REPORT

# ARMOR LAYER DESIGN APPENDIX
# ONONDAGA LAKE

**Prepared for**

Honeywell
PARSONS

**Prepared by**
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**January 2011**

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## LIST OF ACRONYMS AND ABBREVIATIONS

| Abbreviation | Definition |
|---|---|
| 2-D | 2-dimensional |
| ACES | Automated Coastal Engineering System |
| CEM | Coastal Engineering Manual |
| cfs | cubic feet per second |
| COCs | chemicals of concern |
| CRREL | Cold Regions Research and Engineering Laboratory |
| °F | degrees Fahrenheit |
| FEMA | Federal Emergency Management Agency |
| fps | feet per second |
| IDA | instantaneous data archive |
| IRM | Interim Remedial Measure |
| Lake | Onondaga Lake |
| LP3 | Log-Pearson Type III |
| Metro | Metropolitan Syracuse Wastewater Treatment Plant |
| mgd | million gallons per day |
| mph | miles per hour |
| NAVD88 | North American Vertical Datum of 1988 |
| NGVD29 | National Geodetic Vertical Datum of 1929 |
| NOAA | National Oceanic and Atmospheric Administration |
| NYSCC | New York State Canal Corporation |
| NYSDEC | New York State Department of Environmental Conservation |
| NYSDOT | New York State Department of Transportation |
| pdf | probability distribution functions |
| PDI | Pre-Design Investigation |
| RA | Remediation Area |
| ROD | Record of Decision |
| SMS | Surface Water Modeling System |
| SMU | Sediment Management Unit |
| USACE | United States Army Corps of Engineers |
| USEPA | United States Environmental Protection Agency |

| | |
|---|---|
| USGS | United States Geological Survey |
| Wastebed | WB |

# 1 INTRODUCTION

As described in the Record of Decision (ROD), the multi-component sediment cap portion of the Onondaga Lake (Lake) remedial design will consist of separate layers to provide specific functions:

- Chemical isolation from chemicals of concern (COCs) in the underlying sediment (i.e., "chemical isolation layer")
- Protection from physical forces causing erosion (i.e., "armor layer")
- Suitable substrate to promote habitat reestablishment (i.e., "habitat layer")

This report details the design of the sediment cap armor layer; other technical documents present the design of the chemical isolation and habitat layers.

The primary objective of the armor layer is to prevent exposure and erosion of the chemical isolation layer. The potential for erosion of the sediment cap depends on the erosive processes that are likely to occur in Onondaga Lake, as well as the materials comprising the cap layers. Potential erosive processes that may act on the sediment cap within Onondaga Lake include:

- Wind-induced waves due to storm events
- Currents in the Lake resulting from discharge of tributaries and other discharges, as well as from typical lake circulation conditions
- Localized propeller wash from vessels
- Waves generated by passing vessels
- Winter ice buildup and resulting scour processes

Each of these potential erosion processes was evaluated independently to determine the design requirements for the cap armor component. The cap armor layer was then designed to withstand erosion under the range of anticipated conditions for each process. This appendix presents the results of this armor layer design analysis. The appendix is divided into the following sections:

- Section 2 summarizes the armor layer design for each remediation area
- Section 3 describes the armor layer design and performance criteria

- Section 4 presents the evaluation of historical Onondaga Lake water levels to determine the water level to be used for design of the armor layer
- Section 5 presents the wind-generated waves analysis
- Section 6 presents the tributary and lake currents analysis
- Section 7 presents the vessel-impacts analysis (propeller scour and boat wakes)
- Section 8 presents the ice analysis
- Section 9 presents the Sediment Management Unit (SMU) 3 shoreline enhancement analysis
- Section 10 presents the evaluation of the relative stability of littoral zone sediments in water depths from 20 to 30 feet (6 to 9 meters)

# 2 ARMOR LAYER DESIGN FOR ONONDAGA LAKE

Table 2-1 presents a summary of the sediment cap armor layer design.

**Table 2-1**
**Summary of Sediment Cap Armor Layer Design by Remediation Area**

| Range of Water Depths Based on Baseline Lake Level (feet) | A | | B | | C and D | | E | |
|---|---|---|---|---|---|---|---|---|
| | Particle Size | Minimum Thickness (inches) | Particle Size | Minimum Thickness (inches) | Particle Size | Minimum Thickness (inches) | Particle Size | Minimum Thickness (inches) |
| 40.5 to 30.5 | Fine Sand | 3 | Fine Sand | 3 | Fine Sand | 3 | Fine Sand | 3 |
| 30.5 to 20.5 | Fine Sand | 3 | Fine Sand | 3 | Fine Sand | 3 | Medium Sand | 3 |
| 20.5 to 15.5 | Fine Sand | 3 | Fine Sand | 3 | Medium Sand | 3 | Fine Gravel | 3 |
| 15.5 to 10.5 | Fine Sand | 3 | Medium Sand | 3 | Medium Sand | 3 | Fine Gravel | 3 |
| 10.5 to 8.5 | Medium Sand | 3 | Coarse Sand | 3 | Fine Gravel | 3 | Coarse Gravel | 3 |
| 8.5 to 6.5 | Coarse Sand | 3 | Fine Gravel | 3 | Fine Gravel | 3 | Coarse Gravel | 3 |
| 6.5 to surf zone | Fine Gravel | 3 | Fine Gravel | 3 | Fine Gravel | 3 | Cobbles | 6 |
| Within surf zone | Coarse Gravel | 3 | Coarse Gravel | 3.5 | Coarse Gravel | 4 | Cobbles | 6 |

Notes:
1. Sediment type was classified using the Unified Soil Classification System.
2. The surf zone begins at a depth approximately equal to the breaking wave height.
3. The breaking wave depth (surf zone) is approximately 3.5 feet in Remediation Areas (RAs) A and B, 4 feet in RAs C and D, and 7 feet in RA E.
4. Range of water depths referenced to the Onondaga Lake baseline water level of 362.5 feet (see Section 4 of this appendix). The water level used for the armor layer design is 0.5 feet lower than the baseline water level (362.0 feet).
5. The erosion protection layer thickness will be the greater of either 1.5 times the largest particle diameter, or 2 times the median particle diameter. For practical application considerations for construction, the minimum erosion protection layer thickness will be 3 inches (0.25 feet).

The armor layer thickness will be the greater of either 1.5 times the largest particle diameter or 2 times the median particle diameter. For practical application considerations for construction, the minimum armor layer thickness will be 0.25 feet.

The tributary analysis resulted in stable particle sizes of fine gravel for the portions of the cap near the discharge of Ninemile Creek (Remediation Area [RA] A) and Onondaga Creek (RA E). The required particle sizes are less than or equal to the stable particles computed from the wind-wave results. Ninemile Creek and Onondaga Creek are the two largest inflows to the Lake. Evaluation of erosive forces from other tributaries and discharges to the Lake, such as from stormwater and other outfalls, will be evaluated as part of future design efforts, but is not anticipated to result in significant design revisions. The assessment of typical current velocities measured in the Lake (away from the influence of tributary flows) indicated a stable particle size of fine sand, which is less than or equal to the stable particles computed from the wind-wave results.

Based on a review of the types of vessels in Onondaga Lake and operating procedures for these vessels, there will generally be two types of vessel operations over the cap: 1) commercial and recreational vessels operating frequently in the New York State Canal Corporation (NYSCC) navigation channel to the Inner Harbor in RA E; and 2) recreational vessels operating randomly in shallower water depths. The propeller wash analysis indicates that particle sizes in the coarse gravel range (1 to 2 inches) would be required for the armor layer in the NYSCC navigation channel. For the other areas of the cap, recreational vessels will likely operate randomly; that is, these vessels will not start and stop or pass over the same location on a regular basis. Due to the limited area impacted by propeller wash from an individual vessel, significant movement of armor layer is not expected from propeller wash. In addition, in shallow water, a dedicated 1.5- to 2-foot-thick habitat layer is planned above the armor and chemical isolation layers. Any potential disturbance to particles within a localized area is expected to "self-level" soon after disturbance due to natural hydrodynamic conditions within the Lake.

Ice freezing to the bottom of Onondaga Lake is expected in shallow water at the shoreline. In such cases, it is expected that the normal thickening of ice will encounter the bed and freezing will continue. It was determined that the freezing of ice to the lake bottom is limited to water depths of less than 1.5 feet. To protect the chemical isolation layer for the cap, the armor layer and chemical isolation layer will be placed below the ice freezing zone described above. Using a low lake water level of 362.0 feet, the ice freezing zone would be

360.5 feet. The top of the armor layer and chemical isolation layer will be placed below an elevation of 360.5 feet to protect against ice scour.

The final armor layer median particle size ($D_{50}$) and gradation (such as for the sands and gravels) will be selected during the final design phase with consideration of constructability and availability of materials.

# 3 DESIGN AND PERFORMANCE CRITERIA

Setting performance standards for the sediment cap is a necessary first step in developing the design requirements for isolation caps. As described in the United States Environmental Protection Agency's (USEPA's) and the United States Army Corps of Engineers' (USACE's) *Guidance for In-Situ Subaqueous Capping of Contaminated Sediments* (Palermo et al. 1998):

> *"The cap component for stabilization/erosion protection has a dual function. On the one hand, this component of the cap is intended to stabilize the contaminated sediments being capped, and prevent them from being resuspended and transported offsite. The other function of this component is to make the cap itself resistant to erosion. These functions may be accomplished by a single component, or may require two separate components in an in-situ cap."*

In addition, USEPA's *Contaminated Sediment Remediation Guidance for Hazardous Waste Sites* (USEPA 2005) states that:

> *"[t]he design of the erosion protection features of an in-situ cap (i.e., armor layers) should be based on the magnitude and probability of occurrence of relatively extreme erosive forces estimated at the capping site. Generally, in-situ caps should be designed to withstand forces with a probability of 0.01 per year, for example, the 100-year storm."*

As described in the ROD, the sediment cap will be a multi-component cap designed with separate layers to provide chemical isolation of underlying sediment, protection from erosive forces, and suitable substrate for habitat restoration. The erosion protection, or armor layer, is designed to protect the chemical isolation layer (which will be primarily made of sand) from erosional processes such as waves, ice, tributary flows, and propeller wash. The armor layer will be included in the cap design and construction, where needed, above the chemical isolation layer and below the habitat restoration layer. In select locations, a single layer of material may be designed to function as both the armor layer and habitat restoration layer.

The armor layer is designed to provide long-term protection of the chemical isolation layer using methods developed by the USEPA and the USACE specifically for in-situ caps. This includes the methods included in *Armor Layer Design of Guidance for In-Situ Subaqueous Capping of Contaminated Sediments* (Maynord 1998). The armor layer design presented herein involved evaluating the particle size (ranging from sand to cobbles) required to resist a range of erosive forces expected on Onondaga Lake.

Consistent with USEPA guidance and based on ROD requirements and other project-specific considerations, design and performance criteria for the armor layer are listed below:

- The armor layer will be physically stable under conditions predicted to occur based on consideration of 100-year return-interval waves. The 100-year wave is the highest wave that would be expected to occur, on average, once every 100 years.
- The armor layer, specifically the areas potentially impacted by influent from tributaries, will be physically stable under conditions predicted to occur during a 100-year flood flow event.
- The sediment cap will be designed such that the chemical isolation layer will not be negatively impacted by ice.
- The sediment cap will be designed such that the chemical isolation layer is not negatively impacted by erosive forces resulting from propeller scour.

# 4 ONONDAGA LAKE WATER LEVELS

This section provides a summary of the analysis of historical Onondaga Lake water levels for determining an appropriate water level to use for armor layer design.

Onondaga Lake is part of the Erie (Barge) Canal system, and the elevation of the Lake is controlled by a dam on the Oswego River at Phoenix, New York, downstream of the Lake. The United States Geological Survey (USGS) maintains a water level gage on Onondaga Lake at the Onondaga Lake Park Marina Basin in Liverpool, New York (USGS Gage 04240495). Daily mean (average) water level data since October 1970 are available online and can be accessed at http://waterdata.usgs.gov/ny/nwis/dv/?site_no=04240495&agency_cd=USGS&referred_module=sw. It should be noted that the water level data were reported to the National Geodetic Vertical Datum of 1929 (NGVD29). These water levels were converted to the project datum, the North American Vertical Datum of 1988 (NAVD88), by subtracting 0.59 feet.

A frequency analysis was performed on the daily mean water level data from October 1, 1970 to April 1, 2009 (approximately 38 years). Table 4-1 presents the minimum, maximum, mean (average), and median water levels by month. Figure 4-1 presents a time series of Onondaga Lake water levels. Figure 4-2 presents the cumulative frequency distribution. Figure 4-3 presents monthly median water levels for Onondaga Lake.

Based on the measurements collected over the past 38 years, the following observations can be made:

- The mean and median waters level for the Lake were similar at 362.85 feet and 362.58 feet, respectively (Table 4-1)
- The highest lake level was 369.18 feet (on April 28, 1993)
- The lowest lake level was 361.00 feet (on March 12, 1978)
- The median water levels for the late winter/spring months (reflecting higher water levels due to rainfall and snowmelt) are 363.35 feet (April) and 363.20 feet (March)
- The median water levels for summer months (reflecting drier conditions and lower lake levels) are 362.31 feet (August) and 362.30 feet (September)

**Table 4-1**

**Monthly Minimum, Average, and Maximum Onondaga Lake Water Levels**

| Month | Minimum Water Level (feet) | Mean Water Level (feet) | Median Water Level (feet) | Maximum Water Level (feet) |
|---|---|---|---|---|
| January | 361.63 | 362.87 | 362.70 | 366.64 |
| February | 361.33 | 362.87 | 362.68 | 366.74 |
| March | 361.00 | 363.39 | 363.20 | 367.88 |
| April | 361.83 | 363.66 | 363.35 | 369.18 |
| May | 361.44 | 362.98 | 362.63 | 368.33 |
| June | 361.68 | 362.61 | 362.49 | 368.55 |
| July | 361.70 | 362.51 | 362.37 | 368.55 |
| August | 361.73 | 362.35 | 362.31 | 364.58 |
| September | 361.64 | 362.38 | 362.30 | 366.33 |
| October | 361.65 | 362.60 | 362.44 | 366.17 |
| November | 361.85 | 362.86 | 362.73 | 365.78 |
| December | 361.56 | 363.07 | 362.97 | 366.33 |
| Yearly (January to December) | 361.00 | 362.85 | 362.58 | 369.18 |

Notes:

1. Daily mean water levels from October 1, 1970 through April 1, 2009 obtained from http://waterdata.usgs.gov/ny/nwis/uv/?site_no=04240495&agency_cd=USGS.
2. Water levels referenced to the NAVD88 vertical datum.

For the design of the habitat modules, a baseline water level of 362.5 feet is being used. This water level represents the mean water level in Onondaga Lake during the plant growing season (May through October). Based on the analysis above, it can be seen that Onondaga Lake water levels have rarely dropped below 362.0 feet since the mid-1990s (see Figure 4-1). Further, this lake elevation of 362.0 feet also represents an elevation that has been exceeded during approximately 99.6 percent of the analyzed time period. A lake level of 362.0 feet is being used for the armor layer design. In principle, lower water levels correlate to greater forces exerted by storm events on the lake bottom. Therefore, selection of a lake level of 362.0 feet represents a conservative assumption for armor layer design.

# 5 WIND-WAVE ANALYSIS

This section summarizes the wind-wave analysis that was used to determine the 100-year design wave for each remediation area and the resultant particle size(s) necessary for providing stability for the sediment cap armor layer. To resist wind-generated waves, stable particle sizes were computed at various water depths within and outside of the surf zones for each remediation area where sediment caps will be constructed as part of the Lake remedy.

## 5.1 Introduction

Meteorological factors such as changes in barometric pressure and the uneven heating and cooling of the earth produce pressure differences that result in winds. Winds blowing across the surface of bodies of water transmit energy to the water, and waves are formed. The size of these wind-generated waves depends on the wind velocity, the length of time the wind is blowing, and the extent of open water over which it blows (fetch) (USACE 1991).

For the Onondaga Lake wind-generated wave analysis, a return period for episodic events of 100 years has been utilized in the design evaluations of the armor layer to provide a high degree of protection to the sediment cap. Even though higher return frequencies for wind-wave analysis could be considered, the incremental benefits of using a return frequency higher than 100 years is minimal, since the changes in forcing conditions are minimally incremental over frequencies of 100 years, as opposed to those under the 100-year event. The use of 100-year return frequency for erosion protection of contaminated sediment site cap/armor design is also consistent with past practices at national contaminated sites under USEPA-/USACE-/ State-led programs. The wind-wave analysis summarized herein was conducted for the following remediation areas (Figure 5-1):

- RA A
- RA B
- RA C and D
- RA E

The wind-wave analysis consisted of the following major components:

1. Obtaining historical wind speeds and directions proximal to Onondaga Lake
2. Conducting a statistical analysis of wind data to estimate the 100-year return-interval wind speed (i.e., the highest wind speed that would be expected to occur once, on average, every 100 years) for each remediation area
3. Estimating the 100-year wave height and period from the 100-year return-interval wind data
4. Computing the particle size necessary to withstand the erosive forces associated with the 100-year wave outside the surf zone
5. Computing the particle size necessary to resist the erosive forces associated with the 100-year breaking wave within the surf zone

In general, within each remediation area, the sediment cap armor layer size will increase as the water depth decreases due to increasing wave energy. The details of the methodology are presented in Section 5.3. A detailed example calculation is included as Attachment A.

## 5.2 Summary

The wind-wave analysis was conducted to determine armor stone sizes for the sediment cap in RAs A, B, C, D, and E based on the 100-year design wave. Design wave heights were computed using a statistical analysis of 68 years of wind records collected at Hancock International Airport (formerly Syracuse Municipal Airport). The airport is located approximately 5 miles east of Onondaga Lake. Wave-induced horizontal orbital velocities generated by the 100-year wave were computed at different water depths before wave-breaking.

Stable sediment particle sizes were computed for the sediment cap for various water depths both prior to, and following, wave-breaking (in the surf zone). In general, the armor layer size increases as the water depth decreases. The size of the armor layer predicted for Onondaga Lake is generally gravel- to cobble-sized in the surf zone (shallower depths) and sand-sized materials in the deeper zones. Table 5-1 summarizes the particle size for each remediation area.

**Table 5-1**
**Summary of Sediment Cap Armor Layer Design by Remediation Area**

| Range of Water Depths based on Baseline Lake Level (feet) | A | | B | | C and D | | E | |
|---|---|---|---|---|---|---|---|---|
| | Particle Size | Minimum Thickness (inches) | Particle Size | Minimum Thickness (inches) | Particle Size | Minimum Thickness (inches) | Particle Size | Minimum Thickness (inches) |
| 40.5 to 30.5 | Fine Sand | 3 | Fine Sand | 3 | Fine Sand | 3 | Fine Sand | 3 |
| 30.5 to 20.5 | Fine Sand | 3 | Fine Sand | 3 | Fine Sand | 3 | Medium Sand | 3 |
| 20.5 to 15.5 | Fine Sand | 3 | Fine Sand | 3 | Medium Sand | 3 | Fine Gravel | 3 |
| 15.5 to 10.5 | Fine Sand | 3 | Medium Sand | 3 | Medium Sand | 3 | Fine Gravel | 3 |
| 10.5 to 8.5 | Medium Sand | 3 | Coarse Sand | 3 | Fine Gravel | 3 | Coarse Gravel | 3 |
| 8.5 to 6.5 | Coarse Sand | 3 | Fine Gravel | 3 | Fine Gravel | 3 | Coarse Gravel | 3 |
| 6.5 to surf zone | Fine Gravel | 3 | Fine Gravel | 3 | Fine Gravel | 3 | Cobbles | 6 |
| Within surf zone | Coarse Gravel | 3 | Coarse Gravel | 3.5 | Coarse Gravel | 4 | Cobbles | 6 |

Notes:
1. Sediment type was classified using the Unified Soil Classification System.
2. The surf zone begins at a depth approximately equal to the breaking wave height.
3. The breaking wave depth (surf zone) is approximately 3.5 feet in RA A and B, 4 feet in RAs C and D, and 7 feet in RA E.
4. The range of water depths referenced to the Onondaga Lake baseline water level of 362.5 feet (see Section 4 of this appendix). The water level used for the armor layer design is 0.5 feet lower than the baseline water level (362.0 feet).
5. The erosion protection layer thickness will be the greater of either 1.5 times the largest particle diameter, or 2 times the median particle diameter. For practical application considerations for construction, the minimum erosion protection layer thickness will be 3 inches (0.25 feet).

## 5.3 Methodology

This section describes the methodology used to estimate the 100-year return-interval wind speed, the 100-year design wave height and period, and the size and thickness of the armor layer for the sediment cap. The results of the analyses are presented in Section 5.4 below.

### *5.3.1 Wind Analysis Methodology*

Hourly wind measurements (speeds and direction) from 1942 to 2009 were obtained from Hancock International Airport. The airport is located approximately 5 miles east of Onondaga Lake. The winds were measured at the following heights above the ground:

- 1942 to 1949: 57 feet
- 1949 to 1962: 72 feet
- 1962 and 1963: 84 feet
- 1963 to 2009: 21 feet

A wind rose diagram for the data, illustrating how wind speed and direction are typically distributed for the site, is shown on Figure 5-1. As can be seen in this figure, the prevailing winds in the area are from the westerly direction.

The methodology used to estimate winds speeds for wave prediction were consistent with that described in Part II – Chapter 2 of the USACE's *Coastal Engineering Manual* (CEM; USACE 2006). In accordance with the CEM, the measured wind speeds were first converted to hourly averaged wind speeds at heights of 32.8 feet (10 meters) above the ground for predicting waves (USACE 2006). The hourly averaged wind speeds were then converted to 15-minute-averaged wind speeds using procedures outlined in the CEM. In large lakes, the wave generation process tends to respond to average winds over a 15- to 30-minute interval (USACE 2006), because shorter duration gusts are generally not sufficient for significant wave generation. It is assumed that Onondaga Lake represents fetch-limited conditions and not duration-limited conditions for wave growth. Using 15-minute averages produces higher wind speeds than 30-minute averages, so the more conservative 15-minute averaging interval was used in this analysis.

A statistical analysis was then performed on the maximum annual 15-minute-averaged wind speeds to estimate the 100-year return-interval wind speeds (the 100-year design wind speed). For each remediation area, those winds blowing primarily toward the shoreline for that remediation area (i.e., along the possible fetch radials) were considered in each analysis. The following ranges of wind directions were used (where 360° represents due north; see Figure 5-1):

- A: 330° to 100°
- B: 330° to 130°
- C: 0° to 130°
- D: 320° to 30°
- E: 280° to 340°

Five candidate probability distribution functions (pdfs) were fitted to the maximum 15-minute-averaged annual winds during the 68-year period of record to develop representative wind speeds with different return periods, including the 100-year wind speed. The candidate distribution functions evaluated were Fisher-Tippet Type I and Weibull distributions with the exponent k varying from 0.75 to 2.0. The 100-year wind speed to be used in the design was chosen from the distribution that best fit the data.

In addition to the data available from Hancock Airport, data are also available from two meteorological stations installed at Onondaga Lake as part of the Pre-Design Investigation (PDI) studies to measure wind speeds and directions near the Lake. One station was installed at Wastebed (WB) 13 (WB 13 Site) in November 2005, and another was installed along the Lake shore at Willis Avenue (Lakeshore Site) in November 2006 (Parsons 2007a, 2007b). Hourly-averaged wind speed and directions were measured at both sites at an elevation of 10 meters above the ground. Attachment B presents a comparison performed by Parsons of the monthly average and monthly maximum wind speeds between Hancock International Airport, the WB 13 Site, and the Lakeshore Site for 2006 to 2009. The comparisons indicate that the monthly average and monthly maximum wind speeds are higher at Hancock International Airport than at the Lake. In addition, it appears that there is a stronger east-to-west wind at the airport than at the Lakeshore Site. In summary, the 10-meter wind velocities measured at the Lakeshore Site from the north/northwest (which has a long overwater fetch distance) are less than the adjusted wind velocities from the airport, indicating that no important transitional effects have been ignored by using the airport data. Therefore, the long-term measurements collected at Hancock Airport were used for the wind-wave evaluations at Onondaga Lake.

### *5.3.2 Wind-Generated Wave Analysis and Armor Layer Sizing Methodology*

The Onondaga Lake shoreline and bathymetry data used to estimate the longest fetch distance and bathymetric profile for each remediation area were obtained from the proposed restored slopes and from C.R. Environmental as part of the *Onondaga Lake Phase I Pre-Design Investigation Geophysical Survey Report* (C.R. Environmental 2007). Along with the computed 100-year design winds described above, this information was used to estimate the 100-year wave heights and horizontal orbital water velocities at various depths and nearshore slopes. The USACE Automated Coastal Engineering System (ACES) computer program was used to model wave growth and propagation due to winds (USACE 1992). The ACES program was developed in 1992 by the USACE and is an accepted world-wide reference for modeling water wave mechanics and properties. To compute the 100-year design wave height for each remediation area, the 100-year wind was applied along the longest fetch distance for each remediation area.

For each remediation area, the 100-year wave was determined using the ACES Wave Prediction Module and was then transformed along the longest fetch's bathymetric profile using the ACES Wave Transformation Module. This module was used to determine wave heights and horizontal orbital velocities at different water depths and the breaking wave depth. These wave characteristics were then used to determine appropriate stable particle sizes within and outside of the "surf zone." The surf zone is defined as the region in the Lake extending from the location where the waves begin to break to the limit of wave run-up on the shoreline slope. Within the surf zone, wave-breaking is the dominant hydrodynamic process. Outside of the surf zone, the wave-induced horizontal orbital velocities are the dominant force. In general, the surf zone begins at a depth approximately equal to the breaking wave height.

The USEPA's *Armor Layer Design for the Guidance for In-Situ Subaqueous Capping of Contaminated Sediment* (Maynord 1998) was used to compute a representative particle size (diameter) to resist erosion associated with the wave-induced horizontal orbital velocities. This estimate was compared with these two other methods:

- The commonly used Shields diagram presented in Vanoni (1975), which presents stable particle sizes under different flow velocities measured parallel to the particle bed.
- A model for sediment initiation under non-breaking waves on a horizontal bed developed by You (2000). This model was based on experimental data collected for oscillatory flows.

The maximum particle size obtained from these three methods was conservatively selected as the stable sediment particle for the sediment cap armor layer outside of the surf zone.

Due to the amount of turbulence generated by breaking waves in the surf zone, the sediment cap armor layer was modeled as a rubble mound berm (or revetment) in the surf zone. The berm or revetment was assumed to be composed of a rock layer (equivalent to the armor layer) on the top of a chemical isolation layer that would serve as an interface between the revetment core (i.e., the sediment to be capped) and the rock surface (armor layer). The physical properties (e.g., grain size distribution) of the chemical isolation layer (below the armor layer) will be selected to prevent wave-induced turbulence from moving the chemical isolation layer materials into or through the armor layer (i.e. "piping"). Such effects could be minimized by either providing a separate filter layer in between the armor and isolation cap, or through coarsening of the isolation cap material, and/or fine-grading the overall gradation of the armor layer. These options will be reviewed and addressed as part of the future design process.

The ACES Rubble Mound Revetment Design Module was used to compute the armor stone gradation and thickness in the surf zone. ACES assumes that the waves would propagate and break on the slope of the armor layer. The structure is assumed to be permeable, thereby minimizing wave reflection. Stable particle sizes (i.e., armor sizes) for the restored slopes (that are being currently considered for each remediation area) were evaluated using the model.

Revetments used for coastal protection projects are often designed allowing for some maintenance of the armor layer. The revetment design methodology allows varying amounts of displacement (movement) of the armor layer. The amount of displacement considered can be categorized as:

- No displacement – No armor stone displacement (note that this does not account for settlement)
- Minor displacement – Few armor stones displaced (less than 5 percent) and potentially redistributed within or in the near vicinity of the armor layer
- Intermediate displacement – Ranging from moderate to severe; armor stones are displaced without causing exposure of filter layer to direct wave attack

Allowable movement or rocking of armor stones (minor displacement) in the ACES revetment design methodology is based on steeper slopes (from 1.5 horizontal to 1 vertical [1.5H:1V] to 6H:1V) that are typically used for coastal revetments than the relatively milder slopes that are being considered for Onondaga Lake (50H:1V). Since the proposed slopes are milder than the slopes typically evaluated, only the minor displacement maintenance scenario was considered in the analysis.

## 5.4 Results

This section summarizes the results of the wind-wave analysis and armor layer sizing for each remediation area. A detailed example calculation is included as Attachment A. Table 5-2 presents a summary of the 100-year design wind speeds based on various return-interval periods for each remediation area. The 100-year design wind speed varies from 45.0 miles per hour (mph) at RA C to 60.0 mph at RA E.

**Table 5-2**
**100-Year Design Wind Speed by Remediation Area**

| | A | B | C | D | E |
|---|---|---|---|---|---|
| Wind Direction (degrees) | 330° to 100° | 330° to 130° | 0° to 130° | 320° to 30° | 280° to 340° |
| Wind Speed (mph) | 47.7 | 47.9 | 45.0 | 46.5 | 60.0 |

Using the 100-year design wind speed shown in Table 5-2, Table 5-3 presents a summary of the fetch length, the 100-year significant wave height ($H_s$), the 100-year significant wave period ($T_s$), and the corresponding breaking wave height and depth for each remediation area. The 100-year design wave heights ranged from 2.6 feet in RA A to 5.2 feet in RA E. In general, the 100-year wave breaks in depths of 3.4 to 6.7 feet.

**Table 5-3**
**100-Year Design Wave Summary by Remediation Area**

| Remediation Area | Longest Fetch (miles) | Significant Wave Height (feet) | Significant Wave Period (seconds) | Breaking Wave Height (feet) | Breaking Wave Depth (feet) |
|---|---|---|---|---|---|
| A | 2.01 | 2.6 | 2.7 | 2.6 | 3.4 |
| B | 2.43 | 2.8 | 2.9 | 2.9 | 3.6 |
| C | 3.57 | 3.2 | 3.2 | 3.3 | 4.2 |
| D | 3.39 | 3.2 | 3.2 | 3.3 | 4.2 |
| E | 4.66 | 5.2 | 3.9 | 5.3 | 6.7 |

In the sediment cap design, the effects of wind and wave setup were not included so that the resultant design will be more conservative in terms of armor protection. An analysis was performed to evaluate the setup across the surf zone to evaluate the level of conservatism. In addition to the creation of wind-waves, wind can also cause a condition known as "setup" or “setdown.” Wind stress on the water surface can result in a pushing or piling up of water in the downwind direction and a lowering of the water surface in the upwind direction. When the wind blows, water will set up against the land. This setup, superimposed on the normal water level, causes apparent higher-than-normal water levels at the shoreline. When the wind stops, the setup or setdown water surface will return to normal levels (USACE 1991). Wind setup at the shoreline at each remediation area as a result of the 100-year design wind was estimated using two methods: Ippen (1966) and USACE (1997).

In addition to wind setup at the shoreline, as waves shoal and break, the momentum flux in the onshore direction is reduced and results in compensating forces on the water column (Dean and Dalrymple 1991). Wave setup is the superelevation of mean water level in the surf zone caused by wave action (Smith 2003). Similar to wind setup, wave setup causes apparent higher-than-normal water levels at the shoreline. The wave setups for the 100-year design waves were computed using Dean and Dalrymple (1991).

Table 5-4 presents the wind and wave setup in each remediation area. Estimates of the wind setup at the shoreline varies between methods but ranges between 1 and 6 inches in RAs A and B, 2 to 7 inches in RAs C and D, and 4 to 8 inches in RA E. The wave setup across the surf zone ranges from 6 inches in RA A to 1 foot in RA E.

**Table 5-4**
**100-Year Wind and Wave Setup Calculations by Remediation Area**

| Remediation Area | Longest Fetch (miles) | 100-Year Design Wind Speed (mph) | Wind Setup at Shoreline using USACE (1997) (feet) | Wind Set-up at Shoreline using Ippen (1966) (feet) | Wave Setup at Shoreline (feet) |
|---|---|---|---|---|---|
| A | 2.01 | 47.7 | 0.1 | 0.5 | 0.5 |
| B | 2.43 | 47.9 | 0.1 | 0.5 | 0.5 |
| C | 3.57 | 45.0 | 0.1 | 0.5 | 0.6 |
| D | 3.39 | 46.5 | 0.1 | 0.6 | 0.6 |
| E | 4.66 | 60.0 | 0.3 | 0.7 | 1.0 |

Stable sediment particle sizes for the sediment cap armor layer outside of the surf zone were calculated in accordance with the procedure presented in Section 5.3.2 and are presented in Table 5-5. Attachment C presents the calculations (including the computed median particle size, $D_{50}$) for each remediation area. Since RAs C and D have the same design wave height, they have the same stable particle size and, therefore, have been presented together in the table. As can be seen from the calculations, the stable particle sizes for the sediment cap predicted to resist the 100-year wind-induced wave would generally consist of sand-sized particles in water depths deeper than 15 feet. However, gravel-sized particles are predicted in water depths ranging from about 15 feet to the surf zone. Maynord (1998) recommends that the thickness of the armor layer be 1.5 times the maximum particle diameter ($1.5D_{100}$) or twice the median particle diameter ($2D_{50}$), whichever is greater. Due to the relatively small median particle diameter of these materials, it would not be practical to place such a small armor layer thickness consistent with Maynord's recommendations. Therefore, based on constructability considerations, the armor layer outside of the surf zone has been designed with a minimum thickness of 3 inches. It is recognized that this 3-inch design thickness represents a conservative thickness relative to the erosion protection evaluation.

**Table 5-5**
**Summary of Sediment Cap Armor Layer Design by Remediation Area**
**(Outside of Surf Zone)**

| Range of Water Depths (feet) | A | B | C and D | E |
|---|---|---|---|---|
| 40 to 30 | Fine Sand | Fine Sand | Fine Sand | Fine Sand |
| 30 to 20 | Fine Sand | Fine Sand | Fine Sand | Medium Sand |
| 20 to 15 | Fine Sand | Fine Sand | Medium Sand | Fine Gravel |
| 15 to 10 | Fine Sand | Medium Sand | Medium Sand | Fine Gravel |
| 10 to 8 | Medium Sand | Coarse Sand | Fine Gravel | Coarse Gravel |
| 8 to 6 | Coarse Sand | Fine Gravel | Fine Gravel | Coarse Gravel |
| 6 to surf zone | Fine Gravel | Fine Gravel | Fine Gravel | Cobbles |

Notes:
1. Sediment type was classified using the Unified Soil Classification System.
2. The surf zone begins at a depth approximately equal to the breaking wave height.
3. The breaking wave depth (surf zone) is approximately 3.5 feet in RA A and B, 4 feet in RAs C and D, and 7 feet in RA E.

Table 5-6 presents a summary of the median ($D_{50}$) armor stone size and minimum thickness layer for the sediment cap in the surf zone for each remediation area for a restored slope of 50H:1V. The design armor layer thicknesses presented in Table 5-6 are based on the same criteria summarized above for the areas outside of the surf zone (1.5 times $D_{50}$ or 2 times $D_{100}$, whichever is greater).

**Table 5-6**
**Armor Stone Size ($D_{50}$) and Thickness**
**with a Restored Slope of 50H:1V (For Surf Zone Regime)**

| Remediation Area | $D_{50}$ Stone Size (inches) | Thickness of Armor Layer (inches) |
|---|---|---|
| A | 1.5 | 3.0 |
| B | 1.7 | 3.4 |
| C and D | 1.9 | 3.8 |
| E | 3.0 | 6.0 |

Notes:
1. $D_{50}$ = median grain size.
2. Computed using minor displacement (S=3). Minor displacement refers to minimal movement of armor stones and could be related to "rocking" of the armor under extreme wave action. Repairs associated with such events (if any) will be handled as part of a maintenance program.

### *5.4.1 Assessment of Rubble-Mound Revetment Approach in Surf Zone*

As described in Section 5.3.2, the rubble-mound revetment methodology used for assessing stability within the surf zone is based on steeper slopes (from 1.5H:1V to 6H:1V; typical for coastal revetments) than the relatively mild slopes that are being considered for Onondaga Lake (50H:1V). A detailed assessment was performed to verify the use of this method for estimating stable particle sizes in the surf zone for the Onondaga Lake armor layer design.

The ACES methodology is based on van der Meer's (1988) paper titled D*eterministic and Probabilistic Design of Breakwater Armor Layers*. van der Meer suggested using the method for slopes flatter than 4H:1V. The van der Meer method uses wave period, structure permeability, damage, and storm duration. The ACES program assumes an event (N) of 7,000 waves. The equations are valid in the range 1,000< N <7,000, so N = 7,000 represents the limiting value that is used in this ACES application and is conservative. In addition, the typical revetment design and application (in which ACES is often used) involves the revetment extending from below the normal water level to above the normal water level. Waves typically break on the revetment itself. In the Onondaga Lake application, the armor layer will always be below the water level with a 1.5- to 2-foot dedicated habitat layer placed above the armor layer in the surf zone.

The waves in Onondaga Lake are fetch-limited and the surf similarity parameter ($\xi_\beta$) ranges between 0.06 and 0.07, which would indicate that the waves are spilling breakers. In spilling breakers, the wave crest becomes unstable and cascades down the shoreward face of the wave, thus producing a wave that can be characterized as "foamy water." Spilling breakers tend to occur for high-steepness waves on gently sloping beaches. Spilling breakers differ little in fluid motion from unbroken waves and generate less turbulence near the bottom and thus tend to be less effective in suspending sediment than plunging or collapsing breakers (Smith 2003). Since spilling breakers have a similar effect on stone stability as non-breaking waves, a comparison was made with the stable particle size recommended by Maynord (1998) and You (2000) for non-breaking waves, which would be a lower bound for the stable particle size estimate (Figure 5-2). As can been seen on Figure 5-2, the van der Meer method predicts larger stable particle sizes than Maynord (1998) and You (2000). Since the method needs to be extrapolated for flatter slopes (flatter than 6H:1V), only allowing for minor displacement was recommended to be conservative.

Based on this analysis, the use of the rubble-mound revetment equations are appropriate to assess stable particles sizes within the surf zone for Onondaga Lake.

### *5.4.2 Wave Refraction*

As waves approach the shoreline, it is possible for orthogonals (i.e., paths) of wave crests to converge or diverge if the water depth varies laterally in the direction of the wave crests. The shallower water depths tend to slow down the wave phase speed and give the impression that waves are "turning" toward the shallower parts of the shoreline. This turning or bending is known as wave refraction.

The restored slopes in each remediation area will generally be parallel with the shoreline and, therefore, significant wave refraction is not anticipated for the majority of the cap areas within the Lake. However, one area where there may be some wave refraction is in the vicinity of the boundary between RA A and RA B. There may some wave refraction around the "headland" feature at this location for waves approaching from the northeast. However, for the purpose of evaluating the stable particle sizes for the sediment cap, the design wave height was computed by applying the maximum wind speed along the maximum fetch distance for each remediation area. The computed stable particle size was then applied to the entire remediation area (not just portions of the remediation area); that is, larger waves that may impact only a portion of the remediation area that may "bend" toward another portion within the remediation area were not ignored. The maximum 100-year waves that could be generated for the remediation area were applied to cap armor design for the remediation area. Therefore, a wave refraction analysis was not necessary for the cap armor design.

# 6 TRIBUTARY ANALYSIS

This section summarizes the analysis used to evaluate the stable particle sizes for the armor layer of sediment caps to resist currents generated by the tributaries flowing into Onondaga Lake. High flows resulting from rainfall runoff can occur in the tributaries that discharge into Onondaga Lake. These high flows can result in elevated velocities (and associated bed shear stress) near the mouths of these tributaries and have the potential to erode and/or resuspend sediments. This analysis was conducted to refine and optimize cap designs for long-term stability and performance by evaluating the size of armor stone that would resist the erosive forces from the tributary flows (under high-flow events) entering into Onondaga Lake.

## 6.1 Summary

Velocity fields generated by the 100-year flows from Ninemile Creek and Onondaga Creek were modeled using a 2-dimensional (2-D) hydrodynamic model. Particle sizes necessary to withstand the 100-year flood flow were computed for the 100-year flood flow from Ninemile Creek and Onondaga Creek.

As expected, the influence of the tributaries decreases with distance from the tributary mouth into the Lake. The tributary analysis resulted in a stable particle size of coarse-to-fine gravel for the portions of the cap near the discharge of Ninemile Creek (RA A) and fine gravel for portions of the cap near the discharge of Onondaga Creek (RA E). In comparison, the assessment of typical current velocities measured in the Lake (away from the influence of tributary flows) indicated a stable particle size of fine sand. In summary, the stable particle sizes were smaller than the stable particles required to resist the 100-year wind-generated waves (see Section 5). In fact, the armor layer protection based on wind waves is predicted to withstand bottom velocities up to 4 feet per second (fps) and 6 fps at the mouths of Ninemile Creek and Onondaga Creek, respectively.

Honeywell is currently working with the New York State Department of Environmental Conservation (NYSDEC) in realigning Harbor Brook as part of the WB B upland remediation. The East Flume is also being realigned as part of the East Flume Interim Remedial Measure (IRM). These tributaries will be evaluated in the Final Design Submittal once the design specifications (e.g., alignment, channel cross-section, and depth) have been determined. Additionally, the Metropolitan Syracuse Wastewater Treatment Plant (Metro) discharges into RA E and will be evaluated as part of the Final Design Submittal. However, based on the relatively small discharge of these tributaries, the stable particle sizes will likely be smaller than those predicted for Ninemile Creek and Onondaga Creek.

## 6.2 Introduction

Seven creeks and eight industrial or stormwater conveyances discharge to Onondaga Lake. They include:

- Tributary 5A
- Ninemile Creek
- Sawmill Creek
- Bloody Brook
- Ley Creek
- Onondaga Creek
- Harbor Brook
- Metro (four outfalls total)
- East Flume
- Former I-690 Outfall
- Ditch A
- Westside Pumping Station Outlet

Of the seven creeks and eight industrial or stormwater conveyances, sediment caps are proposed at three of the tributary mouths and all of the outfalls. Honeywell evaluated the water current velocities resulting from the tributary and outfall flows as a potential mechanism for cap erosion. These tributaries/outfalls and the respective remediation areas where they enter the Lake include:

- Ninemile Creek in RA A
- Harbor Brook in RA D
- Onondaga Creek in RA E
- East Flume in RA D
- Tributary 5A in RA C

- Westside Pumping Station Outlet in RA C
- Former I-690 Outfall in RA D
- Metro outfalls in RAs D and E

Onondaga Creek and Ninemile Creek are the main contributors to the total freshwater input flow into Onondaga Lake (Exponent 2002), representing 34 percent and 33 percent, respectively, of the total flow. Harbor Brook is a minor tributary contributing only 2.1 percent of the total flow (Exponent 2002). The East Flume is an industrial conveyance that contributes a small percentage of surface water. Metro provides a significant contribution to Onondaga Lake with discharges of flows up to 126 million gallons per day (mgd). For the Onondaga Lake tributary analysis, the design evaluations of the armor layer used a 100-year return period for tributary and outfall flood flows, which provides a high degree of protection to the sediment cap. The analysis presented herein consists of determining the particle size required to resist erosive forces from Ninemile Creek and Onondaga Creek. Honeywell is currently working with the NYSDEC in realigning Harbor Brook as part of the WB B upland remediation. The East Flume is also being realigned as part of the East Flume IRM. These tributaries will be evaluated in the Final Design Submittal once the design specifications (e.g., alignment, channel cross-section, and depth) have been determined. However, based on the relatively small discharge of these tributaries, the stable particle sizes will likely be smaller than those predicted for Ninemile Creek and Onondaga Creek. The need for cap scour protection at the mouths of the eight industrial or stormwater conveyances will be evaluated as part of the Final Design Submittal.

In addition to the tributary and outfall flow analyses, the stable particle size was evaluated for typical Lake currents.

## 6.3 Methodology

This section presents the methods used to compute a stable particle size to resist erosive forces from tributary flood flows. Section 6.3.1 presents the hydrodynamic model used to compute the velocity fields generated by the 100-year flows from Ninemile Creek and Onondaga Creek. Section 6.3.2 presents the methods used to compute stable particle size for the estimated velocity fields associated with tributary flows as well as current velocities observed within the Lake.

Each of these methods is described below. A detailed example calculation is presented in Attachment C.

### 6.3.1 Estimating Current Velocities Using Hydrodynamic Modeling

To determine the stable armor layer particle size in Onondaga Lake, it is necessary to understand the velocity field generated by each tributary to the Lake. The velocity fields generated by the 100-year flows from Ninemile Creek and Onondaga Creek were modeled using the USACE hydrodynamic model, RMA2. The RMA2 model is a 2-D, depth-averaged (i.e., the model computes lateral, not vertical variations in flows), finite-element, hydrodynamic numerical model routinely used by the USACE for hydrodynamic studies and was previously used to estimate stable armor layer sediment size for Onondaga Lake during the Feasibility Study (FS) (Parsons 2004). The RMA2 model was used in conjunction with the Surface Water Modeling System (SMS) for RMA2, which is a pre- and post-processor that includes a graphical interface for display of inputs and results.

The following data were used to develop the hydrodynamic models for Ninemile Creek and Onondaga Creek:

- Creek bathymetry and floodplain topography (within the 100-year flood elevation) for Ninemile Creek and Onondaga Creek
- Proposed bathymetry following remediation for Ninemile Creek
- Estimations of predicted post-remediation bathymetry in Onondaga Lake
- Upstream 100-year Creek flood flow conditions
- Downstream 100-year Onondaga Lake water surface elevations
- Channel and lake bed material types/distributions
- Hydrodynamic calibration parameter values, such as the Peclet number (estimated based on published literature)

Table 6-1 summarizes the input parameters for each model. Each of the inputs is described below.

**Table 6-1**
**Summary of RMA2 Input Parameters**

| Tributary | Upstream BC | Downstream BC | Manning's Roughness Coefficient | | |
|---|---|---|---|---|---|
| | Flow (cubic fps [cfs]) | Water Surface Elevation (feet, NAVD88) | Lake | Tributary | Floodplain |
| Ninemile Creek | 3,756 | 366.96 | 0.03 | 0.035 | 0.1 |
| Onondaga Creek | 4,890 | 366.96 | 0.03 | 0.03 | NA |

Notes:
NA = not applicable.
Peclet numbers between 15 and 40 were used for both hydrodynamic models.

The hydrodynamic models were applied for steady-state flow conditions to provide conservative assumptions of flow and velocity.

#### *6.3.1.1 Model Grid*

Two-dimensional, finite-element model grids were developed for the tributary analysis that extended from the mouths of the tributaries into Onondaga Lake. The Ninemile Creek model grid extended approximately 2,700 feet into the Lake and 5,600 feet along the shore. Figure 6-1 presents the Ninemile Creek model grid, which consists of 2,351 elements and 7,026 nodes. The sediment cap in RA A extends approximately 1,450 feet into the Lake near the mouth of Ninemile Creek, and therefore the Ninemile Creek model grid extends approximately 1,250 feet beyond the proposed sediment cap. The Onondaga Creek model grid extended approximately 2,700 feet into the Lake and 3,900 feet along the shore. Figure 6-2 presents the Onondaga Creek model grid, which consists of 1,098 elements and 3,073 nodes. The sediment cap in RA E extends approximately 1,840 feet into the Lake near the mouth of Onondaga Creek, and therefore the Onondaga Creek model grid extends approximately 860 feet beyond the proposed sediment cap.

The bed elevations at each node of the grid were interpolated from bathymetric contour maps comprised of the proposed restored bathymetry in remediation areas and existing bathymetry measurements collected in 2006 by C.R. Environmental in remaining areas of Onondaga Lake. Limited bathymetry from the National Oceanic and Atmospheric Administration (NOAA) map was applied to Onondaga Creek (NOAA 2001), while planned restored bathymetry and topography collected in 2009 by Thew Associates was applied to Ninemile Creek. Figures 6-3 and 6-4 present the bathymetry used in the hydrodynamic model grids for Ninemile Creek and Onondaga Creek, respectively.

It should be noted that the design and implementation of dredging at the RA E shoreline adjacent to the active rail line is being evaluated due to the stability of this area during dredging. The effect that revisions to the capping surface have on tributary velocities in the vicinity of the Onondaga Creek area will be evaluated as part of the Final Design.

#### *6.3.1.2 Model Boundary Conditions*

The model boundary conditions consisted of upstream 100-year flood flows from the respective tributaries and a downstream 100-year flood water surface elevation in Onondaga Lake.

*Upstream Flow*

The 100-year flood flows were computed for each tributary using peak streamflow data acquired directly from a USGS website (http://nwis.waterdata.usgs.gov/usa/nwis/peak) or computed using the annual peak streamflow from USGS instantaneous data archive (IDA; http://ida.water.usgs.gov/ida/). Streamflow data were gathered from USGS gage titled Ninemile Creek at Lakeland Station (USGS #04240300) for Ninemile Creek and Onondaga Creek at Spencer Street (USGS #04240010) for Onondaga Creek. The 100-year flood flows were estimated using three methods/sources. These three values were reviewed and compared, and the most conservative value was used as the upstream boundary condition. The three methods/sources used were:

- Fitting a Log-Pearson Type III (LP3) probability distribution to the data and estimating the return flow based on the expected value of the distribution at the 99 percent exceedance level
- Using the USGS flood frequency analysis PeakFQ Program (where peak streamflow data were available from USGS)
- Obtaining 100-year flood flow estimates from a USGS report of flood flows for streams in New York State (USGS 2006)

Table 6-2 presents a summary of the estimated 100-year flood flows.

**Table 6-2**
**Computed 100-year Tributary Flows**

| Tributary[a] | Peak Discharge (cfs) for 100-year Return Frequency Flood Flow | | | |
|---|---|---|---|---|
| | LP3 Calculation[b] | PeakFQ Calculation (adjusted)[c] | USGS Flood Report[d] | Select 100-year Flood Flow |
| Ninemile Creek | 3,202 (3,700) | NA[e] | 2,260 | 3,756[f] |
| Onondaga Creek | 4,641 | 4,620 | 4,890 | 4,890 |

Notes:

a. Streamflow data were gathered from USGS gage titled Ninemile Creek at Lakeland Station (USGS #04240300) for Ninemile Creek and Onondaga Creek at Spencer Street (USGS #04240010) for Onondaga Creek.
b. Calculated using Log Pearson Type 3 distribution method. (Values in parentheses represent adjusted value based on review of graphical distribution fit).
c. Calculated using USGS's PeakFQ software adjusted to allow for inclusion of records designated as "All or part of the record affected by Urbanization, Mining, Agricultural changes, Channelization, or other," and "Discharge affected by Regulation or Diversion." PeakFQ typically excludes entries flagged with these qualifiers.
d. Taken from Table 9 of USGS Scientific Investigations Report 2006-5112, *Magnitude and Frequency of Floods in New York.* Page 131.
e. NA – PeakFQ calculations not made because the USGS peak streamflow data for this gage comprised only maximum daily average streamflow measurements as opposed to instantaneous peak flow measurements. Annual peak streamflow data based on maximum daily averages was not considered to be representative of actual peak streamflow conditions and was therefore not used for 100-year flood calculations.
f. A previous 100-year return flow for Ninemile Creek at Lakeland was developed by Limno-Tech, Inc. and presented in the April 2005 *HEC-RAS Model Calibration for Current Conditions and Remedial Scenario Forecasts for Ninemile Creek.* In that document, the 100-year flood flow was presented as 3,756 cfs (Table 6 and Table 8). Associated discussion stated that this was determined via use of the Log Pearson Type 3 method using available USGS data from the period 1990-2004.

### *Downstream Water Surface Elevations*

Onondaga Lake level was assessed as part of the Supplemental FS for Geddes Brook/Ninemile Creek, Operable Unit 1 (Parsons 2008). Upper and lower bound values, representing the range of estimates from two difference data sources (Federal Emergency Management Agency [FEMA] and USGS), were computed as 371.23 feet NAVD88 and 366.96 feet NAVD88, respectively. The lower value of 366.96 feet NAVD88 was conservatively selected for use as the downstream boundary condition in both hydrodynamic models. A sensitivity analysis on the water surface elevation was performed and is described in Section 6.5.

#### 6.3.1.3 Bed Roughness and Turbulent Exchange Coefficient

The Manning's roughness coefficient (Manning's *n*) value is used to represent the bed roughness in the hydrodynamic model. The visual observations of bed materials, as well as input values from previous hydraulic analyses, were used to assign the bed roughness in the model grids (Parsons 2008). Bounding values of Manning's roughness coefficient were evaluated for the channel and floodplains of Ninemile Creek as part of the Supplemental FS (Parsons 2008). The midpoint of the bounding values was selected for application to the RMA2 model. Therefore, Manning's roughness coefficients of 0.035 (range from 0.03 and 0.04) and 0.1 (range of 0.05 and 0.15) were used for the channel and floodplains, respectively, in the Ninemile Creek model. Since the beds of Onondaga Creek and Onondaga Lake are composed of sand and clay, a Manning's roughness coefficient of 0.03, based on published values (such as presented in Chou 1959 and USACE 1996), was used in the model. A sensitivity analysis on the Manning's roughness coefficient was performed and is described in Section 6.5.

Turbulence may be generally defined as the effect of temporal variations in velocity and the momentum exchange associated with their spatial gradients. In particular, turbulence is viewed as the temporal effects occurring at time scales smaller than the model time step. The eddy viscosity terms in the governing equations used in RMA2 actually represent the molecular viscosity and the effects of turbulence from the Reynolds stress terms. The eddy viscosity controls the numerical stability of the solution and the variation of velocities through a cross-section. Turbulence was accounted for in RMA2 by allowing the model to automatically adjust the turbulence exchange coefficient (E) after each solution iteration, based on a provided Peclet number. The Peclet number, which is based on the unique size and calculated velocity within each element, defines the relationship between the average elemental velocity magnitude, elemental length, fluid density, and E. The Peclet number (non-dimensional) is recommended to be between 15 and 40 (USACE 1996). Peclet numbers within this range were selected for the flow simulations.

### 6.3.2 Stable Particle Size to Resist Current Velocities

Representative particle sizes (diameters) to resist erosion associated with current velocities were estimated using two methods:

- The *Armor Layer Design for the Guidance for In-Situ Subaqueous Capping of Contaminated Sediment* (Maynord 1998), which uses current velocity and water depth
- The commonly used Shields diagram presented in Vanoni (1975), which presents stable particle sizes under different flow velocities measured parallel to the particle bed

Stable particles sizes at the mouths of Onondaga Creek and Ninemile Creek were computed using estimated velocities and water depths from the hydrodynamic models. Additionally, the stable particle size necessary to resist typical Lake current velocities was assessed using current velocities measured in the littoral zone (less than 9 meters) in 1987 by Effler (1996). The maximum particle size obtained from these two methods was conservatively selected as the stable sediment particle for the sediment cap armor layer due to current velocities.

## 6.4 Results

This section summarizes the results of the tributary analysis and associated armor layer sizing for each tributary. A detailed example calculation is included as Attachment C.

### 6.4.1 Ninemile Creek

Figure 6-5 presents the 100-year flood flow velocity magnitude for Ninemile Creek. Additionally, Figure 6-6 presents the 100-year flood flow velocity along the approximate discharge centerline from Ninemile Creek into Onondaga Lake. The predicted velocities decrease almost linearly with distance from the mouth of Ninemile Creek. Velocities along the discharge centerline where a sediment cap is proposed ranged from 0.7 to 3.8 fps.

Stable sediment particle sizes for the sediment cap armor layer were calculated in accordance with the procedure presented in Section 6.3.2 and are presented in Table 6-3. The sediment type required to resist the 100-year flood flow ranges from coarse gravel at the nearshore edge of the sediment cap to medium sand at the offshore edge of the sediment cap.

**Table 6-3**
**Stable Particle Sizes along the Discharge Centerline from Ninemile Creek**

| Distance Offshore (feet)[a] | Computed Velocity (fps) | Median Particle Diameter (inches) | | Design Median Particle Size (inches) | Design Median Particle Size (millimeters) | Sediment Type[b] |
|---|---|---|---|---|---|---|
| | | Maynord (1998) | Vanoni (1975) | | | |
| 0 | 3.8 | 1.00 | 0.71 | 1.00 | 25.5 | coarse gravel |
| 79 | 3.4 | 0.77 | 0.59 | 0.77 | 19.5 | coarse gravel |
| 251 | 2.8 | 0.52 | 0.35 | 0.52 | 13.2 | fine gravel |
| 363 | 2.3 | 0.30 | 0.28 | 0.30 | 7.7 | fine gravel |
| 551 | 1.9 | 0.19 | 0.18 | 0.19 | 4.8 | coarse sand |
| 749 | 1.4 | 0.08 | 0.08 | 0.08 | 2.2 | coarse sand |
| 1,038 | 1.1 | 0.05 | 0.06 | 0.06 | 1.6 | medium sand |
| 1,466 | 0.7 | 0.01 | 0.02 | 0.02 | 0.6 | medium sand |
| 1,529 | 0.7 | 0.01 | 0.02 | 0.02 | 0.6 | medium sand |
| 1,922 | 0.6 | 0.01 | 0.02 | 0.02 | 0.4 | fine sand |

Notes:
a. Sediment cap extends approximately 1,450 feet offshore from Ninemile Creek (indicated with shading).
b. Sediment type was classified using the Unified Soil Classification System.

### *6.4.2 Onondaga Creek*

Figure 6-7 presents the 100-year flood flow velocity magnitude for Onondaga Creek. Additionally, Figure 6-8 presents the 100-year flood flow velocity along the approximate discharge centerline from Onondaga Creek into Onondaga Lake. As with Ninemile Creek, the predicted velocities decrease almost linearly with distance from the mouth of Onondaga Creek. In areas where a sediment cap is proposed as the remedy for RA E, velocities along the discharge centerline ranged from 0.9 to 2.7 fps.

Stable sediment particle sizes for the sediment cap armor layer were calculated in accordance with the procedure presented in Section 6.3.2 and are presented in Table 6-4. The sediment type required to resist the 100-year flood flow ranges from fine gravel near the mouth of Onondaga Creek to medium sand at the offshore edge of the sediment cap.

**Table 6-4**
**Stable Particle Sizes along the Discharge Centerline from Onondaga Creek**

| Distance Offshore (feet)[a] | Computed Velocity (fps) | Median Particle Diameter (inches) | | Design Median Particle Size (inches) | Design Median Particle Size (millimeters) | Sediment Type[b] |
|---|---|---|---|---|---|---|
| | | Maynord (1998) | Vanoni (1975) | | | |
| 0 | 2.7 | 0.36 | 0.33 | 0.36 | 9.2 | fine gravel |
| 206 | 2.1 | 0.19 | 0.24 | 0.24 | 6.0 | fine gravel |
| 382 | 1.9 | 0.14 | 0.18 | 0.18 | 4.5 | coarse sand |
| 744 | 1.5 | 0.09 | 0.11 | 0.11 | 2.8 | coarse sand |
| 1,100 | 1.3 | 0.06 | 0.08 | 0.08 | 2.0 | medium sand |
| 1,785 | 0.9 | 0.02 | 0.04 | 0.04 | 1.0 | medium sand |
| 1,990 | 0.8 | 0.02 | 0.03 | 0.03 | 0.8 | medium sand |
| 2,590 | 0.7 | 0.01 | 0.02 | 0.02 | 0.6 | medium sand |

Notes:
a. Sediment cap extends approximately 1,840 feet offshore from Onondaga Creek (indicated with shading).
b. Sediment type was classified using the Unified Soil Classification System.

### *6.4.3 Onondaga Lake Current Velocities*

In addition to evaluating the influence of the tributaries on the stable particle size, the particle size needed to resist current velocities in Onondaga Lake under typical weather conditions were also assessed. Current velocities range from 0.02 to 0.25 fps in the littoral zone (less than 9 meters) as measured in 1987 by Effler (1996). Using the methods presented in Section 6.3.2, these measured velocities result in a stable particle size less than fine sands (Table 6-5).

**Table 6-5**
**Stable Particle Sizes for Typical Onondaga Lake Current Velocities**

| Measured Velocity (fps)[a] | Median Particle Diameter (inches) | | Design Median Particle Size (inches) | Sediment Type[b] |
|---|---|---|---|---|
| | Maynord (1998) | Vanoni (1975) | | |
| 0.17 | <0.001 | <0.004 | 0.004 | fine sand |
| 0.02 | <0.001 | <0.004 | 0.004 | fine sand |
| 0.25 | 0.001 | <0.004 | 0.004 | fine sand |
| 0.04 | <0.001 | <0.004 | 0.004 | fine sand |
| 0.18 | <0.001 | <0.004 | 0.004 | fine sand |
| 0.03 | <0.001 | <0.004 | 0.004 | fine sand |

Notes:
a. Measured velocities include values reported by Effler (1996) in the littoral zone (<9 meters).
b. Sediment type was classified using the Unified Soil Classification System.

## 6.5 Sensitivity Analyses

Sensitivity analyses were performed by varying Manning's roughness coefficient and downstream (e.g., lake) water surface elevation. Table 6-6 presents the various input parameters for the sensitivity simulations. The downstream water surface elevation was varied between 366.96 feet NAVD88 (lower bound 100-year flood level) and 371.23 feet NAVD88 (upper bound 100-year flood level). Manning's roughness coefficient was varied for each material type as shown below:

- Ninemile Creek Channel: 0.03 to 0.04
- Ninemile Creek Floodplains: 0.05 to 0.15
- Onondaga Creek Channel and Onondaga Lake: 0.025 to 0.035

**Table 6-6**
**Summary of Input Parameters for Sensitivity Simulations**

| | | Upstream BC | Downstream BC | Manning's Roughness Coefficient | | |
|---|---|---|---|---|---|---|
| Tributary | Simulation | Flow (cfs) | Water Surface Elevation (feet, NAVD88) | Lake | Tributary | Floodplain |
| Onondaga Creek | Base Run | 4,890 | 366.96 | 0.03 | 0.03 | NA |
| | A | 4,890 | 366.96 | 0.035 | 0.035 | NA |
| | B | 4,890 | 366.96 | 0.025 | 0.025 | NA |
| | C | 4,890 | 371.23 | 0.03 | 0.03 | NA |
| Ninemile Creek | Base Run | 3,756 | 366.96 | 0.03 | 0.035 | 0.1 |
| | A | 3,756 | 366.96 | 0.035 | 0.04 | 0.15 |
| | B | 3,756 | 366.96 | 0.025 | 0.03 | 0.05 |
| | C | 3,756 | 371.23 | 0.03 | 0.035 | 0.1 |

Note:
NA = Not applicable

Tables 6-7 and 6-8 present the results of the sensitivity analysis for Ninemile Creek. A comparison of velocities and stable particle sizes for the range of Manning's roughness coefficients shows the Base Run predicts generally the same material necessary for the armor layer when comparing the Base Run with Simulations A and B (Table 6-7). A slightly coarser material (coarse gravel versus fine gravel) is predicted at the initial 250 feet of the sediment cap with the Base Run and Simulation A as compared with Simulation B. Additionally, a slightly coarser material (medium sand versus fine sand) is predicted at the outer edge of the sediment cap with the Base Run as compared with Simulation A. A comparison of stable particle sizes for differing water surface elevations indicates a larger material would be required near the mouth of Ninemile Creek using the Base Run (lower bound) as compared to Simulation C (upper bound) (Table 6-8). Furthermore, this particle size is below the particle size required to resist wind-generated waves.

**Table 6-7**
**Summary of Sensitivity Analysis for Ninemile Creek - Manning's Roughness Coefficient**

| Distance Offshore (feet)[b] | Manning's Roughness Coefficient | | | | | | Sediment Type from Wind-Wave Analysis[a] |
|---|---|---|---|---|---|---|---|
| | Base Run - Mid Values | | Simulation A - Upper Values | | Simulation B - Lower Values | | |
| | Velocity (fps) | Sediment Type[c] | Velocity (fps) | Sediment Type | Velocity (fps) | Sediment Type | |
| 0 | 3.8 | coarse gravel | 4.1 | coarse gravel | 3.3 | fine gravel | 1.5-inch stone |
| 79 | 3.4 | coarse gravel | 3.6 | coarse gravel | 3.0 | fine gravel | 1.5-inch stone |
| 251 | 2.8 | fine gravel | 2.8 | fine gravel | 2.6 | fine gravel | 1.5-inch stone |
| 363 | 2.3 | fine gravel | 2.2 | fine gravel | 2.1 | fine gravel | 1.5-inch stone |
| 551 | 1.9 | coarse sand | 1.7 | coarse sand | 1.8 | coarse sand | 1.5-inch stone |
| 749 | 1.4 | coarse sand | 1.2 | medium sand | 1.4 | coarse sand | fine gravel |
| 1,038 | 1.1 | medium sand | 0.9 | medium sand | 1.0 | medium sand | medium sand |
| 1,466 | 0.7 | medium sand | 0.6 | fine sand | 0.3 | fine sand | fine sand |
| 1,529 | 0.7 | medium sand | 0.6 | fine sand | 0.2 | fine sand | NA |
| 1,922 | 0.6 | fine sand | 0.5 | fine sand | 0.2 | fine sand | NA |

Notes:
a. See Section 5 for description of wind-wave analysis and results.
b. Sediment cap extends approximately 1,450 feet offshore from Ninemile Creek (indicated with shading).
c. Sediment type was classified using the Unified Soil Classification System.

**Table 6-8**

**Summary of Sensitivity Analysis for Ninemile Creek – Water Surface Elevation**

| Distance Offshore (feet)[b] | Water Surface Elevation (feet, NAVD88) | | | | Sediment Type from Wind-Wave Analysis[a] |
|---|---|---|---|---|---|
| | Base Run - Lower Bound 100-year Flood | | Simulation C - Upper Bound 100-year Flood | | |
| | Velocity (fps) | Sediment Type[c] | Velocity (fps) | Sediment Type | |
| 0 | 3.8 | coarse gravel | 2.0 | fine gravel | 1.5-inch stone |
| 79 | 3.4 | coarse gravel | 1.8 | coarse sand | 1.5-inch stone |
| 251 | 2.8 | fine gravel | 1.5 | coarse sand | 1.5-inch stone |
| 363 | 2.3 | fine gravel | 1.3 | medium sand | 1.5-inch stone |
| 551 | 1.9 | coarse sand | 1.2 | medium sand | 1.5-inch stone |
| 749 | 1.4 | coarse sand | 0.9 | medium sand | fine gravel |
| 1,038 | 1.1 | medium sand | 0.8 | medium sand | medium sand |
| 1,466 | 0.7 | medium sand | 0.7 | medium sand | fine sand |
| 1,529 | 0.7 | medium sand | 0.6 | fine sand | NA |
| 1,922 | 0.6 | fine sand | 0.7 | medium sand | NA |

Notes:
a. See Section 5 for description of wind-wave analysis and results.
b. Sediment cap extends approximately 1,450 feet offshore from Ninemile Creek (indicated with shading).
c. Sediment type was classified using the Unified Soil Classification System.

Tables 6-9 and 6-10 present the results of the sensitivity analysis for Onondaga Creek. A comparison of velocities and stable particle sizes for the range of Manning's roughness coefficients shows similar results for the all three simulations (i.e., Base Run, Simulation A, and Simulation B; Table 6-9). A comparison of stable particle sizes for differing water surface elevations indicates a slightly larger material would be required near the mouth of Onondaga Creek using the Base Run (lower bound) as compared to Simulation C (upper bound). Furthermore, the particle size is below the necessary particle size required to resist wind-generated waves.

## Table 6-9
## Summary of Sensitivity Analysis for Onondaga Creek – Manning's Roughness Coefficient

| Distance Offshore (feet)[c] | Manning's Roughness Coefficient | | | | | | Sediment Type from Wind-Wave Analysis[a] |
|---|---|---|---|---|---|---|---|
| | Base Run - Mid Values | | Simulation A - Upper Values | | Simulation B - Lower Values | | |
| | Velocity (fps) | Sediment Type[d] | Velocity (fps) | Sediment Type | Velocity (fps) | Sediment Type | |
| 0 | 2.7 | fine gravel | 2.7 | fine gravel | 2.7 | fine gravel | fine gravel[b] |
| 206 | 2.1 | fine gravel | 2.0 | fine gravel | 2.1 | fine gravel | fine gravel[b] |
| 382 | 1.9 | coarse sand | 1.8 | coarse sand | 1.9 | coarse sand | fine gravel[b] |
| 744 | 1.5 | coarse sand | 1.5 | coarse sand | 1.6 | coarse sand | fine gravel |
| 1,100 | 1.3 | medium sand | 1.2 | medium sand | 1.4 | coarse sand | fine gravel |
| 1,785 | 0.9 | medium sand | 0.8 | medium sand | 1.0 | medium sand | medium sand |
| 1,990 | 0.8 | medium sand | 0.8 | medium sand | 0.9 | medium sand | fine sand |
| 2,590 | 0.7 | medium sand | 0.6 | fine sand | 0.8 | medium sand | fine sand |

Notes:
a. See Section 5 for description of wind-wave analysis and results.
b. A median stone size of 3 inches is proposed throughout the navigation channel, as it is necessary on the side slopes for protection from wind-waves.
c. Sediment cap extends approximately 1,840 feet offshore from Onondaga Creek (indicated with shading).
d. Sediment type was classified using the Unified Soil Classification System.

**Table 6-10**
**Summary of Sensitivity Analysis for Onondaga Creek – Water Surface Elevation**

| Distance Offshore (feet)[c] | Water Surface Elevation (feet, NAVD88) | | | | Sediment Type from Wind-Wave Analysis[a] |
|---|---|---|---|---|---|
| | Base Run - Lower Bound 100-year Flood | | Simulation C - Upper Bound 100-year Flood | | |
| | Velocity (fps) | Sediment Type[d] | Velocity (fps) | Sediment Type | |
| 0 | 2.7 | fine gravel | 2.1 | fine gravel | fine gravel[b] |
| 206 | 2.1 | fine gravel | 1.7 | coarse sand | fine gravel[b] |
| 382 | 1.9 | coarse sand | 1.5 | coarse sand | fine gravel[b] |
| 744 | 1.5 | coarse sand | 1.3 | medium sand | fine gravel |
| 1,100 | 1.3 | medium sand | 1.1 | medium sand | fine gravel |
| 1,785 | 0.9 | medium sand | 0.8 | medium sand | medium sand |
| 1,990 | 0.8 | medium sand | 0.8 | medium sand | fine sand |
| 2,590 | 0.7 | medium sand | 0.7 | medium sand | fine sand |

Notes:

a. See Section 5 for description of wind-wave analysis and results.
b. A median stone size of 3 inches is proposed throughout the navigation channel, as it is necessary on the side slopes for protection from wind-waves.
c. Sediment cap extends approximately 1,840 feet offshore from Onondaga Creek (indicated with shading).
d. Sediment type was classified using the Unified Soil Classification System.

## 6.6 Wave and Current Interaction

An additional analysis was performed to assess the potential simultaneous combination of erosive forces from wind-generated waves and tributary outflows. The evaluation was performed for RA E, conservatively assuming that two low-frequency, extreme events (a 10-year wind-wave event and the 10-year flood flow from Onondaga Creek) occurred simultaneously. While the probability of this occurrence is extremely low, this calculation was performed to compare the predicted maximum bottom velocities during the combined event with the 100-year wind-wave event.

The hydrodynamic model described above was used to simulate velocities in Onondaga Lake as a result of the 10-year flood flow event in Onondaga Creek. The 10-year return interval wind-generated wave height was computed for RA E following the methodology outlined in

Section 5. The computed 10-year wave has a significant wave height of 3.6 feet and a period of 3.4 seconds.

The first step in this analysis was to compute the wave height transformation from deep water to the location of interest. This change in wave height is quantified by the shoaling coefficient $K_s$, where $c_0$ is the deep water wave celerity (in fps) and $c_g$ is the local group celerity (in fps):

$$K_s = \sqrt{\frac{c_0}{2c_g}}$$

Unna (1942) developed a formulation that allows the local wave speed to be calculated for a wave in a constant depth and uniform current as shown below:

$$c = \frac{1}{2} c_0 \tanh(2kh) \left( 1 + \sqrt{1 + \frac{4U \coth(kh)}{c_0 \tanh(2kh)}} \right)$$

Where:

| | | |
|---|---|---|
| c | = | local wave celerity (fps) |
| U | = | current velocity (fps) |
| k | = | wave number (feet$^{-1}$) |
| h | = | local water depth (feet) |

The local group celerity is related to wave celerity by the equation below:

$$c_g = \frac{c}{2}\left(1 + \frac{2kh}{\sinh(2kh)}\right)$$

The group celerity is used to calculate the shoaling coefficient. The shoaled wave height at each location of interest is then calculated by multiplying the deep water wave height (3.6 feet in this case) by the corresponding shoaling coefficient.

The maximum bottom velocities for a given water depth, wave height, and current velocity were then computed following the numerical method developed by Chaplin (1990). This method is based on wave theory that was first developed by Dean (1965), and utilizes multiple orders of nonlinearity to provide solutions of wave profiles and dynamics for waves from deep water to near breaking conditions, and allows for inclusion of a uniform current. The analysis was performed for water depths up to 30 feet (the water depth at the RA E offshore boundary). The results were compared with the maximum bottom velocities computed for the 100-year wind-wave event. Table 6-11 presents the results of the analysis.

**Table 6-11**
**Wave and Velocity Results for the 10-year Wave and 10-year Flow Combination**

| Water depth (ft) | Opposing Current from Onondaga Creek (fps) | Wave Height (ft) | Maximum Bottom Velocity (fps) | Maximum 100-year Wave Bottom Velocity (fps) |
|---|---|---|---|---|
| 30 | 0.50 | 3.62 | 0.76 | 0.71 |
| 20 | 0.65 | 3.53 | 1.3 | 1.5 |
| 15 | 0.72 | 3.45 | 1.8 | 2.1 |
| 10 | 1.00 | 3.44 | 2.7 | 3.1 |
| 8 | 1.30 | 3.55 | 3.3 | 3.8 |
| 6 | 1.30 | 3.70 | 3.3 | Wave Breaking |

At equivalent depths, the maximum bottom velocities induced by the 10-year flood flow and 10-year wave combination are comparable to or less than those from the 100-year wave event (see Table A-3 of Attachment A). These results indicate that using the 100-year wave event is protective for the design of armor layer material.

# 7 VESSEL EFFECTS ANALYSIS

This section summarizes the analysis used to evaluate the stable particle sizes to resist propeller wash from commercial and recreational vessels that might operate in Onondaga Lake. In addition, an analysis was performed to evaluate the potential for vessel-generated wake waves associated with the vessels that may operate on Onondaga Lake. The analysis was conducted to refine and optimize cap designs for long-term stability and performance by evaluating the size of armor stone that would resist the erosive forces from the propeller wash generated by boats operating on Onondaga Lake.

## 7.1 Summary

A propeller wash and vessel wake analysis was conducted to evaluate the stable particle sizes to resist propeller wash from commercial and recreational vessels that currently, or may in the future, use Onondaga Lake. Both commercial and recreational vessels were evaluated over a range of water depths and operating conditions.

The results of the analysis were compared with the stable particle sizes to resist erosion by wind-generated waves. Based on the analysis, 1- to 2-inch coarse gravel is recommended for the armor layer in the NYSCC navigation channel to resist propeller wash. Outside of the navigation channel, the particle sizes necessary to withstand the wind-generated waves are protective against the expected frequency and magnitude of propeller wash expected under typical operating conditions. In the event that a disturbance to the surface of the cap from localized propeller wash or boat anchor occurs, the disturbed area is expected to "self-level" following removal of the anchor from deposition and redistribution of the habitat layer.

The results of the vessel wake analysis indicate that designing the armor layer to protect the chemical isolation layer from 100-year wind-generated waves will also protect against vessel-generated waves.

## 7.2 Propeller Wash

As a vessel or boat moves through the water, the propeller produces an underwater jet of water. This turbulent jet is known as propeller wash (or propwash). If this jet reaches the bottom, it can contribute to resuspension or movement of bottom particles. Based on a review of the types of vessels and operating procedures for these vessels in Onondaga Lake, there will generally be two types of vessel operations over the sediment cap:

1. Commercial and recreational vessels operating frequently in the NYSCC navigation channel to the Inner Harbor in RA E
2. Recreational vessels operating randomly in shallower water depths

The propeller wash analysis consisted of the following major components:

1. Obtaining information of the types of commercial and recreational vessels that use Onondaga Lake and their operating characteristics
2. Obtaining the vessel characteristics (such as draft and engine horsepower)
3. Selecting representative vessels to be used in the design
4. Computing the particle size necessary to withstand the erosive forces associated with propeller wash at various water depths

The details of the methodology are presented in Section 7.3. A detailed example calculation is included as Attachment D.

## 7.3 Propeller Wash Methodology

This section describes the methodology used to estimate the particle size that will withstand the erosive forces associated with propeller wash. The results of the analyses are presented in Section 7.3 of this appendix.

### *7.3.1 Design Vessels*

A variety of vessels operate in Onondaga Lake, including tugboats, a passenger vessel, and a variety of private recreational vessels. The first step in the analysis was to gather information about these vessels including specific design characteristics and typical operating procedures. The characteristics of various vessels were considered, and representative recreational design vessels were selected for analysis.

There are two types of commercial vessels that use Onondaga Lake – tugboats and a passenger vessel. Discussions with NYSCC representatives and barge operators indicate that

Pellegrino Marine operates two tugs on the Lake: the *Sean* and the *Mavret H*. Mid-Lake Navigation Corporation operates the *Emita II*, a 42-person passenger vessel. Previous discussions with tug operators indicate that their vessels operate in the deeper portion of the Lake and use an average of 25 percent of their horsepower (Parsons 2004). Table 7-1 shows the pertinent dimensions used in the propeller wash for these vessels. These vessels are considered representative of the types of commercial vessels that may use the Lake in the future.

**Table 7-1**
**Commercial Vessel Characteristics**

| Vessel Class | Vessel | Propeller Shaft Depth (feet) | Number of Engines | Engine Horsepower | Propeller Dimensions (feet) | Ducted Propeller |
|---|---|---|---|---|---|---|
| Passenger Vessel | *Emita II* | 5.5 | 1 | 200 | 3.5 | No |
| Tugboat | *Mavret H* | 3 | 1 | 800 | 4.67 | Yes |
| | *Sean* | 3 | 2 | 600 total | 2.2 | No |

In addition to these commercial-type vessels, several different types of recreational vessels operate on Onondaga Lake. The various types of recreational vessels that currently use the Lake and their operational parameters were determined based on discussions with Onondaga County personnel. In general, the vessels can be organized into six general categories:

- Ski and fishing boats
- Bass boats
- High performance/power boats
- Sail boats
- Sports yachts
- Others (pontoon boats/jet skis)

Table 7-2 summarizes the types of vessels from annual tenants from the Onondaga Lake Marina located on the eastern shore of the Lake in Liverpool.

**Table 7-2**
**Types of Recreational Vessels from Onondaga Lake Marina**

| Category | Number of Vessels | % of Total |
|---|---|---|
| Ski/Fishing Boat | 30 | 26 |
| Bass Boat | 29 | 26 |
| Sail Boat | 22 | 19 |
| Sports Yacht | 20 | 18 |
| Other (inflatable, pontoon, jet ski) | 7 | 6 |
| High Performance/Power Boats | 6 | 5 |
| Total | 114 | 100 |

The majority (over 50 percent) of vessels surveyed are characterized as ski/fishing boats and bass boats. Based on discussions with Onondaga County, fishing boats are the primary users of the Lake with sailboats using the Lake frequently on weekends. The larger vessels (high performance power boats and sports yachts) are limited in number and are not frequently used on the Lake. As opposed to these larger vessels, smaller vessels (such as ski/fishing boats and bass boats) can operate in shallower water and may use a significant amount of their available horsepower.

Representative vessels from the ski/fishing, bass boat, and high performance power boat category were used in this propeller wash analysis. Table 7-3 summarizes characteristics of these representative vessels.

**Table 7-3**
**Representative Recreational Vessel Characteristics**

| Vessel Class | Vessel | Propeller Shaft Depth (feet) | Number of Engines | Engine Horsepower | Propeller Dimensions (inches) |
|---|---|---|---|---|---|
| Bass Boat | *Nitro 929* | 1.17 | 1 | 270 | 14.625 |
| Ski and Fishing Boat | *Triumph 191* | 2.5 | 1 | 150 | 16 |
| High Performance Boat | *Baja Outlaw 23* | 2.75 | 1 | 375 | 17 |

### 7.3.2 Design Approach

The propeller wash analysis for the commercial vessels operating in deeper waters was conducted using the methods presented in USEPA's *Armor Layer Design for the Guidance for In-Situ Subaqueous Capping of Contaminated Sediment* (Maynord 1998). These methods are based on the relationships developed by Blaauw and van de Kaa (1978) and Verhey (1983). This USEPA model considers physical vessel characteristics (e.g., propeller diameter, depth of propeller shaft, and total engine horsepower) and operating/site conditions (applied horsepower, water depth, etc.) to estimate propeller-induced bottom velocities at various distances behind the propeller. The model can be used to predict the particle size that would be stable when subjected to the steady-state (i.e., maneuvering vessel where the speed of the vessel is essentially zero) propeller wash from the modeled vessel. In the case of non-steady-state conditions (i.e., moving vessel), the use of this model is conservative since the propeller wash force is transient in nature, only impacting a fixed point on the bottom for a short time.

Certain model components are based on large ocean-going vessels operating at very slow speeds (e.g., maneuvering operations), and therefore are not applicable to much smaller recreational vessels. The methods presented in the USEPA guidance (Maynord 1998) and technical literature (Verhey 1983; Blaauw and van de Kaa 1978) are based on large ocean-going vessels operating at very slow speeds (e.g., maneuvering operations), and therefore are not fully applicable to the smaller, fast-moving recreational vessels that typically operate in the shallower waters of Onondaga Lake. Specifically, the model does not properly consider the angle of the propeller (the propeller angling downward toward the bed as the boat is starting up) or the transient (i.e., moving vessel) nature characteristic of recreational propeller wash. A more detailed analysis of the propeller wash from recreational vessels was conducted using a refined modeling framework specifically developed for evaluating recreational propeller wash.

The refined modeling approach for evaluating the propeller wash from recreational vessels involved adapting the predictive equations developed for the larger vessels (based on USEPA guidance) to address smaller recreational vessels under moving conditions. The refinements were based, in part, on results of a field study where bottom-mounted current meters were used to measure actual bottom velocities of maneuvering and passing recreational vessels in the Fox River. This refined approach was successfully applied and accepted by USEPA

(Region V) for the design of the Lower Fox River remediation to evaluate the effects of propeller wash for the design of the armor layer of a sediment isolation cap (Shaw and Anchor 2007).

Both of the approaches (for maneuvering commercial vessels and transient recreational vessels) summarized above were utilized to evaluate stable particle sizes to resist propeller wash from a range of vessel and operating/site conditions.

## 7.4 Propeller Wash Results

This section summarizes the results of the propeller wash analysis. As described above, a detailed example calculation is included as Attachment D. Based on previous discussions with tugboat operators and Mid-Lakes Navigation representatives, these vessels operate primarily in the deeper portion of the Lake and at 25 percent of their horsepower (Parsons 2004). One area in the future where these types of vessels may operate more frequently is the NYSCC navigation channel leading to the Inner Harbor in RA E. The navigation channel is authorized by the State of New York. At the time of dredging plan development, the authorized channel depth was unknown, and Honeywell awaits confirmation of the authorized channel depth, as well as the side slope configuration, from the NYSCC. For the propeller wash analysis, a water depth of 14 feet was used (an authorized depth of 12 feet plus 2 feet below authorized dredge depth to prevent dredge-induced damage to the cap associated with future navigational dredging). To assess the range of particle sizes that would be stable under varying propeller wash events from large commercial vessels, calculations were made using the USEPA guidance (Maynord 1998) method for a range of applied horsepower (10, 25, and 50 percent of the total installed power) as well as a range of water depths (14 feet, 20 feet, and 30 feet) for the *Emita II* passenger vessel and the *Mavret H* tugboat (representing these vessel classes). These operating conditions are considered conservative since most of the Lake is deeper than 30 feet and these vessels would be limited in operating in the nearshore regions due to their draft. Table 7-4 presents a summary of the stable median particle sizes (D50) for various water depths and applied horsepower for the *Emita II* passenger vessel and the *Mavret H* tugboat.

**Table 7-4**
**Stable Particle Sizes for Commercial Vessels**

| Vessel Class | Representative Vessel | Water Depth (feet) | Applied Horsepower (Percent) | Median Particle Size $D_{50}$ (inches) | Median Particle Size $D_{50}$ (millimeters) | Particle Size Type |
|---|---|---|---|---|---|---|
| Commercial Passenger Vessel | *Emita II* | 14 | 10 | 0.5 | 13 | Fine Gravel |
| | | | 25 | 0.9 | 23 | Coarse Gravel |
| | | | 50 | 1.5 | 37 | Coarse Gravel |
| | | 20 | 10 | 0.2 | 4 | Coarse Sand |
| | | | 25 | 0.3 | 8 | Fine Gravel |
| | | | 50 | 0.5 | 13 | Fine Gravel |
| | | 30 | 10 | 0.1 | 2 | Medium Sand |
| | | | 25 | 0.1 | 3 | Coarse Sand |
| | | | 50 | 0.2 | 4 | Coarse Sand |
| Tugboat | *Mavret H* | 14 | 10 | 1.1 | 27 | Coarse Gravel |
| | | | 25 | 1.9 | 49 | Coarse Gravel |
| | | | 50 | 3.1 | 78 | Cobbles |
| | | 20 | 10 | 0.4 | 11 | Fine Gravel |
| | | | 25 | 0.8 | 21 | Coarse Gravel |
| | | | 50 | 1.3 | 33 | Coarse Gravel |
| | | 30 | 10 | 0.2 | 4 | Coarse Sand |
| | | | 25 | 0.3 | 8 | Fine Gravel |
| | | | 50 | 0.5 | 13 | Fine Gravel |

Notes:
1. Water depth of 14 feet represents operation in the NYSCC navigation channel.
2. Sediment type was classified using the Unified Soil Classification System.

To assess the range of particle sizes that would be stable under varying propeller wash events for recreational vessels, calculations were made using the refined USEPA methodology for a range of applied horsepower (25, 50, 75, and 100 percent of total installed power), as well as a range of water depths to the top of the underlying armor layer for the three representative vessels outlined in Table 7-3. The minimum water depth for vessel operation that was evaluated was approximately 1 foot off each vessel's propeller to the top of the cap (i.e. habitat layer). In shallow water, a dedicated 1.5- to 2-foot-thick habitat layer is planned for placement above the armor and chemical isolation layer. The analysis was performed for water depths to as deep as 10 feet. These scenarios represent the range of typical recreational vessels operating in shallow water. Table 7-5 presents a summary of the stable particle sizes for various water depths and applied horsepower for these vessels.

**Table 7-5**
**Stable Particle Sizes for Recreational Vessels**

| Vessel Class | Representative Vessel | Water Depth to Armor Layer (feet) | Applied Horsepower (Percent) | Median Particle Size $D_{50}$ (inches) | Median Particle Size $D_{50}$ (millimeters) | Particle Size Type |
|---|---|---|---|---|---|---|
| Bass Boat | *Nitro 929* | 4 | 25 | 0.4 | 10 | Fine Gravel |
| | | | 50 | 0.6 | 15 | Fine Gravel |
| | | | 75 | 0.7 | 18 | Fine Gravel |
| | | | 100 | 0.9 | 23 | Coarse Gravel |
| | | 5 | 25 | 0.1 | 3 | Coarse Sand |
| | | | 50 | 0.1 | 3 | Coarse Sand |
| | | | 75 | 0.2 | 5 | Coarse Sand |
| | | | 100 | 0.2 | 5 | Coarse Sand |
| | | 10 | 25 | 0.003 | 0.1 | Fine Sand |
| | | | 50 | 0.004 | 0.1 | Fine Sand |
| | | | 75 | 0.005 | 0.1 | Fine Sand |
| | | | 100 | 0.007 | 0.2 | Fine Sand |
| Ski and Fishing Boat | *Triumph 191* | 5 | 25 | 0.7 | 18 | Fine Gravel |
| | | | 50 | 0.8 | 20 | Coarse Gravel |
| | | | 75 | 0.9 | 23 | Coarse Gravel |
| | | | 100 | 1.1 | 28 | Coarse Gravel |
| | | 6 | 25 | 0.1 | 3 | Coarse Sand |
| | | | 50 | 0.2 | 5 | Coarse Sand |
| | | | 75 | 0.2 | 5 | Coarse Sand |
| | | | 100 | 0.2 | 5 | Coarse Sand |
| | | 10 | 25 | 0.005 | 0.1 | Fine Sand |
| | | | 50 | 0.007 | 0.2 | Fine Sand |
| | | | 75 | 0.007 | 0.2 | Fine Sand |
| | | | 100 | 0.008 | 0.2 | Fine Sand |

| Vessel Class | Representative Vessel | Water Depth to Armor Layer (feet) | Applied Horsepower (Percent) | Median Particle Size $D_{50}$ (inches) | Median Particle Size $D_{50}$ (millimeters) | Particle Size Type |
|---|---|---|---|---|---|---|
| High Performance Boat | *Baja Outlaw 23* | 6 | 25 | 0.2 | 5 | Coarse Sand |
| | | | 50 | 0.3 | 8 | Fine Gravel |
| | | | 75 | 0.4 | 10 | Fine Gravel |
| | | | 100 | 0.5 | 13 | Fine Gravel |
| | | 10 | 25 | 0.01 | 0.2 | Fine Sand |
| | | | 50 | 0.01 | 0.3 | Fine Sand |
| | | | 75 | 0.01 | 0.3 | Fine Sand |
| | | | 100 | 0.02 | 0.4 | Medium Sand |

Notes:
1. Sediment type was classified using the Unified Soil Classification System.
2. The shallowest water depth analyzed for each vessel was approximately 1 foot below the depth of the propeller.

## 7.5 Assessment of Propeller Wash for the Onondaga Lake Cap Design

The propeller wash analysis performed for Onondaga Lake indicates that particle sizes in the coarse gravel range (1 to 2 inches) would be stable in the NYSCC navigation channel when subjected to propeller wash forces from larger commercial vessels operating under the range of potential conditions identified above.

For the other areas of the cap (primarily in the nearshore areas), recreational vessels will likely operate randomly; that is, these vessels will not start and stop or regularly pass over the exact same location on a regular basis, and therefore the cap armor layer will not be subjected to repeated unidirectional propeller wash. Table 7-6 presents a comparison of the stable particle sizes at depths up to 8.5 feet in each remediation area to resist the 100-year wind-generated wave and propeller wash. As can be seen from the table, the particle size(s) predicted to be stable under the propeller wash are comparable to the particle sizes designed to resist wind waves. Due to the limited area impacted by propeller wash from an individual vessel, significant movement of armor layer is not expected from propeller wash. Only 3 percent (approximately 10 acres) of the sediment cap area in RAs A through D have water depths between the surf zone and 5.5 feet. In addition, in shallow water, a dedicated 1.5- to 2-foot-thick habitat layer is planned for placement above the armor and chemical isolation layer. In the event that the habitat materials are disturbed by propeller wash, the disturbed area(s) are expected to "self-level" shortly thereafter due to the natural hydrodynamic process of the Lake, which tends to level out discontinuities in the bottom.

**Table 7-6**
**Comparison of Stable Particle Sizes for Recreational Vessels and Wind-Waves**

| Range of Water Depths Based on Baseline Lake Level (feet) | RA A | RA B | RA C and D | RA E | Range of Stable Particle Sizes for Recreational Vessels |
|---|---|---|---|---|---|
| 8.5 to 6.5 | Coarse Sand | Fine Gravel | Fine Gravel | Coarse Gravel | Coarse Sand |
| 6.5 to 5.5 | Fine Gravel | Fine Gravel | Fine Gravel | Cobbles | Coarse Sand to Fine Gravel |
| 5.5 to 4.5 | Fine Gravel | Fine Gravel | Fine Gravel | Cobbles | Coarse Sand to Coarse Gravel |
| 4.5 to surf zone | Fine Gravel | Fine Gravel | Coarse Gravel | Cobbles | Coarse Sand to Coarse Gravel |
| Within surf zone | Coarse Gravel | Coarse Gravel | Coarse Gravel | Cobbles | Fine to Coarse Gravel |

Notes:

1. Sediment type was classified using the Unified Soil Classification System.
2. The surf zone begins at a depth approximately equal to the breaking wave height.
3. The breaking wave depth is approximately 3.5 feet in RA A and B, 4 feet in RA C and D, and 7 feet in RA E.
4. Range of water depths referenced to the Onondaga Lake baseline water level of 362.5 feet (see Section 4 of this appendix). The water level used for the armor layer design is 0.5 feet lower than the baseline water level (362.0 feet).

## 7.6 Vessel Wake

As indicated in Section 5 of this appendix, wind-generated waves are the dominant waves in Onondaga Lake. Waves can also be generated by a boat moving through the water. These vessel-generated waves are often referred to as wakes. An analysis was performed to evaluate the potential vessel-generated wake wave heights associated with the vessels that may operate on Onondaga Lake. The results of the analysis indicate that designing the armor layer to protect the chemical isolation layer from 100-year wind-generated waves will also protect against vessel-generated waves.

### 7.6.1 Design Approach

Two methods were used in estimating potential vessels wakes:

- Sorensen-Weggel method (Sorensen and Weggel 1984; Weggel and Sorensen 1986) for tugboats and passenger vessels
- Bhowmik et al. (1991) for recreational vessels

The Sorensen-Weggel method is an empirical model (developed from available laboratory and field data on vessel-generated waves) to predict maximum wave height as a function of vessel speed, vessel geometry, water depth, and distance from the sailing line. This model is applicable for various vessel types (ranging from tugboats to large tankers), vessel speeds, and water depths. The method calculates the wave height generated at the bow of a vessel as a function of the vessel speed, distance from the sailing line, water depth, vessel displacement volume, and vessel hull geometry (i.e., vessel length, beam, and draft). The method has been widely tested on different vessels and is recommended for use with vessels having a Froude number between 0.2 and 0.8. The non-dimensional Froude number used in this method is defined as:

$$Fr = \frac{\text{vessel speed}}{\sqrt{\text{g} \times \text{water depth}}}$$

This method is not applicable for vessels moving with higher speeds at smaller water depths (e.g., recreational vessels) because the Froude number is outside the recommended range (0.2 to 0.8).

The Bhowmik et al. (1991) predictive model is based on measurements of waves generated by 12 different recreational boats ranging in length from approximately 11 to 45 feet, with a maximum draft of 2.4 feet in the Illinois and Mississippi Rivers. Vessels included in the Bhowmik et al. studies were a flat-bottom johnboat, a pontoon, a tri-hull, and various V-hulls. Two wave gages were deployed at each of four distances from the sailing line and 246 test runs were conducted. Vessel speeds ranged from 6.2 knots (7.2 mph) to 39.5 knots (45.4 mph). The empirical model relates maximum vessel-generated wave height as a function of vessel speed, draft, length, and distance from the sailing line. The maximum wave height was found to be proportional to the vessel length and vessel draft, and inversely and weakly proportional to the vessel speed. This is a result of the smaller recreational vessels planing at high speeds. The water depth was not found to be significant in the regression analysis, so it was not included in the empirical equation. Because this model is based on measurements of waves generated by 12 different recreational boats, this method was only used for simulations of recreational vessels traveling at various speeds throughout the Lake.

### *7.6.2 Results*

Vessel wakes for a range of vessel operating speeds for representative commercial vessels are presented in Table 7-7. For these calculations, the wave characteristics were estimated at distances of 25, 50, and 100 feet from the sailing line (essentially the centerline) of the vessel. In actuality, distances may be well over 1,000 feet for vessels operating in deeper portions of the Lake. These close distances are considered to be conservative, since wave heights decrease the further you are from the vessel sailing line due to wave propagation and energy dissipation. A detailed example calculation is included as Attachment E. Details are presented below:

**Table 7-7**
**Vessel-Generated Wave Heights for Commercial Vessels**

| Vessel Class | Representative Vessel | Water Depth (feet) | Vessel Speed (mph) | Distance from Sailing Line (feet) | Wave Height (feet) |
|---|---|---|---|---|---|
| Commercial Passenger Vessel | *Emita II* | 14 | 8 | 25 | 1.0 |
| | | | | 50 | 0.8 |
| | | | | 100 | 0.6 |
| | | | 11 | 25 | 1.6 |
| | | | | 50 | 1.3 |
| | | | | 100 | 1.0 |
| | | 30 | 8 | 25 | 1.3 |
| | | | | 50 | 1.0 |
| | | | | 100 | 0.8 |
| | | | 11 | 25 | 2.0 |
| | | | | 50 | 1.7 |
| | | | | 100 | 1.4 |

| Vessel Class | Representative Vessel | Water Depth (feet) | Vessel Speed (mph) | Distance from Sailing Line (feet) | Wave Height (feet) |
|---|---|---|---|---|---|
| Tugboat | *Mavret H* | 14 | 4 | 25 | 0.2 |
| | | | | 50 | 0.1 |
| | | | | 100 | 0.1 |
| | | | 10 | 25 | 2.5 |
| | | | | 50 | 2.0 |
| | | | | 100 | 1.6 |
| | | 30 | 4 | 25 | * |
| | | | | 50 | * |
| | | | | 100 | * |
| | | | 10 | 25 | 3.2 |
| | | | | 50 | 2.6 |
| | | | | 100 | 2.1 |

Note:
* - Froude number <0.2 for this case.

**Commercial Passenger Vessels**: The *Emita II* passenger vessel-generated wave heights were predicted using the Sorensen-Weggel method to range between 0.6 feet to 2.0 feet. These predicted heights were generated in water depths of 14 and 30 feet, and at speeds of 7.0 knots (8 mph) and 9.6 knots (11 mph). Based on conversations with Mid-Lakes Navigation representatives, these are the typical and maximum speeds that the *Emita II* travels in Onondaga Lake. The wave heights were predicted to decrease as the distance from the sailing increases. At a distance of 100 feet from the vessel, the maximum wave height is predicted to be approximately 1.4 feet.

**Tugboats**: The Sorensen-Weggel method was used to predicted wave height generated by the *Mavret H* tugboat ranging between 0.1 feet to 3.2 feet. These predicted heights were generated in similar water depths of 14 and 30 feet, and at speeds of 3.5 knots (4 mph) and 8.7 knots (10 mph). These speeds were considered the range of typical speeds at which tugboats would operate on Onondaga Lake. As described above, these vessels typically operate in the deeper portion of the Lake and at 25 percent of their horsepower. At a distance of 100 feet from the tugboat, the maximum wave height is predicted to be approximately 2.1 feet.

Predicted vessel wakes for a range of vessel operating speeds for representative recreational boats are presented in Table 7-8. Similar to the commercial vessels, the wave characteristics were calculated at distances of 25, 50, and 100 feet from the sailing line (essentially the centerline) of the vessel.

**Table 7-8**
**Vessel-Generated Wave Heights for Recreational Vessels**

| Vessel Class | Representative Vessel | Vessel Speed (mph) | Distance from Sailing Line (feet) | Wave Height (feet) |
|---|---|---|---|---|
| Bass Boat | *Nitro 929* | 8 | 25 | 1.3 |
| | | | 50 | 1.0 |
| | | | 100 | 0.8 |
| | | 12 | 25 | 1.2 |
| | | | 50 | 0.9 |
| | | | 100 | 0.7 |
| Ski and Fishing Boat | *Triumph 191* | 8 | 25 | 1.0 |
| | | | 50 | 0.8 |
| | | | 100 | 0.6 |
| | | 12 | 25 | 0.9 |
| | | | 50 | 0.7 |
| | | | 100 | 0.6 |
| High Performance Boat | *Baja Outlaw 23* | 8 | 25 | 1.7 |
| | | | 50 | 1.3 |
| | | | 100 | 1.0 |
| | | 12 | 25 | 1.5 |
| | | | 50 | 1.2 |
| | | | 100 | 0.9 |
| Sports Yacht | *SeaRay Sundancer* | 8 | 25 | 2.8 |
| | | | 50 | 2.2 |
| | | | 100 | 1.7 |
| | | 12 | 25 | 2.4 |
| | | | 50 | 1.9 |
| | | | 100 | 1.5 |

The Bhowmik et al. method was used to predict waves generated by the *Nitro 929* bass boat, one of the smaller vessels in this class. The predicted wave heights generated by the *Nitro 929* ranged between 0.7 feet to 1.7 feet. These predicted heights were generated at speeds of 7.0 knots (8 mph) and 10.4 knots (12 mph). Likewise, wave heights predicted to be generated by the *SeaRay Sundancer* sports yacht, which is the largest vessel analyzed in this class, ranged between 1.5 feet to 2.8 feet. As described above, the wave heights are inversely proportional to vessel speed. At a distance of 100 feet from the boats, the maximum wave height is predicted to be approximately 1.7 feet.

The 100-year design wind-generated wave heights range from 2.6 feet in RA A to 5.2 feet in RA E. Therefore, the wave analysis focuses on wind-generated waves and not vessel-induced waves.

## 7.7 Anchor Drag and Wading

Commercial vessel anchoring is not expected to occur over the sediment cap and is expected to be controlled via institutional controls to be implemented as part of the remedy, which will dissuade such anchoring over the capped areas. Recreational vessel anchoring over the sediment cap is likely, but will not be subject to the institutional controls applicable to commercial vessel anchoring. However, the armoring component of the sediment cap that underlies the habitat layer and overlies the chemical isolation layer will provide penetration resistance from recreational boat anchors from disturbing the underlying cap. In the event that a disturbance to the surface habitat layer of the cap from a boat anchor occurs, the disturbed area is expected to "self-level" following removal of the anchor as a result of redistribution of the habitat layer caused by the natural hydrodynamics of the Lake.

An analysis was performed to evaluate the sediment cap's ability to support human foot traffic (such as wading into shallow water for fishing or entering or exiting a boat). Shallow water sediment caps were designed to support the weight of an individual walking on the surface, consistent with USEPA and USACE cap design guidance. The safety factor for the sediment cap is 5 to 15 times greater than the required safety under the range of nearshore cap thicknesses, and thus will be stable under worst-case bearing loads. An example calculation is included in Attachment F.

## 8 ICE ANALYSIS

Due to the cold temperatures that occur in Central New York in the winter months, Onondaga Lake typically freezes over in the winter. As a result, the potential effects of ice on the sediment cap were evaluated as part of the armor layer design. This section provides a summary of the analysis of icing conditions on Onondaga Lake and the design of the sediment cap armor layer to resist ice impacts.

Ice engineering is a highly specialized field, and it is important that ice processes be evaluated by an experienced professional. A leading technical center of expertise on ice engineering is the USACE Cold Regions Research and Engineering Laboratory (CRREL), located in Hanover, New Hampshire. The evaluation of ice processes for Onondaga Lake was performed by Dr. George Ashton, former Chief of Research and Engineering Directorate at CRREL, who has over 35 years of experience with ice processes. Dr. Ashton's evaluation was based on a field site visit, reviews of published literature on ice processes, review of historical water temperature measurements, observations of ice formation at Onondaga Lake, and evaluation of data from other lakes. The record of ice cover on the Lake from the winter of 1987/1988 through 2002/2003 was examined (a period of 16 years). Dr. Ashton's evaluation was included in Appendix H of the FS and is included as Attachment G to this appendix.

The primary ice scour mechanism of concern for lakes such as Onondaga Lake is the expansion and contraction of ice associated with temperature changes through the winter and spring before breakup and the subsequent movement and pilings of ice at the shoreline due to wind. Occasional ice pilings along the shore of Onondaga Lake have been observed, but these are of limited height (less than 5 feet) and were not considered severe. In the 16 years of observation, only two cases of ice pilings on the shore were noted.

Formation of frazil or anchor ice is not likely to occur at Onondaga Lake due to the size of the Lake and the low exposure to supercooling. Frazil is ice in very small crystals formed in supercooled (below 32 degrees Fahrenheit [°F]) water. While in the supercooled matrix, it can adhere to most materials. In some cases, this frazil can adhere to the bottom sediments. When attached to the bottom, it is often termed anchor ice. Conditions favoring the formation of frazil ice include cooling of the water to below 32°F and sufficient turbulent mixing (e.g., rapids within a river) to entrain the water and crystals to depth. In Onondaga Lake, it is probable that neither condition occurs. The Lake is not of sufficient size and exposure to develop large wind-driven currents, and it is doubtful that the majority of the Lake becomes supercooled. There may be some limited supercooling of the top surface water during the time of initial ice formation, but this will only occur in the absence of mixing with the warmer water below.

Ice freezing to the bottom of the Lake is expected in shallow water at the shoreline of Onondaga Lake. In such cases, it is expected that the normal thickening of ice will encounter the bed, and freezing will continue. Reported ice thicknesses were sparse in the 16 years of record and rarely greater than 8 inches. Estimates of potential ice thickness (based on the degree–day calculation) ranged from 12 to 18 inches. It was determined by Dr. Ashton that the freezing of ice to the Lake bottom is limited to water depths of less than 18 inches (1.5 feet).

To protect the chemical isolation layer of the sediment cap, dredging and capping have been delineated such that the armor layer and chemical isolation layer will be placed below the ice freezing zone described above. Using a low lake water level of 362.0 feet (see Section 4), the ice freezing zone would be 360.5 feet. The armor layer and chemical isolation layer will be placed below an elevation of 360.5 feet to protect against ice scour.

In summary, the sediment cap has been designed to protect the chemical isolation layer from ice scour.

# 9 SMU 3 SHORELINE ENHANCEMENT

This section provides a summary of the analysis of the stable particle size that is proposed for the habitat enhancement activities along the SMU 3 shoreline in RA B. The purpose of these activities along the estimated 1.5 miles of SMU 3 shoreline is to assist in stabilizing calcite deposits, which will reduce the ongoing periodic resuspension and turbidity in the nearshore areas. The shoreline stabilization activities in this area will be integrated with the remedy for the WBs 1-8 site.

SMU 3 (RA B) is located adjacent to WBs 1-8 in a medium-energy environment. The remedy specified in the ROD for area consists of dredging and capping of select areas, as well as stabilization of the shoreline. It is anticipated that the shoreline stabilization will use a combination of bioengineering techniques to provide a natural shoreline area to create transition zones from the low lying area of WBs 1-8 and SMU 3. However, the FS has not been completed, and no remedial approach has been identified for WB 1-8 at this time.

## 9.1 Summary

The surf zone associated with the 10-year return period was selected as the basis of design for defining the treatment area. This is the area with a 10 percent probability of receiving wave action of the specified size in any year. The short-term, periodic events that cause daily or weekly resuspension of materials that impact aquatic plants are the main focus for these stabilization activities. Larger wave events that occur much less frequently do not have the ongoing, periodic impacts to the offshore area.

The treatment area for stabilizing the substrate will be set at the 2.5-foot contour within SMU 3 (360.0 feet) and will extend up the slope to a higher water level elevation of 365.0 feet (see Section 4). The design event for determining the stable particle size should be greater than the design event used to define the surf zone so that the material placed within the surf zone will be stable. However, the design event should not be so conservative as to require unnecessarily large stone sizes that could limit the habitat suitability of the material. As a result, the 10-year return period was used as the basis of design for determining the stable particle size to balance between stability and particle size. Based on this analysis, graded gravel with a median particle size ($D_{50}$) of 1.3 inches will be placed within the surf zone to stabilize the substrate to reduce resuspension, and at the toe of the slope where bioengineering treatments are anticipated. It should be noted that this material will be placed along the entire SMU 3 shoreline to a water depth of 2.5 feet (based on the baseline Lake water level of 362.5 feet), coincident with the depth that demarks the shallow edge of Module 3. As such, there is no overlap of the shoreline stabilization areas with the limited area of Modules 1, 2, or 3 currently planned for RA B.

## 9.2 Design Wave Heights and Stable Particle Size

The 10-year return interval wind-generated wave height was computed for the SMU 3 shoreline (in RA B) following the methodology outlined in Section 5. Table 9-1 summarizes the 10-year design wind speed, computed wave height, and breaking wave height and depth.

**Table 9-1**
**Design Wave Summary for SMU 3 Shoreline**

| Event | Wind Speed (mph) | Significant Wave Height (feet) | Significant Wave Period (seconds) | Breaking Wave Height (feet) | Breaking Wave Depth (feet) |
|---|---|---|---|---|---|
| 10-year | 37.9 | 2.1 | 2.6 | 2.2 | 2.7 |

The armor stone size and gradation for the surf zone for the 10-year wave was computed using the methods summarized in Section 5. The gradation is summarized in Table 9-2.

**Table 9-2**
**Armor Stone Size ($D_{50}$)**
**with a Slope of 50H:1V (for Surf Zone Regime)**

| Gradation | Stone Size (inches) |
|---|---|
| $D_0$ | 0.6 |
| $D_{15}$ | 1.0 |
| $D_{50}$ | 1.3 |
| $D_{85}$ | 1.6 |
| $D_{100}$ | 2.0 |

Note:
Computed using minor displacement (S=3). Minor displacement refers to minimal movement of armor stones and could be related to "rocking" of the armor under extreme wave action. Repairs associated with such events (if any) will be handled as part of a maintenance program.

# 10 EVALUATION OF 6- TO 9-METER ZONE

This section provides a summary of the analysis of relative stability of littoral zone sediments in water depths from 20 to 30 feet (6 to 9 meters). This stability evaluation is utilized in the IDS to evaluate the appropriate sediment depth to consider in defining remedial boundaries and to support technical evaluations related to evaluating the potential placement of a thin-layer cap in this zone.

The first step in the evaluation is to evaluate the stability of the existing sediments in the 20- to 30-foot water depth portions of RAs A, B, and C and at the RA E/SMU 5 boundary. This evaluation included a review of the Lake morphology, sediment texture data, and the stability of the bed under extreme wave events. This section summarizes these analyses.

## 10.1 Summary

Based on a review of Lake morphology, wind-generated waves, and resuspension potential, the 20- to 30-foot water depth region of RAs A, B, and C are net depositional (e.g., new sediments are expected to accumulate over time). Therefore, surficial sediment concentrations in these areas could be used to delineate the remedial boundaries.

In the 20- to 30-foot water depth region in the vicinity of the RA E/SMU 5 boundary, the analysis suggests that resuspension of the existing fine-grained sediments under an extreme wave event would be generally limited to the surface sediments (within the top 1 foot). Therefore, surficial sediment concentrations in this area could be used to delineate the remedial boundaries.

## 10.2 Evaluation of Potential Bed Stability

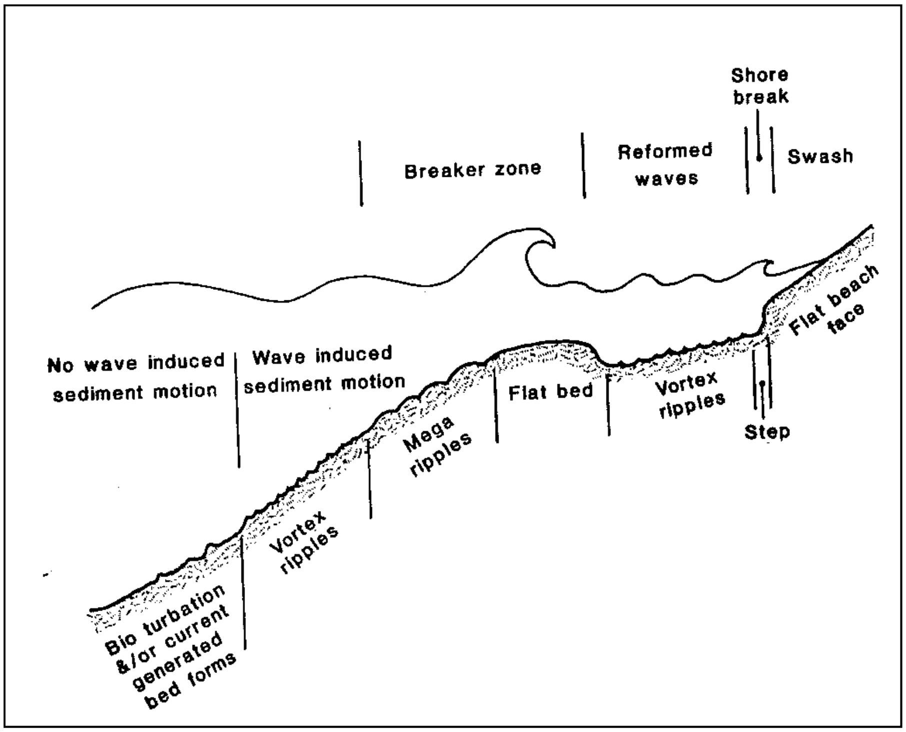
As described by Downing and Rath (1988), many studies have demonstrated that the likelihood of sediment accumulation increases with depth in lakes. Lake bed materials are typically coarser in the high-energy, shallow environments and are usually more fine-grained and flocculated in the deeper water. Effler (1996) reviewed available sediment data in Onondaga Lake and suggested that sediment resuspension would be expected to occur in water depths less than 6 meters (20 feet). Based on their analysis, Effler (1996) concluded that Onondaga Lake regions with depths in excess of 6 to 8 meters (20 to 26 feet) represent the depositional basin of the Lake.

As described in Section 5, the size of wind-generated waves in each remediation area depends on the wind velocity and the fetch distance. To evaluate the relative stability of the existing sediments in the 20- to 30-foot water depth region of each remediation area an analysis was performed on a RA-basis for RAs A, B, and C, and the RA E/SMU 5 boundary. The analysis involved:

1. Reviewing existing sediment texture data in the 20- to 30-foot water depth region to determine the particle size of the sediments.
2. Comparing the horizontal orbital velocities for the 2-year, 10-year, and 100-year design waves in each RA to the commonly used Shields diagram presented in Vanoni (1975), which presents stable particle sizes under different flow velocities measured parallel to the particle bed. The comparison was performed to determine if the existing sediments could potentially be resuspended by wave action.

Details of the wave height and horizontal orbital velocities calculation are presented in Section 5 and Attachment A. It should be noted that Rowan et al. (1992) suggests that critical wave heights to evaluate the mud depositional boundary layer in lakes (i.e., the boundary between the high-energy erosive environment and the low energy depositional areas where fine-grained sediment accumulates) is approximately 77 percent of the maximum wave heights that occur during the one or two largest storms that occur annually. Therefore, for the purpose of this analysis, the 2-year, 10-year, and 100-year extreme events were evaluated.

In addition, in a wave-dominated environment such as Onondaga Lake, the sediment bed outside of the surf zone may move based on a wave's ability to form bedforms. Bedforms are sedimentary structures found on a sediment bed, which may have a large range of sizes and shapes (Nielsen 1992). Examples include bars, dunes, and ripples. The illustration below shows an example bedform distribution on a barred shoreline.




**Example of Bedform Distribution (adapted from Figure 3.2.1 of Nielsen 1992)**

In addition to evaluating the potential for the existing sediments to be resuspended by wave action, an additional evaluation was also performed to determine if the bedforms could develop as a result of wave action in these water depths. In the 20- to 30-foot water depth region, if the wave action is strong enough, vortex ripples can form (see figure above). Vortex ripples are unique to the wave environment, and their scaling is closely tied to wave motion. The size of the vortex ripples is closely linked to the orbital length of the wave-induced fluid motion near the bed. Suspended sediment distribution also tends to scale on ripple height (Nielsen 1992). Specifically, Nielsen (1992) states "…over vortex ripples, the suspended sediment distribution will scale on ripple height, while other bedforms like megaripples and bars, the suspension distribution will scale on flat bed boundary layer thickness which is much smaller than the height of those bedforms." Therefore, if sediment could be resuspended (i.e., if the maximum wave orbital velocities during an extreme wave event exceed the threshold velocities for resuspension of sediments), then the size of the bedforms would suggest the depth at which the bed may be mixed or resuspended.

Sediment texture (i.e., grain size) measurements in the 20- to 30-foot water depth region were available in RAs A, B, and C from the various phases of the PDI. The core locations where measurements were collected in RAs A, B, and C are shown on Figures 10-1, 10-2, and 10-3, respectively. The grain size analysis from Core OL-VC-60054 was used in the analysis for the RA E/SMU 5 boundary.

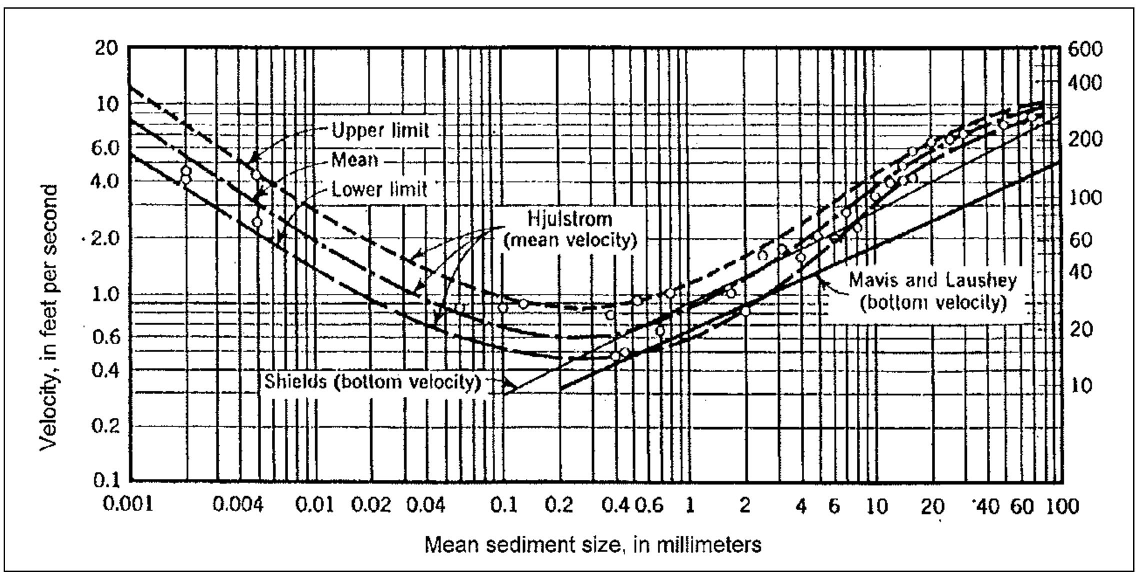
Table 10-1 presents the percentage of fine-grained sediments (defined herein as those materials passing the U.S. no. 200 sieve [0.075 millimeters]) in each segment measured.

**Table 10-1**
**Percentage of Fine Grained Sediments in the 6- to 9-Meter Zone**

| Remediation Area | Core | Depth Interval (ft) | Percent Silt and Clay Size |
|---|---|---|---|
| A | OL-VC-40016 | 9.9-13.2 | 99.2 |
| | | 13.2-16.4 | 99.4 |
| | | 16.5-19.8 | 99.5 |
| | OL-VC-40017 | 0.5-3.3 | 99.8 |
| | | 6.6-9.9 | 99.1 |
| | OL-VC-40018 | 0-3.3 | 99.0 |
| | | 6.6-9.9 | 99.9 |
| | | 16.5-18.6 | 99.2 |
| | OL-VC-40019 | 0.5-3.3 | 98.5 |
| | | 9.9-13.2 | 99.0 |
| | | 16.5-19.8 | 99.4 |
| | OL-VC-40021 | 0.5-3.3 | 98.3 |
| | | 3.3-6.6 | 98.8 |
| | | 13.2-16.5 | 83.5 |
| | OL-VC-40022 | 0.5-3.3 | 98.4 |
| | | 13.2-16.5 | 87.9 |
| | OL-VC-40023 | 3.3-6.6 | 99.6 |
| | | 13.2-16.5 | 90.8 |
| | S302 | 0.3-0.59 | 94.4 |
| | | 0.59-1.59 | 99.3 |
| | | 1.59-2.59 | 98.5 |
| | | 2.59-3.59 | 99.0 |

| Remediation Area | Core | Depth Interval (ft) | Percent Silt and Clay Size |
|---|---|---|---|
| **A** | S302 | 3.59-4.59 | 98.4 |
| | | 4.59-5.59 | 98.4 |
| | | 5.59-6.59 | 98.7 |
| | | 6.59-7.61 | 98.2 |
| **B** | OL-VC-30034 | 0.5-3.3 | 82.2 |
| | | 9.9-13.2 | 99.1 |
| | OL-VC-30035 | 6.6-9.9 | 99.1 |
| | | 16.5-19.6 | 97.9 |
| | OL-VC-30036 | 0.5-3.3 | 97.1 |
| | | 16.5-17.3 | 99.5 |
| | OL-VC-30037 | 0.5-3.3 | 92.6 |
| | | 9.9-13.2 | 96.9 |
| | | 13.2-16.5 | 98.9 |
| **C** | OL-VC-20067 | 0-3.3 | 97.8 |
| | | 6.6-9.9 | 87.9 |
| | OL-VC-20073 | 3.3-6.6 | 97.7 |
| | | 13.2-16.5 | 97.4 |
| | | 16.5-19.3 | 98.5 |
| | OL-VC-20074 | 0-3.3 | 98.4 |
| | | 9.9-13.2 | 98.7 |
| | | 13.2-16.5 | 99.0 |
| | OL-VC-20076 | 0-3.3 | 98.3 |
| | | 9.9-13.2 | 90.2 |
| | OL-VC-20077 | 0-3.3 | 96.4 |
| | | 13.2-16.5 | 99.1 |
| **RA E/SMU 5 Boundary** | OL-VC-60054 | 0.5-3.3 | 97.8 |
| | | 3.3-6.6 | 95.8 |
| | | 6.6-9.9 | 98.3 |
| | | 16.5-18.5 | 99.2 |
| | | **Minimum** | **82.2** |
| | | **Maximum** | **99.9** |
| | | **Average** | **97.0** |

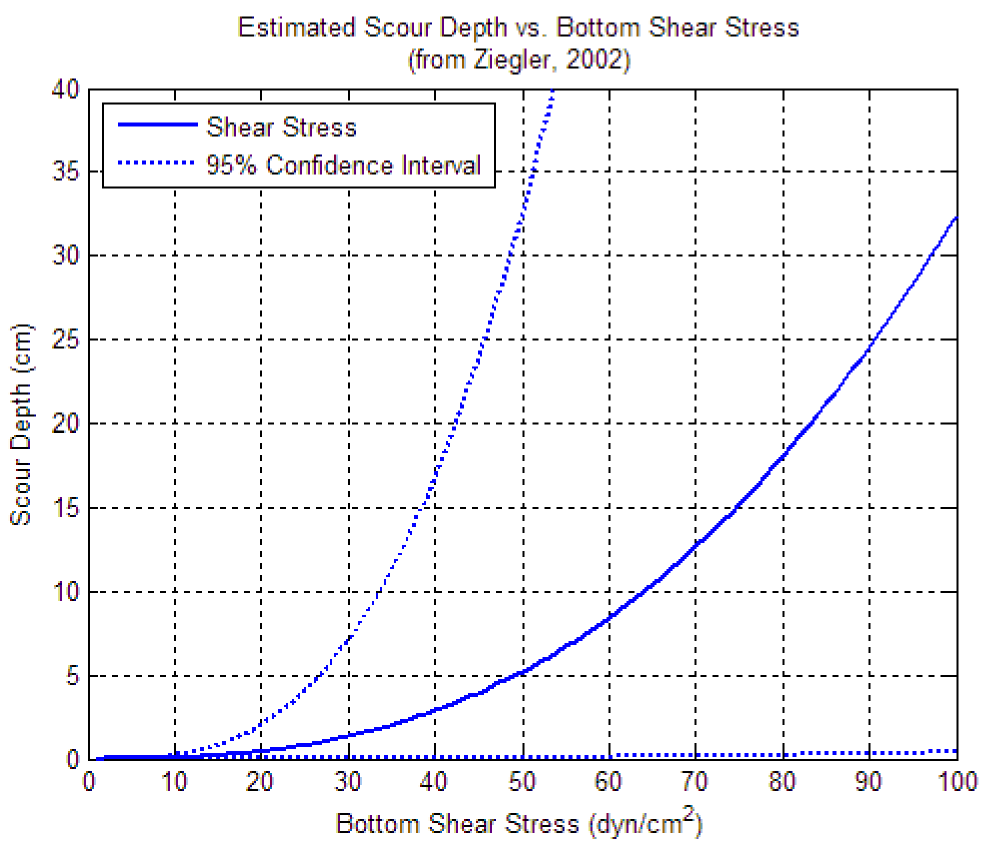
The grain size curves for each core are included in Attachment H. The grain size data indicate that the sediments in the 20- to 30-foot water depth region consist of thick deposits of primarily fine-grained sediments, which is consistent with depositional areas. The percentage of fine-grained sediments ranged from 82.2 to 99.9 percent, with an average of 97.0 percent). As shown on the Shields Diagram for Initiation of Motion (included as Figure A-8 of Attachment A and reproduced below), the velocity required to resuspend fine-grained sediments (with particle sizes of 0.075 millimeters or less) ranges between 0.6 fps (the lower limit) to 1.0 fps (the upper limit). It should be noted that the velocity required to resuspend the fine-grained sediments per the Shields Diagram is greater than that for fine sands due to the typical cohesive nature of these sediments, which provides resistance to erosion. As can be seen from the Shields Diagram, the smaller the particle size in the silts and clay region, the higher the velocity required to resuspend the sediments due to the increasing cohesion. For example, as can been seen from the grain size analysis, the median particle diameter ($D_{50}$) generally ranges from 0.0021 to 0.0257 millimeters. Based on the Shields Diagram, velocities greater than 1 to 3 fps would be necessary to resuspend particles of these sizes due to cohesion.




**Shields Diagram for Initiation of Motion (from Vanoni 1975)**

Tables 10-2, 10-3, and 10-4 present the maximum orbital velocity for the 2-year, 10-year, and 100-year wave for each remediation area in the 20- to 30-foot water depth region. The potential lengths of the vortex ripples for each wave event were also computed using Equation 3.4.1 from Nielsen (1992).

An estimate of potential scour depth in cohesive sediments due wave action was also performed using the methods presented by Ziegler (2002). This method involves determining the bed shear stresses induced by the wave or current forces and using an empirical relationship to estimate the depth of scour based on these forces. Ziegler (2002) presented a depth of scour estimated as a function of bottom shear stress based on erosion-rate measurements of cohesive sediments collected at eight aquatic systems in the United States. The figure below shows the estimated scour depth as a function of bottom shear stress for the average and 95 percent confidence intervals based on the data from these sites. Maximum bottom shear stresses were calculated in the 20- to 30-foot water depths for the 100-year extreme wave event for each remediation area. Table 10-4 presents the results of the analysis, showing the water depth, bottom shear stress, and resulting scour depth for each of the five remediation areas.




**Estimated Scour Depth as a Function of Bottom Shear Stress of Motion (from Ziegler 2002)**

Overall, the results of the analysis are consistent with Effler (1996). An evaluation of the wind-generated waves and sediment texture data suggest that Onondaga Lake regions with depths in excess of 6 to 8 meters (20 to 26 feet) represent the depositional basin of the Lake. Figure 10-4 shows the approximate locations of surficial sediment particle size measurements in Onondaga Lake described by Effler (1996). The locations of the PDI samples have also been included on the figure for comparison. Figure 10-5, adapted from Figure 8.12(b) of Effler (1996), presents the mean particles size by water depth in the Lake based on the surficial particle size measurements. As shown on the figure, the mean particle size in the 6- to 9-meter depth zone is between approximately 0.04 and 0.05 millimeters. This is consistent with the PDI data presented in Table 10-1, which shows that on average 97 percent of sediments in the 6 to 9-meter depth zone are fine-grained sediments (particle sizes of 0.075 mm or less). Using these data, Effler (1996) concluded that Onondaga Lake regions with depths in excess of 6 to 8 meters (20 to 26 feet) represent the depositional basin of the Lake:

> *"The effective depth of wave influence on sediment distributions may be marked by a well-defined change in the slope of the mean particle size – water depth relationship (Sly et al. 1982). In Onondaga Lake, this boundary occurs at a depth of approximately 6 m (Figure 8.12b). Based on this, it is concluded that lake regions with depths in excess of 6-8 m (65-71 percent of the lake area) represent the depositional basin of the lake."*

## Table 10-2
## Horizontal Orbital Velocities and Bedforms in 6- to 9-Meter Zone for the 2-year Wave Event

| Remediation Area | Water depth (feet) | Maximum Orbital Velocity (fps) | Bedform Length (feet) |
|---|---|---|---|
| A | 20 | 0.02 | 0.01 |
| | 30 | 0.00 | 0.00 |
| B | 20 | 0.04 | 0.02 |
| | 30 | 0.00 | 0.00 |
| C | 20 | 0.08 | 0.04 |
| | 30 | 0.01 | 0.01 |
| RA E/SMU 5 Boundary | 20 | 0.30 | 0.18 |
| | 30 | 0.07 | 0.05 |

Note:
The 2-year significant wave height and period for the RA E/SMU 5 boundary is 2.4 feet and 2.9 seconds, respectively. This is based on a fetch distance of 4.1 miles and a 2-year wind speed of 34.8 mph.

## Table 10-3
## Horizontal Orbital Velocities and Bedforms in 6- to 9-Meter Zone for the 10-year Wave Event

| Remediation Area | Water depth (feet) | Maximum Orbital Velocity (fps) | Bedform Length (feet) |
|---|---|---|---|
| A | 20 | 0.07 | 0.03 |
| | 30 | 0.01 | 0.00 |
| B | 20 | 0.13 | 0.07 |
| | 30 | 0.02 | 0.01 |
| C | 20 | 0.24 | 0.14 |
| | 30 | 0.05 | 0.03 |
| RA E/SMU 5 Boundary | 20 | 0.65 | 0.46 |
| | 30 | 0.22 | 0.15 |

Note:
The 10-year significant wave height and period for the RA E/SMU 5 boundary is 3.4 feet and 3.3 seconds, respectively. This is based on a fetch distance of 4.1 miles and a 10-year wind speed of 45.2 mph.

**Table 10-4**

**Horizontal Orbital Velocities and Bedforms in 6- to 9-Meter Zone for the 100-year Wave Event**

| Remediation Area | Water depth (feet) | Maximum Orbital Velocity (fps) | Bedform Length (feet) |
|---|---|---|---|
| A | 20 | 0.21 | 0.11 |
| | 30 | 0.04 | 0.02 |
| B | 20 | 0.32 | 0.18 |
| | 30 | 0.08 | 0.04 |
| C | 20 | 0.54 | 0.35 |
| | 30 | 0.17 | 0.11 |
| RA E/SMU 5 Boundary | 20 | 1.30 | 1.01 |
| | 30 | 0.56 | 0.43 |

Note:
The 100-year significant wave height and period for the RA E/SMU 5 boundary is 4.9 feet and 3.7 seconds, respectively. This is based on a fetch distance of 4.1 miles and a 100-year wind speed of 60 mph.

**Table 10-5**

**Bottom Shear Stresses in 6- to-9 Meter Zone for the 100-year Wave Event**

| Remediation Area | Water Depth (feet) | Maximum Bottom Shear Stress (dynes/square centimeter) | Range of Scour Depths (centimeters) |
|---|---|---|---|
| A | 20 | 0.41 | 0 |
| | 30 | 0.028 | 0 |
| B | 20 | 0.83 | 0 |
| | 30 | 0.082 | 0 |
| C | 20 | 1.9 | 0 – 0.0003 |
| | 30 | 0.30 | 0 |
| D | 20 | 1.9 | 0 – 0.0003 |
| | 30 | 0.30 | 0 |
| E | 20 | 9.6 | 0.0014 – 0.21 |
| | 30 | 2.8 | 0 – 0.0024 |

The results of the analysis indicate that the maximum wave orbital velocities during extreme wave events (the 2-year, 10-year, and 100-year) are less than the threshold velocities for resuspension of fine-grained sediments in RAs A, B, and C. Based on the Ziegler (2002) method, even in the case of the highest estimated shear stress (9.6 dynes per square centimeter), the resulting scour depth is estimated to be less than 0.25 centimeters. The results also indicate that waves do not have the potential to develop significant bedforms in these remediation areas. This would suggest that the 20- to 30-foot water depth region is net depositional.

At the RA E/SMU 5 boundary where the fetch wave energy is greater than in RAs A, B, and C, the results indicate that during the 2-year and 10-year wave events, the maximum wave orbital velocities are less than the threshold velocities for resuspension of fine-grained sediments. This would suggest fine-grained sediment would accumulate as suggested by Rowan et al. (1992) as the mud depositional boundary for lakes. The results also indicate that at the 20-foot water depth, the maximum wave orbital velocity during the 100-year extreme wave exceeds the threshold velocity for resuspension of fine grained sediments. At the 30-foot depth, the maximum wave orbital velocity is less than the threshold velocity for resuspension of fine grained sediments. Based on a bedform analysis (which is a conservative estimate of resuspension potential in cohesive sediments based on a comparison with Ziegler [2002]), the results indicate that resuspension or movement of sediments during an extreme event would be limited to the top foot in this location during the 100-year event. Sediments buried below these surficial sediments are expected to be stable.

# 11 CAP FOR THE STEEP UNDERWATER SLOPE OF NYSDOT TURNAROUND

This section provides a summary of the analysis of the stable particle size that is proposed for the cap on the steep underwater slope along the New York State Department of Transportation (NYSDOT) turnaround area. The cap will extend from elevation 362.5 feet down to the base of the steep slope (approximate elevation of 345.0 feet). The existing underwater slope along this area is as steep as 2H:1V. Using the 100-year return interval wind-generated wave height for the RA C shoreline, the armor stone size and gradation for the surf zone for the 100-year wave was computed using the methods summarized in Section 5. The gradation is summarized in Table 11-1.

**Table 11-1**
**Armor Stone Size ($D_{50}$)**
**with a Slope of 2H:1V (for Surf Zone Regime)**

| Gradation | Stone Size (inches) |
|---|---|
| $D_0$ | 5 |
| $D_{15}$ | 7 |
| $D_{50}$ | 10 |
| $D_{85}$ | 12 |
| $D_{100}$ | 15 |

Note:
Computed using minor displacement (S=3). Minor displacement refers to minimal movement of armor stones and could be related to "rocking" of the armor under extreme wave action. Repairs associated with such events (if any) will be handled as part of a maintenance program.

Since this underwater cover system will extend above elevation 360.5 feet, portions of the cover system will be exposed to ice (see Section 8). The recommended stone size is slightly smaller than the 16-inch minimum recommended in Attachment G (Ashton 2004) to ensure no impacts from ice. Therefore, the area would be inspected annually and repaired if necessary.

Due to the size of the gradation, a 1-foot-thick filter layer consisting primarily of coarse gravels (1 to 2 inches) will be placed between the armor stone and the existing slope.

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# FIGURES

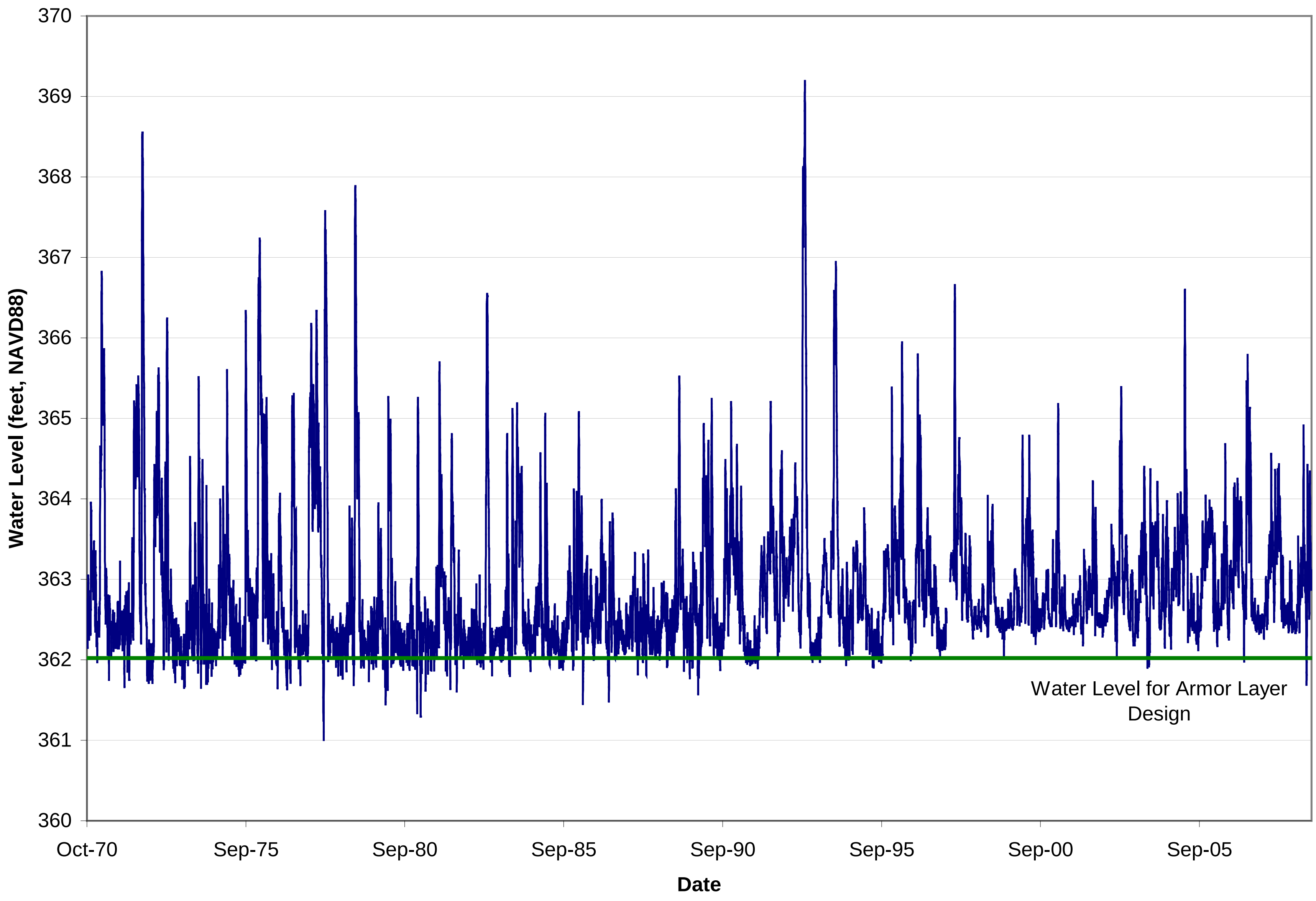


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**Figure 4-1**
Time Series of Onondaga Lake Water Levels 1970-2009
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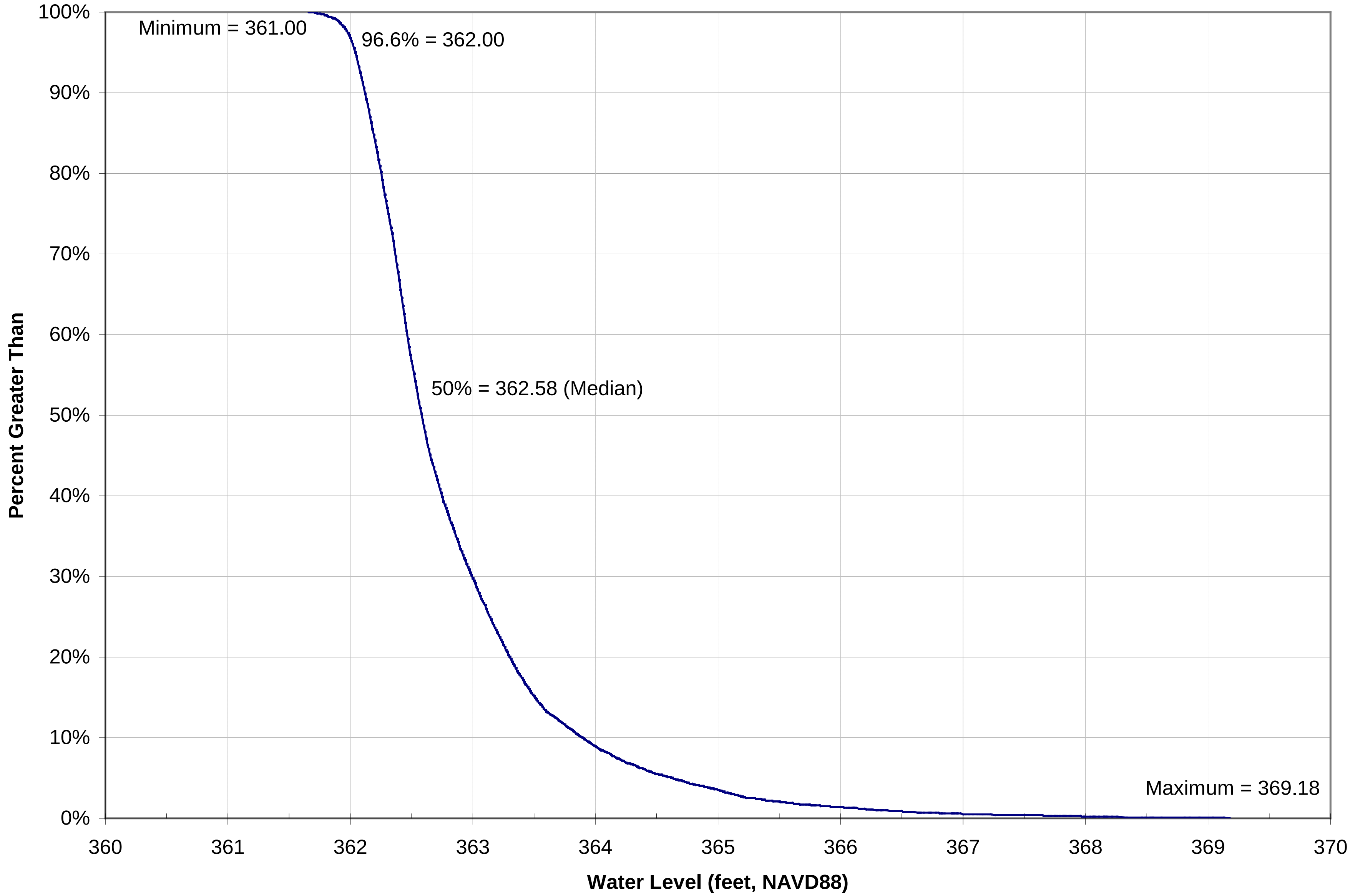


**Figure 4-2**
Cumulative Frequency Distribution of Onondaga Lake Water Levels 1970-2009
Armor Layer Design Appendix
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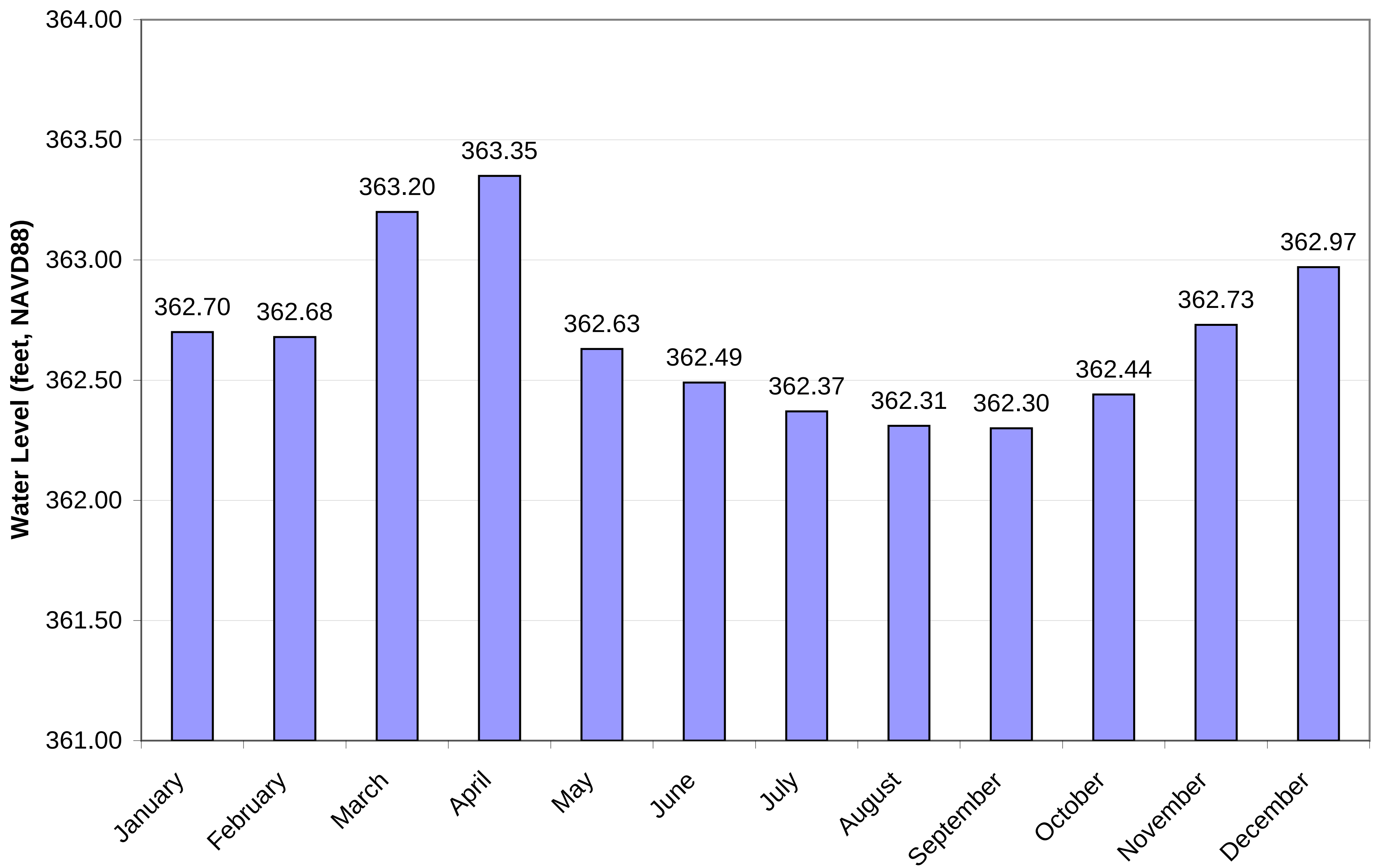


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**Figure 4-3**
Monthly Median Onondaga Lake Water Levels 1970-2009
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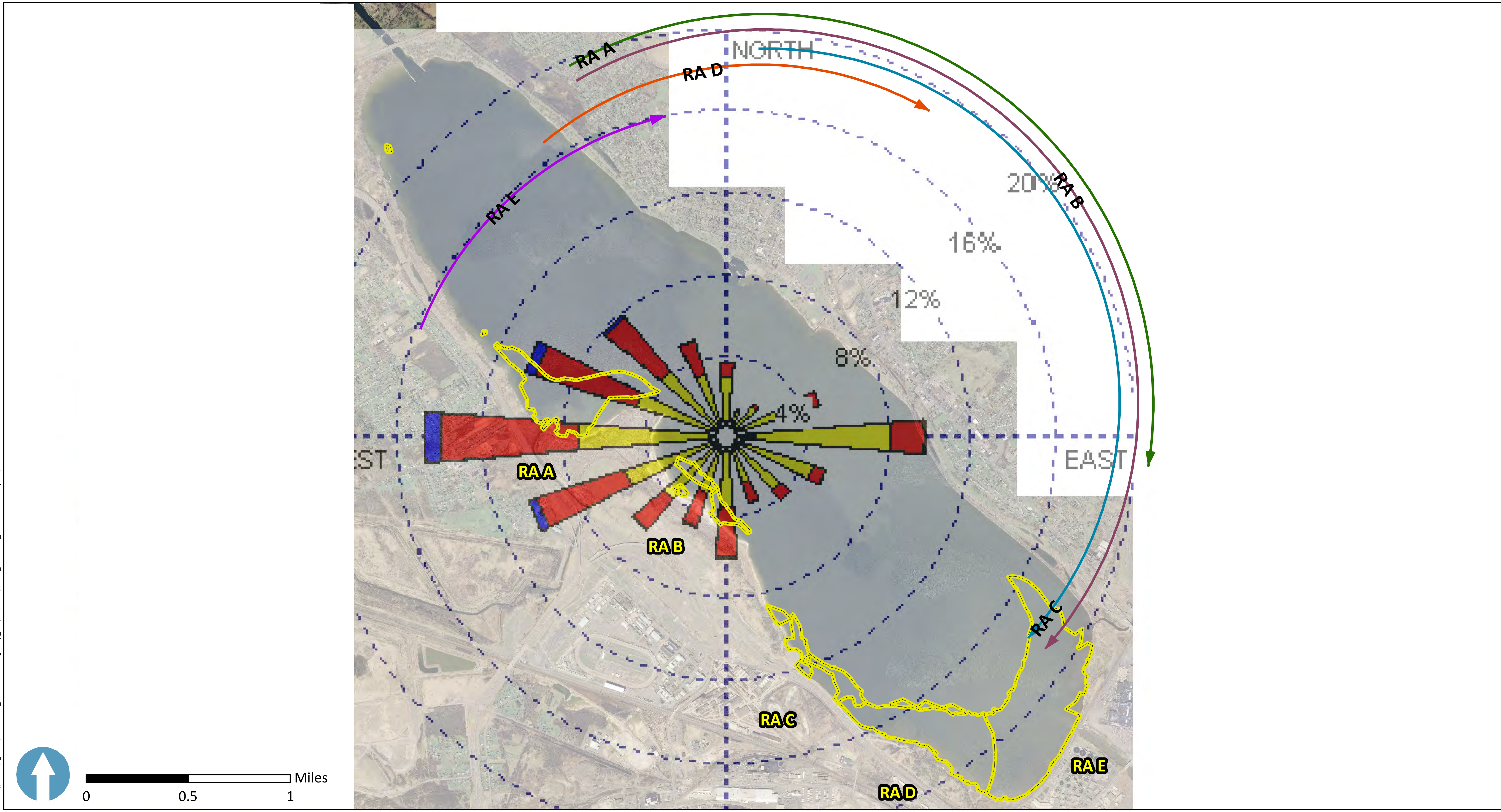



Legend

Remediation Areas

Wind speed
(meters / second)

>= 25
20 - 25
15 - 20
10 - 15
5 - 10
2 - 5
1 - 2
Calms: 0.27%


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**Figure 5-1**
Wind Rose for Onondaga Lake
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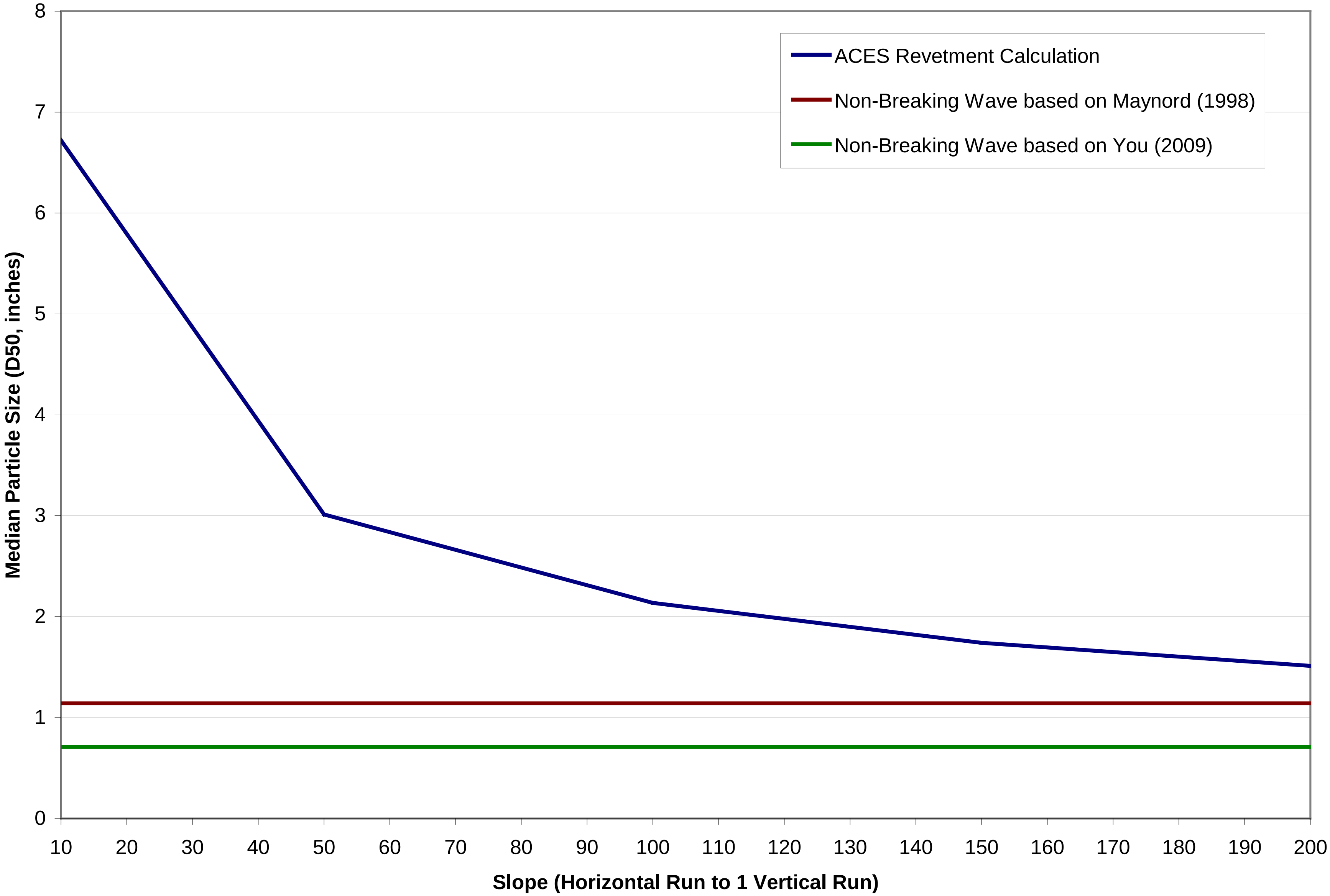


Note: Assumes "Minor Displacement" of Armor Layer (S=3)

**Figure 5-2**

Sensitivity of Median Armor Stone Size (D50) to Slope in Remediation Area E

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**Figure 6-1**
Ninemile Creek Model Grid
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**Figure 6-2**
Onondaga Creek Model Grid
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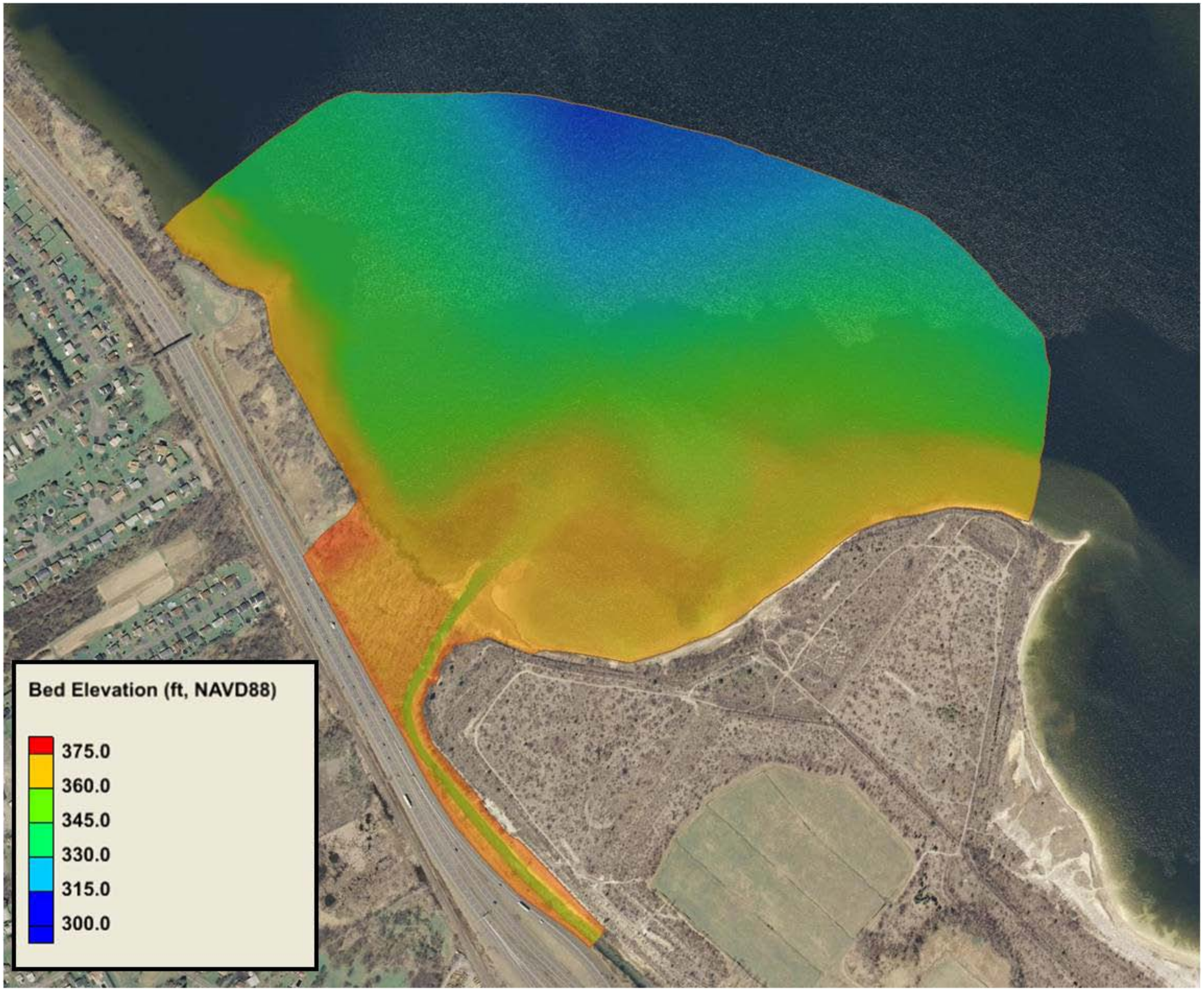


**Figure 6-3**
Model Grid Bathymetry – Ninemile Creek
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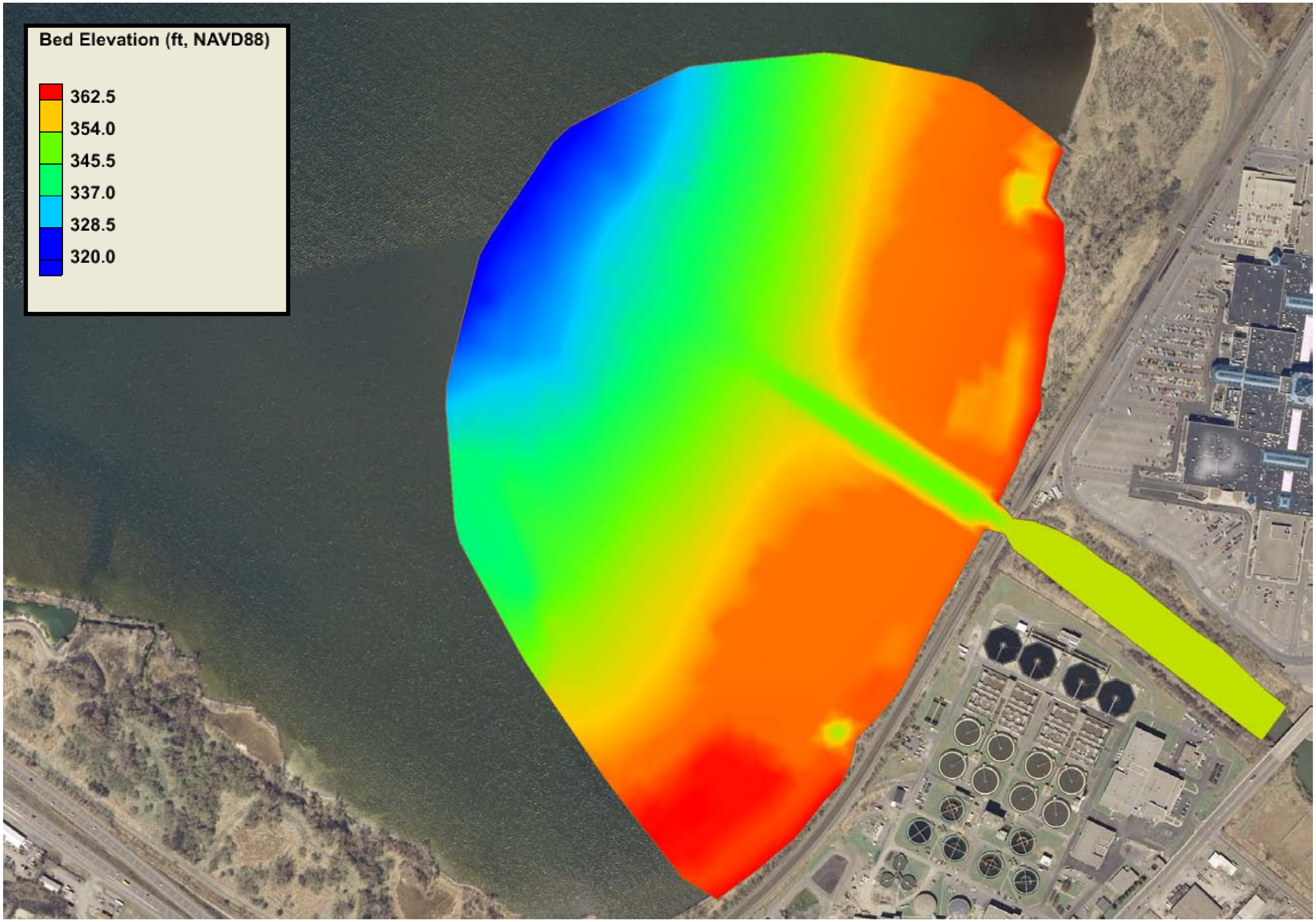


Note: Bathymetry presented as restored bathymetry after 2 years of settlement in water depths >3 feet.

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**Figure 6-4**
Model Grid Bathymetry – Onondaga Creek
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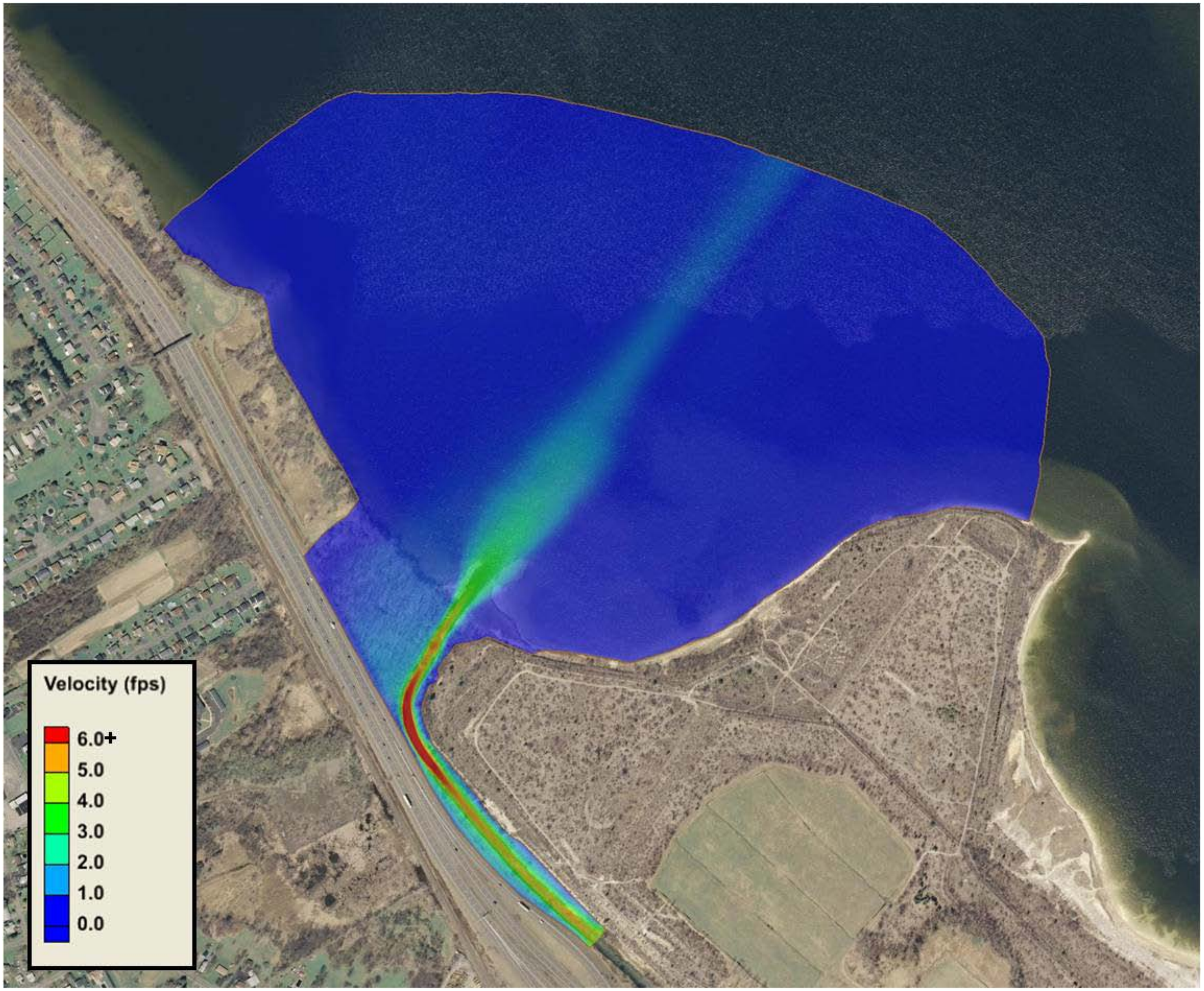


**Figure 6-5**
Computed Velocity Magnitude in Remediation Area A
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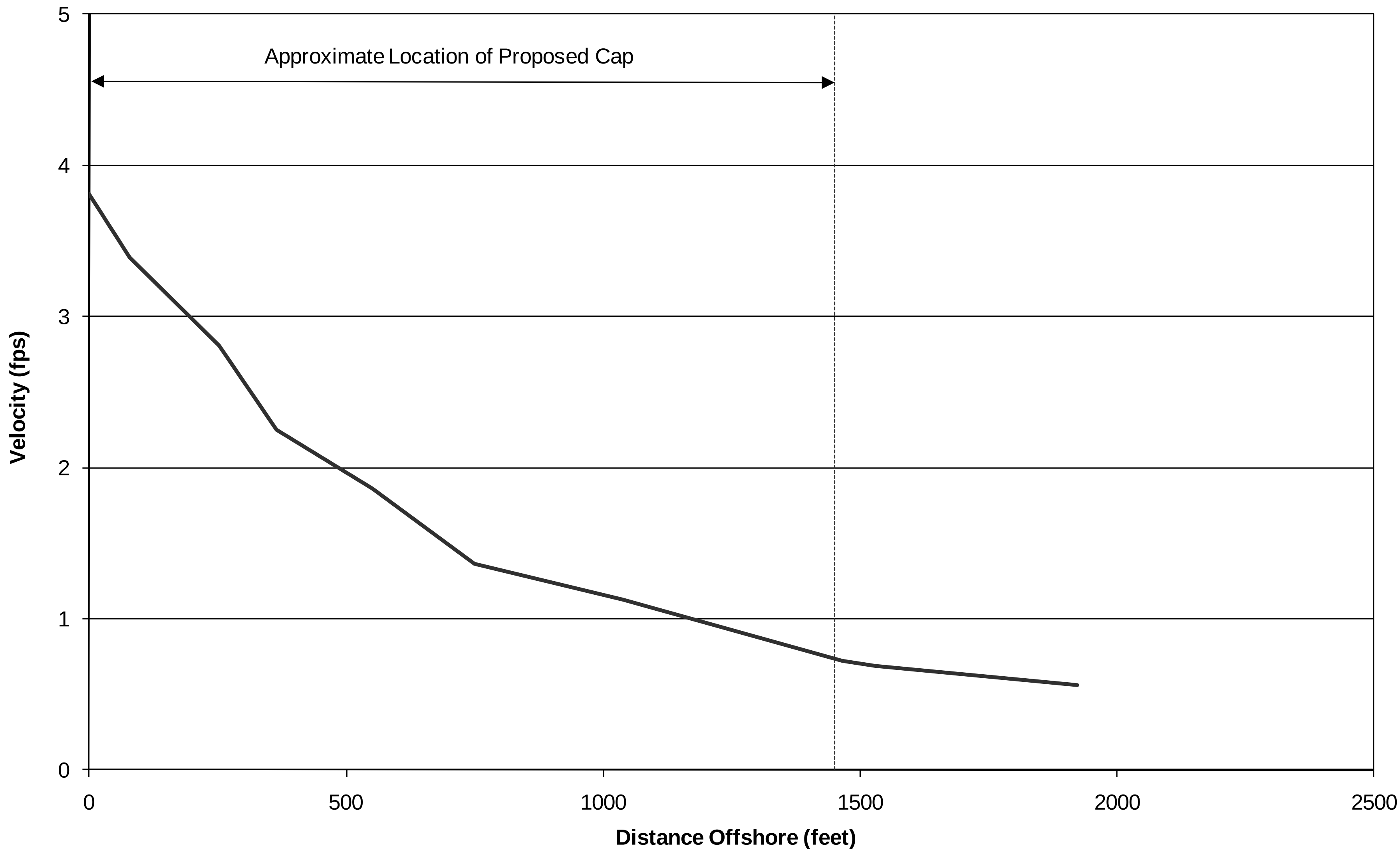


**Figure 6-6**
Computed Velocity along Discharge Centerline from Ninemile Creek
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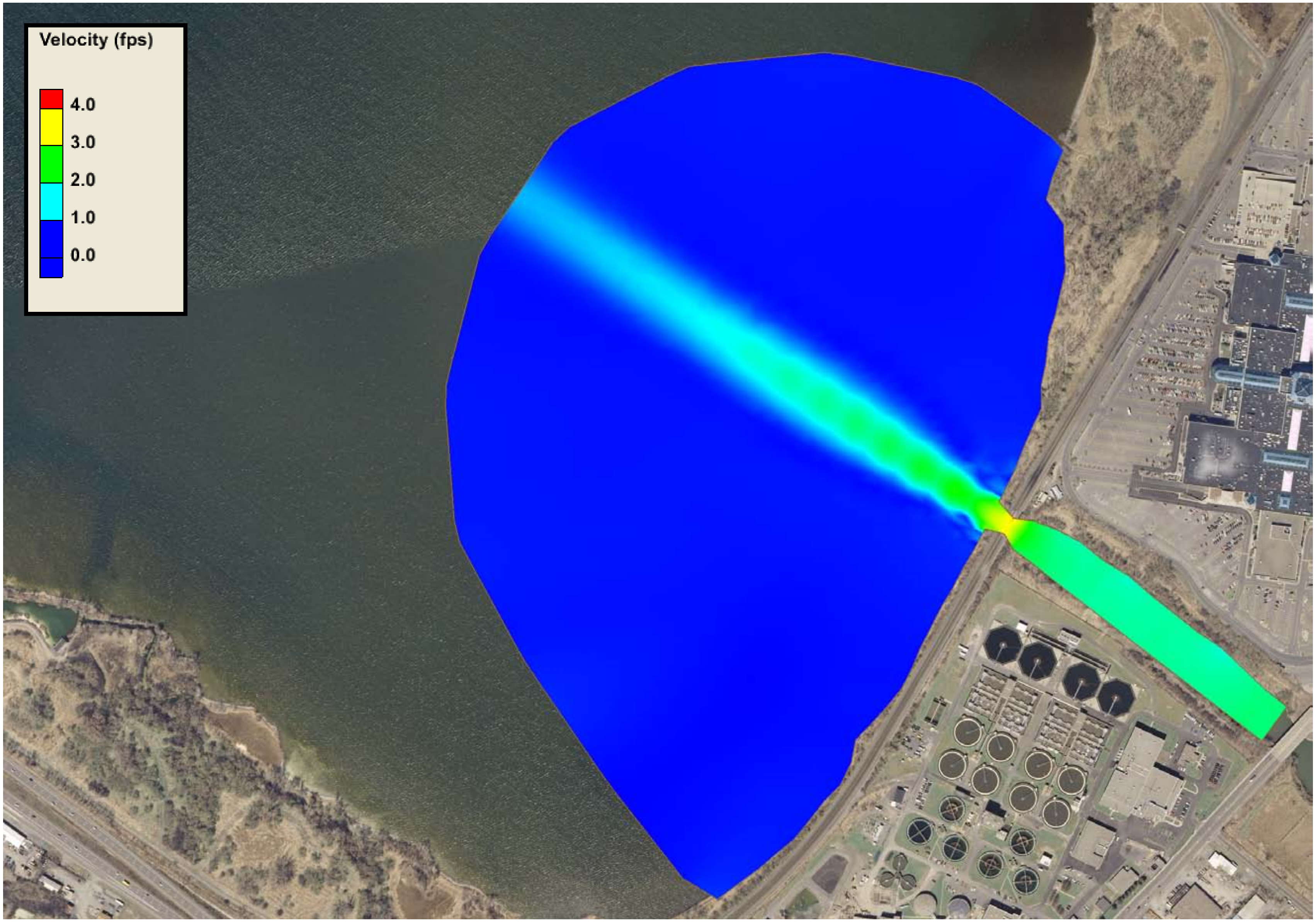


**Figure 6-7**
Computed Velocity Magnitude in Remediation Area E
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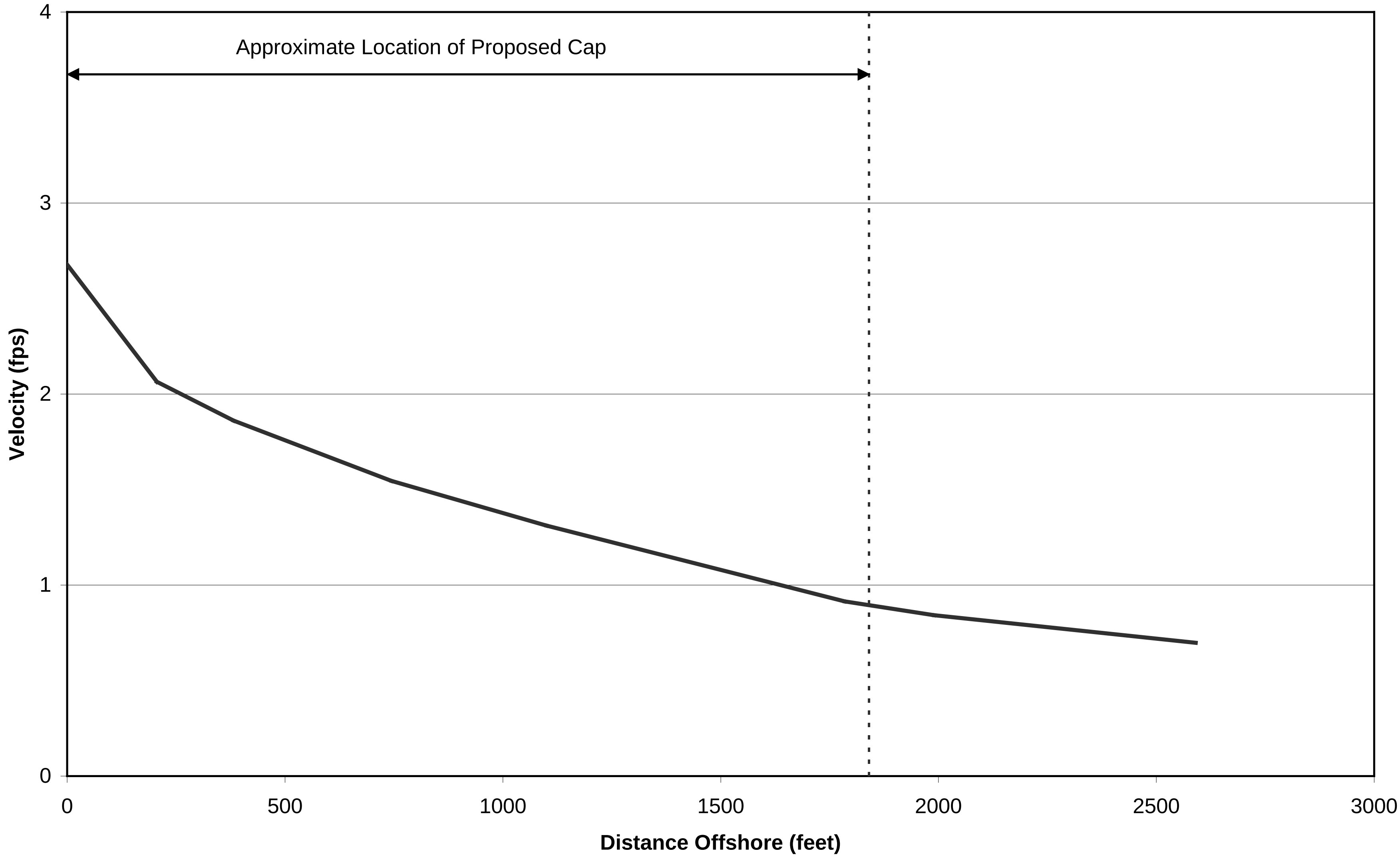


**Figure 6-8**
Computed Velocity along Discharge Centerline from Onondaga Creek
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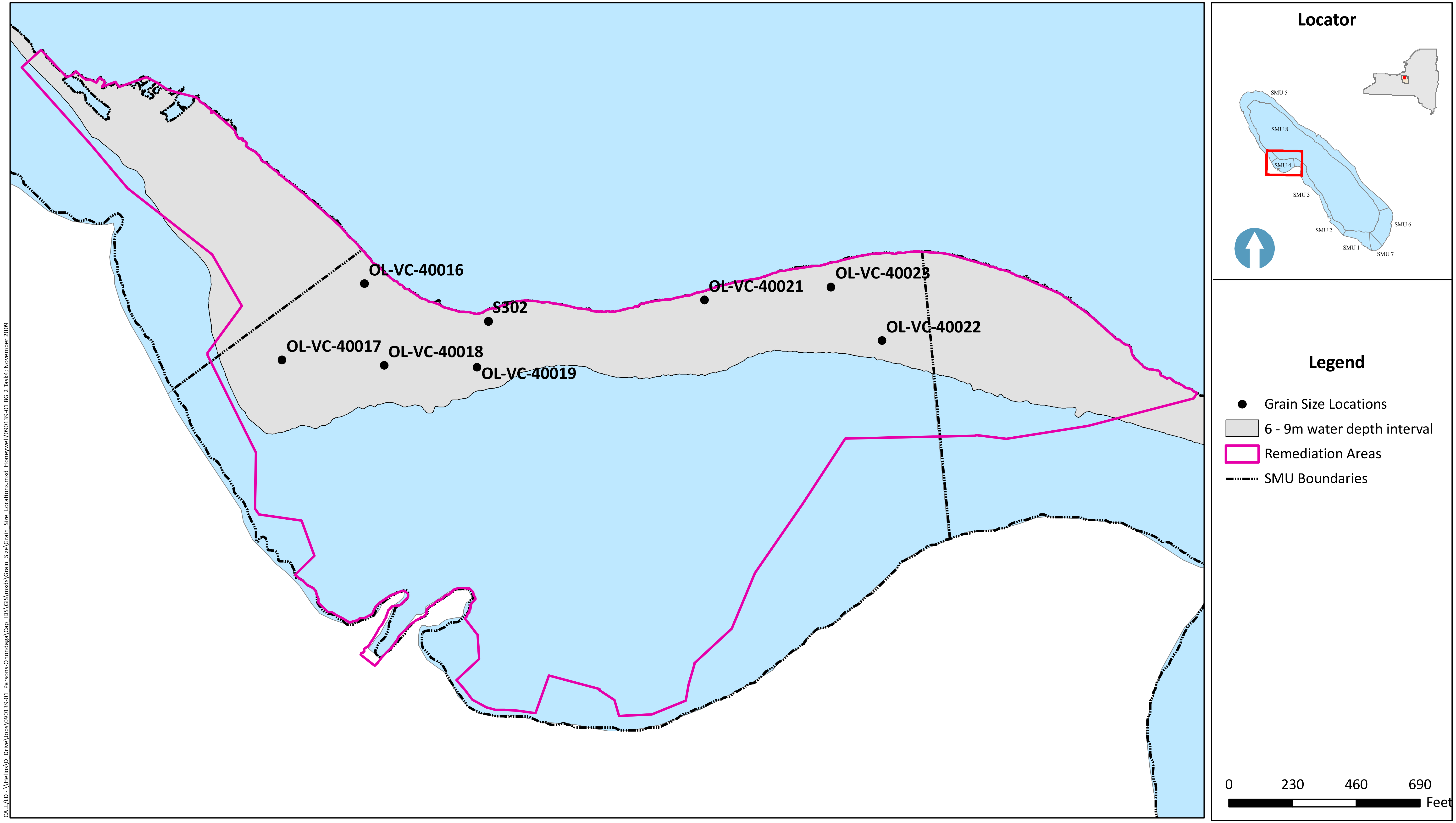


**Figure 10-1**
Grain Size Locations
Remediation Area A
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**Figure 10-2**
Grain Size Locations
Remediation Area B
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**Figure 10-3**
Grain Size Locations
Remediation Area C
Armor Layer Design Appendix
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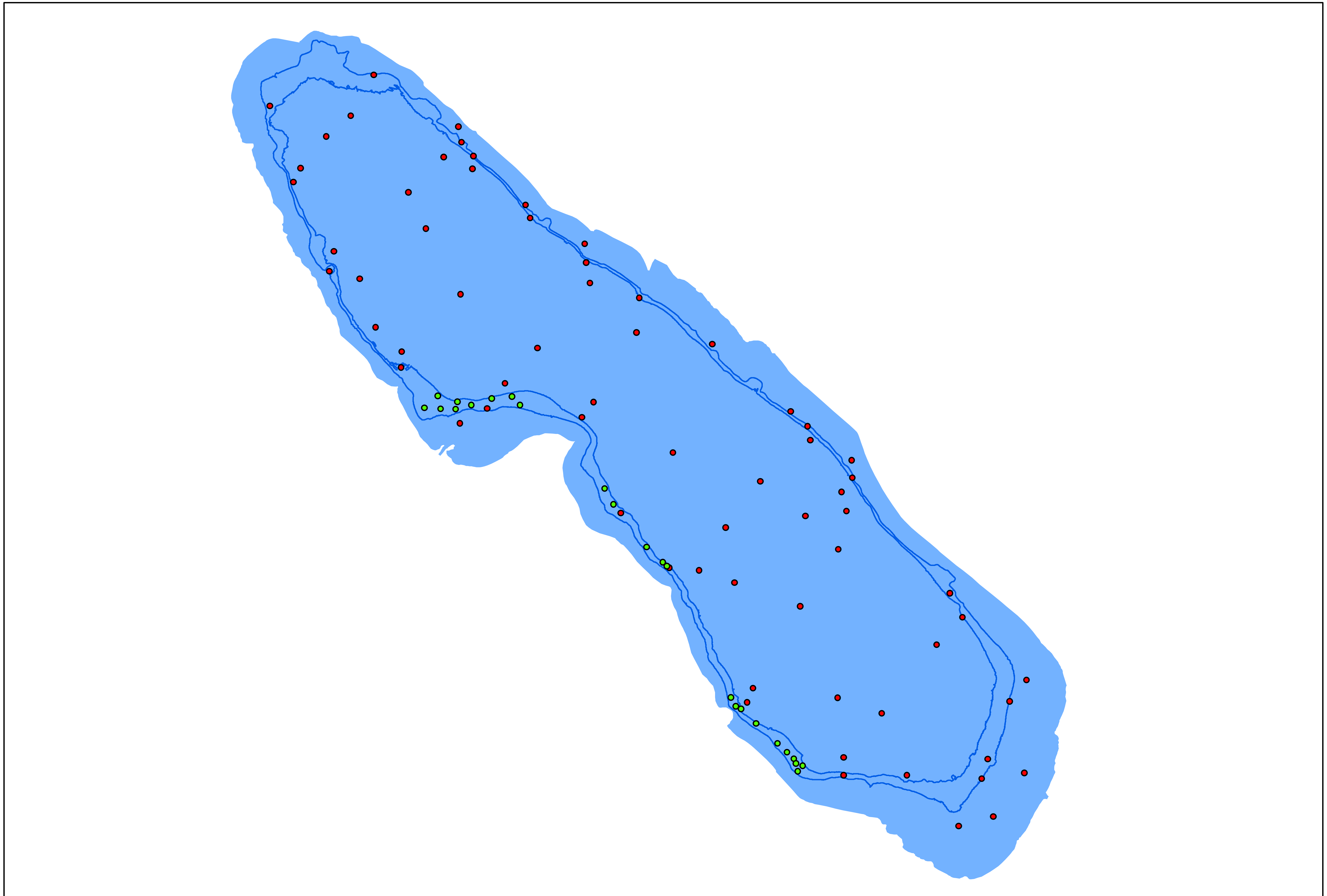

Locator


**Legend**

- Effler Sampling Locations
- PDI Grain Size Locations
- 6 - 9m Water Depth Zone

0 1,000 2,000 3,000 4,000 Feet

090139-01.17 Jul 2010

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**Figure 10-4**
Sampling Locations Described in Effler
Adapted from Effler Figure 8.11(a)
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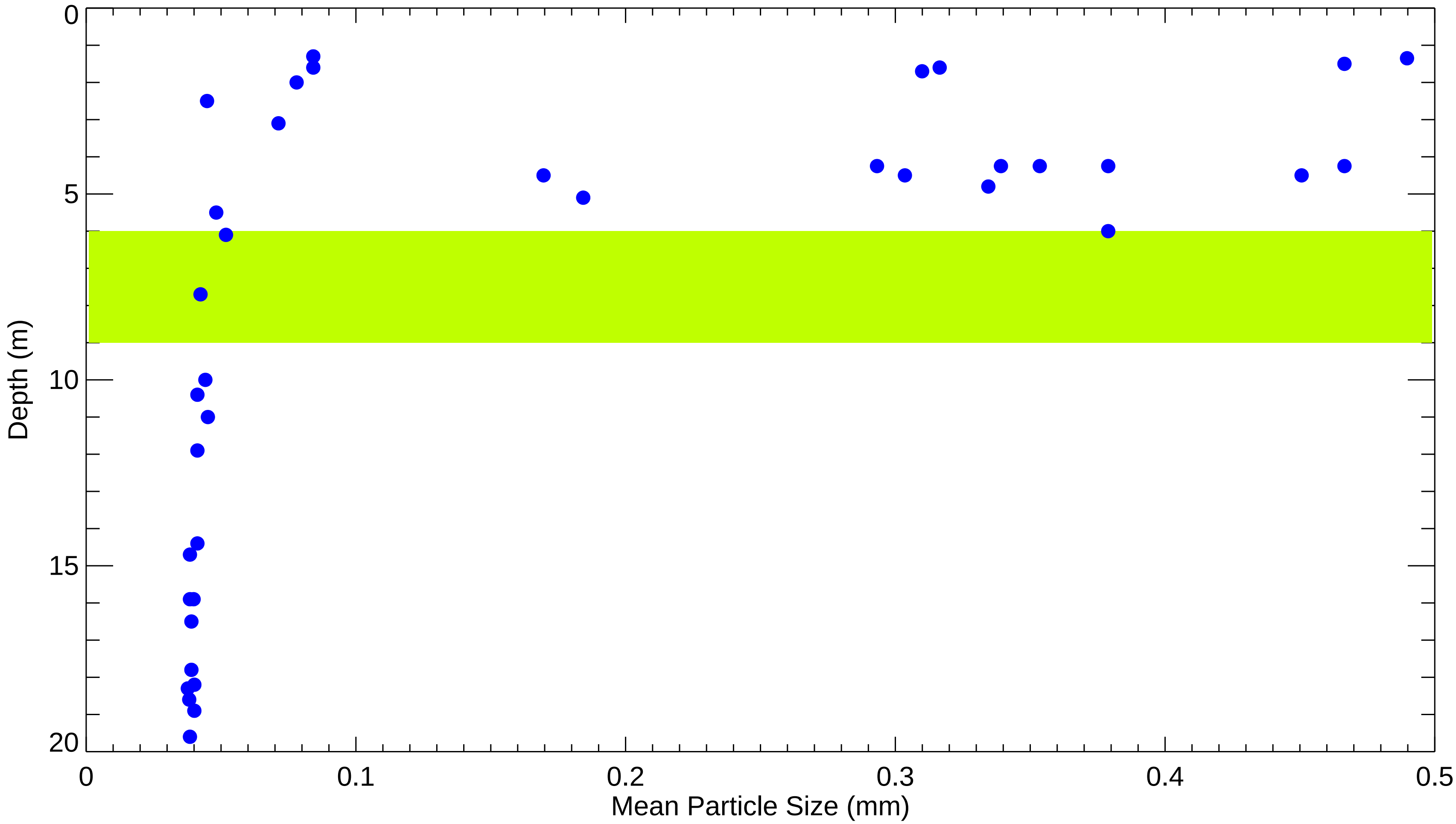


**Figure 10-5**

Effler Mean Particle Size

Adapted from Effler Figure 8.12(b) and highlights the 6-9 m depth zone

Armor Layer Design Appendix

Draft Capping, Dredging and Habitat Intermediate Design