DADE COUNTY, FLORIDA

BEACH EROSION CONTROL
& HURRICANE PROTECTION PROJECT

LIMITED REEVALUATION REPORT
ENGINEERING APPENDIX A
January 2015

US Army Corps
of Engineers
Jacksonville District

U.S. ARMY CORPS OF ENGINEERS
JACKSONVILLE DISTRICT

DADE COUNTY, FLORIDA
BEACH EROSION CONTROL AND HURRICANE PROTECTION PROJECT
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INTRODUCTION

This Appendix contains the engineering support and documentation that is summarized in the main section of this Limited Reevaluation Report (LRR). Included in this appendix are sections discussing physical data and processes throughout the project area. Separate, more detailed sub-appendices provide volumetric projections for the Dade County Beach Erosion Control and Hurricane Protection (BEC & HP) Project throughout the remaining period of Federal participation, and also provide the results of the wave refraction analysis that was conducted to determine the nearshore effects of the proposed offshore borrow areas.

PHYSICAL DATA

General.

The physical data presented in this section includes water levels, wind and wave measurements and hindcasts, and a discussion of recent storms that have affected the project area. Tidal datums in this report are provided by NOAA and predicted storm surge elevations are provided by FEMA. Sea level rise rates were based on Corps of Engineers guidance (Engineering Regulation 1100-2-8162). Wave data used in the wave refraction analysis was derived from the Corps of Engineers' Wave Information Study, an ongoing research effort by the Corps' Engineering Research and Development Center (ERDC).

Water Levels.

General. Changes in water levels along the Miami-Dade County shoreline occur primarily as a result of three separate processes: astronomical tides, storm surges, and long-term sea level rise. Astronomical tides affect the area on a daily basis, while the effects of significant storm surges are much less frequent. The effects of sea-level rise during the remaining years of Federal participation in the Dade County BEC & HP Project are much more gradual and smaller in magnitude than the effects of tides or storm surges, and will be examined separately.

The effects of water levels on the Federal project are important because in addition to the obvious flooding impacts associated with high water levels, waves will generally break closer to shore, causing greater damage; and tidal current velocities through inlets (and the resulting scouring and/or shoaling) are greater when the tidal range is at its highest. The most severe conditions a beach renourishment project will usually be subjected to will be a combination of astronomical high tide coupled with storm surge, while also being subjected to storm wave attack. These conditions occur simultaneously on a fairly frequent basis during the passage of tropical storms and winter 'northeasters'.

Although the most noticeable effects on water levels in the project area are due to astronomical tides and storm surges, the effects of long-term sea level rise cannot be ignored due to its implications on the long-term management of the project. Each of these three processes affecting ocean water levels will be discussed in the following sections.

Astronomical Tides. Astronomical tides are created by the gravitational pull of the moon and sun, and these tides are predictable in magnitude and timing. The National Oceanic and Atmospheric Administration (NOAA) regularly publishes tide tables for selected locations along the coastlines of the United States and around the world. These tables provide the times of high and low tides, as well as predicted tidal amplitudes.

Tides in the Miami-Dade County study area are semidiurnal, meaning that two high tides and two low tides occur during each 24-hour period. Two measures of tidal range are commonly used: the mean tide range and the spring tide range. The mean tide range is defined as the difference between mean high water and mean low water, and represents an average range during the entire monthly lunar cycle. The spring tide range is the average semidiurnal range that occurs semimonthly when the moon is new or full. The range of tidal elevations between successive high and low tides is typically greater at any location during periods of a new or full moon. Both tide ranges are relatively low along the Miami-Dade County Atlantic shoreline - the mean tide range is 2.02 feet and the spring tide range is 2.19 feet.

Elevations of tidal datums in the vicinity of the project area are provided in Table 1. All datum elevations in this table are referenced to mean lower low water (MLLW). These datums are based on values from an active tide station located on Virginia Key, at the University of Miami's Rosenstiel School of Marine and Atmospheric Science. This gage is located near the south end of the project area and was established by the National Oceanic and Atmospheric Administration (NOAA), National Ocean Service (NOS) in January 1994.

Table 1
Tidal Datums – NOAA Station #8723214

Highest Observed Water Level (24 Oct 2005)	=	4.76 Feet
Mean Higher High Water (MHHW)	=	2.19 Feet
Mean High Water (MHW)	=	2.13 Feet
North American Vertical Datum 1988 (NAVD88)	=	1.97 Feet
Mean Tide Level (MTL)	=	1.12 Feet
Mean Sea Level (MSL)	=	1.10 Feet
Mean Low Water (MLW)	=	0.11 Feet
Mean Lower Low Water (MLLW)	=	0.00 Feet
Lowest Observed Water Level (29 Mar 94)	=	-1.31 Feet

Storm Surges. Storm surge is defined as the rise of the ocean surface above the normal astronomical tide level due to storm effects. Strong onshore winds pile up water near the shoreline, resulting in superelevated water levels along the coastal region and inland waterways. In addition, the lower atmospheric pressure that accompanies storms also contributes to a rise in water surface elevation. Extremely high wind velocities coupled with low barometric pressures (such as those experienced in tropical storms, hurricanes, and very strong northeasters) can produce very high, damaging water levels. For example, a peak surge of up to 10.6 feet MLW was measured along the Key Biscayne shoreline in southern Miami-Dade County during the passage of Hurricane Andrew in 1992. Factoring out the 2.0-foot astronomical high tide resulted in a storm surge of 8.6 feet.

Storm surge levels versus frequency of occurrence were calculated for coastal counties throughout Florida by the Federal Emergency Management Agency (FEMA) as part of that agency's Flood Insurance Study (reference 1). An example of the storm surge elevation vs frequency of occurrence curves for Miami-Dade County are shown in Figure 1 below.

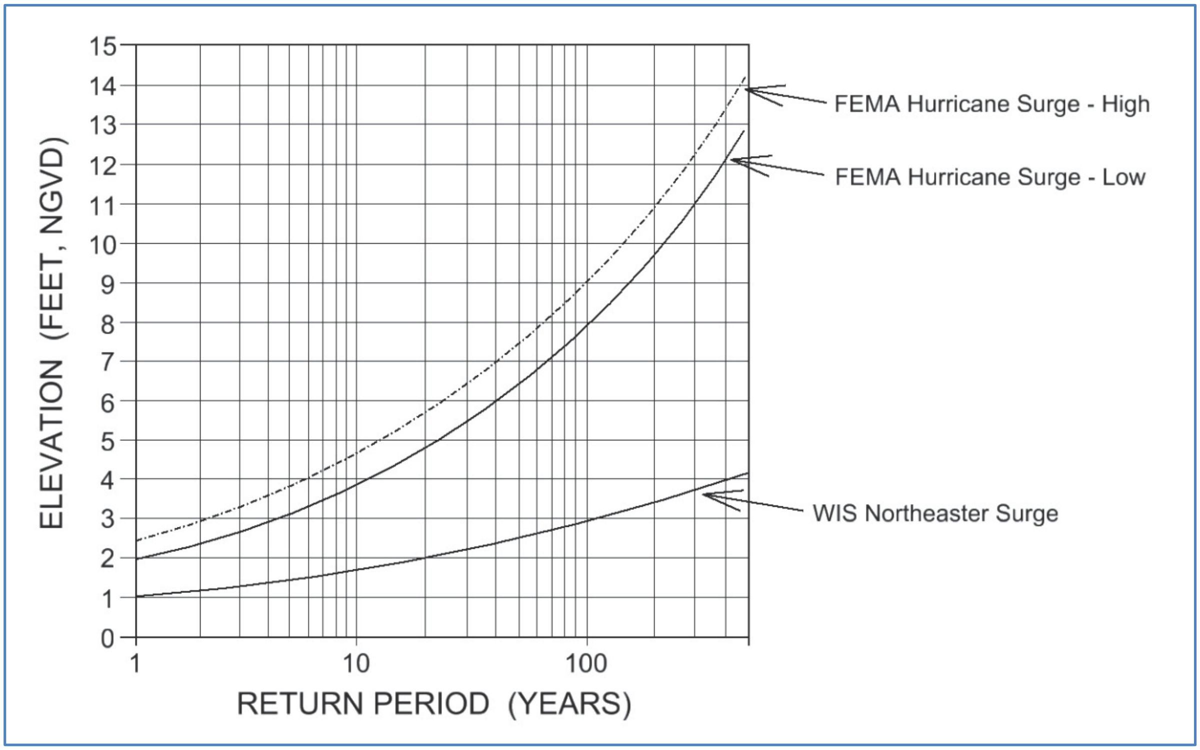

Figure 1. Typical FEMA Storm Surge Frequency Curve, for Miami-Dade County.

Methodology developed by the National Academy of Sciences was used in the development of these series of storm surge curves, which are provided as an example of the magnitude of surge levels that could be expected along the Miami-Dade Atlantic shoreline. Two types of curves are provided in Figure 1, consisting of a pair of hurricane-produced surge levels, and a northeaster-produced surge level. These curves do not include tidal effects, but do include the effects of storm waves, and also include the wave dissipation effects of features such as dunes and vegetation, and coastal structures such as seawalls and buildings.

In this example, the FEMA hurricane surge curves are primarily interpolated from data points for the 10, 50, 100, and 500-year recurrence hurricane events. The 'high' curve was computed for hurricane surges along the southern portion of Miami-Dade County, while the 'low' curve was calculated for surges along the northern portion of the county. The differences between these two curves are mainly due to variations in wave runup levels at the northern and southern ends of the county.

Storm surge levels due to northeasters were generated from the Corps of Engineers' Wave Information Study (WIS) data hindcasts. The 'northeaster' curve shown on this graph was interpolated from data points for the 2, 5, 10, 20, and 50-year recurrence events at Miami Beach. Like the FEMA hurricane data, the WIS northeaster data does not include astronomical tides. Since most northeasters last several days and span many tidal cycles, the appropriate astronomical high tide value should be added to this northeaster curve to represent a realistic 'worse-case' scenario.

The FEMA curves are extrapolated below the 10-year event and the WIS northeaster curve is extrapolated above the 50-year event, so care should be used when determining surge levels based on these extrapolated portions of the storm surge curves. Each of these curves represents the results of surge level calculations based on a given set of 'typical' storm input parameters; for specific storm events re-calculation of these curves is necessary. A regression analysis was used to extrapolate data beyond the stated computational limits.

Sea Level Rise.

General Information. Eustatic sea level change is defined as a global change in the water surface elevations of the world's oceans. The total relative sea level change is the combination of eustatic sea level change and changes in local land surface elevations. The eustatic sea level has varied widely over geologic time, and evidence suggests that sea levels in the past have been both much higher - and much lower - than present levels.

Calculation of SLR Rates. Sea levels have been rising gradually throughout the study area during the entire period of record. The longest water-level record in the Miami Beach area was measured by NOAA gage #8723170. Recorded water levels from this gage span 50 years, extending from 1931 to 1981. During this period the average annual rate of sea level rise was 2.39mm per year, +/- 0.43 mm/yr. It is generally accepted that sea level will continue to rise and that the rate of rise may accelerate due to climatic changes.

The Corps of Engineers provides guidance on the calculation of sea level rise and on its application to the design process. The Corps of Engineers' Engineering Regulation (ER) 1100-2-8162 was issued in December 2013 to establish procedures for projecting sea level rise into the future based on global sea level change rates, local historic sea level change rate, base year of project analysis, and number of years in the period of analysis. This ER requires that three scenarios be examined, which result in low, intermediate, and high predictions of sea level rise. The low value is based on an extrapolation of the local historic sea level rise rate. The intermediate and high values are based on the National Research Council (NRC) sea level rise predictive Curves I and III, respectively.

All three curves are based on the following basic equation for prediction of eustatic sea level rise due to ongoing glacial melting and thermal expansion of ocean water :

$$E(t) = 0.0017t + bt^2$$

In this equation *E(t)* is the eustatic sea level rise (in meters); *t* is the time of the projection into the future using 1992 as a baseline year. 1992 is used as the baseline because it is the midpoint of the previous tidal epoch (1983-2001). The value *b* is a coefficient that varies for each of the three NRC curves (note that only curves I and III are used in this analysis). The coefficient *b* is equal to 2.71E-5 for Curve I; 7.00E-5 for Curve II, and 1.13E-4 for Curve III. This equation assumes a global mean sea level change rate of +1.7 mm/yr. These parameters were used to calculate the following three sea level rise prediction curves as required in ER 1100-2-8162. In Figure 2 the extrapolated historic rate is represented by the green line; the NRC Curves I and III predicted rates are represented by the blue and red lines, respectively. These three curves correspond to the low, intermediate, and high predictions of sea level rise required by ER 1100-2-8162.

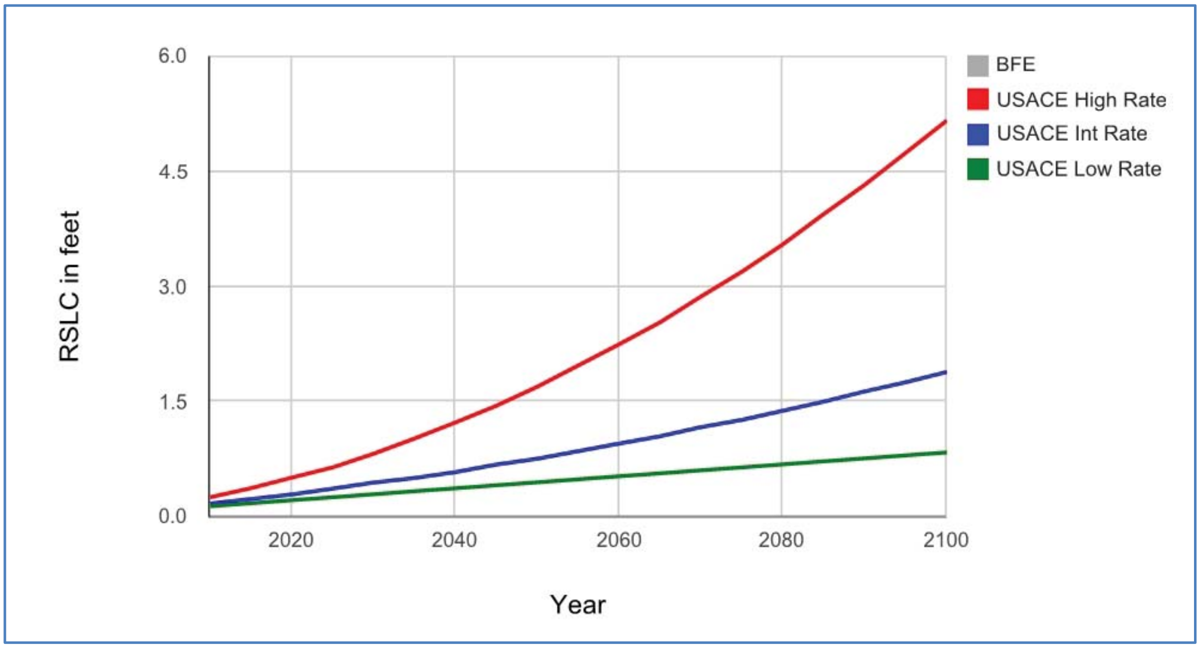

Figure 2. Summary of Predicted Sea Level Rise in Miami Beach by Year 2100.

Most Corps of Engineers shore protection projects, including the Dade County BEC & HP Project, were authorized for a 50-year period of Federal participation. Initial construction of the main segment of the Dade County project was begun in 1975 and the corresponding end of Federal participation will occur in 2025. The Sunny Isles segment of the project was constructed in 1988, and Federal participation in that segment of the project will expire in 2038. As seen in Table 2, the range of predicted values of sea level rise by the year 2025 varies from 0.26 to 0.66 feet. The range of predicted values of sea level rise by year 2038 varies from 0.36 to 1.15 feet. The shaded regions of the table correspond to the years in which the periods of Federal participation expire.

Table 2								
USACE SLR Curves - ER 1100-2-8162								
SLR Curve	2010	2015	2020	2025	2030	2035	2038	2040
USACE - Low	0.14	0.18	0.22	0.26	0.30	0.34	0.36	0.38
USACE - Intermediate	0.17	0.23	0.29	0.36	0.43	0.50	0.55	0.58
USACE - High	0.26	0.38	0.51	0.66	0.83	1.02	1.15	1.23

Shoreline Response to SLR. As sea levels rise (or fall) the shoreline will adjust to remain in equilibrium. Accordingly, both shoreline position and beach fill volume can be expected to change in response to changing water levels. Per Bruun first provided quantitative guidance on this phenomenon in 1962 and versions of his method are still in use today. The Bruun method proposes that the beach profile will tend to stay in equilibrium with the associated water level; that as water levels rise the profile will tend to be displaced upward and landward in response.

The Bruun Rule is applicable mainly to long, straight reaches of shoreline that are provided with with adequate supplies of sand. These conditions are reasonably well met along the Miami-Dade shoreline. Little is known about the rate at which profiles adjust to changing water levels, but since some period of time is required for these cross-shore processes to adjust, these rates of recession should mainly be applied to long-term shoreline changes. The Bruun method provides a means of estimating shoreline changes in the absence of any field data, but actual historical measurements should be used whenever possible, to determine more accurate profile adjustments due to changing water levels.

The Bruun method proposes a two-dimensional formula for calculating the rate of shoreline recession in response to water level increases that takes local topography and bathymetry into account. The Bruun method assumes that as sea levels rise, the beach profile will tend to establish the same bottom depths relative to the surface that existed prior to the rise in water level. To achieve this the natural beach profile will be translated upward and landward to maintain equilibrium. Assuming the longshore transport of sediment in and out of a given reach of shoreline is equal and since the seaward portion of this profile is near the depth of closure, the material required to re-establish the nearshore slope must erode from the upper reaches of the subaerial beach. The shoreline recession (*X*) that results from a given change in water levels can be calculated using the Bruun Rule, which is defined as:

$$X = \frac{-SW_*}{(h_* + B)}$$

Where:

S is the rate of water level change (varies according to Curves I through III above),
B is the berm height above datum (approximately 6 feet),
h_* is the depth of closure below datum (approximately 25 feet), and
W_* is the width of the active profile (about 1,200 feet).

These coefficients were entered into the Bruun equation and the resulting shoreline recession values are presented below in Table 3. As seen in this table the predicted shoreline recession values due to sea level rise (relative to the baseline year of 1992) range from -10.1 to -25.5 feet by the end of the period of Federal participation for the main segment of the Dade County BEC & HP Project in 2025. The recession values range from -13.9 to -44.5 feet by the end of the period of Federal participation for the Sunny Isles segment in 2038. Relative to the year 2015, the predicted recession values range from -3.1 to -10.8 feet along the main segment, and from -6.9 to -29.8 feet for the Sunny Isles segment.

Table 3

Shoreline Recession - Per Bruun Equation (ft)

SLR Curve	2010	2015	2020	2025	2030	2035	2038	2040
USACE - Low	-5.4	-7.0	-8.5	-10.1	-11.6	-13.2	-13.9	-14.7
USACE - Intermediate	-6.6	-8.9	-11.2	-13.9	-16.6	-19.4	-21.3	-22.5
USACE - High	-10.1	-14.7	-19.7	-25.5	-32.1	-39.5	-44.5	-47.6

Volumetric Response to SLR. As sea levels rise (or fall) and the beach profile adjusts, the volume of material contained in the beach berm will adjust accordingly. Guidance on this topic is provided by Corps of Engineers' Engineering Manual (EM) 1110-2-3301. This EM describes a procedure to calculate changes in beach volume (V) based on known values of shoreline recession, as calculated above. The equation presented in EM 1110-2-3301 is :

$$V = (B + h_*)X$$

Where B is the berm height above datum, h_* is the depth of closure below datum, and X is the horizontal translation of the profile. In this case B = 6 ft; h_* = 25 ft; and the values for X are presented in Table 3 above. Based on this methodology, the calculated volumetric changes (in cubic yards per linear foot of shoreline) due to sea level rise are summarized in Table 4.

Table 4

Volumetric Changes - EM 1110-2-3301 (units of cy/lf)

SLR Curve	2010	2015	2020	2025	2030	2035	2038	2040
USACE - Low	6.2	8.0	9.8	11.6	13.3	15.1	16.0	16.9
USACE - Intermediate	7.6	10.2	12.9	16.0	19.1	22.2	24.4	25.8
USACE - High	11.6	16.9	22.7	29.3	36.9	45.3	51.1	54.7

Observed sea level changes over the previous years of the project life have followed the low USACE curve, and this curve will be used as a baseline condition for future volumetric projections. Using a base year of 2015 and extrapolating the unit volumetric changes from Table 4 across the length of the main project segment (57,000 feet in length) and the Sunny Isles segment (13,200 feet in length) yield the following values.

For the main project segment, the total potential volumetric increases as per the intermediate curve are (16.0 - 11.6) x 57,000 ft = 250,800 cy, by the year 2025. Similarly, the potential volumetric increases along the main segment using the USACE high curve are (29.3-11.6) x 57,000 ft = 1,008,900 cy, by 2025. Using the same methodology, the corresponding values for the Sunny Isles segment are 110,880 cy (intermediate curve), and 463,320 cy (high curve), by 2038. These values equate to annual increases of rates of fill placement that vary from 25,080 cy/yr to 100,890 cy/yr for the main segment, and 4,820 cy/yr to 20,140 cy/yr for the Sunny Isles segment. Based on the annual renourishment rate of 190,000 cy/yr for the main segment and 50,000 cy/yr for the Sunny Isles segment (see Sub-Appendix A), these values equate to increases in annual renourishment volumes of 13.2 to 53.1 percent for the main segment, and 9.6 to 40.3 percent for the Sunny Isles segment.

Renourishment of various segments the of the Dade County project will be required several more times before expiration of Federal participation in 2025 and 2038. Monitoring of eustatic sea levels over the past few decades indicates that sea level changes during this period have occurred very slowly, and have most closely followed the low curve as presented above. At this point in time, no acceleration in the rate of water level rise has been observed from any of the relevant gage data. Due to the minimal change in water levels since project construction, there has been no significant impact on the overall management of this project or on any of the other shore protection projects in the region. However, a re-evaluation of the project should be conducted if significant changes in eustatic sea levels do occur. Due to the relatively short timeframes until expiration of the periods of Federal participation for both project segments and based on past project experience, any profile changes due to sea level rise are likely to be minimal, and the higher values presented in the preceding analysis represent a ‘worse-case scenario’ of the effects of sea level rise. Simple measures such as slight adjustments in construction berm elevation may be adequate to address sea level changes in all but the most extreme cases.

Wind Data.

Local winds in the project area are the primary means of generating the small-amplitude, short period waves that impact the south Florida shoreline for much of the year. Miami-Dade County lies at 26 degrees north latitude, within the edges of the tropical tradewind zone. Winds in this zone originate from the northeast, east, and southeast much of the year, with the greatest velocities originating from the northeast (in winter months), and the greatest frequencies of occurrence from the east and southeast (spring, summer, early fall).

Figure 3 shows a summary of wind data from WIS Station 63470, located at latitude 25.833 degrees north, longitude 79.917 degrees west, or approximately 11 nautical miles due east of central Miami Beach. This wind rose summarizes windspeeds, directions, and durations at this station location, and includes the seasonal variations described above. From February through September, winds originate most frequently from the eastern 45-degree sector, with a stronger tendency toward the southeasterly directions from March through August. From October through January winds originate more from the northeast sector as cold fronts move through the region, usually associated with ‘northeasters’ of varying intensities. These winds generate much of the wave energy that drives sediment transport along the beach, and can result in beach erosion along the Miami-Dade shoreline.

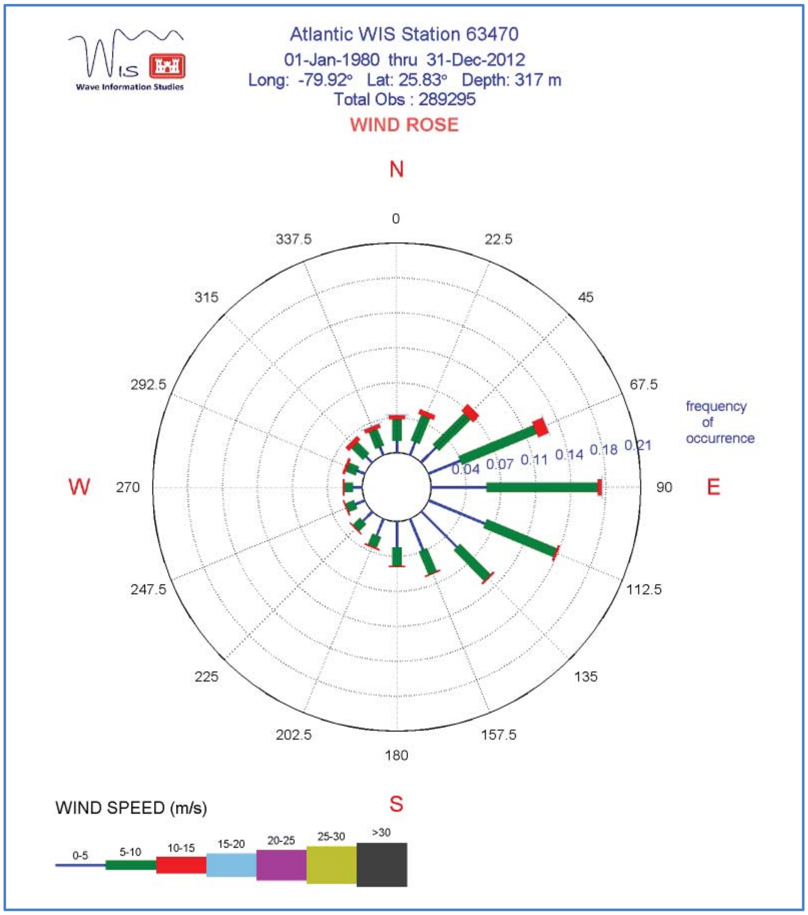

Figure 3. Wind Rose at WIS Station 63470, 11 nm offshore of Miami Beach.

<u>Wave Data.</u>

General. The project area is vulnerable to wave attack from all of the easterly directions. As discussed in the previous section of this report on wind data, the overall climate is moderate and as a result, locally-produced waves are generally small and low-energy relative to most of the east coast of the U.S. The study area is shielded from all but the most northerly open-ocean storm swells by the Bahama Bank (Figure 6), so with a few exceptions distant storm swells do not cause significant damage within the project area. The presence of the Bahama Bank 60 miles offshore also limits the local generation of fetch-limited waves.

Two primary modes of beach erosion are experienced along the Miami-Dade County shoreline: large-scale episodic damages to the project and upland development result from storm wave impacts from the relatively infrequent passage of hurricanes, tropical storms, and strong northeasters; and the much slower but persistent erosional damage to the Federal project which results from the nearly constant occurrence of smaller locally-generated wind waves along the county's shoreline. Both types of wave attack have played a part in the historical erosional damages to the Federal project, and each damage mode will be examined in greater detail in this report.

The largest waves that impact the Miami-Dade County shoreline are produced by nearby tropical disturbances including hurricanes, and the relatively few northeasters which produce strong swells from the most northerly directions. The extreme southerly location of the project area on the Florida peninsula usually results in weaker local effects from the passage of northeasters, since these large-scale weather systems tend to lose strength as they move further south into warmer climates. The reverse is true for hurricanes and other tropical disturbances, and as a result Miami-Dade County lies near the center of a broad corridor of hurricane activity.

Deepwater Wave Conditions. The most detailed long-term database available for this region is the revised Wave Information Study (WIS) wave hindcast produced by ERDC for the Atlantic coast of the U.S. This WIS data was used in this study to define wave characteristics of the area, and is typically used as an input database for numerical shoreline modeling, such as the Wave Refraction Study contained in Sub-Appendix B of this report. This WIS record extends from January 1980 through December 2012, and this 33-year data set includes the effects of tropical disturbances. WIS datasets consist of a hindcast time-series listing of wave height, period and directions at 1-hour intervals throughout the period of record, and various data summary products are available for each station as well.

The locations of all WIS hindcast stations along the southeast Florida coast are shown in Figure 4. The locations of the three primary WIS stations that were used in this study are highlighted in Figure 4 and described in more detail in Table 5. Station 63470 is located 11 nautical miles offshore of central Miami Beach, and was used to define wave characteristics along the Dade County BEC&HP Project. Stations 63450 and 63455 are located offshore of St. Lucie and Martin Counties, respectively, and were used to provide input to the wave refraction analyses of borrow areas in those locations.

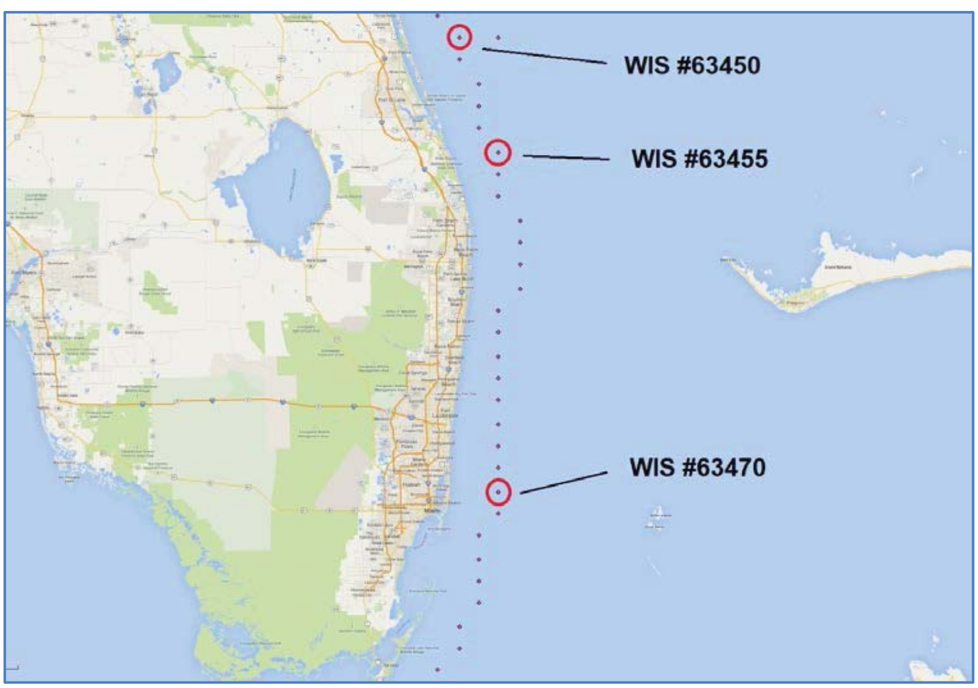

Figure 4. WIS station locations; WIS stations used in this study.

Table 5				
WIS Hindcast Stations Used in This Study				
Station ID	Latitude (N)	Longitude (W)	Depth (m)	Description
63470	25.83	79.92	317	11 nm east of Miami Beach, Miami-Dade County
63455	27.08	79.92	172	14 nm SE of St. Lucie Inlet, Martin County
63450	27.50	80.08	23	11 nm E of Ft. Pierce Inlet, St. Lucie County

This WIS database also includes the sheltering effects of the Bahamas Bank, which lies directly offshore of the southeast Florida coastline (see Figure 6). Inclusion of this feature is considered essential for providing realistic wave input data for numerical shoreline simulation models, as the presence of the Bahama Bank prevents distant ocean swells from reaching the Miami-Dade County shoreline from all but the most northerly directions, and limits the generation of wind waves under a wide variety of conditions.

Analyses of hindcast wave data and field observations of the wave environment along the southeast Florida coast indicate that strong seasonal effects exist, which closely correlates to the wind data characteristics. Waves during the summer months rarely exceed 1-2 feet, and usually originate from the east/southeasterly directions. These locally-generated wind waves occur during the majority of the time throughout the year, with the exception of a shift toward more frequent occurrences of larger storm-generated waves from the northerly directions during the winter months. Historically, calculated net littoral drift rates along the southeast Florida coast have indicated a net transport direction to the south, with a large northerly transport component during the summer months. This bimodal seasonal effect is reflected in the data contained in the wave rose, as shown in Figure 5.

An examination of monthly summaries of WIS hindcast data demonstrates the seasonal variation of deepwater wave heights between the summer and winter months. During the summer months wave heights are generally much lower than in the winter months, and are concentrated in the lowest wave-height band in the summary tables. During the months of August and September the overall wave energy is still low, but several large storm events are typically observed; these are the products of tropical disturbances and early ‘northeasters’. During the winter months (November to March) the overall wave energy is higher, with the majority of wave events in each month falling in the 2.0-3.99 ft range. A greater number of large northerly storm waves are produced by winter ‘northeaster’ storms during this period.

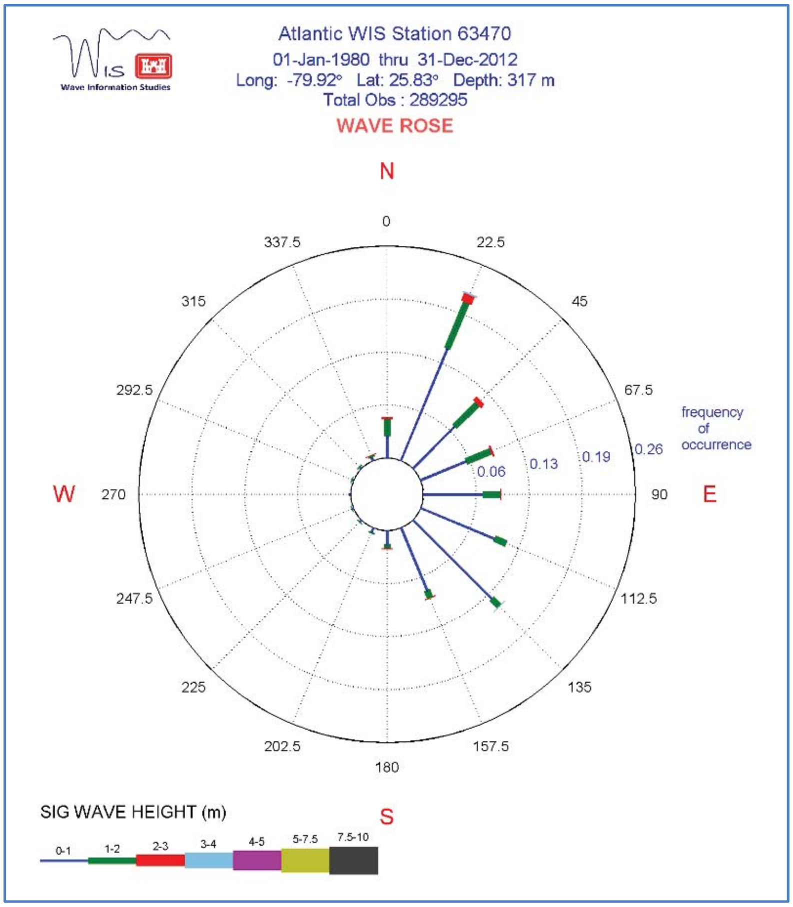

Figure 5. Wave Rose at WIS Station 63470, 11 nm east of Miami Beach.

Throughout the entire database at station 63470, 92 percent of all deepwater wave events fall within the 0 to 1.49 – meter wave height bands (0 – 4.9 feet). Of the larger wave events in the WIS record, only 0.23 percent exceed 3 meters (9.8 feet), and many of these events occur in August and September and are the results of tropical disturbances. An analysis of the WIS database indicates that on average, the southeast Florida coast experiences a relatively low-energy wave environment year-round, but a pronounced increase in overall wave energy occurs between the summer and winter months.

Wave periods show the same seasonality as wave heights; short-period, locally-generated wind waves are common throughout the year, but in the summer months these short period waves occur almost exclusively. During the winter months a shifting towards higher-energy, longer-period storm swells can be seen in the monthly summary of occurrences of peak wave periods.

Shallow-Water Wave Conditions. The wave environment can differ considerably between the deepwater WIS station location and the nearshore region. The offshore location of WIS data is subjected to waves originating from all directions, but the nearshore environment is subjected only to waves originating from the easterly directions due to the sheltering effect of the shoreline and the refractive effects of the nearshore reef system.

The bathymetry offshore of southeast Florida is somewhat unusual for the east coast of the U.S., in that the continental shelf is at its narrowest along Palm Beach, Broward, and Miami-Dade Counties. The typical width of the continental shelf along the Miami-Dade County shoreline is about 3 miles. The effect of this bathymetry can be quite significant on the refraction of waves transiting the shelf from deep water toward the shoreline. Numerous lines of shore-parallel coral reefs extend along the southeast Florida coast, and these reefs can strongly affect the propagation of incident deepwater waves. These reef structures tend to feature irregular bathymetry in both the longshore and cross-shore directions, and can refract the incident waves in complex ways. Coral structures tend to be very rough, and can reduce wave energy via frictional losses. The shallower reefs can cause even greater energy loss by forcing waves to break in waters that would otherwise be too deep for breaking.

Wave refraction analyses from previous studies (reference 2) demonstrate how incident deepwater waves are altered as waves propagate from deepwater to shallow water depths across the south Florida shelf. First, all waves incident from the westerly directions are removed from the nearshore wave record since they are propagating away from the shoreline. As the remaining waves propagate through progressively shallower waters the wave directions become aligned more perpendicular to the depth contours in accordance with Snell's Law. Waves in shallow water will therefore be much more closely aligned to shore-normal than waves in greater water depths. Exceptions to this observation are seen in areas where prominent features (reefs, nearshore sandbars) cause waves to refract in directions away from shore-normal.

In spite of the short distance across the shelf, wave heights are generally reduced as they transit the shelf zone and lose energy due to bottom friction. Wave refraction near the coastline can cause both constructive and destructive wave interference patterns. Wave amplitudes can be increased or decreased over relatively short distances in the alongshore direction, resulting in corresponding areas of erosion or accretion along the beach.

Previous studies have demonstrated the seasonal variations of the South Florida wave environment in the nearshore zone. The shift of incident wave directions from the east-southeast during the summer months to a more east-northeasterly direction in the winter months is easily seen in the model output data. From May through August waves originate from the east and southeast sectors over 80 percent of the time during each of the summer months. Waves originate from the east and northeast directions over 80 percent of the time during the remaining winter months. The resulting longshore transport rates which have been calculated in previous studies verify this seasonal change in direction : most calculated gross transport rates are several times higher than the calculated net rates since the northerly sediment transport along the coastline during the summer months cancels out much of the southerly transport which occurs during the winter months. These calculations are verified by long-term field observations of sediment movement along the Miami-Dade shoreline.

Storm Activity.
Figure 6 is a plot produced using NOAA records, which shows tracks of hurricanes that have passed within 50 nautical miles of the Miami-Dade County shoreline over the period of record, between 1859 and 2013. NOAA records indicate that a total of 76 tropical weather events passed within a 50 nautical mile radius of Miami Beach during that 154-year interval. Of those 76 events, 49 consisted of hurricanes of at least Category 1 intensity (winds over 64 mph). This equates to an average occurrence of a Category 1 (or stronger) storm every 3.1 years. Major hurricanes (Category 3, 4, or 5) occurred 15 times during this period, an average of once every 10.3 years.

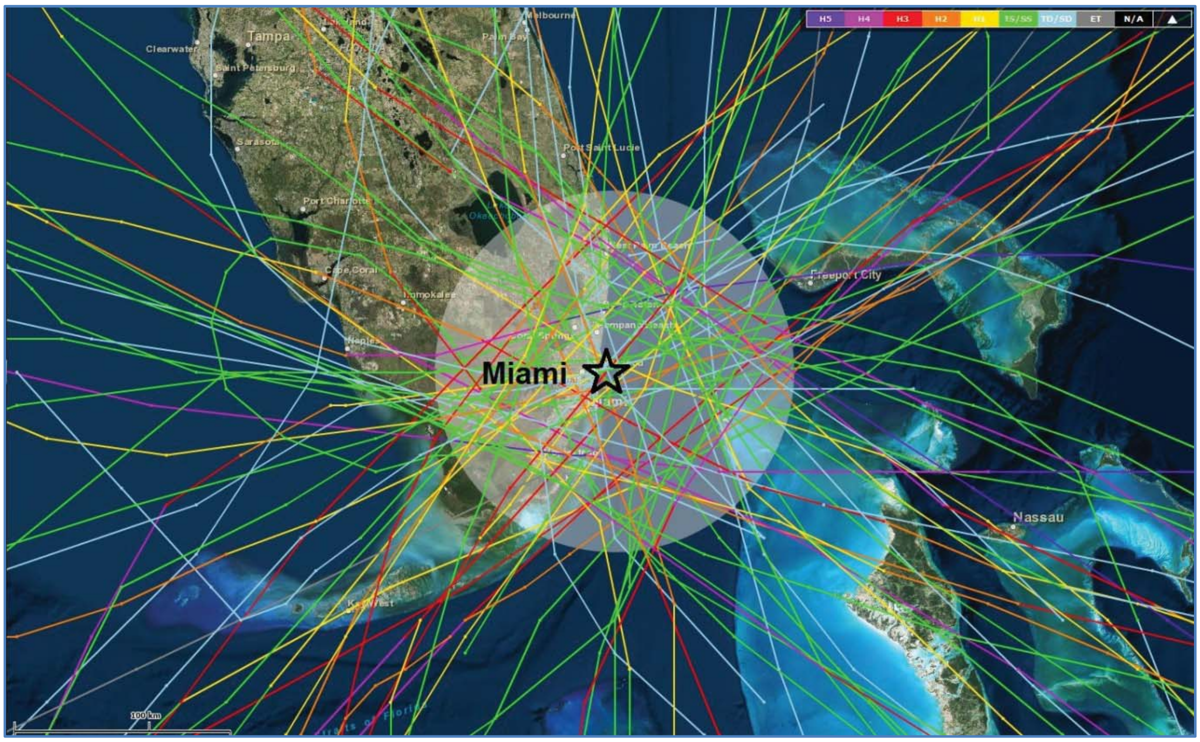

Figure 6. Historical hurricane tracks within 50 nm of Miami Beach, 1859-2013. *(NOAA)*

Several significant storm events have impacted the region since initiation of construction of the Dade County BEC&HP Project in 1978. These include Hurricanes Floyd in 1987, Andrew in 1992, Jerry in 1995, Irene in 1999, and Katrina in 2005. Of these, Andrew was by far the most severe, and had the most damaging effects on the Miami-Dade shoreline. Andrew caused extensive beach erosion, coastal and inland flooding, and extensive property damages due to elevated wind, waves, and water levels.

Storm-generated waves from each of the tropical weather events shown in Figure 6 that occurred during the period 1980-2012 are included in the Wave Information Study hindcasts. This was the primary wave input database used in wave modeling studies for this report, and will be described in greater detail in Sub-Appendix B.

ENGINEERING ANALYSES

General.

The environmental parameters described in previous sections of this report drive sediment movement along the Miami-Dade coastline. As sediment is transported throughout the project, areas of erosion and accretion can result. In order to quantify these changes, annual beach profile monitoring surveys are performed by the County to monitor the condition of the project. Volumetric changes are calculated based on these surveys, as well as the volumes required to re-establish the authorized project dimensions. These surveys are also used as a basis for other studies, such as detailed design of project features, numerical modeling, and development of sediment budgets.

The most recent monitoring survey was performed by Miami-Dade County in January 2014. This survey (and other previous monitoring surveys) were analyzed to determine annual losses, the volumes required to rebuild the construction template along the various segments of the project, and to develop a projection of future renourishment events that will be required to maintain the project throughout the remaining years of Federal participation. These analyses are all contained in Sub-Appendix A. A summary of these analyses is provided below.

Sediment Budgets.

In order to quantify the movement of material along the Miami-Dade County coast, various sediment budgets have been devised during the course of numerous studies. The development of these sediment budgets is a complex process and will not be repeated in this report. However, one comprehensive source of information on sediment movement along the Miami-Dade County shoreline is provided in the *Dade County Evaluation Report – 2001* (reference 2). The sediment budget provided in this report was developed largely by Coastal Systems International in the 1997 study *Dade County Regional Sediment Budget* (reference 3). Summary figures from these reports are provided in Figures 7a and 7b. Although developed in 1997, this analysis is still considered to be representative of sediment movement along the Miami-Dade shoreline today.

As seen in Figures 7a and 7b, the net flow of sediment is from north to south along the Miami-Dade shoreline. The net rates of transport vary with location, but generally range between about 60,000 to 80,000 cy/yr. The gross transport rates can be substantially higher, due to the periodic northward flow of sediment caused by winds/waves from the southerly directions during the summer months, as previously described in this report. Areas of higher and lower net transport are observed as well : the lowest net transport is observed at Bakers Haulover Inlet, and is due primarily to the obstruction created by the inlet. The highest net transport is observed at 32nd St in Miami Beach, and is due to waves breaking at a steeper angle along the curved shoreline.

The volumes of sediment deposited or eroded within each segment of shoreline are also shown in Figures 7a and 7b. These volumes are based on analysis of monitoring surveys and define areas of erosion and deposition throughout the project. It is readily seen that most of the project length is erosive, except for Haulover Park and south Miami Beach.

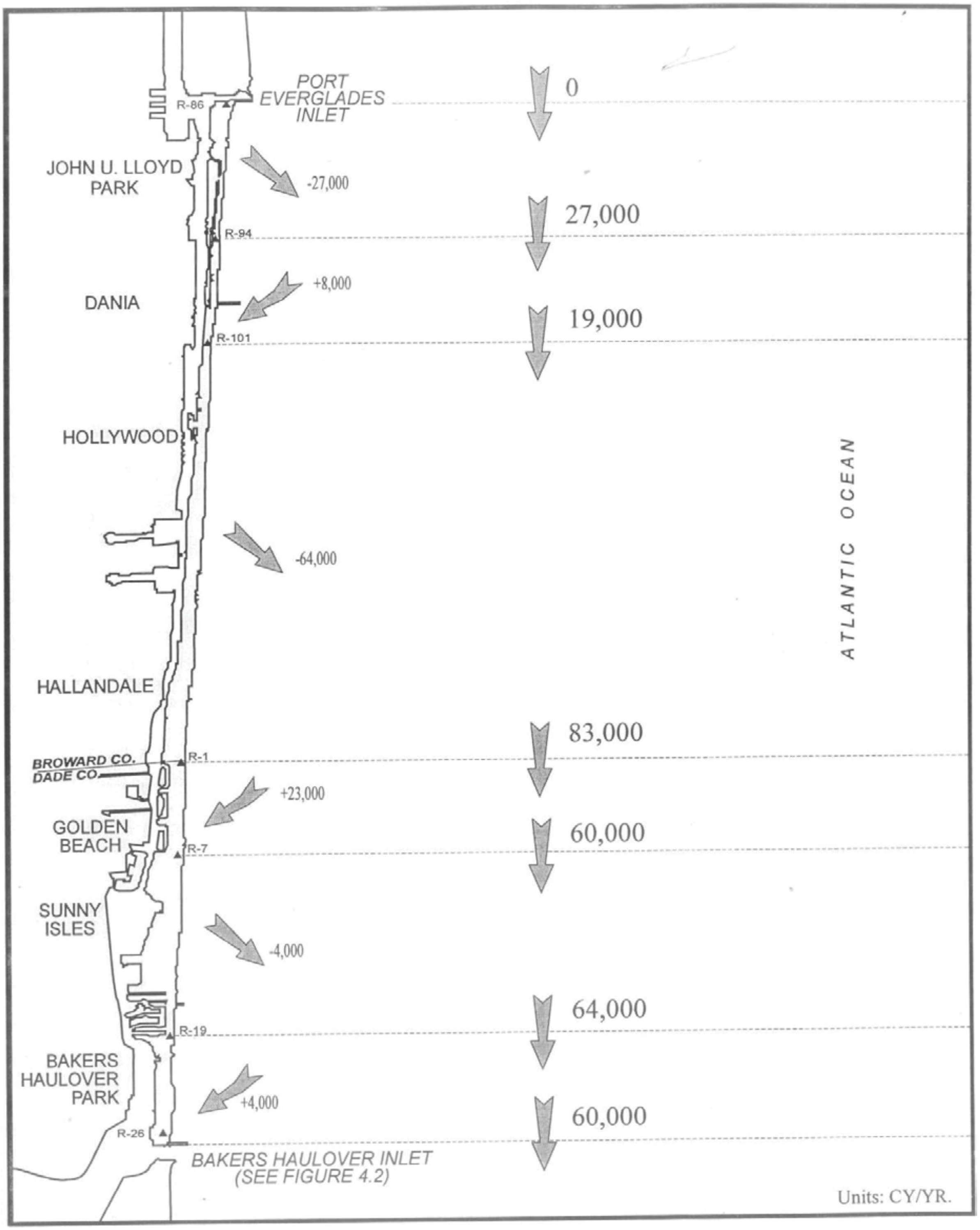

Figure 7a. Calculated sediment budget for southern Broward/northern Dade Counties.

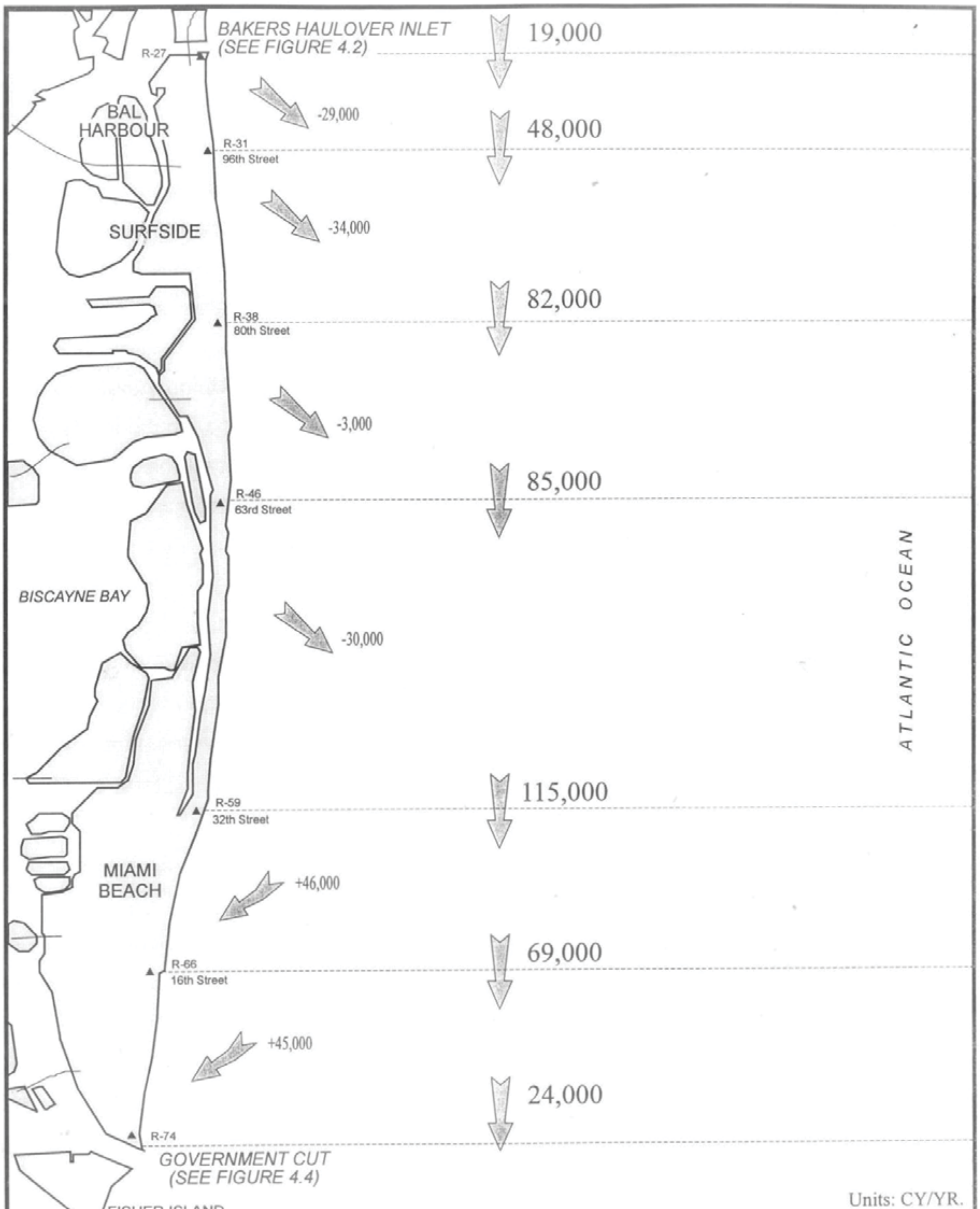

Figure 7b. Calculated sediment budget for southern Miami-Dade County.

Calculated Erosion Rates.

Several decades of monitoring data were examined to determine reliable volumetric change rates to be used to forecast the future volumetric requirements of the Dade County BEC Project. The underlying assumption used in this analysis is that the project will continue to erode in the future as it has in the past. The methodology used in this analysis is described in detail in Sub-Appendix A, and a summary of the findings is provided in Table 6. These values are considered to be slightly conservative (high), as they are intended to capture the erosive effects of storms in high-energy years.

Table 6

Annual Erosion Rates

Segment		Annual Erosion Rate (cy/yr)
Sunny Isles		50,000
Haulover Park		15,000
Bal Harbour		55,000
Surfside		45,000
Miami Beach (HS)		50,000
Miami Beach (NHS)		25,000
TOTAL :		240,000

Present Renourishment Requirements.

The most recent monitoring survey of the Federal project was performed in January 2014. This survey was used as a basis for defining the current condition of the project, and was also used as a baseline for future volumetric projections. Like most monitoring surveys, this survey consisted of beach profile lines extending from the upland region of the beach seaward to a depth of closure, typically in 20-25 feet of water. Profiles were surveyed at every DNR monument along the length of the Federal project, and additional profiles were surveyed at shorter intervals in known areas of erosion. Profile spacing was therefore approximately 1,000 feet along most of the 13.5-mile project, with spacing as little as 200 feet in the areas of interest.

The 2014 survey was first analyzed to determine the volumes of fill required to restore the construction template along the length of the project. This was accomplished simply by overlaying the various construction templates used to construct the project onto the January 2014 survey and calculating the difference. These calculations were performed by project reach, corresponding to the different communities along the length of the project. A summary of the volumes required to fully restore the project to its full construction template is provided in Table 7. From this table, the total volume required to restore the project to its fully-renourished condition is 1,881,140 cubic yards, based on January 2014 conditions. No fill placement is required in Haulover Park because it is accretional, and no fill is required in Bal Harbour because it was fully renourished in early 2014. Additional details of this analysis are provided in Sub-Appendix A.

Table 7		
Volumes to Rebuild Construction Template, 2014		
Project Reach	Monuments	Volume
Sunny Isles	7 - 19.5	447,330
Haulover Park	19.5 - 26.5	-
Bal Harbour	27 - 31.5	-
Surfside	31.5 - 36.5	425,460
Miami Beach (hotspots)	misc.	402,250
Miami Beach (non-HS)	36.5 - 74.5	606,100
Total	7 - 74.5	1,881,140

Future Volumetric Projections - Dade County BEC & HP Project.

A primary feature of this LRR is the projection of future volumetric needs of renourishment throughout the remaining years of Federal participation in the Dade County BEC & HP Project. This was accomplished by using a two-step process. The first step was to calculate the volume required to restore the eroded areas of the project to a fully renourished condition. This analysis was described in the previous section, and the results are presented in Table 7. The second step in this analysis was to project future renourishment needs through the remaining years of Federal participation for each project segment. This was accomplished by using the erosion rates presented in Table 6, and the renourishment intervals developed in Sub-Appendix A.

The results of this analysis are summarized in Table 8. Each projected future renourishment event is shown in this table, throughout the remaining years of Federal participation for each of the two project segments. As seen at the bottom of the table, the total volume of fill required through the end of Federal participation is 3,625,620 cubic yards. It is important to note that this volume represents the amount of fill required on the beach. Due to normal dredging losses an additional 30% is required at the offshore borrow source, so the required volume of offshore borrow material required would be 1.3 x 3,625,620 cy = 4,713,310 cy.

Table 8			
SUMMARY OF PROJECTED RENOURISHMENT EVENTS			
Dade County BEC & HP			
Year	Renourishment Event	Placement Area	Volume, cy
2016	Sunny Isles, Miami Beach Hotspots	R7-R26; R41-46; R50-55; R60-64	1,104,060
2017	Surfside	R32-R36	560,460
2019	Haulover, Bal Harbour, Miami Bch (non-HS)	R27-R31	1,026,100
2021	Miami Beach (HS)	R41-46; R50-55; R60-64	200,000
2022	Surfside	R32-R36	135,000
2025	----- End of period of Federal participation, Main Project segment ---		
2026	Sunny Isles	R7-R26	500,000
2036	Sunny Isles	R7-R26	100,000
2038	----- End of period of Federal participation, Sunny Isles segment ---		
		TOTAL :	3,625,620

Figure 8 graphically shows each individual renourishment event through the remaining years of Federal participation. Development of the quantities and construction schedules that are summarized in Table 8 and Figure 8 are provided in greater detail in Sub-Appendix A.

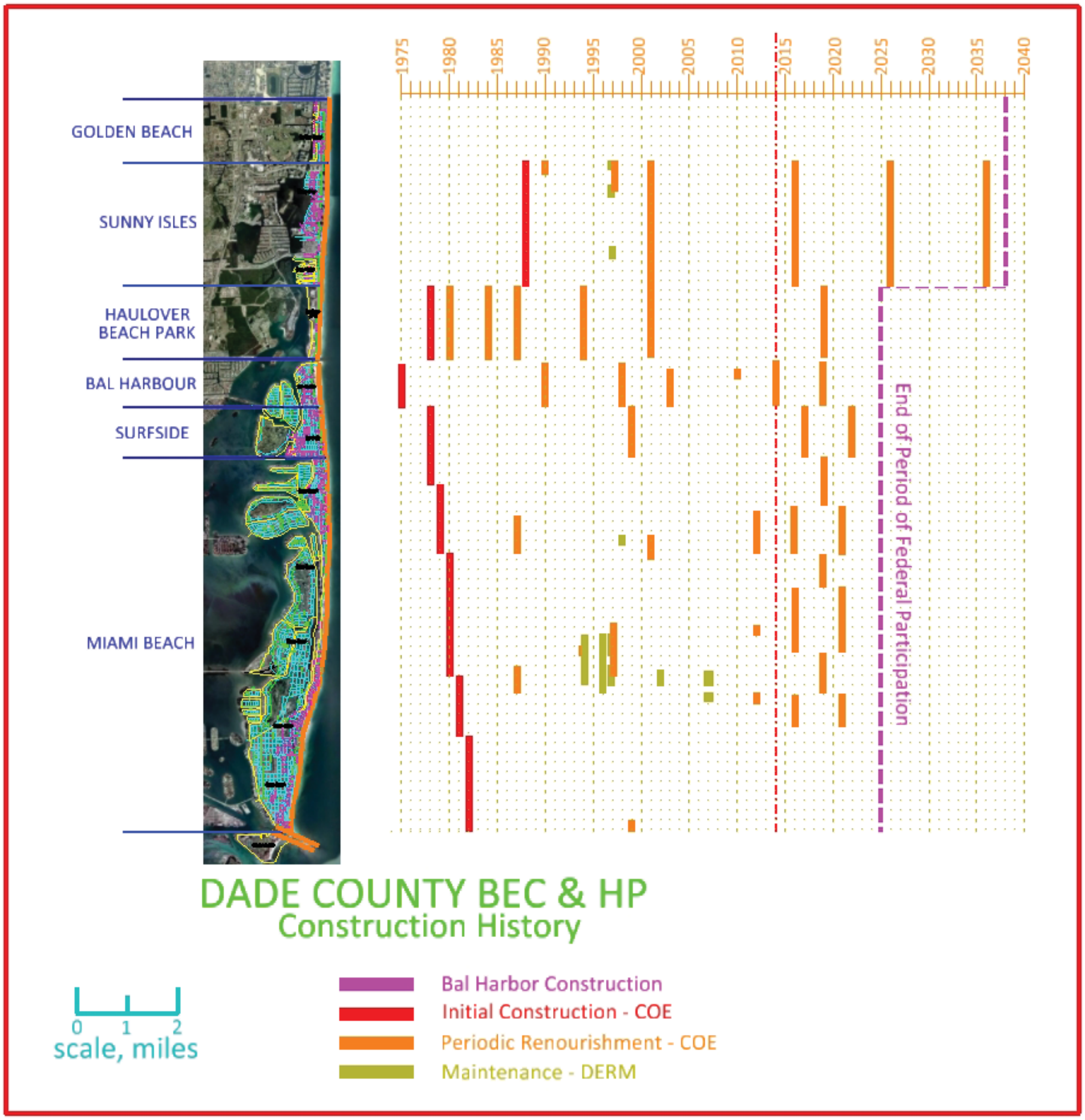

Figure 8. Projected future renourishment events – Dade County BEC & HP Project.

Wave Refraction Analysis.

The Dade County BEC & HP Project has depleted all local sources of offshore borrow material. In order to provide a continued supply of borrow material for periodic renourishment of the project, alternative offshore borrow sites were investigated. Two primary borrow sources have been identified in the Federal waters offshore of St. Lucie County and Martin County, in Borrow Area Zone C. These borrow areas are designated as “SL10-T41” and “M4-R105”, respectively, and are shown in Figure 9.

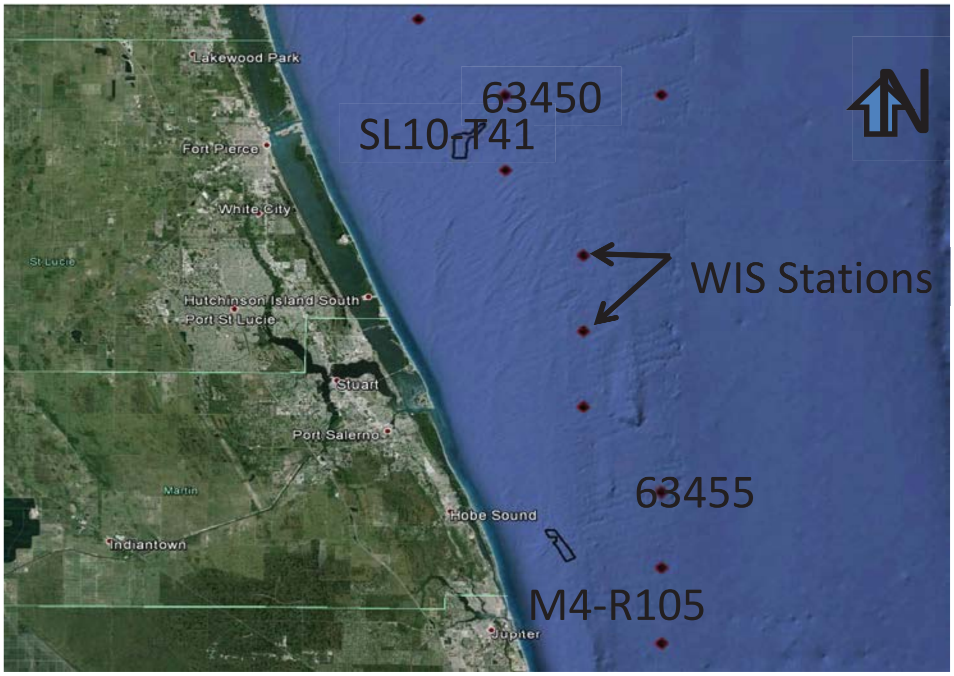

Figure 9. Locations of offshore borrow areas SL10-T41 and M4-R105.

This wave refraction analysis was performed to determine if dredging these borrow areas could have any adverse effects on the nearshore wave environment. As waves enter shallow waters, wave heights and directions become affected by the bathymetry which they propagate across. Due to their depth and proximity to the shoreline the possibility exists that excavation of these borrow areas could alter existing wave refraction patterns to the point that shoreline erosion could be increased in some areas. In response to these concerns this analysis was performed to determine the degree of potential impact of borrow area excavation on existing wave refraction patterns in the nearshore region.

The basic strategy of this study was to evaluate the differences between matched pairs of wave refraction simulations: each pair of simulations would use identical input wave parameters, and would differ only in the bathymetry being used. The first simulation would be performed using existing bathymetry, and the second simulation would represent the borrow area in an excavated condition. Differences between the two output wave fields would then be quantified and evaluated to determine the potential for erosional impacts that could occur along the shoreline. Any changes to wave conditions observed within the active littoral zone would suggest that the proposed dredging could impact sediment transport along the shoreline.

Each of the two proposed borrow areas were evaluated separately in this analysis, due primarily to the large distance between them. The numerical wave refraction model used in this study was CMS-Wave. A more detailed explanation of the CMS-Wave numerical model, input conditions, model setup, execution of model simulations, and model output and analysis are all contained in Sub-Appendix B. Figures 12 through 35 of the Sub-Appendix provide summary output graphs of select model output. Many more model simulations were made than are shown in Sub-Appendix B, but in the interest of brevity much of the repetitive model output was omitted from this report but is retained in output files that can be retreived if needed.

The threshold used in this analysis to determine if wave refractive effects were “significant” was whether any changes in wave height, period, or direction due to borrow area excavation were observed landward of the approximate depth of closure, which defines the seaward limit of the active littoral zone. The significant finding of this wave refraction analysis was that in no case did any impacts associated with borrow area deepening extend landward of the depth of closure. The only events that even approached this limit were the most extreme wave cases, particularly those with exceptionally long wave periods. However, even using the longest wave periods and largest wave heights on record, in no cases were any changes to the nearshore wave environment observed within the depth of closure.

It is also important to note that at every point during this analysis, conservative assumptions (i.e. selected to make the greatest impact) were made as to model input and executions of model simulations. By adopting ‘worst-case scenarios’ at every decision point, further assurance was gained that the model would tend to over-predict any nearshore impacts. Even with this ‘worst-case’ modeling strategy, no impacts could be found inside the active littoral zone. Specific examples of these measures adopted during modeling were that the borrow areas were digitally ‘dredged’ to far greater horizontal and vertical extents than is currently planned; low bottom-friction values were used, input wave conditions tended to focus more on extreme cases which would occur infrequently; more frequently-occurring ‘average’ wave input showed minimal effects and was largely omitted from the study.

Even with overly conservative assumptions throughout the model setup and execution, no impacts were observed landward of the depth of closure, for any of the incident wave conditions. It can therefore be concluded that the excavation of borrow areas SL10-T41 and M4-R105 to their maximum permitted limit will have no erosive impacts on the adjacent shorelines.

CONCLUSIONS

This Engineering Appendix presents relevant physical data and analyses on the Dade County BEC & HP study area, as well as the proposed offshore borrow sites SL10-T41 and M4-R105. The effects of winds, waves, and sea level rise on the project are discussed. The large-scale processes which affect the movement of sediment throughout the project are described and quantified. Recent survey data was used to determine that the placement of 1,881,140 cy of fill would be required to restore the Dade County BEC & HP Project to its full construction template. Historical project performance data was analyzed to determine that a conservative estimate of the annual erosion rate along the entire length of the project is 240,000 cubic yards/year. This rate was used to determine that a total volume of 3,625,620 cubic yards of material would be required to renourish the Dade County BEC & HP Project through the remaining years of Federal participation. Finally, a wave refraction analysis was performed to determine the effects of borrow area deepening on existing wave refraction patterns, since significant changes to the wave environment could potentially result in increased erosion along the adjacent beaches in the lee of the borrow areas. In response to this concern a wide variety of extreme (and commonly-occurring) wave events were simulated, both under existing conditions and with each borrow area in a fully-excavated condition. The results were compared and the study found that even in the most extreme wave conditions, the effects of borrow area deepening were highly localized around the borrow area and did not extend close enough to the shoreline to possibly cause increased erosion.

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1. Flood Insurance Study, Dade County, Florida, and Incorporated Areas, FEMA, November 1987.

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3. Dade County Regional Sediment Budget, Coastal Systems International Inc, January 1997.

4. U.S. Army Corps of Engineers, Coastal Engineering Manual, EM-1110-2-1100, April 2002.

LIMITED REEVALUATION REPORT

ENGINEERING APPENDIX
SUB – APPENDIX A

VOLUMETRIC ANALYSIS
AND PROJECTIONS

LIMITED REEVALUATION REPORT

ENGINEERING APPENDIX
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Dade County BEC&HP
January 2015

Introduction.
Miami-Dade County's offshore borrow areas have been depleted, and a search has been performed to find acceptable alternative borrow sources to sustain the Dade County BEC & HP Project through the remaining years of project life. In support of this effort, projections of the renourishment requirements of the project through the balance of the 50-year project life are required. In this analysis, recent monitoring surveys will be used to establish erosion rates that are currently affecting the project. These rates will be projected forward in time through the remaining years of the project life, to determine the total future sediment requirements of the project. Based on the present state of the project as determined from the most recent monitoring survey in January 2014, historic erosion rates will be used to project individual renourishment events throughout the remainder of the project life. This data will be used to plan for suitable borrow sources to sustain the project through its remaining years.

Years Remaining in Project Life.
The Dade County BEC & HP project was constructed in two main segments, described as follows:
(1) The first ("main") segment extends from Government Cut northward through Haulover Beach Park, covering a distance of 10.7 miles. Construction on this segment began in 1975. This project segment is currently authorized for a 50-year project life, which terminates in the year 2025.
(2) The second segment spans the 2.4-mile length of Sunny Isles. Construction on this segment began in 1988. This project segment is also currently authorized for a 50-year project life, which terminates in the year 2038.

Due to the lack of an available borrow source for the Dade County Project, and the time required to perform the engineering investigations and permitting work to establish a borrow source, the baseline year for constructing the remaining renourishments is presently estimated to be 2016. Using 2016 as the baseline for the purposes of this analysis, 9 years will remain in the project life of the "main" segment of the project, and 22 years will remain in the project life of the Sunny Isles segment of the project.

Surveys - Period of Analysis.
Numerous computations of volumetric changes along the length of the Federal project have been developed since initial project construction. Most of these analyses have been based on periodic monitoring surveys performed by the Corps of Engineers and by Dade County DERM over the years. These surveys were based on beach profiles referenced to Corps monuments in the early years of the project life, and transitioned over to DNR monuments beginning in the early 1990's.

It is normally desirable to use surveys spanning the longest time period available, to establish more reliable longterm shoreline change rates. However, direct comparison of the older surveys (surveyed from Corps monuments) with the newer surveys (surveyed from DNR monuments) is not possible in most cases, as the profile locations and azimuths do not coincide. There is an indirect method of comparison available however: surveys can be used to construct a digital terrain model (dtm), and profiles can be cut across the dtm at any desired location. A dtm constructed using COE profiles can therefore be cut at DNR profile locations to approximate the conditions along DNR profile locations. This data must be used with caution however, since the elevations along such “cut” lines are not actual field measurements, but are elevations interpolated from other locations along the coastline. Other important factors also limit the comparison of older survey data with newer survey data, including the addition of protective structures to the project and the occurrence of extreme storm events.

In response to accelerated erosion observed along several reaches of the project, protective structures were constructed to reduce sediment losses at the north end of Sunny Isles, at 32nd St in Miami Beach, and at the south end of the project at Government Cut. These three structural projects were constructed between 1999-2002. These structures strongly altered the patterns sediment movement in these regions, and for the purposes of determining present-day sedimentation patterns, construction of these structures rendered the older, pre-structure surveys obsolete.

Additionally, two notable periods of unusually severe storm activity have impacted the project. Hurricane Andrew passed across southern Dade County in 1992, causing substantial damage to the project. More recently, in 2004-05 an unusual combination of hurricanes and tropical storms impacted the region; these storms caused greater cumulative losses to the project than the single impact of Hurricane Andrew. Some survey evidence suggests that large volumes of sediment were driven out into deep water by the storms of 2004-05; some of this material may continue to rebound back to shore even today.

Due to the factors discussed above, the time period used for survey analysis to establish future erosion rates should be chosen very carefully. First and foremost, this time period should be representative of present-day conditions along the Dade County shoreline. It should also exclude unusually severe and infrequent storm events. Surveys taken before 2000 cannot be readily compared to more recent surveys due to the different sets of monumentation used. More importantly, the structures added to the project between 1999-2002 have altered sediment flow within the project to the point that surveys taken prior to 2002 no longer represent present-day conditions, and these surveys should be excluded from this analysis, at least in the vicinity of the structures. Finally, the storms of 2004-05 represent an anomaly in terms of sediment movement. The high wave energies associated with these storms caused sediment movement in quantities and patterns that do not represent the longterm “normal” movement of sediment within the project.

Due to these considerations, certain limitations of the periods of survey analysis are recommended. In this discussion, surveys taken prior to 2002 will still be considered, but will be treated more cautiously because of the strong impact the protective structures have had on portions of the project. Surveys taken in the 2004-05 timeframe will also be used in the analysis, but with the understanding that these values represent an upper range of sediment movement due to the extreme storm events that occurred in that period. The most recent full-county beach profile survey was performed in January 2014; this survey will be used as the baseline condition for the following analyses.

Method of Analysis.

The volumetric analysis presented in this report will consist of two phases :

1) Phase 1 : Present Need. As of July 2014 most reaches of the project were in a depleted condition. The January 2014 survey will be used as a baseline to calculate the volume of fill required in the immediate future to restore the project to the full construction template. Realistic assumptions will be made as to construction method, production rates, project costs, environmental restrictions, and practical limitations of construction, all based on nearly 40 years of construction experience along the Dade County shoreline. Due to the large volume of initial fill required and differing priorities of fill placement based on present project condition, this initial phase of project renourishment will be constructed in stages. Construction sequence will be detailed in following sections of this report.

2) Phase 2 : Future Renourishments. Completion of Phase 1 will restore the project to its full construction template. At this point the maintenance schedule developed in Phase 2 will maintain the design level of protection for the remaining years of project life. This schedule will be developed based on predicted erosion rates, and is described as follows.

In general, three different approaches will be taken in an effort to evaluate all relevant data towards establishing projected future erosion rates. As per the preceding discussion, the survey interval 2005 – 2014 will be used as a primary reference. This dataset best represents the project in its current condition for the reasons discussed above, although it may include some rebounding effects from the 2004-05 storms. A second method will be to examine older datasets. In some areas of the project the structural additions that alter 'historic' littoral processes are a great distance away and should have minimal effect. The examination of older survey databases can, in these cases, provide additional data to support a particular choice of erosion rate. Finally, the volume required to reconstruct select project segments can be calculated to establish a third erosion rate. The erosion of a particular project segment since the previous renourishment can be assumed to be a repeatable process, and the measured loss of material from the segment can be used to establish a projected future erosion rate.

Volumetric Analysis.

Phase 1 : Present Need. This analysis defines the present need for fill along the length of the project. The survey used in this analysis was performed in January 2014. This survey was performed by the project's local sponsor (DERM), and consists of 3,000 – ft long profiles taken at every DNR primary and intermediate monument. Profile spacing was therefore 500 feet +/- between adjacent profiles, and areas of particular interest were surveyed in greater detail, with profile spacings of typically about 200 feet.

This Phase 1 analysis consisted of a straightforward calculation of the volumes required to reconstruct the construction template along the various segments of the project. This was accomplished by superimposing the construction template onto the 2014 survey using the MicroStation CADD program. The resulting volumes required to rebuild the construction template were tabulated by project reach in a spreadsheet and are summarized below.

Volumes to Rebuild Construction Template, 2014

Project Reach	Monuments	Volume
Sunny Isles	7 - 19.5	447,330
Haulover Park	19.5 - 26.5	-
Bal Harbour	27 - 31.5	-
Surfside	31.5 - 36.5	425,460
Miami Beach (hotspots)	misc.	402,250
Miami Beach (non-HS)	36.5 - 74.5	606,100
Total	7 - 74.5	1,881,140

Table 1. Volumes to rebuild construction template, by project reach.

In general the *construction* berm was moderately to severely eroded along most reaches of the project in 2014, as indicated by the volumes required to reconstruct this berm. However, only minor erosion into the *design* berm was noted along the project in the 2014 survey. The areas where erosion into the design section occurred were located along limited portions of Sunny Isles and Surfside, and along the two hotspot areas of Miami Beach at 46^{th} and 55^{th} Streets.

Note that no volumes of fill are indicated for placement along either Haulover Park or Bal Harbour. Haulover Park has been gradually accreting in recent years, and only negligible erosion into the lower toe of the front slope was noted in this survey. Bal Harbour was severely eroded at the time of the January 2014 survey but a full renourishment of the project in the spring of 2014 re-established the full construction template along the length of this reach.

The total volume of fill required to reconstruct the construction template is therefore 1,881,140 cy, as of 2014.

Phase 2 : Future Renourishments. The monitoring surveys used in the primary analysis were taken in 2005, 2007, 2009, 2011, and 2014. These surveys span the entire length of the Federal project at approximately 1,000-ft intervals (or less), and typically extend about 3,000 feet offshore. Surveys were plotted in CADD (MicroStation) and volumes between adjacent profile pairs were computed using an average end-area method. The results were tabulated in excel spreadsheets, which are summarized in tables for each project segment throughout the remaining sections of this report.

This analysis will be broken into two primary parts: the "main" segment of the project extending from Government Cut to the north end of Haulover Park, and the second segment extending along Sunny Isles.

This analysis will proceed along the length of the project, from north to south, beginning with Sunny Isles. Due to its shorter length and lack of extensive maintenance history, the Sunny Isles segment is much more straightforward than most other segments of the project. For these reasons this Sunny Isles analysis will provide a more simplified example of the methodology used to project renourishment volumes into the future.

Sunny Isles – Future Renourishment Rate. The Sunny Isles segment of the Dade County BEC & HP project was initially constructed in 1988, and limited portions of Sunny Isles have been renourished in 1990, 1994, 1997, 1998, and 1999. Most of these maintenance events have been concentrated near the north end of the project because of rapid erosion due to end losses in that area. In order to reduce these losses a breakwater was constructed along northern Sunny Isles in 2002 and the full length of Sunny Isles was renourished at that time. This solution proved highly effective and no additional fill placement has been required since construction of the breakwater and beach fill in 2002. The most recent monitoring survey (2014) shows that the Sunny Isles segment is eroding at a much slower rate that in the pre-breakwater era. Some relatively minor erosion into the design template has been noted in the recent survey, but renourishment of this segment may still not be required for several more years.

A summary of monitoring survey data for Sunny Isles is provided in Table 2. This data represents project performance in the post-breakwater era only, since construction of this structure completely altered historic erosion rates and sedimentation patterns along much of the Sunny Isles shoreline. As seen in the table, the "post-breakwater" surveys were taken in 2005, 2007, 2009, 2011, and 2014. Although these surveys were selected in order to represent performance of the Sunny Isles beach fill in the post-breakwater-construction era, this time interval also was selected to exclude the main effects of the severe hurricane impacts of 2004-05. Large volumes of material were eroded from the upland beach profile and driven out into deep water during these events, and some evidence suggests that rebounding/recovery processes have occurred over several years following the impacts of these storms. These effects can be seen in some of the profile data, and are reflected in the large volume of "rebounding" observed during the relatively calm years of 2007 – 2009.

As seen in Table 2 the measured volumetric change rates along Sunny Isles are highly variable, both spatially and temporally. Annual change rates vary from a low of -105,576 cy/yr to a high of +238,005 cy/yr, depending on the survey interval selected. The overall longterm erosion rate along the 2.5-mile length of Sunny Isles as measured between 2005-2014 is +4,573 cy/yr. This rate represents a net accretion and is unsuitable for use as a basis of future volumetric projections. The calculated 2005-2014 rate excludes the erosional effects of the unusually severe hurricane seasons of 2004-05, but it includes the period of post-storm recovery, and may therefore underestimate the ‘true’ ongoing erosion rate.

Volumetric Changes Along Sunny Isles (2005 - 2014)

	2005 vs 2007		2007 vs 2009		2009 vs 2011		2011 vs 2014		2005 vs 2014	
R #	Vol (cy)	Vol/yr (cy/yr)	Vol (cy)	Vol/yr (cy/yr)	Vol (cy)	Vol/yr (cy/yr)	Vol (cy)	Vol/yr (cy/yr)	Vol (cy)	Vol/yr (cy/yr)
7										
	-9932	-5675	36730	25940	-28162	-12087	-1717	-624	-3081	-375
8										
	-557	-318	23349	16490	-26957	-11570	3491	1269	-675	-82
9										
	-11551	-6600	31396	22172	-33595	-14418	31786	11558	18036	2194
10										
	-32186	-18392	42433	29967	-18543	-7958	24301	8837	16005	1947
11										
	-14148	-8085	45258	31962	-28195	-12101	11193	4070	14108	1716
12										
	-586	-335	17378	12272	-24480	-10506	9256	3366	1568	191
13										
	-11304	-6460	30931	21844	-34554	-14830	24948	9072	10021	1219
14										
	-13012	-7436	33027	23324	-17069	-7326	10656	3875	13601	1655
15										
	-33620	-19211	31919	22541	-533	-229	-1825	-663	-4059	-494
16										
	-26626	-15215	25674	18131	-18103	-7769	-1924	-700	-20979	-2552
17										
	-3585	-2049	7445	5258	-9385	-4028	-1058	-385	-6583	-801
18										
	-20950	-11972	9424	6655	6330	2717	4671	1699	-526	-64
19										
	-6700	-3828	2050	1447	8244	3538	-3444	-1252	150	18
19+300										
TOTAL :	**-184757**	**-105576**	**337014**	**238005**	**-225003**	**-96568**	**110332**	**40121**	**37587**	**4573**

Table 2. Measured volumetric changes along Sunny Isles, post-breakwater period.

In order to gain a deeper perspective on the erosion rate, losses from the 2002 beach fill were calculated based on the April 2011 beach profile survey. Based on this survey, losses from the 2002 construction template equaled -447,330 cubic yards along the full length of Sunny Isles. Based on the 12-year interval between construction and survey, the calculated average annual rate is -37,280 cy/yr. This rate is higher than the long-term rate calculated above (2005-2015) in part because it includes the effects of the 2004-05 storms. The volume calculations represented in table 2 also extend further offshore than the construction template, and include accreted material in the nearshore bar system.

An examination of older erosion rates was conducted for comparison, even though these rates may no longer be applicable to this area due to the addition of the breakwater. From the 2001 Dade County Evaluation Study conducted by the COE, the average annual erosion rate along the entire 2.4-mile length of Sunny Isles was -4,200 cy/yr. Over half of this erosion occurred along the northernmost area of the project, which is the region that has now been stabilized by the construction of the breakwater. Excluding the northern half-mile from this analysis eliminated this erosive area and better represents ‘normal’ erosion through the main body of the Sunny Isles fill segment, and gives an average annual rate of -3,008 cy/yr.

To briefly summarize, volumetric change rates along Sunny Isles have proven to be extremely variable, ranging from highly erosive to highly accretionary, depending on the time interval examined. For the most part erosion rates tend to be relatively low along Sunny Isles but can become quite high, occasionally exceeding -100,000 cy/yr. A weighted average will be assumed in order to approximate the most realistic (but still conservative) value possible, and in an attempt to smooth out the extreme fluctuations observed in some of the monitoring data. A weighted average of -50,000 cy/yr will be selected based on the data presented above. This value best approximates the amount of actual erosion measured along the limits of the most recent (2002) beach fill, over a long (12-year) period. Since the effects of the storms of 2004-05 are included in the calculation of this value, -50,000 cy/yr should present a reasonably conservative estimate of future erosion rates along Sunny Isles through the remaining years of the project life.

Sunny Isles – Future Renourishment Needs. The Sunny Isles segment of the Dade County BEC & HP project was originally formulated with a 10-year design renourishment interval. The last renourishment of Sunny Isles has already been in place for over 12 years and portions of this segment of the project are currently in need of renourishment. Based on an examination of the most recent monitoring survey in January 2014, the southern half of Sunny Isles was in acceptable condition but the northern end of the project was eroded to the point that the design template was compromised in several areas. Renourishment of Sunny Isles is therefore recommended at this time, to prevent major damages in the event of a severe storm event.

At the time of the January 2014 survey, the northern portion of Sunny Isles was one of the more severely eroded segments of the Dade County project. The next renourishment of this segment would therefore be constructed as a top priority, and the soonest construction could occur would be in 2016. Following the 2016 renourishment it will be assumed that the projected erosion rate of -50,000 cy/yr will apply through the remaining years of the project life. The 10-year design renourishment interval will apply through the remaining years of project life as well, resulting in future renourishments in 2026 and 2036.

As shown in Table 3, the volume of material required to construct the 2016 fill will be 447,330 cy (as measured from the January 2014 survey), plus two years of future erosion (2014 to 2016) at the projected rate of -50,000 cy/yr. The total volume placed in 2016 will therefore be 447,330 + (2 x 50,000) = 547,330 cy. The volume placed in 2026 will be (10 yrs x 50,000 cy/yr) = 500,000 cy. The volume placed in 2036 will be (2 yrs x 50,000 cy/yr) = 100,000 cy, since only 2 years will remain in the project life in 2036. **The total volume of material required to renourish the Sunny Isles segment through the remaining 24 years of project life is therefore 1,147,330 cy.**

Sunny Isles	
Summary of Future Volumes	
Year	Volume, cy
2016	547330
2026	500000
2036	100000
Total	1147330

Table 3. Summary of future volume placements – Sunny Isles.

Projected erosion rates and future renourishment volumes will now be calculated for the “main“ segment of the BEC & HP project. Since the different sub-reaches of the main segment of the project behave in such different ways, this long reach will be broken into sub-reaches, and areas with different sedimentation characteristics will be analyzed separately. Specifically, different analyses will be presented for Haulover Park, Bal Harbour, Surfside, northern Miami Beach, and southern Miami Beach.

Haulover Park – Future Renourishment Rate. Haulover Park has historically been one of the least-erosive areas of the Dade County BEC & HP project. This segment was initially constructed in 1978, and even during initial construction the volume of fill placed was only 300,000 cy along the 1.1-mile length of this segment. Two small-scale renourishments (<50,000 cy) were performed in 1980 and 1984 using material dredged from the adjacent Federal navigation channel at Bakers Haulover Inlet. One relatively large-scale (235,000 cy) renourishment was performed in 1987, and this segment of the project has not required renourishment since that time.

Table 4 shows volumetric changes measured along Haulover Park between 2005 and 2014, which excludes the period of the 2004-05 storms. Except for the 2005-07 interval, all survey intervals show accretion along this reach. The 2005-2007 interval was likely influenced by the hurricanes of 2005; this erosion rate is not indicative of the long-term performance of this project segment and should not be used to establish longterm future volumetric projections. Neither should the accretionary values during the other time periods be used, other than to provide a verification that the erosion along Haulover Park tends to be low, relative to the other parts of the Dade County project.

The accretionary trend along Haulover Park is largely attributed to the net southward transport of sediment along the Dade County coastline. Southward-directed material tends to “pile up” to the north of the Bakers Haulover Inlet north jetty; corresponding erosion is constantly noted on the south side of the inlet along the Bal Harbour shoreline (see next section).

As an alternative analysis, an examination of older survey data from the 2001 Evaluation Report was conducted for comparison purposes. Based on survey data from 1990-2000, a measured erosion rate of -5,436 cy/yr was calculated along Haulover Park. As with the more recent survey analysis presented above, there was a great deal of variation in erosion rates within this 10-year period, depending on the survey interval selected.

	Volumetric Changes Along Haulover Park (2005 - 2014)									
	2005 vs 2007		2007 vs 2009		2009 vs 2011		2011 vs 2014		2005 vs 2014	
R #	Vol (cy)	Vol/yr (cy/yr)	Vol (cy)	Vol/yr (cy/yr)	Vol (cy)	Vol/yr (cy/yr)	Vol (cy)	Vol/yr (cy/yr)	Vol (cy)	Vol/yr (cy/yr)
19+300										
	-16102	-9201	4926	3478	19814	8504	-8278	-3010	360	1590
20										
	-18500	-10571	11630	8213	28204	12105	-8889	-3232	12444	3993
21										
	-26357	-15061	22853	16139	-682	-293	24420	8880	20234	-352
22										
	-23135	-13220	11746	8295	11034	4735	16871	6135	16515	340
23										
	-23277	-13301	23781	16795	18685	8019	-2259	-821	16931	3543
24										
	-14773	-8442	23050	16278	9359	4017	-2000	-727	15636	3140
25										
	7400	4229	-785	-555	432	185	6811	2477	13859	1174
26										
TOT, HP:	-114745	-65568	97200	68644	86846	37273	26677	9701	95978	11676

Table 4. Measured Volumetric Changes along Haulover Park, 2005-2014.

Finally, another point of view was gained from an examination of the current condition of the project (as per January 2014 survey) versus the construction template, which shows that little or no fill is required to rebuild the construction template along Haulover Park at this time. This construction template was last filled in 1987 with the placement of 235,000 cy, and most of this material remains within the fill template today. This reach of the project was formulated with a five-year renourishment interval. If it is assumed that this placement volume erodes within the next five years to the point where renourishment is required, an erosion rate could be established based on the volume required to rebuild the same template, and the time interval between renourishments. Based on the present state of the project this scenario appears unlikely, so this analysis would present a “worst-case” scenario for this segment of the project. Assuming a renourishment project in 5 years (2019), a time interval of 32 years would exist between subsequent renourishments. Further assuming the renourishment volume would be about the same as in the 1987 project, 235,000 cy would be replaced. The resulting erosion rate is therefore calculated to be -7,340 cy/yr.

In order to remain conservative, the volumes developed in this section are averaged between the ‘low’ values (generally in the accretionary range) and the one ‘high’ erosive value from the 2005-07 interval (-65,568 cy/yr). Due to the overwhelming long-term tendency of this area to accrete, a weighted average of -15,000 cy/yr will be adopted. This value may still be conservative (high), but does account for the possibility of uncharacteristic erosion in the event of severe storm impacts.

Haulover Park – Future Renourishment Needs. Haulover Park (and the remaining reaches of the project to the south) were originally formulated with a 5-year renourishment interval. Historically the renourishment interval at Haulover Park has far exceeded this value. Based on past experience it is very likely that this segment of the project may only require one other renourishment (if any) through the remaining 11 years of the life of this segment of the project.

Assuming a “worst-case’ scenario in which renourishment would be required in the year 2019, the volume required at that time to extend the beach fill through the end of project life in 2025 would be (6 x 15,000 cy/yr) = 90,000 cy. As seen in Table 5, only one renourishment event would occur in the remaining years of project life for this segment, so **the total volume of material required through the remaining 11 years of project life for the Haulover Park segment is 90,000 cy.**

Haulover Park	
Summary of Future Volumes	
Year	Volume, cy
2019	90000
Total	90000

Table 5. Summary of future volume placements – Haulover Park.

<u>Bal Harbour – Future Renourishment Rate</u>. Bal Harbour was the first segment of the Dade County BEC & HP project to be constructed. Initial construction was completed in 1975, and this segment has remained one of the most rapidly-eroding areas of the project to date. This is primarily due to its location on the south side of Bakers Haulover Inlet. The inlet interrupts the predominantly southward flow of sediment, creating a sediment deficit along the Bal Harbour shoreline.

Table 6 shows volumetric changes measured along Bal Harbour between 2005 and 2014, which excludes the period of the 2004-05 storms. As with most of these analyses, the 2005-07 interval represents the highest erosion rates measured during the period of analysis. The erosion rates presented in Table 6 fall within a much narrower range than the rates from the previously analyzed project segments. Rates tend to average to the -30,000 cy/yr range, with the highest losses (-48,548 cy/yr) measured between 2005-07. Note that Bal Harbour was renourished shortly after the 2014 survey was completed; the data contained in table 6 represents pre-project conditions.

Volumetric Changes Along Bal Harbour (2005 - 2014)										
	2005 vs 2007		2007 vs 2009		2009 vs 2011		2011 vs 2014		2005 vs 2014	
R #	Vol (cy)	Vol/yr (cy/yr)	Vol (cy)	Vol/yr (cy/yr)	Vol (cy)	Vol/yr (cy/yr)	Vol (cy)	Vol/yr (cy/yr)	Vol (cy)	Vol/yr (cy/yr)
27										
	-33999	-19428	1174	829	-6825	-2929	-11176	-4064	-50826	-6183
28										
	-24167	-13810	-7324	-5173	-42197	-18110	-28058	-10203	-101747	-12378
29										
	-6868	-3925	-2378	-1679	-15903	-6825	-23159	-8421	-48307	-5877
30										
	-11673	-6670	-2455	-1734	-5126	-2200	-22140	-8051	-41394	-5036
31										
	-8253	-4716	-6442	-4550	3919	1682	-9225	-3355	-20001	-2433
31+520										
TOTAL :	**-84959**	**-48548**	**-17426**	**-12306**	**-66132**	**-28383**	**-93758**	**-34094**	**-262275**	**-31907**

Table 6. Measured Volumetric Changes along Bal Harbour, 2005-2014.

A check of older erosion rates from the 2001 Evaluation Report was conducted for comparison purposes. Based on survey data from 1990-2000, an erosion rate of -54,602 cy/yr was measured. More variation between the individual survey intervals was noted in this older database than with the more recent database presented in Table 6.

As a final check the volume that would be required to reconstruct the 2003 construction template along Bal Harbour was calculated. A total of 228,345 cy would be required, based on analysis of the January 2014 survey (a pay volume of 235,732 cy was actually placed a few months later). Adding in the 33,000 cy placed along Bal Harbour in 2010 from the dredging of Bakers Haulover Inlet and averaging this volume over the 11-year period that it took to erode the template, an erosion rate of (228,345 + 33,000) / 11 yrs = -20,760 cy/yr is calculated.

The three rates calculated from three different databases/methodologies are more internally consistent than the rates observed north of Bakers Haulover Inlet. In general, approximate rates of 20,000, 30,000 and 55,000 are indicated. In order to be conservative for future renourishment needs the ‘high’ rate of 55,000 cy/yr is selected for Bal Harbour, one of the most highly-erosive regions of the project.

Bal Harbour – Future Renourishment Needs. Bal Harbour (and the remaining reaches of the “main” segment of the project) were originally formulated with a 5-year renourishment interval. Based on the selected erosion rate of -55,000 cy/yr, the future renourishment needs of this segment are calculated as follows :

An analysis of the January 2014 survey shows that the Bal Harbour segment was in a highly-eroded condition at that time. However, a complete renourishment of this segment was completed in the spring of 2014 and at the present time the project is in a fully nourished condition. In accordance with the originally-formulated 5-year renourishment interval the next projected renourishment would occur in 2019, and would require the placement of (5 yrs x 55,000 cy/yr) = 275,000 cy. Then one additional small-scale renourishment of 55,000 cy would be required in 2024, prior to the end of the period of Federal participation in 2025.

The proposed borrow area for Bal Harbour is the Bakers Haulover Inlet ebb shoal. As will be discussed in later sections of this report, this borrow area has been used to renourish Bal Harbour in the past, and has been shown to refill at an average rate of about +30,000 cy/yr. For long-range planning purposes the ebb shoal can be assumed to produce 300,000 cy of borrow material every 10 years, so on a long-term basis this borrow area can be used to supply every other renourishment of Bal Harbour. Material dredged from the shoal is usually earmarked for placement along the Bal Harbour shoreline in order to augment natural sand bypassing around Bakers Haulover Inlet and provide nourishment to all beaches south of the inlet. Also, due to the proximity of the Bal Harbour fill area to the Bakers Haulover Inlet borrow area, this operation promotes minimum cost and maximum efficiency, versus transporting the material to more distant fill areas.

The 2014 renourishment was constructed using material from this ebb shoal area. Since the portion of this borrow area permitted for excavation was depleted at that time, it could not be used as a sole source of material for Bal Harbour until 2024. Therefore the material required to construct the 2019 renourishment would be provided from other sources.

As discussed above, two additional renourishments will be required along the Bal Harbour shoreline over the remaining 11 years of project life. The project was fully renourished in 2014; the next renourishment would place 275,000 cy in 2019. Then one final renourishment of 55,000 cy would be required in 2014. However, due to the high cost of mobilizing dredging equipment it is not advisable to place only one single year's worth of renourishment. A much more economically efficient plan would be to place the extra year's volume of material during the 2019 renourishment. This strategy would result in mobilization cost savings of $1M or more.

In addition, some material may become available from the adjacent Bakers Haulover flood shoal / IWW dredging. As will be discussed later in this report this region produces about 9,600 cy/yr on average, and this material is typically placed on the beaches adjacent to the inlet, including Bal Harbour. If available, this material would be placed preferentially on Bal Harbour, and would reduce the volume required from other sources.

Therefore, as shown in table 7, only one additional renourishment event is proposed along Bal Harbour during the remaining years of project life. **The total volume of future placement along Bal Harbour during the 11 remaining years of project life is 330,000 cy.**

Bal Harbour Summary of Future Volumes	
Year	Volume, cy
2019	330000
Total	330,000

Table 7. Summary of future volume placements – Bal Harbour.

Surfside – Future Renourishment Rate. Surfside has historically performed very well, in part because it receives adequate nourishment due to the predominantly southward transport of sediment from Bal Harbour to the north. The Surfside segment of the project was initially constructed in 1978, and has only been renourished once, in 1999. In spite of the relatively low erosion rates along Surfside, the area is currently in need of renourishment.

Table 8 shows volumetric changes measured along Surfside between 2005 and 2014. Again, the 2005-07 interval experienced the highest annual erosion rates measured during this period of analysis. The erosion rates presented in Table 8 show more variability than those observed at Bal Harbour, with some accretion observed in the 2007-09 interval. The average annual losses from 2005-2014 were relatively low, at -17,724 cy/yr.

Volumetric Changes Along Surfside (2005 - 2014)										
	2005 vs 2007		2007 vs 2009		2009 vs 2011		2011 vs 2014		2005 vs 2014	
R #	Vol (cy)	Vol/yr (cy/yr)	Vol (cy)	Vol/yr (cy/yr)	Vol (cy)	Vol/yr (cy/yr)	Vol (cy)	Vol/yr (cy/yr)	Vol (cy)	Vol/yr (cy/yr)
31+520										
	-7824	-4471	-6108	-4313	3716	1595	-8746	-3180	-18962	-2307
32										
	-16075	-9186	-5323	-3759	-5394	-2315	-7696	-2799	-34488	-4196
33										
	-8377	-4787	-6578	-4645	-15442	-6628	-6184	-2249	-36582	-4450
34										
	-7763	-4436	708	500	-13928	-5978	-20733	-7539	-41717	-5075
35										
	-11642	-6652	14012	9896	-19770	-8485	2371	862	-15028	-1828
36										
	-2104	-1202	3592	2537	-5922	-2542	5516	2006	1082	132
36+420										
TOTAL :	**-53786**	**-30735**	**304**	**215**	**-56741**	**-24352**	**-35472**	**-12899**	**-145695**	**-17724**

Table 8. Measured Volumetric Changes along Surfside, 2005-2014.

The older erosion rates from the 2001 Evaluation Report were examined for comparison. Surfside is located approximately midway between the structures added in 2001-02, and littoral processes in the area should be minimally affected by those structures. Based on survey data from 1990-2000, an annual erosion rate of -43,228 cy/yr was measured. Less variation in volumetric change values was seen across the 1990-2000 time interval than across the 2005-2014 interval.

The project was renourished to its full construction template in the one renourishment of this area, in 1999. As a third method of analysis, the volume that would be required to reconstruct the 1999 construction template along Surfside was calculated. A total volume of 425,460 cy would be required, based on the January 2014 survey. Averaging this volume over the 15-year period that it took to erode, an average annual erosion rate of -28,360 cy/yr is calculated.

Based on the foregoing discussion an erosion rate of -45,000 cy/yr is selected for future volumetric projections along Surfside. This represents only a slight rounding-up of the measured erosion rate from the 1990-2000 survey analysis, and gives a slightly conservative estimate of future renourishment needs.

Surfside – Future Renourishment Needs.   Based on the projected erosion rate of -45,000 cy/yr and the design 5-year renourishment interval, the following maintenance schedule is provided :

Based on the present condition of the Surfside shoreline, renourishment is not critical at this time but should be performed in the relatively near future. Assuming construction in 2017, the volume of placement would be 425,460cy (based on the January 2014 survey), plus 3 yrs x -45,000 cy/yr = 135,000 cy, giving a total volume of 560,460 cy.

Future renourishments would then consist of placing 225,000 cy at 5-year intervals. Following construction in 2017, the next renourishment would be required in 2022, and would place only (3 yrs x 45,000) = 135,000 cy of material, since the project life expires in 2025. As shown in table 9. **the total volume required along the Surfside segment throughout the remaining project life is 695,460 cy.**

Surfside	
Summary of Future Volumes	
Year	Volume, cy
2017	560460
2022	135000
Total	695460

Table 9. Summary of future volume placements – Surfside.

<u>*Miami Beach – Future Renourishment Rate*</u>. Miami Beach is the longest segment of the project, with a length of about 7.5 miles. It is also the most complex region, because of the variety of the coastal environment (and the littoral characteristics) along its length. Much of the region is moderately erosional. However some areas are highly erosional, some areas are relatively stable, and still other areas are consistently accretional.

The entire southern reach of the project is consistently accretional and can be completely excluded from any consideration of ever requiring any future beach renourishments. Sediment is transported predominantly from north to south along the Dade County shoreline, and the southern reach of the project forms a large embayment that basically functions as an impoundment basin. Southbound sediment is transported into this area and is blocked from further southward transport by the north jetty at Government Cut (Miami Harbor entrance). These jetties also block wave energy from the south, preventing the northward transport of material out of the area.

The breakpoint between the erosive area and the accretional area is located near survey monument R-65. This monument is located about 2 miles north of the south end of the project. Therefore, of the 7.5-mile length of the Miami Beach segment, the southern 2 miles are consistently accretional and the northern 5.5 miles are consistently erosional. The Miami Beach segment of the project will be divided into these two sub-reaches in this analysis because of the great differences in sedimentation characteristics of these two areas. These two reaches will be addressed separately in the following discussion, beginning with the southern reach :

<u>Southern Sub-Reach of Miami Beach Segment</u>. This accretional area begins at about DNR monument R-65 and extends to the south end of the project at R-74, a distance of about 2 miles. Material steadily impounds into this area, and over the years the berm has grown to a width of over 400 feet in most areas. Due to the constant accretion of southward-moving sediment, this area has never required renourishment since initial project construction. In recent years this area has been used as a borrow source for renourishing some of the erosional beaches to the north. Since these operations move material from this accretionary

zone at the south end of the project back updrift, these are known as ‘backpassing’ operations.

Several rates of impoundment have been calculated for this region in various studies over the years. One of the more widely-accepted values was calculated in the report “*Dade County Regional Sediment Budget, January 1997*”, by Coastal Systems International, Inc. From the county-wide sediment budget developed in this report, deposition rates of +46,000 cy/yr and +45,000 cy/yr were calculated along the northern and southern halves of this accretionary sub-reach, respectively. Therefore the total deposition along the southern sub-reach of the Miami Beach segment of the project was +91,000 cy/yr. This value was based on an analysis of monitoring surveys from 1980-1996. It should be noted that this period of analysis was prior to the 1999 sand-tightening of the north jetty at Government Cut. The jetty was semi-porous at that time and large volumes of sand were known to pass through the structure into the navigation channel prior to sand-tightening.

An updated survey analysis was recently conducted, using surveys from 2005-2014. A summary of the volumetric changes measured from these surveys is presented in Table 10. As seen in this table, annual measured shoaling rates vary from 20,300 cy/yr to 110,900 cy/yr, bracketing the value calculated in the 1997 *Sediment Budget* report. The average value from all surveys was only 54,491 cy/yr during this interval however. It should be noted that these values do not include the material borrowed from the region during Dade County’s two backpassing operations. At the present time no reliable information can be obtained from Dade County regarding the volume of material removed from Lummus Park during the County’s 2007 backpassing operation. The shoaling rate of +51,300 cy/yr therefore excludes this volume and is overly conservative (low). The longterm rate of +91,000 cy/yr from the sediment budget has long been accepted and has proven reasonable over the years.

A rounded rate of +90,000 cy/yr is adopted as the shoaling rate along the full length of Lummus Park. Historically, the rates of deposition are relatively uniform across the length of Lummus Park, so it can be reasonably assumed that this material is uniformly distributed along the southern 2 miles of the project. Since only the northern half (closest to the fill areas) is normally used as a borrow source, the allowable rate of borrowing material is half of the total rate, or 45,000 cy/yr. If the limits of the borrow operation are altered significantly, this rate should be adjusted accordingly.

Since this area consistently accretes and rarely if ever erodes, **the future renourishment volume needs along southern Miami Beach are zero**; the area would be used as an occasional borrow source instead. Since it is seldom economically practical to mobilize construction equipment for very small beach fill projects, this area should be allowed to accumulate a relatively large volume of sediment before backpassing operations are conducted. A practical backpassing operation planned in accordance with this projected future shoaling rate could produce a volume of about 225,000 cy every 5 years with no significant detrimental effects to this reach of shoreline.

	Volumetric Changes Along Miami Beach (2005 - 2014) (Southern Sub-Reach)									
	2005 vs 2007		2007 vs 2009		2009 vs 2011		2011 vs 2014		2005 vs 2014	
R #	Vol (cy)	Vol/yr (cy/yr)	Vol (cy)	Vol/yr (cy/yr)	Vol (cy)	Vol/yr (cy/yr)	Vol (cy)	Vol/yr (cy/yr)	Vol (cy)	Vol/yr (cy/yr)
65										
	238	136	20042	14154	14668	6295	18339	6669	53287	6483
66										
	-10850	-6200	33052	23342	28938	12420	2738	996	53878	6555
67										
	-5765	-3294	28862	20383	15872	6812	20621	7498	59590	7249
68										
	6390	3652	10491	7409	17200	7382	24989	9087	59071	7186
69										
	-9233	-5276	7395	5223	21642	9288	-998	-363	18806	2288
70										
	2413	1379	10551	7451	-38	-16	5610	2040	18535	2255
71										
	26152	14944	13552	9571	-8325	-3573	24304	8838	55683	6774
72										
	20485	11706	15658	11058	-3259	-1399	44643	16234	77526	9431
73										
	5692	3252	17425	12306	-3050	-1309	31470	11444	51537	6270
74										
TOT,sMB:	35523	20299	157027	110895	83647	35900	171716	62442	447913	54491

Table 10. Measured Volumetric Changes along the southern sub-reach of Miami Beach (Lummus Park), 2005-2014.

Northern Sub-Reach of Miami Beach Segment. The 5.6-mile reach of Miami Beach north of DNR-65 is mostly erosional, and to varying degrees. Several erosional hotspot areas are known to exist along this reach, including the 32nd and 63rd St regions (for which structural solutions have been developed and renourishments recently completed), and the 46th and 55th St areas (which have no structural solutions, and no beach fill has been placed recently). The areas between hotspots are generally stable to moderately erosional. Volumetric changes over the 2005-2014 period are shown in Table 11. The yellow stripes along the right-hand margin show the limits of the erosional "hotspot" areas.

As seen in this table, erosion rates during this period were highly variable, both spatially and temporally. The longterm volumetric change rate, averaged over all time periods over the full length of the reach, is only a net accretion of +5,657 cy/yr. This rate represents the natural shoreline change with all beach fills removed, and is largely a result of the considerable gains in material seen during the 2007-2009 interval. However, this positive rate can be misleading : areas of high erosion are balanced by areas of low erosion or accretion. In practice, material tends to be transported out of the hotspot areas (due to wave energy focusing and other factors discussed in the 2001 *Evaluation Report*) and deposited in the regions between the hotspots. These "in-between" areas tend to erode more slowly, or not at all.

	Volumetric Changes Along Miami Beach (2005 - 2014) *(Northern Sub-Reach)*									
	2005 vs 2007		**2007 vs 2009**		**2009 vs 2011**		**2011 vs 2014**		**2005 vs 2014**	
R #	Vol (cy)	Vol/yr (cy/yr)	Vol (cy)	Vol/yr (cy/yr)	Vol (cy)	Vol/yr (cy/yr)	Vol (cy)	Vol/yr (cy/yr)	Vol (cy)	Vol/yr (cy/yr)
36+420										
	-3201	-1829	5465	3860	-9010	-3867	8393	3052	1646	200
37										
	-8524	-4871	3978	2809	6150	2639	-2151	-782	-547	-67
38										
	-13092	-7481	3601	2543	1186	509	-2055	-747	-10360	-1260
39										
	-1661	-949	3895	2751	-11583	-4971	5822	2117	-3526	-429
40										
	-19	-11	-1582	-1117	2185	938	5104	1856	5688	692
41										
	2136	1221	-6262	-4423	-14158	-6077	-1609	-585	-19893	-2420
42										
	1651	943	-9991	-7056	-11790	-5060	-20650	-7509	-40780	-4961
43										
	-25315	-14466	27475	19403	-7259	-3115	-17424	-6336	-22524	-2740
44										
	-29345	-16768	33127	23395	-25885	-11109	527	192	-21576	-2625
45										
	-6066	-3466	-1444	-1019	-14848	-6373	5283	1921	-17075	-2077
46										
	-4246	-2426	22645	15992	-15483	-6645	27923	10154	30839	3752
47										
	3993	2282	28741	20298	-10547	-4527	18086	6577	40274	4899
48										
	307	176	22303	15751	-6055	-2599	13284	4831	29840	3630
49										
	-7834	-4476	18260	12896	-22343	-9589	13479	4901	1563	190
50										
	-11490	-6566	13870	9795	-14651	-6288	-2277	-828	-14549	-1770
51										
	-4840	-2766	10538	7442	-418	-179	-11682	-4248	-6402	-779
52										
	-6303	-3602	20935	14785	-12896	-5535	-8225	-2991	-6489	-789
53										
	-21394	-12225	37545	26515	-27316	-11724	-36490	-13269	-47655	-5798
54										
	2326	1329	11104	7842	-17314	-7431	-62779	-22829	-66663	-8110
55										
	1148	656	5641	3984	-7132	-3061	16360	5949	16018	1949
56										
	2171	1240	13708	9681	5124	2199	5788	2105	26791	3259
57										
	18852	10772	14907	10528	10352	4443	22241	8088	66352	8072
58										
	10934	6248	6938	4899	8510	3652	27528	10010	53909	6558
59										
	6464	3694	7407	5231	3266	1402	16699	6072	33835	4116
60										
	689	394	44114	31154	-17490	-7506	-11543	-4198	15769	1918
61										
	-14230	-8132	50129	35402	-18323	-7864	-8028	-2919	9548	1162
62										
	-23306	-13318	16090	11363	-6159	-2643	1986	722	-11389	-1386
63										
	-12398	-7084	5046	3564	-11093	-4761	1566	569	-16878	-2053
64										
	-2266	-1295	15192	10728	-21260	-9125	29071	10571	20737	2523
65										
TOT, nMB:	-144860	-82777	423378	298996	-266242	-114267	34227	12446	46503	5657

Table 11. Measured Volumetric Changes along the northern sub-reach of Miami Beach, (R-65 to southern limit of Surfside) 2005-2014.

It should be emphasized that the values in this table represent the volumetric changes between adjacent profiles over the time intervals specified, and do not necessarily correspond to the present-day condition of the project. Since all volumes of beach fill placements have been removed to allow the calculation of ‘natural’ background erosion/accretion rates, some of the hotspot areas indicated in this table are actually in acceptable condition at this time, due to the placement of recent beach fills.

Table 11 (and all of the other volumetric change tables presented in this report) therefore represent the performance of the Dade County project in the absence of beach fill placements. Some changes to these sedimentation patterns along northern Miami Beach can be expected in the future however. Construction of the Section 227 Reefball breakwater at 63rd St (R-46) will reduce erosional losses along this northernmost hotspot and will affect the surrounding areas as well. And according to survey data and field observations by the local sponsor, the region around the 32nd St breakwaters (R-60) may finally be beginning to stabilize as natural bypassing of the structure has finally begun. In order to be conservative however, the rates provided in Table 11 will be selectively used to establish future erosion rates.

At the time of the January 2014 survey three of the hotspot areas had been recently (2012) renourished, and were in acceptable condition. The primary remaining hotspots along this reach of shoreline were located at 55th St (R-49) and 46th St (R-54). Each of these areas have been recognized as rapidly-eroding sections of the project for some time. Since each of these areas of rapid erosion behave much differently than the remaining sections of the northern Miami Beach segment of the project, they will be combined into one “hotspot” renourishment group. The remaining areas of the northern Miami Beach segment of the project erode much more slowly (or not at all), and will be combined into a “non-hotspot” renourishment group. Different erosion rates and renourishment schedules will be developed for each of these two groups.

In Table 11, the “hotspot” reaches are delineated by the orange stripe along the right-hand side of the table. Each orange-highlighted reach corresponds to an area of the project that has historically experienced the heavy, persistent erosion. From north to south these are : the 63rd St area, the 55th/46th St areas, and the region immediately south of the 32nd St breakwater. The “non-hotspot” reaches in table 11 are the remaining sections of the northern Miami Beach segment outside of the orange-highlighted areas.

Within the hotspot areas, the average annual erosion rates during the period 2005-2014 are -14,820 cy/yr for the 63rd St area, -17,250 cy/yr for the 55th/46th St areas, and -3,440 cy/yr south of the 32nd St breakwater. The total annual erosion rate of these hotspot areas is therefore -35,510 cy/yr.

The areas between the hotspots are much more stable, primarily because some material eroded from the hotspots renourishes these areas. The annual erosion rate of the non-hotspot areas will be computed by averaging only the negative values; accreting areas will not be counted. Selecting only the erosive values from the summary column of Table 11 yields an annual erosion rate of -2,000 cy/yr. It is suspected that this value is unreasonably low, based on historic knowledge of the project’s performance.

An examination of the older databases shows that the erosion rate along northern Miami Beach as computed in the 1997 CSI sediment budget is -33,000 cy/yr, and this value includes 'hotspot' erosion as well as the regions adjacent to the hotspots. Again, this CSI dataset is based on survey data from 1980-1996. The values presented in this study would tend to corroborate the low erosion values observed along the areas between hotspots in the 2005-2014 database.

However, a third analysis was performed in the 2001 Evaluation Report. Based on surveys taken between 1990 and 2000, an annual erosion rate of -203,100 cy/yr was measured along the northern reach of Miami Beach. The large difference between this value and the previous two datasets is difficult to reconcile, and on closer inspection several factors may contribute to this anomalous erosion rate.

Each of the later two erosion rates spans a period of time before the construction of the 32nd St breakwaters, and includes the effects of Hurricane Andrew. Andrew impacted the Dade County shoreline severely, resulting in high losses of material that were eventually recommended for replacement under FEMA storm damage assistance. Construction of the 32nd St breakwaters had some adverse downdrift effects but still served to lower the overall erosion rate along this region of Miami Beach; this would tend to render the pre-breakwater erosion rates obsolete. Finally, both of these erosion rates included the hotspot areas as well as the non-hotspot areas. Due to these factors the values of the second and third methods may have limited applicability to today's sedimentation patterns, and will be weighted very lightly when determining the overall renourishment rate for the northern Miami Beach segment.

Given the range of erosion rates calculated for this reach of the project, the most prudent option may be to perform a weighted average of these values. The "hotspot" erosion rate from the first two sources will be rounded upward from the -35,000 range to -50,000 cy/yr in order to yield a more conservative estimate. The non-hotspot areas in the post-breakwater era have been demonstrated to erode at a very low rate. However, the only directly measured value is -2,000 cy/yr (from the 2005-2014 interval). This rate will be inflated to -25,000 cy/yr to account for uncertainties due to potential future storm impacts, and to provide a conservative estimate of future losses. This will yield a total annual erosion rate of -75,000 cy/yr for the northern Miami Beach segment. This value will be used to calculate future renourishment volumes, but is likely to decrease when the Reefball Breakwater is completed at 63rd St.

Miami Beach – Future Renourishment Needs. As discussed above, the Miami Beach segment of the Dade County BEC & HP project can be divided into two very different segments: south of R-65 the project is consistently accretional; north of R-65 project performance is highly variable, with limited portions being highly erosional. Monitoring data and other survey analyses have produced an average erosion rate for the area north of R-65 that will be used to project future renourishment needs through the remaining 11 years of project life. To the maximum extent possible, the southern (accretional) reach of the project will be used as a source of fill for some of these operations.

In practice each of the hotspot areas erode so rapidly that they usually begin to encroach on the design template within about 5 years. Assuming that this continues to be the case, renourishments of the hotspot areas will be performed every 5 years for the remainder of the project life. Note that this is consistent with the originally predicted renourishment interval. Due to their much slower erosion rates, the non-hotspot regions can be renourished every 10 years.

Currently, the two remaining hotspots that were not replenished during the 2012 renourishment are in critical need of renourishment, with erosion encroaching upon the design template. These two areas, near 46th and 55th Streets, would be renourished as a top priority in 2016. As shown in Table 1, at the time of the January 2014 survey a total volume of 402,250 cy will be required to restore these two hotspot areas to the full construction template. Factoring in 2 years of additional erosion until the time of construction (pro-rating by alongshore distance, excluding the recently-nourished areas at 63rd and 32nd Streets) yields an additional 22,760 cy/yr x 2 yrs = 45,520 cy. Further adding in the volume required to restore the remaining hotspot areas (which were renourished in 2012) would bring all of the hotspot areas onto the same renourishment schedule, which would significantly reduce future dredge mobilization costs. Again pro-rating by alongshore distance the volume eroded each year using the selected erosion rate is -27,240 cy/yr. Four years of erosion will have affected these areas by the time of construction, resulting in a required volume of 108,960 cy. The total volume required to repair all hotspots in 2016 is therefore 402,250 + 45,520 + 108,960 = 556,730 cy. Based on a 5-year interval the next renourishment would be required in five years, in the year 2021. Since the period of Federal participation expires in 2025, only 4 years' renourishment would be required in 2021, resulting in a placement of only 200,000 cy at that time.

For the remaining areas outside of the erosional hotspots, a volume of 606,100 cy is currently required to rebuild the construction template based on the January 2014 survey. At this time none of these non-hotspot areas of the project are critically eroded or require immediate replenishment, and construction can be deferred for several years. Based on present project conditions the next renourishment of these areas could occur in 2019. This would allow the large mobilization cost to be shared with the projected 2019 renourishments at Haulover Park and Bal Harbour.

The schedule and volumes of all remaining fill placement events are summarized in table 12. In the far right-hand column is a code designating the fill location as either within the hotspot area ("HS") or between the hotspots ("B"). **The total volume required along the northern Miami Beach segment through the remaining 11 years of project life is 1,362,830 cy**. This volume includes all "hotspot" and "non-hotspot" areas.

Northern Miami Beach			
	Summary of Future Volumes		
Year	Volume, cy		Area
2016	556730		HS
2019	606100		B
2021	200000		HS
Total	1,362,830		

Table 12. Summary of future volume placements – northern Miami Beach.

Total Volumes Required.
The total volume of placement over the remaining years of the project life is 1,147,330 cy for the Sunny Isles segment of the project, and 2,478,290 cy for the "main" segment of the project (Haulover Park south to Government Cut). The total volume required along the entire length of the Dade County BEC & HP project is therefore 3,625,620 cy. A detailed volumetric summary is provided in Table 13 below. Table 13 is a compilation of the data presented in tables 3, 5, 7, 9, and 12.

	SUMMARY OF PROJECTED RENOURISHMENT EVENTS		
	Dade County BEC & HP		
Year	Renourishment Event	Placement Area	Volume, cy
2016	Sunny Isles, Miami Beach Hotspots	R7-R26; R41-46; R50-55; R60-64	1,104,060
2017	Surfside	R32-R36	560,460
2019	Haulover, Bal Harbour, Miami Bch (non-HS)	R27-R31	1,026,100
2021	Miami Beach (HS)	R41-46; R50-55; R60-64	200,000
2022	Surfside	R32-R36	135,000
2025	----- End of period of Federal participation, Main Project segment ---		
2026	Sunny Isles	R7-R26	500,000
2036	Sunny Isles	R7-R26	100,000
2038	----- End of period of Federal participation, Sunny Isles segment ---		
		TOTAL :	**3,625,620**

Table 13. Summary of all future projected dredging events for the Dade County BEC & HP project.

Figure 1 illustrates the beach fill placements that have been constructed over the history of the Dade County BEC & HP project, as well as the predicted future placements. A plan view aerial photograph of the Dade County shoreline is shown along the left side of the graphic for orientation. Beach fills corresponding to the specific areas of placement are shown across the right side of the graphic. A timeline extends across the top of the figure to show the year of placement of each fill event. The red dashed line represents the present year – 2014. Previously-completed beach fills are shown on the left side of this red line; future projected fill placements are shown on the right side. The end of the period of Federal participation for each of the two project authorizations is indicated by the purple dashed line. The volumes of fill indicated in this analysis exclude any effects of sea level rise. Additional fill volumes required in the event of accelerated sea level rise scenarios are computed in the "Sea Level Rise" section of the main LRR report.

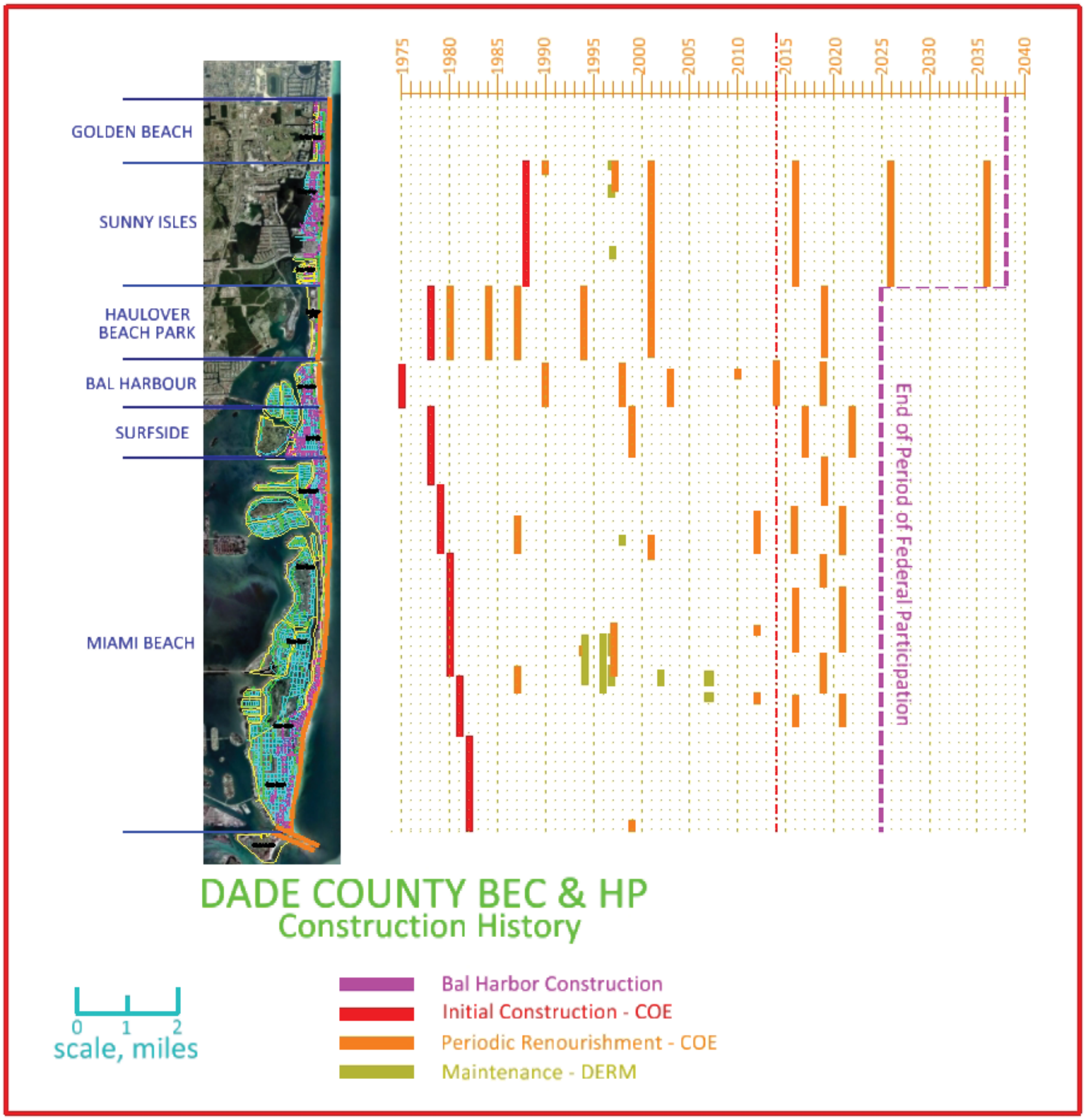

Figure 1. Schematic of past and future (projected) renourishment events.

Borrow Area Analysis.

Borrow areas throughout the coastal regions of Dade County are essentially depleted. At this time all of the offshore borrow areas that have traditionally been used to renourish the Dade County BEC have been completely exhausted. The only other borrow sources known to exist along the Dade County coastal areas are the ebb shoal at Bakers Haulover Inlet, the interior shoals and navigation channels inside the inlet, and Lummus Park, an accretionary region of the shoreline located at the south end of the BEC project.

All three of these borrow areas have been used previously to obtain beach-quality sand for beach placement. The interior shoals and Federal navigation channels at Bakers Haulover Inlet have been dredged approximately once every 5 years, and most material dredged from these shoals is suitable for beach placement. The ebb shoal has been used as a borrow source twice, during the 2003 and 2014 renourishments of Bal Harbour. Lummus Park (a.k.a. "South Beach") has been used several times by both the Corps and the Sponsor. In each case sand from this wide, accretionary beach has been backpassed to erosive reaches of shoreline located short distances to the north.

Each of these three borrow areas has a relatively low capacity, especially when compared to the much larger offshore borrow areas that have been the source of material for construction and maintenance of the project up to this point. Unlike the now-depleted borrow sites offshore of Miami-Dade County, each of these three inshore areas tend to refill naturally with sediment after they are dredged. Ample time must be allowed to pass between successive dredging events however, to allow these nearshore sites to recover.

Alternative offshore borrow sites with adequate capacity to renourish the Dade County project through the end of the period of Federal participation have now been identified, as part of this study. These sites (SL10-T41 and M4-R105) lie over 100 miles north of the project in Borrow Area "Zone C", which extends 90 to 135 miles north of the project area. These sites are generally more expensive to utilize than local sand sources due primarily to the increased transport distance. It is therefore preferable to make maximum usage of these few remaining nearshore local sources when possible. Each of these three remaining local nearshore borrow sites have limited capacity, and are best suited for infrequent, and/or smaller-volume projects. Each of these local sites will be analyzed with regard to capacity and allowable frequency of usage:

Bakers Haulover Inlet – Ebb Shoal. This is the largest of the three remaining inshore borrow areas. The ebb shoal lies predominantly along the north side of the navigation channel through Bakers Haulover Inlet, and has the shape of a large, flattened dome. The shoal is fed almost continuously by southward-moving sediment. In general, sediment moves southward along the beach, enters the navigation channel at Bakers Haulover Inlet by passing around the short north jetty, and depending on the tide is either transported landward into the flood shoal system, or seaward into the ebb shoal system.

As stated previously in this report, the ebb shoal was used as the borrow source for the 2003 and 2014 renourishments of Bal Harbour. In order to avoid interrupting any natural sediment bypassing that may be occurring, the borrow area was selectively dredged to maintain the general shape of the shoal, especially along the seaward edge where most wave energy (and sediment transport) occur. A much less prominent southward extension of this shoal can be seen in bathymetric surveys and aerial photography, extending to the south and eventually connecting to the shoreline along southern Bal Harbour. It is generally accepted that some material is able to bypass the inlet naturally along this pathway, but transport is limited due to the deep channel and strong tidal currents. Some material does bypass however, and maintaining the natural shape of the shoal helps to keep this important sediment pathway open.

The rate of deposition along the shoal has been calculated by use of periodic monitoring surveys performed in 1996, 2002, 2008, and 2011. The differences in volume within the designated dredging area of the shoal were:
- From 1996 to 2002, volume change = +85,022 cy, which equates to +14,170 cy/yr.
- From 2002 to 2008, volume change = -213,915 cy, which equates to -35,650 cy/yr.
- From 2008 to 2011, volume change = +233,345 cy, which equates to +77,780 cy/yr.

The period from 2002 to 2008 will be omitted from this analysis because it included the removal of an undetermined volume of material from the ebb shoal for the 2003 renourishment of Bal Harbour. Measurements of the volumes of material placed in 2003 and 2014 were made at the beach fill location, not at the borrow area. This survey period also included the anomalous effects of the 2004-05 storms.

Performing a weighted average of the 1996-2002 and the 2008-2011 intervals yields an average shoaling rate of +35,380 cy/yr. This is considered to be a highly reliable estimate of the shoaling rate of the ebb shoal, since no construction events influenced sediment transport patterns in the vicinity of the shoal during this time, and no anomalous weather events occurred.

Additionally, the widely-accepted sediment budget developed in the report "*Dade County Regional Sediment Budget, January 1997*", by Coastal Systems International, Inc. quantifies the shoaling rate at the ebb shoal at +32,000 cy/yr. Based largely on this estimate it was calculated that approximately 300,000 cy of material could be borrowed from this shoal every 10 years, with no longterm adverse effects. The updated calculated rate of +35,380 cy/yr is very consistent with this value but will be rounded-down to +30,000 cy/yr to provide a more conservative value. This will be the assumed shoaling rate of Bakers Haulover Inlet ebb shoal for future volumetric projections.

Since relatively little material bypasses southward around the deep channel and strong tidal currents of Bakers Haulover Inlet, dredging this shoal and placing material along the Bal Harbour shoreline amounts to basically a mechanical bypassing operation. Placing material immediately to the south of the inlet is in accordance with good regional sediment management practices, and ensures a healthy shoreline along Bal Harbour as well as the beaches further to the south. Careful consideration should be given before placing this material at any location other than downdrift (south) of Bakers Haulover Inlet.

Bakers Haulover Inlet – Flood Shoal. This is a large region consisting of two Federally-authorized navigation channels and the flood shoal inside Bakers Haulover Inlet. The two navigation channels are the Bakers Haulover Inlet Entrance Channel, and the Atlantic Intracoastal Waterway. These two waterways intersect at the inlet's flood shoal. Since the flood shoal is a rapidly-accreting area, maintenance dredging occurs frequently, on average about once every 5 years. Material dredged from this region is typically beach-quality, since the majority of this material originates from the adjacent beaches.

The longterm average rate of shoaling is +9,600 cy/yr. Thus, the flood shoal region usually produces about 50,000 cy from each 5-year dredging event, and this material is normally pumped directly onto either of the beaches adjacent to the inlet. In the earlier years material was placed primarily to the north along Haulover Park, but in recent years Bal Harbour has received most of the fill, due to a greater need along the beaches to the south side of the inlet.

This annual shoaling rate is very small relative to the other borrow areas and will not be included directly in the long-range volumetric projections. Rather, it is recognized that this material will be placed on one of the adjacent beaches near Bakers Haulover Inlet during each channel maintenance event, and will serve to reduce erosion along the fill placement area. This placement will decrease the need for material from other borrow sources and will therefore provide a margin of safety against depleting future borrow areas.

Lummus Park. The third nearshore borrow area is Lummus Park, alternately referred to as the "southern sub-reach of Miami Beach" in previous sections of this report. This is a wide, accretionary region along the south end of the Dade County BEC & HP project. Southward-moving sediment is deposited in this large embayment, and is prevented from moving further south by the long north jetty at Government Cut (Miami Harbor entrance channel). This area has been accreting since project construction, and berm widths have grown to over 400 feet along most of Lummus Park.

Several large- and small-scale renourishments have been performed in recent years by backpassing sediment from this wide accreting beach northward to erosive areas. These operations have been conducted by the Corps of Engineers, and by DERM, the project's local sponsor. DERM has performed several small-scale truck-haul operations, generally from about 15,000 cy to 50,000 cy in size. The Corps has conducted two larger direct pipeline pumpout projects. Lummus Park has proven to be an effective source of material for erosional areas that are located nearby, within pumping or trucking distances.

Accretion rates along Lummus Park can be calculated by analyzing beach profile monitoring surveys. These surveys have been performed periodically since project construction, and clearly show that the entire area continues to accrete at a steady rate. As discussed in the previous section of this report under the section "Southern Sub-Reach of Miami Beach Segment", this area accretes at an average annual rate of about 91,000 cy/yr. Only the northern half of Lummus Park is used as a borrow source, due to its proximity to the fill areas and to minimize disruption and safety hazards along this popular tourist beach. Since only half of the region can be used as a borrow source, the available rate of extraction of borrow material is reduced by half, to 45,000 cy/yr. The volume available at the 5-year construction interval consistent with renourishment of the nearby erosional hotspots would be 225,000 cy every 5 years. Since Lummus Park was used as a source of 141,159 cy of fill for the 2012 Contract E project, the area will require 3.1 years to refill before it can be used for the next backpassing operation.

Summary.

The total volume of fill required for the remaining years of the Dade County BEC & HP project is 3,625,620 cy. This volume consists of 1,147,330 cy for the Sunny Isles segment of the project, and 2,478,290 cy for the "main" segment of the project (Haulover Park south to Government Cut). The period of Federal participation for the Sunny Isles segment expires in the year 3038, so 24 years of project life remain. The period of Federal participation for the main segment of the project expires in 2015, so 11 years of project life remain. Figure 1 and table 13 provide numerical and graphical summaries of all of the dredging events that are projected to occur during the remainder of the project lives under these two project authorizations.

Note that these values represent the volumes required in place on the beach. Due to losses in the dredging process, a larger volume is required at the borrow area. Typical dredging losses range up to 30%, and would result in a borrow volume of up to 1.3 x 3,625,620 cy = 4,713,310 cy.

Dade County, Florida
Beach Erosion Control and Hurricane Protection Project
Limited Reevaluation Report
Appendix A: Engineering

Borrow Area Wave Modeling Sub Appendix

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1.0 INTRODUCTION

The Dade County Beach Erosion Control Project has depleted all local sources of offshore borrow material. In order to provide continued supply of borrow material for periodic renourishment of the Project, alternative offshore borrow sites were investigated. Two primary borrow sources have been identified that have sediment compatible with the Dade BEC Project. The borrow sites are offshore of St. Lucie County and offshore of Martin County in water depths ranging from 56 to 72 ft (17 to 22 m). Both sites are located within the region designated as Borrow Area "Zone C", which extends from 90 to 135 miles from the project area in Miami-Dade County. The borrow area designated "SL10-T41" by Ousley et al. (2014) is located about 8.4 miles offshore of St. Lucie County (Figure 1). Borrow area "M4-R105" is located about 3.8 miles offshore of Martin County (Figure 2). The notation "SL10-T41" and "M4-R105" will be used throughout this report.

Due to the size and proximity of these sites to the shoreline it is necessary to evaluate whether either of the borrow areas could alter the incident waves at the shoreline. In this study, the impacts that the proposed dredging could cause to wave energy distribution along the shoreline were analyzed using the wave model CMS-Wave. CMS-Wave is a phase averaged 2-D spectral wave transformation model. The model represents changes in wave energy resulting from diffraction, reflection, and energy losses as waves move across the model grid bathymetry.

For each borrow area, a wide variety of average and extreme wave input conditions were examined, both under existing bathymetric conditions, and with the borrow areas fully excavated. Differences in wave energy distribution were plotted for each pair of with- and without-project model simulations, and examined to determine the potential for changes to longshore sediment transport. Any changes to wave conditions observed within the active littoral zone would suggest that the proposed dredging could impact sediment transport along the shoreline.

The following report presents the details of this borrow area wave study. This report details the basic site conditions, the numerical model used in the analysis, model input parameters, calculated output, and interpretation of results.

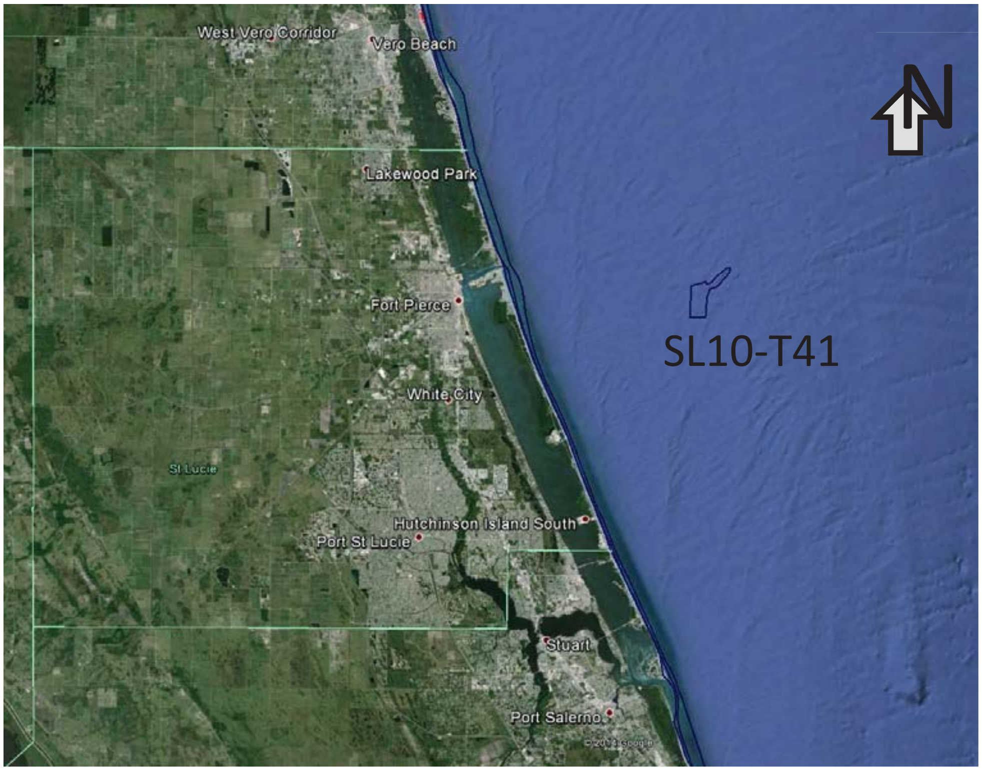

Figure 1. Proposed borrow area offshore of St. Lucie County, FL.

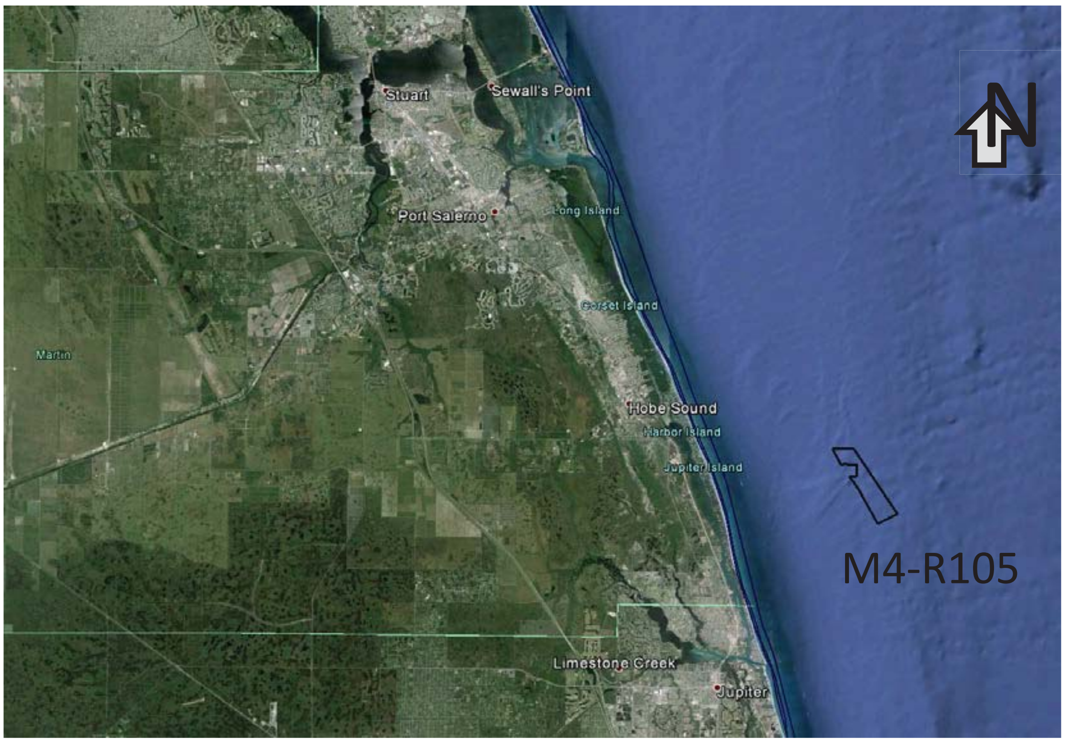

Figure 2. Proposed borrow area offshore of Martin County, FL.

2.0 MODEL SETUP

2.1 Grid Development

Regional bathymetry data from the NOAA Coastal Relief Model (CRM; NOAA 2014) was used to develop the CMS-Wave model grid. The CMS grid around the borrow areas was further refined using new bathymetric survey data collected by Jacksonville District in October 2014 (Survey #14-159). In order to set up the proposed borrow areas in the CMS-Wave model, shape files created during the Sediment Assessment and Needs Determination (SAND) study (Ousley et al., 2014) were imported into SMS. Wave Information Studies (USACE, 2014b) output locations (63449 through 63458, and 63523) were loaded into the map space for reference.

The model domains were created for each proposed borrow area location with a base cell size of 820 ft (250 m). Each grid was oriented approximately shore normal, or 70°E of N for SL10-T41 and 85°E of N for M4-R105. In CMS-Wave, refine points allow the user to increase grid resolution around areas of interest. The wave grid for SL10-T41 used 14 refine points around the borrow area and close to the shoreline inside and outside of the surf zone. Figure 3 shows the extents of the modeling grid and the refine points (in black) used for SL10-T41. Borrow area M4-R105 utilized 11 refine points providing greater model detail around the borrow area and increasing resolution approaching the shoreline (Figure 4). For both modeled sites, the refinements reduced cell sizes at the borrow area to as little as 98 ft (30 m) in the on-offshore direction and 164 ft (50 m) in the alongshore direction. Cell sizes near the shoreline at St. Lucie County were reduced to as little as 33 ft (10 m) in the on-offshore direction and 98 ft (30 m) in the alongshore direction and less than 98 ft (30 m) in the on-offshore direction and 148 ft (45 m) in the alongshore direction near the shoreline of Martin County. The SL10-T41 grid totaled 77,120 cells and the M4-R105 grid totaled 65,560 cells.

In order to compare the existing and proposed dredged conditions, the entire SL10-T41 area was deepened in the model to reflect the deepest possible cut from all of the 2014 core borings (see Figure 5 and Figure 6). The potential cut depth for boring VB-SLC14-06 was 79 ft (24.1 m), or about 13 ft (4 m) below the existing surface (see Appendix D). Other core borings taken within SL10-T41 indicated potential cut depths ranging from 69 to 78 ft (21 to 24 m) below NAVD88. Using a cut depth of 79 ft (24.1 m) over the entire investigation area is well in excess of the area likely to be dredged, and represents a worst case scenario for changes to the wave field in the lee of the borrow area. The total area of SL10-T41 represented in the model grid equaled about 52,699,000 ft^2 and the volume of dredged material was 30,835,000 cy (Table 1). Note that the volumes included in Table 1 are actually the volume of water above the bed as represented in the model, therefore, the volume difference is the important value as it represents the amount of material removed from the borrow area. The average depth change over the entire area was therefore 15.8 ft (4.8 m).

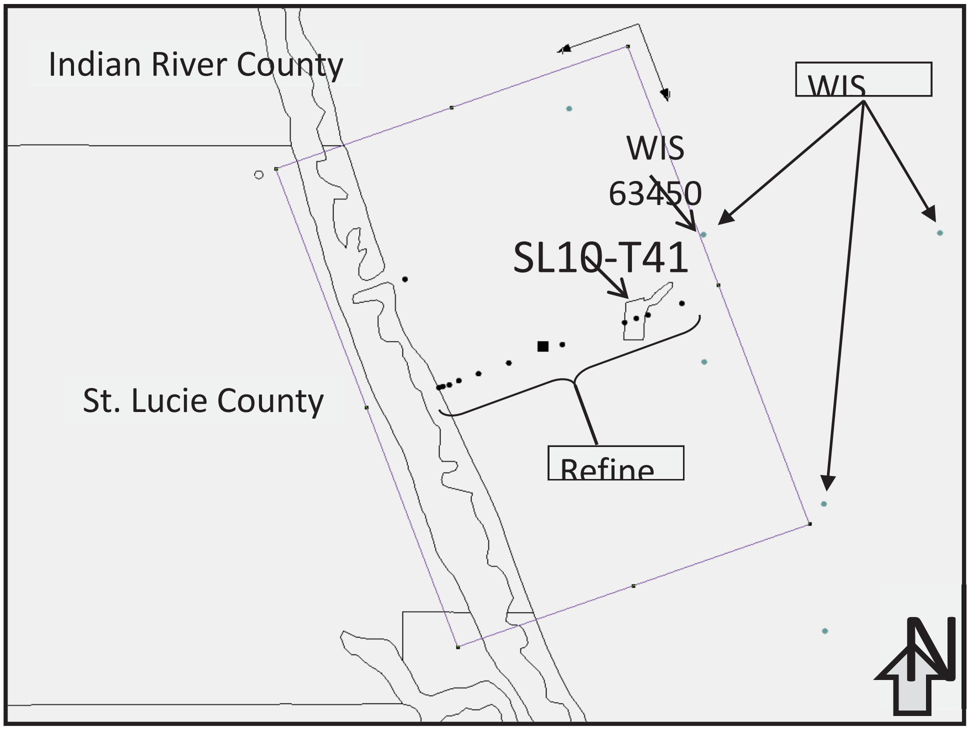

Figure 3. SL10-T41 CMS-Wave modeling grid with refine points.

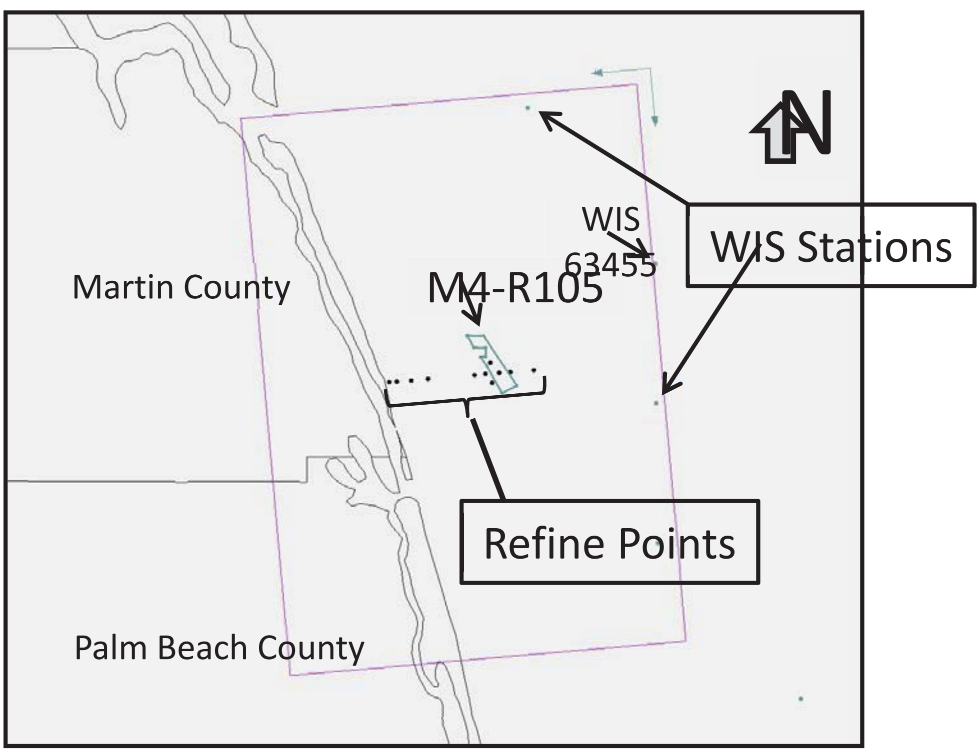

Figure 4. M4-R105 CMS-Wave modeling grid with refine points.

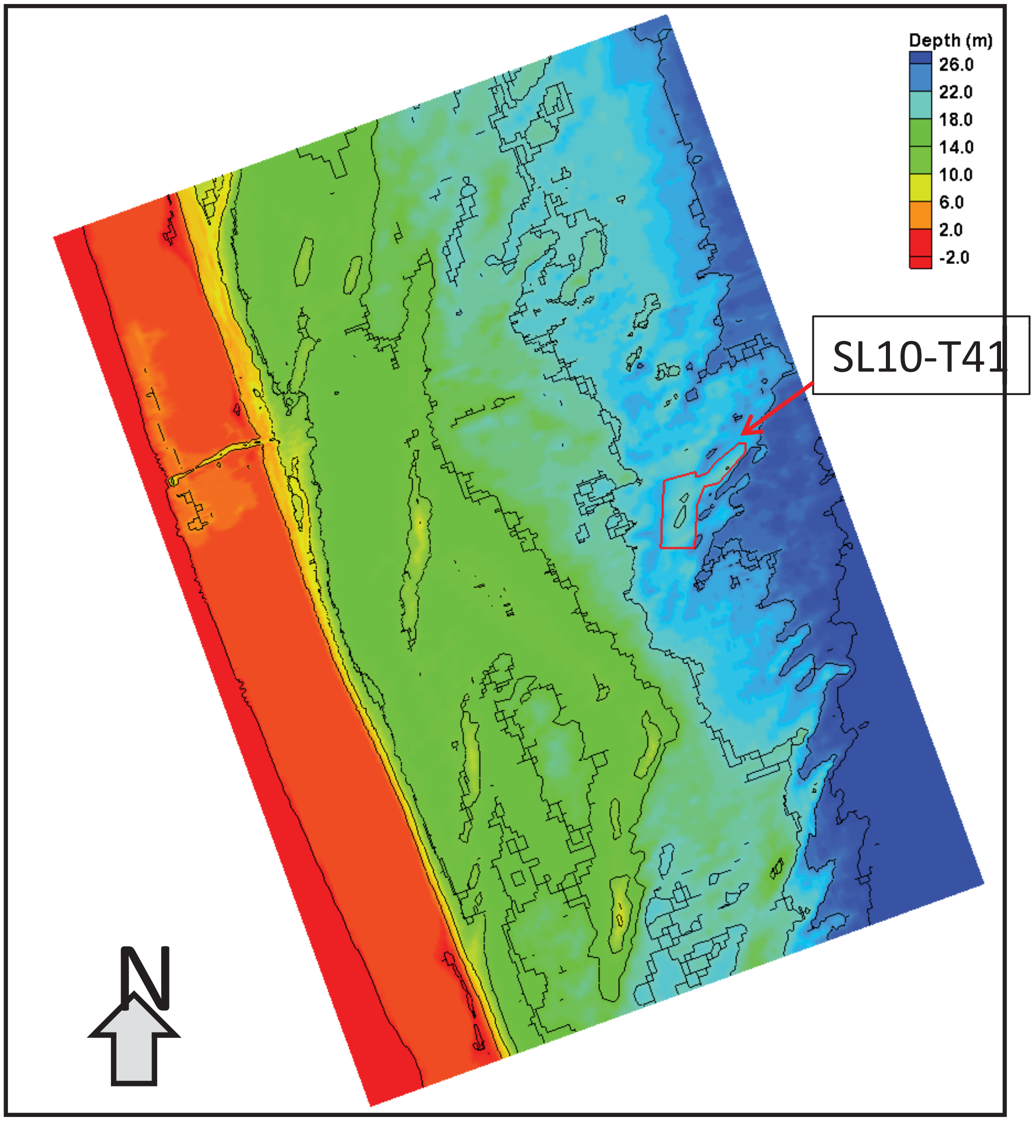

Figure 5. Model bathymetry for SL10-T41 under existing conditions.

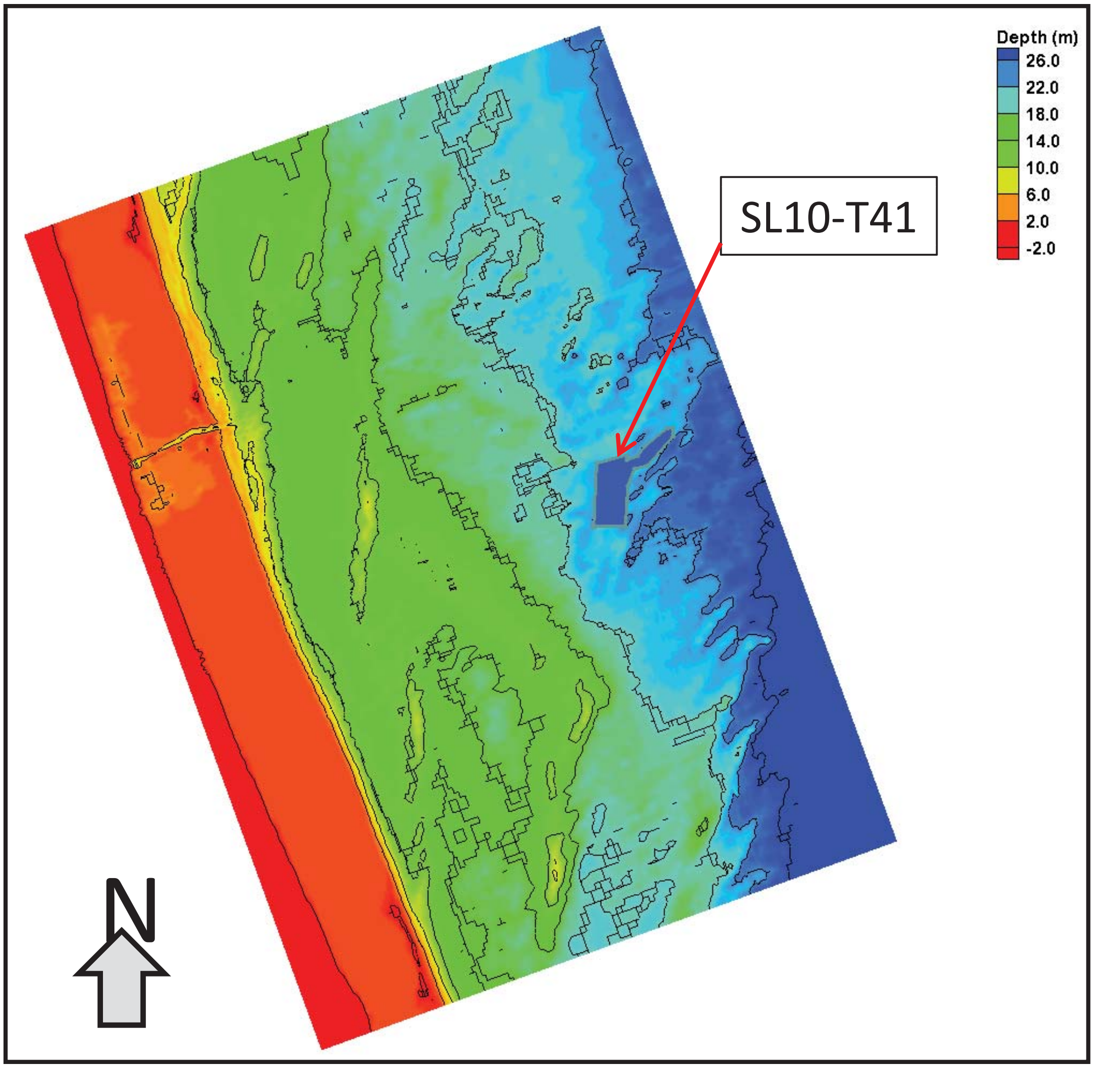

Figure 6. SL10-T41 model bathymetry with deepened investigation area.

Table 1. SL10-T41 modeled borrow area characteristics.

Grid	# of Cells	Area (ft^2)	Volume (cy)	Average Depth meters	Average Depth feet
Existing	1981	52,698,716	123,183,778	19.2	63
Dredged	1981	52,698,716	154,018,308	24.1	79
Difference	0	0	30,834,530	4.8	15.8

Existing core borings around the M4-R105 borrow area and additional borings collected during 2014 were analyzed to determine the suitability of the sediments for use in the Dade County BEC Project. Model bathymetry for the without project condition for potential borrow area M4-R105 is shown in Figure 7. Only the southwestern portion of M4-R105 contained sediments compatible with existing Dade County beach material, as documented in Geotechnical Appendix D. Core borings outside M4-R105 showed favorable material so the potential borrow site was extended landward to include these materials in order to provide the worst case, maximum impact, dredging scenario. The maximum depth of good quality sand in the area was identified as 69 ft (21 m) and the model grid was updated to reflect the with-project condition (Figure 8). The total area covered by the M4-R105 grid equaled about 9,175,000 ft^2 and the volume of dredged material represented in the model was 1,387,000 cy (Table 2). The average depth change over the entire area was therefore 4.1 ft (1.2 m).

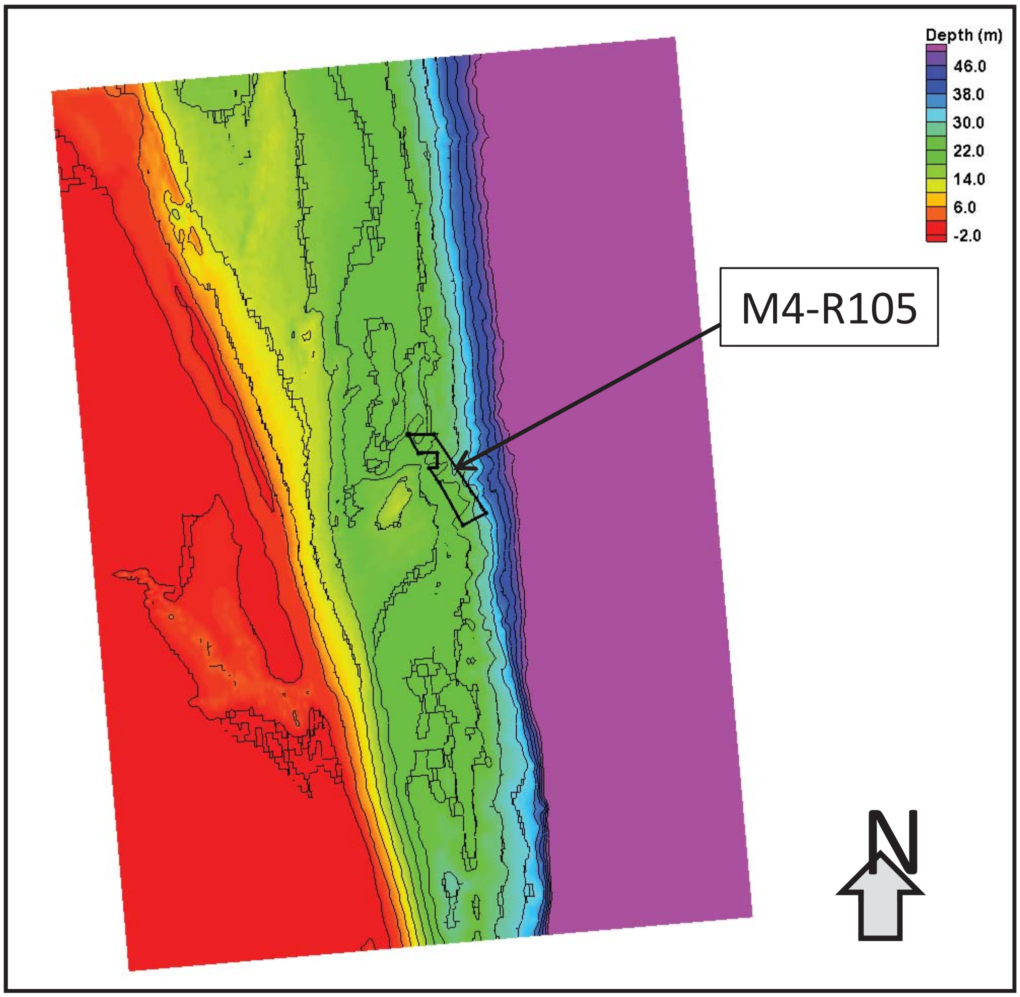

Figure 7. Model bathymetry for M4-R105 borrow area under existing conditions.

Table **2**. M4-R105 modeled borrow area characteristics.

Grid	**# of Cells**	**Area (ft^2)**	**Volume (cy)**	**Average Depth**	
				meters	**feet**
Existing	514	9,175,235	22,017,564	19.7	65
Dredged	514	9,175,235	23,404,960	21.0	69
Difference	0	0	1,387,396	1.2	4.1

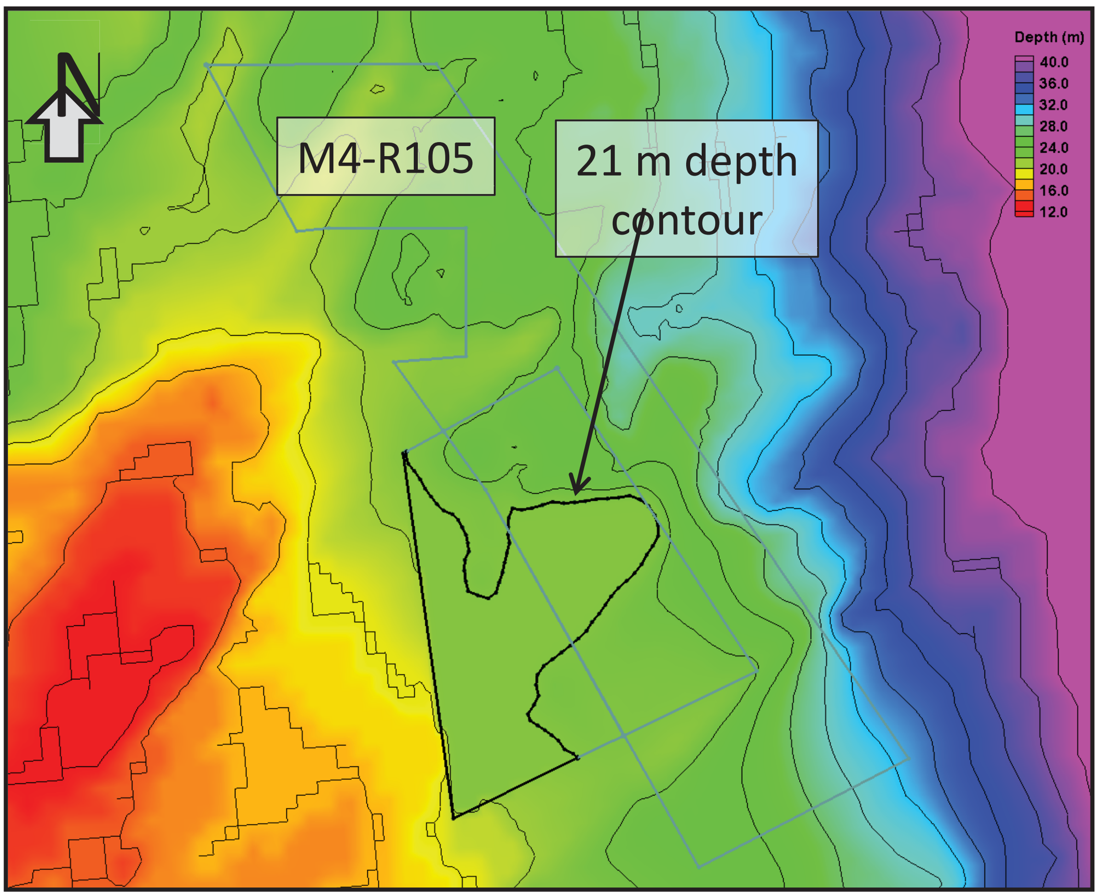

Figure 8. M4-R105 model bathymetry and deepened investigation area.

2.2 Wave Climate

Borrow areas SL10-T41 and M4-R105 are located about 30 miles apart and therefore have similar wave characteristics. A 33-year wave and wind hindcast was developed for various output locations in the vicinity of the proposed borrow areas under the USACE Wave Information Studies (WIS) program (USACE, 2014b; Figure 9). Stations 63450 and 63455 were selected for use in this study due to their proximity to the proposed borrow areas, and because they feature more energetic wave climates versus the stations nearest the borrow areas. More energetic waves are more likely to show the effects of borrow area dredging, and again represent a kind of 'worst-case' scenario for this analysis. Wave heights are important in this analysis because larger waves have exponentially more power to move sediment. Wave period is important because longer-period waves are more strongly affected by changes in bathymetry.

Although these stations are located similar distances offshore, the water depth at station 63450 is 75 ft (23 m) and 564 ft (172 m) for station 63455. For WIS station 63450, waves typically originate from the east or east-northeast (ENE) direction (Figure 10). The predominant wave direction for the 63455 WIS station off of Martin County is from the northeast direction (Figure 11). Wave heights for both stations are typically 0 to 6.5 ft (0 to 2 m) and wave heights greater than 13 ft (4 m) are rare but do occur. In the winter, low pressure systems (Nor'easters) generate storm waves from the northeast (NE) or ENE direction. Storm events from the east and south are due to passing tropical storms which can generate offshore waves up to 29 ft (8.7 m) and 35 ft (10.8 m) at stations 63450 and 63455, respectively. Storm wave return intervals for each WIS station used in this study are presented in Figure 12 and Figure 13.

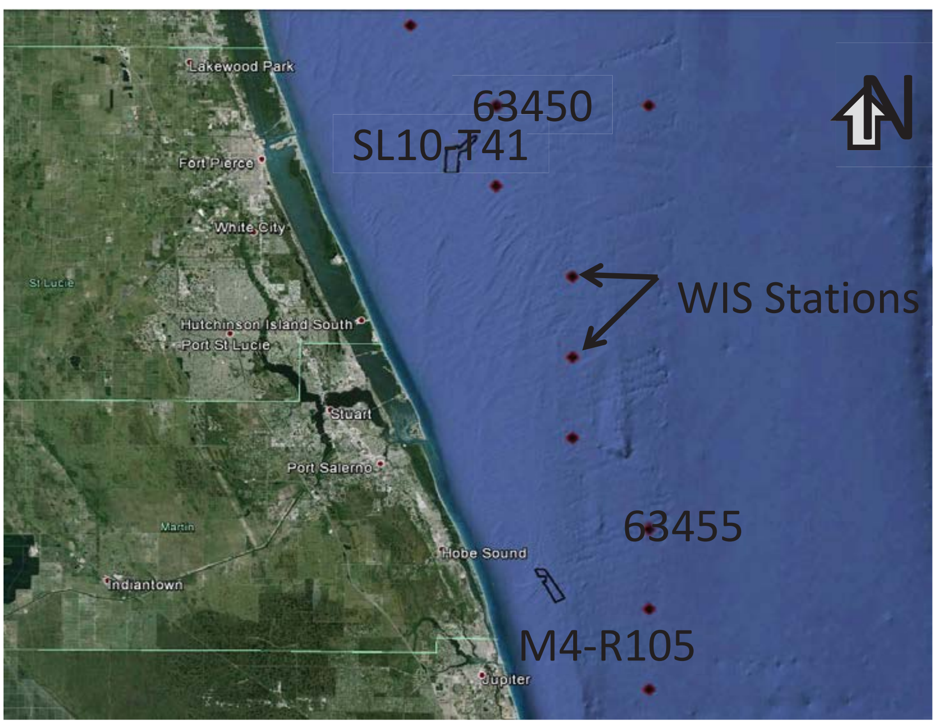

Figure 9. Proposed borrow areas and Wave Information Study output stations.

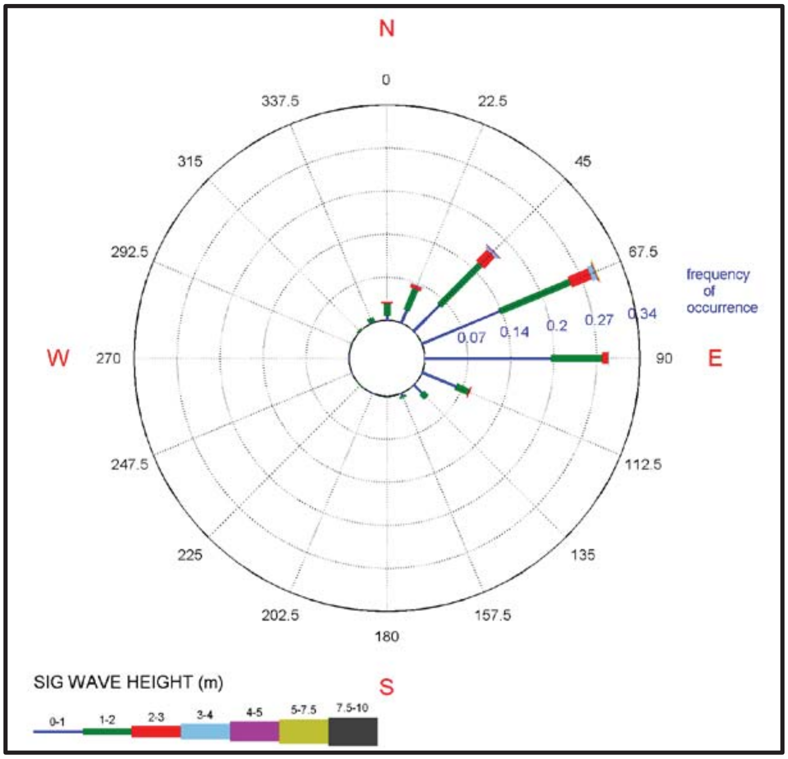

Figure 10. Wave rose for WIS Station 63450 off St. Lucie County, FL.

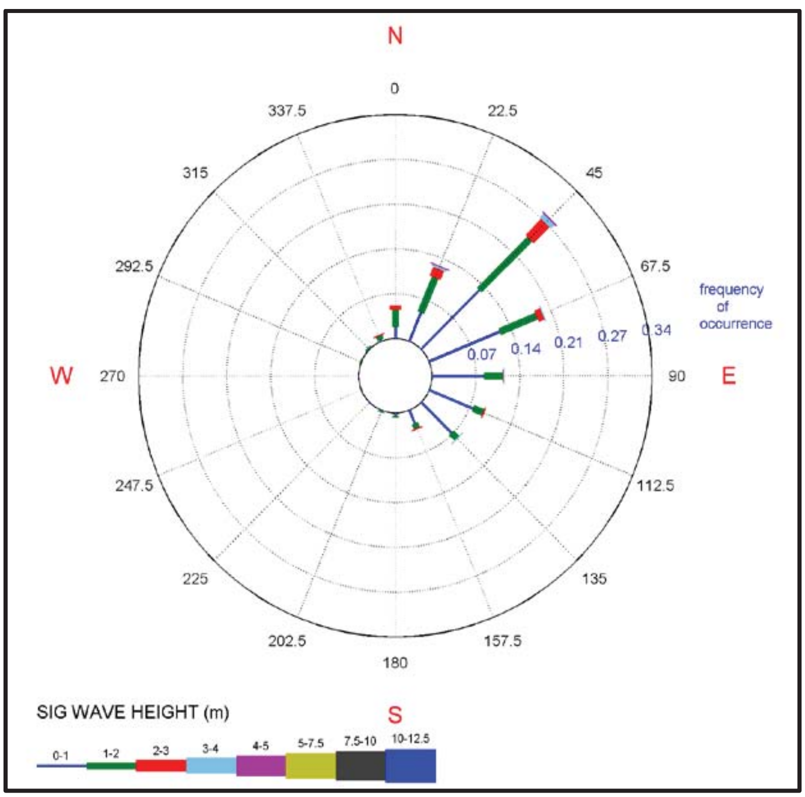

Figure 11. Wave rose for WIS Station 63455 off Martin County, FL.

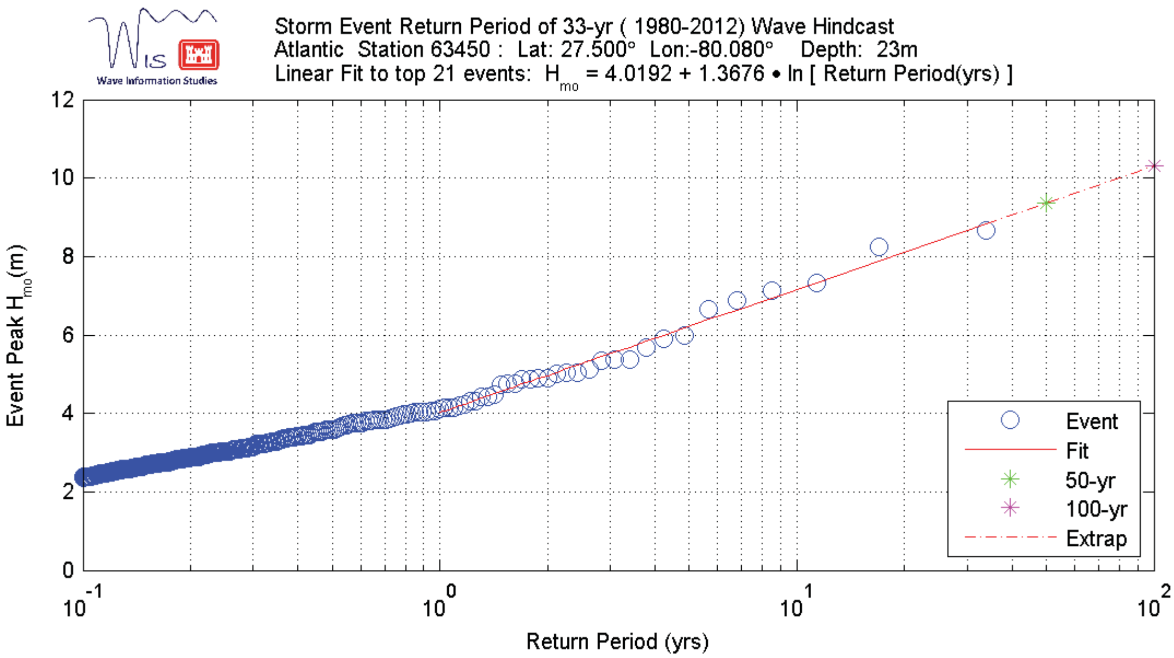

Figure 12. Wave height return period for WIS station 63450.

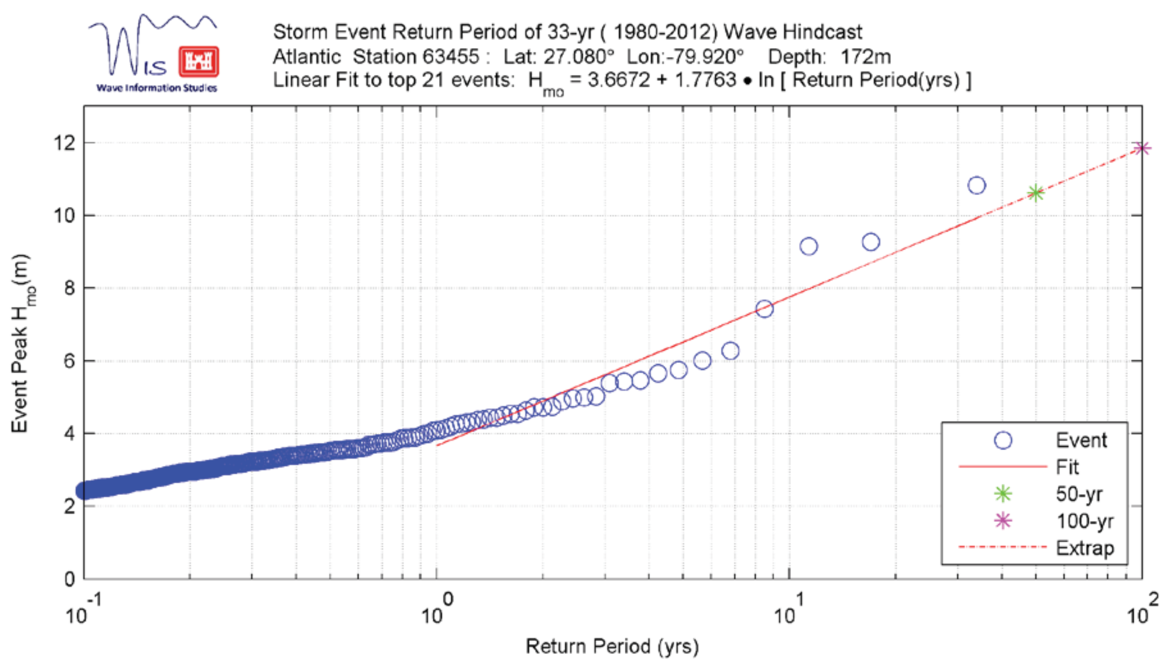

Figure 13. Wave height return period for WIS station 63455.

2.3 Boundary Conditions

WIS station 63450 is located on the seaward edge of the CMS-Wave grid just north of the SL10-T41 investigation area in 75 ft (23 m) water depths referenced to Mean Sea Level (MSL; Figure 3). WIS station 63455 is located on the seaward edge of the CMS-Wave grid just south of the M4-R105 investigation area in water depths of 564 ft (172 m) (MSL; Figure 4). WIS hindcast data for these stations covers the 33-year period from 1 January 1980 through 31 December 2012. Average monthly wave heights and maximum wave heights were analyzed to develop the input wave conditions. Inputs included the largest wave on record, the greatest period event on record (which ranks as the 20th greatest wave height for station 63450 and 13th for station 63455), the greatest monthly average wave height, and finally the overall average wave height, as seen in Table 1 and

Table 5. The monthly average and overall average wave height conditions were varied by direction based on direction bands with frequencies of occurrence greater than 10 percent. Input conditions did not include elevated water levels (storm surge) typically associated with extreme events, another conservative measure. Including storm surge would reduce changes to the wave field caused by dredging the proposed borrow areas since interaction of input waves with the bed would decrease.

The largest wave in the record for both WIS stations occurred in September 2004 during the passage of Hurricane Jeanne with wave heights reaching 29 ft (8.7 m) at a peak spectral period of 15.4 seconds from the direction 71° for WIS station 63450. During the peak of the storm, WIS station 63455 reported wave heights reaching 35 ft (10.8 m) at a peak spectral period of 14.5 seconds from the direction 42°. A corollary to energy and power calculated as height squared and height squared times peak period, respectively, is included in Table 4 and

Table 6 to show the distribution of energy between the input wave conditions. The greatest wave period event occurred in October 1991 during the renowned "Halloween Nor'easter" (also "Perfect Storm") where wave heights for WIS station 63450 reached 16 ft (4.9 m) at a peak spectral period of 19.4 seconds from 62°. During the peak of the storm, WIS station 63455 also reported wave heights reaching 16 ft (5.0 m) at a peak spectral period of 19.1 seconds from the direction 43°. The maximum height and period events were given a second variant by changing the direction to shore normal which was expected to have the most potential for wave field differences between the pre- and post-dredge condition. Since the direction maximum height event for the SL10-T41 site only varied from shore normal by one degree, it was removed from analysis.

Table 3. Input wave parameters for SL10-T41 model grid.

Condition #	Description	Month of Occurrence	Height (m)	Period (s)	Direction (°TN)
1	Max Recorded Energy Event	Sep-04	8.7	15.4	71
2a	Max Period Event	Oct-91	4.9	19.4	62
2b	Max Period, Shore Normal	Oct-91	4.9	19.4	70
3a	Greatest Monthly Average Height	Nov-11	2.3	10.2	70
3b					45
3c					90
4a	Overall Average Height	N/A	1	9	70
4b					45
4c					90

Table 4. Input wave energy and power corollaries for SL10-T41 model grid.

Condition #	Description	Month of Occurrence	Energy Corollary (m^2)	Power Corollary (H^2*T)
1	Max Recorded Energy Event	Sep-04	75	1158
2a	Max Period Event	Oct-91	24	457
2b	Max Period, Shore Normal	Oct-91	24	457
3a	Greatest Monthly Average Height	Nov-11	5	52
3b				
3c				
4a	Overall Average Height	N/A	1	13
4b				
4c				

Table 5. Input wave parameters for M4-R105 model grid.

Condition #	Description	Month of Occurrence	Height (m)	Period (s)	Direction (°TN)
1a	Max Recorded Energy Event	Sep-04	10.8	14.5	42
1b	Max Event, Shore Normal Direction	Sep-04	10.8	14.5	85
2a	Max Period Event	Oct-91	5.0	19.1	43
2b	Max Period, Shore Normal	Oct-91	5.0	19.1	85
3a	Greatest Monthly Average Height	Nov-11	2.1	9.6	85
3b					22.5
3c					45
3d					67.5
4a	Overall Average Height	N/A	1.0	8.3	85
4b					22.5
4c					45
4d					67.5

Table 6. Input wave energy and power corollaries for M4-R105 model grid.

Condition #	Description	Month of Occurrence	Energy Corollary (m^2)	Power Corollary (H^2*T)
1a	Max Recorded Energy Event	Sep-04	117	1701
1b	Max Event, Shore Normal Direction	Sep-04	117	1701
2a	Max Period Event	Oct-91	25	475
2b	Max Period, Shore Normal	Oct-91	25	475
3a	Greatest Monthly Average Height	Nov-11	4	41
3b				
3c				
3d				
4a	Overall Average Height	N/A	1	9
4b				
4c				
4d				

Spectra were generated using the TMA Shallow Water method (Ochi, 1998) with spectral shaping parameters following the default values given in SMS (see Table 7) and the wave parameters found in Table 3 and

Table 5 for SL10-T41 and M4-R105, respectively. As required by the model, wave directions were modified for the shore normal model grid, or relative to 70° for SL10-T41 (85° for M4-R105), with positive angles originating from directions north of shore normal and negative angles originating south of shore normal.

Table 7. Spectral shape default parameters.

Tp (s)	Gamma	nn
10	3.3	4
11	4	8
12	4	10
13	5	12
14	5	16
15	6	18
16	6	20
17	7	22
18	7	26
19	8	28
20	8	30

2.4 Depth of Closure

The depth of closure is commonly defined as the offshore location where sediment movement due to wave action ceases. The depth of closure can be established where repeated beach profile surveys show no change in elevation. Alternatively, empirical methods developed by Hallermeier (1978) and Birkemeier (1985), have been employed which rely on 12-hour annual maximum wave heights and associated wave periods (Table 8). Hallermeier (1981) found that using the average annual wave period provides an adequate estimate for the wave period input required in the empirical depth of closure equations. Birkemeier (1985) also modified the empirical equation to utilize annual average significant wave height, as did Kraus et al. (1999) and Houston (1995).

Table 8. Depth of closure sources and equations.

Source	Equation
Hallermeier (1978)	$d = 2.28H - 68.5\left(\frac{H^2}{gT^2}\right)$
Birkemeier (1985), A	$d = 1.75H - 57.9\left(\frac{H^2}{gT^2}\right)$
Birkemeier (1985), B	$d = 1.57H$
Houston (1995)	$d = 6.75\overline{H_s}$
Kraus et al. (1999)	$d = 8.9\overline{H_s}$
where: H = 12-hour annual maximum wave height. g = acceleration due to gravity (9.81$\frac{m}{s^2}$). T = 12-hour or average annual wave period. $\overline{H_s}$ = average annual significant wave height.	

For both the SL10-T41 and M4-R105 borrow areas, annual 12-hour maxima were selected from the 1-year storm event wave height shown in Figure 12 and Figure 13, or 13.1 ft (4 m) and 13.4 ft (4.1 m), respectively. Average annual significant wave heights were generated from the 63450 and 63455 WIS station outputs and equaled 3.7 ft (1.13 m) and 3.4 ft (1.03 m), respectively. The average wave periods were found to equal 9.0 and 8.3 seconds, respectively. These values constitute the inputs to each equation presented in Table 8. The result of each equation is presented in Table 9 for the St. Lucie County shoreline and Table 10 for the Martin County shoreline. A conservative estimate of 26 ft (8 m) will be used for the depth of closure for both St. Lucie County and Martin County. The depth of closure contour is located about 3,900 ft (1,200 m) offshore of St. Lucie County and 3,300 ft (1,000 m) offshore of Martin County in the lee of the proposed borrow areas based on the input model grid bathymetry. Evaluation of model results will therefore indicate that changes to the wave field inside of the 26 ft (8 m) depth contour due to dredging the proposed borrow areas will cause impacts to the existing sediment transport pattern.

Table 9. Depth of closure calculations for WIS station 63450.

Input Wave	**Wave Period (s)**	**Depth of Closure (m)**				
		Hallermeier (1981)	**Birkemeier A (1985)**	**Birkemeier B (1985)**	**Houston (1995)**	**Kraus et al. (1999)**
$H = 4m$	9.0	7.7	5.8	6.3	*na*	*na*
$\overline{H_s} = 1.13m$	*na*	*na*	*na*	*na*	7.6	10.1

Average (m)	**7.5**
Median (m)	**7.6**
Standard Deviation (m)	**1.6**

Table 10. Depth of closure calculations for WIS station 63455.

Input Wave	**Wave Period (s)**	**Depth of Closure (m)**				
		Hallermeier (1981)	**Birkemeier A (1985)**	**Birkemeier B (1985)**	**Houston (1995)**	**Kraus et al. (1999)**
$H = 4.1m$	8.3	7.6	5.7	6.4	*na*	*na*
$\overline{H_s} = 1.03m$	*na*	*na*	*na*	*na*	7.0	9.2

Average (m)	**7.2**
Median (m)	**7.0**
Standard Deviation (m)	**1.3**

3.0 MODEL RESULTS

Comparisons of the with- and without-project wave heights and wave directions for the most adverse input conditions are displayed below. For wave heights, the with-project model results are normalized by the without-project model results (i.e. dredged conditions / existing conditions) and for wave directions, an arithmetic difference is applied between the with- and without-project model results. The depth of closure (26 ft; 8 m) as well as the 9.8 ft (3 m), 19.7 ft (6 m), and 39.4 ft (12 m) depth contours are included for reference.

3.1 Wave Height Comparisons

The wave conditions that provided for the maximum possible impact to the nearshore sediment transport in both St. Lucie County and Martin County were selected for presentation in this section. For brevity, not all input conditions that were tested are included, however sufficient output to show the variation in results and maximum changes to the wave field are displayed.

The with-project model outputs were normalized by the without-project model outputs to represent changes to the wave field in the vicinity of and landward of the proposed borrow areas. Wave height factors are presented in Figure 14 through Figure 20 for SL10-T41 with values ranging from 0.75 (a 25% decrease in wave height) to 1.15 (a 15% increase in wave height). Figure 21 through Figure 25 present the wave height factors for selected model runs of the M4-R105 borrow area with the same value scale. The wave height factor figures for SL10-T41 include wave conditions 1, 2a, 2b, 3a, 3b, 3c, and 4a (see Table 3). For the M4-R105 site, wave height factor figures were limited to wave conditions 1a, 1b, 2a, 3b, and 4b (see

Table 5) for brevity since the other model runs did not produce any noteworthy changes to the wave field.

An examination of the model output for borrow area SL10-T41 shows that the largest period event on record (wave conditions 2a and 2b) causes the most widespread change to the wave field in the alongshore direction, however, the most energetic event (wave condition 1) extends slightly closer to shore (Figure 14 through Figure 16). The majority of change resulting from Condition 1 occurs 11,500 ft (2.2 miles) seaward of the depth of closure contour. The large wave height of condition 1 results in some wave dissipation (breaking) due to wave steepness. The dissipation occurs in both the with- and without-project conditions in adjacent cells which causes the noise in the output. Condition 2 similarly exhibits some noise seaward of the depth of closure. Wave breaking due to wave steepness occurs in deep water and therefore is not expected to influence sediment motion in the littoral zone. For all modeled input conditions, the major changes to the wave field exist seaward of the 39.4 ft (12 m) depth contour with the exception of the noise, which is still outside of the depth of closure (26 ft; 8 m) contour. Figure 15 and Figure 16 illustrate the minor changes to outputs when the wave field is changed by only 8°. Figure 17 through Figure 19 provide a more comprehensive understanding of impacts due to wave direction as the input wave varies between 45° and 90°. Figure 20 demonstrates the similar changes wave condition 4 causes to the wave field as wave condition 3. Since wave condition 4 results in less impacts, only condition 4a is provided for brevity.

The changes to the wave field observed offshore Martin County follow similar patterns as St. Lucie County due to the similar input conditions. The results show even less impact to the wave field in the lee of the M4-R105 borrow area than were observed at SL10-T41. This was likely due to the reduced area, volume, and average cut depth compared to the SL10-T41 borrow area in addition to marginally deeper depths at the M4-R105 borrow area. Condition 2a shows impacts extending to within about 13,500 ft (2.5 miles) seaward of the depth of closure contour. Similar to the SL10-T41 borrow area, the noise between the with- and without-project conditions is also observed for Condition 1, but is much less pronounced for M4-R105 (Figure 21 and Figure 22).

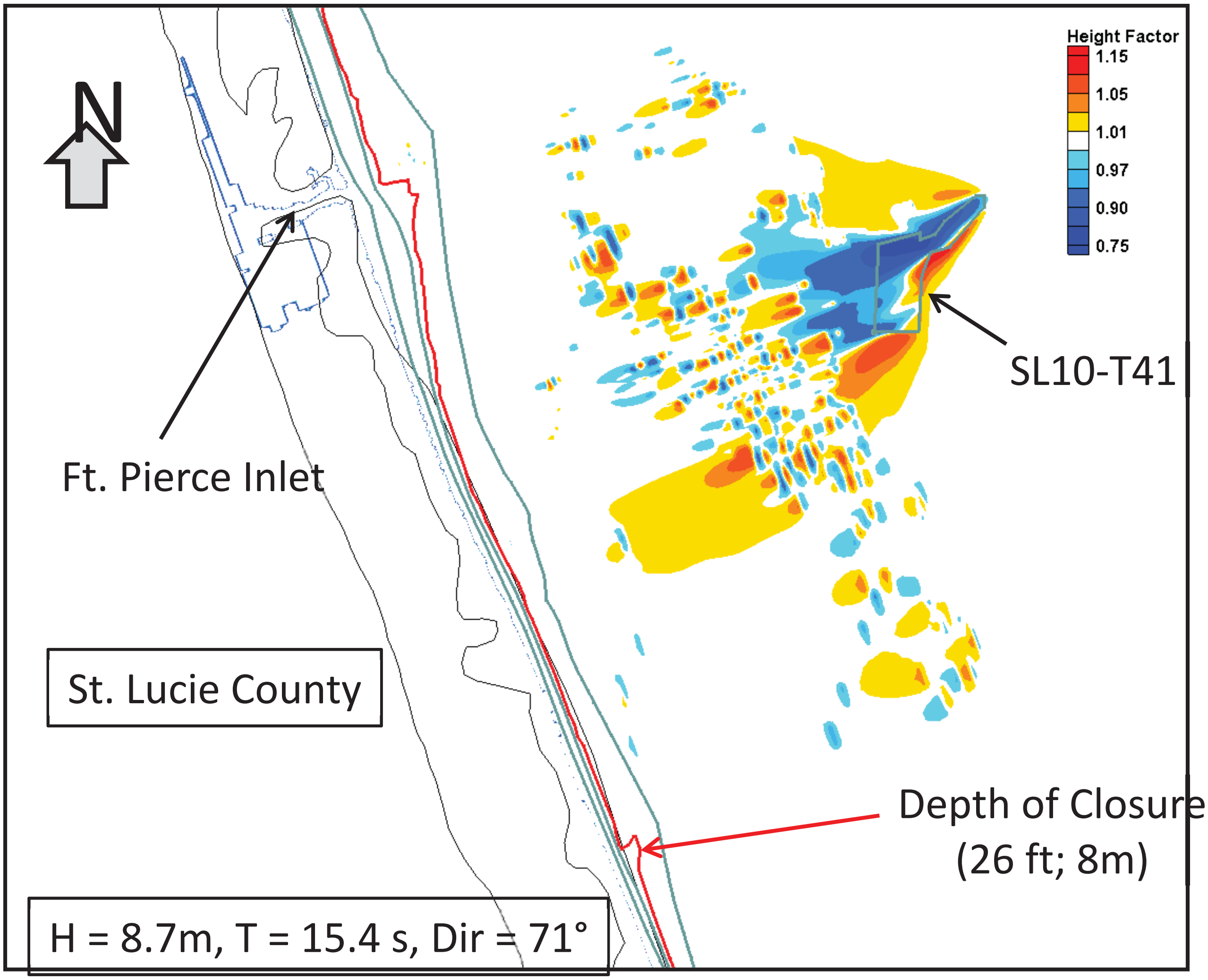

Figure 14. Wave height comparison for wave condition #1 at SL10-T41.

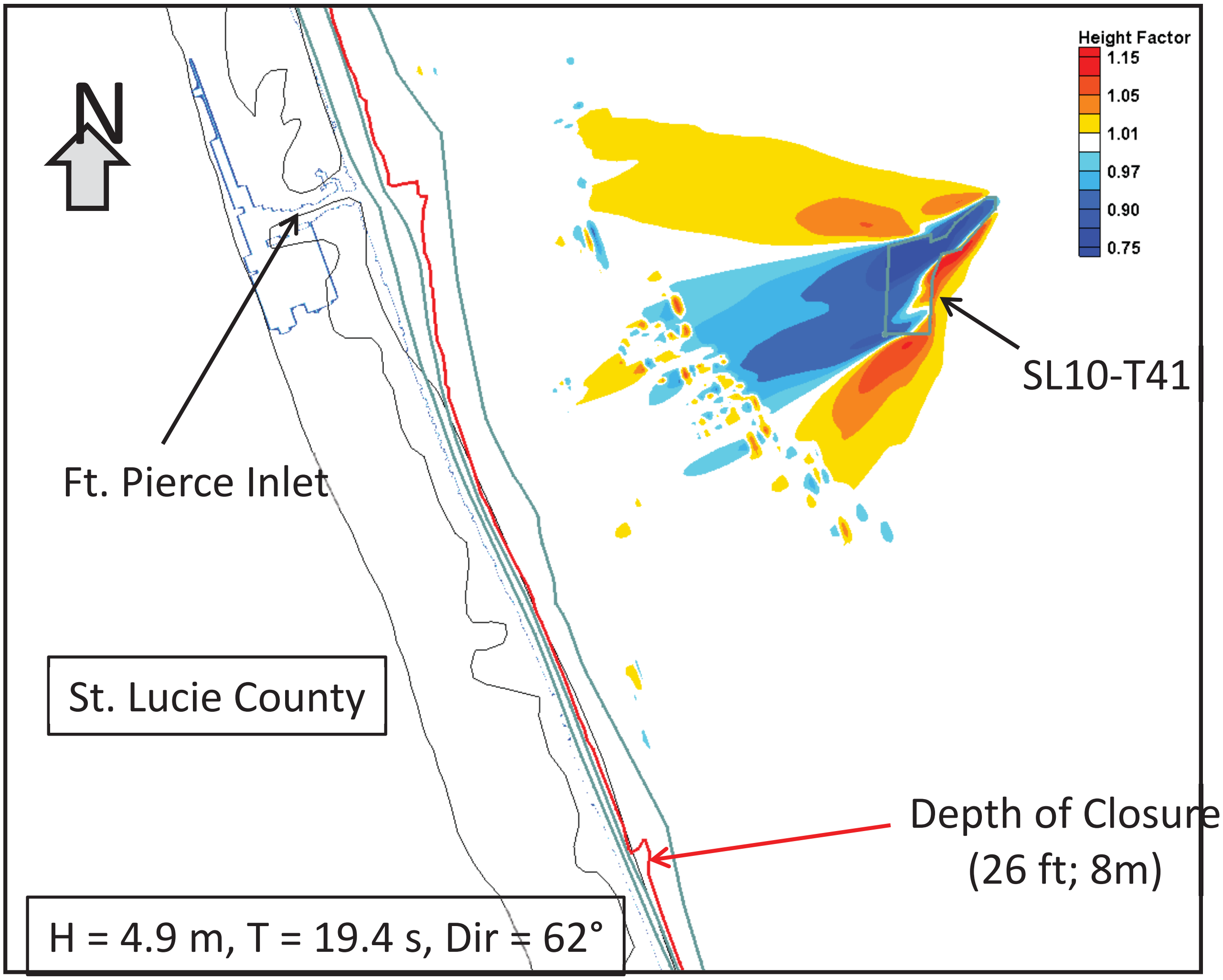

Figure 15. Wave height comparison for wave condition #2a at SL10-T41.

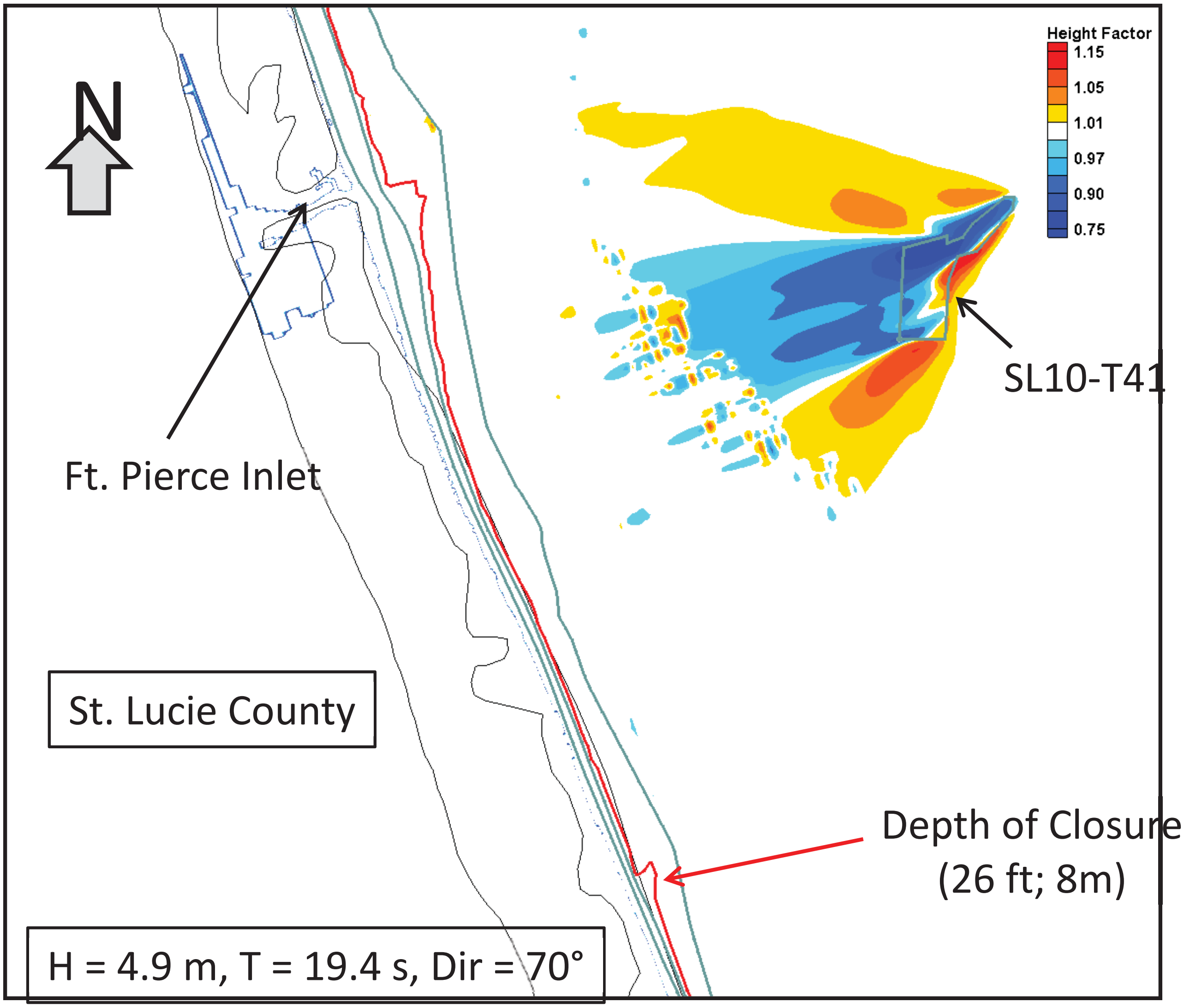

Figure 16. Wave height comparison for wave condition #2b at SL10-T41.

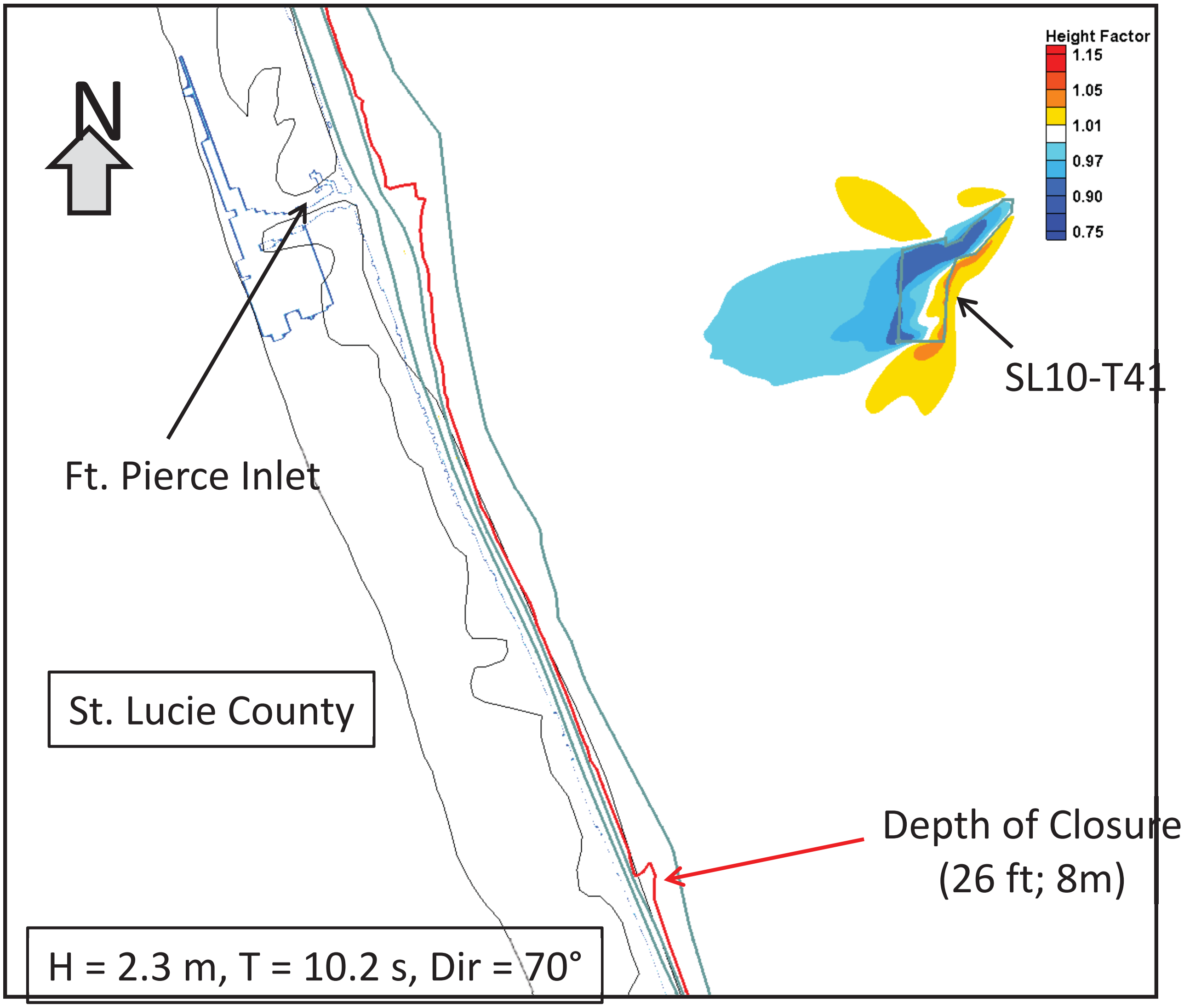

Figure 17. Wave height comparison for wave condition #3a at SL10-T41.

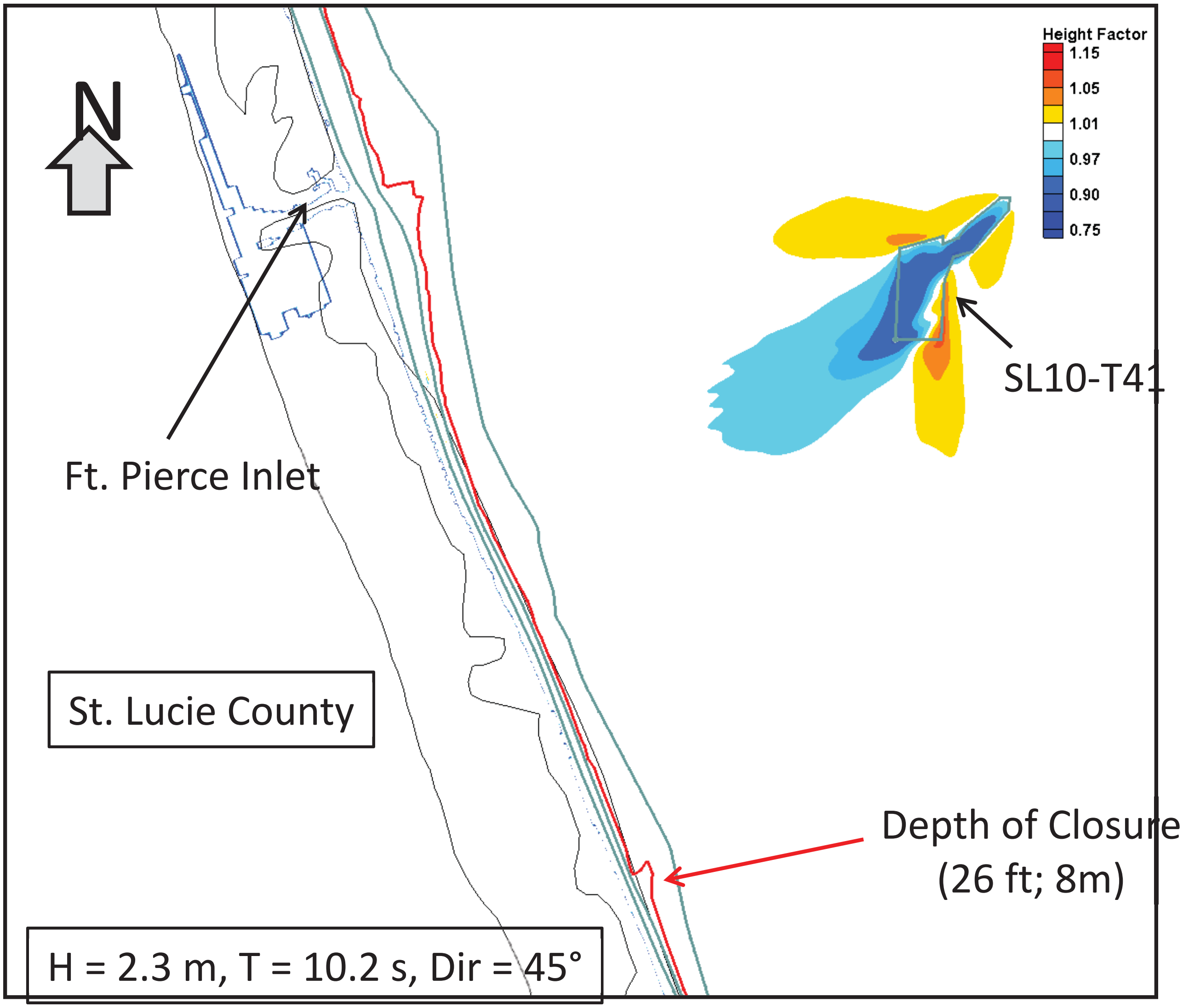

Figure 18. Wave height comparison for wave condition #3b at SL10-T41.

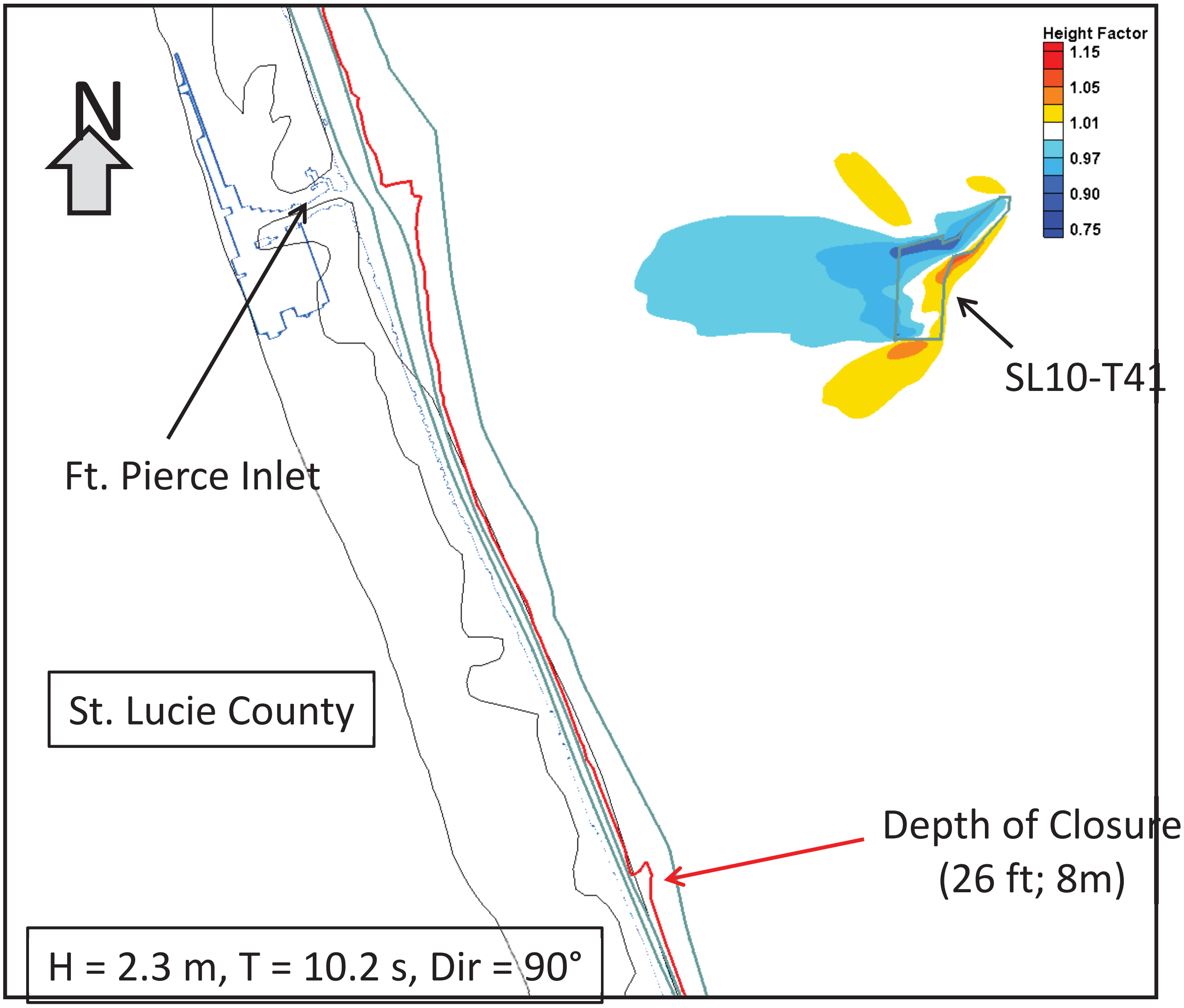

Figure 19. Wave height comparison for wave condition #3c at SL10-T41.

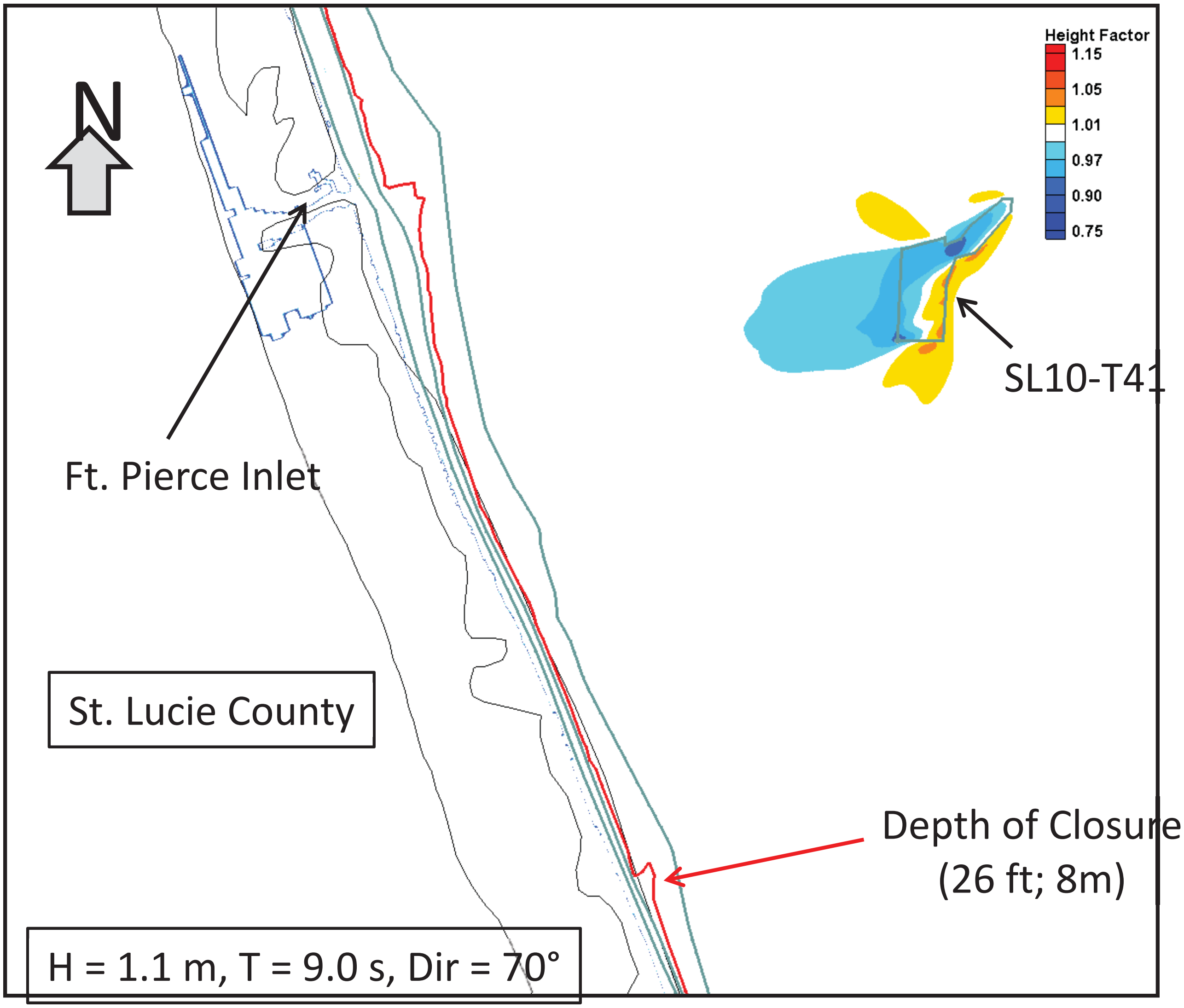

Figure 20. Wave height comparison for wave condition #4a at SL10-T41.

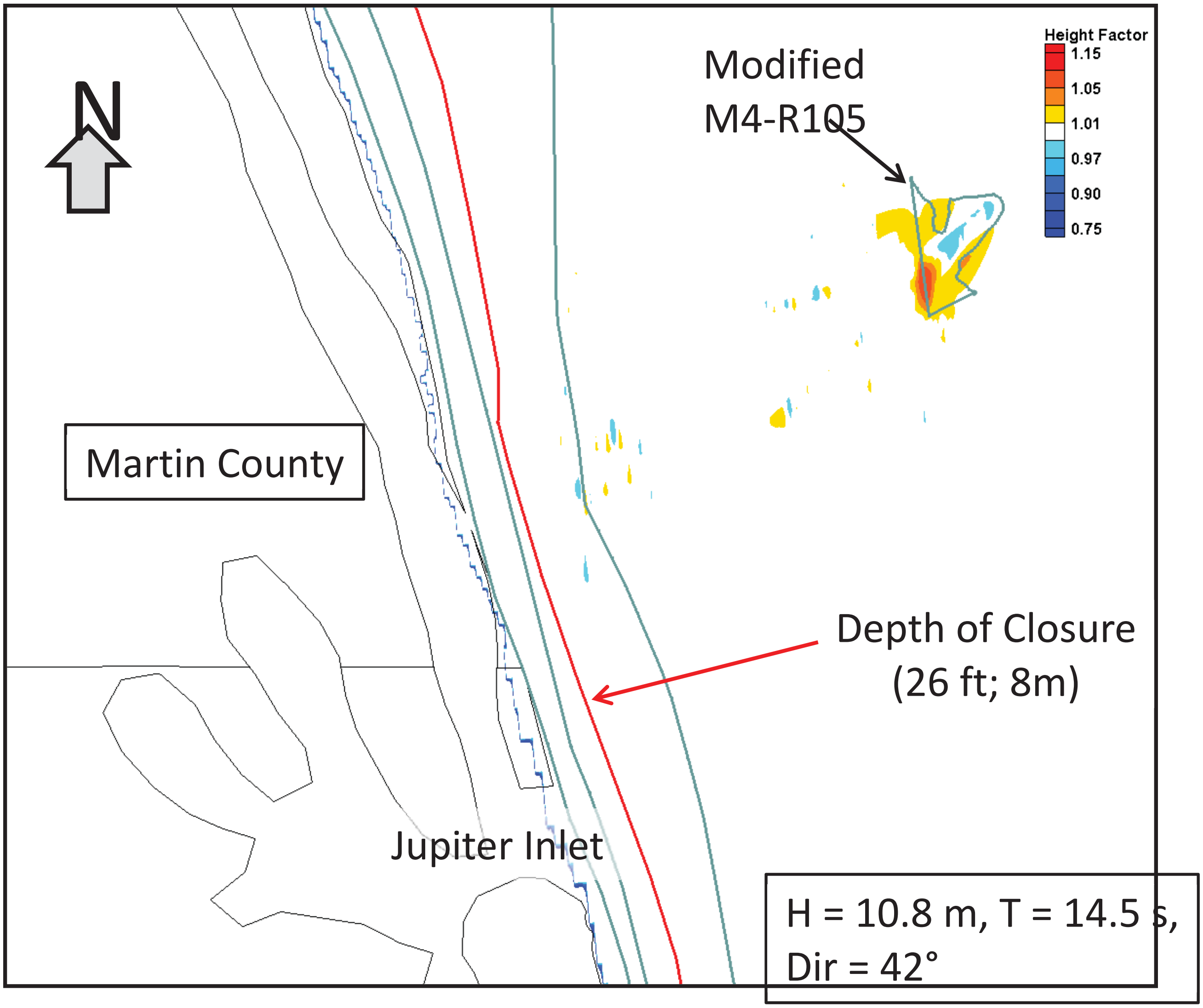

Figure 21. Wave height comparison for wave condition #1a at M4-R105.

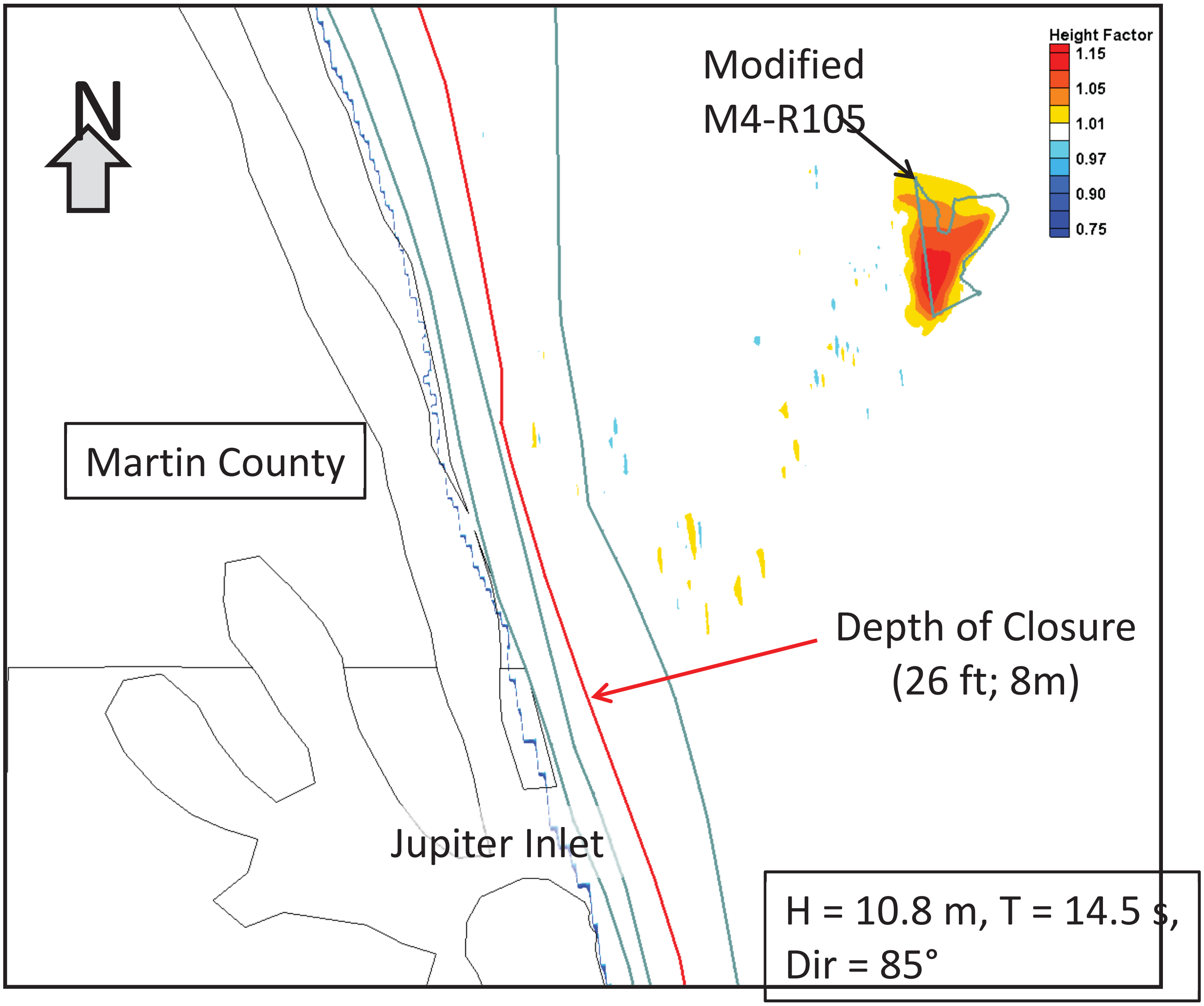

Figure 22. Wave height comparison for wave condition #1b at M4-R105.

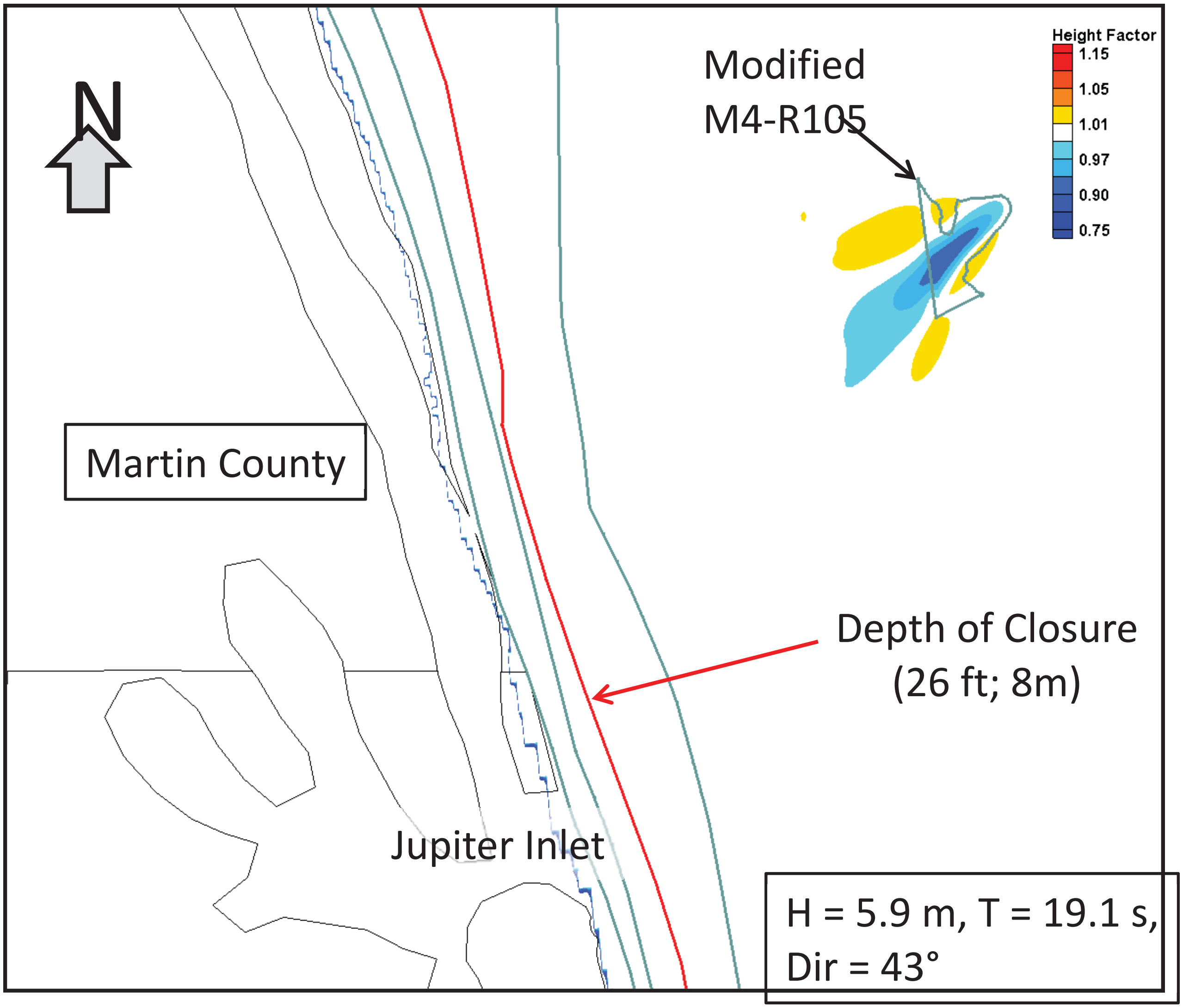

Figure 23. Wave height comparison for wave condition #2a at M4-R105.

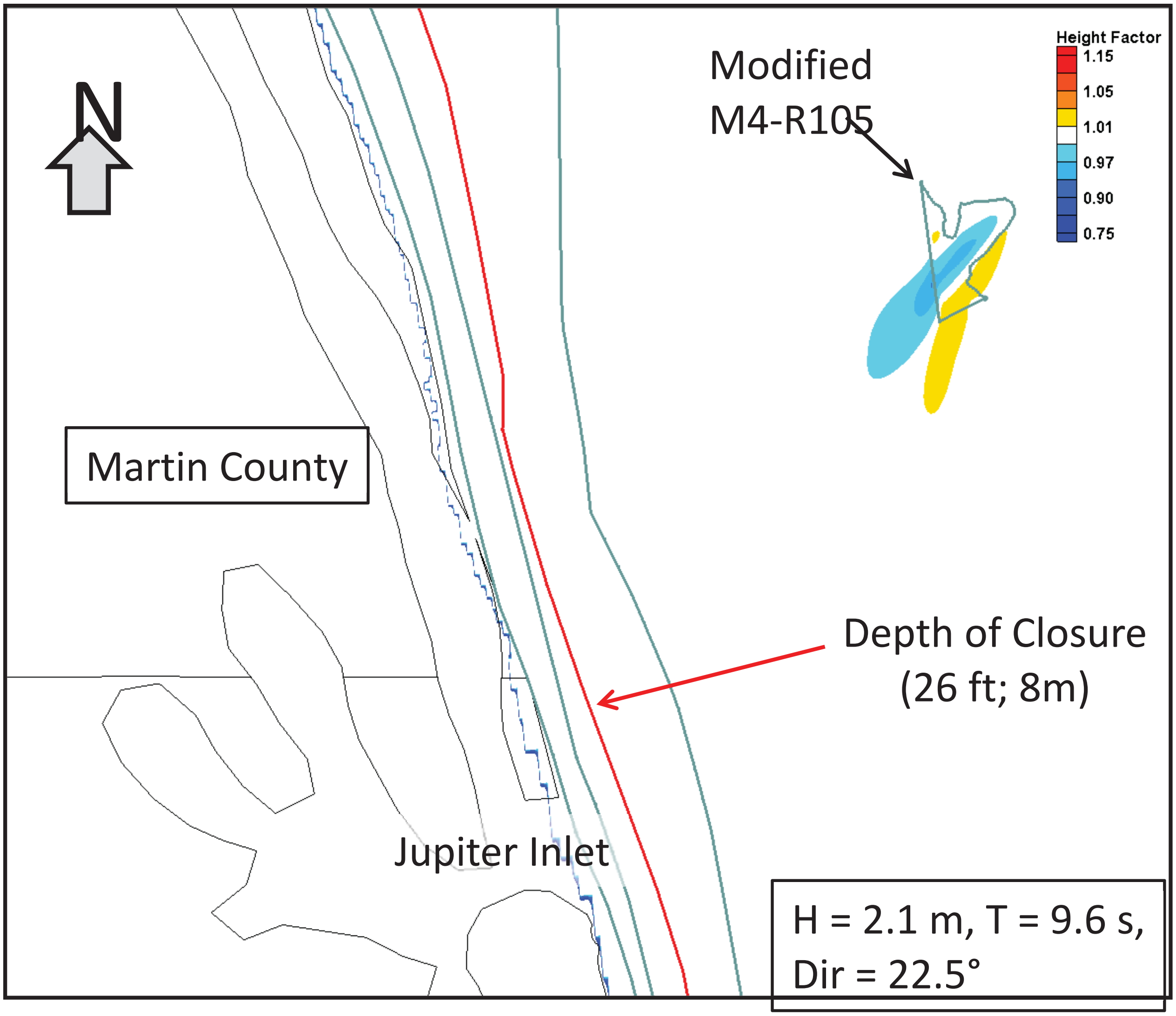

Figure 24. Wave height comparison for wave condition #3b at M4-R105.

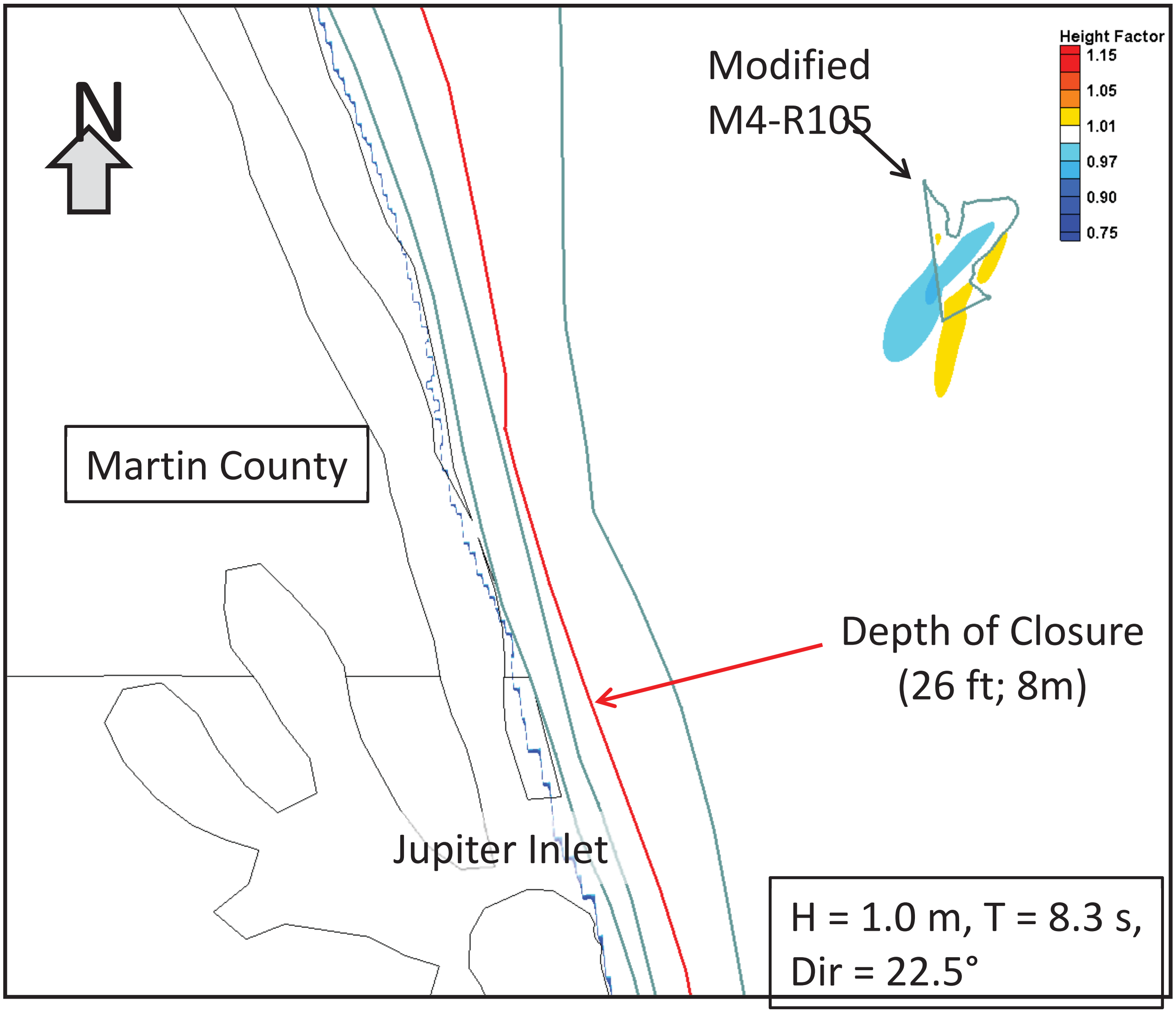

Figure 25. Wave height comparison for wave condition #4b at M4-R105.

3.2 Wave Direction Comparisons

The comparison of with- and without-project conditions indicates that the maximum wave period input Condition 2a resulted in the greatest impact to wave directions landward of the SL10-T41 borrow area. The changes to the wave direction for all model runs of SL10-T41 occur seaward of the 39 ft (12 m) depth contour as well as the depth of closure (8 m) contour (see Figure 26 through Figure 28). Although Conditions 1 and 2b showed significant change around SL10-T41, Condition 2a altered the wave directions closest to the shoreline. Condition 3b represented the greatest impacts to wave directions closest to the shoreline for all of model runs 3 and 4 (i.e. 3a through 3c and 4a through 4c). But it is important to note that in no cases were any changes in the wave field noted landward of the depth of closure.

The M4-R105 borrow area showed greater response to the maximum wave height events (Conditions 1a and 1b) versus the maximum wave period events (Conditions 2a and 2b) as seen in Figure 29 and Figure 30. Monthly average wave conditions (Conditions 3a through 3d) and overall average wave conditions (Conditions 4a through 4d) were only minimally affected by excavation of the M4-R105 borrow area. Figure 31 shows the greatest impact of the model runs for Conditions 3a through 3d and 4a through 4d. Again, in the average and even in the most energetic input wave conditions, no impacts were observed landward of the depth of closure.

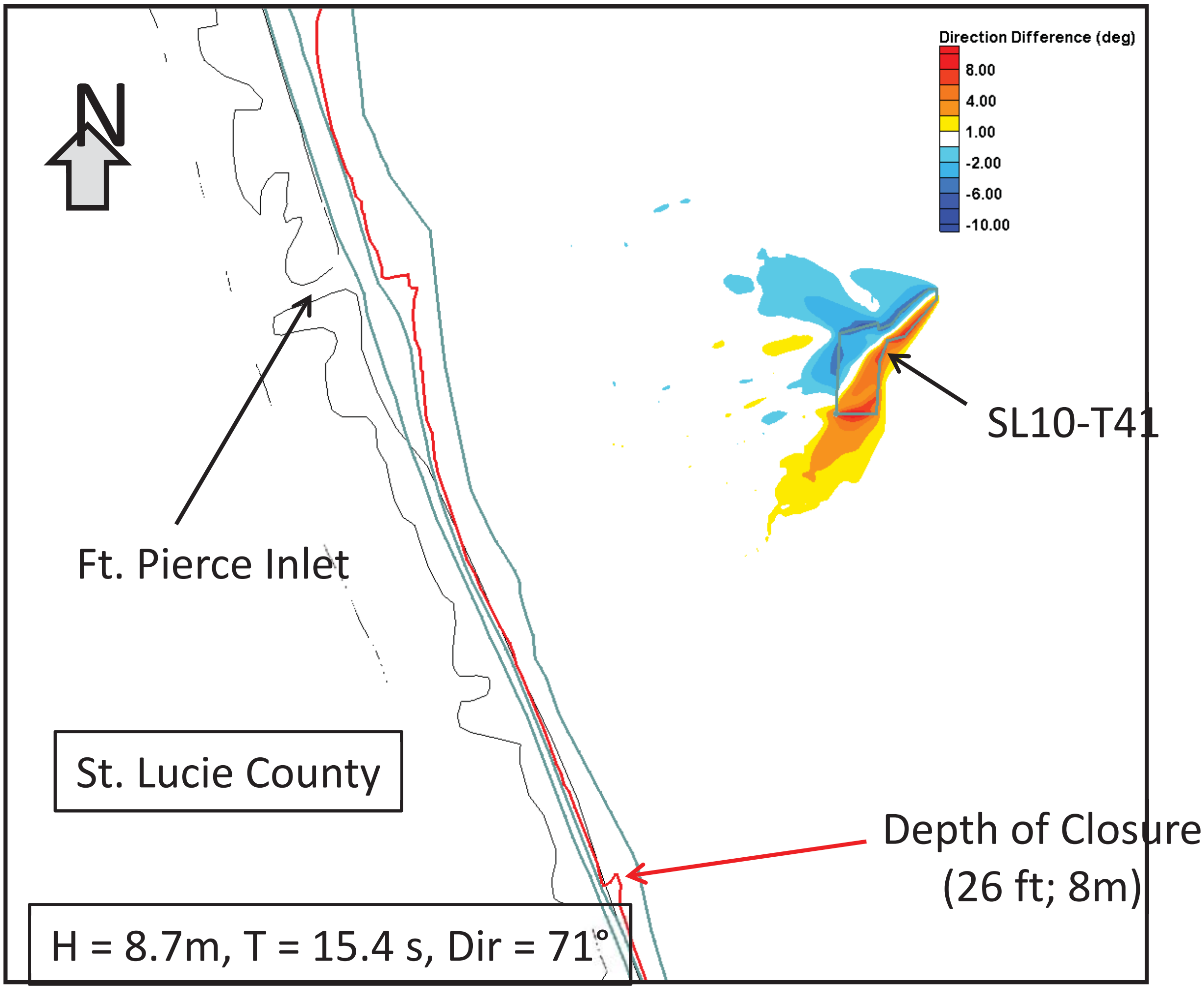

Figure 26. Wave direction comparison for wave condition #1 at SL10-T41.

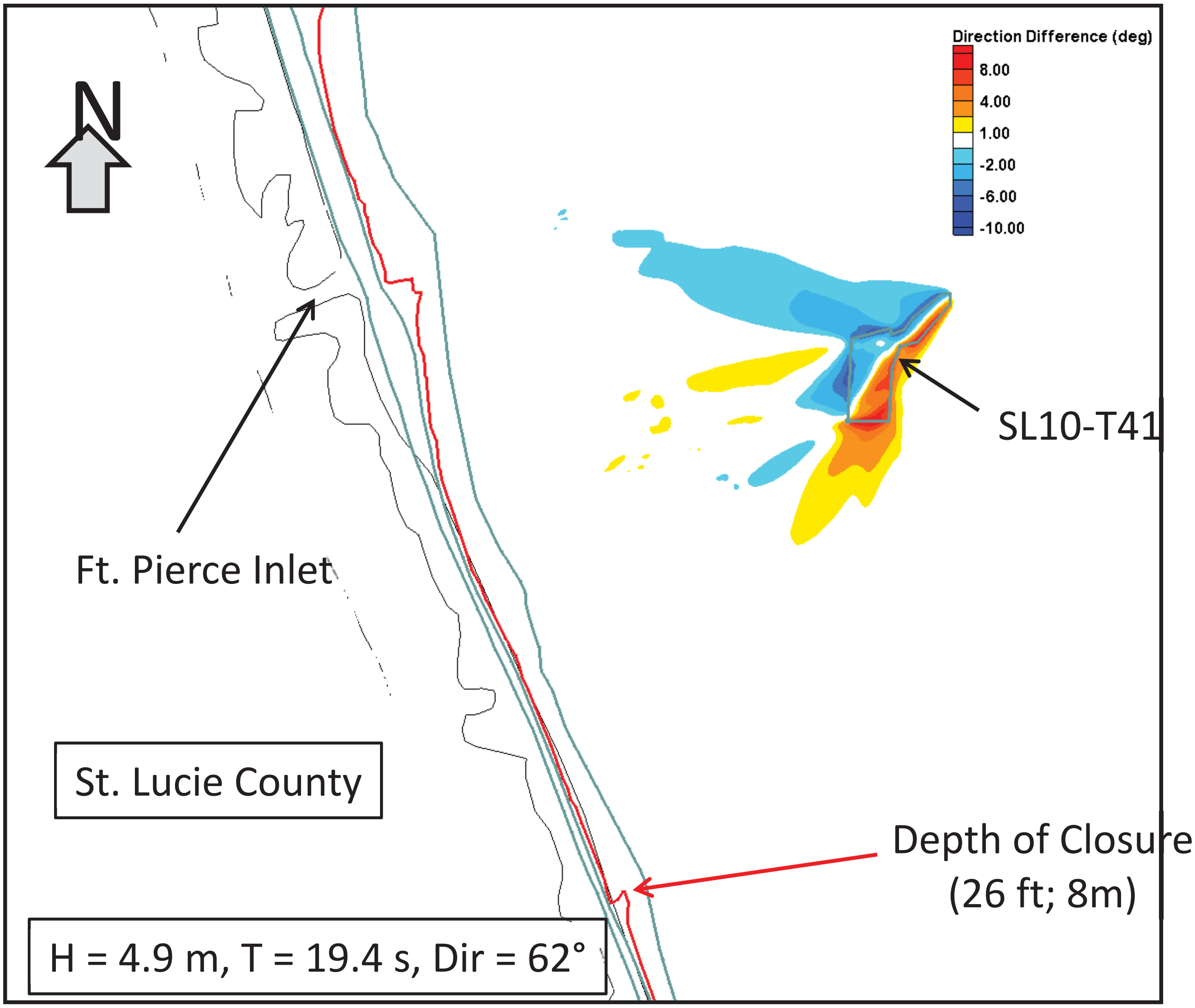

Figure 27. Wave direction comparison for wave condition #2a at SL10-T41.

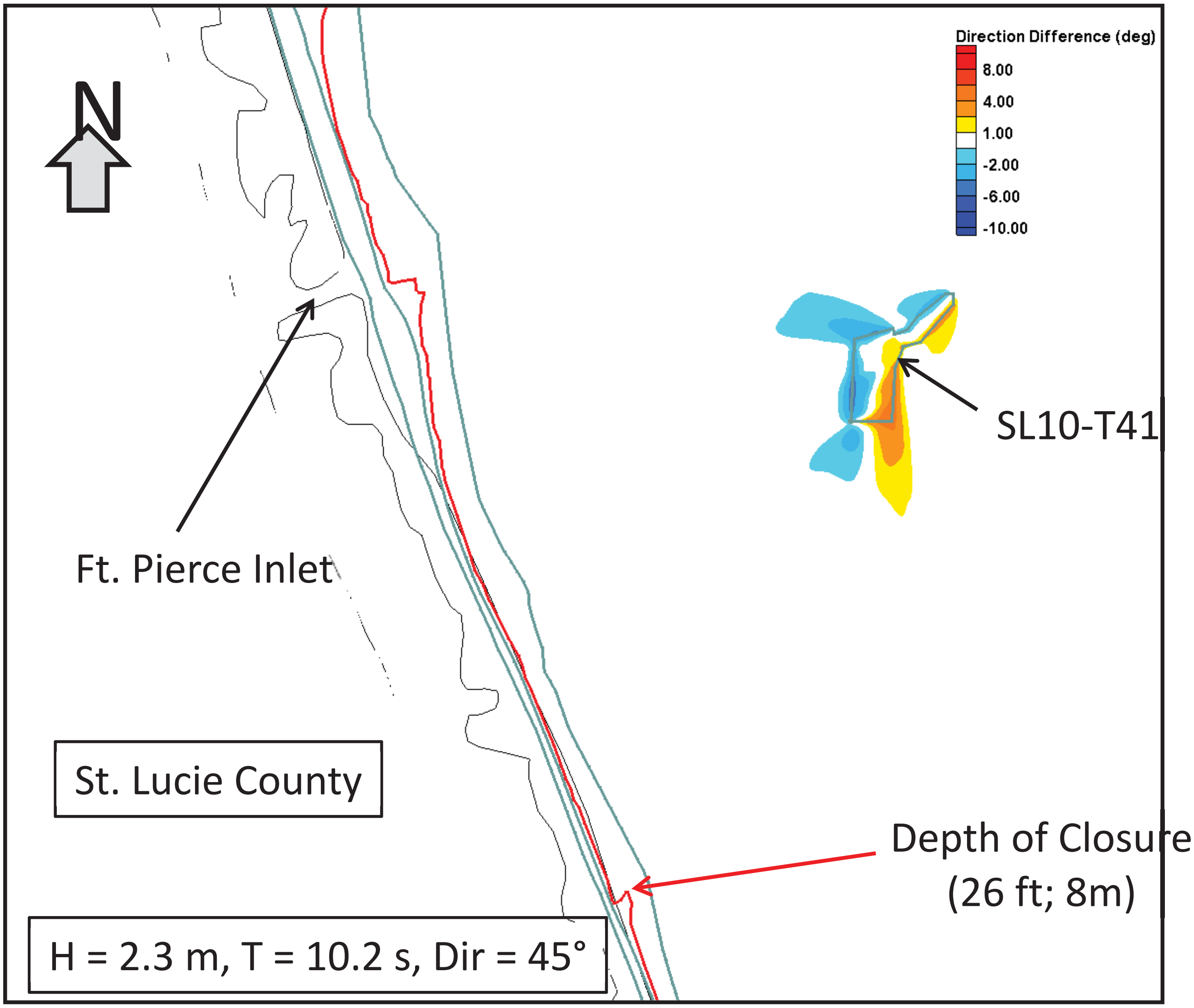

Figure 28. Wave direction comparison for wave condition #3b at SL10-T41.

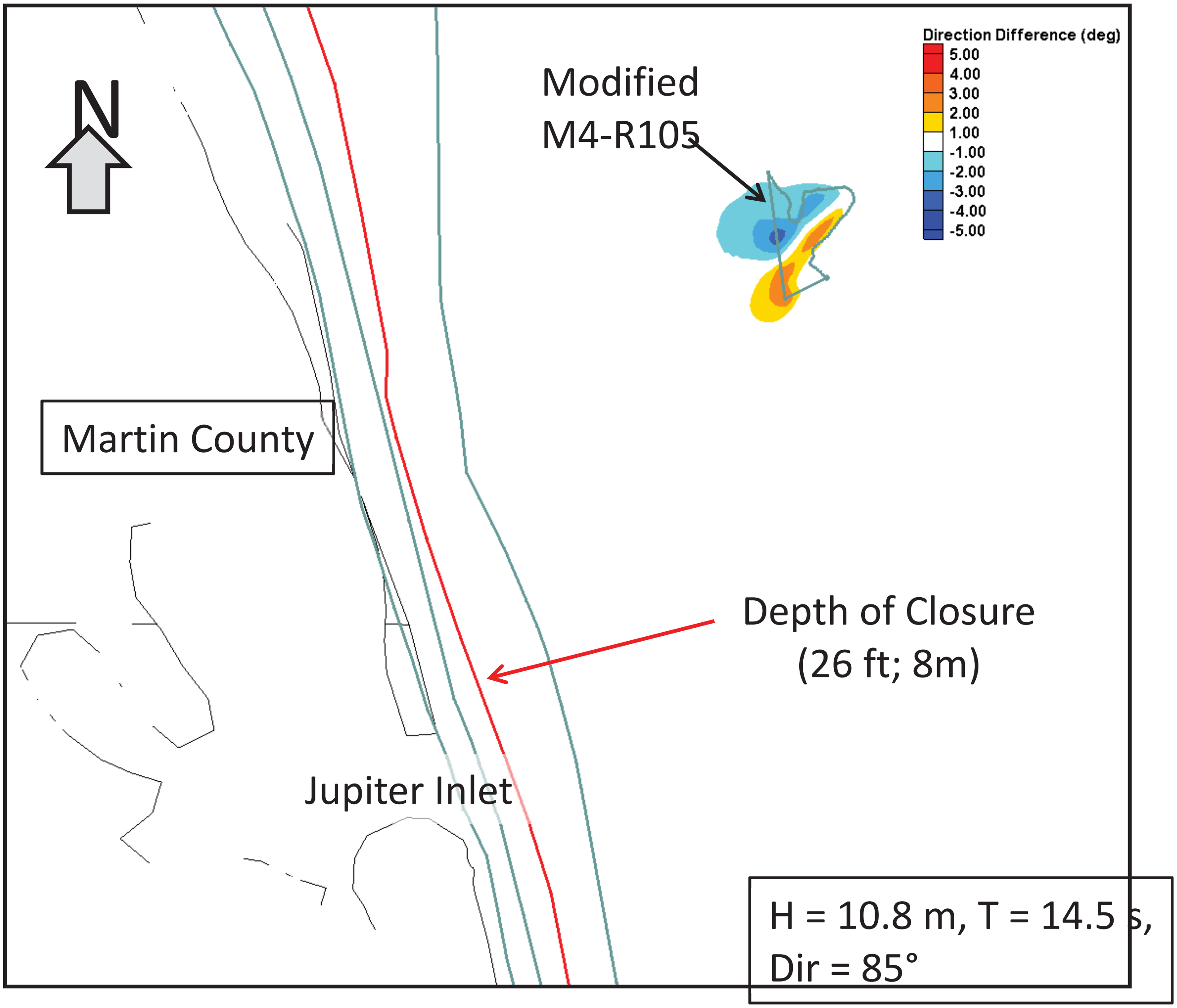

Figure 29. Wave direction comparison for wave condition #1b at M4-R105.

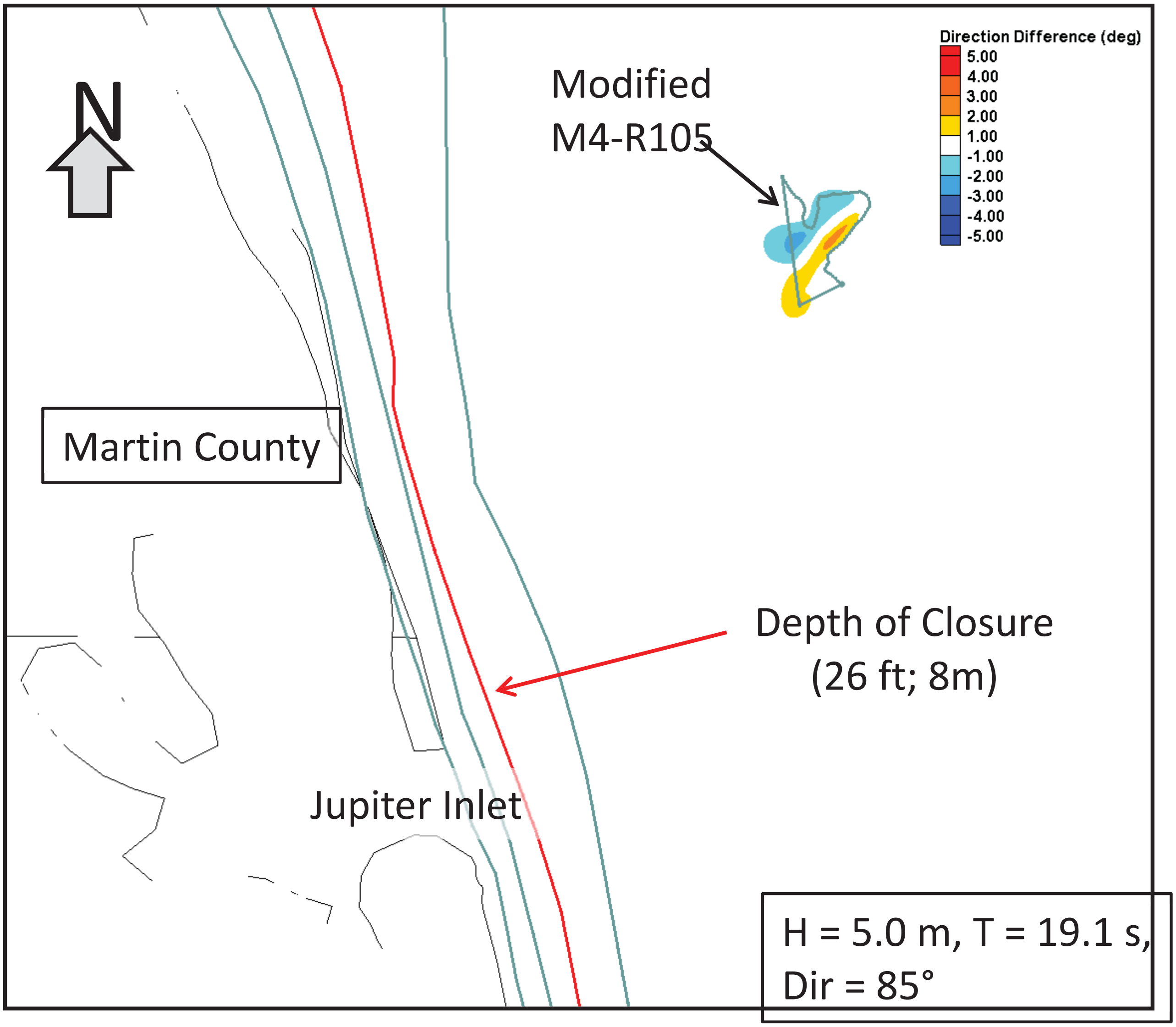

Figure 30. Wave direction comparison for wave condition #2b at M4-R105.

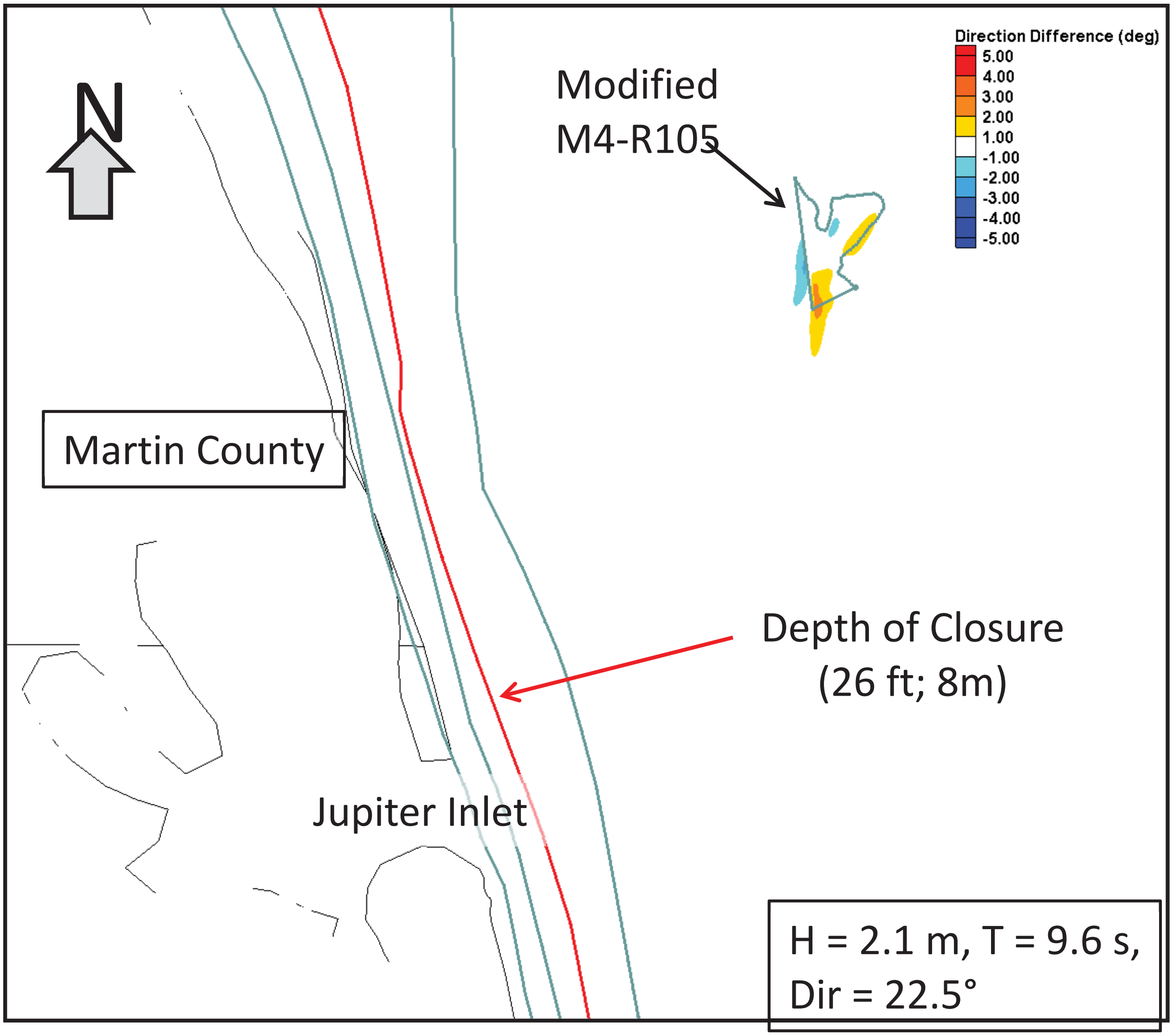

Figure 31. Wave direction comparison for wave condition #3b at M4-R105.

3.3 Sensitivity Analysis

The with- and without-project conditions were tested for sensitivity to the Manning's N friction factor used in CMS-Wave in lieu of a typical model calibration. Model calibration was not possible since collected wave data were not available for the modeled areas. The friction factor was varied from 0.0 to 0.5, but mostly focused around the default parameter value of 0.025, as seen in Table 11. Wave heights were compared at various depths to see how much change resulted due to changing input friction values. The propagation of the wave for input condition #1 resulted in very little difference in wave heights for the range of Manning's N values centered on the default (0.025). For the most extreme value of Manning's N applied (0.500), the input wave was drastically reduced before even arriving at the first comparison location in the lee of the borrow area in 56 ft (17 m) water depths.

Table 11. Wave height variation (ft) with depth and friction factors for SL10-T41 model grid under Condition 1.

Depth (ft)	**Manning's Friction Value**							
	0.000	**0.005**	**0.015**	**0.025**	**0.035**	**0.055**	**0.100**	**0.500**
56	25.7	26.4	26.3	26.3	26.1	25.6	23.1	5.2
44	20.3	20.1	20.1	20.1	20.0	20.3	10.3	4.0
34	14.3	14.6	14.7	14.3	14.7	14.6	15.0	2.6
30	12.2	12.6	12.6	12.6	12.2	12.1	11.9	1.8
27	11.9	12.0	12.0	12.0	11.9	11.8	11.1	1.4
17	7.5	7.5	7.5	7.5	7.5	7.4	7.4	1.3

Although the input friction value was varied, sometimes resulting in drastically reduced wave heights (i.e. for Manning's N value of 0.5), the objective of this study was to quantify the changes to the nearshore transport regime. As presented in the Wave Height Comparison section, wave height factors were created for the with- versus without-project condition to show the spatial extents of the borrow area impacts. Changes to wave height and wave direction inside of the 26 ft (8 m) contour (depth of closure) were not observed for any friction factor investigated nor for any input condition that was modeled. Select outputs for SL10-T41 are presented in Figure 32 to Figure 35 below for friction factors of 0.000 and 0.100 and wave input Conditions 1 and 2a. These figures can be compared with those generated using the default friction value (0.025; see Figure 14 and Figure 15) and very little change is observed. The change in wave field for M4-R105 was also very low. Figure 36 and Figure 37 provide comparison for wave input Condition 2a with Figure 23 as the default friction value (0.025) for reference.

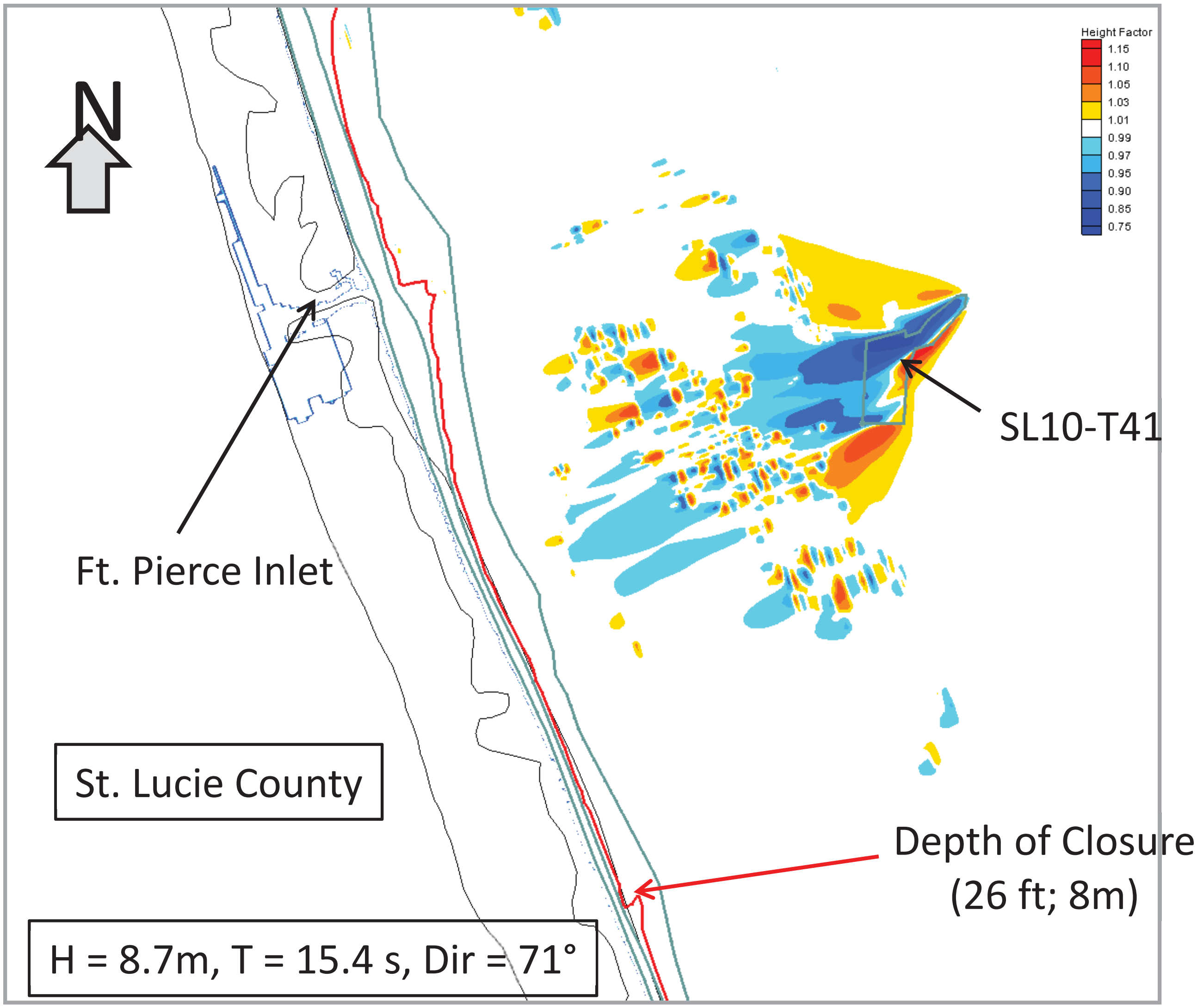

Figure 32. Wave height comparison for SL10-T41 Condition 1 with Manning's N = 0.000.

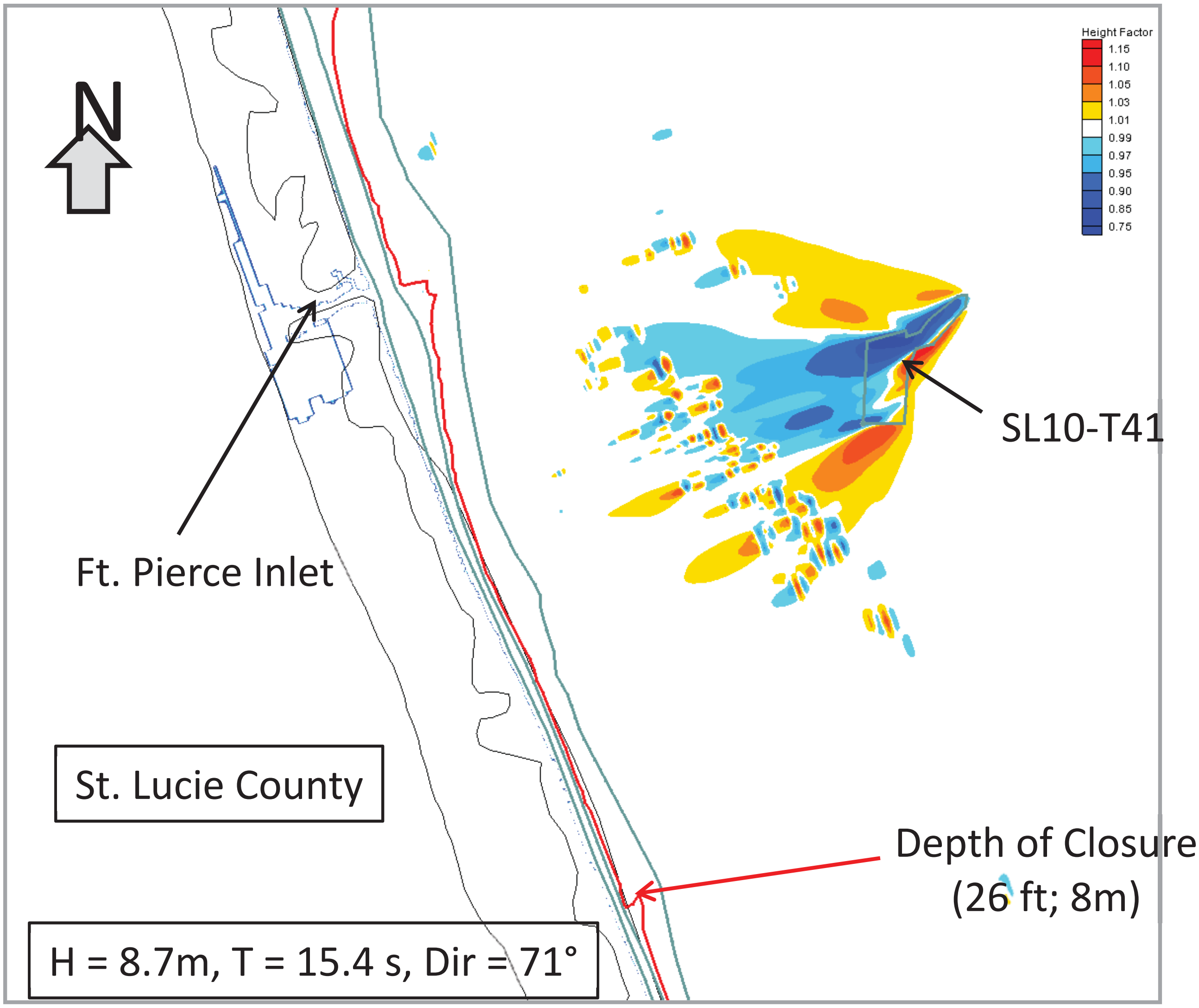

Figure 33. Wave height comparison for SL10-T41 Condition 1 with Manning's N = 0.100.

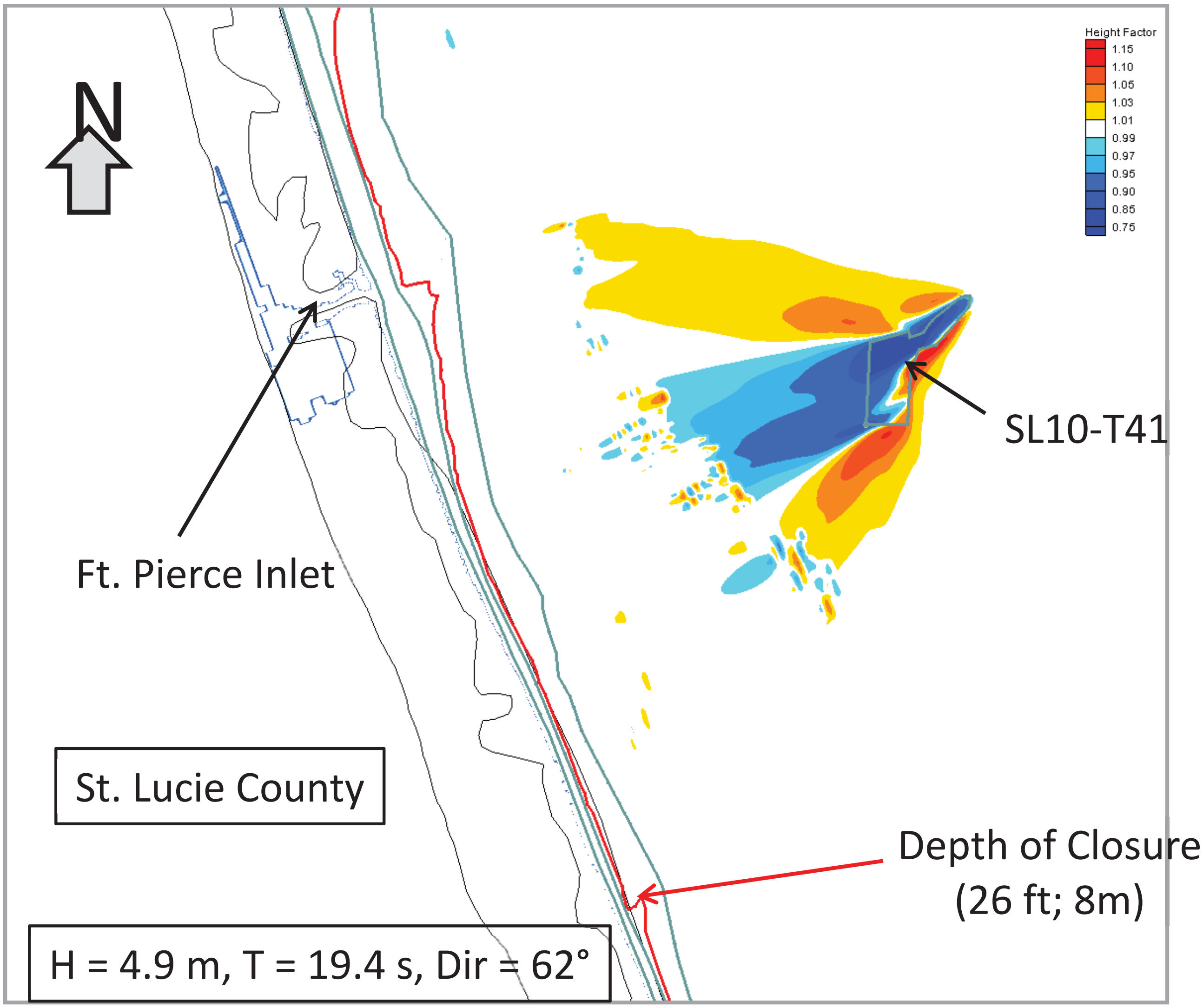

Figure 34. Wave height comparison for SL10-T41 Condition 2a with Manning's N = 0.000.

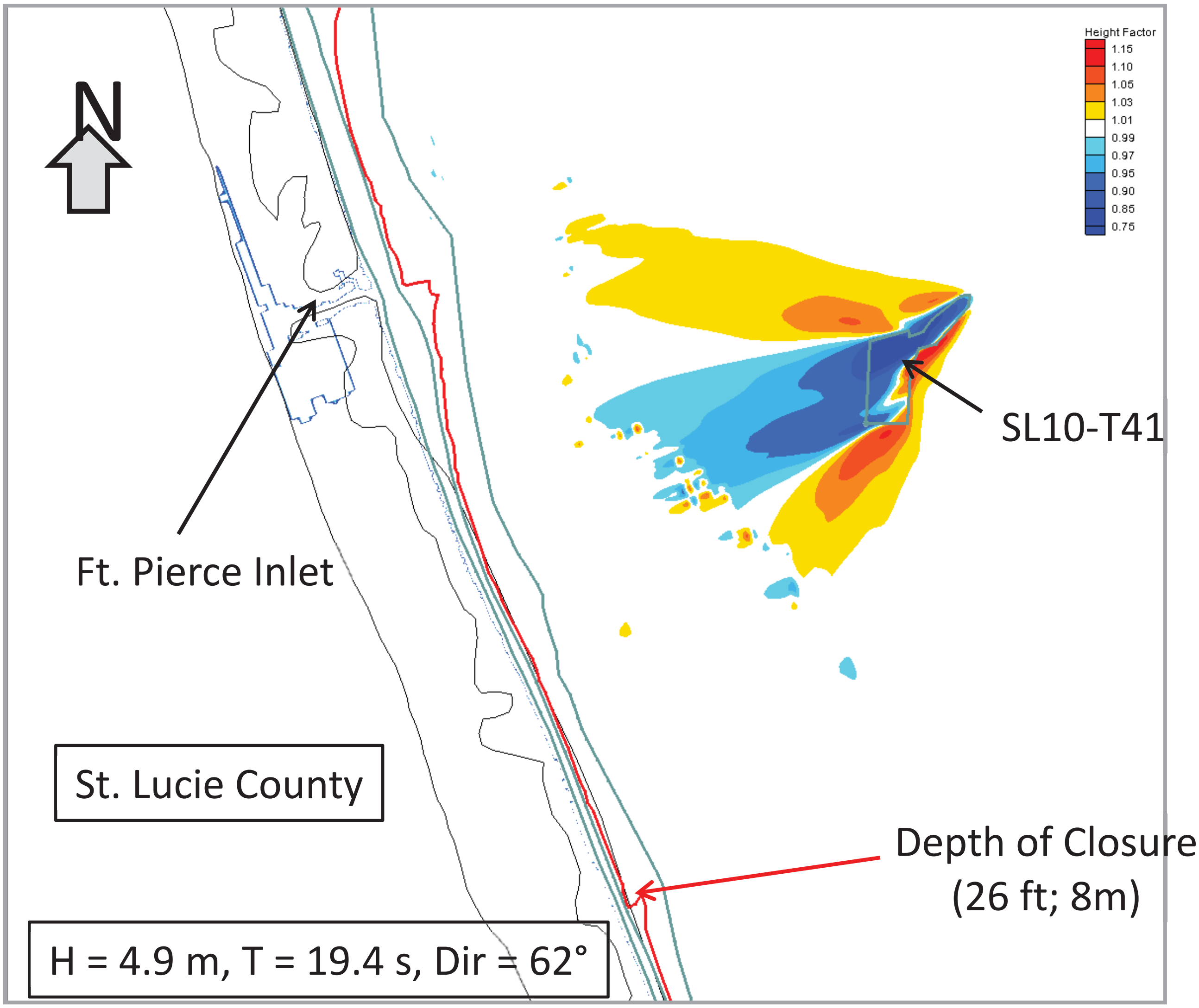

Figure 35. Wave height comparison for SL10-T41 Condition 2a with Manning's N = 0.100.

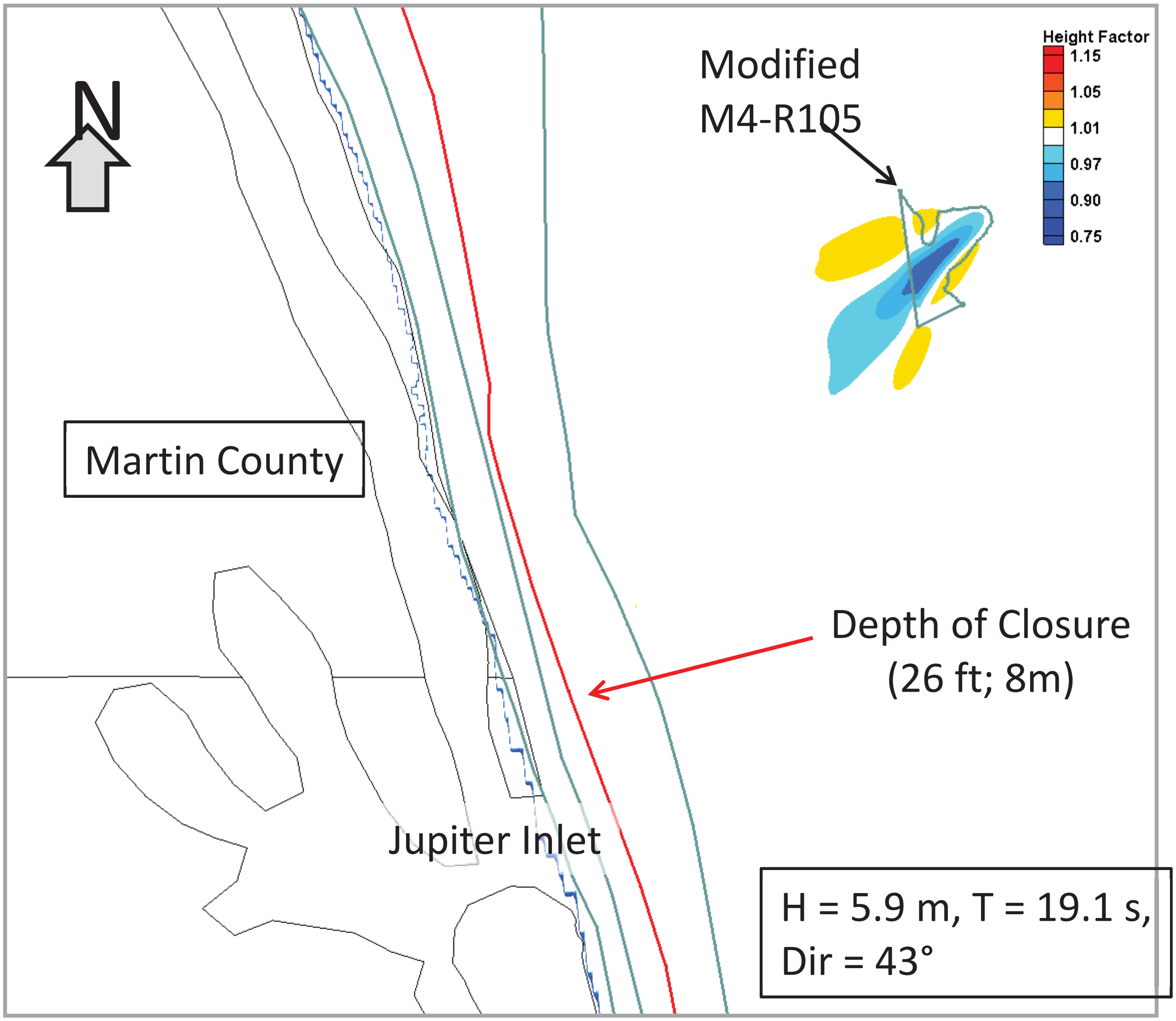

Figure 36. Wave height comparison for M4-R105 Condition 2a with Manning's N = 0.000.

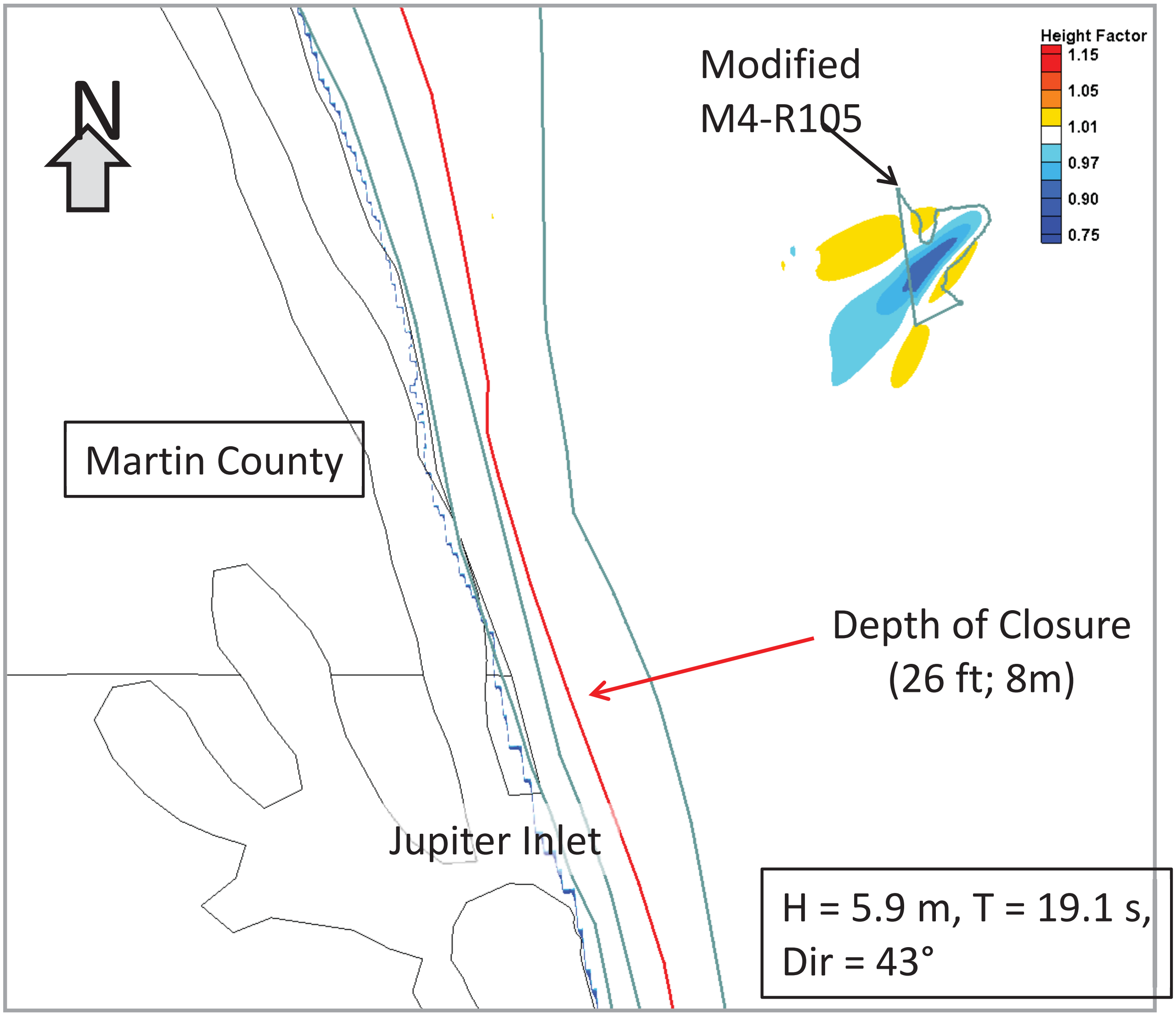

Figure 37. Wave height comparison for M4-R105 Condition 2a with Manning's N = 0.100.

4.0 CONCLUSIONS

Proposed borrow areas SL10-T41 and M4-R105 were assessed for changes to existing wave refraction patterns that could result from excavation. Any such changes could in turn cause impacts to sediment transport patterns observed along the shoreline. Wave heights and wave directions developed from extreme and average conditions observed in the WIS hindcast record for output stations near the borrow area locations were modeled under the existing (without-project) condition and with-project conditions.

Although the M4-R105 borrow area is located closer to the shoreline than SL10-T41, the average depth change over the modeled area equaled only 4.1 ft (1.2 m). Impacts to the wave field did not extend as far from the M4-R105 borrow area as those observed for SL10-T41 where the average depth change equaled 15.8 ft (4.8 m). As modeled, the excavated area of SL10-T41 was 5 times greater than M4-R105 and the volume of material removed was about 30 times greater. Note that both borrow areas were modeled with greater excavation than what is planned for the Dade County BECP. The expected dredging volumes in support of the Dade County BECP (as found in Geotechnical Appendix D) equal 4,600,000 cy and 600,000 cy for SL10-T41 and M4-R105, respectively. The excavated volume as modeled equaled 30,800,000 cy and 1,400,000 cy, respectively, over a greater footprint. Further, in the case of SL10-T41, the cut depth was deeper than that proposed by Dade BECP. Increasing the volume and footprint of the proposed dredged areas during model simulations provided a more conservative analysis of the impacts to the wave field caused by dredging.

Under the most extreme wave conditions, impacts were observed no closer than 11,500 ft (2.2 miles) and 13,500 ft (2.5 miles) seaward of the depth of closure for the SL10-T41 and M4-R105 borrow areas, respectively. There were insignificant changes landward of the 39 ft (12 m) depth contour for both locations that were attributed to model noise as a result of wave breaking due to steepness and would not impact sediment transport at the shoreline. For the more frequently occurring low-energy (average) wave conditions, the effects of borrow area excavation are confined to the immediate vicinity of the excavated area. Therefore, impacts to the sediment transport patterns at the shoreline are not expected due to excavation of SL10-T41 or M4-R105 by the Dade County BECP.

5.0 REFERENCES

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