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## **APPENDIX B – COASTAL ENGINEERING APPENDIX**

### **GENERAL REEVALUATION REPORT (GRR) AND SUPPLEMENTAL ENVIRONMENTAL IMPACT STATEMENT (SEIS) FOR THE POPLAR ISLAND ENVIRONMENTAL RESTORATION PROJECT CHESAPEAKE BAY, TALBOT COUNTY, MARYLAND**

The hydraulic and hydrodynamic modeling for the 575-acre northern lateral expansion of the Poplar Island Restoration Project (PIERP) included the evaluation of four major components:

1. impacts to current velocity and changes to flow patterns (USACE-ERDC, 2005a),
2. changes in the residence time of water in Poplar Harbor, the open-water embayment, and the wetland cells within the proposed northern lateral alignment (USACE-ERDC, 2005a),
3. impacts to shoreline wave heights (USACE-ERDC, 2005b), and
4. life cycle analysis for the dike design (USACE-ERDC, 2005c).

In addition, two previous studies (M&N, 2004; 2003) evaluated the potential for erosion and deposition of cohesive and non-cohesive sediment based on a generic northern alignment for the lateral expansion, as compared to the existing condition (the existing PIERP – 1,140 acres with 50 percent wetlands and 50 percent uplands). The results of each of these studies are summarized in the following sections.

#### **B.1 IMPACTS TO VELOCITY AND CHANGES IN FLOW PATTERN**

The purpose of this study was to assess the response of the surrounding area to the construction of the proposed northern lateral alignment using a numerical hydrodynamic circulation model. The ADvanced CIRCulation (ADCIRC) model was employed to assess the following issues:

- (1) Tidal elevation.
- (2) Current velocity.
- (3) Response of surrounding areas (Coaches Island, mainland) during a storm.
- (4) Changes in flow patterns resulting from the expanded island.
- (5) Feasibility for and functioning of a tidal gut between the existing and expanded island.

The full details and results of the hydrodynamic modeling are presented in *Poplar Island Expansion Flushing Study, Memorandum for the Record* (USACE-ERDC, 2005a). Methods and results of the study are summarized in the following sections.

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### B.1.1 Model Development

**ADCIRC Mesh.** The finite-element mesh used in this study contained approximately 73,500 computational nodes and 142,000 elements (USACE-ERDC, 2005a). Depths on the mesh were referenced to mean tide level (MTL). Figure B-1 shows the ADCIRC mesh for the area directly adjacent to the existing Poplar Island and the proposed expansion area.



**Figure B-1. ADCIRC mesh for the proposed northern lateral alignment**

*Source: (USACE-ERDC, 2005a)*

**Existing Condition.** The existing condition is defined by the most recent bathymetry information available as of December 2004. Bathymetry for the Poplar Island region was obtained from *Poplar Island Environmental Restoration Project, Morphological Modeling* (M&N, 2004). Bathymetry for the remainder of the numerical model domain was obtained from the National Geophysical Data Center (NGDC) Coastal Relief bathymetry database for coastal regions of the Atlantic Ocean.

**Option 1.** This option uses the same finite-element mesh as the existing condition, with modifications in bathymetry in the region of proposed northern lateral alignment. Bathymetry in the region of proposed northern lateral alignment was modified based on the heights of the dikes and depths of the proposed wetland cells. Minor changes were made to the depths in the wetland cells to allow each cell to more appropriately drain through the openings. Water enters the tidal gut formed between the existing island and proposed expansion from one opening on the south side of the expansion area (Figure B-2).

**Option 2.** This option is the same as Option 1, except that an second opening for the tidal gut was created in the outer dike area at the NW end of the tidal gut to allow flow from two ends into the wetland cell area (Figure B-2).



**Figure B-2. Bathymetry for Option 1 (left) and Option 2 (right)**

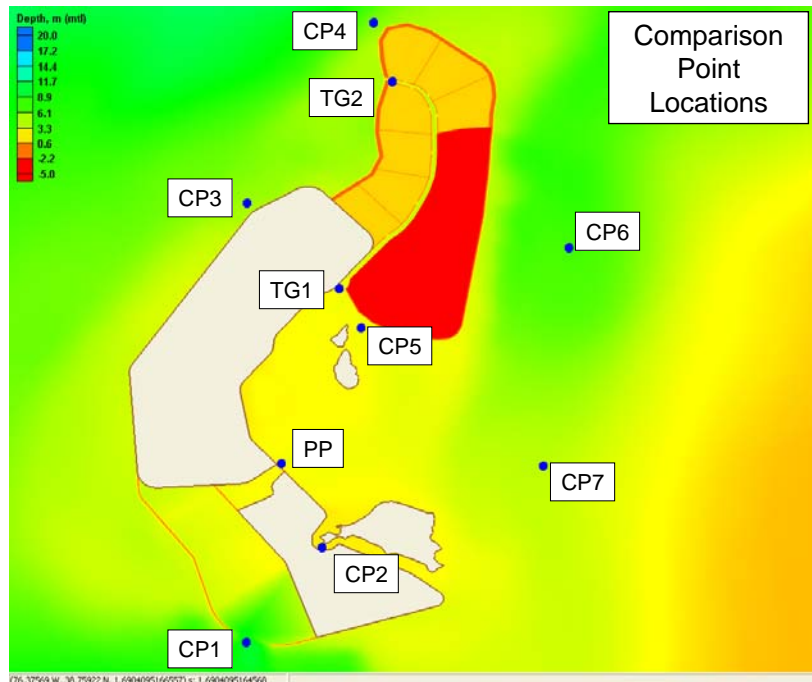
*Source: (USACE-ERDC, 2005a)*

**Definition of simulated events.** Two types of circulation runs – a validation run and a storm run - were conducted for each of the three options (existing condition, one opening in the tidal gut, and two openings in the tidal gut). Initially, the ADCIRC mesh was validated with a 31-day run. For the storm run, Coastal and Hydraulics Laboratory (CHL) staff members chose to run one of a suite of hurricanes developed in a storm-surge study that covered the same domain (Lin et al., 2005). Hurricane Isabel (September 2003) was chosen for this study following discussion between CHL and USACE-Baltimore District personnel, because the intensity of the wind and storm path.

**Validation Process.** The ADCIRC model was validated using data from a gauge placed at the personnel pier in Poplar Harbor from between May 17, 2004 and June 16, 2004. The ADCIRC model was run for the same time period, so a direct comparison could be made. Details of the validation process are presented in USACE-ERDC, 2005a.

**Definition of Comparison Locations for Validation and Storm Event Periods.** Ten locations were selected to compare the existing condition with Options 1 and 2. The “TG” prefix denotes comparison locations in the proposed tidal gut. “CP” denotes other comparison points except the location corresponding to the personnel pier gauge location, designated “PP.” The comparison points are listed and described below (Figure B-3):

1. CP1 – South of Poplar Island
2. CP2 – Southwest Corner of Coaches Island
3. CP3 – Northwest Corner of Poplar Island
4. CP4 – North of Expansion Area
5. CP5 – Area between Expansion Area And Jefferson Island
6. CP6 – Navigation Channel between Expansion Area and Mainland
7. CP7 – Navigation Channel between Mainland and Southeast of Expansion Area
8. TG1 – Tidal Gut Entrance
9. TG2 – North End of Tidal Gut
10. PP – Personnel Pier Gauge Location



**Figure B-3. Points used for comparison of the model results**

*Source: (USACE-ERDC, 2005a)*

### **B.1.2 Water Level and Current Speed Results**

**Water Level Comparisons.** Water levels calculated for the existing condition were compared to those from Options 1 and 2. No significant changes were noted at most comparison locations. Location TG2 showed a slight change in phase for the comparison of Option 1 to the existing condition, which was attributed to its location at the only opening of the tidal gut for Option 1.

**Current Speed Comparisons.** Current speeds calculated for the existing condition were compared to those for Option 1 and Option 2. The only significant changes occurred at CP4,

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TG1, and TG2, as described below. These changes were attributed to the introduction of the new landmass of the proposed lateral expansion, thereby increasing friction in the shallower water and changing current speed and direction.

- a. Location CP4 demonstrated a maximum increase in current speed for both alternatives of approximately 0.11 m/sec from 0.15 m/sec to 0.26 m/sec.
- b. At TG1, the maximum increase in current speed for Option 1 was approximately 0.20 m/sec, from 0.01 to 0.21 m/sec. The increase for Option 2 was 0.12 m/sec, from 0.01 to 0.13 m/sec.
- c. At TG2, there was a decrease in maximum current speed for Option 1 by approximately 0.15 m/sec, from 0.16 to 0.01 m/sec. For Option 2, there was an increase in maximum current speed of approximately 0.27 m/sec, from 0.16 to 0.43 m/sec. The TG2 comparison point is located where there is relatively little flow until the opening of a dike in Option 2. This location explains the reduction of current speed with Option 1 and the increase of current speed with Option 2.

**Water Level Comparisons for a Storm Event.** Water level comparisons of the existing condition to those of Option 1 and Option 2 were completed for the storm event, Hurricane Isabel. Slight water level changes were noted for the PP, CP2, CP5, CP7, TG1 and TG2 comparison locations, which were attributed to location of these comparison points on the eastern side of Poplar Island and either inside or south of the proposed northern lateral expansion. The land mass added in Options 1 and 2 acts as a barrier and promotes retention of the storm surge until the peak of the storm passes and water level starts to lower. The largest increase in water level was 0.15 m over the existing condition for Option 1 at TG2 at the peak of the storm surge. The increase can be attributed to existence of a barrier at the end of the tidal gut in Option 1.

**Current Speed Comparisons for a Storm Event.** Current speed comparisons of the existing condition to Options 1 and 2 were conducted for the storm event, Hurricane Isabel. At the PP, CP1, CP3, and CP4 locations, there were only minor reductions of current speed, on the order of 0.04 to 0.10 m/sec. Four comparison points exhibited notable change in current speed as a direct result of circulation changes induced by introduction of a land mass in the expansion area:

- a. At CP2, which is positioned at the southwest corner of Coaches Island, it was observed for Options 1 and 2 that there was a 0.45 m/sec decrease in calculated current speed during the highest intensity of the storm.
- b. At CP5 and CP6, a decrease in current speed of 0.40 m/sec was observed for both options during the time of highest storm surge, while at the CP7 location there was a slight decrease in speed as the surge was rising and a slight increase as the surge was receding.

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- c. The largest change in current speed was shown to occur for TG1, located at the tidal gut entrance. During the highest water levels, there was an increase of 0.5 m/sec. For the other tidal gut comparison location, TG2, there was a 0.20 m/sec decrease in current speed. This current speed changes occurred for both options.

### **B.1.3 Evaluation of the Open-Water Embayment**

Following the completion of the plan formulation process, a proposal from National Marine Fisheries Service (NMFS) and subsequent discussions with the Environmental Protection Agency (USEPA), Fish and Wildlife Service (USFWS), Maryland Department of Natural Resources (MDNR), and Maryland Department of the Environment (MDE) led to the development and evaluation of an open-water embayment that could be incorporated into a northern lateral alignment.

In the NMFS proposal, the footprint of the northern lateral alignment was the same as those proposed by USACE, but approximately 130 acres of wetland located on the western side of the lateral expansion was designated as an open-water embayment protected by segmented breakwaters and bordered by salt marsh and mudflats. USACE-Baltimore District modified the initial open-water embayment proposed by NMFS to enhance the hydraulic characteristics of the proposal and minimize the impact on the dredged material placement capacity of the lateral expansion. The open-water embayment alignment consists of a 575-acre (nominal area contained within the project footprint) lateral expansion to the north and northeast of the existing project, and a 5-ft vertical raising of the existing upland cells (Cells 2 and 6).

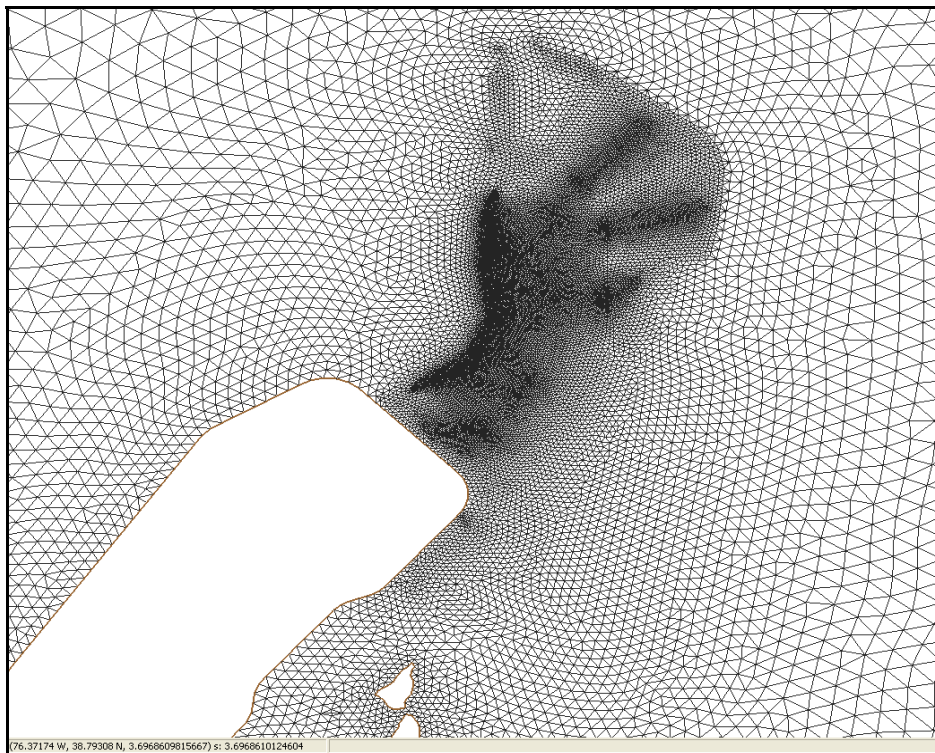
Based on the agency consultation to-date, the open-water embayment could potentially range between 80 to 140 acres in size. Concerns pertaining to specific components of the open-water embayment will be discussed and evaluated further in the next design phase of the project based on additional consultation with each resource agency (USFWS, NMFS, USEPA, MDNR, and MDE) and MPA (the non-Federal sponsor); results of additional hydrodynamic modeling studies; and additional design considerations. However, for the evaluation conducted in this document, the size of the open-water embayment within the northern lateral expansion was estimated to be 130 acres in size.

The alignment with the open-water embayment would be bounded by an armored sand dike similar in cross section to the dikes of the existing project. Approximately 3,400 ft of the western leg of the perimeter dike would be replaced by segmented breakwaters. The breakwater segments are currently proposed to be approximately 200 ft long and separated by about 50-ft, except for one or two larger openings of approximately 200 ft. The breakwater structures will consist of a core of 250-lb underlayer stone and two layers of stone armor having a mean weight of approximately 2,500 lb. A high-strength geotextile sheet would be placed on the Bay bottom to minimize loss of stone into soft or loose surface deposits. Hydraulic analyses will be performed in the next design phase of the project to optimize the breakwater crest height, stone size, and dimension of openings between segments.

The interior dikes of the alignment with the open-water embayment would be constructed to elevation +6.5 ft MLLW using sand and have minimal slope protection provided by established

vegetation. Those dikes forming the perimeter of the proposed open water embayment will be raised to a minimum crest elevation of +9 ft MLLW and will require slope protection to prevent erosion from the exposure along the embayment. The present design assumption is that adequate slope protection can be provided by a double layer of 350-lb stone placed on a bedding layer and a geotextile filter, similar to the protection proposed for the eastern slopes of the expansion dikes. Dike height and slope protection requirements will be refined as hydraulic analyses are completed.

**ADCIRC Mesh.** The finite-element mesh employed for the NMFS alternative differs from that employed in the originally investigated two alternatives, because substantial modification was required. High resolution was added to the ADCIRC mesh in the study area by incorporating drawings of the envisioned project provided by USACE-Baltimore District and CHL staff. This high resolution was used for the Existing Condition and for both storm and non-storm events to ensure one-to-one correspondence of points for comparisons. The mesh for the NMFS alternative differs only by bathymetric variations in the ADCIRC mesh. Depths on the mesh were referenced to mean tide level MTL. Figure B-4 shows the ADCIRC mesh for the area directly adjacent to the existing Poplar Island and the proposed expansion area.

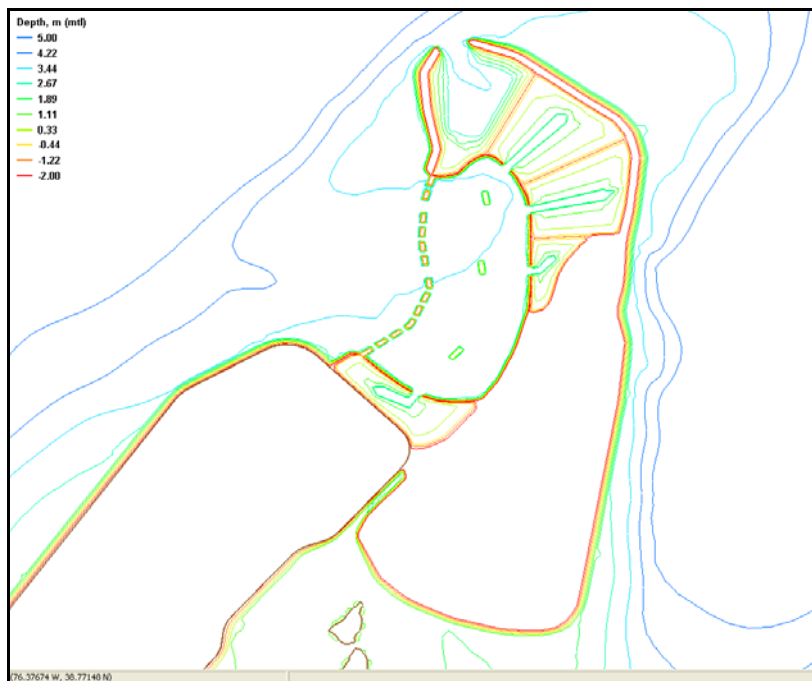


**Figure B-4. ADCIRC mesh used for the northern lateral alignment with the open-water embayment.** *Source: (USACE-ERDC, 2005a)*

**Existing Condition.** As with the analysis for Options 1 and 2, the existing condition is defined by the most recent bathymetry information available as of December 2004. Bathymetry for the Poplar Island region was obtained from *Poplar Island Environmental Restoration Project*,

*Morphological Modeling* (M&N, 2004). Bathymetry for the remainder of the numerical model domain was obtained from the National Geophysical Data Center (NGDC) Coastal Relief bathymetry database for coastal regions of the Atlantic Ocean.

**Open-Water Embayment Alignment.** Bathymetry defining this alternative was taken directly from the updated ADCIRC mesh, with only modifications in bathymetry in the region of the project expansion area. Bathymetry was entered using USACE-Baltimore guidance on the heights of the dikes, breakwaters, rock reefs and depths of the proposed wetland cells. Minor changes were made to the depths in the wetland cells to allow each cell to more appropriately drain through the openings. Figure B-5 shows bathymetry for the alignment with the open-water embayment.



**Figure B-5. Bathymetry for the alignment with the open-water embayment**

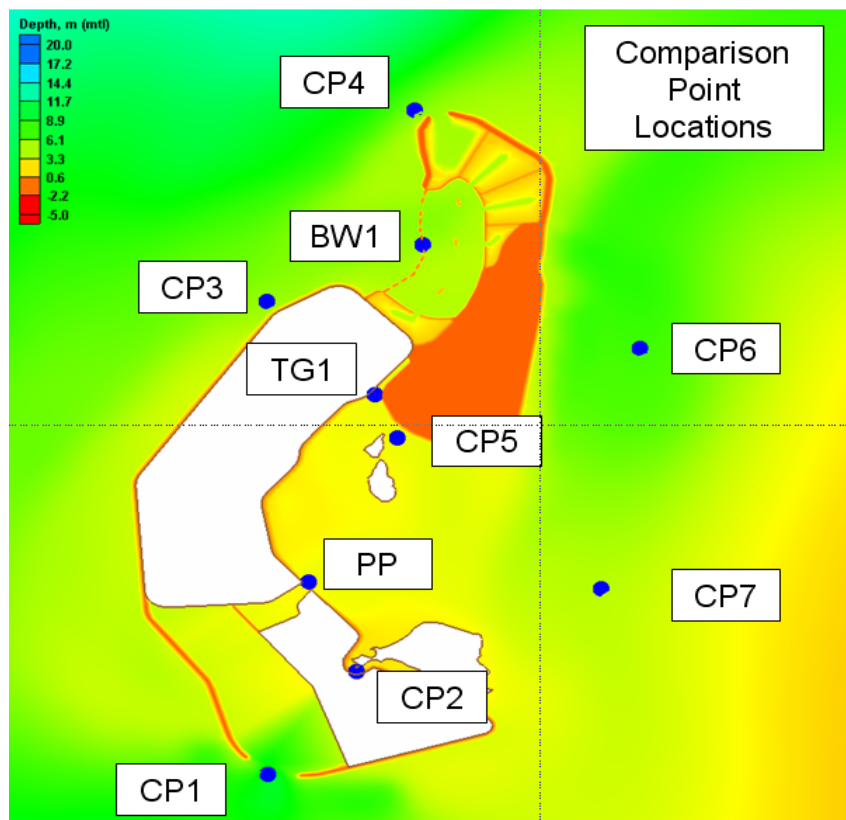
Source: (USACE-ERDC, 2005a)

**Definition of simulated events and validation process.** The simulated events for both the validation process and storm events used to evaluate the alignment with the open-water embayment were the same as those used for Options 1 and 2 (Section B.1.1). Details of the validation process are presented in USACE-ERDC, 2005a.

**Definition of Comparison Locations for Validation and Storm Event Periods.** Ten locations were selected to compare the existing condition with the open-water embayment (Figure B-6). The “TG” prefix denotes comparison location in the proposed tidal gut. “CP” denotes other comparison points except the location corresponding to the Personnel Pier gauge location, designated “PP” and one entrance to the embayment area through one of the wide breakwater gaps, designated “BW1.” The comparison points are listed and described below:



1. CP1 – South of Poplar Island
2. CP2 – Southwest Corner of Coaches Island
3. CP3 – Northwest Corner of Poplar Island
4. CP4 – North of Expansion Area
5. CP5 – Area between Expansion Area And Jefferson Island
6. CP6 – Navigation Channel between Expansion Area and Mainland
7. CP7 – Navigation Channel between Mainland and Southeast of Expansion Area
8. TG1 – Proposed Tidal Gut
9. BW1 – Breakwater Gap Entrance to Embayment Area
10. PP – Personnel Pier Gauge Location



**Figure B-6. Points used for evaluation of the alignment with the open-water embayment**

*Source: (USACE-ERDC, 2005a)*

**Water Level and Current Speed Results.** Water levels calculated for the existing condition were compared to those calculated for the alignment with the open-water embayment. There were no significant changes between the existing condition and the alignment with the open-water embayment for either the validation period or the storm event (USACE-ERDC, 2005a).

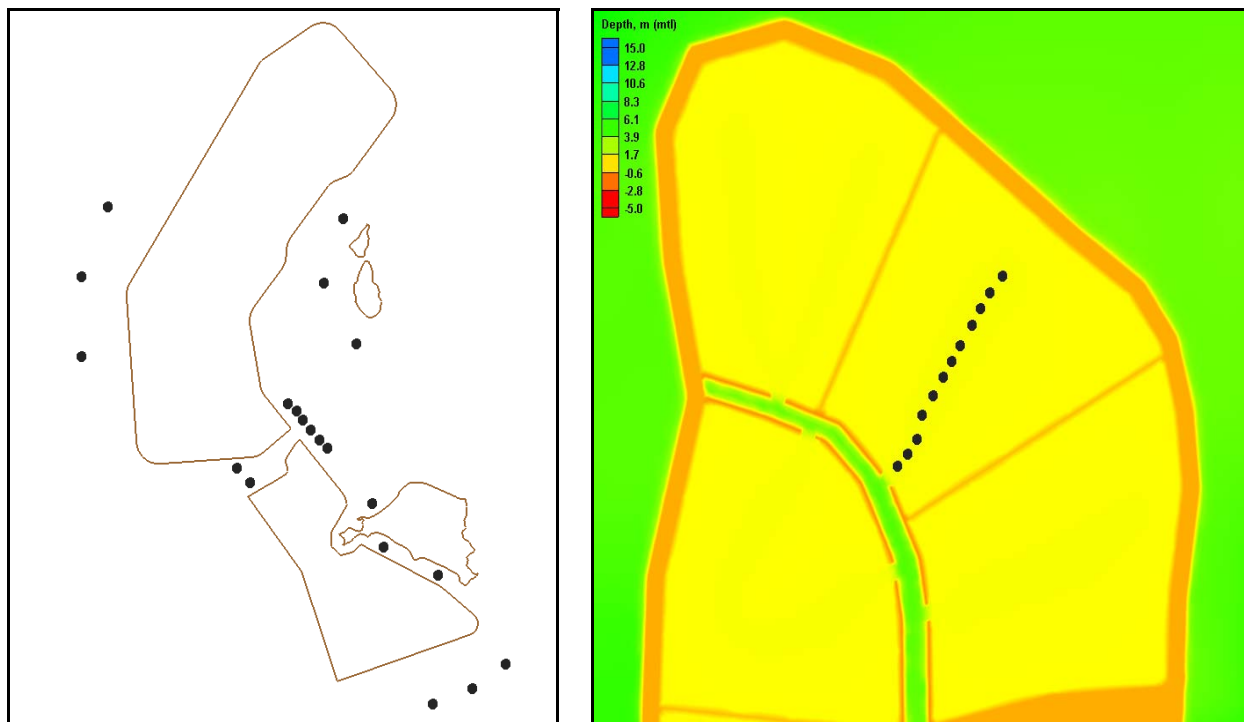
Current speeds calculated for the existing condition were compared to those for the alignment with the open-water embayment. Four of the ten locations (BW1, TG1, CP3 and CP5) had a

significant change in current speed. At these locations, there was a significant decrease in current speeds which can be attributed to the introduction of land in the expansion area, thereby increasing friction in the shallower water and changing current speed and direction (USACE-ERDC, 2005a).

## B.2 PARTICLE TRACKING MODEL AND RESIDENCE TIME IN POPLAR HARBOR, EXPANSION WETLAND CELLS, AND THE OPEN-WATER EMBAYMENT

### B.2.1 Particle Tracking Model

After the hydrodynamics for all events were simulated for the each option, the Particle Tracking Model (PTM) was run to evaluate residence times in Poplar Harbor, and to qualitatively visualize the flushing process in one of the proposed wetland cells within the northern lateral alignment (USACE-ERDC, 2005a). Point sources were defined in the Surface-water Modeling System (SMS) and were placed in two areas, (a) around Poplar Island and Harbor (Figure B-7), and (b) inside one of the wetland cells in the proposed northern lateral alignment (Figure B-7).



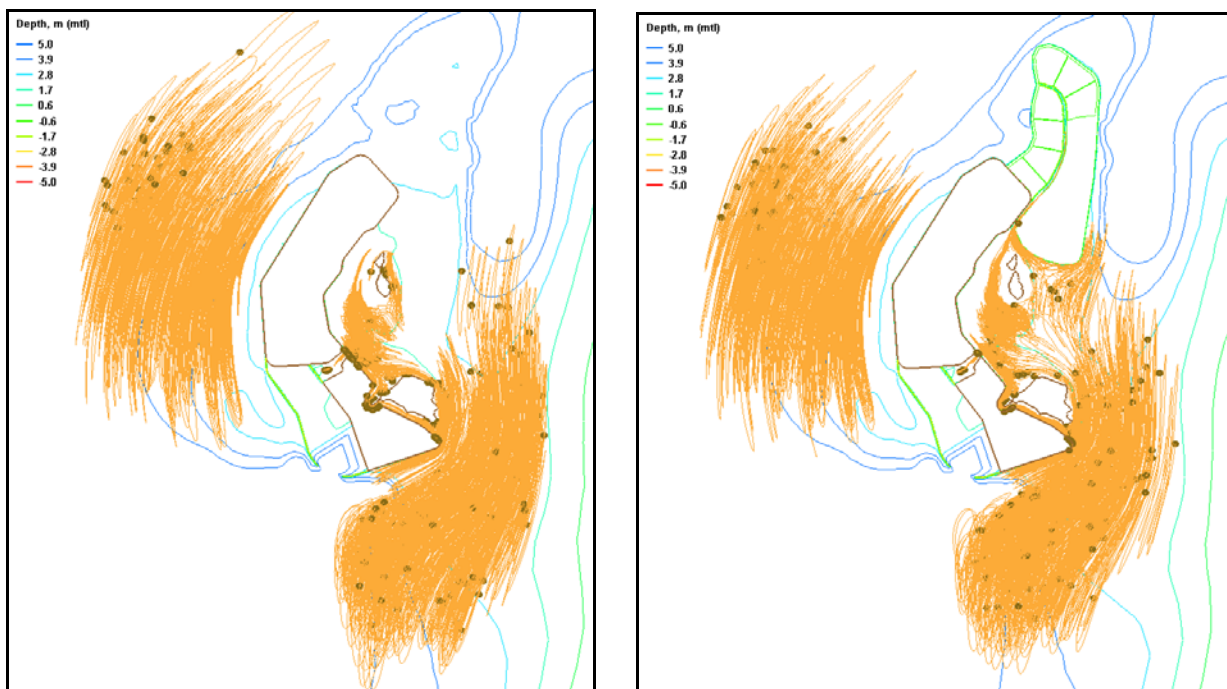
**Figure B-7. Point sources placed around Poplar Island and Harbor area for Option 1, and in the wetland cell in the expansion area for Option 2**

*Source: (USACE-ERDC, 2005a)*

During the initial run on the PTM for each option and event, point sources of neutrally buoyant particles were created with the same attributes [elevation = 1.0 m above bed, number of parcels = 10, parcel mass = 10.0 kg, parcel radius = 1.0 m, particle size = 0.2 mm (not active for neutrally

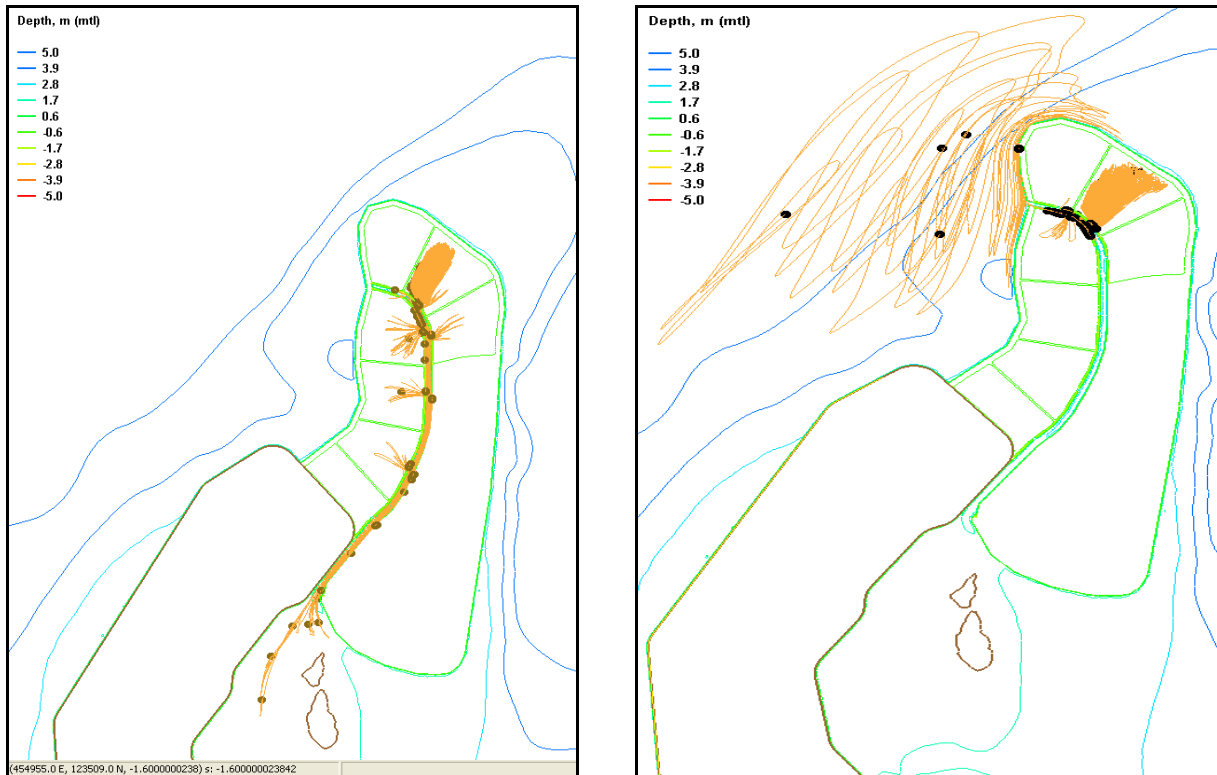
buoyant particles), particle density = 1,035 kg/m<sup>3</sup>, and source type = instantaneous injection] (USACE-ERDC, 2005).

**Particle Tracking Model Results in Poplar Harbor for the Validation Run.** After the PTM was run, a film loop was created to view the movement of each parcel placed around Poplar Harbor. For the validation run, clear periodic or cyclical tidal movement was observed for each of the options. Parcels that were placed on the west and south sides of the PIERP moved more freely than those placed on the east side in the harbor area, which is partially sheltered because of the curved shape of Poplar Island. A comparison of the location and path of particles after eight days for the validation event for the existing condition and Option 1 (Figure B-8) indicated that the particle paths (denoted by the orange lines) were similar, but that there was a wider dispersement of particles for the existing condition.



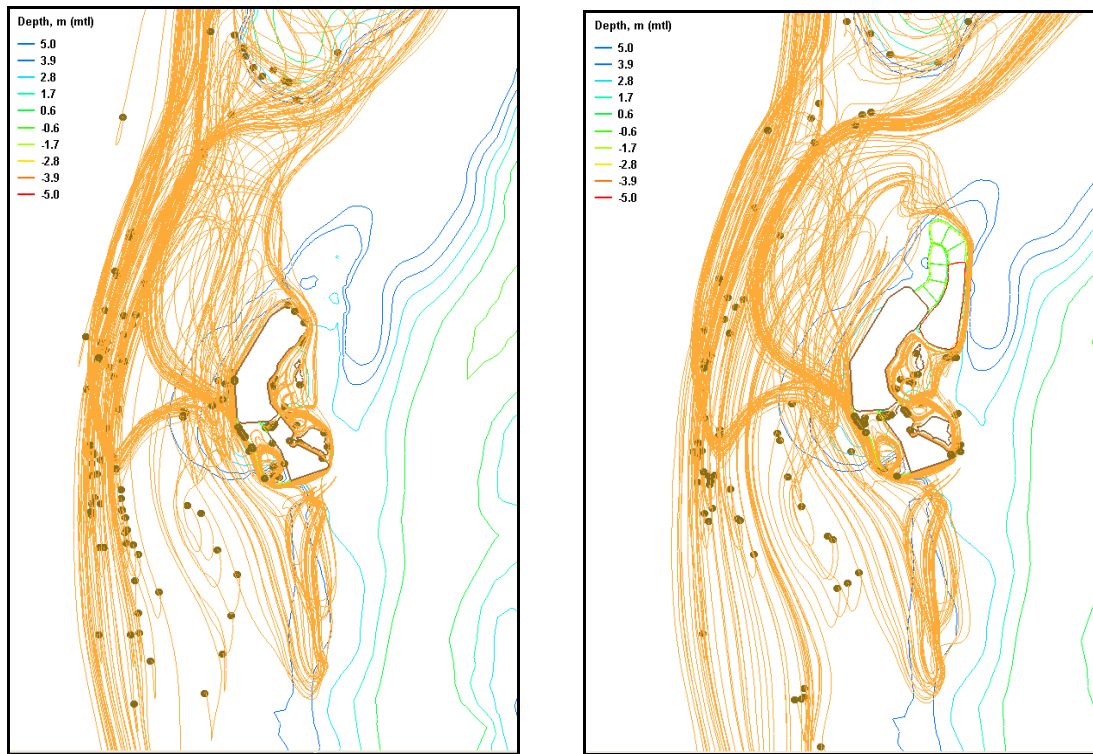
**Figure B-8. Particle track after eight days for existing condition (left) and Option 1 (right)**  
*Source: (USACE-ERDC, 2005a)*

**Particle Tracking Results in an Expansion Wetland Cell for Validation Run.** Particle movement was also modeled for a proposed wetland cell in the northern lateral expansion (Options 1 and 2 only). For Option 1 (Figure B-9), particles moved out from the wetland cell and into others, then migrated south until exiting the expansion area and out into open water. During the Option 2 condition (Figure B-9), many particles remained in the wetland cell, with only a small percentage exiting into the tidal gut where they traveled to other wetland cells and out of the northern tidal gut opening and into open water.



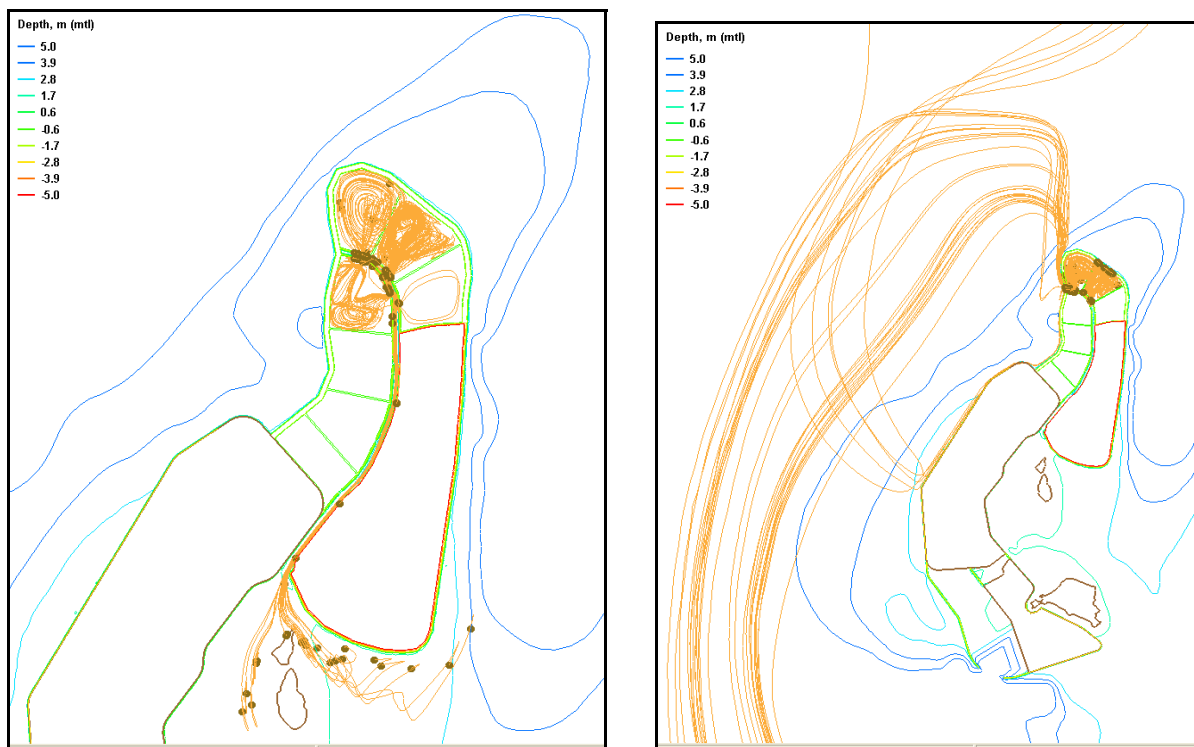
**Figure B-9. Particle track after eight days for Option 1 (left) and Option 2 (right)**  
*Source: (USACE-ERDC, 2005a)*

**Evaluation of Particle Tracking Results in Poplar Harbor for Storm Events.** The results of the PTM for the storm event indicated that many particles were swept away from the island area westward where they entered the main channel of the Chesapeake Bay. Most of the remaining particles were trapped in the large wetland cell at the southwest corner of Poplar Island. No discernable differences in particle paths were observed during the modeled storm event for the existing condition and Option 1 (Figure B-10) for the particles in Poplar Harbor.



**Figure B-10. Particle track for Existing Condition (left) and Option 1 (right) during storm**  
*Source: (USACE-ERDC, 2005a)*

**Evaluation of Particle Tracking Results in the Wetland Cell Area for the Storm.** Particle movement in the wetland cell area is significantly different than particle movement in Poplar Harbor. For Option 1, most of the particles remained in the expansion area and moved between wetland cells, with only a few moving out through the single tidal gut opening as the storm surge receded (Figure B-11). For Option 2, as the water level increased with the storm surge, most particles remained inside the expansion area, with only a few particles exiting through the northern tidal gut opening (Figure B-11). The particles that exited were swept into the main channel of Chesapeake Bay.



**Figure B-11. Particle track for wetland area particles during storm for Option 1 (left) and Option 2 (right)**

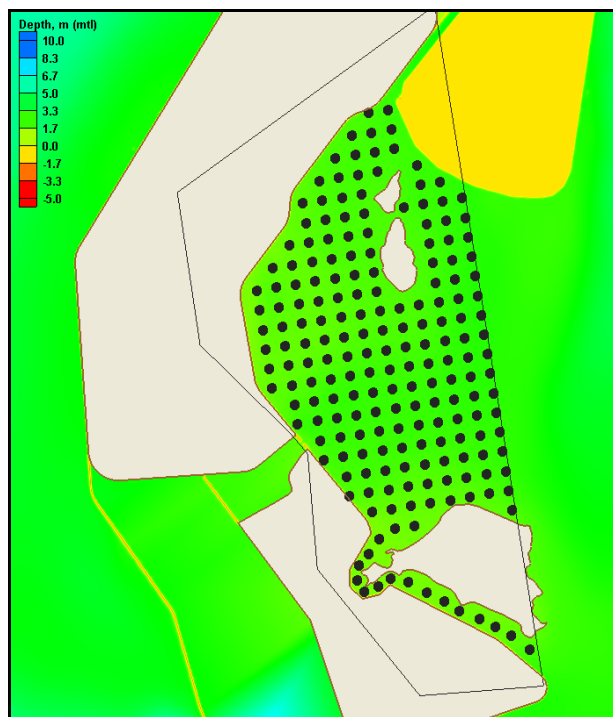
Source: (USACE-ERDC, 2005a)

## B.2.2 Residence Time in Poplar Harbor

Residence time is the average length of time that an entity of interest remains within the limits of a specified water area. Return of particles to the area is taken into account – that is, particles might leave the area, then return. Residence times for Poplar Harbor were calculated by tracking the movement of neutrally buoyant particles, which represent water exchange. All residence times for each particle were summed, and this total residence time determined the average residence time. If particles never exited the reference area, the total simulation time was included.

**Residence Times in Poplar Island Harbor for Validation and Storm Periods.** An updated version of the PTM was developed to allow residence time calculations for neutrally buoyant

particles. For the residence time calculations, a separate set of point sources was developed for the PTM. These sources were only placed inside Poplar Harbor and were regularly spaced 100 m apart, when possible (Figure B-12). A line of particles was also placed in the channel between Coaches Island and Poplar Island. Point source attributes for residence calculations [elevation = 1.0 m above bed, number of parcels = 20, parcel mass = 10.0 kg, parcel radius = 25.0 m, particle size = 0.2 mm (not active for neutrally buoyant particles), particle density = 1,035 kg/m<sup>3</sup>, source type = instantaneous injection] differed slightly from the set used for visualization. The number of parcels for each point source was doubled to 20, and the horizontal parcel radius was increased from 1 m to 25 m. The total number of particles used for this configuration was 3,890.



**Figure B-12. Residence time calculation area and location of point sources for Poplar Harbor**

*Source: (USACE-ERDC, 2005a)*

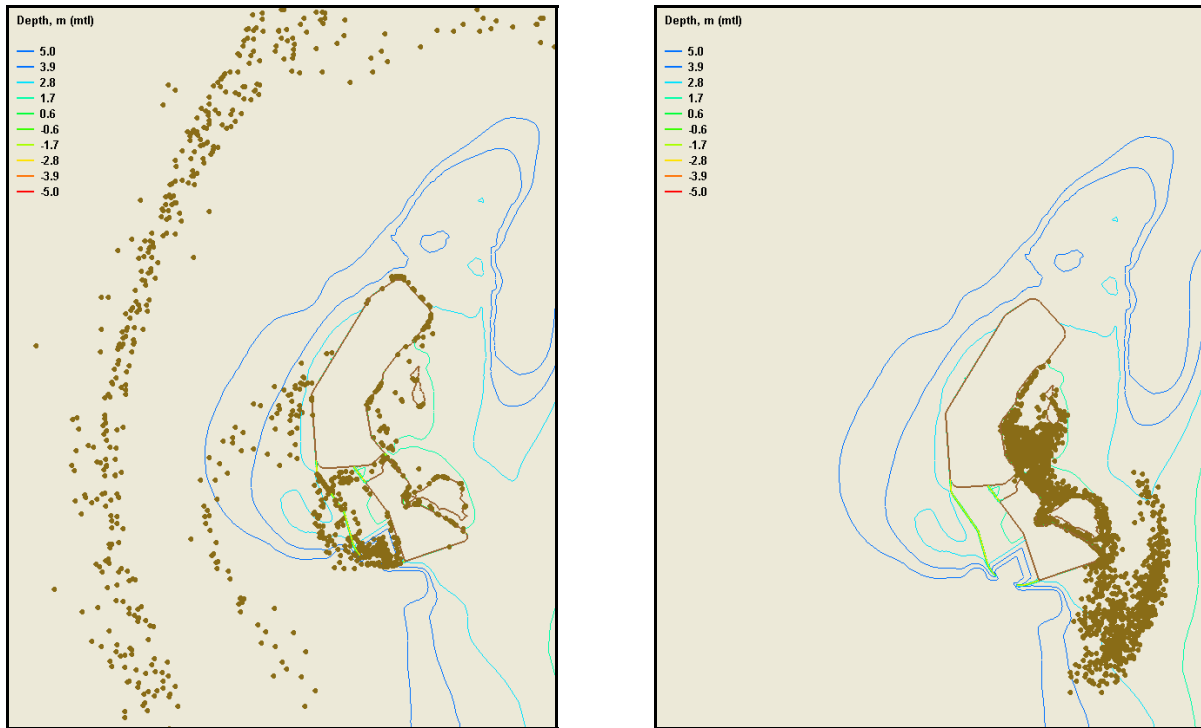
Calculated average residence times for Poplar Harbor are shown in Table B-1. Residence times for the low-energy, eight day reference period of the validation event increased 8.3 percent for Option 1, and increased 15 percent for Option 2 from the baseline (existing condition) residence time of 4.07 days. The residence time for Option 1 is lower because of an increased current from the south end of the tidal gut into the harbor reference area. Option 2 allows two points of egress of flow from the expansion area and thereby has a reduced current from the south end of the tidal gut relative to Option 1.

**Table B-1. Residence times in Poplar Harbor for each Option and event**

*Source: (USACE-ERDC, 2005a)*

	Average Residence Time (days)	
	Validation	Storm
<b>Existing</b>	4.07	0.69
<b>Option 1</b>	4.41	0.91
<b>Option 2</b>	4.68	0.93

During the higher energy, four day storm period, both options produced an increase in residence time of approximately 33 percent over the existing condition for the storm, with a slightly lower increase for Option 1. The substantially shorter reference time for the storm events is expected as the water level and current increased significantly with the storm surge from Hurricane Isabel. Storms thus greatly shorten residence times and enhance flushing. Figure B-13 shows the approximate location of particles for the existing condition after the same 3-day period of the storm event and validation run, respectively.



**Figure B-13. Dispersement of particles after three days of a modeled storm event (left) and after three days of the validation run (right) for the existing condition**

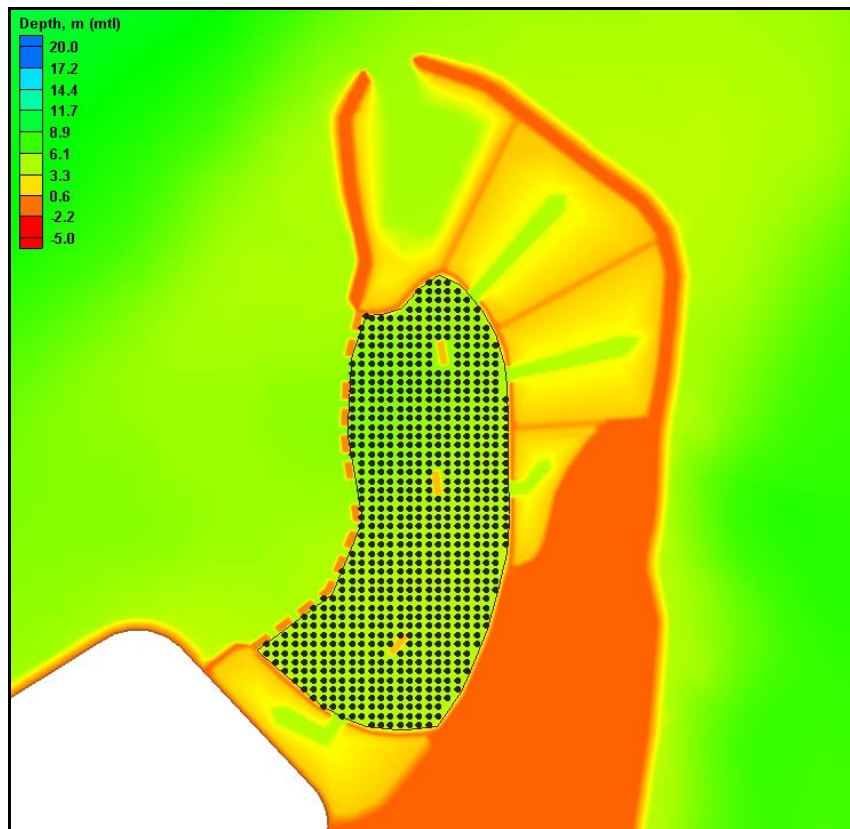
*Source: (USACE-ERDC, 2005a)*



### B.2.3 Residence Time in the Open-Water Embayment

Residence time is the average length of time that an entity of interest remains within the limits of a specified water area. Return of particles to the area is taken into account – that is, particles might leave the area, then return. Residence times were calculated by tracking the movement of neutrally buoyant particles that represent water exchange. Residence times for all particles were summed, and the average residence time was formed. If particles never exited the reference area, the total simulation time was used in the average. Outputs from the particle tracking model (PTM) for each condition and event were analyzed for residence times in the embayment area.

An updated version of the PTM was applied to calculate residence time calculations for neutrally buoyant particles within the open-water embayment. For the open-water embayment residence time calculations, a separate set of point sources was developed for the PTM. These sources were placed inside the calculation area only and were regularly spaced 25 m apart when possible (Figure B-14). Point source attributes for residence calculations [number of parcels = 20, parcel mass = 10.0 kg, parcel radius = 25.0 m, particle size = 0.2 mm (not active for neutrally buoyant particles), particle density = 1,035 kg/m<sup>3</sup> (neutrally buoyant), source type = instantaneous injection] differed slightly from the set used for visualization.



**Figure B-14. Residence time calculation area and location of point sources for the open-water embayment**

*Source: (USACE-ERDC, 2005a)*

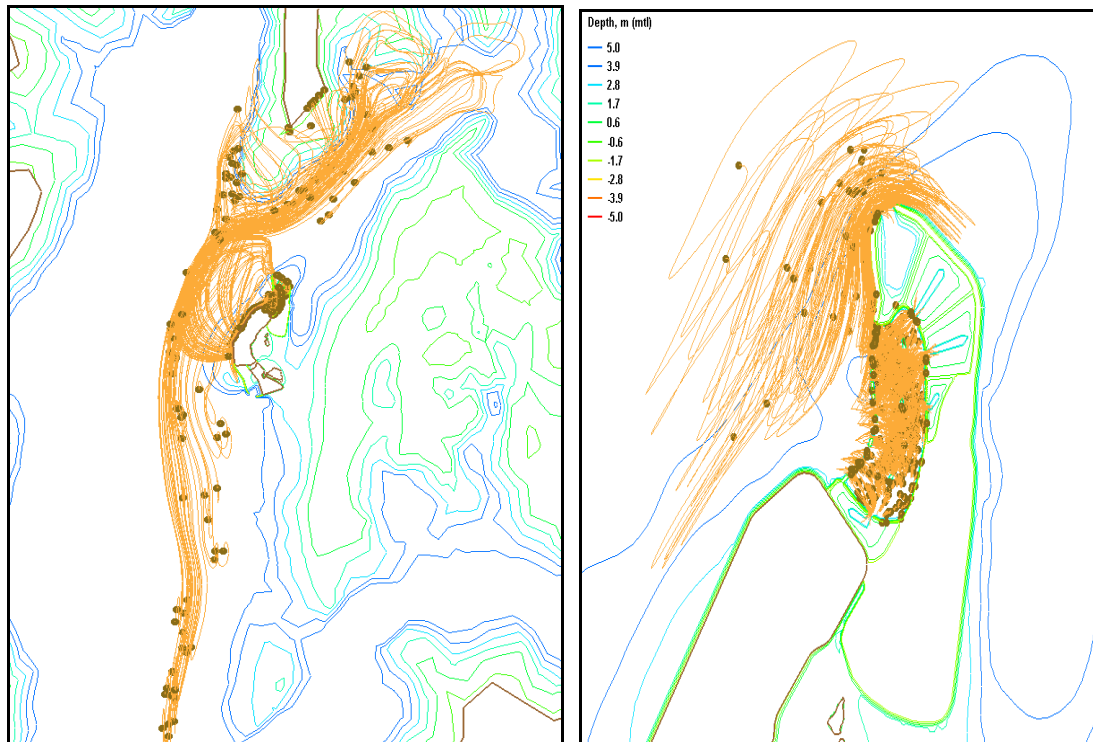
The existing condition has a small residence time (good flushing) for both the validation and storm conditions because the particles move freely in the current, and their travel is unrestricted. The open-water embayment is surrounded on three sides by land or dikes and thereby shelters the area from waves and currents. This sheltering produces a substantially longer residence time for both the validation and storm condition (Table B-2).

**Table B-2. Residence times for the open-water embayment.**

*Source: (USACE-ERDC, 2005a)*

	Average Residence Time (days)	
	Validation	Storm
<b>Existing</b>	0.25	0.18
<b>Open-Water Embayment</b>	3.79	1.10

During the low-energy, 8-day reference period of the validation, the average residence time for the open-water embayment was approximately 15 times longer than for the existing condition. During the higher energy, 4-day storm period, the average residence time for the open-water embayment for the storm decreased by 71 percent, from 3.79 days to 1.1 days. Figure B-15 shows the path of particles and their ending location for both the storm and validation periods for the open-water embayment.



**Figure B-15. Path of particles for the open-water embayment after a modeled storm event (left) and after the validation period (right).**

*Source: (USACE-ERDC, 2005a)*

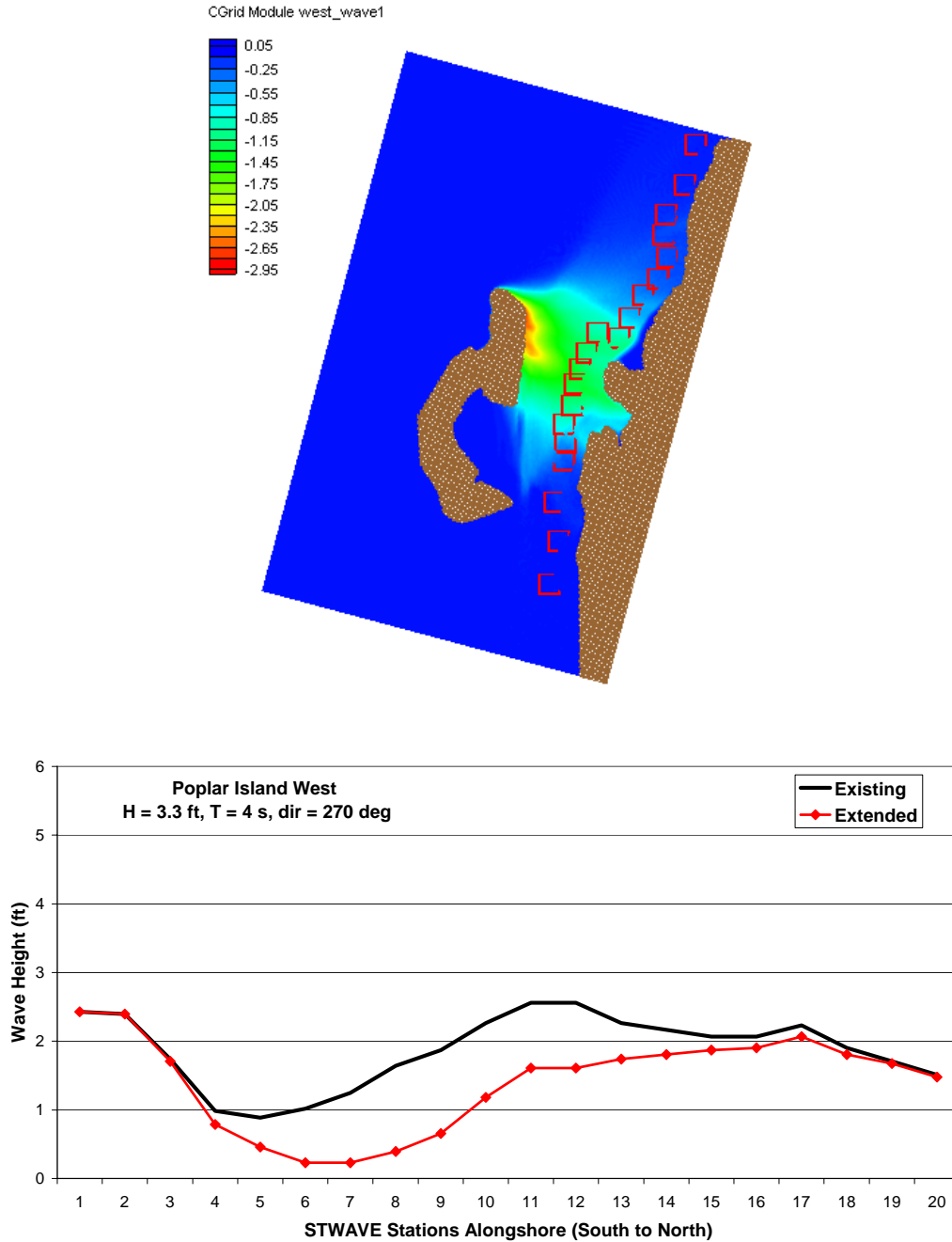
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### **B.3 IMPACTS TO SHORELINE WAVE HEIGHTS**

Potential shoreline impacts from the 575-acre northern lateral alignment were evaluated by modeling the relative difference in wave height at the shoreline between the existing PIERP and the existing PIERP with the lateral expansion using the STWAVE model (Smith, Sherlock, and Resio, 2001). Details of the STWAVE model, model inputs, and sample model results can be found in *Shoreline Impact Study for Poplar Island Expansion. Memorandum for the Record* (USACE-ERDC, 2005b).

Summary plots for each of the modeled scenarios (where negative values indicate a reduction in wave height) and comparison plots of the wave height for the existing and proposed project for 20 points along the shoreline are included in Figures B-16 through B-28. Negative values indicate a reduction in wave height, blue indicates no difference, and reds the maximum difference. Note that the scales vary from figure to figure. Preliminary results of the model indicated that in each case, the maximum difference in wave height for each case is directly in the lee of the lateral expansion, and no increases in wave height along the shoreline were predicted from the lateral expansion, as compared to the conditions from the existing PIERP. The maximum reductions in wave height from the lateral expansion are predicted to be 3-4 ft directly in the lee of lateral expansion. Close to the shore (depth of 9 ft), the maximum reductions in wave height are 1-1.5 ft.

The sheltering effect of the northern lateral expansion was stronger for waves from the north and west, and weaker for waves from the south. Wave height did not increase along the shoreline as a result of the lateral expansion for any cases simulated.

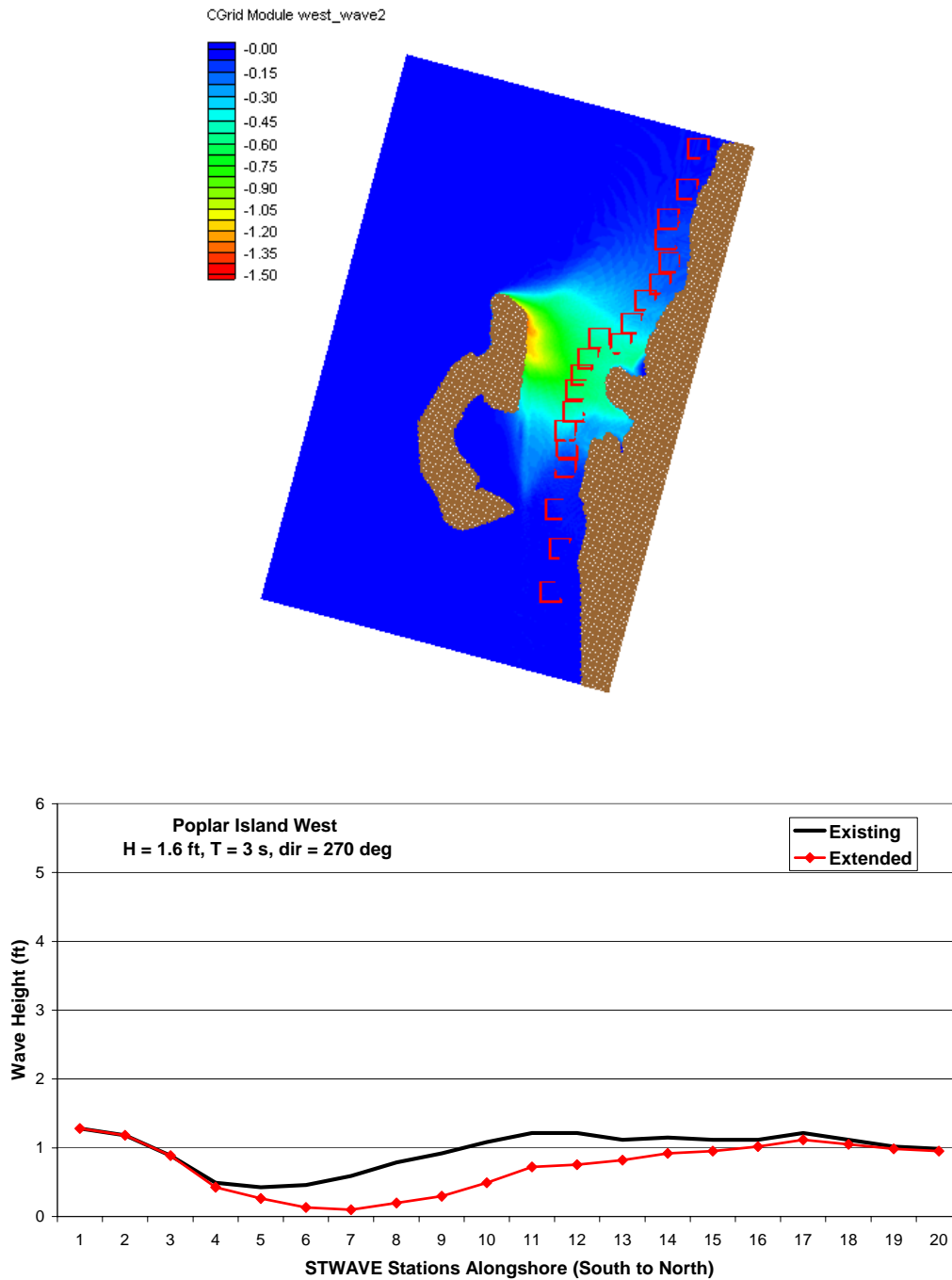


**Figure B-16. Wave Height Difference (ft) and Comparison of Wave Heights for Existing PIERP and the Existing PIERP plus the Lateral Expansion.**

West grid, H = 3.3 ft, T = 4 s, water level = 1.3 ft (MSL), wave direction = 270 deg.

Negative values indicate a reduction in wave height

(Source: USACE-ERDC, 2005b)

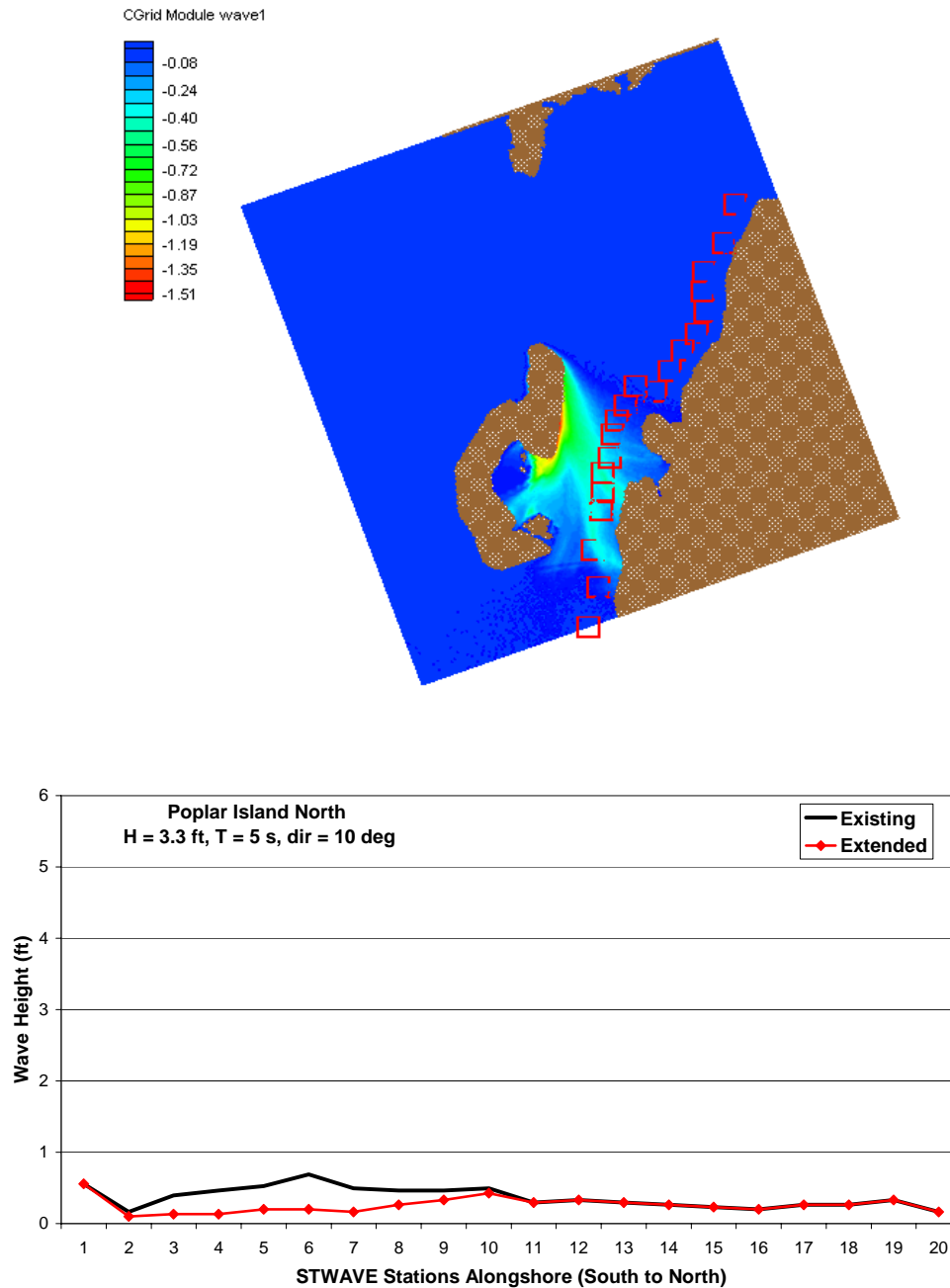


**Figure B-17. Wave Height Difference (ft) and Comparison of Wave Heights for Existing PIERP and the Existing PIERP plus the Lateral Expansion.**

West grid, H = 1.6 ft, T = 3 s, water level = 0.0 ft (MSL), wave direction = 270 deg.

Negative values indicate a reduction in wave height

(Source: USACE-ERDC, 2005b)

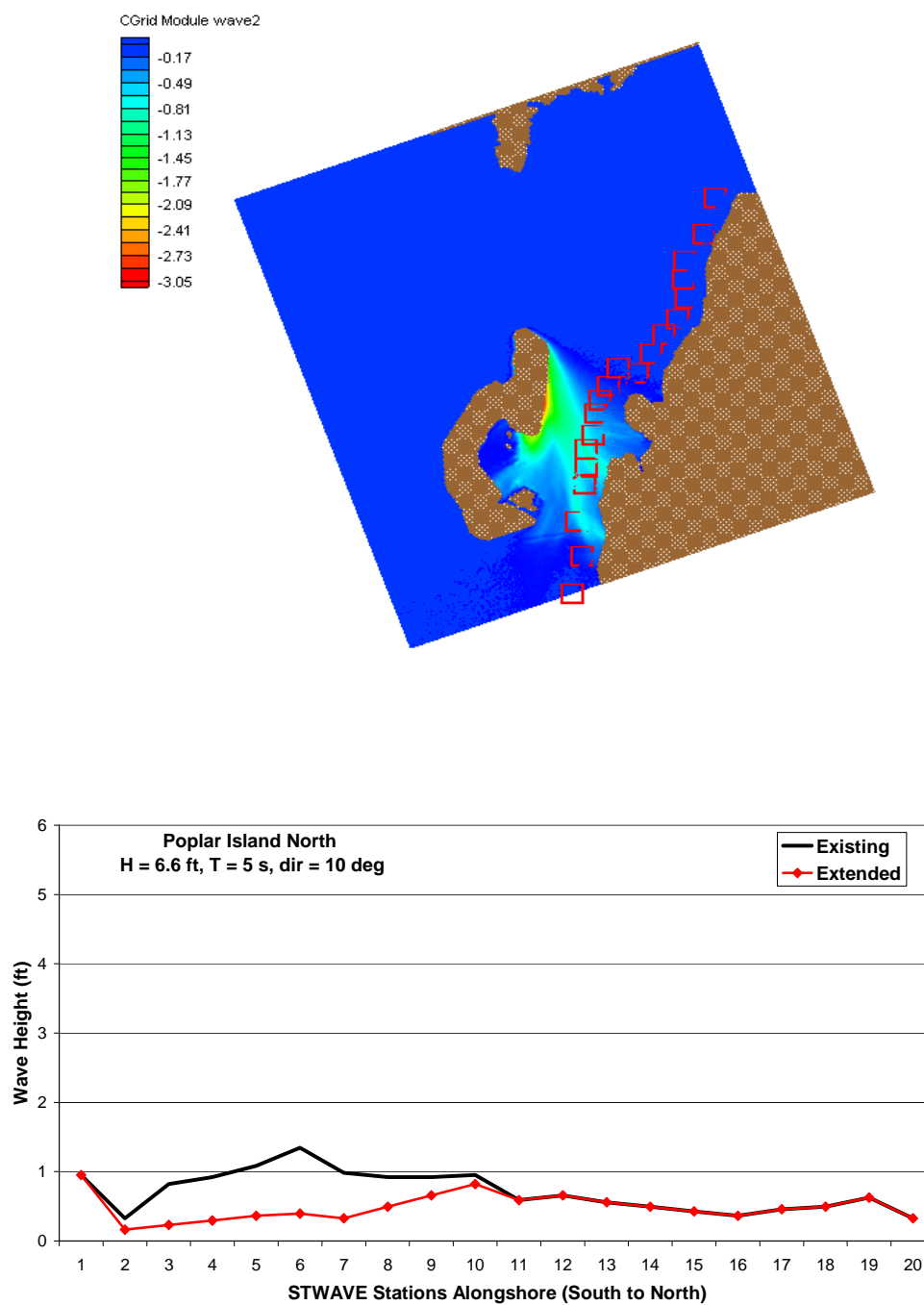


**Figure B-18. Wave Height Difference (ft) and Comparison of Wave Heights for Existing PIERP and the Existing PIERP plus the Lateral Expansion.**

**North grid, H = 3.3 ft, T = 5 s, water level = 1.3 ft (MSL), wave direction = 10 deg.**

Negative values indicate a reduction in wave height

(Source: USACE-ERDC, 2005b)

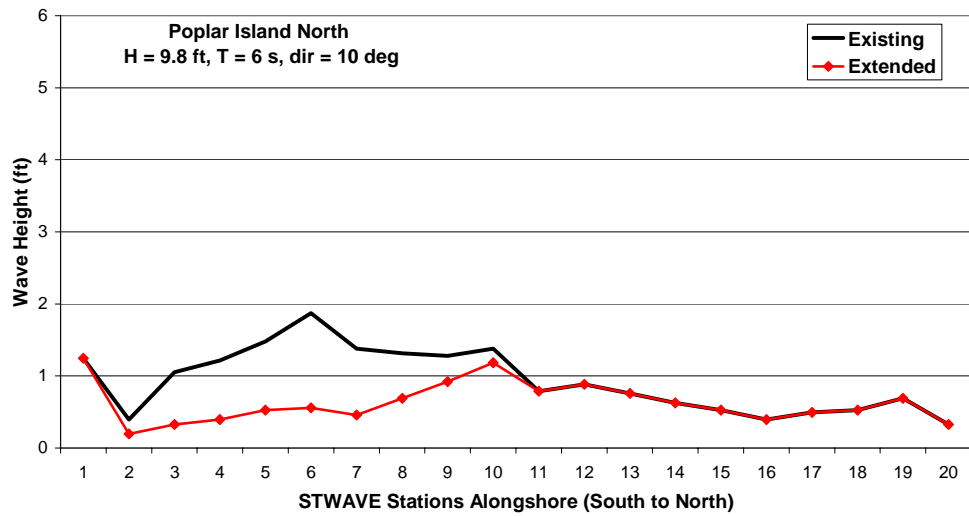
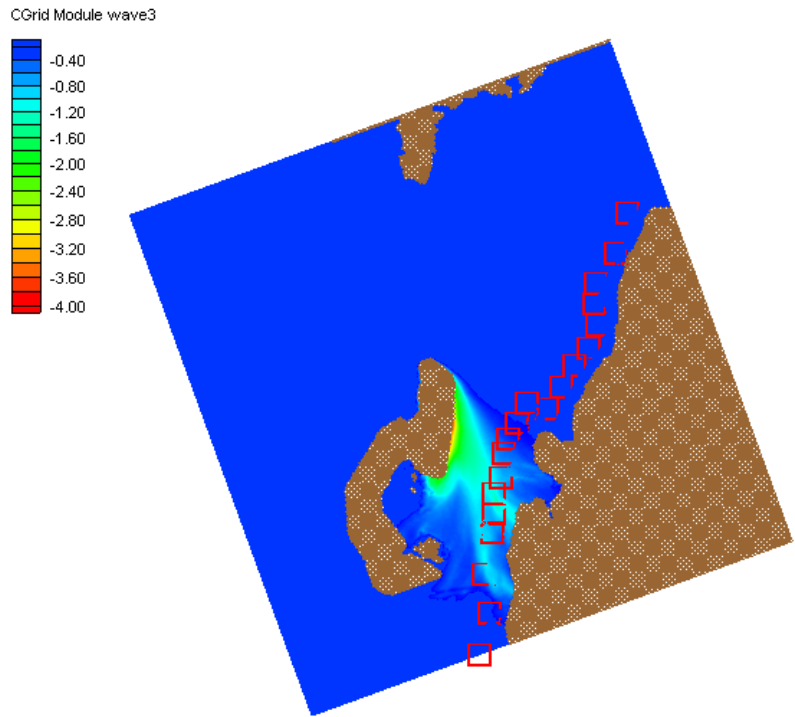


**Figure B-19. Wave Height Difference (ft) and Comparison of Wave Heights for Existing PIERP and the Existing PIERP plus the Lateral Expansion.**

**North grid, H = 6.6 ft, T = 5 s, water level = 1.6 ft (MSL), wave direction = 10 deg.**

Negative values indicate a reduction in wave height

(Source: USACE-ERDC, 2005b)



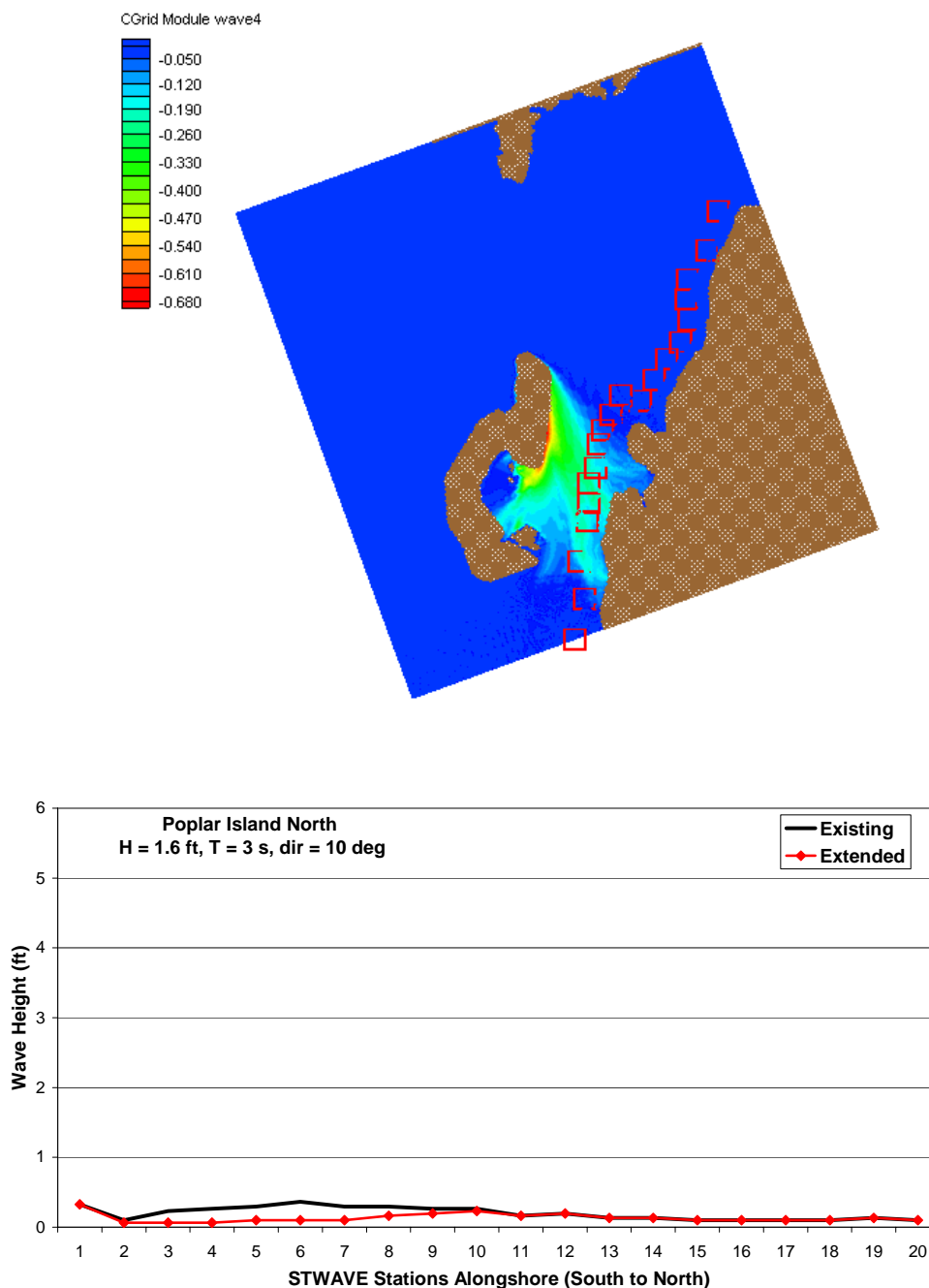
**Figure B-20. Wave Height Difference (ft) and Comparison of Wave Heights for Existing PIERP and the Existing PIERP plus the Lateral Expansion.**

North grid, H = 9.8 ft, T = 6 s, water level = 1.6 ft (MSL), wave direction = 10 deg.

Negative values indicate a reduction in wave height.

(Source: USACE-ERDC, 2005b)



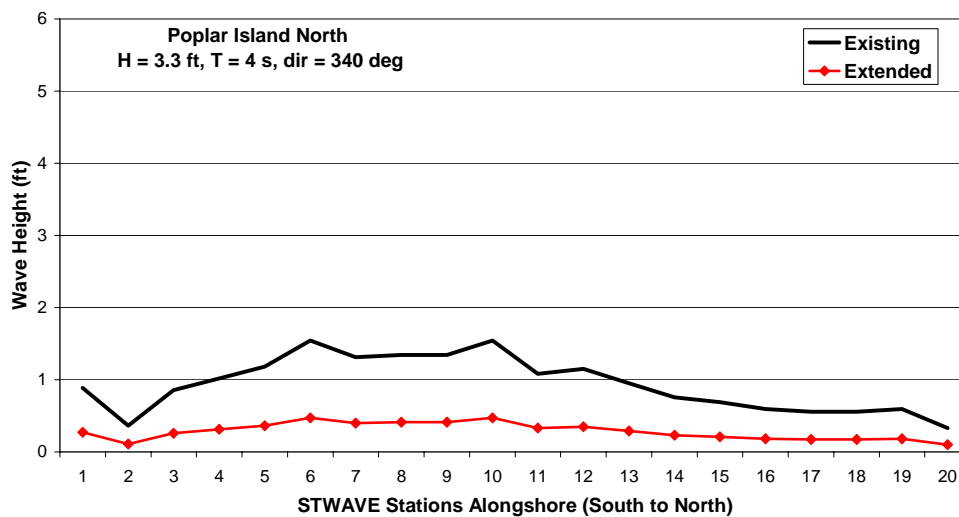
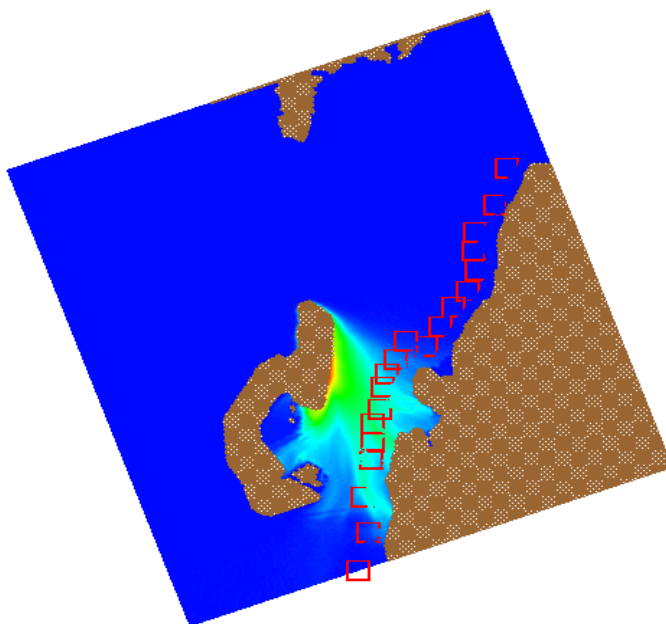
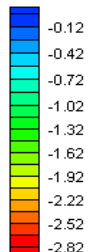


**Figure B-21. Wave Height Difference (ft) and Comparison of Wave Heights for Existing PIERP and the Existing PIERP plus the Lateral Expansion.**

North grid, H = 1.6 ft, T = 3 s, water level = 0.0 ft (MSL), wave direction = 10 deg.  
 Negative values indicate a reduction in wave height.

(Source: USACE-ERDC, 2005b)

CGrid Module wave5

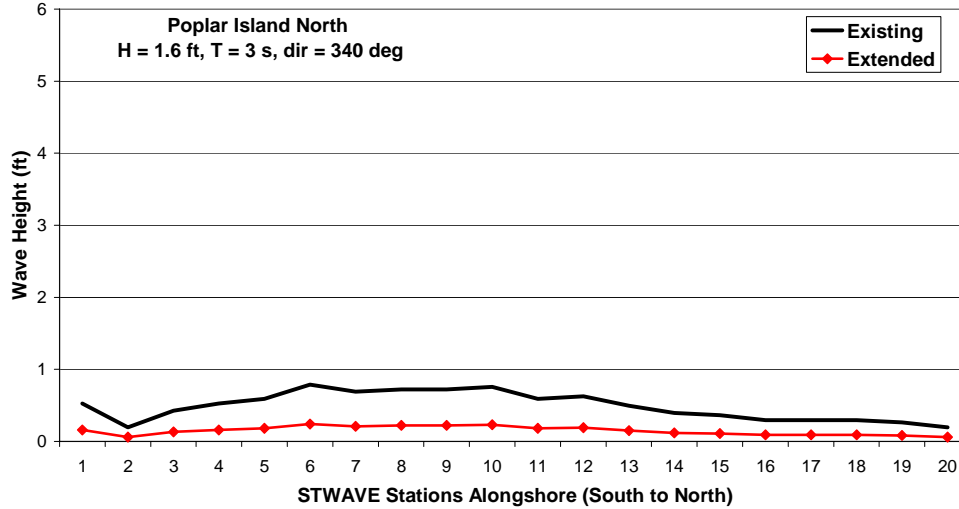
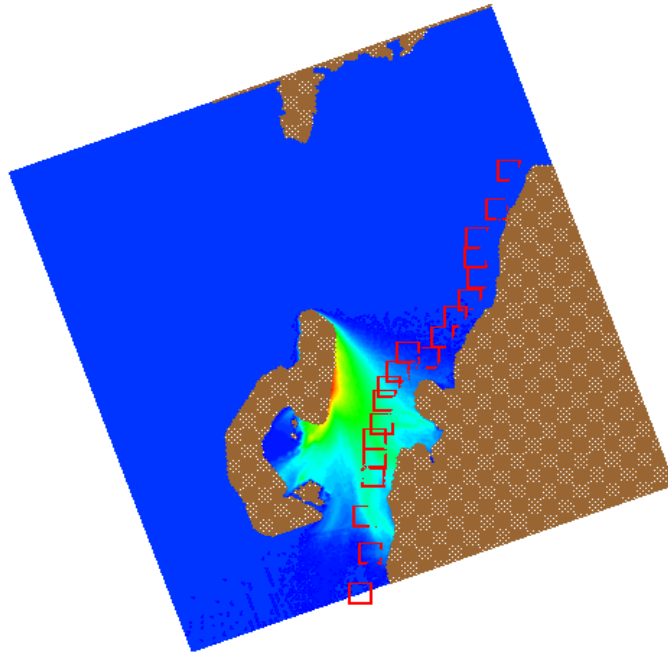
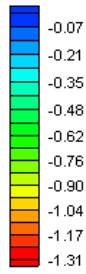


**Figure B-22. Wave Height Difference (ft) and Comparison of Wave Heights for Existing PIERP and the Existing PIERP plus the Lateral Expansion.**

North grid, H = 3.3 ft, T = 4 s, water level = 1.6 ft (MSL), wave direction = 340 deg.  
Negative values indicate a reduction in wave height.

(Source: USACE-ERDC, 2005b)

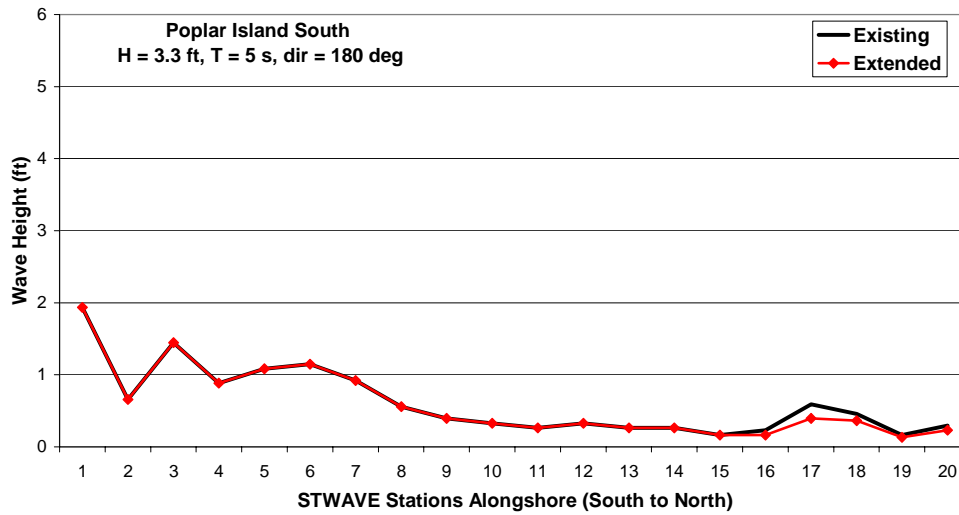
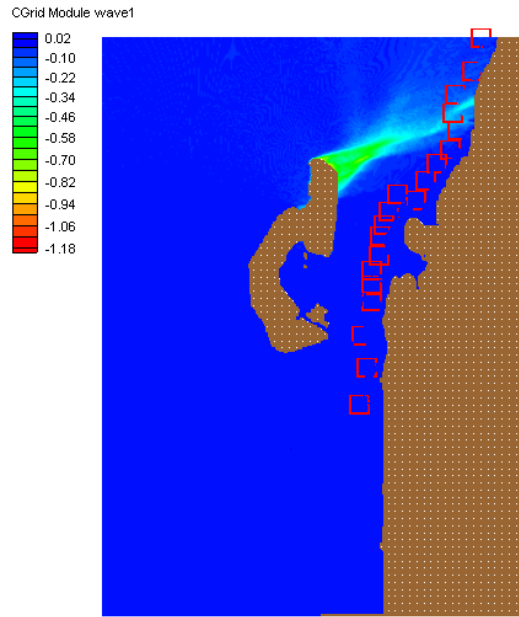
CGrid Module wave6



**Figure B-23. Wave Height Difference (ft) and Comparison of Wave Heights for Existing PIERP and the Existing PIERP plus the Lateral Expansion.**

North grid, H = 1.6 ft, T = 3 s, water level = 0.0 ft (MSL), wave direction = 340 deg.  
Negative values indicate a reduction in wave height.

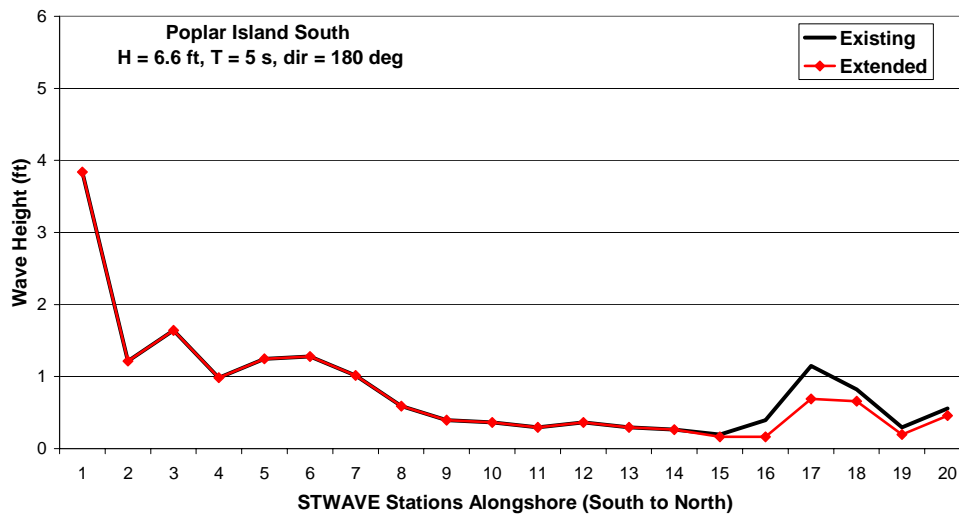
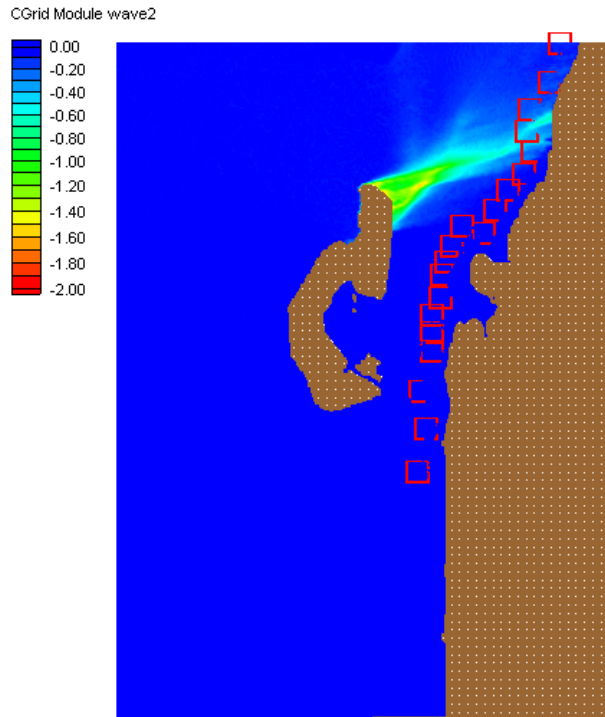
(Source: USACE-ERDC, 2005b)



**Figure B-24. Wave Height Difference (ft) and Comparison of Wave Heights for Existing PIERP and the Existing PIERP plus the Lateral Expansion.**

South grid, H = 3.3 ft, T = 5 s, water level = 1.3 ft (MSL), wave direction = 180 deg.  
 Negative values indicate a reduction in wave height.

(Source: USACE-ERDC, 2005b)

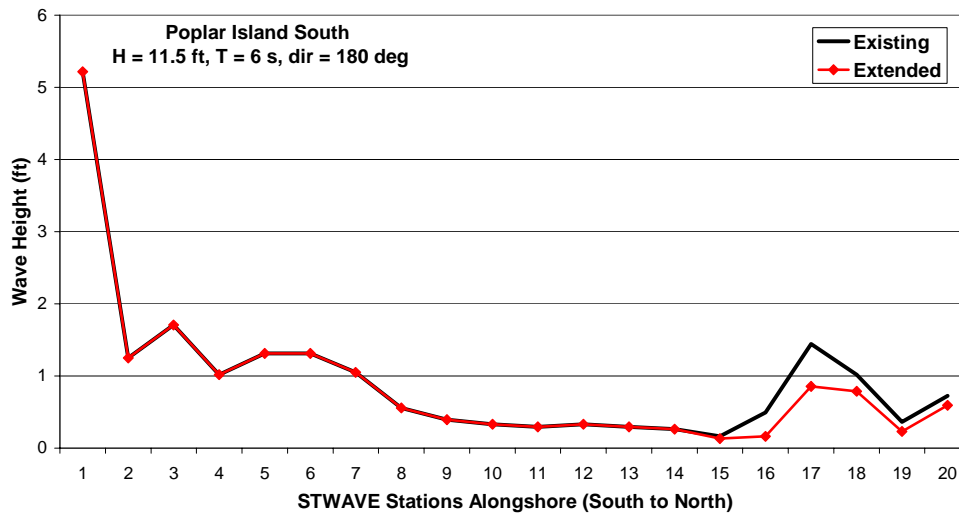
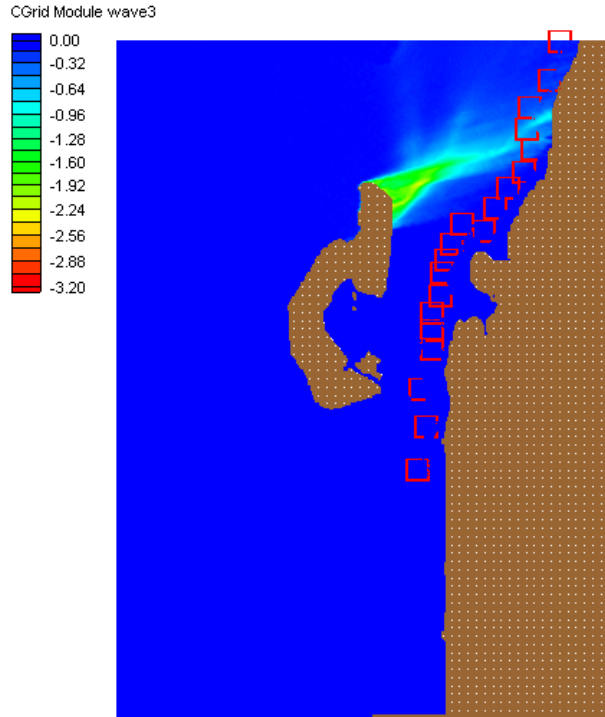


**Figure B-25. Wave Height Difference (ft) and Comparison of Wave Heights for Existing PIERP and the Existing PIERP plus the Lateral Expansion.**

South grid, H = 6.6 ft, T = 5 s, water level = 1.6 ft (MSL), wave direction = 180 deg.

Negative values indicate a reduction in wave height.

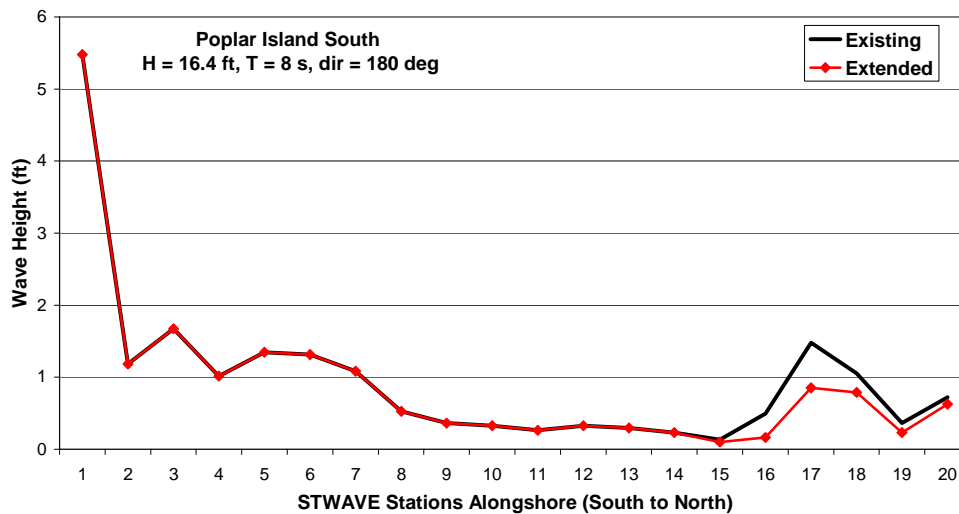
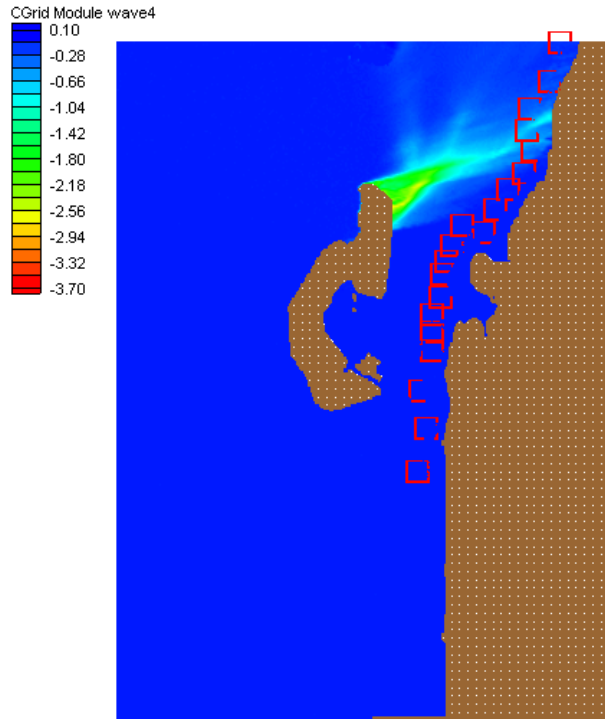
(Source: USACE-ERDC, 2005b)



**Figure B-26. Wave Height Difference (ft) and Comparison of Wave Heights for Existing PIERP and the Existing PIERP plus the Lateral Expansion.**

South grid, H = 11.5 ft, T = 6 s, water level = 3.3 ft (MSL), wave direction = 180 deg.  
 Negative values indicate a reduction in wave height.

(Source: USACE-ERDC, 2005b)

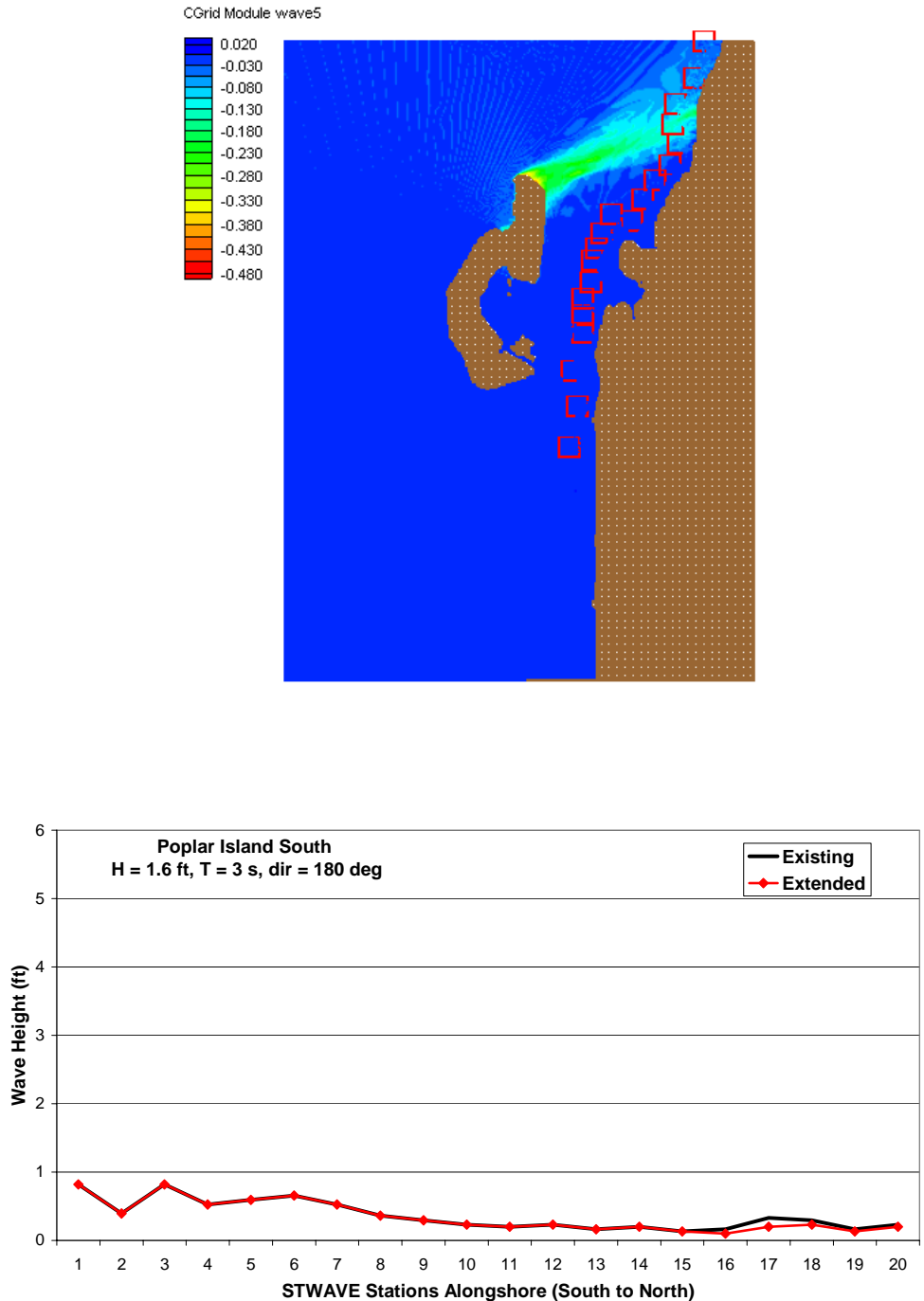


**Figure B-27. Wave Height Difference (ft) and Comparison of Wave Heights for Existing PIERP and the Existing PIERP plus the Lateral Expansion.**

**South grid, H = 16.4 ft, T = 8 s, water level = 6.6 ft (MSL), wave direction = 180 deg.**

Negative values indicate a reduction in wave height.

(Source: USACE-ERDC, 2005b)



**Figure B-28. Wave Height Difference (ft) and Comparison of Wave Heights for Existing PIERP and the Existing PIERP plus the Lateral Expansion.**

**South grid, H = 1.6 ft, T = 3 s, water level = 0.0 ft (MSL), wave direction = 180 deg.**

Negative values indicate a reduction in wave height.

(Source: USACE-ERDC, 2005b)



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## B.4 LIFE CYCLE ANALYSIS FOR THE DIKE DESIGN

The life-cycle analysis for the PIERP dike design was conducted by the U.S. Army Engineer Research and Development Center (USACE-ERDC), Coastal and Hydraulics Laboratory (CHL), for the USACE-Baltimore District. The report *Life Cycle Analysis of Mid Bay and Poplar Island Projects, Chesapeake Bay, Maryland* (USACE-ERDC, 2005c) is summarized in the following sections.

The PIERP is an enhanced habitat with stone revetment around much of its perimeter, about 2.5 miles long and 0.5 miles wide. The PIERP is exposed to waves from all directions, with fetch distances ranging from less than one mile to twenty miles and more, depending on exposure direction. The PIERP is also subject to tides and storm surges, with a mean tide range of 1.2 feet. Extreme storm surges can reach much higher than the range of even extreme high astronomical tides, adding as much as six feet to the astronomical tide level.

The occurrence of extreme conditions at Chesapeake Bay island sites involves an interplay between high winds, elevated water levels, high waves, and shallow water depths. Revetment damage is typically caused by energetic waves directly moving stone on the slope or by high water levels allowing high or moderately high waves to overtop the structure and collapse the crest by undermining the landward side. Extreme water levels do not necessarily coincide with extreme wave heights attacking the various reaches of the island perimeter.

Optimized design of the PIERP revetments to withstand typical storm event conditions is necessary to prevent structural damage and failure. Recent advances in numerical modeling technology have provided tools for significantly improved accuracy of wave and water level estimates for storm events. With present technology, the time variation of winds, waves, and water levels during historical storms can be hindcast based on available historical information.

The approach consisted of the following components:

- a. Identify historical tropical and extratropical storms needed to develop design conditions at Chesapeake Bay project sites.
- b. Acquire wind fields for historical storms identified in *a*, to be used for water level modeling. Open-ocean winds for most storms were available from previous studies.
- c. Adjust wind fields over Chesapeake Bay waters as needed to represent winds over the bay suitable for water level modeling.
- d. Analyze existing historical data from regional anemometers to develop local winds over Chesapeake Bay fetches for wave analysis.
- e. Compute historical storm water levels using the existing ADCIRC numerical model, updating the regional bathymetry and shoreline grid already developed for other USACE-Baltimore District studies at Ocean City Inlet and Assateague Island.

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f. Hindcast historical storm waves using model winds along with measured winds from several area anemometers. Compute historical offshore waves using relationships for wind-wave growth over irregular, restricted fetches.

g. Transform waves through shallow nearshore waters to shore using a spectral wave transformation model (STWAVE).

h. Compute responses for these historical events, such as runup, overtopping as a function of crest height, structure damage as a function of stone size, and required toe stone weight. Use techniques based on recommendations given in the Coastal Engineering Model (CEM).

i. Recreate multiple life cycles of storms and project responses using the Empirical Simulation Technique (EST). Each life cycle represents a possible future condition, which is statistically consistent with historical storm forcing, response, and sequencing information. The EST simulation includes progressive revetment damage due to successive storms that may occur between maintenance opportunities. Realistic maintenance cycles are incorporated into the simulation.

j. Compute life-cycle damage and function for selected designs that appear to be favorable.

A total of 95 historical tropical and extratropical storms were chosen to report predicted water levels under various storm conditions at the PIERP (USACE-ERDC, 2005c). Fifty-two hurricanes were selected from the North Atlantic Hurricane Track Database (1851-2003) for simulation based upon the following criteria: storms with maximum wind speeds greater than 50 knots in the area between 75 and 79 deg W longitude and 36 and 39 deg N latitude. Forty-three northeasters (1954-2003) were identified in the Atmospheric Environmental Service of Canada (AES-40) wind fields (Swail et al., 2000) and in the reanalysis project database (Kalnay et al., 1996) by the U.S. National Centers for Environmental Prediction (NCEP) and the National Center for Atmospheric Research (NCAR). Storms were selected based upon the criteria of peak wind speeds greater than 20 m/sec (66 ft/s) or 10 m/sec (33 ft/s) with durations exceeding three days at the ocean entrance of the Chesapeake Bay.

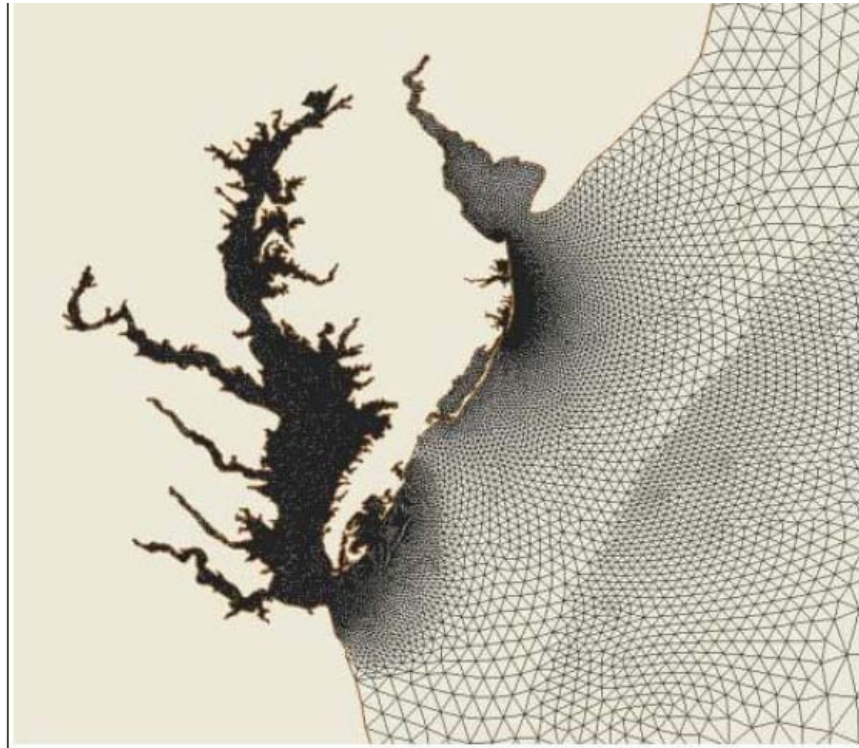
**Water Level Data.** NOAA historical water level data (1996-2003) for Chesapeake Bay was extracted to examine seasonal and daily water level variations, excluding daily tides. These variations were applied to the model results to account for the monthly mean variation of the water level.

**Over-Land Wind Adjustment.** AES-40, NCEP/NCAR and PBL model wind fields are generally accurate for the open coast and ocean applications. In the Chesapeake Bay and adjacent land area, the wind fields needed to be adjusted for over-bay and over-land effects.

**Numerical Grid Development.** The hydrodynamic model ADCIRC (Leutlich et al., 1992) was applied the Chesapeake Bay area for each historical area. A regional scale ADCIRC grid with a rudimentary representation of Chesapeake Bay was developed through previous Coastal Inlets

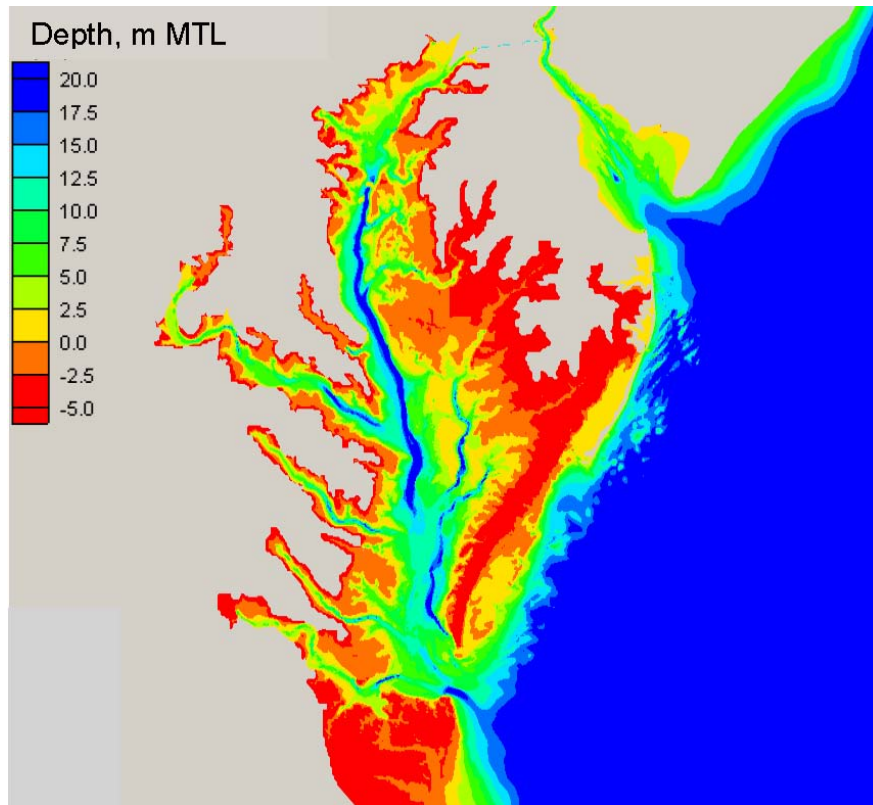
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Research Program (CIRP) and Offshore and Coastal Technologies, Inc. (OCTI) studies and refined in Chesapeake Bay and far-field areas for the present study (Figure B-29) using National Ocean Service (NOS) Digital Navigation Charts (DNC). Further grid development included the incorporation of overbank areas into the Chesapeake Bay tributaries to accurately predict storm surge in these relatively narrow branches of the bay (Figure B-30). The ADCIRC grid was extended to include low land topography data to +10 m (33 ft), mean tide level, from USGS Digital EEM database GTOPO30 -- 30-sec arc resolution. The grid was constructed with a minimum resolution of 50 m (164 ft) and a maximum cell size of 500 m (1640 ft) in the open ocean.



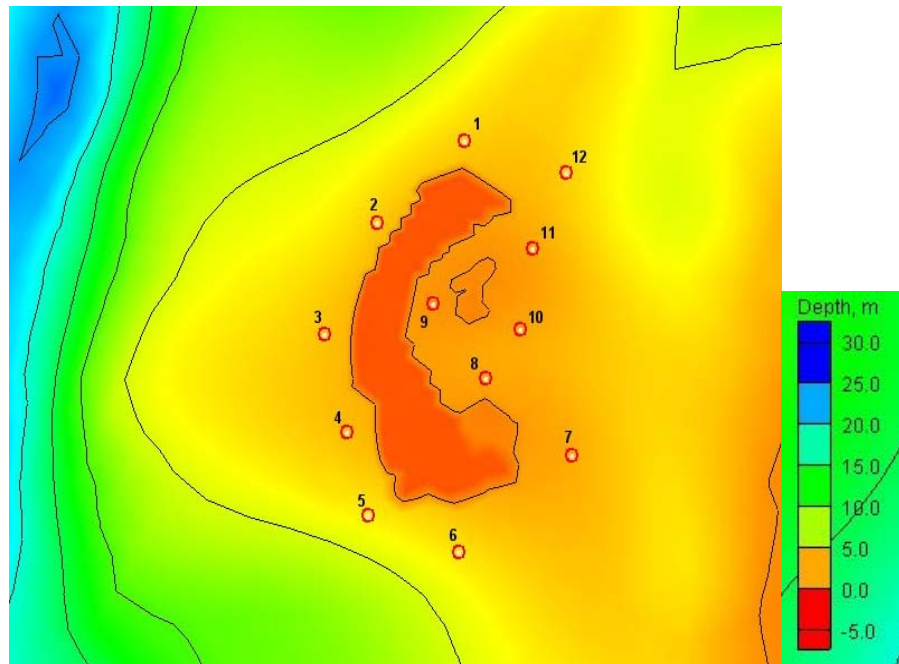
**Figure B-29. Portion revised ADCIRC grid resolution and shoreline**

*Source: (USACE-ERDC, 2005c)*



**Figure B-30. Portion of revised ADCIRC grid bathymetry with overbank extensions**  
*Source: (USACE-ERDC, 2005c)*

The validated model was applied to the suite of 95 storms (hurricanes and northeasters), and maximum water levels were extracted at twelve locations in the vicinity of the PIERP (Figure B-31) for use in the wave modeling and life cycle analysis. Water levels predicted at Poplar Island for 52 hurricanes ranged between 0.33 and 2.44 m (1.1 and 8.0 ft), and water levels the northeasters ranged between 0.19 and 0.98 m (0.6 and 3.2 ft).



**Figure B-31. Twelve Poplar Island locations for saving simulated water levels**  
*Source: (USACE-ERDC, 2005c)*

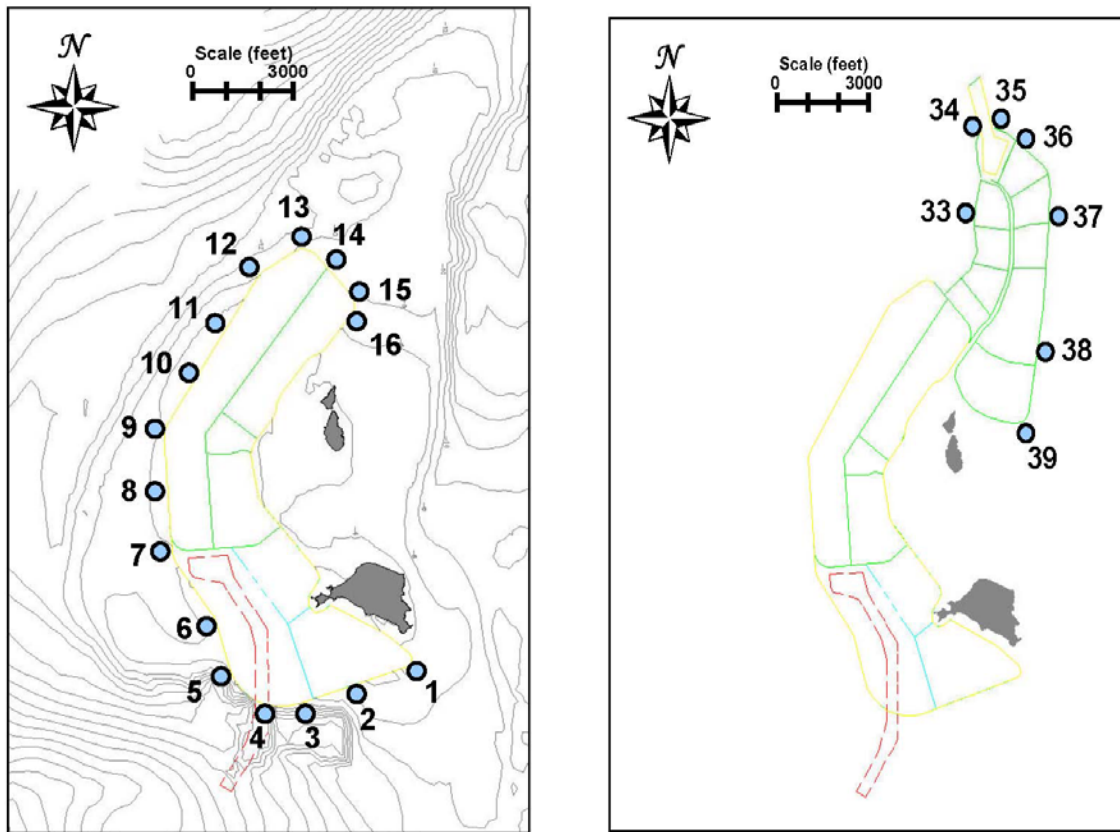
#### B.4.1 Wave Modeling

Life-cycle analysis for the PIERP required wave parameters around each island for a variety of storm conditions. Winds were carefully validated and, in the case of the AES-40 hindcast winds, adjusted to compensate for reduced over-water drag. Three steps were required for wave modeling to produce the life cycle inputs, including a restricted-fetch wave growth model (Smith, 1991) in the Automated Coastal Engineering System (ACES), application of a parametric spectral shape in the Surface Water Modeling system (SMS), and the spectral transformation model STWAVE (Smith et al., 2001). Additional details of the wave modeling transformations can be found in USACE-ERDC (2005c). Results of the STWAVE model indicated that longer period waves interact more strongly with the bottom, resulting in greater refraction (turning of the wave directions) and shoaling (increases in wave height in shallow depths) as compared to shorter period waves. This refraction indicated that there was more turning of the wave direction toward the PIERP for longer period waves.

The time period covered by tropical storms is 148 years (1856-2003), while the time period covered by northeasters is only 50 years (1954-2003). Northeasters are more common than tropical storms and are less likely than tropical storms to be atypically severe. To populate the early tropical storm years with northeasters, the northeasters were folded back. Thus, a 148-year offshore time history of historical storm waves and water levels was created.

For Poplar Island, sixteen points (Stations 1-16) were selected adjacent to shore for design analysis (Figure B-32). Another 16 points (Stations 17-32) were selected further offshore from

each of the points shown, approximately 300 m from shore (not shown). Finally, seven points (Stations 33-39) were selected adjacent to shore for the proposed northern lateral expansion (Figure B-32).

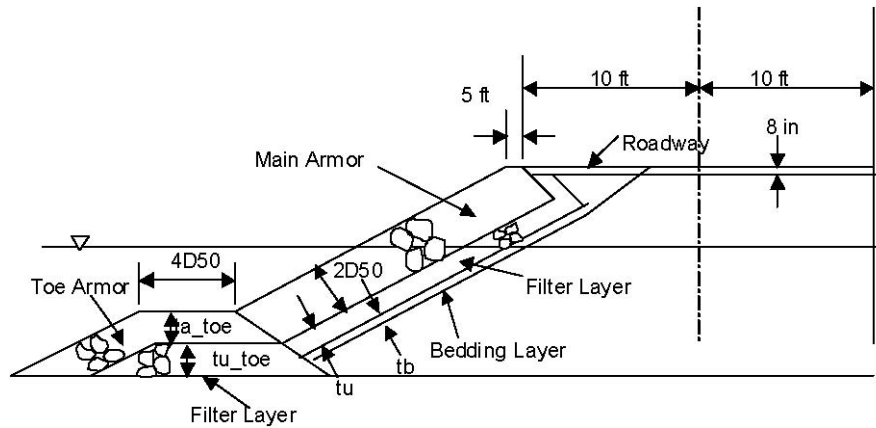


**Figure B-32. Locations at the PIERP used for model result comparisons**

*Source: (USACE-ERDC, 2005c)*

#### **B.4.2 Structural Optimization**

In this study, it is anticipated that the least cost dike structure cross section that prevents breaching during the economic life will provide the structure portion of the NED alternative. Typically, the NED plan is the least expensive alternative over the project economic lifetime, including first cost and maintenance costs as well as extraneous benefits and costs. Therefore, the basic objective of the optimization scheme described herein is to minimize total amortized costs including maintenance and first costs with the constraint that breach failures over the economic life are to be avoided. The constraint of avoiding breaches is required to avoid the large environmental costs (such as loss of sediment contained within the island) associated with a breach-type failure. A typical multi-layer rubble mound revetment (Figure B-33) was assumed for the lateral expansion.



**Figure B-33. General Revetment Cross Section**

Variables that are most influential in the optimization are the crest height, armor stone size and structure slope. Damage occurs primarily as a result of waves attacking and displacing armor stones and as a result of wave overtopping producing scour of the crest. Damage to the armor layer will progress in a predictable and continuous manner until the filter layers are exposed. At that point, the deterioration will accelerate until the structure is breached. It was conservatively assumed that the damage progresses very rapidly from exposure of the filter layers to breaching during one 3-hr increment. Damage to the crest because of overtopping is similar in that damage progresses slowly unless the overtopping exceeds a certain magnitude. At that point, the damage progresses very rapidly to a breached condition. Therefore, it was assumed that, for an unarmored crest, the structure progresses from minor damage to breach within one time step of 3-hrs if the overtopping rate exceeds this value.

### **B.4.3 Results and Recommendations of the Life Cycle Analysis for the Northern Lateral Alignment**

**Waves and Water Levels.** Stations with an open exposure toward the south experience the highest waves. These are also the stations most dominated by hurricanes. North- and east-facing stations along the north end of Poplar Island are less dominated by hurricanes. Return period significant wave height ( $H_s$ ) at stations 33 through 39 follow a smooth variation around the proposed expansion for return periods up to about 40 - 50 years. For longer return periods, the impact of hurricanes causes a noticeable increase in significant wave height at stations 33, 34, and 39 relative to the other stations.

**Structural Analysis.** The structural analysis of poplar island is composed of two primary parts: 1) preliminary analysis using only the historical waves and water levels and 2) final design using the simulated waves and water levels. The preliminary structural optimization for Poplar Island for the historical wave climate is separated into two parts: optimization for least cost and optimization for fewest repairs.

**Upland Cell Stability.** The upland cell analysis consisted of analyzing the compound slope runup for the upland cells corresponding to stations 37 and 38 (Figure B-32). These stations

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show no overtopping breaches for return periods greater than five years, regardless of crest height. The stations also show no runoff on the upland cells for the higher return periods. As such, there is no need to armor the upland cell slopes as long as the crest height of the lower structure is maintained at a reasonably high level of greater than 2.29 m (7.5 ft) MLLW.

**Conclusions and Recommendations.** From the results of the preliminary and ELS investigations summarized above a few conclusions can be made:

1. The least cost solution that provides a reasonable level of protection for the reach represented by stations 33 – 34 (from the southwest start of expansion to northwest entrance) is the 40-yr return period design with an armored crest. This design can be expected to have a 5 percent chance of exceeding two repairs and a 2 percent chance of exceeding three repairs during the 50-yr life cycle. There is little apparent increase in cross sectional cost in going to larger stone, given that stone unit costs and equipment costs do not increase with increasing stone size. However, this is likely not the case. As such, the recommended design for the western side of the expansion is based on station 34 with the 40-yr return period cross section, armored crest at 2.9 m (9.5 ft) and primary slope of 1V:3H.
2. Based on the preliminary design, the northern reach represented by stations 35 – 36 is between the sheltered eastern side and the exposed western side. The required stone sizes are between the two. The water level exposure is low, similar to the eastern side. So the recommended design for this reach is based on station 36 with the 40-yr return period cross section, armored crest at 2.9 m (9.5 ft) and primary slope of 1V:3H.
3. The least cost solution that provides a reasonable level of protection for the reach represented by stations 37 – 38 is based on the station 37 cross section. The least cost alternative corresponds to the 10-yr return period design with an armored crest. The least cost crest height is the lowest at 2.29 m (7.5 ft)
4. The least cost solution that provides a reasonable level of protection for the reach represented by station 39 corresponds to the 50-yr return period design with an armored crest at 3.2 m (10.5 ft).

So it is recommended that three different cross sections be used for the proposed northern lateral expansion. Each of the following designs have a primary armor seaside slope of 1V:3H and a toe seaside slope of 1V:2H. The final design return period was 40 years for all stations except 39, which was 50 years. The final recommended sections are as follows:

Western Reach (stations 33 – 34)

Armor stone weight: 1.04 tonne (2300 lb)

Primary underlayer stone weight: 0.10 tonne (230 lb)

Crest:  $h_c = 2.9$  m (9.5 ft) armored

Toe stone weight: 0.54 tonne (1200 lb)

Toe underlayer weight: 0.054 tonne (120 lb)



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Northern and Southeastern Reaches (stations 35, 36, 39)

Armor stone weight: 0.29 tonne (650 lb)

Primary underlayer stone weight: 0.03 tonne (65 lb)

Crest:  $h_c = 2.9$  m (9.5 ft) armored

Toe stone weight: 0.15 tonne (330 lb)

Toe underlayer weight: 0.015 tonne (33 lb)

Eastern Reach (stations 37 – 38)

Armor stone weight: 0.15 tonne (330 lb)

Primary underlayer stone weight: 0.015 tonne (33 lb)

Crest:  $h_c = 2.9$  m (9.5 ft) armored

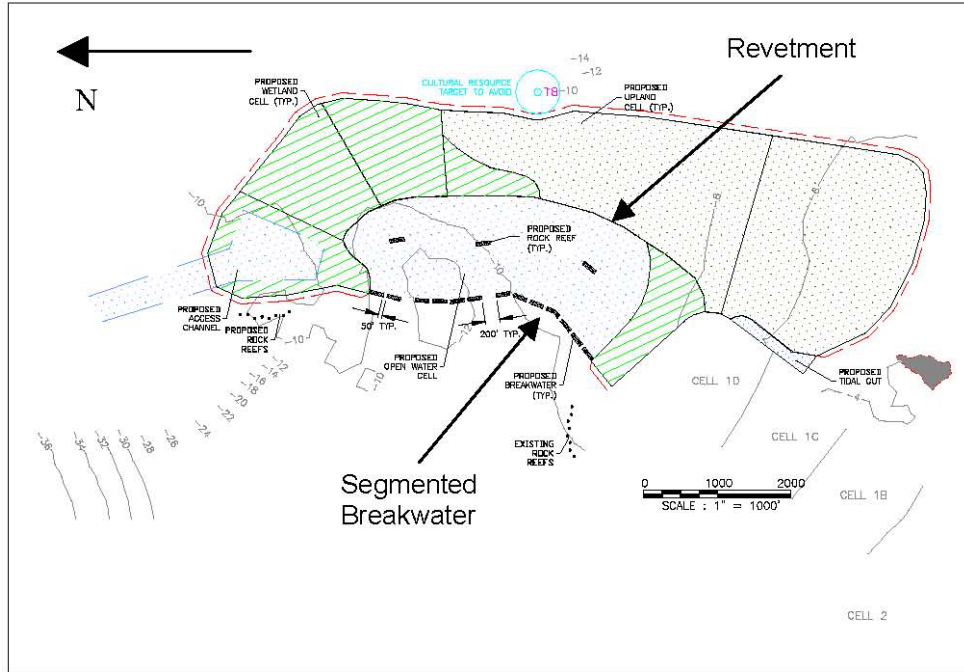
Toe stone weight: 0.054 tonne (120 lb)

Toe underlayer weight: 0.005 tonne (12 lb)

#### **B.4.4 Open-Water Embayment (Offshore Breakwater Alternative)**

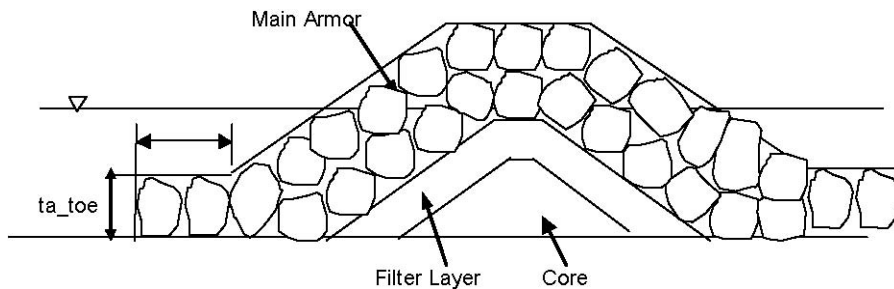
National Marine Fisheries Service (NMFS) proposed a variation for the northern lateral alignment that included an open-water embayment. In the NMFS proposal, the footprint of the northern lateral alignment was the same as those proposed by USACE, but approximately 130 acres of wetland located on the western side of the lateral expansion was designated as an open-water embayment protected by segmented breakwaters and bordered by salt marsh and mudflats. USACE-Baltimore District modified the initial open-water embayment proposed by NMFS to enhance the hydraulic characteristics of the proposal and minimize the impact on the dredged material placement capacity of the lateral expansion. The open-water embayment alignment proposed for the northern lateral alignment of the PIERP (Figure B-34) would provide semi-protected fisheries habitat adjacent to wetland and upland cells, and would increase the trophic interaction between the wetland cells and the open-water embayment within the lateral expansion. The bottom habitat of the open-water embayment would remain essentially undisturbed, preserving the existing bathymetry and benthic habitat. In addition, the construction of small rock reefs within the open-water embayment would provide cover and enhance fish habitat.

Included in the proposal is a shoreline revetment that runs generally north-south in the middle of the lateral expansion. The analysis summarized in the following sections (USACE-ERDC, 2005c) was conducted to provide an initial optimization of the segmented offshore breakwater and revetment using only historical wave conditions.



**Figure B-34. Alignment for the Open-Water Embayment with Offshore Breakwaters**

**Offshore Breakwater Design.** For this effort, it was assumed that the offshore breakwaters would be configured as in Figure B-35, with a structure crest that was three stones wide. The section included a traditional section with two-stones thick armor and filter layers. For the final section, filter material may replace the core so there are only two stone classes. For all analyses, it was assumed that the structure was placed on a geotextile to prevent fines from leaching up through the structure.



**Figure B-35. Typical Section for Offshore Breakwater Structures**

The maximum surge levels in the historical record occurred during Hurricane Hazel. The maximum surge along this reach of Poplar Island was 2.048 m (6.72 ft) MSL or 2.278 m (7.47ft) MLLW (USACE-ERDC, 2005c). This water level dictates the minimum height of the crest for

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the shoreline revetment because the structure would likely be undermined if flooding because of storm surge overtopped the structure. In addition, there would likely be significant damage to the ecosystem if significant flooding occurred. However, no such restriction has been placed on the crest height for the offshore breakwaters. In this investigation, an optimum combination of crest height for the offshore structures and armor size for the shoreline revetment is sought that will minimize costs and maintain functionality of the embayment.

Initial results indicate that the least-cost alternative is the one with the smallest cross-section and the largest armor stone. The steep slope alternatives with seaward and leeward side slopes of 1V:1.5H are the least expensive (USACE-ERDC, 2005c). Of these, the lowest crest is the least expensive. For the least-cost alternative with crest height of 1.83 m (6-ft) MLLW, the largest stone with a return period of 100 years is the least expensive. There is one minor damage repair and no breaches for return periods of 35-100 years. Although the larger stone is least expensive, it has been assumed that this stone is the same unit cost as the smaller stone, which may not be the case. So, the lower return period of 45 years is selected for design. For the next higher crest height of 2.13 m (7-ft) MLLW, the least cost alternative is the 40 year return period and there is no repair for 40 year or greater return periods.

**Revetment analysis.** The shoreline revetment analysis was conducted using similar methods to those that described used for the perimeter dike. The representative cross section is shown in Figure B-35. An unarmored crest with crest width of 7.62 m (25-ft) is assumed.

For design of the revetment, the transmitted wave height past the offshore breakwater was determined. The wave transmission will occur because of wave overtopping, wave diffraction through the gaps, and wave transmission by porous flow through the structures. For this preliminary analysis, it was assumed that wave transmission as a result of porous flow was negligible. Wave transmission from overtopping depends on offshore structure crest height and incident wave height, wave length, and water level. Wave transmission from diffraction depends on gap width and wave period. The overtopping transmission was assumed to be constant along the length of the revetment while that due to diffraction will vary with location.

The analyses indicated that the minimum cost for revetment landward of the 200-ft gap occurs for larger return periods (USACE-ERDC, 2005c). Because some secondary costs of repair are unknown and because repairs are generally not desirable, it is expected that even larger return periods will be more economical. Therefore, a return period of 45 years was selected for the preliminary design.

### **Recommendations for the Open-Water Embayment Offshore Breakwaters and Revetment.**

The recommended stone weights and layer thicknesses for both the offshore breakwater and the revetment section are as follows:

Offshore breakwater design at  $x = 457$  m (1,500 ft)

- Crest Height: 1.83 m (6 ft)
- Armor Weight: 1,697 N (384 lb)
- Armor Thickness: 0.80 m (2.64 ft)
- Filter Layer Weight: 167 N (38 lb)

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Filter Layer Thickness: 0.38 m (1.22 ft)  
Toe Armor Weight: 1,942 N (440 lb)

Revetment design

Crest Height: 2.13 m (7 ft)  
Armor Weight: 1,697 N (384 lb)  
Armor Thickness: 0.80 m (2.64 ft)  
Filter Layer Weight: 167 N (38 lb)  
Filter Layer Thickness: 0.38 m (1.22 ft)  
Toe Armor Weight: 1,942 N (440 lb)

## **B.5 HYDRODYNAMICS AND SEDIMENTATION MODELING**

Hydrodynamic and sedimentation modeling of the existing PIERP was conducted as part of the reconnaissance study by Moffatt & Nichol Engineers (M&N, 2003). Results of that report are described in the following sections.

The Upper Chesapeake Bay Finite Element Model (UCB-FEM) (as developed by Moffatt & Nichol Engineers) was used to characterize the existing hydrodynamic and sedimentation patterns in the vicinity of the existing PIERP. The UCB-FEM was developed based on the USACE finite element hydrodynamics (RMA-2) and sedimentation (SED-2D) models, collectively known as TABS-2 (Thomas and McAnally, 1985). The UCB-FEM model evaluated and predicted areas where erosion and accretion were likely to occur.

Correlation of the hydrodynamic model calibration results relative to NOAA predicted data for tidal elevations and current velocities is generally better than 90 percent. Predicted percent error is typically less than 10 percent for tidal elevations and less than 15 percent for current velocity. These values indicate that the hydrodynamic model is calibrated to acceptable accuracy and performs within allowable error ranges to provide an acceptable representation of hydrodynamic conditions.

SED-2D can be applied to sediment where flow velocities can be considered two-dimensional in the horizontal plane (i.e., the speed and direction can be satisfactorily represented as a depth-averaged velocity), and is useful for both deposition and erosion studies. The program treats two categories of sediment: 1) noncohesive, which is referred to as sand herein; and 2) cohesive, which is referred to as clay. Both sand and clay may be analyzed, but the model considers a single, effective grain size during each simulation. Therefore, a separate model run is required for each effective grain size. Input parameters for the sedimentation model include: settling velocity, water surface elevations, x-velocity, y-velocity, diffusion coefficients bed density, critical shear stresses for erosion, erosion rate constants, and critical shear stress for deposition.

The non-cohesive sediment model was run using 0.1mm (.004 inch) sediment under no-wind conditions. Analysis of results shows negligible sand transport due solely to tidal currents. Modeled non-cohesive sediment transport for existing conditions is negligible for 4- and 13-mph winds for all directions. Sixteen-mph winds, when taken cumulatively with lower wind speeds, account for nearly 90 percent of the yearly wind occurrences and cause significant sediment

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transport for winds from the NNW, N, NNE, S and SW directions, with negligible to moderate sediment transport for winds from other directions.

The cohesive sediment model was run under no wind conditions for a 6-month simulation period at which point the model achieved a dynamic equilibrium (average values and rates remain steady over time). The cohesive sediment model was then run for each of 16 wind directions for wind speeds of 4-, 13-, and 16-mph.

### **B.5.1 Existing Conditions in the Vicinity of the PIERP**

The numerical modeling system used in the reconnaissance study used a database of water depths and bottom material properties to represent the estuarial system. Water depths were represented by nodes located in the horizontal plane, which are interconnected to create elements. The resulting nodal/element network is commonly called a finite element mesh and provides a computerized representation of the estuarial geometry and bathymetry. A detailed description of the model grid, equations, assumptions, input parameters and model calibration is located in *Poplar Island Modifications, Hydrodynamics and Sedimentation Modeling* (M&N, 2003). The finite element mesh used to characterize the existing conditions in the vicinity of the PIERP and the 630-acre northern lateral alignment are shown in Figure B-36.

The UCB-FEM sedimentation model was used to examine transport of non-cohesive and cohesive materials (i.e. sand and clay), which characterize sediment in the vicinity of the PIERP. Examination of model results for both non-cohesive and cohesive sediments indicates that normal tidal currents in the vicinity of the PIERP are insufficient to directly cause sediment suspension and transport. Wind generated waves increase bottom shear stresses significantly and can cause sediment suspension. Various wind speeds were modeled, and 16-mph winds were determined to be the minimum necessary to cause sediment suspension and transport for non-cohesive sediments. Thirteen-mph winds were the minimum necessary to cause substantial sediment suspension and transport for cohesive sediments.

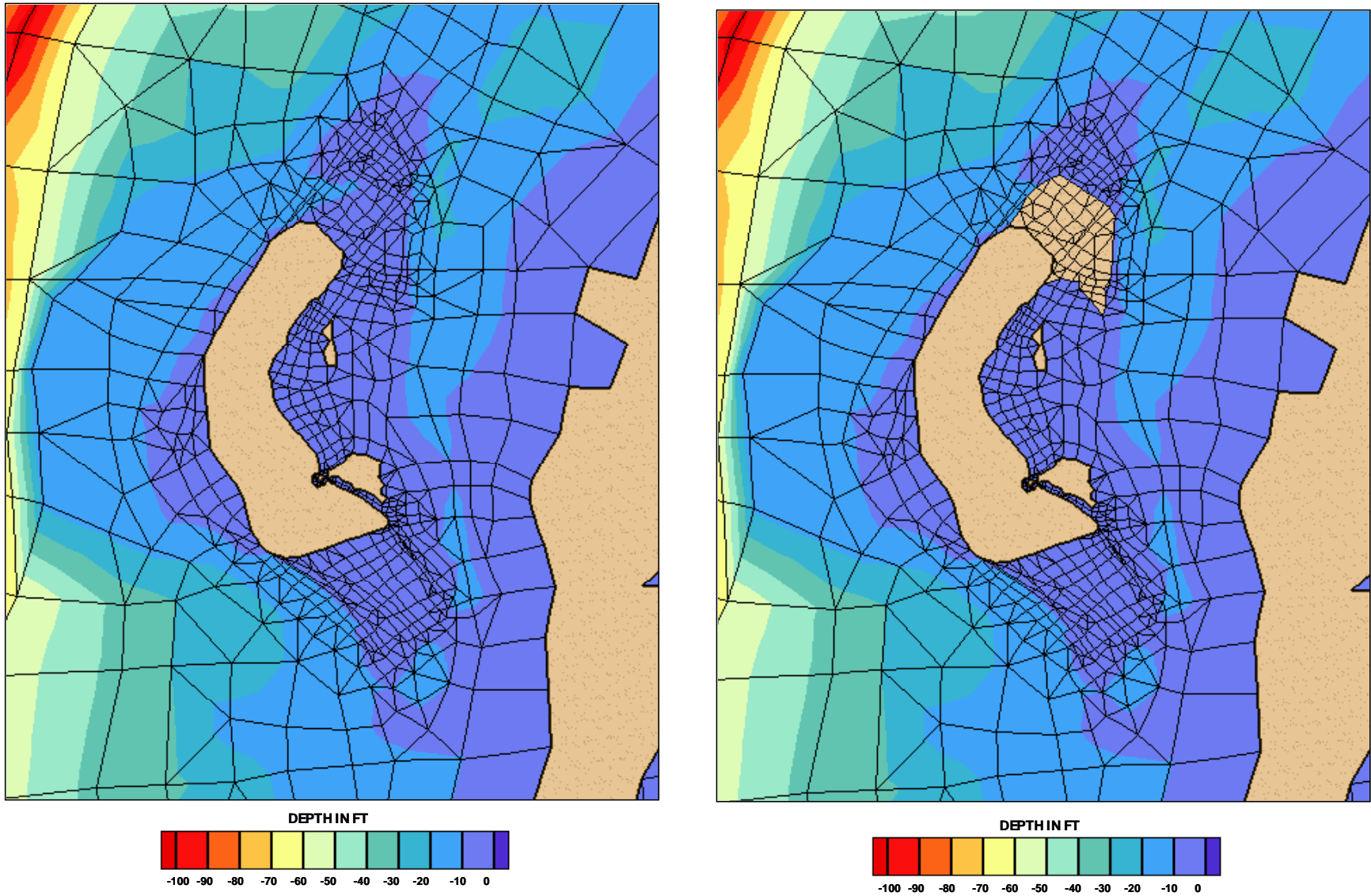
To characterize the sediment erosion and accretion patterns in the vicinity of the PIERP, the UCB-FEM sedimentation model was run for non-cohesive and cohesive sediments for each of 16 wind directions (E, ENE, NE, NNE, N, NNW, NW, WNW, W, WSW, SW, SSW, S, SSE, SE, and ESE) for wind speeds of 4-, 13-, and 16-mph.

Modeled non-cohesive sediment transport in the vicinity of the existing PIERP was negligible for 4- and 13-mph winds for all directions. Sixteen-mph winds, when taken cumulatively with lower wind speeds, account for nearly 90 percent of the yearly wind occurrences and cause significant sediment transport for winds from the NNW, N, NNE, S and SW directions, with negligible to moderate sediment transport for winds from other directions. Model results indicating patterns of accretion and deposition for non-cohesive sediments for 16-mph winds from the NNW, N, NNE, S and SW directions are shown in Figure B-37.

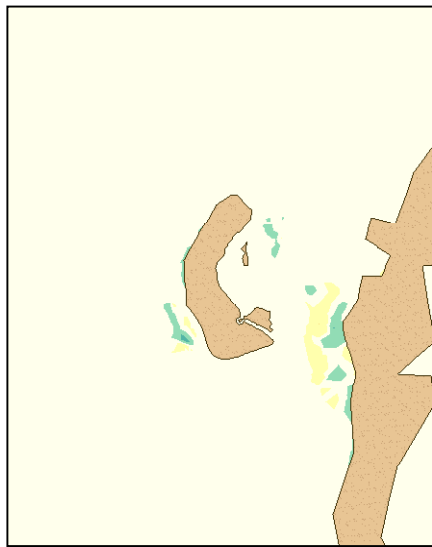
Modeled cohesive sediment transport in the vicinity of the existing PIERP was negligible for 4-mph winds. Thirteen-mph winds cause significant sediment transport for winds from the NNW, N, NNE, NE, S and SW as shown in Figure B-38, with negligible sediment transport for winds

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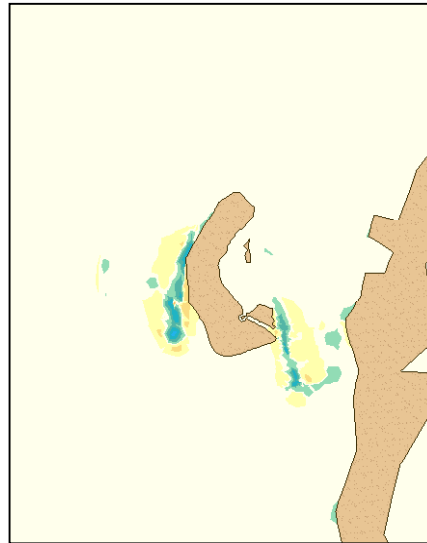
from other directions. Results are shown using a normalized unitless scale due to the empirical use of the sedimentation model and the lack of available data to verify model calibration. In general, for cohesive sediments, the areas of erosion and accretion are larger than for non-cohesive sediment, because the properties of cohesive sediment (shape, plasticity, electric charge) cause the particles to remain in suspension for relatively long periods of time before they settle out.



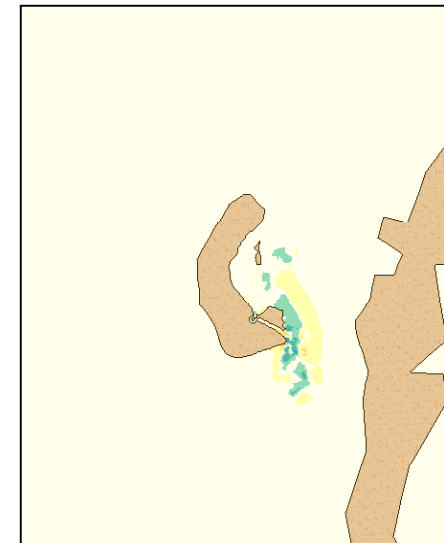
**Figure B-36: The Upper Chesapeake Bay – Finite Element Mesh (UCB-FEM) for the Existing Condition (left) 630-Acre Northern Lateral Alignment (right) (Source: M&N, 2003)**



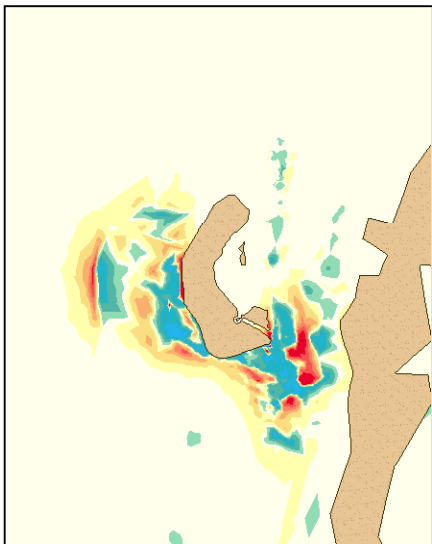
North-Northwest Wind (16 mph)



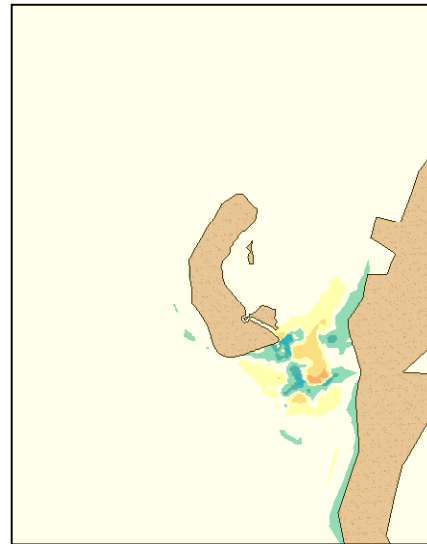
North Wind (16 mph)



North-Northeast Wind (16 mph)



South Wind (16 mph)



Southwest Wind (16 mph)

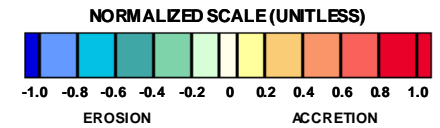
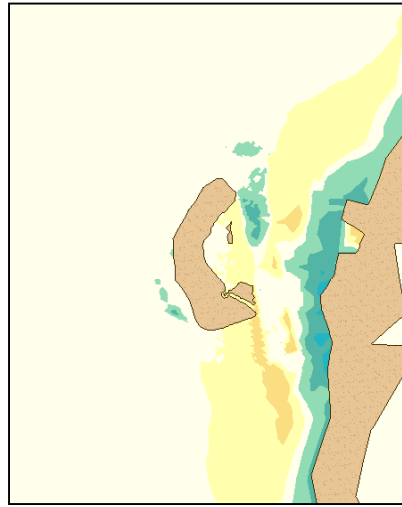
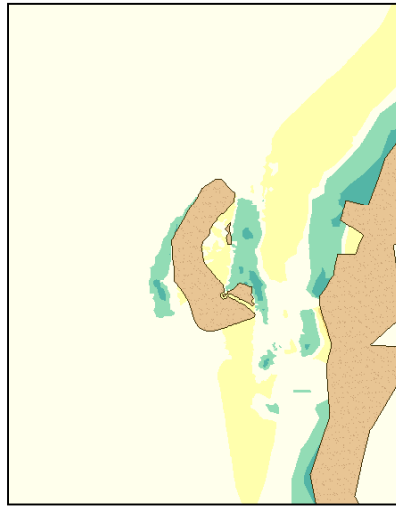


Figure B-37: Non-Cohesive Sediment -- Existing Conditions (Source: M&N, 2003)

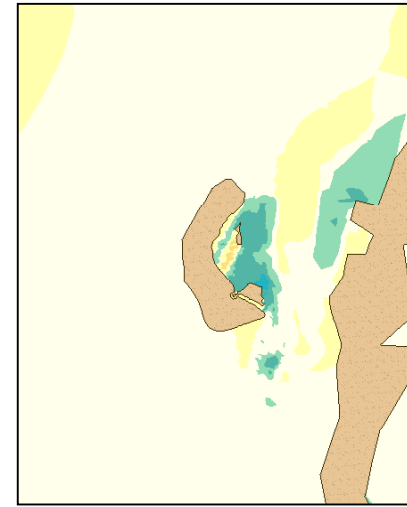




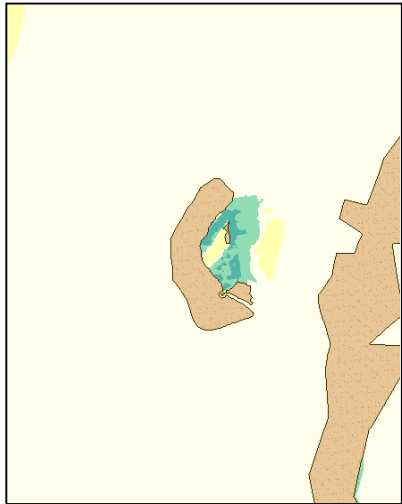
**North-Northwest Wind (13 mph)**



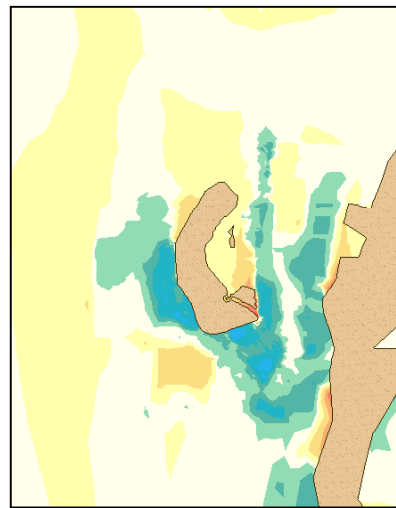
**North Wind (13 mph)**



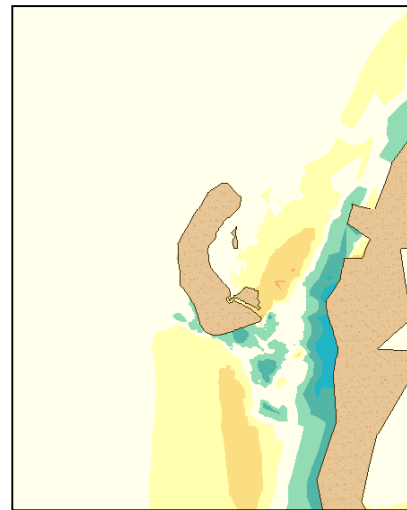
**North-Northeast Wind (13 mph)**



**Northeast Wind (13 mph)**

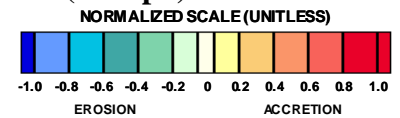


**South Wind (13 mph)**



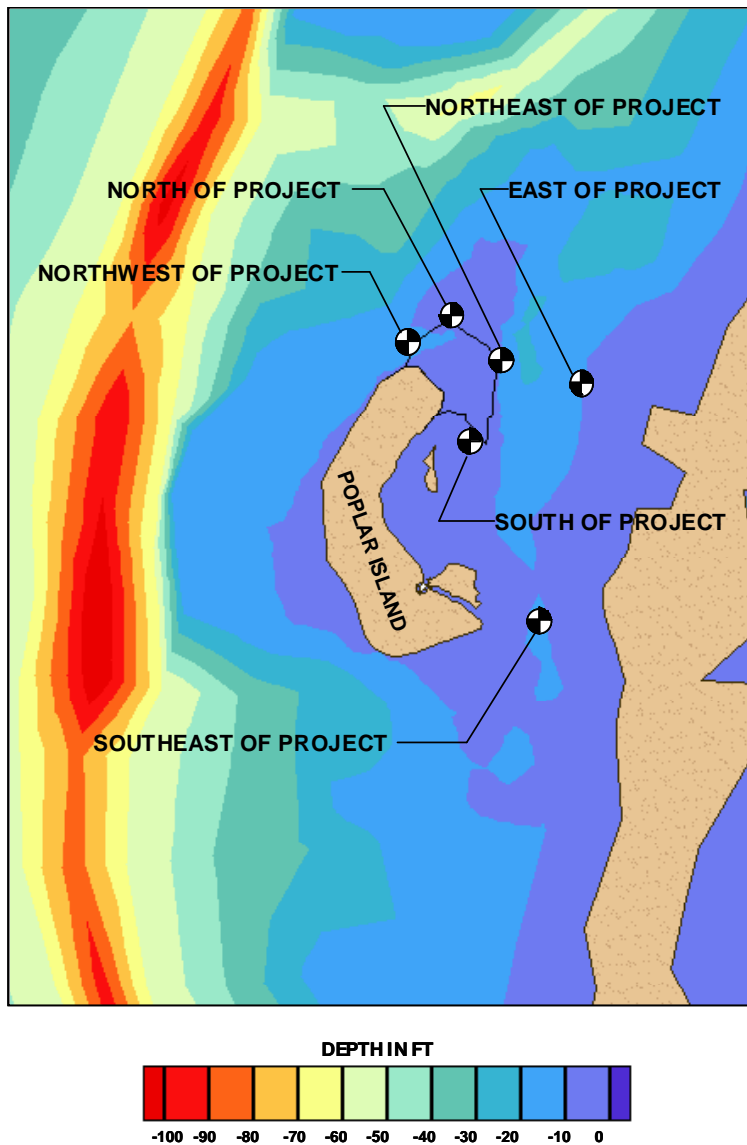
**Southwest Wind (13 mph)**

**FigureB-38: Cohesive Sediment Existing Conditions** (Source: M&N, 2003)



### B.5.2 The Northern Lateral Expansion Alignment

The UCB-FEM model is used to assess impacts by applying identical hydrodynamic input boundary conditions to existing condition and with-project model bathymetry. Hydrodynamic results are then used as input into the sedimentation model, which is also run using identical boundary conditions existing condition and with-project conditions. The input conditions selected represent typical hydrodynamic conditions in the vicinity of Poplar Island. Results of the hydrodynamic simulations are compared numerically at several points around the project site and visually for the entire project vicinity. Figure B-39 shows comparison stations for the 630-acre northern lateral alignment.



**Figure B-39: Results Comparison Locations** (Source: M&N, 2003)

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The UCB-FEM sedimentation model was run for the 630-acre northern lateral alignment to assess erosion and accretion of both non-cohesive and cohesive sediments. Results were normalized to a unitless scale because of the empirical use of the sedimentation model as a result of insufficient local calibration data. Cohesive sediments have properties (shape, plasticity, electric charge) that cause the particles to remain in suspension for relatively long periods of time before they settle out, generally resulting in a larger area affected by sedimentation and erosion from cohesive sediment as compared to non-cohesive sediments.

Hydrodynamic model results indicate that water surface elevations would be unaffected by construction of the 630-acre northern lateral alignment (Figure B-40), and relatively small impacts would occur to current velocities. Figures B-41 and B-42 visually show the differences in peak ebb and flood current velocity, respectively, in the project area due to construction of the project. Following construction of the 630-acre northern lateral alignment, flow would be displaced northward, and current velocity would increase at the northernmost point. Current velocity decreases where flow is blocked by the island, creating an area of increased quiescence to the east, west and immediately south of the 630-acre northern lateral alignment area. During the peak flood tide, shown in Figure B-42, flows are reversed relative to ebb tide currents but patterns of velocity change are similar to those observed for ebb flow conditions.

Currents near the PIERP are on the order of 0.1 to 1.2 ft/sec, and construction of the additional beneficial use area does not significantly change current velocities in the surrounding vicinity. Figure B-43 graphically shows the differences between current velocities at six locations in the vicinity of newly created habitat area.

**Non-Cohesive Sediments.** Figures B-44 through B-45 show sedimentation modeling results for 0.004-inch non-cohesive sediments for 16-mph NNW, N and NNE winds, respectively. Comparison of sedimentation patterns with bathymetry shows that the areas of erosion correspond to shallow water depths while deposition occurs in adjacent deep water areas.

Construction of the 630-acre northern lateral alignment would interrupt the long NNW wind fetch from across the Bay, thereby reducing the rates of erosion and accretion in the lee of the project as shown in Figure B-44. The difference plot in Figure B-44 shows a large area southeast of the project, labeled as both "more sediment" and "less sediment" on the scale, which represents areas that are eroding and accreting under existing conditions and show no sediment transport in the with-project conditions.

Construction of the 630-acre northern lateral alignment would also interrupt a large portion of the long wind fetch from the north, reducing the rates of erosion and accretion southeast of the PIERP as shown in the difference plot of Figure B-45. The region labeled as "more sediment" and "less sediment" on the scale represents areas that are both eroding and accreting, respectively, under existing conditions, and similar to NNW wind conditions show no sediment transport in the with-project conditions.

Figure B-45 shows that construction of the 630-acre northern lateral alignment would interrupt the long wind fetch from the NNE, reducing the rates of erosion and accretion in Poplar Harbor. The difference plot in Figure B-42 shows areas within Poplar Harbor labeled as "more sediment"

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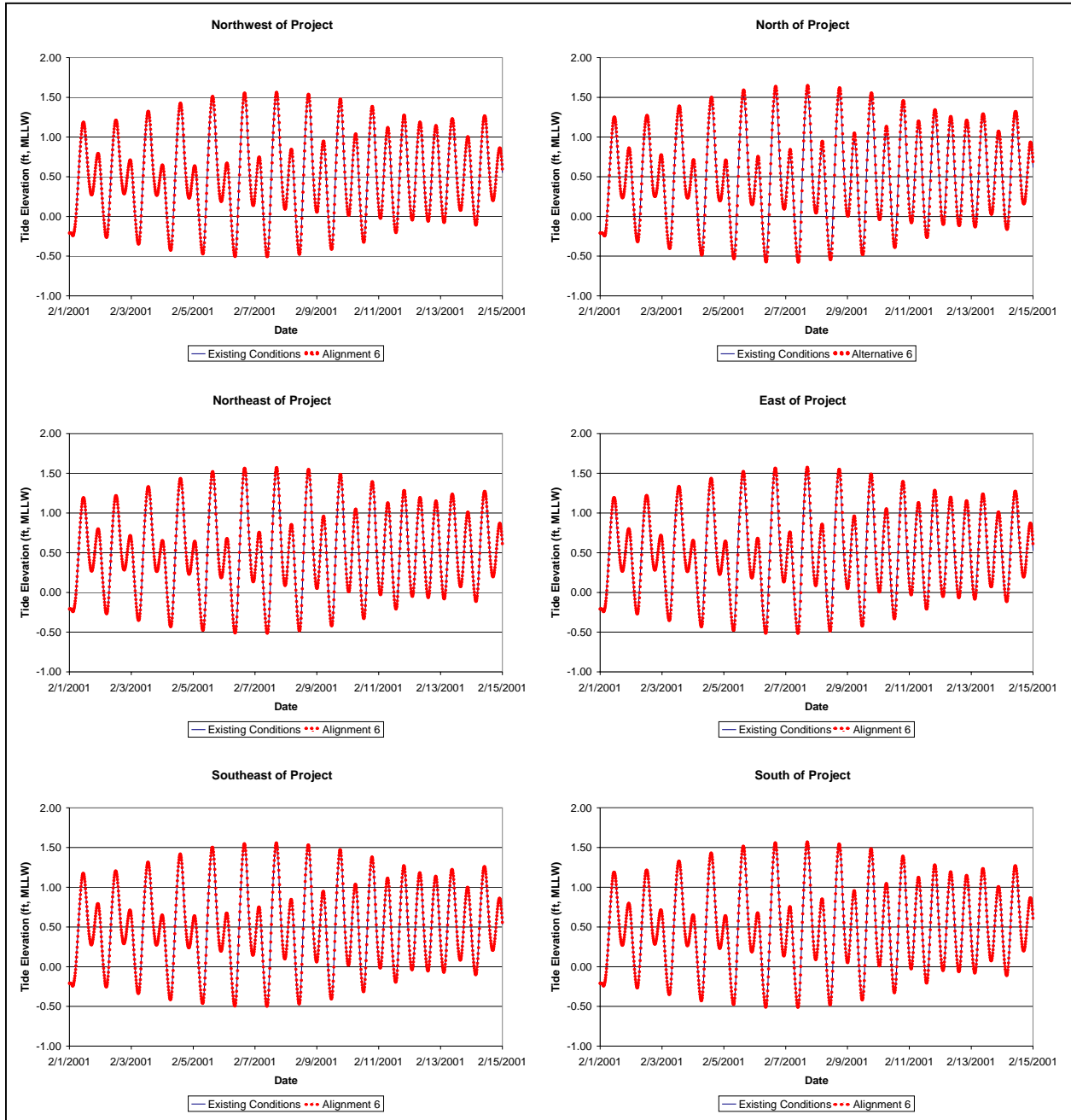
and “less sediment” on the scale, which represent areas that are eroding and accreting, respectively, under existing conditions and show no sediment transport in the with-project conditions.

**Cohesive Sediments.** Figures B-47 through B-50 show sedimentation modeling results for cohesive sediments for 13mph NNW, N, NNE and NE winds, respectively. Figure B-47 shows a virtual absence of sediment movement within Poplar Harbor following construction of the 630-acre northern lateral alignment for NNW winds, and a reduction of sediment movement east and southeast of the island.

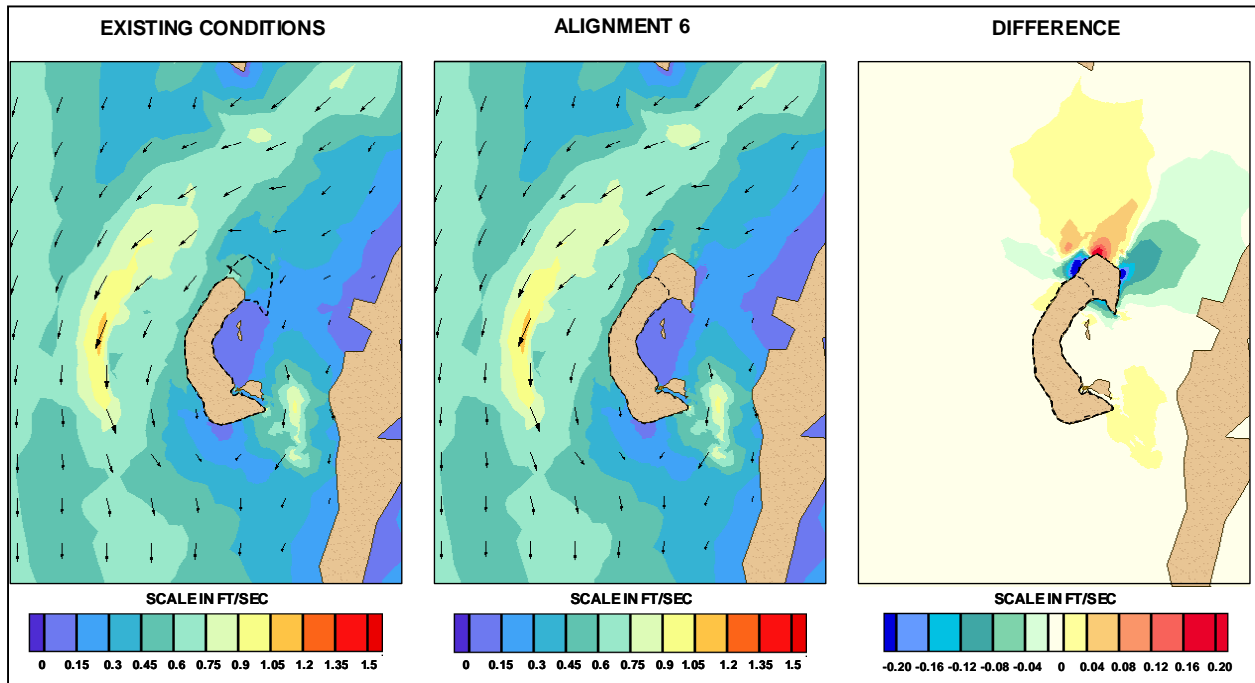
Figure B-48 shows modeling results for 13-mph N winds. This figure shows a potential for increased erosion and decreased accretion north of the 630-acre northern lateral alignment area after construction, because of the increased flow west of the island. The figure also shows decreased erosion of sediment from the shallow areas within Poplar Harbor and decreased deposition in the deeper areas of Poplar Island Narrows east of the 630-acre northern lateral alignment and within the sheltered areas south of the PIERP. Similarly for existing conditions, 13-mph NNE winds (Figure B-49) cause erosion of sediment within Poplar Harbor. Following construction of the 630-acre northern lateral alignment, a large area of Poplar Harbor, including Jefferson Island, is sheltered by the expansion, resulting in decreased erosion of sediment from the shallow areas within Poplar Harbor and decreased deposition in the deeper areas east of Poplar Harbor in the Poplar Island Narrows. Erosion of Jefferson Island is greatly reduced after construction of the 630-acre northern lateral alignment. Modeling results for 13-mph NE winds, Figure B-50, show results similar to NNE winds.

**Model Results for the Northern Lateral Alignment.** Hydrodynamics and sedimentation numerical modeling for the Poplar Island Modifications Reconnaissance Study show that expansion of the PIERP would have minimal impacts on local tidal elevations and current velocities. Tidal elevations would be unchanged, and maximum increase or decrease in current velocity following construction of any alignment would be about 0.2 ft/sec.

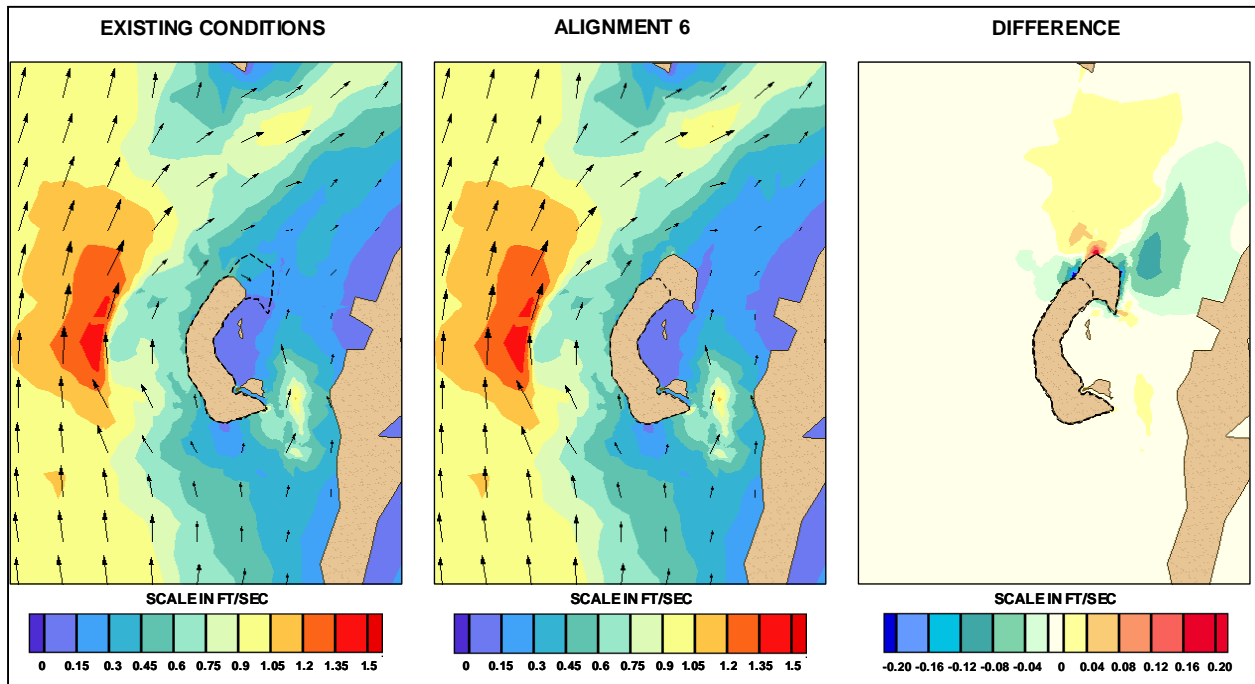
Construction of the 630-acre northern lateral alignment would have a beneficial impact because it would provide shelter to Poplar Harbor from wind and waves coming from the NNW, N, NNE and NE directions, reducing erosion of Jefferson Island and shallow areas of the harbor. This reduction in erosion would likely reduce suspended sediment and improve water clarity within Poplar Harbor.



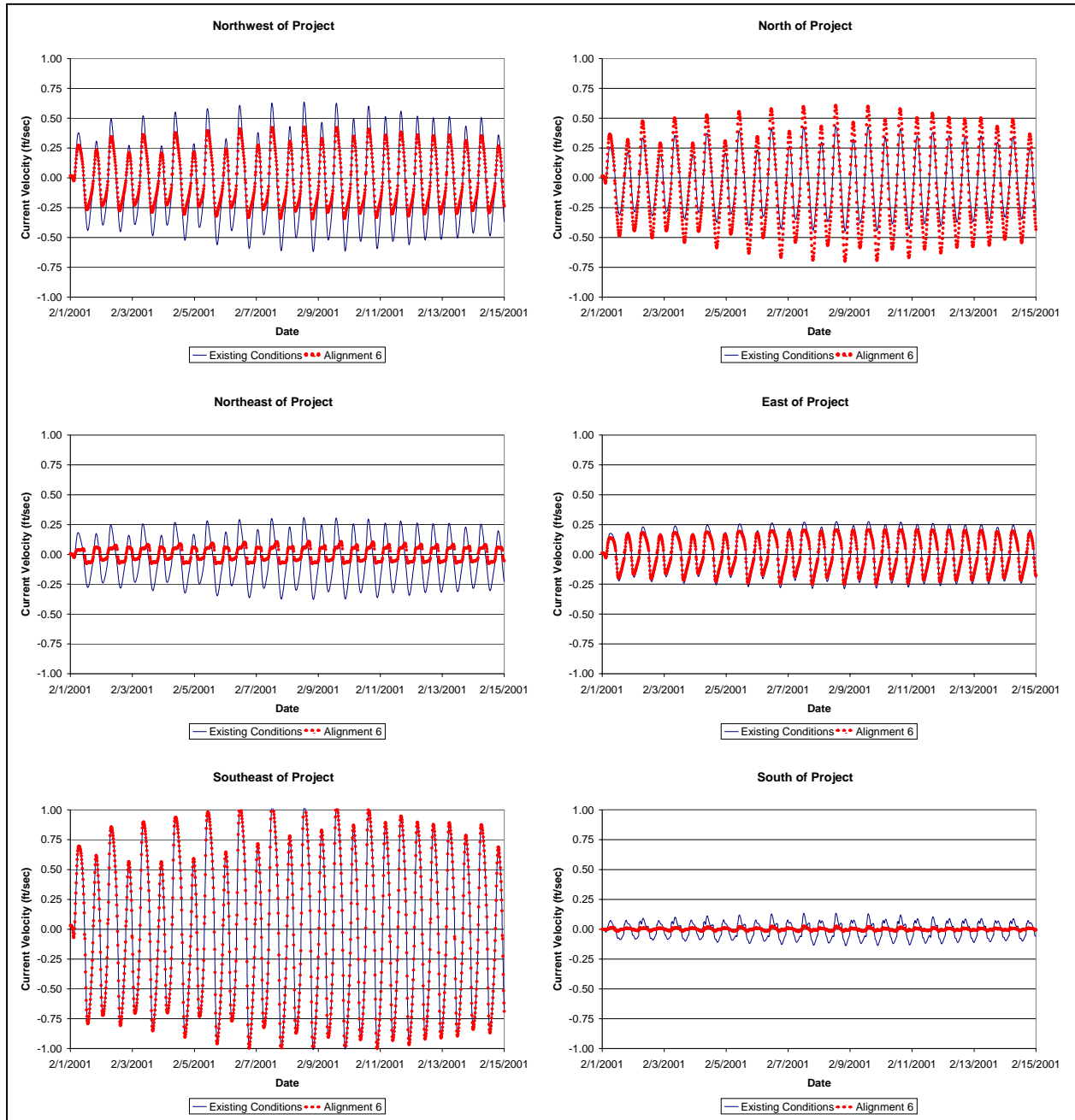
**Figure B-40: Poplar Island Tidal Results Comparison for a 630-Acre Northern Lateral Alignment** (Source: M&N, 2003)



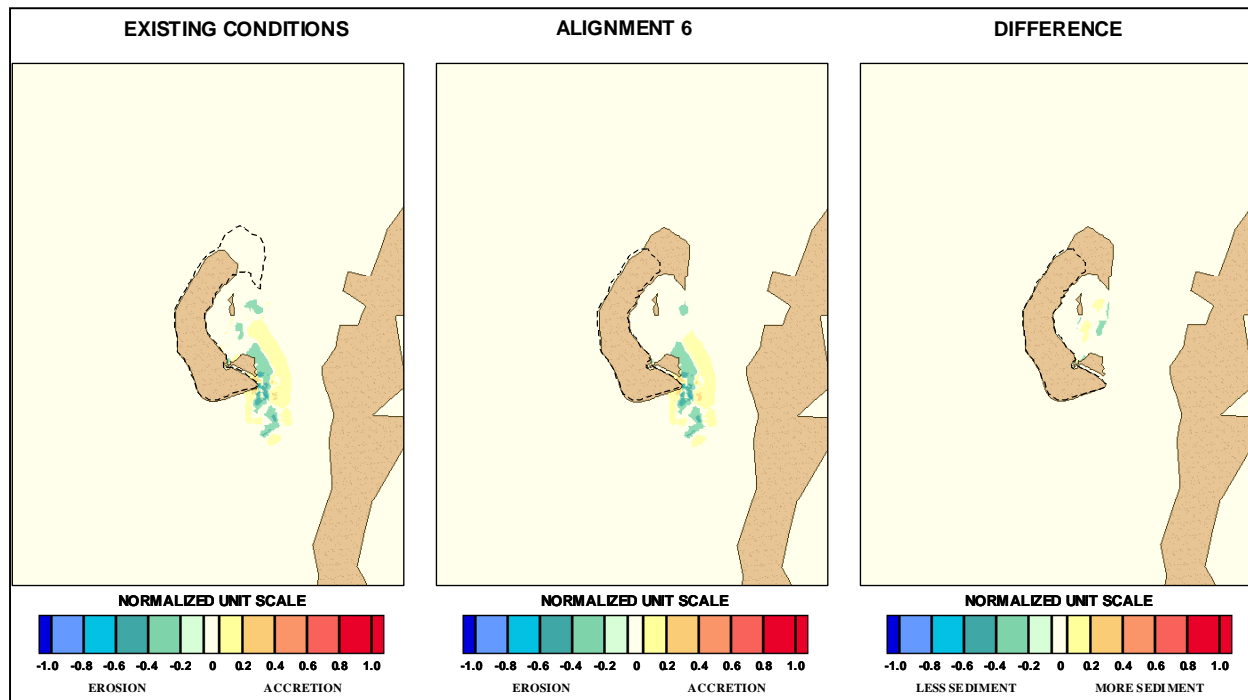
**Figure B-41: Peak Ebb Current Velocity – 630-Acre Northern Lateral Alignment vs. Existing Conditions** (Source: M&N, 2003)



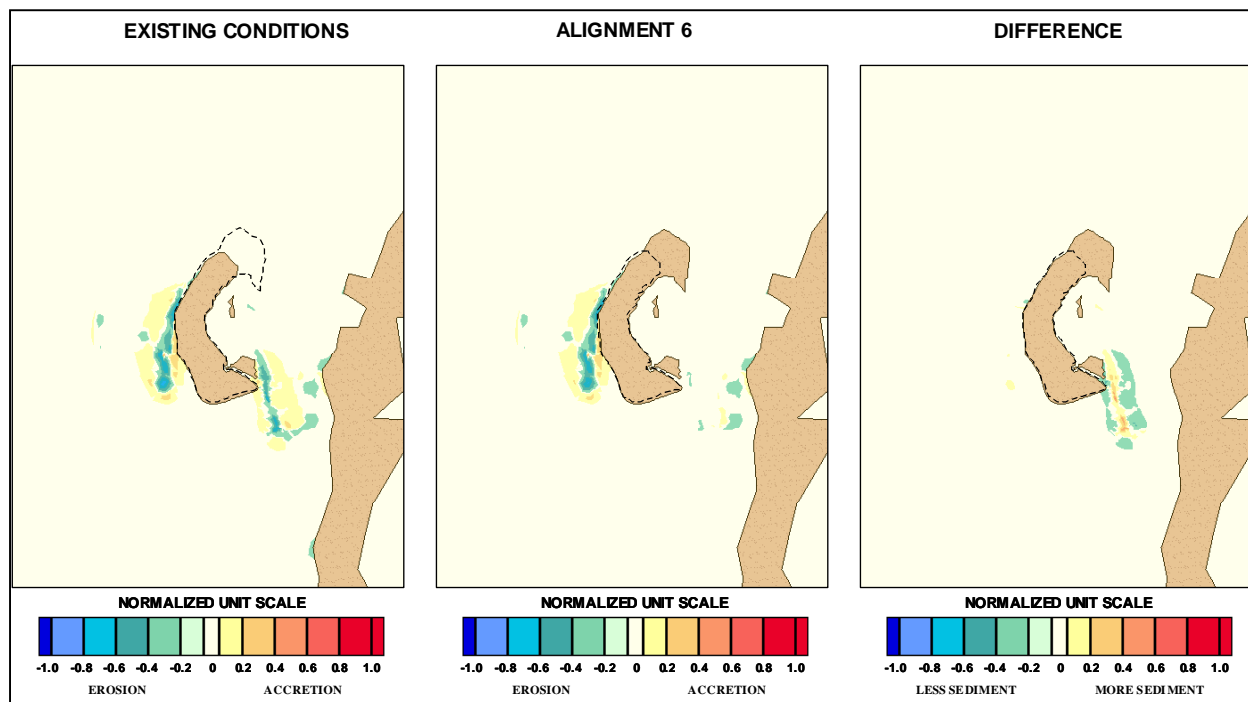
**Figure B-42: Peak Flood Current Velocity – 630-Acre Northern Lateral Alignment vs. Existing Conditions** (Source: M&N, 2003)



**Figure B-43: Poplar Island Current Velocity Results Comparison for a 630-Acre Northern Lateral Alignment** (Source: M&N, 2003)

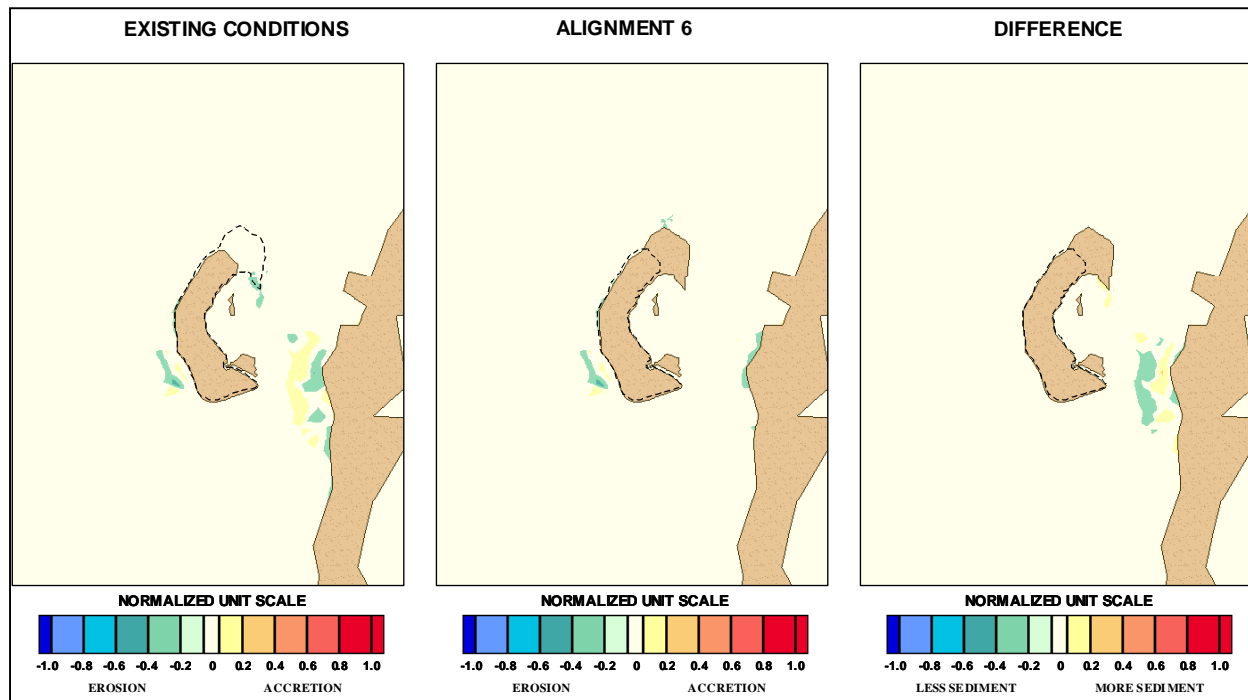


**Figure B-44: Non-Cohesive Sediment – North-Northeast Wind 16 mph – 630-Acre Northern Lateral Alignment vs. Existing Conditions** (Source: M&N, 2003)

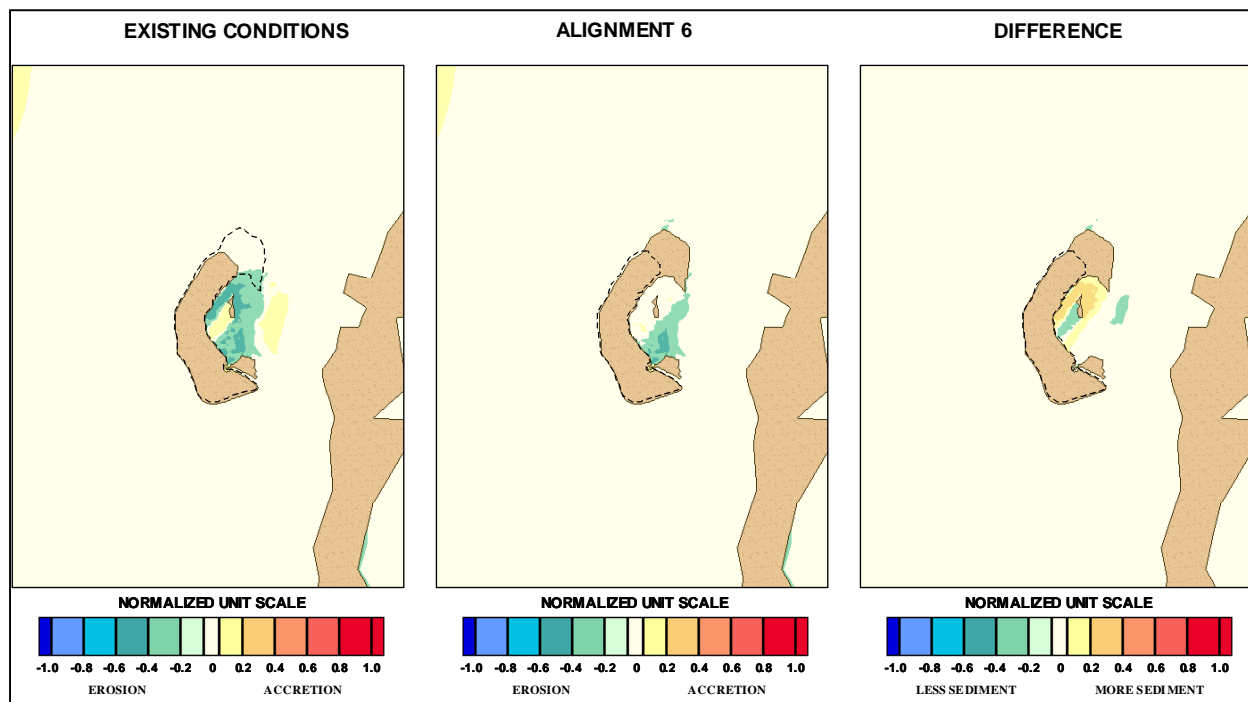


**Figure B-45: Non-Cohesive Sediment – North Wind 16 mph – 630-Acre Northern Lateral Alignment vs. Existing Conditions** (Source: M&N, 2003)

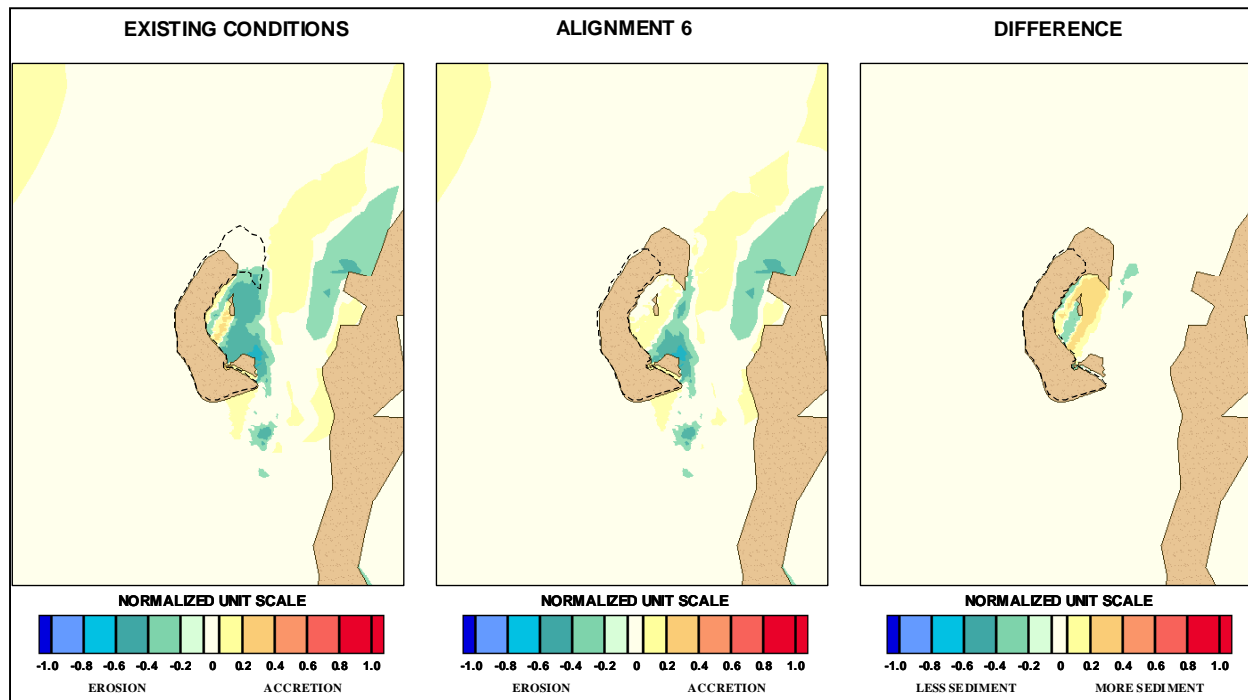




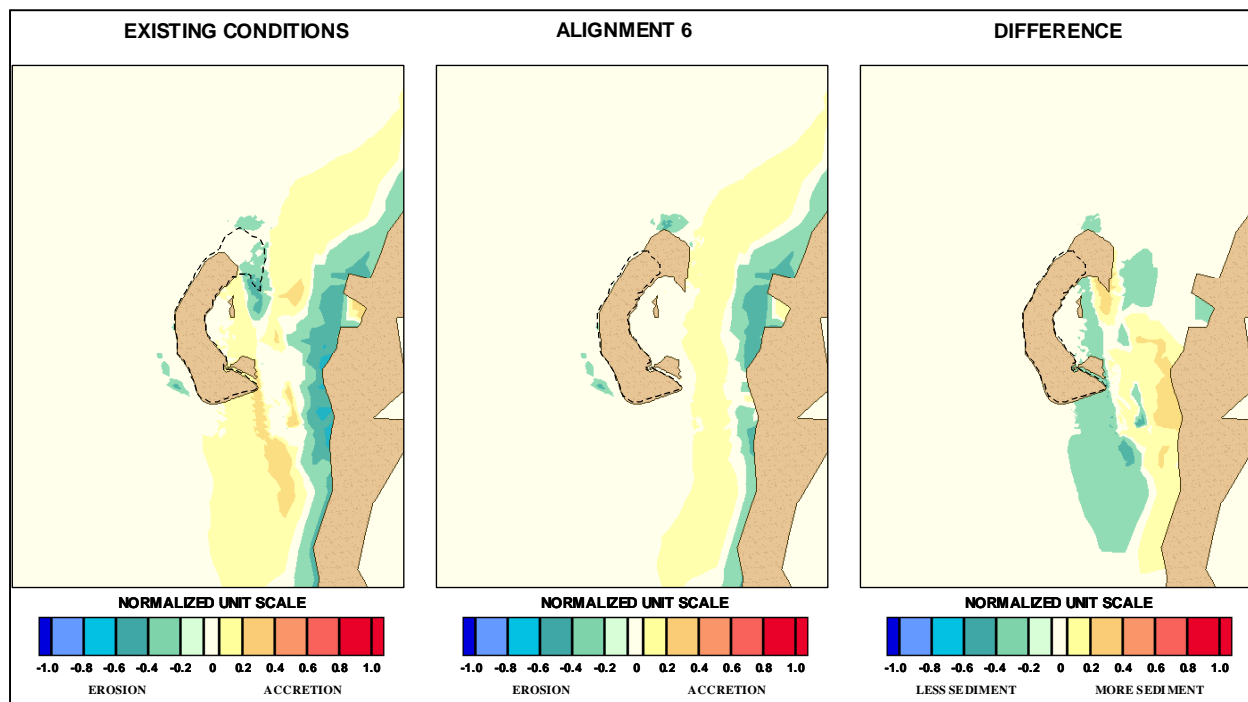
**Figure B-46: Non-Cohesive Sediment – North-Northwest Wind 16 mph – 630-Acre Northern Lateral Alignment vs. Existing Conditions** (Source: M&N, 2003)



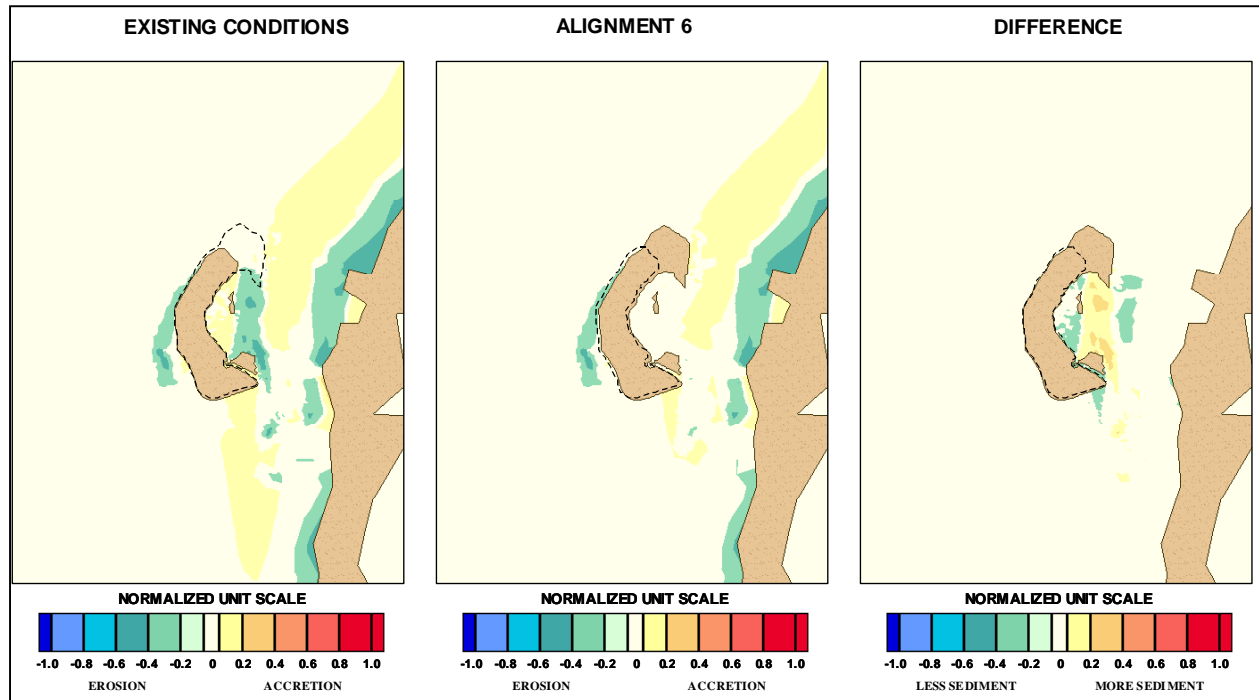
**Figure B-47: Cohesive Sediment – Northeast Wind 13 mph 630-Acre Northern Lateral Alignment vs. Existing Conditions** (Source: M&N, 2003)



**Figure B-48: Cohesive Sediment – North-Northeast Wind 13 mph – 630-Acre Northern Lateral Alignment vs. Existing Conditions** (Source: M&N, 2003)



**Figure B-49: Cohesive Sediment – North Wind 13 mph – 630-Acre Northern Lateral Alignment vs. Existing Conditions** (Source: M&N, 2003)



**Figure B-50: Cohesive Sediment – North-Northwest Wind 13 mph – 630-Acre Northern Lateral Alignment vs. Existing Condition** (Source: M&N, 2003)

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## REFERENCES

- Demirbilek, Z., Zundel, A., and Jones, R. (2004). "Particle Tracking Model (PTM) in the SMS: IV. Tutorial - sample sediment sources," DOER Technical Report, U.S. Army Engineer Research and Development Center, Vicksburg, MS.
- Kalnay, E., *et al.* 1996: The NCEP/NCAR 40-year Reanalysis Project. *Bulletin, American Meteorological Society*, 77(3), 437-471.
- Lin, L., Cialone, M. A., and Bass, G. P. (in press). "Hurricane Isabel Storm Surge Simulation for Chesapeake Bay," Chesapeake Research Consortium Publication 05-160, Heritage Press, Edgewater, MD.
- Moffatt & Nichol Engineers (M&N). 2004. *Poplar Island Environmental Restoration Project, Morphological Modeling*. Prepared for Maryland Port Administration. December.
- Moffatt & Nichol Engineers (M&N). 2003. *Poplar Island Modifications, Hydrodynamics, and Sedimentation Modeling. Final Report January 22, 2003*. Report submitted to Maryland Port Administration and Maryland Environmental Services.
- Smith, J. M. 1991. "Wind-wave Generation on Restricted Fetches," Miscellaneous Paper CERC-91-2, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS, 25 pp.
- Smith, J. M., Sherlock, A. R. and Resio, D. T. 2001. "STWAVE: Steady-State spectral Wave Model User's manual for STWAVE, Version 3.0," ERDC/CHL SR-01-1, US Army Corps of Engineers Engineer Research and Development Center, Vicksburg, MS.
- Swail, V.R., Ceccacci, E.A., and Cox, A.T., 2000. "The AES40 North Atlantic Wave Reanalysis: Validation and Climate Assessment," The 6th International Workshop On Wave Hindcasting and Forecasting, Monterey, California. 1-15.
- Thomas, W.A., W.H. McAnally, Jr. and S.A. Adamec, Jr. 1985. A User's Manual for the Generalized Computer Program, Sediment Transport in Unsteady, 2-Dimensional Flow, Horizontal Plane (STUDH).
- U.S. Army Corps of Engineers, Engineer Research and Development Center (USACE-ERDC), Coastal and Hydraulics Laboratory (CHL). 2005a. *Poplar Island Expansion Flushing Study*. Memorandum for the Record. Prepared for USACE-Baltimore District. 23 May 2005.

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U.S. Army Corps of Engineers, Engineer Research and Development Center (USACE-ERDC), Coastal and Hydraulics Laboratory (CHL). 2005b. *Shoreline Impact Study for Poplar Island Expansion*. Memorandum for the Record. Prepared for USACE-Baltimore District. 17 February 2005.

U.S. Army Corps of Engineers, Engineer Research and Development Center (USACE-ERDC), Coastal and Hydraulics Laboratory (CHL). 2005c. *Life Cycle Analysis of Mid Bay and Poplar Island Projects, Chesapeake Bay, Maryland*. Prepared for USACE-Baltimore District. July. Draft.