment feature may act as a "groin" during periods of easterly transport. Therefore, one of the alternatives being considered is to remove or "shave-off" this attachment feature (Figure 3-14). The removed sediment could be used to restore the West Beach and also placed west of the attachment point. The feature would rebuild in time as sediment would tend to accumulate in this area again. The effects of this scheme on overall ebb shoal morphology and more importantly natural bypassing are very uncertain, even if extensive modeling is performed. For example, removing this feature might increase the amount of sediment that is transported west and out of the West Beach during SE and SSE storm conditions. On the other hand, the plan would be completely reversible, so if monitoring suggests it is not working or it is making the conditions worse, a different alternative could be implemented. Navigation conditions would not be affected. However, shoaling within the deposition basin may increase if additional sediment is available for transport from the West Beach around the west jetty and into the inlet, which may require more frequent dredging.

15. *Existing Practice plus Relocation of the Maritime Center within Shinnecock Bay.*

This alternative would continue the existing inlet management practice as described above (channel and deposition basin maintenance) and it would also include relocating the maritime center located immediately west of the inlet to a new location within Shinnecock Bay. Therefore, this alternative would maintain the status quo with regards to navigation reliability, bypassing, and erosion of the west shoreline immediately adjacent to the inlet. On the other hand, it would reduce the risks of damages and loss of life during extreme storm events, since the new facilities would presumably be relocated to a less exposed area of Shinnecock Bay. It should be noted, however, that facility relocation will most likely require increasing the authorized channel depth for the inner Shinnecock Bay Channel (existing authorized depth is 6 feet) in order to provide access to the relocated facilities to the deeper draft vessels that currently use it (approximately 10 feet draft). It is also unlikely that relocation alone would completely eliminate the need to renourish and protect the West Beach. The area would still be subject to chronic erosion and if the existing cross-section is reduced to the point where breaching occurs during an extreme event, damages to inlet structures and increased flooding are likely to be significant.

3.4.2 Moriches Inlet

Existing Conditions and Design Considerations

Previous sediment budget estimates (Gravens et al., 1999), principally based on shoreline changes from 1979 to 1995, suggest a long-shore sediment transport rate of approximately 184,000 m^3/yr (240,000 cy/yr) entering the inlet from the east under *Existing* (circa 1999) conditions. Analysis of recent surveys (see Section 5.2) suggest that this value might be significantly higher from 1995 to 2001, 238,000 m^3/yr

(312,000 cy/yr). This increase is at least partly due to recent fill placement east of the inlet as part of the Westhampton Interim Project (666,000 m^3/yr or 873,000 cy/yr between 1995 and 2001). The volume of sediment placed in future years along the Westhampton shoreline should be reduced to maintenance levels (approximately 250,000 m^3/yr or 328,000 cy/yr), which would result in approximately 167,000 m^3/yr (219,000 cy/yr) of net westerly transport at the inlet (see Section 5.2).

Note that the recent transport rate is significantly higher than at Shinnecock Inlet. This may explain why, aside from the differences in inlet geometry and deposition basin dimensions, the channel and deposition basin at Moriches Inlet shoals noticeably faster than at Shinnecock Inlet.

Existing inlet management practice consists of infrequent (at least relative to the existing need) channel and deposition basin dredging (Figure 2-3). Recent (1996 and 1998) dredging events have yielded approximately 56,000 m^3/yr (73,000 cy/yr). Most of this material was placed within 2,000 feet of the west jetty, that is, still within the area fronted by the ebb shoal. Similarly to Shinnecock Inlet, it is unlikely that this material is subsequently transported by waves farther west past the ebb shoal attachment area. Nonetheless, and although a small amount of sand accumulates immediately updrift of the inlet, the inlet appears to effectively naturally bypass the balance of the net longshore transport along a pathway that follows a very shallow bypassing bar extending right across the navigation channel in a NE to SW alignment and a large, slightly deeper ebb shoal platform that connects this bar with the shoreline downdrift (see Section 5.2). Dredging events "cut" the NE to SW sand bar in two pieces, the western piece appears to be rapidly moved west and onto the downdrift shoreline, while the eastern bar remnant grows rapidly encroaching the channel again within a few months. Specifically, recent condition surveys (Appendix A) suggest that on July 2000, which was less than two years after the October 1998 dredging event, the leading edge of the shallow ebb shoal feature was encroaching into the channel. By April 2001, this feature had grown across the channel, and navigation conditions had been significantly deteriorated. Between 1996 and 1998, shoaling occurred more rapidly due to a more active wave climate during that period (see Section 5), channel depths were reduced from more than -20 feet MLW in certain areas (apparently the deposition basin was significantly overdredged in 1996) to less than -10 feet MLW in less than two years.

Efficient bypassing is also suggested by ebb shoal volumetric changes computed as part of this study (see Section 5) as well as a recent analysis by Allen et al. (2002) which hint that the ebb shoal is a relatively stable feature.

Navigation, however, is extremely dangerous through the inlet. The entrance channel and deposition basin were last dredged in 2004 (to -14 feet MLW) at the same time as Shinnecock Inlet. Unlike Shinnecock Inlet, however, the deposition basin and channel at Moriches Inlet have shoaled at a very rapid rate. This may be due to several factors, including the smaller size of the deposition basin and perhaps a greater influx of sediment into Moriches Inlet (see above).

1. *Authorized Project*

Authorized channel dimensions (200 feet wide at -10 feet MLW) are adequate for navigation under normal wave conditions at the inlet. In addition, the authorized deposition basin (300 feet wide at -14 MLW) provides for reliable navigation in all but the very highest wave conditions (see Section 7.3). Therefore, if dredging took place as needed to maintain the project, navigation through the inlet would be reliable most of the time.

The shoaling data presented above would suggest that the channel and deposition basin, as authorized, needs to be dredged more frequently than it has in recent years. In fact, the selected plan described in the Moriches Inlet General Design Memorandum (USACE-NAN, 1982) calls for a "seasonal" maintenance schedule which amounts to annual dredging (75,000 m^3/yr or 98,000 cy/yr) and the assumption that the channel and deposition basin will shoal to a depth of less than -10 MLW approximately 7 months after dredging operations. The GDM suggests that dredging take place in the spring, so that depths of less than -10 feet MLW would only occur during the winter months when traffic through the inlet is minimal. Bathymetric changes observed after the 1996 and 1998 dredging events seem to confirm the expected shoaling rates as described in the GDM.

Although this dredging schedule seems to meet the average dredging requirements, waves and longshore transport processes do not always follow the average, and it is anticipated that some years shoaling will occur at a rate higher than the average during spring and summer, which may result in significant impacts to navigation. On the other hand, a continuous shallow sand bar across the channel during the winter months would allow for improved natural bypassing. Other years transport will be relatively small and not enough material will be available within the deposition basin to justify the high mobilization costs of a dredge. Material dredged from the channel and deposition basin would be placed westward of the ebb shoal downdrift attachment area (i.e., at least 8,000 feet from the west jetty).

This alternative also includes rehabilitation of the outer end of the west jetty, as authorized. The 1982 GDM called for repair of the east and west jetties at Moriches Inlet. Subsequent construction, however, did not include this feature. Presently, the west jetty is in significant disrepair. Other features of this alternative, also included in the authorized plan, are maintenance of both jetties and the bayside revetment at Pikes Beach (constructed after the breach in January 1980) and maintenance dredging of the inner bay channel to a depth of -6 feet MLW and a width of 100 feet. All alternatives considered below also include these authorized features.

2. *Authorized Project plus Dredging the Flood Shoal.*

The flood shoal at Moriches Inlet could also be used as an additional source of sand to supplement bypassing from the authorized deposition basin. Recent surveys, however, suggest that presently Moriches Inlet effectively bypasses most of the westerly net long shore transport. Therefore, an additional source of sand might only be required after extreme events that cause localized erosion in areas adjacent to the inlet. If so, the flood shoal could be dredged in conjunction with the deposition basin and the inner channels, which also require periodic maintenance dredging. Periodic dredging of the southeastern edge of the shoal will prevent encroachment of this feature into the inner navigation channel, which is currently much shallower than the authorized -6 feet MLW depth along its authorized alignment. Note that this area of the flood shoal was dredged once as a source of sand for the closure of the breach that opened adjacent to the east jetty in January 1980. Approximately 460,000 m^3 (600,000 cy) were removed (Sorensen and Schmeltz, 1982). Since then the shoal has apparently recovered to its condition prior to dredging.

3. *Channel & Deposition Basin Realignment.*

Similar to Shinnecock Inlet, the unconstrained channel thalweg at Moriches Inlet follows a NE to SW alignment. This alternative includes channel and deposition basin maintenance as needed but on a more NE to SW alignment. As distinct from Shinnecock Inlet, where the purpose of the realignment would be to allow for improved natural bypassing by possibly reducing the amount of sediment lost to the ebb shoal, the realigned channel at Moriches Inlet may also reduce shoaling along the channel and attendant periods of unreliable navigation at the end of each dredging cycle. Aside from this, navigation conditions would be similar to those expected for the authorized plan, although some minor impacts should be expected given that vessels would be slightly more exposed to beam waves.

4. *Relocation of the Deposition Basin (Not Channel)*

Similar to the Authorized Project, this alternative would include periodic dredging but with a realigned/relocated deposition basin. One option would be to relocate the basin adjacent to the navigation channel on the east (updrift) side of the channel. The intent of this alternative would be to prevent sediments from encroaching on the channel from the east, thereby eliminating periods of unreliable navigation at the end of each dredging cycle. Under this scenario, however, the channel might be more susceptible to shoaling during periods of easterly transport.

5. *Increased Dimensions of Deposition Basin*

As described above, the authorized Moriches Inlet project calls for "seasonal" dredging once a year, which, if implemented, would result in unreliable navigation for approximately 5 months a year. In reality, however, the inlet has been dredged less frequently, which has had significant impacts on navigation reliability. An alternative would be to increase the dimensions of the deposition basin so as to increase the period between required dredging operations to a more practical schedule. For example, a deposition basin that allows for dredging every two years would be easier to maintain, particularly if modifications at Shinnecock Inlet reduce the dredging cycle at that inlet from the existing 4 years (or more) to 2 years. This would allow for multiyear contracts combining both inlets, which would significantly reduce the cost of both projects. In fact, this is how the 1998 and 2004 Moriches and Shinnecock Inlets maintenance dredging contract were bid and contracted.

Note, however, that a larger deposition basin may have some negative effects on existing natural bypassing conditions, although these effects could possibly be offset by the expected increase in maintenance dredging and bypassing.

6. *Dredging the Ebb Shoal (outside limits of Deposition Basin) with a Floating Plant*

Similarly to Shinnecock Inlet, an alternative to dredging the updrift fillet is to dredge the required bypass volume from other areas of the ebb shoal such as the updrift lobe or around the seaward slope of the shoal. This action may be easily implemented using a traditional floating plant on contract in combination with regular maintenance of the authorized channel and deposition basin. Note, however, that there appears to be less need to supplement existing bypassing at Moriches Inlet than at Shinnecock Inlet. Accordingly, this action may only be warranted if monitoring surveys subsequent to regular maintenance of the authorized project indicate that the ebb shoal is growing. In that case, a floating dredge could "shave-off" the seaward slope of the shoal in conjunction with a regular maintenance dredging operation.

7. *Semi-fixed Bypass System*

A semi-fixed bypass system similar to that proposed at Shinnecock Inlet and consisting of a jet eductor pump mounted on crawler crane could also be implemented at Moriches Inlet. System elements and operation would be very similar (crane, jet eductor pump, booster pumps, and discharge pipeline). There are, however, some key differences that might make this system less attractive at Moriches Inlet.

As explained above, the net westerly transport at Moriches Inlet in recent years (231,000 m^3/yr or 303,000 cy/yr) has increased significantly from previously reported rates (184,000 m^3/yr or 240,000 cy/yr) although this rate is expected to decline to approximately 167,000 m^3/yr (219,000 cy/yr). In any case, any of these amounts is significantly higher than the design capacity for the system in operation at Indian River Inlet in Delaware (at least 50% higher). In fact, Watson et al (1993) suggest that this type of system is only suitable for sites where the maximum bypass rate is less than 150,000 m3/yr (197,000 cy/yr). Nonetheless, the authors also indicate that bypassing efficiency could be improved by capturing more of the net drift; the system apparently operates only 40% of available days owing to limitations of the amount of littoral material transported and trapped within reach of the crane boom (Watson et al, 1993). Alternatively, the system could be operated in combination with channel and deposition basin maintenance operations.

8. *Truck/Trailer Mounted System*

The truck/trailer mounted bypassing system described above for Shinnecock Inlet could be shared with Moriches Inlet. However, road access to the updrift fillet may be more difficult at Moriches Inlet since the last one mile of road is not paved. Given the relatively large design bypass rate that would be required at Moriches Inlet, the system would have to be supplemented by maintenance of the authorized channel and deposition basin with a floating plant. Note also that the limited capacity of this system would have to be shared between Shinnecock and Moriches Inlet unless two separate systems were used. On the other hand, the system might be used to offset the relatively small capacity of the authorized deposition basin, thus allowing for an annual channel dredging cycle with limited impacts to navigation reliability between dredging operations.

9. *Authorized Project plus Extension the West Jetty*

Extending the west jetty would arguably improve navigation conditions at Moriches Inlet by forcing the ebb tidal jet and channel thalweg into a more north to south alignment that would parallel the existing authorized channel and deposition basin. Pushing the tip of the west jetty into deeper water may also reduce scour. Finally, the jetty extension may, at least initially, reduce sediment transport from the downdrift beach into the inlet. On the other hand, redirection of the ebb jet might make the inlet more hydraulically efficient, reducing the sediment trapping efficiency of the deposition basin and possibly forcing the ebb shoal to grow farther seaward.

10. *Sand Trapping and Bypassing System Updrift*

Similar to Shinnecock Inlet, an alternative bypassing approach that could be used in combination with channel and deposition basin maintenance dredging is to intercept, collect, and bypass sediment from the subaerial and subaqueous littoral material accumulated at the updrift fillet adjacent to the east jetty. As in Shinnecock Inlet, this could be accomplished by means of a conventional cutter-suction pipeline dredge and one of three sediment collection/dredge protection features: (1) jetty opening & nearshore breakwater, (2) weir jetty and sediment trap, or (3) offshore breakwater. Note, however, that the east jetty length and updrift fillet area available for this type of operation at Moriches Inlet are considerable smaller than at Shinnecock Inlet, thus it might be more difficult and/or costly to implement this alternative.

11. *Reduce Authorized Channel Depth*

This alternative is similar to Existing Practice, since presently the channels at Moriches Inlet are not regularly maintained at the authorized depth. Nonetheless, this alternative is being considered in the present study because only recreational vessels and the U.S. Coast Guard typically use the channel. It would, however, require reauthorization of the Moriches Inlet navigation project.

Based on the design dimensions typical of the recreational vessels that use the inlet, the minimum required depth in the channel, absent waves, is -7 feet MLW (see Section 7.3). In the presence of waves, a minimum channel depth of -11 feet MLW is required to maintain safe navigation 90% of the time at MLW (Figure 7-5). Unfortunately, small reductions in channel depth translate in significant increases in "downtime." For example, if the channel were reauthorized to a depth of -9 feet MLW, safe navigation would be maintained less than 30% of the time, although this percentage would be greater during the summer and fall recreational boating season. Further reductions in authorized channel depth would be moot because boaters would probably choose to navigate along the natural channel thalweg and the deeper gaps along the southwest edge of the ebb shoal.

3.4.3 Fire Island Inlet

Existing Conditions and Design Considerations

Previous sediment budget estimates (Gravens et al., 1999), principally based on shoreline changes from 1979 to 1995, suggest a long-shore sediment transport rate of approximately 188,000 m^3/yr (245,000 cy/yr) entering the inlet from the east and under existing (circa 1999) conditions. However, recent shoreline and volumetric changes (1995 to date) suggest that the net westerly transport at Fire Island Inlet might be significantly higher, 295,000 m^3/yr (386,000 cy/yr). This amount is consistent with recent (1997, 1999 and 2001) maintenance dredging operations which have yielded approximately 279,000 m^3/yr (365,000 cy/yr). More recently, the inlet was also dredged in 2003-04 and the next maintenance dredging project is under re-evaluation to be scheduled for the Fall of 2007.

1. *Existing Practice (Dredging of Deposition Basin & Channel)*

This alternative consists of continuing the Existing Practice of channel and deposition basin dredging approximately every two years (Figure 2-5). This practice has recently (1995 to 2001) resulted in dredging of approximately 279,000 m^3/yr (365,000 cy/yr), 80% of which are placed at downdrift at Gilgo Beach and 20% (depending on the need) are placed updrift within Robert Moses State Park. Although there are only a few surveys of the ebb shoal available and most have a very limited coverage, the data suggest that the ebb shoal is still accumulating sediment at a rate of approximately 68,000 m^3/yr (89,000 cy/yr). This may be because part of the material bypassed to Gilgo Beach returns to the ebb shoal, which means that the net bypassing rate at the inlet is on the order of 125,000 m^3/yr (169,000 cy/yr). In other words, even though Existing Practice appears to bypass a significant percentage of the net westerly transport immediately updrift of the inlet, there might still be an opportunity to improve bypassing efficiency.

The shoreline east of the inlet along Robert Moses State Park still suffers from significant fluctuations which may endanger existing park facilities, thus the need for regular backpassing of sediment from the inlet.

Navigation conditions are less than optimal at the inlet under Existing Practice. The dredged channel and deposition basin shoal rapidly after maintenance dredging operations which typically requires the Coast Guard to move the channel buoys about twice a year as the channel is encroached by the westerly moving shallow sand spit that extends from the east jetty. This despite the fact that the channel and deposition basin are typically dredged significantly deeper in areas adjacent to the eastern tip of the shoal than the authorized depth (see figures in Appendix A).

Finally, the relatively long inlet thalweg combined with the continued growth of the eastern sand spit have made the inlet hydraulically inefficient, a condition which, if not addressed, may lead to worsening navigation conditions as well increasing risk of significant breaching and unchecked inlet growth at other locations in Great South Bay during extreme storm events.

2. *Existing Practice plus Discharge farther West*

In concept, if the sediment bypassed to Gilgo Beach is placed farther west, the amount of sediment flowing back towards the ebb shoal might be reduced, which will improve overall bypassing efficiency and arrest ebb shoal growth. The maintenance dredging project completed in 2003-2004 called for placement starting approximately 7,000 feet farther west than the previous project in 2001. This modification in placement location is also included by default in most of the alternatives discussed below.

3. *Optimize Existing Channel and Deposition Basin Configurations*

This alternative consists of optimizing the alignment and dimensions of the existing deposition basin to reduce channel shoaling and possibly to improve navigation. For example, shifting the channel alignment farther north and west to adjacent areas of naturally deeper water may accomplish these two goals (Figure 3-14). Note, however, that also shifting the deposition basin farther west and allowing the sand spit to grow to the eastern edge of this new alignment would only delay the problem for a couple of years. The deposition basin would need to be located where it is or even farther east to allow for sediment trapping and dredging prior to shoaling of the channel itself (Figure 3-14). In addition, a wider basin configuration excavated to a deeper depth and limited to the leading edge of the sand spit may be more efficient that the existing basin which is relatively narrow, long and parallel to the channel alignment. Sediment does not appear to accumulate in significant quantities within the outer, trapezoidally shaped, portion of the existing deposition basin. The optimized deposition basin could be designed so as to minimize wave impacts on navigation along the east to west section of the channel and Oak Beach. Alternatively, if a significant portion of the sand spit were removed, a training dike/breakwater could be built in the area where the tip of the spit is presently located. This dike could provide protection to navigation, dredging operations and Oak Beach.

In theory, this alternative should allow for continuation of existing bypassing rates. Bypassing efficiency would also be improved by placing the material farther west along Gilgo Beach.

4. *Eastern Realignment of Channel and Deposition Basin*

This alternative is similar to the previous one in that it aims at reducing channel shoaling and attendant impacts on navigation by realigning the channel and reconfiguring the deposition basin. The difference is that the channel and basin would be shifted eastward toward the east jetty (Figure 3-15). This would require dredging of all or most of the existing sand spit. As in the previous alternative, the deposition basin would be designed so as to prevent encroaching of the spit (which would tend to grow again) into the channel by enlarging the basin in the areas subject to the most deposition and reducing its footprint (or eliminating it) in areas farther seaward, where it does not appear to serve a significant purpose.

One added benefit of this alternative is that it would improve inlet hydraulic efficiency as compared to the Existing Practice. As in the previous alterative, eastern realignment of the channel and deposition basin should allow for continuation of existing bypassing rates. Bypassing efficiency would also be improved by placing the material farther west along Gilgo Beach.

On the other hand, this alternative requires dredging of valuable environmental habitat. In addition, the spit provides protection to the Oak Beach shoreline from S to SW waves, although a protection structure could also be built to mitigate this effect. These and other impacts are further discussed in Section 3.6 below.

5. *Existing Practice plus Dredging the Flood Shoal*

The flood shoal at Fire Island Inlet, which is defined here as the shallow areas that extend from the thumb, east along Oak Beach past the Robert Moses State Park causeway and into the areas adjacent to the eastern tip of Jones Island, contains a large volume of sediment that has been carried to those areas as the Inlet migrated west. Limited dredging of these areas (assuming that the material is compatible with beach sand) may be a way of supplementing existing bypassing practices in order to achieve 100% bypassing of the net westerly transport arriving at Fire Island Inlet. In addition, dredging some of these shallow shoal areas may improve inlet hydraulic efficiency which would reduce shoaling at the mouth. Dredging within the inlet would also be less susceptible to downtime due to weather and less costly overall than dredging areas of the ebb shoal.

On the other hand, this alternative might have issues with sediment compatibility, and using the flood shoal to increase bypassing does not improve bypassing efficiency or navigation conditions.

6. *Dredging the Ebb Shoal (outside limits of Deposition Basin) with a Floating Plant*

Similarly to Shinnecock Inlet and Moriches Inlet, one alternative to increase bypassing is to remove sediment from other areas of the ebb shoal such as the seaward slope. Ebb shoal dredging may be easily implemented using a traditional floating plant on contract in combination with the regular maintenance of the authorized channel and deposition basin. This alternative would offset the continued growth of the ebb shoal which is now estimated at 68,000 m^3/yr (89,000 cy/yr). However, a program to survey the ebb shoal on a regular basis would need to be implemented before moving forward with this approach.

7. *Semi-fixed Bypass System*

A semi-fixed bypass system could also be implemented at Fire Island Inlet. However, estimated net westerly transport rates are significantly higher at Fire Island Inlet than at Shinnecock or Moriches. Therefore, if a crane-mounted jet system is selected, the system will only provide for dredging and bypassing of approximately 50% of the net littoral drift, the rest would have to be supplemented by channel and deposition basin maintenance operations similar to existing practice, albeit in a longer dredging cycle. Alternatively, a system of several pumps on a trestle (with or without a boom that would increase reach and mobility) could be designed to provide for a design bypass rate closer to the net westerly drift. As explained above, however, regardless of system capacity, it is very unlikely that channel maintenance could be completely eliminated.

8. *Existing Practice plus Extension of the East Jetty*

This alterative calls for extension of the east jetty, which in fact was part of the Federal project authorized in 1971, although this recommendation was never implemented (Figure 3-16). Although extending the jetty would initially reduce the amount of sediment that flows past the tip of the structure, along the sand spit and into the deposition basin and channel, this effect would only be temporary. Therefore, the extended jetty would not significantly reduce dredging needs at the inlet in the long term. The extension, however, may mitigate the need for backpassing (and this continuous rehandling) to RMSP, although historically material has been placed a significant distance away from the jetty, and the effects of jetty lengthening on this area might not be felt for years. In theory, the jetty extension could be designed to initially allow for about 80% of the existing westerly transport at the jetty to pass around. This is the amount that is eventually bypassed via dredging to Gilgo Beach. With time more of the net westerly littoral transport would bypass the jetty and eventually present levels of bypassing would be reached. The main difference then would be that all of the material dredged from the inlet could be bypassed to Jones Island with no further need for backpassing to RMSP.

A feature that could be added to this alternative would be to "prefill" the updrift shoreline to the expected alignment with the extended jetty in place using sediment from the seaward edge of the ebb shoal. This would reduce the temporary impacts on sediment bypassing that the extended jetty might have prior to the shoreline reaching a new equilibrium position.

Needless to say, the performance and effects of this type of alternative are very sensitive to our understanding of average annual longshore sediment transport quantities, which to date, are not accurately determined.

9. *Reconfiguration of the Sore Thumb (and Channel Realignment)*

The "thumb" was originally built in the 1959 to force a southward shift of the navigation channel to prevent continued erosion of the north shoreline of the inlet (i.e., Oak Beach). To that end the thumb has been reasonably successful. In theory, the thumb could be reconfigured (or even eliminated) to address existing conditions and possibly improve inlet hydraulic efficiency. Although removing the thumb will release a significant amount of sediment presently "trapped" immediately west of the thumb and make it available to the downdrift littoral system along Jones Island, other effects on bypassing, shoreline stability and navigation are likely to be negative. The inlet thalweg is likely to shift north possibly impacting Gilgo Beach and Oak Beach. The channel thalweg may also become longer and less hydraulically efficient. Finally, it will be more difficult to maintain the footprint of the existing sand spit, which will tend to grow west faster as the natural channel moves northward and westward.

10. *Sand Trapping and Bypassing System Updrift*

As in Shinnecock and Moriches Inlet, intercepting, collecting, and bypassing sediment from the subaerial and subaqueous littoral material accumulated at the updrift fillet adjacent to the east jetty is an alternative bypassing approach that could be used in combination with channel (and deposition basin) maintenance. As in those two inlets, this could be accomplished by means of a conventional cutter-suction pipeline dredge and one of three sediment collection/dredge protection features: (1) jetty opening & nearshore breakwater, (2) weir jetty and sediment trap or (3) offshore breakwater. Unlike Moriches Inlet and similar to Shinnecock Inlet, Fire Island Inlet has a large area available to implement this type alternative. In fact, the existing sand spit may be incorporated into the design as the area where sediment is dredged from.

11. *Groins East of the Inlet*

Similarly to the east jetty extension, but much more effectively, a series of groins along the eastern end of Robert Moses State Park could be designed to reduce renourishment needs along this area without significantly affecting net westerly transport (which is about 80% of existing dredging rates at the inlet). The groins would have to be pre-filled to the anticipated stable shoreline configuration (Figure 3-17). As with the extension of the east jetty, this type of alternative is very sensitive to miscalculations regarding the existing littoral sand transport conditions. Other impacts are also discussed in sections below.

12. *Move the Inlet back to the Lighthouse Location*

A more drastic approach to addressing the needs at Fire Island Inlet would be to move the inlet eastward to one of its historic locations between its present one and the Fire Island Lighthouse, where it was located in the early 19th century. In theory, ebb and flood shoals associated with the new inlet could be pre-filled (at least partially with material excavated from the construction of the new inlet) to prevent impacts on adjacent shorelines. A bypassing system would have to be implemented too. Finally the inlet would have to be designed so as to minimize detrimental impacts on the hydraulic and water quality conditions of Great South Bay. Clearly, however, the new inlet would be more hydraulically efficient than the existing one assuming both had similar inlet throat widths.

The volume of sediment accumulated in the existing ebb shoal, over 30 million m^3 (41 million cy), would become a natural source of material to the downdrift shoreline that could last for over 100 years assuming existing net westerly transport rates. Navigation reliability could be significantly improved if the new channel, which would be better aligned to incoming waves than the existing one, was adequately maintained.

Obviously, this type of modification has other significant economic, environmental, and social impacts that are addressed in more detail in Section 3.6 below.

3.5 Initial Multiple Criteria Screening Analysis

To screen the remaining inlet modification alternatives and develop a limited set of alternatives that will be subject to detailed modeling and design, a screening analysis was applied. The selection of alternatives for each of the three inlets requires the careful balancing of multiple, sometimes conflicting, factors. Consideration of the different choices becomes a *Multiple Criteria Decision Analysis* (MCDA). MCDA, "an umbrella term used to describe a number of formal approaches that seek to take explicit account of multiple criteria in helping individuals or groups explore decisions that matter" (Belton and Stewart, 2002), has developed rapidly over the last 25 years and a number of different methods with different degrees of complexity have emerged. Note, however, that none of these methods will provide the decision makers with the "right" answer, nor an "objective" analysis that will relieve the decision makers of the responsibility of making difficult judgments. On the other hand, MCDA is an "aid" to decision making that will make subjectivity explicit and help manage it. In other words, MCDA does not eliminate subjectivity (e.g., relative "weight" given to specific criteria), but it makes subjective judgments explicit and the process by which they are taken into account transparent to the decision makers and stakeholders. Above all, MCDA facilitates learning about and understanding of the problems, goals, constraints and preferred course of action. It forces hard thinking about the generation of alternatives, anticipation of future contingencies, examination of secondary effects, illuminates controversy, and may facilitate compromise (Keeney and Raiffa, 1972).

Belton and Stewart (2002) further emphasize the usefulness of MCDA methods as follows:

- MCDA seeks to take explicit account of multiple, conflicting criteria in aiding decision making;
- The MCDA process helps to structure the problem;
- MCDA provides a focus and a language for discussion;
- The principle aim is to help decision makers learn about the problem situation, about their own and others values and judgments, and through organization, synthesis and appropriate presentation of information to guide them in identifying, often through extensive discussion, a preferred course of action;
- The analysis serves to complement and to challenge intuition, acting as a sounding-board against which ideas can be tested. It does not seek to replace intuitive judgment or experience;
- The process leads to better considered, justifiable and explainable decisions. The analysis provides an audit trail for a decision.

There are a number of MCDA models which can be classified into three broad categories (Belton and Stewart, 2002), namely:

1. ***Value measurement models*** in which numerical scores are constructed in order to represent the degree to which one decision option may be preferred to another. Such scores are developed initially for each individual criterion, and are then synthesized.

2. ***Goal, aspiration, or reference models*** in which satisfactory levels of achievement are established for each of the criteria. The process then seeks to discover options which are in some sense closest to achieving the desirable goals and aspirations.

3. ***Outranking models*** in which alternative courses of action are compared pair wise, initially on terms of each criterion, in order to identify to which extent a preference for one over the other can be asserted. Preference information is then aggregated across all criteria therefore establishing the strength of preference of one alterative over another.

A *value measurement* method was used in this study for further screening of the bypassing alternatives that passed the fatal flaw analysis. The simplest value measurement method is the "additive" model:

$$V(a) = \sum_{i=1}^{m} w_i \cdot v_i(a)$$

where:

V(a) is the overall value or score of alterative *a*
$v_i(a)$ is the value score reflecting alternative *a*'s performance on criterion *i*
w_i is the weight assigned to reflect the importance of criterion *i*

Individual criterion scores, criterion weights, and values or score results for each alternative are typically summarized numerically in the form of matrix or visually in the form of bar graphs or "thermometer" scales. However, it should be stressed that the learning and understanding which results from engaging in the screening process is generally far more important than the numerical results.

For this study, a modified version of the standard "additive" model was implemented in the form of an alternative selection decision matrix. As in the standard method, the matrix evaluates each of the alternatives based on their performance with regards to several criteria. In addition, the proposed method weights the resulting overall values according to how well each alternative performs with regards to the stated project needs. In summary, an overall value or score for each alternative is computed based on the following two basic scores:

(1) *Performance Score:* How well does the alternative meet the stated needs (accounting for risk & uncertainty inherent to each alterative and their expected performance), and
(2) *Total Criteria Score:* How beneficial (or adverse) is each alternative with regards to a specific set of criteria.

Each score is computed using the additive model. Raw *Performance* scores are normalized to a scale from 0 to 10 (0 for not meeting any needs, 10 for meeting them 100%), and raw *Total Criteria* scores to scale from 0 to 100 (from a very adverse to a very beneficial alternative with regards to the selected set of criteria). The *Final Score* for each alternative is computed by multiplying the *Performance Score* times the *Total Criteria Score.* Therefore, the maximum possible score is 1000. As such, the proposed value measurement model gives more weight to meeting the stated needs than meeting any specific criteria. This is important because an alternative may be very beneficial with respect to the selected criteria, but it might not meet the stated needs, which would make its implementation moot. Other relevant characteristics of the proposed screening approach are explained in the following paragraphs.

Five general *Criteria Categories* were defined: Environmental, Economical, Recreational, Engineering, and Cultural/Social, each including specific individual criteria (see below). A single weighted average score for each Criteria Category is computed for each alternative based on the raw scores for each specific criteria (e.g., cost). Scores for each criteria category are normalized to scale from 0 to 20. The scores for each Criteria Category are then added up to compute the *Total Criteria Score*. This approach avoids unfairly weighting a specific category (say Engineering) just because more specific criteria were identified for that category as compared to others. In other words, the maximum score for any Criteria Category is the same and is added to compute the *Total Criteria Score* as follows:

Criteria Category	**% of Total Score**
Environmental	20
Economic	20
Recreational	20
Engineering	20
Cultural and Social	20
Total	**100**

The scoring process was based on a "qualitative value scale" method, which assesses the performance of alternatives by reference to descriptive pointers (i.e., word descriptions) to which appropriate values are assigned[2]. The following table shows the descriptive pointers and related grades (on a scale of 1 to 5) used in this analysis. Note that all criteria were assigned the same weight within each general Criteria Category.

Grade/Value	*Description*
5	Very Beneficial
4	Beneficial
3	Neutral
2	Adverse
1	Severely Adverse

Therefore, if an individual alterative was considered "Neutral" with regards to all of the selected criteria, the score for each *General Criteria Category* would be 12 (i.e., 3 normalized to a 0-20 scale) and the *Total Criteria Score* would be 60 (out of a maximum of 100).

The *Performance Score* is computed based on how well each alternative meets the stated needs (on a scale of 0 to 10 as described above) and how much risk & uncertainty is associated with the alternative with regards to those needs (measured in terms of percentage). For example, one alternative may theoretically meet the stated needs (score of 10), but if this type of alternative has never been implemented before either at the inlet in question or anywhere else with similar conditions and it also deviates significantly from existing practice, the risk & uncertainty would be relatively high (e.g., 40% uncertainty), the *Performance Score* would be reduced to 6 (i.e., 60% x 10).

A conceptual example of a screening matrix, only showing a "rolled up" *Total Criteria Score*, is provided in the following table:

Table 3-6: Example Preliminary Screening Matrix

	Project Needs Score (Max. 10)			Risk and Uncertainty (%) (100% is worst)	Performance Score (Max. 10)	Total Criteria Score (Max. 100)	**FINAL SCORE (Max. 1,000)**
	Navigation	Local Erosion	Bypassing				
"Beneficial" Alternative	10	8	8	5	8.2	80	**659**
"Neutral" Alternative	10	8	8	20	6.9	50	**347**
"Adverse" Alternative	4	2	2	50	1.3	30	**40**

[2] Other alternative scoring systems are "quantitative value functions" (performance is scored in terms of a measurable attribute reflecting the criterion of interest, this approach is generally to difficult to implement for most typical criteria) or "direct rating" (performance is scored based on an arbitrary analog scale with no specific definition of performance characteristics)

3.6 Screening Criteria and Alternative Scoring

The next step in the analysis was to develop a comprehensive list of screening criteria, using Multiple Criteria Decision Analysis (MCDA). The process of developing these criteria and specific descriptions for each one are provided in the following paragraphs. An attempt was made to reduce the number of criteria to a reasonable amount that would adequately describe the pros and cons of each alternative by reflecting its impacts on the most relevant environmental, economic, recreational, engineering, social, and cultural conditions in the study area. At this level of this screening process, a concise but representative list of criteria allows for a more objective grading of the different alternatives because it does not unfairly weight very specific issues that happen to be included in the analysis while others (maybe as important) were forgotten or intentionally left out. It also minimizes the possibility of "double counting" the effects on certain issues that might otherwise be included under several different criteria.

One other important consideration in developing this list of criteria was to ensure that screening process would account for relevant New York State Coastal Management Program (CMP) Policies (NYSDOS, 2002). A summary of these policies is provided in Appendix D. Table 3-7 at the end of this section summarizes the correlation between the selected criteria and these policies.

The following paragraphs provide a description of the selected criteria as well as a general description of how the alternatives presented above were scored. Scores and screening results are shown in Tables 3-8, 3-9 and 3-10 in Section 3.7.

3.6.1 Environmental Criteria

1. *Fish and Wildlife*

This criterion reflects impacts of alternative modifications to fish, wildlife, and plants within the study area. It also includes consideration of the possible impacts to several NYDOS-designated significant coastal fish and wildlife habitats within the inlet modifications study area, which include: Southampton Beach, Tiana Beach, Shinnecock Bay, Dune Road Marsh, Moriches Bay, Great South Bay (West), Great South Bay (East) and the Sore Thumb. This criterion reflects any potential impacts to these habitats. Accordingly, this criterion is consistent with the intent of CMP *Fish and Wildlife Policies* (7 to 10). Note that specific impacts to rare and endangered species are accounted for in the next criterion.

The study area includes several habitats (i.e., offshore, nearshore, barrier island, and back bay) and a number of benthic (crustacean and molluscan shellfish and other invertebrates), finfish, mammals, reptiles, shorebirds, waterfowl, and vegetation species. Examples of relevant species that may be impacted by inlet modifications are surf clams, bay scallops, winter and summer flounder, striped bass, cord grass, and eelgrass (i.e., Submerged Aquatic Vegetation, SAV).

In general, most of the alternatives considered in this analysis will have a relatively small impact on fish and wildlife resources and habitats. For example, finfish are highly mobile, and would be able to avoid construction areas. Nonetheless, some bottom fish may be entrained in the intake of a dredge, and increased turbidity during construction will affect the gills of finfish. Benthic organisms offshore and nearshore may also be affected by direct removal from borrow sites or burial in fill placement areas. On the other hand, dredging of areas of the ebb shoal for example might expose food sources that were previously unavailable. In addition, recolonization of these areas would begin almost immediately and new benthic communities would be established in 12 to 18 months (USACE, 1999). Therefore, most alternatives were considered "Neutral" or "Adverse" with regards to Fish and Wildlife, depending on the location and dimensions of the sources of material and placement areas. Alternatives that might induce slightly larger impacts are those that include dredging of flood shoal areas, which may impact existing SAV beds and a number of species (e.g., scallops or hard clams) that depend on eel grass beds for survival. Therefore these alternatives are considered "Severely Adverse" relative to others. Dredging the flood shoals may also, at least indirectly, impact a number of wading and shorebird species. For example, dredging the sand spit at Fire Island would also be considered "Severely Adverse".

Alternatives that directly impact existing significant habitats by, for example, reducing their total area are also considered, in relative terms, "Severely Adverse". These include removing the Sore Thumb at Fire Island Inlet, or constructing relatively large updrift sediment collection and bypass facilities.

Note that these grades are not meant to provide a detailed assessment of the environmental impacts associated with any of these alternatives in absolute terms. They are only supposed to provide an assessment of the relative differences between alternatives for the purposes of screening. Therefore, "Severely Adverse" grades for this and other criteria should not be interpreted literally, but in the context of this analysis.

2. *Rare and Endangered Species*

This criterion refers to potential impacts to rare and endangered species that may be found within the study area; examples include several species of sea turtles, piping plovers and other rare/endangered shorebirds, and endangered and rare plant species (e.g., seabeach amaranth). The criterion is also consistent with CMP Policy 7, *Significant Habitats*, which also includes consideration for rare and endangered species.

Piping plovers and other rare and endangered shorebird species are found in the vicinity of all three inlets. Other special status species, however, are relatively rare or non-existing in the vicinity of the inlets during the expected operation periods: e.g., whales, porpoises and turtles. This is not to say they should not be considered, but it is more likely that plovers and other rare/endangered shorebirds are more sensitive to impacts from the various inlet modification alternatives. Therefore, the scoring of alternatives was generally controlled by the impacts of the alternative on endangered or threatened shorebirds, unless others were specifically impacted (e.g., seabeach amaranth). In general, all of the alternatives will have to incorporate environmental windows into their design and operation that are meant to protect plovers and other shorebird species. For example, fill placement (and thus dredging) cannot take place between April 1 and August 31. Note that the engineering and economical impact of the environmental windows is not accounted for in this criterion but in others below. Therefore, scoring for this criterion is intended to reflect the possible reductions (or increases) in shorebird habitat rather than the direct impacts, assuming that environmental windows would effectively prevent those direct impacts.

Alternatives that significantly reduce existing shorebird habitat (e.g., dredging of the sand spit at Fire Island Inlet or fixed updrift bypass facilities that reduce the size of the updrift fillet) are considered "Severely Adverse" with regards this criterion, others that might slightly reduce available habitat on the updrift side of the inlet by reducing the width of the updrift fillet but might increase the dry beach area on the downdrift, were considered "Neutral". Alternatives that may actually increase the net available habitat area, such as increased downdrift fill placement from sources such as the ebb shoal, or offshore were considered "Beneficial" relative to others.

Alternatives that eliminate the sand spit at Fire Island will also be considered "Severely Adverse" because of potential impacts on rare and endangered seabeach plants such as the seabeach amaranth or seabeach knotweed, which depend on this type of dynamic shoreface environment. Similarly, alternatives that prevent breach formation with structural stabilization are also considered "Severely Adverse" with regards to this criterion.

The potential for impacts on other rare or endangered species such the various species of sea turtles, the diamond back terrapin, or whales is relatively low and similar for the all the considered alternatives. Therefore, the effects with regards to these species were not specifically considered in the scoring for this criterion.

3. *Water Quality*

This criterion reflects inlet modification effects on water quality conditions within Shinnecock, Moriches and Great South Bay. This criterion also considers relevant CMP Water and Air Resources Polices (30 to 44) such as Policy 30, *State and National Water Standards* and Policy 35, *Dredging and Disposal*. Although it is noted that most of these policies are intended for things like sanitary waste water, solid waste, industrial discharges, hazardous materials, etc, which are not features of any of the alternative plans considered in this study. Policy 44 *Tidal and Freshwater Wetlands* is considered in this study as a specific evaluation criterion (see next).

In general, increased hydraulic efficiency at the inlets will lead to improved water quality. Therefore, alternatives that significantly improve this efficiency are considered "Highly Beneficial" (e.g., relocation of Fire Island Inlet). None of the remaining alternatives are expected to significantly reduce efficiency, although some of them might reduce it slightly in order to reduce theoretical sediment losses to the ebb shoal and to increase the trapping efficiency of the deposition basin (e.g., reduced dimension of the deposition basin at Shinnecock Inlet). These alternatives are considered "Adverse" as compared to others. Finally, alternatives that are expected to maintain similar hydraulic efficiency are considered "Neutral."

Water quality impacts related to dredging and placement may include turbidity, sedimentation of fine sediments, and release of fines high in organics are only moderately adverse and short-term. In addition, it is not expected that dredging would affect dissolved oxygen conditions. In other words dredging related to inlet maintenance and bypassing does not produce a long-term significant adverse impact to water quality. Therefore, whether or not an alternative included dredging (almost all do), or the relative size of the dredging project, was not considered in the development of water quality scores.

4. *Tidal and Freshwater Wetlands*

According to NYS definitions and policies, wetland areas include the following ecological zones: coastal fresh marsh; intertidal marsh; coastal shoals, bars and flats; littoral zone; high marsh or salt meadow; and formerly connected tidal wetlands. The study region contains numerous areas officially delineated as tidal wetlands in New York State Department of Environmental Conservation's (NYSDEC) Tidal Wetlands Inventory Map, mostly as coastal shoals, bars and flats. This criterion accounts for any impacts to these areas and as such it is consistent with the intent of CMP Policy 44, *Tidal and Freshwater Wetlands***,** which is to preserve and protect the wetlands as well as the benefits derived from them.

In general, most of the alternatives considered will have a very limited effect on these wetlands, with the exception of alternatives that directly impact some of these resources such as Fire Island Inlet relocation, dredging of flood shoals or sand spits, and significant updrift fillet mining, which are considered "Severely Adverse" relative to others. The rest of the alternatives considered in this study are considered relatively "Neutral."

5. *Sediment Pathways*

This criterion includes a relative broad range of issues related to sediment transport processes within the study area. Specifically, it reflects the degree to which an alternative may or may not restore a natural longshore sediment pathway around the inlet (i.e., natural bypassing). It also considers whether or not an alternative will reduce the amount of sediment that might be lost from the longshore sediment transport system to adjacent areas such as ebb and flood shoals or the extent to which an alternative might reduce or increase erosion in adjacent shorelines or impact natural protective features such as dunes.

On the surface, this criterion seems very similar to the stated project need of reduced erosion (local and farther downdrift). However, note that the intent of this criterion is to reward alternatives that reduce or eliminate erosion by means of restoring natural processes (i.e., natural sediment transport around the inlet), not by using external sand sources (e.g., offshore dredging), stabilization structures, or even dredging. Finally, the criterion also considers whether alternatives will impact natural cross-shore sediment transport processes generally associated to storm events (i.e., overwash and breaching) and related barrier island migration processes. The criterion therefore reflects the intent of CMP Policies 12 and 15.

Alternatives that maximized natural bypassing around the inlets by means of reducing the authorized channel depth and/or deposition basin dimensions (e.g., reduced channel depth at Moriches Inlet or reduced deposition basin at Shinnecock Inlet) were considered “Highly Beneficial.” Alternatives that increased natural bypassing by reducing inlet hydraulic efficiency or optimizing the size and location of the channel and deposition basin (e.g., channel and deposition basin relocation at Shinnecock Inlet) were considered “Beneficial.” Existing condition practices were considered “Beneficial”, “Neutral” or “Adverse” at Moriches, Fire Island, and Shinnecock inlets, respectively. Actions that achieve a significant percentage of the total design bypassing rate by means of updrift “non-floating” plants or dredging of the deposition basin or other areas of the ebb shoal were considered “Neutral” because although they replace the natural bypass system, they do not restore it. Note that these type of alternatives, however, are generally much more effective than others (e.g., optimization of channel and deposition basin to increase natural bypassing), although these benefits are accounted for under separate criteria. Actions including “external” sources of sediment such as offshore dredging were considered “Severely Adverse”, and actions involving the use of flood shoal sediments were considered “Adverse.”

6. Non-Structural Components

This criterion reflects the extent to which the project needs are addressed by non-structural means such as changes in management or maintenance practices versus structural measures such as expansion of existing inlet structures or construction of new ones. Therefore this policy is consistent with the intent of CMP Policy 17, *Non-structural Control Measures.*

In general, alternatives including only small changes to existing dredging practices (e.g., realignment of the channel and deposition basin, redesign of the deposition basin) or reductions in the footprint of existing structures (e.g., shorten the east jetty at Shinnecock Inlet) were considered “Highly Beneficial” with regards to this criterion and thus received a score of 5. Others including modifications or small extensions of existing structures (e.g., spur jetty at Shinnecock Inlet) and/or soft solutions to the localized erosion problem (e.g., flood shoal dredging) were considered “Neutral”. Finally, alternatives that included significant structural increases in order to meet the project needs (e.g., groins along the West Beach at Shinnecock Inlet or east jetty extension at Fire Island) were considered “Adverse” or “Severely Adverse.”

3.6.2 Economic Criteria

7. Lifecycle Costs

This criterion considers the cost over the life of the project including the initial construction costs as well as the present value of future maintenance and operation costs. Consideration of lifecycle costs is consistent with CMP General Policy 18, which safeguards, among others, the economic interests of the State of New York as a local cost sharing sponsor.

Note that the annualized cost of each alternative is weighted similarly to other criteria.

Alternatives that include relatively infrequent dredging and rely mostly on optimized natural bypassing (at the expense of reduced efficiency or impacts on navigation reliability) are considered to be the least expensive options. This is the case, for example, of existing (or similar to) bypassing practices at Shinnecock and Moriches Inlets. Therefore, these types of alternatives will be considered "Highly Beneficial" in terms of lifecycle costs.

As far as the relative costs difference between floating plant alternatives (e.g., some combination of channel, deposition basin and ebb shoal dredging) and semi-fixed bypass alternatives (crane or truck mounted dredging of the updrift fillet), the relative scoring was based on order of magnitude costs estimates found in the literature and more detailed estimates found in previous studies. According to Bruun (1993), annualized costs for fixed and semi-fixed bypassing plants range from \$5 to \$12 per m^3/yr and \$3 to \$7 per m^3/yr for floating plants. Of course, these costs depend on a multitude of project specific variables. Nonetheless, this range of numbers confirms the general knowledge that fixed bypassing system installations are generally less effective and cost more to run than floating systems. Although modern systems such as the crane mounted jet pump at Indian River, Delaware (\$6/m^3/yr), or the new fixed jet pump systems at Nerang, Australia (\$3/m^3/yr), appear to be much more effective than older systems.

However, the estimates for fixed and semi-fixed systems do not typically include maintenance of adjacent channels, and this additional cost may be as high as or higher than the cost of bypassing. For example, Williams et al. (1998) analyzed several sand bypass options at Shinnecock Inlet and concluded that the cost of a floating plant (hopper dredge) alternative operation on a 2 year cycle and discharging onshore west of the ebb shoal attachment would cost \$1.2 million per year (or approximately \$12 per m^3). This cost is higher than the range of costs presented by Bruun (1993) because of the relatively small annual bypass rate (100,000 m^3) required at Shinnecock Inlet. On the other hand, Williams el al. suggest that the annualized cost of the crane mounted pump system would be approximately \$800,000 per year (approximately \$8/m^3/yr). However, their cost analysis does not include the additional cost of maintaining, albeit less frequently, the navigation channel, which would have to be maintained with a floating plant. This additional cost may be on the order an additional \$600,000 per year (\$6/m^3/yr), which would make a semi-fixed system slightly more expensive than a floating system. Therefore, for the purposes of this initial screening analysis, floating plant systems were considered "Beneficial", while semi-fixed systems where considered "Neutral" in terms of lifecycle cost.

Alternatives that require significant first costs to construct relatively large new structures or significant modifications to existing ones are considered to be more costly than others, and therefore "Adverse" or "Severely Adverse," depending on the type, number, and size of the structures. Relocation of existing maritime facilities at Shinnecock Inlet was considered "Severely Adverse". Fire Island Inlet relocation was also considered "Severely Adverse."

8. <u>*Flooding Risk*</u>

The Flooding Risk criterion accounts for the possibility of increased or reduced mainland flooding risks due to changes in tidal range, or changes in the way ocean storm surge propagates through the inlets during extreme storm events. Therefore, this criterion partially reflects the intent of CMP Policy 14, *No Flooding or Erosion Increases***,** (note that consideration of erosion increases is included in Criterion No. 6 and the overall project needs).

All the alternatives included in this analysis (excluding the ones that were eliminated in the fatal flaw analysis) are intended to increase bypassing and reduce erosion. Therefore, all of them have a "Beneficial" impact on flooding risk. In addition, some of the alternatives, particularly at Shinnecock Inlet, may significantly reduce the likelihood of a breach and associated increase in bay flooding along the shoreline immediately downdrift of the inlet. These alternatives, which include stabilization structures and fill placement from offshore, are considered "Highly Beneficial". Note that alternatives that rely on various forms of bypassing from either the ebb shoal or the updrift fillet are "only" considered beneficial because of the possibility that the amount of material bypass by these methods may not be enough to meet the needs of both the West Beach and areas farther downdrift. Dredging the flood shoal at Shinnecock Inlet was considered neutral because although it may increase inlet hydraulic efficiency, it provided a source of material to maintain West Beach and prevent breaching. At Fire Island Inlet it was also considered neutral because a slight increase in hydraulic efficiency might be compensated by a reduced risk of breaching elsewhere. However, dredging the flood shoal at Moriches it was considered "Adverse" because it would increase surge propagation through the inlet with no other beneficial effects.

Alternatives that increase the tidal prism in the bay are considered "Adverse", or "Severely Adverse", depending on the estimated impact. For example, relocation of Fire Island Inlet is considered "Severely Adverse", whereas increasing the size of the deposition basin at Moriches Inlet and maintaining the basin and channel more frequently is considered "Adverse."

9. <u>*Commercial Fisheries*</u>

This criterion reflects the potential effects of inlet modification plans on commercial finfish and shellfish resources. Therefore, the criterion is consistent with the intent of CMP Policy 10, *Commercial Fisheries*.

In general, it is not expected that any of the alternatives considered in this analysis will have a significant impact on commercial fishery resources (see Fish and Wildlife criterion above). Dredging and beach fill construction activities typically related to bypass and channel maintenance would only affect a small area of the total fishing grounds available at the three inlets. Although commercially valuable surf clams may also be affected by dredging and bypassing activities, significant impacts to the population are not expected. Offshore dredging alternatives, however, may have a more significant impact on this resource and thus were considered "Adverse".

Eelgrass meadows support commercially important species such as bay scallops, hard clams, and winter flounder. Therefore, alternatives that include direct impacts to eelgrass (i.e., removal by dredging) are considered "Adverse," at least relative to other alternatives considered herein. These include dredging of flood shoal areas in Shinnecock, Moriches or Great South Bay. Relocating Fire Island Inlet is also considered to have an "Adverse" impact because of the potential destruction of existing eelgrass meadows in the vicinity of the new inlet, even though new ones may form at the existing inlet location, if closed. Note that these alternatives are not considered "Severely Adverse" because they would also increase water quality in the bays which would have a beneficial effect on these fisheries and because the dredging footprint could be optimized to minimize direct impacts.

10. <u>*Waterfront Development and Commercial Fishing Facilities*</u>

Considers the potential effects of inlet modification plans on existing waterfront areas, including small harbors, and specifically commercial fishing facilities. This criterion is particularly relevant in the analysis of alternative plans that include relocation of an inlet and/or associated waterfront development (e.g., relocation of the commercial fishing port at Shinnecock Inlet, or relocation of Fire Island Inlet). It also considers whether relocation will take place within or near areas that, based on their physical, environmental and infrastructure conditions, are suitable for the type of facility that is to be relocated. As such, the criterion reflects the intent of CMP *Development Policies* 1 to 6, where relevant, and CMP Policy 10, *Commercial Fisheries*, which also considers effects on commercial fishing facilities.

Alternatives that directly impact existing waterfront development and particularly commercial fishing facilities are considered “Severely Adverse” (e.g., relocation of the Shinnecock Inlet maritime center). Alternatives that provide for protection of the West Beach and the Shinnecock Inlet commercial fishing facilities by means of increased bypassing or sand placement are considered “Neutral.” Alternatives that provide more reliable protection to these facilities (e.g., nearshore structures along West Beach at Shinnecock Inlet) are considered “Highly Beneficial”. Note that this alternative also reduces the need for dredging, which may reduce impacts on commercial fisheries too.

Alternatives that may promote improvement of existing or development of new facilities at other maritime centers in Moriches Bay (Center and East Moriches Maritime centers) are also considered “Beneficial,” these include all alternatives that improve navigation at Moriches Inlet. Similarly, alternatives that improve navigation reliability at Fire Island Inlet, which serves a number of maritime centers in Great South Bay, are also considered “Beneficial.”

11. <u>*Land Use and Ownership*</u>

Reflects the degree to which a proposed alternative will affect existing land uses and/or will require ownership changes. Generally, alternatives requiring new development in open areas or continuous use of these areas are considered “Adverse” (e.g., semi-fixed bypass plants or updrift sediment trapping and bypass systems), or “Neutral” (e.g., nearshore structures along West Beach). Alternatives that maintain existing use and ownership are considered “Beneficial.” Alternatives that require significant ownership and/or land use changes such relocation of the Shinnecock Inlet maritime center or Fire Island Inlet are considered “Severely Adverse”.

3.6.3 Recreational Criteria

12. <u>*Recreational Fish and Wildlife Resources*</u>

This criterion accounts for the effects of project alternatives on the access to existing consumptive and non-consumptive recreational use of fish and wildlife resources as well as impacts on the quantity of these resources. It also includes a consideration for alternatives that might result in the development of new resources. The criterion reflects the intent of CMP Policy 9, *Recreational Resources*.

In general, alternatives that improve navigation will have a “Beneficial” effect on the access to recreational fish resources. On the other hand, alternatives that reduce navigation reliability are considered “Adverse”. These grades are further weighted according to any direct impacts an alternative might have on these resources.

13. <u>*Water and Foreshore Related Recreation Resources*</u>

This criterion pertains to the protection, maintenance, and improvement of water and foreshore related recreational resources such as surfing, boating, fishing, swimming, sunbathing, sightseeing, bird watching, etc. It also refers to any effects a project alternative might have on public access to these resources and particularly to any publicly-owned lands within the study area. As such, this criterion is consistent with CMP *Public Access and Recreation Policies* 19 to 22.

Increased bypassing would mitigate existing downdrift erosion and it would maintain or extend the beach frontage in the vicinity of all three inlets. Increased bypassing and reduced erosion would also maintain barrier island integrity and existing access roads such as Dune Road. Note that all of the alternatives assume that operations would take place between Labor Day and Memorial Day, therefore impacts on foreshore related recreation during the summer season would be minimal. Therefore, most of the considered alternatives will have positive effects on recreational resources and are considered "Beneficial."

Exceptions are alternatives including offshore and flood shoal sediment sources which were rated "Neutral" because slight differences in sediment grain size might affect nearshore beach profile morphology and recreational value. Also, alternatives that include semi-fixed equipment or facilities are considered "Adverse" and alternatives that include nearshore structures were considered "Severely Adverse," relative to others.

In terms of surfing, since there are number of surf breaks in the vicinity of the three inlets, alternatives that included significant modifications to the inlet structures or adjacent shoals were considered "Adverse" or "Severely Adverse" depending on the other considerations listed above. However, it should be noted that sand and structure placement strategies meant to improve surfing conditions could be considered as part of the detailed design of these alternatives.

3.6.4 Engineering Criteria

14. *Capacity*

Capacity reflects the degree to which a bypassing system will be capable of meeting the design bypass volume requirements. It also accounts for whether the design capacity of the system is at the upper limit of the specific type of system or well within its range of capabilities.

Floating plant alternatives are considered "Highly Beneficial" in terms of capacity. Semi-fixed systems are considered "Neutral" and Truck-mounted pump systems are considered "Severely Adverse". If either of the two latter systems is complemented with a floating plant, the system is considered "Beneficial" or "Neutral", respectively.

Scoring for this criterion is particularly sensitive at Fire Island Inlet where depending on the bypass scheme, dredging required volume within the available environmental window might be difficult or impossible, particularly if having to bypass farther downdrift.

15. *Source Flexibility*

Source flexibility is defined as the ability to remove sand from a relatively large source area without being limited to predefined boundaries imposed by operational and/or technical constraints. This criterion also addresses the ability of land based systems to operate in nearshore areas and areas adjacent to existing structures such as jetties, etc.

Existing deposition basin configurations at Moriches and Fire Island Inlets are considered "Beneficial" because there appears to always be sufficient sediment with the deposition basin to meet design bypass rates. At Shinnecock, however, the deposition basin does not appear to provide sufficient material and it may be considered "Neutral" or "Adverse" depending on the configuration. Non-floating systems are considered "Adverse" because they can only access a limited amount of sediment within their reach on the updrift fillet. Alternatives that include flood shoal and offshore borrow areas are also considered "Adverse" because of the limited resources within those areas. Finally, alternatives that include dredging of the ebb shoal outside the limits of the deposition basin with a floating plant are considered to be the most flexible (i.e., "Highly Beneficial").

16. *Placement Flexibility*

Placement flexibility is defined as the ease with which the bypassing equipment can reach various locations such as the nearshore and onshore areas immediately downdrift of the inlet, or areas farther downdrift. It also reflects the ability to cover a relatively long area.

Floating plant options are considered to be "Beneficial" in terms of placement flexibility, non-floating systems and updrift sand trapping alternatives are considered to be "Adverse" or "Neutral" depending on whether or not their operation is supplemented by a floating plant.

17. *Continuity*

This criterion reflects the ability of the bypassing system to provide continuous bypassing in a manner similar to natural longshore drift. It also accounts for the flexibility of the system with regards to changing conditions, either regulatory (e.g., environmental windows) or physical (e.g., extended periods of reduced or increased longshore transport), while still meeting the design bypass requirements.

Alternatives that include a relatively large percentage of natural bypassing are considered "Highly Beneficial" in terms of longshore drift continuity. Non-floating systems and updrift sand trapping systems are considered "Beneficial". Floating plant dredging on a short cycle (e.g., 1 year) is considered "Neutral," whereas medium (2 years) and long (4 years or more) cycles are considered "Adverse" and "Severely Adverse," respectively.

18. *Performance*

The performance criterion reflects the expected equipment performance based on past experience, expected mechanical reliability, ease of operation, and system availability.

Floating plants are considered highly reliable and available. Their performance, however, might be limited by wave action impacts, particularly if the window of operation is short relative to the volume of sediment to be dredged. Therefore, alternatives that include dredging of the channel, deposition basin, and/or ebb shoal in Moriches and Shinnecock inlets are considered "Highly Beneficial," but the same at Fire Island are considered "Beneficial" or "Neutral" depending on the volume of material and exposure to waves.

The use of semi-fixed systems has been limited to few locations, albeit with some success in recent applications such as Indian River Inlet in Delaware. Therefore this system is considered "Adverse" in terms of performance. Truck-mounted systems are not a proven technology in this type of application and thus they are considered "Severely Adverse", unless they are supplemented by floating plant dredging, which would make it "Adverse".

19. *Reversibility*

This criterion considers to degree to which an inlet modification alternative relies on significant modifications to existing practice that might be difficult to change or reverse if unsuccessful. It also rewards the ability to implement a full range of alternatives at the end of the project life. Therefore, non-structural alternatives and alternatives based on the restoration of existing natural sediment pathways as opposed to alternatives that dictate a future course of action that will be difficult to change are considered "Highly Beneficial". These include, for example, reducing the navigation depth at Moriches Inlet or modifying the deposition basin at Shinnecock Inlet so as to increase natural bypassing. Other alternatives involving various channel and deposition basin configurations are considered "Beneficial" because they would be easy to change.

On the other hand, alternatives that include substantial modifications to exiting facilities and structures, or construction of large new structures, are considered "Severely Adverse". For example updrift collection and bypass systems, which require a number of new structures, or relocation of the inlet, which obviously would be hard to reverse. Non-floating bypass systems are considered "Adverse" because of the initial investment required.

3.6.5 Cultural and Social Criteria

20. <u>*Historic, Cultural, and Scenic Resources*</u>

This criterion reflects impacts on existing historic, archaeological, cultural, and coastal scenic resources. As such, this criterion is consistent with CMP policies 23 to 25, *Historic and Scenic Resources Policies*.

There are no known archeological sites in the immediate vicinity of the three inlets. Although additional investigations may be required for alternatives that include significant dredging in areas that could contain archeological or cultural resources, such as areas of the ebb shoal that have not been previously investigated or dredged, or the area in the vicinity of the Fire Island Lighthouse where the FI Inlet could be relocated. Remote sensing of the borrow area for the West of Shinnecock project has identified two targets that should be avoided during dredging operations (USACE, 1999).

More importantly, the Shinnecock Inlet Maritime center is considered by NY State to be a significant cultural and maritime resource, and in general existing cultural and scenic resources such us commercial fishing, recreational activities, and parks should be maintained at all three inlets. Alternatives that require relocation of this center were considered "Severely Adverse." Alternatives that provide for increased protection of the center as compared to others were considered "Highly Beneficial" (e.g., nearshore structures). Other alternatives were rated in between depending on the level of protection that they provide.

In addition, alternatives that impact existing barrier parks or scenic resources were considered "Severely Adverse" or "Adverse", depending on the relative impact level. All other alternatives were considered "Neutral" with regards to this criterion.

Note that although the Fire Island Light Station, situated approximately five miles east of the western end of Fire Island, is included in the National Register of Historic Resources, none of the alternatives are expected to have an impact on this resource. Inlet relocation in particular would be designed so as to avoid any impacts.

21. <u>*Local Concerns and Public Relations*</u>

This criterion reflects the degree to which an alternative takes into account specific local concerns (if any), or whether the alternative may have an effect on a specific local issue not addressed by any other criteria above. It also considers the potential impacts on the relation between project sponsors and the general public.

In general alternatives that include significant modifications (i.e., inlet relocation, Shinnecock Inlet Marine Center relocation, elimination of the "Shore Thumb") are expected to generate local concerns and opposition (i.e., "Severely Adverse"). In addition, alternatives that do not clearly address some of the local concerns (e.g., navigation at Moriches Inlet, wave impacts at Oak Beach, local shoreline stabilization at West of Shinnecock) are not expected to generate local public support even if they provide for significant bypass increases and as such are considered "Adverse". Finally, some alternatives address these local concerns with a more direct approach that might be more apparent to the public and thus might generate support (e.g., stabilization of the West Beach at Shinnecock Inlet and alternatives that improve navigation conditions at Moriches Inlet).

Most other alternatives are considered relatively “Neutral”. For example, an updrift non-floating bypassing system might generate local support as very visible means to achieve improved bypassing. On the other hand, other locals might oppose it for aesthetic and recreational reasons. Similar pro and con arguments could be made with regards to dredging the flood shoals.

Table 3-7: Correlation between Screening Criteria & CMP Policies

	Environmental Criteria	1. Fish and Wildlife	2. Rare and Endangered Species	3. Water Quality	4. Tidal and Freshwater Wetlands	5. Sediment Pathways	6. Non-structural Measures	**Economic Criteria**	7. Lifecycle Costs	8. Flooding Risk	9. Commercial Fisheries	10. Waterfront Development	11. Land Use and Ownership	**Recreational Criteria**	12. Fish & Wildlife Resources	13. Water & Foreshore Resources	**Engineering Criteria**	14. Capacity	15. Source Flexibility	16. Placement Flexibility	17. Operation Schedule Flexibility	18. Performance	**Cultural & Social Criteria**	19. Historic and Scenic Resources	20. Local Concerns & Public Relations
Development Policies																									
1: Waterfront Revitalization												■													
2: Water-Dependent Uses												■													
3: Major Ports																									
4: Small Harbors												■													
5: Public Services												■													
6: Permit Procedures																									
Fish & Wildlife Policies																									
7: Significant Habitats		■	■	■																					
8: Pollutants		■	■	■																					
9: Recreational Resources		■													■										
10: Commercial Fisheries		■									■														
Flooding and Erosion Hazards Policies																									
11: Siting Structures																									
12: Natural Protective Features						■																			
13: 30-Year Erosion Control Structures																									
14: No Flooding or Erosion Increases										■															
15: Natural Coastal Processes						■																			
16: Use of Public Funds									■																
17: Non-structural Control Measures							■																		
General Policies																									
18: Safeguard the Vital Interests									■																
Public Access Policies																									
19: Water-Related Recreation Resources																■									
20: Public Foreshore																■									
Recreation Policies																									
21: Water-Dependent/Enhanced Recreation																■									
22: Multiple-Use Development																■									
Historic and Scenic Resources Policies																									
23: Historic Preservation																								■	
24: Statewide Scenic Resources																								■	
25: Local Scenic Resources																								■	■
Agricultural Lands Policies																									
26: Conserve & Protect Agricultural Lands																									
Energy and Ice Management Policies																									
27: Energy Facility Siting and Construction																									
28: Ice Management Practices																									
29: Energy Resources Development																									
Water and Air Resources Policies																									
30: State & National Water Quality Standards				■																					
31: LWRP Policies and Constraints				■																					
32: Innovative Sanitary Waste Systems				■																					
33: Stormwater Runoff/ Combined Sewers				■																					
34: Vessel Discharges				■																					
35: Dredging and Disposal				■																					
36: Hazardous Material Spills				■																					
37: Non-point Pollution Discharges				■																					
38: Surface & Ground Industrial Discharges				■																					
39: Solid Waste Management																									
40: Industrial Discharges																									
41: State & National Air Quality Standards																									
42: Clean Air Act – Reclassifications																									
43: Acid Rain																									
44: Tidal and Freshwater Wetlands					■																				

3.7 Screening Matrices

The foregoing discussion provides a description of the selected criteria as well as a general description of how the alternatives were scored. Scores and screening results are shown in Tables 3-8, 3-9 and 3-10. Note that criteria scores to date reflect the A/E's recent findings with regards to coastal processes at the inlets (e.g., ebb shoal growth) and the sediment budgets. The resulting ranking may be somewhat different under different assumptions.

Finally, note that although the resulting ranking depends on a relatively subjective assessment (as is always the case in this type of analysis), the exercise of developing criteria and assigning scores does bring to focus each alterative and the associated pros and cons. More importantly, the results of this screening will only be used to identify alternatives that should be eliminated from further consideration, and also to identify the top alternatives that should be carried forward for more detailed investigations. The screening will not be used to select only the top ranked alternative at each inlet.

3.8 Alternatives Selected for Detailed Design and Analysis

Based on the initial screening, the following alternatives were selected for detailed design and analysis for each inlet and renumbered accordingly. Each of these alternatives would achieve the goals of providing reliable navigation through the Federal navigation channel, restoring natural sediment pathways, and reducing adjacent shoreline erosion, albeit to varying degrees and at different costs. The performance and pros and cons of each alternative will be assessed in Section 8.

3.8.1 Shinnecock Inlet

Alt. 1: Authorized Project (AP) + Dredging the Ebb Shoal
Alt. 2: AP + Nearshore Structures
Alt. 3: AP + Offshore Dredging
Alt. 4: AP + Semi-fixed Bypass System
Alt. 5: AP with Reduced Dimensions of Deposition Basin
Alt. 6: AP + Dredging the Flood Shoal
Alt. 7: AP + Shortening the East Jetty
Alt. 8: AP + West Jetty Spur

3.8.2 Moriches Inlet

Alt. 1: Authorized Project (AP)
Alt. 2: AP + Dredging the Ebb Shoal
Alt. 3: AP + Semi-fixed Bypass System
Alt. 4: AP + Dredging the Flood Shoal

3.8.3 Fire Island Inlet

Alt. 1: Existing Practice/ Authorized Project (AP)
Alt. 2: AP + Dredging the Ebb Shoal
Alt. 3: AP + Optimized Deposition Basin
Alt. 4: AP + Dredging the Flood Shoal

Table 3-8 Shinnecock Inlet - Initial Screeening

Shinnecock Inlet Screening

Scores are from 1 to 5. Higher scores are better! The FINAL SCORE is computed as the "Performance Score" x "Total Criteria Score"

			Project Needs						Screening Criteria: Environmental							Economic						Recreational			Engineering							Cultural & Social		
	FINAL SCORE (Max 1,000)	**RANKING (out of 15)**	Safe navigation	Mitigate Local Erosion (West Beach)	Mitigate Erosion Farther Downdrift (Regional)	***RISK & UNCERTAINTY (%)***	**Performance Score (Max 10)**	**Total Criteria Score (Max 100)**	1. Fish and Wildlife	2. Rare and Endangered Species	3. Water Quality	4. Tidal and Freshwater Wetlands	5. Sediment Pathways	6. Non-Structual Components	**Environmental Score (Max. 20)**	7. Lifecycle Costs	8. Flooding Risk	9. Commercial Fisheries	10. Waterfront & Commercial Fishing Facilities	11. Land Use and Ownership	**Economic Score (Max. 20)**	12. Recreational Fish and Wildlife Resources	13. Water & Foreshore Related Recreation Res	**Recreational Score (Max. 20)**	14. Capacity	15. Source Flexibility	16. Placement Flexibility	17. Continuity	18. Performance	19. Reversibility	**Engineering Score (Max. 20)**	20. Historic and Scenic Resources	21. Local Concerns and Public Relations	**Cultural & Social Score (Max. 20)**
1 Existing Practice plus Offshore Dredging (for West Beach)	**429**	**3**	10	7	7	20	**6.4**	**67**	2	4	3	3	1	4	**11**	3	5	2	4	4	**14**	3	3	**12**	5	2	4	1	5	3	**13**	4	4	**16**
2 Existing Practice plus Dredging the Flood Shoal	**342**	**8**	10	6	7	30	**5.4**	**64**	1	2	5	1	2	4	**10**	5	3	2	4	4	**14**	3	3	**12**	5	2	4	1	5	3	**13**	4	3	**14**
3 Channel & Deposition Basin Realignment	**290**	**15**	8	5	8	40	**4.2**	**69**	3	3	2	3	3	4	**12**	4	4	3	3	4	**14**	2	4	**12**	5	2	4	2	5	4	**15**	4	4	**16**
4 Relocation of the Deposition Basin (Not Channel)	**358**	**6**	8	6	8	30	**5.1**	**70**	3	3	2	3	3	4	**12**	4	4	3	3	4	**14**	2	4	**12**	5	3	4	2	5	4	**15**	4	4	**16**
5 Reduced Dimensions of Deposition Basin	**378**	**5**	8	7	8	30	**5.4**	**70**	4	3	2	3	3	4	**13**	4	4	3	3	4	**14**	2	4	**12**	5	3	4	2	5	4	**15**	4	4	**16**
6 Existing Practice plus Dredging the Ebb Shoal	**512**	**1**	10	8	9	20	**7.2**	**71**	2	4	3	3	2	4	**12**	4	4	3	3	4	**14**	2	4	**12**	5	5	4	2	5	4	**17**	4	4	**16**
7 Semi-fixed Bypass System (plus "reduced" Existing Practice)	**385**	**4**	9	8	9	20	**6.9**	**55**	2	2	3	2	3	4	**11**	3	4	3	4	2	**13**	3	2	**10**	4	2	3	4	3	2	**12**	2	3	**10**
8 Truck/Trailer Mounted System (plus "reduced" Existing Practice)	**328**	**11**	9	8	9	30	**6.1**	**54**	2	2	3	2	3	4	**11**	3	4	3	4	2	**13**	3	2	**10**	3	2	3	4	2	2	**11**	2	3	**10**
9 Existing Practice plus Spur Jetty (West)	**306**	**13**	10	7	5	30	**5.1**	**60**	3	3	4	3	2	3	**12**	4	3	3	3	4	**14**	3	2	**10**	5	2	4	1	5	1	**12**	3	3	**12**
10 Existing Practice plus Shortening the East Jetty	**333**	**9**	8	7	8	20	**6.1**	**54**	2	2	2	2	3	5	**11**	4	4	3	3	3	**14**	2	2	**8**	5	3	4	3	5	1	**14**	2	2	**8**
11 Change Distance between Inlet Jetties	**189**	**17**	8	6	7	50	**3.5**	**54**	3	3	2	2	3	4	**11**	1	4	3	3	3	**11**	3	2	**10**	5	3	4	2	5	1	**13**	3	1	**8**
12 Existing Practice plus Nearshore Structures along West Beach	**440**	**2**	10	10	8	10	**8.4**	**52**	2	1	3	2	2	2	**8**	2	5	3	5	3	**14**	3	2	**10**	5	2	4	1	5	1	**12**	2	2	**8**
13 Sand Trapping & Bypassing System Updrift (plus Existing Practice)																																		
A. Jetty Opening and Nearshore Breakwater	**253**	**16**	10	9	9	40	**5.6**	**45**	1	1	3	1	3	2	**7.3**	1	4	3	4	2	**11**	3	1	**8**	5	4	3	4	2	1	**13**	1	2	**6**
B. Weir Jetty and Sediment Trap	**301**	**14**	10	9	9	30	**6.5**	**46**	1	1	3	1	3	2	**7.3**	2	4	3	4	2	**12**	3	1	**8**	5	3	3	4	3	1	**13**	1	2	**6**
C. Offshore Breakwater	**332**	**10**	10	8	8	20	**6.9**	**48**	2	2	3	2	3	2	**9.3**	1	4	3	4	2	**11**	3	1	**8**	4	4	3	4	4	1	**13**	1	2	**6**
14 Existing Practice plus Dredging the Ponquogue Attachment	**346**	**7**	10	8	8	30	**6.1**	**57**	2	1	5	2	2	5	**11**	4	4	3	3	4	**14**	3	2	**10**	4	4	4	1	3	4	**13**	2	2	**8**
15 Existing Practice plus Relocation of the Maritime Center	**323**	**12**	10	8	8	10	**7.8**	**41**	3	5	3	3	2	3	**13**	1	1	2	1	1	**4.8**	3	1	**8**	5	2	4	1	5	1	**12**	1	1	**4**

Other Alternatives Eliminated due to Fatal Flaws	***Fatal Flaw (s)***
Existing Practice	Continued Local erosion at West Beach
Do Nothing	Impacts to navigation and Shinnecock Inlet Maritime Center
Ebb Shoal Nourishment	Uncertainty with regards to required volumes. Lack of sand resources
Other Non-Floating Bypassing Systems	Cost and performance
Backpassing	No need
Realignment of Complete Inlet System	Risk and uncertainty, other alternatives available
Removal of the Jetties	Navigation, shoreline impacts, breaching, flooding risk, impacts to Shinnecock Inlet Maritime Center
Inlet Closure	Navigation, WQ, increased breaching and flooding risk, impacts to Shinnecock Inlet Maritime Center
Move the Inlet to a new Location	No benefit. Other alternatives available
Relocation of the Maritime Center to Smith Point	Impacts on existing fleet. Navigation. Deeper inner channel requirements
Reduce Authorized Channel Depth	Impacts to navigation and Shinnecock Inlet Maritime Center
Closing the Inlet to Navigation	Impacts to navigation and Shinnecock Inlet Maritime Center

Shinnecock Inlet Screening

	Alternative Plan Description	FINAL SCORE (Max 1,000)	RANKING (out of 17)
6	Existing Practice plus Dredging the Ebb Shoal	512	1
12	Existing Practice plus Nearshore Structures along West Beach	440	2
1	Existing Practice plus Offshore Dredging (for West Beach)	429	3
7	Semi-fixed Bypass System (plus "reduced" Existing Practice)	385	4
5	Reduced Dimensions of Deposition Basin	378	5
4	Relocation of the Deposition Basin (Not Channel)	358	6
14	Existing Practice plus Dredging the Ponquogue Attachment	346	7
2	Existing Practice plus Dredging the Flood Shoal	342	8
10	Existing Practice plus Shortening the East Jetty	333	9
13	C. Offshore Breakwater	332	10
8	Truck/Trailer Mounted System (plus "reduced" Existing Practice)	328	11
15	Existing Practice plus Relocation of the Maritime Center	323	12
9	Existing Practice plus Spur Jetty (West)	306	13
13	B. Weir Jetty and Sediment Trap	301	14
3	Channel & Deposition Basin Realignment	290	15
13	A. Jetty Opening and Nearshore Breakwater	253	16
11	Change Distance between Inlet Jetties	189	17

Table 3-9 Moriches Inlet - Initial Screeening

Moriches Inlet Screening

Scores are from 1 to 5. Higher scores are better! The FINAL SCORE is computed as the "Performance Score" x "Total Criteria Score"

			Project Needs						Screening Criteria: Environmental							Economic						Recreational			Engineering							Cultural & Social		
	FINAL SCORE (Max 1,000)	RANKING (out of 13)	Safe navigation	Mitigate Local Erosion (West Beach)	Mitigate Erosion Farther Downdrift (Regional)	RISK & UNCERTAINTY (%)	Performance Score (Max 10)	Total Criteria Score (Max 100)	1. Fish and Wildlife	2. Rare and Endangered Species	3. Water Quality	4. Tidal and Freshwater Wetlands	5. Sediment Pathways	6. Non-Structual Components	Environmental Score (Max. 20)	7. Lifecycle Costs	8. Flooding Risk	9. Commercial Fisheries	10. Waterfront & Commercial Fishing Facilities	11. Land Use and Ownership	Economic Score (Max. 20)	12. Recreational Fish and Wildlife Resources	13. Water & Foreshore Related Recreation Res	Recreational Score (Max. 20)	14. Capacity	15. Source Flexibility	16. Placement Flexibility	17. Continuity	18. Performance	19. Reversibility	Engineering Score (Max. 20)	20. Historic and Scenic Resources	21. Local Concerns and Public Relations	Cultural & Social Score (Max. 20)
1 Authorized Project	384	10	8	8	8	30	5.6	69	3	3	3	3	3	5	13	4	4	3	4	4	15	3	3	12	5	3	4	3	5	4	16	3	3	12
2 Authorized Project plus Dredging the Flood Shoal	401	6	8	9	10	30	6.3	64	1	2	5	1	2	5	11	5	2	2	4	4	14	3	3	12	5	3	4	3	5	3	15	3	3	12
3 Channel & Deposition Basin Realignment	404	5	7	9	9	30	5.8	69	3	3	2	3	3	5	13	4	4	3	4	4	15	3	4	14	5	2	4	3	5	4	15	3	3	12
4 Relocation of the Deposition Basin (Not Channel)	408	4	8	8	9	30	5.8	70	3	3	2	3	3	5	13	4	4	3	4	4	15	3	4	14	5	3	4	3	5	4	16	3	3	12
5 Increased Dimensions of Deposition Basin	408	3	10	7	7	30	5.6	73	4	3	2	3	3	5	13	4	2	3	4	4	14	4	4	16	5	4	4	2	5	4	16	3	4	14
6 Authorized Project plus Dredging the Ebb Shoal	532	1	9	9	10	20	7.5	71	2	4	3	3	2	5	13	4	4	3	4	4	15	3	4	14	5	5	4	3	5	4	17	3	3	12
7 Semi-fixed Bypass System (plus Authorized Practice)	449	2	9	9	10	20	7.5	60	2	2	3	2	3	5	11	3	4	3	4	2	13	4	2	12	4	2	3	4	3	2	12	2	4	12
8 Truck/Trailer Mounted System (plus Authorized Practice)	384	9	9	9	10	30	6.5	59	2	2	3	2	3	5	11	3	4	3	4	2	13	4	2	12	3	2	3	4	2	2	11	2	4	12
9 Authorized Project plus Extension of the West Jetty	274	13	9	8	5	40	4.4	62	3	3	4	3	2	2	11	3	3	3	4	4	14	4	2	12	5	3	4	2	5	1	13	3	3	12
10 Sand Trapping and Bypassing System Updrift (plus Authorized Project)																																		
A. Jetty Opening and Nearshore Breakwater	285	12	10	9	10	40	5.8	49	1	1	3	1	3	2	7.3	1	4	3	4	2	11	4	1	10	5	4	3	4	2	1	13	1	3	8
B. Weir Jetty and Sediment Trap	338	11	10	9	10	30	6.8	50	1	1	3	1	3	2	7.3	2	4	3	4	2	12	4	1	10	5	3	3	4	3	1	13	1	3	8
C. Offshore Breakwater	387	8	10	9	9	20	7.5	52	2	2	3	2	3	2	9.3	1	4	3	4	2	11	4	1	10	4	4	3	4	4	1	13	1	3	8
11 Reduced Authorized Channel Depth	399	7	5	9	9	10	6.9	58	3	5	3	3	2	5	14	2	4	3	1	4	11	2	1	6	5	4	4	5	5	5	19	3	1	8

Other Alternatives Eliminated due to Fatal Flaws	*Fatal Flaw (s)*
Existing Practice	Lack of Navigation Reliability
Do Nothing	Lack of Navigation Reliability
Ebb Shoal Nourishment	Uncertainty with regards to required volumes. Lack of sand resources. No apparent need at Moriches
Other Non-Floating Bypassing Systems	Cost and performance
Backpassing	No need
Realignment of Complete Inlet System	Risk and uncertainty, other alternatives available
Removal of the Jetties	Navigation, shoreline impacts, breaching, flooding risk, impacts to existing marine centers
Inlet Closure	Navigation, WQ, increased breaching and flooding risk, impacts to existing marine centers
Move the Inlet to a new Location	No benefit. Other alternatives available
Closing the Inlet to Navigation	Impacts to navigation and Shinnecock Inlet Maritime Center

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Moriches Inlet Screening

	Scores are from 1 to 5. Higher scores are better! The FINAL SCORE is computed as the "Performance Score" x "Total Criteria Score"	FINAL SCORE (Max 1,000)	RANKING (out of 13)
6	Authorized Project plus Dredging the Ebb Shoal	532	1
7	Semi-fixed Bypass System (plus Authorized Practice)	449	2
5	Increased Dimensions of Deposition Basin	408	3
4	Relocation of the Deposition Basin (Not Channel)	408	4
3	Channel & Deposition Basin Realignment	404	5
2	Authorized Project plus Dredging the Flood Shoal	401	6
11	Reduced Authorized Channel Depth	399	7
10	C. Offshore Breakwater	387	8
8	Truck/Trailer Mounted System (plus Authorized Practice)	384	9
1	Authorized Project	384	10
10	B. Weir Jetty and Sediment Trap	338	11
10	A. Jetty Opening and Nearshore Breakwater	285	12
9	Authorized Project plus Extension of the West Jetty	274	13

Table 3-10 Fire Island Inlet - Initial Screeening

Fire Inlet Screening

Scores are from 1 to 5. Higher scores are better! The FINAL SCORE is computed as the "Performance Score" x "Total Criteria Score"

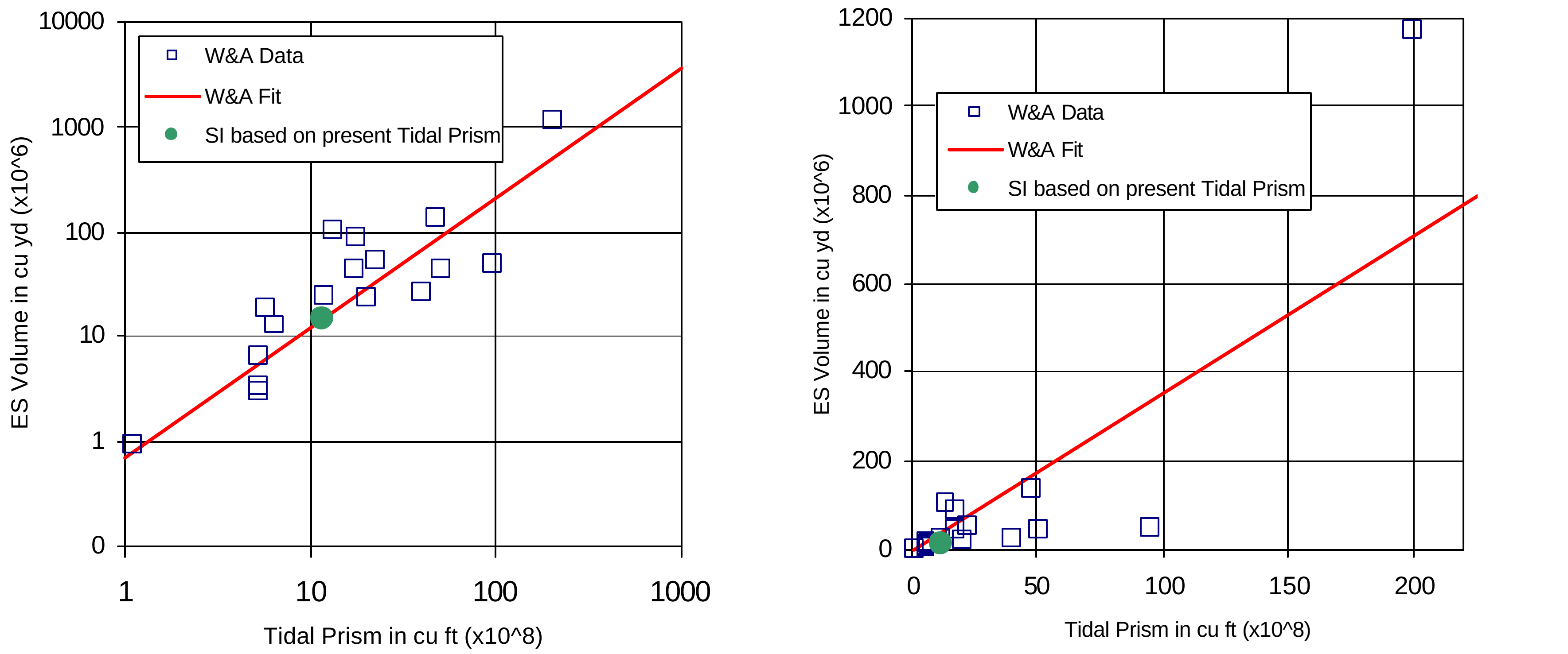
			Project Needs						*Screening Criteria*																									
									Environmental							*Economic*						*Recreational*			*Engineering*							*Cultural & Social*		
Alternative	**FINAL SCORE (Max 1,000)**	**RANKING (out of 14)**	Safe navigation	Mitigate Local Erosion (West Beach)	Mitigate Erosion Farther Downdrift (Regional)	***RISK & UNCERTAINTY (%)***	**Performance Score (Max 10)**	**Total Criteria Score (Max 100)**	1. Fish and Wildlife	2. Rare and Endangered Species	3. Water Quality	4. Tidal and Freshwater Wetlands	5. Sediment Pathways	6. Non-Structural Components	**Environmental Score (Max. 20)**	7. Lifecycle Costs	8. Flooding Risk	9. Commercial Fisheries	10. Waterfront & Commercial Fishing Facilities	11. Land Use and Ownership	**Economic Score (Max. 20)**	12. Recreational Fish and Wildlife Resources	13. Water & Foreshore Related Recreation Res	**Recreational Score (Max. 20)**	14. Capacity	15. Source Flexibility	16. Placement Flexibility	17. Continuity	18. Performance	19. Reversibility	**Engineering Score (Max. 20)**	20. Historic and Scenic Resources	21. Local Concerns and Public Relations	**Cultural & Social Score (Max. 20)**
1 Existing Practice	**413**	**4**	6	8	7	10	**6.3**	**66**	3	3	3	3	3	4	**13**	3	4	3	3	4	**14**	3	3	**12**	4	4	4	2	5	4	**15**	3	3	**12**
2 Existing Practice plus Discharge Farther West	**397**	**5**	6	8	8	20	**5.9**	**68**	3	3	3	3	4	4	**13**	3	4	3	3	4	**14**	3	4	**14**	3	4	4	2	5	4	**15**	3	3	**12**
3 Optimize Existing Channel & Deposition Basin Configurations	**419**	**3**	9	8	8	30	**5.8**	**72**	3	3	3	3	4	4	**13**	4	4	3	4	4	**15**	4	4	**16**	4	5	4	1	5	4	**15**	3	3	**12**
4 Eastern Realignment of Channel and Deposition Basin	**429**	**2**	9	8	8	20	**6.7**	**64**	1	3	4	1	4	4	**11**	4	3	3	4	4	**14**	4	4	**16**	4	5	4	1	5	3	**15**	3	1	**8**
5 Existing Practice plus Dredging the Flood Shoal	**347**	**7**	7	8	9	30	**5.6**	**62**	1	2	4	1	2	4	**9.3**	3	3	2	3	4	**12**	3	4	**14**	3	5	4	2	5	3	**15**	3	3	**12**
6 Existing Practice plus Dredging the Ebb Shoal	**483**	**1**	6	10	10	20	**6.9**	**70**	2	4	3	3	4	4	**13**	3	4	3	3	4	**14**	3	4	**14**	3	5	4	2	5	3	**15**	3	4	**14**
7 Semi-fixed Bypass System (plus "reduced" Existing Practice)	**378**	**6**	7	8	8	10	**6.9**	**55**	2	2	3	2	4	4	**11**	3	4	3	4	2	**13**	3	2	**10**	2	2	3	4	3	2	**11**	2	3	**10**
8 Existing Practice plus Extension of the East Jetty	**314**	**9**	6	9	7	30	**5.1**	**61**	2	4	3	3	2	2	**11**	4	4	3	4	4	**15**	3	3	**12**	4	4	4	2	5	1	**13**	3	2	**10**
9 Reconfiguration of the Sore Thumb (and Channel Realignment)	**208**	**14**	7	6	9	40	**4.4**	**47**	1	1	4	1	4	5	**11**	3	2	3	4	2	**11**	3	1	**8**	4	4	4	2	5	1	**13**	1	1	**4**
10 Sand Trapping and Bypassing System Updrift (plus Existing Practice)																																		
A. Jetty Opening and Nearshore Breakwater	**233**	**13**	8	8	9	40	**5.0**	**47**	1	1	3	1	4	2	**8**	1	4	3	4	2	**11**	4	1	**10**	3	4	3	4	2	1	**11**	1	2	**6**
B. Weir Jetty and Sediment Trap	**276**	**11**	8	8	9	30	**5.8**	**47**	1	1	3	1	4	2	**8**	2	4	3	4	2	**12**	4	1	**10**	3	3	3	4	3	1	**11**	1	2	**6**
C. Offshore Breakwater	**328**	**8**	8	8	9	20	**6.7**	**49**	2	2	3	2	4	2	**10**	1	4	3	4	2	**11**	4	1	**10**	2	4	3	4	4	1	**12**	1	2	**6**
11 Groins East of the Inlet (plus Existing Practice)	**301**	**10**	8	10	8	30	**6.1**	**50**	2	3	3	3	1	1	**8.7**	3	4	3	3	4	**14**	3	1	**8**	4	4	4	2	5	1	**13**	2	1	**6**
12 Move the Inlet Back to the Lighthouse	**245**	**12**	10	8	8	40	**5.2**	**47**	3	3	5	3	3	2	**13**	1	1	2	4	1	**7.2**	4	1	**10**	4	4	4	2	5	1	**13**	1	1	**4**

Other Alternatives Eliminated due to Fatal Flaws	***Fatal Flaw (s)***
Do Nothing	Impacts to navigation and increased breaching and flooding risk
Ebb Shoal Nourishment	Uncertainty with regards to required volumes. Lack of sand resources. No apparent need.
Other Non-Floating Bypassing Systems	Cost and performance
Removal of the Jetties	Navigation, shoreline impacts, breaching, flooding risk.
Inlet Closure	Navigation, WQ, increased breaching and flooding risk, impacts to several marine centers
Relocation of the Maritime Center to Smith Point	Impacts on existing fleet. Navigation. Deeper inner channel requirements
Reduce Authorized Channel Depth	Impacts to navigation and marine centers
Closing the Inlet to Navigation	Impacts to navigation and marine centers. Increased breaching/flooding risk if inlet not maintained

Fire Island Screening DRAFT 9-21-03.xls

Fire Inlet Screening

	Scores are from 1 to 5. Higher scores are better! The FINAL SCORE is computed as the "Performance Score" x "Total Criteria Score"	**FINAL SCORE (Max 1,000)**	**RANKING (out of 13)**
6	Existing Practice plus Dredging the Ebb Shoal	483	1
4	Eastern Realignment of Channel and Deposition Basin	429	2
3	Optimize Existing Channel & Deposition Basin Configurations	419	3
1	Existing Practice	413	4
2	Existing Practice plus Discharge Farther West	397	5
7	Semi-fixed Bypass System (plus "reduced" Existing Practice)	378	6
5	Existing Practice plus Dredging the Flood Shoal	347	7
10	C. Offshore Breakwater	328	8
8	Existing Practice plus Extension of the East Jetty	314	9
11	Groins East of the Inlet (plus Existing Practice)	301	10
10	B. Weir Jetty and Sediment Trap	276	11
12	Move the Inlet Back to the Lighthouse	245	12
10	A. Jetty Opening and Nearshore Breakwater	233	13
9	Reconfiguration of the Sore Thumb (and Channel Realignment)	208	14



Tidal Prism-Ebbshoal Storage Ratio for Moderately Exposed Coasts (Walton & Adams, 1976)

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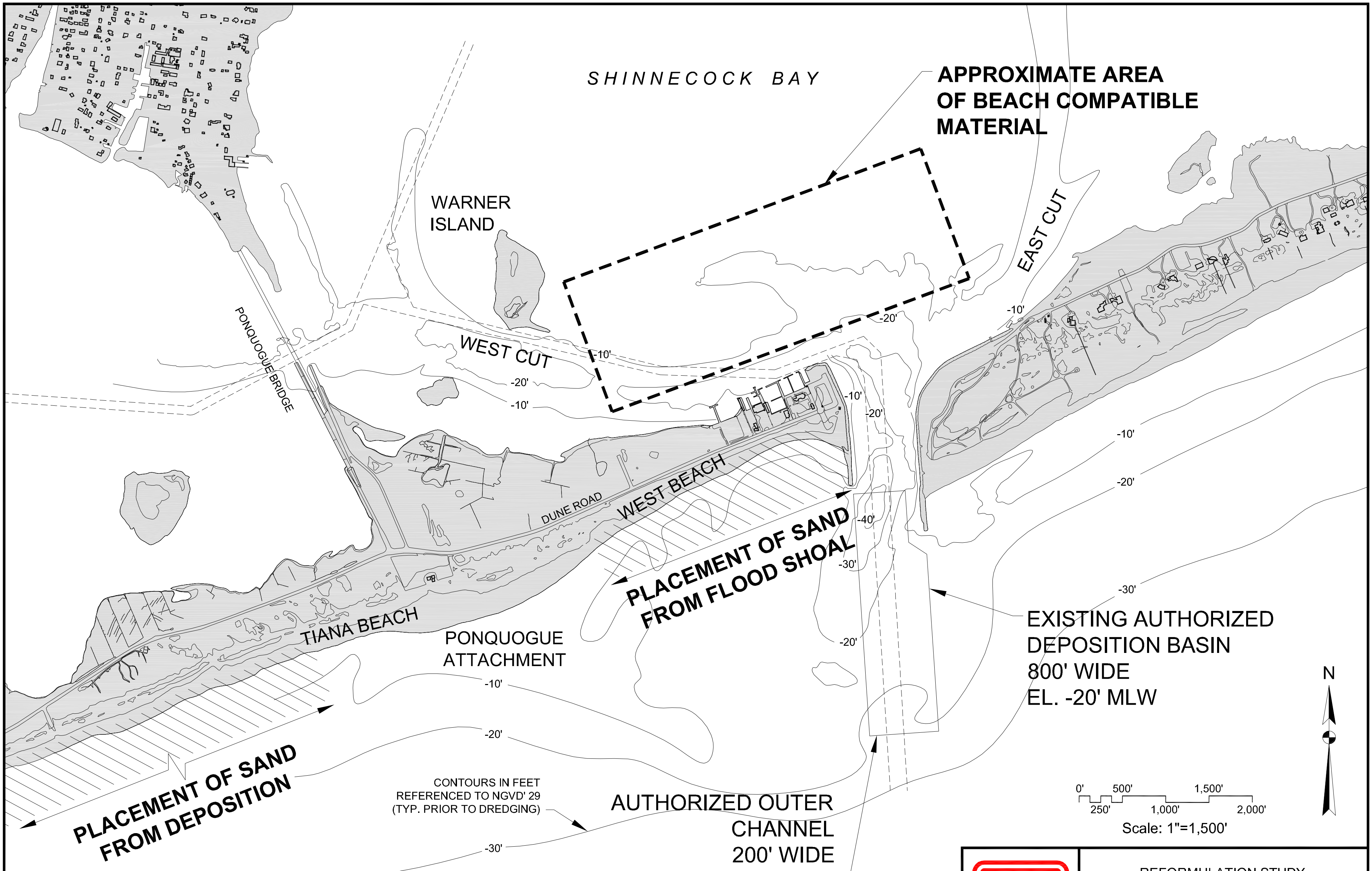
FIGURE 3-1



SHINNECOCK INLET - Alternative 2
EXISTING PRACTICE PLUS DREDGING THE FLOOD SHOAL

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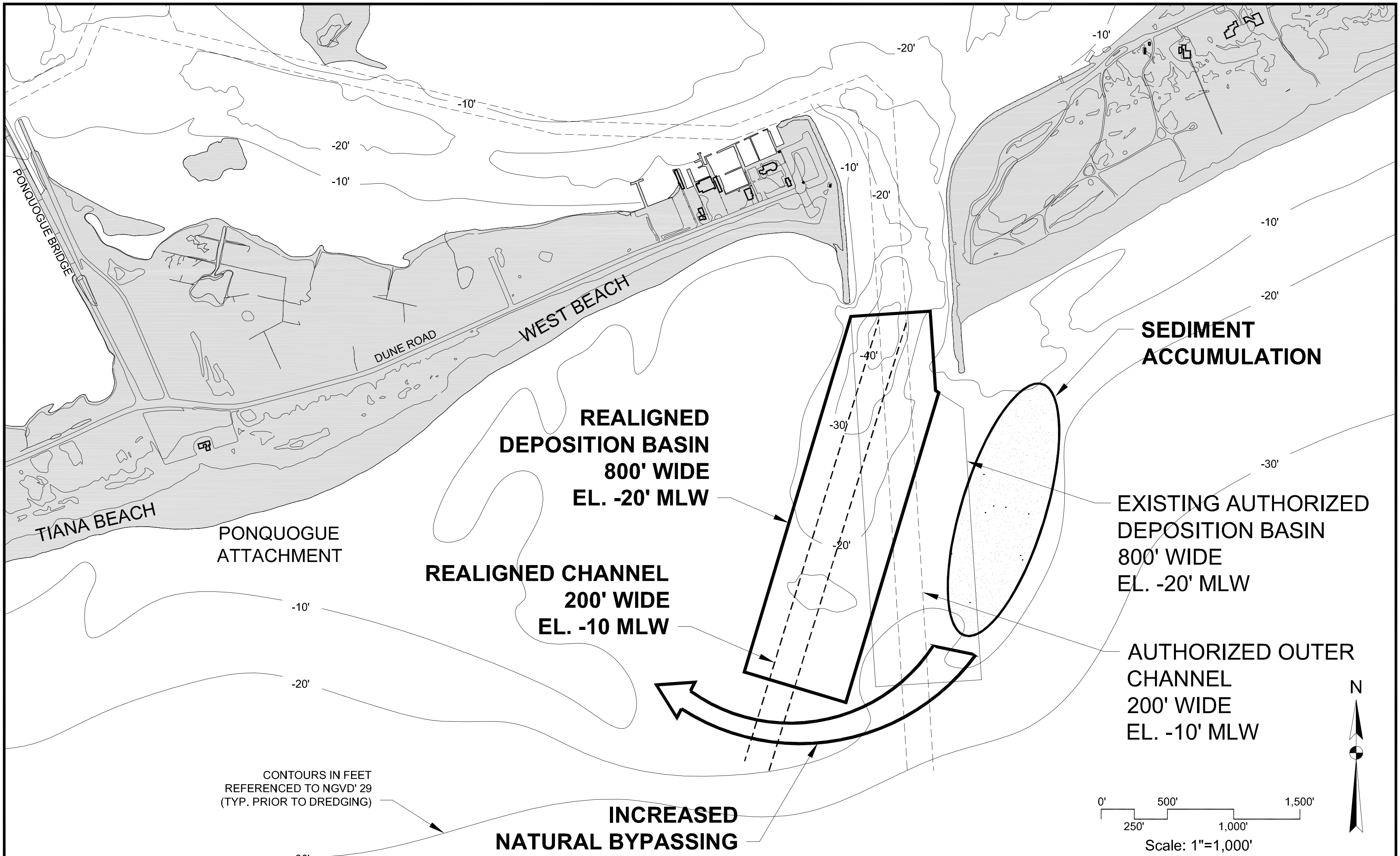
FIGURE 3-2

Filename: P:\3982-28\CAD\dwg\SI ALT No. 2.dwg (Shinnecock)



SHINNECOCK INLET - Alternative 3
CHANNEL & DEPOSITION BASIN REALIGNMENT
ALONG THE "NATURAL" CHANNEL THALWEG

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FIGURE 3-3

Filename: P:\3982-28\CAD\dwg\SI ALT No. 3.dwg (Shinnecock)

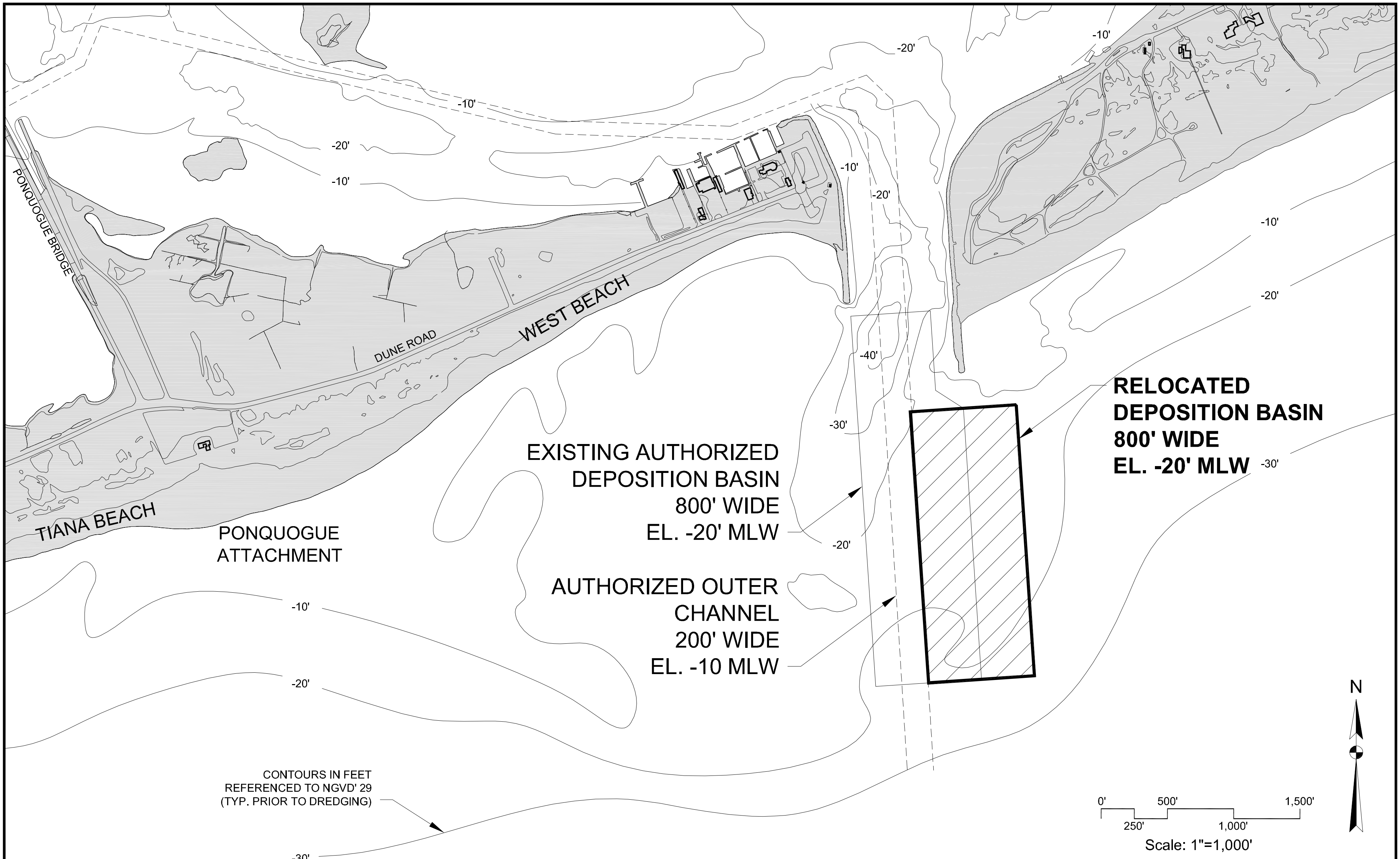

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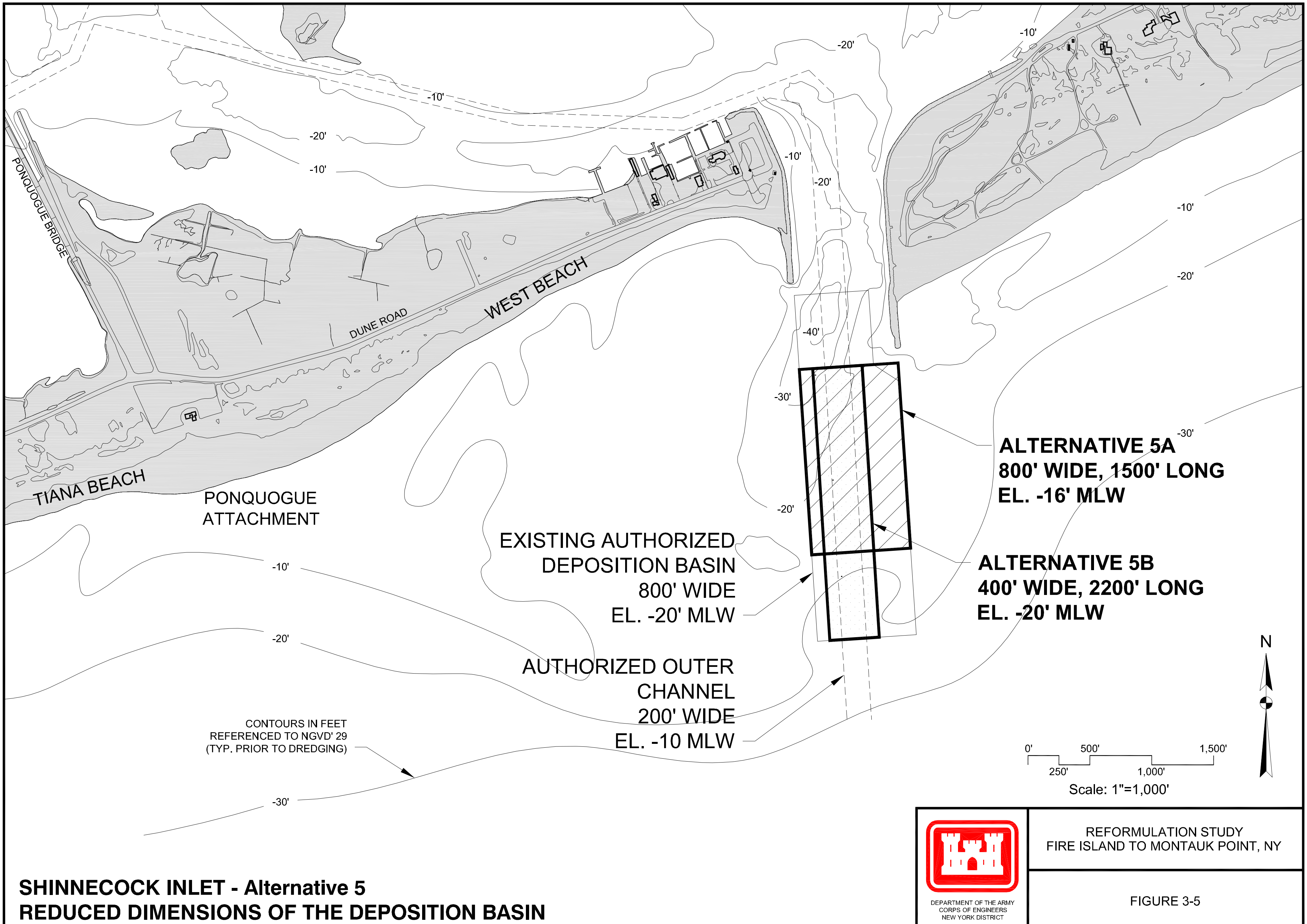
FIGURE 3-4

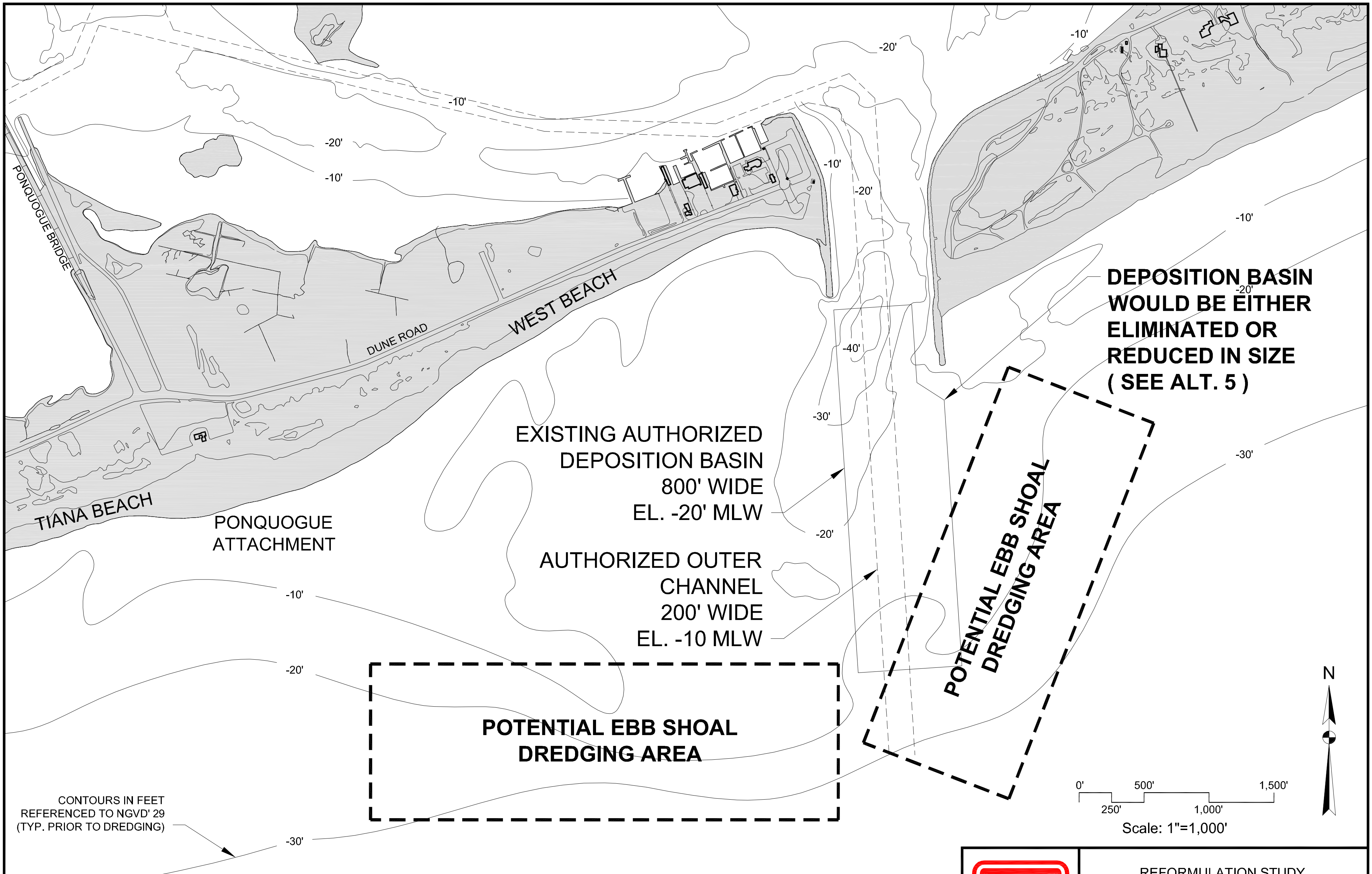
SHINNECOCK INLET - Alternative 4
RELOCATION OF THE DEPOSITION BASIN (NOT CHANNEL)

Filename: P:\3982-28\CAD\dwg\SI ALT No. 4.dwg (Shinnecock)



SHINNECOCK INLET - Alternative 5
REDUCED DIMENSIONS OF THE DEPOSITION BASIN

Filename: P:\3982-28\CAD\dwg\SI ALT No. 5.dwg (Shinnecock)



SHINNECOCK INLET - Alternative 6
DREDGING THE EBB SHOAL

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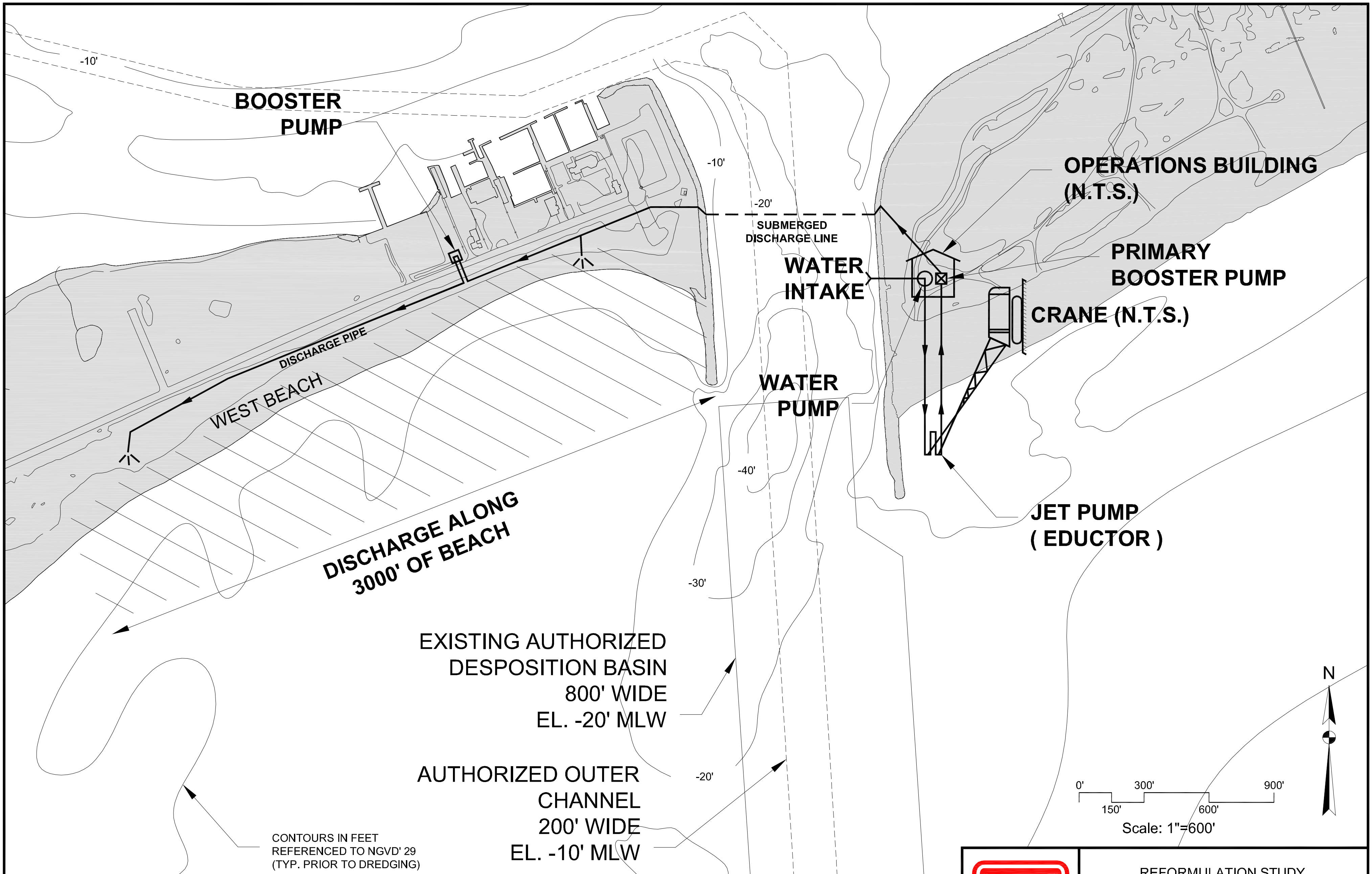
FIGURE 3-6

Filename: P:\3982-28\CAD\dwg\SI ALT No. 6.dwg (Shinnecock)



SHINNECOCK INLET - Alternative 7
SEMI-FIXED BYPASS SYSTEM

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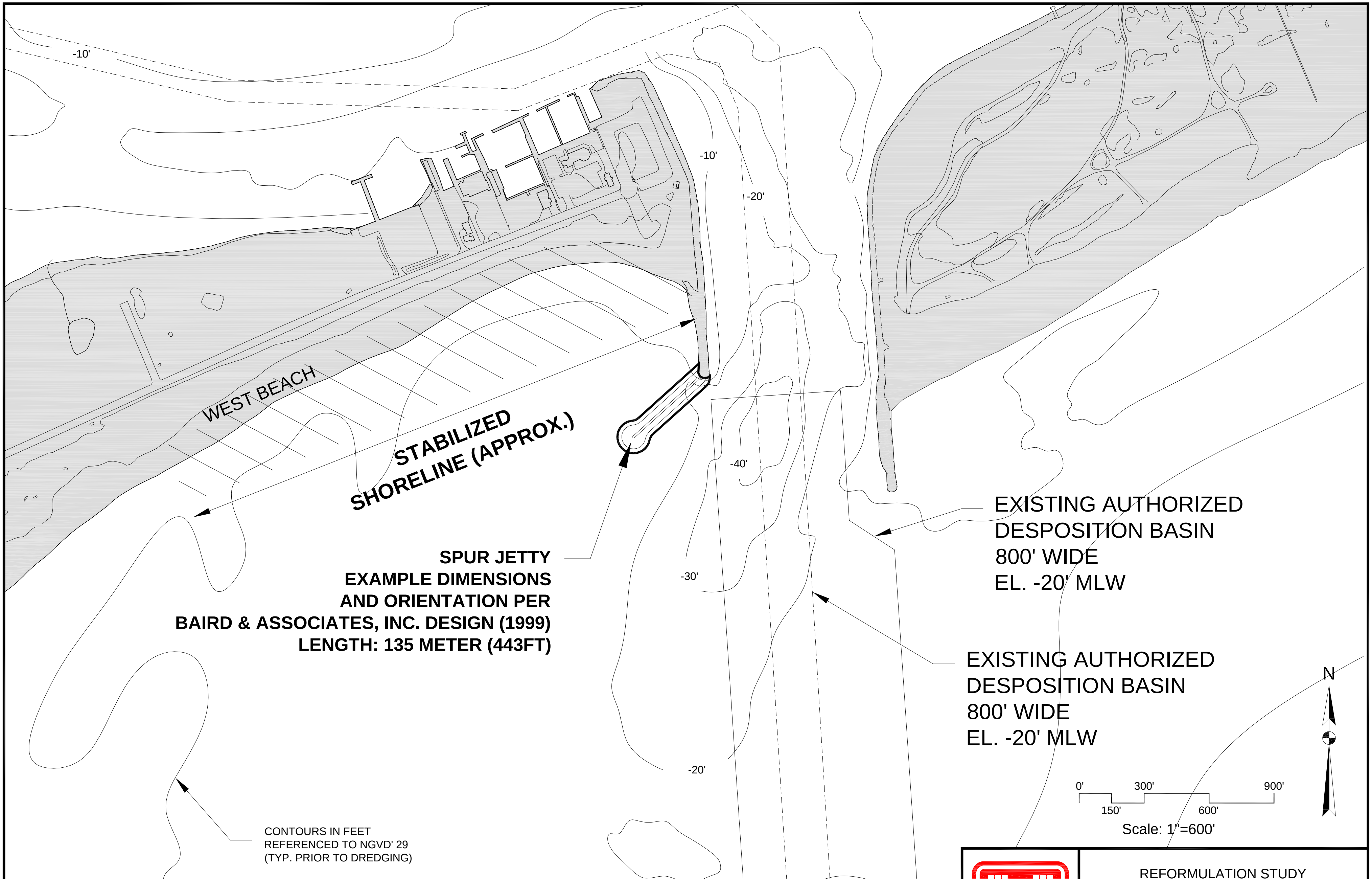
FIGURE 3-7

Filename: P:\3982-28\CAD\dwg\SI ALT No. 7.dwg (Shinnecock)



SHINNECOCK INLET - Alternative 9
EXISTING PRACTICE PLUS SPUR JETTY (WEST)

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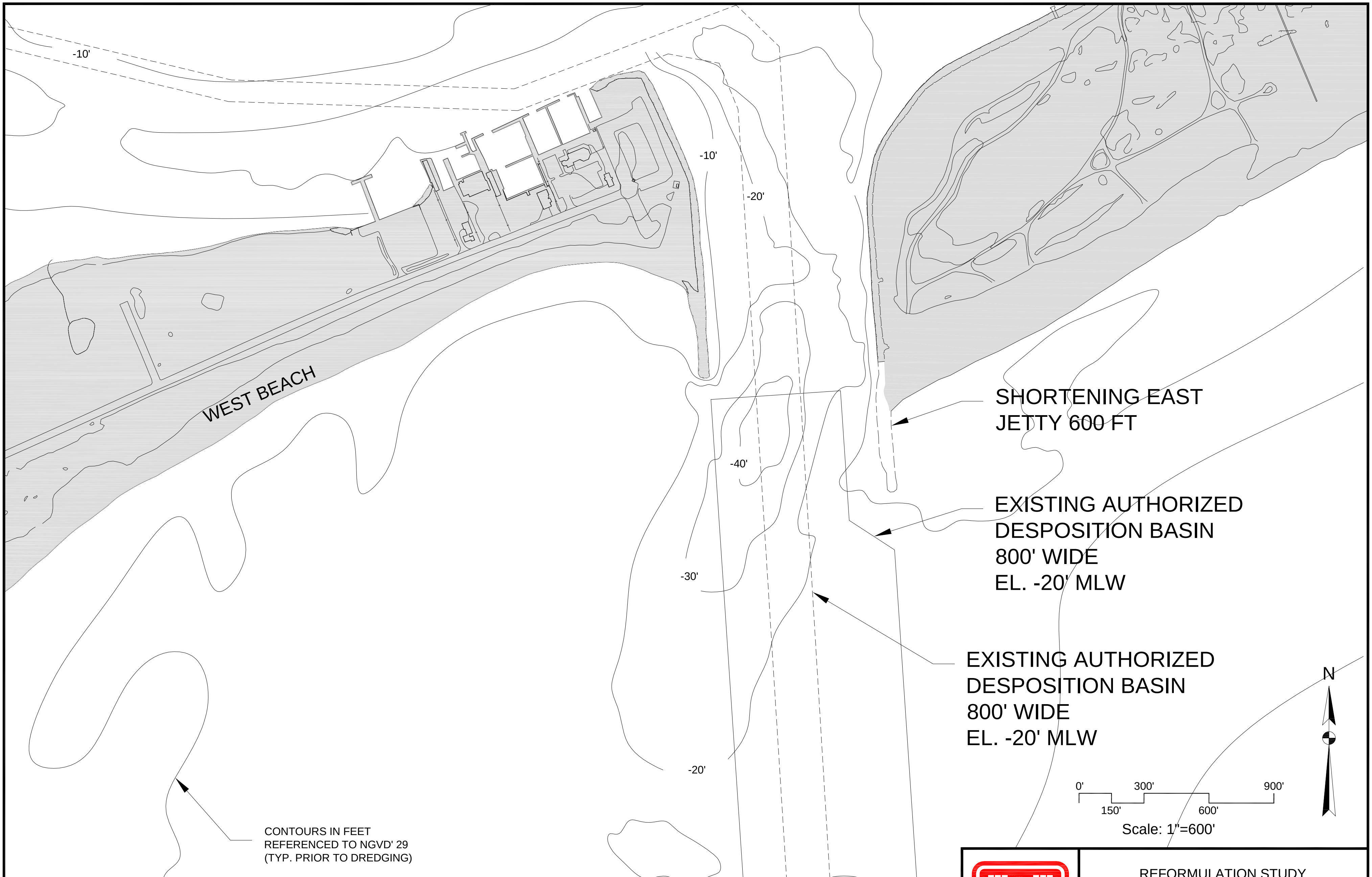
FIGURE 3-8

Filename: P:\3982-28\CAD\dwg\SI ALT No. 9.dwg (Shinnecock)



SHINNECOCK INLET - Alternative 10
EXISTING PRACTICE PLUS SHORTENING EAST JETTY

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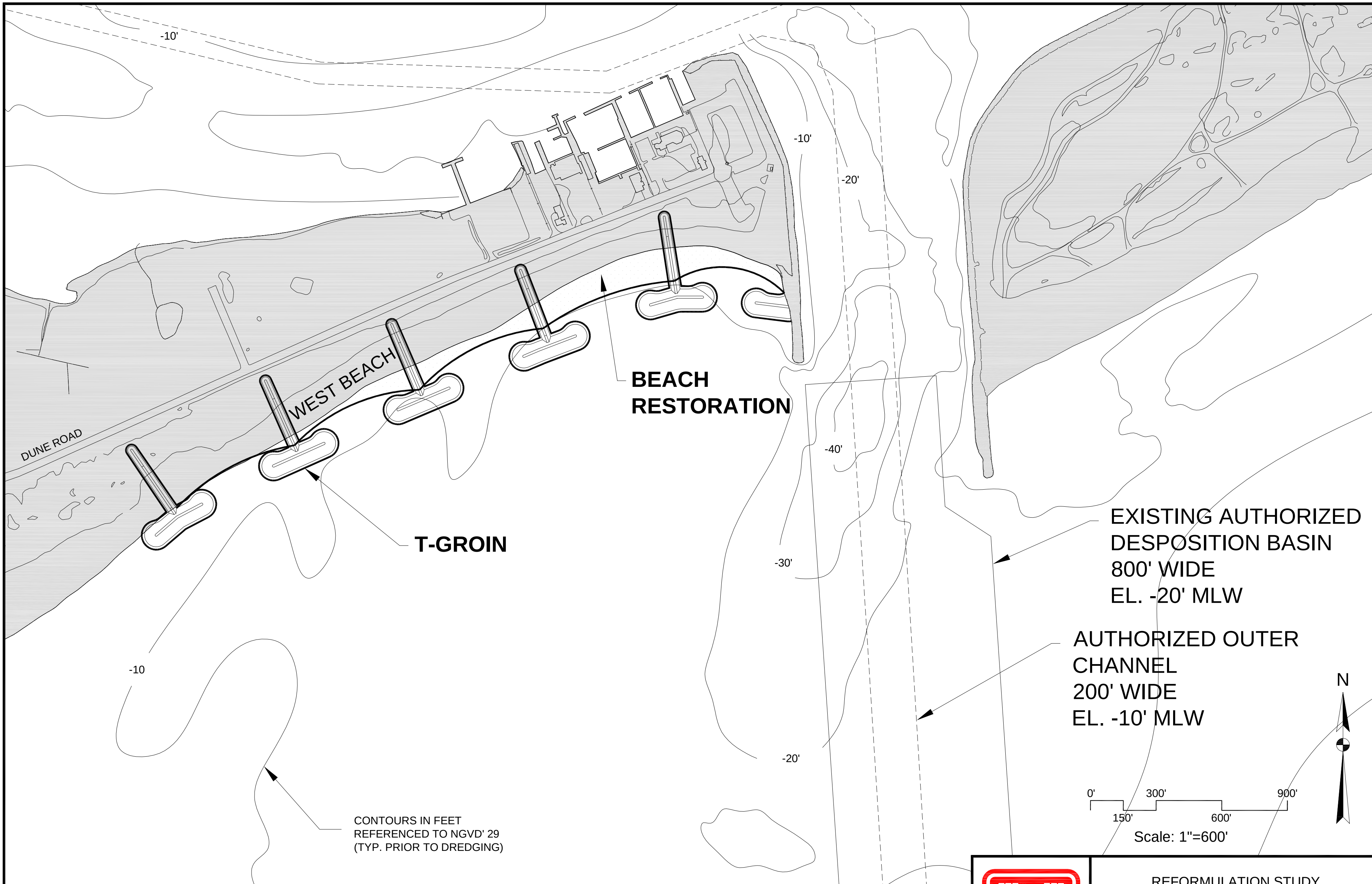
FIGURE 3-9



SHINNECOCK INLET - Alternative 12
EXISTING PRACTICE PLUS NEARSHORE STRUCTURES ALONG WEST BEACH

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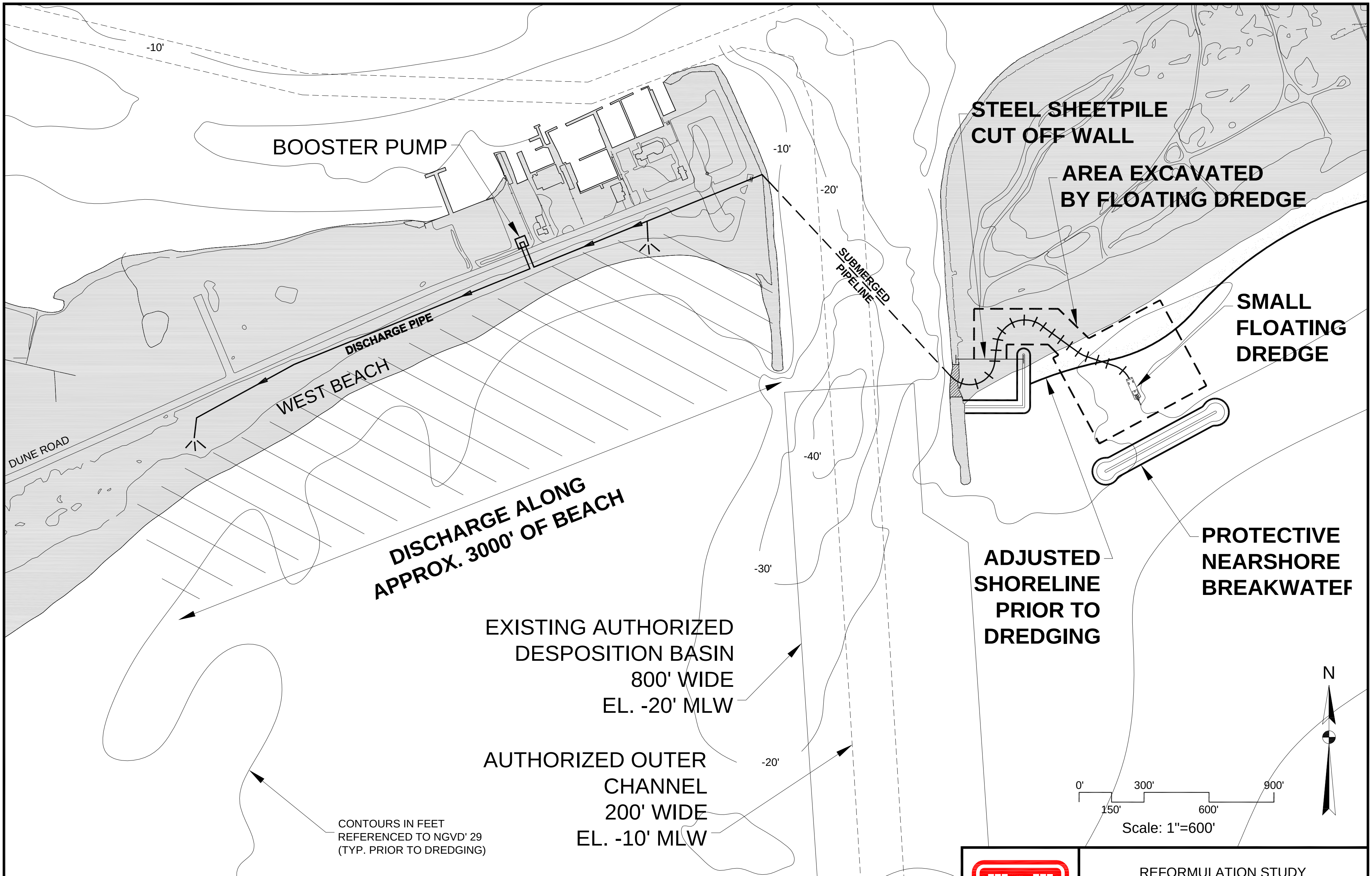
FIGURE 3-10

Filename: P:\3982-28\CAD\dwg\SI ALT No. 12.dwg (Shinnecock)



SHINNECOCK INLET - Alternative 13A
SAND TRAPPING AND BYPASSING SYSTEM UPDRIFT (WITH FLOATING PLANT)
JETTY "OPENING" AND NEARSHORE BREAKWATER

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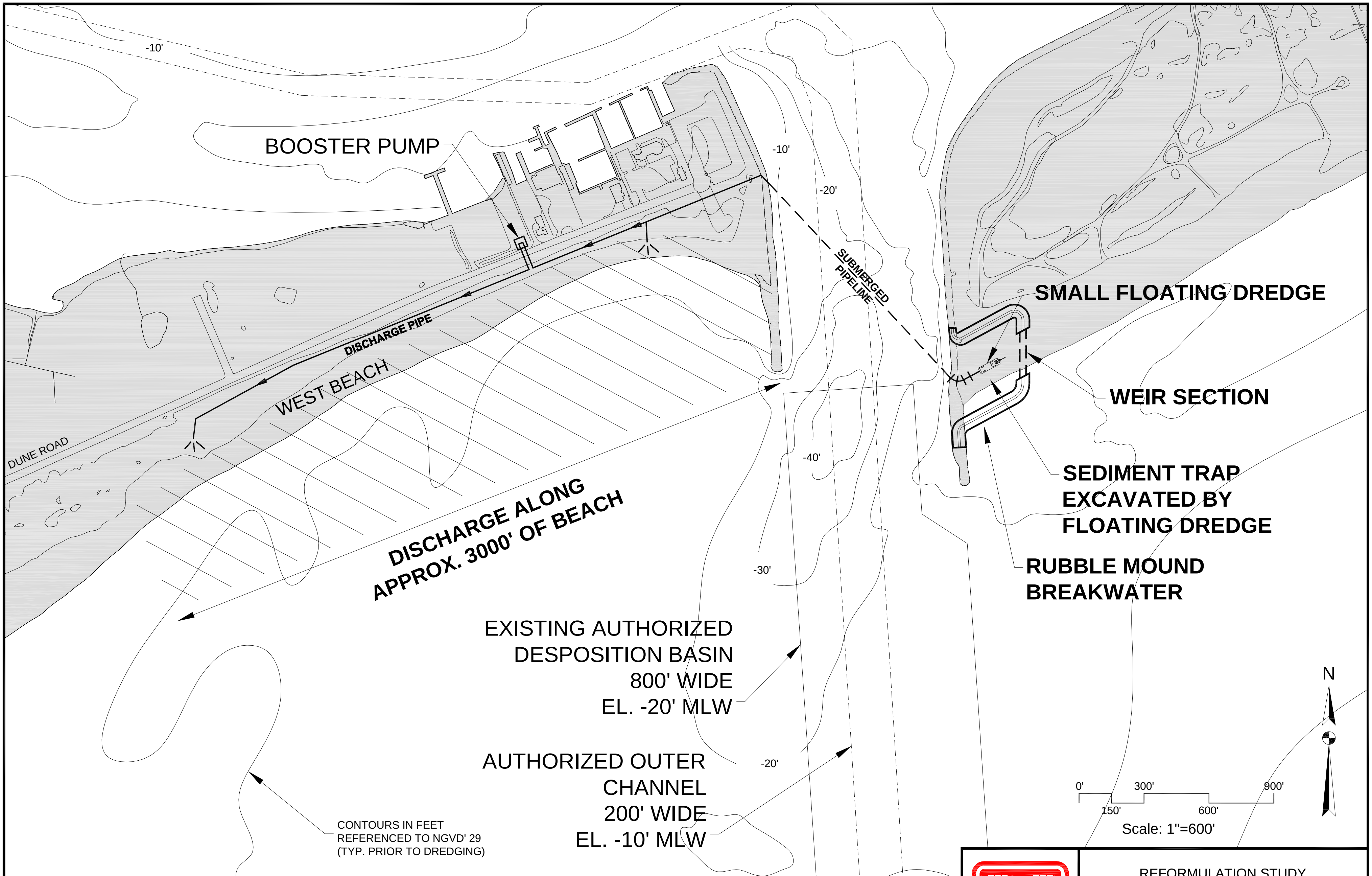
FIGURE 3-11

Filename: P:\3982-28\CAD\dwg\SI ALT No. 13a.dwg (Shinnecock)



SHINNECOCK INLET - Alternative 13B
WEIR JETTY AND SEDIMENT TRAP

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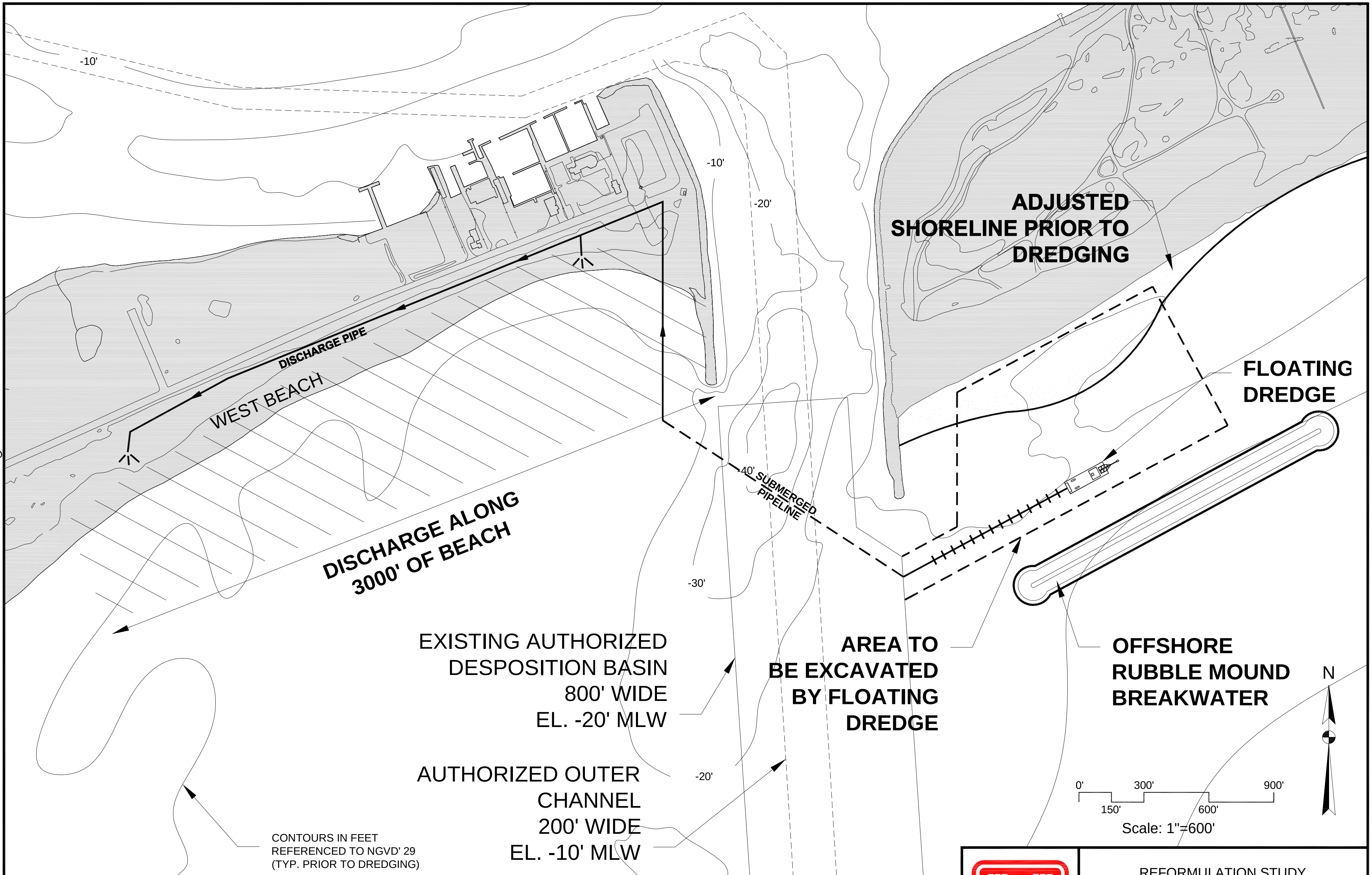
FIGURE 3-12

Filename: P:\3982-28\CAD\dwg\SI ALT No. 13b.dwg (Shinnecock)



SHINNECOCK INLET - Alternative 13C
OFFSHORE BREAKWATER

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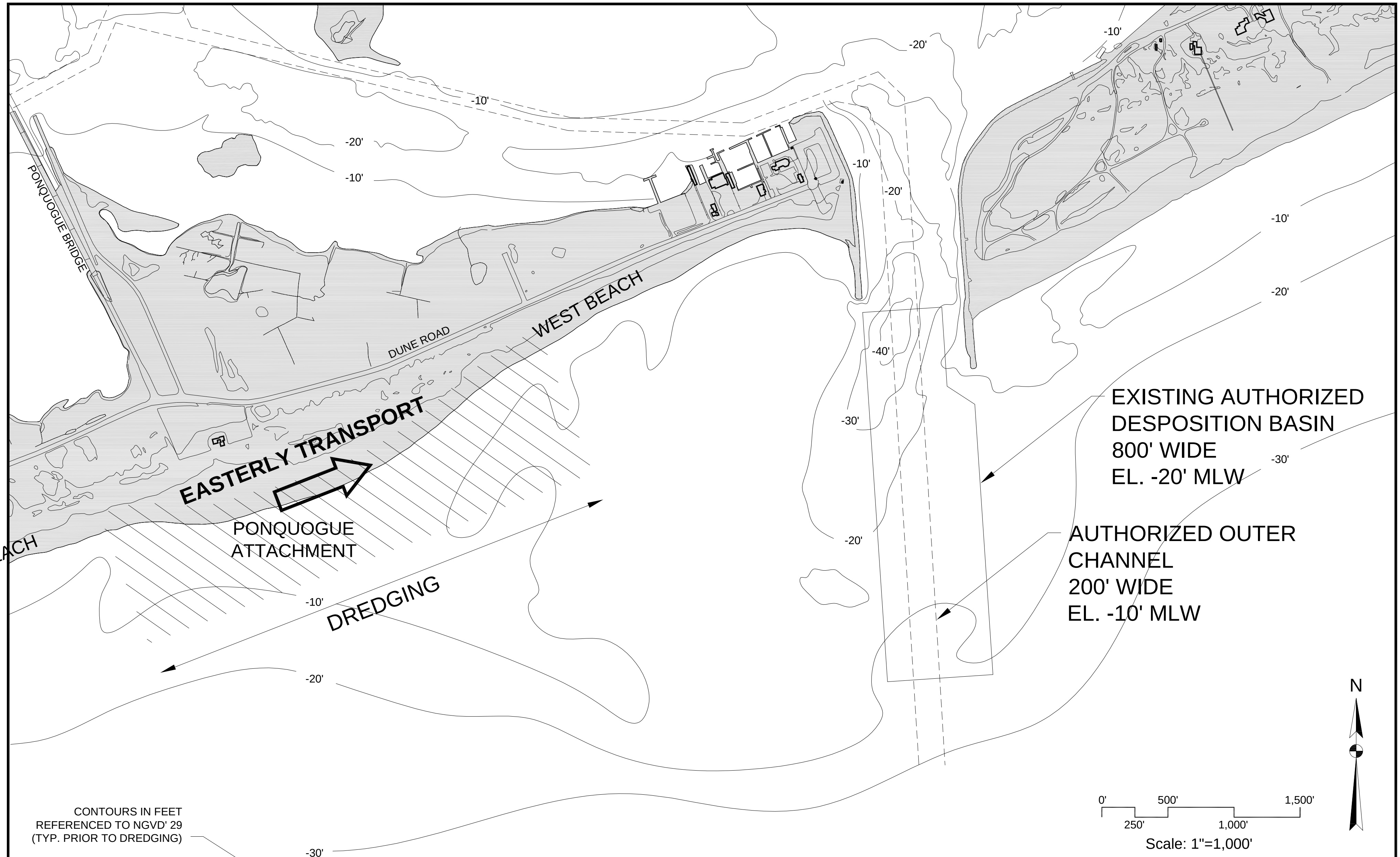
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 3-13

Filename: P:\3982-28\CAD\dwg\SI ALT No. 13c.dwg (Shinnecock)



REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
NEW YORK DISTRICT

FIGURE 3-14

SHINNECOCK INLET - Alternative 14
EXISTING PRACTICE PLUS DREDGING THE PONQUOGUE EBB SHOAL ATTACHMENT

Filename: P:\3982-28\CAD\dwg\SI ALT No. 14.dwg (Shinnecock)

4. INLET MODELING

To assess the efficacy of any proposed inlet modification alternatives, the changes in seabed morphology induced by the alternatives must be estimated. That is, will the proposed modification meet the stated goals of navigation and improved bypassing, without exacerbating existing problems or creating new ones? The most efficient method of calculating effects on sediment transport is using numerical models.

Until recently, the most common modeling approach used to investigate potential morphological changes consisted of inferring possible inlet behavior based on modeled hydrodynamic and wave condition changes or at most extrapolating the values of initial representative short-term sediment transport computations. The latter approach, although an improvement over simple hydrodynamics computations, may result in erroneous change predictions if the initial rates change significantly over time. Moreover, short-term sedimentation models typically exhibit a very scattered pattern of initial sedimentation/erosion patterns that would be "smoothed" over time in a morphological simulation. This approach also cannot account for climate variability and episodic events. Short-term sediment models therefore offer only limited insight into the impacts of any proposed inlet modifications.

Detailed modeling over large space and time scales may provide a reasonable estimate of the expected morphological changes near an inlet under different conditions. However, modeling of all hydrodynamic and wave events along with concomitant morphological changes requires excessively long simulation times. As a result, much of the research on morphological evolution of sandy and muddy coastlines has recently focused on how to make predictions with microscale (process based) models using input and process filtering (reduction) techniques (DeVriend et al., 1993; Whitehouse and Roberts, 1999; EMPHASYS Consortium, 2000). This approach reduces computational intensity by selecting a limited number of representative hydrodynamic and sediment transport conditions to use as input to a microscale process-based model. One example of input filtering is the use of a representative "morphological tide" where the sediment transport and bed evolution is driven by the average tide that would move the same amount of material per cycle as the full tidal time series. Simulation of the hydrodynamics is only required over one tide cycle rather than over a weeks- or months-long simulation.

A similar approach can be used for schematizing the influence of waves in morphological models. These techniques offer the advantage of reduced model run times provided their accuracy has been tested. Even so, state-of-the-art, microscale, sediment transport models cannot explicitly incorporate all of the physical processes that drive morphological evolution. Input and process filtering, in fact, only represent an additional simplification of simplifications already inherent in the formulation of the sediment transport equations.

Representative tidal variation and wave climate forcing may be applied to represent a seasonal wave climate until the morphological changes are so significant that the hydrodynamic conditions have to be recalculated. In this way transport and bottom computations are repeated a number of times, until bottom changes are sufficiently large that a full hydrodynamics computation is required, thereby reducing the number of hydrodynamic runs, the most computationally demanding element of the morphological process.

Morphological evolution is a very difficult process to model given the inherent uncertainties. Nevertheless, if model results are acceptable, the model is a good tool to compare different alternatives and study the differences that each one causes in certain hydrodynamic or morphological variables. The remainder of Section 4 presents methods and results of morphological modeling of the three inlets comprising FIMP. Both morphological methods discussed above, detailed modeling of all processes and input filtering, are tested to develop an effective diagnostic tool.

4.1 DELFT Modeling System

Morphological models of Shinnecock, Moriches and Fire Island Inlets were developed using the morphological model of the general Delft3D modeling system. This model (Delft3D-MOR) fully integrates the effects of waves, currents and sediment transport on morphological evolution. Delft3D-MOR includes the following components:

- *Waves (Waves module*): The HISWA model (Holthuijsen et al, 1989) solves refraction and dissipation of directionally spread random waves. Several computations through a tidal cycle are carried out in one call. Model formulation is similar to STWAVE.
- *Hydrodynamics (Flow Module):* Delft3D Flow is a multidimensional (2D or 3D) hydrodynamic (and transport) simulation program which calculates non-steady flow and transport phenomena that result from tidal and meteorological forcing on a curvilinear, boundary-fitted grid. In 3D simulations, the vertical grid is defined following the sigma coordinate approach. The model solves the Navier-Stokes equations for incompressible fluid under the shallow water and the Boussinesq assumptions. In the vertical momentum equation, the vertical accelerations are neglected, resulting in the hydrostatic pressure equation.
- *Sediment transport (Sand or Silt Module):* This model applies the time dependent results obtained from the Waves and Flow modules to calculate the sediment transport in the curvilinear flow grid. In the case of non-cohesive sediment, the model can either calculate the total transport or account separately for bed-load and suspended sediment transport. A special version of this model may be used to calculate the sediment transport for cohesive material. The implemented sediment transport formulas are: Engelund-Hansen, Meyer-Peter-Muller, Swanby (Ackers-White), General Formula based on Meyer-Peter-Muller, Bijker with Waves, Van Rijn, and Ribberink – Van Rijn.
- *Bottom changes (Bottom Module):* Computes the bed level variation induced by the sediment transport module by solving the bed level continuity equation.

Each component of the model is developed and calibrated separately, then combined to simulate bed morphology. The model allows the simulation of time scales from days to years. The morphological process is built up from morphological time steps, which consist of a simulation of wave-current interaction over a period of time, followed by the computation of the average sediment transport over that period, and the bottom update.

The general processing sequence within Delft3D-MOR is as follows. A morphological time step starts by running the Wave Module using the hydrodynamics and the bathymetry from the previous step. Then, the hydrodynamics are calculated including the wave-current interaction effects. Hydrodynamic results (water surface elevations and currents) are subsequently used to simulate sediment transport over the same period of time. Sediment transport calculated for this period is used to compute bottom changes. The calculated bottom changes determine the bathymetry at the end of the period, which is then used in the subsequent period. Separate processes are coupled via a bottom evolution model based on sediment conservation. The frequency and length for each module run within Delft3D-MOR can be specified separately. This allows for great flexibility in the optimization of total model run times. Delft3D-MOR also permits the specification of dredging scenarios during the morphodynamic simulation.

To improve efficiency, the model can apply the continuity correction method, used to adjust the flow field solution due to small changes in bathymetry. The flow pattern is assumed relatively constant for small bottom changes. Sediment transport is recomputed after adjusting the average flow velocity and wave-induced orbital velocity, which allows the interval between full hydrodynamic simulations to be increased.

The application and calibration of Delft3D-MOR to Shinnecock, Moriches and Fire Island Inlets is presented in the following sections. The mechanics of the modeling system are discussed in detail only for Shinnecock Inlet, but are applicable to all three inlets.

4.2 Shinnecock Inlet

4.2.1 Hydrodynamics

Model Grid

The Shinnecock Inlet Model (SIM) computational grid extents from the east end of the Quogue Canal at the west, to Southampton Beach at the east (~9.1 miles) to approximately 3.2 miles offshore of the inlet's mouth, including Shinnecock Bay and Tiana Bay (see Figure 4-1). The offshore boundaries (hydrodynamic, wave and sediment transport boundaries) are located far enough from the inlet that changes in bathymetry and hydrodynamics near the inlet will not influence the boundary conditions.

SIM is built on a curvilinear computational grid. The offshore boundary is nearly parallel to the shore and the two lateral boundaries located at Quogue Canal and at Southampton Beach are perpendicular to the coast. The grid resolution is variable throughout the model domain. The highest resolution is found at the inlet throat: 13 m grid spacing along the axis of the inlet and 20 m spacing across the throat. Figure 4.2 shows a detail view of the grid at the vicinity of the inlet throat. Along the coast on either side of the inlet the grid size is on the order of 10-20 m. The grid size in the longshore direction increases with distance from the inlet, reaching values of up to 210 m. The grid size also increases incrementally from the coast to the offshore boundary up to maximum spacing of 630 m. The flexibility of the curvilinear grid allows high resolution in the areas of interest, especially at the inlet, the updrift/downdrift beaches, and at the ebb and flood shoals. Computational efficiency is improved by the use of larger grids in other areas. In Shinnecock Bay, the model is defined using a coarser grid except at the Ponquogue Bridge and at the flood shoal areas. Overall, SIM has 11,700 computational points.

Figure 4-1 shows the SIM computational grid. The grid has been constructed following the criteria of orthogonality, and smoothness in both x and y-directions as defined by Delft Hydraulics (Delft Hydraulics, 1999) in order to minimize errors in the finite difference approximation.

Model Bathymetry

The model bathymetry has been constructed using data available from the sources presented in Table 4-1. Data for the offshore part of the model is available from the Geophysical Data System (GEODAS) developed by the National Geophysical Data Center. GEODAS is an interactive database management system for use in the assimilation, storage and retrieval of geophysical data. The GEODAS database contains information from surveys conducted between 1930 and 1980.

Table 4-1: Bathymetric Data Sources for the Shinnecock Inlet Model

Data Source	Description
GEODAS	Geophysical data System developed by the National Geophysical Data Center
SUNY traditional Survey	Traditional boat survey conducted in 1998 by State University of New York at Stony Brook
SHOALS survey	Dense Bathymetry data collected by the Scanning Hydrographic Operational Airborne Lidar System (1994-2001)
Acoustic Data (USACE NAN)	Bathymetric data obtained from an acoustic survey (4-6 – March-1998)
Beach Profiles	Collected for the Atlantic Coast of New York Monitoring Program

Table 4-2 shows the GEODAS hydrographic surveys that cover totally or in part the Shinnecock Inlet area and the survey year.

The general bathymetry of the interior bay has been constructed using hydrographic surveys from the 1930's. The State University of New York at Stony Brook conducted a more recent traditional boat survey in March 1998 which covers portions of the bay. This survey has been used to improve and complement the old hydrographic survey.

High-density SHOALS (Scanning Hydrographic Operational Airborne Lidar System) data are available for the years: 1994, 1996, 1997, 1998, 2000 and 2001. The data cover most of the ebb shoal and part of the downdrift and updrift beaches for each year and a large part of the flood shoal for the last 4 years. Available SHOALS data for Shinnecock inlet are presented in Section 5. The bathymetry of the near shore area obtained from the SHOALS data has been complemented using beach profiles from the Atlantic Coast of New York Monitoring Program.

Table 4-2: GEODAS Hydrographic Surveys for Shinnecock Inlet Area

Data Source	Year	Area Specific
H09550	1975	Fire Island (general area description)
H09551	1975	South Shore Shinnecock Inlet to East Hampton
H06328	1938	Southampton to vicinity of Bellport
H06329	1938	Montauk Point to vicinity of Southampton
H06331	1938	Apps. to New York Harbor South of Block Island to Long Island
H05323	1933	Shinnecock and Quantuck Bays
H05324	1933	Southampton to West Hampton
H05325	1933	Montauk to Southampton
H05379	1933	Great Peconic Bay

All these data sources are integrated within the Delft3D modeling system by interpolating the values into the SIM grid using triangular interpolation. Figure 4-3 presents the SIM model bathymetry derived from the May 1998 SHOALS survey.

Hydrodynamic Data

NOAA's National Ocean Service (NOS) operated three water level gauges in the area of Shinnecock inlet: Shinnecock Bay (from 09/77 to 04/79), Ponquogue Point (from 07/89 to 06/90) and Shinnecock Inlet (from 06/78 to 05/79). As shown in Figure 4-4, two of the stations (Shinnecock Bay and Ponquogue Point) are located in Shinnecock and Tiana Bays, while the third one is located at the ocean side of Shinnecock Inlet. The deployment period was long enough in order to have sufficient data for tidal datum calculation. The mean tide range decreases from 3.33 feet at the inlet to 2.44 feet in Tiana Bay.

Metocean data (which comprises observed measurements of current, wave, sea level and meteorological data) has been extensively collected at the Shinnecock inlet area since 1998. LISHORE provides information on sea and shoreline conditions for Long Island, New York (USACE, 2001). LISHORE began in 1998 as the Shinnecock Inlet Field Monitoring Project. From this initial focus on Shinnecock Inlet and Shinnecock Bay, LISHORE has expanded along the South Shore of Long Island. LISHORE is a database of meteorological/hydrological data, historical data, images, and written information generated by the U.S. Army Corps of Engineers or received from other official sources. The location of the LISHORE stations and other available metocean data in the Shinnecock inlet area are shown in Figure 4-4. The period of record of all Shinnecock hydrodynamic data sources is displayed in Figure 4-5.

Tidal datums computed over the period of record for the LISHORE gauges within Shinnecock Bay are presented in Table 4-3.

Table 4-3: Tidal Datums (ft) for Shinnecock Tide Gauges				
Datum	Ocean Gauge P1 (04/98 – 05/01)	Town Dock P2 (04/98 – 12/01)	Shinnecock Canal P3 (04/98 – 08/99)	Quogue Canal P4 (04/98 – 08/99)
MHHW	3.86	3.49	3.31	2.92
MHW	3.61	3.26	3.08	2.71
MTL	1.88	1.70	1.60	1.40
MLW	0.15	0.14	0.12	0.09
MLLW	0.00	0.00	0.00	0.00

With the exception of P4, where the tidal range is practically identical, tidal ranges calculated from this longer data set are on the order of 0.10 feet larger than those presented in Militello and Kraus, (2001), which were based on only two months of data.

Current velocity data is available from gauges C4 and C2. C4 is a side-looking current profiler set to record the horizontal current in 4-m bins and mounted on the east jetty at mid-depth (approximately 10ft below MTL). The meter has a range of up to 160 m across the inlet. C2 is a side-looking acoustic current meter with a pressure sensor located at the North edge of Ponquogue Bridge. The location of both gauges is shown in Figure 4-4.

Velocity and discharge were measured in transects across the inlet using a boat-mounted Acoustic Doppler Profiler (ADCP) during a number of days in 1997, 1998 and 1999. This measurement program is detailed in Pratt and Stauble (2001). Tidal discharge was computed by multiplying the average velocity in each bin by the bin area, then summing the bin discharges vertically over the water column and then horizontally across the channel. Time series of measured discharges for 4 December 1997 and 22 July 1998 have been obtained from Militello and Kraus (2001).

<u>*Boundary Conditions*</u>

SIM is forced from the three open offshore boundary conditions (east, west, south). The boundaries are defined as time series of water surface elevations constructed from nine major tidal constituents extracted from the high-resolution ADCIRC EastCoast 2001, finite-element tidal model (Luettich, et al, 1995). Table 4-4 lists the amplitude and phase for each constituent at two points along the boundary: the eastern edge and the western edge. Linear interpolation between points is applied along the offshore boundary, while the water surface has no slope on the east and west boundaries.

Note that the model treats the Quogue and Shinnecock canals as closed boundaries, with no flow through them. Although previous studies (Militello and Kraus, 2001) have shown that the inflow through Shinnecock Canal increases the peak ebb discharge by 4 percent, the model was successfully calibrated without including the canal inflow as detailed below.

Table 4-4: Boundary Tide Constituents - Shinnecock Model				
	Eastern BC		Western BC	
Constituents	Amp (m)	Phase (deg)	Amp (m)	Phase (deg)
O1	0.0509	178.3	0.0497	180.8
K1	0.0854	166.5	0.0838	166.8
N2	0.1116	329.8	0.1074	330.8
M2	0.4808	342.3	0.4603	342.9
S2	0.0985	3.5	0.0950	4.3
K2	0.0220	15.0	0.0212	16.0
Q1	0.0099	170.4	0.0100	172.2
M4	0.0122	352.0	0.0150	347.4
M6	0.0134	211.6	0.0130	205.9

Constituents extracted from EastCoast 2001 model (Luettich, et al, 1995)

Calibration

The hydrodynamic model was calibrated for the period 5 November 1998 to 30 November 1998. This period had the greatest number of simultaneously operating measurement stations (see Figure 4-5). Near-field bathymetry used for this period was based on the SHOALS from May 1998 and the SUNY survey of Shinnecock Bay as described in previous sections. The bathymetry did not consider the dredging works of September 1998 (see Table 2-1). In order to study the sensitivity of the hydrodynamic model calibration to this change in bathymetry, an additional model run was performed where the deposition basin was dredged to 22ft. Comparison of simulations with and without the deepening of the deposition basin showed very similar results at all the calibration locations.

Model skill was assessed using the following three error estimates: correlation coefficient, root mean square (RMS) error and percent error.

- *Correlation Coefficient*: Uses the Pearson product moment correlation coefficient, *r*, (a dimensionless index that ranges from -1.0 to 1.0 inclusive) that reflects the extent of a linear relationship between two data sets. This parameter indicates how closely the modeled data is in phase with the calibration data. An index of 1.0 indicates the two data sets are linearly perfectly in phase, an index of –1.0 indicates the data are 180 degrees out of phase.
- *Root Mean Square (RMS) Error*: The square root of the average square of the difference (error) between the data points.

$$RMS = \sqrt{\frac{\sum\left(x_i - y_i\right)^2}{n}}$$

Where $i = 1..n$

- *RMS Error Percentage*: Computes the RMS Error as a percentage of the range of the predicted/measured data. This gives perspective on the magnitude of the RMS error. Elevation error is computed based on mean range, current error is computed based on the mean peak flood/ebb speed.

The calibrated model results were compared to the water level and current measurements at the data station locations. The data has been analyzed to extract the same nine tidal constituents that comprise the boundary condition. A time series constructed from the data-derived constituents is compared to the model output. Results of calibration are shown in Table 4-5. Time series of model output compared to harmonic tides and observed water levels and currents are presented from Figure 4-6 to Figure 4-11.

Table 4-5: Model Calibration Statistics - SIM output vs. harmonic time series

Water level data	Correlation	RMS Error (m)	Percent Error
P1	0.994	0.04	3.6
P2	0.997	0.03	3.2
P3	0.996	0.04	4.1
P4	0.943	0.12	13.0
C1	0.993	0.06	5.8
C2	0.996	0.05	4.3
Current data	Correlation	RMS Error (m/s)	Percent Error
C2	0.913	0.22	14.9
C4	0.960	0.28	15.0

Note: current statistics are based on measured currents, not harmonics

In general, modeled water levels are in very good agreement with the data. The highest error is observed at P4 and it is probably associated to several factors among which the most important could be considering Quogue Canal as a closed boundary in the model and more importantly the lack of recent bathymetric data for west Shinnecock Bay.

Simulated currents were compared to observations at two stations C2 and C4. Modeled current velocity at station C2 very accurately predicts the current phase but slightly overpredicts the peaks in speed In order to compare model results with data available at station C4, the average of the velocity at the eastern most 20 bins was computed and compared with the average of the simulated velocities at the same area. This part of the inlet cross-section presents the highest currents. The model predicts quite accurately the flood current, though slightly overestimates the ebb current. Table 4-5 presents the results of the calibration for station C4. Time series of observed and simulated currents are presented in Figure 4-12.

The model was run for two additional periods corresponding to the dates when ADCP discharge measurements were taken at the inlet: 2-4 December 1997 and 21-23 July 1998. Figure 4-13 shows a comparison of model output to measured discharge. Only graphical comparison of the discharge is made due to the intermittent data collection method. Agreement with measured flows is nonetheless excellent.

4.2.2 Waves

The stationary wave model HISWA (Holthuijsen et al. 1989) is a second generation wave model that computes wave propagation, wave generation by wind, non-linear wave-wave interactions and dissipation for a given bottom topography and stationary wind, water level and current field in waters of deep, intermediate and finite depth. The model accounts for the following physics: Wave refraction over a bottom of variable depth and/or spatially varying ambient currents; depth and current induced shoaling; wave generation by wind; dissipation by depth-induced breaking and/or bottom friction; and wave blocking by strong counter currents. Since the model does not account for pure diffraction effects the wave field computed will generally not be accurate in the immediate vicinity of obstacles and in harbors.

HISWA is based on the action balance equation and wave propagation is based on linear wave theory (including the effect of currents). HISWA wave computations are carried out on a rectangular grid. The results obtained in this rectangular grid are automatically transferred to the hydrodynamic module, which simulates the flow on a curvilinear grid. Nonstationary conditions are simulated with HISWA as quasi-stationary with repeated model runs, i.e. as the flow model progresses in time a stationary wave computation is performed at intermediate time steps. All the wave models presented in this report have been created using the wave model HISWA.

Local Wave Models within Morphological Model

The wave model domain is defined on a number of grids in a common Cartesian coordinate system. The Shinnecock wave model uses three nested wave grids (see Figure 4-14). The first grid (offshore grid) is the coarsest and has a resolution of 50 m in the direction of the wave propagation (x-direction) and 200 m in the direction perpendicular to the wave propagation (y-direction). Because of the limitation on the incoming wave direction that can be resolved with one grid, and in order to cover all the recorded wave directions with the model, three offshore wave grids have been built. The upwave boundary condition is perpendicular to the 165°, 155° and 145° azimuth directions of the three offshore grids. The offshore grids propagate the incoming waves from the upwave boundary condition, located at a depth of approximately 25 meters (82 feet) to the finer grids, thereby providing boundary conditions for the next grid.

The second level of grid resolution (nearshore grid) has a resolution of 25 m in the x-direction and 100 m in the y-direction. This grid propagates waves from the offshore grid and transforms the waves to the nearshore zone. The orientation of the nearshore grid is perpendicular to the 158° azimuth direction.

A third grid (inlet grid) provides even higher resolution in the inlet vicinity. The inlet grid has a resolution of 5 m in the x-direction and 20 m in the y-direction. The orientation of the inlet grid is perpendicular to the 173° azimuth direction. The finer grid permits the calculation of wave breaking and shoaling in the inlet entrance.

Wave Data

Data from four directional wave gauges are available. The location of the data gauges is shown in Figure 4-15. The station farthest offshore is National Data Buoy Center (NDBC) buoy 44025 in approximately 40 m of water. Three nearshore gauges are located at depths of approximately 12 m (gauge ADV1-Shinnecock inlet entrance), 6 m (ADV2-West of the west jetty) and at 9.5 m at the Westhampton beach (NY001). The NDBC 44025 buoy has been recording standard meteorological data during the periods 1975-1981 and 1991 to the present. In addition, this buoy has been recording additional spectral wave data since 1996. The NY001 gauge has been recording data since 1994, and the two LISHORE data gauges (ADV1/ADV2) data are available from 1998 to the present. Figure 4-5 shows the period of record for all data sources.

Boundary Conditions (Regional Wave Model)

Wave boundary conditions for the morphological model boundaries at a depth of 25 m have been generated from two different data sources: NY001 at a depth of 9.5 m and buoy 44025 at 40m. The transformation to 25 m was carried out using a regional wave transformation model. The model covers the area from Montauk Point at the East to West of Jones Inlet. This model contains two grids parallel to the coast, with different resolution. The first grid propagates the wave from the offshore boundary located at a depth of approximately 45 m to the second finer grid which simulates the wave transformation in the nearshore. Figure 4-15 presents the extent and bathymetry of the regional wave model.

<u>*Model Skill Assessment*</u>

Regional Wave Model

Model output at the location of NY001 was compared to observations in order to test the skill of the regional wave model. Input to the model was based on measured wave conditions at NOAA 44025. The period selected for calibration was February 1999. Comparison of wave height and direction between model output and observations at NY001 are presented in Figure 4-16. Gaps in the data and model correspond to periods during which waves are directed offshore. Overall, model results appear to compare very well with observations. Wave heights are very accurately reproduced, even for storm events. Wave direction is also well predicted, varying by about 5 degrees for very oblique incident wave directions.

Local Wave Models within Morphological Model

Figure 4-17 presents comparison of observed and modeled wave parameters at ADV1 based on wave conditions at NY001 estimated by back-propagating NY001 measurements to 25 m using the results from the regional model. The local wave models reproduce the wave height at ADV1 reasonably well. Wave direction at ADV1 shows good agreement for the South and Southeast waves but it seems to predict Southwest waves too much towards the south. This is probably because the high ebb and flood currents at the inlet mouth where ADV1 is located are not included in these wave model runs and therefore any wave current interaction has been neglected. In addition, it has to be mentioned that the measurement error for both ADV1 and NY001 data were not considered in this assessment. Differences between measurement error for ADV1 and NY001 also contribute to the discrepancies between simulated and observed values of ADV1.

4.2.3 Sediment Transport & Morphology

<u>*Model Set-up*</u>

The hydrodynamic and wave models have been described and calibrated in previous sections. The sediment transport formulation used in the morphological model of Shinnecock inlet is based on the well-known Bijker sediment transport formulation (Bijker, 1971), which accounts for the effect of waves. The model calculates separately suspended and bed load transports. For this application, a constant grain size distribution was applied throughout the domain: D50 =0.430 mm and D90=0.690 mm, which represents the average for the Shinnecock inlet area. These values are based on studies presented by Pratt and Stauble (2001) and McCormick (1971). The grain settling velocity is 0.063 m/s and the porosity 0.4. Selected sand density was 2,650 kg/m3.

Boundary conditions are only required at the open boundaries. Separate boundary conditions are required for suspended and bed load transport. For the suspended sediment transport, the boundary condition during inflow is defined as a concentration equal to equilibrium concentration, and during outflow it is equal to upstream concentration. For the bed load transport, a bed level condition is imposed where the bed level remains constant at the boundary segment.

Two approaches to model calibration were tested:

1. *Detailed Time-series Modeling.* In order to optimize model calibration, and given that a significant amount of bathymetric, sediment, hydrodynamic and wave data is available at Shinnecock Inlet, the model was initially set-up to simulate morphological changes based on a detailed time series of

measured waves and predicted tides. Specifically, a period between two recent SHOALS surveys (13 August 1997 to 28 May 1998) was selected.

2. *Input Filtering.* A second approach was also used to calculate the morphological changes for this period. This approach is based on and input filtering approach that forces the model with a set of representative tidal and wave conditions which generate morphological changes similar to those generated with the complete, detailed time series of waves and predicted water levels.

The following sections describe each approach and provide detailed results.

Model Calibration through Detailed Time-Series Modeling

The detailed model uses a morphological step of 1 hour. During this period the wave conditions are constant. Before the start of the process, the wave direction and initial bathymetry are defined. As explained in Section 4.2.2, the wave model uses three offshore wave grids, the active grid is a function of the incident wave direction. Each morphological step is a morphological model in itself. The bathymetry is created from the end of the previous step. After each step, a restart file is saved and used to initialize the next step with the latest bathymetry. This process provides a smooth transition between morphological steps.

Each morphological step contains the following model runs:

1. Steady wave conditions are calculated using the water level and current fields from the last time step of the previous 1-hour period.
2. The hydrodynamic model is then run for a 1-hour period using the wave radiation stresses computed from the wave model.
3. The sediment transport model computes the average sediment transport for the 1-hour period.
4. Using the average sediment transport calculated in the previous step, the bottom changes for 1-hour are computed and the bathymetry is updated.

Figure 4-18 and Figure 4-19 show morphological changes from 13 August 1997 to 28 May 1998 as measured with the SHOALS system and as simulated with the morphological model, respectively. The model does an adequate job reproducing the observed morphological changes, given the general shortcomings of state-of-the-art sediment transport models and the complexity of the inlet system, particularly with regard to some of the most important features such as the east lobe, the outer channel, and the deposition basin. The model accurately simulates the N/S to NE/SW shift in channel alignment as well as the accumulation in the deposition basin. Areas where the model may require additional improvement are the bypassing bar on the west side of the channel and the channel throat. Overall, results suggest that the model may be used to investigate short- and long-term impacts of any proposed inlet modifications.

Model Calibration through Input Filtering

The second approach used to simulate the morphological changes at Shinnecock Inlet was based on input filtering techniques. Instead of the real tidal variability at the boundaries, this technique applies a representative tide that generates the same morphological changes that will take place if the full tidal variability is applied. In addition the wave climate for the simulation period is described by a discrete number of wave conditions which are applied for a specific time. The idea is to reproduce the same net transport as observed in a morphological simulation using the full tidal variability and wave climate but at a much smaller computational cost. Another important advantage is that for each morphological time step the transport and bottom computations could be repeated a number of times using the "continuity correction" method, until bottom changes are so large that a full hydrodynamics computation is required. The continuity correction method is based on the assumption that the velocity patterns are not significantly influenced by small bed level changes. The continuity correction is applied as follows. Before the new hydrodynamic and wave fields are calculated, the depth at every grid point is stored. Then, sediment transport is calculated, the bathymetry updated, and the difference in depth is computed at every grid point. If either the maximum difference (the threshold is specified in absolute value) or if the relative change (the threshold is specified in percentage of change) throughout the domain remains below certain threshold the continuity correction method is applied. Note that only one of the two methods (absolute or relative) can be used in the same model run. A new depth is computed by recalculating the sediment transport with the adapted velocity and orbital velocity fields. If this threshold has not been met after a number of applications of the continuity correction, the full hydrodynamic and wave computations will be performed using the latest computed bathymetry. The model used a relative depth change of 10% and a maximum number of 6 consecutive "continuity correction" applications. Figure 4-20 presents an example of the morphological model process tree.

The following sections described the calculation of the representative tide and the representative wave conditions.

Representative Tide

One representative spring/neap cycle has been selected from a longer time series (see top panel of Figure 4-21). This cycle contains 27 double tide intervals as shown in the center panel of Figure 4-21. The results from the morphological model run for each cycle have been compared to the morphological results of the complete spring/neap cycle. The same initial bathymetry is used in all these morphological runs. The parameters to be compared among the different morphological runs are:

- Correlation between the bottom changes in all grid points for each double-tide period and for the spring-neap period. This parameter indicates if the overall pattern is represented correctly.
- The slope of the linear regression between the bottom changes at all points over the spring-neap period and those over the selected period. This can be seen as a time-scale factor, i.e. it provides an indication of the relation between the magnitude of the changes obtained from each tide and the total changes in the spring-neap cycle.
- The weighted standard deviation, which is an indication of the magnitude of the bottom changes.

The correlation and slope of the linear regression between the morphological changes obtained for each cycle run and the full spring-neap run are presented in the bottom panel of Figure 4-21. It can be observed that the highest correlation was obtained for the transition tides from spring to neap and also from neap to spring. The lowest correlation was obtained for the neap tide cases. In addition, it is also observed in the bottom panel of Figure 4-21 that the tides during spring tide will overpredict morphological changes in the order of 2.5 times larger than those expected with the spring/neap cycle. On the other hand for cycles within neap tide will underestimate the total changes by up to 50 %. The tidal cycle with the higher correlation (9, 10 and 16) will overpredict significantly the changes, therefore, a cycle with a still high correlation (0.927) but at the same time a slope very close to one (0.997) has been selected. The selected cycle (TIDE 8) is shown in Figure 4-21. The values for correlation, slope and standard deviation for each cycle are presented in Table 4-6.

Table 4-6: Comparison of modeled transport from a spring/neap cycle and the individual cycles				
TIDE #	Correlation	Slope	Standard Deviation	Weighted STD
TIDE 1	0.908	1.039	0.037	0.038
TIDE 2	0.894	1.111	0.034	0.038
TIDE 3	0.881	1.226	0.030	0.037
TIDE 4	0.870	1.372	0.027	0.037
TIDE 5	0.862	1.508	0.024	0.036
TIDE 6	0.867	1.505	0.024	0.036
TIDE 7	0.894	1.301	0.029	0.038
TIDE 8	0.928	0.997	0.039	0.039
TIDE 9	0.949	0.726	0.055	0.040
TIDE 10	0.950	0.535	0.075	0.040
TIDE 11	0.937	0.419	0.094	0.039
TIDE 12	0.922	0.359	0.108	0.039
TIDE 13	0.916	0.342	0.113	0.039
TIDE 14	0.923	0.366	0.106	0.039
TIDE 15	0.936	0.435	0.091	0.039
TIDE 16	0.946	0.567	0.070	0.040
TIDE 17	0.942	0.785	0.050	0.040
TIDE 18	0.915	1.121	0.034	0.038
TIDE 19	0.883	1.532	0.024	0.037
TIDE 20	0.851	1.792	0.020	0.036
TIDE 21	0.839	1.729	0.020	0.035
TIDE 22	0.846	1.505	0.024	0.036
TIDE 23	0.868	1.296	0.028	0.037
TIDE 24	0.888	1.142	0.033	0.037
TIDE 25	0.906	1.032	0.037	0.038
TIDE 26	0.919	0.959	0.040	0.039
TIDE 27	0.929	0.905	0.043	0.039

Representative Waves

The wave schematization method applied in this section is based on van Duin (2002) and consists of two steps: (1) division of the given wave climate in a number of sectors, and (2) calculation of the required simulation time of the schematized waves in order to obtain the same overall net sediment transport as would be obtained using the real data. The following calculation uses wave data at the Shinnecock inlet wave model boundary (approximately at 25 meters of depth) that has been transformed using the regional wave model forced with NDBC 44025 wave data at the offshore boundary. The selected period coincides with the period between the two recent SHOALS surveys (May 28, 1997 to August 13, 1998).

A directional sector from 90° to 220° was initially divided into equal sectors of 10°. A number of 10° sectors were then combined, obtaining 4 sectors with percentage of occurrence between 20 and 30 %. The frequency distributions for each 10°-sector and for the four multidirectional sectors are presented in Table 4-7 considering 11 wave height cases.

Table 4-7: Frequency distribution of wave direction versus wave height. Shinnecock Inlet (1997-98 period)

	<90	<100	<110	<120	<130	<140	<150	<160	<170	<180	<190	<200	<210	<220	>220	TOT
<0.5	0.19	0.45	2.05	2.16	1.64	1.06	1.12	0.93	0.32	0.17	0.09	0.17	0.13	0.15	0.76	**11.39**
<1.0	0.63	0.93	4.79	6.25	3.65	3.08	2.80	2.65	1.57	1.79	1.25	1.32	1.40	1.49	2.50	**36.10**
<1.5	0.97	0.58	1.47	3.73	2.80	2.18	2.03	2.61	1.77	1.36	1.01	0.97	0.73	1.08	1.90	**25.19**
<2.0	1.19	0.80	0.48	1.12	0.67	0.63	0.67	1.01	0.95	0.88	0.78	0.69	0.39	0.24	0.76	**11.28**
<2.5	0.52	1.58	0.82	0.71	0.78	0.47	0.52	0.73	0.67	0.39	0.35	0.32	0.13	0.21	0.41	**8.61**
<3.0	0.09	0.80	0.56	0.39	0.50	0.21	0.45	0.41	0.28	0.04	0.04	0.07	0.11	0.06	0.13	**4.14**
<3.5	0.00	0.21	0.71	0.41	0.21	0.06	0.07	0.30	0.11	0.00	0.00	0.02	0.00	0.06	0.07	**2.22**
<4.0	0.00	0.00	0.32	0.26	0.07	0.00	0.04	0.07	0.04	0.00	0.00	0.00	0.00	0.00	0.02	**0.82**
<4.5	0.00	0.00	0.06	0.00	0.02	0.06	0.00	0.04	0.00	0.00	0.02	0.02	0.00	0.00	0.00	**0.21**
<5.0	0.00	0.00	0.00	0.04	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	**0.04**
>5.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
TOT	**3.60**	**5.35**	**11.26**	**15.07**	**10.35**	**7.74**	**7.70**	**8.75**	**5.71**	**4.62**	**3.54**	**3.58**	**2.89**	**3.28**	**6.56**	
	20.21%			25.41%		27.89%				24.48%						

For each directional sector a wave direction and two wave heights are chosen. The selected wave heights correspond to an average and a high wave height.

To calculate the required simulation time for each schematized wave the sediment transport is computed for each of the selected combinations and for each of the wave/height combination of Table 4-7. The sediment transport has been calculated using Delft3D, as the total cross-shore sediment transport (suspended and bed load) that the wave/height combination generates on a typical profile measured at Southampton beach in Long Island. The net sediment transport for each directional sector, $S_{net}(i)$ is calculated using the expression:

$$S_{net}(i) = \sum_j N \times P(j) \times S(j)$$

where:

$S_{net}(i)$ – Total net transport per directional sector "i"
S(j) –Transport of wave height/direction combination "j" per unit time
P(j) – Probability of occurrence of wave height/direction combination "j"
N – Total number of time units of the complete study period

The weighted transport for each of the 8 selected combinations is calculated using the following expressions.

$$S_{av}^{w}(i) = S_{av}(i)\left[\frac{P_{av(i)}}{P_{av}(i) + P_{hi}(i)}\right]$$

or

$$S_{hi}^{w}(i) = S_{hi}(i)\left[\frac{P_{hi(i)}}{P_{av}(i) + P_{hi}(i)}\right]$$

where:

$S^w_{av}(i)$ – Weighted transport of average wave height/direction combination "i" per unit time
$S^w_{hi}(i)$ – Weighted transport of average high height/direction combination "i" per unit time
$S_{av}(i)$ – Transport of average wave height/direction combination "i" per unit time
$S_{hi}(i)$ – Transport of high wave height/direction combination "i" per unit time
$P_{av}(i)$ – Probability of occurrence of average wave height/direction combination "i"
$P_{hi}(i)$ – Probability of occurrence of high wave height/direction combination "i"

The total representative transport for each of the four directional sectors is calculated as the sum of the weighted transports for the average and high wave height/direction combinations times the total time for the sector.

$$S_{rep}(i) = \left[S^w_{av}(i) + S^w_{hi}(i)\right] t_{old}(i)$$

where:

$S_{rep}(i)$ – Weighted representative transport per directional sector
$t_{old}(i)$ – Old simulation time per directional sector

The ratio, R(i), between the total transport per directional sector and the weighted representative transport.

$$R(i) = \frac{S_{net}(i)}{S_{rep}(i)}$$

The new simulation time per directional sector is calculated using:

$$t_{new}(i) = R(i) \times t_{old}(i)$$

The time per directional sector has to be distributed between the average and high combinations using the probability of occurrence of each of them

$$t^{av}_{new}(i) = t_{new}(i)\left[\frac{P_{av(i)}}{P_{av}(i) + P_{hi}(i)}\right]$$

or

$$t^{hi}_{new}(i) = t_{hi}(i)\left[\frac{P_{hi(i)}}{P_{av}(i) + P_{hi}(i)}\right]$$

Table 4-8 presents the values of the different variables for the eight selected combinations including the number of days that each condition has to be simulated in order to obtain the same total net transport for the simulation period. The new net transport per sector using the representative wave conditions can be computed by:

$$S^{new}_{net}(i) = t^{av}_{new}(i)S_{av}(i) + t^{hi}_{new}(i)S_{hi}(i) \approx S_{net}(i)$$

Table 4-8: Shinnecock Inlet Representative Wave Calculation Variables								
Combination	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
Sector	1		2		3		4	
Height (m)	1	3.5	1	3.5	1	3	1	2.5
Direction(^{o})	105		115		145		210	
Period (s)	9	10	9	10	9	10	9	10
S_{net} (m^3)	59,911		57,254		20,113		33,870	
S_{av} or S_{hi} (m^3/s)	0.0045	0.1152	0.0043	0.1059	0.0019	0.0342	0.0044	0.0509
P_{av} or P_{hi} (%)	4.79	0.71	6.25	0.41	2.80	0.45	1.4	0.13
S^w_{av} or S^w_{hi} (m^3/s)	0.0040	0.0148	0.0040	0.0065	0.0016	0.0047	0.0040	0.0043
S_{rep} (m^3)	95,900		67,453		44,572		51,498	
R	0.62		0.85		0.45		0.66	
t_{old} (day)	59.0		74.2		81.4		71.5	
t_{new} (day)	36.9		63.0		36.8		47.0	
t^{av}_{new} or t^{hi}_{new} (day)	32.1	4.7	59.1	3.9	31.7	5.1	43.0	4.0

The simulation of the morphological process consists of running one wave condition after another for the period of time calculated for each of them. The order in which the wave conditions were simulated was 1-3-5-7-2-4-6-8, which is starting from east to west and from the average conditions to the extreme wave conditions. Sensitivity tests showed that although some differences could be observed when a different order was used, the final results were not very sensitive to the selected order of the wave conditions.

Figure 4-22 shows the morphological changes from 28 May 1997 to 13 August 1998 simulated with the input filtering morphological model. Comparison with Figure 4-18 shows that the results are very similar to those obtained with the full description of tides and wave climate. Moreover, comparison with Figure 4-19 indicates that the model does an adequate job reproducing the observed morphological changes. Since the simulation using the input filtering techniques requires significantly less computational effort, and given the results from both cases are very similar, long-term morphological simulations (longer than one year) will use this technique. Short-term simulations during storm conditions (in the order of days) will use the detailed time-series methodology.

4.2.4 Existing Conditions

The following paragraphs describe the observed patterns in the model results as regards hydrodynamics, waves, sediment transport, and morphology at Shinnecock Inlet. Results are based on the calibrated models described earlier in Section 4. Measured rates of sediment transport and morphology changes in the field are presented in Section 5.

Hydrodynamics

Figure 4-23 illustrates the modeled current speed and vectors during a typical peak flood tide at Shinnecock Inlet. Current speeds are relatively low over the ebb shoal (0.3-0.7 m/s) and do not increase significantly until the immediate vicinity of the jetties. In the throat of the inlet currents are strong (over 2 m/s). Currents remain high over the flood shoal (1.0 m/s) and into the channel past the commercial fishing docks and Ponquogue Bridge (0.9 – 1.0 m/s). Because of the relatively shallow bay inside the inlet, the deeper channels attract more flow and consequently have higher currents. Velocities remain lower outside the inlet because flow is drawn from all directions over relatively constant depths. Flow accelerates through the constriction caused by the inlet and the fixed jetties.

Figure 4-24 shows modeled flow patterns during a typical ebb tide. Flow is drawn from the interior of Shinnecock Bay and ejected through the inlet. Flow speeds in the interior channels are similar to flood tide conditions (0.9 – 1.0 m/s). Flow is constricted from the bay into the throat of the inlet. Velocities in the throat are again high (over 2 m/s), but now the flow velocity is maintained out over the ebb shoal as a jet. The values of the ebb velocities are smaller than those during flood. This corresponds to the definition of Shinnecock Inlet as a flood dominated inlet (Militello and Kraus, 2001). The alignment of the jet principally follows the alignment of the deposition basin, skewed a bit to the west, probably due to the offset of the western jetty. Morphological modeling results show that the channel tends to align with the flow, relocating to the west in a more NE-SW alignment, between maintenance dredging projects.

Waves and Sediment Transport Potential

Figure 4-25 to Figure 4-28 display wave shoaling/refraction coefficients and initial sediment transport rates for the four wave sectors delineated for the morphological model input filtering (see Section 4.2.3). For Shinnecock Inlet, the four schematized wave directions are 105°, 115°, 145°, and 210° clockwise from north (Nautical convention) at a depth of 25 m. Each plot shows, in the lower frame, wave refraction/shoaling coefficients (*KrKs*) for a 1-meter, 9-second offshore wave (the wave used as the average morphological wave condition for each principal direction). The top frame of each plot shows the resulting tidally-averaged sediment transport rates. These rates were computed by combining the bottom stresses resulting from currents averaged over the representative tide, wave orbital velocity, and radiation stress-induced currents. The sediment transport rates represent the initial potentials at the beginning of the morphological modeling. Note that in a conventional sediment transport modeling effort, these rates would be extrapolated over time to compute the bed change. In the morphological analysis the rates are altered to account for the bed evolution and its effects on waves and currents.

Figure 4-25 plots the wave patterns and sediment transport patterns resulting from a wave with an offshore direction of 105 degrees. This wave condition occurs approximately 20% of the time. The waves are traveling obliquely to the shoreline. The wave direction vectors in the lower panel of the figure show the nearshore waves are oriented toward the northwest. The plot shows waves breaking along the shoreline east of the inlet and on the eastern jetty. Waves shoal up on the eastern side of the deposition basin, over the east lobe of the ebb shoal. Waves focus on and shoal over the west lobe of the ebb shoal and break on the shoreline west of the inlet. Wave heights are greatly reduced in the throat of the inlet due to sheltering of the eastern jetty and the fast currents in the throat.

The upper panel of Figure 4-25 shows the results of the sediment transport potential due to this wave condition. Longshore transport of sand is very strong on the eastern shoreline due to wave breaking. The oblique angle of incidence of the waves increases the strength of the westward flow. Transport at the eastern jetty is also strong, showing transport into the inlet entrance and the deposition basin. There is strong transport potential over the west lobe of the ebb shoal. Transport vectors are directed along the shoal toward where the ebb shoal welds to the shoreline. On the west side coastline, there is a moderate longshore transport toward the west, from the fillet on the western jetty toward the ebb shoal and from ebb shoal west toward Westhampton Beach. There is also significant transport potential in the throat of the inlet and in the entrance channel/deposition basin and around the west jetty. These potentials are mainly due to the tidal currents. Wave heights in these areas are not great and the water depths are typical large (>15 feet). The depth averaged tidal currents, however, are strong (Figure 4-23 and Figure 4-24). Tidal average transport potentials in the channel are directed outward south of the west jetty and inward north of the jetty. The strength of the potentials lessens away from the throat of the inlet. It is expected that some deposition may occur in these areas along the gradient of the potential.

Figure 4-26 shows modeled wave patterns and transport potentials for waves arriving from 115° (ESE). Wave patterns are similar to those from Figure 4-25. A notable difference is that waves from this angle strike the coastline west of the inlet more perpendicularly. The resulting longshore transport is weaker westward from the inlet. Transport potential between the deposition basin and the west lobe of the ebb shoal is stronger.

Figure 4-27 reports modeled wave patterns and transport potentials for a wave direction of 145° (SE). Waves arrive nearly perpendicular to the offshore contours. Wave coefficients are greater, since waves largely do not refract. Because of the normal approach, the longshore transport travels both east and west along the coastline. Nodal points develop on either side of the inlet, coinciding to the points where the ebb shoal meets the shoreline. Outward of the nodal points, longshore transport is away from the inlet. Inside the nodal points, transport is toward the inlet.

Figure 4-28 shows modeled wave patterns and sediment transport potentials for waves from 210° (SW). Waves point at an eastward angle to the coast in this orientation. Strong eastward longshore transport occurs on the shoreline on both sides of the inlet. Transport on the west lobe of the ebb shoal occurs closer to shore and is directed more toward the inlet. Transport along the bottom of the deposition basin is strong from the east jetty to the west side of the basin, before decreasing in intensity. Sediments transported away from the jetty are expected to deposit on east side of the deposition basin.

4.3 Moriches Inlet

4.3.1 Hydrodynamics

<u>*Model Grid*</u>

The Moriches Inlet Model (MIM) extends from Smith Point to the Quantuck canal at Westhampton beach. The model covers approximately 13 miles west to east. From the inlet mouth, the model extends about 3.3 miles offshore.

MIM was constructed similarly to SIM, using a curvilinear grid and variable grid resolution throughout the model domain. As with SIM, the highest resolution is found at the inlet throat: 12 m grid spacing along the axis of the inlet and 17 m spacing across the throat. Along the coast on either side of the inlet the grid size is on the order of 10-20 m. The grid size in the alongshore direction increases with distance from the inlet, reaching values of up to 355 m. The grid size also increases incrementally from the coast to the offshore boundary up to maximum spacing of 560 m. In total, MIM has a total of 8,703 grid cells. Figure 4-29 shows the MIM computational grid and Figure 4-30 shows a detail view of the model grid at the vicinity of the inlet.

<u>*Model Bathymetry*</u>

Two sources of data were used to construct the bathymetry for the Moriches Model. GEODAS data, as explained in section 4.2.1, was used to construct the entire model domain excluding the inlet where more recent SHOALS data was used. Table 4-9 shows the data block and the area where it was applied. Figure 4-31 shows the bathymetry of MIM.

Table 4-9: Bathymetric Data Sources for the Moriches Inlet Model		
Data Source	Year	Area Specific
SHOALS	1996	Inlet
H05324	1933	Eastern part of the Moriches Back Bay – From the inlet to Westhampton Beach
H05322	1933	Western part of the Moriches Bay – from Smith Point Bridge to the Inlet
H06328	1938	Off-Shore area
ACNYMP	1998	Westhampton and Fire Island Beach profiles were used to generate more accurate bathymetry for the near shore areas.

Hydrodynamic Data

LISHORE stations P6 (Moriches Coast Guard Station) and P7 (Smith Point Bridge) were primarily used for the hydrodynamic model calibration for MIM. Water level time series data were available commencing from April 2000.

Tidal datums have been computed over the period of record for the LISHORE gauges for these two stations, P6 and P7. The results of the analysis are presented in Table 4-10.

Table 4-10: Tidal Datums (ft) for Moriches Tide Gauges		
Datum	Coast Guard P6 (04/00 – 11/01)	Smith Point P7 (04/00 – 11/01)
MHHW	2.61	1.48
MHW	2.39	1.33
MTL	1.24	0.71
MLW	0.10	0.09
MLLW	0.00	0.00

However, for calibration purposes, the tidal harmonic constituents were extracted for the data at P6 and P7 and new time series were re-generated based on 9 constituents. These 9 constituents are the same as those used to define the boundary of the model and are explained in more detail in the following section. The tidal constituents are presented in Table 4-11.

The locations of the hydrodynamic data stations for Moriches Bay are shown in Figure 4-32.

Boundary Conditions

MIM is forced by water surface elevation from the three open boundary conditions located offshore. The boundary is based on a time series of water surface elevations constructed from 9 major tidal constituents extracted from the high-resolution ADCIRC EastCoast 2001, finite-element tidal model (Luettich, et al, 1995). Table 4-12 lists the amplitude and phase for each constituent at two points along the boundary: the eastern edge, and the western edge. Linear interpolation between points is applied along the offshore boundary, while the water surface is non-sloping on the east and west. These boundaries are used for the model calibration as well as for the long-term morphological and the calculation of the representative tidal boundary.

Table 4-11: Calibration Station Tide Constituents – Moriches Model				
	P6 – Moriches Coast Guard		P7 – Smith Point Bridge	
Constituents	Amp (m)	Phase (deg)	Amp (m)	Phase (deg)
O1	0.0375	201.84	0.0268	239.81
K1	0.0642	193.57	0.0447	234.59
N2	0.0625	9.75	0.0292	56.80
M2	0.3071	25.23	0.1591	74.76
S2	0.0499	57.60	0.0239	104.02
K2	0.0134	54.23	0.0064	112.13
Q1	0.0065	216.04	0.0028	239.98
M4	0.0097	2.16	0.0081	40.76
M6	0.0072	118.64	0.0025	143.63

Table 4-12: Boundary Tide Constituents - Moriches Model				
	Eastern BC		Western BC	
Constituents	Amp (m)	Phase (deg)	Amp (m)	Phase (deg)
O1	0.0516	177.46	0.0538	175.08
K1	0.086	166.47	0.0881	166.47
N2	0.1135	329.63	0.1195	329.12
M2	0.4895	342.16	0.5177	341.89
S2	0.1001	3.42	0.1052	3.11
K2	0.0224	14.79	0.0236	14.18
Q1	0.0099	169.75	0.01	167.71
M4	0.011	353.93	0.0077	3.63
M6	0.0134	214.73	0.0132	227.14

Constituents extracted from EastCoast 2001 model (Luettich, et al, 1995)

Calibration

The hydrodynamic model was calibrated for the period 20 April 2000 to 15 May 2000. The calibration result plots are shown in Figure 4-33 to Figure 4-34. Comparisons to current and discharge were not possible as there were no measurements in this model area. Simulated discharge and currents are presented in Figure 4-35.

Model skill is assessed using the following three error calculations: correlation coefficient, root mean square (RMS) error and percent error. For a full description of the statistical parameters, see Section 4.2.1. The three statistics for calibration at P6 and P7 are presented in Table 4-13.

Table 4-13: Calibration Statistics for Moriches Model

Station	Correlation	RMS Error (m)	Percent Error
Moriches Coast Guard – P6	0.99	0.04	4.9
Smith Point Bridge – P7	0.96	0.05	11.5

In general, model predicted water levels in the two available measurement locations are in very good agreement with the data.

4.3.2 Waves

The Moriches wave model has been built using the HISWA model (See Section 4.2.2 for a description of this model).

Local Wave Models within Morphological Model

Similarly to the Shinnecock wave model, the Moriches wave model uses three nested wave grids (see Figure 4-36). The first grid (offshore grid) is the coarsest and has a resolution of 50 m. in the direction of the wave propagation (x-direction) and 200 m. in the direction perpendicular to the wave propagation (y-direction). Only one offshore wave grid has been used in this case, since this grid is able to resolve all the incoming representative wave direction calculated from the input filtering technique. The grid orientation in this case is perpendicular to the 155° azimuth direction. The offshore grid propagates the incoming waves from the upwave boundary condition, located at a depth of approximately 25 meters (82 feet) to the finer grids, thereby providing boundary conditions for the next grid.

The second level of grid resolution (nearshore grid) has a resolution of 25 m in the x-direction and 100 m in the y-direction. This grid propagates boundary conditions obtained from the offshore grid and calculates the wave conditions in the nearshore zone. The orientation of the nearshore grid is perpendicular to the 159° azimuth direction.

A third grid (inlet grid) provides even higher resolution at the inlet area. The inlet grid has a resolution of 5 m in the x-direction and 20 m in the y-direction. The upwave boundary condition of the inlet grid is perpendicular to the 167° azimuth direction. The finer grid permits the calculation of wave breaking and shoaling in the inlet entrance.

Wave Data

No wave data were available near Moriches Inlet. The two wave gauges already described for Shinnecock inlet were used: the National Data Buoy Center (NDBC) buoy 44025 at a water depth of approximately 40 meters and the Westhampton beach gauge (NY001) at a depth of 9.5 meters. The location of the data gauges is shown in Figure 4-15.

Boundary Conditions

The model boundaries at a depth of 25 m have been generated from the output of the regional wave transformation model using NDBC 44025 data and based on an analysis similar to that used for the Shinnecock Inlet Model.

4.3.3 Sediment Transport & Morphology

<u>Model Set-up</u>

The sediment characteristics applied in SIM were also used for MIM

<u>Model Calibration through Input Filtering</u>

Morphological changes at Moriches Inlet were simulated using input filtering techniques similar to those described for Shinnecock Inlet. Specifically, the same procedure that was applied to calculate the representative tide and waves at Shinnecock Inlet has been used here.

The selected calibration period was October 1998 (post-dredge conditions) to July 2000. Representative waves were computed for this period according to the same procedure applied at Shinnecock Inlet. Specifically, the frequency distributions for each 10°-sector and for the four selected multidirectional sectors are presented in Table 4-14.

Table 4-14: Frequency distribution of wave direction versus wave height. Moriches Inlet (1998-00 period)

	<90	<100	<110	<120	<130	<140	<150	<160	<170	<180	<190	<200	<210	<220	>220	TOT
<0.5	0.37	0.28	1.11	2.19	2.09	1.65	1.53	1.05	0.74	0.26	0.21	0.34	0.22	0.11	0.20	12.35
<1.0	1.71	1.36	3.32	4.55	4.18	3.59	3.04	2.82	2.68	3.29	3.17	2.56	2.19	1.44	1.61	41.51
<1.5	1.79	1.45	1.54	2.32	1.91	1.61	1.92	1.75	1.96	2.81	2.51	1.93	1.73	1.23	1.35	27.80
<2.0	0.44	0.63	0.43	0.83	0.41	0.73	0.82	0.74	1.01	1.02	0.75	0.65	0.80	0.36	0.67	10.29
<2.5	0.23	0.47	0.49	0.33	0.36	0.32	0.64	0.58	0.25	0.32	0.32	0.32	0.15	0.20	0.32	5.28
<3.0	0.04	0.42	0.34	0.18	0.08	0.04	0.23	0.23	0.04	0.04	0.07	0.07	0.04	0.04	0.14	2.00
<3.5	0.00	0.02	0.08	0.02	0.03	0.01	0.09	0.12	0.02	0.01	0.04	0.00	0.00	0.03	0.04	0.49
<4.0	0.00	0.00	0.00	0.00	0.00	0.00	0.04	0.04	0.04	0.02	0.01	0.01	0.00	0.00	0.00	0.14
<4.5	0.00	0.00	0.00	0.00	0.01	0.02	0.01	0.02	0.03	0.01	0.00	0.00	0.00	0.00	0.00	0.09
<5.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.02
>5.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.03
TOT	4.57	4.62	7.32	10.43	9.07	7.97	8.31	7.36	6.77	7.78	7.07	5.88	5.13	3.40	4.34	
	26.94%			25.34%		21.90%				25.81%						

Table 4-15 presents the values of the different variables for the eight selected combinations including the number of days that each condition has to be simulated in order to obtain the same total net transport for the simulation period.

Table 4-15: Moriches Inlet Representative Wave Calculation Variables								
Combination	1	2	3	4	5	6	7	8
Sector	1		2		3		4	
Height (m)	1	3	1	3	1	3	1	2.5
Direction(^{o})	110		135		165		210	
Period (s)	9	10	9	10	9	10	9	10
S_{net} (m^3)	83,733		56,626		14,516		75,212	
$S_{av\ or}\ S_{hi}$ (m^3/s)	0.0045	0.0787	0.0028	0.0516	0.0009	0.0179	0.0044	0.0509
$P_{av\ or}\ P_{hi}$ (%)	3.32	0.34	3.59	0.04	2.68	0.04	2.19	0.15
$S^w_{av\ or}\ S^w_{hi}$ (m^3/s)	0.0040	0.0074	0.0028	0.0006	0.0009	0.0003	0.0041	0.0032
S_{rep} (m^3)	177,121		50,804		15,192		109,476	
R	0.47		1.11		0.96		0.69	
t_{old} (day)	180.5		169.8		146.7		172.9	
t_{new} (day)	85.3		189.3		140.2		118.8	
$t^{av}_{new\ or}\ t^{hi}_{new}$ (day)	77.4	8	187.0	2.3	138.0	2.3	111.2	7.6

As described for Shinnecock Inlet, the simulation of the Moriches Inlet morphological process consists of running one wave condition after another for the period of time calculated for each of them. The order in which the wave conditions were simulated was 1-3-5-7-2-4-6-8, which is, starting from east to west and from the average conditions to the extreme wave conditions. Figure 4-37 and Figure 4-38 show morphological changes from October 1998 to May 2000 as measured with the available surveys and SHOALS data and simulated with the morphological model, respectively.

The morphological model is able to reproduce most of the most important morphological changes observed in Figure 4-37. Deposition on the navigation channel and on the east and offshore areas of the ebb shoal is well reproduced by the model. The model also reproduces the displacement westwards of the shoal located west of the channel. Areas were the model that may require additional improvement are the deposition basin, where the model seems to accumulate less material than observed and the channel throat. Overall, results suggest that the model may be used to investigate short- and long-term impacts of any proposed inlet modifications.

4.3.4 Existing Conditions

The following paragraphs describe the observed patterns in the model results as regards hydrodynamics, waves, sediment transport, and morphology at Moriches Inlet. Results are based on the calibrated models described earlier in Section 4. Measured rates of sediment transport and morphology changes in the field are presented in Section 5.

<u>*Hydrodynamics*</u>

Figure 4-39 and Figure 4-40 present current patterns and velocities for the peak flood and peak ebb tide, respectively, during the representative morphological tide. Hydrodynamic patterns are similar to those of Shinnecock inlet. The inlet throat experiences high currents (1.0 – 2.0 m/s) on both flood and ebb tide. Similarly to Shinnecock inlet, maximum flood velocities are larger than maximum ebb. The velocities in the interior channels are higher during ebb tide, while during flood the incoming flow spreads out over the flood shoal at about 1.0 m/s. Currents over the ebb shoal on the flood tide are lower (0.5 m/s) than during the ebb tide jet (0.9 – 1.3 m/s).

<u>*Waves and Sediment Transport Potential*</u>

Figure 4-41 to Figure 4-44 show wave and sediment transport patterns for waves arriving from ESE to SW. Patterns are generally similar to those for Shinnecock Inlet, with strong westward longshore transport east and west of the inlet for waves arriving from 110° and 135°. Node points in longshore transport form for waves arriving from 165°, and longshore transport shifts to the east for waves arriving from the SW.

Waves break over the west lobe of the ebb shoal from all wave directions. Transport potentials over the shoal are active, with several areas of high potential and large gradients. Transport vectors along the edge of the deposition basin are generally southward for all wave directions, further westward vectors are in towards shore and along the crest of the shoal, and vectors are directed offshore and westward nearer to the shore.

An interesting feature of Moriches Inlet is that the shoreline between west jetty and the point where the west lobe of the ebb shoal attaches to the shore is oriented normal to waves arriving from the SE. The longshore transport in this area is low for SE waves, and the shoreline is likely in equilibrium with the predominant wave direction.

For SW waves, the refracted wave vectors are oriented parallel with the deposition basin. Waves breaking over the east lobe of the ebb shoal direct sediment transport toward the east side of the inlet, reversing the direction of transport predominant from the SE and S waves.

Transport potential in the inlet is driven mainly by the average tidal currents. In contrast with Shinnecock where transport inside the jetties is directed inward and outside directed outward, transport at Moriches is directed inward on the western side of the throat and outward on the eastern side.

4.4 Fire Island Inlet

4.4.1 Hydrodynamics

<u>*Model Grid*</u>

The Fire Island Inlet Model (FIIM) extends from the Great Island in the west of the Great South Bay to Smith Point in the east. The model covers approximately 35 miles west to east in the back bay and about 18 miles offshore. From the inlet mouth, the model extends about 8.5 miles offshore.

FIIM was constructed similarly to SIM and MIM, using a curvilinear grid and variable grid resolution throughout the model domain. The highest resolution is found at the inlet throat: 35 m grid spacing along the axis of the inlet and 40 m spacing across the throat. Along the coast on either side of the inlet the grid size is on the order of 50-60 m. The grid size in the alongshore direction increases with distance from the inlet, reaching values of up to 800 m. The grid size also increases incrementally from the coast to the offshore boundary up to maximum spacing of 500 m. In total, FIIM has a total of 14,816 grid cells. Figure 4-45 shows the overall computational grid for Great South Bay, Figure 4-46 shows the resolution of the model at Fire Island Inlet.

<u>*Model Bathymetry*</u>

A number of sources of data were used to construct the bathymetry for the Fire Island Model. GEODAS data, as explained in Section 4.2.1, was used to construct the entire model domain excluding the inlet where more recent data were used. Table 4-16 shows the data sources and the extent of the coverage. Figure 4-47 shows the bathymetry of the Great South Bay model used for the hydrodynamic calibration and the morphological model. This bathymetry, representative of the beginning of 2001, was constructed using the 2001 condition survey, SUNY survey 2001-2002, Spring 2001 profiles from the Atlantic Coast of New York Monitoring Program and GEODAS data.

Table 4-16: Bathymetric Data Sources for the Fire Island Inlet Model

Data Source	Year	Area Specific
Condition Survey	March-2001	Inlet Throat
SHOALS	1996	Inlet Periphery
GEODAS	1933-1938	Great South Bay – from great Island to Smith Point
GEODAS	1933-1938	Off-Shore area
Atlantic Coast of New York Monitoring Program	Spring 2001	Profiles were used to generate more accurate bathymetry for the near shore areas.
SUNY survey	Dec 2001 – March 2002	Inlet Throat and Ebb shoal

Hydrodynamic Data

LISHORE station P8 (Fire Island Coast Guard Station) was primarily used for the hydrodynamic model calibration of FIIM. Water level time series data were available commencing from January 2000.

Tidal datums have been computed over the period of record for the P8 LIShore gauge. The results of the analysis are presented in Table 4-17.

Table 4-17: Tidal Datums (ft) for Fire Island Tide Gauges

Datum	Coast Guard P6 (01/00 – 11/01)	WES TG6
MHHW	2.30	0.98
MHW	2.14	0.87
MSL	1.15	0.46
MTL	1.11	0.46
MLW	0.09	0.05
MLLW	0.00	0.00

Table 4-18: Calibration Station Tide Constituents – Fire Island Inlet Model

	P8 – Fire Island Coast Guard		WES – TG6	
Constituents	Amp (m)	Phase (deg)	Amp (m)	Phase (deg)
O1	0.0300	194.3	0.02133	174.7
K1	0.0560	190.4	0.03657	169.2
N2	0.0632	341.2	0.02438	290.7
M2	0.2859	4.8	0.1250	311.9
S2	0.0502	27.2	0.00610	12.7
K2	0.0156	23.3	0.01219	80.1
Q1	0.0079	177.8	-	-
M4	0.0062	235.8	-	-
M6	0.0028	177.5	-	-

For calibration purposes, the tidal harmonic constituents were extracted for the data P8 and new time series were re-generated based on 9 constituents. These 9 constituents are the same as those used to define the boundary of the model and are explained in more detail in the following section. A second station, TG6, was used to assess the model calibration in Great South Bay. The time series for TG6 was constructed from tidal constituents calculated by WES based on measurements at the TG6 location. The tidal constituents for both stations are presented in Table 4-18.

The locations of the hydrodynamic data stations for Great South Bay are shown in Figure 4-48.

Boundary Conditions

FIIM is forced by water surface elevation from the semicircular open boundary conditions located offshore. The boundary is based on a time series of water surface elevations constructed from 9 major tidal constituents extracted from the high-resolution ADCIRC EastCoast 2001, finite-element tidal model (Luettich, et al, 1995). Table 4-19 lists the amplitude and phase for each constituent at three points along the boundary: the eastern edge, south-edge, and the western edge. Linear interpolation between points is applied on the boundaries.

Open boundary conditions are specified at the east (Smith Point) and west (Great Beach) limits of the model. At the east, the model has been forced with the tide generated from constituents computed from LISHORE P7 data. The western boundary has been forced with tides generated for the WES TG4 station. Table 4-20 lists the amplitude and phase for each constituent at these two Great South Bay boundaries.

Table 4-19: Offshore Boundary Tide Constituents – Fire Island Inlet Model

Constituents	East BC		South BC		West BC	
	Amp (m)	Phase (deg)	Amp (m)	Phase (deg)	Amp (m)	Phase (deg)
O1	0.0574	173.4	0.0599	174.06	0.0607	172.86
K1	0.091	167.18	0.0921	168.26	0.0935	168.16
N2	0.1284	329.76	0.1321	331.08	0.1368	330.94
M2	0.5584	342.75	0.5751	344.18	0.5966	344.01
S2	0.1131	4.01	0.1163	5.47	0.1207	5.36
K2	0.0256	14.72	0.0265	16.02	0.0275	15.64
Q1	0.0101	165.63	0.0104	164.97	0.0103	164.49
M4	0.0033	44.23	0.0023	111.14	0.0044	139.58
M6	0.0116	255.63	0.0094	280.01	0.0109	302.92

These boundaries are used for the model calibration as well as for the long-term morphological and the calculation of the representative tidal boundary.

Table 4-20: Great South Bay Boundary Tide Constituents Fire Island Model				
	Eastern Boundary at Smith Point (P7)		Western Boundary at Great Beach (TG4)	
Constituents	Amp (m)	Phase (deg)	Amp (m)	Phase (deg)
O1	0.0268	239.8	0.03353	123.6
K1	0.0447	234.6	0.08839	119.1
N2	0.0292	56.8	0.08229	228.1
M2	0.1591	74.8	0.4143	242.2
S2	0.0239	104.0	0.0213	308.8
K2	0.0064	112.1	0.05486	12.8
Q1	0.0028	240.0	-	-
M4	0.0081	40.8	-	-
M6	0.0025	143.6	-	-

Constituents extracted from EastCoast 2001 model (Luettich, et al, 1995)

<u>*Calibration*</u>

The hydrodynamic model was calibrated for the period 20 April 2000 to 15 May 2000. The calibration result plots are shown in Figure 4-49 and Figure 4-50. As there are no current and discharge measurements in this model area, comparisons between modeled and observed currents were not possible. Simulated discharge and currents are presented in Figure 4-51.

Model skill is assessed using the following three error calculations: correlation coefficient, root mean square (RMS) error and percent root mean square error (rms). For a full description of the statistical parameters, see Section 4.2.1. The three statistics for simulation results at P8 and WES TG6 are presented in Table 4-21.

Table 4-21: Simulation Statistics for Fire Island Inlet Model			
Station	Correlation	RMS Error (m)	Percent Error
Fire Island Coast Guard – P8	0.99	0.035	4.98
WES: TG6	0.97	0.025	8.42

Results presented in Table 4-21 show that the model predicts correctly the water levels at the available observations points, and it can therefore be assumed that it is able to accurately reproduce the tidal variability at Fire Island inlet and Great South Bay.

4.4.2 Waves

The Fire Island Inlet wave model has been built using the HISWA model (see Section 4.2.2).

<u>*Local Wave Models within Morphological Model*</u>

The Fire Island wave model uses three nested wave grids into an offshore grid (see Figure 4-52). The offshore grid is the coarsest and has a resolution of 50 m in the direction of the wave propagation (x-direction) and 200 m in the direction perpendicular to the wave propagation (y-direction). Only one offshore wave grid has been used in this case, since this grid is able to resolve all the incoming representative wave direction calculated from the input filtering technique. The grid orientation in this case is perpendicular to the 165° azimuth direction. The offshore grid is used to propagate the incoming waves from the upwave boundary condition, located at a depth of approximately 25 meters (82 feet) to the finer grids, thereby providing boundary conditions for the next grid.

The second level of grid resolution includes two nearshore grids and an inlet grid, all with a resolution of 25 m in the x-direction and 50 m in the y-direction. This grid propagates boundary conditions obtained from the offshore grid and calculates the wave conditions in the nearshore zone and at the inlet throat. The wave boundary condition is perpendicular to the 165° and 162° azimuth direction for the east and west nearshore grids respectively. The inlet grid has enough resolution to resolve the bathymetric features that are represented in that area in the hydrodynamic model. The orientation of the inlet grid is perpendicular to the 177° azimuth direction.

<u>*Wave Data*</u>

At this time, no wave data were available in the vicinity of Fire Island Inlet. Only the National Data Buoy Center (NDBC) buoy 44025 at a water depth of approximately 40 meters, already described for SIM. The location of the data gauge is shown in Figure 4-15.

<u>*Boundary Conditions*</u>

The model boundaries at a depth of 25 m have been generated from the output of the regional wave transformation model using NDBC 44025 data and based on an analysis similar to that used for the Shinnecock Inlet Model.

4.4.3 Sediment Transport & Morphology

<u>*Model Set-up*</u>

Sediment characteristics applied in SIM and MIM were used for FIIM.

<u>*Model Calibration through Input Filtering*</u>

Morphological changes at Fire Island Inlet were simulated using input filtering techniques similar to those described for Shinnecock Inlet. Representative tide and waves were used to predict long-term morphological changes.

The selected calibration period was March 2001 to December 2001. Representative waves were computed for this period according to the same procedure applied at Shinnecock Inlet. Specifically, the frequency distribution for each 10°-sector and for the four selected multidirectional sectors are presented in Table 4-22.

Table 4-22: Frequency distribution of wave direction versus wave height. Fire Island Inlet (2001 period)																
	<90	<100	<110	<120	<130	<140	<150	<160	<170	<180	<190	<200	<210	<220	>220	TOT
<0.5	0.38	0.36	0.76	1.60	2.02	1.76	0.60	0.58	0.25	0.15	0.31	0.25	0.18	0.15	0.29	9.66
<1.0	1.22	1.09	3.78	5.04	5.24	5.44	4.44	3.80	2.06	1.95	2.16	2.78	3.06	2.27	1.78	46.11
<1.5	1.71	1.33	1.60	5.29	4.98	1.47	0.76	1.98	1.76	1.26	1.47	1.62	1.58	1.02	1.71	29.56
<2.0	0.49	1.07	0.51	1.22	1.07	0.24	0.02	0.31	0.78	0.93	1.00	0.58	0.38	0.60	0.95	10.15
<2.5	0.11	0.42	0.20	0.11	0.13	0.02	0.02	0.05	0.16	0.02	0.11	0.29	0.11	0.09	0.82	2.66
<3.0	0.00	0.07	0.24	0.20	0.09	0.04	0.00	0.00	0.09	0.00	0.00	0.04	0.02	0.02	0.13	0.93
<3.5	0.00	0.04	0.22	0.09	0.05	0.13	0.00	0.00	0.05	0.04	0.00	0.02	0.00	0.00	0.02	0.65
<4.0	0.00	0.00	0.00	0.00	0.02	0.05	0.04	0.00	0.00	0.02	0.00	0.00	0.00	0.00	0.00	0.13
<4.5	0.00	0.00	0.00	0.02	0.00	0.00	0.07	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.09
<5.0	0.00	0.00	0.00	0.02	0.00	0.02	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.05
>5.0	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02
TOT	3.91	4.38	7.31	13.59	13.60	9.17	5.98	6.73	5.17	4.35	5.06	5.58	5.33	4.15	5.69	
	29.19%			22.77%		22.23%				25.81%						

Table 4-23 presents the values of the different variables for the eight selected combinations including the number of days that each condition has to be simulated in order to obtain the same total net transport for the simulation period.

Table 4-23: Fire Island Inlet Representative Wave Calculation Variables								
Combination	1	2	3	4	5	6	7	8
Sector	1		2		3		4	
Height (m)	1	3	1	2.5	1	2.5	1	2
Direction(^{o})	110		130		160		210	
Period (s)	9	9	9	9	9	9	9	9
S_{net} (m^3)	40,702		22,687		837		30,546	
$S_{av\ or}\ S_{hi}$ (m^3/s)	0.0045	0.0787	0.0033	0.0378	0.0002	0.0267	0.0044	0.0291
$P_{av\ or}\ P_{hi}$ (%)	3.78	0.24	5.24	0.13	3.8	0.05	3.06	0.38
$S^w_{av\ or}\ S^w_{hi}$ (m^3/s)	0.0041	0.0045	0.0032	0.0009	0.0002	0.0001	0.0039	0.0032
S_{rep} (m^3)	63,517		23,480		1,316		46,139	
R	0.64		0.97		0.64		0.66	
t_{old} (day)	84.9		66.3		64.7		75.1	
t_{new} (day)	54.4		64.0		41.2		49.7	
$t^{av}_{new\ or}\ t^{hi}_{new}$ (day)	51.2	3.2	62.5	1.5	40.6	0.6	44.2	5.5

As it was already described for Shinnecock and Moriches inlets, the simulation of the Fire Island inlet morphological process consists of running one wave condition after another for the period of time calculated for each of them. The order in which the wave conditions were simulated was 1-3-5-7-2-4-6-8, which is, starting from east to west and from the average conditions to the extreme wave conditions. Figure 4-53 shows morphological changes between the condition survey of March 2001 and the SUNY survey of December 2001- February 2002. A limited number of bathymetric data sets were available at Fire Island inlet, as shown in Table 4-16 compared with Shinnecock and Moriches inlets. The morphological changes for the aforementioned period were only calculated for the extent of the March 2001 which has a much smaller extent that the SUNY survey. Figure 4-53 shows small changes throughout the channel and the deposition basin, with the exception of the area located northwest of the navigation channel where significant accretion is observed. Areas of significant erosion along the navigation channel are the consequence of the dredging events that took place while the SUNY survey was being conducted. Figure 4-54 presents the morphological changes simulated with the model corresponding to the same period.

Comparison of Figure 4-53 and Figure 4-54 show that model does a good job in representing the main morphological features that can be observed in the data: small changes in general possibly indicating ebb shoal maturity and significant deposition north-west of the navigation channel. Note that the aforementioned dredging event observed in the measurements was not included in the model.

4.4.4 Existing Conditions

The following paragraphs describe the observed patterns in the model results as regards hydrodynamics, waves, sediment transport, and morphology at Fire Island Inlet. Results are based on the calibrated models described earlier in Section 4. Measured rates of sediment transport and morphology changes in the field are presented in Section 5.

<u>*Hydrodynamics*</u>

Figure 4-55 and Figure 4-56 present current patterns and velocities for the peak flood and peak ebb tide, respectively, during the representative morphological tide. The character of Fire Island Inlet is very different from the other two inlets. Fire Island is much older than Moriches or Shinnecock. The inlet is oriented east-west instead of north-south. Velocities are higher through the throat and interior channel during flood tide than during ebb tide. Because the throat of the inlet is wider than either Moriches of Shinnecock, peak velocities are lower (1.5 m/s). Velocities over the ebb shoal are higher during ebb than flood, but the velocity vectors fan out over the shoal more than in the other inlets because the deposition basin is not oriented with the ebb flow.

<u>*Waves and Sediment Transport Potential*</u>

Figure 4-57 to Figure 4-60 show wave and sediment transport patterns for waves arriving from ESE to SW. Waves and sediment transport along the shoreline east of the inlet behaves similarly to the other two inlets. For waves from ESE to S, longshore sediment transport is directed eastward. For SW waves longshore transport is eastward.

In the mouth of the inlet and west of the inlet, the transport patterns are different from the other inlets. From the tip of the Federal jetty to the end of Democrat Point, longshore transport at the shoreline is always directed into the inlet. Waves from all directions breaking on this segment of shore direct transport inward; this is reinforced by the direction of the tidal current during flood tide. This segment of shore is protected from tidal currents during ebb tide. This may explain the rapid shoaling in the deposition basin and the growth of Democrat Point.

In the throat of the inlet, transport potentials are negligible. This is due to the lower average velocities in the throat. This indicates that there is likely little sediment exchange through the inlet.

West of the inlet, there is little longshore transport except during SW waves when there is a moderate transport potential toward the inlet. Between the point where the west side of the ebb shoal welds to shore and the northern jetty, there is a mild longshore return transport toward the jetty. These results indicate that the shoreline of Fire Island west of the inlet appears to be in equilibrium with the predominant wave direction. The transport potentials over the ebb shoal are much milder than in the other two inlets and follow generally the orientation of the average tidal currents regardless of wave direction. This would seem to indicate a slow outward growth of the ebb shoal, but that the shoal is more or less in equilibrium with the wave climate.

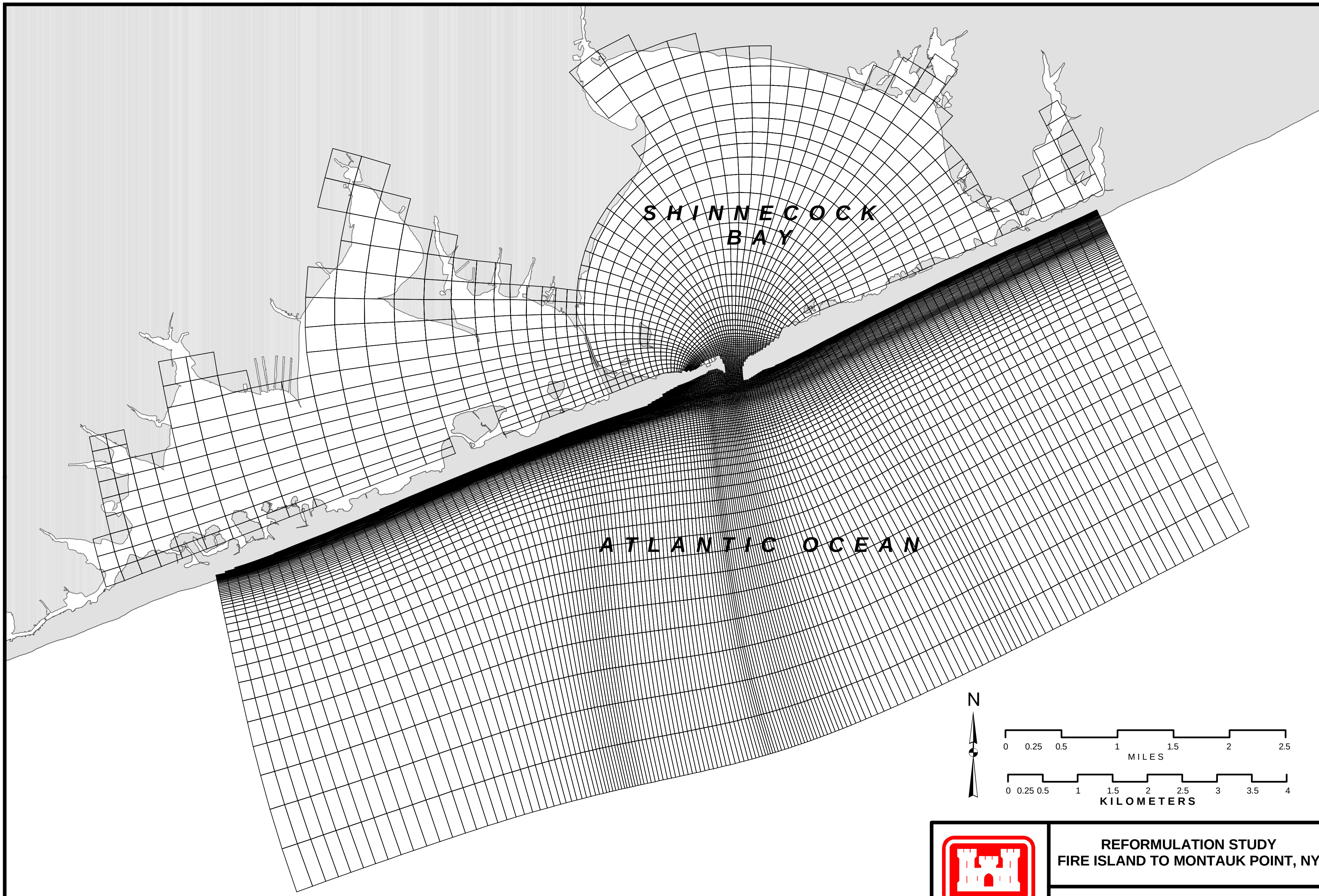


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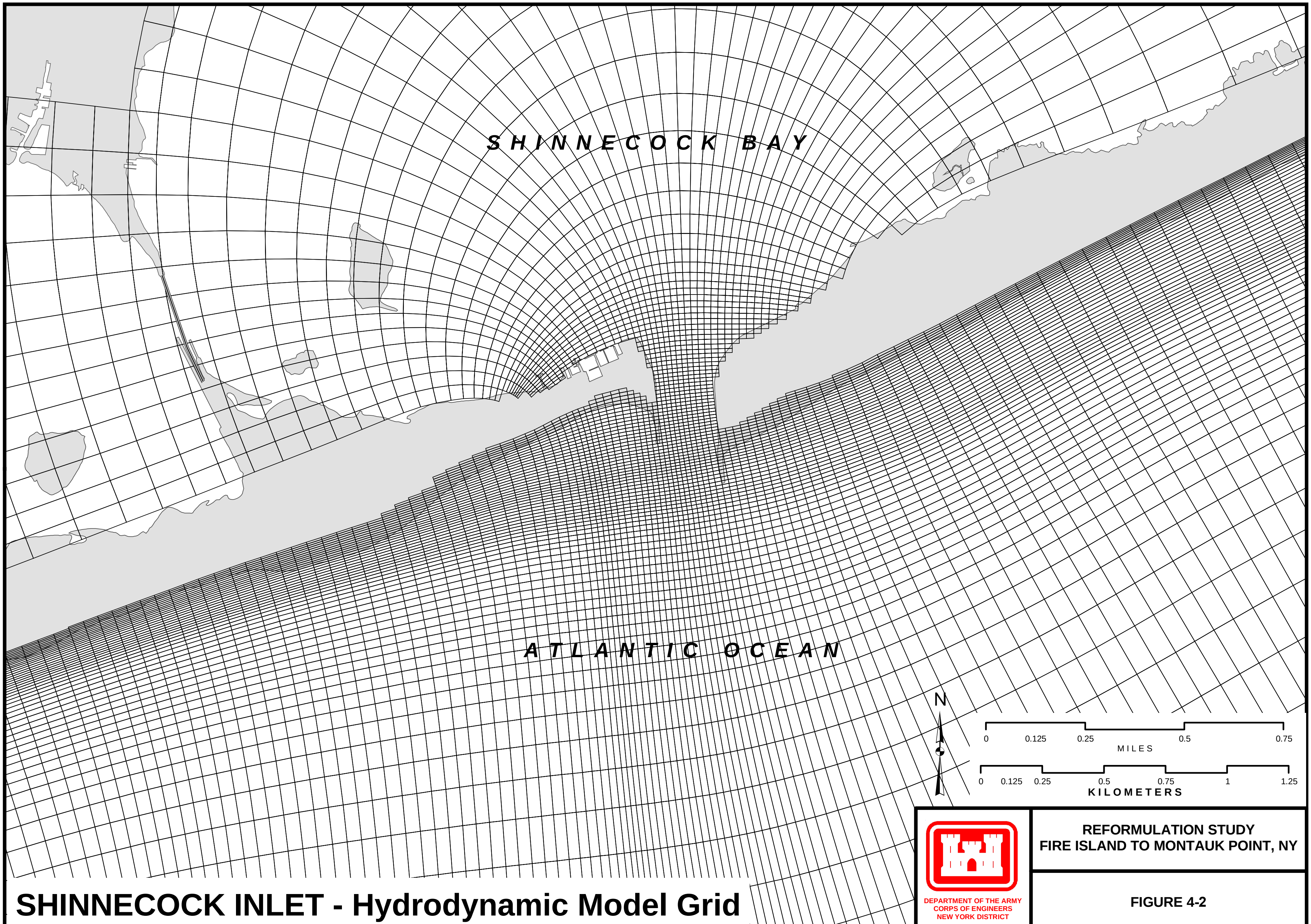
FIGURE 4-1

SHINNECOCK BAY - Hydrodynamic Model Grid

Filename: CAD/3982-23/Report2002/Shinn_Model_Grids.mxd



SHINNECOCK INLET - Hydrodynamic Model Grid

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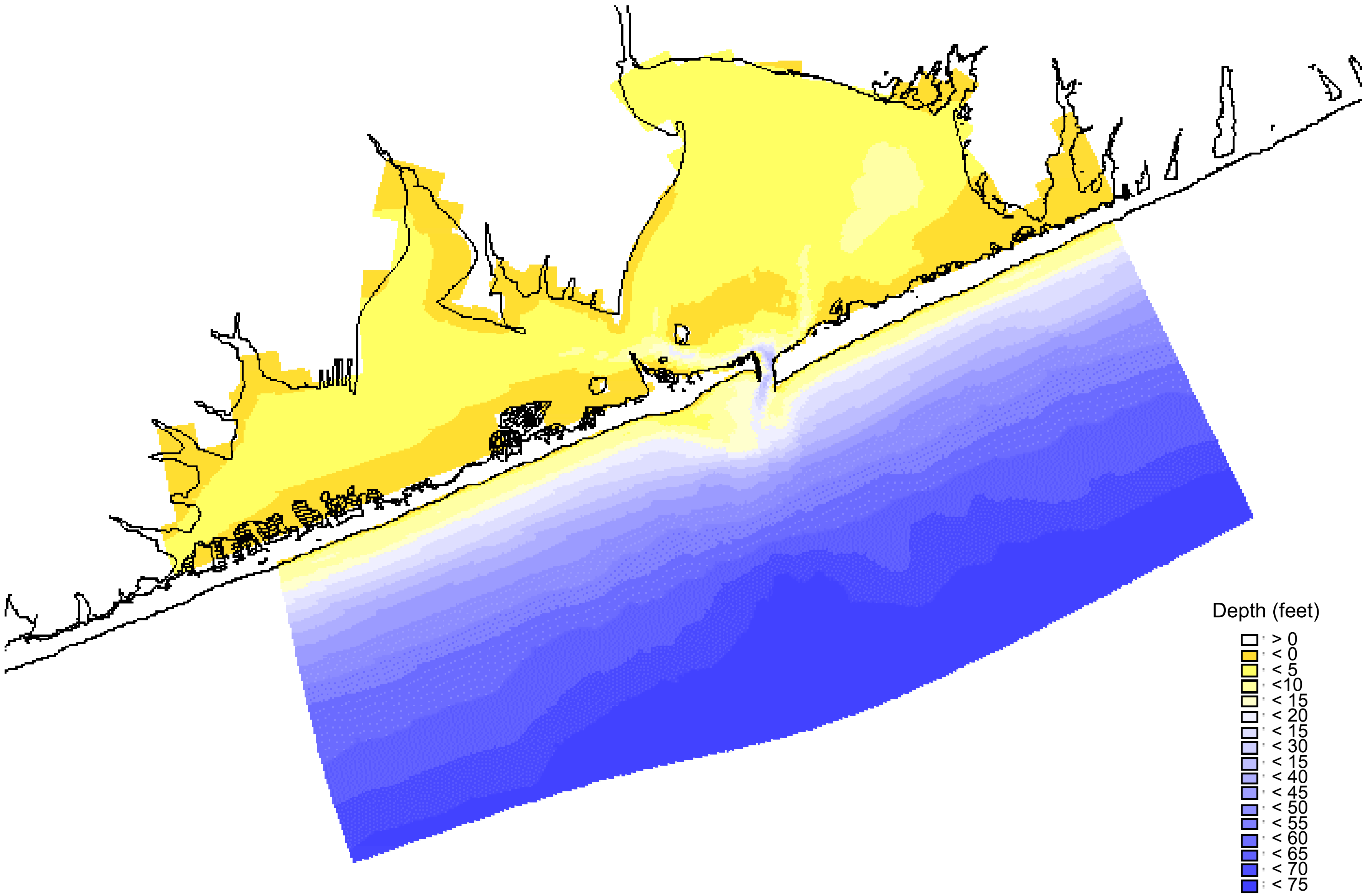
FIGURE 4-2

Filename: CAD/3982-23/Report2002/Shinn_Inlet-Grid.mxd



Shinnecock Inlet – Hydrodynamic Model Bathymetry

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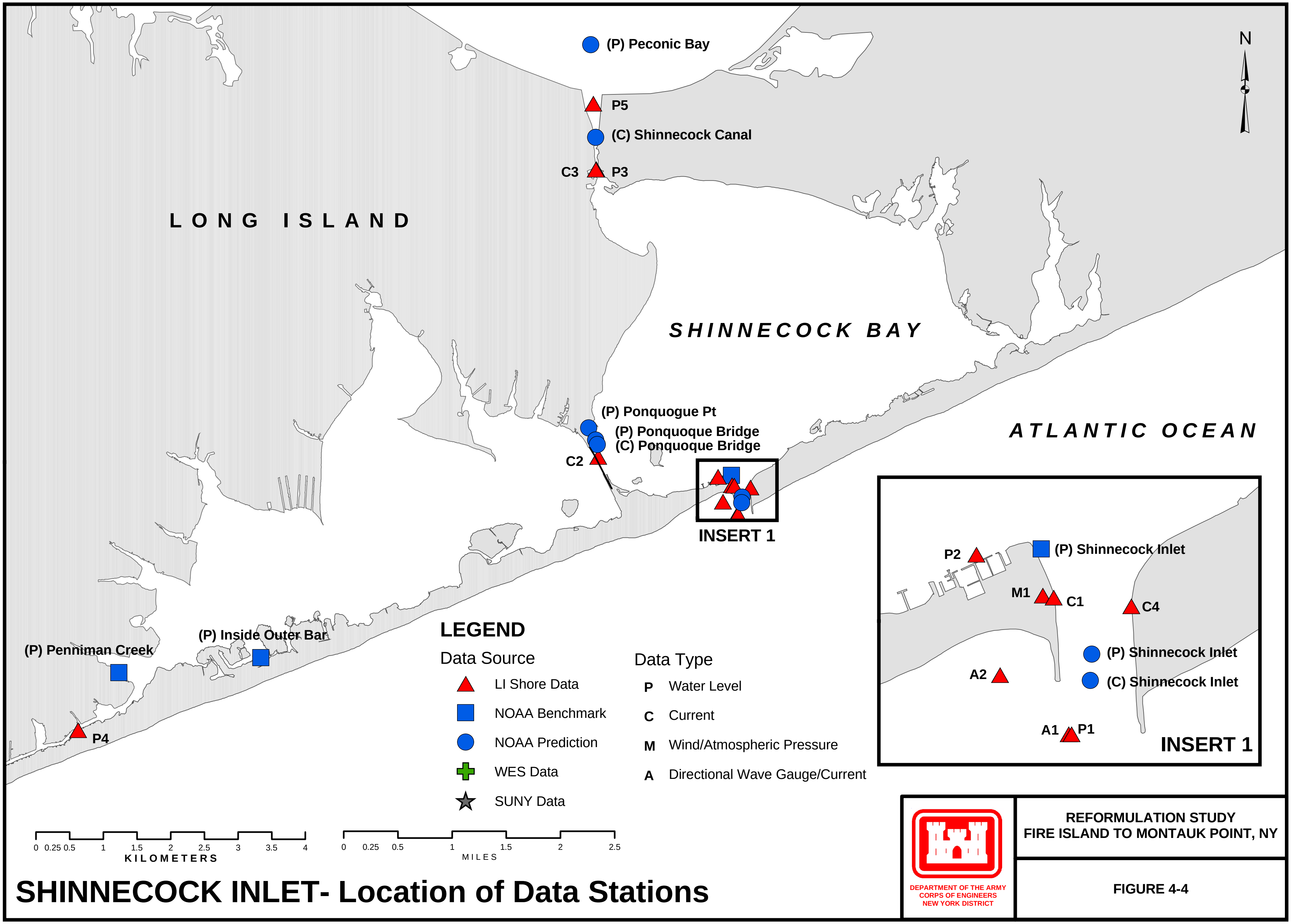
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 4-3

File pat/File name



SHINNECOCK INLET- Location of Data Stations

Filename: CAD/3982-23/Report2002/Shinn_Data-Location.mxd

Note: 44025 data continuously available since 1991

Period of Record for Shinnecock Data Stations

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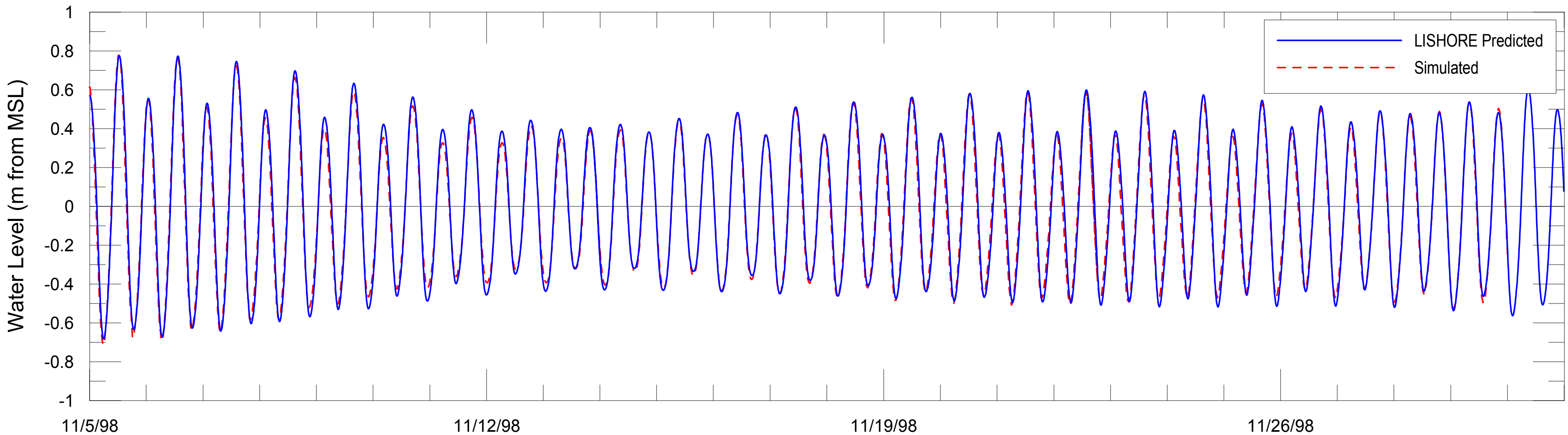
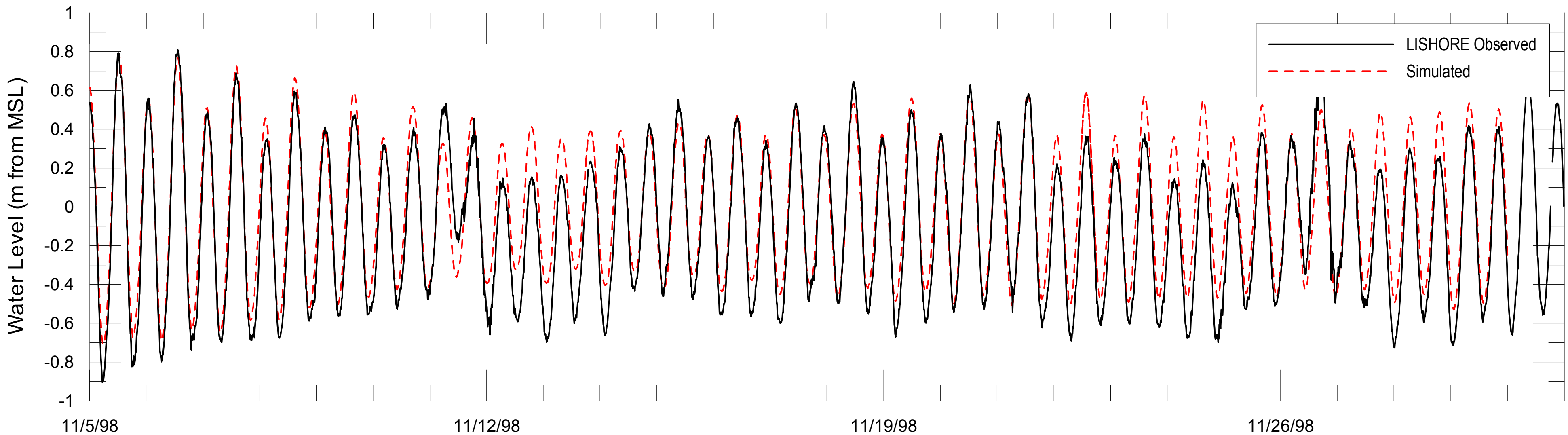
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 4.5

File pat/File name





Comparison of Predicted, Observed and Simulated Water Levels at Station P1

DEPARTMENT OF THE ARMY
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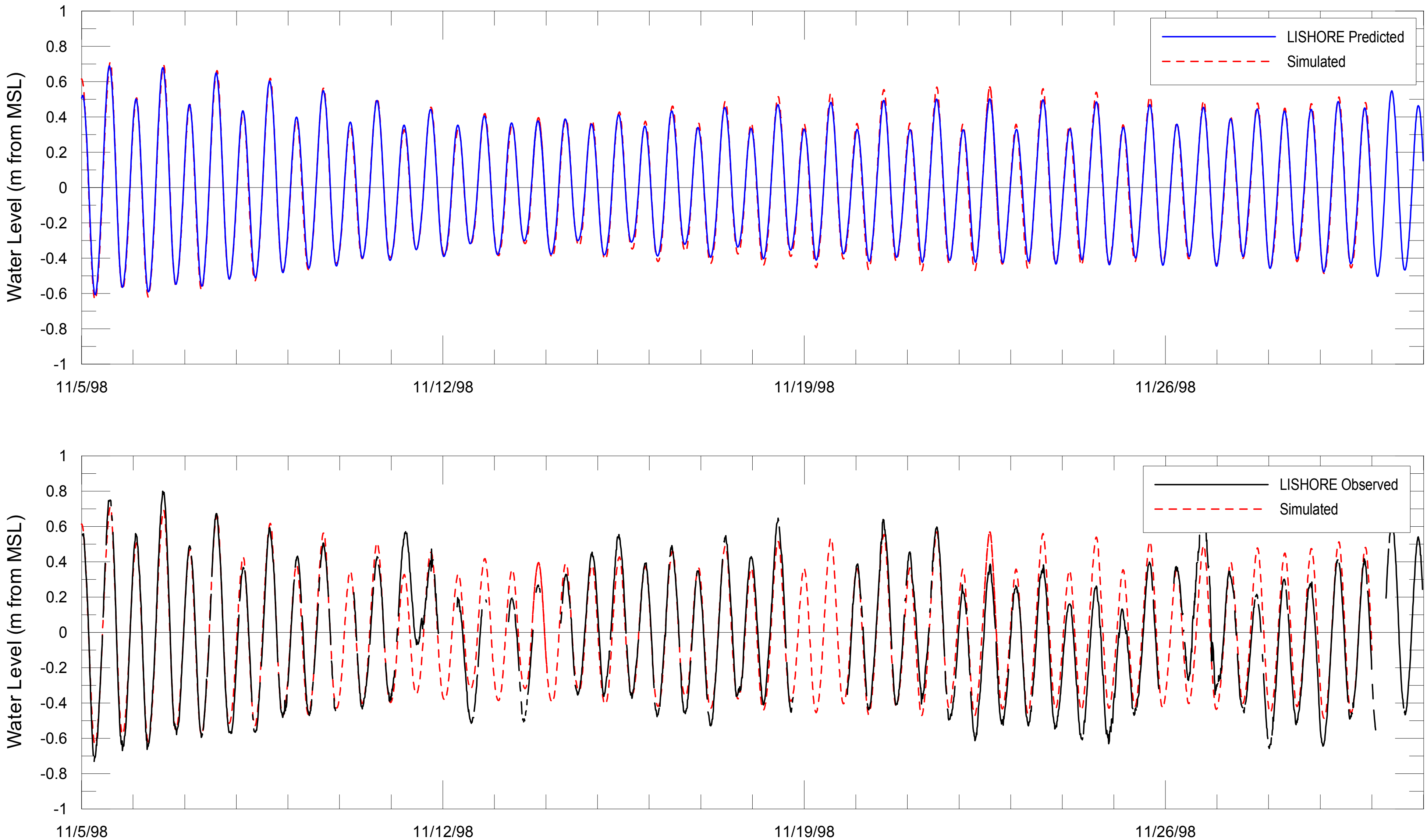
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 4-6



Comparison of Predicted, Observed and Simulated Water Levels at Station P2

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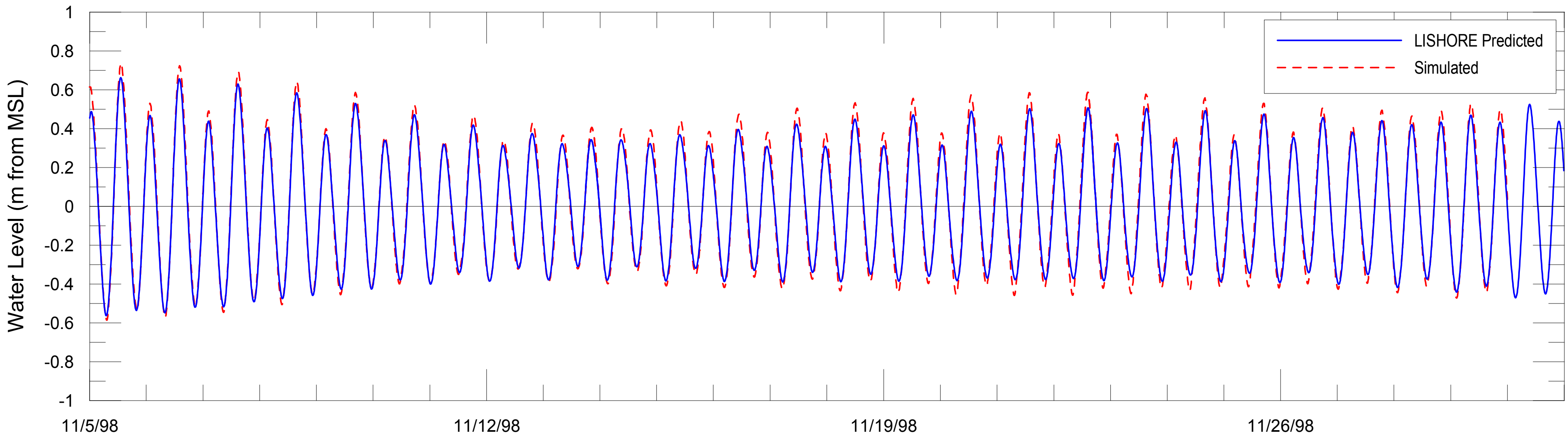
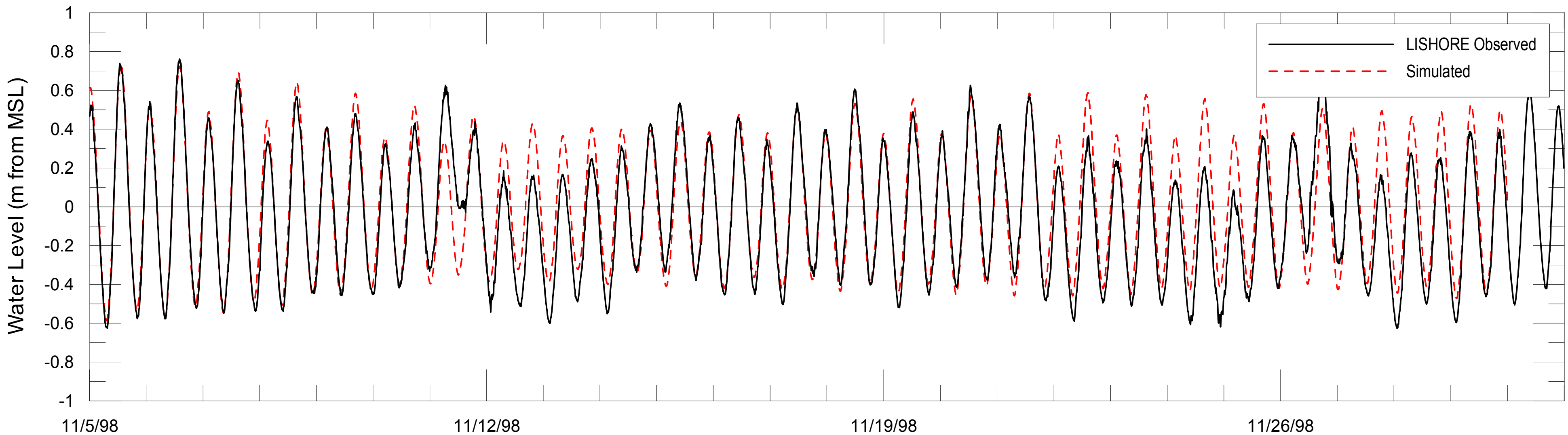
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 4-7





Comparison of Predicted, Observed and Simulated Water Levels at Station P3

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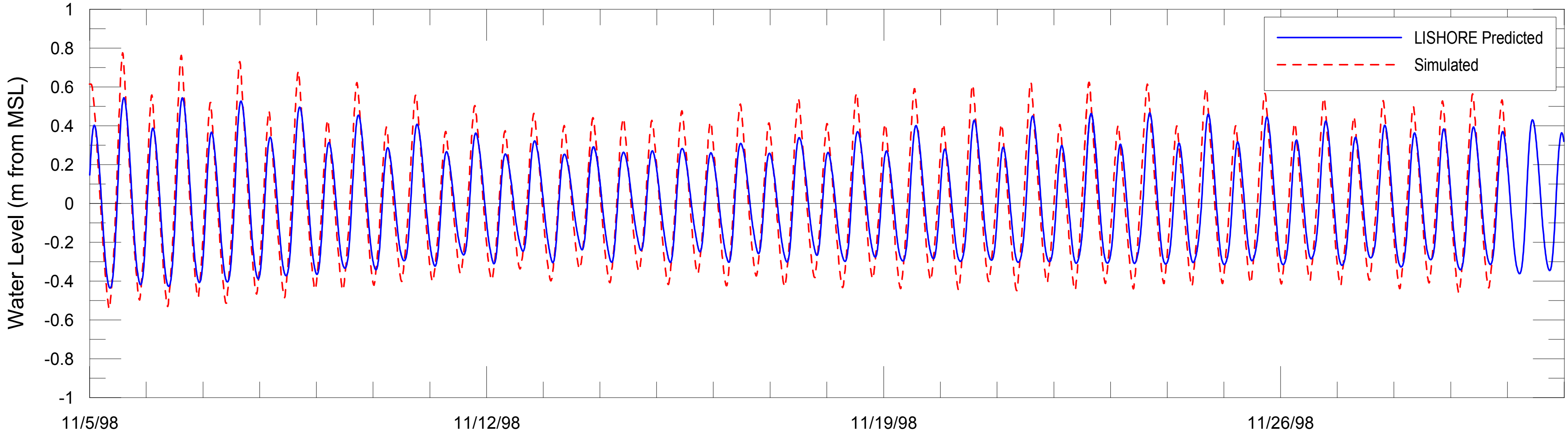
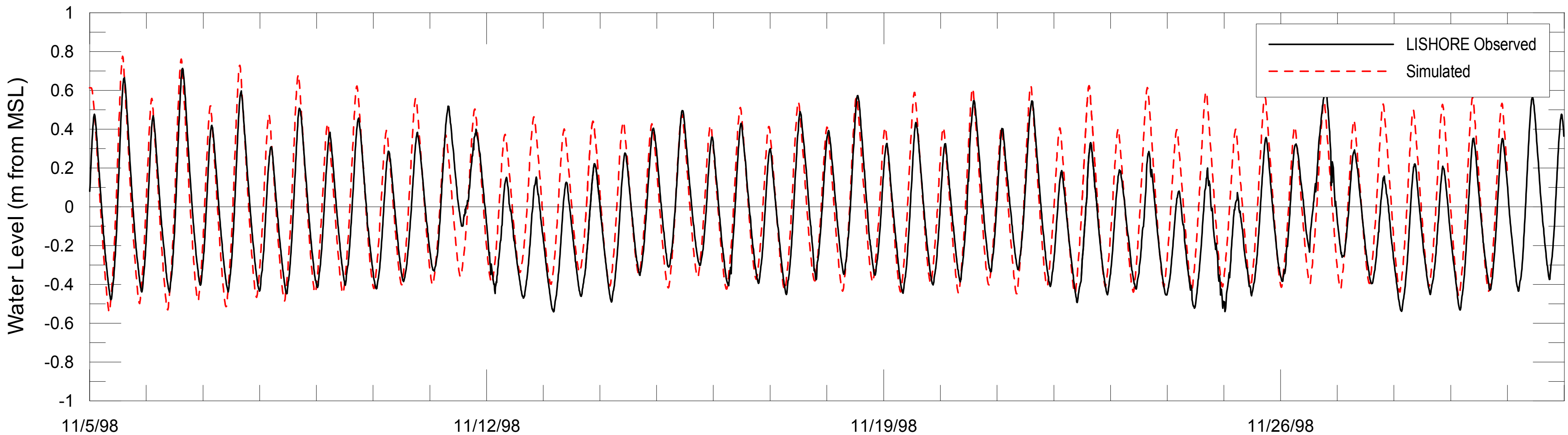
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 4-8





Comparison of Predicted, Observed and Simulated Water Levels at Station P4

DEPARTMENT OF THE ARMY
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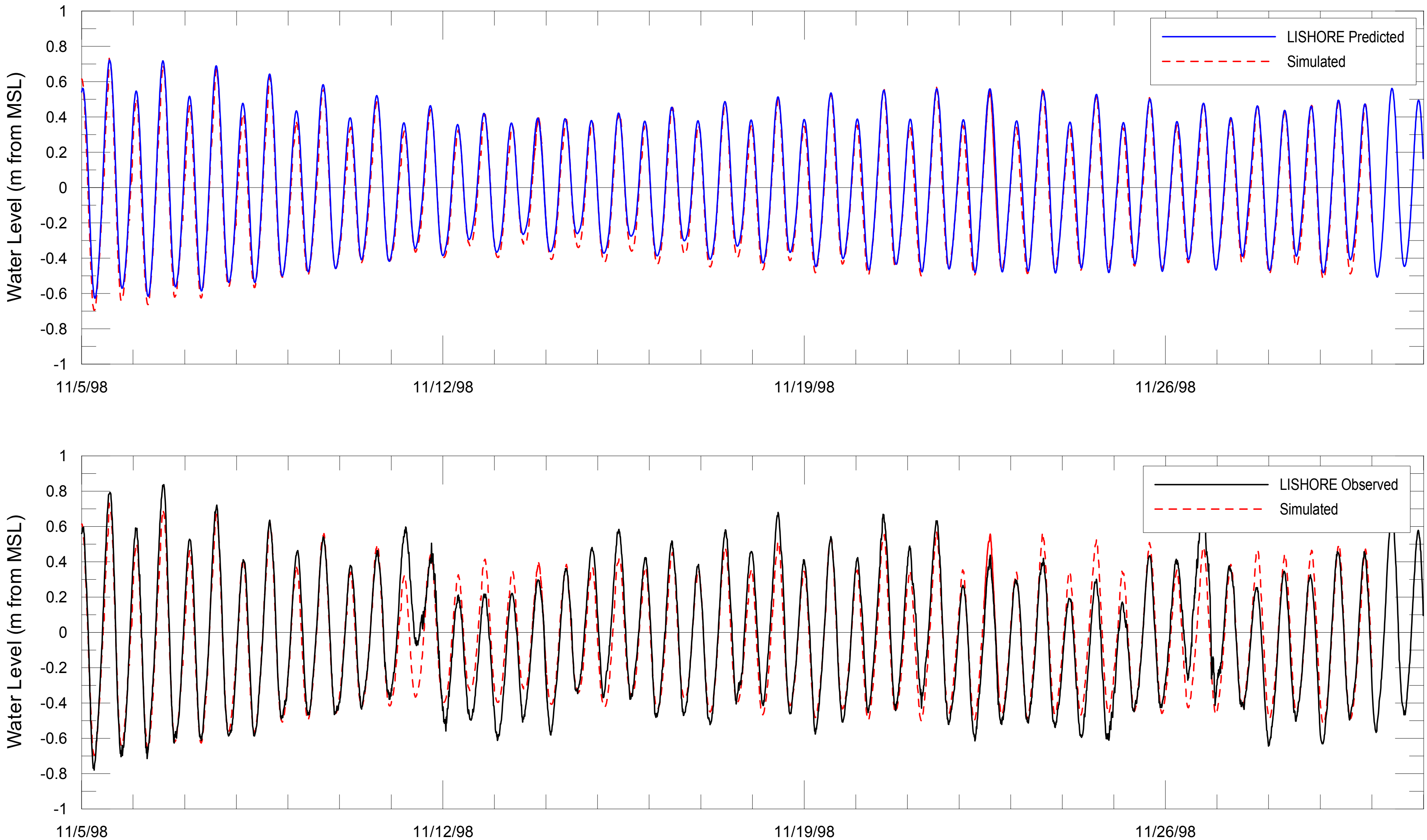
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 4-9



Comparison of Predicted, Observed and Simulated Water Levels at Station C1

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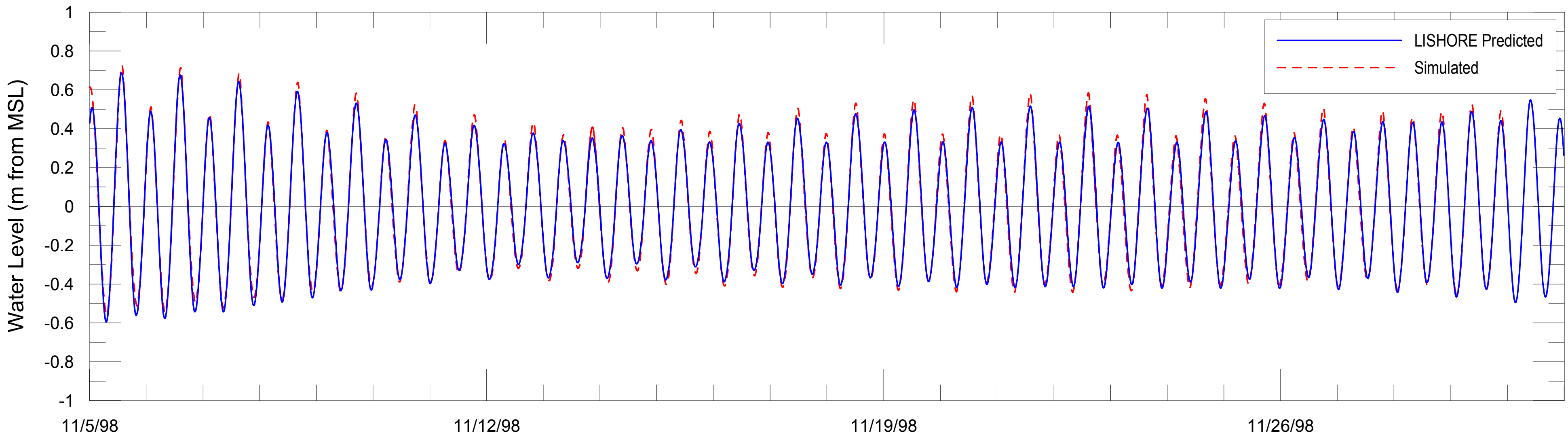
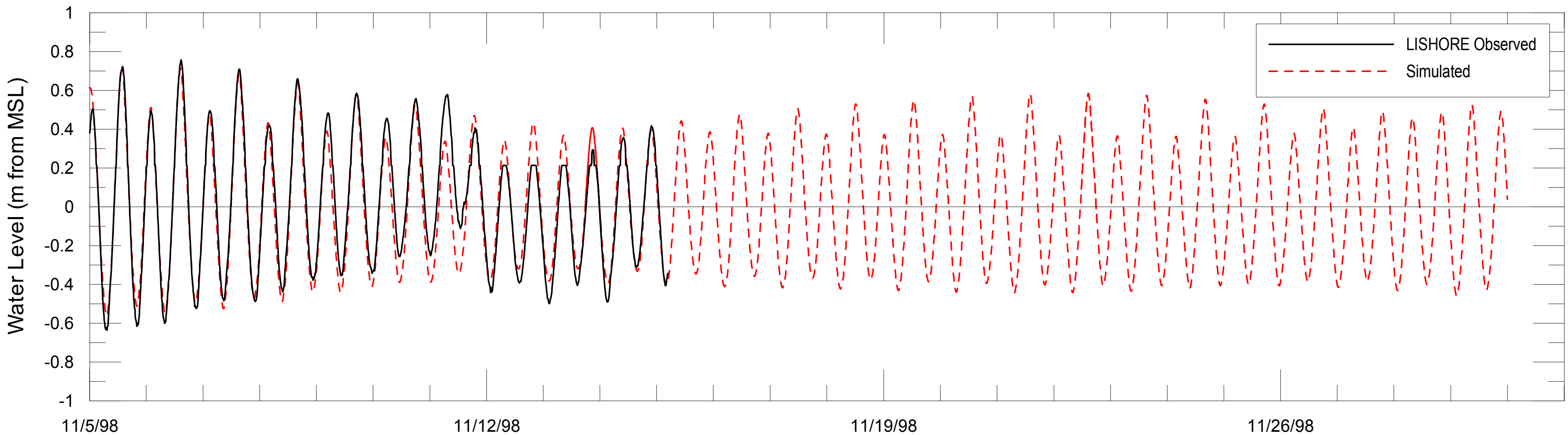
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 4-10





Comparison of Predicted, Observed and Simulated Water Levels at Station C2

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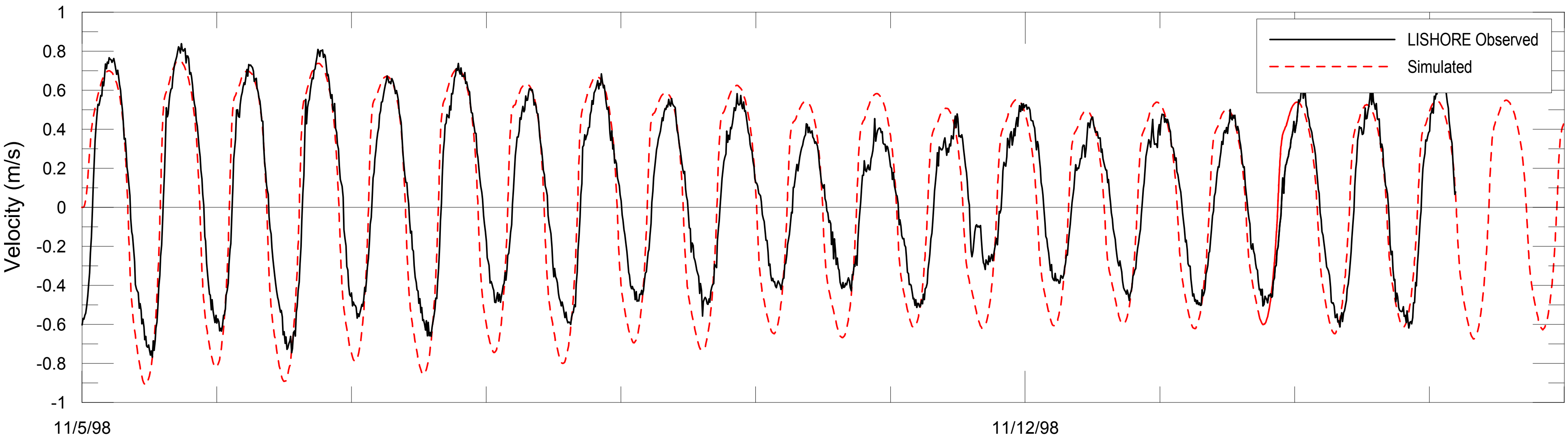
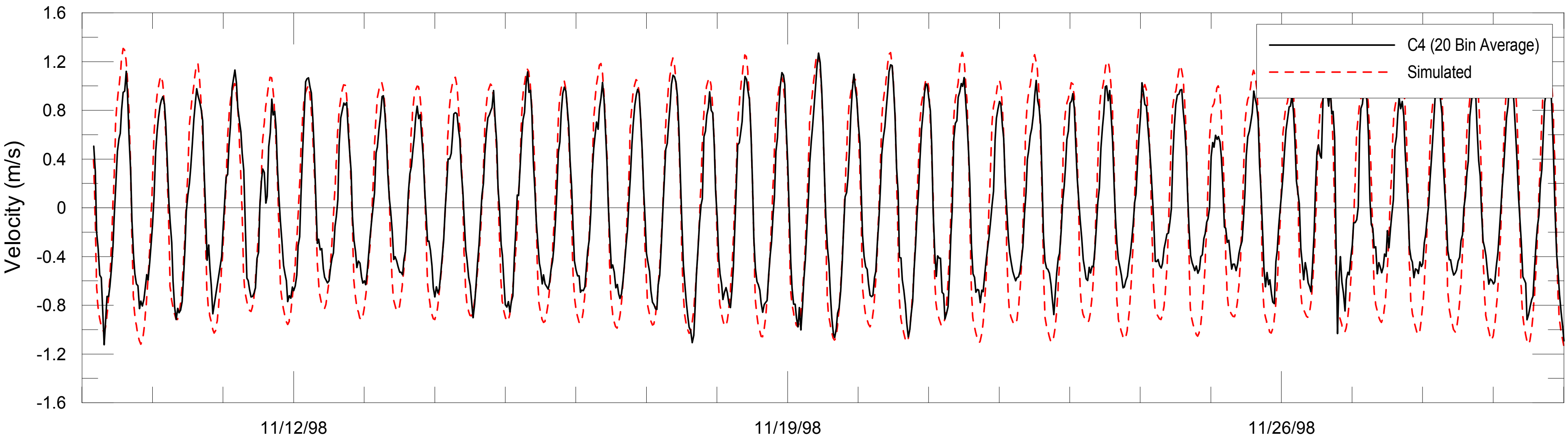
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 4-11





Comparison of Observed and Simulated Currents at Stations C2 and C4

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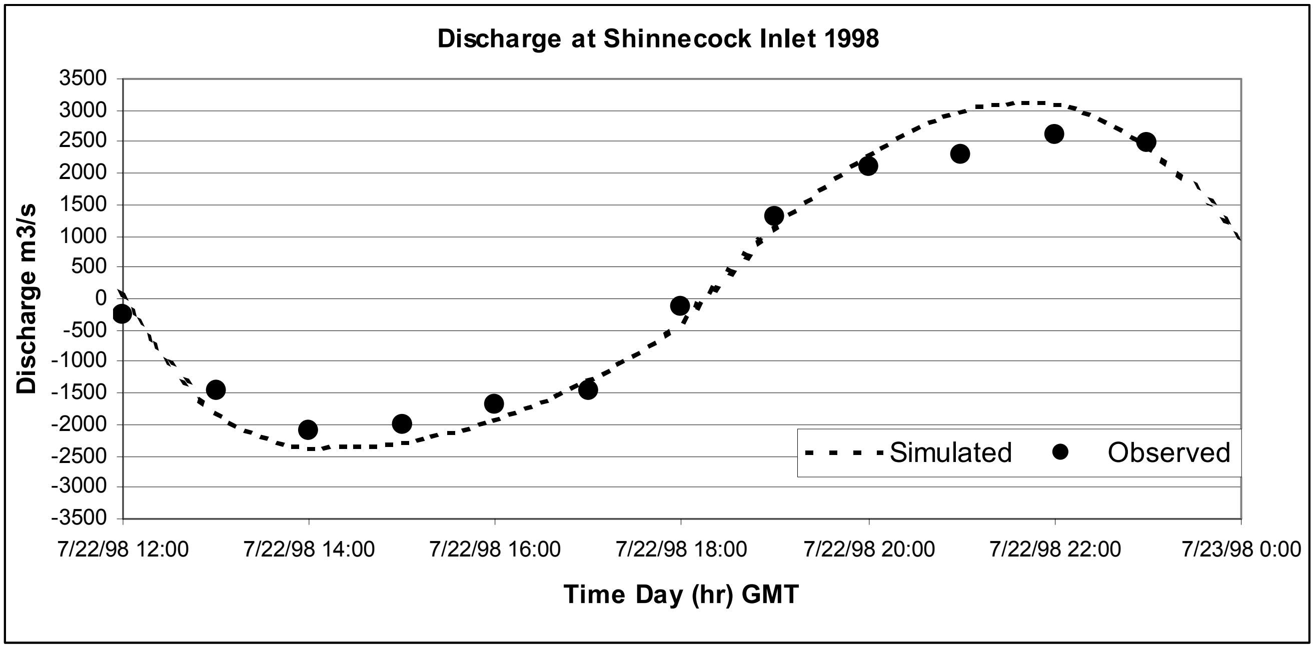
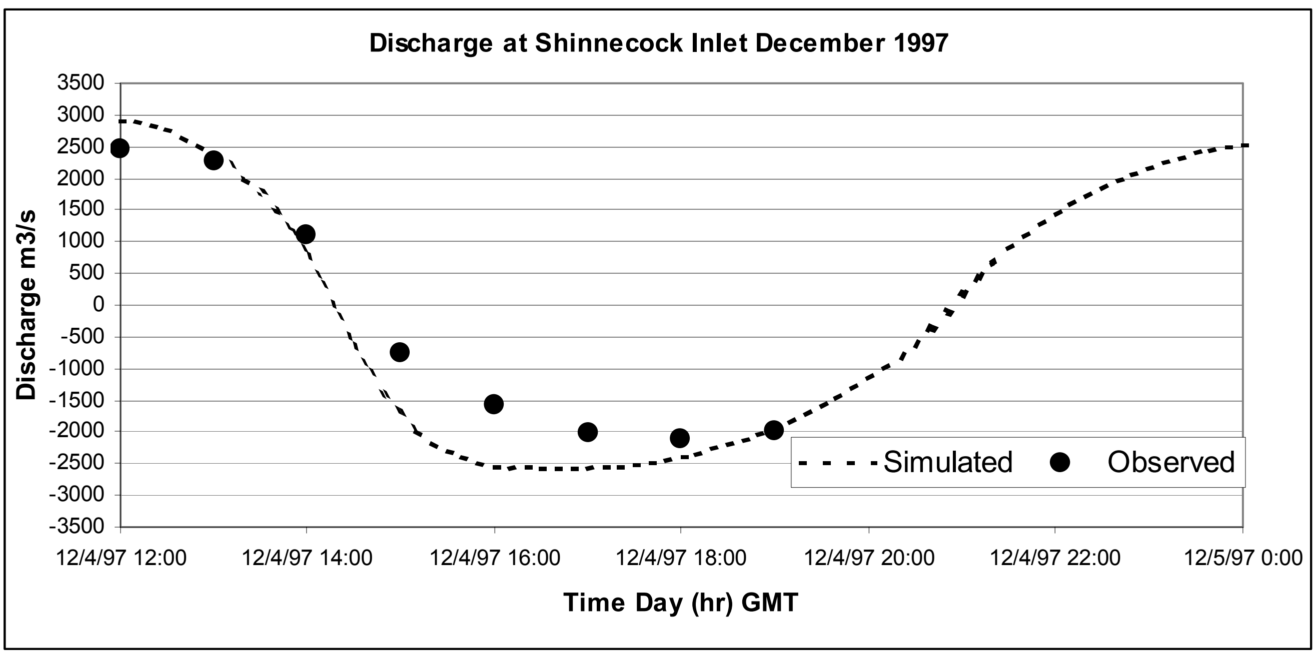
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 4-12





Comparison of observed and simulated discharges at Shinnecock Inlet

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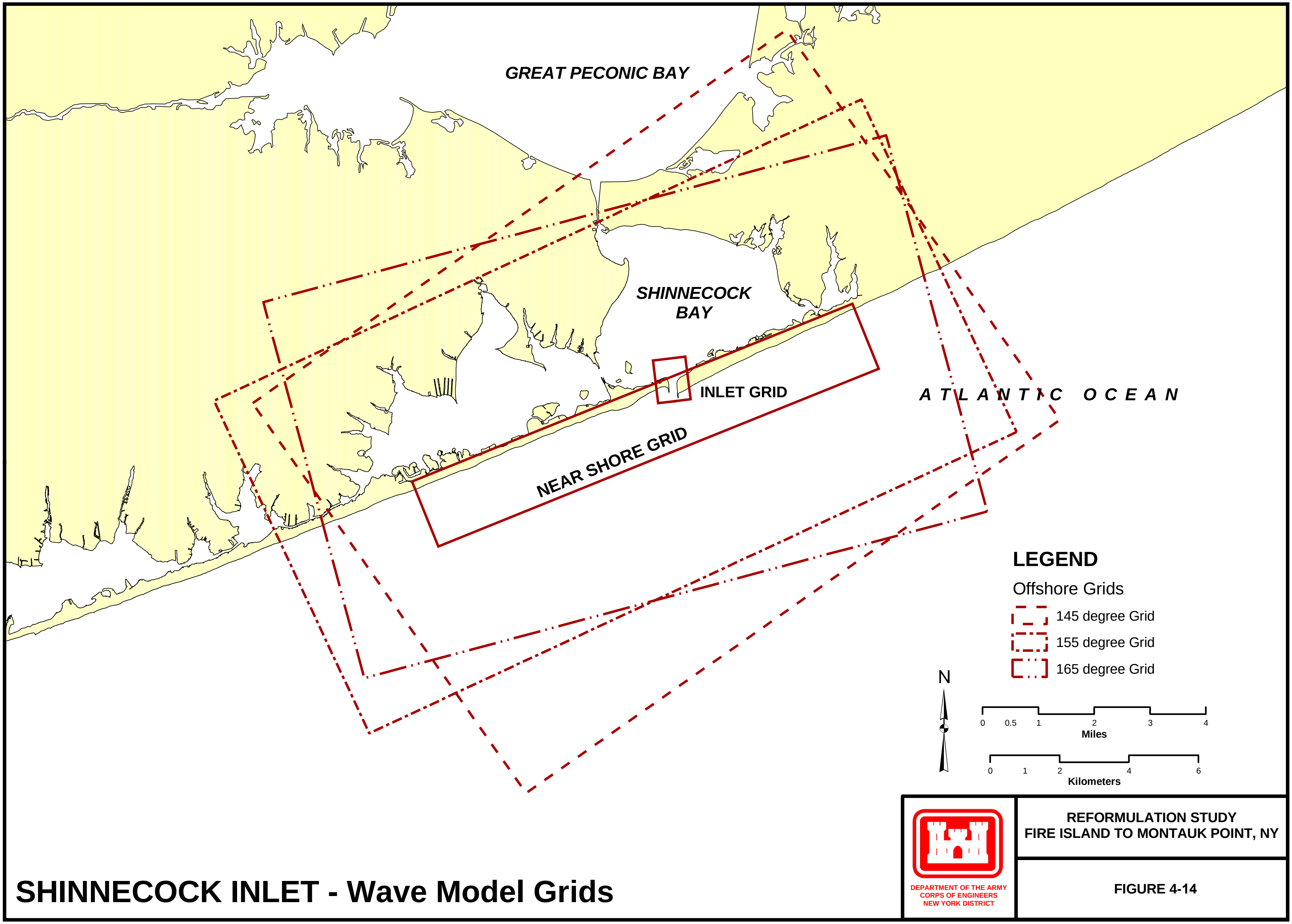
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

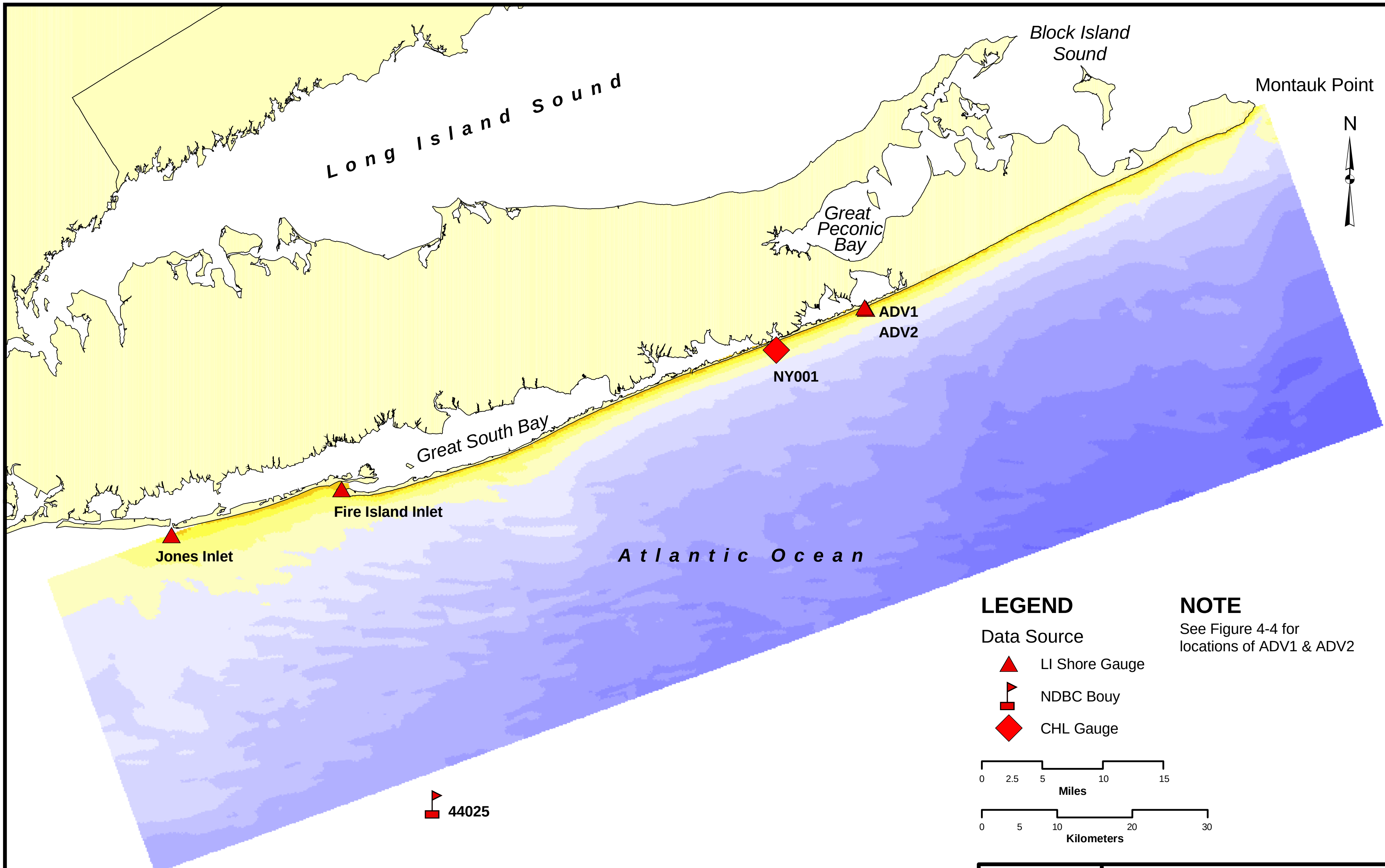
FIGURE 4-13

File pat/File name



SHINNECOCK INLET - Wave Model Grids

Filename: CAD/3982-23/Report2002/Fig4_8.mxd



Regional Wave Model Bathymetry & Wave Station Locations

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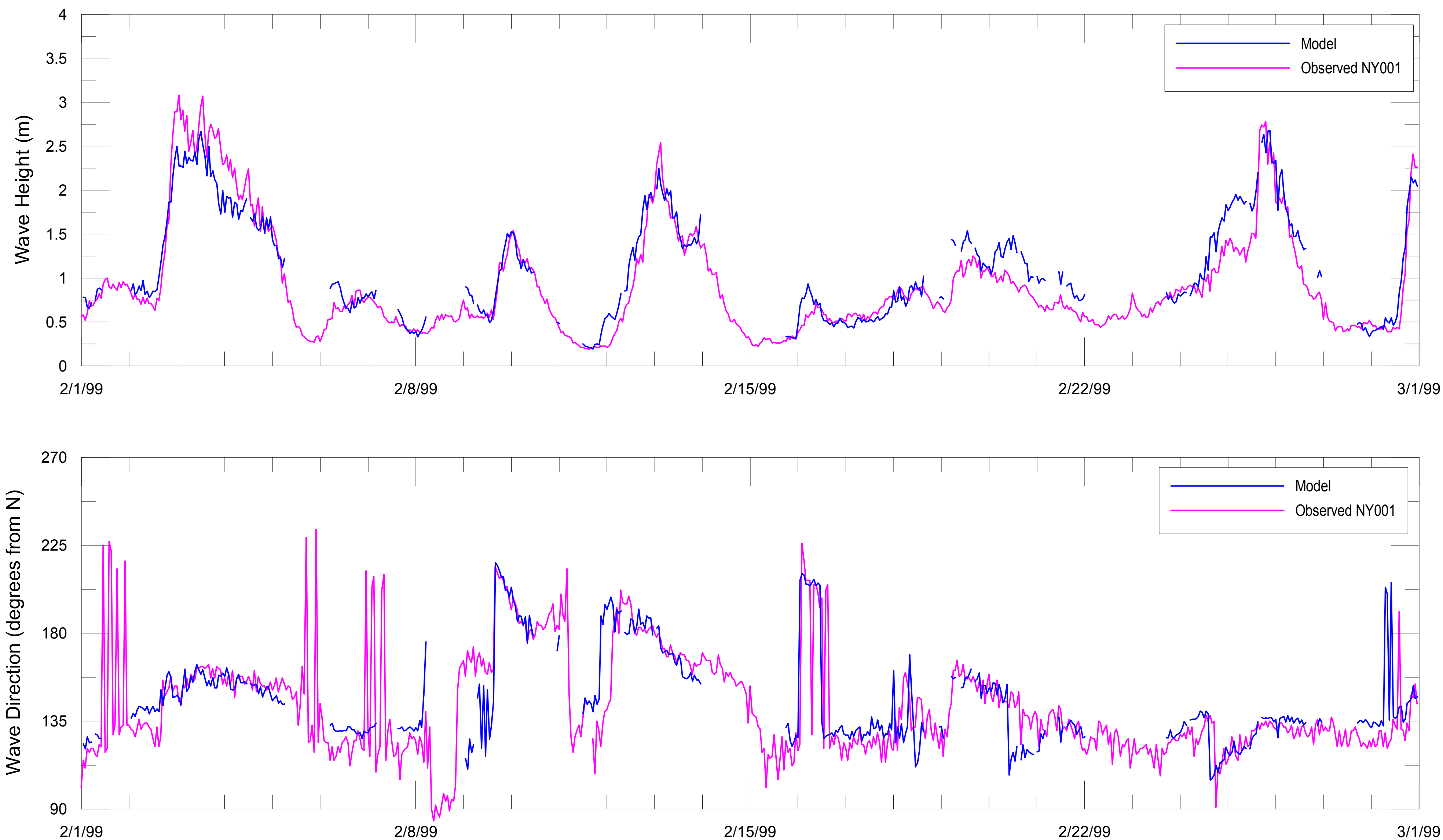
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 4-15

Filename: 3982-23\Report 2002\Fig4_9.mxd



Regional Wave Model versus Observations at NY001

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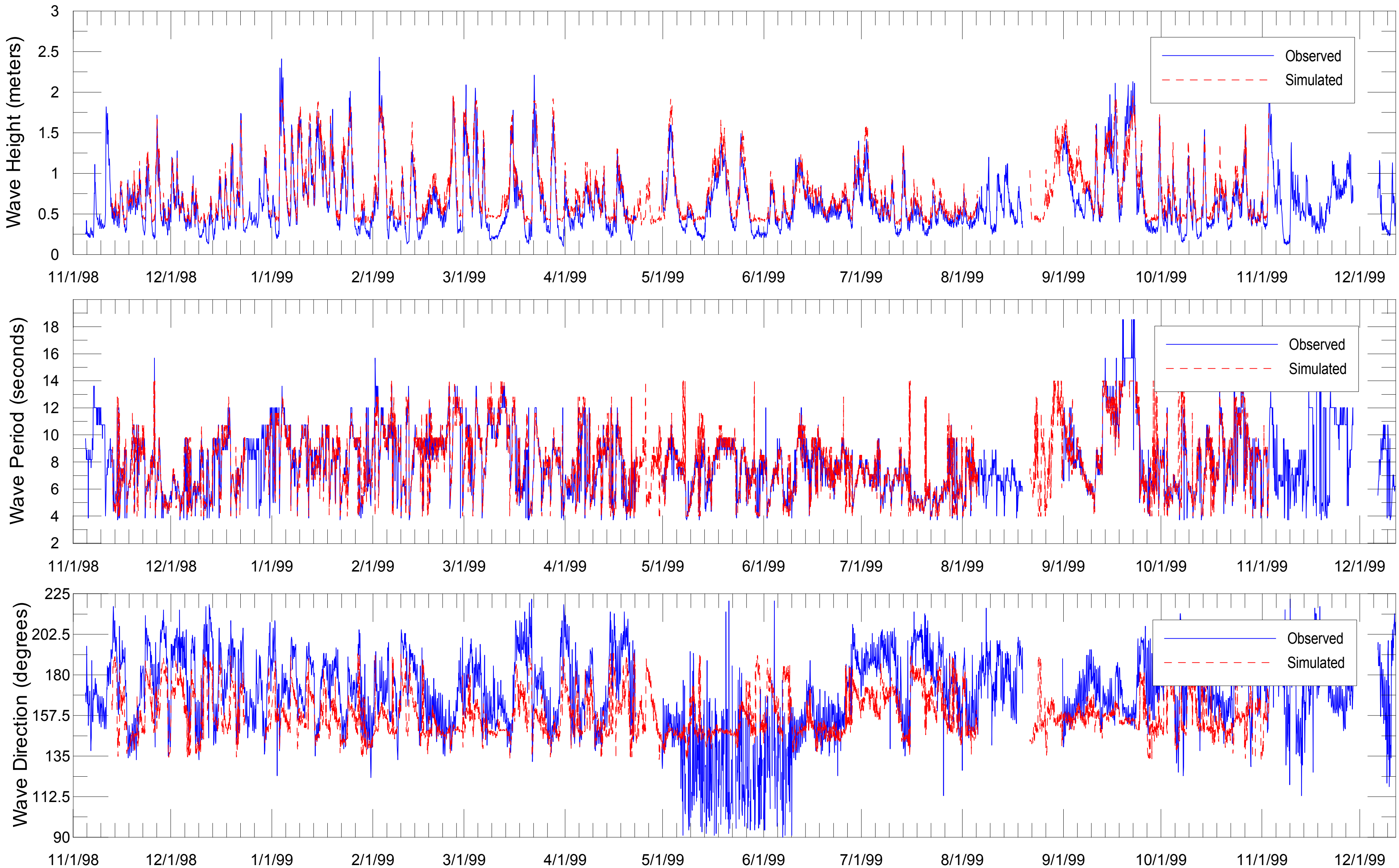
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 4-16



Wave Calibration at ADV1

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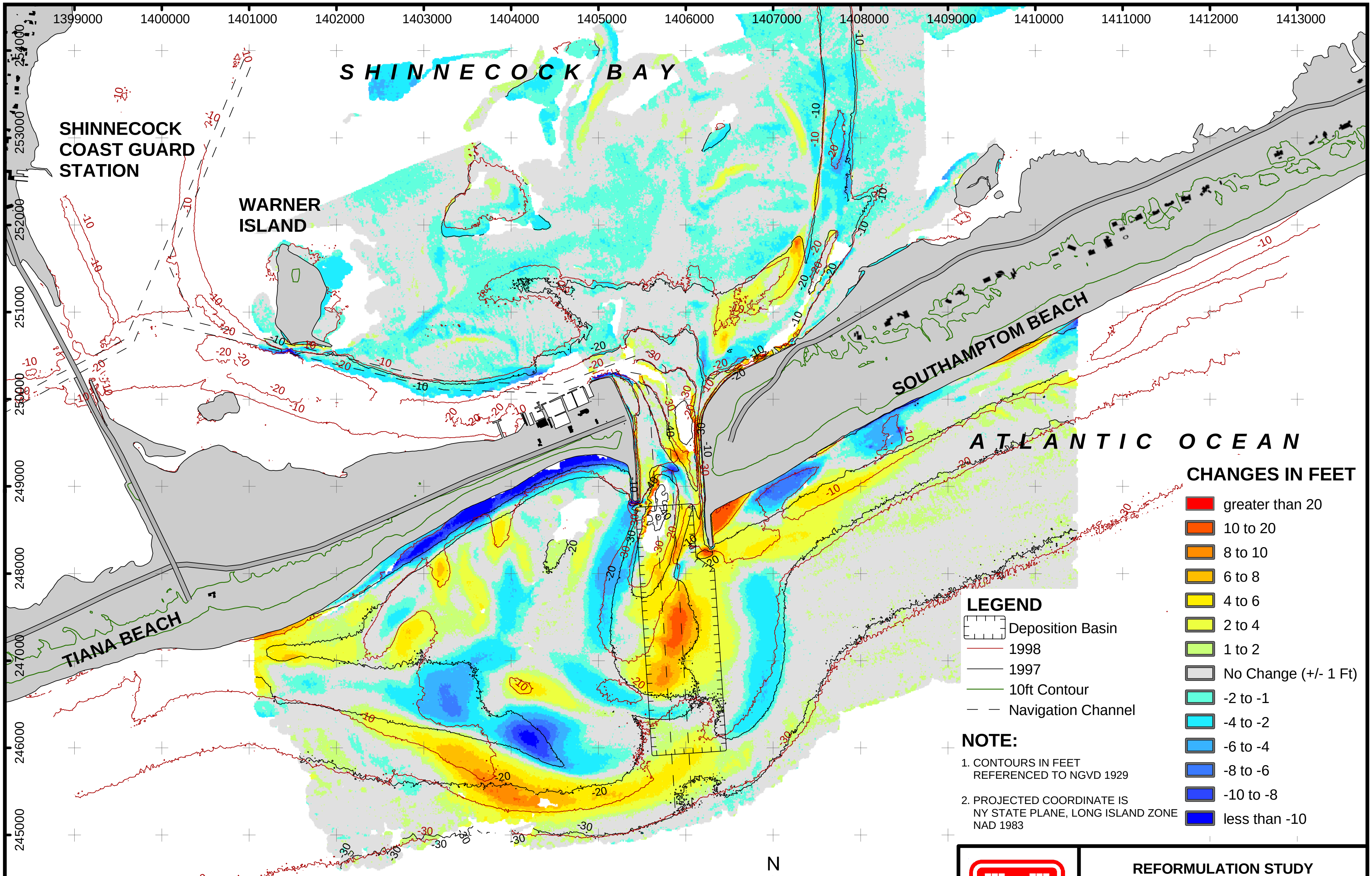
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 4-17



SHINNECOCK INLET - Measured Bathymetric Changes (8/13/1997 to 5/28/1998)

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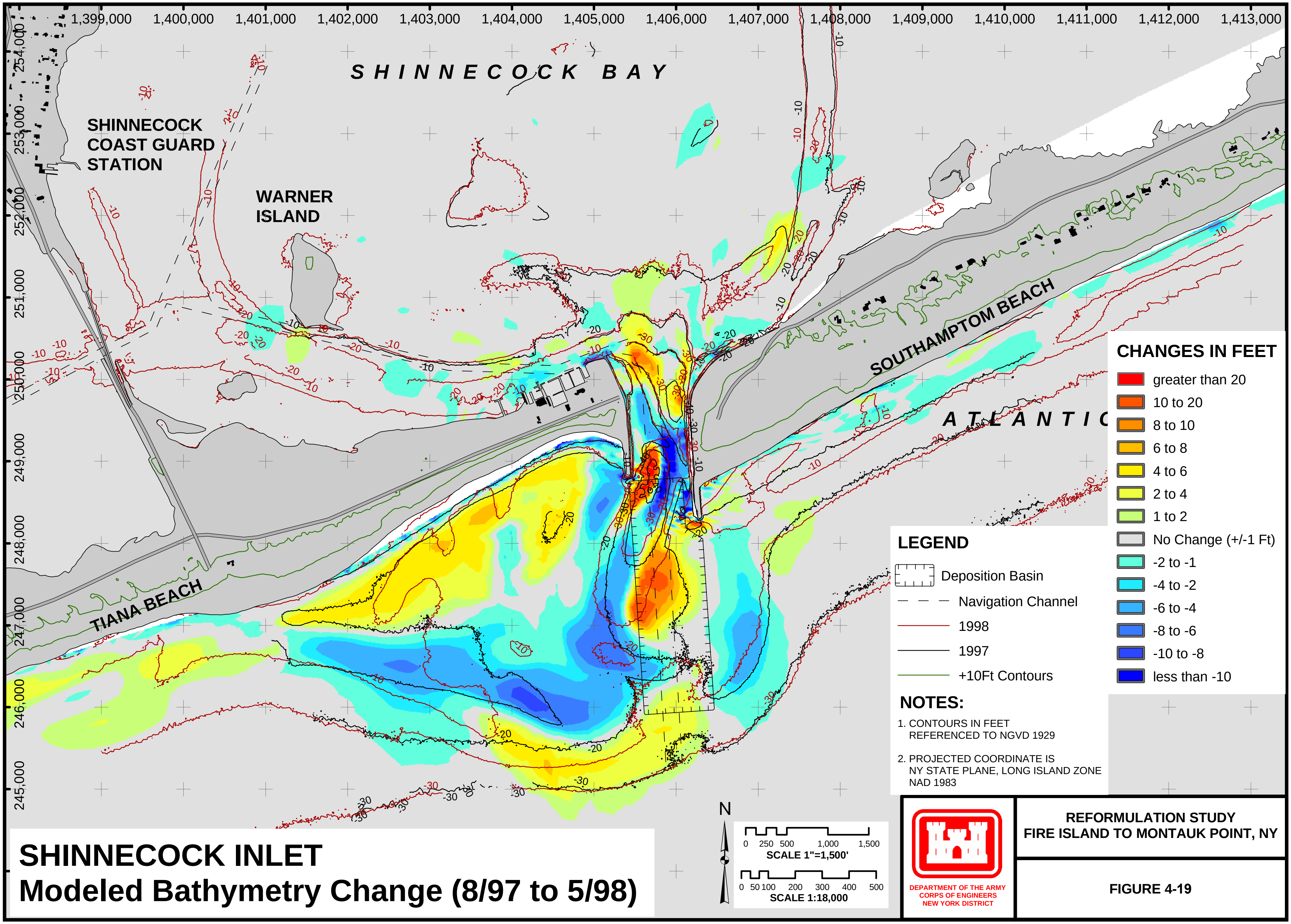
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 4-18

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Shin_Diff.mxd

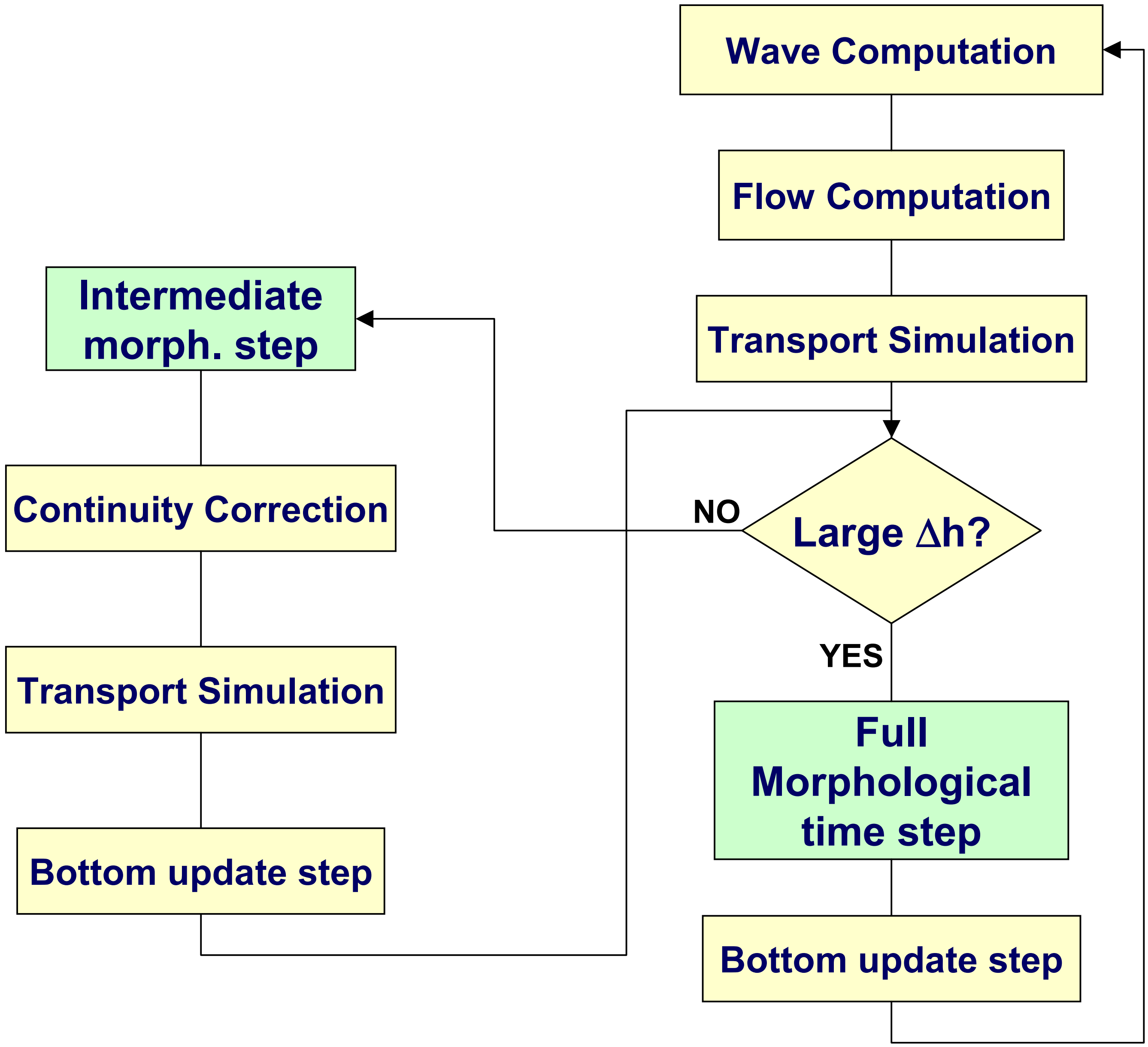


SHINNECOCK INLET
Modeled Bathymetry Change (8/97 to 5/98)

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 4-19

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Shin_98-97.mxd



Delft-MOR Process Tree

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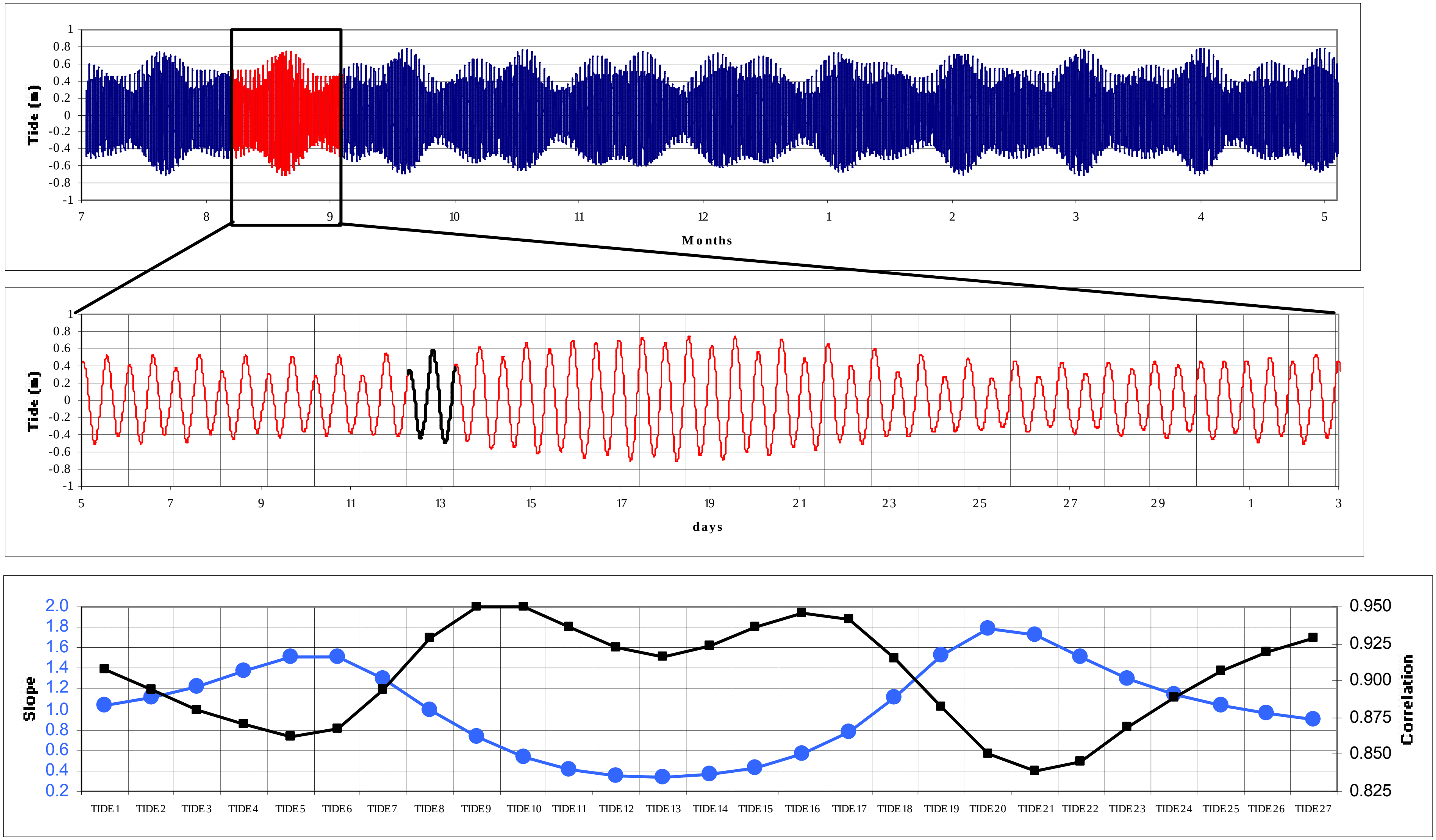
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 4-20

File pat/File name



Shinnecock Inlet – Representative Tide Calculation

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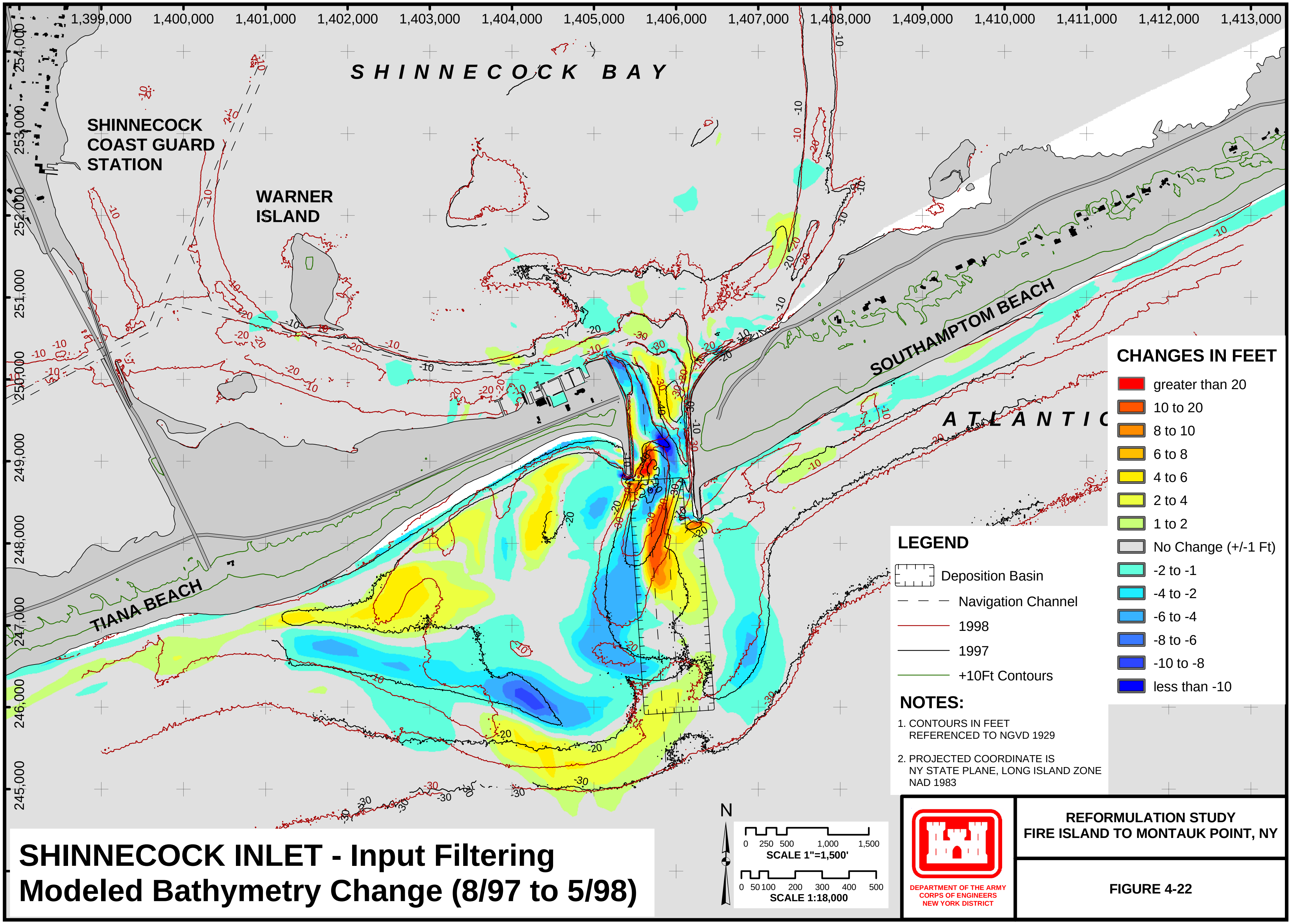
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

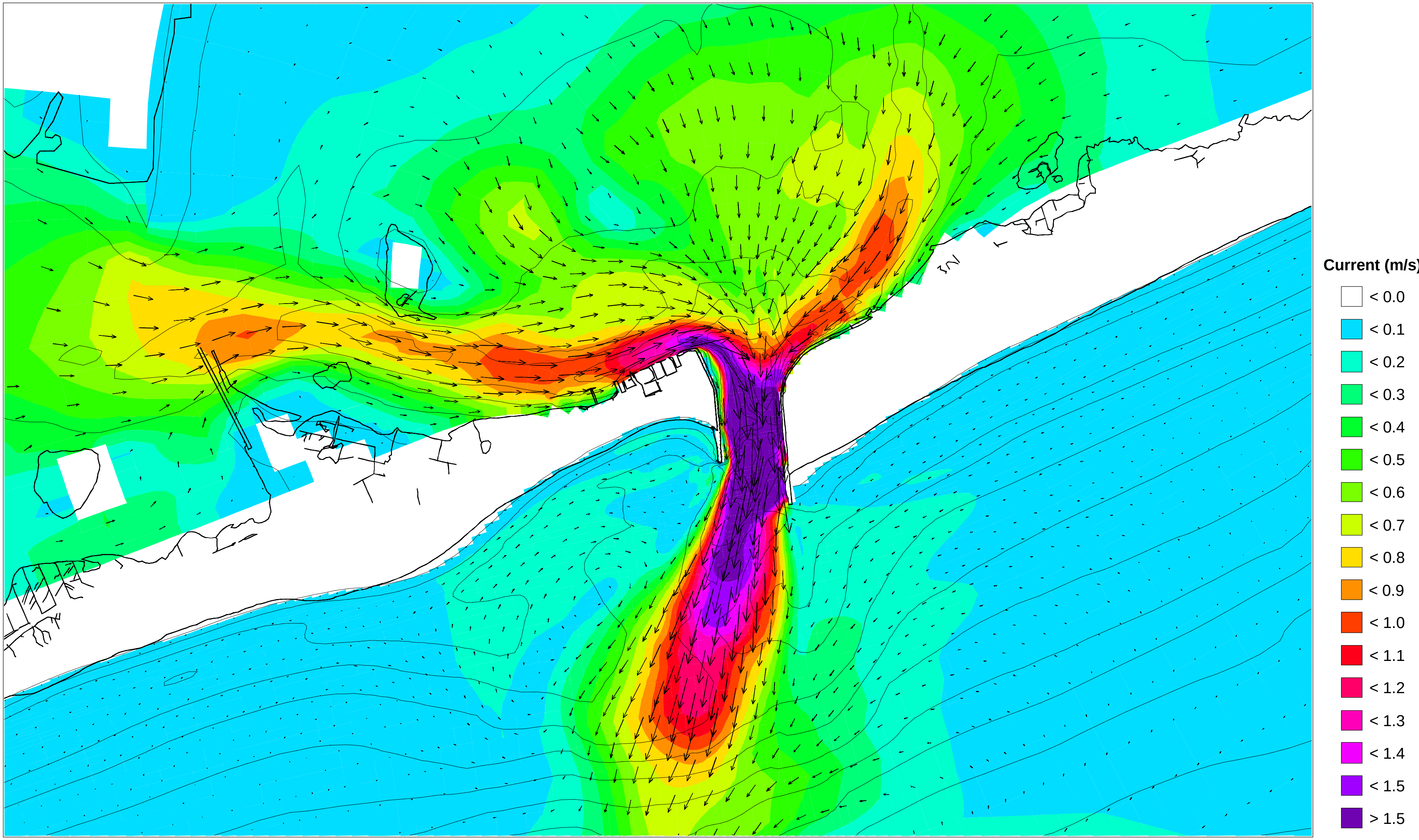
FIGURE 4.21

File pat/File name



SHINNECOCK INLET - Input Filtering Modeled Bathymetry Change (8/97 to 5/98)

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Shin_98-97.mxd



Shinnecock Inlet Flow Velocity – Peak Ebb Tide

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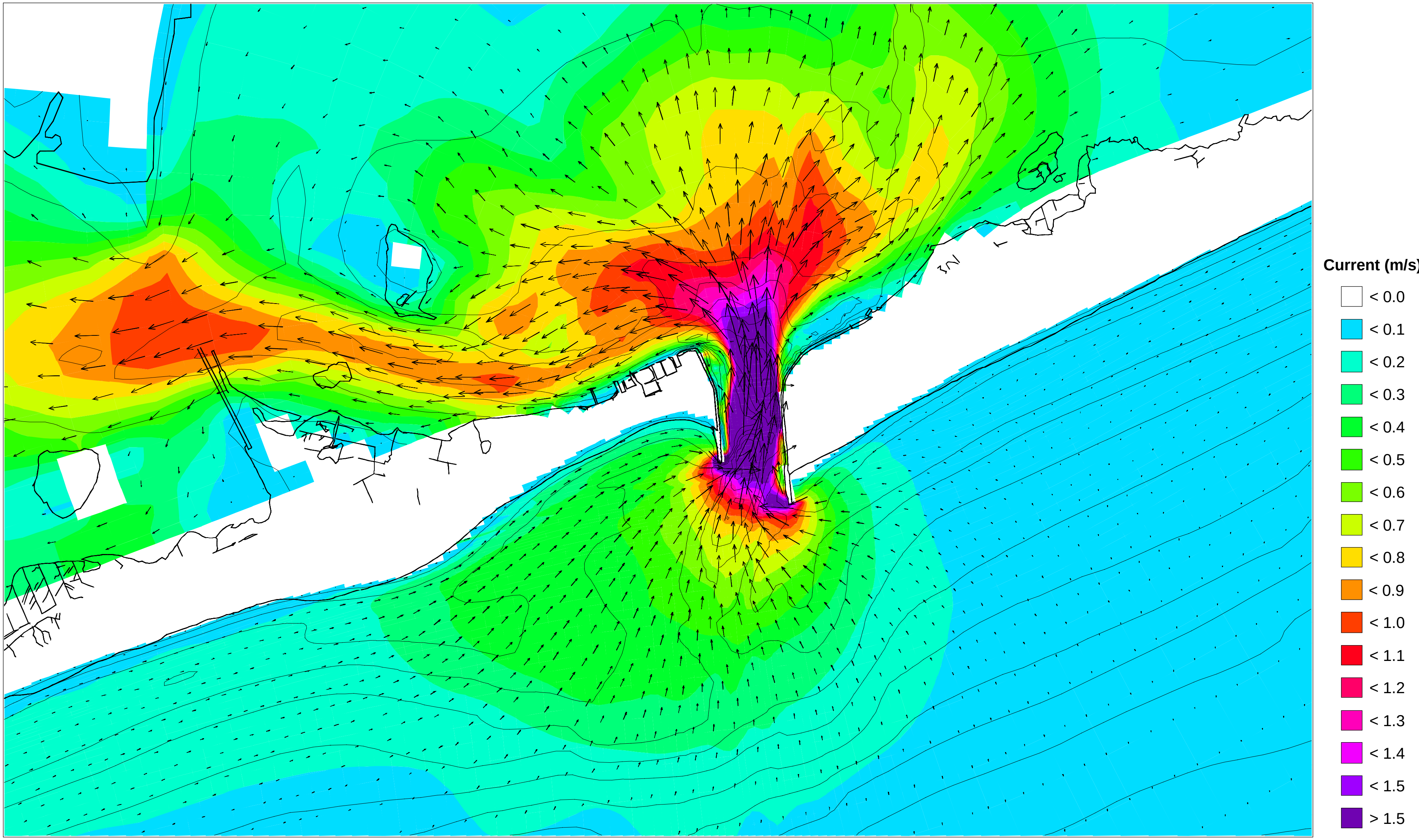
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 4-23



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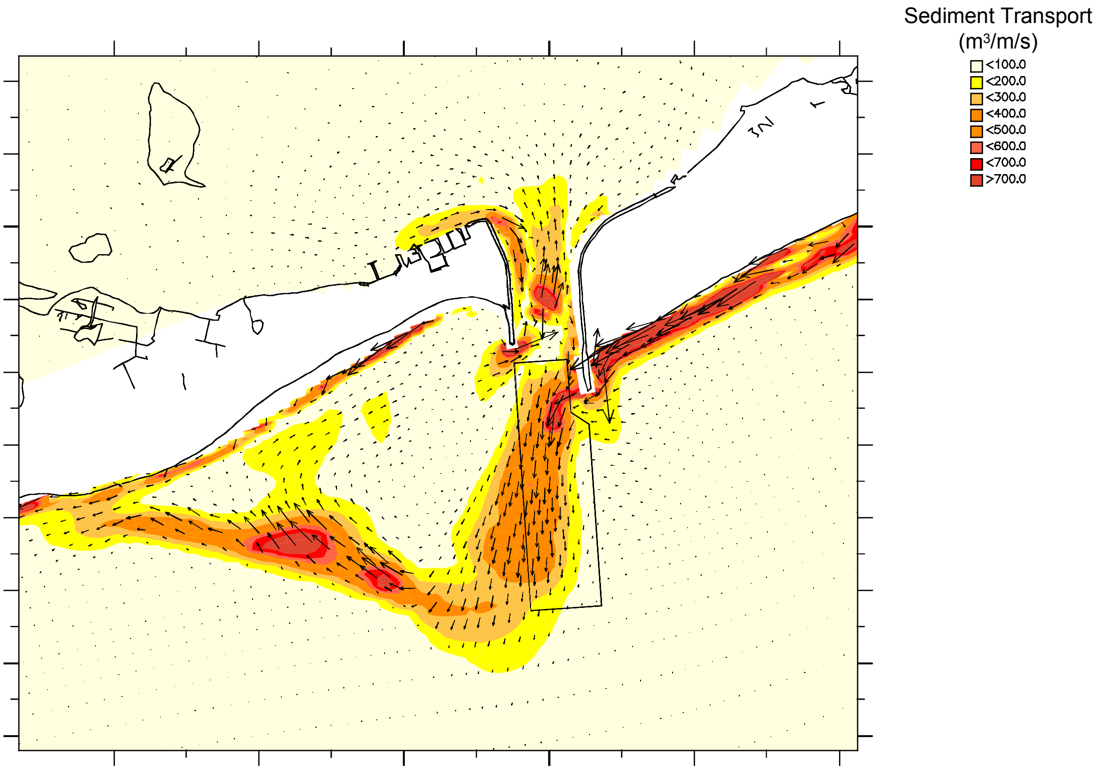
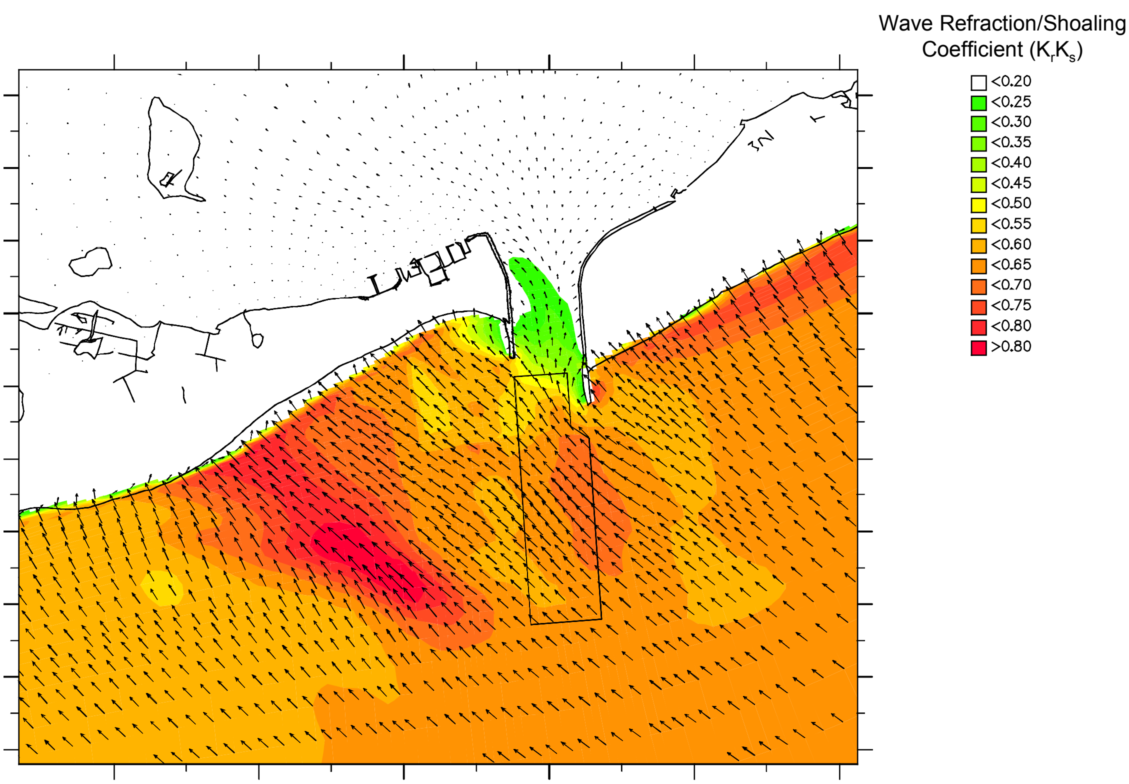
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 4-24

Shinnecock Inlet Flow Velocity – Peak Flood Tide

File pat/File name





Shinnecock Inlet- Sediment Transport and Waves (105° from North)

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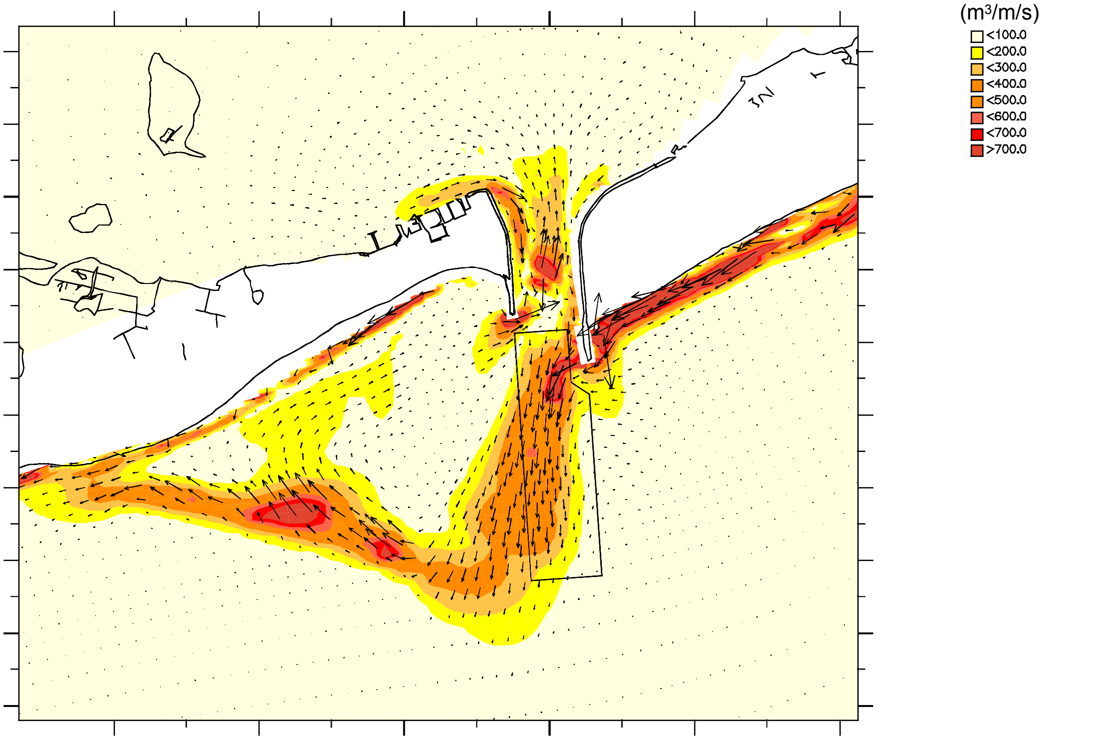
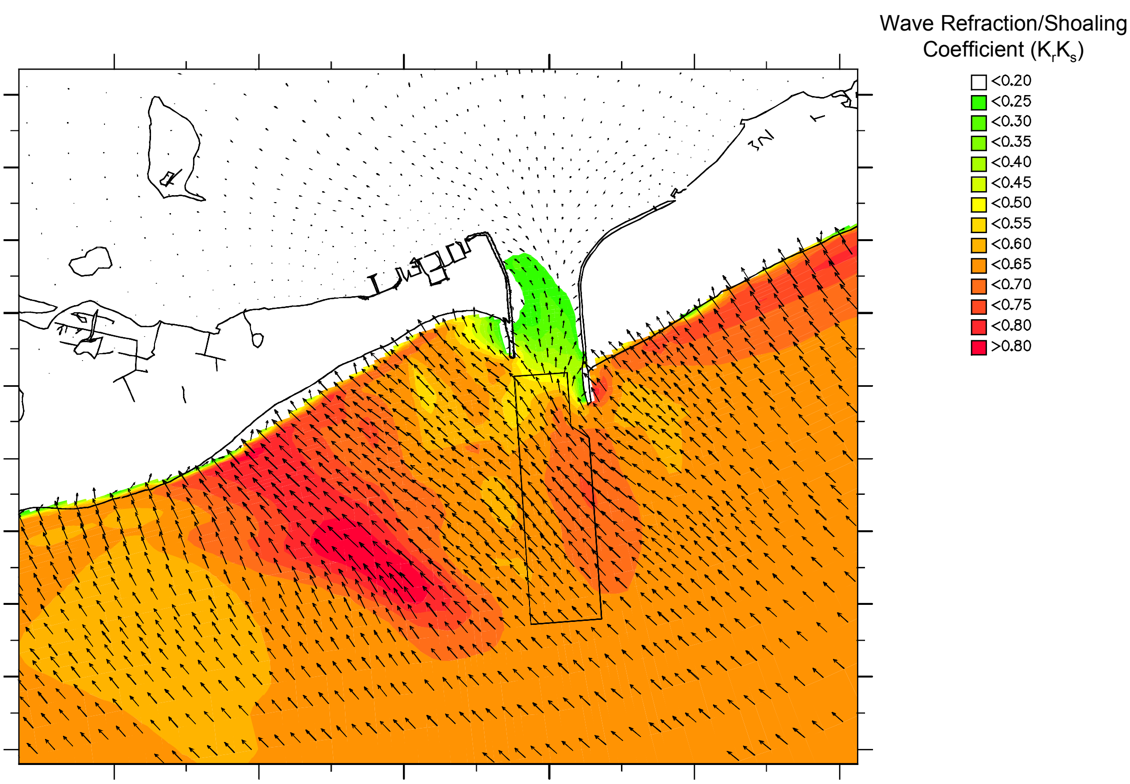
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 4-25

File pat/File name





Shinnecock Inlet- Sediment Transport and Waves (115° from North)

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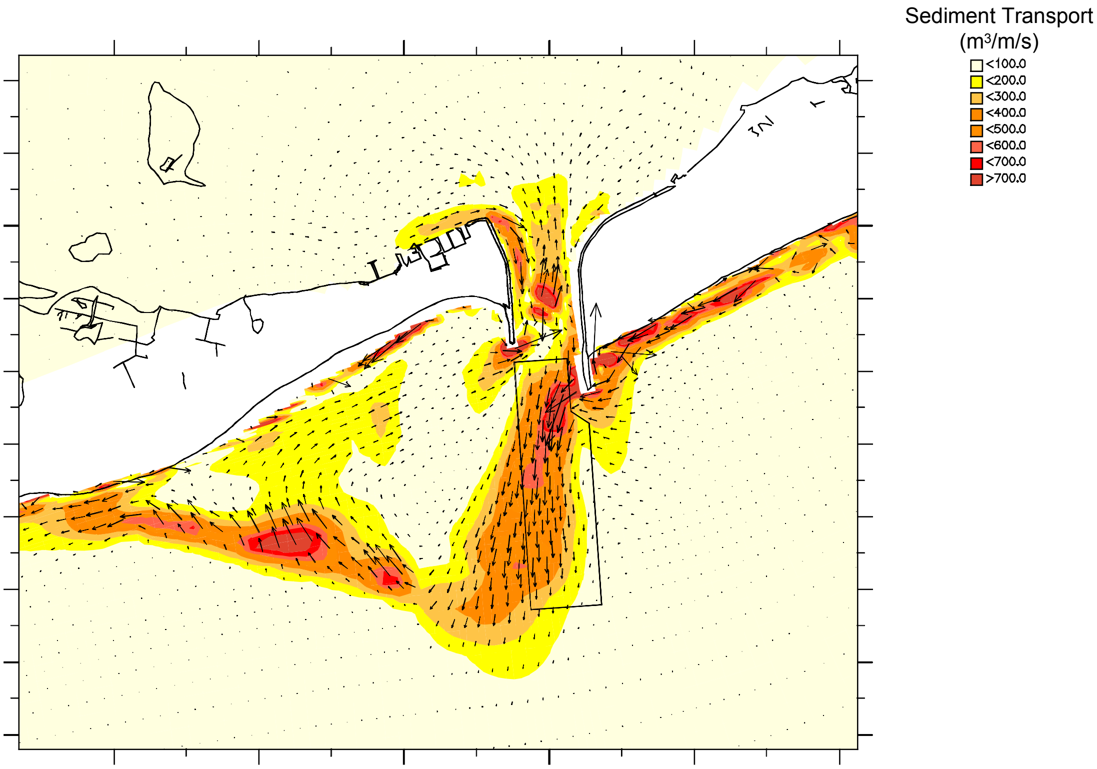
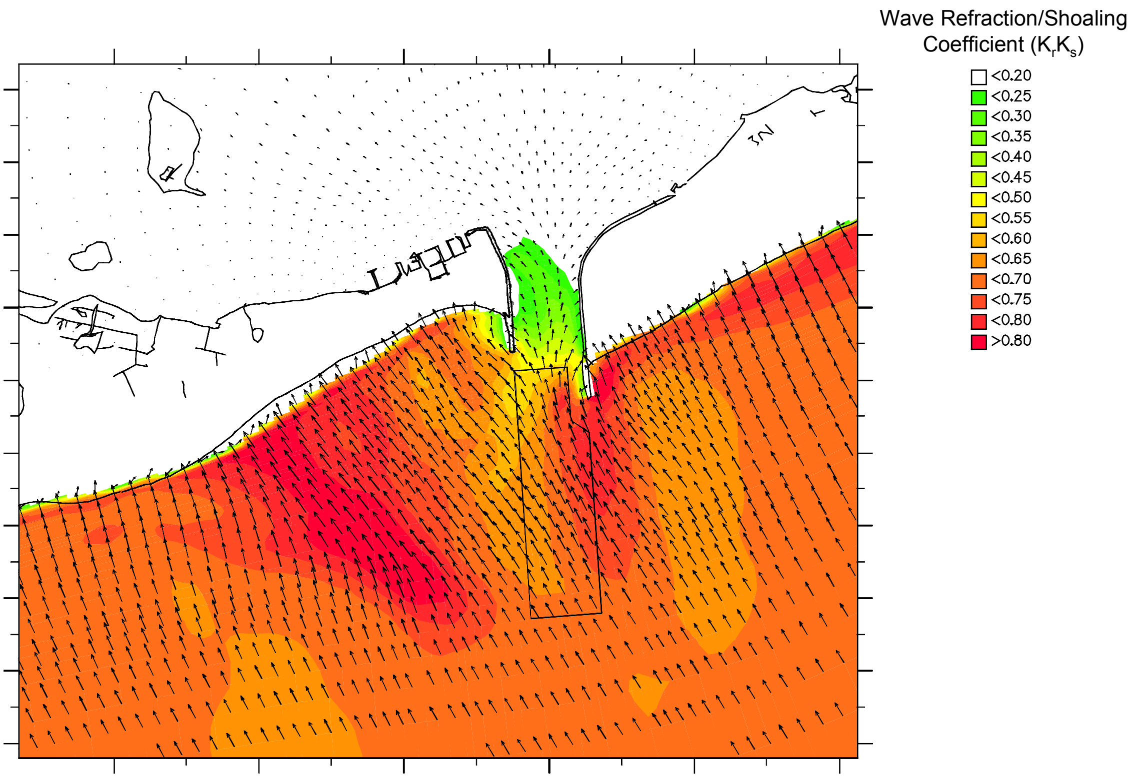
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 4-26

File pat/File name





Shinnecock Inlet- Sediment Transport and Waves (145° from North)

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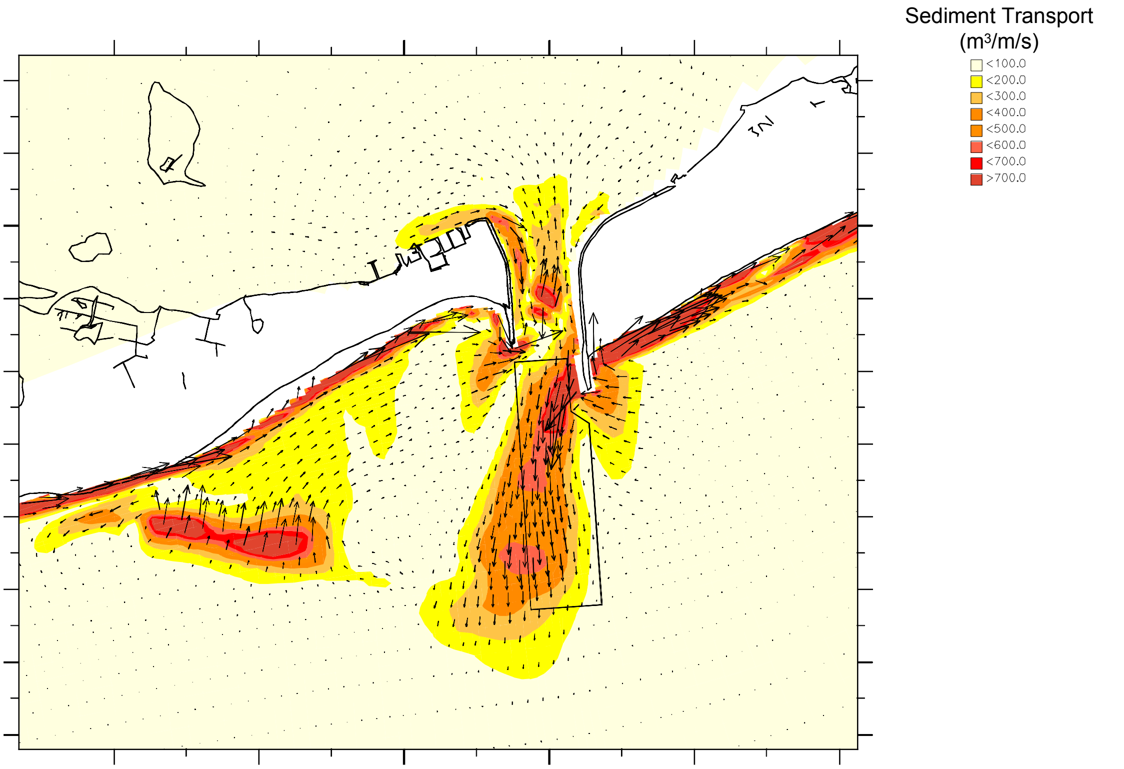
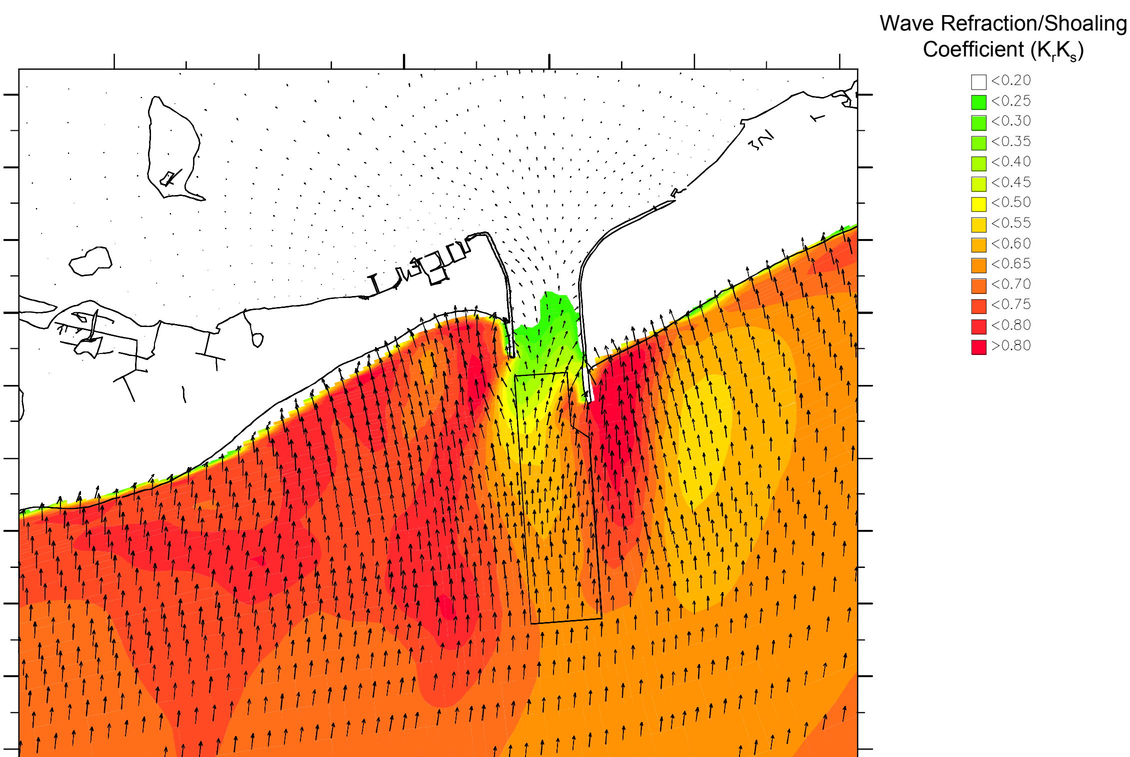
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 4-27

File pat/File name





Shinnecock Inlet- Sediment Transport and Waves (210° from North)

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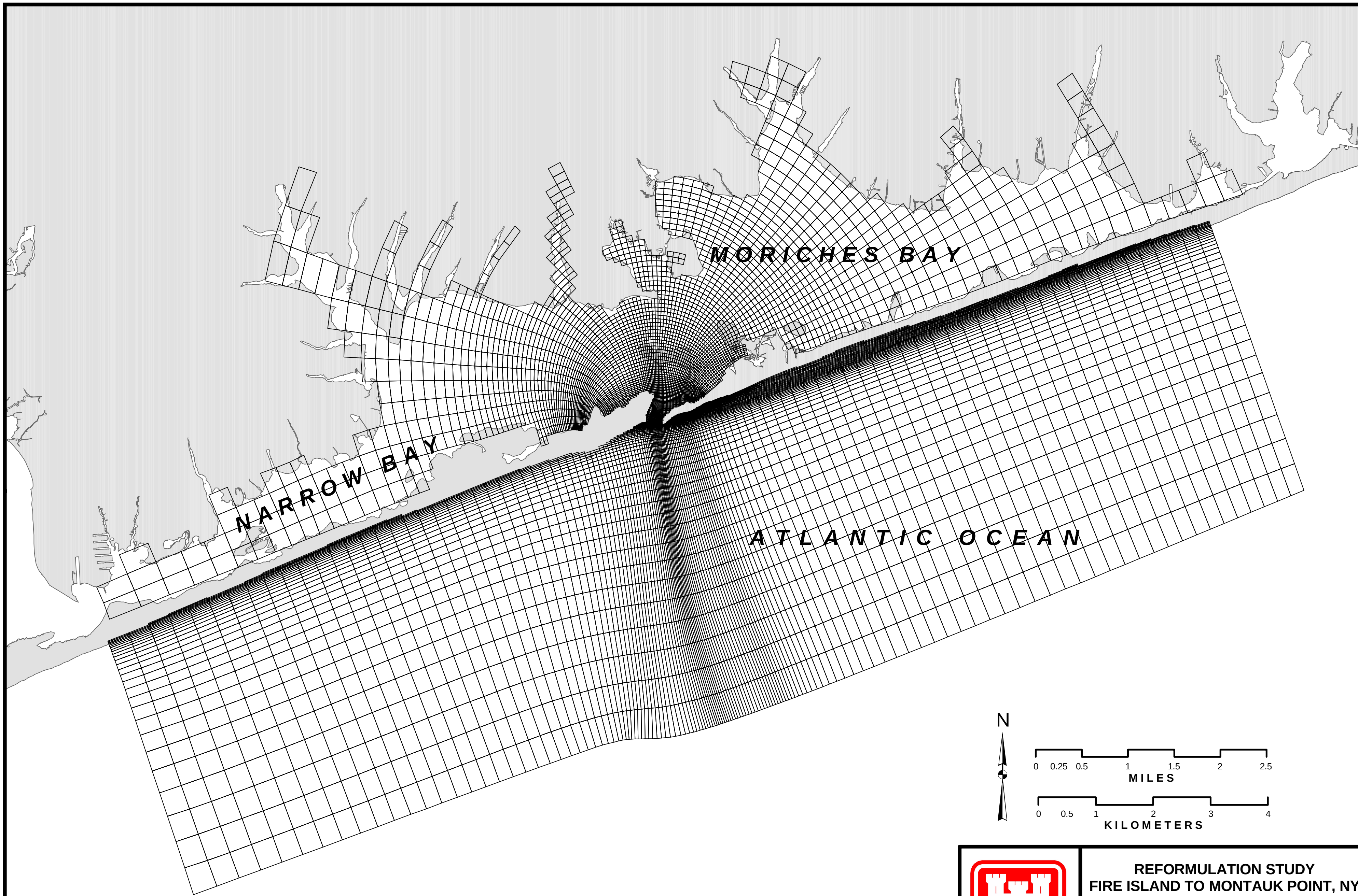
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 4-28

File pat/File name



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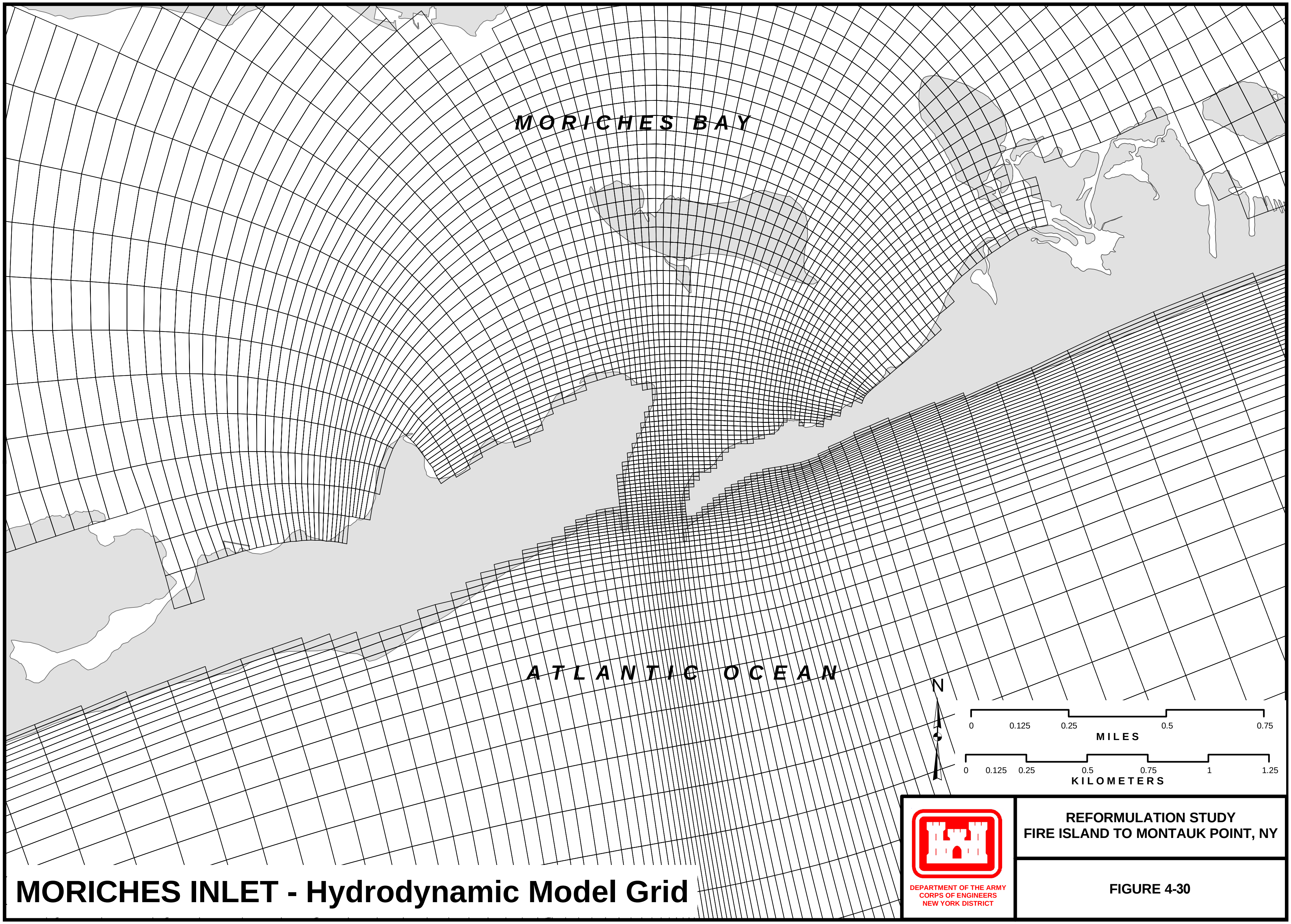
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 4-29

MORICHES BAY - Hydrodynamic Model Grid

Filename: CAD/3982-23/Report2002/Moriches_Model-Grid.mxd



MORICHES INLET - Hydrodynamic Model Grid

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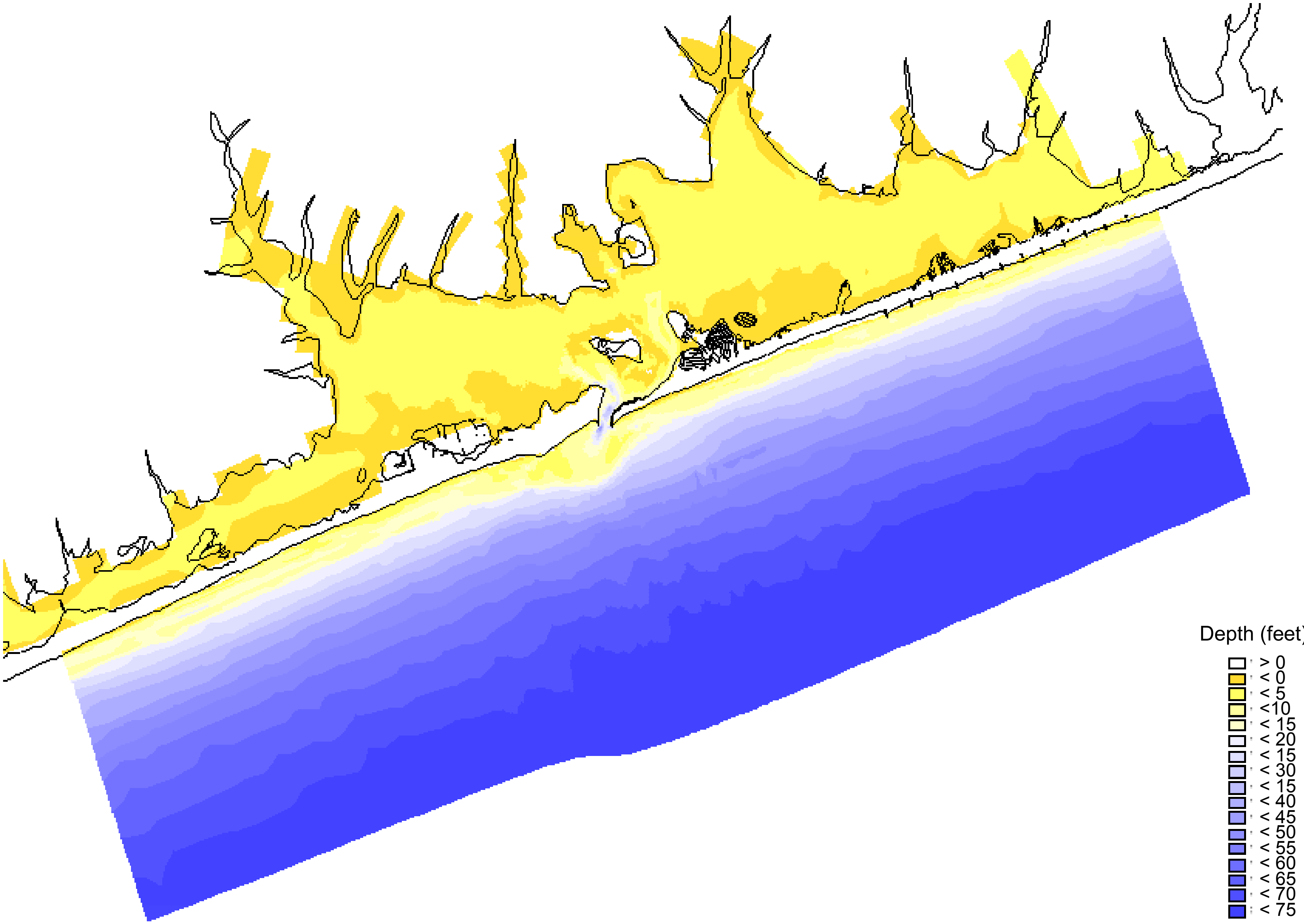
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 4-30

Filename: CAD/3982-23/Report2002/Moriches_Inlet-Grid.mxd



Moriches Inlet – Hydrodynamic Model Bathymetry

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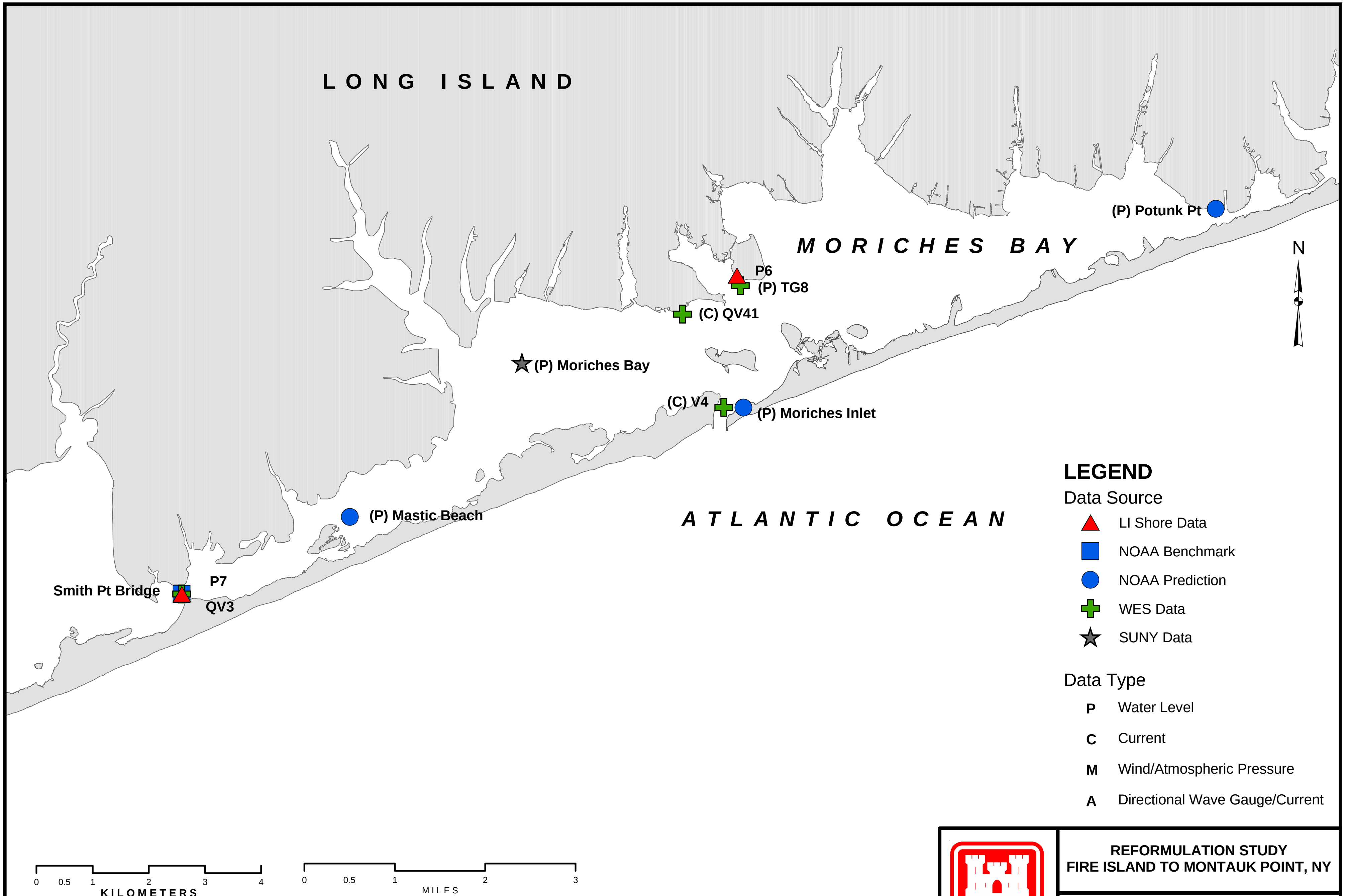
FIGURE 4-31

File pat/File name



MORICHES INLET- Location of Data Stations

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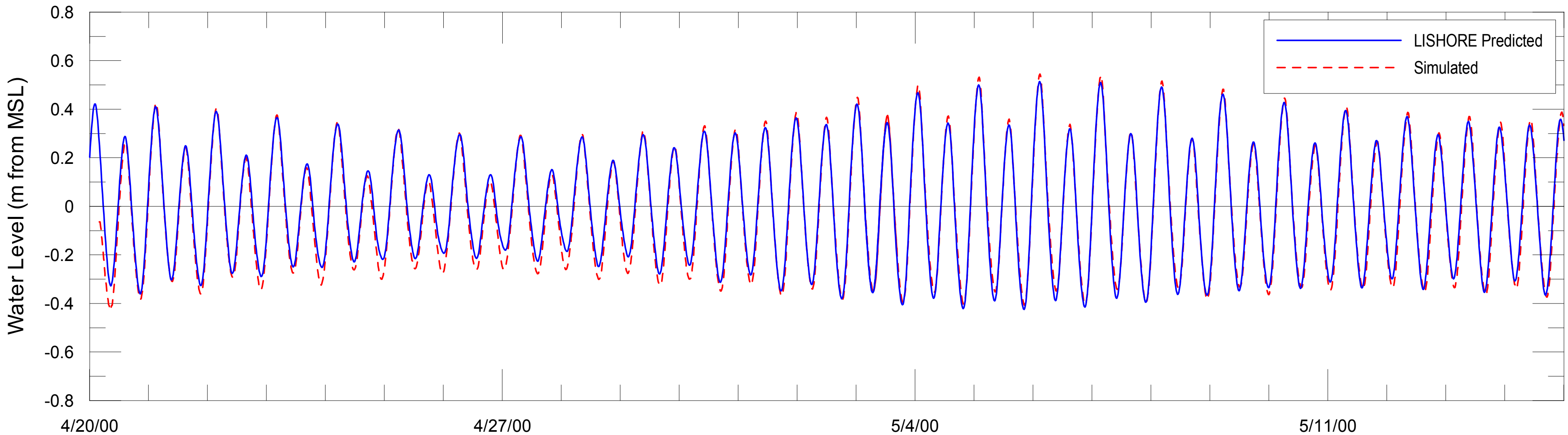
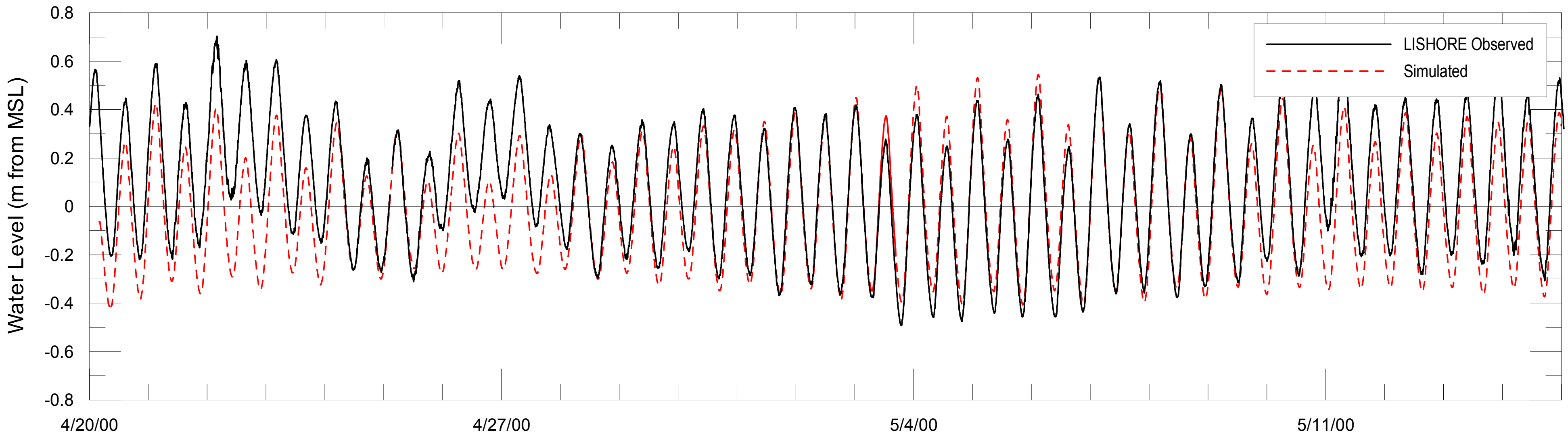
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 4-32

Filename: CAD/3982-23/Report2002/Moriches_Data-Location.mxd





Comparison of Predicted, Observed and Simulated Water Levels at Station P6

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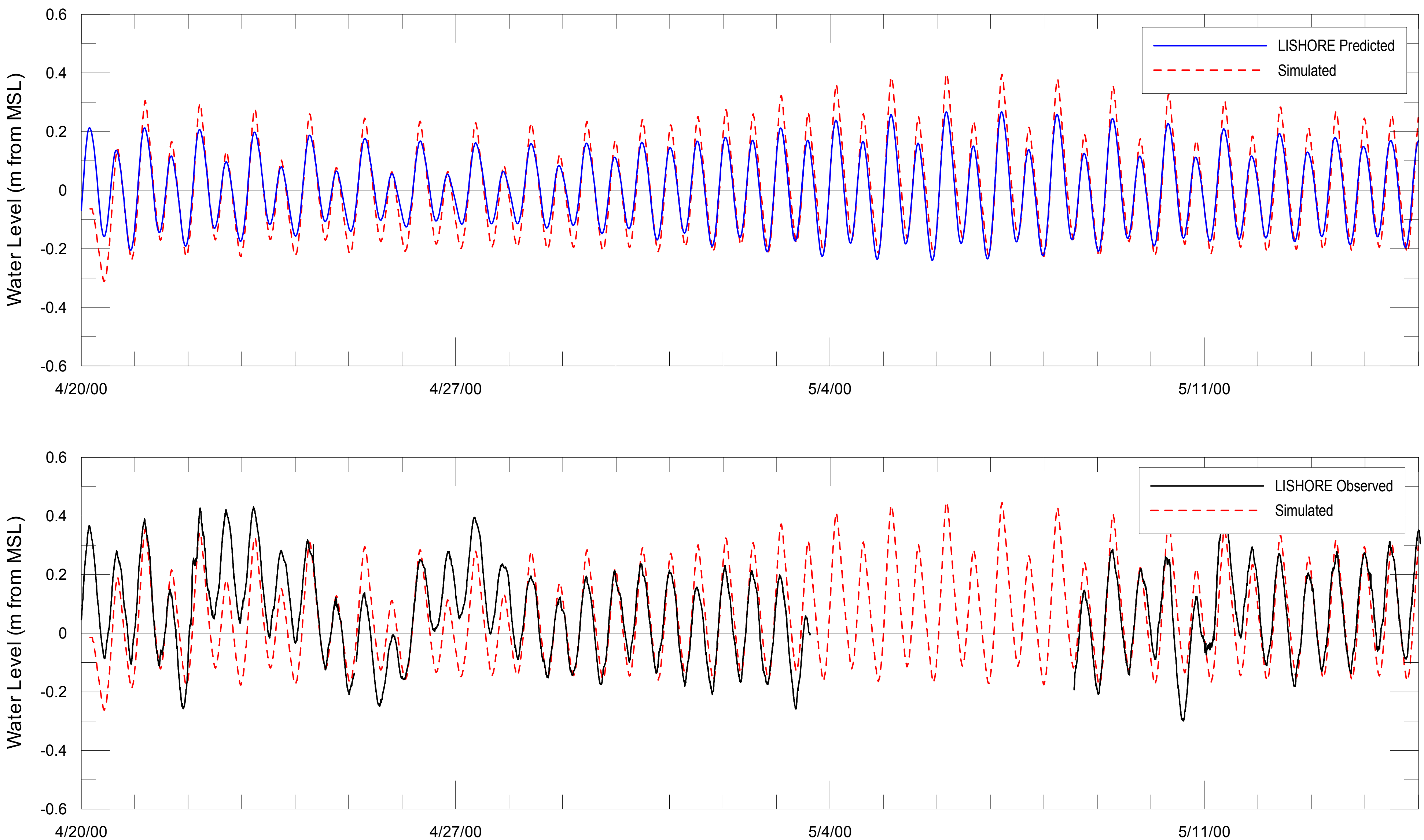
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 4-33



Comparison of Predicted, Observed and Simulated Water Levels at Station P7

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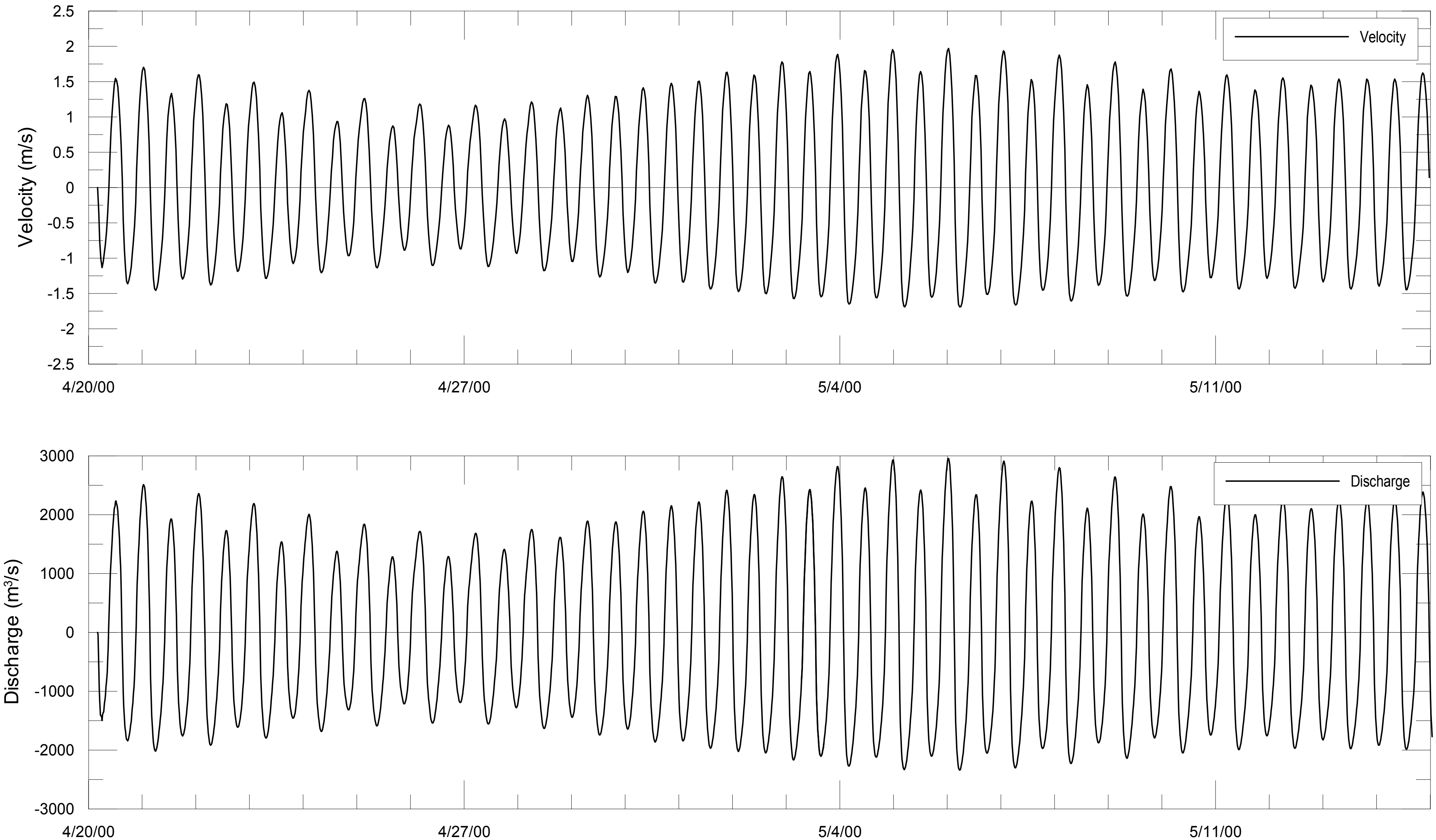
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 4-34



Simulated Velocity and Depth Averaged Discharge at Moriches Inlet

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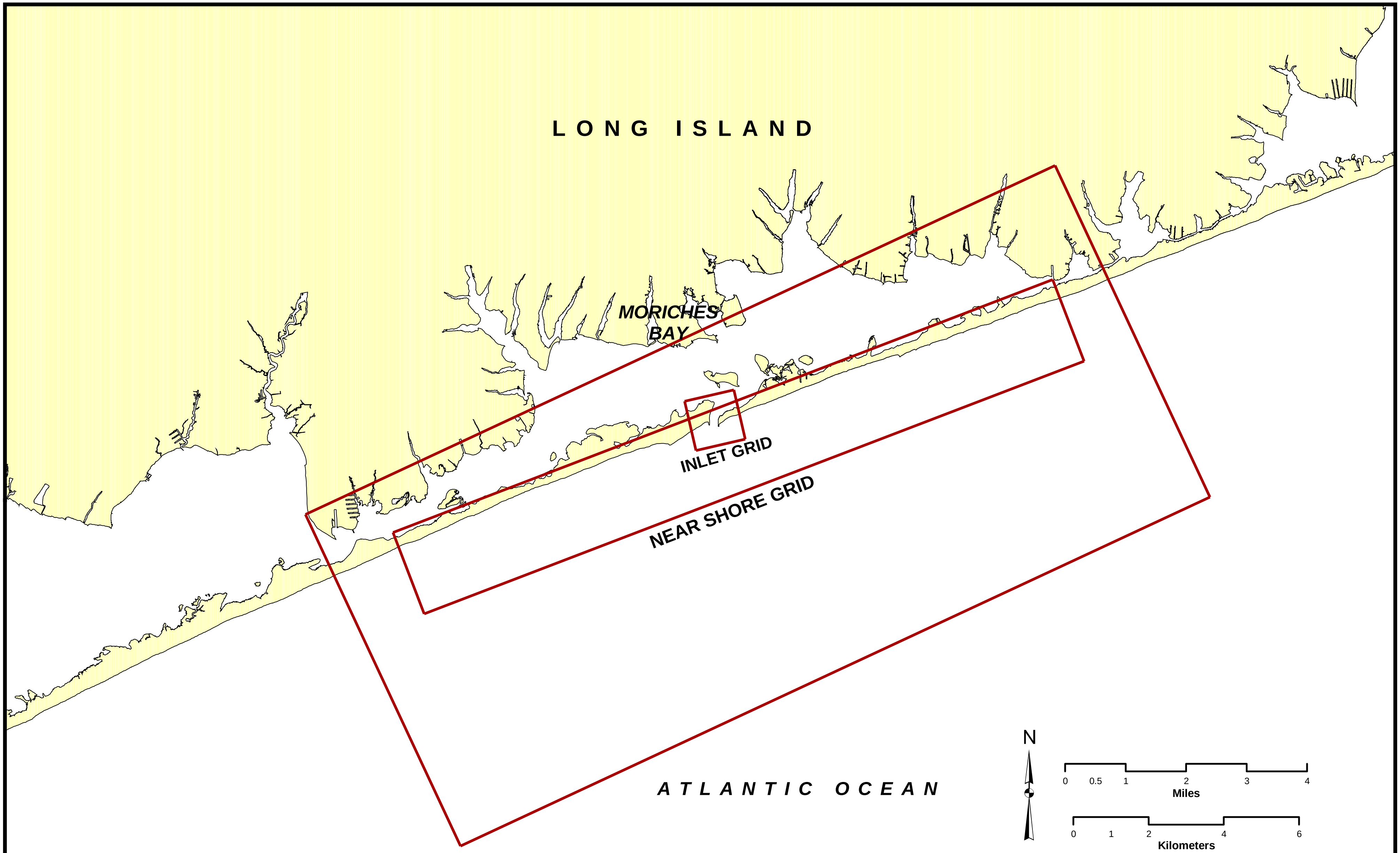
FIGURE 4-35



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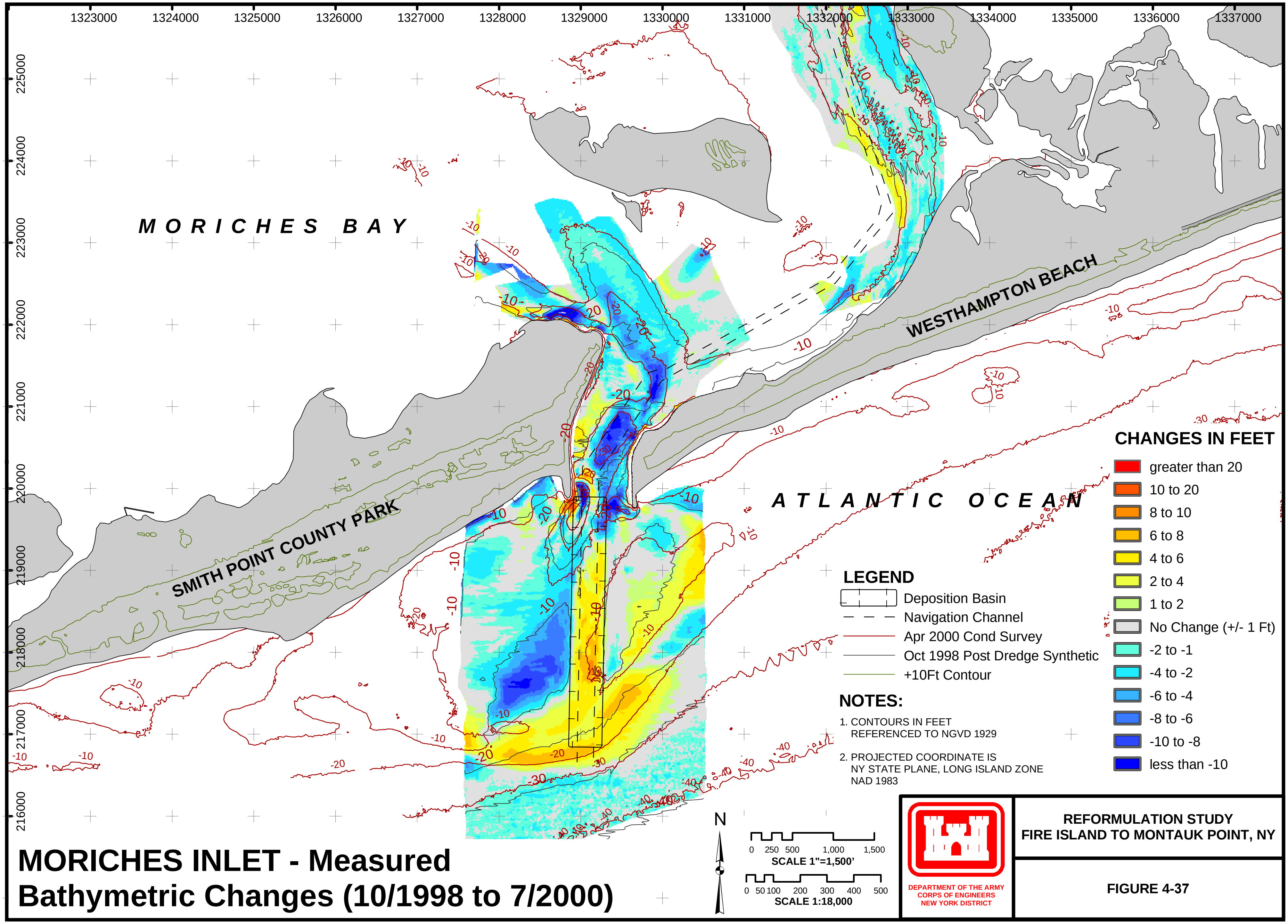
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 4-36

MORICHES INLET - Wave Model Grids

Filename: R:\GIS\3982 FIMP\3982-28 Inlet Modifications\Reports_MXDfiles\Dec2002\Wave_Grid_Moriches.mxd

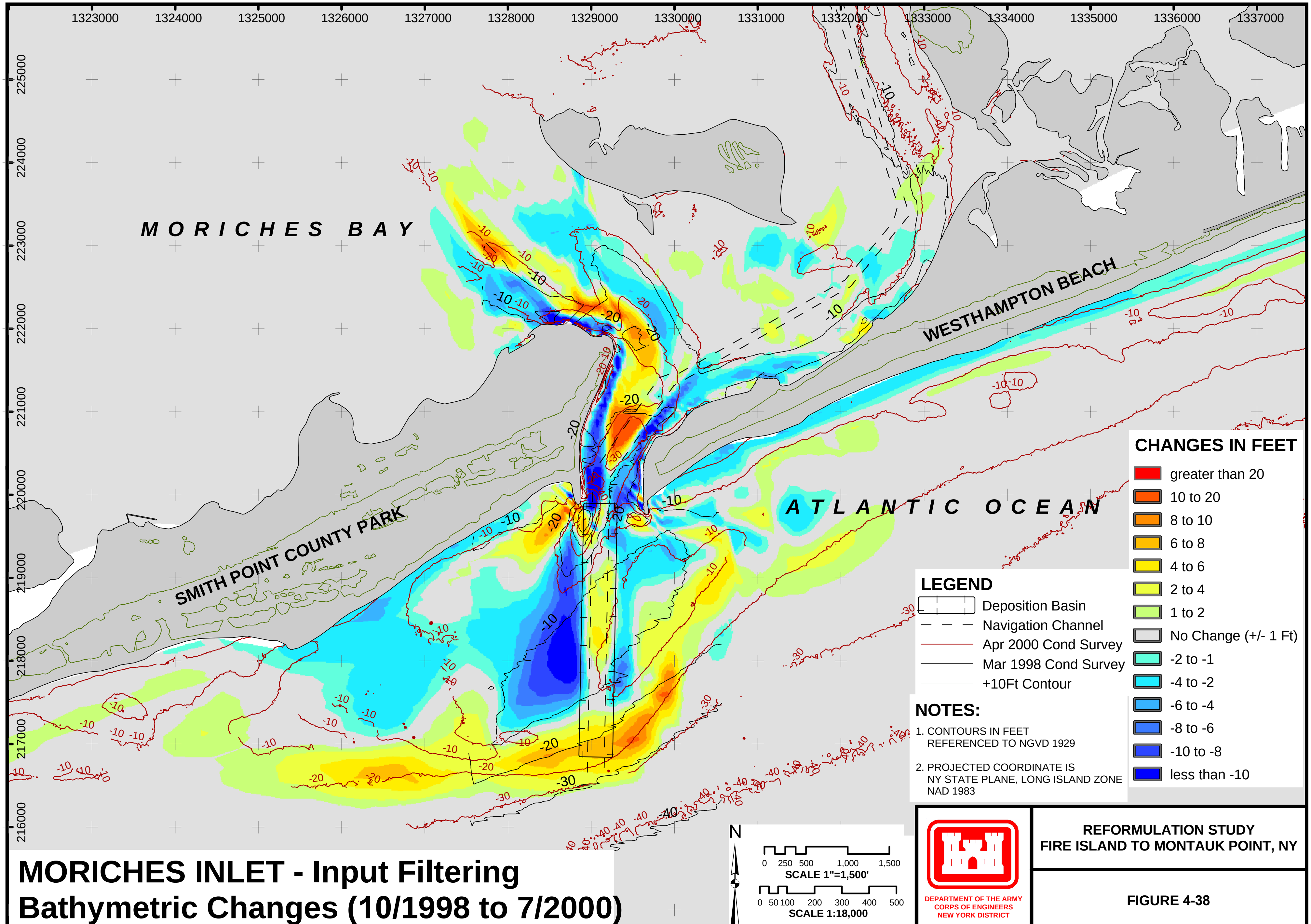


MORICHES INLET - Measured Bathymetric Changes (10/1998 to 7/2000)

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 4-37

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Moriches_Diff.mxd

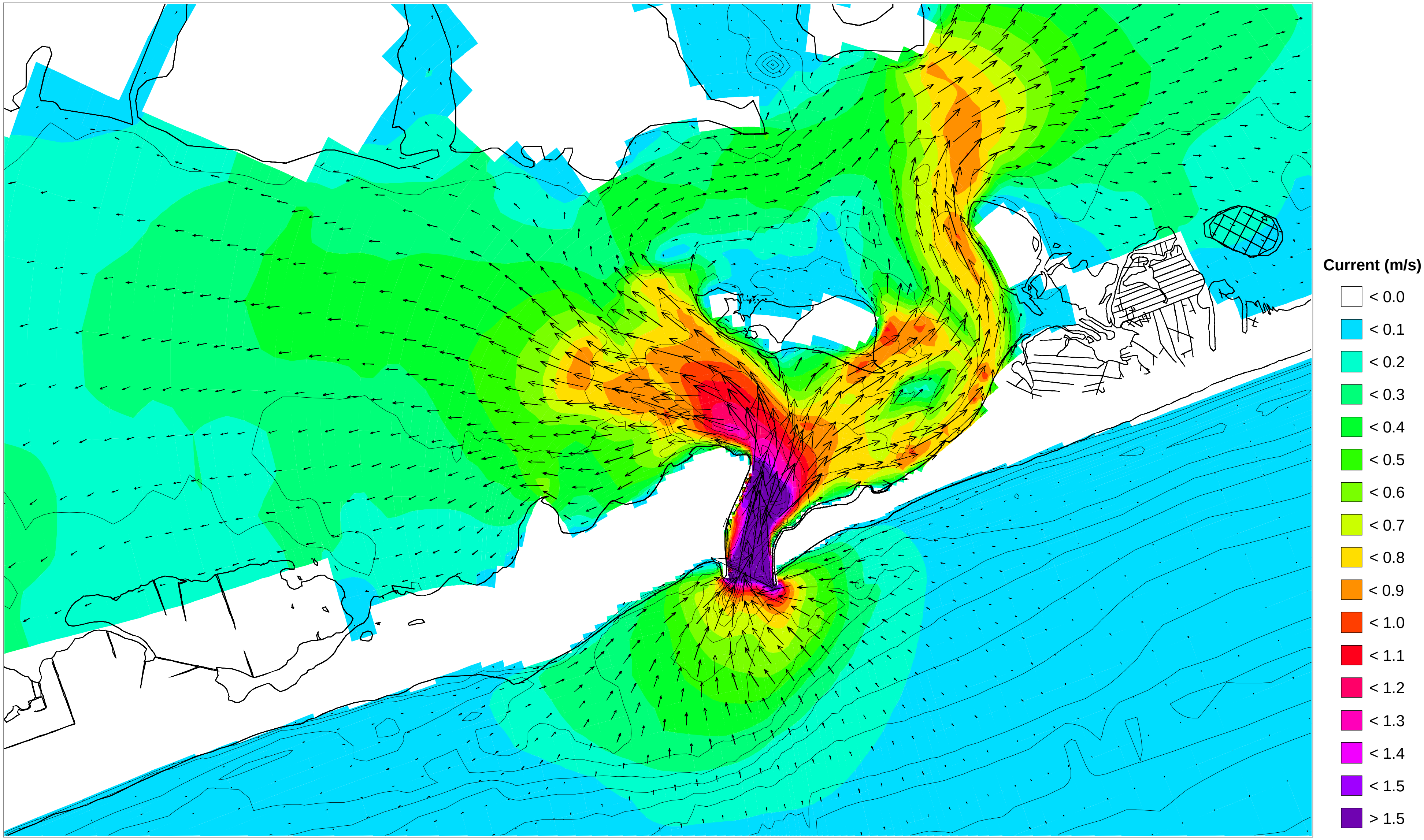


MORICHES INLET - Input Filtering Bathymetric Changes (10/1998 to 7/2000)

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 4-38

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Moriches_Diff.mxd



Moriches Inlet Flow Velocity – Peak Flood Tide

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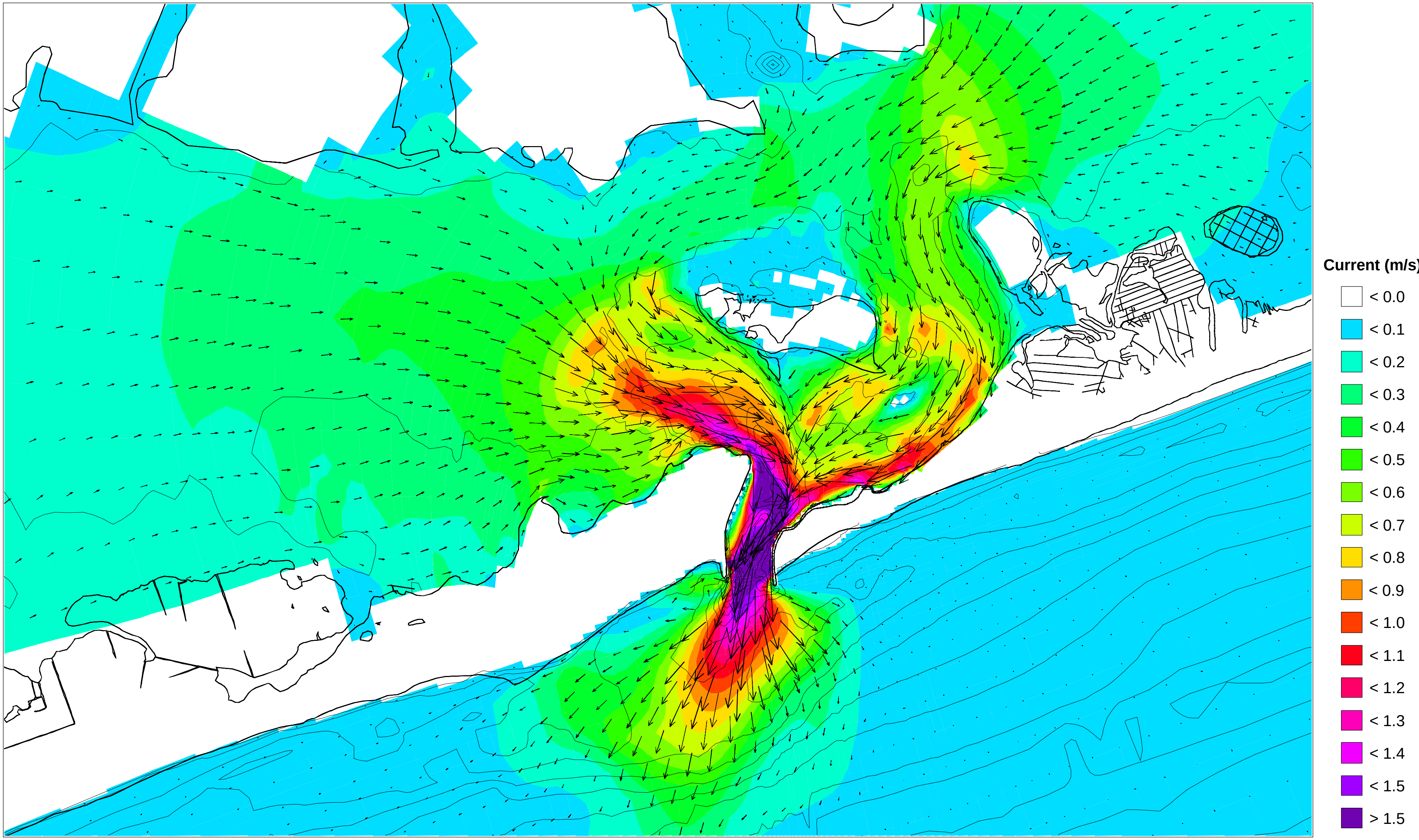
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 4-39

File pat/File name



Moriches Inlet Flow Velocity – Peak Ebb Tide

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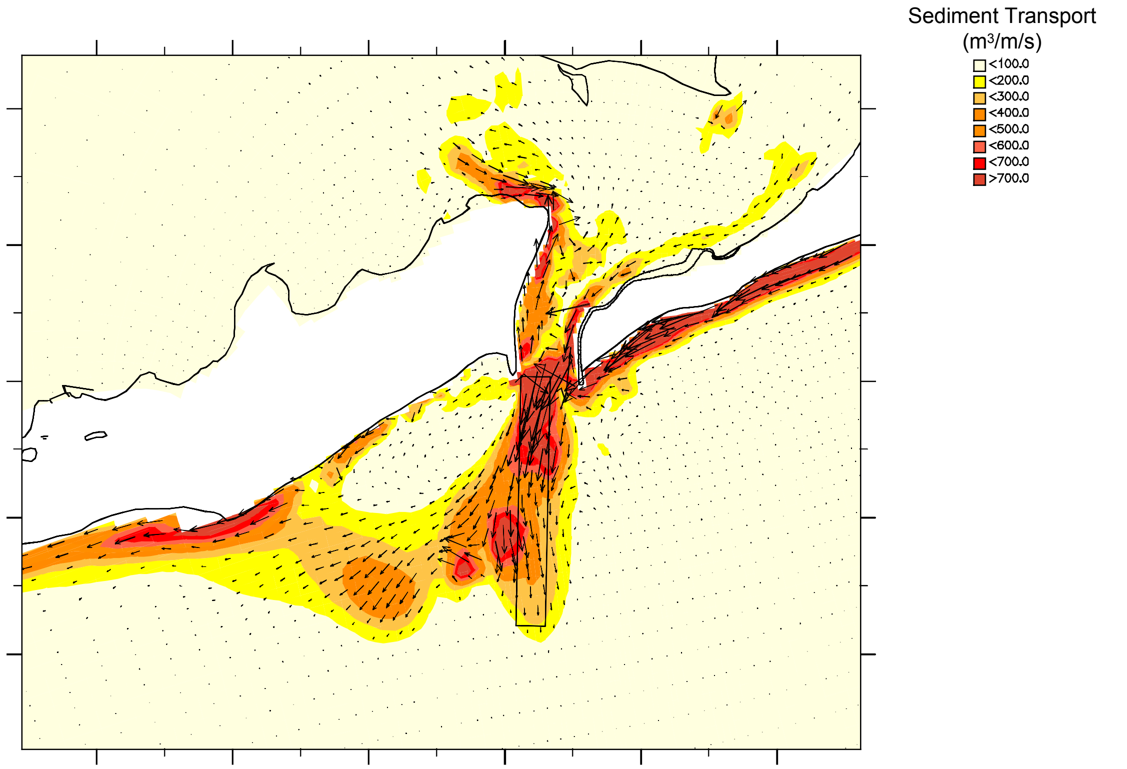
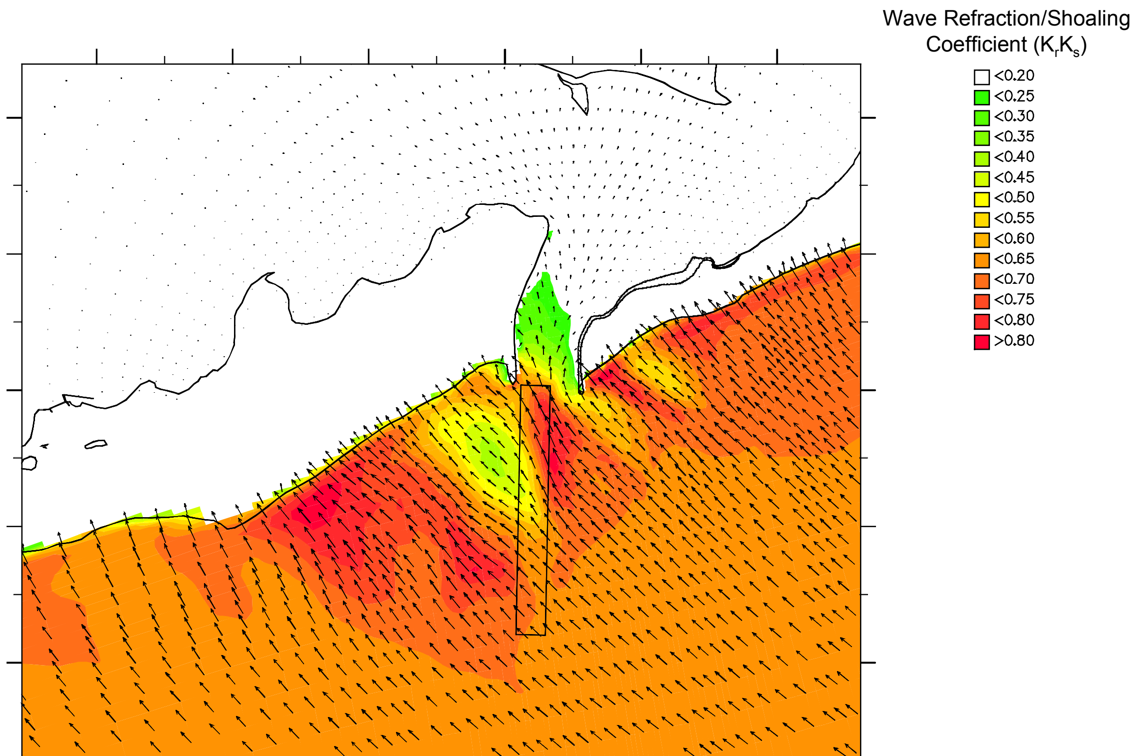
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 4-40





Moriches Inlet- Sediment Transport and Waves (110° from North)

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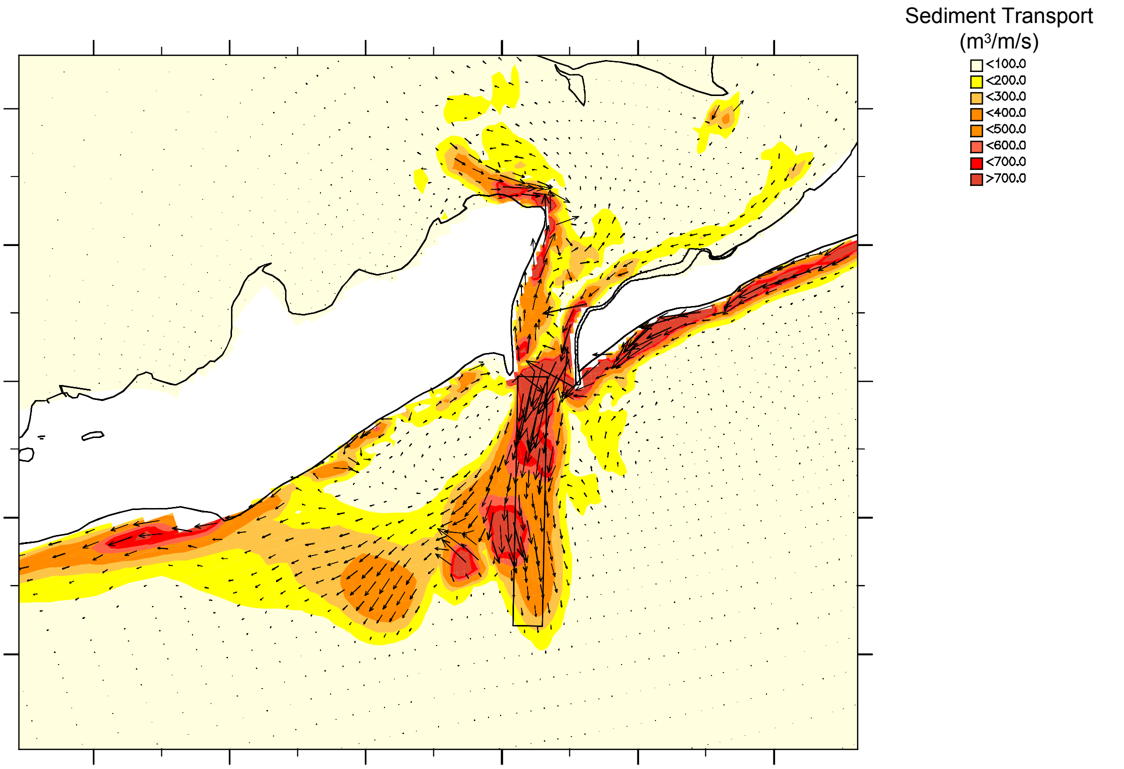
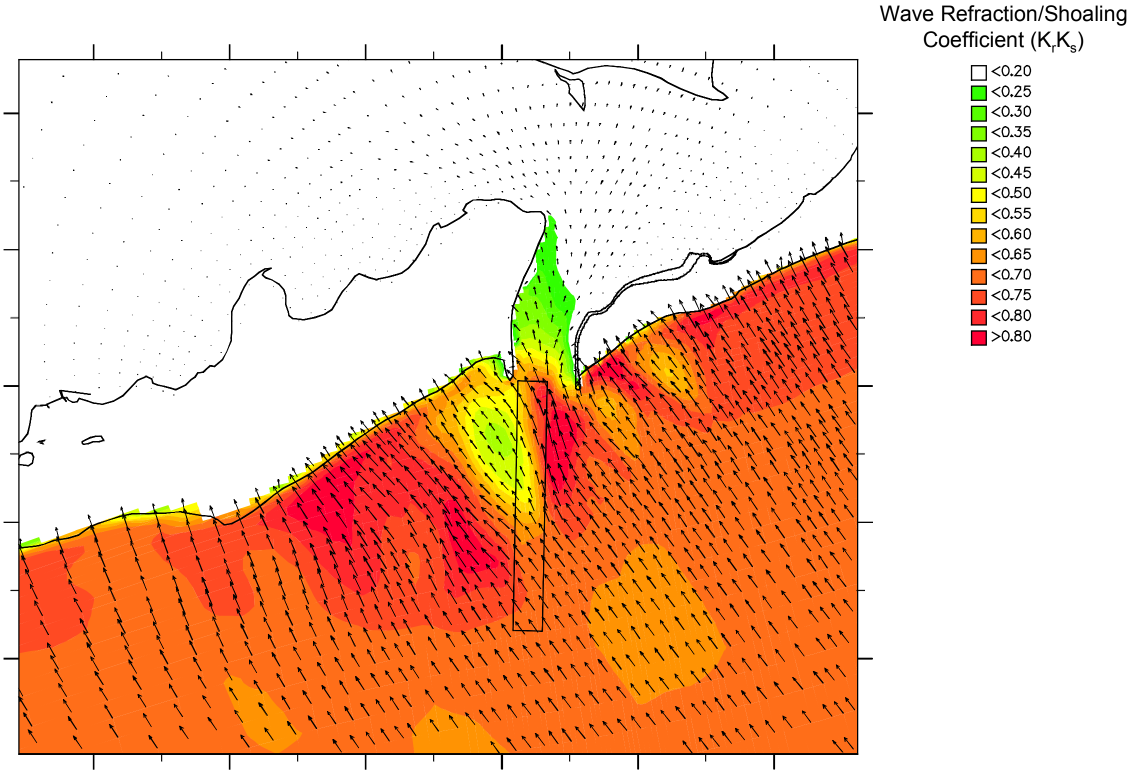
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 4-41

File pat/File name





Moriches Inlet- Sediment Transport and Waves (135° from North)

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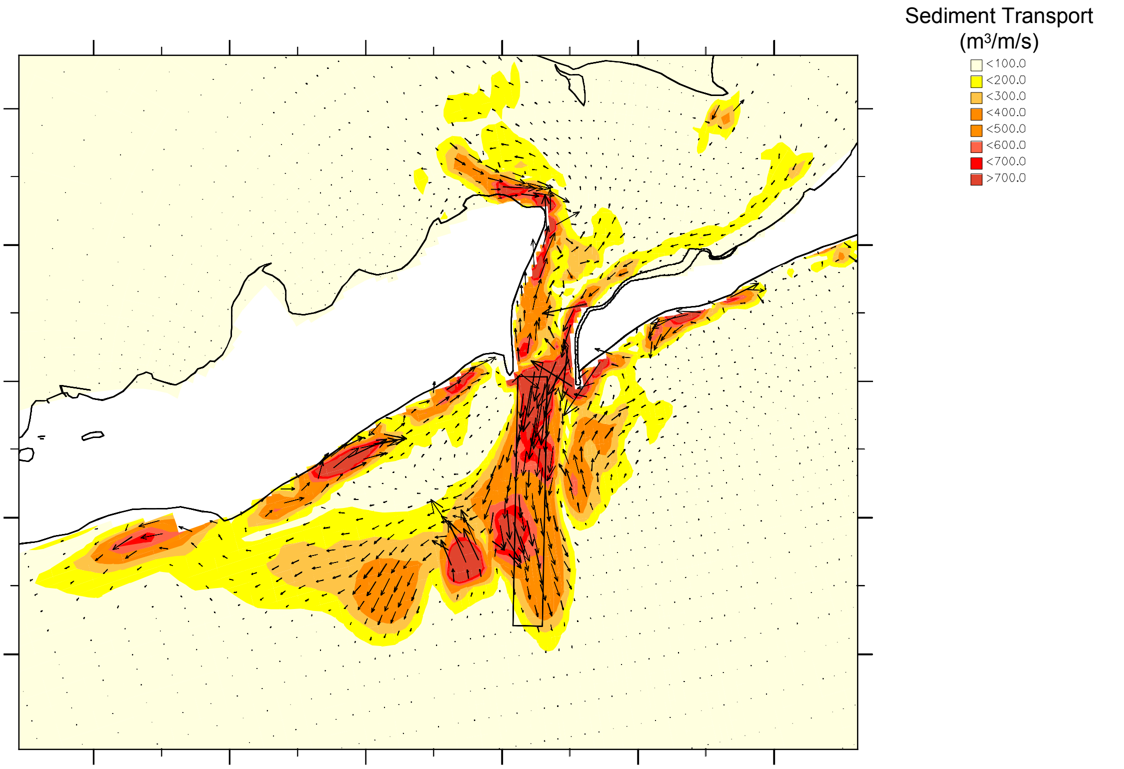
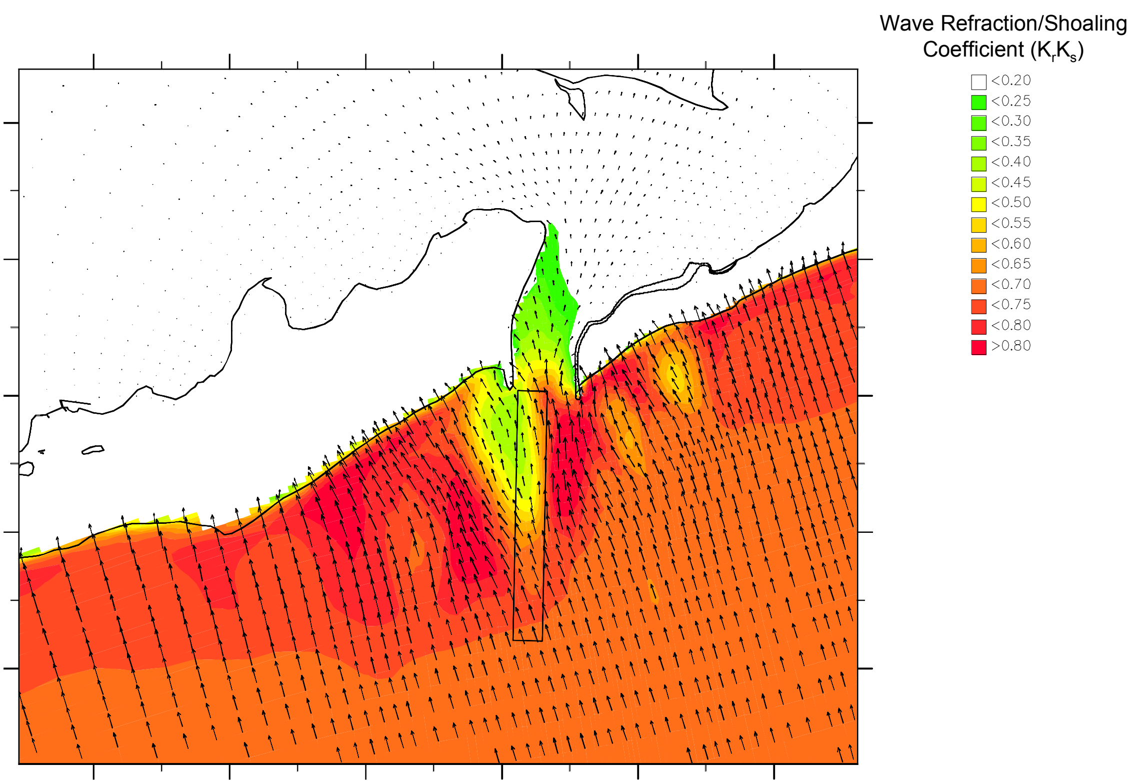
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 4-42

File pat/File name





Moriches Inlet- Sediment Transport and Waves (165° from North)

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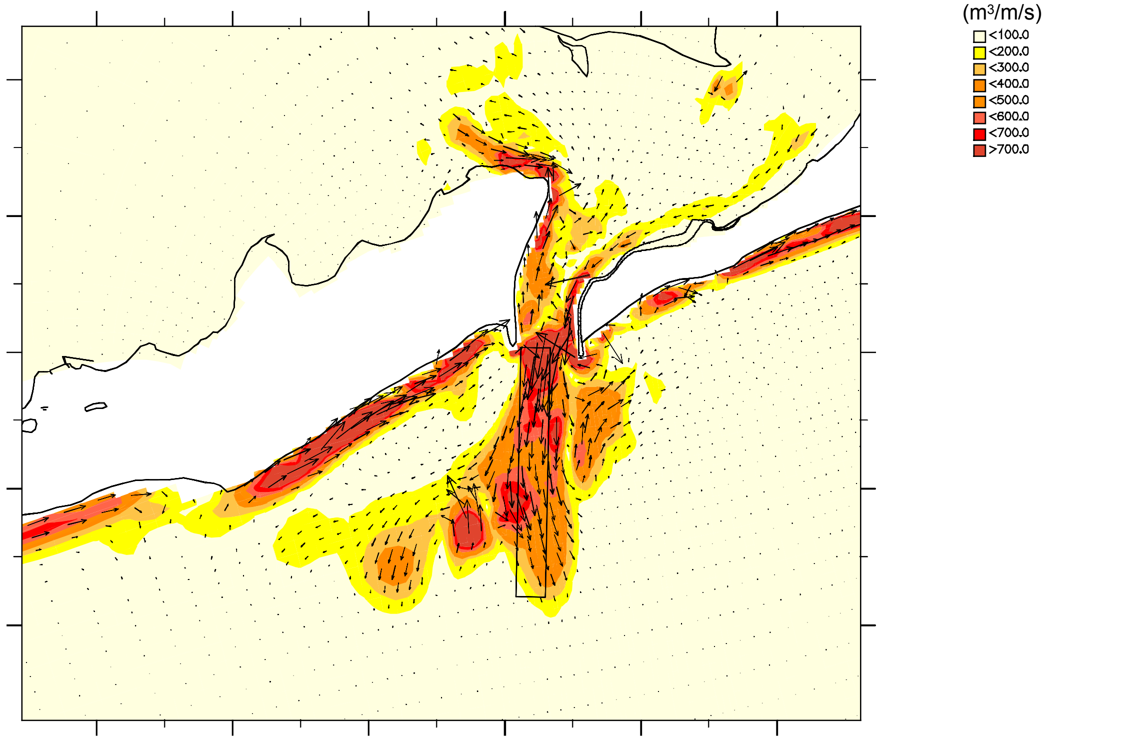
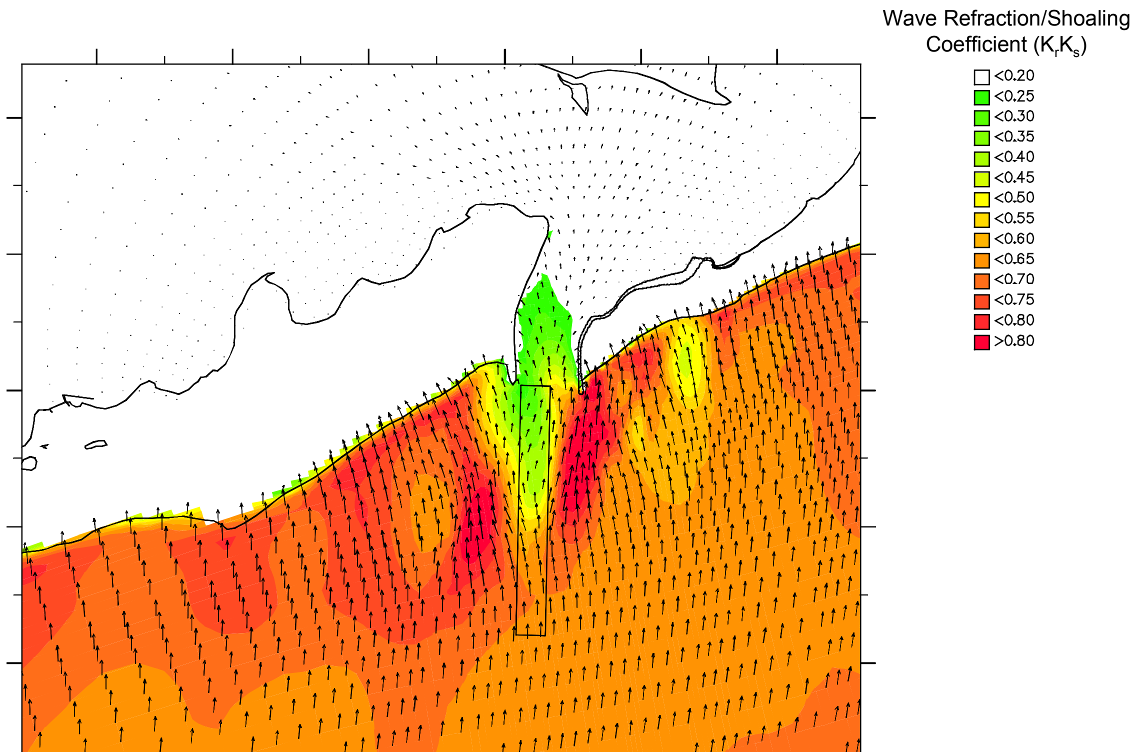
FIGURE 4-43

File pat/File name





Moriches Inlet- Sediment Transport and Waves (210˚ from North)

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FIGURE 4-44

File pat/File name

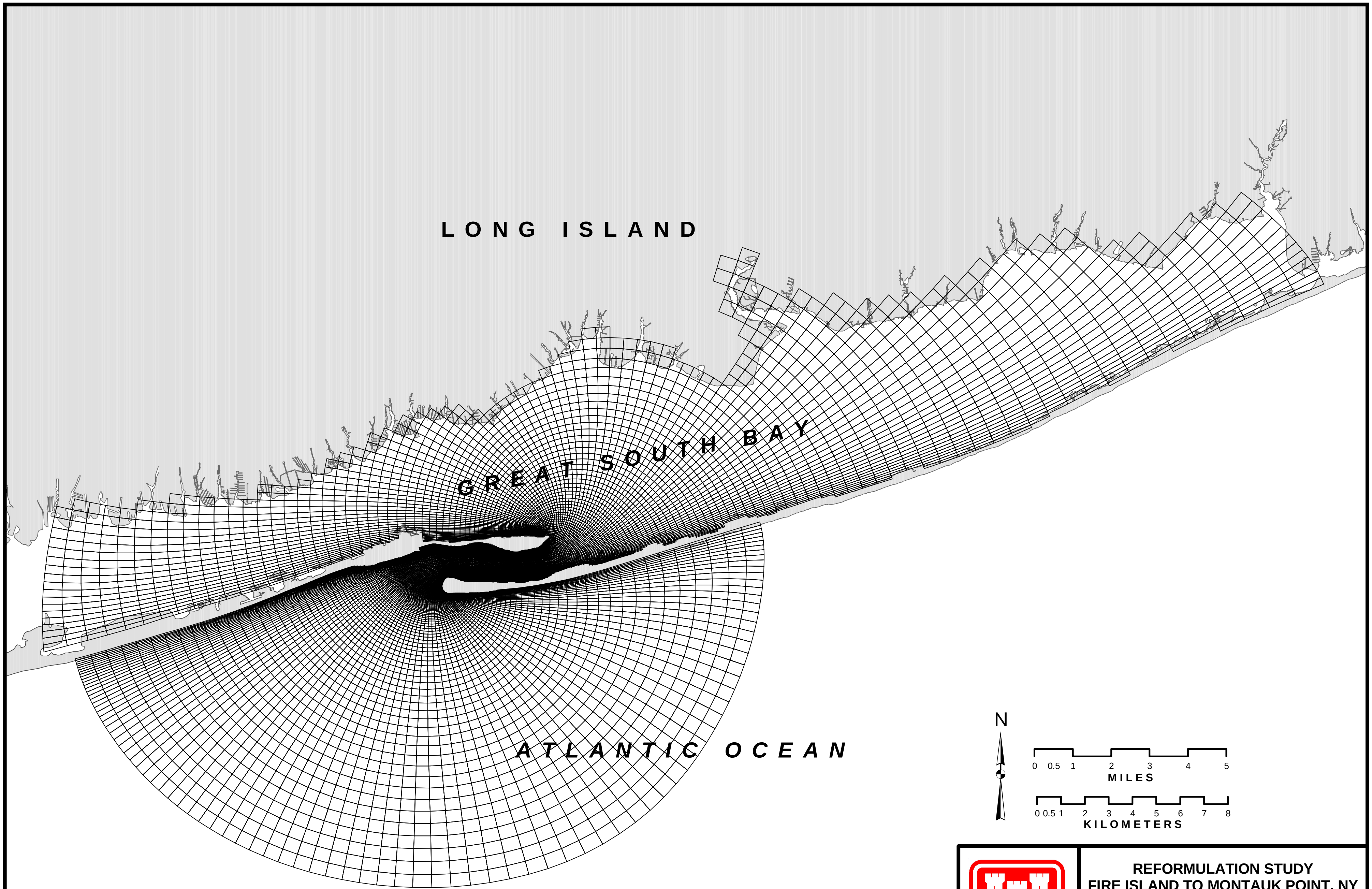

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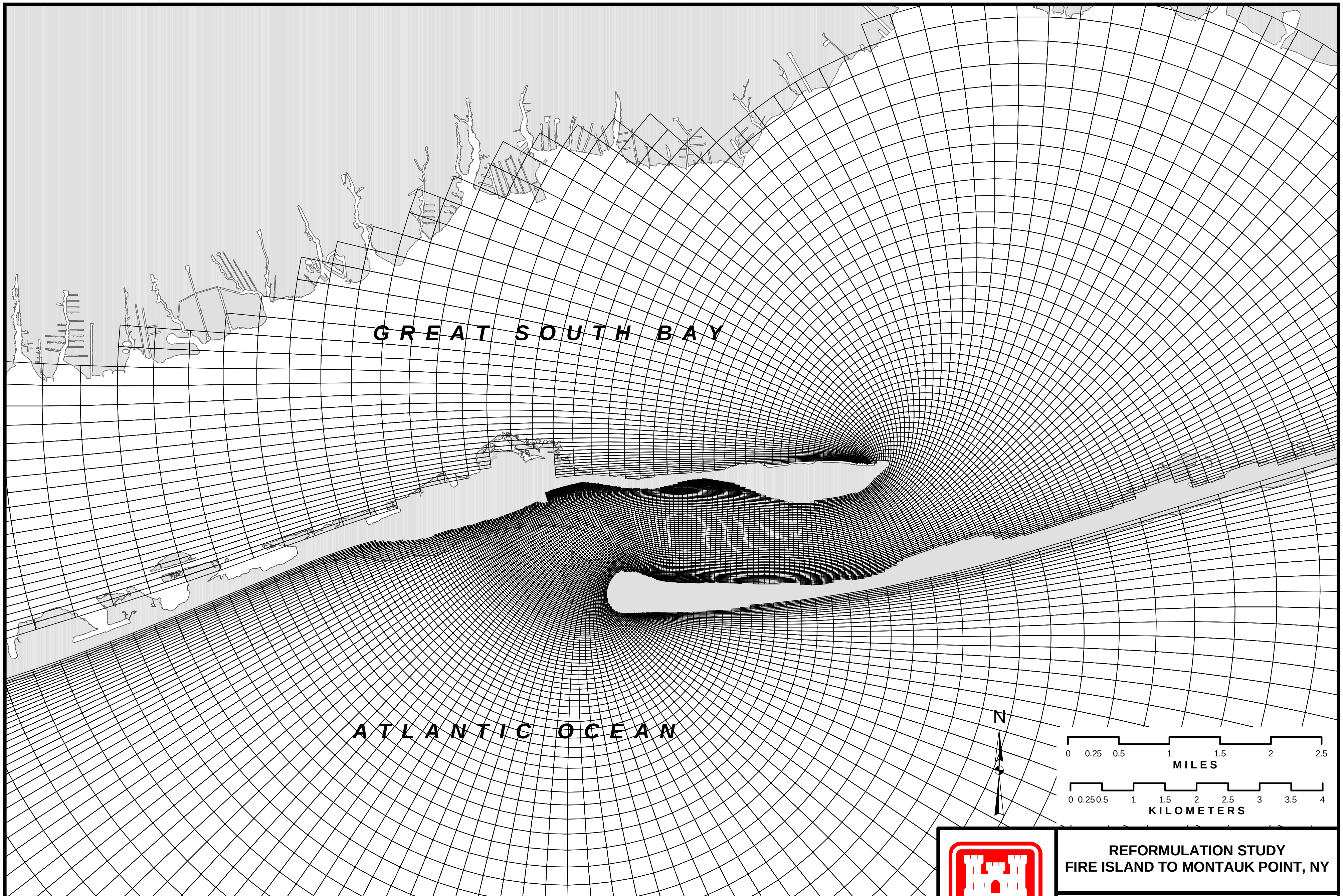
FIGURE 4-45

GREAT SOUTH BAY - Hydrodynamic Model Grid

Filename: R:\GIS\3982 FIMP\3982-28 Inlet Modifications\Reports_MXDfiles\Dec2002\FireIsl_Model-Grid.mxd



FIRE ISLAND INLET - Hydrodynamic Model Grid

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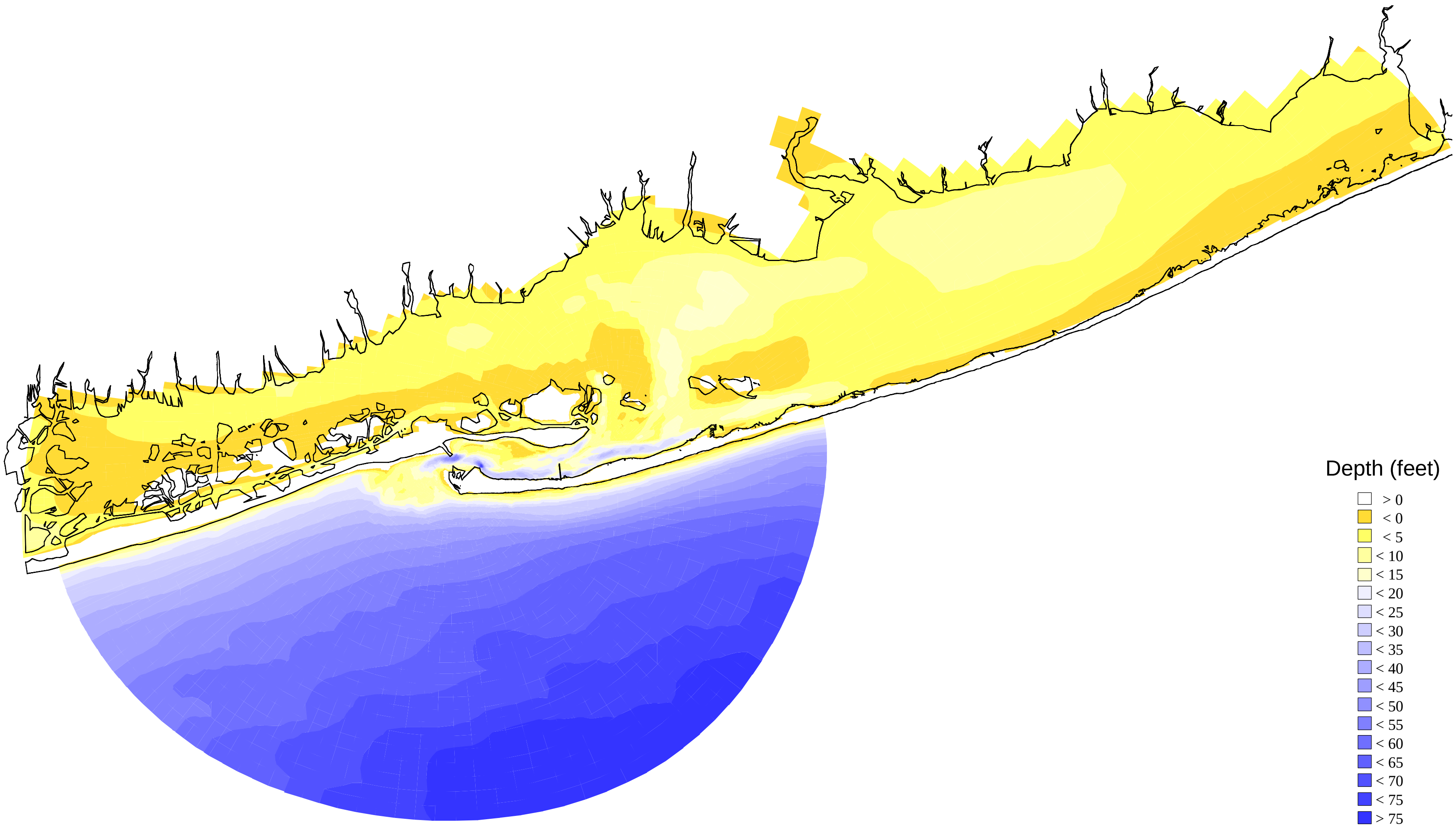
FIGURE 4-46

Filename: R:\GIS\3982 FIMP\3982-28 Inlet Modifications\Reports_MXDfiles\Dec2002\FireIsl_Inlet-Grid.mxd



Great South Bay – Hydrodynamic Model Bathymetry

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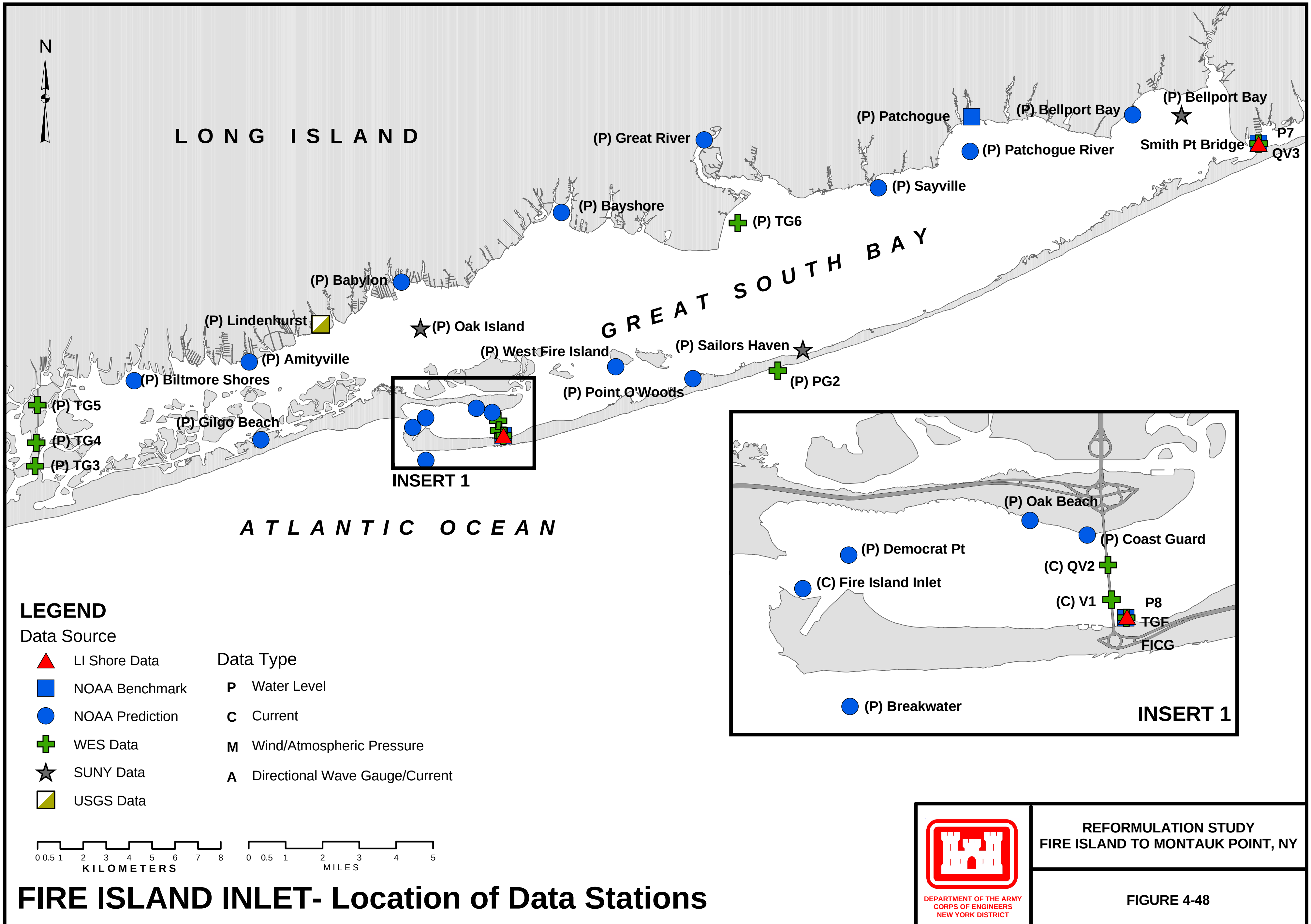
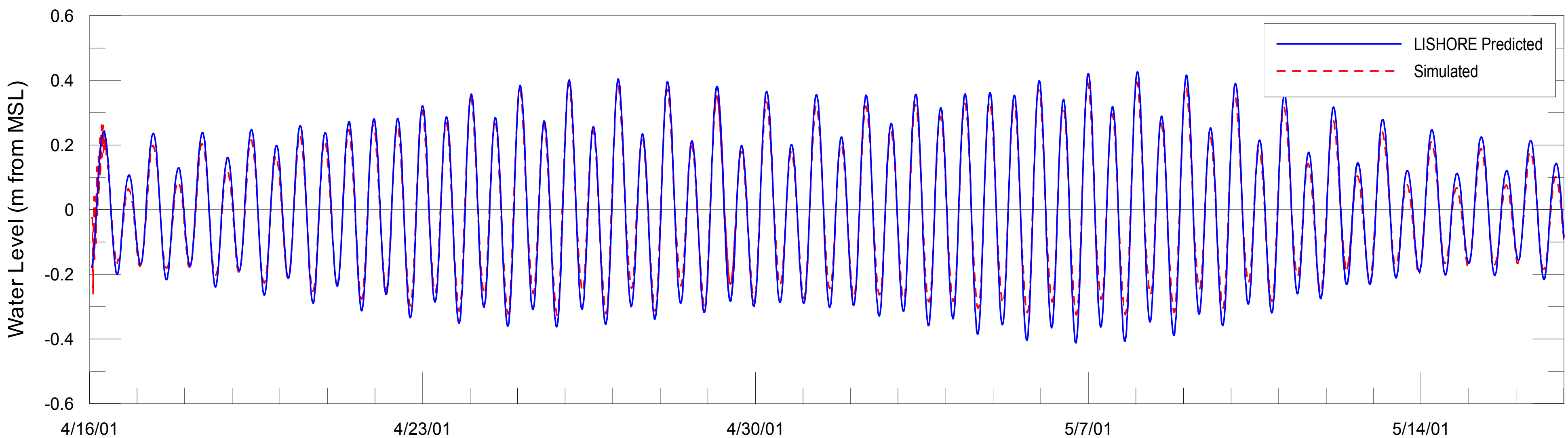
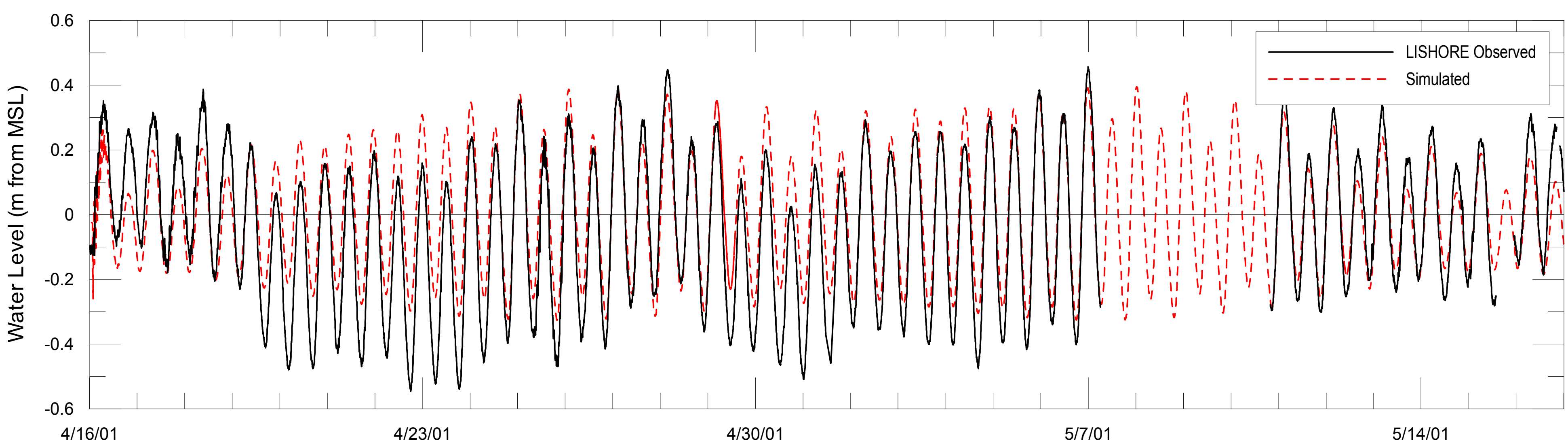
FIGURE 4-47



Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/FireIsl_Data-Location.mxd





Comparison of Predicted, Observed and Simulated Water Levels at Station P8 (FI CGS)

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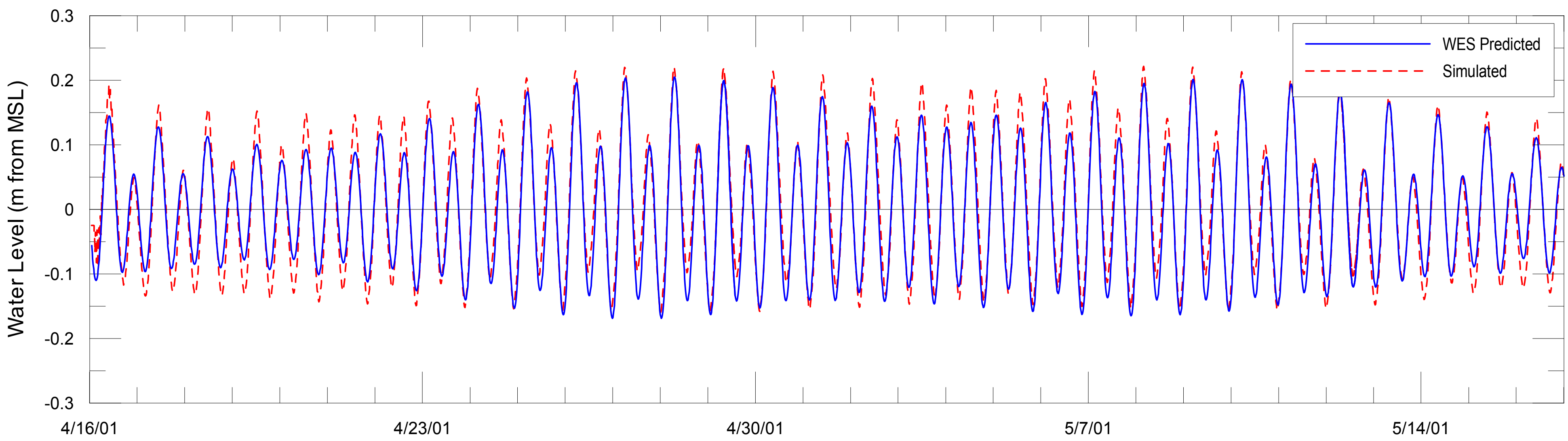
FIGURE 4-49



Comparison of Predicted and Simulated Water Levels at WES Station TG6

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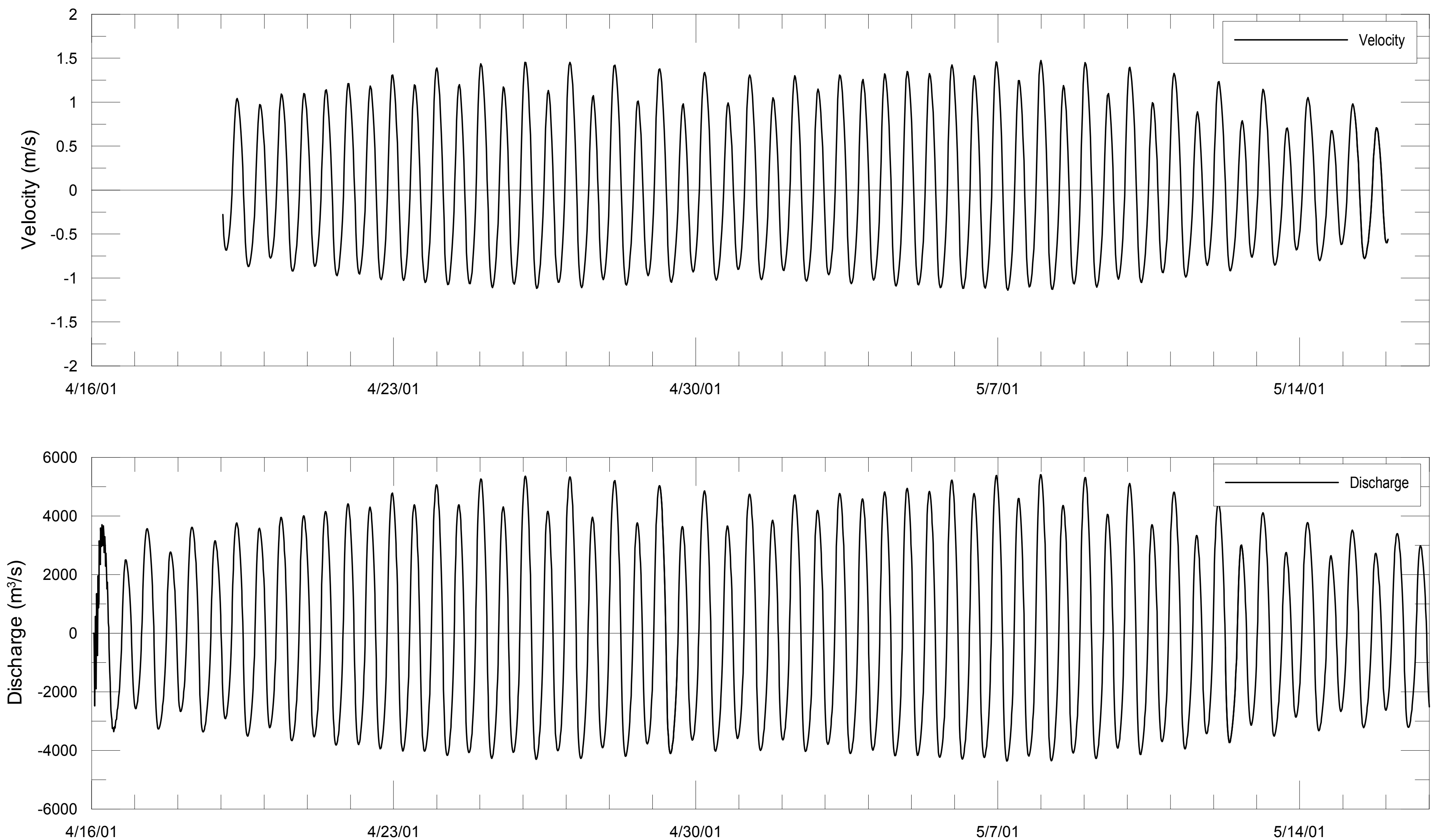
FIGURE 4-50



Simulated Velocity and Depth Averaged Discharge at Fire Island Inlet

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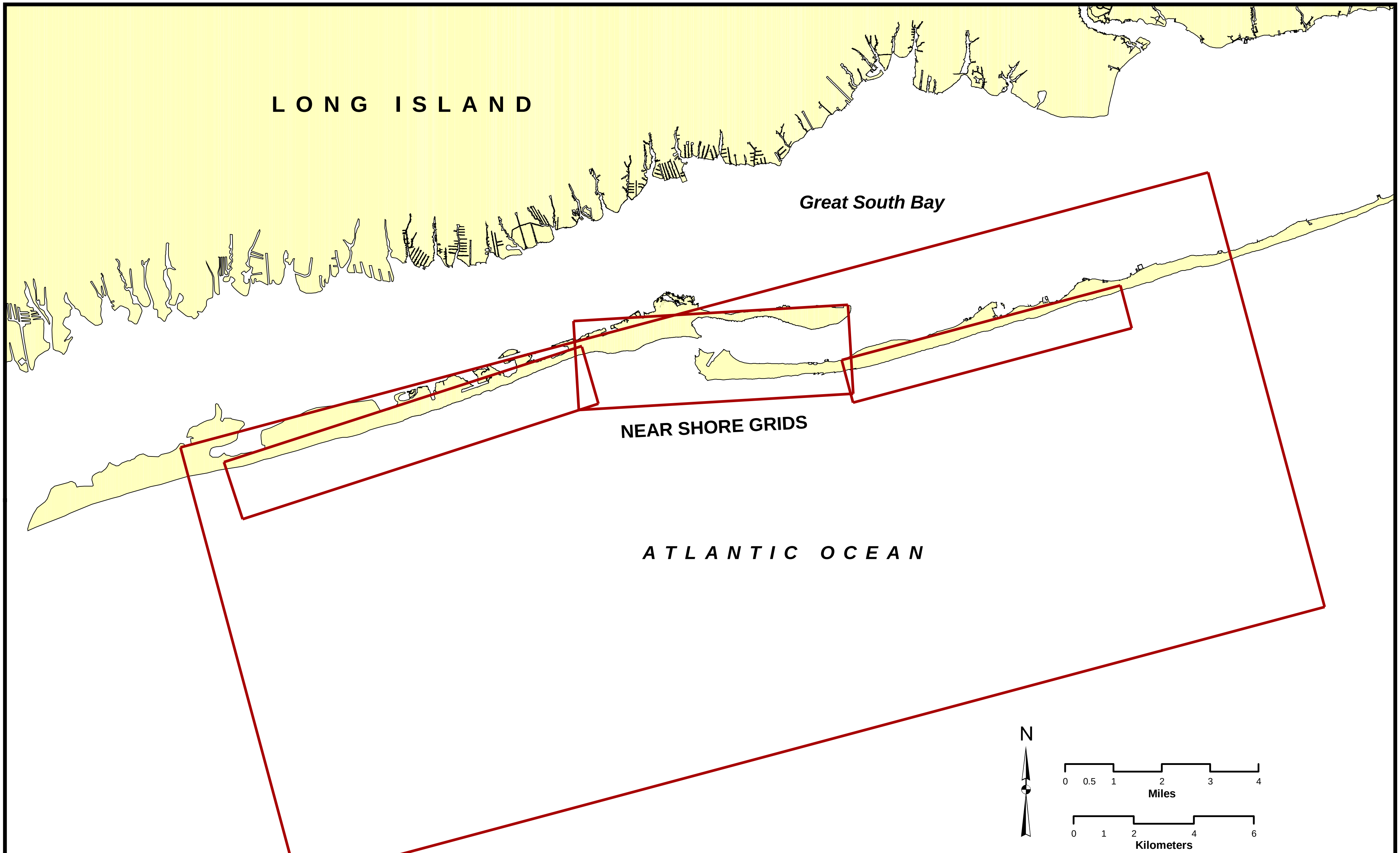
FIGURE 4-51



FIRE ISLAND INLET - Wave Model Grids

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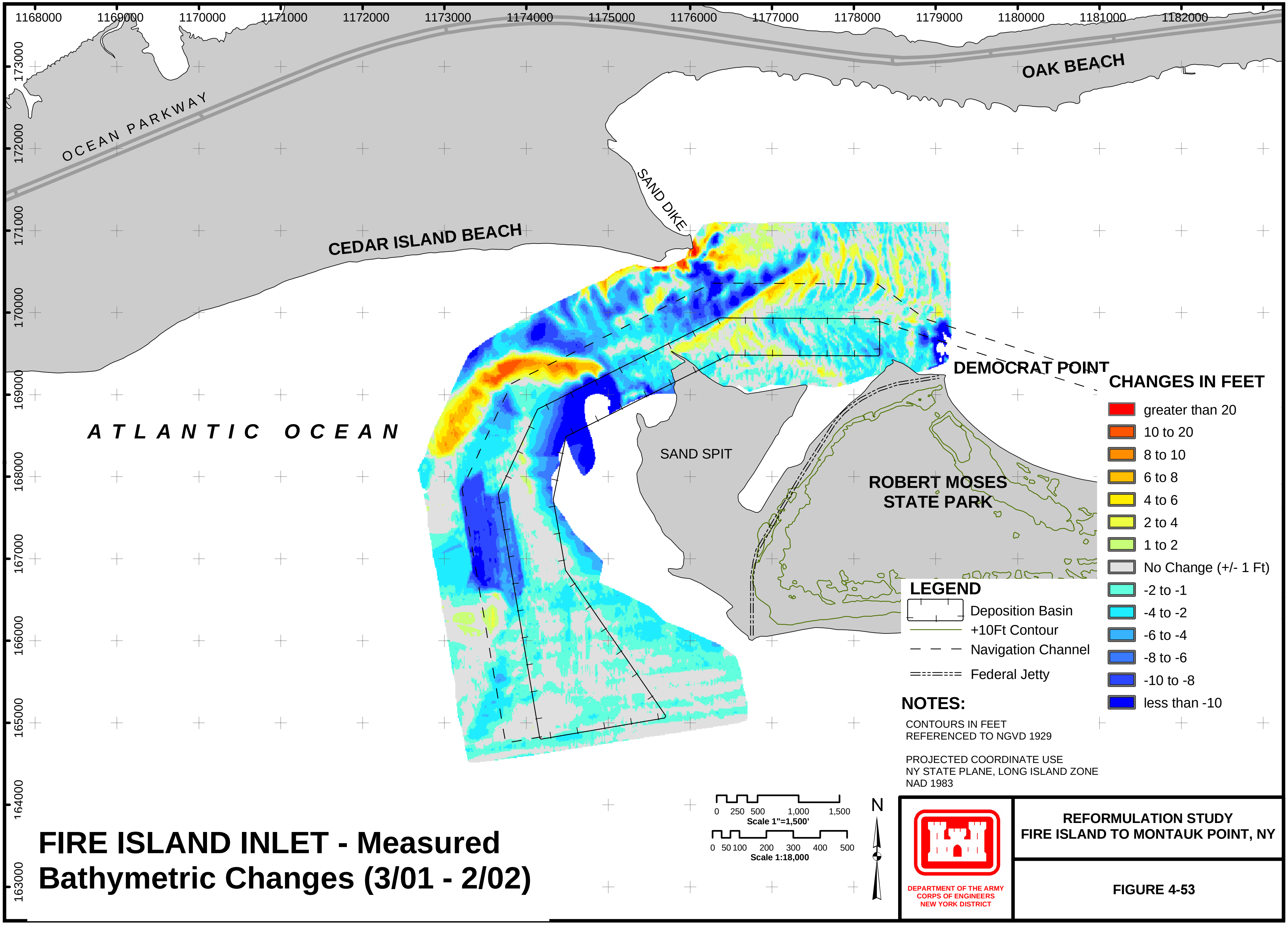
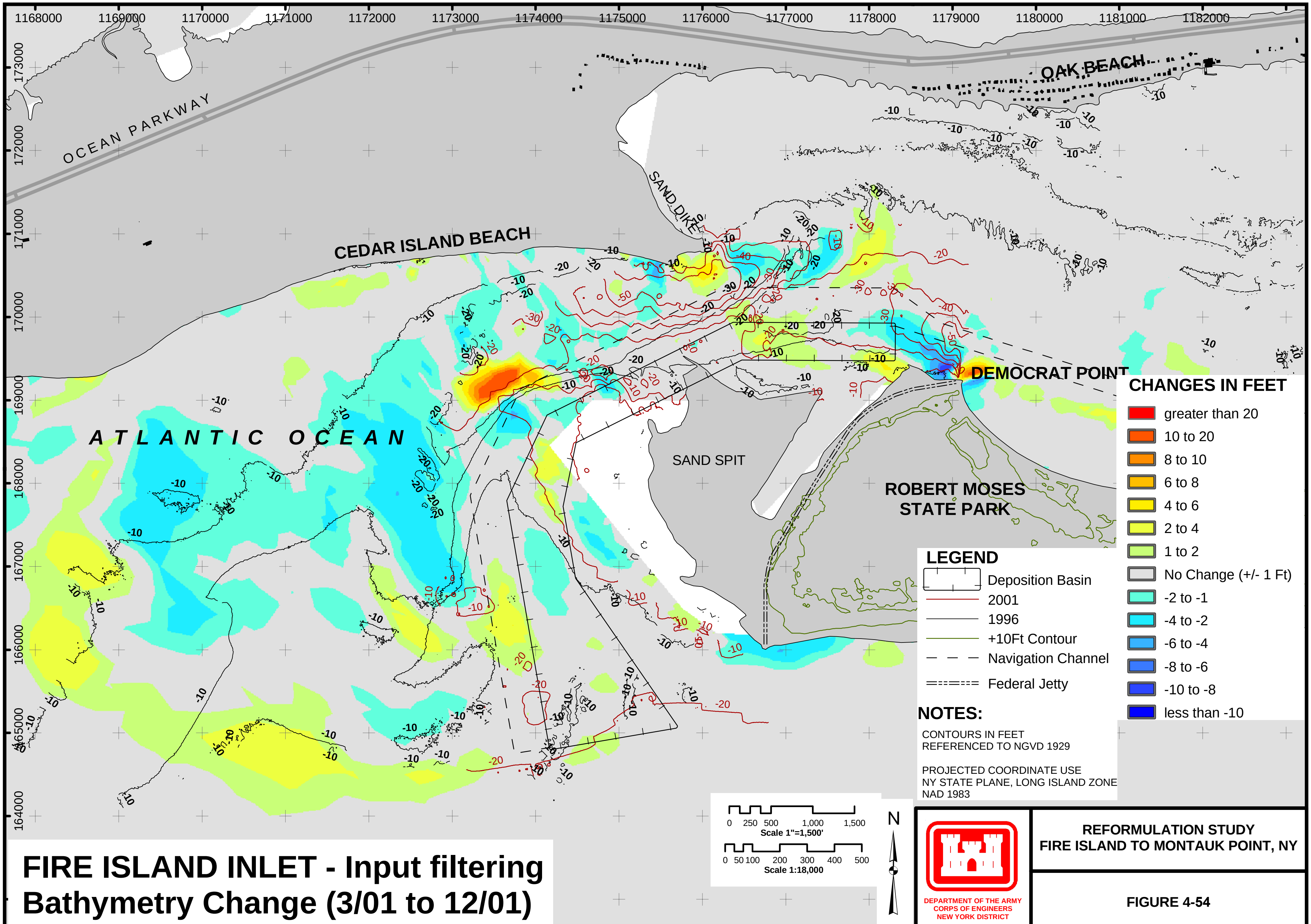
FIGURE 4-52

Filename: R:\GIS\3982 FIMP\3982-28 Inlet Modifications\Reports_MXDfiles\Dec2002\Wave_Grid_FireIsland.mxd



FIRE ISLAND INLET - Measured Bathymetric Changes (3/01 - 2/02)

FIGURE 4-53



FIRE ISLAND INLET - Input filtering Bathymetry Change (3/01 to 12/01)

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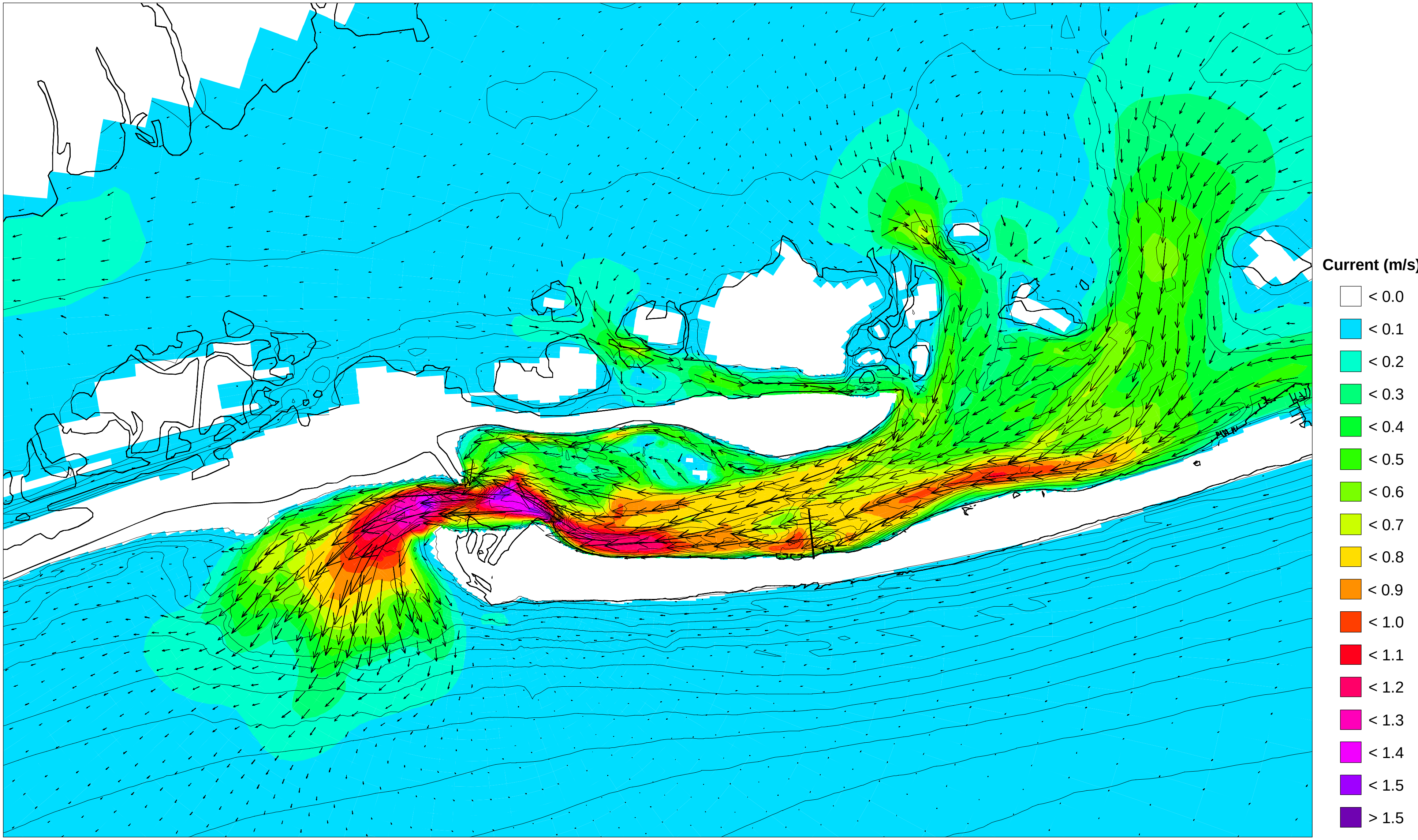
FIGURE 4-54

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/FireIsl_Diff.mxd



Fire Island Inlet Flow Velocity – Peak Ebb Tide

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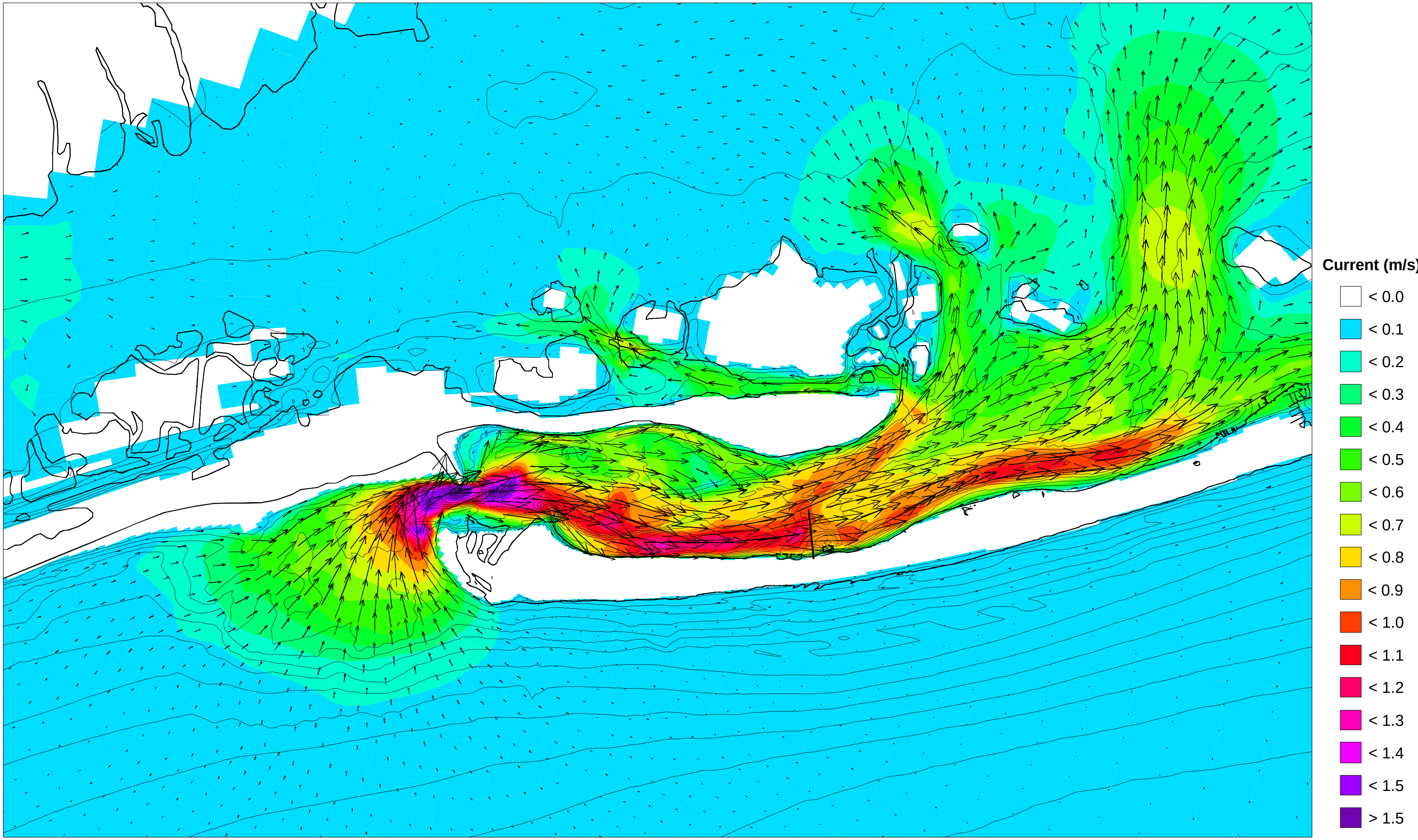
FIGURE 4-55

File pat/File name



Fire Island Inlet Flow Velocity – Peak Flood Tide

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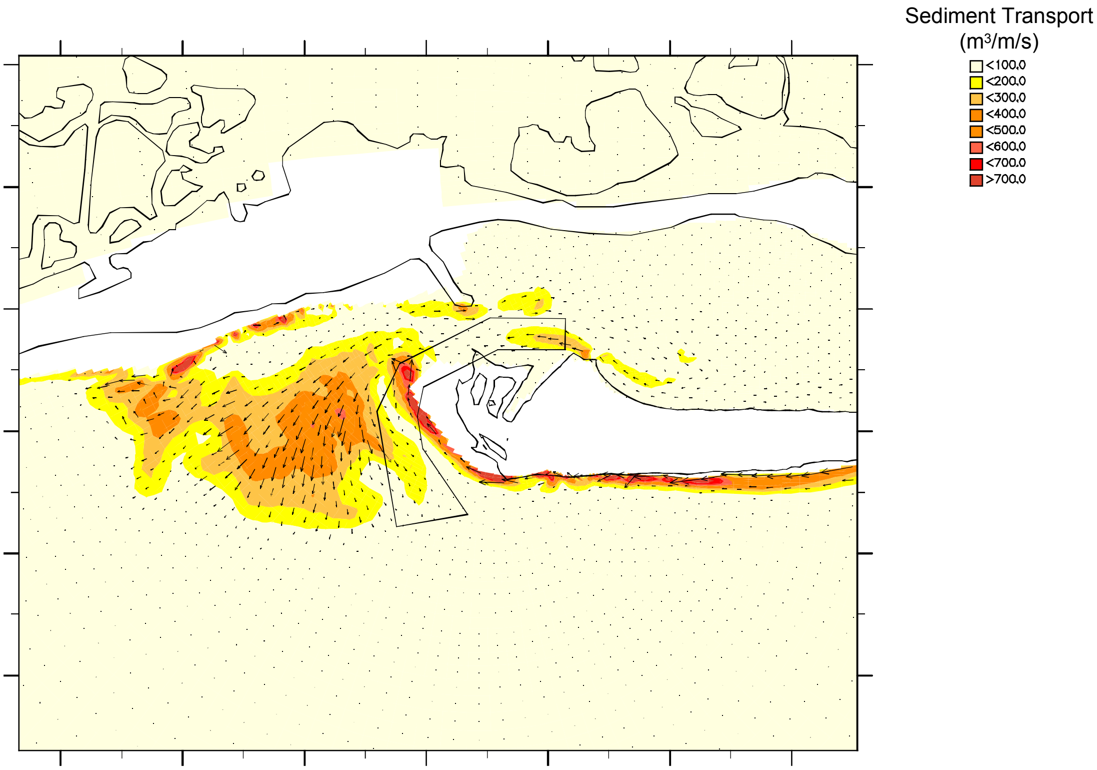
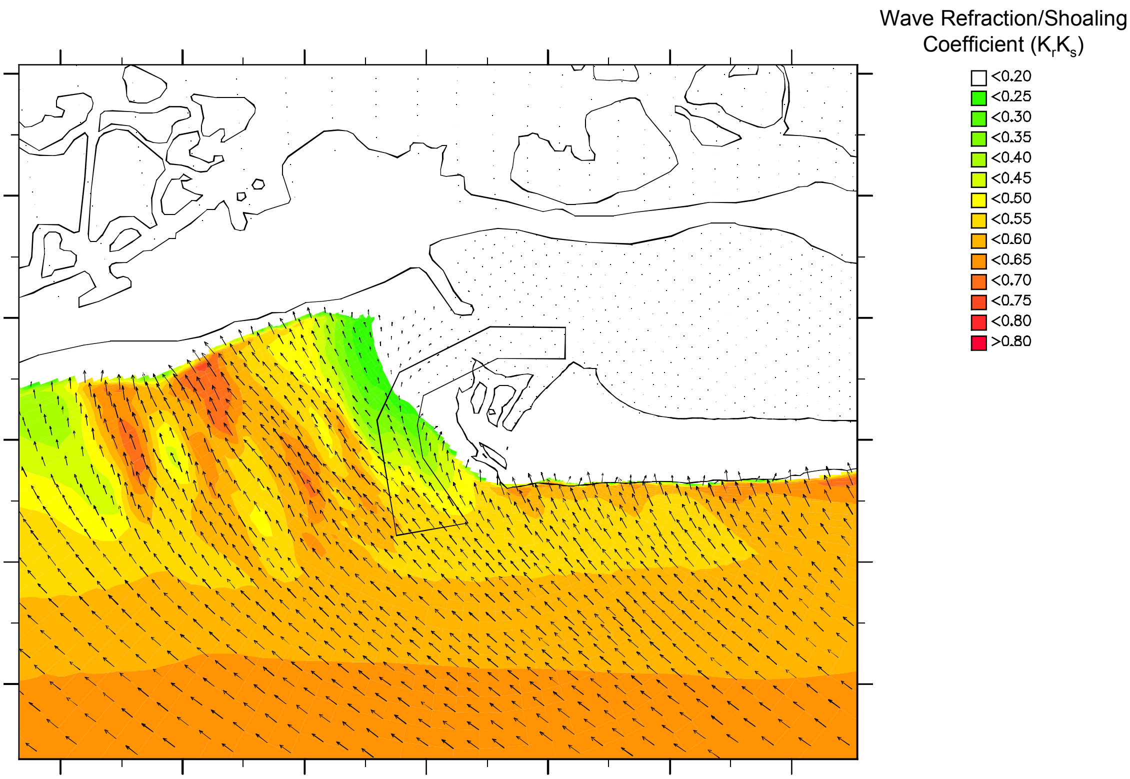
FIGURE 4-56

File pat/File name





Fire Island Inlet- Sediment Transport and Waves (110˚ from North)

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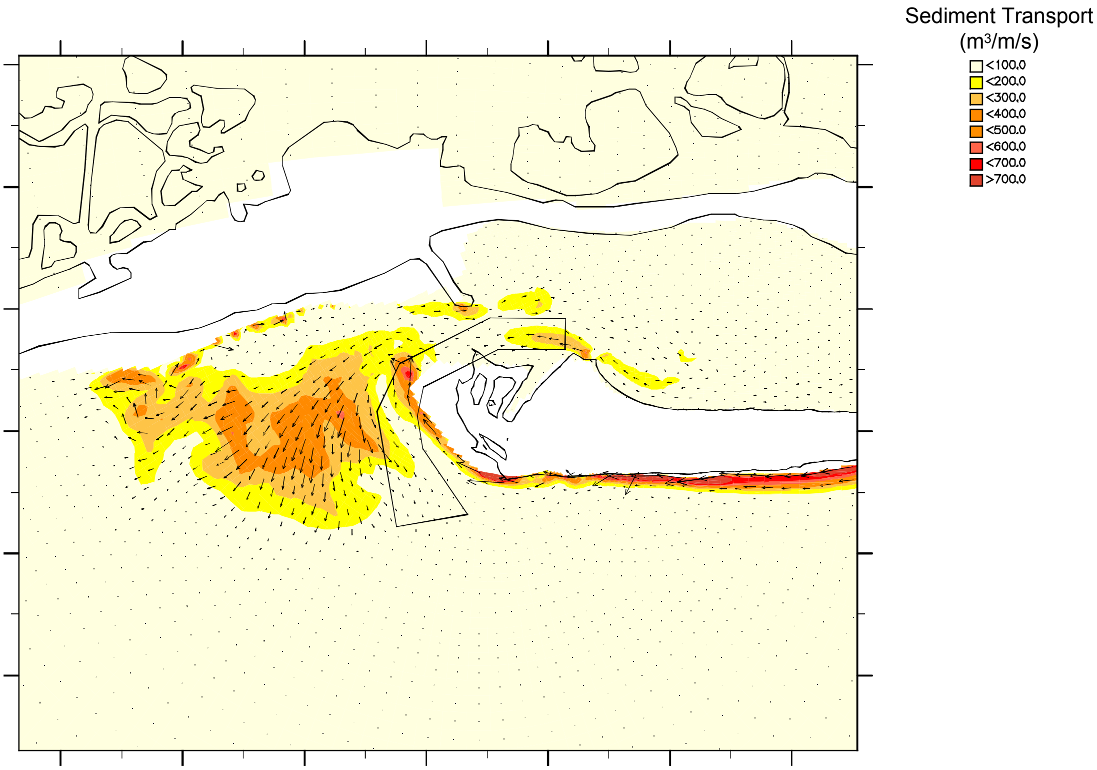
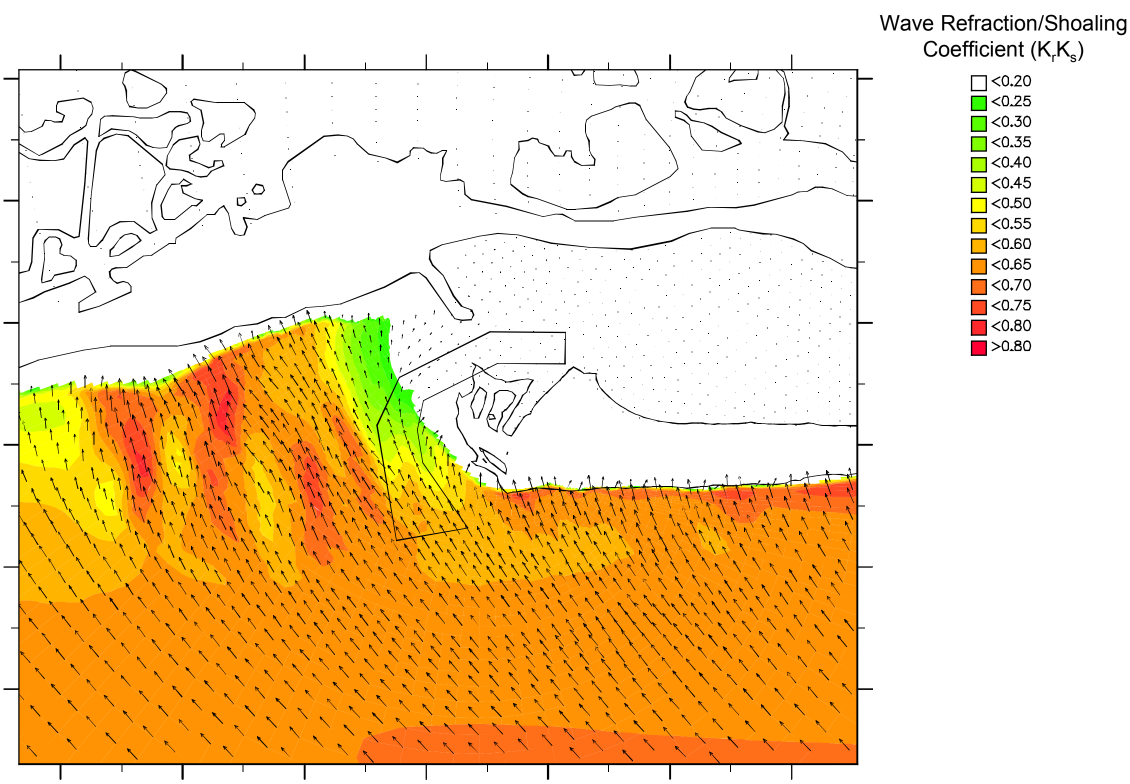
FIGURE 4-57

File pat/File name





Fire Island Inlet- Sediment Transport and Waves (130˚ from North)

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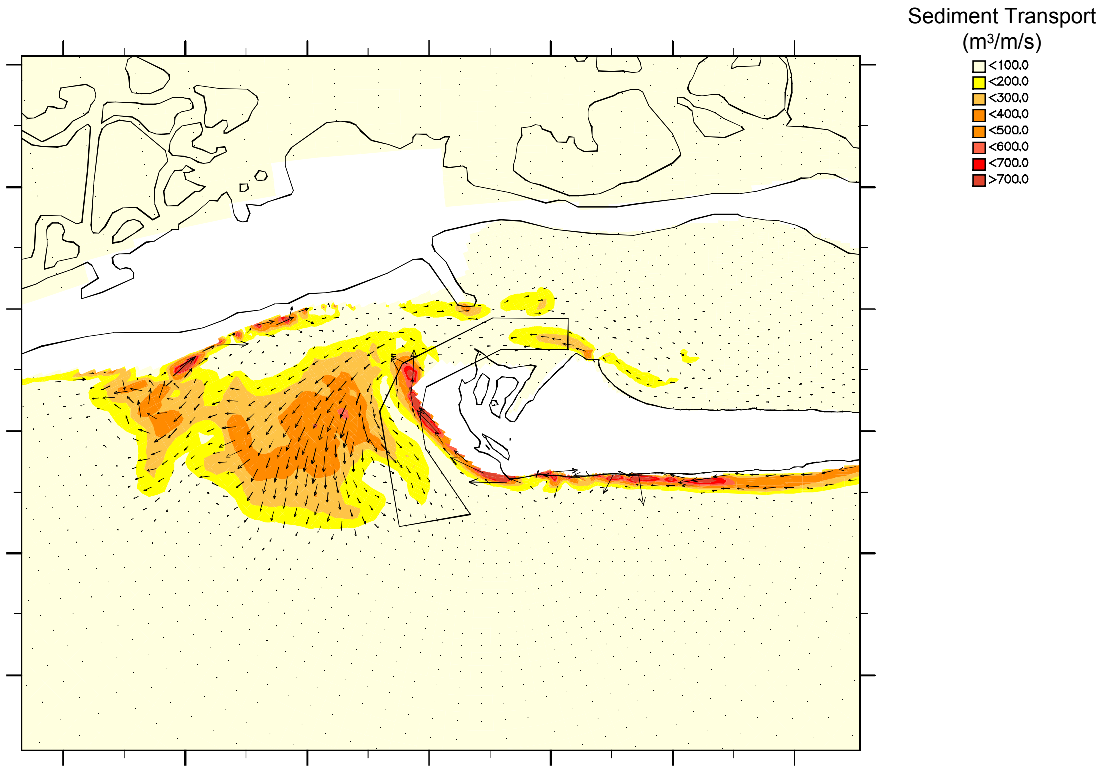
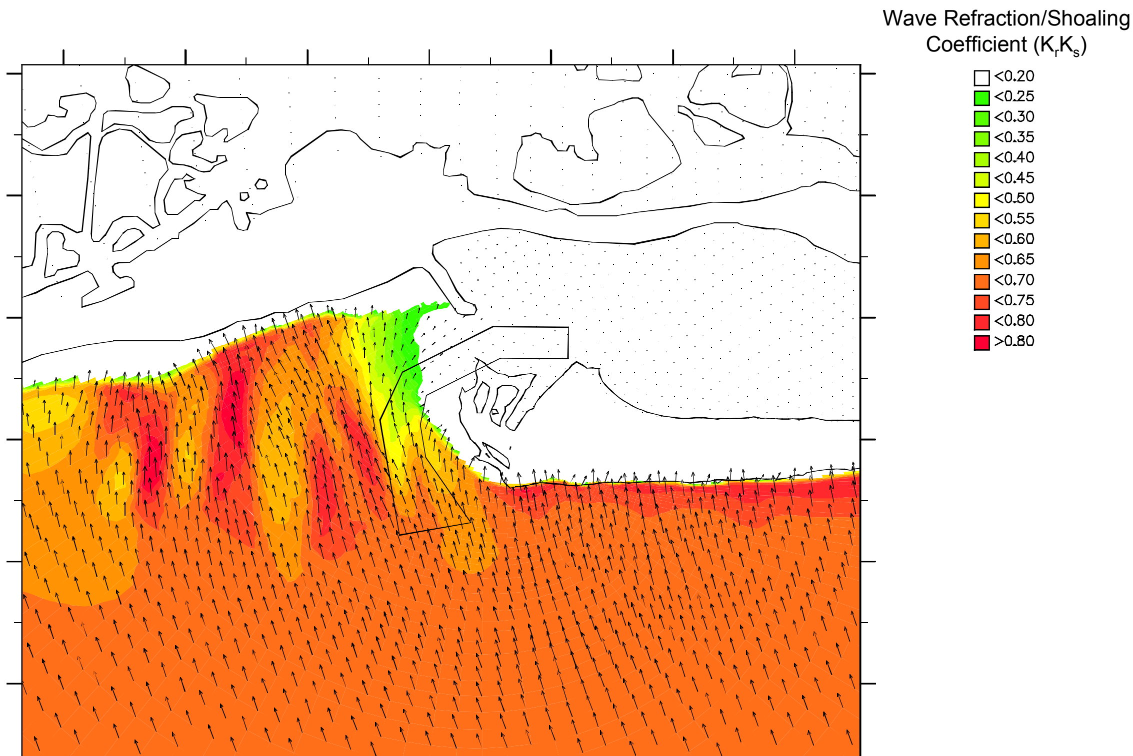
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 4-58

File pat/File name





Fire Island Inlet- Sediment Transport and Waves (160˚ from North)

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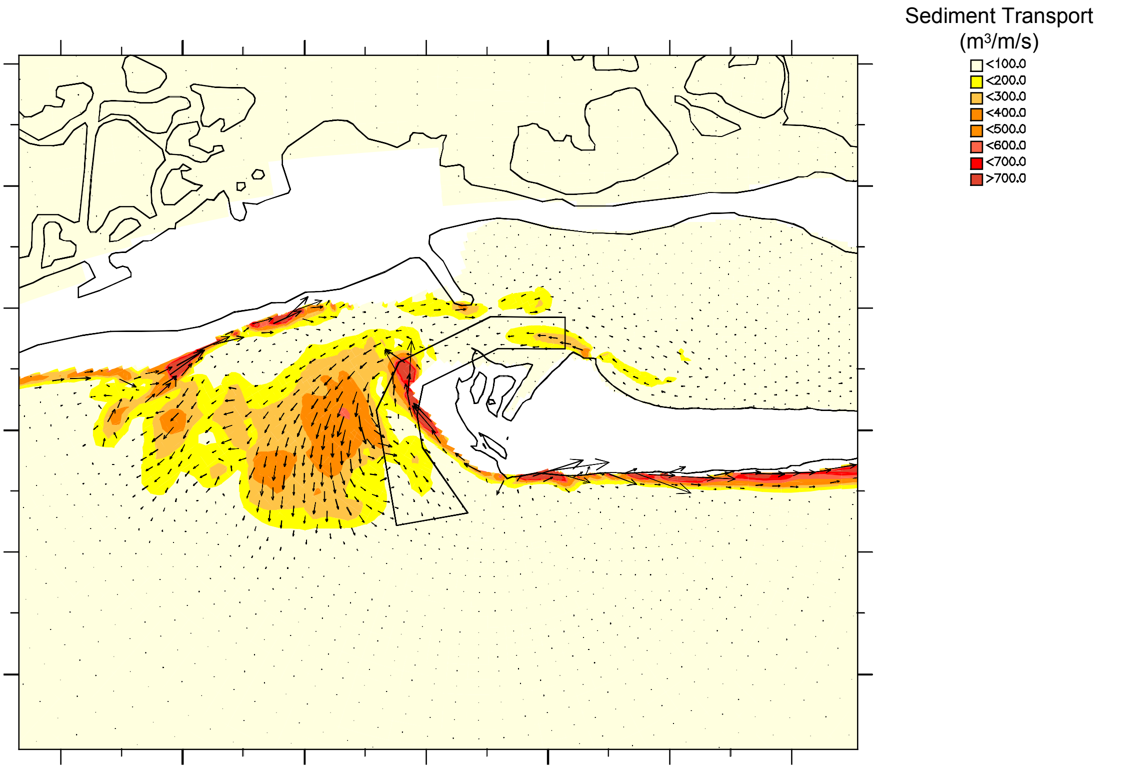
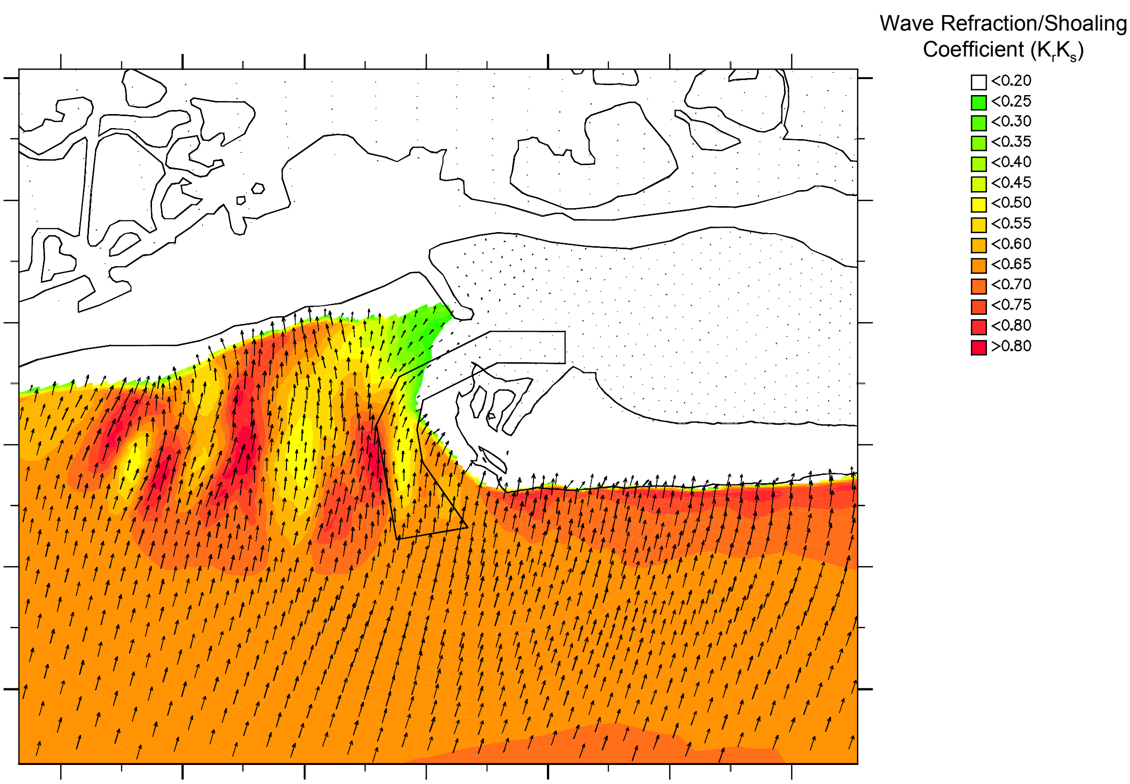
FIGURE 4-59

File pat/File name





Fire Island Inlet- Sediment Transport and Waves (210˚ from North)

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FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 4-60

File pat/File name

5. INLET DYNAMICS

An analysis of tidal inlet dynamics provides a framework for understanding the processes at work at the inlet and adjacent shorelines. This portion of the report addresses each inlet in terms of hydrodynamics, morphology, and sediment budget. Existing conditions as determined from available hydrographic data and modeling results are presented as well as estimates of future without project and future improved conditions based on extrapolation of existing conditions and modeling results for specific inlet modification scenarios.

5.1 Shinnecock Inlet

5.1.1 Existing Condition

Shinnecock Inlet has been the focus of numerous study programs in recent years, and as a result data availability and quality at the inlet are relatively good. This section describes recent bathymetric surveys and changes in inlet morphology from survey to survey. Hydraulic parameters are examined to determine theoretical cross-sectional area and ebb shoal capacity. Escoffier's (1940) method is applied to analyze the stability of Shinnecock Inlet.

Bathymetric Records

At Shinnecock Inlet, a comprehensive monitoring program has included frequent Scanning Hydrographic Operational Airborne Lidar Survey (SHOALS) surveys. Recent records include surveys dated July 1994, May 1996, August 1997, May 1998, July 2000, and July 2001 and are shown in Appendix A. Additional relevant survey data obtained using traditional hydrographic survey methods is available for 1933, June 1984, June 1989, August 1991, December 1992, August 1994, August 1998, September 1998, April 2000, and April 2001. Figures showing these additional surveys are included in Appendix A.

Hydraulic Analysis

Well-known relationships between tidal inlet prism and morphology are applied to Shinnecock Inlet. The analysis is designed to identify the theoretical equilibrium state of the inlet in terms of minimum cross-sectional area and volume of the ebb shoal. Current measured conditions at the inlet can then be compared with the theoretical equilibrium values to qualitatively assess probable future conditions.

Measured Conditions

Recent minimum cross-sectional areas, approximate average depths and estimated ebb shoal volumes are presented in Table 5-1. Minimum cross-sectional areas are measured using the surveys presented in Appendix A. The SHOALS 1994 survey is omitted from this analysis and further quantitative assessment due to questions about the vertical datum resolution (Morang, personal communication, June 2002). Average channel depth is estimated by dividing the cross-sectional area by the stabilized inlet channel width of 800 feet. Ebb shoal volume is estimated using cumulative measured changes in bathymetric survey data.

Table 5-1: Shinnecock Inlet Existing Hydraulic Characteristics			
Date	**Measured Minimum Cross-Sectional Area (sq. ft)**	**Average Depth (ft)**	**Estimated Ebb Shoal Volume[1] (cy x 10^6)**
1996	15,170	19.0	3.7
1997	15,750	19.7	7.7
1998	14,670	18.4	8.8
Dredging, June – September 1998, 440,000 cy removed from Deposition Basin			
2000	17,750	22.2	6.8
2001	17,510	21.9	5.8

[1]Ebb shoal volume estimated by comparisons of bathymetric data with bathymetry surveyed in 1933, before Shinnecock Inlet opened. Comparisons necessarily include all oceanside inlet effects on bathymetry; some surveys may cover more of the shoal than other surveys. The 1996 survey has the least coverage. See Appendix A for inlet survey coverage. 1998, 2000 and 2001 have similar coverage..

Tidal Prism - Theoretical Conditions

Theoretical equilibrium conditions for the existing tidal prism at Shinnecock Inlet are calculated as described below based on the most recent data available.

Existing tidal prism is estimated using the bay tidal amplitude as:

$$P = 2a_B A_B \qquad \text{Equation 5-1}$$

where

a_B = tidal amplitude in the bay, and

A_B =area of the bay.

Bay tidal amplitude at Shinnecock Inlet is based on an analysis of data collected as part of the Long Island Shore (LISHORE) monitoring program. Data has been collected at three pressure tide gauges in Shinnecock Bay (see Figure 4-4 for gauge locations). Table 5-2 lists tidal datums computed using these data sets; based on this analysis, tide range in the bay averages 2.9 feet. Shinnecock Bay surface area is approximately 14.5 square miles. These values and Equation 5-1 yield an estimated tidal prism of 1,170 x 10^6 ft^3.

Table 5-2: Tidal Datums for Shinnecock Inlet and Bay Tide Gauges[1]				
Tidal Datum (ft)	**P1[2] Ocean**	**P2[3] Town Dock**	**P3[4] Shinnecock Canal**	**P4[5] Quogue Canal**
MHHW	3.86	3.49	3.31	2.92
MHW	3.61	3.26	3.08	2.71
MSL	1.86	1.66	1.56	1.31
MTL	1.88	1.70	1.60	1.40
MLW	0.15	0.14	0.12	0.09
MLLW	0.00	0.00	0.00	0.00

[1] Monitoring at stations P3 and P4 ended in August 1999; monitoring at P1 and P2 ended on May 2001 and January 2002.
[2] Based on observations from April 1998 - May 2001.
[3] Based on observations from April 1998 - December 2001.
[4] Based on observations from April 1998 – August 1999.
[5] Based on observations from May 1998 – August 1999.

Theoretical equilibrium cross-sectional area is calculated for an Atlantic Coast Inlet as (Jarrett, 1976):

$$A_c = 7.75x10^{-6} P^{1.05} \qquad \text{Equation 5-2}$$

where

A_c = minimum cross-sectional area (square feet), and
P = tidal prism (cubic feet).

The estimated existing tidal prism of 1,170 x 10^6 ft^3 and Equation 5-2 result in a theoretical minimum cross-sectional area of 25,800 ft^2 at Shinnecock Inlet.

Ebb shoal capacity is estimated for a moderately exposed Atlantic Coast inlet (Walton and Adams, 1976) as:

$$V = 10.5x10^{-5} P^{1.23} \qquad \text{Equation 5-3}$$

where

V = volume of sand stored in the ebb shoal (cubic yards), and
P = tidal prism (cubic feet).

Equation 5-3 applied using the estimated existing tidal prism of 1,170 x 10^6 ft^3 gives a theoretical ebb shoal capacity of 15 x 10^6 cubic yards. Therefore, this analysis would suggest that the ebb shoal volume is only 40-60% of the estimated equilibrium volume. However, as explained in Section 3.3, there is a significant level of uncertainty associated with this estimate and the actual equilibrium volume could be much smaller or even larger. Therefore, one must be very careful not to rely too much in this type of analysis when formulating inlet management alternatives. Additional analysis of historic Shinnecock Inlet ebb shoal growth is presented in the following paragraphs.

Is the Shinnecock Inlet Ebb Shoal Accreting?

Previous work (USACE-NAN, 1998; Militello and Kraus 2001) has suggested that the ebb shoal at Shinnecock Inlet is still growing and has yet to reach a dynamic equilibrium. Volumetric change comparisons completed as part of the current study, however, indicate that the ebb shoal may not be accumulating a significant amount of sediment and might actually be closer to equilibrium than previously thought.

Morang (2001) conducted a detailed analysis of the changes in the ebb shoal at Shinnecock Inlet. His study compared regional bathymetric data collected by the United States Coast and Geodetic Survey (USCGS) in 1933, before the inlet opened, to more recent data sets from 1949, 1984, 1994, 1996, 1997, and 1998. Morang delineated a series of reference squares, each measuring 1000 feet by 1000 feet square and aligned with the NAD 1983 State Plane Long Island grid. Each survey was compared with the baseline USCGS 1933 survey. Cut and fill volumes were determined in each square using terrain modeling software and summed. The results are presented in Table 5-3.

Table 5-3: Change in Shinnecock Ebb Shoal Volume (after Morang 2001)

Survey Date	Cut (yd^3)	Fill (yd^3)	Total (yd^3)
Jul-Aug 1949	17,500	1,043,000	1,025,000
June 1984	747,000	5,245,000	4,498,000
May 1996	856,000	8,446,000	7,590,000
Aug 1997	712,000	8,544,000	7,832,000
May 1998	933,000	9,385,000	8,453,000

Morang's table seems to indicate that the ebb shoal is continuing to grow and that it gained substantial material from 1984 to 1998. However, in his report, Morang notes that *"the fact that measured volume was greater in 1998 than 1997 or 1996 may be due to slightly greater survey coverage."* After an uncertainty assessment Morang states that *"the volumes computed for 1996, 1997, and 1998 cannot be considered statistically different, and no inferences should be made regarding ebb shoal growth from these three data points."*

In order to examine the changes in the ebb shoal from 1984 to 1998, the reference squares from Morang (2001) were reproduced in a GIS software package. The 1984 and 1998 surveys were directly compared within the bounds of the boxes to determine the changes from survey to survey. The 1933 to 1984 and 1933 to 1998 survey comparisons were reproduced as well. Volumetric comparisons are presented in Table 5-4. Figure 5-1, Figure 5-2 and Figure 5-3 show the patterns of erosion and accretion.

Table 5-4: Change in Shinnecock Ebb Shoal Volume from 1984 to 1998

Survey Dates	Overlapping Coverage Area (Acres)	Cut (cy)	Fill (cy)	Total (cy)
1933 to 1984	480	1,162,000	6,308,000	5,147,000
1933 to 1998	780	1,232,000	10,031,000	8,798,000
1984 to 1998	480	1,290,000	1,710,000	420,000

If the 1933 to 1984 and 1933 to 1998 ebb shoal volume estimates are compared, it would appear that the ebb shoal grew 3.7 million cy from 1984 to 1998, or roughly 260,000 cy/yr. However, as Morang (2001) commented in his report, the relative extents of each survey can skew the results significantly. As shown in Table 5-4 the 1993-1998 overlapping coverage area is approximately 63% larger than the 1993-1984 overlapping coverage area. A direct comparison of the 1984 and 1998 surveys (i.e., based on overlapping survey coverage) yields a significantly smaller increase of 420,000 cubic yards from 1984 to 1998, or 30,000 cy/yr.

These results suggest that the ebb shoal may now be closer to equilibrium than previously thought. In fact, the ebb shoal volume estimates presented in Table 5-1, computed using the same methodology as Morang (2001), do not suggest substantial accumulation. These results lead us to hypothesize that the ebb shoal at Shinnecock Inlet may not be accumulating as much sediment as previously thought. Detailed survey comparisons and volumetric changes are summarized in the following sections.

Inlet Stability Analysis

As a complement to the discussion above, Escoffier's method for assessing inlet stability (Escoffier 1940) is applied to existing and historic conditions at Shinnecock Inlet. The Escoffier method requires information about inlet geometry (cross-sectional area, length and width), ocean tidal amplitude, bay surface area, and friction coefficient. Inlet cross-sectional areas are shown in Table 5-1. Shinnecock Bay surface area of 14.5 square miles and ocean tidal amplitude of 1.75 feet as measured at LISHORE station P1 (see Table 5-2) are employed for Shinnecock Inlet. A friction coefficient of 0.0035 is selected, and the physical inlet length is approximated as 2,500 feet.

Tidal amplitude is assumed constant over the 1996 to 2001 time period. Results of the inlet stability evaluation for existing and recent cross-section at Shinnecock Inlet are presented in Figure 5-4. Measured cross-section data are superimposed on the curve. The equilibrium flow velocity shown is approximately 3.3 ft/sec, a value suggested by Bruun (1978), Van de Kreeke (1984) and others as appropriate for most inlets.

The closure curve for Shinnecock Inlet summarizes the stability of the inlet in recent years. Shinnecock Inlet cross-sectional area remained fairly stable from 1996 to 1998 before dredging (dredging occurred in September 1998 after the May 1998 SHOALS survey). The most recent surveys (2000 and 2001) show that the inlet appears to be maintaining an increased cross-sectional area. The position of these cross-sections on the stability curve indicates that Shinnecock Inlet is stable with relatively high velocities for the given ocean tidal range and inlet and bay geometry. Theoretically inlet cross-sectional area has the potential to continue to increase; stable cross-section values are on the order of 30,000 ft^2. [This value is slightly larger than the estimate of equilibrium cross section using Equation 5-2, 25,800 ft^2.] If inlet cross-sectional area increases, velocities would be expected to decrease. Since the width of the inlet is fixed at about 800 feet by the position of the jetties, channel depth may increase to an average of 38 feet. As visible in the figures included in Appendix A, parts of the channel near the northeast corner of the east jetty and east of the west jetty already exceed this depth. With the current alignment of the channel, increased inlet scouring could deepen these areas.

Recent Volume Changes (1995 to 2001)

To track morphological changes in areas surrounding Shinnecock Inlet and to develop an inlet sediment budget, the inlet area was divided into nine cells. These cells, shown in Figure 5-5, were delineated following distinct morphological features of the inlet complex. The Updrift Beach (UBCH) extends from 600 meters east of the inlet to 3,200 meters east of the inlet and corresponds to the Shinnecock East (SE) cell in the previous sediment budget work done at CHL (Gravens et al, 1999). The Updrift Lobe (UL) extends from the east jetty to 600 meters east of the inlet and offshore to approximately the -12 m (–40 feet) contour. The Deposition Basin (DB) is as defined by the USACE for maintenance of the navigation channel. The Channel Throat (CT) extends through the mouth of the inlet and is bound by the jetties on either side. The Near Field Flood Shoal (NFFS) is a potential source of sand for mining operations, and was therefore tracked as a feature separate from the Flood Shoal (FS). The West Beach (WB) extends from the west jetty about 900 meters to the west and corresponds to CHL stationing W23.9 km to W24.8 km. The Downdrift Lobe (DL) extends from 900 m west of the inlet an additional 1,500 meters west and corresponds to CHL stationing W22.4 km to W23.9 km. The DL follows the bypassing bar from seaward of the deposition basin to the Ponquogue attachment point. The Downdrift Beach (DBCH) corresponds to CHL stationing W21.6 km to W22.4 km, extending about 800 meters west of the DL.

Recent bathymetry changes at Shinnecock Inlet are relatively well documented through a number of recent surveys. Appendix A shows plots of the surveys used in this study and Appendix B details the data sources for all data used in the inlet sediment budget formulation. To calculate volume changes in the sediment budget cells over the largest possible spatial extent, a series of "synthetic" topography and bathymetry grids were created for Shinnecock Inlet. Each grid had 10-foot (3.3 meter) spacing and was referenced to the New York State Plane Long Island NAD 83 coordinate system and NGVD 1929 vertical datum. The data sources used for each of the grids is detailed in Table 5-5. Plots of each of the synthetic topography/bathymetry grids are shown in Figure 5-6 to Figure 5-11 [6 figures, 1996, 1997, 1998 pre-dredge, 1998 post-dredge, 2000, 2001]. Again, the 1994 survey was omitted from this analysis due to questions about the vertical datum resolution (A. Morang, pers. comm. June 2002).

The synthetic grids were used to compute recent volume changes in sediment budget cells UL, DB, CT, NFFS, FS, WB and DL. The UBCH and DBCH cells had poor coverage for most surveys; therefore shorelines and beach profiles were used instead to compute volume changes in those cells. Table 5-6 presents computed volumetric changes from the synthetic grid comparisons. For consistency and to make comparisons with the previous sediment budget work by Gravens et al (1999) easier, the changes as well as the inlet sediment budgets are presented in metric units. Figure 5-12 to Figure 5-15 [1996-7, 1997-8, 1998-2000, 2000-01] show erosion and accretion patterns from the synthetic grid comparisons.

Figures show that from year to year, changes can vary greatly. Some of this variability is due to the fact that the inlet is a very dynamic system; however, resolution of different datums in the available data sets may also be a cause. For example, the May 1996 to July 2001 comparison yields erosion in all of the cells examined. This is difficult to believe, and it be may a result of datum differences. The SHOALS surveys in 1996 and 2000 were datum-adjusted using different methodologies. The same applies for the September 1998 to July 2000 comparison. Apparently the 1996, 1997 and 1998 SHOALS surveys were all corrected using the same methodology. The methodology, however, was changed for the 2000 and 2001 SHOALS surveys. Generally, comparisons of surveys corrected in the same manner were given more credence in developing the inlet sediment budget.

Volume changes in cells UBCH and DBCH were examined using both shoreline data and survey profiles. Shorelines digitized from aerial photographs in March 1995 and April 2001 were obtained from earlier work at CHL and new digitizing efforts at CENAN, respectively. These two datasets were used with the baselines previously established by CHL (Gravens et al., 1999) to compute shoreline change rates in each cell. Beach profile data collected in Fall 1995, Spring 1998, and Spring 2001 were compared as well. Results of the updrift and downdrift beach comparisons are presented in Table 5-7.

Table 5-5: Synthetic Grids and Source Data for Shinnecock Inlet Sediment Budget

Synthetic Grid		Data Source 1		Data Source 2		Data Source 3		Data Source 4	
Shinnecock Inlet Spring 1996		SHOALS Survey 23 May-2 June 1996		ACNYMP Profiles March-April 1996 W36-44, P1-5					
Filename	shin96synthgd	Datum, Conversion to NGVD	NGVD	Datum, Conversion to NGVD	NGVD				
Shinnecock Inlet Spring-Summer 1997		SHOALS Survey 13 August 1997		ACNYMP Profiles March-April 1997 W36-44, P1-5					
Filename	shin97synthgd	Datum, Conversion to NGVD	NGVD	Datum, Conversion to NGVD	NGVD				
Shinnecock Inlet Spring 1998 (Pre-Dredge)		SHOALS Survey 28 May 1998		Condition Survey, 4-6 March 1998		ACNYMP Profiles February-March 1998 W36-50, P1-5, SH1-2			
Filename	shin98synthgd	Datum, Conversion to NGVD	NGVD	Datum, Conversion to NGVD	MLW 1.5 ft below NGVD	Datum, Conversion to NGVD	NGVD		
Shinnecock Inlet Fall 1998 (Post-Dredge)		Post-Dredge Soundings, 25, 29 Sept 1998		SHOALS Survey 28 May 1998		Condition Survey, 4-6 March 1998		ACNYMP Profiles February-March 1998 W36-50, P1-5, SH1-2	
Filename	shin98postgrd	Datum, Conversion to NGVD	MLW 1.5 ft below NGVD	Datum, Conversion to NGVD	NGVD	Datum, Conversion to NGVD	MLW 1.5 ft below NGVD	Datum, Conversion to NGVD	NGVD
Shinnecock Inlet Spring-Summer 2000		SHOALS Survey, July 2000		Condition Survey, 13 April 2000					
Filename	Shin00synthgd	Datum, Conversion to NGVD	NGVD	Datum, Conversion to NGVD	MLW 1.5 ft below NGVD				
Shinnecock Inlet Spring-Summer 2001		SHOALS Survey, July 2001		Condition Survey, 5 April 2001		ACNYMP Profiles April 2001 W36-44,37, P1-5			
Filename	Shin01Synthgd	Datum, Conversion to NGVD	MSL 0.5 ft above NGVD	Datum, Conversion to NGVD	MLW 1.5 ft below NGVD	Datum, Conversion to NGVD	NGVD		

Table 5-6: Volumetric Changes from Synthetic Grid Comparisons: Shinnecock Inlet						
Inlet Segment	**From**	**To**	**Cut (m^3)**	**Fill (m^3)**	**Net (m^3)**	**Net/yr (m^3/yr)**
Updrift Lobe	May-96	May-98	-185,000	220,000	35,000	17,000
Deposition Basin	May-96	May-98	-22,000	212,000	190,000	95,000
Channel Throat	May-96	May-98	-110,000	30,000	-80,000	-40,000
West Beach	May-96	May-98	-239,000	59,000	-180,000	-90,000
Downdrift Lobe	May-96	May-98	-586,000	600,000	14,000	7,000
Near Field Flood	May-96	May-98	-126,000	3,000	-123,000	-62,000
Flood Shoal	May-96	May-98	-236,000	7,000	-229,000	-114,000
Updrift Lobe	August-97	May-98	-151,000	273,000	122,000	155,000
Deposition Basin	August-97	May-98	-9,000	234,000	225,000	285,000
Channel Throat	August-97	May-98	-27,000	41,000	14,000	18,000
West Beach	August-97	May-98	-321,000	68,000	-252,000	-320,000
Downdrift Lobe	August-97	May-98	-369,000	516,000	147,000	187,000
Near Field Flood	August-97	May-98	-119,000	12,000	-106,000	-135,000
Flood Shoal	August-97	May-98	-596,000	104,000	-492,000	-624,000
Updrift Lobe	September-98	July-00	-290,000	130,000	-160,000	-91,000
Deposition Basin	September-98	July-00	-72,000	75,000	3,000	2,000
Channel Throat	September-98	July-00	-65,000	10,000	-55,000	-31,000
West Beach	September-98	July-00	-89,000	41,000	-48,000	-27,000
Downdrift Lobe	September-98	July-00	-718,000	162,000	-556,000	-316,000
Near Field Flood	September-98	July-00	-42,000	43,000	1,000	0
Flood Shoal	September-98	July-00	-108,000	288,000	180,000	102,000
Updrift Lobe	July-00	July-01	-150,000	152,000	2,000	2,000
Deposition Basin	July-00	July-01	-31,000	52,000	21,000	21,000
Channel Throat	July-00	July-01	-31,000	55,000	24,000	24,000
West Beach	July-00	July-01	-79,000	33,000	-45,000	-45,000
Downdrift Lobe	July-00	July-01	-377,000	240,000	-137,000	-137,000
Near Field Flood	July-00	July-01	-169,000	14,000	-155,000	-155,000
Flood Shoal	July-00	July-01	-456,000	178,000	-278,000	-278,000
Updrift Lobe	May-96	July-01	-391,000	171,000	-221,000	-43,000
Deposition Basin	May-96	July-01	-164,000	9,000	-155,000	-30,000
Channel Throat	May-96	July-01	-157,000	17,000	-139,000	-27,000
West Beach	May-96	July-01	-306,000	187,000	-119,000	-23,000
Downdrift Lobe	May-96	July-01	-1,126,000	462,000	-664,000	-130,000
Near Field Flood	May-96	July-01	-212,000	1,000	-211,000	-41,000
Flood Shoal	May-96	July-01	-290,000	5,000	-285,000	-56,000

Table 5-7: Volumetric Changes from Shoreline and Profile Comparisons: Shinnecock Inlet					
Inlet Segment	**From**	**To**	**Data Used**	**Volume Change Rate (m^3/m/yr)**	**Net/yr (m^3/yr)**
Updrift Beach	Fall 1995	Spring 2001	P3, P4, P5	42	108,000
Updrift Beach	Spring 1995	Spring 2001	Shorelines, Stations 0.6 to 3.2	11	27,000
Downdrift Beach	Fall 1995	Spring 2001	W36, W37	2	2,000
Downdrift Beach	Spring 1995	Spring 2001	Shorelines, Stations 21.6 to 22.4	-15	-12,000

Recent Engineering Events

A comprehensive list of engineering activities throughout the FIMP area from 1995 to 2002 is presented in Appendix C. Table 5-8 lists the dredging and fill events taking place from stations 21.6 km west of Shinnecock to 3.2 km east of Shinnecock, corresponding to the inlet sediment budget cells described above. Dredging generally takes place in the DB cell, and fill is placed in the WB and DL cells.

Table 5-8: Recent (1995 to 2001) Engineering Events in the Vicinity of Shinnecock Inlet					
Date	**Locality**	**Stationing (km)**		**Volume Placed or Removed (m^3)**	**Comments**
		West Boundary	**East Boundary**		
Feb-Mar 1997	East Flood Shoal Channel	NA	NA	191,150	Material dredged from East Flood Shoal Channel
Feb-Mar 1997	West of Shinnecock Inlet	24.375	24.8	191,150	100% placed in WB (Sta. 23.9-24.8), material from East Flood Shoal Channel dredging.
27 Jun – 11 Jul 1998	Shinnecock Inlet	NA	NA	-26,000	Phase 1 dredging, 26,000 m^3 removed from entrance channel and deposition basin above –4.3 m contour, placed in surf zone of wet beach from 150 m west of west jetty to 550 m west of west jetty
27 Jun – 11 Jul 1998	West of Shinnecock Inlet	24.1	24.65	26,000	100% placed in WB (Sta. 23.9-24.8) material from Shinnecock Inlet dredging
13-25 Sep 1998	Shinnecock Inlet	NA	NA	-310,400	Phase 2 dredging, 310,400 m^3 removed from entrance channel and deposition basin above –6.7 contour, placed on west beach between west jetty and 1070 m west
13-25 Sep 1998	West of Shinnecock Inlet	23.73	24.8	310,400	84% placed in WB (Sta. 23.9-24.8); 16% placed in DL (Sta. 22.4-23.9), material from Shinnecock Inlet dredging

Wave Climate and Longshore Sediment Transport (LST) in Recent Years

Measured morphological changes at Shinnecock and Moriches Inlets show significant variability from period to period. This variability is particularly apparent at Shinnecock Inlet, possibly due to a relatively complete survey record in the last decade. For example, Figure 5-13 and Figure 5-14 show bathymetry changes at Shinnecock Inlet from 1997 to 1998 and 2000 to 2001, respectively. Clearly, the changes over the deposition basin and ebb shoal in general are much more pronounced in the 1997-98 period. This is particularly important when one considers that the deposition basin was dredged in the fall of 1998. Inlet morphology in 2000 (less than 2 years after dredging) appears to be considerably more out of equilibrium than in 1997 (4 years after dredging 1993). Therefore, one would expect morphological changes to be more pronounced in 2000-01 than 1997-98. That is, of course, if the wave climate was similar during both periods. After a review of available wave records for the two periods, it was in fact concluded that the wave climate and associated potential Longshore Sediment Transport (LST) was much more active in 1997-98.

The correlation between inlet morphological changes and wave induced LST was further investigated to allow for better interpretation of those changes in the context of the inlet sediment budgets.

LST Analysis

Although the inlet sediment transport models developed in Section 4 allow for computation of sediment transport rates along the shoreline, a more computationally efficient methodology was applied in this analysis. First, recorded wave conditions at the offshore NOAA gauge were transformed nearshore using the regional wave model described in Section 4. Then computed nearshore wave conditions were used as input to LITPACK, a deterministic sediment transport model developed by the Danish Hydraulic Institute (DHI). LITPACK allows for the simulation of a large number of wave/current scenarios and for the combination of these simulations into predictions of LST. Two major components of LITPACK were used in this study:

1. A hydrodynamic model that includes propagation, shoaling, and breaking of waves, calculation of driving forces due to radiation stress gradients, wave set-up and the longshore current velocities.

2. A sediment transport model that includes the combined effects of waves and currents and computes total sediment load as the sum of bed and suspended load.

In order to simplify the problem and save computational time, the model assumes that the conditions are uniform along a uniform coast.

Input to the model includes:

- Beach profile
- Wave climate
- Sediment characteristics along the profile
- Tides
- Additional currents
- Winds

A profile representative of the bathymetry east of Shinnecock Inlet was used as input to the model, with representative sediment grain sizes as presented by Gravens et al (1999). Hourly wave climate from the NOAA gauge and predicted tide conditions were also used to compute hourly transport (in m^3/sec), net and gross accumulated transport (in m^3) for each period between available surveys at Shinnecock Inlet. Figure 5-16 show measured wave height and direction, as well as the hourly transport and accumulated net transport for the 1997-98 period. Table 5-9 presents computed net, gross, easterly and westerly LST magnitudes and rates for each period. The weighted average net transport from 1994 to 2001 was computed at 233,000 m^3/yr (westerly-directed).

Table 5-9: Computed LST (m^3/yr) East of Shinnecock Inlet										
	From	To	From	To	From	To	From	To	From	To
	7/19/94	6/2/96	6/2/96	8/13/97	8/13/97	5/28/98	5/28/98	7/1/00	7/1/00	7/1/01
# of days	684		437		288		765*		365	
NET (m^3)	468,263		225,688		445,866		288,778		192,303	
GROSS (m^3)	1,105,906		646,630		644,678		691,029		433,906	
NET (m^3/yr)	249,877		188,504		565,073		137,783		192,303	
GROSS (m^3/yr)	590,140		540,091		817,040		329,707		433,906	
G/N	2.36		2.87		1.45		2.39		2.26	
Westerly (m^3/yr)	420,009		364,298		691,057		233,745		313,104	
Easterly (m^3/yr)	-170,131		-175,794		-125,984		-95,962		-120,801	
W/E	2.47		2.07		5.49		2.44		2.59	
Positive and negative numbers represent westerly and easterly transport, respectively * Wave record has significant data gaps										

An estimate of LST was also computed for the 1997-98 period using the energy flux approach as described in the Coastal Engineering Manual (CEM) (USACE, 2002) resulting in approximately 6 times the transport computed with LITPACK. Note that this estimate is based on a recommended default value of K=0.77. Therefore, a value of K=0.77/6=0.13 would result in LST estimates similar to those computed with LITPACK. This lower K value is similar to that calibrated by Gravens et el. (1999) in their GENESIS application to existing conditions along the FIMP shoreline. Specifically, they calibrated the *K1* parameter in GENESIS to an average value of 0.2, similar to that computed in this effort and significantly lower than the default value recommended in the CEM.

Results presented in Table 5-9 confirm that the wave climate and associated sediment transport was significantly more active during the 97-98 period. It is also interesting to note that the ratio of westerly to easterly transport is significantly larger for this period too. In other words, westerly transport accounts for most of the gross difference in transport between this and other periods. Apparently this resulted in significant changes at ebb shoal and accumulation within deposition basin. Note, however, that the net accumulation in the inlet (UL, DB, WB and DL) over this period was 242,000 m^3, whereas the computed net westerly transport was 445,866 m^3. In other words, approximately half of the net LST appears to have bypassed the inlet. Even if one considers that the models are not perfect and the actual net westerly transport might have been lower, it would appear that the inlet has allowed for more bypassing in recent years existing conditions than previously assumed.

During a relatively calm period such as 2000-01, westerly transport is approximately 2.6 times larger than easterly. More importantly, the inlet (UL, DB, WB and DL) appears to erode slightly (-159,000 m^3). Finally, the 1996-97 period represents a somewhat average condition in which the ebb shoal accumulates a small amount of sand.

Influence of Relative Sea Level Rise

Only limited research has been conducted regarding the influence of relative sea level rise on the morphological evolution of coastal inlets. One of the most extensive studies was recently conducted by van Goor (2001) who focused his research on whether or not inlets and tidal basins of the Dutch Wadden Sea can keep pace with rising sea level. His work and theories also extend to more general inlet configurations which are representative of the inlets along the south shore of Long Island.

According to van Goor, a tidal inlet system is in a state of dynamic equilibrium under a steady relative sea level rise. A depth increase by sea level rise is compensated by a depth decrease due to sediment accretion over the inlet system. This concept is similar to the Bruun Rule for beach profiles (Bruun, 1962), which states that beaches follow a characteristic profile shape based on the wave climate and the type of sediment and that when sea level rises sand is eroded from the upper beach and deposited farther seaward to re-establish the profile at a higher elevation. This transfer also results in net shoreline recession. In the case of an inlet, sea level rise and the associated demand for sand creates a sediment sink in the littoral sediment transport system.

Table 5-10 shows the estimated sediment demand for each morphological feature at Shinnecock Inlet (excluding the updrift and downdrift beaches, for which the effects for sea level rise are analyzed using Bruun's Rule). The Nearshore Flood Shoal, East Channel, and Flood Shoal cells were aggregated into one, all-inclusive, Flood Shoal feature. Sediment demand was computed as the area of each feature (which, according to van Goor, does not change under the influence of sea level rise only) times the rate of sea level rise. The latter was estimated at 0.003 m/yr (i.e., 0.3 m or roughly 1 feet in 100 years) based on over 90 years of tidal records at the Battery in New York City. Although slightly different future relative sea level rise estimates may be developed based on different assumptions or data records (e.g., tidal records at Sandy Hook, NJ), the estimate based on the Battery was used to maintain consistency with previous work (e.g., Gravens et al., 1999). The approximate area of each inlet feature was determined based on the polygons shown in Figure 5-5.

Table 5-10: Sediment Demand due to Relative Sea Level Rise: at Shinnecock Inlet

Sediment Budget Cell	Area (x 1000 m^2)	Sediment Demand/ Accretion (m^3/yr)
Updrift Lobe	1,048	3,000
Deposition Basin	207	< 1,000
Channel Throat	149	< 1,000
Flood Shoal	4,195	13,000
West Beach	437	1,000
Downdrift Lobe	2,227	7,000
Total	8,263	24,000

Recent (1995-2001) Inlet Sediment Budget

Computed volume changes within each cell and engineering activity records were used to develop a sediment budget for the inlet. The budget provides a balance of sediment movement for the inlet built upon balance for each cell or control volume, which expressed as (adapted from Rosati and Kraus, 1999):

$$\sum Q_{IN} - \sum Q_{OUT} - \sum \Delta V + P - R = residual \qquad \text{Equation 5-4}$$

where all terms are expressed as a volume or as a volumetric change rate. Q_{IN} are the sources (e.g., bluff erosion, incoming LST) to the control volume, conversely, Q_{OUT} are the sinks (e.g., Sea Level Rise, outgoing LST) to the control volume. ΔV is the net volume change within the cell, P and R are the amounts of material placed in and removed from the cell, respectively, and *residual* represents the degree to which the sediment budget is balanced. For a balanced budget, the residual is zero. This framework is the basis for all of the update work.

A qualitative sediment budget that illustrates the assumed sediment pathways between the different inlet cells is presented in Figure 5-17. The following paragraphs present the sediment budget results for the 1995 to 2001 period for each of the inlet cells. Sediment input to the UBCH cell (net westerly LST of

279,000 m^3/yr) is taken from the *Recent (1995-2001)* sediment budget developed and part of this study and presented in detail in Section 0 below.

Updrift Beach (UBCH) [Sta. M0.6 to M3.2]

As shown in Table 5-7, the updrift beach apparently accreted from 1995 to 2001. The magnitude of the change varies depending of the data source from 27,000 m^3/yr based on shoreline data to 108,000 m^3/yr based on profile data (three profiles available in this cell, two of them long). The *Historical (1979-95)* sediment budget from the previous work (Gravens et al., 1999) also indicated accretion in this cell, with a magnitude of 27,000 m^3/yr, although some placement (6,000 m^3/yr) occurred during the *Historical* period. This value was reduced to zero in CHL's *Existing (c. 1999)* condition sediment budget.

The updated results indicate that this area continued to accrete during the 1995-01 period. Considering sea level rise as in the previous work by Gravens et al. (1999)[3], the total sediment accumulation in this cell using shoreline change data would be 27,000 m^3/yr + 6,000 m^3/yr = 33,000 m^3/yr. This estimate may be compared with the volume changes computed based of profile data (108,000 m^3/yr), which directly account for changes due to sea level rise. Based on the previous findings the estimate based on shoreline data was chosen as reasonable and representative of recent (i.e., 1995 to 2001) conditions, although it is clear that there is significant uncertainty with regards to short-term changes. Changes based on profile data appear to yield unrealistically large accumulation values in this case. Writing Equation 5-4 specifically for this cell and entering these values gives,

$$Q_{W\_UBCH} - Q_{UBCH\_UL} - \Delta V_{UBCH} + P_{UBCH} - R_{UBCH} = residual$$
$$279{,}000 - Q_{UBCH\_UL} - 33{,}000 + 0 - 0 = residual = 0$$
$$\therefore\ Q_{UBCH\_UL} = 246{,}000 \text{ m3/yr}$$

That is, after accretion in the UBCH cell, net westerly longshore transport into the UL cell is 246,000 m^3/yr.

Updrift Lobe (UL)

Measured volume changes in the updrift lobe range from erosion to accretion (Table 5-6). Erosion at a rate of –91,000 m^3/yr was obtained comparing the September 1998 to July 2000 synthetic grid and at a rate of –43,000 m^3/yr from May 1996 to July 2001. Accretion at a rate of 17,000 m^3/yr was suggested by the May 1996 to May 1998 comparison, 155,000 m^3/yr using the August 1997 to May 1998 comparison and 2,000 m^3/yr from the July 2000 to July 2001 comparison.

An average rate of change for the 1996-2001 period can be obtained by using a weighted average of the May 1996 to May 1998 comparisons (before dredging) plus the September 1998 to July 2000 comparison (after dredging), and the July 2000 to July 2001 comparison. Using this approach the average rate of change is -29,000 m^3/yr, which was computed as follows: [2(17,000 m^3/yr) + 2(–91,000 m^3/yr) + 2,000 m^3/yr]/5. However, as mentioned above, the SHOALS surveys in 1996, 1997, and 1998 were datum-adjusted using a methodology different from that used in 2000 and 2001. Therefore, the September 1998 to July 2000 comparison may yield erroneous accretion/erosion rates throughout the inlet system. Instead, it is assumed that the 1998 to 2000 period is characterized by changes with magnitude equal to the average of the May 1996 to May 1998 changes and July 2000 to July 2001 changes. This average

[3] As in Gravens et al. (1999), volume changes due to relative sea level rise were incorporated into the sediment budget using Bruun's Rule (Bruun, 1962). Based on a relative sea level rise rate of 0.003 m/yr, the estimated volumetric loss rate is 2.31 m^3/yr per meter of shoreline.

would equal 10,000 m^3/yr, which herein is considered representative of the 1998 to 2000 period. Therefore, a revised weighted average for the 1996-2001 period yields 11,000 m^3/yr, which is computed as follows: [2(17,000 m^3/yr) + 2(10,000 m^3/yr) + 2,000 m^3/yr]/5. This is considered a more reasonable estimate as it shows slight accretion of the updrift lobe as opposed to erosion, a result more consistent with the expected morphological behavior of the ebb shoal. Note that sediment demand due to sea level rise (approximately 3,000 m^3/yr) is already included in this estimate because it should be captured by the survey comparisons. Writing Equation 5-4 specifically for this cell results in,

$$Q_{UBCH\_UL} - Q_{UL\_DB} - Q_{UL\_DL} - \Delta V_{UL} + P_{UL} - R_{UL} = residual$$

Model results (Section 4.2) and morphological changes in recent years suggest no significant direct sand transport from UL to the Downdrift Lobe (DL). Thus, it is assumed that $Q_{UL\_DL} \approx 0$, and entering the rest of the values as described in the preceding paragraph gives,

$$246{,}000 - Q_{UL\_DB} - 0 - 11{,}000 + 0 - 0 = residual = 0$$
$$\therefore\ Q_{UL\_DB} = 235{,}000\ \ \mathrm{m^3/yr}$$

That is, a net westerly transport of 235,000 m^3/yr leaves the UL and entered the Deposition Basin (DB) during the 1995 to 2001 period.

Flood Shoal (FS), Near Field Flood Shoal (NFFS), and East Channel (EC)

The volume changes in Table 5-6 show significant erosion of the FS and NFFS cells. This unexpected result is attributed to difficulties in resolving the tidal datums in the bay and inconsistencies from survey to survey. The flood shoal complex (including tidal flood channels) may be eroding slightly but it is unlikely that hundreds of thousands of cubic meters of sediment are lost from the flood shoal every year. Given the level of uncertainty regarding changes over the flood shoal, and in order to simplify the sediment balance, the following analysis and results assume one Flood Shoal (FS) cell, which also includes the Near Field Flood Shoal and the tidal flood channels.

In 1997, 191,150 m^3 were dredged from the East Channel (EC) and placed on the West Beach. Review of recent surveys in August 1997, July 2000, and July 2001 show that the dredged area has basically recovered since that time. Considering the time period from May 1996 to July 2001 for consistency with other comparisons, the dredging/infilling rate for this area was calculated as 191,150 m^3/(5.17 yrs between May 1996 and July 2001) = 37,000 m^3/yr. The source of the sand is uncertain at this time. For the purposes of the sediment budget, however, one may assume that this material comes from the littoral cells by way of the deposition basin and channel (Q_{CT_EC} = 37,000 m^3/yr) and that the flood shoal is accreting at the rate induced by relative sea level rise (ΔV_{FS} =13,000 m^3/yr). Note that this is very different than that the net erosion of 19,000 m^3/yr previously computed by Morang (1999). Nonetheless, net accretion in flood shoal is considered to be more consistent with general inlet morphology and evolution principles.

Writing Equation 5-4 specifically for the combined flood shoal (FS) and the East Cut (EC) cell results in,

$$\mathrm{FS:}\ \pm Q_{CT\_FS} - Q_{FS\_EC} - \Delta V_{FS} + P_{FS} - R_{FS} = residual$$
$$\mathrm{EC:}\ Q_{CT\_EC} + Q_{FS\_EC} - \Delta V_{EC} + P_{EC} - R_{EC} = residual$$

The balance equation for EC assumes that $Q_{OUT} \approx 0$.

Entering the values and assumptions described above gives,

$$\text{FS}: \ \pm Q_{CT\_FS} - 0 - (13{,}000) + 0 - 0 = residual = 0$$
$$\text{EC}: \ 37 + 0 - 0 + 0 - 37{,}000 = residual = 0$$

and

$$\therefore Q_{CT\_FS} = 13{,}000 \ \text{m}^3/\text{yr}$$
$$\therefore Q_{CT\_EC} = 37{,}000 \ \text{m}^3/\text{yr}$$

Channel Throat (CT)

Table 5-6 shows changes in the inlet channel throat ranging from –40,000 m^3/yr (May 1996 to May 1998) to 24,000 m^3/yr (July 2000 to July 2001). A weighted average of changes yields [2(-40,000 m^3/yr) + 2(-8,000 m^3/yr)+24,000 m^3/yr]/5 = -18,000 m^3/yr. This number is difficult to reconcile with the fact that this area is not known to be eroding. It may be explained by the fact that SHOALS survey coverage in the deeper parts of the channel is sparse or not available in the May 1996, May 1998 and July 2001 surveys, therefore results may be inaccurate and misleading. This cell is considered fairly stable and therefore volume change rate of 0 m^3/yr is assumed for the 1995 to 2001 period. Writing Equation 5-4 for the CT results in,

$$Q_{DB\_CT} - Q_{CT\_EC} - Q_{CT\_FS} - \Delta V_{CT} + P_{CT} - R_{CT} = residual = 0$$
$$Q_{DB\_CT} - 37{,}000 - 13{,}000 - 0 + 0 - 0 = 0$$
$$\therefore \ Q_{DB\_CT} = 50{,}000 \, \text{m}^3/\text{yr}$$

West Beach (WB) [Sta. W23.9 km - W24.8 km]

The West Beach shows erosion for all comparisons in Table 5-6. Two placement events between 1995 and 2001 in this area also confirm a chronic erosion problem. In 1997 and 1998, 191,150 m^3 and 286,700 m^3 were placed in this cell, respectively. Considering the time period from May 1996 to July 2001 the placement rate is (191,150 + 286,700 m^3)/5.17 years = 92,000 m^3/yr.

Volume changes from May 1996 to May 1998 span a fill event, as do changes from September 1998 to July 2000. From May 1996 to May 1998, 191,150 m^3 of material were placed. The net volume change for that period of time was -180,000 m^3. To get the erosion rate, the net volume change plus the fill is divided by the time between surveys to obtain [-180,000 m^3 - 191,150 m^3]/(2 yrs) = -186,000 m^3/yr.

Again, the 1998-2000 erosion rate was computed as the average of the 1996-98 and 2000-01 rates, (-186,000 m^3/yr –45,000 m^3/yr)/2=-116,000 m^3/yr, and a weighted average erosion rate for the 1996 to 2001 period was computed as [2(-186,000 m^3/yr) + 2(-116,000 m^3/yr) –45,000 m^3/yr]/5 = -130,000 m^3/yr. Note that this value represents an erosion rate accounting for fill placement. In other words, considering the average fill rate over this period, the actual (i.e., observed) rate of volumetric change at WB would be ΔV_{WB} = -130,000 m^3/yr + 92,000 m^3/yr = -38,000 m^3/yr.

Shoreline morphology, model results (see Section 4.2) and recent changes characterized by chronic erosion suggest that net influx of sediment into the WB from any of the adjacent cells is relatively small. Therefore, for simplicity it was assumed that this influx was zero (Q_{IN} = 0) and that eroded sediment is transported into either the Deposition Basin (DB) or the Downdrift Lobe (DL). Note that this assumption does not affect the overall sediment balance for the inlet and the net effect it has on the regional sediment budget. Therefore, writing Equation 5-4 for the WB gives,

$$0 - Q_{WB\_DL} - Q_{WB\_DB} - \Delta V_{WB} + P_{WB} - R_{WB} = residual = 0$$

Shoreline morphology and model results also suggest that net westerly transport from WB toward DL is relatively small. Therefore, it was also assumed that Q_{WB_DL} is zero. Note that if some transport from WB to DL was assumed it would only reduce the transport from WB to DB and then (see below) the transport from DB to DL. However, ultimately the total net transport influx to DL would be the same. Entering values in the balance equation gives,

$$0 - 0 - Q_{WB\_DB} - (-38{,}000) + 92{,}000 - 0 = residual = 0$$
$$\therefore\ Q_{WB\_DB} = 130{,}000\ \mathrm{m^3/yr}$$

Deposition Basin (DB)

The deposition basin at Shinnecock was established in October 1990 and dredged again in Jan-May 1993 and September of 1998. The deposition basin was also recently dredged in 2004 (approximately 275,000 $m^{3)}$. Examining Table 5-6 shows that volume change rates in the deposition basin range from –30,000 m^3/yr (from 1996 to 2001) to 285,000 m^3/yr (from August 1997 to May 1998). Note that the May 1996 to July 2001 period includes one dredging event in 1998, resulting in a dredging rate of approximately 65,000 m^3/yr. More importantly, as mentioned above, the 1996 to 2001 and 1998 to 2000 comparisons may provide inaccurate estimates of volume change due to datum inconsistencies.

The August 1997 to May 1998 time period was very active in terms of sediment transport, due to a particularly vigorous wave climate. The July 2000 to July 2001 time span, on the other hand, is characterized by little morphological change and milder waves. The May 1996 to May 1998 comparison includes milder conditions from 1996 to 1997 as well as the active transport of 1997 to 1998. The wave climate patterns and observed morphological changes appear to suggest that filling of the deposition basin is episodic and linked to active wave conditions. The deposition basin may fill slowly for several years and then experience a large amount of accumulation during a particularly stormy year.

A representative rate of accumulation (net of dredging) in the DB was obtained by using a weighted average of the 1996-98 and 2000-01 changes as [2(95,000 m^3/yr) + 2(58,000 m^3/yr) + 21,000 m^3/yr]/5 = 65,000 m^3/yr. This is the same methodology applied to obtain volume changes in the UL and WB cells. That is, the rate of change from 1998 to 2000 was computed as the average of the 1996-98 and 2000-01 rates, (95,000 m^3/yr+21,000 m^3/yr)/2=58,000 m^3/yr.

The accumulation rate happens to be equal to the average dredging rate between May 1996 and July 2001, 336,000 m^3/(5.17 yrs) = 65,000 m^3/yr. This result seems reasonable given the fact that, on average, the deposition basin is dredged at approximately the rate it accumulates sediment, such that the net accumulation in the deposition basin is zero. Writing Equation 5-4 for the DB gives,

$$Q_{UL\_DB} + Q_{WB\_DB} - Q_{DB\_CT} - Q_{DB\_DL} - \Delta V_{DB} + P_{DB} - R_{DB} = residual = 0$$

Entering values into this equation gives,

$$235{,}000 + 130{,}000 - 50{,}000 - Q_{DB\_DL} - 0 + 0 - 65{,}000 = residual = 0$$
$$Q_{DB\_DL} = 250{,}000 \text{ m}^3/\text{yr}$$

That is, changes suggest that approximately 250,000 m^3/yr of sand were transported toward the Downdrift Lobe (DL) during the 1995 to 2001 period.

Downdrift Lobe (DL) [Sta. 22.4 km to 23.9 km]

DL volume change rates shown in Table 5-6 vary from –316,000 m^3/yr from September 1998 to July 2000 to 285,000 m^3/yr from August 1997 to May 1998. A weighted average of 1996-98 and 2000-01 change rates calculated in the same manner as those computed for the UL and DB yields –51,000 m^3/yr, [2(96-98 change) + 2(average of 96-98 and 00-01 change) + 00-01 change]/5.17. In 1998, 49,700 m^3 of fill were placed in the DL cell extending from the fill in the WB cell. This corresponds to a fill rate of 10,000 m^3/yr from May 1996 to July 2001 (49,700 m^3/5.17 years).

It is helpful to consider the qualitative changes shown in Figure 5-12 to Figure 5-15 when evaluating the sediment transport patterns of the downdrift lobe. From May 1996 to August 1997 accretion is evident across the shallow crest of the lobe, hereon referred to as the bypassing bar, combined with erosion up- and down-drift. More erosion is evident just east of the attachment point. Fill from the dredging of the East Channel in 1997 is also evident east of the attachment point. From August 1997 to May 1998 it appears that the accretion feature on the 1996-1997 comparison has eroded and shifted to the west. The area of erosion on the 1996-1997 comparison near the attachment point has filled, possibly with sediment from the erosion and migration of the west beach fill. Although possible datum questions make the September 1998 to July 2000 comparison less useful quantitatively, qualitatively it appears that the bypassing bar has begun to migrate onshore, with erosion at its former location and accretion at the attachment point and west. In addition, the west beach fill seems to be moving offshore towards the deposition basin or attachment bar. From 2000 to 2001 there is little change; what may be material from the fill project has continued to accumulate in the bypassing bar, while the area just east of the bypassing bar eroded as the sediment moved west. Additional accumulation west of the attachment point is also noted.

It is theorized that before the dredging in September 1998, the bypassing bar was shifting back and forth in response to waves and currents, with bypassing at the attachment point and downdrift due to wave action. After the dredging, it appears that the bypassing bar experienced a reduction in sediment influx and as a result gradually moved onshore at the attachment point, eroding from its former position. Sediment from the west beach fill project appears to be replenishing the bypassing bar to some degree possibly by way of the deposition basin.

These qualitative results seem to indicate that bypassing is occurring in an episodic fashion. The fill placed in the WB and DL cells appears to be eventually bypassing to the attachment point and west although episodic erosion of the bypassing bar is likely just after dredging of the deposition basin. Weighted average changes for the study period suggest erosion, although the area has the potential to rapidly accumulate sand under the right conditions as indicated by changes from August 1997 to May 1998. General inlet morphodynamic principles also make the possibility of a long-term eroding trend very unlikely. Nonetheless, the sediment balance for the relatively short 1995-2001 study period would be as follows:

$$Q_{DB\_DL} + Q_{WB\_DL} + Q_{UL\_DL} - Q_{DL\_DBCH} - \Delta V_{DL} + P_{DL} - R_{DL} = residual = 0$$
$$250{,}000 + 0 + 0 - Q_{DL\_DBCH} - (-51{,}000) + 10{,}000 - 0 = 0$$
$$\therefore Q_{DL\_DBCH} = 311{,}000 \text{ m}^3\text{/yr}$$

In summary, the net westerly transport from the DL toward the Downdrift Beach (DBCH) was 311,000 m^3/yr during the 1995-2001 study period.

Alternatively, the relatively small accretion rate of 7,000 m^3/yr as computed in the DL cell from May 1996 to May 1998 and consistent with sediment demand estimates due to sea level rise (see Table 5-10) could be considered more representative of *Existing Conditions* and “normal” tidal inlet morphodynamic processes. Entering this alternative value in the sediment balance equation gives,

$$250{,}000 + 0 + 0 - Q_{DL\_DBCH} - 7{,}000 + 10{,}000 - 0 = 0$$
$$\therefore Q_{DL\_DBCH} = 253{,}000 \text{ m}^3\text{/yr}$$

That is, the net westerly transport from the DL toward the Downdrift Beach (DBCH) was 253,000 m^3/yr during the 1995-2001 study period.

That is, 311,000 m^3/yr were estimated to bypass the inlet system, , which includes the channel, deposition basin, shoals and west beach, during the 1995-01 period under the first DL scenario and 253,000 m^3/yr under the second DL scenario. These amounts represent 126% and 103% of the updrift net longshore sediment transport rate entering the Updrift Lobe, respectively. These two alternative sediment budgets representative of the recent changes observed between 1995 and 2001 are presented in Figure 5-18 and Figure 5-19.

Downdrift Beach (DBCH) [Sta. 21.6 km to 22.4 km]

The downdrift beach volume changes computed using shoreline change and profiles are shown in Table 5-7. Profile data (two profiles over 800 m of shoreline) indicate 2,000 m^3/yr accretion in this area, while the shoreline changes indicate –12,000 m^3/yr of erosion. When using the shoreline change an impoundment of 2,000 m^3/yr due to sea level rise must also be considered, giving a net loss within this cell of –10,000 m^3/yr. Given that two profiles were available within this relatively short cell, the value calculated using profile changes, 2,000 m^3/yr, was considered representative of the 1995-2001 period. Note that this value also matches the estimated accumulation due to sea level rise. Entering values in Equation 5-4 gives,

$$Q_{DL\_DBCH} - Q_{DBCH\_West} - \Delta V_{DBCH} + P_{DBCH} - R_{DBCH} = residual = 0$$
$$311{,}000 - Q_{DBCHB\_West} - 2{,}000 + 0 - 0 = 0$$
$$\therefore Q_{DBCHB\_West} = 309{,}000 \text{ m}^3\text{/yr}$$

or entering values from DL alternative budget,

$$253{,}000 - Q_{DBCH\_West} - 2{,}000 + 0 - 0 = 0$$
$$\therefore Q_{DBCH\_West} = 251{,}000 \text{ m}^3\text{/yr}$$

Summary of Results for the Recent (1995-2001) Sediment Budget

Relevant conclusions from the *Recent (1995-2001)* sediment budget at Shinnecock Inlet are as follows:

- Apparently, the Updrift Beach accreted significantly (33,000 m^3/yr) over the 1995-01 period. Even if these changes are accurate, and not an error due to survey and/or shoreline delineation inaccuracies, it is unlikely that this condition will be sustainable in long-term. An *Existing (c.2001)* condition budget should assume that this area is relatively stable (see below).
- The Updrift Lobe appears to be slightly accretional.
- The West Beach area continues to erode, apparently at a slightly faster rate than the fill placement rate (the net deficit for the 1995-01 period was 38,000 m^3/yr), although this result might be artifice of the selected averaging period. Beach fill and erosion should approximately balance in the long-term. Note, however, that this does not mean that the west beach shoreline is stable; on the contrary, this reach suffers chronic erosion at the greatest rate observed within the FIMP project area.
- As expected, dredging and shoaling within the Deposition Basin approximately balance out during this period.
- It is difficult to assess sediment transport pathways and quantities in the Flood Shoal complex. Therefore, and until better quality survey data over a larger extent is available, it is recommended that the flood shoal be assumed to be accreting at the rate required to keep up with sea level rise.
- The Downdrift Lobe appears to bypass sediment to the west in episodic fashion, with periodic accretion and erosion events that seem to result in net sediment transport from the shoal to the attachment point and westward. A net trend is difficult to assess based on the short-term data available since construction of the deposition basin and the realignment of the channel. Nonetheless, over the longer term, the Downdrift Lobe is considered to be slightly accretional.

Existing (c. 2001) Condition Sediment Budget

An *Existing (c. 2001)* condition sediment budget for Shinnecock Inlet also reflecting updated regional sediment budget results (see Section 6) is presented in Figure 5-20. Note that the *Existing (c. 2001)* regional sediment budget suggests a net westerly transport rate of 157,000 m^3/yr entering the updrift (east) boundary of the inlet system (instead of 279,000 m^3/yr in the *Recent* sediment budget). Also note that proposed *Existing (c.2001)* condition change rates for the ebb shoal and deposition basin are based mostly on changes measured during the sediment budget update period of 1995 to 2001. It is fully acknowledged that six years is a relatively short period of time to develop accurate and reliable estimates of change for these features. However, this was the only period of time for which data representative of current inlet management practices (the channel and deposition basin were built in 1990) and morphology were available. Datasets prior to construction of the deposition basin are not considered illustrative of existing conditions and comparison with recent surveys might suggest erosion or accretion patterns directly related to the new channel configuration but not representative of future trends. The only other extensive dataset available in the 1990's is the 1994 SHOALS survey, but as explained above this survey was not used due to concerns regarding the vertical datum (Morang, personal communication, June 2002).

Specific assumptions for the *Existing (c. 2001)* Shinnecock Inlet sediment budget are as follows:

- Stability of the Updrift Beach. This means that only enough sediment to keep up with sea level rise is accumulated within this reach (i.e., approximately 6,000 m^3/yr).
- Slightly accreting Updrift Lobe, ΔV_{UL} = 11,000 m^3/yr, based on data from the 1995 to 2001 period; roughly 3,000 m^3/yr as a result of sea level rise.
- No dredging of the East Cut flood shoal channel.
- Stable Flood Shoal with accumulation due to sea level rise (ΔV_{FS} =13,000 m^3/yr),
- Net shoreline stability at the West Beach through continued beach fill placement. Accumulation due to sea level rise (ΔV_{WB} = 1,000 m^3/yr),

- Continued dredging of the Deposition Basin at a rate of 65,000 m^3/yr. Accumulation due to sea level rise is less than 1,000 m^3/yr)
- Stable Channel Throat. Accumulation due to sea level rise is less than 1,000 m^3/yr and therefore it was not accounted for in the budget.
- Relatively stable Downdrift Lobe accreting slightly due to sea level rise (ΔV_{DL} = 7,000 m^3/yr). Therefore, the total accumulation rate over the ebb shoal complex (including the West Beach cell) is assumed to be approximately 19,000 m^3/yr. Note that this is roughly two thirds of the accretion rate measured between 1984 and 1998. This reduction seems reasonable considering that the ebb has probably adjusted further to the channel and deposition basin changes implemented in 1990.
- Stable Downdrift Beach with slight volume accumulation due to sea level rise (ΔV_{DBCH} = 2,000 m^3/yr)

Under these assumptions, approximately 79% (119,000 m^3/yr) of the net updrift westerly transport entering the Updrift Lobe (151,000 m^3/yr) bypasses the inlet system, which includes the channels, deposition basin, shoals and west beach. The remaining 21% (32,000 m^3/yr) accumulates within the inlet shoals. Therefore, this new *Existing (c. 2001)* budget suggests that the net overall impact of Shinnecock Inlet to the regional sediment budget is reduced as compared to the conclusions presented in the previous work by USACE-NAN (1998) and adopted by Gravens et al. (1999) which suggested that 57,000 m^3/yr accumulate within the inlet. This finding seems consistent with the idea that the inlet system has further adjusted to the configuration implemented in 1990 when the deposition basin was dredged and the channel was realigned. However, only continued monitoring and additional detailed survey data over the next decade or so will confirm or refute this finding.

Finally, it should be noted that this *Existing (c. 2001)* condition budget will be temporarily altered by the recent implementation of the West of Shinnecock Interim (WOSI) Storm Damage Project. This project provides beach fill to the West Beach cell from an offshore borrow site instead of the deposition basin. Sand dredged from the deposition basin is placed farther west beyond the direct area of influence from the inlet. Initial beach fill placement as part of WOSI was completed in March 2005. The project includes two additional renourishments for a period not to exceed 6 years. Note that the net effect in terms of the sediment budget will be relatively minor as far as the inlet itself. The WB cell will continue to erode and sand will flow back mostly into the deposition basin, which will continue to be dredged periodically. The only difference is that Tiana Beach, downdrift of the inlet, will receive an additional influx of sediment roughly equal to the amount of material dredged from the offshore borrow site. It is assumed that after WOSI ends (i.e., 2011) conditions will gradually revert back to *Existing* as described above.

5.2 Moriches Inlet

5.2.1 Existing Condition

Unfortunately Moriches Inlet has been monitored less frequently than Shinnecock Inlet so fewer hydrographic surveys exist and spatial coverage is more limited. Tidal elevation measurements are also scarcer at Moriches Inlet. This section describes three recent bathymetric surveys and changes in inlet morphology from survey to survey. Hydraulic and inlet stability analysis are conducted following methods outlined in Section 5.1.1 to determine theoretical cross-sectional area, ebb shoal capacity and to identify trends toward closure or scour.

Bathymetric Records

At Moriches Inlet, two SHOALS surveys are available for recent years dated May 1996 and July 2000. Additional survey data obtained using traditional hydrographic survey methods is available on March 1981, March 1998, September 1998, October 1998, April 2000, April 2001, and April 2002. Figures showing these surveys are included in Appendix A.

Hydraulic Analysis

Tidal inlet prism and morphology relationships are applied to Moriches Inlet. The analysis follows methods outlined in Section 5.1.1 and is designed to assess the theoretical equilibrium state of the inlet. Current measured conditions at the inlet are compared with the theoretical equilibrium values to qualitatively assess probable future conditions.

Measured Conditions

Recent minimum cross-sectional areas, approximate average depths and estimated ebb shoal volumes are presented in Table 5-11. Minimum cross-sectional areas are measured using the surveys presented in Appendix A. It is noted that the cross-sectional area from 1998 shown in Table 5-11 may be underestimated due to lack of survey coverage in shallow depths. Average depth is obtained by dividing the cross-sectional area by the stabilized inlet channel width of 800 feet. Ebb shoal volume is estimated using comparisons with 1933 bathymetric survey data, which was collected shortly after inlet formation. Note estimated volumes are somewhat smaller than previously documented (USACE-NAN, 1998). These differences are related to available survey coverage and confidence on interpolation between available datasets. For example, previous estimates based on 1996 data include a large coverage west of the inlet based on a few available profiles. In this study it was determined that those profiles do not offer sufficient data density and that interpolation between them does not provide a reliable estimate of volume changes. It was judged that the approach followed in this study is more reliable, even though it does not account for the complete ebb shoal. Moreover, the estimates presented in Table 5-11 are similar to those recently computed by Allen et al. (2002) using fairly detailed hydrographic surveys in 1995, 1996 and 1999.

Overall, estimated ebb shoal volumes for Moriches Inlet are roughly half to two thirds of those estimated for Shinnecock Inlet (4 million cy at Moriches Inlet versus 6-8 million at Shinnecock). Some of this relatively large difference may be to differences in tidal prism and ebb shoal capacity. However, careful inspection of the available bathymetry data suggest that most of the additional volume at the Shinnecock Inlet ebb shoal is located within the updrift reaches of the shoal (east of the updrift jetty alignment) and the adjacent beaches. Increased accumulation in this area at Shinnecock Inlet is likely due to a significantly longer updrift jetty structure and smaller sediment influx at Moriches Inlet due to the effects of the Westhampton Groin field. In addition, in the case of Moriches Inlet, the 1933 survey already included a small amount of ebb shoal accumulation, albeit at the former inlet position, which also results in smaller net increase in ebb shoal volume from 1933 to date.

Table 5-11: Moriches Inlet Existing Hydraulic Characteristics

Year	Measured Minimum Cross-Sectional Area (sq. ft)	Average Depth (ft)	Estimated Ebb Shoal Volume[1] (cy x 10^6)
Dredging, 1996, 256,600 cy removed			
1996	13,600	17.1	1.9
1998	11,750	14.8	2.4
Dredging, 1998, 186,200 cy removed			
2000	15,000	18.8	4.1[2]

[1]Ebb shoal volume estimated by comparisons of bathymetric data with bathymetry surveyed in 1933, just after Moriches Inlet opened. Comparisons necessarily include all oceanside inlet effects on bathymetry; some surveys may cover more of the shoal than other surveys. See Appendix A for inlet survey coverage.
[2] The 2000 survey has the best coverage of the inlet and ebb shoal features.

Tidal Prism - Theoretical Conditions

Theoretical equilibrium conditions for the existing tidal prism at Moriches Inlet are calculated based on the most appropriate data available.

At Moriches Inlet, NOAA tidal records are considered outdated—the secondary stations in Moriches Bay are based on observations taken from 1939-1940 and from 1940-1941 for Potunk Point and Mastic Beach, respectively (Tide.Predictions@noaa.gov, pers. comm. July 2002). NOAA records indicate tide ranges in the bay of about 0.5 feet; this is considered a fairly substantial underestimate. Therefore, bay tidal records from the LI Shore monitoring program at Moriches Coast Guard Station and Smith Point Bridge are used to estimate tide range in the bay. Instrument locations are shown in Figure 4-32. Average tidal range in Moriches Bay obtained using these records is 1.8 feet.

Table 5-12: Tidal Datums for Moriches Bay Tide Gauges

Tidal Datum (ft)	P6[1] Moriches Coast Guard Station	P7[1] Smith Point Bridge
MHHW	2.62	1.48
MHW	2.39	1.33
MSL	1.24	0.74
MTL	1.24	0.71
MLW	0.10	0.09
MLLW	0.00	0.00

[1] Based on observations from April 2000 – November 2001.

Using Equation 5-1 and the approximate surface area of Moriches Bay, 16 square miles, tidal prism is estimated as 800 x 10^6 ft^3.

Theoretical minimum cross-sectional area is calculated with a tidal prism value of 800 x 10^6 ft^3 and Equation 5-2 as 17,300 ft^2 at Moriches Inlet.

Equation 5-3 and the tidal prism value, 800 x 10^6 ft^3, are used to calculate a theoretical ebb shoal capacity at Moriches Inlet of 9 x 10^6 cy. Therefore, this analysis would suggest that the ebb shoal volume is less 50% of the estimated equilibrium volume (for the available survey coverage). However, as explained in Section 3.3, there is a significant level of uncertainty associated with this estimate. As with Shinnecock Inlet, one must be very careful not to rely too much in this type of analysis when formulating inlet management alternatives.

After dredging in 1996, the minimum cross-sectional area was nearly 80% of the theoretical cross-sectional area. The 1998 cross-sectional area appears to have decreased to close to 70% of the equilibrium value, but this may be due to an underestimate of the area where no data exists in shallow water at the edges of the channel in the 1998 condition survey. After dredging in 1998, the inlet shows a minimum cross-sectional area of almost 90% of the theoretical value. Overall, it appears that Moriches Inlet is nearly at an equilibrium state. However, the inlet may have the capacity to the potential to scour the inlet to deeper channel depths as well as to trap some additional sediment in the ebb shoal.

<u>Inlet Stability Analysis</u>

Escoffier's method for assessing inlet stability (Escoffier 1940) is applied to existing and historic conditions at Moriches Inlet. Moriches Bay surface area of 16 square miles and ocean tidal amplitude of 1.75 feet are considered for Moriches Inlet. A friction coefficient of 0.0035 is employed and inlet length is approximated as 2,500 feet. Ocean tidal amplitude is assumed constant over the 1996 to 2000 time period.

Figure 5-21 presents the inlet closure curve for recent and existing cross-section data at Moriches Inlet and includes measured cross-sections. The equilibrium flow velocity shown is approximately 3.3 ft/sec, a value suggested by Bruun (1978), Van de Kreeke (1984) and others as appropriate for most inlets.

The closure curve for Moriches Inlet is similar to that for Shinnecock Inlet (Figure 5-4). The curve suggests that Moriches Inlet is stable with a tendency to scour. Moriches Inlet has been dredged twice in recent years, in 1996 and in 1998. The 1996 dredging was conducted before the 1996 survey; the 1998 dredging occurred after the 1998 survey. Figure 5-21 suggests that the inlet shoaled from 1996 to 1998, however, cross-sectional area shown for 1998 may be underestimated due to lack of survey coverage in shallow depths. The inlet has remained fairly stable in recent years and cross-sectional area appears to be slightly increasing. Presently, Moriches Inlet has a cross-sectional area of approximately 15,000 ft^2 and continued stability of the inlet is anticipated. Figure 5-21 indicates a stable cross-sectional area of about 33,000 ft^2, nearly twice the theoretical equilibrium value of 17,300 ft^2 calculated using Equation 5-2. If the inlet were to reach a cross-sectional area of 33,000 ft^2, average depth considering the fixed width of 800 feet would be 42 feet. Recently, areas near the west jetty are the only ones approaching this depth, as visible in Figure 5-26 to Figure 5-28.

<u>Recent Volume Changes (1995-2001)</u>

Eight sediment budget cells were delineated to represent Moriches Inlet and the adjacent beaches. These cells are presented in Figure 5-22. The Updrift Beach (UBCH) extends from 600 meters east of Moriches Inlet to 3,200 meters east of the inlet, corresponding to the ME sediment budget cell in previous work by CHL. The Updrift Lobe (UL) reaches from the east jetty to 600 meters east of the inlet, extending to approximately the –40 foot contour. The Deposition Basin (DB) cell is as defined by the USACE for navigation channel maintenance. The Channel Throat (CT) is bound by the east and west jetties. The Flood Shoal (FS) extends into the back bay. The West Beach (WB) extends from the west jetty 600 m to the west, and corresponds to CHL stationing FI48.6 km to FI49.2 km. The Downdrift Lobe (DL) reaches from 600 m west of the inlet another 1800 m, CHL stationing FI46.8 km to FI48.6 km. The Downdrift Beach (DBCH) extends 800 meters west of the DL, from CHL stations FI46 km to FI46.8 km.

Available bathymetric surveys, shoreline data and beach profile surveys were employed to formulate the Moriches Inlet sediment budget. Plots of all available bathymetric surveys are presented in Appendix A. Appendix B describes each data set used in sediment budget preparation. Bathymetric surveys from 1996, 1998, 2000, 2001 and 2002 were used in combination with beach profile surveys to create a series of synthetic grids for Moriches Inlet. Each grid had 10-foot (3.3-m) spacing and was referenced to the New York State Plane Long Island NAD 83 coordinate system and NGVD 1929. Data sources used for each of the grids are listed in Table 5-13. Figure 5-23 to Figure 5-28 show synthetic grid bathymetry/topography for Spring 1996, Spring 1998 (pre-dredge), Fall 1998 (post-dredge), Spring-Summer 2000, Spring, 2001, and Spring-Summer 2002.

Table 5-13: Synthetic Grids and Source Data for Moriches Inlet Sediment Budget

Synthetic Grid		Data Source 1		Data Source 2		Data Source 3	
Moriches Inlet Spring 1996 (Post-Dredge)		SHOALS Survey, 22-23 May 1996		ACNYMP Profile Survey, March 1996 F80-84; W1-6			
Filename	mi96synthgrd	*Datum, Conversion to NGVD*	NGVD	*Datum, Conversion to NGVD*	NGVD		
Moriches Inlet Spring 1998 (Pre-Dredge)		Condition Survey, 11-13 March 1998		ACNYMP Profile Survey, February-March 1998 F80-84			
Filename	mi98synthgrd	*Datum, Conversion to NGVD*	MLW 0.4 ft below NGVD	*Datum, Conversion to NGVD*	NGVD		
Moriches Inlet 1998 (Post-Dredge)		Condition Survey, 20 October 1998		Condition Survey, 11-13 March 1998		ACNYMP Profile Survey, February-March 1998 F80-84	
Filename	mipstdrggd98	*Datum, Conversion to NGVD*	MLW 1.7 ft below NGVD	*Datum, Conversion to NGVD*	MLW 0.4 ft below NGVD	*Datum, Conversion to NGVD*	NGVD
Moriches Inlet Spring-Summer 2000		SHOALS Survey, 3, 5-8 July 2000		ACNYMP Profile Survey, April 2000 W1-5, W720, W740, WHV7-9			
Filename	mi00synthgrd	*Datum, Conversion to NGVD*	NGVD	*Datum, Conversion to NGVD*	NGVD		
Moriches Inlet Spring 2001		Condition Survey, 6, 15-16 April 2001		ACNYMP Profile Survey, April 2001 F80-84, W1-5			
Filename	mi01synthgd	*Datum, Conversion to NGVD*	MLW 0.3 ft below NGVD	*Datum, Conversion to NGVD*	NGVD		
Moriches Inlet Spring-Summer 2002		Condition Survey, 6-7 April 2002					
Filename	mi02condgd	*Datum, Conversion to NGVD*	MLW 0.3 ft below NGVD				

Volume change rates were developed by comparing the synthetic grids in sediment budget cells UL, DB, CT, FS, WB, and DL. Table 5-14 presents volumetric changes calculated with the synthetic grid comparisons. Because UB and DBCH extend past most survey coverage, volume change rates in those cells were assessed with shoreline and beach profile data. Shorelines used were digitized from aerial photographs dated March 1995 and April 2001 at CHL and CENAN, respectively. Beach profile data collected in Fall 1995, Spring 1998, and Spring 2001 as part of the ACNYMP were also compared. Table

5-15 shows the volume change rates calculated using both data sets for UB and DBCH. Figure 5-29 to Figure 5-32 [S96-S98, F98-S00, S00-S01, S01-S02] are plots of the spatial distribution of the topographic and bathymetric changes from the synthetic grid comparisons.

Table 5-14: Volumetric Changes from Synthetic Grid Comparisons: Moriches Inlet

Inlet Segment	From	To	Cut (m3)	Fill (m3)	Net (m3)	Net/yr (m3/yr)
Updrift Lobe	May-96	March-98	-125,000	78,000	-47,000	-26,000
Deposition Basin	May-96	March-98	-13,000	172,000	159,000	88,000
Channel Throat	May-96	March-98	-57,000	62,000	5,000	3,000
West Beach	May-96	March-98	-69,000	31,000	-38,000	-21,000
Downdrift Lobe	May-96	March-98	-36,000	54,000	18,000	10,000
Flood Shoal	May-96	March-98	-73,000	62,000	-11,000	-6,000
Updrift Lobe	October-98	July-00	-95,000	176,000	81,000	48,000
Deposition Basin	October-98	July-00	-20,000	99,000	79,000	47,000
Channel Throat	October-98	July-00	-161,000	26,000	-135,000	-79,000
West Beach	October-98	July-00	-95,000	15,000	-80,000	-47,000
Downdrift Lobe	October-98	July-00	-196,000	133,000	-63,000	-37,000
Flood Shoal	October-98	July-00	-340,000	66,000	-274,000	-161,000
Updrift Lobe	May-96	July-00	-270,000	211,000	-59,000	-14,000
Deposition Basin	May-96	July-00	-15,000	117,000	102,000	25,000
Channel Throat	May-96	July-00	-161,000	28,000	-133,000	-32,000
West Beach	May-96	July-00	-154,000	6,000	-148,000	-36,000
Downdrift Lobe	May-96	July-00	-196,000	133,000	-63,000	-15,000
Flood Shoal	May-96	July-00	-249,000	99,000	-150,000	-36,000
Updrift Lobe	July-00	April-01	-56,000	222,000	166,000	210,000
Deposition Basin	July-00	April-01	-5,000	60,000	55,000	70,000
Channel Throat	July-00	April-01	-37,000	100,000	63,000	80,000
West Beach	July-00	April-01	-26,000	54,000	28,000	35,000
Downdrift Lobe	July-00	April-01	-59,000	193,000	134,000	170,000
Flood Shoal	July-00	April-01	-21,000	91,000	70,000	89,000
Updrift Lobe	April-01	April-02	-73,000	97,000	24,000	25,000
Deposition Basin	April-01	April-02	-26,000	24,000	-2,000	-2,000
Channel Throat	April-01	April-02	-25,000	52,000	27,000	28,000
West Beach	April-01	April-02	-34,000	15,000	-19,000	-19,000
Downdrift Lobe	April-01	April-02	-69,000	130,000	61,000	63,000
Flood Shoal	April-01	April-02	-34,000	37,000	3,000	3,000

Note that comparison of the surveys on July 2000 (SHOALS survey) and April 2001 ("traditional" hydrographic condition survey) appears to indicate a very high accretion rate over most of the overlapping survey coverage. This period does not correspond to a particularly active wave climate and changes over this area during other periods are much less significant. Therefore, the changes are considered unrealistic and likely to be due to differences in the vertical datum reduction methodology for both surveys.

Table 5-15: Volumetric Changes from Shoreline and Profile Comparisons: Moriches Inlet					
Inlet Segment	**From**	**To**	**Data Used**	**Volume Change Rate (m3/m/yr)**	**Net/yr (m3/yr)**
Updrift Beach	Fall 1995	Spring 2001	W2-W6, W740, WHV9 WHV8, WHV7, W720	29	75,000
Updrift Beach	Spring 1995	Spring 2001	Shorelines, Stations 0.6 to 3.2	22	57,000
Downdrift Beach	Fall 1995	Spring 2001	F80	71	57,000
Downdrift Beach	Spring 1995	Spring 2001	Shorelines, Stations 46 to 46.8	-20	-16,000

Recent Engineering Events

Table 5-16 presents the dredging and fill events in the Moriches Inlet area, from stations 46 km west of Moriches to 3.2 km east of the inlet (Gravens et al., 1999). The complete list of engineering activities for the FIMP area can be found in Appendix C. The DB cell is generally dredged with placement primarily in DL and partial placement in WB and UBCH.

Table 5-16: Recent (1995 to 2001) Engineering Events in the Vicinity of Moriches Inlet					
Date	**Locality**	**Stationing (km)**		**Volume Placed or Removed (m^3)**	**Comments**
		West Boundary	**East Boundary**		
1995	Great Gun Beach	47.5	47.875	30,600	Predominantly mud; **not included** in budget calculations, 100% placed in DL (Sta. 46.8 -48.6)
Jan, Feb and Mar 1996	Moriches Inlet	NA	NA	-196,210	Moriches Inlet dredged; total 196,210 m^3 removed: 20,000 m^3 stockpiled 1.6 km east of inlet; 176,210 m^3 placed in offshore berm (below -1.5 m NGVD) approximately 1.6 km west of inlet
Jan, Feb and Mar 1996	West of Moriches Inlet	47.9	49.225	176,210	From Moriches Inlet dredging, placed in offshore berm (below -1.5 m NGVD) approximately 1.6 km west of inlet, 46% placed in WB (Sta. 48.6-49.2) and 54% in DL (Sta. 46.8 -48.6)
1996	East of Moriches	1.6	1.6	20,000	From Moriches Inlet dredging – stockpiled (see above), **not included** in budget calculations 100% placed in UBCH.
Oct 1998	Moriches Inlet	NA	NA	-142,350	Moriches Inlet dredged, 142,350 m^3 removed.
Oct 1998	West of Moriches Inlet	48.7	49	142,350	Beach fill from Moriches Inlet dredging, 100% placed in WB (Sta. 48.6-49.2)

Influence of Relative Sea Level Rise

Table 5-17 shows the estimated sediment demand (i.e., accretion) for each morphological feature at Moriches Inlet (excluding the updrift and downdrift beaches, for which the effects for sea level rise are analyzed using Bruun's Rule). As in the case of Shinnecock Inlet, sediment demand was computed as the area of each feature times the rate of sea level rise. The approximate area of each inlet feature was determined based on the polygons shown in Figure 5-22.

Table 5-17: Sediment Demand due to Relative Sea Level Rise: Moriches Inlet		
Sediment Budget Cell	**Area (x 1000 m^2)**	**Sediment Demand/ Accretion (m^3/yr)**
Updrift Lobe	1,227	4,000
Deposition Basin	98	< 1,000
Channel Throat	1,910	< 1,000
Flood Shoal	4,726	14,000
West Beach	2,240	<1,000
Downdrift Lobe	2,179	7,000
Total	8,644	25,000

Recent (1995-2001) Inlet Sediment Budget

Computed volume changes within each cell and engineering activity records were used to develop a *Recent* sediment budget for Moriches Inlet representative of the period from 1995 to 2001. A qualitative sediment budget that illustrates the assumed sediment pathways between the different inlet cells is presented in Figure 5-33. The following paragraphs present the sediment budget results for the 1995 to 2001 period for each of the inlet cells. The resulting sediment budget is shown in Figure 5-34. Sediment input to the UBCH cell (net westerly LST of 437,000 m^3/yr) is taken from the *Recent (1995-2001)* sediment budget developed and part of this study and presented in Section 6.

Updrift Beach (UBCH) [Sta. W0.6 km to W3.2 km]

The volume change rates for the UBCH cell presented in Table 5-15 indicate that this area accreted from 1995 to 2001. This is not surprising given that a large quantity of sand was placed updrift during this period as part of the Westhampton Interim Project (a total of 3.6 million m^3 or 567,000 m^3/yr). Profile data shows the magnitude of the change as 75,000 m^3/yr (based on ten profiles) and shoreline data indicate a change rate of 57,000 m^3/yr, or 63,000 m^3/yr considering losses due to sea level rise rate. Since the two estimates are very close, the average was selected (69,000 m^3/yr) as representative of the changes in this cell. Writing Equation 5.4 specifically for this cell and entering values gives,

$$Q_{W\_UBCH} - Q_{UBCH\_UL} - \Delta V_{UBCH} + P_{UBCH} - R_{UBCH} = residual$$
$$437{,}000 - Q_{UBCH\_UL} - 69{,}000 + 0 - 0 = residual = 0$$
$$\therefore\ Q_{UBCH\_UL} = 368{,}000\ \mathrm{m^3/yr}$$

That is, after impoundment in the UBCH, net westerly longshore transport into UL is 368,000 m^3/yr.

Updrift Lobe (UL)

The updrift lobe is accretional in two of the five grid comparisons and erosional in the other three (Table 5-14). Comparisons from October 1998 to July 2000, July 2000 to April 2001, and April 2001 to April 2002 show accretion rates of 48,000 m^3/yr, 210,000 m^3/yr and 25,000 m^3/yr, respectively. Erosion is observed in the May 1996 to March 1998 comparison, and the May 1996 to July 2000 comparison. From May 1996 to March 1998, it appears that the updrift lobe eroded at a rate of 26,000 m^3/yr while the deposition basin filled in (see Figure 5-29). However, it also looks as though the UL may have accreted just east of the survey coverage, extending the small accretional lobe visible at approximately the –20 foot (6 m) contour. The best coverage available is from SHOALS surveys in May 1996 and July 2000;

comparison of these two surveys yields an erosion rate of -14,000 m^3/yr. The changes from 1996 to 2000 were considered representative of the 1995 to 2001 time span based on the extent of coverage.

Writing Equation 5-4 specifically for this cell results in,

$$Q_{UBCH\_UL} - Q_{UL\_DB} - Q_{UL\_DL} - \Delta V_{UL} + P_{UL} - R_{UL} = residual$$

Model results (Section 4.3) and morphological changes in recent years suggest no significant sand transport from UL directly to the Downdrift Lobe (DL) around the seaward edge of the ebb shoal. Thus, it is assumed that $Q_{UL\_DL} \approx 0$, and entering the rest of the values proposed in preceding paragraphs gives,

$$368{,}000 - Q_{UL\_DB} - 0 - (-14{,}000) + 0 - 0 = residual = 0$$
$$\therefore\ Q_{UL\_DB} = 382{,}000\ \text{m}^3/\text{yr}$$

That is, a net westerly transport of 382,000 m^3/yr exits the UL and enters the Deposition Basin (DB).

Flood Shoal (FS)

The evolution of the FS is difficult to assess with the surveys available. The July 2000 survey is the only one with significant coverage of the flood shoal. The volume change rates in Table 5-14show accretion and erosion, but mostly indicate changes near the Channel Throat. In addition, it is often difficult to resolve vertical datums in the FS area and survey methods may be different. Previous estimates suggest that the FS is eroding at a rate of –23,000 m^3/yr (USACE/NAN, 1998). This update does not confirm or refute this number; it is within the range of values presented in Table 5-14. However, most or all of the FS erosion noted in this previous estimate, which was included in the *Historic (1979-1995)* sediment budget, was apparently due to updrift bayside shoreline erosion between the 1950's and early 1980's. After the 1980 breach and subsequent construction of a bay shoreline revetment, erosion halted. Therefore, the FS is not considered to have been eroding between 1995 and 2001 and in fact an accretion rate equal to the sediment demand due to sea level rise was assumed (14,000 m^3/yr).

Writing Equation 5-4 specifically for this cell and entering this value gives,

$$\pm Q_{FS\_CT} - \Delta V_{FS} + P_{FS} - R_{FS} = residual$$
$$-Q_{FS\_CT} - 14{,}000\ \text{m}^3/\text{yr} + 0 - 0 = 0$$
$$\therefore\ Q_{CT\_FS} = 14{,}000\ \text{m}^3/\text{yr}$$

Channel Throat (CT)

Volume change rates reported in Table 5-14 for the CT range from –79,000 m^3/yr of erosion (October 1998 to July 2000) to 80,000 m^3/yr of accumulation (July 2000 to April 2001). These numbers are thought to be variable due to difficulties in resolving the deepest parts of the channel in the SHOALS survey coverages and lack of resolution through the inlet throat in the condition surveys. The area is considered fairly stable and a volume change rate of 0 m^3/yr is therefore assigned.

Writing Equation 5-4 specifically for this cell and entering this value gives,

$$Q_{DB\_CT} - Q_{CT\_FS} - \Delta V_{CT} + P_{CT} - R_{CT} = residual$$
$$Q_{DB\_CT} - 14{,}000\ \mathrm{m^3/yr} - 0 + 0 - 0 = 0$$
$$\therefore\ Q_{DB\_CT} = 14{,}000\ \mathrm{m^3/yr}$$

West Beach (WB) [Sta. FI48.6 km to FI49.2 km]

The WB cell shows erosion for all comparisons except the July 2000 to April 2001 comparison (as explained above, there appears to be differences in the vertical datum reduction methodology between these two surveys and therefore any results based in a comparison between them are considered suspect). From May 1996 to March 1998, the WB erosion was –21,000 m^3/yr; from March 1998 to July 2000 (which spans the fill in fall of 1998), the rate of volume change was –47,000 m^3/yr. Accounting for the fill, the actual erosion rate was -96,000 m^3/yr for that period. The WB erosion rate from April 2001 to April 2002 was –19,000 m^3/yr. Assuming that the 2001-2002 erosion rate can be applied to the 2000-2002 time period, a weighted average erosion rate can be calculated as [2(–21,000 m^3/yr) + 2(–96,000 m^3/yr) + 2(–19,000 m^3/yr)]/6 years = -45,000 m^3/yr.

Fill was placed in the WB cell in early 1996, before the May 1996 SHOALS survey, and in fall of 1998. An average fill rate (P_{WB}) can be calculated as [81,060 m^3+142,350 m^3]/6 years from 1996 to 2002 = 37,000 m^3/yr. Note that the computed volume change rate (-45,000 m^3/yr) does not include the contribution of the two beach fills. Therefore, a true measure of the volumetric changes between 1996 and 2002, i.e., $\Delta V_{WB,}$ can be computed as -45,000 m^3/yr +37,000 m^3/yr = –8,000 m^3/yr.

The WB adjacent to Moriches Inlet, although also slightly erosive, appears to be more stable than the WB adjacent to Shinnecock Inlet. Nevertheless, it was assumed that the net influx of sediment into the WB from any adjacent cells is zero (Q_{IN} = 0) and that sediment eroded from this cell moves into either the Deposition Basin (DB) or the adjacent Downdrift Lobe (DL) cell. Therefore, writing Equation 5-4 for the WB gives,

$$0 - Q_{WB\_DL} - Q_{WB\_DB} - \Delta V_{WB} + P_{WB} - R_{WB} = residual$$

Similarly to Shinnecock Inlet, it is assumed that no transport occurs from the WB to DL (Q_{WB_DL} = 0). Note that if some transport from WB to DL was assumed it would only reduce the transport from WB to DB and then the transport from DB to DL. However, ultimately the total net transport influx to DL would be the same. Entering values in the balance equation gives,

$$0 - 0 - Q_{WB\_DB} - (-8{,}000) + 37{,}000 - 0 = residual = 0$$
$$\therefore\ Q_{WB\_DB} = 45{,}000\ \mathrm{m^3/yr}$$

Deposition Basin (DB)

Changes in cell DB presented in Table 5-14 appear to vary with wave climate and proximity in time to previous dredging events. The deposition basin at Moriches Inlet was dredged twice during the 1995-2001 period[4]: early spring of 1996 and in the fall of 1998 (Table 5-16). Comparison of the post-dredging survey of May 1996 with the March 1998 synthetic grid yields an accumulation rate of 88,000 m^3/yr. The 1998 dredging removed 142,350 m^3 [71,000 m^3/yr over two years]; close to the measured volume change.

[4] Moriches Inlet was also recently dredged in 2004, when approximately 190,000 m^3 were removed from the deposition basin.

From October 1998 (just after dredging) to July 2000 the accumulation rate is reduced to 47,000 m^3/yr. The difference in accumulation rate may be explained by the fact that the 1997 to 1998 season was quite active in terms of wave energy and calmer conditions existed from 1998 to 2000. An average of the two gives 68,000 m^3/yr, close to the volume change rate of 70,000 m^3/yr obtained from the July 2000 to April 2001 comparison, although there are concerns about the datum correction correspondence of those two surveys. The April 2001 to April 2002 comparison gives a change rate of –2,000 m^3/yr. This is a small amount and considering uncertainty may correspond to no change. In other words, it appears that as of 2002 the deposition basin has reached an approximate equilibrium and is probably ready to be dredged again.

The approximate rate of dredging at the inlet from May 1996 to April 2001 (while the deposition basin was still accumulating sediment) is [196,210 m^3 + 142,350 m^3]/5.08 yrs = 67,000 m^3/yr. As expected, this is roughly equal to the accumulation rate. If the inlet had continued to be dredged every two to three years this rate would be considered representative. However, the inlet was not dredged between 1986 and 1996 and after 1998 the inlet was not dredged again until 2004. An approximate dredging rate based on the total volume dredged in the *Recent* sediment budget study period averaged over the 6 years in this period was assumed: [196,210 m^3 + 142,350 m^3]/6 yrs = 56,000 m^3/yr.

$$Q_{UL\_DB} + Q_{WB\_DB} - Q_{DB\_CT} - Q_{DB\_DL} - \Delta V_{DB} + P_{DB} - R_{DB} = residual = 0$$

Entering values into this equation gives,

$$382{,}000 + 45{,}000 - 14{,}000 - Q_{DB\_DL} - 0 + 0 - 56{,}000 = residual = 0$$
$$Q_{DB\_DL} = 357{,}000\ \mathrm{m^3/yr}$$

That is, changes suggest that approximately 357,000 m^3/yr of sand are transported from the DB toward the Downdrift Lobe (DL).

Downdrift Lobe [Sta. FI46.8 km to FI48.6 km]

The volume change rates in Table 5-14 show accretion from May 1996 to March 1998, July 2000 to April 2001 and April 2001 to April 2002. The October 1998 to July 2000 and May 1996 to July 2000 comparisons both show erosion. The May 1996 to July 2000 comparison has the best coverage over the DL cell and it yields a volume change rate of -15,000 m^3/yr. However, none of the comparisons has coverage extending to the attachment point, therefore these estimates are not considered representative of the changes in 1995-2001. Fill from dredging the deposition basin was placed in the DL in early 1996, with an average placement rate of 95,150 m^3/6 years = 16,000 m^3/yr.

An examination of the successive change plots (Figure 5-29 to Figure 5-32) shows a cycle of changes in the DL. From May 1996 (just after dredging) to March 1998 (before dredging) the deposition basin fills back completely and the shallow bypassing bar characteristic of Moriches Inlet forms again across the channel and deposition basin. This relative quick change is probably due to the active wave climate over this period, particularly in 1997 to 1998. Note that the “isolated” portion of the bypassing bar that remains west of the deposition basin after the dredging in 1996 appears to be “static” between 1996 and 98. We hypothesize, however, that this feature did in fact move west toward the attachment point and simply reformed in the same place as the bypassing bar was reestablished across the deposition basin, giving the impression when looking at the surveys that this feature remained in place.

On the other hand, from October 1998 (after dredging) to July 2000, the morphological changes appear to be significantly different. The isolated western portion of the bypassing bar erodes while the deposition basin fills in and the seaward "tip" of the ebb shoal accretes and extends seaward perhaps as a result of increased flow along the channel/deposition basin. In other words, this period appears to be dominated by tidal flow induced morphological changes as well as a relatively mild wave climate that is sufficient to erode and translate the remnant western portion of the bypassing bar, but not active enough to completely restore the bypassing bar. Unfortunately, this "translation", as opposed to net erosion, cannot be conclusively proven due to the lack of complete survey coverage in 1998. Further analysis using available long-profiles, however, seems to support this hypothesis (Table 5-18). Profile based volumetric changes from the WB and DL in Fall 1998 (post-dredging/fill) and Spring 2000 suggest that while the WB area (profiles F83 and F84) eroded from Fall 1998 to Spring 2000, the attachment point continued to grow (profile F82). The erosion at Profile F81 west of the attachment point may be due to spreading of the fill placed in 1998.

Table 5-18: Profile Change Rates in DL and WB, Moriches Inlet.

Profile	Station	From	To	Volume Change Rate (m3/m/yr)
F81	47550	Spring 1998	Fall 1998	768
F81	47550	Fall 1998	Spring 2000	-275
F82	48000	Spring 1998	Fall 1998	-4
F82	48000	Fall 1998	Spring 2000	66
F83	48750	Spring 1998	Fall 1998	86
F83	48750	Fall 1998	Spring 2000	-183
F84	49150	Spring 1998	Fall 1998	117
F84	49150	Fall 1998	Spring 2000	-15

From July 2000 to April 2001, the bypassing bar appears to be substantially restored, and the seaward extension of the ebb shoal is partially reversed (erosion). Finally, from April 2001 to April 2002, the bypassing bar is fully restored to its original location prior to dredging in 1998. The seaward portion of the ebb shoal has also retreated north to a position also similar to that in the Spring 1998.

As explained above, there are not enough hydrographic surveys with sufficient coverage to yield a reliable estimate of the volume changes in this cell during the 1995 to 2001 period. However, available data does suggest that the downdrift lobe is fairly stable in terms of net volume although with significant morphological variability as a result of dredging events and changes in the wave climate. Therefore, a relatively small accretion rate of 7,000 m^3/yr based on sediment demand estimates due to sea level rise was assumed as representative of this period (see Table 5-17).

Based on this assumption, the balance equation for this cell is as follows,

$$Q_{DB\_DL} + Q_{WB\_DL} + Q_{UL\_DL} - Q_{DL\_DBCH} - \Delta V_{DL} + P_{DL} - R_{DL} = residual = 0$$
$$357{,}000 + 0 + 0 - Q_{DL\_DBCH} - 7{,}000 + 16{,}000 - 0 = 0$$
$$\therefore\ Q_{DL\_DBCH} = 366{,}000\ \mathrm{m^3/yr}$$

That is, the net westerly transport from the DL toward the Downdrift Beach (DBCH) is 366,000 m^3/yr.

Therefore, the total amount of sediment bypassing the inlet system, which includes the channel, deposition basin, shoals and west beach, is then 366,000 m^3/yr (99% of the westerly influx of 368,000 m^3/yr). In other words, the *Recent (1995-2001)* sediment budget suggests that the inlet bypasses most or all of the material entering the Updrift Lobe. This is also consistent with recent findings by Allen et al (2002), who analyzed a series of recent hydrographic surveys of the inlet in 1995, 1996 and 1999[5] and concluded that the volume of the ebb tidal delta at Moriches Inlet *"remains reasonably constant"* and that *"the ebb-tidally dominated inlet bypasses sediment in discrete morphologic units of shoals moving downdrift along the outer bar …"*.

Downdrift Beach [Sta. FI46 km to FI46.8 km]

Volume change rates in the DBCH calculated with shoreline and profile data (only one short profile is available) are presented in Table 5-15. Shoreline data show some erosion in this reach (-16,000 m^3/yr) and profile data suggests significant accretion (57,000 m^3/yr), although this estimate is based on only one profile. Losses due to sea level rise in this 800 m reach account for less than 1,000 m^3/yr.

Given that only one short profile is available, a representative change rate was computed by averaging the shoreline and profile data. This approach yields an accretion rate of 21,000 m^3/yr for this reach. Writing the balance equation for this cell and entering these values gives,

$$Q_{DL\_DBCH} - Q_{DBCH\_West} - \Delta V_{DBCH} + P_{DBCH} - R_{DBCH} = residual = 0$$
$$366{,}000 - Q_{DBCH\_West} - 21{,}000 + 0{,}000 - 0 = 0$$
$$\therefore\ Q_{DBCH\_West} = 345{,}000\ \mathrm{m}^3/\mathrm{yr}$$

Summary of Results for the Recent (1995-2001) Sediment Budget

Relevant conclusions from the *Recent (1995-2001)* sediment budget at Moriches Inlet are as follows:

- The Updrift Beach accreted significantly (69,000 m^3/yr) over the 1995-01 period, most likely as a result of increased sediment supply from the east; a total of 3.6 million m^3 or 567,000 m^3/yr were placed updrift as part of the Westhampton Interim Project during this period.
- The ebb shoal (including the Updrift and Downdrift lobes) was very dynamic and slightly erosive during the 1995 to 2001 period (net change of -7,000 m^3/yr). The morphology of the Downdrift Lobe in particular appears to be very sensitive to dredging of the channel and deposition basin.
- The West Beach adjacent to Moriches Inlet, although also slightly erosive, was more stable than the WB adjacent to Shinnecock Inlet during this period.
- As in the case of Shinnecock Inlet, it was difficult to assess sediment transport pathways and quantities in the Flood Shoal complex with the available data. Therefore, it was assumed that the flood shoal is relatively stable and only accreting enough sediment to offset the effects of sea level rise.
- During the 1995 to 2001 period the inlet system, which includes the channel, deposition basin, shoals and west beach, bypassed roughly 99% of the net easterly influx of sediment to the inlet. This finding is also consistent with the conclusions of recent investigations by Allen et al (2002).

<u>*Existing (c. 2001) Condition Sediment Budget*</u>

An *Existing (c. 2001)* condition sediment budget for Moriches Inlet which also incorporates the *Existing (c. 2001)* regional sediment budget results (see Section 6) is presented in Figure 5-35. Note that the

[5] Note that unfortunately these surveys were not readily available at the time this study was completed.

Existing (c. 2001) regional sediment budget suggests a net westerly transport rate of 267,000 m^3/yr entering the Updrift Beach (instead of 437,000 m^3/yr in the *Recent* sediment budget).

Other specific assumptions for the *Existing (c. 2001)* Moriches Inlet sediment budget are as follows:

- Construction of the Westhampton Interim Project has apparently caused significant accumulation of sediment in the Updrift Beach cell (69,000 m^3/yr in the 1995 to 2001 period). However, it is assumed that this accretion rate, which equates to 27 m^3/m/yr (volume change) or 2.3 m/yr (shoreline change), is not sustainable in the medium- to long-term. The previous *Existing (c. 1999)* sediment assumed that this cell would accrete at rate of 29,000 m^3/yr, including losses to sea level rise. This estimate is still considered reasonable and representative of *Existing* conditions. Accretion caused by the initial construction of the Westhampton Interim project (2.9 million m^3) and the first renourishment (723,000 m^3) should taper off with time as only subsequent renourishment volumes are placed updrift and the capacity of this cell to accumulate sediment is reduced as it fills up.
- It was assumed that the recent erosion measured on the Updrift Lobe would not continue and that this feature is relatively stable, with only slight accumulation due to sea level rise, ΔV_{UL} = 4,000 m^3/yr.
- A similar assumption was made for the Flood Shoal (ΔV_{FS} =14,000 m^3/yr) and the Downdrift Lobe (ΔV_{DL} = 7,000 m^3/yr). In other words, inlet shoals are considered relatively stable in the long-term. It was assumed that fairly efficient bypassing measured during the 1995 to 2001 period would continue under *Existing* conditions. Note that ebb shoal changes between 1981 (i.e., before construction of the new channel and deposition basin in 1986) and 2000 suggest that the ebb shoal accumulated approximately 16,000 m^3/yr over that 19 year period. Under the assumptions stated above, the ebb shoal (Updrift and downdrift lobes) would accumulate 11,000 m^3/yr. This slight reduction seems reasonable considering the shoal appears to have largely adjusted to the new channel and deposition basin built in 1986.
- The West Beach was also assumed to be relatively stable. Accumulation due to sea level rise in this cell will be less than 1,000 m^3/yr and therefore it was not accounted for in the budget.
- Continued dredging of the Deposition Basin at the recent rate of 56,000 m^3/yr was assumed as representative of *Existing* conditions. Note, however, that the dredging rate will obviously depend on the dredging interval. Available surveys in the 1990's suggest that the deposition basin "fills up" within two to three years of dredging. On the other hand, after the dredging in 1986 and 1998 it took 10 and 6 years, respectively, to dredge the inlet again.
- Also note that for the purposes of this budget it was assumed that most of this material (40,000 m^3/yr) dredged from the inlet would continue to be placed along the West Beach although this sand is not necessarily required to maintain a stable shoreline in that area. In any event, if the material were placed farther west within the Downdrift Lobe cell or the beaches beyond to the west, the net impact of Moriches Inlet on the sediment budget would not change. Finally, accumulation due to sea level rise in this cell will be less than 1,000 m^3/yr and therefore it was not accounted for in the budget.
- Stable Channel Throat. Accumulation due to sea level rise is less than 1,000 m^3/yr and therefore not accounted for in the sediment budget.
- The computed accretion rate of 21,000 m^3/yr within the Downdrift Beach during 1995-2001 period does not seem sustainable in the medium- to long-term. On the other hand, erosion is not likely either since Moriches Inlet appears to be effectively bypassing most or all of the net westerly longshore sediment transport arriving at the inlet. Therefore a relatively stable shoreline with accumulation only due to sea level rise (2,000 m^3/yr) was assumed as representative of *Existing* conditions.

Under these assumptions, approximately 89% (213,000 m^3/yr) of the net westerly transport under *Existing (c. 2001) Conditions* bypasses the inlet system, which includes the channel, deposition basin, shoals and west beach. The other 11% (25,000 m^3/yr) is assumed to accumulate within the ebb and flood shoals as a result of sea level rise. Therefore, this new *Existing* condition budget suggests that overall net impact of Moriches Inlet to the regional sediment budget is reduced as compared to the conclusions presented in the previous work by USACE-NAN (1998) and adopted by Gravens et al. (1999) which suggested that 69,000 m^3/yr accumulate within the inlet. Apparently the ebb shoal has adjusted to the changes implemented in 1986 and it has reached a relatively stable volume (apart from sea level rise effects) since its opening by a storm in 1931 and stabilization with jetties in 1953.

5.3 Fire Island Inlet

5.3.1 Existing Condition

Data at Fire Island Inlet is sparse, with only two hydrographic surveys available for recent years and limited recent tidal data available for the area. This section describes the two recent bathymetric surveys and changes in inlet morphology from survey to survey. Hydraulic and inlet stability analysis are conducted following methods outlined in section 5.1.1 to determine theoretical cross-sectional area, ebb shoal capacity and to identify trends toward closure or scour.

Bathymetric Records

At Fire Island Inlet, one SHOALS survey is available on May 1996. Additional survey data obtained using traditional hydrographic survey methods is available on March 2001, March 2002, and Dec 2001 to March 2002 (MSRC-SUNY Survey). Figures showing these surveys are included in Appendix A.

Hydraulic Analysis

Tidal inlet prism and morphology relationships are applied to Fire Island Inlet. The analysis follows methods outlined in section 5.1.1 and is designed to assess the theoretical equilibrium state of the inlet. Current measured conditions at the inlet are compared with the theoretical equilibrium values to qualitatively assess probable future conditions.

Measured Conditions

Recent minimum cross-sectional areas, approximate average depths and estimated ebb shoal volumes are presented in Table 5-19. Minimum cross-sectional areas are measured using the surveys presented in Appendix A. It is noted that the cross-sectional area from 1996 shown in Table 5-19 may be underestimated due to lack of survey coverage in the deep water of the channel. Average depth is obtained by dividing the cross-sectional areas by the approximate inlet channel width of 1,600 feet in 1996 and 2,000 feet in 2001. Ebb shoal volume was estimated using cumulative measured changes in bathymetric survey data. For ebb shoal evolution from 1924 to 1996 see USACE-NAN (1998).

Table 5-19: Fire Island Inlet Existing Hydraulic Characteristics

Year	Measured Minimum Cross-Sectional Area (sq. ft)	Average Depth (ft)	Estimated Ebb Shoal Volume (cy x 10^6)
1996	22,600	14.3	41
Dredging, 1997, 1,081,861 cy removed			
Dredging, 1999 to 2000, 1,107,718 cy removed			
2001	51,500	26.1	42
Dredging Oct 2001 to Feb 2002, 1,490,784 cy removed			

Tidal Prism - Theoretical Conditions

Theoretical equilibrium conditions for the existing tidal prism at Fire Island Inlet are presented in this section. The inlet morphology-tidal prism relationships used required tide range estimates in Great South Bay. Tide range data from NOAA predictions (NOAA 2002) may be flawed; secondary stations in Great South Bay are based on observations primarily from the 1930s and 1950s (Tide.Predictions@noaa.gov, personal communications, June 2002). Recent monitoring at the Fire Island Coast Guard Station as part of the LI Shore program gives tidal range of 1.4 feet. As shown in Table 5-12, tide range observed at the Smith Point Bridge gauge (between Moriches and Great South Bays) is 1.2 feet. Data in Great South Bay reported by NOAA averages 0.75 feet. Averaging these three data sources yields a value for tidal range in Great South Bay of approximately 1 foot.

Using the average tidal range in Great South Bay and the approximate surface area, 110 square miles, Equation 5-1 estimates a Great South Bay tidal prism of 3070 x 10^6 ft^3.

Theoretical minimum cross-sectional area is calculated using the tidal prism estimate of 3070 x 10^6 ft^3 and Equation 5-2 to obtain 70,800 ft^2 for Fire Island Inlet.

Equation 5-3 and the tidal prism value, 3070 x 10^6 ft^3, result in a theoretical ebb shoal capacity at Fire Island Inlet of 49 x 10^6 cy.

These theoretical values can be compared with the measured quantities presented in Table 5-19. Direct comparison assumes that tidal prism has not changed from 1996 to 2001 and assumes that the tidal amplitude values reported by NOAA are valid for Great South Bay. In reality, the tidal prism may vary and could be numerically modeled for a more accurate representation. Continuing work will attempt to verify the water levels used to calculate the theoretical parameters for Fire Island Inlet.

In 1996, Fire Island Inlet was characterized by a cross-section smaller than all other reported values since inlet formation (Moffatt & Nichol Engineers and URS Consultants 1998). This may be the result of a lack of survey coverage through the inlet channel, resulting in an underprediction of cross-sectional area. The 1996 value was just 32% of the theoretical equilibrium cross-section. After dredging events in 1997 and 1999-2000, the minimum inlet cross-section increased to 73% of the theoretical equilibrium value. Estimates of the ebb shoal volume in 1996 indicate that the inlet was at about 84% of the theoretical equilibrium capacity.

<u>*Inlet Stability Analysis*</u>

To further investigate the existing condition at Fire Island Inlet, stability analysis using Escoffier's method (Escoffier 1940) is applied to existing and historic conditions. Results are presented in Figure 5-36. Inlet cross-sectional areas are as shown in Table 5-19. Great South Bay surface area of 110 square miles and ocean tidal amplitude of 2.05 feet are considered for Fire Island Inlet. A friction coefficient of 0.0035 is chosen and inlet length is approximated as 20,000 feet. See Appendix B for calculation details.

The closure curves for Fire Island Inlet presented in Figure 5-36 indicate that the inlet has been marginally stable for recent years. The two different curves reflect varying inlet widths in 1996 and 2001, 1,600 feet and 2,000 feet, respectively. Marginal stability means that the inlet is stable with a tendency to shoal and may close if events cause accumulation of sediment and decrease cross-sectional area. This marginal stability is indicated by the relative position of the measured inlet cross-sectional areas relative to the closure curves. The measured cross-sections are location just inside of the equilibrium interval, characterized by marginally stable cross-sections of 18,000 to 20,000 ft^2. Evaluation of Fire Island Inlet stability is difficult because dredging operations have been performed on a nearly annual basis. This dredging has artificially increased inlet cross-sections, which makes interpretation of stability inconclusive. The 1996 estimate of cross-sectional area may be an underestimate due to lack of survey coverage in the deep water of the channel. Presently, Fire Island Inlet is characterized by a cross-section that is larger than all other historical values (see Moffatt & Nichol Engineers and URS Consultants, 1998). This is likely a result of dredging operations.

<u>Recent Volume Changes (1995-2001)</u>

Seven sediment budget cells were delineated to represent Fire Island Inlet and the adjacent beaches. These sediment budget cells are presented in Figure 5-37. The Updrift Beach (UBCH) extends from just east of the inlet about 3,700 m to the traffic circle at Robert Moses State Park (RMSP), corresponding to CHL stationing 0.075 km to 3.8 km. The Sand Spit (SS) extends from the jetty across the spit about 800 m. The Deposition Basin (DB) is just west of the SS and is as defined by the USACE for maintenance of the navigation channel. Two cells comprise the roughly 5,000 m long and 1,500 m wide channel formed by the overlapping western and eastern ends of Fire Island and Jones Island, respectively. One cell, named Channel (CH), follows the navigation channel along the north shoreline of Fire Island. The second, named Oak Beach (OB), covers the relatively shallow water fronting the eastern end of Jones Island, including Gilgo Beach and Captree State Park. Technically, the extensive shoals located immediately east of this channel also form part of the Fire Island Inlet system. However, there is little or no data in this area. Moreover, it is uncertain whether these shoals continue to be directly affected by inlet processes (e.g., accreting through influx of littoral sediments carried by flood currents) or whether they are relict, relatively static, features associated with historic inlet migration from east to west. Therefore, a cell over this area was not considered for the sediment budget update. Note that previous sediment budgets for Fire Island Inlet did not consider changes to shoals either. The Ebb Shoal (ES) cell extends westward from the DB, north to Cedar Island Beach along Cedar Island about 4,000 m and offshore to approximately the -9 m (–30 foot) contour. The Downdrift Beach (DBCH) reaches westward of the ES about 3,700 m.

Although there has been frequent dredging at Fire Island Inlet, complete survey coverage of the entire inlet complex, including the ebb and flood shoals, is sparse. Primary source data for volume change calculations at Fire Island Inlet consist of a May 1996 SHOALS survey and a 2001-02 single- and multi-beam survey conducted by SUNY. All data sources available for Fire Island Inlet are listed in Appendix B. Two synthetic grids were created based on the 1996 and 2001-2002 surveys; the grids and data sources are detailed in Table 5-20. Because of uncertainty associated with triangulation of sparse SHOALS and profile data across the inlet throat and across the ebb shoal, a grid created with just the May 1996 SHOALS survey was also used for volume change comparisons. Plots of each of the topography/bathymetry grids used for comparisons are presented in Figure 5-38 and Figure 5-39 [1996 SHOALS and 2001 synthetic]. Figure 5-40 shows the grid comparison.

The grids were used to compute volume changes in sediment budget cells DB, ES, OB, and CH. Table 5-21 shows the computed grid volume changes and volume change rates. UBCH and DBCH extend farther than survey coverage, therefore shoreline and profile data were used to compute volume change rates in those cells. Volume change rates for UBCH and DBCH are shown in Table 5-22.

Finally, recent changes in the Sand Spit cell were estimated based on synthetic grids generated using a 1995 topography dataset with sub-aerial elevation data for this area, the May 1996 SHOALS bathymetry, the 2001 bathymetry, and a 2001 LIDAR topography dataset, also with sub-aerial elevation data for the spit. Gaps in coverage between the topography and bathymetry data were filled using GIS software and assumptions regarding the morphology of the spit supported by analysis of available aerial photos. The various datasets as well as the synthetic grids are shown in Figure 5-41 and Figure 5-42. Figure 5-43 shows the changes from 1995/96 to 2001 within this cell. As shown in Table 5-21, there is significant accumulation within the spit cell (75,000 m^3/yr) during the analysis period. Most of the accumulation appears to have taken place as the spit grew westward toward the eastern edge of the channel/deposition basin. In fact, as of 2001, the sub-aerial part of the spit extended right up against the channel. Therefore, as long as maintenance dredging continues it appears unlikely that the spit will continue to accumulate sediment at this rate (see *Existing (c. 2001)* sediment budget discussion).

Table 5-20: Synthetic Grids and Source Data for Fire Island Inlet Sediment Budget

Synthetic Grid		Data Source 1		Data Source 2		Data Source 3	
Fire Island Inlet May 1996 SHOALS		SHOALS Survey, 24-26 May 1996					
Filename	fi1996shshft	*Datum, Conversion to NGVD*	Shift of –3.5 ft applied as above				
Fire Island Inlet 2001		Condition Survey, 15-20 Mar 2001		Multi-Beam Hydro Survey (Roger Flood SUNY Survey), Dec 2001		ACNYMP Profile Survey, March-April 2001 JI10,12-16,18; F1-5	
Filename	fi2001synthgd	*Datum, Conversion to NGVD*	MLW 2 ft below NGVD	*Datum, Conversion to NGVD*	MLLW 0.41 ft below NGVD bayside; MLLW 1.71 ft below NGVD oceanside	*Datum, Conversion to NGVD*	NGVD

Table 5-21: Volumetric Changes from Grid Comparisons: Fire Island Inlet

Inlet Segment	From	To	Cut (m3)	Fill (m3)	Net (m3)	Net/yr (m3/yr)	Grids
Sand Spit	95/96	2001	306,372	628,356	421,799	75,000	Synthetic 95/96-Synthetic 01
Deposition Basin	May-96	March-01	-209,315	173,146	-36,169	-8,000	SHOALS 96-Synthetic 01
Channel	May-96	March-01	-116,215	23,933	-92,282	-19,000	SHOALS 96-Synthetic 01
Oak Beach	May-96	March-01	-284,001	515,496	231,496	48,000	SHOALS 96-Synthetic 01
Ebb Shoal	May-96	March-01	-1,556,380	1,884,674	328,294	68,000	SHOALS 96-Synthetic 01

Table 5-22: Volumetric Changes from Shoreline and Profile Comparisons: Fire Island Inlet

Inlet Segment	From	To		Volume Change Rate (m3/m/yr)	Net/yr (m3/yr)
Updrift Beach	Fall 1995	Spring 2001	F2-F5	-9	-34,000
Updrift Beach	Spring 1995	Spring 2001	Shorelines, Stations 0.075 to 3.8	-1	-4,000
Downdrift Beach	Fall 1995	Spring 2001	JI18, JI19, JI21	14	51,000
Downdrift Beach	Spring 1995	Spring 2001	Shorelines, Gilgo Stations 4.4 to 8.025	-1	-4,000

Recent Engineering Events

Fire Island Inlet is dredged on a fairly regular basis, as shown in Table 5-23, which lists dredging and fill events from Gilgo Beach to Robert Moses State Park from 1995 to 2002. A list of all engineering activities in the FIMP area for that time span is presented in Appendix C.

Table 5-23: Recent (1995 to 2001) Engineering Events in the Vicinity of Fire Island Inlet

Date	Locality	Stationing (km)		Volume Placed or Removed (m^3)	Comments
		West Boundary	East Boundary		
1997	Fire Island Inlet	NA	NA	-827,140	Fire Island Inlet dredged, 827,140 m^3 removed, 549,650 m3 to Gilgo and 277,490 m3 to Robert Moses State Park (RMSP)
1997	Robert Moses State Park	0.6	3.8	277,490	Fill from Fire Island Inlet dredging
1999 to 2000	Fire Island Inlet	NA	NA	-846,910	Fire Island Inlet dredged, 846,910 m^3 removed, 743,400 m3 to Gilgo and 103,510 m3 to RMSP
1999 to 2000	Robert Moses State Park	0.6	3.8	103,510	Fill from Fire Island Inlet dredging
Oct 2001 to Feb 2002	Fire Island Inlet	NA	NA	1,139,780	Fire Island Inlet dredged, 1,139,780 m^3 removed, 1,013,790 m3 to Gilgo and 125,990 m3 to RMSP
Oct 2001 to Feb 2002	Robert Moses State Park	0.6	3.8	125,990	Fill from Fire Island Inlet dredging

Influence of Relative Sea Level Rise

Table 5-24 shows the estimated sediment demand (i.e., accretion) for each morphological feature at Fire Island Inlet (excluding the updrift and downdrift beaches, for which the effects for sea level rise are analyzed using Bruun's Rule). As in the case of Shinnecock and Moriches Inlet, sediment demand was computed as the area of each feature times the rate of sea level rise. The approximate area of each inlet feature was determined based on the polygons shown in Figure 5-37. Note that for reasons explained above, the shoals east of Captree State Park were excluded from this analysis.

Table 5-24: Sediment Demand due to Relative Sea Level Rise: Fire Island Inlet

Sediment Budget Cell	Area (x 1000 m^2)	Sediment Demand/ Accretion (m^3/yr)
Ebb Shoal	7,399	23,000
Deposition Basin	4,450	1,000
Oak Beach	4,945	15,000
Channel	2,903	9,000
Total	24,982	48,000

Recent (1995-2001) Inlet Sediment Budget

Computed volume changes within each cell and engineering activity records were used to develop a sediment budget for Fire Island Inlet. A conceptual sediment budget that illustrates the assumed sediment pathways between the different inlet cells is presented in Figure 5-44. The following paragraphs present the sediment budget results for the 1995 to 2001 period for each of the inlet cells. The resulting sediment budget is shown in Figure 5-45. Sediment input to the UBCH cell (net westerly LST of 318,000 m^3/yr) is taken from the *Recent (1995-2001)* sediment budget developed and part of this study and presented in Section 6.

Updrift Beach (UBCH) [Sta. FI0.075 km to FI3.8 km]

Volume change rates based on both profile and shoreline change data (Table 5-22) indicate that the UBCH cell eroded during the 1995 to 2001 period, although profile data suggest greater erosion (-34,000 m^3/yr) than shoreline changes (-4,000 m^3/yr). In fact, considering SLR, approximately 9,000 m^3/yr, the net volume change in this cell would be +5,000 m^3/yr. Since less than one profile per km of shoreline was available in this cell, an average of the net change rate based on shoreline data and based on profile data (i.e., -14,000 m^3/yr) was assumed as representative of the changes in the UBCH cell during the 1995-2001 period.

In addition, part of the material dredged from the Fire Island Inlet channel and deposition basin is typically placed in the UBCH cell. In recent years (1997 to 2002) fill has been placed within this cell at a rate of roughly 62,000 m^3/yr. It was also assumed that there is no direct sediment exchange between the UBCH cell and the Ebb Shoal (ES) or the Deposition Basin (DB), only between the UBCH cell and the Sand Spit (SS). Applying the sediment balance equation this cell and entering values gives,

$$Q_{W\_UBCH} - Q_{UBCH\_SS} - \Delta V_{UBCH} + P_{UBCH} - R_{UBCH} = residual$$
$$318{,}000 - Q_{UBCH\_SS} - (-14{,}000) + 62{,}000 - 0 = residual = 0$$
$$\therefore\ Q_{UBCH\_SS} = 394{,}000\ \mathrm{m^3/yr}$$

Therefore, the estimated net westerly longshore transport rate around the Democrat Point jetty and into the Sand Spit cell during the 1995-2001 period is 394,000 m^3/yr.

Sand Spit (SS)

As explained above, a comparison of two synthetic grids built using available topography and bathymetry datasets in 1995, 1996, and 2001 suggest that the spit accumulated sand at a rate of 75,000 m^3/yr during this period. Applying the sediment balance equation to this cell and entering values gives,

$$Q_{UBCH\_SS} - Q_{SS\_DB} - \Delta V_{SS} + P_{SS} - R_{SS} = residual$$
$$394{,}000 - Q_{SS\_DB} - 75{,}000 + 0 - 0 = residual = 0$$
$$\therefore\ Q_{SS\_DB} = 319{,}000\ \mathrm{m^3/yr}$$

That is, the resulting sediment transport westward from the SS to the DB is estimated at 319,000 m^3/yr.

Channel (CH) and Oak Beach (OB)

The volume change rates computed from the SHOALS 1996 and synthetic 2001 grids (Table 5-21) indicate some accumulation in the OB cell and some erosion/scour in the CH. There is, however, sparse coverage of these two cells for the comparison time period. The comparison shown in Figure 5-40 indicates some accretion east of the “Thumb”, but changes near the beach are not captured by the available survey coverage. Given the lack of data it was assumed that these two cells are dynamically stable and only accumulate material at a rate sufficient to meet the demand induced by sea level rise.

Applying the sediment balance equation to these two cells, which were combined into one for the purposes for the sediment budget, and entering values gives,

$$Q_{DB\_OBCH} - \Delta V_{OBCH} + P_{OBCH} - R_{OBCH} = residual = 0$$
$$Q_{DB\_OBCH} - 24{,}000 + 0 - 0 = residual = 0$$
$$\therefore\ Q_{DB\_OBCH} = 24{,}000\ \text{m}^3/\text{yr}$$

Deposition Basin (DB)

Bathymetric comparisons show very little net change within the DB (–8,000 m^3/yr, see Table 5-21) between 1996 and 2001. This is probably because both the May 1996 SHOALS and March 2001 condition surveys were conducted about the same length of time after dredging events in 1994-1995 and 1999-2000. From May 1996 to March 2002 a total of 2,813,830 m^3 of material was removed[6], a rate of approximately 375,000 m^3/yr. Applying the sediment balance equation to this cell and entering values gives,

$$Q_{SS\_DB} - Q_{DB\_ES} - Q_{DB\_OBCH} - \Delta V_{DB} + P_{DB} - R_{DB} = residual = 0$$
$$319{,}000 - Q_{DB\_ES} - 24{,}000 - (-8{,}000) + 0 - 375{,}000 = residual = 0$$
$$\therefore\ Q_{DB\_ES} = -72{,}000\ \text{m}^3/\text{yr}$$

In other words, based on the assumed updrift influx of sediment to the inlet (319,000 m^3/yr) sediment balance at the Deposition Basin for this period requires an influx of 72,000 m^3/yr from the ebb shoal cell to balance the DB cell sediment budget. A possible explanation for this somewhat counterintuitive result is that sediment transport reversals (i.e., transport from west to east) combined with placement of large quantities of material dredged from the Deposition Basin on the downdrift beach but relatively close to the ebb shoal may be causing a significant influx of sediments from the west into the DB. Alternatively, the influx of sediment from the east due to net longshore transport may be larger that the estimate developed as part of the *Recent (1995-2001)* sediment budget, either due to inaccuracies in the estimates of volume changes (i.e., more erosion occurred during this period) or due the additional influx of sediment from an offshore source (see Section 6). More sediment from the east would require less sediment from the west to balance the DB budget.

Ebb Shoal (ES)

According to volume changes from 1996 to 2001 (see Table 5-21) the ebb shoal cell accreted at a rate of 68,000 m^3/yr during this period. It appears from the comparison shown in Figure 5-40 that there is a significant amount of scour along the natural channel west of the “Thumb”. It also seems like sand has shifted towards the west over the middle part of the ES. It should be noted that the 1996 SHOALS survey does not cover the Cedar Beach shoal. Therefore, changes in this area are not included in the accretion rate of 68,000 m^3/yr.

This accumulation rate is significantly smaller that previously reported by M&N in USACE-NAN (1998). Therefore, changes of the ES were further investigated to confirm these new findings. Specifically, available profile surveys at stations JI12, JI13, JI15 and JI16 were examined in detail. Results are summarized in Table 5-25. From 1995 to 1998, significant erosion was observed at JI12 (the only profile across the channel west of the “Thumb” with long profile data). On the other hand, moderate accretion was observed from 1998 to 2001. There are no data available for JI13 from 1995 to 1998, but accretion

[6] Fire Island Inlet was recently dredged in 2004 (831,000 m^3)

from 1998 to 2001 is similar to the JI12 value. Station JI15 just west of the attachment point at Cedar Beach shows accumulation from 1995 to 1998 and slight erosion from 1998 to 2001. JI16 at the western edge of the DL shows some erosion from 1995 to 2001.

Table 5-25: Profile Change Rates in ES Cell, Fire Island Inlet

Profile	From	To	Volume Change Rate (m3/m/yr)
JI12	October-95	July-98	-750
JI12	July-98	January-01	180
JI13	October-98	January-01	160
JI15	October-95	July-98	390
JI15	July-98	January-01	-10
JI16	April-95	January-01	-60

It is assumed that while additional accumulation may be taking place near the attachment point (just east of JI15), the volume change rate of 68,000 m^3/yr is representative of the accumulation in the ES. The real change rate may be slightly greater but is not thought to be as large as previous estimates have indicated (USACE-NAN, 1998). Apparently, this previous estimate was partly based on interpolation of available beach profile surveys which may have overestimated the western extent of the ebb shoal in 1996 and resulted sediment "apparent" accumulation over this area that was not real.

Applying the sediment balance equation to this cell and entering values gives,

$$Q_{DB\_ES} - Q_{ES\_DBCH} - \Delta V_{ES} + P_{ES} - R_{ES} = residual = 0$$
$$-72{,}000 - Q_{ES\_DBCH} - 68{,}000 + 0 + -0 = residual = 0$$
$$\therefore\ Q_{ES\_DBCH} = -140{,}000\ \mathrm{m}^3/\mathrm{yr}$$

That is, unless additional material entered the inlet from the east during this period, a balanced sediment budget suggests that that approximately 140,000 m^3/yr are transported into the Ebb Shoal from the west as net easterly directed sediment transport. As explained above, this somewhat counterintuitive result may be due to inaccuracies in the *Recent (1995-2001)* regional sediment budget, an offshore sediment supply, or a real sediment transport reversal in this area as a result of significant volumes of sand being placed in the DBCH cell (see below).

Downdrift Beach [Extends approximately 3.6 km](DBCH)

Volume changes based on shoreline data indicate mild erosion (-4,000 m^3/yr) while profile data suggest accretion (51,000 m^3/yr) (Table 5-22) within this cell. Incorporating losses due to sea level rise into the volume changes from shoreline data suggests net accumulation within DBCH of 4,000 m^3/yr. Since less than one profile per km of shoreline was available in this cell, an average of the net change rate based on shoreline data and based on profile data (i.e., 28,000 m^3/yr) was assumed as representative of the changes in the UBCH cell during the 1995-2001 period.

In addition, most of the sand dredged from the Fire Island Inlet channel and deposition basin, 313,000 m^3/yr, is placed in this cell. Applying the sediment balance equation to this cell and entering values gives,

$$Q_{ES\_DBCH} - Q_{DBCH\_West} - \Delta V_{DBCH} + P_{DBCH} - R_{DBCH} = residual = 0$$
$$-140{,}000 - Q_{DBCH\_West} - 28{,}000 + 313{,}000 + -0 = residual = 0$$
$$\therefore\ Q_{DBCH\_West} = 145{,}000\ \text{m}^3/\text{yr}$$

That is, unless additional material entered the inlet from the east during this period, there are only 145,000 m^3/yr of net westerly transport at the western boundary of the DBCH cell.

Summary of Results for the Recent (1995-2001) Sediment Budget

Relevant conclusions from the *Recent (1995-2001)* sediment budget at Fire Island Inlet are as follows:

- Although there is significant uncertainty based on the available data, it appears the Updrift Beach eroded slightly (14,000 m^3/yr) over the 1995-01 period. Even if accurate of the 1995-2001 period, this value is considered atypical of this cell, which is appears to be fairly stable. Material dredged from the Fire Island Inlet was placed in this cell at a rate of roughly 62,000 m^3/yr.
- A synthetic compilation of available topography and bathymetry datasets in 1995, 1996, and 2001 suggest that the spit accumulated 75,000 m^3/yr during this period.
- Changes in the Channel (CH) and Oak Beach (OB) cells are very difficult to assess based on the available data. Therefore, it was assumed that these two cells only accumulate material at a rate sufficient to meet the demand induced by sea level rise (24,000 m^3/yr in total for the two cells).
- The Deposition Basin (DB) was dredged roughly every two years at a rate of 375,000 m^3/yr during the *Recent* sediment budget study period. The net volume change was -8,000 m^3/yr.
- Limited survey coverage in 1996 and 2001 suggest that the ebb shoal cell accreted at a rate of 68,000 m^3/yr during this period. Although the coverage does not include a large portion of the ebb shoal fronting Cedar Beach, available profile surveys in that area suggest that overall accumulation in the ebb shoal is not much greater than the estimate above. More importantly, the data does not seem to support the previous estimate by M&N (USACE-NAN, 1998) of 535,000 m^3/yr, which was adopted in both the *Historic (1979-1995)* and *Existing (c. 1999)* regional sediment budgets by Gravens et al. (1999).
- Dredging rates in the Deposition Basin, accumulation in the ebb shoal, and losses due to sea level rise suggest that the net amount of sediment entering the inlet through Democrat Point from the east during the 1995-2001 period was not enough to balance the budget. Unless an additional influx of sediment from the east (due to increased erosion or an offshore source of sediment) is assumed, 140,000 m^3/yr entering the Ebb Shoal cell from the west are required to balance the budget. Note that this latter scenario may be plausible given that approximately 313,000 m^3/yr are placed very close to the shoal on the Downdrift Beach cell (see below), and some of this material may be flowing back into the ebb shoal. Also note that this issue is different than the problems previous investigators had trying to match up historic spit growth or updrift fillet accumulation with incoming LST estimates based on volume changes along Fire Island to Montauk Point. In that case, a net sediment transport reversal would not contribute to westerly spit growth or accumulation within the updrift fillet after construction of the Democrat Point breakwater in the 1940's.
- Accretion at a rate of 28,000 m^3/yr is considered representative of the changes in the DBCH cell during the 1995-2001 period. Approximately 313,000 m^3/yr was placed in this cell during this period, resulting in a net westerly transport at the western end of the FIMP area of 145,000 m^3/yr.

<u>*Existing (c. 2001) Condition Sediment Budget*</u>

An *Existing (c. 2001)* condition sediment budget for Fire Island Inlet which also incorporates the *Existing (c. 2001)* regional sediment budget results (see Section 6) is presented in Figure 5-46. Note that the

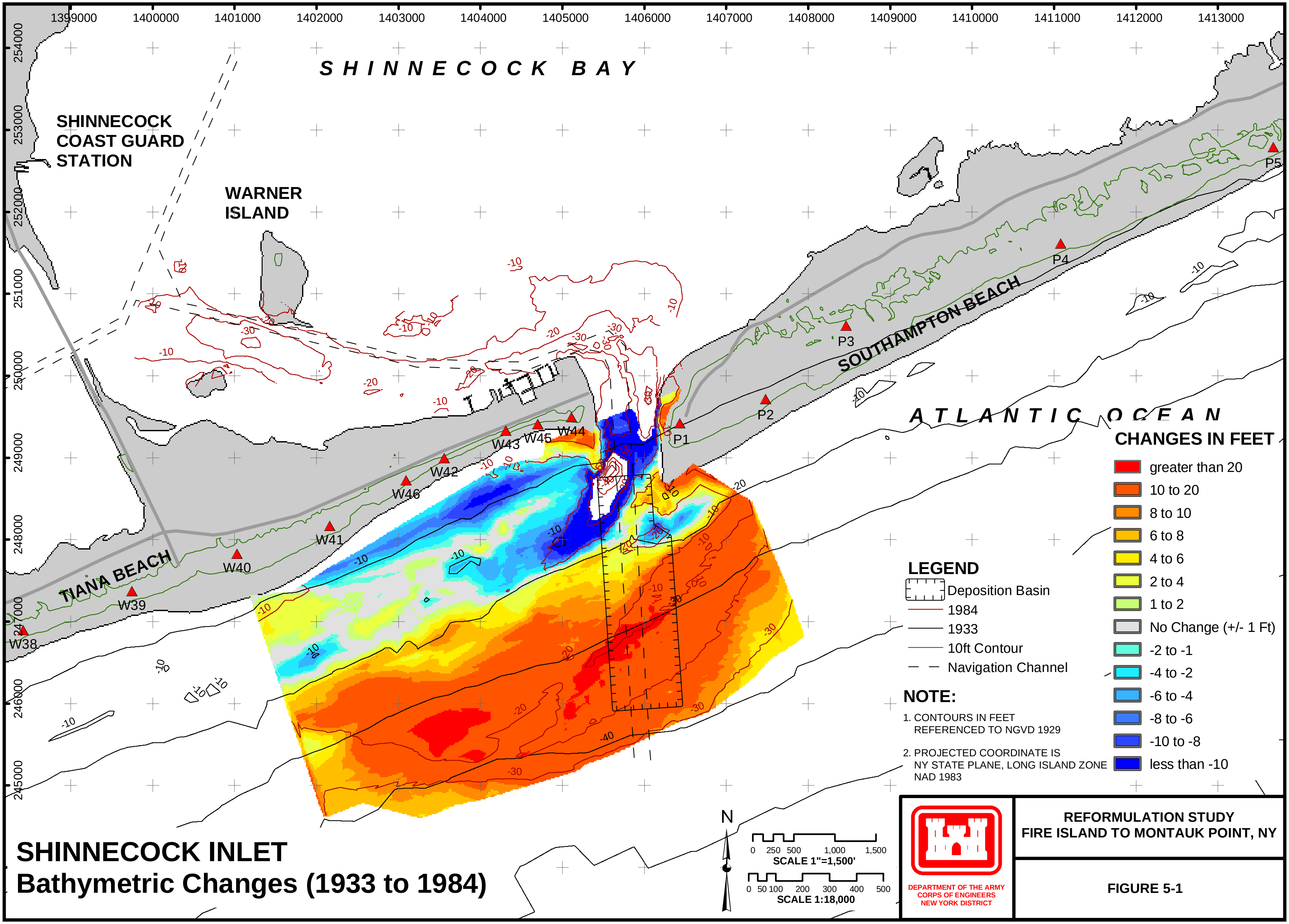
Existing (c. 2001) regional sediment budget suggests a net westerly transport rate of 351,000 m^3/yr entering the Updrift Beach (instead of 318,000 m^3/yr in the *Recent* sediment budget).

Other specific assumptions for the *Existing (c. 2001)* Fire Island Inlet sediment budget are as follows:

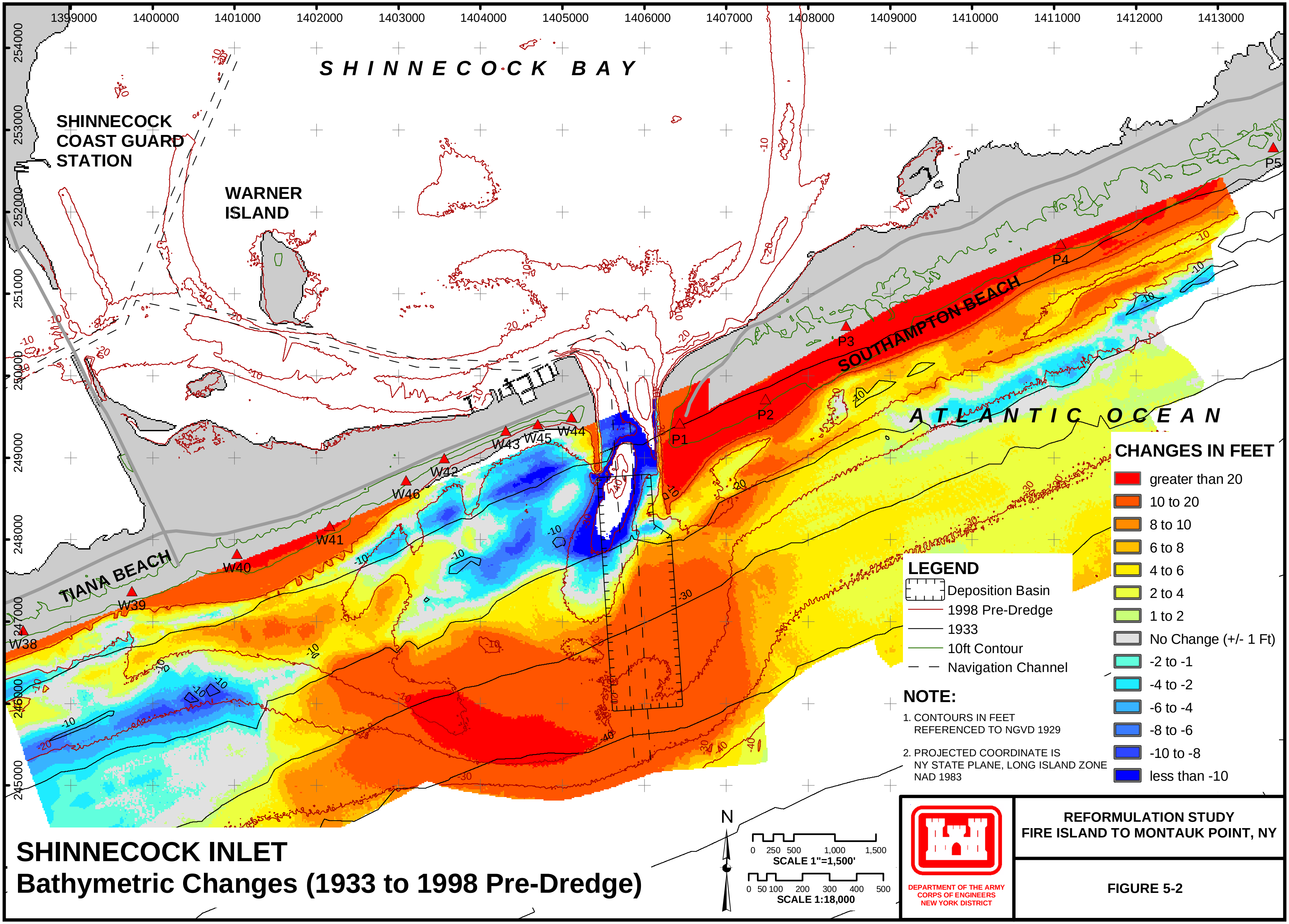
- Stability of the Updrift Beach. This means that only enough sediment to keep up with sea level rise is accumulated within this reach (i.e., approximately 9,000 m^3/yr). Placement within this cell was assumed to continue at the recent rate of roughly 62,000 m^3/yr.
- Available data suggest that the Sand Spit cell has accumulated a significant amount of sediment in recent years (75,000 m^3/yr between 1995 and 2001). However, if the channel and deposition basin continue to be maintained in their current position, this cell will eventually run out room to grow and further accumulate sediment. In fact, as of 2001 it appears that there is very little room left for growth. Therefore, a significantly reduced accumulation rate (15,000 m^3/yr) was assumed under *Existing* conditions as representative of the medium- to long-term evolution of the spit.
- As in the *Recent* sediment budget, it was assumed that under *Existing* conditions the Channel (CH) and Oak Beach (OB) cells accumulate material at a rate sufficient to meet the demand induced by sea level rise (24,000 m^3/yr in total for the two cells).
- Continued dredging of the Deposition Basin at the recent rate of 375,000 m^3/yr with only small net change in volume due to SLR (1,000 m^3/yr) was assumed as representative of *Existing* conditions.
- Accumulation within the ebb shoal was also assumed to continue in the medium- to long-term at the 1995-2001 rate of 68,000 m^3/yr.
- Under the foregoing assumptions the proposed *Existing (c. 2001)* condition sediment budget requires a significantly smaller reversal in net sediment transport direction at the boundary between the Ebb Shoal cell and the Downdrift Beach cell to balance the inlet budget, 79,000 m^3/yr versus 140,000 m^3/yr. This reduction owes mostly to an increase in incoming westerly longshore transport at the updrift inlet system boundary and a reduction in the amount of sediment lost to the Sand Spit cell.
- The estimated accretion rate of 28,000 m^3/yr within the Downdrift Beach during 1995-2001 period was also assumed to continue in the medium- to long-term and thus it was assumed to be representative of *Existing (c. 2001)* conditions.

Under these assumptions, approximately 59% (206,000 m^3/yr) of the net westerly transport under *Existing (c. 2001)* conditions bypasses the inlet system, which includes the channels, deposition basin, the ebb shoal, and the adjacent updrift and downdrift beaches[7]. Note this bypassing occurs mostly by mechanical means, i.e., dredging of the deposition basin and placement on the downdrift beach. The other 41% (145,000 m^3/yr) is assumed to accumulate within the ebb shoal, the channels and the adjacent beaches, some of it as a result of sea level rise. Therefore, this new *Existing* condition budget suggests that overall net impact of Fire Island Inlet to the regional sediment budget is reduced as compared to the conclusions presented in the previous work by USACE-NAN (1998) and adopted by Gravens et al. (1999) which suggested that as much as 535,000 m^3/yr accumulate within the inlet system. As explained above, total accumulation within Fire Island Inlet and the adjacent beaches may very well be greater than 145,000 m^3/yr. However, the recent data does not support a value as large as previously reported in USACE-NAN (1998) and Gravens et al. (1999). In fact, as explained below in Section 6, numerical analysis also performed by Gravens et al. (1999) suggests a potential longshore sediment transport to the west on the order of 200,000 m^3/yr along the Jones Island shoreline, which is consistent with the results of the *Existing (c. 2001)* sediment budget developed herein. On the other hand, accumulation of 535,000 m^3/yr in Fire Island Inlet as assumed in previous *Existing (c. 1999)* sediment budget, forced the net longshore sediment transport in Jones Island to be 466,000 m^3/yr to the <u>east</u>.

[7] Note that at Fire Island Inlet, changes on the adjacent updrift and downdrift beaches were included in the bypassing efficiency calculation because all the sand dredged from the Deposition Basin was placed within these two cells

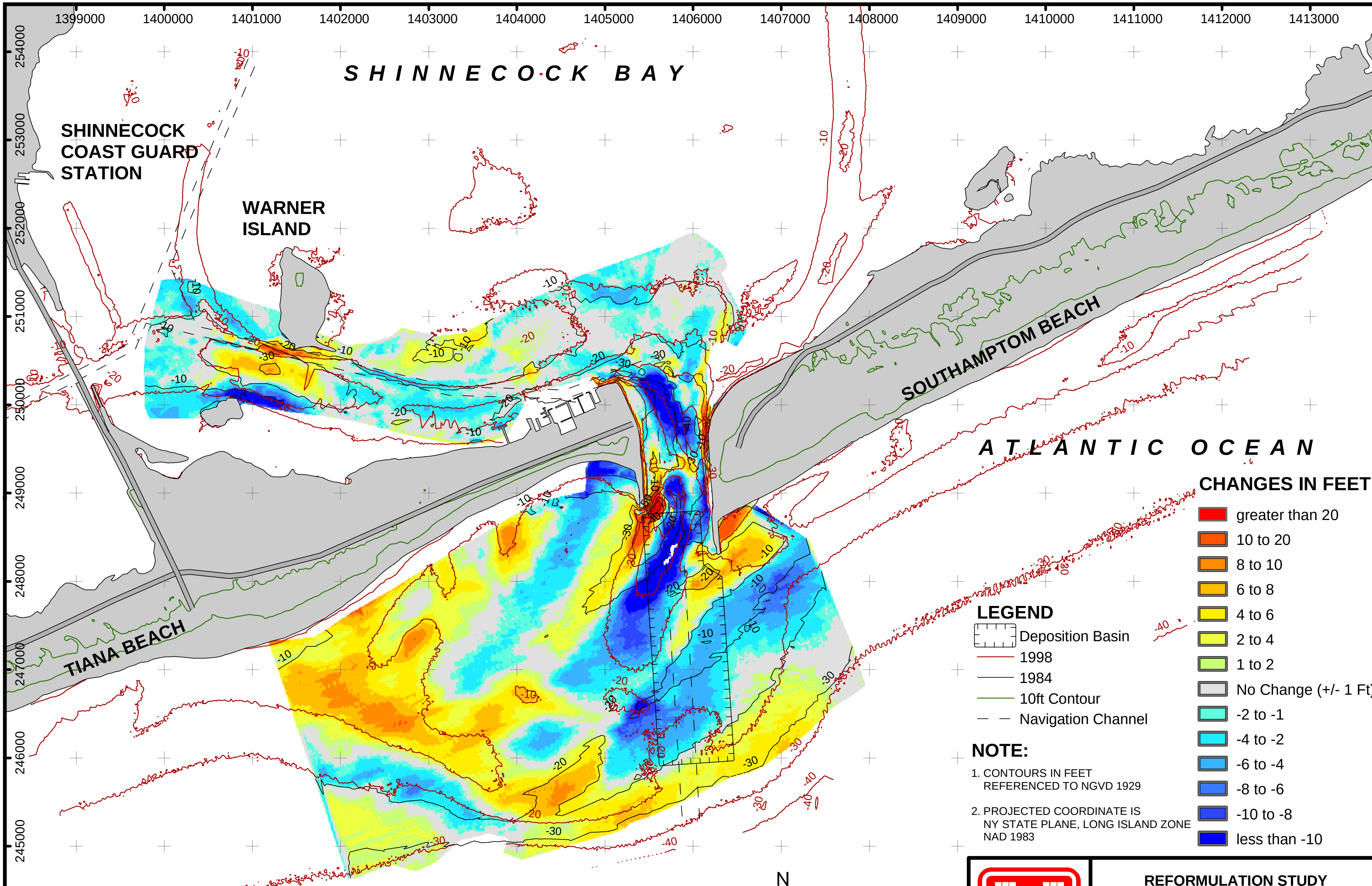


Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Shin_Diff.mxd



SHINNECOCK INLET
Bathymetric Changes (1933 to 1998 Pre-Dredge)

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Shin_Diff.mxd



SHINNECOCK INLET
Bathymetric Changes (1984 to Spring 1998)

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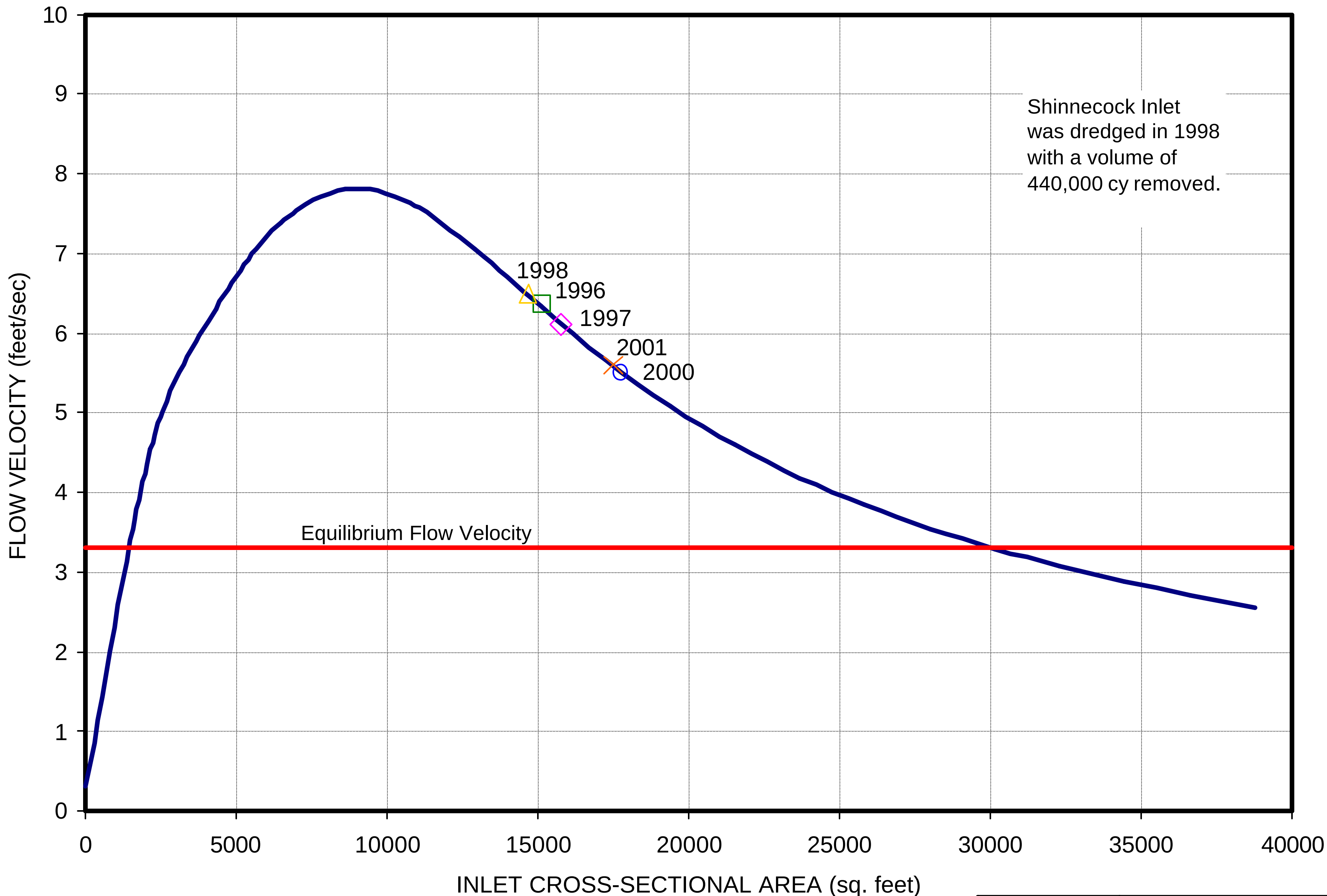
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 5-3

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Shin_Diff.mxd



SHINNECOCK INLET - Stability Analysis, Existing Conditions

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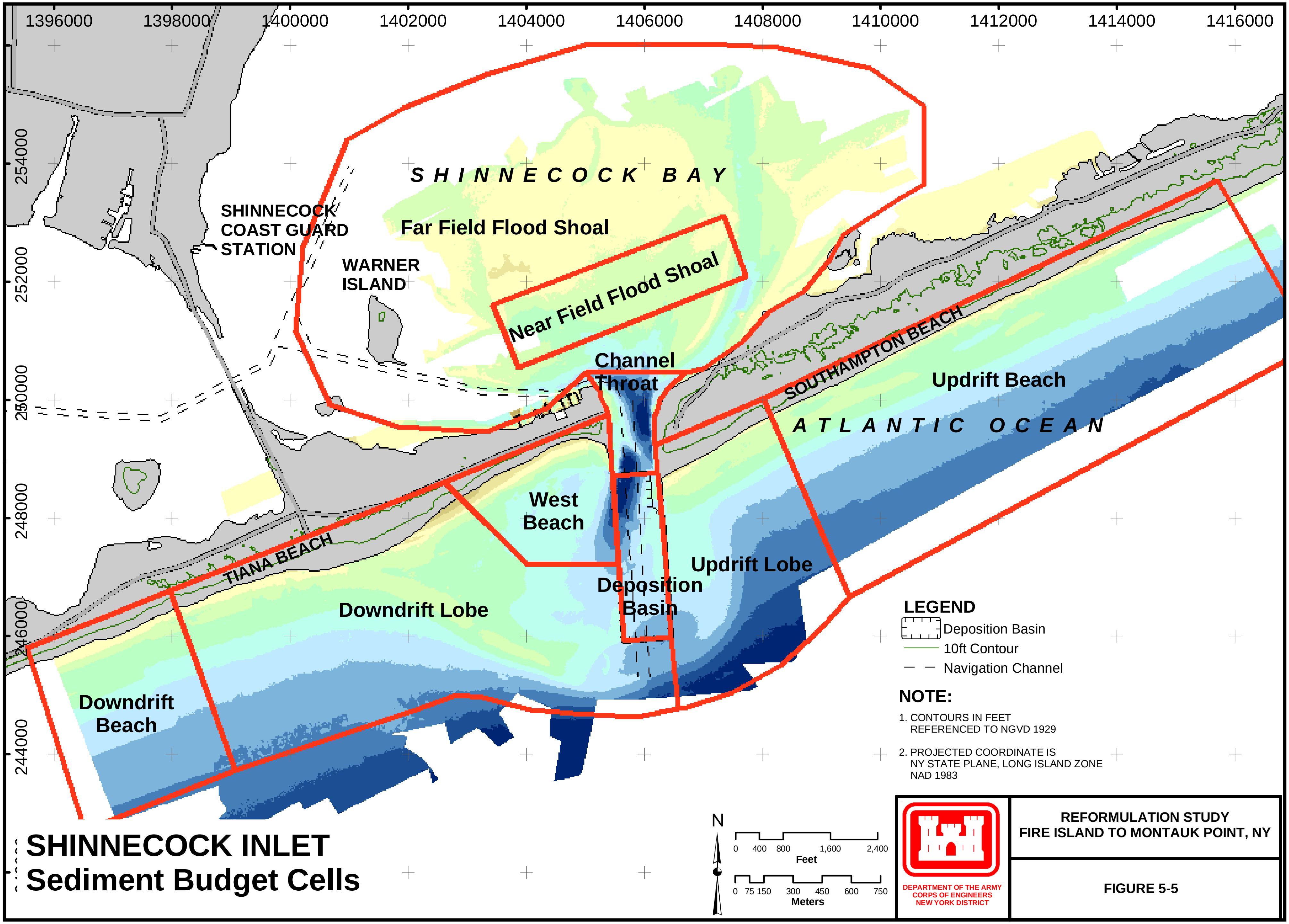
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

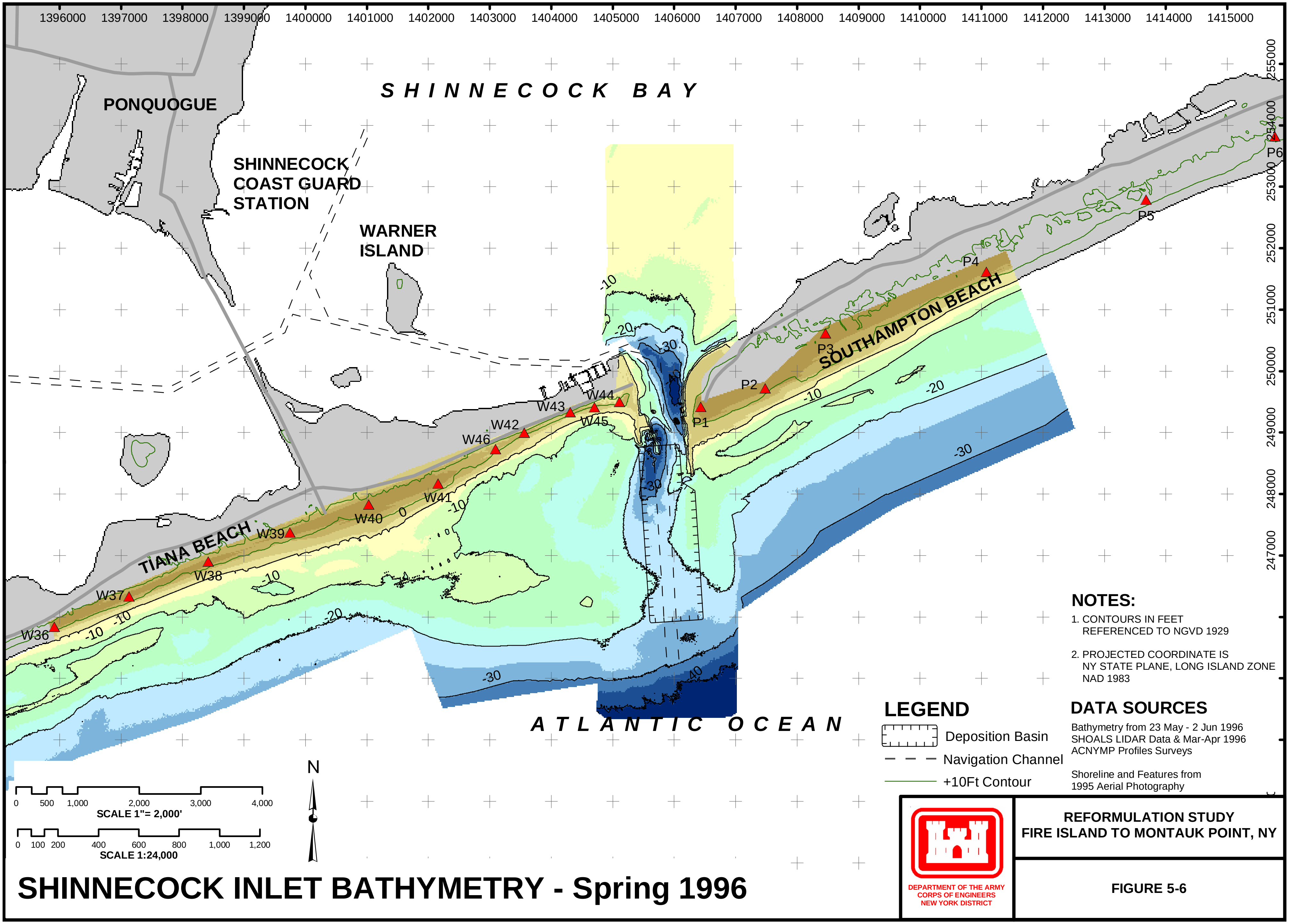
FIGURE 5-4

FILE: 3982-28 FIMP/Reports/Report2002/SIStability.ppt



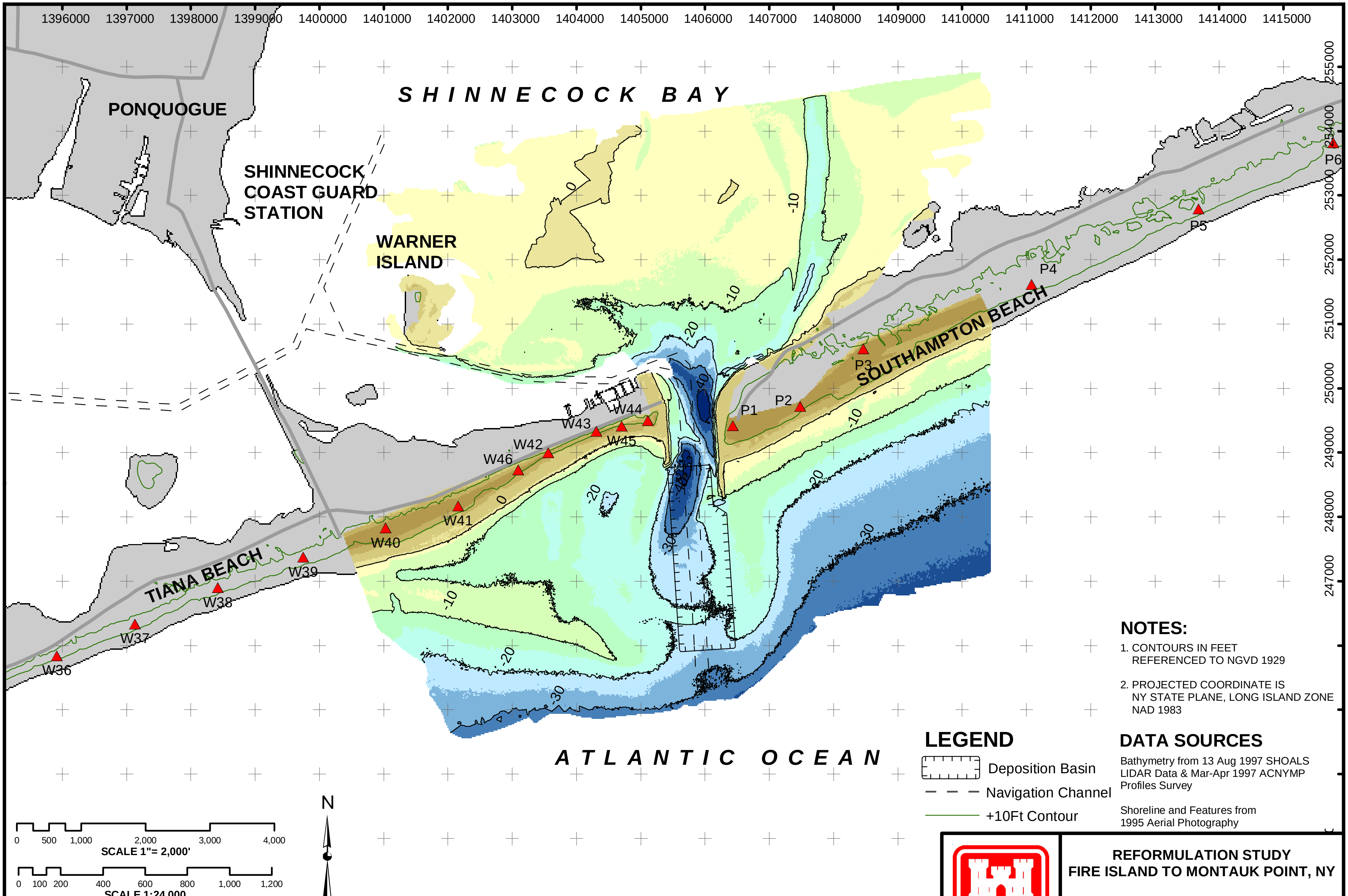
SHINNECOCK INLET
Sediment Budget Cells

Filename: 3982-28/ReportDec2002/Shinn_Cells.mxd



SHINNECOCK INLET BATHYMETRY - Spring 1996

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Shin_Depth.mxd



NOTES:

1. CONTOURS IN FEET REFERENCED TO NGVD 1929
2. PROJECTED COORDINATE IS NY STATE PLANE, LONG ISLAND ZONE NAD 1983

LEGEND

Deposition Basin
Navigation Channel
+10Ft Contour

DATA SOURCES

Bathymetry from 13 Aug 1997 SHOALS LIDAR Data & Mar-Apr 1997 ACNYMP Profiles Survey

Shoreline and Features from 1995 Aerial Photography

SHINNECOCK INLET BATHYMETRY - Spring-Summer 1997

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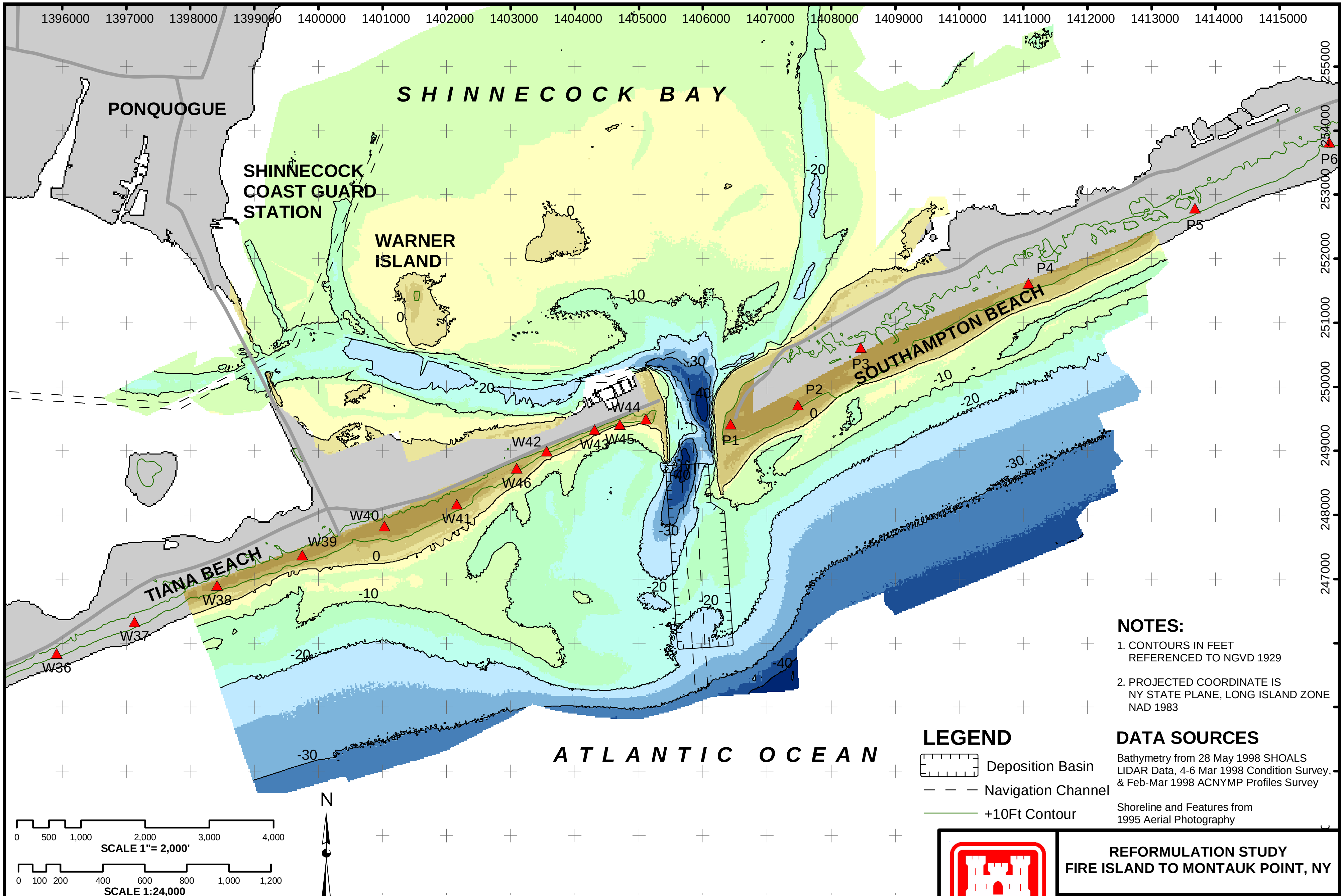
REFORMULATION STUDY FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 5-7

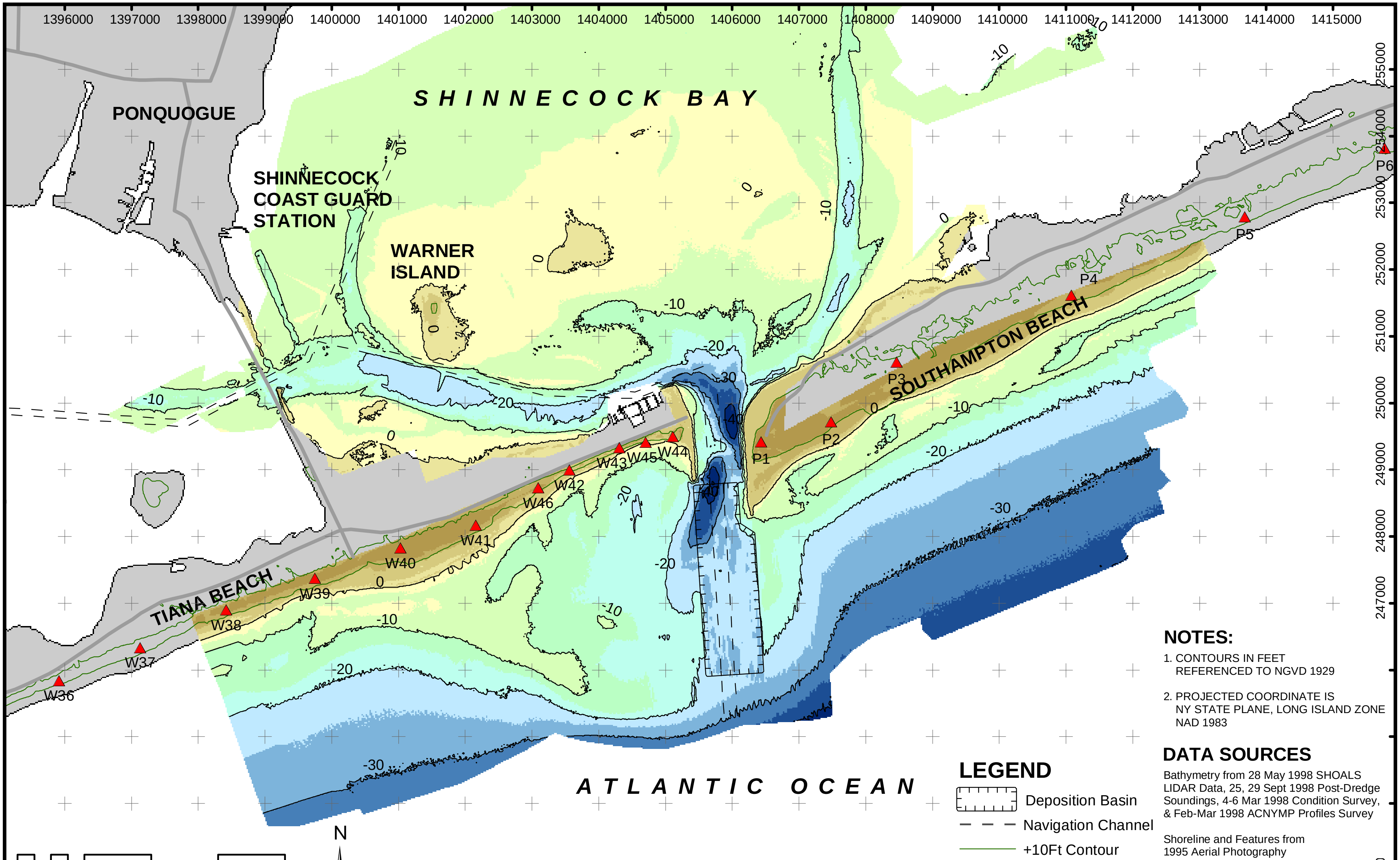
Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Shin_Depth.mxd



SHINNECOCK INLET BATHYMETRY - Spr 1998 (Pre-Dredge)

FIGURE 5-8

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Shin_Depth.mxd



NOTES:

1. CONTOURS IN FEET REFERENCED TO NGVD 1929
2. PROJECTED COORDINATE IS NY STATE PLANE, LONG ISLAND ZONE NAD 1983

DATA SOURCES

Bathymetry from 28 May 1998 SHOALS LIDAR Data, 25, 29 Sept 1998 Post-Dredge Soundings, 4-6 Mar 1998 Condition Survey, & Feb-Mar 1998 ACNYMP Profiles Survey

Shoreline and Features from 1995 Aerial Photography

LEGEND

- Deposition Basin
- Navigation Channel
- +10Ft Contour

0 500 1,000 2,000 3,000 4,000
SCALE 1"= 2,000'

0 100 200 400 600 800 1,000 1,200
SCALE 1:24,000

SHINNECOCK INLET BATHYMETRY- Fall 1998(Post-Dredge)

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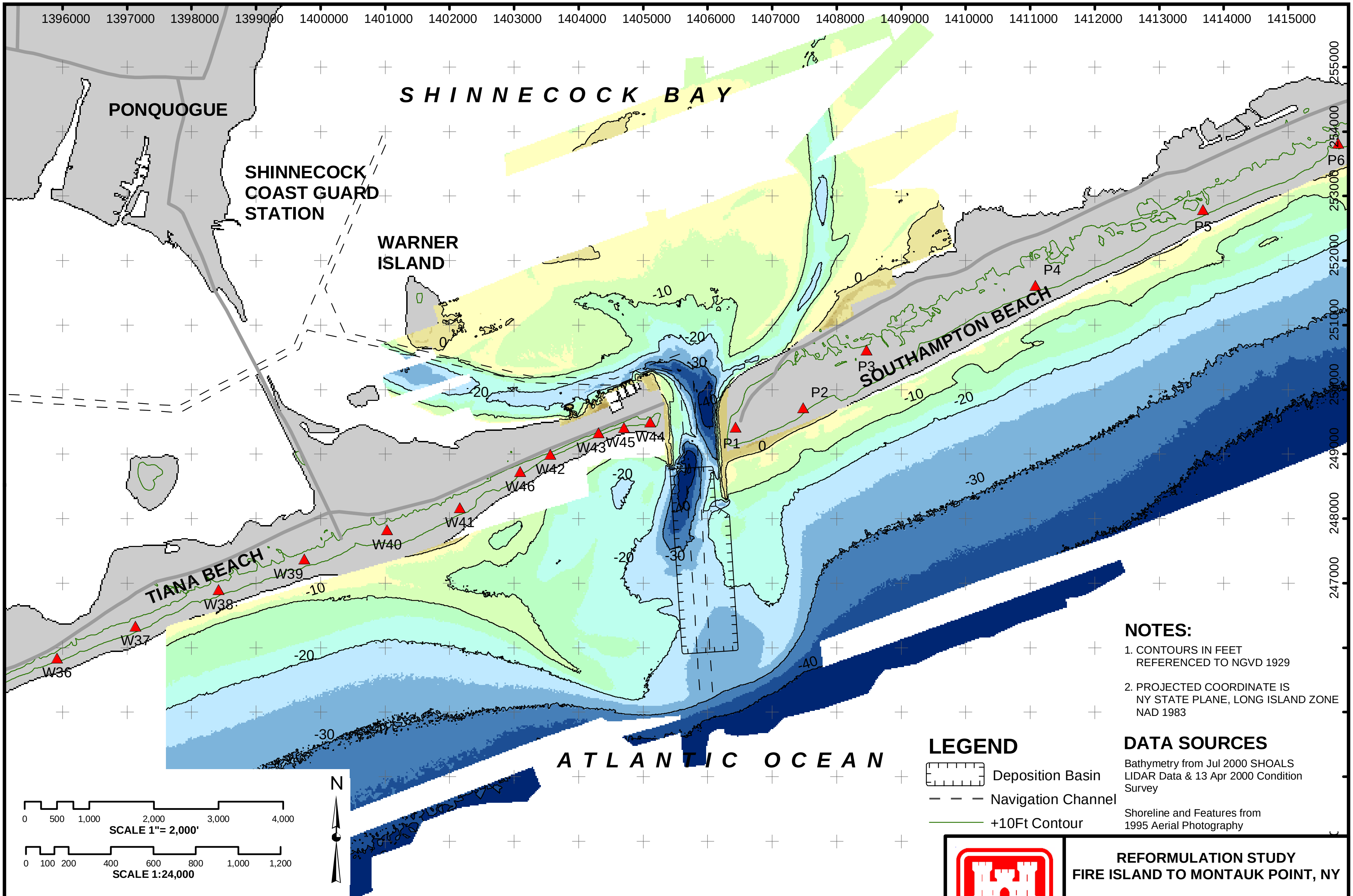
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 5-9

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Shin_Depth.mxd



SHINNECOCK INLET BATHYMETRY- Spring-Summer 2000

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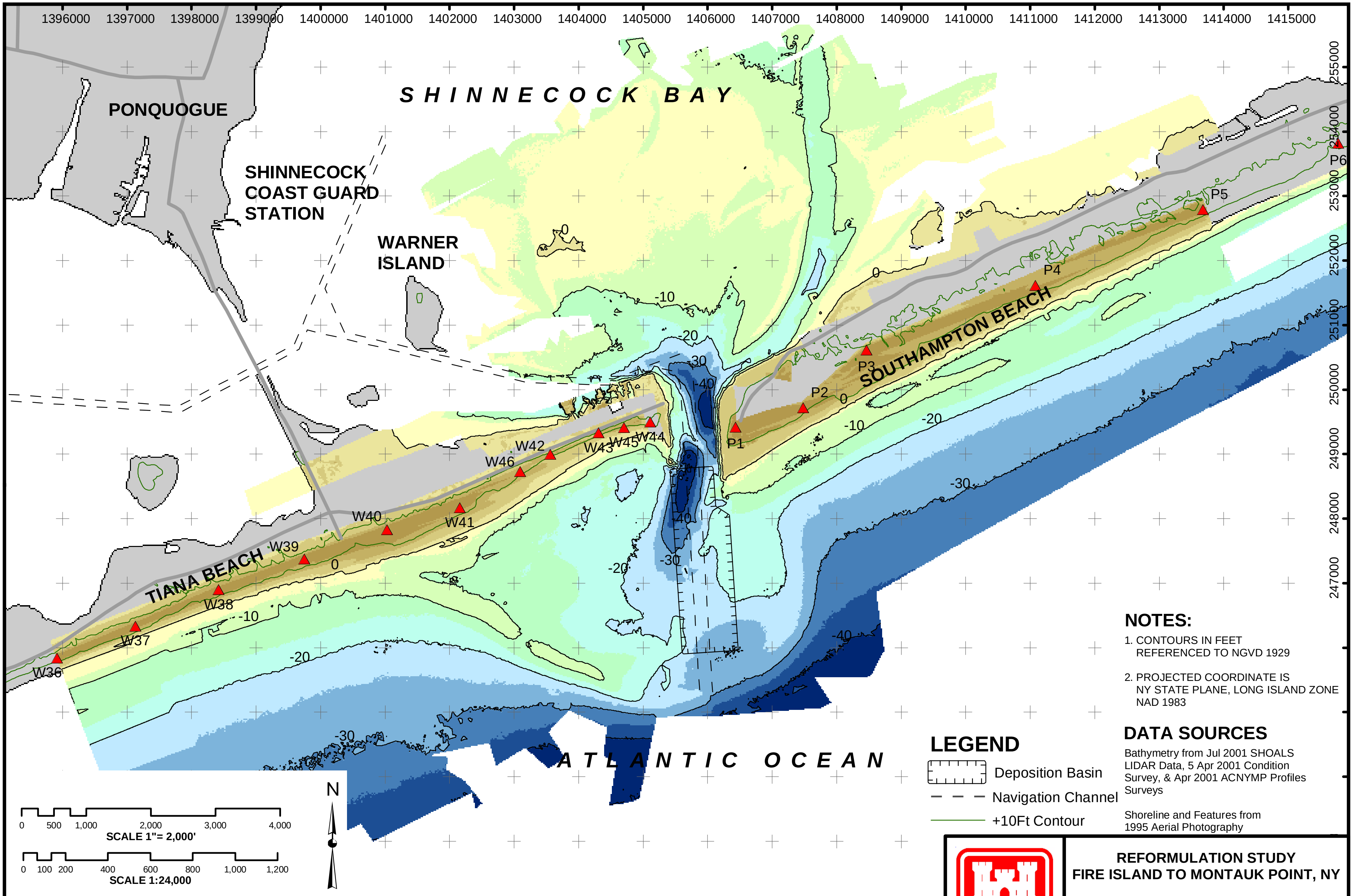
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 5-10

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Shin_Depth.mxd



NOTES:

1. CONTOURS IN FEET REFERENCED TO NGVD 1929
2. PROJECTED COORDINATE IS NY STATE PLANE, LONG ISLAND ZONE NAD 1983

DATA SOURCES

Bathymetry from Jul 2001 SHOALS LIDAR Data, 5 Apr 2001 Condition Survey, & Apr 2001 ACNYMP Profiles Surveys

Shoreline and Features from 1995 Aerial Photography

LEGEND

Deposition Basin
Navigation Channel
+10Ft Contour

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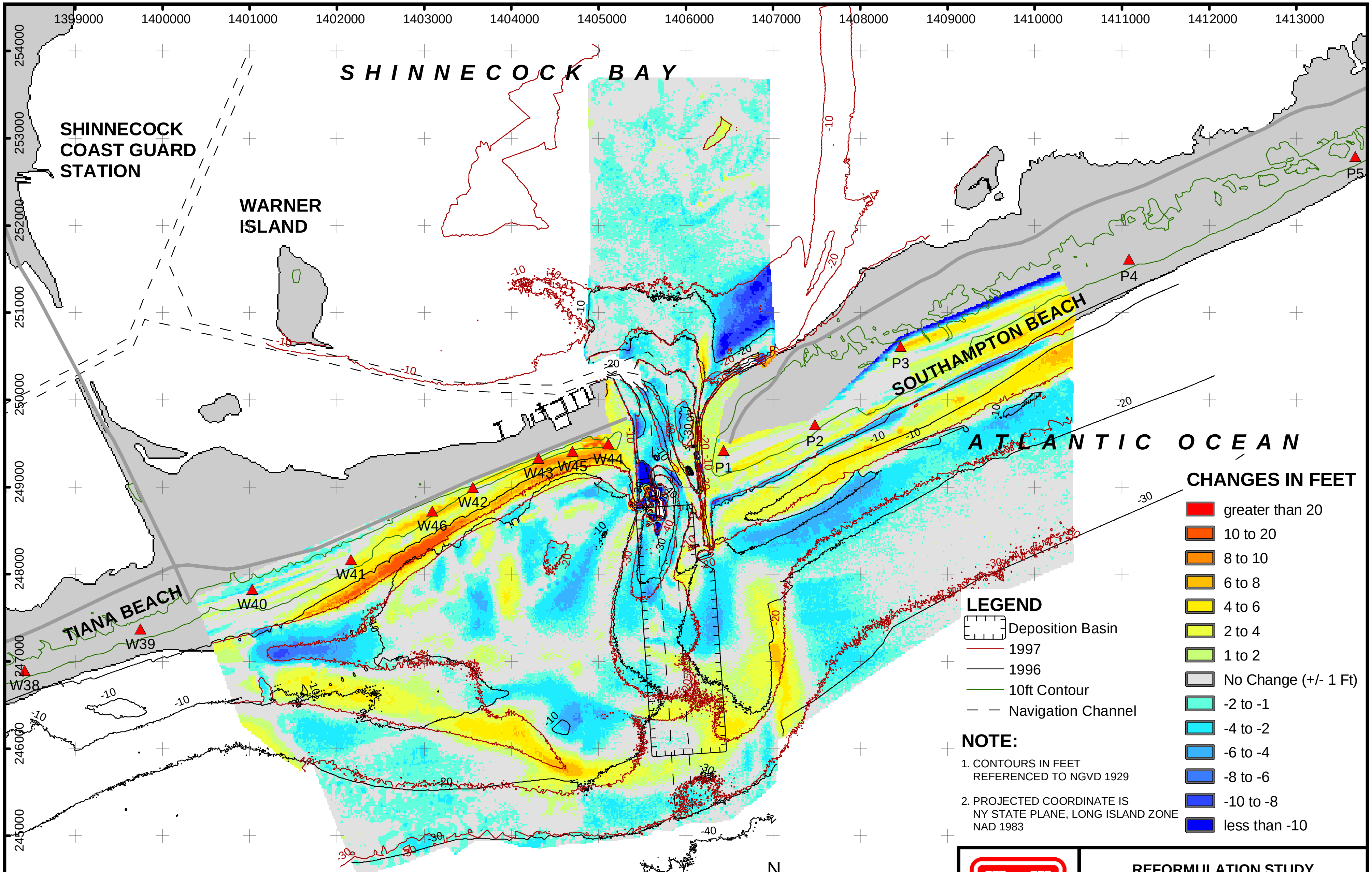
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 5-11

SHINNECOCK INLET BATHYMETRY- Spring-Summer 2001

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Shin_Depth.mxd



SHINNECOCK INLET
Bathymetric Changes (1996 to 1997)

N

0 250 500 1,000 1,500
SCALE 1"=1,500'

0 50 100 200 300 400 500
SCALE 1:18,000

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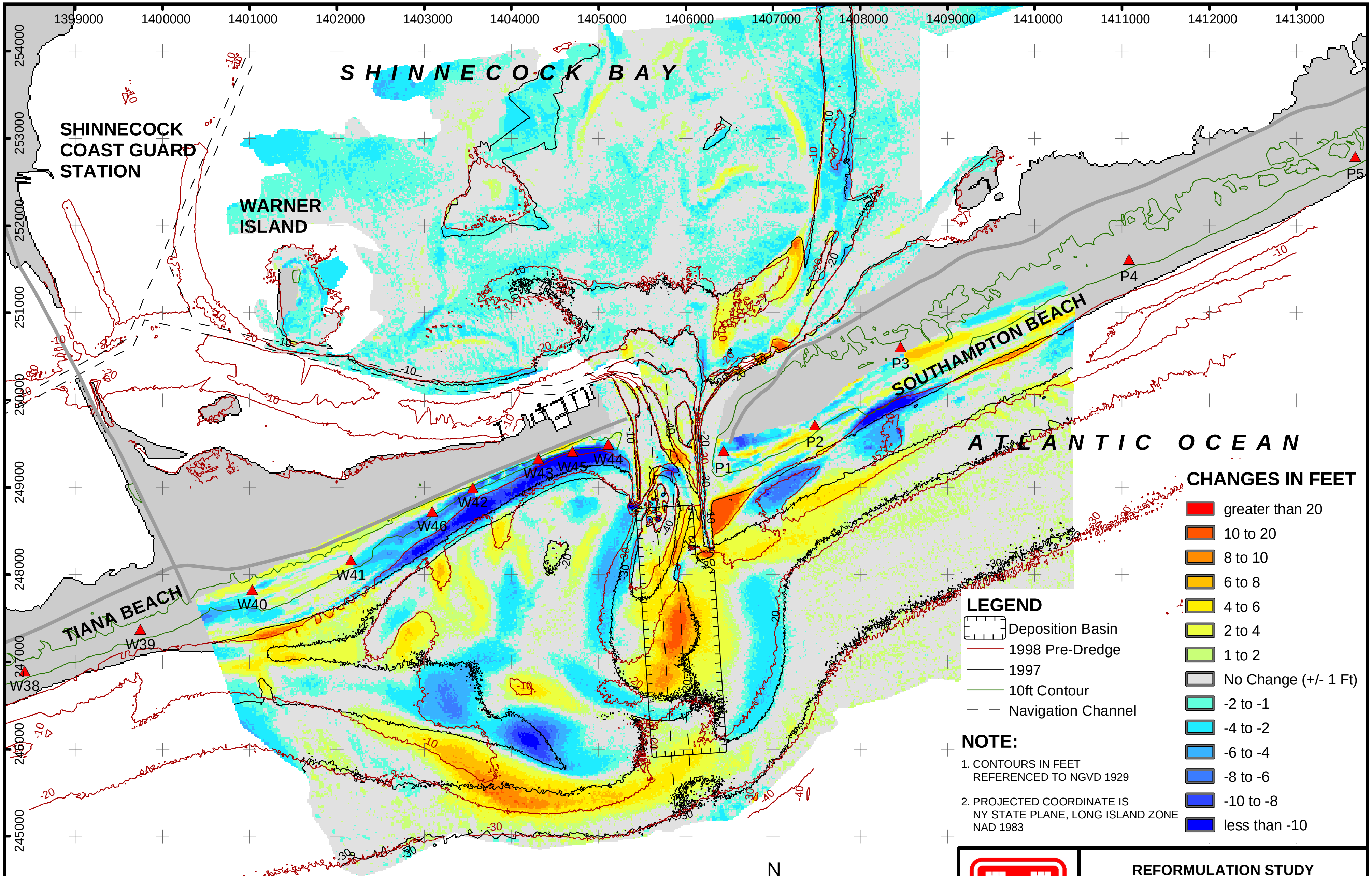
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 5-12

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Shin_Diff.mxd



SHINNECOCK INLET
Bathymetric Changes (1997 to 1998 Pre-Dredge)

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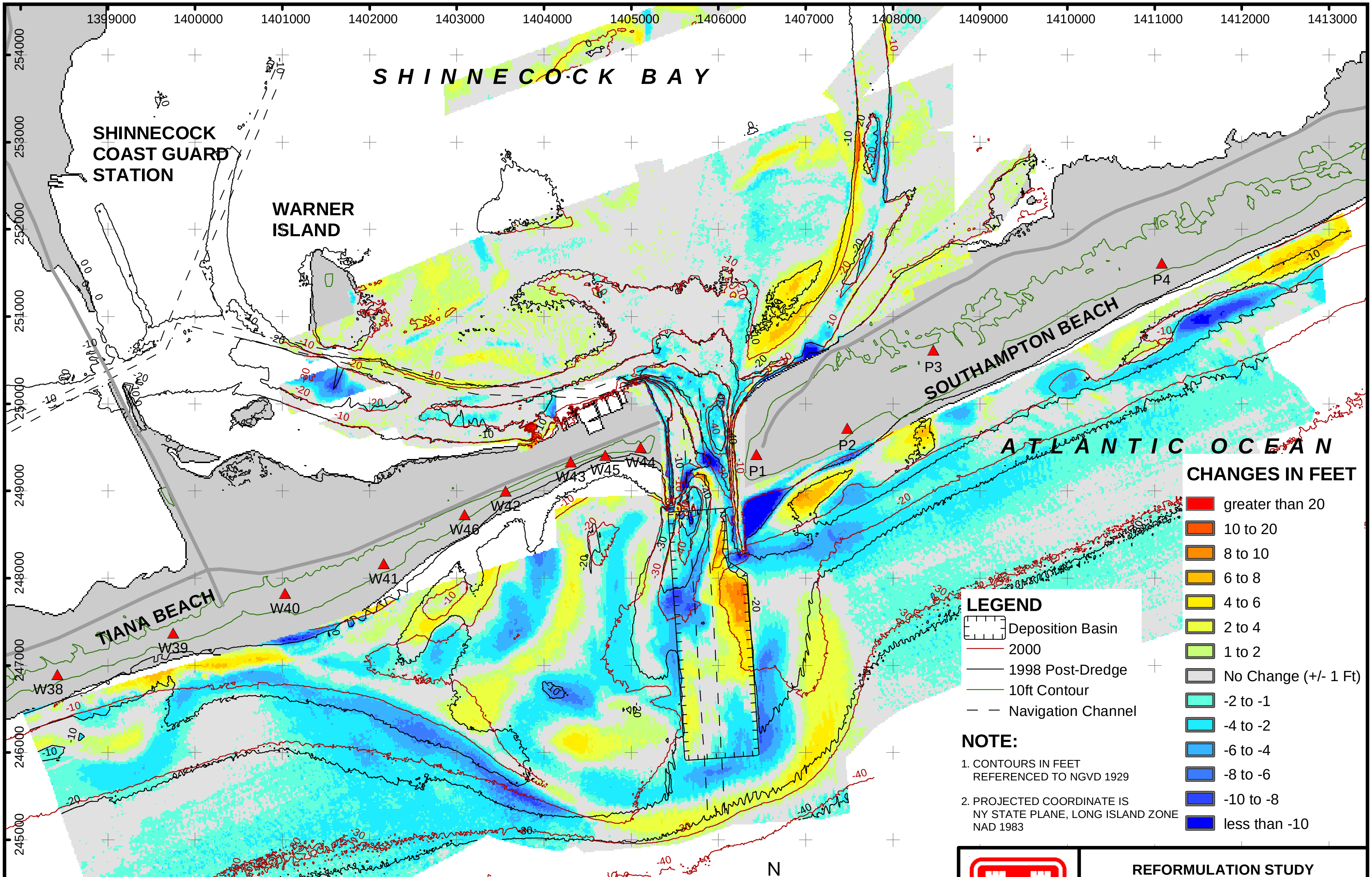
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 5-13

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Shin_Diff.mxd



NOTE:

1. CONTOURS IN FEET REFERENCED TO NGVD 1929
2. PROJECTED COORDINATE IS NY STATE PLANE, LONG ISLAND ZONE NAD 1983

SHINNECOCK INLET
Bathymetric Changes (1998 Post-Dredge to 2000)

N

0 250 500 1,000 1,500
SCALE 1"=1,500'

0 50 100 200 300 400 500
SCALE 1:18,000

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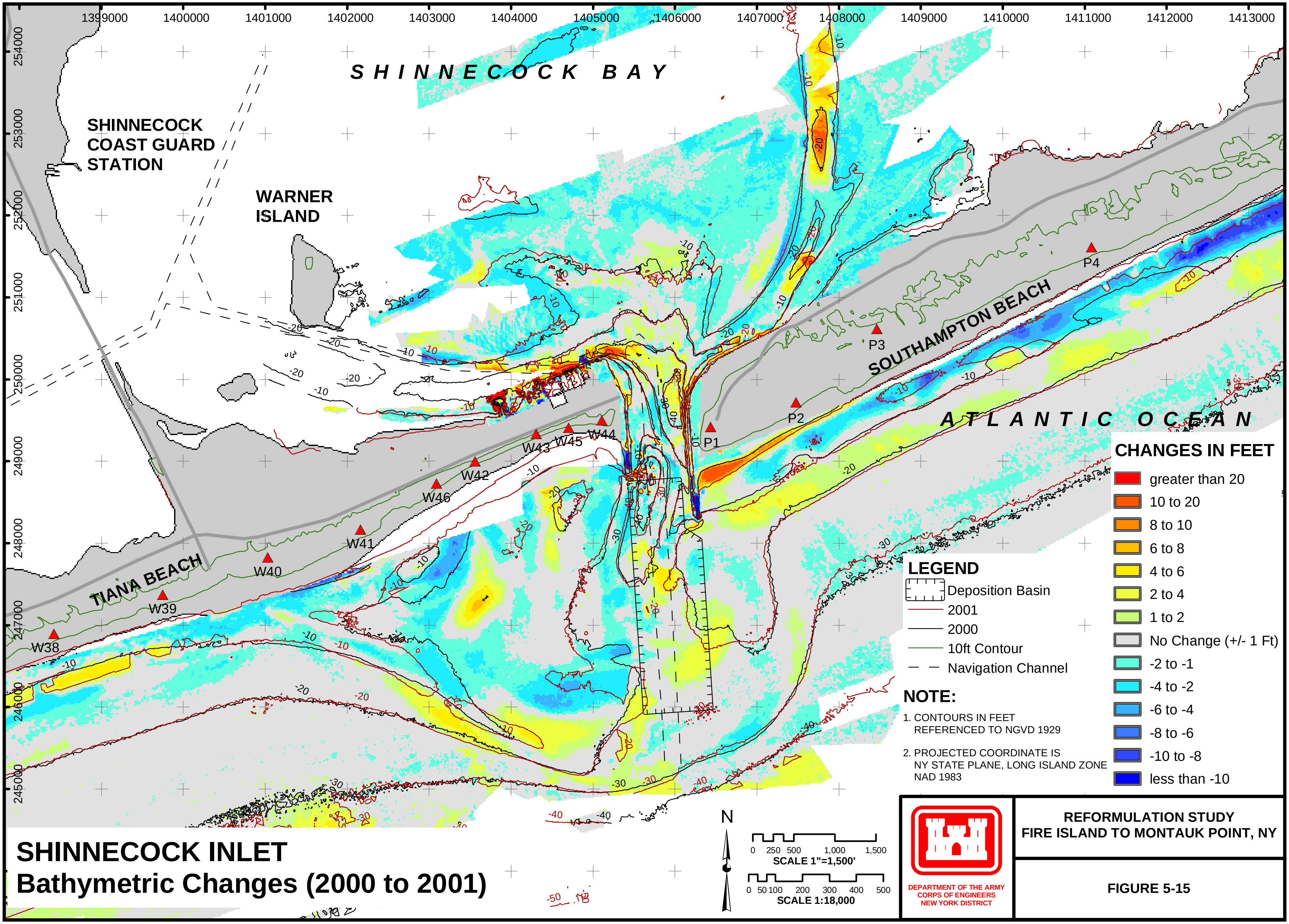
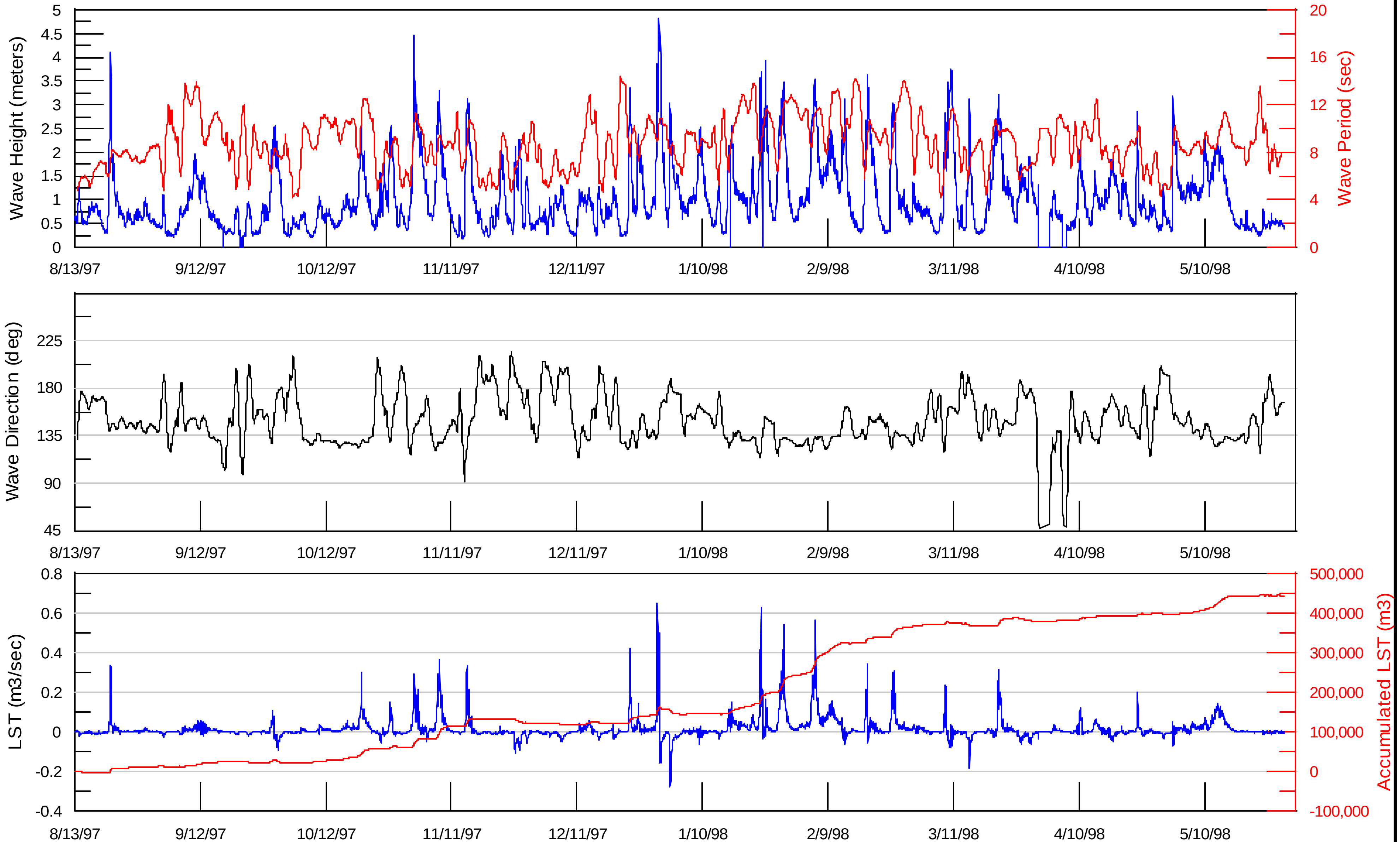
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 5-14

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Shin_Diff.mxd



Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Shin_Diff.mxd



Wave Climate & LST East of Shinnecock Inlet (1997-98)

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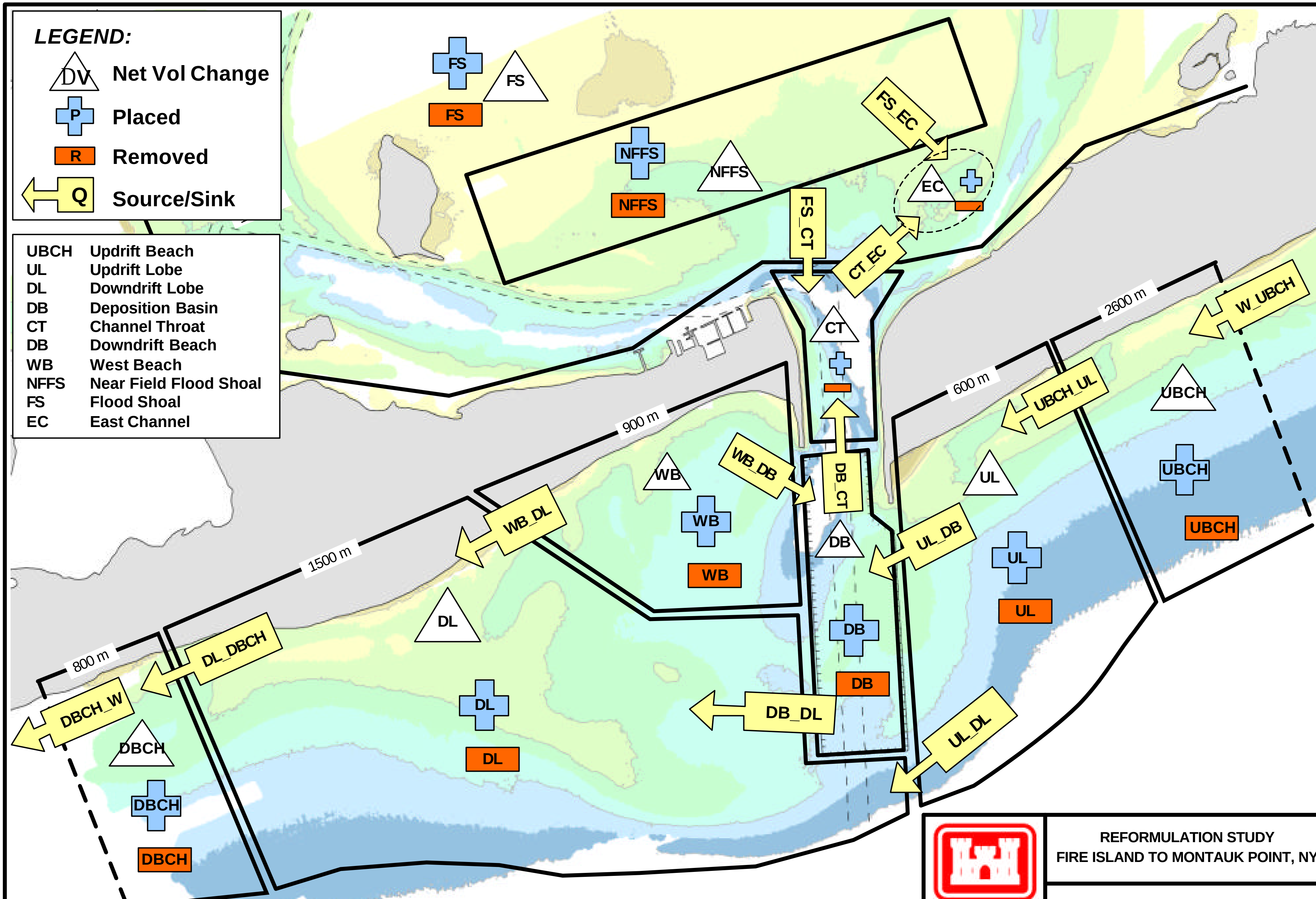
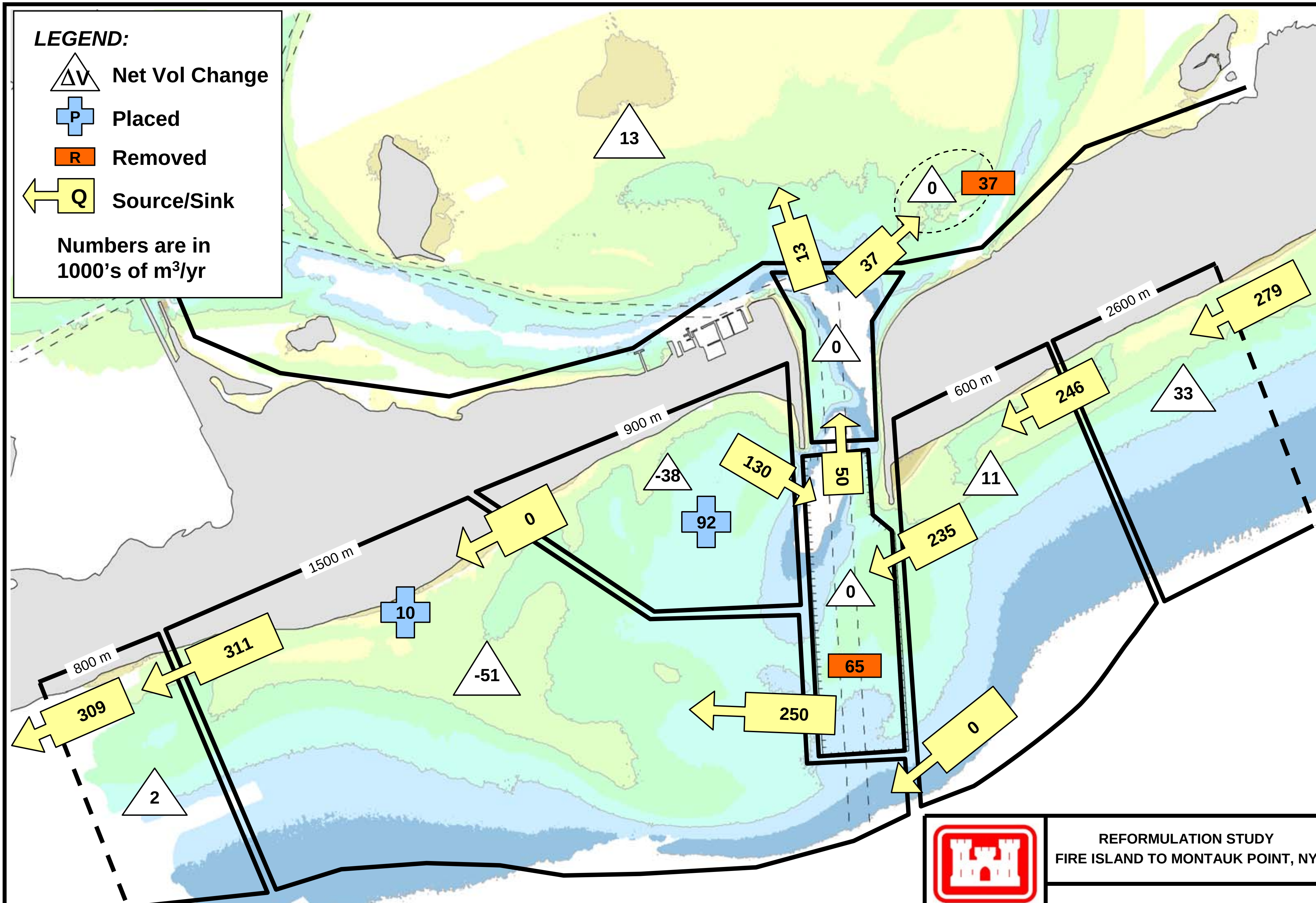
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 5-16



Shinnecock Inlet Conceptual Sediment Budget



Shinnecock Inlet *Recent (1995/01)* Sediment Budget - Alt # 1

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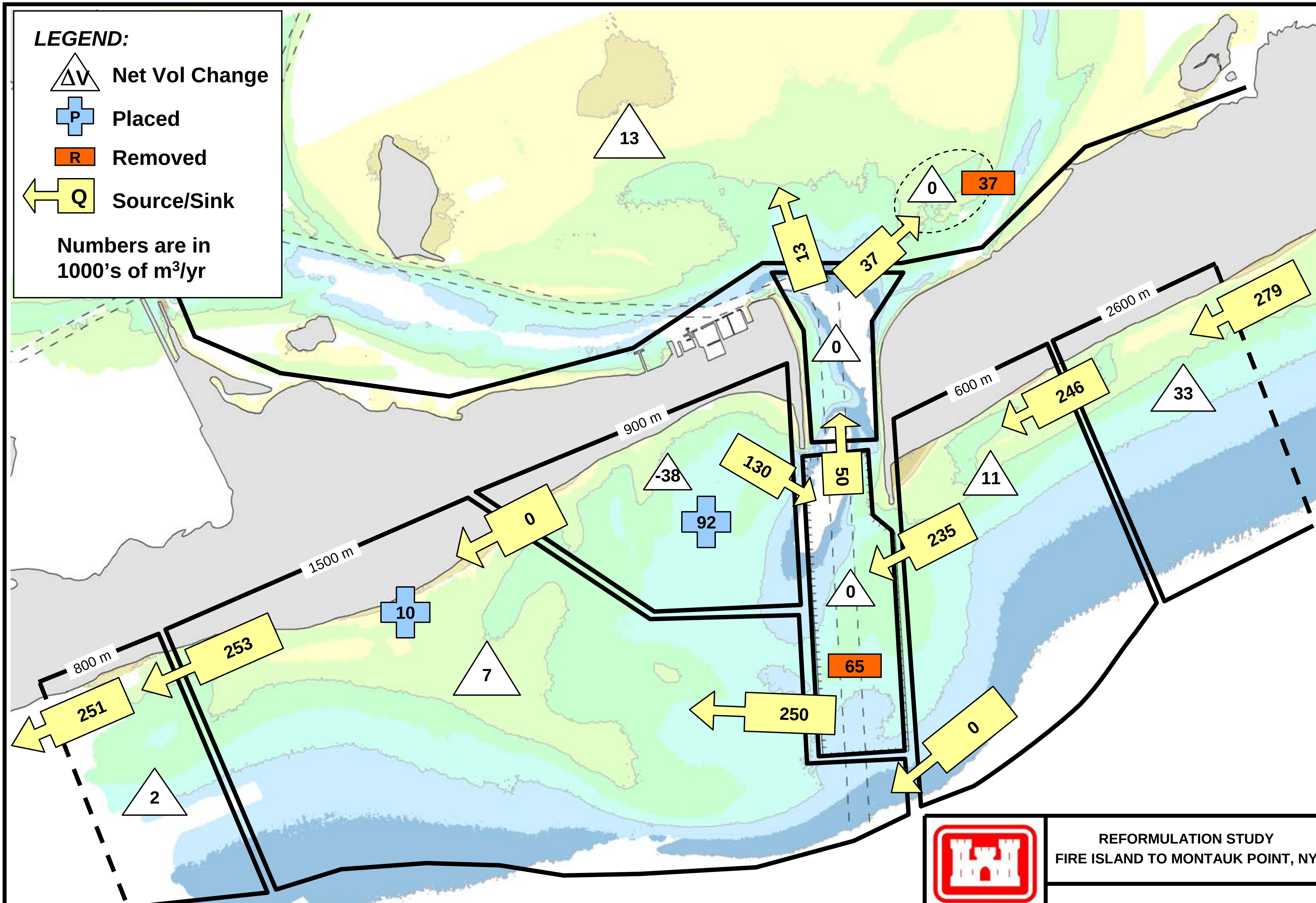
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 5-18



Shinnecock Inlet Recent *(1995/01)* Sediment Budget - Alt # 2

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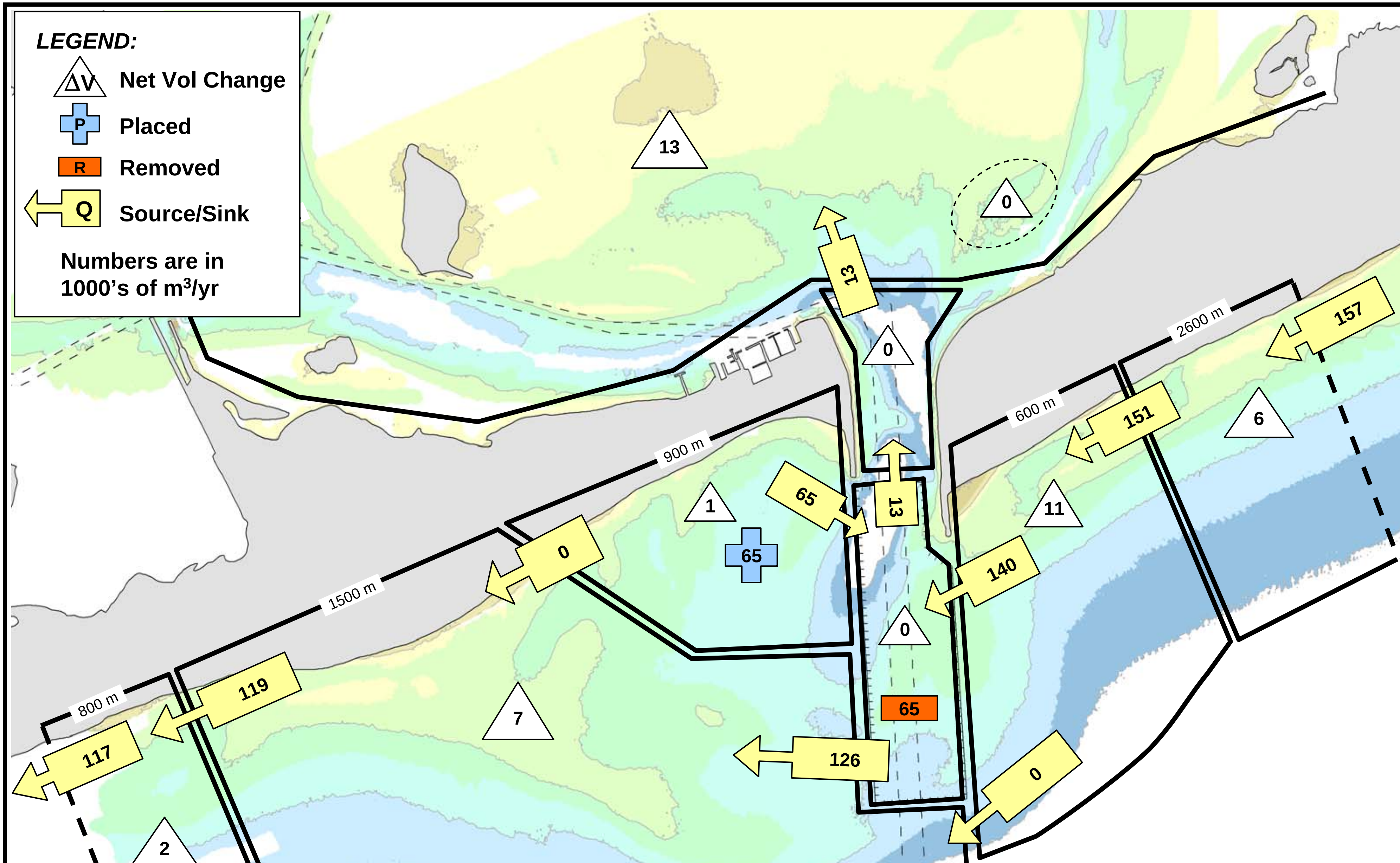
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 5-19



Shinnecock Inlet *Existing (c. 2001)* Sediment Budget

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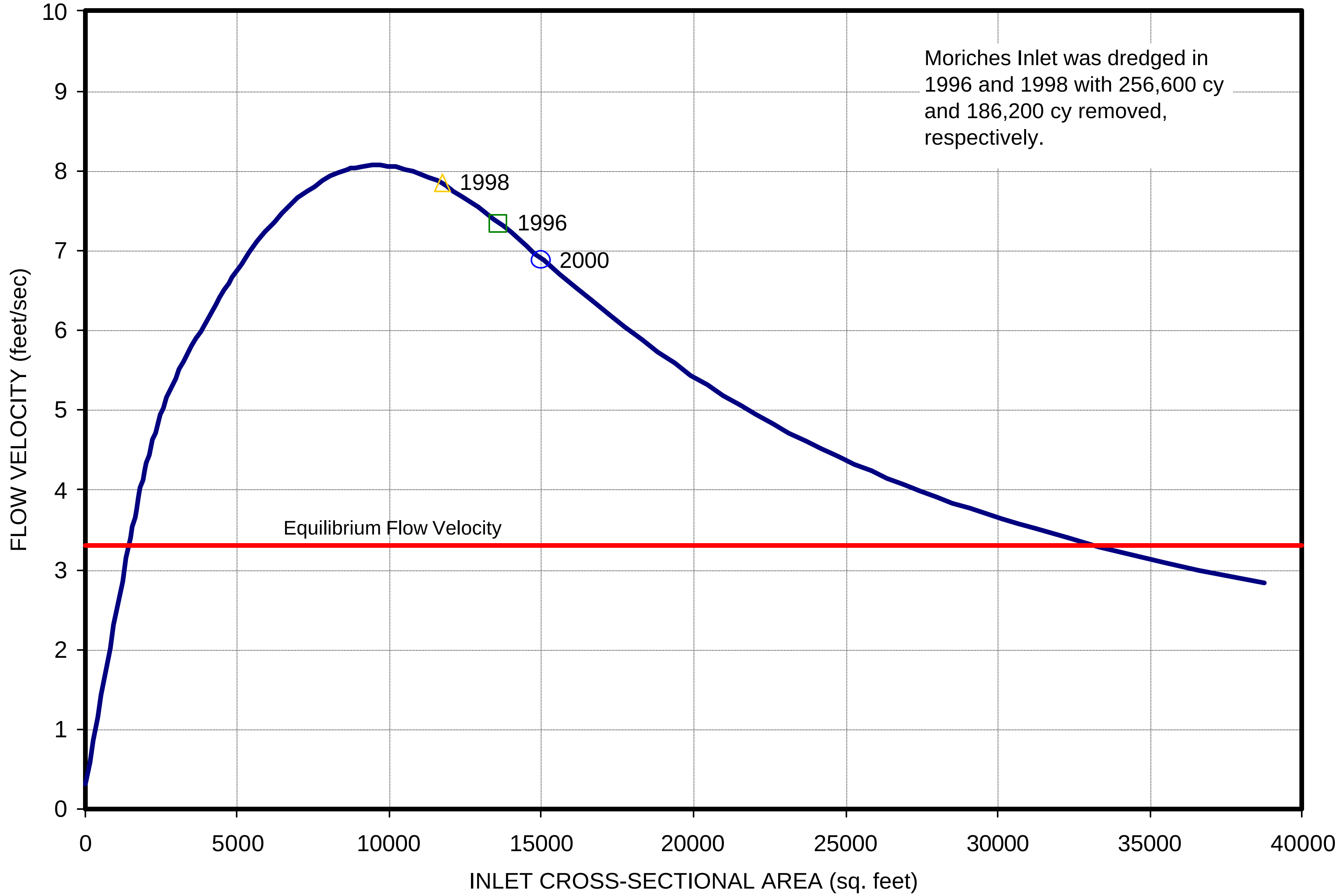
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 5-20



MORICHES INLET - Stability Analysis, Existing Conditions

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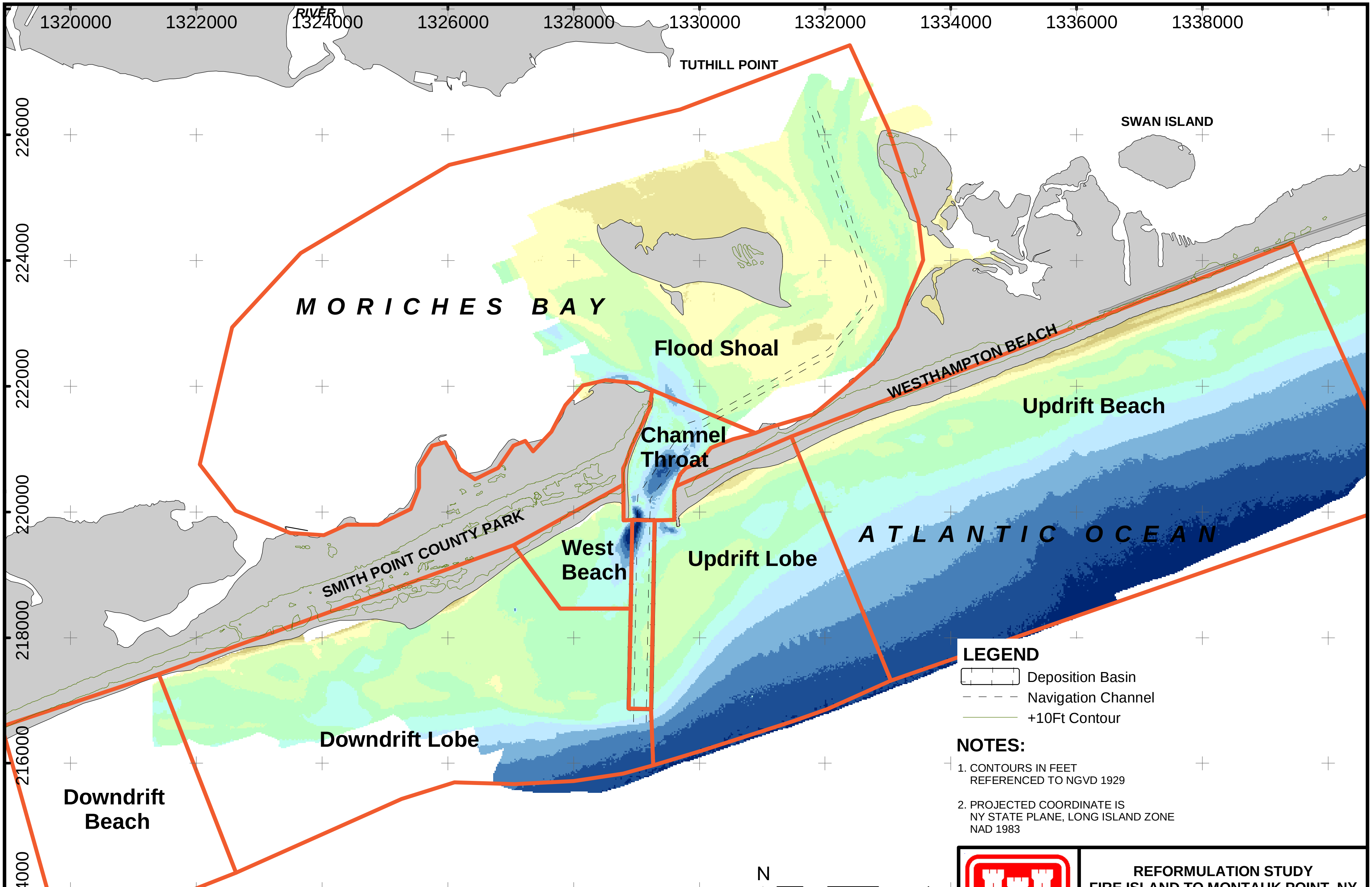
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 5-21

FILE: 3982-28 FIMP/Reports/Report2002/MIStability.ppt



LEGEND

- Deposition Basin
- Navigation Channel
- +10Ft Contour

NOTES:

1. CONTOURS IN FEET REFERENCED TO NGVD 1929
2. PROJECTED COORDINATE IS NY STATE PLANE, LONG ISLAND ZONE NAD 1983

N

0 400 800 1,600 2,400 Feet

0 75 150 300 450 600 750 Meters

MORICHES INLET
Sediment Budget Cells

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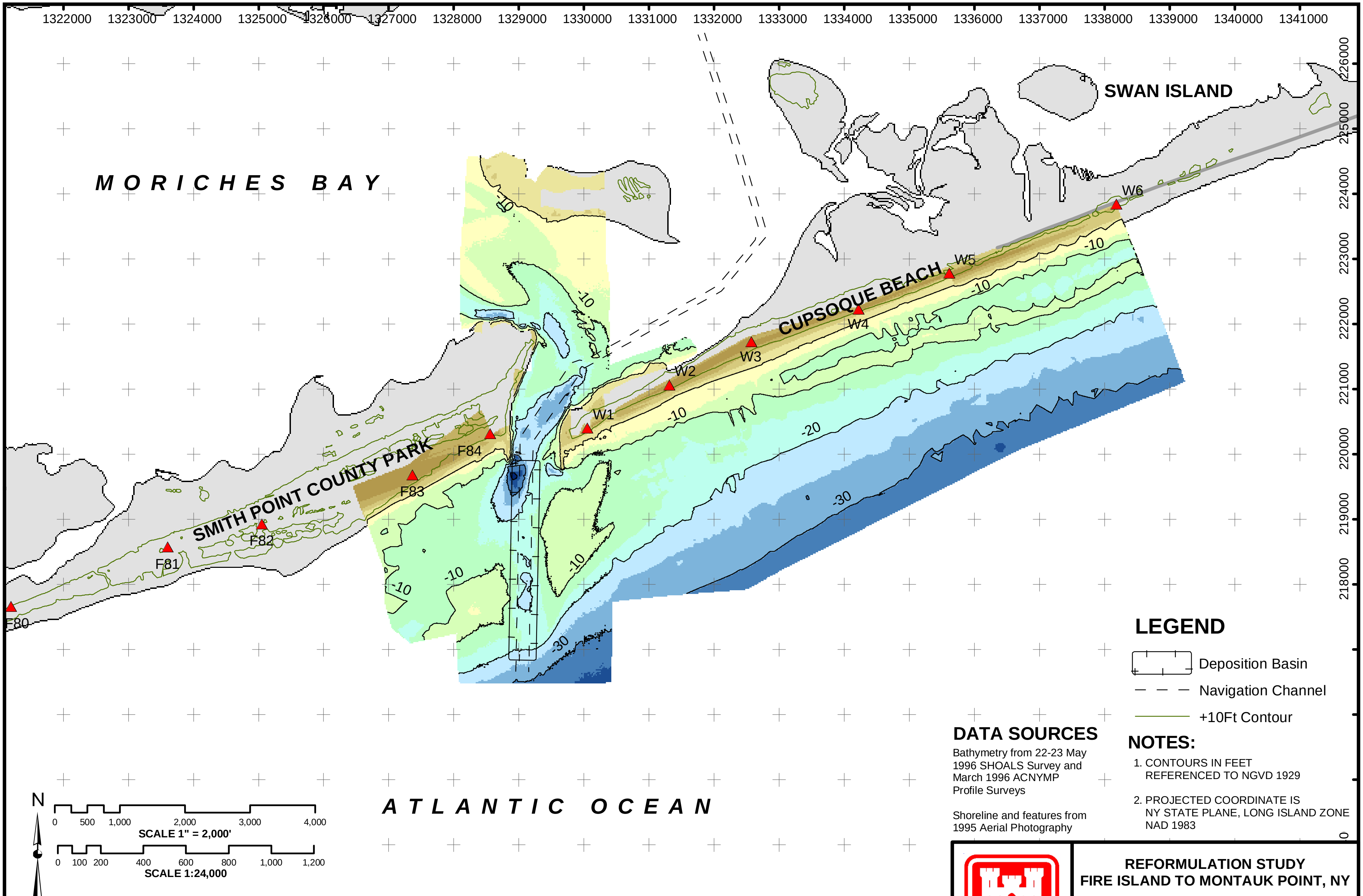
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 5-22

Filename: 3982-28/ReportDec2002/Moriches_Cells.mxd



LEGEND

- Deposition Basin
- Navigation Channel
- +10Ft Contour

DATA SOURCES

Bathymetry from 22-23 May 1996 SHOALS Survey and March 1996 ACNYMP Profile Surveys

Shoreline and features from 1995 Aerial Photography

NOTES:

1. CONTOURS IN FEET REFERENCED TO NGVD 1929
2. PROJECTED COORDINATE IS NY STATE PLANE, LONG ISLAND ZONE NAD 1983

MORICHES INLET BATHYMETRY - SPRING 1996

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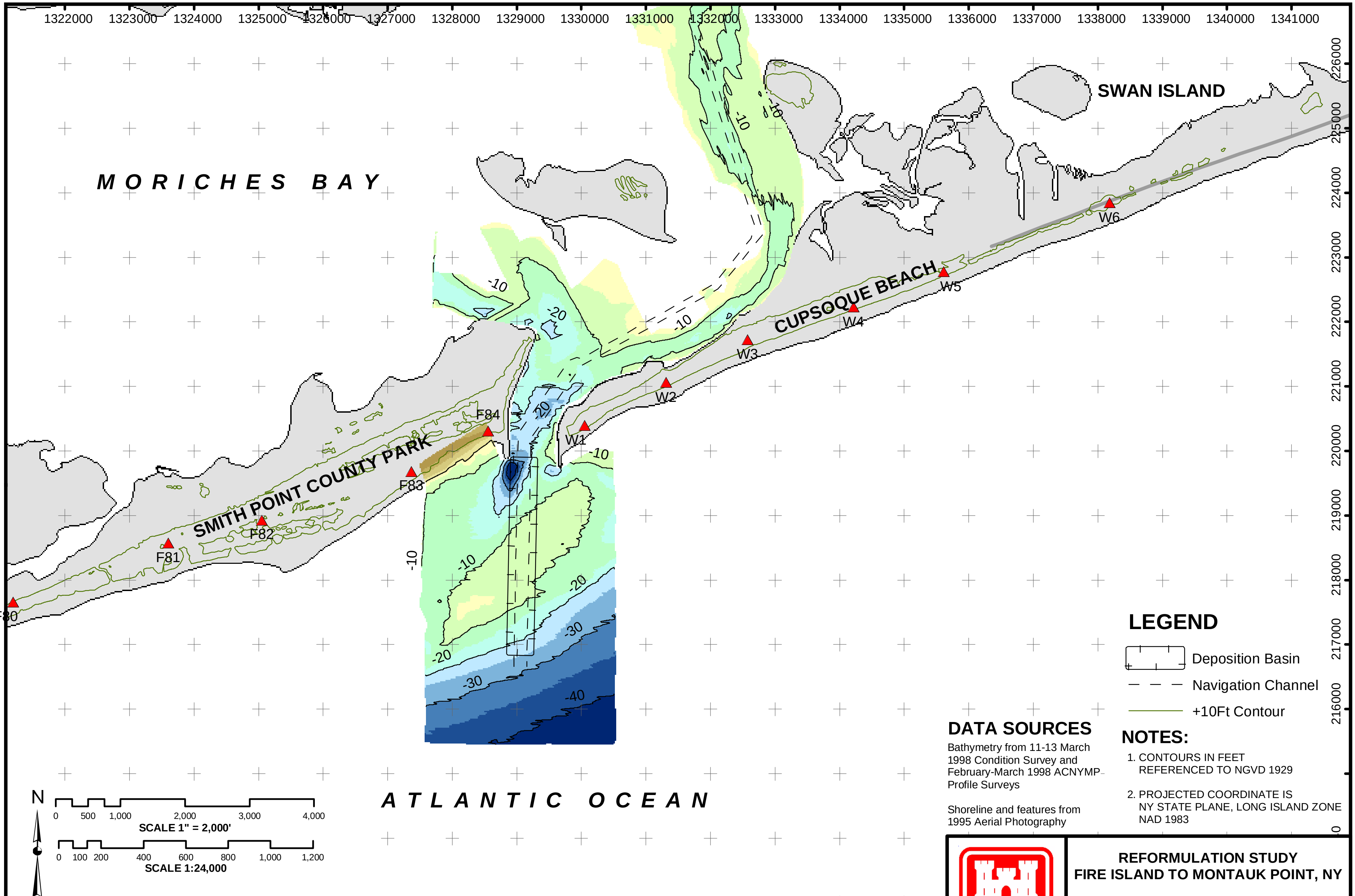
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 5-23

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Moriches_Depth.mxd



DATA SOURCES

Bathymetry from 11-13 March 1998 Condition Survey and February-March 1998 ACNYMP Profile Surveys

Shoreline and features from 1995 Aerial Photography

NOTES:

1. CONTOURS IN FEET REFERENCED TO NGVD 1929
2. PROJECTED COORDINATE IS NY STATE PLANE, LONG ISLAND ZONE NAD 1983

MORICHES INLET BATHYMETRY - SPRING 1998

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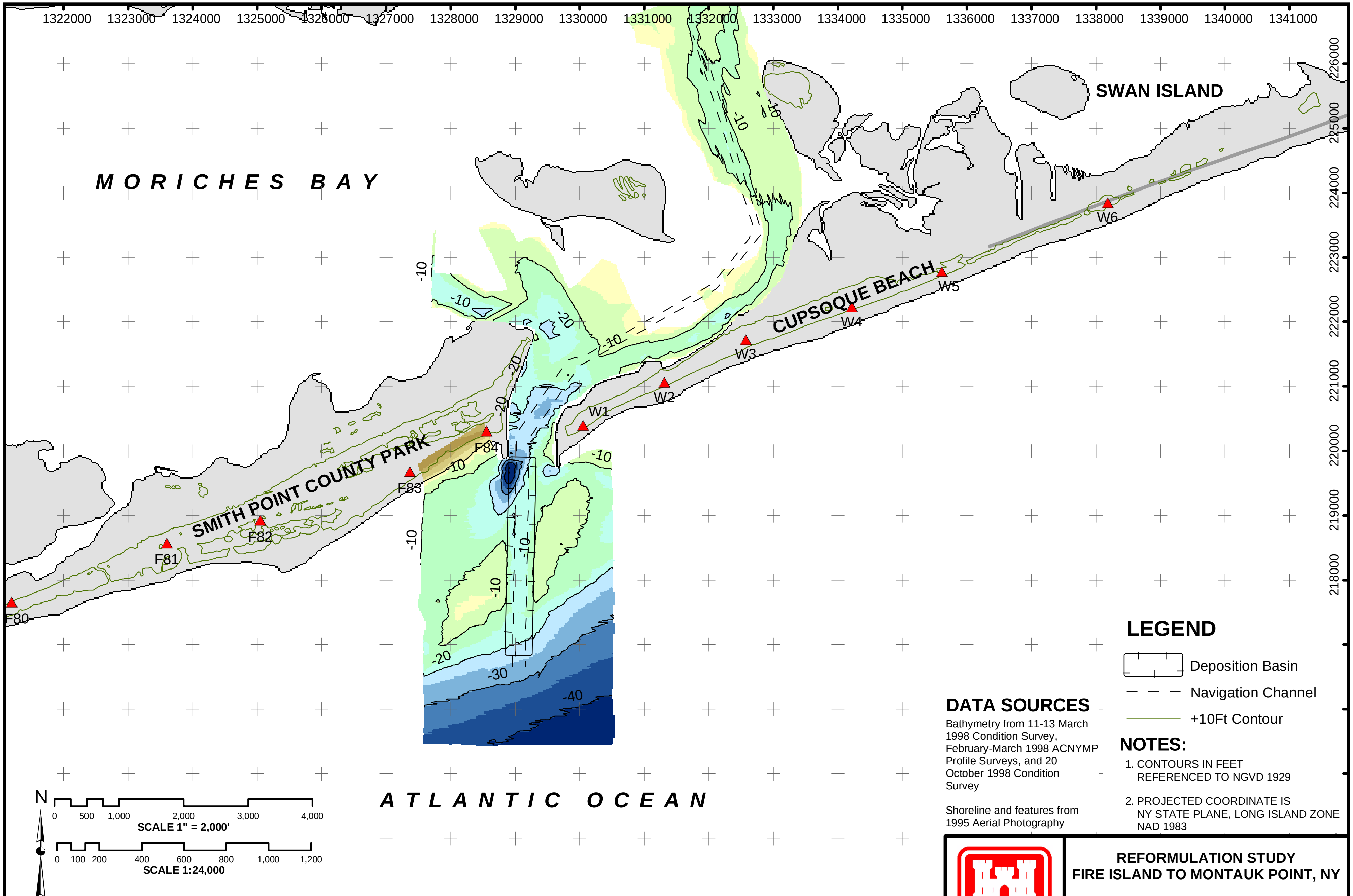
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 5-24

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Moriches_Depth.mxd



LEGEND

Deposition Basin

Navigation Channel

+10Ft Contour

DATA SOURCES

Bathymetry from 11-13 March 1998 Condition Survey, February-March 1998 ACNYMP Profile Surveys, and 20 October 1998 Condition Survey

Shoreline and features from 1995 Aerial Photography

NOTES:

1. CONTOURS IN FEET REFERENCED TO NGVD 1929
2. PROJECTED COORDINATE IS NY STATE PLANE, LONG ISLAND ZONE NAD 1983

MORICHES INLET BATHYMETRY - FALL 1998

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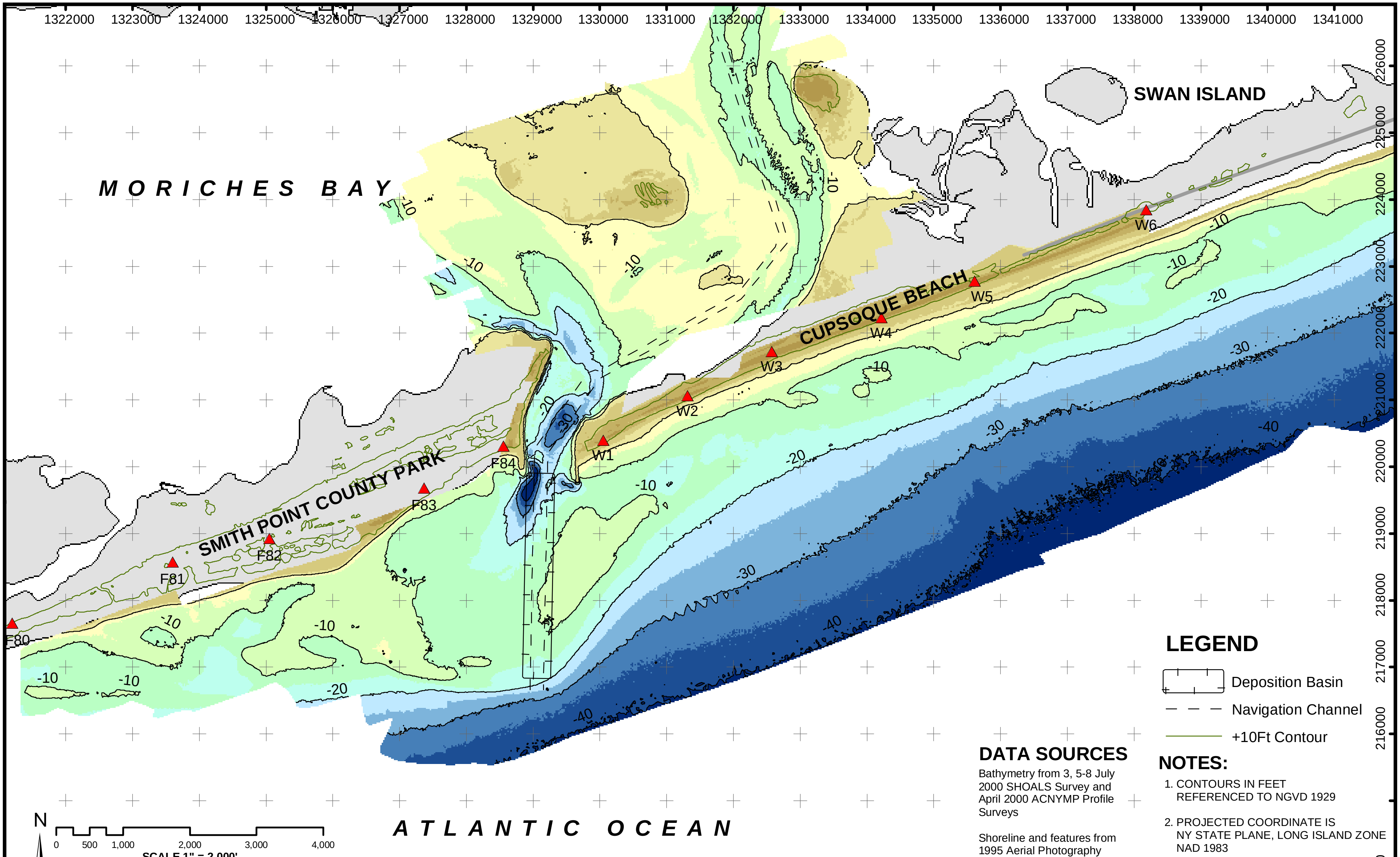
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 5-25

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Moriches_Depth.mxd



MORICHES INLET BATHYMETRY - SPRING-SUMMER 2000

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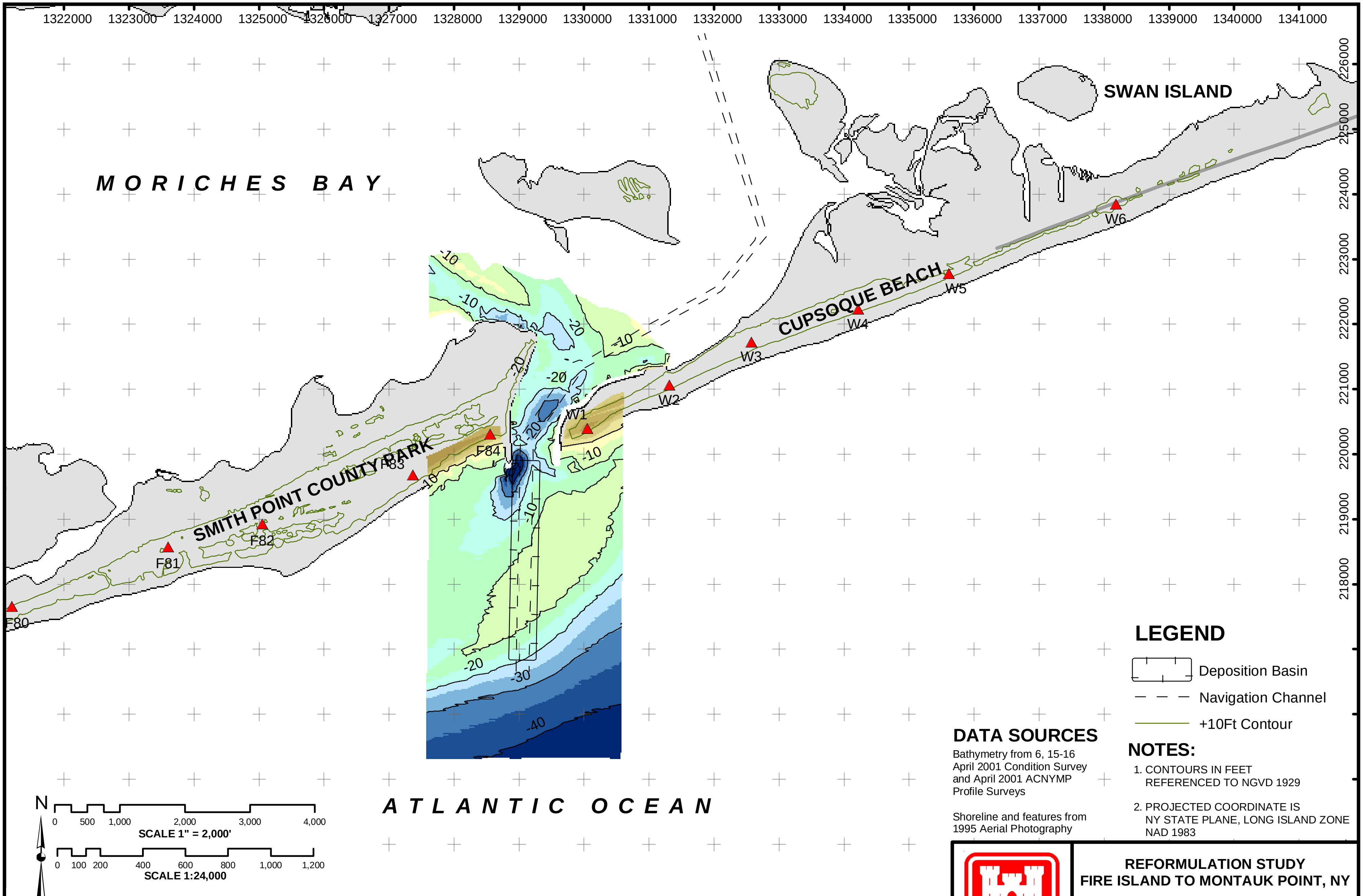
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 5-26

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Moriches_Depth.mxd



DATA SOURCES

Bathymetry from 6, 15-16 April 2001 Condition Survey and April 2001 ACNYMP Profile Surveys

Shoreline and features from 1995 Aerial Photography

NOTES:

1. CONTOURS IN FEET REFERENCED TO NGVD 1929
2. PROJECTED COORDINATE IS NY STATE PLANE, LONG ISLAND ZONE NAD 1983

MORICHES INLET BATHYMETRY - SPRING 2001

DEPARTMENT OF THE ARMY
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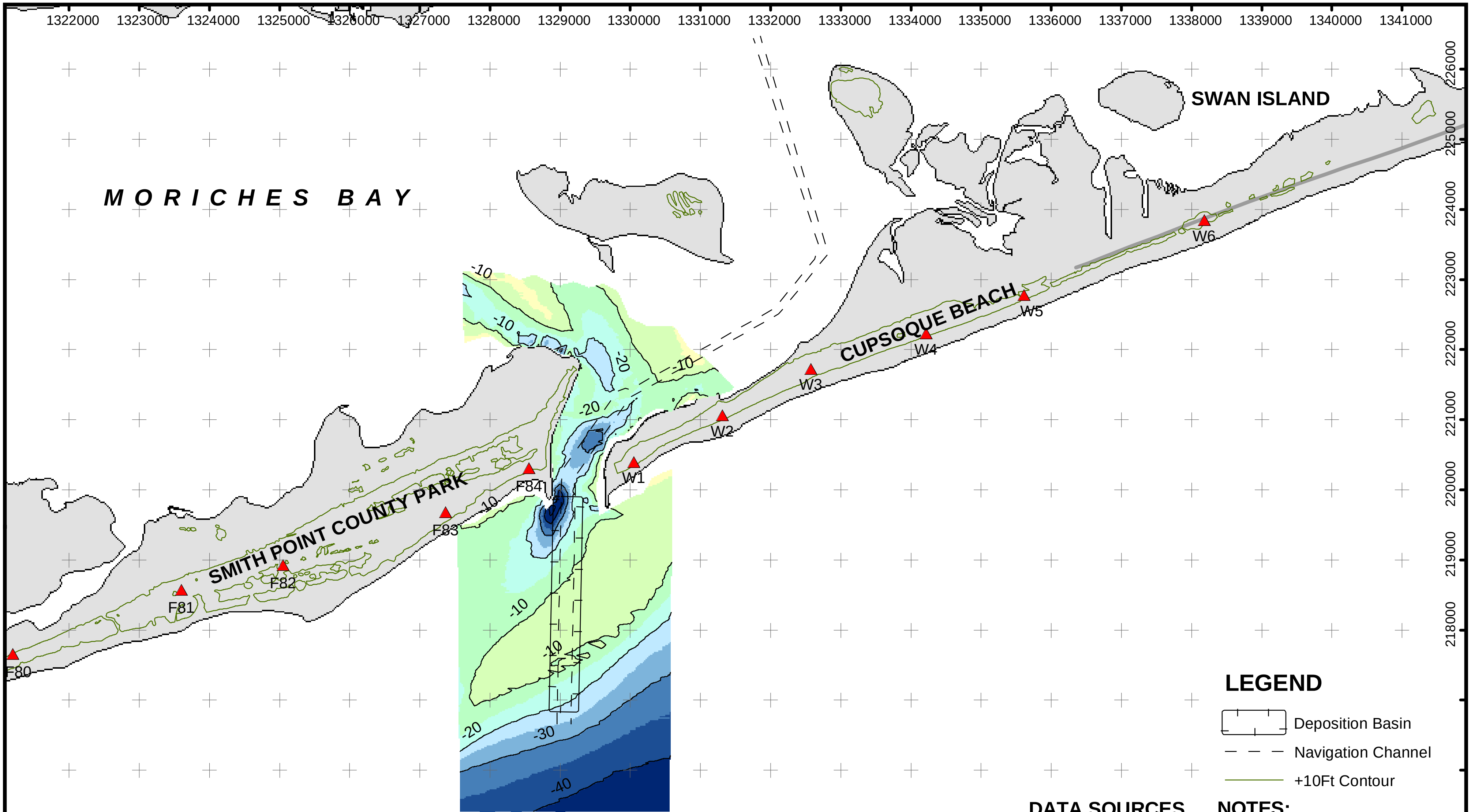
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 5-27

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Moriches_Depth.mxd



LEGEND

Deposition Basin
Navigation Channel
+10Ft Contour

DATA SOURCES

Bathymetry from 6-7 April 2002 Condition Survey

Shoreline and features from 1995 Aerial Photography

NOTES:

1. CONTOURS IN FEET REFERENCED TO NGVD 1929
2. PROJECTED COORDINATE IS NY STATE PLANE, LONG ISLAND ZONE NAD 1983

N

0 500 1,000 2,000 3,000 4,000
SCALE 1" = 2,000'

0 100 200 400 600 800 1,000 1,200
SCALE 1:24,000

MORICHES INLET BATHYMETRY - SPRING 2002

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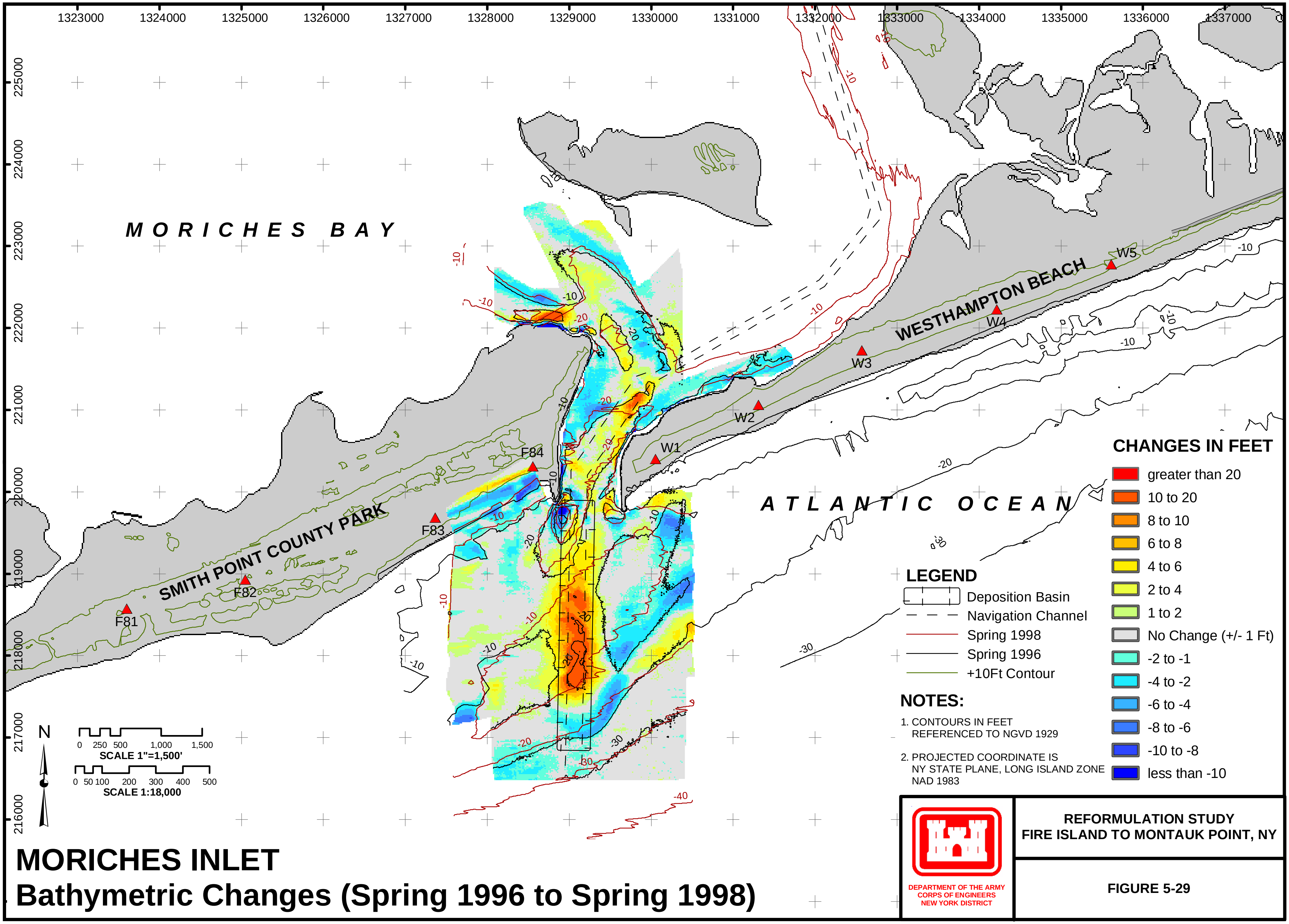
NEW YORK DISTRICT

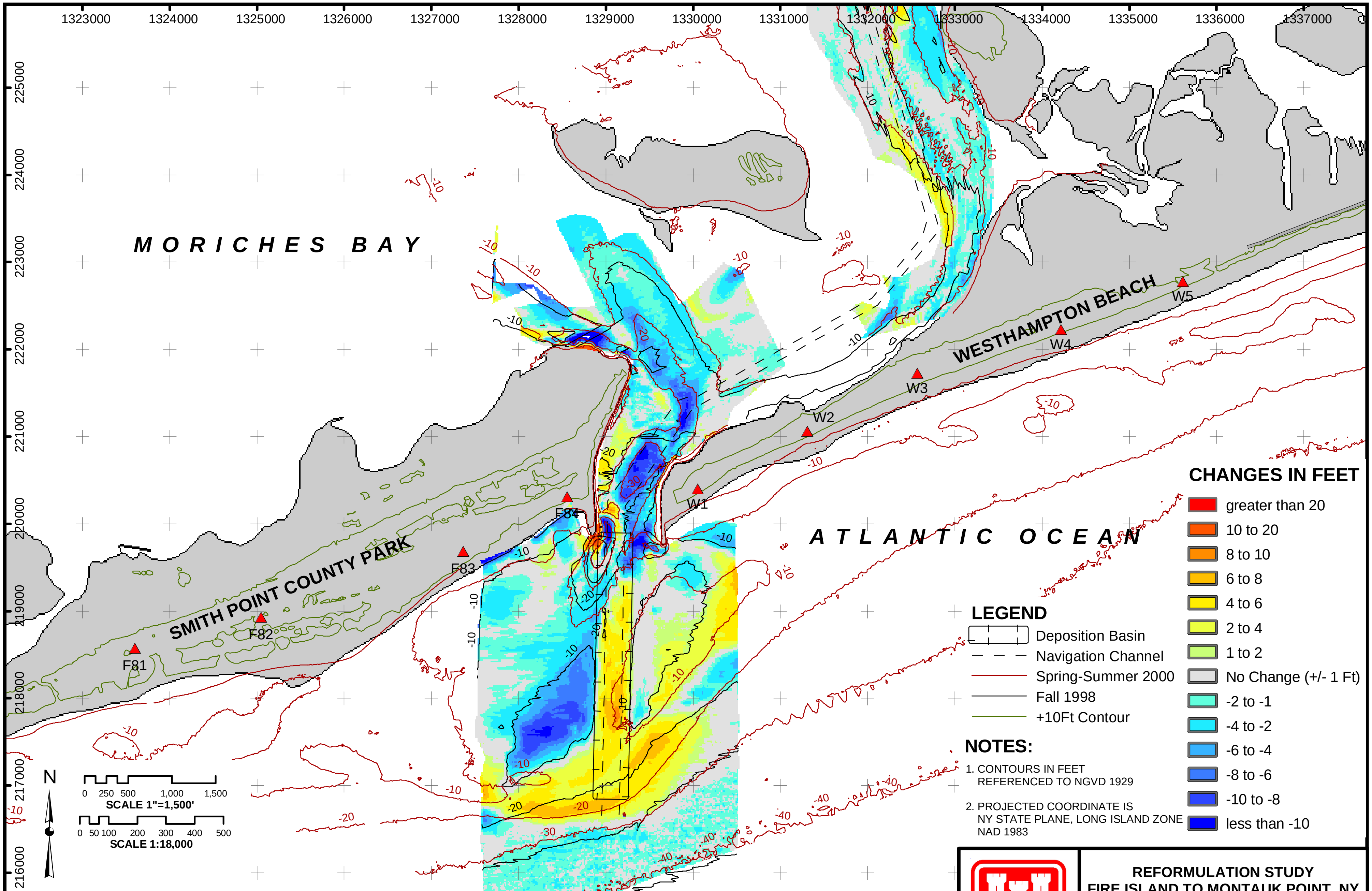
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 5-28

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Moriches_Depth.mxd



Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Moriches_Diff.mxd



MORICHES INLET
Bathymetric Changes (Fall 1998 to Spring-Summer 2000)

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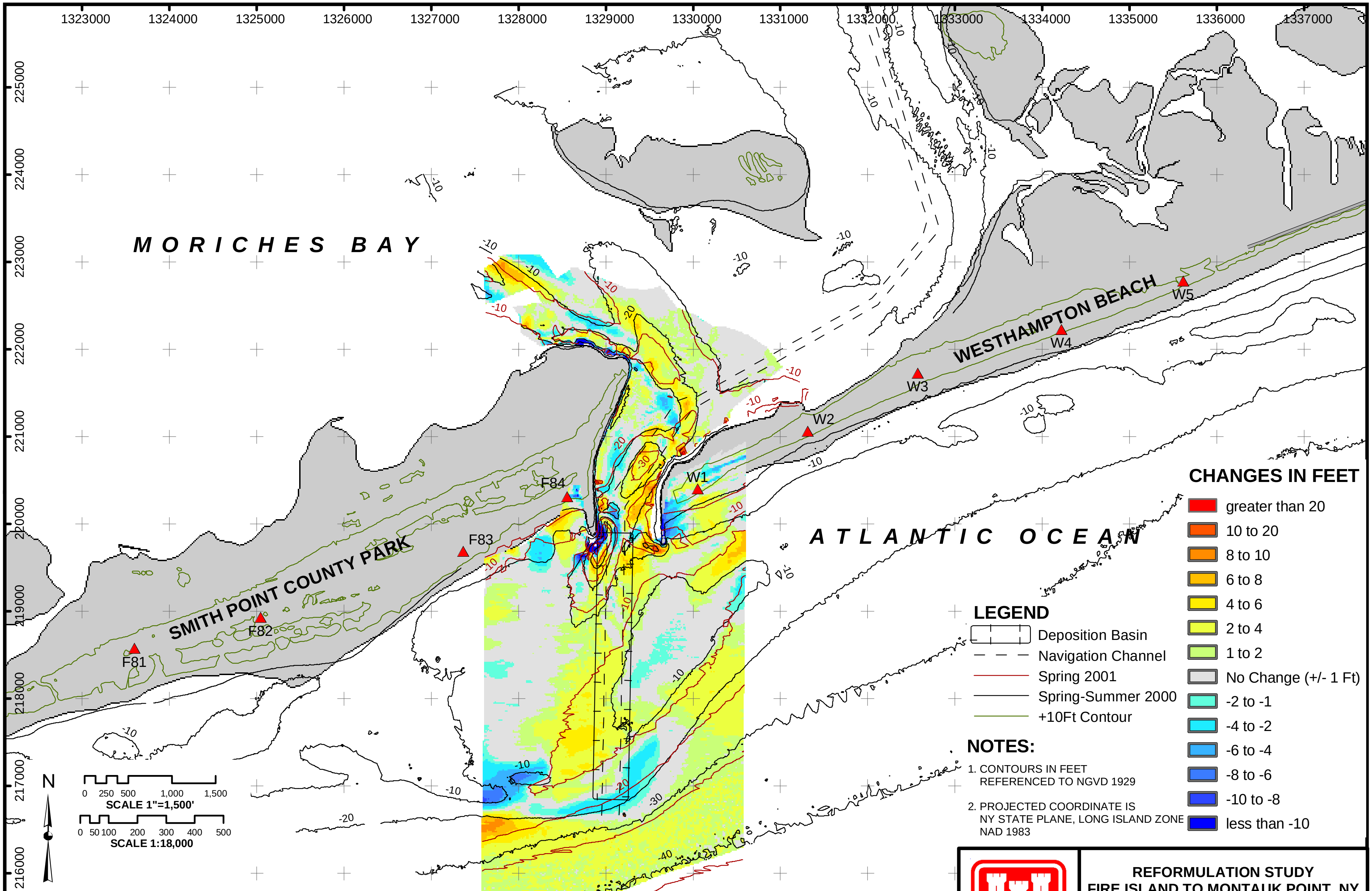
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 5-30

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Moriches_Diff.mxd



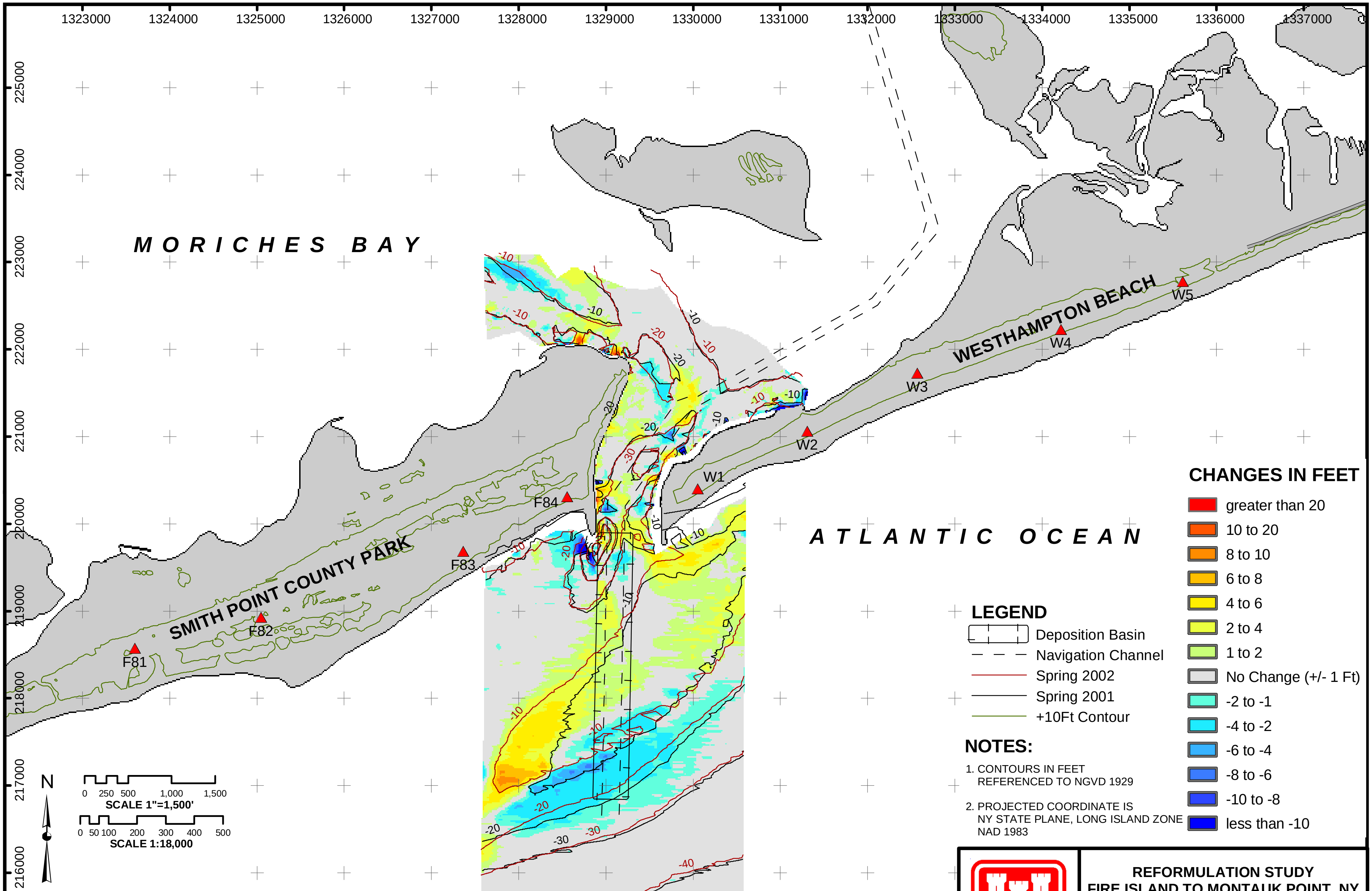
MORICHES INLET

Bathymetric Changes (Spring-Summer 2000 to Spring 2001)

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 5-31

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Moriches_Diff.mxd



MORICHES INLET
Bathymetric Changes (Spring 2001 to Spring 2002)

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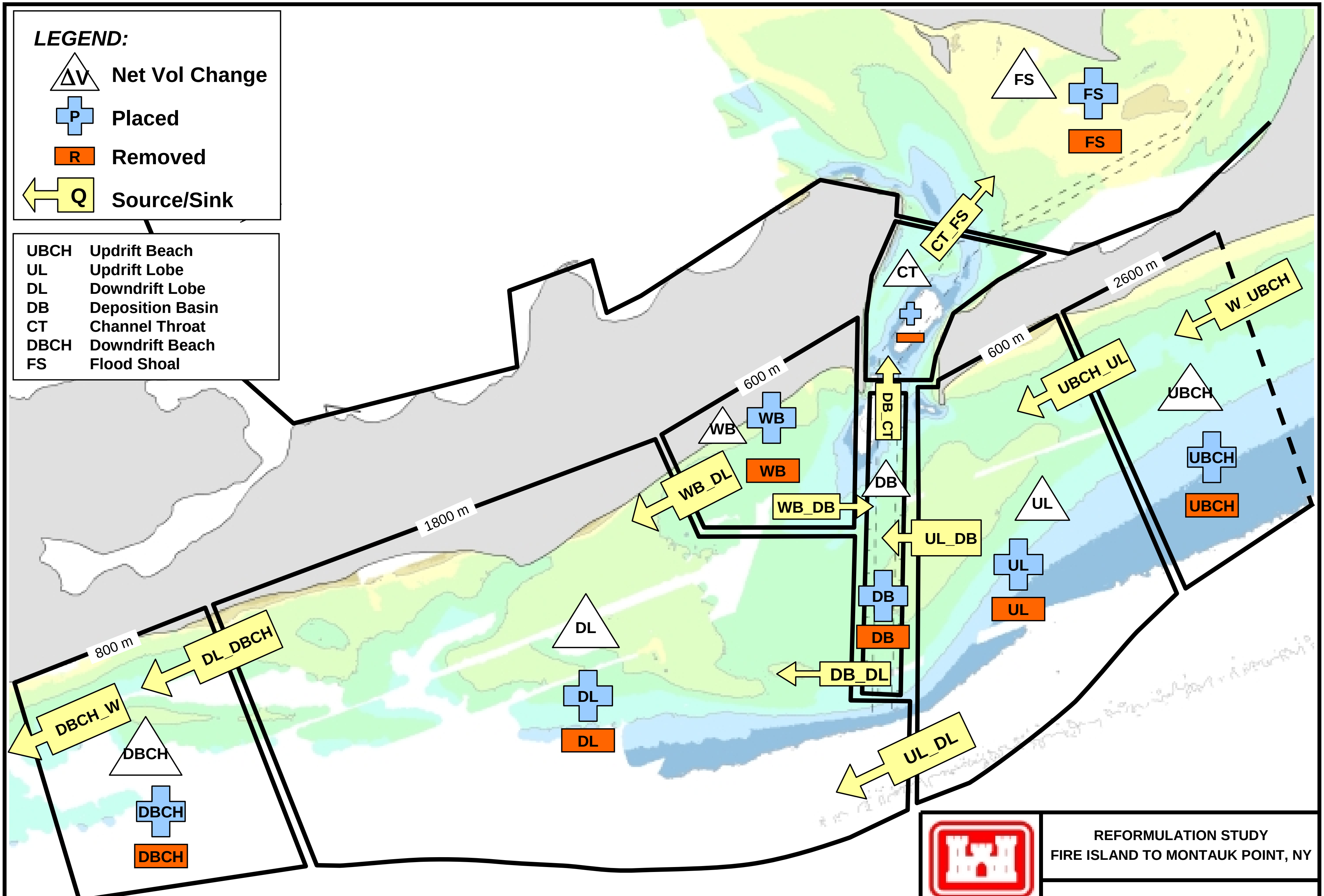
NEW YORK DISTRICT

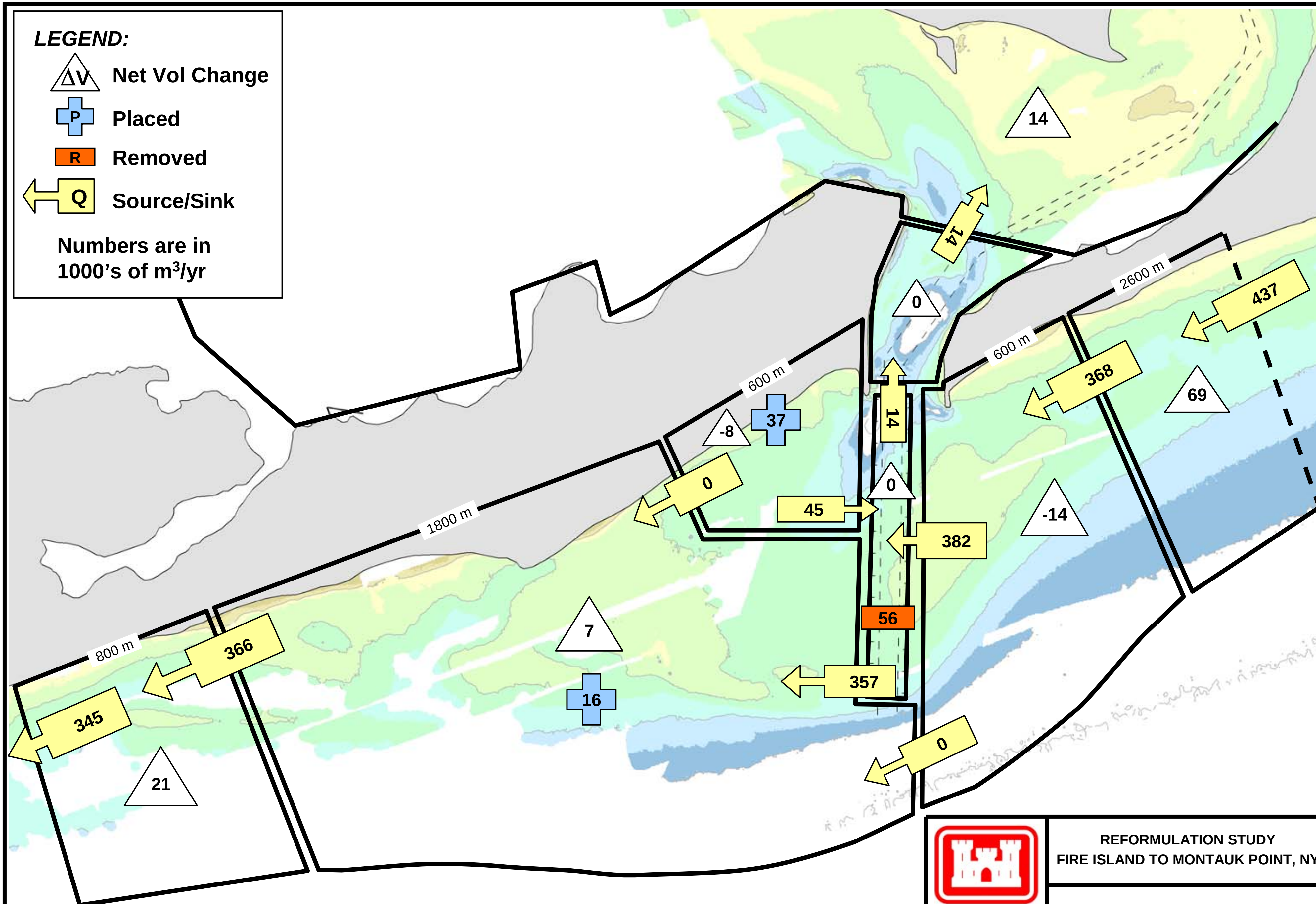
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 5-32

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Moriches_Diff.mxd



Moriches Inlet Conceptual Sediment Budget



Moriches Inlet *Recent (1995-2001)* Sediment Budget

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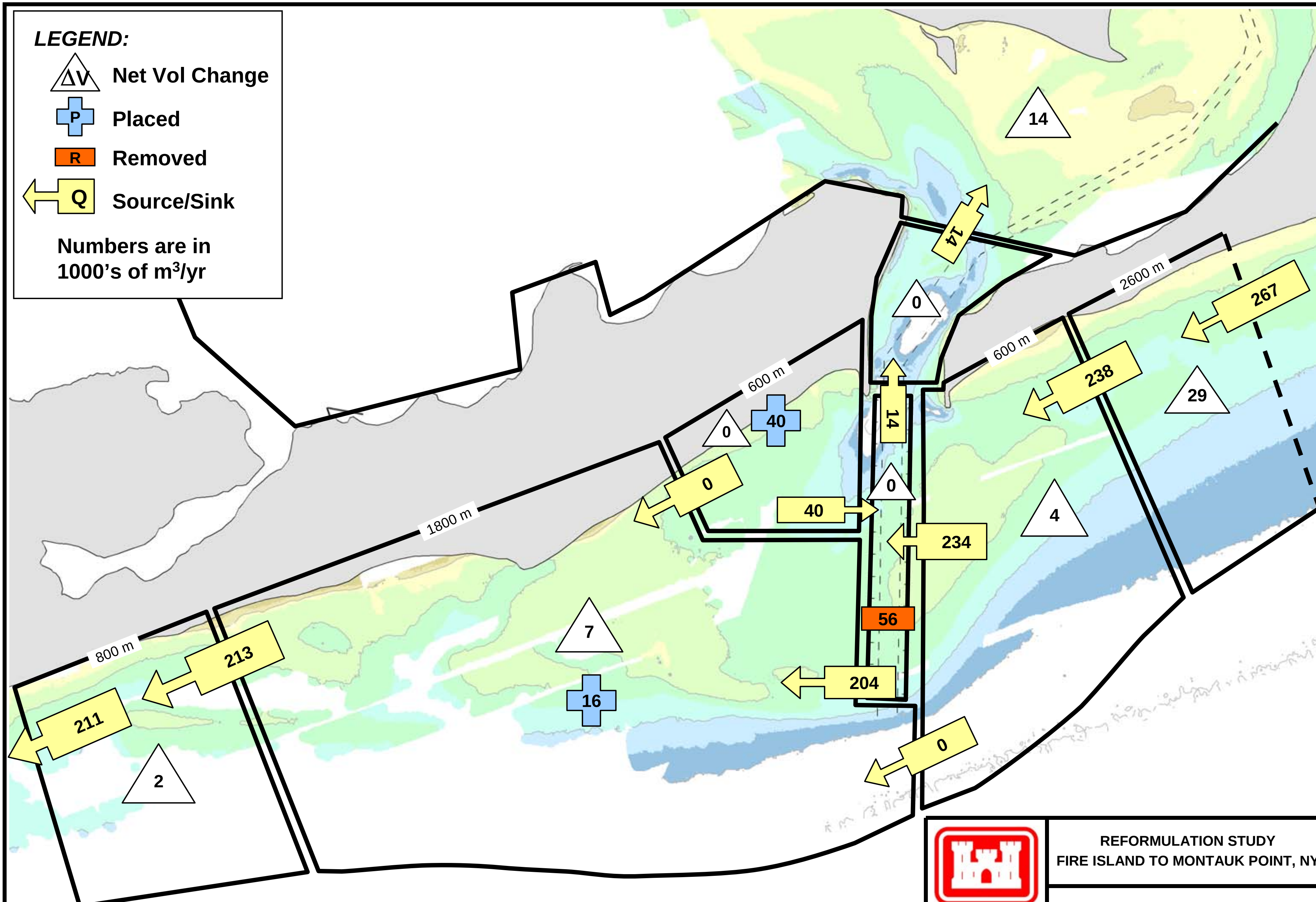
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

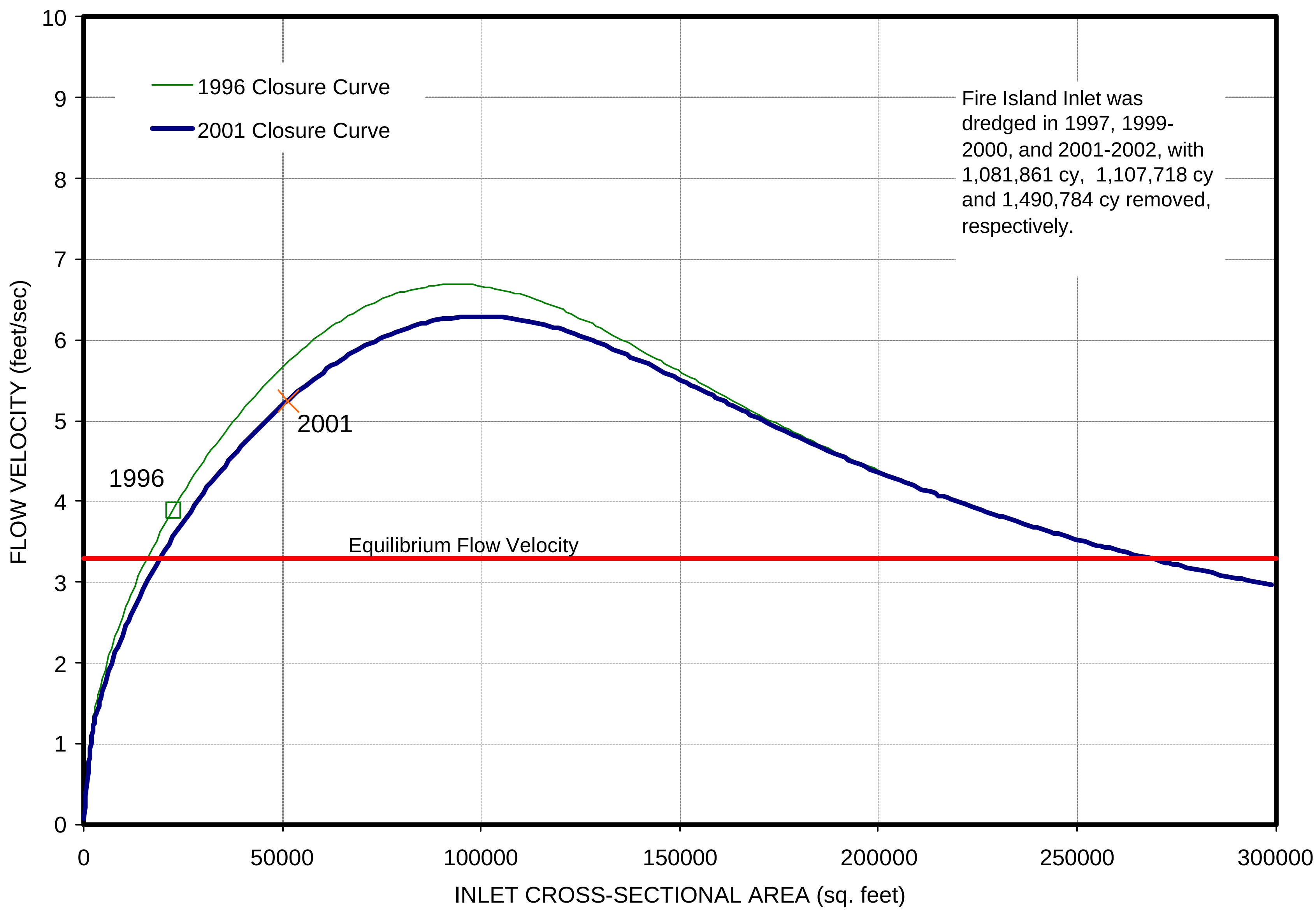
FIGURE 5-34



REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 5-35

Moriches Inlet *Existing (c. 2001)* Sediment Budget



FIRE ISLAND INLET - Stability Analysis, Existing Conditions

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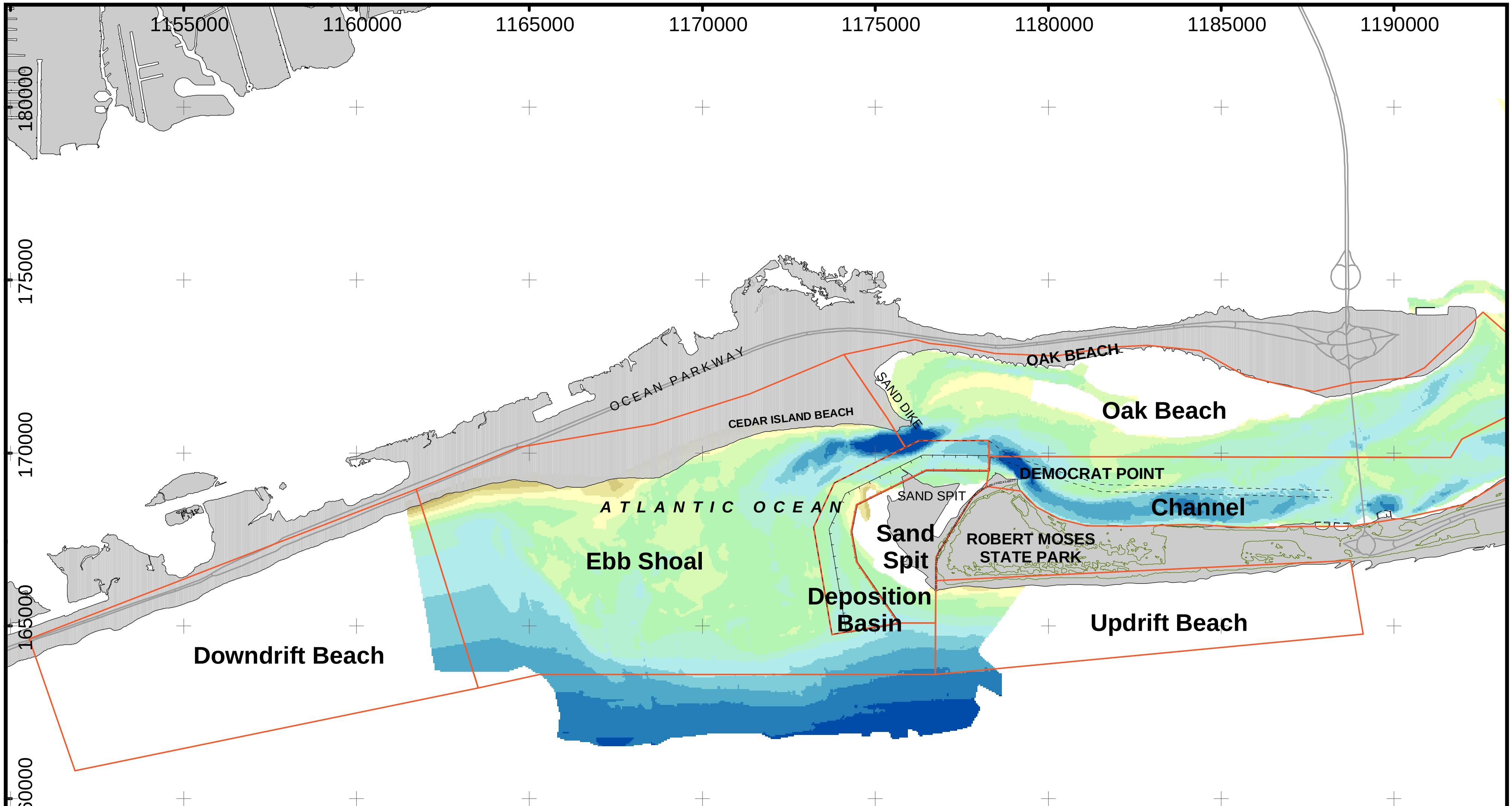
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 5-36

FILE: 3982-28 FIMP/Reports/Report2002/FIIStability.ppt



LEGEND

Deposition Basin
+10Ft Contour
Navigation Channel
Federal Jetty

NOTES:

CONTOURS IN FEET
REFERENCED TO NGVD 1929

PROJECTED COORDINATE USE
NY STATE PLANE, LONG ISLAND ZONE
NAD 1983

FIRE ISLAND INLET Sediment Budget Cells

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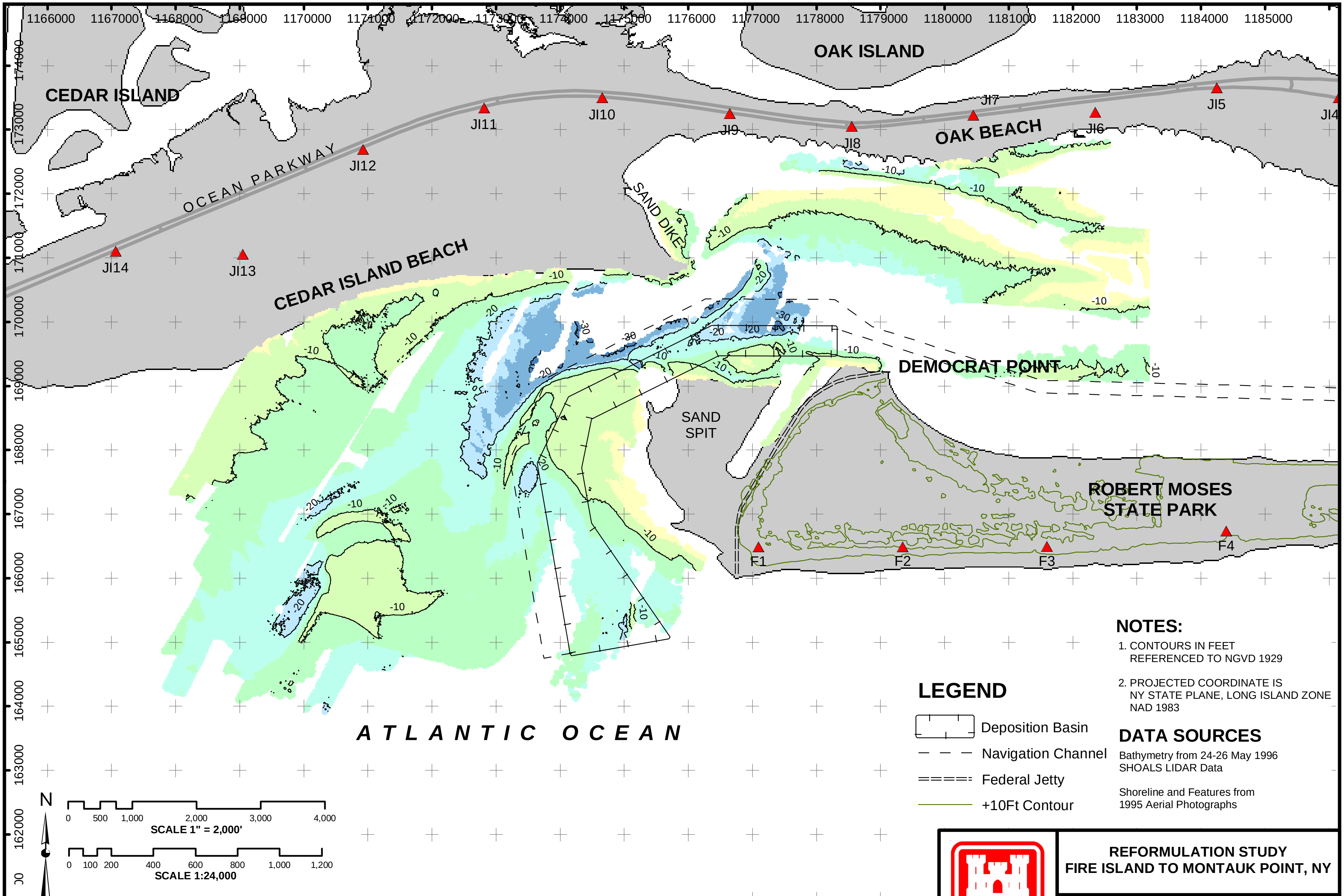
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 5-37

Filename: 3982-28/ReportDec2002/FireIsl_Cells.mxd



NOTES:

1. CONTOURS IN FEET REFERENCED TO NGVD 1929
2. PROJECTED COORDINATE IS NY STATE PLANE, LONG ISLAND ZONE NAD 1983

LEGEND

- Deposition Basin
- Navigation Channel
- Federal Jetty
- +10Ft Contour

DATA SOURCES

Bathymetry from 24-26 May 1996 SHOALS LIDAR Data

Shoreline and Features from 1995 Aerial Photographs

FIRE ISLAND INLET BATHYMETRY - May 1996

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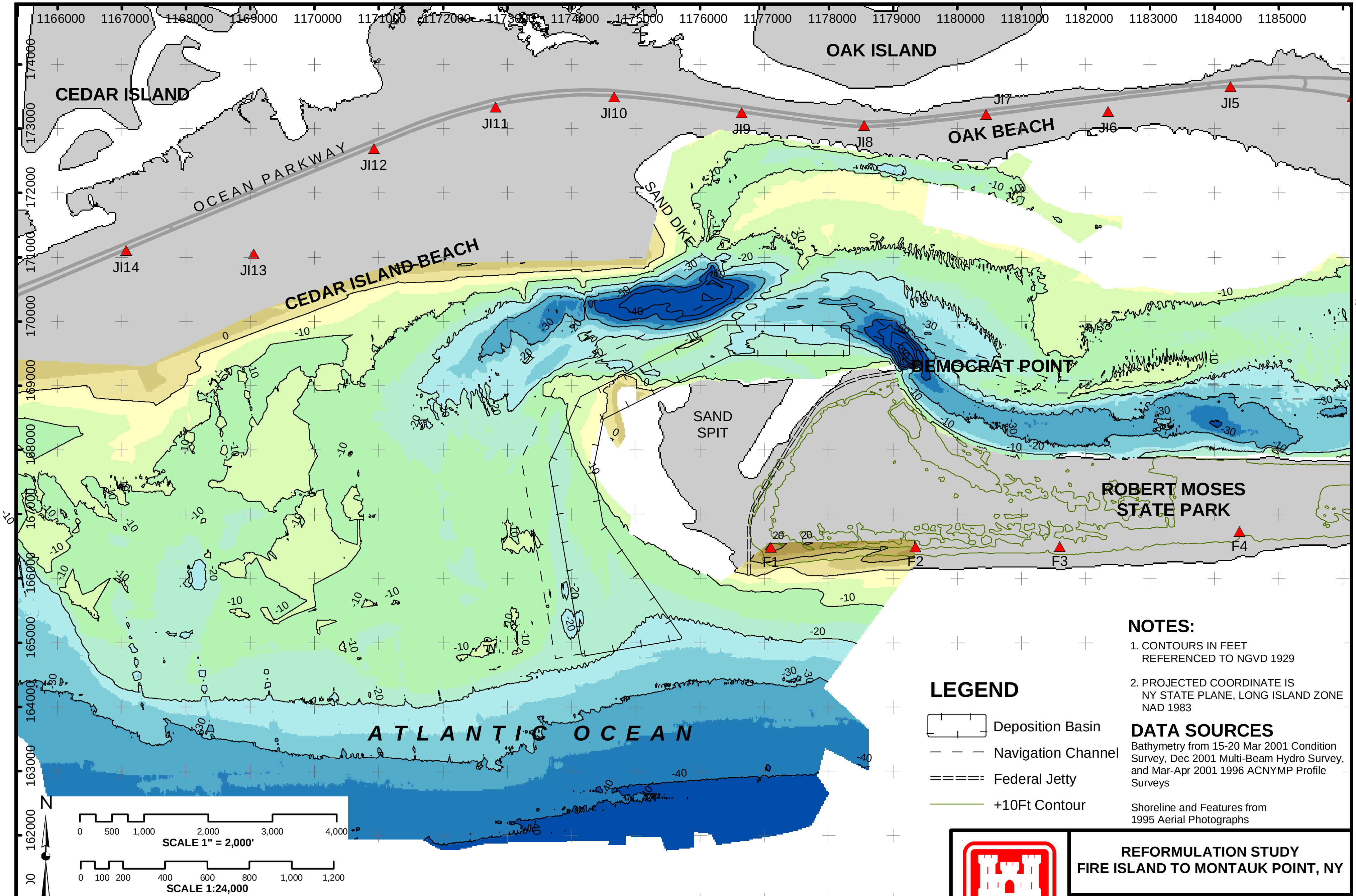
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 5-38

Filename: GIS/3982 FIMP/3982-23/Reports_MXDFiles/FireIsl_Depth.mxd



FIRE ISLAND INLET BATHYMETRY - 2001

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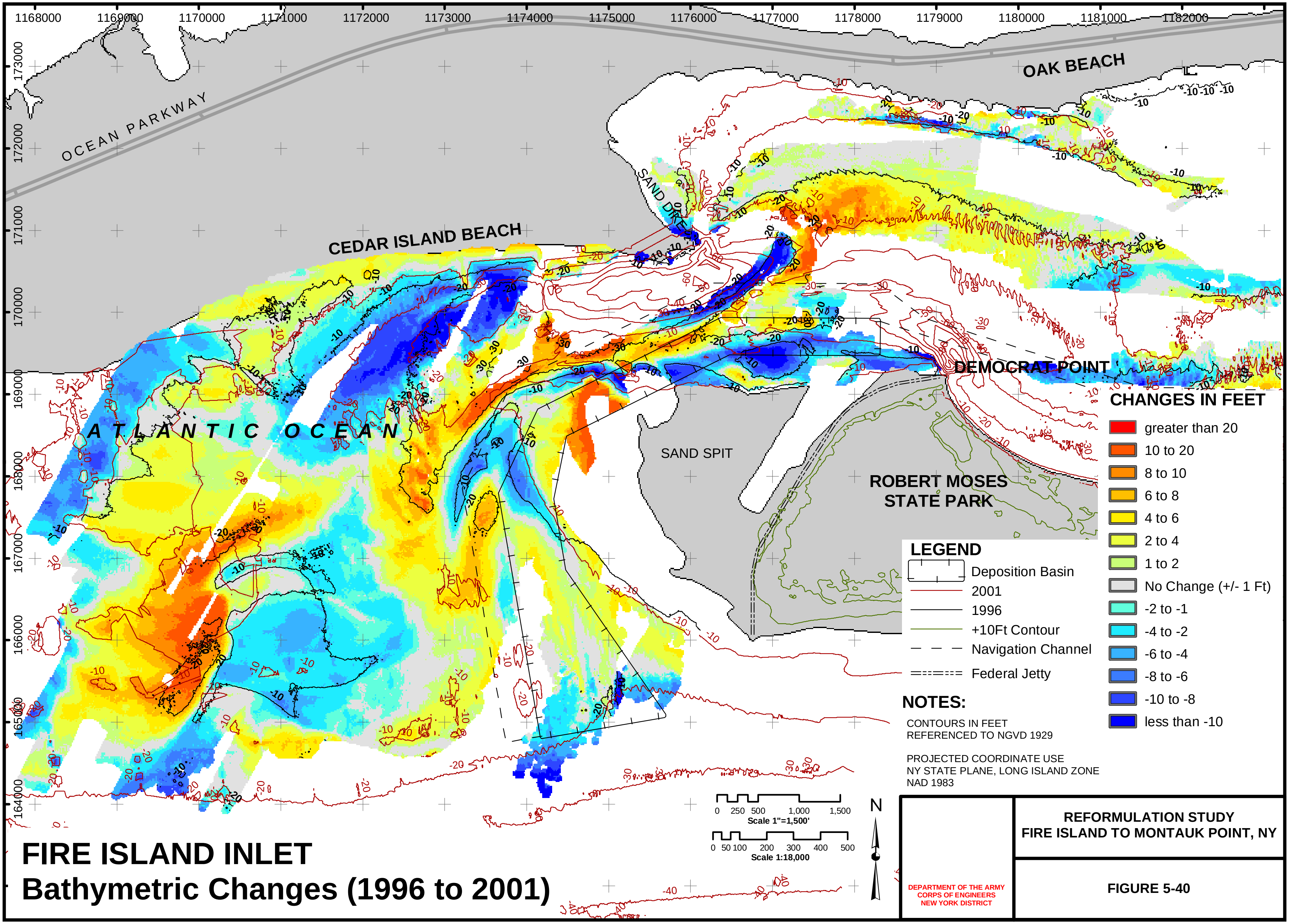
FIRE ISLAND TO MONTAUK POINT, NY

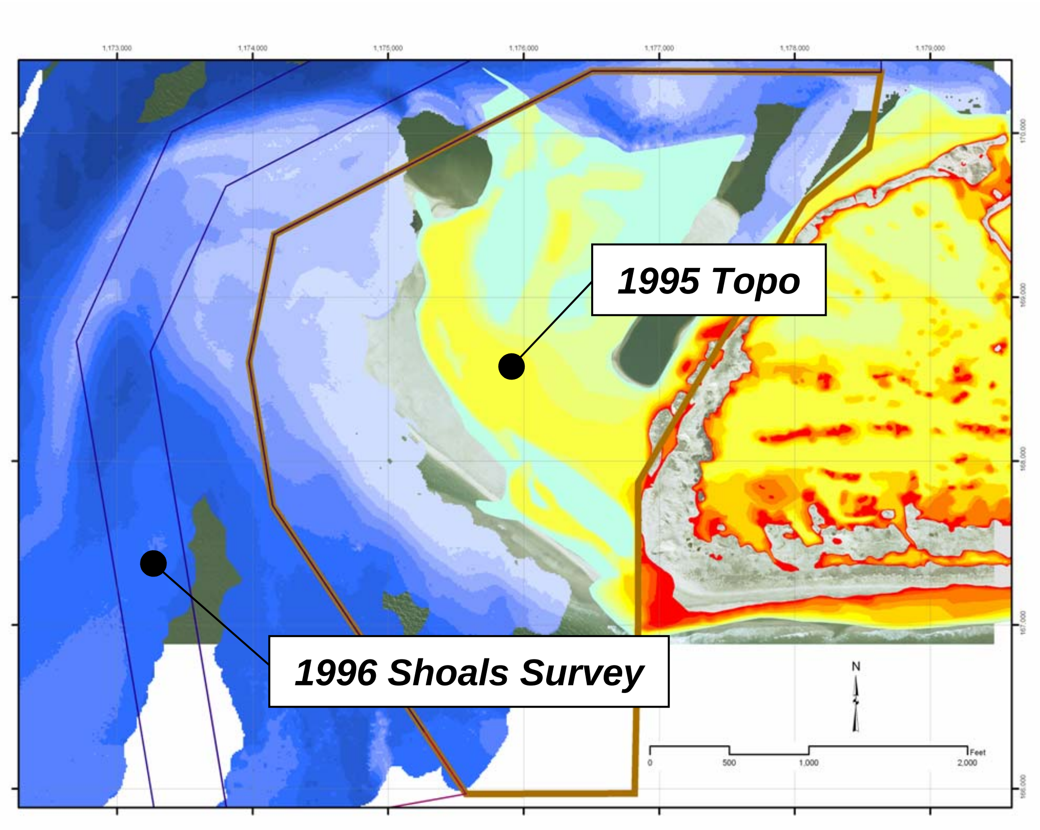
FIGURE 5-39

Filename: GIS/3982 FIMP/3982-23/Reports_MXDFiles/FireIsl_Depth.mxd

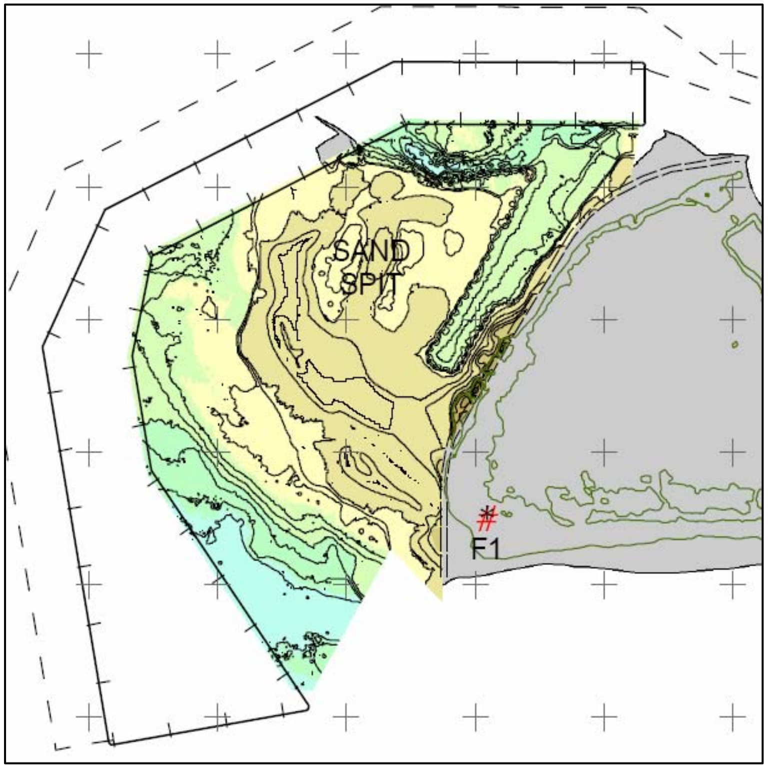


Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/FireIsl_Diff.mxd

Available Individual Datasets



Synthetic Topo/Bathy Generated in GIS



Fire Island Inlet – Sand Spit Elevations (1995/96)

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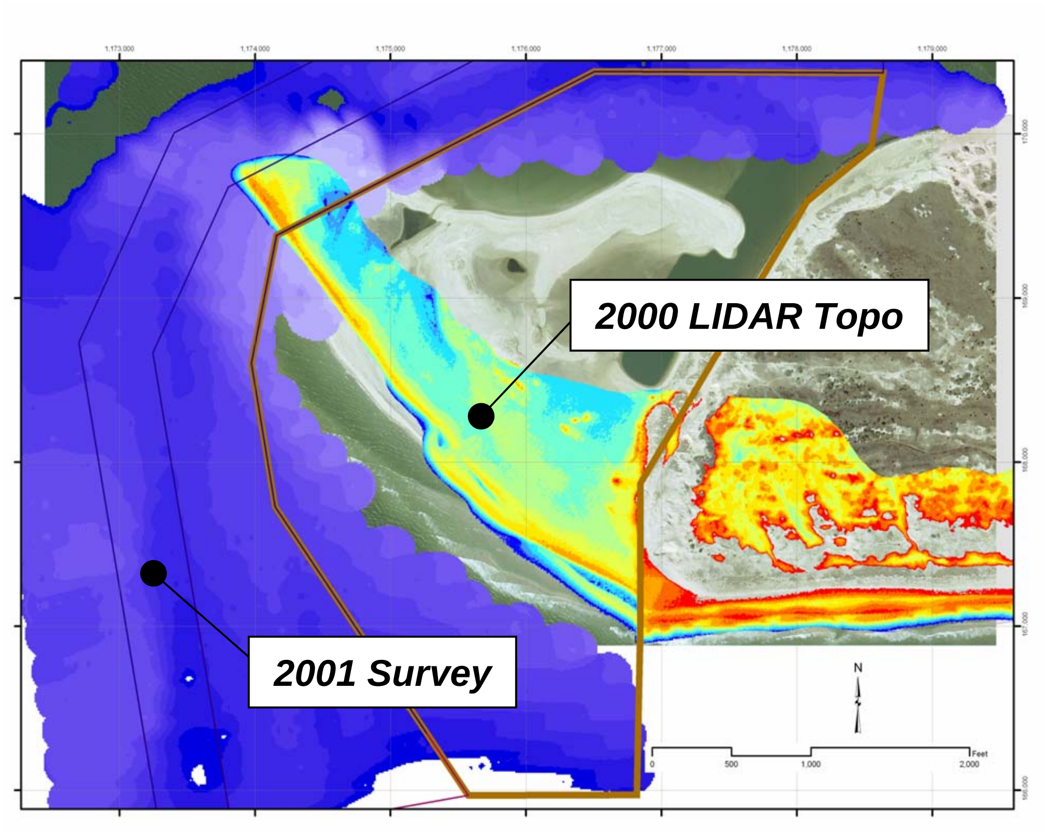
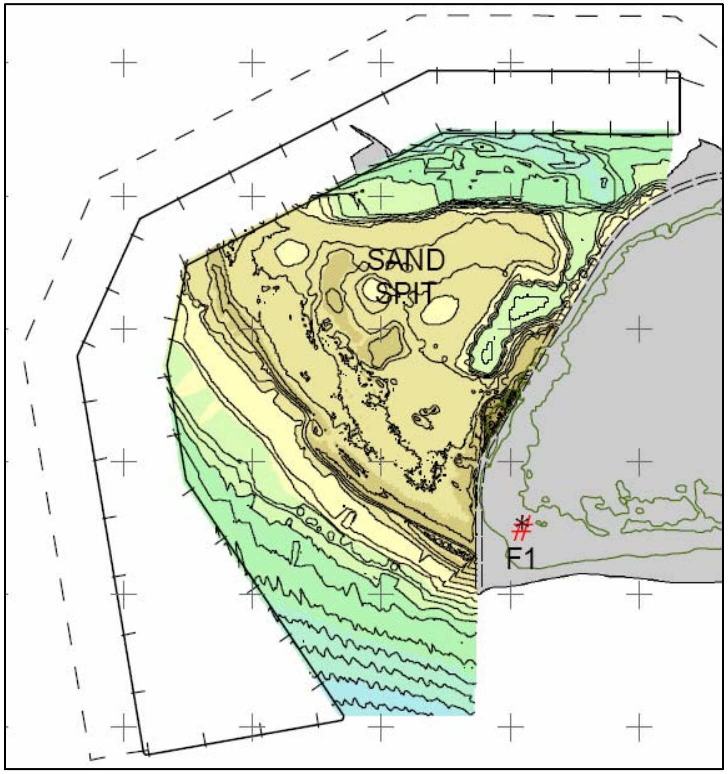
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 5-41

Available Individual Datasets



Synthetic Topo/Bathy Generated in GIS



Fire Island Inlet – Sand Spit Elevations (2000/01)

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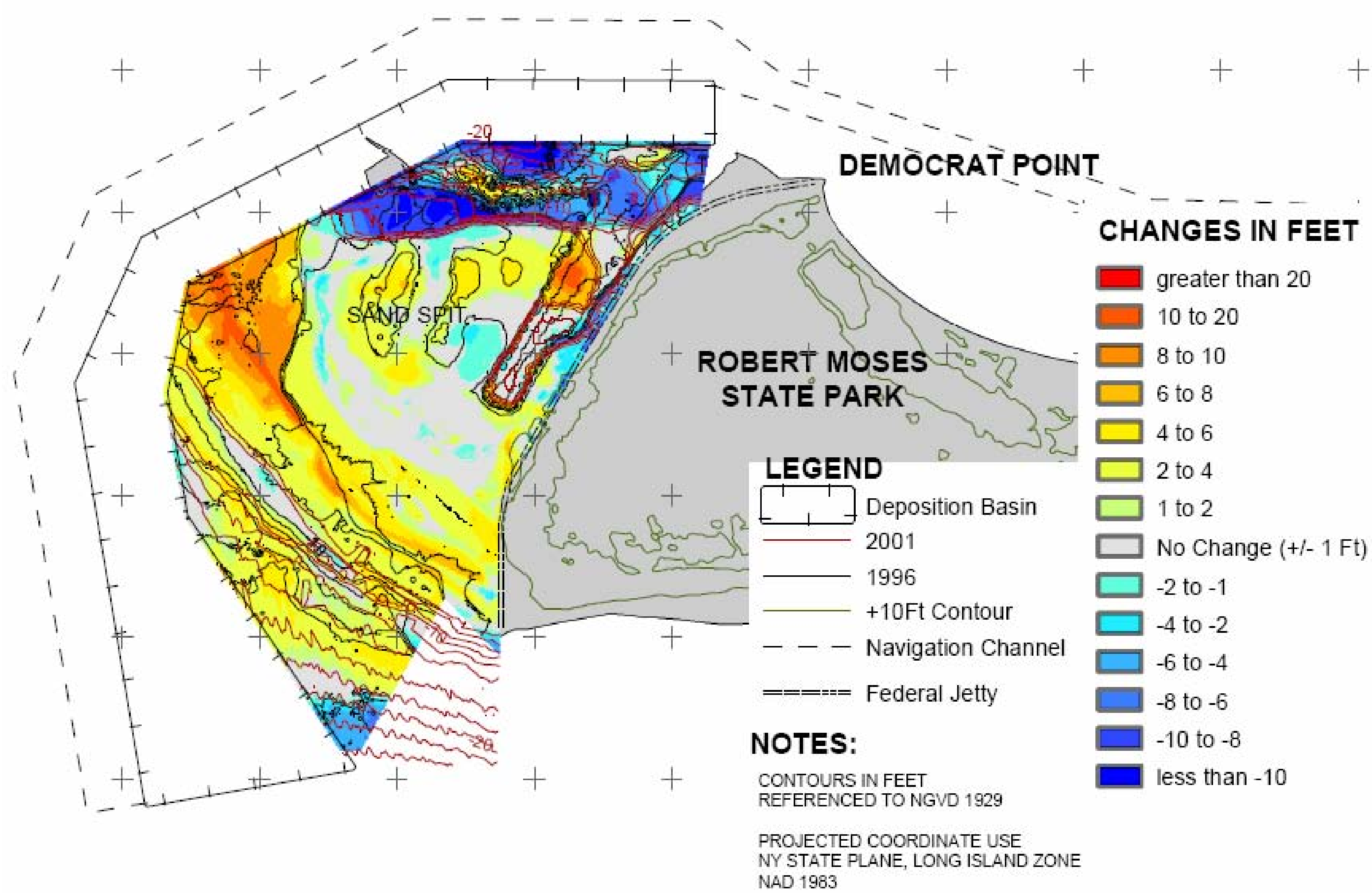
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 5-42



Fire Island Inlet – Sand Spit Changes (95/96-01)

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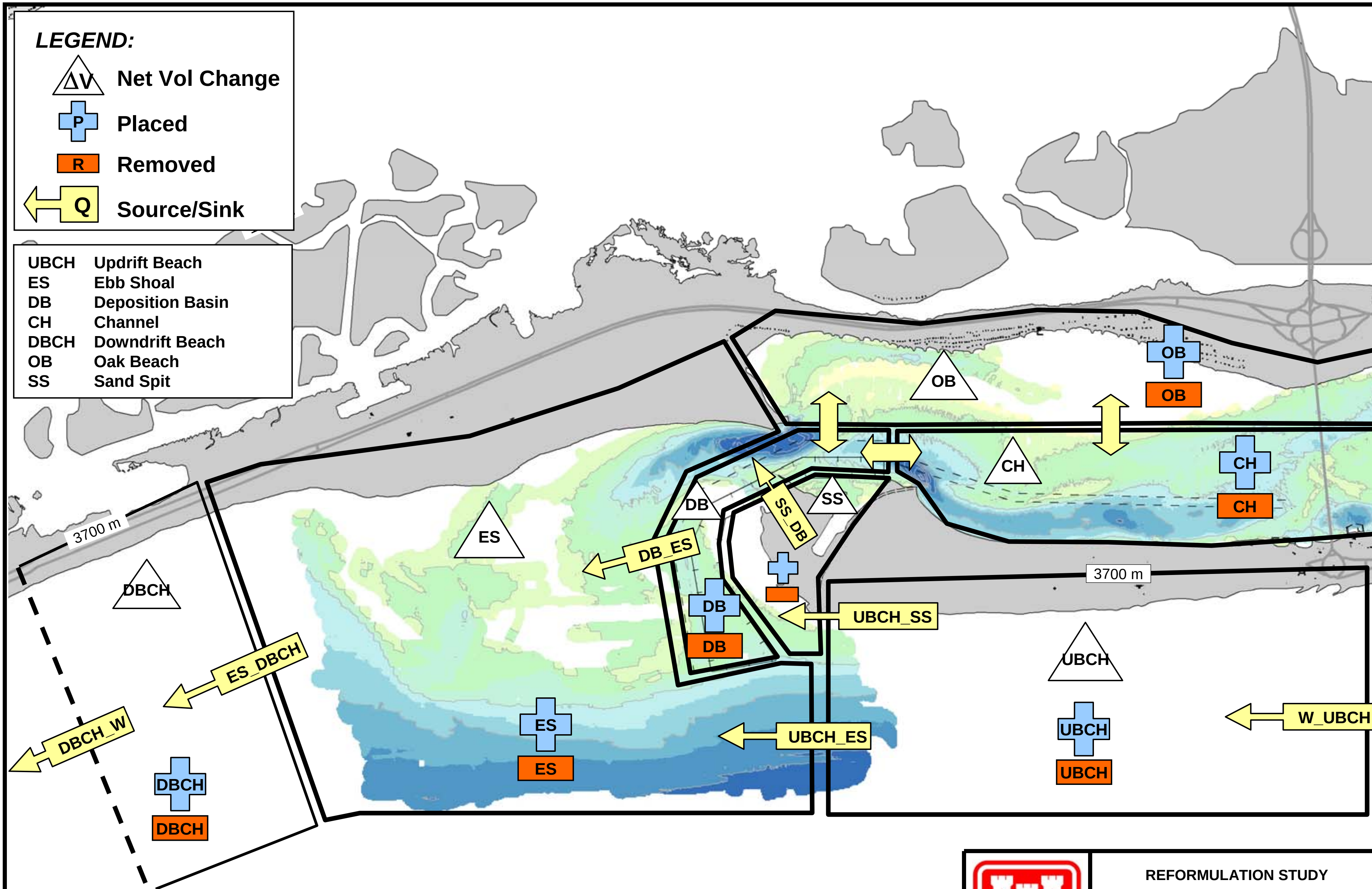
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 5-43



REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

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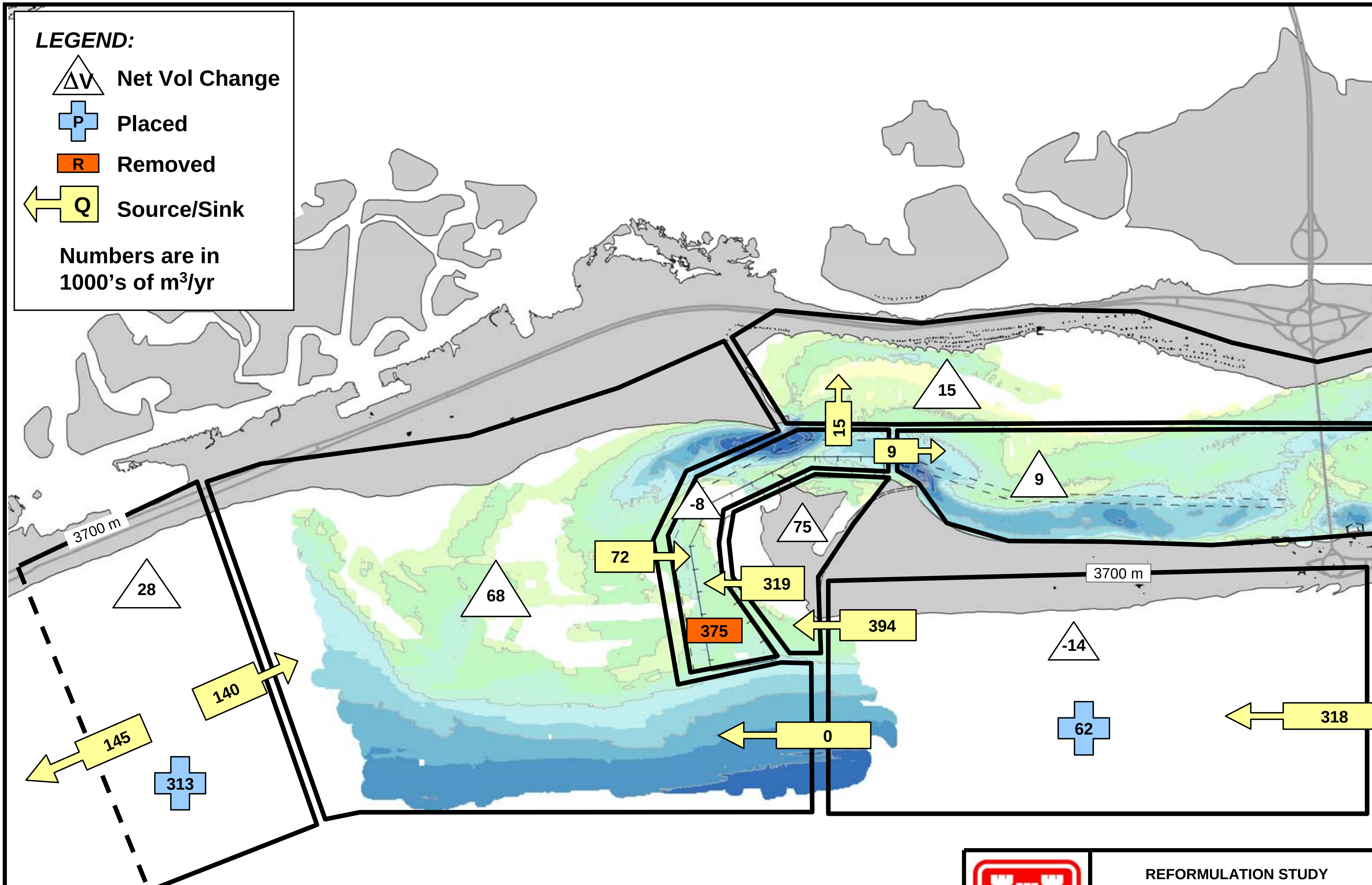
FIGURE 5-44

Fire Island Inlet Conceptual Sediment Budget



REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

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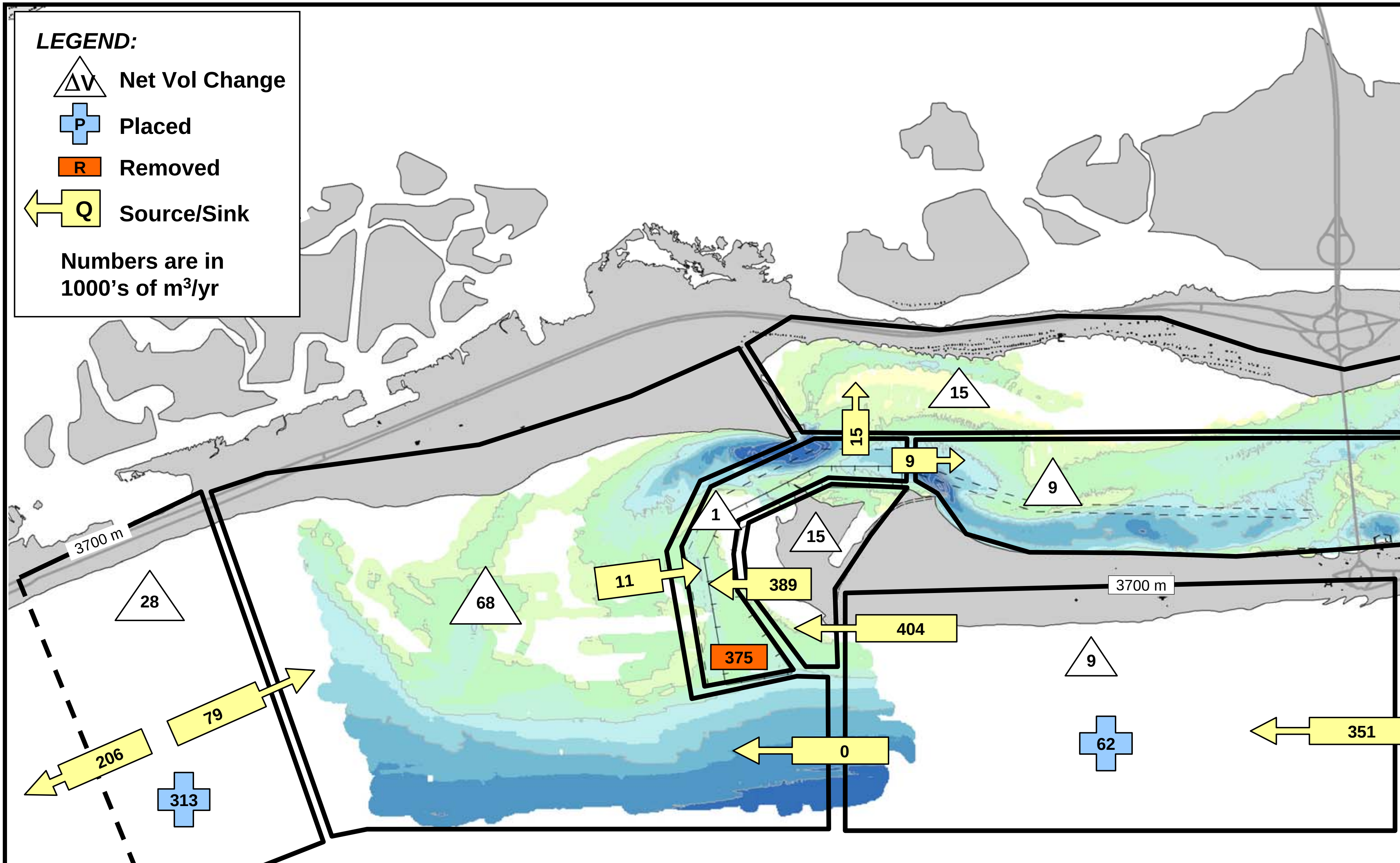
FIGURE 5-45

Fire Island Inlet *Recent (1995-2001)* Sediment Budget



Fire Island Inlet *Existing (c. 2001)* Sediment Budget

6. REGIONAL SEDIMENT BUDGET

This section presents the development of a regional sediment budget for the 133-km shoreline from Fire Island Inlet to Montauk Point. This project area includes sections of mainland (58 km) and barrier island beaches (75 km) and three stabilized inlets. The goals of this task were (1) to develop a sediment budget representative of recent morphological changes and beach/inlet management practices and (2) to expand this analysis by incorporating medium- to long-term (10-30 years) historic trends and ongoing management practices and engineering activities to develop a regional sediment budget representative of *Existing* conditions. This budget will be used to assist in the planning, design, and formulation of shore protection and storm damage reduction measures for the FIMP project area.

6.1 Previous Work

6.1.1 Gravens et al. (1999)

As part of the FIMP Reformulation Study, Gravens et al. (1999) developed a *Historical* sediment budget representative of coastal sediment transport pathways and magnitudes during the 1979 to 1995 period. In addition, the authors developed an *Existing* sediment budget reflecting littoral transport processes along the barrier island and inlets as of the time of their study (c. 1999). Both budgets were based on an analysis of the mainland and barrier island shorelines within the FIMP project area conducted by the Coastal Hydraulics laboratory (CHL), and an analysis of the three inlets contained in the FIMP project area conducted by Moffatt and Nichol (M&N) (see USACE-NAN, 1998). The authors applied shoreline position data available in 1979, 1983 and 1995 to derive estimates of volume change for each sediment budget cell by assuming the shoreline translated parallel to itself over the active profile depth. The latter is measured as the difference in elevation between the top of the seaward-most active berm and the depth of closure. Gravens et al. used profile data in 1979 and 1995 to compute an active profile depth of 10.5 (34.4 feet) as representative of the beach profiles within FIMP. The two budgets are referred to herein as the *Historical (1979-95)* and *Existing (c. 1999)* sediment budgets.

Gravens et al. divided the 133-km project shoreline extending from Fire Island Inlet to Montauk Point into three major morphological reaches (Figure 6-1): (1) Montauk Reach extending from Montauk Point in the east to Shinnecock Inlet in the west (58.1 km), (2) Westhampton Reach extending from Shinnecock Inlet to Moriches Inlet (24.8 km), (3) Fire Island Reach extending from Moriches Inlet to Fire Island Inlet (49.5 km). The Montauk Reach (M) is characterized by high bluffs rising more than 25 m above NGVD from Montauk Point to Montauk Beach (budget cell M5), which is located approximately 8 km to the west of Montauk Point. These bluffs, which are formed by a Pleistocene outcropping, are considered to be a source of material to the littoral sediment transport system. The shoreline to the west for about 6 km is characterized by a beach and dune system backed by mainland (budget cell M4). The next 30 km are characterized by a sandy beach backed by mainland and several ponds and small bays which are not typically connected to the ocean, unless during and immediately after storms, or after having been opened by locals to improve water quality (budget cells M3 and M2). The remaining 13 km of the Montauk Reach are characterized by a barrier island beach, which fronts the eastern half of Shinnecock Bay (budget cell M5 and the updrift beach at Shinnecock Inlet).

The westernmost 8.6 km in the Westhampton Reach (downdrift beach cell at Shinnecock Inlet and budget cell W4) include a stretch of barrier island fronting the western half of Shinnecock Bay and the narrow canal that connects Shinnecock Bay and Quantuck Bay. This cell includes the undeveloped area within Shinnecock Inlet Park and the developed communities of Tiana and Hampton Beach. The barrier continues west 2.1 km (budget cell W3) to the start of the Westhampton groin field, a 5.5 km stretch of barrier island (budget cell W2) stabilized between 1965 and 1970 with 15 groins (one additional, short, groin was recently added in 1998 as part of the Westhampton Interim Project). The remaining 5.2 km of barrier island in the Westhampton Reach (budget cell W1 and the updrift beach at Moriches Inlet) include Pikes Beach and Cupsogue County Park.

The eastern portion of Fire Island (roughly 7.2 km including the downdrift beach at Moriches Inlet and budget cell FI3) is characterized by mostly undeveloped barrier island including Smith Point County Park and roughly the eastern two thirds of the Otis Pike Wilderness Area, both part of the Fire Island National Seashore. The next budget cell along central Fire Island (FI2) is roughly 15 km long and it includes the western one third of the Wilderness Area and alternating developed and undeveloped regions of Fire Island from the Watch Hill Visitor Center to Cherry Grove. The remaining 17 km of Fire Island (budget cell FI1 and the updrift beach at Fire Island Inlet) include a relatively continuous stretch of developed barrier island (roughly 8 km from Oakleyville to Kismet) flanked by two undeveloped regions: Sunken Forest to the east and the Fire Island Lighthouse tract and Robert Moses State Park to the west.

The *Historical (1979-95)* and *Existing (c. 1999)* regional sediment budgets are reproduced in Figure 6-2. Conclusions from their study are literally reproduced in the following paragraphs. For a more detailed discussion see Gravens et al. (1999).

The Historical [1979-1995] and Existing [c. 1999] condition sediment budgets provide estimates of net longshore sand transport rates, include engineering activities (beach fill placement and dredging), and sources and sinks representative of the Fire Island to Montauk Point study area. These sediment budgets indicate net LST that fall within accepted ranges as derived by previous researchers and as calculated through independent analyses herein. As compared to earlier sediment budget formulations, differences (such as west of the Westhampton Groin Field) appear reasonable given knowledge of the engineering activities and coastal processes occurring during the time periods representative of the Historical (1979 to 1995) and Existing (~1999) conditions. East- and west-directed components of the net longshore sand transport rate can be derived from the potential longshore sand transport rate calculations, as discussed in Chapter 6 of this report.

Beach fill placement (and/or transfer of littoral material to adjacent beaches) is a significant process and constitutes an important mechanism in maintaining the study area beaches. The majority of the beach fill placement most likely occurs through dredging of the inlets and bays, and placement on the adjacent beaches, in effect, a mechanical bypassing (or backpassing) mechanism. From 1933 to 1979 and 1979 to 1995, the cumulative rate of beach fill placed from Montauk Point to Fire Island was 295,000 and 309,000 cu m/year, respectively. Estimating that only 25 percent of fills placed to close breaches reflects an alongshore movement of littoral material reduced the 1979 to 1995 value to 208,000 cu m/year. Similar values for the 1979 to 1997 time period are 468,000 (total fill) and 357,000 cu m/year (adjusted for breach fill). These rates of beach fill placement are of the same order as estimates of the net longshore sand transport rate at Fire Island Inlet (Taney (1961a,b): 344,000 cu m/year; RPI (1985): 240,000 cu m/year; Kana (1995): 360,000 cu m/year; growth rate of Democrat Point prior to stabilization (this study): 159,000 to 238,000 cu m/year; impoundment rate at Fire Island East jetty (this study): 385,000 cu m/year (high; may include ebb shoal welding)). Thus, on a regional scale, future projects must maintain this nourishment rate to preserve present-day beach conditions.

Shoals and the inner shelf offshore of central Fire Island have been postulated by other researchers as a required source for solving the regional sediment budget. The sediment budgets formulated herein do not require an offshore source to formulate net longshore sand transport rates within an accepted range. However, incorporation of a lower-bound estimate (75,000 cu m/year) for the offshore source also agrees with the accepted range for net longshore transport at Fire Island Inlet. However, integration of the upper-bound estimate results in net longshore sand transport rates at Fire Island Inlet which exceed accepted values. It is concluded that a source of sediment offshore of central Fire Island may exist, but its contribution to the littoral zone is of the order 75,000 cu m/year.

6.2 Other Studies (reproduced from Gravens et al., 1999)

Previous studies, including sediment budgets, pertinent to the Fire Island to Montauk Point Reformulation study include work by Taney (1961a,b), Research Planning Institute (RPI 1983), and Kana (1995). A brief review of these studies and a few others prepared by Gravens et al. (1999) is reproduced for the most part literally in the following paragraphs.

6.2.1 Taney (1961a,b)

Taney discusses littoral transport processes for the south shore of Long Island, providing geomorphic support for the general east-to-west direction of (net) longshore sediment transport based on migration of inlets prior to stabilization, and impoundment at jetties east of the inlets after stabilization. However, he mentions two locations in which there appears to be a reversal in (net) longshore sediment transport, one of which is within the FIMP region. Immediately west of Fire Island Inlet, a reversal in (net) longshore sediment transport occurs due to tidal currents and wave refraction of the shoal at the mouth of the inlet. He emphasizes that the littoral drift rate varies with distance alongshore.

Taney (1961a) estimates a littoral transport rate for three locations along Long Island, two of which are within the FIMP study reach, based on two methods: (a) Method 1 is the accretion rate updrift of a littoral barrier, up until impoundment capacity, using periodic profiles; and (b) Method 2 is the product of the average annual growth of the updrift shore of an inlet and the average inlet depth. Method 1 is considered more accurate, due to the fact that Method 2 cannot account for the quantity of sediment that bypasses the inlet or is lost to the flood or ebb shoals. Of course, it must be recognized that the equilibrium state of the shoreline updrift of the location of interest affects the estimate. For example, Taney discusses the lack of advancement at Democrat Point from 1930-34 and the possible correlation of this with the opening of Moriches Inlet, which occurred in 1931. Taney (1961b) also estimates a volume of 76,500 m^3/yr from the wave-cut moraine bluffs at Montauk Point.

At Moriches Inlet, Taney estimated approximately 230,000 m^3/yr for the (assumed to be net) littoral transport rate. Note that this estimate reflects conditions prior to the construction of the Westhampton Groin Field. The net longshore sand transport rates for Fire Island Inlet range from 122,000 to 460,000 m^3/yr with 344,000 m^3/yr considered the "most acceptable estimate." Taney does not present his calculations, although data for growth of the updrift spit prior to (and after) stabilization are provided in figures and tables, and profile data are provided in two appendices. (See "Analyses – Impoundment at Democrat Point for a net littoral transport estimate calculated using Taney's data" in Gravens et al., 1999)

Taney concludes that "the present rate of littoral drift is much greater than can be derived from this source" (the headland bluffs). "Streams do not contribute sediments to the system," and "the shoreward movement of the nearshore bottom sediments is questionable." "Therefore, the great difference between the estimates of the amount of sediments moving and that supplied by the bluff unit of the headlands section would indicate that a source of beach material in addition to the bluffs is required. It appears that the only remaining sources of supply of littoral materials are the existing beaches, and possibly a small portion of the nearshore bottom."

6.2.2 Panuzio (1968)

In a general paper about the south shore of Long Island, Panuzio discusses longshore sand transport rates (assumed to be net) for various locations along the study shoreline: Shinnecock Inlet 230,000 m^3/yr (300,000 cy/yr), Moriches Inlet 267,000 m^3/yr (350,000 cy/yr), and Fire Island Inlet 460,000 m^3/yr (600,000 cy/yr). Presumably, these rates were derived from impoundment of littoral material at jetties, migration of pre-stabilized inlet spits, and wave refraction calculations. Panuzio also gives an evaluation *of the Westhampton Groins 1 through 11 18 months after construction. East of Groin 4, the beach accreted, and west of Groins 4 through 11 (Groins 12 through 15 had not yet been constructed) eroded. West of the Groin 11, a reversal in the direction of net longshore sand transport was believed to occur due to the trapping and filling of the updrift groins.*

6.2.3 Research Planning Institute (1983)

The Research Planning Institute (1983) prepared a sediment budget in support of CENAN to aid in the design of future beach erosion control, hurricane protection, and inlet navigation projects for the Fire Island Inlet to Montauk Point project reach. In formulating the sediment budget, several criteria were used to select data used in the budget: historical data were given preference over theoretical calculations; data representing stabilized inlets were favored over pre-stabilization data; and extreme ("rare") events such as the 1980 Moriches Breach were considered perturbations to the normal long-term trends of the sediment budget, and therefore were considered inappropriate in making future beach maintenance decisions. In reviewing the available data, controlled profile data measured in June 1955 and December 1979 were determined to best meet the data selection criteria, although profile data from June 1933, January 1940, and June 1967 were also applied to develop intermediate sediment budgets.

Analysis of the profile data set indicated an "inflection point" (shift from onshore losses to offshore gains) at approximately –25 ft Mean Sea Level (MSL). Based on this, and to meet CENAN requirements for offshore cells separated at 6 ft depth intervals, -24 ft MSL was taken as the seaward boundary of the littoral transport cells. Thus, in the on-offshore direction, volumetric changes for three lenses were represented in the sediment budget: between the profile baseline to Mean High Water (MHW), representing the dune and visible portion of the beach; MHW to Mean Low Water (MLW), representing the intertidal beach type; and MLW to –24 ft MSL, which included the offshore bar. As requested by CENAN, volumetric changes were also reported for regions defined by: profile baseline to MHW; MHW to Mean Sea Level (MSL); MSL to MLW; MLW to –6 ft contour; and four other segments at 6-ft intervals out to the –30 ft MSL contour. The authors found a good correlation between the MSL contour movement and unit width volume change V, with MSL (ft/year) = 7.5 V (cy/ft/year). In the alongshore direction, 25 fixed compartments/sub-compartments were established based on the availability of profile data and existing morphological features (e.g., inlets). Annualized volumetric changes calculated using the profile data formed the primary basis for formulation of the sediment budget, and were calculated for each alongshore compartment and on-offshore lens. Other quantities applied in the budget are discussed below:

- *Originally, longshore sediment transport rates as calculated from wave energy flux were planned for use with the sediment budget. However, the net direction of longshore sediment transport as estimated using wave energy flux did not agree with geomorphic evidence. Thus, these data were only used as a guide for the magnitude of longshore sediment transport rates. Instead, longshore sediment transport rates as inferred from 1940 to 1955 impoundment rates updrift of Fire Island Inlet (306,000 m^3/yr, or 400,000 cy/yr) formed the basis for calculation of longshore sediment transport rates at each alongshore compartment.*
- *Dredge and fill records were applied in solving the budget, despite incomplete fill/disposal records. Assumptions about disposal/fill quantities and locations, which averaged approximately 2.5 m^3/m/year (1 cy/ft/year) between 1955 and 1979, were made to complete the budget.*
- *A theoretical quantity for the offshore loss of sediment due to profile adjustment because of sea level rise was applied based on an equation from Hands (1981), assuming a sea level rise rate of 3 cm/year (0.01 ft/year) based on New York Harbor data. The rate of offshore sediment loss due to sea level rise ranged from 0.59 to 0.76 m^3/yr (0.77 to 0.99 cy/year).*
- *Losses to the littoral budget due to washovers and breaches were estimated from historical data or narratives. These estimates were determined by estimating a planview area for the washover deposit,*

and assuming an average thickness ranging from 0.3 to 0.45 m (1.0 to 1.5 ft) based on published results. For breach channels, a typical depth was estimated to be 0.9 to 1.5 m (3 to 5 ft) which would have removed a sand wedge from 2.4 to 3.7 m (8 to 12 ft) thick. The authors discuss the severity of the 1938 hurricane, which is the storm of record for the project area. Making several assumptions, the authors estimate that this storm removed 468,000 m^3 (612,000 cy) from Westhampton Beach.

Sediment budgets are presented for ten time periods, based on pairing each profile survey date with a subsequent profile data set. [The authors developed a] sediment budget for the 1955 to 1979 time period, which is recommended by the authors as representing the most "typical" long-term conditions for the project area. Major conclusions for this 24.5-year period are summarized:

- *For the project area, the beach above MHW gained 100,000 m^3/yr (130,000 cy/yr), which is less than the estimated fill quantity (320,000 m^3/yr or 420,000 cy/yr).*
- *Approximately 25,600 m^3/yr (33,500 cy/yr) were lost from the control volume due to overwashes and breaches. If the Moriches 1980 breach were included, this quantity would double.*
- *Approximately 229,000 m^3/yr (300,000 cy/yr) was lost between the baseline and the -18.3 m (-24 ft) MSL contour.*
- *From Montauk to Southampton, 7.5 m3/m/year (-3 yd3/ft/year) was lost, with approximately 50 percent of this average derived from the offshore lens*
- *Shinnecock, Westhampton, and Moriches Inlet compartments gained 6.9 m3/m/year (2.75 cu yd/ft/year), with beach fill projects contributing 4.5 m3/m/year (1.8 cu yd/ft/year).*
- *•On average, central Fire Island was stable (less than 0.08 m3/m/year or 0.1 cu yd/ft/year loss), although individual compartments experienced gains and losses.*
- *Western Fire Island experienced large net losses (14.8 m3/m/year or 5.9 cu yd/ft/year), with 85 percent of this from the offshore lens.*
- *On Fire Island, western compartments from Sunken Forest to Robert Moses State Park lost over 23,000 m^3/yr (30,000 cy/yr) above MHW.*
- *Democrat Point gained sediment at 20.5 m^3/m/year (8.2 cy/ft/year).*

The authors conclude by discussing sensitivity of the sediment budget to various input quantities.

- *They discuss the small differential between the average net loss to the project area (1.9 m^3/yr or 0.75 cy/yr) and the apparent input averaging 0.76 m^3/yr (1.0 cy/yr). If these values are in error by 20 to 25 percent, it could significantly affect longshore sediment transport calculations.*
- *The average estimate for offshore losses due to sea level rise is comparable to the average net loss for the entire project reach. Reducing the average rate by half would have a cumulative increase of 134,000 m^3/yr (175,000 cy/yr) for the entire project reach.*
- *Estimates for overwash and breach quantities represent only 5 10 percent of the annualized volume changes, and therefore assumptions made in these calculations have relatively little effect on the sediment budget.*
- *The authors estimate that dredge and fill volumes are probably accurate within ±25 percent, which would produce up to ±0.63 m^3/m/year (±0.25 cu yd/ft/year) error in the budget.*

6.2.4 USACE-NAN (1987)

In a sediment budget formulated for Shinnecock Inlet, USACE-NAN (1987), estimated a net longshore sand transport rate 1 km east of the inlet equal to 230,000 m^3/yr, and a net longshore sand transport 1.8 km west of the inlet equal to 189,000 m^3/yr.

6.2.5 Nersesian and Bocamazo (1992)

In another sediment budget for Shinnecock Inlet, Nersesian and Bocamazo estimated the net transport east of Shinnecock equal to 281,000 m^3/yr.

6.2.6 Williams and Morgan (1993)

These authors used sedimentological evidence from four offshore and 11 onshore samples along Fire Island to quantitatively link two of the offshore samples, representing buried glacial to fluvioglacial lobes of the Huntington-Centreport Pleistocene channel, to the immediately-onshore or slightly downdrift onshore samples. Although representing only two offshore sample data points, these results provide some evidence that offshore sediment may be a contributor to the sediment budget of Fire Island. Taney (1961) had cited westward littoral drift as the dominant mechanism introducing sand-sized material from wave-cut moraine bluffs at Montauk Point (76,500 m^3/yr). However, to satisfy the sediment budget, an additional 152,000 m^3/yr at Moriches Inlet and 45,500 m^3/yr (Taney 1961) to 408,500 m^3/yr (Panuzio 1968) at Fire Island Inlet would be required to balance the sediment budget. RPI (1983) indicated a 300,000 m^3/yr deficit from west-central Fire Island to Fire Island Inlet. Based on these results, the authors speculate that there may also be an eastern Fire Island on-offshore sedimentological link (specifically, the Smithtown-Brookhaven Pleistocene channel).

6.2.7 Kana (1995)

Kana updated RPI's (1983) sediment budget from Fire Island Inlet to Montauk Point by including volumetric changes as calculated using profile data seaward of the -7.3 m MSL contour out to depth of closure (determined to vary between -9.1 m MSL along Fire Island and Westhampton Beach to -12.2 m MSL in the vicinity of Montauk Point). (Note that RPI (1985) had calculated a line sink at -24 ft MSL due to equilibrium profile adjustment; this rate of profile adjustment ranged from 1.9 to 2.5 m^3/m/year. Kana's (1995) formulation extended to the depth of closure, meaning that there was no profile adjustment included.)

The sediment budget was based on comparative profile data from June 1955 to December 1979, and was calculated using 25 alongshore cells, with a width of 7.6 km (excluding inlets). Each cell was represented by 3 to 5 long profiles. This time period represents "present-day conditions" (at that time) after inlet stabilization and construction of groin fields, and was sufficiently removed from the storms of 1960 and 1962 to represent typical conditions.

Montauk Point was estimated to provide 110,000 m^3/yr. The east fillet at Shinnecock Inlet was determined to grow at 220,000 m^3/yr, which agrees with Panuzio (1968). To solve the budget, a reversal in net longshore sediment transport was determined to occur west of the Westhampton Groin Field, resulting in 85,000 m3/yr net longshore sediment transport to the east. Net longshore sediment transport rates at Fire Island Inlet were determined to be 360,000 m^3/yr (agrees with USACE, 1958; Panuzio, 1969; RPI, 1985). Major conclusions of the study were:

- *The magnitude of net longshore sediment transport does not increase uniformly in magnitude from the source at Montauk Point.*
- *The groin field at Westhampton interrupts all net (eastbound) longshore sediment transport, resulting in a reversal in this region.*
- *Net longshore sediment transport rates at Moriches Inlet are lower than previously reported.*
- *The middle portion of Fire Island (20 km east of Fire Island Inlet) had a lower net longshore sediment transport rate (110,000 m^3/yr) than expected. Severe erosion of eastern Fire Island is feeding the central portion; of this erosion, 87 percent is between MLW and -9.1 m MSL. Abandoned Fire Island Inlet shoals appear to have been a significant source of sediment through the early 1900s. However, because of the erosion of west Fire Island beaches, this source appears to be largely gone.*

6.2.8 Schwab et al (1999)

The authors present results of geologic mapping of the inner continental shelf offshore of Fire Island based on high-resolution sidescan-sonar imaging and subbottom profiling. Their results indicate that the inner continental shelf offshore of Watch Hill, the oldest (1,200 years) and most stable part of the barrier system from Shinnecock Inlet to Fire Island Inlet, most likely behaved as a headland during times of lower sea level. Erosion of this headland during the past 10,000 years furnished sediment to the inner continental shelf downdrift and was reworked into a series of shoreface-attached sand ridges. These ridges are 5-m thick immediately west of the outcrop, and less than 1 m thick or absent in other regions.

Previous sediment budgets have indicated that (net) longshore sediment transport rates along the Fire Island barrier are roughly 200,000 m^3/yr, whereas approximately 360,000 m^3/yr is believed to be passing into Fire Island Inlet. The authors suggest that the deficit in previous sediment budgets can be accounted for by an onshore sediment flux from the shoreface-attached sand ridges. Schwab speculated that the magnitude of the onshore sediment flux ranges from 75,000 to 390,000 m^3/yr[8], and feeds into the littoral system for a region extending from just west of Watch Hill through Point of Woods, Fire Island.

6.3 Overview of the Present Work

The purpose of the present work is to modify the sediment budgets developed by Gravens et al. by considering more recent data, especially new conditions and management practices at the three inlets in the FIMP project area. First, a sediment budget for the period 1995 to 2001, herein referred to as the *Recent (1995-2001)* regional sediment budget, was developed for the project shoreline and the three inlets. This budget was based on the 1995 shoreline previously digitized by CHL, a recent (2001) shoreline digitized from orthorectified aerial photography by CENAN, short (i.e., wading depth) and long (i.e., to or beyond depth of closure) beach profile surveys collected in 1995 and 2001 by CENAN, and several inlet surveys collected between 1995 and 2002 by CENAN and others. This short-term sediment budget was prepared to assess any recent changes in the previously identified medium- to long-term trends. Note, however, that these short-term results cannot, in general, be used to predict long-term or even medium-term sediment transport trends. Thus, a new sediment budget incorporating the long-term trends identified by Gravens et al., recent changes, and existing shoreline and inlet management practices was also developed. This new “representative” budget is referred to herein as the *Existing (c. 2001)* sediment budget and should be considered an “update” of the of the *Existing (c. 1999)* conditions budget developed by Gravens et al.

The reach definitions for most of the cells of the regional budget remain similar to the ones in the Gravens et al. analysis to facilitate assimilation of the previous estimates and comparisons with the previous sediment budgets. The inlet cells (Fire Island Inlet, Moriches, and Shinnecock) encompass the sub-divisions specified in the inlet sediment budget presented in Section 5. Table 6-1 lists the beginning and ending stations (from east to west starting at Fire Island Inlet) for each of the regional sediment budget cells. For consistency and to make comparisons with previous work easier, the current sediment budget update is also presented in metric units.

[8] The estimate published in the final Coastal Sediment paper was 84,000 to 396,000 m^3/yr

Table 6-1: Regional Sediment Budget Cell Stations		
Morphologic Zone	**CHL Stationing (km east of each inlet)**	**Regional Stationing (km east of Fire Island Inlet)**
Fire Island Inlet	0	0
	0.075	0.075
UBCH-FI	0.075	0.075
	3.8	3.8
FI1	3.8	3.8
	17	17
FI2	17	17
	32	32
FI3	32	32
	46	46
DBCH-M	46	46
	46.8	46.8
Moriches	46.8	46.8
	0.6	50
UBCH-M	0.6	50
	3.2	52.6
W1	3.2	52.6
	5.1	54.5
W2	5.1	54.5
	10.8	60.2
W3	10.8	60.2
	12.9	62.3
W4	12.9	62.3
	21.6	71
DBCH-S	21.6	71
	22.4	71.8
Shinnecock	22.4	71.8
	0.6	74.8
UBCH-S	0.6	74.8
	3.2	77.4
M1	3.2	77.4
	13	87.2
M2	13	87.2
	24	98.2
M3	24	98.2
	44	118.2
M4	44	118.2
	50	124.2
M5	50	124.2
	58.1	132.3

6.4 Methodology and Data Sources

The basic sediment budget equation for a control volume, or cell, is expressed as (adapted from Rosati and Kraus, 1999):

$$\sum Q_{IN} - \sum Q_{OUT} - \sum \Delta V + P - R = residual \qquad \text{Equation 6-1}$$

where all terms are expressed as a volume or as a volumetric change rate. Q_{IN} are the sources (e.g., bluff erosion, incoming Longshore Sediment Transport, LST) to the control volume, conversely, Q_{OUT} are the sinks (e.g., outgoing LST) to the control volume. ΔV is the net volume change within the cell, P and R are the amounts of material placed in and removed from the cell, respectively, and *residual* represents the degree to which the sediment budget is balanced. For a balanced budget, the residual is zero. This framework is the basis for the analysis presented herein.

All data sets available for the development of the regional sediment budget update are listed in Appendix B. A detailed description of the data available for formulation of the inlet sediment budgets is presented in Section 5. Data sources used specifically for the development of the sediment budget along the mainland and barrier island sections of the project area are as follows:

6.4.1 Beach Profile Data

Beach profiles collected by CENAN throughout the FIMP study area on several separate dates (March 1995, October 1995, March 1996, October 1996, March 1997, March 1998, October 1998, March 1999, October 1999, March 2000, and March 2001). These profile datasets were available as part of the Atlantic Coast of New York Erosion Monitoring Program (ACNYMP). ACNYMP was initiated in 1995 to collect information and data on beach changes and coastal processes along the 210-kilometer ocean shoreline of Long Island, New York. As of 2005 the program had collected over 3,400 beach profile surveys at 426 locations and semi-annual aerial photo surveys of the entire shoreline. Data and information such as historical shorelines, topography, locations of structures, flood zone delineations, etc. from other coastal projects have also been compiled and digitized as part of this effort (Tanski and Pendergrass, 2005).

All of the profile survey data are horizontally referenced to the NAD83 in New York Long Island Zone 3104 State Plane coordinates. Vertically the profile data are referenced to the North American Geodetic Vertical Datum of 1929 (NGVD). The method of survey employed for most of these surveys include rod and transit for the land portion of the survey while the offshore portion of the survey was conducted using a survey sled fitted with a GPS and pulled by a boat. The vertical accuracy of the sled survey method is generally less than 0.03 m (0.1 ft).

The March 1995 beach profile survey consists of 126 sled lines (extending approximately 9 m (30 feet) NGVD water depth), including 78 profiles coinciding with the 1979 profile survey. The October 1995 and subsequent surveys consist of 213 profile lines, of which 68 are long lines extending to -9 m (-30 feet) NGVD. The remaining 145 profile survey lines are wading surveys that extending to approximately -0.6 m (-2 feet) NGVD. The spacing between these most recent survey lines ranges between about 200 m and 1 km with most lines spaced about 600 m apart. The profile survey lines are generally spaced closer together in the vicinity of the inlets and in the developed portions of Fire Island and west of the groin field on the Westhampton barrier. The beach profile survey lines are divided among four main project reaches as follows: “Fire Island” – 84 lines (24 long lines and 60 short), “Westhampton” – 44 lines (18 long lines and 26 short), “Ponds” (from Shinnecock Inlet to just east of Georgica Pond) – 42 lines (14 long lines and 28 short), and “Montauk” (from East Hampton to Montauk Point) – 43 lines (12 long lines and 31 short).

6.4.2 Shoreline Data

Gravens et al. (1999) compiled and analyzed a total of 13 historical shoreline position datasets as part of their study. 10 shoreline position data sets (1830, 1870, 1887, February-May 1933, October 1938, March 1962, and December 1979) were mapped by Leatherman and Allen (1985). The other three datasets (April 1983, March 1988, and March/April 1995) were compiled by Gravens et al. (1999) by digitizing the interpreted high water line shoreline position on digitally rectified scanned aerial photography. Specifically, aerial photographs dated April 1983, March 1988, and March/April 1995 were used to create a shoreline position data set for each time period. Details about the origin of the aerial photography are given in Gravens et al (1999).

The aerial photographs were scanned to create a digital image rectified using visible points with known coordinates and subsequently used to digitize the High Water Line (HWL) shoreline. Note that the HWL shoreline is not referenced to a specific elevation or datum, but to a notable feature that represents the upward limit reached by the water (Gravens et al., 1999). For their study, Gravens et al visually interpreted the HWL shoreline as the seaward-most berm crest. This morphologic feature was generally identifiable in the images by a color differentiation on the beach. This procedure was believed to result in shoreline positions compatible with the historical shoreline position data depicted on the U.S. Coast and Geodetic Survey maps from which the shoreline position data prior to and including 1933 were derived and the shorelines digitized by Leatherman and Allen (1985), which according to their description represent "mean high tide shorelines."

One additional shoreline dataset was compiled and analyzed by CENAN as part of this study by also digitizing the HWL on high-resolution orthoimagery[9] collected in April 2001 by the New York Statewide Digital Orthoimagery Program. The images have a resolution of 20 cm (0.5 feet) per pixel and can be easily viewed with GIS software.

Finally, a series of mean high water shoreline position surveys on Fire Island were provided by Dr. James R. Allen, U.S. Geological Survey (USGS). These shoreline position data sets were collected by Dr. Allen by traversing the shoreline with an All-Terrain-Vehicle (ATV) equipped with a Global Position System (GPS). The data sets consist of the horizontal (northing, easting) position of the ATV as it was driven along the berm crest. GPS shoreline position data sets provided by Dr. Allen correspond to the following dates: August 1993, September 1994, August 1995, November 1996, January 1997, May 1997, September 1997, January 1998, and September 2001.

The same baseline established by Gravens et al. was used in this study as reference for all the shoreline position data, including the data from USGS and the April 2001 shoreline. Detailed baseline information provided by Gravens et al. (1999) is summarized in Table 6-2.

Table 6-2: Shoreline Analysis Baseline Information

Shoreline Segment	Point of Origin		Point of Termination		Orientation (deg)	Length (m)
	Easting (m)	Northing (m)	Easting (m)	Northing (m)		
Gilgo	346350.00	49300.00	359587.26	53857.95	N 71 E	14000
Fire Island	358192.34	51878.58	405468.26	68156.99	N 71 E	50000
Westhampton	404995.00	67720.00	428334.51	76679.20	N 69 E	25000
Montauk	428275.00	76400.00	480220.68	102867.65	N 63 E	58000

6.4.3 Volume Changes

In order to develop the *Recent (1995-2001)* and *Existing (c. 2001)* sediment budgets, volume changes in each cell were computed using three data sources: (1) long profiles, (2) a combination of long and short (i.e. wading) profiles, and (3) digitized shorelines. Volume differences were divided by the time between surveys to obtain a volume change rate. Where short profiles were used to supplement the long profiles, volume changes across the subaerial portion of the profile were summed, a contour change rate was calculated at shoreline and multiplied by the approximate depth to closure, 7.0 m (Gravens et al., 1999),

[9] Digital Orthoimagery is vertical aerial imagery that has had all distortions caused by ground elevation changes and camera distortions removed through computer processing and placed in a digital format that can be used with computer applications. A digital orthoimage combines the rich information content of an aerial photo with the accuracy and spatial registration of a map. (http://www.nysgis.state.ny.us/gateway/orthoprogram/ortho_options.htm)

then added to the subaerial changes and divided by the time between surveys. Shoreline change rate was multiplied by the active profile depth, 10.5 m (Gravens et al., 1999) to obtain a volume change rate. In general, volume changes based on profile data were preferred over changes based on shoreline data if profile density was adequate (at least one profile per km of shoreline). However, this approach was modified in areas where additional shoreline data was available (Fire Island) or where changes based on profile data seemed unrealistic based on previous sediment budgets, net longshore sediment transport computed with GENESIS (see below) or basic understanding of coastal processes in the FIMP area.

6.4.4 Sea Level Rise

Cross-shore sediment losses due to sea level rise were incorporated as in Gravens et al. (1999) (after Bruun, 1962). Specifically, a volumetric loss rate due to relative sea level rise of 2.3 m^3/m/yr based on relative SLR rate of 0.003 m/yr was applied to all ocean shoreline cells in the shoreline-based volume change analysis. Therefore the total sediment sink along the shorelines due to sea level rise is estimated to be roughly 305,000 m^3/yr. 134,000 m^3/yr from Montauk Point to Shinnecock Inlet, 57,200 m^3/yr from Shinnecock to Moriches Inlet, and 114,000 m^3/yr along Fire Island (Gravens et al., 1999). Note that for the profile-based volume change analysis sea level rise should be directly accounted for since the data usually extends past the depth of closure and therefore was not considered separately. See Section 5 for details regarding treatment for sea level rise at the inlets.

6.4.5 Contribution of Montauk Point Bluffs

Gravens et al. (1999) presents estimates of a sediment source from the Montauk Bluffs on the order of 30,000 m^3/yr, obtained using shoreline change and profile data as well as sediment grain size analysis. In this update, available profile data, which includes the face of the bluff, were used to quantify volume changes throughout the Montauk Bluff area. Therefore, these volume changes are used in the update directly without separate consideration of the exact bluff contribution.

6.4.6 Offshore Sediment Source

A number of previous studies (e.g., Williams, 1986, Williams and Meisburger, 1987, Williams and Morgan, 1993, Schwab et al. 1999, Schwab et al. 2000) suggest the possibility of a contribution of sediment to the coastal sediment budget from offshore sources. Based on previous studies and data obtained from high-resolution sea-floor mapping techniques, Schwab et al. (1999) suggest that the late Holocene evolution and modern behavior of the Fire Island barrier system is linked directly to the geologic framework of the inner-continental self. Specifically, onshore sediment flux from an inner shelf source consisting of a series of shoreface-attached reaches has increased the sediment volume available to maintain island stability between Watch Hill and Point O' Woods and for historic Fire Island spit growth west of Point O' Woods. This source was created during Holocene marine transgression from a modern sediment deposit formed from erosion of a subaerial headland formed during times of lower sea level. Therefore the authors suggest that this inshore sediment flux must be considered in order to develop a realistic regional sediment budget. Moreover, although the authors believe that published sediment budgets for Fire Island are, at best, semi-quantitative, they use them to develop an estimate of the onshore flux of approximately 80,000 m^3/yr to 396,000 m^3/yr, apparently in order to explain the spit progradation at Democrat Point. Schwab et al. (2000) narrow this estimate based an average longshore sediment transport rate along Fire Island of approximately 200,000 m^3/yr (Kana, 1995) and suggest that an additional 200,000 m^3/yr from an offshore source is required to balance the sediment budget. Note, however, that neither Schwab et al. nor any of the other studies listed above are able to identify the specific processes controlling this onshore flux of sediment.

In contrast, the authors suggest that east of Watch Hill there is only Pleistocene and Holocene channel-fill material available in the inner-shelf. Apparently this sediment is coarser grained and thus less mobile and not a source to the coastal sediment budget. There are other modern sand ridges off the coast between

Moriches Inlet Southampton and between Napeague and Montauk Point. However, these sources are located seaward of the 18-m contour, stranded there due to late Holocene marine transgression, and therefore not connected to the shoreface.

Finally, the authors refute Kana's hypothesis that erosion of a large "relict" ebb-tidal delta west of Point O' Woods could have provided sufficient material to the littoral zone to explain the observed spit growth at Democrat Point. They argue that there is no evidence that this ebb-tidal delta existed because there is no modern analog along southern Long Island and because microtidal, wave-dominated shoreline should be expected to have large flood tidal deltas and small ebb-tidal deltas.

As explained in Section 6.6, the present study also recognizes the possibility of an offshore sediment source based on estimated volume changes and computed potential longshore sediment transport rates, although this source was not required to yield longshore sand transport rates at Fire Island Inlet falling within an accepted range.

6.4.7 Overwash and Breaching Losses to the Bays

There were not significant storm events over the 1995 to 2001 period requiring estimates of overwash fan volume and/or other losses to the system from storms. Therefore, the *Recent (1995-2001)* regional sediment budget did not include sediment losses caused by overwash or breaching. Gravens et al. (1999) accounted for bay losses in their *Historic (1979-1995)* sediment budget, which included the 1980 breach at Moriches Inlet and the 1992 breaches at Pikes Beach (at total of approximately 100,000 m^3/yr for the Westhampton barrier), but not in their *Existing (c. 1999)* sediment budget. Kana (1995) in his 1955 to 1975 budget also suggest that annualized losses to the bays by inlets or washovers were insignificant over the period, despite a series of significant storms, the largest of which was the 6-8 March 1962 Nor'easter, which caused significant overwash and a breach in Westhampton Beach. Kana computed volume estimates for washovers and breaches based on analysis of storm histories, vertical photographs, and assumptions regarding the thickness of the washover deposits. In the end the total annualized contribution was relatively small: 25,000 m^3/yr or 0.2 m^3/m/yr (RPI, 1985). Therefore, for the purposes of this study, the contribution of breaches and overwashes was also neglected in the formulation of the Existing (c. 2001) sediment budget.

Note, however, that very large storm effects may cause significant impacts and a large removal of sediment from the barrier island to the backbay. For example, RPI (1985) estimated that the upwards of 750,000 m^3/yr were removed from the Westhampton littoral cell during the 1938 hurricane (Kana, 1995). These, however, are considered extraordinary events that one should be aware of but that are obviously impossible to predict and thus difficult to incorporate into a sediment budget representative of *Existing* conditions.

6.4.8 Wind-blown Sediment Transport

Wind-blown sediment transport processes directly affect the morphology of the beach and barrier island by building and reshaping the dry beach and the dunes through onshore sand transport. In their study, Gravens et al. (1999) assumed that the dune system within the FIMP area is relatively well-established and vegetated. Therefore they assumed that the contribution of sand transport to these littoral sediment was minor and it was neglected. A similar assumption was made in this study.

6.4.9 Inlet Sediment Budgets

Sediment budget cells at each of the three inlets have been updated and are discussed in detail in Section 5. Beach profile and shoreline change data were used to assess volume change in shoreline cells adjacent to the inlets as discussed above. Bathymetric survey comparisons were conducted using a series of synthetic grids at each inlet.

6.4.10 Engineering Activities

Details of engineering activities and beach fill placement from 1998 to 2002 were obtained from CENAN and other state and local stakeholders and are presented in Appendix C. Activities from 1995 to 1998 were compiled by Gravens et al. (1999) and are also included in Appendix C. Activities prior to 1995 are presented in Gravens et al. (1999).

6.4.11 Uncertainty

Volume changes and sediment transport quantities required for the formulation of a coastal sediment budget cannot be measured directly and therefore values of such quantities have to be obtained through indirect and/or incomplete measurements (e.g., shorelines or beach profiles), with predictive formulas, or through estimates based on experience and judgment. According to Kraus and Rosati (1998a), these values can be considered as consisting of two terms: (1) Best Estimate ± (2) Uncertainty. The values presented in the following sections are considered a "Best Estimate" and are based on various sources including incomplete measurements (beach profiles, inlet surveys), indirect measurements (shorelines), numerical estimates of longshore sediment transport, and numerous assumptions regarding coastal processes and sediment transport pathways within the FIMP project area, particularly at the three inlets.

"Uncertainty" consists of *error* and *true uncertainty* (Kraus and Rosati, 1998a). A main source of error is limitation in measurement process or instrument. Typically the estimates of the potential error associated with the various instruments (e.g., total stations, GPS, lasers, echosounders, etc.) and measurement processes (land surveys, boat surveys, sled profile surveys, SHOALS surveys, aerial mapping, orthorectifying, digitizing, etc.) can be quantified. However, true uncertainty in estimates of coastal engineering quantities is more difficult to determine and, unfortunately, generally much more significant than error because it includes natural temporal (daily, seasonal, annual) variability and spatial variability (alongshore and across shore) as well as many unknowns (e.g., grain size, past and future wave climate) and variability imposed by choices regarding various definitions which are necessary to compute these estimates (e.g., average shoreline orientation, berm location, depth of closure, etc.)

Kraus and Rosati (1998a) provide various representative examples of uncertainty analysis and show that uncertainty in sediment budget can be large. In fact, the maximum uncertainty computed by the authors was greater than the estimates themselves and the "best" uncertainty was only about 50% smaller. This despite the fact that some of the assumed "input" uncertainty values are relatively small compared to other published estimates. For example, in their uncertainty analysis example, the "best" (rms) estimate of uncertainty regarding the active profile depth was 0.3 m for an assumed value of 8 m. However, Morang et al. (1999) estimated error associated with profile interpretation at 0.15 m, short-term temporal variability at more than 2 m, and spatial variability along the FIMP area at 3 m.

Other examples include uncertainty associated with shoreline mapping using orthorectified aerials. Leatherman and Allen (1985) estimated the total uncertainty associated with their 1979 FIMP shoreline dataset to be 6 m (5 m due to accurate mapping of HWL and 1 m due to equipment/operator error during digitization) , although is not clear whether or not uncertainty due to scanning and orthorectification of the original aerial photos was included. Nonetheless, a 6 m uncertainty in the shoreline is equivalent to a 30,000 m^3/yr over a 20 year period along a 10 km beach cell.

Although profile surveys significantly reduce errors due to measurement, uncertainties regarding depth of closure, and seasonal onshore-offshore volume changes, they also introduce a significant source of uncertainty regarding alongshore spatial variability unless very dense coverage is available. As explained below there were approximately 0.8 long profiles per km of shoreline available for the period 1995-2001. Density increases to 1.5 profiles per km if short (i.e., wading) profiles are also included. For comparison, Kana's budget for the 1955-1979 period was developed with 0.5 profiles per km. However, unlike the 1995 and 2001 profiles collected by CENAN and available for this study, the profiles used by Kana were collected from unique monuments which meant that the profiles in 1955 and 1979 did not exactly overlap.

Uncertainty with regards to volume changes at the inlets is particularly large and difficult to estimate. As explained in Section 5, most of the available surveys lack sufficient coverage of the inlet channels and shoals. This is particularly true of the flood shoals. In addition, differences in survey (e.g., SHOALS versus a boat-based bathymetric survey) and datum reduction methods add another source of error an uncertainty. Minimum performance standards for USACE hydrographic surveys (HQUSACE, 2002) require that RMS errors for a depth measurements should not exceed 0.3 m (1 foot) for acoustic system surveys performed in depths between 5 and 13 m over a "soft" bottom (including sand). Byrnes et al. (2002) present an example of uncertainty analysis for volume changes at Ocean City Inlet for the period 1977/78 to 2000. The authors computed uncertainty estimates for the ebb-tidal complex on the order of 20% of volume change estimates (roughly 800,000 m^3 out of 4,300,000 m^3). A significant percentage despite the fact that lack of coverage or differences in survey and datum reduction methods were not an issue in their example.

It should be noted that that none of the previous studies have completely addressed the issue of uncertainty. Gravens et al. (1999) accounted for uncertainty in their potential longshore sand transport rate calculations based on the Wave Information Study (WIS) hindcast database (1976-1994 period) and wave modeling (see below). Specifically, the authors computed the standard deviation in the net LST and divided it by the square root of the number of yearly averages to give a representative decadal-scale variability. This value ranged from 30,000 m^3/yr to 40,000 m^3/yr and the authors selected a "conservative" value of 40,000 m^3/yr, which was incorporated into their sediment budget and specifically the net LST rate calculations based on computed volume changes. However, as explained above, this uncertainty only addresses a small part of the total error and true uncertainty associated with the formulation of sediment budgets.

Given the myriad of data sources used in this study and the fact that most of the uncertainty is not easy to identify much less calculate (e.g., lack of overlapping coverage at the inlet surveys or differences in datum correction methods) an attempt to quantify the total uncertainty associated with the volume changes and longshore sediment transport rates presented below was not made. Instead, based on the various estimates of uncertainty presented above and the volume change estimates presented in the flowing section and in Section 5 it was concluded that uncertainty represents a significant percentage of the estimates included in the proposed sediment budgets, perhaps as much as the estimates themselves in some cases. Nonetheless, it is judged that the proposed sediment budgets provide a realistic, albeit only semi-quantitative, description of the sediment transport processes that can be used to assist in the planning, design, and formulation of shore protection and storm damage reduction measures for the FIMP project area.

6.5 *Recent (1995-2001)* Regional Sediment Budget

6.5.1 Volume Change Rates

Volume change rates for the 1995-2001 period within the regional sediment budget cells (see Figure 6-1 and Table 6-1) were computed using the long profile data, long and short profile data, and shoreline data described above. Results of the regional volume change analysis are presented in Table 6-3. Volume change rates from each data source are plotted in Figure 6-3 through Figure 6-6.

Table 6-3 illustrates how significant is the uncertainty associated with volume change estimates. For example, the changes computed along Fire Island using USACE shorelines digitized from available aerial photos in the spring of 1995 and 2001 are remarkably different than the changes computed with field data collected by USGS using ATVs and GPS in late summer on 1995 and 2001. Some of these differences are probably due to methodology (scanning and digitizing the HWL on an aerial is very different than "driving" the HWL in the field) and some due to seasonal effects on the onshore/cross-shore distribution of sediment.

Table 6-3:Volume Change Rates by Reach and Data Source (1995 to 2001)

Morphologic Zone	Stationing (km east of each inlet)	Long Profile Density (Profiles/km)	Short & Long Profile Density (Profiles/km)	Volume Change Rate (Long Profiles) 1000 m3/yr	Volume Change Rate (Long & Short Profiles) 1000 m3/yr	Volume Change Rate (USACE Shoreline Change) 1000 m3/yr	Volume Change Rate (USGS Shoreline Change) 1000 m3/yr	Sea Level Rise (1000 m3/yr)
Fire Island Inlet	0 0.075							
UBCH-FII	0.075 3.8	0.81	0.81	26	-34	-4	204	8
FI1	3.8 17	0.53	1.48	-139	137	-152	248	28
FI2	17 32	0.57	1.77	131	89	-142	344	31
FI3	32 46	0.32	0.93	-402	-420	-294	111	29
DBCH-MI	46 46.8	--	1.25	--	57	-16	--	0
Moriches Inlet	46.8 0.6							
UBCH-MI	0.6 3.2	2.50	2.88	151	75	57	--	6
W1	3.2 5.1	2.11	2.89	123	343	237	--	4
W2	5.1 10.8	1.14	1.75	411	255	162	--	12
W3	10.8 12.9	0.71	1.19	-122	-25	-4	--	4
W4	12.9 21.6	0.57	1.15	-255	-57	-146	--	18
DBCH-SI	21.6 22.4	--	2.50	--	2	-12	--	2
Shinnecock Inlet	22.4 0.6							
UBCH-SI	0.6 3.2	0.96	1.15	59	108	27	--	6
M1	3.2 13	0.36	1.07	33	-59	-165	--	17
M2	13 24	0.55	1.50	21	-150	-254	--	19
M3	24 44	0.43	0.93	-265	-425	-105	--	34
M4	44 50	0.50	1.08	89	82	-32	--	10
M5	50 58.1	0.06	0.86	-73	-80	68	--	14

Also worth noting are the differences between volumes computed from shoreline data and profiles. Unfortunately we can only speculate as to which of the two datasets is more accurate, because as explained in the previous section each has its pros and cons. However, as explained above, volume changes based on profile data were preferred over changes based on shoreline data if profile density was at least one profile per km of shoreline.

6.5.2 Sediment Budget

The Recent (1995-2001) sediment budget was developed cell by cell from east to west. The volume changes presented in Table 6-3 were used with results of potential sediment transport calculations to build the regional budget. This process was based not only on the calculations themselves, but also on previous work and engineering judgment.

Montauk Point provides a convenient boundary condition for longshore sediment transport estimates and sediment budget formulation. Specifically, if zero longshore transport at the east end of the Montauk bluffs morphological reach (M5) is assumed, transport rates at the western end of that reach and at the boundaries between reaches farther west can be computed by solving the sediment budget equation for each reach. Therefore, the regional sediment budget was developed by starting at Montauk and progressing west until reaching Fire Island Inlet. A very similar approach was used in developing most of previous sediment budgets (e.g., Gravens et., 1999 and Kana, 1995). Computed transport rates at the updrift boundary of the inlet cells were also compared to previous estimates based on updrift jetty impoundment or updrift spit growth (Fire Island Inlet) and, in the case of Shinnecock Inlet, with a numerical estimate of potential longshore sediment transport.

Montauk Reach: M5 [Sta. M50 to M58.1]

The M5 cell is the farthest east, extending west 8.1 km from Montauk Point. It includes, from east to west, the Montauk Point bluff region, the community of Ditch Plains and the bluffs between Ditch Plains and Montauk Beach. In this cell, volume changes computed using the long and short profiles together (Table 6-3) are considered representative of existing conditions, a value of –80,000 m^3/yr (or -10 m^3/yr/m). Density of short and long profiles is nearly 1 per km, which less than preferred. However, shoreline change analysis shows accretion in this area, which is not a result supported by previous sediment budgets and or consistent with the idea of this reach being a source of sediment to the south shore of Long Island. The *Existing (c. 1999)* regional sediment budget suggested net erosion within this cell at the rate of -61,000 m^3/yr (-75,000 m^3/yr due to shoreline erosion plus 14,000 m^3/yr due to sea level rise). Therefore, the estimate of –80,000 m^3/yr for the 1995-2001 based on profile data seems reasonable.

A small amount of fill, 1,000 m^3/yr, was placed within this cell during this period M5. All dredging and fill projects for the regional budget are detailed in Appendix C. The sediment budget is balanced assuming net transport to the west, with no net influx of sediment from the east.

Applying the sediment balance equation to this cell and entering values gives,

$$Q_{W\_M5} - Q_{M5\_M4} - \Delta V_{M5} + P_{M5} - R_{M5} = residual$$
$$0 - Q_{M5\_M4} - (-80{,}000) + 1{,}000 - 0 = residual = 0$$
$$\therefore\ Q_{M5\_M4} = 81{,}000\ \mathrm{m}^3/\mathrm{yr}$$

Montauk Reach: M4 [Sta. M44 to M50]

The M4 cell extends 6 km and it includes the shoreline between Montauk Beach and Hither Hills State Park. Again, volume changes computed using long and short profiles together (82,000 m^3/yr or 14 m^3/m/yr) are considered representative of the 1995-2001 period. Note that there is more than one short or long profile per km of shoreline. Long (only) profile analysis yields a similar volume change rate. However, shoreline change analysis shows some erosion in this area (32,000 m^3/yr). The *Existing (c.*

1999) regional sediment budget suggested net accumulation within this cell at the rate of 7,000 m^3/yr (-3,000 m^3/yr due to shoreline erosion plus 10,000 m^3/yr due to sea level rise).

There was no placement or removal of material in this cell from 1995 to 2001. The sediment budget is balanced assuming transport to the west, with sediment incoming from M5.

Applying the sediment balance equation to this cell and entering values gives,

$$Q_{M5\_M4} - Q_{M4\_M3} - \Delta V_{M4} + P_{M4} - R_{M4} = residual$$
$$81{,}000 - Q_{M4\_M3} - (82{,}000) + 0 - 0 = residual = 0$$
$$\therefore\ Q_{M4\_M3} = -1{,}000\ \mathrm{m}^3/\mathrm{yr}$$

The negative sign indicates that transport from west to east (i.e., a reversal in the direction of net longshore sediment transport) is necessary to balance this cell. This may be simply due to uncertainty in the numbers, however, to create a balanced budget, the necessary 1,000 m^3/yr is assumed to come from the west (M3).

Montauk Reach: M3 [Sta. M24 to M44]

The M3 cell extends 20 km and it includes, from east to west, Hither Hills State Park, Napeague Beach, Napeague State Park, Amagansett and the beaches east of Hook Pond. Volume changes computed using shoreline changes (Table 6-3) are considered representative of 1995-2001 conditions, a value of –71,000 m^3/yr (or -4,000 m^3/m/yr), including losses due to sea level rise (34,000 m^3/yr). Although there is more than one short and long profile per km for this reach, both calculation methodologies using profiles yield a large amount of erosion. However, this cell is not considered to be particularly erosive. In fact, the previous *Existing (c. 1999)* regional sediment budget suggested net sediment accumulation within this cell at the rate of 26,000 m^3/yr (-8,000 m^3/yr due to shoreline erosion plus 34,000 m^3/yr due to sea level rise). Moreover, M3 is updrift (i.e., east) of the groins adjacent to Georgica Pond, which makes a long-term trend of erosion within this cell very unlikely.

There was no placement or removal of material in this cell from 1995 to 2001. The sediment budget is balanced assuming net transport to the west, with 1,000 m^3/yr of sediment transported to the east into M4.

Applying the sediment balance equation to this cell and entering values gives,

$$Q_{M4\_M3} - Q_{M3\_M2} - \Delta V_{M3} + P_{M3} - R_{M3} = residual$$
$$-1{,}000 - Q_{M3\_M2} - (-71{,}000) + 0 - 0 = residual = 0$$
$$\therefore\ Q_{M3\_M2} = 70{,}000\ \mathrm{m}^3/\mathrm{yr}$$

Montauk Reach: M2 [Sta. M13 to M24]

The M2 cell extends 11 km, from 13 km east of Shinnecock Inlet to 24 km east of the inlet, including the shorelines that front Hook Pond, Georgica Pond, Sagaponack Lake, and Mecox Bay. Volume changes computed using short and long profile changes (1.5 profiles per km) are considered representative of conditions between 1995 and 2001 (–150,000 m^3/yr or -14 m^3/m/yr). Fairly sparse long profile data (0.5 profiles per km) yield slight accretion in this cell (21,000 m^3/yr), and shoreline change data indicate significant erosion (–254,000 m^3/yr), which is considered larger than reasonable, although it does appear that this cell is eroding overall. The *Existing (c. 1999)* regional sediment budget suggested net erosion within this cell at the rate of -43,000 m^3/yr (-62,000 m^3/yr due to shoreline erosion plus 19,000 m^3/yr due to sea level rise).

There was no placement or removal of material in this cell from 1995 to 2001. The sediment budget is balanced assuming net transport to the west.

Applying the sediment balance equation to this cell and entering values gives,

$$Q_{M3\_M2} - Q_{M2\_M1} - \Delta V_{M2} + P_{M2} - R_{M2} = residual$$
$$70{,}000 - Q_{M2\_M1} - (-150{,}000) + 0 - 0 = residual = 0$$
$$\therefore\ Q_{M2\_M1} = 220{,}000\ \mathrm{m}^3/\mathrm{yr}$$

Montauk Reach: M1 [Sta. M3.2 to M13]

The M1 cell extends 9.8 km from Sta. M3.2 to Sta. M13, including most of the barrier island fronting the eastern half of Shinnecock Bay. Volume changes computed using short and long profile changes (-59,000 m^3/yr or -6,000 m^3/m/yr) are considered representative of the 1995-2001 period. There is adequate profile density, approximately one per km of shoreline. Sparse long profile data (0.4 profiles per km) suggest accretion in this cell, and shoreline change data indicate erosion. The erosion rate based on shoreline data (-148,000 m^3/yr) is considered too large, although it is believed that, like cell M2, this cell is generally eroding. The *Existing (c. 1999)* regional sediment budget suggested net erosion within this cell at a rate of -26,000 m^3/yr (-43,000 m^3/yr due to shoreline erosion plus 17,000 m^3/yr due to sea level rise).

There was no placement in or removal of material from this cell from 1995 to 2001. The sediment budget is balanced assuming net transport to the west.

Applying the sediment balance equation to this cell and entering values gives,

$$Q_{M2\_M1} - Q_{M1\_UBCH-S} - \Delta V_{M1} + P_{M1} - R_{M1} = residual$$
$$220{,}000 - Q_{M1\_UBCH-S} - (-59{,}000) + 0 - 0 = residual = 0$$
$$\therefore\ Q_{M1\_UBCH-S} = 279{,}000\ \mathrm{m}^3/\mathrm{yr}$$

Shinnecock Inlet: UBCH [Sta M0.6 to M3.2]

The Updrift Beach (UBCH) cell extends 2.6 km from Sta. M0.6 to Sta. M3.2. All three analysis methodologies (long profiles, long & short profiles, and shoreline data) suggest accretion in this area, which is consistent with previous sediment budgets and accumulation updrift of the east jetty at Shinnecock Inlet. Nonetheless, volume changes computed based on shoreline analysis (33,000 m^3/yr or 13,000 m^3/m/yr) are considered representative of the 1995-2001 period. See Section 5 for additional details.

Applying the sediment balance equation to this cell and entering values gives,

$$Q_{M1\_UBCH-SI} - Q_{UBCH-SI\_SI} - \Delta V_{UBCH-SI} + P_{UBCH-SI} - R_{UBCH-SI} = residual$$
$$279{,}000 - Q_{UBCH-SI\_SI} - (33{,}000) + 0 - 0 = residual = 0$$
$$\therefore\ Q_{UBCH-SI\_SI} = 246{,}000\ \mathrm{m}^3/\mathrm{yr}$$

Estimates of sediment transport from 1995 to 2001 using LITPACK yield a net westerly transport of 233,000 m^3/yr in this area (see Section 5). Therefore, the net transport presented above appears to be a reasonable estimate for this period and the assumptions and choices made regarding volume changes within each reach appear to be supported. Note that this net transport value is significantly higher than the estimate from the previous *Existing (c. 1999)* budget (130,000 m^3/yr) but is still within the range of other previously published estimates (68,000 to 304,000 m^3/yr, see Section 6.2). Also note that this difference is not considered unusual considering that the *Recent* estimate is only representative of a relatively small period of time (1995 to 2001) and not long-term conditions.

Shinnecock Inlet

For the purposes of developing the regional budget, most of the inlet cells presented in Section 5 were collapsed into a single Shinnecock Inlet cell. According to the changes between 1995 and 2001, Shinnecock Inlet, including changes over the shoals, channels, deposition basin and the West Beach cell, was a small source of sediment during that period, contributing 7,000 m^3/yr to the regional sediment budget. Refer to Section 5 for additional details.

Applying the sediment balance equation to this cell and entering values gives,

$$Q_{UBCH-SI\_SI} - Q_{SI\_DBCH-SI} - \Delta V_{SI} + P_{SI} - R_{SI} = residual$$
$$246{,}000 - Q_{SI\_DBCH-SI} - (-7{,}000) = residual = 0$$
$$\therefore\ Q_{SI\_DBCH-SI} = 253{,}000\ \mathrm{m}^3/\mathrm{yr}$$

Shinnecock Inlet: DBCH [Sta W21.6 to W22.4]

The Downdrift Beach (DBCH) cell extends 800 m from Sta. W21.6 to Sta. W22.4. Volume changes computed using long and short profile changes (Table 6-3) are considered representative of conditions from 1995 to 2001, a fairly stable beach with slight accretion of 2,000 m^3/yr. A detailed discussion of the DBCH changes is presented in Section 5. No fill was placed in this cell between 1995 and 2001.

Applying the sediment balance equation to this cell and entering values gives,

$$Q_{SI\_DBCH-SI} - Q_{DBCH-SI\_W4} - \Delta V_{DBCH-SI} + P_{DBCH-SI} - R_{DBCH-SI} = residual$$
$$253{,}000 - Q_{DBCH-SI\_W4} - (2{,}000) + 0 - 0 = residual = 0$$
$$\therefore\ Q_{DBCH-SI\_W4} = 251{,}000\ \mathrm{m}^3/\mathrm{yr}$$

Westhampton Reach: W4 [Sta. 12.9 to 21.6]

The W4 cell extends 8.7 km from Sta. W12.9 to Sta. W8.7 and it includes Tiana Beach and Hampton Beach. Volume changes computed using long and short profiles together (–57,000 m^3/yr) are considered representative of the 1995-2001 period. There is more than one short and long profile per km. Relatively sparse long profile coverage (0.6 profiles per km) and shoreline data suggest larger erosion rates in this area which are not considered representative (see Table 6-3). The previous *Existing (c. 1999)* regional sediment budget suggested net erosion within this cell at a rate of –54,000 m^3/yr (-72,000 m^3/yr due to shoreline erosion plus 18,000 m^3/yr due to sea level rise).

There was no placement or removal of material in this reach, which is located updrift of the Westhampton Interim project area, from 1995 to 2001. The sediment budget is balanced assuming transport to the west, with sediment incoming from cell DBCH-SI.

Applying the sediment balance equation to this cell and entering values gives,

$$Q_{DBCH-DSI\_W4} - Q_{W4\_W3} - \Delta V_{W4} + P_{W4} - R_{W4} = residual$$
$$251{,}000 - Q_{W4\_W3} - (-57{,}000) + 0 - 0 = residual = 0$$
$$\therefore\ Q_{W4\_W3} = 308{,}000\ \mathrm{m}^3/\mathrm{yr}$$

Westhampton Reach: W3 [Sta. W10.8 to W12.9]

The W3 cell is immediately updrift (east) of the Westhampton groin field and it extends 2.1 km from Sta. W10.8 to Sta. W12.9. Volume changes computed using shoreline changes (–4,000 m^3/yr of erosion balanced with about 4,000 m^3/yr lost to sea level rise, i.e., no net change) are considered representative of the 1995-2001 period. Both long profile and long and shore profile analysis show larger erosion volume changes in this area and are considered less probable considering previous estimates and the stabilizing effect of the groin field located downdrift. The previous existing conditions regional sediment budget indicated accumulation at the rate of 8,000 m^3/yr (including sediment lost to sea level rise, 4,000 m^3/yr) in W3.

There was a small amount of fill placement in this cell as part of the Westhampton Interim project from 1995 to 2001 (23,000 m^3/yr, see Appendix C). The sediment budget is balanced assuming transport to the west, with sediment incoming from W4.

Applying the sediment balance equation to this cell and entering values gives,

$$Q_{W4\_W3} - Q_{W3\_W2} - \Delta V_{W3} + P_{W3} - R_{W3} = residual$$
$$308{,}000 - Q_{W3\_W2} - (0) + 23{,}000 - 0 = residual = 0$$
$$\therefore\ Q_{W3\_W2} = 331{,}000\ \mathrm{m}^3/\mathrm{yr}$$

Westhampton Reach: W2 [Sta. W5.1 to W10.8]

The W2 cell extends from 5.7 km from Sta. W5.1 to Sta. W10.8 including Westhampton groin field. Volume changes computed using long profile changes (411,000 m^3/yr) are considered representative of the 1995-2001 period given the significant amount sand that was placed within this reach as part of the Westhampton Interim project (see below) and the stabilizing influence of the groins and the additional fill placed downdrift in cell W1. Long profile density in this cell is better than 1 per km for this cell, and in this case it was considered more appropriate to use long profile data in order to capture cross-shore profile equilibration after fill placement. The previous *Existing (c. 1999)* conditions regional sediment budget indicated accumulation at the rate of 132,000 m^3/yr (including sediment lost to sea level rise, 12,000 m^3/yr) in W2.

There was a significant amount of fill placement from 1995 to 2001 in this cell as part of the Westhampton Interim project: 407,000 m^3/yr (see Appendix C). The sediment budget is balanced assuming transport to the west, with sediment incoming from W3.

Applying the sediment balance equation to this cell and entering values gives,

$$Q_{W3\_W2} - Q_{W2\_W1} - \Delta V_{W2} + P_{W2} - R_{W2} = residual$$
$$331{,}000 - Q_{W2\_W1} - (411{,}000) + 407{,}000 - 0 = residual = 0$$
$$\therefore\ Q_{W2\_W1} = 327{,}000\ \mathrm{m^3/yr}$$

Westhampton Reach: W1 [Sta. W3.2 to W5.1]

The W1 cell extends 1.9 km from Sta. W3.2 to Sta. W5.1, including Pikes Beach and Cupsogue. Similarly to cell W2, volume changes computed using long profile changes (123,000 m^3/yr) are considered representative of the 1995-2001 period. Long profile density is very good (2.1 profiles/km). In addition, accumulation in W1 seems reasonable as a result of the fill placement and gradual recovery of this cell after the decades of net sediment losses due to the Westhampton groin field. The previous *Existing (c. 1999)* conditions regional sediment budget suggested accumulation at a rate of 80,000 m^3/yr (including sediment lost to sea level rise, 4,000 m^3/yr) in W1.

There was placement in this cell as part of the Westhampton Interim project area from 1995 to 2001 at a rate of 233,000 m^3/yr (see Appendix C). The sediment budget is balanced assuming transport to the west, with sediment incoming from cell W2.

Applying the sediment balance equation to this cell and entering values gives,

$$Q_{W2\_W1} - Q_{W1\_UBCH-MI} - \Delta V_{W1} + P_{W1} - R_{W1} = residual$$
$$327{,}000 - Q_{W1\_UBCH-MI} - (123{,}000) + 233{,}000 - 0 = residual = 0$$
$$\therefore\ Q_{W1\_UBCH-MI} = 437{,}000\ \mathrm{m^3/yr}$$

Moriches Inlet: UBCH [Sta 0.6 to 3.2]

The Moriches Inlet Updrift Beach (UBCH-MI) cell at Moriches Inlet extends 2.6 km from Sta. 0.6 to Sta. 3.2. An average of volume changes computed using short and long profiles and shoreline data (69,000 m^3/yr, including losses from sea level rise of 6,000 m^3/yr) are considered representative of the 1995-2001 period. A detailed discussion of the UBCH changes is presented in Section 5.2.1.

Applying the sediment balance equation to this cell and entering values gives,

$$Q_{W1\_UBCH-MI} - Q_{UBCH-MI\_MI} - \Delta V_{UBCH-MI} + P_{UBCH-MI} - R_{UBCH-MI} = residual$$
$$437{,}000 - Q_{UBCH-MI\_MI} - (69{,}000) + 0 - 0 = residual = 0$$
$$\therefore\ Q_{UBCH-MI\_MI} = 368{,}000\ \mathrm{m^3/yr}$$

This value is significantly larger than the estimate from the previous *Existing (c. 1999)* conditions budget, 184,000 m^3/yr, and also higher than the range of values published in the literature (45,000 m^3/yr to 267,000 m^3/yr, see Section 6.2). However, note that this *Recent* budget is only representative of the 5-year period between 1995 and 2001, during which roughly 3.6 million m^3 (567,000 m^3/yr) were placed updrift within 13 km of the inlet. Therefore, this result is considered representative of *Recent (1995-2001)* conditions.

Moriches Inlet

Most of the Moriches Inlet sediment budget cells presented in Section 5.2.1 were collapsed into one overarching Moriches Inlet cell for the regional budget. During the 1995-2001 period total volume changes in the inlet add up to a net change of 2,000 m^3/yr, which includes dredging and fill. Refer to Section 5.2.1 for a complete discussion of inlet sediment budget details.

Applying the sediment balance equation to this cell and entering values gives,

$$Q_{UBCH-MI\_MI} - Q_{MI\_DBCH-MI} - \Delta V_{MI} + P_{MI} - R_{MI} = residual$$
$$368{,}000 - Q_{MI\_DBCH-MI} - 2{,}000 = residual = 0$$
$$\therefore\ Q_{MI\_DBCH-MI} = 366{,}000\ \mathrm{m^3/yr}$$

<u>*Moriches Inlet: DBCH [Sta FI46 to FI46.8]*</u>

The Moriches Inlet Downdrift Beach (DBCH-MI) cell extends 800 m from Sta. FI46 km to Sta. FI46.8 km. An average of volume changes computed using short and long profiles and shoreline data (21,000 m^3/yr) is considered representative of the 1995-2001 period. A detailed discussion of the DBCH changes is presented in Section 5. No fill was placed in this cell during the 1995 to 2001 period.

Applying the sediment balance equation to this cell and entering values gives,

$$Q_{MI\_DBCH-MI} - Q_{DBCH-MI\_FI3} - \Delta V_{DBCH-MI} + P_{DBCH-MI} - R_{DBCH-MI} = residual$$
$$366{,}000 - Q_{DBCH-MI\_FI3} - (21{,}000) + 0 - 0 = residual = 0$$
$$\therefore\ Q_{DBCH-MI\_FI3} = 345{,}000\ \mathrm{m^3/yr}$$

<u>*Fire Island Reach: FI3 [Sta. FI32 to FI46]*</u>

The FI3 cell is approximately 14 km long and it includes most of Smith Point County Park (all but the beaches adjacent to Moriches Inlet) and roughly the eastern two thirds of the Otis Pike Wilderness Area, including the Old Inlet area. Volume changes computed using an average of shoreline changes developed from USACE and NPS shoreline data (–63,000 m^3/yr, including sediment lost to sea level rise, 29,000 m^3/yr) are considered representative of the 1995-2001 period. Short and long profiles (0.9 profiles/km) yield an extremely large erosion rate which does not seem representative given that, according to the recent data reviewed as part of this study and work by others (Allen et al., 2002), Moriches Inlet appears to have bypassed sediment effectively from 1995 to 2001. Given this recent increase in bypassing is also seems reasonable that the erosion in this cell be lower that the estimate in the previous *Historic (1979-1995)* sediment budget which suggested net erosion within this cell at a rate of -116,000 m^3/yr (including sediment lost to sea level rise, 29,000 m^3/yr).

Nonetheless, significant differences between the various datasets, particularly the NPS shoreline data collected by Allen et al. (2002), illustrate the relatively high level of uncertainty associated with shoreline and volume changes estimates in Fire Island. Results from the various datasets are only slightly better correlated along the reminder of Fire Island.

There was a small amount of placement in this cell (see Appendix C) at a rate equivalent to 13,000 m^3/yr. The sediment budget is balanced assuming net transport to the west.

Applying the sediment balance equation to this cell and entering values gives,

$$Q_{DBCH-MI\_F3} - Q_{FI3\_FI2} - \Delta V_{FI3} + P_{FI3} - R_{FI3} = residual$$
$$345{,}000 - Q_{FI3\_FI2} - (-63{,}000) + 13{,}000 - 0 = residual = 0$$
$$\therefore\ Q_{FI3\_FI2} = 421{,}000\ \mathrm{m^3/yr}$$

<u>*Fire Island Reach: FI2 [Sta. FI17 to FI32]*</u>

The FI2 cell extends roughly 15 km from Sta. FI17 Sta. FI 32 and it includes the western one third of the Wilderness Area, the Watch Hill Visitor's Center, the central Fire Island communities of Davis Park, Water Island, Fire Island Pines and Cherry Grove, and several large undeveloped Federal tracts (Carrington Tract, Talisman/Barrett Beach). Similarly to cell FI3, volume changes computed using an average of shoreline changes developed from USACE and NPS data (accumulation of 132,000 m^3/yr, including 31,000 m^3/yr due to sea level rise) are considered representative of the 1995-2001 period. This number is similar to the value obtained using long profiles only, and slightly larger than the value obtained from short and long profiles, which also suggest accretion in this area (Table 6-3). The previous *Existing (c. 1999)* conditions regional sediment budget indicated accumulation in this cell at the rate of 60,000 m^3/yr (including sediment lost to sea level rise, 31,000 m^3/yr).

There was fill placement in this cell from 1995 to 2001 at a rate of 104,000 m^3/yr, mostly within Fire Island Pines (see Appendix C). The sediment budget is balanced assuming net transport to the west.

Applying the sediment balance equation to this cell and entering values gives,

$$Q_{FI3\_FI2} - Q_{FI2\_FI1} - \Delta V_{FI2} + P_{FI2} - R_{FI2} = residual$$
$$421{,}000 - Q_{FI2\_FI1} - (132{,}000) + 104{,}000 - 0 = residual = 0$$
$$\therefore\ Q_{FI2\_FI1} = 393{,}000\ \mathrm{m^3/yr}$$

<u>*Fire Island Reach: FI1 [Sta. FI3.8 to FI17]*</u>

The FI1 cell extends roughly 13.2 km from Sta. FI3.8, approximately at the traffic circle at Robert Moses State Park (RMSP), to Sta. FI17. FI1 includes all of the western Fire Island communities from Kismet to Point O' Woods as well as the eastern half of RMSP, the Fire Island Lighthouse Tract, Sunken Forest, Sailor's Haven Visitors Center and a few other small undeveloped Federal tracts. Again, volume changes computed using an average of shoreline changes developed from USACE and NPS data (accumulation of 75,000 m^3/yr, including 28,000 m^3/yr due to sea level rise) are considered representative of the 1995-2001 period. Sparse long profile data suggest erosion while denser long and short profile data show accretion at a slightly higher rate than the one considered representative (Table 6-3). The previous *Existing (c. 1999)* conditions regional sediment budget indicated accumulation at the rate of 8,000 m^3/yr (including sediment lost to sea level rise, 28,000 m^3/yr) in F1.

Although fill was placed within this cell in 1994 and again in 2003, there no placement in this cell from 1995 to 2001. The *Recent* sediment budget is balanced assuming net transport to the west.

Applying the sediment balance equation to this cell and entering values gives,

$$Q_{F2\_F1} - Q_{F1\_UBCH-FII} - \Delta V_{F1} + P_{F1} - R_{F1} = residual$$
$$393{,}000 - Q_{F1\_UBCH-FII} - (75{,}000) + 0 - 0 = residual = 0$$
$$\therefore\ Q_{F1\_UBCH-FII} = 318{,}000\ \mathrm{m^3/yr}$$

Fire Island Inlet: UBCH [Sta FI0.075 to Sta. FI3.8]

The Updrift Beach (UBCH) at Fire Island Inlet cell extends from just east of the Democrat Point breakwater to 3.8 km east of the inlet. An average of the volume changes based on USACE shoreline data and based on profile data (i.e., -14,000 m^3/yr) was assumed as representative of conditions in this cell during the 1995-2001 period.

In addition, part of the material dredged from the Fire Island Inlet channel and deposition basin is typically placed in the UBCH cell. In recent years (1997 to 2002) fill has been placed within this cell at a rate of roughly 62,000 m^3/yr. Applying the sediment balance equation to this cell and entering values gives,

$$Q_{F1\_UBCH-FII} - Q_{UBCH-FII\_FII} - \Delta V_{UBCH-FII} + P_{UBCH-FII} - R_{UBCH-FII} = residual$$
$$318{,}000 - Q_{UBCH-FII\_FII} - (-14{,}000) + 62{,}000 - 0 = residual = 0$$
$$\therefore\ Q_{UBCH-FII\_FII} = 394{,}000\ \mathrm{m}^3/\mathrm{yr}$$

As in the case of Moriches Inlet, the estimated westerly transport arriving at Fire Island Inlet (394,000 m^3/yr) during the 1995-2001 period is significantly higher than the value from the previous *Existing (c. 1999)* conditions budget, which was 182,000 m^3/yr. However, it fits better within the rage of other previously published estimates (344,000 m^3/yr to 460,000 m^3/yr).

Fire Island Inlet

For the regional budget, the cells presented in Section 5.3.1 are collapsed into an overarching Fire Island Inlet cell. The total volume changes in the inlet during the 1995 to 2001 period add up to a net accumulation (i.e., sink) of 159,000 m^3/yr. Refer to Section 5.3.1 for a complete discussion of inlet sediment budget details.

Applying the sediment balance equation to this cell and entering values gives,

$$Q_{UBCH-FII\_FII} - Q_{FII\_DBCH-FII} - \Delta V_{FII} + P_{FII} - R_{FII} = residual$$
$$394{,}000 - Q_{FII\_DBCH-FII} - 159{,}000 + 0 - 375{,}000 = residual = 0$$
$$\therefore\ Q_{FII\_DBCH-FII} = -140{,}000\ \mathrm{m}^3/\mathrm{yr}$$

In other words, unless additional material entered the inlet from the east during this period, a balanced sediment budget suggests that approximately 140,000 m^3/yr are transported into the Ebb Shoal from the west as net easterly directed sediment transport. As explained above, this somewhat counterintuitive result may be due to inaccuracies in the *Recent (1995-2001)* regional sediment budget, an offshore sediment supply, or a real sediment transport reversal in this area as a result of significant volumes of sand being placed in the DBCH cell (see below).

Fire Island Inlet: DBCH [Extends approximately 3.6 km]

Volume changes at the beaches west of Fire Island indicate mild erosion (-4,000 m^3/yr) from shoreline position data and accretion (51,000 m^3/yr) from profile changes. Considering sea level rise as in the previous work, approximately 8,000 m^3/yr would be lost to SLR in this cell. If SLR losses are incorporated into the volume changes from shoreline change data, DBCH would be trapping 4,000 m^3/yr of sediment. Since less than one profile per km of shoreline was available in this cell, an average of the net change rate based on shoreline data and based on profile data (i.e., 28,000 m^3/yr) was assumed as representative of the changes in the UBCH cell during the 1995-2001 period.

In addition, most of the sand dredged from the Fire Island Inlet channel and deposition basin, 313,000 m^3/yr, is placed in this cell. Applying the sediment balance equation to this cell and entering values gives,

$$Q_{FII\_DBCH-FII} - Q_{DBCH-FII\_West} - \Delta V_{DBCH-FII} + P_{DBCH-FII} - R_{DBCH-FII} = residual = 0$$
$$-140{,}000 - Q_{DBCH-FII\_West} - 28{,}000 + 313{,}000 + -0 = residual = 0$$
$$\therefore\ Q_{DBCH-FII\_West} = 145{,}000\ \mathrm{m^3/yr}$$

<u>*Summary of Results for the Recent (1995-2001) Regional Sediment Budget*</u>

Estimated volume change rates and fill rates in each of the sediment budget cells from east (Montauk Point) to west (Fire Island Inlet) for the *Recent (1979-1995)* regional sediment budget are summarized in Table 6-4 and Figure 6-7, with longshore transport rates at the western boundary of each cell. The data sources used to determine the volume change rates are also indicated.

Qualitatively, this budget is similar to previous studies in that it shows increasing transport from east to west and it also shows that erosion along the beaches from Montauk Point to Southampton is the main source for a relatively large net westerly directed longshore sediment transport rate at updrift of Shinnecock Inlet (68,000 to 304,000 m^3/yr shown in previous studies, see Section 6.2). The budget also shows erosion along the two barrier island reaches downdrift of Shinnecock and Moriches Inlet: W4 (Tiana Beach) and FI3 (Smith Point County Park and the eastern end of the Wilderness Area), respectively. In fact, erosion rates in reach W4 are very similar to those shown in Kana (1995) and in Gravens et al. (1999), which were approximately 50,000 to 60,000 m^3/yr. On the other hand, erosion rates in the FI3 cell during the 1995-2001 period were roughly half of those shown in those two studies (100,000 to 120,000 m^3/yr). As explained above, this new result seems reasonable considering that Moriches Inlet appears to have been bypassing sand fairly efficiently in recent years.

In fact, perhaps the most significant difference between the *Recent (1995-2001)* budget and previous studies (particular Gravens et al., 1999 and USACE-NAN, 1999) is that Shinnecock and Moriches Inlet, and to smaller extent the Westhampton groin field, do not appear to be intercepting as much of the westerly sand flow as they had in the past. This seems reasonable considering that these two inlets have now been open for more than 70 years and stabilized with rock jetties for over 50 years. And although recent inlet modifications at Moriches Inlet (1986) and Shinnecock Inlet (1990) caused profound changes to the configuration of the channel and the ebb shoal, they do not appear to have caused a significant net increase in ebb shoal volume (see Section 5). However, this finding should be viewed somewhat skeptically until additional surveys are collected and analyzed over the next decade or so to confirm or refute it. Additional discussion regarding expected medium- to long-term trends at the inlets is presented in the following section.

As in the previous studies, particularly in Kana (1995), central Fire Island shoreline (cell F2) appears to be fairly stable or even slightly accreting. The *Recent (1995-2001)* budget also shows net accretion in western Fire Island (75,000 m^3/yr in cell FI1), whereas Gravens et al. suggested very little net accumulation (8,000 m^3/yr) and Kana showed significant erosion (more than 150,000 m^3/yr) despite some fill (roughly 25,000 m^3/yr) being placed in this area during the analysis period for that budget (1955-1979). Kana also shows high erosion rates within Robert Moses State Park between 1955 and 1979 (42,000 m^3/yr) despite fill at rate of 14,000 m^3/yr.

Table 6-4: *Recent (1979-1995)* Regional Sediment Budget Summary					
Morphologic Zone	**Stationing (km east of each inlet)**	**Volume Change Rate (1000 m^3/yr)**	**Fill Rate (1000 m^3/yr)**	**LST (1000 m^3/yr)**	**Volume Change Rate Data Source (All shoreline change rates include sea level rise effects)**
M5	58.1 to 50	-80	1	81	Short/Long Profiles
M4	50 to 44	82	--	-1	Short/Long Profiles
M3	44 to 24	-71	--	70	Shoreline Change-USACE data
M2	24 to 13	-150	--	220	Short/Long Profiles
M1	13 to 3.2	-59	--	279	Short/Long Profiles
UBCH-S	3.2 to 0.6	33	--	246	Shoreline Change-USACE data
Shinnecock Inlet		-7	--[10]	253	See Section 5.1
DBCH-S	22.4 to 21.6	2	--	251	Short/Long Profiles
W4	21.6 to 12.9	-57	--	308	Short/Long Profiles
W3	12.9 to 10.8	0	23	331	Shoreline Change-USACE data
W2	10.8 to 5.1	411	407	327	Long Profiles
W1	5.1 to 3.2	123	233	437	Long Profiles
UBCH-M	3.2 to 0.6	69	--	368	Average of Short/Long Profiles and Shoreline Change-USACE data
Moriches Inlet		2	--[10]	366	See Section 5.2
DBCH-M	46.8 to 46	21	--	345	Average of Short/Long Profiles and Shoreline Change-USACE data
FI3	46 to 32	-63	13	421	Average of Shoreline Change-NPS and Shoreline Change-USACE data
FI2	32 to 17	132	104	393	Average of Shoreline Change-NPS and Shoreline Change-USACE data
FI1	17 to 3.8	75	--	318	Average of Shoreline Change-NPS and Shoreline Change-USACE data
UBCH-FI	3.8 to 0.075	-14	62	394	Average of Short/Long Profiles and Shoreline Change-USACE data
Fire Island Inlet		159	-375	-140	See Section 5.3
DBCH-FI		28	313	145	Average of Short/Long Profiles and Shoreline Change-USACE data

Computed net westerly transport entering Fire Island Inlet between 1995 and 2001 (394,000 m^3/yr) compares favorably with the range of estimates (including Panuzio, 1969; RPI, 1985; Kana, 1995) prior to Gravens et al. (1999), which shows a significantly lower estimate of 194,000 m^3/yr. Increased

[10] Dredge and Fill (including 3,000 m^3/yr to an upland stockpile) are roughly in balance

sediment supply from updrift as a result of more efficient bypassing around Shinnecock and Moriches Inlet and, more importantly, the Westhampton groin field, combined with a large amount of fill placed at Westhampton may be at least partially responsible for increased westerly transport along Fire Island and at Fire Island Inlet between 1995 and 2001. In previous studies these large westerly transport estimates were arrived at on the basis of historic spit growth analysis at Fire Island and updrift fillet accumulation after construction of the Democrat Point breakwater, however updrift volume changes from Fire Island to Montauk Point did not support that much transport at Fire Island and thus required other sources of sediment such as an offshore supply. Kana (1995) speculated that up until the early 1900s the source of this sediment was an abandoned delta off western Fire Island whereas between 1979 and 1995 this relict source had largely disappeared and the foreshore in western Fire Island was being "cannibalized" instead. Note that the more recent spit growth and impoundment analysis performed by Gravens et al. (1999) suggest slightly lower longshore sediment transport rates than Taney (1961a,b): 159,000 to 300,000 m^3/yr based on spit growth[11] and 385,000 m^3/yr based on impoundment at Democrat Point. The authors considered the latter estimate to be most likely "high" because it probably included "some contribution due to onshore welding of the eastern portion of the Fire Island ebb shoal" after construction of the east jetty.

Note that the fact the *Recent (1979-1995)* sediment budget does not necessarily require an offshore sediment source to yield an estimate of net westerly transport arriving at Fire Island Inlet that matches estimates based on spit growth prior to stabilization or impoundment at Democrat Point. However, this does not necessarily mean that there is no offshore source. In fact, accumulation within the inlet and dredging rates still yield a somewhat low westerly transport rate on Gilgo Beach downdrift of Fire Island Inlet (145,000 m^3/yr), which would be increased by an offshore source of sediment.

6.6 *Existing (c. 2001)* Regional Sediment Budget

As explained above, the *Recent* sediment budget is only representative of the 1995-2001 period and should not be used to predict medium- to long-term trends (10-20 year) in the FIMP area. A new *Existing* sediment budget was developed for that purpose. This *Existing (c. 2001)* regional sediment budget incorporates, to the extent possible, relevant long-term trends identified in Gravens et al. (1999) as well as recent changes shown in the 1995-2001 sediment budget, including relatively new inlet and shoreline management practices such as the deposition basin at Shinnecock Inlet and the Westhampton Interim Project.

To develop this new *Existing (c. 2001)* regional sediment budget, the *Recent (1995-2001)* regional budget was used in conjunction with the previous *Historic (1979-1995)* and *Existing (c. 1999)* regional sediment budget developed by Gravens et al. In most cases, estimates of volume change rates for the barrier island cells under *Existing (c. 2001)* conditions were computed as a prorated average of the *Recent (1995-2001)* and *Historic (1979-1995)* changes, which effectively results in an estimate of the long-term (1979 to 2001) changes in that cell. 1995-2001 estimates alone where used in cells where the recent trends are considered more representative of existing and future conditions (e.g., FI3). At the inlets, an attempt was made to account for recent management and morphological evolution changes without discounting previously identified long term trends and established theories regarding the impacts that inlets have on longshore sediment transport and a barrier island processes.

It was assumed that beach fill practices in Montauk Beach (cell M5), Westhampton, and Fire Island (mostly at Fire Island Pines, the westernmost Fire Island communities, and RMSP) would continue at rate similar to that in the 1990s and early 2000s. Of course, large storms or specific hot spots may require placement of fill in areas that did not receive fill during that period (e.g., Ocean Beach) which would

[11] Gravens et al. (1999) developed two estimates based on different active beach depths. See Gravens et al. (1999) for details.

affect the sediment budget at least temporarily. Assumptions regarding the behavior of the fill placed at Westhampton Beach were made based on previous work by Gravens et al. (1999) and the changes observed so far since project construction in 1996-97.

Finally, computed longshore sediment transport rates were compared with results from previous studies and checked against estimates developed by Gravens et al. (1999) using the Wave Information Study (WIS) 1976 to 1994 wave hindcast database and the GENEralized model for SImulating Shoreline change (GENESIS) developed by Hanson and Kraus (Hanson, 1987; Hanson and Kraus, 1989). Gravens et al. calculated net and gross LST rates from Fire Island to approximately 6 km west of Montauk Point. Their model was calibrated such that the magnitude of the potential sediment transport rate at Fire Island Inlet agreed with accepted rates. Therefore the long-term accuracy of these computed potential transport rates is limited by the accuracy of the accepted rates at Fire Island inlet and the degree to which the wave climate in the 1976 to 1994 is representative of average long-term conditions. Nonetheless, results of the *Existing (c. 2001)* conditions sediment budget were checked against the model results and assumptions and/or results were modified, if necessary.

The proposed *Existing (c. 2001)* conditions regional sediment budget is summarized in Table 6-5 and Figure 6-8. This budget reflects coastal processes, inlet management activities, and beach fill placement rates assumed to be representative of the present time (c. 2001) and medium- to long-term conditions in the FIMP project area. Major assumptions used to develop this budget and some of the most significant results obtained were as follows:

6.6.1 Montauk Cells

As in previous studies and in the *Recent (1979-1995)* regional sediment budget it was assumed that sediment transport generally increases from east to west from the initial source at Montauk. From Montauk Point to Southampton (cells M5, M4, M3, M2 and M1), volume changes were computed as a prorated average of the *Recent (1995-2001)* and *Historic (1979-1995)* changes. This approach seemed reasonable given that shoreline management practices have not changed significantly in this reach since construction of the Georgica Pond groins in the 1960s. This approach results in a net erosion volume of 156,000 m^3/yr within the M reach, which, combined with placement of 1,000 m^3/yr in M5, yields a net longshore sand transport arriving at cell UBCH-SI of 157,000 m^3/yr. Note that most of the erosion occurs in cells M5, M2 and M1, whereas M4 and M3 appear to be fairly stable overall.

As explained in Section 5.1, the Updrift Beach cell at Shinnecock Inlet (UBCH-SI) is considered to be stable and continued impoundment in this cell due to the east jetty is not expected in the medium- to long-term. Therefore, it was assumed that only a volume of sediment enough to offset sea level rise and maintain the existing shoreline position would accumulate within this cell (6,000 m^3/yr). Gravens et al. made a similar assumption in their *Existing (c. 1999)* sediment budget. This results in 151,000 m^3/yr entering Shinnecock Inlet under *Existing (c. 2001)* conditions. This estimate falls in the middle of the range of other previously published estimates (68,000 to 304,000 m^3/yr, see Section 6.2) and is only slightly higher than the value proposed by Gravens et al. (130,000 m^3/yr), which was mostly based on the potential net LST computations using GENESIS. Note that the value estimated by Kana (1995) was 220,000 m^3/yr, which was also similar to Panuzio (1968).

Table 6-5: *Existing (c. 2001)* Regional Sediment Budget Summary					
Morphologic Zone	**Stationing (km east of each inlet)**	**Volume Change Rate (1000 m^3/yr)**	**Fill Rate (1000 m^3/yr)**	**LST (1000 m^3/yr)**	**Volume Change Rate Data Source**
M5	58.1 to 50	-90	1	91	Prorated Average[12]
M4	50 to 44	26	--	65	Prorated Average
M3	44 to 24	1	--	64	Prorated Average
M2	24 to 13	-70	--	134	Prorated Average
M1	13 to 3.2	-23	--	157	Prorated Average
UBCH-S	3.2 to 0.6	6	--	151	Assumed based on relative shoreline stability.
Shinnecock Inlet		32	--[13]	119	See Section 5.1
DBCH-S	22.4 to 21.6	2	--	117	Assumed based on relative shoreline stability
W4	21.6 to 12.9	-55	--	172	Prorated Average
W3	12.9 to 10.8	5	--	167	Prorated Average
W2	10.8 to 5.1	100	125	192	Assumed. See text below
W1	5.1 to 3.2	50	125	267	Assumed. See text below
UBCH-M	3.2 to 0.6	29	--	238	Equal to *Existing (c. 1999)*
Moriches Inlet		25	--[13]	213	See Section 5.2
DBCH-M	46.8 to 46	2	--	211	Assumed based on relative shoreline stability
FI3	46 to 32	-63	--	274	Equal to 1995-2001
FI2	32 to 17	78	100	296	Prorated Average
FI1	17 to 3.8	25	--	351	Prorated Average
UBCH-FI	3.8 to 0.075	9	69	404	Assumed based on relative shoreline stability
Fire Island Inlet		108	-375	-79	See Section 5.3
DBCH-FI		28	313	206	Equal to 1995-2001

6.6.2 Shinnecock Inlet

Based on recent inlet management practices, available surveys and assumptions regarding accumulation due to sea level rise, it was concluded in Section 5.1 that under *Existing (c. 2001)* conditions approximately 79% (119,000 m^3/yr) of the net updrift westerly transport entering the ebb shoal at Shinnecock Inlet (151,000 m^3/yr) bypasses the inlet system, which includes the channels, deposition basin, shoals and west beach. The remaining 21% (32,000 m^3/yr) accumulates within the inlet shoals. As

[12] Volume change rate computed as a prorated average of the *Recent (1995-2001)* and *Historic (1979-1995)* changes.
[13] Dredge and Fill are roughly in balance.

explained above, this finding seems consistent with the idea that the inlet system is more mature and has further adjusted to the configuration implemented in 1990 when the deposition basin was dredged and the channel was realigned. Note that the prorated average of 1979-1995 and 1995-2001 changes would be 49,000 m^3/yr, so considering that the inlet may now be closer to a dynamic equilibrium, 32,000 m^3/yr seem reasonable.

As explained in Section 5.1, the Downdrift Beach cell at Shinnecock Inlet (UBCH-SI) is also considered to be stable with enough accumulation to offset sea level rise and maintain the existing shoreline position (2,000 m^3/yr). This results in a net longshore transport rate entering the Westhampton reach of 117,000 m^3/yr, only slightly higher than the net potential LST rate computed by Gravens et al. (1999) using GENESIS.

6.6.3 Westhampton Cells

Because of similar shoreline management practices and lack of major engineering works between 1979 and 2001, volume changes in cells W4 and W3 were also computed as a prorated average of the *Recent (1995-2001)* and *Historic (1979-1995)* changes. The *Existing (c. 2001)* condition sediment budget continues to show erosion within in cell W4 (Tiana Beach) at rate very similar to the previous *Historic (1979-1995)* and *Existing (c. 1999)* sediment budgets (54,000 m^3/yr), whereas the cell W3 (Hampton Beach) continues to be stable, probably as a result of the stabilizing effect that the Westhampton groin field located immediately downdrift has on this shoreline. The net erosion for cells W4 and W3 combined is 50,000 m^3/yr, whereas the previous *Existing (c. 1999)* sediment budget showed 46,000 m^3/yr and Kana (1995) showed a slightly larger value of 63,000 m^3/yr. This moderate erosion contributes to an increase in the longshore sediment transport rate entering Westhampton groin field. The new *Existing (c. 2001)* condition budget yields a net westerly LST value into this cell of 167,000 m^3/yr compared to 111,000 m^3/yr in the previous *Existing (c. 1999)* budget and 164,000 m^3/yr in Kana (1995). The estimates in this new *Existing (c. 2001)* condition budget and in Kana compare very well with the net LST value computed with GENESIS which is roughly 160,000 m^3/yr (Gravens et al., 1999)

Note that the *Existing (c. 2001)* condition budget proposed herein will be temporarily altered by the recent implementation of the West of Shinnecock Interim (WOSI) Storm Damage Project. Initial beach fill placement as part of WOSI was completed in March 2005. The project includes two additional renourishments for a period not to exceed 6 years. During this period material dredged from the Shinnecock Inlet deposition basin will be placed farther downdrift and closer to Tiana Beach. Therefore, it is expected that Tiana Beach will receive an additional influx of sediment (See Section 5.1). It is also assumed that after WOSI ends (i.e., 2011) conditions will gradually revert back to *Existing* as described above.

The Westhampton Interim project includes periodic nourishment (3- to 6-year interval), as necessary to ensure the integrity of the project design, for up to 30 years since its original construction in 1997 (i.e., 2027). The first renourishment was completed in February 2001 and included placement of 723,000 m^3. The second nourishment was in 2005 and approximately 641,000 m^3 were placed. The long-term average renourishment rate will be approximately 250,000 m^3/yr. Placement area will likely include a portion of the Westhampton groin field (cell W2) and the downdrift barrier beach (cell W1, Pikes Beach). Although exact fill placement patterns will be determined prior to each renourishment cycle, for the purposes of the sediment budget it was assumed that approximately half the volume will be placed in each of the two cells (i.e., 125,000 m^3/yr).

The previous *Existing (c. 1999)* conditions budget assumed that 50% of the renourishment fill in cell W2 would move alongshore, with the remaining 50% captured by the groin field or moving across-shore. In addition, the previous budget assume that 75% of the renourishment fill in cell W1 would move alongshore, with the remaining 25% accumulating in this cell. Recent data collected since initial project construction in 1997 and the first renourishment in 2001 suggest that more material will accumulate in these two cells. Specifically, between 1995 and 2001 accumulation in cells W2 and W1 was 411,000 m^3/yr and 123,000 m^3/yr, respectively. These volumes are equivalent to roughly 100% and 50% of the volume placed in these cells. It is not likely that the groin field and downdrift beach will continue to accumulate sand as effectively, so for the purposes of the *Existing (c. 2001)* conditions budget it was assumed that cell W2 would retain 80% of the fill (100,000 m^3/yr) and cell W1 would retain 40% (50,000 m^3/yr).

The net effect of the sediment balance within cells W2 and W1 on the longshore sediment transport entering the Updrift Beach cell at Moriches Inlet (UBCH-MI) is an increase of 100,000 m^3/yr, from 167,000 m^3/yr to 267,000 m^3/yr. As explained in Section 5.2, the relatively high recent (1995-2001) accretion rate of 69,000 m^3/yr at cell UBCH-MI is probably not sustainable in the medium- to long-term. The previous *Existing (c. 1999)* sediment budget assumed that this cell would accrete at rate of 29,000 m^3/yr, including losses to sea level rise. This estimate is still considered reasonable and representative of *Existing* conditions. Therefore, the net longshore sediment transport entering Moriches Inlet is 238,000 m^3/yr. This estimate is larger than the estimate from the previous *Existing (c. 1999)* conditions budget, 184,000 m^3/yr, but within the range of values published in the literature (45,000 m^3/yr to 267,000 m^3/yr, see Section 6.2). The low end of this range corresponds to the values computed by Kana (1995), which covered the period from 1955 to 1979, including the first decade after construction of the groin field, when trapping effects were greatest. On other hand, a relatively high net LST entering the inlet seems reasonable considering the recent and expected future influence of the Westhampton Interim project. In addition, net potential longshore sediment transport computed with GENESIS at this location is slightly higher 200,000 m^3/yr (Gravens et al., 1999), which compares reasonably well with *Existing* conditions estimate. Moreover, Taney (1961a,b) estimated approximately 230,000 m^3/yr for the littoral transport rate entering Moriches Inlet. Note that this was under conditions prior to the construction of the Westhampton groin field and therefore similar to Existing conditions with the Westhampton Interim Project in place.

6.6.4 Moriches Inlet

Based on recent inlet management practices, available surveys and assumptions regarding accumulation due to sea level rise, it was concluded in Section 5.2 that under *Existing (c. 2001)* conditions approximately 89% (213,000 m^3/yr) of the net westerly transport conditions bypasses the Moriches Inlet system, which includes the channel, deposition basin, shoals and west beach. The remaining 11% (25,000 m^3/yr) is assumed to accumulate within the ebb and flood shoals as a result of sea level rise. As explained above, the ebb shoal has apparently adjusted to the changes implemented in 1986 and it has reached a relatively stable volume (apart from sea level rise effects) since its opening by a storm in 1931 and stabilization with jetties in 1953. The Downdrift Beach cell at Moriches Inlet (UBCH-MI) is also considered to be relatively stable (accumulation of 2,000 m^3/yr). This results in a net longshore transport rate of 211,000 m^3/yr entering the rest of Fire Island, a value only slightly higher than the potential LST rate computed by Gravens et al. (1999).

6.6.5 Fire Island Cells

Because of this apparent increase in bypassing efficiency at Moriches Inlet as well as the significant increase in sediment entering Moriches Inlet and therefore Fire Island, it was assumed that the recent (1995-2001) erosion rate within cell FI3 (Smith Point County Park and the eastern two thirds of the Wilderness Area) is representative of *Existing (c. 2001)* conditions. This rate, 63,000 m^3/yr, is about half of that value computed by Kana (1995) and Gravens et al (1999). Reduced erosion also seems more consistent with potential net LST rates in this area computed by Gravens et al., which actually suggest a decrease in longshore sediment transport and accumulation of sediment.

From west of Bellport Beach in the Wilderness Area to the traffic circle at RMSP (cells FI2 and FI1) volume changes were computed as a prorated average of the *Recent (1995-2001)* and *Historic (1979-1995)* changes. Therefore, central Fire Island shoreline (cell F2) continues to be stable in the Existing (c. 2001) sediment budget and actually it was computed to accumulate 78,000 m^3/yr. Note that potential net LST rates computed with GENESIS (Gravens et al., 1999) actually suggest increasing longshore transport and therefore erosion within this cell. This difference between potential transport and transport computed based on volume changes may be explained, at least partly, if in fact an offshore source of sediment exists from Watch Hill to Point O' Woods as suggested by Schwab et al. (1999). Approximately 200,000 m^3/yr would be required to offset the erosion predicted by GENESIS while still accreting 78,000 m^3/yr within this cell. In fact, Schwab et al. speculated that the magnitude of the onshore sediment flux ranges from 75,000 to 390,000 m^3/yr, so 200,000 m^3/yr seems a plausible number. Note that Gravens et al. (1999) suggested that the lower end of this range (75,000 m^3/yr) as reasonable instead based on results from their sediment budget and Fire Island spit growth estimates. In any case, as explained below, an offshore source of sediment was not required to balance the *Existing (c. 2001)* conditions budget at Fire Island Inlet or to yield reasonable estimates of longshore transport entering and exiting the inlet. Therefore, the *Existing (c. 2001)* condition regional budget presented herein does not explicitly include this offshore source although its possible existence and contribution to the nearshore sediment transport system is recognized.

Beach fill was assumed to continue at roughly the same rate within cell FI2, 100,000 m^3/yr, most of which is placed within Fire Island Pines. The most recent fill project at this community was in November 2003 (380,000 m^3). Prior to that, in 1997, 513,000 m^3 were placed at the same location.

According to the *Existing (c. 2001)* condition regional budget, cell FI1 (western Fire Island) also appears to be slightly accreting (25,000 m^3/yr). Even though beach fill was not placed within this cell during the 1995-2001 period (see above), fill has been historically placed in this reach. Specifically, a total of approximately 2 million m^3 of sand were placed between 1933 and 2001 (Kana, 1995; Gravens et al. 1999). Approximately half of this volume (1 million m^3) was placed between 1933 and 1979 with the other half between 1979 and 2001. In addition, approximately 534,000 m^3 were placed along the communities of Saltaire, Fair Harbor, Dunewood and Lonelyville in November 2003. The previous *Existing (c. 1999)* sediment budget assumes placement within this cell at rate of 80,000 m^3/yr based on *Historic (1979-1995)* placement rates. This estimate was also used in the new *Existing (c. 2001)* sediment budget. However, it should be noted that historically fill has been placed in this cell shortly after major storms such as Hurricane Donna in 1960, the 1962 Nor'easter and a series of the storms in the early 1990's including the Dec 1992 Nor'easter. Therefore, actual placement rates will probably depend on future storm cycles.

As explained in Section 5.3, the Updrift Beach at Fire Island Inlet (UBCH-FI), which extends approximately from the traffic circle to the jetty at Democrat Point is considered stable under existing conditions, which include fill placement, accumulating enough sand to offset the effects of sea level rise and maintain the existing shoreline position (9,000 m^3/yr). This value is similar that proposed by Gravens et al in their *Existing (c. 1999)* condition sediment budget. Placement within this cell was assumed to continue at the recent rate of roughly 62,000 m^3/yr.

Therefore, the net longshore sediment transport entering Fire Island Inlet is 404,000 m^3/yr. Although this estimate is significantly larger than the estimate from the previous *Existing (c. 1999)* conditions budget, 188,000 m^3/yr, like the estimate under *Recent (1995-2001)* conditions, it fits better within the rage of other previously published estimates (344,000 m^3/yr to 460,000 m^3/yr). It also compares very well with net the potential longshore sediment transport rate computed with GENESIS (Gravens et al., 1999) at this location which suggests a value of nearly 400,000 m^3/yr. Note that, as explained above, the *Existing (c.*

2001) condition regional budget as formulated herein does not require the contribution of an offshore source of sediment to obtain a westerly transport rate entering the inlet that matches spit growth and updrift impoundment estimates. However, this is not to say that this source does not exist. In fact, as stated above, potential net LST gradients computed with GENESIS in central Fire Island (cell F2) suggest that this source could be on the order of 200,000 m^3/yr.

6.6.6 Fire Island Inlet

Based on historic and recent inlet management practices, a limited number of incomplete surveys and assumptions regarding accumulation due to sea level rise, it was concluded in Section 5.3 approximately 59% (206,000 m^3/yr) of the net westerly transport under *Existing (c. 2001)* conditions bypasses the inlet system, which includes the channels, deposition basin, the ebb shoal, and the adjacent updrift and downdrift beaches[14]. The other 41% (145,000 m^3/yr) accumulates within the ebb shoal, the channels and the adjacent beaches, some of it as a result of sea level rise. The number of bathymetry surveys and the coverage of these surveys at Fire Island Inlet is not as good as at Moriches or Shinnecock Inlet. There is only one survey (a multi-beam hydrographic survey performed in 2001 by Stony Brook University) with adequate ebb shoal coverage. The others, including a SHOALS survey in 1996, do not include a significant portion of the western flank of the ebb shoal. Condition surveys and pre- and post-dredging surveys are typically performed over the channel and deposition basin areas only, so coverage is even more limited (see Section 5.3 and Appendix A). Therefore, total sediment accumulation within the inlet shoals may be greater than 145,000 m^3/yr, although it seems very unlikely that it would be as high as the previous estimate by USACE-NAN (1998) of 535,000 m^3/yr.

Moreover, the net westerly longshore transport downdrift of inlet estimated with the sediment budget (206,000 m^3/yr) appears to compare reasonably well with potential net LST rates computed with GENESIS along central Fire Island (10 to 20 km east of Fire Island Inlet), which has a similar shoreline orientation as the downdrift shoreline at Jones Island. Therefore, unless there is additional sediment entering Fire Island Inlet (from an offshore source for example), the net accumulation rate computed at Fire Island Inlet seems reasonable.

6.6.7 Conclusions

An *Existing (c. 2001)* conditions sediment budget presenting estimates of volume changes and longshore sediment transport rates for 18 beach cells and 3 inlets within the FIMP study area was developed using available survey data. The budget incorporates, to the extent possible, relevant long-term trends identified in previous studies as well as recent changes, including relatively new inlet and shoreline management practices such as the deposition basin at Shinnecock Inlet and the Westhampton Interim Project.

Most estimates of volume change rates for the beach cells were computed as a prorated average of the *Recent (1995-2001)* and *Historic (1979-1995)* changes, which effectively results in an estimate of the long-term (1979 to 2001) changes in that cell. 1995-2001 estimates alone where used in cells where the recent trends are considered more representative of existing and future conditions (e.g., FI3). At the inlets, an attempt was made to account for recent management and morphological evolution changes without discounting previously identified long term trends and established theories regarding the impacts that inlets have on longshore sediment transport and a barrier island processes.

Overall, this budget shows longshore sediment transport rates that fall within the range of previously published estimates (e.g., 151,000 m^3/yr, 238,000 m^3/yr, and 404,000 m^3/yr entering Shinnecock,

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[14] Note that at Fire Island Inlet, changes on the adjacent updrift and downdrift beaches were included in the bypassing efficiency calculation because all the sand dredged from the Deposition Basin was placed within these two cells

Moriches, and Fire Island Inlets, respectively). Transport appears to increase from east to west and the initial source of sediment feeding the net longshore sediment transport from east to west appears to be erosion along the beaches from Montauk Point to Southampton, specifically in cells M5, M2, and M1.

The budget suggests that the effects of the Westhampton groin field have been largely offset by the construction of the Westhampton Interim Project. Specifically, the estimate of sediment entering Moriches Inlet (238,000 m^3/yr) is higher than values presented in other recent studies (e.g., Kana, 1995) and very similar to the estimate by Taney (1961a,b) of 230,000 m^3/yr under conditions prior to the construction of the Westhampton groin field.

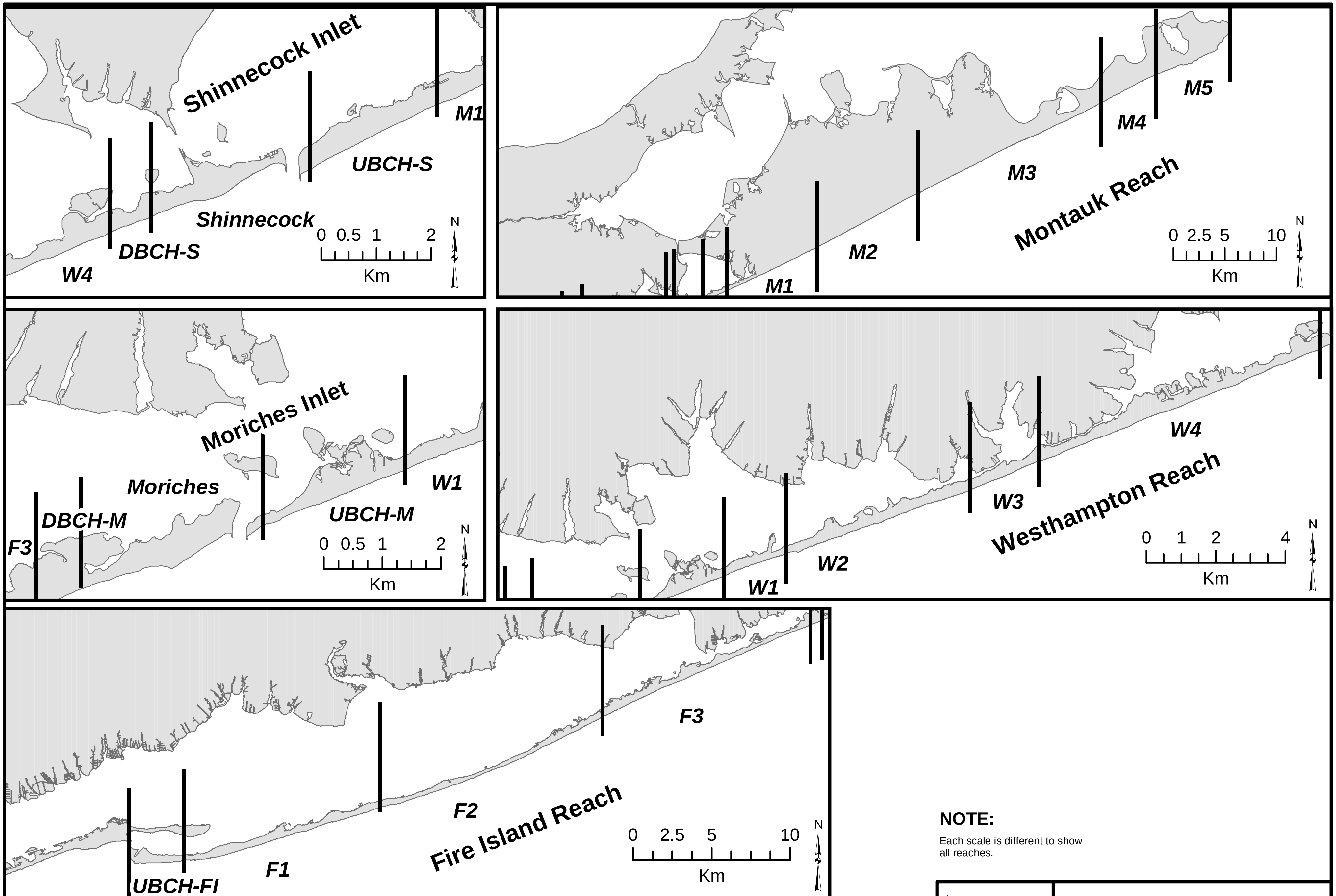
Also similarly to previous studies, the *Existing (c. 2001)* condition budget suggest erosion along the two barrier island reaches downdrift of Shinnecock and Moriches Inlet: W4 (Tiana Beach) and FI3 (Smith Point County Park and the eastern end of the Wilderness Area), respectively, albeit at somewhat smaller rates, particularly at cell FI3. This reduction may be a result of increased bypassing at Shinnecock and Moriches Inlet in recent years.

Nonetheless, the three inlets in the FIMP study area, particularly Fire Island Inlet, continue to be a sediment sink. Specifically, available surveys and assumptions regarding the effects of sea level rise on inlet morphology suggest that Shinnecock, Moriches, and Fire Island Inlet accumulate 32,000, 25,000, and 108,000 m^3/yr, respectively. Therefore, the total loss to the system is 165,000 m^3/yr, which represents a significant percentage of the average longshore sediment transport along the FIMP shoreline.

On the other hand, approximately 431,000 m^3/yr of beach fill dredged from offshore sources are placed along the shoreline between Montauk Point to Fire Island Inlet, mostly as part of the Westhampton Interim Project (250,000 m^3/yr).

The *Existing (c. 2001)* condition regional budget does not explicitly include an offshore sediment source because it was not required to balance the budget at Fire Island Inlet or to yield reasonable estimates of longshore transport entering and exiting the inlet. Although its possible existence and contribution to the nearshore sediment transport system is recognized. Specifically, differences between potential net transport computed with GENESIS and transport computed based on volume changes in central Fire Island suggest an onshore sediment flux of approximately 200,000 m^3/yr to explain the well documented relative shoreline stability in this area. This value matches the estimate suggested by Schwab et al. (2000) based on the sediment budget by Kana (1995). However, Gravens et al. (1999) suggested a lower value, 75,000 m^3/yr, based on results from their sediment budget and Fire Island spit growth estimates

A relatively large number of data sources were used to develop this sediment budget, including shorelines digitized from aerial photography, shorelines surveyed using an ATV and a GPS system, beach profile surveys, boat-based bathymetric surveys, and LIDAR surveys. There are obvious benefits associated with a large dataset, such a spatial and temporal coverage. However, large differences in the results obtained from each dataset (e.g., volume changes based on shoreline vs. profile data) also underscore the significant level of uncertainty associated with this type of study. Although a detailed quantitative analysis was not possible because many of the individual uncertainty contributions cannot be determined (e.g., uncertainty due to lack of survey coverage at the inlets or due to differences in datum reduction methodologies), it is judged that the uncertainty in the estimates presented above is significant, perhaps as much as the estimates themselves in some cases. Even so, it is concluded that the proposed *Existing (c. 2001)* condition sediment budget provides a realistic, albeit semi-quantitative, description of the sediment transport processes that can be used to assist in the planning, design, and formulation of shore protection and storm damage reduction measures for the FIMP project area.



NOTE:

Each scale is different to show
all reaches.

Regional Sediment Budget Reaches

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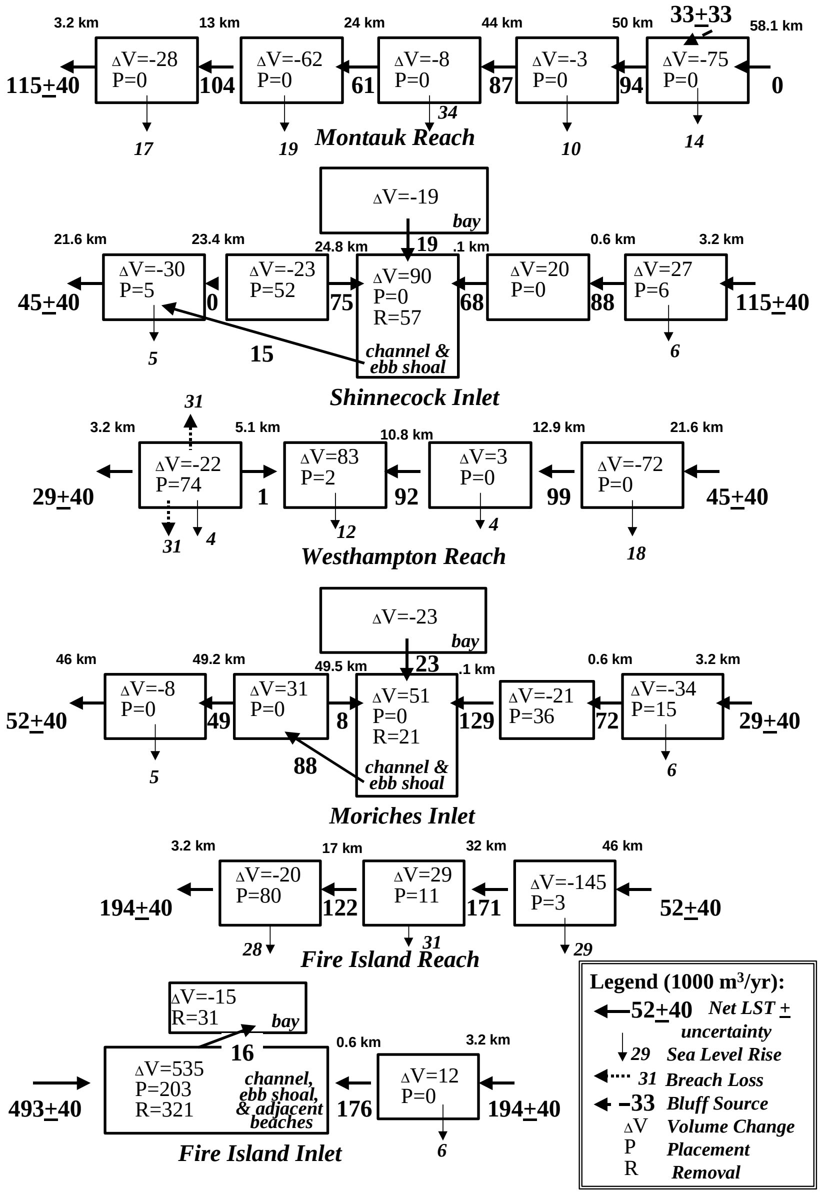
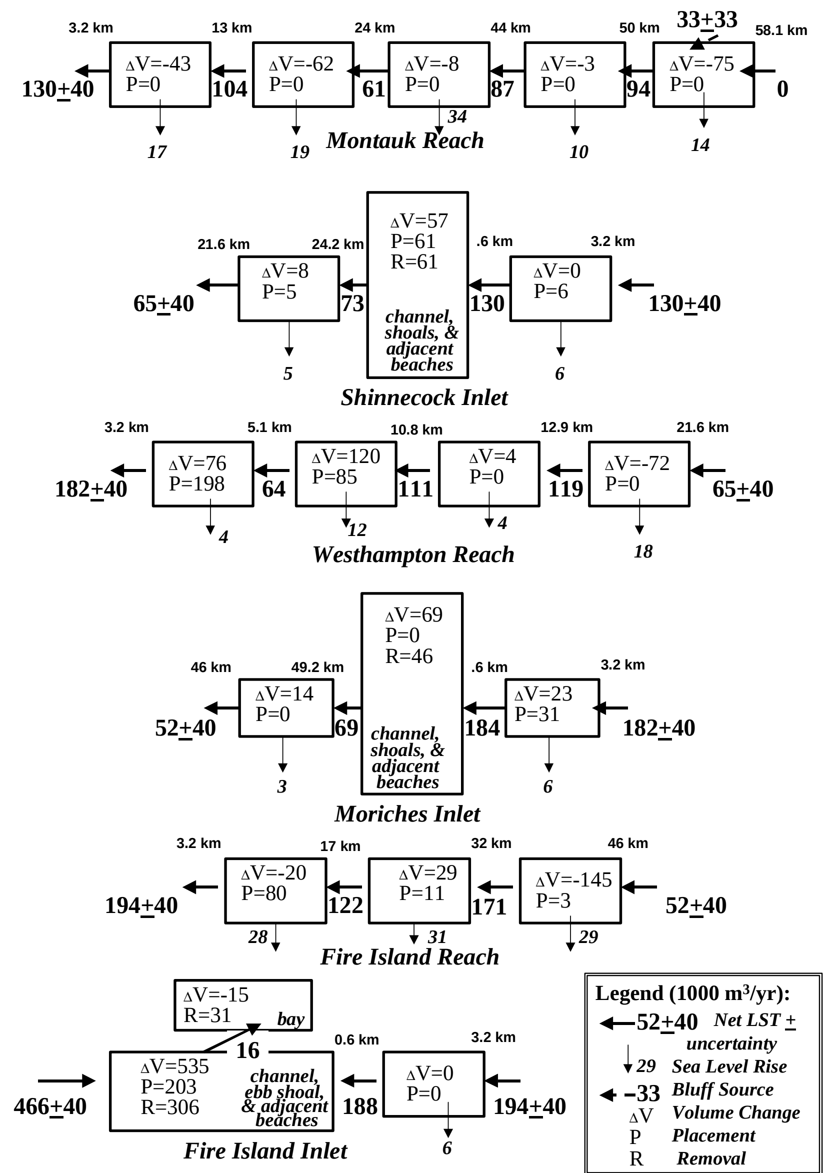
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 6-1

Filename: GIS/FIMP/BudgetCells.mxd



Historical sediment budget



Existing condition sediment budget

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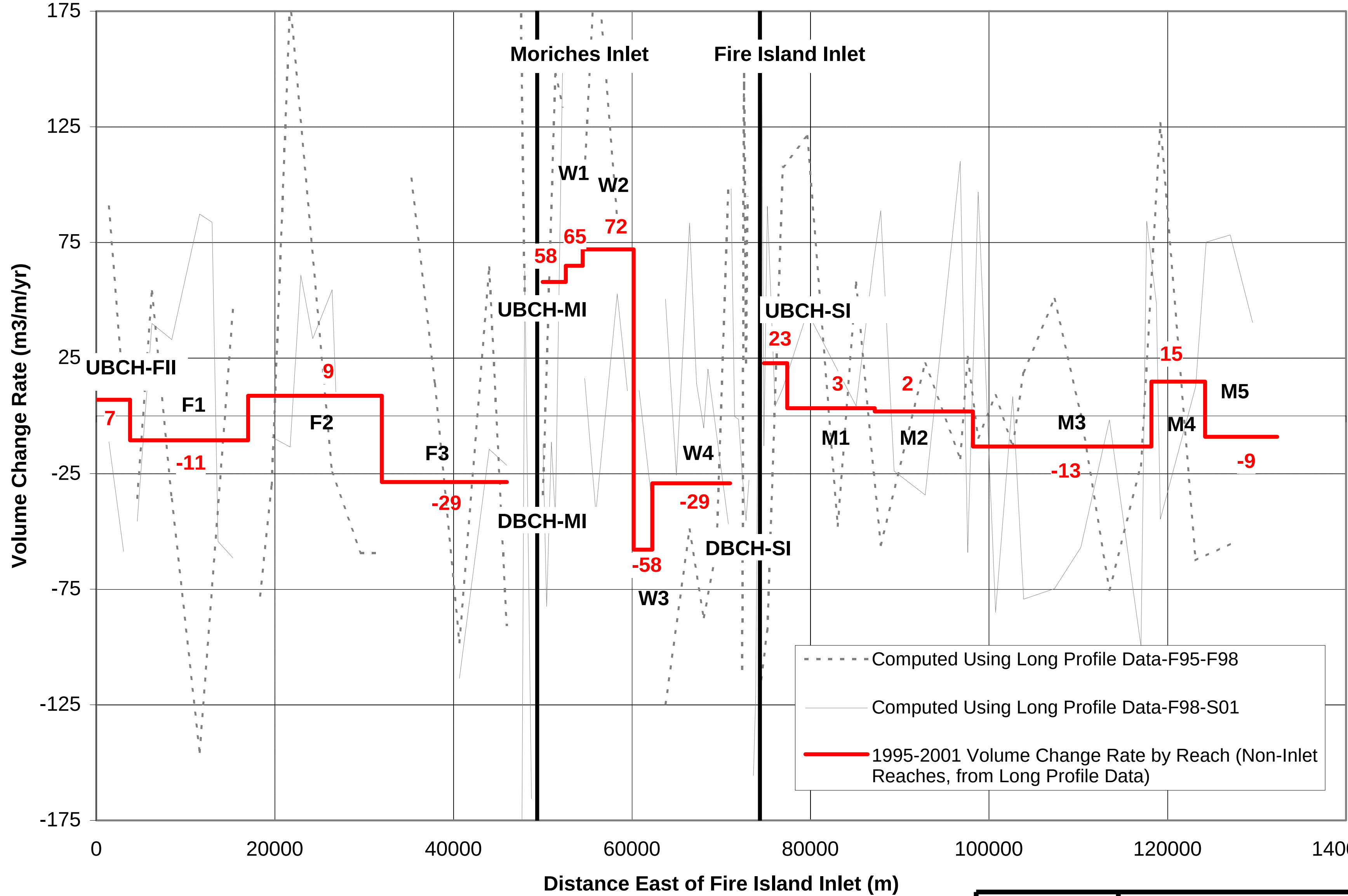
Previous Regional Sediment Budgets-CHL

FIGURE 6-2



Volume Change Rates, 1995-2001
Long Profile Change Data

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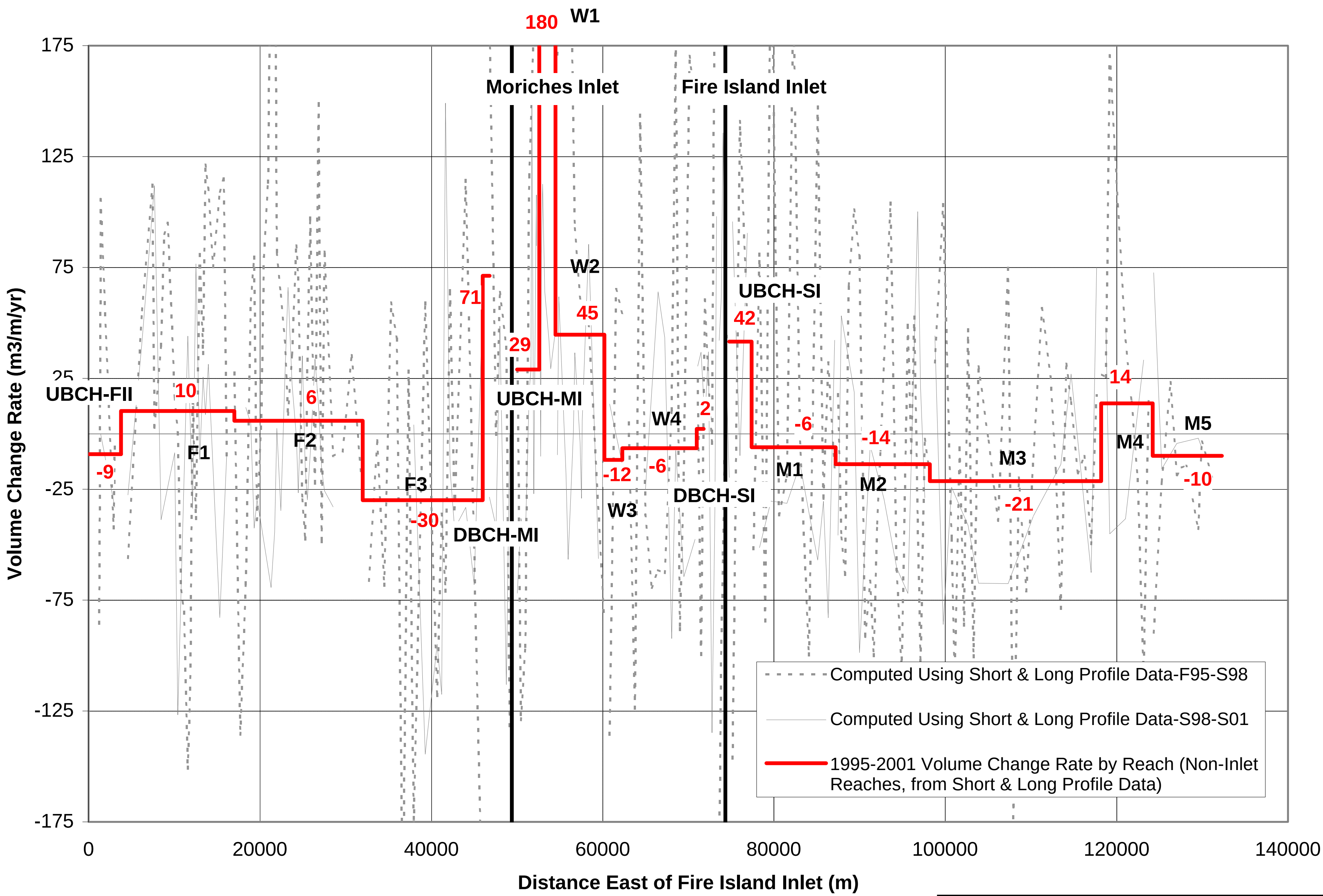
FIGURE 6-3



Volume Change Rates, 1995-2001
Short & Long Profile Change Data

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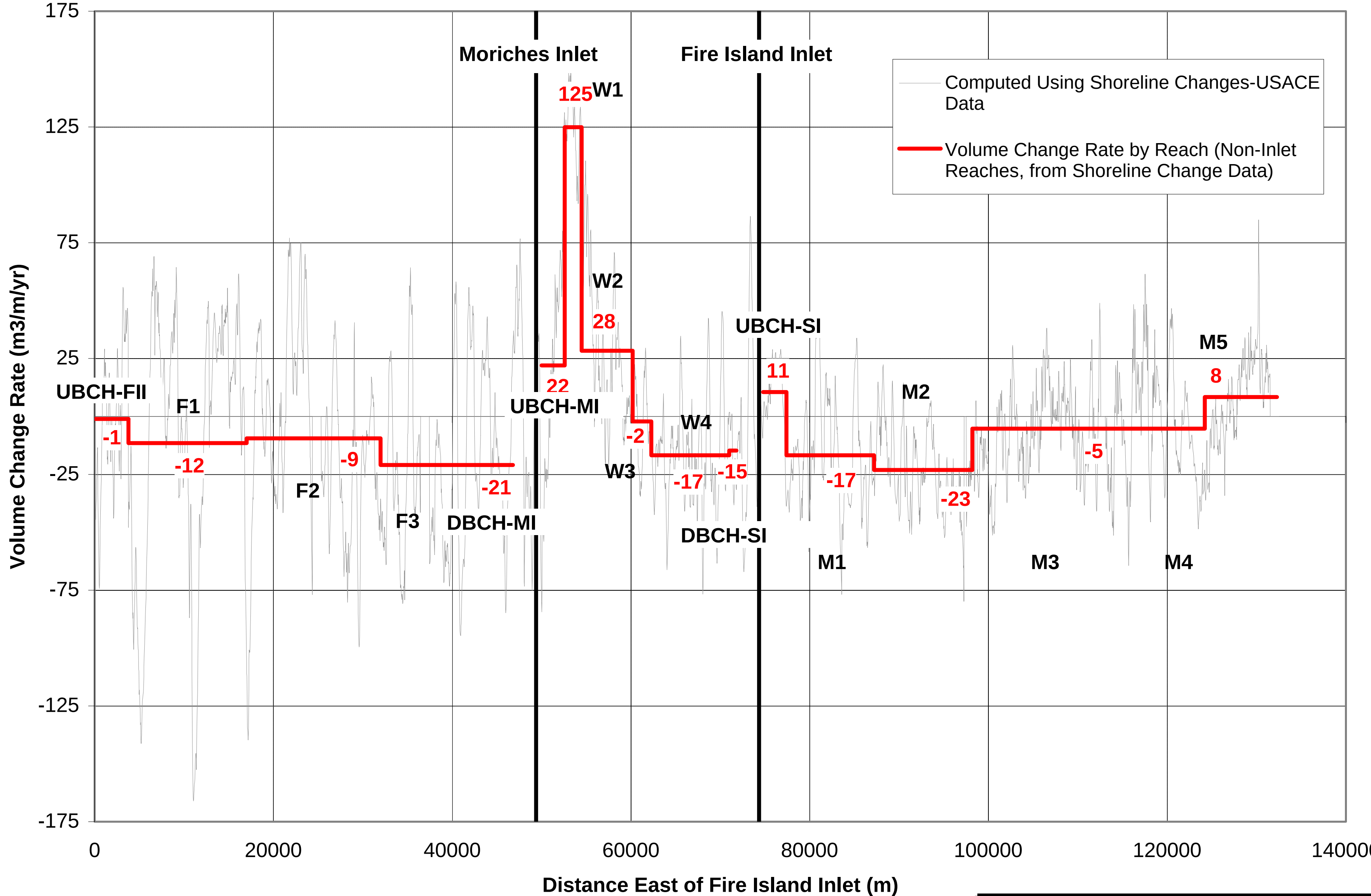
FIGURE 6-4



Volume Change Rates, 1995-2001
USACE Shoreline Change Data

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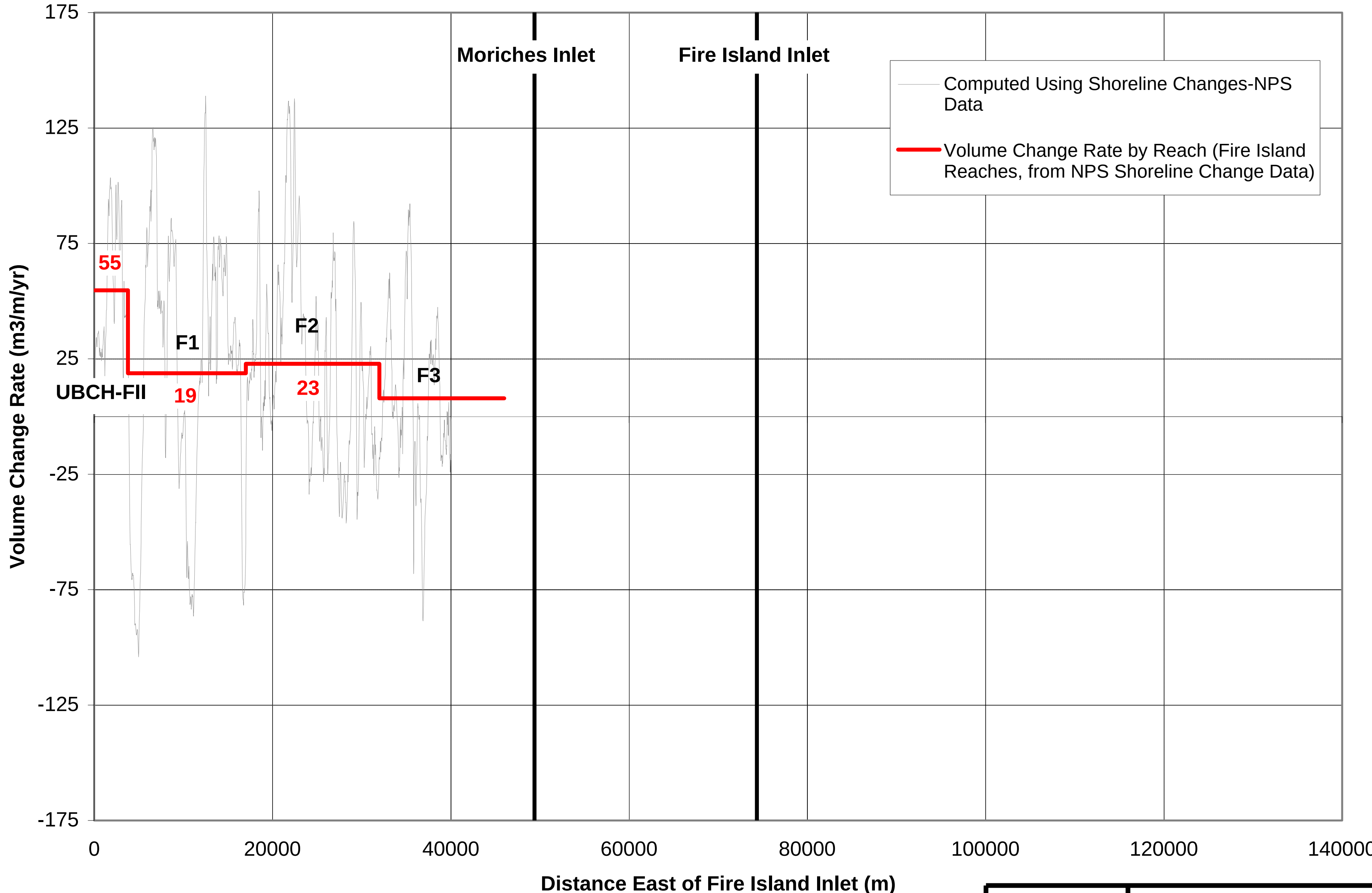
FIGURE 6-5



Volume Change Rates, 1995-2001
NPS Shoreline Change Data

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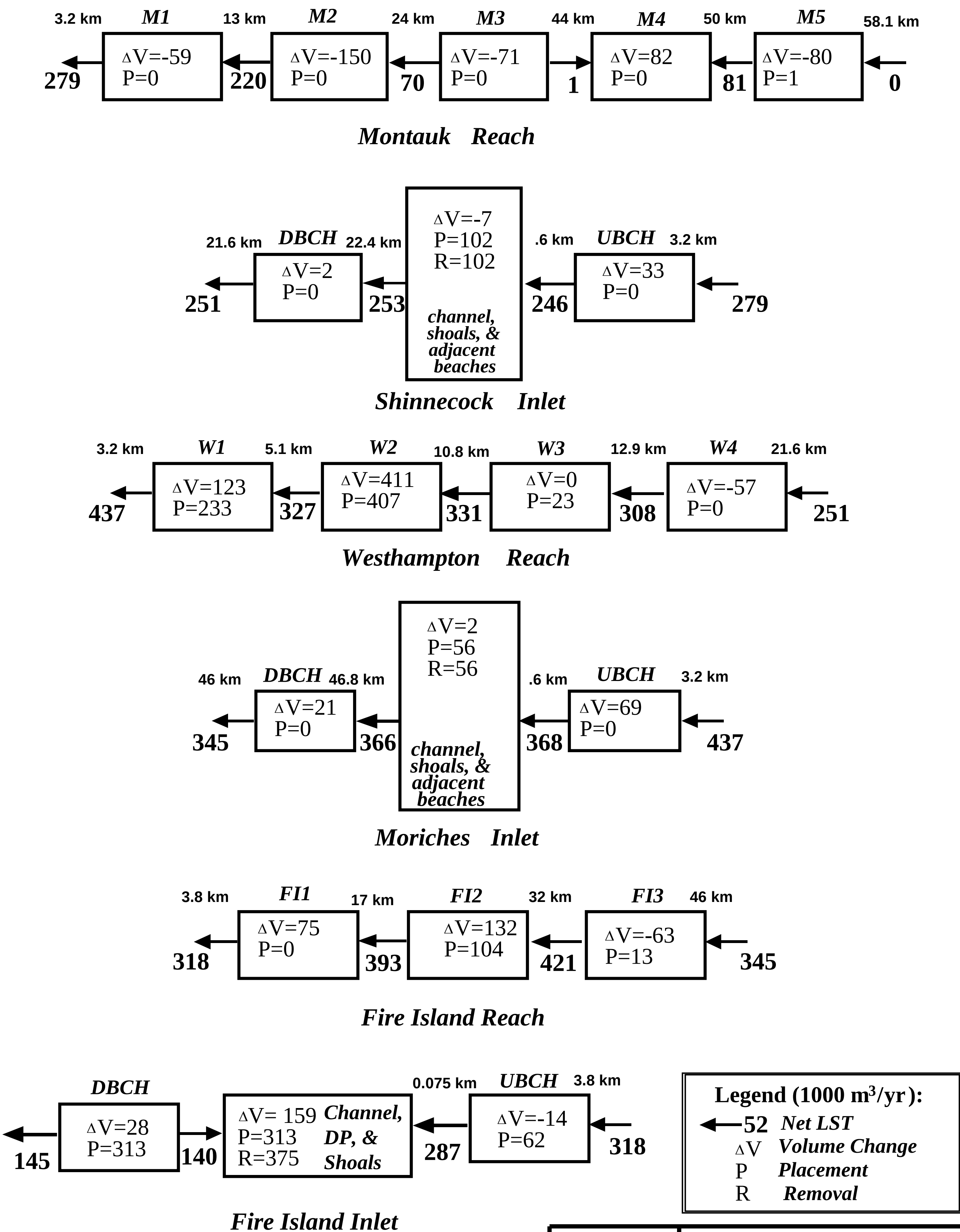
FIGURE 6-6

3982-28/Reports/ReportDec2002/Ch6Figs.ppt



Recent (1995-2001) Regional Sediment Budget

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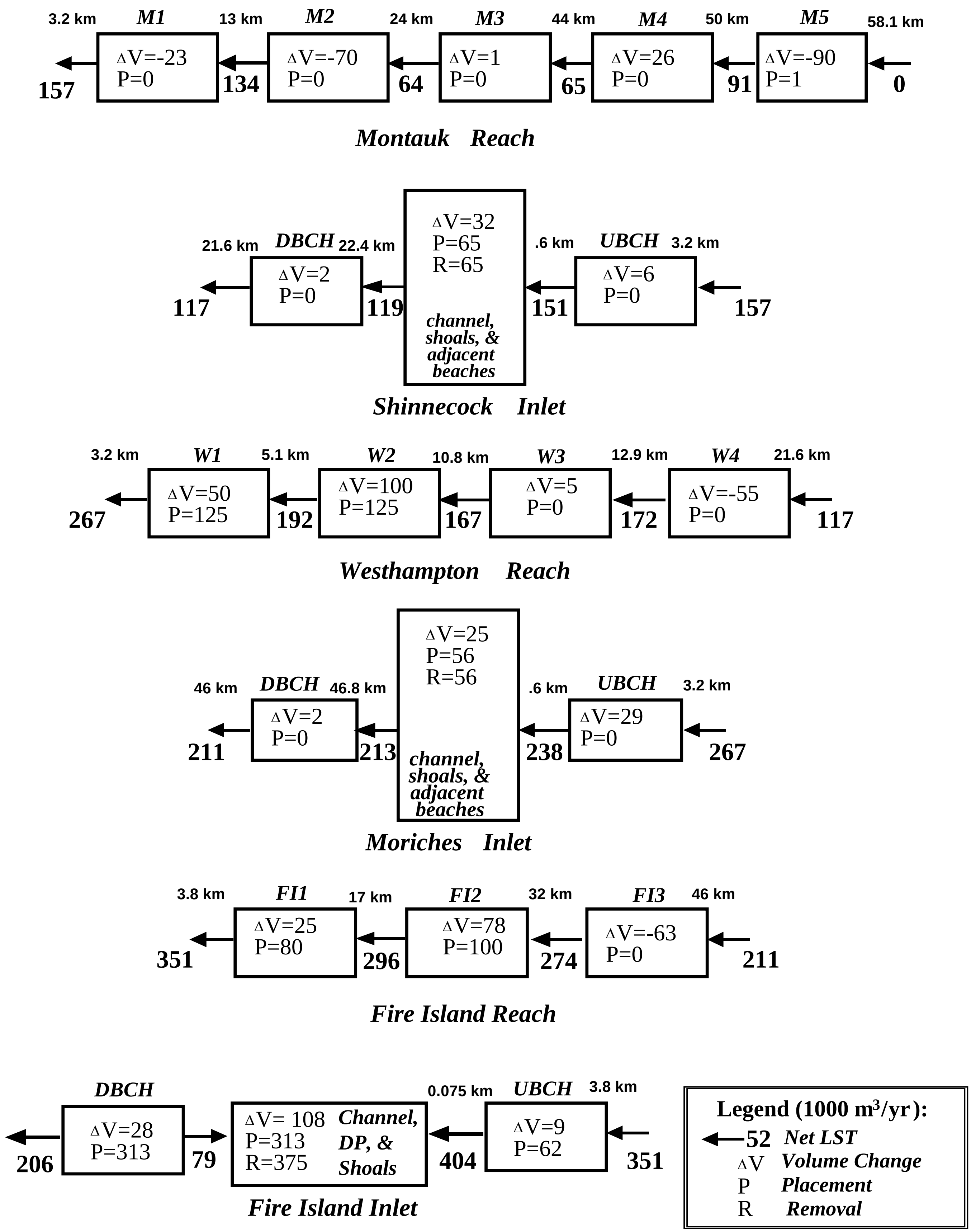
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 6-7

P:/3982-28 FIMP/Reports/ReportDec2002/Reg_Budget_Figs.ppt



Existing (c. 2001) Regional Sediment Budget

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REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 6-8

P:/3982-28 FIMP/Reports/ReportDec2002/Reg_Budget_Figs.ppt

7. NAVIGATION

This section presents an analysis of the navigation conditions of the inlets and how navigation may be affected by implementation of inlet modifications. A vessel transit analysis is conducted for selected design vessels at each inlet. Existing entrance channels and navigation conditions are discussed

7.1 Methodology

Navigation through Shinnecock Inlet, Moriches Inlet and Fire Island Inlet was evaluated for safe passage. The navigation requirements for each inlet are evaluated as follows:

7.1.1 Design Vessel

A representative vessel was chosen to control the required dimensions of the navigation channels through the inlets. Typically, this is the largest draft vessel anticipated to transit the channel on a regular basis.

7.1.2 Ship Motion Model

RAOs (Response Amplitude Operators) were developed for the design vessel. RAOs convert incident waves into vessel motions as a function of wave period. A frequency domain program, SCORES II, was used to derive the RAOs for the design vessel.

SCORES II was originally developed for the offshore industry to calculate stresses in ship hulls. The primary function of the model is to predict the wave-induced motions of a vessel in both the lateral and vertical planes. SCORES uses a Lewis-form, section-line representation of the hull form to calculate the added mass and damping coefficients of the subject vessel. The program computes the RAOs, both amplitude and phase, for all six degrees of freedom of a vessel with constant speed and heading with respect to the incident waves. The version of SCORES implemented for this study includes calculation of finite water depth effects and second order drift forces, developed for the Naval Civil Engineering Laboratory. The program has been verified by comparison with both laboratory model tests and theoretical computations (Hydromechanics, 1981).

The simulation protocol for this study subjects the design vessel to spectral sea states in 30-degree incident angle increments from bow to stern. The wave spectra have unit amplitude and periods ranging from 4 – 16 seconds. Vertical motion RAOs were predicted for 3 points along the keel of the vessel: at the bow, at the stern, and at the deepest draft point.

Probabilistic vessel response to incident sea states was evaluated spectrally. Scores uses a wave frequency spectrum to predict statistical vessel response. For this study, a 2-parameter spectrum has been defined. The spectrum is defined by:

$$S_1(\omega) = A \cdot B\omega^{-5} e^{-B\omega^{-4}}$$

where:

$$A = 0.25 H_{1/3}^2$$

$$B = (0.817 \cdot \frac{2\pi}{T})^4$$

Inputs to the spectrum are significant wave height ($H_{1/3}$) and mean period (T). SCORES output includes vessel response amplitude statistics. For evaluation of channel navigation, the average of the upper 10% of vessel response amplitude ($a_{1/10}$) was extracted from the model.

7.1.3 Channel Dimension Calculation

Based on the 1/10 response amplitude, draft of the vessel, and standard underkeel clearance, the recommended channel depth was calculated for incrementing wave height based on Coastal Engineering Manual (CEM) methods (USACE, 2002). Required channel widths were determined based on Permanent International Association of Navigation (PIANC, 1997) and CEM guidelines.

7.1.4 Existing Conditions

The existing authorized navigation channels were reviewed for depth, width, and navigability. The wave models developed in Section 4 were utilized to transform 10 years of wave data at NDBC Buoy 44025 to three points along the inlet entrance channel. Wave climate statistics were calculated at each point. Combining the vessel response functions and the modeled wave climates in the inlets, probability exceedance curves were developed for vertical vessel motion. The exceedance curves were used to evaluate the percent of time that the existing channels are not navigable for the design vessels. The following sections present the results of the navigation analysis for each inlet.

7.2 Shinnecock Inlet

The Shinnecock maritime center is home port for 30-35 commercial, deep-water fishing vessels and it is estimated that an additional 10-20 transient vessels off-load at the port at from time to time. Smaller clam boats and gill-net boats also dock at the maritime center. The center is also home to charter fishing boats and recreational vessels. Together, the boats comprise the second largest commercial fishing fleet in the South Shore Estuary, after Freeport/Point Lookout.

7.2.1 Design Parameters

Commercial, deep-water fishing boats are the largest vessels using Shinnecock inlet. Vessels range in size from 60-90 feet with estimated drafts of 10 feet. CENAN has identified three vessels as representative of the Shinnecock Fleet:

Table 7-1: Shinnecock Fishing Fleet Representative Vessels*

Vessel	Length (ft)	Beam (ft)	Hull Depth (ft)	Est. Draft (ft)	Gross Tonnage
Patriot	74.5	23.0	12.5	10.5	162
Hope	77.5	23.2	11.8	10.0	168
Second Generation	64.8	22.0	10.3	9.0	119

* Data from US Coast Guard vessel documentation database (USCG, 2002).

A typical east coast fishing vessel hull has been assumed for the SCORES vessel response study. The selected vessel has length overall of 79 feet, a beam of 22 feet, and a draft on the keel of 10 feet. The boat model displaces 170 tons at full draft.

The RAO from the SCORES analysis of the vessel are shown in Figure 7-1. The figure shows the amplification of vertical motion of the keel relative to the amplitude and period of the incident waves for vessel speeds of 0, 4, 8, and 12 knots, and wave directions from bow-on to stern-on. The maximum RAO is 1.3 at the keel for waves with periods greater than 5 seconds incident on the bow or stern. In other words, the vertical motion of the vessel about the still water level may be as much as 1.3 times the incident wave amplitude. The vessel response at the bow stern of the vessel was greater than at the keel, however because the hull depth less at the bow and stern, the overall depth of water required by the boat is controlled by the vertical motion of the keel.

however because the hull depth less at the bow and stern, the overall depth of water required by the boat is controlled by the vertical motion of the keel.

Roll response of the vessel becomes large for waves incident to the vessel at a greater than 45 degree angle. For incident wave angles less than 45 degrees the maximum response factor is 3.2 degrees per foot of wave amplitude.

Channel depth requirements for the design vessel were determined by combining vessel draft and motions under a given wave condition plus the effects of vessel squat and suggested safety clearance to the channel bottom. Squat for shallow draft vessels is taken as 1.0 feet for entrance channels (USACE, 2002). Required safety clearance for soft, sandy bottoms is 2 feet (USACE. 2002). Adding the design vessel draft plus squat allowance and draft, the minimum required depth in the channel, absent waves, is -13 feet MLW, 3 feet greater than the authorized channel depth of -10 feet MLW.

Figure 7-2 shows roses of wave climate at three points along the channel through Shinnecock Inlet derived from wave modeling described in Section 4. Combining the wave climate at each of these points with the spectral vessel response functions, a probability of keel motions was developed. Figure 7-3 shows the probability of the vessel exceeding the 2-foot safety clearance for given channel depths at each point in the channel. Probabilities of exceedance are based on the 1/10 vessel amplitude response (see Section 7.1). If the deposition basin is included in the channel depth calculation, the available depth post dredging is -20 feet MLW, and the channel may be in transited in all but the very highest wave conditions if the vessel is kept at a moderate forward speed (less than 8 knots).

An interesting characteristic of the vessel response function is that vessel response increases markedly with increasing forward speed. This is most important for a vessel traveling from the outer channel to the inner channel. As vessel speed increases, the encounter frequency of the incoming waves decreases. In essence, the waves appear to be elongated from the vessel frame of reference. As a result, the vessel responds as if it were in larger seas than it truly is. The response is less severe for vessels traveling less than 8-knots true speed, as the wave celerity is greater than the vessel speed.

If the deposition basin is half full, i.e. a channel depth of approximately -15 feet MLW, downtime increases to about 70% of the time in the outer and middle channel (not accounting for additional depth due to tides). This result indicates that vertical motion of the vessel may exceed two feet under most wave conditions. Transiting the channel at a tide stage above MLW would reduce downtime to some degree.

Required channel width is typically determined as a multiplier of boat breadth to reflect navigation conditions. Two methods are presented herein, the first using PIANC guidelines (PIANC, 1995), the second using methods discussed in CEM part V (USACE, 2002). Table 7-2 presents the PIANC guidelines.

Table 7-2: Required Shinnecock Entrance Channel Width (PIANC, 1997)*	
Basic Width, Good Maneuverability	1.3B
Cross Wind, Moderate	0.4B
Cross Current, Negligible	0.0B
Longitudinal Current, Strong	0.2B
Aids to Navigation, Good	0.1B
Bottom Surface, Soft	0.1B
Depth Waterway, shallow	0.2B
Two-way Traffic	
Vessel Speed, Moderate	1.6B
Moderate Traffic	0.2B
Bank Clearance	0.5B x 2
TOTAL	**5.1B**

* Width specified as multiple of vessel beam

On the other hand, from the CEM, for a two-way channel with strong currents in a trench channel, the required width is 6.5B (Table V-5-10, USACE 2002).

Using a beam of 24 feet for the design vessel, the two methods yield a channel width of 117 feet and 156 feet, respectively.

7.2.2 Existing Condition

The navigation conditions at Shinnecock Inlet are good according to USCG personnel (personal communication, 2002). The east bar of the ebb shoal encroaches on the channel, but the channel has remained clear in recent years due to dredging efforts. Navigation lights are operated at the end of each jetty, but no buoys are maintained outside the inlet due to the wide, straight channel.

The Federal authorized channel depth of the Shinnecock entrance channel is currently -10 feet MLW and the marked channel is 200 feet wide (See Figure 2-1). Available depths and widths are typically much greater in the channel due to the presence of the deposition basin (800 feet wide). For example, in the 2000 SHOALS survey, minimum available depth in the entrance channel was -20 feet MLW. However, the 2002 condition survey shows minimum depth in the middle of the channel at -15.3 feet MLW, although the eastern quarter of the channel had shoaled to -12.3 feet MLW. In the last five years, available depth of the channel has varied, but it appears the channel typically shoals to approximately -15 feet MLW prior to dredging. Two shoals establish the minimum available depth within the channel limits: the dynamic ebb shoal outside the inlet that requires periodic maintenance dredging and the permanent shoal along the west jetty in the throat of the inlet. Maximum measured currents in the inlet have exceeded 4.5 knots on the flood tide; typical currents are on the order of 3 knots.

The authorized channel width meets the required channel widths for the wave, current, and traffic conditions in the inlet.

As established in Section 7.2.1, the required depth of the Shinnecock entrance channel for safe navigation of the design vessel, absent waves, is 13 feet - 3 feet deeper than the authorized channel depth. However, 13 feet is typically available in the channel due to the presence of the deposition basin. To provide for safe navigation 90% of the time in actual wave conditions, a channel depth of 18.5 feet is required. If the deposition basin shoals to -15 feet MLW across the full width, the channel will meet safe navigation standards only 25-30% of the time. Transiting at slower speeds and at elevated tide levels would help to mitigate downtime as the channel shoals.

Without any modification to the inlet, and maintaining current dredging intervals, the future navigation conditions at Shinnecock will remain similar to existing conditions.

7.3 Moriches Inlet

While officially closed for navigation, Moriches Inlet serves as a conduit for recreational boating between Moriches Bay and the Atlantic.

7.3.1 Design Parameters

There is limited information available on the design vessel for Moriches Inlet. CENAN reports one party boat, *Rosie*, utilizing the inlet. The Coast Guard vessel information database indicates the *Rosie* has a length of 61.6 feet and breadth of 17.5 feet, but only drafts approximately 3 feet. However, Coast Guard personnel report that *Rosie* operates primarily within the confines of Moriches Bay and rarely uses the inlet (USCG personal communication, 2002).

The vessels most likely to use Moriches Inlet for navigation are personal recreational watercraft. The NY Department of State reports that over 3000 recreational boats greater than 16 feet utilize the Bay (Steadman, 1999). The 1983 Moriches GDM lists the maximum vessel draft using the inlet as 4 feet with a maximum breadth of 12 feet. The same dimensions have been assumed for this study to represent the typical recreational watercraft (the beam of *Rosie* is not considered to be representative of the typical inlet traffic).

It is difficult to model the motions of small recreational craft accurately. There is large variability in the construction of such vessels and hull forms often exceed the design limits of programs like SCORES, which were designed for the modeling of larger displacement vessels. For this study, a typical allowance is applied to account for vessel vertical motion response to waves: one-half the incident wave height (USACE, 2002). Vessel motion probabilities to determine channel depth are based on the 1/10 vessel amplitude response, which in the case of small recreational craft is taken to be equivalent to one-half the average of the upper 10% of waves in a given spectrum ($H_{1/10}$).

Similarly to the Shinnecock analysis, channel depth requirements for the design vessel are determined by combining vessel draft and motions under a given wave condition plus the effects of vessel squat and suggested safety clearance to the channel bottom. Squat for shallow draft vessels is taken as 1.0 feet for entrance channels (USACE, 2002). Required safety clearance for soft, sandy bottoms is 2 feet (USACE, 2002). Summing the two, the minimum required depth in the channel, absent waves, is -7 feet MLW.

Figure 7-4 shows roses of wave climate at three points along the channel through Shinnecock inlet derived from wave modeling described in Section 4. Combining the wave climate at each of these points with the vessel response function ($0.5H_{1/10}$), a probability of keel motions was developed. Figure 7-5 shows the probability of the vessel exceeding the 2-foot safety clearance for given channel depths at each point in the channel.

Required channel widths as a function of vessel breadth are identical to those given in Section 7.2.1: 5.1B – 6.5B. Using a beam of 12 feet for the design vessel, the two methods yield a channel width of 61 feet and 78 feet, respectively.

7.3.2 Existing Condition

When the entrance channel and deposition basin were dredged in 1998 (to -14 feet MLW), the condition survey in April 2002, showed a controlling depth of only -6.1 feet MLW. Due to the rapid shoaling of the channel, navigation charts list Moriches inlet as officially closed to navigation (NOS Chart 12352). Nonetheless, recreational boats continue to use the inlet, approximately 200 boats on a summer weekend, although transiting the ebb shoal can be difficult. According to USCG personnel, the Coast Guard operates a rescue boat out of Group Moriches that transits the inlet on a regular basis (USCG personal communication, 2002). The 27-foot boat drafts 3 feet and navigation through the inlet is challenging even at this size. The 49-foot long, 5-foot draft USCG buoy tender operating out of Group Moriches must be docked at Fire Island Station because it cannot navigate Moriches Inlet. Interior channels are in worse condition and are typically much less than the authorized 6-foot depth. The Coast Guard has received numerous complaints from the public on the condition of the inlet.

The Coast Guard provides no aids to navigation as the inlet is officially closed. Boats navigate through the ebb shoal by looking for gaps in the breaking waves. According to USCG personnel, the gap in the ebb shoal as of November 2002 is approximately 250 feet wide and 6-7 feet deep. Depths over the shoal are as shallow 2-3 feet. Boats navigating the shoal typically take a magnetic heading of 220 degrees from the end of the inlet jetties.

The Federal authorized channel depth of the Moriches entrance channel is currently -10 feet MLW and the marked channel is 200 feet wide (See Figure 2-3). Available depths are currently less than the authorized depth. The channel condition survey of 6-7 April 2002 reported a minimum depth of 6.1 feet in mid-channel, similar to USCG reported depths in 2002.

The minimum required depth of the Moriches entrance channel for safe navigation of the design vessel, absent waves, is 7.0 feet. The channel through the ebb shoal is often at or less than this level. In the presence of waves, a minimum channel depth of 11 -feet MLW is required to maintain safe navigation 90% of the time at MLW (Figure 7-5). If the deposition basin is included in the channel depth calculation, the available depth post dredging is -14 feet MLW, and the channel may be in transited in all but the very highest wave conditions.

The authorized channel width meets or exceeds the required channel widths for the wave, current, and traffic conditions in the inlet (see section 7.3.1).

Without change in current practices, navigation at the inlet is likely to remain poor. However, if the channel were maintained at the authorized depth (-10 feet MLW), navigation conditions would generally be good and downtime would be less than 30%. This would require, at a minimum, a reduction in the interval between maintenance dredging.

7.4 Fire Island Inlet

The Captree Boat Basin on the east end of Fire Island is home port for 32 commercial vessels, including 14 head boats, 14 charter boats, four dive boats, and two sail boats.

7.4.1 Design Parameters

The fishing vessels have the largest draft of any using the inlet. These boats are generally smaller than the Shinnecock fleet but larger than the Moriches fleet with vessel lengths of 50-60 feet and drafts of about 6 feet with wood or fiberglass hulls. CENAN has identified two vessels as representative of those using Fire Island Inlet:

Table 7-3: Fire Island Fishing Fleet Representative Vessels*					
Vessel	**Length (ft)**	**Beam (ft)**	**Hull Depth (ft)**	**Est. Draft (ft)**	**Gross Tonnage**
Sarah Beth	55.2	17.5	6.9	5.5	45
Happy Hooker	43	15	8.0	6.0	34

* Data from US Coast Guard vessel documentation database (USCG, 2002).

For the vessel response analysis, the fishing vessel hull used in the Shinnecock analysis was scaled to the match the length of the Fire Island Inlet vessel *Sarah Beth*, 55 feet. The modeled vessel has an overall length of 55 feet, a beam of 15.2 feet, and a draft on the keel of 6.5 feet. The vessel displaces 55 tons on full draft.

The results of the SCORES analysis of the vessel are shown in Figure 7-6. The figure shows the amplification of vertical motion of the keel relative to the amplitude and period of the incident waves for vessel speeds of 0, 5, and 9 knots. The maximum RAO is 1.5 at the keel for waves incident on the bow or stern, with periods greater than 10 seconds.

Roll response of the vessel becomes large for waves incident to the vessel at a greater than 30 degree angle. For incident wave angles less than 30 degrees, the maximum response function is 3.2 degrees per foot of wave amplitude.

Channel depth requirements for the design vessel are determined by combining vessel draft and motions under a given wave condition with the effects of vessel squat and the suggested safety clearance to the channel bottom. Squat for shallow draft vessels is taken a 1.0 feet for entrance channels (USACE, 2002). Required safety clearance for soft, sandy bottoms is 2 feet (USACE, 2002). Summing the design vessel draft and squat allowance, the minimum required depth in the channel, absent waves, is -9.5 feet MLW.

Figure 7-7 shows roses of wave climate at three points along the channel through Fire Island inlet derived from wave modeling described in Section 4. Combining the wave climate at each of these points with the spectral vessel response functions, a probability of keel motions are developed. Figure 7-8 shows the probability of the vessel exceeding the 2-foot safety clearance for given channel depths at each point in the channel. Probabilities of exceedance are based on the 1/10 vessel amplitude response (see section 7.1). If the deposition basin is included in the channel depth calculation, the available depth post dredging is -14 feet MLW, and the channel may be in transited in all but the very highest wave conditions if the vessel is kept at a moderate forward speed (less than 8 knots).

Channel width is determined as a multiplier of boat beam to reflect navigation conditions. Two methods are presented herein, the first using PIANC guidelines (PIANC, 1995), the second using methods discussed in CEM part V (EM 1110-2-1100, 2001). Table 7-3 presents the PIANC guidelines.

Using the CEM method, for a two-way channel with strong currents in a trench channel, the required width is 6.5B. Using a beam of 18 feet for the design vessel, the two methods yield a required channel width of 105 feet and 117 feet, respectively.

Table 7-4: Required Fire Island Inlet Entrance Channel Width (PIANC, 1995)*	
Basic Width, Good Maneuverability	1.3B
Cross Wind, Moderate	0.4B
Cross Current, Moderate	0.7B
Longitudinal Current, Strong	0.2B
Aids to Navigation, Good	0.1B
Bottom Surface, Soft	0.1B
Depth Waterway, shallow	0.2B
Two-way Traffic	
Vessel Speed, Moderate	1.6B
Moderate Traffic	0.2B
Bank Clearance	0.5B x 2
TOTAL	**5.8B**

*Width specified as multiple of vessel beam

7.4.2 Existing Condition

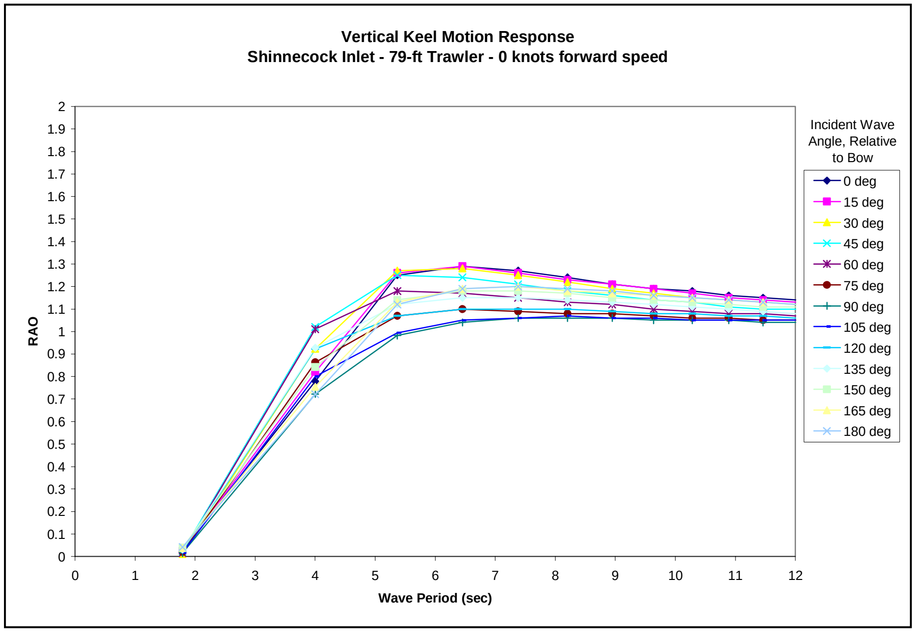
The morphology of Fire Island Inlet is highly dynamic and the channel is subject to rapid shoaling. As a result, the navigation channel alignment has been realigned several times in the last 30 years. The present channel alignment is shown in Figure 2-5. USCG personnel report that buoys placed in 17 feet of water at the edge of the dredged area will shoal to 10 feet in 6 months and 6 feet within two years. Waves of 5-6 feet will break on the buoys marking the west side of the channel. Buoys are typically left in place for one year then moved west due to shoaling. The light at the tip of the Federal jetty has been discontinued since it is now in the middle of the Democrat Point shoal. Navigation conditions at night are difficult. Typical currents in the inlet are on the order of 3 knots.

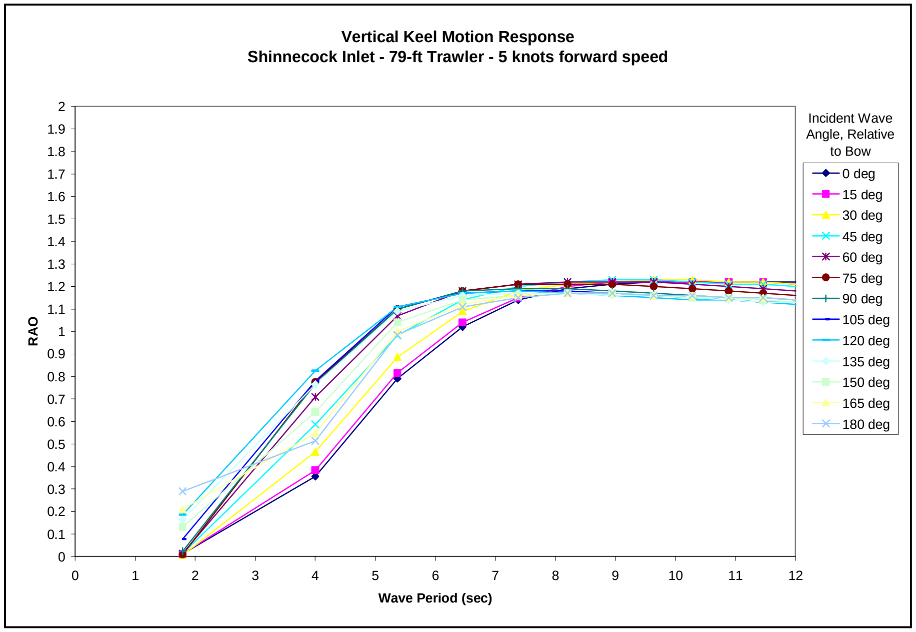
The existing channel has an authorized depth of -10 feet MLW and a width of 450 feet. Available depth near the channel entrance is often deeper due to the presence of the deposition basin, which is currently dredged to a design depth of -14 feet MLW. While the Federal channel does not overlap the deposition basin as with the other two inlets, the channel and the deposition basin are commonly both dredged to -14 feet MLW. Recent surveys indicate that available depth is reduced to -10 feet MLW or less around the tip of the existing sand spit prior to maintenance dredging (see Figure 5-28).

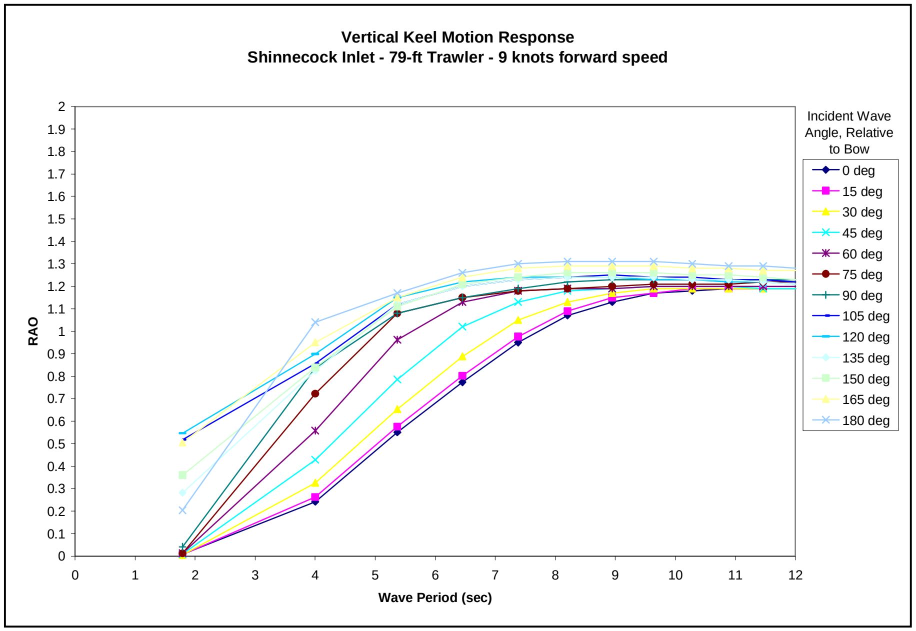
The required depth of the Fire Island Inlet entrance channel for safe navigation of the design vessel, absent waves, is 9.5 feet. A depth of at least -10 feet MLW is typically available. According to modeling results, vessel vertical movement will encroach into the 2-foot safety margin 90% of the time at this depth. In order to maintain an open, safe channel 90% of the time, the minimum recommended depth at MLW is 13.5 feet (see Figure 7-8). In short, the authorized channel depth does not meet the required depths at the inlet, but sufficient depth is realized with the deposition basin.

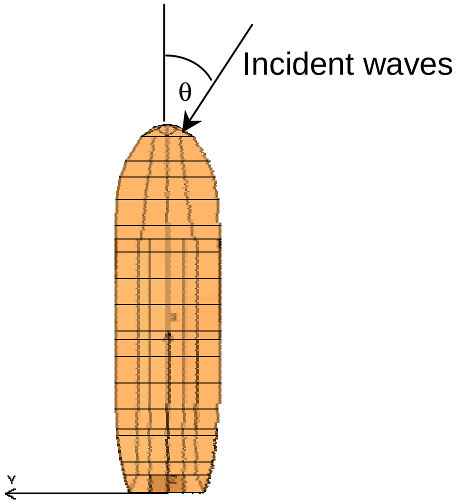
The authorized channel width far exceeds the required channel widths for the wave, current, and traffic conditions in the inlet.

Without any modification to the inlet configuration or dredging program, navigation conditions will remain similar to existing conditions.









Vertical Motion RAO – Shinnecock Design Vessel

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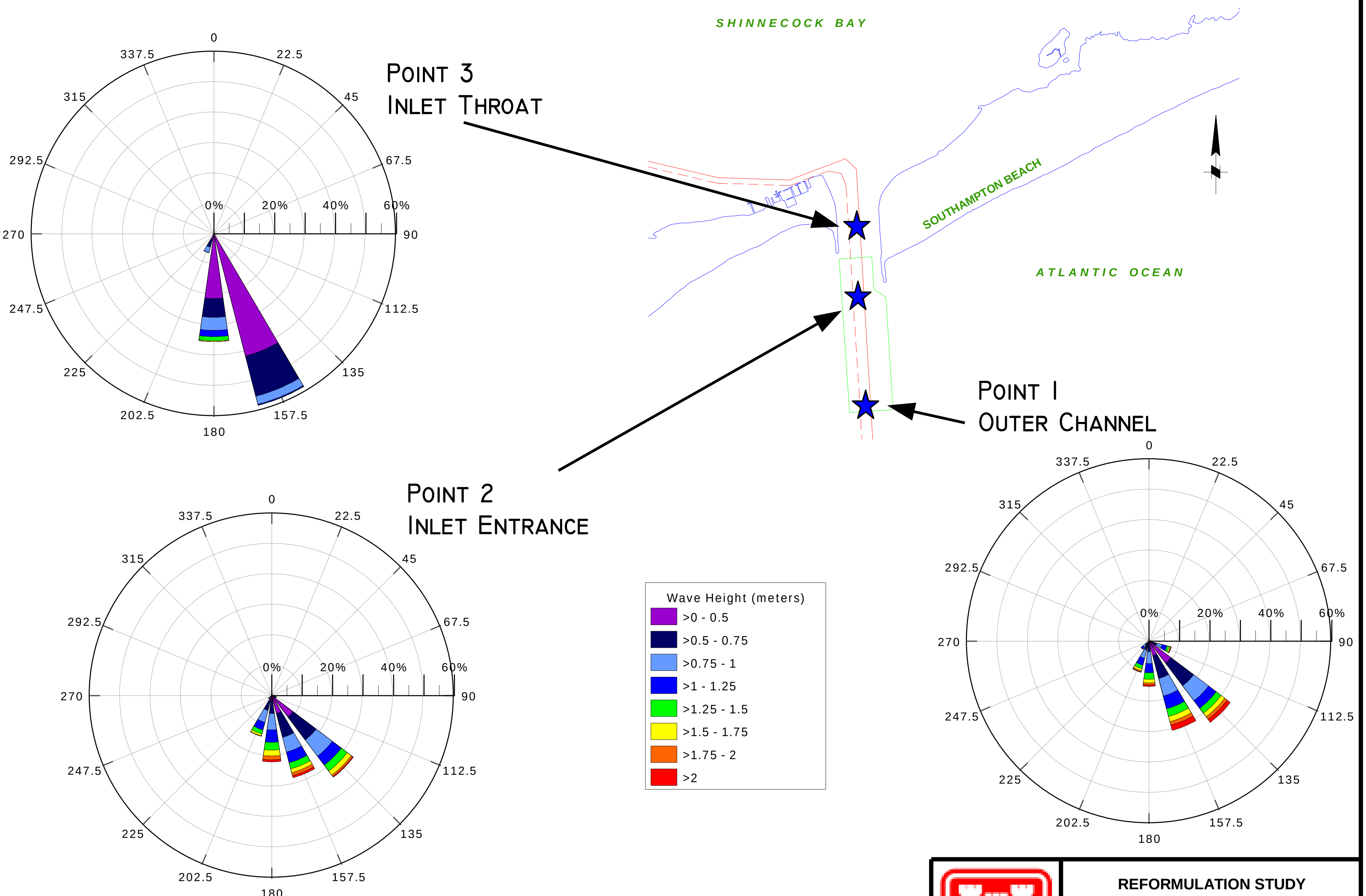
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 7.1

File pat/File name



Wave Climate – Shinnecock Inlet

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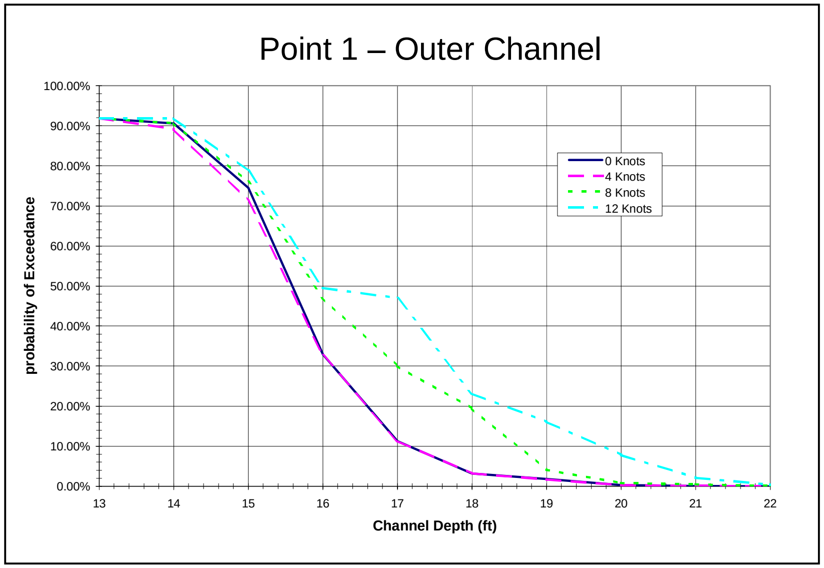
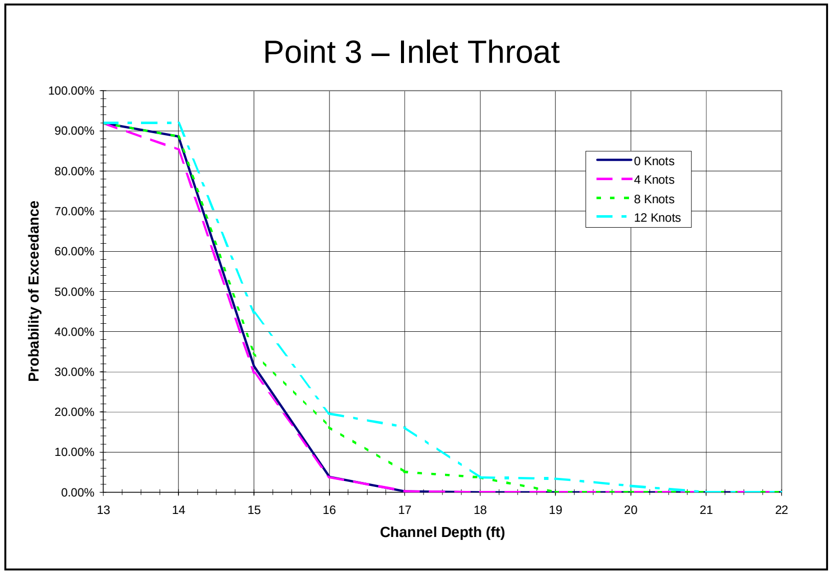
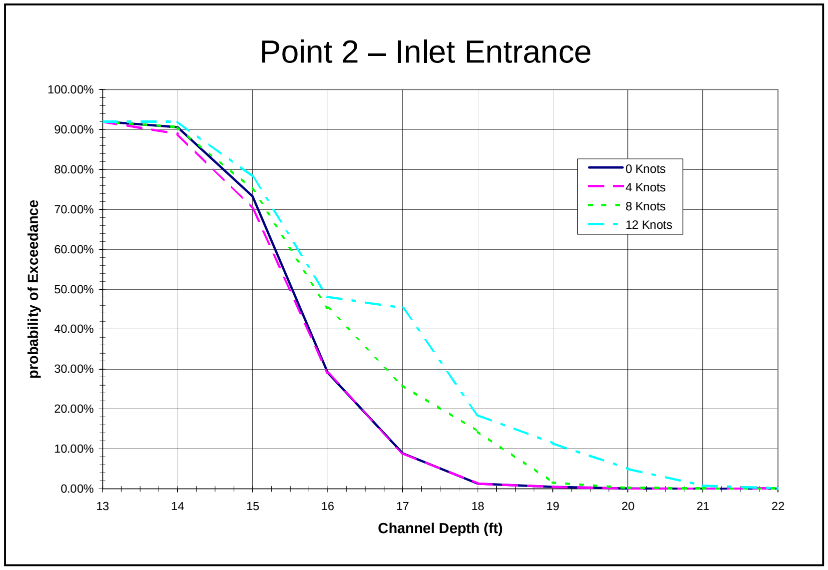
FIGURE 7.2

File pat/File name







Safe Navigation Depth Exceedance Probability

Shinnecock Design Vessel

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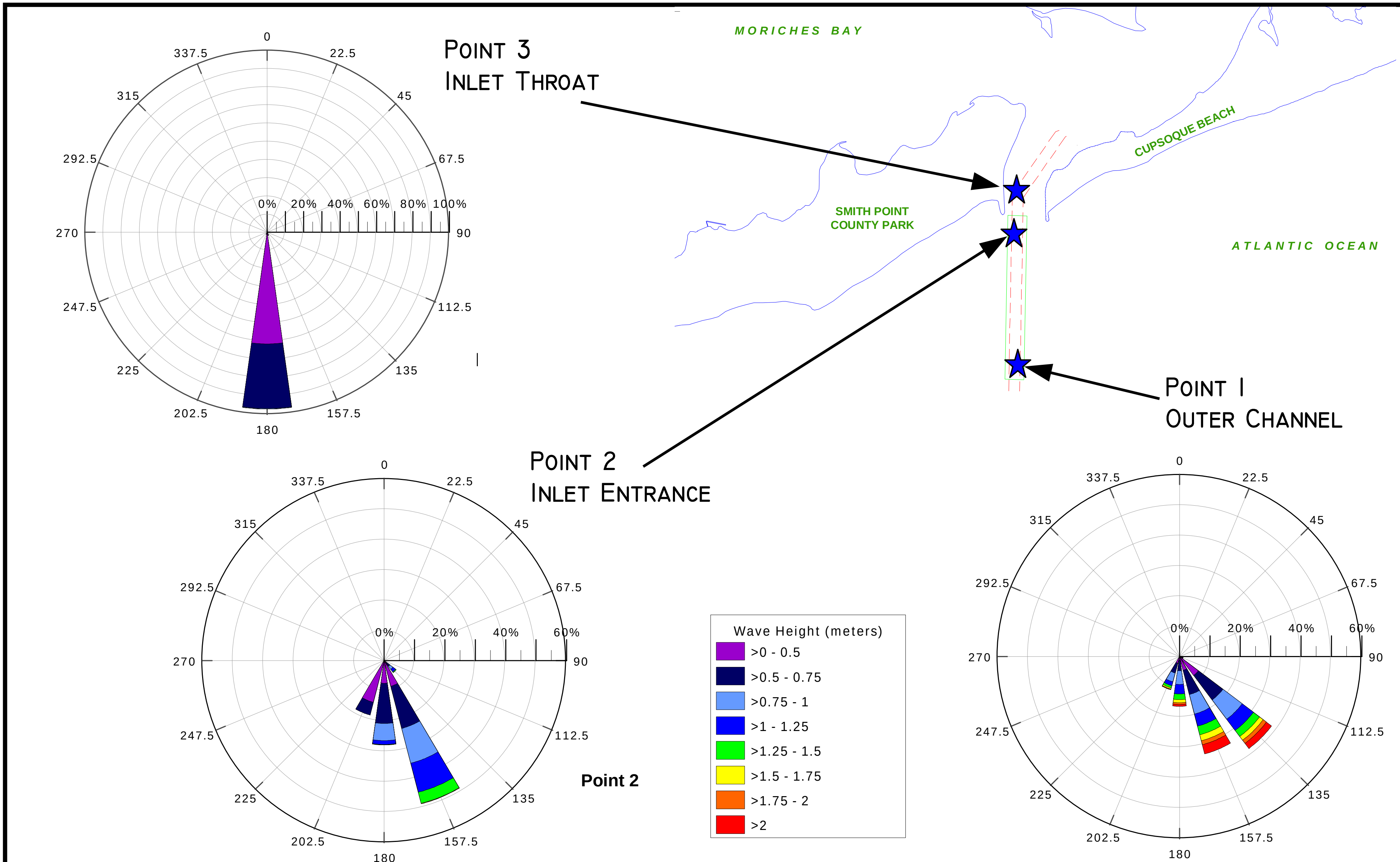
FIGURE 7.3

File pat/File name



Wave Climate – Moriches Inlet

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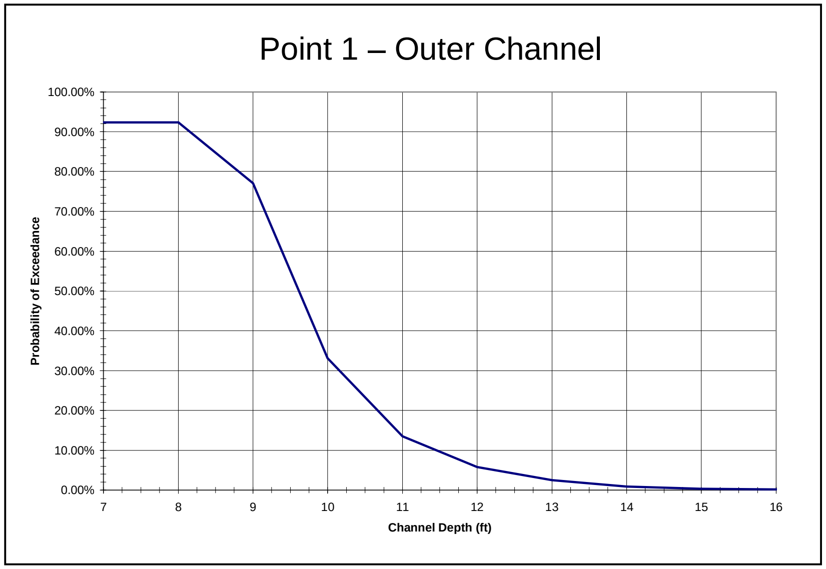
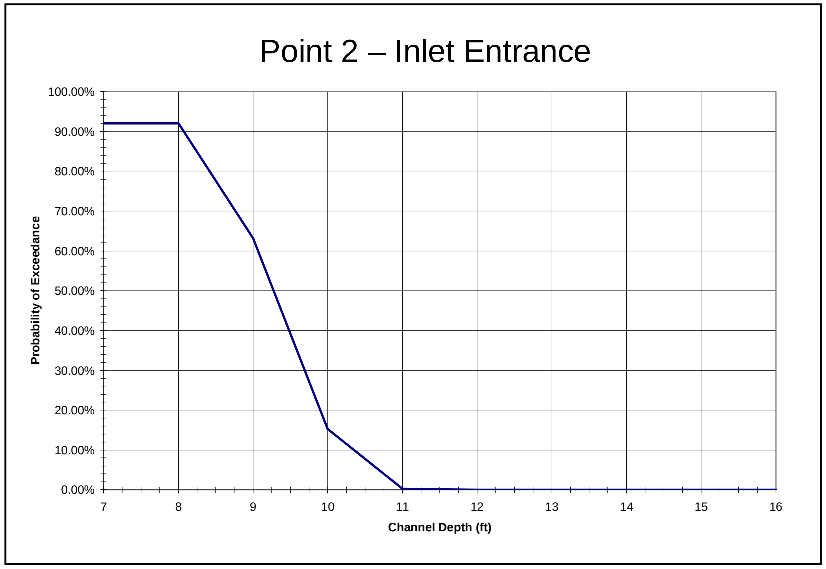
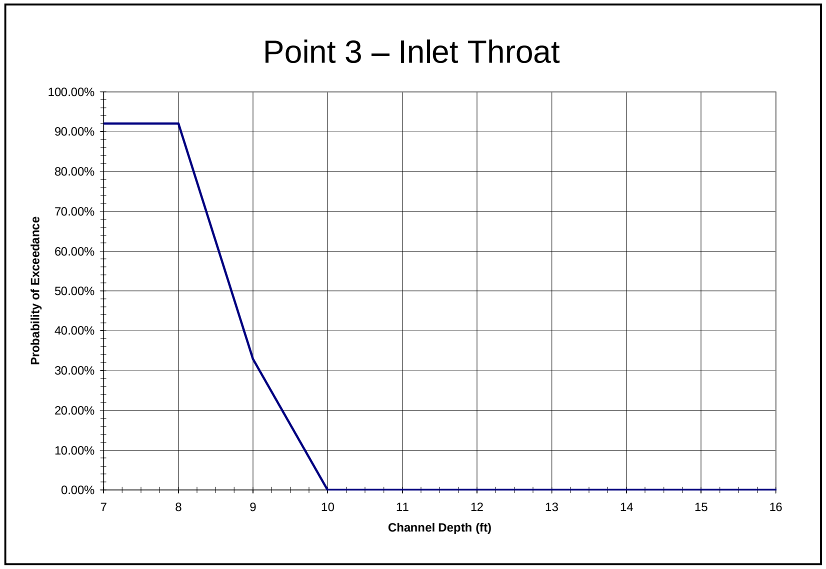
FIGURE 7.4

File pat/File name







Safe Navigation Depth Exceedance Probability

Moriches Design Vessel

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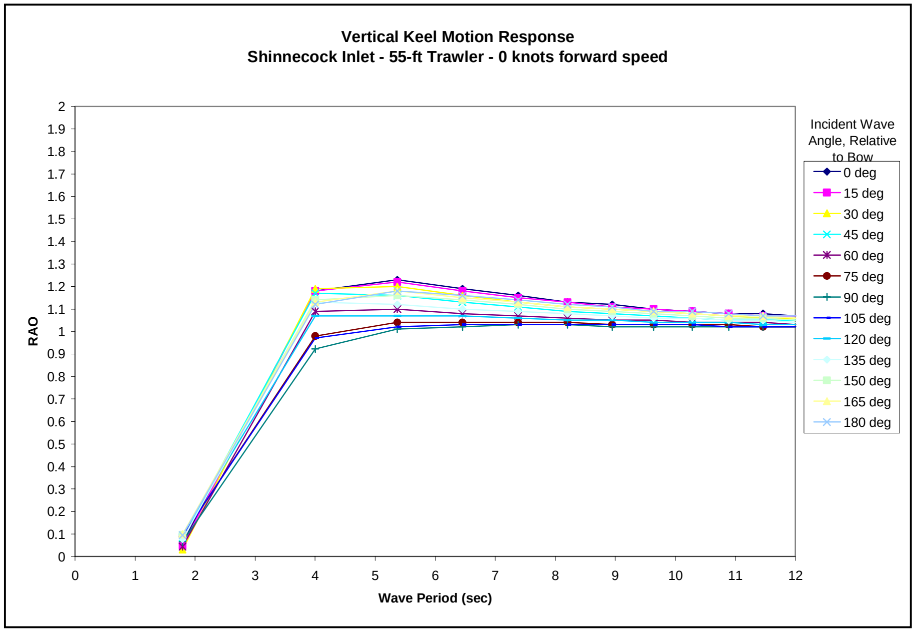
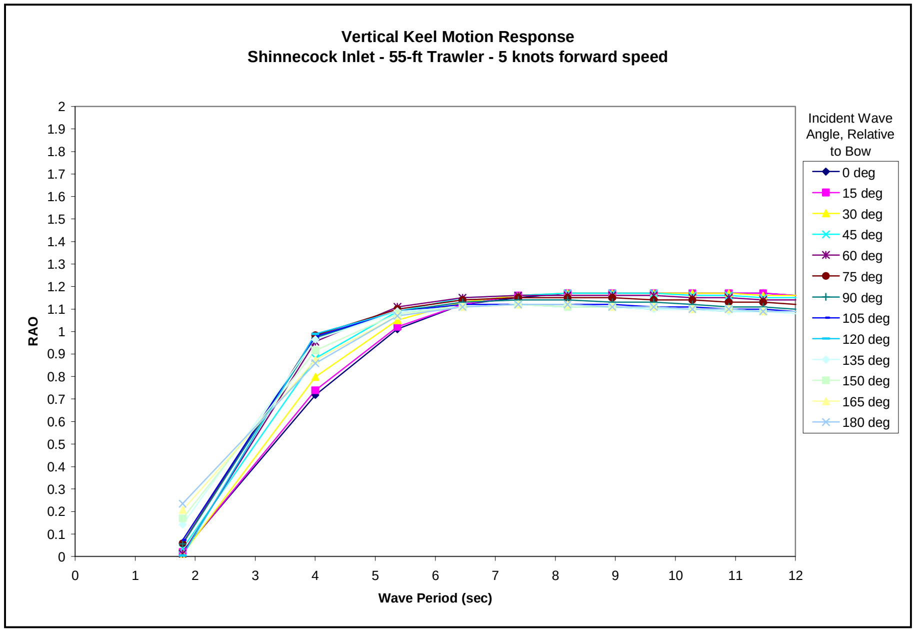
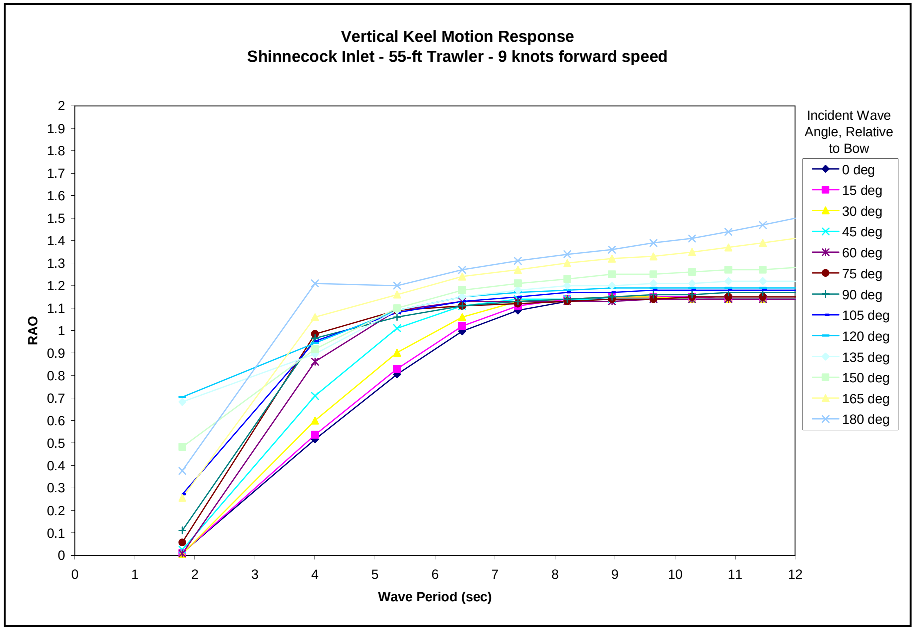
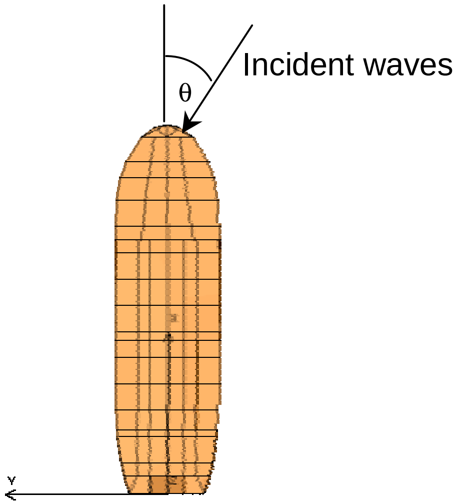
FIGURE 7.5









Vertical Motion RAO – Fire Island Design Vessel

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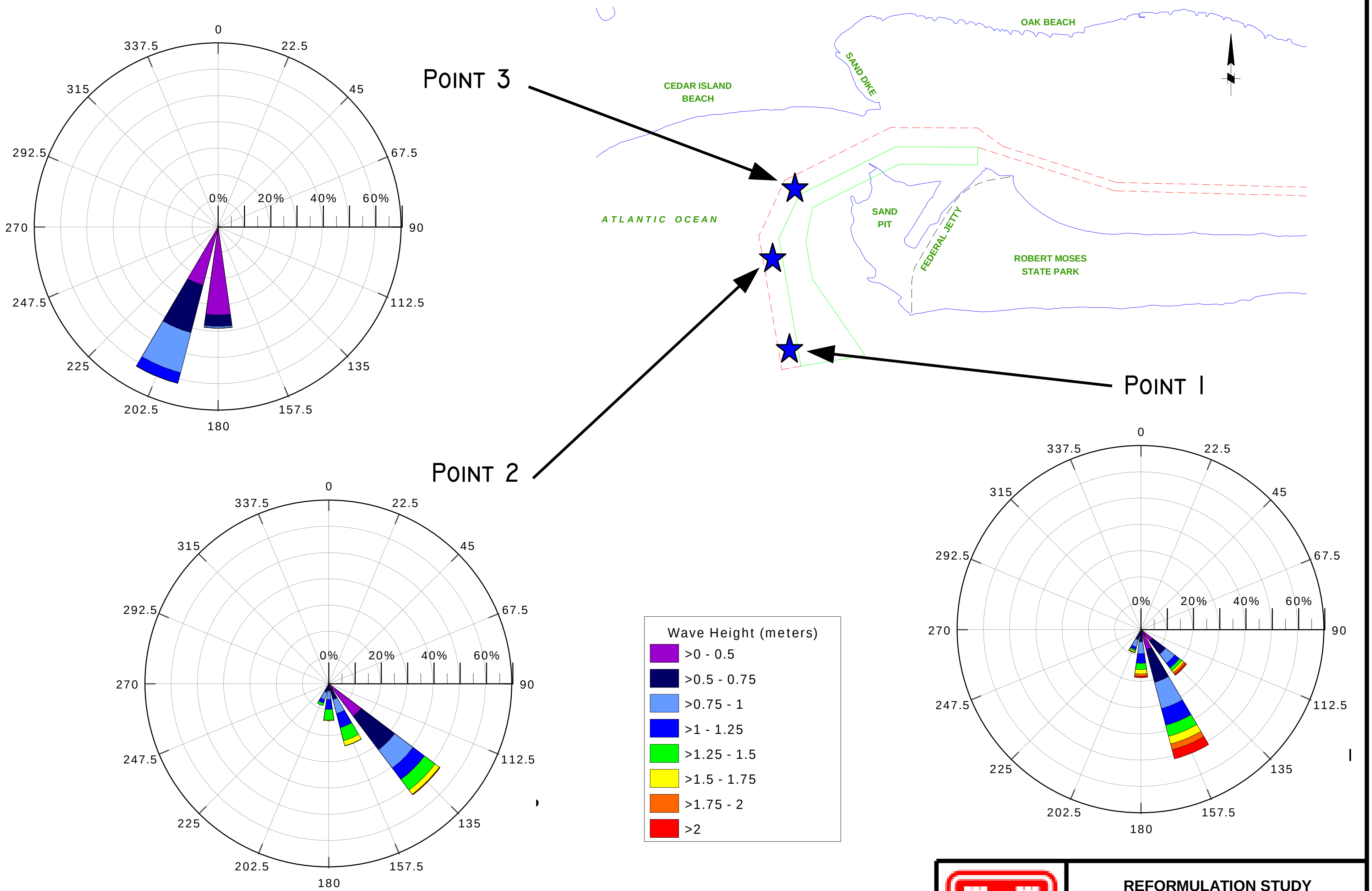
FIGURE 7.6

File pat/File name



Wave Climate – Fire Island Inlet

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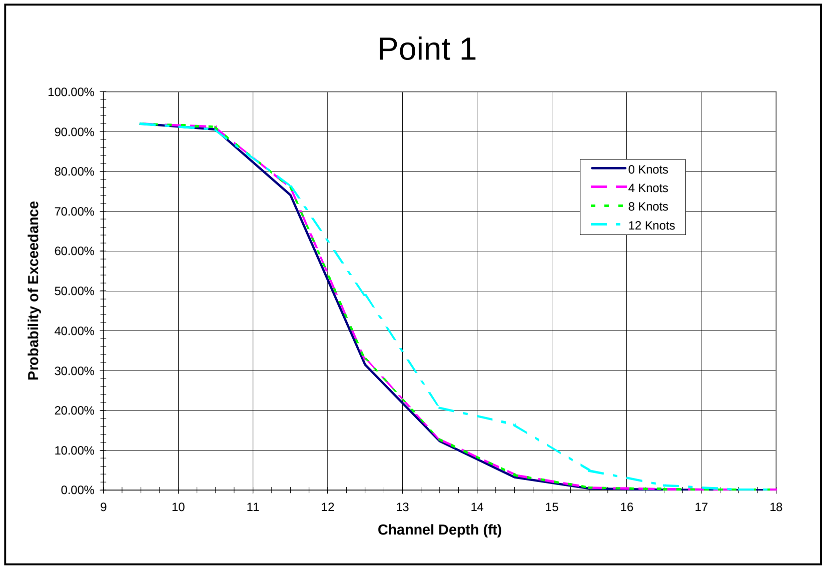
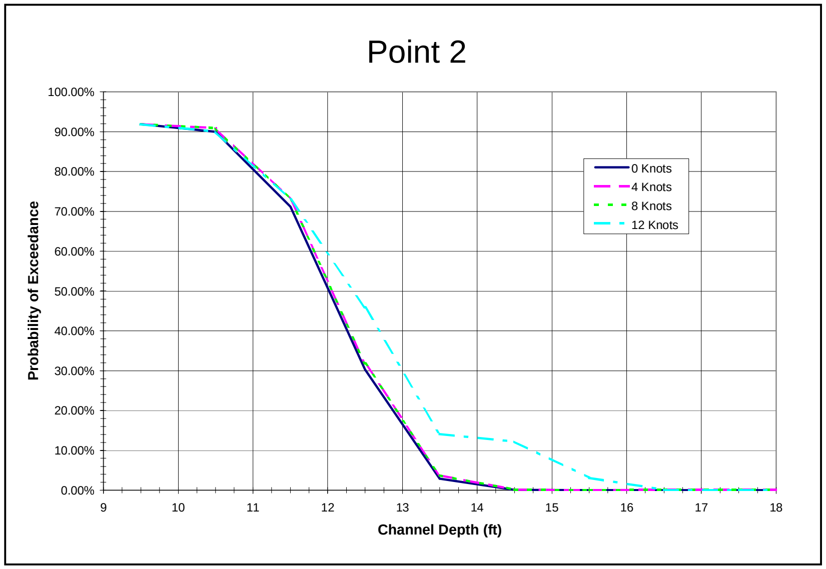
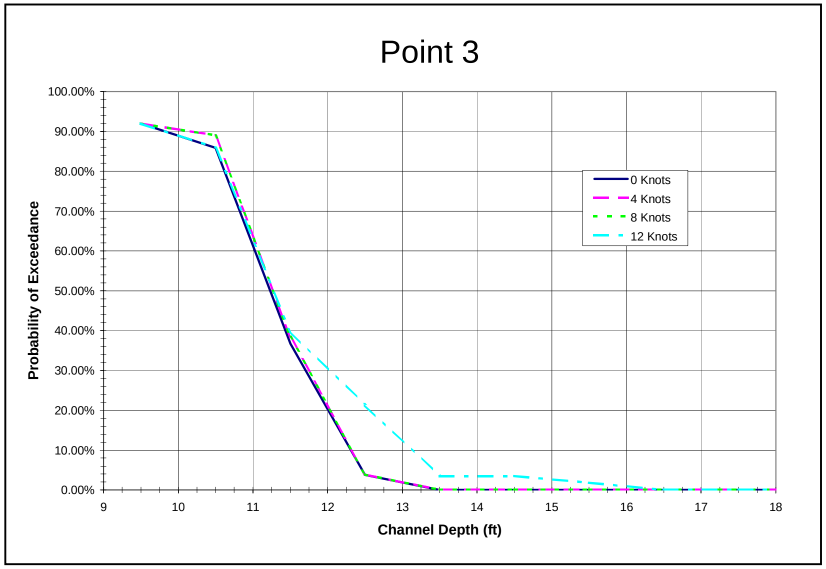
FIGURE 7.7

File pat/File name







Safe Navigation Depth Exceedance Probability

Fire Island Design Vessel

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FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 7.8

File pat/File name

8. DETAILED ANALYSIS OF SELECTED INLET ALTERNATIVES

8.1 Cost Estimation

The following paragraphs summarize the basic parameters and assumptions used to develop the cost estimates presented in this section. Detailed spreadsheets and additional backup estimates for each alterative are presented in Appendix E.

The interest rate used in all the costs estimates is 5.125%. "Subtotal Costs" shown in the tables below account for 15% uncertainty. Engineering and design fees are 7% of the subtotal, while supervision and administration fees were a percentage of the subtotal, given by the following formula:

$$S\,\&\,A\% = \frac{17 - \left(2.1 * \log\left(\frac{Subtotal}{1000}\right)\right)}{100}$$

For nearshore dredging (i.e., within the deposition basin, ebb shoal, and/or flood shoal) at Shinnecock and Moriches Inlets, a mobilization cost of $1,000,000 is assumed for a typical 30" cutterhead dredge. Unit prices at these two inlets are determined using standard cost estimation techniques employed by dredging contractors, which take into account the quantity of material to be dredged and the distance to the placement site. Unit price ranges from $5.00 to $5.70 per cubic yard. These estimates are in line with recent bids, which range from $2.50 to $6.11 per cubic yard at Shinnecock and $4.85 to $5.44 per cubic yard at Moriches. At Fire Island Inlet, where a relatively large number of similar dredging operations have been conducted since 1990, traditional cost estimates are complicated by a large distance between dredging and placement sites and the likely use of booster pumps. Therefore, average mobilization costs and unit prices specific to each placement location were used. The averages are based on four recent similar dredging contracts; the average unit price and mobilization cost for placement downdrift at Gilgo beach is $7.30/cy and $2,608,000 and the average unit price and mobilization cost for placement updrift at Robert Moses State Park is $4.20/cy and $593,000.

Note that the costs for each alternative take into account the cost of the existing bypassing deficit at each inlet by assuming this deficit would have to be eventually made up by importing sand from an offshore source. This approach allows for a valid comparison between alternatives. Otherwise, alternatives which do nothing to offset the deficit, including Existing Practice, would appear less costly than they really are compared to others that directly reduce or eliminate this deficit.

Offshore dredging utilizes the FIMP offshore borrow sites identified in Borrow Source Investigations (September 29, 2005). As explained above, it is assumed that offshore dredging is used to offset the longshore sediment transport deficit identified in the sediment budget for each alternative. As with nearshore dredging, a typical mobilization cost of $1,000,000 is assumed. The unit price for offshore dredging at each inlet is the same as used in the beach fill cost estimates (March, 2006) because the dredged volumes and borrow site locations are similar. The unit price for Shinnecock is $6.50/cy, for Moriches, $7.00/cy, and for Fire Island, $9.00/cy.

Costs for additional structures (e.g. the T-groins or Spur Jetty) are based on recent unit prices for rock. Initial construction, annual, and overhaul and replacement costs for the semi-fixed bypassing plants considered at Shinnecock and Moriches Inlet are based on the work by Williams et al (1998) are scaled to 2005 price levels.

8.2 Shinnecock Inlet

Based on the results from the preliminary screening, the following alternatives were selected for further consideration and detailed analysis at Shinnecock Inlet:

Alt. 1: Authorized Project (AP) + Dredging the Ebb Shoal
Alt. 2: AP + Nearshore Structures
Alt. 3: AP + Offshore Dredging
Alt. 4: AP + Semi-fixed Bypass System
Alt. 5: AP with Reduced Dimensions of Deposition Basin
Alt. 6: AP + Dredging the Flood Shoal
Alt. 7: AP + Shortening the East Jetty
Alt. 8: AP + West Jetty Spur

Each of these alternatives would achieve the goals of providing reliable navigation through the Federal navigation channel, restoring natural sediment pathways, and reducing adjacent shoreline erosion, albeit to varying degrees and at different costs. The performance of each alterative as regards to navigation and sand bypassing was investigated using analytical (sediment budget) and numerical (Delft3D) modeling tools. In addition, detailed costs estimates were developed for each alternative. A summary of pros and cons for each alternative is also presented in the following sections. Finally, a recommendation is made.

8.2.1 Summary of Existing Conditions

The *Existing (c. 2001)* condition sediment budget for Shinnecock Inlet (see Section 5 and Figure 5-20) suggests a net westerly transport rate of 157,000 m^3/yr entering the updrift (east) boundary of the inlet system. The budget also suggests that approximately 75% (117,000 m^3/yr) of the net updrift westerly transport entering the Updrift Beach bypasses the inlet system, including the channels, deposition basin, shoals and adjacent beaches. The remaining 25% (40,000 m^3/yr) accumulates within the inlet shoals and adjacent beaches. The budget includes dredging of the deposition basin at a rate of 65,000 m^3/yr. With the exception of the last dredging event in 2004, material dredged from the deposition basin has been placed on the west beach immediately adjacent to the west jetty.

The design capacity of the deposition basin (Figure 2-1) is approximately 350,000 m^3, and, according to the General Design Memorandum (GDM) (USACE-NAN, 1988) and project authorization, the originally anticipated dredging interval was 1.5 years. These design characteristics were based on an anticipated gross sediment transport rate of approximately 300,000 m^3/yr. This estimate of the gross sediment transport rate (roughly two times the net transport in the *Existing* sediment budget) is probably not far off according to numerical sediment transport calculations (see Section 5). However, since 1990 the deposition basin has only been dredged three times, in 1993, 1998, and 2004 and as of January 2006 the deposition basin appeared to have accumulated enough material to be dredged again[15]. Therefore, the actual dredging interval has been roughly 4 years, instead of the anticipated 1.5 years. Rapid shoaling between March 2004 and January 2006 may owe to less dredging in 2004[16], significant storms in the fall of 2004, and/or the fact that approximately 555,000 $m^3$of fill were placed immediately west of the inlet as part of the WOSI project in 2004/05.

Further, modeling results confirm that accumulation in the deposition basin is not as high as anticipated. Specifically, Delft3D morphological simulations under "representative" wave conditions based on long-term wave records suggest that roughly 80,000 m^3/yr accumulate in the deposition basin during the first

[15] USACE-NAN controlling depth report: http://www.nan.usace.army.mil/business/buslinks/navig/cdr.htm
[16] The dredge volume in 2004 was somewhat smaller than in 1993 and 1998 because 2/3 of the deposition basin was dredged to -19 ft. MLW vs. -20 ft.

two years following a dredge event, but not much more after that (see Figure 8-1). Over a four year period the net accumulation is roughly 64,000 m^3/yr, which is consistent with the *Existing (c. 2001)* conditions sediment budget. Model results also show that the maximum capacity of the basin (assuming 95,000 m^3 for 2 feet of payable overdredge) is on the order of 325,000 m^3 (230,000 m^3 above design depth of -20 feet MLW), which is very similar to the design capacity assumed in the GDM.

Available survey data (see Section 5 and Appendix A) also indicate that there are periods of time when morphological changes and accumulation in the basin are very slow. For example, after dredging in 1993, changes were relatively slow for the following 4 years. In fact, as of August 1997 there were only approximately 30,000 m^3 above the design depth (-20 feet MLW). On the other hand, approximately 200,000 m^3 accumulated in the basin between August 1997 and May 1998 (9 months). In September of 1998, 311,000 m^3 (including allowable overdredge) were dredged and as of the summer of 2001 (almost 3 years later), a very small volume of sand had accumulated in the deposition basin, which was only dredged again in March 2004 (231,000 m^3). Therefore, most of this accumulation took place between the summer of 2001 and the fall of 2003. Numerical calculations based on available wave data (see Section 5) also confirm large year-to-year variability in longshore sediment transport (LST) rates. Specifically, gross LST between August 1997 and May 1998 was over 500,000 m^3/yr, or almost three times the gross transport between June 1996 and August 1997 or May 1998 and July 2001 (less than 200,000 m^3/yr during both periods).

Therefore, analysis and selection of the alternatives needs to account for the fact that the "filling-time" of the deposition basin appears to be anywhere from less than 2 years to 5 years depending mostly on wave climate conditions.

To date, most of the material excavated from the channel and deposition basin has been placed along the shoreline immediately west of the inlet (west beach), between the west jetty and the Ponquogue ebb shoal reattachment point, a shoreline reach that suffers from chronic erosion. Only during initial project in 1990 and as part of the 2004 dredging project, was sand placed farther west at Ponquogue beach (approximately 200,000 m^3 and 231,000 m^3, respectively). Surveys and numerical model results suggest that a large percentage of the material placed on the west beach may return to the inlet instead being transported farther west. As such, Existing Practice[17], although successful in maintaining a minimum level of protection at the west beach, is not very efficient in terms of preventing large shoreline fluctuations or providing hydraulic bypassing farther downdrift, although recent surveys arguably suggest that this system results in relatively efficient natural bypassing of the net longshore sediment transport balance (see Section 5). In other words, the chronic erosion at west beach is not necessarily related to a lack of overall bypassing but instead may be due to the geometric configuration of the inlet system and local wave and current conditions.

It is also evident that the Existing Practice has been very successful in providing for continuous reliable navigation (see Section 5) without a need for the emergency dredging projects which were typical of historic channel maintenance practices prior to project implementation in 1990. This success is due to the straight channel alignment and greater than required depths, both maintained because of the deposition basin.

[17] Note that "Existing Practice" refers to typical dredging and placement operations since construction of the currently Authorized Project in 1990. Specifically, Existing Practice consists of maintenance of a channel and deposition basin per the Authorized Project dimensions (200 feet wide navigation channel to an elevation of -10 ft MLW enveloped by a deposition basin 800 feet wide by 2,600 feet long, to an elevation of -20 ft MLW) but on a 4 year dredging interval instead of the anticipated 1.5 year interval. In addition, under Existing Practice the Authorized Project's goal of placing 80% of the dredged material in the littoral zone approximately 5,000 feet west of the Inlet has not been implemented and most of the material has been placed within 4,000 feet of the west jetty.

The following sections present a detailed analysis of each of the alternatives shortlisted at Shinnecock Inlet in Section 3.

8.2.2 Alternative 1: Authorized Project (AP) + Dredging the Ebb Shoal

As explained in Section 3, the premise behind this alternative is to offset the deficit in the *Existing (c. 2001)* sediment budget (40,000 m^3/yr) by dredging areas of the ebb shoal outside the deposition basin such as the updrift lobe or around the seaward slope of the shoal. Dredging the ebb shoal and placing the sediment beyond the area of direct influence of inlet processes would theoretically restore longshore sediment transport process. It would also serve to constrain or even reverse sediment accumulation in the ebb shoal. On the other hand, this alternative may have some adverse effects on ebb shoal morphology, natural bypassing, and adjacent shorelines if the amount of material dredged from the ebb shoal is too large.

The potential borrow area is roughly 6,000 feet long and located seaward of the -20 foot NGVD contour (See Figure 8-2) to minimize impacts on nearshore wave climate conditions and existing sediment transport processes which, according to the sediment budget, currently account for a large percentage of the bypassing around the inlet. Dredging could be performed to the -35 foot contour (NGVD). Based on the recent bathymetry data, the volume of sediment available within this area is roughly 1 million m^3. However, it would only be necessary to dredge within a much smaller area (roughly 1,000 feet long) to produce the amount necessary for each operation. The optimum borrow site location could be selected prior to each dredging event based on condition surveys which would need to include coverage of the ebb shoal. These surveys would be used to identify areas of ebb shoal growth or recovery from previous dredging operations as candidates for additional dredging. A sketch of the conceptual design for this alternative is shown in Figure 8-2.

Dredging could be easily performed using a traditional floating plant (hydraulic or hopper dredge) on contract. Dredging could easily conform to a design bypass schedule and quantity, as opposed to dredging from the deposition basin, which, as already experienced, does not accumulate sediment at the same rate every year. More frequent dredging and placement would reduce shoreline fluctuations along West Beach and farther downdrift.

Ebb shoal dredging would have to be supplemented with dredging of the authorized channel and deposition basin to provide adequate navigation reliability. Dredging of the ebb shoal could take place on the same frequency as the channel and deposition basin (4 years on average) (Alternative 1A) or on a more frequent interval (e.g., every 2 years) but coinciding with dredging the channel and deposition basin every other operation to reduce costs (Alternative 1B).

<u>Summary of Modeling Results</u>

The Delft3D morphological model was used to simulate future sedimentation in the dredged areas and to assess potential impacts to adjacent inlet cells such as the West Beach and the Downdrift Beach. Results for a simulation after dredging roughly 160,000 m^3/yr (i.e., a typical operation in a 4 year dredging cycle) are shown in Figure 8-3. This model results confirm that the dredged area would recover over time. More importantly, a detailed volumetric analysis indicates that changes in other cells (including the Deposition Basin, West Beach, and Downdrift Beach) would be similar to *Existin*g (c. 2001) conditions, which supports the conceptual sediment budget presented below.

A more conservative condition which assumes that roughly 1 million m^3 were removed from the ebb shoal was also simulated (Figure 8-4). These results also confirm gradual though slower recovery of the dredged area and minimal impacts on the adjacent inlet cells.

Sediment Budget

A conceptual sediment budget for this alternative is presented in Figure 8-5. As shown in this figure, the LST deficit (40,000 m^3/yr) is made up by dredging the ebb shoal and placing the material downdrift. Note that it is assumed that the dredging interval (2 or 4 years) would have a negligible effect on the average annual sediment budget. However, as discussed above, more frequent bypassing would reduce existing swings in shoreline position. It is assumed that over time dredging would offset ongoing accumulation in the ebb shoal (18,000 m^3/yr) and possibly reverse historical accumulation at a rate of up to 22,000 m^3/yr. This "erosion" of the ebb shoal would offset future accumulation in the Updrift Beach (6,000 m^3/yr), Flood Shoal (13,000 m^3/yr), West Beach (1,000 m^3/yr) and Downdrift Beach (2,000 m^3/yr) which is assumed would continue. Thus, over a 50 year project life as much as 1.1 million m^3 could be dredged from the ebb shoal. Note that is still less than 20% of the total volume in the ebb shoal, accumulated since it opened in 1938.

Sand dredged from the deposition basin could continue to be placed immediately west of the inlet or part of it could be placed farther downdrift, assuming that the sand dredged from the ebb shoal is placed on the West Beach. The net effect on the sediment budget would be similar; however, more frequent dredging from the ebb shoal may be better suited to offset erosion along the West Beach while a large percentage of the sand from the deposition basin could be placed downdrift, past the Ponquogue ebb shoal attachment. Regardless, pumping distance will be less than 2 miles and the costs associated with each option would be very similar.

Costs

First costs and annual costs developed for Alternative 1A (4 year cycle) and 1B (2 year cycle) and are summarized in Table 8-1 and Table 8-2, respectively. Costs for Existing Practice and the Authorized Project are shown in Table 8-3 for comparison. Note that Existing Practice costs take into account the cost of the existing bypassing deficit (40,000 m^3/yr) by assuming this deficit would have to be eventually made up by importing sand from an offshore source. This is just a way to assign a cost to the existing bypassing deficit and it does not mean that offshore dredging is currently part of the Existing Practice at Shinnecock Inlet. Also note that this cost does not account for damages related to a more or less eroded shoreline condition downdrift.

Increased dredging frequency at the ebb shoal (every 2 vs. every 4 years) increases the number of dredge mobilizations and demobilizations and thus the annual costs increase by about $322,000/yr (21%) as compared to dredging on a 4 year cycle. The cost of dredging the ebb shoal on a two year cycle is actually comparable to the cost of Existing Practice including dredging offshore every 10 years to offset the existing LST deficit.

Table 8-1: Cost Summary for SI Alternative 1A: AP + Ebb Shoal Dredging every 4 years

Plan Component	Dredging Interval (years)	Quantity (1000x m^3 (cy))	Mob/ Demob ($000)	Unit Cost ($/$m^3$ ($/cy))	Subtotal Cost ($000)	E&D and S&A ($000)	Total Cost Per Operation ($000)	Annual Cost ($000)
Channel & Deposition Basin Dredging	4	260 (340)	$1,000	$6.55 ($5.00)	$3,105	$518	$3,623	$1,033
Ebb Shoal Dredging	4	160 (210)	Same contract	$6.55 ($5.00)	$1,208	$212	$1,419	$405
							Grand Total	$1,438

Table 8-2: Cost Summary for SI Alternative 1B: AP + Ebb Shoal Dredging every 2 years								
Plan Component	**Dredging Interval (years)**	**Quantity (1000x m^3 (cy))**	**Mob/ Demob ($000)**	**Unit Cost ($/$m^3$ ($/cy))**	**Subtotal Cost ($000)**	**E&D and S&A ($000)**	**Total Cost Per Operation ($000)**	**Annual Cost ($000)**
Channel & Deposition Basin Dredging	4	260 (340)	Same contract	$6.55 ($5.00)	$1,955	$334	$2,289	$653
Ebb Shoal Dredging	2	80 (105)	$1,000	$6.55 ($5.00)	$1,754	$301	$2,055	$1,107
							Grand Total	$1,760

Table 8-3: Cost Summary for SI Existing Practice with AP								
Plan Component	**Dredging Interval (years)**	**Quantity (1000x m^3 (cy))**	**Mob/ Demob ($000)**	**Unit Cost ($/$m^3$ ($/cy))**	**Subtotal Cost ($000)**	**E&D and S&A ($000)**	**Total Cost Per Operation ($000)**	**Annual Cost ($000)**
Channel & Deposition Basin Dredging	4	260 (340)	$1,000	$6.55 ($5.00)	$3,105	$518	$3,623	$1,033
LST Deficit	10	400 (524)	$1,000	$8.50 ($6.50)	$5,067	$822	$5,889	$767
							Grand Total	$1,800

<u>Summary of Pros and Cons</u>

Pros and cons associated with this alterative are summarized as follows:

Project Needs

Dredging the ebb shoal would offset the existing longshore sediment transport deficit and restore (in terms of average volume per year) longshore sediment transport processes downdrift of the inlet. Continued dredging of the deposition basin would be used to mitigate local erosion of the west beach. Depending on future performance, which is to be assessed by routine monitoring surveys, part of the sediment from the deposition basin could be placed farther downdrift beyond the ebb shoal attachment point. Conversely, ebb shoal material could be occasionally placed on the west beach if necessary.

Continued dredging of the deposition basin to -20 feet MLW would maintain navigation reliability through the inlet.

Risk & Uncertainty

This alternative would entail very little risk and uncertainty as compared to others since it involves continuation of Existing Practice under the Authorized Project dimensions and bypassing would be improved using proven dredging technology with relatively well known costs, schedules, performance, and environmental effects. Uncertainty regarding accurate estimates of ebb shoal growth and its effects on the sediment budget and long shore sediment transport processes could be managed through regular monitoring surveys of the ebb shoal and dredging in areas of observed growth.

Potential impacts on nearshore waves and littoral processes can be minimized by monitoring future morphological changes and managing the dredging program accordingly.

Environmental Criteria

Most of the shortlisted alternatives scored similarly with regards to environmental criteria. Nonetheless, ebb shoal dredging has some distinct advantages over some of the others in that it does not involve construction of hard structures, dredging of more environmentally sensitive areas such as the flood shoal, or reducing beach width along the updrift beach. Effects on water quality (flushing) would be insignificant because it would not affect existing tidal prism (discharge) through the inlet.

Although the annual sediment bypass deficit would be made up with sand from the ebb shoal, accumulation in the adjacent beaches and the flood shoal would continue and, technically, the natural "sediment pathways" would not be fully restored. In that sense this alternative would only be slightly better than existing conditions.

Economic Criteria

This alternative would maintain safe access to the existing commercial fishing facilities and would reduce existing flooding risks by reducing the potential for a breach downdrift. It would not have any impacts on land use or ownership. More importantly, it would reduce the cost of providing for 100% bypassing across the inlet compared to Existing Practice, which it has been assumed would require periodic placement of sand from and offshore source to offset the existing net LST deficit.

Compared to other alternatives it is also relatively inexpensive because of the savings associated with using the same equipment to maintain the channel and deposition basin and dredge the ebb shoal.

Recreational Criteria

This alternative would maintain navigation reliability and thus it would not impact access to recreational fish resources. Water and foreshore related recreational resources such as surfing, boating, fishing, swimming, sunbathing, sightseeing, birdwatching, etc. would also be maintained or improved. Specifically, increased bypassing would increase the beach area downdrift of the inlet and within the Shinnecock Inlet County Park. Sand from the ebb shoal will be very compatible with sand on the beach and will not significantly affect the profile or the texture of the sand as other alternatives might. Public access to this resource will not be affected either. Permanent construction on the beach will not be required.

Impacts on surfing can be minimized by dredging only the seaward edge of the shoal so that nearshore wave breaking patterns are not significantly affected.

Engineering Criteria

This is perhaps the category where this alternative offers some of the most important advantages, particularly in the case of a relatively short ebb shoal dredging cycle. First, dredging has a distinct advantage over other artificial bypassing systems in terms of capacity. Dredging a total of roughly 420,000 m^3 per event (assuming a 4-year cycle) is not an issue with the typical ocean-going dredging equipment available. Source flexibility is also an important advantage of this alternative given the relatively large size of the area of ebb shoal that could be dredged.

A hydraulic or hopper dredge also offers placement flexibility since they can be easily used to place material at various locations such as the nearshore and onshore areas immediately downdrift of the inlet, and areas farther downdrift past the Ponquogue attachment point.

Continuity, or the ability of the bypassing system to provide continuous bypassing in a manner similar to natural longshore drift, is perhaps the most significant disadvantage of this alternative. Periodic dredging and placement is not the same as natural longshore drift and it certainly not as "continuous" as a bypassing plant. Nonetheless, this problem may be partially mitigated by a more frequent dredging cycle (every 2 years) and the fact that continued dredging, regardless of wave climate and erosion downdrift, can be used to build a "buffer" that will offset subsequent erosion waves. This is not easy to do by means of only dredging the deposition basin because sand only accumulates in the basin after significant storms occur and thus the erosion has already taken place downdrift.

As explained above, floating dredge plants are considered highly reliable and are readily available. Thus this alternative also scores high in terms of performance.

Finally, this alternative does not rely on significant investments or modifications that might be difficult to change or reverse if unsuccessful. It also provides the ability to implement a full range of alternatives at the end of the project life without dictating a future course of action that will be difficult to change.

8.2.3 Alternative 2: AP + Nearshore Structures (T-groins) along the West Beach

As explained in Section 3, the premise of this alternative is that by stabilizing the west beach with a series of T-Groins, future sand placement needs in this area will be minimized and the sand excavated from the deposition basin could be placed past the ebb shoal attachment area. Dredging of the authorized channel and deposition basin to provide adequate navigation reliability and bypassing would continue, albeit at a slightly lower rate since less sand would flow from the west beach back into the deposition basin.

Groins, usually constructed as a series, are coastal structures that act to retard longshore sediment transport by impounding sand between them. At the west beach, a groin system would reduce beach nourishment requirements as compared to a purely dredging alternative. In particular, T-groins would trap sand on the west beach, bring the shoreline into an equilibrium planform, and minimize offshore losses of sand (as compared to standard groins).

The proposed plan consists of 5 full T-groins and one spur that would be anchored to the west jetty. A gap length of 325 feet and a "T" length of 300 feet are proposed. The maximum crest elevation is +6.0 feet NGVD with a T-crest width of 14 feet. The T-portion of the groin is to be constructed of two layers of 1-ton core stone and two layers of 14-ton armor stone. The portion of the T-groin perpendicular to shore would be constructed of 50 to 1500 lb core stone and one layer of 7-ton capstone. A sketch of the conceptual design for this alternative is shown in Figure 8-6.

<u>Sediment Budget</u>

A conceptual sediment budget for this alternative is presented in Figure 8-7. As shown on this figure, it is assumed that the existing LST deficit (40,000 m^3/yr) would continue because the T-groins would not do anything to offset it. Accumulation on the shoals and adjacent beaches would continue as under existing conditions. Periodic renourishment along the west beach would be reduced from 65,000 m^3/yr to 10,000 m^3/yr. It is assumed that dredging of the deposition basin would be slightly reduced (from 65,000 m^3/yr to 50,000 m^3/yr). In addition, sand dredged from the deposition basin would be placed farther downdrift, past the ebb shoal attachment point. The net effect of the structures on the sediment budget would be a trade-off between less "natural" bypassing from the deposition basin to the updrift lobe and beyond (86,000 m^3/yr vs. 126,000 m^3/yr) and more "mechanical" bypassing (40,000 m^3/yr vs. 0 m^3/yr).

<u>Costs</u>

First costs and annual costs developed for this alternative are summarized in Table 8-4. The cost of building the T-groins is based on recent prices used in the cost estimate for construction of similar structures at Coney Island. The cost of initial fill from an offshore borrow site is also included in the construction cost. Overall, Table 8-4 indicates that this alternative is significantly more costly that than continuing the authorized project ($1.1 M/yr more, or a 60% increase). Reduced dredging volumes at the deposition basin lower the dredging costs slightly (just over $100,000/yr), but not nearly enough to offset the cost of the structures ($1.3 M/yr).

Table 8-4: Cost Summary for SI Alternative 2: APD + Nearshore Structures (T-groins)								
Plan Component	**Dredging Interval (years)**	**Quantity (1000x m^3 (cy))**	**Mob/ Demob ($000)**	**Unit Cost ($/$m^3$ ($/cy))**	**Subtotal Cost ($000)**	**E&D and S&A ($000)**	**Total Cost Per Operation ($000)**	**Annual Cost ($000)**
Channel & Deposition Basin Dredging	4	200 (262)	$1,000	$6.55 ($5.00)	$2,657	$447	$3,103	$885
LST Deficit	10	400 (524)	$1,000	$8.50 ($6.50)	$5,067	$822	$5,889	$767
T-groins					$20,889	$3,118	$24,007	$1,341
							Grand Total	$2,993

<u>*Summary of Pros and Cons*</u>

Pros and cons associated with this alterative are summarized as follows:

Project Needs

Constructing the T-groins would essentially eliminate the chronic erosion problem along the west beach and it would free up the most of the sand now being placed there to be directly bypassed to the beaches downdrift of the inlet. However, according to the proposed *Existing (c. 2001)* and with-project sediment budgets, the net effect would be a continuation of the existing longshore sediment transport deficit, because accumulation in the shoals and beaches adjacent to the inlet would continue at roughly the same rate. Moreover, part of the existing “natural” bypassing across the inlet would be replaced with “mechanical” bypassing from the deposition basin. Mechanical, or artificial, bypassing is not continuous, although it has the advantage of being a process that can be easily monitored and measured. Estimates of natural bypassing will always have to rely on indirect measurements (i.e., beach and shoal surveys) and/or analytical/numerical assessments.

In addition, it is possible that less sand placement on the west beach and subsequent transport into the deposition basin and updrift lobe of the ebb shoal would slightly reduce accumulation in the ebb shoal and therefore reduce the LST deficit. However, only future monitoring could confirm or refute this theory.

Continued dredging of the deposition basin to -20 feet MLW would maintain navigation reliability through the inlet.

Risk & Uncertainty

The general performance of T-groins as regards to shoreline stabilization is now fairly well understood and documented. Therefore, there is a relatively small amount of risk and uncertainty associated with this alternative. In addition, Existing Practice under the Authorized Project would continue and therefore there would very little risk and uncertainty associated with navigation reliability.

Finally, as explained above, one aspect where uncertainty would be somewhat reduced as compared to existing conditions would be in the amount of sand that actually bypasses the inlet, since this estimate would not solely rely on indirect measurements as it does now. On the other hand, uncertainty with regards to the volume of material that actually accumulates in the shoals and to a lesser extent the adjacent beaches would continue to be problematic. In other words, the LST deficit could in fact be higher than predicted.

Environmental Criteria

This alternative is perhaps the worst of the shortlisted ones regarding this aspect of the evaluation, mostly because it is based on a structural solution to the erosion problem at the west beach. In addition, as explained above, sediment pathways would not be further restored from existing conditions. On the other hand, this alternative would not require dredging of any additional areas either on the shoals or the updrift beach. Finally, effects on water quality (flushing) would be insignificant because it would not affect existing tidal prism (discharge) through the inlet.

Economic Criteria

Like most of the others, this alternative would maintain safe access to the existing commercial fishing facilities. However, unless the LST deficit is offset though periodic offshore dredging, it would not reduce existing flooding risks downdrift by reducing the potential for a breach. More importantly, the cost of providing for 100% bypassing across the inlet compared to existing conditions, would increase significantly compared to existing conditions and many of the other shortlisted alternatives.

On the other hand, flooding risks in Shinnecock Bay due to a breach through the west beach could be somewhat reduced due to the additional protection offered by the structures.

Recreational Criteria

This alternative would maintain navigation reliability and thus it would not impact access to recreational fish resources. However, construction of the T-groins will impact foreshore related recreational resources such as surfing, swimming, sunbathing, and sightseeing.

Engineering Criteria

Dredging and bypassing capacity related to the deposition basin will be the same as under Existing Conditions since the same type of equipment would be used (cutterhead or hopper dredges). In fact, average annual dredging volumes will be reduced from 65,000 m^3/yr to 50,000 m^3/yr. On the other hand, source flexibility would continue to be an issue since sand would only be dredged from the deposition basin, where the accumulation rate is not constant from year to year.

Placement flexibility would be a plus for this alterative since the dredge could easily place sand in areas downdrift past the Ponquogue ebb shoal attachment point and along the west beach. As with all other alternatives that rely mostly on dredging of the deposition basin, continuity would be a drawback. In fact, this could be an even bigger issue for this alternative since less material is likely to accumulate in the deposition basin and, depending on the wave climate, a relatively long period of time may elapse between consecutive dredging events, although on average it has been assumed that the dredging cycle would remain at approximately 4 years. Performance of the T-groins should not be an issue, since they “lock” the shoreline in place.

Finally, the largest drawback of this alternative is that it requires a significant modification of the inlet system and a large initial investment that will be nearly impossible to reverse.

8.2.4 Alternative 3: AP + Offshore Dredging for the West Beach

This alternative is based on the concept of mitigating erosion along the west beach and the net LST deficit across the inlet by “importing” sand from an offshore source. More sand dredged from the channel and deposition basin would be placed west of the Ponquogue attachment point and the west beach would be maintained through a combination periodic offshore and deposition basin dredging.

Sediment Budget

A conceptual sediment budget for this alternative is presented in Figure 8-8. As shown in this figure, it is assumed that the existing LST deficit (40,000 m^3/yr) would be offset by periodic dredging from offshore. Further, it is assumed the 40,000 m^3/yr would be placed on the west beach and that this volume would be augmented with 25,000 m^3/yr from the deposition basin in order to meet the renourishment needs. The rest (approximately 45,000 m^3/yr) would be directly bypassed farther west past the ebb shoal attachment. Accumulation on the shoals and adjacent beaches would continue as under existing conditions.

Costs

First costs and annual costs developed for this alternative are summarized in Table 8-5. The only difference between this alternative and Existing Practice (Table 8-3) is that offshore dredging would be performed every two years to provide for more continuous protection of the west beach. This frequency increase makes this alternative more expensive than continuing the Existing Practice (approximately $0.5 M/yr more, or a 27% increase).

Table 8-5: Cost Summary for SI Alternative 3: AP + Offshore Dredging for West Beach

Plan Component	Dredging Interval (years)	Quantity (1000x m^3 (cy))	Mob/ Demob ($000)	Unit Cost ($/$m^3$ ($/cy))	Subtotal Cost ($000)	E&D and S&A ($000)	Total Cost Per Operation ($000)	Annual Cost ($000)
Channel & Deposition Basin Dredging	4	260 (340)	$1,000	$6.55 ($5.00)	$3,105	$518	$3,623	$1,033
Offshore Dredging	2	80 (105)	$1,000	$8.50 ($6.50)	$1,935	$379	$2,314	$1,247
							Grand Total	$2,280

Summary of Pros and Cons

Pros and cons associated with this alterative are summarized as follows:

Project Needs

Offshore dredging in combination with continued dredging of the deposition basin would mitigate local erosion of the west beach, offset the existing longshore sediment transport deficit and restore (in terms of average volume per year) longshore sediment transport processes downdrift of the inlet. It is worth noting again that the LST deficit would be offset by offshore dredging, but that accumulation of sand in the shoals and adjacent beaches would continue. Therefore, unlike Alternative 1 (ebb shoal dredging), this alternative does not "balance" the sediment budget by reducing accumulation within the inlet.

Unlike any other alternative investigated in this study, this alternative meets the stated project needs by using a source of sediment arguably exterior to the natural littoral system (i.e., the offshore borrow area), not by directly improving the bypassing efficiency of the inlet system or by reducing erosion along west beach. As such, this alternative may not be sustainable in the long term.

Continued dredging of the deposition basin to -20 feet MLW would maintain navigation reliability through the inlet.

Risk & Uncertainty

As with Existing Practice, uncertainty with regards to the volume of material that actually accumulates in the shoals and to a lesser extent the adjacent beaches would be problem. In other words, the LST deficit could in fact be higher than predicted. There is also some uncertainty associated with the performance of beach fill on the west beach.

Environmental Criteria

One environmental drawback of this alternative compared to Existing Practice and others is the need for dredging offshore, which may induce some minimal additional impacts. As with most of the other alternatives, effects on water quality (flushing) would be insignificant because it would not affect existing tidal prism (discharge) through the inlet.

Although the annual sediment bypass deficit would be made up with sand from offshore, accumulation in the inlet shoals and adjacent beaches would continue and, natural "sediment pathways" would not be restored. In that sense this alternative would be the same as existing conditions.

Economic Criteria

This alternative would maintain safe access to the existing commercial fishing facilities and would reduce existing flooding risks by reducing the potential for a breach downdrift. It would not have any impacts on land use or ownership. Although the cost of providing for 100% bypassing across the inlet would be increased compared to Existing Practice, flooding risks would be reduced because of the more continuous bypassing operation based on a 2 year offshore dredging cycle as opposed to the 10 year cycle.

Recreational Criteria

This alternative would maintain navigation reliability and thus it would not impact access to recreational fish resources. Water and foreshore related recreational resources such as surfing, boating, fishing, swimming, sunbathing, sightseeing, birdwatching, etc. would also be maintained or improved. Specifically, increased bypassing would increase the beach area downdrift of the inlet and within the Shinnecock Inlet County Park. However, sand from offshore will not be 100% compatible with sand on the beach and may affect the profile shape or the texture of the sand. Public access to these resources will not be affected either. Permanent construction on the beach will not be required.

Engineering Criteria

As in the case of previous alternatives, dredging and bypassing capacity would not be an issue. However, source flexibility would be restricted both at the inlet (when shoaling in the deposition basin is slow) and offshore (limited borrow site availability). Placement flexibility would be an advantage since ocean dredges can easily place material on the west beach and in areas farther downdrift past the Ponquogue attachment point. As in the case of Alternative 1, the lack of continuity would be a disadvantage of this alternative.

Also like Alternative 1, this alternative does not rely on significant investments or modifications that might be difficult to change or reverse if unsuccessful. It also provides the ability to implement a full range of alternatives at the end of the project life without dictating a future course of action that will be difficult to change.

8.2.5 Alternative 4: AP + Semi-fixed Bypass System

The main advantage of a semi-fixed bypassing system is that it could provide for more continuous bypassing. The semi-fixed plant at Indian River Inlet, Delaware served as the model for the proposed system. A crawler crane would be used to position a jet pump on the updrift fillet near MLW. A pump at the inlet would supply the jet pump with clean water from the inlet. The jet pump would discharge to a booster pump, which would then pump the slurry to the downdrift beach. The maximum estimated capacity of the proposed system is 100,000 m^3/yr (131,000 cy/yr). Sand would be bypassed to the West Beach and, with the help a second booster pump, farther downdrift past the ebb shoal attachment point. A sketch of the conceptual design for this alternative is shown in Figure 8-9.

Sediment Budget

A conceptual sediment budget for this alternative is presented in Figure 8-10. As shown in this figure, it is assumed that approximately 100,000 m^3/yr would be directly bypassed from the updrift fillet to the west beach and the downdrift beach (25,000 m^3/yr and 75,000 m^3/yr, respectively). Theoretically, this would reduce the volume of sediment entering the deposition basin and possibly the rate of accumulation in the basin (albeit by a small amount). It is further assumed that future accumulation on the updrift fillet would be reduced to zero from 11,000 m^3/yr. Sand dredged from the deposition basin (40,000 m^3/yr) would continue to be placed on the West Beach to supplement plant bypassing (for a total of 65,000 m^3/yr). Accumulation in the flood shoal, updrift beach, downdrift lobe, and updrift beach, is also assumed to continue at the existing rate. Therefore, the net effect of the bypassing plant on overall bypassing efficiency would be a reduction of the LST deficit from 40,000 m^3/yr to 29,000 m^3/yr.

Costs

Overall costs for this alternative are summarized in Table 8-6. Initial construction, annual, and overhaul and replacement costs for the semi-fixed bypassing plants proposed at Shinnecock and Moriches Inlets are based on the work by Williams et al (1998). All costs are scaled from 1997 to 2005 price levels. The design detailed by Williams is assumed to be applicable for both inlets for the purposes of cost estimation. Additional details are provided in Appendix E.

Note that costs assume that dredging of the deposition basin would continue on a 4 year interval despite the reduction in accumulation rate. In addition, and as in other alternatives, the equivalent costs associated with the LST deficit at the inlet (29,000 m^3/yr in this case) are computed based on a 10 year dredging interval.

Overall this alternative is relatively expensive compared to Existing Practice (42% more) and others despite the reduction in the LST deficit and the deposition basin dredging because of the added cost of the bypassing plant ($1.1 M/yr). It should be noted, however, that reduced risks and damages downdrift associated with more continuous bypassing are not accounted for in these costs.

Table 8-6: Cost Summary for SI Alternative 4: AP + Semi-fixed Bypass System								
Plan Component	**Dredging Interval (years)**	**Quantity (1000x m^3 (cy))**	**Mob/ Demob ($000)**	**Unit Cost ($/$m^3$ ($/cy))**	**Subtotal Cost ($000)**	**E&D and S&A ($000)**	**Total Cost Per Operation ($000)**	**Annual Cost ($000)**
Channel & Deposition Basin Dredging	4	160 (210)	$1,000	$6.55 ($5.00)	$2,358	$399	$2,756	$838
Initial Construction					$3,764	$621	$4,385	$245
Annual Bypass System Cost	Cont.	100 (131)			$616	$112	$728	$768
Overhaul & Replacement								$52
LST Deficit	10	290 (379)	$1,000	$8.50 ($6.50)	$3,983	$655	$4,638	$656
							Grand Total	$2,559

<u>Summary of Pros and Cons</u>

Pros and cons associated with this alterative are summarized as follows:

Project Needs

Plant bypassing in combination with continued dredging of the deposition basin would mitigate local erosion of the west beach and partially offset the existing longshore sediment transport deficit. However, some accumulation of sand in the shoals and adjacent beaches would continue and downdrift erosion would not be fully mitigated unless there is also placement from offshore. Continued accumulation in the ebb shoal is consistent with experience at Indian River Inlet, where recent surveys confirm that the ebb shoal has continued to grow despite a fairly successful bypassing program.

Continued dredging of the deposition basin to -20 feet MLW would maintain navigation reliability through the inlet.

Risk & Uncertainty

As with Existing Practice and all other alternatives except the ones that include dredging the shoals, uncertainty with regards to the volume of material that actually accumulates in the shoals and to a lesser extent the adjacent beaches would still be problem. In other words, the LST deficit could in fact be higher than predicted. On the other hand, uncertainty with regards to the volume of sand that actually bypasses the inlet would be significantly reduced, since this estimate would not solely rely on indirect measurements as it does now.

Environmental Criteria

Although the annual sediment bypass deficit would not be eliminated and, technically, natural "sediment pathways" would not be restored (sediment would move via a pipeline from east to west), one aspect of the natural bypassing system, continuity, would be significantly improved as compared to Existing Practice and other alternatives. However, as explained above, accumulation in the inlet shoals is expected to continue.

As with most of the other alternatives, effects on water quality (flushing) would be insignificant because this alternative would not affect existing tidal prism (discharge) through the inlet.

Economic Criteria

This alternative would maintain safe access to the existing commercial fishing facilities and would reduce existing flooding risks by reducing the potential for a breach downdrift. It would not have any impacts on land use or ownership. Although the cost of providing for 100% bypassing across the inlet would be increased compared to Existing Practice and some of the other alternatives, flooding risks and damages would be arguably reduced because of the more continuous bypassing operation, although this effect is difficult to quantify.

Recreational Criteria

This alternative would maintain navigation reliability and thus it would not impact access to recreational fish resources. Specifically, increased bypassing would increase the beach area in downdrift of the inlet and within the Shinnecock Inlet County Park with sand that would be 100% compatible. Recreational resources such as swimming, sunbathing, and sightseeing, would be impacted at the updrift fillet due to bypass plant operations, presence of equipment (crane, pipes, pump) and some of the permanent construction that will be required (e.g., pump house). These impacts would extend to the downdrift beaches because of sand discharge operations, although the effect should be relatively small.

Engineering Criteria

Capacity would be a potential issue for this alternative. The actual bypassing rate for the plant at Indian River Inlet between 1990 and 2006 has been approximately 60,000 m^3/yr, and although lessons learned at this facility could be applied at Shinnecock Inlet and equipment improvements could be made, it is clear that capacity will be more of an issue for this alternative than others.

Source flexibility may also be a problem in that the area that can be accessed by the crane and jet pump is limited. On the other hand, placement flexibility should not be an issue since the system of booster pumps and pipe would allow for placement anywhere between the west jetty and approximately 7,000 feet west of the west jetty. One drawback of this alternative is the initial investment required, which would not be reversible. The principal engineering advantage of this alternative would be continuity, which would reduce shoreline fluctuations both at the west beach and farther downdrift.

Finally, this alternative provides the ability to implement a full range of alternatives at the end of the project life.

8.2.6 Alternative 5: Reduced Dimensions of Deposition Basin

The design capacity of the existing basin is approximately 350,000 m^3 and the anticipated dredging interval was 1.5 years, which would have resulted in dredging and bypassing of roughly 230,000 m^3/yr. However, since 1990, the deposition basin has been dredged only 3 times (1993, 1998, and 2004), and the actual dredging and bypassing rate has been 65,000 m^3/yr on roughly a 4 year cycle. A smaller deposition basin would have less storage capacity and thus reduce the dredging interval, thereby increasing continuity and potentially reducing risk downdrift. Note, however, that more frequent dredging would not be likely to increase the existing annualized dredging rate of 65,000 m^3/yr.

Every two feet of reduction in depth in the deposition basin translates to roughly 100,000 m^3 less in capacity. Thus a deposition basin at -18 feet MLW would have a 225,000 m^3 capacity, while one at -16 feet MLW would store roughly 125,000 m^3 (both estimates include 2 feet of allowable overdredge). These two options were considered as alternatives. Dredging frequencies for Alternative 5A (DB at -18 feet MLW) and Alternative 5B (DB at -16 feet MLW) would be 3 and 2 years respectively. A sketch of the conceptual design for this alternative is shown in Figure 8-11.

Note that other deposition reduction alternatives consisting of reductions in width, particularly along the east side of the channel were not considered because, as shown by experience at Moriches Inlet and even recently at Shinnecock when only 2/3 of the basin was dredged, shoaling on the east of the basin could occur fairly rapidly thus encroaching on the channel alignment.

Summary of Modeling Results

Modeling results (Figure 8-12) confirm that accumulation in the deposition basin would continue at roughly the same rate as *Existing (c. 2001)* and therefore, on average, the -18 MLW and -16 MLW deposition basins will require dredging every 3 and every 2 years, respectively.

Sediment Budget

A conceptual sediment budget for this alternative (regardless of depth in the deposition basin) is presented in Figure 8-13. As shown in this figure, it is assumed that although dredging in the deposition basin would take place more frequently as a result of the reduced basin capacity, the annualized sediment budget would remain roughly the same as existing conditions.

Costs

Costs associated with these two alternatives are summarized in Table 8-7 and Table 8-8. The increase in the dredging frequency increases the annualized cost of dredging the deposition basin by roughly $130,000 (7%) and $340,000 (19%) per year for Alternatives 5A and 5B, respectively. Note that the additional costs associated offsetting the LST deficit would remain the same ($767,000/yr).

Table 8-7: Cost Summary for SI Alternative 5A: -18 ft MLW Deposition Basin

Plan Component	Dredging Interval (years)	Quantity (1000x m^3 (cy))	Mob/ Demob ($000)	Unit Cost ($/$m^3$ ($/cy))	Subtotal Cost ($000)	E&D and S&A ($000)	Total Cost Per Operation ($000)	Annual Cost ($000)
Channel & Deposition Basin Dredging	3	195 (255)	$1,000	$6.55 ($5.00)	$2,616	$440	$3,056	$1,130
LST Deficit	10	400 (524)	$1,000	$8.50 ($6.50)	$5,067	$822	$5,889	$767
							Grand Total	$1,897

Table 8-8: Cost Summary for SI Alternative 5B: -16 ft MLW Deposition Basin

Plan Component	Dredging Interval (years)	Quantity (1000x m^3 (cy))	Mob/ Demob ($000)	Unit Cost ($/$m^3$ ($/cy))	Subtotal Cost ($000)	E&D and S&A ($000)	Total Cost Per Operation ($000)	Annual Cost ($000)
Channel & Deposition Basin Dredging	2	130 (170)	$1,000	$6.55 ($5.00)	$2,128	$362	$2,489	$1,341
LST Deficit	10	400 (524)	$1,000	$8.50 ($6.50)	$5,067	$822	$5,889	$767
							Grand Total	$2,108

<u>*Summary of Pros and Cons*</u>

Pros and cons associated with this alterative are summarized as follows:

Project Needs

Reduced deposition basin dimensions and more frequent dredging (2-3 years instead of 4) would potentially improve the average condition of the west beach by decreasing the interval between renourishments. However, it would not offset the existing longshore sediment transport deficit or improve longshore sediment transport processes downdrift of the inlet beyond existing conditions. So although dredging of the deposition basin to -16 feet MLW would maintain navigation reliability through the inlet, by itself, this alternative does not fully address all of the project needs.

Risk & Uncertainty

There would be some risk and uncertainty with regards to the actual dredging interval should the deposition basin be reduced to -16 or -18 feet MLW. However, should the dredging interval become too “short” (less than two years), the depth of the basin could be increased again. In other words, this plan is easily reversible.

Environmental Criteria

Hydrodynamic model simulations indicate that a shallower deposition basin will not have a significant effect on tidal prism or water quality (flushing). Dredging frequency would be increased, but since dredging would take place over the same area as before, no additional environmental impacts would occur.

Sediment pathways are not expected to be significantly affected by this alternative either, although theoretically a shallower basin may allow for some restoration of the natural east west transport.

Economic Criteria

This alternative (even with a deposition basin at -16 feet MLW) would maintain safe access to the existing commercial fishing facilities, so long as dredging takes place at the required frequency. It would also reduce risks on the west beach since nourishment would be more frequent. However, the costs of providing 100% bypassing across the inlet would increase as shown above.

Recreational Criteria

This alternative would maintain navigation reliability and thus it would not impact access to recreational fish resources. Water and foreshore related recreational resources such as surfing, boating, fishing, swimming, sunbathing, sightseeing, birdwatching, etc. would also be maintained. Public access to these resources will not be affected either. Permanent construction on the beach will not be required.

Engineering Criteria

As with other alternatives that are based on use of a floating dredge plant, capacity is an advantage for this alternative since the required dredging volume per operation is well within the capabilities of existing dredges. Continuity would be slightly improved over existing conditions (particularly for the -16 feet MLW alternative), but bypassing, particularly to the west beach, would not be continuous. Implementation of this alternative would be easy to change or reverse if unsuccessful.

8.2.7 Alternative 6: AP + Dredging the Flood Shoal

Similarly to Alternative 1, the premise behind this alternative is to offset the deficit in the *Existing (c. 2001)* sediment budget (40,000 m^3/yr) by dredging areas of the flood shoal. Dredging the flood shoal and placing the sediment beyond the area of direct influence of inlet processes would theoretically restore longshore sediment transport process. It would also serve to constrain or even reverse sediment accumulation in the flood shoal.

Sand could be removed from a relatively large area of compatible material along the south edge of the flood shoal. The amount of compatible material within this area is between 700,000 and 1,450,000 m^3, depending on the depth and area of the cut (OCTI, 1999; Militello and Kraus, 2001). It would only be necessary to dredge within a much smaller area to produce the amount necessary for each dredging operation. The optimum borrow site location could be selected prior to each dredging event based on condition surveys which would need to include coverage of the flood shoal. These surveys would be used to identify areas of shoal growth or recovery from previous dredging operations as candidates for additional dredging. A sketch of the conceptual design for this alternative is shown in Figure 8-14.

Dredging could be easily performed using a traditional floating plant (hydraulic or hopper dredge) on contract. Dredging could easily conform to a design bypass schedule and quantity, as opposed to dredging from the deposition basin, which, as already experienced, does not accumulate sediment at the same rate every year. More frequent dredging and placement would reduce shoreline fluctuations along West Beach and farther downdrift.

Flood shoal dredging would have to be supplemented with dredging of the authorized channel and deposition basin to provide adequate navigation reliability. Dredging of the flood shoal could take place on the same frequency as the channel and deposition basin (4 years on average) (Alternative 6A) or on a more frequent interval (e.g., every 2 years) but coinciding with dredging the channel and deposition basin every other operation to reduce costs (Alternative 6B).

Modeling Results

The Delft3D morphological model was used to simulate future sedimentation in the dredged areas and to assess potential impacts to other inlet cells. The simulation was performed under a conservative condition assuming in which over 1 million m^3 were removed from the area of compatible material of the flood shoal (Figure 8-15). Simulation results confirm gradual recovery of the dredged area and minimal impacts on other inlet cells. Specifically, detailed volumetric analysis indicates that changes in other cells (including the deposition basin, west beach, and downdrift beach) would be similar to *Existing (c. 2001)* conditions, which supports the conceptual sediment budget presented above.

Sediment Budget

A conceptual sediment budget for this alternative is presented in Figure 8-16. As shown in this figure, the LST deficit (40,000 m^3/yr) is made up by dredging the flood shoal and placing the material downdrift. Note that it is assumed that the dredging interval (2 or 4 years) would have a negligible effect on the average annual sediment budget. However, as discussed above, more frequent bypassing would reduce existing swings in shoreline position. It is assumed that over time dredging would offset ongoing accumulation in the flood shoal (13,000 m^3/yr) and possibly reverse historical accumulation at a rate of up to 27,000 m^3/yr. This "erosion" of the flood shoal would offset future accumulation in the Updrift Beach (6,000 m^3/yr), ebb shoal (18,000 m^3/yr), West Beach (1,000 m^3/yr) and Downdrift Beach (2,000 m^3/yr), which is assumed would continue. Thus, over a 50 year project life, as much as 1.35 million m^3 could be dredged from the flood shoal, which is roughly the amount of compatible sand identified in this area.

Sand dredged from the deposition basin could continue to be placed immediately west of the inlet or part of it can be placed farther downdrift, assuming that the sand dredged from the flood shoal is placed on the West Beach. The net effect on the sediment budget would be similar; however, more frequent dredging from the flood shoal may be better suited to offset erosion along the West Beach while a large percentage of the sand from the deposition basin could be placed downdrift, past the Ponquogue ebb shoal attachment. Regardless, pumping distance will be less than 2 miles and the costs associated with each option would be very similar.

Costs

First costs and annual costs developed for Alternative 6A (4 year cycle) and 6B (2 year cycle) and are summarized in Table 8-9 and Table 8-10, respectively. As in the case of ebb shoal dredging, increasing dredging frequency at the flood shoal (every 2 vs. every 4 years) increases the number of dredge mobilizations and demobilizations and thus the annual costs increase by about $322,000/yr (21%) as compared to dredging on a 4 year cycle. The cost of dredging the flood shoal on a two year cycle is also comparable to the cost of Existing Practice including dredging offshore every 10 years to offset the existing LST deficit.

Table 8-9: Cost Summary for SI Alternative 6A: AP + Flood Shoal Dredging (every 4 years)

Plan Component	Dredging Interval (years)	Quantity (1000x m^3 (cy))	Mob/ Demob ($000)	Unit Cost ($/$m^3$ ($/cy))	Subtotal Cost ($000)	E&D and S&A ($000)	Total Cost Per Operation ($000)	Annual Cost ($000)
Channel & Deposition Basin Dredging	4	260 (340)	$1,000	$6.55 ($5.00)	$3,105	$518	$3,623	$1,033
Flood Shoal Dredging	4	160 (210)	Same contract	$6.55 ($5.00)	$1,208	$212	$1,419	$405
							Grand Total	$1,438

Table 8-10: Cost Summary for SI Alternative 6B: AP + Flood Shoal Dredging (every 2 years)

Plan Component	Dredging Interval (years)	Quantity (1000x m^3 (cy))	Mob/ Demob ($000)	Unit Cost ($/$m^3$ ($/cy))	Subtotal Cost ($000)	E&D and S&A ($000)	Total Cost Per Operation ($000)	Annual Cost ($000)
Channel & Deposition Basin Dredging	4	260 (340)	Same contract	$6.55 ($5.00)	$1,955	$334	$2,289	$653
Flood Shoal Dredging	2	80 (105)	$1,000	$6.55 ($5.00)	$1,754	$301	$2,055	$1,107
							Grand Total	$1,760

Summary of Pros and Cons

Pros and cons associated with this alterative are summarized as follows:

Project Needs

Similar to Alternative 1, dredging the flood shoal would offset the existing longshore sediment transport deficit and restore (in terms of average volume per year) longshore sediment transport processes downdrift of the inlet. Continued dredging of the deposition basin could be used to mitigate local erosion of the west beach and would continue to provide navigation reliability through the inlet. Dredging the flood shoal would reduce encroachment of this morphological feature into the east and west cut channels, thereby improving navigation reliability and reducing maintenance requirements along those two inner waterways.

Risk & Uncertainty

Similarly to Alternative 1, this alternative would entail very little risk and uncertainty as compared to others since it involves continuation of Existing Practice under the Authorized Project dimensions and bypassing would be improved using proven dredging technology with relatively well known costs, schedules, performance, and environmental effects. However, uncertainty with regards to flood shoal growth rates, optimum dredging rates, and its effects on the sediment budget may be more difficult to manage than in the case of Alternative 1 because of the difficulties associated with surveying relatively shallow water in the back bay and the fact that the flood shoal extents are somewhat unclear compared to the ebb shoal.

Environmental Criteria

As shown by Militello and Kraus (2001) flood shoal dredging, even if performed outside of shallow habitat areas, may induce some hydrodynamic impacts that extend beyond the dredging footprint. Thus this alternative is considered slightly worse than others with regards to dredging impacts. In addition, as the volume of sand removed from the shoal accumulates over time, impact on hydrodynamics (increased tidal prism) may increase, although numerical model simulations suggest that these effects would be relatively small.

Although the annual sediment bypass deficit would be made up with sand from the flood shoal, accumulation in the adjacent beaches and ebb shoal would continue and, technically, the natural "sediment pathways" would not be fully restored. So as in the case of Alternative 1, this alternative would only be slightly better than existing conditions in that sense.

Economic Criteria

This alternative would maintain safe access to the existing commercial fishing facilities and would reduce existing flooding risks by reducing the potential for a breach downdrift. It would not have any impacts on land use or ownership. More importantly, it would reduce the cost of providing for 100% bypassing across the inlet compared to existing conditions, which it has been assumed would require periodic placement of sand from and offshore source to offset the existing net LST deficit.

Compared to other alternatives it is also relatively inexpensive because of the savings associated with using the same equipment to maintain the channel and deposition basin and dredge the ebb shoal.

Recreational Criteria

This alternative would maintain navigation reliability and thus it would not impact access to recreational fish resources. Water and foreshore related recreational resources such as surfing, boating, fishing, swimming, sunbathing, sightseeing, birdwatching, etc. would also be maintained or improved. Specifically, increased bypassing would increase the beach area in downdrift of the inlet and within the Shinnecock Inlet County Park. However, sand from the flood shoal will not be fully compatible with the beach sand, which might have some effects on the profile and the texture of the sand. Public access to this resources will not be affected either. Permanent construction on the beach will not be required.

Engineering Criteria

Similar to all the alternatives that rely on a floating plant for bypassing, capacity would not be an issue. Source flexibility, however, is not as good, as in the case of Alternative 1, since some of the easily accessible areas of the flood shoal have been identified as not suitable for beach fill.

A floating plant (most likely a cutterhead dredge in this case) would offer placement flexibility since it can be easily used to place material at various locations including the west beach and the downdrift beach past the Ponquogue attachment point.

Continuity, or the ability of the bypassing system to provide continuous bypassing in a manner similar to natural longshore drift, is perhaps the most significant disadvantage of this alternative. Periodic dredging and placement is not the same as natural longshore drift and it certainly not as "continuous" as a bypassing plant. Nonetheless, this problem may be partially mitigated by a more frequent dredging cycle (every 2 years) and the fact that continued dredging, regardless of wave climate and erosion downdrift, can be used to build a "buffer" that will offset subsequent erosion waves. This is not easy to do by means of only dredging the deposition basin because sand only accumulates in the basin after significant storms occur and thus the erosion has already taken place downdrift.

As explained above, floating dredge plants are considered highly reliable and available. Thus this alternative also scores high in terms of performance.

Finally, this alternative does not rely on significant investments or modifications that might be difficult to change or reverse if unsuccessful. It also provides the ability to implement a full range of alternatives at the end of the project life without dictating a future course of action that will be difficult to change.

8.2.8 Alternative 7: AP + Shortening the East Jetty

As explained in Section 3, this alternative includes the shortening of the east jetty approximately 600 feet to allow additional sediment transport into the inlet and deposition basin during periods of westerly transport. Theoretically, this plan may result in improved bypassing, as this additional volume of sediment would be either bypassed naturally or collected in the deposition basin and available for dredging and bypassing. A sketch of the conceptual design for this alternative is shown in Figure 8-17.

Modeling Results

The Delft3D morphological model was used to simulate the effects of shortening the east jetty on sediment transport and shoaling in the deposition basin. As shown in Figure 8-18, at the end of year 1 in the simulation, the area that has shoaled to less than -15 feet MSL (roughly -13.3 feet MLW) is significantly larger than under Existing Conditions (Figure 8-1). More importantly, almost double the amount of sand accumulates in the deposition basin (roughly 150,000 m^3 during the first year). Considering that Delft3D is not fully capable of simulating the shoreline retreat that that would occur along the updrift beaches, it is very likely that this volume of sand would be even larger, and therefore dredging for this alternative would most likely be required on an annual basis in order to keep the channel navigable.

Sediment Budget

A conceptual sediment budget for this alternative based on the *Existing (c.2001)* conditions sediment budget and the morphological modeling results described above is presented on Figure 8-19. As shown in this figure, it is assumed that shortening the east jetty would cause erosion on the Updrift Lobe and Updrift Beach, which could theoretically be sufficient to offset accumulation in the Flood Shoal, Downdrift Lobe, West Beach and Downdrift beach Cells (23,000 m^3/yr) and thus eliminate the existing LST deficit.

As shown above, sedimentation in the Deposition Basin would increase to 150,000 m^3/yr. Part of this volume would be placed on the West Beach at the current rate (65,000 m^3/yr) and the remainder (85,000 m^3/yr) could be bypassed directly to the Downdrift Beach past the Ponquogue attachment.

<u>Costs</u>

A summary of the costs for this alternative is presented in Table 8-11. As explained above, dredging frequency would likely increase to once per year as a result of jetty shortening. On the other hand, the net LST deficit for the inlet could be completely offset by an additional influx of sand from the updrift beaches as they erode in response to jetty shortening. Thus, it was assumed that this alternative would not incur the additional costs of offsetting an LST deficit through offshore dredging. Overall, this alternative would be $1.1 M/yr more the Existing Practice, mostly because of the additional dredging costs associated with annual dredge mobilizations and a larger dredging rate. Shortening of the east jetty represents an additional $115,000/yr in annualized costs.

Table 8-11: Cost Summary for SI Alternative 7: AP + Shortening the East Jetty

Plan Component	Dredging Interval (years)	Quantity (1000x m^3 (cy))	Mob/ Demob ($000)	Unit Cost ($/$m^3$ ($/cy))	Subtotal Cost ($000)	E&D and S&A ($000)	Total Cost Per Operation ($000)	Annual Cost ($000)
Channel & Deposition Basin Dredging	1	150 (196)	$1,000	$6.55 ($5.00)	$2,277	$386	$2,663	$2,799
East Jetty Shortening (600 ft)					$1,754	$302	$2,056	$115
							Grand Total	$2,914

<u>Summary of Pros and Cons</u>

Pros and cons associated with this alterative are summarized as follows:

Project Needs

Shortening the east jetty offsets the LST deficit and partially mitigates local erosion of the west beach through increased placement frequency. On the other hand, navigation through the inlet would be likely to deteriorate because of the increased influx of sediments from the east and a greatly reduced dredging cycle. Modeling results indicate that under large storm conditions channel depths could be rapidly reduced. The jetty could obviously be shortened a smaller distance to better balance navigation and dredging/bypassing needs. However, a similar result could be accomplished by reducing the depth of the deposition basin and increasing dredging frequency. Moreover, the latter would be easily reversible while the former would not.

Risk & Uncertainty

There is significant uncertainty regarding the effects of shortening the jetty on sediment transport and accumulation in the deposition basin. As explained above, the available models do not fully account for updrift shoreline retreat and thus can only be used to provide an estimate of the average accumulation. Conditions could be significantly worse than predicted, particularly during large storms.

Environmental Criteria

This alternative would "release" some of the material accumulated updrift of the inlet and thus partially restore some of the LST deficit caused by it. Sediment pathways, however, will not be fully restored, and in fact, as shown in the sediment budget, it is likely that more artificial vs. natural bypassing will be required (albeit more frequently than under existing conditions). As explained above, this alternative would also impact the beaches updrift. Water quality in the bay would not be affected.

Economic Criteria

As long as dredging is performed as needed, this alternative would maintain safe access to the existing commercial fishing facilities and would reduce existing flooding risks by reducing the potential for a breach downdrift. However, as explained above, there is significantly more uncertainty regarding the actual performance of this alternative as regards navigation compared to others. More importantly, costs to provide 100% bypassing across the inlet and navigation would increase significantly compared to existing conditions and most of the other alternatives considered herein.

Recreational Criteria

Beach-related recreational resources would be significantly impacted on the updrift beach due to shoreline retreat and potential loss of beach area. Navigation reliability could also be impacted and thus affect access to recreational fish resources. Surfing in the areas adjacent to the east jetty would also be impacted. Increased bypassing would increase the beach area downdrift of the inlet and within the Shinnecock Inlet County Park. Public access to these resources will not be affected.

Engineering Criteria

Capacity and flexibility of this alternative would be similar to existing conditions and other alternatives that mostly rely on the deposition basin to provide bypassing volumes and a floating plant to perform the dredging. Continuity, however, would be significantly improved by this alternative, given the increased dredging frequency. Shortening the east jetty would be a significant modification that would be difficult and costly to reverse if unsuccessful.

8.2.9 Alternative 8: AP + West Jetty Spur

The premise of this alternative is that a west jetty spur would increase stability of the West Beach by blocking waves from SE and SSE. In theory, the spur could reduce the amount of sediment that is lost to the channel and deposition basin during periods of south waves. Increased stability along the West Beach would allow for a larger share of the sand excavated from the deposition basin to be placed farther downdrift. A sketch of the conceptual design for this alternative is shown in Figure 8-20.

Modeling Results

The spur configuration proposed W.F. Baird & Associates (1999), which consists of a 135 meter (440 feet) long rubble mound structure extending from the seaward tip of the existing jetty and oriented 132 degrees counterclockwise, was tested with the morphological model. Results are presented in Figure 8-21. As shown in this figure, the expected effect of the spur on the west beach is the accumulation of sand in the lee of the spur, resulting in a more stable beach in this area. However, Delft3D as explained in Section 4, cannot simulate shoreline movement and thus it is not clear whether or not the stabilizing effects caused by the spur would extend sufficiently far west to "anchor" this shoreline in place and thereby significantly reduce future erosion.

More importantly, the model shows that accumulation in the deposition basin would be reduced as compared to Existing Conditions. However, this reduction does not entirely owe to a smaller influx of sediments from the west beach. Some of the material (approximately 10,000 m^3/yr) previously deposited in the deposition basin appears to be carried farther offshore and deposited on the seaward edge of the downdrift ebb shoal lobe. Model results suggest that the slightly increased training of the ebb jet as a result of spur construction is the cause of this change.

Sediment Budget

A conceptual sediment budget for this alternative based on the *Existing (c. 2001)* conditions sediment budget and the morphological modeling results described above is presented in Figure 8-22. Significant changes compared to existing conditions include the assumption that less fill will be required on the West Beach on annual basis (35,000 m^3/yr vs. 65,000 m^3/yr), less sand will accumulate in the Deposition Basin (55,000 m^3/yr vs. 65,000 m^3/yr), and the more sand will accumulate in the Downdrift Lobe (17,000 m^3/yr vs. 7,000 m^3/yr).

The overall effect of these changes on the net LST deficit for the inlet will be an increase in this deficit from 40,000 m^3/yr to 50,000 m^3/yr.

Costs

Because of the similarity in the construction material requirements, unit costs associated with the recent Coney Island project were used to estimate the cost of the west jetty spur construction. Quantities are based upon the work done by Baird. A summary of the costs associated with this alternative is presented in Table 8-12. Details are presented in Appendix E.

Note the cost increase associated with the increased LST deficit ($146,000/yr) and the cost associated with construction of the spur ($343,000/yr). The grand total for this alternative is $391,000/yr more expensive than existing conditions.

Table 8-12: Cost Summary for SI Alternative 8: AP with + West Jetty Spur

Plan Component	Dredging Interval (years)	Quantity (1000x m^3 (cy))	Mob/ Demob ($000)	Unit Cost ($/$m^3$ ($/cy))	Subtotal Cost ($000)	E&D and S&A ($000)	Total Cost Per Operation ($000)	Annual Cost ($000)
Channel & Deposition Basin Dredging	4	220 (288)	$1,000	$6.55 ($5.00)	$2,806	$470	$3,276	$935
LST Deficit	10	500 (654)	$1,000	$8.50 ($6.50)	$6,039	$970	$7,009	$913
Spur Construction					$5,291	$856	$6,147	$343
							Grand Total	$2,191

Summary of Pros and Cons

Pros and cons associated with this alterative are summarized as follows:

Project Needs

As explained above, this alternative would potentially increase the existing LST deficit, even if it mitigates local erosion of the west beach. Navigation conditions, despite a small increase in current velocity induced by the slow training effect of the spur, should remain similar to existing conditions.

According to Baird, one added benefit also suggested in the study is the reduction in scour potential at the new tip of the jetty, which would be located in deeper water, and presumably subject to lower tidal current velocities. Modeling results suggest scour at the tip the new spur, but to a lesser extent than experienced at the existing jetties. However, scour at the tip of the existing jetty does not appear to be reduced significantly either.

Risk & Uncertainty

There is significant uncertainty regarding the effects of spur on the west beach. Modeling results suggest a clear improvement in the lee of the structure, but it is unclear whether or not that effect would extend farther west and be sufficient to significantly reduce renourishment needs.

Environmental Criteria

The flow training effect of the spur would likely increase accumulation in the ebb shoal and thus further affect natural sediment pathways. Water quality in the bay would not be affected.

Economic Criteria

This alternative would maintain safe access to the existing commercial fishing facilities and would potentially reduce existing flooding risks by reducing the potential for a breach on the West Beach. However, as explained above, there is significantly more uncertainty regarding the actual performance of this alternative regarding the west beach as compared to others. Costs to provide 100% bypassing across the inlet and navigation would be relatively high because of the cost of the spur and the additional costs associated with the increased LST deficit.

Recreational Criteria

Beach-related recreational resources could improve along the west beach due to the stabilizing effects of the spur. Access to recreational fish resources should remain the same. Surfing in the areas adjacent to the west jetty and spur could be impacted. Public access to these resources will not be affected. However, unless the increased LST deficit is made up through offshore dredging, the beach area downdrift of the Ponquogue attachment point could be reduced.

Engineering Criteria

Capacity, flexibility, and continuity of this alternative would be similar to existing conditions and other alternatives that mostly rely on the deposition basin to provide bypassing volumes and a floating plant to perform the dredging on a 4 year cycle. However, constructing the spur will be a significant modification that would be difficult and costly to reverse if unsuccessful.

8.2.10 Annual Costs Summary and Recommended Alternative

Table 8-13 summarizes the costs for each shortlisted alternative. According to this table, the least expensive alternatives are those that maintain the Authorized Project dimensions (AP) and offset the existing LST (40,000 m^3/yr or 52,000 cy/yr) deficit by dredging the ebb shoal or the flood shoal on a 4 year cycle. In fact, dredging the inlet shoals appears to be the only effective and reliable way to completely eliminate this deficit. Other alternatives do not achieve a 100% reduction (i.e., semi-fixed bypassing plant) or include too much uncertainty on this issue (i.e., shortening the east jetty). Dredging the inlet shoals also provides the ability to implement a full range of alternatives at the end of the project life without dictating a future course of action that will be difficult to change.

Table 8-13: Summary of Annual Cost for Shinnecock Inlet Alternatives	
Plan	**Annual Cost ($000)**
SI Existing Practice with AP	*$1,800*
Alt 1A. AP + Ebb Shoal Dredging every 4 years	$1,438
Alt 1B. AP + Ebb Shoal Dredging every 2 years	$1,760
Alt 2. AP + Nearshore Structures (T-groins)	$2,993
Alt 3. AP + Offshore Dredging for West Beach	$2,280
Alt 4. AP + Semi-fixed Bypass System	$2,559
Alt 5A. -18 ft MLW Deposition Basin	$1,897
Alt 5B. -16 ft MLW Deposition Basin	$2,108
Alt 6A. AP + Flood Shoal Dredging every 4 years	$1,438
Alt 6B. AP + Flood Shoal Dredging every 2 years	$1,760
Alt 7. AP + Shortening the East Jetty	$2,799
Alt 8. AP with + West Jetty Spur	$2,191

Overall, dredging the shoals outside the limits of the channel and deposition basin would entail very little risk and uncertainty as compared to others since it involves continuation of existing practice under the authorized project dimensions and bypassing would be improved using proven dredging technology with relatively well known costs, schedules, performance, and environmental effects. Uncertainty regarding accurate estimates of ebb shoal growth and its effects on the sediment budget and long shore sediment transport processes could be managed through regular monitoring surveys of the ebb shoal and dredging in areas of observed growth.

Potential impacts on nearshore waves and littoral processes, which modeling results suggest are insignificant, can be also be minimized by monitoring future morphological changes and managing the dredging program accordingly.

The Authorized Project combined with dredging the inlet shoals also offers the advantage of being easily reversible, particularly in the case of the ebb shoal. Morphological modeling simulations suggest that the shoals would recover over time, and neither alternative requires a new capital improvement or significant upfront costs. Of the two shoals, dredging the ebb shoal is the preferred option because it reduces uncertainty and potential environmental impacts. Dredging the ebb shoal would offset the existing longshore sediment transport deficit and restore (in terms of average volume per year) longshore sediment transport processes downdrift of the inlet. Continued dredging of the deposition basin would be used to mitigate local erosion of the west beach. Depending on future performance, which would be assessed by regular monitoring surveys, part of the sediment from the deposition basin could be placed farther downdrift beyond the ebb shoal attachment point. Conversely, ebb shoal material could be occasionally placed on the west beach if necessary. Continued dredging of the deposition basin to -20 feet MLW would maintain navigation reliability through the inlet.

One potential disadvantage of dredging the shoals is lack of bypassing continuity, particularly on a 4 year cycle. However, a 2-year cycle could be combined with a shallower deposition basin (at -16 feet MLW) to provide for cost effective solution that would improve continuity and eliminate the LST deficit across the inlet. Only shortening the east jetty (dredging on 1 year cycle) or a bypassing plant could provide for more continued bypassing. However, both would be more expensive, less reliable, and not easily reversible. A two-year dredging cycle would also be much closer to the 1.5-year cycle originally anticipated in the current project authorization. This trade-off between more continues bypassing and slightly increase average annual costs could be managed and modified, if necessary, in the future depending on actual performance and costs.

Costs for this recommended alternative combining dredging of the ebb shoal and a shallower deposition basin are presented in Table 8-14. Note that dredging both the deposition basin and the ebb shoal at the same frequency (i.e., one mobilization) and eliminating the costs of the LST deficit brings the cost of this alternative below that of existing conditions, despite doubling the dredging frequency. Other potentially negative issues associated with the other alternatives aside from the increased annual costs are summarized below.

Table 8-14: Costs for SI Recommended Alternative: -16 ft MLW DB + Ebb Shoal Dredging

Plan Component	Dredging Interval (years)	Quantity (1000x m^3 (cy))	Mob/ Demob ($000)	Unit Cost ($/$m^3$ ($/cy))	Subtotal Cost ($000)	E&D and S&A ($000)	Total Cost Per Operation ($000)	Annual Cost ($000)
Channel & Deposition Basin Dredging	2	130 (170)	$1,000	$6.55 ($5.00)	$2,128	$362	$2,489	$1,341
Ebb Shoal Dredging	2	80 (105)	Same Contact	$6.55 ($5.00)	$604	$110	$714	$384
							Grand Total	$1,726

Dredging the flood shoal (Alt. 6) would be very similar in terms of meeting the stated goals to the selected alternative; however, it does have increased uncertainty with regards to morphodynamics, optimum dredging rates and its effects on the sediment budget may be more difficult to understand and manage than in the case of dredging the ebb shoal. In addition, modeling results show that flood shoal dredging, if significant in extent and depth, may induce some hydrodynamic impacts that extend beyond the dredging footprint potentially affecting navigation and increasing the tidal prism through the inlet (i.e., the potential for increased flood elevations exists). There is also more uncertainty regarding sediment compatibility. Typically, ebb shoal sediments are very compatible with the beach material, whereas the flood shoal sands tend to be finer. Finally, flood shoal dredging would have to be performed closer to environmentally sensitive areas.

Offshore dredging (Alt. 3) combination with continued dredging of the deposition basin would mitigate local erosion of the west beach, offset the existing longshore sediment transport deficit but accumulation of sand in the shoals and adjacent beaches would continue. Therefore, unlike Alternative 1 (ebb shoal dredging), this alternative does not “balance” the sediment budget by reducing accumulation within the inlet.

A semi-fixed bypassing plan (Alt. 4) in combination with continued dredging of the deposition basin would mitigate local erosion of the west beach and partially offset the existing longshore sediment transport deficit. However, some accumulation of sand in the shoals and adjacent beaches would continue and downdrift erosion would not be fully mitigated unless there is also placement from offshore. Continued accumulation in the ebb shoal is consistent with experience at Indian River Inlet, where recent surveys suggest that the ebb shoal has continued to grow despite continuous bypassing.

Capacity would be a potential issue for this alternative. The actual bypassing rate for the plant at Indian River Inlet between 1990 and 2006 has been somewhat lower than anticipated (approximately 60,000 m^3/yr), and although lessons learned at this facility could be applied at Shinnecock Inlet and equipment improvements could be made, it is clear that capacity will be more of an issue for this alternative than for dredging alone. Source flexibility may also be a problem in that the area that can accessed by the crane and jet pump is limited. Finally, the initial investment required would not be recoverable.

Shortening the east jetty (Alt. 7) offsets the LST deficit and partially mitigates local erosion of the west beach through increased dredging and placement frequency. On the other hand, navigation through the inlet would be likely to deteriorate because of the increased influx of sediments from the east. Modeling results indicate that under large storm conditions channel depths could be reduced rapidly. The jetty could obviously be shortened a smaller distance to better balance navigation and dredging/bypassing needs. However, a similar result could be accomplished by reducing the depth of the deposition basin and increasing dredging frequency. Moreover, the latter would be easily reversible while the former would not. There is also a significant amount of uncertainty regarding the actual effect that shortening the jetty would have on shoaling and navigation conditions within the channel and deposition basin.

A spur of the west jetty (Alt. 8) would completely stabilize the west beach, sand placement in this area is likely to be required in the future. More importantly, modeling results show that accumulation in the deposition basin would be reduced as compared to existing conditions. Some of the material (approximately 10,000 m^3/yr) previously deposited in the deposition basin appears to be carried farther offshore and deposited on the seaward edge of the downdrift ebb shoal lobe. Model results suggest that the slightly increased training of the ebb jet as a result of spur construction is the cause of this change. Finally, this alternative is worse than other with regards to environmental impacts because it requires a structure.

Constructing the T-groins (Alt. 2) would essentially eliminate the chronic erosion problem along the west beach and it would free up the most of the sand now being placed there to be directly bypassed to the beaches downdrift of the inlet. However, it is uncertain what their net effect would be on the sediment budget and whether or not the existing longshore sediment transport deficit would be reduced. More importantly, like Alt. 8 (spur), the T-groins are considered to have a significantly greater environmental impact.

8.3 Moriches Inlet

Based on the result from the preliminary screening, the following alternatives were selected for further consideration and detailed analysis at Shinnecock Inlet:

Alt. 1: Authorized Project (AP)
Alt. 2: AP + Dredging the Ebb Shoal
Alt. 3: AP + Semi-fixed Bypass System
Alt. 4: AP + Dredging the flood shoal

8.3.1 Summary of Existing Conditions

Previous sediment budget estimates (Gravens et al., 1999), principally based on shoreline changes from 1979 to 1995, suggest a long-shore sediment transport rate of approximately 184,000 m^3/yr (240,000 cy/yr) entering the inlet from the east under *Existing* (*c. 1999*) conditions. Analysis of recent surveys (see Section 5.2) suggests that this value might be significantly higher from 1995 to 2001, 238,000 m^3/yr (312,000 cy/yr). This increase is at least partly due to recent fill placement east of the inlet as part of the Westhampton Interim Project (666,000 m^3/yr or 873,000 cy/yr between 1995 and 2001). The volume of sediment placed in future years along the Westhampton shoreline should be reduced to maintenance levels (approximately 250,000 m^3/yr or 328,000 cy/yr), which would result in approximately 167,000 m^3/yr (219,000 cy/yr) of net westerly transport at the inlet (see Section 5.2).

The *Existing* (c. 2001) condition sediment budget for Moriches Inlet suggests a net westerly transport rate of 267,000 m^3/yr entering the updrift (east) beach. The budget also suggests that approximately 79% (211,000 m^3/yr) of the net updrift westerly transport bypasses the inlet system, including the channels, shoals, and adjacent beaches. The remaining 21% (56,000 m^3/yr) accumulates within the inlet shoals and adjacent beaches. The budget includes dredging of the deposition basin at a rate of 56,000 m^3/yr. Typically, material dredged from the deposition basin has been placed on the west beach relatively close to the west jetty.

Note that the recent net longshore sediment transport rate is significantly higher than at Shinnecock Inlet. This may explain why, aside from the differences in inlet geometry and deposition basin dimensions, the channel and deposition basin at Moriches Inlet shoals noticeably faster than at Shinnecock Inlet.

Existing inlet management practice consists of infrequent (at least relative to the existing navigation need and Authorized project) channel and deposition basin dredging (Figure 2-3). Sediment dredged from the inlet in recent years has been placed within 2,000 feet of the west jetty, that is, still within the area fronted by the ebb shoal. Similarly to Shinnecock Inlet, it is unlikely that this material is subsequently transported by waves farther west past the ebb shoal attachment area. Nonetheless, and although a small amount of sand accumulates immediately updrift of the inlet, the inlet appears to effectively naturally bypass the balance of the net longshore transport along a pathway that follows a very shallow bypassing bar extending right across the navigation channel in a NE to SW alignment and a large, slightly deeper ebb shoal platform that connects this bar with the shoreline downdrift (see Section 5.2). Dredging events "cut" the NE to SW sand bar in two pieces; the western piece appears to be rapidly moved west and onto the downdrift shoreline, while the eastern bar remnant grows rapidly encroaching the channel again within a few months. Specifically, recent condition surveys (Appendix A) suggest that on July 2000, which was less than two years after the dredging event in October 1998, the leading edge of the shallow ebb shoal feature was encroaching into the channel. By April 2001, this feature had grown across the channel, and navigation conditions had been significantly deteriorated. Between 1996 and 1998, shoaling occurred more rapidly due to a more active wave climate during that period (see Section 5), channel depths were reduced from more than -20 feet MLW in certain areas (apparently the deposition basin was significantly overdredged in 1996) to less than -10 feet MLW in less than two years.

Fairly efficient bypassing is also suggested by ebb shoal volumetric changes computed as part of this study (see Section 5.2) as well as a recent analysis by Allen et al. (2002) which hint that the ebb shoal is a relatively stable feature.

Navigation, however, is extremely dangerous through the inlet. In 1998, Moriches Inlet was dredged to -14 feet MLW at the same time as Shinnecock Inlet. Unlike Shinnecock Inlet, however, the deposition basin and channel at Moriches Inlet shoaled at a very rapid rate. This may be due to several factors, including the smaller size of the deposition basin and perhaps a greater influx of sediment into Moriches Inlet.

Similarly less than 8 months after the last dredging event in February 2004, available controlling depth reports that the shoal had formed again and dredging was required (3.7 feet minimum depth). The previous time Moriches Inlet was dredged (1998), shoaling was minimal for 2 years and then occurred rapidly between summer 2000 and spring 2001.

8.3.2 Alternative 1: Authorized Project (AP)

The premise of this alternative is simply implementation of the Authorized Project. According to the navigation analysis presented in the previous section, authorized channel dimensions (200 feet wide at -10 feet MLW) are adequate for navigation under normal wave conditions at the inlet. In addition, the authorized deposition basin (300 feet wide at -14 feet MLW) would provide for reliable navigation in all but the very highest wave conditions. Therefore, if dredging took place as needed to maintain these authorized project dimensions, navigation through the inlet would not be a problem most of the time.

However, available survey data suggest that the channel and deposition basin need to be dredged more frequently than in recent years in order to maintain the authorized dimensions. In fact, the selected plan described in the Moriches Inlet General Design Memorandum (USACE-NAN, 1982) calls for a "seasonal" maintenance schedule which amounts to annual dredging (75,000 m^3/yr or 98,000 cy/yr) and the assumption that the channel and deposition basin will shoal to a depth of less than -10 feet MLW approximately 7 months after dredging operations. The GDM suggests that dredging take place in the spring, so that depths of less than -10 feet MLW would only occur during the winter months when traffic through the inlet is minimal. Bathymetric changes observed after the 1996 and 1998 dredging events seem to confirm the expected shoaling rates as described in the GDM.

Although this dredging schedule seems to meet the average dredging requirements, waves and longshore transport do not match the average every year, and it is anticipated that some years shoaling may occur at a rate higher than the average during spring and summer, which may result in significant impacts to navigation. On the other hand, a continuous shallow sand bar across the channel during the winter months would allow for improved natural bypassing. Other years transport may be relatively small and not enough material will be available within the deposition basin to justify the high mobilization costs of a dredge. Material dredged from the channel and deposition basin would be placed westward of the ebb shoal downdrift attachment area (i.e., at least 8,000 feet from the west jetty). Dredging can easily be performed using a traditional floating plant (hydraulic or hopper dredge) on contract. A dredging interval of 1 year is conservatively assumed in order to ensure maintenance of navigation conditions. A conceptual sketch of this alternative is shown in Figure 8-23.

This alternative also includes rehabilitation of the outer end of the west jetty, as authorized. The 1982 GDM called for repair of the east and west jetties at Moriches Inlet. Subsequent construction, however, did not include this feature. Presently, the west jetty is in significant disrepair. Other features of this alternative, also included in the authorized plan, are maintenance of both jetties and the bayside revetment at Pikes Beach (constructed after the breach in January 1980) and maintenance of the inner bay channel to a depth of -6 feet MLW and a width of 100 feet. All alternatives considered for Moriches Inlet also include these authorized features.

Modeling Results

The Delft3D morphological model was used to simulate future sedimentation in the dredged areas (deposition basin and authorized channel) and to assess potential impacts to adjacent inlet cells. Initial bathymetric conditions (Figure 8-24) used in the morphological model represent post-dredge conditions where the NE to SW sand bar is cut into two pieces. Simulation results indicate that the western piece of the sand bar appears to be rapidly moved west and onto the downdrift shoreline, while the eastern bar remnant grows rapidly, encroaching the channel again within 1 to 2 years. The model also shows that in less than 4 years, if dredging does not occur, the sand bar will recover to the pre-dredge condition with depths of less than -7 feet MLW.

Sediment Budget

A conceptual sediment budget for this alternative is presented in Figure 8-25. As shown in this figure, the LST deficit is the same as under *Existing (c. 2001)* conditions (56,000 m^3/yr). In this alternative it is assumed that sand dredged from the Deposition Basin would be placed further down drift but if needed, the sand could be used to periodically nourish the West Beach if monitoring surveys indicate erosion.

The distance required for either option is less than 2 miles and thus the costs associated with either placement would be very similar.

The volume dredged from the Deposition Basin is expected to increase when dredged on a one year dredging cycle (75,000 m^3/yr vs. 56,000 m^3/yr with Existing Practice).

<u>*Costs*</u>

First costs and annual costs were developed for Alternative 1 and are presented in Table 8-15. Costs for Existing Practice are shown in Table 8-16 for comparison. Note that Existing Practice costs take into account the cost of the existing bypass deficit (56,000 m^3/yr) by assuming this deficit would have to be eventually made up by importing sand from an offshore source. This is just a way to assign a cost to the existing bypassing deficit and it does not mean that offshore dredging is currently part of the Existing Practice at Moriches Inlet. Also note that both cost estimates do not account for damages related to a more or less eroded shoreline condition downdrift. The bypassing deficit estimated under this alternative is accounted for in the same fashion.

Table 8-15: Cost Summary for MI Alternative 1: Authorized Project (AP)

Plan Component	Dredging Interval (years)	Quantity (1000x m^3 (cy))	Mob/ Demob ($000)	Unit Cost ($/$m^3$ ($/cy))	Subtotal Cost ($000)	E&D and S&A ($000)	Total Cost Per Operation ($000)	Annual Cost ($000)
Channel & Deposition Basin Dredging	1	75/ (98)	1,000	$7.45/ ($5.70)	$1,792	$308	$2,100	$2,208
LST Deficit	10	560/ (732)	1,000	$8.54/ ($7.00)	$7,043	$1,121	$8,164	$1,064
							Grand Total	$3,272

Increasing the dredging frequency of the deposition basin and navigation channel (every 1 year vs. every 4 years) increases the number of dredge mobilizations and demobilizations and thus the annual costs increase by about $1,186,000 (57%) as compared to Existing Practice.

Table 8-16: Cost Summary Existing Practice (EP) at Moriches Inlet

Plan Component	Dredging Interval (years)	Quantity (1000x m^3 (cy))	Mob/ Demob ($000)	Unit Cost ($/$m^3$ ($/cy))	Subtotal Cost ($000)	E&D and S&A ($000)	Total Cost Per Operation ($000)	Annual Cost ($000)
Channel & Deposition Basin Dredging	4	224/ (293)	$1,000	$7.45 ($5.70)	$3,071	$512	$3,583	$1,022
LST Deficit	10	560/ (732)	$1,000	$9.20 ($7.00)	$7,043	$1,121	$8,164	$1,064
							Grand Total	$2,086

8.3.3 Alternative 2: AP + Ebb Shoal Dredging

One alternative to facilitate sediment bypassing and offset the LST deficit is to dredge from the ebb shoal and place the material downdrift of the ebb shoal attachment. Areas such as the updrift lobe or around the seaward slope of the shoal are candidates for dredging (Figure 8-26). Dredging from these relatively large areas will provide a guaranteed source of sediment that conforms to a design bypass schedule and quantity, as opposed to dredging from a deposition basin which, as already experienced, does not accumulate sediment at the same rate every year. On the other hand, this alternative may have some adverse effects on ebb shoal morphology, natural bypassing, and adjacent shorelines if the amount of material dredged from the ebb shoal is too large.

Similarly to Shinnecock Inlet, the potential borrow area is located seaward of the -20 foot NGVD contour to minimize impacts on nearshore wave climate conditions and existing sediment transport processes. Dredging could be performed to the -35 foot contour (NGVD). The optimum borrow site location could be selected prior to each dredging event based on condition surveys which would need to include coverage of the ebb shoal. These surveys would be used to identify areas of ebb shoal growth or recovery from previous dredging operations as candidates for additional dredging.

Ebb shoal dredging would be supplemented with dredging of the deposition basin and channel (i.e., Authorized Project). The dredging contract could be structured to include ebb shoal dredging on a regular cycle but perhaps not as frequently as the main channel and deposition basin (e.g., every 2 years instead of every year).

As with all Moriches Inlet alternatives, this alternative also includes rehabilitation of the outer end of the west jetty, maintenance of both jetties and the bayside revetment at Pikes Beach, and maintenance of the inner bay channel to a depth of -6 feet MLW and a width of 100 feet, as authorized.

Modeling Results

The Delft3D morphological model was used to simulate future sedimentation in the dredged areas and to assess potential impacts to adjacent inlet cells. Results for a simulation considering the conservative condition of dredging a large area of the ebb shoal are presented in Figure 8-27. Model results indicate that the dredged area would recover over time. More importantly, a detailed volumetric analysis indicates that changes in other cells (including the Deposition Basin, West Beach, and Downdrift Beach) would be similar to Existing Conditions. Additional model results also indicate that dredging a smaller area of the ebb shoal more frequently will lead to a faster recovery of the dredged area and negligible impacts on the adjacent inlet cells.

Sediment Budget

A conceptual sediment budget for this alternative is presented in Figure 8-28. As is shown in this figure, the LST deficit (56,000 m^3yr) is made up by dredging the ebb shoal and placing the material downdrift. The ebb shoal at Moriches Inlet contains both a shallow bypassing bar that extends across the navigation channel and ebb shoal platform that connects this bar to the shoreline. Dredging activity could occur over both pieces of the ebb shoal. It is assumed that over time, dredging would offset ongoing ebb shoal accumulation (11,000 m^3/yr). This "erosion" of the ebb shoal would offset future accumulation in the Updrift Beach (29,000 m^3/yr), Flood Shoal (14,000 m^3/yr), and Downdrift Beach (2,000 m^3/yr). The total volume dredged from the ebb shoal over the project life would be 2.25 million m^3.

In this alternative it is assumed that sand dredged from the deposition basin or ebb shoal would be placed farther downdrift, but if needed, the sand could be used to periodically nourish the west beach if monitoring surveys indicate erosion.

Costs

First costs and annual costs developed for Alternative 2 are summarized in Table 8-17. The cost estimates assume that both navigation and ebb shoal dredging occur on a 1 year cycle. This alternative is slightly less expensive (14%) than maintaining the Authorized Project (Alternative 1) but more expensive than Existing Practice (34%). It should be noted, however, that reduced risks and damages downdrift associated with more continuous bypassing are not accounted for in these costs.

Table 8-17: Cost Summary for MI Alternative 2: AP + ES Dredging

Plan Component	Dredging Interval (years)	Quantity (1000x m^3 (cy))	Mob/ Demob ($000)	Unit Cost ($/$m^3$ ($/cy))	Subtotal Cost ($000)	E&D and S&A ($000)	Total Cost Per Operation ($000)	Annual Cost ($000)
Channel & Deposition Basin Dredging	1	75/ (98)	$1,000	$7.45/ ($5.70)	$1,792	$308	$2,100	$2,208
Ebb Shoal Dredging	1	56/ (73)	Same contract	$7.45/ ($5.70)	$479	$88	$566	$595
							Grand Total	$2,803

8.3.4 Alternative 3: AP + Semi-fixed Bypass System

A semi-fixed bypass system similar to that proposed at Shinnecock Inlet and consisting of a jet eductor pump mounted on crawler crane could be implemented at Moriches Inlet. System elements and operation would be very similar (crane, jet eductor pump, booster pumps, and discharge pipeline). With this system, sand would be bypassed with the help of a second booster pump downdrift past the ebb shoal attachment point. A conceptual sketch of this alternative is shown in (Figure 8-29).

There are some key differences that might make this system less attractive at Moriches Inlet. The net westerly transport at Moriches Inlet in recent years is significantly higher than the design capacity for typical semi-fixed bypassing system (at least 50% higher). To compensate, the system could be operated in combination with channel and deposition basin maintenance operations.

Sediment Budget

A conceptual sediment budget for this alternative is presented in Figure 8-30. As shown in this figure, it is assumed that approximately 100,000 m^3/yr would be directly bypassed from the updrift fillet to the Downdrift Beach, west of the ebb shoal attachment point. Theoretically, this would reduce the volume of sediment entering the Deposition Basin and possibly the rate of accumulation in the basin (albeit by a small amount). It is further assumed that future accumulation on the updrift fillet would be reduced to zero from 4,000 m^3/yr and on the Updrift Beach to 15,000 m^3/yr from 29,000 m^3/yr. Sand dredged from the Deposition Basin (reduced to 55,000 m^3/yr) would continue to be placed downdrift of the ebb shoal attachment point to supplement plant bypassing (for a total of 155,000 m^3/yr). Accumulation in the Flood Shoal, Downdrift Lobe, and Downdrift Beach is assumed to continue at the *Existing (c. 2001)* rate. Therefore, the net effect of the bypassing plant on overall bypassing efficiency would be a reduction in the LST deficit from 56,000 m^3/yr to 38,000 m^3/yr.

Costs

Overall costs for this alternative are summarized in Table 8-18. Initial construction, annual, and overhaul and replacement costs for the semi-fixed bypassing plants are based on work by Williams et al (1998). All costs are scaled from 1997 to 2005 price levels. The design detailed by Williams is assumed to be applicable for the purposes of cost estimation. Additional details are provided in Appendix E. Note that costs assume that dredging of the deposition basin would continue on a 1 year interval despite the reduction in accumulation rate. In addition, as in other alternatives, the equivalent costs associated with the LST deficit at the inlet (38,000 m^3/yr in this case) are computed based on a 10 year dredging interval.

Overall this alternative is almost twice as expensive as Existing Practice and is more expensive than other alternatives despite the reduction in the LST deficit and the deposition basin dredging because of the added cost of the bypassing plant. It should be noted, however, that reduced risks and damages downdrift associated with more continuous bypassing are not accounted for in these costs.

Table 8-18: Cost Summary for MI Alternative 3: AP + Semi-fixed Bypass System

Plan Component	Dredging Interval (years)	Quantity (1000x m^3 (cy))	Mob/ Demob ($000)	Unit Cost ($/$m^3$ ($/cy))	Subtotal Cost ($000)	E&D and S&A ($000)	Total Cost Per Operation ($000)	Annual Cost ($000)
Channel & Deposition Basin Dredging	1	55/ (72)	$1,000	$7.45/ ($5.70)	$1,622	$280	$1,902	$1,999
Initial Construction					$3,764	$621	$4,385	$245
Annual System Operation	Cont.	100 (131)			$616	$112	$728	$768
Overhaul and Replacement								$52
LST Deficit	10	380/ (497)	$1,000	$8.54/ ($7.00)	$5,151	$835	$5,986	$780
							Grand Total	$3,844

8.3.5 *Alternative 4: AP + Dredging the Flood Shoal*

Similarly to Alternative 2, the premise behind this alternative is to offset the LST deficit in the *Existing* (*c. 2001*) sediment budget (56,000 m^3/yr) by dredging areas of the flood shoal. Dredging the flood shoal and placing the sediment beyond the area of direct influence of inlet processes would theoretically restore longshore sediment transport process. It would also serve to constrain or even reverse sediment accumulation in the flood shoal.

Sand could be removed from a relatively large area of compatible material along the southeastern edge of the flood shoal (Figure 8-31). Periodic dredging of the southeastern edge of the shoal will prevent encroachment of this feature onto the inner navigation channel, which is currently much shallower than the authorized -6 feet MLW depth along its authorized alignment. Note that this area of the flood shoal was dredged once as a source of sand for the closure of the breach that opened adjacent to the east jetty in January 1980. Approximately 460,000 m^3 (600,000 cy) were removed (Sorensen and Schmeltz, 1982). Since then the shoal has apparently recovered to its condition prior to dredging. The optimum borrow site location could be selected prior to each dredging event based on condition surveys which would need to include coverage of the flood shoal. These surveys would be used to identify areas of shoal growth or recovery from previous dredging operations as candidates for additional dredging.

Dredging could be easily performed using a traditional floating plant (hydraulic or hopper dredge) on contract. Dredging could conform to a design bypass schedule and quantity, as opposed to dredging from the deposition basin, which, as already experienced, does not accumulate sediment at the same rate every year.

Flood shoal dredging would have to be supplemented with dredging of the authorized channel and deposition basin to provide adequate navigation reliability. Dredging of the flood shoal could take place on the same frequency as the channel and deposition basin (every year) or on a less frequent interval (e.g., every 2 years) but coinciding with dredging the channel and deposition basin every other operation.

As with all Moriches Inlet alternatives, this alternative also includes rehabilitation of the outer end of the west jetty, maintenance of both jetties and the bayside revetment at Pikes Beach, and maintenance of the inner bay channel to a depth of -6 feet MLW and a width of 100 feet, as authorized.

Modeling Results

The Delft3D morphological model was used to simulate future sedimentation in the dredged areas and to assess potential impacts to adjacent inlet cells. The simulation was performed under a conservative condition assuming that roughly one (1) million m^3 were removed from the flood shoal (Figure 8-32). Simulation results and detailed volumetric analysis confirm gradual recovery of the dredged area and minimal impacts on the other inlet cells, which supports the conceptual sediment budget presented below.

Sediment Budget

A conceptual sediment budget for this alternative is presented in Figure 8-33. As shown in this figure, the LST deficit (56,000 m^3/yr) is made up by dredging the flood shoal and placing the material downdrift. It is assumed that over time dredging would offset ongoing accumulation in the flood shoal (14,000 m^3/yr) and possibly reverse historical accumulation at a rate of up to 42,000 m^3/yr. This "erosion" of the flood shoal would offset future accumulation the Updrift Beach (29,000 m^3/yr), ebb shoal (11,000 m^3/yr), and Downdrift Beach (2,000 m^3/yr), which is assumed would continue. Thus, over a 50 year project life, as much as 2.1 million m^3 could be dredged from the flood shoal.

In this alternative it is assumed that sand dredged from the deposition basin or flood shoal would be placed further downdrift but if needed, the sand could be used to periodically nourish the west beach if monitoring surveys indicate erosion. The distance required for either option is less than 2 miles and thus the costs associated with either placement would be very similar.

Costs

First costs and annual costs developed for Alternative 4 are summarized in Table 8-19. The cost estimates assume that both navigation and flood shoal dredging occur on a 1 year cycle. The cost of Alternative 4 is less than the cost of Alternative 1; however, both are more expensive than Existing Practice due to the increased number of mobilizations/demobilizations. It should be noted, however, that reduced risks and damages downdrift associated with more continuous bypassing are not accounted for in these costs.

Table 8-19: Cost Summary for MI Alternative 4: AP + Flood Shoal Dredging

Plan Component	Dredging Interval (years)	Quantity (1000x m^3 (cy))	Mob/ Demob ($000)	Unit Cost ($/$m^3$ ($/cy))	Subtotal Cost ($000)	E&D and S&A ($000)	Total Cost Per Operation ($000)	Annual Cost ($000)
Channel & Deposition Basin Dredging	1	75/ (98)	$1,000	$7.45/ ($5.70)	$1,792	$308	$2,100	$2,208
Flood Shoal Dredging	1	56/ (73)	Same contract	$7.45/ ($5.70)	$479	$88	$566	$595
							Grand Total	$2,803

8.3.6 *Costs Summary & Recommended Alternative*

Table 8-20 summarizes the costs for each shortlisted alternative. Similarly to Shinnecock Inlet, the least expensive alternatives are those that maintain the Authorized Project features and offset the existing LST deficit (56,000 m^3/yr or 73,000 cy/yr) by dredging the ebb shoal or the flood shoal. Existing Practice, which include dredging to the authorized project dimensions every four years on average (instead of the yearly dredging frequency established in the Authorized Project), is actually the least costly alternative, but it does not meet the goal of reliable navigation and the sediment bypassing is only partially restored because the sediment is placed too close to the inlet.

Table 8-20: Summary of Annual Cost for Moriches Inlet Alternatives

Plan	Annual Cost ($000)
Existing Practice	*$2,086*
Alt 1. Authorized Project	$3,272
Alt 2. Alt APD + Ebb Shoal Dredging	$2,803
Alt 3. APD + Semi-fixed Bypass System	$3,844
Alt 4 APD + Flood Shoal Dredging	$2,803

Maintaining reliable navigation would require more frequent dredging, as anticipated in the design of the Authorized Project, which recommended a one year dredging cycle. Recent data confirms that the deposition basin can be completely filled within months of dredging. For example, by October 2004 (i.e., 8 months after dredging in February 2004) the shoal had formed again over the channel and deposition basin and dredging was required. Only the dredging in 1998 seemed to last a little longer, although a survey in the summer of 2000 already showed the ebb shoal bar encroaching on the channel from the east, with depths shallower than -10 feet MLW.

At Moriches Inlet, Alternative 2 is recommended. Navigation conditions will be maintained as authorized with an annual dredging event in the navigation channel (200 feet wide, -10 feet MLW) and deposition basin (800 feet wide, -14 feet MLW). The ebb shoal will be dredged and the material should be placed farther downdrift than the ebb shoal reattachment point in order to mitigate erosion west of this point and to reduce flooding risk. This alternative is more costly than Existing Practice (which does not maintain the Authorized Project dimensions), but provides a greater level of flood protection downdrift and more importantly, unlike Existing Practice, it provides for reliable navigation.

Dredging the ebb shoal on a regular cycle to increase bypassing has less risk and uncertainty as compared to other alternatives since it involves continuation of Existing Practice under the authorized project dimensions and bypassing would be improved using proven dredging technology with relatively well known costs, schedules, performance, and environmental effects. Additionally, dredging allows for flexibility by, for example, potentially extending the interval between dredging events during relatively calm wave years such as the 1998 to 2000 period. Finally, it provides the ability to implement a full range of alternatives at the end of the project life. Nonetheless, similarly to Shinnecock Inlet, monitoring of ebb shoal recovery and erosion of the downdrift beaches will be a critical element of this plan.

Arguably, increasing the deposition basin dimensions could be used to maintain a channel for at least one year or perhaps even two and thus to reduce average annual costs and improve navigation. However, as suggested by the GDM that supports the current authorization, further improvements in navigation do not appear to be warranted and a larger deposition basin may have unintended effects on the sediment budget for the inlet. Nonetheless, actual performance of the project on a 1-year dredging cycle should be monitored and, if needed, the dimensions and/or layout of the deposition basin could be reassessed.

Dredging the flood shoal (Alt. 4) instead of the ebb shoal has similar drawbacks at Moriches as at Shinnecock Inlet: increased uncertainty with regards to morphodynamics, optimum dredging rates, effects on the sediment budget and potential impacts on hydrodynamics and flooding.

The main drawbacks of the semi-fixed bypass system (Alt. 3) are capacity and costs. At Moriches Inlet the net westerly longshore sediment transport immediately updrift of the inlet is 238,000 m^3/yr, which is more than double the capacity of this type of bypassing systems (estimated at 100,000 m^3/yr). Therefore, with a semi-fixed bypassing plant annual dredging in the channel and deposition basin will continue to be required, albeit at a reduced rate. More importantly, sediment would continue to accumulate in the inlet shoals since the system would not capture and transfer 100% of the littoral drift. The resulting deficit, albeit somewhat reduced from *Existing (c.2001)* conditions, would still have to be offset by periodic dredging from other sources (e.g., offshore). Note that combining ebb shoal dredging with a semi-fixed bypassing plant would also offset the LST deficit, but at a higher cost than dredging alone.

A semi-fixed bypassing plant would provide for more continuous bypassing. However, continuity is not as much of issue for the dredging alternatives in this case given the recommended yearly dredging cycle. Dredging also allows for flexibility by, for example, potentially extending the interval between dredging events during relatively calm wave years such as the 1998 to 2000 period.

8.4 Fire Island Inlet

The following alternatives remain under consideration at Fire Island Inlet:

Alt. 1: Existing Practice/Authorized Project (AP)
Alt. 2: AP + Dredging the Ebb Shoal
Alt. 3: Optimized Deposition Basin
Alt. 4: AP + Dredging the Flood Shoal

8.4.1 Summary of Existing Conditions

Previous sediment budget estimates (Gravens et al., 1999), largely based on shoreline changes from 1979 to 1995, suggest a long-shore sediment transport rate of approximately 188,000 m^3/yr (245,000 cy/yr) entering the inlet from the east and under existing (circa 1999) conditions. However, recent shoreline and volumetric changes (1995 to date) suggest that the net westerly transport at Fire Island Inlet might be significantly higher, 295,000 m^3/yr (386,000 cy/yr). This amount is consistent with recent (1997, 1999 and 2001) maintenance dredging operations which have yielded approximately 279,000 m^3/yr (365,000 cy/yr). More recently, the inlet was also dredged in 2003-04 and the next maintenance dredging project is scheduled for the fall of 2007, although insufficient funds may prevent project implementation according to the most recent USACE Project Fact Sheet (May 2007).

The *Existing (c. 2001)* condition sediment budget for Fire Island Inlet suggests a net westerly transport rate of 351,000 m^3/yr entering the updrift (east) boundary of the inlet system. The budget also suggests that approximately 59% (206,000 m^3/yr) of the net updrift westerly transport bypasses the inlet system, including the channels, shoals, and adjacent beaches. The remaining 41% (145,000 m^3/yr) accumulates within the inlet shoals and adjacent beaches. The budget includes dredging of the deposition basin at a rate of 375,000 m^3/yr. The channel and deposition basin are typically dredged every two years. One significant difference between Fire Island Inlet and Moriches or Shinnecock Inlet is that it is maintained based on a multi-purpose authorization: navigation and beach erosion. In fact, the deposition basin is dredged based on the volume of material need for placement on Jones Island according to the beach erosion authorization (roughly 500,000 cy/yr).

Navigation conditions are not optimal at the inlet under existing conditions. The western side of dredged channel and deposition basin shoal rapidly after maintenance dredging operations as the westerly moving shallow sand spit that extends from Democrat Point breakwater encroaches on the channel. These changes typically require the Coast Guard to move the channel buoys about twice a year.

8.4.2 Alternative 1: Existing Practice/Authorized Project (AP)

This alternative consists of continuing the current practice of channel and deposition basin dredging approximately every two years under the Authorized Project (Figure 8-34). Sediment bypassed to Gilgo Beach on Jones Island (313,000 m^3/yr or 410,000 cy/yr) would be placed farther west than in some of the previous bypassing events (at least 3 miles west of Democrat Point), which would theoretically reduce the amount of sediment flowing back towards the ebb shoal, thus improving overall bypassing efficiency and slowing down ebb shoal growth. This approach was apparently implemented during the maintenance dredging project conducted in 2004, which called for placement starting approximately 7,000 feet farther west than the previous project in 2001. This placement location is also included by default in the rest of the alternatives discussed.

The shoreline east of the inlet along Robert Moses State Park still suffers from significant fluctuations which may endanger existing park facilities, so under this alternative backpassing to this area will also continue as in the recent past; typically 20% of the sediment recently dredged from Fire Island Inlet has been placed in this area (62,000 m^3/yr or 81,000 cy/yr).

Modeling Results

The Delft3D morphological model was used to simulate future sedimentation in the dredged areas (deposition basin and authorized channel) and to assess potential impacts to adjacent inlet cells. Simulation results are presented in Figure 8-35. After two years of simulation the model shows sediment accumulation at the navigation channel as a consequence of the encroaching of both the ebb shoal (from the west) and the sand spit (from the east). Model results and detailed volumetric analysis indicate that during the simulation sand continues to accumulate in the deposition basin and the ebb shoal, while the downdrift beach experiences a net loss of sand as a consequence of the bypassing deficit.

Sediment Budget

A conceptual sediment budget for this alternative is presented in Figure 8-36. As shown in this figure, the LST deficit is the same as under *Existing (c. 2001)* conditions (145,000 m^3/yr). As explained above, under Existing practice sand dredged from the deposition basin would be bypassed to Gilgo Beach and backpassed to Robert Moses State Park.

Costs

First costs and annual costs were developed for Alternative 1 and are presented in Table 8-21; note that the cost for Alternative 1 and Existing Practice are the same as this alternative is a continuation of Existing Practice under the current, multi-purpose, project authorization. Note that the cost of the existing bypass deficit (145,000 m^3/yr) is taken into account by assuming this deficit would have to be eventually made up by importing sand from an offshore source on a 4 year cycle. This is just a way to assign a cost to the existing bypassing deficit and it does not mean that offshore dredging is currently part of the Existing Practice at Fire Island Inlet. Also note that this cost does not account for damages related to a more or less eroded shoreline condition downdrift.

Table 8-21: Cost Summary for Existing Practice/Authorized Project (AP)								
Plan Component	**Dredging Interval (years)**	**Quantity (1000x m^3 (cy))**	**Mob/ Demob ($000)**	**Unit Cost ($/$m^3$ ($/cy))**	**Subtotal Cost ($000)**	**E&D and S&A ($000)**	**Total Cost Per Operation ($000)**	**Annual Cost ($000)**
Channel & Deposition Basin Dredging – Updrift Placement	2	124 (162)	$593	$5.50 ($4.20)	$1,464	$254	$1,718	$926
Channel & Deposition Basin Dredging – Downdrift Placement	2	626 (819)	$2,608	$9.60 ($7.30)	$9,875	$1,542	$11,417	$6,151
LST Deficit	4	580 (759)	$1,000	$11.80 ($9.00)	$9,006	$1,414	$10,419	$2,972
							Grand Total	$10,049

8.4.3 Alternative 2: AP + Dredging the Ebb Shoal

Similarly to Shinnecock and Moriches Inlets, one alternative to increase bypassing is to remove sediment from part of the ebb shoal, such as the seaward slope. Ebb shoal dredging may be easily implemented using a traditional floating plant on contract and in combination with the regular maintenance of the authorized channel and deposition basin. This alternative would offset the continued growth of the ebb shoal, which is now estimated at 68,000 m^3/yr (89,000 cy/yr). However, a program to survey the ebb shoal on a regular basis would need to be implemented before moving forward with this approach. A conceptual sketch of this alternative is shown in Figure 8-37.

The potential borrow area is roughly 12,000 feet long and 1,500 feet wide and located seaward of the -20 foot NGVD contour to minimize impacts on nearshore wave climate conditions and existing sediment transport processes. Dredging could be performed to the -35 foot contour (NGVD). Based on the recent bathymetry data, the volume of sediment available within this area is roughly 4.5 million m^3. Therefore, it would only be necessary to dredge within a much smaller area to produce the amount necessary for each dredging operation. The optimum borrow site location could be selected prior to each dredging event based on condition surveys which would need to include coverage of the ebb shoal. These surveys would be used to identify areas of ebb shoal growth or recovery from previous dredging operations as candidates for additional dredging.

Ebb shoal dredging would be supplemented with dredging of the deposition basin and channel (i.e., Authorized Project).

<u>*Modeling Results*</u>

The Delft3D morphological model was used to simulate the effect of dredging the ebb shoal on the inlet morphology and also to assess potential impacts to adjacent inlet cells. Two dredging alternatives were simulated: dredging a volume of 290,000 m^3 to offset the LST deficit for a two year cycle and dredging all the sediment available (to a depth of -35 ft NGVD) at the potential borrow site area (around 4.5 million m^3). Results from these simulations are presented in Figure 8-38 and Figure 8-39 respectively. Although in both cases a recovery of the dredged area is observed, a much faster recovery takes place for the smaller area as expected.

<u>*Sediment Budget*</u>

A conceptual sediment budget for this alternative is presented in Figure 8-40. As is shown in this figure, the LST deficit (145,000 m^3yr) is made up by dredging the ebb shoal and placing the material downdrift. It is assumed that overtime dredging would offset ongoing ebb shoal accumulation (68,000 m^3/yr). This "erosion" of the ebb shoal would offset future accumulation in the Updrift Beach (9,000 m^3/yr), Oak Beach (15,000 m^3/yr), Channel (9,000 m^3/yr), Sand Spit (15,000 m^3/yr), Deposition Basin (1,000 m^3/yr), and Downdrift Beach (28,000 m^3/yr). The total volume dredged from the ebb shoal over the project life would be 3.9 million m^3.

In this alternative it is assumed that sand dredged from the ebb shoal would be placed downdrift of existing inlet processes in order to offset the LST deficit. Alternatively, some of the dredged sand from the ebb shoal could be placed at Gilgo Beach and some of the material dredged from the deposition basin placed downdrift of the inlet system. The distance required for either option is similar and thus the costs associated with either placement would be very similar.

<u>*Costs*</u>

First costs and annual costs developed for Alternative 2 are summarized in Table 8-22. The cost estimates assume that both navigation and ebb shoal dredging occur on a 2 year cycle. The cost of dredging the ebb shoal on a 2 year cycle is less ($972,000/yr) than the cost of Existing Practice including dredging offshore every 10 years to offset the existing LST deficit.

Table 8-22: Cost Summary for AP +Dredging the Ebb Shoal

Plan Component	Dredging Interval (years)	Quantity (1000x m^3 (cy))	Mob/ Demob ($000)	Unit Cost ($/$m^3$ ($/cy))	Subtotal Cost ($000)	E&D and S&A ($000)	Total Cost Per Operation ($000)	Annual Cost ($000)
Channel & Deposition Basin Dredging – Updrift Placement	2	124 (162)	$593	$5.50 ($4.20)	$1,464	$254	$1,718	$926
Channel & Deposition Basin Dredging – Downdrift Placement	2	626 (819)	$2,608	$9.60 ($7.30)	$9,875	$1,542	$11,417	$6,151
Ebb Shoal Dredging	2	290 (379)	Same contract	$9.60 ($7.30)	$3,182	$530	$3,711	$2,000
							Grand Total	$9,077

8.4.4 Alternative 3: Optimized Deposition Basin

This alternative consists of optimizing the alignment and dimensions of the existing deposition basin to reduce channel shoaling and possibly to improve navigation. A wider deposition basin configuration at the leading edge of the sand spit may be more efficient that the existing basin which is relatively narrow in this area and thus is not sufficient to prevent significant channel shoaling in this area between dredging events. This alternative should allow for continuation of existing bypassing rates. Bypassing efficiency would also be improved by placing the material farther west along Gilgo Beach. A conceptual sketch of this alternative is shown in Figure 8-41.

Modeling Results

The Delft3D morphological model was used to simulate the effect of the aforementioned optimization to the deposition basin on the inlet morphology and also to assess potential impacts to the different inlet cells. Model results are presented in Figure 8-42. Model results and detailed volumetric analysis indicate that a larger volume of sand will accumulate on the deposition basin during the first few years and also an slight increase in sedimentation will occur at the downdrift lobe. No differences are observed at the downdrift beach when compared to other alternatives. Less accumulation is observed at the flood shoal for this alternative, probably due to the increase accumulation of sand on the deposition basin.

Sediment Budget

A conceptual sediment budget for this alternative is presented in Figure 8-43. As shown in this figure, the LST deficit is the same as under *Existing* conditions (145,000 m^3/yr). In this alternative it is assumed that sand dredged from the deposition basin would be bypassed to Gilgo Beach and backpassed to Robert Moses State Park.

Costs

First costs and annual costs developed for Alternative 3 are summarized in Table 8-23. The cost estimate assumes that the bypass deficit will be made up by importing sand from an offshore source on a 4 year cycle. Because dredging volumes and intervals are not modified from Existing Practice, the cost of this alternative is the same as Alternative 1.

Table 8-23: Cost Summary Optimized Deposition Basin

Plan Component	Dredging Interval (years)	Quantity (1000x m^3 (cy))	Mob/ Demob ($000)	Unit Cost ($/$m^3$ ($/cy))	Subtotal Cost ($000)	E&D and S&A ($000)	Total Cost Per Operation ($000)	Annual Cost ($000)
Channel & Deposition Basin Dredging – Updrift Placement	2	124 (162)	$593	$5.50 ($4.20)	$1,464	$254	$1,718	$926
Channel & Deposition Basin Dredging – Downdrift Placement	2	626 (819)	$2,608	$9.60 ($7.30)	$9,875	$1,542	$11,417	$6,151
LST Deficit	4	580 (759)	$1,000	$11.80 ($9.00)	$9,006	$1,414	$10,419	$2,972
							Grand Total	$10,049

8.4.5 Alternative 4: AP+Dredging the Flood Shoal

The flood shoal at Fire Island Inlet, which is defined here as the shallow areas that extend from the thumb, east along Oak Beach past the Robert Moses State Park causeway and into the areas adjacent to the eastern tip of Jones Island, contains a large volume of sediment that has been carried to those areas as the inlet migrated west. Limited dredging of these areas (assuming that the material is compatible with beach sand) may be a way of supplementing existing bypassing practices in order to achieve 100% bypassing of the net westerly transport arriving at Fire Island Inlet. A conceptual sketch is shown in Figure 8-44.

<u>*Modeling Results*</u>

The Delft3D morphological model was used to simulate future sedimentation in the dredged areas and to assess potential impacts to other inlet cells associated to Alternative 4. Simulation results are presented in Figure 8-45. Simulation results confirm gradual recovery of the dredged area of the flood shoal and minimal impacts on other inlet cells. Specifically, detailed volumetric analysis indicates that changes in other cells (including the Deposition Basin, the Downdrift Lobe and the Downdrift Beach) would be similar to existing conditions.

<u>*Sediment Budget*</u>

A conceptual sediment budget for this alternative is presented in Figure 8-46. As shown in this figure, the LST deficit (145,000 m^3/yr) is made up by dredging the flood shoal (Channel and Oak Beach cells) and placing the material downdrift on Gilgo Beach or updrift at Robert Moses State Park. It is assumed that over time dredging would offset ongoing accumulation in the flood shoal (24,000 m^3/yr). This "erosion" of the flood shoal would offset future accumulation the Updrift Beach (9,000 m^3/yr), ebb shoal (68,000 m^3/yr), Deposition Basin (1,000 m3/yr), and Downdrift Beach (28,000 m^3/yr), which is assumed would continue. Thus, over a 50 year project life, as much as 5.3 million m^3 would be dredged from the flood shoal.

<u>*Costs*</u>

First costs and annual costs developed for Alternative 4 are summarized in Table 8-24. The cost estimates assume that both navigation and flood shoal dredging occur on a 2 year cycle. This alternative is slightly more expensive than dredging the ebb shoal due to the increased distance to the downdrift placement site.

Table 8-24: Cost Summary for AP + Dredging the Flood Shoal

Plan Component	Dredging Interval (years)	Quantity (1000x m^3 (cy))	Mob/ Demob ($000)	Unit Cost ($/$m^3$ ($/cy))	Subtotal Cost ($000)	E&D and S&A ($000)	Total Cost Per Operation ($000)	Annual Cost ($000)
Channel & Deposition Basin Dredging – Updrift Placement	2	124 (162)	$593	$5.50 ($4.20)	$1,464	$254	$1,718	$926
Channel & Deposition Basin Dredging – Downdrift Placement	2	626 (819)	$2,608	$9.60 ($7.30)	$9,875	$1,542	$11,417	$6,151
Flood Shoal Dredging	2	290 (379)	Same contract	$7.65 ($10.00)	$4,359	$713	$5,072	$2,732
							Grand Total	$9,809

8.4.6 Recommended Alternative

Table 8-25 summarizes the costs for each shortlisted alternative. Note that Alternative 1 essentially represents continuation of the Existing Practice under the current, multi-purpose, project authorization. All four alternatives have similar costs although Alternatives 1 and 3 are slightly more costly because of the need to offset the estimated LST deficit (145,000 m^3/yr or 190,000 cy/yr) by means offshore dredging on a 4 year cycle instead of dredging the ebb shoal or flood shoal. The average annual cost associated with this offshore dredging is $2,972,000.

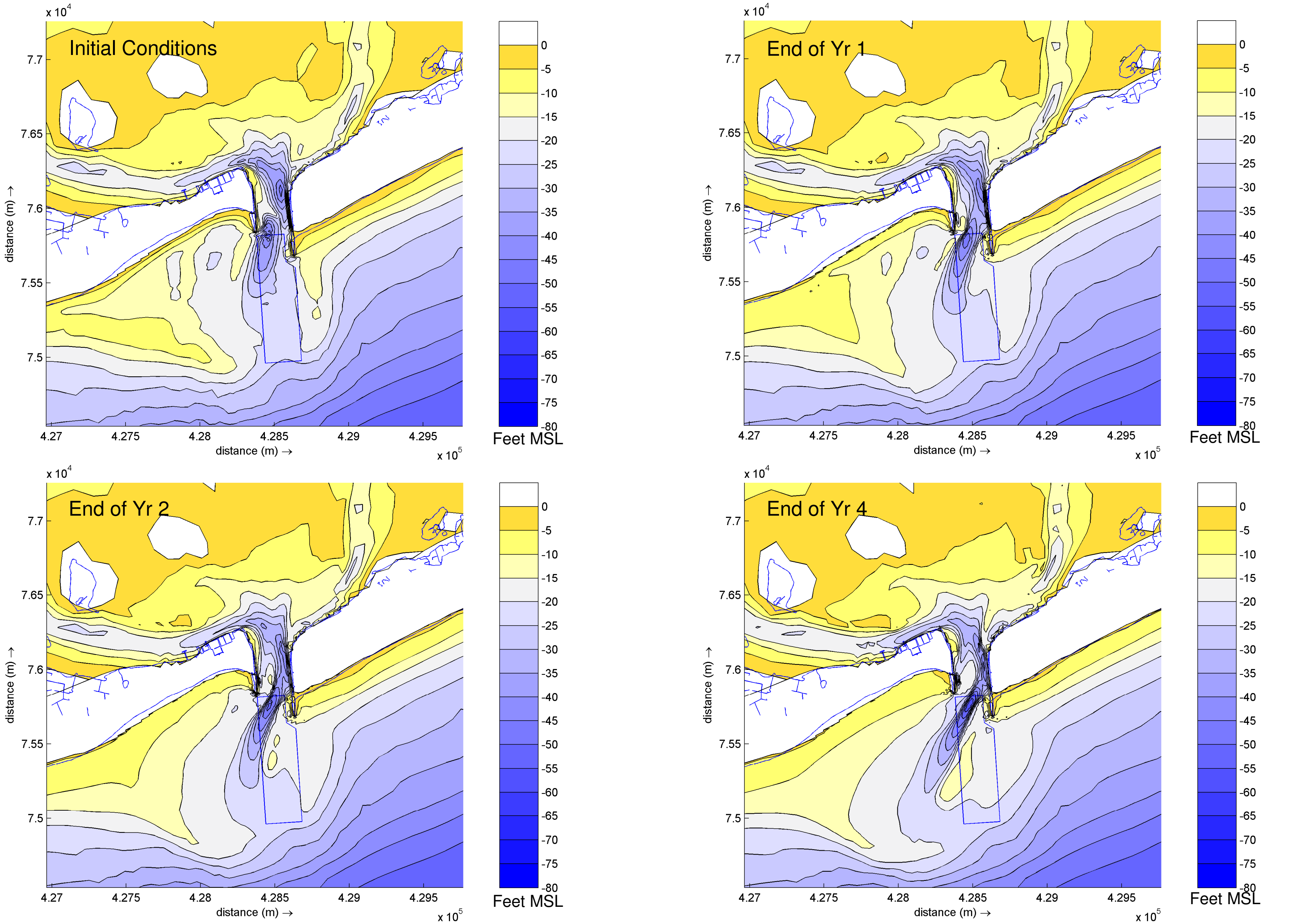
Table 8-25: Summary of Annual Cost for Fire Island Inlet Alternatives	
Plan	**Annual Cost ($000)**
Alt 1. Existing Practice/ Authorized Project (AP)	$10,049
Alt 2. AP + Dredging the Ebb Shoal	$9,077
Alt 3. Optimized Deposition Basin	$10,049
Alt. 4 AP + Dredging the Flood Shoal	$9,809

Available morphological data, model simulations, and sediment budget analyses do not suggest any significant benefits (e.g., increased bypassing, reduced maintenance dredging or improved navigation) associated with a complete realignment of the channel and/or deposition basin. However, a slightly wider deposition basin at the western tip of the existing sand spit will limit encroachment of this feature into the navigation channel at the end of each dredging cycle. Therefore, the recommended plan for Fire Island Inlet consists of combining Alternatives 1 and 4 and continuing the recent practice of placing all of the dredged material at least three miles west of Democrat Point.

Future placement of some of the dredged material along Robert Moses State Park (i.e., backpassing) on an as needed basis will depend on future shoreline changes and infrastructure protection requirements. A more detailed breakdown of the costs for this recommended plan is presented in Table 8-26. Note that the slight change in the deposition basin will not change the costs compared to Alternative 2. Initial dredging in the expansion area will likely be offset with less dredging along the deposition basin farther offshore.

As in the case in Shinnecock and Moriches Inlet, this alternative provides the most reliable, flexible, and cost-effective means for maintaining navigation and offsetting the existing LST deficit. Given the volumes and distances involved the only other feasible alternative would be to dredge the flood shoal or offshore. Dredging offshore would be more expensive and would not directly eliminate the existing sediment sink at Fire Island Inlet. Dredging the flood shoal may be technically feasible, but its dynamics are poorly understood at this time due to lack of comprehensive bathymetry data, and geomorphic, hydrodynamic, and environmental impacts associated with dredging this feature may be significant. Moreover, dredging the flood shoal, particularly in areas east of the Robert Moses Causeway, would be more costly than dredging the ebb shoal because of the increased transport distance.

Table 8-26: Costs for FII Recommended Alternative: AP + Ebb Shoal Dredging & DB Expansion								
Plan Component	**Dredging Interval (years)**	**Quantity (1000x m^3 (cy))**	**Mob/ Demob ($1000s)**	**Unit Cost ($/$m^3$ ($/cy))**	**Subtotal Cost ($1000s)**	**E&D and S&A ($1000s)**	**Total Cost Per Operation ($1000s)**	**Average Annual Cost ($1000s)**
Channel & Deposition Basin Dredging – Updrift Placement	2	124 (162)	$593	$5.50 ($4.20)	$1,464	$254	$1,718	$926
Channel & Deposition Basin Dredging – Downdrift Placement	2	626 (819)	$2,608	$9.60 ($7.30)	$9,875	$1,542	$11,417	$6,151
Ebb Shoal Dredging	2	290 (379)	Same contract	$9.60 ($7.30)	$3,182	$530	$3,711	$2,000
							Grand Total	$9,077



SI Authorized Project: Modeled Morphological Evolution

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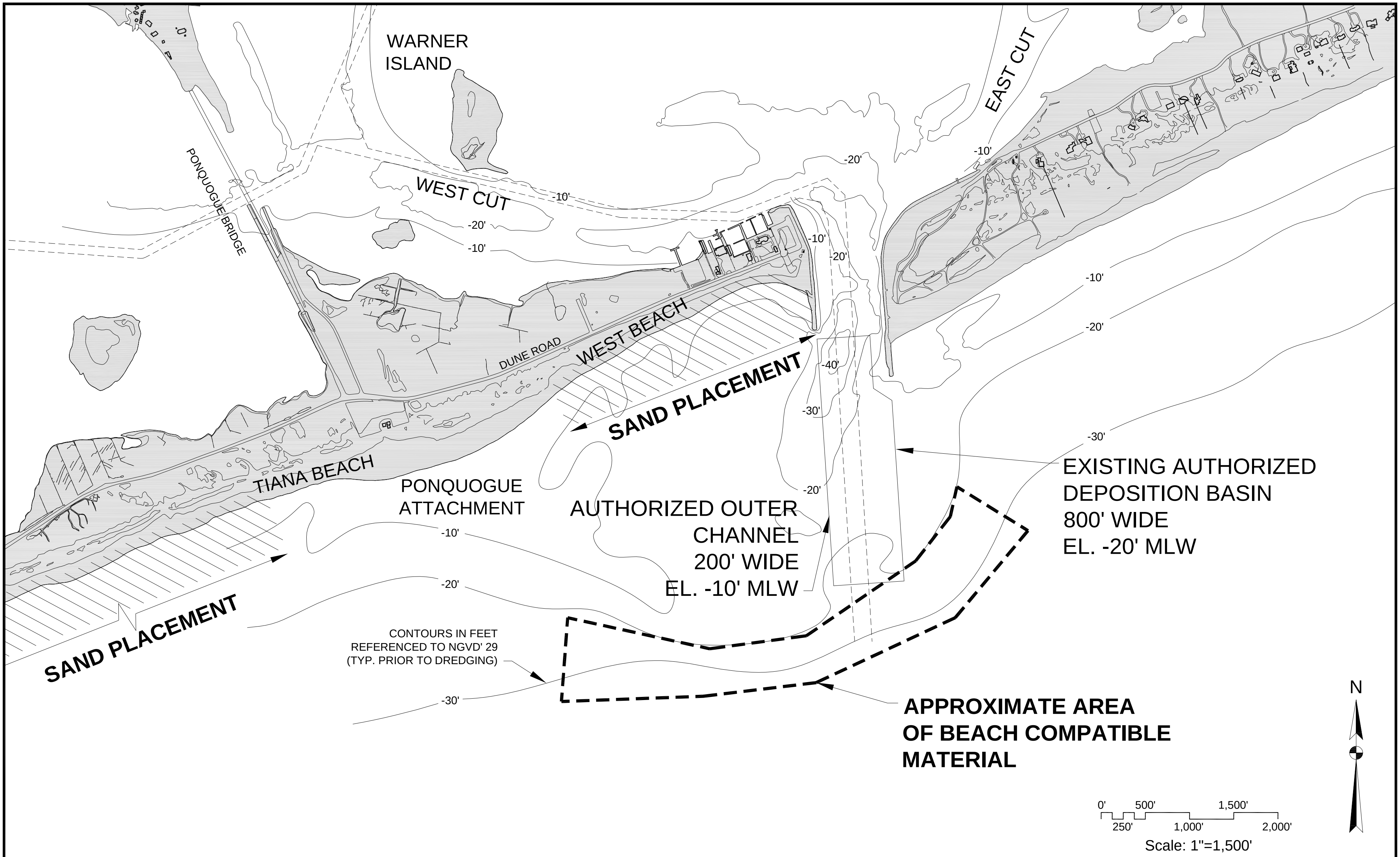
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 8-1



SHINNECOCK INLET - Alternative 1
DREDGING THE EBB SHOAL

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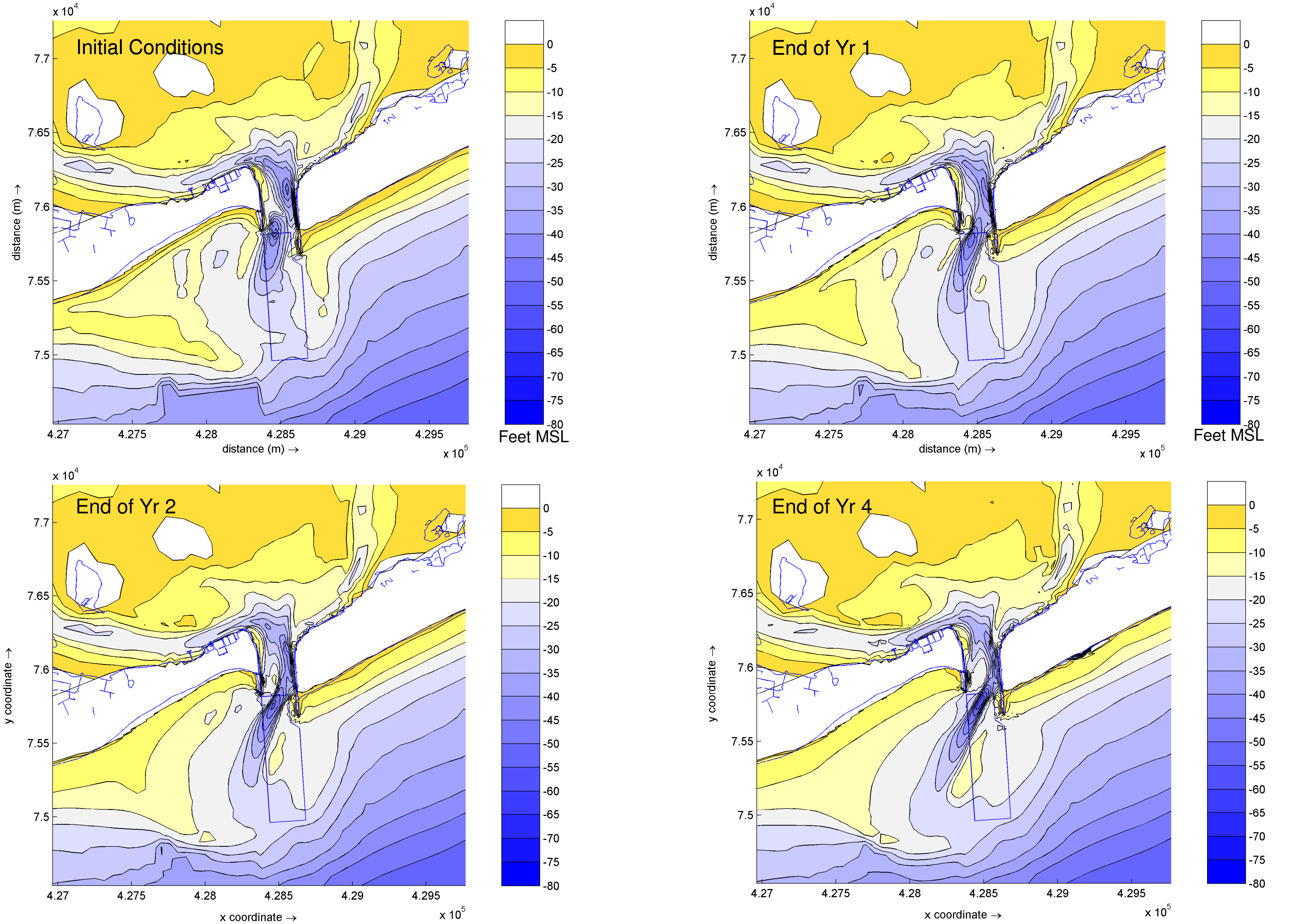
FIGURE 8-2



SI Alt. 1. Dredging the Ebb Shoal: Modeled Morphological Evolution (Small Dredged Area)

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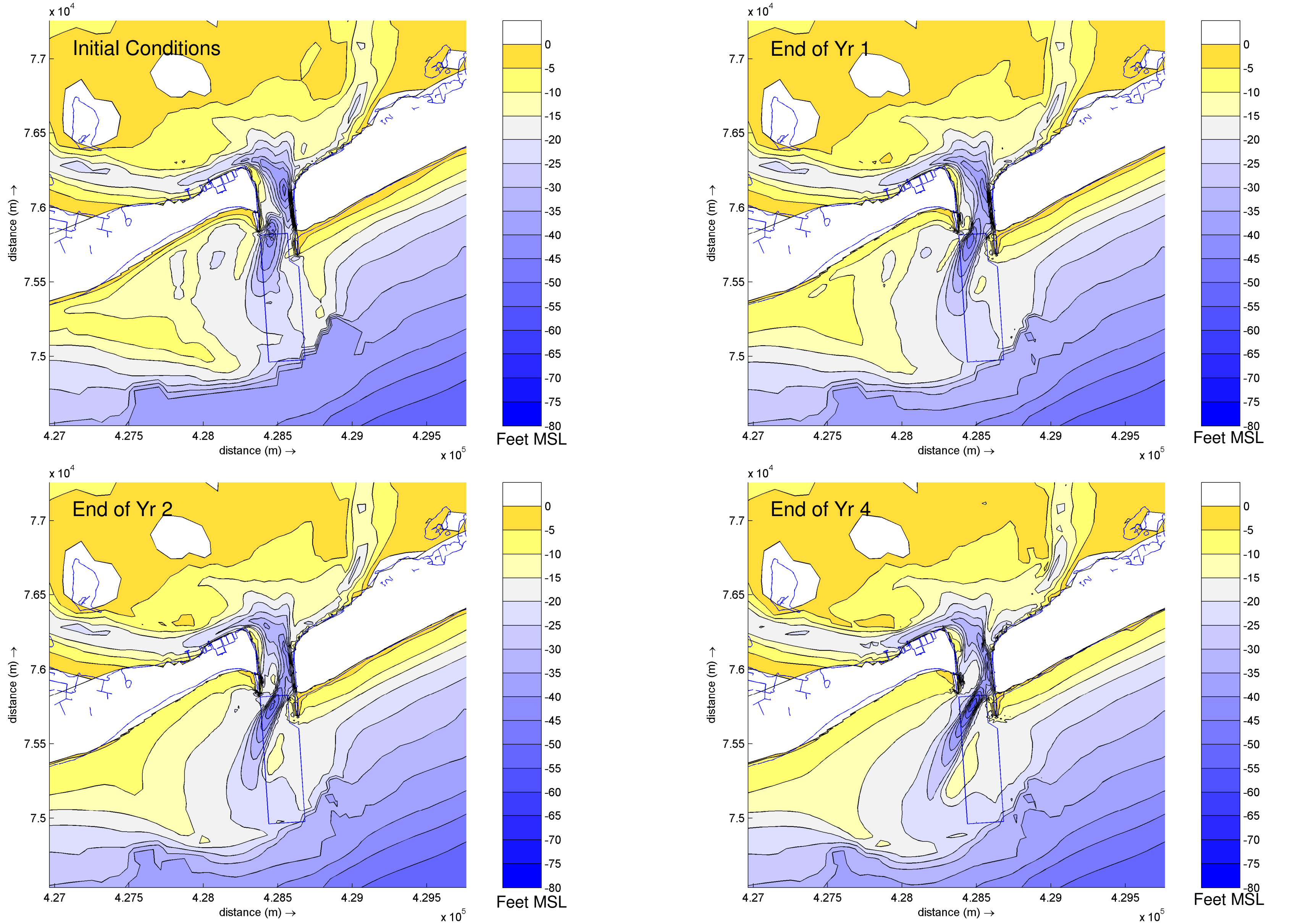
FIGURE 8-3	



SI Alt. 1. Dredging the Ebb Shoal: Modeled Morphological Evolution (Large Dredged Area)

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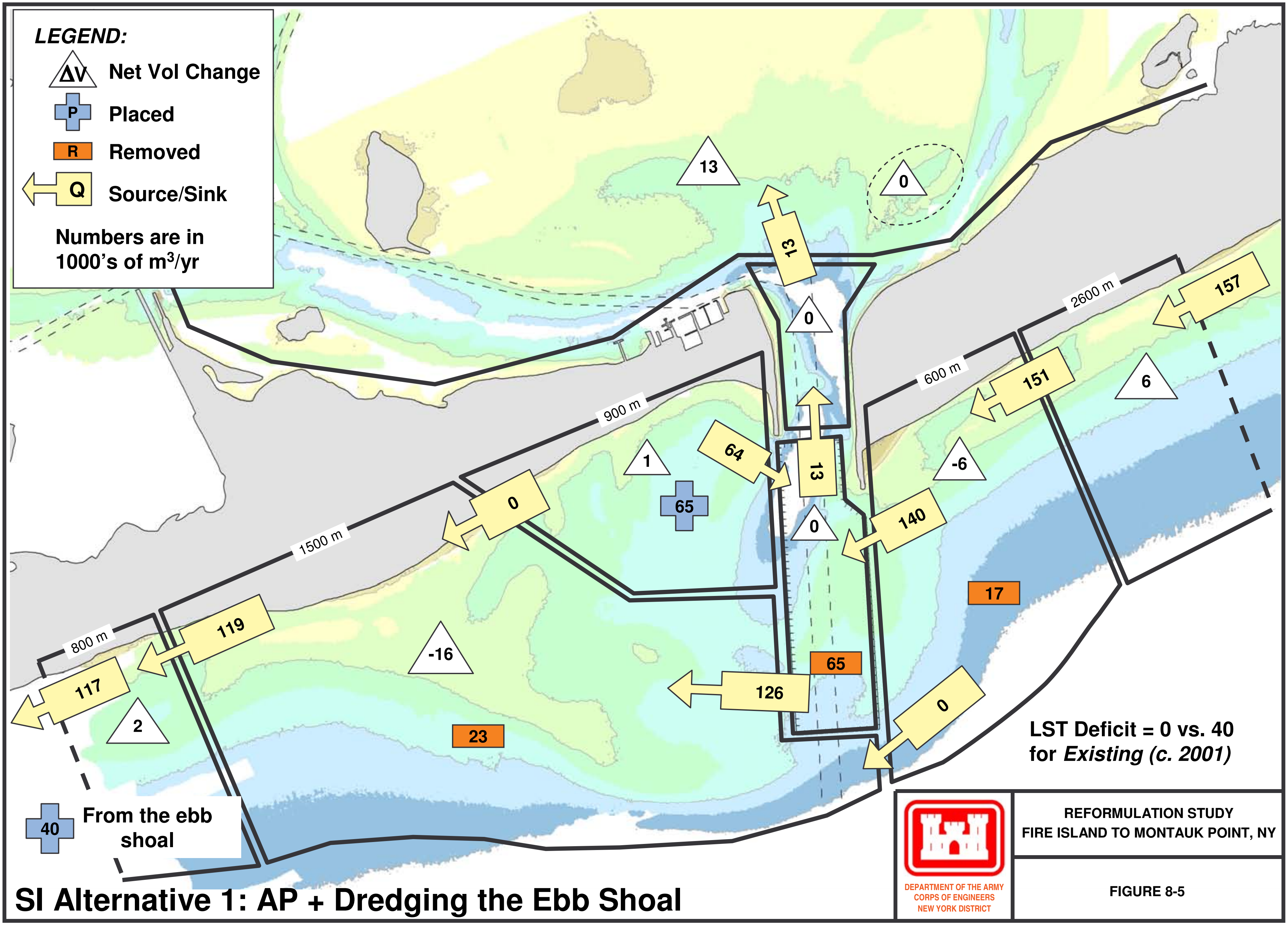
FIGURE 8-4



SI Alternative 1: AP + Dredging the Ebb Shoal

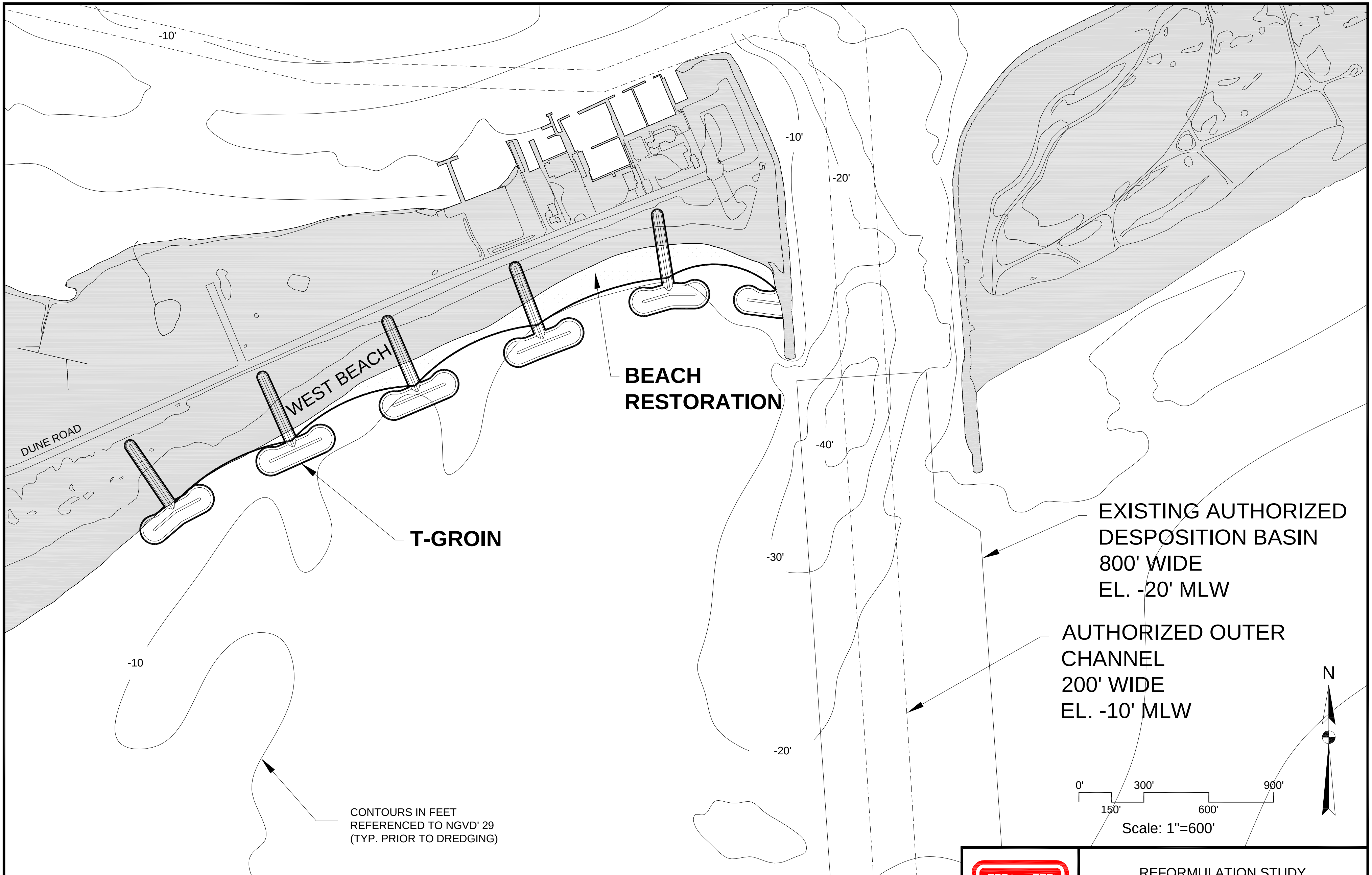


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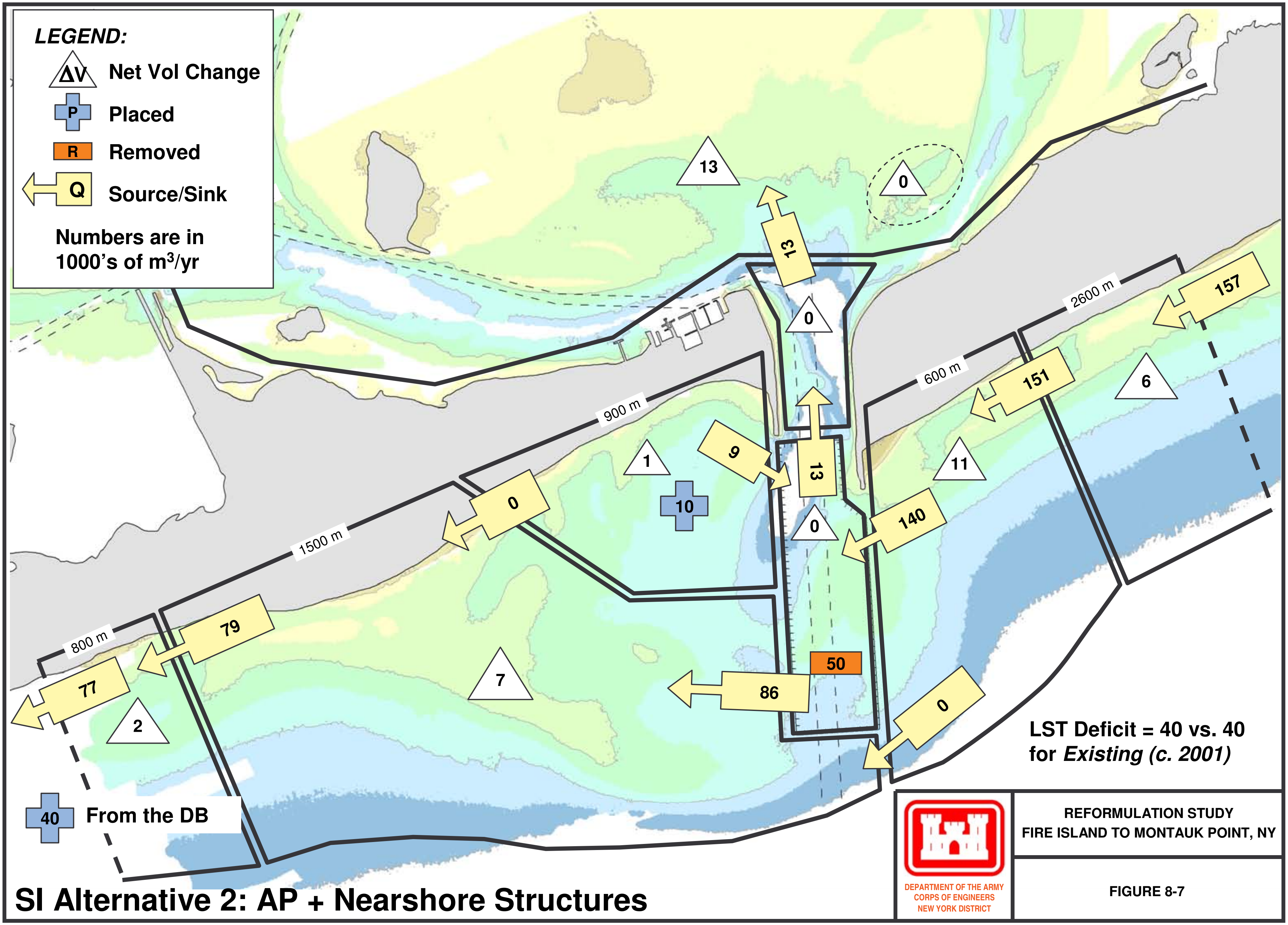
FIGURE 8-6

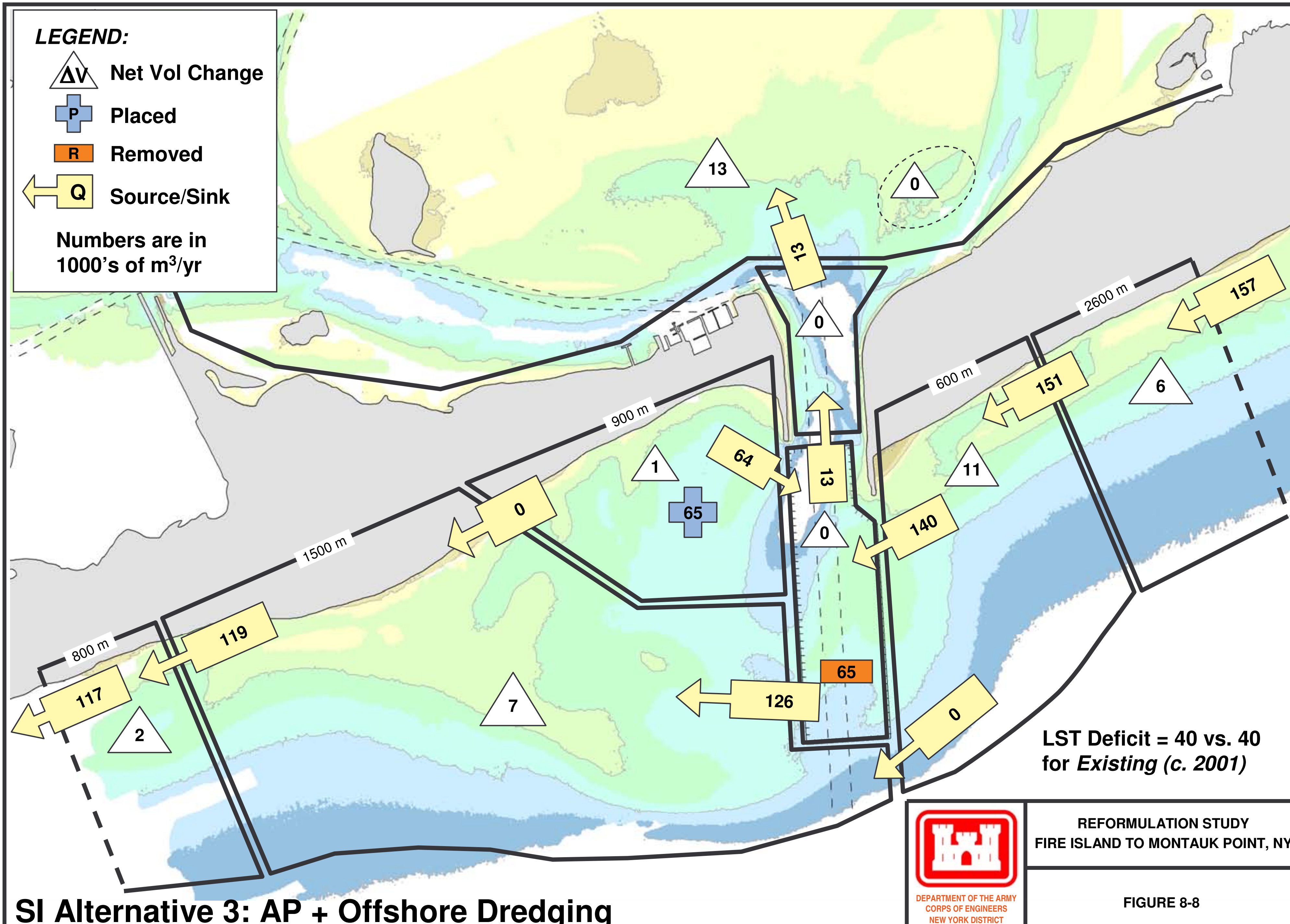
SHINNECOCK INLET - Alternative 2
EXISTING PRACTICE PLUS NEARSHORE STRUCTURES ALONG WEST BEACH

Filename: P:\3982-28\CAD\dwg\SI ALT No. 2.dwg (Shinnecock)



SI Alternative 2: AP + Nearshore Structures

FIGURE 8-7



SI Alternative 3: AP + Offshore Dredging

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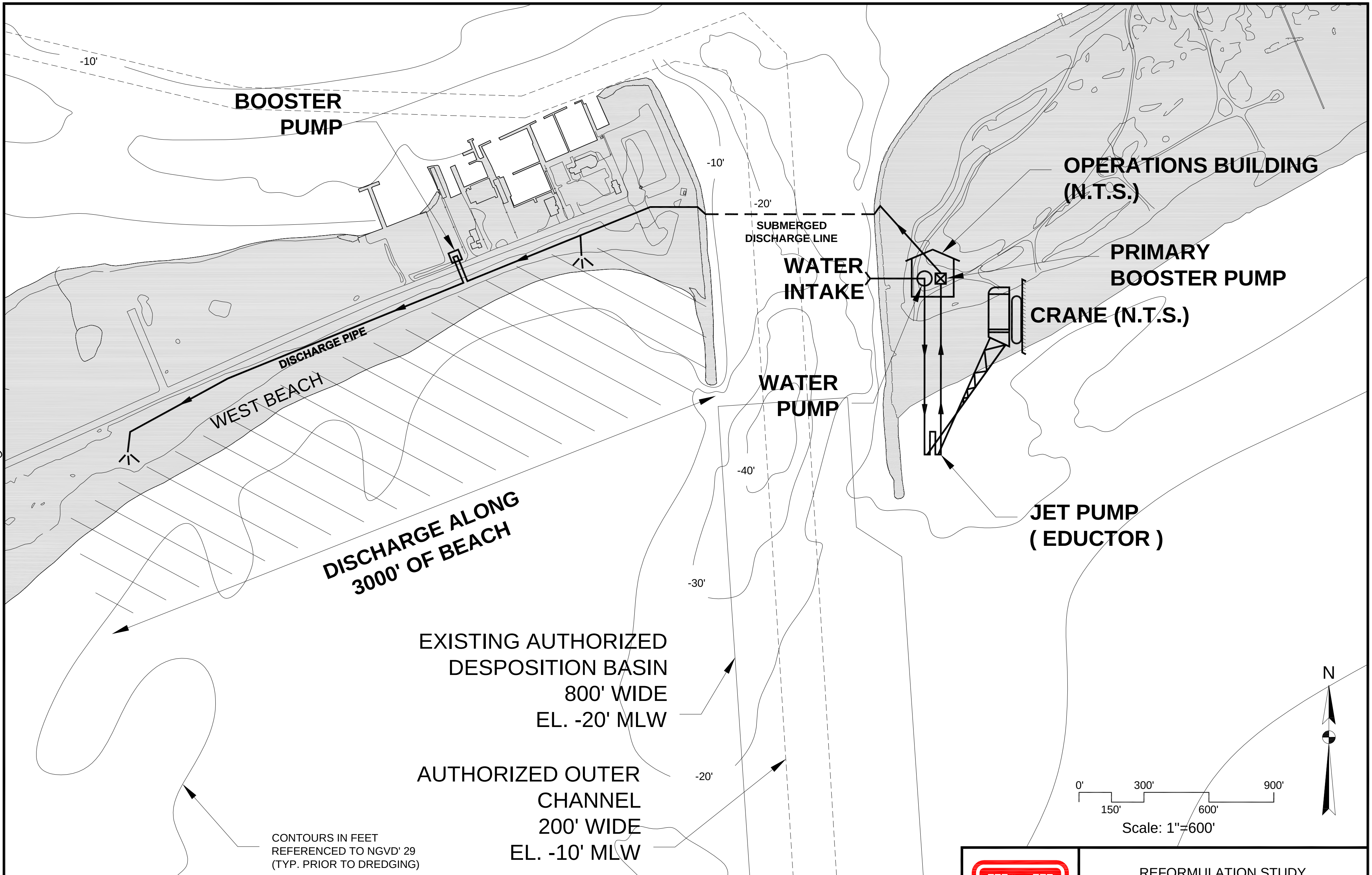
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 8-8



SHINNECOCK INLET - Alternative 4
SEMI-FIXED BYPASS SYSTEM

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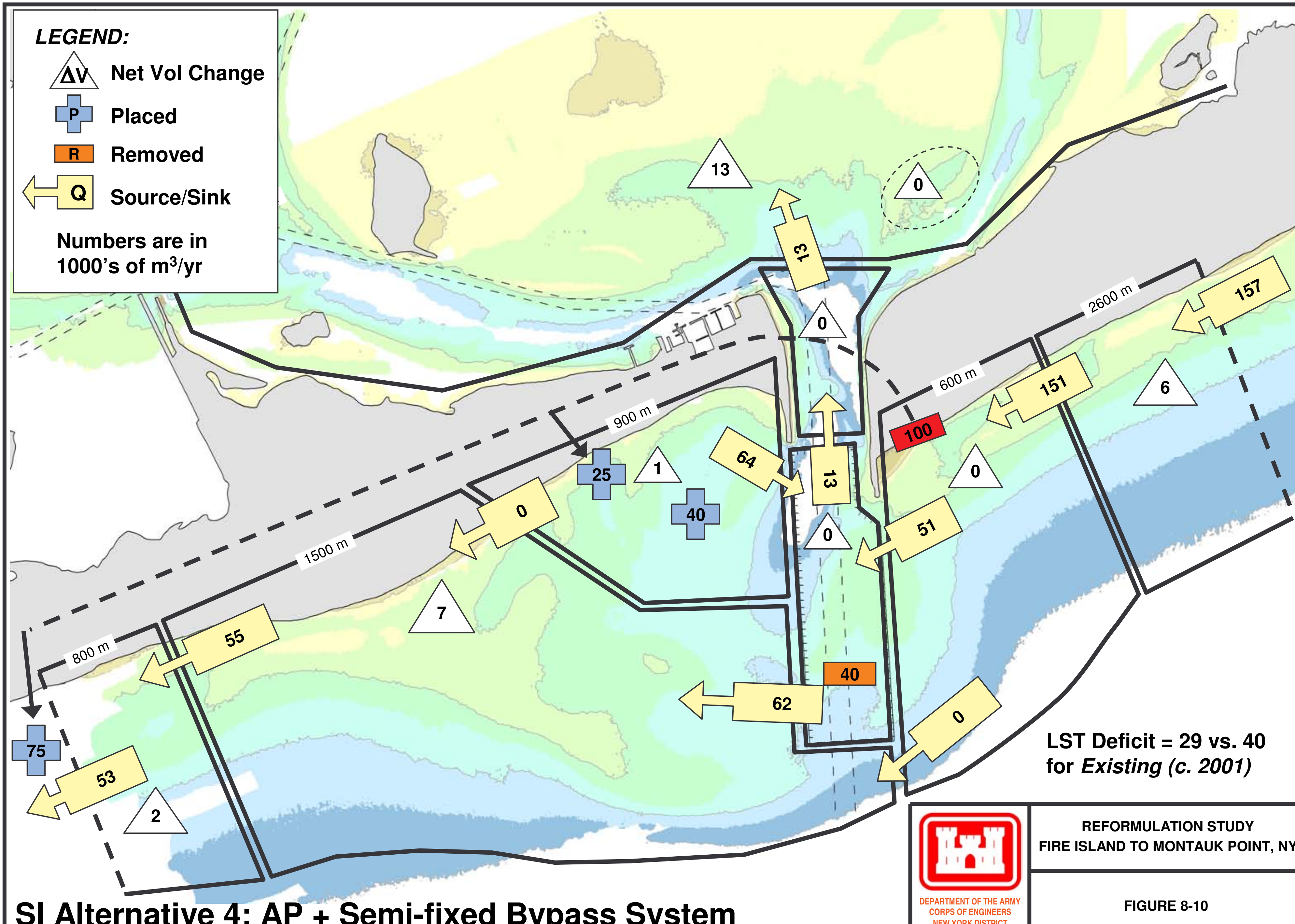
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 8-9

Filename: P:\3982-28\CAD\dwg\SI ALT No. 4.dwg (Shinnecock)



SI Alternative 4: AP + Semi-fixed Bypass System

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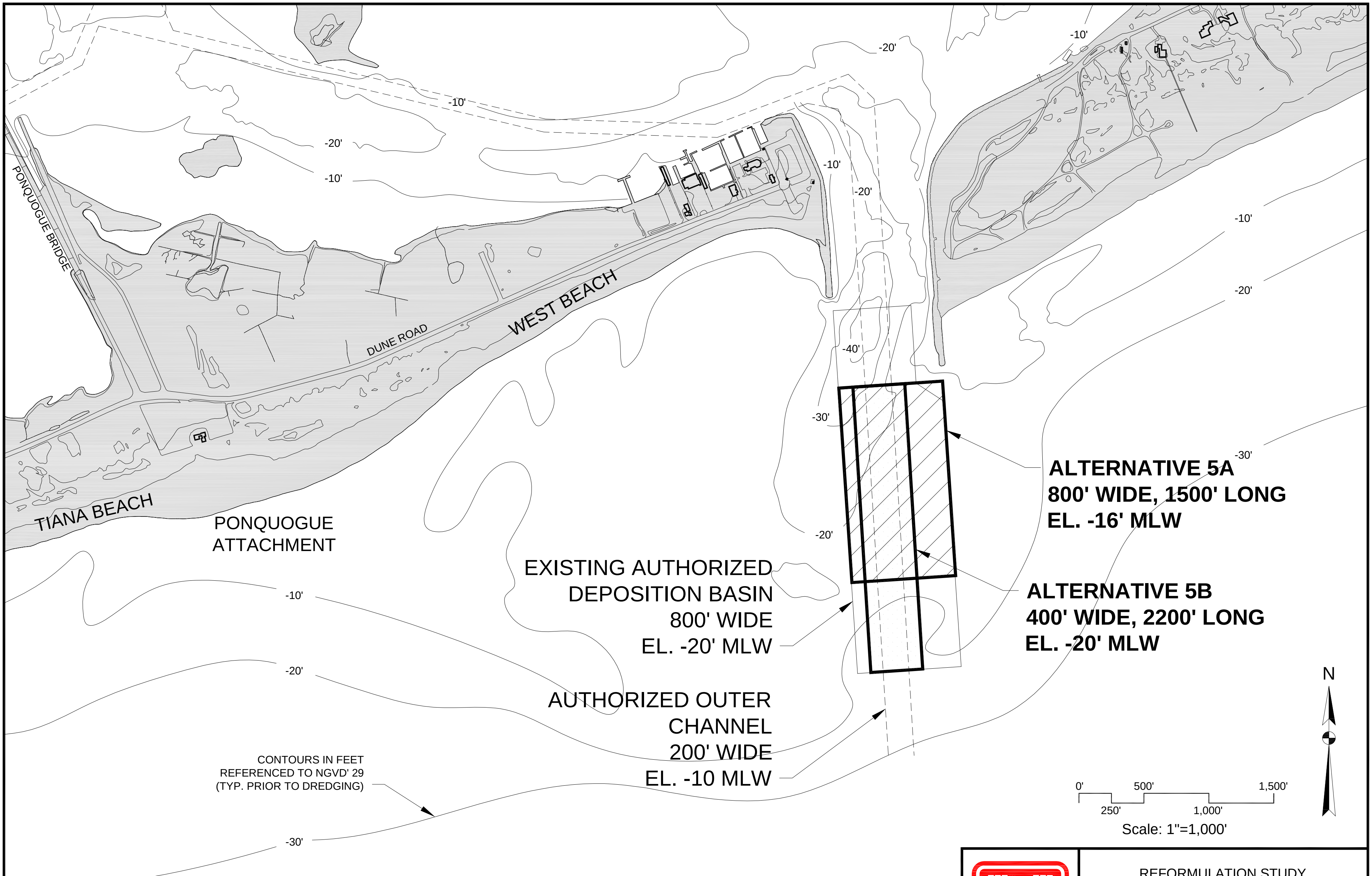
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 8-10



SHINNECOCK INLET - Alternative 5
REDUCED DIMENSIONS OF THE DEPOSITION BASIN

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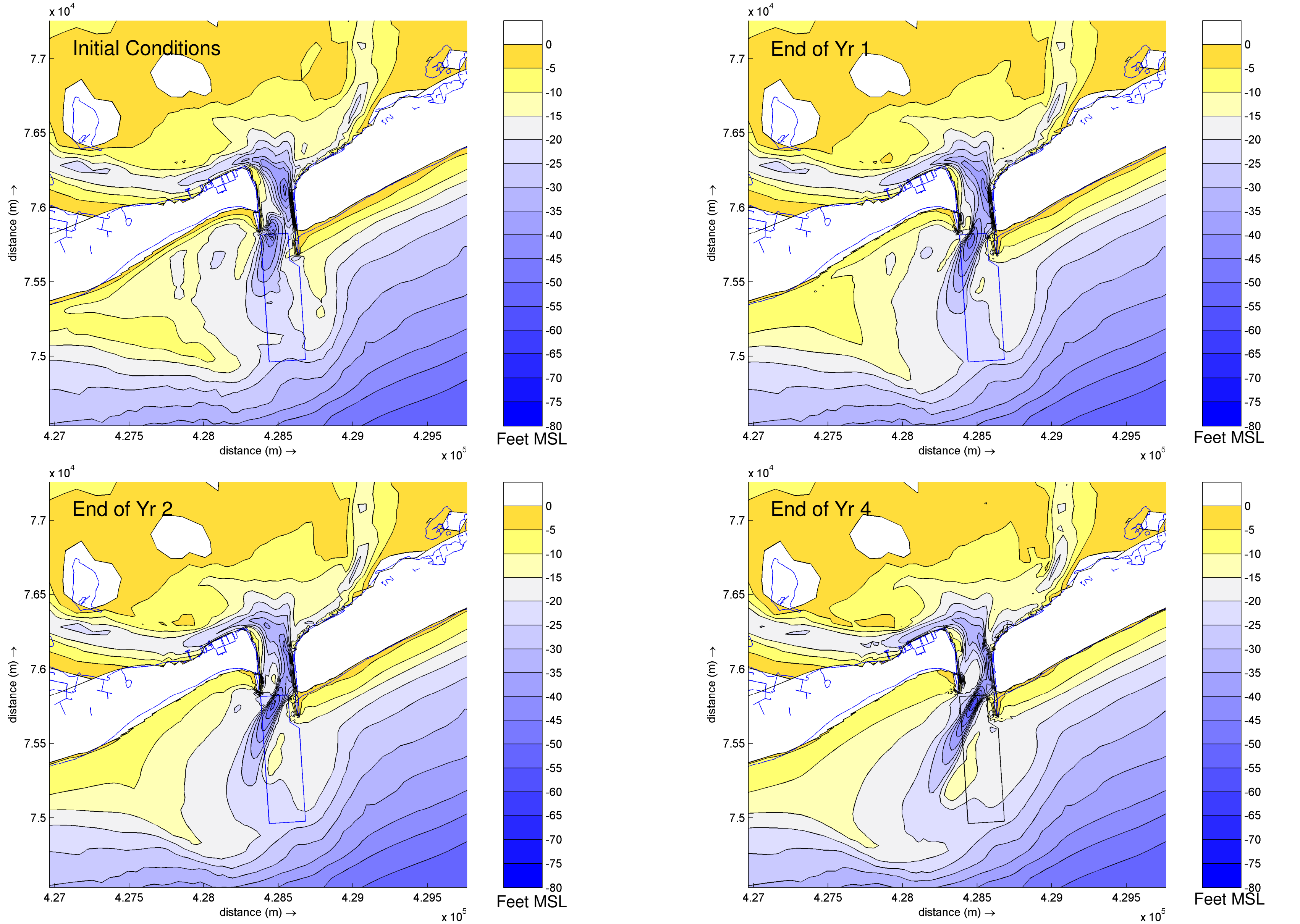
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 8-11

Filename: P:\3982-28\CAD\dwg\SI ALT No. 5.dwg (Shinnecock)



SI Alt. 5. Reduced Dimensions of Deposition Basin: Modeled Morphological Evolution

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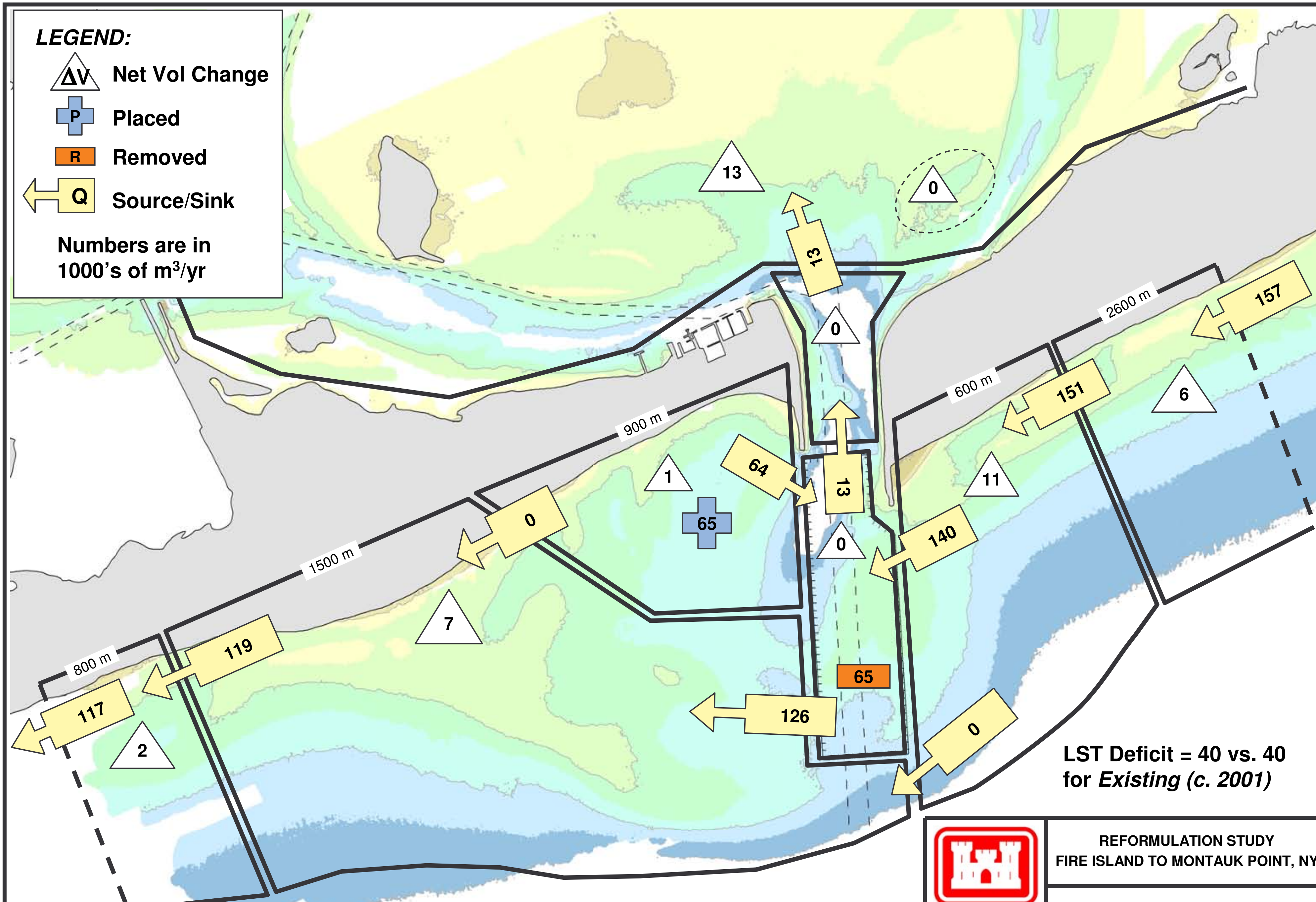
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 8-12



SI Alternative 5: Reduced Dimensions of Deposition Basin

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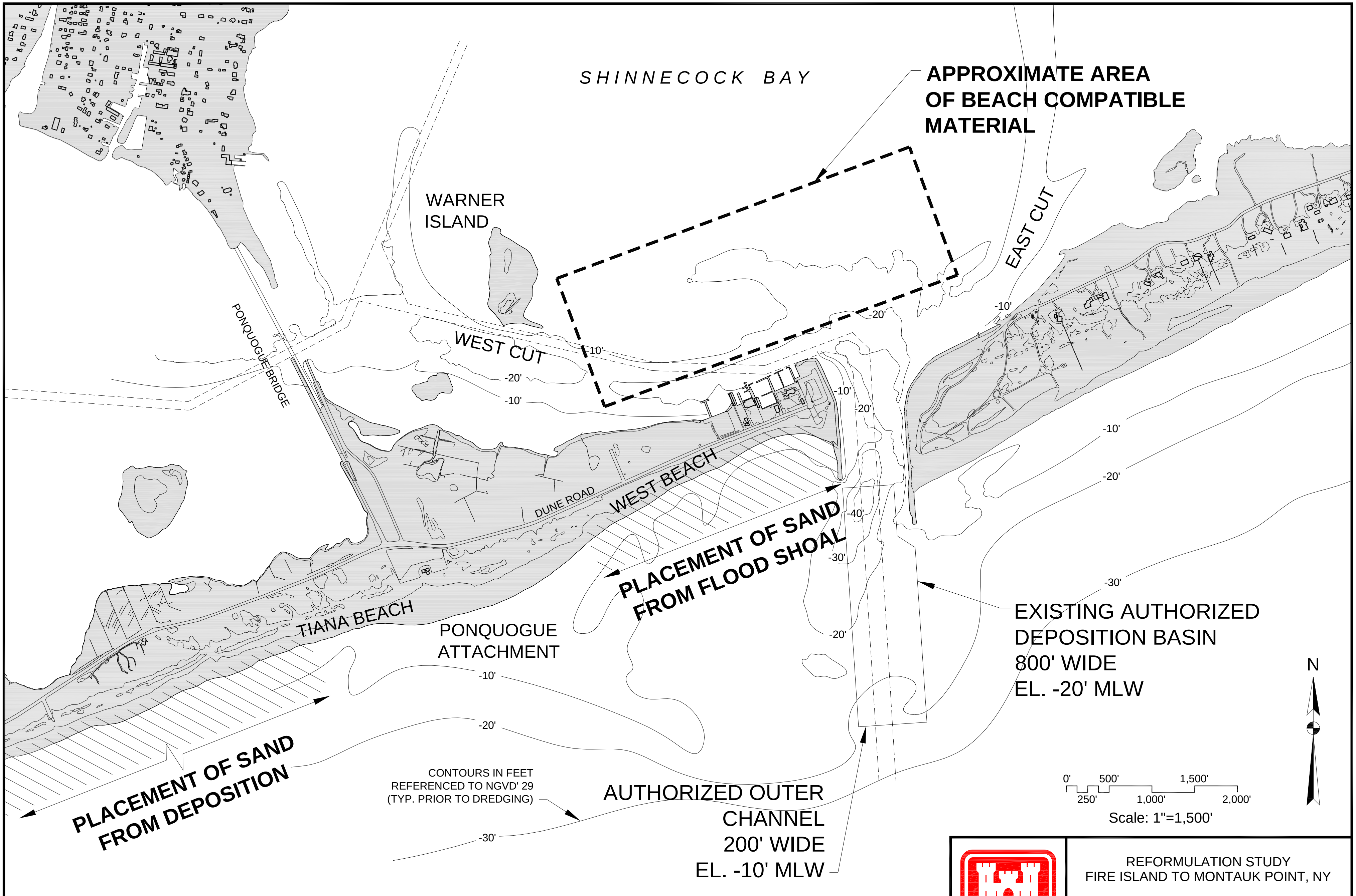
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 8-13

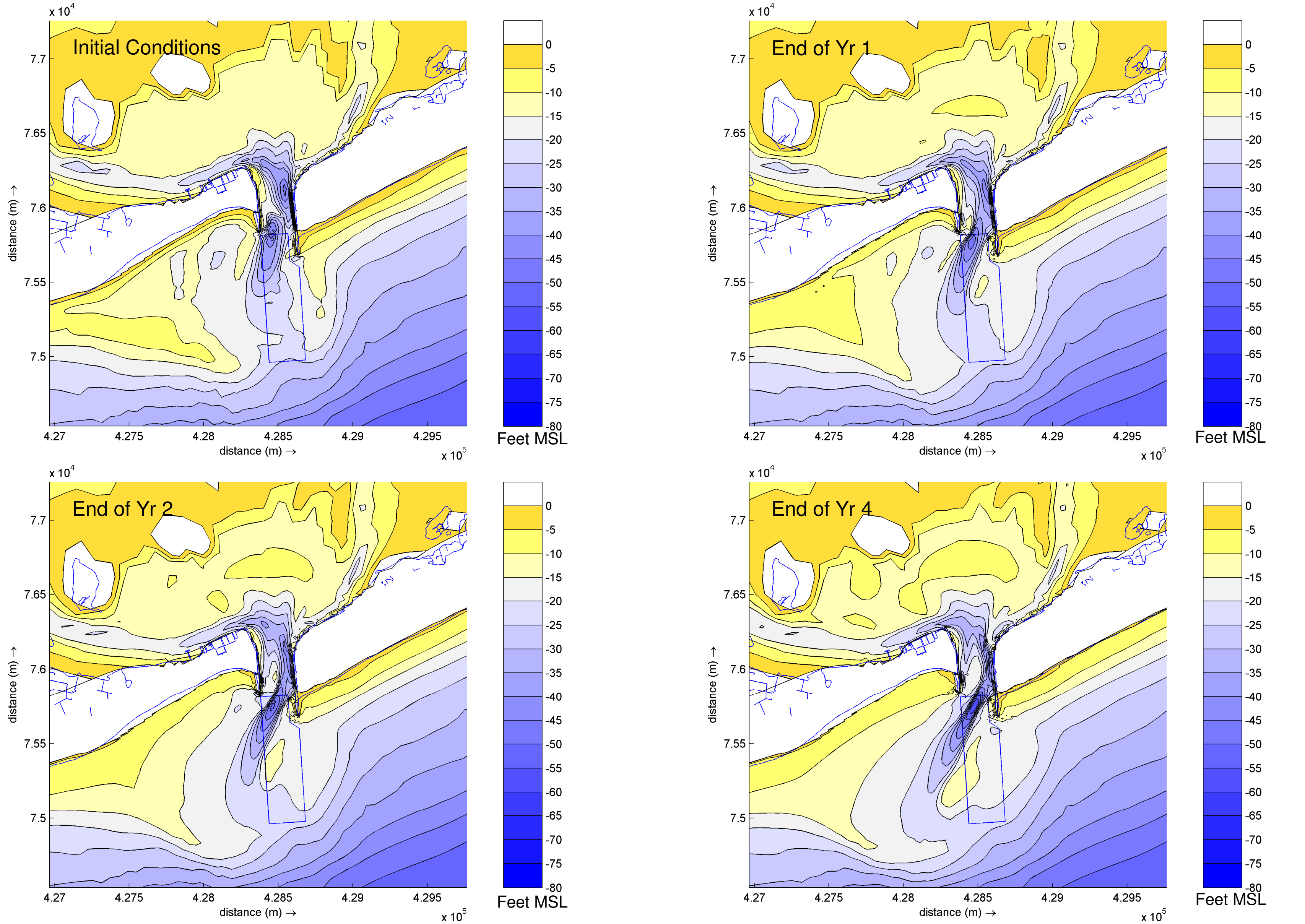


SHINNECOCK INLET - Alternative 6
EXISTING PRACTICE PLUS DREDGING THE FLOOD SHOAL

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 8-14

Filename: P:\3982-28\CAD\dwg\SI ALT No. 6.dwg (Shinnecock)



SI Alt. 6. Dredging the Flood Shoal: Modeled Morphological Evolution

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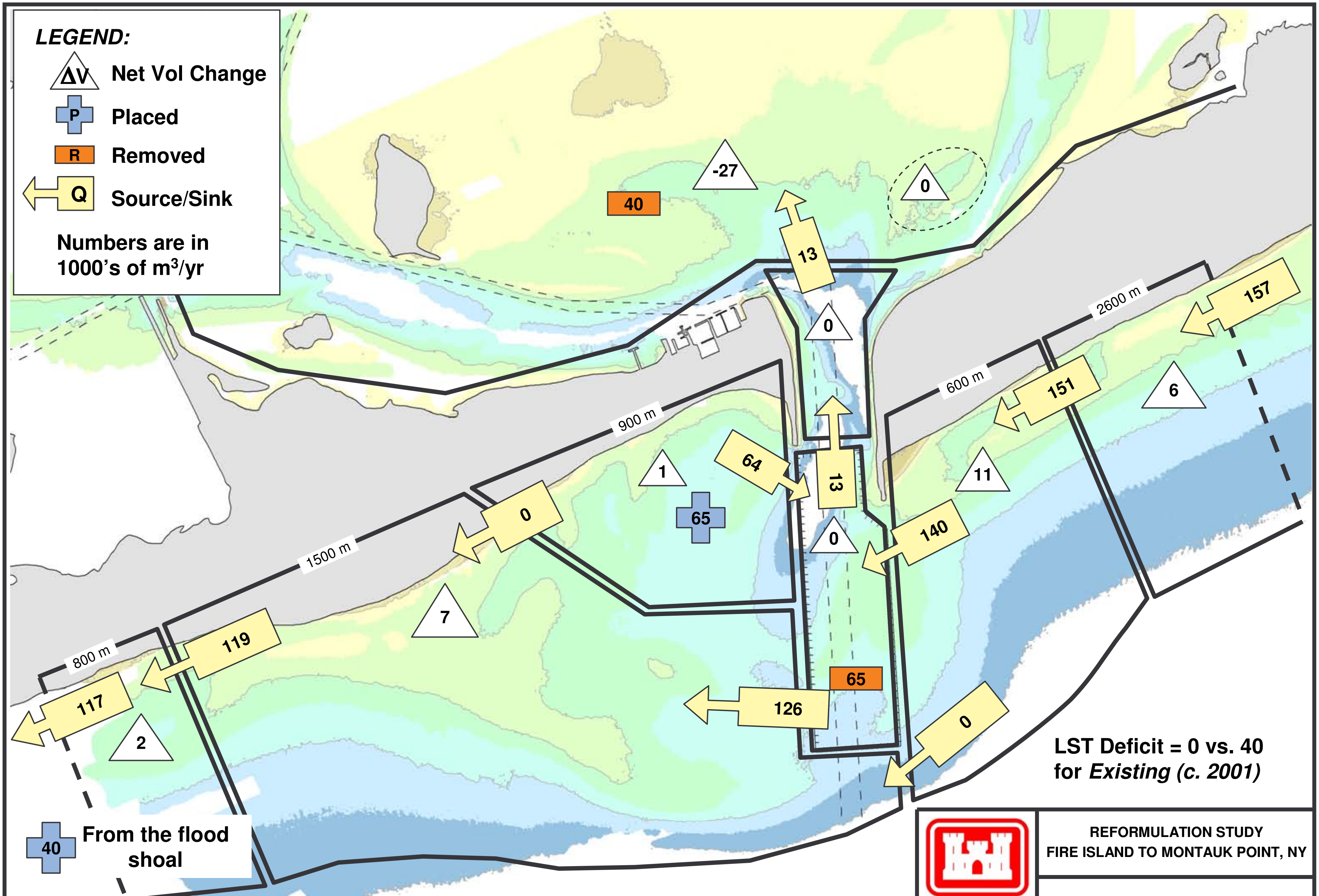
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

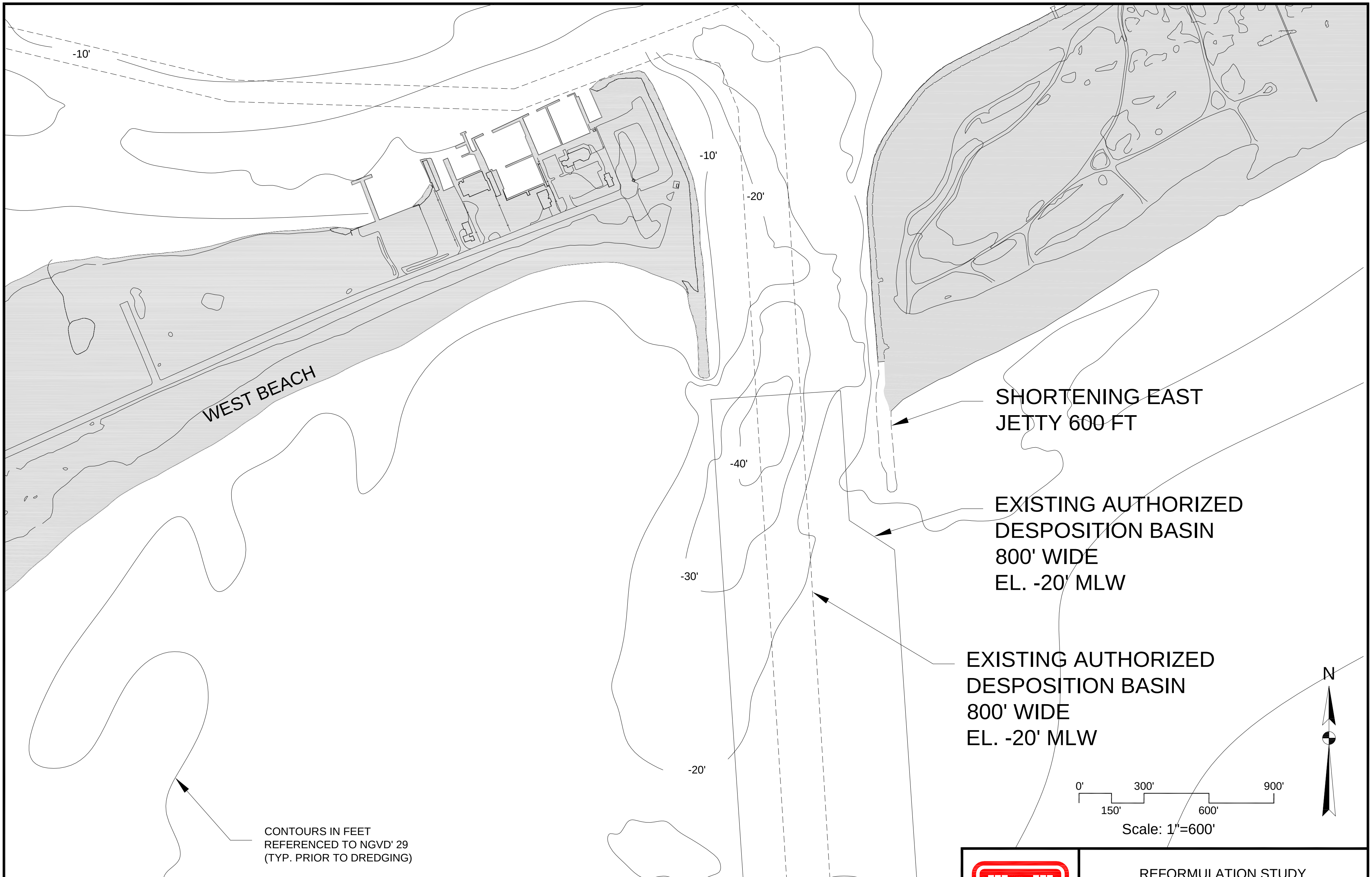
FIGURE 8-15



SI Alternative 6: AP + Dredging the Flood Shoal

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 8-16



SHINNECOCK INLET - Alternative 7
EXISTING PRACTICE PLUS SHORTENING EAST JETTY

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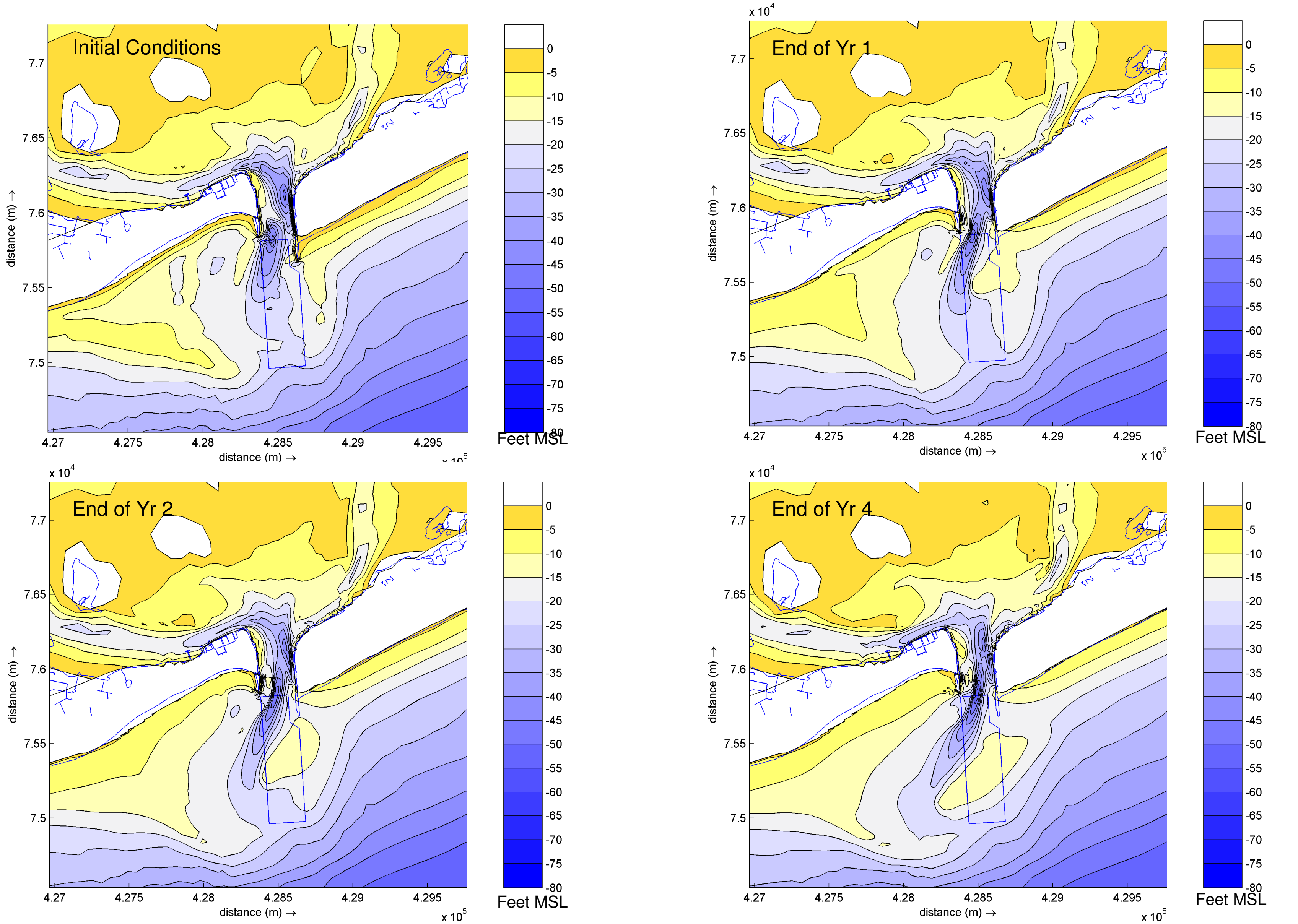
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 8-17



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FIRE ISLAND TO MONTAUK POINT, NY

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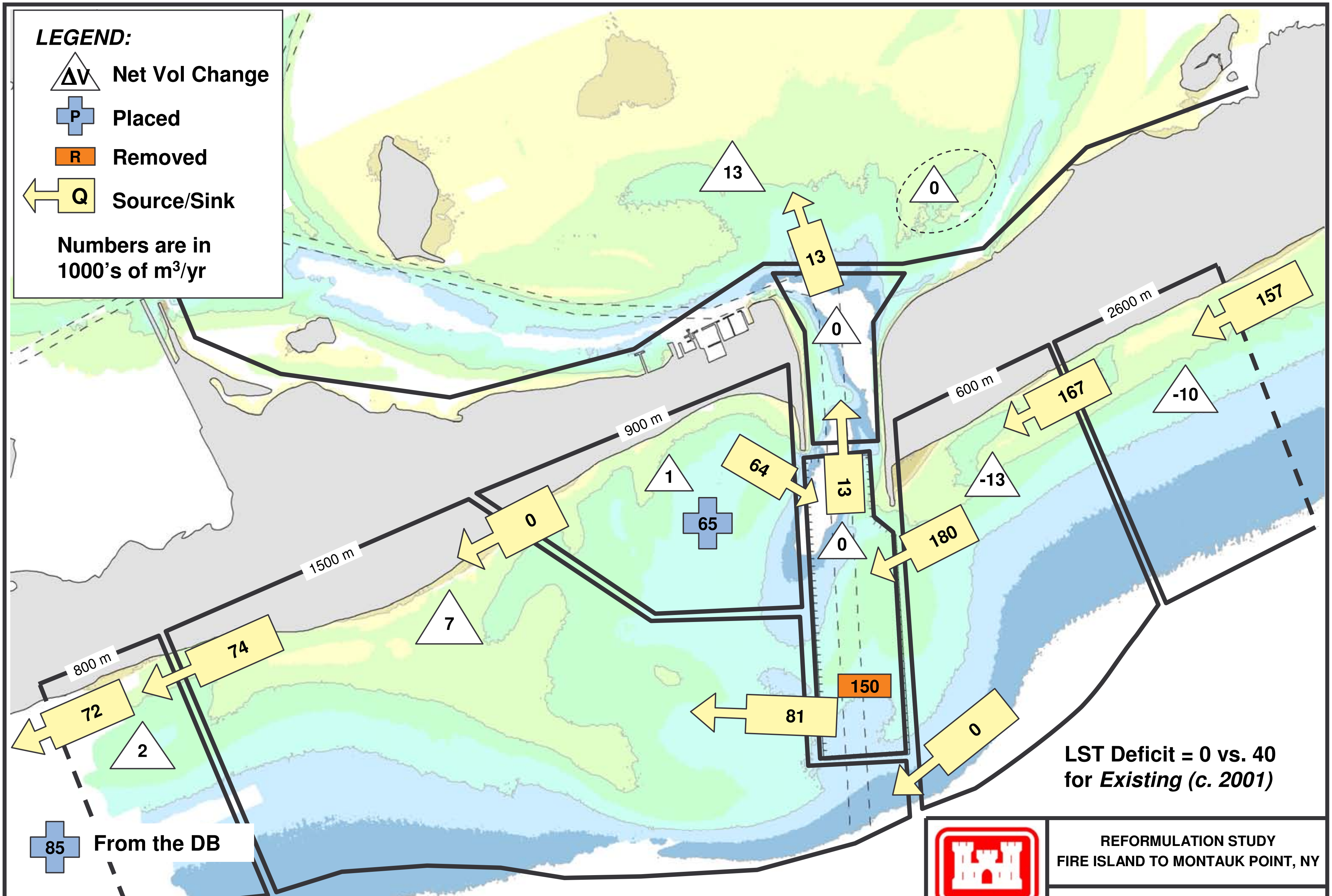
FIGURE 8-18

SI Alt. 7. Shortening the East Jetty: Modeled Morphological Evolution



SI Alternative 7: AP + Shortening the East Jetty

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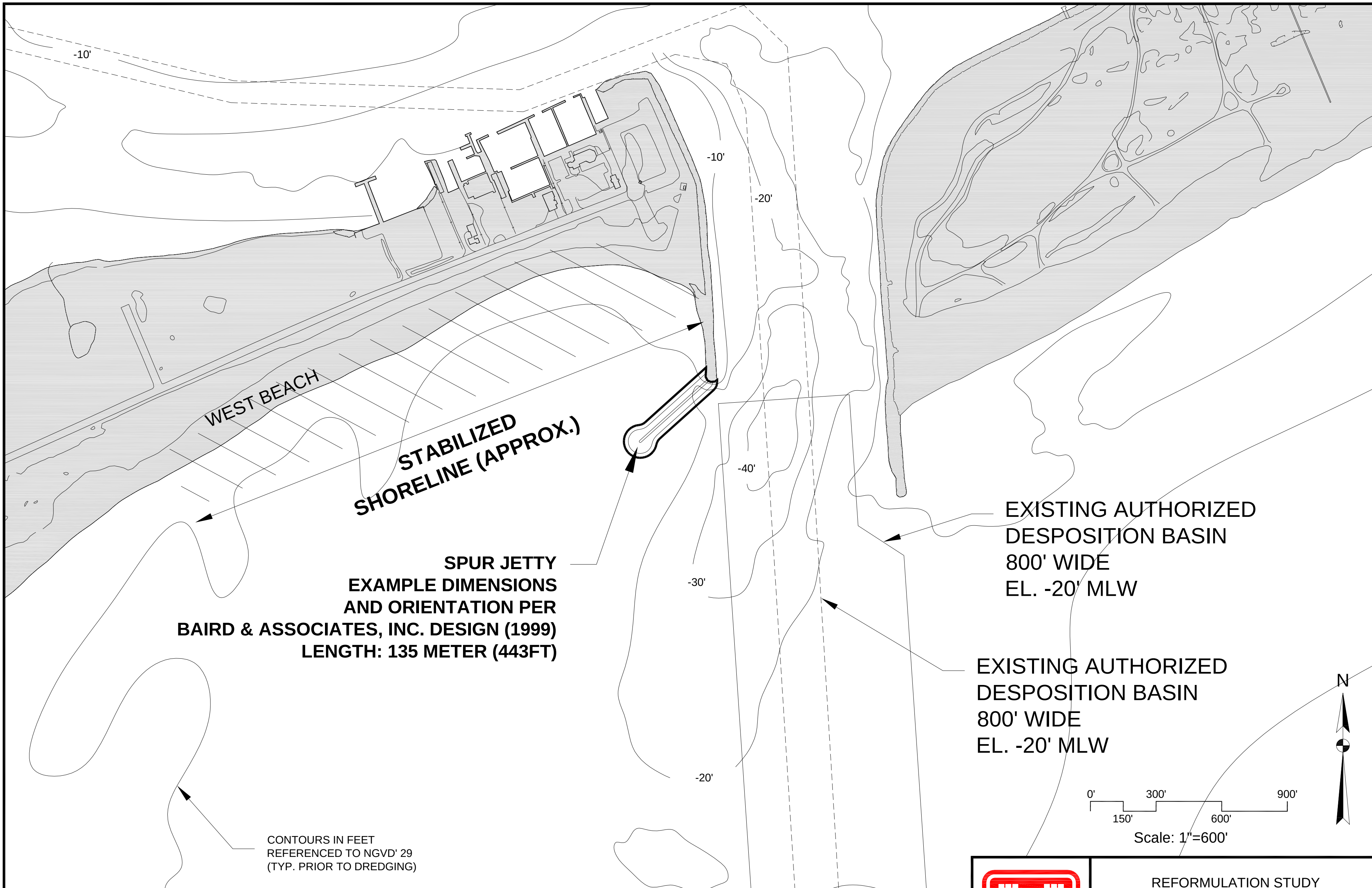
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 8-19



SHINNECOCK INLET - Alternative 8
EXISTING PRACTICE PLUS SPUR JETTY (WEST)

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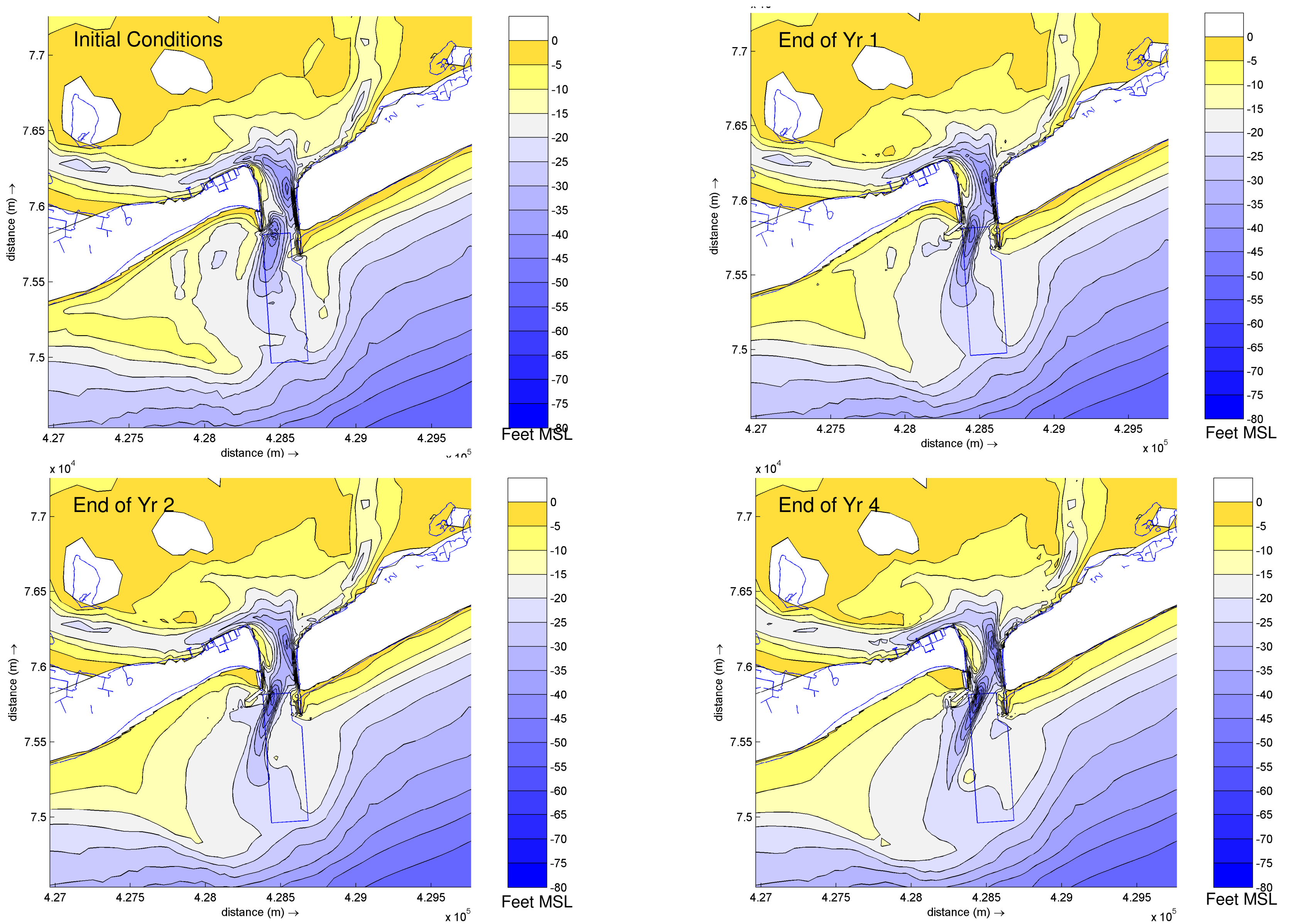
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 8-20

Filename: P:\3982-28\CAD\dwg\SI ALT No. 8.dwg (Shinnecock)



SI Alt. 8. West Jetty Spur: Modeled Morphological Evolution

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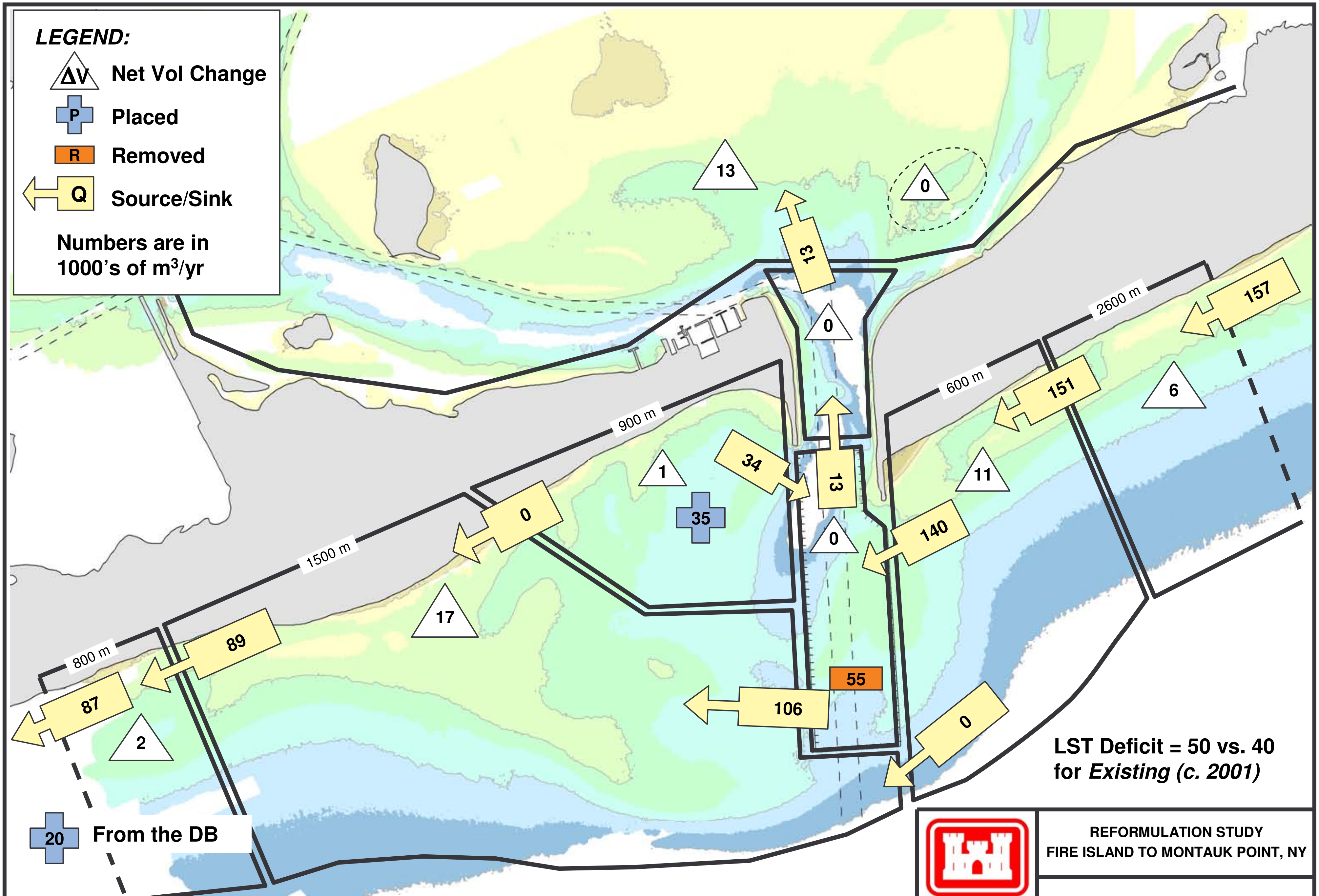
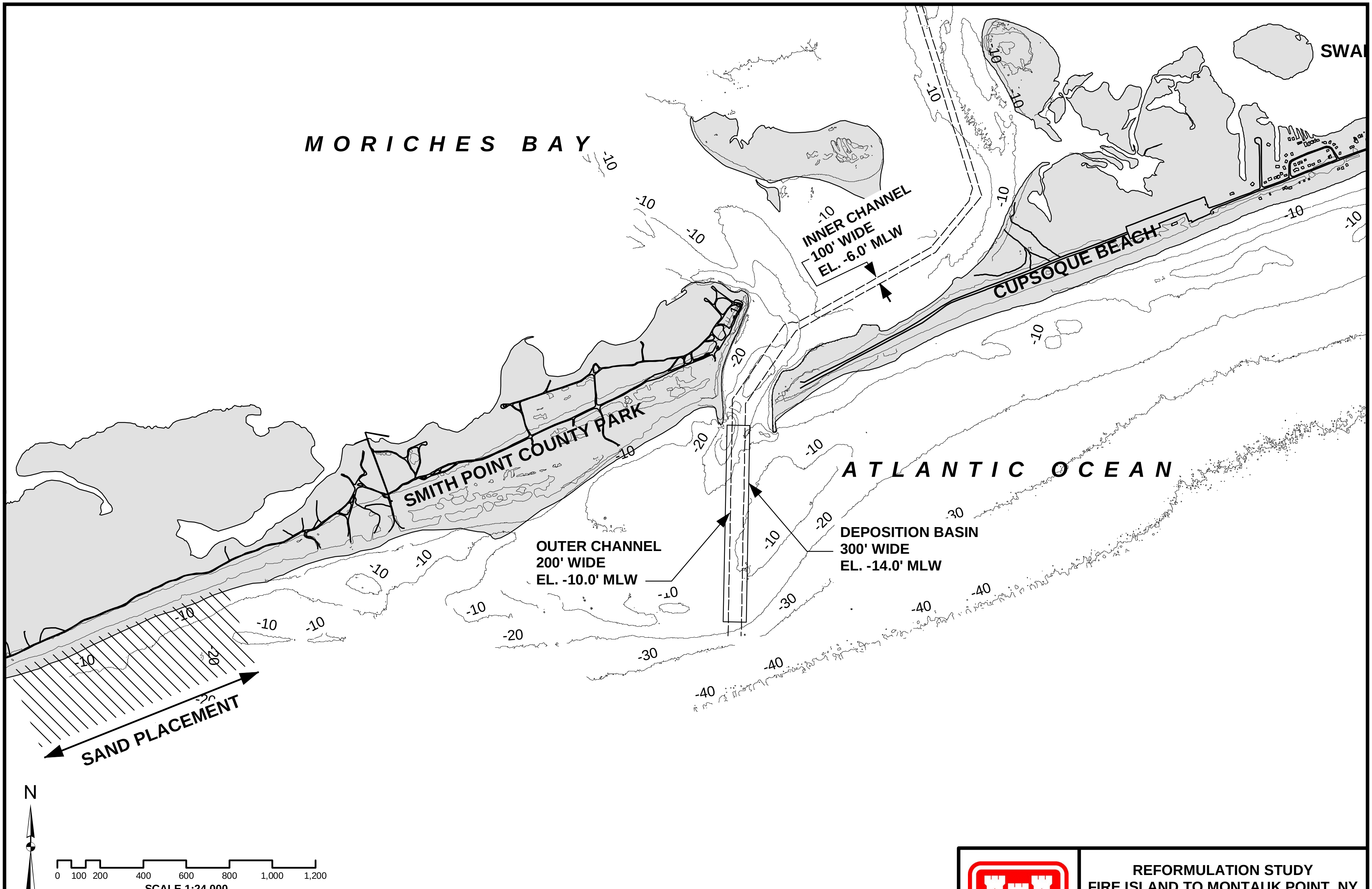
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 8-21



SI Alternative 8: AP + West Jetty Spur



MORICHES INLET - Alternative 1
Authorized Project (AP)

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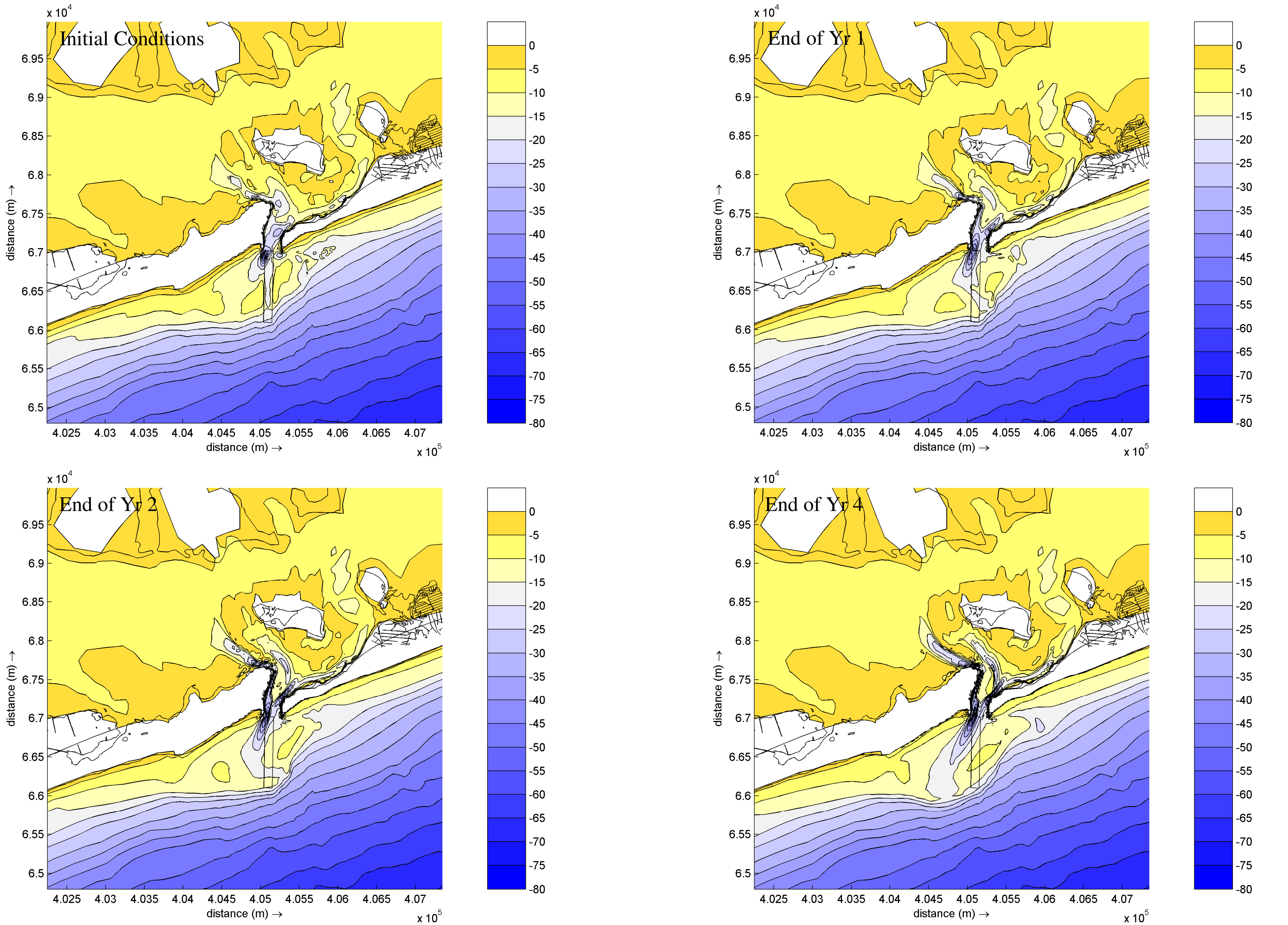
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 8-23

Filename: P:\3982-28\Work\Figures-Inlet Runs\Alternatives\Selected Inlet Alternatives\Moriches\MXD\M_ALT1_APD.mxd



MI Alt. 1 Authorized Project: Modeled Morphological Evolution

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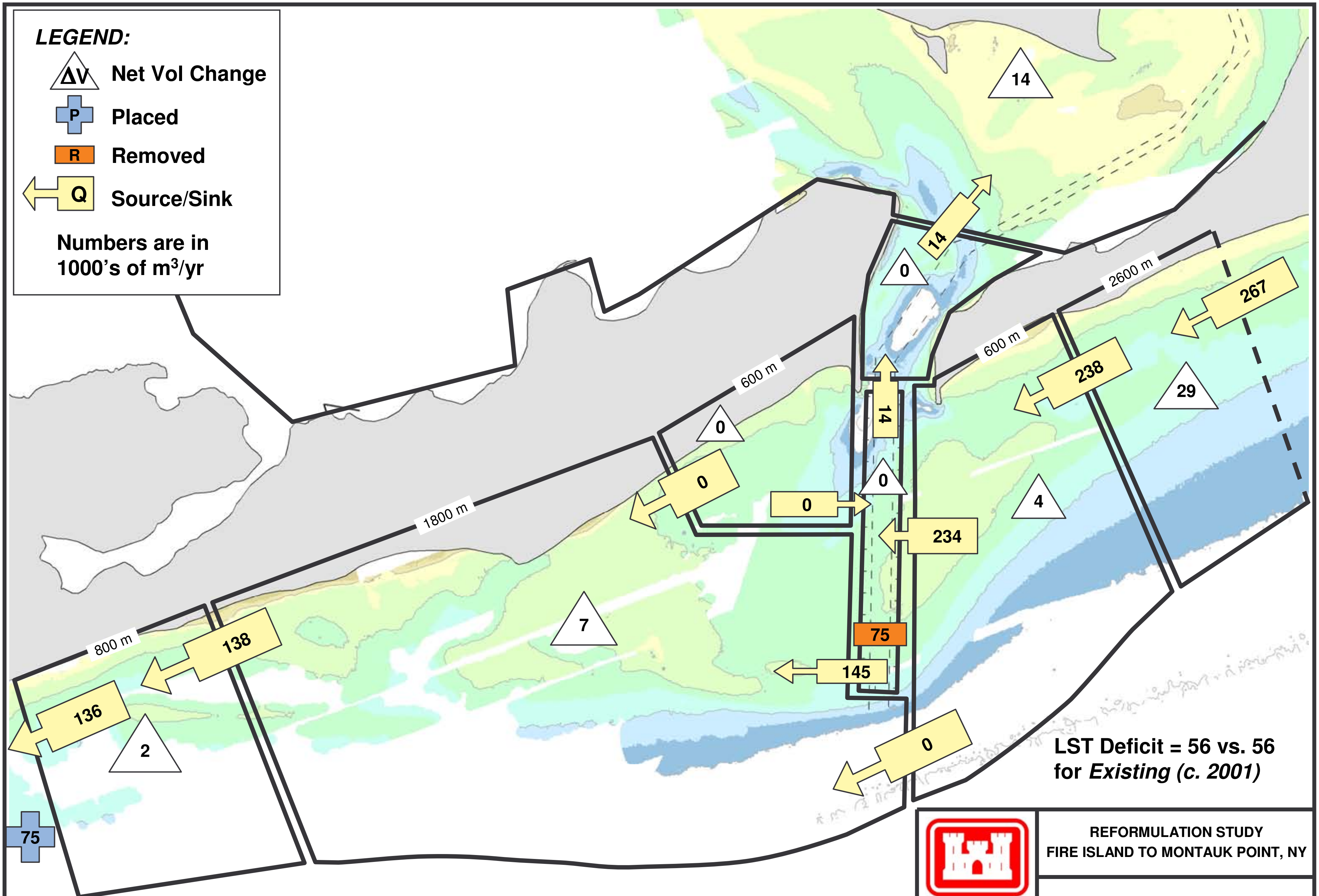
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 8-24



MI Alt. 1: Authorized Project Dimensions

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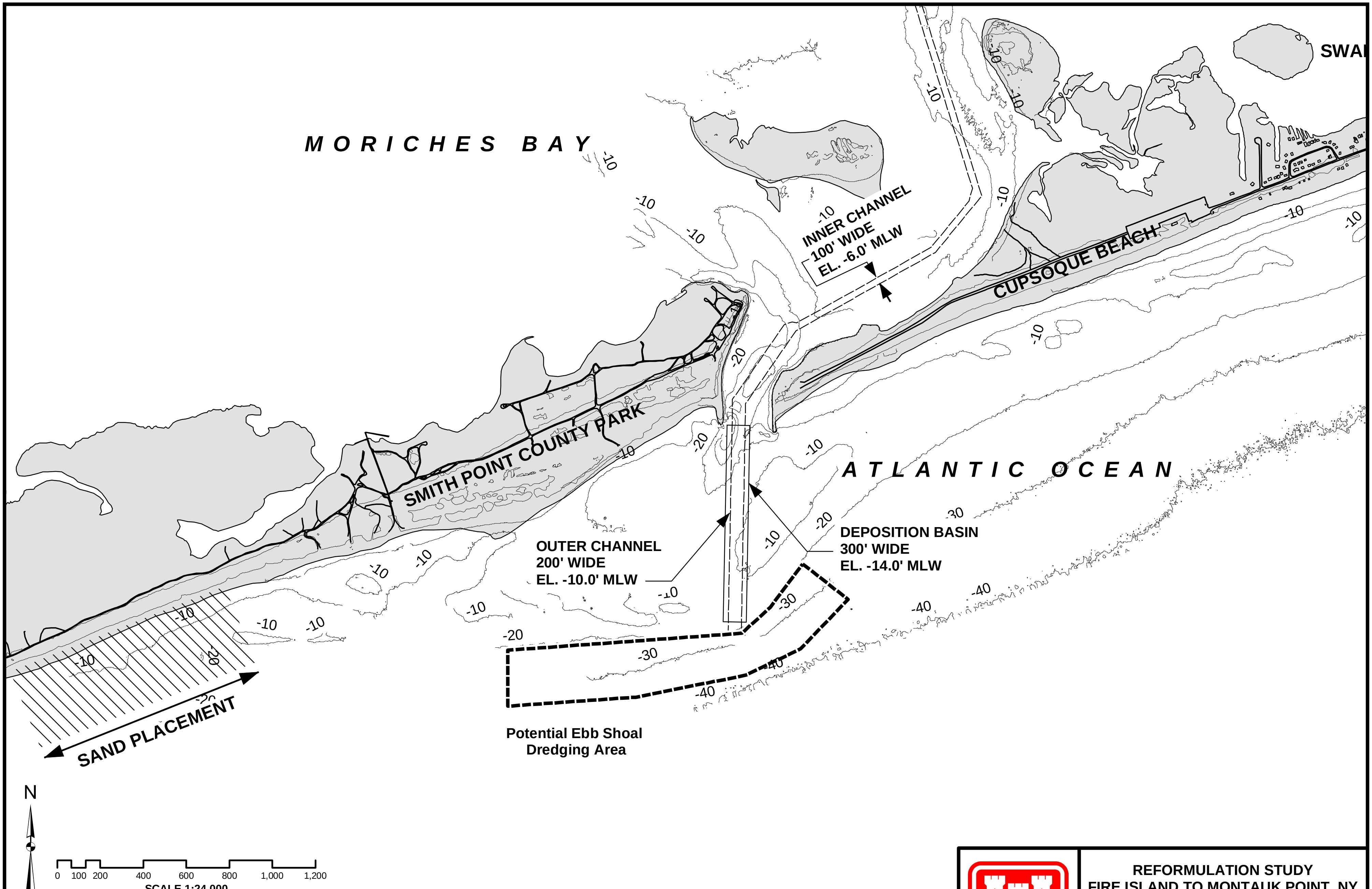
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 8-25



MORICHES INLET - Alternative 2
AP + Dredging the Ebb Shoal

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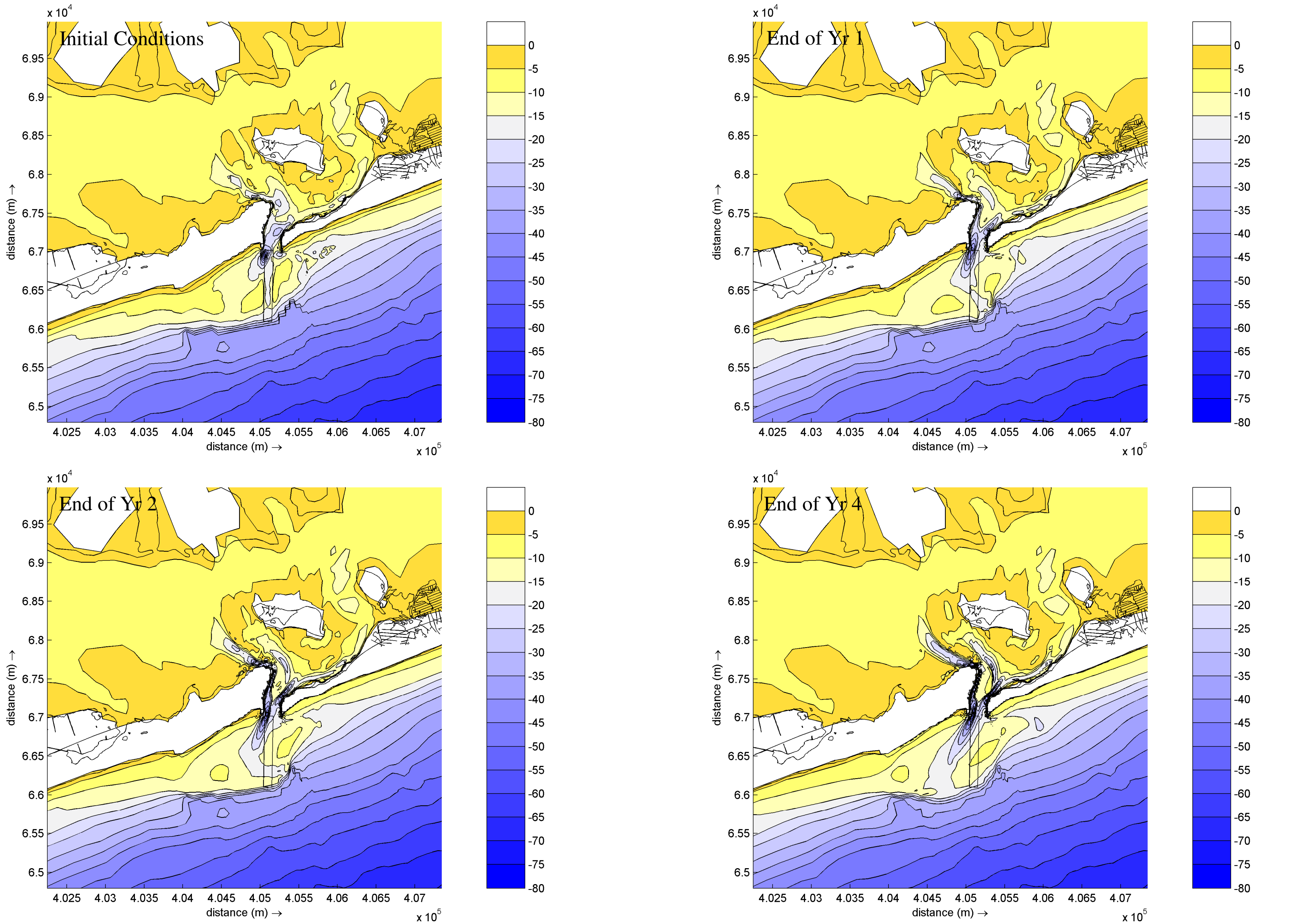
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 8-26

Filename: P:\3982-28\Work\Figures-Inlet Runs\Alternatives\Selected Inlet Alternatives\Moriches\MXD\M_ALT2_EbbShoal.mxd



MI Alt. 2 Dredging the Ebb Shoal: Modeled Morphological Evolution

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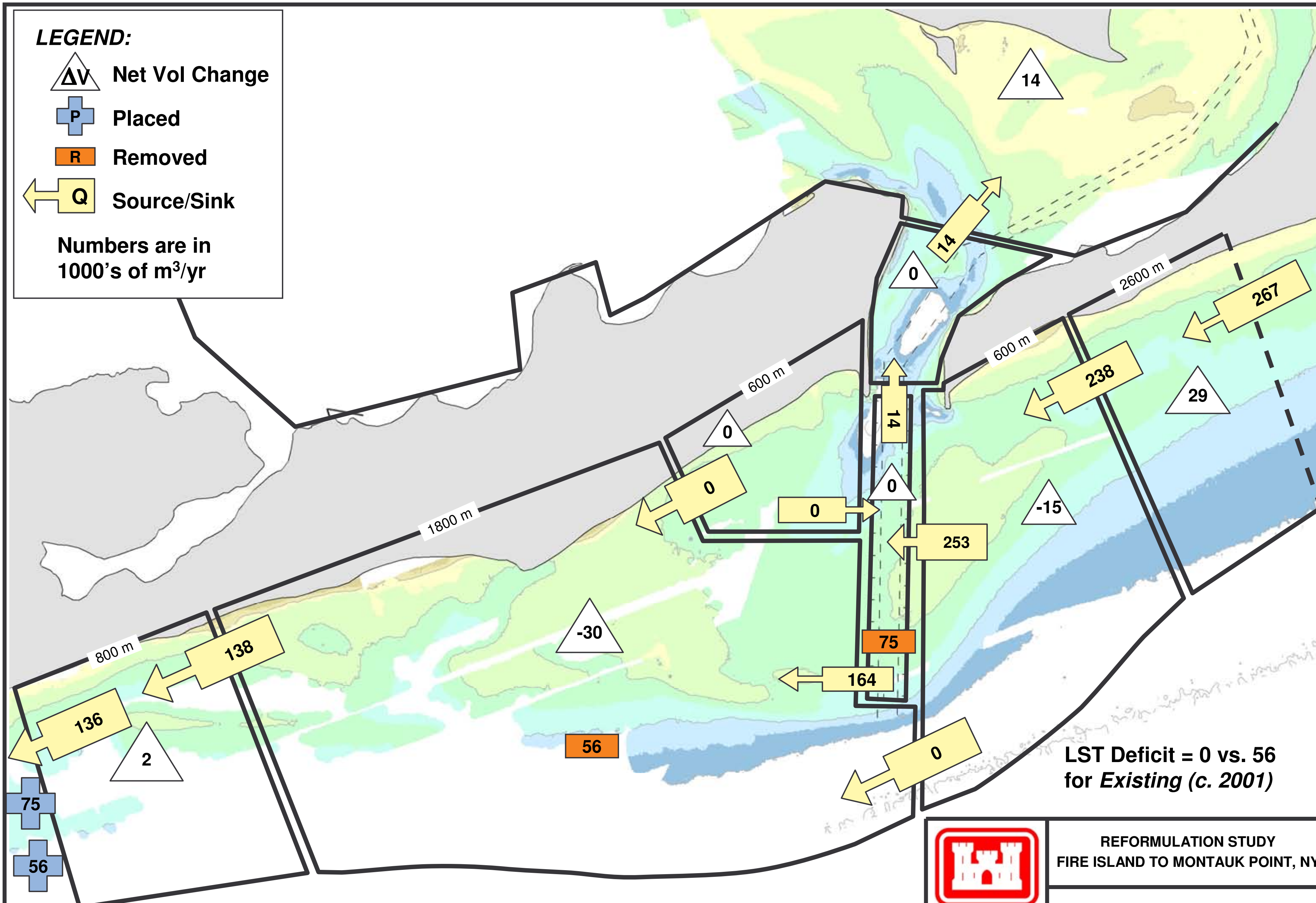
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 8-27



REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

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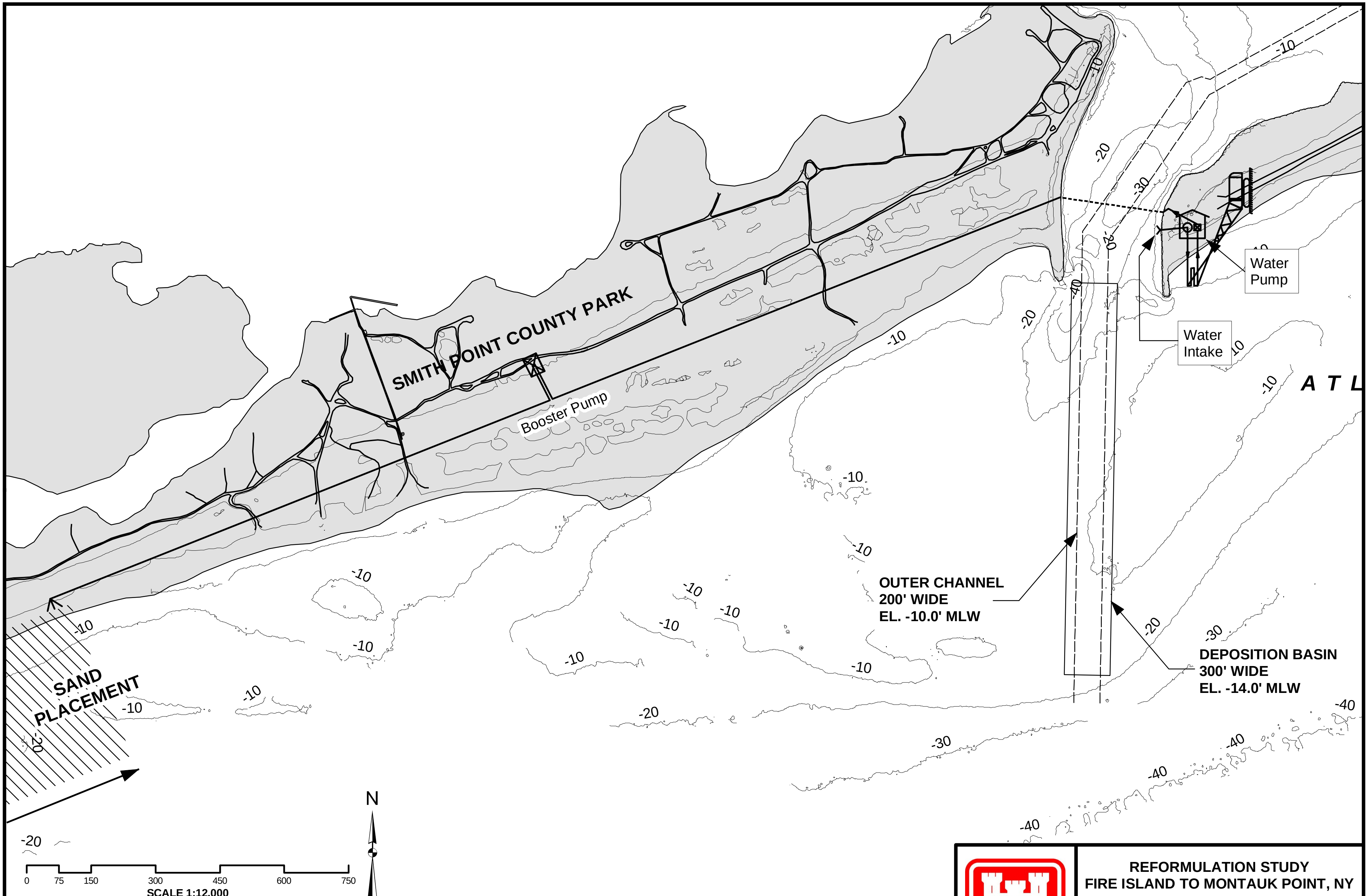
FIGURE 8-28

MI Alt. 2: AP + Dredging the Ebb Shoal



MORICHES INLET - Alternative 3
AP + Semi-Fixed Bypass System

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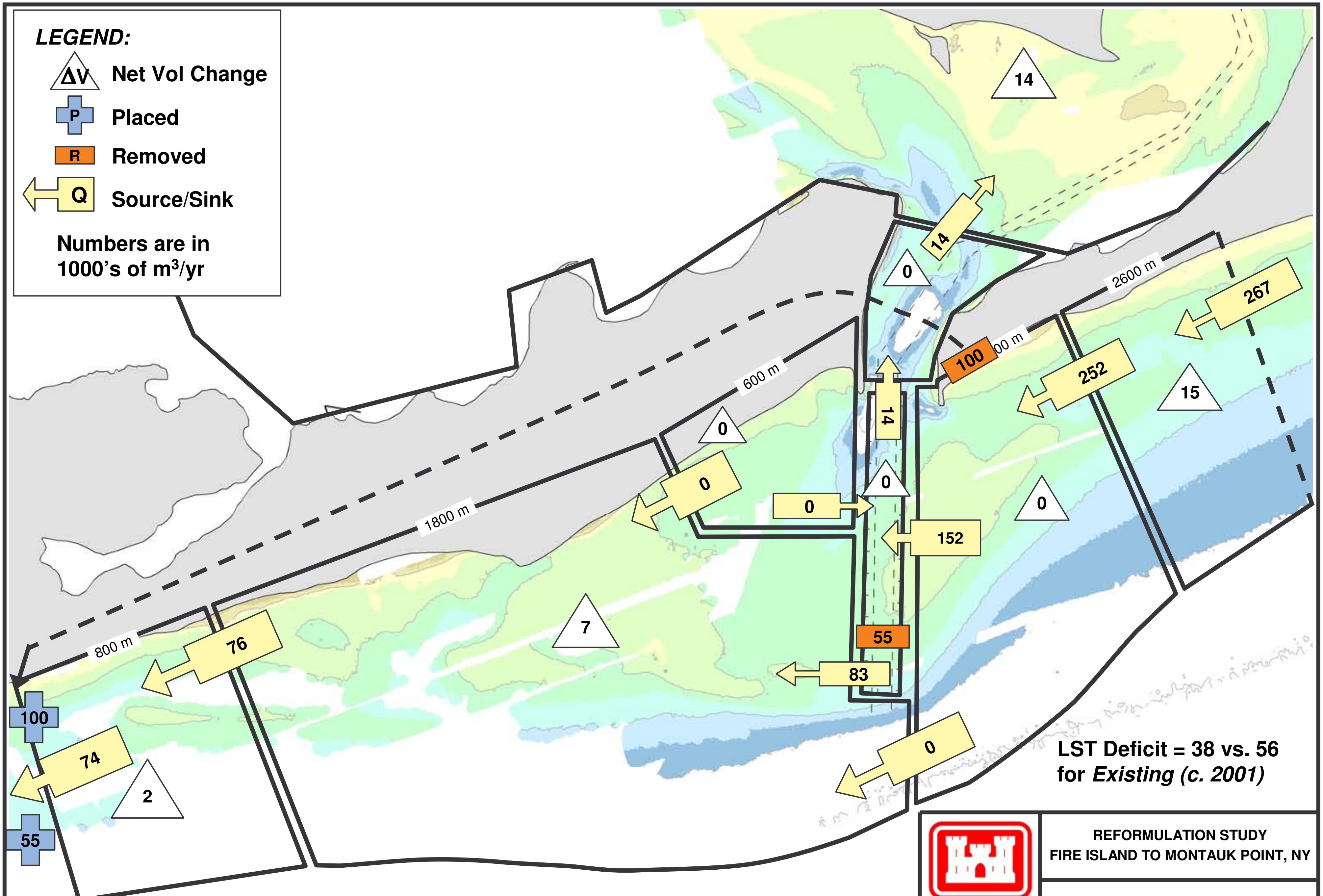
FIRE ISLAND TO MONTAUK POINT, NY

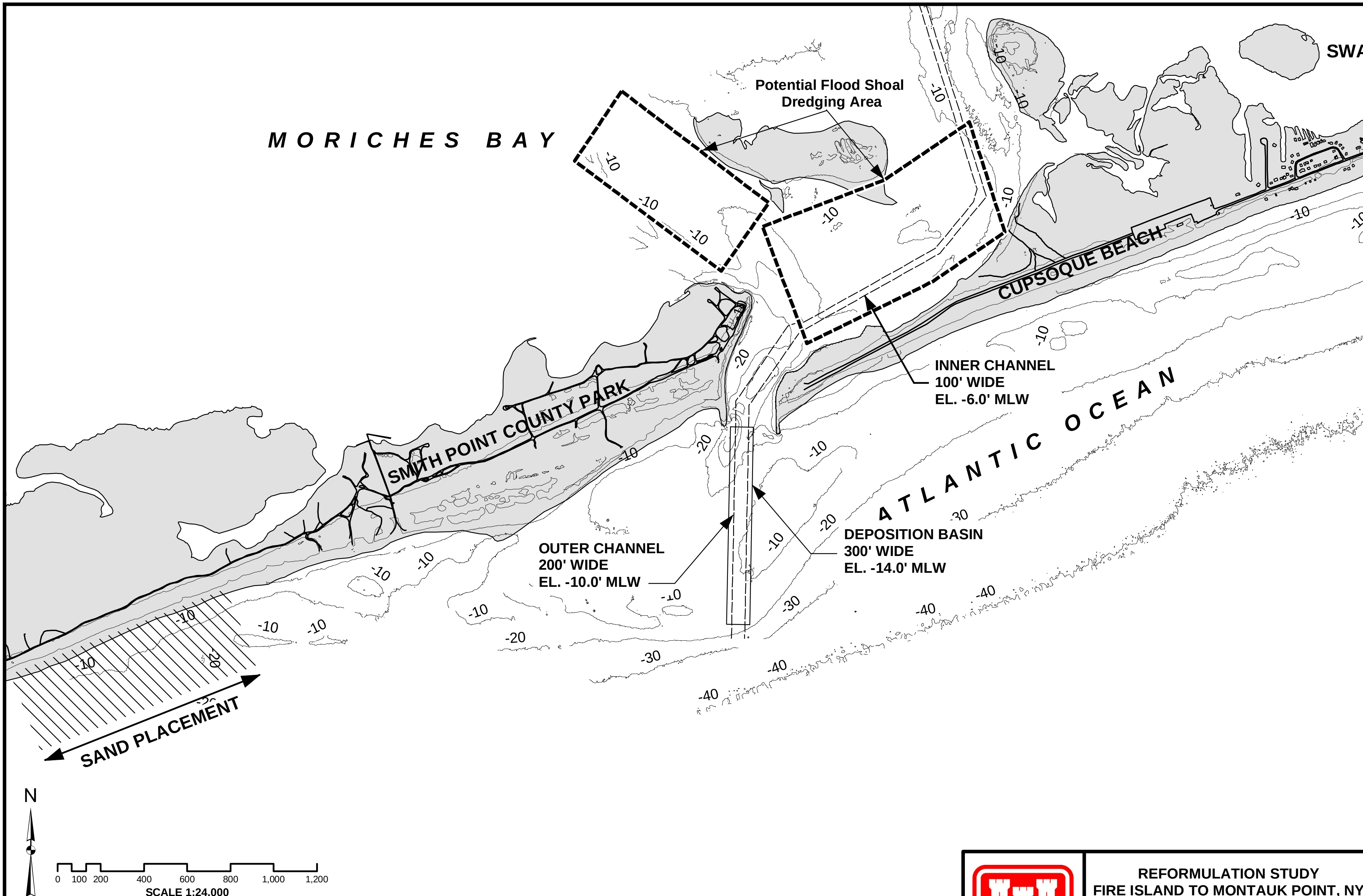
FIGURE 8-29



MI Alt. 3: AP + Semi-fixed Bypass System

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 8-30



MORICHES INLET - Alternative 4
AP + DREDGING THE FLOOD SHOAL

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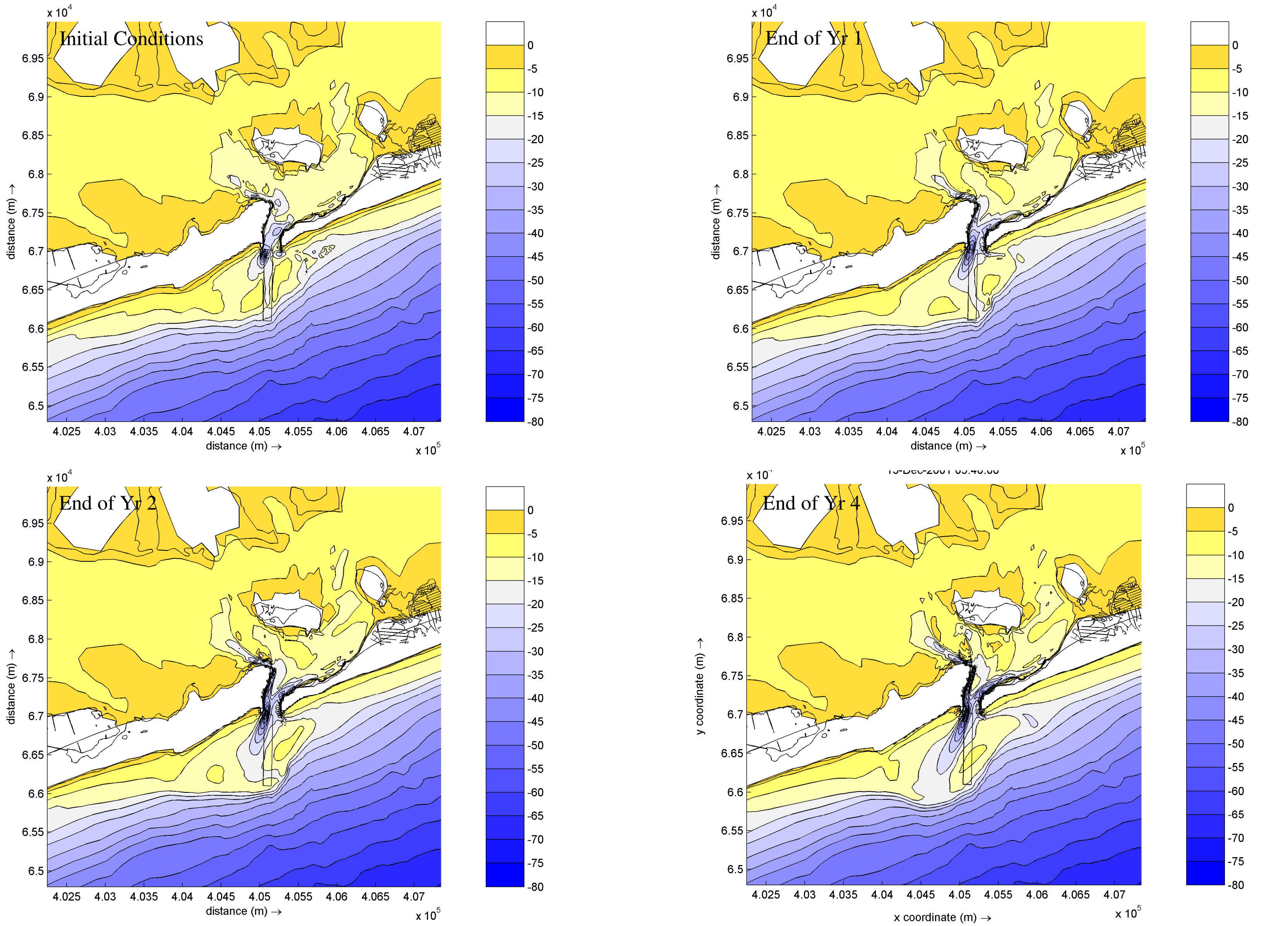
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 8-31

Filename: P:\3982-28\Work\Figures-Inlet Runs\Alternatives\Selected Inlet Alternatives\Moriches\MXD\M_ALT4_FloodShoal.mxd



MI Alt. 4 Dredging the Flood Shoal: Modeled Morphological Evolution

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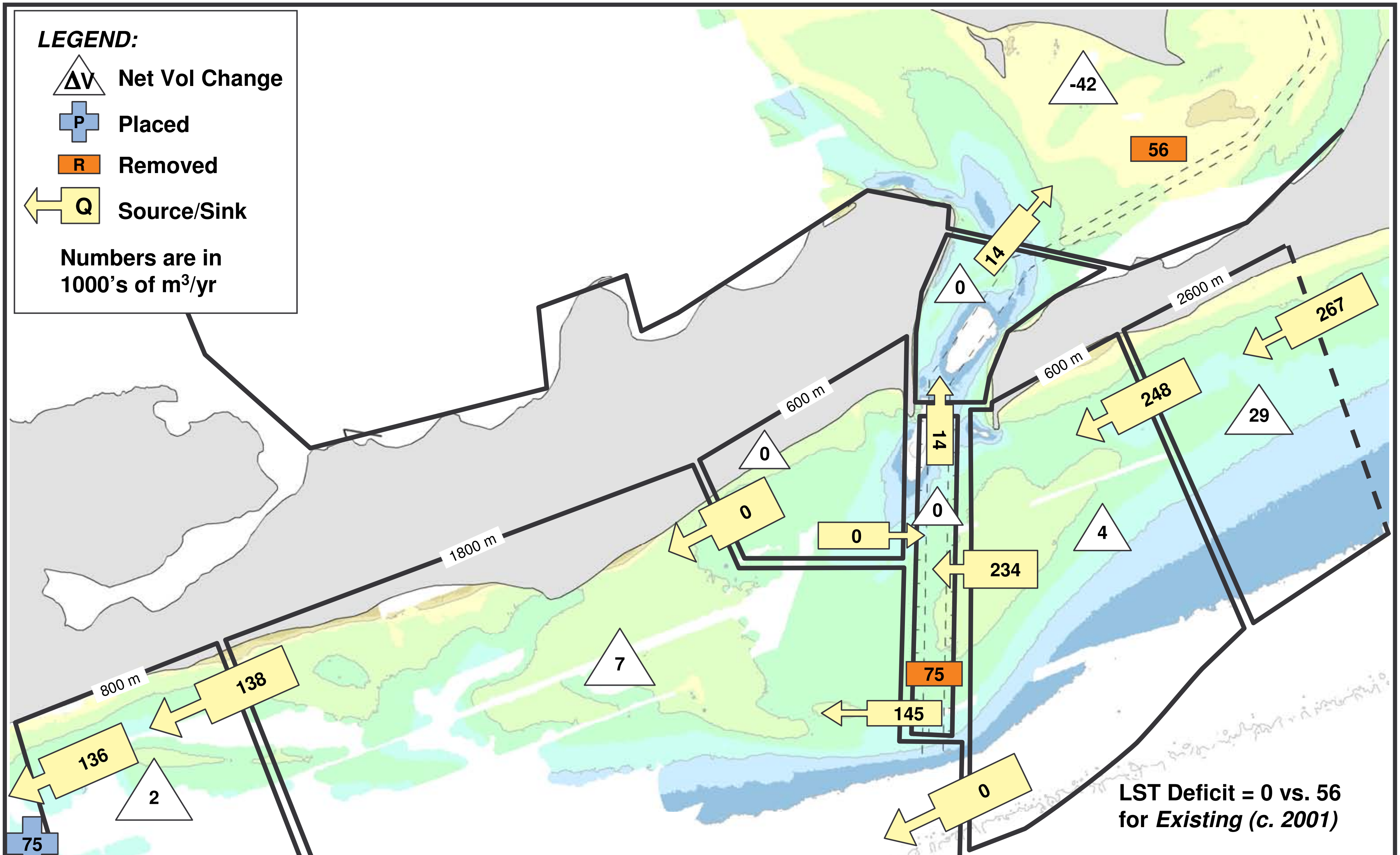
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 8-32



MI Alt. 4: AP + Dredging the Flood Shoal

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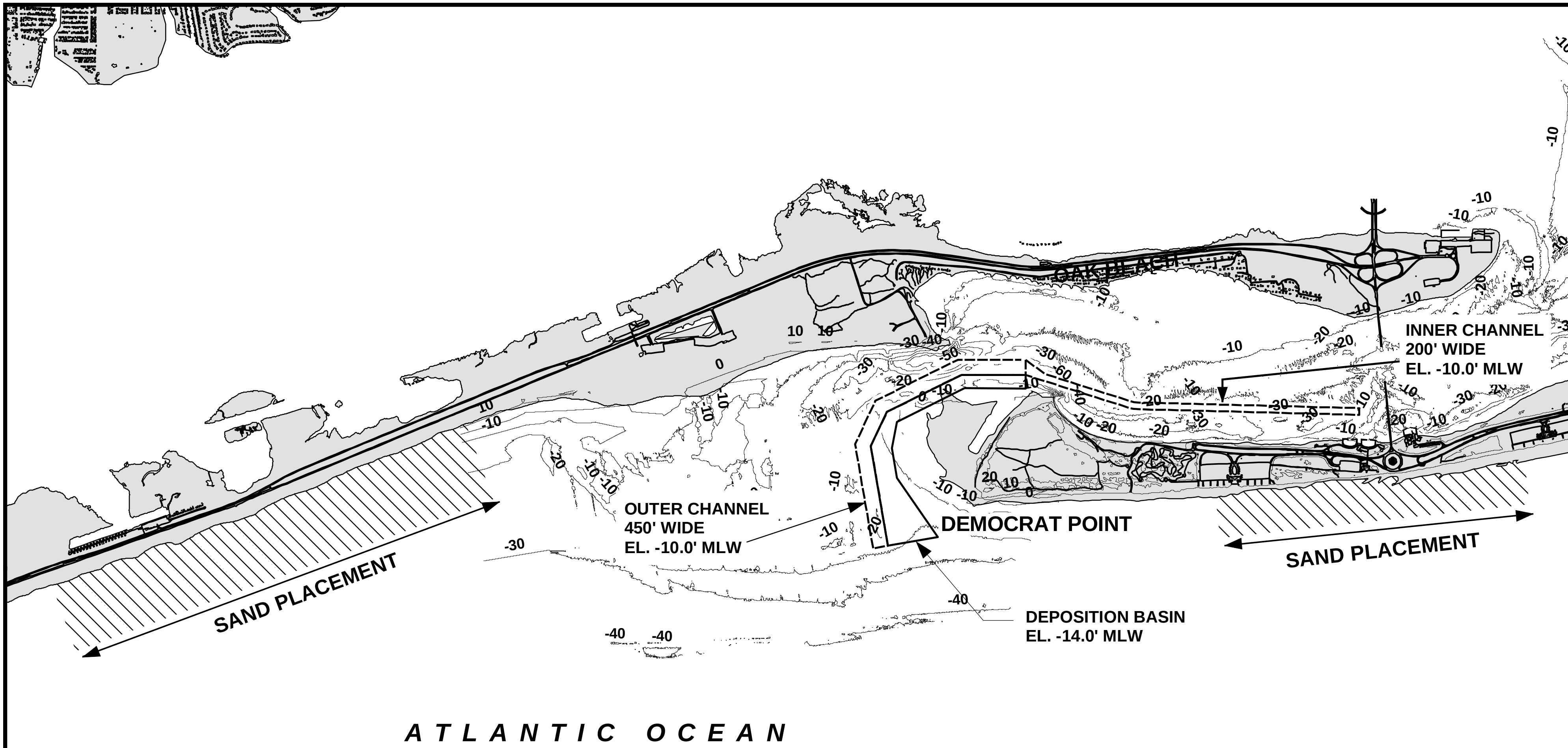
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 8-33



N

0 1,250 2,500 5,000

SCALE 1:54,000

FIRE ISLAND INLET - Alternative 1
Authorized Project Dimension (APD)

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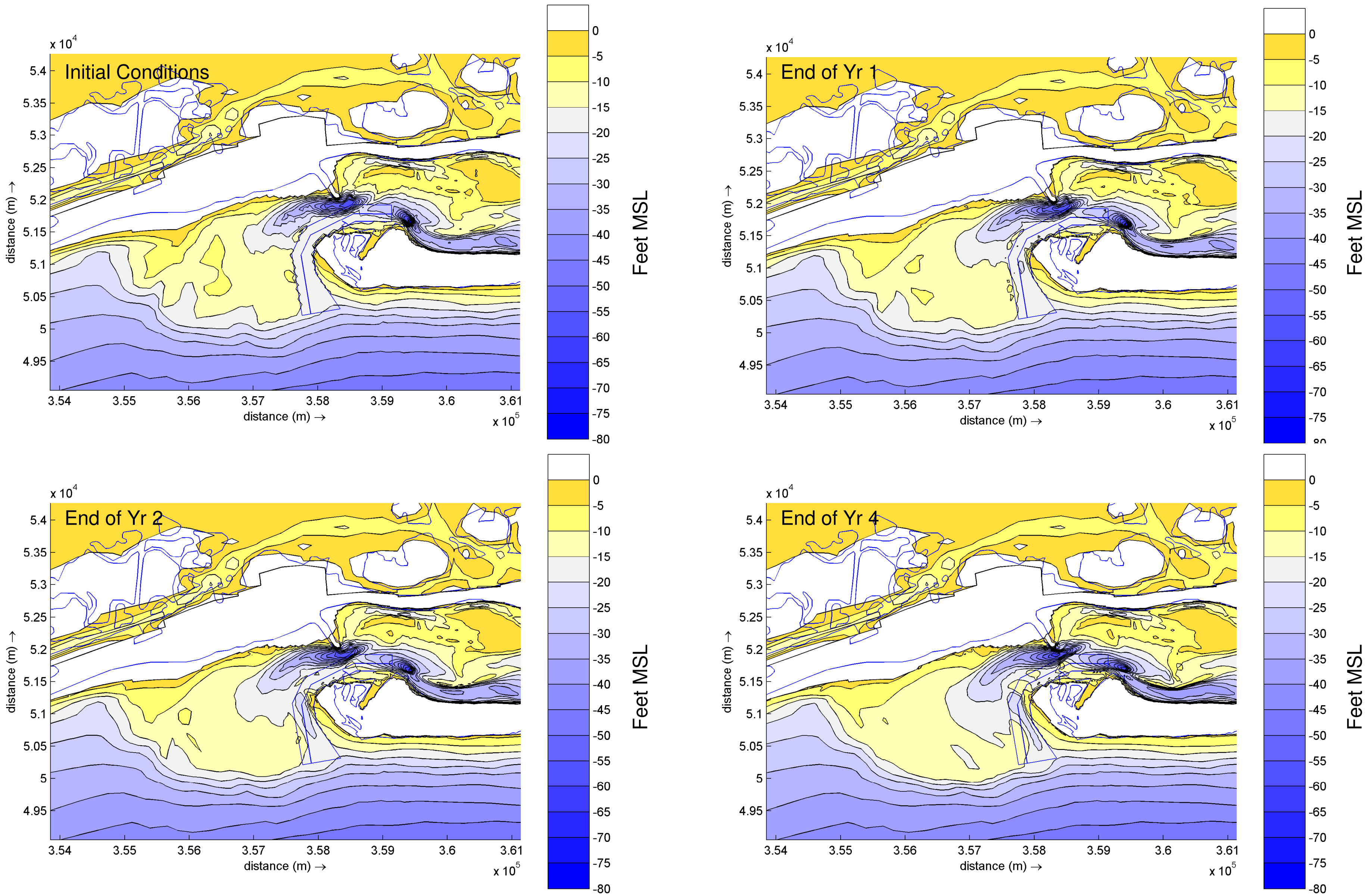
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 8-34

Filename: P:\3982-28\Work\Figures-Inlet Runs\Alternatives\Selected Inlet Alternatives\Moriches\MXD\FI_ALT1_APD.mxd



FII Alt. 1: Authorized Project /Existing Practice: Modeled Morphological Evolution

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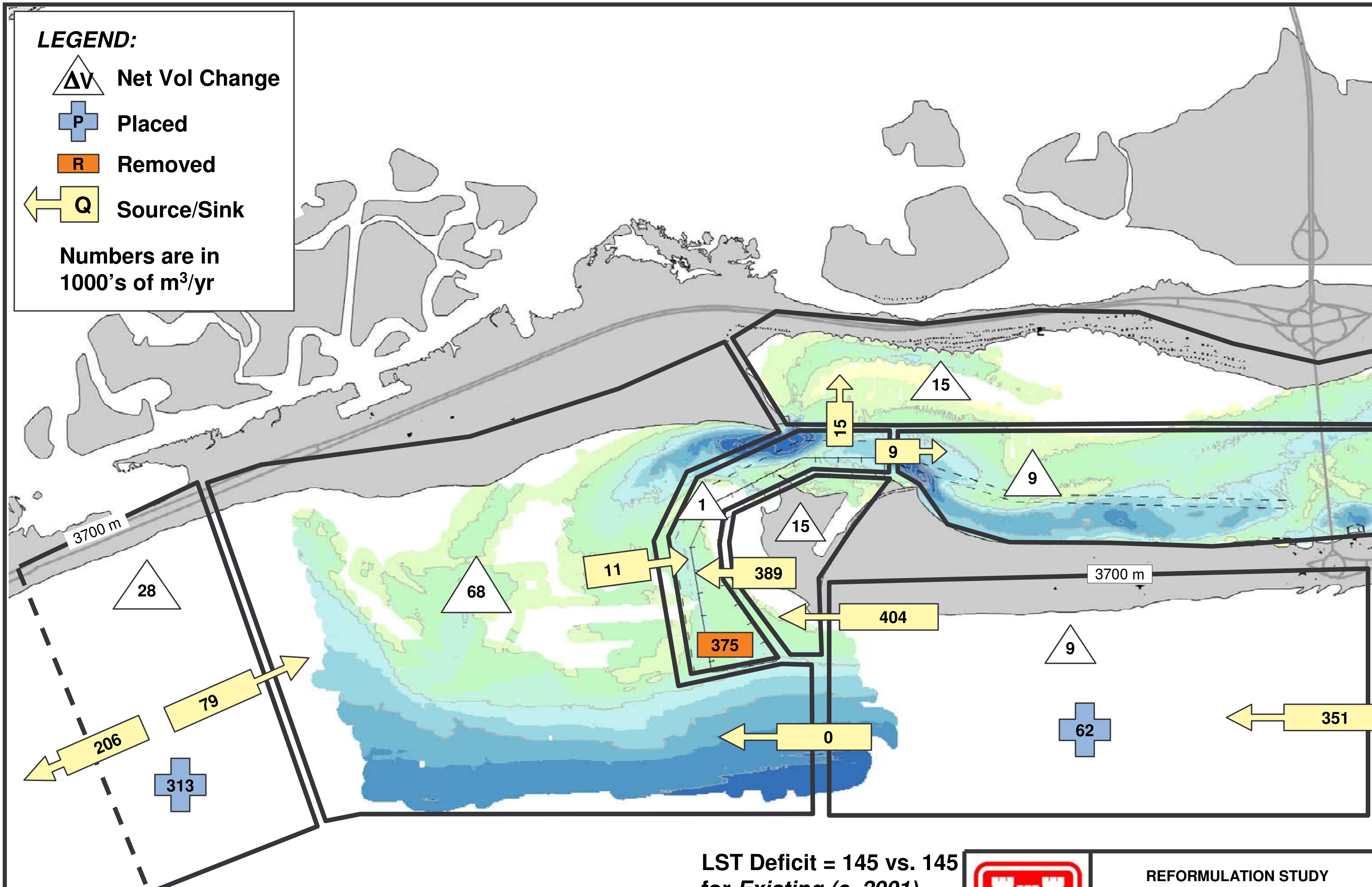
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 8-35



LST Deficit = 145 vs. 145
for *Existing (c. 2001)*

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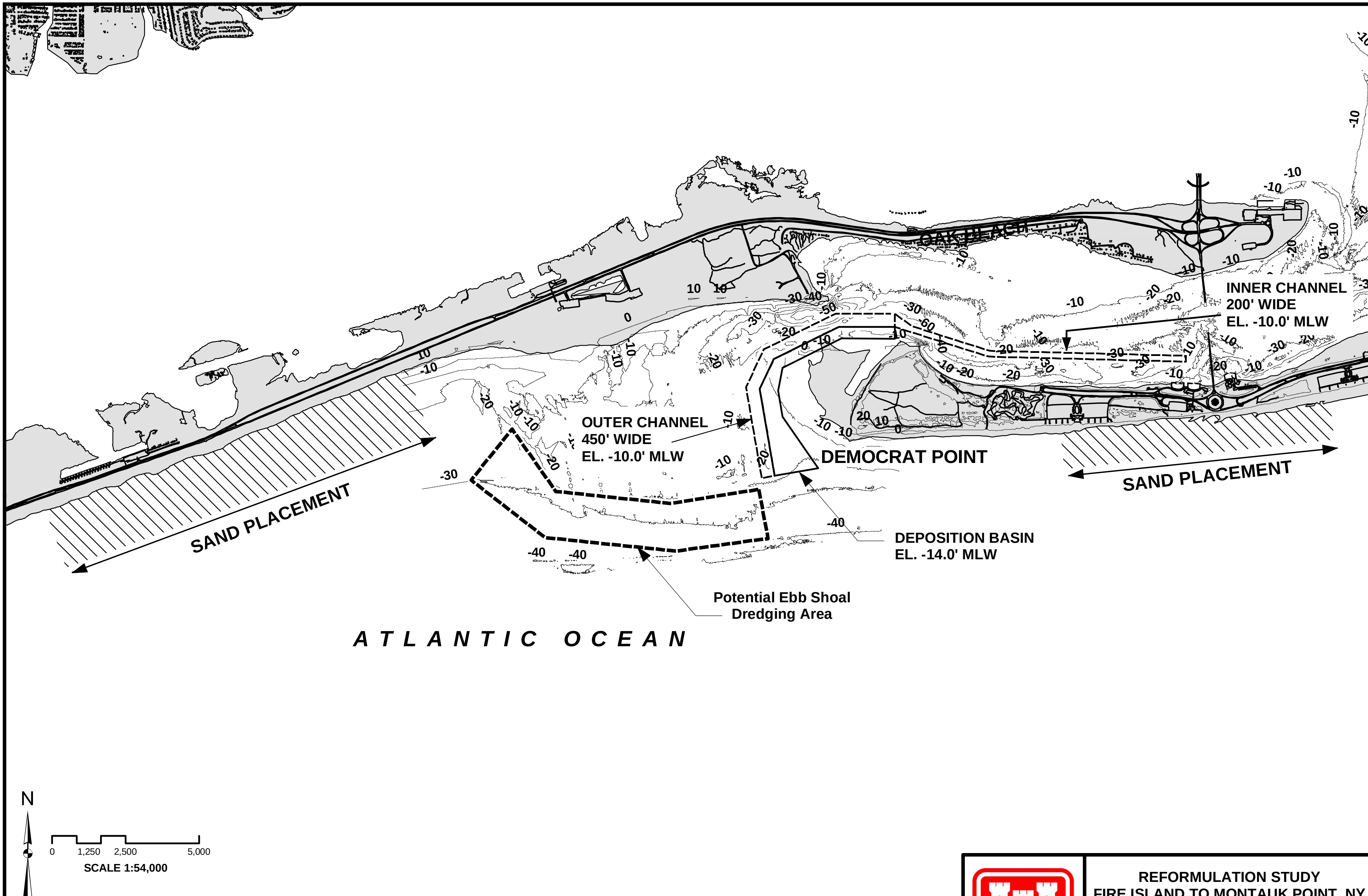
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 8-36

FII Alt. 1: AP/Existing Practice



FIRE ISLAND INLET - Alternative 2
AP+Dredging the Ebb Shoal

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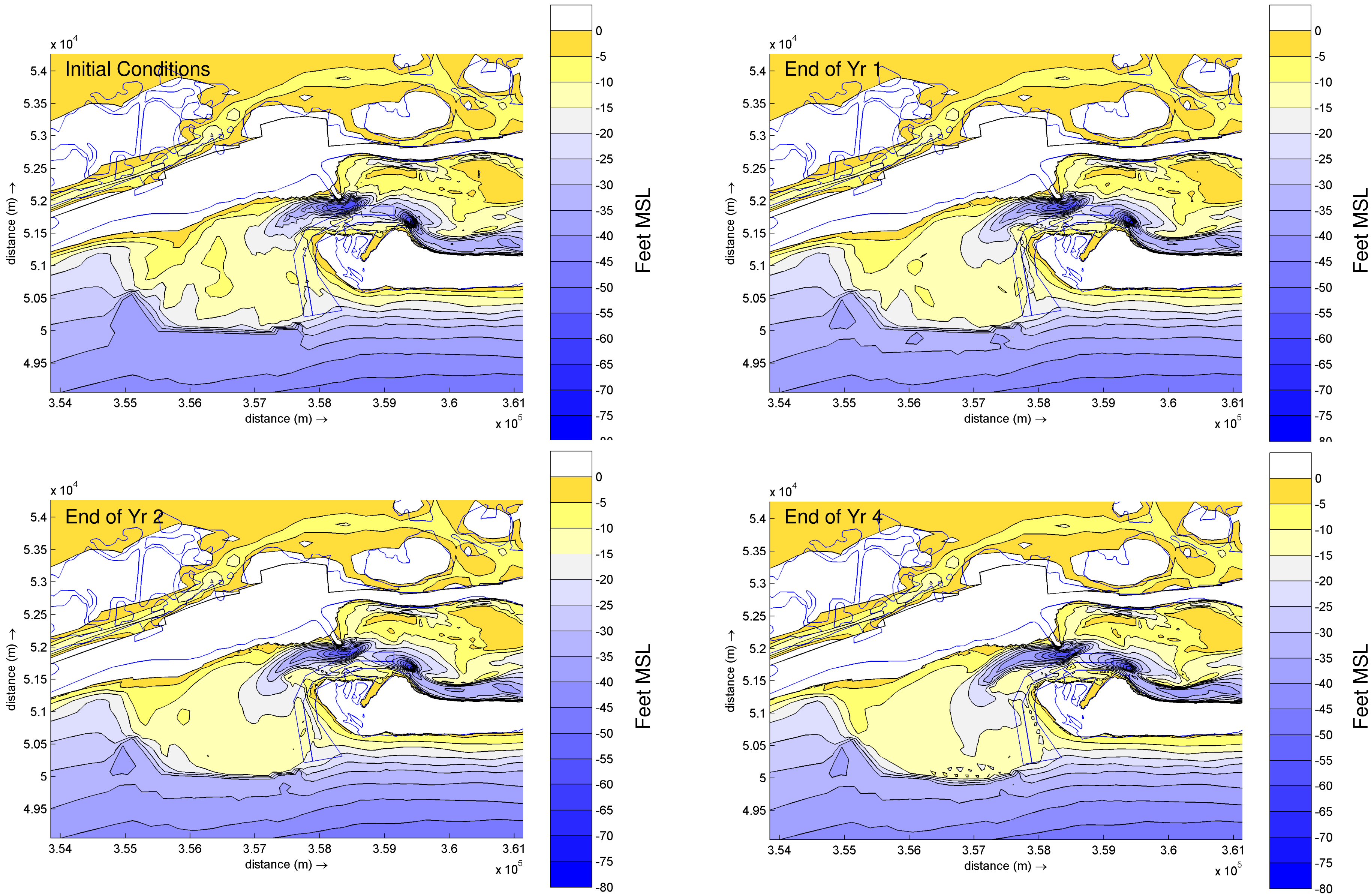
FIGURE 8-37

Filename: P:\3982-28\Work\Figures-Inlet Runs\Alternatives\Selected Inlet Alternatives\Moriches\MXD\FI_ALT2_EbbShoal.mxd



FII Alt. 2: Dredging the Ebb Shoal: Modeled Morphological Evolution

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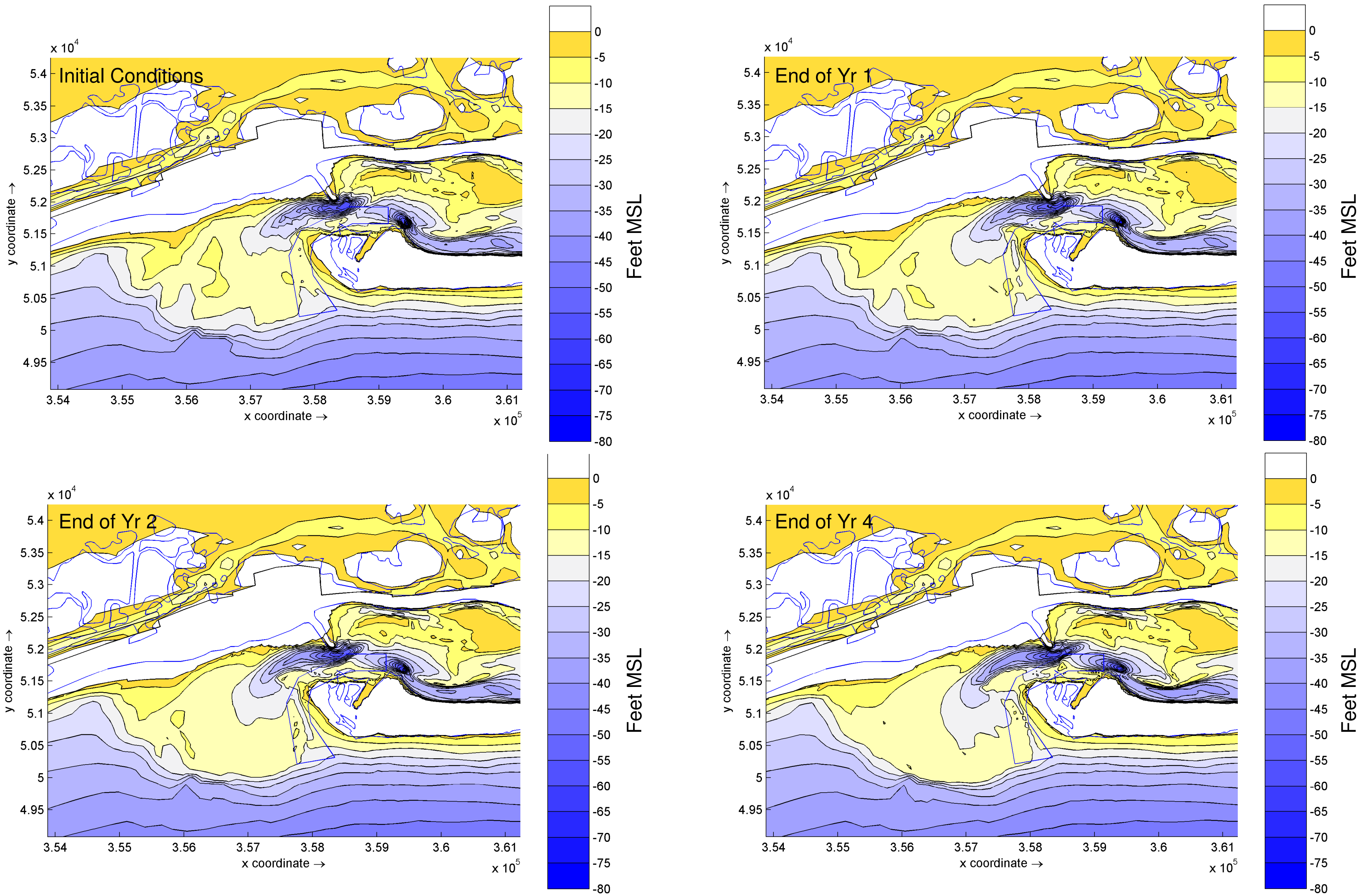
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 8-38



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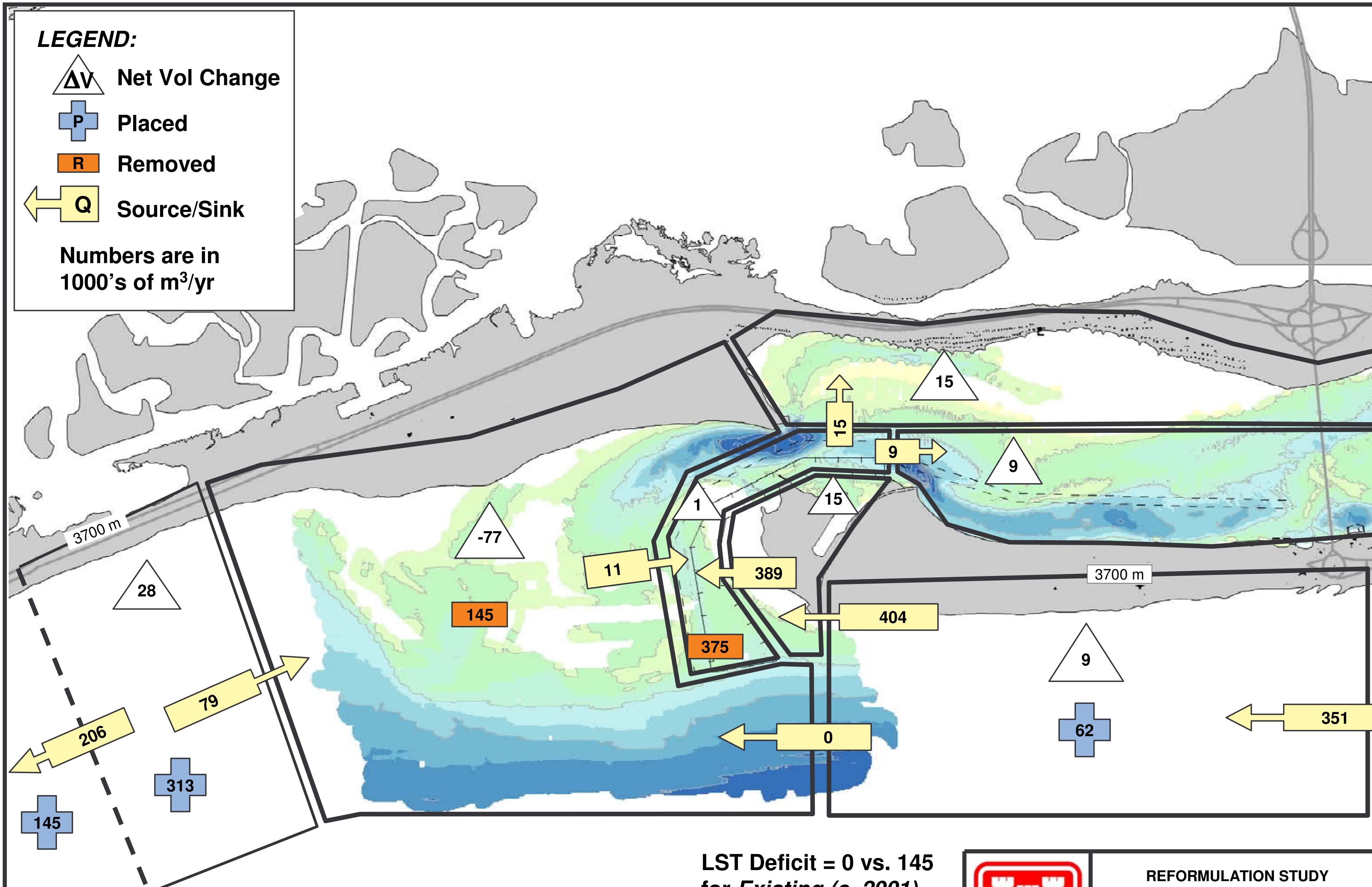
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 8-39

FII Alt. 2: Dredging the Ebb Shoal Small: Modeled Morphological Evolution



LST Deficit = 0 vs. 145
for *Existing (c. 2001)*

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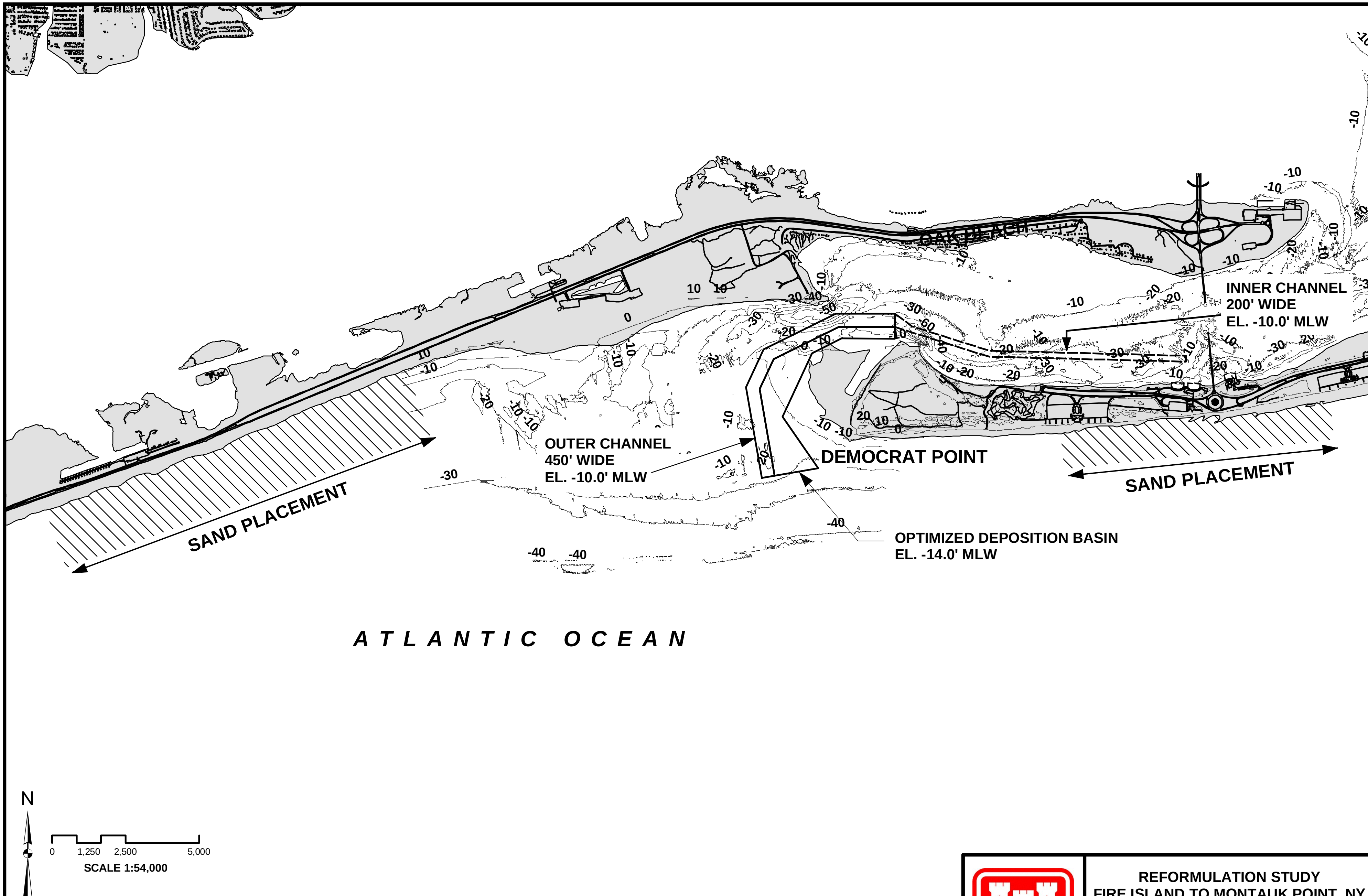
FIGURE 8-40

FII Alt. 2: AP + Dredging the Ebb Shoal



FIRE ISLAND INLET - Alternative 3
AP+Optimized Deposition Basin

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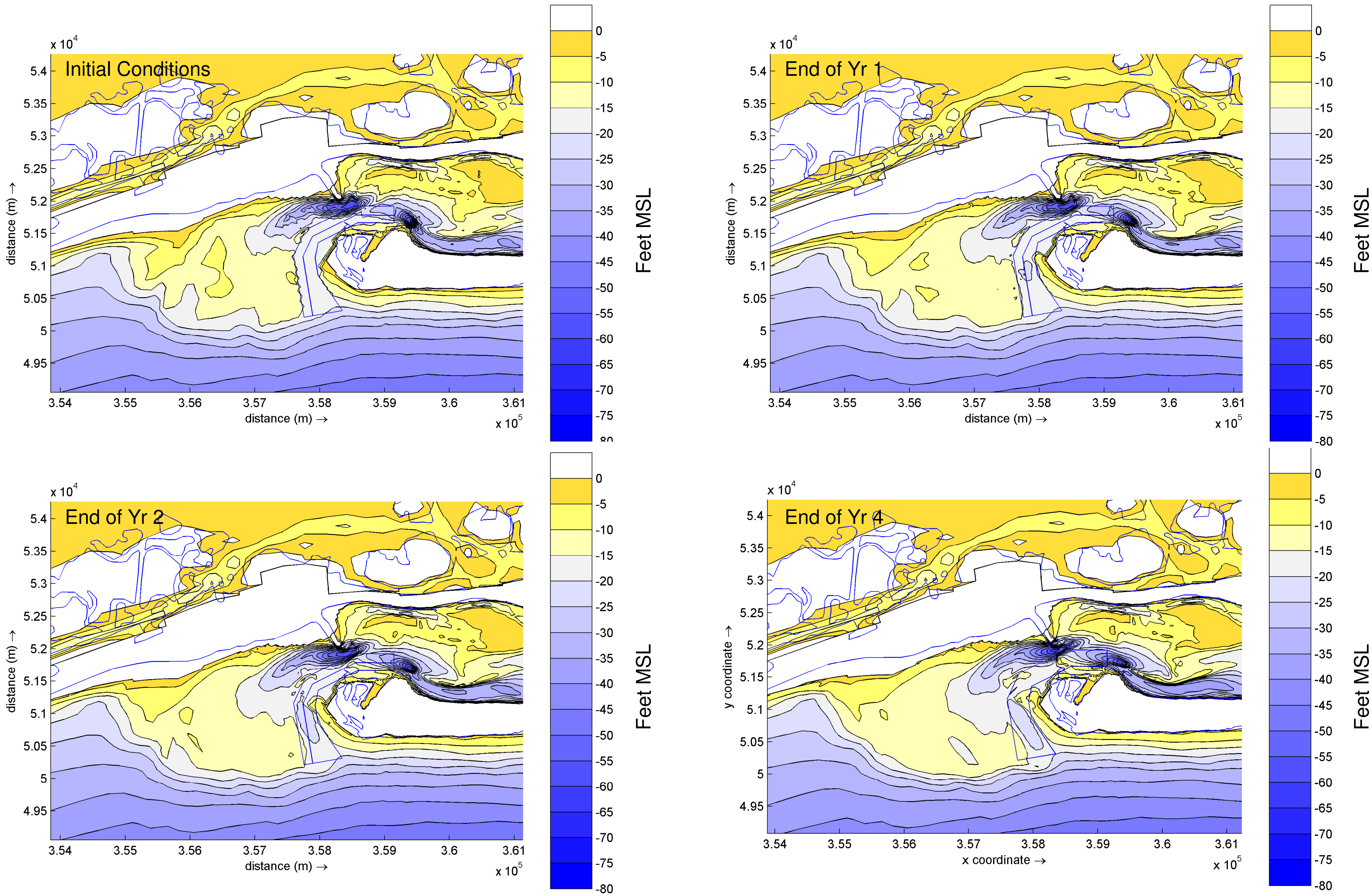
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 8-41

Filename: P:\3982-28\Work\Figures-Inlet Runs\Alternatives\Selected Inlet Alternatives\Moriches\MXD\FI_ALT3_Optimized.mxd



FII Alt. 3: Optimized Channel and/or Deposition Basin Configuration (Extended East): Modeled Morphological Evolution

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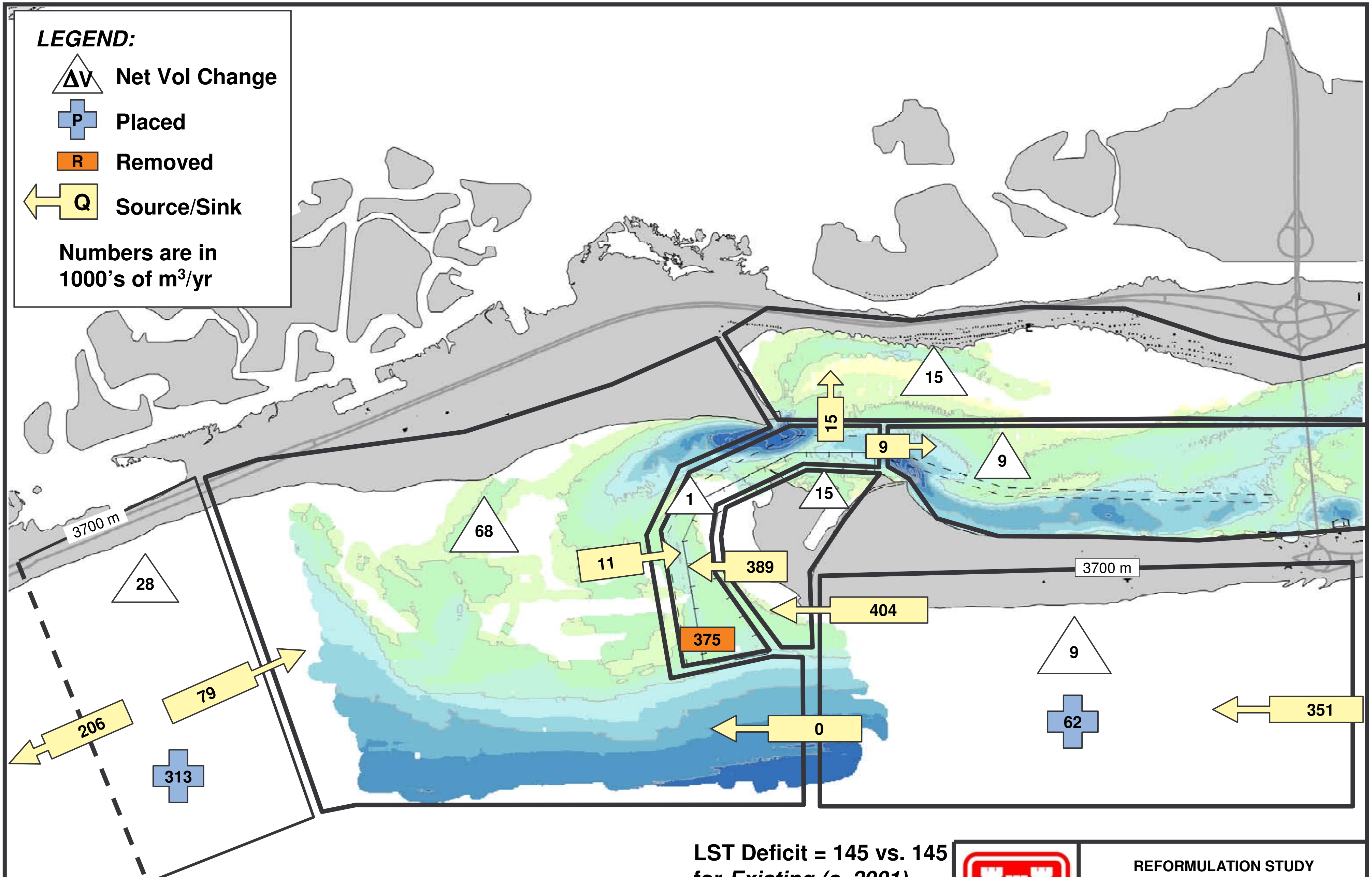
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 8-42



LST Deficit = 145 vs. 145
for *Existing (c. 2001)*

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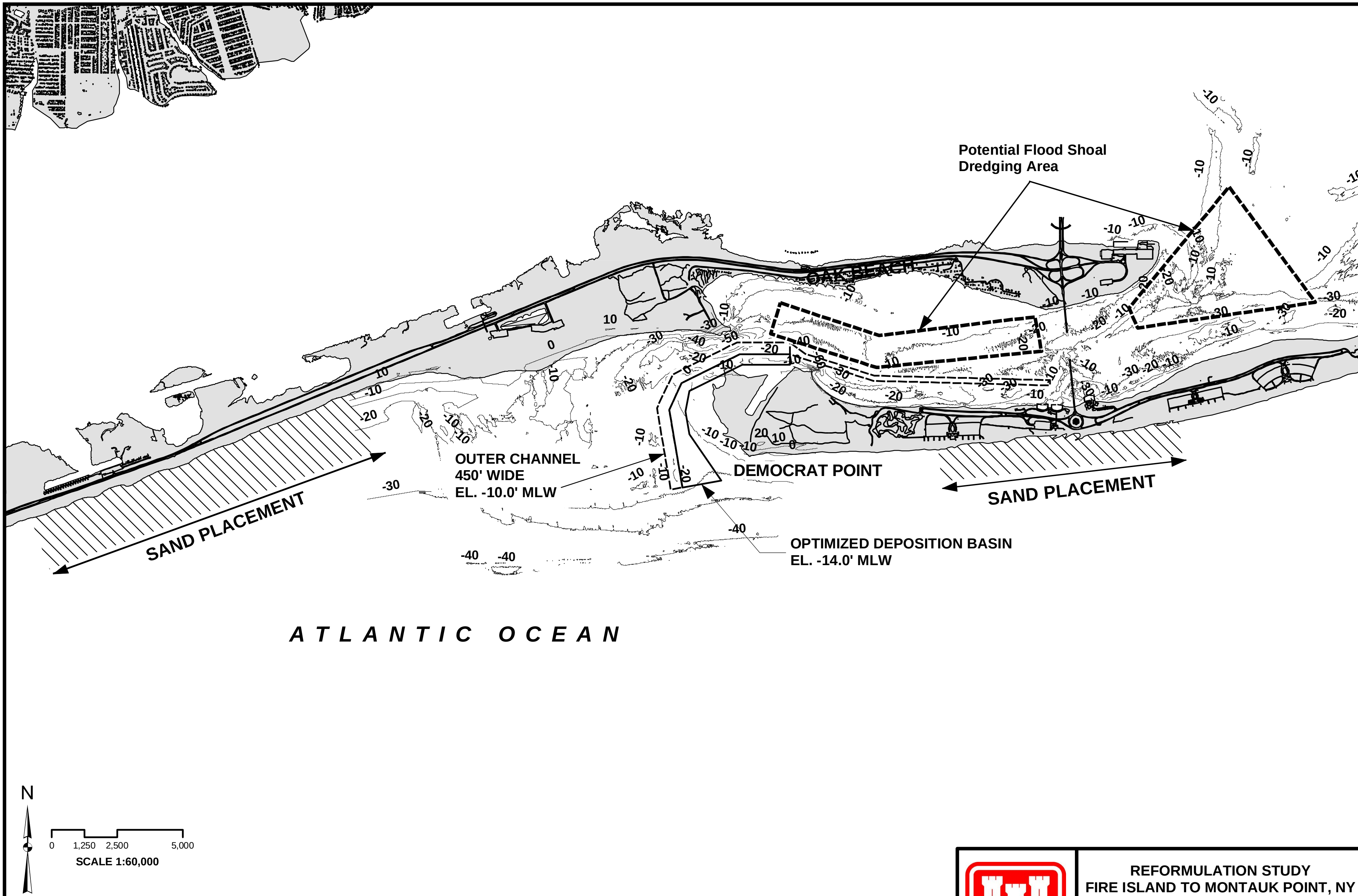
FIGURE 8-43

FII Alt. 3: Optimized Deposition Basin



FIRE ISLAND INLET - Alternative 4
AP+Dredging the Flood Shoal

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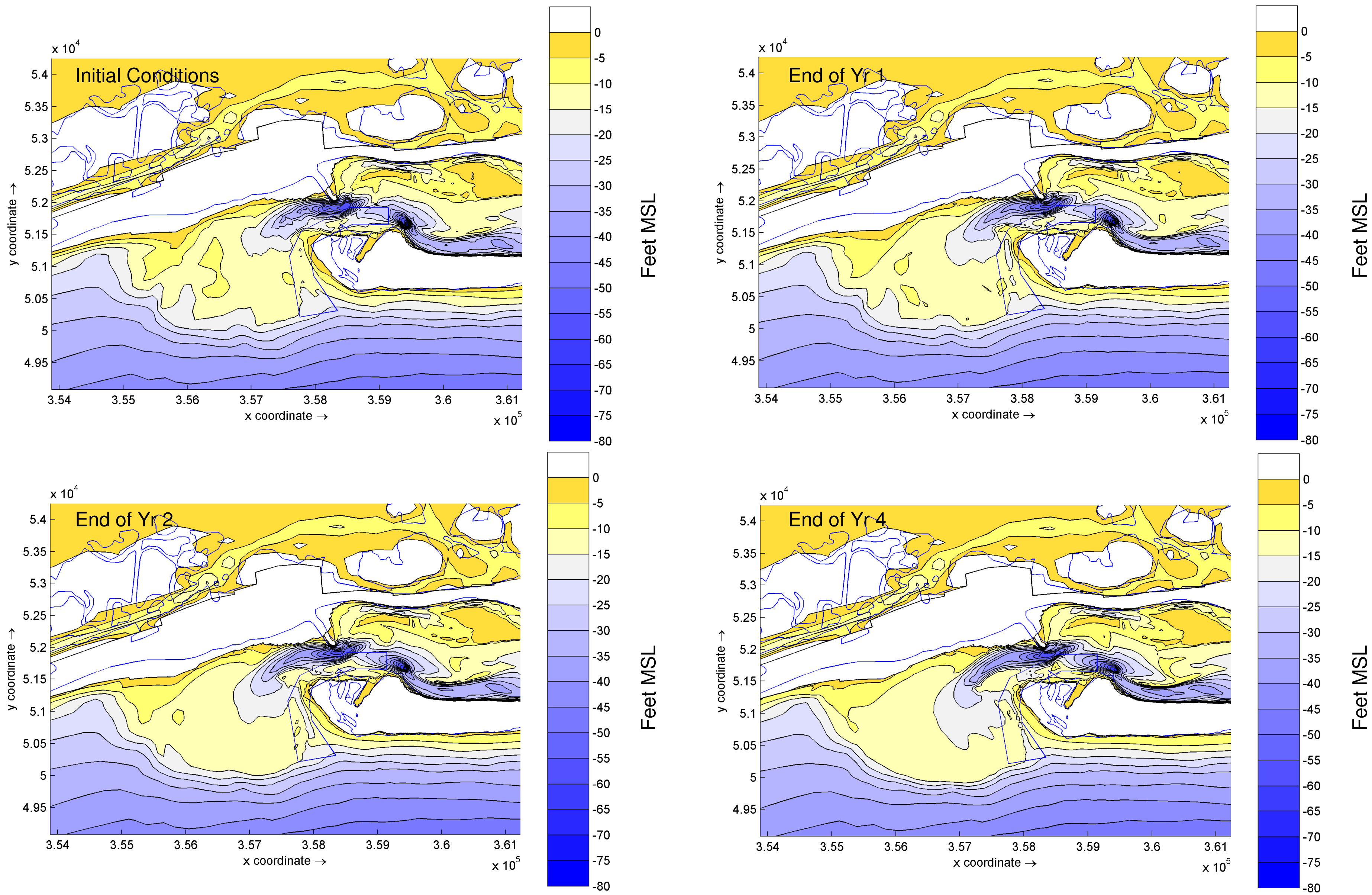
FIGURE 8-44

Filename: P:\3982-28\Work\Figures-Inlet Runs\Alternatives\Selected Inlet Alternatives\Moriches\MXD\FI_ALT4_FloodShoal.mxd



FII Alt. 4: Dredging the Flood Shoal: Modeled Morphological Evolution

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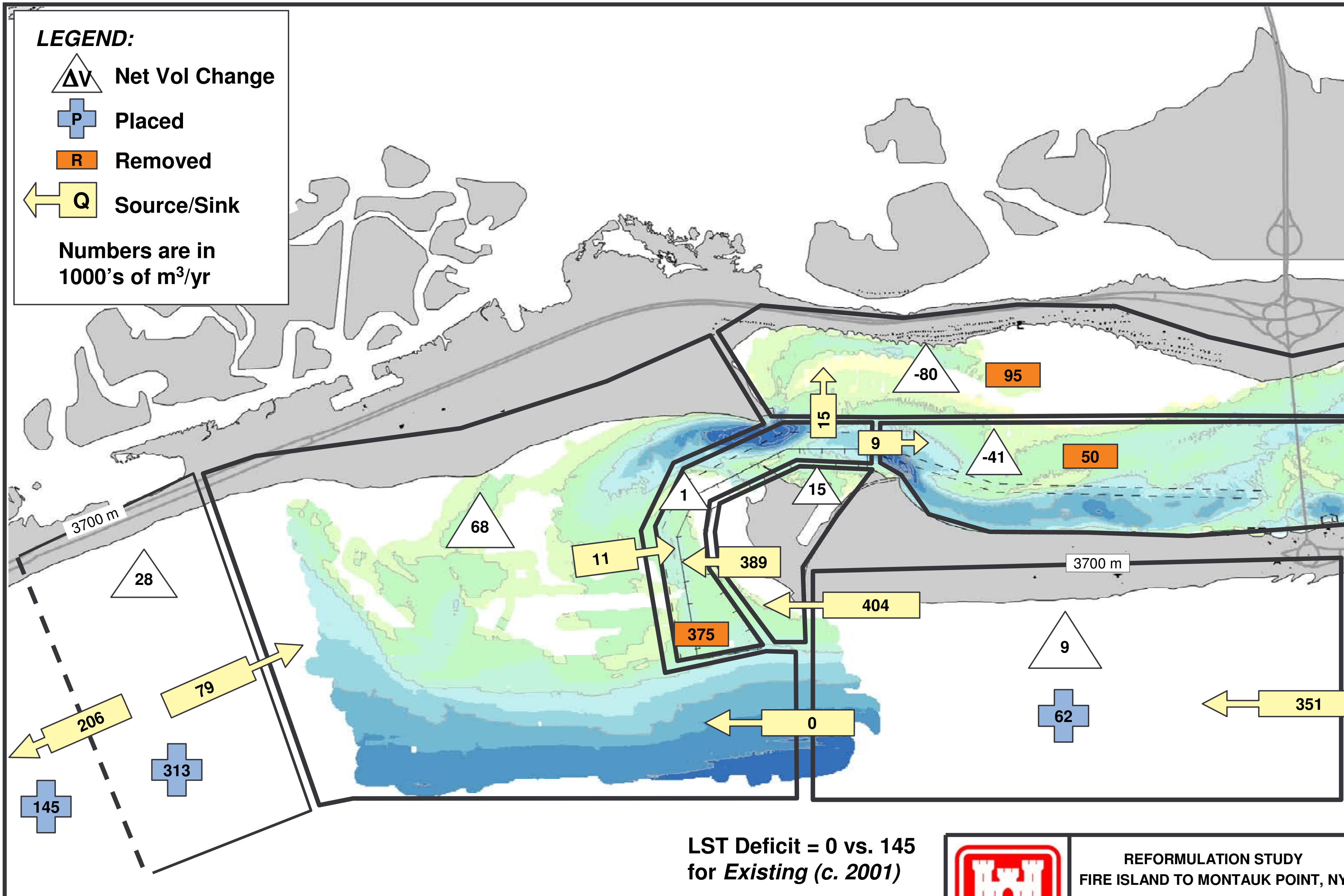
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE 8-45



FII Alt. 4: AP + Dredging the Flood Shoal

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FIGURE 8-46

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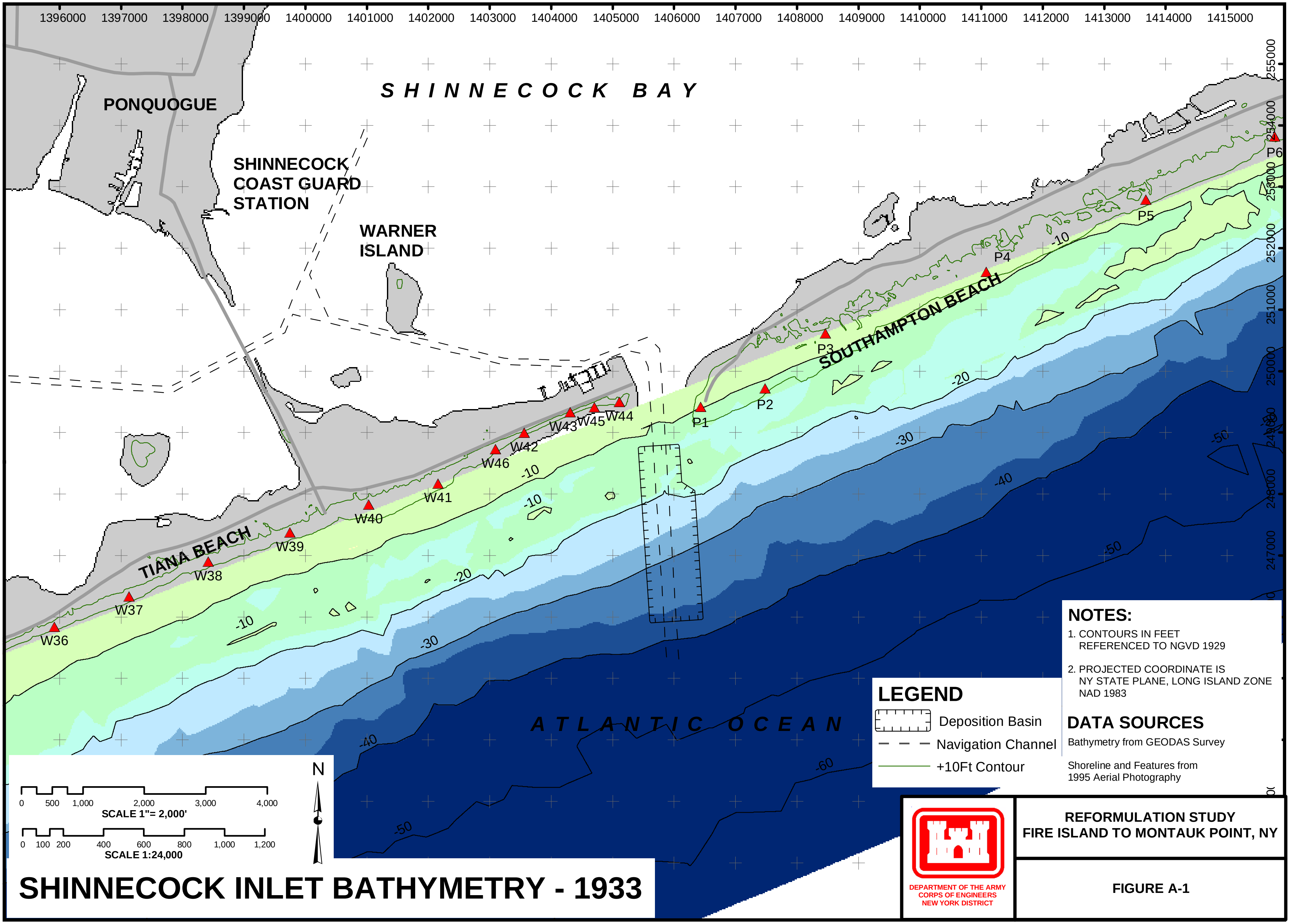
Appendix A – Inlet Surveys


SHINNECOCK BAY
PONQUOGUE
SHINNECOCK COAST GUARD STATION
WARNER ISLAND
TIANA BEACH
SOUTHAMPTON BEACH
ATLANTIC OCEAN
W36
W37
W38
W39
W40
W41
W46
W42
W43
W45
W44
P1
P2
P3
P4
P5
P6
NOTES:
1. CONTOURS IN FEET REFERENCED TO NGVD 1929
2. PROJECTED COORDINATE IS NY STATE PLANE, LONG ISLAND ZONE NAD 1983
LEGEND
Deposition Basin
Navigation Channel
+10Ft Contour
DATA SOURCES
Bathymetry from GEODAS Survey
Shoreline and Features from 1995 Aerial Photography
SCALE 1"= 2,000'
SCALE 1:24,000
N
SHINNECOCK INLET BATHYMETRY - 1933
DEPARTMENT OF THE ARMY CORPS OF ENGINEERS NEW YORK DISTRICT
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY
FIGURE A-1
Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Shin_Depth.mxd

Shinnecock Inlet Bathymetry - June 1984

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE A-2

File pat/File name

Shinnecock Inlet Bathymetry - June 1989

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE A-3

File pat/File name

Shinnecock Inlet Bathymetry - August 1991

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE A-4

File pat/File name

Shinnecock Inlet Bathymetry - December 1992

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE A-5

File pat/File name

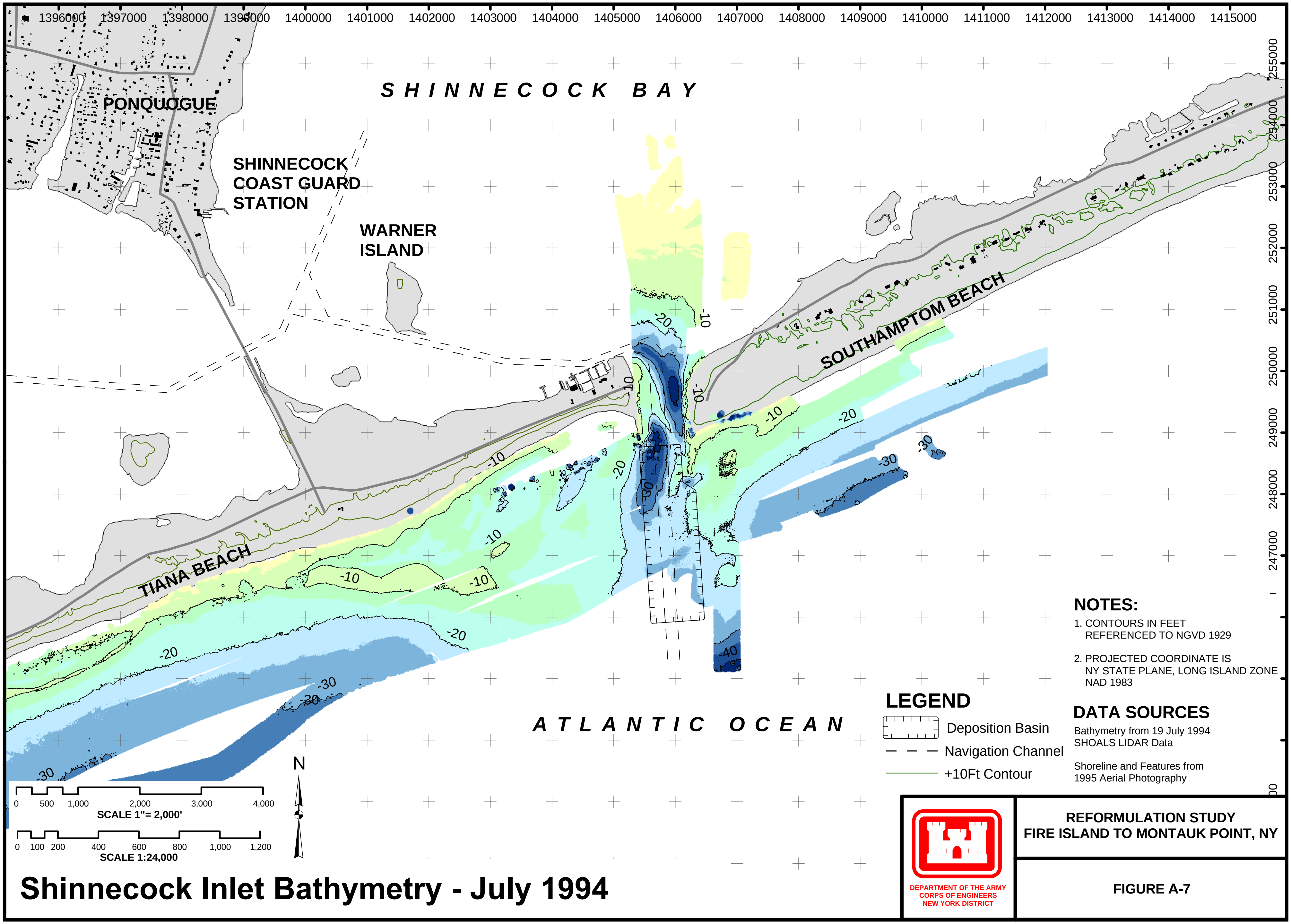
Shinnecock Inlet Bathymetry - August 1994

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE A-6

File pat/File name



Shinnecock Inlet Bathymetry - July 1994

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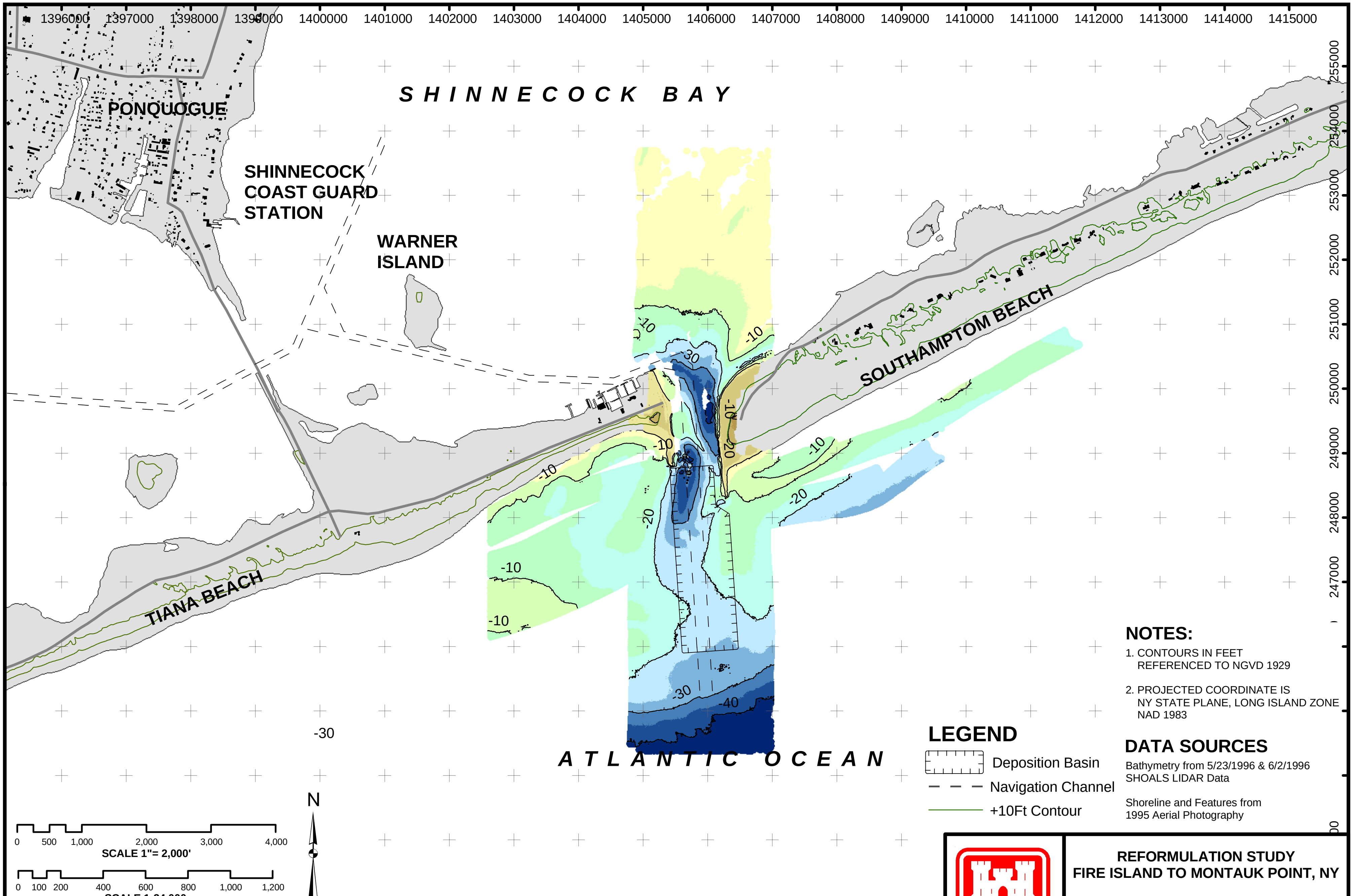


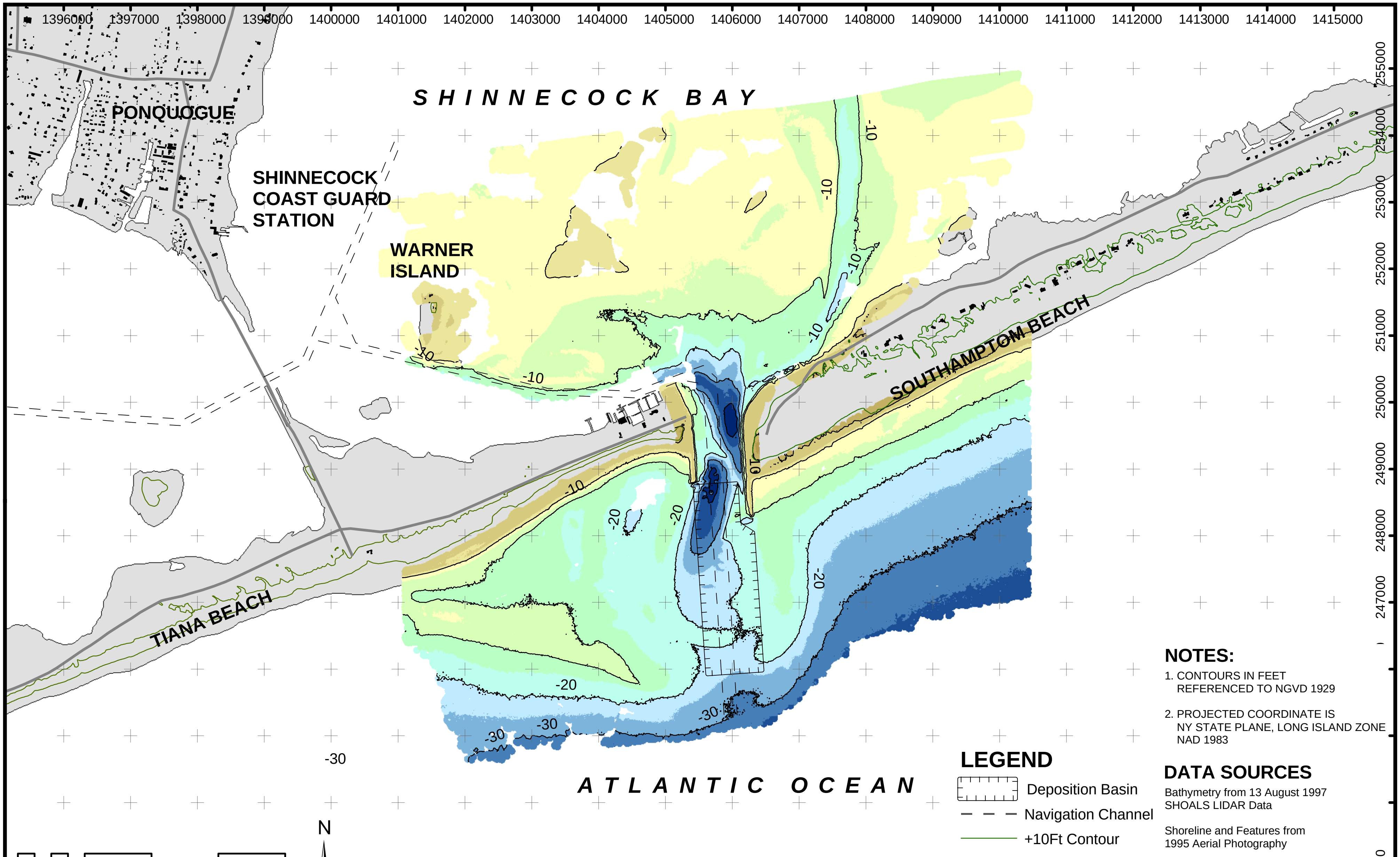
Shinnecock Inlet Bathymetry - June 1996

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE A-8

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Shin_Depth.mxd



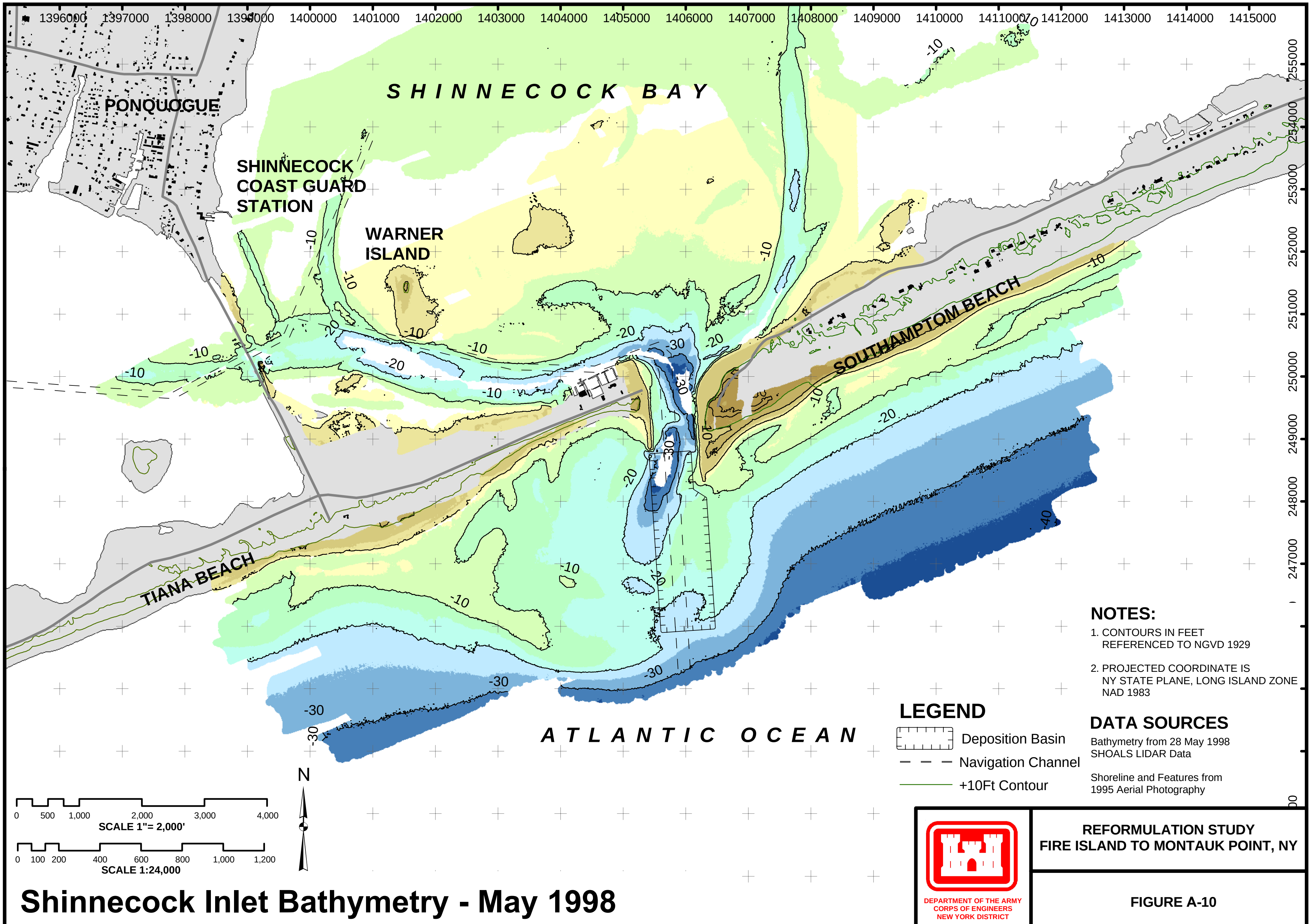
Shinnecock Inlet Bathymetry - August 1997

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE A-9

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Shin_Depth.mxd



NOTES:

1. CONTOURS IN FEET REFERENCED TO NGVD 1929
2. PROJECTED COORDINATE IS NY STATE PLANE, LONG ISLAND ZONE NAD 1983

LEGEND

- Deposition Basin
- Navigation Channel
- +10Ft Contour

DATA SOURCES

Bathymetry from 28 May 1998 SHOALS LIDAR Data

Shoreline and Features from 1995 Aerial Photography

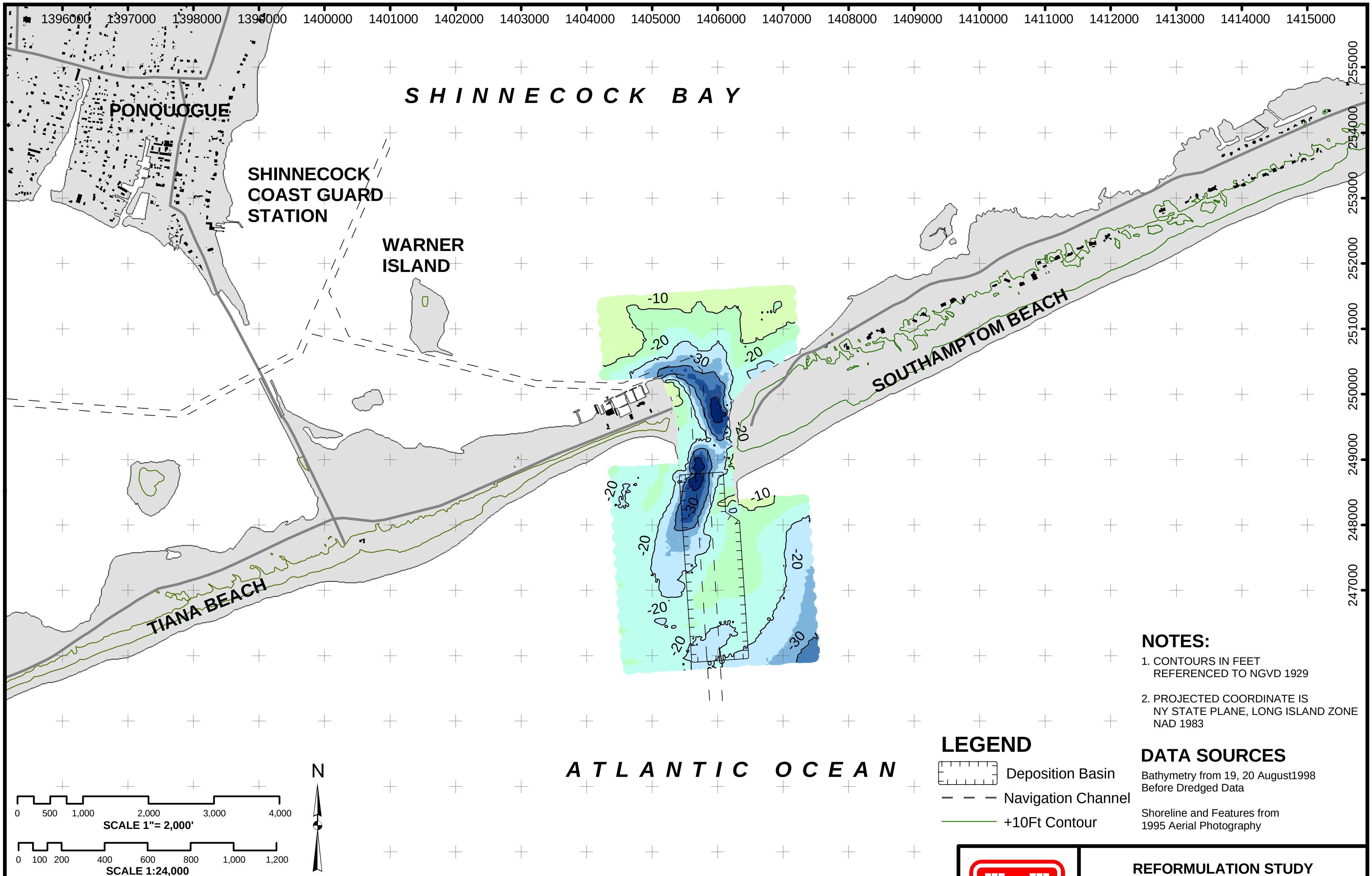
Shinnecock Inlet Bathymetry - May 1998

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE A-10

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Shin_Depth.mxd



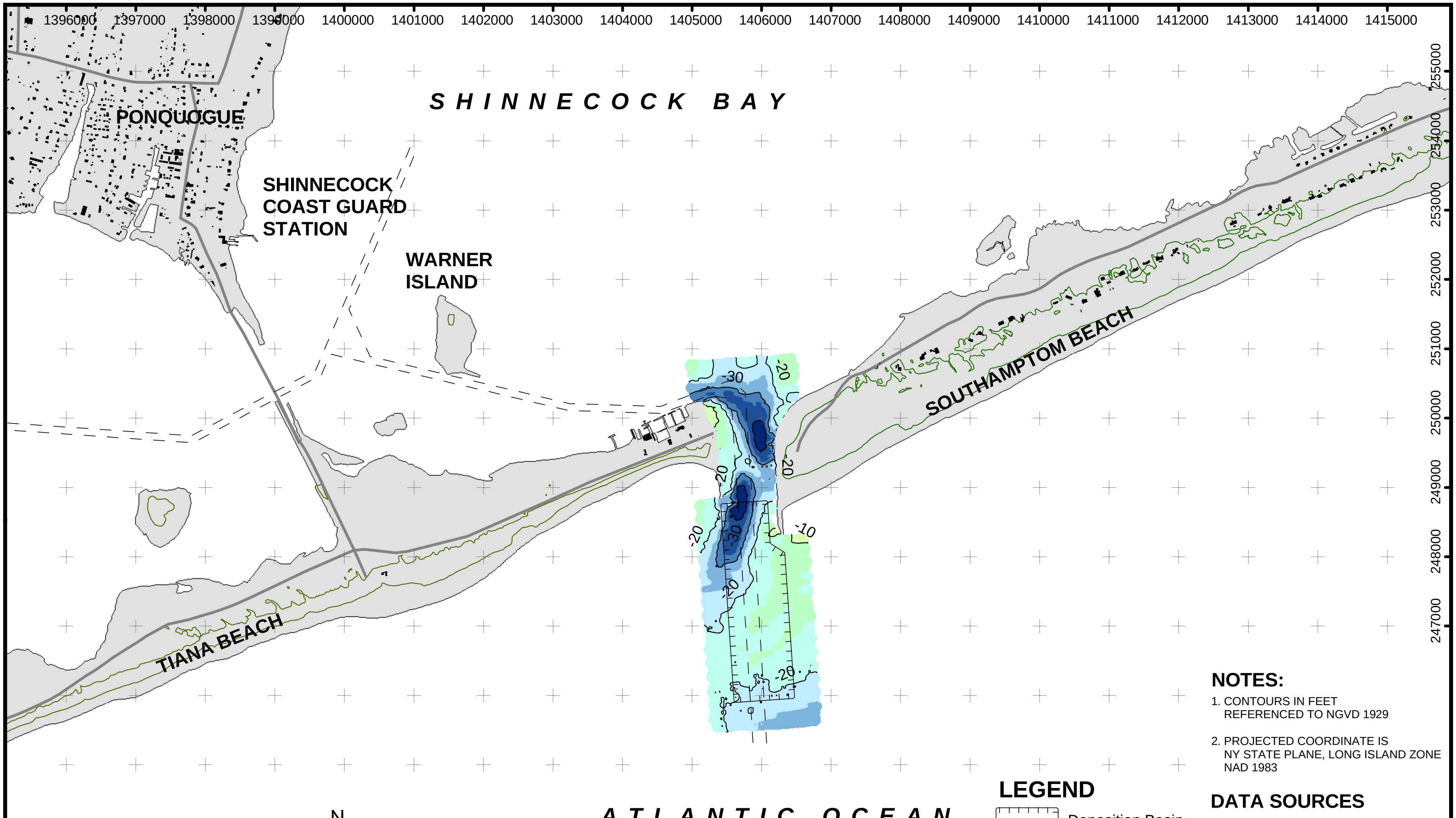
Shinnecock Inlet Bathymetry - August 1998
Before Dredged Survey

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE A-11

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Shin_Dredge.mxd



NOTES:

1. CONTOURS IN FEET REFERENCED TO NGVD 1929
2. PROJECTED COORDINATE IS NY STATE PLANE, LONG ISLAND ZONE NAD 1983

LEGEND

Deposition Basin
Navigation Channel
+10Ft Contour

DATA SOURCES

Bathymetry from 4, 8 & 9 September 1998 Before Dredged Data

Shoreline and Features from 1995 Aerial Photography

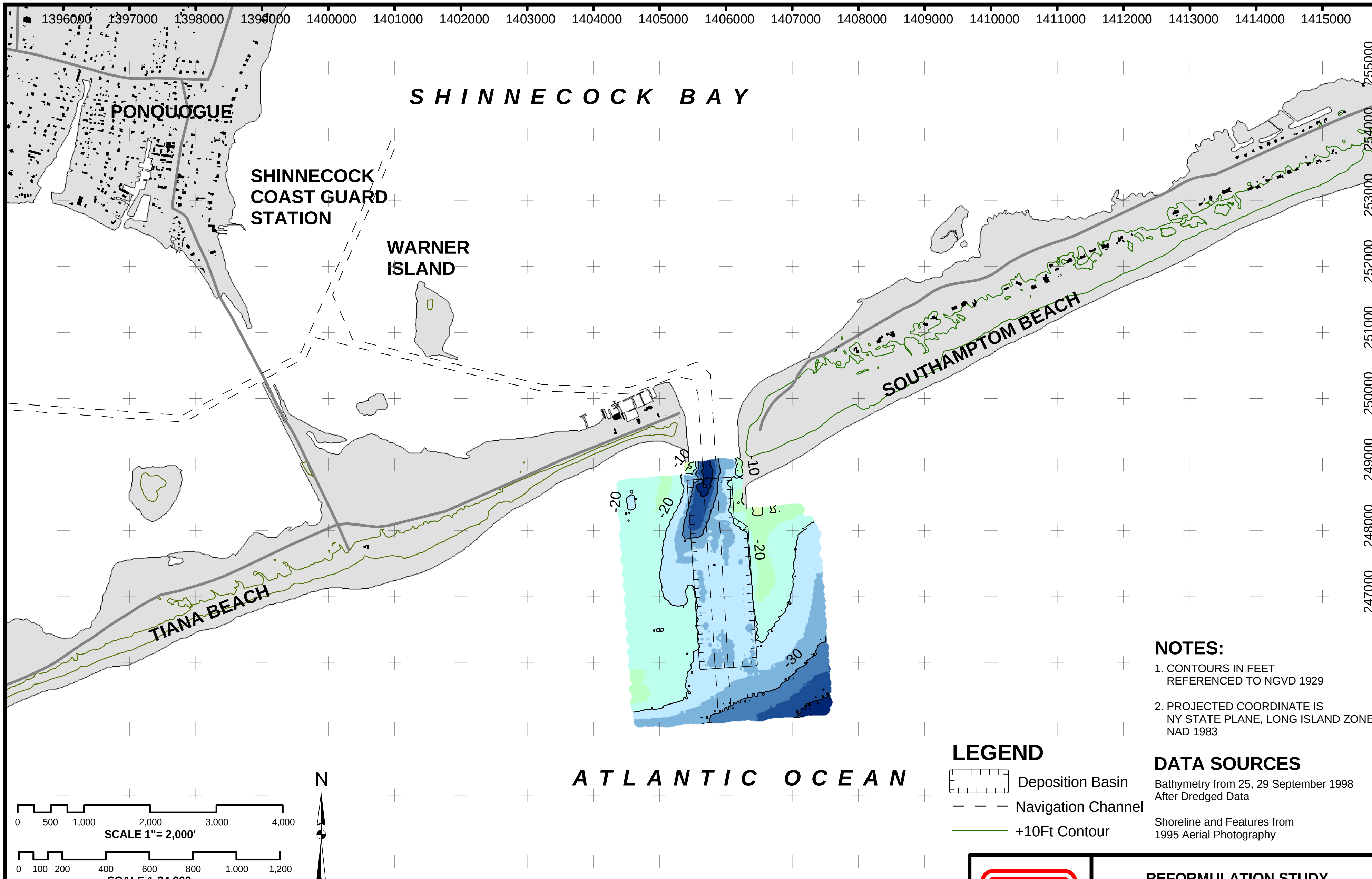
Shinnecock Inlet Bathymetry - September 9 1998 Before Dredged Survey

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE A-12

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Shin_Dredge.mxd



NOTES:

1. CONTOURS IN FEET
   REFERENCED TO NGVD 1929
2. PROJECTED COORDINATE IS
   NY STATE PLANE, LONG ISLAND ZONE
   NAD 1983

LEGEND

- Deposition Basin
- Navigation Channel
- +10Ft Contour

DATA SOURCES

Bathymetry from 25, 29 September 1998
After Dredged Data

Shoreline and Features from
1995 Aerial Photography

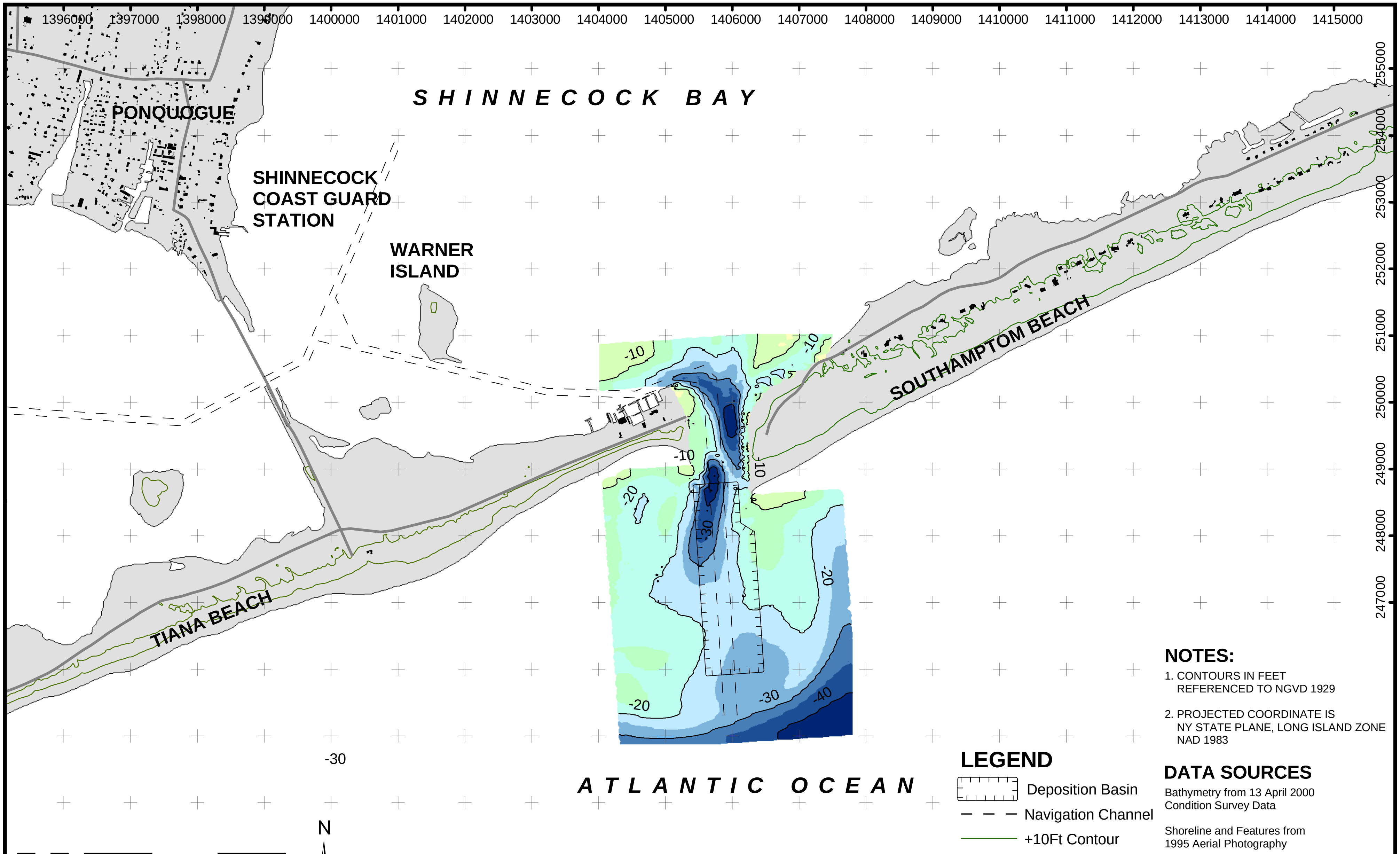
DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE A-13

Shinnecock Inlet Bathymetry - September 29 1998
After Dredged Survey

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Shin_Dredge.mxd



NOTES:

1. CONTOURS IN FEET REFERENCED TO NGVD 1929
2. PROJECTED COORDINATE IS NY STATE PLANE, LONG ISLAND ZONE NAD 1983

LEGEND

Deposition Basin

Navigation Channel

+10Ft Contour

DATA SOURCES

Bathymetry from 13 April 2000 Condition Survey Data

Shoreline and Features from 1995 Aerial Photography

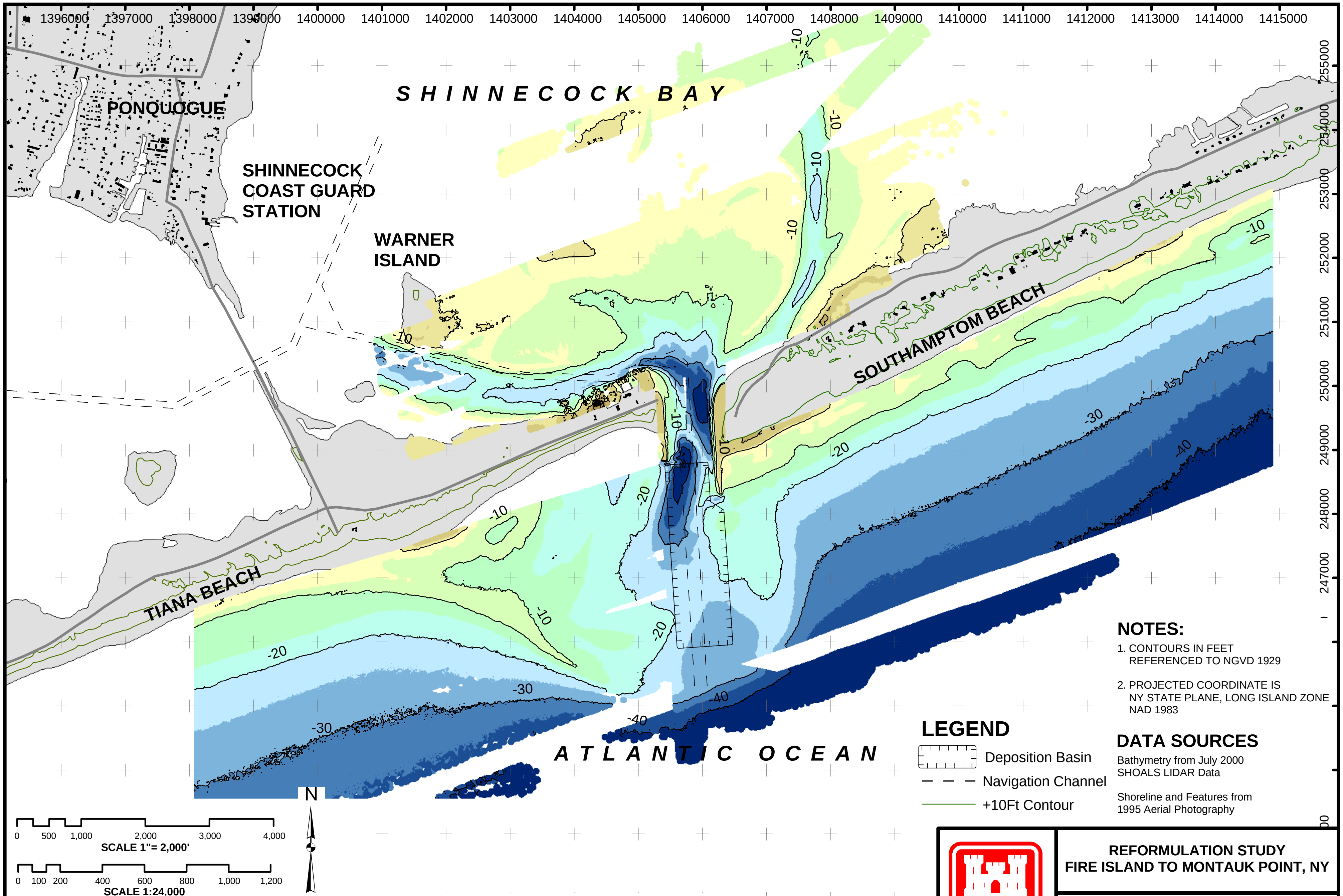
DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE A-14

Shinnecock Inlet Bathymetry - April 2000

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Shin_CondSurvey.mxd



Shinnecock Inlet Bathymetry - July 2000

NOTES:

1. CONTOURS IN FEET REFERENCED TO NGVD 1929
2. PROJECTED COORDINATE IS NY STATE PLANE, LONG ISLAND ZONE NAD 1983

LEGEND

- Deposition Basin
- Navigation Channel
- +10Ft Contour

DATA SOURCES

Bathymetry from July 2000 SHOALS LIDAR Data

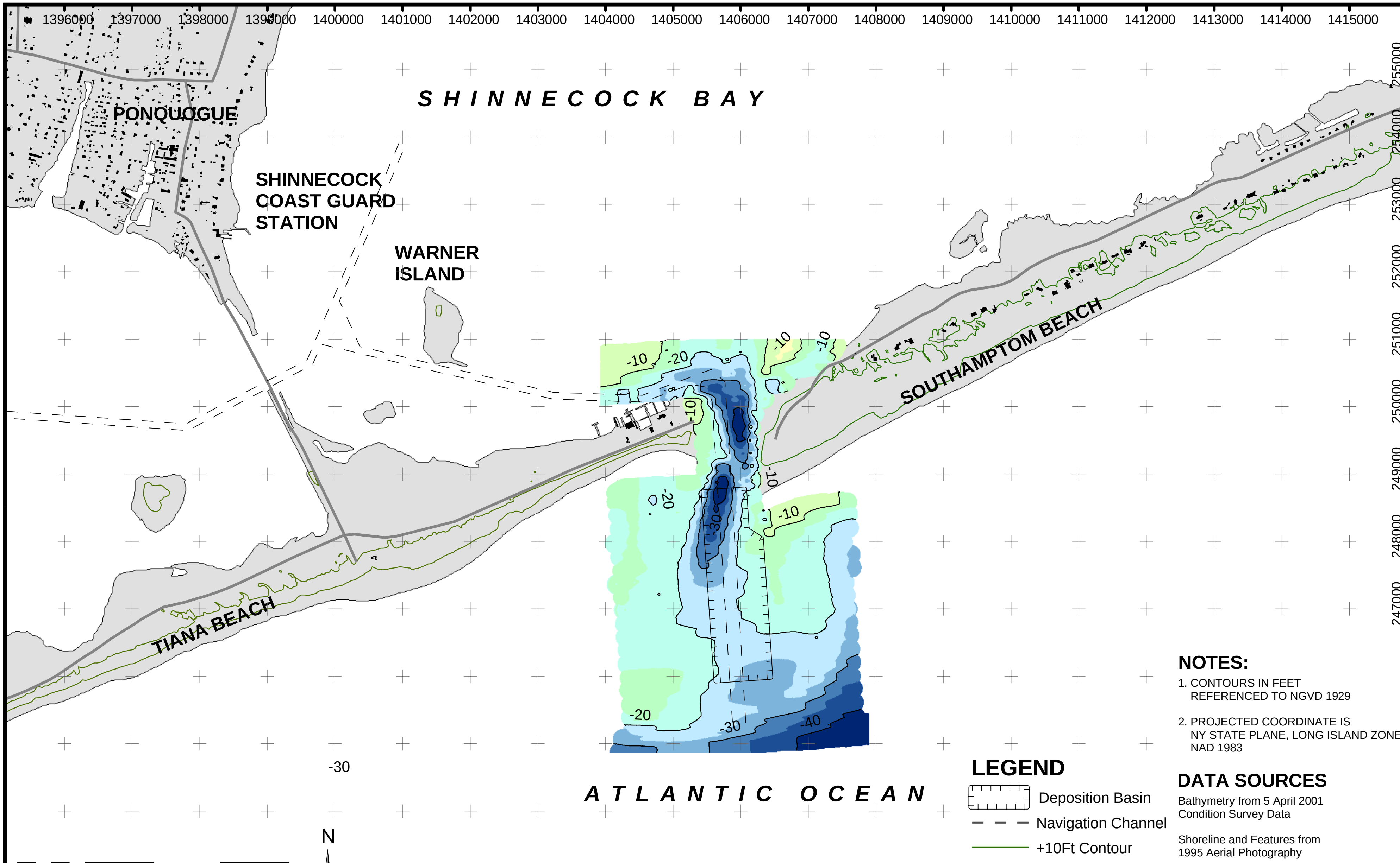
Shoreline and Features from 1995 Aerial Photography

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE A-15

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Shin_Depth.mxd



NOTES:

1. CONTOURS IN FEET REFERENCED TO NGVD 1929
2. PROJECTED COORDINATE IS NY STATE PLANE, LONG ISLAND ZONE NAD 1983

LEGEND

Deposition Basin
Navigation Channel
+10Ft Contour

DATA SOURCES

Bathymetry from 5 April 2001 Condition Survey Data

Shoreline and Features from 1995 Aerial Photography

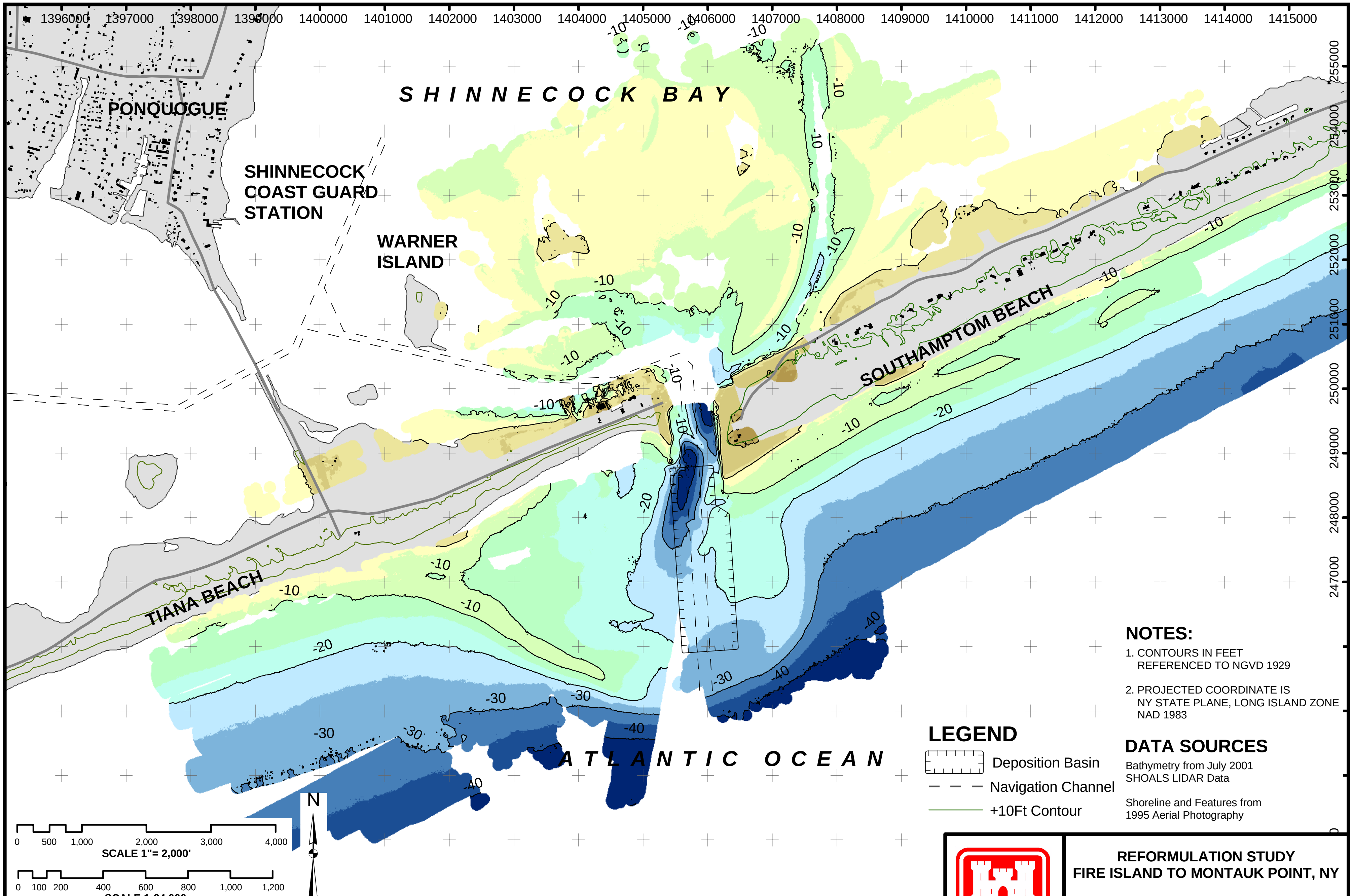
Shinnecock Inlet Bathymetry - April 2001

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE A-16

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Shin_CondSurvey.mxd



Shinnecock Inlet Bathymetry - July 2001

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE A-17

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Shin_Depth.mxd

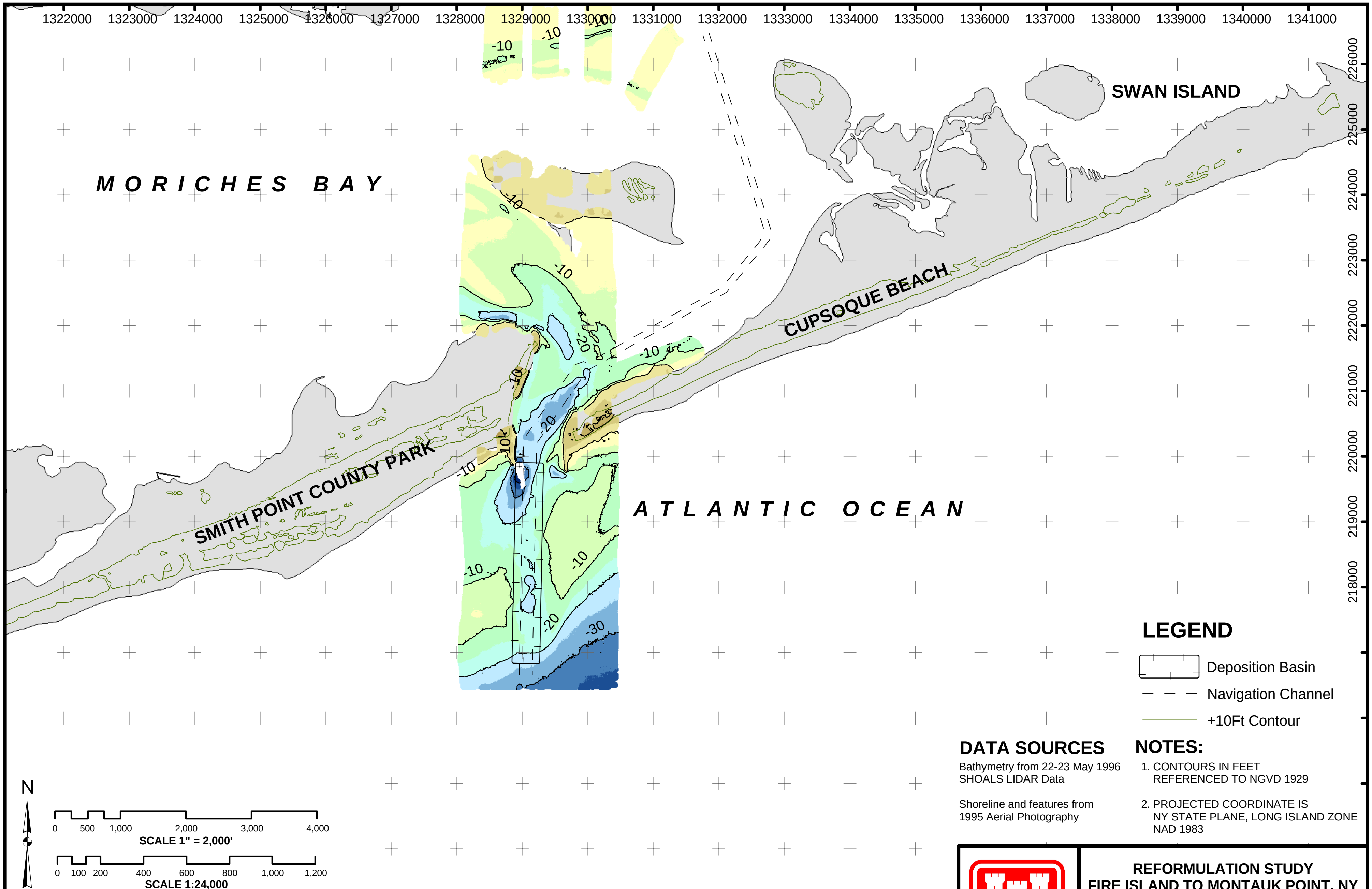
Moriches Inlet Bathymetry - March 1981

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE A-18

File pat/File name



LEGEND

Deposition Basin

Navigation Channel

+10Ft Contour

DATA SOURCES

Bathymetry from 22-23 May 1996
SHOALS LIDAR Data

Shoreline and features from
1995 Aerial Photography

NOTES:

1. CONTOURS IN FEET
   REFERENCED TO NGVD 1929
2. PROJECTED COORDINATE IS
   NY STATE PLANE, LONG ISLAND ZONE
   NAD 1983

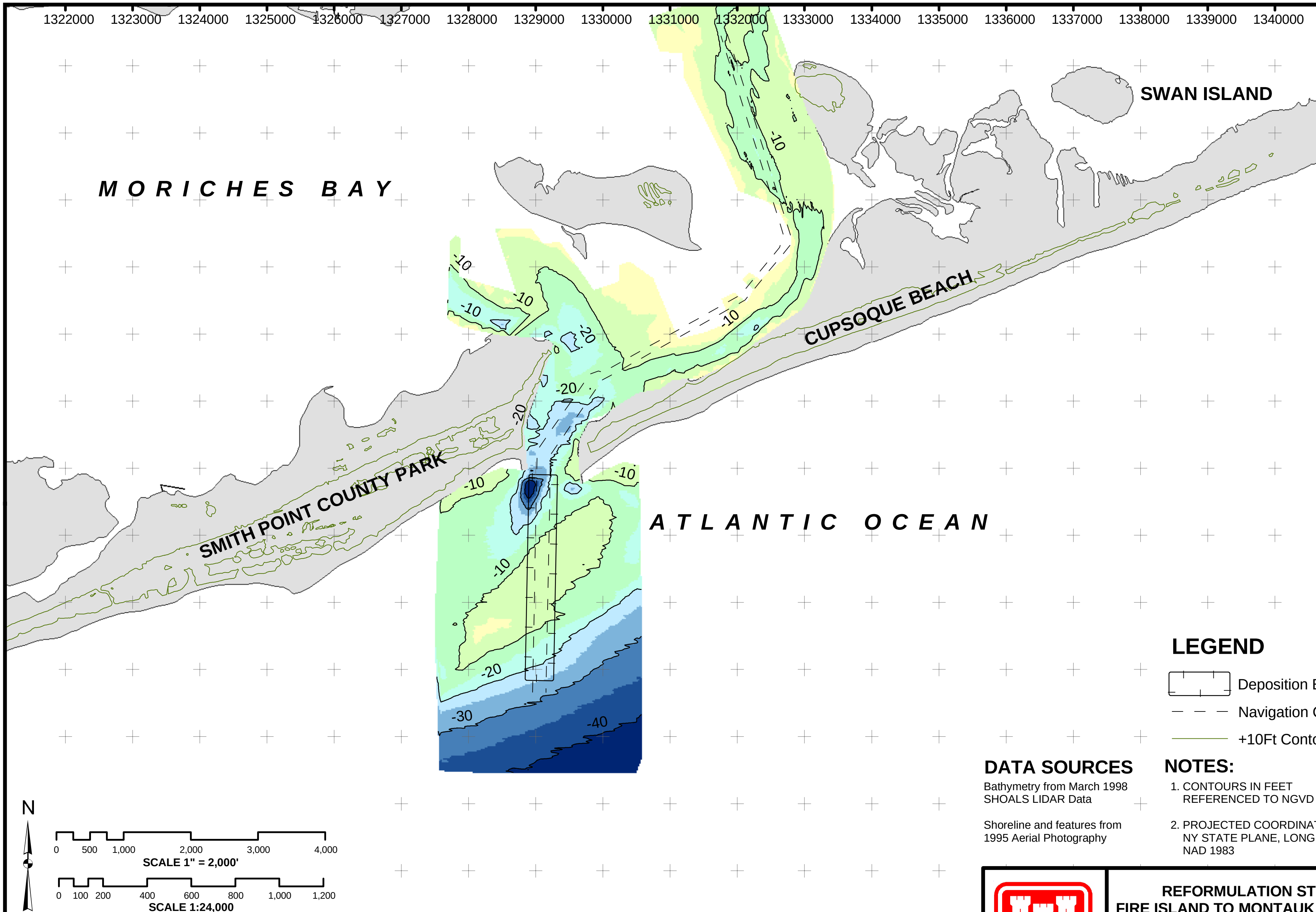
Moriches Inlet Bathymetry - May 1996

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE A-19

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Moriches_Depth.mxd



DATA SOURCES

Bathymetry from March 1998
SHOALS LIDAR Data

Shoreline and features from
1995 Aerial Photography

NOTES:

1. CONTOURS IN FEET
REFERENCED TO NGVD 1929

2. PROJECTED COORDINATE IS
NY STATE PLANE, LONG ISLAND ZONE
NAD 1983

Moriches Inlet Bathymetry - March 1998

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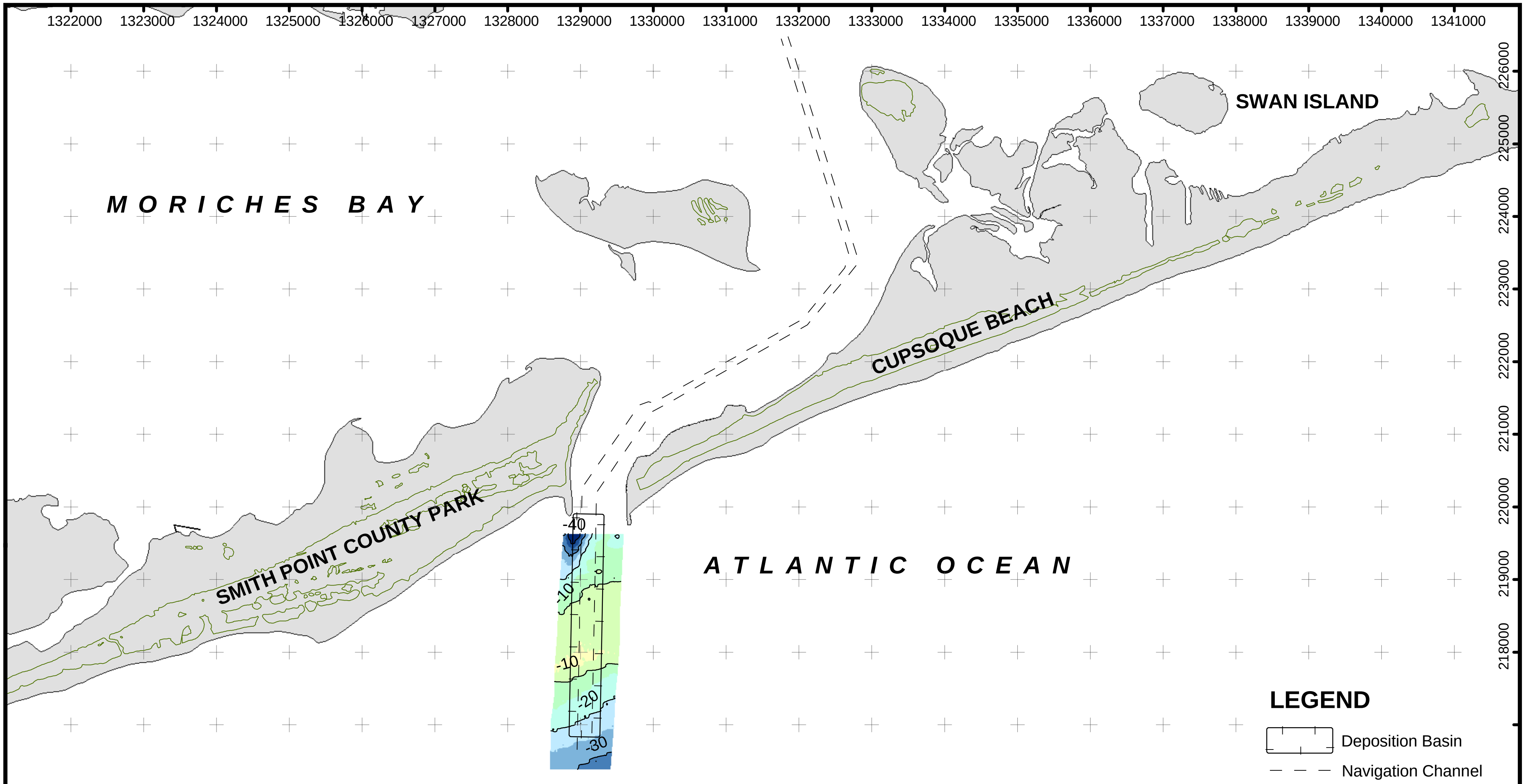
NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

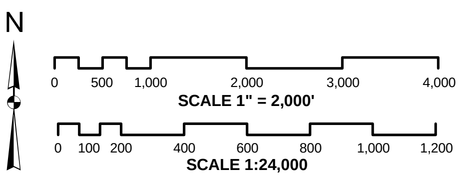
FIGURE A-20

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Moriches_Depth.mxd



LEGEND

- Deposition Basin
- Navigation Channel
- +10Ft Contour



DATA SOURCES

Bathymetry from 9/21/1998
Pre-dredge Survey Data

Shoreline and features from
1995 Aerial Photography

NOTES:

1. CONTOURS IN FEET REFERENCED TO NGVD 1929
2. PROJECTED COORDINATE IS NY STATE PLANE, LONG ISLAND ZONE NAD 1983

Moriches Inlet Bathymetry - September 1998

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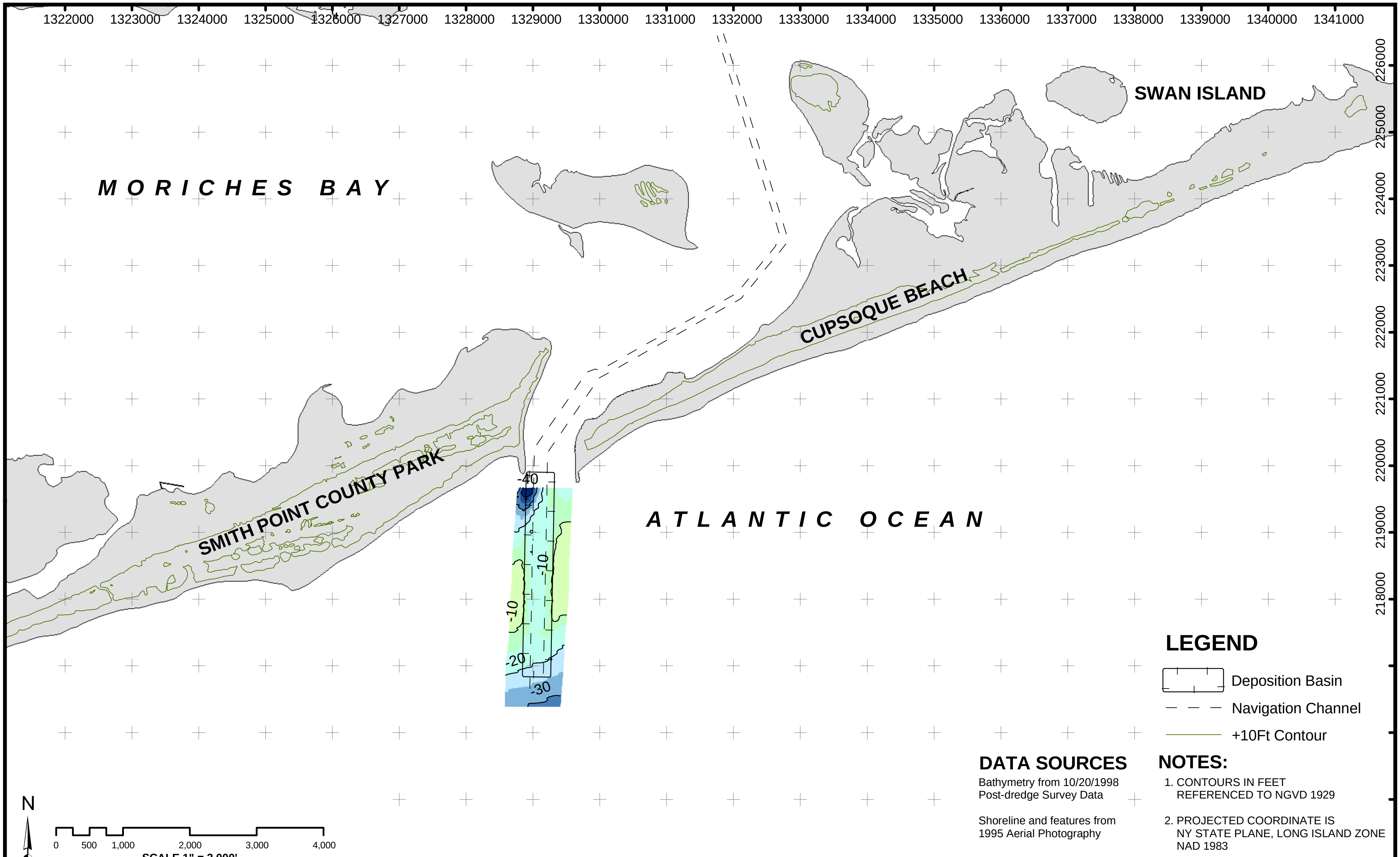
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE A-21

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Moriches_Depth.mxd



N

0 500 1,000 2,000 3,000 4,000
SCALE 1" = 2,000'

0 100 200 400 600 800 1,000 1,200
SCALE 1:24,000

Moriches Inlet Bathymetry - October 1998

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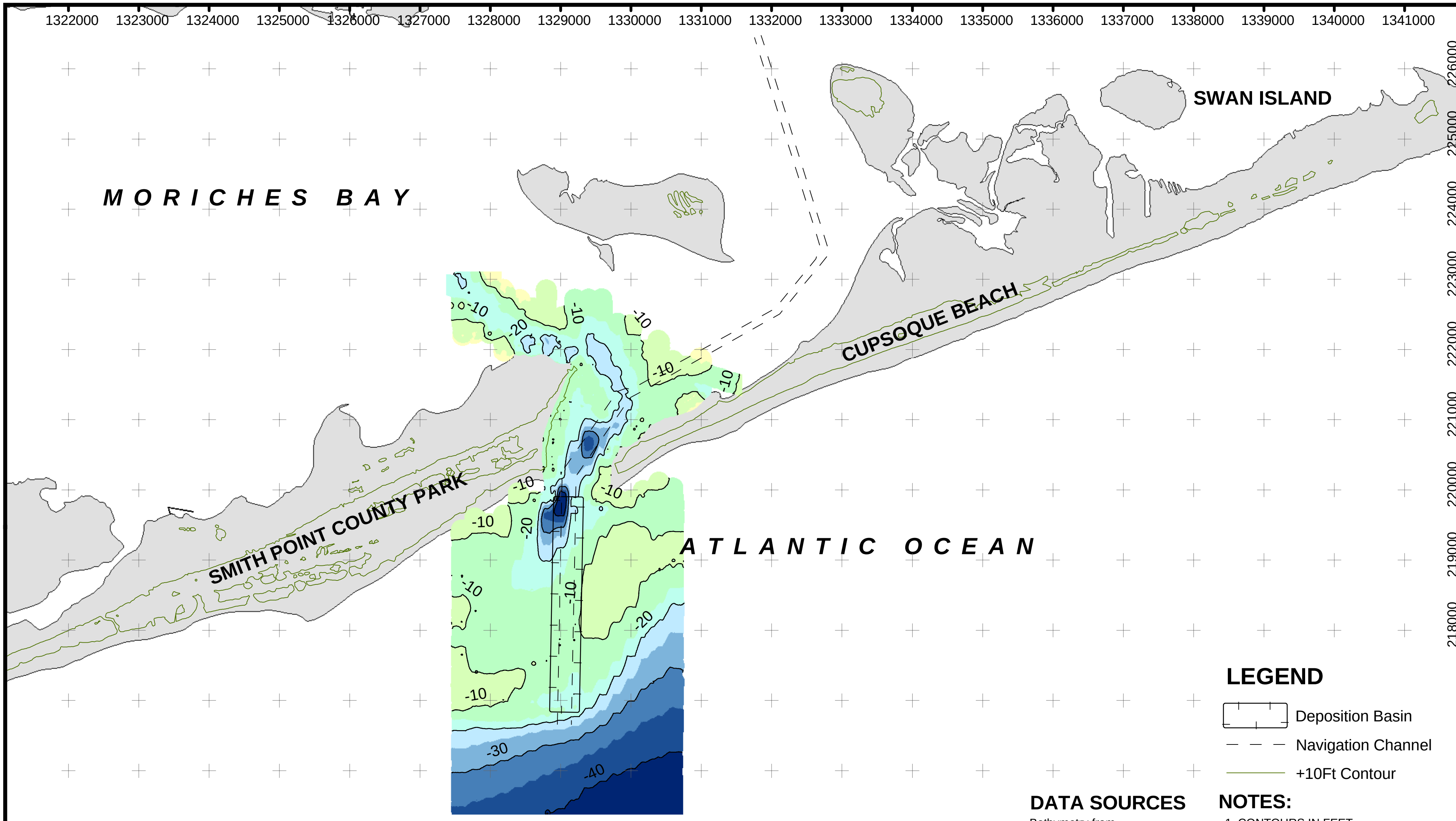
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE A-22

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Moriches_Depth.mxd



DATA SOURCES

Bathymetry from
14-15 April and 1 May 2000
Condition Survey Data

Shoreline and features from
1995 Aerial Photography

NOTES:

1. CONTOURS IN FEET REFERENCED TO NGVD 1929
2. PROJECTED COORDINATE IS NY STATE PLANE, LONG ISLAND ZONE NAD 1983

Moriches Inlet Bathymetry -April 2000

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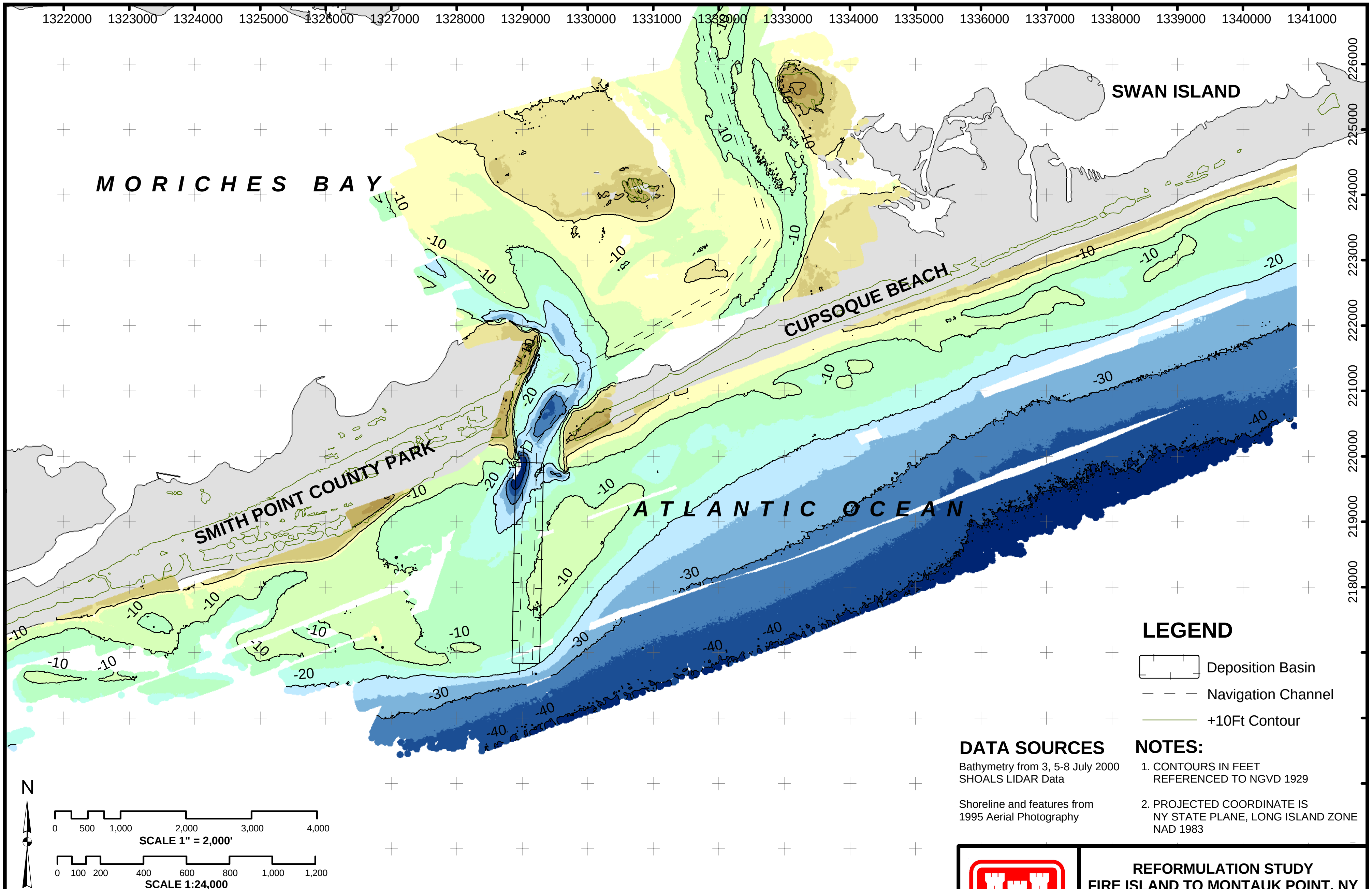
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE A-23

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/MI_CondSurvey.mxd



LEGEND

- Deposition Basin
- Navigation Channel
- +10Ft Contour

DATA SOURCES

Bathymetry from 3, 5-8 July 2000
SHOALS LIDAR Data

Shoreline and features from
1995 Aerial Photography

NOTES:

1. CONTOURS IN FEET REFERENCED TO NGVD 1929
2. PROJECTED COORDINATE IS NY STATE PLANE, LONG ISLAND ZONE NAD 1983

Moriches Inlet Bathymetry - July 2000

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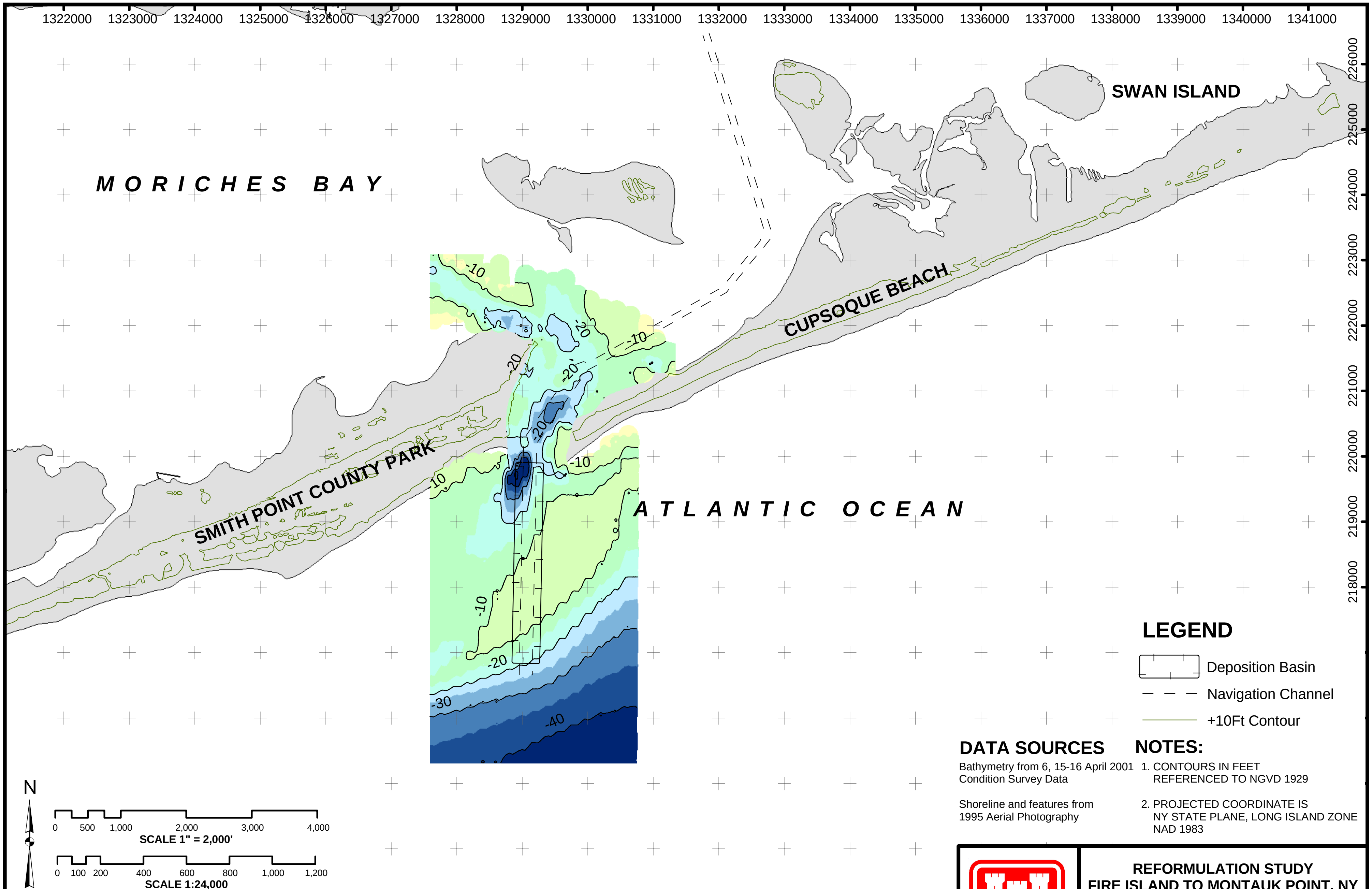
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE A-24

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/Moriches_Depth.mxd



DATA SOURCES

Bathymetry from 6, 15-16 April 2001 Condition Survey Data

Shoreline and features from 1995 Aerial Photography

NOTES:

1. CONTOURS IN FEET REFERENCED TO NGVD 1929
2. PROJECTED COORDINATE IS NY STATE PLANE, LONG ISLAND ZONE NAD 1983

Moriches Inlet Bathymetry - April 2001

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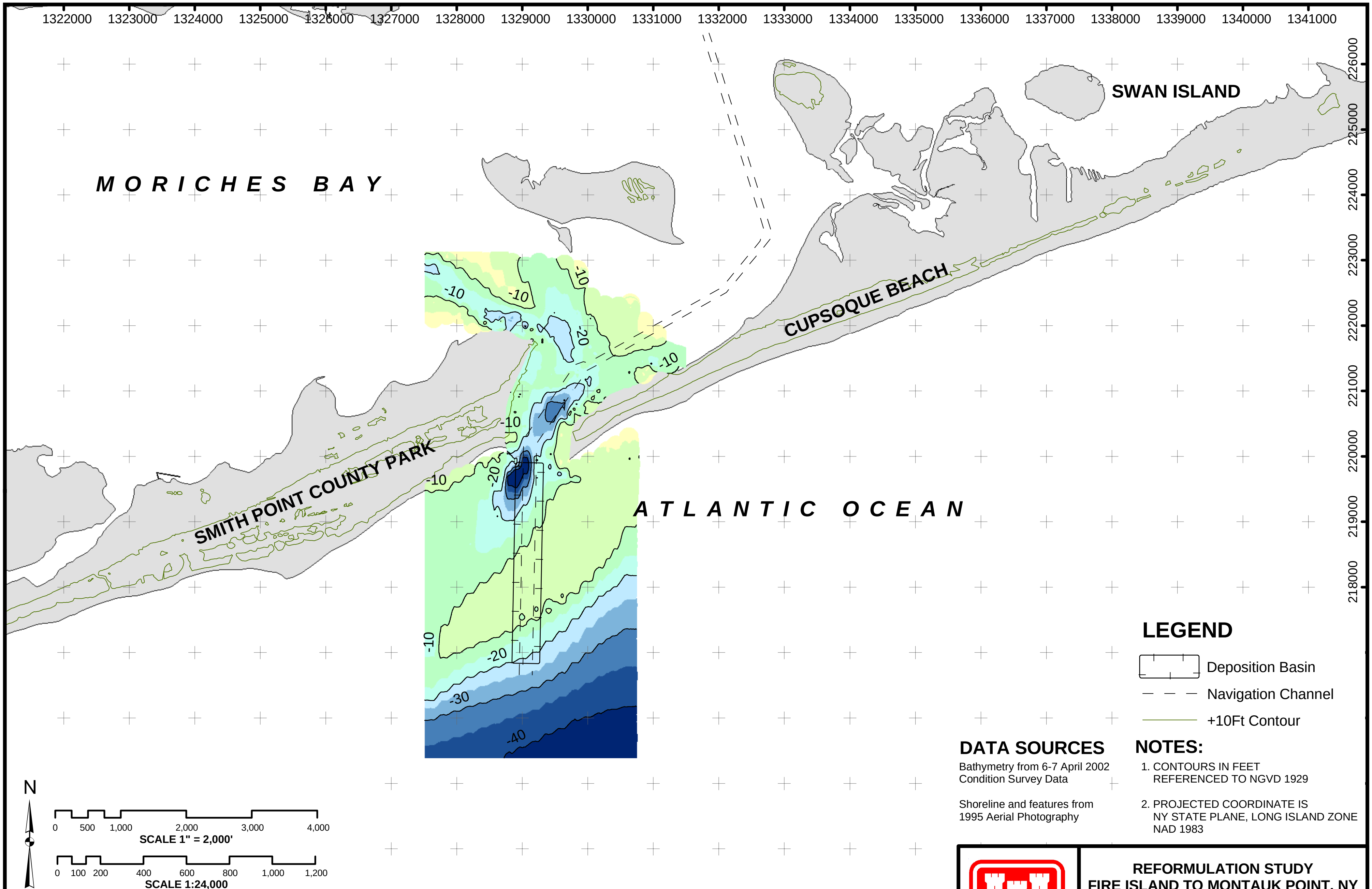
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE A-25

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/MI_CondSurvey.mxd



DATA SOURCES

Bathymetry from 6-7 April 2002
Condition Survey Data

Shoreline and features from
1995 Aerial Photography

NOTES:

1. CONTOURS IN FEET REFERENCED TO NGVD 1929
2. PROJECTED COORDINATE IS NY STATE PLANE, LONG ISLAND ZONE NAD 1983

Moriches Inlet Bathymetry - April 2002

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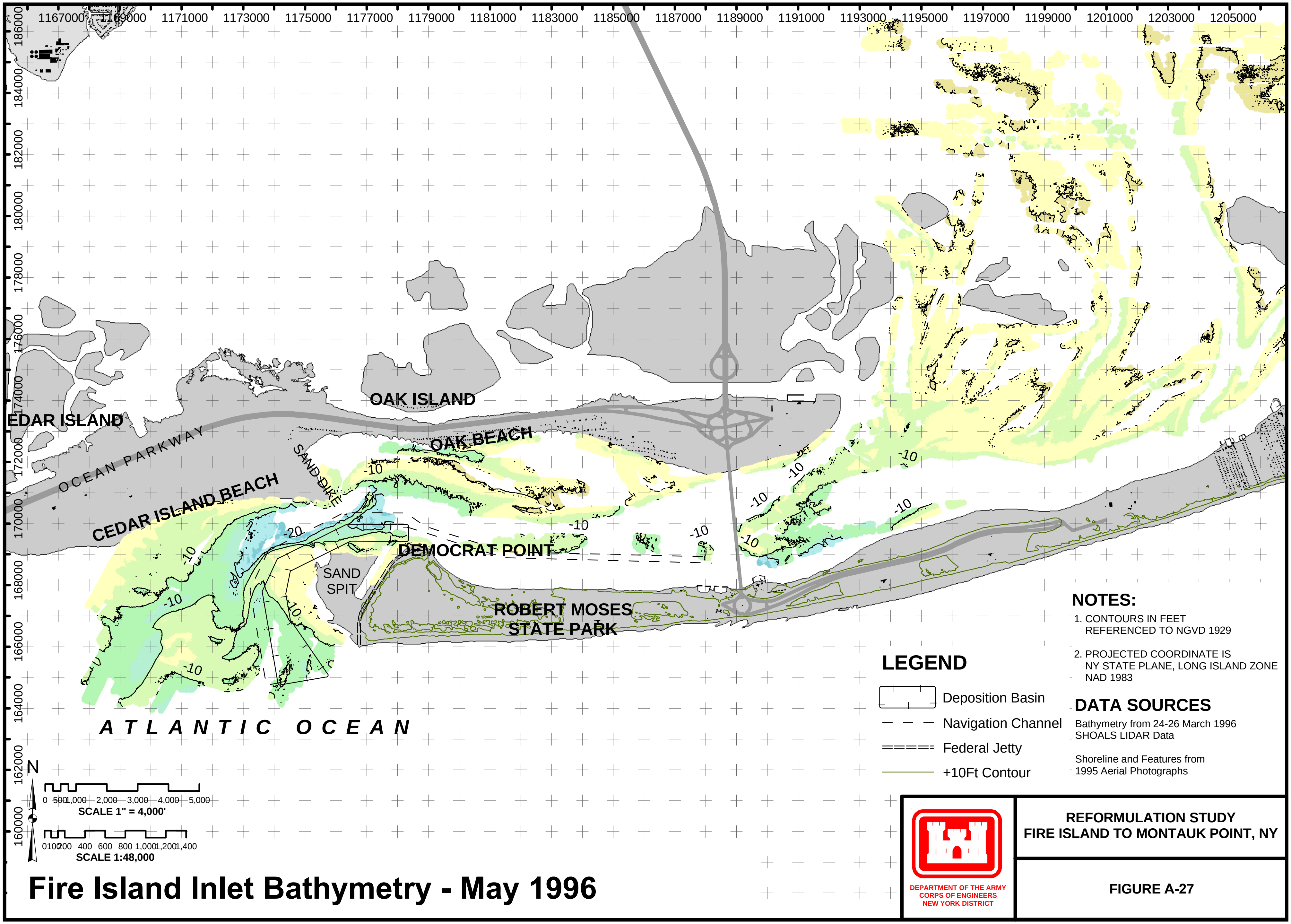
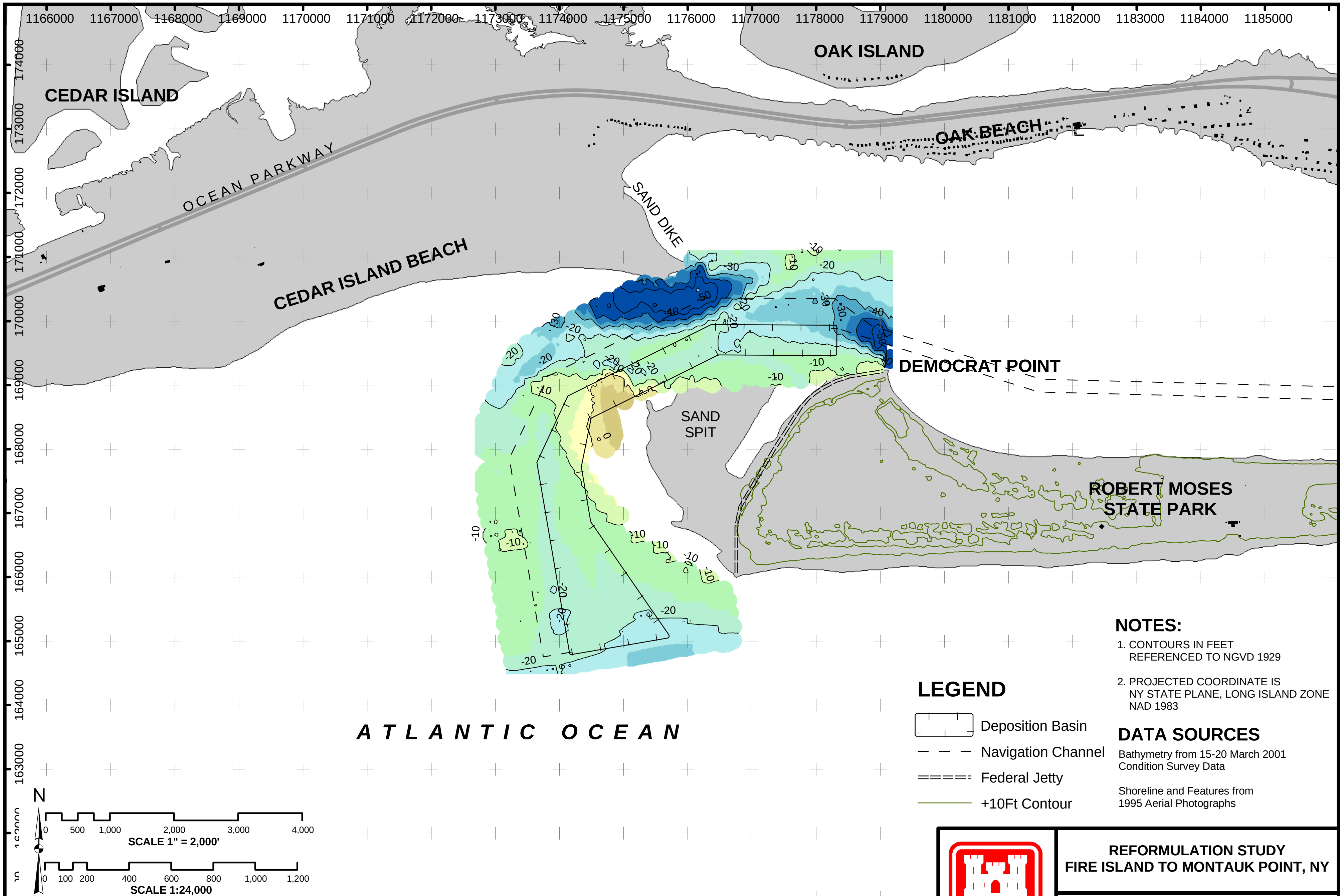
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE A-26

Filename: GIS/3982 FIMP/3982-28/Reports_MXDFiles/MI_CondSurvey.mxd



Filename: GIS/3982 FIMP/3982-23/Reports_MXDFiles/FireIsl_RogerFlood.mxd



NOTES:

1. CONTOURS IN FEET REFERENCED TO NGVD 1929
2. PROJECTED COORDINATE IS NY STATE PLANE, LONG ISLAND ZONE NAD 1983

LEGEND

- Deposition Basin
- Navigation Channel
- Federal Jetty
- +10Ft Contour

DATA SOURCES

Bathymetry from 15-20 March 2001 Condition Survey Data

Shoreline and Features from 1995 Aerial Photographs

Fire Island Inlet Bathymetry - March 2001

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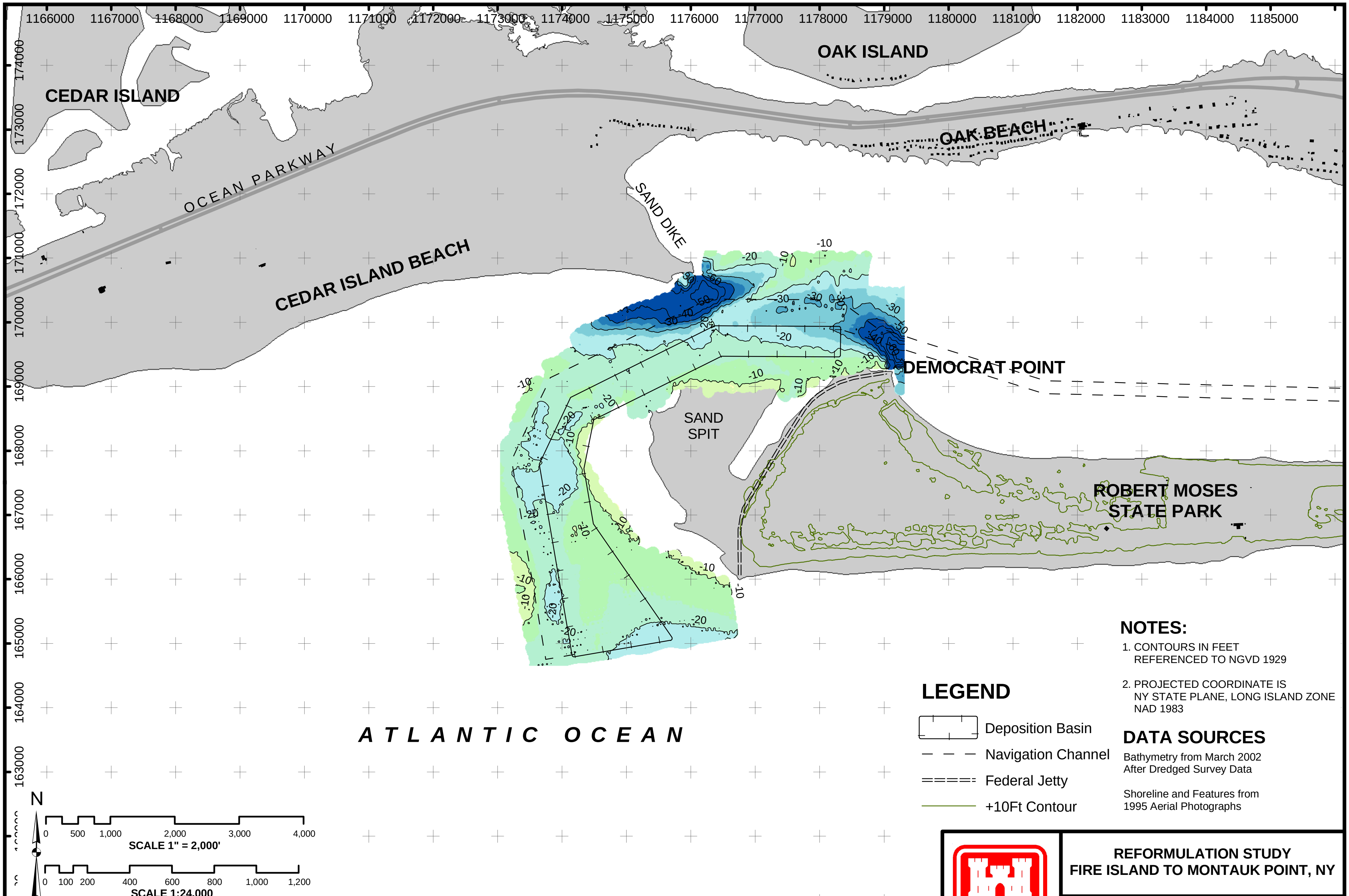
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE A-28

Filename: GIS/3982 FIMP/3982-23/Reports_MXDFiles/FireIsl_Depth.mxd



NOTES:

1. CONTOURS IN FEET REFERENCED TO NGVD 1929
2. PROJECTED COORDINATE IS NY STATE PLANE, LONG ISLAND ZONE NAD 1983

DATA SOURCES

Bathymetry from March 2002 After Dredged Survey Data

Shoreline and Features from 1995 Aerial Photographs

Fire Island Inlet Bathymetry - March 2002

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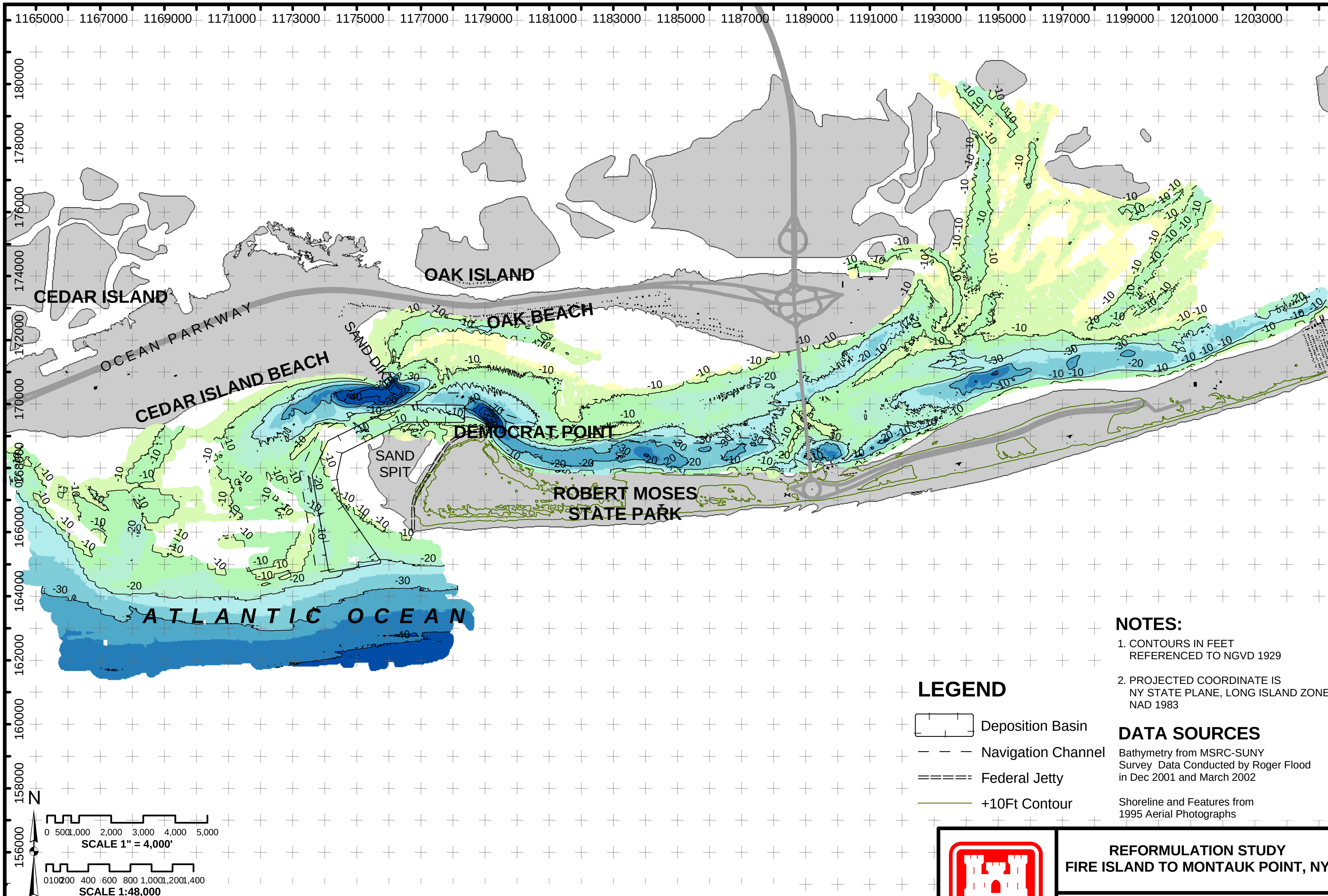
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE A-29

Filename: GIS/3982 FIMP/3982-23/Reports_MXDFiles/FireIsl_Depth.mxd



NOTES:

1. CONTOURS IN FEET REFERENCED TO NGVD 1929
2. PROJECTED COORDINATE IS NY STATE PLANE, LONG ISLAND ZONE NAD 1983

LEGEND

- Deposition Basin
- Navigation Channel
- Federal Jetty
- +10Ft Contour

DATA SOURCES

Bathymetry from MSRC-SUNY Survey Data Conducted by Roger Flood in Dec 2001 and March 2002

Shoreline and Features from 1995 Aerial Photographs

Fire Island Inlet Bathymetry - Dec 2001 to March 2002 (a)

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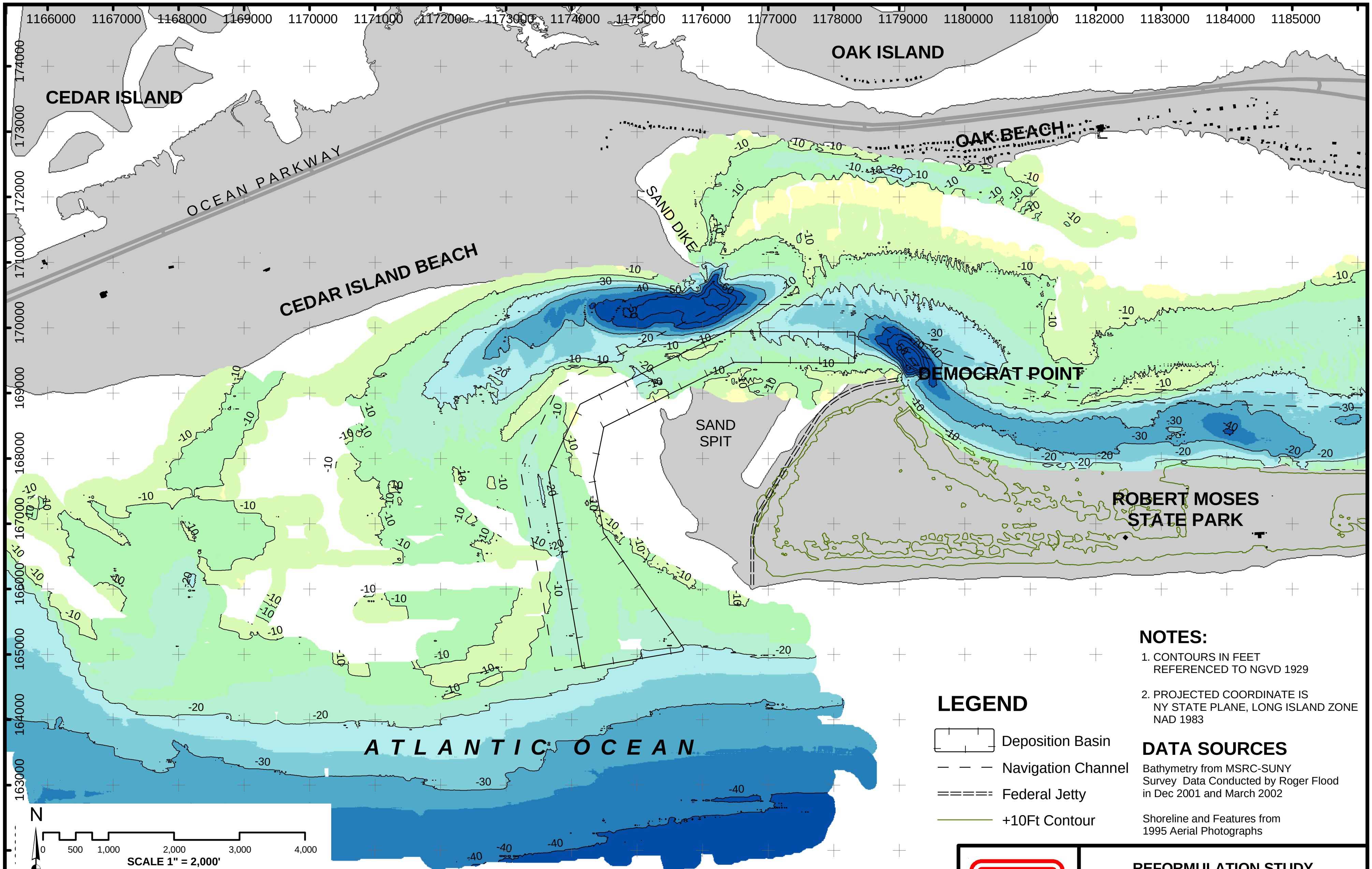
REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE A-30

Filename: GIS/3982 FIMP/3982-23/Reports_MXDFiles/FireIsl_RogerFlood.mxd



LEGEND

- Deposition Basin
- Navigation Channel
- Federal Jetty
- +10Ft Contour

NOTES:

1. CONTOURS IN FEET REFERENCED TO NGVD 1929
2. PROJECTED COORDINATE IS NY STATE PLANE, LONG ISLAND ZONE NAD 1983

DATA SOURCES

Bathymetry from MSRC-SUNY Survey Data Conducted by Roger Flood in Dec 2001 and March 2002

Shoreline and Features from 1995 Aerial Photographs

Fire Island Inlet Bathymetry - Dec 2001 to March 2002 (b)

DEPARTMENT OF THE ARMY
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NEW YORK DISTRICT

REFORMULATION STUDY
FIRE ISLAND TO MONTAUK POINT, NY

FIGURE A-31

Filename: GIS/3982 FIMP/3982-23/Reports_MXDFiles/FireIsl_RogerFlood.mxd

Appendix B – Data Sources

Table B-1 Data Inventory for Sediment Budget Update

Geographic Coverage	Data Type	Dates	Filename(s)	Datum	Comments
Fire Island Inlet	SHOALS Hydro survey	24-26 May 1996	fireis96.pts	State Plane Long Island, meters, NAD 83 NGVD	Survey adjusted –3.5 ft at all points to account for apparent systematic datum shift
Fire Island Inlet	Condition Survey	15-20 March 2001	FICond_2001.zip	State Plane Long Island, feet, NAD 83, NGVD (file adjusted using MLW 2 ft below NGVD)	Zip file contains grid and ASCII files
Fire Island Inlet	Single and Multi-beam Hydro Survey	December 2001-March 2002	Various	NAD83 (the maps are UTM, Zone 18), the vertical datum is MLLW. Oceanside points were adjusted to reflect MLLW is 1.71 ft below NGVD; Bayside points used 0.41 ft below NGVD	Data from Roger Flood, SUNY
Fire Island Inlet	Post-dredge survey	2002	FIPost2002.zip	State Plane Long Island, feet, NAD 83, NGVD (file adjusted using MLW 2 ft below NGVD)	Survey obtained from W. Vanterpool, Zip file contains grid and ASCII files
Moriches Inlet	Historical Survey (GEODAS)	1933	03F11621.txt 03F11622.txt 03F11623.txt 03F11627.txt	Lat-Long NAD 27, meters MLW	MLW of 1933 is estimated to be 1.7 ft below NGVD.
Moriches Inlet	Moriches Inlet, Long Isl. Condition Survey - After Beach Restoration	February-April 1981	morich81grd.zip	State Plane Long Island, feet, NAD 83, NGVD	Grid created from contours digitized from hardcopy map
Moriches Inlet	SHOALS Hydro Survey	22-23 May 1996	morich96.pts, MOR_SHIN.PTS	State Plane Long Island, meters, NAD 83, NGVD	
Moriches Inlet	Condition survey	11-13 March 1998	MORA-1.XYZ, MORA-2.XYZ, MORA-3.XYZ, MORA-4.XYZ	State Plane Long Island NAD 83 and Mean Low Water. The Corps of Engineers has established MLW as being 0.40 below NGVD. 29 for this project. Units are feet.	The 0.4 ft conversion is for the bay side benchmark, this survey was conducted using RTK GPS. Obtained from: Daniel W. Rogers, L.S.
Moriches Inlet	Pre-Dredge Survey	21 September 1998	1878.dgn, sep2198ngvd.zip	State Plane Long Island, feet, NAD 83, MLW, MLW is given as 1.7 ft below NGVD on the dgn file	dgn file with soundings, zip file is ArcView grid generated from soundings
Moriches Inlet	Post-Dredge Survey	20 October 1998	1890.dgn, oct2098ngvd.zip	State Plane Long Island, feet, NAD 83, MLW, MLW is given as 1.7 ft below NGVD on	dgn file with soundings, zip file is ArcView grid generated from soundings

Table B-1 Data Inventory for Sediment Budget Update					
Geographic Coverage	**Data Type**	**Dates**	**Filename(s)**	**Datum**	**Comments**
				the dgn file	
Moriches Inlet	SHOALS Hydro Survey	3, 5-8 July 2000	mori1_r.xyz, morizone1b.xyz, zone2_r.xyz, zone2b.xyz, zone3_r.xyz, zone3b.xyz, zone4_r.xyz, zone4b.xyz, zone5_r.xyz zone5b.xyz	State Plane Long Island, feet, NAD 83, NGVD	Zones 1 & 2 - Moriches Inlet Zones 3 & 4 - coast between Moriches & Shinnecock Zone 5 - Shinnecock Inlet
Moriches Inlet	Condition Survey	14-15 April, 1 May 2000	moriche1.dgn; moriche2.dgn	State Plane Long Island, feet, NAD 83, MLW	MLW is stated to be 0.3 ft below NGVD on the *.dgn files
Moriches Inlet	Condition Survey	6,15-16 April 2001	2155Sh1.dgn 2155Sh2.dgn	State Plane Long Island, feet, NAD 83, MLW	MLW is stated to be 0.3 ft below NGVD on the *.dgn files
Moriches Inlet	Condition Survey	6-7 April 2002	2277-dgn.ZIP	State Plane Long Island, feet, NAD 83, MLW	MLW is stated to be 0.3 ft below NGVD on the *.dgn files
Shinnecock Inlet	Historical Survey (GEODAS)	1933	03F11622.xyz 03F11623.xyz	Lat-Long NAD 27, meters MLW	
Shinnecock Inlet	Condition Survey	June 11, 1984	s6-11-84.pts	State Plane Long Island, feet, NAD 83 NGVD 29	Obtained from A. Morang, WES
Shinnecock Inlet	Condition Survey	15-22 Nov and 1,2,7-10 Dec 1989	SI-1289.DAT, SI-1289b.xyz	State Plane Long Island, feet, NAD 27 Mean Low Water	MLW is 1.2 feet below NGVD 29. Obtained from A. Morang, WES
Shinnecock Inlet	Condition Survey	August 1991	Sh-0891.dat	State Plane Long Island, feet, NAD 27 NGVD 29	Obtained from A. Morang, WES
Shinnecock Inlet	Condition Survey	December 1992	200.DAT, 200X.DAT, 50.DAT	State Plane Long Island, feet, NAD 27, vertical datum not confirmed	Obtained from A. Morang, WES
Shinnecock Inlet	Condition Survey	August 3-9, 1994	SH-0894A.DAT	State Plane Long Island, feet, NAD 27 NGVD 29	Obtained from A. Morang, WES
Shinnecock Inlet	Condition Survey	1994	SH-0894B.DAT	State Plane Long Island, feet, NAD 27 NGVD 29	Obtained from A. Morang, WES
Shinnecock Inlet	Condition Survey	October 4-5 and 9-11, 1995	SH-1095.DAT	State Plane Long Island, feet, NAD 27 NGVD 29	Obtained from A. Morang, WES
Shinnecock Inlet	SHOALS Hydro Survey	23 May to 02 June, 1996	shin96a.pts shin96b.pts	State Plane Long Island, feet, NAD 27, NGVD	
Shinnecock Inlet	SHOALS Hydro Survey	13 Aug 1997	shin97.pts	State Plane Long Island, feet, NAD 27, NGVD	
Shinnecock Inlet	Condition survey	4-6 March 1998	S-Mar98.pts	State Plane Long Island, feet, NAD 83, Mean Low Water, which is 1.5 ft below NGVD (1929)	Datum information in header of ASCII file
Shinnecock Inlet	"Echo Soundings Before Dredging"	19,20 Aug 1998	1859xyz.dat	State Plane Long Island, feet, NAD 83, Mean Low Water, which is 1.5 ft below NGVD	Datum information on hard copy map

Table B-1 Data Inventory for Sediment Budget Update

Geographic Coverage	Data Type	Dates	Filename(s)	Datum	Comments
				(1929)	
Shinnecock Inlet	"Before Dredging"	4,8,9 Sept 1998	1866xyz.dat	State Plane Long Island, feet, NAD 83, Mean Low Water, which is 1.5 ft below NGVD (1929)	Datum information on hard copy map
Shinnecock Inlet	"Echo Soundings After Dredging"	25,29 Sept 1998	1884xyz.dat	State Plane Long Island, feet, NAD 83, Mean Low Water, which is 1.5 ft below NGVD (1929)	Datum information on hard copy map
Shinnecock Inlet	SHOALS Hydro Survey	28 May 1998	shin98.pts	State Plane Long Island, feet, NAD 27, NGVD	
Shinnecock Inlet	SHOALS Hydro Survey	Jul 2000	See files listed in Moriches 2000 survey, files cover both inlets	State Plane Long Island, feet, NAD 83, NGVD	Zones 1 & 2 - Moriches Inlet Zones 3 & 4 - coast between Moriches & Shinnecock Zone 5 - Shinnecock Inlet
Shinnecock Inlet	SHOALS Hydro Survey	Jul 2001	020* (various)	State Plane Long Island, feet, NAD 83, Mean Sea Level	Mean Sea Level is taken to be 0.5 ft above NGVD
Shinnecock Inlet	Condition Survey	13 April 2000	SI_Cond00pnt.txt	State Plane Long Island, feet, NAD 83, NGVD	Converted from MLW using MLW is 1.5 ft below NGVD
Shinnecock Inlet	Condition Survey	5 April 2001	SI_Cond01pnt.txt	State Plane Long Island, feet, NAD 83, NGVD	Converted from MLW using MLW is 1.5 ft below NGVD
"Montauk"	Profile Data	S95, F95, S96, F96, S97, S98, F98, S01	ACNYMP.dbf (Coastal View database);Montauk.BM (BMAP from A. Morang);Mz.zip (Zipped text files)	State Plane Long Island, feet, NAD 83, NGVD	ACNYMP.dbf has data problems; BMAP file is corrected but not x,y,z; MZ.zip contains 43 text files in z, easting, northing, dist from monument format;
"Ponds"	Profile Data	S95, F95, S96, F96, S97, S98, F98, S01	ACNYMP.dbf (Coastal View database);Ponds.BM (BMAP from A. Morang);Pz.zip (Zipped text files)	State Plane Long Island, feet, NAD 83, NGVD	ACNYMP.dbf has data problems; BMAP file is corrected but not x,y,z; Pz.zip contains 55 text files in z, easting, northing, dist from monument format;
"Westhampton"	Profile Data	S95, F95, S96, F96, S97, S98, F98, S99, F99, S00, S01	ACNYMP.dbf (Coastal View database);Westhampton.BM (BMAP from A. Morang);Wh.zip (Zipped text files)	State Plane Long Island, feet, NAD 83, NGVD	ACNYMP.dbf has data problems; BMAP file is corrected but not x,y,z; Wh.zip contains 193 text files in z, easting, northing, dist from monument format;
"Fire Island"	Profile Data	S95, F95, S96, F96, S97, S97, S98, F98, S01	ACNYMP.dbf (Coastal View database); FI_a.BM (Fire Island West-BMAP from A. Morang); FI_b.BM (Fire Island East BMAP); Fi.zip (Zipped text files)	State Plane Long Island, feet, NAD 83, NGVD	ACNYMP.dbf has data problems; BMAP file is corrected but not x,y,z; Fi.zip contains 147 text files in z,

Table B-1 Data Inventory for Sediment Budget Update					
Geographic Coverage	**Data Type**	**Dates**	**Filename(s)**	**Datum**	**Comments**
					easting, northing, dist from monument format;
West of Shinnecock	Short Profiles	November 1998 to July 2002	post-Transect1.BM; post-Transect10.BM; post-Transect3.BM; post-Transect7.BM; post-Transect9.BM; volspostnourishment.xls	Station-Elevation information, vertical datum is NGVD, units are meters.	Profiles in BMAP format from Brian Batten at SUNY; xls data analysis
FIMP area	Aerial Photos	1995, 1996, 1997, 1998, 1999, 2000, 2001	Various		Photographs are not rectified.
FIMP area	Rectified Aerial Photos	April 2001	*.tif	State Plane Long Island NAD 83, feet	From Suffolk County, NY,
FIMP area	Topographic Survey	1995	Misc ArcView		Detailed survey from Erdman-Anthony.
FIMP area	Historic Shoreline Data	1830, 1870, 1887 (no months available), Feb-May 1933, Oct 1938, Mar 1962, Dec 1979, April 1983, March 1988, and March/April 1995	FI_FIMP.xls, G_FIMP.xls, MP_FIMP.xls, WH_FIMP.xls Readme.doc	State Plane Long Island NAD 83, meters	XLS files and Doc readme
FIMP area	Shoreline Data	April 2001	SentShoreLines.ZIP	State Plane Long Island NAD 83, feet	Digitized by Daniel J. Kriesant Hydraulic Engineer USACE New York District
Fire Island Area	Shoreline Data	1979, 8/1993, 8/1995, 8/1996, 9/1994, 9/1997, 9/1998, 9/1999, 9/2001	FiXYYmhw, where X is the last digit of the year and YY is the month, eg, fi308mhw is August 1993.	UTM Zone 18, NAD 83, meters	Arc View Coverages
West of Shinnecock	Profile Data	Mar 4-7 2002	WofShinn2002_xyz.3D; WofShinn2002_profiles.dgn	NGVD29 and NAD83 State Plane Long Island (feet).	Metadata also included in directory (*.met)
ACNYMP Area	Short Profile Data	Spring 2002	ASCII.zip – text files, also *.bm, BMAP files, *.met, metadata files	NGVD29 and NAD83 State Plane Long Island (feet).	Metadata also included in directory (*.met), Note from Jen Irish that problems have been found with this data set.

Appendix C – Engineering Activities

Table C-1. Engineering Event Log: 1995 to 2002

Date	Locality	Stationing (km) West Boundary	Stationing (km) East Boundary	Volume Placed (m^3)	Source of Information	Comments
Fire Island to Moriches Inlet						
Feb 1997-Apr 1997	Fire Island Inlet	0	0	0	CENAN Records (6)	Fire Island Inlet dredged, 827,140 m^3 removed, 549,650 m^3 to Gilgo and 227,490 m^3 to Robert Moses State Park (RMSP)
Nov 1999 to Mar 2000	Fire Island Inlet	0	0	0	CENAN Records (6)	Fire Island Inlet dredged, 846,910 m^3 removed, 743,400 m^3 to Gilgo and 103,510 m^3 to RMSP
Dec 2001 to Mar 2002	Fire Island Inlet	0	0	0	CENAN Records (6)	Fire Island Inlet dredged, 1,139,780 m^3 removed, 1,013,790 m3 to Gilgo and 125,990 m3 to RMSP
1997	Robert Moses State Park	0.6	3.8	277,490	CENAN Records (6)	Fill from Fire Island Inlet dredging
1999 to 2000	Robert Moses State Park	0.6	3.8	103,510	CENAN Records (6)	Fill from Fire Island Inlet dredging
Dec 2001 to Mar 2002	Robert Moses State Park	0.6	3.8	125,990	CENAN Records (6)	Fill from Fire Island Inlet dredging
1997	Fire Island Pines	19.9	21.8	513,524	CP&E (6)	Beach Fill from Fire Island Pines borrow area
Nov 2003	Fire Island Pines	19.9	21.8	380,000		Beach Fill from Fire Island Pines borrow area
1996	Water Island and Barrett Beach	24.1	26.025	57,300	Suffolk County (1)*	Predominantly sand, from bay boat channels
Jan 1996	Smith Point Boardwalk	39.65	39.8	41,400	Suffolk County (2)*	Fill from Smith Point Marina
Jun 1996	Smith Point Boardwalk	39.65	39.8	5,000	Suffolk County (2)*	Fill from Smith Point Marina
				5,900		Fill from Orchard Neck
				2,400		Fill from Calverton National Cemetery
Dec 1996 – Jan 1997	Smith Point Boardwalk	39.65	39.8	8,300	Suffolk County (2)*	Fill from Calverton National Cemetery
2001	Smith Point County Park Boardwalk	39.7	38.8	7,646	CENAN Fact Sheet (6)	Fill from Long Island Intracoastal Waterway, could be 7,646 to 11,468 m^3
1995	Great Gun Beach	47.5	47.875	30,600	Suffolk County (1)*	Predominantly mud; not included in budget calculations

Table C-1. Engineering Event Log: 1995 to 2002

Date	Locality	Stationing (km) West Boundary	Stationing (km) East Boundary	Volume Placed (m^3)	Source of Information	Comments
1996	West of Moriches Inlet	47.9	49.225	176,210	CENAN Records (6)*(updated)	From Moriches Inlet dredging, placed in offshore berm (-1 to -2 m) approximately 1.6 km west of inlet
Oct 1998	West of Moriches Inlet	48.7	49	142,350	NY State Department of Environmental Conservation (7)	Beach fill from Moriches Inlet dredging
Moriches Inlet to Shinnecock Inlet						
Jan, Feb and Mar 1996	Moriches Inlet	0	0	0	CENAN Records (6)* (updated)	Moriches Inlet dredged; total 196,210 m^3 removed: 20,000 m^3 stockpiled 1.6 km east of inlet; 176,210 m^3 placed in offshore berm (below –1.5 m NGVD) approximately 1.6 km west of inlet
Oct 1998	Moriches Inlet	0	0	0	CENAN Records (6)	Moriches Inlet dredged, 142,350 m^3 removed.
1996	East of Moriches	1.6	1.6	20,000	CENAN Records (6)* (updated)	From Moriches Inlet dredging – stockpiled (see above), not included in budget calculations
May-Sep 1996	Westhampton Beach	3.2	6.5	1,912,000	CENAN Records (6)*	Westhampton Interim Project, from offshore borrow area
1997	Westhampton Beach	6.525	11.2	1,012,300	CENAN Records (6)*	Westhampton Interim Project, from offshore borrow area
Nov 2000 to Mar 2001	Westhampton	3.2	11.2	723,259	CENAN Records (6)	Westhampton Interim Project, from offshore borrow area
1996	Quogue	12.125 (assumed)	19.125 (assumed)	0	Village of Quogue (5)*	35-m geotube installed at base of seaward side of dune after winter storms of 1994/95; covered with sand; planted with beach grass; site of 1994 entry
1996	Quogue	12.125 (assumed)	19.125 (assumed)	1,200	Village of Quogue (5)*	Restoration for 28 m of dune; upland sand
1996	Quogue	12.125 (assumed)	19.125 (assumed)	0	Village of Quogue (5)*	Two stacked geotubes installed at base of dune 61-m length; tubes covered and planted with grass
1996	Quogue	12.125 (assumed)	19.125 (assumed)	0	Village of Quogue (5)*	Permit approved for Patented Subsurface Dune restoration System at 188 Dune Road; involves placement of 20 sand-filled containers along face of dune
1997	West of Shinnecock Inlet	24.375	24.8	191,150	NY State*	From Shinnecock East Cut

Table C-1. Engineering Event Log: 1995 to 2002						
Date	**Locality**	**Stationing (km)**		**Volume Placed (m^3)**	**Source of Information**	**Comments**
		West Boundary	**East Boundary**			
27 Jun – 11 Jul 1998	West of Shinnecock Inlet	24.1	24.65	26,000	CENAN Records (6)	From Shinnecock Inlet dredging
13-25 Sep 1998	West of Shinnecock Inlet	23.73	24.8	310,400	CENAN Records (6)	From Shinnecock Inlet dredging
Shinnecock Inlet to Montauk Point						
27 Jun – 11 Jul 1998	Shinnecock Inlet	0	0	0	CENAN Records (6)	Phase 1 dredging, 26,000 m^3 removed from entrance channel and deposition basin above –4.3 m contour, placed in surf zone of wet beach from 150 m west of west jetty to 550 m west of west jetty
13-25 Sep 1998	Shinnecock Inlet	0	0	0	CENAN Records (6)	Phase 2 dredging, 310,400 m^3 removed from entrance channel and deposition basin above –6.7 contour, placed on west beach between west jetty and 1070 m west
1995	Ditch Plains	51.6	52.65	1,530	CENAN Records (6)	From stockpile of 3-Mile Harbor spoils
1997	Ditch Plains	51.6	52.65	1,380	CENAN Records (6)	From stockpile of 3-Mile Harbor spoils
2001	Ditch Plains	51.6	52.65	1,530	CENAN Records (6)	Dredged from 3-Mile Harbor

(1) Mr. William D. Lifford, Suffolk County Department of Public Works, 335 Yaphank Avenue, Yaphank, NY 11980.
(2) Mr. Daniel J. Pendzick, Suffolk County Department of Public Works, 335 Yaphank Avenue, Yaphank, NY 11980.
(3) CENAN Operations files (S.U. McKnight), courtesy of Dr. Andrew Morang, CHL.
(4) Annual Report of Chief of Engineers, 1992, courtesy of Dr. Andrew Morang, CHL.
(5) Ms. Thelma Georgeson, Mayor, Village of Quogue, P.O. Box 926, Quogue NY, 11959-0926.
(6) CENAN records, courtesy of Ms. Christina Rasmussen, CENAN.
(7) Mr. William W. Daley, Director, Bureau of Flood Protection, NY State Department of Environmental Conservation, 625 Broadway, Albany, NY 12233-3507.
*Published in Gravens, et al. (1999)

Appendix D – Summary of NYSDOS CMP Policies

DEVELOPMENT POLICIES

Policy 1 **Waterfront Revitalization:** Restore, Revitalize, And Redevelop Deteriorated And Underutilized Waterfront Areas For Commercial, Industrial, Cultural, Recreational, And Other Compatible Uses.

Policy 2 **Water-Dependent Uses:** Facilitate The Siting Of Water-Dependent Uses And Facilities On Or Adjacent To Coastal Waters.

Policy 3 **Major Ports:** Further Develop The State's Major Ports Of Albany, Buffalo, New York, Ogdensburg, And Oswego As Centers Of Commerce And Industry, And Encourage The Siting, In These Port Areas, Including Those Under The Jurisdiction Of State Public Authorities, Of Land Use And Development Which Is Essential To, Or In Support Of, The Waterborne Transportation Of Cargo And People.

Policy 4 **Small Harbors:** Small Harbors: Strengthen The Economic Base Of Smaller Harbor Areas By Encouraging The Development And Enhancement Of Those Traditional Uses And Activities Which Have Provided Such Areas With Their Unique Maritime Identity.

Policy 5 **Public Services:** Encourage The Location Of Development In Areas Where Public Services And Facilities Essential To Such Development Are Adequate.

Policy 6 **Permit Procedures:** Expedite Permit Procedures In Order To Facilitate The Siting Of Development Activities At Suitable Locations.

FISH AND WILDLIFE POLICIES

Policy 7 **Significant Habitats:** Significant Coastal Fish And Wildlife Habitats Will Be Protected, Preserved, And Where Practical, Restored So As To Maintain Their Viability As Habitats.

Policy 8 **Pollutants:** Protect Fish And Wildlife Resources In The Coastal Area From The Introduction Of Hazardous Wastes And Other Pollutants Which Bio-Accumulate In The Food Chain Or Which Cause Significant Sublethal Or Lethal Effect On Those Resources.

Policy 9 **Recreational Resources:** Expand Recreational Use Of Fish And Wildlife Resources In Coastal Areas By Increasing Access To Existing Resources, Supplementing Existing Stocks, And Developing New Resources.

Policy 10 **Commercial Fisheries:** Further Develop Commercial Finfish, Shellfish, And Crustacean Resources In The Coastal Area By Encouraging The Construction Of New, Or Improvement Of Existing On-Shore Commercial Fishing Facilities, Increasing Marketing Of The State's Seafood Products, Maintaining Adequate Stocks, And Expanding Aquaculture Facilities.

FLOODING AND EROSION HAZARDS POLICIES

Policy 11 **Siting Structures:** Buildings And Other Structures Will Be Sited In The Coastal Area So As To Minimize Damage To Property And The Endangering Of Human Lives Caused By Flooding And Erosion.

Policy 12 **Natural Protective Features:** Activities Or Development In The Coastal Area Will Be Undertaken So As To Minimize Damage To Natural Resources And Property From Flooding

And Erosion By Protecting Natural Protective Features Including Beaches, Dunes, Barrier Islands And Bluffs.

Policy 13 **30-Year Erosion Control Structures:** The Construction Or Reconstruction Of Erosion Protection Structures Shall Be Undertaken Only If They Have A Reasonable Probability Of Controlling Erosion For At Least Thirty Years As Demonstrated In Design And Construction Standards And/Or Assured Maintenance Or Replacement Programs.

Policy 14 **No Flooding or Erosion Increases:** Activities And Development, Including The Construction Or Reconstruction Of Erosion Protection Structures, Shall Be Undertaken So That There Will Be No Measurable Increase In Erosion Or Flooding At The Site Of Such Activities Or Development, Or At Other Locations.

Policy 15 **Natural Coastal Processes:** Mining, Excavation Or Dredging In Coastal Waters Shall Not Significantly Interfere With The Natural Coastal Processes Which Supply Beach Materials To Land Adjacent To Such Waters And Shall Be Undertaken In A Manner Which Will Not Cause An Increase In Erosion Of Such Land.

Policy 16 **Use of Public Funds:** Public Funds Shall Only Be Used For Erosion Protective Structures Where Necessary To Protect Human Life, And New Development Which Requires A Location Within Or Adjacent To An Erosion Hazard Area To Be Able To Function, Or Existing Development; And Only Where The Public Benefits Outweigh The Long Term Monetary And Other Costs Including The Potential For Increasing Erosion And Adverse Effects On Natural Protective Features.

Policy 17 **Non-structural Control Measures:** Non-Structural Measures To Minimize Damage To Natural Resources And Property From Flooding And Erosion Shall Be Used Whenever Possible.

GENERAL POLICY

Policy 18 **Safeguard State Interests:** To Safeguard The Vital Economic, Social And Environmental Interests Of The State And Of Its Citizens, Proposed Major Actions In The Coastal Area Must Give Full Consideration To Those Interests, And To The Safeguards Which The State Has Established To Protect Valuable Coastal Resource Areas.

PUBLIC ACCESS POLICIES

Policy 19 **Water-Related Recreation Resources:** Protect, Maintain, And Increase The Level And Types Of Access To Public Water-Related Recreation Resources And Facilities.

Policy 20 **Public Foreshore:** Access To The Publicly-Owned Foreshore And To Lands Immediately Adjacent To The Foreshore Or The Water's Edge That Are Publicly-Owned Shall Be Provided And It Shall Be Provided In A Manner Compatible With Adjoining Uses.

RECREATION POLICIES

Policy 21 **Water-Dependent/Water-Enhanced Recreation:** Water-Dependent And Water-Enhanced Recreation Will Be Encouraged And Facilitated, And Will Be Given Priority Over Non-Water-Related Used Along The Coast.

Policy 22 **Multiple-Use Development:** Development, When Located Adjacent To The Shore, Will Provide For Water-Related Recreation, Whenever Such Use Is Compatible With Reasonably Anticipated Demand For Such Activities, And Is Compatible With The Primary Purpose Of The Development.

HISTORIC AND SCENIC RESOURCES POLICIES

Policy 23 **Historic Preservation:** Protect, Enhance And Restore Structures, Districts, Areas Or Sites That Are Of Significance In The History, Architecture, Archaeology Or Culture Of The State, Its Communities, Or The Nation.

Policy 24 **Statewide Scenic Resources:** Prevent Impairment Of Scenic Resources Of Statewide Significance.

Policy 25 **Local Scenic Resources:** Protect, Restore Or Enhance Natural And Man-Made Resources Which Are Not Identified As Being Of Statewide Significance, But Which Contribute To The Overall Scenic Quality Of The Coastal Area.

AGRICULTURAL LANDS POLICY

Policy 26 **Conserve Agricultural Lands:** Conserve And Protect Agricultural Lands In The State's Coastal Area.

ENERGY AND ICE MANAGEMENT POLICIES

Policy 27 **Energy Facility Siting and Construction:** Decisions On The Siting And Construction Of Major Energy Facilities In The Coastal Area Will Be Based On Public Energy Needs, Compatibility Of Such Facilities With The Environment, And The Facility's Need For A Shorefront Location.

Policy 28 **Ice Management Practices:** Ice Management Practices Shall Not Interfere With The Production Of Hydroelectric Power, Damage Significant Fish And Wildlife And Their Habitats, Or Increase Shoreline Erosion Or Flooding.

Policy 29 **Energy Resources Development:** Encourage The Development Of Energy Resources On The Outer Continental Shelf, In Lake Erie And In Other Water Bodies, And Ensure The Environmental Safety Of Such Activities.

WATER AND AIR RESOURCES POLICIES

Policy 30 **State and National Water Quality Standards:** Municipal, Industrial, And Commercial Discharge Of Pollutants, Including But Not Limited To, Toxic And Hazardous Substances, Into Coastal Waters Will Conform To State And National Water Quality Standards.

Policy 31 **LWRP Policies and Constraints:** State Coastal Area Policies And Management Objectives Of Approved Local Waterfront Revitalization Programs Will Be Considered While Reviewing Coastal Water Classifications And While Modifying Water Quality Standards; However, Those Waters Already Overburdened With Contaminants Will Be Recognized As Being A Development Constraint.

Policy 32 **Innovative Sanitary Waste Systems:** Encourage The Use Of Alternative Or Innovative Sanitary Waste Systems In Small Communities Where The Costs Of Conventional Facilities Are Unreasonably High, Given The Size Of The Existing Tax Base Of These Communities.

Policy 33 **Stormwater Runoff/ Combined Sewers:** Best Management Practices Will Be Used To Ensure The Control Of Stormwater Runoff And Combined Sewer Overflows Draining Into Coastal Waters.

Policy 34 **Vessel Discharges:** Discharge Of Waste Materials Into Coastal Waters From Vessels Subject To State Jurisdiction Will Be Limited So As To Protect Significant Fish And Wildlife Habitats, Recreational Areas And Water Supply Areas.

Policy 35 **Dredging and Disposal:** Dredging And Filling In Coastal Waters And Disposal Of Dredged Material Will Be Undertaken In A Manner That Meets Existing State Permit Requirements, And Protects Significant Fish And Wildlife Habitats, Scenic Resources, Natural Protective Features, Important Agricultural Lands, And Wetlands".

Policy 36 **Hazardous Material Spills:** Activities Related To The Shipment And Storage Of Petroleum And Other Hazardous Materials Will Be Conducted In A Manner That Will Prevent Or At Least Minimize Spills Into Coastal Waters; All Practicable Efforts Will Be Undertaken To Expedite The Cleanup Of Such Discharges; And Restitution For Damages Will Be Required When These Spills Occur.

Policy 37 **Non-point Pollution Discharges:** Best Management Practices Will Be Utilized To Minimize The Non-Point Discharge Of Excess Nutrients, Organics And Eroded Soils Into Coastal Waters.

Policy 38 **Surface and Ground Industrial Discharges:** The Quality And Quantity Of Surface Water And Groundwater Supplies, Will Be Conserved And Protected, Particularly Where Such Watersconstitute The Primary Or Sole Source Of Water Supply.

Policy 39 **Solid Waste Management:** The Transport, Storage, Treatment And Disposal Of Solid Wastes, Particularly Hazardous Wastes, Within Coastal Areas Will Be Conducted In Such A Manner So As To Protect Groundwater And Surface Water Supplies, Significant Fish And Wildlife Habitats, Recreation Areas, Important Agricultural Land, And Scenic Resources.

Policy 40 **Industrial Discharges:** Effluent Discharged From Major Steam Electric Generating And Industrial Facilities Into Coastal Waters Will Not Be Unduly Injurious To Fish And Wildlife And Shall Conform To State Water Quality Standards.

Policy 41 **State and National Air Quality Standards:** Land Use Or Development In The Coastal Area Will Not Cause National Or State Air Quality Standards To Be Violated.

Policy 42 **Clean Air Act – Reclassifications:** Coastal Management Policies Will Be Considered If The State Reclassifies Land Areas Pursuant To The Prevention Of Significant Deterioration Regulations Of The Federal Clean Air Act.

Policy 43 **Acid Rain:** Land Use Or Development In The Coastal Area Must Not Cause The Generation Of Significant Amounts Of Acid Rain Precursors: Nitrates And Sulfates.

Policy 44 **Tidal and Freshwater Wetlands:** Preserve And Protect Tidal And Freshwater Wetlands And Preserve The Benefits Derived From These Areas.

Appendix E – Costs

E1. INTRODUCTION

This Appendix presents cost estimates, including initial construction, annual maintenance, and total annual costs, for the detailed inlet modification alternatives presented in Section 0. The following sections summarize the basic parameters and assumptions used to develop these cost estimates.

The interest rate used in all the costs estimates is 5.125%. "Subtotal Costs" shown in the following tables include a 15% allocation for uncertainty. Engineering and design (E&D) fees are 7% of the subtotal, while supervision and administration (S&A) fees are a percentage of the subtotal, given by the following formula:

$$S\ \&\ A\% = \frac{17 - \left(2.1 * \log\left(\frac{Subtotal}{1000}\right)\right)}{100}$$

The cost estimate of all alternatives that do not eliminate the longshore sediment transport (LST) deficit, take it into account by assuming that it would be eventually made up by dredging sand from an offshore borrow site and placing it downdrift of the inlet system. It is assumed that the dredging to compensate for the LST deficit would occur on a ten year cycle for Moriches and Shinnecock Inlet and four year cycle for Fire Island Inlet. Note that this method of assigning a cost to the LST deficit does not mean that offshore dredging is a part of the alternative.

The cost estimates do not take into account damages related to a more or less eroded shoreline condition downdrift (or updrift) of the inlet.

E2. SHINNECOCK INLET

Based on results from the preliminary screening, eight alternatives were considered for detailed analysis at Shinnecock Inlet. Each alternative can achieve the goals of providing reliable navigation through the federal navigation channel, restoring natural sediment pathways, and reducing adjacent shoreline erosion, albeit to varying degrees and at different costs. Table E-1 presents the estimated annual cost for each alternative as well as Existing Practice.

Table E-1. Summary of Annual Cost for Shinnecock Inlet Alternatives

Plan	Annual Cost ($000)
SI Existing Practice	*$1,800*
Alt 1A. Authorized Project (AP) + Ebb Shoal Dredging every 4 years	$1,438
Alt 1B. AP + Ebb Shoal Dredging every 2 years	$1,760
Alt 2. AP + Nearshore Structures (T-groins)	$2,993
Alt 3. AP + Offshore Dredging for West Beach	$2,280
Alt 4. AP + Semi-fixed Bypass System	$2,559
Alt 5A. -18 ft MLW Deposition Basin	$1,897
Alt 5B. -16 ft MLW Deposition Basin	$2,108
Alt 6A. AP + Flood Shoal Dredging every 4 years	$1,438
Alt 6B. AP + Flood Shoal Dredging every 2 years	$1,760
Alt 7. AP + Shortening the East Jetty	$2,799
Alt 8. AP + West Jetty Spur	$2,191

For Shinnecock Inlet, two types of dredging are considered: nearshore and offshore. For nearshore dredging (i.e., within the deposition basin, ebb shoal, or flood shoal), a mobilization/demobilization cost of $1,000,000 is assumed for a typical 30” cutterhead dredge. The unit price at Shinnecock Inlet ($5.00/cy) was determined using standard cost estimation techniques employed by dredging contractors which take into account the quantity of material to be dredged and the distance to the placement site. At Shinnecock Inlet, a production rate of 25,200 cubic yards per work-day is used. The daily cost of a 30” cutterhead dredge is assumed to be $120,000, which is the same daily rate used in the Breach Closure Plan cost estimates (March 2006). From the cost per day, the volume required and the production rate, the unit price per cubic yard were computed. The estimated mobilization/demobilization cost and the unit price per cubic yard are in line with the three most recent nearshore dredging contracts executed at Shinnecock Inlet, which are summarized in Table E-2.

Table E-2. Summary of Recent Nearshore Dredging Contracts at Shinnecock Inlet

Year	Mobilization & Demobilization	Unit Price ($/cy)	Volume (cy)
1993	$380,000	$2.50	500,017
1998	$350,000	$6.11	405,139
2004	$1,149,000	$4.85	302,509

Offshore dredging utilizes the FIMP offshore borrow sites identified in the Borrow Source Investigations Report (September 29, 2005). Offshore dredging is specifically used in Alternative 3 and is also used to assign a cost to the LST deficit associated with each alternative. As with nearshore dredging, a typical mobilization cost of $1,000,000 is assumed. The unit price per cubic yard for offshore dredging ($6.50) is based on the beach fill cost estimates (March 2006) for Reach SB-1D (Shinnecock Inlet Park - West) because the dredged volumes, borrow site locations, and distances to placement sites are similar. It is assumed that borrow site 5B will be utilized; however, if another borrow site that is further away must be used, the unit price per cubic yard would increase.

E2.1 Existing Practice

The primary cost in Existing Practice is from the dredging of the channel and deposition basin. Under Existing Practice, it is assumed that 260,000 m^3 are dredged every four years and are placed either immediately west of the inlet on the west beach or downdrift of the ebb shoal attachment. A cost is also assigned to the LST deficit, which results from Existing Practice. Separate mobilization/demobilization costs are assumed for the two dredging activities.

Table E-3. Cost Summary for SI Existing Practice

Plan Component	Dredging Interval (years)	Quantity (1000x m^3 (cy))	Mob/ Demob ($000)	Unit Cost ($/m^3 ($/cy))	Subtotal Cost ($000)	E&D and S&A ($000)	Total Cost Per Operation ($000)	Annual Cost ($000)
Channel & Deposition Basin Dredging	4	260 (340)	$1,000	$6.55 ($5.00)	$3,105	$518	$3,623	$1,033
LST Deficit	10	400 (524)	$1,000	$8.50 ($6.50)	$5,067	$822	$5,889	$767
							Grand Total	$1,800

E2.2 Alternative 1: Authorized Project (AP) + Dredging the Ebb Shoal

Under each version of Alternative 1, 260,000 m^3 are dredged from the channel and deposition basin. It is assumed that the dredged material will be placed either on the west beach or downdrift of the ebb shoal attachment. Under Alternative 1, the ebb shoal will also be dredged every two or four years and the material will be placed either immediately downdrift of the inlet at the west beach or downdrift of the ebb shoal attachment in order to increase sand bypassing. Under the management practice described in Alternative 1, there will not be a deficit in longshore sediment transport. Because all dredging activity is nearshore, it is assumed that deposition basin dredging and ebb shoal dredging contracts can be combined when possible to reduce the number of mobilizations/demobilizations and optimize costs.

First costs and annual costs developed for Alternative 1A (4 year cycle) and 1B (2 year cycle) and are summarized in Table E-4 and Table E-5, respectively. Increasing dredging frequency at the ebb shoal (every 2 vs. every 4 years) increases the number of dredge mobilizations and demobilizations and thus the annual costs increase by about $322,000/yr (21%) as compared to dredging on a 4 year cycle. The cost of dredging the ebb shoal on a two year cycle is also comparable to the cost of Existing Practice including dredging offshore every 10 years to offset the existing LST deficit; however the additional benefit of increasing continuity of bypassing has not bee included.

Table E-4. Cost Summary for SI Alternative 1A: APD + Ebb Shoal Dredging every 4 years

Plan Component	Dredging Interval (years)	Quantity (1000x m^3 (cy))	Mob/ Demob ($000)	Unit Cost ($/m^3 ($/cy))	Subtotal Cost ($000)	E&D and S&A ($000)	Total Cost Per Operatio n ($000)	Annua l Cost ($000)
Channel & Deposition Basin Dredging	4	260 (340)	$1,000	$6.55 ($5.00)	$3,105	$518	$3,623	$1,033
Ebb Shoal Dredging	4	160 (210)	Same contract	$6.55 ($5.00)	$1,208	$212	$1,419	$405
							Grand Total	$1,438

Table E-5. Cost Summary for SI Alternative 1B: APD + Ebb Shoal Dredging every 2 years

Plan Component	Dredging Interval (years)	Quantity (1000x m^3 (cy))	Mob/ Demob ($000)	Unit Cost ($/m^3 ($/cy))	Subtota l Cost ($000)	E&D and S&A ($000)	Total Cost Per Operation ($000)	Annual Cost ($000)
Channel & Deposition Basin Dredging	4	260 (340)	Same contract	$6.55 ($5.00)	$1,955	$334	$2,289	$653
Ebb Shoal Dredging	2	80 (105)	$1,000	$6.55 ($5.00)	$1,754	$301	$2,055	$1,107
							Grand Total	$1,760

E2.3 Alternative 2: AP + Nearshore Structures (T-groins) along the West Beach

The cost of the nearshore structures, maintenance dredging, and the LST deficit components of Alternative 2 are estimated according to Table E-6. As a result of T-groin construction, dredging from the channel and deposition will decrease by 60,000m^3 over the four year dredging cycle. It is assumed that part of the material dredged from the channel and deposition basin is placed on the west beach, while the remainder is placed downdrift of the ebb shoal attachment. A cost is assigned to the LST deficit according to the offshore dredging formulation. Separate mobilization/demobilization costs are required for each dredging activity.

The cost estimate for building the T-groins on West Beach is based upon 2005 cost estimates for the construction of T-groins at Coney Island. T-groin required construction volumes are contained in the March 1999 document, "West of Shinnecock Inlet Draft Decision Document." The cost of initial design fill from offshore borrow sites is also included in the construction cost. The volumes and costs of the initial construction components are summarized in Table E-7.

Table E-6. Cost Summary for SI Alternative 2: APD + Nearshore Structures (T-groins)

Plan Component	Dredging Interval (years)	Quantity (1000x m^3 (cy))	Mob/ Demob ($000)	Unit Cost ($/$m^3$ ($/cy))	Subtotal Cost ($000)	E&D and S&A ($000)	Total Cost Per Operation ($000)	Annual Cost ($000)
Channel & Deposition Basin Dredging	4	200 (262)	$1,000	$6.55 ($5.00)	$2,657	$447	$3,103	$885
LST Deficit	10	400 (524)	$1,000	$8.50 ($6.50)	$5,067	$822	$5,889	$767
T-groins					$20,889	$3,118	$24,007	$1,341
							Grand Total	$2,993

Table E-7. Initial Construction Cost for T-groins

Plan Component	Quantity		Unit Price	Price
Required Easements				$100,000
Survey, Appraisal & Administration				$100,000
Mob/Demob & Site Preparation				$112,405
Install & Remove Access Stone				$175,000
Excavation	104,000	cy	$15.24	$1,584,960
Filter Fabric	41,000	sf	$27.54	$1,129,140
Bedding Stone	26,641	tons	$55.82	$1,487,101
Core Stone	11,250	tons	$55.92	$629,100
1-ton Armor Stone	8,693	tons	$87.12	$757,334
5-ton Armor Stone	3,774	tons	$116.89	$441,143
7-ton Armor Stone	13,838	tons	$116.89	$1,617,524
14-ton Armor Stone	65,683	tons	$116.89	$7,677,686
Design Beach Fill	302,910	cy	$6.50	$2,648,200
SUBTOTAL				$20,888,800

E2.4 Alternative 3: AP + Offshore Dredging for the West Beach

First costs and annual costs developed for this alternative are summarized in Table E-8. The only difference between this alternative and Existing Practice (Table E-3) is that offshore dredging would be performed every two years to provide for more continuous protection of the west beach. This frequency increase makes this alternative more expensive than continuing the Existing Practice (approximately $0.5 M/yr more, or a 27% increase).

Table E-8. Cost Summary for SI Alternative 3: AP + Offshore Dredging for West Beach								
Plan Component	**Dredging Interval (years)**	**Quantity (1000x m^3 (cy))**	**Mob/ Demob ($000)**	**Unit Cost ($/m^3 ($/cy))**	**Subtotal Cost ($000)**	**E&D and S&A ($000)**	**Total Cost Per Operation ($000)**	**Annual Cost ($000)**
Channel & Deposition Basin Dredging	4	260 (340)	$1,000	$6.55 ($5.00)	$3,105	$518	$3,623	$1,033
Offshore Dredging	2	80 (105)	$1,000	$8.50 ($6.50)	$1,935	$379	$2,314	$1,247
							Grand Total	$2,280

E2.5 Alternative 4: AP + Semi-fixed Bypass System

Alternative 4 incurs costs not only from initial construction and maintenance costs of the bypass system, but also from the still necessary dredging of the channel and the deposition basin and the assigned cost of the LST deficit. All costs are summarized in Table E-9.

It is assumed that the dredging of the channel and deposition basin would continue on a 4 year interval despite the reduction in accumulation rate. In addition, and as in other alternatives, the equivalent costs associated with the LST deficit at the inlet (29,000 m^3/yr in this case) are computed based on a 10 year dredging interval. Separate mobilization/demobilization costs are assumed for the nearshore and offshore dredging components.

Initial construction cost, annual cost, and overhaul and replacement costs for the semi-fixed bypassing plant proposed at Shinnecock Inlet is based on the work performed by Williams et al (1998) to asses the appropriateness of such a plant for this location. All plant costs, which are summarized in Table E-10 through Table E-12, are scaled from 1997 to 2005 price levels. As these tables illustrate, labor costs for installation and annual operation of the plant are significant.

Overall Alternative 4 is relatively expensive compared to Existing Practice (42% more) and other alternatives. The decreased costs from the reduction in the LST deficit and the deposition basin dredging are outweighed by the added cost of the bypassing plant ($1.1 M/yr). It should be noted, however, that reduced risks and damages downdrift associated with more continuous bypassing are not accounted for in these costs.

Table E-9. Cost Summary for SI Alternative 4: AP + Semi-fixed Bypass System								
Plan Component	**Dredging Interval (years)**	**Quantity (1000x m^3 (cy))**	**Mob/ Demob ($000)**	**Unit Cost ($/$m^3$ ($/cy))**	**Subtotal Cost ($000)**	**E&D and S&A ($000)**	**Total Cost Per Operation ($000)**	**Annual Cost ($000)**
Channel & Deposition Basin Dredging	4	160 (210)	$1,000	$6.55 ($5.00)	$2,358	$399	$2,756	$838
Initial Construction					$3,764	$621	$4,385	$245
Annual Bypass System Cost	Cont.	100 (131)			$616	$112	$728	$768
Overhaul & Replacement								$52
LST Deficit	10	290 (379)	$1,000	$8.50 ($6.50)	$3,983	$655	$4,638	$656
							Grand Total	$2,559

Table E-10. Detailed Costs of a Semi-fixed Bypass System: Initial Construction				
Plan Component	**Quantity**		**Unit Price**	**Price**
Mobilization/Demobilization	1		$45,853	$45,853
Operations and Building				
Structure	1500	sf	$95.68	$143,522
Utilities				$14,751
Security Measures				$9.568
Mechanical Equipment				
Water Supply Pump & Engine	1	ea	$58,472	$58,472
Booster Pump & Engine	2	ea	$247,177	$494,354
Booster Pump House	200	sf	$95.68	$19,136
Jet Pumps	2	ea	$66,446	$132,891
Flushing Water Pump & Engine	1	ea	$20,199	$20,199
Air Compressor	1	ea	$16,123	$16,123
Instrumentation & Gages	1	ea	$28,572	$28,572
135-ton Crawler Crane	1	ea	$823,924	$823,924
Vehicle	1	ea	$27,794	$27,794
Electrical Equipment	1	ea	$73,903	$73,903
Eductor & Discharge Piping				
12 inch HDPE	8,800	ft	$13.77	$121,164
12 inch HDPE	1,600	ft	$13.77	$22,030
Butt fusion equipment rental, training, etc.	1	ea	$2,690	$2,690
Pipe delivery charges	1	ea	$10,375	$10,375
. *HDPE & Steel fittings*	1	ea	$20,749	$20,749
Valves	1	ea	$56,536	$56,536
Installation	1	ea	$960,606	$960,606
Access Road & Parking Area	1	ea	$87,531	$87,531
Miscellaneous	1	ea	$82,484	$82,484
SUBTOTAL				$3,763,900

Table E-11. Detailed Costs of a Semi-fixed Bypass System: Annual Operating Costs	
Plan Component	**Price**
Operating Crew	$448,283
Utilities	$1,329
Plant Fuel	$15,370
Vehicles (including fuel)	$1,537
Maintenance	$69,194
SUBTOTAL	$616,000

Table E-12. Detailed Costs of a Semi-fixed Bypass System: Overhaul and Replacement Costs			
Plan Component	**Interval (years)**	**Price**	**Annual Cost**
Crawler Crane A	6	$17,931	$2,603
Crawler Crane B	10	$10,246	$763
Caterpillar Diesel Engine for 3 Booster Pumps	20	$75,311	$2,117
Slurry Booster Pumps A	2	$17,931	$2,117
Slurry Booster Pumps B	4	$14,089	$3,232
Caterpillar Diesel Engine, water supply pump	16	$20,109	$833
Motive Water Supply Pump A	10	$6,020	$448
Motive Water Supply Pump B	4	$1,281	$294
Air Compressor A	10	$1,537	$114
Air Compressor B	15	$1,153	$52
Gauges A	5	$1,281	$225
Gauges B	10	$5,123	$381
Flow Instrumentation	10	$14,089	$1,049
Crane-mounted VHF density Meter	15	$2,562	$115
¾-ton 4WD Pickup (Diesel)	6	$27,794	$4,035
Slurry Gate Valve	6	$1,537	$223
All other gate valves	10	$4,611	$343
Jet Pump A	2	$3,714	$1,794
Jet Pump B	8	$1,281	$132
Jet Pump C	6	$1,793	$260
Jet Pump D	6	$27,794	$4,035
Jet Pump E	8	$2,562	$265
Jet Pump F	8	$2,562	$265
Jet Pump G	8	$1,793	$185
Pipeline A	5	$25,616	$4,507
Pipeline B	12	$110,149	$6,806
Pipeline C	4	$1,281	$294
Pipeline D	5	$961	$169
Pipeline E	6	$2,562	$372
Pumphouse A	25	$3,586	$101
Pumphouse B	25	$3,842	$108
SUBTOTAL			$51,500

E2.6 Alternative 5: AP + Reduced Dimensions of Deposition Basin

Alternative 5A and 5B recommend reducing the dimensions of the deposition basin in order to increase the dredging frequency to provide more continuous sediment bypassing downdrift of the ebb shoal attachment. Alternative 5A consists of a deposition basin at -18 ft MLW with a 225,000 m^3 capacity that would necessitate dredging every 3 years. Alternative 5B consists of a deposition basin at-16 ft MLW that would necessitate dredging every 2 years.

Costs associated with these two alternatives are summarized in Table E-13 and Table E-14. The increase in the dredging frequency increases the annualized cost of dredging the deposition basin by roughly $130,000 (7%) and $340,000 (19%) per year for Alternatives 5A and 5B, respectively. Note that the additional cost from offsetting the LST deficit would remain the same as under Existing Conditions ($767,000/yr). Similarly, separate mobilizations/demobilization costs must be assumed for nearshore and offshore dredging.

Table E-13. Cost Summary for SI Alternative 5A: -18 ft MLW Deposition Basin

Plan Component	Dredging Interval (years)	Quantity (1000x m^3 (cy))	Mob/ Demob ($000)	Unit Cost ($/$m^3$ ($/cy))	Subtotal Cost ($000)	E&D and S&A ($000)	Total Cost Per Operation ($000)	Annual Cost ($000)
Channel & Deposition Basin Dredging	3	195 (255)	$1,000	$6.55 ($5.00)	$2,616	$440	$3,056	$1,130
LST Deficit	10	400 (524)	$1,000	$8.50 ($6.50)	$5,067	$822	$5,889	$767
							Grand Total	$1,897

Table E-14. Cost Summary for SI Alternative 5B: -16 ft MLW Deposition Basin

Plan Component	Dredging Interval (years)	Quantity (1000x m^3 (cy))	Mob/ Demob ($000)	Unit Cost ($/$m^3$ ($/cy))	Subtotal Cost ($000)	E&D and S&A ($000)	Total Cost Per Operation ($000)	Annual Cost ($000)
Channel & Deposition Basin Dredging	2	130 (170)	$1,000	$6.55 ($5.00)	$2,128	$362	$2,489	$1,341
LST Deficit	10	400 (524)	$1,000	$8.50 ($6.50)	$5,067	$822	$5,889	$767
							Grand Total	$2,108

E2.7 Alternative 6: AP + Dredging the Flood Shoal

Under each version of Alternative 6, 260,000 m^3 are dredged from the channel and deposition basin. It is assumed that the dredged material will be placed either on the west beach or downdrift of the ebb shoal attachment. It is assumed that the flood shoal will be dredged every two or four years and that the material will be placed either immediately downdrift of the inlet at the west beach or downdrift of the ebb shoal attachment in order to reduce the LST deficit. Under this management practice, there will not be a deficit in longshore sediment transport. Because all dredging activity is nearshore, it is assumed that deposition basin dredging and flood shoal dredging contracts will be combined when possible to reduce the number of mobilizations/demobilizations and optimize costs.

First costs and annual costs developed for Alternative 6A (4 year cycle) and 6B (2 year cycle) and are summarized in Table E-15 and Table E-16, respectively. As in the case of ebb shoal dredging, increasing dredging frequency at the flood shoal (every 2 vs. every 4 years) increases the number of dredge mobilizations and demobilizations and thus the annual costs increase by about $322,000/yr (21%) as compared to dredging on a 4 year cycle. The cost of dredging the flood shoal on a two year cycle is also comparable to the cost of Existing Practice including dredging offshore every 10 years to offset the existing LST deficit; however the additional benefit of increasing continuity of bypassing has not bee included.

Table E-15. Cost Summary for SI Alternative 6A: APD + Flood Shoal Dredging (every 4 years)

Plan Component	Dredging Interval (years)	Quantity (1000x m^3 (cy))	Mob/ Demob ($000)	Unit Cost ($/$m^3$ ($/cy))	Subtotal Cost ($000)	E&D and S&A ($000)	Total Cost Per Operation ($000)	Annual Cost ($000)
Channel & Deposition Basin Dredging	4	260 (340)	$1,000	$6.55 ($5.00)	$3,105	$518	$3,623	$1,033
Flood Shoal Dredging	4	160 (210)	Same contract	$6.55 ($5.00)	$1,208	$212	$1,419	$405
							Grand Total	$1,438

Table E-16. Cost Summary for SI Alternative 6B: APD + Flood Shoal Dredging (every 2 years)

Plan Component	Dredging Interval (years)	Quantity (1000x m^3 (cy))	Mob/ Demob ($000)	Unit Cost ($/$m^3$ ($/cy))	Subtotal Cost ($000)	E&D and S&A ($000)	Total Cost Per Operation ($000)	Annual Cost ($000)
Channel & Deposition Basin Dredging	4	260 (340)	Same contract	$6.55 ($5.00)	$1,955	$334	$2,289	$653
Flood Shoal Dredging	2	80 (105)	$1,000	$6.55 ($5.00)	$1,754	$301	$2,055	$1,107
							Grand Total	$1,760

E2.8 Alternative 7: AP + Shortening the East Jetty

A summary of the costs for this alternative is presented in Table E-17. As a result of jetty shortening, dredging frequency would likely increase to once per year. The net LST deficit for the inlet could be completely offset by an additional influx of sand from the updrift beaches as they erode in response to jetty shortening. Thus, it was assumed that this alternative would not incur the additional costs of offsetting an LST deficit through offshore dredging.

Shortening of the east jetty represents an additional $115,000/yr in annualized costs. Under this alternative, the East Jetty would be shortened by 500 feet. The weight of stone removed is estimated to be 24,000 tons, based upon the typical rebuilt jetty section in Plate No. 5 of the 1987 Shinnecock General Design Memorandum. The cost estimate for demolition of the jetty is based upon demolition cost estimates used for the recent Coney Island construction. The unit price per ton of demolition is $63.56. Including E&D and S&A, the total initial cost of shortening the east jetty is $2,056,000.

Table E-17. Cost Summary for SI Alternative 7: AP + Shortening the East Jetty								
Plan Component	**Dredging Interval (years)**	**Quantity (1000x m^3 (cy))**	**Mob/ Demob ($000)**	**Unit Cost ($/$m^3$ ($/cy))**	**Subtotal Cost ($000)**	**E&D and S&A ($000)**	**Total Cost Per Operation ($000)**	**Annual Cost ($000)**
Channel & Deposition Basin Dredging	1	150 (196)	$1,000	$6.55 ($5.00)	$2,277	$386	$2,663	$2,799
East Jetty Shortening (600 ft)					$1,754	$302	$2,056	$115
							Grand Total	$2,914

E2.9 Alternative 8: AP + West Jetty Spur

A summary of the costs associated with Alternative 8 is presented in Table E-18. Channel and deposition basin dredging proceeds at the rate of 55 m^3/yr, which is 10 m^3/yr less than under Existing Conditions due to reduced sedimentation in the deposition basin as a result of spur construction. Alternative 8, however, increases the LST deficit from 40 m^3/yr under Existing Conditions to 50 m^3/yr. The cost associated with the LST deficit is $913,000/yr, representing an increase of $146,000/yr over Existing Conditions.

Because of the similarity in the construction material requirements, unit costs associated with the recent Coney Island T-groin construction project were used to estimate the cost of the west jetty spur construction. Construction quantities are based upon the design for the high-crested spur by Baird Associates. Quantities and costs are presented in Table E-19.

The total cost for this alternative is $391,000/yr (22%) more expensive than Existing Conditions.

Table E-18. Cost Summary for SI Alternative 8: APD with + West Jetty Spur								
Plan Component	**Dredging Interval (years)**	**Quantity (1000x m^3 (cy))**	**Mob/ Demob ($000)**	**Unit Cost ($/$m^3$ ($/cy))**	**Subtotal Cost ($000)**	**E&D and S&A ($000)**	**Total Cost Per Operation ($000)**	**Annual Cost ($000)**
Channel & Deposition Basin Dredging	4	220 (288)	$1,000	$6.55 ($5.00)	$2,806	$470	$3,276	$935
LST Deficit	10	500 (654)	$1,000	$8.50 ($6.50)	$6,039	$970	$7,009	$913
Spur Construction					$5,291	$856	$6,147	$343
							Grand Total	$2,191

Table E-19. Initial Construction Cost for the West Jetty Spur				
Plan Component	**Quantity**		**Unit Price**	**Price**
Survey, Appraisal & Administration				$150,000
Mob/Demob & Site Preparation				$112,405
Filter Fabric		sf	$27.54	$104,101
Bedding Stone		tons	$55.82	$173,042
1 ton Core Stone		tons	$87.12	$1,167,408
½-2 ½ ton Filter Stone		tons	$87.12	$766,656
6-12 ton Armor Stone		tons	$116.89	$1,765,039
9-15 ton Armor Stone		tons	$116.89	$362,359
SUBTOTAL				$5,291,000

E3. MORICHES INLET

Based on results from the preliminary screening, four alternatives were considered for detailed analysis at Moriches Inlet. Each alternative can achieve the goals of providing reliable navigation through the Federal navigation channel, restoring natural sediment pathways and reducing adjacent shoreline erosion, albeit to varying degrees and at different costs. Table E-20 presents the estimated annual cost for each alternative as well as Existing Practice.

Table E-20. Summary of Annual Cost for Moriches Inlet Alternatives	
Plan	Annual Cost ($000)
MI Existing Practice	*$2,086*
Alt 1. Authorized Project	$3,272
Alt 2. AP + Ebb Shoal Dredging	$2,803
Alt 3. AP + Semi-fixed Bypass System	$3,844
Alt. 4 AP + Flood Shoal Dredging	$2,803

For Moriches Inlet, two types of dredging are considered: nearshore and offshore. For nearshore dredging (i.e., within the deposition basin, ebb shoal, or flood shoal), a mobilization/demobilization cost of $1,000,000 is assumed for a typical 30" cutterhead dredge. The unit price at Moriches Inlet ($5.70/cy) was determined using standard cost estimation techniques employed by dredging contractors which take into account the quantity of material to be dredged and the distance to the placement site. At Moriches Inlet, a production rate of 22,200 cubic yards per work-day is used. The daily cost of a 30" cutterhead dredge is assumed to be $120,000, which is the same daily rate used in the Breach Closure Plan cost estimates (March 2006). From the cost per day, the volume required, the production rate and the unit price per cubic yard were computed.

The estimated mobilization/demobilization cost and the unit price per cubic yard are slightly higher than two of recent nearshore dredging contracts executed at Moriches Inlet, which are summarized in Table E-21. For the mobilization/demobilization costs, this is most likely because the cost was split (unevenly) between Moriches and Shinnecock Inlets as they were contracted together. Additionally, the prices here have not been adjusted to 2005 price levels.

Offshore dredging utilizes the FIMP offshore borrow sites identified in the Borrow Source Investigations Report (September 29, 2005). Offshore dredging is used to assign a cost to the LST deficit associated with each alternative. As with nearshore dredging, a typical mobilization cost of $1,000,000 is assumed. The unit price per cubic yard for offshore dredging ($7.00) is based on the beach fill cost estimates

(March 2006) for Reach MB-1B (Smith Point Country Park) because the dredged volumes, borrow site locations, and distances to placement sites are similar. It is assumed that borrow site 3A will be utilized; however, if another borrow site that is further away must be used, the unit price per cubic yard would increase.

Table E-21. Summary of Recent Nearshore Dredging Contracts at Moriches Inlet			
Year	**Mobilization & Demobilization**	**Unit Price ($/cy)**	**Volume (cy)**
1998	$100,000	$5.44	186,518
2004	$246,000	$4.85	250,250

E3.1 Existing Practice

The primary cost of continuing Existing Practice is from the dredging of the channel and the deposition basin. Under Existing Practice, 224,000 m^3 are dredged every four years and are placed on the west beach. A cost is assigned to the LST deficit assuming a dredging cycle of 10 years. Separate mobilization/demobilization costs are assumed for the two dredging activities because two different types of dredges are needed (nearshore and offshore). Table E-22 summarizes the estimated annual cost of Existing Practice at Moriches Inlet. This cost is less than all alternatives investigates, but it should be noted that Existing Practice fails to maintain navigation conditions through the inlet.

Table E-22. Cost Summary for MI Existing Practice								
Plan Component	**Dredging Interval (years)**	**Quantity (1000x m^3 (cy))**	**Mob/ Demob ($000)**	**Unit Cost ($/$m^3$ ($/cy))**	**Subtotal Cost ($000)**	**E&D and S&A ($000)**	**Total Cost Per Operation ($000)**	**Annual Cost ($000)**
Channel & Deposition Basin Dredging	4	224/ (293)	$1,000	$7.45 ($5.70)	$3,071	$512	$3,583	$1,022
LST Deficit	10	560/ (732)	$1,000	$9.20 ($7.00)	$7,043	$1,121	$8,164	$1,064
							Grand Total	$2,086

E3.2 Alternative 1: Authorized Project (AP)

The primary cost in implementing the Authorized Project (Alternative 1) is from the dredging of the channel and deposition basin. Under the Authorized Project, 75,000 m^3 are dredged every year and are placed either downdrift of the ebb shoal attachment. A cost is also assigned to the LST deficit. (56,000 $^{3/}$yr) Separate mobilization/demobilization costs are assumed for the two dredging activities because two different types of dredges are needed (nearshore and offshore).

Table E-23 shows the annual costs incurred for this alternative. Implementing the Authorized Project at Moriches Inlet will be more expensive than Existing Practice, due to the increased number of mobilization/demobilizations. Sediment bypassing is not improved under Alternative 1.

Table E-23. Cost Summary for MI Authorized Project								
Plan Component	**Dredge Interval (years)**	**Quantity (1000x m^3 (cy))**	**Mob/ Demob ($000)**	**Unit Cost ($/m^3 ($/cy))**	**Subtotal Cost ($000)**	**E&D and S&A ($000)**	**Total Cost Per Operation ($000)**	**Annual Cost ($000)**
Channel & Deposition Basin Dredging	1	75/ (98)	1,000	$7.45/ ($5.70)	$1,792	$308	$2,100	$2,208
LST Deficit	10	560/ (732)	1,000	$8.54/ ($7.00)	$7,043	$1,121	$8,164	$1,064
							Grand Total	$3,272

E3.3 Alternative 2: AP + Dredging the Ebb Shoal

Under Alternative 2, 75,000 m^3 are dredged from the channel and deposition basin. It is assumed that the dredged material will be placed downdrift of the ebb shoal attachment. The ebb shoal will also be dredged every year (56,000 m^3) and the material will also be placed downdrift of the ebb shoal attachment in order to increase sand bypassing. Under the management practice described in Alternative 2, there will not be a deficit in longshore sediment transport. Because all dredging activity is nearshore, it is assumed that deposition basin dredging and ebb shoal dredging contracts can be combined to reduce the number of mobilizations/demobilizations and optimize costs.

Annual costs developed for Alternative 2 and are summarized in Table E-24. The cost of Alternative 2 is less than the cost of Alternative 1; however, both are more expensive than Existing Practice due to the increased number of mobilizations/demobilizations. It should be noted, however, that reduced risks and damages downdrift associated with more continuous bypassing are not accounted for in these costs.

Table E-24. Cost Summary for MI Alternative 2: AP + Ebb Shoal Dredging								
Plan Component	**Dredging Interval (years)**	**Quantity (1000x m^3 (cy))**	**Mob/ Demob ($000)**	**Unit Cost ($/m^3 ($/cy))**	**Subtotal Cost ($000)**	**E&D and S&A ($000)**	**Total Cost Per Operation ($000)**	**Annual Cost ($000)**
Channel & Deposition Basin Dredging	1	75/ (98)	$1,000	$7.45/ ($5.70)	$1,792	$308	$2,100	$2,208
Ebb Shoal Dredging	1	56/ (73)	Same contract	$7.45/ ($5.70)	$479	$88	$566	$595
							Grand Total	$2,803

E3.4 Alternative 3: APD + Semi-fixed Bypass System

Alternative 3 incurs costs not only from initial construction and maintenance costs of the bypass system, but also from the still necessary dredging of the channel and the deposition basin and the assigned cost of the LST deficit. All costs are summarized in Table E-25.

It is assumed that the dredging of the channel and deposition basin would occur on a 1 year interval despite the reduction in accumulation rate. In addition, and as in other alternatives, the equivalent costs associated with the LST deficit at the inlet (38,000 m^3/yr in this case) are computed based on a 10 year dredging interval. Separate mobilization/demobilization costs are assumed for the nearshore and offshore dredging components.

Initial construction cost, annual cost, and overhaul and replacement costs for the semi-fixed bypassing plant proposed at Moriches Inlet is based on the work performed by Williams et al (1998) to asses the appropriateness of such a plant for this location. All plant costs, which are the same as for Shinnecock Inlet and are summarized in Table E-10 through Table E-12,, are scaled from 1997 to 2005 price levels. As these tables illustrate, labor costs for installation and annual operation of the plant are significant.

Overall Alternative 3 is relatively expensive compared to Existing Practice (63% more), Alternative 1 (17% more) and the other alternatives. The decreased costs from the reduction in the LST deficit and the deposition basin dredging are outweighed by the added cost of the bypassing plant ($1.1 M/yr). It should be noted, however, that reduced risks and damages downdrift associated with more continuous bypassing are not accounted for in these costs.

Table E-25. Cost Summary for MI Alternative 3: AP + Semi-fixed Bypass System

Plan Component	Dredging Interval (years)	Quantity (1000x m^3 (cy))	Mob/ Demob ($000)	Unit Cost ($/m^3 ($/cy))	Subtotal Cost ($000)	E&D and S&A ($000)	Total Cost Per Operation ($000)	Annual Cost ($000)
Channel & Deposition Basin Dredging	1	55/ (72)	$1,000	$7.45/ ($5.70)	$1,622	$280	$1,902	$1,999
Initial Construction					$3,764	$621	$4,385	$245
Annual System Operation	Cont.	100 (131)			$616	$112	$728	$768
Overhaul and Replacement								$52
LST Deficit	10	380/ (497)	$1,000	$8.54/ ($7.00)	$5,151	$835	$5,986	$780
							Grand Total	$3,844

E3.5 Alternative 4: AP + Dredging the Flood Shoal

Under Alternative 4, 75,000 m^3/yr are dredged from the channel and deposition basin as specified in the Authorized Project. It is further assumed in this alternative that the flood shoal will be dredged every year and that the material from both dredge events will be placed downdrift of the ebb shoal attachment in order to reduce the LST deficit. Under this management practice, there will not be a deficit in longshore sediment transport. Because all dredging activity is nearshore, it is assumed that deposition basin dredging and flood shoal dredging contracts will be combined to reduce the number of mobilizations/demobilizations and optimize costs.

First costs and annual costs developed for Alternative 6 in Table E-26. The cost of Alternative 4 is less than the cost of Alternative 1; however, both are more expensive than Existing Practice due to the increased number of mobilizations/demobilizations. It should be noted, however, that reduced risks and damages downdrift associated with more continuous bypassing are not accounted for in these costs.

Table E-26. Cost Summary for SI Alternative 6A: AP + Flood Shoal Dredging (every 4 years)								
Plan Component	**Dredging Interval (years)**	**Quantity (1000x m^3 (cy))**	**Mob/ Demob ($000)**	**Unit Cost ($/$m^3$ ($/cy))**	**Subtotal Cost ($000)**	**E&D and S&A ($000)**	**Total Cost Per Operation ($000)**	**Annual Cost ($000)**
Channel & Deposition Basin Dredging	1	75/ (98)	$1,000	$7.45/ ($5.70)	$1,792	$308	$2,100	$2,208
Flood Shoal Dredging	1	56/ (73)	Same contract	$7.45/ ($5.70)	$479	$88	$566	$595
							Grand Total	$2,803

E4. FIRE ISLAND INLET

Based on results from the preliminary screening, four alternatives were considered for detailed analysis at Fire Island Inlet. Each alternative can achieve the goals of providing reliable navigation through the federal navigation channel, restoring natural sediment pathways, and reducing adjacent shoreline erosion, albeit to varying degrees and at different costs. Table E-27 presents the estimated annual cost for each alternative, which includes continuation of Existing Practice.

Table E-27. Summary of Annual Cost for Fire Island Inlet Alternatives	
Plan	Annual Cost ($000)
Alt 1. Existing Practice/ Authorized Project (AP)	$10,049
Alt 2. AP + Dredging the Ebb Shoal	$9,077
Alt 3. Optimized Deposition Basin	$10,049
Alt. 4 AP + Dredging the Flood Shoal	$9,809

For Fire Island Inlet, two types of dredging are considered: nearshore and offshore. The unit price and mobilization and demobilization costs of nearshore dredging were computed as an average of recent bids for each placement location (updrift or downdrift beach). Unlike Moriches or Shinnecock Inlet, dredging operations at Fire Island Inlet are complicated by a large distances between dredging and placement sites and the likely use of booster pumps. Using four recent dredging contracts, summarized in Table E-28 and Table E-29, the average unit price and mobilization cost for placement downdrift at Gilgo Beach is $7.30/cy and $2,608,000 and the average unit price and mobilization cost for placement updrift at Robert Moses State Park is $4.20/cy and $593,000.

Offshore dredging at Fire Island Inlet utilizes the FIMP offshore borrow sites. A typical mobilization cost of $1,000,000 is assumed. The unit price per cubic yard for offshore dredging ($9.00) is based on the beach fill cost estimates (March 2006) for Reach GSB-1A (Robert Moses State Park) because the dredged volumes, borrow site locations, and distances to placement sites are similar. It is assumed that borrow site 2C will be utilized; however, if another borrow site that is further away must be used, the unit price per cubic yard would increase.

Table E-28. Summary of Recent Nearshore Dredging Contracts at Fire Island Inlet for Placement on Gilgo Beach (2005 Price Levels)			
Year	**Mobilization & Demobilization ($)**	**Unit Price ($/cy)**	**Volume (cy)**
1996	$2,455,134	$6.52	719,000
1999	$2,447,807	$8.36	972,000
2001	$2,655,075	$7.87	1,445,000
2003	$2,875,039	$6.41	953,000
Average	$2,608,300	$7.30	1,022,000

Table E-29. Summary of Recent Nearshore Dredging Contracts at Fire Island Inlet for Placement on Robert Moses State Park (2005 Price Levels)			
Year	**Mobilization & Demobilization ($)**	**Unit Price ($/cy)**	**Volume (cy)**
1996	$90,773	$3.03	363,000
1999	$316,424	$6.41	135,000
2001	$956,682	$4.50	165,000
2003	$1,007,855	$2.92	136,000
Average	$592,900	$4.20	200,000

E4.2 Alternative 1: Existing Practice/ Authorized Project

Alternative 1 maintains the Authorized Project dimensions (as is Existing Practice) by dredging the channel and deposition basin on a 2 year cycle and placing the material both updrift and downdrift of the inlet. Nearshore dredging is used for this maintenance dredging.

A cost is also assigned to the LST deficit on a four year cycle as the volume of the deficit at Fire Island Inlet is much larger than at Shinnecock or Moriches Inlets. Separate mobilization/demobilization costs are assumed for the two dredging activities because two different types of dredges are needed (nearshore and offshore). Costs are summarized in Table E-30.

Table E-30. Cost Summary for Existing Practice/Authorized Project (AP)								
Plan Component	**Dredging Interval (years)**	**Quantity (1000x m^3 (cy))**	**Mob/ Demob ($000)**	**Unit Cost ($/$m^3$ ($/cy))**	**Subtotal Cost ($000)**	**E&D and S&A ($000)**	**Total Cost Per Operation ($000)**	**Annual Cost ($000)**
Channel & Deposition Basin Dredging – Updrift Placement	2	124 (162)	$593	$5.50 ($4.20)	$1,464	$254	$1,718	$926
Channel & Deposition Basin Dredging – Downdrift Placement	2	626 (819)	$2,608	$9.60 ($7.30)	$9,875	$1,542	$11,417	$6,151
LST Deficit	4	580 (759)	$1,000	$11.80 ($9.00)	$9,006	$1,414	$10,419	$2,972
							Grand Total	$10,049

E4.2 Alternative 2: AP + Dredging the Ebb Shoal

Under Alternative 2, 750,000 m^3 are dredged from the channel and deposition basin in continuation of the Authorized Project. As such, part of the material will be backpassed to Robert Moses State Park and part will be placed on Gilgo Beach, downdrift of the inlet. Under Alternative 2, the ebb shoal will also be dredged every year and the material will also be placed downdrift of the ebb shoal attachment in order to increase sand bypassing and eliminate the LST deficit. Because all dredging activity is nearshore, it is assumed that navigation dredging and ebb shoal dredging contracts can be combined to reduce the number of mobilizations/demobilizations and optimize costs.

Annual costs developed for Alternative 2 and are summarized in Table E-31. This alternative is less expensive (10%) than continuation of Existing Practice (Alternative 1). It should be noted, however, that reduced risks and damages downdrift associated with more continuous bypassing are not accounted for in these costs.

Table E-31. Cost Summary for FII Alternative 2: AP + Ebb Shoal Dredging

Plan Component	Dredging Interval (years)	Quantity (1000x m^3 (cy))	Mob/ Demob ($000)	Unit Cost ($/$m^3$ ($/cy))	Subtotal Cost ($000)	E&D and S&A ($000)	Total Cost Per Operation ($000)	Annual Cost ($000)
Channel & Deposition Basin Dredging – Updrift Placement	2	124 (162)	$593	$5.50 ($4.20)	$1,464	$254	$1,718	$926
Channel & Deposition Basin Dredging – Downdrift Placement	2	626 (819)	$2,608	$9.60 ($7.30)	$9,875	$1,542	$11,417	$6,151
Ebb Shoal Dredging	2	290 (379)	Same contract	$9.60 ($7.30)	$3,182	$530	$3,711	$2,000
							Grand Total	$9,077

E3.3 Alternative 3: AP + Optimized Deposition Basin

Alternative 3 recommends widening the deposition basin to the east in the vicinity of the sand spit that is just west of Democratic Point. The purpose is to reduce the rapidity of shoal encroachment on the navigation channel so as to maintain more reliable navigation conditions. The deposition basin depth would not be modified under this alternative. Maintenance navigation would continue on a 2 year cycle, with fill placement occurring both updrift (Robert Moses State Park) and downdrift (Gilgo Beach) of the inlet.

The LST deficit would not eliminated under this alternative and flooding risks would not be improved or made worse as compared to Existing Practice (Alternative 1). A cost is also assigned to the LST deficit on a four year cycle. Separate mobilization/demobilization costs are assumed for the two dredging activities because two different types of dredges are needed (nearshore and offshore). Costs are summarized in Table E-32. The cost of this alternative is the same as continuing Existing Practice.

Table E32. Cost Summary for FII Alternative 3: Optimized Deposition Basin

Plan Component	Dredging Interval (years)	Quantity (1000x m^3 (cy))	Mob/ Demob ($000)	Unit Cost ($/$m^3$ ($/cy))	Subtotal Cost ($000)	E&D and S&A ($000)	Total Cost Per Operation ($000)	Annual Cost ($000)
Channel & Deposition Basin Dredging – Updrift Placement	2	124 (162)	$593	$5.50 ($4.20)	$1,464	$254	$1,718	$926
Channel & Deposition Basin Dredging – Downdrift Placement	2	626 (819)	$2,608	$9.60 ($7.30)	$9,875	$1,542	$11,417	$6,151
LST Deficit	4	580 (759)	$1,000	$11.80 ($9.00)	$9,006	$1,414	$10,419	$2,972
							Grand Total	$10,049

E4.4 Alternative 4: AP + Dredging the Flood Shoal

Under Alternative 4, 750,000 m^3/yr are dredged from the channel and deposition basin as specified in the Authorized Project on a two year cycle. This material will be placed both updrift and downdrift of the inlet to mitigate erosion at Gilgo Beach and Robert Moses State Park. It is further assumed in this alternative that the flood shoal will be dredged every two years and the material will be placed downdrift of the ebb shoal attachment point in order to reduce the LST deficit. Under this management practice, there will not be a deficit in longshore sediment transport. Because all dredging activity is nearshore, it is assumed that deposition basin dredging and flood shoal dredging contracts will be combined.

First costs and annual costs developed for Alternative 4 in Table E-33. This alternative is slightly less expensive (2%) than continuation of Existing Practice (Alternative 1). It should be noted, that reduced risks and damages downdrift from more continuous bypassing are not accounted for in these costs.

Table E-33. Cost Summary for FII Alternative 4: AP + Flood Shoal Dredging

Plan Component	Dredging Interval (years)	Quantity (1000x m^3 (cy))	Mob/ Demob ($000)	Unit Cost ($/$m^3$ ($/cy))	Subtotal Cost ($000)	E&D and S&A ($000)	Total Cost Per Operation ($000)	Annual Cost ($000)
Channel & Deposition Basin Dredging – Updrift Placement	2	124 (162)	$593	$5.50 ($4.20)	$1,464	$254	$1,718	$926
Channel & Deposition Basin Dredging – Downdrift Placement	2	626 (819)	$2,608	$9.60 ($7.30)	$9,875	$1,542	$11,417	$6,151
Flood Shoal Dredging	2	290 (379)	Same contract	$7.65 ($10.00)	$4,359	$713	$5,072	$2,732
							Grand Total	$9,809