APPENDIX B

ENGINEERING REPORT

ENGINEERING REPORT
FIGURE EIGHT ISLAND INLET AND SHORELINE MANAGEMENT PROJECT

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ENGINEERING REPORT
FIGURE EIGHT ISLAND INLET AND SHORELINE MANAGEMENT PROJECT

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B-2	Delft3D Model Results, Part 2 – Model Results Given Present (Feb. – March 2012) Conditions
C	GENESIS Model Results

ENGINEERING REPORT
FIGURE EIGHT ISLAND INLET AND SHORELINE MANAGEMENT PROJECT

1.0 INTRODUCTION

Figure Eight Island is one of a number of barrier islands located along the North Carolina coast in New Hanover County. Figure Eight Island is bordered by Rich Inlet to the north and Mason Inlet to the south (Figure 1-1). The Figure "8" Beach Homeowners Association has an interest in developing a long-term Beach Protection and Management Plan that covers the 4.9 miles of oceanfront shoreline. Approximately 22,130 feet of the Figure Eight Island oceanfront shoreline is developed. Two low-lying spits extend from the developed section of the island toward the adjacent inlets. The northern spit extending towards Rich Inlet is currently ~ 2,100 feet long and the southern spit that extends toward Mason Inlet is ~ 1,500 feet. Both areas are characterized by severe shoreline change.

Rich Inlet is a relatively large inlet that separates Hutaff Island, an undeveloped barrier to the northeast, from Figure Eight Island extending to the southwest. The inlet drains an expansive marsh-filled lagoon where two large tidal creeks, Nixon and Green Channels, connect the inlet to the Atlantic Intracoastal Waterway (AIWW). Although it is relatively stable, Rich Inlet has the capability to promote considerable oceanfront shoreline changes through complex linkages to ebb channel movement and ebb-tidal delta shape changes. Currently, Figure Eight Island is confronted with serious management issues that concern inlet hazard zones and the severe recurring oceanfront erosion. Even though the inlet has been a fairly stable feature since the early 1990's, there have been substantial shoreline changes along both sides of the inlet and the adjacent oceanfront.

At least 11 known beach nourishment projects of varying size have been completed along various shoreline segments of Figure Eight Island since June of 1984 to mitigate erosion. Nourishment activities have increased since the mid to late 1990's due to changes within Mason and Rich Inlets systems and the increase in storm activity. These projects combined have placed an estimated total volume of approximately 4 million cubic yards of beach fill along the island. The island's shoreline maintenance projects have typically involved mitigation efforts along erosion hot spots along the northern and southern segments of the island.

2.0 PROBLEM IDENTIFICATION

The Homeowners Association and island residents have struggled with the continuing problems associated with Rich and Mason Inlets, including long-term chronic erosion that has been exacerbated by a series of hurricanes in the 1990's. The Association is continuing to explore inlet management and beach renourishment options to: (1) preserve the integrity of its infrastructure, (2) provide protection to the existing development, which thereby would maintain or increase property values, and (3) ensure the continued use of the oceanfront beach and its adjacent navigable waterways. Information contained in this report provides a framework for formulating a Long-Term Figure Eight Island Beach Management Strategy.

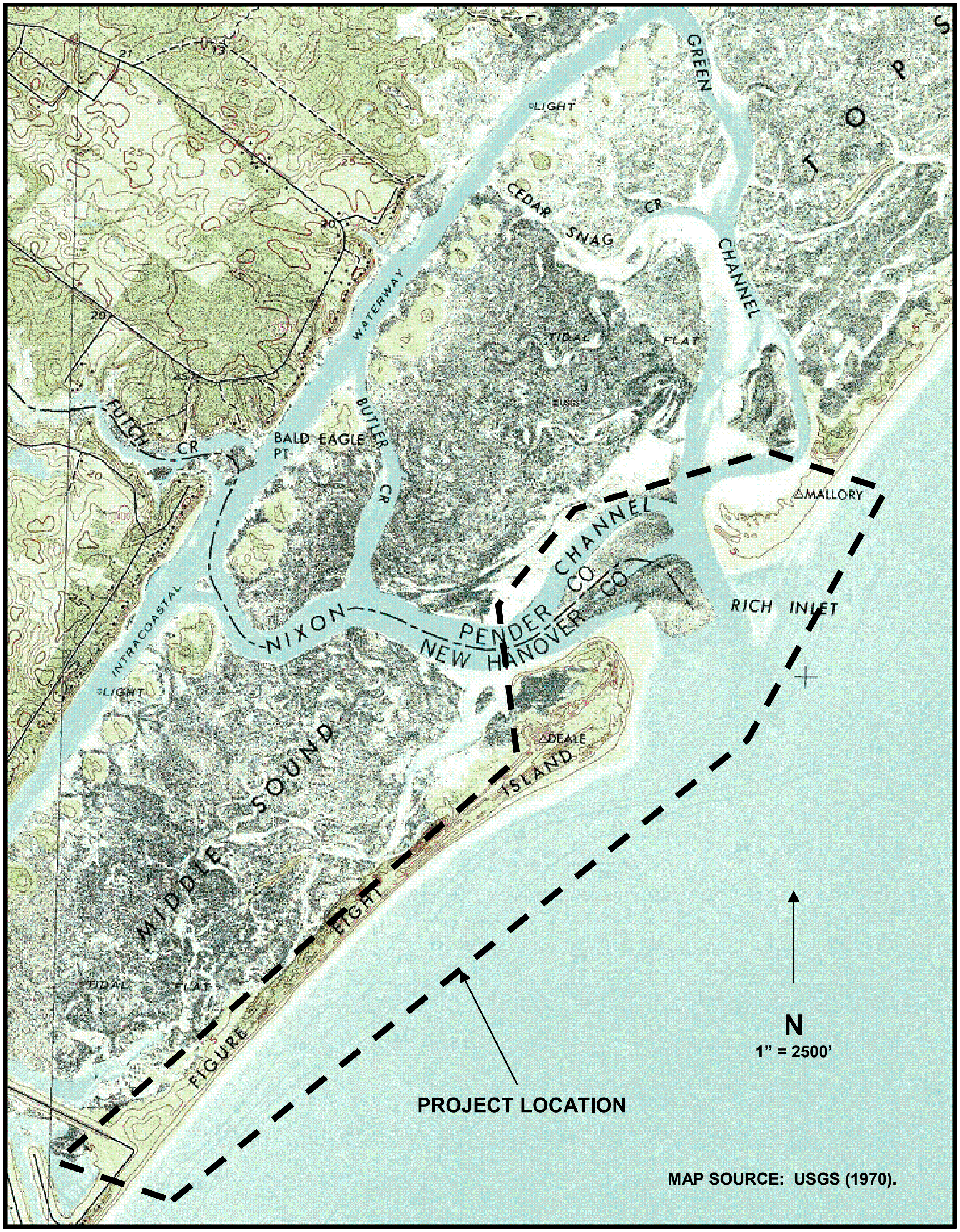


FIGURE 1-1: Figure Eight Island Project Location.

3.0 COASTAL CONSISTENCY

The consistency of this project with the Coastal Barrier Resources Act and Coastal Barrier Improvement Act of 1990 will be discussed in the Environmental Impact Statement for the project.

4.0 PHYSICAL CHARACTERISTICS OF THE PROJECT AREA

4.1 General Description

Barrier islands, such as Figure Eight Island, are composed of unconsolidated fine to medium sized quartz and shell material that is in a constant state of flux due to wind, waves, currents and storms. The oceanfront beach and the backing dunes are deposits of sand that are constantly changing their shape, and hence position with time as they respond to coastal processes.

Figure Eight Island is located within the southern coastal unit that extends from Cape Lookout to Sunset Beach, NC. The continental shelf sediment between Cape Lookout and Cape Fear is locally known as Onslow Bay. The sediment cover in Onslow Bay is generally thin as indicated by a large frequency of rock outcrops.

4.2 Tides

Ocean tides on Figure Eight Island are semi-diurnal, with a spring-neap variation of 28 days. Oceanfront tides are based on the NOAA tide gage and benchmark on Johnny Mercer's Pier in Wrightsville Beach. This benchmark is the closest oceanfront tidal benchmark established by NOAA. Tidal datums at Wrightsville Beach appear in Table 4-1. The mean tidal range is approximately 4.1 feet.

TABLE 4-1

NOAA (2003) OCEANFRONT TIDAL DATUMS
WRIGHTSVILLE BEACH, NC

TIDAL DATUM	ELEVATION (feet MLLW)	ELEVATION (feet NGVD)	ELEVATION (feet NAVD)
MEAN HIGHER HIGH WATER (MHHW)	4.64	3.01	2.05
MEAN HIGH WATER (MHW)	4.29	2.66	1.70
NORTH AMERICAN VERTICAL DATUM-1988 (NAVD)	2.59	0.96	0.00
MEAN TIDE LEVEL (MTL)	2.22	0.59	-0.37
MEAN SEA LEVEL (MSL)	2.22	0.59	-0.37
NATIONAL GEODETIC VERTICAL DATUM-1929 (NGVD)	1.63	0.00	-0.96
MEAN LOW WATER (MLW)	0.15	-1.47	-2.43
MEAN LOWER LOW WATER (MLLW)	0.00	-1.63	-2.59

Additional water level measurements were collected May 25-July, 2005 by Gahagan & Bryant Associates (GBA). These measurements covered 7 different locations within Rich Inlet and the Atlantic Intracoastal Waterway (AIWW). The locations of the 7 tide gages appear in Figure 4-1. Tidal datums based on the measurements appear in Table 4-2. The water levels measured by GBA were used to calibrate and verify the current, water level, and bathymetric change model for Rich Inlet. Tidal ranges inside the AIWW range from 3.2 to 3.6 feet. The tidal range in the throat of the inlet is approximately 3.7 feet. Tides in the AIWW lag the Wrightsville Beach tides by approximately 1 hour. Tides in the throat of Rich Inlet lag the Wrightsville Beach tides by approximately 30 minutes.

TABLE 4-2
INTERIOR TIDAL DATUMS
RICH INLET, NC

GBA Tide Gage	NC-NAD83 Easting (feet)	NC-NAD83 Northing (feet)	MHHW (feet NAVD)	MHW (feet NAVD)	MTL (feet NAVD)	MLW (feet NAVD)	MLLW (feet NAVD)
Green Channel	2388810	206816	1.9	1.3	-0.3	-2.0	-2.3
Nixon Channel	2383594	200566	2.2	1.6	-0.2	-1.9	-2.2
Inlet Throat	2388940	202433	2.2	1.7	-0.2	-2.0	-2.3
AIWW North	2387756	211356	2.0	1.5	-0.2	-1.8	-2.0
AIWW South	2378296	199045	2.3	1.7	-0.1	-1.9	-2.1
AIWW Middle	2382804	208892	2.1	1.5	-0.1	-1.8	-2.0
AIWW Figure Eight Bridge	2374595	193390	2.2	1.7	-0.1	-1.9	-2.2

NOTE: These datums are based on a limited set of water level measurements in 2005 and have not been officially certified by NOAA.

4.3 Currents

Currents were measured by GBA during a spring tidal period on June 21, 2005 (Figure 4-2) using boat-mounted Acoustic Doppler Current Profilers (ADCPs). In the throat of the inlet and Green Channel, the currents were flood-dominated. In Nixon Channel, the currents appeared to be ebb-dominated.

- In the throat of the inlet, the peak currents were 3.2 feet/second during flood and 2.7 feet/second during ebb, with a principal axis of 319º/139º.
- In Green Channel, the peak currents were 3.0 feet/second during flood and 2.0 feet/second during ebb, with a principal axis of 341º/161º.
- In Nixon Channel, the peak currents were 1.7 feet/second during flood and 1.8 feet/second during ebb, with a principal axis of 280º/100º.

The current measurements by GBA were utilized to calibrate current, water level, and bathymetric change model for Rich Inlet. Flow patterns in Rich Inlet were then analyzed using the calibrated model. A review of the flow patterns appears in the Delft3D modeling study.

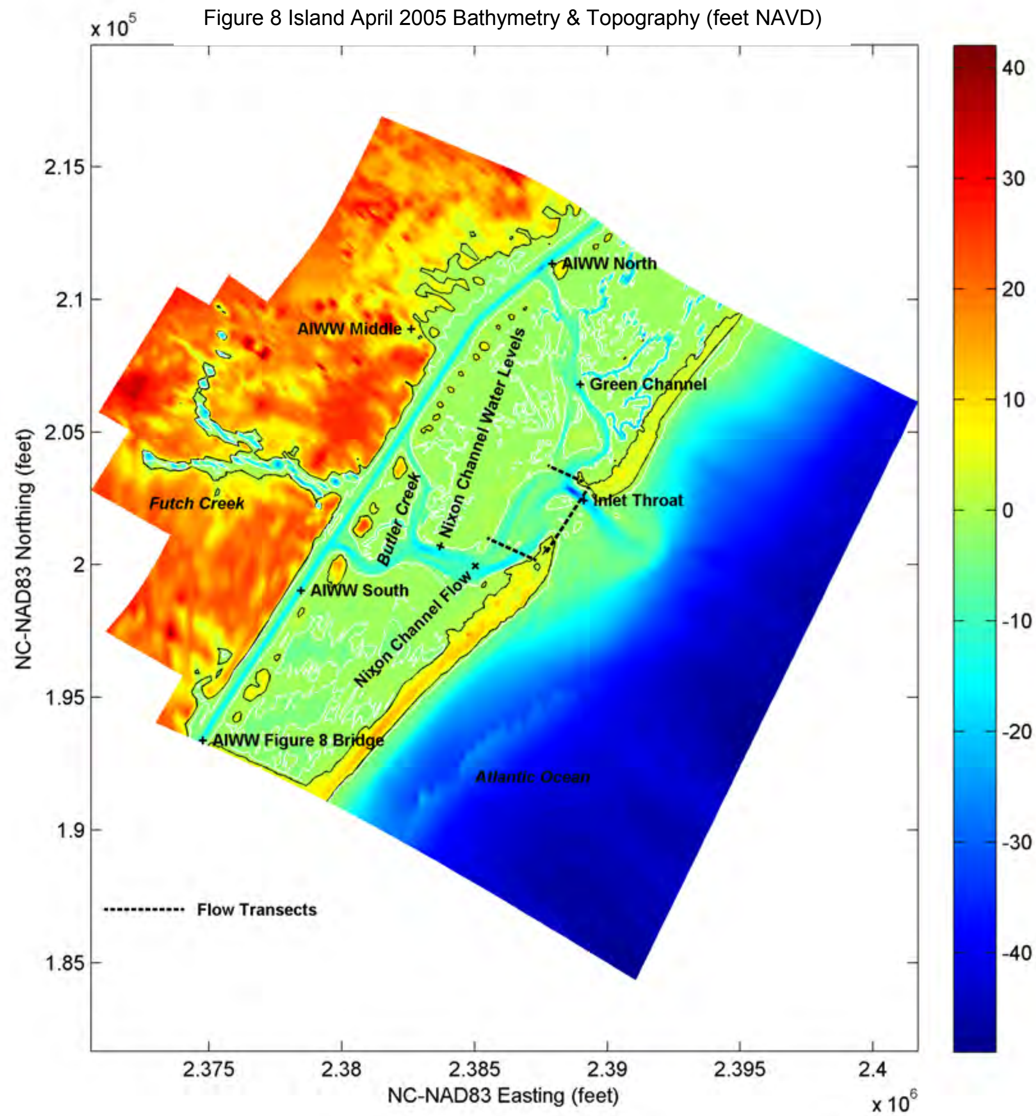


FIGURE 4-1: Tide Gage and Current (Flow) Meter Locations in Rich Inlet.

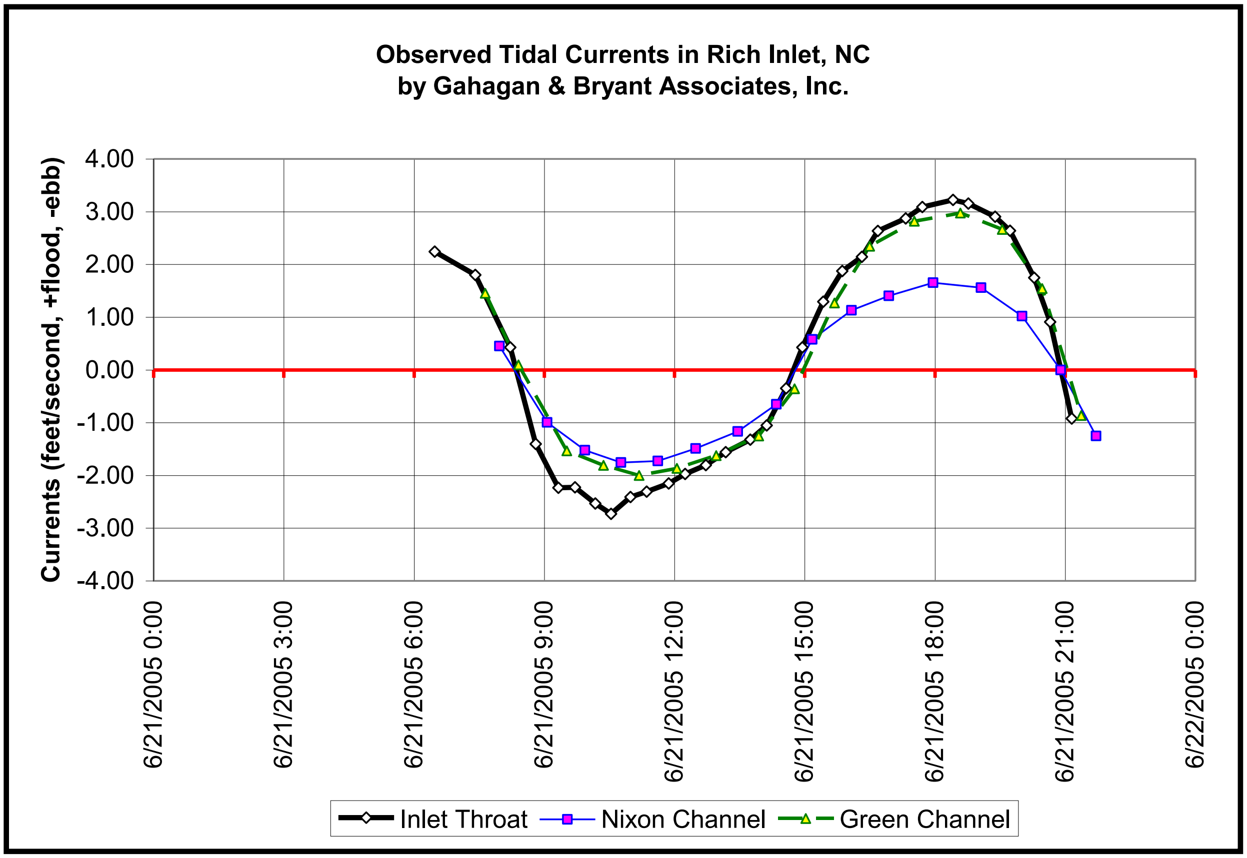


FIGURE 4-2: Tidal Currents during Spring Tide, Rich Inlet, NC.

4.4 Waves

Annual wave statistics at Figure Eight Island are based on the 2002-2005 wave observations at buoy OB3M (UNCW, 2007). The location of this gage is 34º06.133'N, 77º45.049'W at a depth of 52 feet (Figure 4-3). The root-mean-square wave height offshore is 3.3 feet, with a corresponding period and direction of 7.1 seconds and 139º (southeast). The principal direction bands are from the east-southeast and the southeast. The highest waves occur in February during the northeaster season and in August and September during hurricane season. During the summer, waves tend to approach from the south-southeast, driving the sediment transport towards the northeast. During the winter, waves tend to approach from the east-southeast, driving the sediment transport towards the southwest. Annual wave statistics appear in Tables 4-3 and 4-4 and in Figures 4-4 to 4-6.

FIGURE 4-3: Figure Eight Island, NC Wave Gages and Hindcast Stations.

TABLE 4-3

2002-2005 MONTHLY WAVE STATISTICS AT WAVE BUOY OB3M
FIGURE EIGHT ISLAND, NC

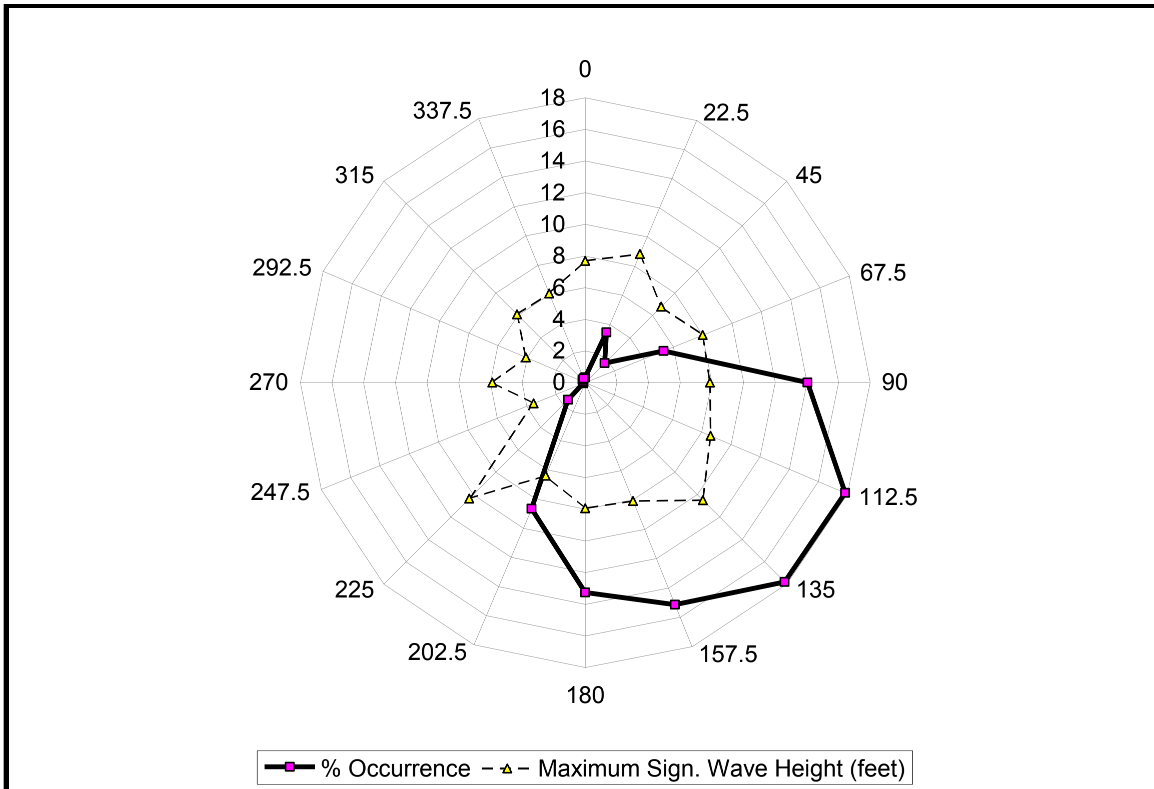
	Wave Height (feet)			Peak Wave Period (sec.)			Peak Wave Direction (deg.)		
	Mean	RMS	Max	Mean	Max	of Highest	Avg. #1*	Avg. #2**	of Highest
January	2.3	2.4	5.2	6.9	10.6	4.7	128	128	65
February	4.3	4.7	10.4	7.4	11.6	8.5	103	96	220
March	3.2	3.4	6.9	7.2	16.0	6.4	114	111	116
April	2.8	3.1	7.0	7.0	12.8	9.1	136	138	144
May	2.8	3.1	7.6	7.0	12.8	6.7	144	136	128
June	2.6	2.8	6.4	6.6	10.6	6.0	153	147	202
July	2.5	2.7	6.1	6.6	10.6	2.0	162	159	156
August	2.9	3.1	10.5	6.6	25.6	8.0	140	134	139
September	3.9	4.1	8.1	8.0	18.2	16.0	124	124	147
October	3.1	3.3	5.8	8.1	16.0	5.8	112	113	118
November	2.8	3.1	7.7	7.5	14.2	8.5	119	119	153
December	3.0	3.4	8.5	6.8	18.2	8.0	127	125	103
AVG.	3.0	3.3	10.5	7.1	25.6	8.0	134	127	139

Notes: * Average direction #1 is a simple average of the wave direction.
** Average direction #2 is the direction of the average wave energy flux.

TABLE 4-4

2002-2005 DIRECTIONAL WAVE STATISTICS AT WAVE BUOY OB3M
FIGURE EIGHT ISLAND, NC

Angle	%	Wave Height (feet)			Peak Wave Period (sec.)		
Band (deg.)	Occur.	Mean	RMS	Max	Mean	Max	of Highest
0	0.3	2.8	3.1	7.7	3.6	4.9	4.9
22.5	3.4	3.3	3.7	8.8	6.7	18.2	7.5
45	1.7	3.3	3.5	6.8	4.8	9.8	5.5
67.5	5.3	3.7	4.0	8.0	5.7	16.0	6.4
90	14.0	3.4	3.7	7.9	6.9	16.0	7.5
112.5	17.8	2.9	3.2	8.6	8.1	18.2	8.0
135	17.8	2.9	3.2	10.5	8.5	18.2	8.0
157.5	15.1	2.9	3.2	8.1	7.5	18.2	16.0
180	13.3	2.8	3.0	7.9	6.5	25.6	7.5
202.5	8.6	2.6	2.8	6.4	5.3	16.0	6.0
225	1.5	2.8	3.1	10.4	4.8	16.0	8.5
247.5	0.1	2.5	2.6	3.5	4.1	7.1	7.1
270	0.1	3.2	3.5	5.9	4.7	5.5	4.7
292.5	0.1	2.8	2.9	4.1	4.8	8.5	3.6
315	0.2	3.7	3.9	6.1	4.2	6.4	6.4
337.5	0.3	2.7	2.9	6.1	5.3	18.2	5.3



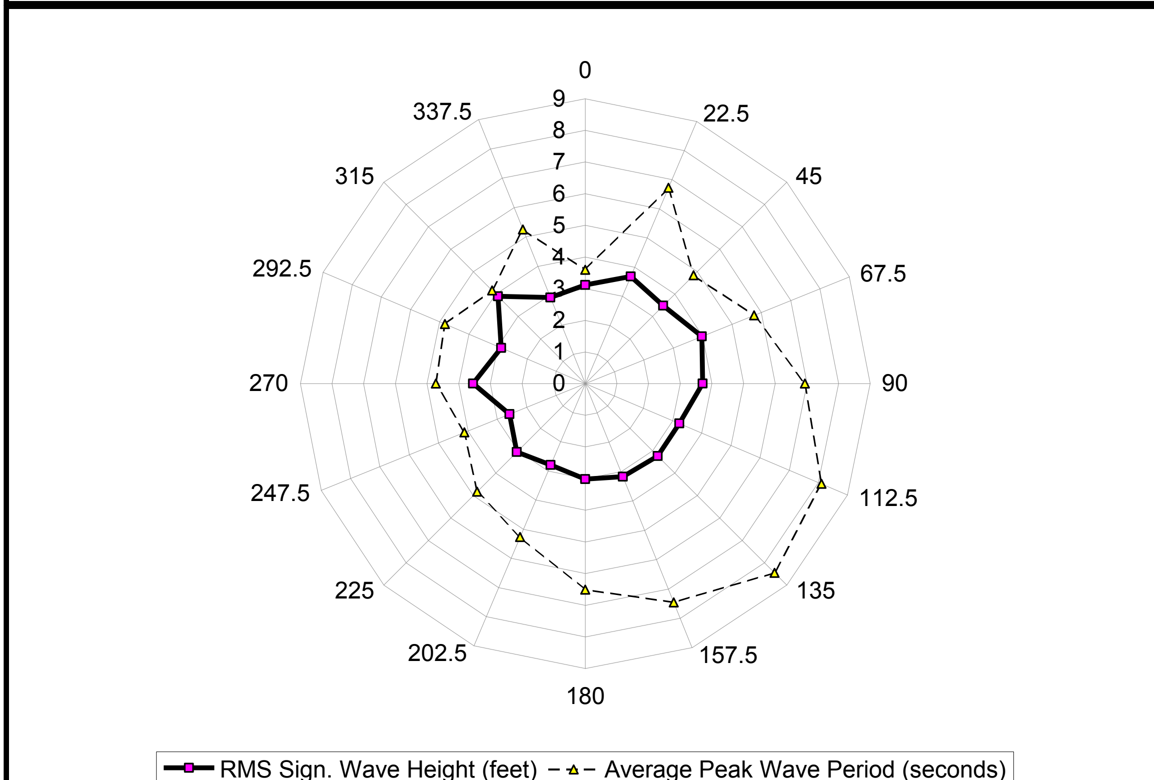


FIGURE 4-4: Directional Wave Statistics, Wave Buoy OB3M, Figure Eight Island, NC.

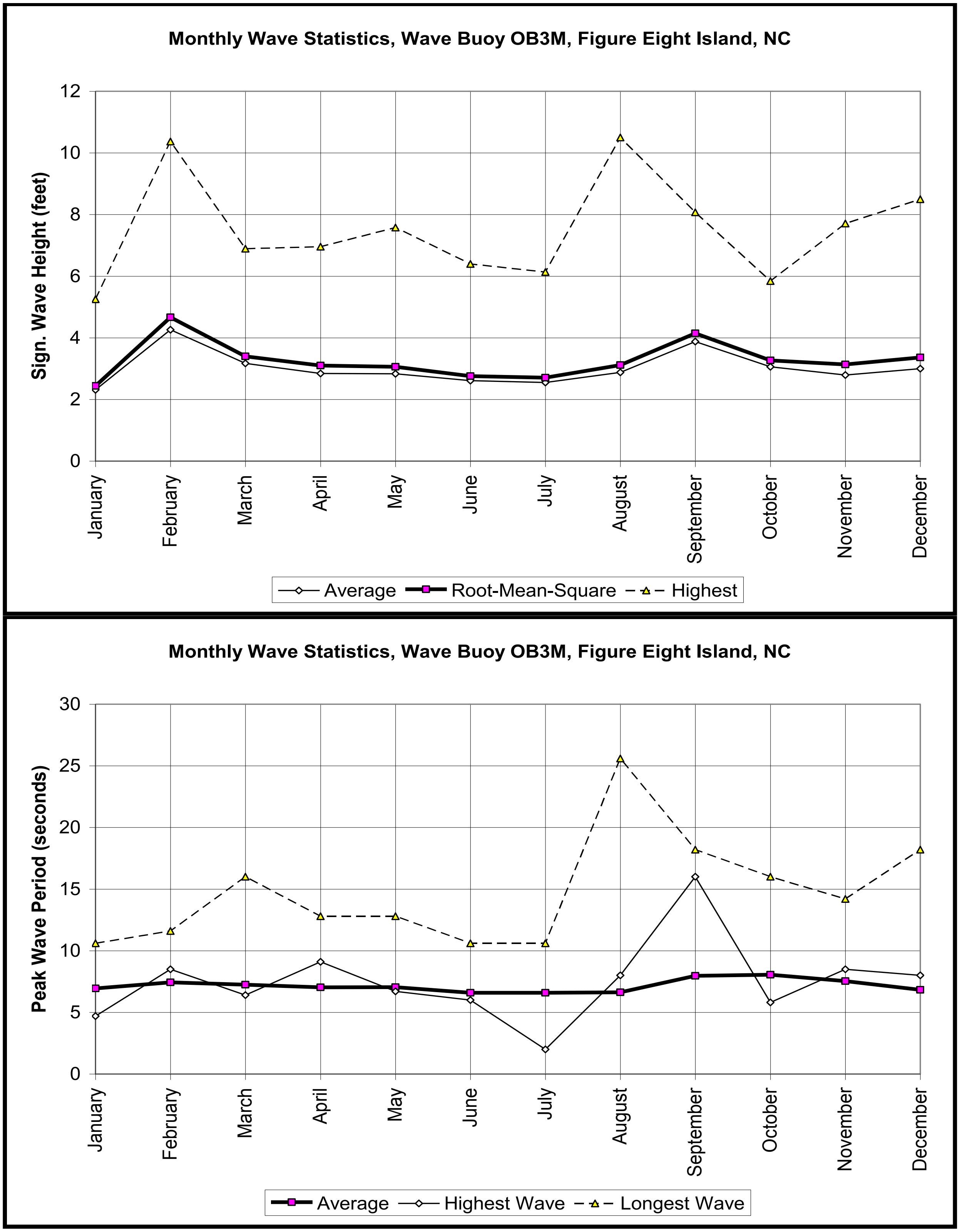


FIGURE 4-5: Monthly Wave Height and Wave Period, Figure Eight Island, NC.

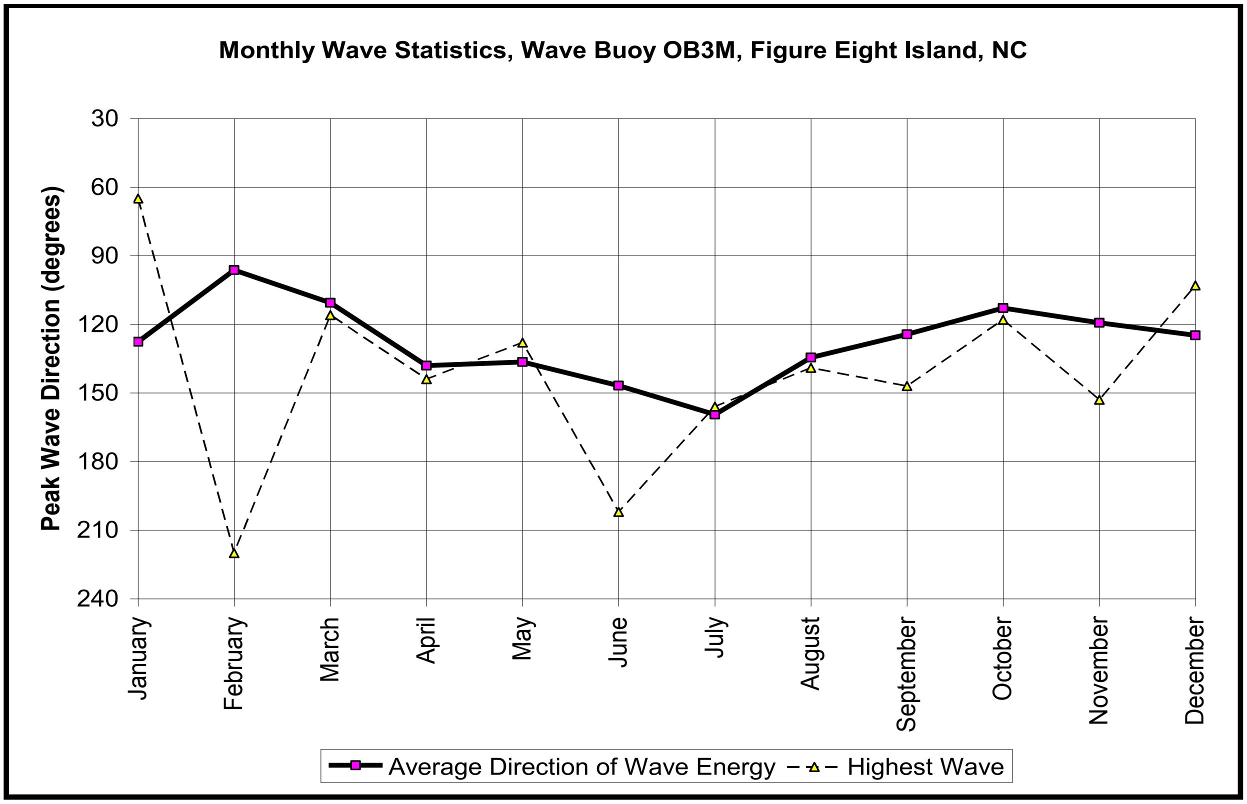
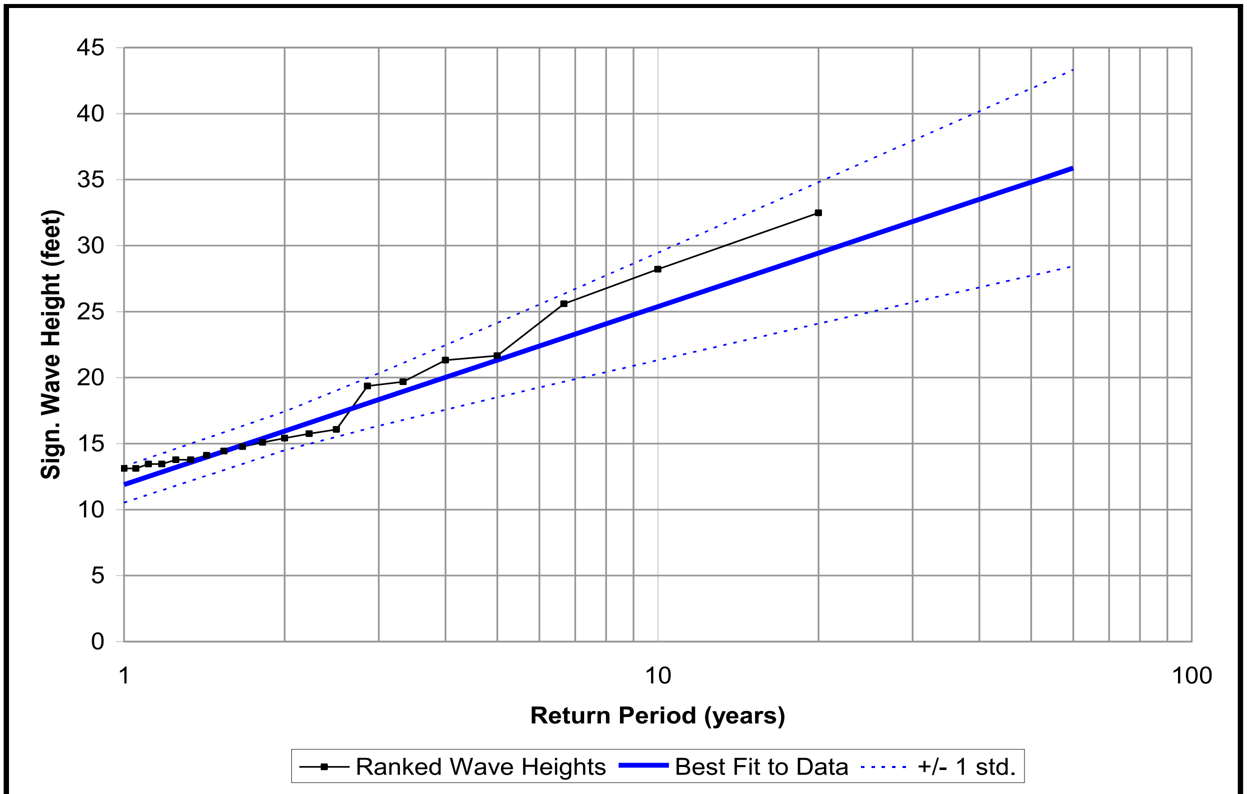


FIGURE 4-6: Monthly Wave Direction, Figure Eight Island, NC.

For numeric modeling purposes, wave conditions during storms were based on the 20 year wave hindcast record at Wave Information System (WIS) Station 296 (Figure 4-3). Wave conditions during severe storms were estimated in terms of return period. The return period represents the chance of a given wave event being exceeded in any given year. For example, the 20 year wave has a 1 on 20 chance of being exceeded in any given year. To delineate the wave height and wave period versus return period, the 20 highest wave events were taken from the wave record. A Weibull distribution was then estimated for the highest 20 wave events. The resulting wave heights and wave periods given the return period appear in Figure 4-7 and Table 4-5.



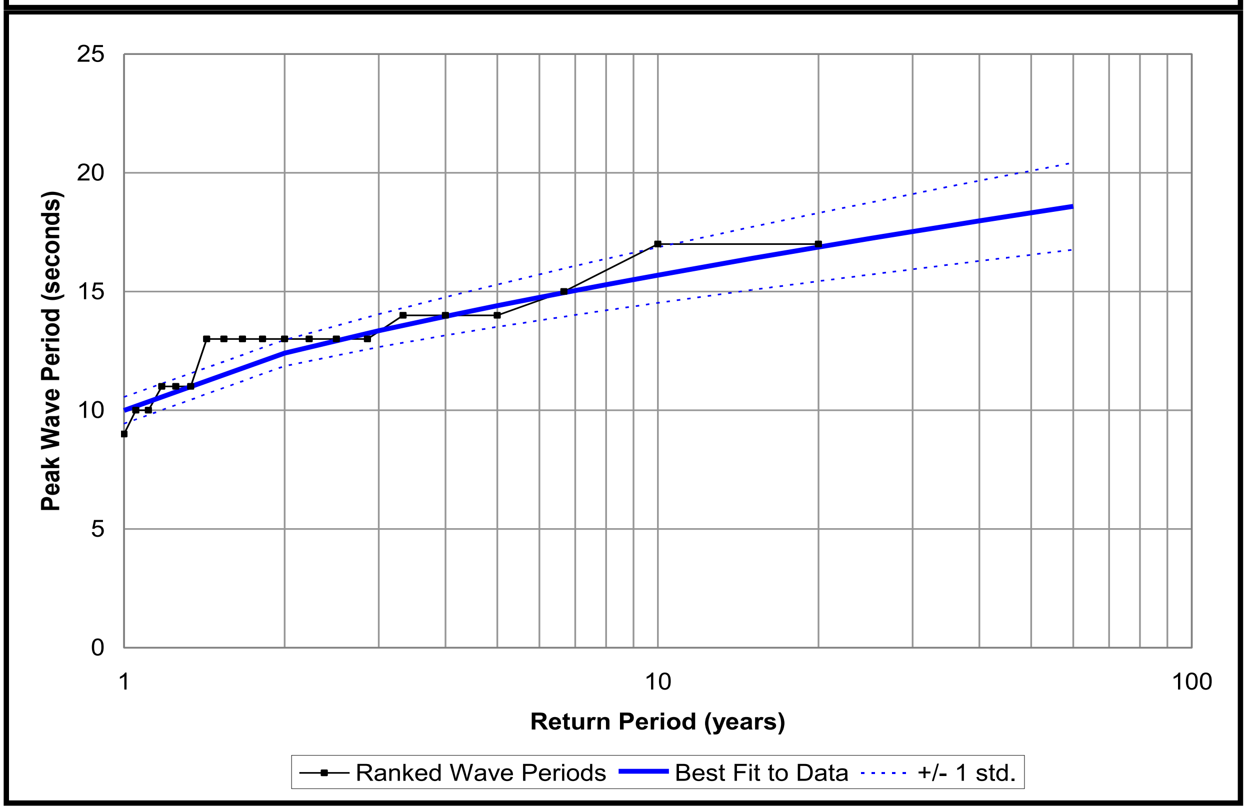


FIGURE 4-7: Storm Wave Statistics, Hindcast Station WIS296, Figure Eight Island, NC.

TABLE 4-5

1980-1999 STORM WAVE STATISTICS
HINDCAST STATION WIS296
FIGURE EIGHT ISLAND, NC

Return Period (years)	Wave Height H_{mo} (feet)	+/- σ	Wave Period T_p (sec.)	+/- σ
1	**11.9**	**1.4**	**10.0**	**0.6**
2	16.0	1.5	12.4	0.5
3	18.3	2.0	13.4	0.7
4	20.0	2.4	14.0	0.8
5	**21.3**	**2.8**	**14.4**	**0.9**
6	22.4	3.1	14.8	1.0
7	23.3	3.4	15.0	1.0
8	24.1	3.7	15.3	1.1
9	24.8	3.9	15.5	1.1
10	**25.4**	**4.1**	**15.7**	**1.2**
15	27.8	4.8	16.4	1.3
20	**29.4**	**5.4**	**16.9**	**1.4**
25	30.7	5.8	17.2	1.5
30	31.8	6.1	17.5	1.6
35	32.7	6.4	17.8	1.6
40	33.5	6.7	18.0	1.7
45	34.2	6.9	18.2	1.7
50	**34.8**	**7.1**	**18.3**	**1.8**
60	35.9	7.4	18.6	1.8

4.5 Storm Surge

Storm surge is defined as the rise of the sea surface above its astronomical tide level due to storm forces. The elevation that the storm surge reaches is known as the storm stage. The increase elevation is attributable to a variety of factors, including waves, wind shear stress, and atmospheric pressure. Storm stages are an important factor governing the performance of a beach fill during storms.

The Federal Emergency Management Agency (FEMA) released a Flood Insurance Study on April 3, 2006 for New Hanover County, North Carolina. The study detailed the storm stage elevations for 10, 50, 100, and 500 year storms. Oceanfront storm stages appear in Table 4-6 and Figure 4-8. The numerical models used in this study utilize offshore water levels as an input and calculate wave setup as an output. Accordingly, the stage values in Table 4-6 do not include wave setup. Detailed discussions of the SBEACH and Delft3D models appear in later sections of this report.

TABLE 4-6

OCEAN STORM STAGES
FIGURE EIGHT ISLAND, NC

FEMA Transect	Location	Storm Stage in feet NAVD given return period in years (excluding wave setup)			
		10	50	100	500
58	Approximately 2,430' south of intersection of Pipers Neck Rd. and Sounds Pt.	5.7	8.7	9.9	12.4
59	Approximately 645' southeast of intersection of Pipers Neck Rd. and Little Neck Rd.	5.7	8.7	9.9	12.4
60	Approximately 290' southeast of intersection of Saltmeadow Rd. and S. Beach Rd.	5.7	8.7	9.9	12.4
61	Approximately 720' northeast of intersection of S. Beach Rd. and Banks Rd.	5.7	8.7	9.9	12.4
62	Approximately 960' northeast of intersection of S. Beach Rd. and Backfin Pt.	5.7	8.7	9.9	12.4
63	Approximately 590' east of intersection of N. Beach Rd. and Bayberry Pl.	5.5	8.6	9.9	12.3
64	Approximately 1610' northeast of intersection of N. Beach Rd. and Salters Rd.	5.4	8.5	9.8	12.3
65	Approximately 1250' southwest of intersection of N. Beach Rd. and Clamdigger Point Rd.	5.3	8.5	9.8	12.3
66	Approximately 830' southeast of intersection of Surf Ct. and N. Beach Rd.	5.3	8.5	9.8	12.3
67	Approximately 520' east of intersection of N. Beach Rd. and Oyster Catcher Rd.	5.3	8.5	9.8	12.3
	Minimum Average Maximum	5.3 5.5 5.7	8.5 8.6 8.7	9.8 9.9 9.9	12.3 12.4 12.4

Source: FEMA (2006).

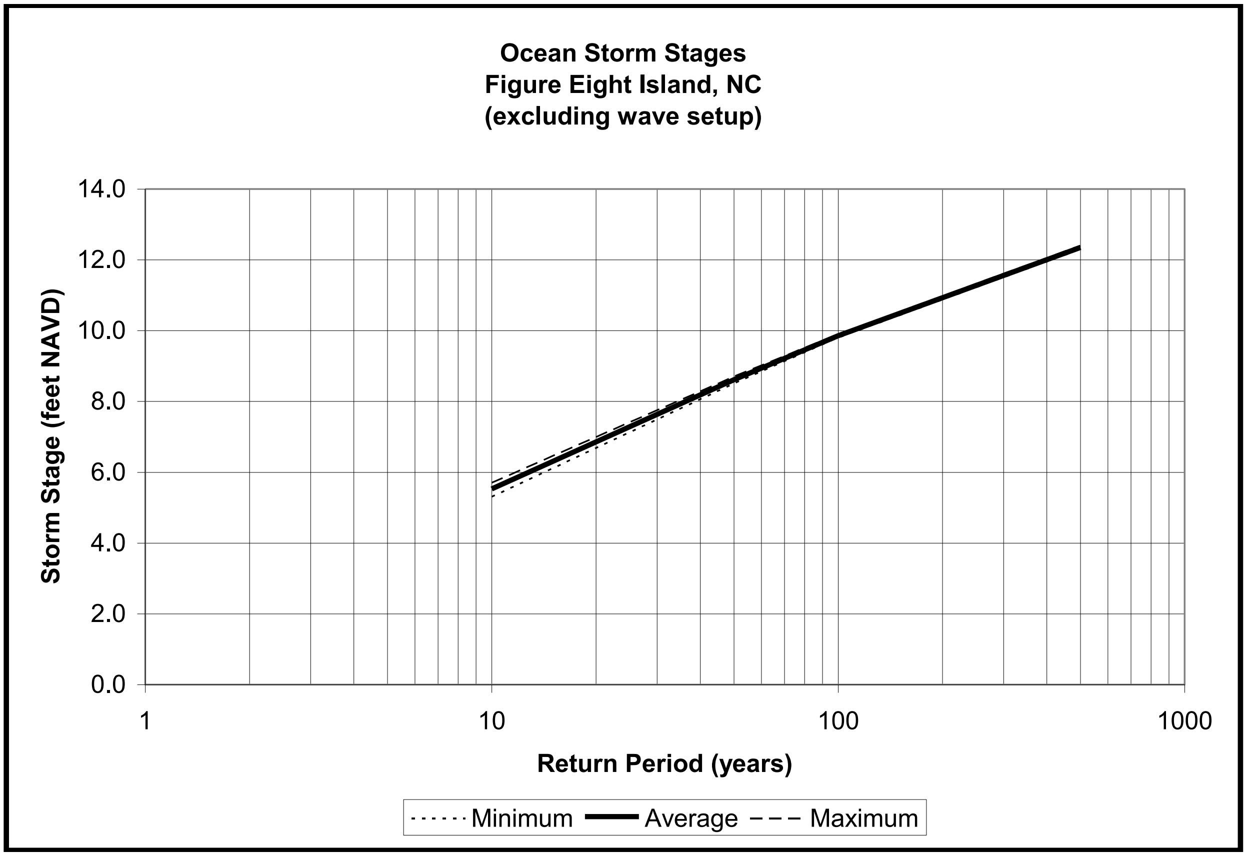


FIGURE 4-8: Ocean Storm Stages, Figure Eight Island, NC.

4.6 Depth of Closure

The depth of closure is defined as the "depth beyond which repetitive profile or topographic surveys (collected over several years) do not detect significant vertical sea bed changes. This is generally considered the seaward limit of littoral transport" (Morang and Szuwalski, 2003). The depth of closure is typically estimated by either comparing historic profiles and observing where the profiles close (pinch out and have no elevation difference) or using empirical equations, such as the ones developed by Hallermeier (1978) or Birkemeier (1985).

Historic profiles of Figure Eight Island were compared for surveys taken in October 2004, April and October 2005, and April 2006. The profiles appeared to close at an average depth of -24 feet NAVD, with closure depths ranging from -17 feet to -31 feet NGVD. This estimate was consistent with the established depth of closure for Topsail Beach (Figure 4-3), which was also -24 feet NAVD (USACE, 2006).

Empirical equations were also used to estimate the depth of closure for the project area. The Hallermeier (1978) and Birkemeier (1985) empirical equations are based on the significant wave event that is exceeded 12 hours per year (H_e and T_e). Hallermeier's equation is Equation 1, while Birkemeier's equation is shown as Equation 2.

Hallermeier's equation:

$$h_* = 2.28H_e - 68.5\left(\frac{H_e^{\,2}}{gT_e^{\,2}}\right)$$ [Equation 1]

Birkemeier's equation:

$$h_* = 1.75H_e - 57.9\left(\frac{H_e^{\,2}}{gT_e^{\,2}}\right)$$ [Equation 2]

The 12-hour wave event at WIS Station 296 (between 1980 and 1999) was found to have a significant wave height (H_e) of 15.8 feet and a period (T_e) of 12.5 seconds. The ACES linear wave transformation program suggests that this wave is transformed to a 21.9-foot wave near the shoreline. Application of Hallermeier's equation suggests that the depth of closure is -43.4 feet, MSL while Birkemeier's equation suggests that the depth of closure is -32.8 feet, MSL.

Based on experience, the depths of closure based on these two equations appear to be an overestimate of the depth to which sediment would be transported following a beach nourishment project. The established depth of closure for Topsail Beach (USACE, 2006) is the same as the survey-based value for Figure Eight Island. Accordingly, -24 feet NAVD has been chosen as the depth of closure for the development of this project.

4.7 Relative Sea Level Rise

The rate of sea level rise applicable to Figure Eight Island was determined from the average of sea level change rates observed at Beaufort, NC (0.0089 ft/yr), and Wilmington, NC (0.0067 ft/yr). The observed sea level trends are available from: http://tidesandcurrents.noaa.gov. The period of sea level observations used to establish these rates are 61 years for Beaufort, NC and 79 years for Wilmington, NC. The average rate of rise for these two stations is 0.0078 ft/yr.

The impacts of sea level rise on shoreline changes along Figure Eight Island due to a relative rise in sea level of 0.0078 ft/yr were based on the well known Brunn Rule (Brunn, 1962). Per Brunn theorized that as sea level rises, the beach profile attempts to reestablish the same bottom depths relative to the surface of the sea that existed prior to the rise in sea level. The quantity of material needed to reestablish the beach profile must be derived from erosion of the shore. This theory is expressed by the equation:

$$\Delta x = ab/(e+d)$$

where:

Δx = rate of shoreline recession due to sea level rise.
e = elevation of the beach berm (+ 6 feet NAVD).
d = limiting depth between predominant nearshore and offshore material transport characteristics (-24 feet NAVD).
a = rate of sea level rise (0.0078 ft/yr)
b = distance from the initial shoreline to the limiting depth (average about 2,000 feet for Figure Eight Island).

For Figure Eight Island, the rate of shoreline erosion (Δx) associated with a sea level rise rate of 0.0078 ft/yr is equal to about 0.5 ft/year.

A recent study completed by the North Carolina Coastal Hazards Science Panel (2015) evaluated possible increases in the rate of rise of sea level due to changing climate conditions as projected by the IPCC (2013). For Beaufort, NC, the Panel estimated a possible rate of sea level to range from 0.0181 ft/year to 0.0208 ft/year for low and high greenhouse gas emissions, respectively. Similarly, the Panel projected possible rates of 0.0161 ft/year and 0.0189 ft/year for Wilmington, NC based on IPCC low and high greenhouse gas emissions. The average of these projections for application to Figure Eight Island results in possible future rates of sea level rise of between 0.0171 ft/year and 0.0199 ft/year. Inserting these rates in the Brunn Rule results in a range of possible shoreline retreat rates due to sea level rise of between 1.1 ft/year to 1.3 ft/year for the low and high greenhouse gas emission scenarios, respectively.

4.8 Native Beach Grain Size

To evaluate the materials presently on the beach, sand samples were collected in September 2007 from profiles F80+00, 10+00 (F120+00), 50+00 (F160+00), and 90+00 (F200+00) on Figure Eight Island. Due to several beach fill projects constructed along Figure Eight Island prior to sampling, these samples did not represent the "native materials" as defined by the North Carolina Technical Standards for Beach Fill Projects (15A NCAC 07H.0312). After discussion with State representatives, it was decided that sampling of the adjacent barrier island, Hutaff Island would be necessary to determine native composites. Additional samples were taken from profiles 160+00 (H1), H2, and H3 on Hutaff Island in September, 2007, along with the samples collected on Figure Eight Island. All profiles were sampled at the following locations:

- Dune
- Toe Of Dune
- Mid-Berm
- +2.0 to +3.0 feet NAVD
- Mean High Water
- Mean Tide Level
- Mean Low Water
- -6 feet NAVD
- -8.8 feet NAVD
- -11.6 feet NAVD
- -14.4 feet NAVD
- -17.2 feet NAVD
- -20 feet NAVD

The existing "beach" composites on Figure Eight Island are summarized in Table 4-7, along with the native composites on Hutaff Island. The locations of each sand sample appear in Figure 4-9.

TABLE 4-7

**EXISTING BEACH COMPOSITES
FIGURE EIGHT ISLAND AND
HUTAFF ISLAND, NC**

PROFILE	Mean Grain Size (mm)	Mean Grain Size (Φ)	Sorting (Φ)	% Silt	% Carbonate
F80+00	0.19	2.40	0.66	0.96	7.9
10+00 (F120+00)	0.18	2.45	0.55	1.03	5.4
50+00 (F160+00)	0.18	2.45	0.50	1.13	4.8
90+00 (F200+00)	0.18	2.47	0.46	1.04	5.9
Figure Eight Island December 2007 "Beach" Composite	**0.18**	**2.44**	**0.55**	**1.04**	**6.0**
160+00 (H1)	0.20	2.33	0.64	0.89	6.9
H2	0.19	2.41	0.59	0.97	5.9
H3	0.24	2.03	1.16	1.14	17.0
Hutaff Island December 2007 Native Composite	**0.21**	**2.26**	**0.85**	**1.00**	**9.9**

The native material on Hutaff Island is fine sand and exhibits a mean grain size of 0.21 mm, a sorting value of 0.85Φ, a carbonate content of 10%, and a low silt content of 1%. The "beach" material on Figure Eight Island is also fine sand, and exhibits a mean grain size of 0.18 mm, a sorting value of 0.55Φ, a carbonate content of 6%, and a silt content of 1%. The "beach" material on Figure Eight Island is slightly finer than the truly native material on Hutaff Island. However, the difference between the two composites is not large, and suggests that the fill placed in 2006 has mixed with the native material. A more detailed discussion of the materials presently on the beach appears in the Geotechnical Investigation for this study.

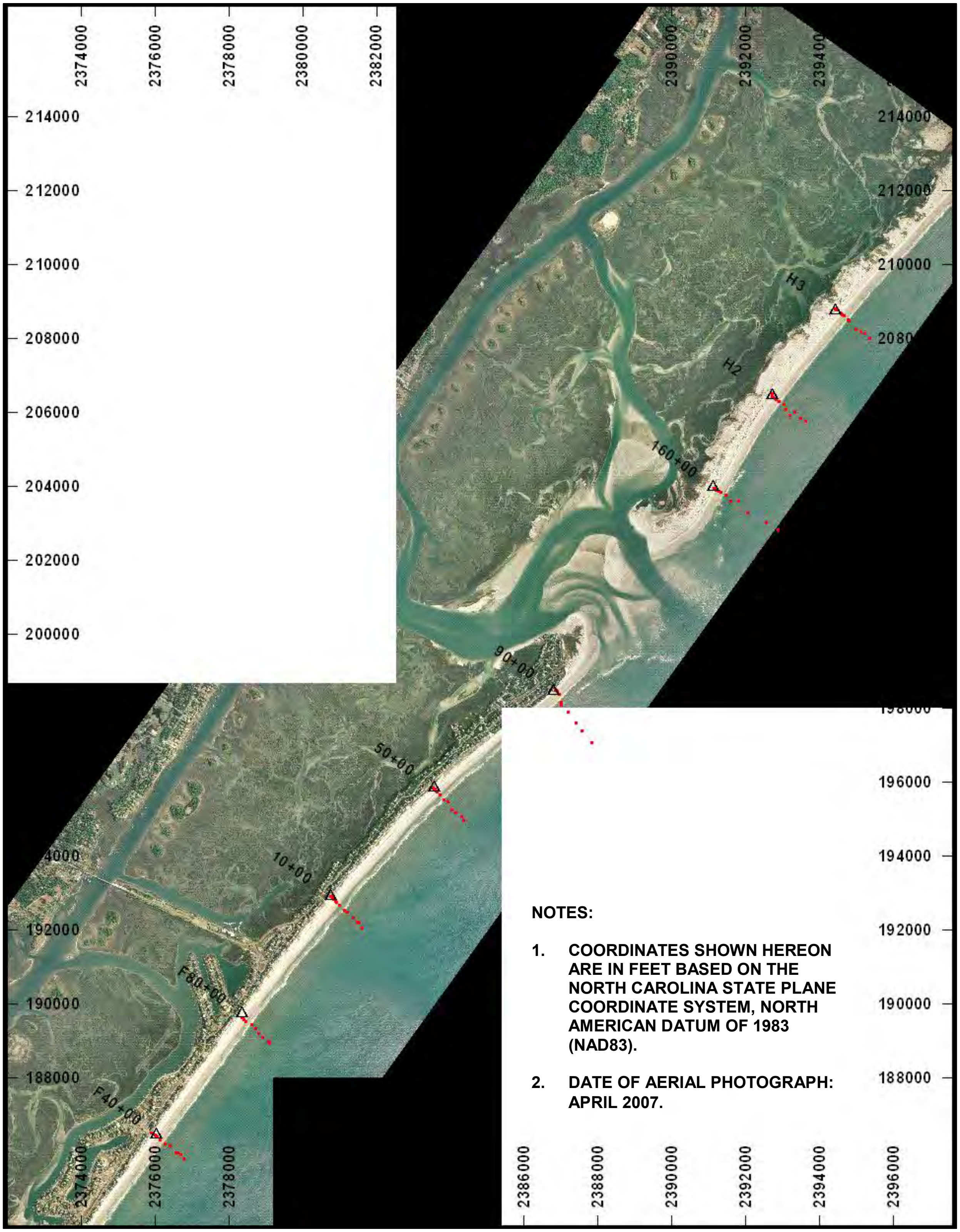


FIGURE 4-9: December 2007 Sand Samples, Figure Eight Island and Hutaff Island, NC.

4.9 Inlet Grain Size

In general, the material in Rich Inlet is fine sand. Based on the geotechnical information, the mean grain sizes of the material in the dredge cuts for Rich Inlet range from 0.18 to 0.30 mm, with sorting values ranging from 0.44 to 1.16Φ, and silt contents on the order of 1%. The composite for the dredge cuts has a mean grain size of 0.24 mm, a sorting value of 0.83Φ, and a silt content of 1%. A more detailed discussion of the materials in the dredge cuts appears in the final Geotechnical Investigation for this study.

4.10 Tidal Prism of Rich Inlet

Several estimates of the tidal prism have been developed for Rich Inlet (Table 4-8). Two sets of estimates appeared in a study by Cleary and Knierim (2003). One set was based on an Acoustic Doppler Current Profiler (ADCP) survey, and the second set was based on empirical relationships between tidal range and tidal prism.

TABLE 4-8

RICH INLET TIDAL PRISM ESTIMATES
FIGURE EIGHT ISLAND, NC

SOURCE / METHOD	TIDAL PRISM THROUGH INLET THROAT (cubic feet)					
	SPRING TIDES		AVG. TIDES		NEAP TIDES	
	FLOOD	EBB	FLOOD	EBB	FLOOD	EBB
Cleary & Knierim (2003)						
ADCP Survey	797,000,000	690,000,000	603,000,000	562,000,000	329,000,000	430,000,000
Empirical Relationships	645,000,000	652,000,000	469,000,000	434,000,000	318,000,000	247,000,000
Gahagan & Bryant (2005) Measurements	1,101,000,000	560,000,000	N/A	N/A	N/A	N/A
Delft3D Model with Waves April 2006 Conditions	N/A	N/A	653,000,000	697,000,000	N/A	N/A

Tidal prism estimates were also estimated based on a later ADCP survey by Gahagan & Bryant (2005). The depth-averaged currents (Figure 4-2) were combined with concurrent water levels and survey data (Figure 4-1) to evaluate the flow rate through the inlet throat in cubic feet per second. Flow rates were then integrated over the flood and ebb cycles shown in Figure 4-1. A final set of tidal prism estimates was based on the Delft3D modeling results. The tidal prism estimates varied widely. However, based on the values in Table 4-8, the average tidal prism was on the order of 560,000,000 cubic feet. A further discussion of the tidal prism appears in the Delft3D modeling study.

5.0 CHANNEL EVOLUTION

Erosion and accretion along relatively stable inlets such as Rich Inlet are related to complex cyclical changes in the shape of the ebb-tidal deltas. Cycles are associated with the repositioning and realignment of the ebb channel and corresponding position and size changes of the marginal flood channels and where swash bars welded onto the adjacent shorelines (FitzGerald, 1984; Cleary, 1994, 1996, and 2002; Cleary and Marden, 1999; Cleary et al., 1989).

Rich Inlet drains an extensive estuary filled with tidal marsh where two large tidal creeks, Nixon and Green Channels, connect the inlet to the Atlantic Intracoastal Waterway (AIWW). It is an example of a relatively stable inlet where the repositioning and realignment of the ebb channel leads to dramatic erosion on one or both adjacent beaches. Erosion occurs as the shape of the offshore sand shoals changes thereby affecting impact of incoming waves on the nearby beaches. Historic map and geomorphic data indicate the inlet has been a relatively stable feature over the past century. The large drainage area that includes portions of the bar-built lagoon and Pages Creek estuary enhances the inlet's stability.

A GIS-based analysis of historic aerial photographs dating from 1938 to 2003 was undertaken by Cleary and Jackson (2004) to quantify shoreline changes, their connection to the inlet's migration, and the system changes of the inlet. Cleary provided an update of this analysis which appears in Sub-Appendix A.

5.1 Historic Channel Alignment (Cleary and Jackson, 2004)

"The recent movement of the ebb (entrance) channel has been confined to a ~0.30 mile wide pathway. The ebb-tidal delta is situated on Oligocene siltstone that crops out along the ebb delta's outer margin in water depths of 30 feet. The width of the inlet throat reached a maximum of 2,673 feet in October of 1989 and a minimum of 920 feet in February of 2001. The average with the inlet throat since 1938 was 2,000 feet.

Since 1938, the position of the ebb (entrance) channel has remained within a 1,600 foot wide migration corridor, indicating that Rich Inlet has been relatively stable. Through the period from 1938 to 2003, the orientation of the ebb channel across the outer portion of the ebb-tidal delta has fluctuated between 83˚ and 181˚. Between 1938 and 1993, the ebb channel was oriented predominately in a southeasterly direction between 112˚ and 181˚ before realigning to a more easterly orientation of 103˚ in 1996. The ebb channel's alignment and position prior to the mid-1990s promoted the development of a one-mile long zone of accretion along the Figure Eight Island oceanfront immediately south of the inlet. During the period from 1993 to 1996, the ebb channel rapidly migrated 1,056 feet northeast at a rate of 308 feet per year. Between August 1996 and February 1998, the ebb channel shifted 147 feet further to the northeast before reversing its migration direction to the southwest in June 1998. Inspection of aerial photographs shows that between June 1998 and February 2002, the ebb channel migrated a distance of 588 feet to the southwest at a rate of 160 feet per year.

While the ebb channel tracked to the northeast between March 1993 and February 1998, the northern spit of Figure Eight Island elongated, dramatically reducing the inlet's width. Although the migration direction changed to the southeast in June 1998, the orientation of the ebb channel continued to be deflected in a northeasterly direction before reaching alignment of 83˚ in October 2000. A breach of the ebb-tidal delta occurred in the latter part of 2000 that resulted in a shore-normal repositioning of the ebb channel. Between February 2001 and March 2003, the outer segment of the ebb channel was continually deflected from its 156˚ alignment in early 2001 to an alignment of 190˚ by early 2003. During late 2003 and early 2004, the ebb channel was reoriented to a shore normal alignment.

Previous studies have shown that the position and orientation of the ebb channel has controlled the shape and ebb tidal delta and ultimately dictates the shoreline changes along the adjacent oceanfront shorelines of Figure Eight Island.

In order to reverse the current erosion trend and promote accretion along the northern oceanfront of Figure Eight Island, the ebb channel must assume a position that approximates the location of the ebb (entrance) channel imaged in 1980 and maintain a near shore-normal orientation of ~145 degrees. For this repositioning to occur the ebb channel must migrate ~1,300 feet to the southwest."

5.2 Location of Ebb Shoal Apex (Cleary and Jackson, 2004)

"The position and alignment of the ebb channel has controlled the symmetry of the ebb-tidal delta and its apex. The changes in the shape of the ebb-tidal delta and in the position of its apex (seaward protrusion) since 1938 are depicted in Figure [5-1]…. Changes in the position of the apex, with time, are a function of the complex interplay of ebb channel (inlet) migration and the deflection of the outer ebb channel. Storms are also thought to contribute to the observed changes in the shape of the ebb-tidal delta. Regardless of the mechanism, the position of the ebb-tidal delta's apex plays a major role in the controlling the manner in which waves impact the oceanfront shorelines in the immediate vicinity of the inlet.

The location of the apex generally coincides with the point where the ebb channel crosses the periphery of the ebb-tidal delta. Deflection of the ebb channel since 1938 has caused a shift in the position of the apex and shape change of the ebb tidal delta across a ~5,100 foot wide zone. As ebb channel migration occurred, the entire offshore shoal complex was continuously being reconfigured along the with adjacent barrier shorelines as they responded to the changes in wave approach and sand supply. The current ebb-tidal delta shape has controlled the erosion since 1997. The zone of maximum erosion along the oceanfront shorelines has generally shifted eastward through time as the ebb channel has migrated to the northeast. The northeasterly shift of the channel has not only dictated the shape of the offshore shoals that afford protection for the end of the island, but simultaneously this shift has controlled the location where large swash bar complexes attach to the shoreline.

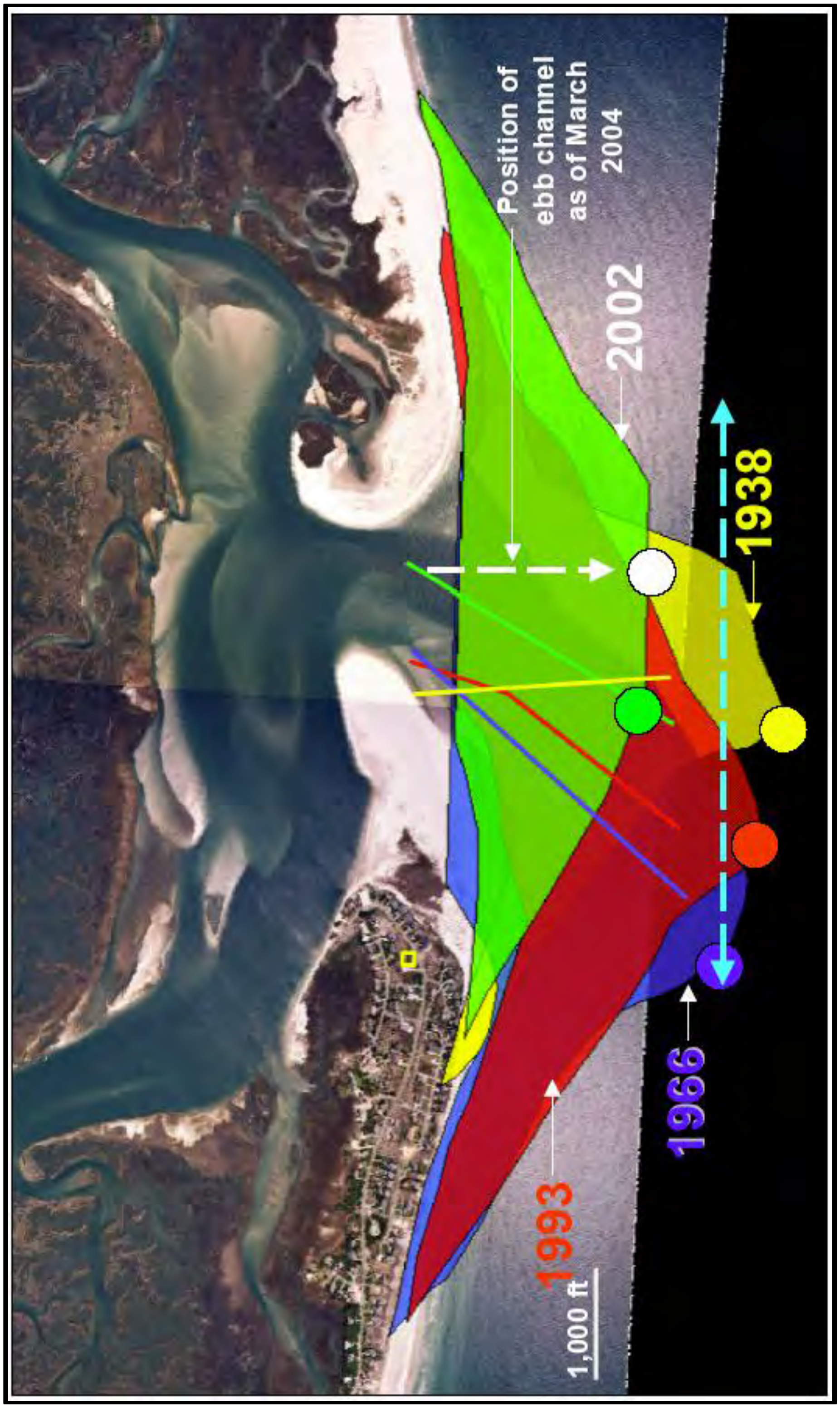


FIGURE 5-1: Aerial photograph (March 2002) with shapes of ebb deltas (as defined by zone of breaking waves), ebb channel positions and apex of ebb deltas (colored dots). The white arrow and dot represent approximate position of the ebb channel and apex in March 2004. Dashed light blue arrow delineates the width of the zone of deflection of the ebb delta apex (dots) (Cleary and Jackson, 2004).

A repositioning of the ebb channel toward Figure Eight Island will lead to a seaward shift and repositioning of the apex to the southwest. The consequences of this net change will reverse the erosion trend that has characterized the oceanfront since 1997.

Any future modification of the inlet should consider the ebb channel's optimum position alignment and the consequent ebb-tidal delta symmetry and related potential shoreline changes. The most felicitous ebb channel position and alignment for shoreline accretion on Figure Eight Island is a configuration where the ebb channel is shore normal and is positioned along the southern portion of its migration pathway, ~1,300 feet to 1,500 feet southwest of its current potion. Any plans that result in a substantial deviation from the above configuration will lead to increased shoreline retreat along a position of the erosion hot-spot. If and when the ebb channel attains the aforementioned position, the ebb-tidal delta will begin to reconfigure and thereby cause a southwesterly shift in large volumes of sand and in the wave sheltering effects of the offshore shoal complex. It must be understood that it is likely there will be a lag effect in terms of the movement of the ebb channel and the timing of the positive impacts along the oceanfront. The lag is primarily due to the time needed for the remobilization of the enormous volume of sediment retained in the ebb-tidal delta that currently lies northeast of the erosion hot-spot. There is a high probability that a breach across the undeveloped spit could occur that will shorten the time lag considerably. The morphology of the inlet depicted on recent photographs and observations made during recent over-flight indicate that the spit is highly vulnerable to breaching when it is narrow."

6.0 SHORELINE CHANGE ANALYSIS

The Figure Eight Island shoreline is a dynamic feature in a constant state of flux due to changes in wave energy and sediment supply. When viewed in terms of decades or on the century scale, a complex set of factors, which operate in concert, have dictated shoreline change along both the oceanfront and inlet shorelines. Under the combined influence of cumulative storm impacts, waves, and inlets, the island has generally become erosional, although certain sections of the island accrete. Sea level rise also contributes to the erosion rates along the island. However, in comparison to the other forces driving erosion, the contribution of sea level rise, which is estimated to be around -0.5 ft/yr given historic sea level trends over the past half century, is minor. Even when taking into account possible increases in the rate of sea level rise, the shoreline recession rates attributable to sea level rise would still be less than 1.3 ft/yr. Much of the northern section of Figure Eight Island is characterized by multiple sets of dune ridges that reflect the buildup of the beach that is related to the influence of Rich Inlet. The presence of large intact dunes provides protection from flooding due to increased water levels and overtopping during storms.

During the late 1990s the complex interplay between the northeasterly migration of the channel and the continuing realignment of its outer segment has resulted in a shift of the breakwater effect of the ebb-tidal delta and a repositioning of it to the northeast. Consequently, the Figure Eight Island oceanfront was no longer afforded protection from wave attack. As a result, the northern 4,500 foot segment of the oceanfront, which has a history of net accretion, began to experience severe erosion.

In the fall of 2000, an ebb delta breaching event occurred that repositioned the ebb channel and initiated a southwestward trek of the inlet and promoted erosion along the downdrift Figure Eight Island shoreline. Between 2001 and 2003 the shoreline retreat averaged ~10 feet. In an effort to mitigate the chronic recession, 350,000 cubic yards of fill material was placed along the erosion zone and the area to the south in February and March 2001 (Cleary and Jackson, 2004). Much of the beach fill was lost by November 2001. In late 2001, erosion continued and reached critical proportions and as a last resort, large sand bags were placed along a number of the endangered homes in the area. The entirety of this shoreline stretch is now armored with a wall of sand bags. Additional fill was placed along this area in 2005 and 2006 (GBA, 2006). However, the shoreline response through March 2008 was similar to the shoreline change after the 2001 project (Figure 6-1).

FIGURE 6-1: North End of the Sandbagged Area, 4-7 Inlet Hook Road, March 18, 2008.

Oceanfront shoreline changes on Figure Eight Island since the October 1999 Light Detection and Ranging (LIDAR) survey by NOAA appear in Figure 6-2 and Table 6-1. The effect of beach fill (Table 6-2) was removed from these shoreline changes. In general, the northern and southern ends of the island erode, while the middle of the island accretes. Aside from the various beach fills, the northern end of the island (profiles 40+00 – 110+00) retreated 2 to 52 feet per year between October 1999 and April 2007. By contrast, the 3,000 foot segment on the south end of Hutaff Island advanced 15 feet/year between October 1999 and April 2005 (Table 6-3). Between April 2005 and April 2007, a large erosion loss occurred on southern Hutaff Island due to Hurricane Ophelia (October 2005) and the formation of a swash channel into Rich Inlet. Nevertheless, over the past 8 years as whole, the north end Figure Eight Island has experienced more erosion than the south end of Hutaff Island.

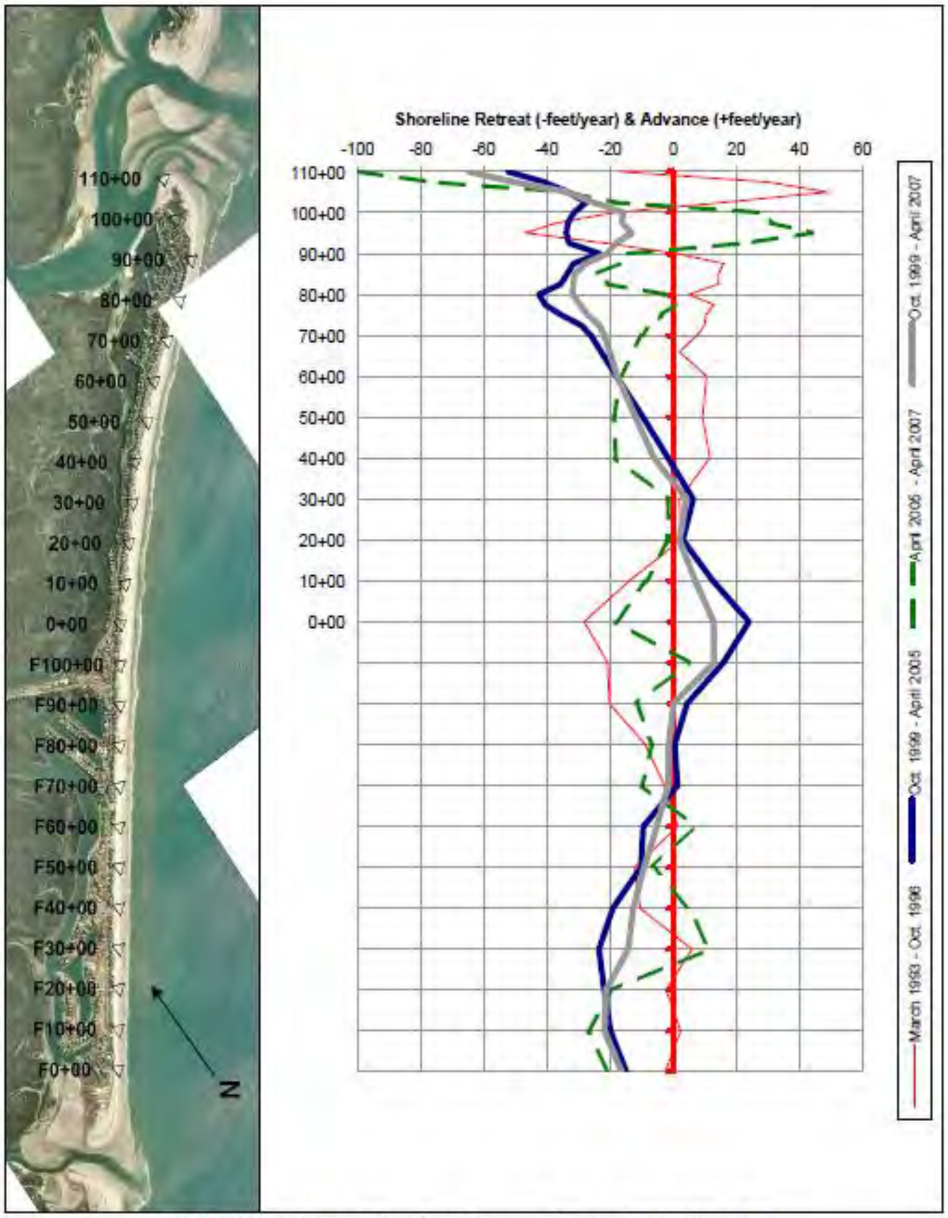


FIGURE 6-2: Ocean Shoreline Changes, Figure Eight Island, NC.

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TABLE 6-1

OCEAN SHORELINE CHANGES
(adjusted for beach fills)
FIGURE EIGHT ISLAND, NC

Profile Line	Beach Length (feet)	Shoreline Retreat (-feet/year) & Advance (+feet/year)				
		Mar 1993 to Oct 1996	Oct 1999 to Apr 2005	Apr 2005 to Apr 2007	Oct 1999 to Apr 2007	1999-2007 Worst Case
F0+00	500	-2	-14.8	-20.5	-16.3	-20.5
F10+00	1,000	2	-19.8	-27.0	-21.7	-27.0
F20+00	1,000	-2	-21.9	-19.4	-21.2	-21.9
F30+00	1,000	6	-23.5	11.3	-14.2	-23.5
F40+00	1,000	-10	-19.1	5.1	-12.7	-19.1
F50+00	1,000	-12	-10.2	-6.4	-9.2	-10.2
F60+00	1,000	1	-9.5	6.7	-5.1	-9.5
F70+00	1,000	-3	1.4	-9.8	-1.6	-9.8
F80+00	1,000	-9	0.5	-6.6	-1.4	-6.6
F90+00	1,000	-20	4.4	-11.6	0.2	-11.6
F100+00	1,000	-21	15.9	4.9	12.9	4.9
0+00	1,000	-28	24.1	-18.1	12.8	-18.1
10+00	1,000	-14	12.6	-8.5	7.0	-8.5
20+00	1,000	3	3.1	-1.4	1.9	-1.4
30+00	1,000	2	6.4	-1.7	4.2	-1.7
40+00	750	12	-1.6	-18.1	-6.0	-18.1
45+00	500	10	-5.5	-18.4	-9.0	-18.4
50+00	750	9	-9.5	-18.7	-12.0	-18.7
60+00	800	11	-17.9	-16.7	-17.6	-17.9
66+00	500	2	-22.8	-12.6	-20.1	-22.8
70+00	325	8	-26.0	-9.9	-21.7	-26.0
72+50	250	10	-29.2	-7.4	-23.4	-29.2
75+00	250	10	-35.5	-3.6	-27.0	-35.5
77+50	250	13	-40.5	0.9	-29.5	-40.5
80+00	250	5	-42.5	-2.5	-31.8	-42.5
82+50	250	14	-35.5	-20.9	-31.6	-35.5
85+00	250	14	-33.5	-24.5	-31.1	-33.5
87+50	250	16	-31.5	-17.0	-27.7	-31.5
90+00	250	1	-23.3	-14.1	-20.8	-23.3
92+50	250	-21	-32.9	22.6	-18.1	-32.9
95+00	250	-47	-33.9	44.0	-13.1	-33.9
97+50	250	-37	-33.3	30.9	-16.2	-33.3
100+00	250	-18	-31.4	27.4	-15.8	-31.4
102+50	250	16	-27.5	-18.6	-25.2	-27.5
105+00	250	49	-33.4	-33.8	-33.5	-33.8
107+50	250	30	-41.0	-77.3	-50.7	-77.3
110+00	125	-17	-52.3	-99.6	-64.9	-99.6
F0+00 to F90+00	9,000	-4	-11.9	-6.9	-10.6	-15.9
F90+00 to 45+00	6,500	-9	9.5	-7.5	5.0	-7.5
45+00 to 66+00	2,100	9	-14.1	-17.0	-14.9	-18.9
66+00 to 105+00	3,900	1	-32.0	-1.5	-23.9	-32.0

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TABLE 6-2

FIGURE EIGHT ISLAND BEACH FILLS 1993 - PRESENT

Project Date	Type of Project	Volume (c.y.)	Source	Profiles
Feb. 1993	Beach nourishment	274,000	Nixon Channel	60+00 to 105+00
January 1997	Storm recovery	Not avail.	Nixon Channel	15+00 to 105+00
March 1998	Channel dredging	450,000	Banks Channel & Middle Sound	INN15+00 to 90+00
March 1999	Beach nourishment	785,000	Banks Channel	INN15+00 to 87+50
March 2001	Beach nourishment	350,000	Nixon Channel	0+00 to 90+00
Jan.-Feb. 2002	Mason Inlet relocation	390,000	Mason Inlet	F0+00 to F100+00
March 2003	Channel dredging	50,000	Banks Channel & AIWW	INN10+00 to F14+00
March 2003	Sandbag placement*	30,000	Banks Channel & AIWW	80+00 to 97+50
Spring 2005	Channel dredging	183,000	Mason Inlet	F12+00 to F57+00
November 2005	Beach nourishment	261,235	Nixon Channel	30+00 to 95+00
April 2006	Beach nourishment	148,969	Mason Creek & AIWW	F-4+00 to F24+00
Spring 2006	Beach nourishment	179,175	Banks Channel	F24+00 to F80+00
Spring 2009	Channel Dredging	295,000	Nixon Channel	67+00 to 95+00
Spring 2009	Beach Nourishment	176,000	Mason Inlet	F-2+00 to F100+00
Jan-Mar 2011	Channel Dredging	275,000	Nixon Channel	0+00 to 95+00

Sources: All projects prior to 2005 - Cleary & Jackson (2004), Chapter 5.
Spring 2005 channel dredging - Gahagan & Bryant (2005).
November 2005 and subsequent projects - Gahagan & Bryant (2006).
* The 30,000 c.y. was placed outside the active beach profile and not incorporated in the shoreline retreat rates.

TABLE 6-3

OCEAN SHORELINE CHANGES
HUTAFF ISLAND, NC

Profile Line	Beach Length (feet)	Shoreline Retreat (-feet/year) & Advance (+feet/year) Mar 1993 to Oct 1996	Oct 1999 to Apr 2005	Apr 2005 to Apr 2007	Oct 1999 to Apr 2007
145+00	125	-5	-11.2	-35.6	-17.7
147+50	250	0	-4.6	-82.6	-25.4
150+00	250	-2	5.6	-109.0	-24.9
152+50	250	-3	8.5	-118.2	-25.3
155+00	250	-2	5.5	-102.1	-23.2
157+50	250	-9	10.8	-94.2	-17.2
160+00	250	-20	14.8	-82.4	-11.1
162+50	250	-26	16.2	-67.7	-6.2
165+00	250	-29	19.9	-52.7	0.5
167+50	250	-36	23.3	-40.3	6.3
170+00	250	-36	30.1	-30.5	14.0
172+50	250	-40	34.9	-35.8	16.1
175+00	125	-36	34.5	-31.9	16.8
145+00 to 175+00	3,000	-19	14.7	-70.8	-8.1

The erosional period on the north end of Figure Eight Island started in 1997. Since 1997, the main channel of Rich Inlet has moved towards its present location near Hutaff Island. However, in 1993, the main channel of the inlet was located closer to Figure Eight Island, as shown in Figure 5-4. Shoreline changes between 1993 and 1996 appear in Figure 6-2, Table 6-1, and Table 6-3. During this period, the northern half of Figure Eight Island (profiles 20+00 to 90+00 and 102+50 to 107+50) was accretional. The only erosion hotspot was located north of Inlet Hook Road (profiles 92+50 to 100+00). Conversely, the south end of Hutaff Island was erosional during this period. In general, a "comparison of the shoreline change data for Figure Eight Island and Hutaff Island for various periods since 1938 indicates that the updrift and downdrift barriers generally have opposing erosion/accretion trends. The major reversals in the accretion patterns and the onset of erosion are directly related to changes in the position of the ebb channel." (Cleary and Jackson, 2004, p. 146).

7.0 VOLUMETRIC CHANGE ANALYSIS

Volumetric changes along Figure Eight Island are based on the April 2005, April 2006, and April 2007 monitoring surveys by Gahagan & Bryant (2006, 2007). Available surveys prior to October 2004 were taken above wading depth (-4' NAVD) only, rendering them insufficient for a true volumetric change analysis.

Volume changes between April 2005 and April 2007 appear in Table 7-1 and Figure 7-1. Volume changes were computed using Beach Morphology Analysis Package Version 2.0 (BMAP, Sommerfeld, et al, 1994). The plotting routine within BMAP was utilized to evaluate the limits beyond which the apparent profile changes were dominated by survey error.

Between April 2005 and April 2007, Figure Eight Island gained 136,800 cubic yards (see Table 7-1, column 3). However, over 589,000 cubic yards of material was placed on the island (Table 6-2) between these dates. Without the beach fill, the island would have lost 452,900 cubic yards (see Table 7-1, column 5), equal to an average erosion rate of 10 c.y./year/foot. Most of the island was erosional between April 2005 and April 2007. Natural gains were limited to a few isolated areas near Bayberry Place (0+00), profiles 20+00 to 30+00, Surf Court (75+00), and Rich Inlet (105+00). The highest erosion rates occurred near Mason Inlet (INN15+00 to F20+00) and Inlet Hook Road (90+00). Moderate erosion occurred between profile 35+00 and Surf Court (70+00).

On the southern end of Hutaff Island (145+00 to 170+00), the beach lost 399,700 cubic yards. As noted earlier, this erosion was caused by Hurricane Ophelia (October 2005) and the formation of a swash channel into Rich Inlet. Based on a comparison of Tables 6-3 and 7-1, the 2005-2007 erosion patterns were not typical of the long term trend since 1999. Furthermore, they were considerably higher than the 1938-1998 erosion rates compiled by Cleary (2008).

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TABLE 7-1

OCEANFRONT VOLUME CHANGES
APRIL 2005 - APRIL 2007
FIGURE EIGHT & HUTAFF ISLAND, NC

Profile Line	Beach Length (feet)	April 2005 - April 2007 Volume Change (c.y.)			2005-2007 Volume Change Rates (c.y./year)
		Surveyed Changes	Beach Fills	Adjusted Changes	
INN15+00					
	100	-5,600	0	-5,600	-2,800
F-4+00					
	400	-16,400	9,300	-25,700	-12,850
F0+00					
	100	-2,700	5,300	-8,000	-4,000
F1+00					
	400	-8,000	23,400	-31,400	-15,700
F5+00					
	500	-3,200	29,200	-32,400	-16,200
F10+00					
	200	400	11,700	-11,300	-5,650
F12+00					
	200	900	11,700	-10,800	-5,400
F14+00					
	600	5,400	35,100	-29,700	-14,850
F20+00					
	400	4,800	23,400	-18,600	-9,300
F24+00					
	500	5,700	22,000	-16,300	-8,150
F29+00					
	100	1,100	2,900	-1,800	-900
F30+00					
	1,000	12,900	29,400	-16,500	-8,250
F40+00					
	1,000	18,400	29,400	-11,000	-5,500
F50+00					
	200	3,900	5,900	-2,000	-1,000
F57+00					
	800	6,800	23,500	-16,700	-8,350
F60+00					
	1,000	1,100	29,400	-28,300	-14,150
F70+00					
	1,000	6,000	29,400	-23,400	-11,700
F80+00					
	500	2,200	7,300	-5,100	-2,550
F85+00					
	500	-2,700	0	-2,700	-1,350
F90+00					

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TABLE 7-1

OCEANFRONT VOLUME CHANGES
APRIL 2005 - APRIL 2007
FIGURE EIGHT & HUTAFF ISLAND, NC

Profile Line	Beach Length (feet)	April 2005 - April 2007 Volume Change (c.y.) Surveyed Changes	Beach Fills	Adjusted Changes	2005-2007 Volume Change Rates (c.y./year)
	500	-3,900	0	-3,900	-1,950
F95+00					
	500	-1,300	0	-1,300	-650
F100+00					
	1,000	5,400	0	5,400	2,700
0+00					
	500	500	0	500	250
5+00					
	500	-9,100	0	-9,100	-4,550
10+00					
	1,000	-13,000	0	-13,000	-6,500
20+00					
	1,000	3,500	0	3,500	1,750
30+00					
	500	4,200	10,900	-6,700	-3,350
35+00					
	500	7,400	21,800	-14,400	-7,200
40+00					
	500	9,000	21,800	-12,800	-6,400
45+00					
	500	9,000	21,800	-12,800	-6,400
50+00					
	1,000	15,100	43,500	-28,400	-14,200
60+00					
	600	10,300	26,100	-15,800	-7,900
66+00					

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TABLE 7-1

OCEANFRONT VOLUME CHANGES
APRIL 2005 - APRIL 2007
FIGURE EIGHT & HUTAFF ISLAND, NC

Profile Line	Beach Length (feet)	April 2005 - April 2007 Volume Change (c.y.) Surveyed Changes	Beach Fills	Adjusted Changes	2005-2007 Volume Change Rates (c.y./year)
	400	10,100	17,400	-7,300	-3,650
70+00					
	250	8,800	10,900	-2,100	-1,050
72+50					
	250	12,300	10,900	1,400	700
75+00					
	250	13,200	10,900	2,300	1,150
77+50					
	250	11,400	10,900	500	250
80+00					
	250	8,600	10,900	-2,300	-1,150
82+50					
	250	4,900	10,900	-6,000	-3,000
85+00					
	250	800	10,900	-10,100	-5,050
87+50					
	250	-3,600	10,900	-14,500	-7,250
90+00					
	250	-7,300	8,200	-15,500	-7,750
92+50					
	250	-10,400	2,700	-13,100	-6,550
95+00					
	250	-8,000	0	-8,000	-4,000
97+50					
	250	-200	0	-200	-100
100+00					
	250	6,000	0	6,000	3,000
102+50					
	250	10,600	0	10,600	5,300
105+00					
	250	9,300	0	9,300	4,650
107+50					
	250	2,200	0	2,200	1,100
110+00					

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TABLE 7-1

OCEANFRONT VOLUME CHANGES
APRIL 2005 - APRIL 2007
FIGURE EIGHT & HUTAFF ISLAND, NC

Profile Line	Beach Length (feet)	April 2005 - April 2007 Volume Change (c.y.)			2005-2007 Volume Change Rates (c.y./year)
		Surveyed Changes	Beach Fills	Adjusted Changes	
INN15+00 to F0+00	500	-22,000	9,300	-31,300	-15,650
F0+00 to F90+00	9,000	53,000	319,000	-266,000	-133,000
F90+00 to 45+00	6,500	2,700	54,500	-51,800	-25,900
45+00 to 66+00	2,100	34,400	91,400	-57,000	-28,500
66+00 to 105+00	3,900	57,200	115,500	-58,300	-29,150
105+00 to 110+00	500	11,500	0	11,500	5,750
FIGURE 8 ISLAND INN15+00 to 110+00	22,500	136,800	589,700	-452,900	-226,450

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TABLE 7-1

OCEANFRONT VOLUME CHANGES
APRIL 2005 - APRIL 2007
FIGURE EIGHT & HUTAFF ISLAND, NC

Profile Line	Beach Length (feet)	April 2005 - April 2007 Volume Change (c.y.) Surveyed Changes	Beach Fills	Adjusted Changes	2005-2007 Volume Change Rates (c.y./year)
145+00					
	250	-26,800	0	-26,800	-13,400
147+50					
	250	-30,500	0	-30,500	-15,250
150+00					
	250	-34,100	0	-34,100	-17,050
152+50					
	250	-37,800	0	-37,800	-18,900
155+00					
	250	-39,500	0	-39,500	-19,750
157+50					
	250	-39,100	0	-39,100	-19,550
160+00					
	250	-38,700	0	-38,700	-19,350
162+50					
	250	-38,400	0	-38,400	-19,200
165+00					
	250	-35,800	0	-35,800	-17,900
167+50					
	250	-31,100	0	-31,100	-15,550
170+00					
	250	-26,300	0	-26,300	-13,150
172+50					
	250	-21,600	0	-21,600	-10,800
175+00					
SOUTHERN HUTAFF IS. 145+00 to 175+00	3,000	-399,700	0	-399,700	-199,850

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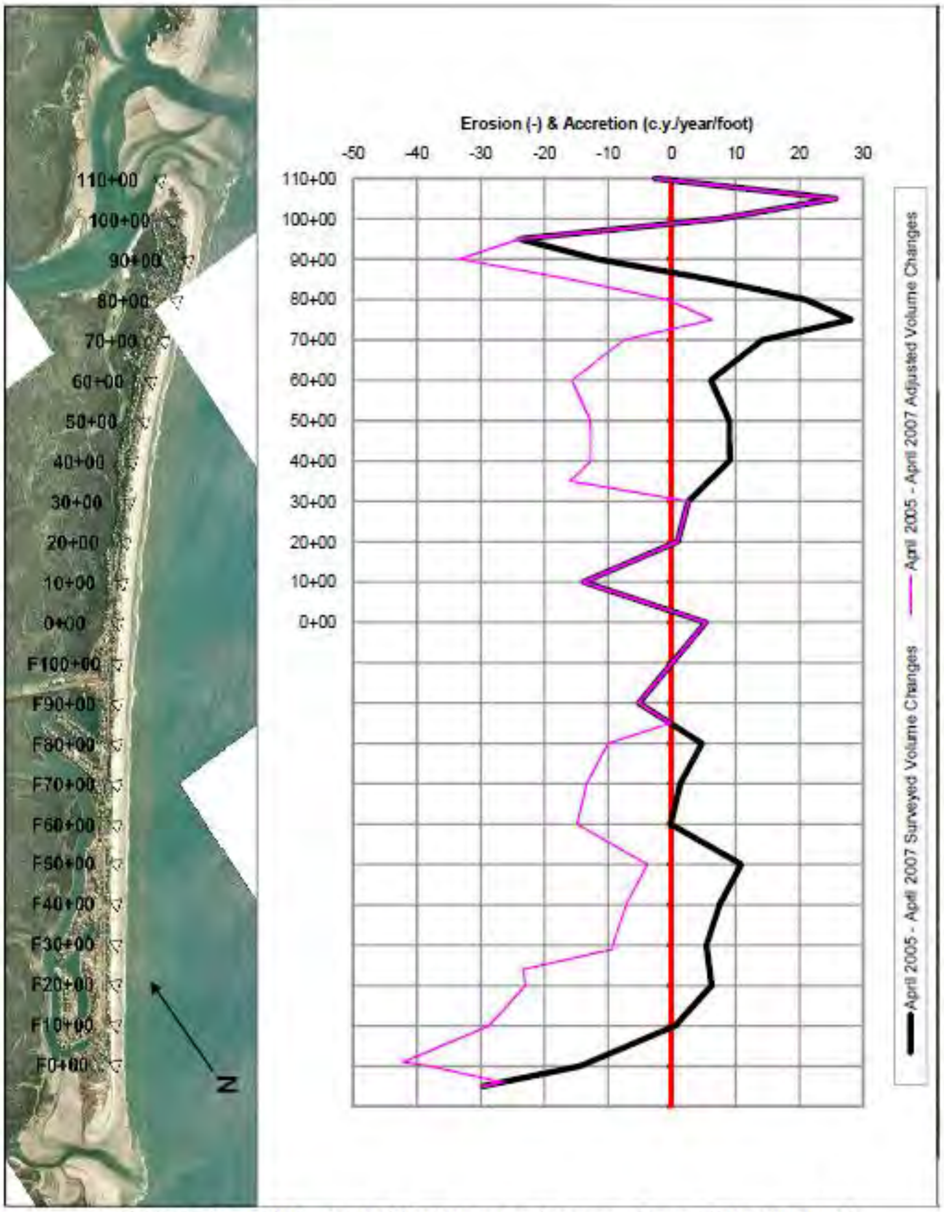

FIGURE 7-1: April 2005 - April 2007 Volumetric Changes, Figure Eight Island, NC.

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8.0 LITTORAL BUDGET

8.1 April 2005 – April 2007 Sediment Budget

Based on the volumetric changes in the previous section, two sediment budgets were developed to map the movement of material along Figure Eight Island and Rich Inlet: April 2005-April 2007 and October 1999-April 2007. For the shorter time period, changes on the oceanfront beaches were based on the erosion rates appearing in Figure 7-1 and Table 7-1. These changes were dominated by Hurricane Ophelia (September 2005) and beach nourishment operations on the northern and southern ends of the island. Volumetric changes near Rich Inlet were based on the April 2005, April 2006, and April 2007 surveys (Figures 8-1 to 8-3). To map the movement of material in Rich Inlet, the inlet and ebb shoal complex was divided into the following cells, which appear in Figure Eight-1:

- Outer Ebb Shoal.
- Existing Channel.
- Southwest Flood Channels.
- Inlet Interior.

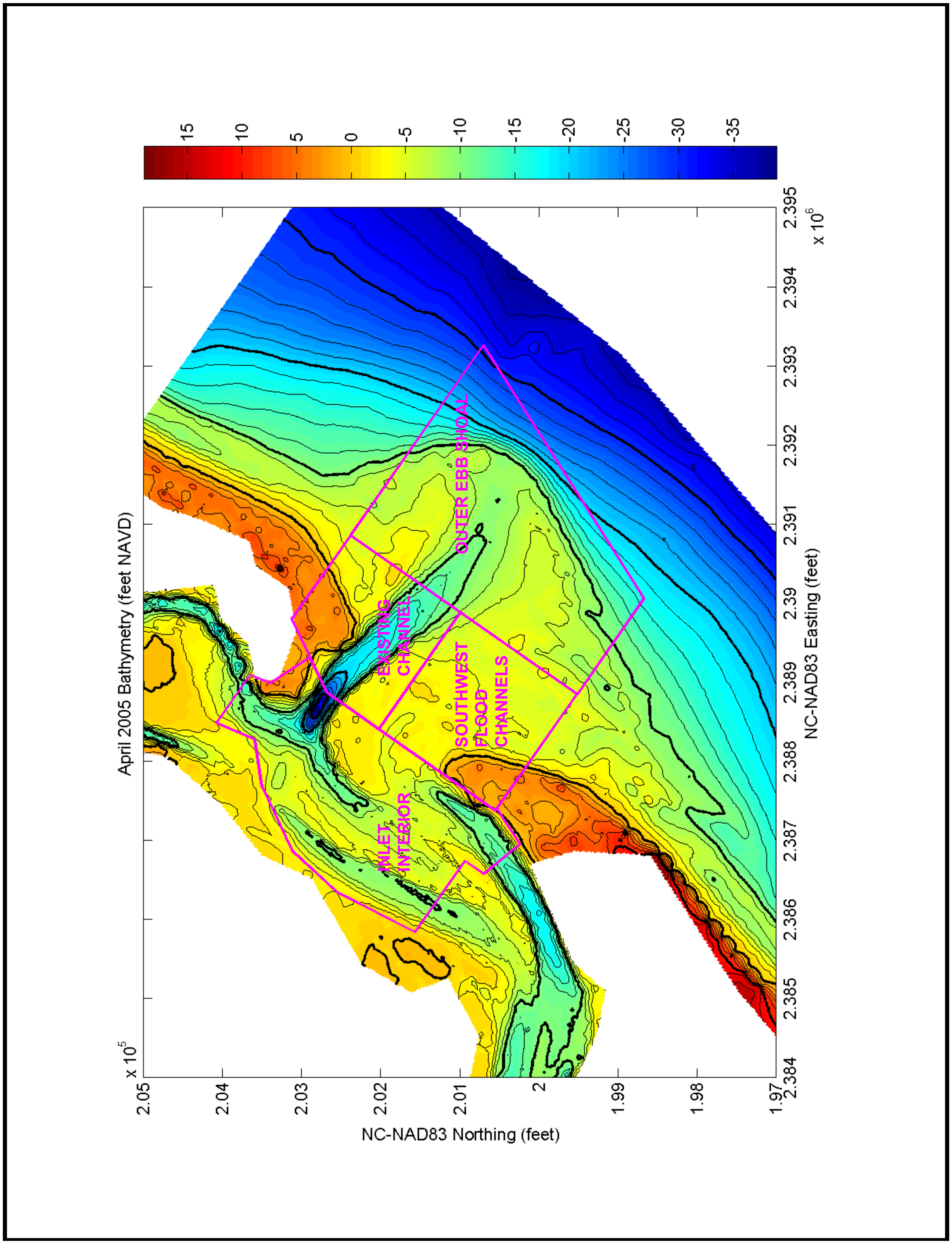


FIGURE 8-1: April 2005 Bathymetry, Figure Eight Island and Rich Inlet, NC.

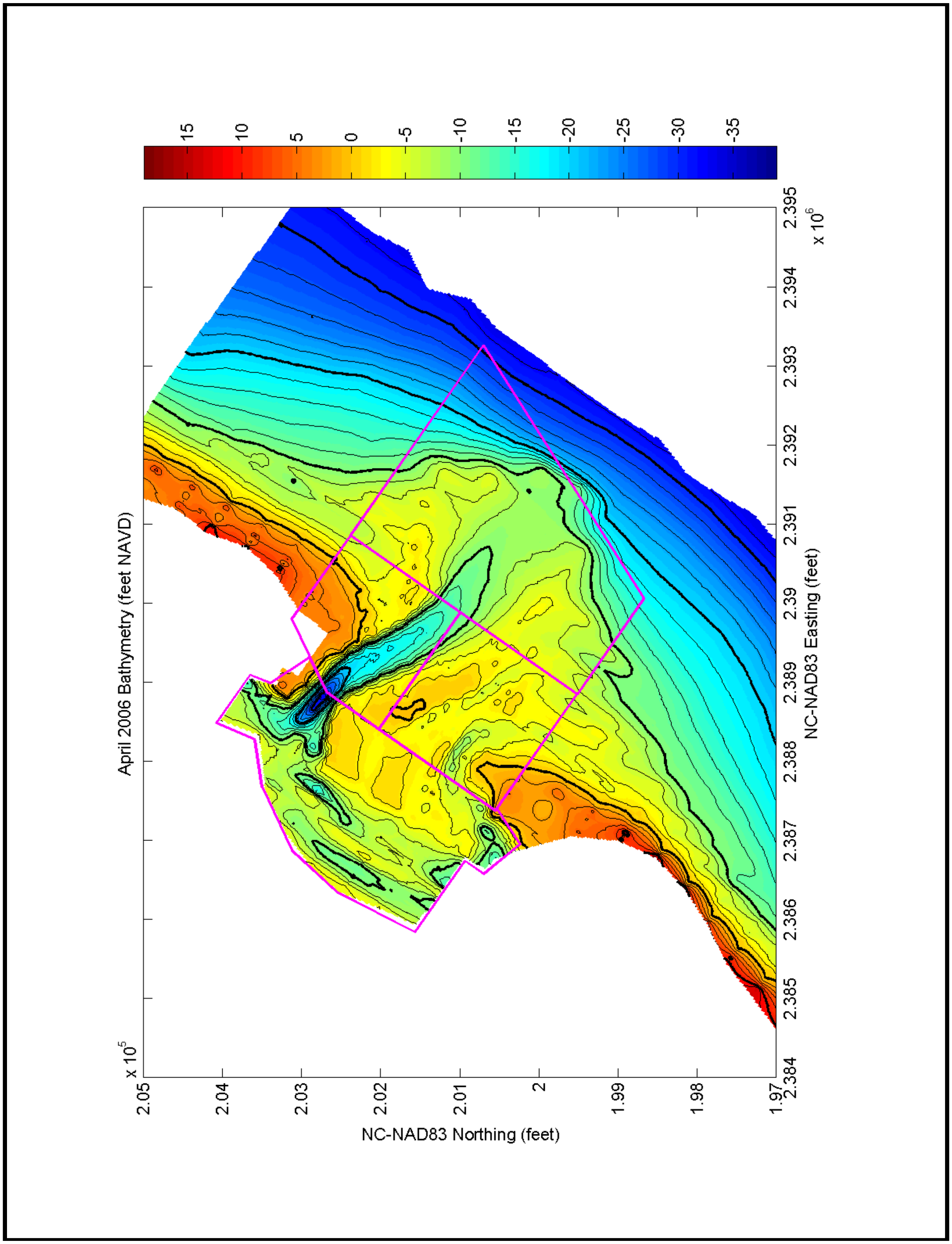


FIGURE 8-2: April 2006 Bathymetry, Figure Eight Island and Rich Inlet, NC.

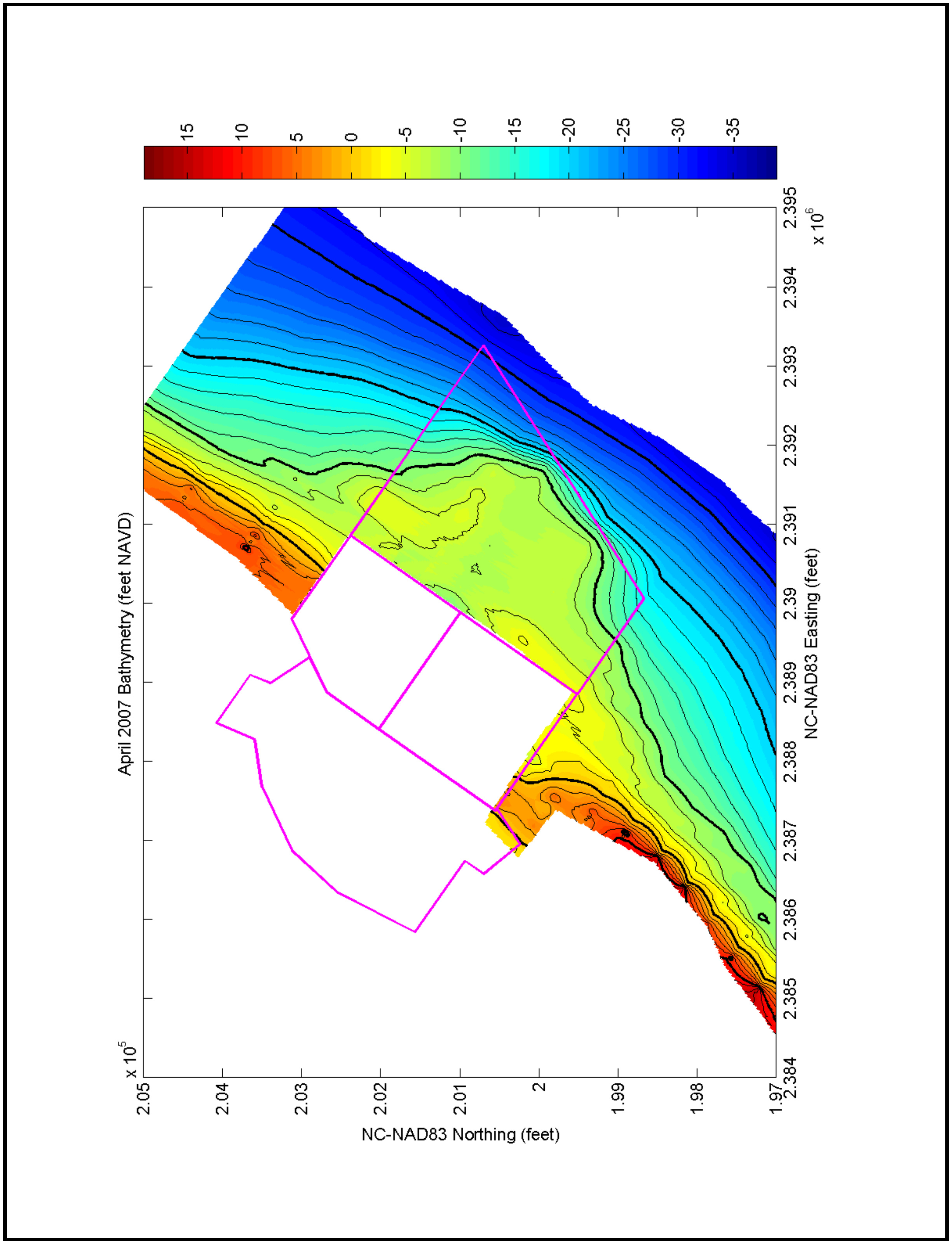


FIGURE 8-3: April 2007 Bathymetry, Figure Eight Island and Rich Inlet, NC.

The locations of these cells were based on the morphology of the inlet and the limits of the 2005, 2006, and 2007 surveys. Changes within the Outer Ebb Shoal were based on the April 2005 and April 2007 surveys. In the other inlet cells, the April 2007 survey did not provide sufficient coverage or spacing to realistically depict the bathymetry. Accordingly, changes in the other 3 inlet cells were based on the April 2005 and April 2006 surveys.

Sediment budget cells along the beach were based on the proposed beach fill layouts, discussed later in this report. South of the beach disposal area, additional cells were delineated based on the available survey data. Oceanfront sediment budget cells are listed in Table 8-1:

TABLE 8-1

OCEANFRONT SEDIMENT BUDGET CELLS
FIGURE EIGHT ISLAND, NC

Profile Lines	Beach Length (feet)	Description
INN10+00 to INN15+00	500	Undeveloped beach near Mason Inlet (1999-2007 only)
INN15+00 to F0+00	500	188 Beach Road S to 184 Beach Road S (wide lots)
F0+00 to F90+00	9,000	184 Beach Road S to 8 Beach Road S
F90+00 to 45+00	6,500	8 Beach Road S to 292 Beach Road N
45+00 to 66+00	2,100	292 Beach Road N to Surf Court
66+00 to 105+00	3,900	Surf Court to Inlet Hook Roads (Rich Inlet erosion hotspot)
105+00 to 110+00	500	Undeveloped beach near Rich Inlet
145+00 to 175+00	3,000	Southern Hutaff Island

Transport rates between the various cells in Rich Inlet were generally based on preliminary Delft3D model results between April 2005 and April 2007. Transport rates on Hutaff Island were then determined based on the observed volume changes (Table 7-1) and the amount of material entering Rich Inlet. Transport rates on Figure Eight Island were determined based on the volumetric changes in Figure 7-1. Between 2005 and 2007, a high erosion area was centered near profile 95+00 (Inlet Hook Road). Accreting areas were located on either side of this erosion hotspot, suggesting the presence of a nodal point, or the transport of material away from profile 95+00 in either direction. Based on the other observed volume changes and fill quantities on the island, transport rates along the remainder of the island were estimated.

The April 2005 – April 2007 sediment budget appears in Figure 8-4. Over the 2 year period, the south end of Hutaff Island lost 199,850 c.y./year. Most of this material went into the Rich Inlet complex, which gained 182,000 c.y./year. Within the inlet complex, the Existing Channel was the primary pathway for offshore transport of sediment, and the Southwest Flood Channels were the primary pathway for the inland transport of sediment.

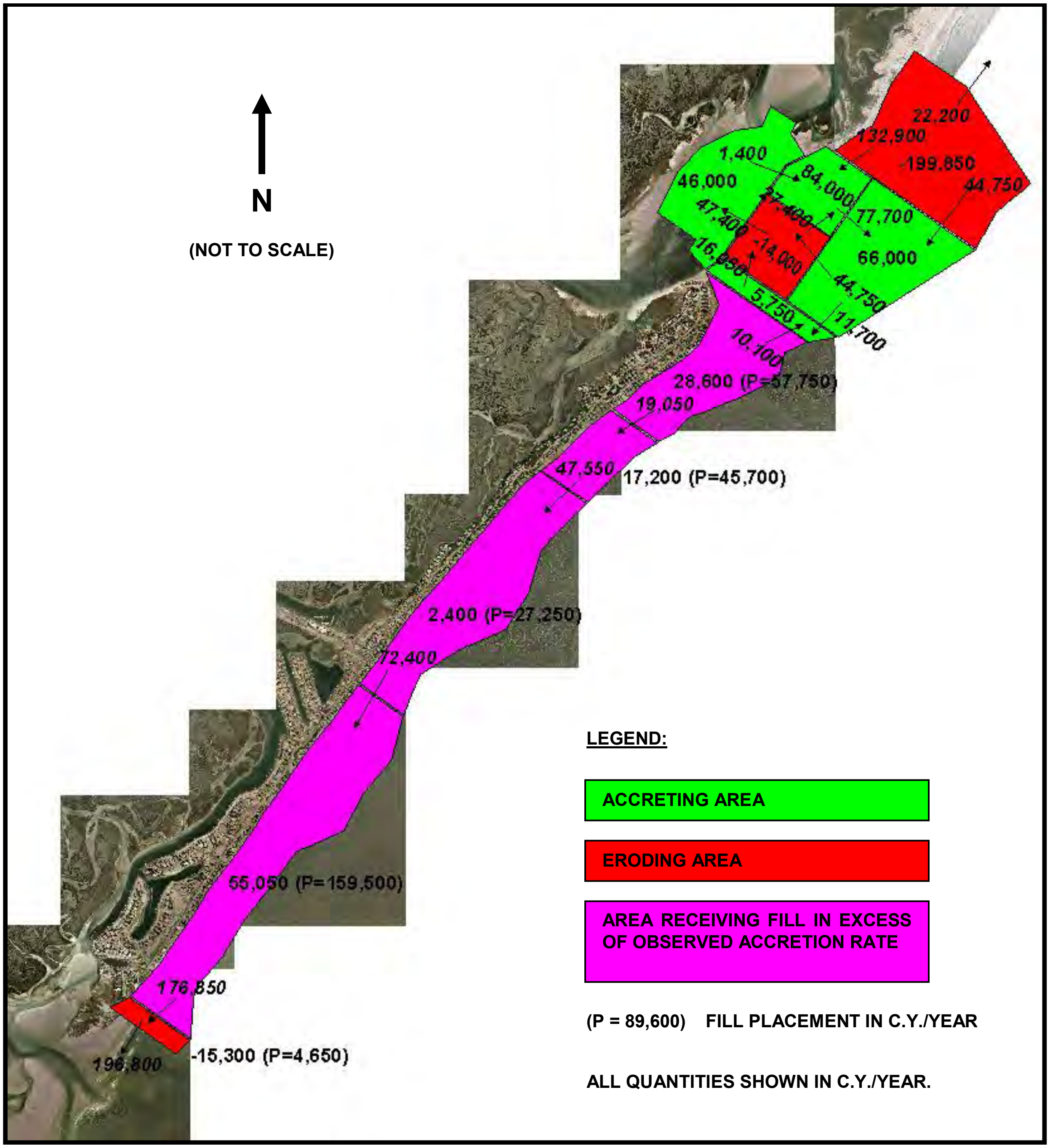


FIGURE 8-4: Figure Eight Island April 2005 – April 2007 Sediment Budget.

Along Figure Eight Island, the net transport was towards the south. Between profile 95+00 (Inlet Hook Road) and F-4+00 (south end of Beach Road), there was a consistent increase in the sediment transport rate from 0 to 196,800 c.y./year.

8.2 October 1999 – April 2007 Sediment Budget

For the longer time period, changes on the oceanfront beaches were based on the 1999-2007 shoreline changes. A detailed bathymetric survey of Rich Inlet prior to 2004 was not available. Accordingly, the inlet and ebb shoal was schematized as a single cell, with volumetric changes estimated based on sediment transport along the adjacent beaches.

The October 1999 – April 2007 sediment budget appears in Figure 8-5. During the 7½ year period, the highest rates of retreat occurred near profiles 80+00 (Comber Road) and 110+00 (Rich Inlet) (Figure 6-2). Accordingly, profile 80+00 (Comber Road) was assumed to be a nodal point, with transport of material away from the area in either direction. Given the observed shoreline changes and beach fills (Table 6-2), the estimated sediment transport was 63,200 c.y./year to northeast at profile 105+00 and 37,100 c.y./year to the southwest at profile 66+00 (Surf Court). Based on the other observed changes and fill quantities on the island, sediment transport rates along the remainder of the island were estimated.

South of profile 66+00 (Surf Court), the net sediment transport was from northeast to southwest. Between Backfin Point (F80+00) and 268 Beach Road North (35+00), there was an accreting area characterized by a decreasing rate of sediment transport. However, the direction of sediment transport was towards the southwest along this reach. South of Backfin Point (F80+00), the beaches were erosional, with an increasing rate of sediment transport towards the southwest.

The net sediment transport near Mason Inlet (INN10+00) was less than the 2005-2007 sediment budget. However, it was consistent with the migration pattern of Mason Inlet prior to 2002, which moved 2,200 feet southwest between 1985 and 2002 (Erickson, Kraus, and Carr, 2003), or approximately 129 feet/year. Based on the inlet migration rate, a +6 foot NAVD berm elevation, a -24 foot NAVD depth of closure, and a cross-shore width of 900 feet, the equivalent sediment transport would be 129,000 c.y./year. This value was close to the sediment transport rate of 142,900 c.y./year in Figure 8-5.

On the south end of Hutaff Island, the net transport rates between 1999 and 2007 were low. Transport rates at profile 175+00 were based on preliminary Delft3D model results for the 5-year, without-project scenario. Transport rates into Rich Inlet were then determined based on the observed shoreline changes between 1999 and 2007. Given the transport rates on either side of Rich Inlet, the inlet and ebb shoal gained approximately 120,600 c.y./year between October 1999 and April 2007. While the gain was 2/3 the combined value shown in Figure 8-4, it was based on erosion rates that were more representative of the study area than the 2005-2007 rates.

8.3 Summary

Based on the two sediment budgets, Rich Inlet is a sediment sink that gains 100,000 to 200,000 c.y./year. The source of this material alternates between the adjacent beaches on Figure Eight Island and the adjacent beaches on Hutaff Island. The recent source is primarily Hutaff Island.

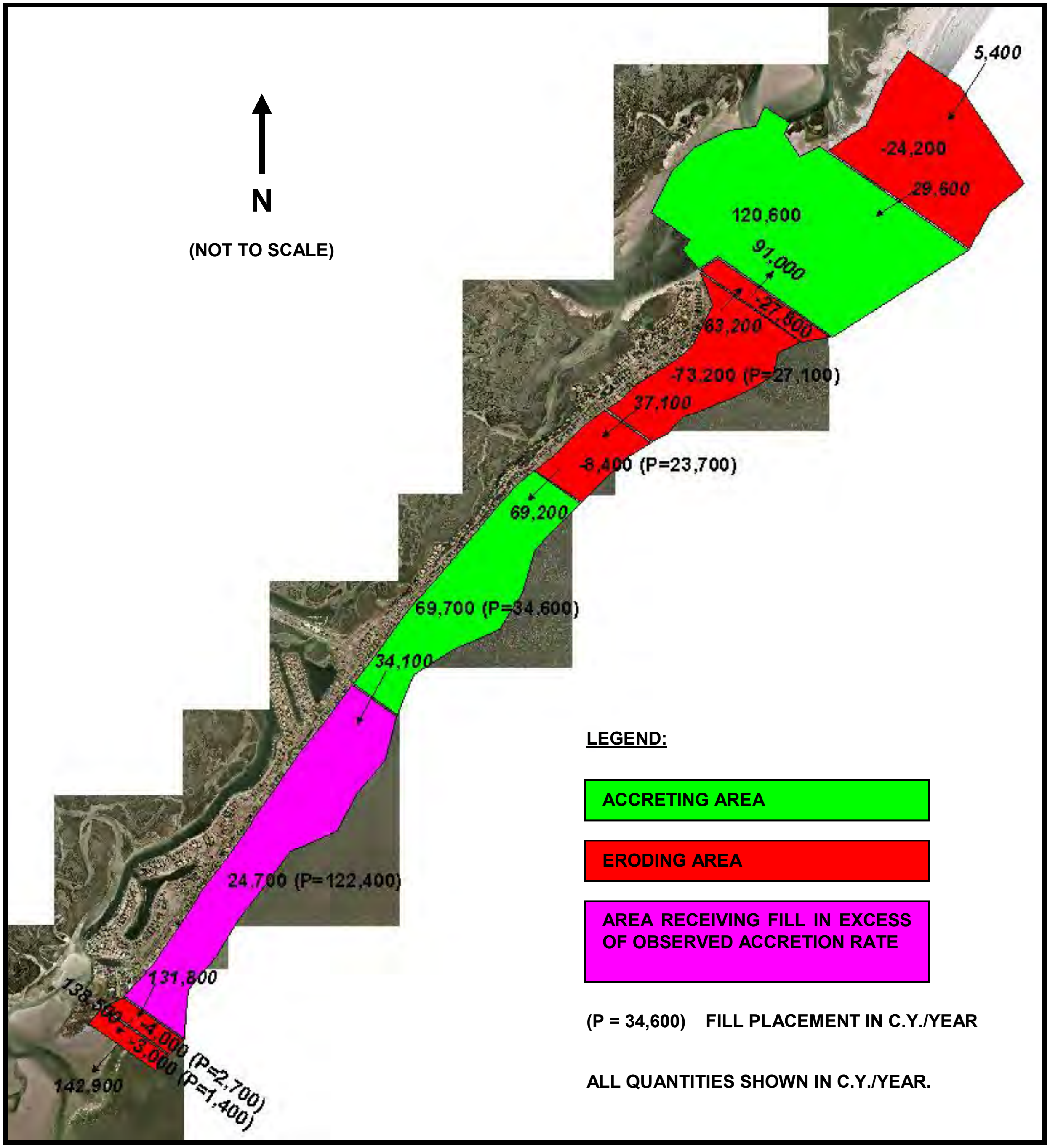


FIGURE 8-5: Figure Eight Island October 1999 – April 2007 Sediment Budget.

Near the northern end of Figure Eight Island, there is a nodal point, at which eroding sediments spread towards both the northeast and the southwest. This nodal point has shifted towards the northeast since 1999, but currently lies near Inlet Hook Road (profile 95+00). Along the rest of Figure Eight Island, the predominant sediment transport is towards the southwest. Sediment transport rates just north of Mason Inlet (profile F-4+00) vary from 142,900 to 196,800 c.y./year. Given the general erosion patterns around Rich Inlet, the northeasterly sediment transport on Topsail Island (USACE, 2006, p. 31), and the southwesterly transport near Mason Inlet, the area surrounding Rich Inlet functions as a nodal point on regional basis.

9.0 PROJECT DESIGN

The main text of the Environmental Impact Statement presents the following alternatives to address chronic erosion on Figure Eight Island:

1. Alternative 1 - No Action.

2. Alternative 2 - Abandon/Retreat.

3. Alternative 3 - Rich Inlet Management and Beach Fill.

4. Alternative 4 - Beach Fill without Management of Rich Inlet.

5. Terminal Groin Options:
    - Alternative 5C - Terminal Groin at a More Northerly Location with Beach Fill from Nixon Channel and a New Channel Connecting to Gorge of Rich Inlet

    - Alternative 5D - Terminal Groin at a More Northerly Location with Beach Fill from the Previously Permitted Area in Nixon Channel and Other Sources

The designation of the terminal groin alternatives as 5C and 5D is the result of changes made in the original terminal groin proposal presented in the Draft Environmental Impact Statement (DEIS) issued in January 2012. The terminal groin alternatives presented in the DEIS were designated as 5A and 5B and had the terminal groin positioned relatively close to the northern most homes on the north end of Figure Eight Island. During the review process for the DEIS, several of the property owners expressed concerns of the potential negative aesthetics of the structure and the possible impact on public access to the extreme north end next to Rich Inlet. As a result, the Figure "8" Beach HOA agreed to consider moving the terminal groin 420 feet north of the DEIS position. The new position of the terminal groin resulted in a new round of model investigations to evaluate the potential impacts of the new location relative to the impacts associated with the DEIS location.

Alternative 5C essentially replaces 5A in the DEIS since it would involve constructing the beach fill along the ocean shoreline and the shoreline of Nixon Channel using material obtained from maintenance of the navigation channel in Nixon Channel (previously permitted area) and a new channel connecting Nixon Channel with the gorge of Rich Inlet. For Alternative 5C, the beach fill along the ocean shoreline is slightly longer than the beach fill for 5A given the more northerly position of the terminal groin. In like manner, Alternative 5D replaces 5B presented in the DEIS. For Alternative 5D, the terminal groin is also positioned farther north and the material to construct the beach fills along the ocean shoreline and the Nixon Channel shoreline would be derived from maintenance of the previously permitted area in Nixon Channel.

The presentation of the evaluation of the terminal groin alternatives presented below is limited to the evaluation of Alternatives 5C and 5D. Model results for these alternatives as well as Alternatives 2, 3, and 4 are provided in Sub-Appendix B-1 for the 2006 initial conditions and Sub-Appendix B-2 for the 2012 initial conditions. The results of preliminary Delft3D model test performed for the DEIS using the 2006 initial conditions are provided in Sub-Appendix B. In Sub-B the nomenclature used for some of the Alternatives differ from the ones presented in this document. For example, the Alternatives designated as 5a-1, 5a-2, etc. were variations of Alternative 5A presented in the DEIS.

The evaluation of the relative impacts of the various alternatives on Rich Inlet and the adjacent shorelines was based on the results obtained from the Delft3D model. The Delft3D model is discussed in detail below. The initial conditions used in the formulation and evaluation of the alternatives was the 2006-07 conditions of Rich Inlet and the adjacent shoreline as these conditions represent the "worst case" conditions with respect to shoreline changes along the north end of Figure Eight Island. The Delft3D model was also run using conditions that existed in March 2012 as the initial condition. Simulations using the 2012 initial conditions were limited to Alternatives 2, 3, 4, and 5D.

Alternative 1 assumes that the present strategies to manage the island's shoreline in Table 6-2 will continue into the future. Alternative 2 assumes that there will be no more beach fill, dune maintenance, inlet maintenance, or sand bag placement operations. Accordingly, this alternative is the true "Without-Project" scenario. Alternative 3 implements the recommended modification of the Rich Inlet ocean bar channel proposed by Cleary (Sub-Appendix A – Rich Inlet Update), which is further detailed in this section. Dredged material from the inlet modification would be strategically placed along the north half of the island to mitigate for the erosion occurring since the late 1990s. Alternative 4 has a beach fill similar to Alternative 3 with the fill material to be taken from an offshore source as well as from maintenance of the existing navigation channel in Nixon Channel. In this regard, potential offshore sand sources have been identified by Cleary (Cleary, 2000) but have not been investigated in detail. Alternative 5 utilizes a terminal groin to create an accretion fillet on the extreme north end of Figure Eight Island and reduce erosion rates from the beach fill placed north of Bridge Road to Rich Inlet. This alternative includes beach fill material from maintenance of the previously permitted area in Nixon Channel and a new channel connector between Nixon Channel and the inlet gorge (Alternative 5C) and fill from maintenance of the previously permitted area in Nixon Channel (Alternative 5D).

9.1 Alternative 3 – Rich Inlet Management and Beach Fill

9.1.1 Channel Location

Many of the erosion problems on the northern half of Figure Eight Island are to due changes in the location and alignment of the ebb shoal and main entrance channel at Rich Inlet. Based on thorough analysis of inlet characteristics between 1938 and 2001, reported by Cleary and Jackson (2004) and an update of that analysis that includes changes between 2001 and 2007 prepared by Dr. Cleary for this report, which is provided in Sub-Appendix A, a recommended optimum channel location was developed which is shown in Figure 9-1. This channel is located in the middle of the inlet approximately 2600 feet northeast of N. Beach Road (536 block).

Based on the trends observed by Cleary and Jackson (2004) and more recently by Cleary (Sub-Appendix A), relocating the channel will also shift the ebb shoal, providing a buffer against wave-driven erosion. As noted in Section 6, the south end of Hutaff Island is eroding partly due to the formation of a swash channel. The formation of the swash channel has partially depleted the ebb shoal on the north side of Rich Inlet. On the other hand, when the north side of the ebb shoal is fully intact, the south end of Hutaff Island accretes. Given these observations, relocating the channel as shown is Figure 9-1 is a possible means of controlling erosion on the north end of Figure Eight Island without using structures.

9.1.2 Closure Dike

To ensure a successful relocation of the channel, it is necessary to close the existing channel. This task will be accomplished by building a closure dike out of the material dredged from the relocated channel. The Delft3D modeling results in a later section of this report show that without a dike, the existing channel will continue to carry the flow through Rich Inlet. The modeling results also show that the dike must be of sufficient size to remain in place for more than a few months. The closure dike at Rich Inlet will have the following dimensions:

Crest Elevation = +6 feet NAVD
Crest Width = 450 feet
Side Slopes ~ 1 vertical on 20 horizontal

9.1.3 Entrance Channel Dimensions

To establish dimensions of the ebb/entrance channel, an inlet stability analysis has been conducted. The inlet stability analysis utilizes two curves (Figure 9-2): the O'Brien curve and the Escoffier curve. The O'Brien curve is an empirical relationship between tidal prism and the cross-sectional area at the throat of the inlet. The Escoffier curve is a theoretical relationship between the tidal current velocity and the cross-sectional area. Currents at the inlet throat were measured by Gahagan & Bryant Associates, Inc. in June 2005. The most recent survey of the inlet throat was taken by Gahagan & Bryant Associates, Inc. in April 2006. As shown in Figure 9-2, the observed flood currents and cross-sectional area fall on the Escoffier curve.

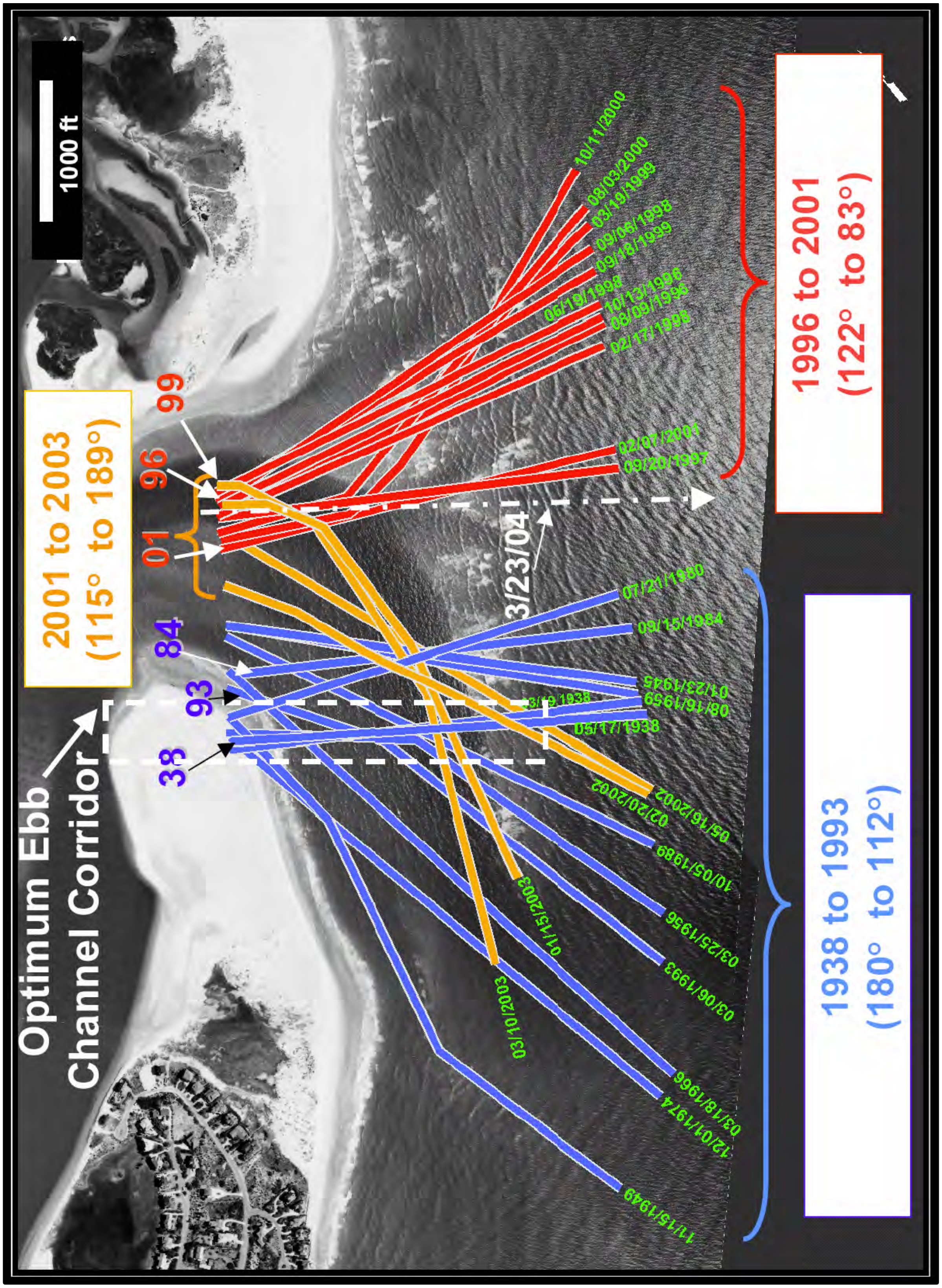


FIGURE 9-1: Rich Inlet Optimum Channel Location (Cleary and Jackson, 2004).

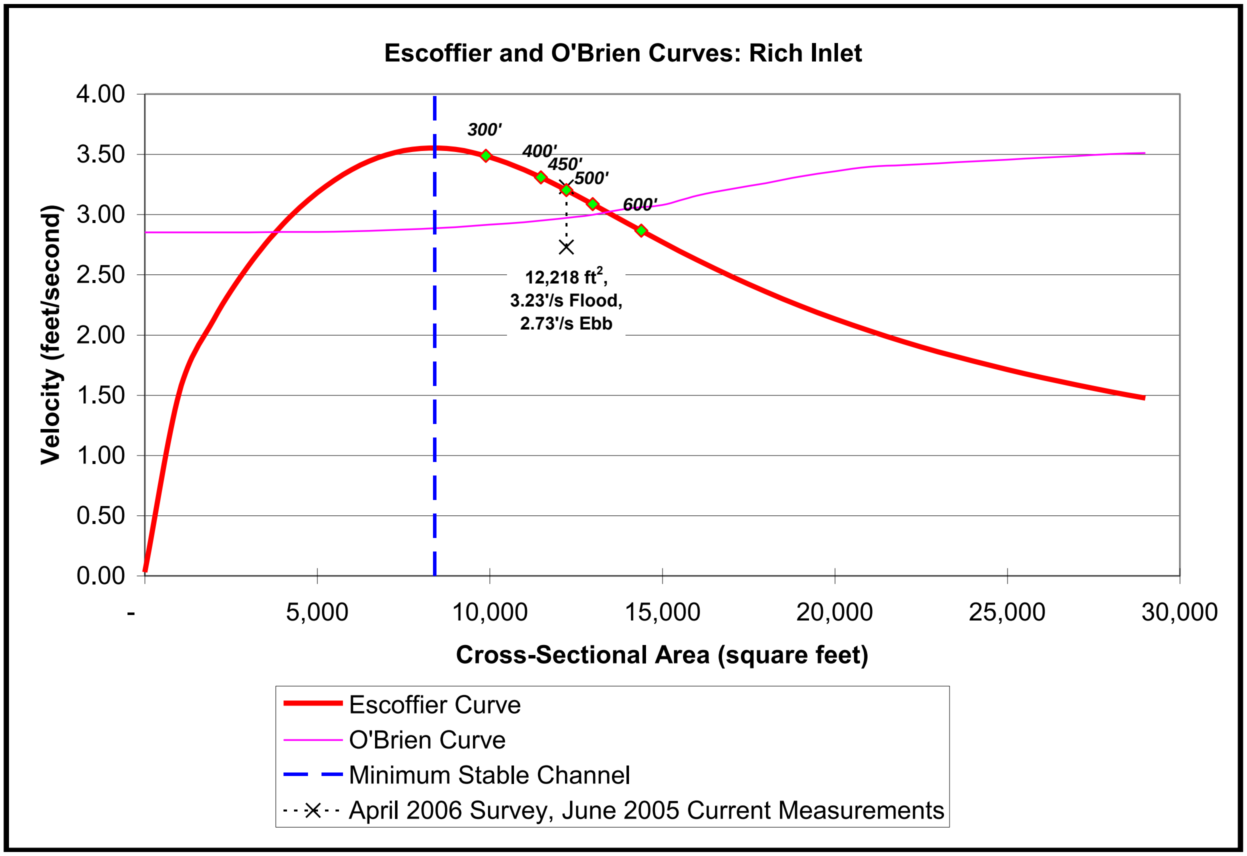


FIGURE 9-2: Inlet Stability Curves for Rich with Bottom Widths Given a Design Depth of -19 feet NAVD and Side Slopes of 1V:5H.

The O'Brien curve crosses the Escoffier curve at two points. The left point is the unstable equilibrium, which corresponds to a cross-sectional area of 3,800 square feet. Any deviation from that point immediately sets into action forces which tend to further increase or aggravate the deviation (Escoffier, 1940). If the deviation is a reduction in the cross-sectional area, the inlet closes. The right point is the stable equilibrium, which corresponds to a cross-sectional area of 13,400 square feet. Any deviation from that point immediately sets into action forces which tend to restore the channel to its initial condition (Escoffier, 1940). Between the two crossing points, the Escoffier curve peaks at a cross-sectional area of 8,400 square feet. This value represents the minimum cross-sectional area for the inlet to remain stable.

The initial designs and preliminary model simulations for Rich Inlet assumed a design depth of -17 feet NAVD. However, based on conversations with dredge contractors, a design depth of -19 feet NAVD was found to be easier and less expensive to construct. Thus, the design depth was modified to -19 feet NAVD, with side slopes of 1 vertical on 5 horizontal.

The closure dike will reduce the cross-sectional area by 8,600 square feet (Figure 9-3) which would reduce the cross-sectional area of the inlet to approximately 3,600 square feet. Based on the stability analysis presented above, this would result in an unstable inlet. For the inlet to remain stable, its cross-sectional area needs to be at least 8,400 square feet. This can be accomplished using a design cross-section with a bottom width of 300 feet. However, the 300 foot bottom width does not offer an appropriate safety factor. Furthermore, it does not restore the cross-section to present size. A bottom width of 500 to 600 feet achieves the stable equilibrium of 13,400 square feet. However, this size is not the most cost-effective, and creates a larger project footprint. A bottom width of 450 feet was selected and restores the cross-sectional to its present size. Natural forces can then be allowed to increase the cross-section.

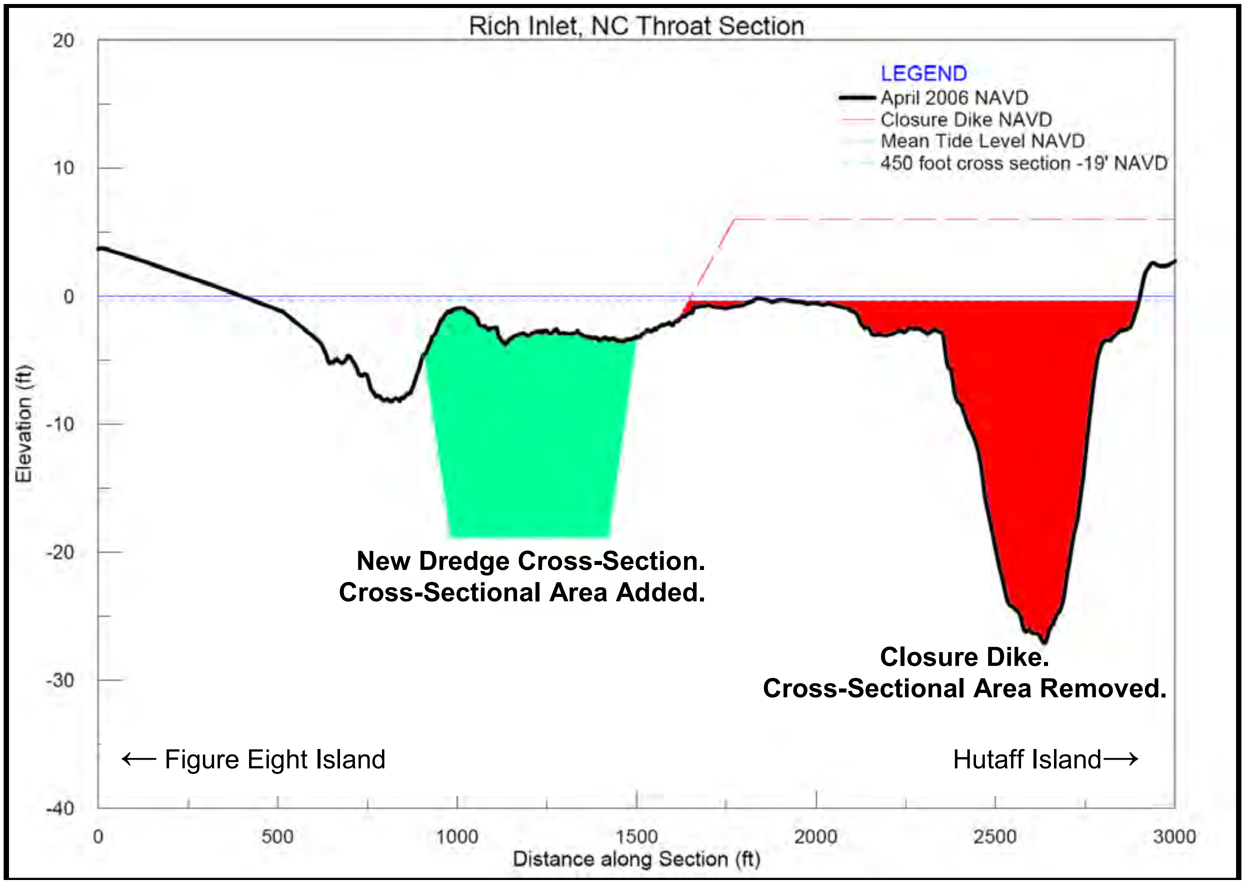


FIGURE 9-3: Cross-Section of the Inlet Throat.

9.1.4 Side Channels

Flow through Rich Inlet is carried into the Atlantic Intracoastal Waterway (AIWW) primarily through Nixon Channel and Green Channel with some flow migrating through the salt marsh area immediately north of the inlet. Nixon Channel lies to the south of the entrance channel and runs from east to west. Green Channel lies north of the entrance channel and runs from south to north. To ensure a successful relocation of the entrance channel, it is necessary to dredge connecting cuts from the entrance channel in Nixon Channel and Green Channel.

9.1.4.1 Dredging Option 1

Dredging Option 1 appears in Figure 9-4, and features an entrance channel through the middle of Rich Inlet, a connecting cut into Nixon Channel, a connecting cut into Green Channel, and a narrow extension of entrance channel towards the salt marsh bounded by Nixon Channel, Green Channel, and the Intracoastal Waterway. Although this Dredging Option provides connecting cuts in Nixon Channel and Green Channel, extension of the entrance channel is not necessary to maintain adequate flow through Nixon Channel and Green Channel. Cleary (2008) has noted that the salt marsh facing the entrance of the main channel of Rich Inlet has been eroding. Preliminary Delft3D model results have shown that much of the flow going through Green Channel is directed to and from the entrance channel through the entrance channel extension instead of the Green Channel connecting cut. This could worsen the erosion of the salt marsh and could make the Green Channel connecting cut more difficult to maintain. Finally, the extension of the entrance channel increases the project footprint and the area impacted during construction, with few added benefits. For these reasons, Dredging Option 1 has been dropped from consideration.

9.1.4.2 Dredging Option 2

Dredging Options 2A and 2B (Figure 9-5) dredge a new entrance channel through the middle of Rich Inlet. The new entrance channel is located midway between Figure Eight Island and Hutaff Island approximately 1,300 feet southwest of the existing (April 2006) channel. The length of the cut is 3,500 feet, and the bottom width is 500 feet given the old design depth of -17 feet NAVD. The new entrance channel runs along a bearing of 142° / 322° (northwest-southeast). At the northern end of the entrance channel, the dredge cut splits into two smaller channels connecting into Nixon Channel and Green Channel. The connection into Nixon Channel runs on a bearing of 64° / 244° (west-southwest to east-northeast) and has a bottom width of 275 feet given the old design depth of -17 feet NAVD. The connection into Green Channel runs on a bearing of 14° / 194° (north-northeast to south-southwest) and has a bottom width of 225 feet given the old design depth of -17 feet NAVD. Under Dredging Option 2A, the connections to Nixon Channel and Green Channel are 3,800 and 2,000 feet long, respectively. Under the shorter Dredging Option 2B, the connections to Nixon Channel and Green Channel are 1,700 and 1,400 feet long, respectively.

Dredging Options 2A and 2B provide sufficient connections from Nixon Channel and Green Channel into the entrance channel without the unnecessary dredging of Dredging Option 1. Flow into Nixon Channel and Green Channel would occur through the corresponding connecting cuts, and would not increase the erosion observed by Cleary (2008) along the interior salt marsh. At the north end of Beach Road North, seven (7) parcels face Nixon Channel (address numbers 538 to 552). The seven (7) parcels are located at Nixon Channel profiles RIN17+00 to RIN25+00. Due to the shifting of Nixon Channel, these properties are currently experiencing high rates of erosion. The high erosion rates have prompted the placement of sandbags along three (3) of the parcels. Dredging Option 2A can sufficiently address the erosion problem along this area, as detailed in the Delft3D modeling study. Dredging Option 2B cannot, since the deep section of the channel is not moved away from the threatened properties. In Green Channel, the difference in cut volume between Dredging Options 2A and 2B is 17-18%. Thus, the corresponding difference in performance would be negligible. Accordingly, if Dredging Option 2 became the Preferred Dredging Option, the design for Nixon Channel would be Dredging Option 2A, and the design for Green Channel would be Dredging Option 2B.

FIGURE 9-4: Rich Inlet Dredging Option 1 under Alternative 3.

FIGURE 9-5: Rich Inlet Dredging Options 2A and 2B under Alternative 3.

9.1.4.3 Dredging Option 3

Dredging Option 3 appears in Figures 9-6 and 9-7. This Dredging Option features only one connecting cut, which runs from the entrance channel into Nixon Channel. Because there is no connecting cut into Green Channel, it does not provide for adequate flow into Green Channel. Presently, Green Channel connects directly into the existing channel. However, if Dredging Option 3 were constructed with the closure dike across the existing channel, there would be no direct connection between Green Channel and the relocated entrance channel, as shown on the contour map in Figure 9-7. Thus, among all the Dredging Options proposed, Dredging Option 3 represents the greatest departure from the existing conditions. For this reason, Dredging Option 3 has been dropped from consideration.

9.1.4.4 Dredging Option 4

Dredging Options 4A and 4B appear in Figure 9-8. Dredging Options 4A and 4B also dredge a new entrance channel through the middle of Rich Inlet. The seaward end of the entrance channel is at the same location Dredging Options 2A and 2B, and its bearing is the same. However, its length is 4,600 feet. Along the first 3,500 feet, the bottom width is 500 feet given the old design depth of -17 feet NAVD. Along the remainder of the entrance channel, the bottom width is 300 feet given the old design depth of -17 feet NAVD. Where the 500 foot wide section ends, there is a connection into Nixon Channel. This connection runs on the same bearing as Dredging Options 2A and 2B. However, its bottom width is 200 feet given the old design depth of -17 feet NAVD. Under Dredging Options 4A and 4B, the connection to Nixon Channel is 3,800 feet and 1,700 feet long, respectively. There is no direct connection to Green Channel. All side slopes are 1 vertical on 5 horizontal.

Dredging Options 4A and 4B provide a direct connection between Nixon Channel and the entrance channel. The entrance channel ends along a natural channel that runs between Nixon Channel and Green Channel along the salt marsh. The longer entrance channel and this natural channel provide an indirect connection into Green Channel.

The difference between Dredging Options 4A and 4B is the length of the connecting cut into Nixon Channel. For reasons similar to Dredging Option 2, it is necessary to dredge the longer cut into Nixon Channel to address the erosion problem at 538-552 Beach Road North. Accordingly, Dredging Option 4B has been dropped from consideration.

9.1.4.5 Preferred Dredging Option

The two viable Dredging Options are Dredging Option 2 and Dredging Option 4A. Dredging Option 4A can reduce the erosional stresses on the north end of Figure Eight Island. However, it does not offer a direct conduit for flow between Green Channel and the entrance channel. Furthermore, it could accelerate erosion along the salt marsh area facing the entrance of the inlet. For this reason, Dredging Option 4A is not the Preferred Dredging Option. Accordingly, the Preferred Dredging Option for Rich Inlet is Dredging Option 2, with the following variations:

FIGURE 9-6: Rich Inlet Dredging Option 3 under Alternative 3.

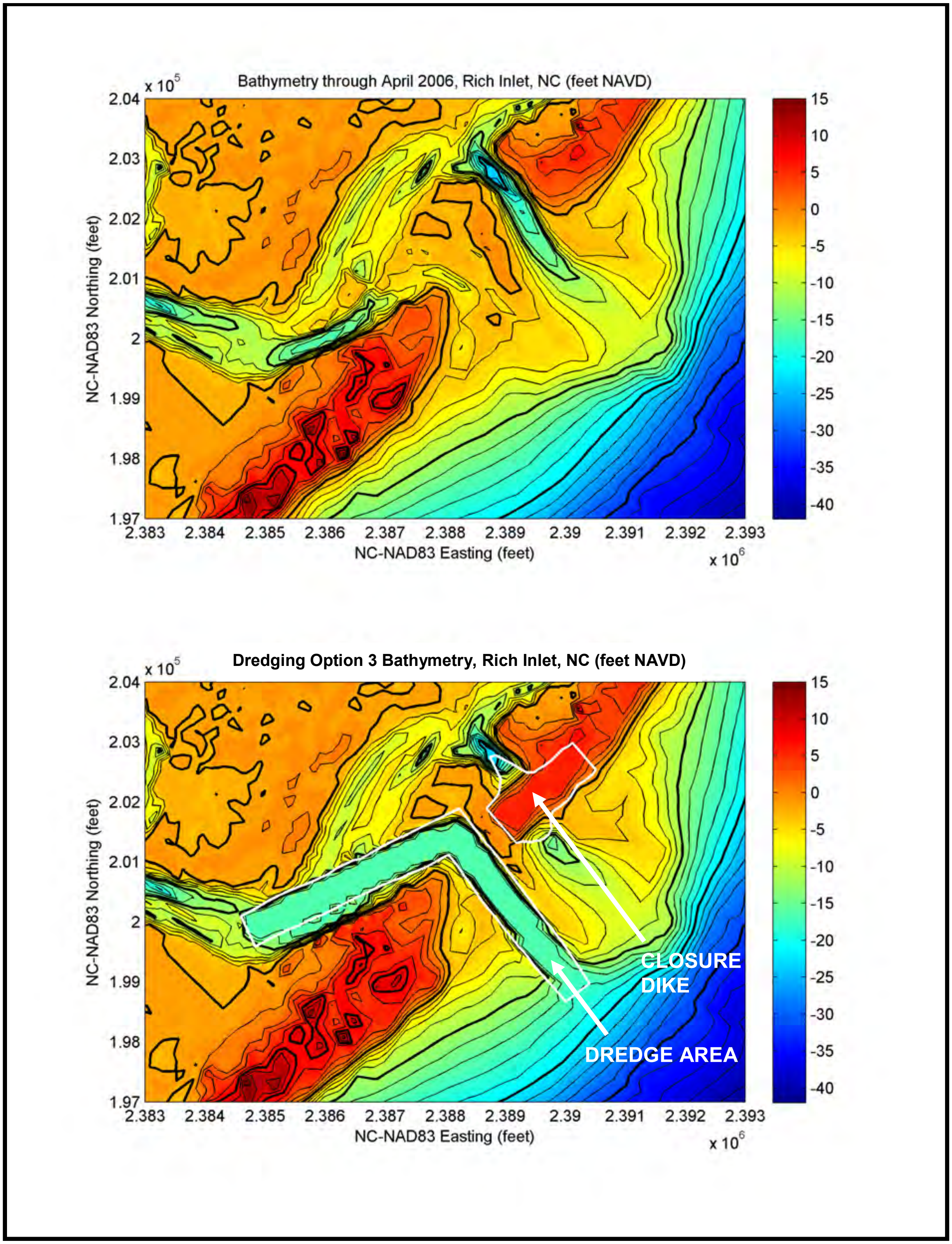


FIGURE 9-7: Bathymetric Contours Given Rich Inlet Dredging Option 3 under Alternative 3.

FIGURE 9-8: Rich Inlet Dredging Options 4A and 4B under Alternative 3.

- Dredging Option 2A inside the entrance channel and Nixon Channel.
- Dredging Option 2B inside the connection to Green Channel.

By dredging a long cut through Nixon Channel, Dredging Option 2A is able to reduce the erosion stress at 538-552 North Beach Road by shifting the flow towards the middle of the channel. Near Green Channel, the shorter Dredging Option 2B eliminates dredging in the interior of Green Channel, while maintaining a conduit for flow between Green Channel and the entrance channel.

To make the project easier to construct, the design depth was changed from -17 to -19 feet NAVD. This change allowed reduction in the bottom width from 500 to 450 feet in the entrance channel and 275 to 240 feet in the Nixon Channel cut. To improve the efficiency of the Green Channel connecting cut, the centerline of the cut was shifted slightly to the west, and the bottom width was changed to 300 feet. This change was able to ensure that the amount of flow going through Green Channel would be similar to the present conditions.

9.1.5 Channel Design Summary under Alternative 3

The Preferred Dredging Option for Rich Inlet features an entrance channel, with 2 side cuts connecting the entrance channel to Nixon Channel and Green Channel. Based on the inlet stability analysis, modeling results, and inquiries regarding feasible dredge depths, the design of Alternative 3's relocated channel in Rich Inlet may be summarized by the following:

- Dredge Depth = -19 feet NAVD + 1 foot overdepth.

- Bottom width & length:

  - Entrance Channel (inlet throat) = 450 feet x 3,500 feet.
  - Nixon Channel = 240 feet x 3,800 feet.
  - Green Channel = 300 feet x 1,400 feet.

- Dredge Volume = 1,786,500 c.y. + 156,400 c.y. overdepth based on the most recent (April 2009 to March 2012) survey = 1,942,900 c.y. total. The Nixon Channel connector contains 27,900 c.y. of clay.

- Closure Dike:

  - Crest Elevation = +6 feet NAVD + 0.5 foot tolerance.
  - Crest Width = 450 feet.
  - Side Slopes = 1 vertical on 20 horizontal (assumed).
  - Volume = 393,000 c.y. + 24,000 c.y. tolerance based on March 2012 survey = 417,000 c.y. total.

- Upland Disposal:
  - 29,700 c.y. clay from Nixon Channel

- Oceanfront Disposal Area:
  - Berm Elevation = +6 feet NAVD + 0.5 foot tolerance.
  - Construction Berm Width = varies.
  - Side Slopes:
    - 1 vertical on 5 horizontal in the dune fill area
    - 1 vertical on 10 horizontal above mean high water (+1.7' NAVD)
    - 1 vertical on 20 horizontal (assumed) below mean high water
  - Fill Length = 12,501 feet.
  - Volume = 1,146,900 c.y. + 43,800 c.y. dune fill based on March 2012 survey = 1,190,700 c.y. total.

- Nixon Disposal Area:
  - Berm Elevation = +6 feet NAVD + 0.5 foot tolerance.
  - Construction Berm Width = varies.
  - Side Slopes:
    - 1 vertical on 5 horizontal
  - Fill Length = 1,400 feet.
  - Volume = 57,000 c.y. + 5,400 c.y. tolerance based on the 2010 before dredging and May-June 2010 LIDAR surveys = 62,400 c.y. total.

A plan view of the dredge cuts and disposal areas appear in Figures 9-8 to 9-10. Typical cross-sections appear in Figures 9-11 and 9-12.

9.1.6 Beach Fill Design under Alternative 3

Based on the 2009-2012 surveys, Alternative 3's Preferred Dredging Option will remove approximately 1,942,900 c.y. from Rich Inlet. Filling the closure dike will require 417,000 c.y., based on the 2012 survey. Also, a pocket of clay, containing 29,700 c.y. was discovered in a section of the Nixon Channel connector which is not beach compatible and would have to be deposited in an upland disposal site located on the south side of the intersection of Nixon Channel and the AIWW. Accordingly, there will be at least 1,496,200 c.y. available to nourish the Figure Eight Island ocean shoreline north of Bridge Road and the Nixon Channel shoreline.

The following options were considered for beach disposal areas:

1. Fill along the entire length of Beach Road (F-5+00 to 105+00, 22,000 feet), and no fill along Nixon Channel.

2. Fill from the intersection of Beach Road and Bayberry Place to Rich Inlet (0+00 to 105+00, 10,500 feet), and no fill along Nixon Channel.

3. Fill from the intersection of Beach Road and Beachbay Lane to Rich Inlet (F90+00 to 105+00, 12,501 feet), with a small fill area along Nixon Channel near the north end of Beach Road (1,400 feet). This option also includes a small amount of dune fill between profiles 77+50 and 95+00 for increased storm protection.

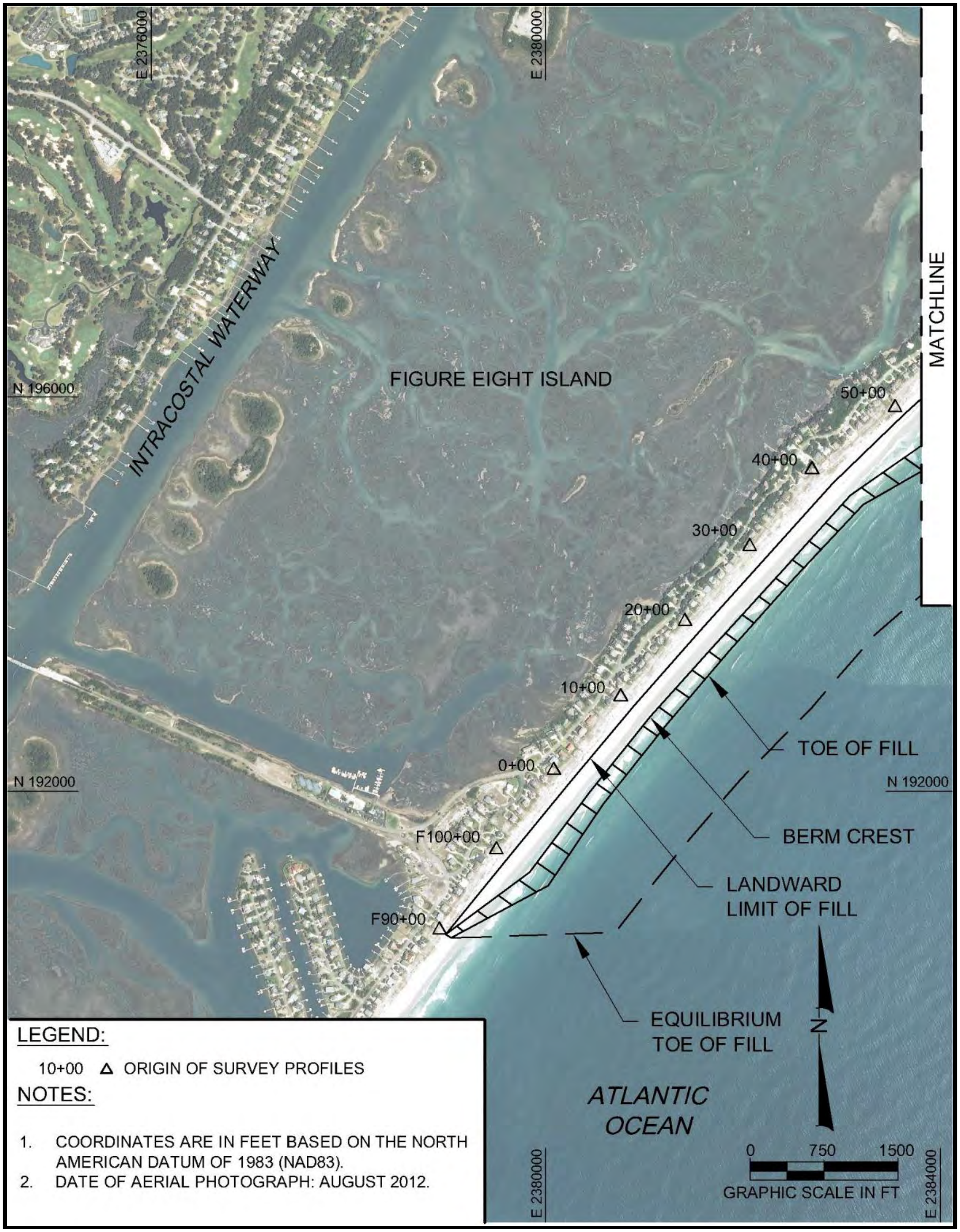


FIGURE 9-9A: Alternative 3 Preferred Dredging Option and Beach Fill Layout.

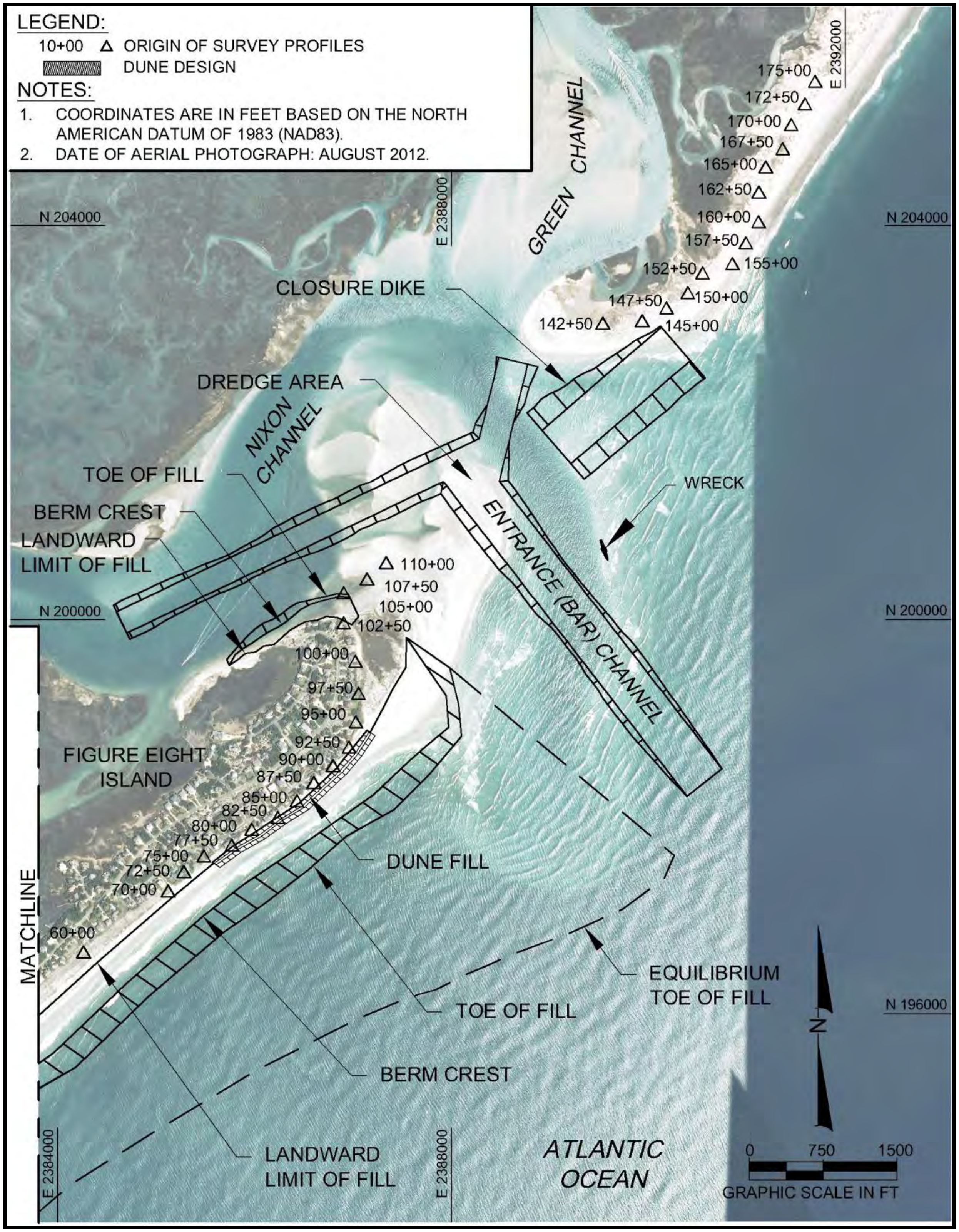


FIGURE 9-9B: Alternative 3 Preferred Dredging Option and Beach Fill Layout.

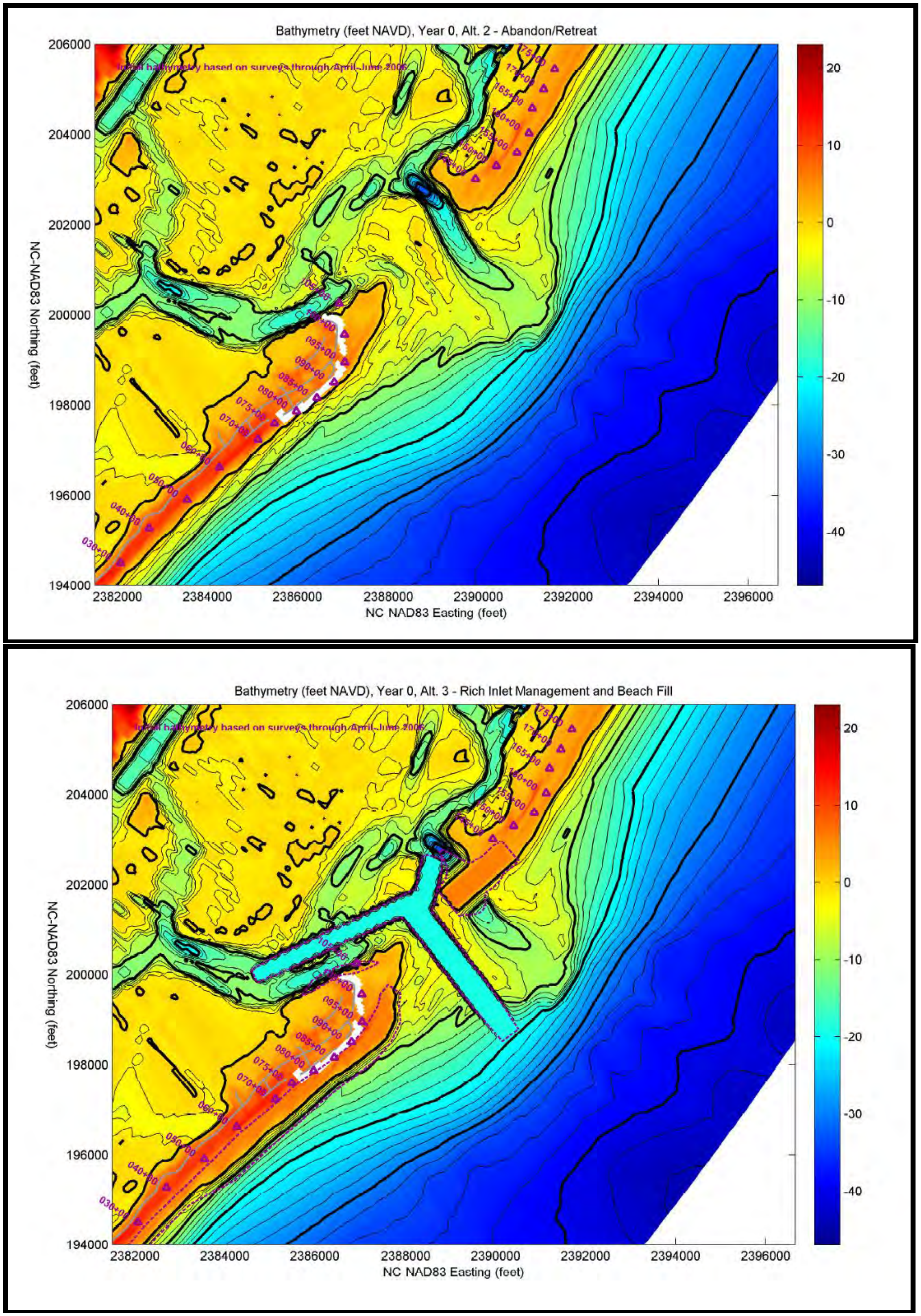
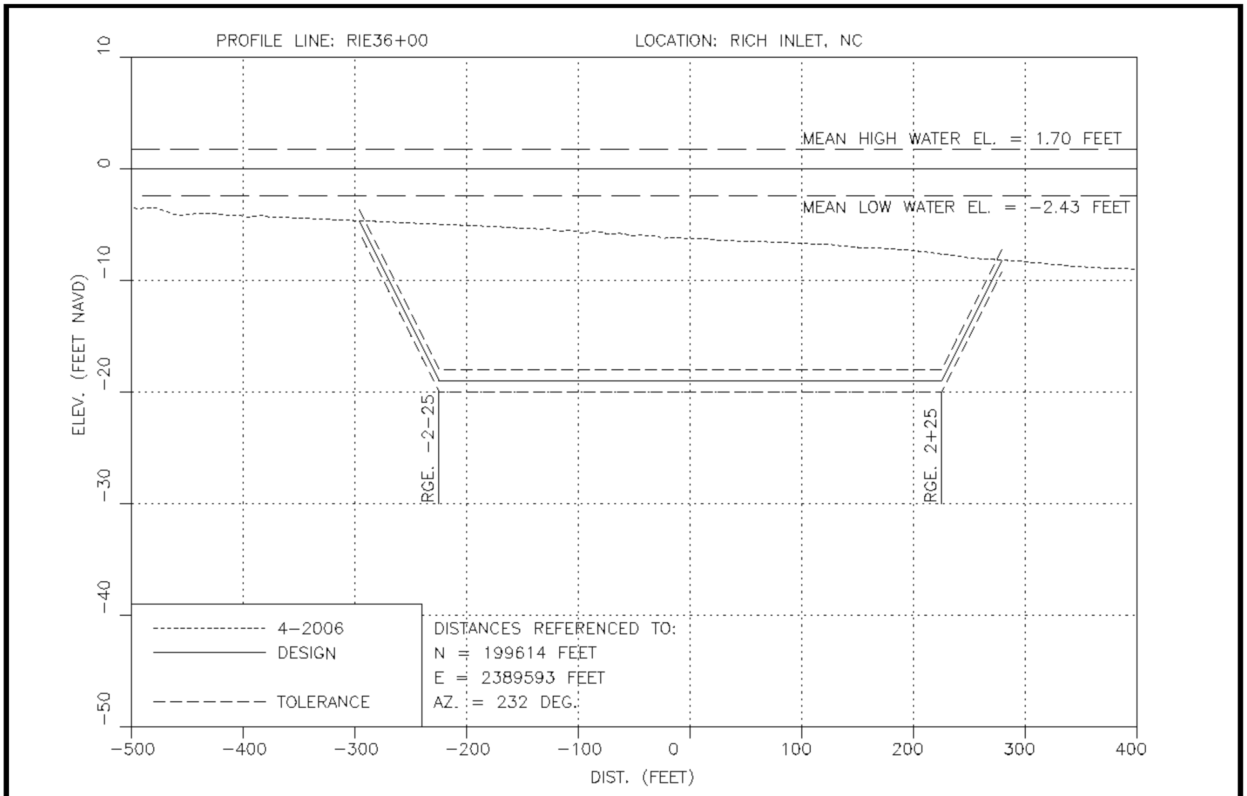


FIGURE 9-10: 2006 Bathymetric Contours and Modification Given Dredging Option No. 2 under Alternative 3.



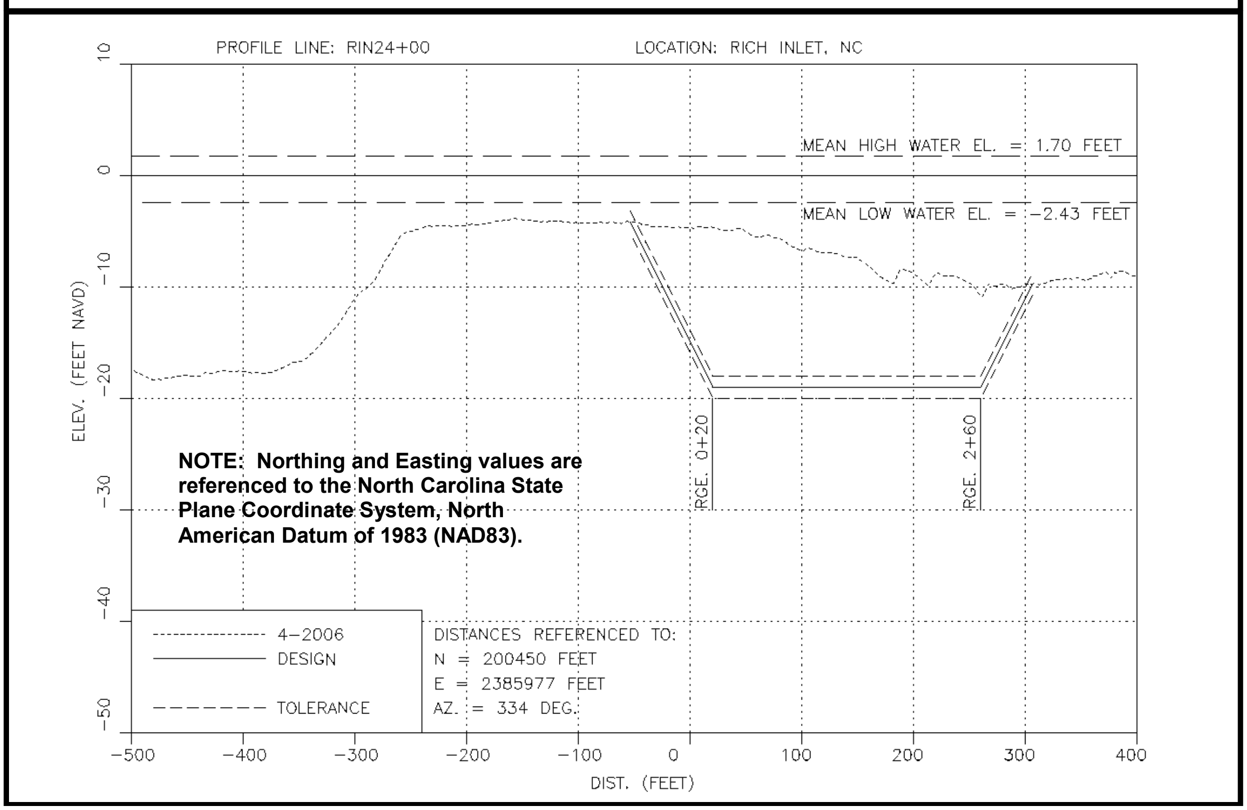


FIGURE 9-11: Typical Cross-Sections in the Entrance Channel (top) and Nixon Channel (bottom), Alternative 3.

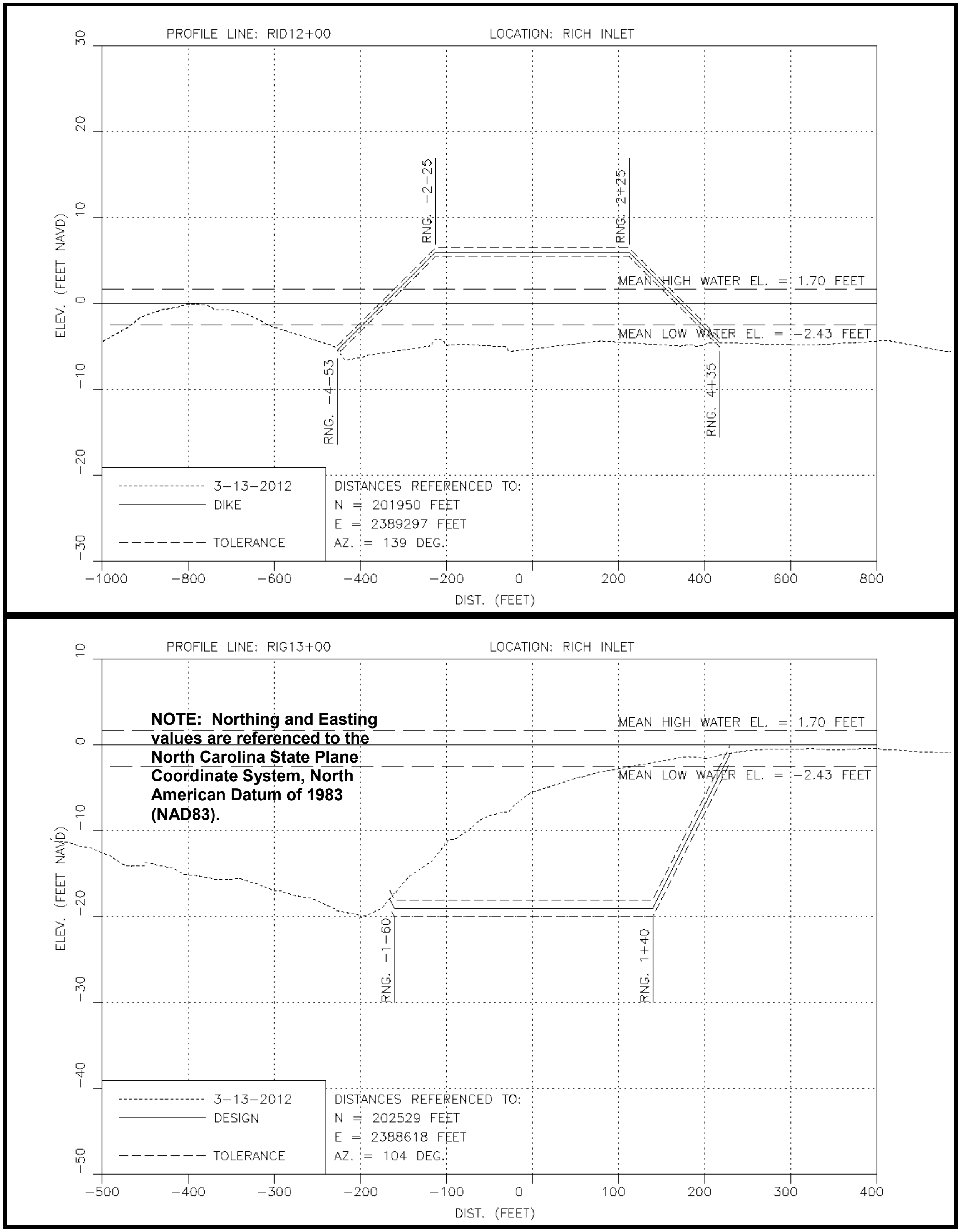


FIGURE 9-12: Typical Cross-Sections in Green Channel (top) and the Closure Dike (bottom), Alternative 3.

Alternative 3 utilizes the 3rd option above. Placing fill along the entire length of Beach Road (option 1) using a pipeline or dustpan dredge would increase the cost of dredging, especially if booster pumps were required. On the other hand, starting the fill at the intersection of Beach Road and Bayberry Place (profile 0+00) (option 2) would leave a gap in the managed shoreline between the Mason Inlet disposal area (profiles F0+00 to F100+00) (ATM, 1999) and the Rich Inlet disposal area. Finally, neither the first nor the second fill options address the high erosion area along Nixon Channel. The 3rd fill option places material along Nixon Channel to address the high erosion rates at the north end of Beach Road. In addition, it utilizes the existing maintenance program at Mason Inlet to economically manage the oceanfront shoreline as a whole. Accordingly, the 3rd fill option is the one included in Alternative 3.

9.1.6.1 Cross-Sectional Volume and Sand Compatibility

Cross-section sizes along the oceanfront shoreline are based on the "Worst Case" retreat rates in Table 6-1. The averages of those values by reach are:

- Beachbay Lane to 282 Beach Road North (F90+00 to 40+00), 9.2 feet/year.
- 302 Beach Road North to 530 Beach Road North (50+00 to 100+00), 24.8 feet/year.

The design berm elevation is +6 feet NAVD, which is approximately equal to the seaward toe of dune along the oceanfront beach fill area. The seaward limit of cross-shore spreading is assumed to be equal to the depth of closure, -24 feet NAVD.

The final quantity needed to determine the cross-section size is the overfill factor. The overfill factor indicates the proportion of fill required to compensate for differences between the grain sizes of the fill source and the existing beach. An overfill factor of 1.0 indicates that no extra fill is required. An overfill factor of 1.28 indicates that the fill volume must be increased 28% to achieve the same performance as material identical to the existing beach. Overfill factors in Table 9-1 are based on the beach composites in Table 4-7, the preliminary inlet composite for the dredge cuts, and the Shore Protection Manual (James-Krumbein) Overfill and Renourishment Factor (USACE, 1986). The higher overfill factor, based on the existing material along Figure Eight Island, is 1.044.

TABLE 9-1
OVERFILL FACTORS
FIGURE EIGHT ISLAND, NC

Composite	Mean Grain Size (mm)	Sorting (Φ)	Overfill Factor Ra
Figure Eight Island (F80+00 to 90+00)	0.18	0.55	1.044
Hutaff Island (H1 to H3)	0.21	0.85	1.000
Dredge Area (Figure 9-8)	0.24	0.83	

Based on the averaged retreat rates above, the design berm elevation (+6' NAVD), the cross-shore spreading limit (-24' NAVD), and an overfill factor of 1.044, cross-section sizes along oceanfront shoreline appear in Table 9-2. Cross-section sizes and fill volumes exclude the upper tolerance.

Cross-section sizes along the Nixon Channel shoreline are based on shoreline retreat rates between 1993 and 2005 (USGS, 1993; NOAA, 2005) (Table 9-3). Full size cross-sections extend 800 feet from profiles RIN17+00 to RIN25+00. The eastern taper section is 500 feet long, extending from profiles RIN12+00 to RIN17+00. The western taper section is 500 feet long, extending from profiles RIN25+00 to RIN30+00. The assumed cross-shore spreading limit along Nixon Channel is also -24 feet NAVD. Although this is deeper than the scour hole along the fill area, the deeper value provides a factor of safety against the high spreading losses that will occur due to the short fill length. Given the averaged retreat rate in Table 9-3, the design berm elevation (+6' NAVD), the assumed cross-shore spreading limit (-24' NAVD), and an overfill factor of 1.044, cross-section sizes along the Nixon Channel shoreline appear in Table 9-4. Cross-section sizes and fill volumes exclude the upper tolerance.

9.1.6.2 Profile Shape

The shapes of the construction templates along the beach were based on the post-construction profiles following the 2005 Bogue Inlet Channel Erosion Response Project. Beach slopes on those profiles averaged 1 vertical on 8 horizontal above wading depth, and 1 vertical on 23 horizontal below wading depth.

In the oceanfront fill area, the specified beach slope above the waterline is 1 vertical on 10 horizontal along the oceanfront fill area. For planning purposes, a beach slope of 1 vertical on 20 horizontal below the waterline is assumed. However, it should be noted that contractors are not able to control the beach slope below the waterline. Accordingly, the beach slope below the waterline is strictly an estimate based on the performance of a previous project in the region.

The design dune cross-section along Comber Road and Inlet Hook Road (profiles 77+50 to 95+00) has side slopes of 1 vertical on 5 horizontal. The crest width of the dune cross-section is 25 feet. To prevent sand from blowing into the upland properties, the dune crest elevation will be similar to the existing dune elevations along the dune fill area, which is approximately +15 feet NAVD. Overall, the dune location in Figure 9-8 is an approximation. The exact dune locations and crest elevations will be determined based on the conditions at the project site immediately prior to construction.

In the Nixon Channel fill area, the specified side slope is 1 vertical on 5 horizontal. This slope is roughly based on the existing bank slope along the scour hole. The assumed slope below the waterline is equal to the specified side slope above the waterline. Representative cross-sections along both fill areas appear in Figures 9-13 and 9-14.

TABLE 9-2

OCEANFRONT BEACH DISPOSAL AREA, ALTERNATIVE 3
BASED ON MARCH 2012 SURVEY
FIGURE EIGHT ISLAND / RICH INLET, NC

Profile Line	Fill Length (feet)	Design Retreat Rate (feet/year)	Adjusted Berm Width (feet)	Fill Distribution (c.y./foot)			Fill Volume (c.y./foot)		
				Beach	Dune	TOTAL	Beach	Dune	TOTAL
F90+00			0.0	0.0	0.0	0.0			
	1,000						26,800	0	26,800
F100+00		-9.2	46.2	53.5	0.0	53.5			
	1,001						53,600	0	53,600
0+00		-9.2	46.2	53.5	0.0	53.5			
	1,000						53,500	0	53,500
10+00		-9.2	46.2	53.5	0.0	53.5			
	1,000						53,500	0	53,500
20+00		-9.2	46.2	53.5	0.0	53.5			
	1,000						53,500	0	53,500
30+00		-9.2	46.2	53.5	0.0	53.5			
	1,000						53,500	0	53,500
40+00		-9.2	46.2	53.5	0.0	53.5			
	1,000						98,600	0	98,600
50+00		-24.8	123.8	143.6	0.0	143.6			
	1,000						143,600	0	143,600
60+00		-24.8	123.8	143.6	0.0	143.6			
	1,000						143,600	0	143,600
70+00		-24.8	123.8	143.6	0.0	143.6			

TABLE 9-2 (continued)

OCEANFRONT BEACH DISPOSAL AREA, ALTERNATIVE 3
BASED ON MARCH 2012 SURVEY
FIGURE EIGHT ISLAND / RICH INLET, NC

Profile Line	Fill Length (feet)	Design Retreat Rate (feet/year)	Adjusted Berm Width (feet)	Fill Distribution (c.y./foot)			Fill Volume (c.y./foot)		
				Beach	Dune	TOTAL	Beach	Dune	TOTAL
	250						35,900	0	35,900
72+50		-24.8	123.8	143.6	0.0	143.6			
	250						35,900	0	35,900
75+00		-24.8	123.8	143.6	20.1	163.6			
	250						35,900	5,400	41,300
77+50		-24.8	123.8	143.6	23.0	166.5			
	250						35,900	5,400	41,300
80+00		-24.8	123.8	143.6	20.5	164.0			
	250						35,900	5,200	41,100
82+50		-24.8	123.8	143.6	21.3	164.8			
	250						35,900	5,400	41,300
85+00		-24.8	123.8	143.6	22.1	165.7			
	250						35,900	5,600	41,500
87+50		-24.8	123.8	143.6	22.7	166.2			
	250						35,900	5,700	41,600
90+00		-24.8	123.8	143.6	23.2	166.8			

TABLE 9-2 (continued)

OCEANFRONT BEACH DISPOSAL AREA, ALTERNATIVE 3
BASED ON MARCH 2012 SURVEY
FIGURE EIGHT ISLAND / RICH INLET, NC

Profile Line	Fill Length (feet)	Design Retreat Rate (feet/year)	Adjusted Berm Width (feet)	Fill Distribution (c.y./foot)			Fill Volume (c.y./foot)		
				Beach	Dune	TOTAL	Beach	Dune	TOTAL
	250						35,900	5,700	41,600
92+50		-24.8	123.8	143.6	22.1	165.7			
	250						35,900	5,400	41,300
95+00		-24.8	123.8	143.6	21.0	164.5			
	250						35,900	0	35,900
97+50		-24.8	123.8	143.6	0.0	143.6			
	250						35,900	0	35,900
100+00		-24.8	123.8	143.6	0.0	143.6			
	250						26,900	0	26,900
102+50		-24.8	61.9	71.8	0.0	71.8			
	250						9,000	0	9,000
105+00			0.0	0.0	0.0	0.0			
Oceanfront F90+00 to 105+00	12,501			91.7	3.5	95.2	1,146,900	43,800	1,190,700

TABLE 9-3
SHORELINE CHANGES ON THE SOUTH SIDE OF NIXON CHANNEL

Profile Line	Profile Origin (NC-NAD83) Easting (feet)	Northing (feet)	Azimuth (deg.)	Shoreline Changes (feet/year) March 1993 to October 2005	October 1996 To October 2005	DESIGN
RIN12+00	2387059.4	200966.8	334.5	-N/A-	7.1	7.1
RIN13+00	2386969.2	200923.7	334.5	-N/A-	-9.3	-9.3
RIN14+00	2386879.0	200880.6	334.5	-N/A-	-14.9	-14.9
RIN15+00	2386788.7	200837.5	334.5	-N/A-	-22.5	-22.5
RIN16+00	2386698.5	200794.4	334.5	-N/A-	-N/A-	-12.8
RIN17+00	2386608.2	200751.3	334.5	-3.0	-N/A-	-3.0
RIN18+00	2386518.0	200708.2	334.5	-1.8	-N/A-	-1.8
RIN19+00	2386427.8	200665.1	334.5	-7.4	-N/A-	-7.4
RIN20+00	2386337.5	200622.0	334.5	-8.2	-N/A-	-8.2
RIN21+00	2386247.3	200578.9	334.5	-8.8	-N/A-	-8.8
RIN22+00	2386157.1	200535.8	334.5	-8.6	-N/A-	-8.6
RIN23+00	2386066.8	200492.7	334.5	-8.8	-N/A-	-8.8
RIN24+00	2385976.6	200449.6	334.5	-8.5	-N/A-	-8.5
RIN25+00	2385886.4	200406.5	334.5	-9.8	-N/A-	-9.8
RIN26+00	2385796.1	200363.4	334.5	-10.8	-N/A-	-10.8
RIN27+00	2385705.9	200320.3	334.5	-9.4	-N/A-	-9.4
RIN28+00	2385615.6	200277.2	334.5	-8.7	-N/A-	-8.7
RIN29+00	2385525.4	200234.1	334.5	-8.6	-N/A-	-8.6
RIN30+00	2385435.2	200191.0	334.5	-7.7	-N/A-	-7.7
AVERAGE						-8.6

TABLE 9-4

NIXON CHANNEL BEACH DISPOSAL AREA, ALTERNATIVE 3
BASED ON 2010 LIDAR & NIXON CHANNEL SURVEYS
FIGURE EIGHT ISLAND / RICH INLET, NC

Profile Line	Fill Length (feet)	Design Retreat Rate (feet/year)	Adjusted Berm Width (feet)	Beach Fill Distr. (c.y./foot)	Beach Fill Volume (c.y.)
RIN12+00*					
RIN13+00*					
RIN14+00*					
RIN15+00*					
RIN16+00		-8.6	34.2	39.7	
	100				4,500
RIN17+00		-8.6	42.8	49.6	
	100				5,000
RIN18+00		-8.6	42.8	49.6	
	100				5,000
RIN19+00		-8.6	42.8	49.6	
	100				5,000
RIN20+00		-8.6	42.8	49.6	
	100				5,000
RIN21+00		-8.6	42.8	49.6	
	100				5,000
RIN22+00		-8.6	42.8	49.6	
	100				5,000
RIN23+00		-8.6	42.8	49.6	
	100				5,000
RIN24+00		-8.6	42.8	49.6	
	100				5,000
RIN25+00		-8.6	42.8	49.6	
	100				4,500
RIN26+00			34.2	39.7	
	100				3,500
RIN27+00			25.7	29.8	
	100				2,500
RIN28+00			17.1	19.8	
	100				1,500
RIN29+00			8.6	9.9	
	100				500
RIN30+00			0.0	0.0	
Nixon Channel RIN16+00 to RIN30+00	1,400			40.7	57,000

*NOTE: The preliminary design included fill from profiles RIN12+00 to RIN30+00. To reduce impacts to a small tidal creek at the east end of this area, fill between profiles RIN12+00 and RIN16+00 was deleted.

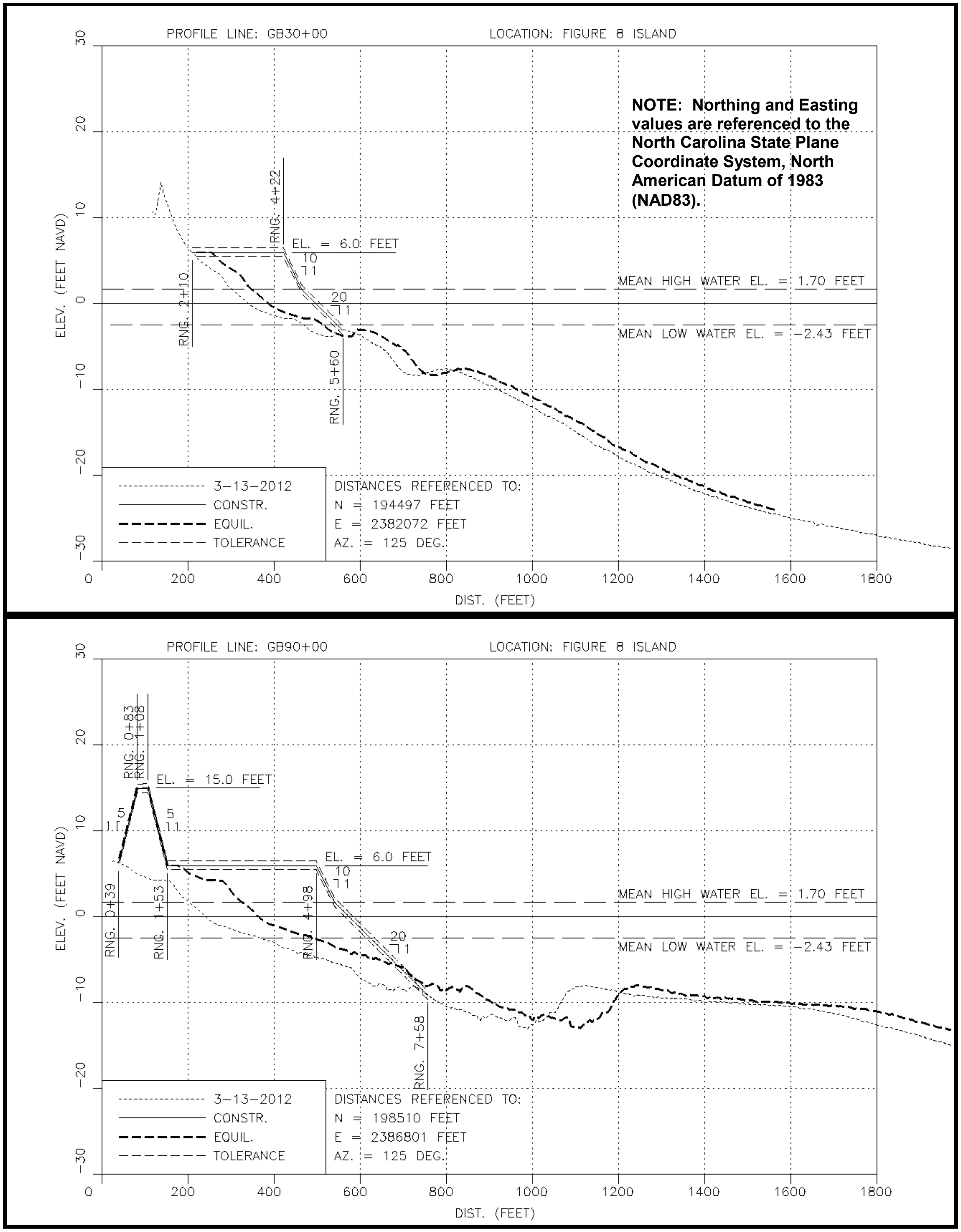


FIGURE 9-13: Representative Cross-Sections along the Oceanfront Beach Fill Area, Alternative 3.

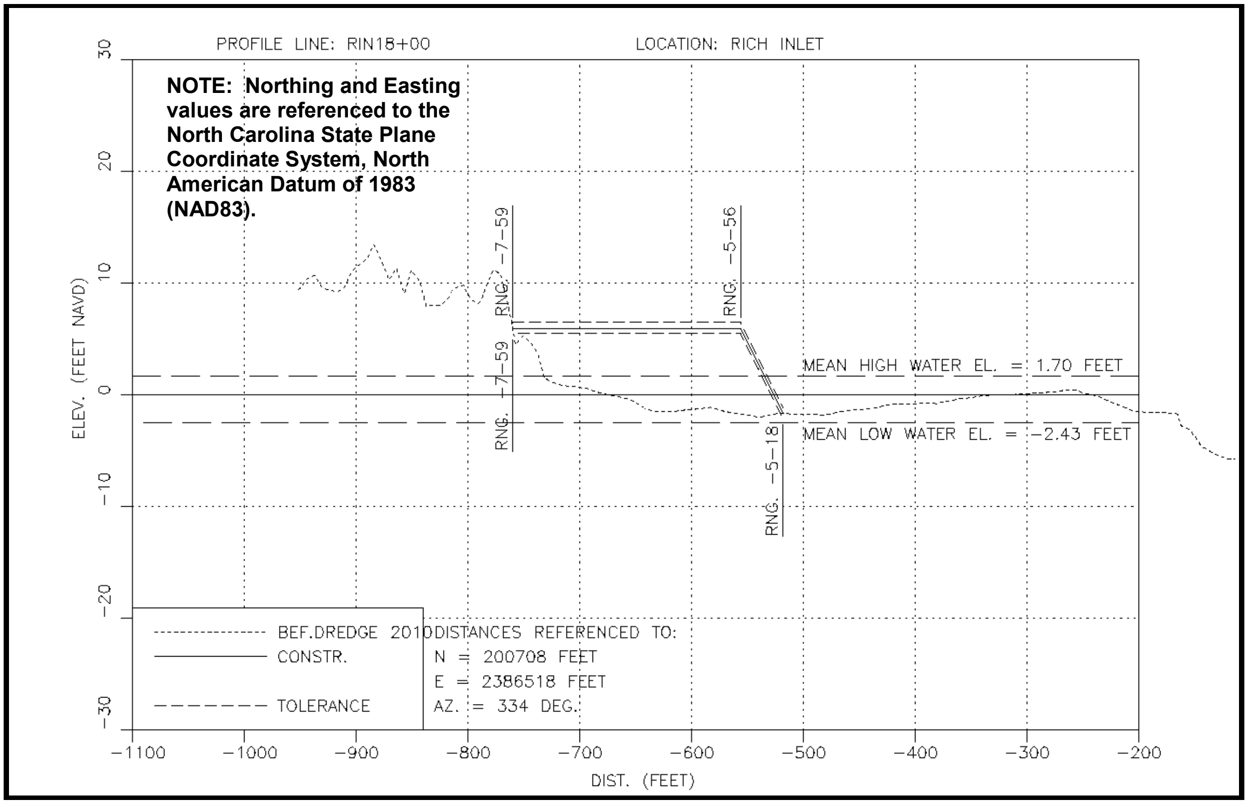


FIGURE 9-14: Representative Cross-Section along the Nixon Channel Fill Area, Alternative 3.

9.2 Alternative 4 –Beach Nourishment without Inlet Management

Alternative 4 would include a beach fill along the ocean shoreline between Rich Inlet and Bridge Road and a fill along the Nixon Channel shoreline immediately behind the north end of Figure Eight Island and periodic nourishment to maintain the fills. The size of the beach fill along the ocean shoreline associated with Alternative 3 was dictated by the volume of material that would be removed to move the inlet ocean bar channel to a preferred position and alignment and modify the channels leading into both Nixon and Green Channels. For Alternative 4, the size of the beach fill was based on the modeled performance of a fill between Rich Inlet and Bridge Road without any modifications to Rich Inlet. In this regard, the size of the beach fill modeled under Alternative 4 was similar to Alternative 3. However, analysis of the model results found this beach fill to be over designed for the area between stations F90+00 and 80+00 and under designed for the area north of station 80+00. As a result, the beach fill under Alternative 4 was modified to address shoreline erosion issues resulting in a smaller initial beach fill between F90+00 and 80+00 and a larger fill between 80+00 and 100+00. Since Alternative 4 does not include any modification to the Rich Inlet ocean bar channel, material to construct and maintain the beach fills would have to be obtained from other sources which are evaluated below.

Also, due to the high rates of loss from the fill obtained from the model results for the area between 80+00 and 100+00, the beach fill design for Alternative 4 was based on a four-year periodic nourishment cycle. The total initial beach fill volume along the ocean shoreline from Rich Inlet to Bridge Road would be 864,300 cubic yards based on the 2006 conditions and 911,300 based on the 2012 conditions. Design berm width and fill placement densities along the ocean shoreline are given in Table 9-5 with the fill distribution provided in Table 9-6. Beach fill placement rates and design berm widths for Alternative 4 are provided in Table 9.5 with the layout of the beach fill shown in Figures 9-15A and 9-15B. The beach fill along Nixon Channel would be the same as Alternative 3 or 57,000 cubic yards. Including the Nixon Channel beach fill, the total beach fill volume for Alternative 4 would be 921,300 cubic yards based on the 2006 conditions and 968,300 cubic yards based on the 2012 conditions.

TABLE 9-5
ALTERNATIVE 4
BEACH FILL PLACEMENT VOLUMES AND DESIGN BERM WIDTHS

Shoreline Segment (Baseline Stations)	Placement Volume (cy/lf)	Design Berm Width (ft)
105+00 to 100+00 (transition)	0 to 200	0 to 172
100+00 to 82+50	200	172
82+50 to 80+00 (transition)	200 to 100	172 to 86
80+00 to 70+00	100	86
70+00 to 60+00 (transition)	100 to 50	86 to 43
60+00 to 30+00	50	43
30+00 to 20+00 (transition)	50 to 20	43 to 17
20+00 to F100+00	20	17
F100+00 to F90+00 (transition)	20 to 0	17 to 0

Material to construct and maintain the beach fill under Alternative 4 would be derived from maintenance dredging of the previously permitted area in Nixon Channel, the potential offshore borrow areas identified by Dr. Cleary as described in Chapter 3 of this document, and the three northern AIWW disposal sites also discussed in Chapter 3. Due to the relative small volume available from the three AIWW disposal sites, these sites would be held in reserve and only used for periodic nourishment if the volume of material shoaling the existing permit area in Nixon Channel is insufficient to meet nourishment requirements or other concerns over the removal of the material from Nixon Channel prevent its use. Also, the relatively high rate of periodic nourishment rates for Alternative 4 indicated by the model results would require the continued use of the offshore borrow sites in order to satisfy the nourishment requirements.

Based on the Delft3D model results discussed later in this document, renourishment of the fill areas under Alternative 4 are expected to be the following:

- Oceanfront fill area:
    - Profiles 60+00 to 105+00: 764,000 cubic yards every 4 years given the 2006 initial conditions and 508,000 cubic yards every 4 years given the 2012 initial conditions.
    - Profiles F90+00 to 60+00: Deferred until deemed necessary based on future monitoring surveys.
- Nixon Channel fill area: 24,000 cubic yards every 4 years.

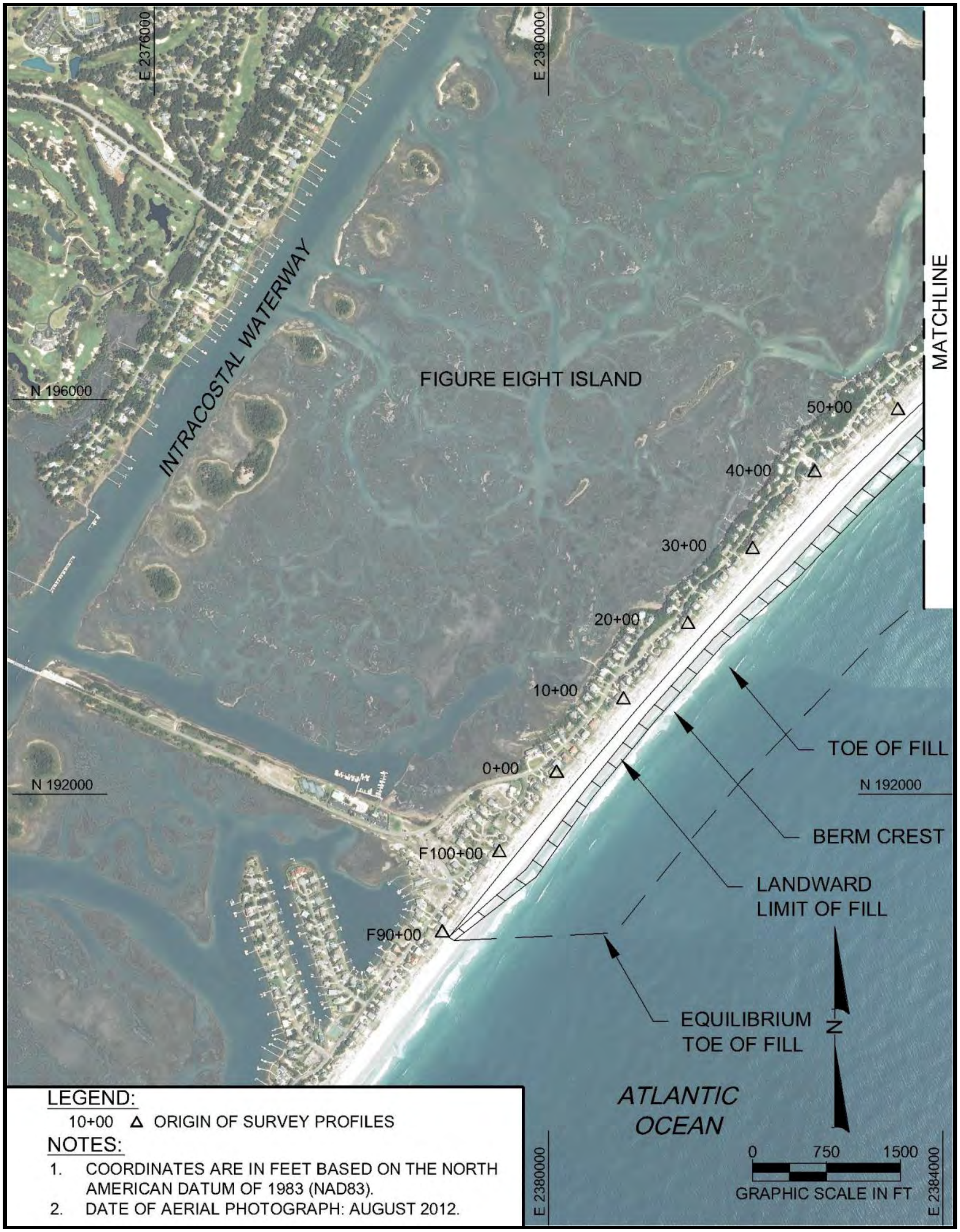


FIGURE 9-15A: Alternative 4 Beach Fill Layout

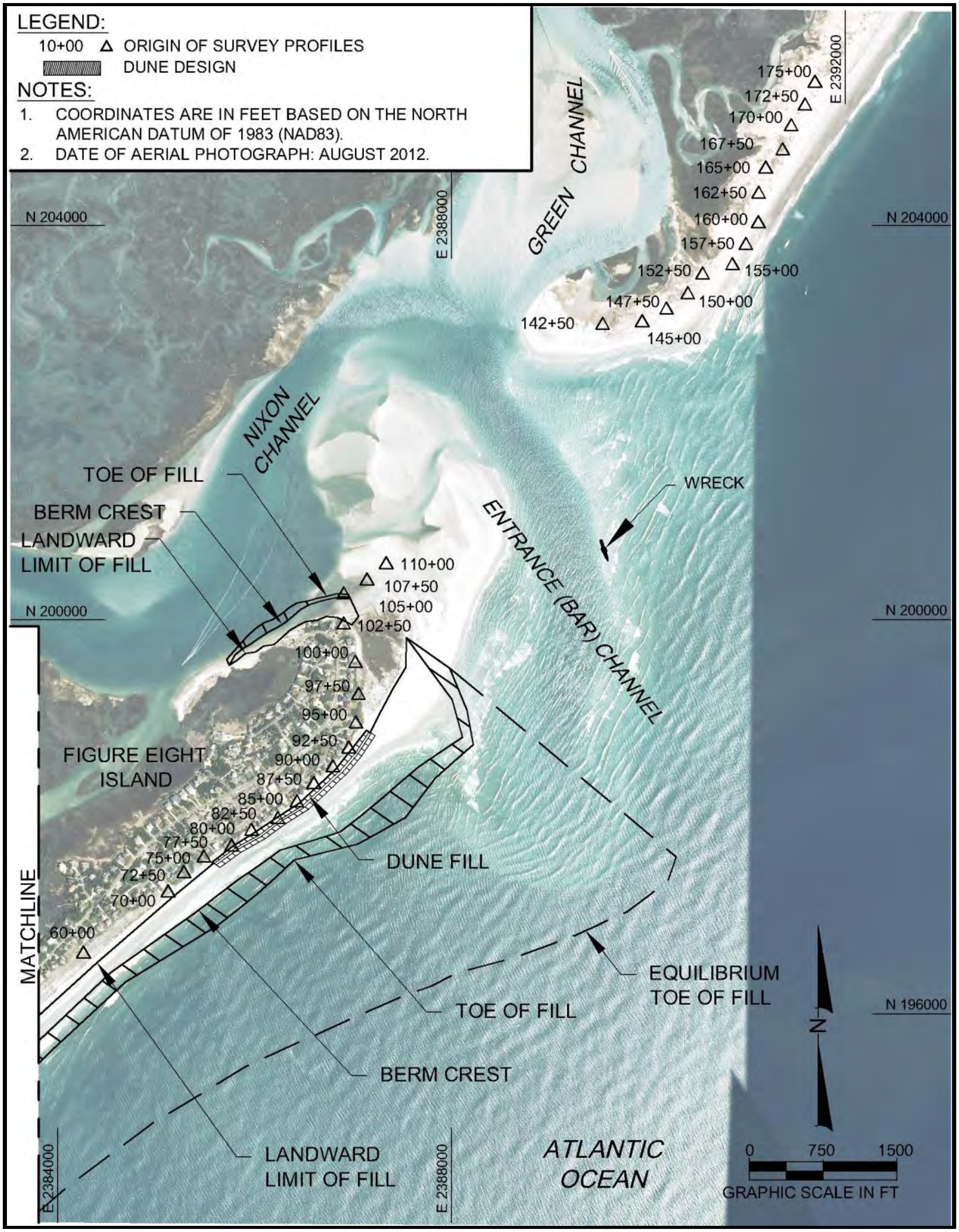


FIGURE 9-15B: Alternative 4 Beach Fill Layout

TABLE 9-6

OCEANFRONT BEACH DISPOSAL AREA, ALTERNATIVE 4
BASED ON MARCH 2012 SURVEY
FIGURE EIGHT ISLAND / RICH INLET, NC

Profile Line	Fill Length (feet)	Adjusted Berm Width (feet)	Fill Distribution (c.y./foot)			Fill Volume (c.y./foot)		
			Beach	Dune	TOTAL	Beach	Dune	TOTAL
F90+00		0.0	0.0	0.0	0.0			
	1,000					10,000	0	10,000
F100+00		17.2	20.0	0.0	20.0			
	1,001					20,000	0	20,000
0+00		17.2	20.0	0.0	20.0			
	1,000					20,000	0	20,000
10+00		17.2	20.0	0.0	20.0			
	1,000					20,000	0	20,000
20+00		17.2	20.0	0.0	20.0			
	1,000					35,000	0	35,000
30+00		43.1	50.0	0.0	50.0			
	1,000					50,000	0	50,000
40+00		43.1	50.0	0.0	50.0			
	1,000					50,000	0	50,000
50+00		43.1	50.0	0.0	50.0			
	1,000					50,000	0	50,000
60+00		43.1	50.0	0.0	50.0			
	1,000					75,000	0	75,000
70+00		86.2	100.0	0.0	100.0			

TABLE 9-6 (continued)

OCEANFRONT BEACH DISPOSAL AREA, ALTERNATIVE 4
BASED ON MARCH 2012 SURVEY
FIGURE EIGHT ISLAND / RICH INLET, NC

Profile Line	Fill Length (feet)	Adjusted Berm Width (feet)	Fill Distribution (c.y./foot) Beach	Dune	TOTAL	Fill Volume (c.y./foot) Beach	Dune	TOTAL
	250					25,000	0	25,000
72+50		86.2	100.0	0.0	100.0			
	250					25,000	0	25,000
75+00		86.2	100.0	20.1	120.1			
	250					25,000	5,400	30,400
77+50		86.2	100.0	23.0	123.0			
	250					25,000	5,400	30,400
80+00		86.2	100.0	20.5	120.5			
	250					37,500	5,200	42,700
82+50		172.4	200.0	21.3	221.3			
	250					50,000	5,400	55,400
85+00		172.4	200.0	22.1	222.1			
	250					50,000	5,600	55,600
87+50		172.4	200.0	22.7	222.7			
	250					50,000	5,700	55,700
90+00		172.4	200.0	23.2	223.2			

TABLE 9-6 (continued)

OCEANFRONT BEACH DISPOSAL AREA, ALTERNATIVE 4
BASED ON MARCH 2012 SURVEY
FIGURE EIGHT ISLAND / RICH INLET, NC

Profile Line	Fill Length (feet)	Adjusted Berm Width (feet)	Fill Distribution (c.y./foot)			Fill Volume (c.y./foot)		
			Beach	Dune	TOTAL	Beach	Dune	TOTAL
	250					50,000	5,700	55,700
92+50		172.4	200.0	22.1	222.1			
	250					50,000	5,400	55,400
95+00		172.4	200.0	21.0	221.0			
	250					50,000	0	50,000
97+50		172.4	200.0	0.0	200.0			
	250					50,000	0	50,000
100+00		172.4	200.0	0.0	200.0			
	250					37,500	0	37,500
102+50		86.2	100.0	0.0	100.0			
	250					12,500	0	12,500
105+00		0.0	0.0	0.0	0.0			
Oceanfront F90+00 to 105+00	12,501		69.4	3.5	72.9	867,500	43,800	911,300

9.3 Alternative 5C – Terminal Groin at a More Northerly Location with Beach Fill from Nixon Channel and a New Channel Connecting to Gorge of Rich Inlet

During the 2011 legislation session, the North Carolina Legislature passed Session Law 2011-387, Senate Bill 110 which allows consideration of terminal groins adjacent to tidal inlets. The legislation limited the number of terminal groins to four (4) statewide and included a number of provisions and conditions that must be met in order for the groins to be approved and permitted. In 2013, the State Legislature passed the Coastal Policy Reform Act of 2013 (SL2013-384) that modified some of the requirements included in the 2011 legislation.

The purpose of the terminal groin is to create a permanent accretion fillet immediately adjacent to the inlet by controlling tide induced or influenced sediment transport off the extreme north end of the island. In so doing, the groin and associated accretion fillet would create a relatively stable shoreline position immediately south of the inlet with an alignment comparable to the shoreline farther south. The elimination or reduction in tide induced or influenced sediment transport off the extreme north end of the island should improve the performance and longevity of beach fills placed on the northern half of Figure Eight Island but would not prevent littoral transport, i.e., wave induced sediment transport from moving past the terminal groin and into Rich Inlet. In this regard, a terminal groin would not address shoreline management problems along the entire island therefore; a shoreline management alternative that includes a terminal groin must include beach nourishment.

9.3.1 Formulation of Alternative 5C

Alternative 5C, which positions the terminal groin 420 feet north of the terminal groin position presented in the DEIS, evolved through the development of Alternative 5A. One element included in the development of Alternative 5A was consideration of three possible channel extensions from the previously permitted area in Nixon Channel to the gorge of Rich Inlet. The three channel options included:

- Dredging Option 1 – 660-740 foot wide connecting cut.
- Dredging Option 2 – 600 foot wide connecting cut.
- Dredging Option 3 – 395-416 foot wide connecting cut.

The purpose of the new channel was to:

- Facilitate navigation between the existing entrance channel and Nixon Channel.
- Provide for a straight flow pattern through Nixon Channel, to reduce the severity of erosion along the end of N. Beach Road.

Through an initial screening process involving simulations in the Delft3D model, Dredging Option 2, shown in Figure 9-16, was selected. As noted, the selected dredging option is also applicable to Alternative 5C. Dredging Option 2 provides a sufficient amount of fill material to pre-fill the groin and provide nourishment of the beach south to Bridge Road. In addition, the channel was found to be more conducive to navigation, with a depth of at least -10 feet NAVD maintained at the seaward end of Nixon Channel over the 5-year maintenance cycle. Overall, Dredging Option 2 represents the best balance of performance, cost, and impact.

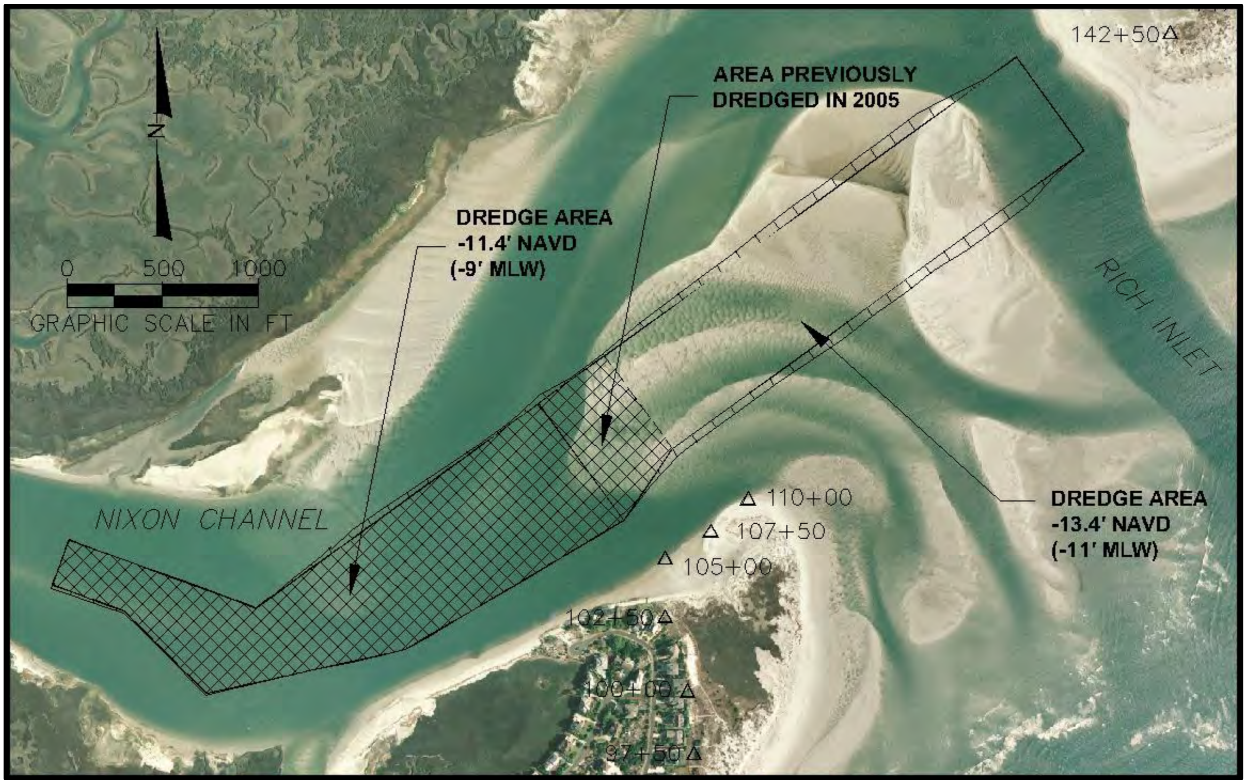


Figure 9-16. Dredging Option 2 – Alternative 5A and 5C.

Additional options for Alternative 5A involving the length of the terminal groin, its performance with and without beach fill, and orientations toward Figure Eight Island were also evaluated. The results of the Delft3D model simulations for these alternatives/options are presented graphically in Sub-Appendix B.

The model evaluations considered two possible lengths each measured from the April 2007 mean high water shoreline. The two lengths evaluated were 700 feet and 1200 feet. Based on the model results, the shorter terminal groin option was selected. Also, the model results for with the terminal groin oriented toward Figure Eight Island did not produce any significant improvement of the performance of the beach fill along the northern end of the island. Therefore, the preferred alignment of the terminal groin would be approximately perpendicular to the shoreline.

The results of the screening process for Alternative 5A, primarily the selection of the dredging option and the orientation of the terminal groin and its general overall length were incorporated into the design of Alternative 5C.

9.3.1 Description of Alternative 5C

Alternative 5C includes a 1,300-foot terminal groin located near baseline station 105+00 or in the more northerly position relative to Alternatives 5A and 5B presented in the DEIS. The terminal groin would include a 995-foot shore anchorage section extending landward of the 2007 mean high water shoreline and a 305-foot section extending seaward of the 2007 mean high water shoreline. The shore anchorage section would be constructed with sheet pile (steel or concrete) while the seaward section would be of rubblemound construction. The landward 100 feet of the shore anchorage section would include a 10-foot wide scour protection mat on both sides of the sheet pile. The beach fill for Alternative 5C would be constructed with material obtained from maintenance of the previously permitted area in Nixon Channel and construction of a new channel connecting Nixon Channel with the gorge of Rich Inlet as shown in Figure 9-16.

Excavation of the previously permitted area in Nixon Channel and the new channel connecting Nixon Channel with the gorge of Rich Inlet would involve the removal of 994,400 cubic yards given the 2006 initial conditions and 1,077,100 cubic yards of material for the 2012 initial conditions. An estimated 29,700 cubic yards of clay is included in this total volume. The clay material would be deposited in an upland disposal site. This would leave 964,700 cubic yards of sandy material given the 2006 conditions and 1,047,400 cubic yards of sandy material under the 2012 conditions.

9.3.2 Beach Fill Areas

Based on the most recent surveys and an allowable overdepth of one-foot, excavation of the dredge area in Figure 9-16 will provide 1,047,400 cubic yards of beach compatible material and 29,700 cubic yards of clay which would be deposited in an upland disposal site. Alternative 5C would provide a beach fill along the shoreline of Nixon Channel and along the oceanfront extending from Beachbay Lane (F90+00) to the terminal groin located near station 105+00.

Although the maintenance cycle of the project will be 5-years, a large volume is required to pre-fill the terminal groin and provide beach fill south to station F90+00. By straightening the shoreline immediately south of the terminal groin and reducing the direct impact of tidal currents along the extreme north end of the island, the terminal groin should reduce erosion rates at the island's northern end while allowing wave induced sediment transport to pass over, around, and/or through the terminal groin. Between profile 75+00 (south of Surf Court) and the terminal groin, fill distributions are based on the volume of material that would be placed to pre-fill the groin fillet. South of profile 75+00, fill distributions are based on 3 years of erosion, given the retreat rates in Tables 9-2 and 9-5, a berm elevation of +6 feet NAVD, a depth of closure equal to -24 feet NAVD, and an overfill factor of 1.044 (Table 9-1). The 3 year assumption was simply used as a means of apportioning the fill within the available volume discussed above. Based on the model results discussed later in the report, the amount of fill south of Surf Court should be sufficient for preventing erosion into the present shoreline over a 5 year period.

The fill area along the Nixon Channel shoreline contains 57,000 cubic yards. The distribution of the fill along the Nixon Channel shoreline is provided in Table 9-7.

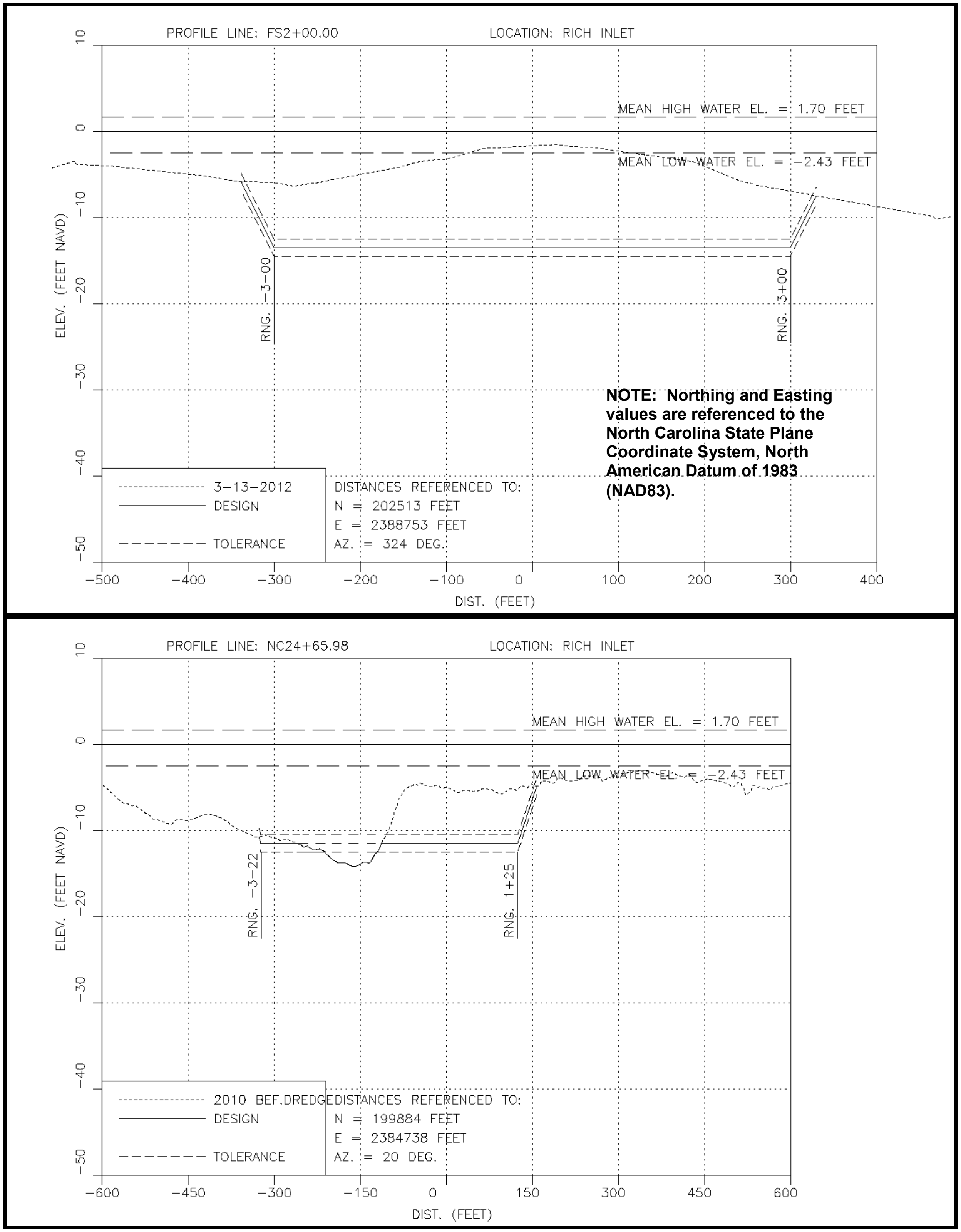


FIGURE 9-17: Representative Dredging Cross-Sections, Preferred Dredging Option (2), Alternative 5C.

TABLE 9-6

OCEANFRONT BEACH DISPOSAL AREA
ALTERNATIVE 5C
FIGURE EIGHT ISLAND / RICH INLET, NC

Profile Line	Fill Length (feet)	Design Retreat Rate (feet/year)	Adjusted Berm Width (feet)	Fill Distribution (c.y./foot)			Fill Volume (c.y.)		
				Beach	Dune	Total	Beach	Dune	Total
F90+00		-9.2	0.0	0.0	0.000	0.000			
F100+00	1,000	-9.2	36.9	42.8	0.000	42.8	21,400	0	21,400
0+00	1,001	-9.2	36.9	42.8	0.000	42.8	42,900	0	42,900
10+00	1,000	-9.2	36.9	42.8	0.000	42.8	42,800	0	42,800
20+00	1,000	-9.2	36.9	42.8	0.000	42.8	42,800	0	42,800
30+00	1,000	-9.2	36.9	42.8	0.000	42.8	42,800	0	42,800
40+00	1,000	-9.2	36.9	42.8	0.000	42.8	42,800	0	42,800
50+00	1,000	-24.8	99.0	114.9	0.000	114.9	78,800	0	78,800
60+00	1,000	-24.8	99.0	114.9	0.000	114.9	114,800	0	114,800
70+00	1,000	-24.8	99.0	114.9	0.000	114.9	114,800	0	114,800
72+50	250	-24.8	99.0	114.9	0.000	114.9	28,700	0	28,700
75+00	250	-24.8	99.0	114.9	20.1	134.9	28,700	0	28,700
77+50	250	-24.8	99.0	114.9	23.0	137.8	28,700	5,400	34,100
80+00	250	-24.8	99.0	114.9	20.5	135.3	28,700	5,400	34,100
82+50	250	-24.8	99.0	114.9	21.3	136.1	28,700	5,200	33,900
85+00	250	-24.8	99.0	114.9	22.1	136.9	28,700	5,400	34,100
87+50	250	-24.8	99.0	114.9	22.7	137.5	28,700	5,600	34,300
90+00	250	-24.8	99.0	114.9	23.2	138.1	28,700	5,700	34,400
92+50	250	-24.8	99.0	114.9	22.1	137.0	28,700	5,700	34,400
95+00	250	-24.8	99.0	114.9	21.0	135.8	28,700	5,400	34,100
97+50	250	-24.8	99.0	114.9	0.0	114.9	28,700	0	28,700
100+00	250	-24.8	99.0	114.9	0.0	114.9	28,700	0	28,700
102+50	250	-24.8	99.0	114.8	0.0	114.8	28,700	0	28,700
105+00	250	-24.8	99.0	114.8		114.8	28,700	0	28,700
Ocean front F90+00 to 105+00	12,501						945,700	43,800	989,500

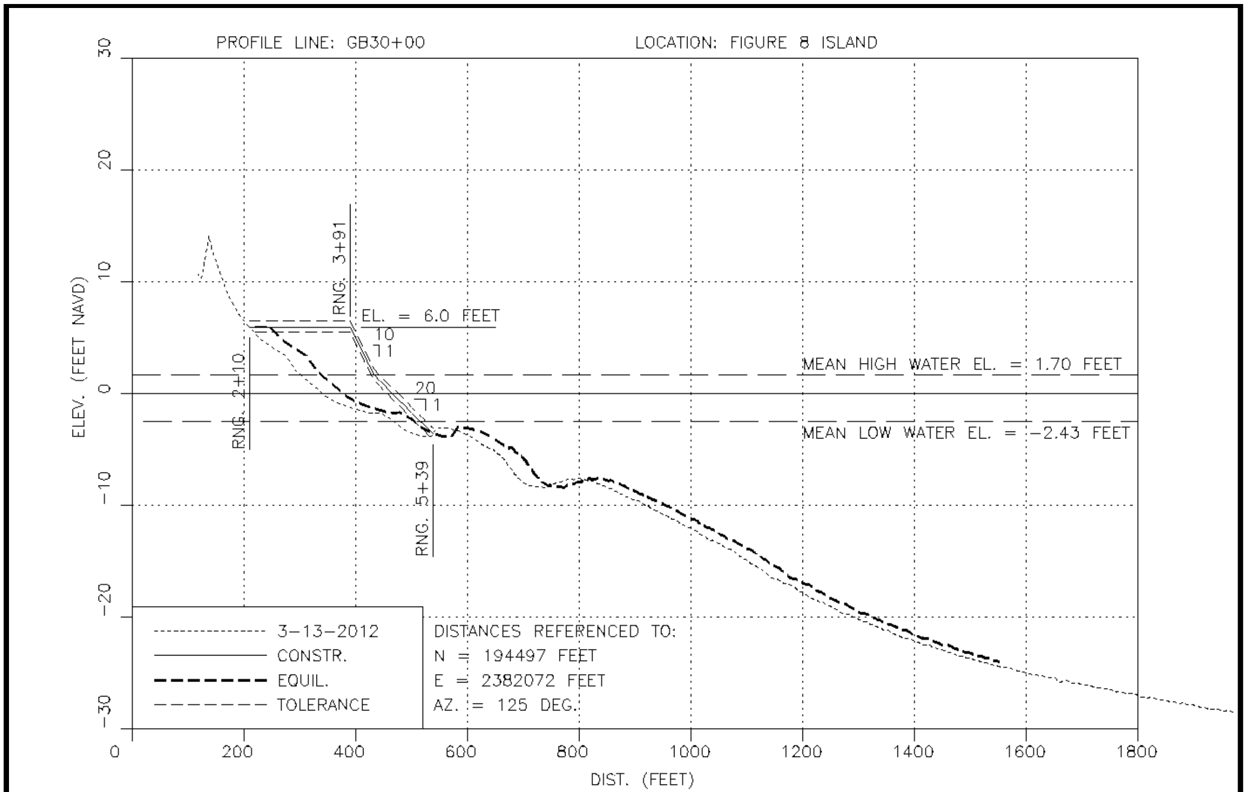
TABLE 9-7

NIXON DISPOSAL AREA
ALTERNATIVE 5C
FIGURE EIGHT ISLAND / RICH INLET, NC

Profile Line	Fill Length (feet)	Beach Fill Distribution (c.y./foot)	Beach Fill Volume (c.y.)
RIN15+00		0	0
RIN16+00	100	39.7	4,500
RIN17+00	100	49.6	5,000
RIN18+00	100	49.6	5,000
RIN19+00	100	49.6	5,000
RIN20+00	100	49.6	5,000
RIN21+00	100	49.6	5,000
RIN22+00	100	49.6	5,000
RIN23+00	100	49.6	5,000
RIN24+00	100	49.6	5,000
RIN25+00	100	49.6	4,500
RIN26+00	100	39.7	3,500
RIN27+00	100	29.8	2,500
RIN28+00	100	19.8	1,500
RIN29+00	100	10.0	500
Total	1,400		57,000

9.3.5 Profile Shape

Profile shapes along the fill area are based on the same assumptions as those of Alternative 3. Representative cross-sections appear in Figures 9-17 through 9-18.



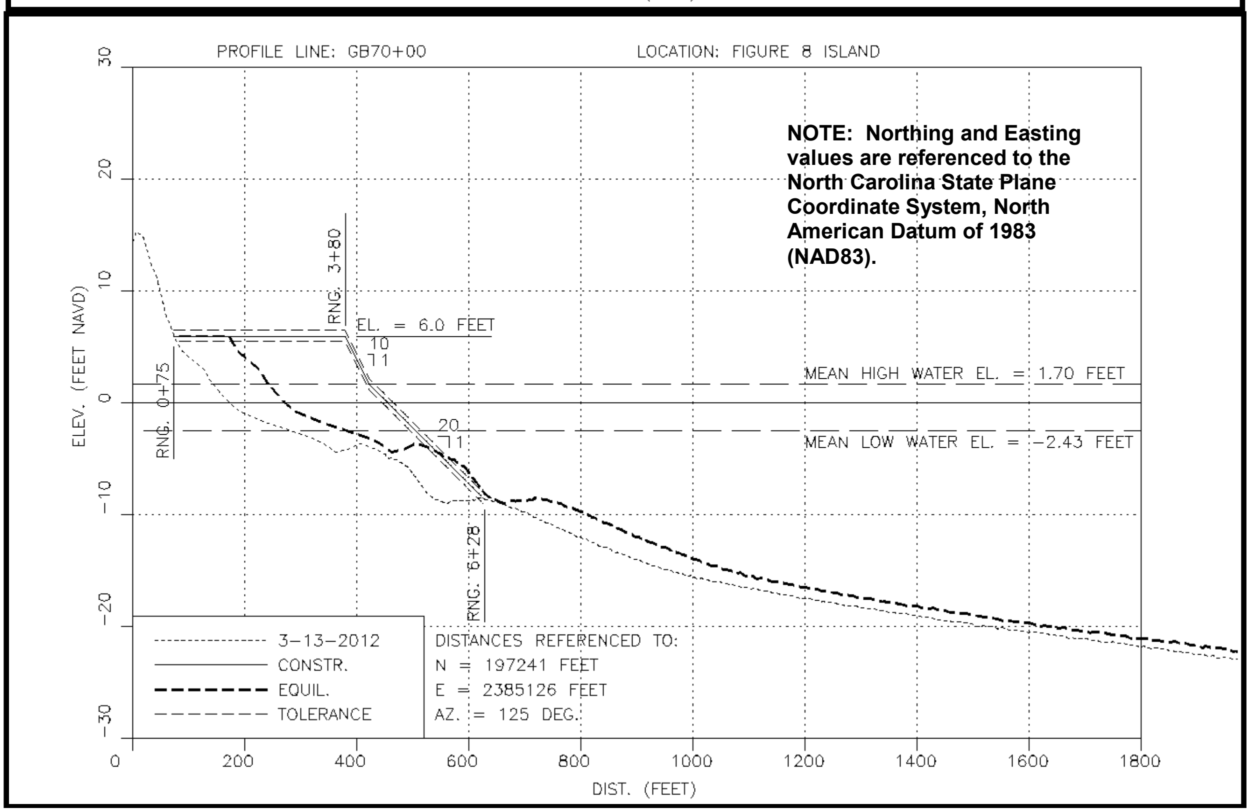


FIGURE 9-18: Representative Cross-Sections along the Oceanfront Beach Fill Area, Alternative 5C.

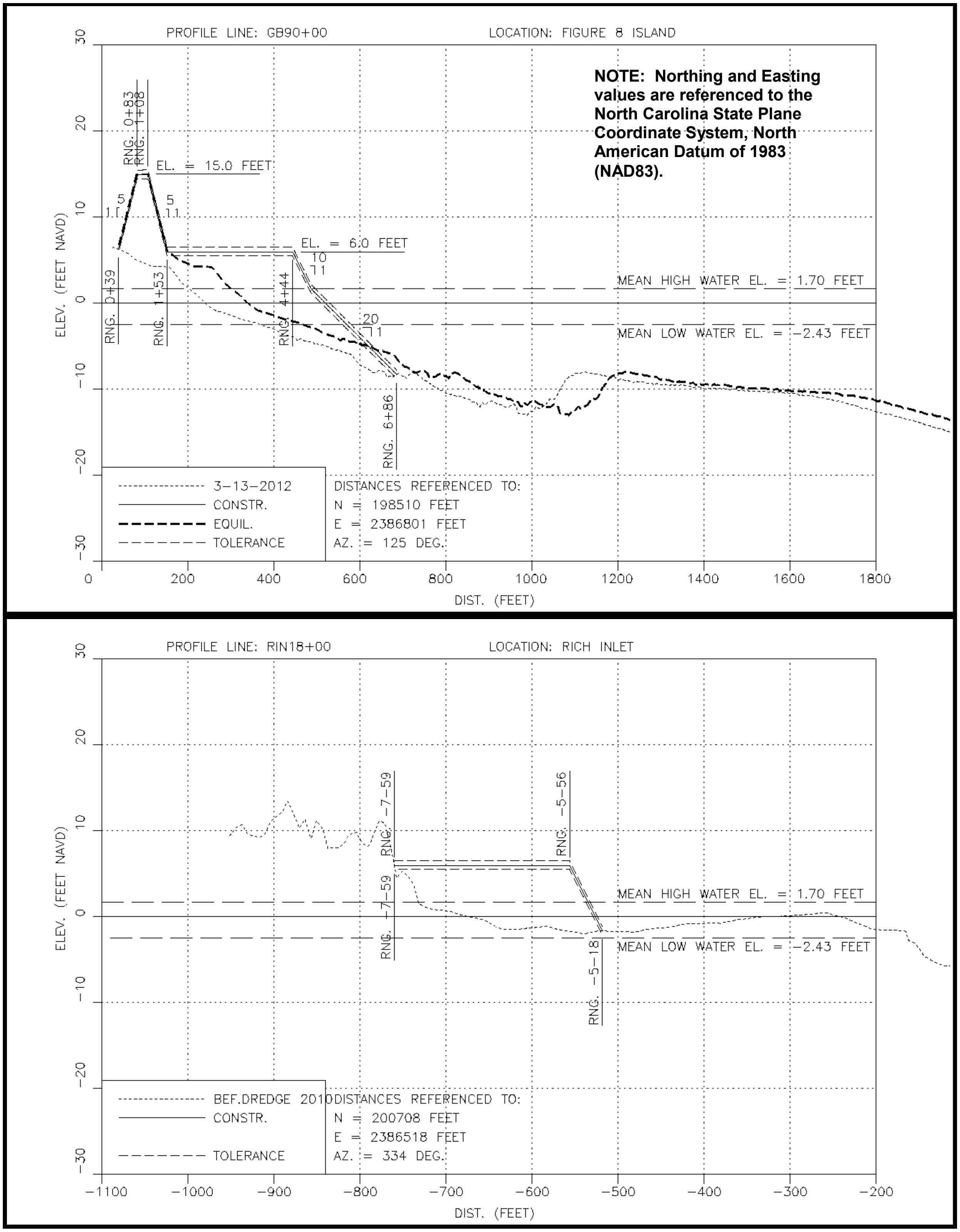


FIGURE 9-19: Representative Cross-Sections along the North End of Figure Eight Island including the Nixon Channel shoreline, Alternative 5C.

9.2.6 Design Summary for Alternative 5C

Based on the various features discussed above, the dredging and groin option for Alternative 5C can be summarized by the following:

- Terminal groin length = 1,300 feet with 305 feet extending seaward from the April 2007 shoreline and 995 feet landward of the April 2007 shoreline.

- Terminal groin footprint (bottom surface area) = 0.7 acres.

- Groin crest elevation:
  - Landward shore anchorage segment (995 feet): +1.5 feet NAVD first 795 feet landward of the April 2007 MHW shoreline and 0.5 feet NAVD last 200 feet.
  - Rubblemound segment 305 feet seaward of April 2007 MHW shoreline: +6 feet NAVD.

- Groin material: Sheet Pile (concrete or steel) for shore anchorage section and Granite quarry stone for seaward 305-foot segment. Armor stone ranging from 7.5 tons to 12.5 tons.

- Dredge cut depth in Nixon Channel and Channel Connector:
  - East section of dredge cut: -13.43 feet NAVD (-11 feet MLW) + 1 foot overdepth.
  - West section of dredge cut: -11.43 feet NAVD (-9 feet MLW) + 1 foot overdepth.

- Dredged cut bottom width:
  - East end of dredge cut: 600 feet.
  - Bending section of dredge cut: 250 to 754 feet.
  - West end of dredge cut: 250 feet.

- Dredge cut length: 6,156 feet.

- Dredge Volume = 994,400 c.y. based on the 2006 survey and 1,077,100 c.y. based on the 2012 surveys.

- Volume of clay to be deposited in upland disposal area = 29,700 c.y.

- Net volume of beach quality material (sandy material) = 907,700 c.y. for the 2006 condition and 990,400 c.y. for the 2012 condition.

- Oceanfront Disposal Area:
  - Berm Elevation = +6 feet NAVD + 0.5 foot tolerance.
  - Construction Berm Width = varies.
  - Side Slopes:
    - 1 vertical on 5 horizontal in the dune fill area
    - 1 vertical on 10 horizontal above mean high water (+1.7' NAVD)
    - 1 vertical on 20 horizontal (assumed) below mean high water

- Fill Length = 12,500 feet (Station F90+00 to 105+00).
- Volume = 850,700 c.y. based on the 2006 conditions and 933,400 c.y. for the 2012 conditions.

- Nixon Disposal Area:
  - Berm Elevation = +6 feet NAVD + 0.5 foot tolerance.
  - Construction Berm Width = varies.
  - Side Slopes:
    - 1 vertical on 5 horizontal
  - Fill Length = 1,400 feet.
  - Volume = 57,000 c.y.

A plan view of Alternative 5C as whole appears in Figures 9-20A and 9-20B.

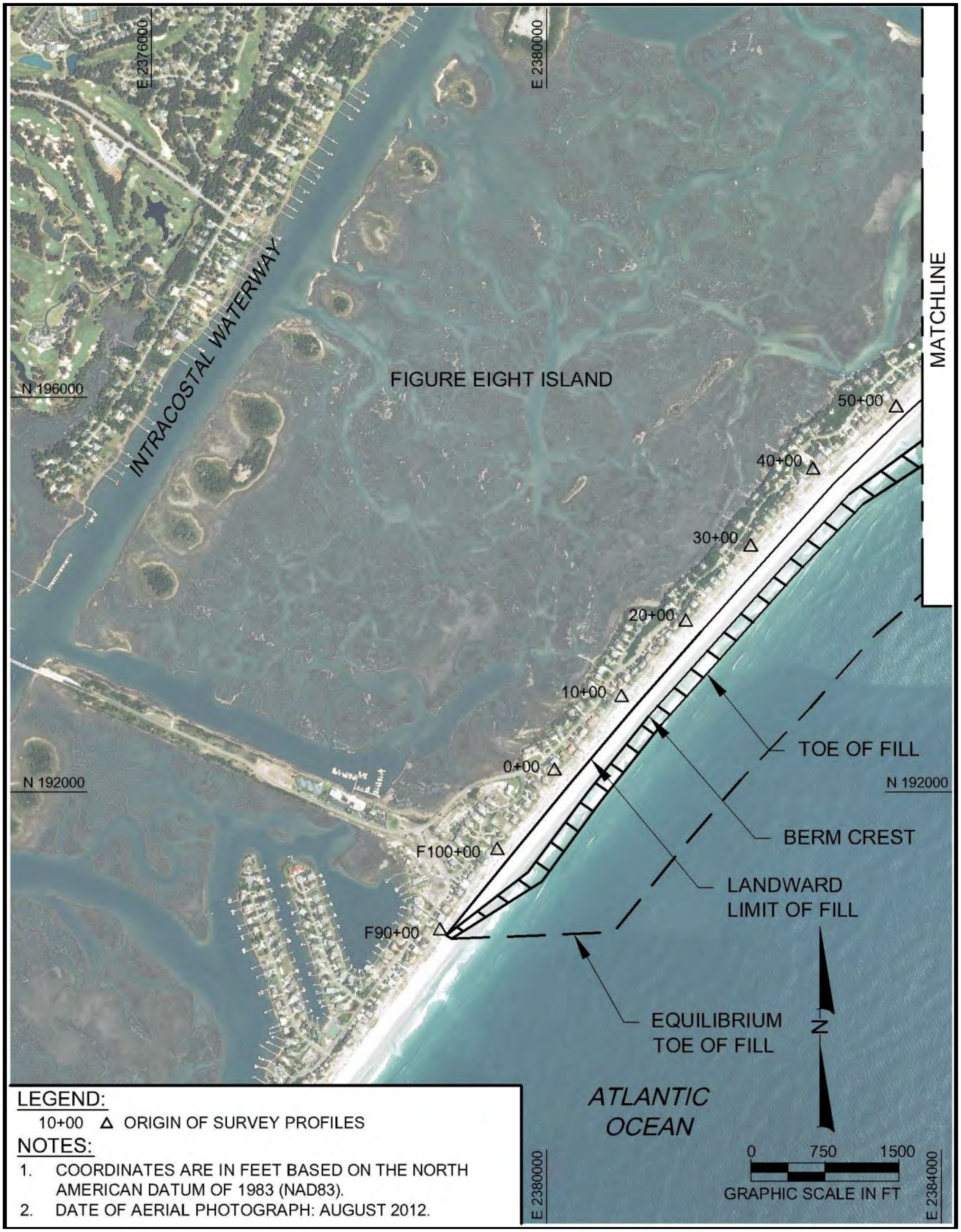


FIGURE 9-20A: Alternative 5C
Dredging and Groin Option and Beach Fill Layout.

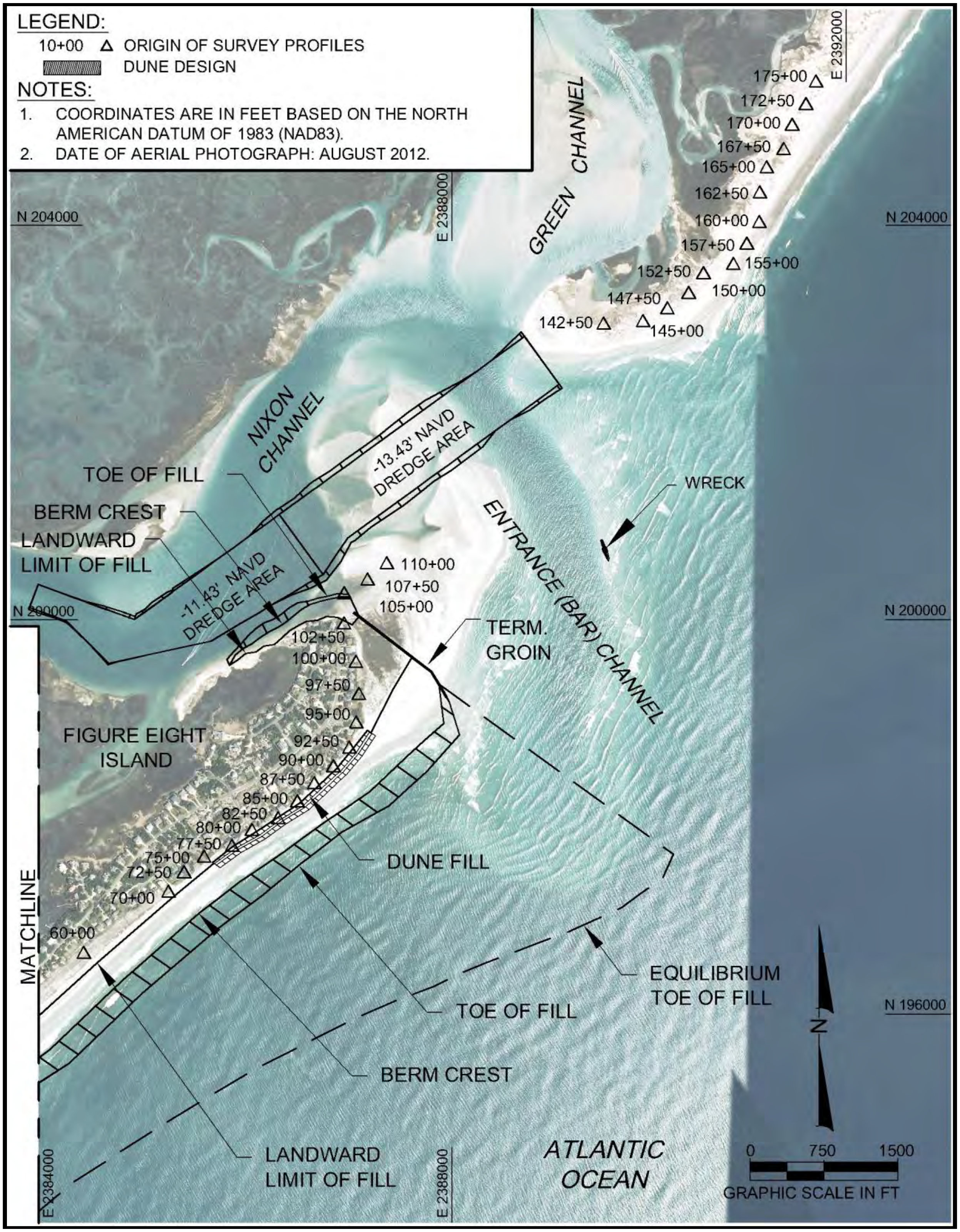


FIGURE 9-20B: Alternative 5C
Dredging and Groin Option and Beach Fill Layout.

9.4 Alternative 5D (Applicant's Preferred Alternative): Terminal Groin at a More Northerly Location with Beach Fill from the Previously Permitted Area in Nixon Channel and Other Sources

Alternative 5D includes a terminal groin in the more northerly location and the same beach fill along Nixon Channel as Alternatives 5C. The ocean shoreline beach fill for Alternative 5D would extend from station 60+00 (approximately 322 Beach Road North) to the terminal groin (station 105+00).

9.4.1 Beach Fill Design

The volume of material needed to construct the beach fill along the ocean shoreline would be 237,500 cubic yards with 57,000 cubic yards needed along the Nixon Channel shoreline resulting in a total beach fill volume of 294,500 cubic yards for Alternative 5D. Fill volumes would be the same for both the 2006 and 2012 conditions. Placement volumes and design berm widths for the ocean shoreline beach fill are provided in Table 9.8 with total volumes for the fill given in Table 9.9. Alternative 5D does not include an artificial dune in the sandbag area.

Table 9.8 Alternative 5D beach fill placement volumes and design berm widths.

Shoreline Segment (Baseline Stations)	Placement Volume (cy/lf)	Design Berm Width (ft)
60+00 to 70+00 (transition	0 to 20	0 to 17
70+00 to 77+50	20	17
77+50 to 80+00 (transition)	20 to 80	17 to 69
80+00 to 105+00 (terminal groin)	80	69

9.4.2 Alternative 5D Plan Formulation

Two terminal groin lengths were evaluated for Alternative 5D, one having the same length as Alternative 5C (1,300 feet) and the other 200-feet longer (1,500 feet). Based on the Delft3D model results, discussed below, volume losses from the beach fill with the 1,300-foot terminal groin occurred rather rapidly with only 6% of the fill placed above the -6-foot NAVD depth contour remaining at the end of the 5-year simulation. Over the whole active profile, that is from the berm crest seaward to the depth of closure (-24 ft NAVD), the entire fill was removed by the end of year 3. For the 1,500-foot structure and the same beach fill design as used in the evaluation of the 1,300-foot structure, the Delft3D model indicated the longer terminal groin was able to retain 27.5% of the fill placed above the -6-foot NAVD depth contour through year 5 of the simulation. The improved performance of the fill, particularly above the -6 foot NAVD depth contour, resulted in the selection of the 1,500-foot terminal groin for Alternative 5D.

The 1,500-foot terminal groin would include a 995-foot shore anchorage section and a seaward section that would project 505 feet seaward of the 2007 mean high water shoreline. The shore anchorage section would be constructed with either steel or concrete sheet pile while the seaward section would be of rubblemound construction. The landward 100 feet of the shore anchorage section would have a 10-foot wide stone scour protection apron on both sides.

The material to construct the beach fills would be obtained from maintenance of the previously permitted area in Nixon Channel. The plan layout for Alternative 5D is shown in Figure 9.21 with typical profiles of the ocean shoreline beach fill shown in Figures 9.22 and 9.23.

TABLE 9-9
OCEANFRONT BEACH DISPOSAL AREA
ALTERNATIVE 5D
FIGURE EIGHT ISLAND / RICH INLET, NC

Profile Line	Fill Length (feet)	Fill Distribution CY/LF	Total Volume CY
60+00	1,000	0	0
70+00	250	20	10,000
72+50	250	20	5,000
75+00	250	20	5,000
77+50	250	20	5,000
80+00	250	80	12,500
82+50	250	80	20,000
85+00	250	80	20,000
87+50	250	80	20,000
90+00	250	80	20,000
92+50	250	80	20,000
95+00	250	80	20,000
97+50	250	80	20,000
100+00	250	80	20,000
102+50	250	80	20,000
105+00	250	80	20,000
TOTAL	4,500		237,500

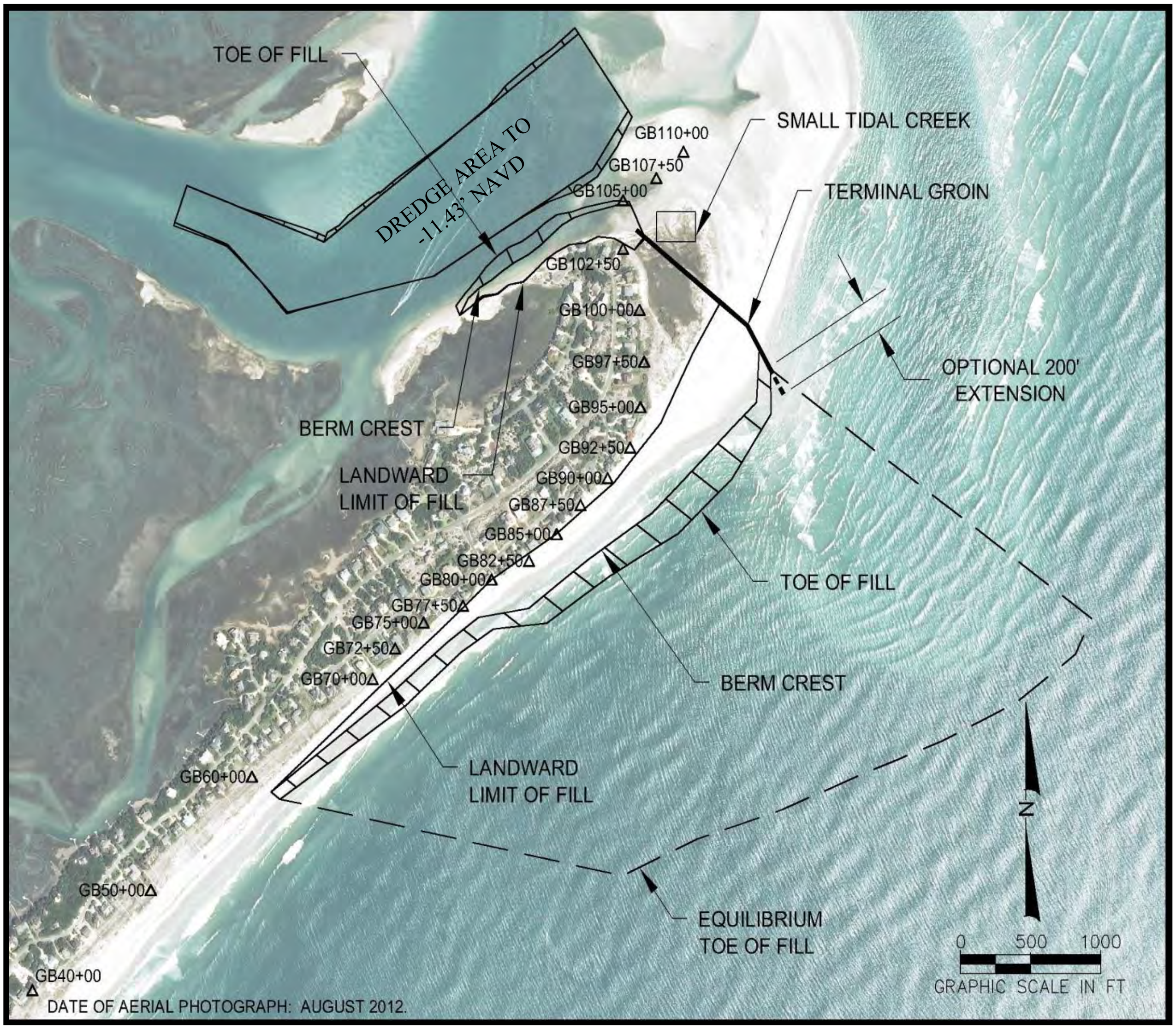


FIGURE 9-21: Alternative 5D Dredging and Groin Option and Beach Fill Layout.

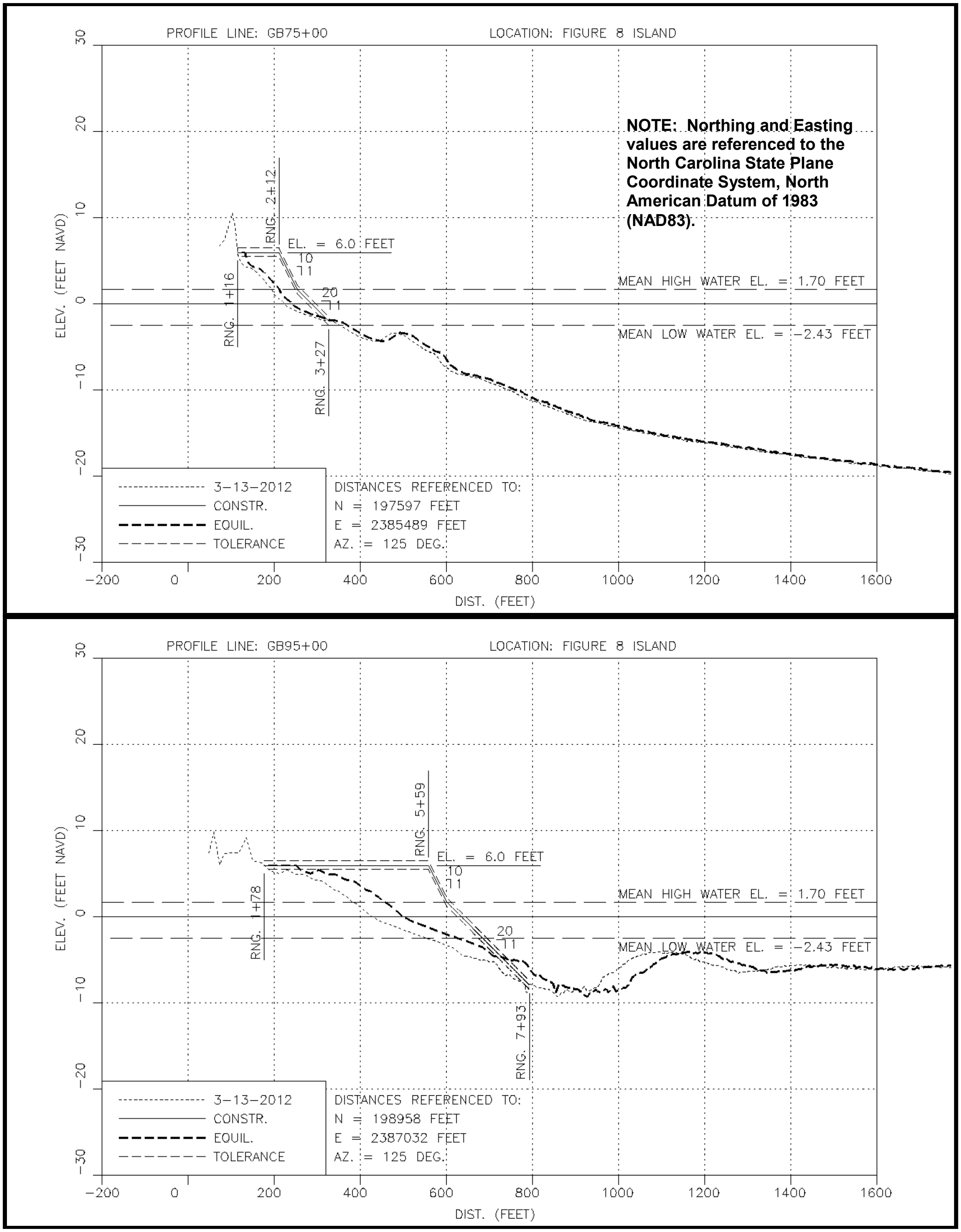


FIGURE 9-22: Representative Cross-Sections along the Oceanfront Beach Fill Area, Alternative 5D.

9.4.3 Structural Design of the Terminal Groin.

The following description of the design of the terminal groin for Alternative 5D, the Applicant's Preferred Alternative, is based on preliminary design considerations and the latest survey information which are subject to change during the preparation of detailed plans and specifications. However, the size of the structures footprint and the required construction corridor presented below are representative of the final design for Alternative 5D.

The total length of the Alternative 5D terminal groin would be 1,500 feet of which only 505 feet would project seaward of the 2007 mean high water shoreline position. The landward 995 feet of the structure would be constructed with sheet pile, either steel or concrete, and would have a top elevation of just below the elevation of the existing ground. In general, the top elevation of the sheet pile will vary from +0.5 feet NAVD for the first 200 feet on the landward end to +1.5 ft NAVD over the remaining 795 feet. The sheet pile section will begin near the Nixon Channel shoreline and end near the position of the 2007 mean high water line. To account for possible scour around the landward end of the shore anchorage section, a 10-foot wide rubble scour protection apron would be installed along both sides of the landward most 100 feet of the anchorage section. The toe apron would be installed at a depth of approximately -2 ft NAVD and would require the excavation of approximately 300 cubic yards. Material excavated for the toe apron would be used to bury the toe protection stone following placement.

A total of 22,200 square feet of sheet pile would be required for the shore anchorage section. Note the amount of sheet pile could vary based on the final design characteristics. The present preliminary design for the sheet pile would penetrate to a depth of -21 feet NAVD. Detailed design considerations would include soil borings along the alignment of the proposed structure to obtain soil characteristics as well as assumptions with regard to possible future positions of the south shoulder of Rich Inlet relative to the sheet piles. The assumed position of the south shoulder of the inlet would dictate soil and water loadings on the piles and hence dictate how deep the piles would need to be driven for stability.

The seaward 505 feet of the structure would be constructed with loose armor stone placed on top of a layer of foundation stone comprised of quarry-run material (generally 12-inch diameter or less) or possibly a wire-mesh mat filled with similar size stone. The top elevation of the rubblemound structure would not exceed +6.0 feet NAVD which is an elevation roughly equivalent to the elevation of the natural beach berm near Rich Inlet. Again, the final design of the rubblemound portion of the structure is subject to change given conditions near the time of actual construction.

The loose nature of the armor stone would be designed to facilitate the movement of littoral material through the structure. A profile of the terminal groin is shown on Figure 9-23. Figure 9-23 shows both the April 2007 profile for baseline station 105+00, which was used as a basis for the terminal groin design, and the March 2012 profile that reflects the accretion that has occurred on the north end of Figure Eight Island since 2010. A typical cross-section of the rubblemound portion is shown in Figure 9-24.

As shown on Figure 9-24, the rubblemound section of the structure would include a 25-foot wide scour protection mat along the inlet side to protect the structure against undermining should the channel through Rich Inlet migrate next to the structure. Based on this preliminary design, construction of the rubblemound portion of the terminal groin would require around 8,500 tons of armor stone, 2,900 tons of bedding stone, and 200 tons for the scour apron around the landward end of the shore anchorage section for a total of 11,600 tons of stone. Construction of the seaward portion of the terminal groin would require excavation of approximately 7,900 cubic yards to create an 82-foot wide trench to a depth of -5.5 ft NAVD. The excavated material would be returned to the trench, partially burying the structure, once construction is complete.

The concept design for the terminal groin presented here is intended to allow littoral sand transport to move over, around, and through the structure once the accretion fillet south of the terminal groin is artificially filled. This would be accomplished by setting the maximum crest elevation of the terminal groin to +6 feet NAVD, which is an elevation slightly above the natural berm elevation, and constructing the structure with large voids between adjacent stones. The relatively short length of the terminal groin seaward of the 2007 mean high water shoreline would also facilitate movement of sediment around the seaward end of the structure. The seaward 200 feet to 300 feet of the structure should be visible at all stages of the tide from both sides of the structure, however, the remaining portions of the structure would be buried below ground and would not be visible from the south side. While the north side of the rubblemound section may project a foot or two above ground, during normal weather conditions, wind-blown sand is expected to accumulate along the north side of the structure partially burying the exposed section.

The shore anchorage section would be completely below ground and would not be visible. The only time the shore anchorage section could be visible would be in the unlikely event the entire north end of the island is eroded back to the position of the sheet piles.

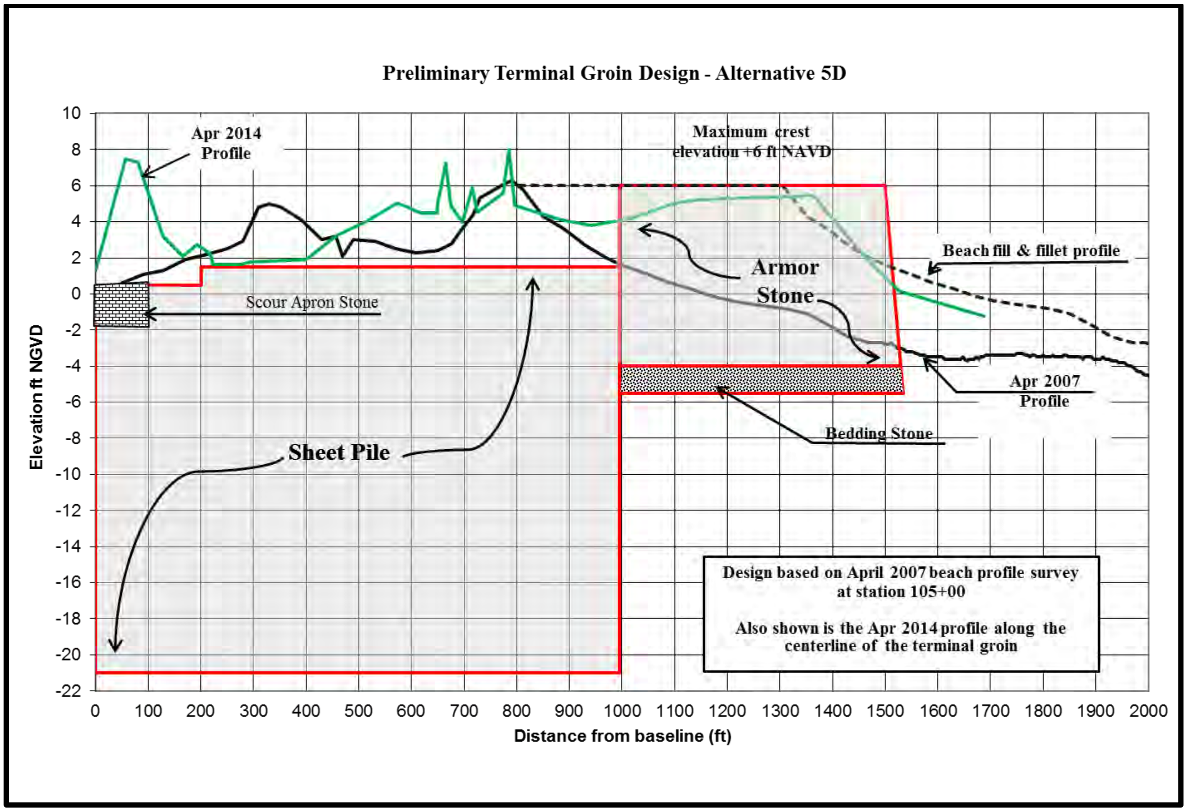


Figure 9-23. Profile of terminal groin for Alternative 5D.

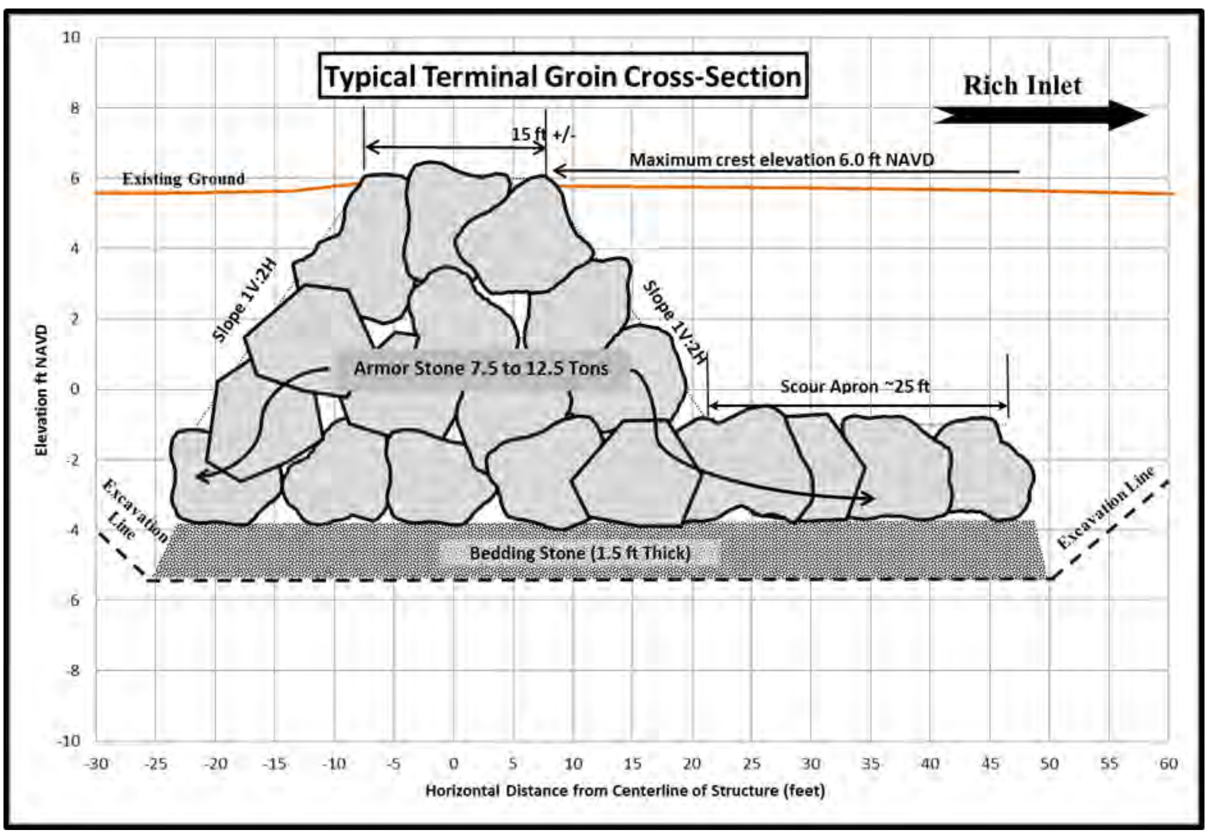


Figure 9-24. Typical terminal groin cross-section.

10.0 PROJECT PERFORMANCE DURING STORMS

Beach erosion and shoreline recession occurs during severe storm events. The performance of the project based on the Delft3D model results later in this report is based on wave cases which utilized records of both average and above-average waves between 1999 and 2007. The "second opinion" of project performance based on the GENESIS model results utilizes wave records given at 3 hour intervals between 2000 and 2009. In those results, erosion due to longshore transport variations is estimated for both average and above-average waves explicitly. Further details regarding both the Delft3D and GENESIS models appear in the sections to follow.

11.0 LONG-TERM PROJECT PERFORMANCE – DELFT3D MODEL STUDY

To evaluate the long-term performance of the various alternatives in Section 9.0, this study utilizes an advanced 2D/3D integrated modeling environment known as Delft3D (WL | Delft, 2005). Delft3D consists of two models that run together to estimate wave transformation, currents, water level changes, sediment transport, erosion, and deposition. Waves in Delft3D are simulated using SWAN (Simulating Waves Nearshore), an advanced wave transformation model that simulates breaking, shoaling, refraction, diffraction, wind stress, and bottom friction. Delft3DFLOW simulates currents, water level changes, erosion, sediment transport, erosion, and deposition based on the forcing of the tides, storm surges, waves, and winds. Delft3DFLOW and SWAN run simultaneously, exchanging wave, water level, current, and bottom depth values. Delft3D can simulate relevant coastal processes over short-term (days-storms) or long term (seasons-years) time scales.

11.1 Wave Model Calibration

Waves in the Delft3D modeling package were simulated using SWAN. Wave transformation estimates within the model utilized a spectral wave approach that treated each observed wave as a superposition of individual waves with varying frequencies and periods.

The primary inputs to the SWAN model were the bottom bathymetry, the time-dependent water levels, and the offshore waves. Additional inputs were the wave breaking coefficients, the bottom roughness scale, the diffraction coefficients, and the non-linear triad coefficients that governed wind effects. The parameter with the largest effect on the transformed wave field was the bottom roughness scale, which governed the bottom friction. Accordingly, calibration of the SWAN model was performed by examining the effect of bottom roughness on the nearshore wave height.

Several wave gages have been deployed in the region at various times, albeit separated by large distances (~ 20 to 50 miles) (Figure 11-1). Thus, the SWAN model was calibrated on a regional basis. Calibration runs were based on an easterly wave event at offshore wave gage LEJ3 (Figure 11-1) in July 2006. Concurrent wave measurements were taken at nearshore wave gage ILM1 (Figure 11-1), located on Johnny Mercer's Pier in Wrightsville Beach. The offshore waves, water levels, and wind velocities used in the model appear in Figures 11-2 to 11-4. Given the information that was available, wind velocities and water levels were assumed to be uniform over the model grid.

Calibration runs were conducted using bottom roughness scale from 0.00075 m to 0.05 m (0.2 inches to 13 inches). A reasonable agreement between the simulated and observed wave heights at gage ILM1 was achieved with a bottom roughness scale of 0.01 m (2.5 inches). The average difference between the observed and simulated wave height at gage ILM1 was -0.1 feet, with a root-mean-square difference of 0.4 feet. Matching the nearshore wave direction was more difficult. Simulated waves at gage ILM1 were more oblique to the shoreline than the observed waves. This occurred due to the tendency of the model to refract the waves parallel to the shoreline, as shown in Figure 11-5. The effect was more pronounced in the second half of the run, when there was a significant difference between wave periods at gages LEJ3 and ILM1. As shown in Figure 11-4, the prevailing winds at LEJ3 were from the northeast during the calibration period. Thus, the wind direction, combined with the bathymetry, had a large influence on the simulated wave direction. Based on the available information, a uniform wind velocity was assumed over the model grid. However, given the 48 mile distance between gages LEJ3 and ILM1, local variations in the wind speed and direction were likely during the calibration period. Overall, differences between the simulated and measured wave direction at gage ILM1 were probably due to the assumption of uniform winds.

Verification runs were based on a southerly wave event at offshore wave gage 41013 (Figure 11-1) in June 2004. Typical wave patterns during this event appear in Figure 11-6. Concurrent wave measurements were taken at nearshore wave gage OB3M (Figure 11-1). The offshore waves, water levels, and wind velocities used in the model appear in Figures 11-7 to 11-9. Similar to the calibration, wind velocities and water levels were assumed to be uniform over the model grid. The bottom roughness scale was set to 0.01 m (2.5 inches). Overall, agreement between the model results and the observations at OB3M was good. The average difference between the observed and simulated wave height at gage OB3M was +0.4 feet, with a root-mean-square difference of 0.6 feet. The average difference between the observed and simulated wave direction at gage OB3M was +1 degree. The verification showed that the SWAN model was able to accurately estimate nearshore wave heights, with reasonable approximations of the nearshore wave direction given a relatively uniform wind field. Based on the results in Figures 11-2, 11-3, 11-7 and 11-8, the calibrated SWAN model was judged to be suitable for estimating project performance.

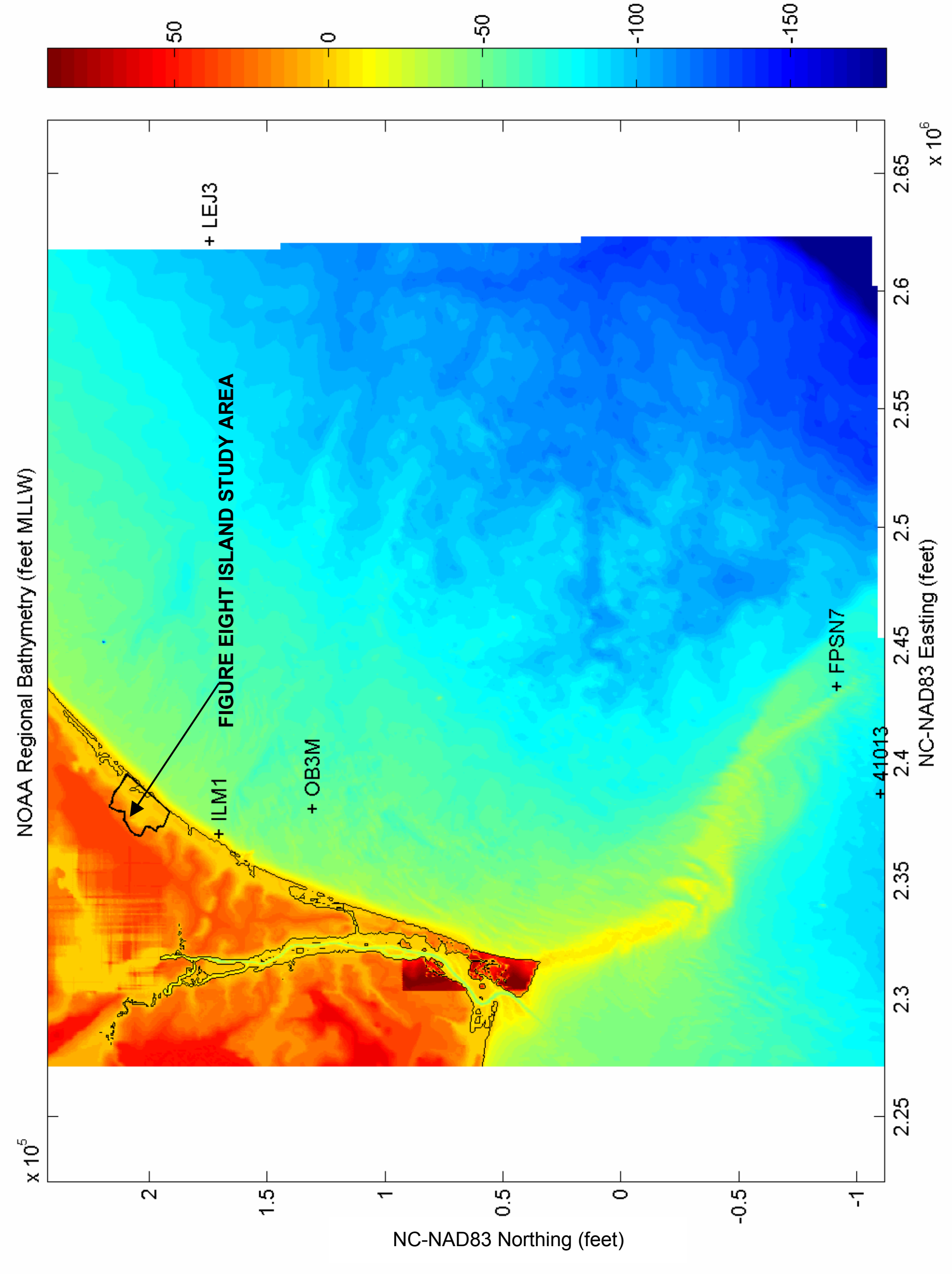


FIGURE 11-1: Wave Calibration Bathymetry based on NOAA (2006) Regional Grid, Figure Eight Island, NC.

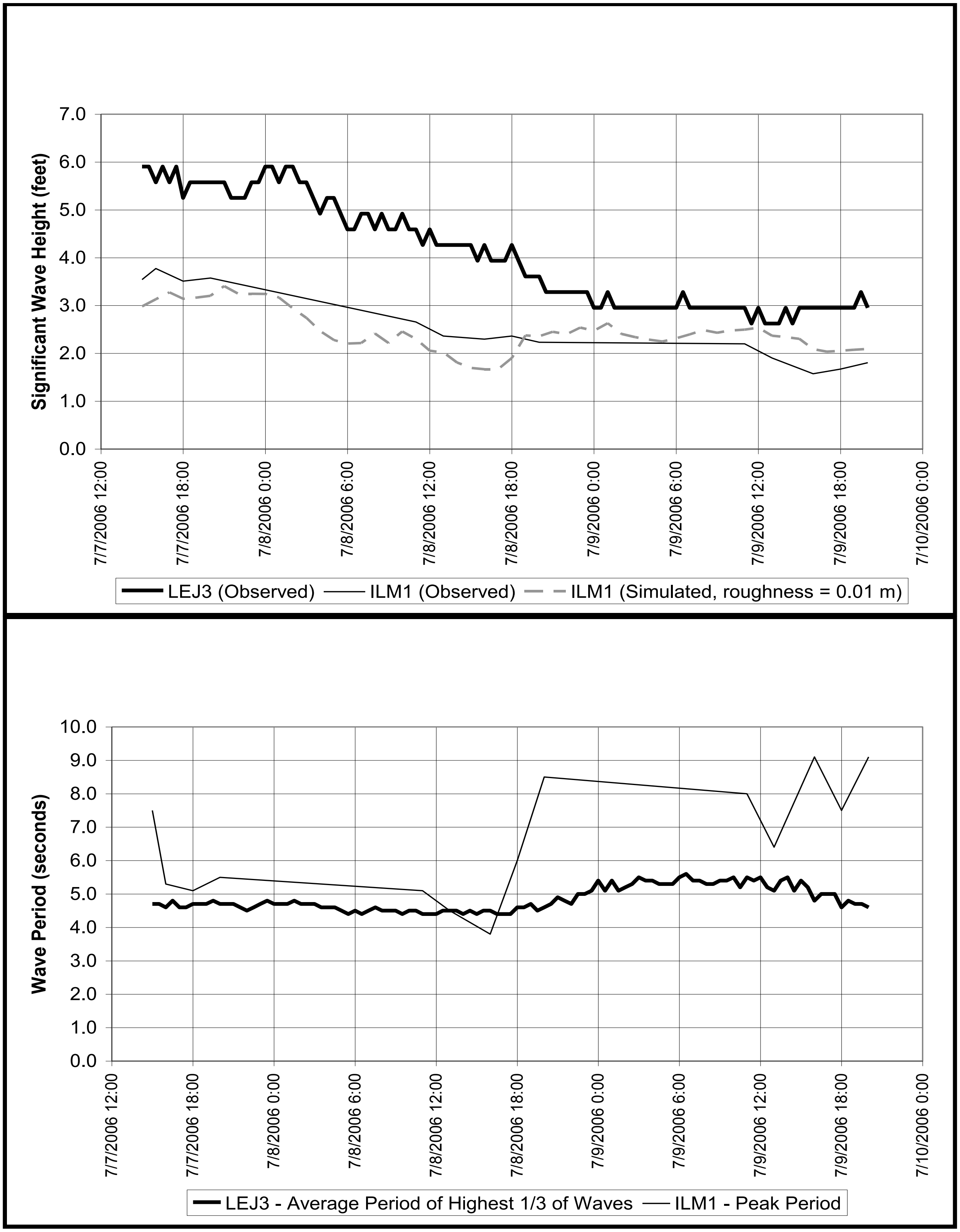
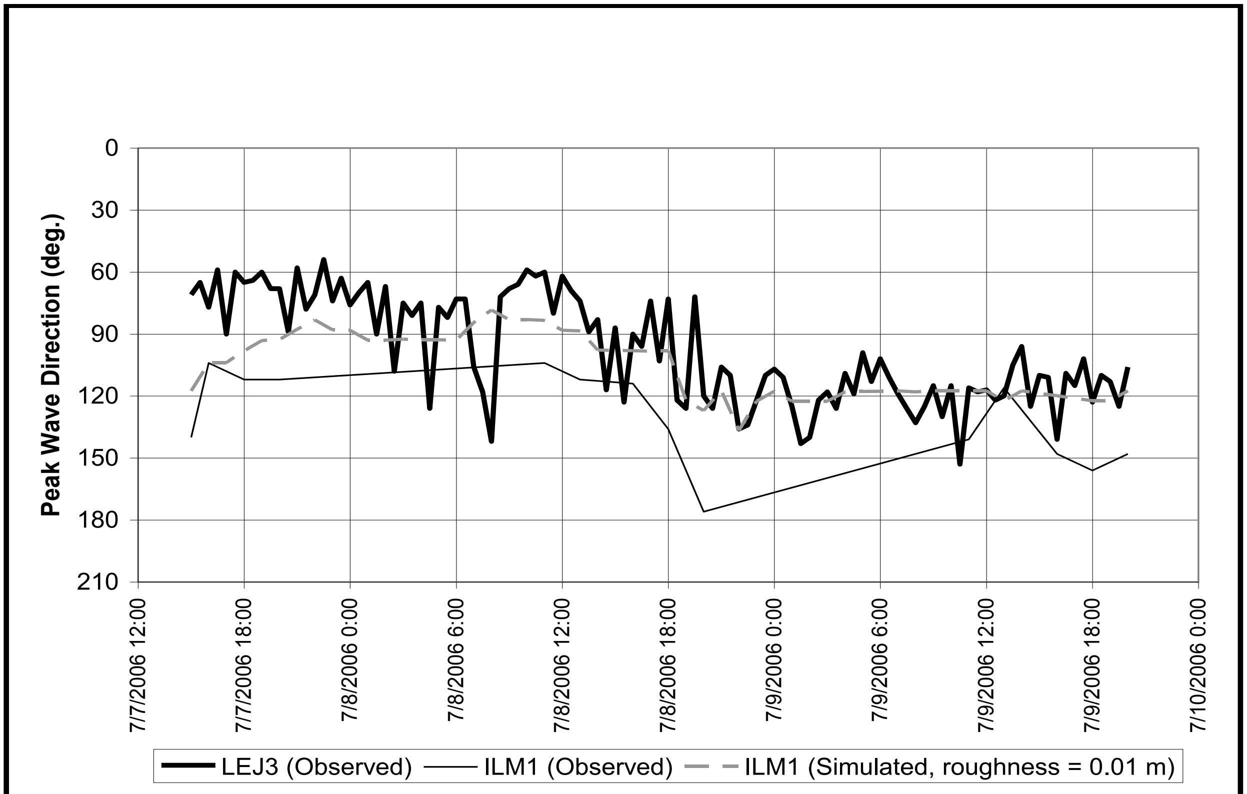


FIGURE 11-2: Delft3D-SWAN Calibration, Wave Height and Wave Period.



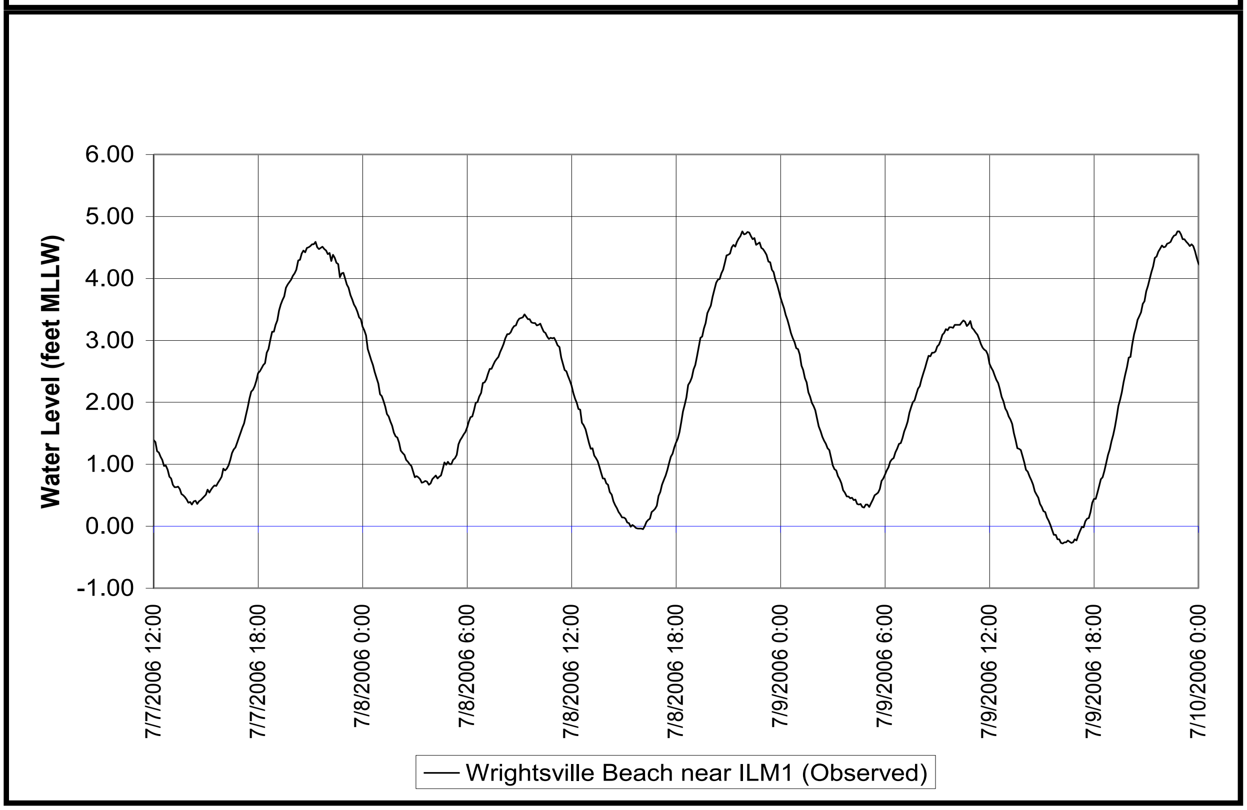


FIGURE 11-3: Delft3D-SWAN Calibration, Wave Direction and Water Level.

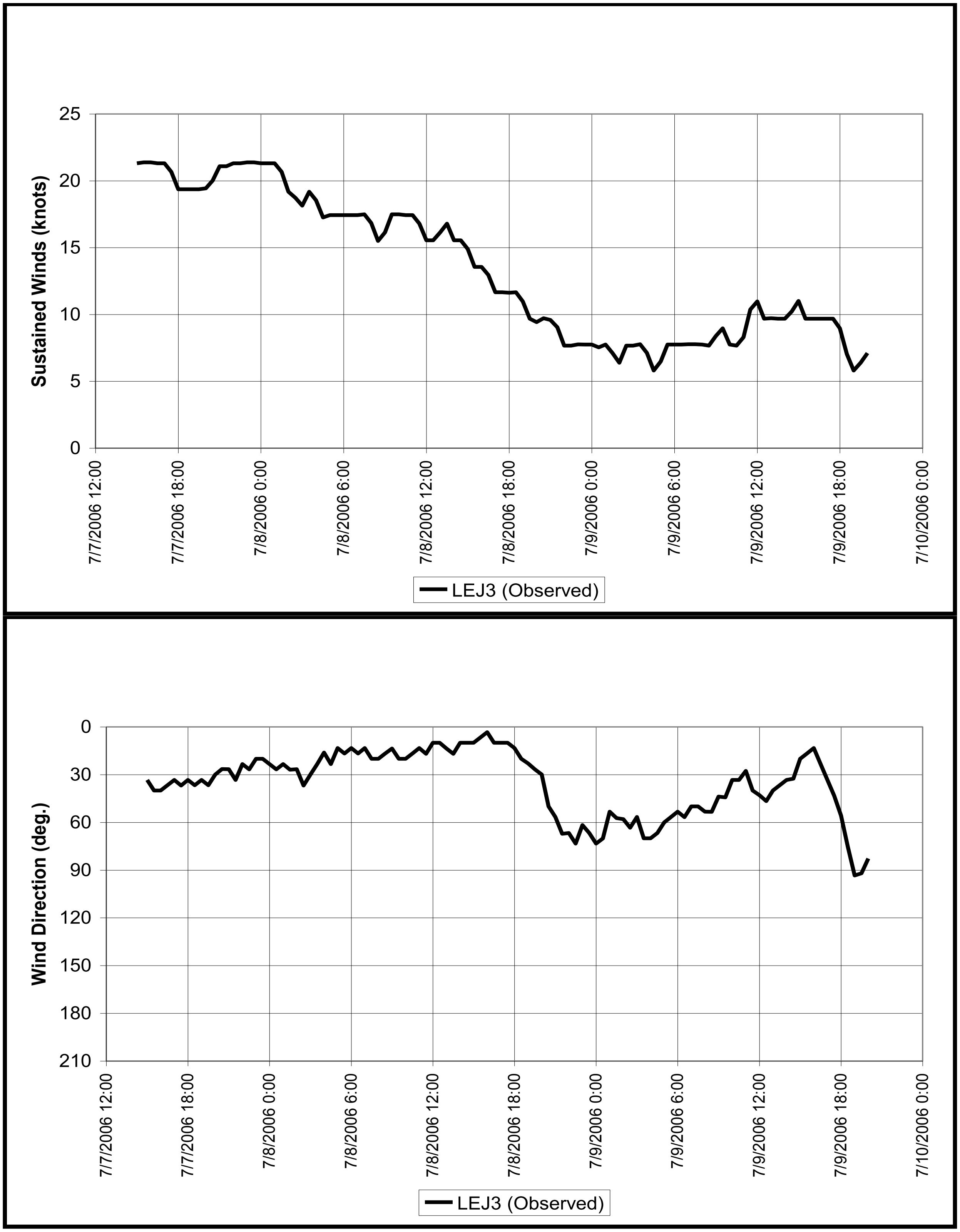


FIGURE 11-4: Delft3D-SWAN Calibration, Wind Velocity.

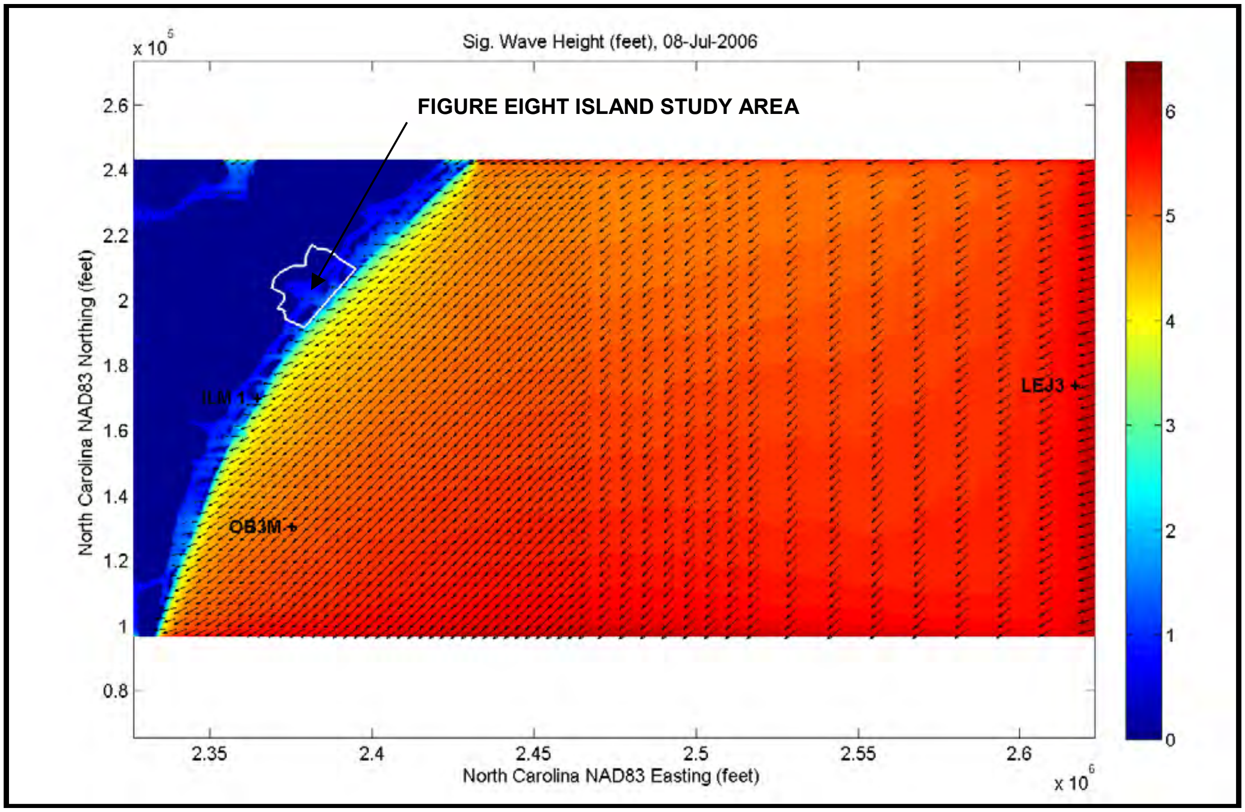


FIGURE 11-5: Typical Wave Calibration Results, Figure Eight Island, NC.

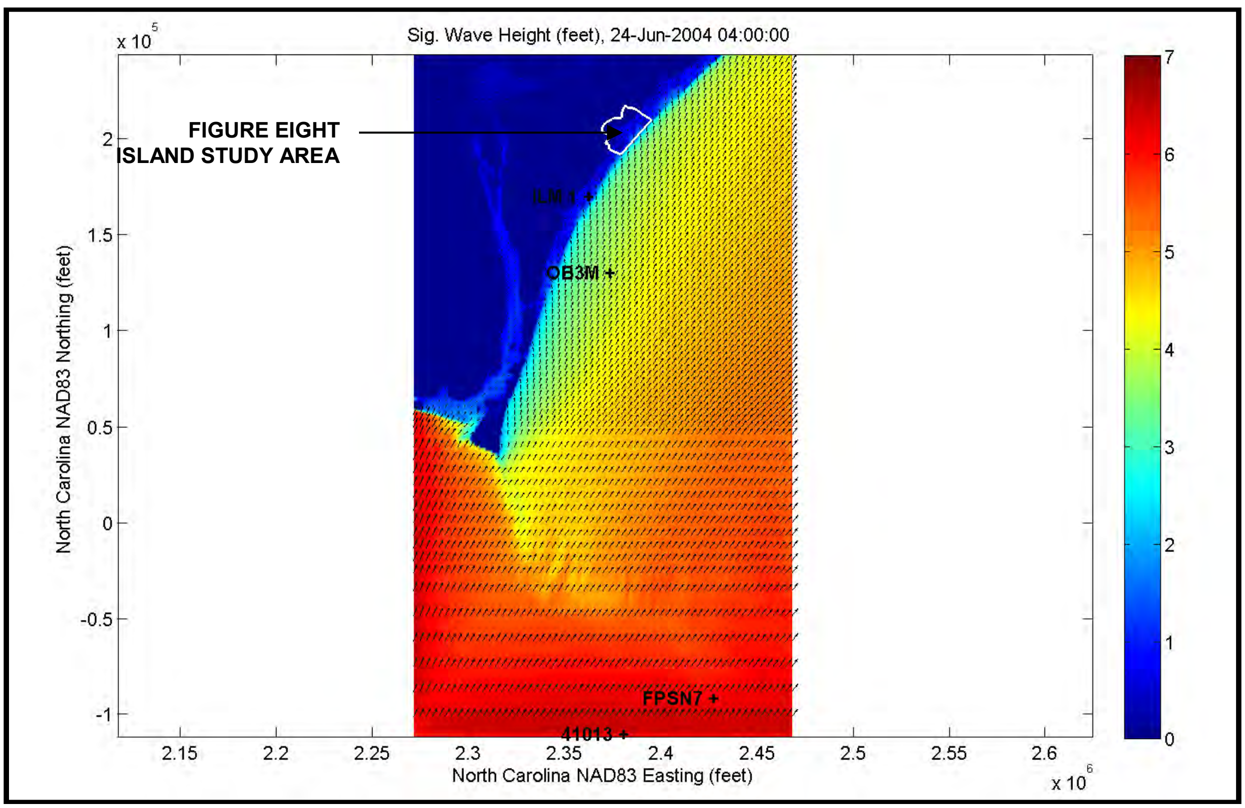


FIGURE 11-6: Typical Wave Verification Results, Figure Eight Island, NC.

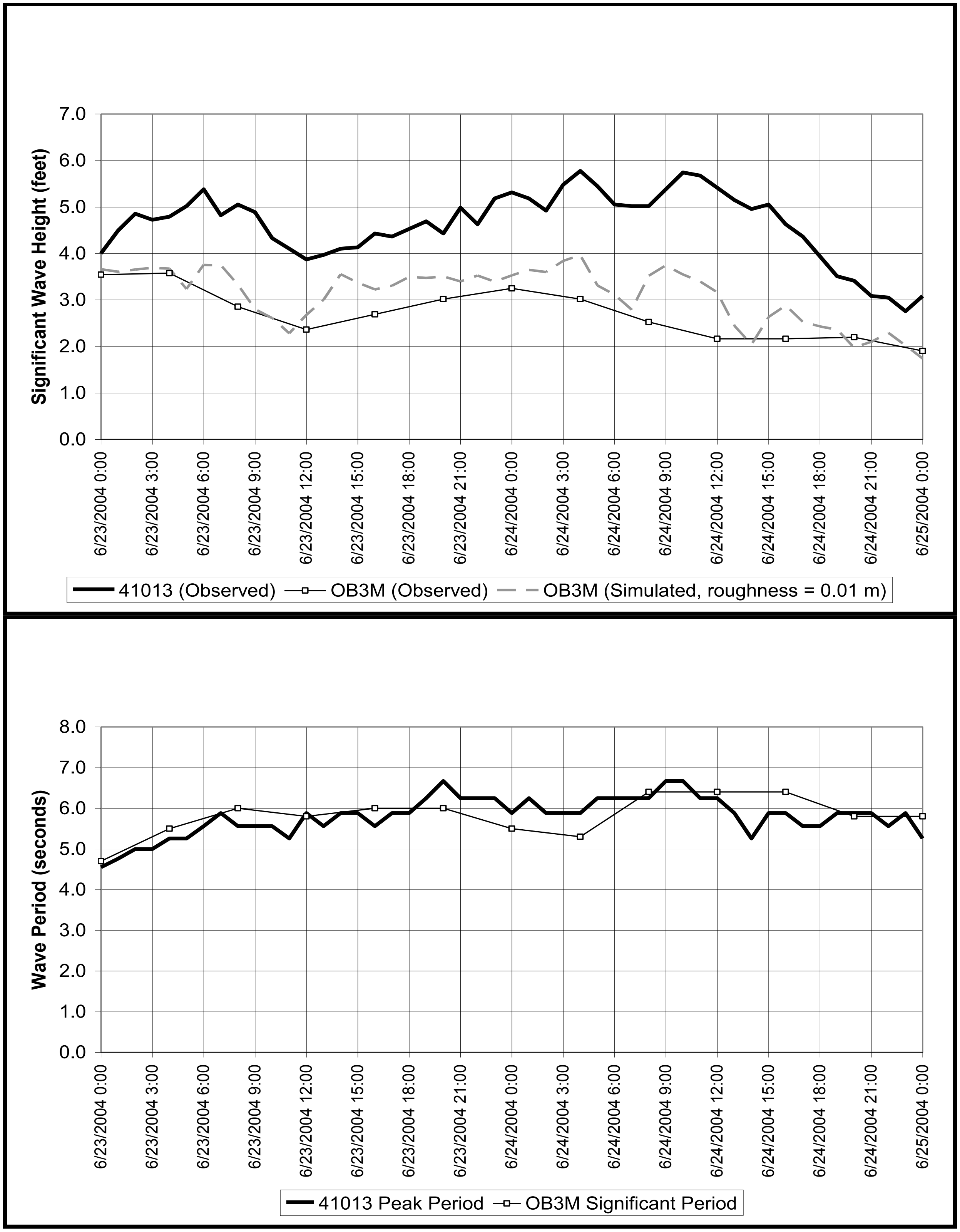


FIGURE 11-7: Delft3D-SWAN Verification, Wave Height and Wave Period.

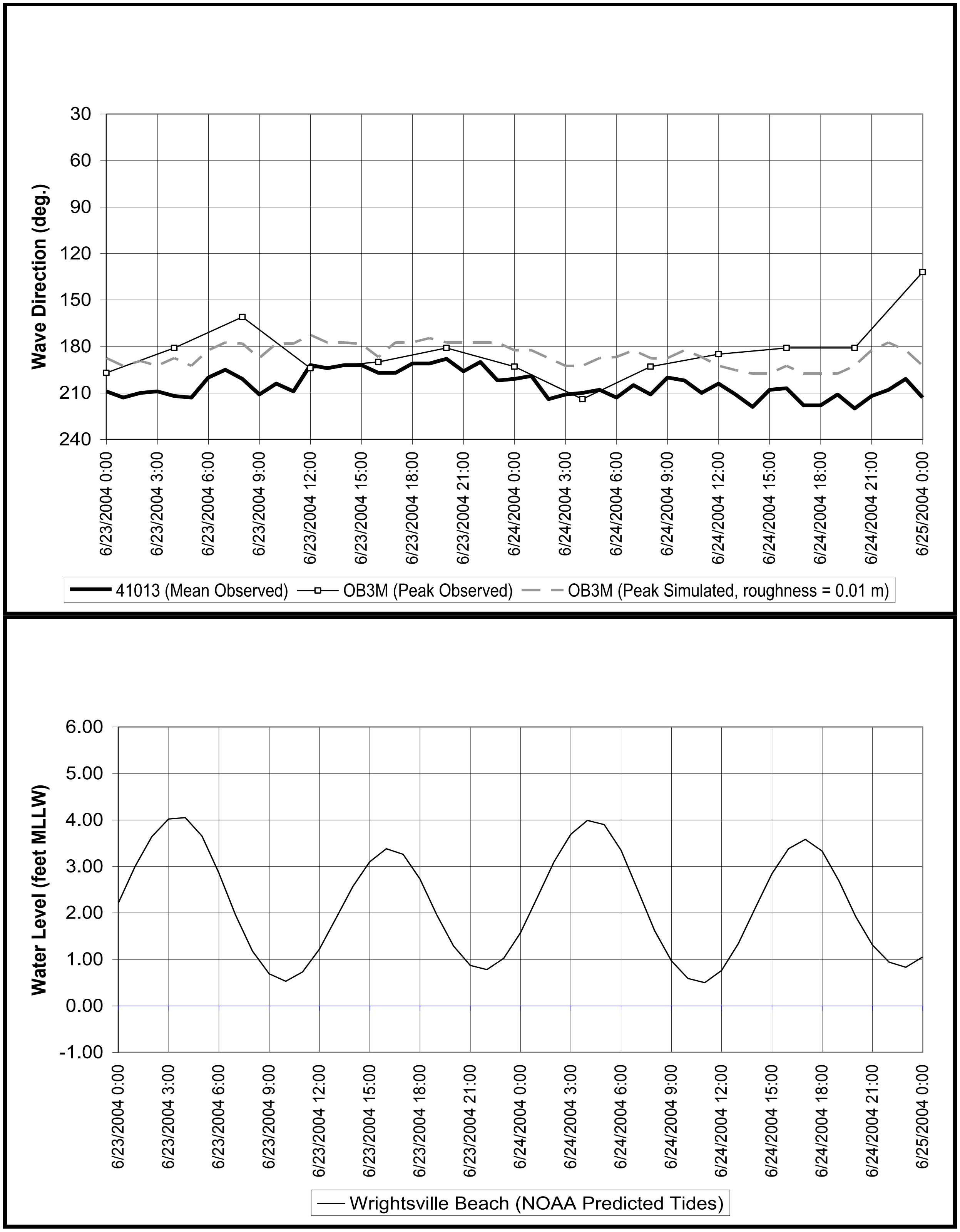
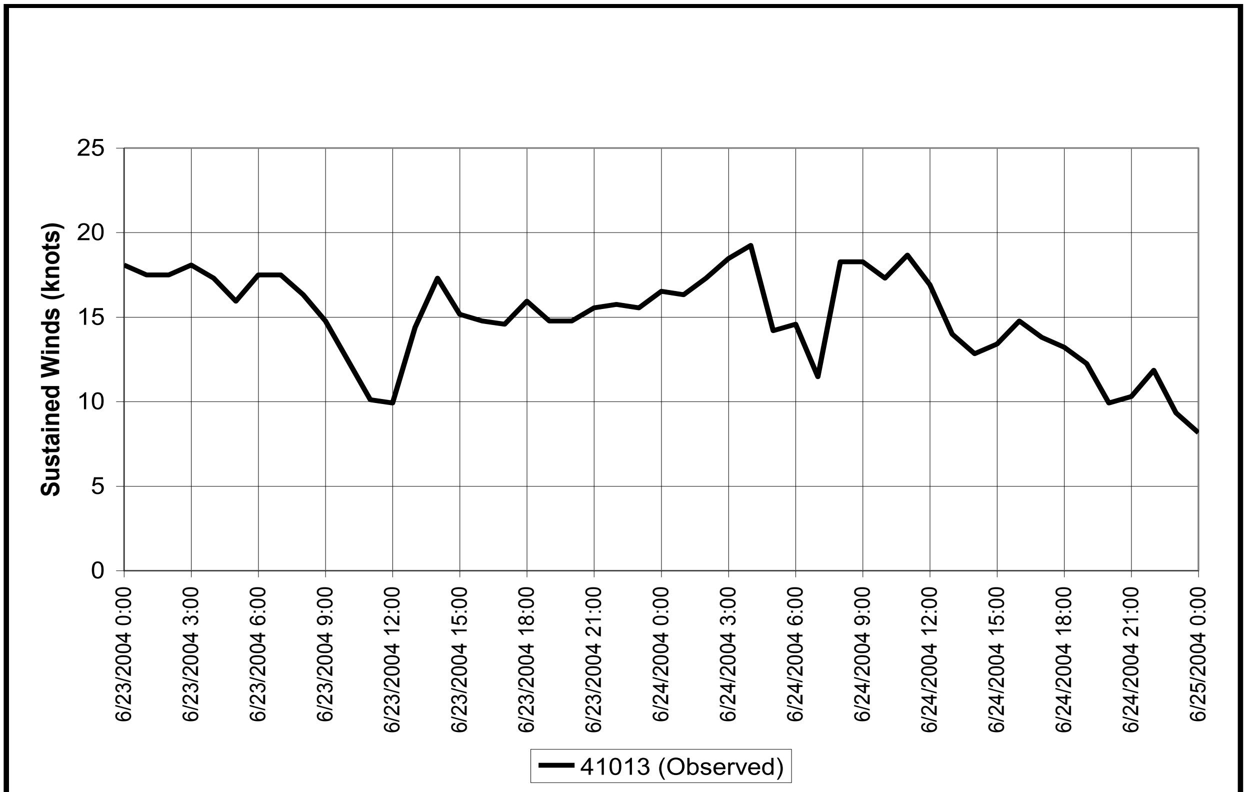


FIGURE 11-8: Delft3D-SWAN Verification, Wave Direction and Water Level.



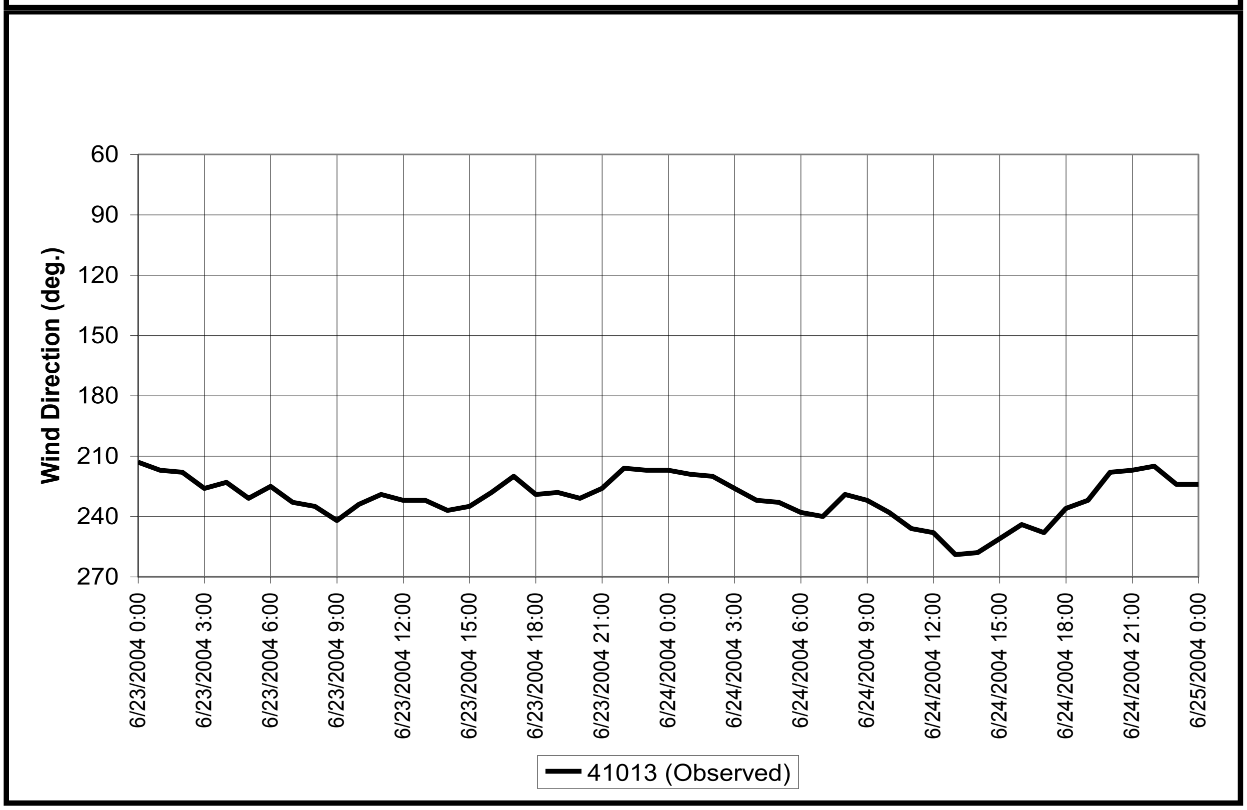


FIGURE 11-9: Delft3D-SWAN Verification, Wind Velocity.

11.2 Current and Water Level Calibration

11.2.1 Grids

Currents and water levels in the Delft3D modeling package were simulated using Delft3DFLOW. The model's currents and water levels were calibrated friction using a set of water level and current measurements provided by Gahagan & Bryant (2006) (see Section 4.3). Water levels were measured at seven (7) tide gages deployed May 25 - July 7, 2005, as shown in Figure 4-1. In addition, velocities were measured at three (3) locations on June 21, 2005 using boat-mounted Acoustic Doppler Current Profilers (ADCPs). Observed currents were reported by Gahagan & Bryant on a depth-averaged basis. The calibration run was performed using Delft3DFLOW in conjunction with SWAN, to account for the influence of both waves and tides.

Four grids were used in the flow calibration and subsequent model runs (Table 11-1 and Figures 11-10 to 11-16):

- Regional Wave Grid. The purpose of this grid was to simulate wave transformation over the region extending from Ocracoke, NC to Pawleys Island, SC. The offshore grid boundary generally followed the -500 foot NAVD depth contour. By simulating wave transformation over this area, it was possible to account for the influence of Cape Lookout and Cape Fear on the local wave patterns (Figures 11-10 through 11-12).

- Intermediate Wave Grid. The purpose of this grid (Figures 11-10, 11-11, and 11-13) was to provide more detailed wave information along the boundaries of the Local Wave Grid. This Intermediate Wave Grid extended from Surf City to Masonboro Island.

- Local Wave Grid. The purpose of this grid was to provide detailed wave information along the project area in shallow water. This grid extended from the midpoint of Hutaff Island to Mason Inlet. Wave transformation estimates along this grid were fed into the Delft3DFLOW model to estimate the wave-driven currents. Currents and water levels estimated by the Delft3DFLOW model were fed into the SWAN model to account for the influence of tidal currents and water level changes over this grid. Over the other two wave grids, tidal currents and water level changes were neglected by the SWAN model (Figures 11-10, 11-11, and 11-14).

- Flow Grid. This grid was utilized to estimate tidal currents and water level changes. Like the Local Wave Grid, this grid extended from Hutaff Island to Mason Inlet. However, to include all of the area drained by Rich Inlet, the grid was extended towards the west (Figures 11-15 and 11-16).

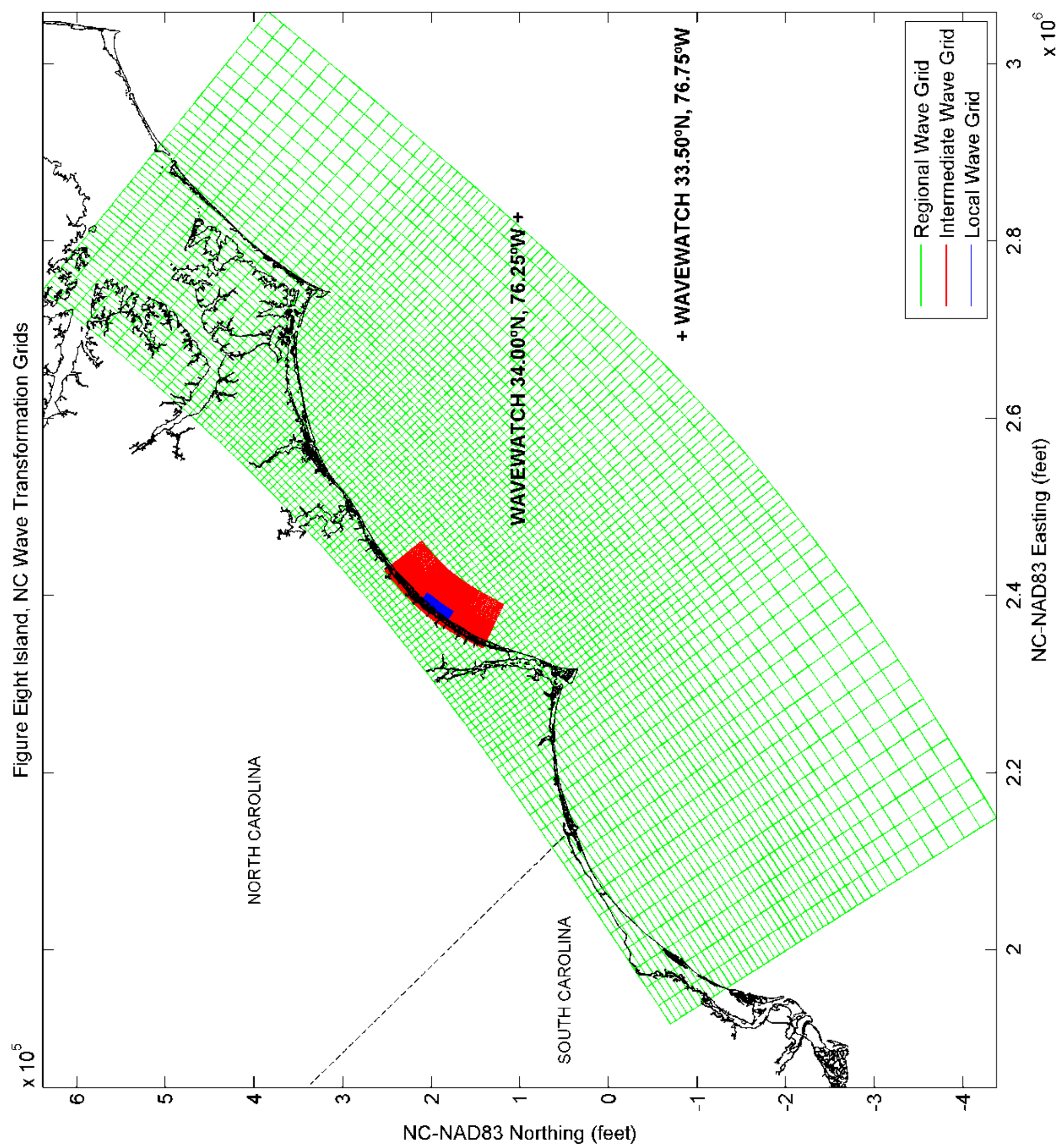


FIGURE 11-10: Wave Transformation Grids used in Delft3DFLOW Calibration and Subsequent Model Runs.

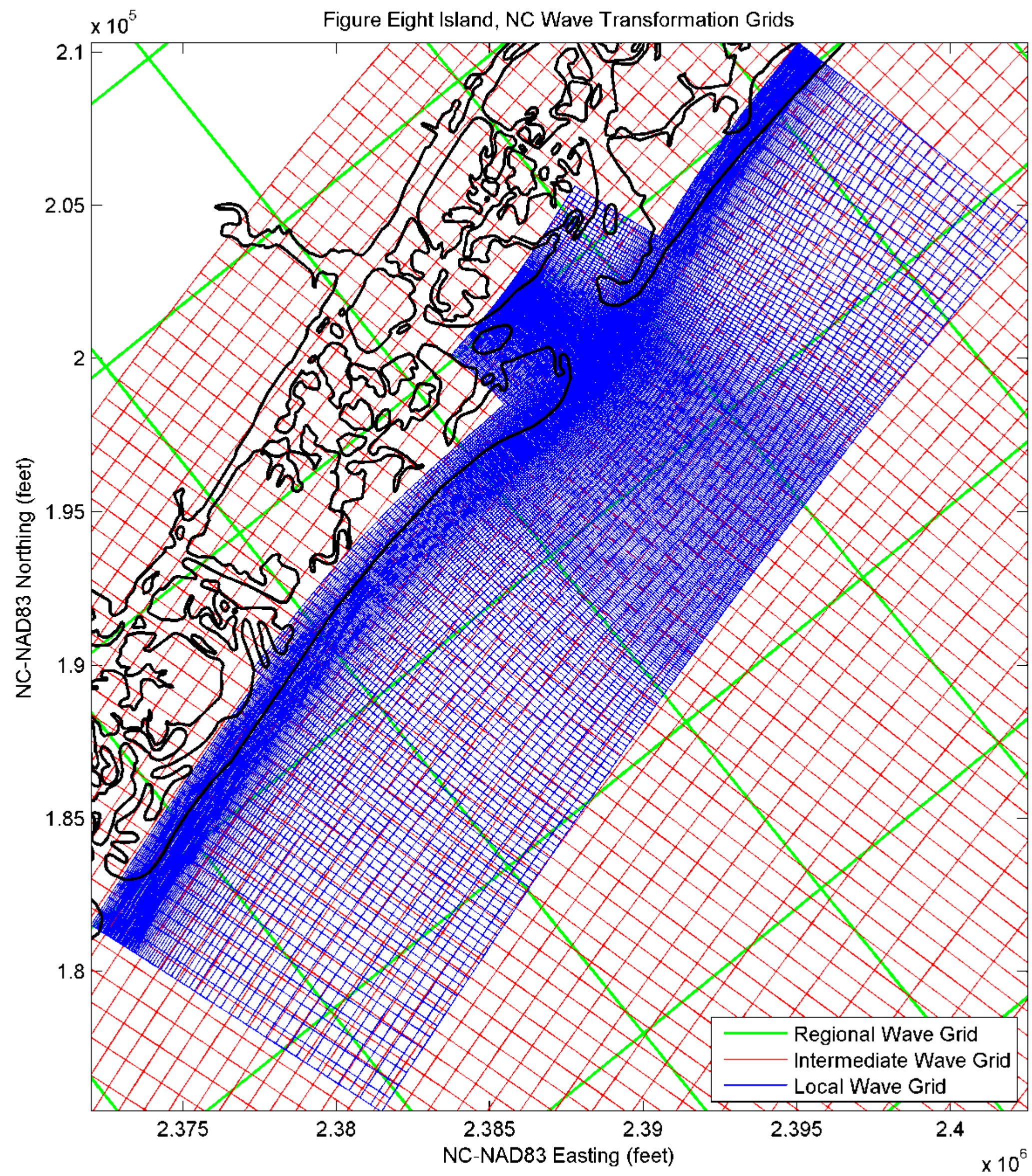


FIGURE 11-11: Wave Transformation Grids used in Delft3DFLOW Calibration and Subsequent Model Runs (closeup).

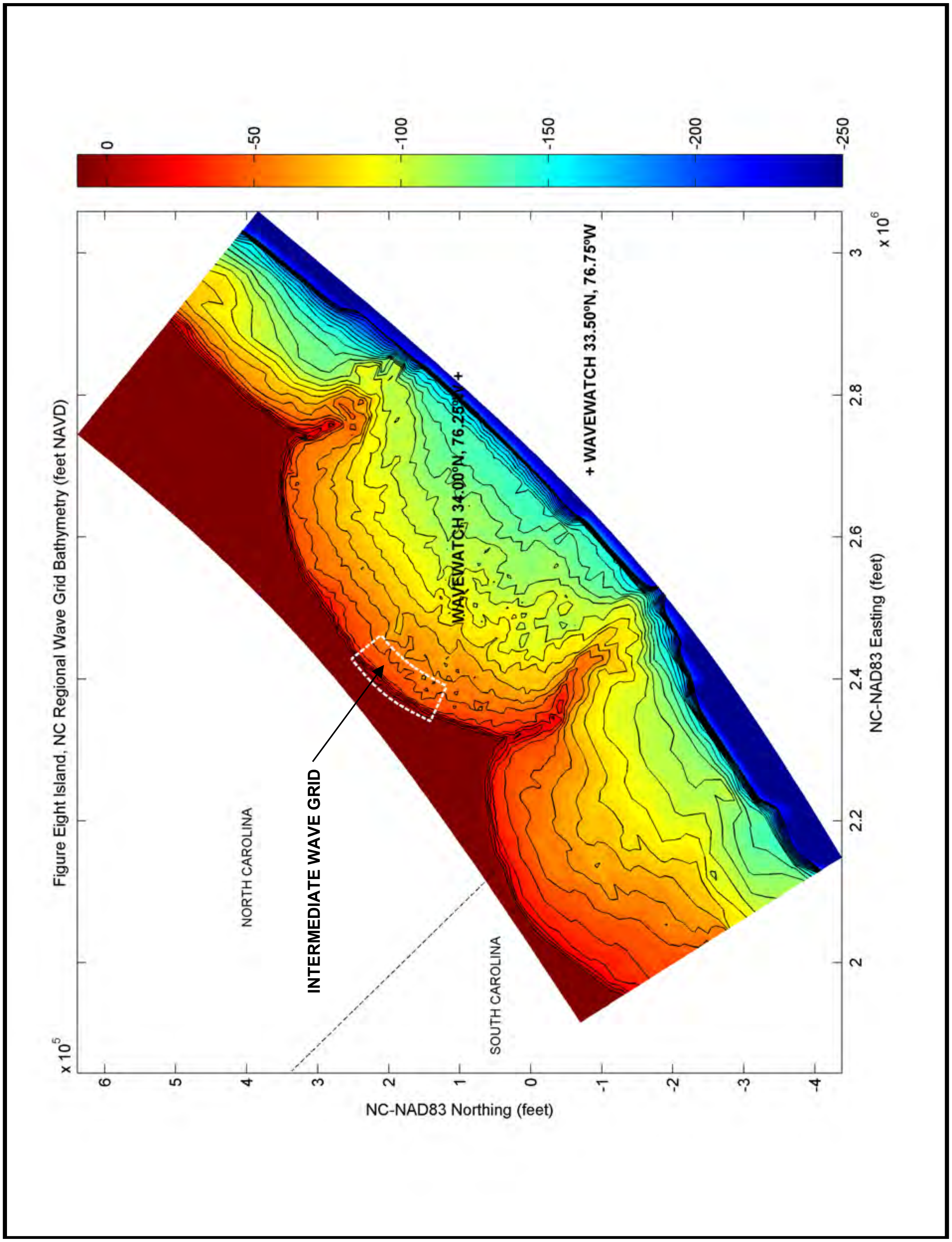


FIGURE 11-12: Bathymetry over the Regional Wave Grid.

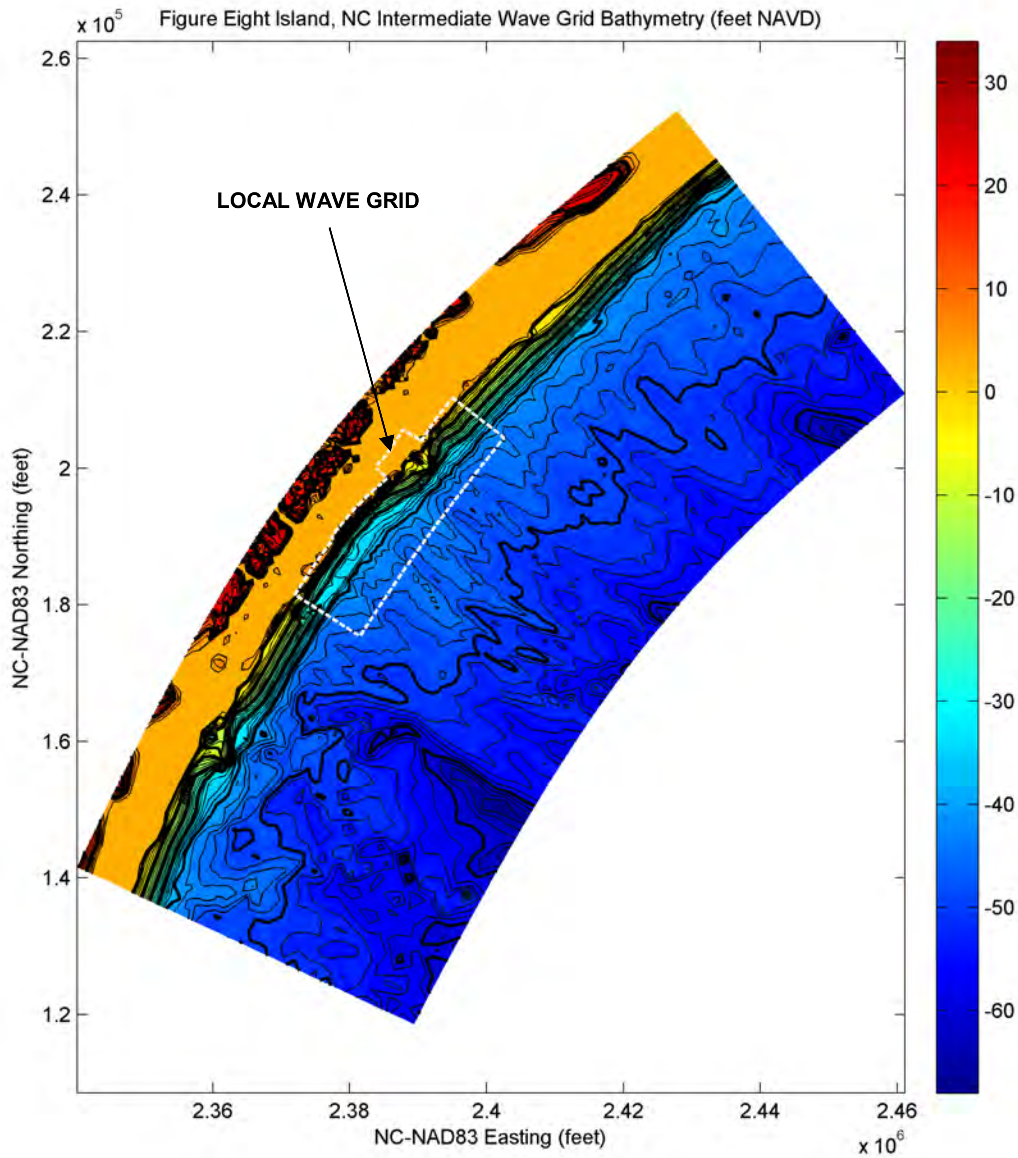


FIGURE 11-13: Bathymetry over the Intermediate Wave Grid.

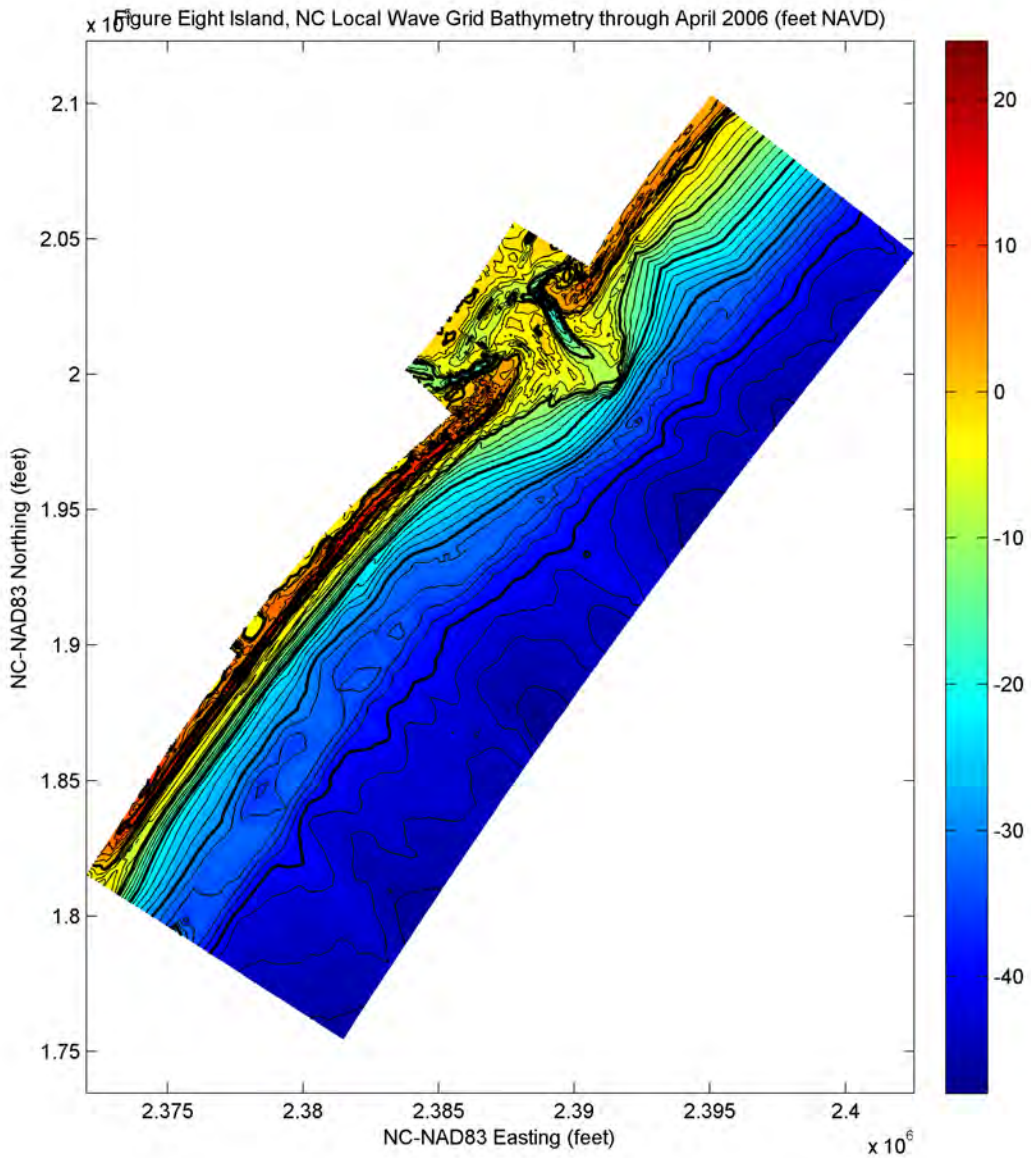


FIGURE 11-14: Bathymetry over the Local Wave Grid.

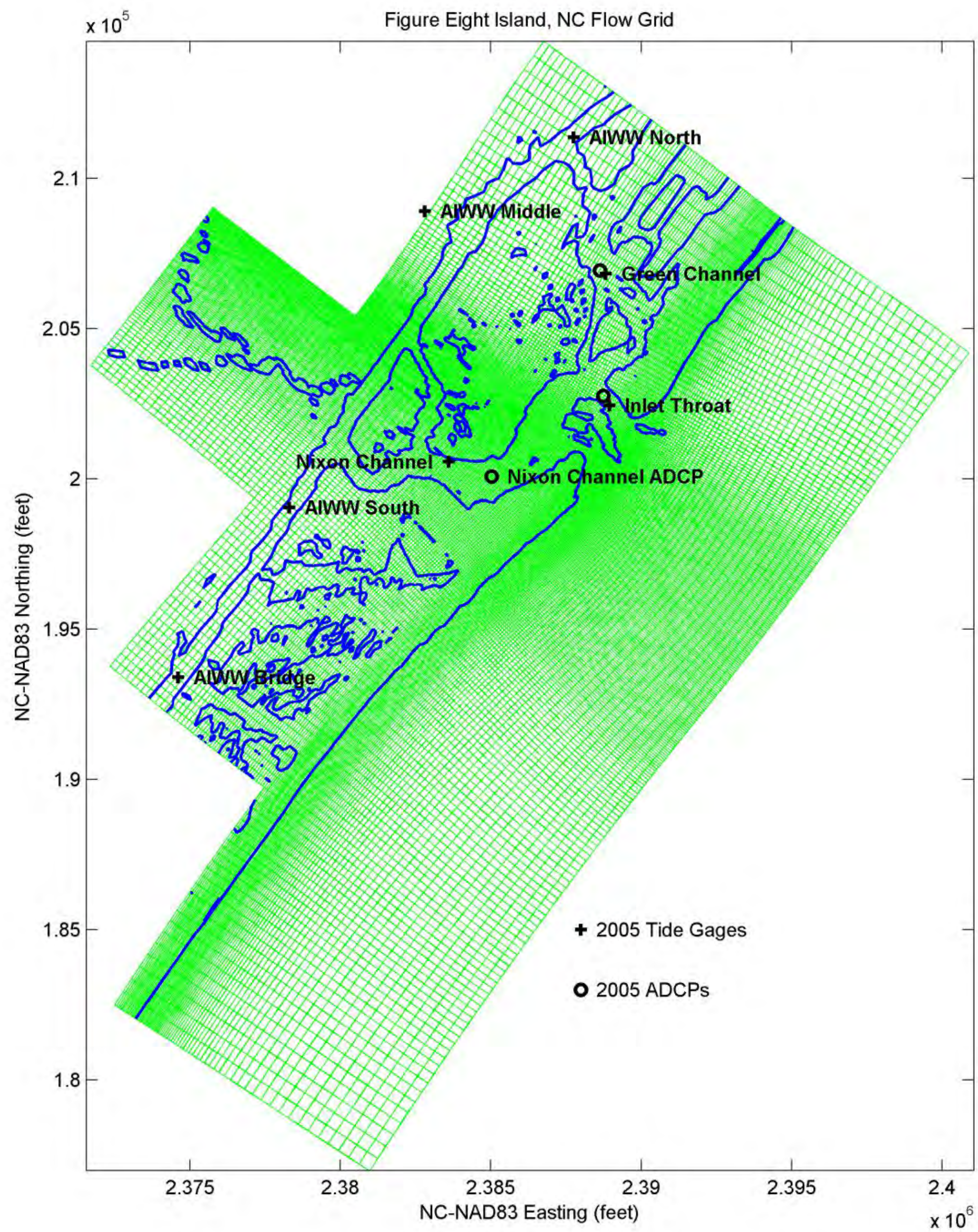


FIGURE 11-15: Flow Grid.

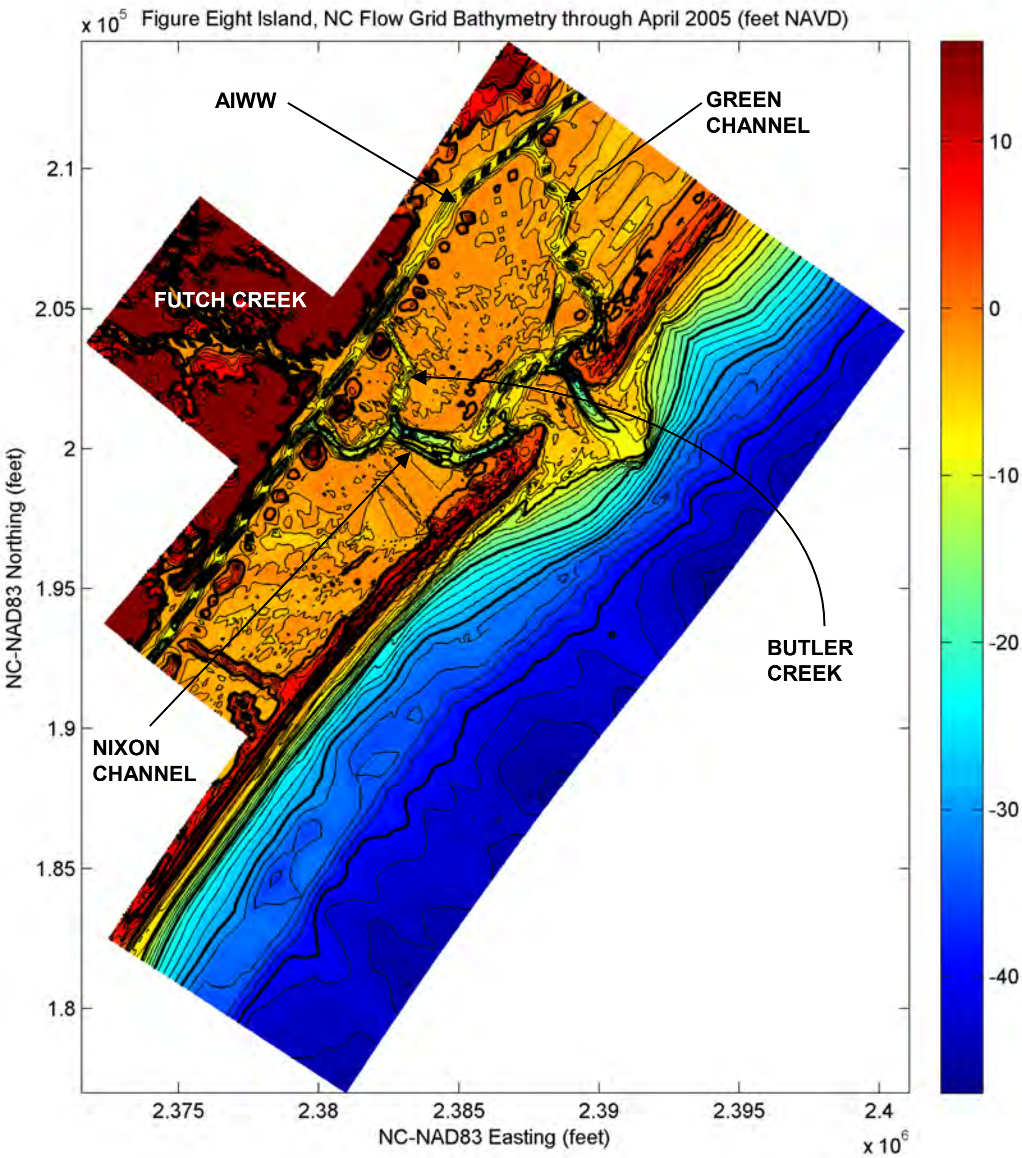


FIGURE 11-16: Bathymetry over the Flow Grid.

TABLE 11-1

GRIDS USED IN DELFT3D MODEL
FIGURE EIGHT ISLAND, NC

Grid	Longshore Grid Cells	Cross-Shore Grid Cells	Longshore Grid Spacing (feet)	Cross-Shore Grid Spacing (feet)
Regional Wave Grid	101	47	6,977 - 23,366	6,575 - 23,375
Intermediate Wave Grid	113	54	582 - 2,194	629 - 2,144
Local Wave Grid	248	93	38 - 542	40- 420
Flow Grid	248	153	33 - 575	41 - 415

Bathymetry over the Regional and Intermediate wave grids was based on the NOAA (2006) Regional Grid (Figure 11-1). Within the Flow Grid and Local Wave Grid, the bathymetry during the calibration runs was updated to depict the conditions during calibration period (May-July 2005). Accordingly, the primary data source used to fill these grids was the April 2005 survey by Gahagan & Bryant (2006). Elevations outside April 2005 survey area were estimated from:

- The October 2005 Light Detection and Ranging (LIDAR) survey of Pender County by NOAA.
- The June 2006 survey of the Mason Inlet area by Gahagan & Bryant.
- The August 2004 LIDAR survey of Pender County by NOAA.
- The March 2002 digital elevation model produced by the North Carolina Floodplain Mapping Program.
- The NOAA (2006) Regional Grid (Figure 11-1).

The 2005 bathymetry appears in Figure 11-16. The primary bathymetric features are the inlet throat, Green Channel, Nixon Channel, the AIWW, and Futch Creek. The main channel through the inlet throat and the ebb shoal ranges from -20 to -35 feet NAVD and runs from southeast to northwest. At the landward end, it splits into Green Channel, which runs from south to north, and Nixon Channel, which runs from east to west. Both channels, which end at the AIWW, are approximately 2 miles long with a typical depth of -15 feet NAVD. In Green Channel, the channel splits in two between the Inlet Throat and Green Channel tide gages. At the landward end of Nixon Channel, Butler Creek provides a secondary connection to the AIWW. Typical depths in Butler Creek are -14 feet NAVD. Futch Creek flows into the AIWW midway between Nixon Channel and Butler Creek. The marsh between Figure Eight Island and the AIWW ranges from 1 to 1.5 miles wide. Typical elevations in the marsh are on the order of 0 feet NAVD.

During the current and water level calibration, the Delft3DFLOW model was run in three-dimensional model. Five vertical layers were assumed at each grid point, with each layer equal to 20% of the water depth.

11.2.2 Model Forcing

To calibrate the currents and water levels in Delft3DFLOW, flow patterns were simulated between May 19, 2005, 8:00 PM EDT and June 30, 2005, 8:00 PM EDT. Sediment transport, erosion, and deposition were assumed to be negligible during this period. Water levels on the offshore boundary of the Flow Grid were assumed to be equal to the measured water levels by NOAA at Wrightsville Beach (see Figures 4-3 and 11-17). Waves on the offshore boundary of the Regional Wave Grid were taken from the NOAA Wavewatch forecast for the Western North Atlantic at 33.50°N, 76.75°W, -488' NAVD (see Figures 11-12, 11-17, and 11-18). Uniform wind velocities were assumed, based on measurements by NOAA at the Wrightsville Beach tide gages (see Figures 4-3 and 11-18).

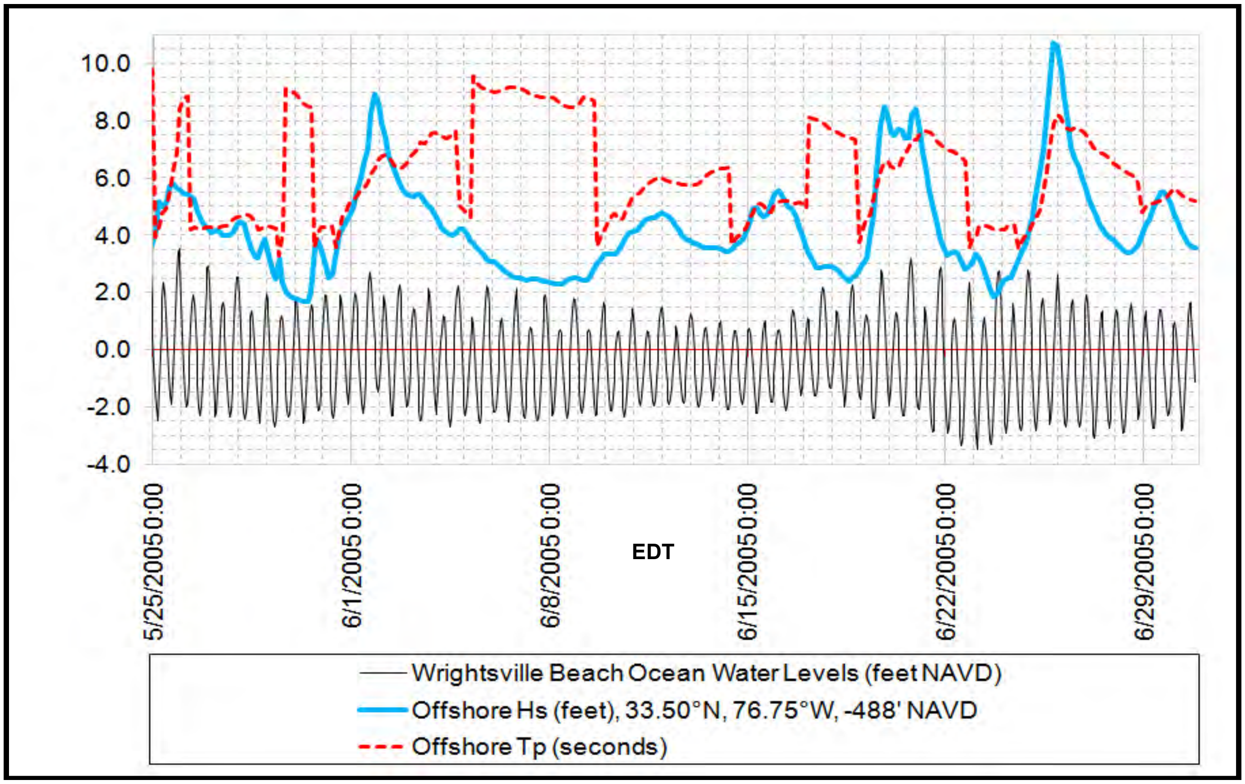


FIGURE 11-17: Offshore Waves and Water Levels during the Delft3DFLOW Calibration.

In both the SWAN and Delft3DFLOW models, the assignment of the upcoast and downcoast boundary conditions followed the standard modeling practices. On the northern and southern boundaries of the flow grid, zero gradient boundary conditions were assumed. Currents and water levels just outside the northern and southern boundaries were assumed to be equal to the corresponding values immediately inside. On the northeastern and southwestern boundaries of the Regional Wave Grid, the wave heights and directions outside the surf zone were assumed to be equal to their corresponding values on the offshore boundaries.

11.2.3 Calibration and Verification Results

To calibrate and verify the water levels and currents, Chezy's bottom friction coefficient was varied (see Figure 11-19). All other model parameters were set to their default values. Chezy's bottom friction coefficient was related to Manning's n based on the following:

Chezy's bottom friction = (Depth in meters$^{1/6}$) / (Manning's n)

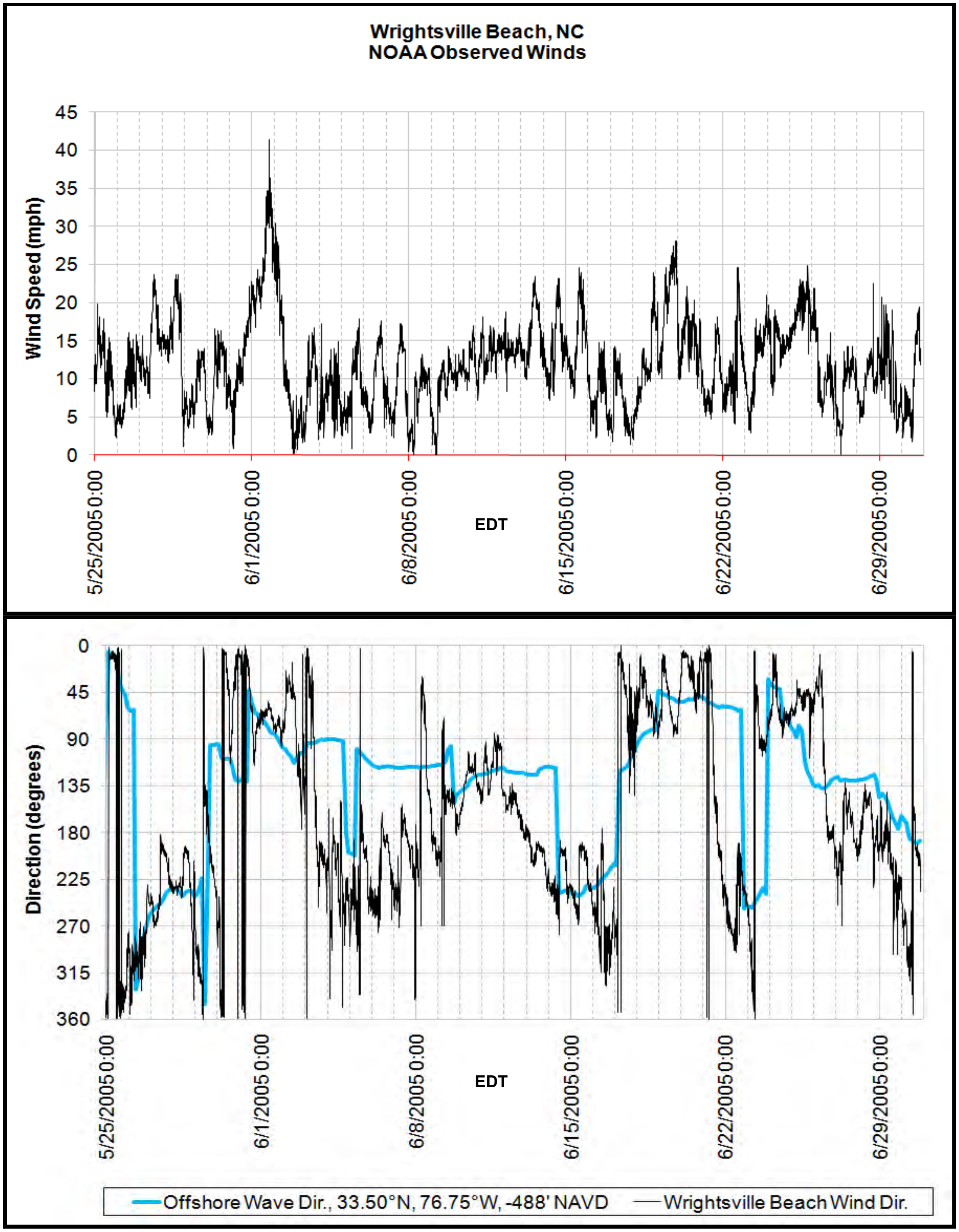


FIGURE 11-18: Wind Velocities and Offshore Waves Directions during the Delft3DFLOW Calibration.

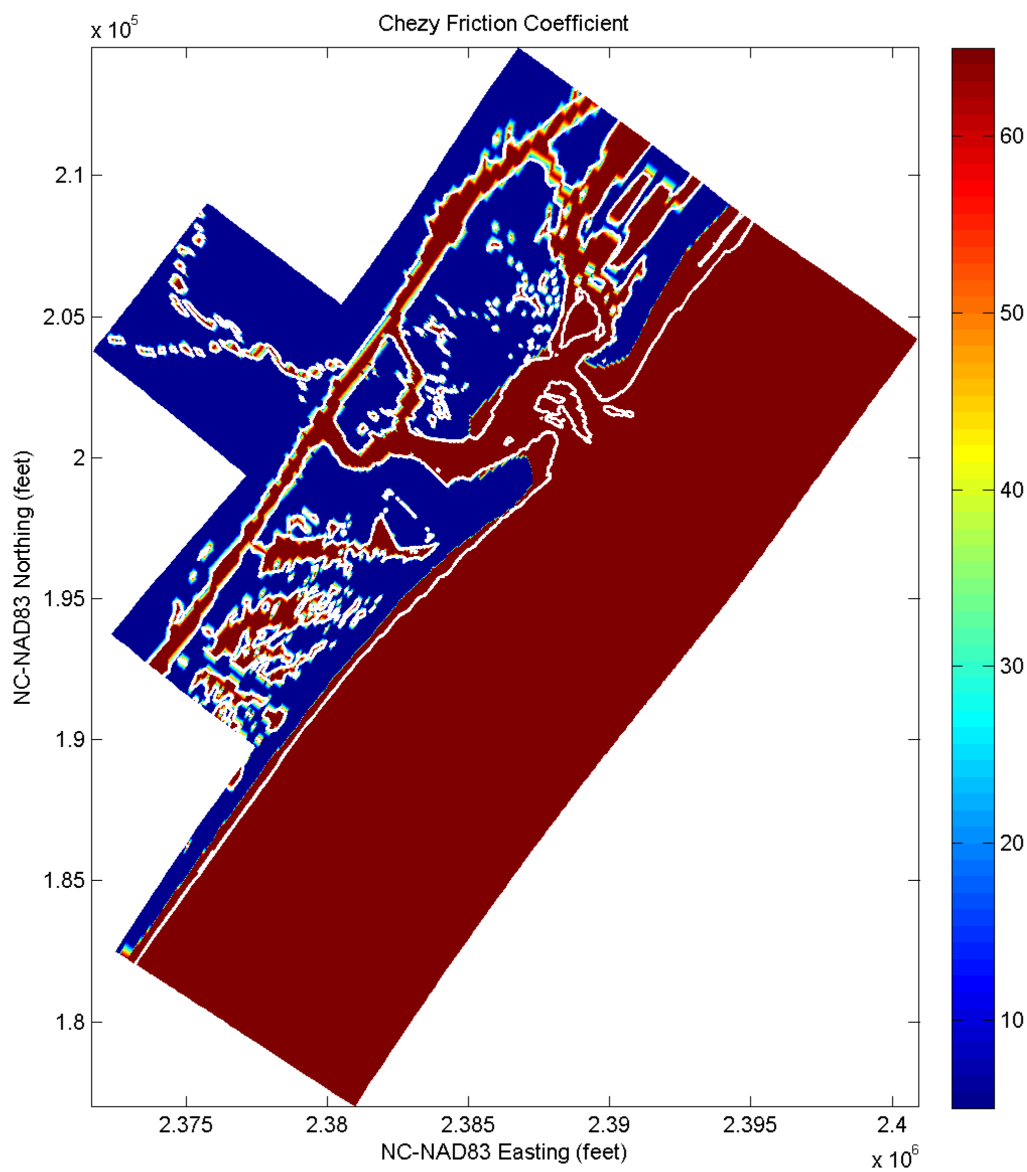


FIGURE 11-19: Final Bottom Friction Mapping for Delft3DFLOW Model.

Within the salt marsh and upland areas, the bottom friction coefficient was equal to 5. The equivalent value of Manning's n given a mean high water elevation of 1.7 feet NAVD and a bottom grade elevation of 0' NAVD would be 0.179. Elsewhere, the bottom friction coefficient was equal to 65, which was the model's default value. The equivalent value of Manning's n given a mean high water elevation of 1.7 feet NAVD and a bottom grade elevation of -15' NAVD would be 0.020.

Model results during spring tides on June 21, 2005 were used to calibrate the model. Agreement between the observed currents and the simulated currents was in the Inlet Throat and Nixon Channel was good (see Figures 11-20 to 11-21). Within Green Channel, differences between the simulated and observed currents occurred due to the location of the Green Channel ADCP (see Figures 11-15 and 11-22). This ADCP was deployed near the junction of the two forks within Green Channel and a side channel into the salt marsh. This location was characterized by complex currents in the model (see Figure 11-23). If the Green Channel ADCP had been deployed further inland, the model results would have been closer to the observations. Overall, the velocities predicted the by the model were reasonable within the areas being considered for dredging.

Simulated and observed water levels appear in Figures 11-24 to 11-26. Agreement between the measured and observed water levels was very good at all tide gages deployed by Gahagan & Bryant.

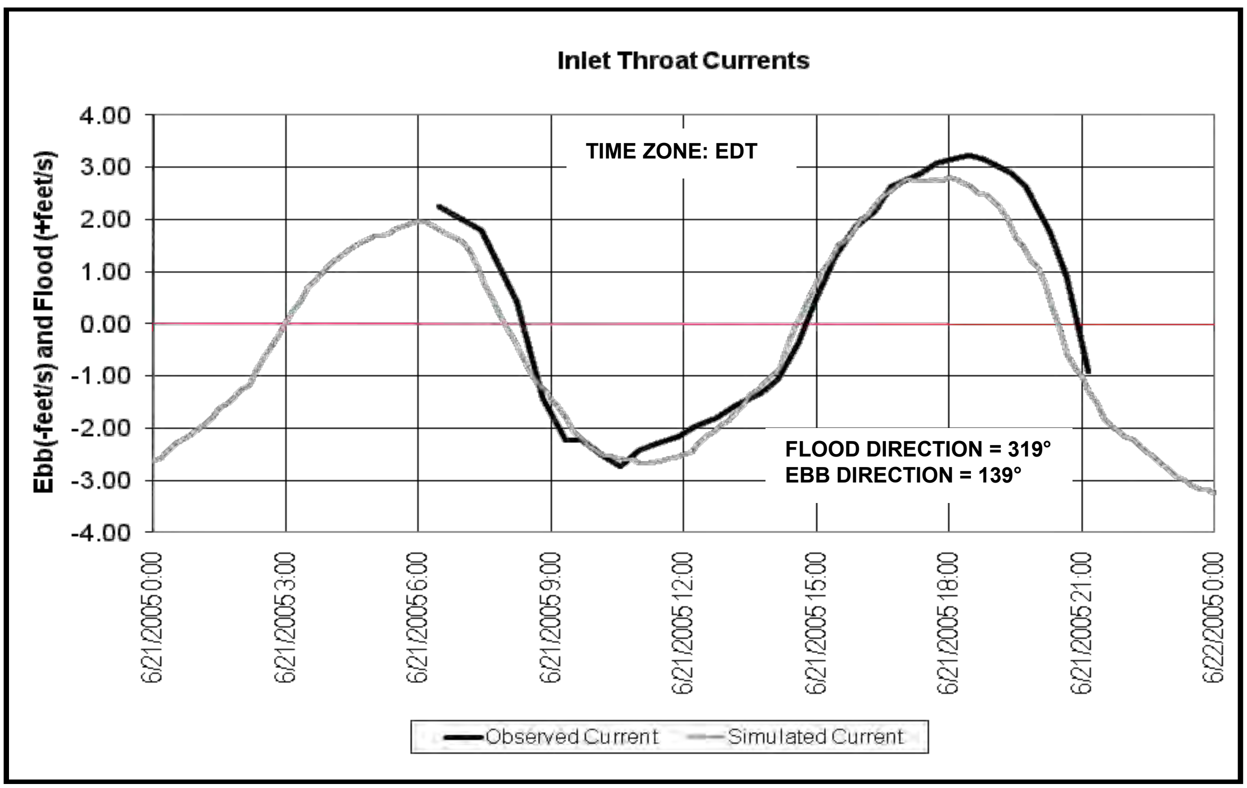


FIGURE 11-20: Simulated and Observed Currents at the Inlet Throat ADCP.

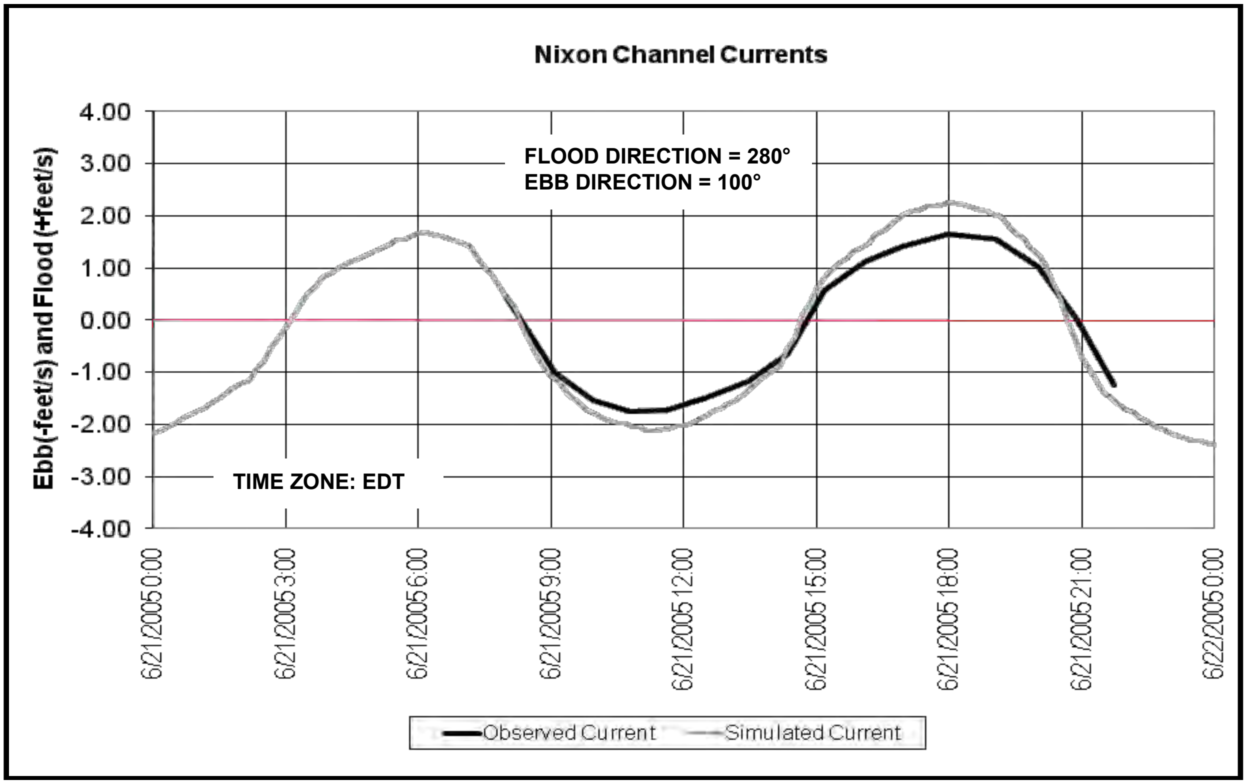


FIGURE 11-21: Simulated and Observed Currents at the Nixon Channel ADCP.

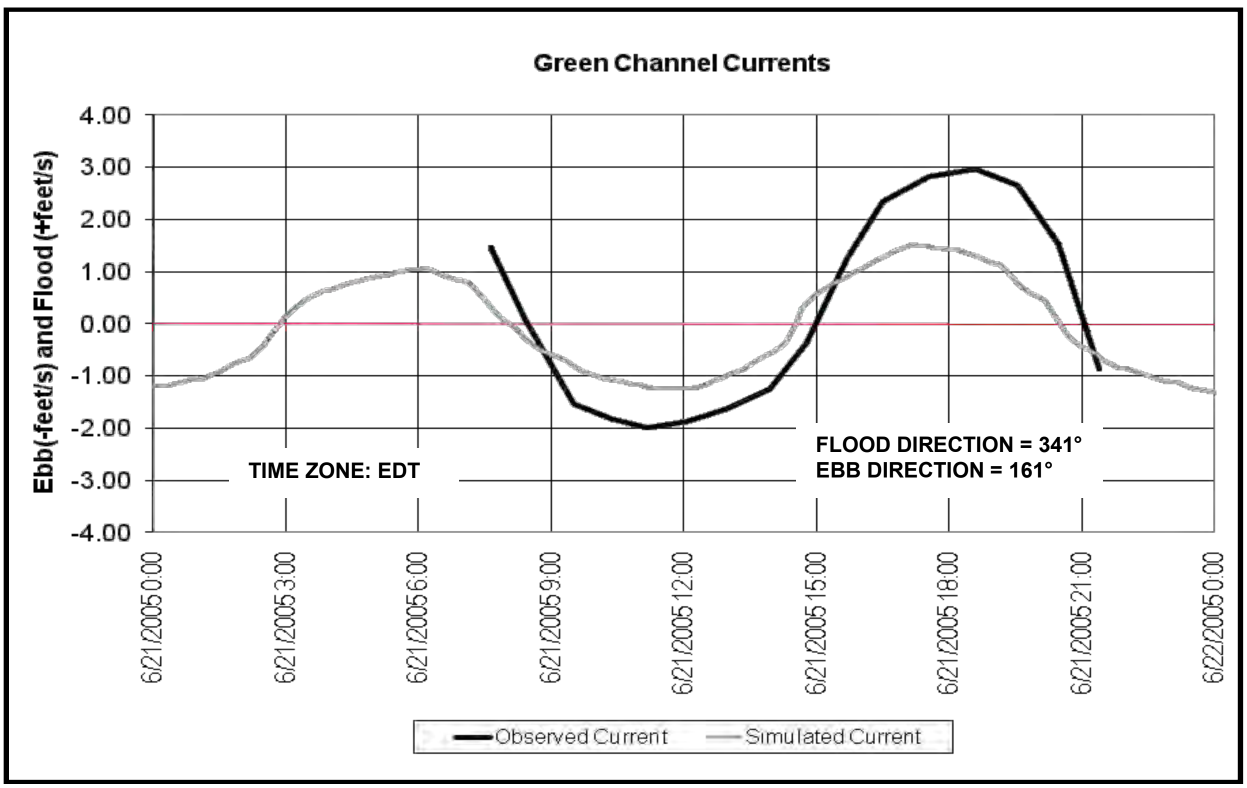


FIGURE 11-22: Simulated and Observed Currents at the Green Channel ADCP.

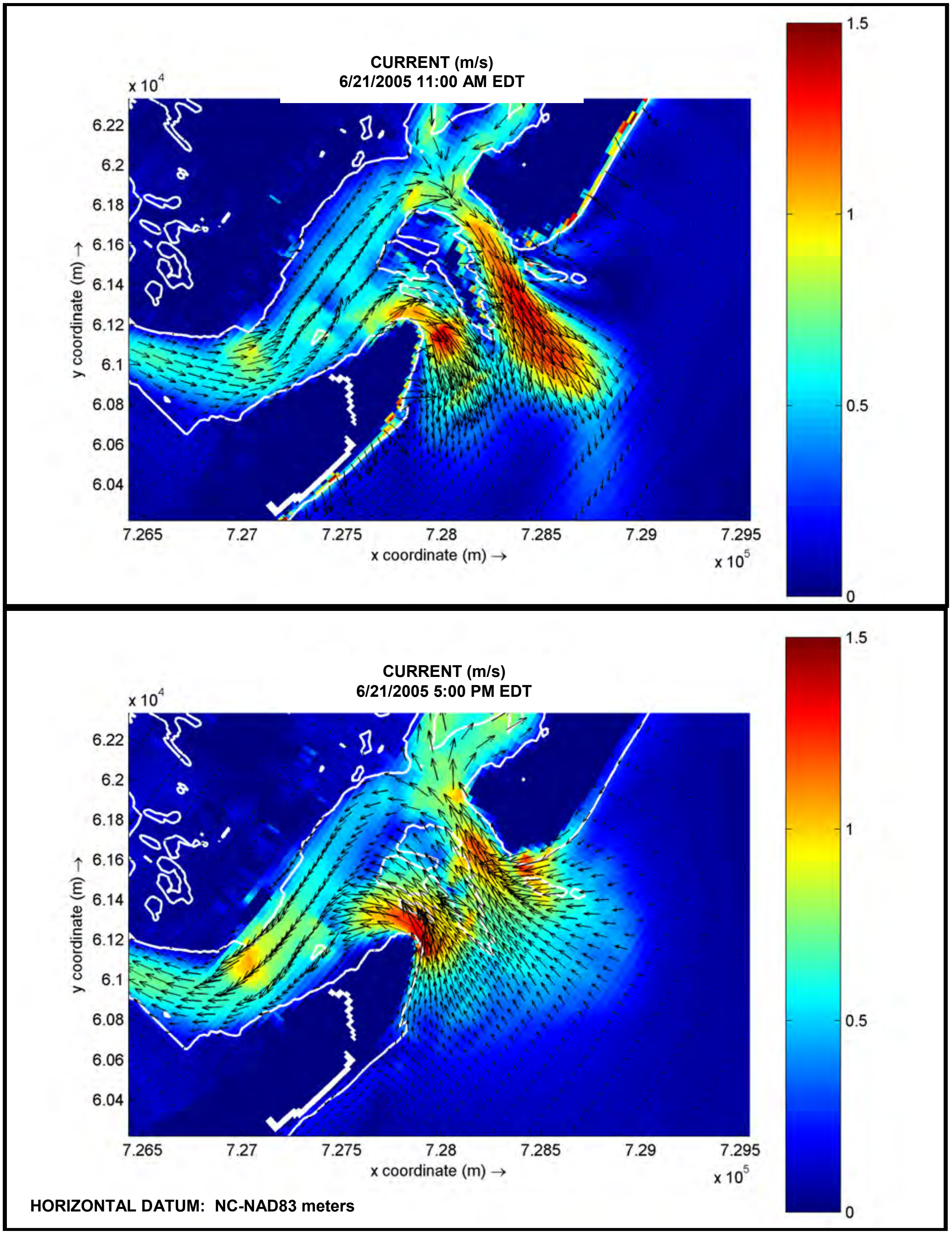


FIGURE 11-23: Typical Simulated Currents during Spring Tides.

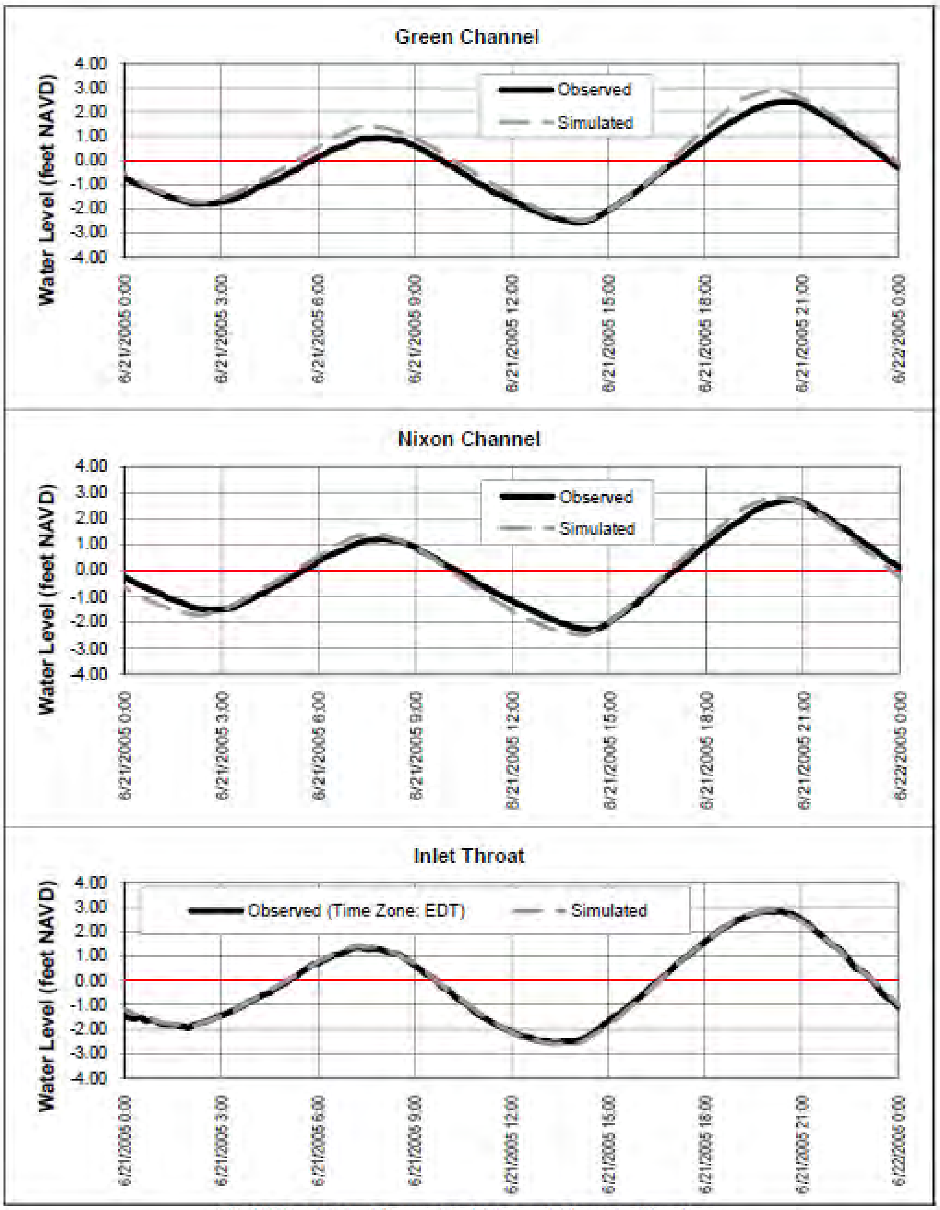


FIGURE 11-24: Figure Eight Island Flow Calibration, Inlet Throat & Channel Tide Gage Comparisons.

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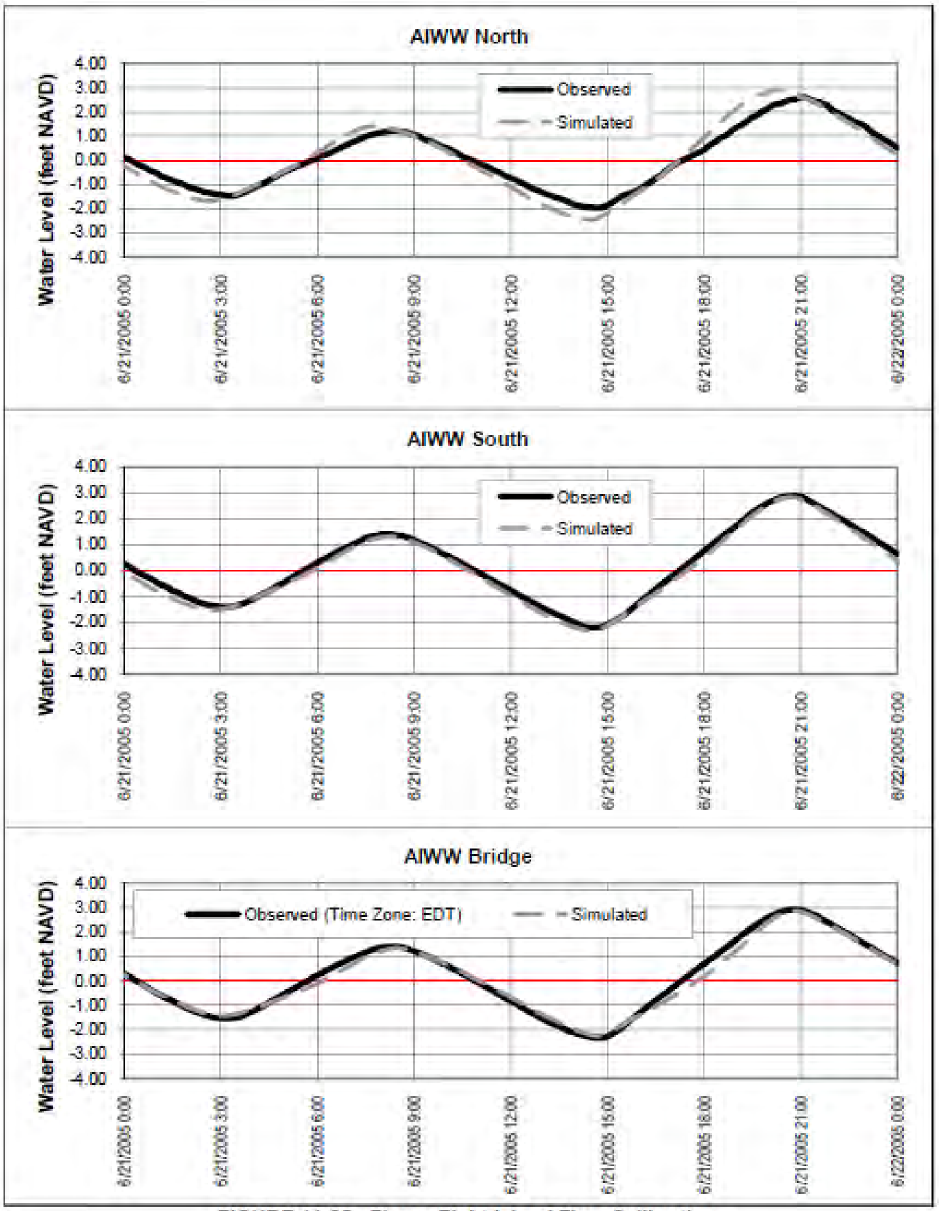

FIGURE 11-25: Figure Eight Island Flow Calibration, Atlantic Intracoastal Waterway Tide Gage Comparisons.

COASTAL PLANNING & ENGINEERING, INC.

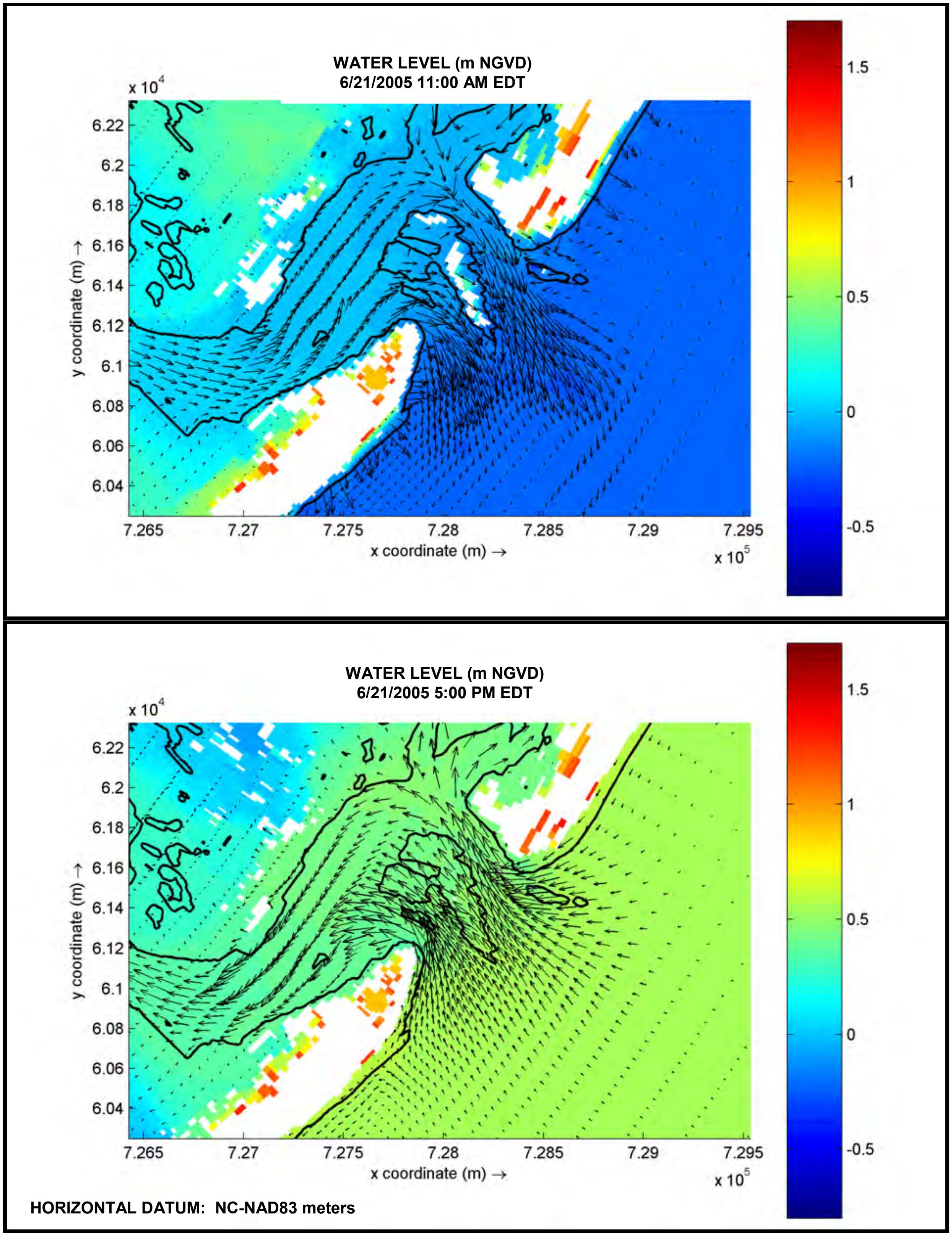

FIGURE 11-26: Typical Water Levels during Spring Tides.

Model results during neap tides on June 13, 2005 were used to verify the model (Figures 11-27 to 11-30). During neap tides, the simulated water levels agreed very well with the observed water levels. Thus, the flow model was able to predict the water levels during both neap tides and spring tides with a high level of confidence (see Table 11-2). Given the overall results from the calibration and verification periods, the flow model provided a sufficient description of the flow patterns in Rich Inlet. Accordingly, the remaining model runs in this study utilized the Delft3DFLOW model with the bottom friction values in Figure 11-19.

TABLE 11-2
DELFT3D CURRENT AND WATER LEVEL
CALIBRATION & VERIFICATION SUMMARY

ADCP	Mean Error (feet/second)	RMS Error (feet/second)
Currents, June 21, 2005 6:30 am EDT to June 21, 2005, 9:40 pm EDT:		
Inlet Throat	0.32	0.59
Nixon Channel	-0.04	0.35
Green Channel	0.28	1.03
Tide Gage	**Mean Error (feet)**	**RMS Error (feet)**
Water Levels, May 25, 2005, 10:10 am EDT to June 30, 2005, 8:00 pm EDT:		
Green Channel	0.16	0.26
Nixon Channel	-0.02	0.19
Inlet Throat	-0.08	0.18
AIWW North	-0.04	0.28
AIWW South	-0.12	0.20
AIWW Middle	-0.10	0.20
AIWW Bridge	-0.05	0.23

11.3 Erosion and Deposition Calibration

Sediment transport, erosion, and deposition in the Delft3D modeling package were simulated using Delft3DFLOW. The calibration of sediment transport, erosion, and deposition was based on the volume changes between April 2005 and the present. Parameters examined during the calibration included the following:

- The approximation of the tides.
- The delineation of the wave cases.
- The use of wind stress in both Delft3DFLOW and SWAN.
- The sediment transport parameters within Delft3DFLOW.

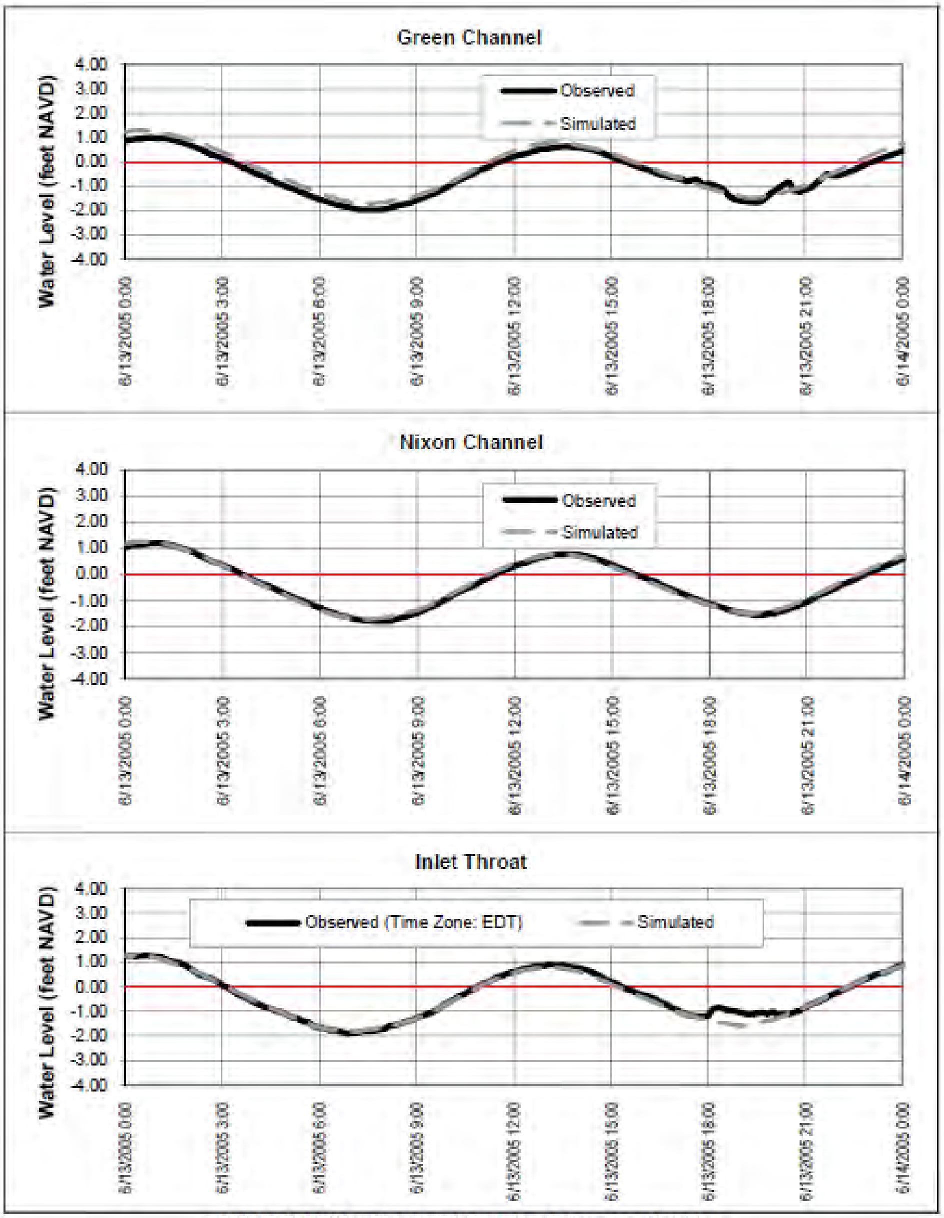

FIGURE 11-27: Figure Eight Island Flow Verification,
Inlet Throat & Channel Tide Gages.

COASTAL PLANNING & ENGINEERING, INC.

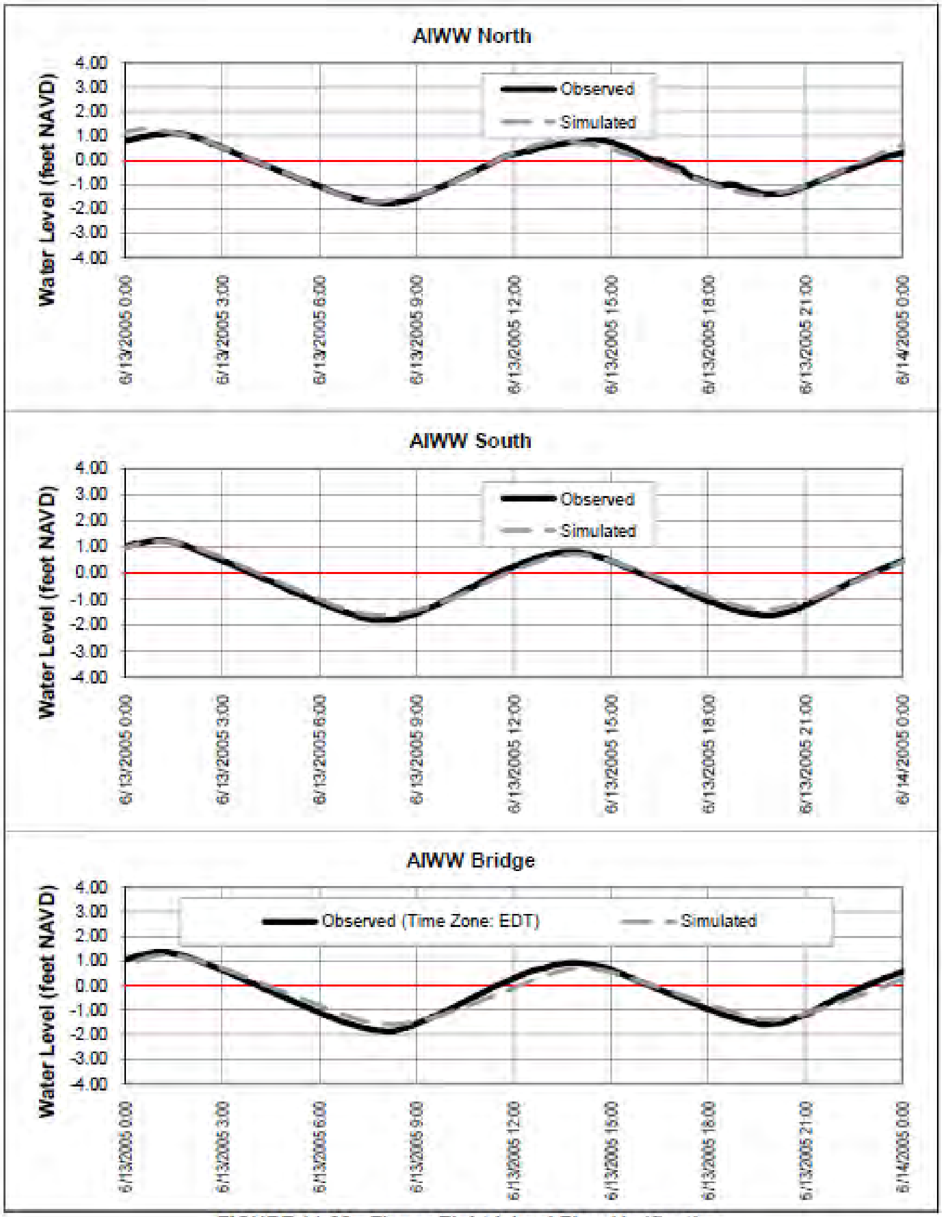

FIGURE 11-28: Figure Eight Island Flow Verification, Atlantic Intracoastal Waterway Tide Gages.

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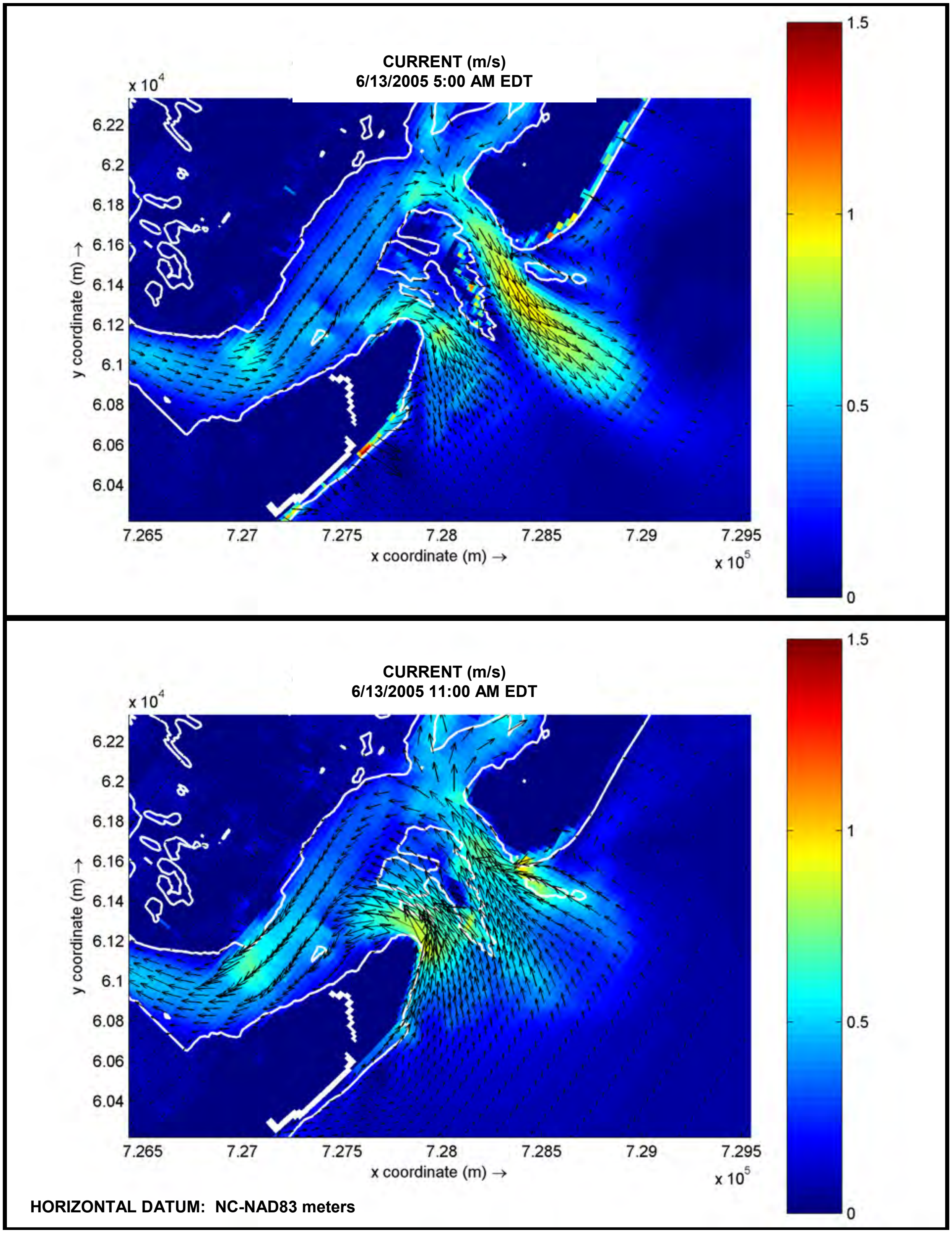

FIGURE 11-29: Typical Simulated Currents during Neap Tides.

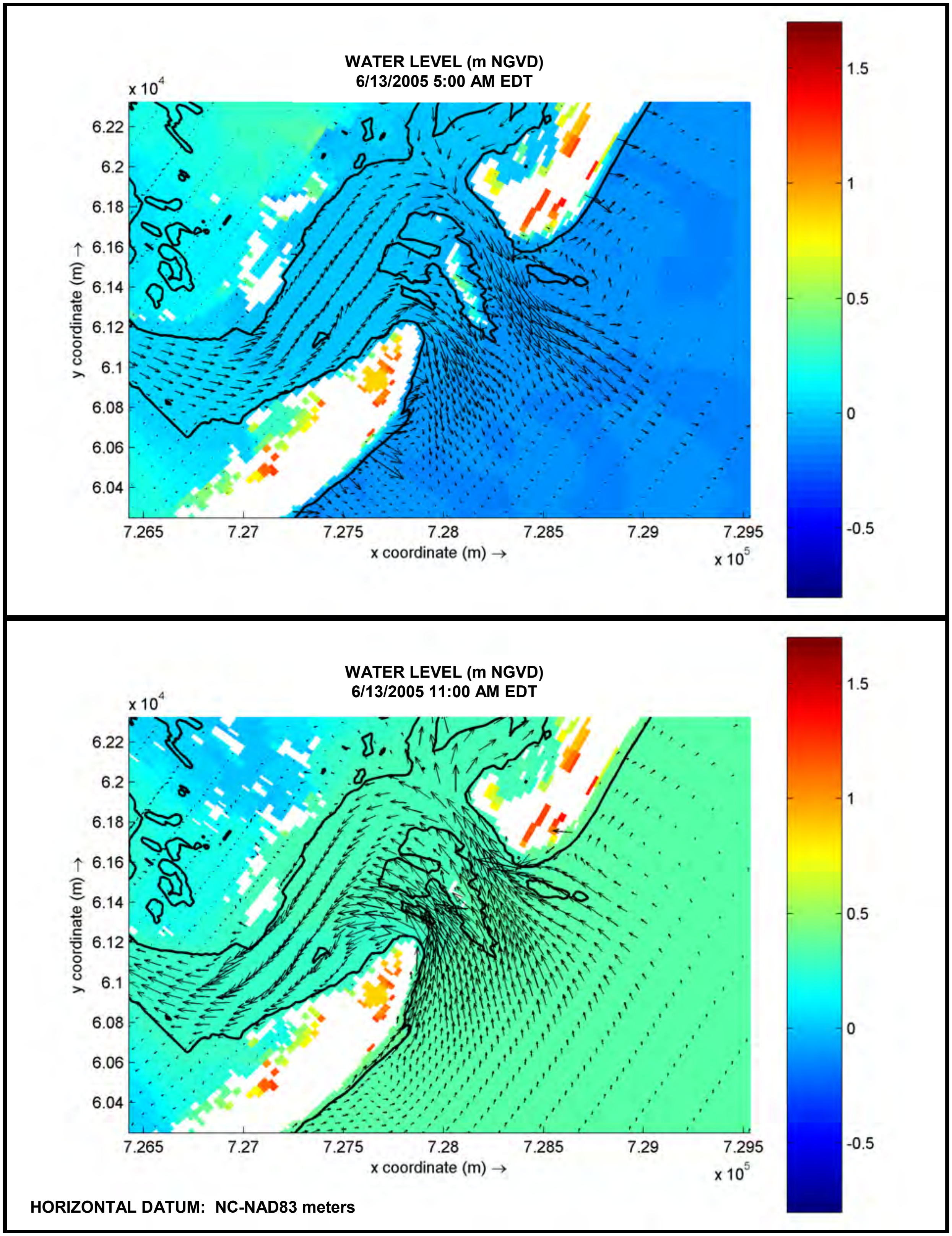

FIGURE 11-30: Typical Water Levels during Neap Tides.

11.3.1 Tides

Ideally, 2-5 years of bathymetric changes could be simulated using a 2-5 year model run. However, a 2-5 year model run using Delft3DFLOW would require 2-3 months of computational time, even under the best circumstances. To reduce the amount of computational time, a number of methods have been developed so that 5 years of bathymetric changes can be simulated using a 3-7 week model run, which can be completed in 2-7 days.

The first of these methods is the simplification of the tides. As long as a simplified tide with single harmonic produces the same residual transport as 14-15 days of predicted tides, the spring-neap tidal cycle can be approximated using a simplified tide:

$$\eta \approx \eta_o + A \cos(2\pi t/T)$$

where

η = water level
η_o = mean tide level
A = tidal amplitude
t = time
T = tidal period

To select the best simplified tide, several simulations were conducted using two methodologies (see Table 11-3):

- The Lesser (2009) approach using M2 and C1 tidal harmonics (M2C1 in Table 11-3).

- The mean tidal amplitude ± 20% and the M2 tidal period of 745 minutes (12.42 hours).

TABLE 11-3
SIMPLIFIED TIDE SCHEMES TESTED

Tide scheme	Amplitude (feet)	Period (min)
M2C1	2.16	1490
M2C1 (-20%)	1.72	1490
M2C1 (+20%)	2.59	1490
Mean	2.07	745
Mean (-20%)	1.66	745
Mean (+20%)	2.48	745

The first simulation consisted of 15 days predicted tides based on the harmonics in Table 11-4. The remaining simulations consisted of 15 days of simplified tides characterized a single amplitude and tidal period. Waves were neglected during these simulations, and default sediment transport parameters were utilized.

TABLE 11-4
TIDAL CONSTITUENTS BASED ON WATER LEVEL MEASUREMENTS
TAKEN IN THE INLET THROAT, MAY 25 – JULY 7, 2005

	Period (hours)	Amplitude (feet)	Phase (degrees)
M2	12.42	1.77	244.1
N2	12.66	0.41	243.4
K1	23.93	0.40	116.3
O1	25.82	0.18	147.9
S2	12.00	0.17	254.6
MM	661.31	0.14	331.9
MSF	354.37	0.13	290.0
M4	6.21	0.07	148.9
MU2	12.87	0.07	163.9
Q1	26.87	0.06	172.5
L2	12.19	0.04	215.6
MS4	6.10	0.04	214.7
M6	4.14	0.03	53.1
M3	8.28	0.03	190.0
MN4	6.27	0.03	75.7
NO1	24.83	0.02	170.7
2MN6	4.17	0.02	61.0
SN4	6.16	0.02	63.0

Although all 6 tidal schemes in Table 11-3 were tested, tides along the regional are semi-diurnal (see Figures 11-20, 11-21, 11-22, 11-24, 11-25, 11-27, and 11-28). Accordingly, the results of the M2C1 tests are not shown. Test results based on the 745 minute tidal schemes appear in Figures 11-30 to 11-34. The best results were achieved using the mean tidal amplitude of 2.07 feet and a tidal period of 745 minutes (12.42 hours). As shown in Figure 11-34, differences in sedimentation patterns between 15 days predicted tides and 15 days simplified tides (T = 745 minutes, A = 2.07’) were small (± 1 foot) or negligible.

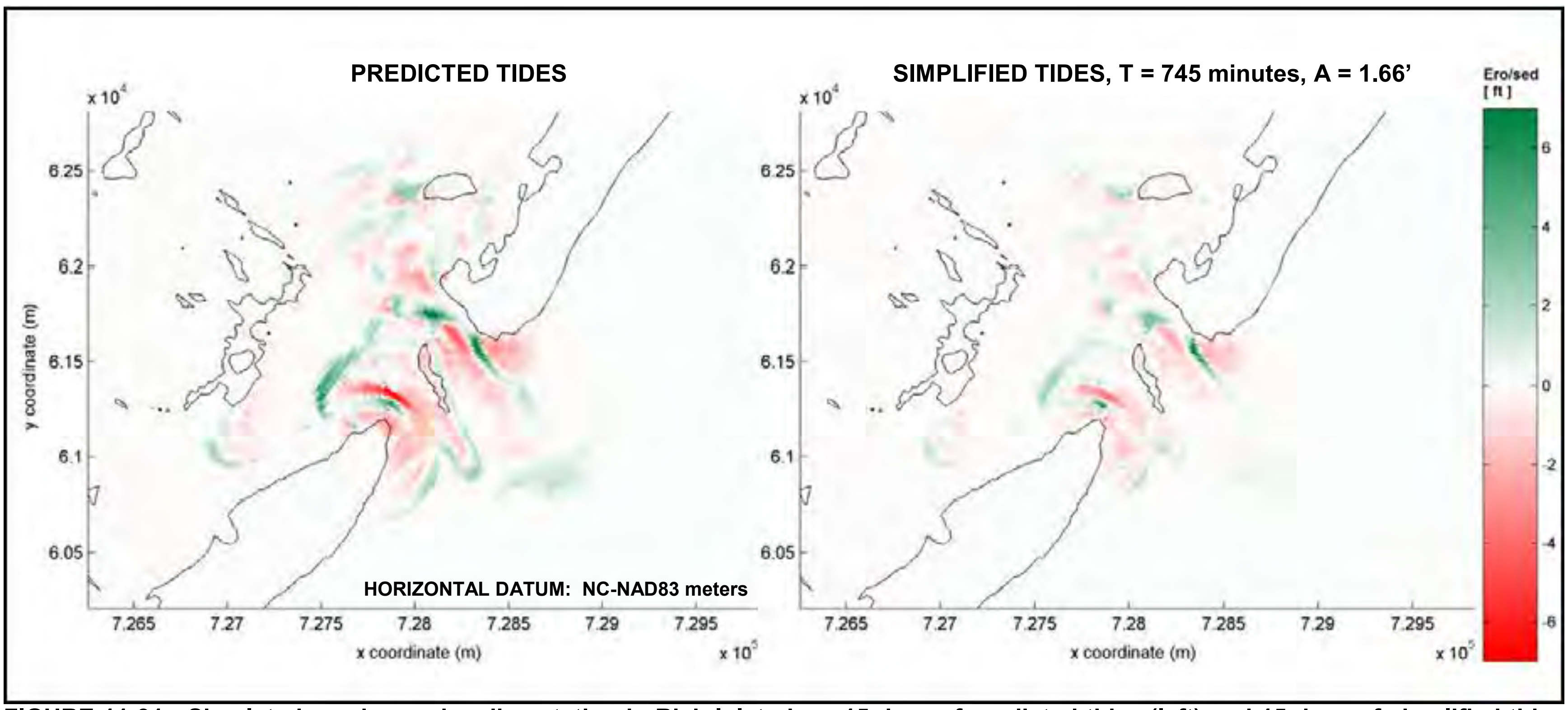

FIGURE 11-31: Simulated erosion and sedimentation in Rich Inlet given 15 days of predicted tides (left) and 15 days of simplified tides assuming T = 745 minutes (12.42 hours) and A = 1.66' (mean – 20%) (right).

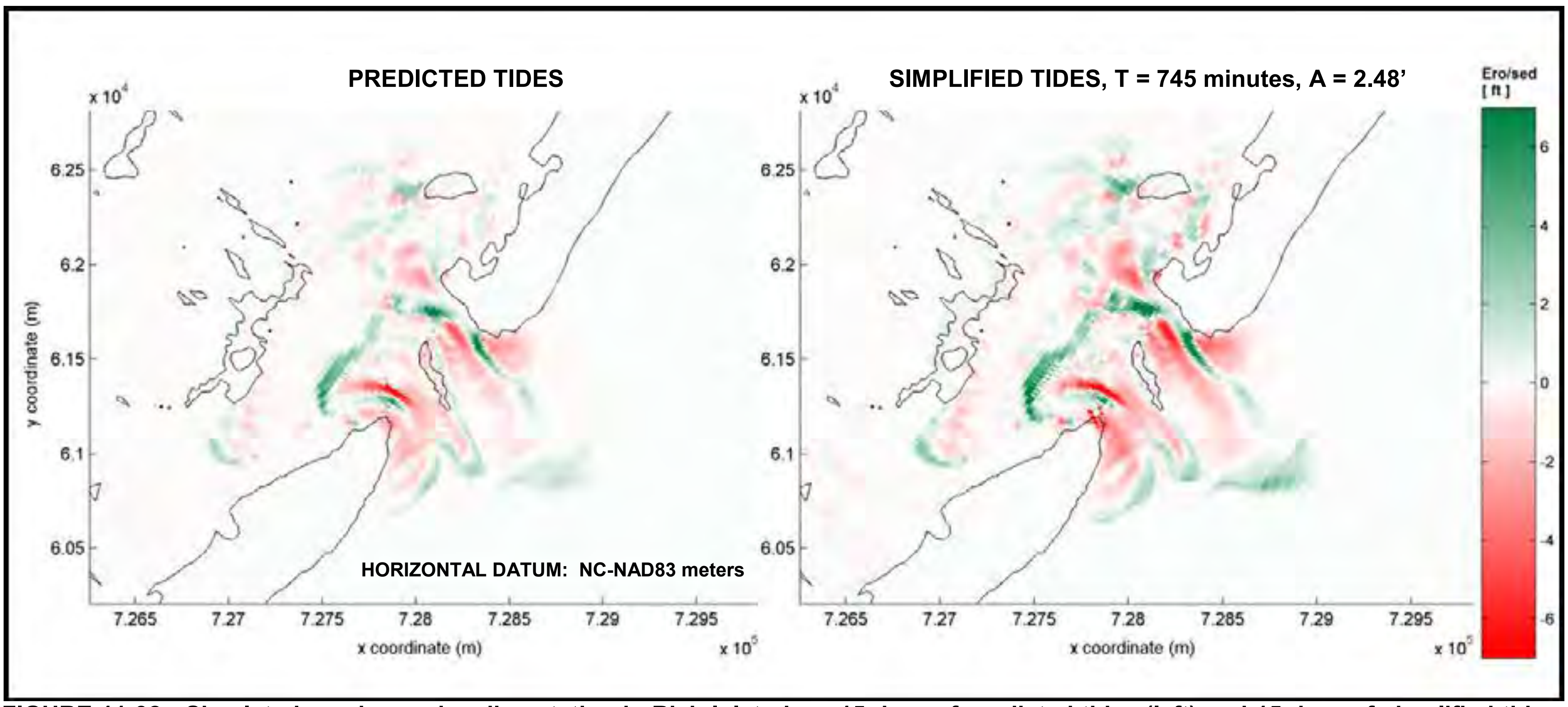

FIGURE 11-32: Simulated erosion and sedimentation in Rich Inlet given 15 days of predicted tides (left) and 15 days of simplified tides assuming T = 745 minutes (12.42 hours) and A = 2.48' (mean + 20%) (right).

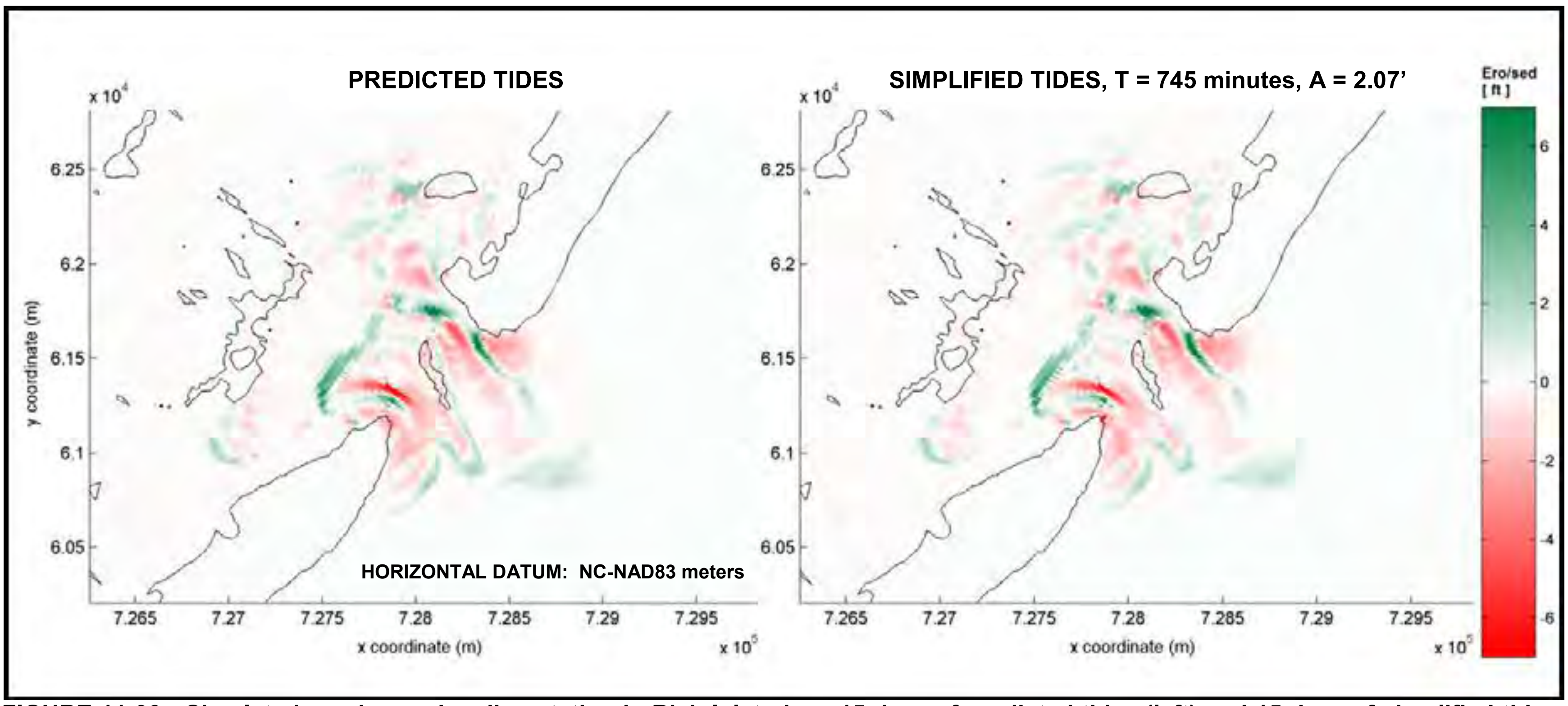

FIGURE 11-33: Simulated erosion and sedimentation in Rich Inlet given 15 days of predicted tides (left) and 15 days of simplified tides assuming T = 745 minutes (12.42 hours) and A = 2.07' (mean) (right).

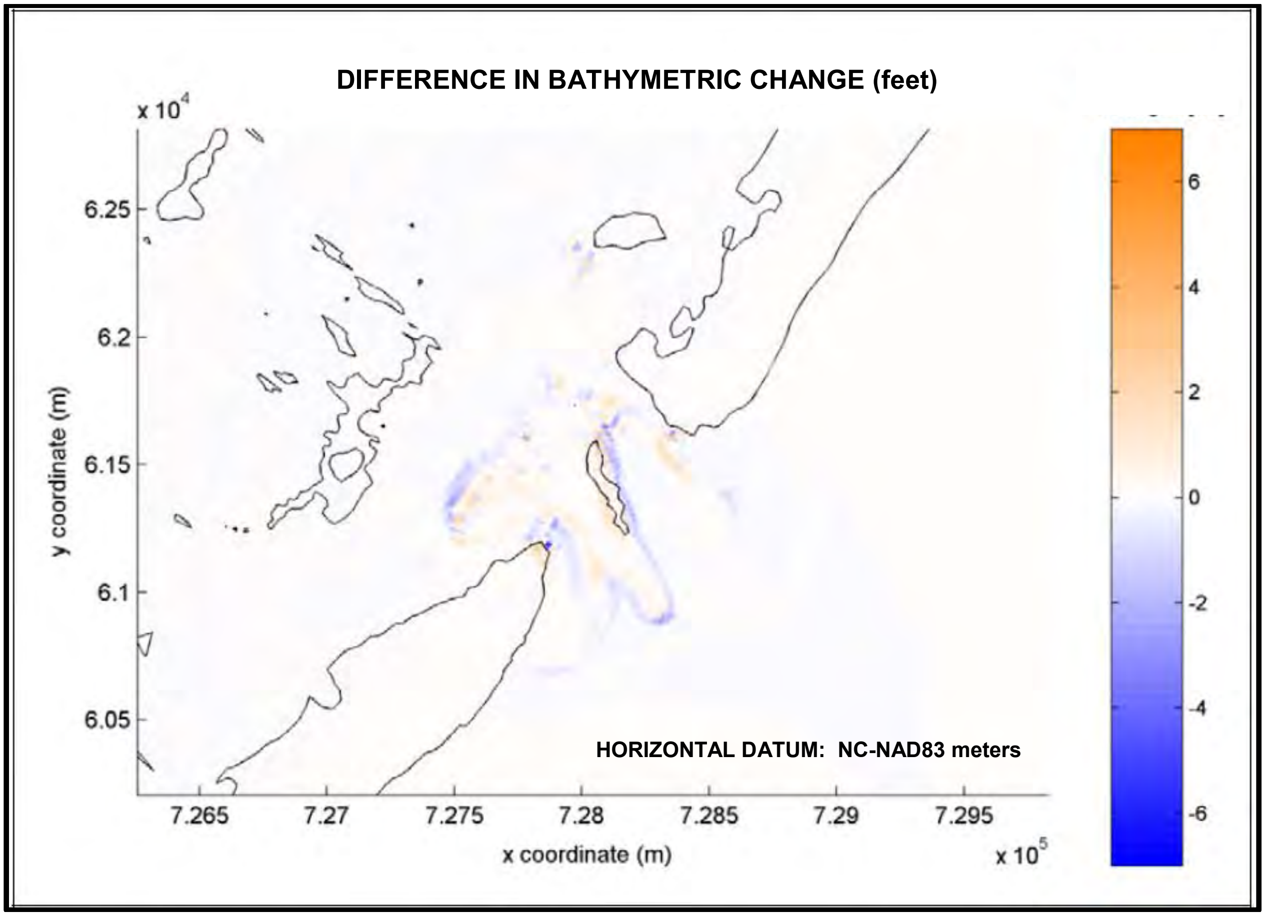

FIGURE 11-34: Differences in bathymetric change given 15 days of predicted tides versus 15 days of simplified tides assuming T = 745 minutes (12.42 hours) and A = 2.07' (mean). A difference of zero indicates that simplified tides lead to the same bathymetric changes as the predicted tides.

11.3.2 Wave Cases

The waves used to calibrate sediment transport, erosion, and deposition were based on the NOAA Global Wavewatch forecast at 34.00ºN, 76.25ºW (see Figure 11-12). The depth at site was approximately -644 feet NAVD. As noted earlier, it is not practical to simulate 2-5 years of bathymetric changes using a 2-5 year time series of offshore water levels and waves to drive the model. Instead, the Delft3D model is typically run for a shorter period of time, using 10-75 representative wave cases to approximate the general wave climate during the period of interest (i.e.: Lesser, et al., 2004; Benedet and List, 2008).

Potential wave climates for the project area were based on the forecast wave record at 34.00°N, 76.25°W between October 1999 and April 2007. All waves propagating from the landward direction bands (200° to 360° and 0° to 55°) were ignored, along with all waves smaller than 1.64 feet (0.5 m). The remaining wave records were divided into wave height and direction classes, with each wave class containing an equal amount of wave energy (in KW-Hours/m). This method, known as the Energy Flux Method, characterized each wave record based on the energy flux:

$E_p \approx 1.56\ T_p \rho g H_s^2 / 2$ (deep water assumption)

$\text{Energy} = E_p \Delta t$

Where

E_p = energy flux
T_p = peak wave period
ρ = sea water density (1025 kg/m^3)
g = gravitational acceleration (9.81 m/s^2)
H_s = significant wave height
Δt = interval between wave records (3 hours)

To simulate 1 year of sediment transport, erosion, and deposition, each wave case was run for 1 to 3 tidal cycles per year, which were characterized by a single harmonic (see previous section). Sediment transport values were then scaled by a Morphological Acceleration Factor, so that 1 to 8 weeks of the simulation would be equivalent to 1 year of erosion (i.e.: Lesser, et al., 2004; Benedet and List, 2008):

$M = T_{study\ period} / T_{model\ period}$

where

M = Morphological Acceleration Factor
$T_{study\ period}$ = (length of the study period) x (percent occurrence for each wave case)
$T_{model\ period}$ = duration of the wave case in the model simulation

Lower M values were used for the higher waves, during which the majority of the significant bathymetric changes occurred. Conversely, higher M values were used for the more frequent, but smaller waves. This schematization was consistent with the standard practices used within the Delft3D modeling community.

Based on the method above, 3 wave climates were delineated:

1. A 12-case wave climate.
2. A 20-case wave climate.
3. A 70-case wave climate that approximated the full time series of waves between October 1999 and April 2007.

To determine which wave climate would be the most appropriate, preliminary Delft3D-FLOW simulations using each wave climate were performed. Since the objective of this task was to determine how many wave cases would be necessary, sediment transport was activated within the Delft3D model, but changes to the seafloor elevation were not. Default sediment transport parameters were also utilized. These settings ensured that the sediment transport rates from each wave climate would not be biased by the erosion or deposition that would theoretically occur during the various wave cases. Average longshore sediment transport values were then extracted from the output of each simulation. Finally, sediment transport values based on the first two wave climates were compared to those of the 70-case wave climate (Figure 11-35).

Since erosion and deposition were not considered in this task, the results in Figure 11-35 were not intended to be compared to the sediment budgets in Figures 8-4 or 8-5. However, the results of the test showed that it would be possible to use a 12-case wave climate in the subsequent phases of the model calibration and the future conditions simulations. Wave cases appear in Table 11-5 and Figure 11-36.

TABLE 11-5
OCTOBER 1999 TO APRIL 2007
WAVE CLIMATE
34.00ºN, 76.25ºW, -644' NAVD

Wave Case	Hs (feet)	Tp (sec.)	Wave Dir. (deg.)	Frequency (days/year)	Tidal Cycles in Model per Year	Morph. Acceler-ation Factor	Wind Speed (mph)	Wind Dir. (deg.)
#1	4.5	7.9	64.2	21.5	2	20.7	3.6	77.1
#2	8.3	9.5	63.4	5.3	1	10.2	8.7	45.0
#3	11.8	10.2	63.6	2.4	1	4.6	19.9	45.0
#4	3.6	8.0	91.3	32.6	2	31.5	2.5	120.3
#5	6.0	8.5	89.1	11.1	1	21.5	6.1	61.7
#6	10.0	9.2	85.7	3.6	1	7.0	9.2	15.0
#7	3.2	7.5	122.1	44.7	3	28.8	4.0	166.4
#8	7.0	7.5	128.5	9.2	1	17.8	5.6	147.4
#9	14.7	9.5	130.9	1.6	1	3.2	4.7	155.4
#10	4.5	5.4	181.7	30.3	2	29.3	8.4	206.9
#11	8.4	7.0	177.8	6.6	1	12.8	13.9	232.2
#12	13.4	8.2	178.7	2.2	1	4.3	18.3	240.2

The smallest wave cases have heights in the range of 3.2 feet (1 m). The intermediate wave heights are in the range of 7.5 feet (2.3 m), and the highest waves are in the 12.5 foot (3.8 m) range. Peak wave periods vary from 5.4 to 10.1 seconds, and the wave direction varies from 63 to 181 degrees. The wind associated to the representative wave conditions was defined as the mean wind of each wave class (selected by Energy Flux Method). Each repetition of the 12 wave cases corresponded to 1 year of sediment transport, erosion, and deposition.

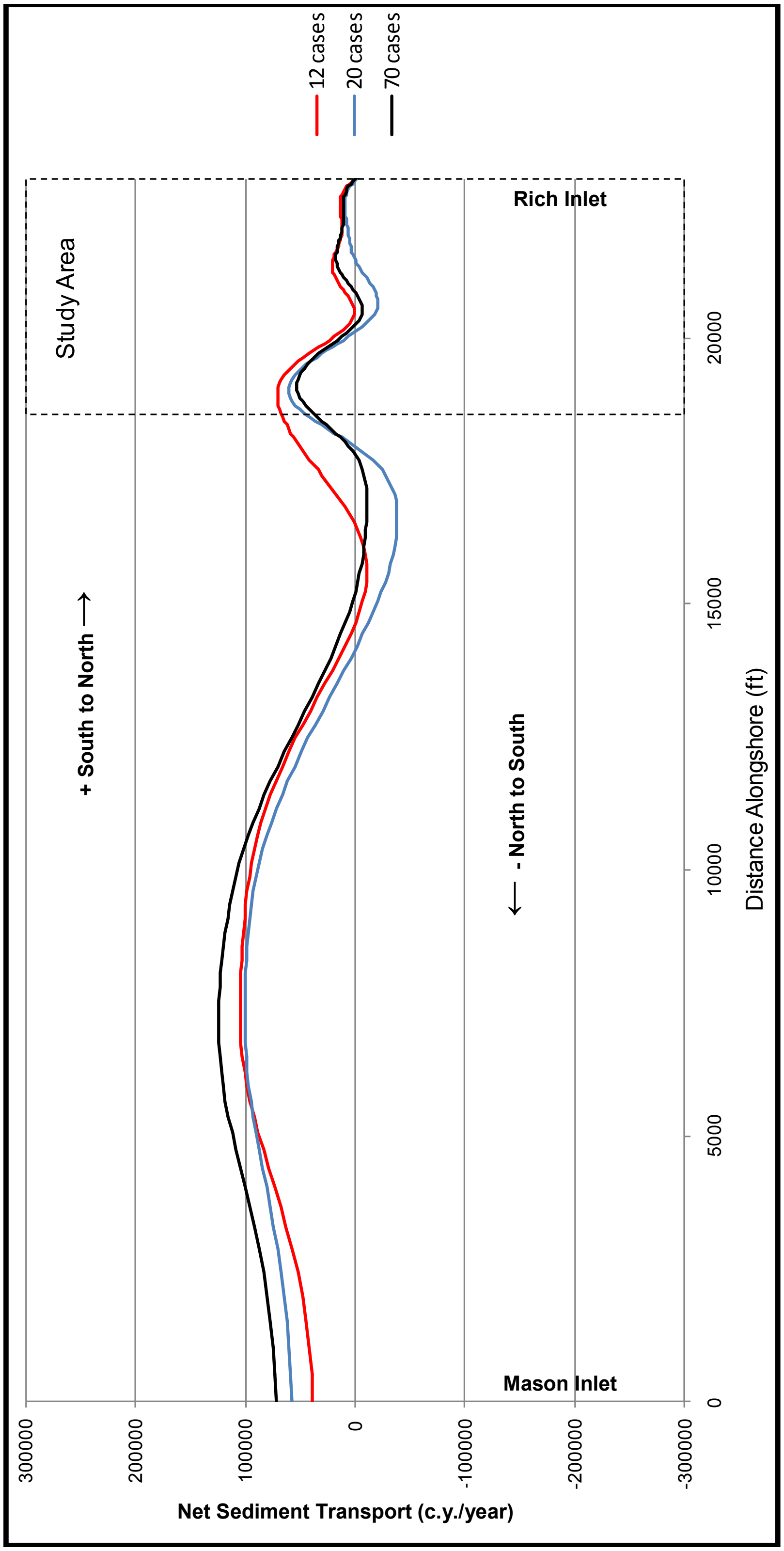

FIGURE 11-35: Theoretical Longshore Sediment Transport along Figure Eight Island Based on Wave Climates with 12, 20, and 70 Cases.

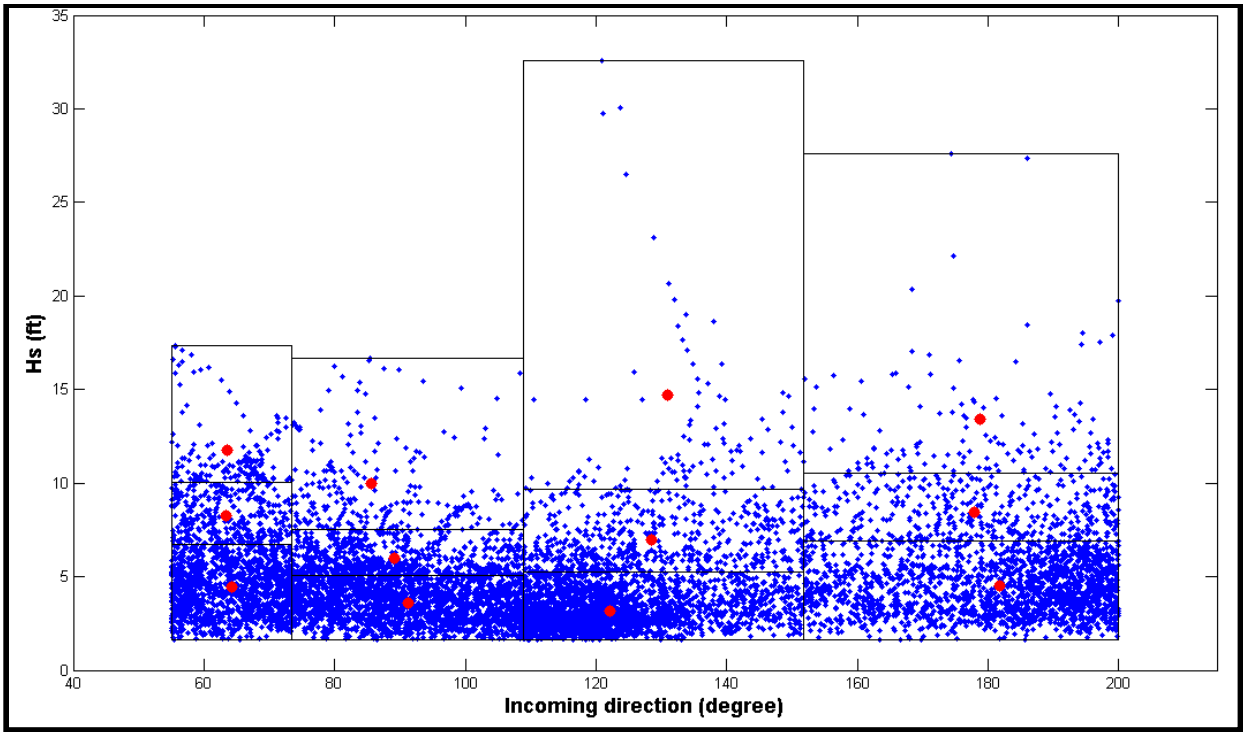

FIGURE 11-36: October 1999 To April 2007 Wave Climate, 34.00ºN, 76.25ºW, -644' NAVD, with Representative Wave Cases (red) for 12 Wave Classes (black squares).

11.3.3 Wind Stress

Both the SWAN and Delft3DFLOW models utilize wind stress formulations. In SWAN, wind stress governs the growth and generation of waves within the model grids. In Delft3DFLOW, shear stresses due to wind can be activated to partially govern the currents.

A large number of simulations were conducted how the model would perform if wind stress were:

1. Neglected in both models.
2. Considered in the Delft3DFLOW model but neglected in the SWAN model.
3. Considered in the SWAN model but neglected in the Delft3DFLOW model.
4. Considered In both models.

In each simulation, bathymetric changes were activated within Delft3DFLOW.

Sediment transport estimates given the first scenario were similar to those in Figure 11-35, which predicted net sediment transport towards the north along most of the island. While this was consistent with the two sediment budgets (Figures 8-5 and 8-4) at Rich Inlet, it was not consistent with the two sediment budgets elsewhere. Net sediment transport estimates under the second and third scenario appear in Figure 11-37. Similar to the first scenario, the direction of the net sediment transport was not consistent with the two sediment budgets. However, when

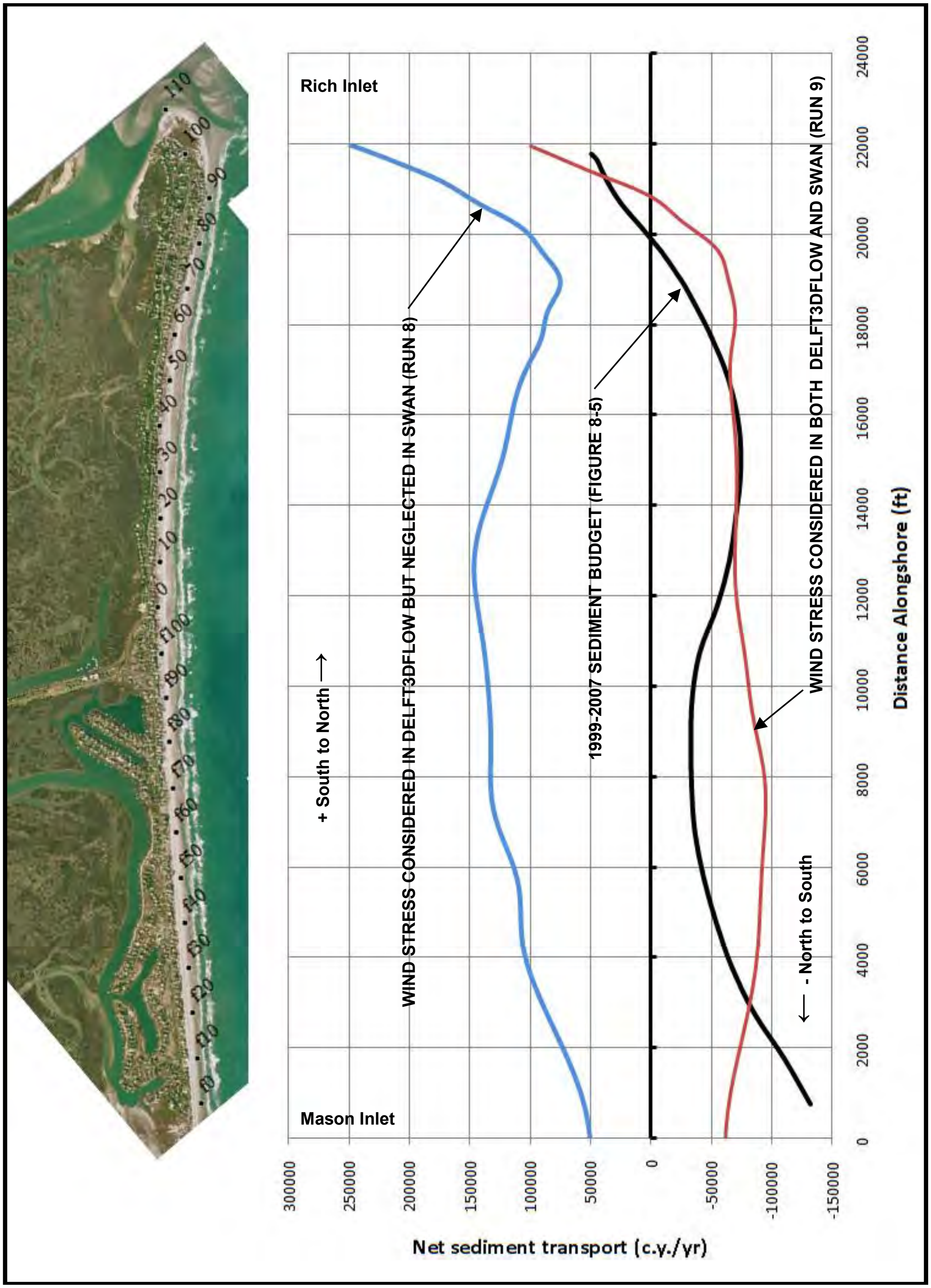

FIGURE 11-37: Sensitivity of Net Sediment Transport to the Activation of Wind Stress in Delft3DFLOW and SWAN.

wind stress was activated within both SWAN and Delft3DFLOW, the simulated sediment transport was closer to the 1999-2007 sediment budget. Subsequent simulations found that wind stress was not a critical factor in the Delft3DFLOW model, even though its application was necessary in the SWAN model. Accordingly, the final calibration run (not shown in Figure 11-37) utilized wind stress in the SWAN model but neglected wind stress in the Delft3DFLOW model. The results of the final calibration run are discussed in the next section.

11.3.4 Sediment Transport Parameters and Other Model Settings

The final phase of the calibration process considered the various sediment transport parameters in the model, along with the sequencing of the wave cases, the time step, the grid spacing, and other model settings. Over 40 calibration runs were performed during this phase. The final calibration run utilized the April 2005 survey as the primary bathymetric data source for the initial conditions, followed by the other data sources listed in Section 11.2.1. Grids were identical to those used in Figures 11-10 and 11-11. The duration of the model run was from April 2005 to April 2012.

A comparison of the simulated and observed volume changes on Figure Eight Island between April 2005 and October 2008 appear in Figure 11-38. Overall, the simulated volume changes are consistent with the observed volume changes. Both indicate a high level of erosion on the north end of the island (Surf Court to Rich Inlet, 70+00 to 110+00), mild erosion between profiles 30+00 and 70+00, and stable beaches between Backfin Point Road (F80+00) and profile 30+00. The model results do not follow the observed changes exactly. However, all of the general erosion patterns along the island's beaches are represented.

On Hutaff Island, the volume changes between April 2005 and April 2007 were anomalous due to the formation of a swash into Rich Inlet during Hurricane Ophelia in October 2005 (see Section 7.0). Since the 12 wave cases in Table 11-5 did not specifically include a Category 1 hurricane, a direct comparison of the model results to the storm-dominated changes was not appropriate. However, the model results followed the general erosion patterns on Hutaff Island between 1996 and 2000, which were characterized by accretion on the south end of the island (profiles 145+00 to 175+00) and erosion to the north (see Figure 11-39).

Net sediment transport during the final calibration run appears in Figure 11-40. In general, the sediment transport predicted by the model on the north end of Figure Eight Island is consistent with the short-term sediment budget in Figure 8-4.

Based on the results in Figures 11-38 to 11-40, the Delft3DFLOW and SWAN model provide a realistic description of the waves (Figures 11-41 and 11-42), currents (Figures 11-23 and 11-29), and erosion patterns (Figures 11-38 and 11-39) along Figure Eight Island and Hutaff Island. Accordingly, the model setup in Tables 11-5 and 11-6 was adopted to evaluate the various erosion control alternatives in Section 9.0.

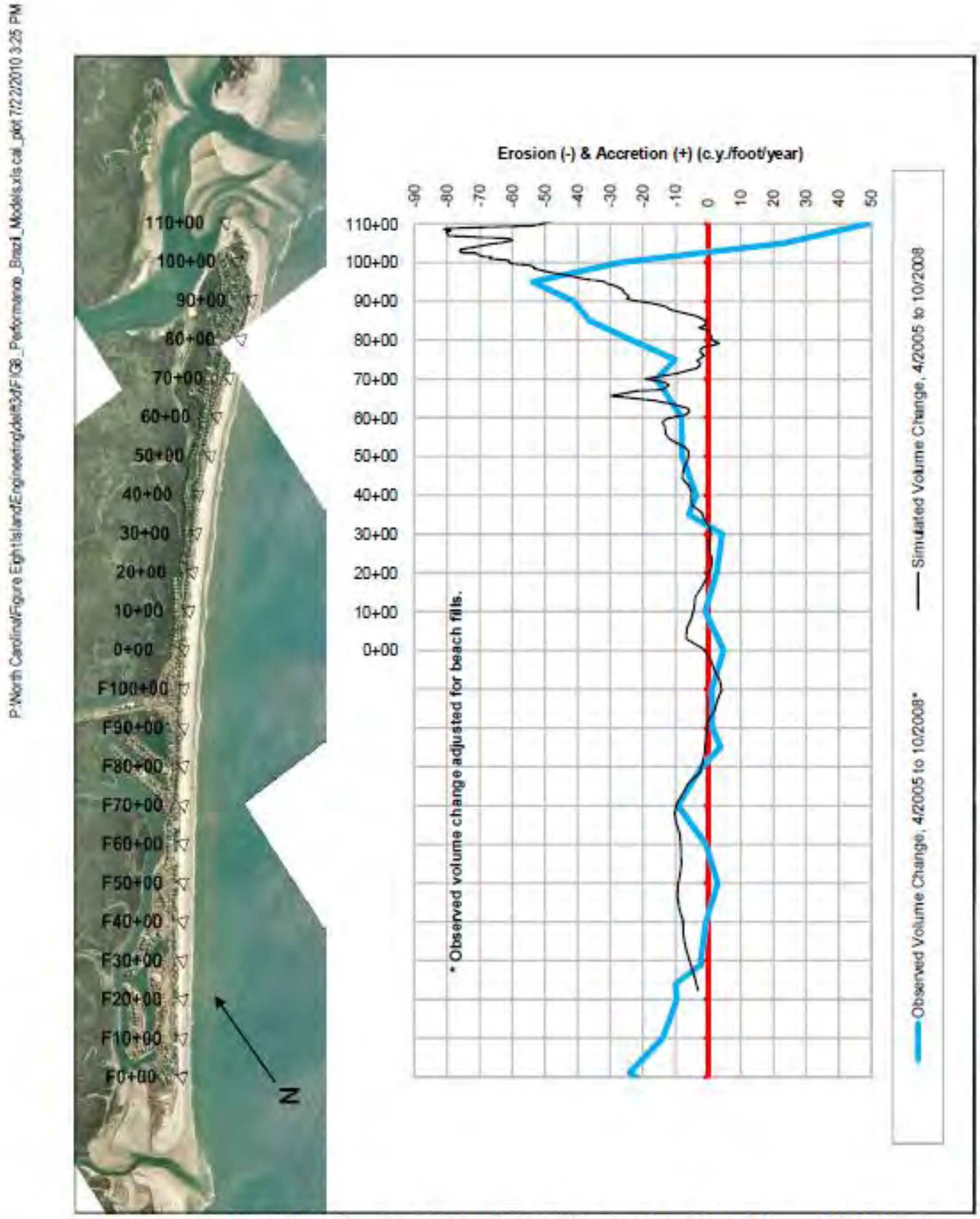

FIGURE 11-38: Delft3D Erosion & Deposition Calibration Results on Figure Eight Island.

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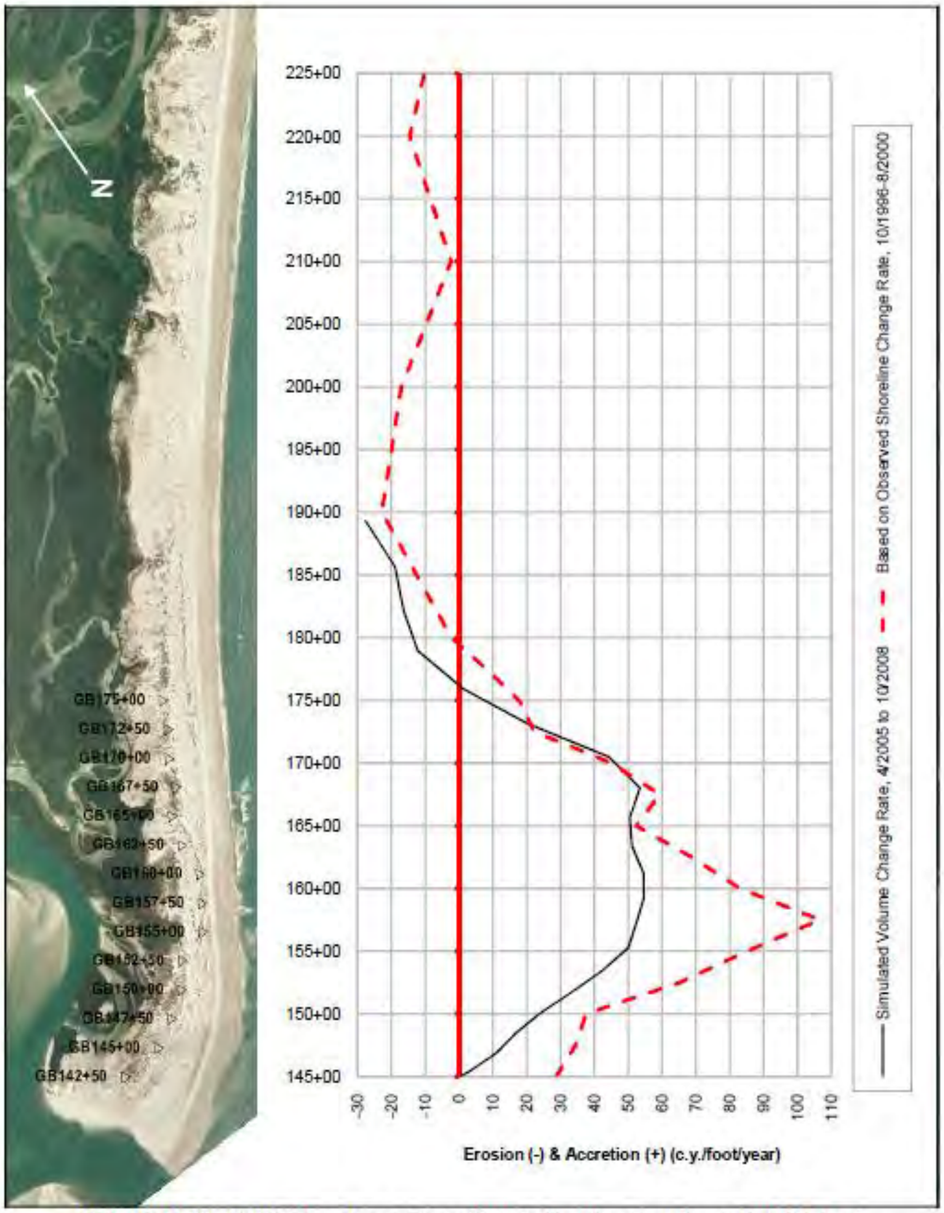

FIGURE 11-39: Delft3D Erosion & Deposition Calibration Results on Hutaff Island.

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FIGURE 11-40: Comparison of the Net Longshore Sediment Transport Based on the Final Delft3D Calibration Run and the 2005-2007 Sediment Budget.

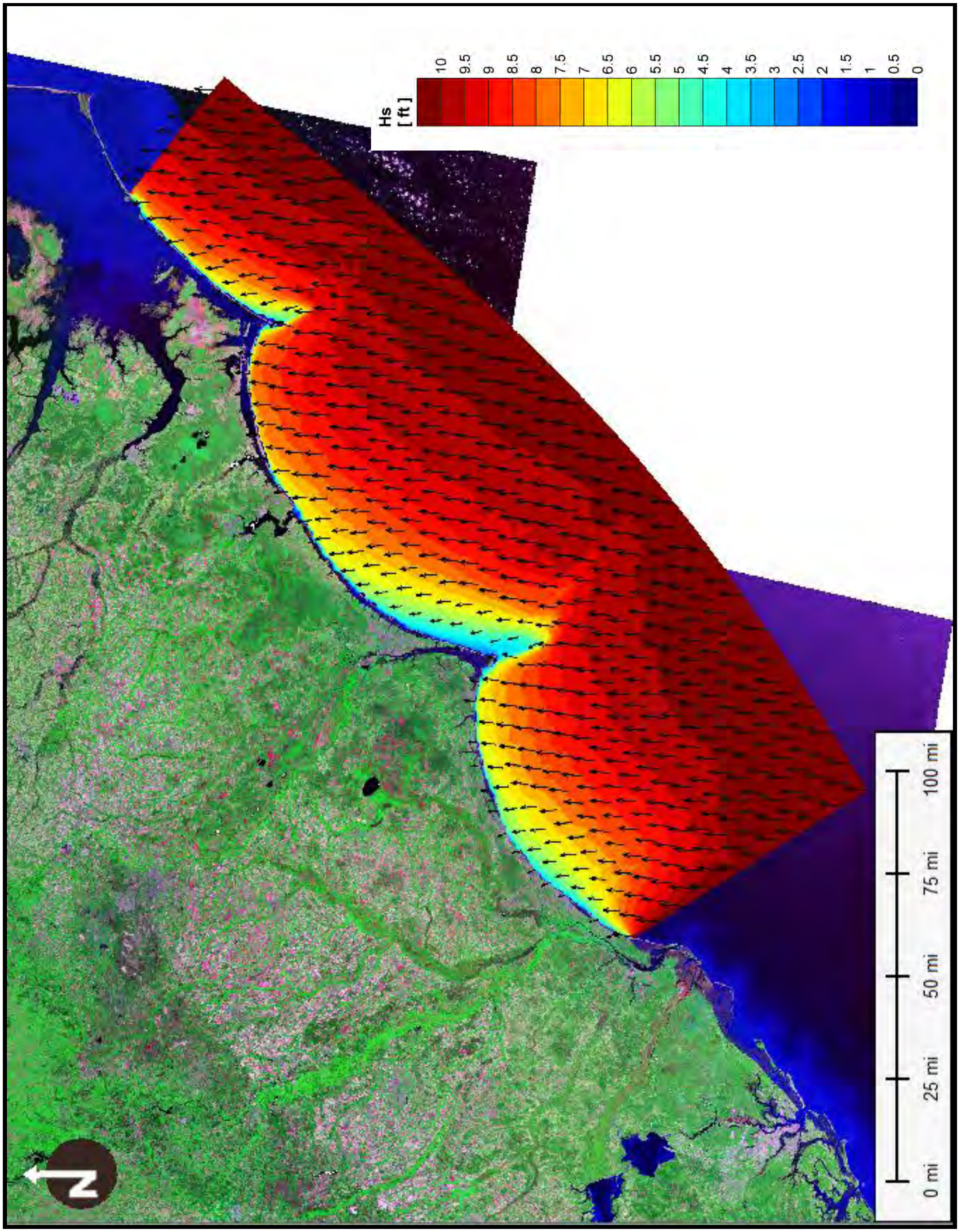


FIGURE 11-41: Typical Wave Transformation Patterns on the Regional Wave Grid (Offshore Boundary Condition - Hs: 10.3 feet; Tp: 7.3 seconds; Dir: 187 degrees).

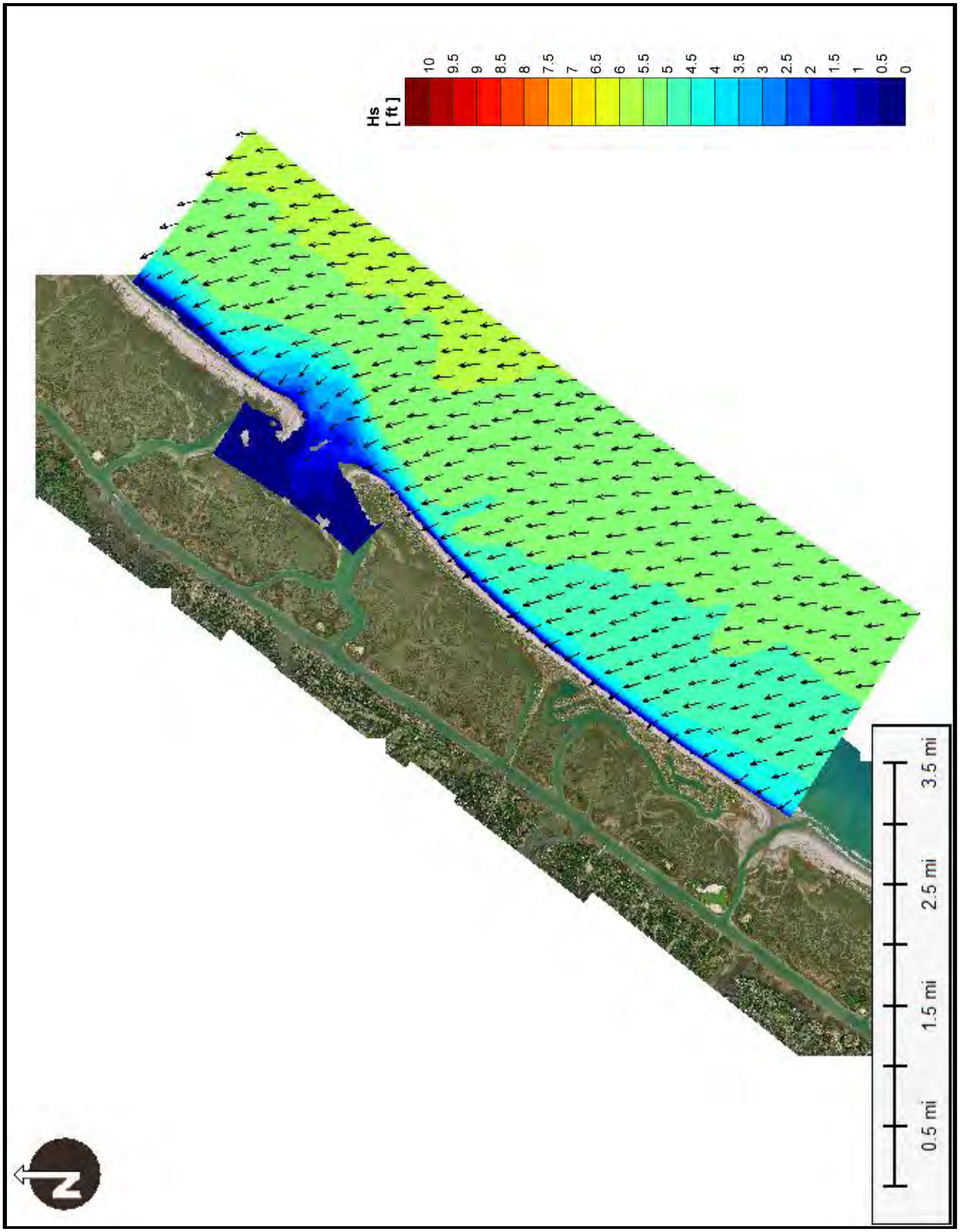


FIGURE 11-42: Typical Wave Transformation Patterns on the Local Wave Grid (Offshore Boundary Condition - Hs: 10.3 feet; Tp: 7.3 seconds; Dir: 187 degrees).

TABLE 11-6
DELFT3FLOW AND SWAN MODEL SETUP
FIGURE EIGHT ISLAND, NC

SWAN model parameters	
Gravity	9.81 m/s² (32.2 feet/s²)
Water Density	1025 kg/m³ (64 lbm/foot³)
Min. Depth for Computations	0.05 m (0.16 feet)
Spectra Type	JONSWAP
Peak Enhancement Factor	3.3
Directional Space	0 to 360 deg.
Number of Direction Bands	36
Lowest Frequency	0.05 hz
Highest Frequency	1 Hz
Number of Frequency Bands	24
Depth Induced Breaking - α_b	1
Depth Induced breaking – γ (H_b/d_b)	0.73
Bottom Friction Roughness Scale	0.01 m (0.4")
Diffraction Smoothing Coefficient	0.2
Diffraction Smoothing Steps	5
Frequency Shift	Activated
Refraction	Activated
Wind growth	Activated
Whitecapping	Activated
Quadruplets	Activated
Percent Accuracy to Accept Iteration	95%
Max. Number of Iterations	15
DELFT3DFLOW Hydrodynamic Parameters	
Number of Vertical Layers	5
Time Step	30 seconds
East Boundary Type	Water level – Harmonic
East Boundary Amplitude & Period	2.17 feet / 745 minutes
East Boundary Reflection Parameter α	0
North Boundary Type	Zero Gradient (Neumann)
South Boundary Type	Zero Gradient (Neumann)
Gravity	9.81 m/s² (32.2 feet/s²)
Water Density	1025 kg/m³ (64 lbm/foot³)
Roughness Chezy	(see Figure 11-19)
Stress Formulation Due To Wave Forces	Fredsoe
Horizontal Eddy Viscosity	5 m^2/s (52 $foot^2$/s)
3-D Turbulence Model	K-Epsilon
Advection Scheme For Momentum	Cyclic
Advection Scheme For Transport	Cyclic
Horizontal Forester Filter	Activated
Freshwater Discharges	No

TABLE 11-6 (continued)
DELFT3FLOW AND SWAN MODEL SETUP
FIGURE EIGHT ISLAND, NC

DELFT3DFLOW Sediment Transport and Morphology Parameters	
Reference Density for Hindered Setting	1600 kg/m³ (99.9 lbm/foot³)
Specific Density	2650 kg/m³ (165.4 lbm/foot³)
Dry Bed Density	1600 kg/m³ (99.9 lbm/foot³)
Median Diameter	0.3 mm
Update Bathymetry During Simulation	Yes
Spin Up Period	725 minutes
Min. Depth for Sediment Calculation	0.1 m (4")
VanRijn Reference Height Factor	1 (2")
Threshold Sediment Thickness	0.05 m
Estimated Ripple Height Factor	2
Dry Cell Erosion Factor (THETSD)	1
Multiplication Factor For Suspended Sed. Ref. Concentration (SUS)	1.4
Multiplication Factor For Bed-Load Transport Vector Magnitude (BED)	0.8
Wave-Related (Orbital Motions) Suspended Sed. Transport Factor (SUSW)	0.1
Wave-Related (Orbital Motions) Bed-Load Sed. Transport Factor (BEDW)	0.1
Horizontal Eddy Diffusivity	2 m^2/s (22 $foot^2/s$)

11.4 Future Conditions

Model results given the 1999-2007 wave cases in Table 11-5 and the "worst case" inlet survey (April 2006) are detailed in Sub-Appendix B1 and below. The model results discussed below should be interpreted in relative terms by comparing the model results for the No Action Alternative (Alternative 2) to the results obtained for the other alternatives. In this regard, all model simulations for formulation of the alternatives and evaluating impacts of the alternatives were based on "worst case" conditions that existed along the north end of Figure Eight Island in 2006-07. At that time, the bar channel of Rich Inlet had migrated to a point near the south end of Hutaff Island and the channel had assumed an alignment toward Hutaff Island. Under these inlet bar channel conditions, the north end of Figure Eight Island normally experiences severe erosion. It is these "worst case" conditions the beach and inlet management plan is addressing.

In 2010, the bar channel of Rich Inlet assumed an alignment toward the north end of Figure Eight Island which has resulted in an ephemeral build-up of material along the north end of the island. Given the historic behavior of Rich Inlet, as discussed by Dr. William J. Cleary in Sub-Appendix A of the Engineering Report (Appendix B), this condition is not expected to prevail for any substantial period of time and the channel will again swing toward Hutaff Island resulting in a renewed round of severe erosion.

If implementation of one of the management alternatives occurs within the near future, the conditions at the time of implementation will likely be similar to the conditions existing in 2012. Therefore, Delft3D model simulations were conducted using 2012 inlet and shoreline data as the initial model conditions. The model simulations with the 2012 initial conditions were run for Alternatives 2, 3, 4, and 5D.

11.4.1 Alternative 1 – No Action

Alternative 1 assumes that the present strategies to manage the island's shoreline in Table 6-2 will continue into the future. As shown in Table 6-2, dredging and fill operations around Figure Eight Island are highly variable in terms of timing and quantity, since they are dependent on decisions made by the Association, State agencies, and the Federal government. This sort of uncertainty cannot be incorporated into the Delft3D model. For this reason, Alternative 1 was not simulated.

11.4.2 Alternative 2 – Abandon/Retreat

Alternative 2 assumes that there will be no more beach fill, dune maintenance, inlet maintenance, or sand bag placement operations. Accordingly, this alternative is the true "Without-Project" scenario, and is the basis for evaluating the performance and impacts of the other alternatives. It is important to note that Alternative 2 does *not* approximate what occurred between 2006 and 2012.

In general, the model results suggest that given eroded conditions similar to those in 2006, the main channel of Rich Inlet would migrate towards the middle of the inlet (Figure 11-43 and Sub-Appendix B1). As part of this process, the flood channel on the southwestern side of the inlet, which connects Nixon Channel to the ocean, would start to close. Within Nixon Channel, the depth near the north end of Beach Road would have increased from -16 feet NAVD to -23 feet NAVD. These changes would be accompanied gains on the southern tip of Hutaff Island and severe erosion and shoreline retreat on the north end of Figure Eight Island (see Figure 11-44).

Under a scenario similar to the 2012 conditions, the model results suggest that the main channel of Rich Inlet would change its orientation from north-northwest/south-southeast to west-northwest/east-southeast (see Figure 11-45). These changes would be accompanied by losses on the southern end of Hutaff Island and gains on the sandy area on the south side of Rich Inlet (see Figure 11-46). However, losses would also occur along the beach between profiles 90+00 (Inlet Hook Road) and 105+00 due to the shifting of the ebb shoal. In addition, the south end of Green Channel could shoal in (see Figure 11-45), which is consistent with observations by Dr. William Cleary. Overall, the simulated changes around Rich Inlet given the 2012 conditions are similar to those that occurred between 1993 and 1999 (see Figure 11-47). In both cases, the channel of the inlet switches its orientation, resulting in a shifting of the ebb shoal and narrowing of the beach near Inlet Hook Road.

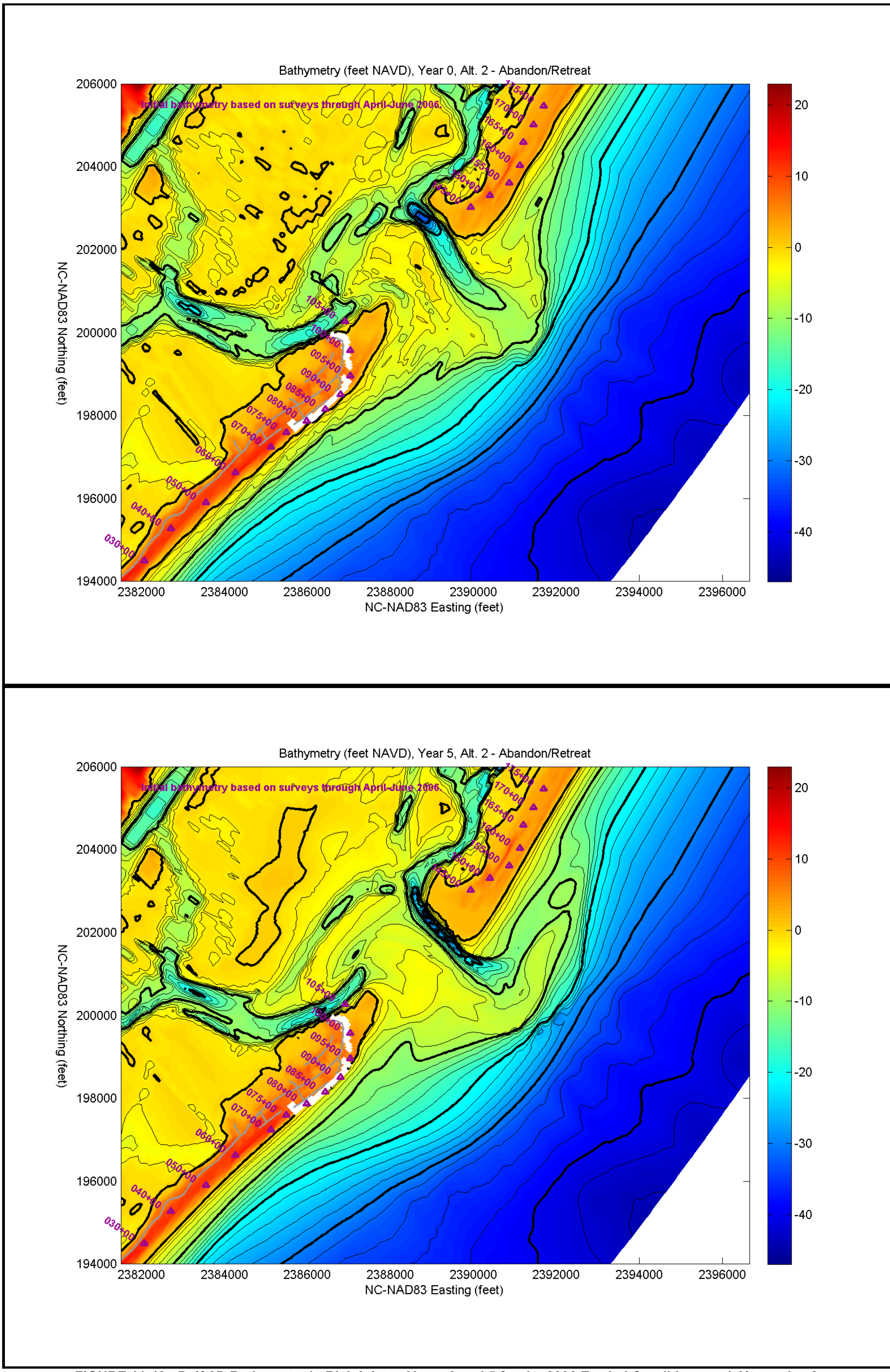


FIGURE 11-43: Delft3D Bathymetry in Rich Inlet at Years 0 and 5 for the 2006 Eroded Conditions and Alternative 2.

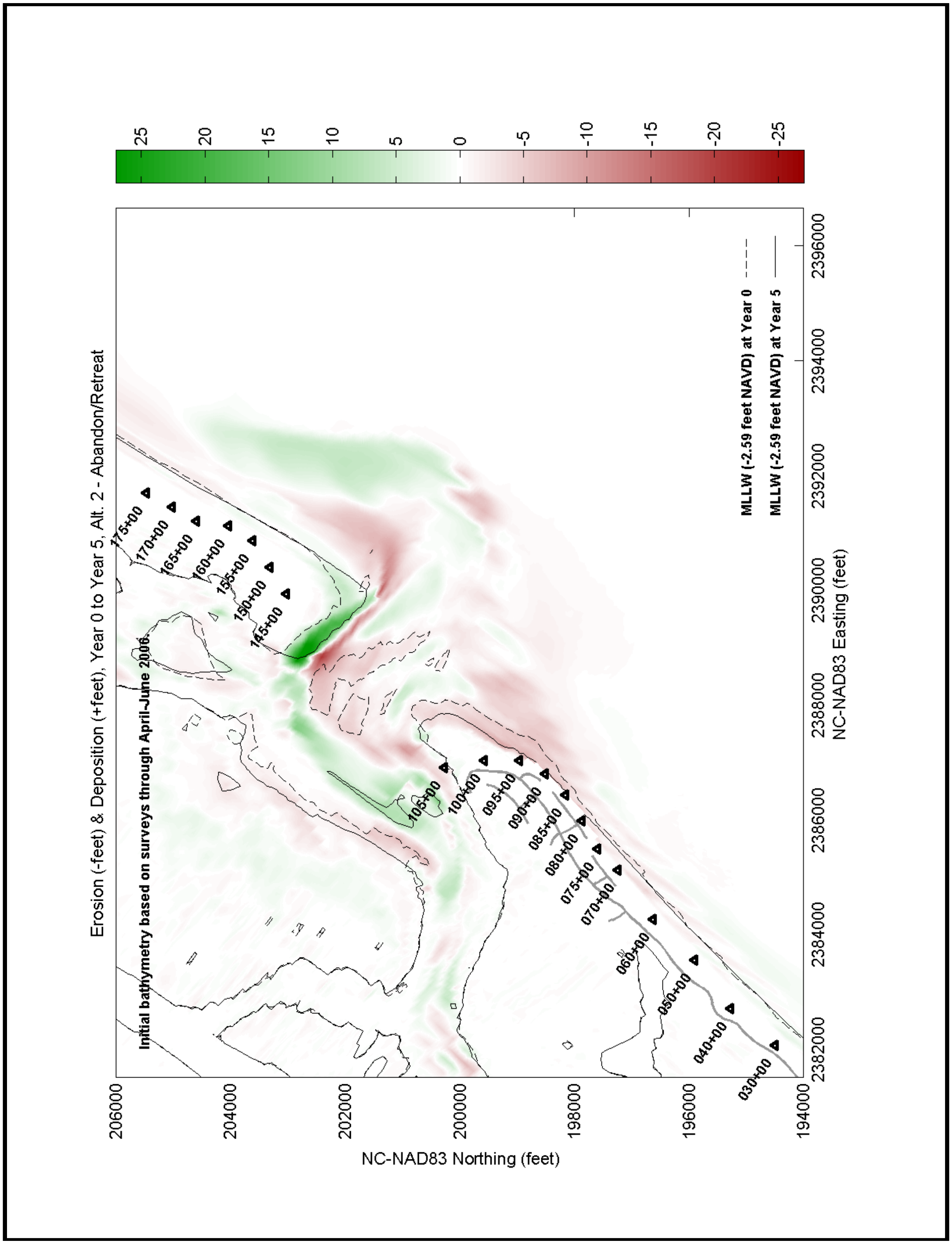
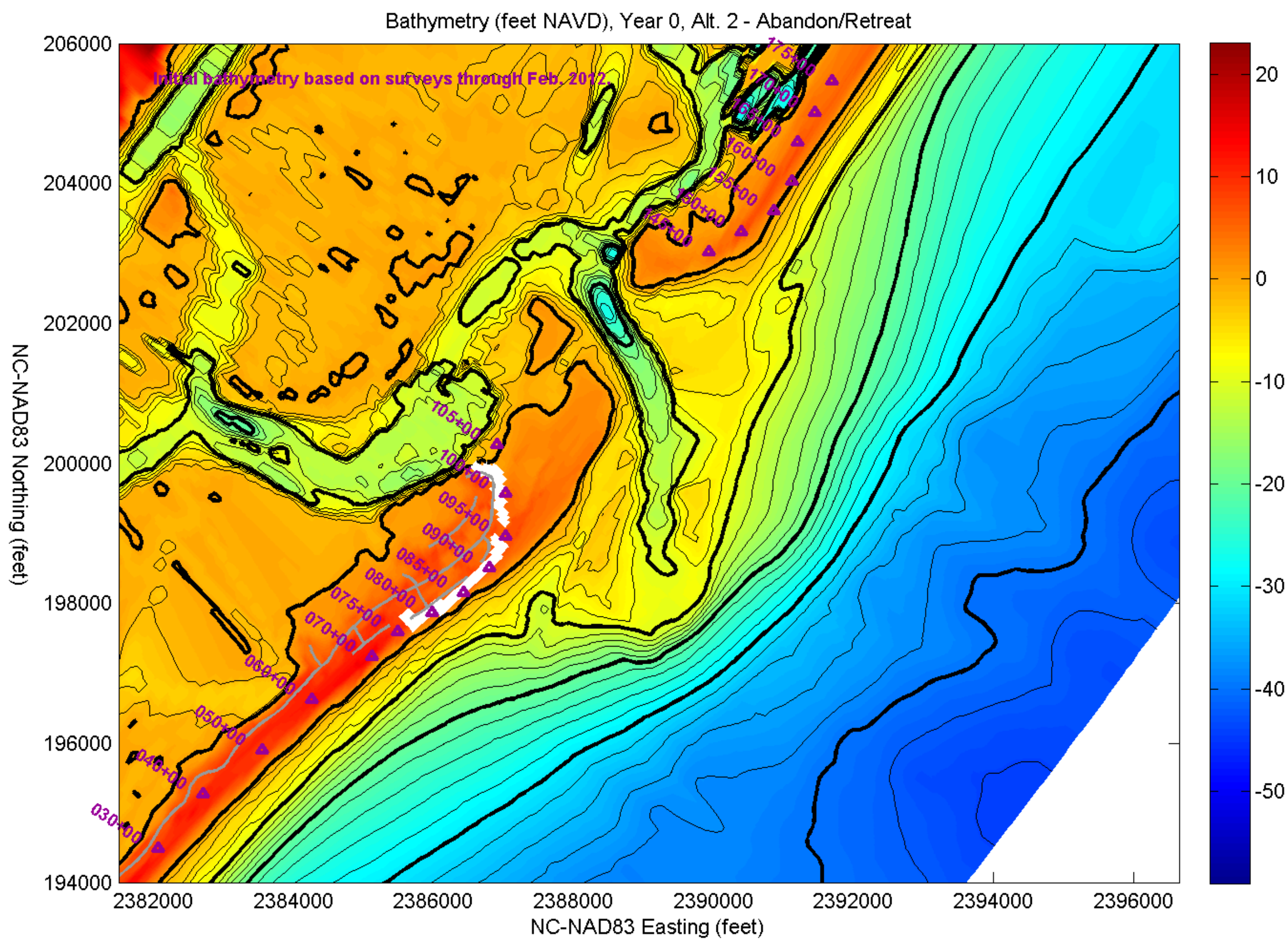


FIGURE 11-44: Delft3D Erosion and Deposition for the 2006 Eroded Conditions and Alternative 2.



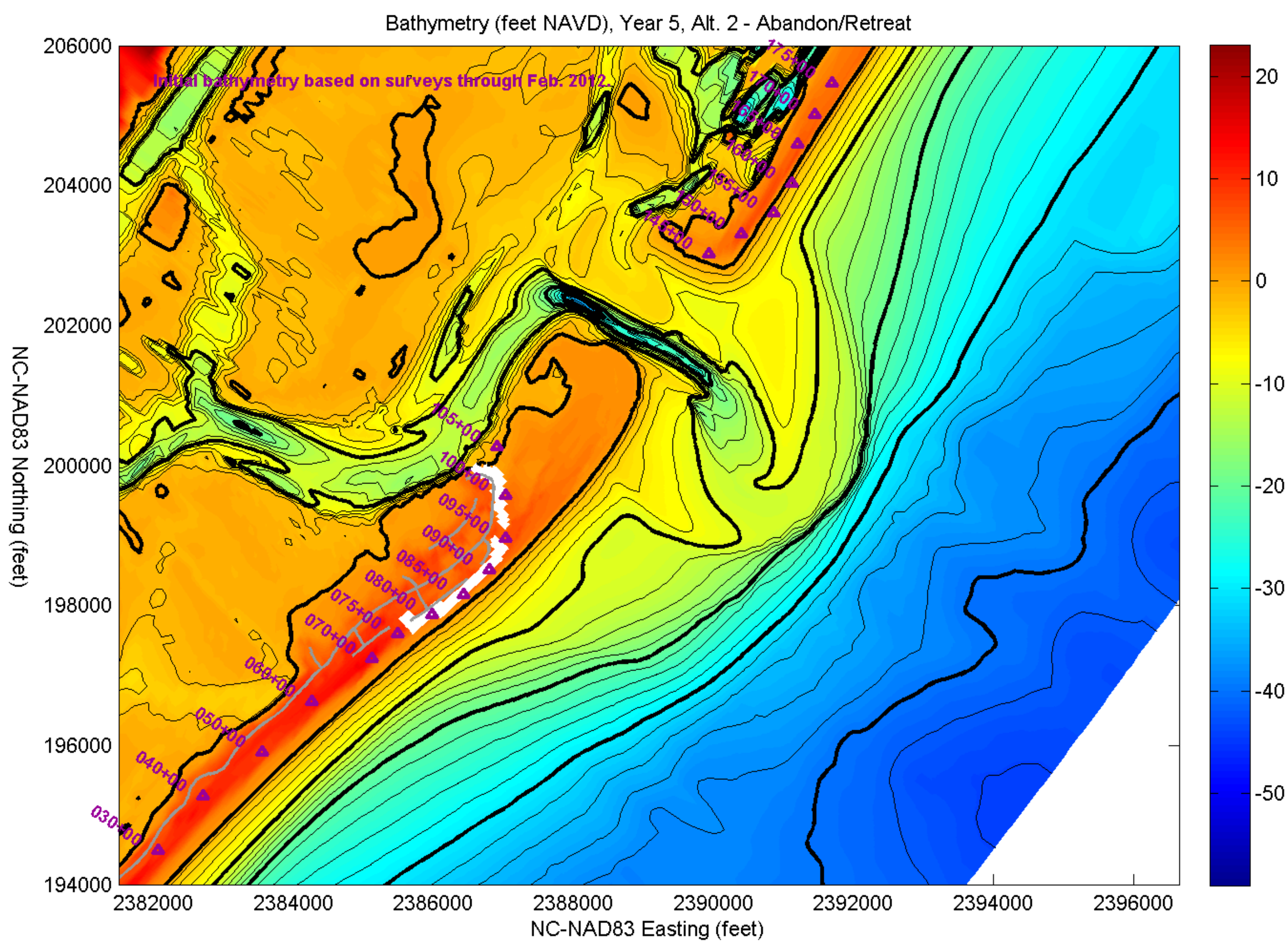


FIGURE 11-45: Delft3D Bathymetry in Rich Inlet at Years 0 and 5 for the 2012 Conditions and Alternative 2.

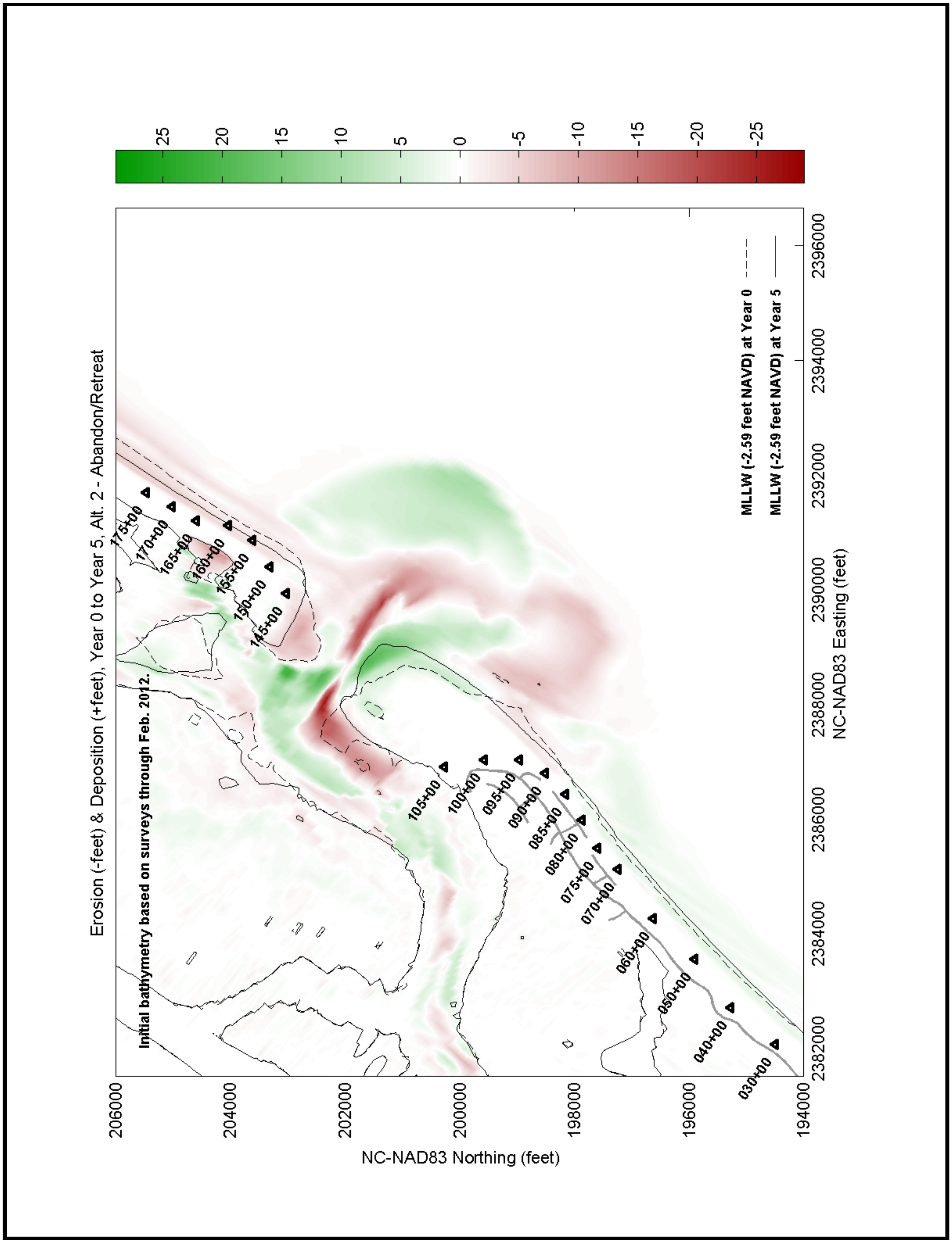


FIGURE 11-46: Delft3D Erosion and Deposition for the 2012 Conditions and Alternative 2.

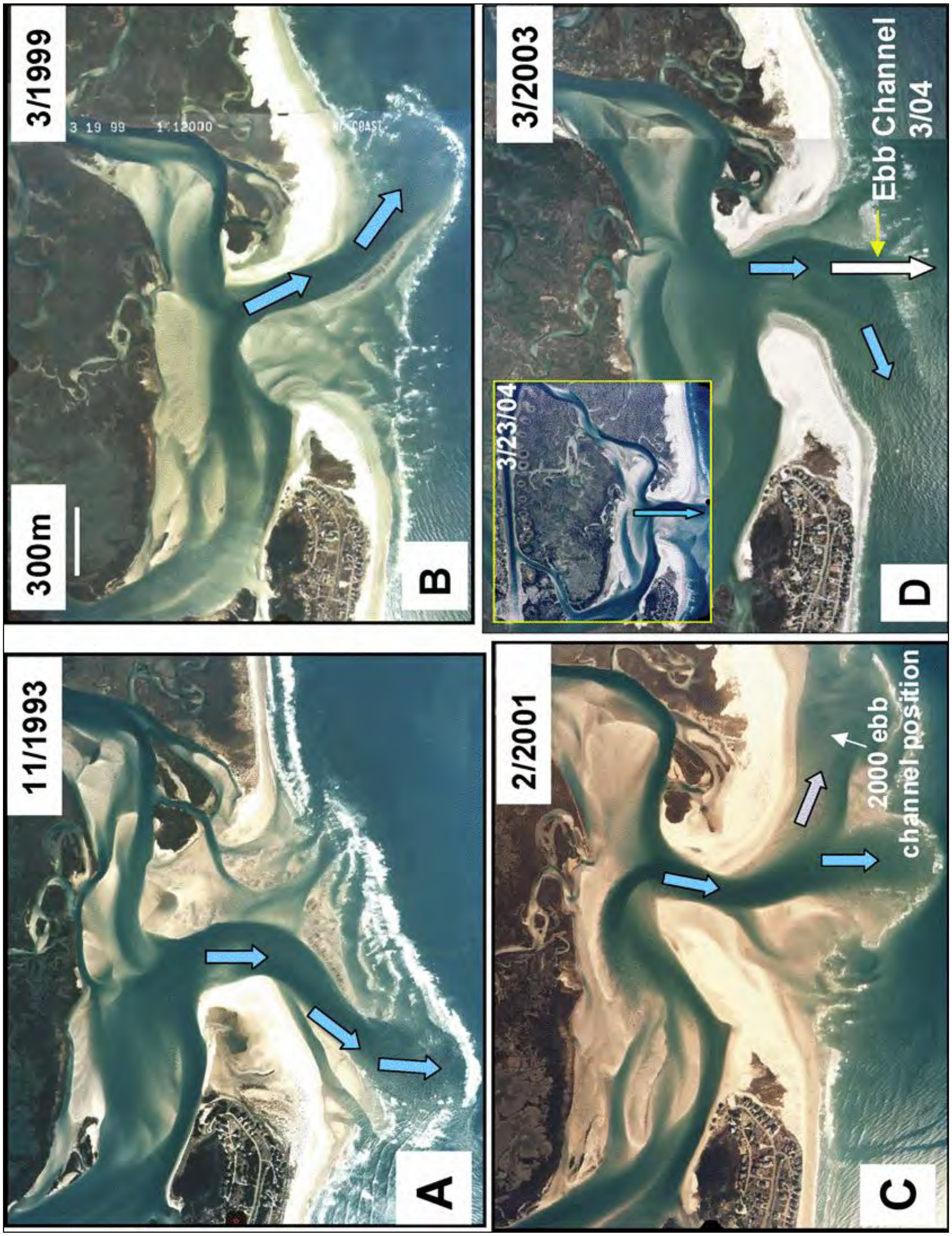


FIGURE 11-47: Aerial photographs of Rich Inlet (11/1993-3/2004). Photographs A-D depict shoreline changes related to deflection of ebb channel (blue arrows) and subsequent repositioning and reorientation through ebb delta breaching in late 2002 (C) and late 2003 after channel deflected toward Figure Eight Island (D). Insert in D shows ebb channel as of March 2004 (Figure and caption from Cleary & Jackson, 2004).

Simulated volume changes along the beach given the 2006 eroded conditions appear in Figure 11-48, Table 11-7, and Sub-Appendix B1. Table 11-7 also includes model indicated volume changes for the other alternatives which will be referenced in the discussion of each respective alternative. Table 11-8 provides the percent of beach fill remaining within two beach segments on Figure Eight Island for all the alternatives that include beach fill. Simulated volume changes along the beach given the 2012 conditions also appear in Sub-Appendix B2.

Model results for Alternative 3 appear in Sub-Appendix B1, Table 11-7, Table 11-8, Table 11-9, and Figures 11-48 through 11-54. Since the Delft3D-FLOW model is not able to incorporate renourishment during a model simulation, these results neglect renourishment at Year 5. It should also be noted that in the model simulations, the beach fill along Nixon Channel was based on a preliminary design with 65,000 cubic yards from profiles RIN12+00 to RIN30+00, rather than the final design with 57,000 cubic yards from profiles RIN16+00 to RIN30+00 only. Given the resolution of the Delft3D-FLOW model, the differences between the preliminary design and the final design do not have a large effect on the model results.

If Alternative 3 were constructed under eroded conditions similar to those in 2006, the straight contours of the initial dredge cut would evolve into a broad arc (see Figure 11-49). The connecting cut into Green Channel could become more constricted over time, although it would not shoal in completely. Within Nixon Channel, the depth near the north end of Beach Road would be similar to the Year 0 condition, allowing some of the fill placed along the adjacent fill area to remain in place at Year 5 (see Figure 11-50). North of profile 85+00 (13 Comber Road), erosion into the pre-construction beach face could occur (see Figure 11-50). However, the degree of erosion would be less than what would occur under the Abandon/Retreat scenario (see Figure 11-51), and the net volume changes over the active profile as a whole (Table 11-7, Figure 11-48) suggest that except for the north taper, complete loss of fill would not occur before Year 5. Refilling of the designated dredge cut would provide enough material for renourishment (see Figure 11-49 and Table 11-9). On Hutaff Island, erosion rates could increase south of profile 175+00. However, 2/3 of the closure dike that would adjoin the southern tip of the island would remain in place (see Figures 11-49 through 11-51).

If Alternative 3 were constructed under conditions similar to those in 2012, the main channel of Rich Inlet would evolve to a west-northwest/east-southeast orientation (see Figure 11-52). The connection between the entrance channel and Green Channel would remain open, fulfilling the intent of that design feature. Along Nixon Channel, much of the fill placed at Year 0 would still be remaining at Year 5 (see Figure 11-53). However, along the oceanfront, erosion into the pre-construction profile could occur by Year 5 north of profile 85+00 (13 Comber Road) (see Figure 11-53 and Sub-Appendix B1). The degree of erosion would be greater than what would occur under the Abandon/Retreat scenario (see Figure 11-54). Along Hutaff Island, project-related impacts north of profile 145+00 would be relatively small (see Figure 11-54).

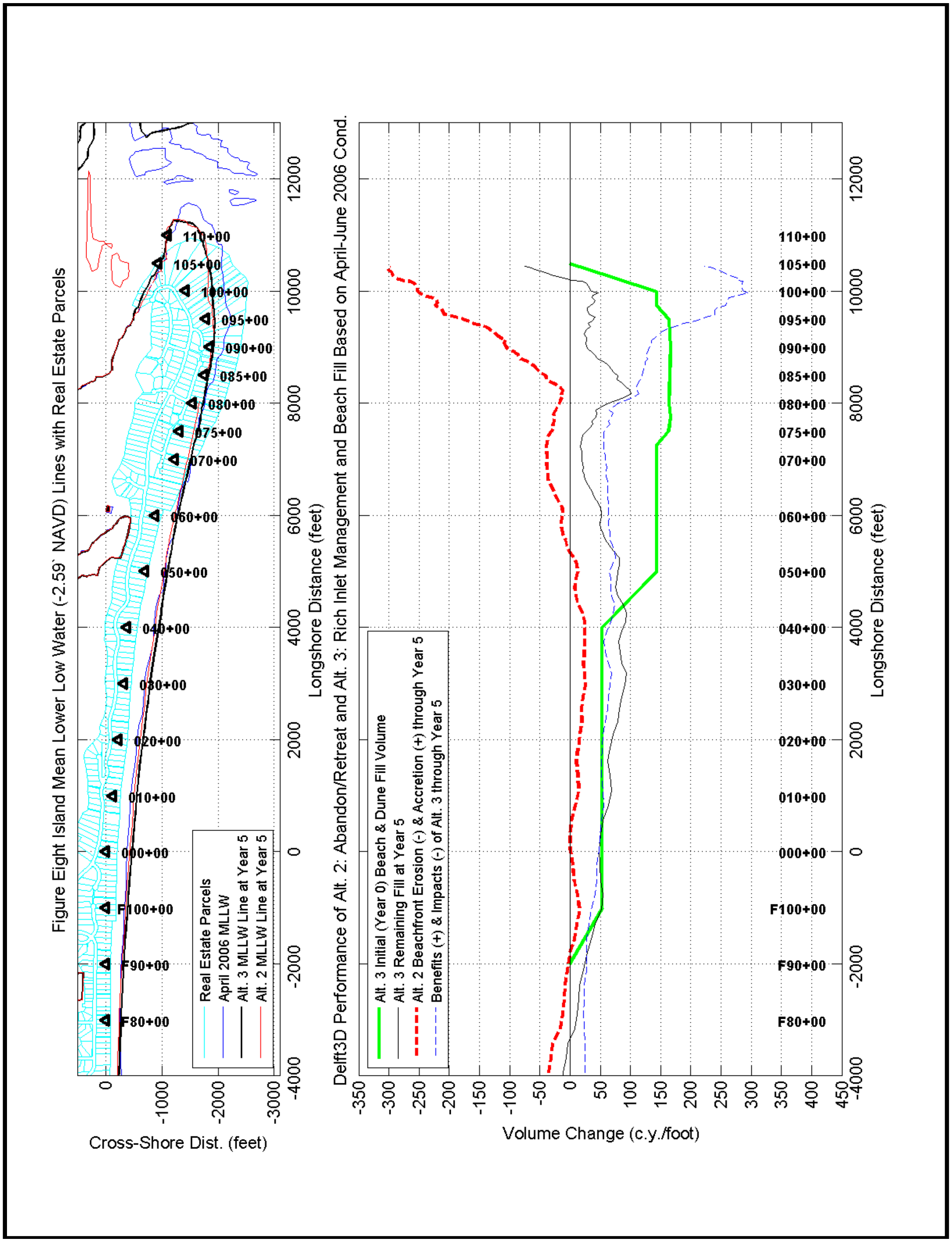


FIGURE 11-48: Delft3D Volume Changes for the 2006 Eroded Conditions and Alternatives 2 and 3.

TABLE 11-7
DELFT3D VOLUME CHANGES GIVEN THE 2006 ERODED CONDITIONS

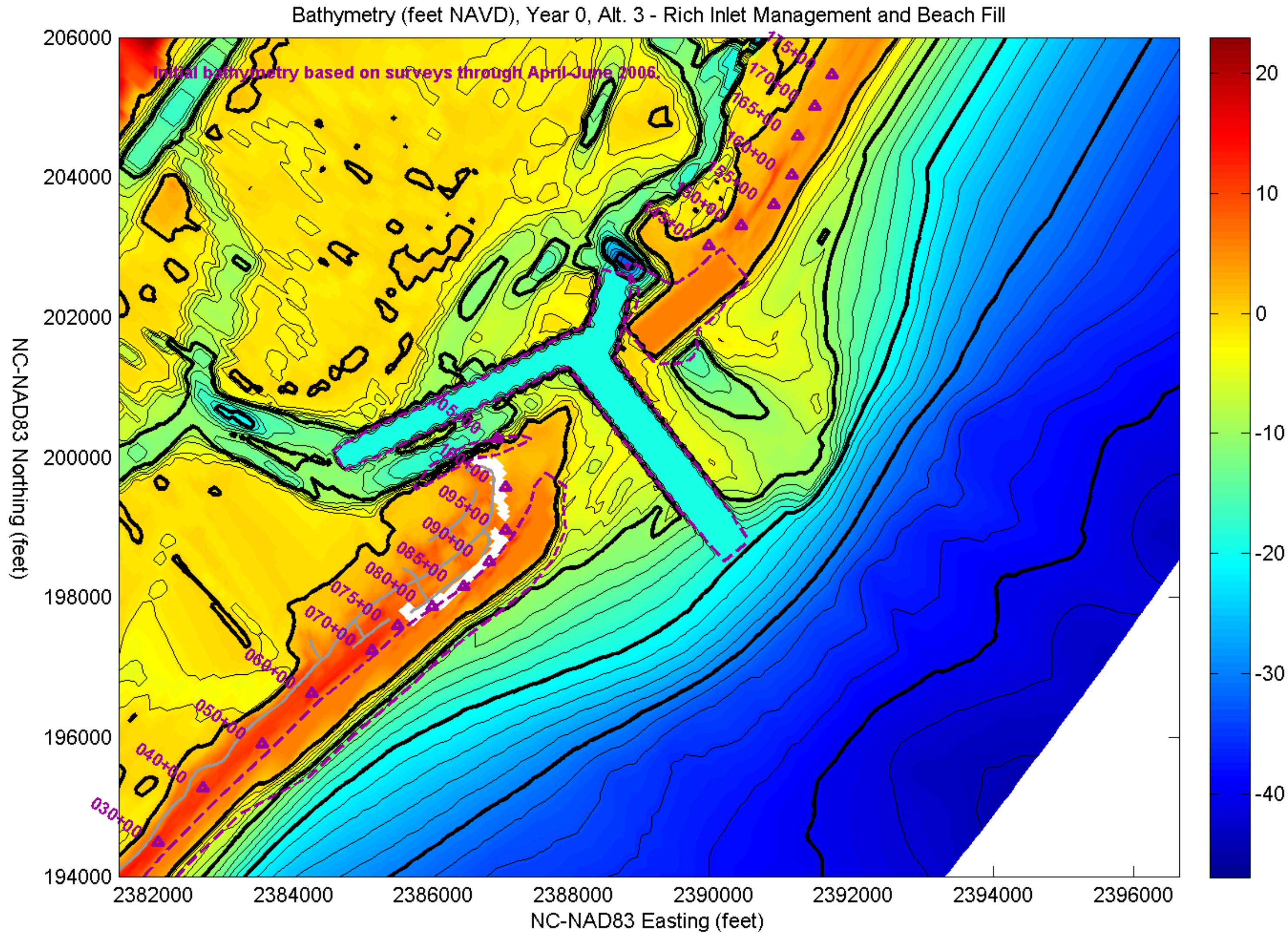
Profile Lines	Beach Length (feet)	Volume Change (cubic yards) At the end of Year 5 of the Simulation Alt. 2	Alt. 3	Alt. 4	Alt. 5C	Alt. 5D	Alt. 5B in DEIS
FIGURE EIGHT ISLAND							
F90+00 to 60+00	8,001	+88,000	-11,000	+148,000	+102,000	+316,000	+251,000
60+00 to 105+00	4,500	-420,000	-495,000	-881,000	-467,000	-288,000	-257,000
HUTAFF ISLAND							
148+60 to 175+00	2,640	+265,000	-155,000	+285,000	-165,000	+365,000	+360,000
175+00 to 215+00	4,000	-175,000	-130,000	-150,000	-260,000	-100,000	-105,000

TABLE 11-8
DELFT3D PERCENT OF BEACH FILL REMAINING GIVEN THE 2006 ERODED CONDITIONS

Alternative	Shoreline Segment	Percent of Beach Fill Remaining After Year: 0 (Fill Volume cy)	1	2	3	4	5
2	F90+00 to 60+00	0	NA	NA	NA	NA	NA
	60+00 to 105+00	0	NA	NA	NA	NA	NA
3	F90+00 to 60+00	537,000	99.4%	108.2%	109.9%	106.7%	98.0%
	60+00 to 105+00	654,000	72.2%	60.9%	51.1%	43.3%	24.5%
4	F90+00 to 60+00	255,000	124.3%	151.4%	165.1%	168.6%	158.0%
	60+00 to 105+00	656,000	57.0%	30.5%	6.4%	-16.3%	-34.3%
5C	F90+00 to 60+00	429,000	104.4%	116.6%	121.4%	124.9%	123.8%
	60+00 to 105+00	479,000	64.3%	41.5%	25.9%	13.4%	2.5%
5D	F90+00 to 60+00	0	NA	NA	NA	NA	NA
	60+00 to 105+00	238,000	80.2%	45.0%	24.3%	10.4%	-21.2%
5B (DEIS)	F90+00 to 60+00	0	NA	NA	NA	NA	NA
	60+00 to 105+00	198,000	59.6%	33.8%	10.1%	-7.6%	-29.8%

TABLE 11-9
DELFT3D DREDGE MAINTENANCE VOLUMES
EIS ALT. 3 WITH PREFERRED DREDGING OPTION
2006 ERODED CONDITIONS

Year	Re-Dredging Volume to Design Depth (-19' NAVD) (c.y.)			
	Entrance Channel	Nixon Channel	Green Channel	TOTAL
0	0	0	0	0
1	202,000	10,000	72,000	284,000
2	430,000	20,000	173,000	623,000
3	571,000	70,000	142,000	783,000
4	641,000	103,000	132,000	876,000
5	666,000	121,000	140,000	927,000



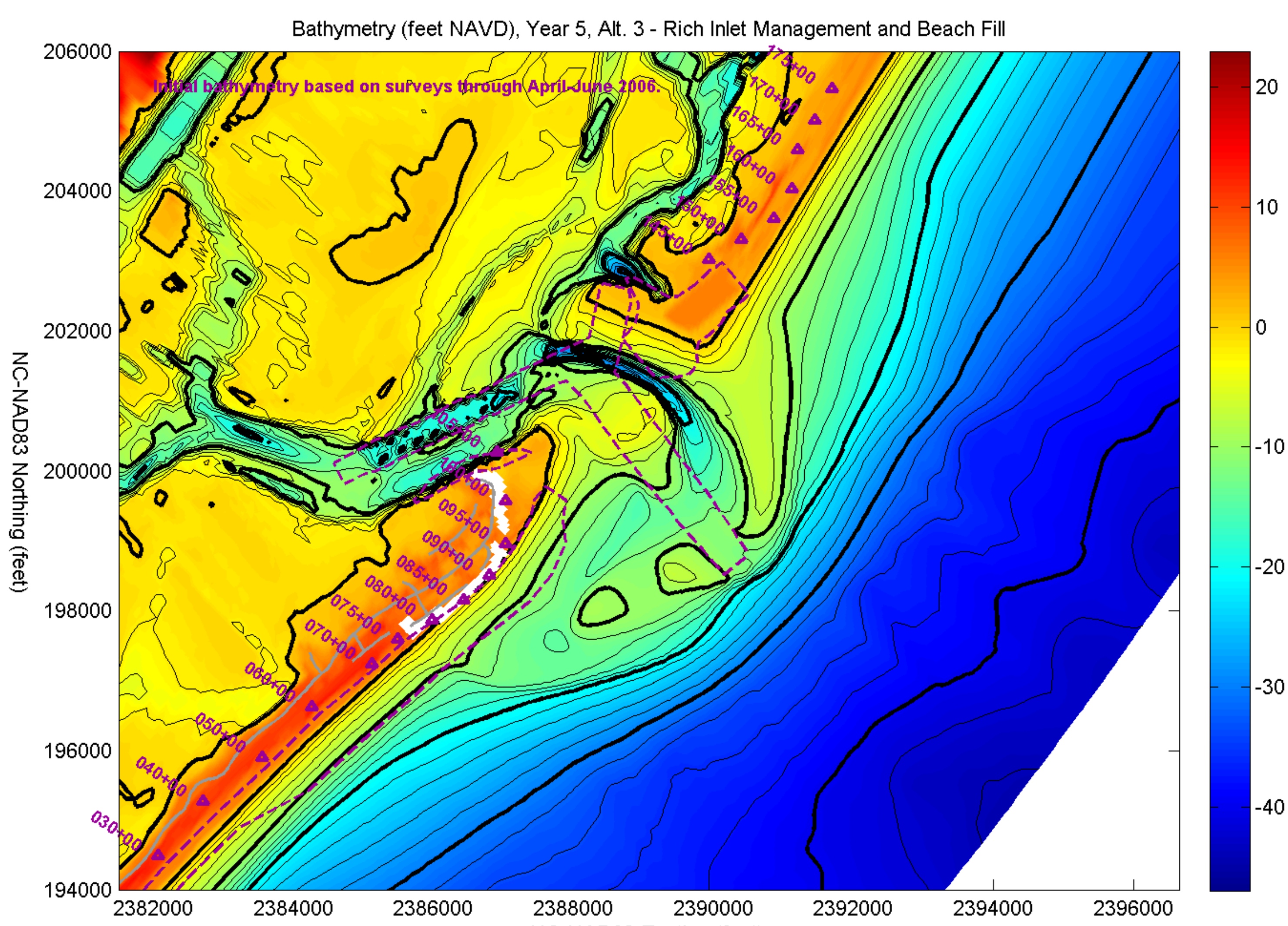


FIGURE 11-49: Delft3D Bathymetry in Rich Inlet at Years 0 and 5 for the 2006 Eroded Conditions and Alternative 3.

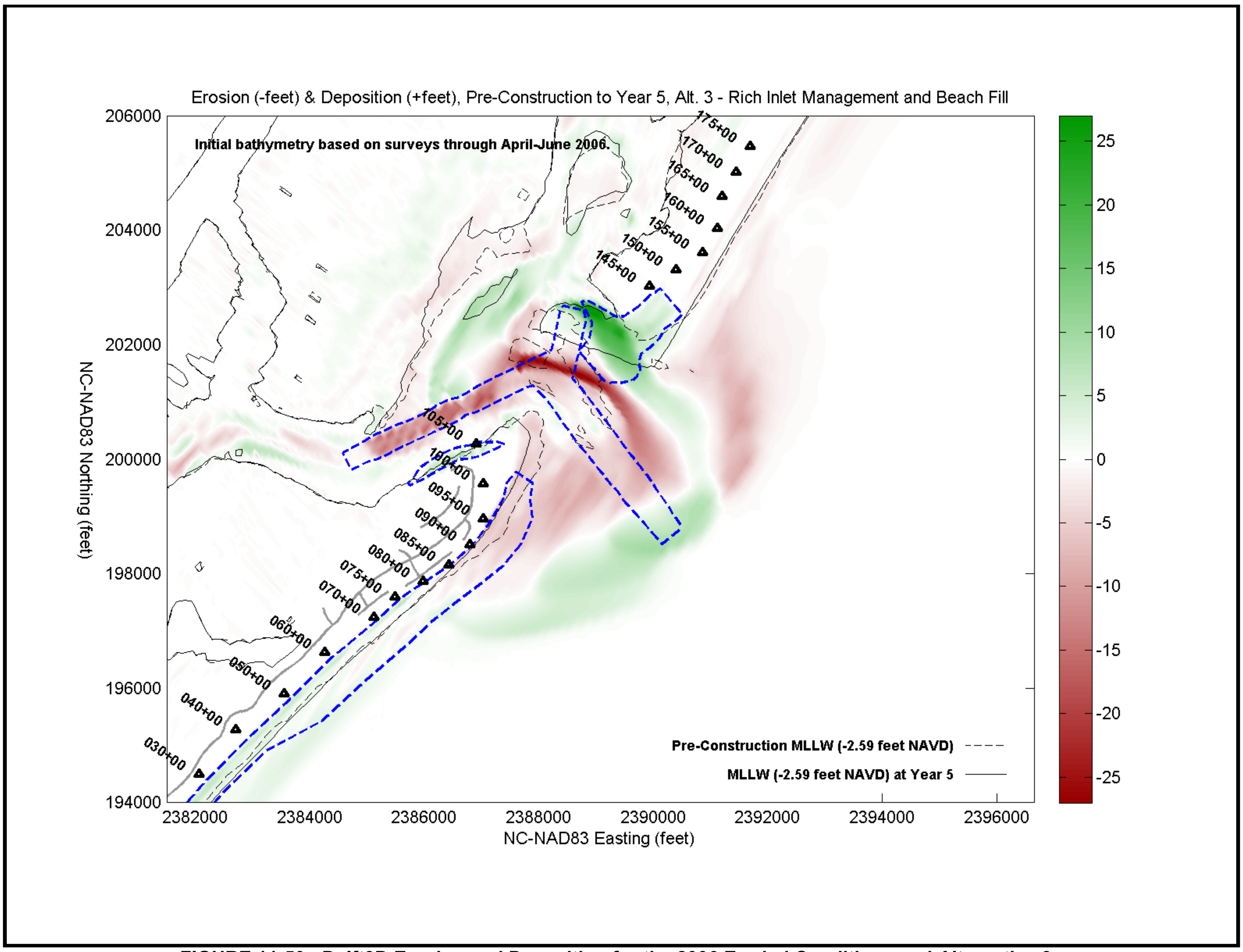


FIGURE 11-50: Delft3D Erosion and Deposition for the 2006 Eroded Conditions and Alternative 3.

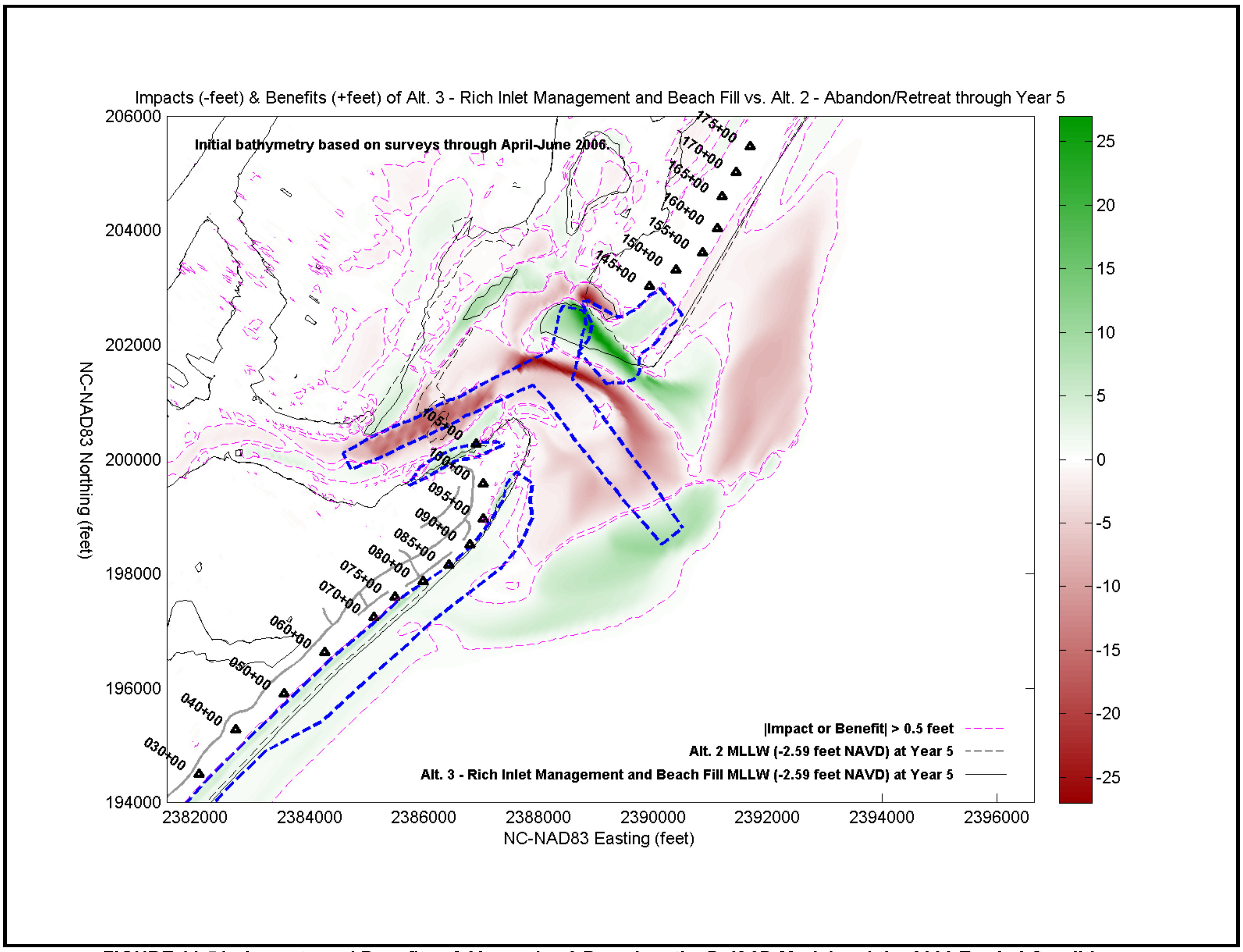
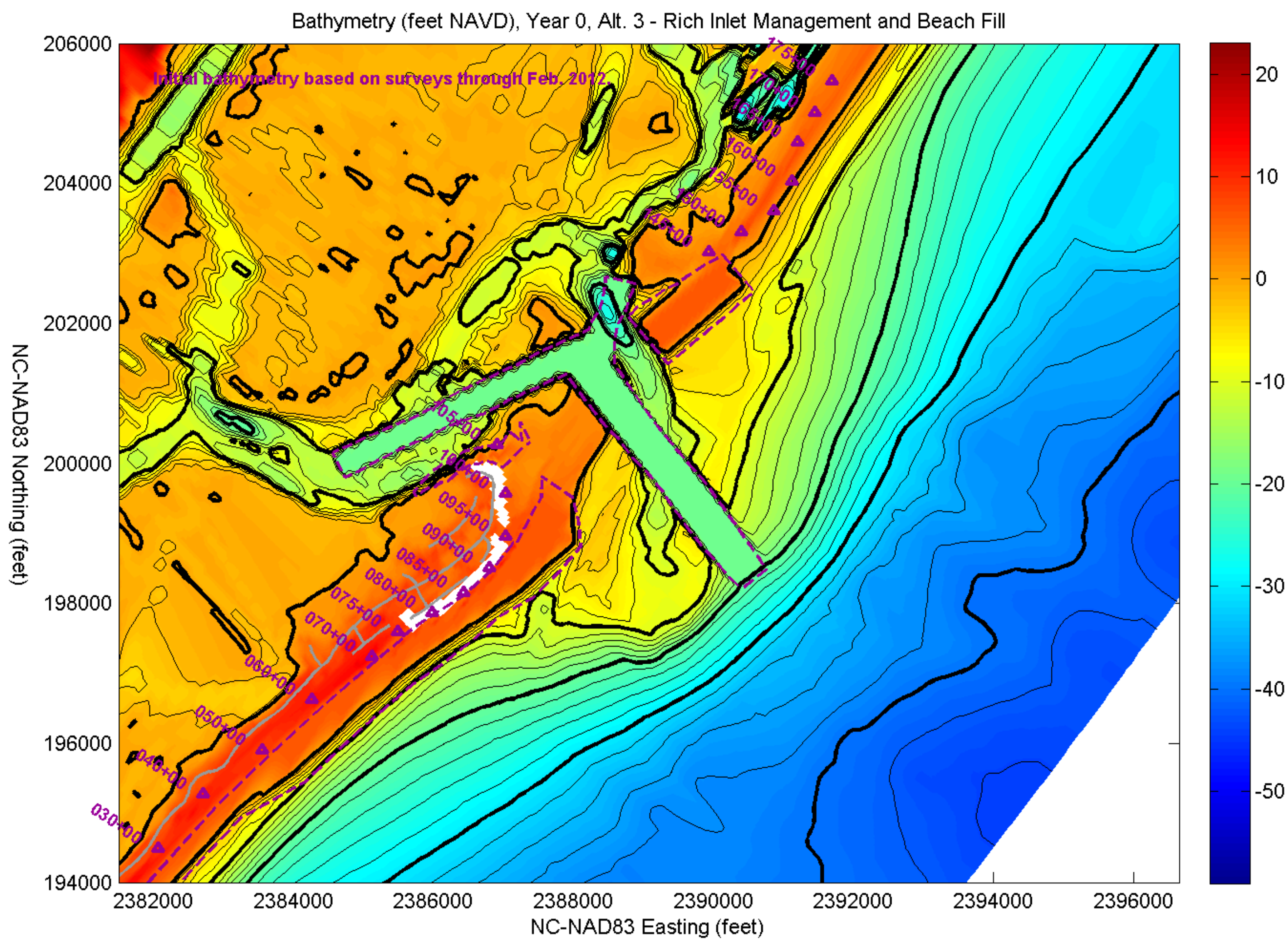


FIGURE 11-51: Impacts and Benefits of Alternative 3 Based on the Delft3D Model and the 2006 Eroded Conditions.



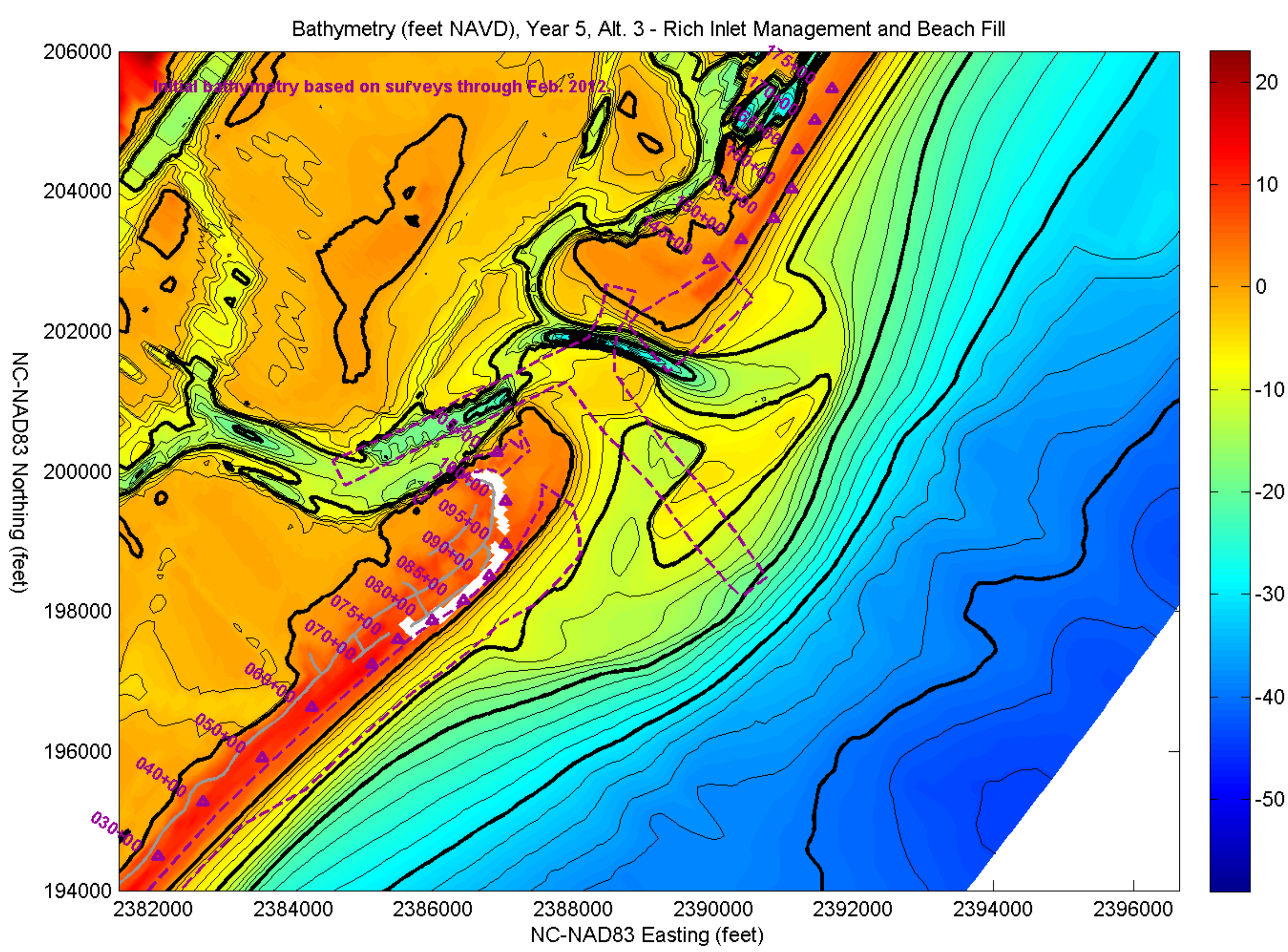


FIGURE 11-52: Delft3D Bathymetry in Rich Inlet at Years 0 and 5 for the 2012 Conditions and Alternative 3.

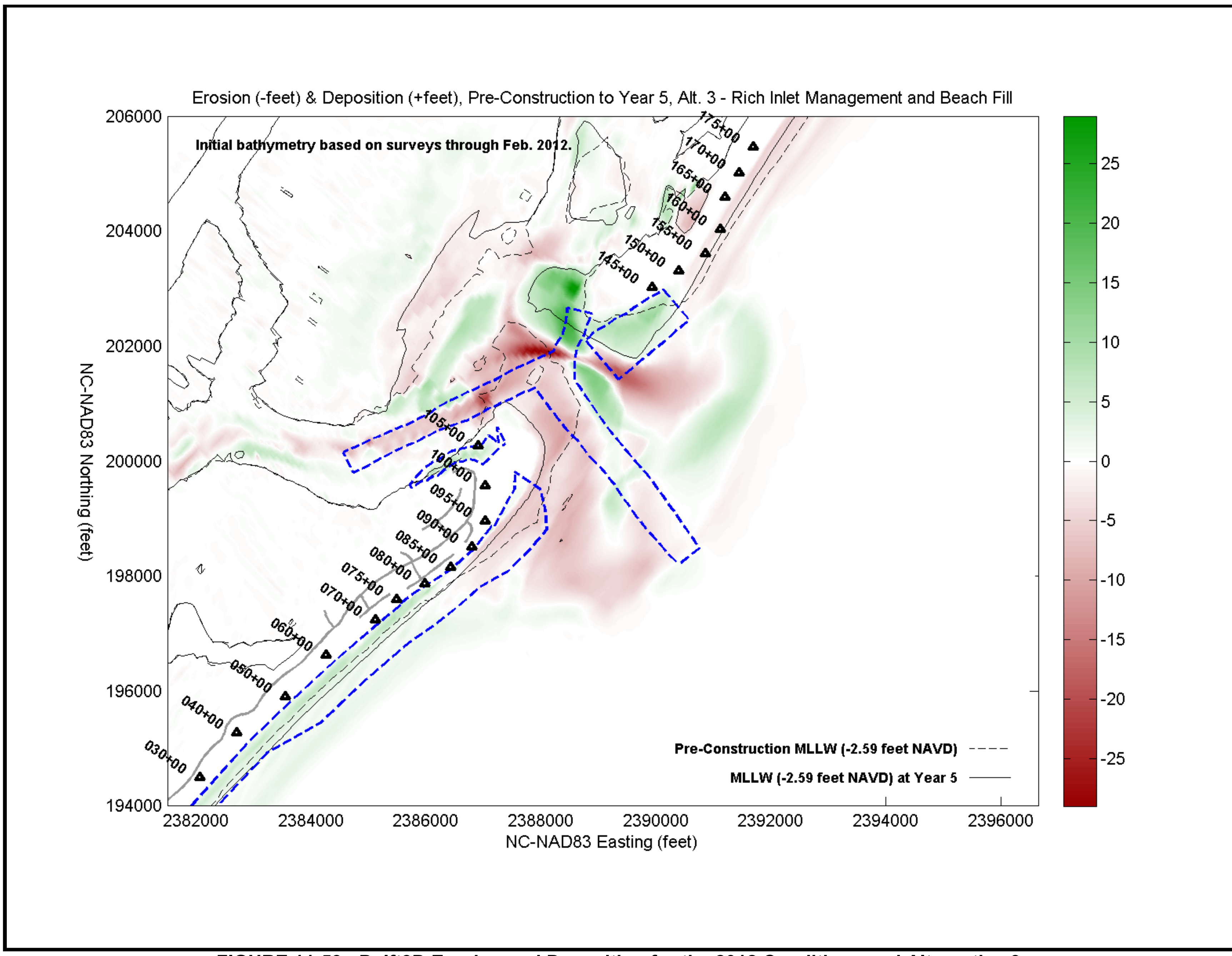


FIGURE 11-53: Delft3D Erosion and Deposition for the 2012 Conditions and Alternative 3.

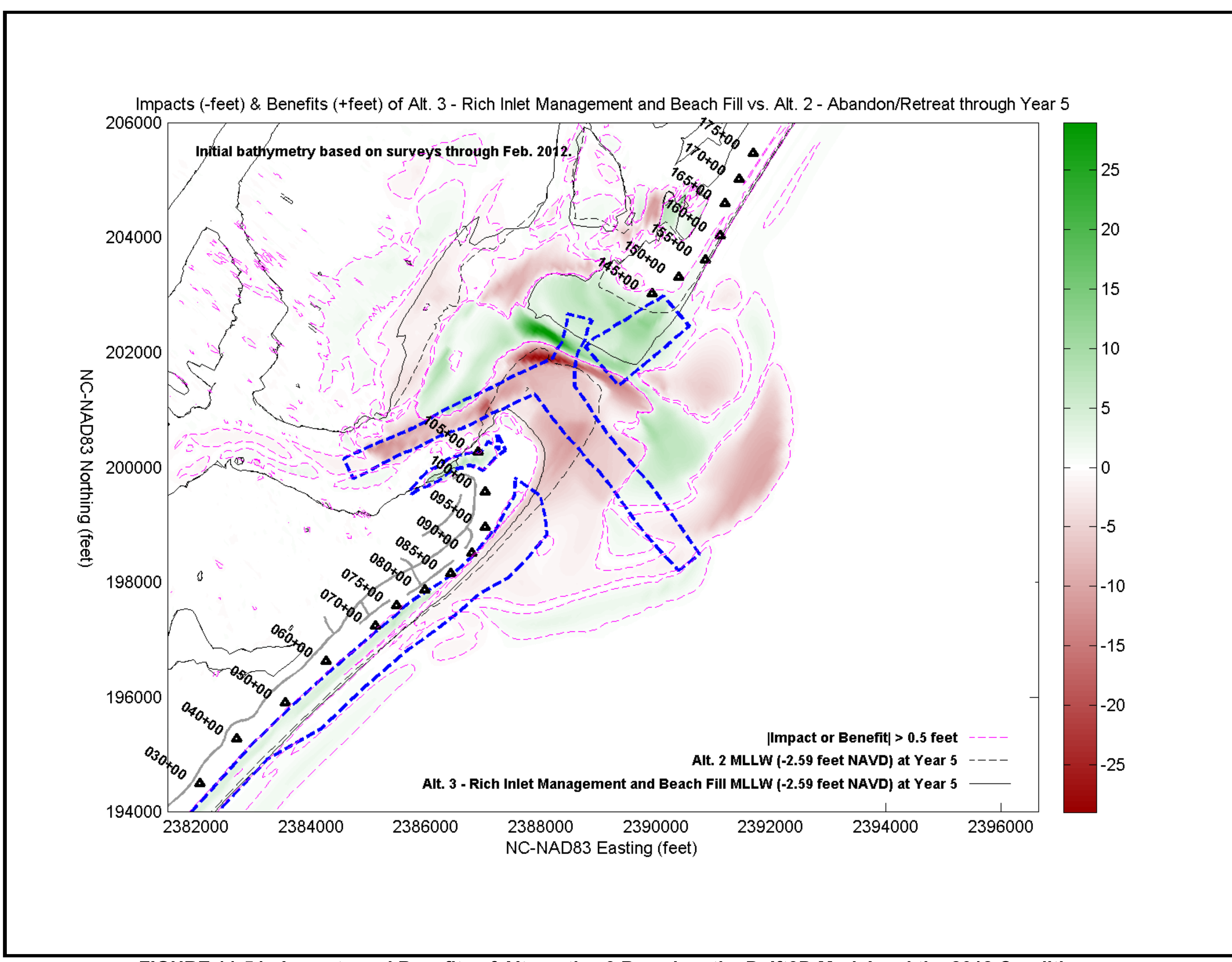


FIGURE 11-54: Impacts and Benefits of Alternative 3 Based on the Delft3D Model and the 2012 Conditions.

11.4.4 Alternative 4 – Beach Fill without Management of Rich Inlet

Model results for Alternative 4 appear in Sub-Appendix B1, Table 11-7, Table 11-8, and Figures 11-55 through 11-61. Since the Delft3D-FLOW model is not able to incorporate renourishment during a model simulation, these results neglect renourishment at Year 4. Similar to Alternative 3, the beach fill along Nixon Channel in the model simulations was based on a preliminary design with 65,000 cubic yards from profiles RIN12+00 to RIN30+00, rather than the final design with 57,000 cubic yards from profiles RIN16+00 to RIN30+00 only.

If Alternative 4 were constructed under eroded conditions similar to those in 2006, Rich Inlet would behave in a manner similar to that of the Abandon/Retreat scenario (compare Figures 11-55 and 11-43). This would also be the case if Alternative 4 were constructed under conditions similar to those in 2012 (compare Figures 11-58 and 11-46).

Volume changes given eroded conditions similar those in 2006 are summarized in Table 11-7 and Figure 11-61. Beach fill performance given Alternative 4 is provided in Table 11-8. North of profile 82+50 (8 Comber Road), erosion into the pre-construction profile would occur by Year 5 or earlier (see also Figure 11-56). The degree of erosion into the pre-construction profile would be considerably higher than that of Alternative 3 (see Table 11-7). However, erosion into pre-construction profile under Alternative 4 would be lower than the erosion obtained for the Abandon/Retreat scenario (see Table 11-7 and Figures 11-57 and 11-61).

Given conditions similar to those in 2012, erosion into pre-construction profile through Year 5 only occurs north of profile 95+00 (Inlet Hook Road) (see Figure 11-59). Since there is no dredging in Rich Inlet, negative impacts to the beach do not occur as they do under Alternative 3 (compare Figure 11-60 with Figure 11-54).

Alternatives 3 and 4 have similar fill layouts. Overall, the model results for the two alternatives suggest that given eroded conditions similar to those in 2006, Alternative 3 performs better (see Table 11-8, Figure 11-51, and Figure 11-57). However, given conditions similar to those in 2012, Alternative 4 appears to perform better than Alternative 3 (see Figure 11-54 and Figure 11-60). The differences in these results are due to the manner in which the dredge cut for Alternative 3 modifies the bathymetry in Rich Inlet, which, in turn, affects the erosion patterns on the adjacent beaches.

None of the alternatives that were simulated matched the sequence of man-made interventions that took place between 2006 and 2012. However, Alternative 4 is the most similar. The differences between Alternative 4 and the actual sequence of events between 2006 and 2012 are the following:

- Fill was placed in two successive operations towards the middle of the study period, rather than a single fill operation at the beginning of the study period.

- The amount of fill was less than the design volume for Alternative 4 (compare Tables 6-1 and 9-5).

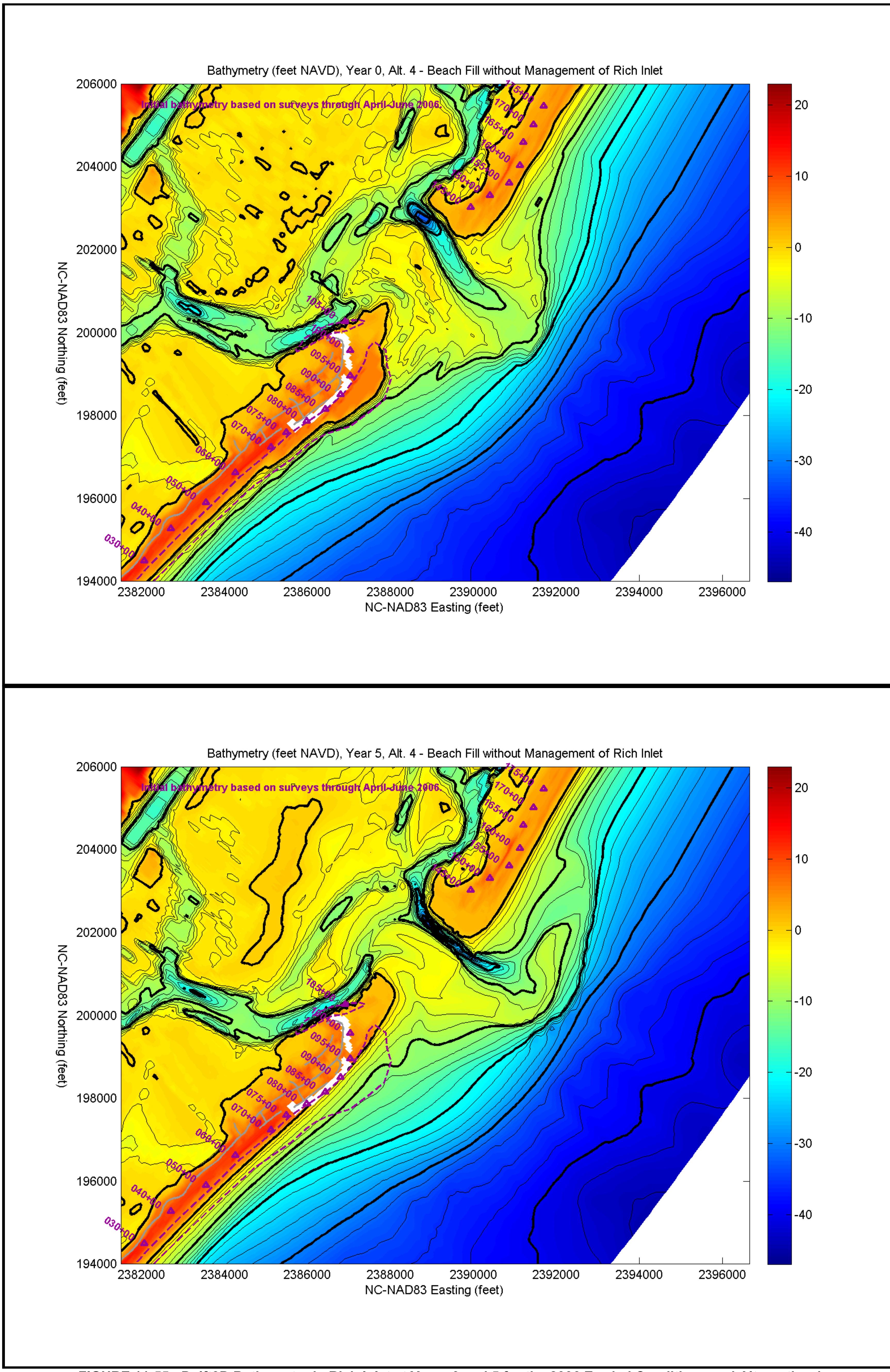


FIGURE 11-55: Delft3D Bathymetry in Rich Inlet at Years 0 and 5 for the 2006 Eroded Conditions and Alternative 4.

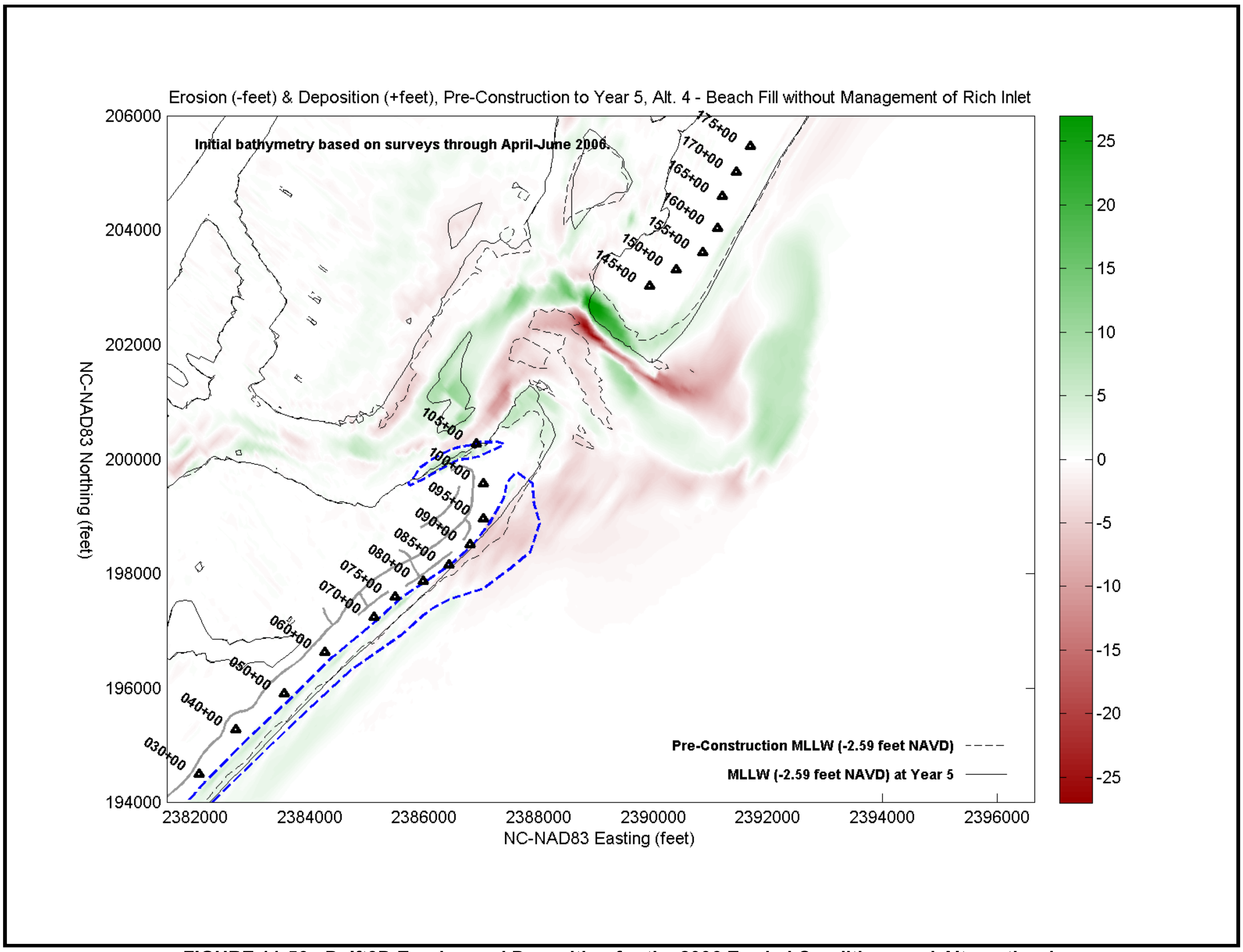


FIGURE 11-56: Delft3D Erosion and Deposition for the 2006 Eroded Conditions and Alternative 4.

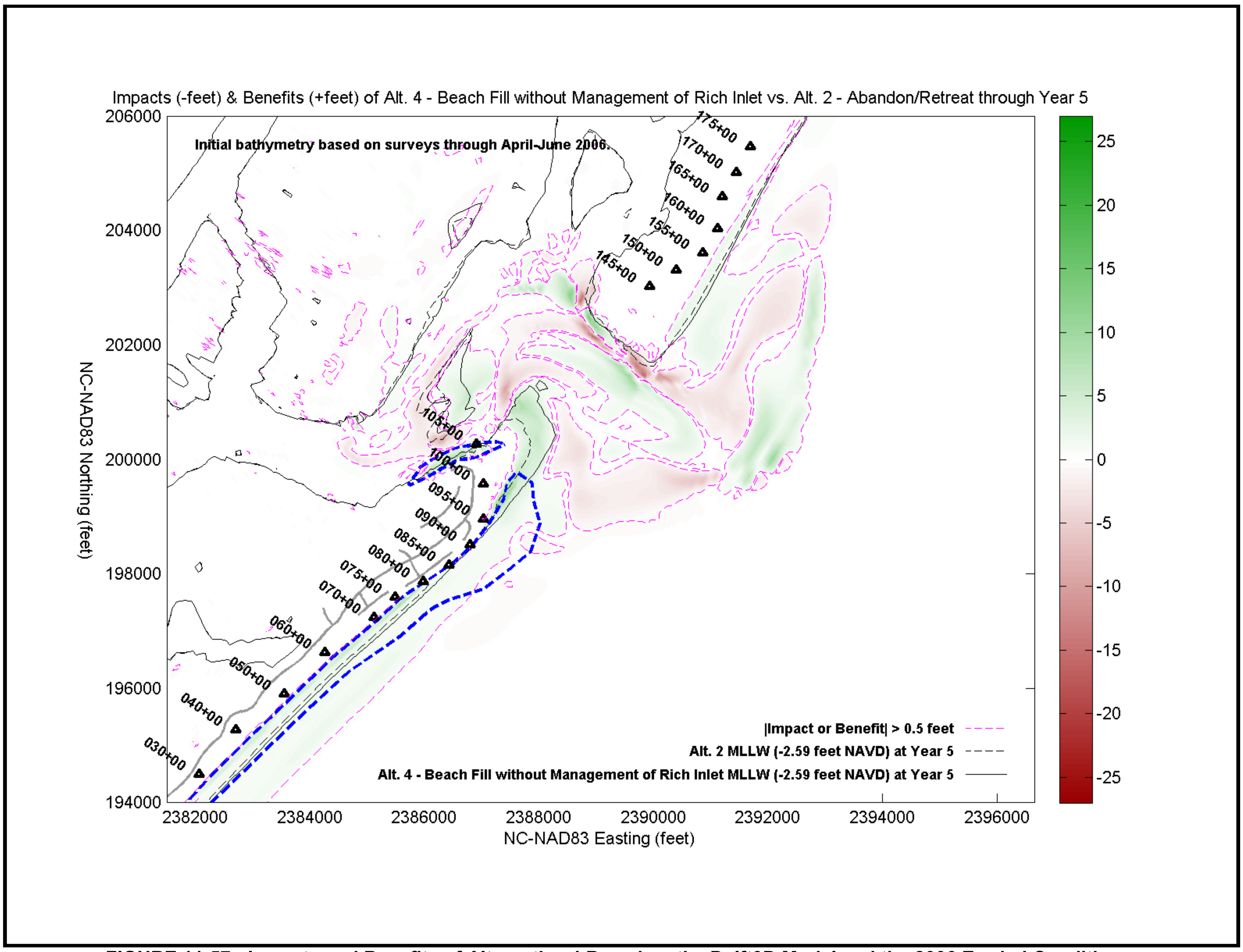


FIGURE 11-57: Impacts and Benefits of Alternative 4 Based on the Delft3D Model and the 2006 Eroded Conditions.

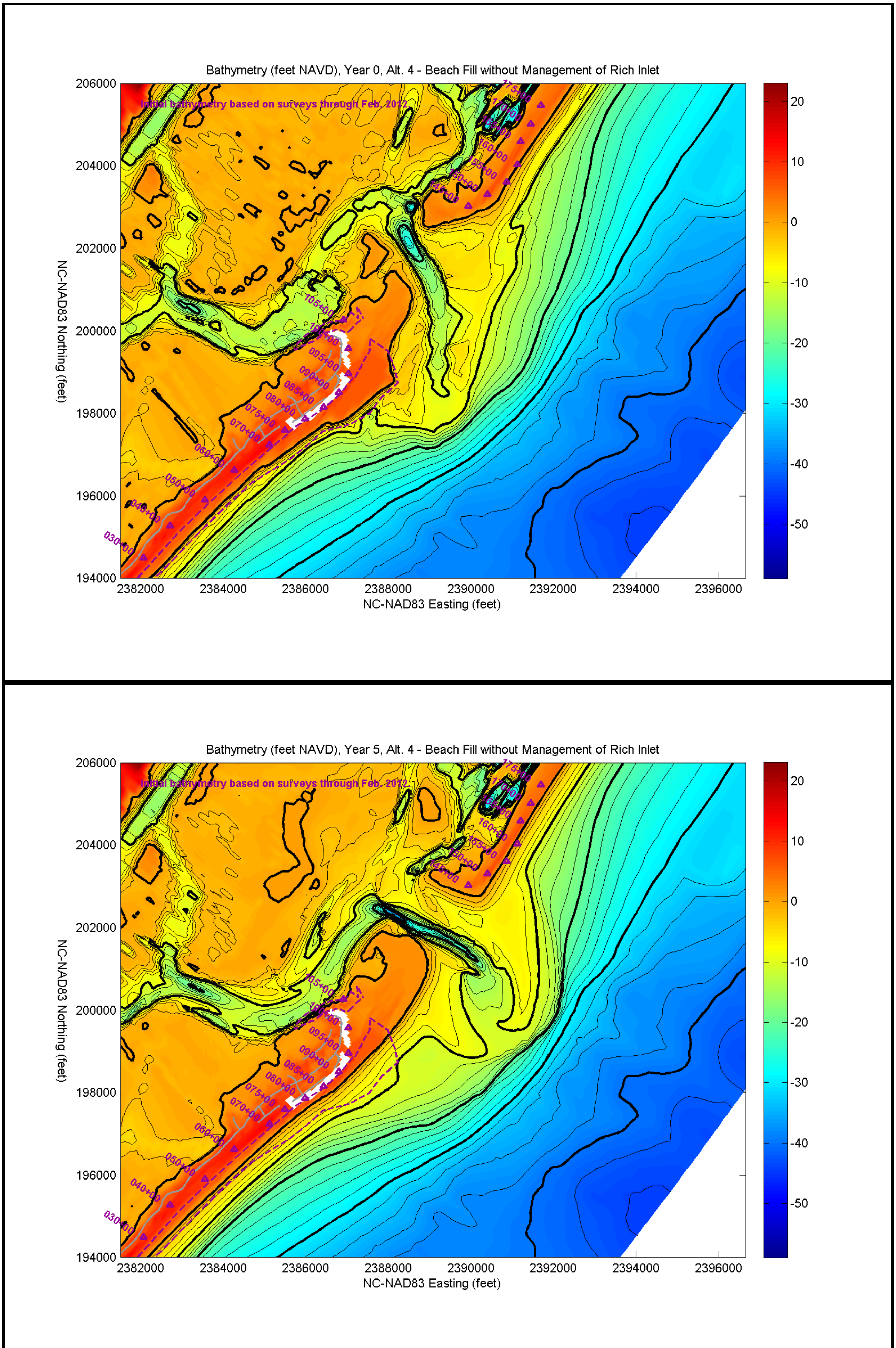


FIGURE 11-58: Delft3D Bathymetry in Rich Inlet at Years 0 and 5 for the 2012 Conditions and Alternative 4.

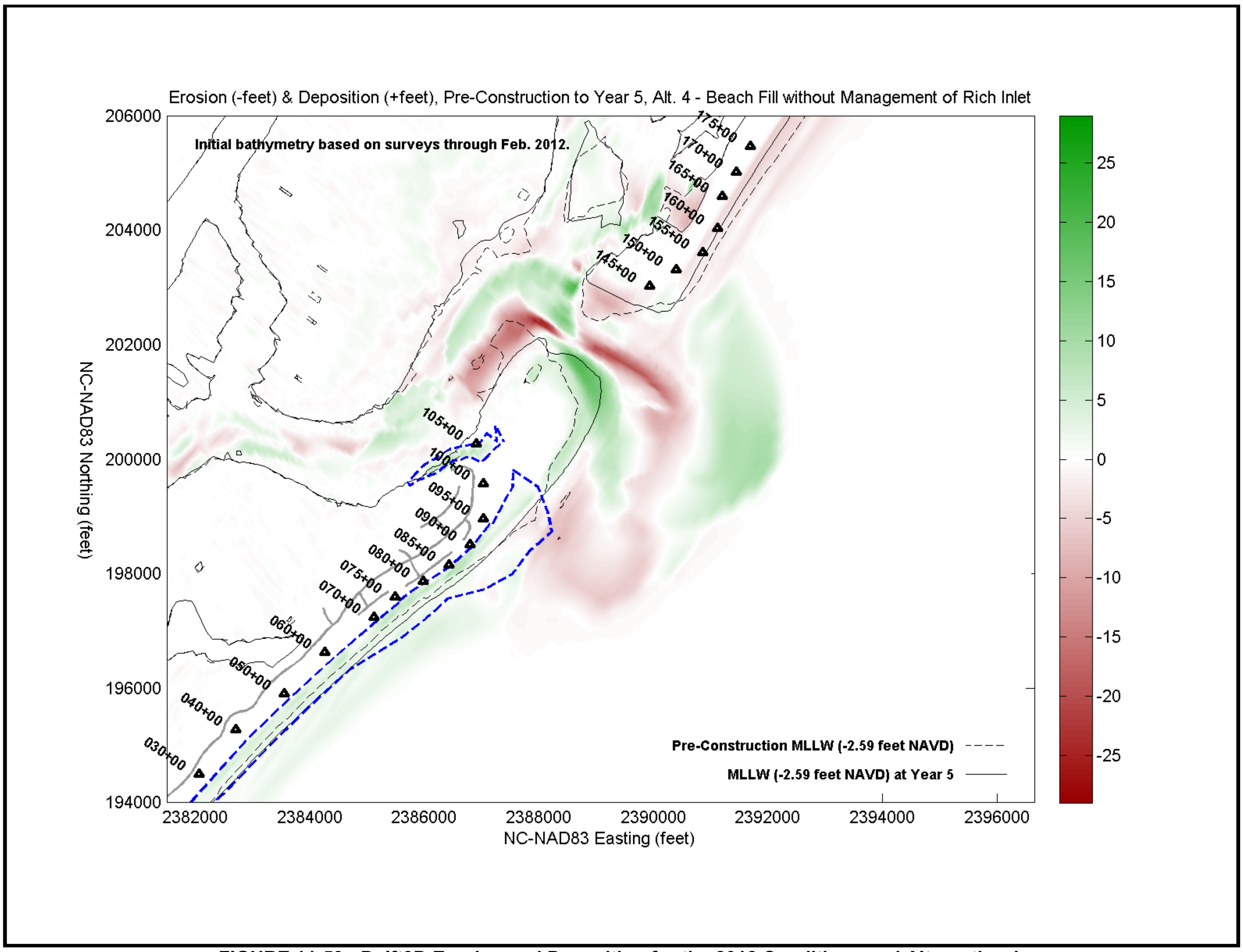


FIGURE 11-59: Delft3D Erosion and Deposition for the 2012 Conditions and Alternative 4.

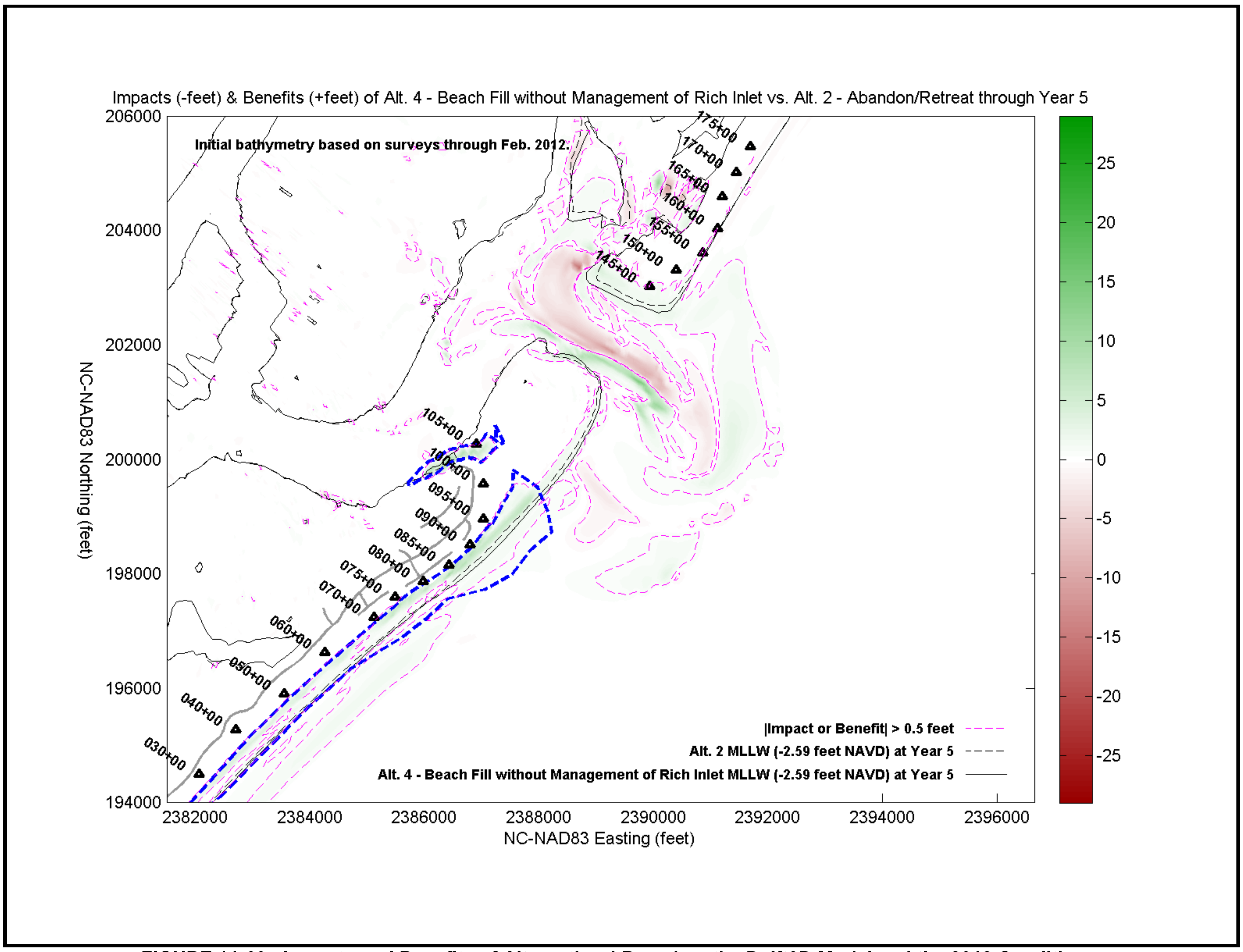


FIGURE 11-60: Impacts and Benefits of Alternative 4 Based on the Delft3D Model and the 2012 Conditions.

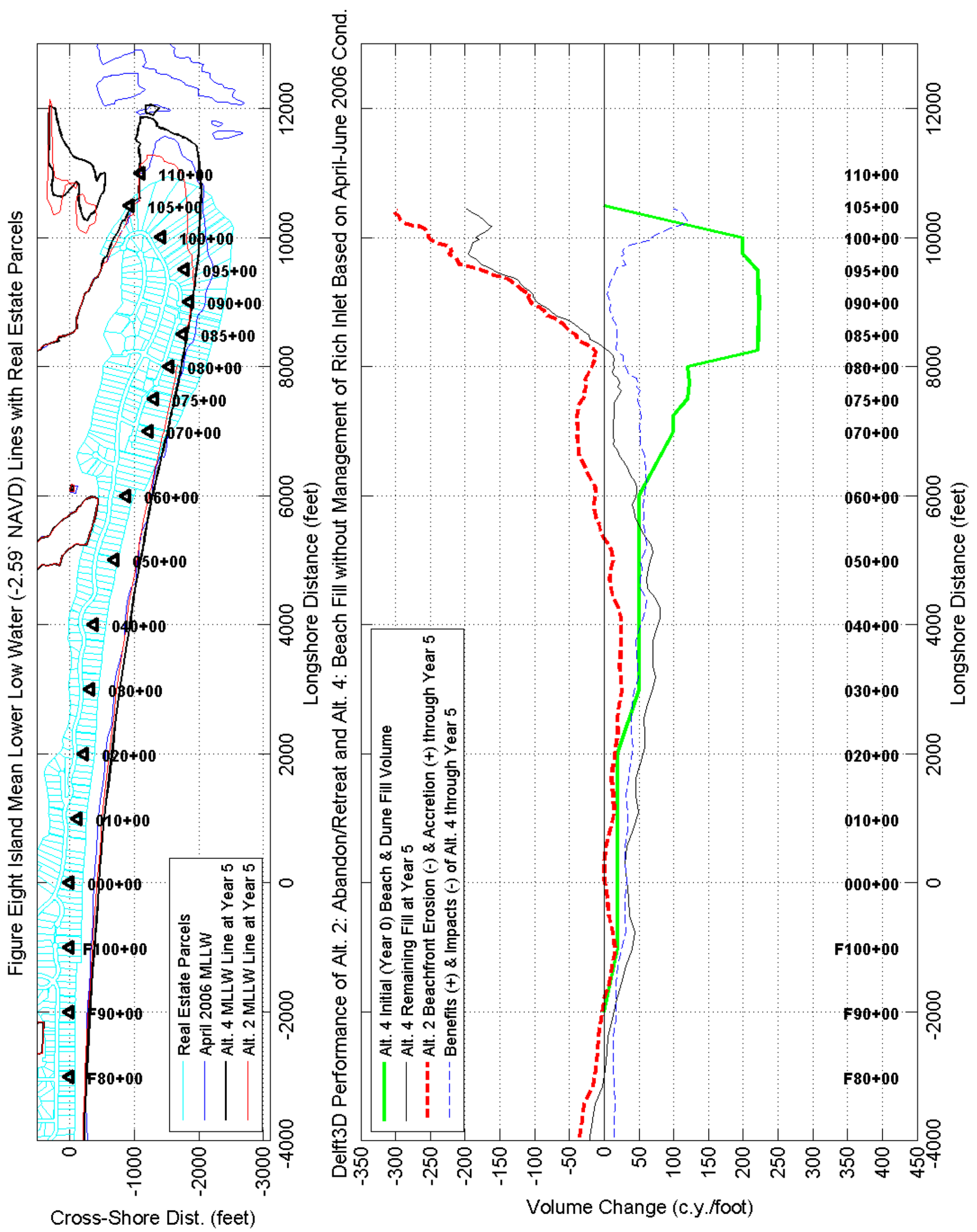


FIGURE 11-61: Delft3D Volume Changes for the 2006 Eroded Conditions and Alternatives 2 and 4.

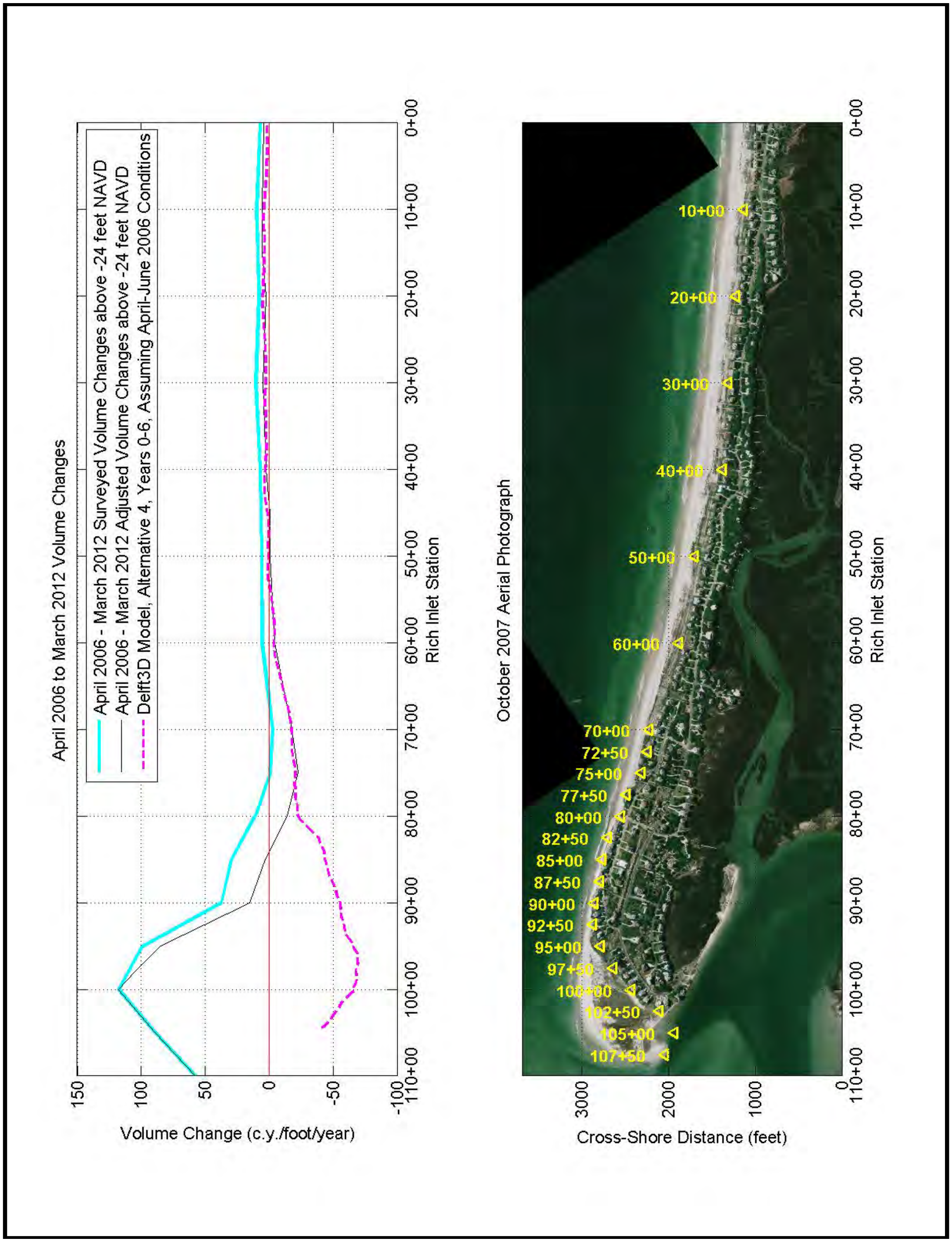


FIGURE 11-62: Comparison of Delft3D Results for Alternative 4 to Observed Volume Changes between 2006 and 2012.

Despite these differences, the model results for Alternative 4 can be used to evaluate how well the model could estimate the changes occurring from 2006 to 2012 (see Figure 11-62).

South of profile 77+50 (Comber Road), there is excellent agreement between the observed volume changes adjusted for beach fill (Figure 11-62, thin, solid line) and the model results (Figure 11-62, dashed line). North of profile 77+50, the model results suggest erosion, while the 2006 and 2012 beach surveys generally indicate accretion. It should be noted that the model was calibrated during a period of erosion along the majority of this segment (see Figure 11-38). For this reason, the model tends to estimate erosion along north of profile 77+50, rather than accretion. It should also be noted that the timing and quantity of the beach fills placed in 2009 and 2010 do not match the placement scenario of Alternative 4, in which all fill is placed at Year 0.

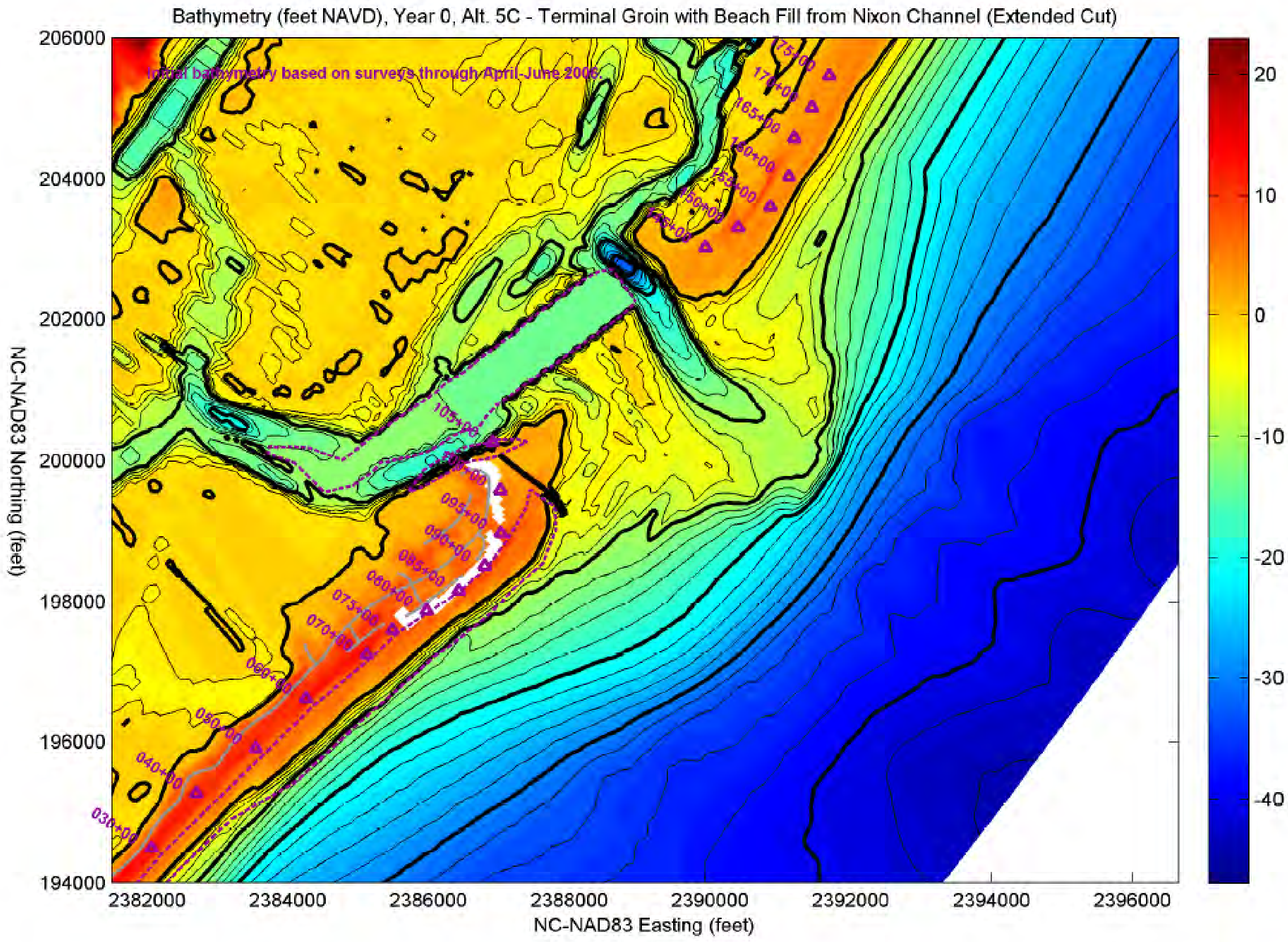
Given the results shown in Figure 11-62, the Delft3D model's estimated erosion rates on the north end of Figure Eight Island are conservative; the erosion estimates are high in comparison to the present trends. Overall, this result confirms that the model results are best used for comparisons between various alternatives, rather than absolute predictions of future volume changes.

11.4.5 Alternative 5C - Terminal Groin at a More Northerly Location with Beach Fill from Nixon Channel and a New Channel Connecting to Gorge of Rich Inlet

Model results for Alternative 5C appear in Sub-Appendix B1, Table 11-7, Table 11-8, Table 11-10, and Figures 11-63 through 11-65. Since the Delft3D-FLOW model is not able to incorporate renourishment during a model simulation, these results neglect renourishment at Year 5. Similar to Alternative 3, the beach fill along Nixon Channel in the model simulations was based on a preliminary design with 65,000 cubic yards from profiles RIN12+00 to RIN30+00, rather than the final design with 57,000 cubic yards from profiles RIN16+00 to RIN30+00 only.

TABLE 11-10
DELFT3D DREDGE MAINTENANCE VOLUMES
EIS ALT. 5C - 2006 ERODED CONDITIONS

Year	Re-Dredging Volume to Design Depth (c.y.)		
	-11' MLW (-13.43' NAVD) Cut	-9' MLW (-11.43' NAVD) Cut	TOTAL
0	0	0	0
1	129,000	7,000	136,000
2	392,000	19,000	411,000
3	382,000	42,000	424,000
4	430,000	82,000	512,000
5	365,000	122,000	487,000



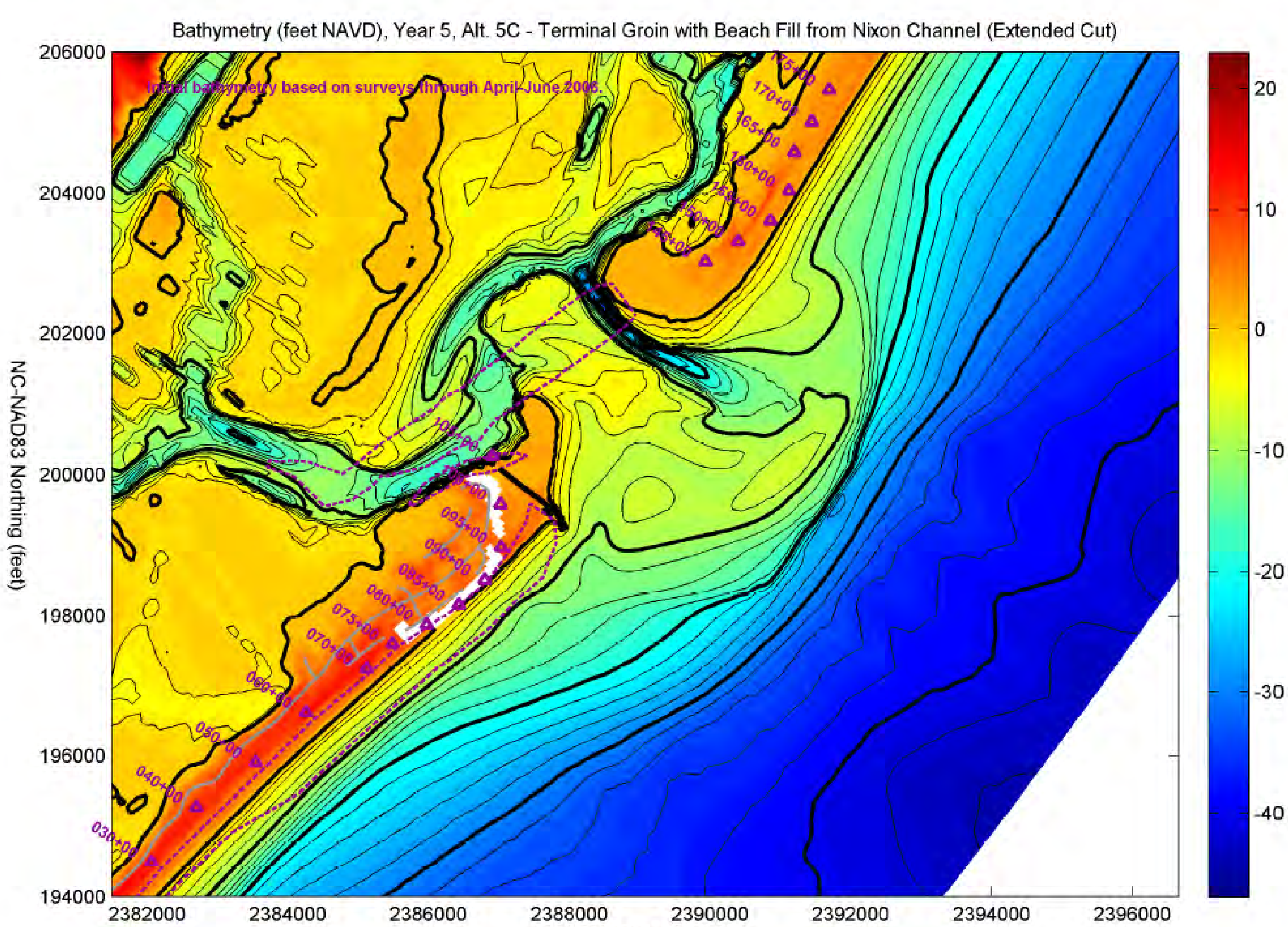


FIGURE 11-63: Delft3D Bathymetry in Rich Inlet at Years 0 and 5 for the 2006 Eroded Conditions and Alternative 5C.

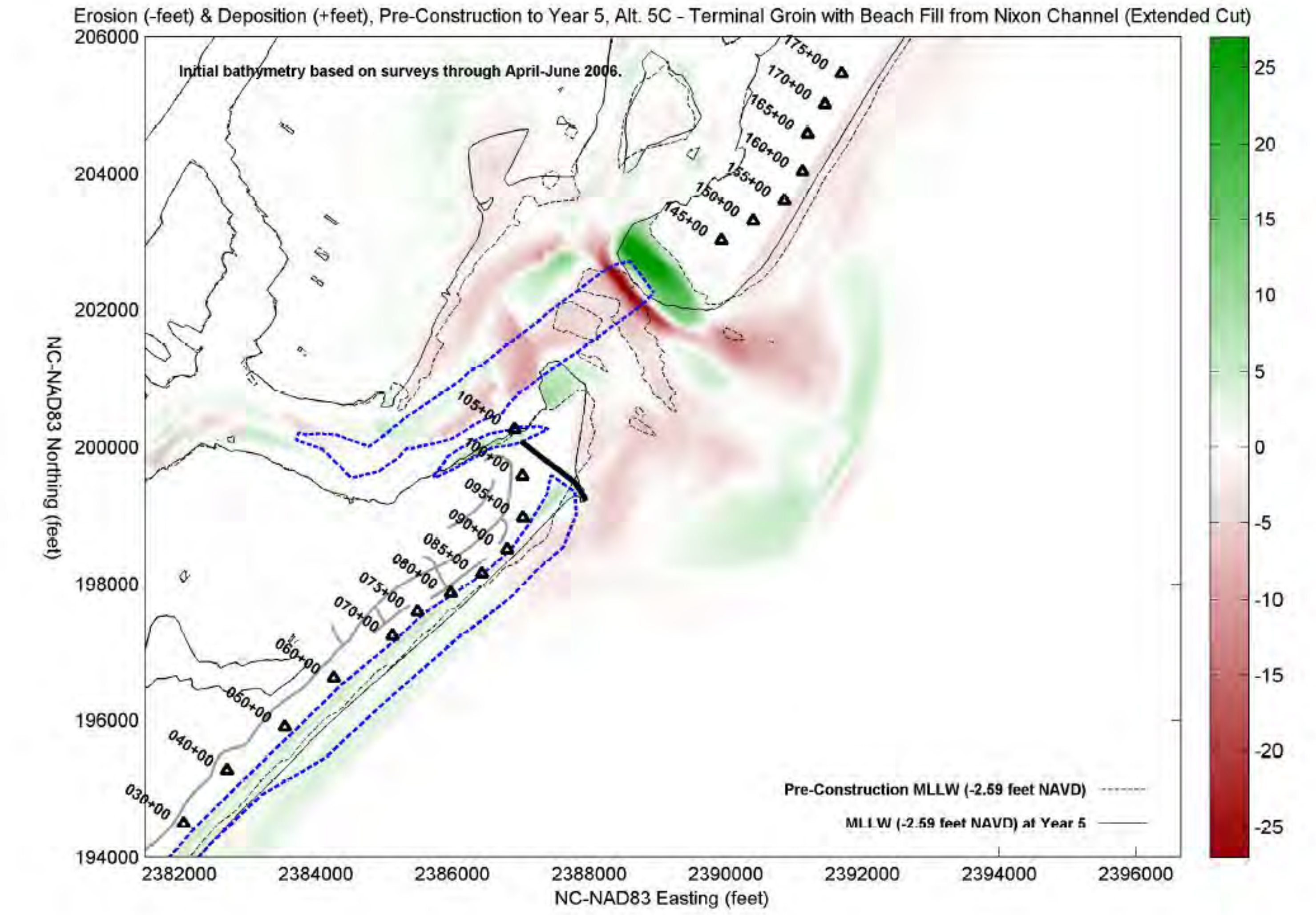


FIGURE 11-64: Delft3D Erosion and Deposition for the 2006 Eroded Conditions and Alternative 5C.

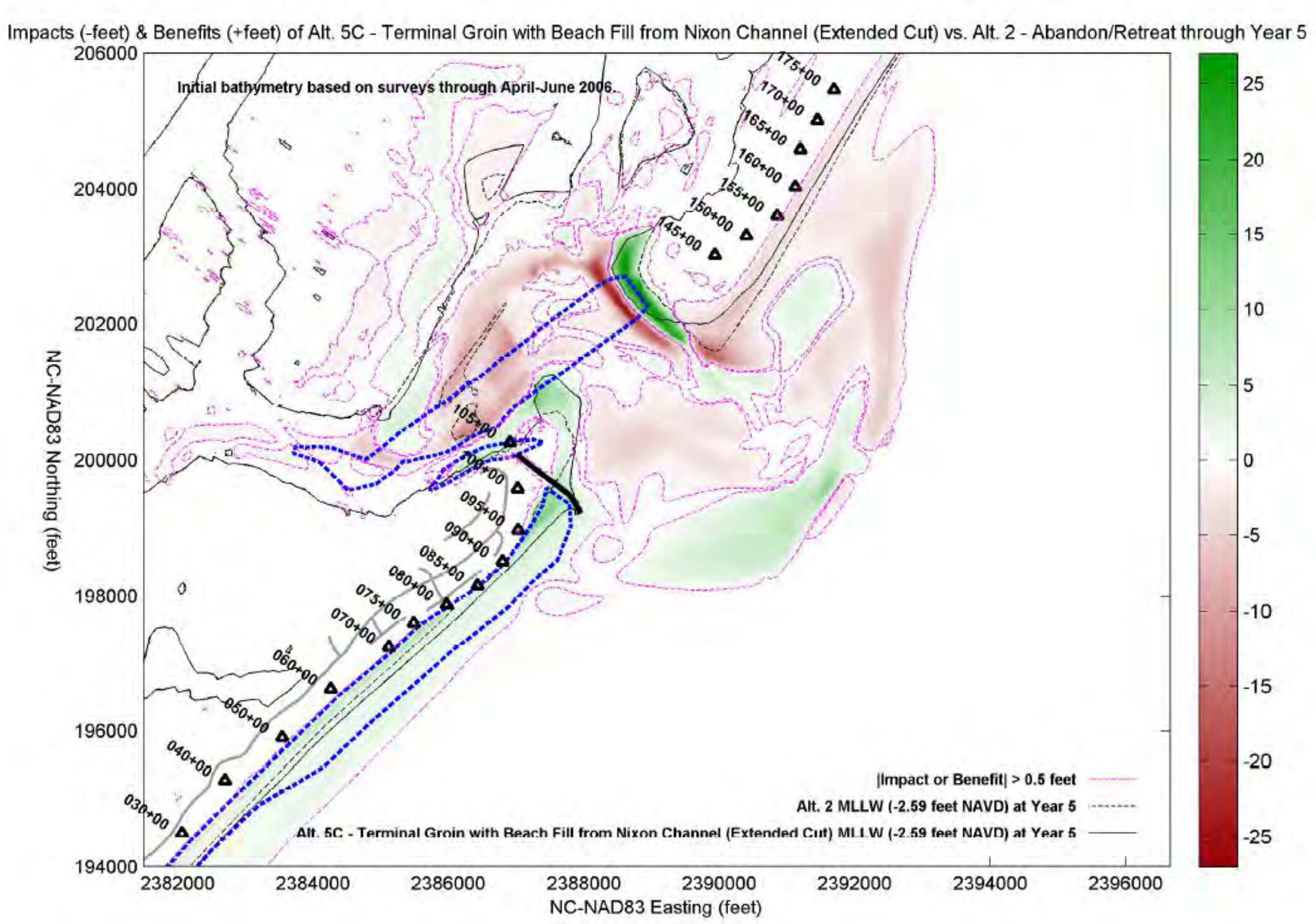


FIGURE 11-65: Impacts and Benefits of Alternative 5C Based on the Delft3D Model and the 2006 Eroded Conditions.

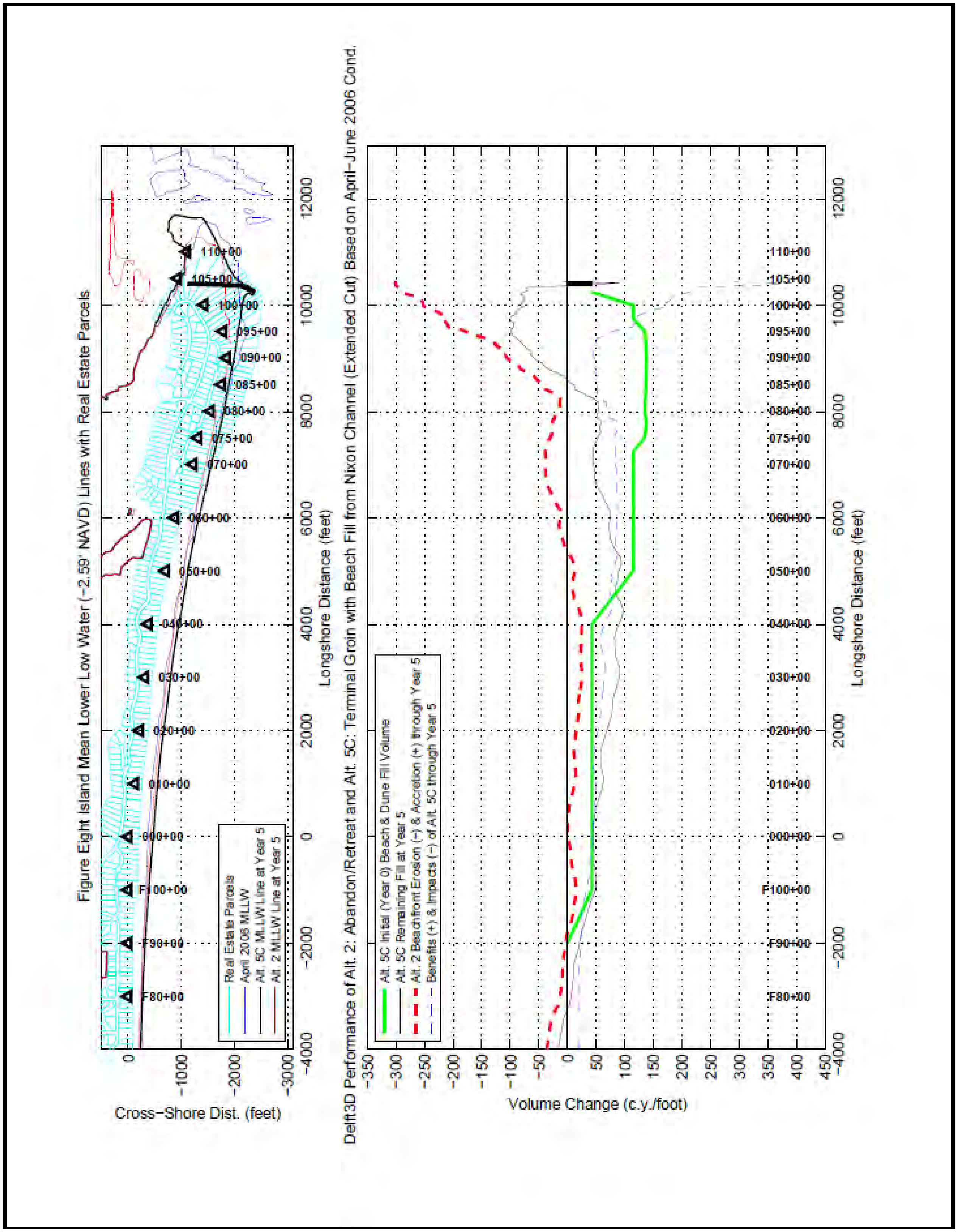


FIGURE 11-66: Delft3D Volume Changes Given the 2006 Eroded Conditions and Alternatives 2 and 5C.

The terminal groin was incorporated into the Delft3D-FLOW model by raising the grid cells along the structure to +6 feet NAVD, and setting the erodible sediment depth along those same grid cells to zero. This ensured that:

- Overtopping of the structure, if any, would be properly estimated in the Delft3D-FLOW model.
- The structure would remain in the model over the entire duration of the model run.

In the SWAN model, the terminal groin was represented as a sloped "dam" with a crest elevation of +6 feet NAVD and negligible wave reflection.

Model simulations for Alternative 5C were only conducted for the 2006 critically eroded condition. Due to the lack of support for this alternative by the Figure "8" Beach HOA, simulating Alternative 5C given the 2012 conditions was determined not to be necessary.

If Alternative 5C were constructed under eroded conditions similar to those in 2006, the main channel of Rich Inlet would have an orientation similar to that of the Abandon/Retreat scenario. However, there would be some differences in the contours of the ebb shoal, along with a more open connection between Nixon Channel and the main channel of the inlet (compare Figure 11-63 with Figure 11-45). The differences in the ebb shoal contours would be due to the beach fill material placed on the north end of Figure Eight Island and the manner in which the terminal groin would deflect the longshore transport off the north end of the island, along with dredging-related changes to the flow through Rich Inlet. The more open connection between Nixon Channel and the main channel of the inlet would be due to the extension of the 2010 cut towards the main channel of the inlet, which would migrate landward over time.

Another key difference between Alternative 5C and Alternative 2 is the development of the spit north of the terminal groin location (profile 105+00). Under Alternative 5C, the spit is longer at the end of Year 5 than it is under Alternative 2 (see Figures 11-65, 11-63, and 11-45). This result is due to the large amount of fill placed along the north end of Figure Eight Island, and suggests that with a sufficient amount of pre-filling, partial bypassing of the terminal groin would occur.

On Hutaff Island, the model results suggest that given Alternative 5C, erosion rates would be higher than those under the Abandon/Retreat scenario (see Figure 11-65 and Table 11-7). This result would be due to effect of the terminal groin on the sediment transport off the north end of Figure Eight Island, changes in the flow through Rich Inlet associated with the design cut, and the resulting changes in the development of the ebb shoal.

In terms of fill performance, the model results suggest that south of profile 85+00 (13 Comber Road), erosion into the pre-construction beach profile (see Table 11-7 and 11-64) will not occur by Year 5. North of profile 85+00, erosion into the pre-construction beach profile could occur within 5 years. However, the degree of erosion would be 2/3 less than what would occur under the Abandon/Retreat scenario (see Table 11-17). Thus, the beach fill and the terminal groin would still provide a benefit (see Figure 11-15). It is important to note that north of profile

85+00, the model results are very conservative (see Figure 11-62); the degree of erosion could be less than what the model suggests if the alternative were constructed under critically eroded conditions. Based on Table 11-9, infilling of the design cut would be just enough to renourish the project at Year 5.

11.4.6 Alternative 5D – Terminal Groin at a More Northerly Location with Beach Fill from the Previously Permitted Area in Nixon Channel and Other Sources

Model results for Alternative 5D appear in Sub-Appendix B1, Table 11-7, Table 11-8, Table 11-11, and Figures 11-67 through 11-73. This alternative constructs a 1,500 foot long terminal groin with 237,500 c.y. of fill along the oceanfront and 57,000 of c.y. of fill along the interior shoreline of Nixon Channel (see Table 9-7 and Figure 9-28). The groin was incorporated into the Delft3D-FLOW and SWAN model in the same manner as Alternative 5C. Similar to the other alternatives, renourishment at Year 5 was neglected.

TABLE 11-11

DELFT3D DREDGE MAINTENANCE VOLUMES
EIS ALT. 5D
2006 ERODED CONDITIONS

Year	Re-Dredging Volume to Design Depth (c.y.)
0	0
1	31,000
2	78,000
3	105,000
4	120,000
5	134,000

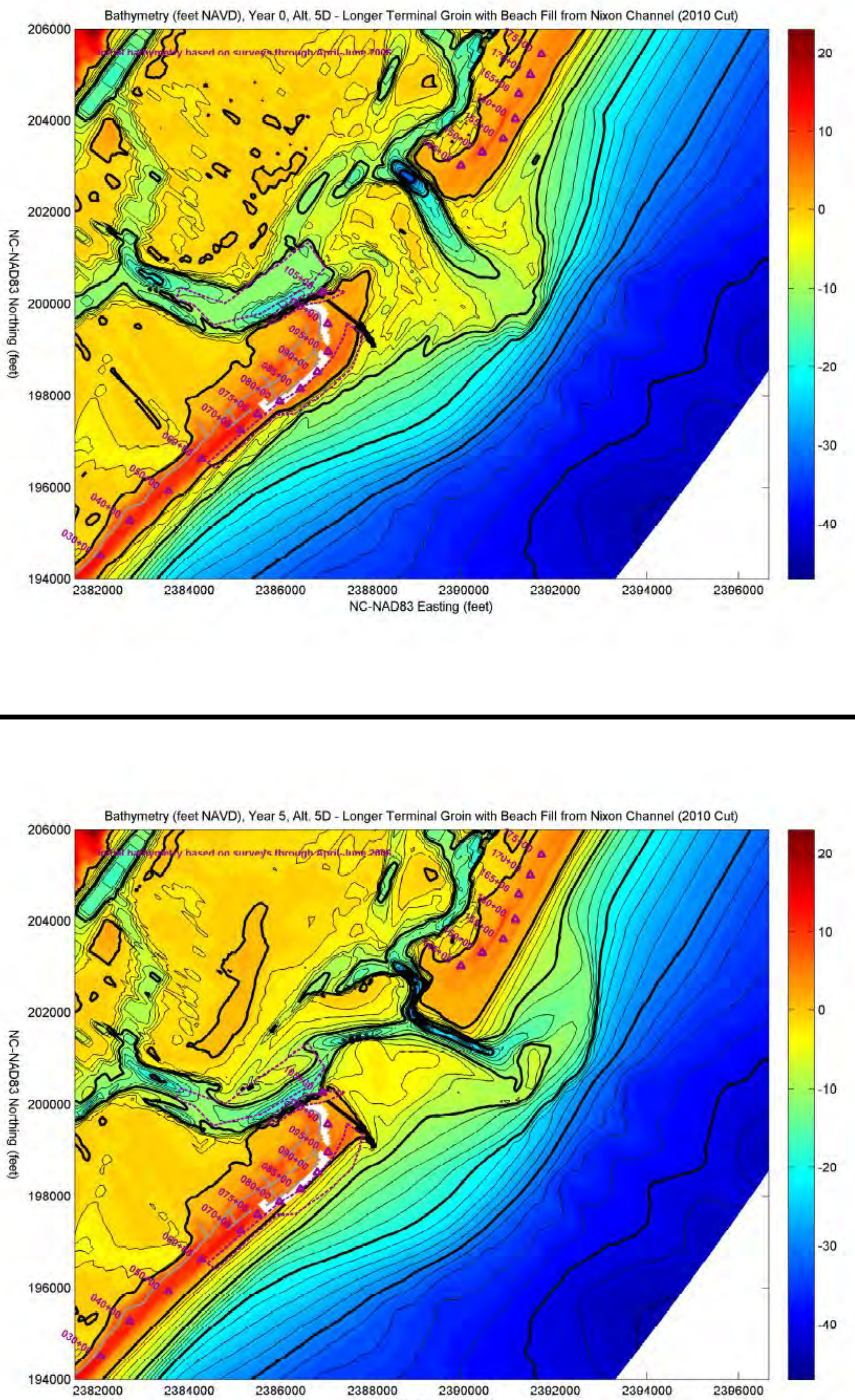


FIGURE 11-67: Delft3D Bathymetry in Rich Inlet at Years 0 and 5 for the 2006 Eroded Conditions and Alternative 5D

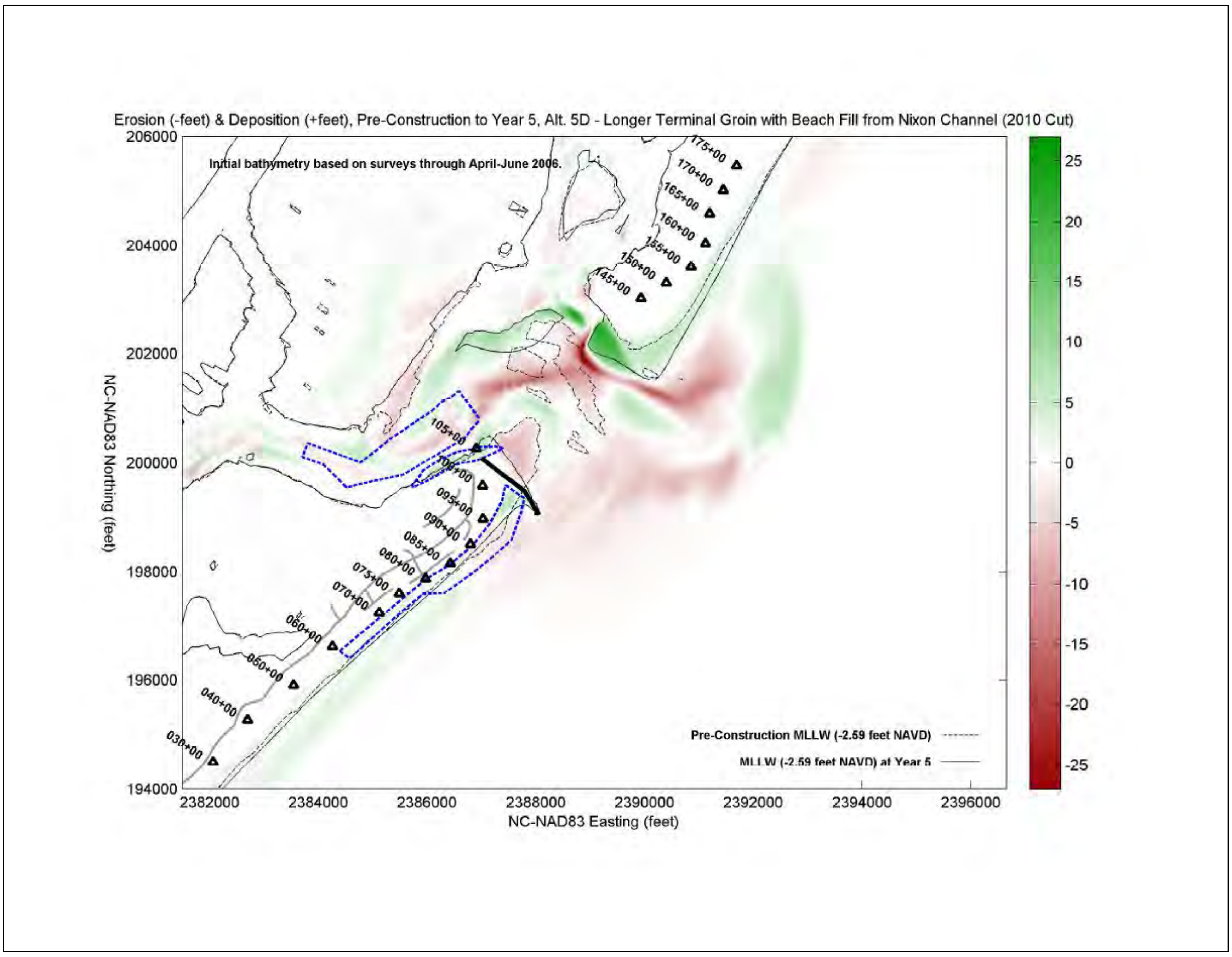


FIGURE 11-68: Delft3D Erosion and Deposition for the 2006 Eroded Conditions and Alternative 5D.

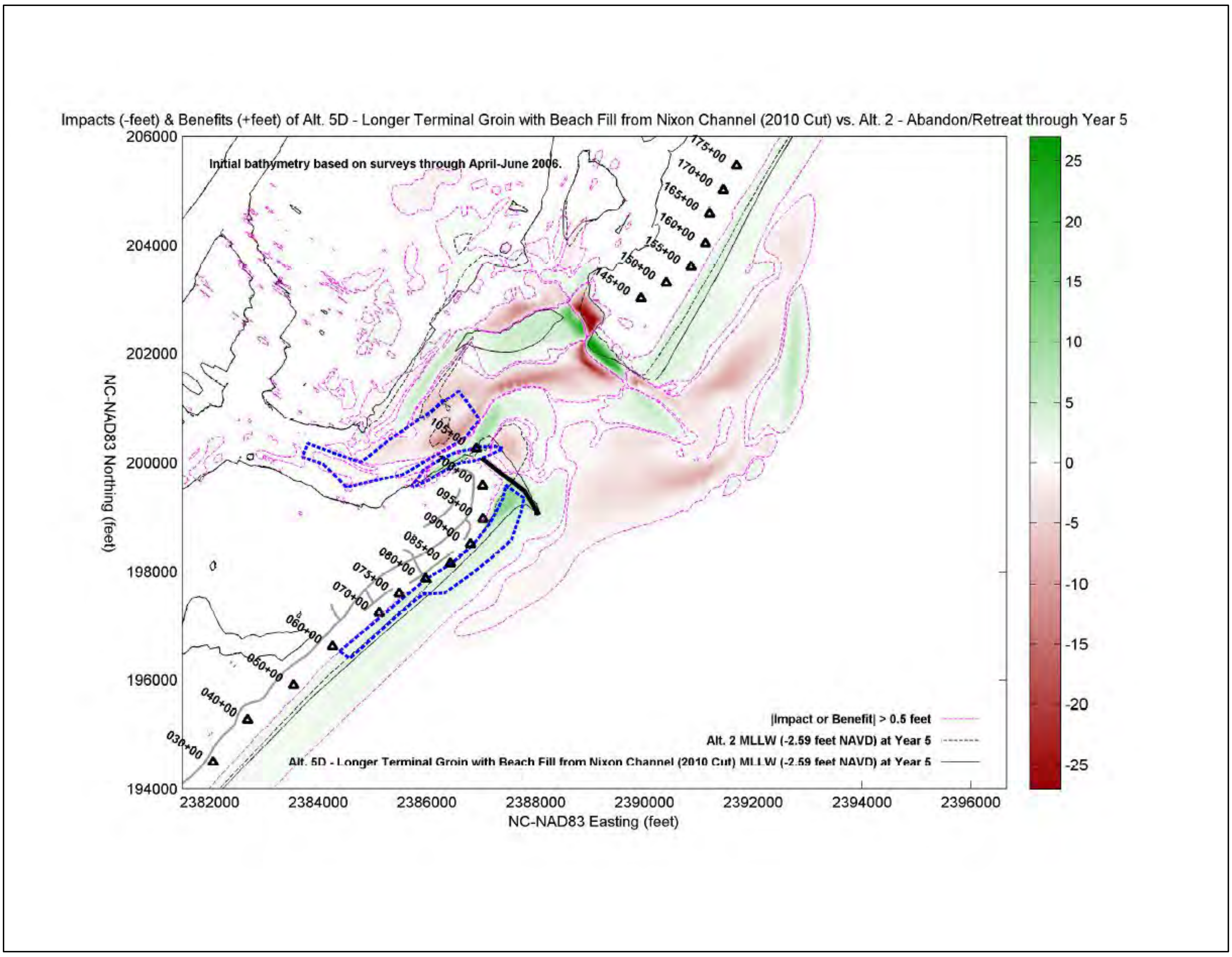
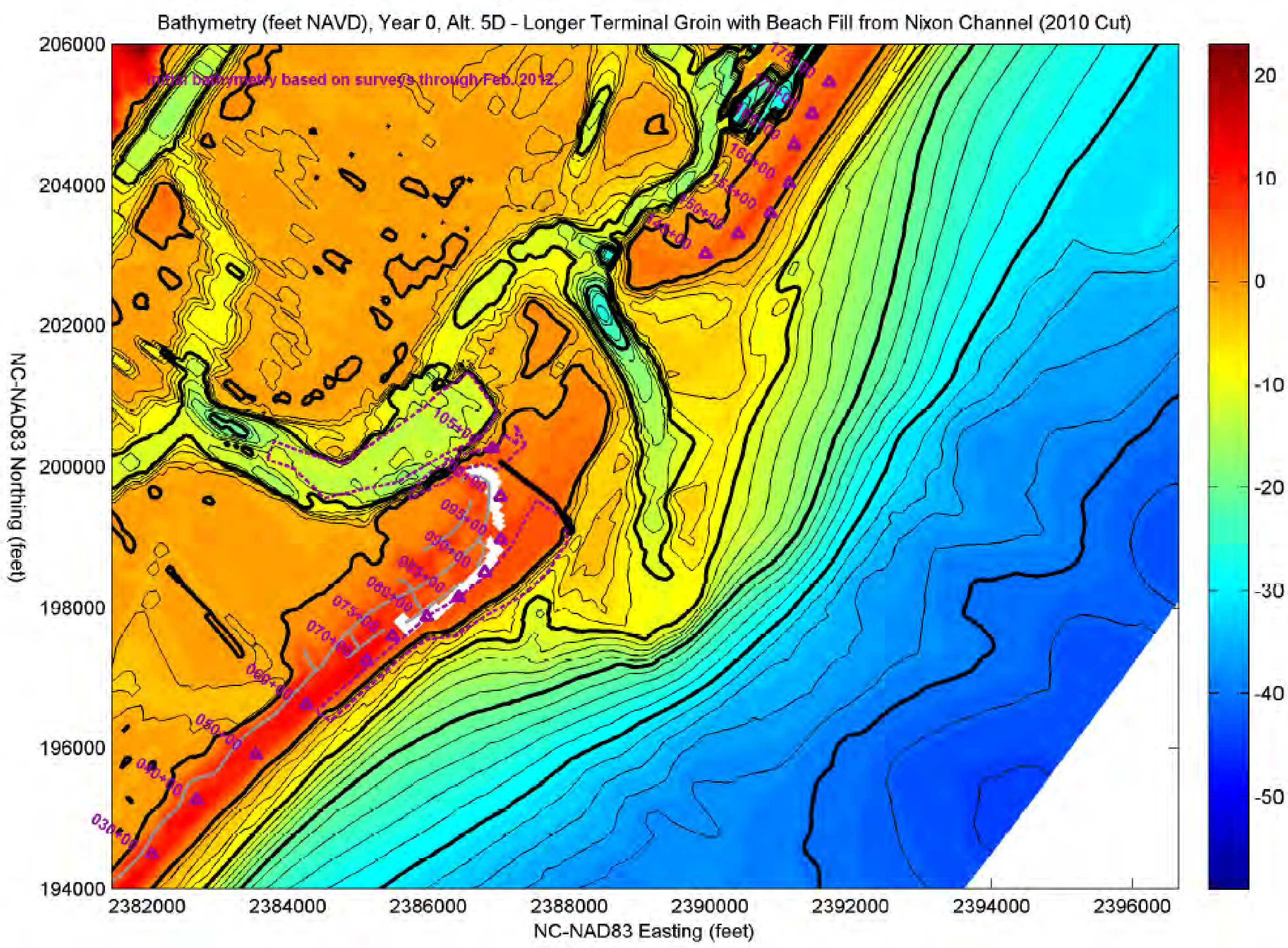


FIGURE 11-69: Impacts and Benefits of Alternative 5D Based on the Delft3D Model and the 2006 Eroded Conditions.



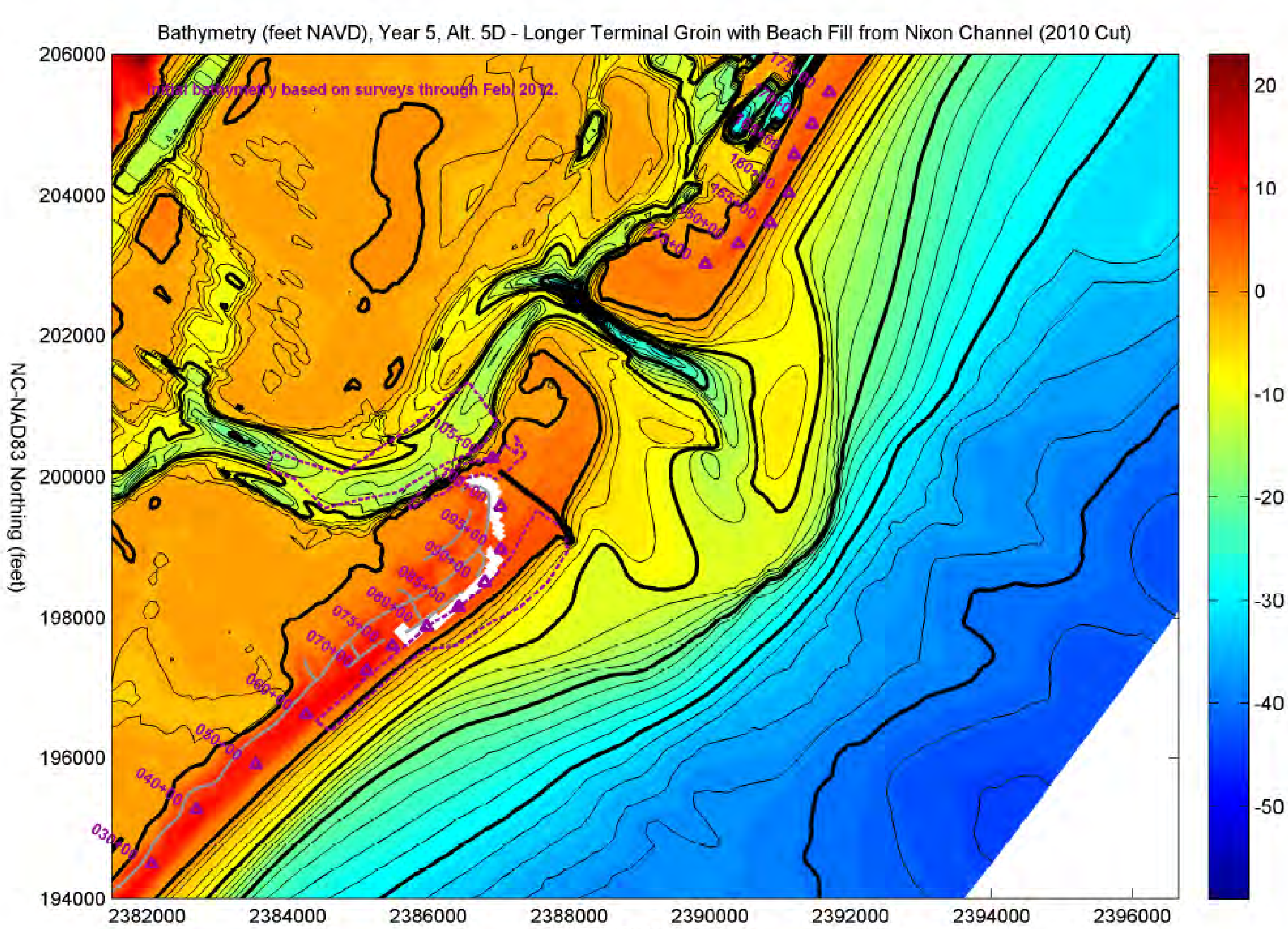


FIGURE 11-70: Delft3D Bathymetry in Rich Inlet at Years 0 and 5 for the 2012 Conditions and Alternative 5D.

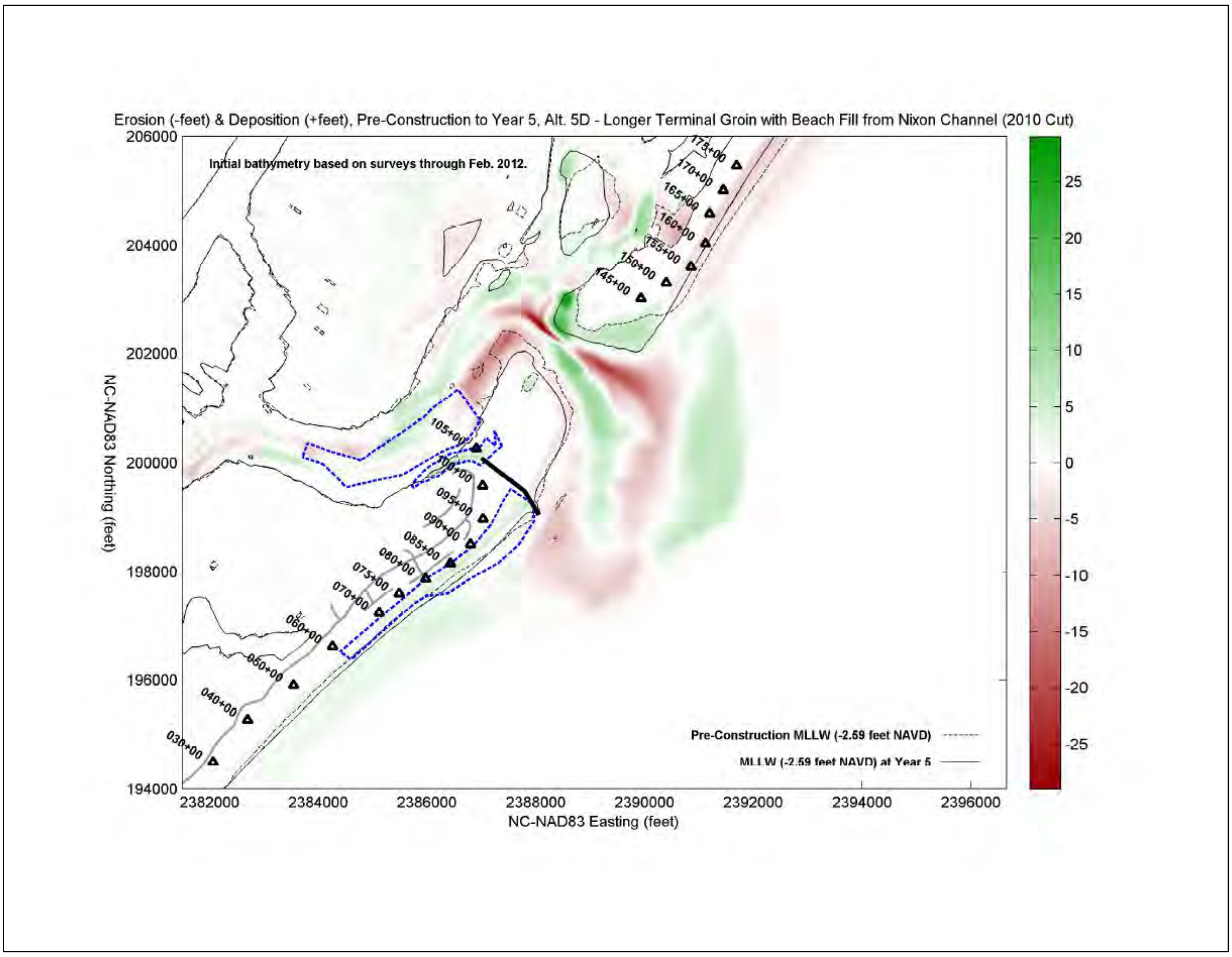


FIGURE 11-71: Delft3D Erosion and Deposition for the 2012 Conditions and Alternative 5D.

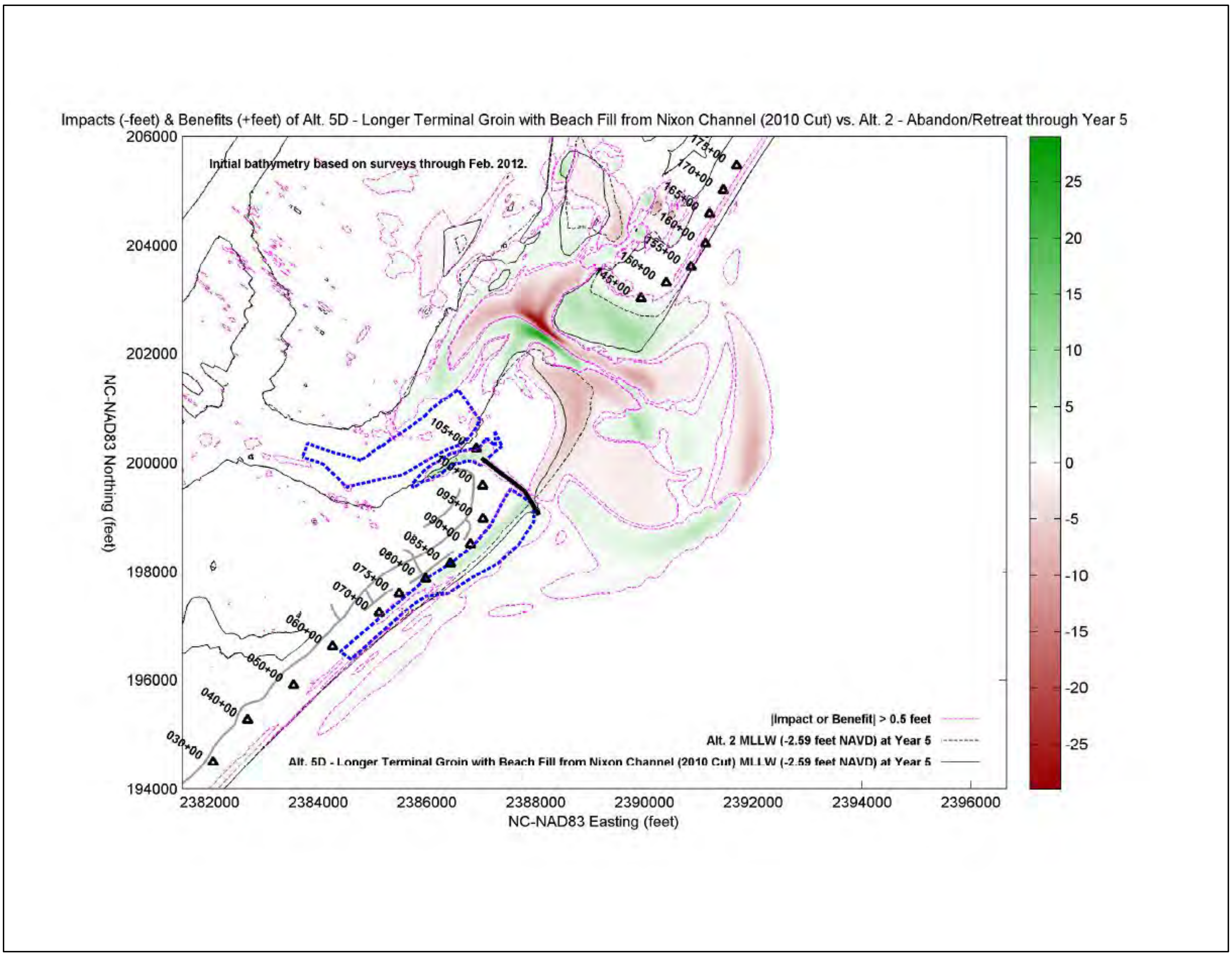


FIGURE 11-72: Impacts and Benefits of Alternative 5D Based on the Delft3D Model and the 2012 Conditions.

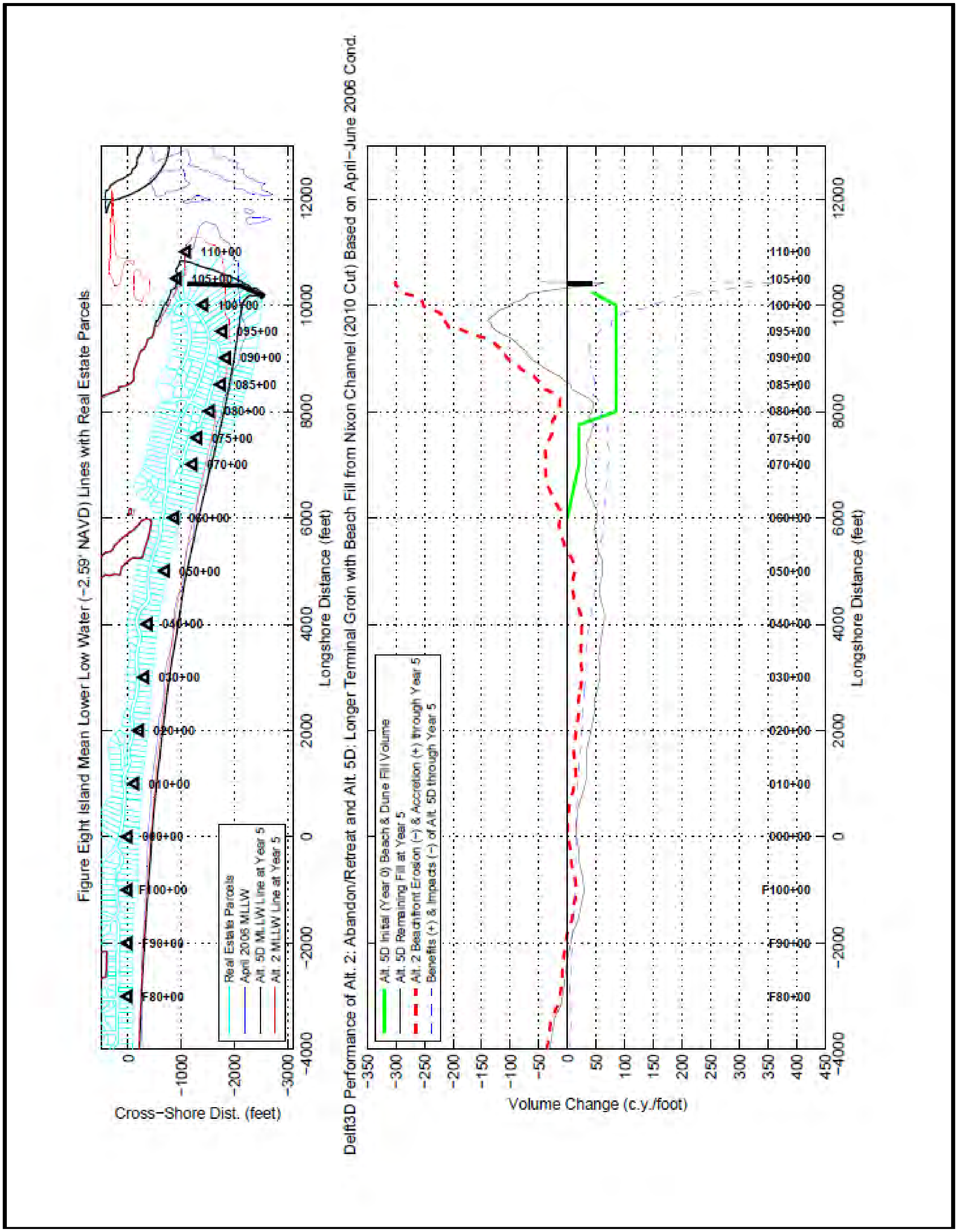


FIGURE 11-73: Delft3D Volume Changes Given the 2006 Eroded Conditions and Alternatives 2 and 5D.

If Alternative 5D were constructed under eroded conditions similar to those in 2006, most of the spit north of the terminal groin location (profile 105+00) would be lost over time (see Figure 11-67). The main channel of Rich Inlet would assume a west-northwest/east-southeast orientation, with relatively deep (> -10 feet NAVD) and continuous connections to Nixon Channel and Green Channel. Along Nixon Channel, much of the beach fill placed at Year 0 would still be in place at Year 5 (see Figure 11-68). Along the oceanfront fill area, erosion into the pre-construction profile would not occur south of profile 85+00 (13 Comber Road) over the first 5 years. North of profile 85+00, erosion into the pre-construction profile could occur if the project were built under critically eroded conditions (see Figure 11-68 and Table 11-7 and Table 11-8). However, the degree of erosion would be half of what would occur under the Abandon/Retreat scenario (see Table 11-7). Along Hutaff Island, adverse, project-related impacts would be minimal (see Figure 11-69 and Table 11-7).

Based on Table 11-11, additional sources may be needed to renourish the project at Year 5. This result is more conservative than historic filling rates might suggest. Based on the most recent dredging operations in Nixon Channel (Table 6-1), the dredge cut could refill faster than what the model suggests. Likewise, fill losses from the oceanfront may be lower than what the model suggests (see Figure 11-62). Annual monitoring will be essential for evaluating what the true renourishment needs will be and the amount of material available for beach renourishment.

If Alternative 5D were constructed under conditions similar to those in 2012, most of the spit north of the terminal groin would remain intact, except for some losses along the interior shorelines of the spit and some minor losses along its oceanfront shoreline (see Figures 11-71 and 11-72). However, the spit would be smaller in size that what would occur under the Abandon/Retreat scenario (see Figure 11-72). The main channel would have an orientation similar to the Abandon/Retreat scenario (compare Figures 11-70 and 11-45). However, it would be located somewhat further north, allowing for a more open connection with Green Channel. At the same time, Hutaff Island would be somewhat longer, even if oceanfront erosion rates north of profile 150+00 are slightly higher (see Figure 11-72). Along Nixon Channel, most of the fill placed at Year 0 would still be remaining at Year 5 (see Figure 11-71). Along the oceanfront fill area, erosion into the pre-construction profile could occur north of profile 95+00 (Inlet Hook Road) within the first 5 years (see Figure 11-71). However, south of profile 95+00, erosion into the pre-construction shoreline would be prevented.

In general, more fill is retained on the beach due to the longer groin length. Given critically eroded conditions similar to those in 2006, impacts to the spit north of the terminal groin are similar under either alternative. Given conditions similar to those in 2012, Alternative 5D reduces the surface area of the spit by roughly 25% (see Figure 11-72). Similar to the difference in performance, the difference in impact is due to the longer groin length.

11.5 Tidal Prisms & Flow Distributions

Average tidal prisms over the model simulation period appear in Table 11-12 and Figures 11-75 to 11-77. Tidal prisms are provided for the Inlet Throat, Nixon Channel, and Green Channel (Figure 11-74).

In comparison to Table 4-8, tidal prism estimates based on the Delft3D model do not exhibit a large degree of variation with respect to either time or alternative. Alternative 4, which does not include dredging in Rich Inlet, would have the least impact on tidal prism based on the model results. Alternative 3, which features the largest amount of dredging, would have the largest effect on tidal prism, with small increases in the prism through the entrance channel (0 to 7% versus Alt. 2), small increases in the prism through Nixon Channel (4 to 9% versus Alt. 2), and small decreases through Green Channel (3 to 8% versus Alt. 2). The terminal groin alternatives (5A, 5B-1, and 5B-2) also tend to increase flow in Nixon Channel and decrease flow in Green Channel versus Alternative 2, but to a lesser degree than Alternative 3. These results are due to the layouts of the design cuts. Under Alternative 3, more dredging occurs in Nixon Channel than near Green Channel. Under the terminal groin alternatives, dredging is limited to Nixon Channel. Removal of material from Nixon Channel slightly increases the flow capacity of this waterway, with less of the flow occurring through Green Channel as a result. However, in all cases, project-induced changes in the average tidal prism are 10% or less, and well within the variability shown in Table 4-8.

TABLE 11-12
TIDAL PRISM ESTIMATES GIVEN APRIL-JUNE 2006 INITIAL CONDITIONS & AVERAGE TIDES

Years after Construction	Inlet Entrance Avg. (feet^3)	Inlet Entrance +/- σ	Nixon Channel Avg. (feet^3)	Nixon Channel +/- σ	Green Channel Avg. (feet^3)	Green Channel +/- σ
	Alternative 2 – Abandon / Retreat					
0 to 1	502,800,000	10,700,000	280,000,000	6,600,000	179,500,000	3,800,000
1 to 2	496,900,000	12,100,000	276,700,000	6,900,000	177,900,000	4,500,000
2 to 3	473,900,000	12,200,000	277,600,000	6,600,000	179,300,000	4,700,000
3 to 4	506,100,000	12,200,000	279,600,000	7,300,000	183,600,000	4,900,000
4 to 5	505,900,000	13,500,000	275,700,000	9,000,000	184,600,000	4,500,000
5 to 6	509,000,000	11,300,000	276,100,000	8,300,000	184,400,000	3,900,000
6 to 7	507,600,000	13,400,000	270,500,000	9,200,000	184,600,000	4,700,000
Years after Construction	Inlet Entrance Avg. (feet^3)	Inlet Entrance +/- σ	Nixon Channel Avg. (feet^3)	Nixon Channel +/- σ	Green Channel Avg. (feet^3)	Green Channel +/- σ
	Alternative 3 – Rich Inlet Management and Beach Fill					
0 to 1	506,100,000	9,000,000	290,900,000	5,700,000	172,600,000	3,200,000
1 to 2	509,400,000	10,300,000	293,600,000	6,600,000	173,100,000	3,600,000
2 to 3	507,000,000	9,700,000	294,900,000	5,600,000	169,500,000	3,900,000
3 to 4	509,700,000	11,500,000	295,200,000	6,900,000	170,700,000	4,200,000
4 to 5	509,400,000	11,600,000	295,600,000	7,500,000	169,600,000	4,300,000
5 to 6	520,500,000	12,600,000	301,600,000	11,200,000	173,900,000	4,600,000
6 to 7	509,100,000	15,600,000	287,600,000	15,100,000	175,000,000	4,300,000
Years after Construction	Inlet Entrance Avg. (feet^3)	Inlet Entrance +/- σ	Nixon Channel Avg. (feet^3)	Nixon Channel +/- σ	Green Channel Avg. (feet^3)	Green Channel +/- σ

	Alternative 4 – Beach Fill without Management of Rich Inlet					
0 to 1	502,300,000	11,100,000	279,600,000	6,800,000	179,500,000	3,900,000
1 to 2	496,000,000	11,600,000	275,100,000	6,600,000	178,600,000	4,600,000
2 to 3	471,300,000	11,900,000	273,700,000	6,700,000	180,900,000	4,400,000
3 to 4	503,400,000	12,100,000	278,000,000	7,100,000	183,800,000	4,700,000
4 to 5	500,700,000	12,200,000	274,300,000	7,100,000	184,100,000	4,600,000
5 to 6	504,700,000	10,600,000	276,100,000	6,400,000	184,200,000	4,000,000
6 to 7	498,800,000	11,200,000	268,300,000	6,700,000	185,200,000	4,400,000

TABLE 11-12 (continued)
TIDAL PRISM ESTIMATES GIVEN APRIL-JUNE 2006 INITIAL CONDITIONS & AVERAGE TIDES

Years after	Inlet Entrance		Nixon Channel		Green Channel	
Construction	Avg. ($feet^3$)	+/- σ	Avg. ($feet^3$)	+/- σ	Avg. ($feet^3$)	+/- σ
	Alternative 5C - Terminal Groin with Beach Fill from Nixon Channel (Extended Cut)					
0 to 1	509,400,000	10,300,000	291,100,000	6,300,000	175,300,000	3,400,000
1 to 2	504,600,000	9,500,000	285,200,000	6,200,000	178,600,000	4,600,000
2 to 3	500,900,000	9,800,000	280,300,000	6,100,000	181,200,000	3,600,000
3 to 4	503,000,000	11,300,000	283,700,000	7,500,000	179,300,000	4,500,000
4 to 5	499,000,000	12,000,000	280,800,000	7,600,000	178,900,000	4,400,000
5 to 6	513,600,000	11,500,000	292,900,000	7,100,000	178,500,000	4,100,000
6 to 7	518,900,000	12,800,000	296,400,000	7,800,000	177,700,000	4,700,000
Years after Construction	**Inlet Entrance Avg. ($feet^3$)**	**+/- σ**	**Nixon Channel Avg. ($feet^3$)**	**+/- σ**	**Green Channel Avg. ($feet^3$)**	**+/- σ**
	Alternative 5D – 1300-ft Terminal Groin with Beach Fill from Nixon Channel (2010 Cut)					
0 to 1	505,200,000	10,800,000	284,100,000	6,900,000	177,600,000	3,600,000
1 to 2	501,200,000	11,900,000	282,500,000	7,000,000	175,700,000	4,400,000
2 to 3	506,700,000	12,100,000	287,800,000	6,900,000	174,900,000	4,600,000
3 to 4	514,000,000	13,600,000	291,600,000	7,700,000	177,100,000	5,300,000
4 to 5	515,300,000	12,800,000	289,600,000	7,100,000	179,600,000	5,300,000
5 to 6	519,700,000	11,500,000	290,400,000	7,000,000	181,400,000	4,500,000
6 to 7	521,600,000	11,200,000	288,500,000	6,800,000	183,600,000	4,500,000
Years after Construction	**Inlet Entrance Avg. ($feet^3$)**	**+/- σ**	**Nixon Channel Avg. ($feet^3$)**	**+/- σ**	**Green Channel Avg. ($feet^3$)**	**+/- σ**
	Alternative 5D – 1500-ft Terminal Groin with Beach Fill from Nixon Channel (2010 Cut)					
0 to 1	505,400,000	10,700,000	284,400,000	6,800,000	177,600,000	3,700,000
1 to 2	503,100,000	11,800,000	284,700,000	6,900,000	175,300,000	4,300,000
2 to 3	508,000,000	11,200,000	288,500,000	6,700,000	175,200,000	4,000,000
3 to 4	515,000,000	13,200,000	291,500,000	7,500,000	177,100,000	5,200,000
4 to 5	515,000,000	13,000,000	289,300,000	7,700,000	178,300,000	4,700,000
5 to 6	520,100,000	10,800,000	290,000,000	6,500,000	181,300,000	4,200,000
6 to 7	523,300,000	11,700,000	290,100,000	7,200,000	183,100,000	4,500,000

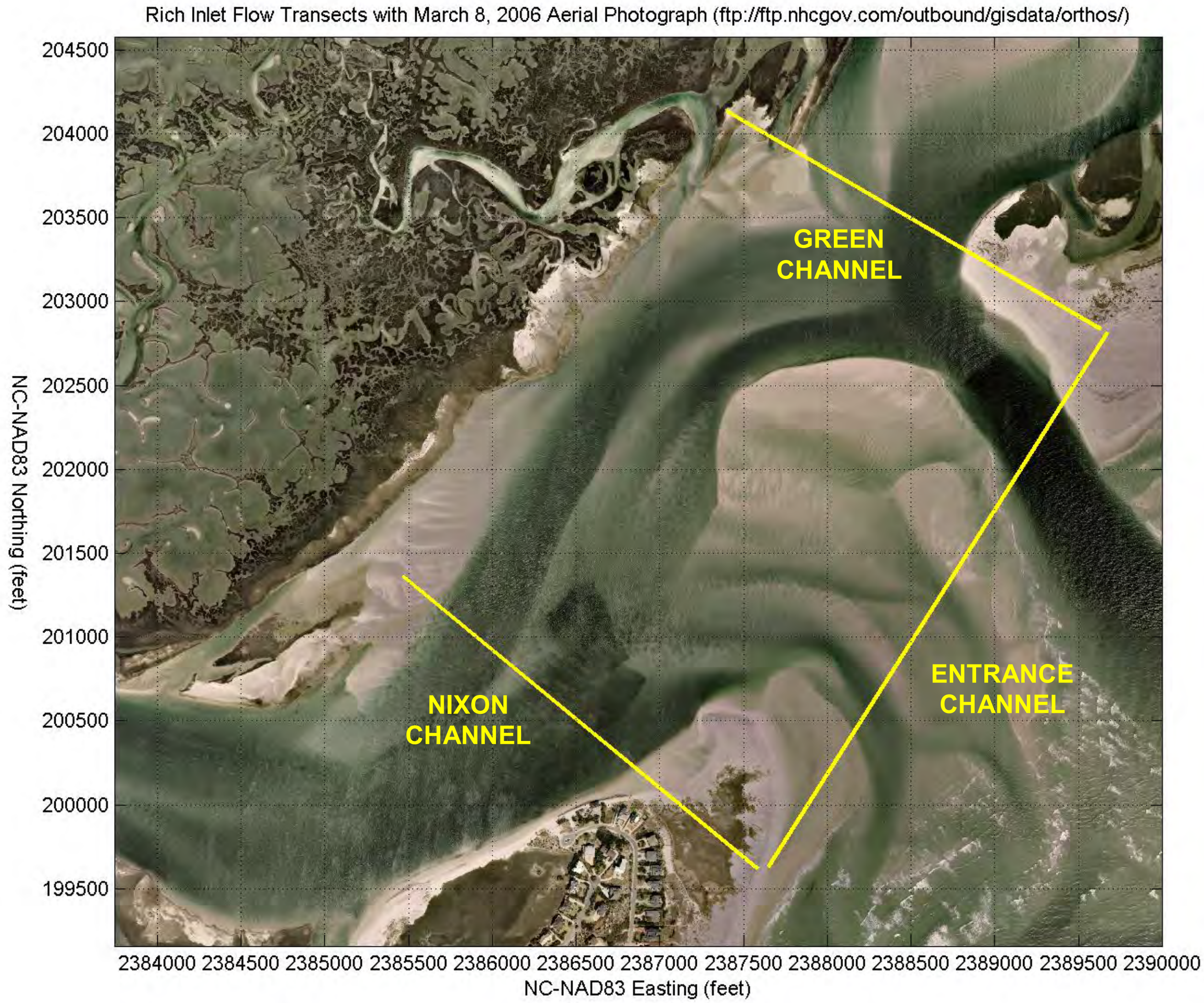


FIGURE 11-74: Rich Inlet Flow Transects.

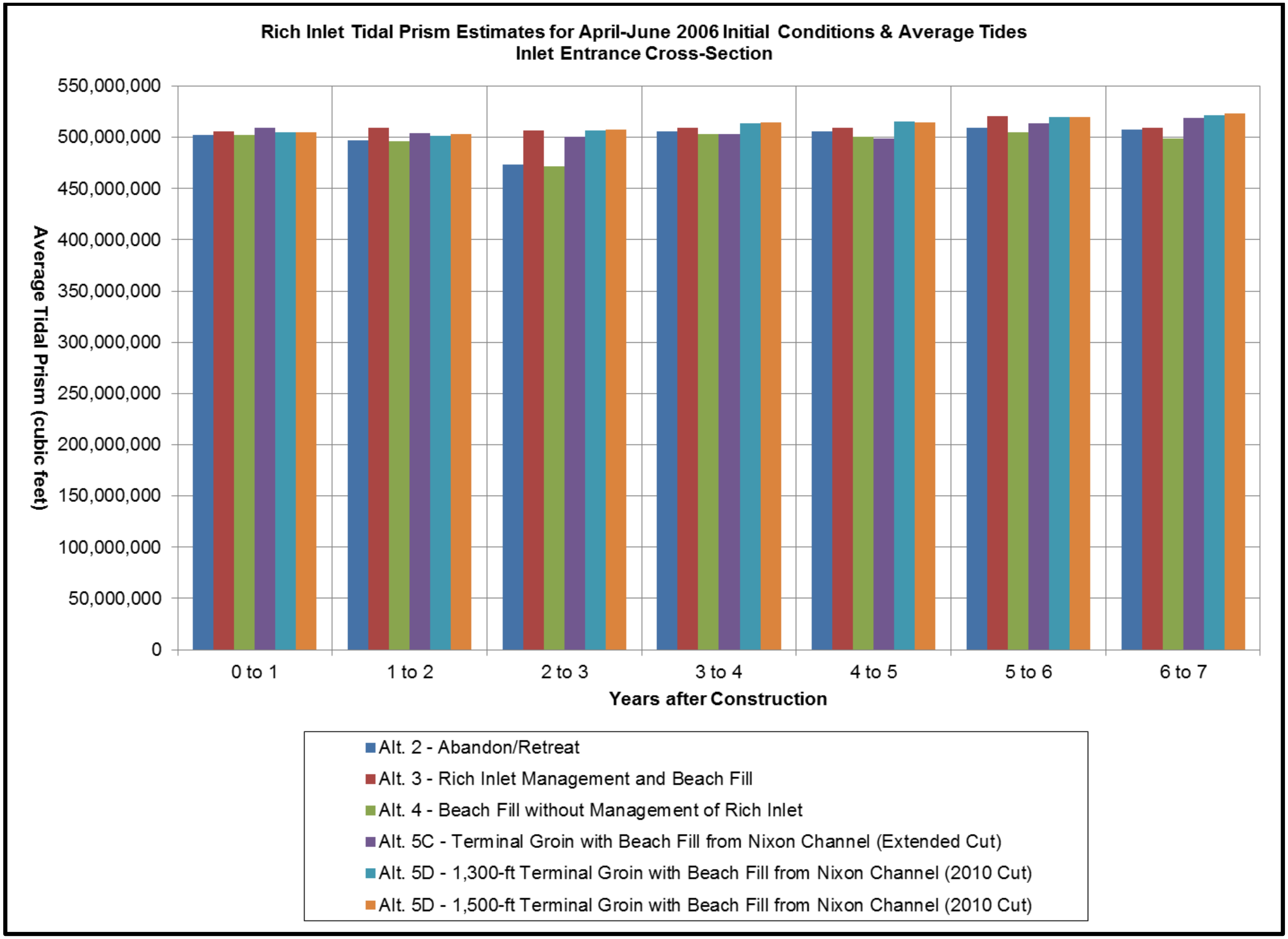


FIGURE 11-75: Tidal Prism Estimates for the Entrance Channel of Rich Inlet.

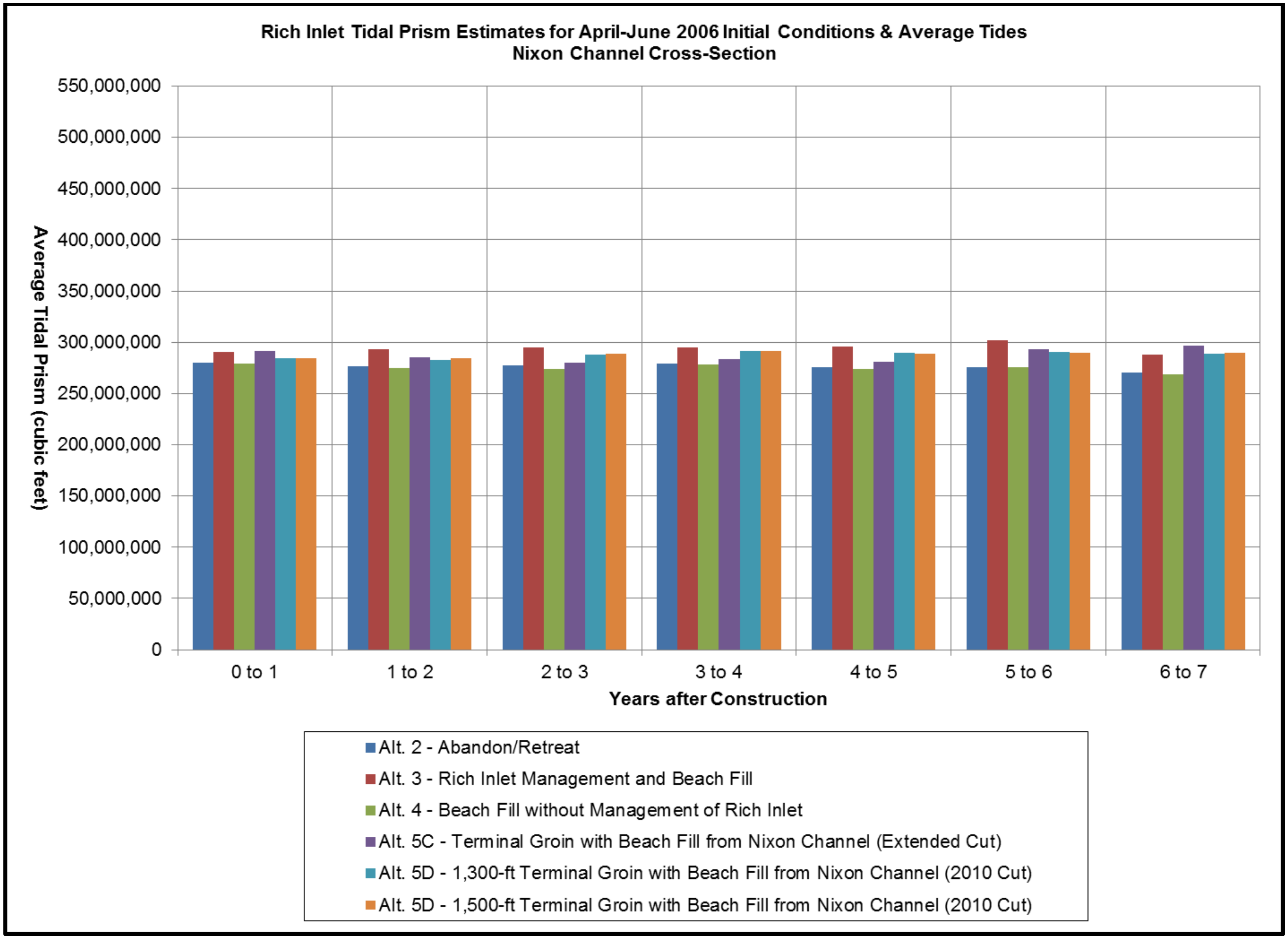


FIGURE 11-76: Tidal Prism Estimates for Nixon Channel.

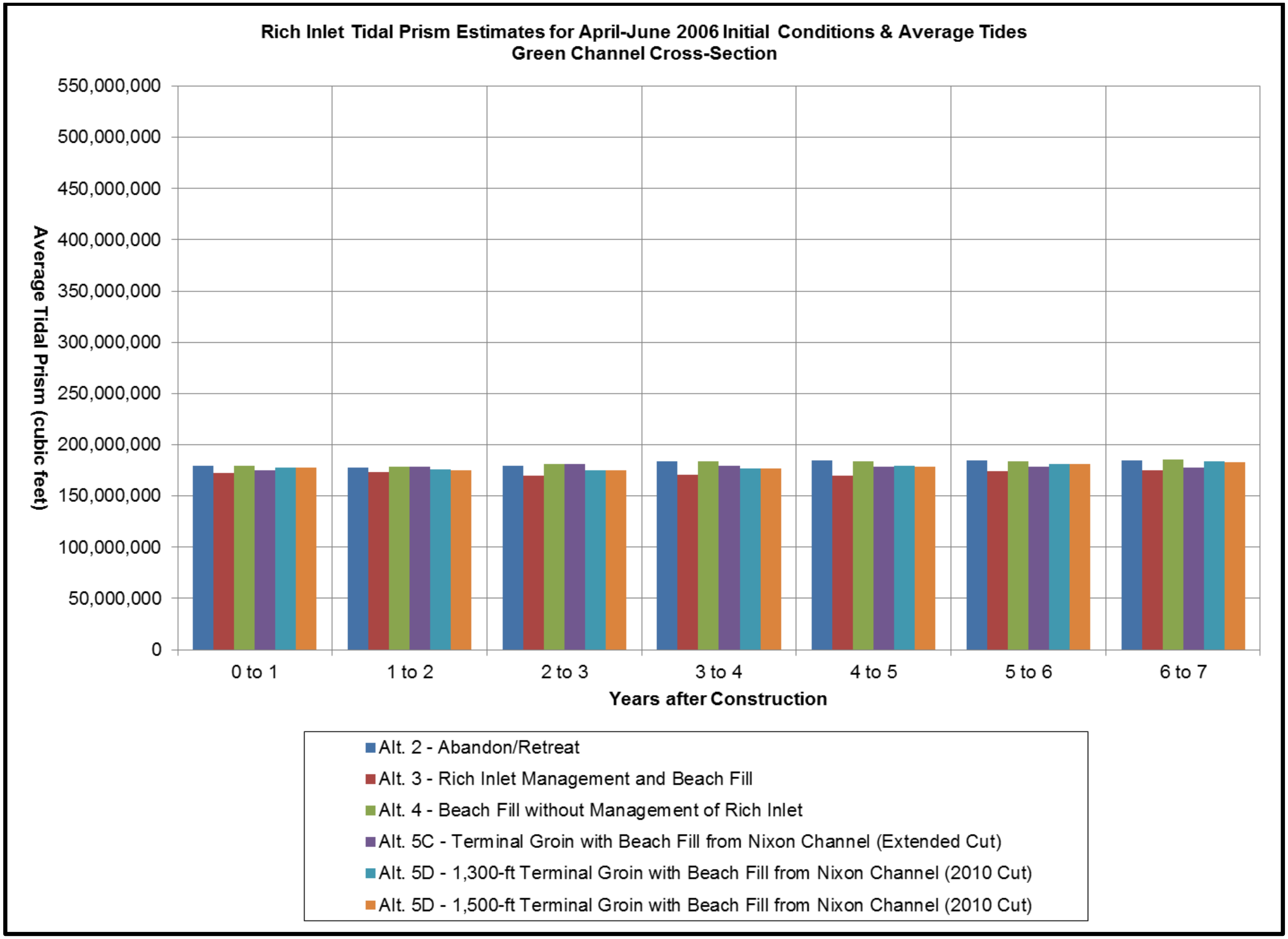


FIGURE 11-77: Tidal Prism Estimates for Green Channel.

11.6 Primary and Secondary Impact Areas

The primary impact areas are the areas falling within the beach fill templates and dredge cuts for each alternative (see Table 11-13). The secondary impact areas are based on the areas in which the vertical difference between Alternative 2 and Alternatives 3, 4, 5A, 5B-1, or 5B-2 in a given year was 0.5 feet or more (see Sub-Appendix B1 and Figures 11-51, 11-54, 11-57, 11-60, 11-65, 11-69, and 11-72). Secondary impacts include the longshore and cross-shore spreading of beach fill and the adjustment of the bottom bathymetry in Rich Inlet to the dredged conditions.

TABLE 11-13

PRIMARY IMPACT AREAS

Project Feature	Primary Impact Area (acres) Based on 2006 Critically Eroded Conditions				
	Alt. 3	Alt. 4	Alt. 5C	Alt. 5D 1300-ft	Alt. 5D 1500-ft
Oceanfront Fill Area	140.2	115.4	125.6	31.7	31.7
Nixon Channel Fill Area	7.4	7.4	7.4	7.4	7.4
Closure Dike	36.5	-N/A-	-N/A-	-N/A-	-N/A-
Dredge Cuts	92.3	-N/A-	77.6	44.7	44.7
TOTAL	276.4	122.8	210.6	83.8	83.8
Project Feature	**Primary Impact Area (acres) Based on 2012 Conditions**				
	Alt. 3	**Alt. 4**	**Alt. 5C**	**Alt. 5D 1300-ft**	**Alt. 5D 1500-ft**
Oceanfront Fill Area	146.9	119.2	127.5	40.0	40.0
Nixon Channel Fill Area	8.7	8.7	8.7	8.7	8.7
Closure Dike	29.6	-N/A-	-N/A-	-N/A-	-N/A-
Dredge Cuts	95.1	-N/A-	88.3	46.1	46.1
TOTAL	280.3	127.9	224.5	94.8	94.8

TABLE 11-14

DELFT3D SECONDARY IMPACT AREAS

Year after Construction	Secondary Impact Area (acres) Given 2006 Critically Eroded Conditions and						
	Alt. 1	Alt. 2	Alt. 3	Alt. 4	Alt. 5C	Alt. 5D 1300-ft	Alt. 5D 1500-ft
0	Not Simulated	0	276	123	211	84	84
1		0	875	366	732	210	266
2		0	1065	460	960	457	514
3		0	1238	569	1071	685	755
4		0	1345	690	1185	863	879
5		0	1433	813	1231	996	1055
6		0	1468	841	1329	1076	1112
7		0	1519	928	1337	1099	1147

Year after Construction	Secondary Impact Area (acres) Given 2012 Conditions and Alt. 1	Alt. 2	Alt. 3	Alt. 4	Alt. 5C	Alt. 5D 1300-ft	Alt. 5D 1500-ft
0		0	280	128		95	95
1		0	880	298		217	216
2	Not	0	997	376	Not	370	393
3		0	1158	405		607	620
4	Simulated	0	1216	515	Simulated	756	811
5		0	1212	564		823	894
6		0	1248	632		963	1037
7		0	1307	700		1095	1207

12.0 OCEANFRONT BEACH FILL PERFORMANCE BASED ON THE GENESIS MODEL

12.1 Background

To provide a "second opinion" regarding the performance and impact of the channel modification and terminal groin alternatives, this Shoreline Management study utilizes the Generalized Model for Simulating Shoreline Change (GENESIS). GENESIS can incorporate the effects of groins, revetments, seawalls, breakwaters, and offshore bathymetry. Inputs to the model include shoreline locations, structure locations, a time series of offshore waves, and, if desired, a set of wave refraction coefficients and refracted wave angles.

GENESIS determines shoreline changes relative to a fixed baseline based on the wave-driven, longshore sediment transport. The model assumes that shoreline change is directly proportional to volume change, the profile shape is relatively constant with time, the berm elevation is uniform, and the depth of closure is uniform. As such, it is a "one-line" model that calculates shoreline position rather than bathymetric changes. The primary advantage of the GENESIS model is its ability to rapidly simulate (1-5 minutes) long-term (5-20 year) shoreline changes using a narrow grid spacing (10-50 feet).

Transport rates are calculated using the USACE (1990) formula (CERC Equation), with an additional term to account for longshore variations in the breaking wave height. To calibrate the model, three longshore transport coefficients are determined:

1. Coefficient K1 governs the transport resulting from changes in the shoreline orientation. K1 typically ranges from 0.1 to 2 and has the largest influence on the model's results (Hanson and Kraus, 1991; CPE, 2007). If GENESIS is being used with a wave transformation model that includes bottom friction, the K1 values tend to be larger.
2. Coefficient K2 governs the transport resulting from variations in the breaking wave height (Hanson and Kraus, 1991). K2 typical ranges from 0 to the value of K1.

The GENESIS baseline for Figure Eight Island appears in Figure 12-1. The baseline extends from profile F0+00 near the south end of Beach Road to profile 110+00 near Rich Inlet. The length of the baseline is 22,000 feet, with a grid spacing of 25 feet. The purpose of the long baseline is to accommodate the spreading of beach fill material given the placement of beach fill between 8 Beach Road S and Rich Inlet (profiles F90+00 to 110+00).

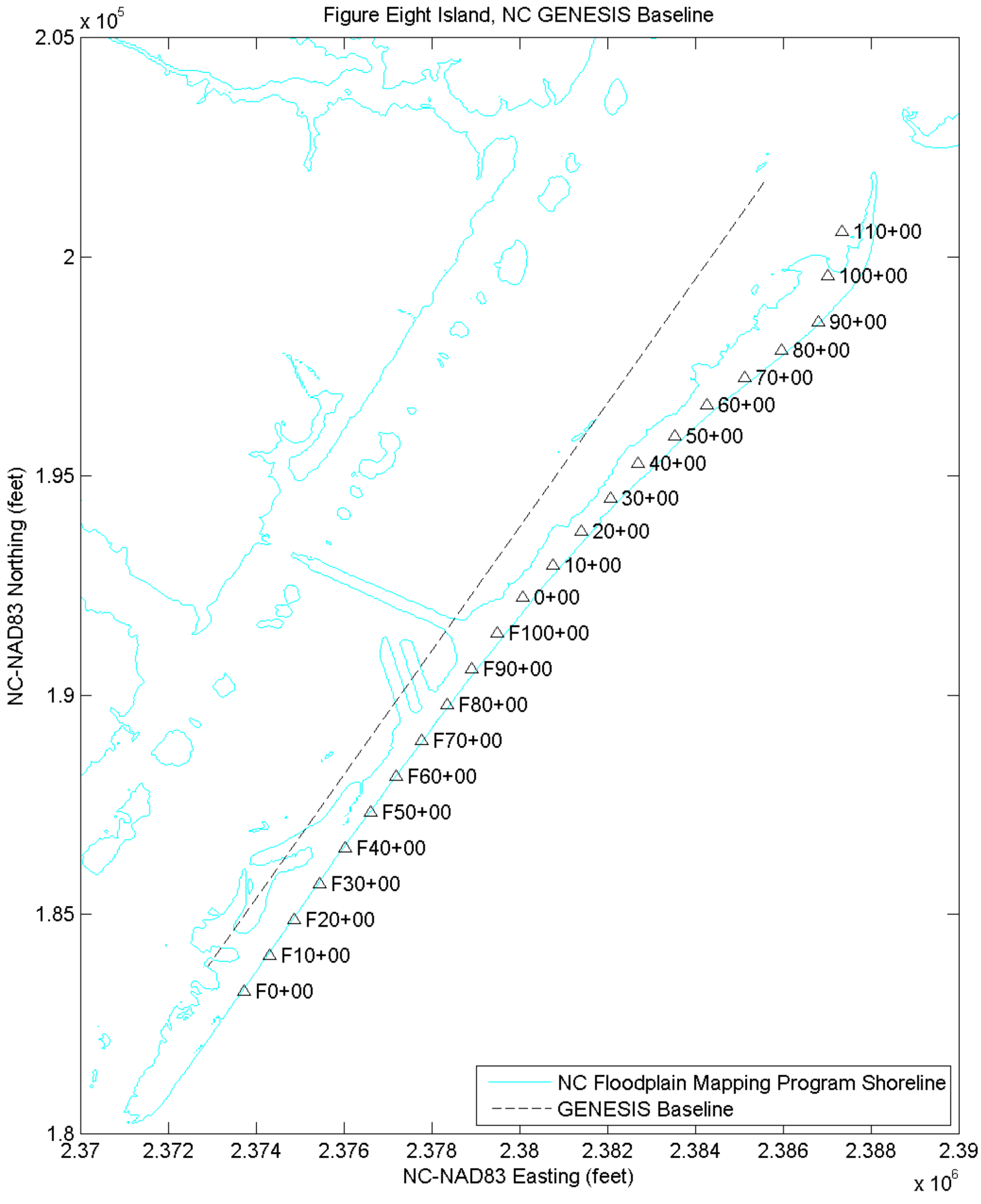


FIGURE 12-1: Figure Eight Island, NC GENESIS Baseline.

12.2 Wave Data

The wave data used in the GENESIS model was taken from the NOAA Western North Atlantic Wavewatch forecast at 34.00ºN, 76.25ºW, -644 feet NAVD (see Figure 11-12). This location was the same forecast node used in the Delft3D calibration. The record at this site extended from July 1, 1999 to December 31, 2012.

To determine the nearshore waves, the wave record was divided into the following wave height, period, and direction classes:

- Significant wave height classes: 0 to 6.4 feet, 6.4 to 10 feet, 10 to 35 feet.

- Peak wave period classes: 0-5 seconds, 5-7 seconds, 7-9 seconds, 9-11 seconds, 11-13 seconds, 13-15 seconds, 15-17 seconds, 17-23 seconds.

- Wave direction classes: 35-58°, 58-80°, 80-103°, 103-125°, 125-148°, 148-170°, 170-193°, 193-215°.

Each wave height classes contained an equal amount of wave energy in KW-Hours/m (see Section 11.3.2). The wave period and direction classes were based on typical divisions used in GENESIS modeling studies. Although the divisions above created 192 height, period, and direction classes, only 127 actually contained wave data. The average wave in each class (Table 12-1) was then transformed to the depth of closure (-24 feet NAVD) using the SWAN model. Refraction coefficients were then calculated based on the ratios of the transformed wave heights to the offshore wave heights in Table 12-1. The grids, bathymetries, and parameters used in the SWAN model were identical to those in Table 11-6 and Figures 11-10 to 11-15.

12.3 Model Calibration

The calibration of the GENESIS model was based on the shoreline and volume changes between April 2007 and October 2008. The April 2007 shoreline was used as the initial condition. A berm elevation of +6 feet NAVD was assumed, along with a closure depth of -24 feet NAVD and an average grain size of 0.18 mm (see Table 4-7). The sandbags along the north end of the island were neglected. When these were included in the model as a “seawall”, their effect was grossly overstated.

To determine the values of K1 and K2, several GENESIS runs were performed using K1 values ranging from 2 to 7. The best results were achieved by setting K1 equal to 2. Changing the value of K2 from 0 to 2 led to smoother shoreline and volume changes with respect to distance. It also provided for better results when the proposed groin was included in subsequent simulations (see Hanson and Kraus, 1991, p. 53).

In general, the agreement between the simulated and observed changes was good (Figures 12-2 and 12-3).

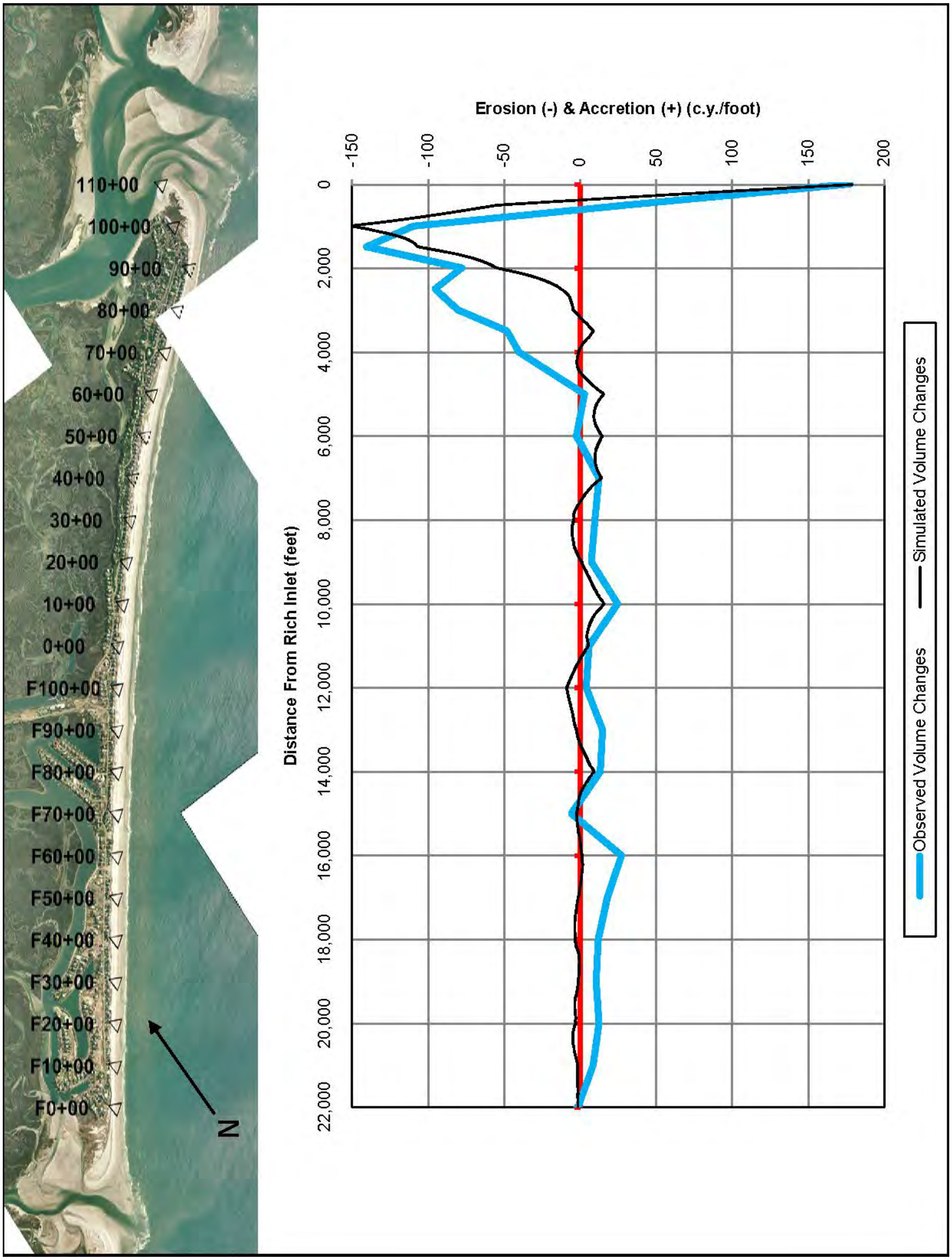


FIGURE 12-2: GENESIS Model Calibration, April 2007 to October 2008.

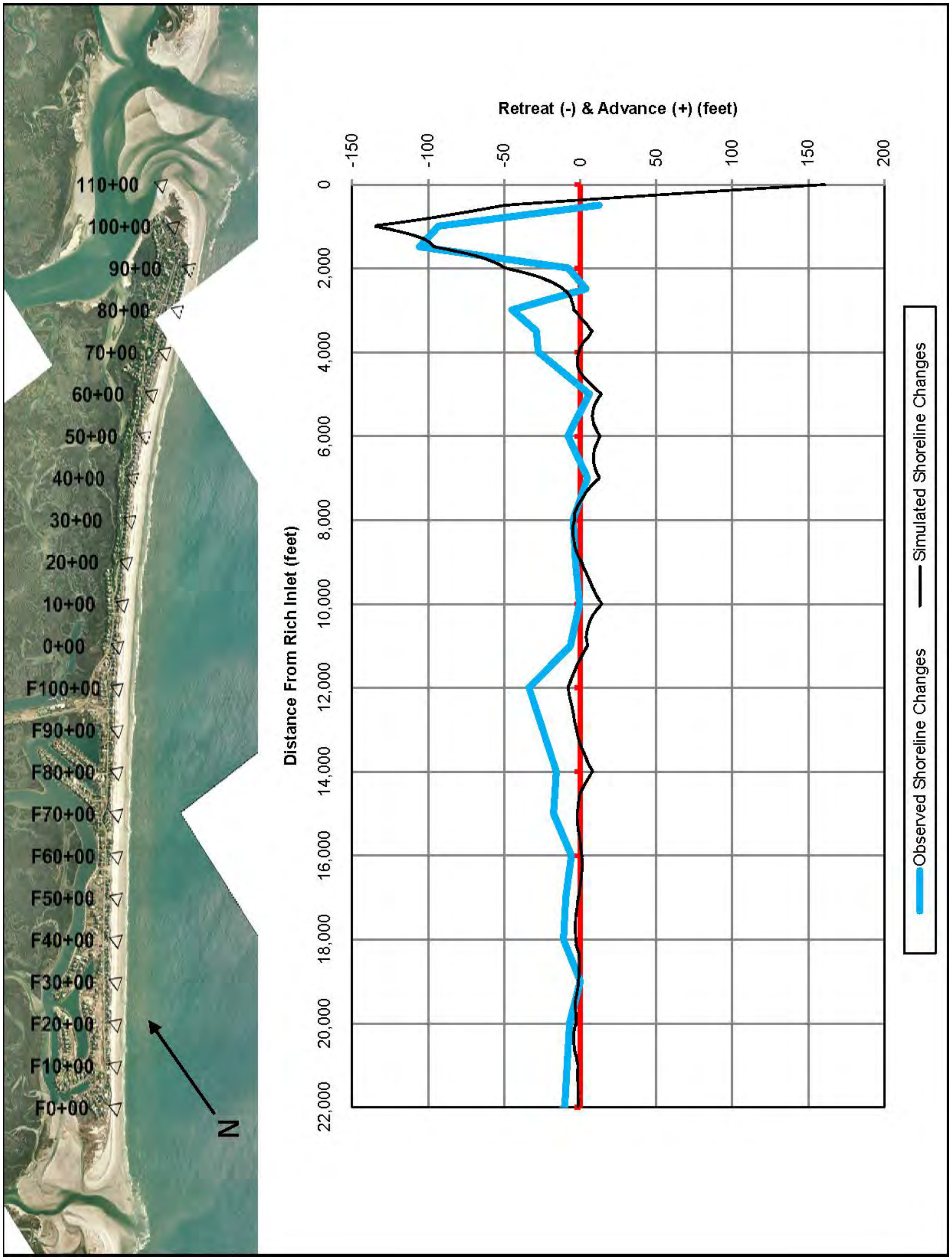


FIGURE 12-3: GENESIS Model Calibration, April 2007 to October 2008.

TABLE 12-1

WAVE CASES FOR GENESIS MODEL

Case #	Hs (feet)	Tp (sec.)	Dir. (°)
10101	4.1	4.4	46
20101	4.1	4.4	69
30101	4.1	4.4	91
40101	4.1	4.4	114
50101	4.1	4.4	136
60101	4.1	4.4	159
70101	4.1	4.4	181
80101	4.1	4.4	204
10201	4.1	6.0	46
20201	4.1	6.0	69
30201	4.1	6.0	91
40201	4.1	6.0	114
50201	4.1	6.0	136
60201	4.1	6.0	159
70201	4.1	6.0	181
80201	4.1	6.0	204
10301	4.1	8.0	46
20301	4.1	8.0	69
30301	4.1	8.0	91
40301	4.1	8.0	114
50301	4.1	8.0	136
60301	4.1	8.0	159
70301	4.1	8.0	181
80301	4.1	8.0	204
10401	4.1	9.8	46
20401	4.1	9.8	69
30401	4.1	9.8	91
40401	4.1	9.8	114
50401	4.1	9.8	136
60401	4.1	9.8	159
70401	4.1	9.8	181
10501	4.1	11.7	46
20501	4.1	11.7	69
30501	4.1	11.7	91
40501	4.1	11.7	114
50501	4.1	11.7	136
10601	4.1	13.7	46
20601	4.1	13.7	69
30601	4.1	13.7	91
40601	4.1	13.7	114
50601	4.1	13.7	136
20701	4.1	15.6	69
40701	4.1	15.6	114
50701	4.1	15.6	136
10102	7.8	4.4	46
50102	7.8	4.4	136
60102	7.8	4.4	159
70102	7.8	4.4	181
80102	7.8	4.4	204
10202	7.8	6.0	46
20202	7.8	6.0	69
30202	7.8	6.0	91
40202	7.8	6.0	114
50202	7.8	6.0	136
60202	7.8	6.0	159
70202	7.8	6.0	181
80202	7.8	6.0	204

Case #	Hs (feet)	Tp (sec.)	Dir. (°)
10302	7.8	8.0	46
20302	7.8	8.0	69
30302	7.8	8.0	91
40302	7.8	8.0	114
50302	7.8	8.0	136
60302	7.8	8.0	159
70302	7.8	8.0	181
80302	7.8	8.0	204
10402	7.8	9.8	46
20402	7.8	9.8	69
30402	7.8	9.8	91
40402	7.8	9.8	114
50402	7.8	9.8	136
60402	7.8	9.8	159
70402	7.8	9.8	181
80402	7.8	9.8	204
10502	7.8	11.7	46
20502	7.8	11.7	69
30502	7.8	11.7	91
40502	7.8	11.7	114
50502	7.8	11.7	136
60502	7.8	11.7	159
70502	7.8	11.7	181
80502	7.8	11.7	204
10602	7.8	13.7	46
20602	7.8	13.7	69
30602	7.8	13.7	91
40602	7.8	13.7	114
50602	7.8	13.7	136
60602	7.8	13.7	159
20702	7.8	15.6	69
40702	7.8	15.6	114
40802	7.8	17.4	114
10203	12.2	6.0	46
20203	12.2	6.0	69
30203	12.2	6.0	91
40203	12.2	6.0	114
50203	12.2	6.0	136
60203	12.2	6.0	159
70203	12.2	6.0	181
80203	12.2	6.0	204
10303	12.2	8.0	46
20303	12.2	8.0	69
30303	12.2	8.0	91
40303	12.2	8.0	114
50303	12.2	8.0	136
60303	12.2	8.0	159
70303	12.2	8.0	181
80303	12.2	8.0	204
10403	12.2	9.8	46
20403	12.2	9.8	69
30403	12.2	9.8	91
40403	12.2	9.8	114
50403	12.2	9.8	136
60403	12.2	9.8	159
70403	12.2	9.8	181
80403	12.2	9.8	204

TABLE 12-1 (continued)

WAVE CASES FOR GENESIS MODEL

Case #	Hs (feet)	Tp (sec.)	Dir. (°)
10503	12.2	11.7	46
20503	12.2	11.7	69
50503	12.2	11.7	136
60503	12.2	11.7	159
70503	12.2	11.7	181
20603	12.2	13.7	69
30603	12.2	13.7	91
40603	12.2	13.7	114
50603	12.2	13.7	136
60603	12.2	13.7	159
70603	12.2	13.7	181
40703	12.2	15.6	114
50703	12.2	15.6	136

The only exception was the area between Surf Court and Comber Road (profiles 65+00 to 90+00), where the model predicted a stable beach instead of an eroding beach. At all other locations, the model results were generally consistent with the observed shoreline and volume changes.

12.4 Model Verification

The verification of the GENESIS model was based on the shoreline and volume changes between April 2006 and April 2007. This period was preceded by beach fill operations on the northern and southern thirds of the island (see Table 6-2). Observed volume change patterns were characterized by an erosion hotspot on the north end of the island, stability in the middle of the island, and erosion on the southern third of the island. The April 2006 shoreline was used as the initial condition on the northern half of the island, and the June 2006 shoreline was used as the initial condition on the southern half of the island. The values of K1 and K2 were identical to those used in the final calibration run, and the existing sandbags were neglected.

Along most of Figure Eight Island, shoreline changes during the verification period were characterized by the change in the beach profile shape following the various beach fill operations (see Figure 12-5). Since this process was not included in the GENESIS model, differences between the simulated and observed shoreline changes occurred in several locations. However, on the northern and central sections of the island, agreement between the simulated and observed volume changes was good (Figure 12-4). The overall volume change patterns that occurred between April 2006 and April 2007 were reproduced by the model. On the southern third of the island (profiles F0+00 to F70+00), the GENESIS model tended to predict stable beaches instead of eroding beaches. This was due to the fact that the waves and tidal currents in Mason Inlet were not incorporated into the SWAN and GENESIS models.

Overall the calibration and verification showed that the GENESIS model is able to simulate the observed shoreline and volume changes after the beach profiles have adjusted to their equilibrium shape. During the initial adjustment period, which ranges from 1-3 years, the GENESIS model is best used as a volume change model. Based on the results presented in Figures 12-2 to 12-5, the GENESIS model is suitable for providing a “second opinion” regarding

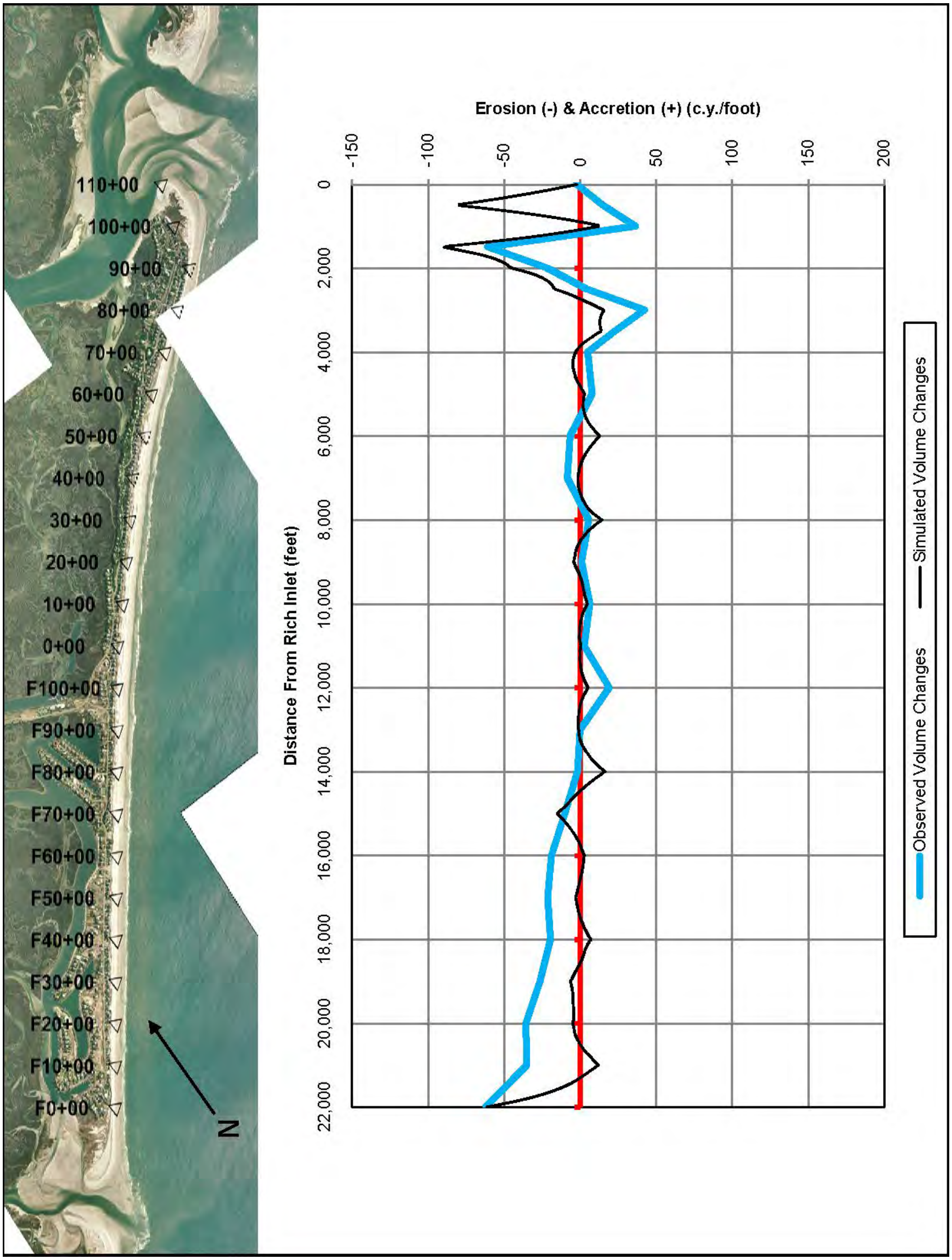


FIGURE 12-4: GENESIS Model Verification, April 2006 to April 2007.

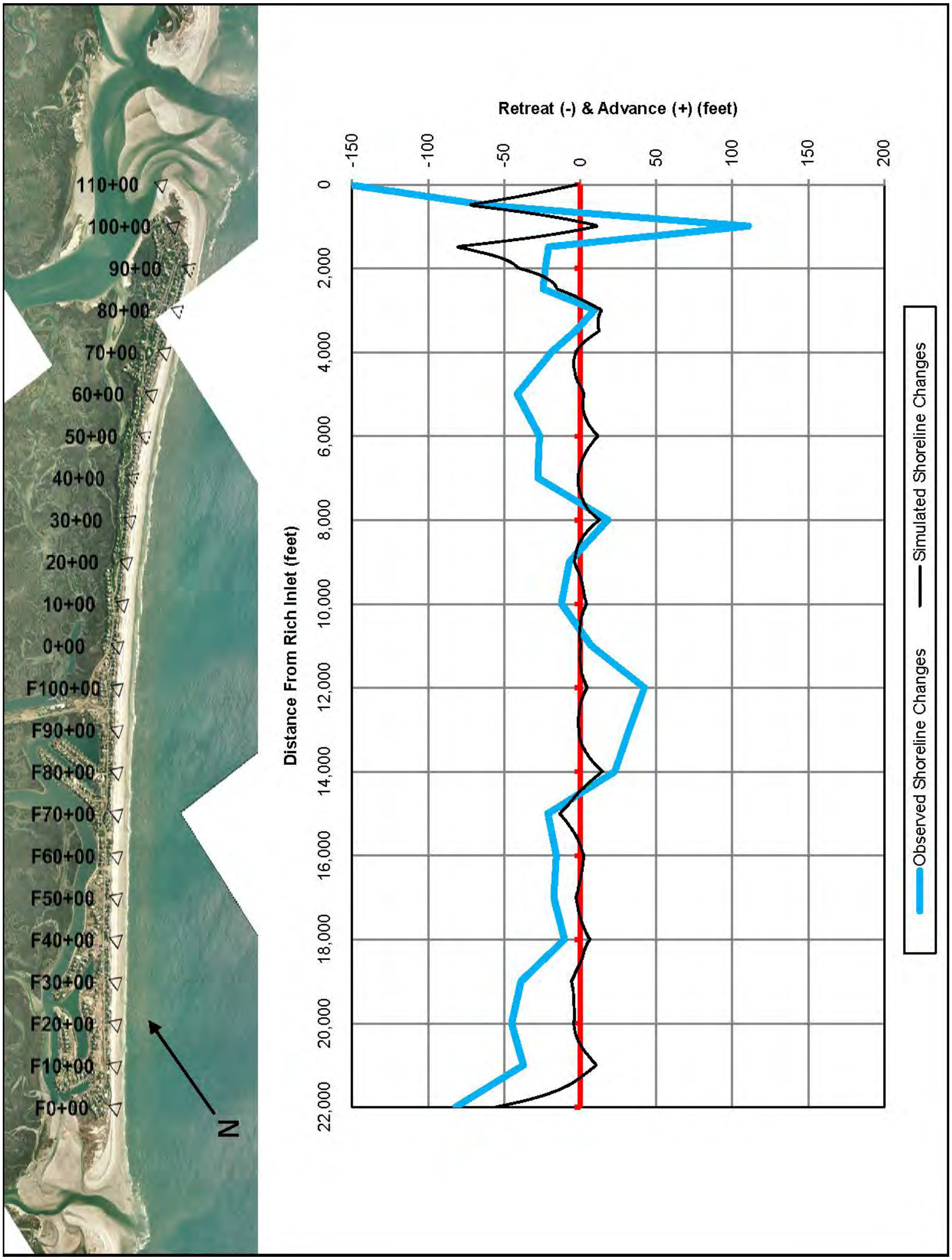


FIGURE 12-5: GENESIS Model Verification, April 2006 to April 2007.

beach fill performance over a 10 year study period on the northern and middle sections of Figure Eight Island.

12.5 Performance of the Alternatives

Using the calibrated GENESIS model, shoreline changes were estimated given the following alternatives:

- Alt. 2 - Abandon/Retreat
- Alt. 3 - Rich Inlet Management and Beach Fill
- Alt. 4 - Beach Fill without Management of Rich Inlet
- Alt. 5C - Terminal Groin with Beach Fill from Nixon Channel (Extended Cut)
- Alt. 5D – 1,300-foot Terminal Groin with Beach Fill from Nixon Channel (2010 Cut)
- Alt. 5D – 1,500 foot Terminal Groin with Beach Fill from Nixon Channel (2010 Cut)

and the following conditions:

- April 2007 critically eroded conditions.
- March 2012 conditions.

Similar to the Delft3D model results, it is important to note that once the project has been constructed, the project area will have changed relative to either set of conditions (April 2006 or March 2012). Unlike the Delft3D model, the GENESIS model is able to incorporate the effects of beach fill during the middle of a simulation. However, neither model is able to *predict* the occurrence of beach fill operations, hurricanes, tropical storms, or northeasters in future years. The GENESIS model can only estimate the effects of such events based on assumptions provided as input. These assumptions are detailed below. Given the various assumptions required to run the GENESIS model, the results in Sub-Appendix C are best suited for comparisons between alternatives. They cannot and should not be used to provide absolute predictions of the future.

12.5.1 Waves

To account for risk and uncertainty, 10 runs were performed for each scenario using random sequences of annual waves (Table 12-2). An additional run was then conducted using the actual wave sequence between 1999 and 2009, for a total of 11 runs. The 11 simulations were then averaged to provide the mean shoreline positions and confidence intervals appearing in Sub-Appendix C. To provide information regarding long-term changes, the duration of each simulation was 10 years.

TABLE 12-2

RANDOM SEQUENCES OF ANNUAL WAVES USED IN FUTURE CONDITIONS SIMULATIONS

Year of Project Life	Years from Wave Record Used in Random Wave Sequence in Run # …										
	1	2	3	4	5	6	7	8	9	10	11
0	2010	2005	2003	2009	2011	2004	2007	2003	2003	2005	1999
1	2011	2011	2001	2009	2012	2010	2006	2011	2010	2001	2000
2	2002	2010	2001	2003	2007	2007	2000	2002	2005	2003	2001
3	2011	2012	2010	2008	2002	2007	2004	2010	2011	2001	2002
4	2008	2008	2008	2008	2002	2011	2002	2006	2002	2002	2003
5	2001	2000	2004	2002	2003	2003	2010	2012	2003	2003	2004
6	2003	2010	2011	2001	2010	2009	2004	2001	2002	2005	2005
7	2007	2011	2000	2006	2003	2009	2006	2005	2002	2001	2006
8	2011	2008	2005	2012	2010	2005	2002	2001	2010	2011	2007
9	2012	2009	2005	2004	2003	2007	2007	2012	2007	2011	2008
10	2002	2009	2009	2007	2011	2001	2003	2000	2007	2006	2009

12.5.2 Alternatives 1 and 2

Alternative 1 assumes that the present strategies to manage the island's shoreline in Table 6-1 will continue into the future. Although the GENESIS model can incorporate the effect of beach fill during the middle of a simulation, dredging and fill operations around Figure Eight Island are highly variable in terms of timing and quantity (see Table 6-1). As such, they are difficult to predict with any degree of certainty. Since the input required to simulate Alternative 1 cannot be formulated with a sufficient degree of certainty, Alternative 1 was not simulated in the GENESIS model. Instead, Alternative 2 was used as the "Absolutely No Action" scenario by which to evaluate the other alternatives. It is important to note that Alternative 2 does *not* approximate what occurred between 2007 and 2012 (see Table 6-1).

The initial conditions for the critically eroded version of Alternative 2 were based on the April 2007 beach profile survey. Initial conditions for the 2012 scenario were based on the March 2012 survey on the northern half of Figure Eight Island and the August 2012 aerial photograph on the southern half of Figure Eight Island. In both scenarios, the effects of the existing sandbags were neglected. To account for changes in the ebb shoal between 2007 and 2012, the refraction coefficients for the 2012 scenario were updated by re-running the wave cases in Table 12-1 using the 2012 bathymetry (see Figure 11-45, top half). The refraction coefficients for the 2007 scenario were identical to the ones used in the original calibration of the GENESIS model, which were based on the 2006 bathymetries shown in Figures 11-12 through 11-14. Model results at Year 5 given Alternative 2 appear in Figures 12-6 and 12-7.

In general, the model results suggest that given eroded conditions similar to those in 2007, severe erosion would continue if the existing sandbags were removed. Oceanfront properties between profiles 80+00 and 95+00 (13 Comber Road to Inlet Hook Road) would be lost to erosion, with the further possibility of losing Inlet Hook Road itself (see Figure 12-6).

FIGURE 12-6: GENESIS Year 5 Conditions Given Alternative 2 under April 2007 Critically Eroded Conditions.

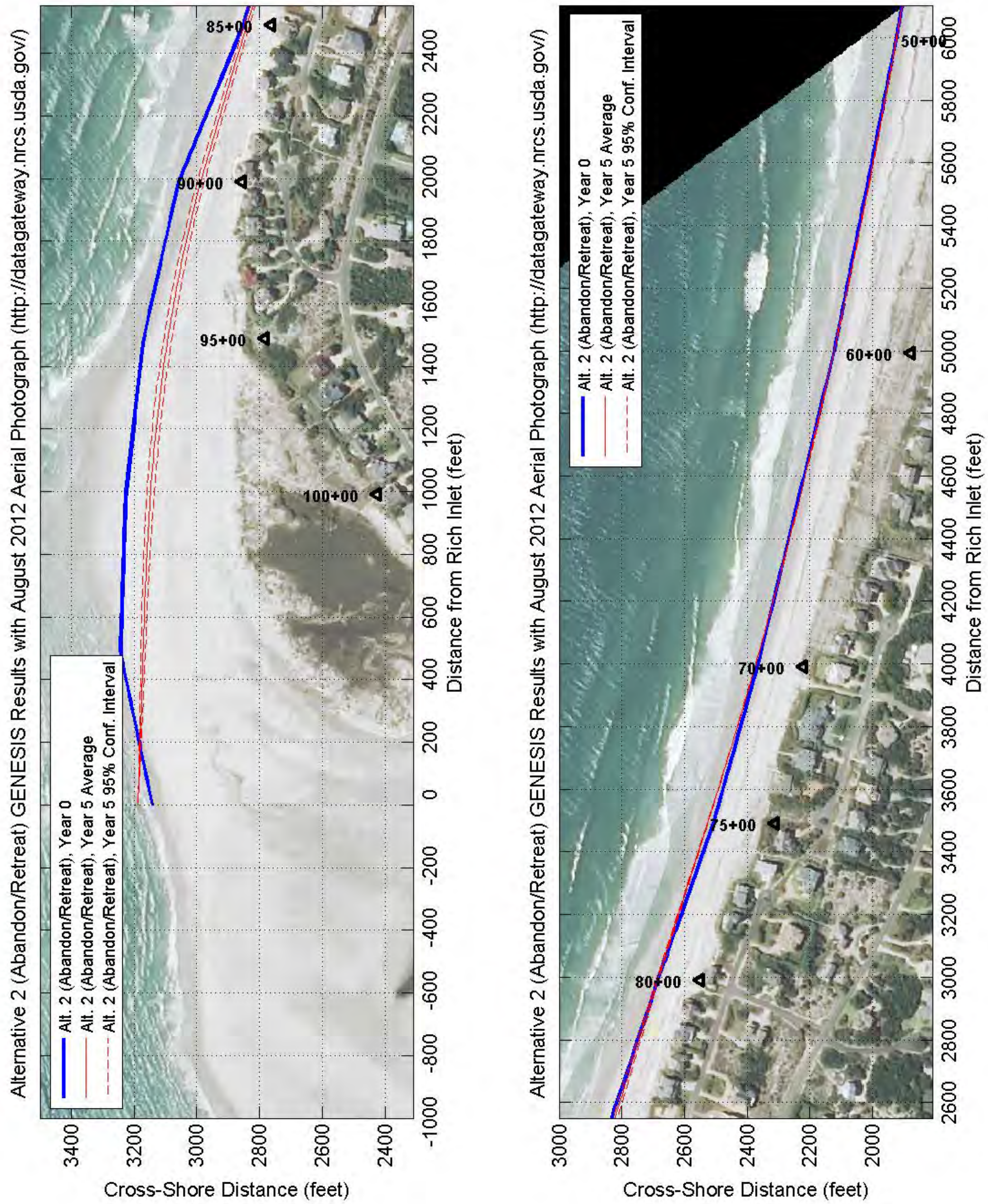


FIGURE 12-7: GENESIS Year 5 Conditions Given Alternative 2 under 2012 Conditions.

Under conditions similar to those in 2012, the model suggests that erosion could occur north of profile 80+00 (13 Comber Road) and Rich Inlet. Although this area has gained material since 2007, some of the gains have occurred due to the placement of beach fill (see Table 6-1 and Figures 6-2b and 7-1b). When the effects of beach fill are removed, the survey data suggests that an erosion hotspot still exists near the north end of Figure Eight Island (see Figures 6-2b and 7-1b). The primary difference between the model results and the survey data is not whether an erosion hotspot exists, but, rather, where it is centered. The survey data suggests that the erosion hotspot is centered between profiles 75+00 and 80+00 (see Figures 6-2b and 7-1b), while the GENESIS model results suggest that the erosion hotspot is centered further north (see Figure 12-7). Overall, the model results suggest that the existing beach would be wide enough to prevent erosion-related losses to upland properties at the north end of the island (see Figure 12-7 and Sub-Appendix C). However, given the differences between the model results in Figure 12-7 and the observed erosion patterns (Figures 6-2b and 7-1b), this finding should be confirmed using future monitoring surveys.

12.5.3 Alternative 3

Alternative 3 was incorporated into the GENESIS model by widening the 2007 and 2012 shorelines based on the "adjusted berm width" in Column 4 of Table 9-2. Since the GENESIS model did not include cross-shore transport, it was necessary to assume that the adjustment to an equilibrium beach profile shape (see Figure 9-13) would occur shortly after construction. For this reason, the "adjusted berm width" in Table 9-2 was used to develop the initial conditions, rather than beach widths based on the construction templates (see Figure 9-13). Renourishment of profiles 60+00 to 105+00 was implemented after the end of Year 5.

Preliminary simulations examined the sensitivity of the GENESIS and SWAN models to dredging in Rich Inlet. Specifically, the 2006 bathymetry (Figure 11-43, top half) was replaced with the post-construction bathymetry under Alternatives 3 and 5A (top halves of Figures 11-49 and 11-63). Using the SWAN model and the 3 different bathymetries, refraction coefficients and wave directions were computed along the -24 foot NAVD contour. Although dredging altered the wave patterns within the inlet, it did not substantially change the refraction coefficients and wave directions along the GENESIS model domain. Had the wave transformation estimates for the GENESIS model been based on the bathymetries at Years 2 or 5, inlet dredging would have altered the refraction coefficients. However, the GENESIS model would no longer be independent from the Delft3D-FLOW model. For these reasons, the refraction coefficients and nearshore wave angles for Alternatives 3, 4, 5C, and 5D with both the 1,300-ft and 1,500-ft terminal groins were the same as those for Alternative 2.

GENESIS model results for Alternative 3 appear in Figures 12-8 through 12-10. Given eroded conditions similar to those in 2007, the model suggests that by Year 5 erosion into the pre-construction shoreline will have occurred north of Comber Road (see Figures 12-8 and 12-10). This finding is consistent with the Delft3D model results (see Figure 11-48). Without the existing sandbags in place, a number of homes along Comber Road could be lost to erosion. However, the risk of loss is less than what would occur under an "absolutely no action" scenario (see Figure 12-8).

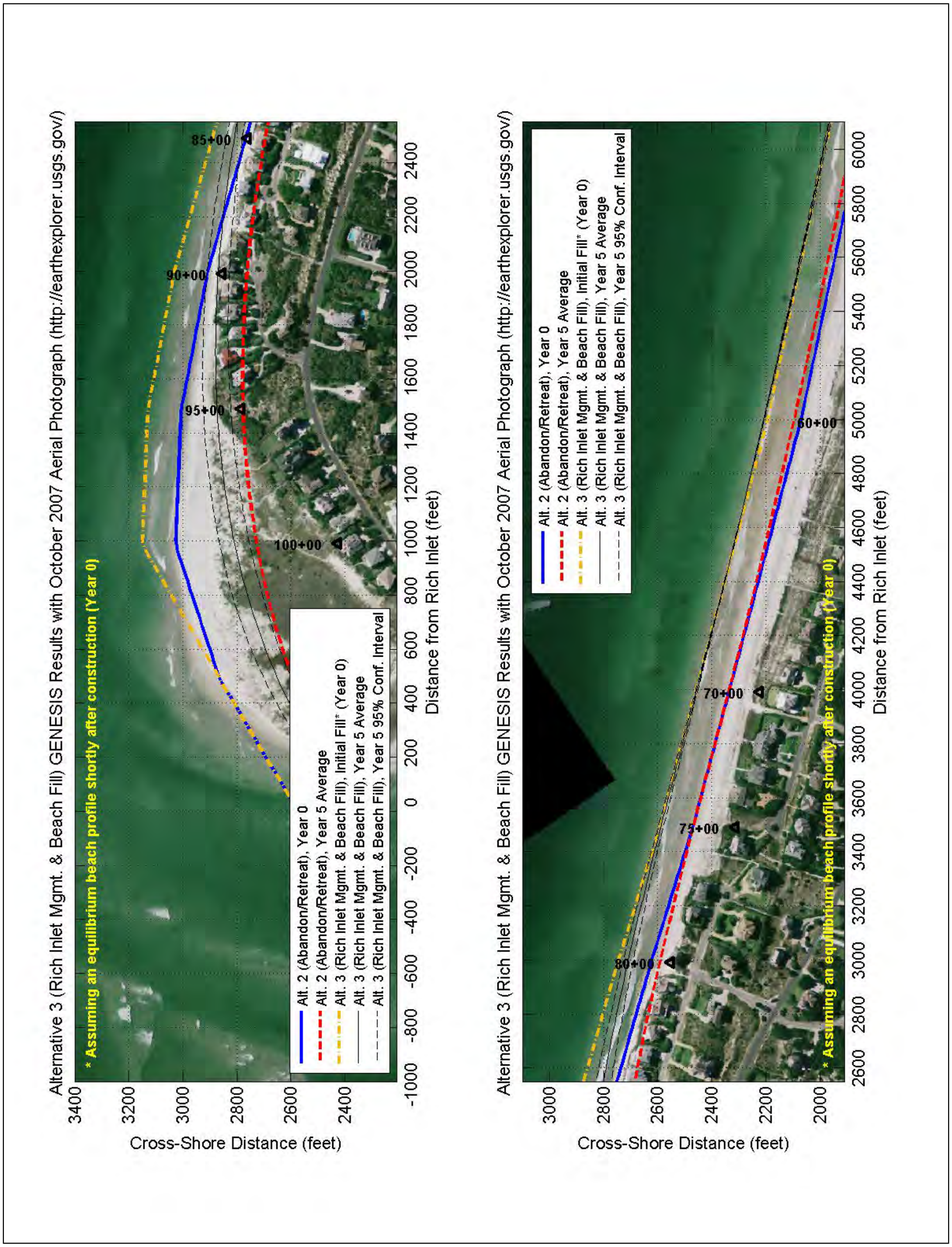


FIGURE 12-8: GENESIS Year 5 Conditions Given Alternative 3 under April 2007 Critically Eroded Conditions.

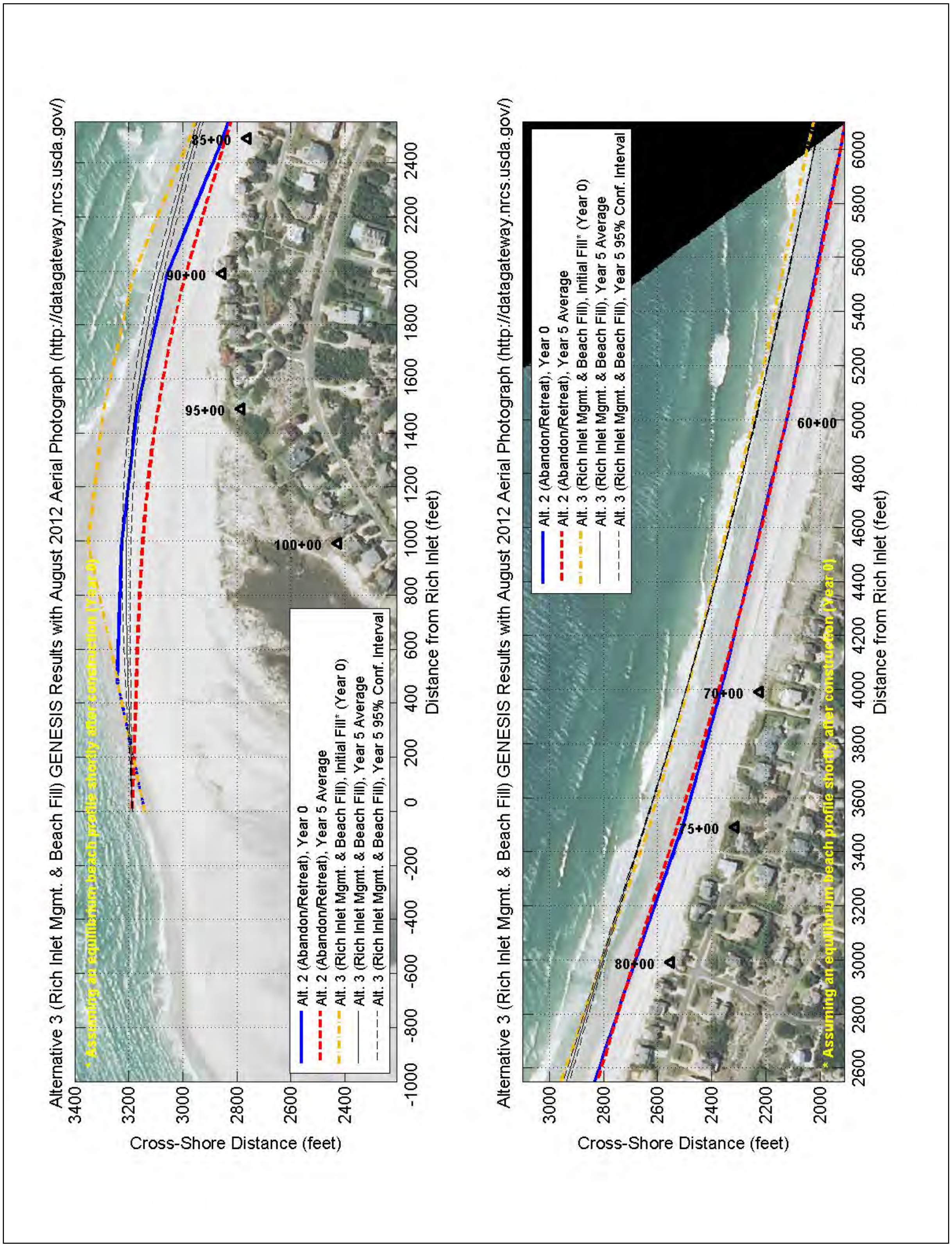


FIGURE 12-9: GENESIS Year 5 Conditions Given Alternative 3 under 2012 Conditions.

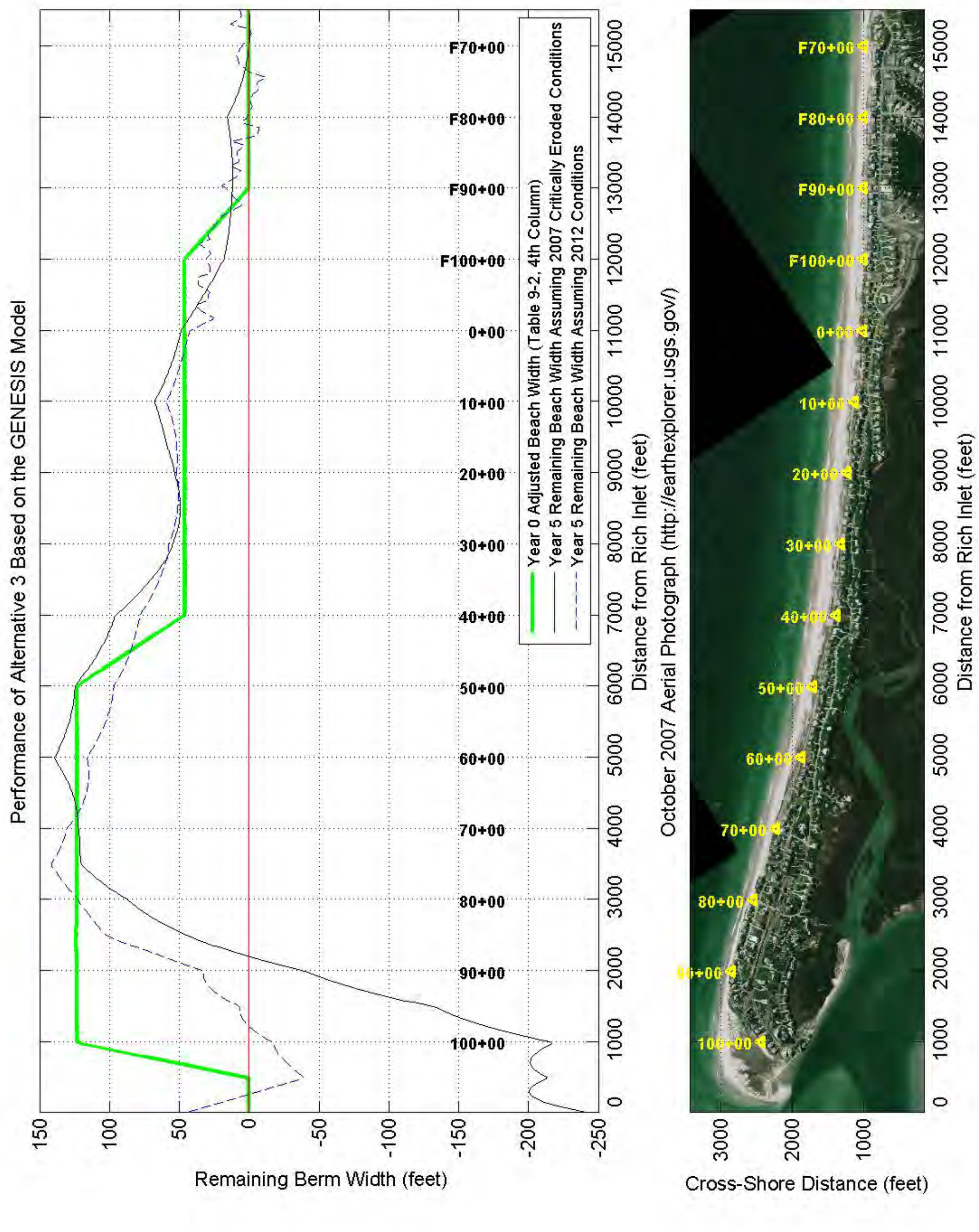


FIGURE 12-10: Remaining Beach Width Given Alternative 3 Based on the GENESIS Model.

Given conditions similar to those in 2012, the model suggests that erosion into the pre-construction shoreline over the first 5 years would be limited to the area north of profile 95+00 (see Figures 12-9 and 12-10). Given the distances between the upland buildings and the 2012 shoreline, the erosion would not pose a risk to upland development (see Figure 12-9). Along Comber Road, more erosion might occur than what the model suggests. However, given the amount of fill and the location of the 2012 shoreline, this risk appears to be manageable (see Figure 12-9).

12.5.4 Alternative 4

Alternative 4 was incorporated into the GENESIS model by widening the 2007 and 2012 shorelines based on the "adjusted berm width" in Column 3 of Table 9-5. Renourishment of profiles 60+00 to 105+00 was implemented after the end of Year 4. The effects of beach renourishment on the model results are illustrated in Figures 12-11 and 12-12, which show the results of the model at Years 4 and 5. Simulated beach widths at Year 5 are significantly greater than those at Year 4 due to the placement of fill on profiles 60+00 to 105+00 between Years 4 and 5.

GENESIS model results for Alternative 4 appear in Figures 12-11 through 12-13. Given eroded conditions similar to those in 2007, the model suggests that by Year 4, erosion into the pre-construction shoreline would occur north of profile 90+00 (see Figures 12-11 and 12-13). However, the risk of losing upland buildings due to erosion appears to be low (see Figure 12-11 and Sub-Appendix C). Given conditions similar to those in 2012, the model suggests that erosion into the pre-construction shoreline over the first 4 years would be limited to the taper sections at either end of the fill area (see Figures 12-12 and 12-13).

12.5.5 Alternative 5C

The beach fill for Alternative 5C was incorporated into the GENESIS model by widening the 2007 and 2012 shorelines based on the "adjusted berm width" in Column 3 of Table 9-6. Renourishment of profiles 60+00 to 102+50 was implemented after the end of Year 5.

As shown in Figures 12-14 and 12-15, the proposed terminal groin alignment is at an angle to the shoreline and the model's baseline. In cases such as these, the model's developers recommend that the structure be treated as a combination of an offshore breakwater and a diffracting groin (see Figure 12-16). Accordingly, the terminal groin was simulated as a diffracting groin with an effective permeability of 37% and an adjoining, offshore breakwater.

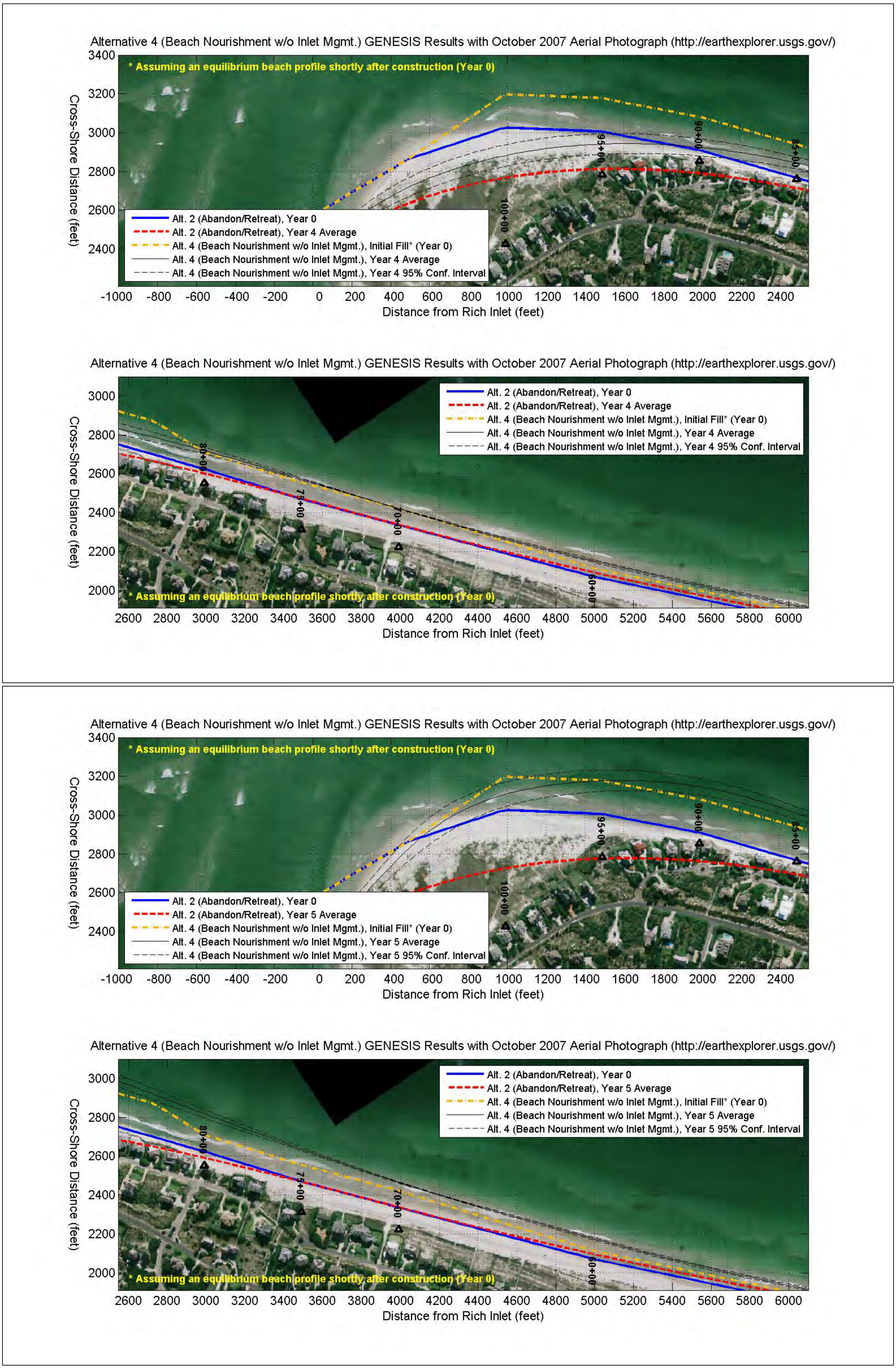


FIGURE 12-11: GENESIS Year 4 & 5 Conditions Given Alternative 4 under April 2007 Critically Eroded Conditions.

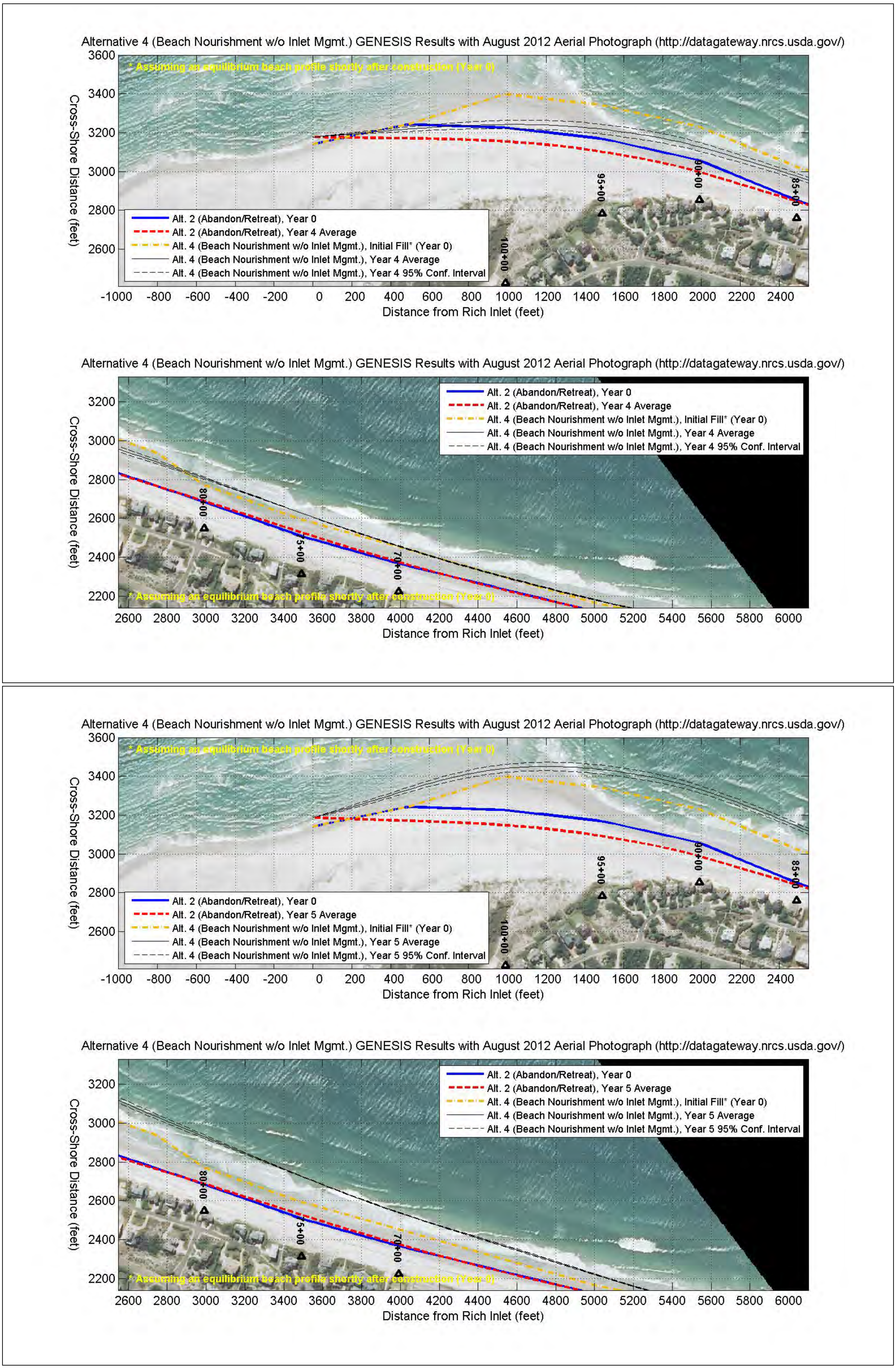


FIGURE 12-12: GENESIS Year 4 & 5 Conditions Given Alternative 4 under 2012 Conditions.

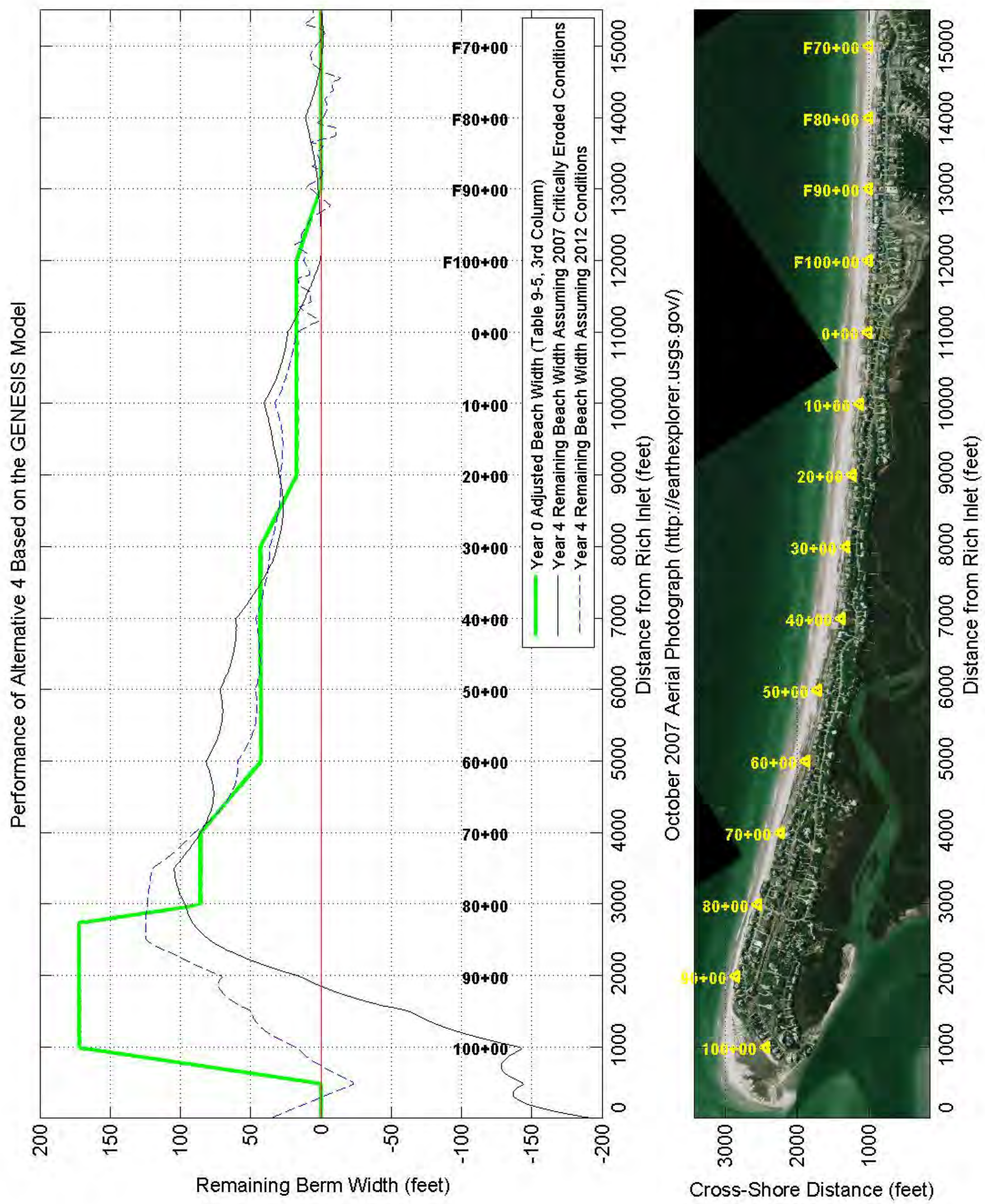


FIGURE 12-13: Remaining Beach Width Given Alternative 4 Based on the GENESIS Model.

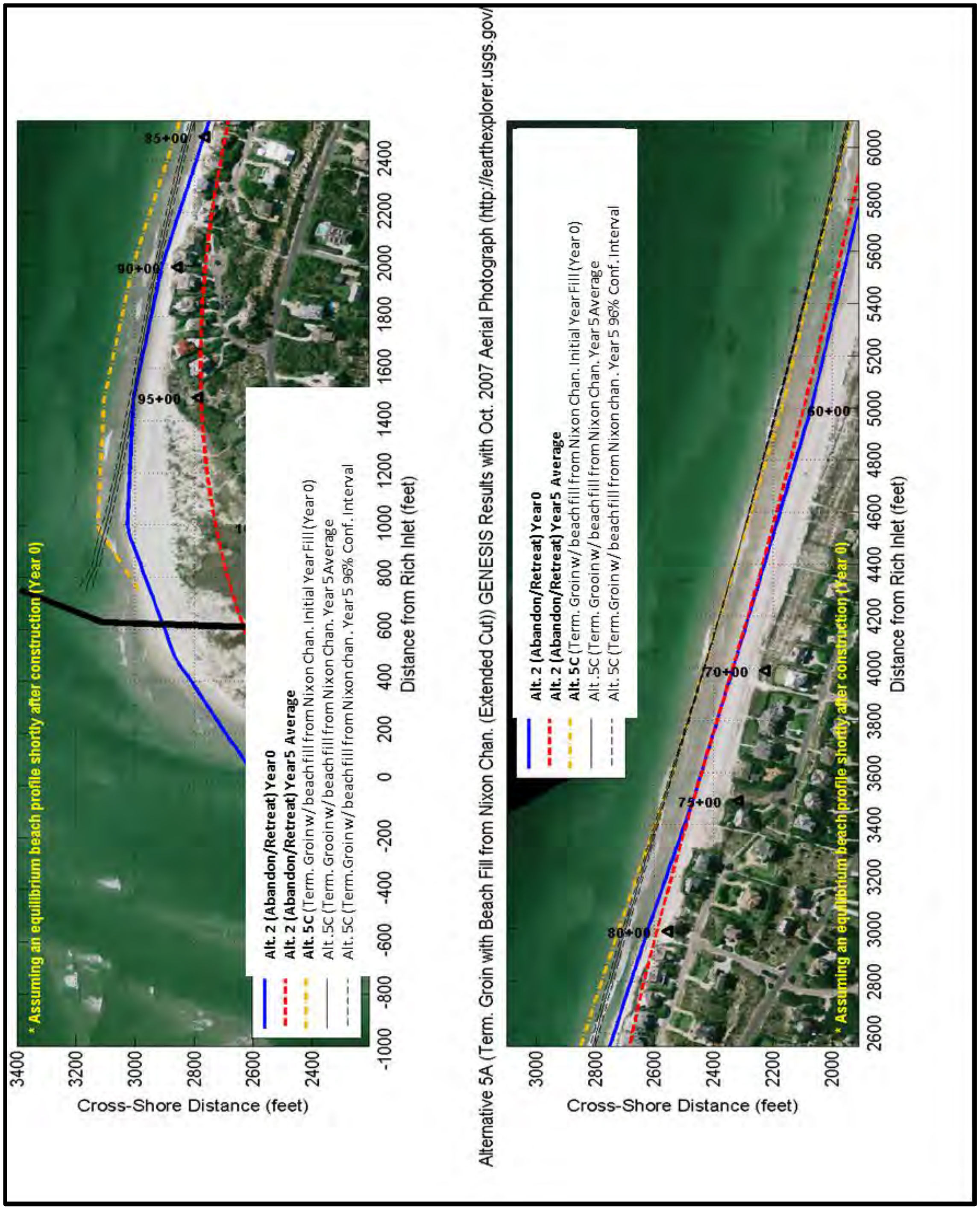


FIGURE 12-14: GENESIS Year 5 Conditions Given Alternative 5C under April 2007 Critically Eroded Conditions.

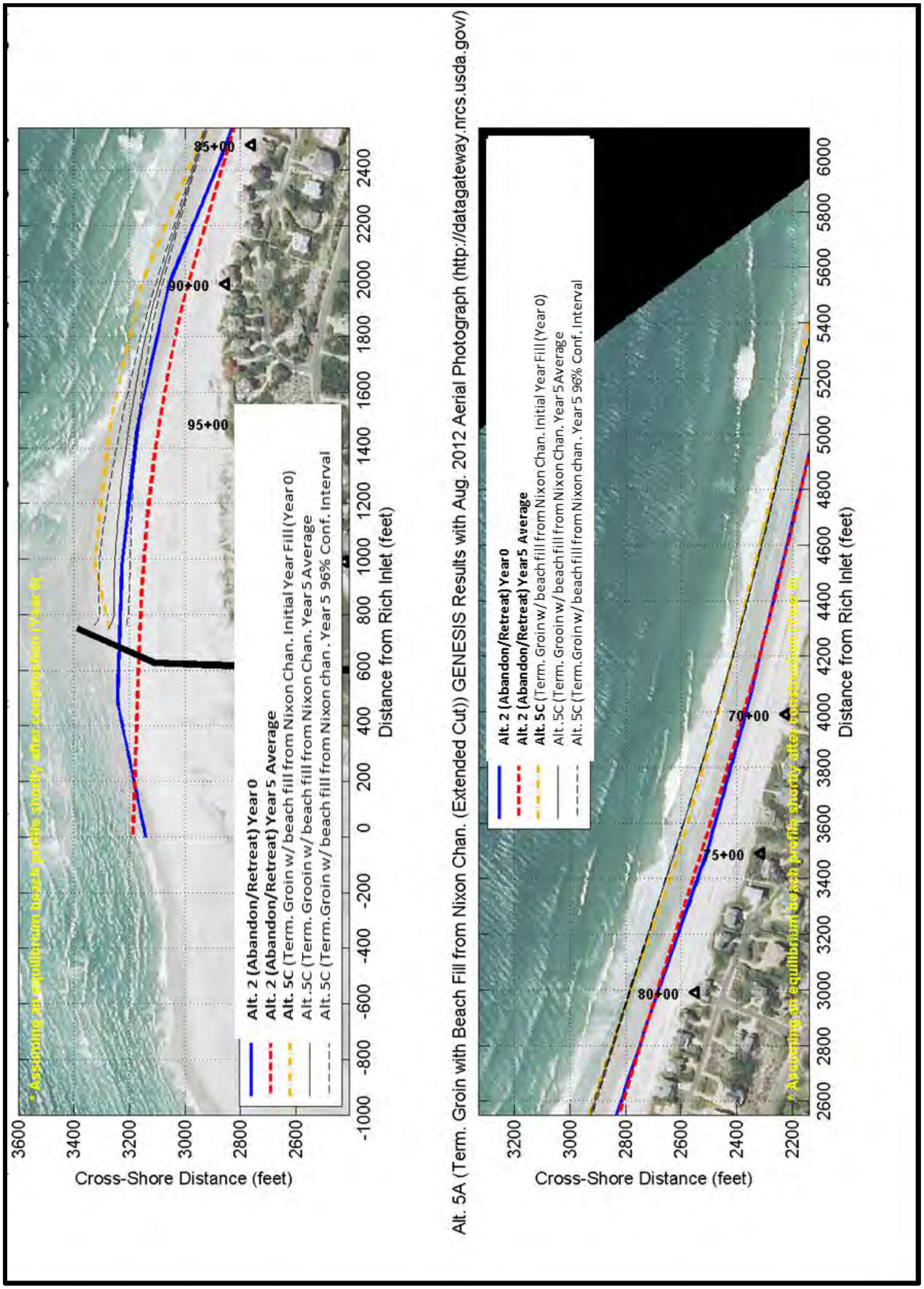


FIGURE 12-15: GENESIS Year 5 Conditions Given Alternative 5C under 2012 Conditions.

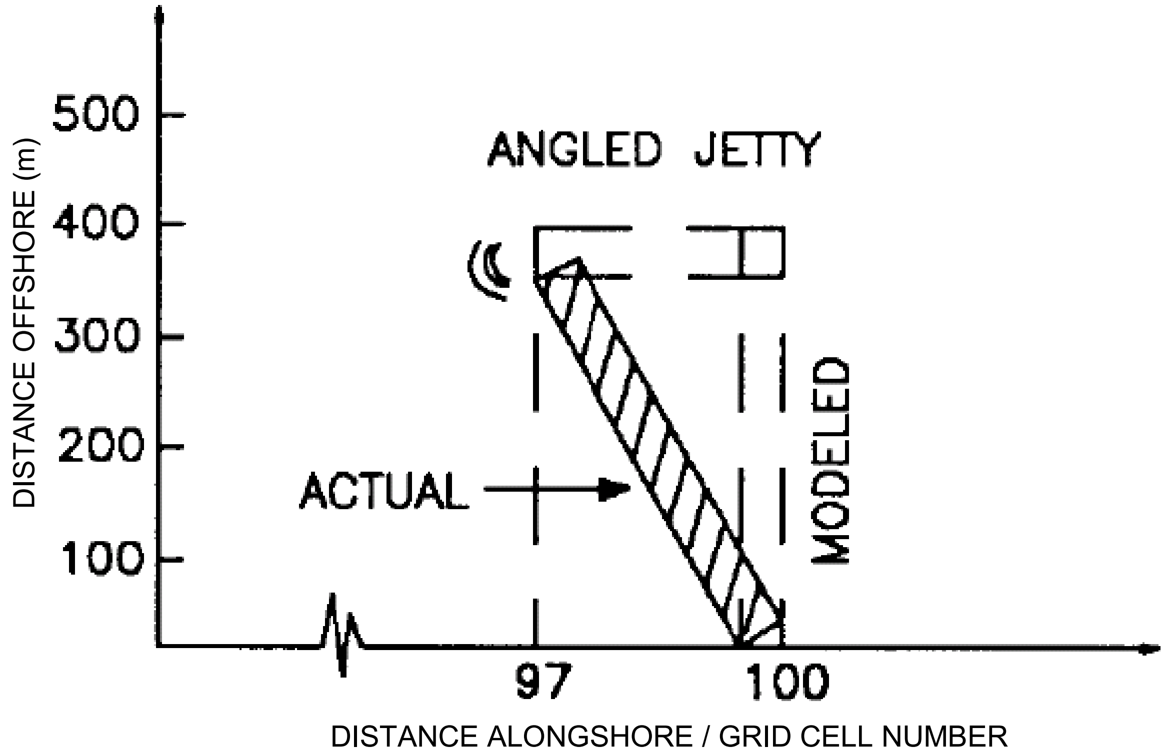


FIGURE 12-16: Recommended Representation of an Angled Groin or Jetty in the GENESIS Model (Hanson and Kraus, 1991, Figure 34, page 144).

GENESIS results for Alternative 5C appear in Figures 12-14, 12-15, and 12-17. Given eroded conditions similar to those in 2007, the GENESIS model suggests that the initial beach fill and the terminal structure will be able to prevent erosion into the pre-construction shoreline (see Figures 12-14 and 12-17). This result is more optimistic that the Delft3D model, which suggests that erosion into the pre-construction shoreline could occur by Year 5 at some locations (see Figure 11-64, Figure 11-66, and Table 11-7). However, both models suggest that Alternative 5C would provide more benefits to the project area than Alternative 3 under a critically eroded scenario, even though the initial fill volume (Table 9-6 versus 9-2) is less.

Given conditions similar to those in 2012, the GENESIS model also suggests that the initial beach fill and the terminal structure will be able to prevent erosion into the pre-construction shoreline (see Figures 12-15 and 12-17). In this case, terminal groin has a smaller effect on beach fill performance than it would under the critically eroded scenario. This is due to the fact that the wider condition of the beach results in a groin that is shorter relative to the initial shoreline.

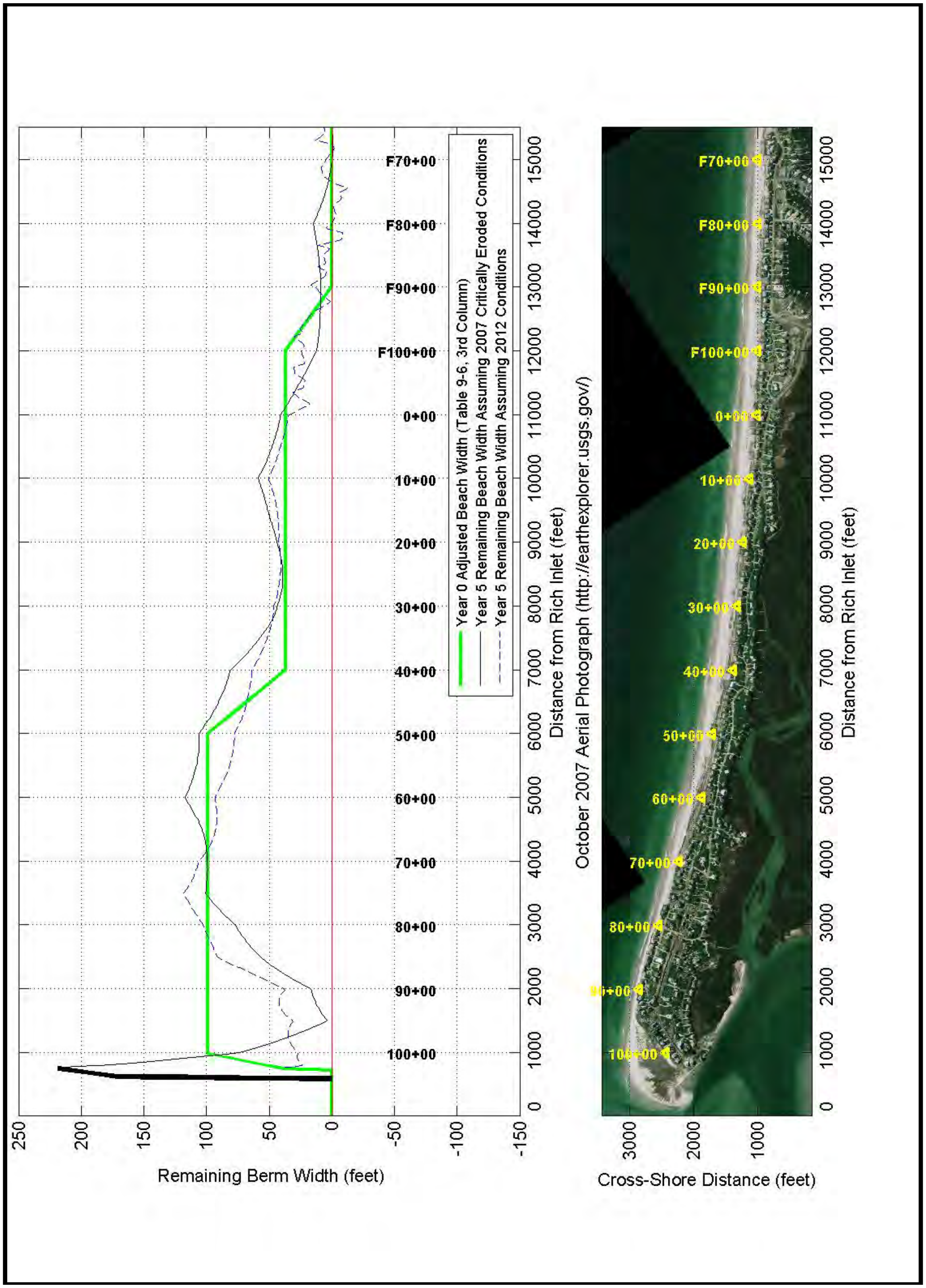


FIGURE 12-17: Remaining Beach Width Given Alternative 5C Based on the GENESIS Model.

12.5.5 Alternatives 5D-1 (1,300-ft terminal groin) and 5D-2 (1,500-ft terminal groin)

The beach fill for Alternatives 5D-1 (1.300-ft terminal groin) and 5D-2 (1,500-ft terminal groin) was incorporated into the GENESIS model by widening the 2007 and 2012 shorelines based on the "adjusted berm width" in Column 3 of Table 9-7. The terminal groins under Alternatives 5D-1 (1,300-ft) and 5D-2 (1,500-ft) were included in the GENESIS model in the same manner as they were for Alternative 5C (see Figure 12-16). For Alternative 5D-1 (1,300-ft), which included the shorter 1,300 foot groin, renourishment of profiles 60+00 to 105+00 was implemented after the end of Year 4. For Alternative 5D-2 (1,500-ft), which included the longer 1,500 foot groin, renourishment of profiles 60+00 to 105+00 was implemented after the end of Year 5. GENESIS model results for Alternatives 5D-1 (1,300-ft) and 5D-2 (1,500-ft appear in Figures 12-18 through 12-23.

In general, the GENESIS model suggests that given either alternative, there will be a limited amount of erosion into the pre-construction shoreline (see Figures 12-20 and 12-23). This is the case for both the April 2007 critically eroded scenarios and the 2012 scenarios. Under the 2012 scenarios, erosion into the pre-construction does not pose a risk to upland development (see Figures 12-19 and 12-22). Under the April 2007 critically eroded scenarios, there are 4 oceanfront homes near the south end of Inlet Hook Road (profile 90+00) that could be at risk of erosion-related damage at Year 4 or 5 (see Figures 12-18 and 12-21). However, the additional results in Sub-Appendix C suggest after the first renourishment operation, the erosion into the pre-construction shoreline over the remainder of the 10 year study period is unlikely.

A direct comparison of Alternatives 5D-1 (1,300-ft) and 5D-2 (1,500ft) appears in Figure 12-20, which shows the performance of the beach fill through Year 4. Under the 2012 scenarios, the GENESIS model suggests that Alternative 5D-2 (1,500-ft), which includes the longer 1,500 foot groin, performs slightly better than Alternative 5D-1 (1,300-ft). Under the April critically eroded scenarios, the GENESIS model suggests that Alternative 5D-1 (1,300-ft), which includes the shorter 1,300 foot groin, performs slightly better than Alternative 5D-2 (1,500-ft). Under either set of scenarios, the differences between two alternatives fall within the uncertainty ranges shown in Figures 12-18, 12-19, 12-21, and 12-22, suggesting that neither alternative is better than the other in terms of beach fill performance. This finding is somewhat contrary to the Delft3D results, which suggest that Alternative 5D-2 (1,500-ft) retains more fill on the beach (see Table 11-7).

The difference between the two models is likely due to the limitations of the GENESIS model versus the Delft3D model. The Delft3D model includes the effects of waves, tidal currents, longshore transport, cross-shore transport, and changes in the offshore bathymetry. The GENESIS model assumes that shoreline and volume changes occur due to longshore currents driven primarily by waves, and that the offshore bathymetry does not change significantly over time. Given these considerations, the Delft3D model results, which suggest that Alternative 5D-2 (1,500-ft) retains more fill on the beach (see Table 11-7), should be given more weight than the GENESIS results.

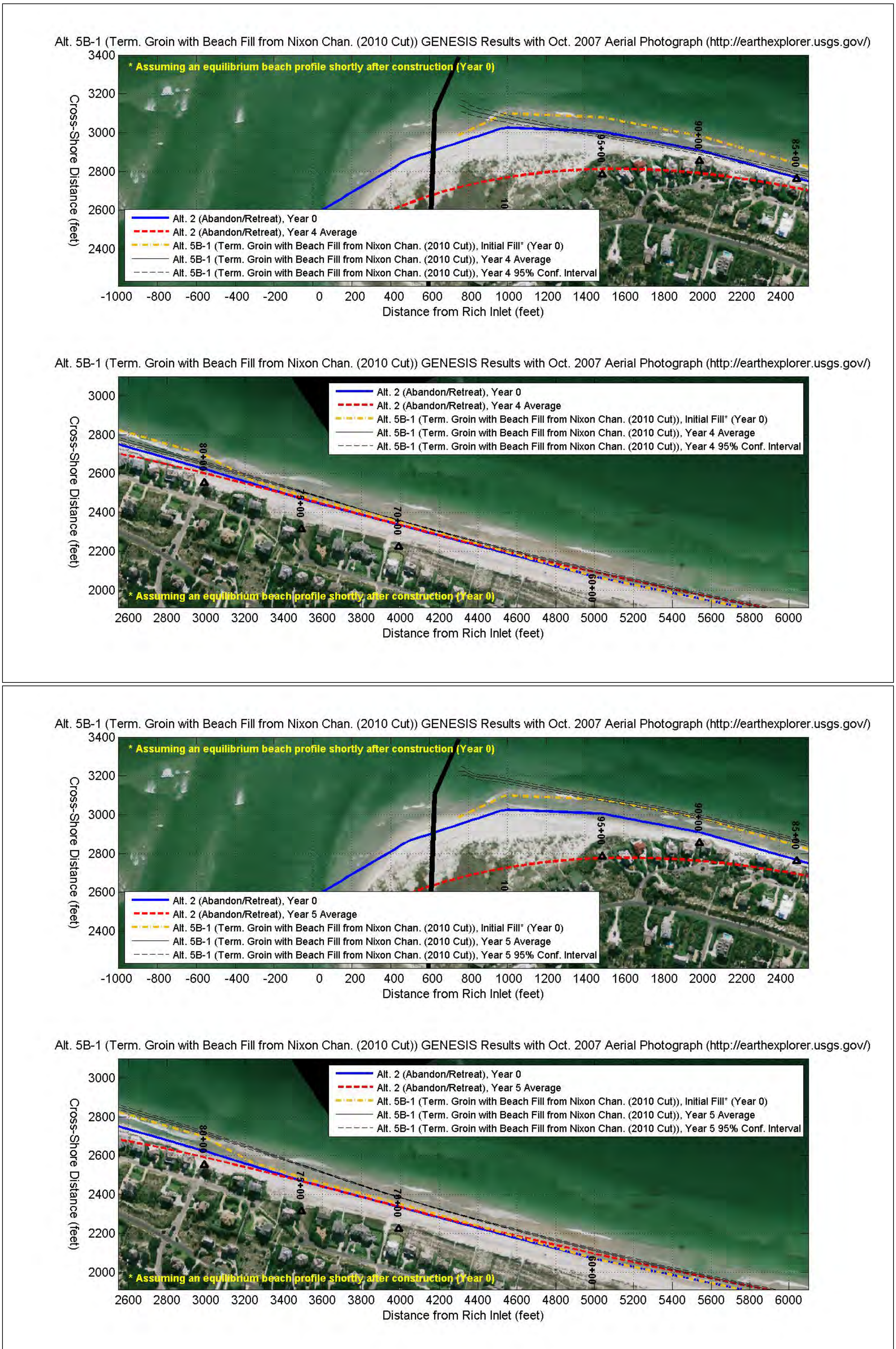
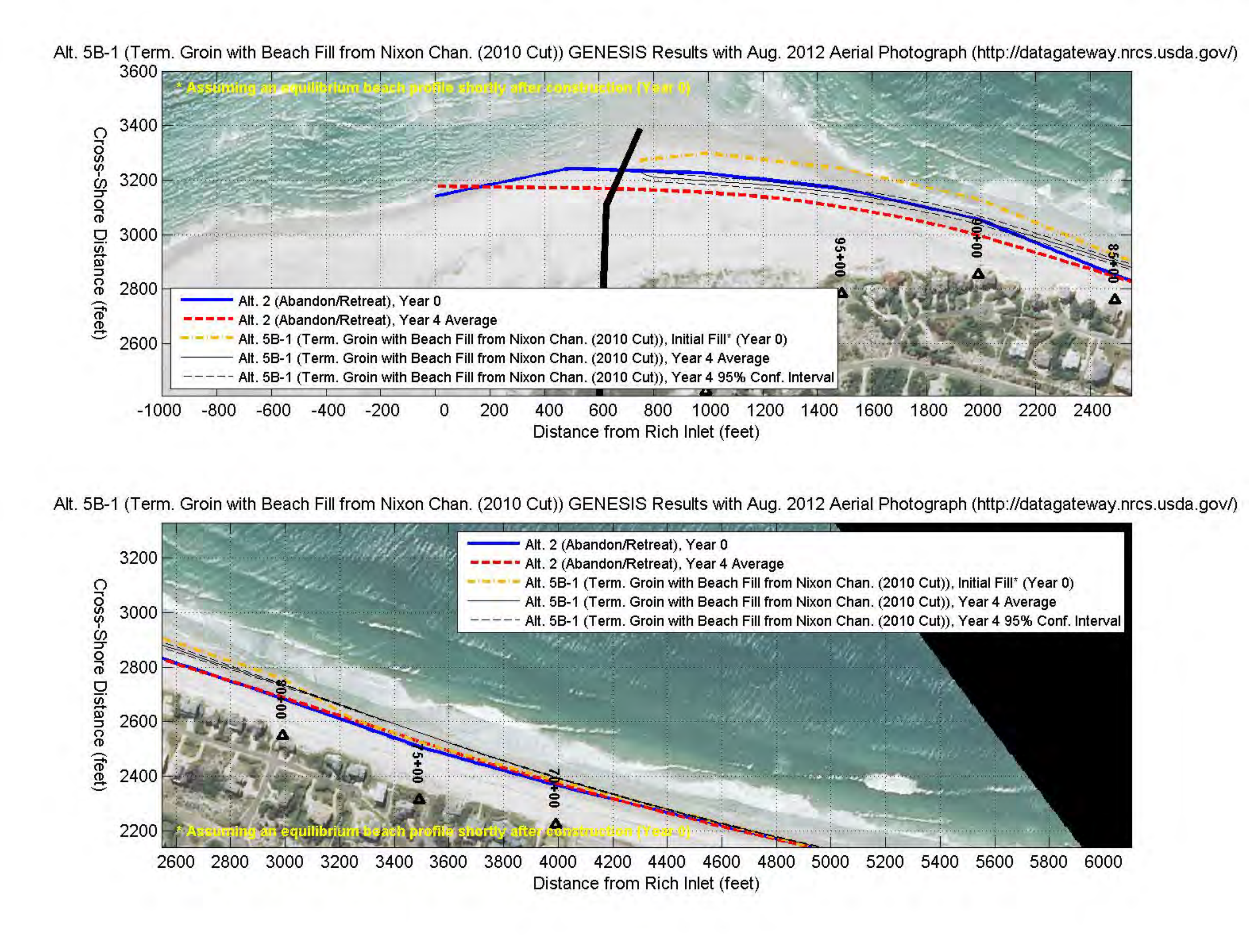


FIGURE 12-18: GENESIS Year 4 & 5 Conditions Given Alternative 5D-1 (1,300 ft) under April 2007 Critically Eroded Conditions.



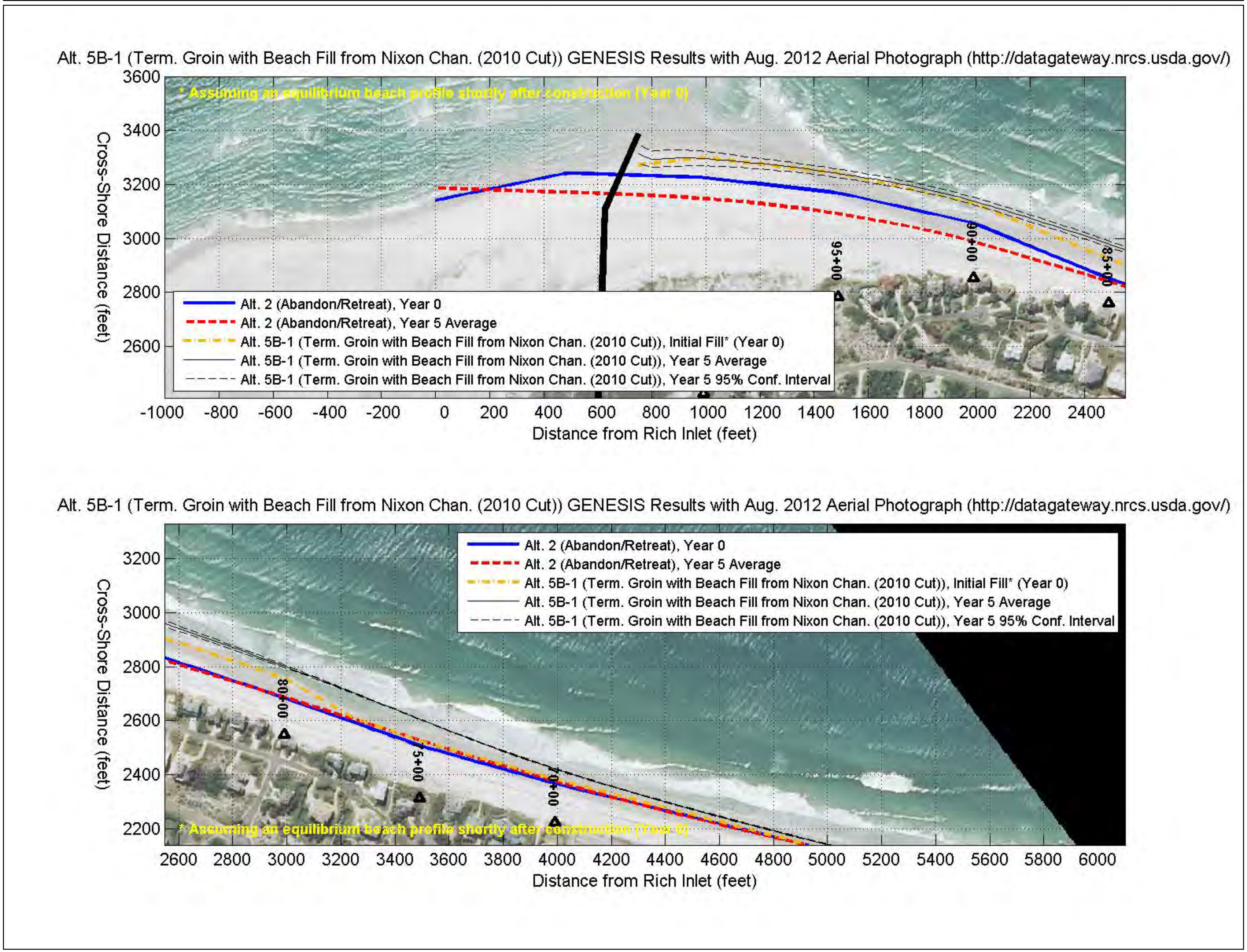


FIGURE 12-19: GENESIS Year 4 & 5 Conditions Given Alternative 5D-1 (1,300-ft) under 2012 Conditions.

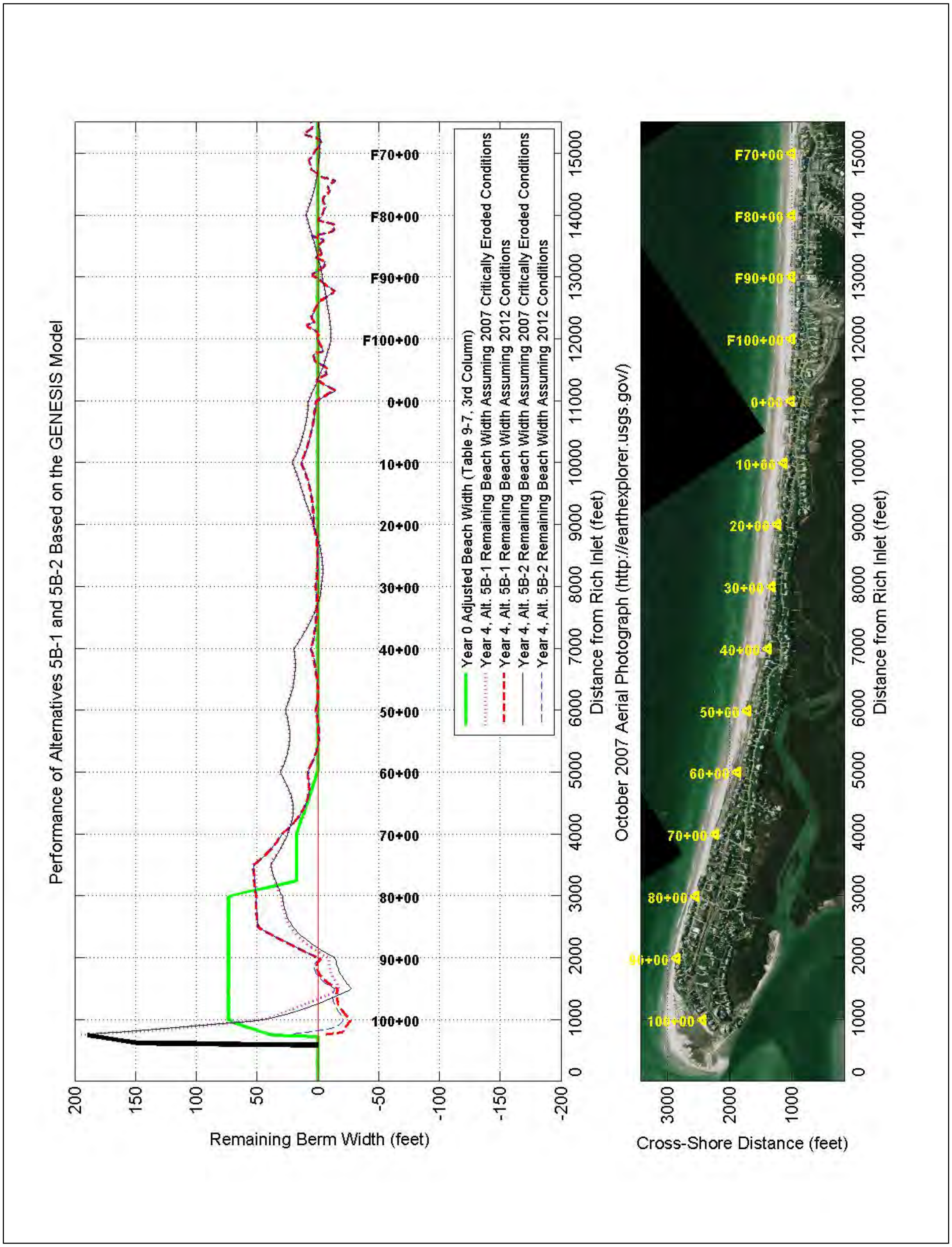


FIGURE 12-20: Remaining Beach Width at Year 4 Given Alternatives 5D-1 (1,300-ft) and 5D-2 (1,500-ft) Based on the GENESIS Model.

FIGURE 12-21: GENESIS Year 5 Conditions Given Alternative 5D-2 (1,500-ft) under April 2007 Critically Eroded Conditions.

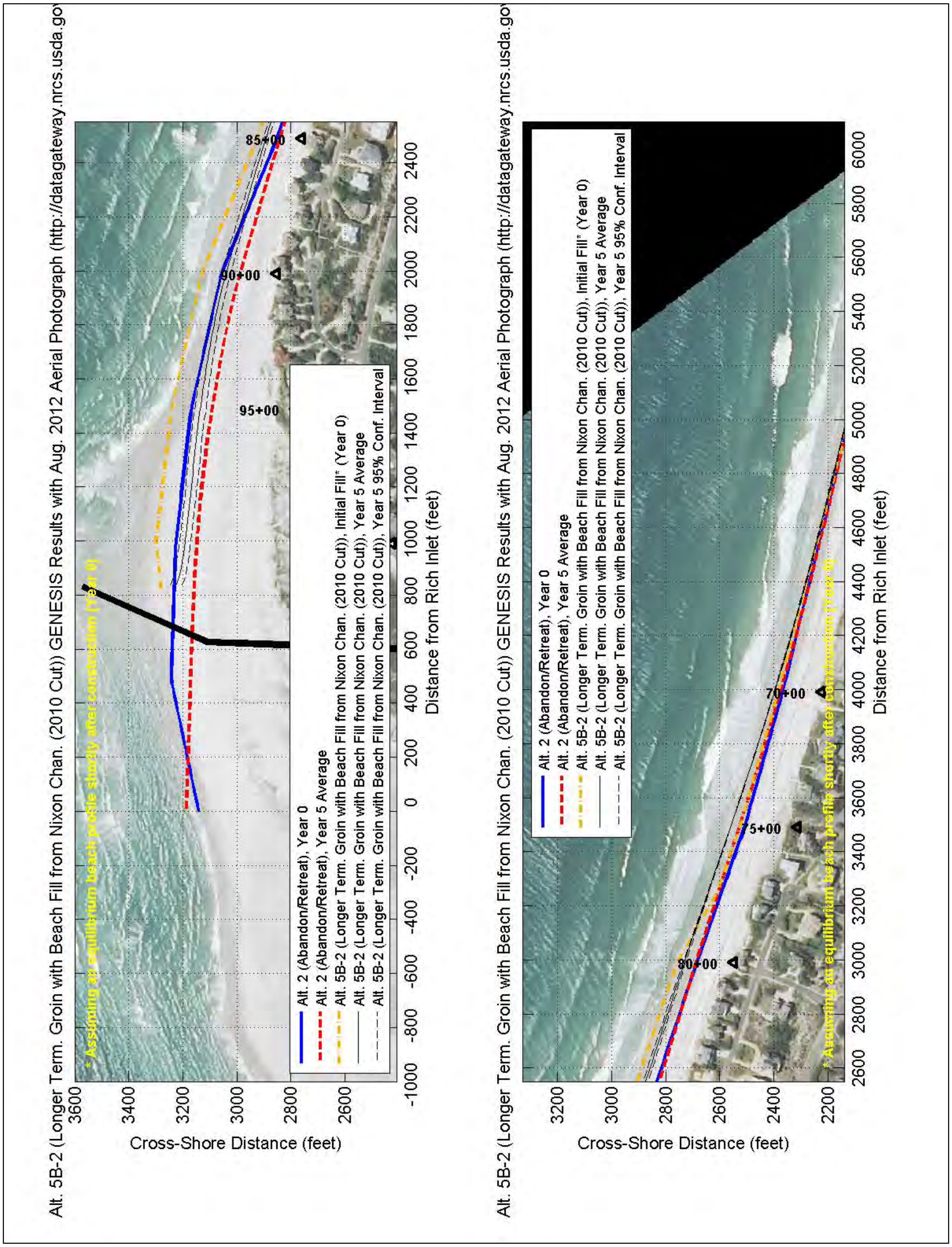


FIGURE 12-22: GENESIS Year 5 Conditions Given Alternative 5D-2 (1,500-ft) under 2012 Conditions.

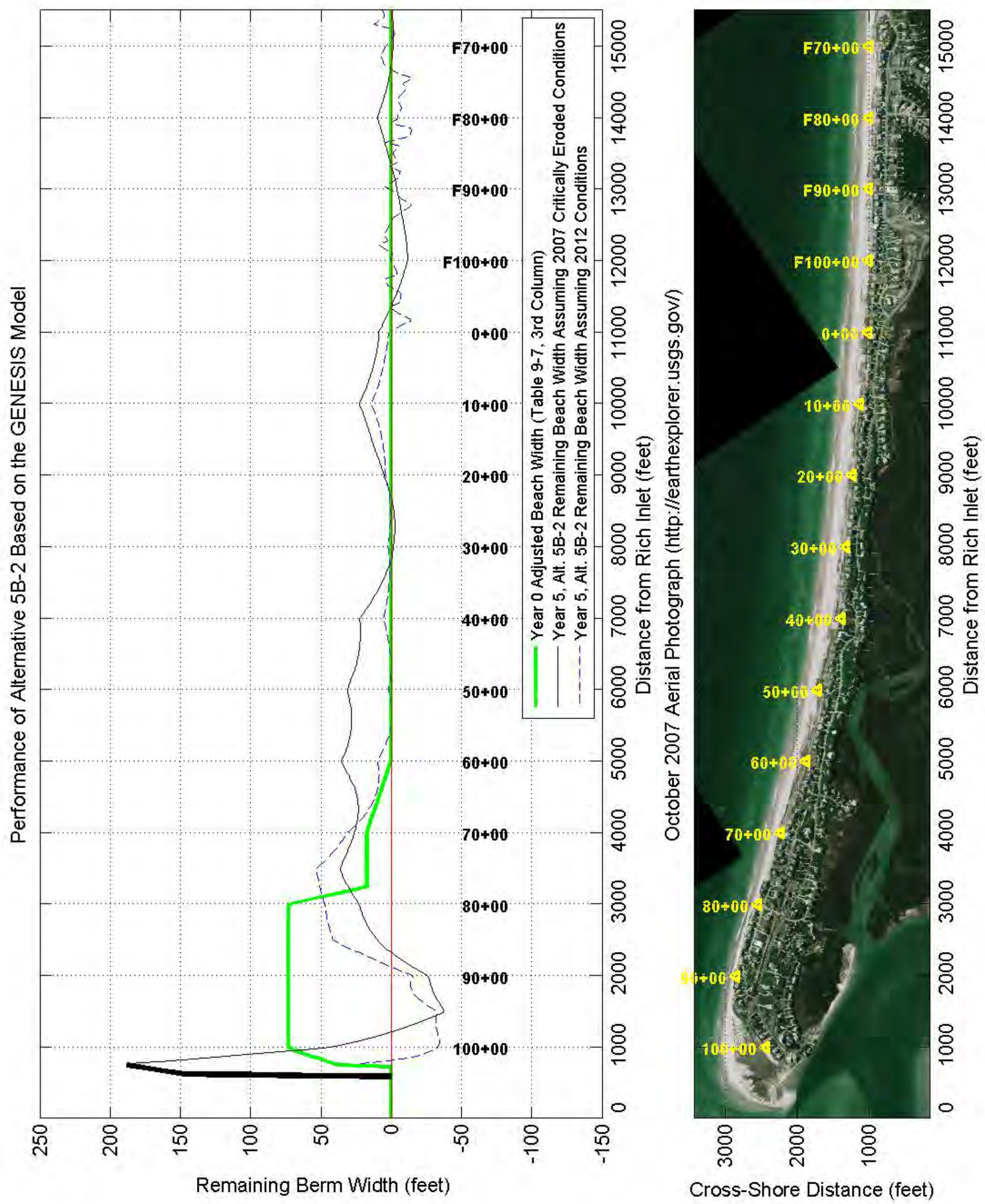


FIGURE 12-23: Remaining Beach Width at Year 5 Given Alternative 5D-2 (1,500-ft) Based on the GENESIS Model.

12.5.5 Summary

While 5-year predictions of the GENESIS and Delft3D-FLOW models differ in their details, they both suggest similar trends in the performance of Alternatives 2, 3, and 5D. The general findings of one model generally support the other. Recommendations based on the model results and the historical erosion analysis in Sections 6 and 7 appear in the final conclusions and recommendations of this report.

13.0 COST ESTIMATES

The following tables provide opinions on costs for Alternatives 3, 4, 5C, and 5D. Costs are provided for both the 2006 and 2012 conditions of Rich Inlet and Figure Eight Island.

Table 13-1a
Cost Estimate – Alternative 3
Rich Inlet Management with Beach Fill
2006 Conditions

First Cost				
Item	Unit	Quantity	Unit Cost	Cost
Beach fill from Green and Inlet Channel				
Mobilization and Demobilization	LS	1	$2,806,000	$2,806,000
Dredging (Beach Fill)	CY	1,462,900	$7.03	$10,279,000
Sub-Total (Beach Fill)				$13,085,000
Construct Dike – Upland Disposal of Clay				
Additional Mob & Demob – Pipe	LS	1	$230,000	$230,000
Modify Upland Disposal Site	Job	1	$288,000	$288,000
Dredging – Dike & Upland Disposal	CY	460,800	$7.03	$3,271,000
Sub-Total Dike & Upland Disposal				$3,789,000
Dune Vegetation	LF	1,250	$2.30	$3,000
Total Construction Cost				$16,843,000
Engineering & Design (P&S)				$150,000
Construction Oversight				$120,000
Total First Cost				**$17,113,000**
Periodic Channel Maintenance and Beach Nourishment (Every 5 years)				
Mobilization and Demobilization	LS	1	$2,806,000	$2,806,000
Dredging Entrance Channel & Beach Fill	CY	666,000	$7.03	$4,679,000
Sub-Total				$7,485,000
Engineering & Design (P&S)				$100,000
Construction Oversight				$120,000
Total Periodic Dredging Cost				**$7,705,000**

Table 13-1b
Cost Estimate – Alternative 3
Rich Inlet Management with Beach Fill
2012 Conditions

First Cost				
Item	Unit	Quantity	Unit Cost	Cost
Beach fill from Green and Inlet Channel				
Mobilization and Demobilization	LS	1	$2,806,000	$2,806,000
Dredging (Beach Fill)	CY	1,477,500	$7.03	$10,382,000
Sub-Total (Beach Fill)				$13,188,000
Construct Dike – Upland Disposal of Clay				
Additional Mob & Demob – Pipe	LS	1	$230,000	$230,000
Modify Upland Disposal Site	Job	1	$288,000	$288,000
Dredging – Dike & Upland Disposal	CY	465,400	$7.03	$3,271,000
Sub-Total Dike & Upland Disposal				$3,789,000
Dune Vegetation	LF	1,250	$2.30	$3,000
Total Construction Cost				$15,048,000
Engineering & Design (P&S)				$150,000
Construction Oversight				$120,000
Total First Cost				**$17,250,000**
Periodic Channel Maintenance and Beach Nourishment (Every 5 years)				
Mobilization and Demobilization	LS	1	$2,806,000	$2,806,000
Dredging Entrance Channel & Beach Fill	CY	666,000	$7.03	$4,679,000
Sub-Total				$7,485,000
Engineering & Design (P&S)				$100,000
Construction Oversight				$120,000
Total Periodic Dredging Cost				**$7,705,000**

Table 13-2a
Cost Estimate – Alternative 4
Beach Nourishment without Inlet Management
2006 Conditions

First Cost				
Item	Unit	Quantity	Unit Cost	Cost
Hopper Dredge – Offshore Borrow Areas				
Mobilization and Demobilization	LS	1	$2,436,000	$2,436,000
Dredging (Beach Fill)	CY	521,300	$13.30	$6,656,000
Sub-Total (Offshore Borrow Areas)				$9,092,000
18-inch Pipeline Dredge – Nixon Channel				
Mobilization and Demobilization	LS	1	$558,000	$558,000
Dredging – Nixon Channel	CY	400,000	$6.80	$2,719,000
Sub-Total Nixon Channel				$3,277,000
Dune Vegetation	LF	1,250	$2.30	$3,000
Total Construction Cost				$12,372,000
Engineering & Design (P&S)				$150,000
Construction Oversight				$170,000
Total First Cost				**$13,692,000**
Periodic Channel Maintenance and Beach Nourishment (Every 4 years)				
Hopper Dredge – Offshore Borrow Areas				
Mobilization and Demobilization	LS	1	$2,436,000	$2,436,000
Dredging (Beach Fill)	CY	328,000	$12.77	$4,188,000
Sub-Total (Offshore Borrow Areas)				$6,624,000
18-inch Pipeline Dredge – Nixon Channel				
Mobilization and Demobilization	LS	1	$558,000	$558,000
Dredging – Nixon Channel	CY	400,000	$6.80	$2,719,000
Sub-Total Nixon Channel				$3,277,000
Total Construction Cost				$9,901,000
Engineering & Design (P&S)				$100,000
Construction Oversight				$170,000
Total 4-year Nourishment Cost				**$10,171,000**

Table 13-2b
Cost Estimate – Alternative 4
Beach Nourishment without Inlet Management
2012 Conditions

First Cost				
Item	Unit	Quantity	Unit Cost	Cost
Hopper Dredge – Offshore Borrow Areas				
Mobilization and Demobilization	LS	1	$2,436,000	$2,436,000
Dredging (Beach Fill)	CY	568,300	$12.77	$7,256,000
Sub-Total (Offshore Borrow Areas)				$9,692,000
18-inch Pipeline Dredge – Nixon Channel				
Mobilization and Demobilization	LS	1	$558,000	$558,000
Dredging – Nixon Channel	CY	400,000	$6.80	$2,719,000
Sub-Total Nixon Channel				$3,277,000
Dune Vegetation	LF	1,250	$2.30	$3,000
Total Construction Cost				$11,951,000
Engineering & Design (P&S)				$150,000
Construction Oversight				$170,000
Total First Cost				**$14,292,000**
Periodic Channel Maintenance and Beach Nourishment (Every 4 years)				
Hopper Dredge – Offshore Borrow Areas				
Mobilization and Demobilization	LS	1	$2,436,000	$2,436,000
Dredging (Beach Fill)	CY	388,000	$12.77	$4,954,000
Sub-Total (Offshore Borrow Areas)				$7,390,000
18-inch Pipeline Dredge – Nixon Channel				
Mobilization and Demobilization	LS	1	$558,000	$558,000
Dredging – Nixon Channel	CY	400,000	$6.80	$2,719,000
Sub-Total Nixon Channel				$3,277,000
Total Construction Cost				$10,667,000
Engineering & Design (P&S)				$100,000
Construction Oversight				$170,000
Total 4-year Nourishment Cost				**$7,821,000**

Table 13-3a
Cost Estimate – Alternative 5C
Terminal Groin with Beach Fill from Maintenance of the Nixon Channel Navigation Channel and Connector Channel
2006 Conditions

First Cost				
Item	Unit	Quantity	Unit Cost	Cost
18-inch Pipeline – Nixon Channel & Beach Fill				
Mobilization and Demobilization	LS	1	$1,156,000	$1,156,000
Dredging (Channel & Beach Fill)	CY	994,400	$7.65	$7,605,000
Dune Vegetation	LF	1,250	$2.30	$3,000
Sub-Total (Channel & Beach Fill)				$9,396,000
Engineering & Design (P&S)				$100,000
Construction Oversight				$120,000
Total First Cost Channel & Beach Fill				**$8,984,000**
Terminal Groin				
Groin Construction	LF	1,300	$2,300	$2,990,000
Engineering & Design (P&S)				$200,000
Construction Oversight				$220,000
Total First Cost Terminal Groin				**$3,410,000**
Total First Cost Alternative 5C				**$12,394,000**
Periodic Channel Maintenance & Beach Nourishment (Every 5 years)				
Mobilization and Demobilization	LS	1	$1,156,000	$1,156,000
Dredging	CY	495,000	$7.65	$3,786,000
Total Periodic Dredging Cost				$4,942,000
Engineering & Design (P&S)				$100,000
Construction Oversight				$120,000
Total Periodic Cost (every 5 years)				**$5,162,000**

Table 13-3b
Cost Estimate – Alternative 5C
Terminal Groin with Beach Fill from Maintenance of the Nixon Channel Navigation Channel and Connector Channel
2012 Conditions

First Cost				
Item	Unit	Quantity	Unit Cost	Cost
18-inch Pipeline – Nixon Channel & Beach Fill				
Mobilization and Demobilization	LS	1	$1,156,000	$1,156,000
Dredging (Channel & Beach Fill)	CY	1,077,000	$7.65	$8,237,000
Dune Vegetation	LF	1,250	$2.30	$3,000
Sub-Total (Channel & Beach Fill)				$9,396,000
Engineering & Design (P&S)				$100,000
Construction Oversight				$120,000
Total First Cost Channel & Beach Fill				**$9,616,000**
Terminal Groin				
Groin Construction	LF	1,300	$2,300	$2,990,000
Engineering & Design (P&S)				$200,000
Construction Oversight				$220,000
Total First Cost Terminal Groin				**$3,410,000**
Total First Cost Alternative 5C				**$13,026,000**
Periodic Channel Maintenance & Beach Nourishment (Every 5 years)				
Mobilization and Demobilization	LS	1	$1,156,000	$1,156,000
Dredging	CY	495,000	$7.65	$3,786,000
Total Periodic Dredging Cost				$4,942,000
Engineering & Design (P&S)				$100,000
Construction Oversight				$120,000
Total Periodic Cost (every 5 years)				**$5,162,000**

Table 13-4a
Cost Estimate – Alternative 5D
Terminal Groin with Beach Fill From Other Sources
2006 Conditions

First Cost				
Item	Unit	Quantity	Unit Cost	Cost
18-inch Pipeline Dredge – Nixon Channel				
Mobilization and Demobilization	LS	1	$558,000	$558,000
Dredging – Nixon Channel	CY	294,500	$6.80	$2,001,000
Sub-Total Nixon Channel				$2,559,000
Engineering & Design (P&S)				$150,000
Construction Oversight				$170,000
Total Construction Beach Fill & Dune				**$2,879,000**
Terminal Groin				
Groin Construction	LF	1,500	$2,760	$4,140,000
Engineering & Design (P&S)				$200,000
Construction Oversight				$220,000
Total First Cost Terminal Groin				**$4,560,000**
Total First Cost Alternative 5D				**$7,439,000**
Periodic Channel Maintenance & Beach Nourishment (Every 5 years)				
Mobilization and Demobilization	LS	1	$558,000	$558,000
Dredging	CY	320,000	$6.80	$2,175,000
Total Periodic Dredging Cost				$2,733,000
Engineering & Design (P&S)				$100,000
Construction Oversight				$170,000
Total Periodic Cost (every 5 years)				**$3,003,000**

Table 13-4b
Cost Estimate – Alternative 5D
Terminal Groin with Beach Fill From Other Sources
2012 Conditions

First Cost				
Item	Unit	Quantity	Unit Cost	Cost
18-inch Pipeline Dredge – Nixon Channel				
Mobilization and Demobilization	LS	1	$558,000	$558,000
Dredging – Nixon Channel	CY	294,500	$6.80	$2,001,000
Sub-Total Nixon Channel				$2,559,000
Engineering & Design (P&S)				$150,000
Construction Oversight				$170,000
Total Construction Beach Fill & Dune				**$2,879,000**
Terminal Groin				
Groin Construction	LF	1,500	$2,760	$4,140,000
Engineering & Design (P&S)				$200,000
Construction Oversight				$220,000
Total First Cost Terminal Groin				**$4,560,000**
Total First Cost Alternative 5D				**$7,439,000**
Periodic Channel Maintenance & Beach Nourishment (Every 5 years)				
Mobilization and Demobilization	LS	1	$558,000	$558,000
Dredging	CY	255,000	$6.80	$1,733,000
Total Periodic Dredging Cost				$2,291,000
Engineering & Design (P&S)				$100,000
Construction Oversight				$170,000
Total Periodic Cost (every 5 years)				**$2,561,000**

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