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WIGGINS PASS, COLLIER COUNTY, FL
NUMERICAL MODELING OF WAVE PROPAGATION, CURRENTS AND MORPHOLOGY CHANGES
PHASE II: NUMERICAL MODELING OF ALTERNATIVES REPORT

PREPARED FOR:

COLLIER COUNTY
WIGGINS PASS MODELING EVALUATION WORKING GROUP
AND
COASTAL ZONE MANAGEMENT DEPARTMENT

PREPARED BY:

COASTAL PLANNING & ENGINEERING, INC.

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Cover page aerial photograph provided by Capt. John E. Findley, Pelican Isle Yacht Club. Taken by Florida Aerial Service February 2008.

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INTRODUCTION

A numerical modeling study was conducted to evaluate potential solutions to improve inlet navigation conditions and reduce erosion on adjacent beaches at Wiggins Pass, Collier County, FL (Figure 1). A numerical model known as Delft3D, developed by Deltares (Delft, The Netherlands), with its sub-modules for the simulation of waves, currents, sediment transport and coastal morphology (bathymetry) change was used in this study.

The study consisted of the analysis of different inlet channel dredging designs (non structural alternatives) to evaluate the effectiveness of each design in achieving the main project goals of improved navigation and erosion mitigation. The numerical modeling activities are divided into three main phases: (1) Field measurements of waves and currents using ADCPs and bathymetry measurements; (2) Numerical model calibration of waves, flows and morphology: and (3) Numerical model production runs. Each phase of the work was coordinated with a special committee of the Collier County Coastal Advisory Committee (CAC) at four meetings in April, June, August and October 2008. The Wiggins Pass Modeling Evaluation Work Group consisted of citizens of Collier County (volunteers) representing boating, environmental and local interest groups; and a representative from the State Parks Service. Interim PowerPoint presentations of modeling results were also given to members of FDEP staff during the study. Over 15 alternatives were tested and discussed with the work group at four meetings. The minutes of these meetings are presented in Appendix IV, which also lists the committee members.

The Group Leader, Heidi Kulpa, a CAC board member developed the purpose of the study for the group:

1. Provide a safe channel for boating.
2. Address the erosion at Barefoot Beach.
3. Lengthen the dredge cycle and accomplish it with the least effect on the environment.
4. The solution needs to be economically effective.

The study indicates that improvement of navigation conditions and mitigation of erosion of Wiggins Pass can be achieved by redesigning the navigation channel and modifying the placement location of the dredged material. A straight navigation channel with smaller dimensions than the current dredge template that extends across the flood shoal is the optimal channel configuration. This new channel location is aided by temporary sand dikes blocking the existing channel meander and swash (marginal) channels that wrap around the end of Barefoot Beach. These two features allow for maximum flow efficiency by directing all the flows to a single channel outlet and reducing sand lost from Barefoot Beach to the navigation channel. Also, fill placement closer to Barefoot Beach, between R15 and R13, is recommended as a means of mitigating erosion problems at this location. Past disposal practices placed dredged material where it encourages a sediment deficit in the inlet's vicinity.

Figure 1. Location diagram of Wiggins Pass.

STUDY AREA

Collier County is located on the Gulf of Mexico in Southwest Florida, approximately 115 miles south of the entrance of Tampa Bay and about 120 miles directly west of Fort

Lauderdale, Florida (Figure 1). The Everglades lie to the southeast. Collier County has 35 miles of accessible coastline, which consists of a series of barrier islands and mainland extensions sub-divided by lagoons, rivers, and inlets. North of the Ten Thousand Islands, Collier County has 9 inlets, Wiggins Pass being the farthest north

Wiggins Pass is located 5 miles north of Clam Pass and 5.9 miles south of Big Hickory Pass. Wiggins Pass is a natural inlet which provides access to the Gulf of Mexico for a number of embayments and rivers along three main interior channels. The eastern channel connects to the Cocohatchee River. The southern channel connects to Water Turkey Bay and Vanderbilt Lagoon, and the northern channel connects to the lagoon between Little Hickory Island and the mainland. This northern lagoon consists of a number of embayments including May Flower, Palm Vines, Little Hickory Bays, and eventually connects to Big Hickory Pass. There is no interior waterway connection to Clam Pass. Population centers near the pass include Vanderbilt Beach and Naples Park to the south and Bonita Shores to the north. Wiggins Pass is bounded by public lands which extend 7950 linear feet to the north and 6,010 linear feet to the south. Barefoot Beach State Park and Lely Barefoot Beach lie to the north and Delnor Wiggins Pass State Recreation Area is to the south (Figure 1).

Wiggins Pass is currently dredged at regular intervals of approximately 2 years to maintain navigable depths for recreational boaters. The pass provides boat access to the Gulf from inland waterways and lagoons at Vanderbilt Beach, the Cocohatchee River, Wiggins Pass Marina and several other private and public marinas and ramps.

BACKGROUND

The dredging history of Wiggins Pass is summarized in detail by Coastal Planning & Engineering, Inc. (CPE 1995) and Humiston and Moore (H&M 2007), and a brief summary is presented below.

The U.S. Army Corps of Engineers (USACE) initiated a study of improvements for small boat navigation at Wiggins Pass Inlet in 1978, which was completed in 1980. The study described the existing conditions at the pass as a hazard to navigation of small boats due to shallow waters and shifting banks and recommended dredging for improvement of navigation safety. In 1982, Coastal Engineering Consultants (CEC) performed a hydraulic study and applied for a permit to dredge the pass in a configuration that reassembled the USACE recommended plan. The USACE recommended plan consisted of a 200 ft wide channel with 8 ft deep MLW (9.68 ft NAVD) on the Gulf side and a eastern interior channel with 5 ft MLW (6.68 NAVD) depth with a width varying from 200 ft at its intersection to 50 ft at its easternmost part.

Wiggins Pass was first dredged to the specifications described above in 1984 following the study of CEC (1982). The dredged material was placed on Delnor Wiggins State Park. CEC (1989) reported on the results of Wiggins Pass after the initial dredging. Their findings indicated that the sand that was placed south of the Inlet at Delnor Wiggins State Park migrated towards the inlet, forming a spit that interrupted portions of the navigation channel. By 1989, erosion problems Barefoot Beach, started to become apparent. After this event, the channel was dredged at periodic intervals between 1 to 4 years.

CPE (1995) developed an inlet management plan where several alternatives for navigation and channel maintenance improvement were evaluated. CPE (1995) recommended channel modifications that included increased channel depth to -12 ft MLW (13.68 NAVD) to provide an additional 4 ft of advanced maintenance dredging and an extra 50 feet channel width in the Gulfside to serve as a deposition basin in order to increase the interval between dredging events to maintain an operational channel with 8 feet MLW. CPE (1995) also recommended placing the disposal material to the south and north of Wiggins Pass to mitigate for erosion of shorelines depending on monitored conditions.

H&M (2004) conducted a feasibility study to evaluate channel dredging practices and erosion at Barefoot Beach. This study recommended adjustment of the dredging template to smaller dimensions similar to the dimensions recommended by the USACE (1980). They also recommended further study of the use of erosion control structures at Barefoot Beach to mitigate Gulf shoreline erosion and flood shoal dredging to mitigate for erosion of Barefoot Beach interior shoreline.

H&M (2007) conducted a hydrodynamic study utilizing two flow models: a tidal-induced flow model (ADCIRC) and wave-tide flow model (CMS). H&M recommended the installation of coastal protection structures (segmented breakwaters, t-groins or jetty) on Barefoot Beach to contain local erosion and a readjustment of the channel dimensions to shallower depths and narrower widths. It was also suggested that the channel be realigned towards the southwest, and in order to enhance hydraulic efficiency and reduce dredging requirements, the channel should be extended by dredging across the flood shoal.

CPE was hired in 2008 to evaluate the effects of various channel configurations on dredging requirements and controlling depths for navigation and erosion of adjacent shorelines. Channel configurations evaluated by CPE, which are described later in this document, included channel realignment, modification of depth and width, interior dredging through the flood shoal, north and south connections, and interior sand dikes. Disposal strategies were also evaluated and are summarized in this report.

For visualization of past inlet configurations the reader is referred to Figures 2, 3, 4, and 5. In 1970, Wiggins Pass Inlet had a semi-symmetrical ebb shoal with natural bypassing. The bypassing bar attached to Barefoot Beach (north) forming a bulge, suggesting sediment bypassing from south to north. The flood shoal was less developed compared to current conditions, with a less pronounced curve in the flood shoal channel. The south ebb shoal lobe was less developed than the northern ebb shoal.

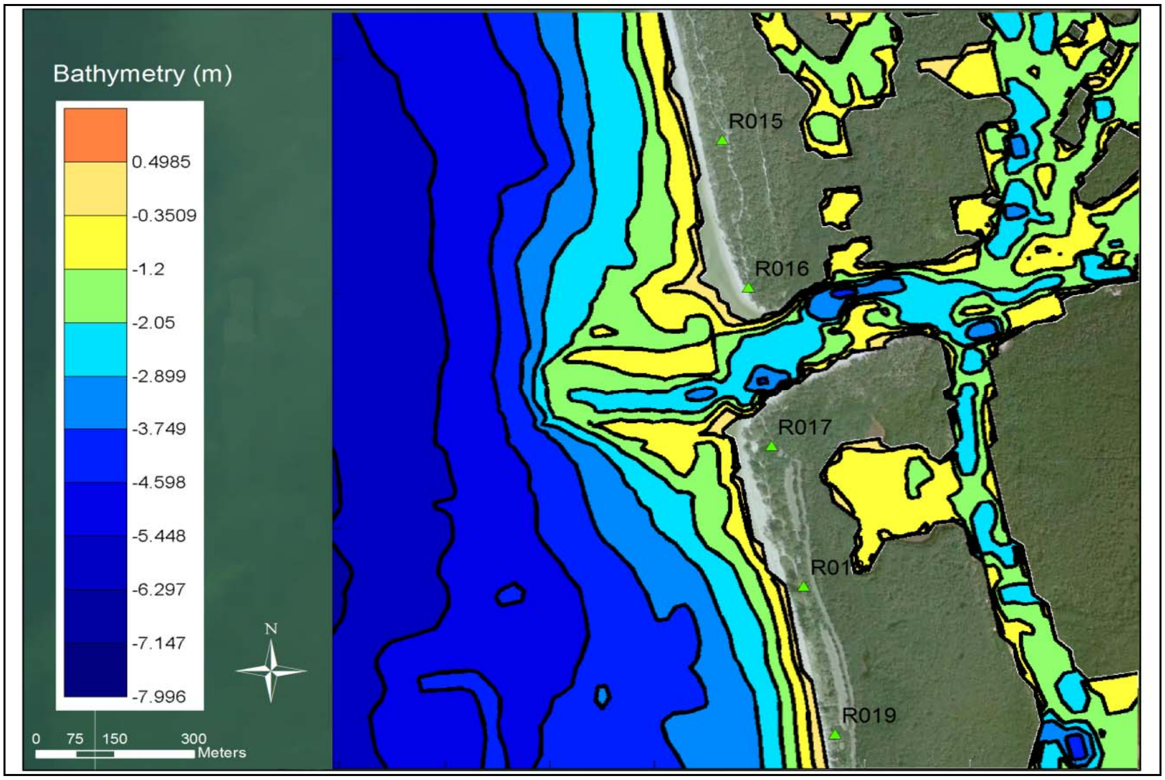


Figure 2. Wiggins Pass bathymetry in the 1970's. Survey data is in m MSL. Horizontal datum is NAD83, Florida State Plane East.

Present inlet configuration (Figures 3, 4 and 5) shows a development of the flood shoal, which forces the flows against the interior shoreline of Barefoot Beach. This allows for the development of the southern ebb shoal lobe and a retraction of the northern ebb shoal lobe. It is believed that the cause for the retraction of the northern ebb shoal lobe is twofold. The main cause is the interruption of the natural bypassing processes due to the dredging of the navigation channel, which has interrupted the natural bypassing of sand from the south lobe to the north lobe; a process that occurs mainly during high-energy south wave events. Some sand from this area may also be lost to the navigation channel during northern wave events and localized transport reversals induced by the ebb shoal morphology. Dredge disposal practices contributed to a deficit of sand just north of the inlet, since sand was either disposed of south of the inlet or too far north. The sediment transports pattern proved to be more complex than the simple north to south net assumed in earlier studies.

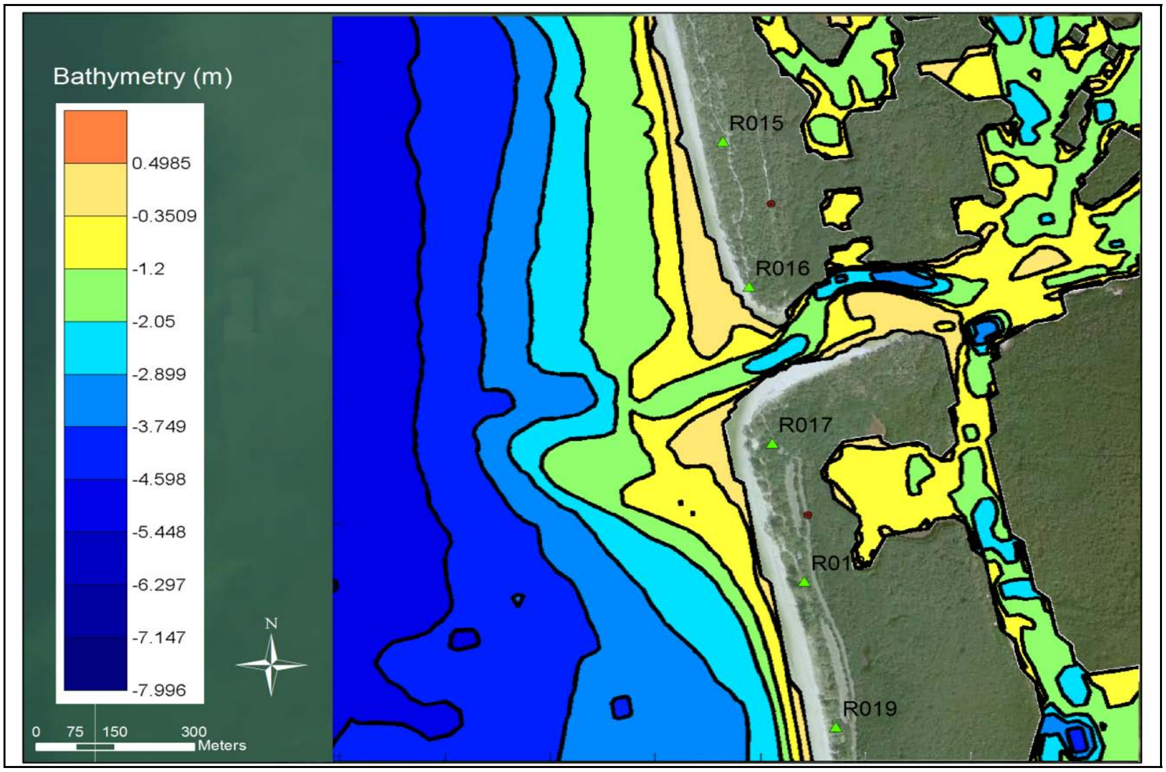


Figure 3. Wiggins Pass bathymetry, October 2004 (Lidar), between dredging events that occurred in 2003 and 2005. Survey data is in m MSL. Horizontal datum is NAD83, Florida State Plane East.

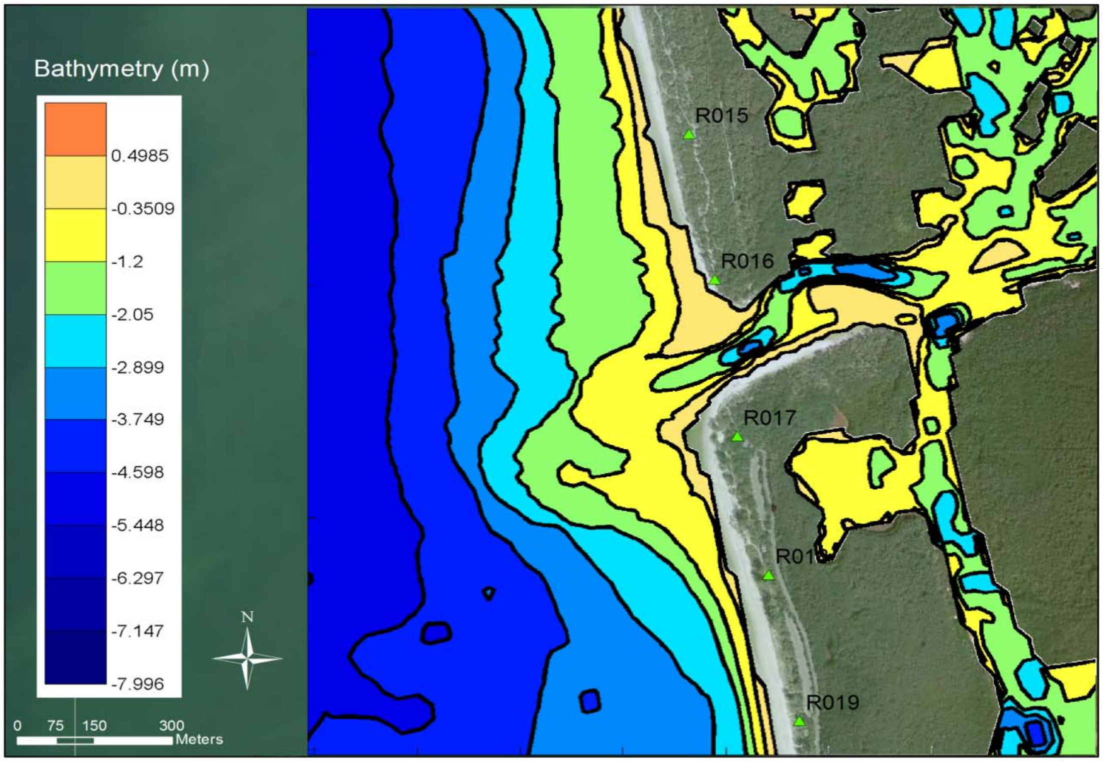


Figure 4. Wiggins Pass bathymetry, January 2007, before the 2007 dredging event and about 2 years after the 2005 dredging event. Survey data is in m MSL. Horizontal datum is NAD83, Florida State Plane East.

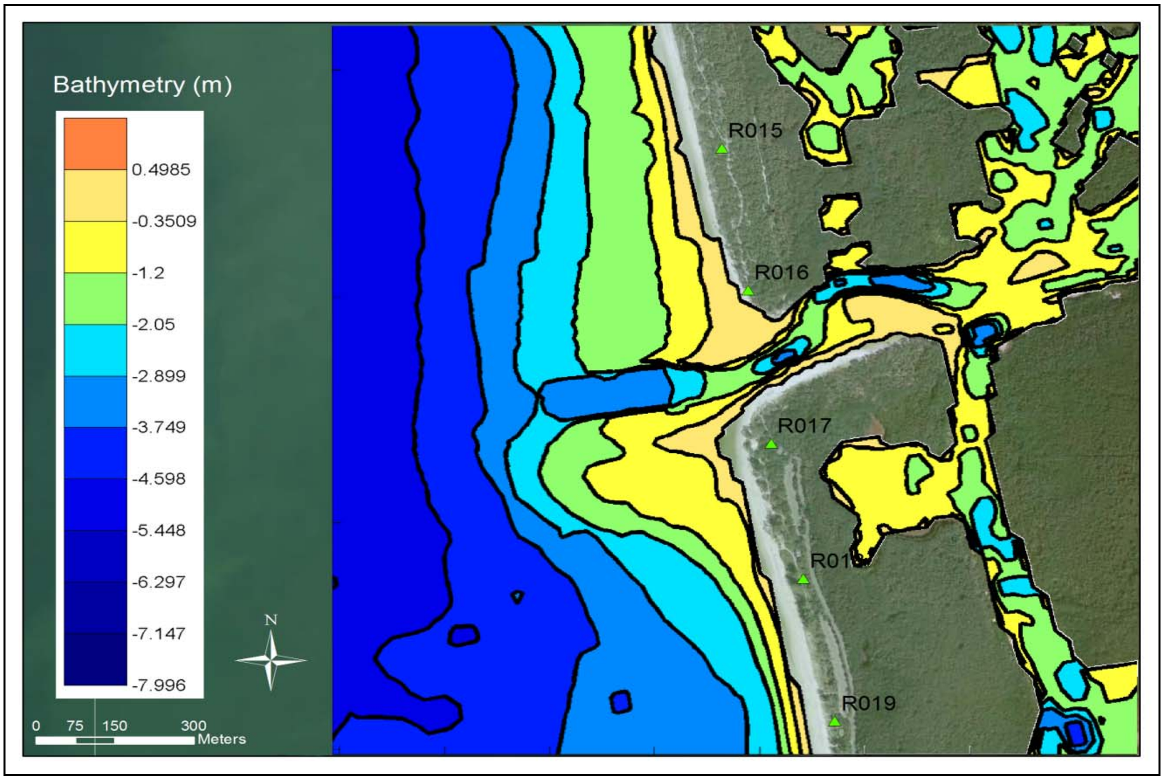


Figure 5. Wiggins Pass bathymetry, February 2007, post-dredge survey. Survey data is in m NAVD, referenced to MSL. Horizontal datum is NAD83, Florida State Plane East.

FIELD MEASUREMENTS

In order to properly calibrate the numerical model, field measurements of waves, currents, tides, and bathymetry were conducted. These measurements are described in this section.

ADCP MEASUREMENTS

CPE deployed two ADCPs (Acoustic Doppler Current Profiler) at the study area to calibrate the wave and current components of the numerical model. The instruments utilized were a Nortek-Aquadopp 1 MHz and a Nortek-AWAC-1 MHz. The location of the deployments is shown in Figure 6. The AWAC was located in 11 m (36 ft) water depth NAVD, and the Aquadopp was located in 4 m (13 ft) NAVD. Pictures of the instruments utilized during deployment are shown in Figure 7.

Measured water levels are shown in Figure 8, and measured current velocities are shown in Figure 9. The tides fluctuated between -0.8 m (2.62 ft) and 0.5 m (1.64 ft). The offshore ADCP (AWAC) and the nearshore ADCP (Aquadopp) presented the same tidal phases, but small differences in amplitude (Figure 8).

Measured currents are shown in Figures 9 and 10. Due to the orientation of Wiggins Pass both u and v components (east-west and north-south components) have significant speeds (Figure 9). The vectorial sum of these components indicates that ebb currents reach up to 1.5 m/s (4.92 ft/s) during flood tide. This velocity is significantly above the 1 m/s threshold, which is referred to as the "equilibrium velocity" for tidal inlets. Normally, an inlet throat with such strong currents would either scour deeper, or become wider; however, this is not observed at Wiggins Pass possibly due to obstructions to scouring such as a consolidated bed or very coarse sediments.

Wave measurements are shown in Figures 11 and 12. The wave roses in Figure 11 show that wave heights varied from 0.1 to 0.9 m (0.3 to 3 ft) in the offshore instrument and from 0.1 to 0.56 m (0.3 to 1.8 ft) in the inlet instrument. The offshore wave directions were from the southwest, while wave directions at the inlet were from the west, due to refraction and diffraction of incident waves by the ebb shoal and inlet channel.

The most significant wave event occurred between May 18 and May 22, 2008. The time-series of wave height and direction during this time period is shown in Figure 12. This event is subsequently used for wave model calibration.

Bathymetry Measurements

For model calibration purposes and characterization of current inlet conditions, CPE conducted a bathymetric survey of Wiggins Pass and its interior channels in June 2008. Materials and methods used in the bathymetry survey are described in CPE (2008). The results of this bathymetric survey are shown in Figures 13 and 14. A comparison of the CPE (2008) bathymetric survey with the 2007 post-dredge survey is shown in Figure 15.

The June 2008 survey shows that the channel has migrated to the southwest, and an ebb-shoal lobe/ridge has crossed the dredged channel (Figures 13 and 14). This has lead to a portion of the dredge template offshore, being cut off from the inshore channel and a migration of the channel to the southwest, following the alignment of the strong ebb current mentioned in the previous section. The current controlling depths in the navigation channel are in the range of 1.2 m (4 ft) NAVD, but can be shallow across the ridge.

Figure 15 shows the bathymetric change between the post dredge survey of February 2007 and the June 2008 survey. Sedimentation is observed within the dredge template, along with a general lowering of the ebb shoal in the surroundings of the dredged area. The total volume of sedimentation within the dredged template is 21,600 m^3/yr (28,200 cy/yr).

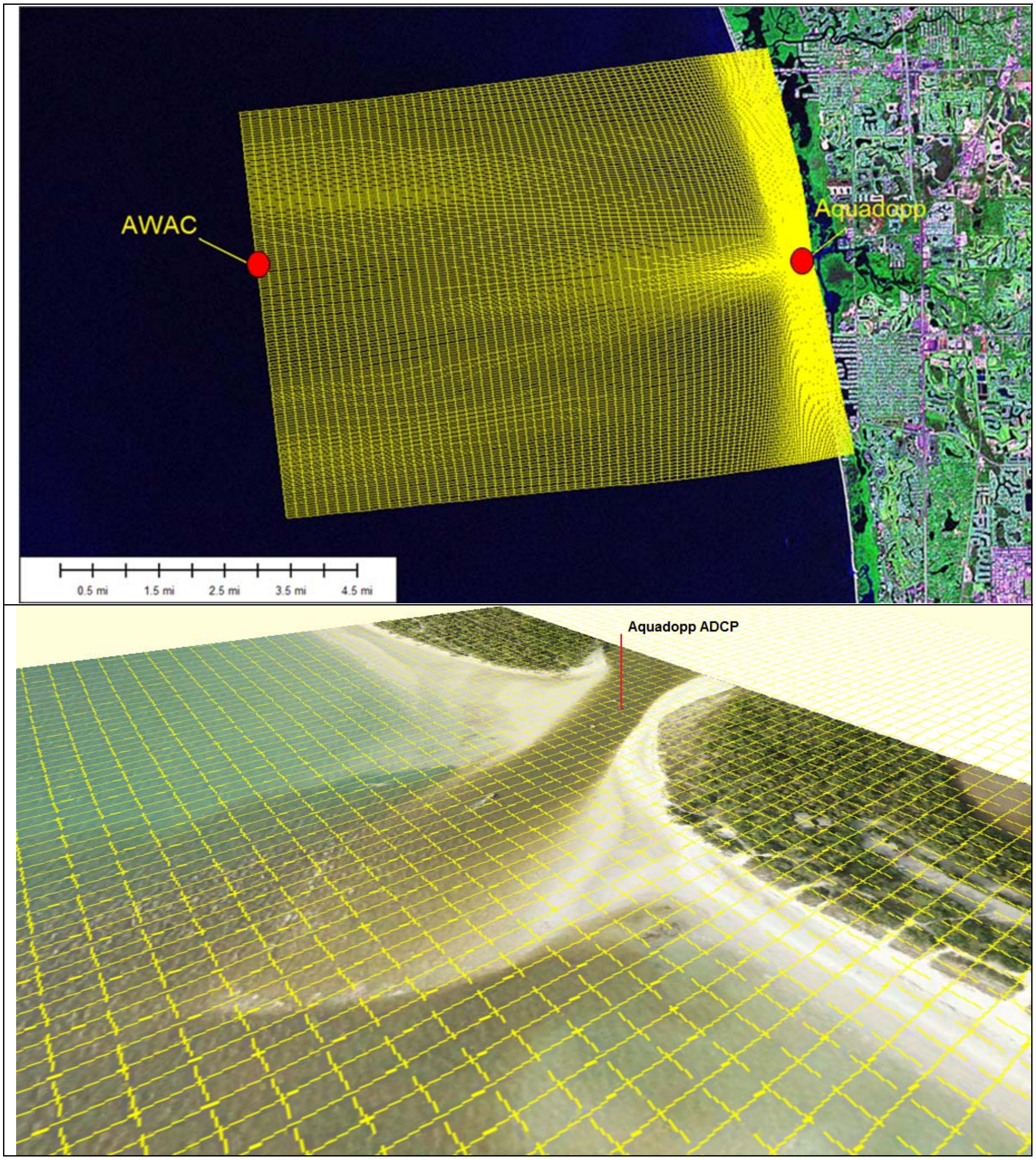


Figure 6. Location of ADCP deployments

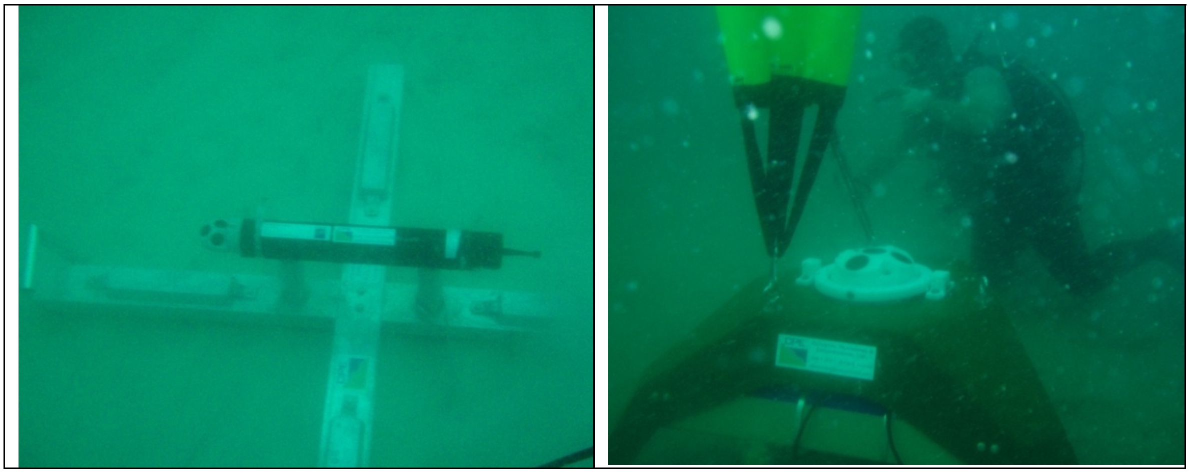


Figure 7. Pictures of the ADCPs utilized in the study. The Aquadopp is shown on the left the AWAC on the right

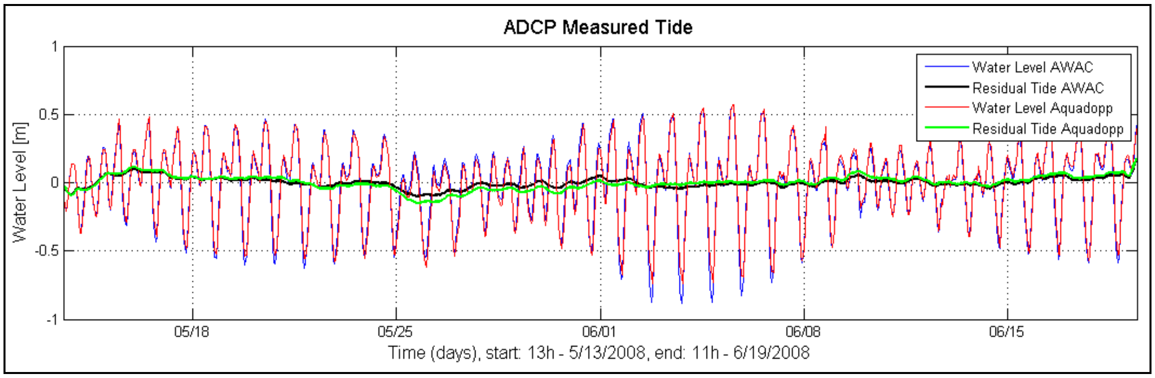


Figure 8. Tide measurements, Wiggins Pass, FL.

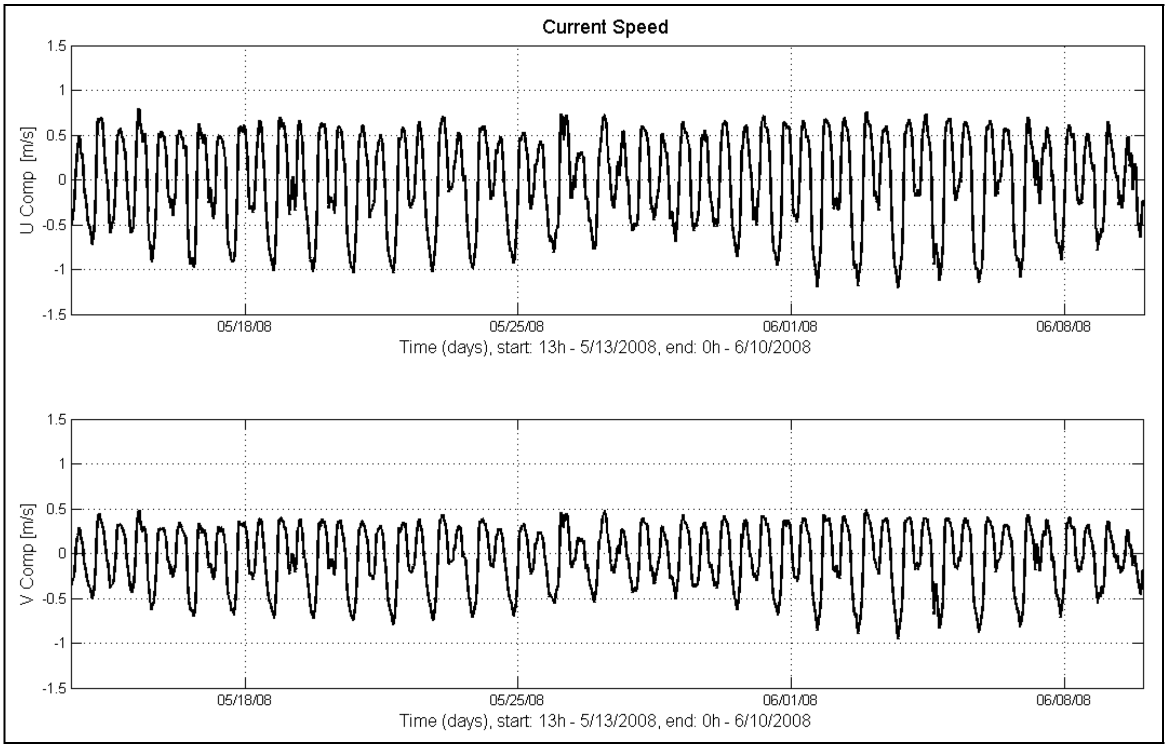


Figure 9. Measured tidal currents, depth averaged. U velocities indicate east-west currents and V velocities indicate north-south currents.

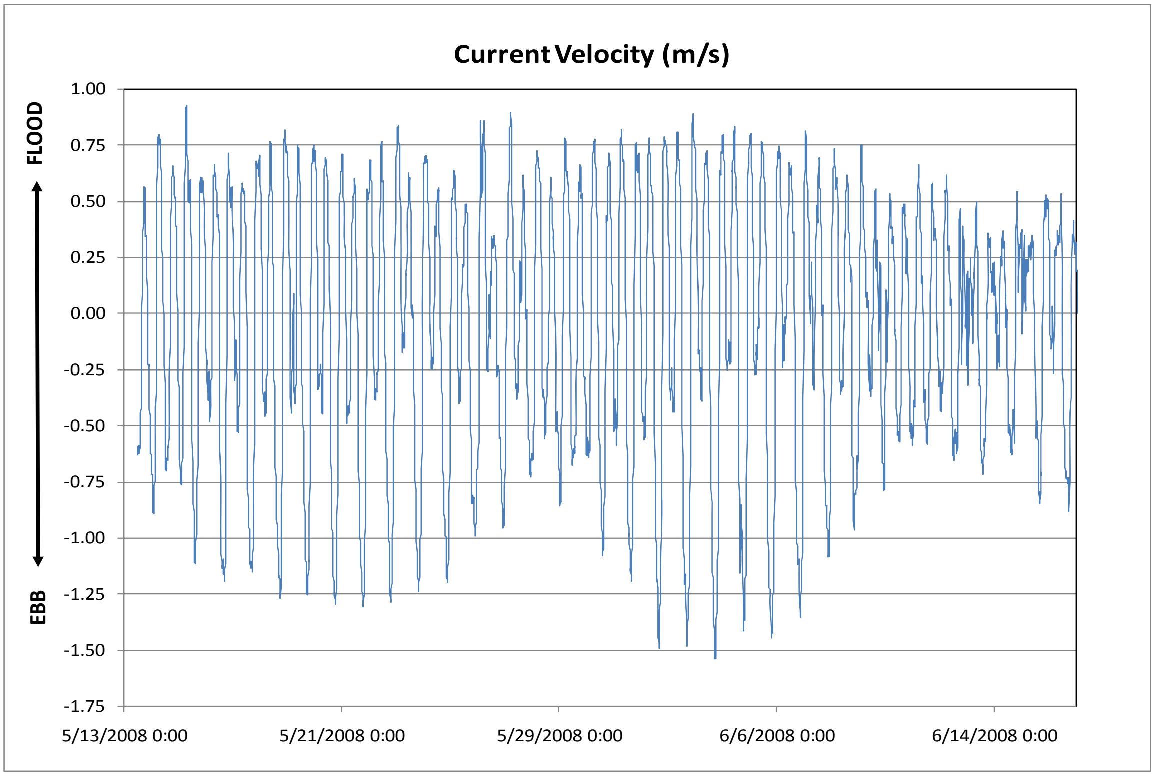


Figure 10. Measured tidal currents, depth averaged. Ebb velocities reach 1.5 m/s (4.9 m/s) during spring tide.

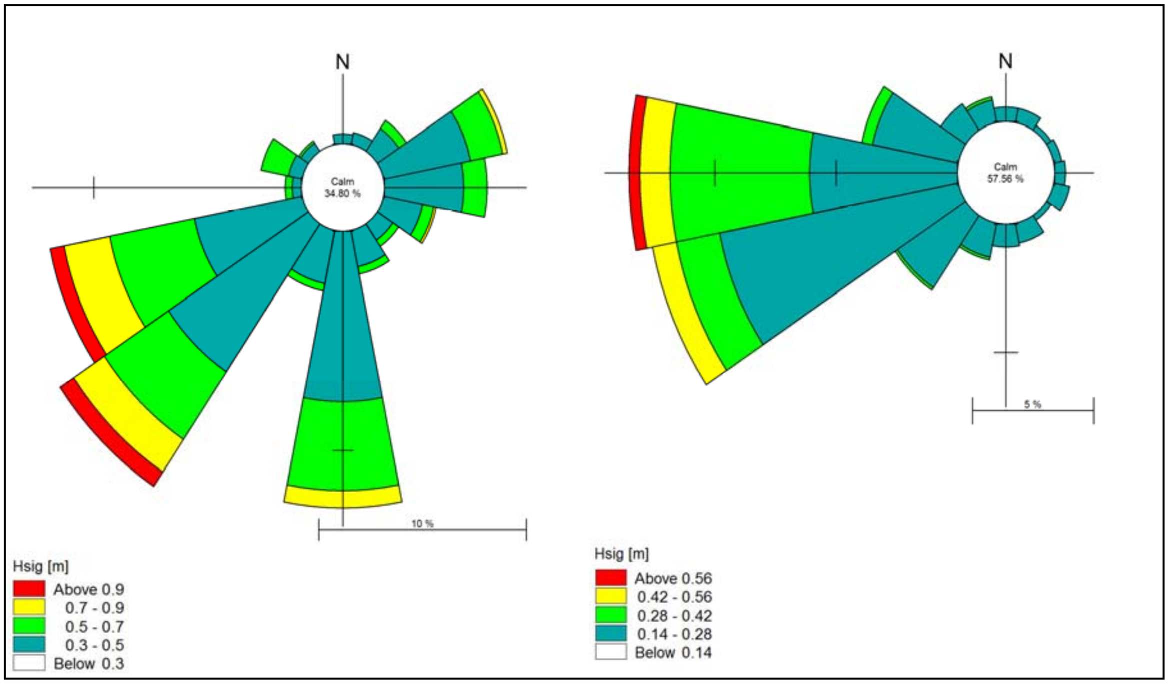


Figure 11. Wave rose for the significant wave height measured by the AWAC (left) and Aquadopp (right) from May 13, 2008 to June 19, 2008.

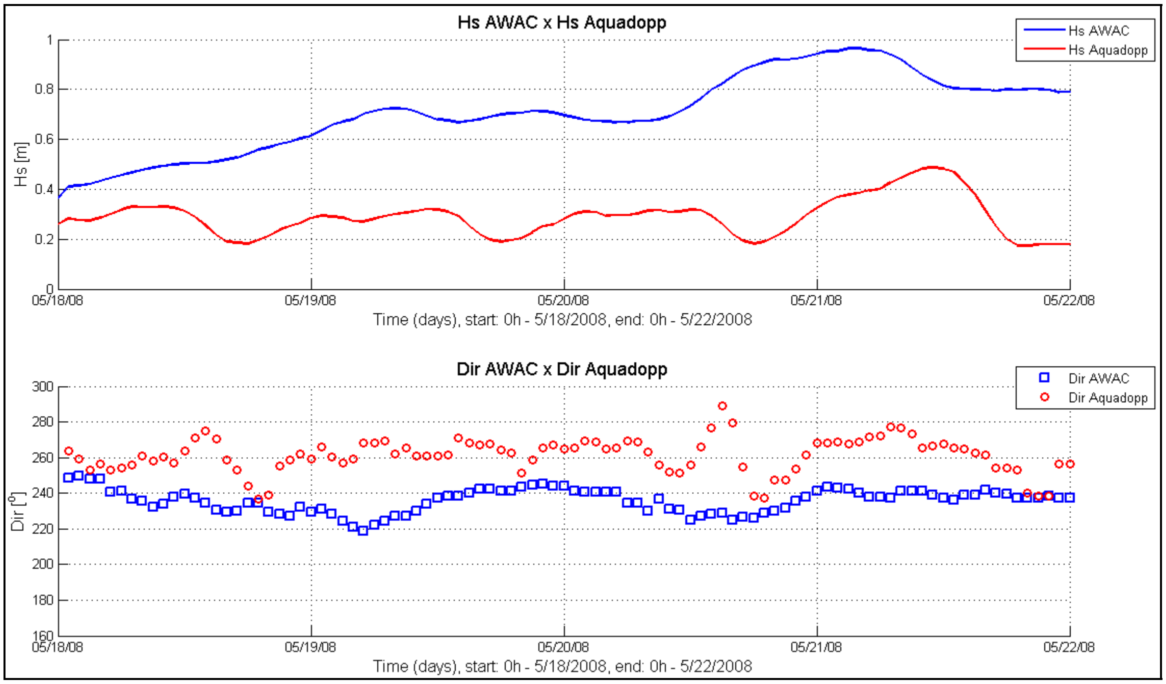


Figure 12. Wave height (above) and direction (below) measurements.

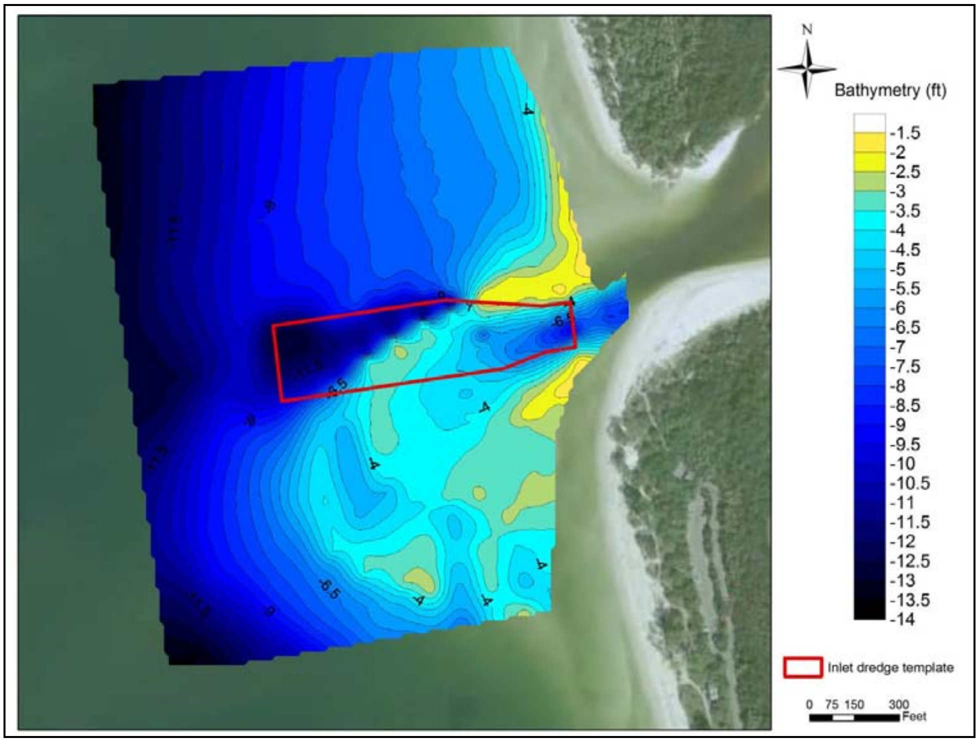


Figure 13. Two-dimensional contour plot of the bathymetric survey conducted by CPE (2008). The elevations are referenced to the datum NAVD88, in feet. Note the ridge in yellow cutting across the channel. This is the major impediment to navigation.

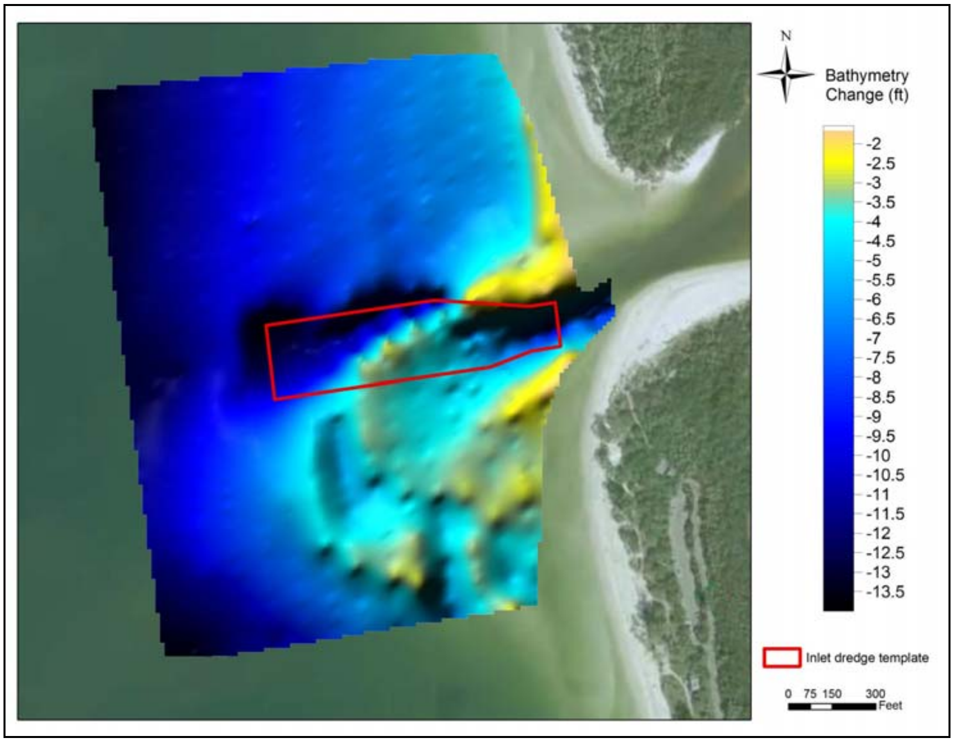


Figure 14. Three-dimensional surface plot of the bathymetric survey conducted by CPE (2008). The elevations are referenced to the datum NAVD88, in feet.

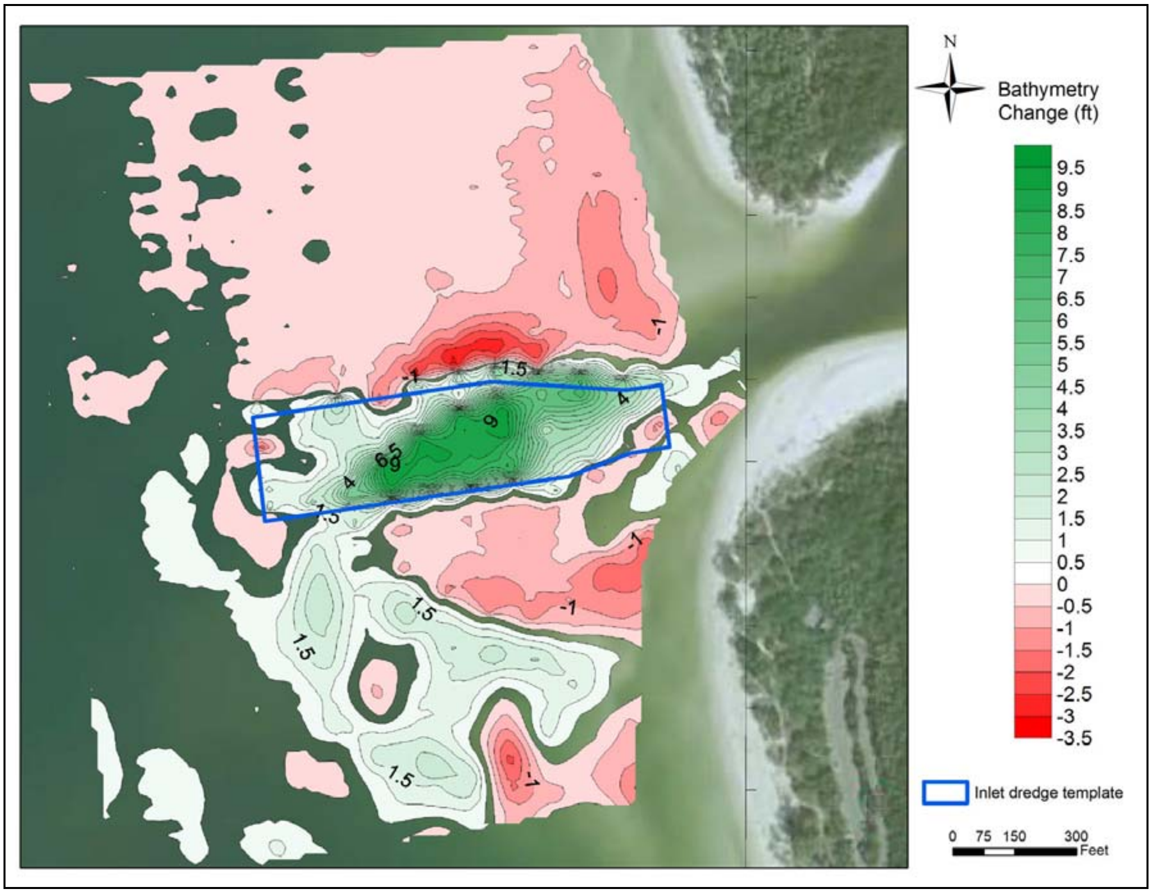


Figure 15. Bathymetry change (in ft) between 2007 (post dredge survey) and June 2008.

Other Measurements

During the ADCP deployment, some sediment samples of the channel were obtained to qualitatively characterize the channel area (Figure 16). In contrast to neighboring beach grain sizes, which range from 0.2 mm to 0.35 mm, the sediments in the channel area and inner ebb shoal range are coarser, with sizes of at least 0.8 mm. Historic vibracores (Appendix A1) show an ebb shoal with coarse shell and sand over a layer of fine sand.

Environmental surveys of the study area were conducted to characterize consolidated substrate within the study area and identify seagrass and fish communities. The results of these environmental surveys are presented in Appendix I.

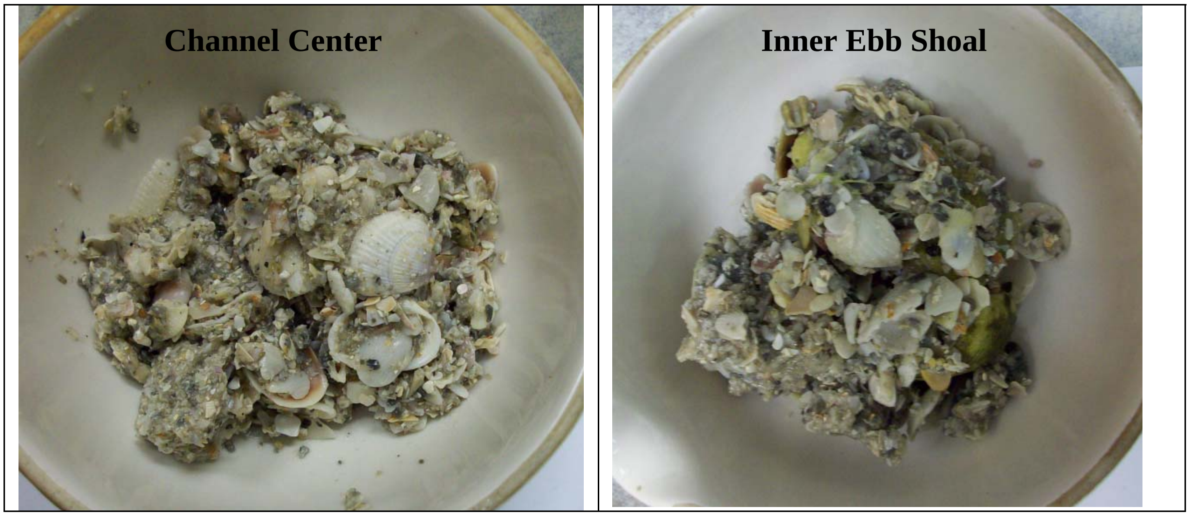


Figure 16. Sand samples obtained in the inlet center and inner ebb shoal.

NUMERICAL MODEL SETUP

This section described the technical set-up of the model necessary for review of the results by technical reviewers with a basic understanding of models. It also illustrates how the model parameters were adjusted (calibrated) to match measured conditions at the project area.

Sources of Input Data

Bathymetry data was obtained from a variety of sources. The deepwater bathymetry was obtained from the NOAA-Geodas database, which utilizes the U.S. Coastal relief model of Davies (1994). The nearshore data was obtained from the recent lidar surveys conducted by the U.S. Army Corps of Engineers, which can be obtained from http://maps.csc.noaa.gov/TCM/ (2004 data). The 2007 pre- and post-dredging survey data were obtained from local surveys conducted under contract to the County, and the 2008 survey data was obtained from a survey conducted by CPE on June 2008.

Computational Grids and Settings

Three different model grids were created and nested to develop the regional wave transformation process, detailed shallow water wave propagation processes, circulation patterns in the coastal and estuarine region, and the morphological changes in the channel and adjacent beaches of Wiggins Pass. Morphology is the change in land form (topography and bathymetry) predicted by the model.

The computational grids are presented Figure 17. The regional wave grid has 62 x 160 elements, with 8585 active grid cells. The offshore boundary was extended to be colocated at the WIS292 coordinates. This grid has the x-coordinate oriented perpendicularly to the shoreline of Wiggins Pass region Figure 17). The regional wave model was necessary in order to include the effects of Sanibel Islands and its offshore shoals on the magnitude of northern waves at Wiggins Pass, which is discussed in later sections of this report.

The local wave grid has 61x172 elements, with 7767 active grid cells, which can be seen in Figure 18. The hydrodynamic and morphologic grid has 166x264 elements and 36492 active grid cells and is represented in Figure 19. The model was run in 3D mode with five vertical layers. The space-varying curvilinear grids follow Deltares (former WL Delft) guidelines for smoothing and orthogonality (Appendix II).

Model parameter settings were defined during calibration as described in the following sections of this report. Model settings are also summarized in Appendix II.

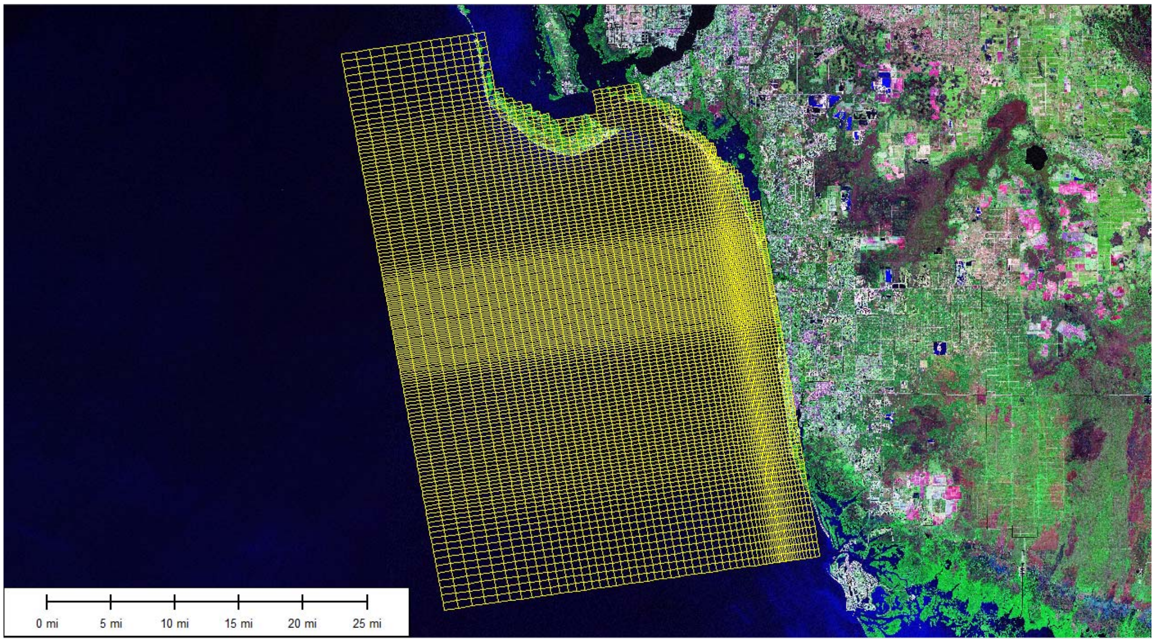


Figure 17. Regional wave grid.

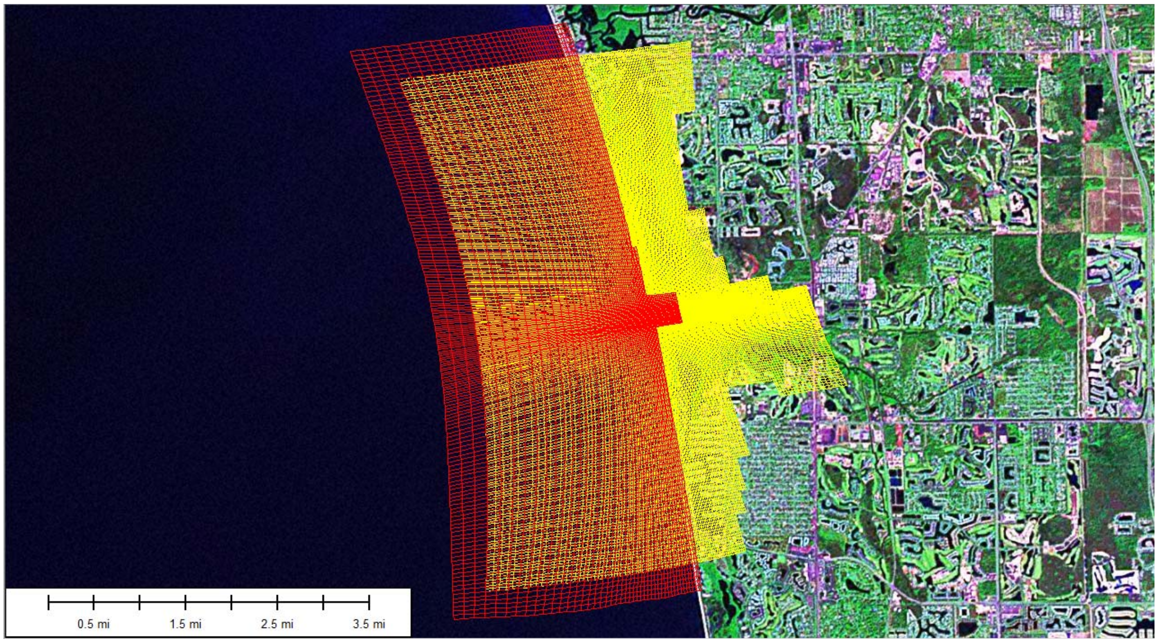
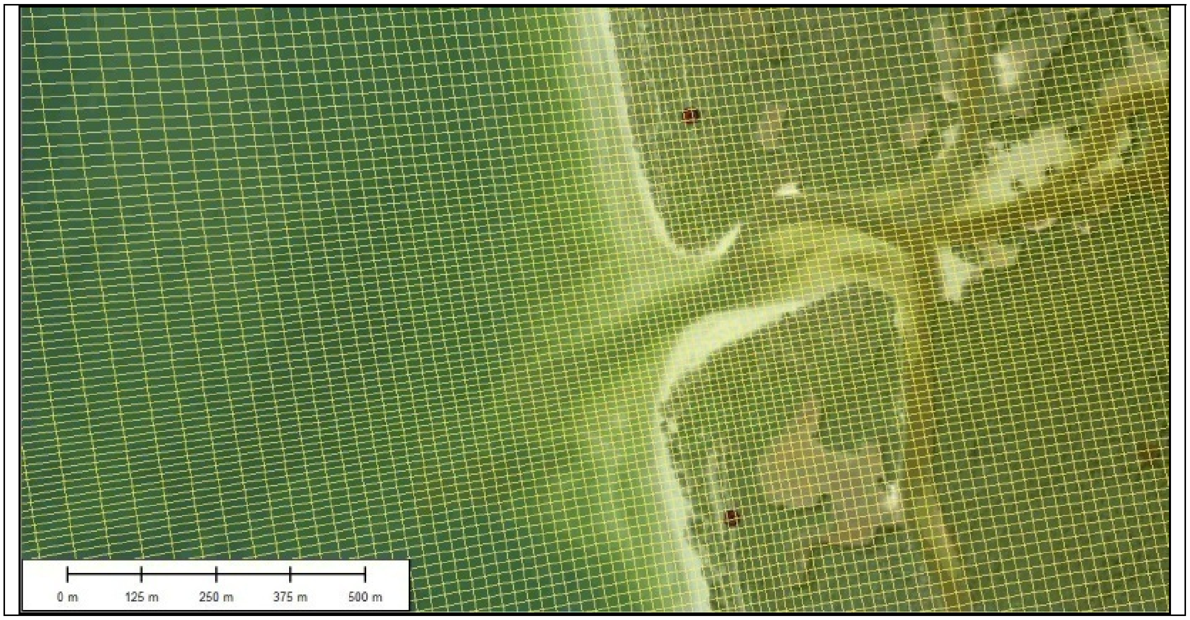


Figure 18. Local wave and hydrodynamic grid.



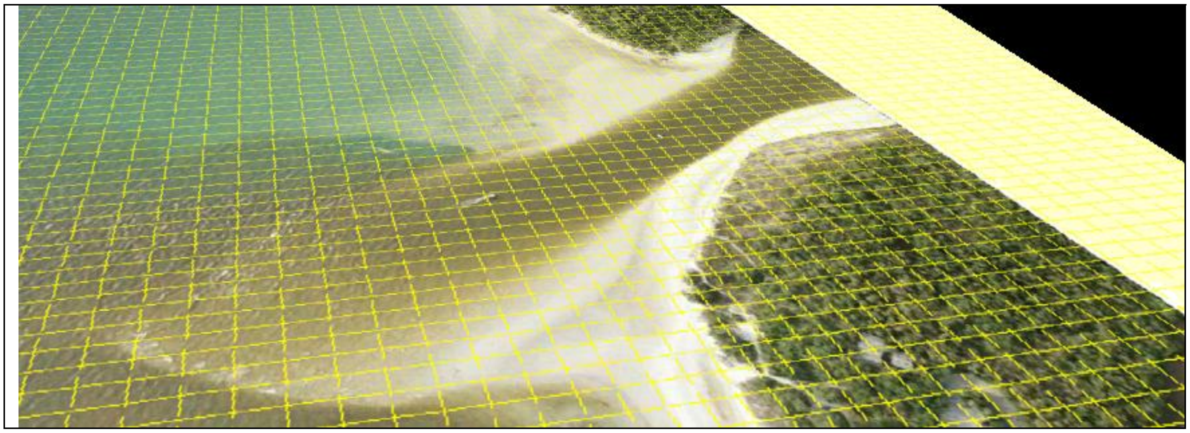
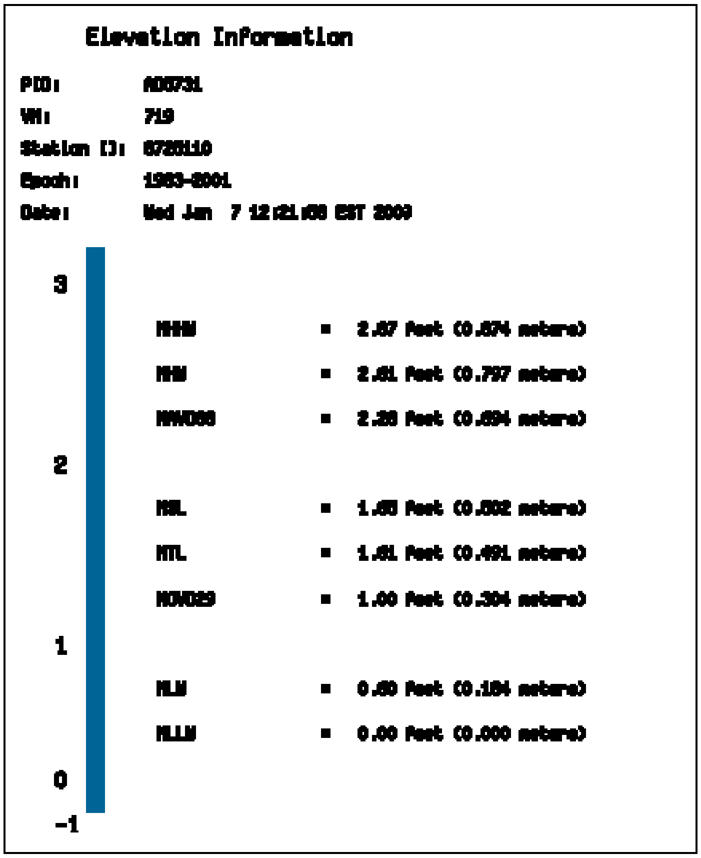


Figure 19. Detailed views of the hydrodynamic and morphology grid at Wiggins Pass.



Elevation Information

PID: AD0731
VM: 719
Station ID: 8725110
Epoch: 1983-2001
Date: Wed Jan 7 12:21:08 EST 2009

MHHW = 2.87 feet (0.874 meters)
MHW = 2.61 feet (0.797 meters)
NAVD88 = 2.28 feet (0.694 meters)
MSL = 1.65 feet (0.502 meters)
MTL = 1.61 feet (0.491 meters)
NGVD29 = 1.00 feet (0.304 meters)
MLW = 0.60 feet (0.184 meters)
MLLW = 0.00 feet (0.000 meters)

Figure 20. Tidal datums used in this study, Naples NOAA station. Numerical model simulations were conducted in MSL, which is about 0.19 m (0.63 ft) below NAVD88. Displayed tidal datums are referenced on 1983-2001 Epoch.

Wind and measured water level data were obtained from NOAA station 8725110, Naples, FL, located at the Naples Pier. Tidal datum information from the Naples Pier was used to convert the different data sources to the same vertical datum. The tidal datum elevations are shown in Figure 20. The numerical model was run in Mean Sea Level (MSL), and the channel design components were developed with reference to Mean Low Water (MLW) and converted to MSL to be implemented in the model. Elevations are presented within the report model units (meters-MSL) and in ft-NAVD.

Offshore wave information for wave climate characterization was obtained from the USACE/Wave Information Studies (WIS) for stations #292 and #297 from 1980 to 1999, located at 82.33° W / 26.33° N and -82.08° W / 26.17° N, respectively (Figure 21, http://www.frf.usace.army.mil/cgi-bin/wis/pac/pac_main.html). The second hindcast dataset was obtained from NOAA/WAVEWACH III reanalysis program from 1999 to 2007 for the station located at the following coordinates: 82° W / 26.25° N.

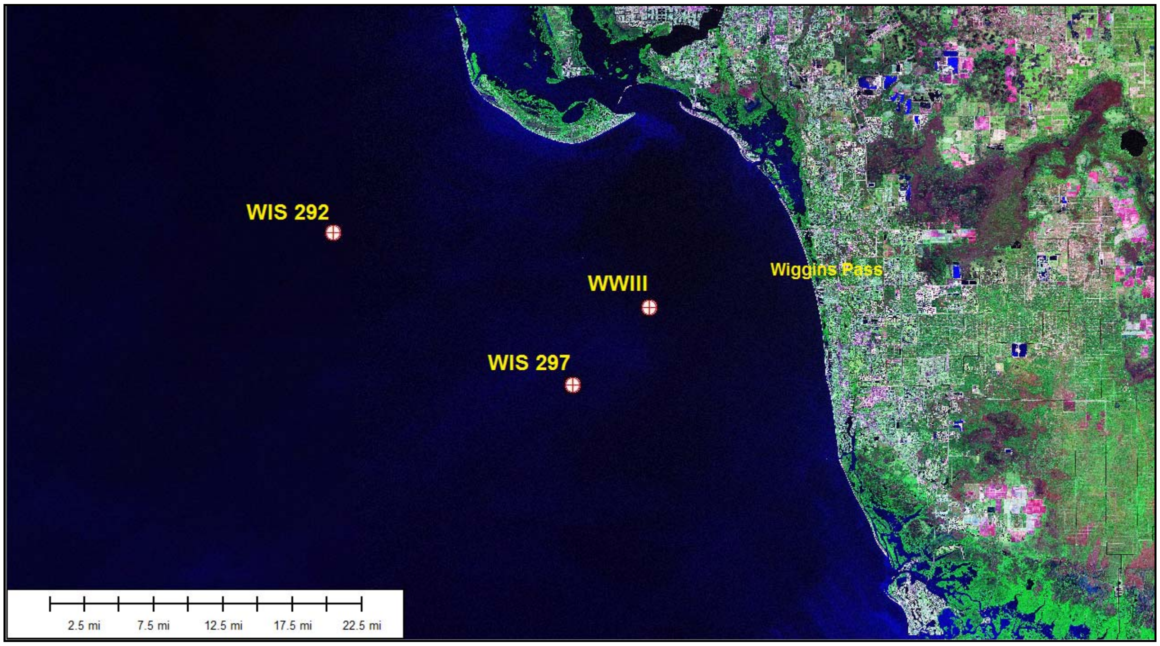


Figure 21. Location of wave hindcast stations for WIS #292 and #297 and NOAA WWIII.

Nearshore and inlet wave and current measurements utilized for calibration purposes were measured by ADCPs deployed by CPE as described in this document.

NUMERICAL MODEL CALIBRATION

Waves Calibration

As described in the measurements section, two different gauges were deployed simultaneously from 13 May 2008 to 19 June 2008 for measurement of wave parameters in intermediate (AWAC-ADCP, offshore instrument) and shallow waters (Aquadopp ADCP, nearshore instrument).

Wave model calibration simulations utilized the spectral model SWAN; the model used by the Delft3D waves module. Calibration simulations were conducted with the following boundary conditions: significant wave height, peak period, peak direction, directional spreading, spectral shape, and peak enhancement factor obtained from the offshore instrument (AWAC-ADCP). The model results were compared with measurements at the nearshore instrument (Aquadopp-ADCP). To obtain a reasonable correlation between the distant measurement stations, several simulations were conducted to enable a sensitivity analysis of wave model processes and parameters (Appendix III). The main objective of this sensitivity analysis was to obtain nearshore wave height and direction values from the model that are in general agreement with the observed ADCP values.

Sensitivity runs were conducted to test different values of bottom friction and effects of wave transformation processes such as wave-current interaction, depth-induced breaking, wind-growth, and white capping. The four day period between May 18 and May 22, 2008 (see Figure 12) was used for calibration simulations. A separate wave model grid, extending from the AWAC-ADCP to the AQUADOPP-ADCP was prepared for the wave model calibration (Figure 22).

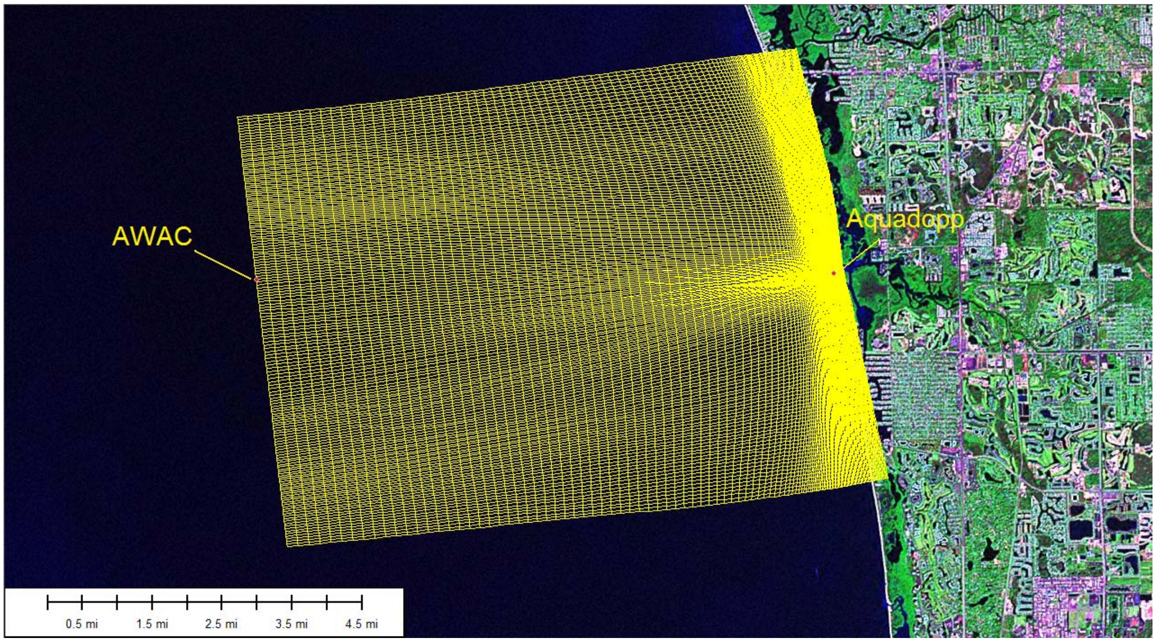


Figure 22. Location diagram showing wave model grid used in the calibration process, AWAC and Aquadopp ADCPs.

Results from two calibration simulations (initial and final) are shown in Figures 23 and 24. Additional results are shown in Appendix III. Improvement in the correlation between data and model were observed when the wind growth and wave-current interaction processes were included in the simulation. The Jonswap bottom friction coefficient was lowered from 0.067 to 0.0335, and the depth induced breaking alpha parameter was reduced from 1 to 0.5. Due to steep slopes and shallow water depths across the ebb shoal, the waves were breaking and dissipating all energy with an alpha parameter of 1. Modification of the alpha parameter to 0.5 allowed for some wave energy to penetrate into the inlet and resulted in better agreement between the measurements and model (Figure 24).

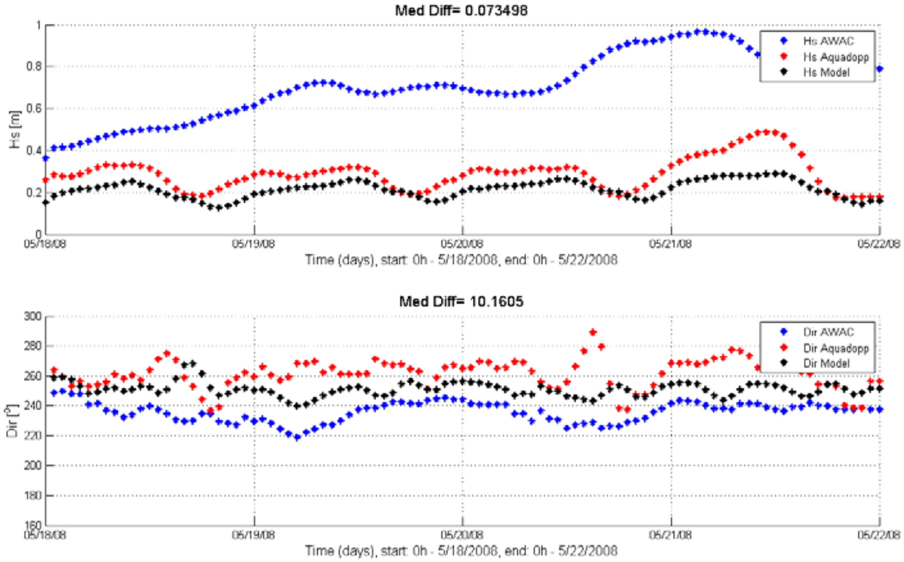


Figure 23 – Significant wave height (upper panel) and peak wave direction (lower panel) measured at AWAC, Aquadopp and simulated by the Delft3D model. Scenario with wave-current interaction and without wind-growth processes. Jonswap bottom friction coefficient = 0.067, depth-induced breaking alpha parameter = 1 and depth-induced breaking gamma parameter = 0.73. These are default model parameters.

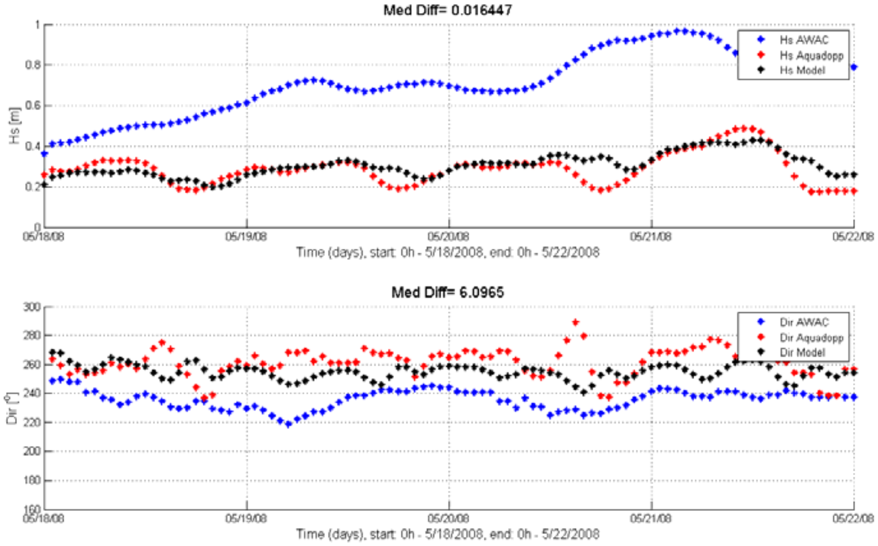


Figure 24 – Significant wave height (upper panel) and peak wave direction (lower panel) measured at AWAC, Aquadopp and simulated by the Delft3D model. Final calibration run with wave-current interaction and wind-growth processes activated. Jonswap bottom friction coefficient = 0.0335, depth-induced breaking alpha parameter = 0.5 and depth-induced breaking gamma parameter = 1.

Calibration of Inlet Flows

Inlet flows were simulated for the entire measurement period. The hydrodynamic calibration simulations consisted of a sensitivity analysis of bottom roughness, which was expressed by the Chezy value. The default Chezy value in Delft3D is 65 $m^{0.5}s^{-1}$ and, roughness in the Chezy equation can be defined as:

$$C_{2DH} = \frac{h^{1/6}}{n}$$

where h is the water depth and n is the Manning's coefficient. A lower Chezy value corresponds with a larger bottom roughness and thus a larger resistance, conversely a higher Chezy value corresponds with smaller roughness and less bed resistance.

The default value of 65 $m^{0.5}s^{-1}$ provided a reasonable agreement between measured and predicted water levels, but a relative difference in magnitude and phase between measured and predicted U and V current components was observed (Figure 25). Modification of Chezy coefficients were evaluated in a series of calibration runs in Appendix II. Reasonable results were observed for all values of Chezy coefficients tested. The smallest difference between measurements and model were obtained with a Chezy value of 30 $m^{0.5}s^{-1}$, but Chezy values of 40 $m^{0.5}s^{-1}$ (Figure 26) and 55 $m^{0.5}s^{-1}$ also produced acceptable results (see Appendix III). Appendix III and Figures 25 and 26 indicate that the best results between measured and modeled currents were achieved when bottom roughness was increased. This is probably due to the presence of very coarse sediments with shell fragments in the bed.

The good agreement between measured and predicted currents demonstrates the capabilities of Delft3D to simulate currents within the study area. With measured water levels on the open ocean boundary and model domains built to cover all the back-barrier water bodies, the model is able to replicate the magnitude and phase of observed tidal currents effectively.

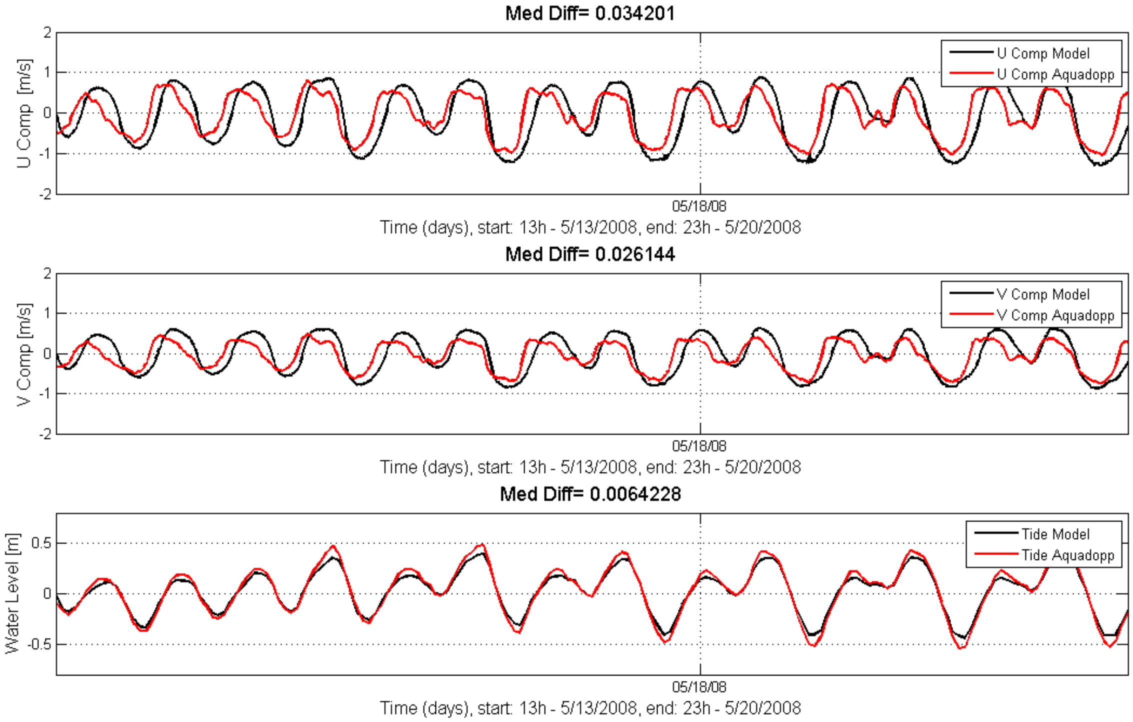


Figure 25 – Comparison between measured and simulated current U component, current V component and water level, Wiggins Pass, May 13 to May 23. Chezy = 65 $m^{0\,5}s^{-1}$ (model default value).

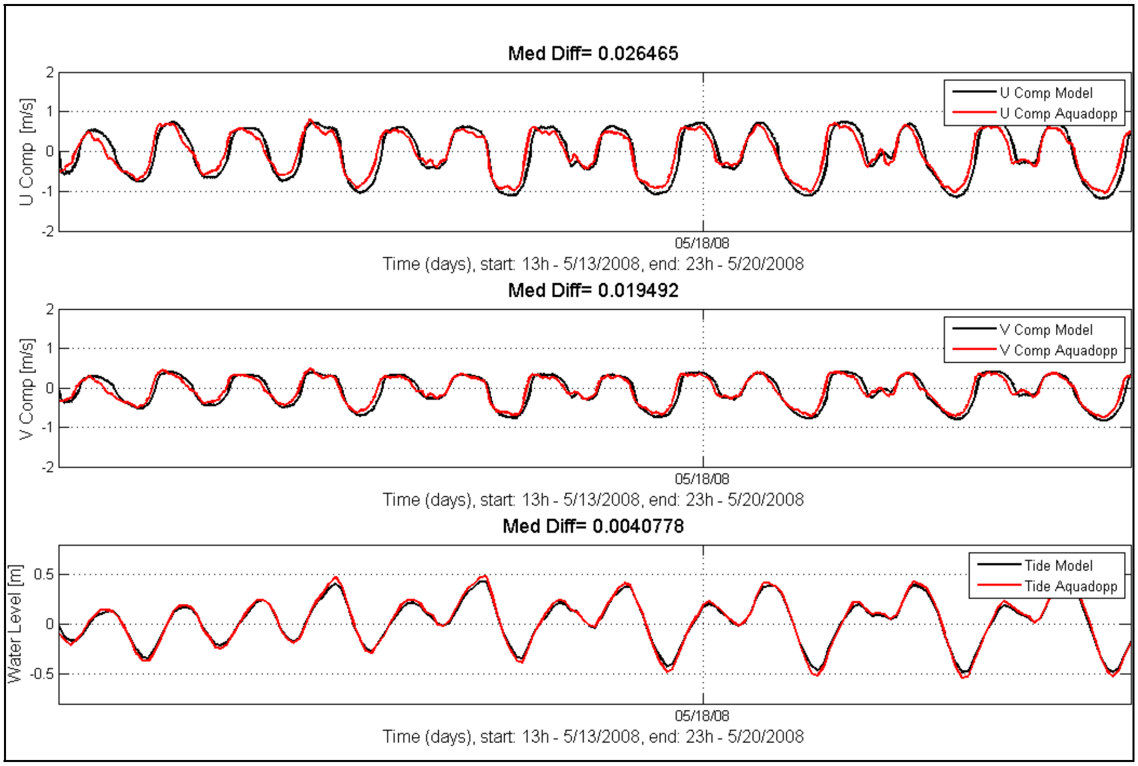


Figure 26 – Comparison between measured and simulated current U component, current V component and water level, Wiggins Pass, May 13 to May 23. Chezy = 40 $m^{0.5}s^{-1}$.

Calibration of Morphology Change

In order to simulate one year of morphological change, a series or schematization of the input boundaries conditions (waves and tides) were needed. Before describing the calibration of the simulated morphology, a brief description of wave and tide schematization is presented.

Wave Climate Schematization

To simulate one year of morphological change, an annual wave climate was developed using the longest wave record available (20 years of WIS data at station 292 and 10 years of WavewatchII data). To visualize the inter-annual variability of the waves in the study area, please refer to Appendix V.

The wave record was first filtered to eliminate all the waves that are directed offshore and that are smaller than 0.4 m (1.3 ft). The remaining wave records, which are significant for sediment transport, were divided into four direction classes and four height classes and 14 wave cases were developed. The wave cases that occurred less than 1 day per year were combined with its nearest class neighbor, and 12 wave cases resulted in the final wave climate. This wave climate was used during calibration simulations and production simulations, and the original 12 wave cases were slightly adjusted during calibration so an accurate representation of the measured morphology during the calibration period could be simulated. The wave cases used in the simulation are shown in Table 1. Significant wave height, peak period, and mean wave direction were obtained directly from the wave record. The smallest waves had heights in the range of 0.4 m (1.3 ft), the intermediate wave heights were in the range of 1.3 m (4.3 ft), and the highest waves fluctuated around 2.4 m (7.9 ft) (Table 1). Wave periods varied from 4 to 8 s, and wave direction varied from 194 degrees to 316 degrees (Table 1). Directional spreading was defined based on the peak wave period, similar to Hartog *et al.* (2008) and Benedet and List (2008). Waves with greater periods, swell, which are characteristically well sorted seas, used a narrow directional spreading value, and waves with smaller periods, commonly referred to as *"wind seas"* which are disorganized seas with waves from many directions used a wider directional spreading.

To decrease the time needed for the morphological computation, a morphological acceleration factor (morfac) was used, as described in Lesser et al (2004) and Benedet and List (2008). The morfac is a multiplication factor, which multiplies the sediment exchanges to and from the seabed, per flow timestep. A wave case that occurs 14 days a year, for example, can be simulated over two tide cycles (one day) with a morfac value of 14 to obtain a model simulation of 14 days. It is common practice between Delft3D users to use lower morfac values for high wave cases, when the most significant morphological changes occur, and to use higher morfac values for smaller wave cases, where little change takes place.

Table 1. Wave climate developed based on the long-term wave record.

Significant Wave height (hs)	Peak Wave Period (Tp)	Mean Wave Direction (degrees)	Directional spreading (power cos)	Percentage of Record (%)	Morfac Utilized
0.4	4.43	194	4	9.2%	14
1.34	5.31	198	4	1.8%	5.5
0.48	4.36	228	4	10.2%	15.5
1.28	5.92	224	4	0.6%	1.9
0.4	5.17	263	4	9.2%	28
1.48	7.64	270	15	1.1%	3.4
0.44	4.8	293	4	24.5%	24.8
1.46	6.87	292	15	6.3%	19.3
2.47	8.07	292	25	2.6%	7.97
0.43	4.47	320	4	27.6%	21
1.41	5.75	323	10	5.7%	17.4
2.39	7.18	316	15	1.0%	3.14

Tide Schematization

Tides at Wiggins Pass are mixed semi-diurnal which consists of 2 high and 2 low tides each day of differing height. Initial simulations used a mean tide, fluctuating from mean high water to mean low water, similar to the methodology described in by Hartog *et al.* (2008) and Benedet *et al.* (2007) and Benedet and List (2008). Simulations of morphology with the mean tide produced acceptable results, and this tide schematization was used throughout the study for long-term morphological simulations. Towards the end of this study the authors became familiarized with a novel method of schematization of mixed semi-diurnal tides, as described by Lesser (2008). This method combines diurnal and semi-diurnal astronomical constituents to calculate a representative morphological tide with two main components: the M2-Lunar and an artificial constituent named C1 that combines diurnal constituents. Tests were conducted using the Lesser (2008) approach and the mean tide approach and it was found that both methods were consistent in producing similar results in term of residual tide transport.

Calibration Simulations

The morphology calibration period extended between the 2007 post-dredge survey (February, 2007) and June 2008. The morphology calibration goal was to reproduce the trends observed in Figure 15 in terms of location of erosion and deposition patterns and to simulate the amount of channel shoaling observed during this time-period. A sediment budget was developed to assist this process. Thirty-two calibration simulations were conducted to achieve a reasonable morphological calibration. Parameters modified during calibration included:

- Sediment transport parameters contained in the Delft3D .mor file. These included ThetSD (dry cell erosion factor), SUS (suspended sediment transport factor), BED (bedload sediment transport factor), SUSW (wave-orbital induced suspended transport), BEDW (wave-orbital induced bedload transport).
- Flow viscosity and diffusivity.
- Sediment composition. Simulations started with a uniform grain size of 0.35 mm. After field investigations, it was observed that coarser sediments occurred within the navigation channel and inner shoal. This area was mapped with a coarser sediment fraction during calibration (1 mm), which leaded to improved results in terms of channel mobility and beach erosion.
- Number of vertical computational layers in the 3D flow model.

The final settings used in the model are shown in Appendix II. The final calibration simulation is shown in Figures 27 and 28.

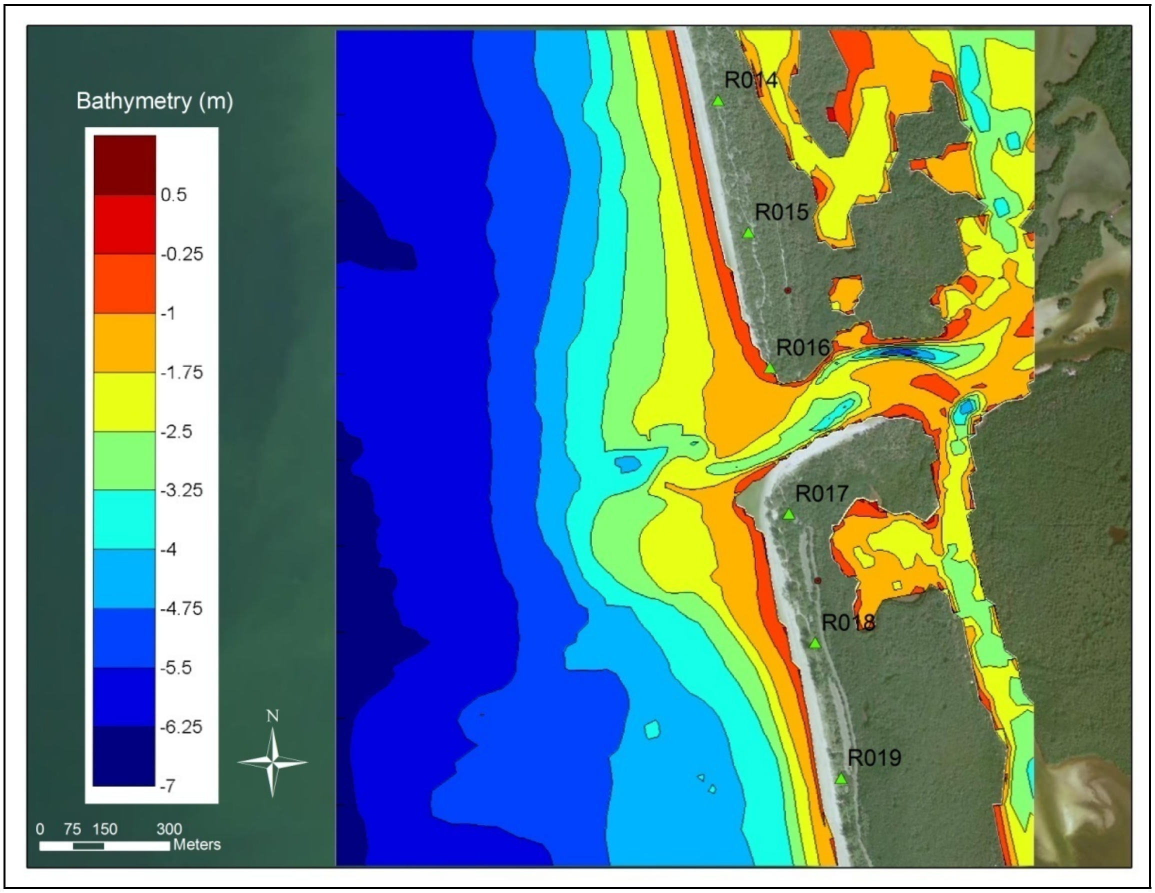


Figure 27. Simulated morphology, calibration run #32.

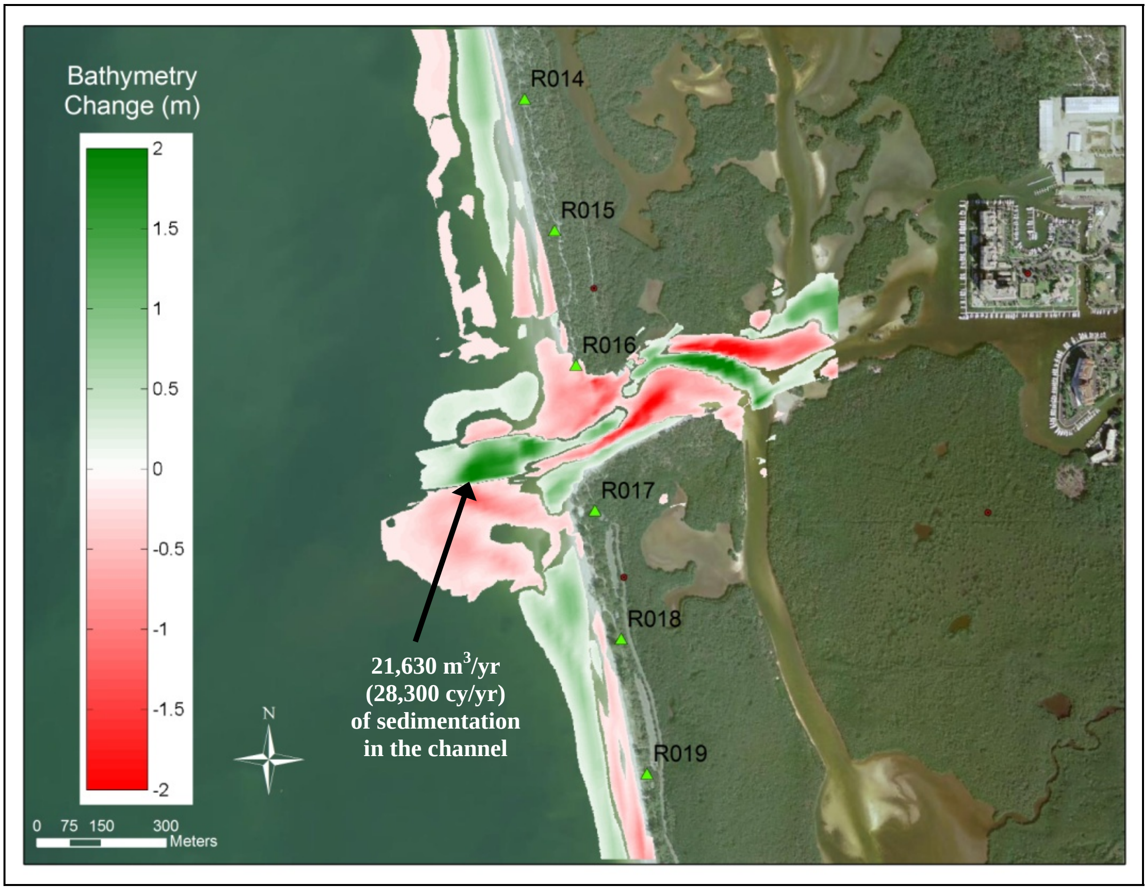


Figure 28. Simulated morphology change, calibration run #32

The final calibration simulation achieved the morphology calibration goals and reproduced the main features observed at Wiggins Pass 1.5 years after dredging, which include:

- An ebb shoal ridge interrupting the dredged channel template.
- The simulated shoaling rate was 21,630 m^3/yr (28,300 cy/yr), against 20,180 m^3/yr 26,400 cy/yr) measured shoaling rate. The predicted shoaling rate is within 10% of the measured shoaling rate.
- There is overall erosion in the ebb shoal around the navigation channel and erosion along Barefoot Island.

The calibrated morphological model with waves and currents was then used to simulate various scenarios of inlet channel configuration and beach placement location, as described in the following pages.

PRODUCTION RUNS AND SELECTION OF ALTERNATIVES

The numerical modeling efforts described in this report were reviewed by a committee during the development of the study. After an initial organization meeting, three meetings with the reviewing committee were conducted during the study development (see Appendix IV). During these meetings, the steps of the modeling effort to be conducted in the following two months after each meeting were defined. In this section of the report, the results presented in each meeting are summarized, followed by the results of the selected alternative. The final selected alternative was chosen by the modeling review committee based on results presented by CPE.

Meeting #1 – June 2008

Regional Wave Transformation Processes

Meeting #1 was conducted on June 5, 2008. Preliminary numerical modeling results prior to model calibration were presented at this meeting. During the numerical model setup, the numerical modelers noticed the significant land and submarine morphology features that extend far offshore and modify incoming waves from the north and northwest quadrants (Figure 29). Due to the presence of Sanibel Island and the offshore sand shoals (Figure 29), Wiggins Pass is mostly sheltered from northern waves with angles greater than 300^o, and all the waves between 280^o and 300^o are transformed over the shallow offshore bathymetric features before they reach Wiggins Pass.

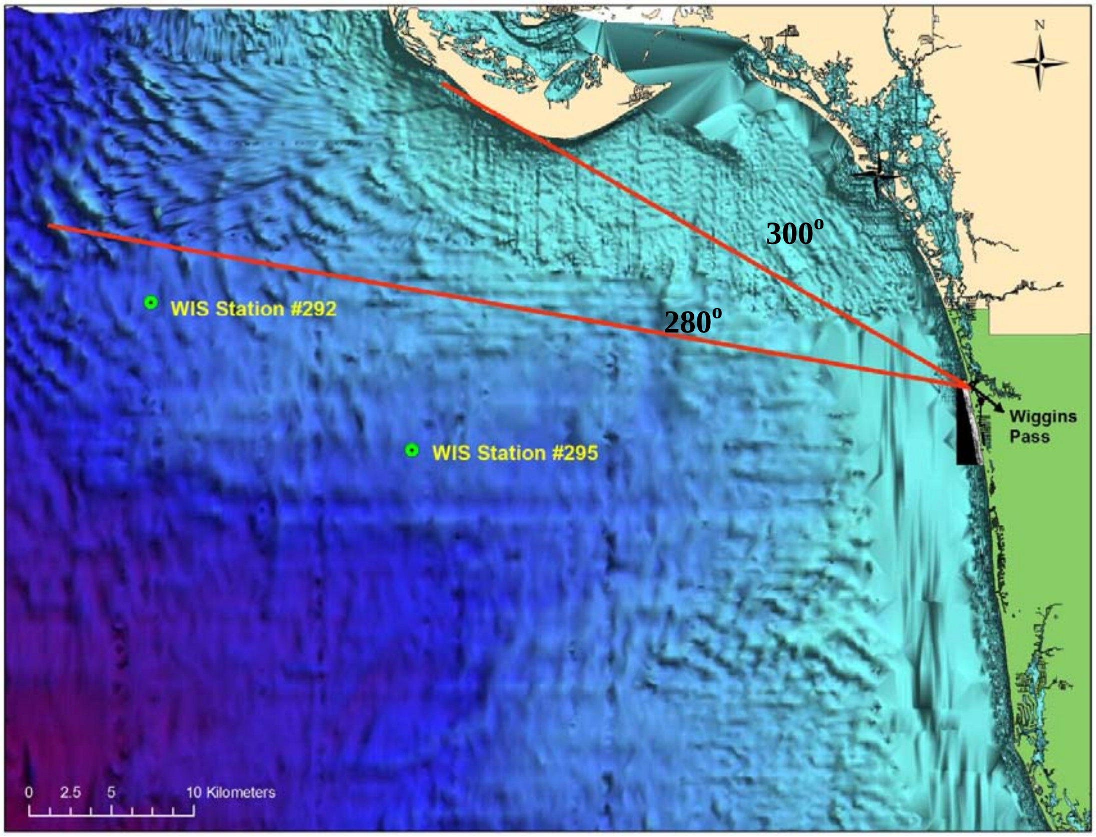


Figure 29. Regional bathymetry showing details of marine geomorphology offshore of Wiggins Pass in relation to the location of WIS wave hindcast stations.

Hindcast wave models used to develop the WIS data series, or WavewatchII data did not account for wave transformation processes over shallow water. These hindcast wave models calculate wave statistics in deep water, and in order to obtain an accurate representation of shallow water waves, a nearshore wave model, such as SWAN, needed to be utilized.

The WIS hindcast stations offshore of Wiggins Pass are located landward of important shelf features, that cause wave attenuation by bottom friction and transformation by refraction, diffraction and shoaling of incoming waves from northern quadrants. These effects, however, are not accounted for in the WIS or WavewatchII data. Therefore, in order to obtain an accurate representation of nearshore waves at Wiggins Pass one needs to transform the hindcast waves over the shallow bathymetric features that lie offshore. This required a large computational domain for the wave model, as shown in Figure 17.

Previous studies at Wiggins Pass (USACE, 1980, CPE, 1995, H&M, 2007) did not account for wave transformation over the shallow shoals offshore of Sanibel Island or the effect of Sanibel Island itself on the northern waves because smaller computational model domains were used. This practice, however, leads to overestimation of northern waves at

Wiggins Pass and may cause an overestimation of north-south transport at the pass and its adjacent shorelines. USACE (1980), for example, clearly states that the calculated littoral drift (alongshore transport) rates in their report "does not take into account the hydraulic effects of Wiggins Pass Inlet nor does it adequately represent modification of alongshore energy flux associated with wave refraction over the inlet and offshore shoals".

Figures 30, 31, 32 and 33 show wave model results obtained utilizing the large regional wave grid. Figure 30 shows that a high angle northwestern wave (300^o) with offshore height of 2 m (6.6 ft) reaches Wiggins Pass with less than 1 ft (0.33 m). If the wave direction is west-northwest (285^o, Figure 31) the same 2 m (6.6. ft) wave reaches Wiggins with about 0.75 m (2.5 ft). Both waves from northern quadrants refract significantly over the shallow offshore bathymetric features and reach Wiggins at almost shore-normal angles (Figures 30 and 31).

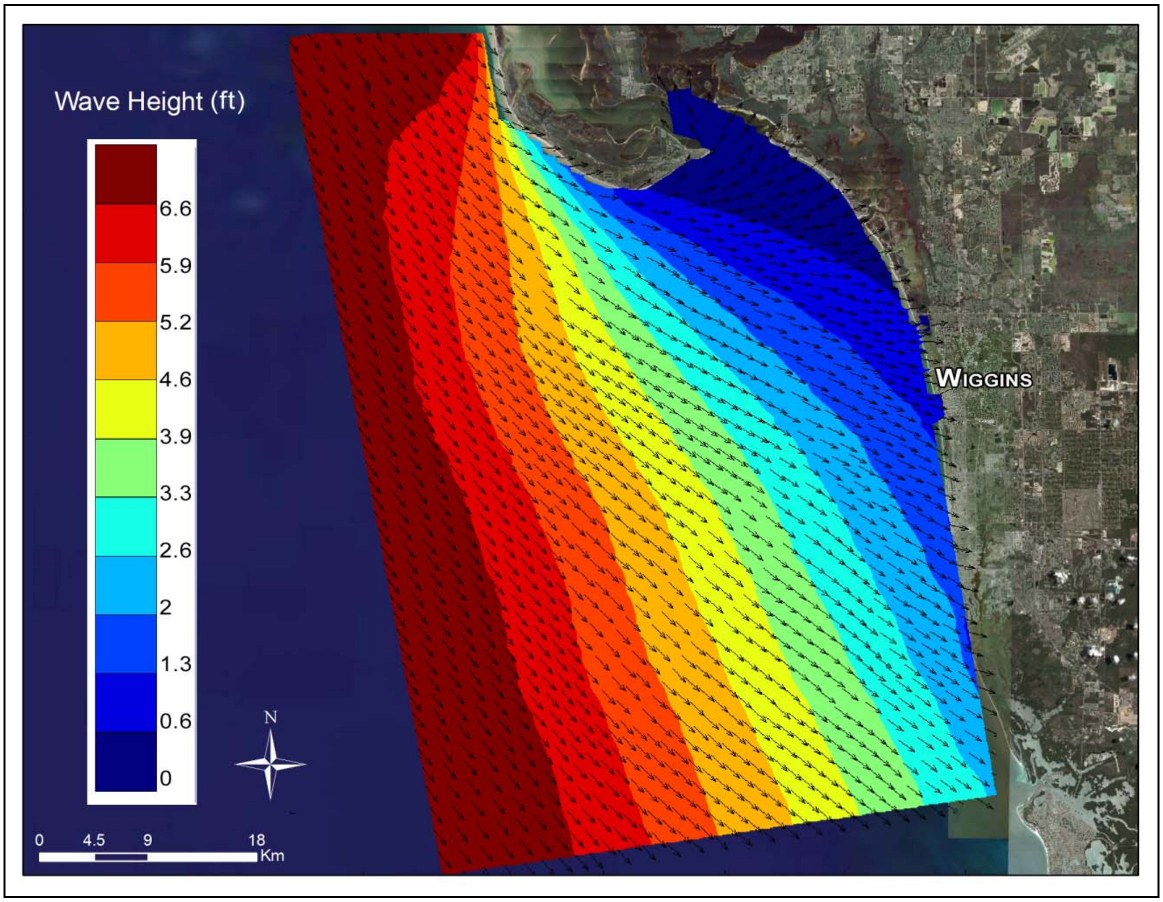


Figure 30. Results from a simulation of northwestern waves (6.6 ft Hs, 6 s TP, 300^o, 15^o directional spreading, Jonswap spectra).

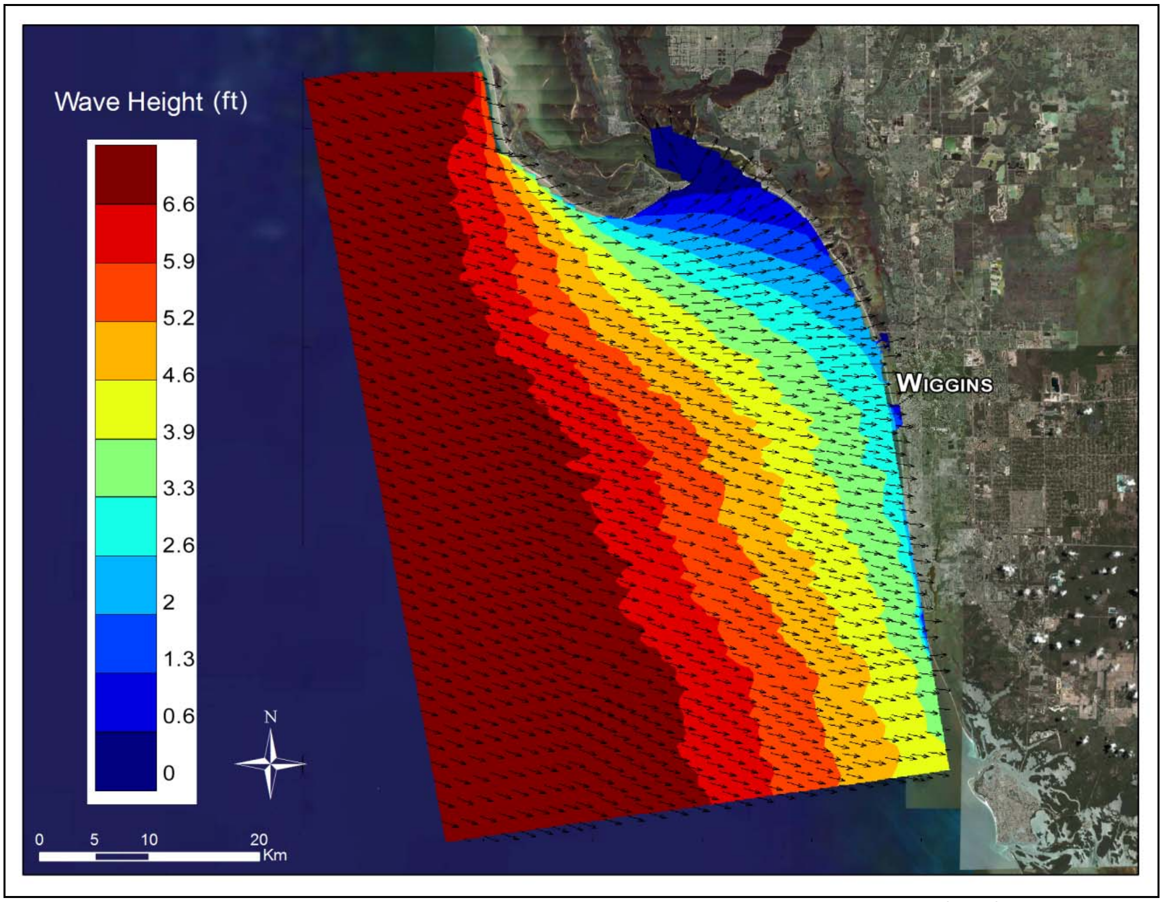


Figure 31. Results from a simulation of west-northwestern waves (6.6 ft Hs, 6 s TP, 285°, 15° directional spreading, Jonswap spectra).

In contrast, waves from southern quadrants reach Wiggins with more energy (in relation to northern waves) and at high angles. A 2 m (6.6 ft) wave offshore, coming from the west-southwest (255°, Figure 32) reaches Wiggins Pass with a wave height of more than 1 m (3.3 ft) (Figure 32), and a 2 m (6.6 ft) wave offshore, coming from the southwest (235°, Figure 33) reaches Wiggins with a wave height about 1 m (3.3 ft) (Figure 33). These results show the northern waves are significantly refracted and dissipated before they reach Wiggins Pass due to shallow offshore bathymetric features, while southern waves reach Wiggins with more energy and at high angles. This behavior has implications for sediment transport at the Pass, as discussed in other sections of this report. To capture this large-scale effect, large model computational grids, which are costly due to long and intensive computer iterations, are required. This is the main reason why this important process may have been overlooked by previous authors who worked with Wiggins Pass.

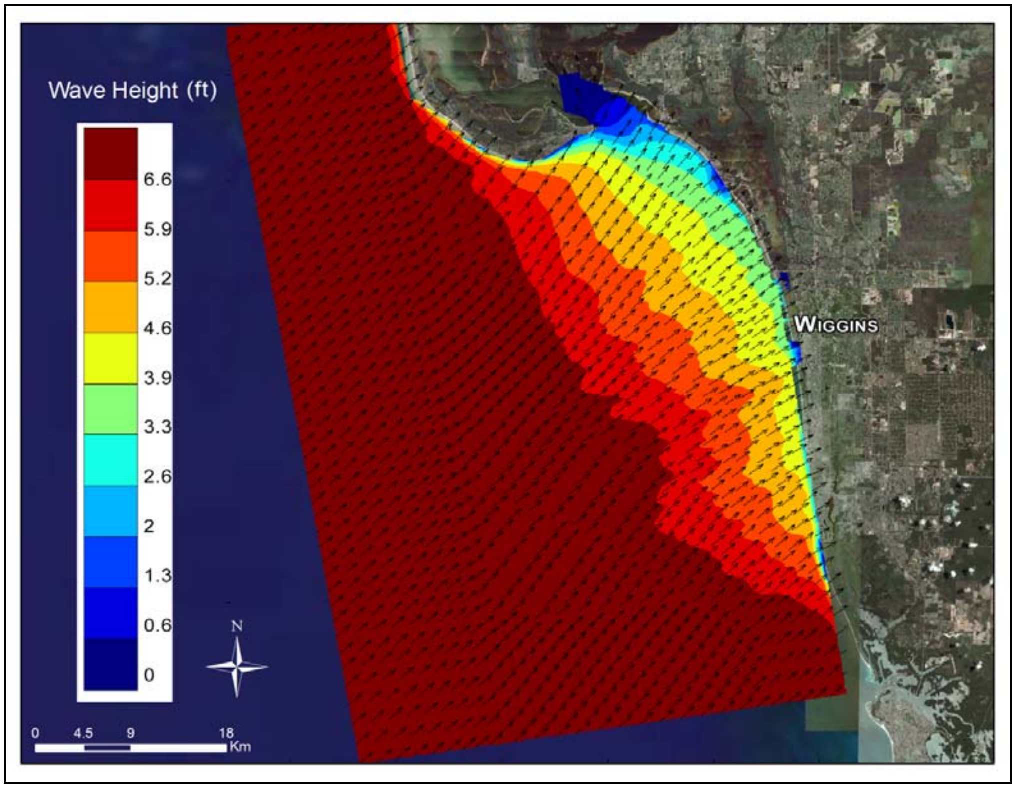


Figure 32. Results from a simulation of west-southwestern waves (6.6 ft Hs, 6 s Tp, 255°, 15° directional spreading, Jonswap spectra).

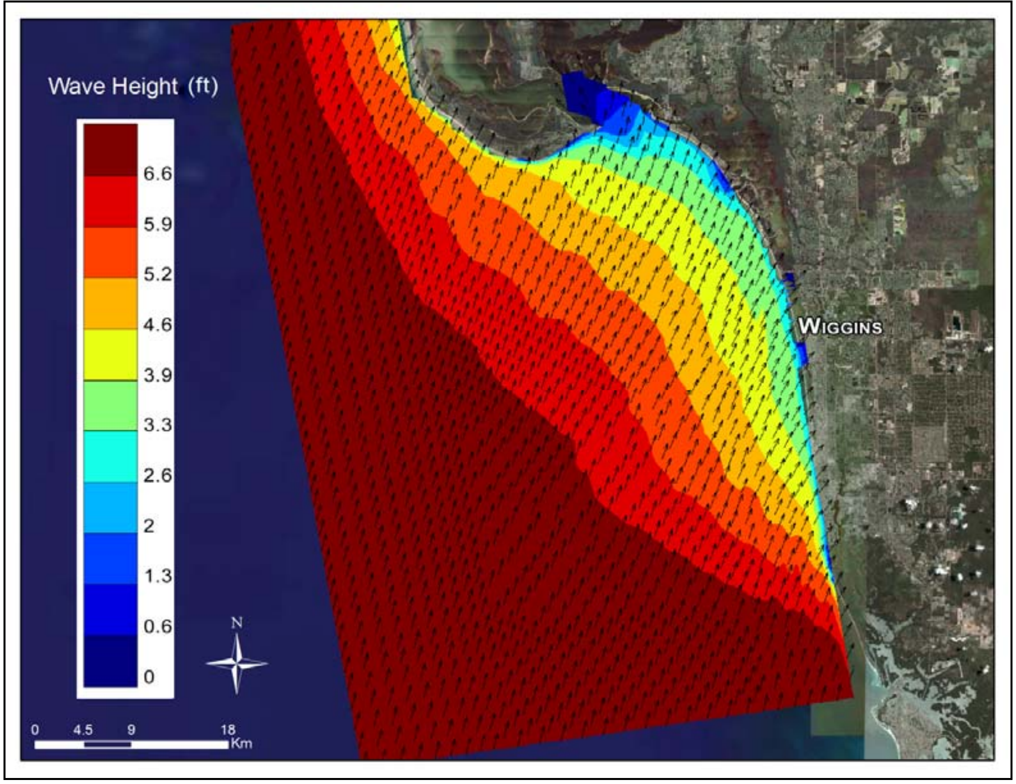


Figure 33. Results from a simulation of southwestern waves (6.6 ft Hs, 6 s Tp, 235°, 15° directional spreading, Jonswap spectra).

Net Sediment Transport Direction

In order to simulate the annual wave climate (Table 1) over the regional and local wave, flow and morphology grids (Figures 17 and 18) were used to calculate net sediment transport at Wiggins Pass (Figures 34 and 35). These simulations are mostly qualitative and were used to evaluate sediment transport patterns, not the absolute magnitudes. Results show that the net sediment transport direction north of Wiggins Pass is from south to north. There is a transport reversal between monument R16 to the end of Barefoot Beach caused by the ebb shoal. South of Wiggins Pass, the net transport direction is poorly defined and is a function of wave transformation over the shallow shoals. Net transport direction becomes north to south only around profile monument R22, about 1.6 km (1 mile) south of the pass.

Figures 34 and 35 provide some theoretical explanations for the erosion problems at Barefoot Beach. Net sediment transport direction north of the pass is from south to north. Therefore, the sediment is leaving Barefoot Beach and being feed to northern beaches. There is a small sediment transport reversal between R16 and the inlet, this causes sediment loss from the south end of Barefoot Beach to the inlet. There is little to no sediment bypassing to Barefoot Beach, due to the presence of a dredged navigation channel. This shows that Barefoot Beach is losing sediments from both sides and not receiving significant sediment input. To complicate matters, the sediment currently being dredged and placed north of Wiggins Pass is disposed around profile monument R12, where sediment transport direction is south to north. This sediment never makes its way back to Barefoot Beach.

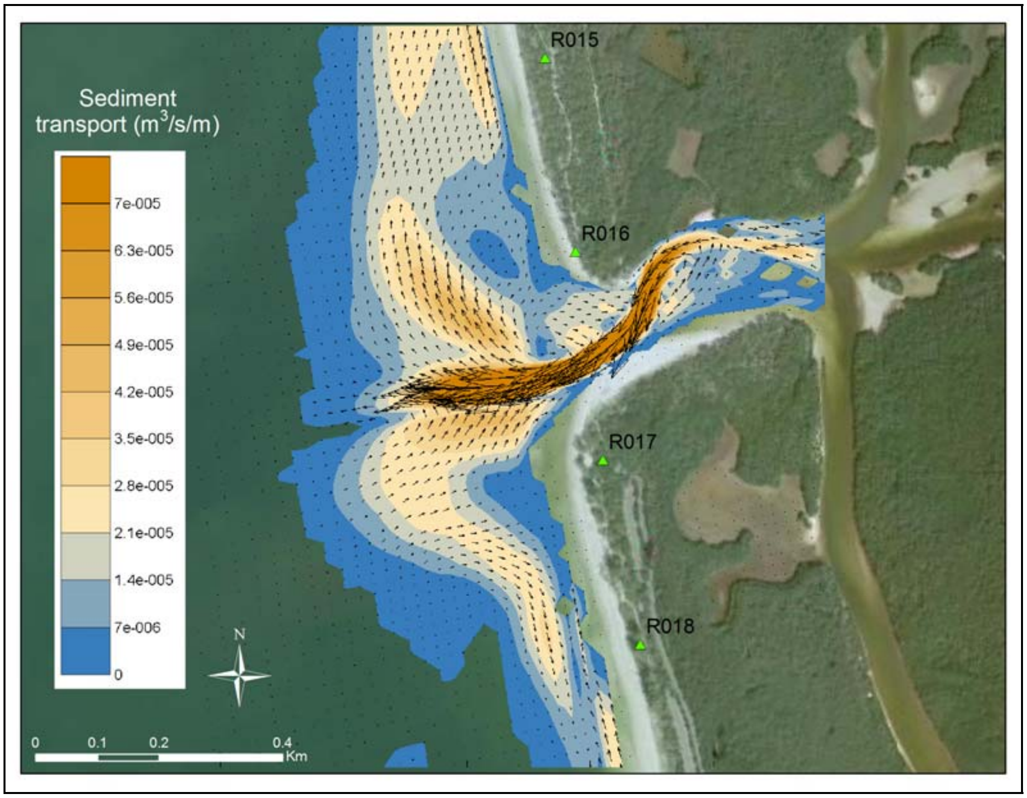


Figure 34. Net annual sediment transport direction, Wiggins Pass, FL.

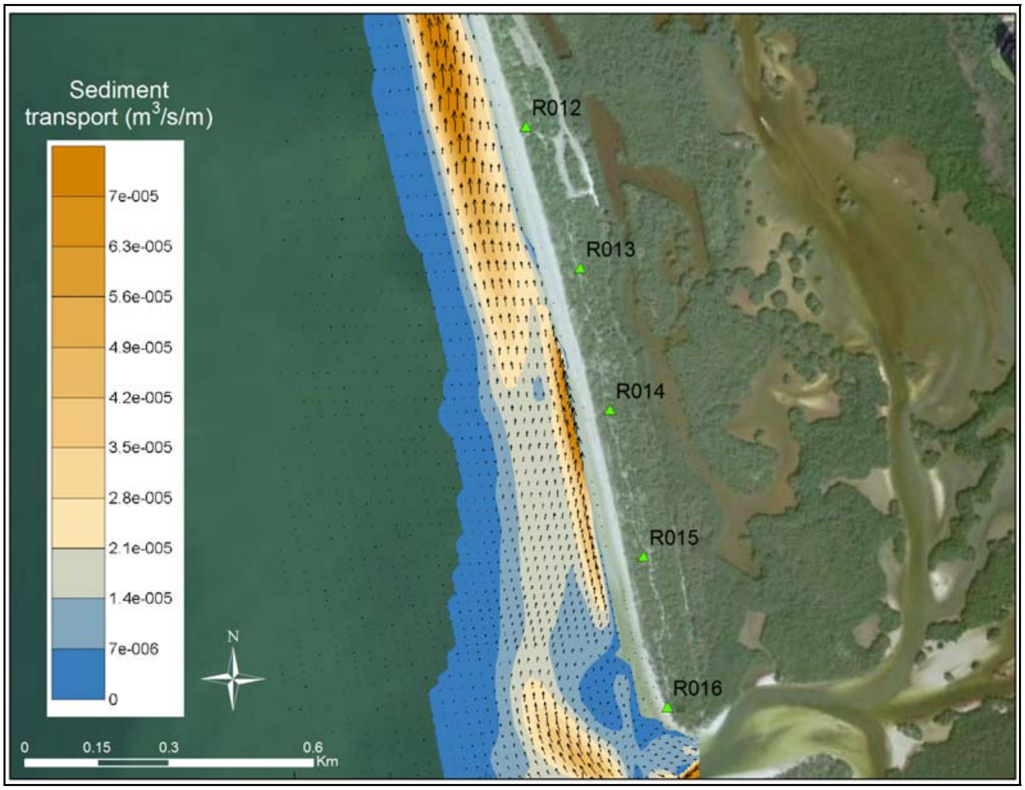


Figure 35. Net annual sediment transport direction, Wiggins Pass, FL.

Gross sediment transport patterns are also affected by the complex offshore geomorphology. During high angle southwestern waves, there is a large sediment transport potential from the south to the north (Figure 36), but most of the sediment that is transported south to north at Delnor Wiggins State Park is trapped by the navigation channel. The small amount of sediment that is not trapped by the navigation channel and bypassed to the north reaches Barefoot Beach between monuments R15 and R14 and is transported further to the north (Figure 36). In contrast, during west-northwest waves (Figure 37) the transport is north to south, south of Wiggins Pass, north to south between R16 and the end of Barefoot Beach but there is little alongshore sediment transport potential from Barefoot Beach to the north.

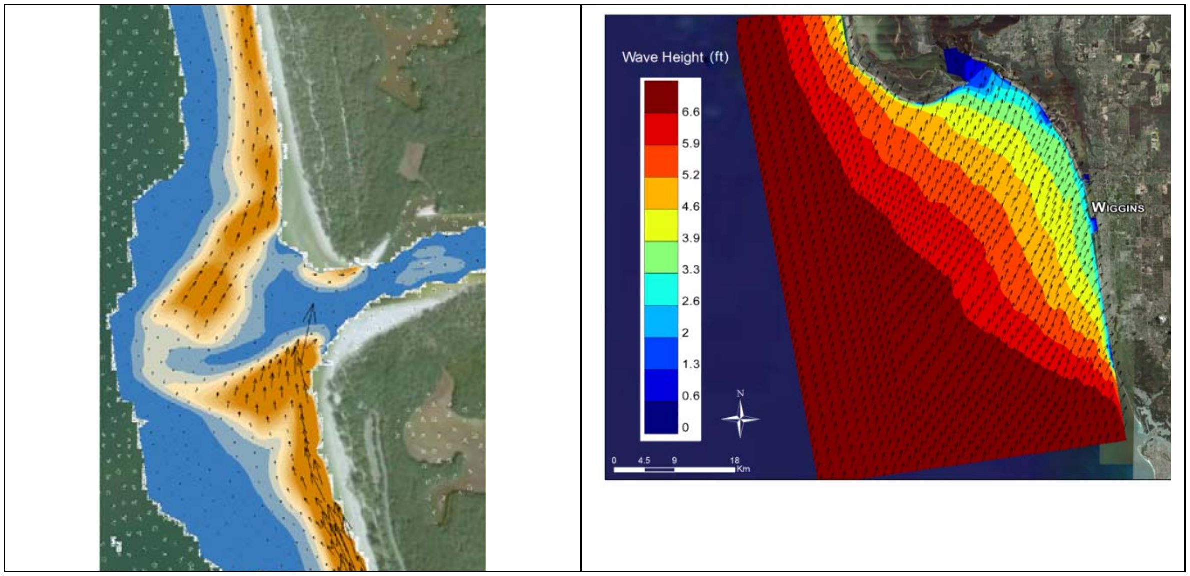


Figure 36. Gross sediment transport (left plot) during high angle southwestern waves (right plot).

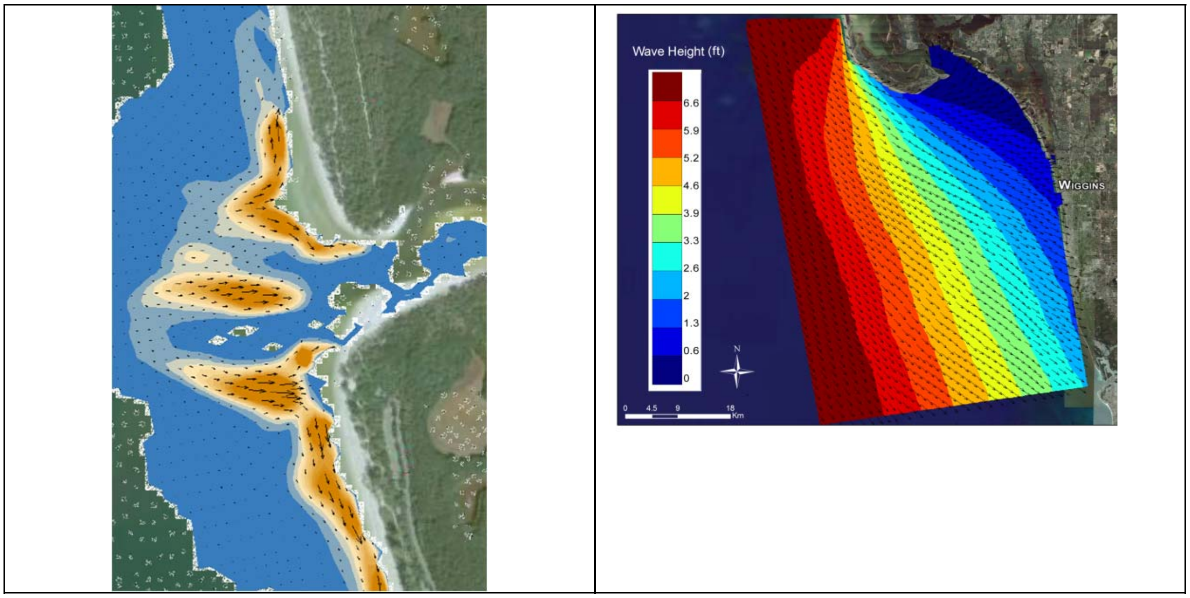


Figure 37. Gross sediment transport (left plot) during high angle northwestern waves (right plot).

Morphology Simulations

One year morphology simulations were conducted for the June meeting #1, to evaluate relative morphological patterns that result from distinct historical bathymetries before the finalization of calibration of the morphology module of Delft3D. The bathymetry measurements utilized for these simulations are shown in Figure 38. The simulated bathymetries after one year are shown in Figure 39.

The one year simulation using the 1970s bathymetry (Figure 39) resulted in a small realignment of the channel to the southwest. The ebb shoal lobe almost crosses the channel on its outer section after one year. The ebb shoal lobe feature that is crossing the channel (green shades in Figure 39) has a depth between 1 m MSL (3.8 ft NAVD) and 1.8 m MSL (6.5 ft NAVD), however, after one year, there is still a small section of the channel connecting to the open Gulf with depths greater than 1.8 m MSL (6.5 ft NAVD).

The one year simulation using the 2004 Lidar bathymetry (Figure 39) resulted in a very shallow and poorly defined channel after 1 year. The green shade that marks the zone between 1 m MSL (3.8 ft NAVD) and 1.8 m MSL (6 ft NAVD) is crossing the channel over a wide zone.

The one year simulation using the 2007 pre-dredge bathymetry resulted in the worst channel configuration for navigation. The green shade zone crosses the channel over a wide section and the shallower waters between 0.15 m MSL (1.12 ft NAVD) and 1 m MSL (3.8 ft NAVD) are starting to obstruct the navigation channel (Figure 39).

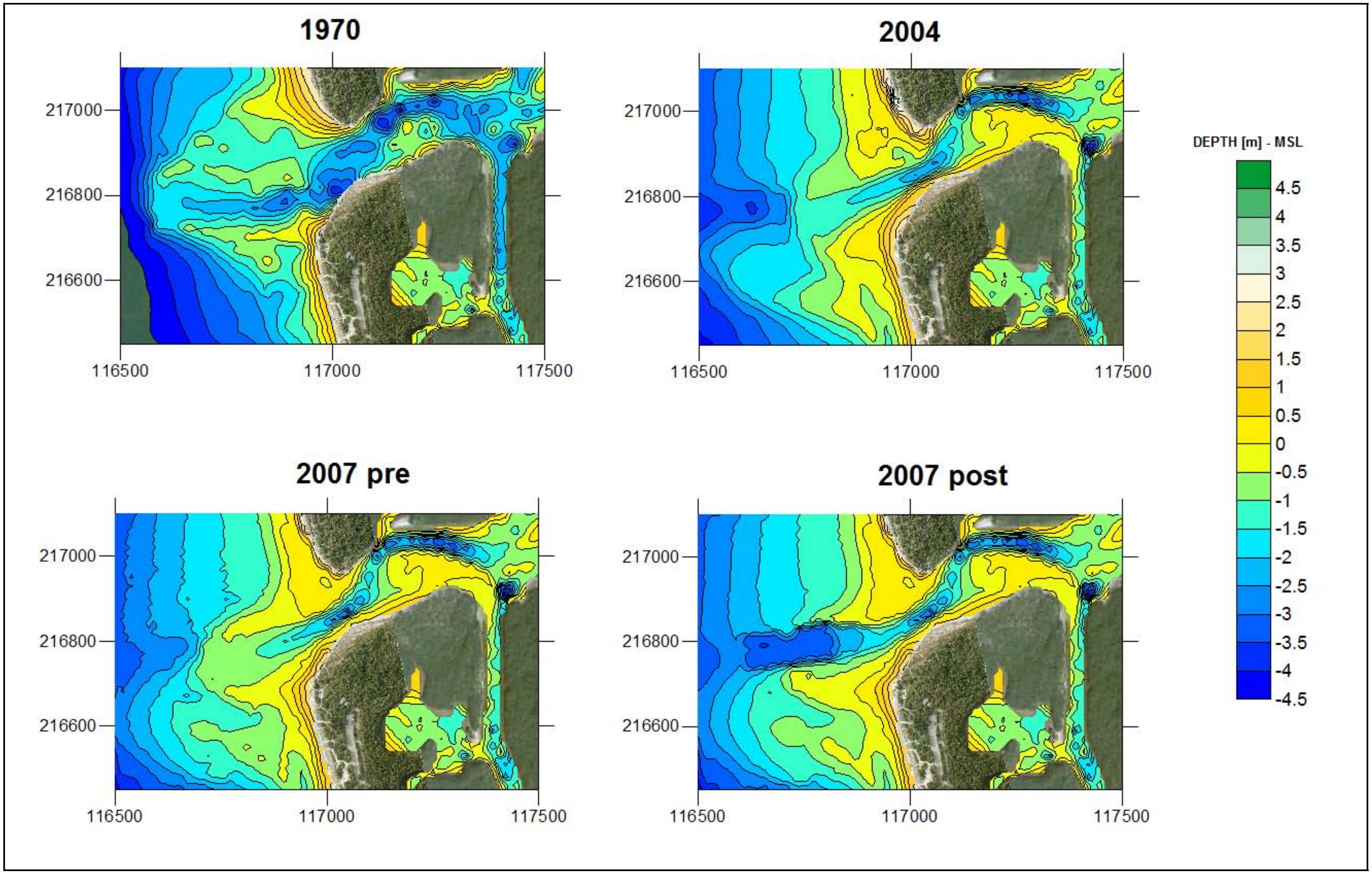
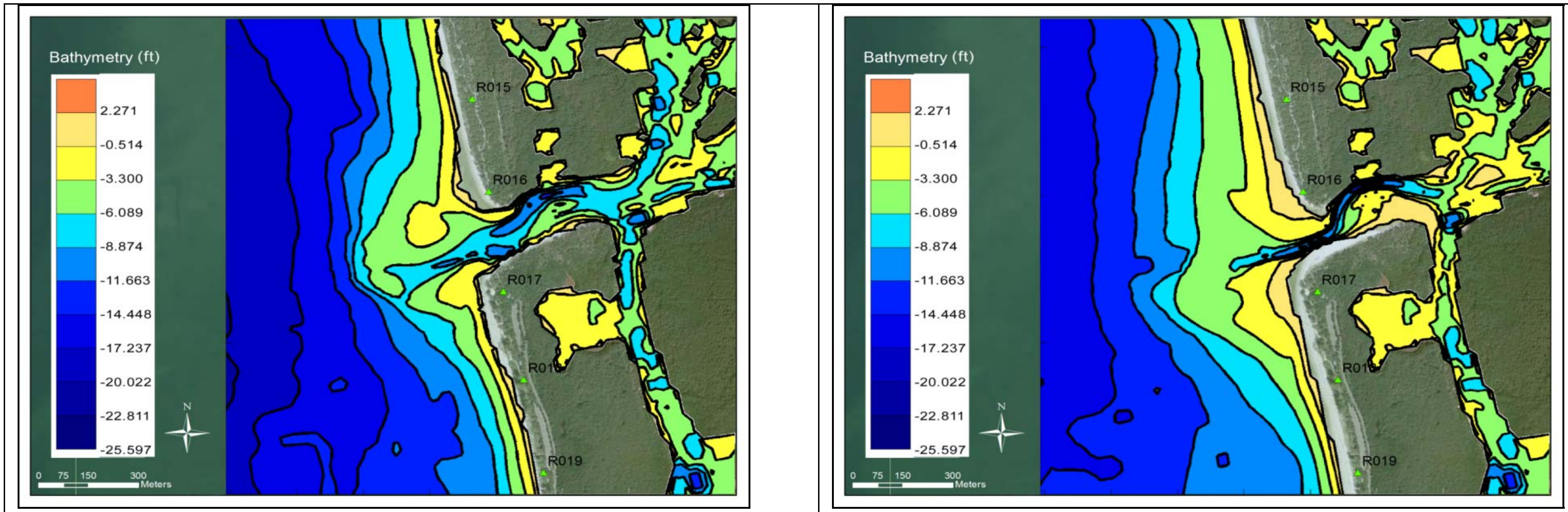
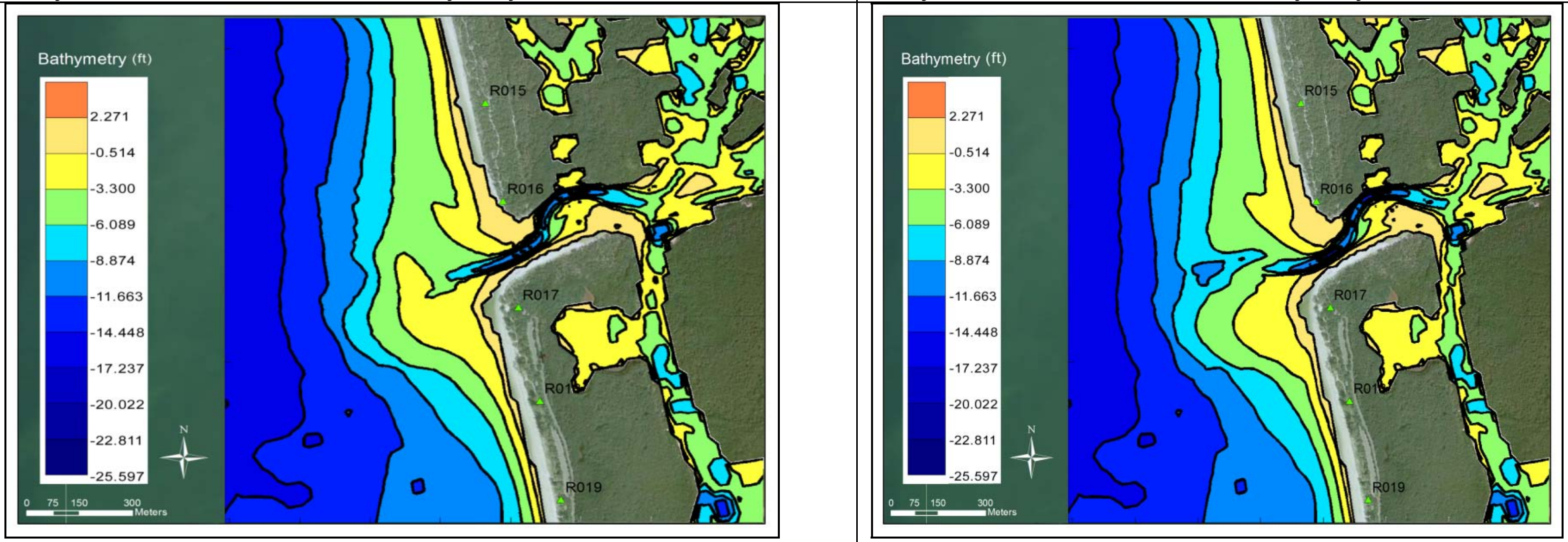


Figure 38. Historical bathymetries simulated and presented in meeting #1.



One year simulation based on 1970's bathymetry

One year simulation based on 2004 bathymetry



One year simulation based on 2007 pre-dredge bathymetry

One year simulation based on 2008 post-dredge bathymetry

Figure 39. Simulations conducted using historical bathymetric configurations

The one year simulation using the 2007 post-dredge bathymetry showed a shoal ridge crossing the dredged channel and leading to blockage to the outer section of the dredged channel. The green shade zone crosses the channel over a narrow section (Figure 39).

These simulations with historic bathymetries indicate that the 1970 configuration is the most efficient configuration in terms of channel depth maintenance, similar to what has been hypothesized by previous authors (H&M, 2007). However, the water depths resulting from the simulation using the 1970 configuration violate minimum draft requirements for safe small boat navigation at Wiggins Pass, which may pose a hazard to navigation. The main features that distinguish the 1970 channel configuration from that of the present conditions include a less developed flood shoal, allowing a straighter ebb-jet and smaller channel cross-sectional area (smaller channel dimensions).

Based on the results of this meeting, potential scenarios for navigation improvement and mitigation of erosion at Barefoot Beach were defined. These scenarios included changing fill placement location, modifying channel dimension to allow for smaller cross-sectional area, and conducting modification in the flood shoal and interior channel meander to allow for a straight channel configuration and a straight ebb jet similar to the 1970 configuration. Between the June 2008 meeting and the August 2008 meeting, ADCP data and a new bathymetry survey would become available, thus allowing for model refinement through calibration.

Meeting #2 – August 2008

Meeting #2 was conducted on the 12th of August 2008. Between meeting #1 and meeting #2, the morphology model was calibrated according to results from a local bathymetry survey (about 1.5 years after dredging), and the wave and flow models were calibrated according to ADCP results. Results from the calibration have been presented in a previous section of the report.

After model calibration and implementation of the new measured bathymetry, a slightly modified sediment transport pattern resulted (Figure 40). North of Wiggins Pass the sediment transport is south to north. There is a prominent reversal and swash (flood) channel between R16 and the southern end of Barefoot Beach. There is a secondary swash channel in front of the northern tip of Delnor Wiggins State Park Beach. The net sediment transport is north to south between R17 and R18, and undefined with small magnitude between R18 and R20. The net transport becomes north to south of R21. Barefoot Beach loses sand to the navigation channel by the sediment reversal and swash channel pathway at R16 and due to an increased south to north sediment transport potential from R15 to the north (Figure 40).

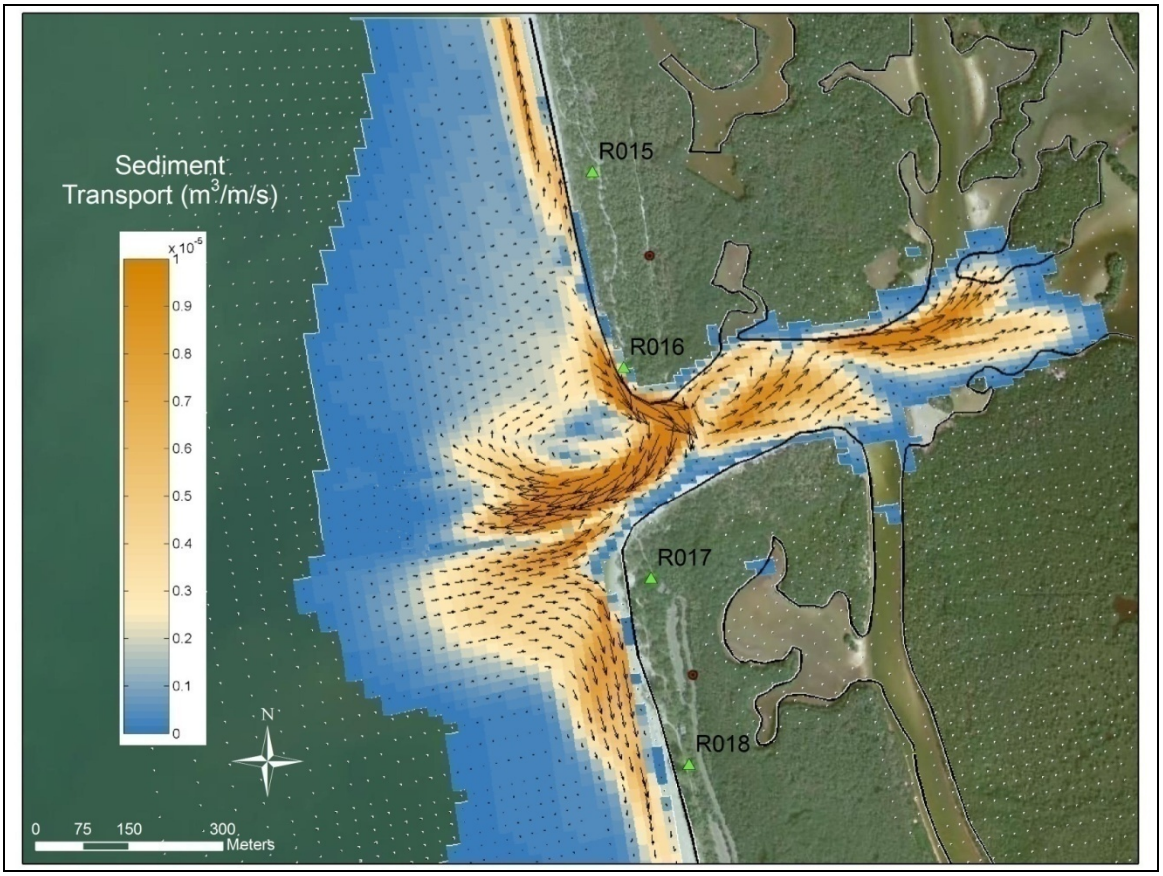


Figure 40. Patterns and magnitude of net annual sediment transport potential. The darker the color brown and the larger the arrows, equates to higher sediment transport.

Eight channel design alternatives were simulated in this phase of the study. The best alternatives were selected for further analysis (long-term simulations) based on a decision matrix. The alternatives simulated included:

- <u>Alternative 1:</u> Current permit plan based on 2007 port-dredge survey - Gulf channel is 250 ft wide, with deepest section at about 4.3 m MSL (14.7 ft NAVD or 13 ft MLW) deep (Figure 41).
- <u>Alternative 2:</u> Current permit plan with a cut across the existing flood shoal. The flood shoal cut is 30 m (100 ft) wide by 2.14 m MSL (7.7 ft NAVD or 6 ft MLW according to USACE recommendations) deep (Figure 42).
- <u>Alternative 3:</u> Current permit plan with a cut across the flood shoal (similar to Alternative #2), a cut to Cocohathee Channel, a small cut connecting to the south and temporary sand dikes blocking flows to the old channel meanders. The temporary (training) sand dikes were implemented in the model to an elevation of 1 m MSL (2.33 ft NAVD), which corresponds to 0.6 m (2 ft) above MHW (Figure 43). The flood shoal cut and other interior channel cuts are 30 m (100 ft) wide by

2.14 m MSL (7.7 ft NAVD or 6 ft MLW according to USACE recommendations) deep (Figure 43).

- Alternative 4: Gulf channel re-aligned to the southwest with 46 m (150 ft) wide and 3.7 m MSL (12.6 ft NAVD or 11 ft MLW) deep (Figure 44).
- Alternative 5: Gulf channel re-aligned to the southwest (same channel as Alternative 4) with a cut across the existing flood shoal. 2.14 m MSL (7.7 ft NAVD or 6 ft MLW according to USACE recommendations) deep (Figure 45).
- Alternative 6: Gulf channel re-aligned to the southwest with flood shoal cut (same as Alternative 5) with cuts to Cocohathee Channel, a small cut connecting to the south and temporary sand dikes blocking flows to the old channel meanders. The temporary sand dikes were implemented in the model to an elevation of 1 m MSL (2.33 ft NAVD), which corresponds to 0.6 m (2 ft) above MHW (Figure 46). The flood shoal cut and other interior channel cuts are 30 m (100 ft) wide by 2.14 m MSL (7.7 ft NAVD or 6 ft MLW according to USACE recommendations) deep (Figure 46).
- Alternative 7: Gulf channel straightened and elongated extending to the channel throat with 61 m (200 ft) wide and 3.7 m MSL (12.6 ft NAVD or 11 ft MLW) deep (Figure 47).
- Alternative 8: Gulf channel similar to Alternative 7 with a cut across the flood shoal (similar to Alternative 2), a cut to Cocohathee Channel, a small cut connecting to the south and temporary sand dikes blocking flows to the old channel meanders. The temporary sand dikes were implemented in the model to an elevation of 1 m MSL (2.33 ft NAVD), which corresponds to 0.6 m (2 ft) above MHW (Figure 48). The flood shoal cut and other interior channel cuts are 30 m (100 ft) wide by 2.14 m MSL (7.7 ft NAVD or 6 ft MLW according to USACE recommendations) deep (Figure 48).

Plots showing the initial bathymetry and the final bathymetry after 1 year for each scenario are shown in Figures 41 to 48. Based on the results of the simulations of the eight scenarios described above, a decision matrix that focused on engineering aspects was developed (Table 2). Erosion and deposition trends are described relative to the simulation with the current dredge plan, which was conducted using the 2007 post-dredge bathymetric survey as a baseline.

Scenarios 4 and 5 resulted in a treacherous channel with shallow controlling depths similar to the current dredging template and negative shoreline effects. It appears that channel realignment to the southwest exacerbated the effects of the swash (flood) channel adjacent to Barefoot Beach, and more of the southern end of Barefoot Beach was eroded in relation to the current dredging plan.

There was an improvement in the efficiency of the current channel dredging design when a cut across the flood shoal was included (Alternative 2, Table 2 and Figure 42). The smaller channel realigned to the southwest also performed better than the current plan (Alternative 1) when combined with the flood shoal cut and interior sand dikes (Alternative 6, Table 2 and Figure 46). Due to the smaller channel dimensions associated with this alternative and consequently smaller quantity of capital dredging, this alternative led to one of the smallest amounts of channel shoaling (Table 2).

Alternative 7, which consisted of a longer straight channel with smaller cross-sectional dimensions than the current dredging plan, showed improved channel efficiency and no adjacent shoreline impacts. The channel template of Alternative 7 was combined with the cut across the flood shoal and temporary sand dikes to develop Alternative 8. This alternative, together with Alternative 3, were the best alternatives in terms of channel efficiency (maintenance of deeper controlling depths and a straight channel configuration). Both caused positive impacts on Barefoot Beach and no negative impacts to the shoreline of Delnor-Wiggins State Park (Table 2, Figures 46 and 51). Because Alternative 8 achieved similar results as Alternative 3, with smaller cross-sectional dimensions of the Gulf channel, it was selected as the best alternative during this phase of the study.

A comparison of the predicted bathymetry for Alternative 1 and Alternative 8 is shown in Figure 49. The improvements in the navigation channel depth and width resulting from Alternative 8 are clearly visible in this figure. After 1 year, the minimum controlling depths within the Gulf part of the channel for Alternative 1 is 1.6 m MSL (5.8 ft NAVD) versus a controlling depth of 2.5 m MSL (8.8 ft NAVD) for Alternative 8. This is a consequence of better channel stability, which retards the formation of a shoal ridge across the Gulf channel.

Table 2. Decision matrix presented on August 2008, created based on the simulation of eight initial inlet modification alternatives.

Alternative		**Outer Channel Controlling depth**	**Outer Channel Infill Rate**	**Channel Stability**	**North Shoreline (R15-R16)**	**South Shoreline (R17-R18)**
No.	**Alternative Description**					
		ft-MSL	cy/yr	Description		Description
	Monitoring Survey Feb 2007 to June 2008	4 to 7	26,400	Poor	erosive	slightly accretional
Alt. 1	Permitted Channel	4 to 7	28,200	Poor	=	=
Alt. 2	Permitted Channel + Flood shoal cut	6 to 9	26,468	Intermediate	=	=
Alt. 3	Permitted Channel + flood shoal cut + sand dikes	6 to 10	30,552	Optimal	+	=
Alt. 4	150 ft realigned channel	4 to 7	19,600	Poor	-	-
Alt. 5	150 ft realigned channel + flood shoal cut	5 to 8	16,504	Poor	-	=
Alt. 6	150 ft realigned channel + flood shoat cut + sand dikes	5 to 8	19,290	Intermediate	+	=
Alt. 7	200 ft straight channel	6 to 10	33,618	Intermediate	-	=
Alt. 8	200 ft straight channel + flood shoat cut + sand dikes	6 to 10	29,979	Optimal	+	=

Existing Gulf channel is 250 feet wide.

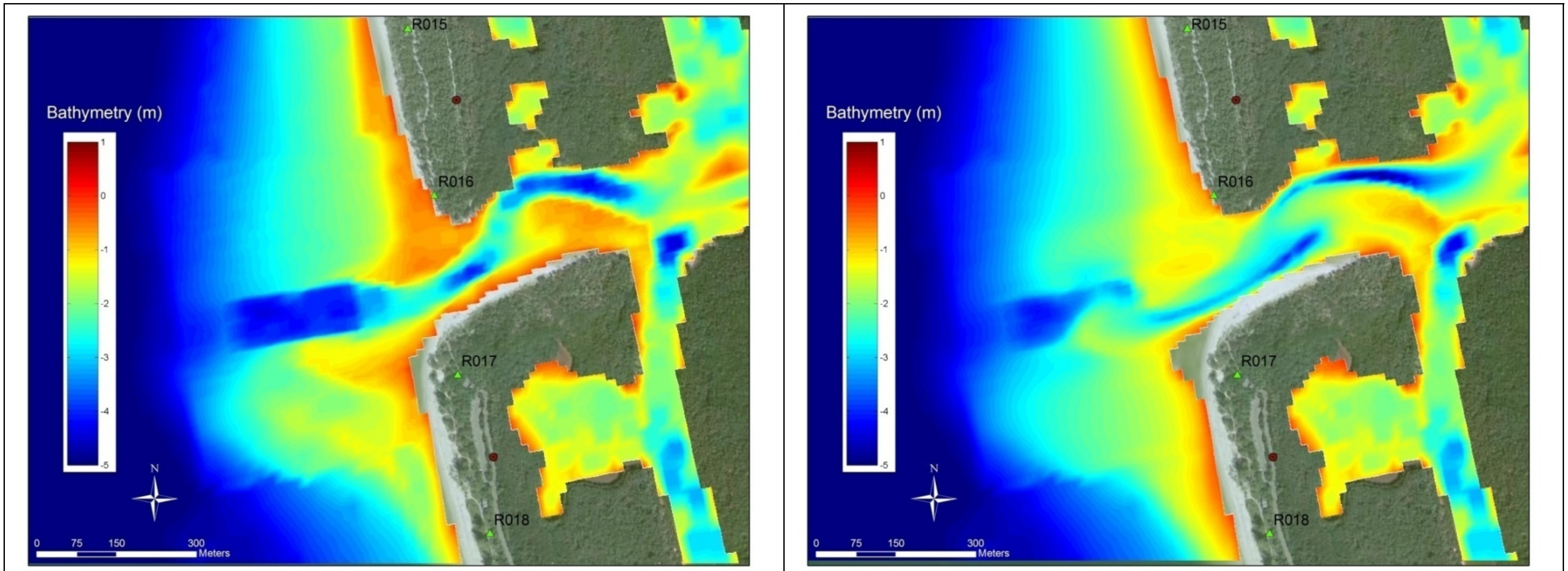


Figure 41. Initial bathymetry (left) and bathymetry simulated after 1 year (right) Alternative 1.

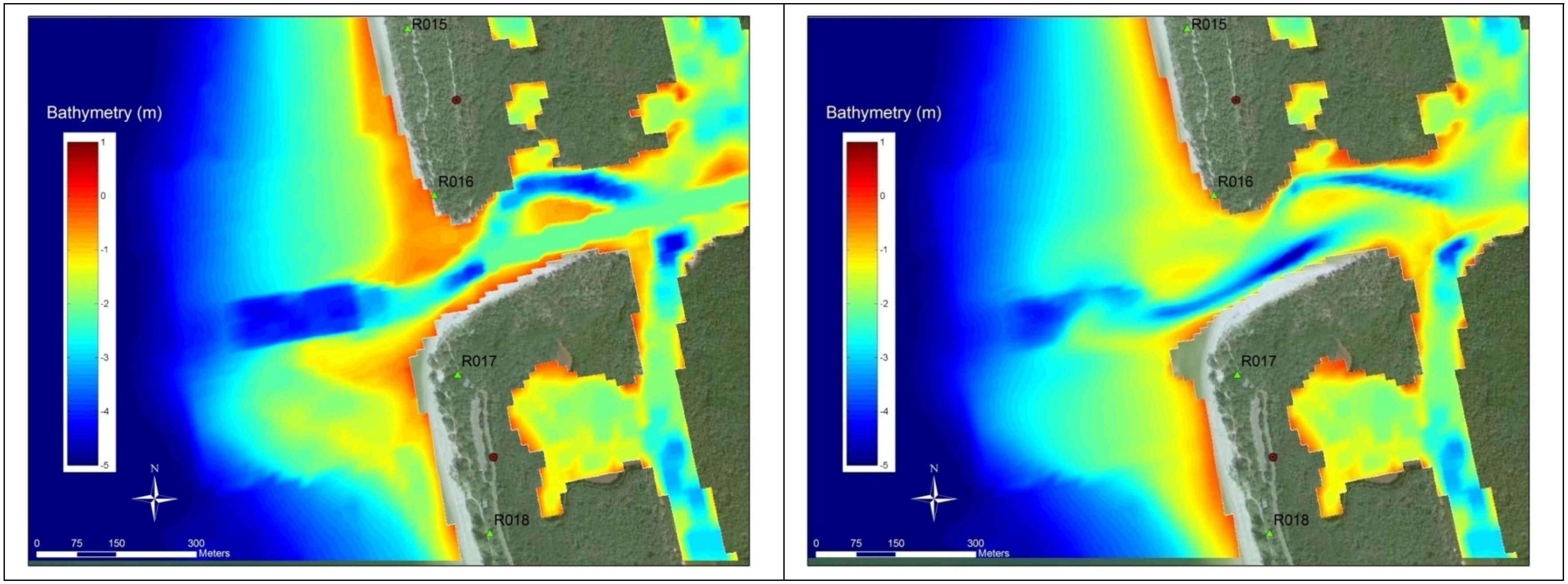


Figure 42. Initial bathymetry (left) and bathymetry simulated after 1 year (right) Alternative 2.

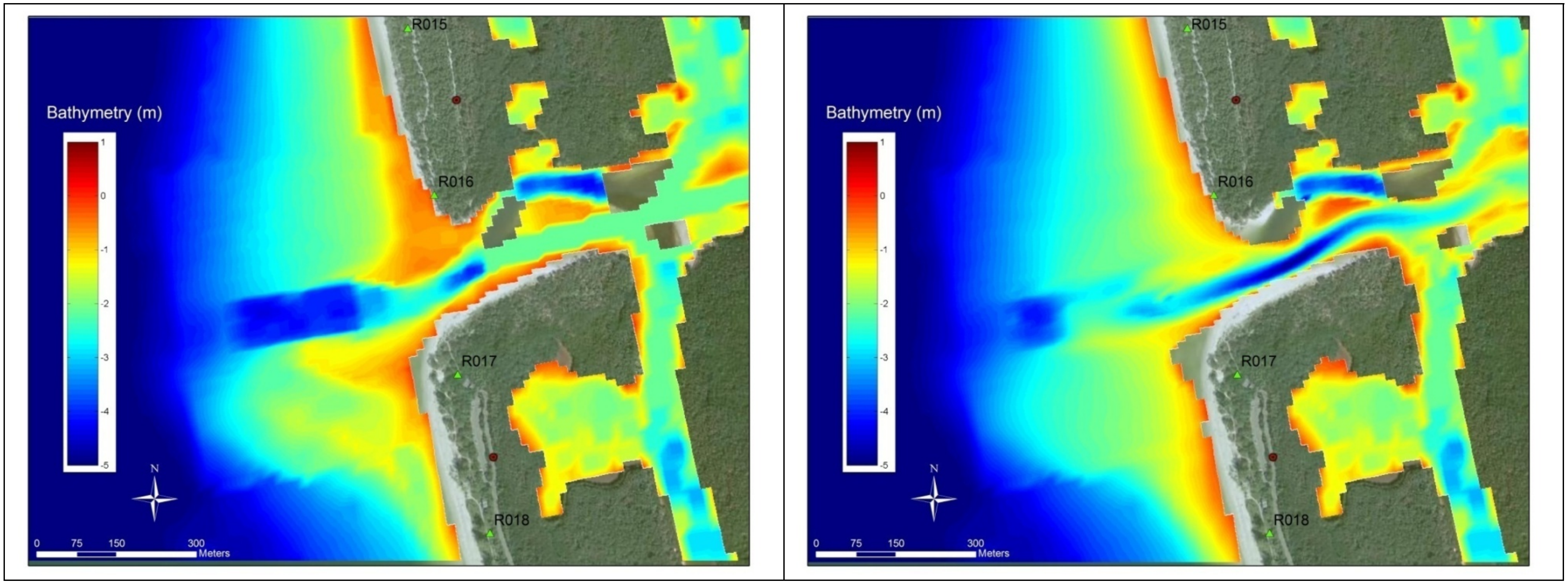


Figure 43. Initial bathymetry (left) and bathymetry simulated after 1 year (right) Alternative 3. Areas in the interior that do not show color shadings are above 0 MSL and indicate the location of the sand dikes.

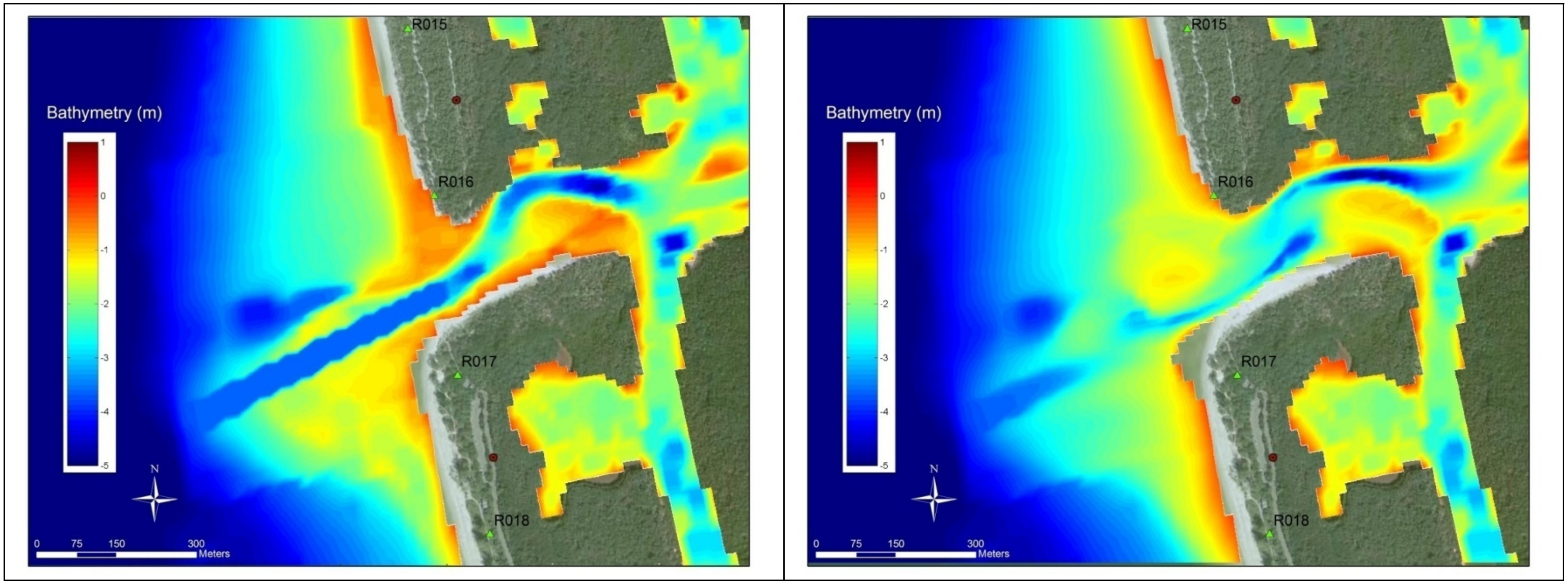


Figure 44. Initial bathymetry (left) and bathymetry simulated after 1 year (right) Alternative 4.

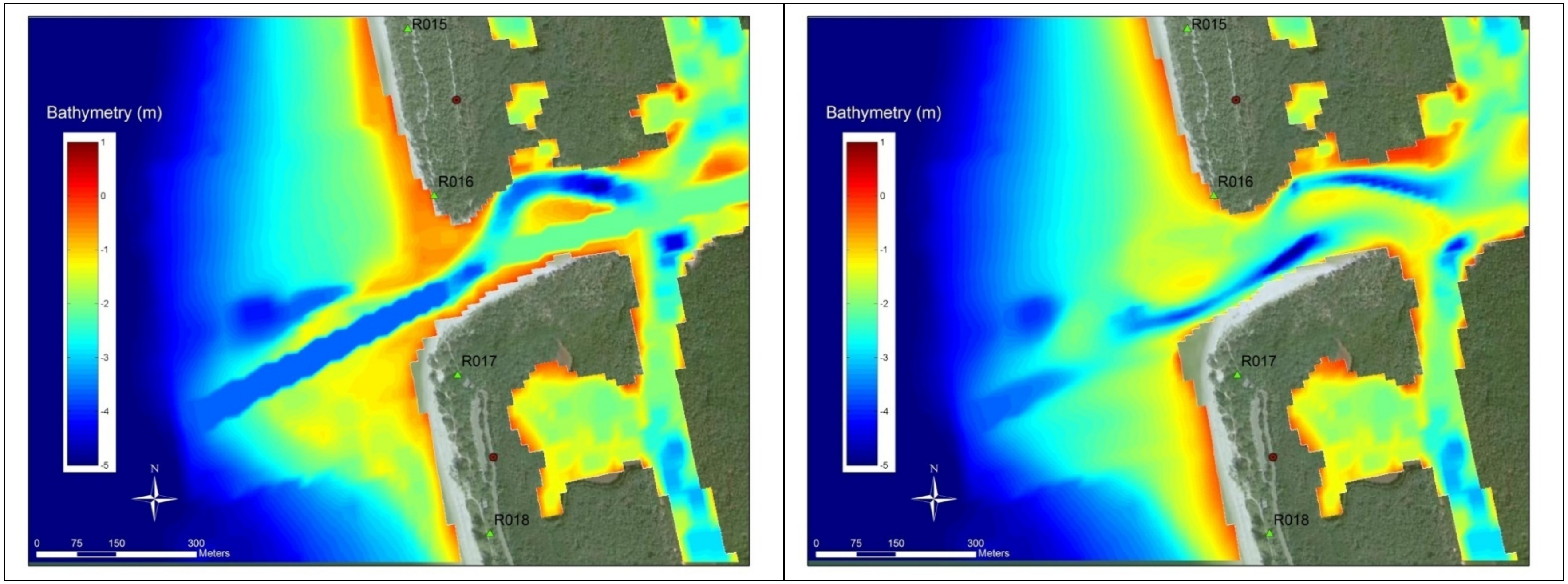


Figure 45. Initial bathymetry (left) and bathymetry simulated after 1 year (right) Alternative 5.

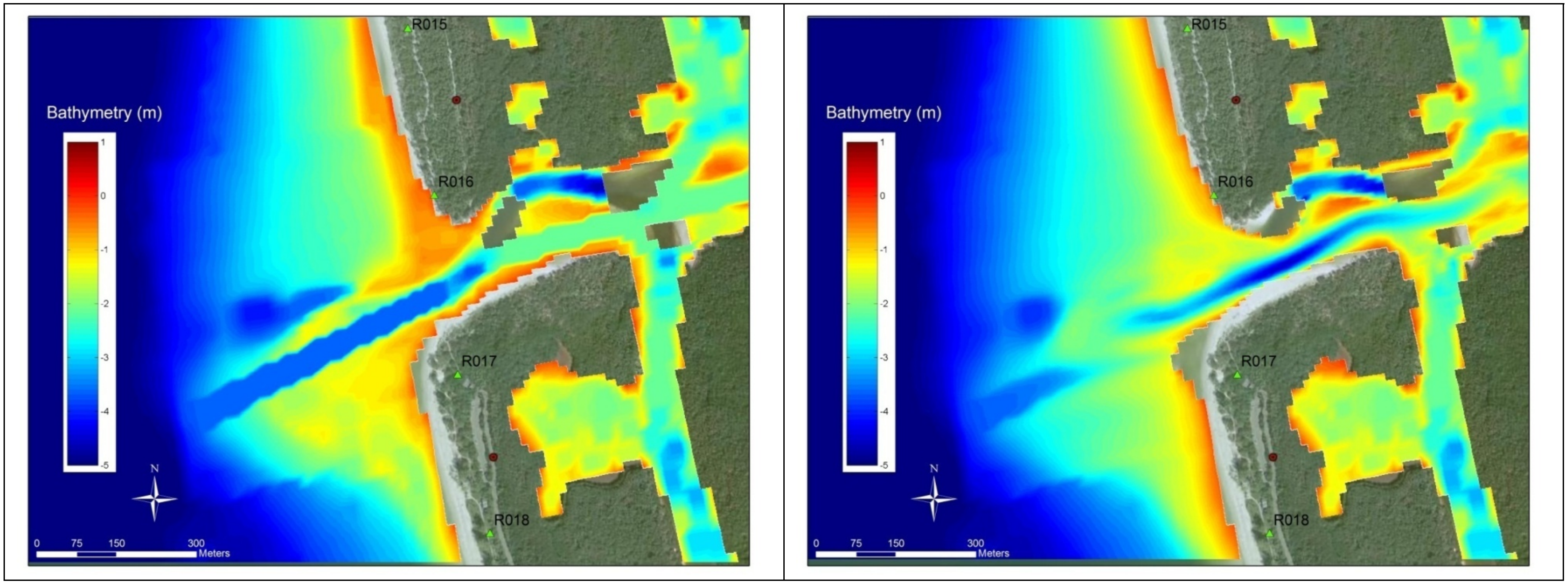


Figure 46. Initial bathymetry (left) and bathymetry simulated after 1 year (right), Alternative 6. Areas in the interior that do not show color shadings are above 0 ft MSL and indicate the location of the sand dikes.

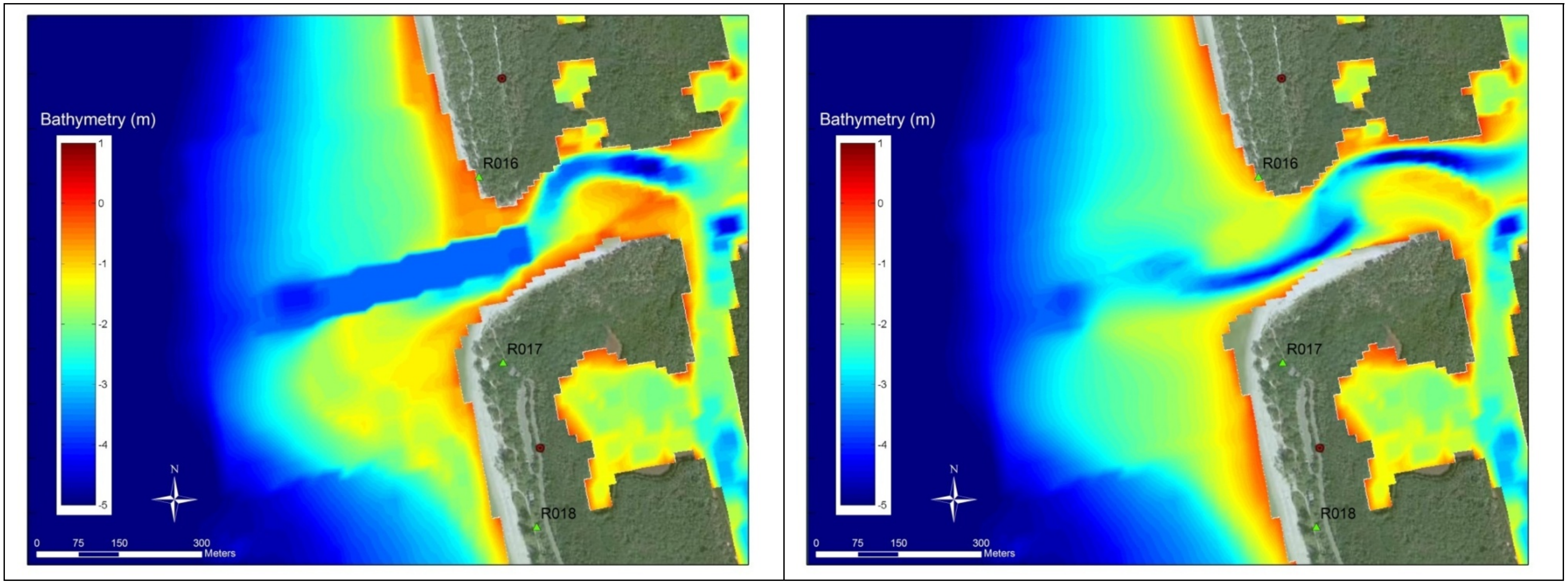


Figure 47. Initial bathymetry (left) and bathymetry simulated after 1 year (right), Alternative 7.

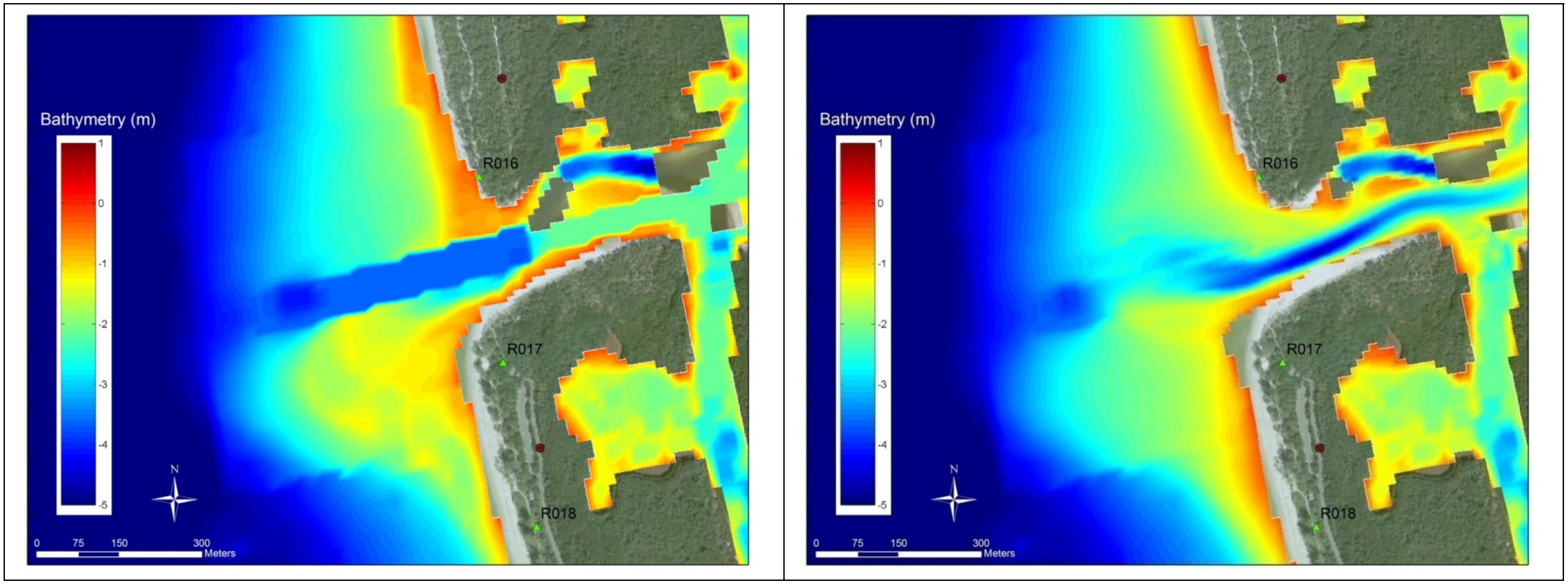


Figure 48. Initial bathymetry (left) and bathymetry simulated after 1 year (right), Alternative 8. Areas in the interior that do not show color shadings are above 0 MSL and indicate the location of the sand dikes.

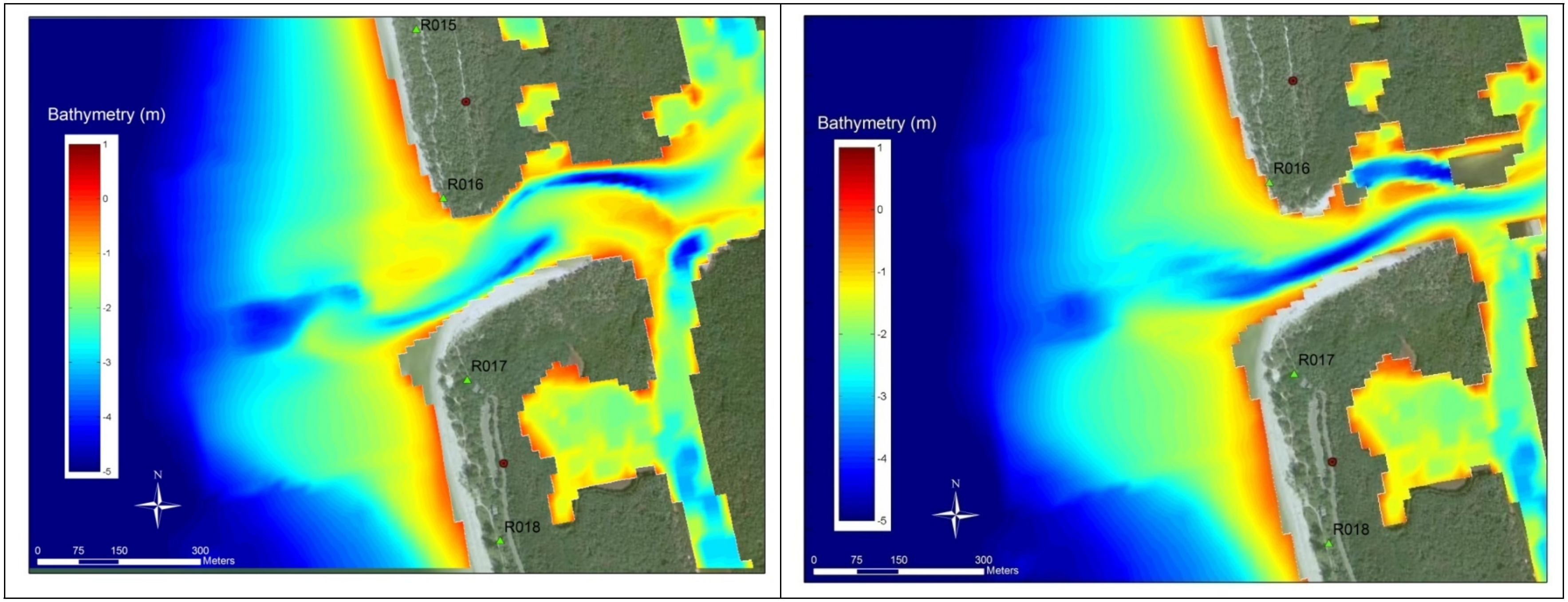


Figure 49. Comparison of final morphology predicted after 1 year for Alternative 1, the existing dredge template design (left), and for Alternative 8, the modified channel with interior inlet modifications that included a flood shoal cut and temporary sand dikes (right). The minimum controlling depth after 1 year for Alternative 1 is 6 m (5.5 ft) versus a controlling depth of 2.5 m (8.3 ft) for Alternative 8.

The predicted bathymetry for Alternative 8 was subtracted from the predicted bathymetry from Alternative 1 (Figure 49) in order to calculate relative elevation changes. Relative elevation changes provide an indication of the location of erosion and deposition caused by the alternative, in relation to the do-nothing scenario (current dredge design template). In relation to Alternative 1, Alternative 8 caused scouring across the navigation channel (red shades, Figure 50). Deposition was observed at the southern tip of Barefoot Beach, which was caused by spreading of the temporary sand dike (Figure 50). No significant impacts, positive or negative, were observed on the shorelines further to the south and north of the navigation channel.

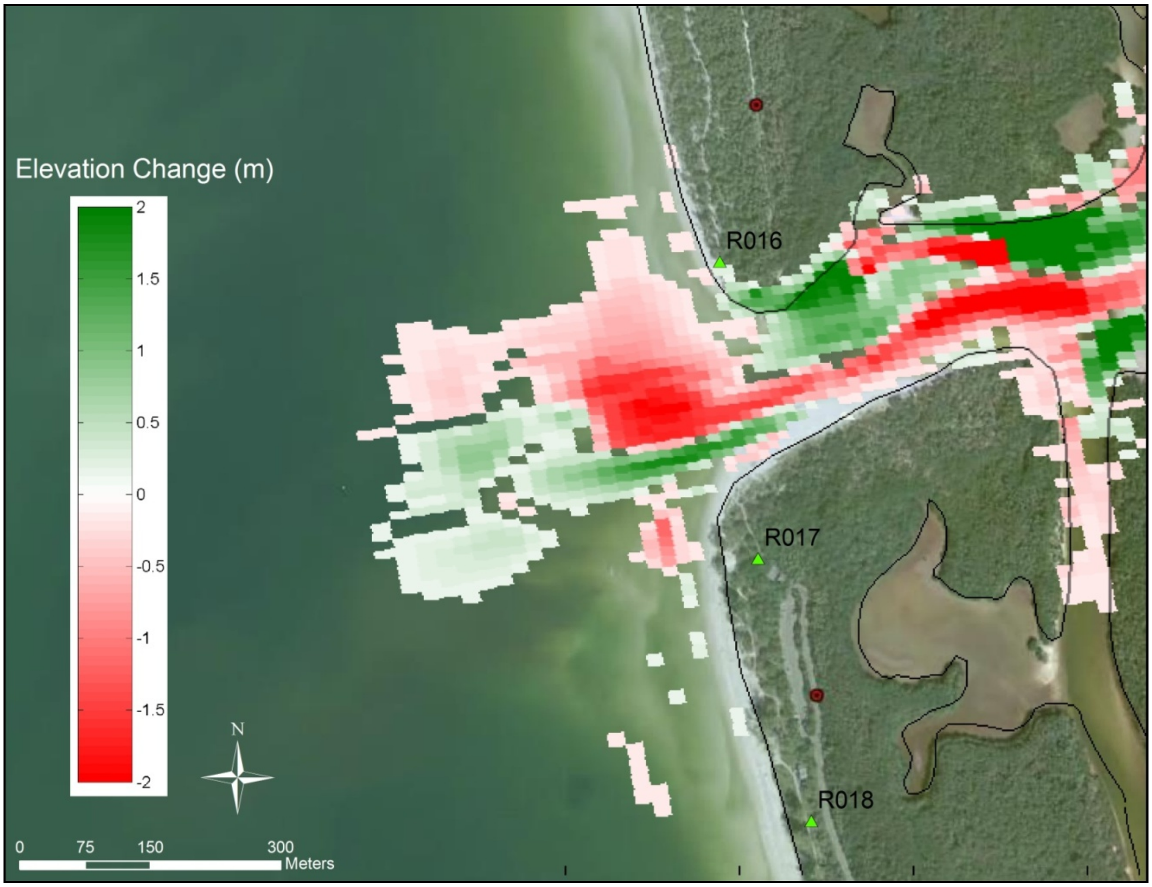


Figure 50. Relative bathymetry change calculated from subtracting the predicted bathymetry for Alternative 8 (best scenario) by the predicted bathymetry for Alternative 1 (current plan), or, predicted bathymetry Alternative 8 – predicted bathymetry Alternative 1.

The results of this initial screening of alternatives indicated that a cut across the flood shoal, and re-dimensioning the channel to a smaller cross-sectional area, as hypothesized by previous studies, was beneficial for channel stability and depth maintenance. Maximum efficiency of these modifications, however, is observed when the old meandering channel is constricted by temporary sand dikes so that a dual channel system is avoided, and the flows are concentrated in a single channel improving hydraulic efficiency. The sand dike on the southern tip of Barefoot Island, on the Gulf mouth of the old meandering channel, had beneficial effects in mitigating erosion pressure on the southern end and inlet shoreline of Barefoot Beach. Based on these observations, it seems that a optimal channel configuration is a straight channel that connects to the interior water bodies through a straight cut across the existing flood shoal, combined with some restrictions (sand dikes) to avoid a dual channel system and other small hydraulic connections (Cocohatchee cut and south channel cut).

For the next phase of the study, a refinement of the best alternatives along with long-term simulations of 1 year, 2 years, 4 years were recommended. These were coupled with a simulation of a wave climate based on a time-period with intense hurricane activity. A simulation of fill placement closer to the inlet (between R13 and R15), and led to a refined decision matrix based on fewer alternatives. Based on the results presented to date, the modeling review committee selected four alternatives for further analysis which are described in the next section of this report.

Meeting #3 – October 2008

A third meeting was conducted on the 22nd of October 2008 to review the results of the alternatives selected by the modeling review committee on August 2008. The refined alternatives simulated were:

- <u>Alternative 1:</u> Current permit plan based on 2007 port-dredge survey - Gulf channel 250 ft wide, with deepest section at about 4.3 m MSL (14.7 ft NAVD or 13 ft MLW) deep.
- <u>Alternative 2:</u> Gulf channel similar straightened and elongated with a cut across the flood shoal (similar to former Alternative #8), a cut to Cocohathee Channel, a small cut connecting to the south and temporary sand dikes blocking flows to the old channel meanders. The Gulf Channel is 61 m (200 ft) wide and 3.67 m MSL (12.6 ft NAVD, or 11 ft MLW) deep. The temporary sand dikes were implemented in the model to an elevation of 1 m MSL (2.33 ft NAVD), which corresponds to 0.6 m (2 ft) above MHW. The flood shoal cut and other interior channel cuts are 30 m (100 ft) wide by 2.14 m MSL (7.7 ft NAVD or 6 ft MLW according to USACE recommendations) deep.
- <u>Alternative 3:</u> Current permit plan in the Gulf channel with all the interior features of Alternative 2 (interior channel cuts and sand dikes).
- <u>Alternative 4:</u> Gulf channel re-aligned to the southwest with 61 m (200 ft) wide and 3.7 m MSL (12.6 ft NAVD, or 11 ft MLW) deep (former Alternative 5 with adjustments).

These alternatives are shown in Figure 51.

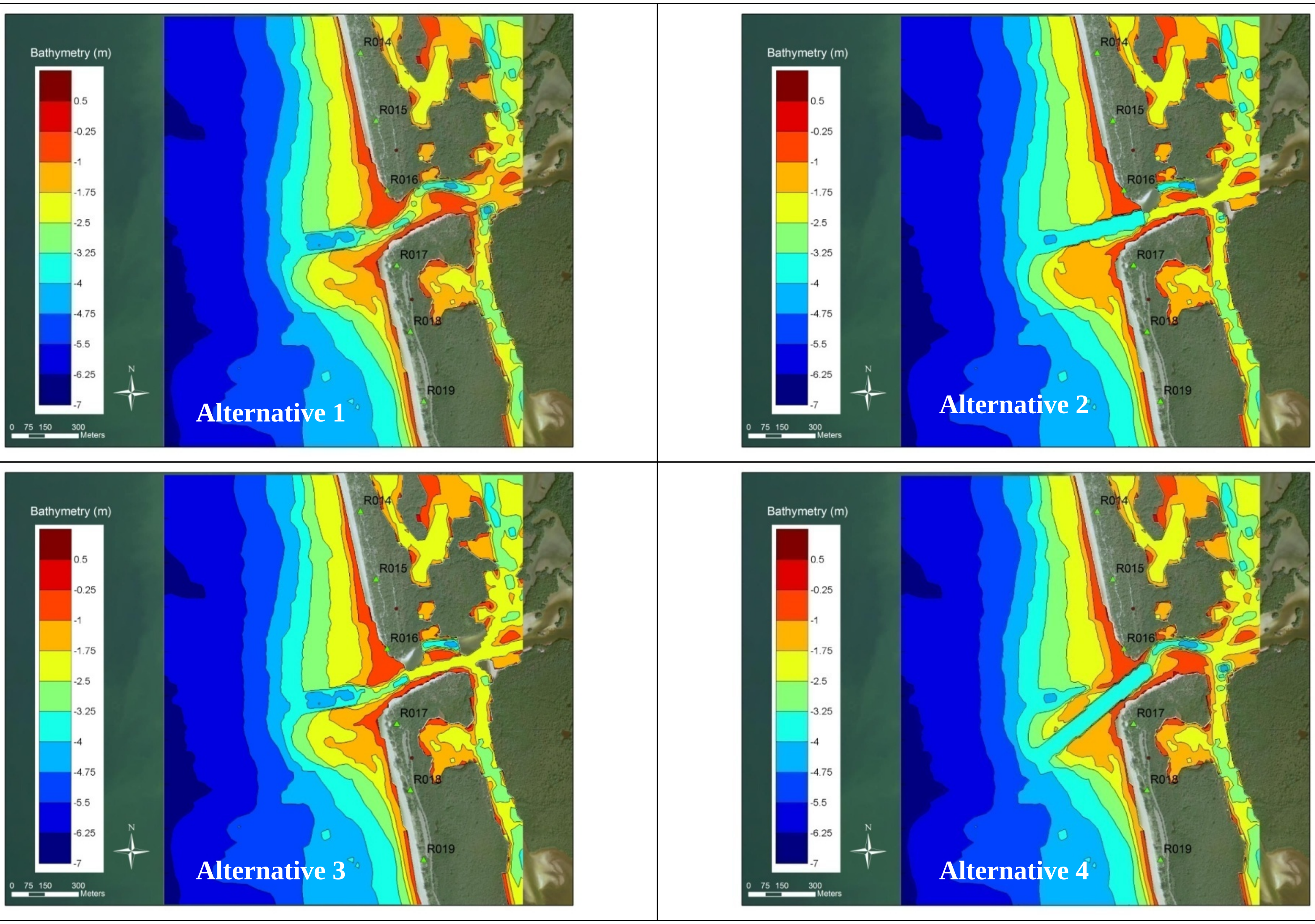


Figure 51. Four alternatives simulated for the third modeling reviewing committee meeting.

One Year Simulation

The one year simulation indicates that greater navigation depths were maintained after one year for Alternatives 2 and 4 (Figure 52 and Table 3) than Alternatives 1 and 3. The controlling depths for these alternatives are approximately 2.5 m MSL (8 ft NAVD), while the controlling depths for Alternatives 1 and 3 were 1.75 m MSL (6 ft NAVD) (Figure 52). Alternative 2 resulted in a wider and straighter channel than Alternative 4, as shown in Figure 52.

Erosion and sedimentation plots (Figure 53) show that Alternative 4 caused the greatest amount of erosion at Barefoot Beach. Alternatives 2 and 3, (which had a straighter channel with interior sand dikes), were the most favorable in terms of reduction of erosion at the southern end of Barefoot Beach and its interior shorelines (Figure 53). All the alternatives had similar effects on the shoreline at Delnor Wiggins State Park (Figure 53).

Table 3 shows that Alternatives 1 to 3 exhibited similar volumes of shoaling after one year, with Alternative 2 being slightly less than the others. Alternative 4 demonstrated the greatest amount of shoaling. After one year, approximately 29,741 m^3 (38,900 cy) shoaled in the navigation channel under this scenario against 20,640 m^3 (27,000 cy) for Alternative 2.

Ebb-tidal flows at the end of the morphological period, before morphological changes, for each alternative are shown in Figure 54. Stronger ebb currents, up to 1.2 m/s (3.9 ft/s) are observed in Alternatives 2 and 3 in relation to Alternatives 1 and 4. This is due to the smaller cross-sectional area created by the flood shoal cut combined with temporary sand dikes placed to block the old channel pathway.

Net annual sediment transport for Alternatives 2 and 4 are shown in Figure 55. Under Alternative 2, the reversal near the south end of Barefoot Island and the amount of transport over the swash (flood) channel is more prominent than in Alternative 4 (Figure 55). The re-aligned channel in Alternative 4 also causes the ebb jet to wrap around the northern end of Delnor-Wiggins State Park, inhibiting a flood channel (as evidence by the net onshore-offshore transport) in this location as seen in Alternative 2. Transport patterns along the beach are similar for both alternatives.

The relative changes caused by Alternatives 2 and 4, in relation to the baseline simulation (Alternative 1, existing dredging template), are shown in Figure 56. The plots clearly show that, in relation to the baseline, Alternative 2 causes deposition between R16 and the southern tip of Barefoot Beach (spreading of sand dike), a deepening of the navigation channel, and no noticeable effects on Delnor-Wiggins State Park. In relation to the baseline, Alternative 4 causes additional erosion along Barefoot Beach and some erosion stress against the north end of Delnor Wiggins State Park (Figure 56).

Results from one year simulations indicate that Alternative 2 is the only alternative that achieves improved channel efficiency with positive effects on Barefoot Beach and no significant effects on Delnor Wiggins State Park. Alternative 4, even though producing improved navigation conditions in relation to the current plan, caused adverse effects on the adjacent shorelines.

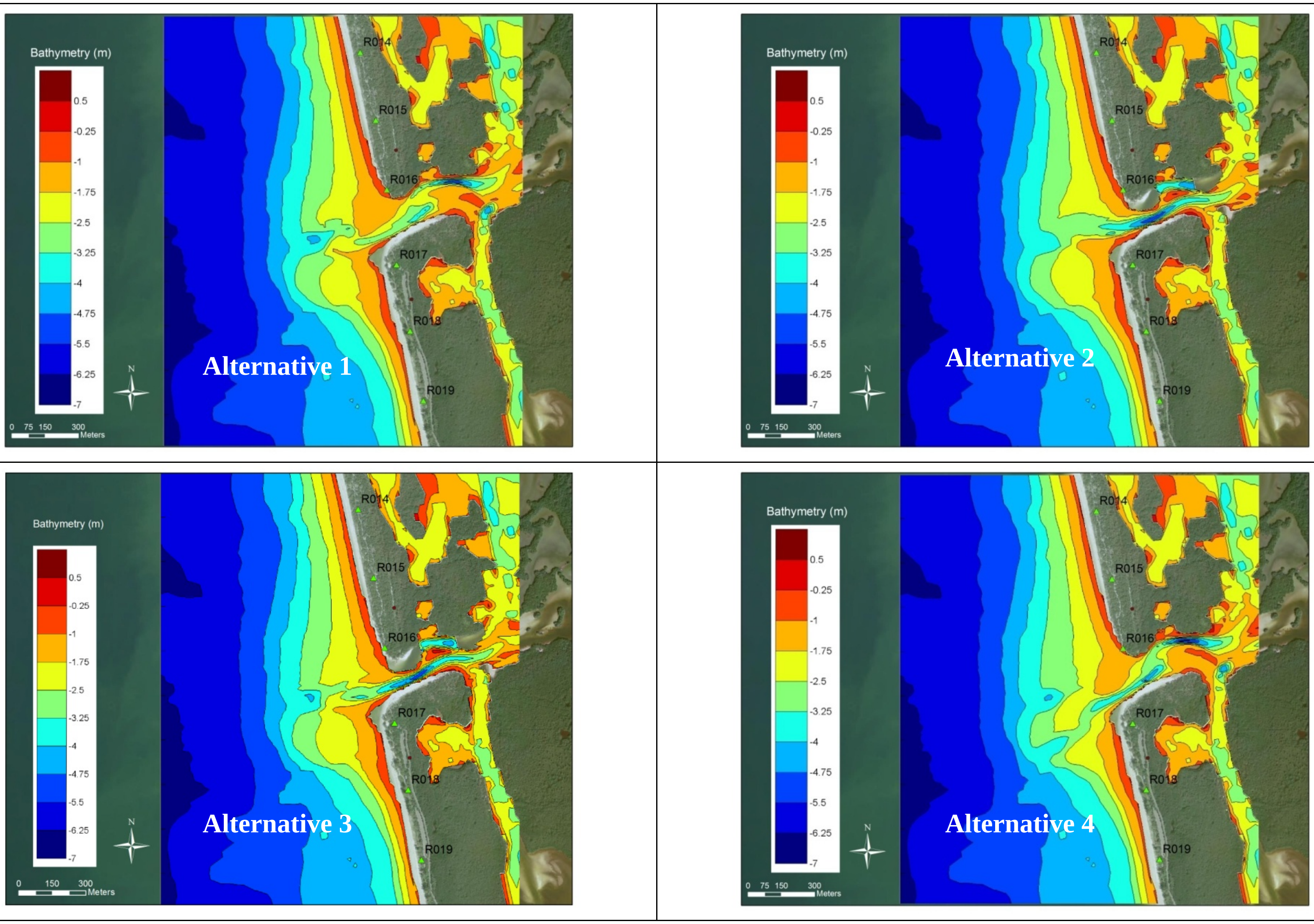


Figure 52. Bathymetry simulated after one year, for Alternatives 1 to 4.

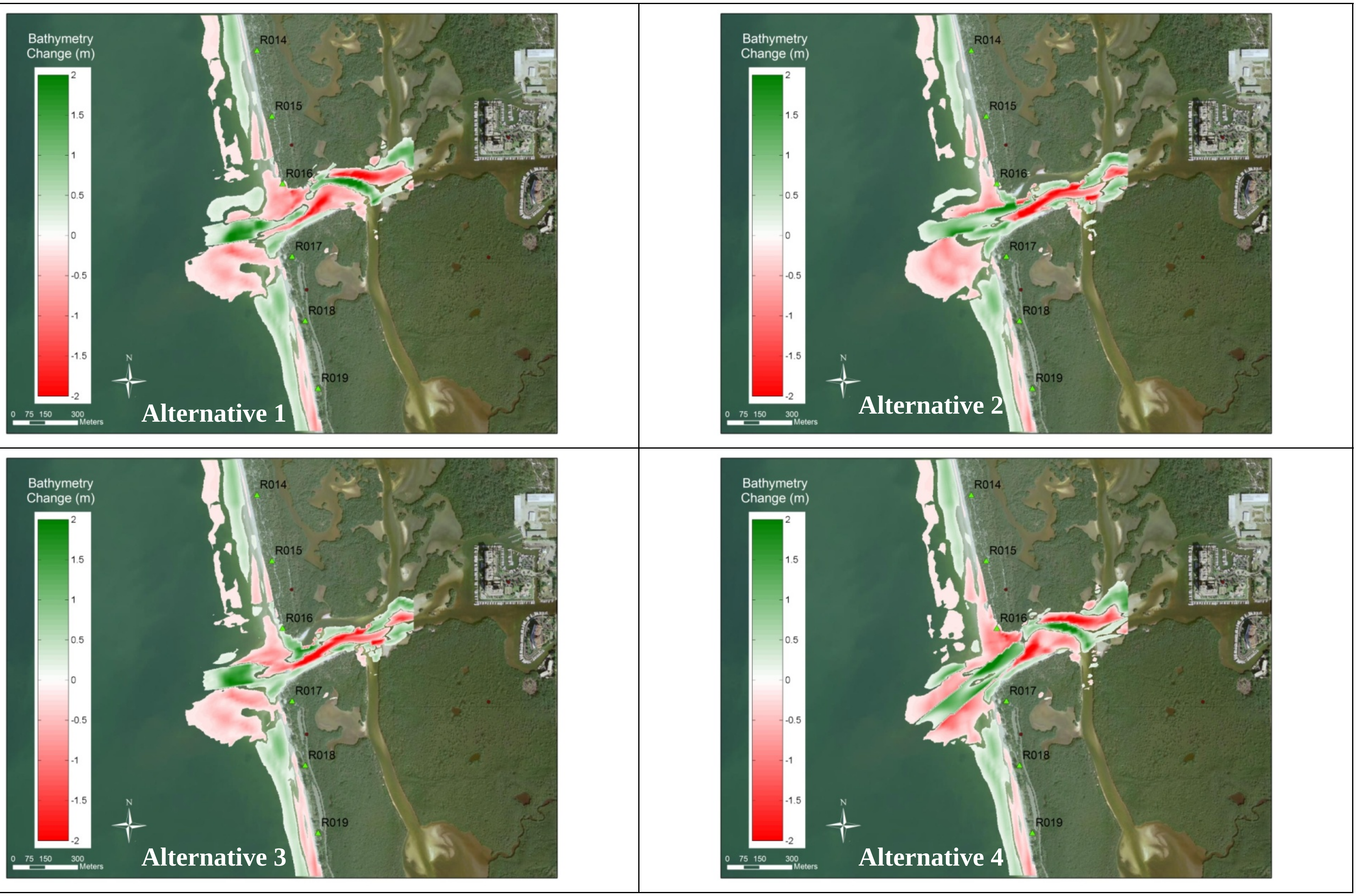


Figure 53. Erosion and sedimentation simulated after one year, Alternatives 1 to 4.

Table 3. Decision Matrix based on results from 1 year simulations.

Alternative		Outer Channel Controlling depth	Outer Channel Infill Rate	Channel Stability	North Shoreline (R15-R16)	South Shoreline (R17-R18)	Limestone Removal
No.	**Alternative Description**						
		Ft-NAVD	CY/yr	Description		Description	
	Monitoring Survey Feb 2007 to June 2008	4 to 7	26,400	Poor	Erosive	slightly accretional	
Alt 1	Permitted channel	~6	28,300	Poor	=	=	No
Alt 2	200’ Straight channel + flood shoal cut and sand dikes	~8	27,000	Good	+	=	Some
Alt 3	Permitted channel + flood shoal cut and sand dikes	~7	29,100	Intermediate	+	=	No
Alt 4	200’ Re-aligned channel	~8	38,900	Good	-	+/-	Yes
	Alt 1 + Beach Fill	~6	30,100	Poor	+	=	No

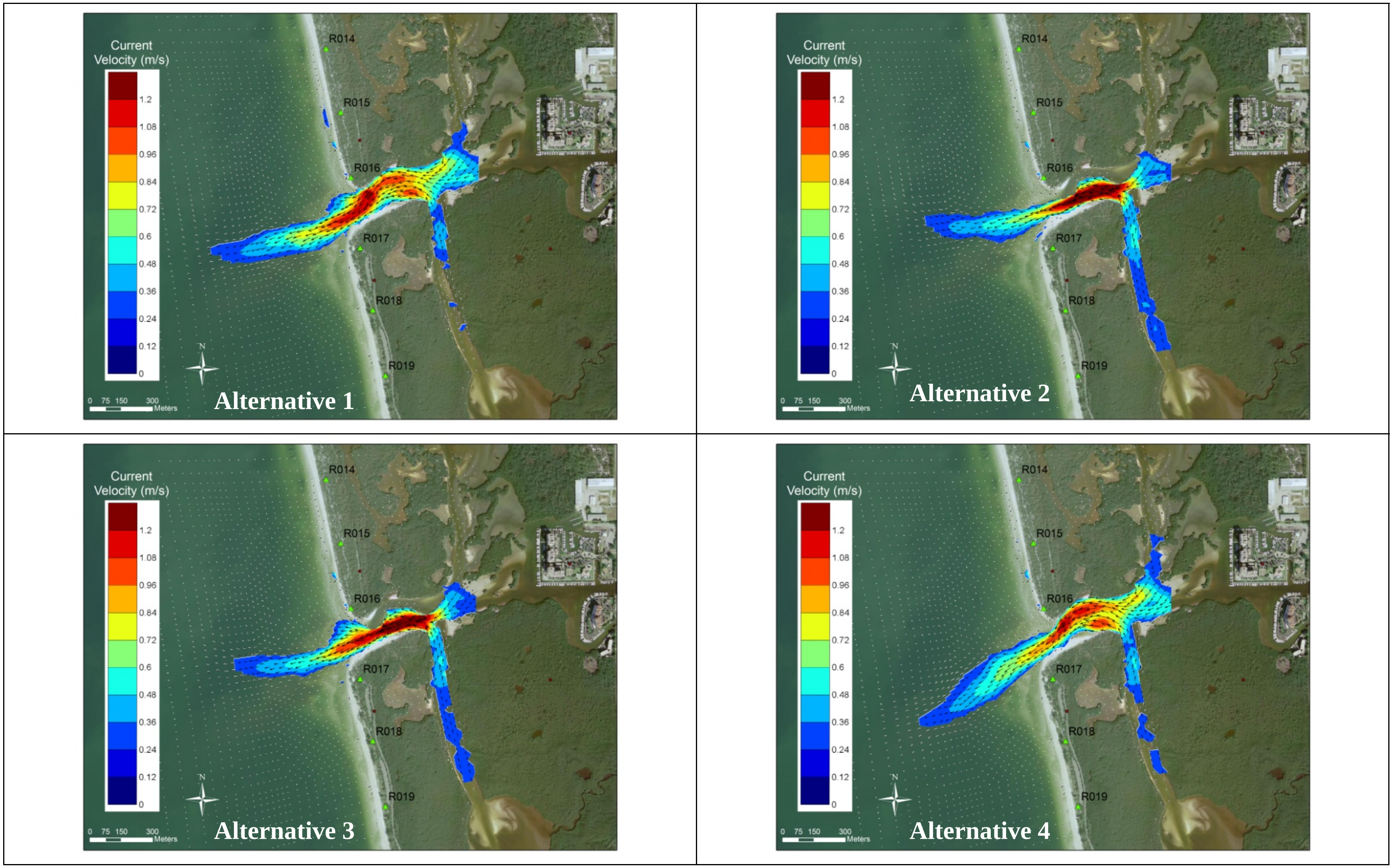


Figure 54. Ebb-tidal flows for all the alternatives simulated at the end of the model start up period, before morphological changes take place.

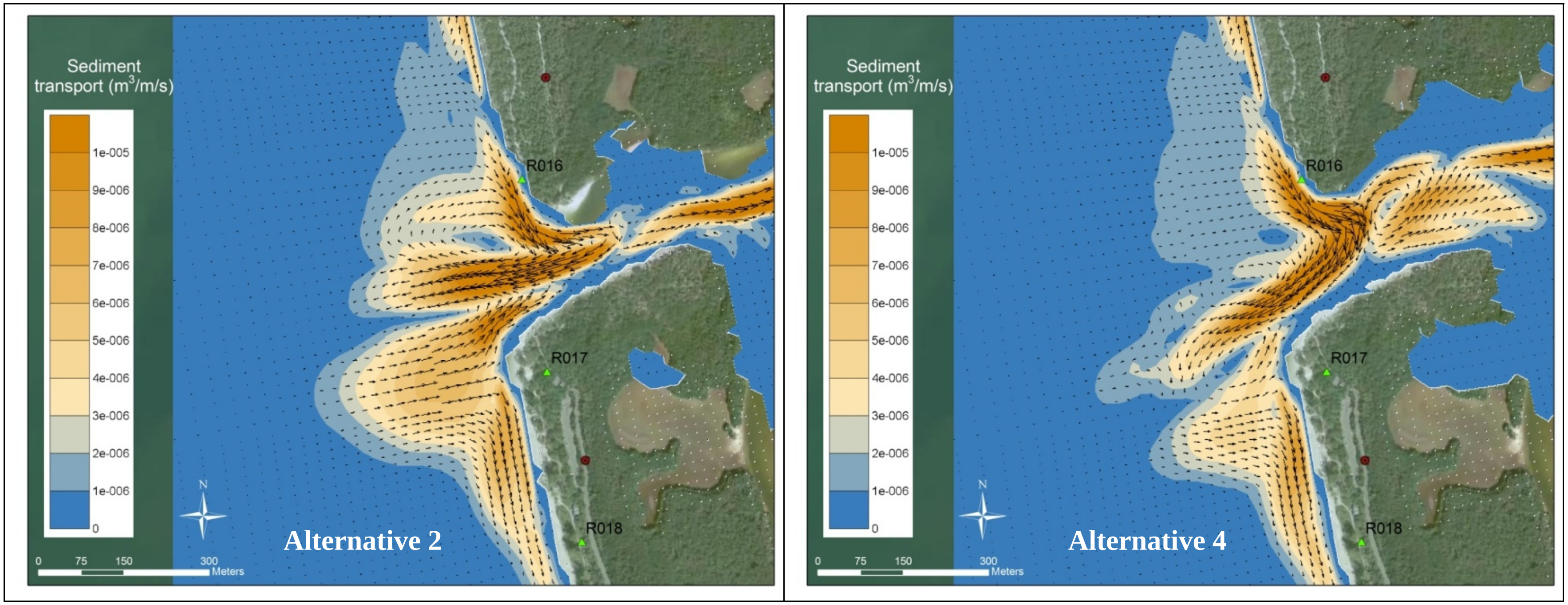


Figure 55. Net annual sediment transport potential for Alternative 2 (left) and Alternative 4 (right).

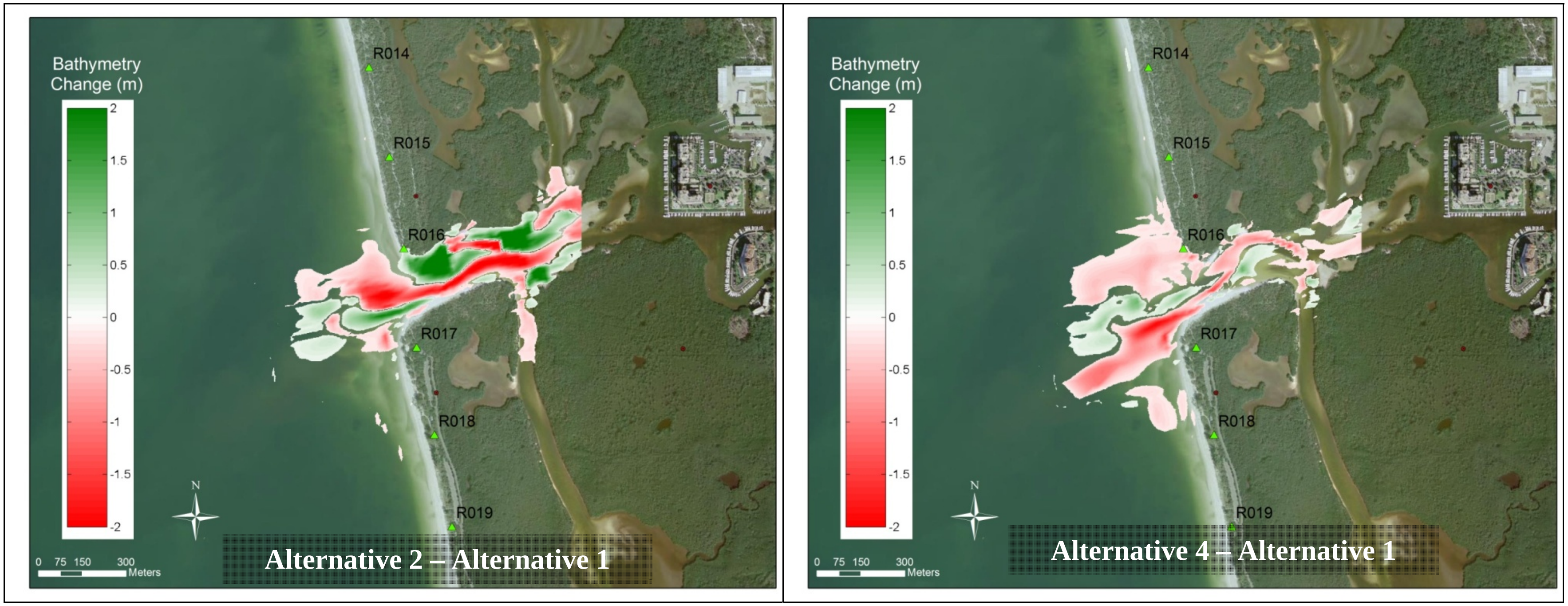


Figure 56. Relative bathymetric changes. On the left the predicted 1 yr bathymetry for Alternative 2 is subtracted from the predicted bathymetry from Alternative 1 showing the relative bathymetry changes induced by Alternative 2 in relation to the baseline (Alternative 1, or current dredge plan). On the right the predicted 1 year bathymetry from Alternative 4 is subtracted from the predicted bathymetry from Alternative 1. Even though Alternative 4 resulted in an improved channel without the need for flood shoal dredging it caused negative effects (additional erosion) on Barefoot Beach and in the northern end of Delnor Wiggins State Park.

Two Year and Four Year Simulations

Results from the two-year and four-year simulations confirmed those of the one year simulations. Alternative 2 results in the best navigable channel configuration after the simulation period, with positive effects at the southern end of Barefoot Beach and negligible effects on Delnor-Wiggins State Park. Of all the alternatives simulated, the one that produced the least favorable navigation conditions after 4 years is the baseline simulation, which consists of the current dredge plan. All other alternatives showed some navigation improvement over the baseline; however, Alternative 2 was the only one to provide navigation improvements combined with positive effects on the southern end of Barefoot Beach and no negative effects on Delnor Wiggins State Park (see Figures 57, 58 and 59).

At the end of the simulation period controlling depths on the Gulf channel were in the range of 1 m MSL (3.9 ft NAVD) for Alternative 1 (existing plan) and 1.75 m MSL (6 ft NAVD) for the other three alternatives (Table 4). However, the channel was straighter with fewer obstructions in Alternative 2 than in Alternatives 3 and 4 (see Figures 57 and 58). Alternatives 2 and 3 were the most favorable for erosion mitigation of Barefoot Beach. Alternative 4, on the other hand, was the least favorable and increased erosion at Barefoot Beach in relation to the baseline simulation (Figure 59).

Annual sedimentation rates decreased for the longer simulation periods. Most of the sediment fills in the channel within the first two-years after dredging. After this time-period the channel reaches a quasi-equilibrium where sediment is only carried around in response to fluctuations in wave and tidal forcings, but minimal additional sedimentation occurs. Annual sedimentation rates were smaller for Alternative 2, followed by Alternatives 1, 3 and 4. Similar to the one-year simulations, the sedimentation rate was greater for Alternative 4 (southwesterly channel) in relation to the other alternatives tested.

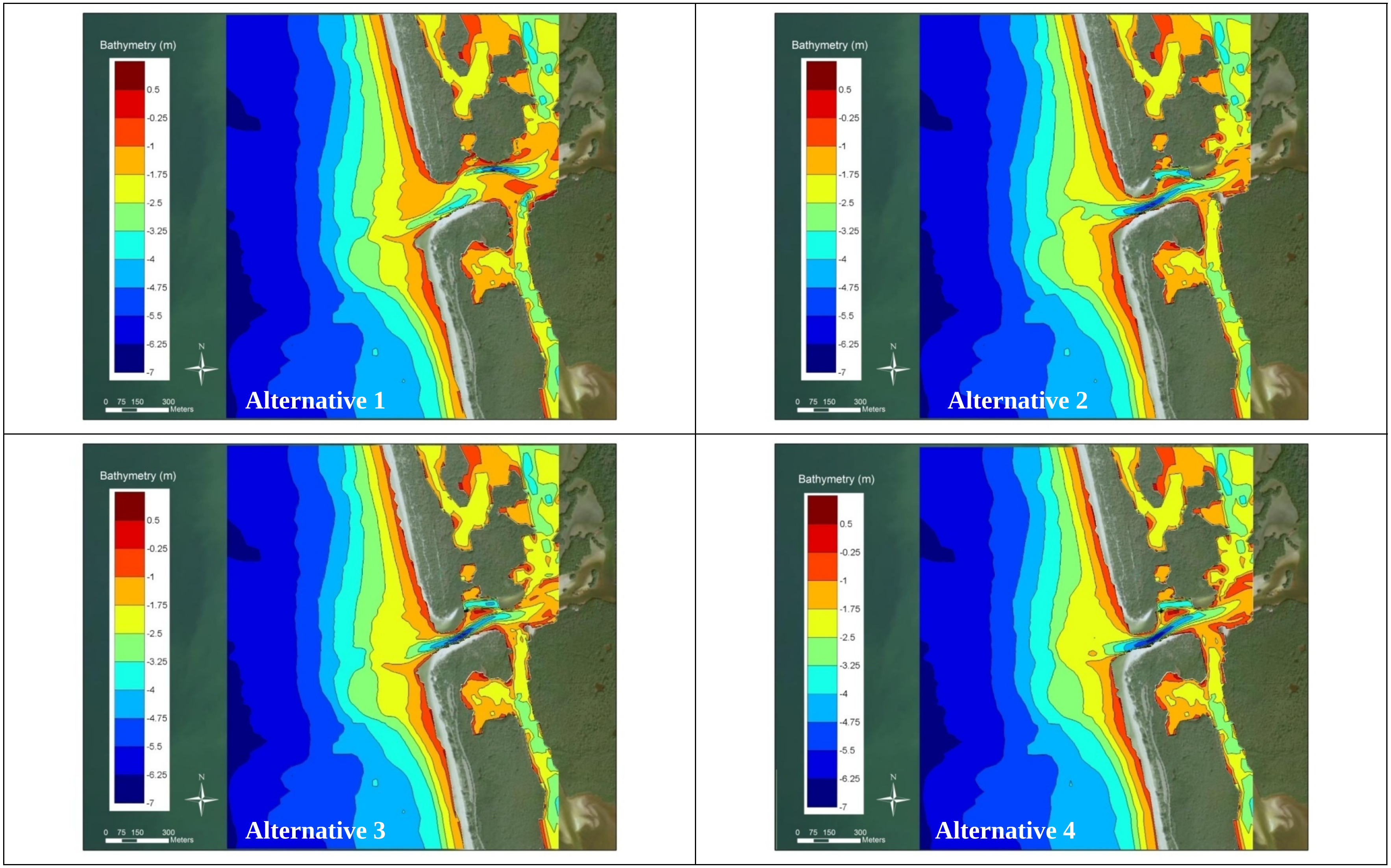


Figure 57. Bathymetry simulated after two years, Alternatives 1 to 4.

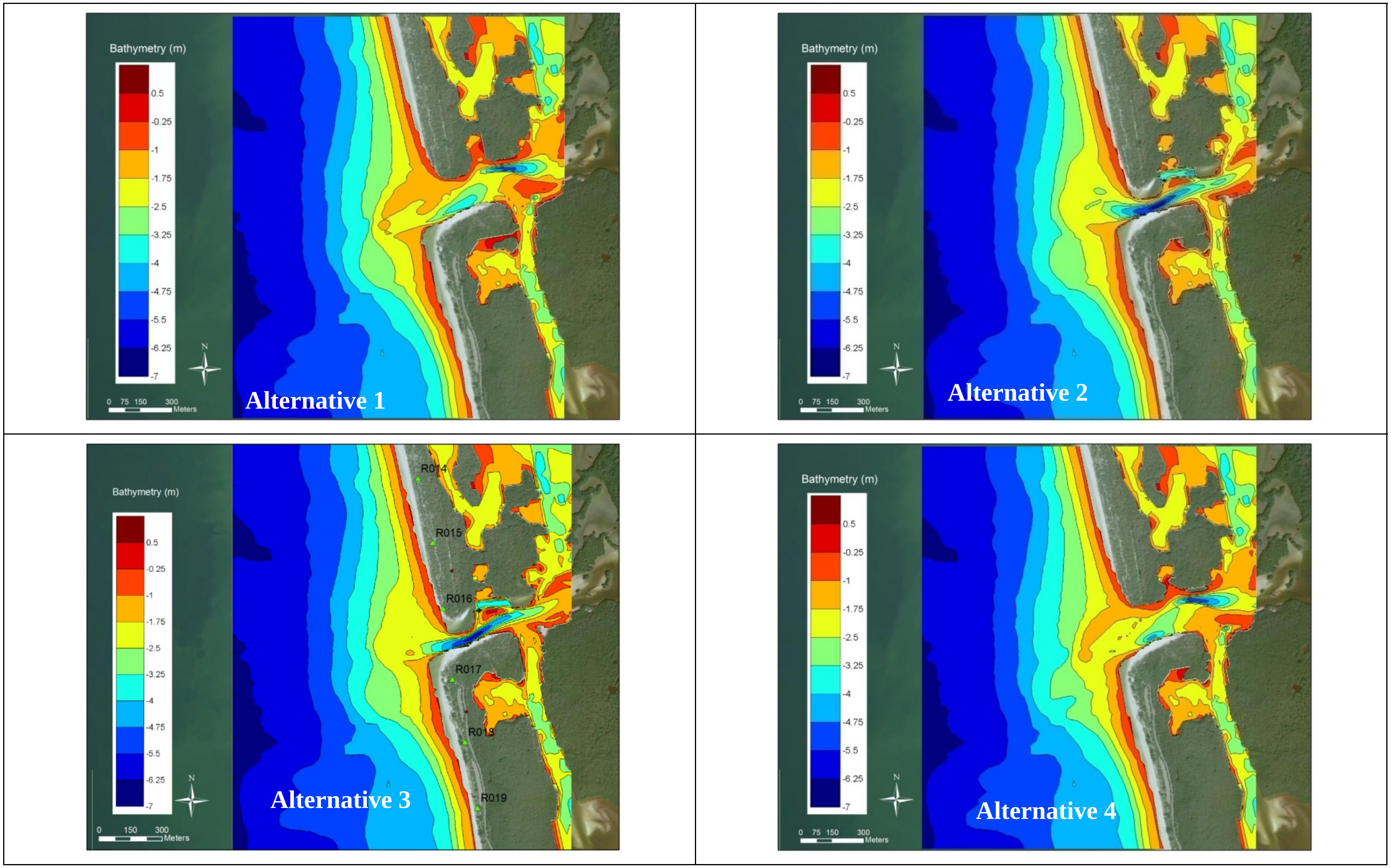


Figure 58. Bathymetry simulated after four years, Alternatives 1 to 4.

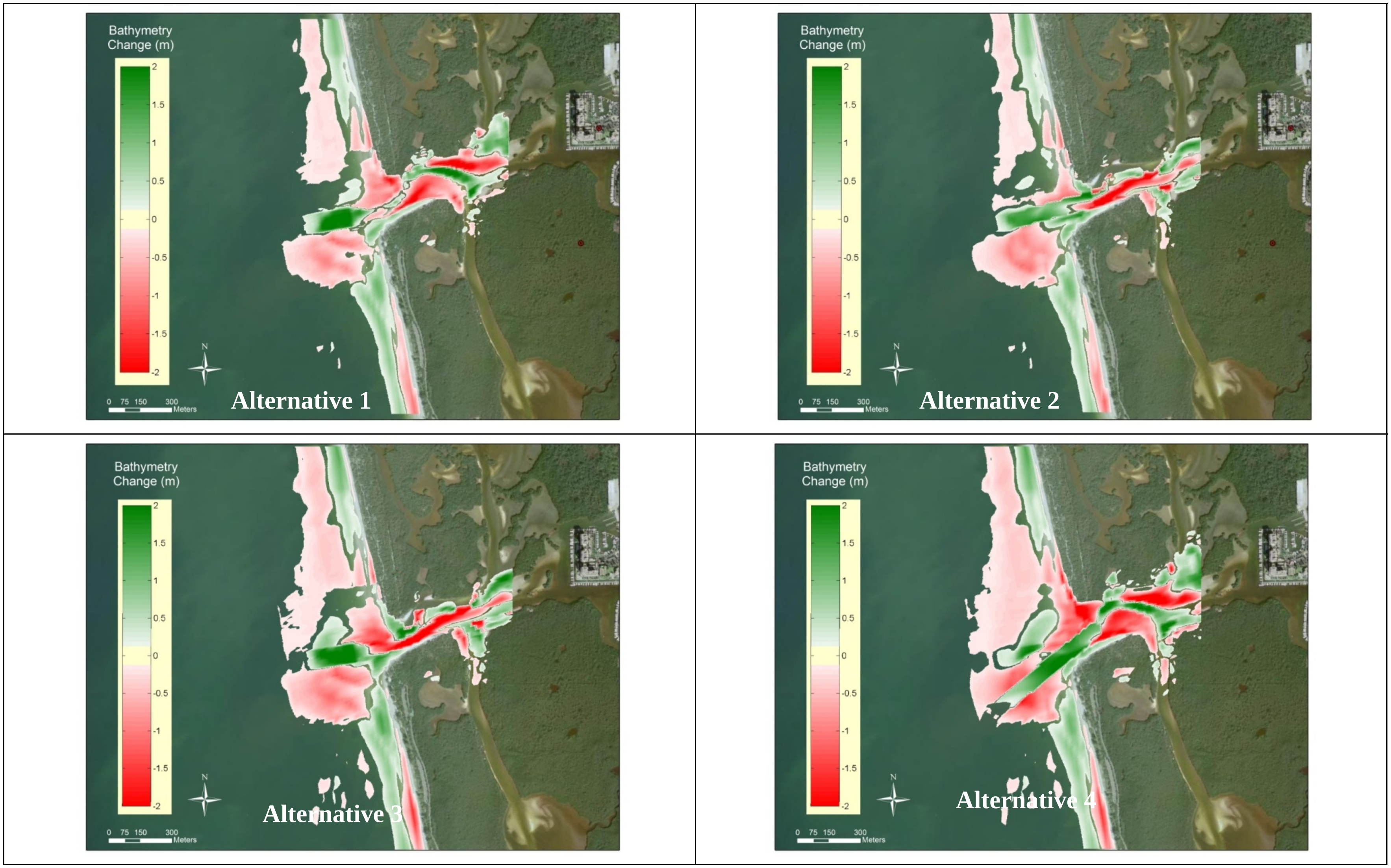


Figure 59. Bathymetry change simulated after four years, Alternatives 1 to 4.

Table 4. Decision matrix elaborated based on the results of the four years simulation.

No.	**Alternative Description**	**Outer Channel Controlling depth**	**Outer Channel Infill Rate**	**Channel Stability**	**North Shoreline (R15-R16)**	**South Shoreline (R17-R18)**	**Limestone Removal**
		ft-NAVD	cy/yr				
Alt 1	Permitted channel	3 to 5	12,800	Poor	Erosional	Stable	No
Alt 2	200' Straight channel + flood shoal cut and sand dikes	4 to 7	8,600	Optimal	Slight accretion	Stable	Some
Alt 3	Permitted channel + flood shoal cut and sand dikes	4 to 7	12,800	Optimal	Slight accretion	Stable	No
Alt 4	200' Re-aligned channel	4 to 7	14,400	Intermediate	Highly erosional	Stable	Yes

Simulation of Beach Fill Placement between R13 and R15

The baseline scenario, Alternative 1, was simulated in combination with placement of 45,800 m^3 (60,000 cy) of material between R13 and R15 to evaluate the effects of fill placement at Barefoot Beach on inlet sedimentation rates. It was assumed a fill grain size of 0.4 mm versus the 0.35 mm existing since the fill sediments are likely to come from the ebb shoal and channel dredging, where coarser material is found. The initial and final bathymetries simulated are shown in Figure 60. The figure shows that final morphological configuration of the inlet is similar to the simulation without the beach fill, with the fill spread slightly in both alongshore directions.

Placement of 45,800 m^3 (60,000 cy) of material between R13 and R15 caused a small increase in channel sedimentation volume for the simulation period (one year) from 21,640 m^3 (28,300 cy) to 23,000 m^3 (30,100 cy) due to beach fill placement. The increased sedimentation amount (1,360 m^3 or 1800 cy) is equivalent to about 6% of the placed material and considered insignificant.

A subtraction of the final predicted bathymetry for Alternative 1, without the beach fill, from the predicted bathymetry for Alternative 1, with the beach fill, is shown in Figure 64. The dark green area is the initial placement location; the light green areas indicated fill spreading locations. The fill spread laterally and cross-shore to the north and south of the placement location, with most of it spreading to the north (Figure 61). This fill placement location directly benefited Barefoot Beach without significant negative effects on channel maintenance.

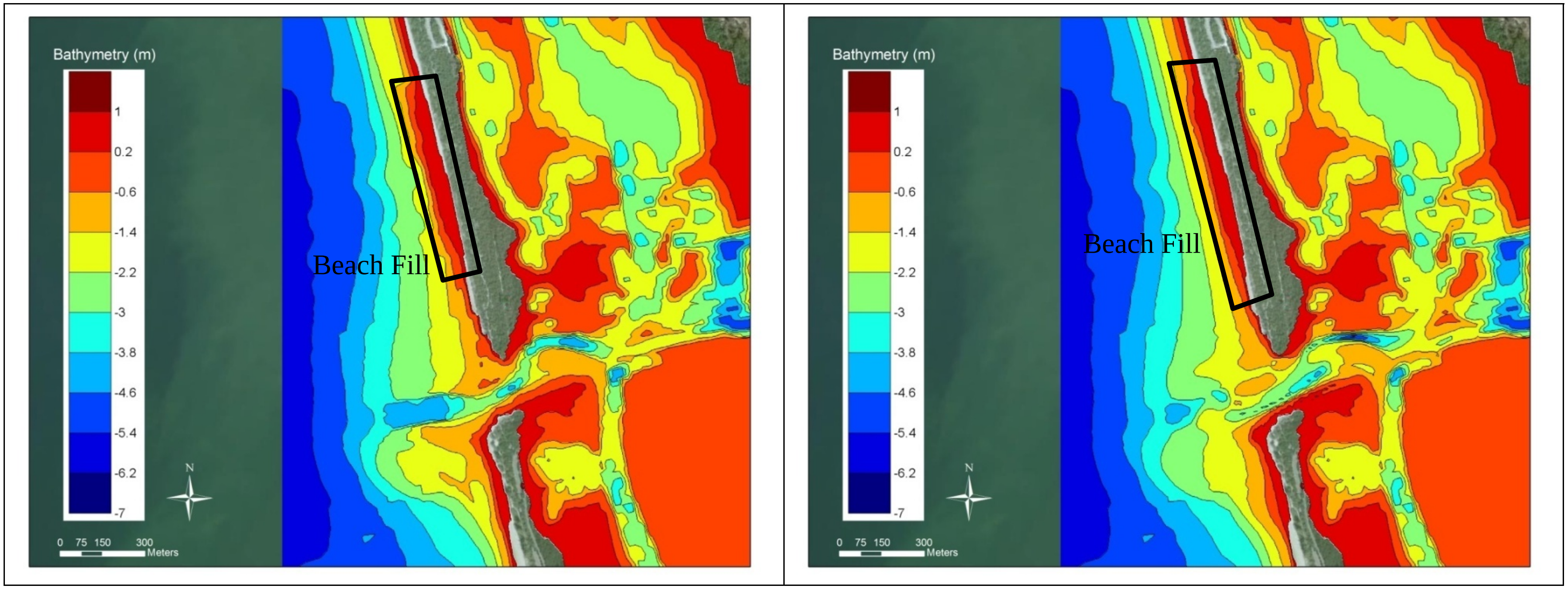


Figure 60. Start bathymetry (left) and simulated bathymetry after one year (right), simulation of Alternative 1 with beach fill placement between R13 and R15.

Figure 61. Relative bathymetry changes for the one-year simulation of Alternative 1, with the beach fill, in relation to the one-year simulation of Alternative 1, without the beach fill.

Hurricane Wave Climate

A two-year wave climate was developed based on the two very active years in terms of Hurricane and Tropical storm activity in Florida (2004 and 2005). Many hurricanes made landfall or passed through the Gulf during this time period, including Hurricanes Charlie, Frances, Ivan, and Jeanne in 2004 and Dennis, Katrina, Rita, Wilma in 2005.

Table 5 shows the two-year wave climate derived based on 2004 and 2005 wave data. This wave climate is much more energetic than the long-term wave climate shown in Table 1, with deepwater wave heights reaching 3.8 m (12.5 ft).

Results from the two year extreme-wave climate simulation are shown in Figure 62. The results indicate that, no matter the channel configuration, if a two year series of extreme storm events affects the study area, similar to what was observed in 2004-2005, navigation becomes impaired and emergency dredging for channel maintenance is required.

Table 5. Wave climate developed based on WavewatchIII data from two active years in terms of hurricane and tropical storm activity (2004 and 2005).

Hs	Tp	MeanDir	DirSpr	Percentage of Record (%)	Morfac
2.2	9.4	266	25	0.4%	1.3
0.2	2	257	10	11.0%	39.0
0.5	5.1	292	4	13.5%	47.9
1.3	6.6	290	15	3.9%	27.7
2.4	7.9	287	25	0.6%	4.4
0.2	2	257	10	11.0%	39.0
3.8	8.4	295	25	0.3%	2.2
0.4	4.1	319	4	8.4%	29.8
1.3	5	319	4	0.7%	4.9
2.3	5.7	317	4	0.1%	1.0
3.9	7	316	15	0.1%	1.0
0.4	4.2	196	4	3.4%	24.2
1.3	5.4	192	4	0.3%	2.4
2.5	8.6	198	25	0.1%	1.0
3.7	8.8	191	25	0.1%	1.0
0.2	2	257	10	11.0%	39.0
0.5	5	228	4	3.4%	24.7
1.4	7.7	230	15	1.3%	9.4
2.5	9.6	230	25	0.4%	3.2
0.2	2	257	10	11.0%	39.0
0.4	5.4	265	4	5.9%	42.5
1.5	7.2	265	15	1.9%	13.6
3.7	12.7	229	25	0.1%	1.0
0.2	2	257	10	11.0%	39.0

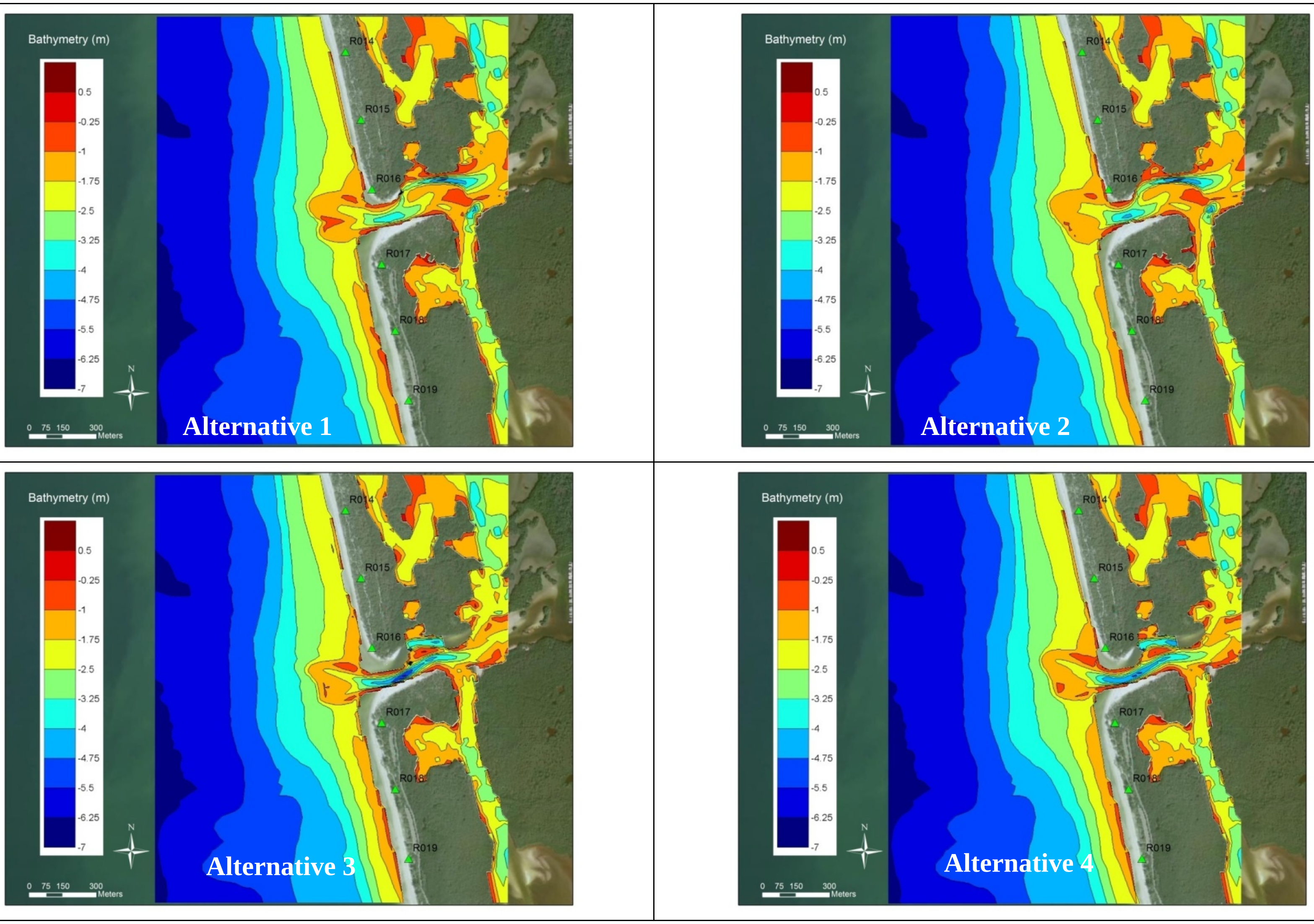


Figure 62. Start bathymetry (left) and simulated bathymetry after one year (right), simulation of Alternative 1 with beach fill placement between R13 and R15.

Results presented at the October 2008 meeting showed that, of all the alternatives tested to date, the only alternative that achieved the project goals of improved navigation and erosion mitigation consists of a straight channel with smaller dimensions than the current dredge template aided by interior dredging and temporary sand dikes. The re-designed Gulf access channel is 61 m (200 ft wide) versus 76 m (250 ft wide) of the current permitted template and 3.67 m MSL (12.6 ft NAVD or 11 ft MLW) deep, versus 4.28 m MSL (14.6 ft NAVD or 13 ft MLW) of the current permitted template. This re-dimensioned channel connects to the interior water body by a cut across the flood shoal. For this configuration to be efficient, the flows of the existing meander have to be blocked by temporary (training) sand dikes so a dual channel system is avoided.

Fill placement between R13 and R15 resulted in minimum effects on channel sedimentation and positive impacts at Barefoot Beach.

Hurricane conditions can severely impair navigation at Wiggins Pass and requires additional emergency maintenance for all channel designs.

Recommendations for the final phase of the study included optimizing the preferred alternative and simulating it for 1 year, 2 years and 4 years with fill placement between R13 and R15, refine the decision matrix alternative, and write the numerical modeling report with the final recommendations from CPE. Based on the results presented to date, the modeling review committee selected Alternative 2 for further refinement and final numerical model simulations.

SIMULATIONS OF THE SELECTED ALTERNATIVE

The selected alternative (Alternative 2 of the October 2008 meeting) was simulated for 1 year, 2 years, and 4 years. Simulations were conducted for the final alternative with and without beach fill placement between R13 and R15. The selected alternative was slightly redesigned by CPE to optimize hydraulic efficiency and minimize dredging volumes. The components of this alternative are shown in Figure 63. The dredge volumes associated with each new channel component are shown in Table 6. The location and volumes associated with the dikes and meander fill are conceptual, and warrant detailed design in a future phase.

Table 6 shows the total cut volume is 58,900 m^3 (77,100 cy). Fill volumes have not been estimated. Cut volumes may be modified by eliminating the dimensions of the Cocohatchee Channel and the south channel and reducing the dimensions of the interior channel across the flood shoal. The fill placed on Barefoot Beach should match the fill dredged from the ebb shoal channel, while the interior dredge volumes will be used to establish the north flood shoal, fill in the flood channel and build dikes. These volumetric and design modifications should be evaluated during final channel design and permitting phases.

Table 6. Dimensions of the different components of the final alternative simulated.

Feature	Width (start, ft)	Width (end)	Elevation (ft MLW)	Elevation (ft NAVD)	Volume (m^3)	Volume (cy)
Gulf Channel	240	160	-11	-12.68	40,300	52,600
Interior Channel (flood shoal cut)	160	120	-6	-7.68	14,600	19,100
Cocohatchee connection	120	120	-6	-7.68	3,300	4,400
Turkey Bay Connection	100	100	-6	-7.68	800	1,000
Total Cut Volume	N/A	N/A	N/A	N/A	**58,900**	**77,100**

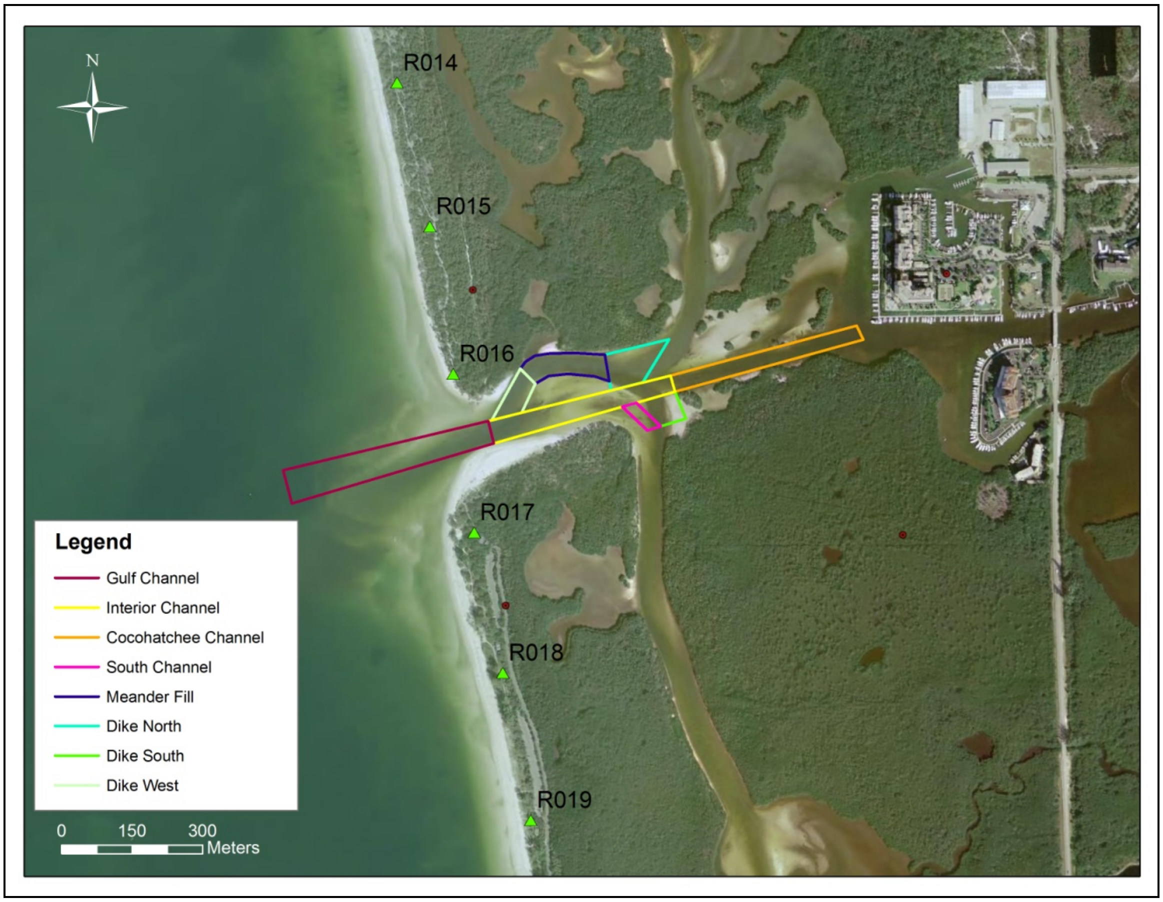


Figure 63. Components of the final alternative overlaying an aerial photograph of Wiggins Pass.

Results of this redesigned selected alternative (former Alternative 2) are very similar to the results of the original Alternative 2, described in the previous section of this report (October 2008 meeting). This alternative still achieved improved navigation conditions, positively impacting the south end of Barefoot Beach, and did not have negative impacts to the shorelines of Delnor Wiggins State Park. It will also reduce long term impacts based on average annual dredge quantities and avoid environmental impacts based on the results described in Appendix I.

Controlling depths after implementing the alternative are about 2.5 m MSL (8.5 ft NAVD), with a channel width of about 55 m (180 ft) at its narrowest Gulf section (Figure 67). Barefoot Beach is slightly accretional between R16 to the south end of Barefoot Island and slightly erosional between R16 and R15. However, the magnitude of erosion is less than the previous simulations conducted with the existing dredge permit template (see Figures 53, 56 and 59 of Alternative 2). The shoreline to the south of Wiggins Pass is accretional between the north end of Delnor Wiggins State Park to R18 and erosional between R18 and R19 (Figure 68). The shoreline change patterns for this alternative are similar to those of the baseline simulation (see Figure 56), showing no positive or negative effects at the Delnor Wiggins State Park Beach.

Two year simulations show that controlling depths still ranged from 2.5 m MSL (8 ft NAVD), but the narrowest section of the Gulf channel is about 18 m (60 ft) and can be hard to find by boaters (Figure 64). Depths around the narrowest section range were around 1.75 m MSL (6 ft NAVD). Shoreline change patterns are similar to the one-year simulations.

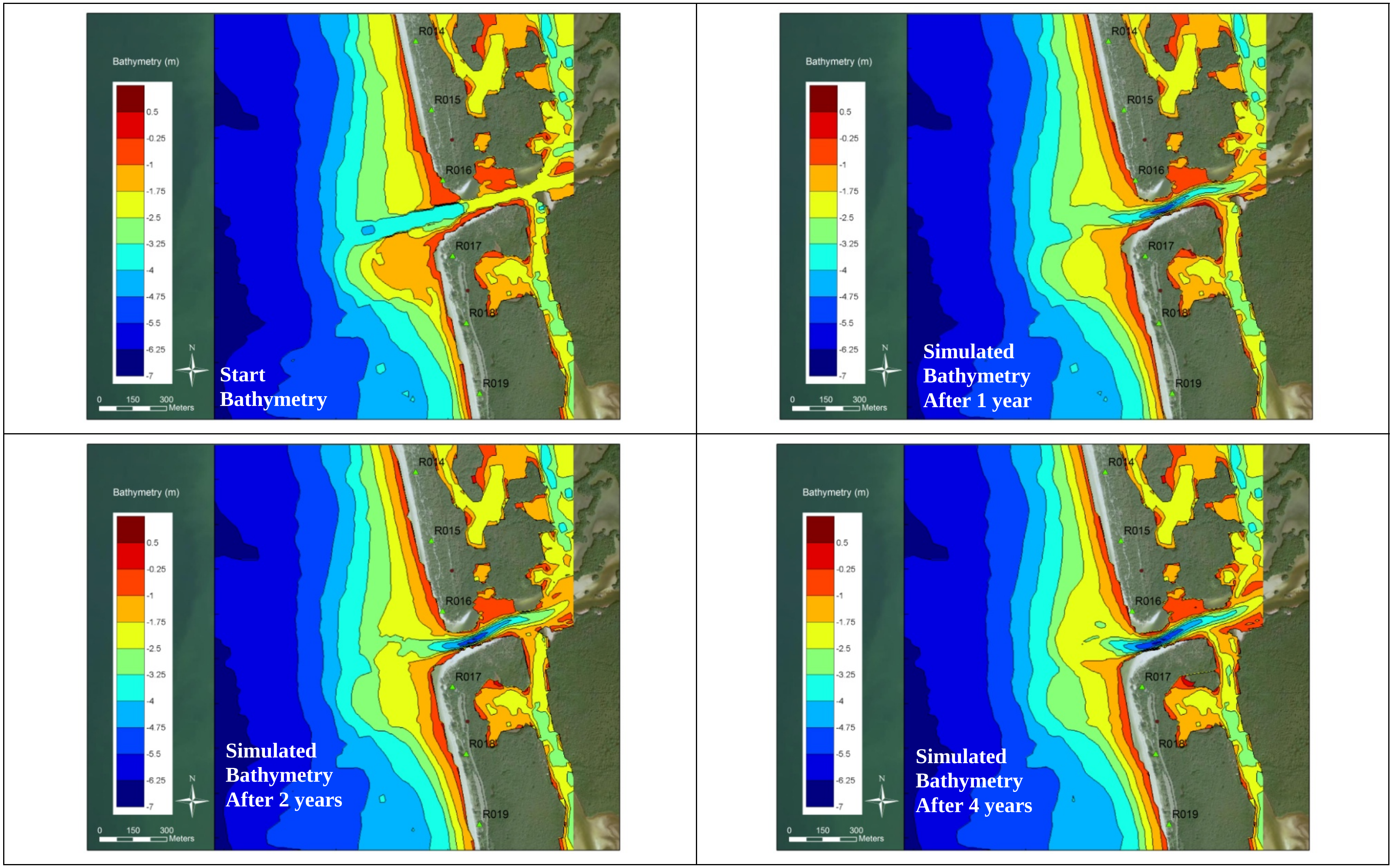
Four year simulations show controlling depths ranging around 1.75 m MSL (6 ft NAVD) with a straight channel orientation. Shoreline change patterns for the four year simulations are similar to the two-year simulations but of greater magnitude (Figure 65).

It is worth noting that the NAVD datum at Wiggins Pass is 0.69 m (2.28 ft) above MLLW. A channel depth of 6 ft NAVD is equivalent to a depth of 3.72 ft during MLLW (low spring tide).

Simulations with the beach fill show similar channel behavior as the simulations without beach fill (see Figures 66 and 67). The color scale of the figures with fill were slightly modified so that elevations above 0.5 m are also shown. Because of this change in the color-scale, it is also possible to visualize the sand dikes (dark red shades in the interior channel in the top left of Figure 66). Figure 67 shows erosion of the fill between R15 and R13, deposition at R12 due to fill spreading to the north, and almost complete reduction of the erosion between R16 and R15 due to fill spreading to the south. Beach fill placement benefited Barefoot Beach mitigating the erosion observed between R16 and R15 (Figures 67 and 68). Figure 68 shows that one year after fill placement, the material placed originally between R13 and R15 had spread alongshore reaching R11 to the north and R16 to the south. Most of the fill alongshore spreading was to the north, but a small portion spread to the south mitigating for erosion previously observed between R15 and

R16. These results indicate that fill placement between R13 and R15 is adequate for erosion mitigation at Barefoot Beach without negatively affecting the shoreline rates of the navigation channel. It is worth reminding that in the future, some percentage of the fill material may need to be placed on the southern section of the Delnor Wiggins State Park, to mitigate for the erosion observed from profile monument R18 to the south.

Table 7 shows the sedimentation volumes for 1 year, 2 years, and 4 years, with and without beach fill. Annual sedimentation rates decrease exponentially with time. Most of the sedimentation occurs in the first two years after dredging, and there is little additional sedimentation from year 2 to year 4 (Table 7). With a stable channel orientation, a longer lasting and deeper path through the ebb shoal that boaters can locate is possible. The simulations with the beach fill indicate a small increase in sedimentation volume, generally less than a 5% increase, indicating that most of the fill placed between R13 and R15 is carried to the north or kept at Barefoot Beach. Maintenance dredging within the interior channels will be small to negligible once the initial cut is made, similar to the existing conditions; as long as some migration of the channel is tolerated.


Start Bathymetry

Simulated Bathymetry After 1 year

Simulated Bathymetry After 2 years

Simulated Bathymetry After 4 years

Figure 64. Start bathymetry (top left), simulated bathymetry after one year (top right), simulated bathymetry after two years (bottom left) and simulated bathymetry after four years (bottom right), selected alternative without beach fill.

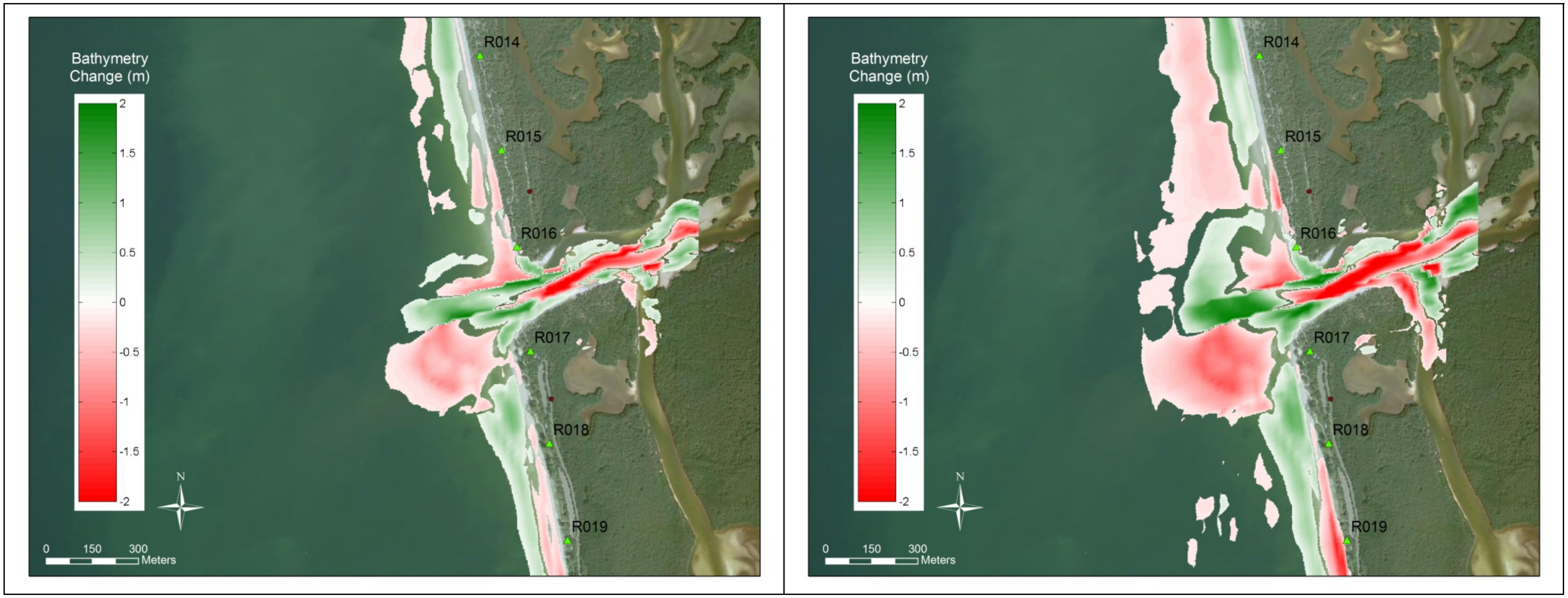


Figure 65. Bathymetry change for one year (left) and four years (right), selected alternative without beach fill.

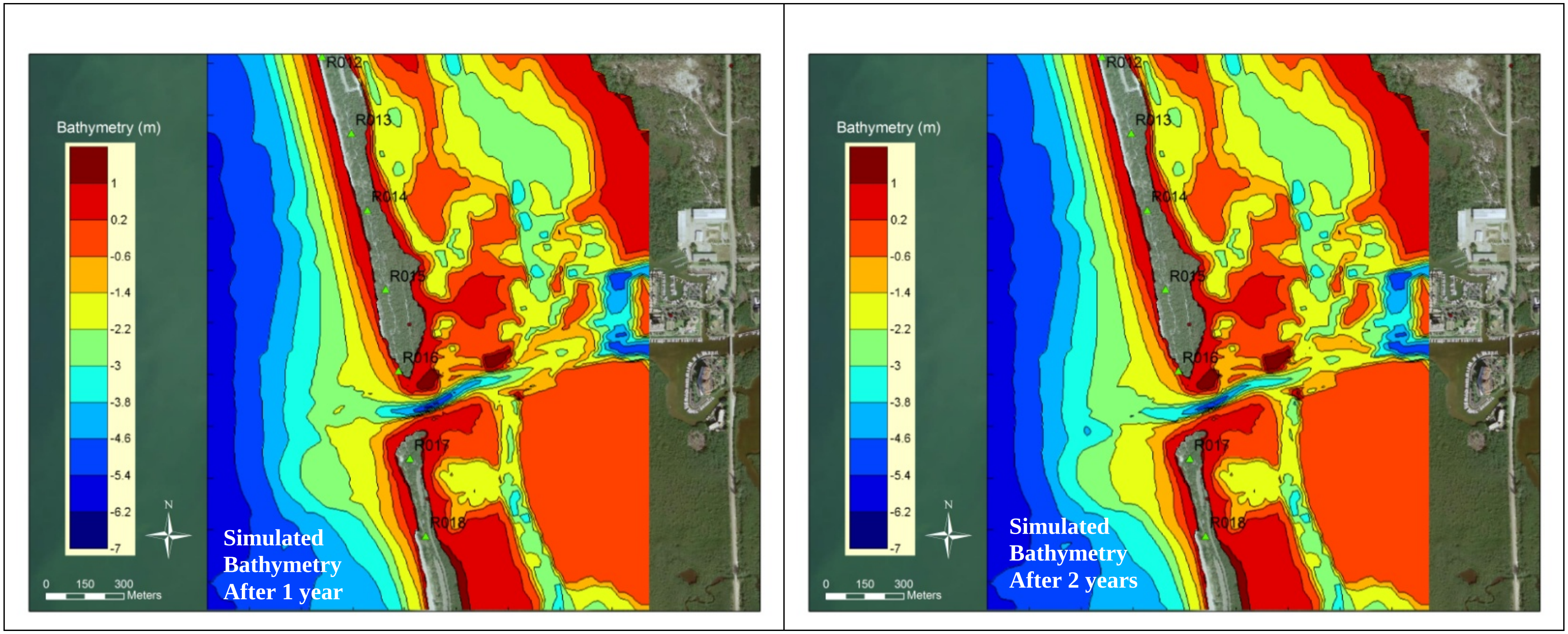


Figure 66. Bathymetry after one year (top left), simulated bathymetry after two years (top left), selected alternative with beach fill.

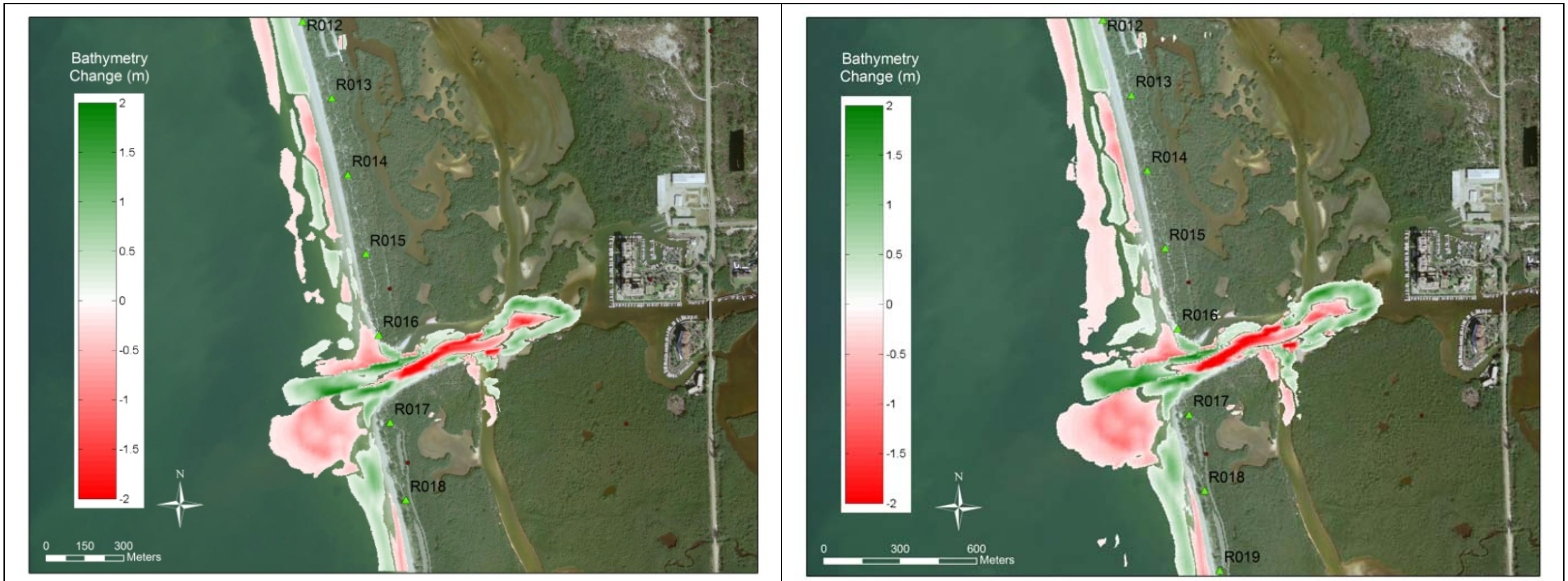


Figure 67. Bathymetry change for one year (left) and two years (right), selected alternative with beach fill.

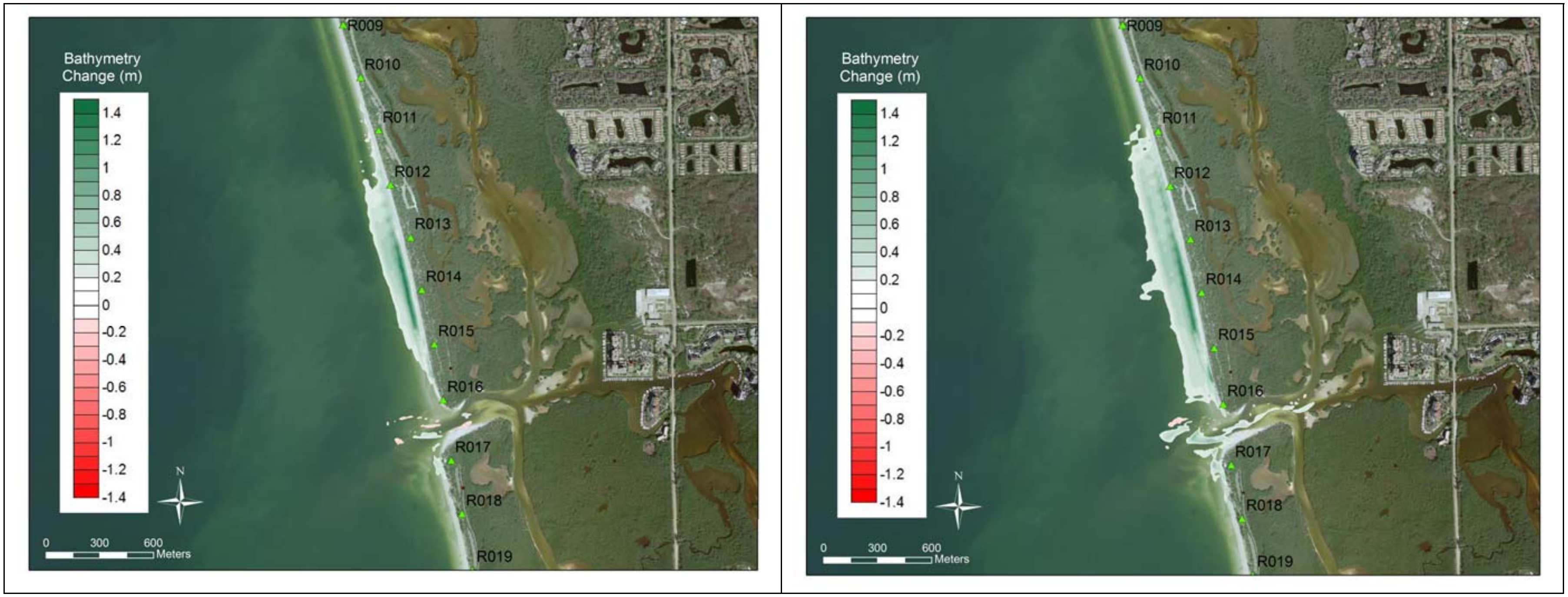


Figure 68. Relative changes for one year and two years (simulation with beach fill minus simulation without beach fill).

Table 7. Decision matrix for the selected alternative

Alternatives		Gulf Channel Controlling Depth	Gulf Channel Width at Controlling Depth	Gulf Channel Sedimentation	Gulf Channel Sed. Rate	Channel Stability	North Shoreline Effects*	South Shoreline Effects*	Limestone Removal**
	Description	ft-NAVD	Ft	cy	cy/yr		**(R15-R16)**	**(R17-R18)**	
Selected Alternative	One year, without beach fill	7 to 10	180	26,000	26,000	Optimal	Slightly Positive	Neutral	No
Selected Alternative	Two years, without beach fill	7 to 10	50	34,300	17,200	Good	Slightly Positive	Neutral	No
Selected Alternative	Four years, without beach fill	~6	N/A	40,500	10,100	Good	Slightly Positive	Neutral	No
Selected Alternative	One year, with beach fill	7 to 10	180	26,600	26,600	Optimal	Positive	Neutral	No
Selected Alternative	Two years, with beach fill	7 to 10	50	36,600	18,300	Good	Positive	Neutral	No

*Effects in relation to baseline simulation (current dredging template)

**Gulf Channel follows same pathway as current permitted channel however it has smaller dimensions therefore limestone removal is not expected. This conclusion however relies on the acquisition of jet probes or vibracores along the flood shoal cut.

CONCLUSIONS AND RECOMMENDATIONS

A comprehensive field measurement and numerical modeling study was conducted to evaluate non-structural engineering solutions to improve navigation conditions at Wiggins Pass and mitigate erosion at the adjacent shoreline north of the navigation channel, Barefoot Beach.

Erosion problems at Barefoot Beach are caused by the same processes of other inlets along the Florida coast. A dredged navigation channel traps sediment that otherwise would bypass to this beach and attach to it in the form of a bypassing bar. Sediment transport potential increases from the R15 to the north, suggesting erosion along this area. A sediment reversal at the southern end of Barefoot Beach, caused by the ebb shoal, also causes additional sediment loss to the navigation channel. Dredge disposal practices also contributed to the sediment deficit in this region. To solve this erosion problems there are three possible solutions: 1. Beach fill placement, 2. Erosion Control Structures and 3. Allow the channel to fill in and re-form a bypassing bar. Solution #3 is in conflict with navigation interests in the area, solution number #2 in conflict with beach user and environmental groups, therefore only solution #1 is feasible. The numerical model study conducted here indicated that fill can be placed closer to the inlet between R13 and R15 without negative effects on channel maintenance.

Several channel design alternatives were tested to evaluate maintenance of controlling depths, channel stability, and sedimentation rates. Of all the alternatives tested the current permitted dredge template was the least efficient configuration, therefore modifications to the current channel design template are required. The reasons for the poor performance of the existing dredge template are briefly described in the next paragraph.

Because of the large flood shoal development over the last thirty years the navigation channel at Wiggins Pass meanders significantly before reaching the Gulf Coast. The channel meandering causes strong flows against the inlet shoreline of Barefoot Beach, causing erosion problems at this location. The ebb jet has a southwesterly direction, causing the natural channel to realign to the southwest after dredging events. This leads to an abandonment of the outer section of the dredged channel as the channel migrates southwest, similar to what was observed in the bathymetric survey of June 2008. This takes the form of a shoal ridge across the channel, which inhibits navigation. A similar phenomenon was observed after previous dredging events as reported by local boaters.

The abandoned part of the dredged channel becomes a dredge pit in the nearshore, and it shoals at an accelerated rate. The natural channel, that realigns to the southwest after dredging, on the other hand, is curved, and has many shallows shoals, becoming dangerous to navigation. Most times local boaters have to navigate through the realigned channel, and find the right path through the shoal to reach the open Gulf. This is a dangerous maneuver that can lead to accidents and groundings. In addition to these navigation hazards, there is a marginal flood channel that wraps around the southern end of Barefoot Beach and eroding sediments from this area, which are lost into the navigation channel.

Potential modifications to the plan were tested using the numerical model described in this report, combined with engineering judgment and expertise built up by previous consultants working in this area.

It was found that a dredge template that follows the natural ebb jet, to the southwest, is more efficient than the current dredging template, however this channel configuration increase erosion of adjacent shorelines. If a channel orientation similar to the current plan is to be continued, modifications of the interior inlet morphology are required so the ebb jet comes out of the inlet throat in a shore normal angle, following the existing dredging template. Improvements can be achieved by re-designing the dredging template to smaller dimensions, dredging through the flood shoal, and plugging the existing channel meander with temporary sand dikes to avoid a dual channel system. This configuration leads to increased inlet efficiency and some mitigation of erosion at Barefoot Beach interior shoreline and southern end. In addition to these inlet modifications it is recommended to place fill material closer to the inlet on Barefoot Beach, between profile monuments R13 and R15. Because of the wave shadow effect of offshore morphological features there is little sediment transport potential towards Wiggins Pass from the north, the current sediment placement location (vicinity R12) does not help the erosion problem at Barefoot Beach, since the dredged sediments move to the north after disposed form this region.

Before implementation of the alternative selected it must be subjected to a final design phase, which will include selected vibracores; and detailed design of the channel, dikes, and disposal locations and dimensions while avoiding environmental resources. Implementation of the alternatives will depend on the permitting process with FDEP and will include not only the physical aspects, focused here, but also environmental and geotechnical aspects. If full implementation of the selected alternative is not feasible due to permitting concerns, than some components of this alternative, like re-dimensioning of the dredge template and sediment placement closer to the inlet would readily improve the situation.

Traditionally, coastal structures (jetties, groins and breakwaters) are used to solve this type of problem. It is recommended to implement the proposed soft engineering solution (re-engineer the channel according to selected alternative), and monitor its performance through a few dredging cycles. Incremental refinements to the plan should be made based monitoring to address actual performance and better meet the objectives of improving navigation, reduction impacts to the adjacent beach while maintaining environmental and cost efficiency objectives.

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