POPLAR ISLAND ENVIRONMENTAL RESTORATION PROJECT: COASTAL ENGINEERING DESIGN AND CONSTRUCTION

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ABSTRACT

The Poplar Island Environmental Restoration Project is one of the success stories regarding the development of a cost-effective and environmentally acceptable dredged material placement site for the Port of Baltimore. The project involved the restoration of an eroded island located along the eastern shore of the Chesapeake Bay in the U.S. This paper addresses the coastal engineering aspects of the planning, design and construction of the project. Specific issues addressed in the paper include: (1) detailed design of the containment dikes with explanation of the optimization design procedures used; (2) physical model testing of the containment structures; (3) construction methodology and costs for the containment structures; and (4) numerical modeling of the impacts of the island development on hydrodynamics, water quality and sedimentation. The paper will also evaluate the performance of the dikes and armor protection in response to high winds and water levels resulting from Hurricane Isabel that passed through the region from September 18, 2003 through September 20, 2003. The importance of this paper is that it serves to summarize the design methods, design, costs and hydrodynamic impacts of one of the largest dredged material placement projects in the United States.

Keywords: Coastal engineering, dike design, modeling, hydrodynamics, optimization

INTRODUCTION

The project consists of the reconstruction of tidal wetland and upland habitats by making a beneficial use of dredged material removed from the southern Bay approach channels to the Port of Baltimore. The purpose of this paper is to present the coastal engineering aspects of the project. Emphasis is placed on factors that govern the design of the perimeter dikes and the physical impacts of the island footprint on areas in and around Poplar Island. This paper presents a description of the selected alignment, the components of the dike design, the results of the dike design optimization analysis, a reliability analysis of the design, the results of physical model test for the design, and an evaluation of hydrodynamic conditions at the site.

The objectives of this beneficial use site include an optimization of the volumetric capacity of the site for dredged material, preparation of a cost-effective design within available funding, restoration of Poplar Island to approximately its 1847 footprint, creation/restoration of desirable habitat, and design of all aspects of the site in an environmentally acceptable manner (GBA-M&N JV 1995). Figure 1 shows the location of the Poplar Island project area.

Several site alignments were examined jointly through a series of discussions with the Maryland Port Administration (MPA), the U.S. Army Corps of Engineers, Baltimore District (USACE), the Maryland Environmental Service (MES) and the Poplar Island Working Group (PIWG). The optimal island orientation was selected based on wave exposure, water depth, and protection to adjacent water and land. A 450 hectare alignment was selected as the proposed project; a cost optimization analysis was performed to revise the alignment to a more cost-effective alternative, which was further revised during construction due to geotechnical considerations.

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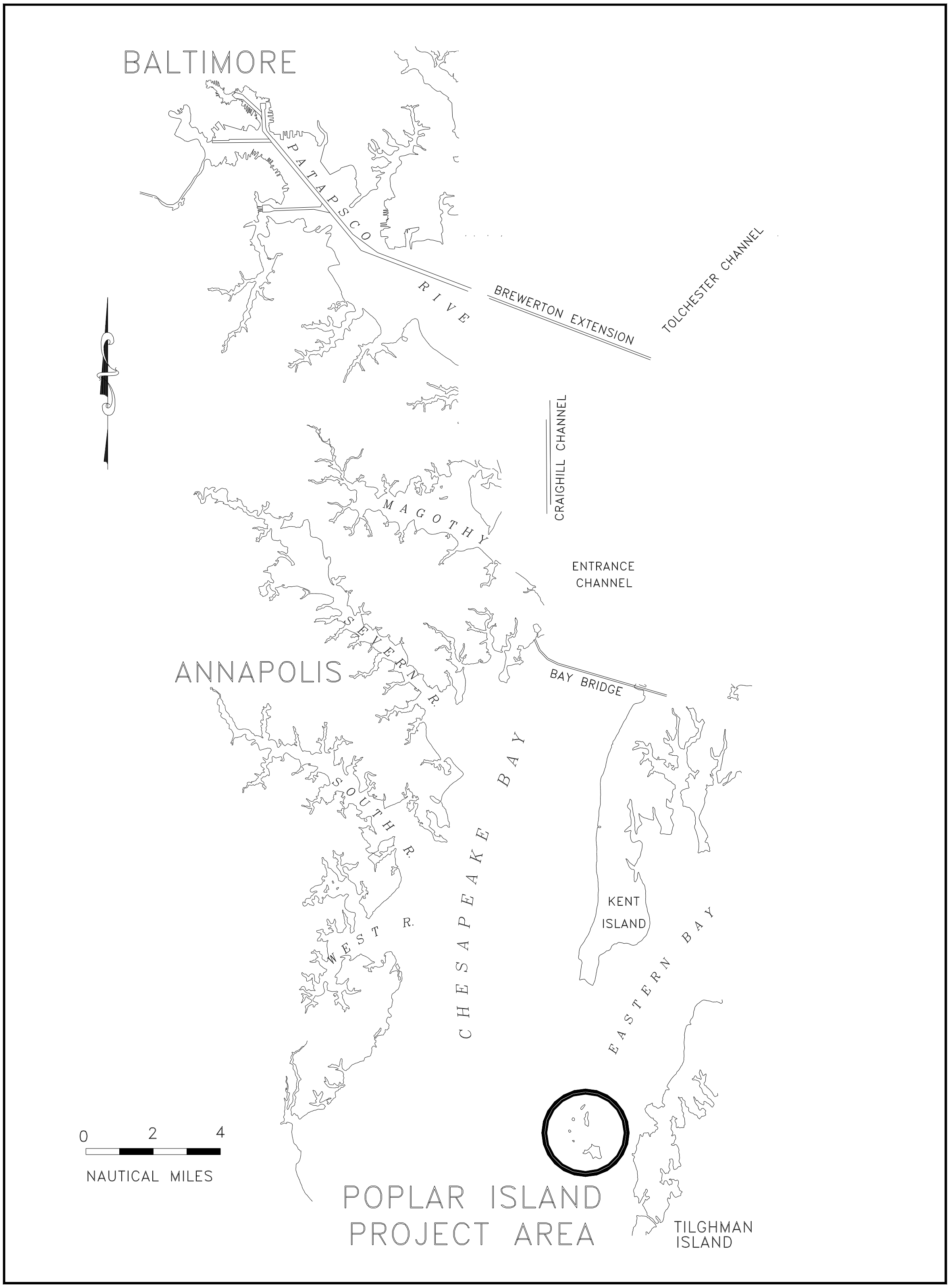

Figure 1. Poplar Island project area.

CONTAINMENT DIKE DESIGN METHODOLOGY

Environmental site conditions that are relevant to the design were evaluated to provide design parameters. These are summarized below.

Bathymetry

Water depths within the project area range from 0.61 to 3.66 m below Mean Lower Low Water (MLLW). The project area is generally flat with slopes on the order of 1:300 to 1:500. Water depths along the proposed eastern dike perimeter in Poplar Harbor are between 0.46 and 1.07 m. Water depths along the proposed western dike perimeter are between 1.52 and 3.05 m. The average depth of water within the project area is approximately 2.13 m.

Wind Conditions

Design winds for the site were developed on the basis of data collected at Baltimore-Washington International (BWI) airport from 1951 through 1982 (32 years). These winds, which can exceed 40 m/sec (90 mi/hr) during a 100-year storm, were used to develop design wave conditions. Predominant wind direction is from the northwest. Table 1 presents the design wind speeds used for the wave hindcast.

Table 1. Design wind speeds (m/sec) per direction and return period.

Return Period	Direction							
	N	NE	E	SE	S	SW	W	NW
5	18	17	14	17	16	21	22	24
10	21	20	17	20	19	25	24	26
25	26	25	21	26	24	31	27	30
50	31	30	25	31	28	37	29	33
100	36	34	29	37	33	43	31	36

Water Levels

Normal water levels at the site are dictated by astronomical tides which have a mean range of 0.55 m from MLLW to Mean Higher High Water (MHHW). Extreme water levels are dictated by storm surge, which is the temporary rise in water level generated either by large-scale extra-tropical storms known as northeasters, or by hurricanes. The rise in water level results from wind action, the low pressure of the storm disturbance and the Coriolis force. Wave setup is a term used to describe the rise in water level due to wave breaking.

A comprehensive evaluation of storm-induced water levels for several Chesapeake Bay locations was conducted by the Virginia Institute of Marine Science (1978) as part of the Federal Flood Insurance Program. Results of this study are summarized in the following Table 2 for selected locations. The closest station locations to Poplar Island are Matapeake on Kent Island, approximately 20.9 km due north, and Chesapeake Beach on the western shore of the Bay, approximately 16.1 km southwest. In the absence of other data, it was assumed that the storm tides for Poplar Island are the mean values of these two locations; the 100-year water level is about 2.13 m MLLW.

Table 2. Water level elevation per return period for Chesapeake Bay locations(m, MLLW).

Return Period (Years)	Baltimore	Annapolis	Chesapeake Beach	Matapeake	Solomons Island	Cambridge
10	1.36	1.33	1.17	1.33	1.14	1.30
50	2.18	2.00	1.69	2.00	1.57	1.66
100	2.58	2.30	1.97	2.30	1.78	1.91
500	3.37	2.97	2.51	2.91	2.24	2.39

Wave Conditions

Poplar Island is exposed to wind-generated waves approaching from all directions. The longest fetch distances to which the site is exposed correspond to the north and south directions, thus the highest waves approach from these directions. Hindcast waves were computed to indicate that 25-year return period waves from the north direction have a significant height (H_s) of 2.2 m and a peak spectral wave period (T_p) of 5.2 seconds. From the south direction, the 25-year return period H_s and T_p are 2.1 m and 5.4 seconds, respectively. Table 3 presents the offshore significant wave heights in the project vicinity, and Table 4 presents the peak spectral wave periods.

Table 3. Offshore significant wave heights (m) per direction and return period.

Return Period (yr)	N	NE	E	SE	S	SW	W	NW
5	1.5	1.0	0.5	0.5	1.5	1.4	1.3	1.5
10	1.8	1.2	0.5	0.6	1.7	1.6	1.4	1.6
25	2.2	1.5	0.7	0.8	2.1	2.0	1.6	1.8
50	2.6	1.8	0.8	1.0	2.5	2.4	1.7	2.0
100	3.0	2.0	1.0	1.2	2.9	2.8	1.9	2.2

Table 4. Peak spectral wave period (sec) per direction and return period.

Return Period (yr)	N	NE	E	SE	S	SW	W	NW
5	4.5	3.6	2.3	2.4	4.6	4.1	3.9	4.1
10	4.8	3.9	2.4	2.6	4.9	4.4	4.0	4.3
25	5.2	4.2	2.6	2.9	5.4	4.7	4.2	4.5
50	5.6	4.5	2.8	3.1	5.7	5.0	4.3	4.6
100	5.9	4.8	3.0	3.3	6.0	5.4	4.5	4.8

These wave heights represent deep water conditions some distance offshore of the dikes, which are located in relatively shallow water. The specific water depths fronting dike sections varies for dikes facing the northwest and southwest quadrants are located in low water depths of 1.5 to 3.0 m. Dikes facing the northeast and southeast quadrants are located in low water depths of 0.6 to 1.2 m. Given the relatively shallow depths fronting the dikes, these structures are exposed to some breaking waves. Thus wave heights are reduced, and nearshore significant wave heights from the north and south quadrants range from about 1.4 m for a 5-year storm to about 1.9 m for a 100-year storm. Similarly, nearshore significant waves from the southeast range from less than 0.7 m for a 5-year event to about 1.1 m for a 100-year event. Maximum depth limited or breaking waves from the north and south range from 2.4 m for a 5-year storm to 3.5 m for a 100-year storm. The maximum southeast breaking wave heights range from 1.2 m for a 5-year event to 1.8 m for a 100-year event. The nearshore significant wave heights (i.e. at the dikes) are shown in Table 5.

Table 5. Nearshore significant wave heights (m) per direction and return period.

Return Period (yr)	N	NE	E	SE	S	SW	W	NW
5	1.4	1.3	0.7	0.8	1.4	1.3	1.3	1.3
10	1.4	1.3	0.8	0.8	1.5	1.4	1.3	1.4
25	1.5	1.4	0.9	0.9	1.6	1.5	1.4	1.5
50	1.7	1.6	1.0	1.1	1.7	1.6	1.6	1.6
100	1.8	1.7	1.1	1.2	1.9	1.8	1.7	1.7

Currents

Tidal currents in the vicinity of Poplar Island are relatively weak (less than 0.3 m/sec) Construction of the project will change current patterns and circulation in the vicinity of Poplar, Coaches and Jefferson Islands comparable to conditions circa 1847.

Soil Conditions

Soil types at the site consist of four basic stratums. Stratum 1 is a surficial silty sand. Stratum 2 is a soft to hard silty clay. Stratum 3 is a stiff silty clay with pockets of sand. Stratum 4 is a very soft gray silty clay. A sizable pocket of silty fine sands, with 0 to 2 m of silty clay overburden, was encountered in the southern portion of the site, adjacent to Coaches Island. A stratum of surficial, very soft silty clay was encountered northeast of the site. A pocket of cemented sands (ironite) was encountered west of South Central Poplar Island.

DIKE DESIGN OPTIMIZATION ANALYSIS

The project required the construction of a perimeter dike both to contain dredged materials as they are placed and to provide protection from wave action for the developed habitats. Interior dikes have been constructed to separate upland and tidal wetland habitat and to partition the site into manageable cells. Both the perimeter and interior dikes were constructed of sand borrowed from within the site alignment. Perimeter dikes are protected from wave attack by rock slope protection on the exposed portions with an armored toe dike to provide additional protection during and after construction. A plan view of the constructed island is shown in Figure 2.

Initial construction costs for the project site are primarily the dike construction costs. Accordingly, a detailed cost optimization analysis was conducted to develop cost-effective designs for both the Western Perimeter Dike (dike segment exposed to waves from the north, west and south) and the Eastern Perimeter Dike (dike segment exposed to the relatively low-energy waves from the east). The purpose of the dike optimization analyses was to select return periods for waves and water levels that provide for an optimal balance between initial capital and maintenance/repair costs. Additionally, the analyses addressed the selection of the optimal structure side slope.

The optimization analysis consisted of estimating damage for three dike components: (1) wave-induced primary armor layer damage, (2) armor layer damage due to wave runup and overtopping, and (3) wave-induced toe armor damage. Wave-induced damage refers to the removal of stone from the armor layer by direct wave attack. Damage associated with overtopping occurs as the dike rock layers are undermined by overtopping waves. It is important to note that the optimization analyses only considered damage to the primary armor layer. Damage to the underlayer stone and sand core were not included in the analyses. Figure 3 shows a schematic of the principal components and the geometry of the dike used for the analyses.

Sixty design cross sections were analyzed for the dike design. The design cross sections correspond to ten return periods (5, 10, 15, 20, 25, 30, 35, 40, 50, and 100 years) and six side slopes (1.5:1, 2.0:1, 2.5:1, 3.0:1, 3.5:1 and 4.0:1). Computations were performed for each of the sixty design cases using methods published by de Waal and van der Meer (1992) for runup and overtopping and van der Meer (1988) for armor stone. The required crest elevation was based on allowable overtopping rates published in CIRIA/CUR (1991).

The cost optimization analysis indicates that the optimal structure slope for the perimeter dike ranges from 3:1 to 4:1. Example optimization results are shown in Figure 4. Overall, the optimal design return period for the Western Perimeter Dike is about 35 years, however, the optimal return period for the primary armor stone is 25 years. The optimal design return period for the armored eastern dike is about 50 years. Similarly, the optimal return period for the design of the eastern dike armor stone is 50 years.

Figure 5 shows the typical dike section design for the western side of the island based on the results of the optimization analysis. Specific design criteria for the western dike are a crest elevation of 3.2 MLLW, 1.4 metric ton armor stone, 3:1 side slope, 0.9 metric ton toe stone and a sand core. Figure 6 shows the typical dike section design for the eastern side of the island. Specific design criteria for the eastern dike are a crest elevation of 2.4 m MLLW, 110 kg armor stone, 3:1 side slope, and a sand core.

An important consideration for design is that during the life of the structure there is a risk of dike damage and/or failure. A reliability, or probability, analysis provides a means to assess the risk of damage to the various dike elements throughout the course of the project life. The reliability analysis shows that the structure has more than a 90% chance that it will suffer damage that will require maintenance over the 100-year design life. This finding is to be expected and has been incorporated into the optimization analysis and long-term maintenance costs for the project presented in this report.

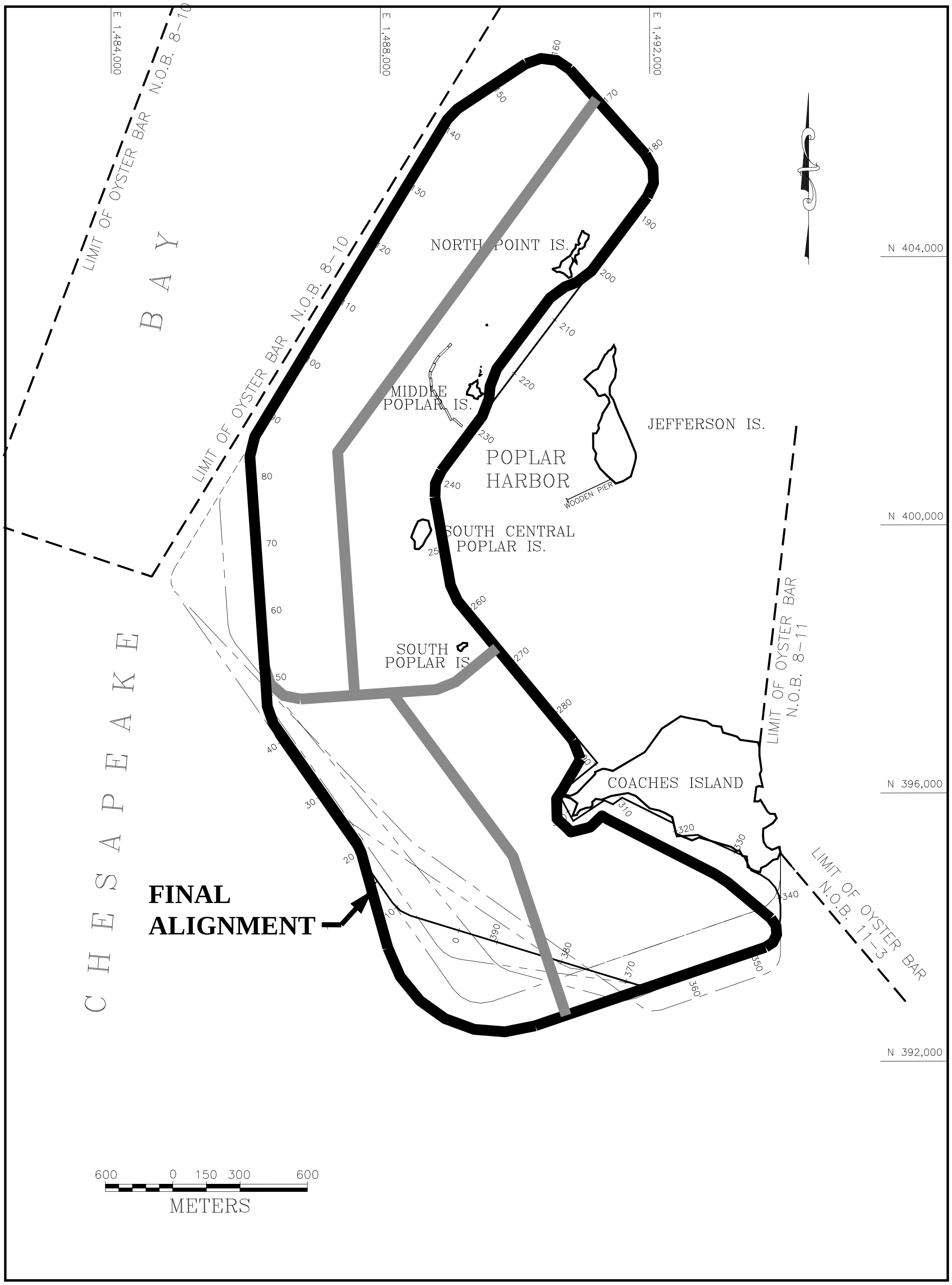

Figure 2. Project alignment analysis and final constructed alignment.

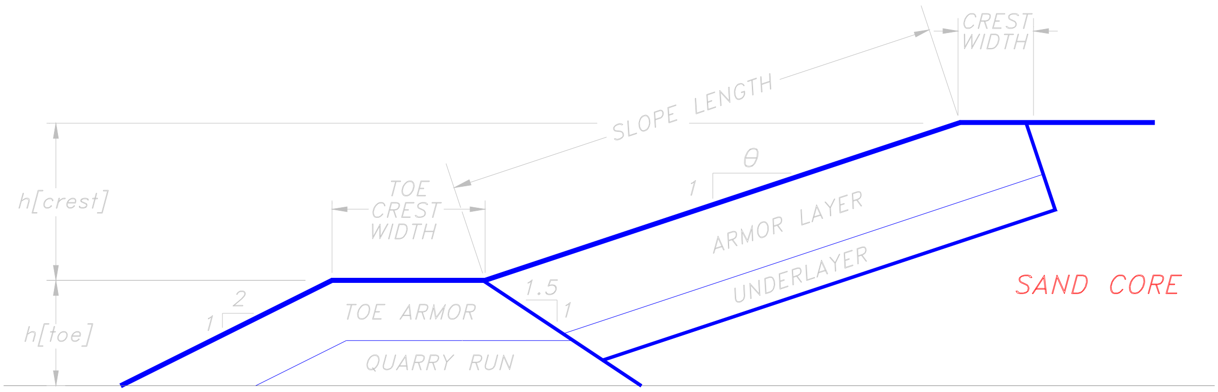

Figure 3. Schematic dike section for optimization analysis.

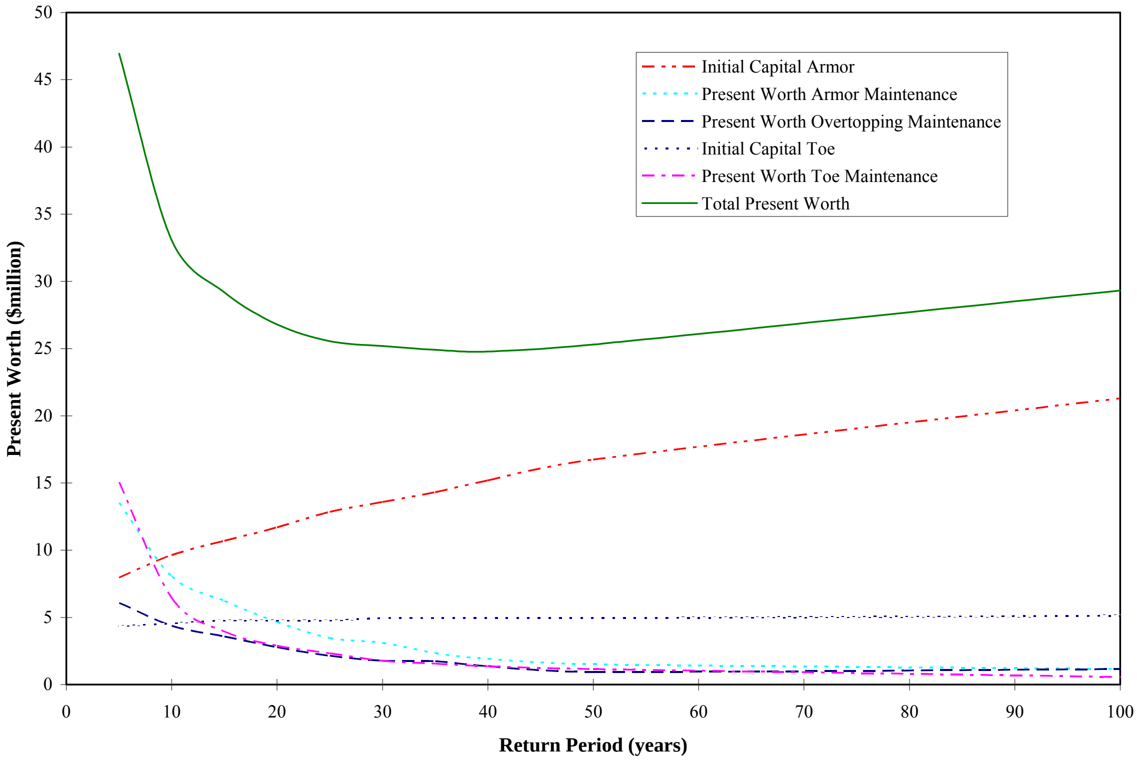

Figure 4. Optimization results.

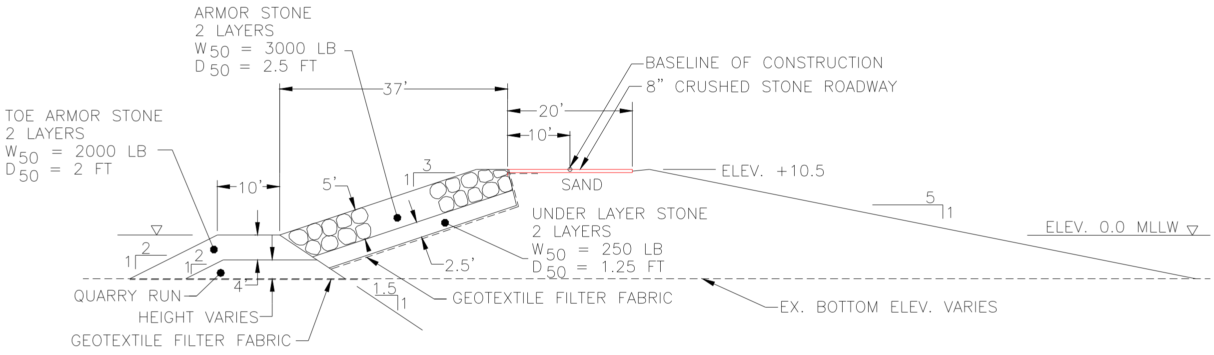

Figure 5. Typical western dike section.

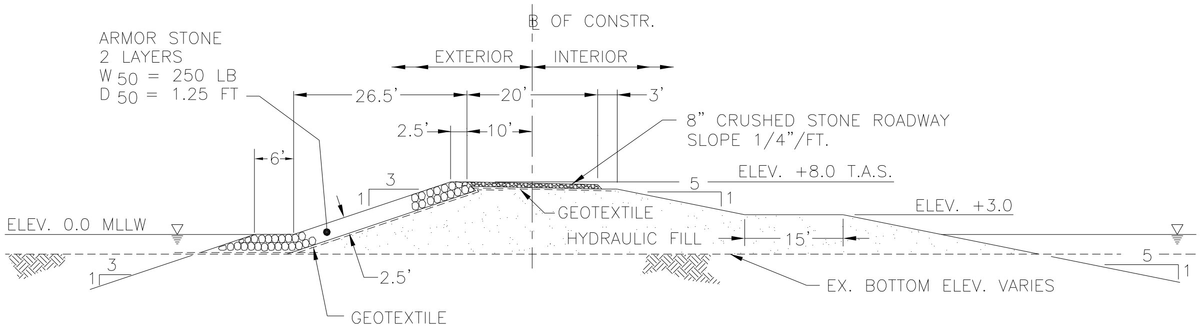

Figure 6. Typical eastern dike section.

PHYSICAL MODEL TESTING

Physical model testing was conducted to assess the hydraulic stability of the dike. Data obtained from the physical model testing were used to verify and finalize the design. Pertinent data included measurement and verification of the proposed armor stone size and gradation to evaluate hydraulic stability and construction methodology, measurement of the significant and maximum wave height at the structure, measurement of the volume of water overtopping the structure for a given return period water level, observance of rock movement and/or rock displacement, and video tape recordings of the testing events. Model testing was conducted using a 10 to 1 geometric scale for 25-year, 50-year and 100-year return period storm events.

The hydraulic modeling facilities at the University of Delaware, Center for Applied Coastal Research, Ocean Engineering Laboratory were used for the physical model testing program. The facilities include an approximately 110 cu m tank having dimensions of 30 m long, 2.44 m wide and 1.5 m deep, hydraulically-driven random wave generator, test measurement equipment, computers, software, utilities and miscellaneous hardware necessary for model construction. Maximum design conditions allowable in the tank are 1.2 m water depth and 0.6 m wave height which limited the largest practical scale to a 10:1 ratio. Rock, sand, textiles, wood and other miscellaneous materials were used to construct the scale model. Table 6 presents the model scale factors and dimensions and Figure 7 shows a schematic layout of the model test setup. The random wave generator is a piston-type wave paddle driven by a hydraulic system and controlled by computer. The wave generator has a 1.75 m stroke to allow for generation of solitary waves and wave adsorption capability, and the hydraulic cylinder is 4 m long. A controlling program RANDWGEN converted a displacement time series to a digital voltage time series that was subsequently converted to a continuous analog signal which was then transmitted to the hydraulic actuator. A calibration curve of paddle displacement vs. voltage signal was developed to provide a means for adjustment of the wave generation during the testing program.

Table 6. Model scales based on Froude Law for 7 ft water depth section.

Item Description	Scale	Ratio	Prototype Dimension	Model Dimension
Dike height	l_r	1/10	17.5 ft	1.75 ft
Dike width (toe to toe)	l_r	1/10	171 ft	17.1 ft
Water depth	$h_r = l_r$	1/10	11.84 ft	1.18 ft
Wave height	$H_r = l_r$	1/10	5.1 ft	0.51 ft
Wavelength	$L_r = l_r$	1/10	95.67 ft	9.57 ft
Wave period and time	$T_r = (l_r)^{1/2}$	$1/\sqrt{10}$	5.4 sec	1.71 sec
Weight per unit length (a)	$w_r = l_r^2$	1/100	10.5 ton/ft	210 lb/ft
Weight of armor unit (a)	$W_r = l_r^3$	1/1000	1.5 ton	3 lb
Size (D_{50}) of armor unit (a)	$d_r = l_r$	1/10	2.7 ft	3.25 inches
Overtopping amount per wave per unit length	$Q_r = l_r^2$	1/100	27 liter/m (2.16 gal/ft)	0.27 liter/m (0.022 gal/ft)
Overtopping rate per unit length	$q_r = l_r^{3/2}$	$1/\sqrt{1000}$	5 liter/m/sec (0.4 gal/ft/sec)	0.16 liter/m/sec (0.013 gal/ft/sec)

(a) Assumes that specific gravity of material for both the prototype and the model are identical

Wave data were collected using several wave gages mounted above the tank at selected locations in the experimental setup. Gages were calibrated by recording free surface displacement vs. voltage; calibration curves were prepared similarly to those made for the wave generator. One critical gage placement was a three-gage array located at the toe of the dike section to confirm that the wave height corresponds to the design wave height and to also measure wave reflectance from the dike structure. Another placement location was at the start of the slope to measure the offshore wave height. Gages were calibrated daily prior to testing.

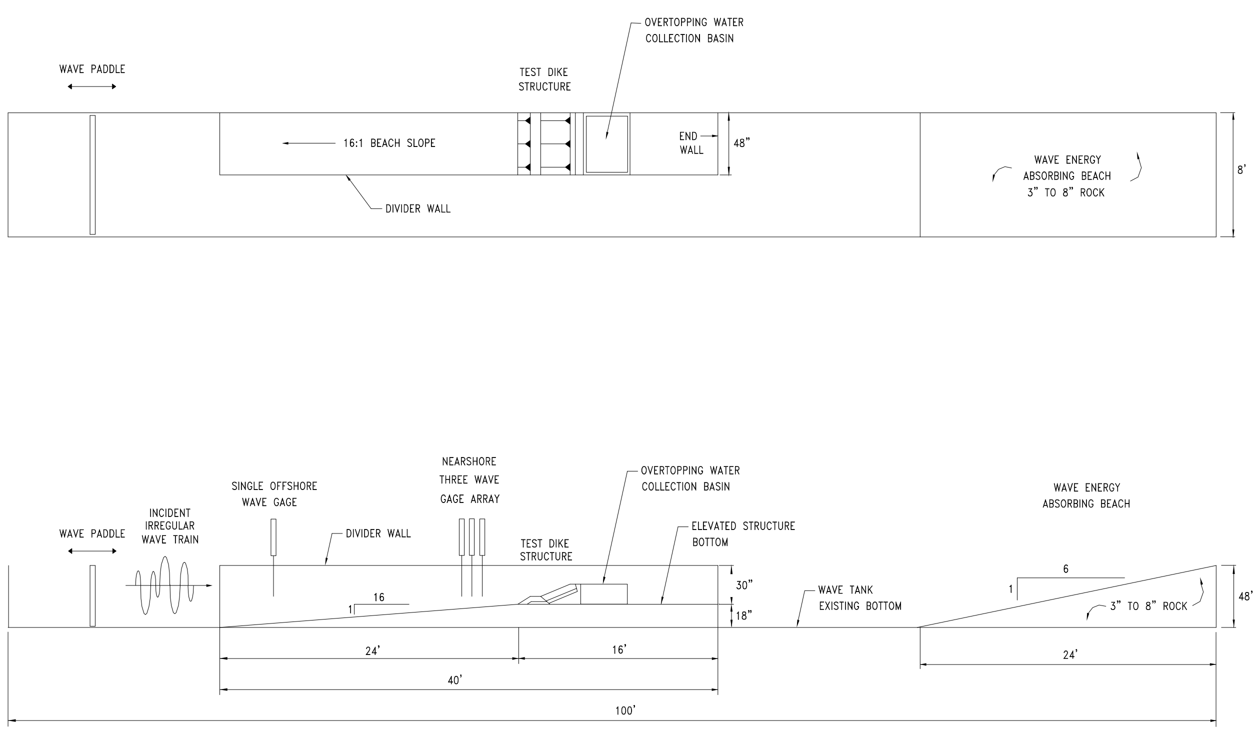

Figure 7. Physical model testing schematic arrangement.

During all test runs, some movement of the rocks was observed. This movement consisted of rocking of the stones as sufficiently large waves broke and/or passed over the individual stone. Rocking of the stones was limited to those that were placed without sufficient contact against adjacent rocks to prevent movement, and for the most part was not significant throughout most of the structure. Due to the relatively flat slope of the structure, no rocks were observed to fall out of the section. Thus, results of the physical model test confirm the armor stone size selected for the dike design.

Overtopping of the structure for the 25-year return period test cases primarily consisted of splashing as waves broke on the structure side slope. On rare occasions, as when relatively large waves were grouped together, waves for this test overtopped and washed over the structure crest. For the 50-year and 100-year return period test cases, overtopping of the structure consisted primarily of washing over the structure crest from both breaking and surging waves. This phenomenom is primarily due to the increased water level (i.e. storm surge) during these tests. Significant overtopping was observed when several large waves were grouped together. It was judged that considerable damage would occur to the structure as a result of this overtopping.

The results also show that the crest height is adequate for the optimized design section, and that considerable overtopping will be associated with the higher water levels (i.e. storm surge) that will occur during larger return period (less frequent) storm events. Figure 8 presents a graph of the overtopping data collected for the study that shows this trend. This figure shows that overtopping volume for the 25-year event tends toward the minimum predicted whereas for the 100-year event the overtopping volume tends toward the maximum predicted. These results were used during dike design to specify higher crest elevations.

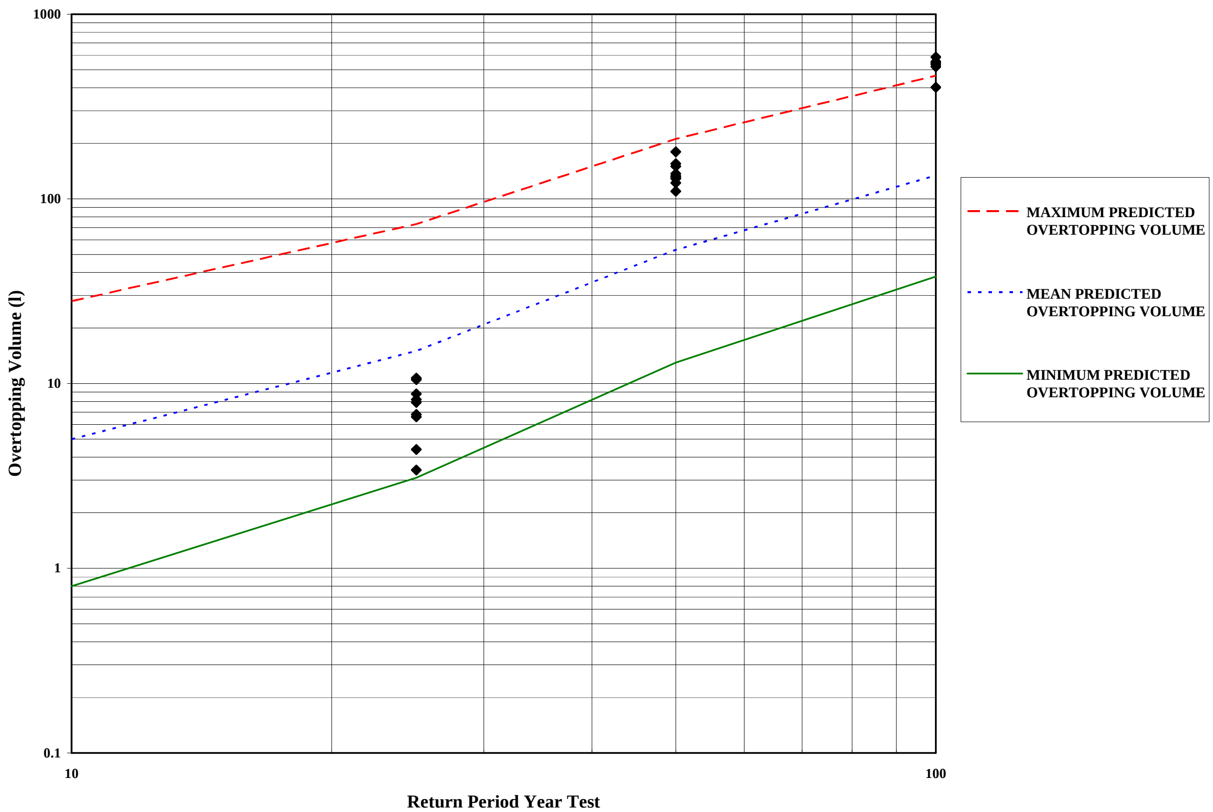

Figure 8. Measured overtopping volume compared to predicted for dike section in 7 ft water depth.

CONSTRUCTION METHODOLOGY AND COSTS

The containment dikes for the dredged material were constructed in two phases. The first phase covered an area of 260 hectares and included construction of over 7.6 km of armored perimeter (exterior) dike, over 3.4 km of uarmored interior dike, and a breakwater over 0.7 km in length between the Phase I island and Coaches Island to protect Poplar Harbor. The breakwater was redesigned during construction as an armored exterior dike for Phase II. Material quantities for major components of Phase I included over 2.4 million cubic meters of sand fill, 280,000 square meters of geotextile, and over 450,000 metric tons of rock for armor layer, underlayer, and roadway. Phase I construction of the Poplar Island Restoration Project was awarded to Kiewit Construction, Inc. for $45.4 million in January 1998; the construction was completed in March 2000.

In September 1998, construction of the rock toe dike began using backhoes which removed the stone from supply barges and placing the stone In mid-October 1998, hydraulic dredging of the borrow areas began with placement of the material into the stockpile area. Dredging was performed by Great Lakes Dredge & Dock Co. using the 76 cm diameter cutter suction dredge *Illinois*. Average production was about 23,000 cubic meters per day. The sand was excavated from the stockpile with backhoes, placed into dump trucks, hauled to the end of the dike, dumped and spread into place with dozers. Dike sand shaping and rock placement on the slope began in December 1998. Hydraulic equipment using rock grabs were effective to achieve the specified armor stone special placement, where the outer layer of rock was placed with the long axis perpendicular to the slope (see Figure 9).

Phase II construction was awarded in April 2000 to Tidewater Construction Corp. for $37.5 million; construction was completed February 2002. This phase covered the remaining 190 hectares of island area. This phase included construction of about 4.6 km of armored perimeter dike and about 2.4 km of unarmored interior dike. Material quantities for major components of Phase II included 1.3 million cubic meters of sand fill, 150,000 square meters of geotextile, and over 250,000 metric tons of rock for armor layer, underlayer, and roadway. Similar to Phase I, the borrow material was dredged hydraulically and placed into a stockpile. The sand was then excavated from the stockpile with backhoes, placed into dump trucks, hauled to the end of the dike, dumped and spread into place with dozers. The sand was subsequently shaped and the rock was placed with a hydraulic rock grab using special placement where the outer layer of rock was placed with the long axis perpendicular to the slope.

Figure 9. Armor stone placement using hydraulic rock grab.

HYDRODYNAMIC NUMERICAL MODELING

Applications of tidal hydrodynamics, constituent transport and sedimentation were developed for the Poplar Island area and its environs. The purpose of the modeling effort was to assess the impacts of island development on tidal flows, residence times and sedimentation in the vicinity of the island. The numerical modeling system used in this study was the US Army Corps of Engineers hydrodynamics (RMA-2), constituent transport (RMA-4) and sedimentation (STUDH) models - TABS-2. The models require that the estuarial system be represented by a network of nodal points (i.e. points defined by coordinates in the horizontal plane and water depth) and elements (i.e. areas made up by connecting adjacent nodal points). Nodes can be connected to form 2-D (3 or 4 nodes) or 1-D elements (2 nodes). The resulting nodal/element network is called a finite element mesh and provides a computerized representation of the estuarial geometry and bathymetry. Figure 10 presents the finite element mesh used for the modeling effort.

Figure 10. Finite element mesh.

Changes in tidal current flow from existing conditions are as would be expected. Specifically, the presence of the restored island has several impacts. First, the waters presently flowing through the island complex are forced to travel around the island footprint. This tendency reduces flows within Poplar Harbor and increases flow on the exterior edges of the island footprint. During flood flow, water that passes through Poplar Harbor under existing conditions will split in the vicinity of the southernmost point of the proposed dike alignment due south of Coaches Island. After construction, this split flow is then trained along the southwest dike of the island restoration and the southern and eastern shorelines of Coaches Island. The increases in flow velocities relative to existing conditions are on the order of 3 cm/sec, which is relatively small and not significant (less than 10 percent change). A small portion of the flow continues to travel through a tidal channel created as part of the project; this flow is on the order of 3 cm/sec and provides enough flushing to maintain ambient water quality without causing erosion of the shoreline within the channel.

Flow during ebb splits at the northern end of the restored island and is trained along the northwest dike and the eastern portion of Coaches Island. Ebb flows fronting the northwest dike are increased about 3 cm/sec relative to existing conditions. Flow velocities on the eastern shoreline of Coaches Island are increased 0 to 3 cm/sec relative to existing conditions.

Insofar as the purpose of this project is to restore Poplar Island, the differences in hydrodynamic conditions for the proposed alignment and those associated with the 1847 footprint have been examined. The results show that conditions associated with the Poplar Island area hydrodynamics are similar to that which existed in 1847. In this sense, the proposed island restoration can be viewed as a restoration of historical hydrodynamic conditions. Figure 11 shows velocity vectors for peak ebb and peak flood conditions after construction of Poplar Island.

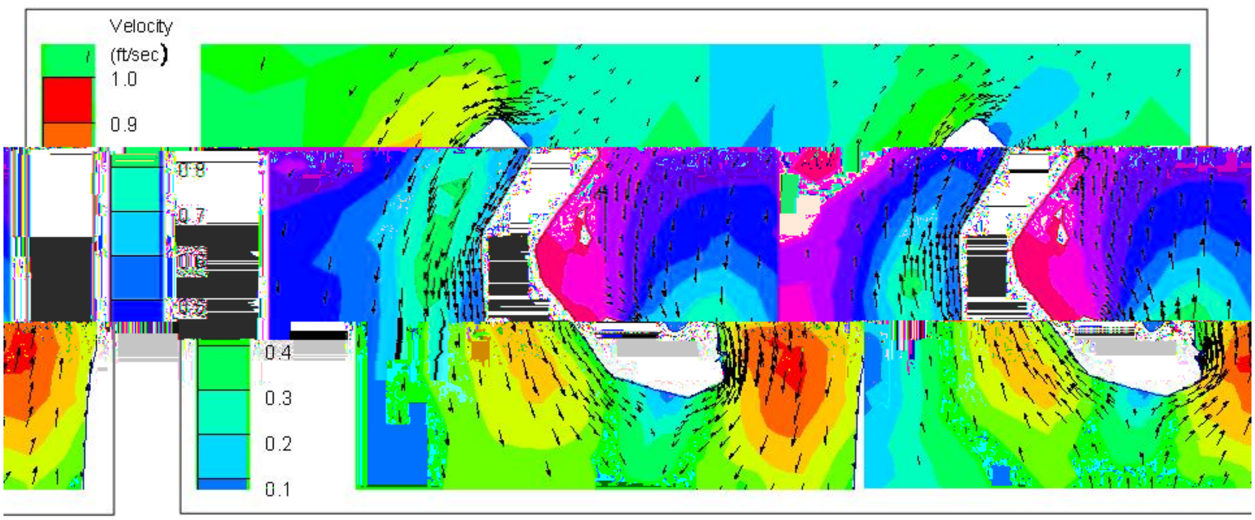

Figure 11. Peak ebb (left) and peak flood (right) velocity magnitude and direction.

A 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 project site. Examination of model results for both non-cohesive and cohesive sediments indicates that normal tidal currents in the vicinity of Poplar Island 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 7 m/s winds were determined to be the minimum necessary to cause sediment suspension and transport for non-cohesive sediments. Winds at 6 m/s were the minimum necessary to cause substantial sediment suspension and transport for cohesive sediments.

Winds from the NNW, N and NNE directions cause significant sediment transport for sand with negligible sediment transport for winds from other directions. Winds from the NNW, N, NNE, and NE cause significant sediment transport for clay. Figure 12 shows sand transport due to north winds and clay transport due to north-northeast winds. In general, for cohesive sediments the areas of erosion and accretion are larger than for non-cohesive sediment, as properties of cohesive sediment (shape, plasticitiy, electric charge) cause the particles to remain in suspension for relatively long periods of time before they settle out.

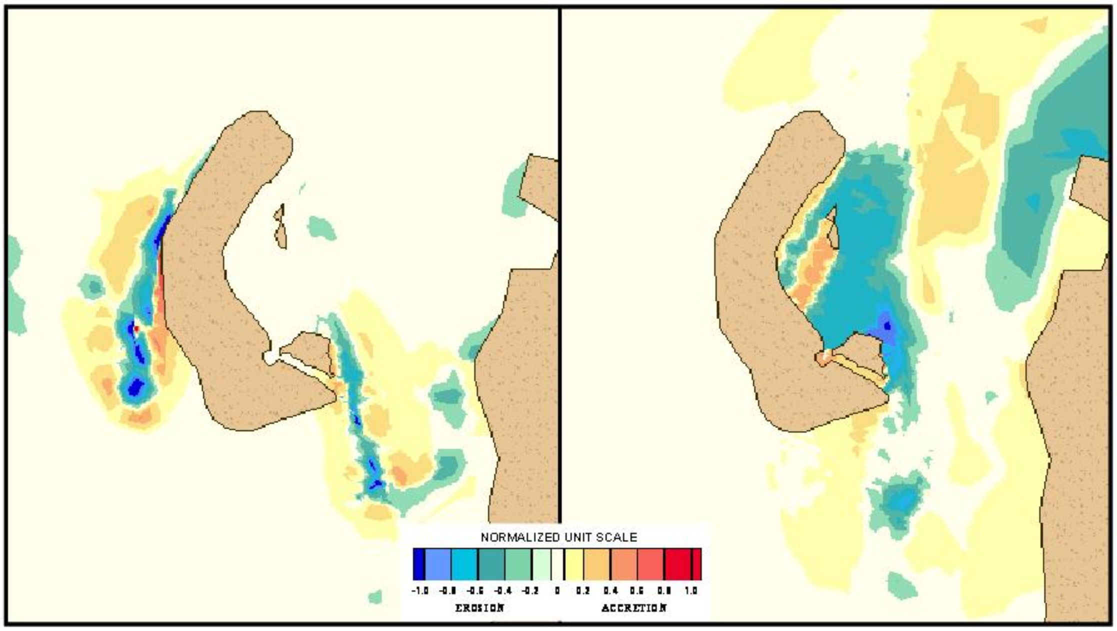

Figure 12. Erosion and accretion potentials s and due to North wind at 7 m/s (left) and clay due to NNE wind at 6 m/s (right).

HURRICANE ISABEL

Beginning September 18, 2003 and ending September 20, 2003, Poplar Island was subject to high water levels and high wind speeds associated with Hurricane Isabel that passed through the region. Data were obtained from recording stations around the Chesapeake Bay and used to evaluate impacts to the coastal engineering aspects of the project. Figure 13 shows water level data obtained from the National Oceanic and Atmospheric Association (NOAA) web site. These data are for Baltimore, Annapolis, Cambridge and Solomons. They show that for Baltimore, Annapolis and Cambridge water levels were 2.5 m MLLW, 2.2 m MLLW and 1.9 m MLLW, respectively. Note that data collection for Solomons stopped on September 18th at about 8 pm as the water levels were still rising, so the highest level was not reported. Table 2 above shows that the 100-yr water levels from the VIMS report are 2.6 m, 2.3 m and 1.9 m. The data indicate that water levels were on the order of a 100-yr return period event. Estimates for Poplar Island indicate that water levels approached 2.3 m MLLW which is higher than the design 100-year water level elevation of 2.1 m MLLW.

Figure 14 presents data for wind speeds and direction at Thomas Point Light, the nearest location to Poplar Island in the Chesapeake Bay. The figure shows that the highest sustained winds coincident with the highest water levels were from the southeast shifting to from the south directions at about 22 m/sec. The highest gusts from these directions were over 25 m/sec. Thus the site was exposed to the highest winds from a long fetch direction. The design winds presented in Table 1 indicate that the winds were on the order of 25- to 50-year return period strengths.

Damage to the coastal protection included a 30 m complete breech of a section of the eastern dike (see Figure 6) in the northern portion of the island. This breech section was adjacent to the transition to the western dike section that had a crest elevation about 0.5 m higher. The other damage area was a 120 m section of the typical western dike section (see Figure 5) on the southeastern portion of the island where the dike was overtopped and sand was eroded from behind the large armor stone. Note that the armor stone itself was not damaged and remained in place. This section was also adjacent to a dike section to the west that was constructed to an elevation about 0.6 m higher.

Costs to repair the northeast breech was about \$300 k, which included replacing sand, geotextile, roadway stone and 110 kg armor stone. Costs to repair the erosion on the southern end was about \$800 k and included sand, geotextile, roadway stone and 110 kg underlayer stone (no armor stone was repaired or replace). These costs are about 1.5 percent of the initial construction costs, less than the 4 percent overtopping damage repair cost used in the optimization analysis described above.

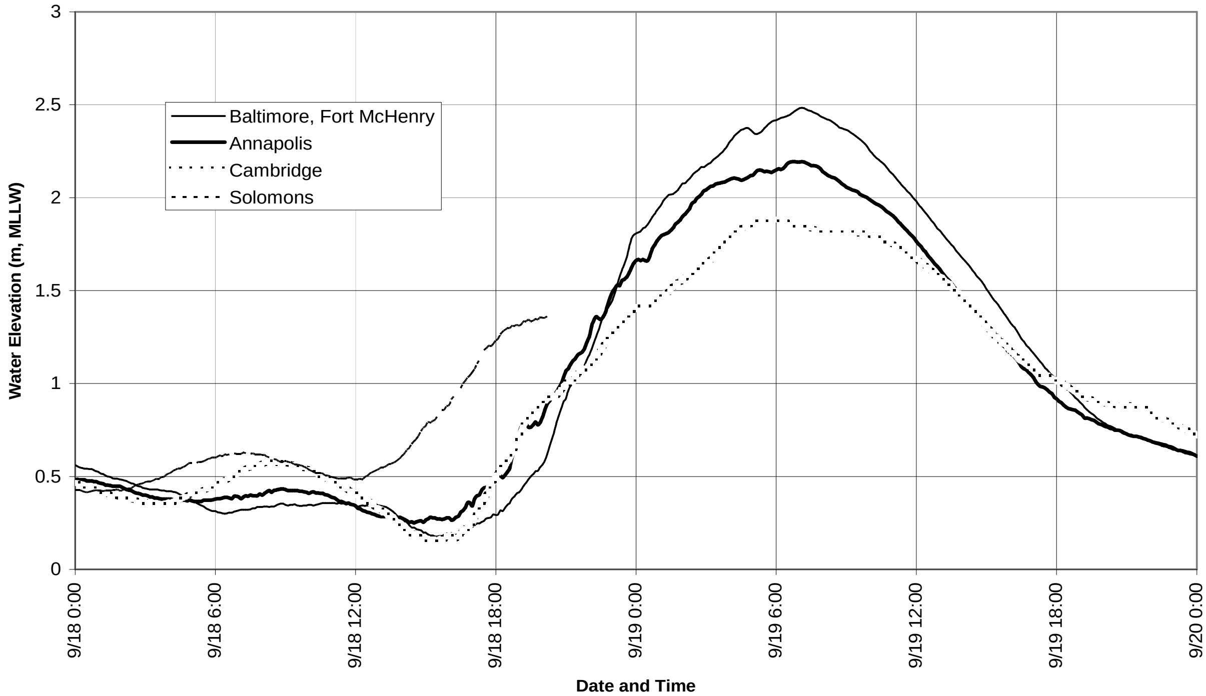

Figure 13. Water elevations during hurricane Isabel, September 2003.

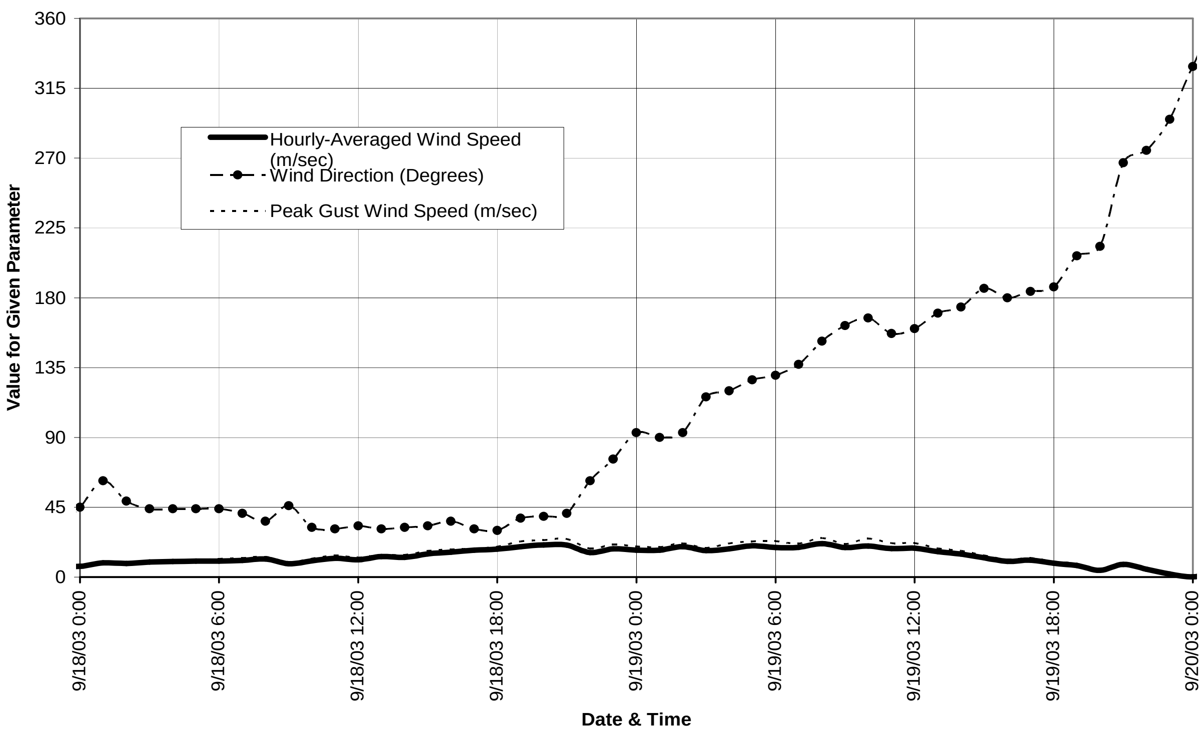

Figure 14. Wind speeds and direction during Hurricane Isabel Thomas Point Light station TPLM2, September 2003.

CONCLUSIONS

Initial costs for the project are primarily the dike construction costs, which were dictated by coastal and geotechnical engineering issues. Soil conditions impacted dike construction in areas where they were weak, requiring either excavation of the unsuitable material or over-building the sand. For Poplar Island, most soil conditions were firm. Optimization of the dike design provided an objective means for saving initial construction costs. The coastal engineering design was optimized based on initial construction costs and long-term maintenance costs. Analysis of the repair costs to the dikes following damage from Hurricane Isabel indicate that the optimization was justified as the predicted repair costs were greater than actual.

Tidal flow patterns around the 1847 island footprint are similar to that for the proposed project, and the proposed island restoration can be viewed as a restoration of historical hydrodynamic conditions. In general, residence times around the island area are on the order of 5~7 days relative to the given boundaries. Residence time distribution over the entire system is not affected by the Poplar Island alignment. In the vicinities of the Poplar Harbor area, however, residence times for the project condition increase slightly on the order of 0.1-0.2 day relative to the existing condition. Comparison between the project and the 1847 footprint shows almost no difference of residence times, which is consistent with the flow patterns. As a result, it was judged that there would be little difference in overall water quality for the with-project condition relative to existing conditions. This prediction has been substantiated through continued water sampling and analysis.

Winds from the NNW, N and NNE directions cause significant sediment transport for sand with negligible sediment transport for winds from other directions. Winds from the NNW, N, NNE, and NE cause significant sediment transport for clay. Figure 15 shows an aerial photo of the completed project taken July 2003.

Figure 15. Aerial view of completed Poplar Island (2003).

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