Hydrologic and Hydraulic Analysis Report

Blind Brook Watershed Study

City of Rye, New York

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Executive Summary

The City of Rye (City), located 7 miles north of New York City, lies in the eastern part of central Westchester County. The City has experienced significant property damage due to flooding along Blind Brook which flows through the City. Several studies have been conducted to analyze the flooding along Blind Brook, and to provide recommendations to reduce/ mitigate the flooding caused by the Brook. The purpose of this report is threefold:

- to provide an assessment of the recommendations made to the City in previous studies;
- evaluate additional flood reduction measures not previously proposed;
- evaluate the operation of the sluice gate recently installed at Bowman Avenue Dam and suggest operational improvements.

First, Parsons Brinckerhoff (PB) analyzed the six reports previously submitted to the City and summarized the purposed objective of each report, the findings and recommendations, the hydrologic and the hydraulic modeling method utilized in each report. Then, by using GIS mapping and the information obtained from a field visit to the Blind Brook watershed, PB reconstructed the subdivided hydrologic models for the Upper Blind Brook watershed. PB then selected 10 potential sites for new detention areas and then studied the impact these 10 areas would have on downstream flooding. The detention areas to study were broken down into 5 regions, SW1-Airport (2 detention areas), SW1-SUNY (2 detention areas), SW1-PepsiCo (1 detention area), SW2-Hutchinson River Parkway (3 detention areas) and SW2 (2 detention areas). The identified detention areas along the Blind Brook were evaluated first individually. For this analysis, it was found that the SW1-SUNY detention basins and SW2 detention basins potentially provide significant water surface elevation reductions at five downstream locations. Then the two most effective detention regions were evaluated cumulatively to provide a sense of incremental mitigation benefits if implemented over time. PB also studied the effect of resizing the upper pond at the Bowman Avenue Dam. Two alternatives were analyzed at this location that examined the effects of increasing the storage volume of the upper pond on the downstream water surface elevation for various storm events. Hydraulic analysis results showed that between the two resized pond alternatives, Cases C and D, the incremental benefit gained with the maximized resized alternative (Case D) is insignificant. PB also performed the construction cost estimate for resizing the Upper Pond and two detention ponds on SUNY-Purchase. The cost and water surface elevation reductions table is provided for those two proposed improvements.

After consulting with engineers from the City, PB studied two scenarios of a new optimal operational rule of the sluice gate. The location of the gauge measuring water surface elevations that controlled the opening or closing the sluice gate would be moved to a point downstream of the dam, and upstream of the flood prone area of Indian Village (upstream of I-95) and downstream of I-287. The optimal elevation to close sluice gate for each storm event was obtained by analyzing the existing conditions maximum water surface elevation for the corresponding storms at the gauge location. The final optimal water surface elevations that trigger closing the sluice gate was found by using a trail-and-error process from a wide range of values. The final result showed that the water surface elevation reductions have been increased for 10, 25, 50 and 100-year storm when compared with previous study conducted in 2012. Operation of gauge based on water surface elevations at a location downstream of I-287 can provide more water surface elevation reductions at all downstream locations for 5, 10 and 25-year storms than the gauge location at Indian Village.

Terms and Glossary

1. Bankfull discharge: The discharge of a river which is just contained within its banks.
2. Dikes: A natural or artificial slope or wall to regulate water levels, also called a levee.
3. Drainage area: A geographic and hydrologic subunit of a watershed.
4. Energy equation: An expression of the work-energy theorem: the work done by the fluid pressure is equal to the change in kinetic energy of the flow.
5. Flood retarding: Same meaning as “flood detention”.
6. Headwaters: The small streams and upland areas that are the source of larger streams and rivers.
7. HEC-RAS: A one-dimensional computer program developed by the US Army Corps of Engineers that models the hydraulics of water flow through natural rivers and other channels.
8. Hurricanes: A tropical cyclone is a rapidly-rotating storm system characterized by a low-pressure center, strong winds, and a spiral arrangement of thunderstorms that produce heavy rain.
9. Hydraulic: A topic in applied science and engineering dealing with the mechanical properties of liquids and open channel flow.
10. Hydrograph: A continuous plot of the surface runoff flow versus time.
11. Hydrologic: The science dealing with the disposition of water on the earth.
12. Infiltration rate: A measure of the rate at which soil is able to absorb rainfall or irrigation flows. It is measured in inches per hour or millimeters per hour.
13. Main Stem: In hydrology, a main stem is "the primary downstream segment of a river, as contrasted to its tributaries".
14. Non-contact activities: An activity where a participant should have no possible means of coming into contact with a body of water.
15. Nor’easters: A macro-scale storm along the upper East Coast of the United States and Atlantic Canada. It gets its name from the direction the wind is coming.
16. Out-of-bank: Refers to the floodplain areas outside of a river banks.
17. Ponds: A body of standing water, either natural or man-made, that is usually smaller

than a lake.

18. Primary and secondary contact recreation: People can swim in the water body without risk of adverse human health effects (such as catching waterborne diseases from raw sewage contamination). People can perform activities on the water (such as boating) without risk of adverse human health effects from ingestion or contact with the water.

19. Reservoirs: A natural or artificial lake, storage pond or impoundment from a dam which is used to store water.

20. River mouth: A part of a river where it flows into the sea, river, lake, reservoir or ocean.

21. Runoff Curve Numbers: An empirical parameter, based on soil type and land use, used in hydrology for predicting direct runoff or infiltration from rainfall excess.

22. Sluice gate: A wood or metal barrier sliding in grooves that are set in the sides of a structure such as a dam. Sluice gates commonly control water levels and flow rates in rivers and canals.

23. Storm event: A disturbed state of an astronomical body's atmosphere especially affecting its surface, and strongly implying severe weather.

24. Subwatershed: Watersheds may contain smaller geographic subdivisions that drain into the river or other water body.

25. Time of Concentration: A concept used in hydrology to measure the response of a watershed to a rain event. It is defined as the time needed for a drop of water to flow from the most remote point in a watershed to a designated point.

26. TR-20: A computer program developed by Natural Resources Conservation Service (NRCS) for the generation and routing of runoff hydrographs.

27. Transmission: Same meaning as infiltration.

28. Tributary: A stream or river that flows into a main stem (or parent) river or a lake.

29. Watershed: An area of land where surface water from a rain, melting snow or ice converges to a single geographic point at a lower elevation, usually the exit point of the basin, where the waters join another water body, such as a river, lake, reservoir, estuary, wetland, sea, or ocean.

30. WinTR-20: A Windows based computer program that computes direct runoff and develops hydrographs resulting from a synthetic or natural rainstorm.

1. Introduction

1.1 Objectives

The Blind Brook Watershed, a tributary to the Long Island Sound, contains portions of the City of Rye, the Village of Rye Brook, the Town/Village of Harrison, the Village of Port Chester and the City of White Plains (Figure 1), which are all vulnerable to flooding during heavy rainfall events. In particular, the portion of Blind Brook running through the City of Rye experiences significant flooding and property damage due to the more extreme rainfall events associated with nor'easters and hurricanes. The frequency of flooding has resulted in repetitive property loss, and as a result of these losses approximately 500 residents rely on Federal Emergency Management Agency's (FEMA) National Flood Protection Agency (NFPA) flood insurance as their safety net in the event of a flood.

The City of Rye, is located in approximately 7 miles north of New York City, and has an area of approximately 6 square miles and lies in the eastern part of central Westchester County (Figure 2). The City has conducted several studies to analyze the flooding along Blind Brook and to provide recommendations to reduce the flooding along Blind Brook. Recommendations with supporting analysis have been provided to the City of Rye and are the subject of review under this project. The purpose of this report is to provide a professional assessment of the recommendations made to the City of Rye in those previous studies, and Parsons Brinckerhoff (PB) will layout a series of next steps toward implementing flood reduction measures along Blind Brook.

Figure 1 - Location Map (Credit: U.S. Geological Survey, National Map Viewer and Download Platform)

Figure 2 - Vicinity Map (City Limits of Rye, Credit: Google Map)

1.2. Watershed Description

The Blind Brook watershed, originating from the Westchester County Airport, has a drainage area of 10.9 square miles (6,976 acres) at its river mouth to Milton Harbor. The watershed is located in Westchester County, New York (96.7% of total area), with a portion in Fairfield County, Connecticut (3.3% of total area, Figure 3). The length of the Blind Brook watershed is approximately 9 miles and its width varies from 0.5 miles to 2 miles. In the United States Geological Survey (USGS) Hydrologic Unit Code (HUC) system, the Blind Brook Watershed has a 6^{th} level, or subwatershed level code of HUC-12 011000060405.

The main municipalities within the Blind Brook watershed include the City of Rye, City of White Plains, Village of Rye Brook, Village of Port Chester, Town of Harrison and Village of Harrison. The headwaters of the Blind Brook originate south of Rye Lake, NY. It continues south flowing past the State University of New York (SUNY) Purchase Campus, PepsiCo Company, and then crosses the Hutchinson River Parkway, and joins with the west tributary of Blind Brook at Harrison, New York. The Brook then flows through Rye, NY, and empties into the Long Island Sound. The main stem and tributaries within the Blind Brook watershed are shown in the figure below (Figure 3).

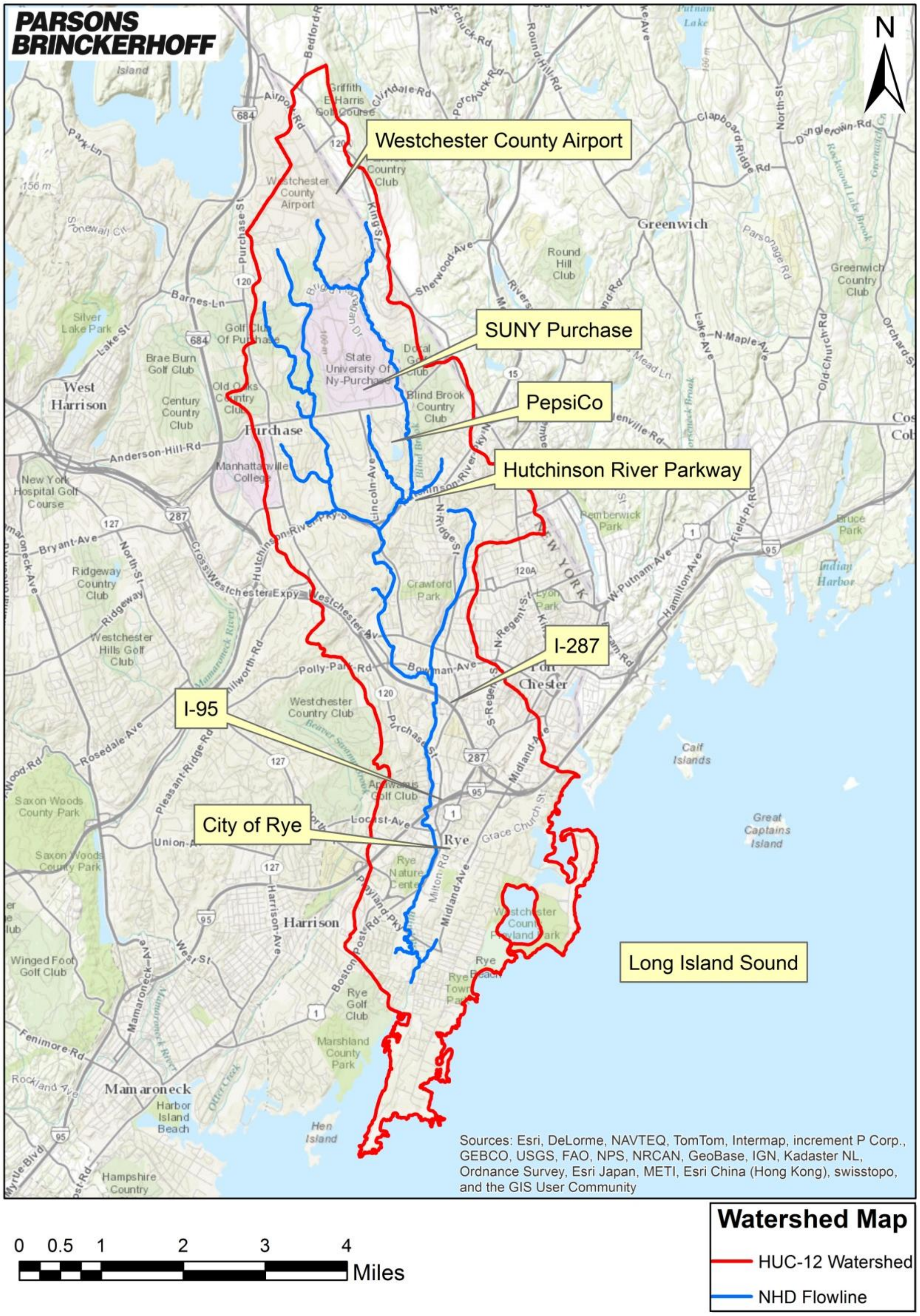


Figure 3 - HUC-12 011000060405 Blind Brook Watershed Map (USGS Data)

1.3 Land Use and Soil Data

Existing land use within the study area is based on the National Land Cover Database 2006 (NLCD_2006) land use data from USGS's national viewer website. NLCD_2006 is a 16-class land cover classification scheme that has been applied consistently across all 50 states and Puerto Rico at a spatial resolution of 90 feet. NLCD_2006 is based primarily on the unsupervised classification of Landsat Enhanced Thematic Mapper+ (ETM+) circa 2006 satellite data. NLCD_2006 improves on NLCD_92 in that it is comprised of three different elements: land cover, percent developed impervious surface and percent tree canopy density. NLCD_2006 also uses improved classification algorithms, which have resulted in data with more precise rending of spatial boundaries between the land cover classes.

The existing land uses in the Blind Brook Watershed include Open Water, Developed: Open Space, Developed: Low Density Residential, Developed: Medium Density Residential, Developed: High Density Residential, Barrel Land, Deciduous Forest, Evergreen Forest, Mixed Forest, Shrub/Scrub, Grassland/Herbaceous, Pasture/Hay, Woody Wetlands, Emergent Herbaceous Wetlands. The breakdown of existing land use categories for this watershed is shown in Figure 4. By running statistical analysis, the watershed contains 34.1% urban area. The existing development condition land use map (Figure 5) was subsequently developed.

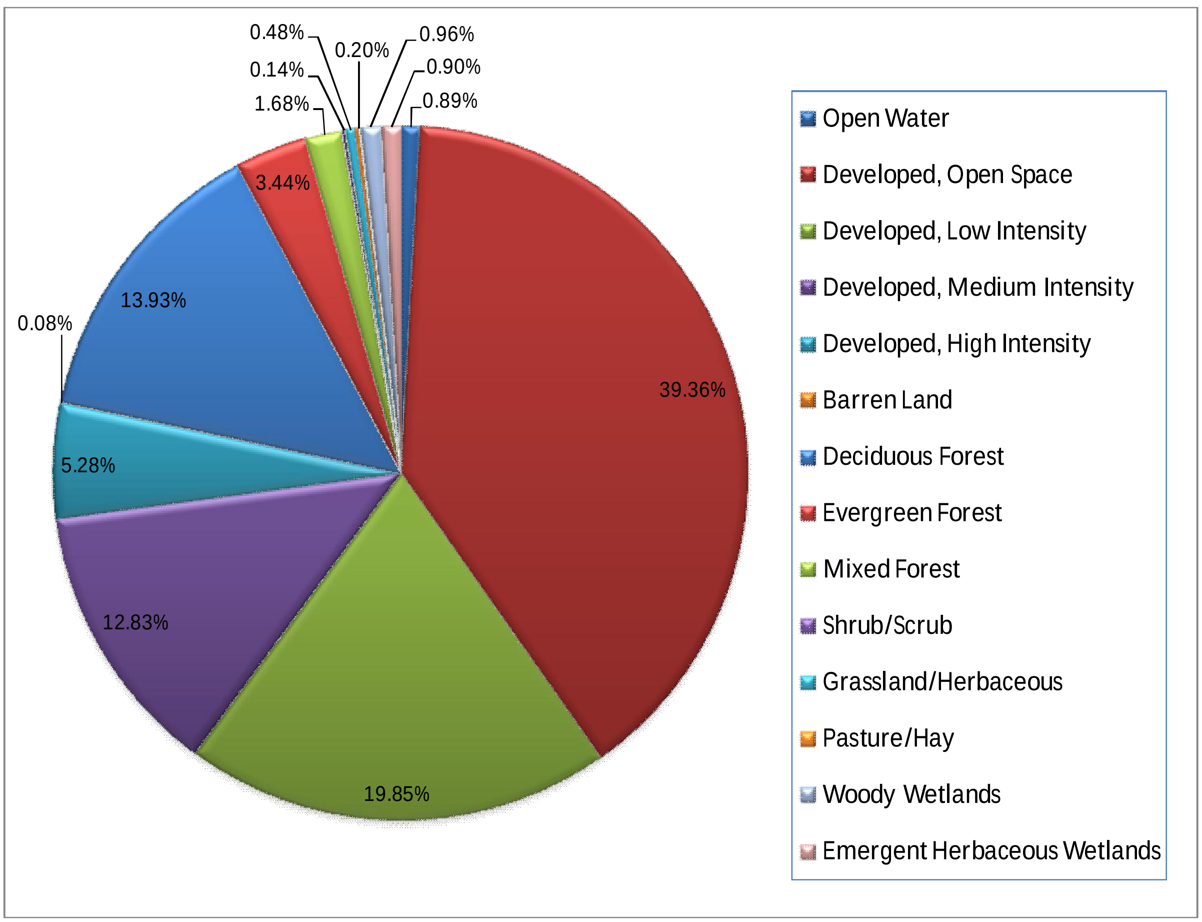


Figure 4 - Breakdown of Existing Condition Land Use

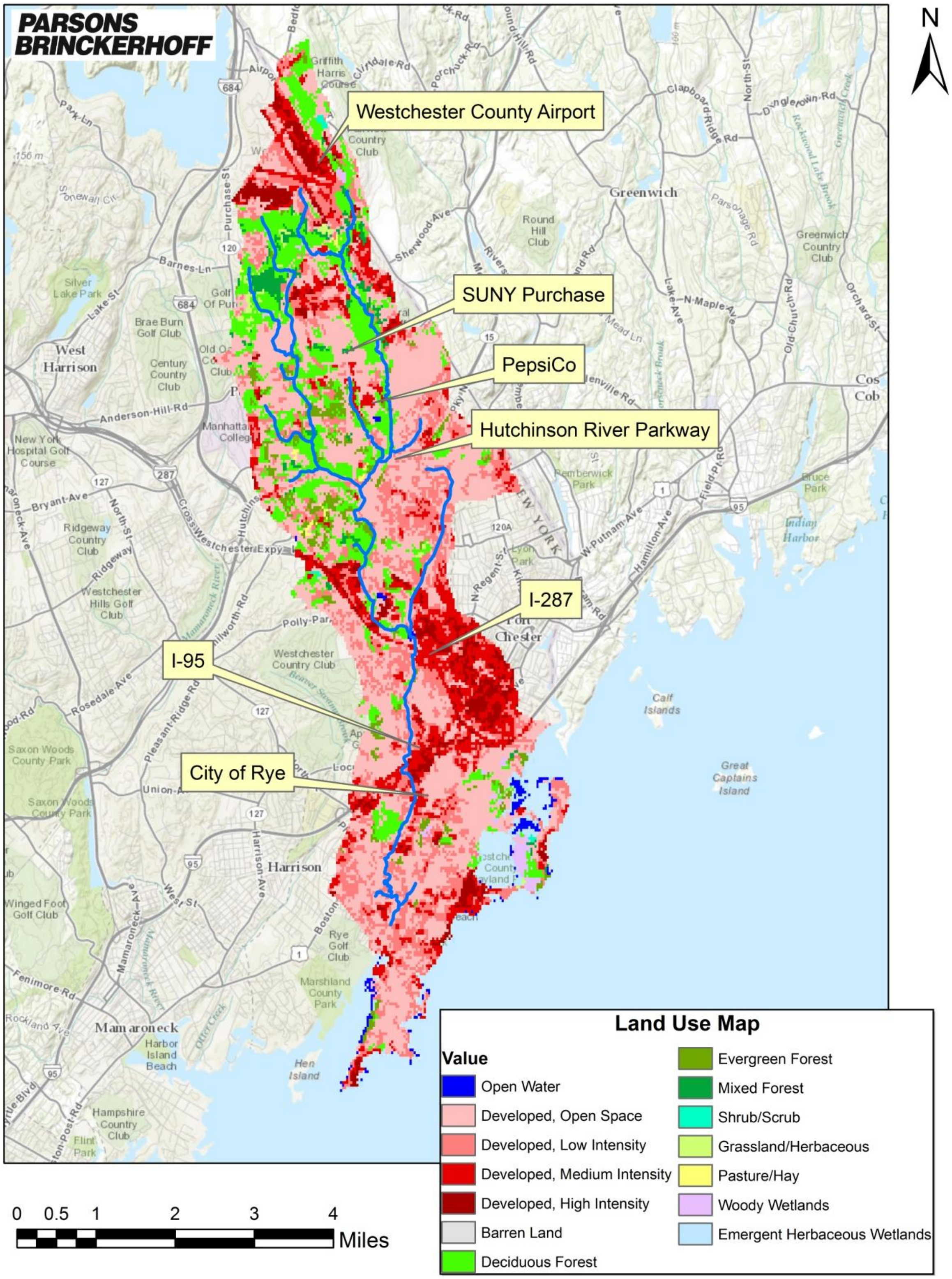


Figure 5 - Existing Condition Land Use Map (NLCD_2006 Data)

The soil types within the Blind Brook Watershed are based on Soil Survey Geography (SSURGO) data which is the most up-to-date database in Natural Resources Conservation Service (NRCS) Web Soil Survey website (Figure 6 and 7). SSURGO classification deals with the systematic categorization of soils based on distinguishing characteristics as well as criteria that dictate choices in use. The majority of the soils within the study watershed are Type B, then Type C, and Type A. Type B soils have moderate infiltration rate when thoroughly wetted. These soils have a moderate rate of water transmission (0.5-8.0 inches/hour). Appendix A listed all the soil type from the Web Soil Survey Website.

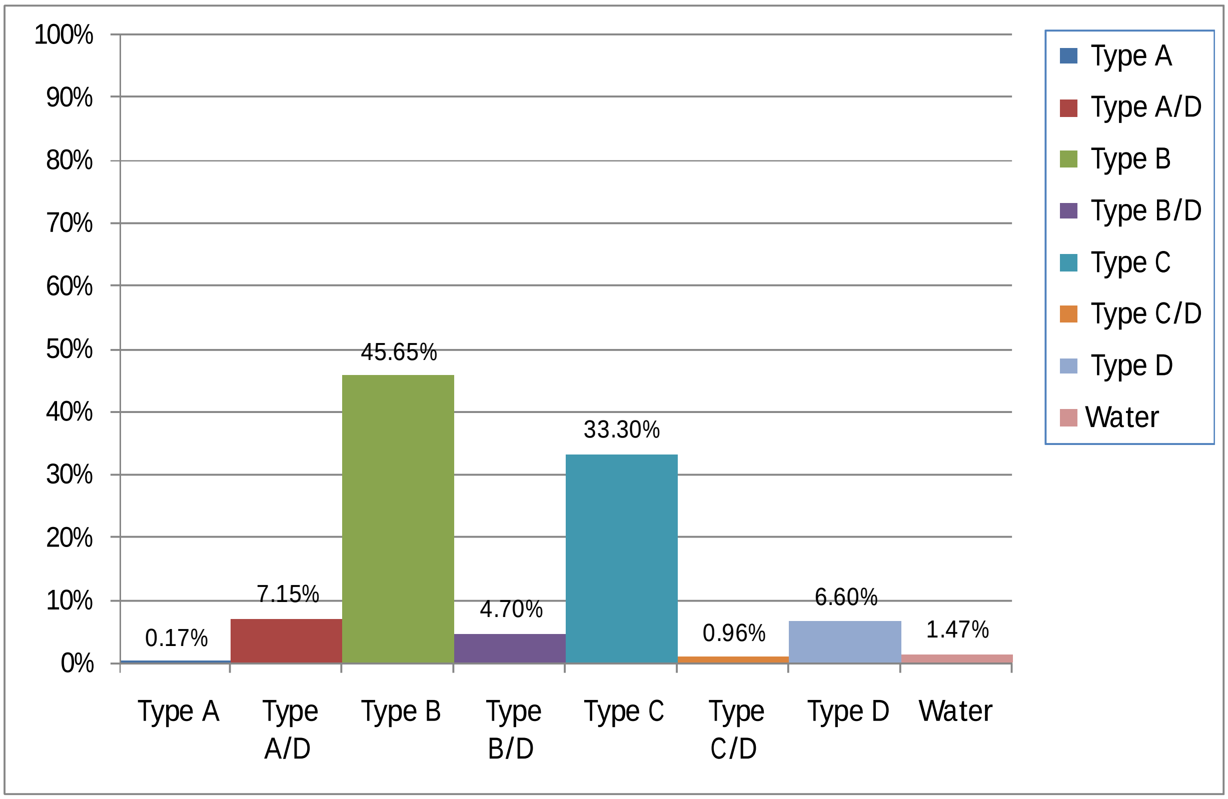


Figure 6 - Soil Breakdowns in Blind Brook Watershed

1.4 Stream Classification

The main stem of the Blind Brook and all its tributaries have been designated as *Classification C* by Article 15, Environmental Conservation Law Implementing Regulations, Code 6NYCRR Part 608, New York State Department of Environmental Conservation. *Classification C* is for waters supporting fisheries and suitable for non - contact activities. These waters shall be suitable for fish, shellfish, and wildlife propagation and survival. The water quality shall be suitable for primary and secondary contact recreation, although other factors may limit the use for these purposes.

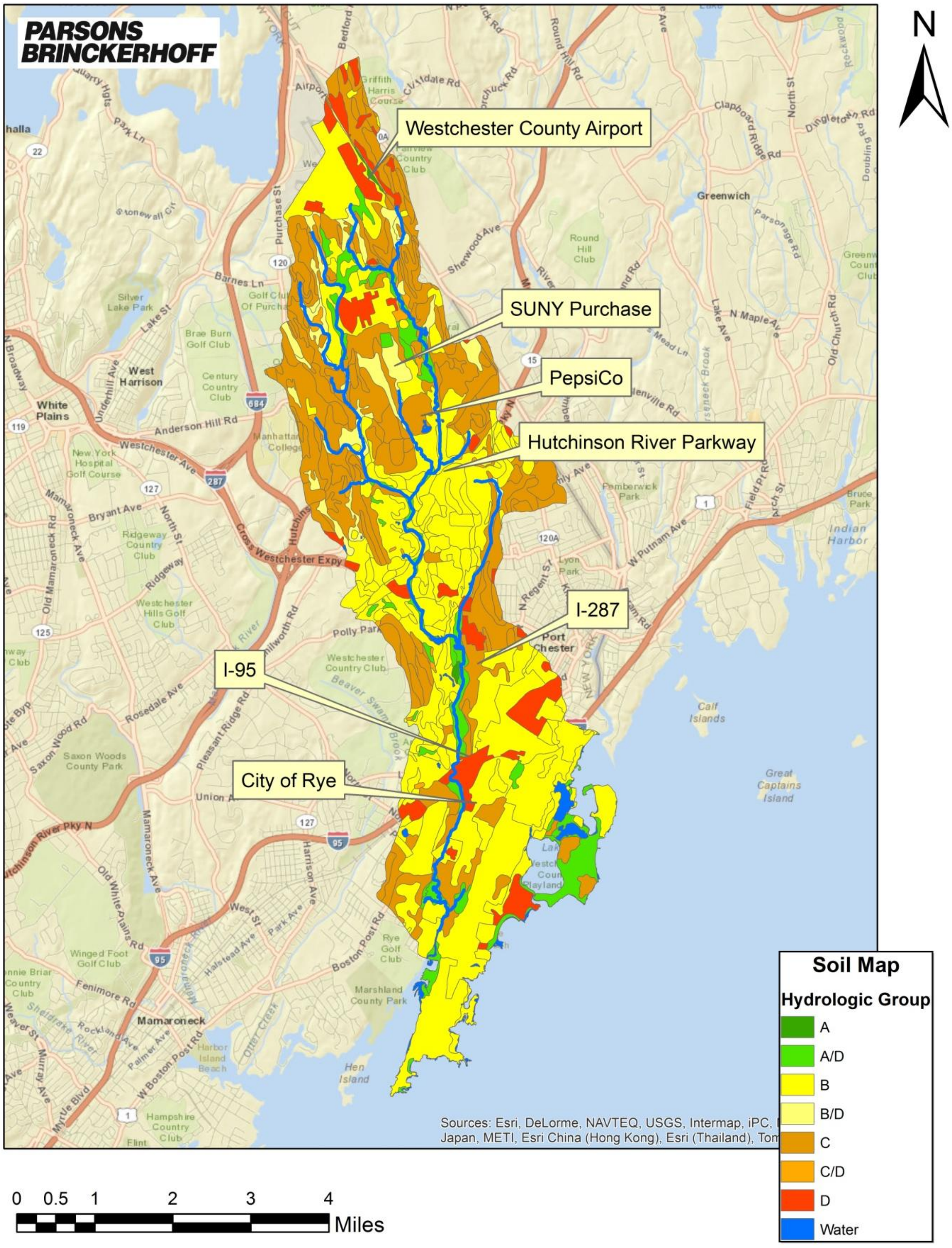


Figure 7 - SSURGO Soil Map (USGS Web Soil Survey)

1.5 FEMA Studies

Hydrologic and hydraulic studies of streams and rivers are often performed by the Federal Emergency Management Agency (FEMA) as part of the National Flood Insurance Program (NFIP). The Blind Brook and adjacent areas within the City of Rye are shown on the Flood Insurance Rate Map Community-Panel Number 36119C0356F and 36119C0352F (Figure 8-1, 8-2 and 8-3). The effective date of these Flood Insurance Rate Maps is September 27, 2007. Areas adjacent to the brook are labeled as Zone X or Zone AE. The floodplain limits were developed using approximate methods, i.e, flooding sources with low development potential or minimal flood hazard were studied using approximate methods. No detailed discharge values for the Blind Brook were provided in the FEMA Insurance Study for the Westchester County.

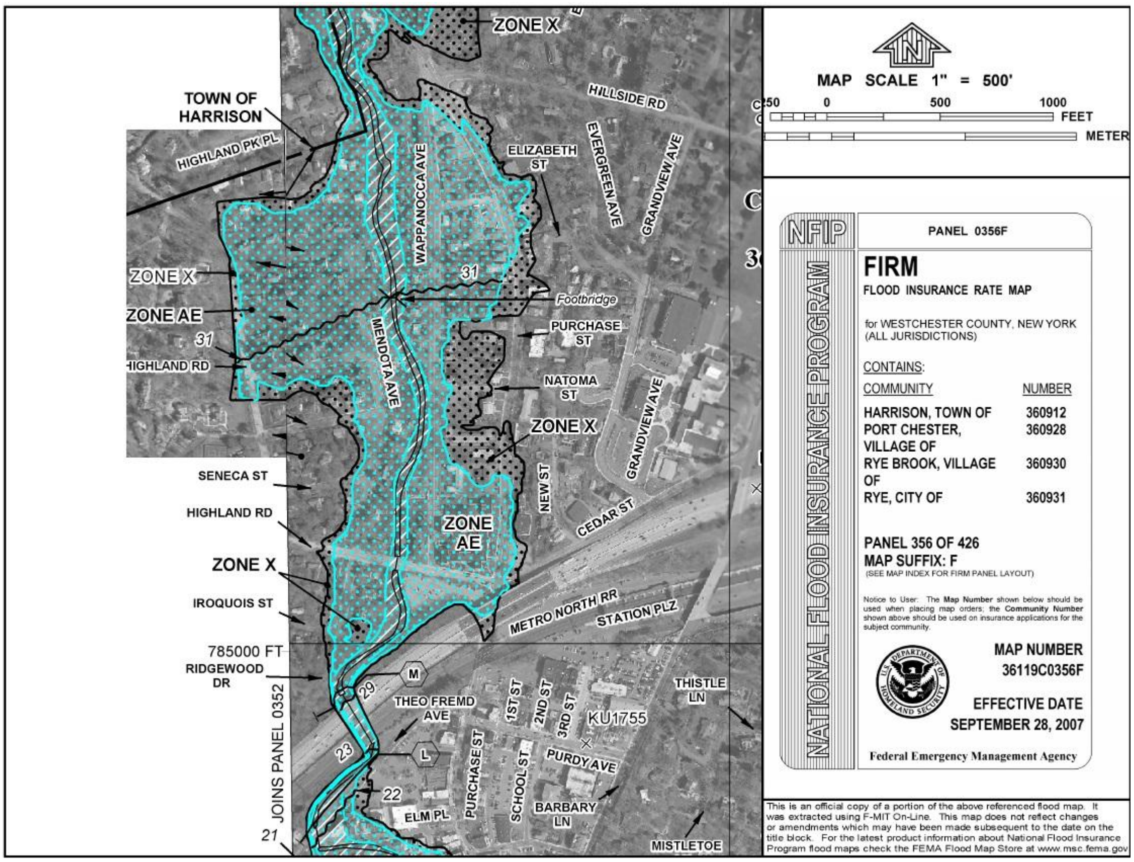
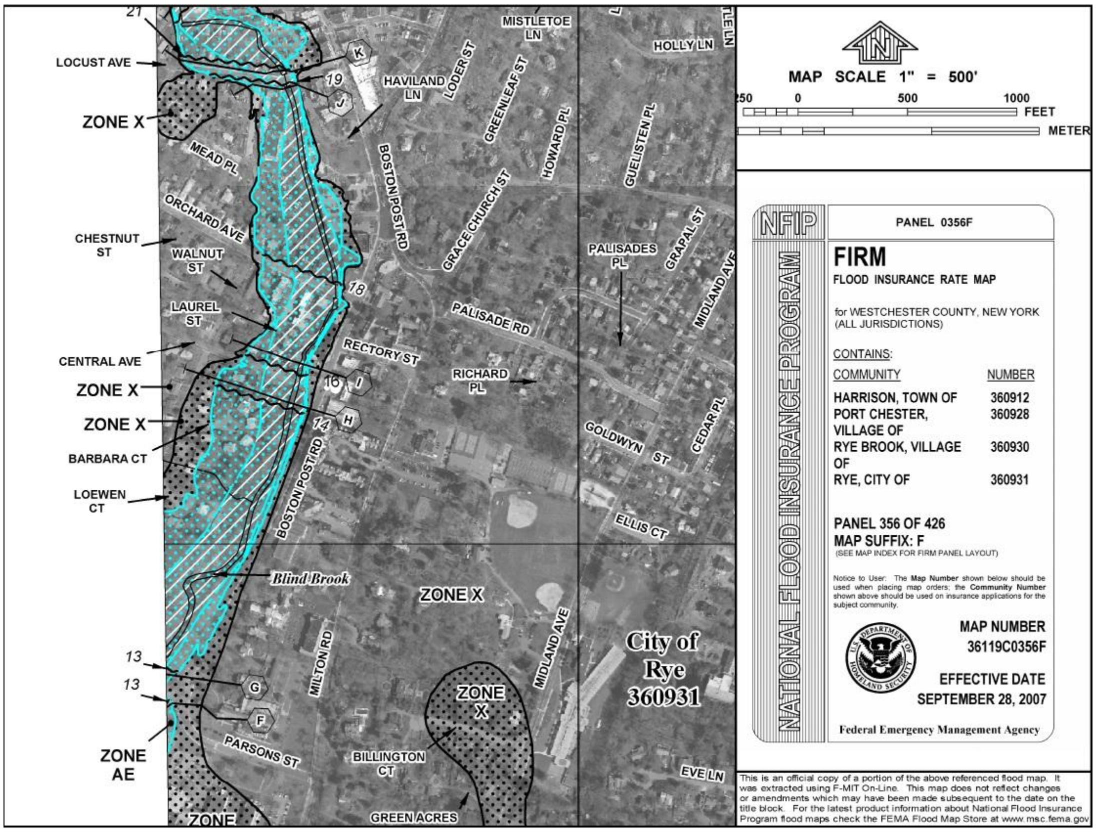


Figure 8-1 - Flood Insurance Rate Map



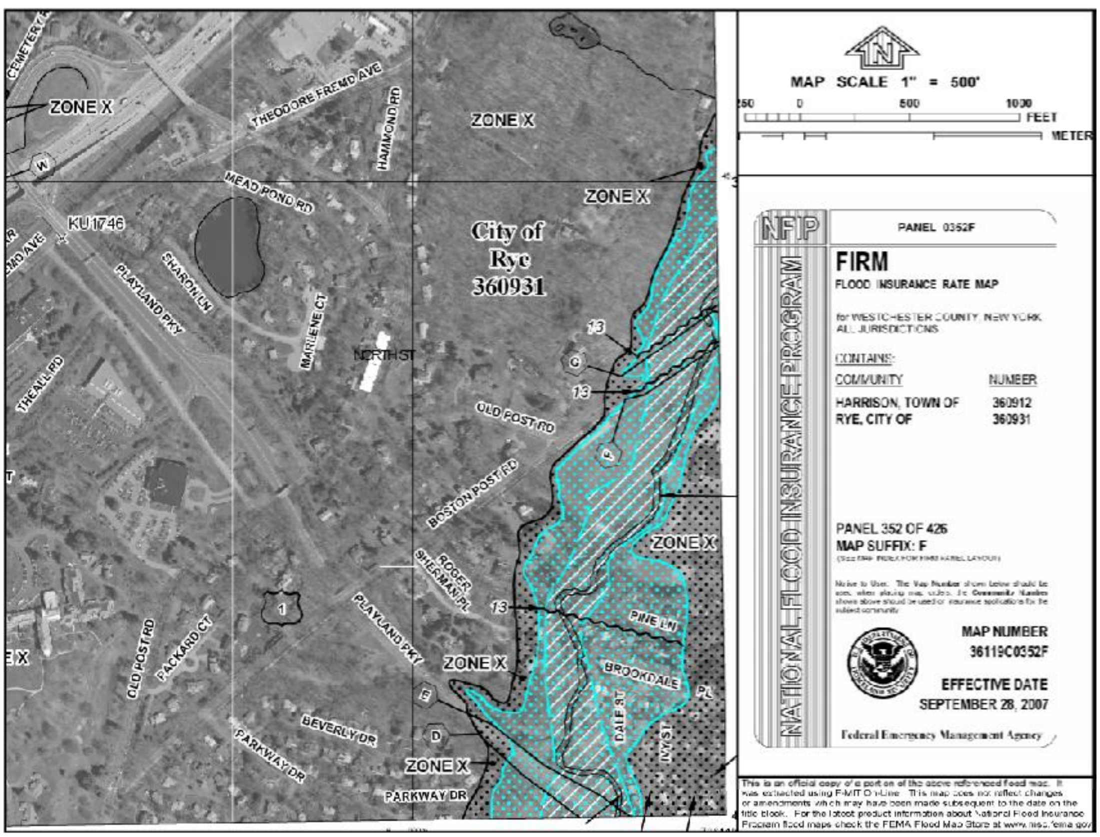


Figure 8-2 and 8-3 - Flood Insurance Rate Map (Community Panel Number: 36119C0356F and 36119C0352F)

1.6 Wetlands

According to National Wetland Inventory (NWI), there are wetlands in the immediate vicinity of the project. Detailed classification by U.S. Fish and Wildlife Service definition is mainly *Fresh Water Pond* along the main stem of the Blind Brook (Figure 9). Any impact to these areas will require authorization by the US Army Corp of Engineers (COE) and the New York Department of the Environmental Conservation (NY DEC), and may require mitigation.

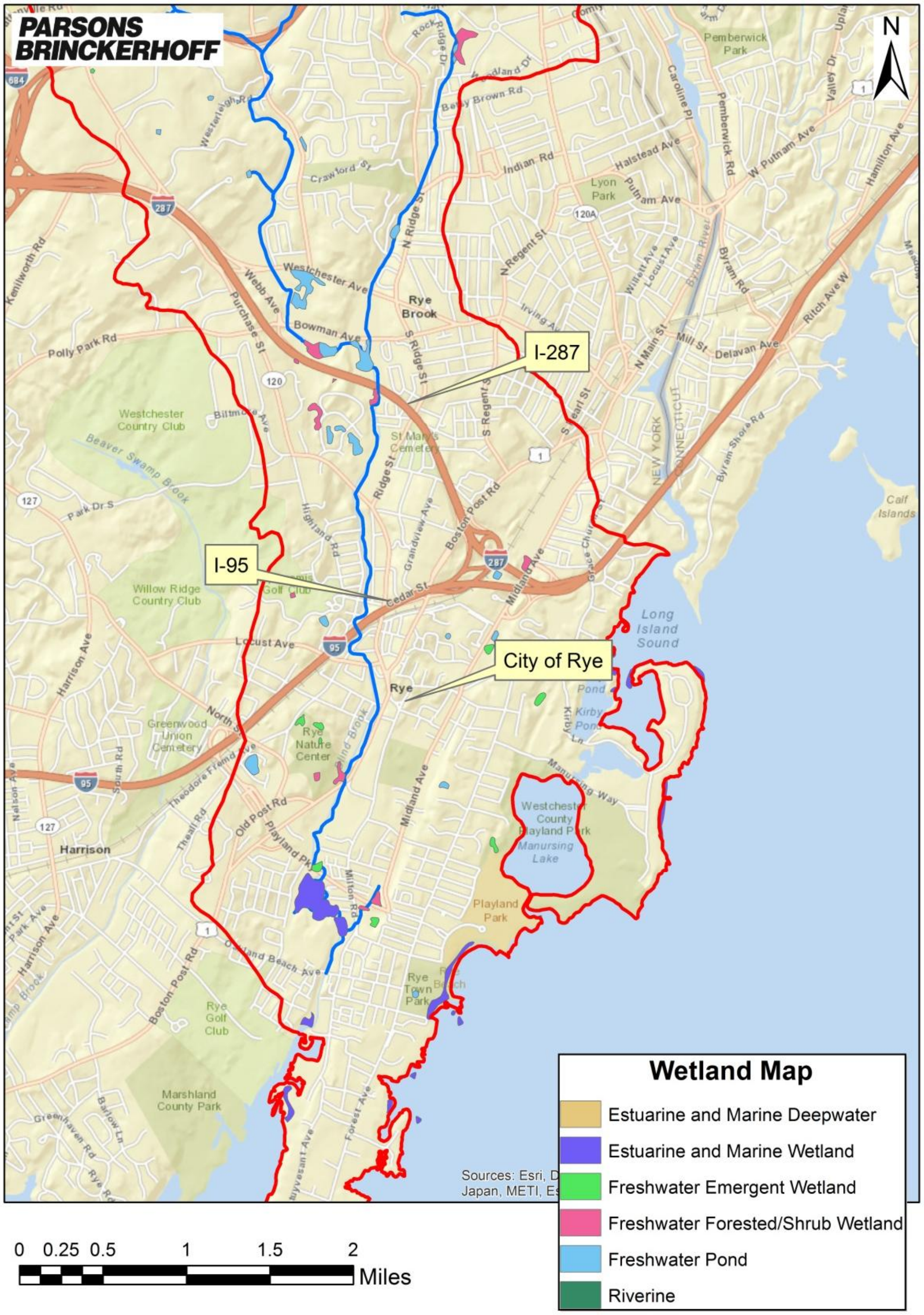


Figure 9 - Wetland Map (U.S. Fish and Wildlife Service)

1.7 Reference Datum

The North American Vertical Datum (NAVD) of 1988 was used in this study as was the horizontal North American Datum (NAD) of 83/91. The projection system is State Plane New York East – Feet.

1.8 Storm Events and Rainfall Depth

Rainfall distributions and depths are taken from National Weather Service's Technical Report No. 40 (TP-40) precipitation data. TP-40 provides storm durations for analysis from 5-minute to 60-day durations at average recurrence intervals from 1-year to 1,000-years. These estimates are based on review of annual maximum series then converted to a partial duration series. These estimates are based on improvements from a denser, more modern network with a longer period of recorded data, enhanced analysis techniques, and better application of spatial interpolation. The storm events and rainfall depth used in the analysis is listed as the followings; 2-year storm: 3.5 inches, 5-year storm: 4.3 inches, 10-year storm: 5.0 inches, 25-year storm: 5.7 inches, 50-year storm: 6.4 inches, 100-year storm: 7.2 inches.

1.9 Bankfull Channel Dimensions (U.S. Fish & Wildlife Service)

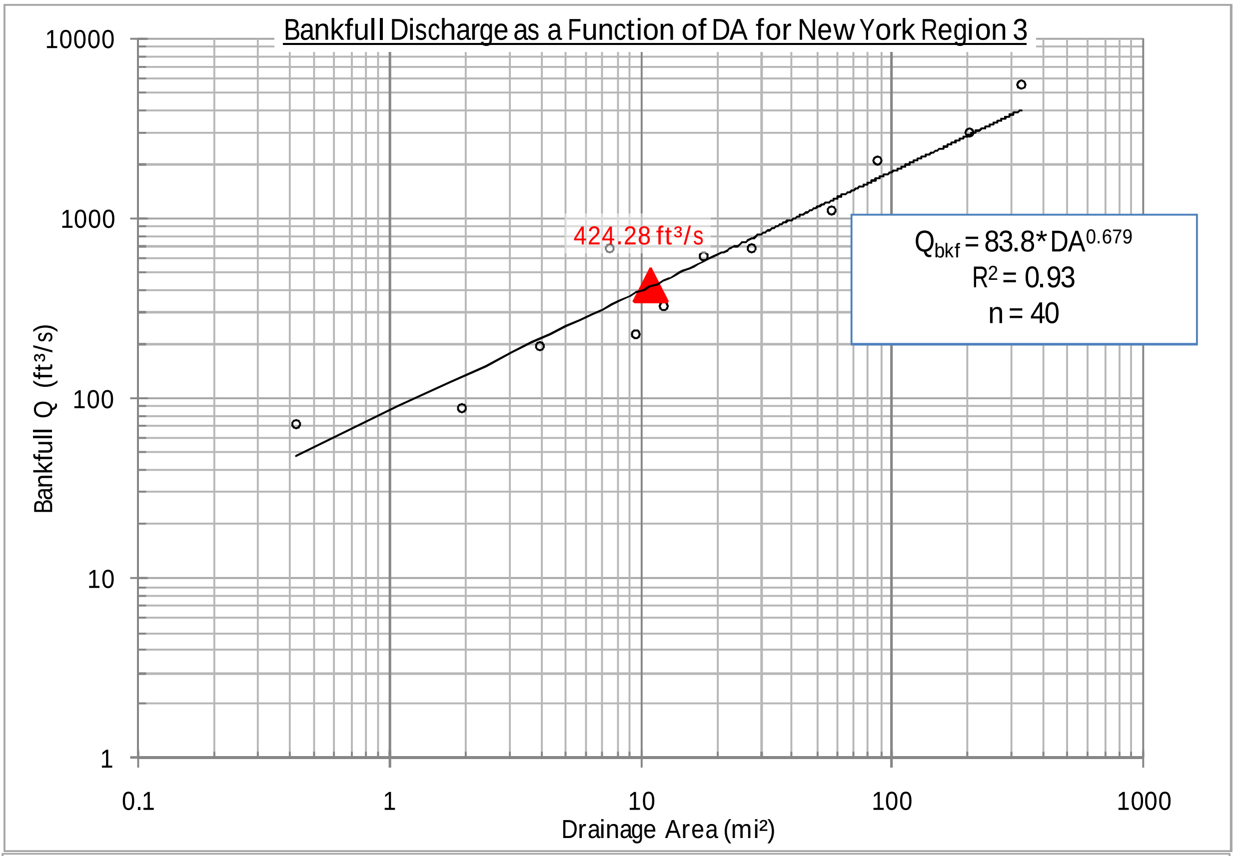
The U.S. Fish & Wildlife Service (USFWS), "Bankfull Discharge and Channel Characteristics of Streams in New York State" has developed relationships between drainage area and bankfull discharge and channel dimensions within New York State. The Blind Brook watershed is identified as being in Region 3 in this report. Region 3 refers to a diverse region that covers the Lower Hudson Valley from the Long Island Sound in Westchester County to the Catskills and includes large metropolitan cities such as Yonkers and the rural landscape of Sullivan County.

Using the relationship defined in the report, the bankfull discharges and channel dimensions can be approximated. The values offered below are for informational purposes only and not intended for design. Table 1 lists the bankfull discharges and channel dimensions calculated as functions of drainage areas for the Blind Brook watershed. Figure 10 shows the graphic view of results in Table 1, and the field data collected to develop the regression equations used to calculate the variables in the table.

Table 1 - Bankfull Discharges and Channel Dimensions

Watershed	DA (mi^2)	Q_{bkf} (cfs)	XS Area (ft^2)	Channel Width (ft)	Channel Depth (ft)
Blind Brook Watershed	10.9	424.28	132.35	48.21	2.74

Where: DA is Drainage Area (mi^2), Q_{bkf} is the bankfull discharge (ft^3/s), and XS stands for Cross Section.



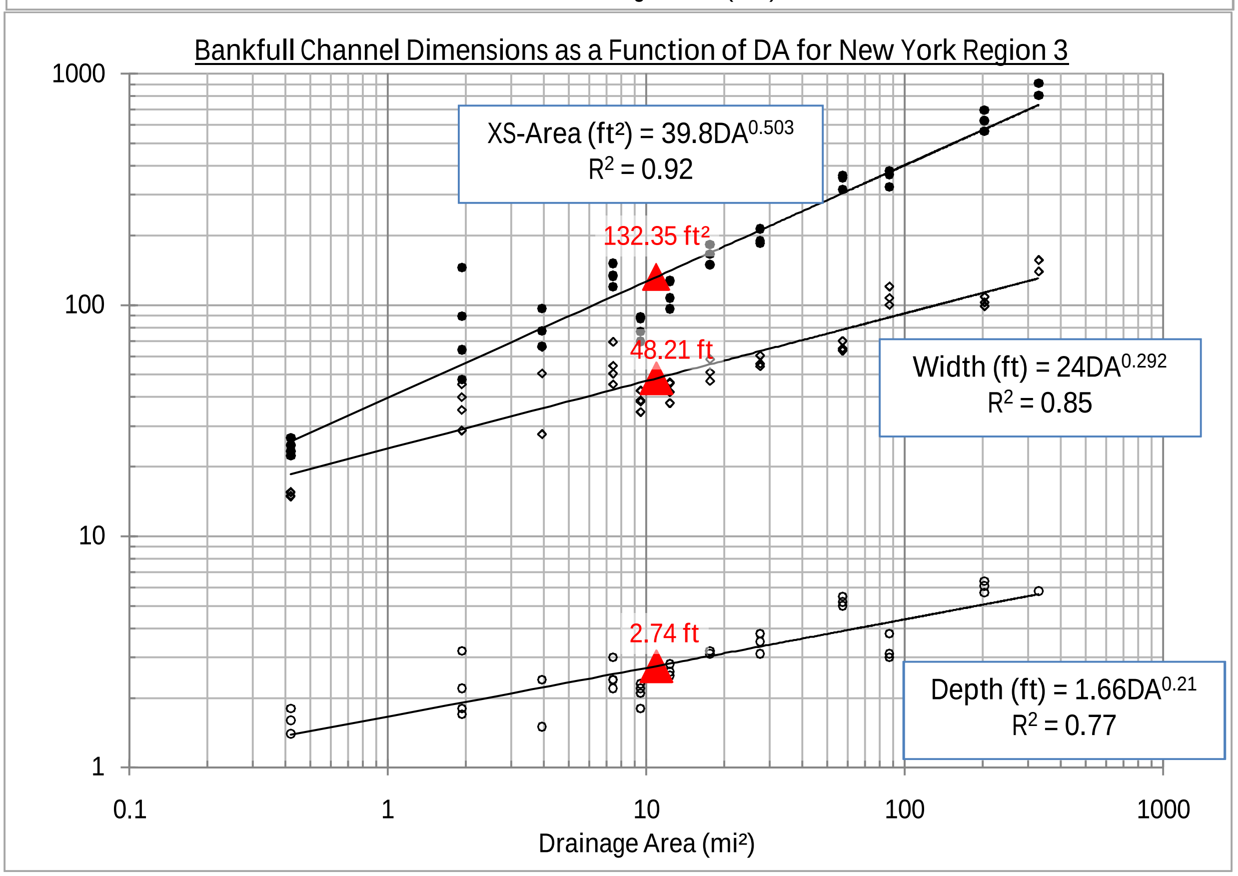


Figure 10-1 and 10-2 - Bankfull Dimensions of Blind Brook Watershed

2. Review of Existing Reports

Parsons Brinckerhoff (PB) has reviewed and analyzed reports previously submitted to the City of Rye. These reports are listed below:

1. Report 1: Watershed Plan and EIS – Blind Brook Watershed, USDA – Soil Conservation Service, July 1979;
2. Report 2: Project Report - Flood Mitigation Study – Bowman Avenue Dam Site, Chas. H. Sells, Inc., March 12, 2008;
3. Report 3: Project Report – Flood Mitigation Study – Lower Pond Supplemental, Chas. H. Sells, Inc., March 12, 2008;
4. Report 4: Hydrologic and Hydraulic Analysis – Bowman Avenue Dam Project – Study for Resizing the Upper Pond Reservoir, Paul C. Rizzo Engineering, New York, PLLC, September 21, 2012;
5. Report 5: Update to the 1999 Storm Water Management Plan: Westchester County Airport, TRC Engineers, Inc., December, 2010;
6. Report 6: PepsiCo Supplemental Storm Water Pollution Prevention Plan (SWPPP), John Meyer Consulting, PC, 2012.

This chapter summarizes the objectives, findings and recommendations, hydrologic and hydraulic modeling methods for each report identified above. A summary of the improvements with a list tabulating the hydraulic parameters, such as flow discharges and water surface elevations for a given storm event are also provided at the end of each report review.

Hydrology and hydraulics modeling yield different results because modeling requires the use of significant engineering judgment and interpretation which is based on observations, and experiences of the modeler. Flow calculations and water surface elevation calculations within a defined region should achieve the same order of magnitude. As a result, PB reviewed the modeling output and provided an explanation of why difference exists for flow rates and water surfaces between various reports.

2.1 Review of Report 1

("USDA Soil Conservation Service. Watershed Plan and Environmental Impact Stetement: Blind Brook Watershed", Westchster County, New York and Fairfield County, Connecticut, July 1979).

1. Purpose of the Report: The purpose of the watershed plan and environmental impact statement was to present the resource-related problems and needs within the Blind Brook Watershed in accordance with the Watershed Protection and Flood Prevention Act (PL 83-566). Incorporated within the watershed plan was an environmental impact statement for the Blind Brook Watershed. The plan describes the criteria for the selection of a plan of action to address the resource related problems and states the outcome of implementing the proposed plan of action.
2. Report 1's Findings and Recommended Improvements: The proposed plan of action provides for the implemention of two types of solutions. The first solution was the use of land treatment facilities such as sediment control measures to reduce erosion rates on construction sites. The plan required that all future lot plans be submitted to the Westchester County Soil

and Water Conservation District (SWCD) for review. The intention of such a review would require land treatment and runoff control measures for all future plot plans. The second solution included the installation of two flood retarding structures and four dikes throughout the brook. It was estimated that the installation of these structural measures would reduce flood damage by up to 73% for storm events up to the magnitude of the 100-year frequency. Sediment concentration at the mouth of the watershed would be reduced from 43 to 39 milligrams per liter.

3. Hydrologic and Hydraulic Analysis Methods: Three historic flood events, each associated with a hurricane, were evaluated in this report. The first flood event, which occurred on September 14, 1938, (unnamed hurricane) resulted in high tide flooding, as well as out-of-bank flooding. The flood events of June 19, 1972, (Hurricane Agnes) and of September 26, 1975, (Hurricane Eloise) occurred in the absence of high tide conditions; consequently, floodwater damages were relatively limited in Reach 1 (upstream of the City of Rye). Table C in the report shows peak discharges (in cubic feet per second) that were recorded at Gaging Station 01300000, located upstream from the Blind Brook Bridge on Theodore Fremd Avenue.

4. Final Results of the Report:

Table 2 - Peak Discharges at USGS Station "01300000"

Storm Date	Peak Discharges (cfs)	Estimated Frequency	Percent Chance
June 19, 1972	2,320	60 years	1.7
September 26, 1975	2,280	55 years	1.8

2.2 Review of Report 2

("Project Report – Flood Mitigation Study – Bowman Avenue Dam Site", Chas. H. Sells, Inc., March 12, 2008)

1. Purpose of the Report: This project involved a feasibility analysis of various flood damage reduction measures at the Bowman Avenue Dam site. This initiative is consistent with the City of Rye's (City) Flood Mitigation Plan dated November 2001 in which the City identified conceptual level improvements at the Bowman Avenue Dam as being part of a comprehensive plan to provide downstream flood control. This study assessed the feasibility, costs and benefits associated with conceptual flood control alternatives described below. It was the intent of this report to aid the City in implementing meaningful flood mitigation measures and to provide documentation necessary for securing Hazard Mitigation Grant Program (HMGP) funding.

2. Report 2's Findings and Recommended Improvements: Several alternatives were investigated as part of this report analysis. Each alternative was compared based on its benefit in terms of relative flow reduction, and lowering of downstream water surface elevations versus overall cost and impacts.

The preferred alternative (Alternative A), from a short-term perspective, consists of the installation of an automated sluice gate at the Bowman Avenue Dam. An automated sluice gate has the ability to vary the outlet opening, thus providing the optimum orifice size for the flow rate in the stream. For 2-year flood to 100-year flood, the orficice diameter ranges from 20.2 inches to 139.1 inches. The sluice gate would be automatically controlled based on water surface elevations measured at a gauge mounted at the dam. Based on the analysis, this alternative provides the most cost effective means to reduce water surface elevations downstream.

Other alternatives (Alternative B and C), including maximizing the storage potential of the Upper Pond behind the Bowman Avenue Dam in conjunction with the sluice gate, and dredging 2-ft of sediment accumulated in the Upper Pond resulted in a further reduction of downstream water surface elevations. The budgetary construction cost for these alternatives is estimated at $10 - $15 million. However, it should be noted that the cost/benefit of this alternative heavily relies on the limit of rock excavation and the presence of contaminated material. Further subsurface investigation including rock probes and soil testing is necessary.

3. Hydrologic and Hydraulic Analysis Methods: Blind Brook and East Branch Blind Brook were studied by detailed hydrologic and hydraulic methods for FEMA's preliminary flood insurance study (FIS) for Westchester County. Backup data was made available to Chas. H. Sells, Inc. (Sells) through Michael Baker, Jr., Inc. The area studied in this report on Blind Brook is from I-95 (south) to Interstate 1-287 (north). For the study of this reach of Blind Brook, base data from the FIS model was used as presented with the exception of flow rates. For this analysis the flow rate used was developed in Sells' August 2007 Hydrologic Report using WinTR-20 software. Sells' August 2007 Hydrological Report determined that the discharge rates in this reach of Blind Brook are greater than those used by FEMA for the existing conditions, and they believed the discharge values developed in their report are a more accurate representation of actual flood events based on methodology, calibration, and historical information.

   The software used for developing water surface profiles for Blind Brook and East Branch Blind Brook is the USACE's HEC-RAS program. All other data including cross sections, distances between cross sections, Manning's n values, bridge geometry, ineffective flow areas, etc. was applied as represented in the FEMA study. The model created was used as the baseline model for this report. The boundary condition (starting point of the backflow analysis) for each altemative was determined from a rating curve (included in Appendix C) developed from the existing FIS HEC-RAS water surface elevations at a section located approximately 850 feet downstream of Interstate I-95.

4. Final Results of the Report:

   Table 3 below listed the peak discharge and water surface elevation results of the existing conditon vs. three proposed alternatives. As it can be seen from this table, Alternative A provides the most water surface elevation reductions at all four downstream locations.

Table 3 - Final Results of Exsting Report 2

Existing Conditions			
Return Periods	Locations	Discharge (cfs)	W.S. Elevation (ft)
2-Year Storm	I-95 (U/S)	N/A	20.77
	Highland Rd. (U/S)	N/A	21.41
	Purchase St. (U/S)	N/A	25.65
	I-287 (D/S)	N/A	31.07
5-Year Storm	I-95 (U/S)	N/A	22.95
	Highland Rd. (U/S)	N/A	24.19
	Purchase St. (U/S)	N/A	27.20
	I-287 (D/S)	N/A	32.15
10-Year Storm	I-95 (U/S)	1,982	24.59
	Highland Rd. (U/S)	1,982	25.88
	Purchase St. (U/S)	1,663	28.33
	I-287 (D/S)	1,663	32.73
25-Year Storm	I-95 (U/S)	N/A	26.93
	Highland Rd. (U/S)	N/A	27.78
	Purchase St. (U/S)	N/A	30.06
	I-287 (D/S)	N/A	33.44
50-Year Storm	I-95 (U/S)	3,078	30.56
	Highland Rd. (U/S)	3,078	31.01
	Purchase St. (U/S)	2,767	31.91
	I-287 (D/S)	2,767	34.11
100-Year Storm	I-95 (U/S)	3,583	32.17
	Highland Rd. (U/S)	3,583	32.60
	Purchase St. (U/S)	3,346	33.44
	I-287 (D/S)	3,346	34.97
Alternative A – Optimize Orifice Opening			
Return Periods	Locations	Discharge (cfs)	W.S. Elevation (ft)
2-Year Storm	I-95 (U/S)	N/A	20.08
	Highland Rd. (U/S)	N/A	21.43
	Purchase St. (U/S)	N/A	25.65
	I-287 (D/S)	N/A	31.07
5-Year Storm	I-95 (U/S)	N/A	22.36
	Highland Rd. (U/S)	N/A	23.35
	Purchase St. (U/S)	N/A	26.61
	I-287 (D/S)	N/A	31.62
10-Year Storm	I-95 (U/S)	1,789	23.79
	Highland Rd. (U/S)	1,789	25.24
	Purchase St. (U/S)	1,344	27.73
	I-287 (D/S)	1,344	32.27

25-Year Storm	I-95 (U/S)	N/A	26.19
	Highland Rd. (U/S)	N/A	27.20
	Purchase St. (U/S)	N/A	29.21
	I-287 (D/S)	N/A	32.87
50-Year Storm	I-95 (U/S)	2,461	26.41
	Highland Rd. (U/S)	2,461	27.39
	Purchase St. (U/S)	2,458	30.18
	I-287 (D/S)	2,458	33.66
100-Year Storm	I-95 (U/S)	3,274	31.12
	Highland Rd. (U/S)	3,274	31.57
	Purchase St. (U/S)	3,117	32.55
	I-287 (D/S)	3,117	34.54

Alternative B – Optimize Orifice Opening, Maximum Upper Pond Volume			
Return Periods	Locations	Discharge (cfs)	W.S. Elevation (ft)
2-Year Storm	I-95 (U/S)	N/A	20.66
	Highland Rd. (U/S)	N/A	21.29
	Purchase St. (U/S)	N/A	25.52
	I-287 (D/S)	N/A	30.09
5-Year Storm	I-95 (U/S)	N/A	21.89
	Highland Rd. (U/S)	N/A	22.72
	Purchase St. (U/S)	N/A	26.27
	I-287 (D/S)	N/A	31.29
10-Year Storm	I-95 (U/S)	1,289	22.12
	Highland Rd. (U/S)	1,289	23.04
	Purchase St. (U/S)	933	26.45
	I-287 (D/S)	933	31.47
25-Year Storm	I-95 (U/S)	N/A	24.73
	Highland Rd. (U/S)	N/A	26.01
	Purchase St. (U/S)	N/A	28.28
	I-287 (D/S)	N/A	32.51
50-Year Storm	I-95 (U/S)	2,176	25.32
	Highland Rd. (U/S)	2,176	26.51
	Purchase St. (U/S)	2,049	29.00
	I-287 (D/S)	2,049	33.20
100-Year Storm	I-95 (U/S)	2,877	30.07
	Highland Rd. (U/S)	2,877	30.52
	Purchase St. (U/S)	2,798	31.54
	I-287 (D/S)	2,798	34.08

Alternative C – Optimize Orifice Opening, Maximum Upper Pond Volume, Dredge 2-ft Sediment Material from Upper Pond			
Return Periods	Locations	Discharge (cfs)	W.S. Elevation (ft)
2-Year Storm	I-95 (U/S)	N/A	20.32
	Highland Rd. (U/S)	N/A	20.93
	Purchase St. (U/S)	N/A	25.32
	I-287 (D/S)	N/A	30.79
5-Year Storm	I-95 (U/S)	N/A	21.79
	Highland Rd. (U/S)	N/A	22.70
	Purchase St. (U/S)	N/A	26.22
	I-287 (D/S)	N/A	31.25
10-Year Storm	I-95 (U/S)	1,112	21.48
	Highland Rd. (U/S)	1,112	21.72
	Purchase St. (U/S)	908	26.14
	I-287 (D/S)	908	31.41
25-Year Storm	I-95 (U/S)	N/A	24.62
	Highland Rd. (U/S)	N/A	25.92
	Purchase St. (U/S)	N/A	28.20
	I-287 (D/S)	N/A	32.47
50-Year Storm	I-95 (U/S)	2,167	25.29
	Highland Rd. (U/S)	2,167	25.41
	Purchase St. (U/S)	2,042	28.98
	I-287 (D/S)	2,042	33.19
100-Year Storm	I-95 (U/S)	2,861	30.04
	Highland Rd. (U/S)	2,861	30.08
	Purchase St. (U/S)	2,787	31.51
	I-287 (D/S)	2,787	34.06

2.3 Review of Report 3

("Project Report – Flood Mitigation Study – Lower Pond Supplemental", Chas. H. Sells, Inc., March 12, 2008)

1. Purpose of the Report: Chas. H. Sells, Inc. (Sells) was retained by the City of Rye to investigate additional flood storage alternatives consistent with the conclusions and recommendations for the Lower Pond downstream of the Bowman Avenue Dam. This report was considered a supplement to the afore-mentioned study, "Flood Mitigation Study - Bowman Avenue Dam Site", dated March 12, 2008. This report evaluated two alternatives designed to maximize the storage potential of the Lower Pond, one is a gravity based while the other is mechanical.

   Additional alternatives studied as part of this analysis included: 1. Alternative A: Increasing the storage area at the Lower Pond site, gravity based, providing an outlet structure with the principal spillway at elevation 31.0'; 2. Alternative B: increasing the storage area at the Lower Pond site, mechanical based, providing an outlet structure with the principal spillway at elevation 31.0’; 3. Alternative C: Increasing the storage area at the Lower Pond, gravity based, providing an outlet structure with the principal spillway at elevation 35.5’; 4. Alternative D: Increasing the storage area at the Lower Pond, gravity based, providing an outlet structure with the principal spillway at elevation 35.5’, and optimizing the Bowman Avenue dam outlet.

2. Report 3’s Findings and Recommended Improvements: The results of the analysis indicate that the gravity-based and mechanical-based alternatives (Alternatives A & B) both provide comparable flow reductions. For 2, 5- and 10-year frequency storms, there is a 2% to 6% reduction in flows (calculated at I-95). For the 25, 50- and 100-year storms, flow reductions were less; ranging from 0% to 4%. Alternative C showed a higher flow reduction at I-95 with a 22% and 8% reduction for 5 and 10-year frequency storms, respectively. However, similar to Alternatives A and B, this alternative had less of an effect on the lower frequency events. For the 25, 50 and 100-year storms the reductions ranged from 2% to 8%.

   Based on the analysis, Alternative D provides the largest discharge reduction. This was accomplished by increasing the Lower Pond storage and providing a fixed outlet structure with the principal spillway at elevation 35.5 and combining this effect with optimizing the Bowman Avenue dam outlet. Whereas Alternative D showed comparable flow reductions for the 2, 5 and 10-year storms it had a greater impact on the 25, 50 and 100-year events (10%, 23% and 9% reductions respectively). This is primarily attributed to the influence of the sluice gate. Alternative D was further analyzed to calculate downstream water surface elevations.

3. Hydrologic and Hydraulic Analysis Methods: The existing and Bowman Avenue dam outlet optimization conditions WinTR-20 models used in March 12, 2008 report were modified to account for the new Lower Pond modification alternates. The model is based on available TR-20 data included in the 1979 Flood Insurance Study Backup information for the City of Rye. The backup information includes drainage areas, Runoff Curve Numbers and time of concentration for each sub watershed and the model schematic. Although this data is from the 1970's and might not represent existing conditions, including the extent of natural and manmade changes that have occurred in the watershed, for the purpose of determining inflow/outflow rate at the Bowman Avenue Dam, the available data is considered valid . This is the same data that was used in the April 2007 ACOE report.

   The software used by FEMA FIS for developing water surface profiles for Blind Brook and East Branch Blind Brook is the US Army Corp of Engineers HEC-RAS software. HEC-RAS is a more recent developed windows version of the DOS based HEC-2 computer program. The program is designed to perform one dimensional hydraulic calculations of natural and manmade channels. Water surface profiles are computed using an iterative procedure called the standard step method. The water surface elevations are calculated from section to section by solving the Energy equation. The bridge modeling approach chosen in the FIS is the "momentum" for low flows and "pressure and/or weir" for high flows. The boundary condition (starting point of the backflow analysis) for each alternative was determjned from a rating curve developed from the existing FIS HEC-RAS water surface elevations at a section located approximately 850 feet downstream of Interstate I-95. The water surface elevations for the existing, recommended outlet optimization at Bowman dam and Alternate D alternatives were computed and presented in the Appendix of this report.

4. Final Results of the Report:

Table 4 below listed the peak discharge and water surface elevation results of Alternative C and D. As it can be seen from this table, Alternative D provides the most water surface elevation reductions at all four downstream locations.

Table 4 - Final Results of Exsting Report 3

Alternate C – Optimize Orifice Opening at Bowman Dam			
Return Periods	Locations	Discharge (cfs)	W.S. Elevation (ft)
2-Year Storm	I-95 (U/S)	928	20.80
	Highland Rd. (U/S)	928	21.43
	Purchase St. (U/S)	781	25.65
	I-287 (D/S)	781	31.51
5-Year Storm	I-95 (U/S)	1,344	22.36
	Highland Rd. (U/S)	1,344	23.35
	Purchase St. (U/S)	999	26.61
	I-287 (D/S)	999	32.10
10-Year Storm	I-95 (U/S)	1,789	23.89
	Highland Rd. (U/S)	1,789	25.24
	Purchase St. (U/S)	1,344	27.73
	I-287 (D/S)	1,344	32.82
25-Year Storm	I-95 (U/S)	2,403	26.19
	Highland Rd. (U/S)	2,403	27.2
	Purchase St. (U/S)	1,775	29.21
	I-287 (D/S)	1,775	33.48
50-Year Storm	I-95 (U/S)	2,461	26.41
	Highland Rd. (U/S)	2,461	27.39
	Purchase St. (U/S)	2,458	30.18
	I-287 (D/S)	2,458	34.58
100-Year Storm	I-95 (U/S)	3,274	31.12
	Highland Rd. (U/S)	3,274	31.57
	Purchase St. (U/S)	3,117	32.55
	I-287 (D/S)	3,117	35.70

Alternate D – Increasing the Storage Area at the Lower Pond			
Return Periods	Locations	Discharge (cfs)	W.S. Elevation (ft)
2-Year Storm	I-95 (U/S)	871	20.54
	Highland Rd. (U/S)	871	21.17
	Purchase St. (U/S)	666	25.4
	I-287 (D/S)	666	31.09
5-Year Storm	I-95 (U/S)	1,163	21.66
	Highland Rd. (U/S)	1,163	22.66
	Purchase St. (U/S)	850	26.16
	I-287 (D/S)	850	31.71
10-Year Storm	I-95 (U/S)	1,633	23.33
	Highland Rd. (U/S)	1,633	24.66
	Purchase St. (U/S)	1,270	27.38
	I-287 (D/S)	1,270	32.68
25-Year Storm	I-95 (U/S)	2,328	25.9
	Highland Rd. (U/S)	2,328	26.98
	Purchase St. (U/S)	1,765	29.05
	I-287 (D/S)	1,765	33.47
50-Year Storm	I-95 (U/S)	2,385	26.12
	Highland Rd. (U/S)	2,385	27.16
	Purchase St. (U/S)	2,407	30.15
	I-287 (D/S)	2,407	34.5
100-Year Storm	I-95 (U/S)	3,255	31.06
	Highland Rd. (U/S)	3,255	31.51
	Purchase St. (U/S)	3,096	32.51
	I-287 (D/S)	3,096	35.67

2.4 Review of Report 4

("Hydrologic Analysis in Study for Resizing the Upper Pond Reservoir", Paul C. Rizzo Engineering – New York, PLLC, September 21, 2012.)

1. Purpose of the Report: Paul C. Rizzo Engineering (RIZZO) was retained by Sells to perform a Hydrologic and Hydraulic (H&H) analysis to determine the potential benefits of resizing the Upper Pond in order to increase temporary storage capacity. The watershed was subdivided into six sub-watersheds according to topographic and hydrologic conditions (Figure 11). The intention was to retime the storm water flows in order to decrease water surface profiles within the Blind Brook watershed between Interstates I-287 and I-95. RIZZO was also asked to consider optimizing the sluice gate operation to increase potential benefits from the new sluice gate.

Table 5 - Subwatershed Parameters in RIZZO's Report (2012)

Sub-watersheds	Drainage Areas (mi^2)	CN	Time of Concentration (hr)	Storage Coefficient (hr)
Blind Brook Country Club (SW1)	2.10	75	3.01	1.24
Lincoln Avenue (SW2)	3.28	73	2.51	0.89
Bowman Avenue (SW3)	1.46	74	1.4	0.56
I-287 (SW4)	1.19	75	1.89	0.99
Purchase Street (SW5)	0.68	67	1.16	0.63
I-95 (SW6)	0.44	62	0.77	0.43

* The red highlighted box in Table 5 indicates the Upper Blind Brook Watershed that will be further analyzed in the following chapter.

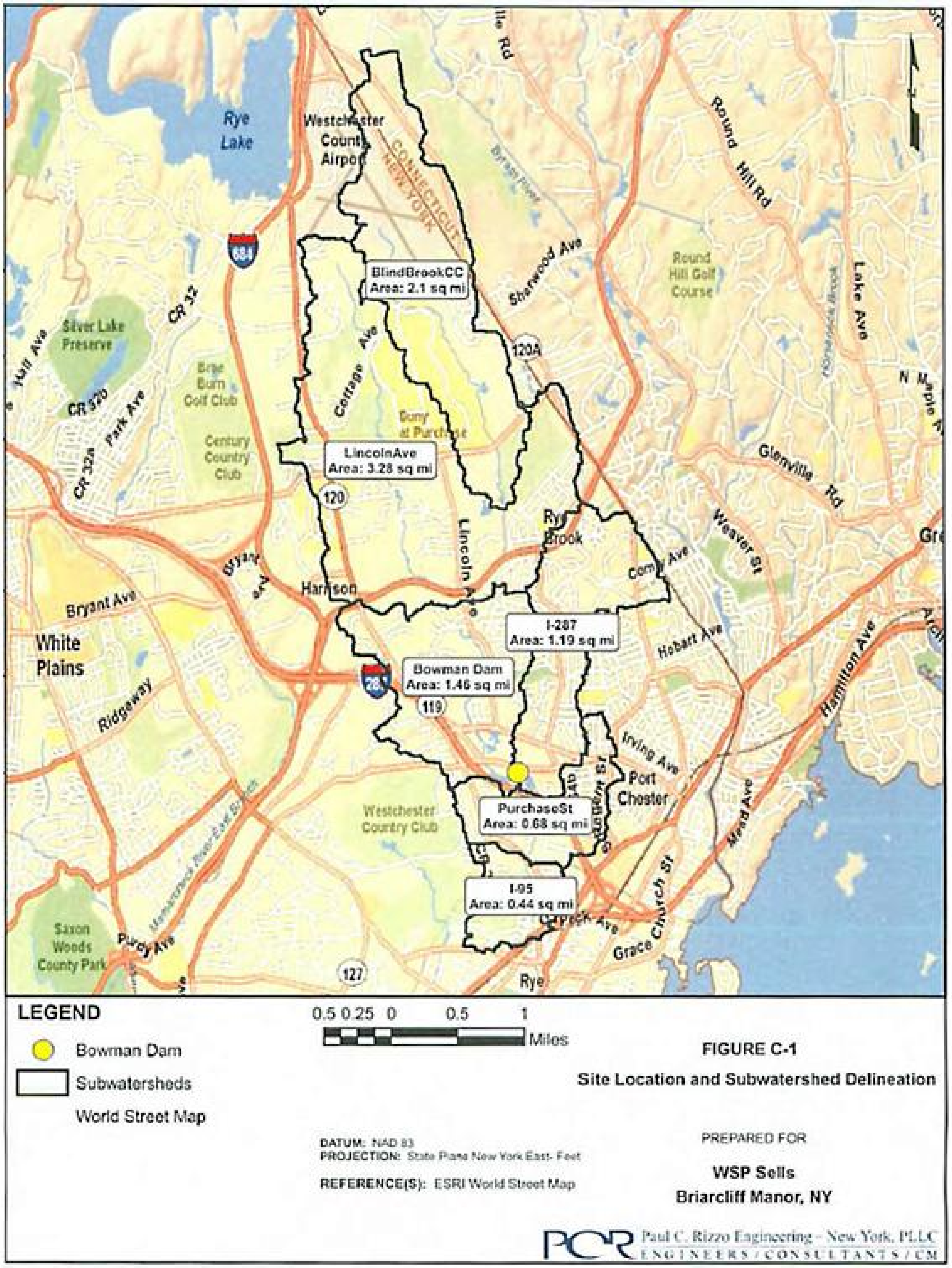


Figure 11 - Site Location and Sub-watershed Delineation (RIZZO Report, 2012)

2. Report 4's Findings and Recommended Improvements: Output from the RIZZO's hydrologic analysis consists of hydrographs obtained for the six sub-watersheds composing the Blind Brook Watershed. The hydrographs are used as input data for the hydraulic analysis

(unsteady HEC-RAS model) described in that report. Table 6 presents the hydrograph peak flows for storm events ranging from 2 to 100-year return periods at selected points within the watershed.

Table 6 - Hydrograph Peak Values in CFS (RIZZO Report, 2012)

Storm Event	Blind Brook & Lincoln Av. (SW1&SW2)	Bowman Avenue (SW3)	I-287 (SW4)	Purchase Street (SW5)	I-95 (SW6)
2-years	1054.2	540.9	336.4	221.3	163.9
5-years	1602.8	790.6	491.3	328	244.1
10-years	2073.2	1001.9	622.2	420.1	314.1
25-years	2539.7	1165.2	723.2	492.2	369.3
50 years	3023.6	1359.7	843.5	578.9	436.1
100 years	3560.7	1585.9	983.3	680.8	515.1

* The red highlighted box in Table 6 indicates the Upper Blind Brook Watershed that will be analyzed further in the following chapter.

The 1.32 mi^2 Airport drainage areas in the *Update to the 1999 Storm Water Management Plan Westchester County Airport*, contributes a significant amount of discharge when compared with peak discharges from the Blind Brook & Lincoln Ave. drainage areas (2.1 mi^2 and 3.28 mi^2, respectively) in the RIZZO report. For 100-year flood of the Existing Condition (2010), the ratio of Airport discharges vs. SW1 & SW2 discharges is computed as: 1595.1 cfs / 3560.7 cfs = 45 %. For 100-year flood of Proposed Condition (2011), this ratio is: 1318.8 cfs / 3560.7 cfs = 37 %.

The analysis aims to determine the potential impacts of the proposed resized Upper Pond Alternatives on water surface profiles between Interstates I-287 and I-95. It also considers an optimized sluice gate operating sequence. Water levels corresponding to the 2-year, 5-year, 10-year, 25-year, 50-year and 100-year return storm events were determined. The following scenarios were analyzed:

- Case A: Existing Condition
- Case B: Sluice Gate Installation
- Case C: Proposed Resized Upper Pond
- Case D: Proposed Maximized Resized Upper Pond
- Case E: Combination Resized Upper Pond and Sluice Gate

Results of this study show a potential reduction in downstream water elevations resulting from the sluice gate installation for large storm events (i.e. floods with return periods between 25 and 100 years). Overall, water elevations are projected to be approximately 6 inches lower after sluice gate installation for the 50- and 100-year return period floods.

Results also show that between the two pond alternatives. Cases C and D, the incremental benefit gained with the maximized resized alterative (Case C) is insignificant. By implementing the smaller resized pond alternative (Case D), potential water elevations are between 8 and 10 inches lower for the smaller storm events (i.e. 2 to 10 year period of return storms) and around 4 or 5 inches lower for the larger storm events (i.e. 50 and 100 year period of return). Case E, which models the smaller resized pond alternative with the sluice gate installed, shows overall potential water surface level decrease of 10 to 15 inches between I-287 and I-95 during larger storm events.

3. Hydrologic and Hydraulic Modeling Methods: In the hydrologic analysis, peak discharges from various precipitation events were computed using Hydrologic Modeling System software HEC-HMS (version 3.4) developed by US Army Corps of Engineers (USACE).

   The Geographic Information System (GIS) software ArcGIS was used to manage and analyze the most current topographic and hydrologic data available in order to create the HEC-HMS model. Arc-Hydro tools (ArcGIS Application) were used to delineate the watershed. The U.S. Department of Agriculture-Natural Resources Conservation Service (NRCS)-Curve Number (CN) and the Snyder Transform Methods were used to model the hydrologic loss and to transform the rainfall excess into runoff hydrographs.

   Finally inflow hydrographs obtained from HEC-HMS for the different storm events serve as input into the HEC-RAS model. The model starts in the vicinity of Crawford Park and ends approximately 800 feet downstream of I-95. This is an unsteady flow HEC-RAS model which runs equation with an implicit finite difference method to calculate equation solutions. This approach allows storm event to be routed within rivers while modeling hydrograph variation in space and time as well as the flood wave attenuation. The results represent a very accurate modeling of real flooding phenomena.

4. Final Results of the Report

   Table 7 below listed the peak discharge and water surface elevation results of the existing conditon (Case A) vs. four proposed alternatives (Case B, C, D and E). As it can be seen from this table, Case D and Case E provide the most water surface elevation reductions at all five downstream locations.

Table 7 - Final Results of Exsting Report 4

Return Periods	Locations	Existing Cond. (CASE A)	Sluice Gate Inst. (CASE B)	Resized Upper Pond-Alt. 2 (CASE C)	Max. Upper Pond-Alt. 1 (CASE D)	Resized Upper Pond-Alt 2 & SG (CASE E)
2-Year Storm	D/S of I-287	33.8	33.8	33.2	33.1	33.2
	Purchase Street	28.3	28.3	27.7	27.6	27.7
	Mendota Avenue	24.9	24.9	24.4	24.3	24.4
	Highland Road	24.5	24.5	23.8	23.7	23.8
	U/S I-95	23.4	23.4	22.9	22.9	22.9
5-Year Storm	D/S of I-287	34.5	*	34.1	34.0	34.0
	Purchase Street	29.8	*	29.0	28.8	28.8
	Mendota Avenue	26.6	*	25.7	25.5	25.5
	Highland Road	26.5	*	25.5	25.3	25.3
	U/S I-95	24.7	*	23.8	23.7	23.7
10-Year Storm	D/S of I-287	35.1	*	34.9	34.9	34.9
	Purchase Street	31.0	*	30.6	30.5	30.5
	Mendota Avenue	27.8	*	27.3	27.3	27.3
	Highland Road	27.7	*	27.2	27.2	27.2
	U/S I-95	26.1	*	25.5	25.4	25.4
25-Year Storm	D/S of I-287	35.5	35.4	35.4	35.4	35.3
	Purchase Street	31.7	31.6	31.5	31.4	31.4
	Mendota Avenue	28.7	28.6	28.2	28.2	28.3
	Highland Road	28.6	28.5	28.2	28.1	28.3
	U/S I-95	27.3	27.2	26.8	26.7	26.9
50-Year Storm	D/S of I-287	35.9	35.7	35.9	35.9	35.5
	Purchase Street	32.5	32.1	32.3	32.3	31.9
	Mendota Avenue	29.8	29.4	29.4	29.4	29.0
	Highland Road	29.8	29.3	29.4	29.3	28.9
	U/S I-95	28.7	28.2	28.2	28.2	27.7
100-Year Storm	D/S of I-287	36.3	36.1	36.2	36.2	35.9
	Purchase Street	33.2	33.0	33.1	33.1	32.6
	Mendota Avenue	31.2	30.8	30.8	30.8	30.1
	Highland Road	31.2	30.7	30.8	30.7	30.0
	U/S I-95	30.2	29.7	29.7	29.7	28.9

2.5 Review of Report 5

("Update to the 1999 Storm Water Management Plan: Westchester County Airport" by TRC Engineers, 2010.)

1. Purpose of the Report: The report entitled "Update to the 1999 Storm Water Management Plan", was completed by TRC Engineers, Inc. in December 2010. The report provides an updated stormwater management analysis for the Westchester County Airport ("Airport") that analyzes and compares existing conditions and planned development with the conditions that were documented in the Airport 1999 Storm Water Management Plan (1999 SWMP). The report establishes the hydrologic conditions for the Airport as of 2010 to determine the effectiveness of existing stormwater quantity mitigation measures as well as the need for future ones; to determine if the existing Airport stormwater management system is being impacted by upstream properties; to analyze the stormwater impacts of existing and proposed actions at the Airport, and; to present the measures required to mitigate those impacts and reduce peak runoff rates from the Airport.

2. Report 5's Findings and Recommended Improvements: In the 1999 SWMP, the "study area" or limits of study, included the Airport property, approximately 70 acres of offsite properties in New York immediately south of the Airport, and approximately 300 acres of offsite properties immediately north and east of the Airport in the Town of Greenwich, Connecticut. The Airport is situated within two major drainage basins – Rye Lake (RL) and Blind Brook (BB). The pre-1987 Condition model refers to those which existed prior to February of 1987, which was before the improvements to the Airport that were detailed in the 1987 EA/FGEIS for the 1986 Westchester County Airport Master Plan Update. In stormwater management terms, Pre-1987 Conditions are those that existed prior to the diversion of runoff at the Airport from the Rye Lake watershed to the Blind Brook watershed.

   The 1999 SWMP recommended the diversion of runoff from the Rye Lake/Kensico Reservoir watershed to the Blind Brook watershed, as well as providing for water quality treatment and the attenuation of peak rates of runoff associated with the modernization and improvement projects undertaken at the Airport property.

   Existing Conditions (2010) model was established for the Blind Brook and Rye Lake drainage sub areas. This was accomplished by updating the 1999 Full Development Conditions with supplemental information such as land uses, cover types, topography, detention basin storage and outlet data, and known diversions from onsite and offsite drainage systems. Using the updated hydrologic parameters noted above, a hydrologic model of the Existing Conditions (2010) for the Rye Lake and Blind Brook drainage areas was created for the 1-, 2-, 10- and 100-year, 24-hour storms. Not all of the diversion of runoff from the Rye Lake drainage basin to the Blind Brook drainage basin occurred (143 acres of 157 acres planned is diverted in this model).

   Based on the results of the hydrologic modeling of Existing Conditions (2010) summarized above, various options to modify the Airport's stormwater management system were analyzed in the report so that the impacts of the 10- and 100-year storm events could be mitigated under Proposed Conditions (2011) model. The improvements recommended in this scenario to Detention Basins A and B included providing full-depth expansion through excavation at the southeast corner of the basin adjacent to the Perimeter Access Road / reconstruction of the earthen spillway, the embankment slope along the eastern and southern perimeters and the two basin outlet structures. These actions will improve the performance of the existing stormwater management system during the 10- and 100-year storm events, improve downstream hydrologic conditions within the Blind Brook headwaters, and provide additional capacity to undertake future projects at the Airport.

The future Planned Capital Projects (2011) model refers to various modernization and improvement projects proposed by Westchester County at the Airport. In addition to the improvements to the detention basins, the following proposed capital projects are planned: the creation of a permanent baggage screening area in the Main Terminal, a consolidated deicing pad on the west side of the airport, the reconstruction of the South Airport Rescue Fire Fighting (ARFF) Road, and the redevelopment of the former Air National Guard (ANG) site. Incorporating these future projects would result in a proposed increase of approximately 4.7 acres of impervious surface area within the Airport. All of these projects would, be located in the Blind Brook watershed. Thus, associated stormwater runoff would drain to the Blind Brook. Furthermore, an evaluation was undertaken to determine if the proposed detention basin improvements would also mitigate the increased stormwater runoff that would be generated by these projects. A hydrologic model of future conditions with the additional proposed projects for the Blind Brook drainage area was created for the 1-, 2-, 10-, and 100-year, 24-hour storms.

3. Hydrologic Modeling Methods: The *Update to the 1999 Storm Water Management Plan* is based on the development of hydrologic models that compare “Existing Conditions (2010)” to the “Pre-1987” and “1999 Full Development” model as documented in the 1999 Storm Water Management Plan prepared by Dvirka and Bartilucci (“1999 SWMP”). The Update to the 1999 Storm Water Management Plan has been designed in accordance with the methodology and criteria found in the following publications:
   - "Urban Hydrology for Small Watersheds" (Technical Release No. 55), published by the United States Department of Agriculture (USDA), Natural Resources Conservation Service (NRCS) (formerly Soil Conservation Service, SCS), dated June 1986.
   - “Computer Program for Project Formulation Hydrology” (Technical Release No. 20), published by the United States Department of Agriculture (USDA), Natural Resources Conservation Service (NRCS) (formerly Soil Conservation Service, SCS), May 1965, revised 1983.
   - New York State Stormwater Management Design Manual, last revised August 2010.

   Table 8 in *Update to the 1999 Storm Water Management Plan: Westchester County Airport* listed the tributary sub areas under Existing Conditions (2010), with the acreage on the Airport, off the Airport in New York, off the Airport in Connecticut, and total acreage for each sub area. The total drainage area to the Blind Brook Watershed is 844.73 acres (1.32 mi^2).

Table 8 - Summary of Drainage Areas in TRC 2010 Report

Sub Area	On Airport (Ac.)	Off Airport In New York (Ac.)	Off Airport In Connecticut (Ac.)	Total Area (Ac.)
		Blind Brook		
BB-1AF	7.45			7.45
BB-1BF	17.69		51.17	68.86
BB-1CF			39.15	39.15
BB-1DF	0.17		6.91	7.08
BB-1EF			8.14	8.14
BB-1FF			19.50	19.50
BB-1GF	8.11		1.83	9.94
BB-2F	10.11	43.01		53.12
BB-3F	33.54			33.54
BB-4F	25.66		2.53	28.19
BB-5F	3.38		33.67	37.05
BB-6F	4.78			4.78
BB-7FA	98.17			98.17
BB-7FB	89.91			89.91
BB-7FC	8.17	5.66		13.83
BB-8FA	158.08			158.08
BB-8FB	18.92			18.92
BB-8FC	9.20			9.20
BB-8FD	15.26			15.26
BB-9F	3.16	0.71		3.87
BS-A			15.86	15.86
BS-B1 bypass			10.90	10.90
BS-B1 det			4.99	4.99
BS-B2			13.58	13.58
BS-B3			11.23	11.23
BS-C	4.74		13.93	18.67
BS-D det			4.66	4.66
BS-D chan			18.00	18.00
BS-D pond			21.42	21.42
BS-D bypass			1.38	1.38
Sub Total 1	516.50	49.38	278.85	844.73

The proposed improvements to Detention Basins A and B would enable the Airport to better manage stormwater from their facility under Existing Conditions (2010), reduce peak runoff rates downstream, mitigate stormwater impacts of existing and proposed actions at the Airport as well as impacts from existing actions in offsite areas within the study area, and provide the additional storage capacity needed to undertake the planned Airport capital projects.

4. Final Results of the Report:

Table 9 in the report shows the peak discharges computed for the area draining to Blind Brook Watershed.

TABLE 9
UPDATE TO THE 1999 STORM WATER MANAGEMENT PLAN
WESTCHESTER COUNTY AIRPORT
COMPARISON OF PEAK DISCHARGE RATES (cfs)

Location of Confluence	1-Year Storm				2-Year Storm				10-Year Storm				100-Year Storm			
	Pre-1987	Exist. (2010) Cond	Prop. (2011) Cond	Future (2011) Cond w/Capital Projects	Pre-1987	Exist. (2010) Cond	Prop. (2011) Cond	Future (2011) Cond w/Capital Projects	Pre-1987	Exist. (2010) Cond	Prop. (2011) Cond	Future (2011) Cond w/Capital Projects	Pre-1987	Exist. (2010) Cond	Prop. (2011) Cond	Future (2011) Cond w/Capital Projects
CON 1 - Rye Lake	216.3	**116.3**	**116.3**	**116.3**	330.2	**179.7**	**179.7**	**179.7**	594.8	**372.3**	**372.3**	**372.3**	1062.2	**763.9**	**763.9**	**763.9**
CON 3 - On Blind Brook @ Basin A discharge	256.1	**108.4**	**103.8**	**105.8**	399.2	**167.7**	**163.1**	**165.6**	626.1	**344.2**	**281.8**	**286.8**	994.4	**683.8**	**495.6**	**504.5**
CON 3A - On Blind Brook @ Basin B discharge	N/A	**151.8**	**141.1**	**143.8**	N/A	**279.8**	**260.2**	**264.0**	N/A	**741.0**	**531.9**	**536.2**	N/A	**1286.7**	**1016.6**	**1026.0**
L1 - West Branch of Blind Brook @ Lincoln Ave.	266.4	**152.8**	**142.1**	**144.7**	417.2	**281.4**	**261.8**	**265.6**	672.7	**743.5**	**535.0**	**539.6**	1095.5	**1292.5**	**1023.2**	**1032.5**
L2 - On Lincoln Avenue 470 feet east of L1	21.6	**25.2**	**25.2**	**25.2**	35.2	**38.7**	**38.7**	**38.7**	68.2	**70.2**	**70.2**	**70.2**	128.5	**125.5**	**125.5**	**125.5**
L3 - On Lincoln Avenue 1000 feet east of L1	5.1	**3.4**	**3.4**	**3.9**	8.3	**5.4**	**5.4**	**6.0**	16.0	**10.3**	**10.3**	**11.1**	30.3	**19.3**	**19.3**	**20.1**
CON 5 - Intersection of East Branch Blind Brook & Lincoln Avenue	41.8	**66.5**	**66.5**	**67.3**	69.6	**106.2**	**106.2**	**107.1**	138.0	**194.5**	**194.5**	**195.7**	264.6	**378.8**	**378.8**	**380.4**
CON 6 - Intersection of East and West Branches of Blind Brook	321.3	**205.6**	**199.9**	**202.6**	517.0	**359.4**	**341.5**	**346.1**	853.7	**950.5**	**724.6**	**722.7**	1451.7	**1595.1**	**1318.8**	**1330.7**

2.6 Review of Report 6

("PepsiCo Project Renew Amended Phase I: Supplemental Stormwater Pollution Prevension Plan", John Meyer Consultanting, PC, Feburary 2013)

1. Purpose of the Report: The Stormwater Pollution Prevention Plan (hereinafter referred to as the "Phase I SWPPP") has been prepared for the 152 acre PepsiCo World Headquarters site, located in the Purchase area of the Town of Harrison, New York. PepsiCo's property currently consists of seven (7) interconnected, three (3) story office buildings surrounding a series of landscaped courtyards in the center of the Property. There is significant open space on the property, highlighted by the publically accessible Donald M. Kendall Sculpture Gardens and the existing "P" pond. The proposed improvements represent the first major renovations to the entire campus since it was constructed in the late 1960s. The stormwater improvements have been designed in accordance with the requirements of the New York State Department of Environmental Conservation (NYSDEC) SPDES General Permit GP-10-001.

   The purpose of this report is to examine and mitigate impacts of the proposed amended PepsiCo-Project Renew development and associated site improvements on the local watershed. This study includes an analysis of existing drainage conditions within the analysis area and describes proposed drainage conditions after development of the project. It also includes temporary improvements to be used throughout construction to minimize erosion and sediment transport.

2. Report 6's Findings and Recommended Improvements: The site is approximately 152 acres and is bound by Anderson Hill Road to the north, Lincoln Avenue to the west, the Blind Brook to the east and existing residences to the south. Based on the Westchester County Soil Survey, all on-site soils are moderately well drained and belong to hydrological group "B" or "C". The soil types, boundaries and drainage areas/designations are depicted on the Drainage Areas Maps DA-1 and DA-2 in Appendix H. Four separate Design Points (1 through 4) were identified for comparing peak rates of runoff in existing and proposed conditions. Similarly, four separate drainage areas were identified in existing conditions based on the existing drainage divides at the site. The numbers included in the name of each drainage area correspond to the Design Point they drain towards.

   The proposed Phase I improvements consist of the following:
   - Renovations of the seven (7) existing office buildings, totaling approximately 450,000 square feet.
   - Construction of fire access roads to provide access to the exterior of the buildings for emergency equipment.
   - A northern and southern expansion of the existing parking facilities in the northeast portions of the property, including an infiltration basin and new drainage pipes to convey a portion of these existing lots to the new stormwater infiltration basin.
   - Minor interior modifications to portions of the existing parking areas.
   - Relocation of the existing material storage area to the rear of the former nursery facility.

- Upgrades to the existing westernmost curb-cut on the nursery property to provide a construction access road / future maintenance drive and to the new material storage area. The upgraded driveway will require the demolition of several existing structures, and will connect to the main parking area for construction access and future maintenance purposes.
- Landbanked parking, consisting of an expansion of the existing parking area located adjacent to the existing P-pond.
- Wetland mitigation in the northeast corner of the site, between the former nursery site and campus parking and in the vicinity of the piped outlet from the existing P-Pond.

The proposed drainage improvements include a variety of stormwater practices, such as vegetated swales, stormwater management ponds with forebays and biofilters. After treatment for water quality and peak rate attenuation, stormwater discharges from the ponds will utilize low velocity level-spreaders which will drain to the existing wetlands, existing wetland buffers or proposed wetland mitigation areas. The vegetated practices and overland discharges provide multiple opportunities for water quality enhancement and infiltration in addition to the proposed stormwater management basins.

3. Hydrologic Modeling Methods: Runoff rates were calculated based upon the standards set forth by the United States Department of Agriculture Natural Resources Conservation Service Technical Release 55, Urban Hydrology for Small Watersheds (TR-55), dated June 1986. The methodology set forth in TR-55 considers a multitude of characteristics for watershed areas including soil types, soil permeability, vegetative cover, time of concentration, topography, rainfall intensity, ponding areas, etc. The 1-, 2-, 10-, 25- and 100-year storm recurrence intervals were reviewed in the design of the stormwater management facilities (in Appendices A & B of Hydrologic Calculations). Anticipated drainage conditions were analyzed taking into account the rate of runoff which will result from the construction of parking areas and other impervious surfaces associated with the site development.

4. Final Results of the Report:

   The final results of Report 6 showed the reductions in peak rates of runoff from proposed to existing conditions at Design Point #4 are shown in the table below.

Table 10 - Percent Reductions in Peak Rates of Runoff at Design Point (DP-4)

Storm Recurrence Interval	Existing Peak Runoff Rate (cfs)	Proposed Peak Runoff Rate (cfs)	Percent Reduction (%)
1-year	7.79	5.60	28.1
2-year	14.91	11.68	21.7
10-year	33.71	28.14	16.5
25-year	47.73	41.97	12.1
100-year	70.50	69.85	0.92

2.7 Conclusions Based on Review

Based on the review of the six existing reports, and for the purpose of studying the optimized gate operations and detention analysis, the following conclusions were provided. This chapter also includes an explanation of recommended improvements, effects on the flooding elevations in Blind Brook and potential technical issues for each of the reports.

1. In general, the results of previous reports agree with each other. The same magnitudes of the hydrologic and hydraulic results were achieved among the previous studies report.
2. Report 2, “Project Report – Flood Mitigation Study – Bowman Avenue Dam Site”, completed by Chas. H. Sells, Inc. in March 2008, studied the ability of the automated sluice gate to vary the size of the outlet opening, thus providing the optimum orifice size for the flow rate in the stream. It provided some insights in developing the optimal gate operations rules for various storm events studied.
3. Report 3, “Project Report – Flood Mitigation Study – Lower Pond Supplemental”, completed by Chas. H. Sells, Inc. in March 2008, developed the potential water surface elevation reduction by increasing the Lower Pond volume in conjunction with optimizing the openings of the automatic sluice gate. As it should be noted in Table 4 on Page 9, there is 4.44 ft reduction of 50-year water surface elevation at upstream of I-95, due to the change of the overtopping regime of I-95, the regime changes from energy flow to pressure and/or weir flow, The reason for this abrupt changes in water surface elevation is because of bridge modeling approach in HEC-RAS model was not properly set.
4. Report 4, “Hydrologic Analysis in Study for Resizing the Upper Pond Reservoir”, completed by Paul C. Rizzo Engineering – New York, PLLC, (RIZZO study) in September 2009, used the most up-to-date approaches for hydrologic and hydraulic modeling methods, including distributed parameter based subdivided watershed and unsteady flow modeling. The RIZZO models were all set properly, and the results are reasonable. The model was utilized as our base model for studying optimal gate operation rules and detention study. However, the soil data in RIZZO model was based on USGS 2006 STATSGO Hydrologic Soil data (Figure C-3), which is not the latest data. In the present study, SSURGO soil data from USGS web soil survey will be used.
5. Report 5, “Update to the 1999 Storm Water Management Plan: Westchester County Airport” completed by TRC Engineers, 2010, can be used to further accurately model the Upper Blind Brook watershed. By incorporating this model (drainage area = 1.32 square miles) into RIZZO’s hydrologic model, and with the detailed hydrologic components set up, it will be very convenient to evaluate the potential benefits by increasing the capacity of existing stormwater management ponds on the airport properties. However, Pond 78P in this model was not properly set. The broad-crested weir of this pond was set at elevation 348.60’, which is higher than the maximum storage volume at elevation 348.0’.
6. Report 6, “PepsiCo Project Renew Amended Phase I: Supplemental Stormwater Pollution Prevention Plan” completed by John Meyer Consulting, PC, 2012, only computation of SWM to Design Point 4 was provided. The drainage area to this study point is only 26.9 acres, or 0.04 square miles. As compared with the 0.33 square miles drainage areas of PepsiCo, this drainage area is minor. For this reason Report 6 will not be used in building the hydrologic models.

3. Hydrologic and Hydraulic Analysis of Additional Detention Areas

This part of report utilized the hydrologic model provided in the report entitled “Update to the 1999 Storm Water Management Plan”, by TRC Engineers, Inc. in December 2010 (TRC report), in conjunction with the hydrologic and hydraulic models provided in the report entitled “Hydrologic and Hydraulic Analysis: Bowman Avenue Dam Project, Study for Resizing the Upper Pond Reservoir”, by Paul C. RIZZO Engineering in September 2012 (RIZZO report) to analyze the proposed impacts of additional detention facilities on Blind Brook Watershed.

Based on the models mentioned above, a set of new subwatershed hydrologic models of the 5.38 square miles of the Upper Blind Brook Watershed were created based on ten potential detention areas. The model used in this analysis further divided the subwatershed to better define the overall watershed. There are seven subwatershed areas, SW1-Airport, SW1-SUNY, SW1-PepsiCo, SW2-U/S, SW2-Edgar Bronfman Lake, SW2-D/S and SW2-Hutchinson River Parkway defined to represent the contributing area for each potential detention basin. The model also includes TRC report with detailed subdivided airport drainage areas, and the onsite Stormwater Management (SWM) components such as ponds and reservoirs. The models were first run for the Existing Conditions (2010) as mentioned in TRC report, and the results were found to be in agreement with those results from the hydrologic models in RIZZO report with two subwatersheds approach. The models were then run for the 2011 Future Planned Capital Projects (Future Condition 2011 in Report 5 by TRC) with the airport SWM improvements, to obtain the full flood hydrographs of various storm events as the PB existing condition. New unsteady HEC-RAS plans were created, with the existing condition hydrographs from the new hydrologic model as the upstream boundary conditions. Maximum water surface elevations were computed at five downstream locations along the Blind Brook, namely, downstream of I-287, Purchase Street, Highland Road, Mendota Avenue and upstream of I-95 for existing condition scenario.

For the detention analysis, by using GIS mapping and the information obtained from a field visit to the Blind Brook, PB selected 10 potential detention areas to study the detention effect on the flood peak discharges in five subwatershed areas as the followings, SW1-Airport (2 detentions), SW1-SUNY (2 detentions), SW1-PepsiCo (1 detention), SW2-Hutchinson River Parkway (3 detentions) and SW2 (2 detentions). The addition of identified detention areas along the Blind Brook were evaluated both individually and collectively by the most effective detention areas to provide a sense of incremental benefits of implementation over time. The water surface elevation differences between PB existing conditions vs. five proposed conditions were compared and analyzed.

3.1 Hydrologic Analysis Based on New Watershed Subdivision

PB first obtained the hydrologic models in HydroCAD Version 10.1 from TRC Engineers in December 2013. There are four scenarios of the models, Pre-1987 Condition, Existing Condition 2010, Proposed Condition 2011 and Future Condition 2011. For each scenario, there are two model input files, one for 1- and 2-year events, other one for 10- and 100-year events. The drainage areas for each scenario are summarized and listed in the table below. As it can be seen from this table, the drainage areas to Blind Brook Watershed for Existing Condition 2010, Proposed Condition 2011 and Future Condition 2011 are all very close to 1.32 square miles. The overall runoff curve numbers for those three scenarios are all close to 82.

Table 11 - Drainage Areas for Four Scenarios in TRC Report (2010)

	Rye Lake DA (acres/mi^2)	Blind Brook DA (acres/mi^2)	Total (acres/mi^2)
Pre-1987 Condition	337.28 / 0.53	721.92 / 1.13	1,059.2 / 1.66
Existing Condition 2010	193.03 / 0.30	845.09 / 1.32	1,038.12 / 1.62
Proposed Condition 2011	193.03 / 0.30	845.09 / 1.32	1,038.12 / 1.62
Future Condition 2011	193.03 / 0.30	846.36 / 1.32	1,039.39 / 1.62

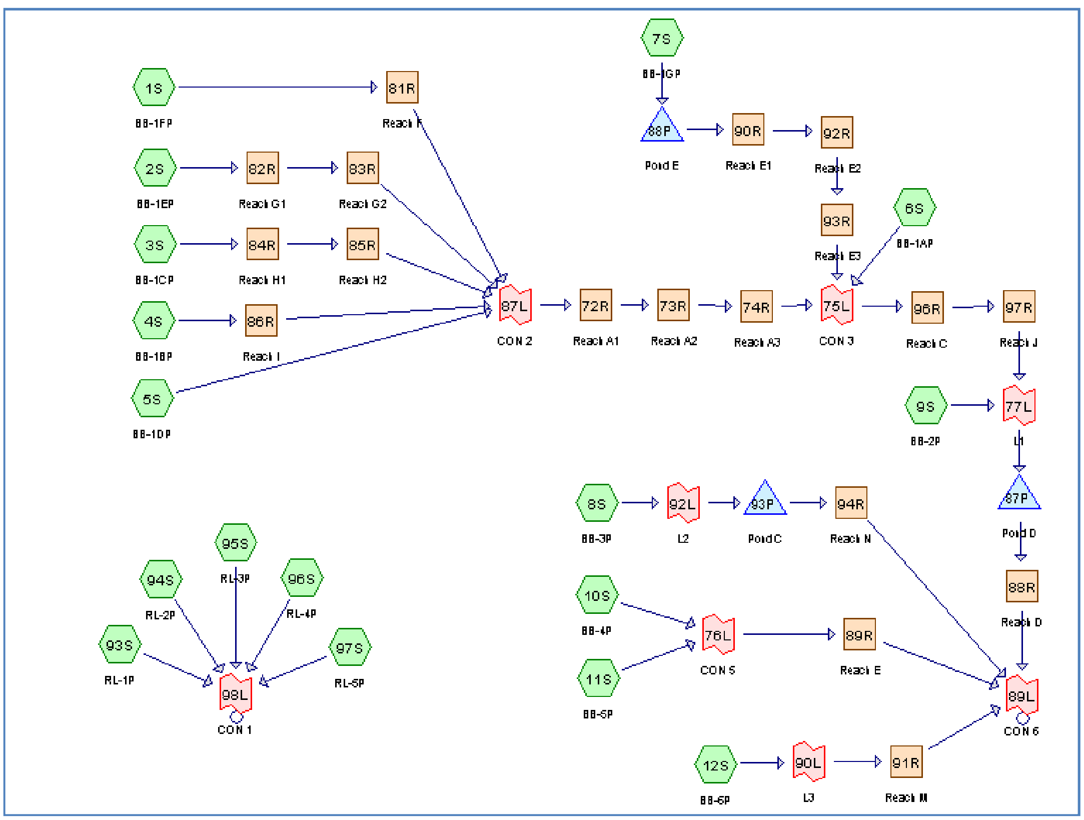


Figure 12 - HydroCAD Model for Pre-1987 Condition from TRC Report (2010)

PB also obtained the hydrologic models of the Blind Brook Watershed used in the RIZZO report, which were modeled using Hydrologic Modeling System software HEC-HMS (version 3.4) developed by US Army Corps of Engineers (USACE). There are six sub-watersheds in the model according to topographic and hydrologic conditions. In the upper reach of the brook, the watershed was subdivided into two sub-watersheds, 2.1 mi^2 (SW1) and 3.28 mi^2 (SW2). The 1.32 mi^2 drainage area of the TRC model was fully contained in the upstream part of sub-watershed SW1 in RIZZO's hydrologic model. Figure 13 below show the RIZZO hydrologic model in HEC-HMS and the drainage area maps.

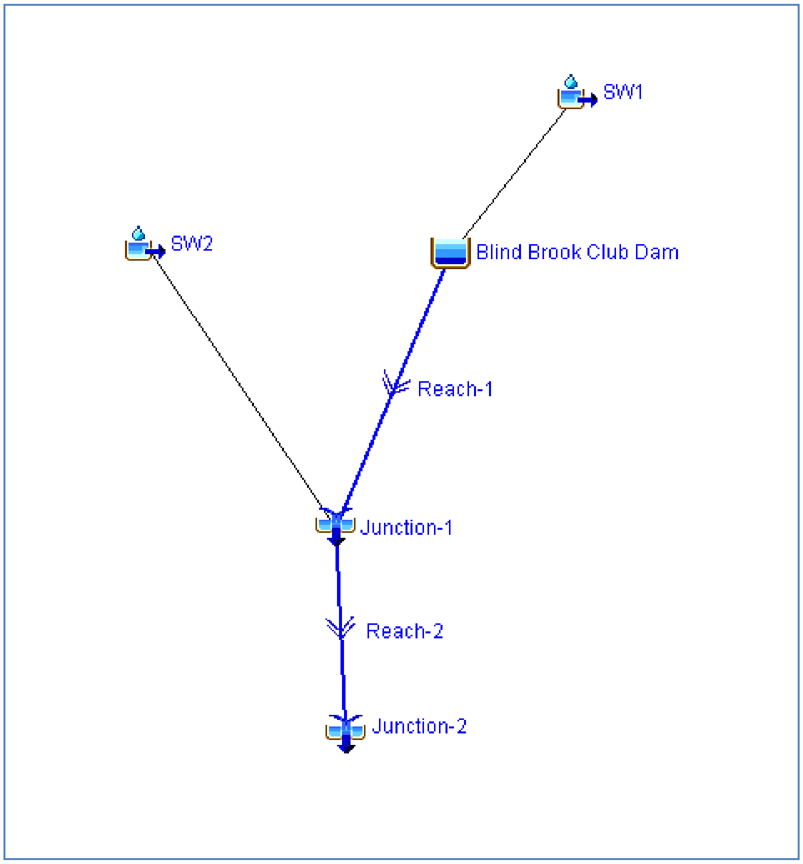


Figure 13 - HEC-HMS Model Schematic for SW1 and SW2 from RIZZO Report (2012)

Based on the models mentioned above, PB created a set of new hydrologic models in HydroCAD for the Upper Blind Brook Watershed which includes drainage area SW1 and SW2 (Figure 14). These models contain detailed subdivided drainage areas considering 10 potential detention areas, and onsite airport Stormwater Management (SWM) components such as ponds and reservoirs. Based on the contour information from USGS topography, NLCD_2006 land use data and SSURGO soil mentioned in Chapter 1. The details of drainage areas, curve numbers and time of concentration were also computed in WinTR-55 (Table 6). Blind Brook Club Reservoir and two reach routing components in RIZZO's HEC-HMS model were also successfully replicated in the new hydrologic model in HydroCAD. The model set contains six sub-models, as shown in the Figure 14 below.

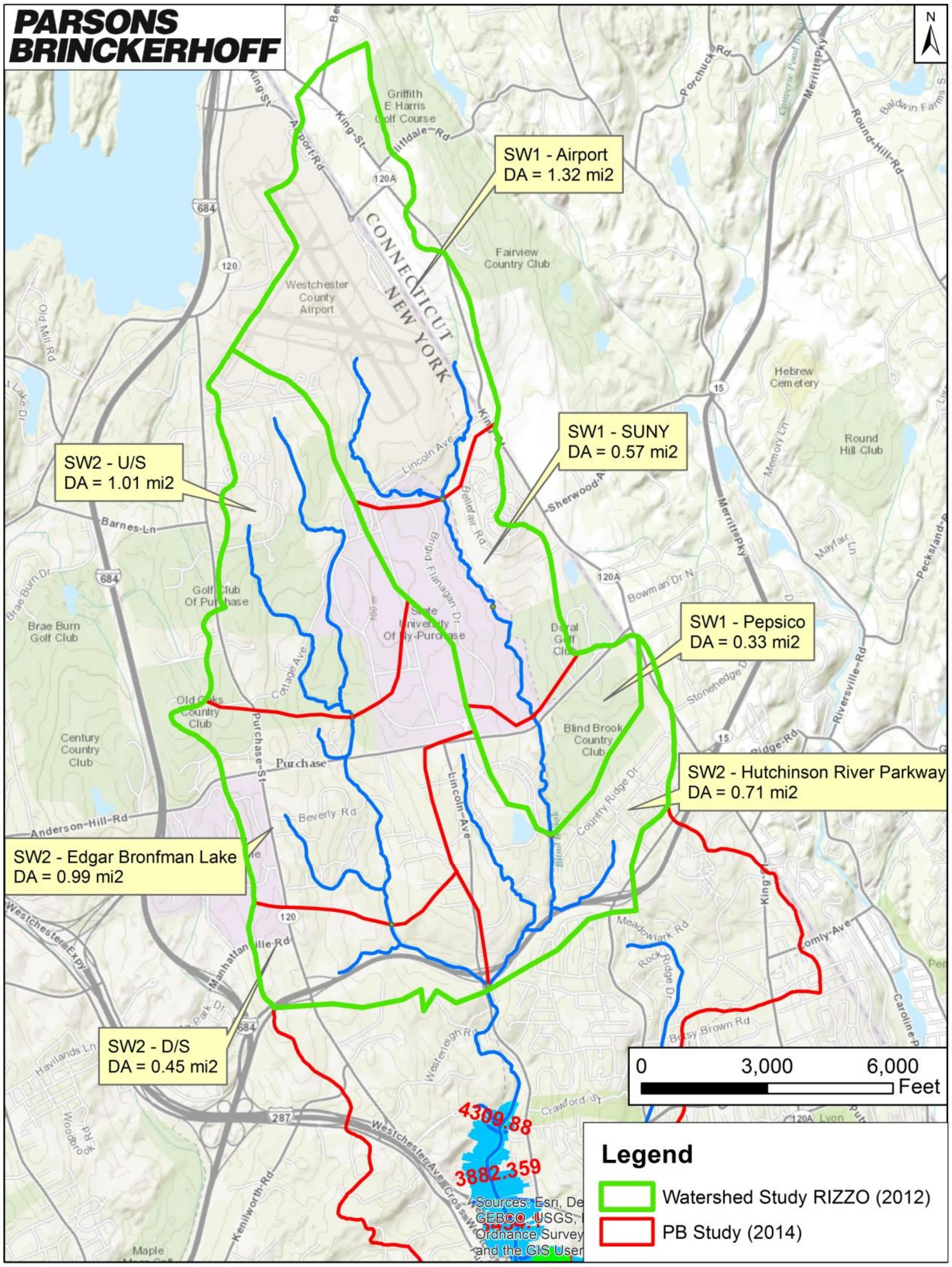


Figure 14 - Subdivided Watersheds of SW1 and SW2 in PB Study

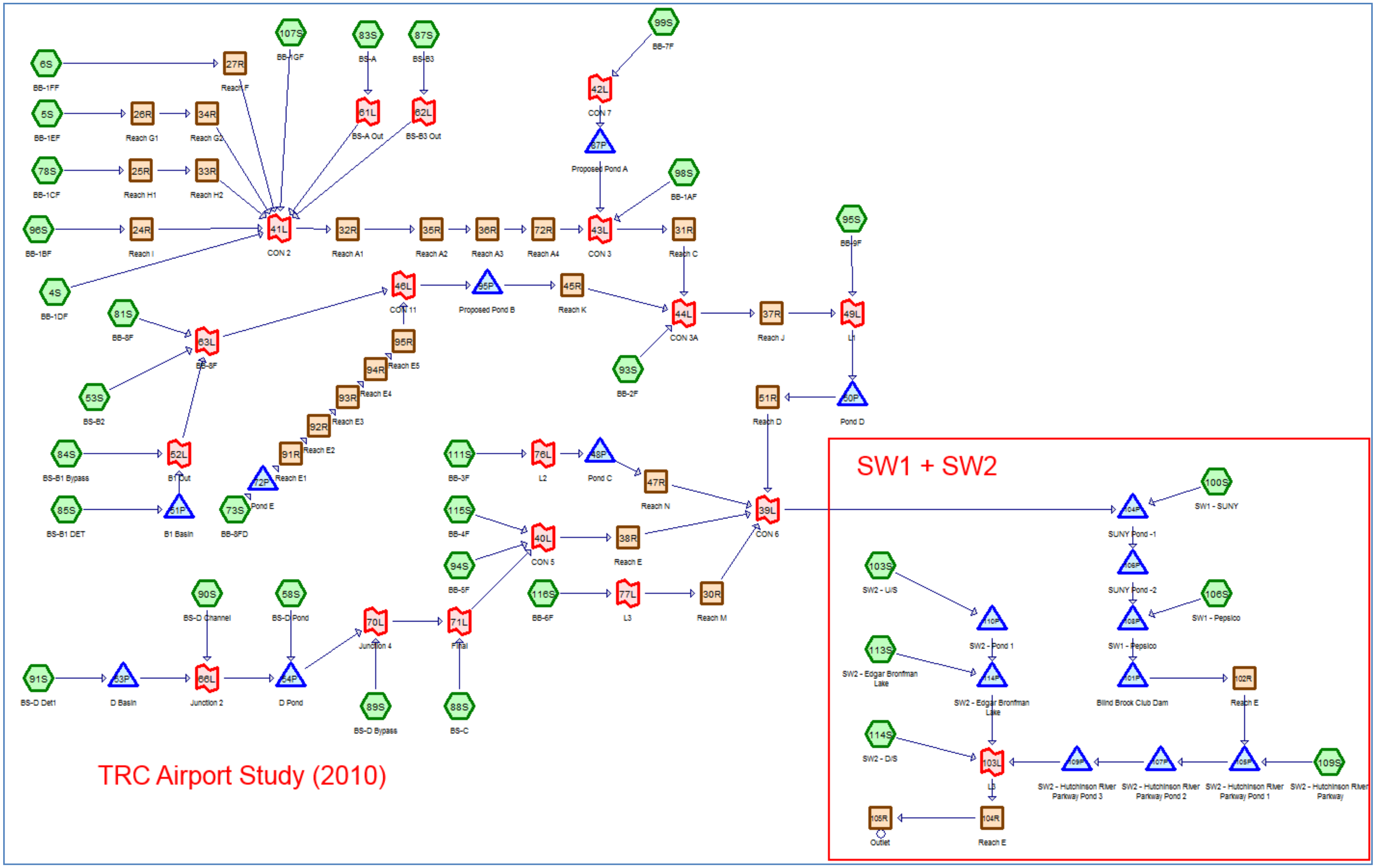


Figure 15 - HydroCAD Schematic for SW1 and SW2 with detailed Airport Study

Table 12 - Detailed Sub-watershed Information for SW1 and SW2

Sub-watershed	Drainage Area (mi^2)	Runoff Curve Number	T_c (min)
SW1-Airport	1.32	-	-
SW1-SUNY	0.57	79	100
SW1-PepsiCo	0.33	75	90
SW2-U/S	1.01	70	150
SW2- Edgar Bronfman Lake	0.99	74	110
SW2- D/S	0.45	70	110
SW2- Hutchinson River Parkway	0.71	73	130

These models were first run for the Existing Conditions (2010) as desired in TRC report, and the results were found to be in agreement with those from hydrologic models in RIZZO report. The comparison of the model results are listed in the table below. The main reason for differences of the peak discharges between the two models is due to different watershed subdivision and various timings of peak discharges from each subwatershed. The use of Clark's unit hydrograph in the RIZZO model also contributes to the peak discharges difference computed. Since Clark's unit hydrograph is currently not available in HydroCAD model, the SCS unit hydrograph was used instead.

Table 13 - Comparison of Peak Discharges from the New Hydrologic Model and RIZZO Model for Existing Condition (2010)

Peak Discharges	Existing 2010 RIZZO Study - Two Subwatersheds (SW1 and SW2) (cfs)	Existing 2010 PB Study - TRC Airport DA, Six Subwatersheds (cfs)	% Difference Existing RIZZO (2010) vs. Existing PB (2010)
2-Year Storm	1,054	992	-5.88%
5-Year Storm	1,603	1,559	-2.74%
10-Year Storm	2,073	2,180	5.16%
25-Year Storm	2,540	2,709	6.65%
50-Year Storm	3,024	3,233	6.91%
100-Year Storm	3,561	3,824	7.39%

The models were then run for Future Planned Capital Projects (Future Condition 2011) with the airport SWM improvements, to obtain the reduced peak hydrographs. Based on the hydrologic models mentioned above, PB created a set of new unsteady HEC-RAS models, with the full hydrographs obtained above as the upstream boundary condition. Maximum water surface elevations were computed at five downstream locations along the Blind Brook, namely, downstream of I-287, Purchase Street, Highland Road, Mendota Avenue and upstream of I-95 for existing condition. The locations for water surface elevation comparisons are shown in the figure below.

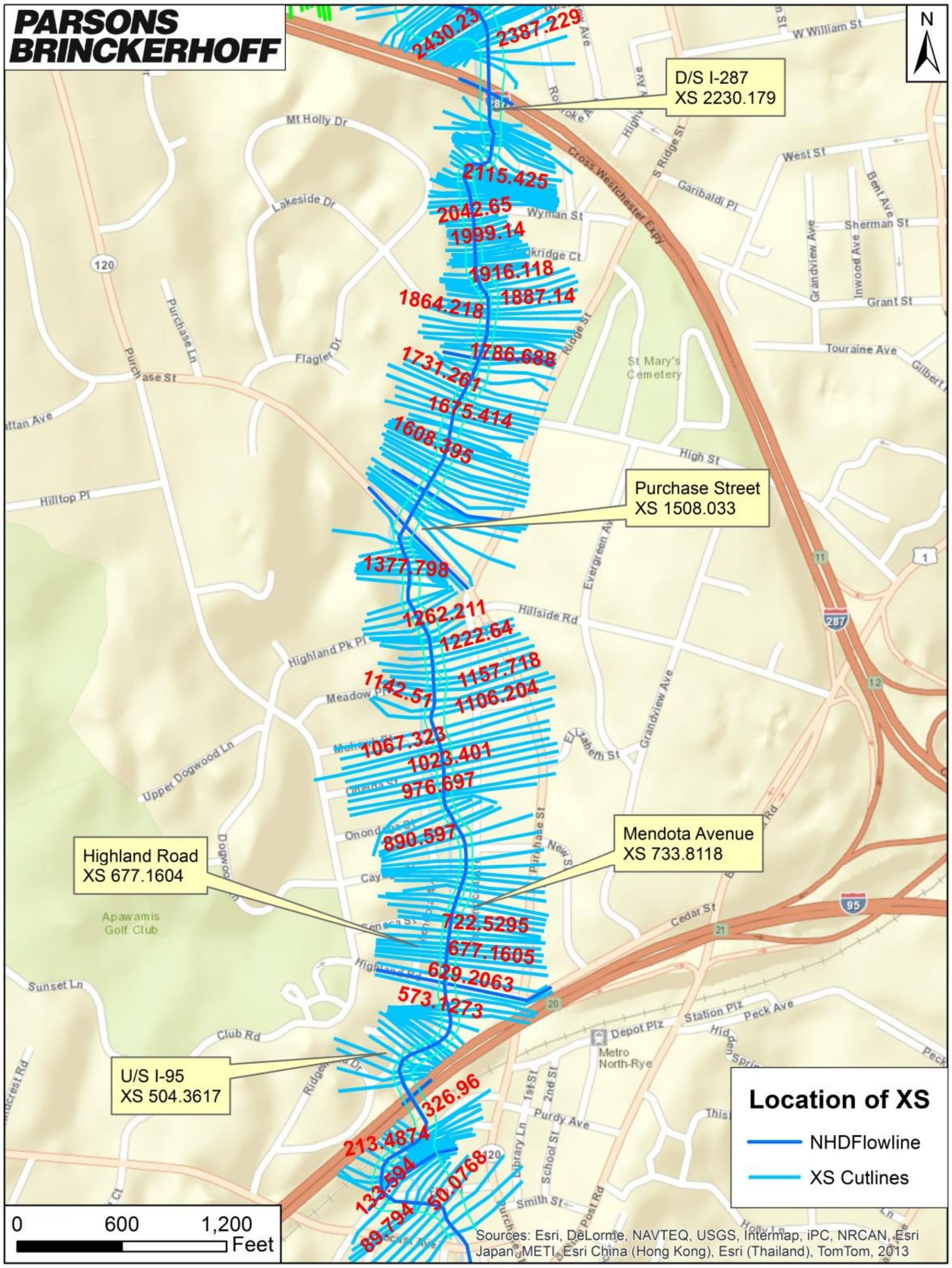


Figure 16 - Locations for Comparing the Water Surface Elevations in HEC-RAS

3.2 Description of Additional Detentions

The studies performed provide an understanding of how the existing Blind Brook stream system functions. The alternatives recommended in the Report 2 by Chas. H Sells, 2008 focused on the Bowman Avenue Dam and potential detention volume on the upstream side of the dam. Costs associated with the improvements were also provided in RIZZO's report, and as noted in the meeting with City of Rye, on such improvements. In this study, the next step was to look for detention areas further up in the watershed, where more land is available. The caveat being, stay on State or County land simply due to land acquisition costs and EIS permitting fees, and minimizing the cost of construction with the exception of possible opportunities on the PepsiCo property. The addition of identified detention areas along the Blind Brook can then be evaluated both individually and collectively to provide a sense of incremental benefits of implementation over time.

Using GIS mapping from Pictometry Online Version 1.10.2 and having performed a field visit to the Blind Brook, we have provided examples on aerial photos showing potential detention areas worth investigation. The five sites presented were to determine the feasibility of these sites to provide flood control and the next steps to a conceptual look at the improvement they can provide. The concept level input relied on readily available GIS data and engineering judgment to establish storage volume verse elevation.

In the aerial images below (Figure 17), five detention regions were identified with various numbers of detention ponds as the followings:

- Westchester County Airport: 2 detention ponds (expanding existing SWM ponds)
- SUNY Purchase: 2 detention areas with low stabilized earth berms
- PepsiCo: 1 detention area with low stabilized earth berm
- Hutchinson River Parkway Right-Of-Way: 3 detention areas with low stabilized earth berms
- Sub-watershed SW2: 2 detention ponds (expanding existing reservoirs)

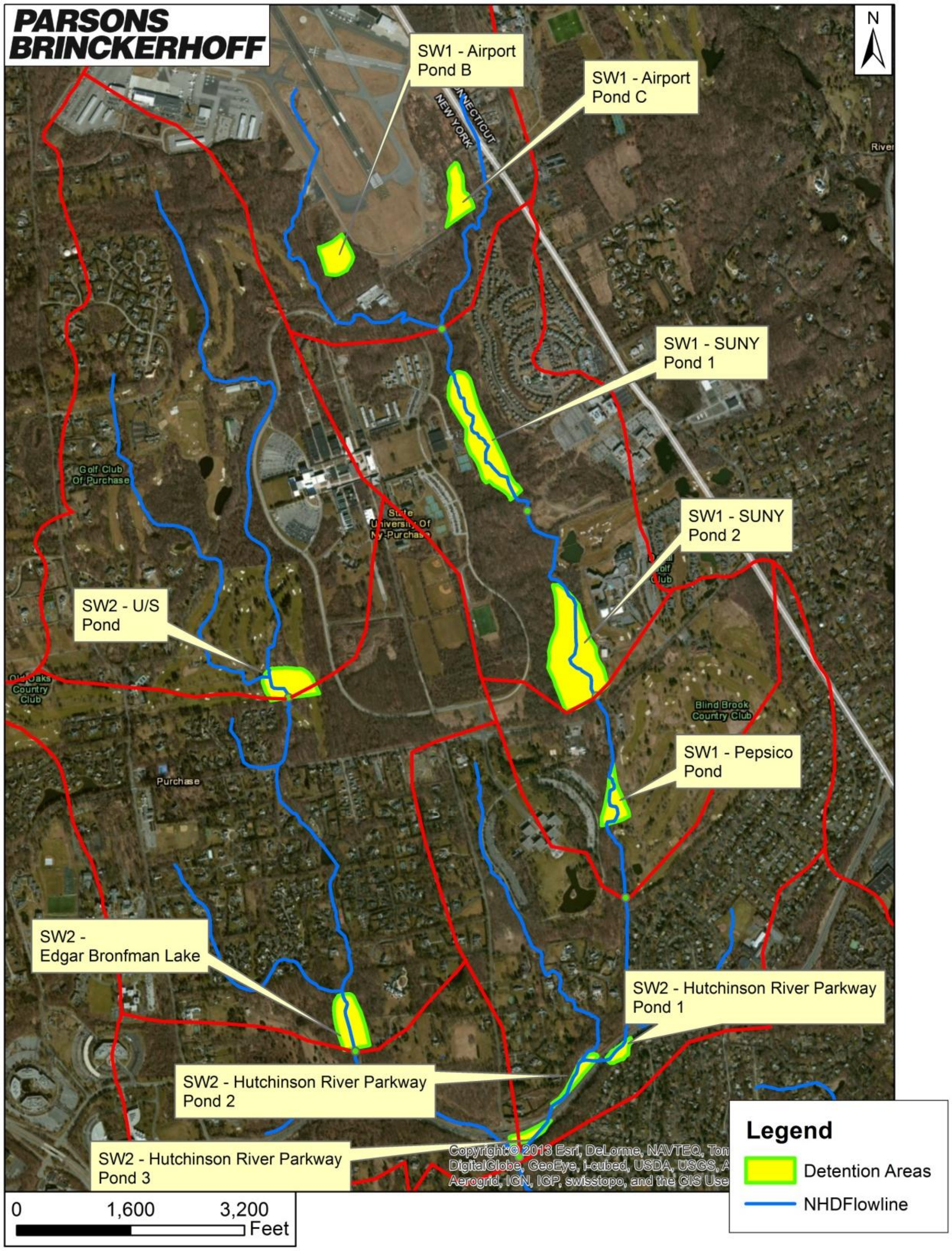
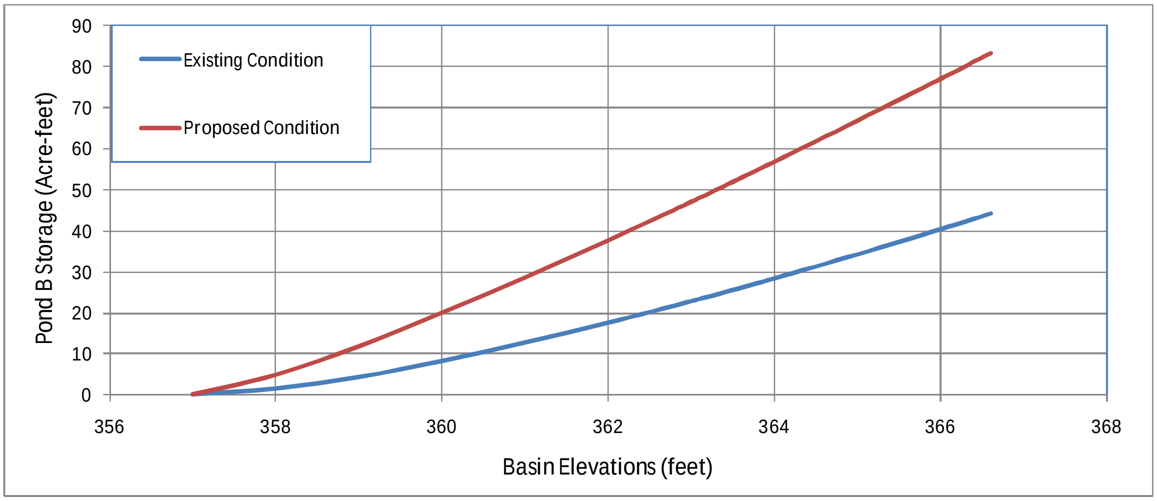


Figure 17 - Locations for 10 Potential Detentions in Upper Blind Brook Watershed

3.2.1 Westchester County Airport – The Blind Brook starts in the vicinity of the Westchester Airport as two channels that converge at a confluence located just south of

Lincoln Avenue. From the TRC Report, two detention ponds, Pond B and Pond C currently exist, one east and one west of the runway. The additional detention would build upon existing ponds and could potentially increase the detention volume at the airport over what exists today. The location of the additional detention is shown in Appendix D, Figure D-1. The slide slope of proposed pond is assumed to be 3:1. The elevation vs. storage relationship for existing and proposed condition for Pond B and Pond C are shown below.



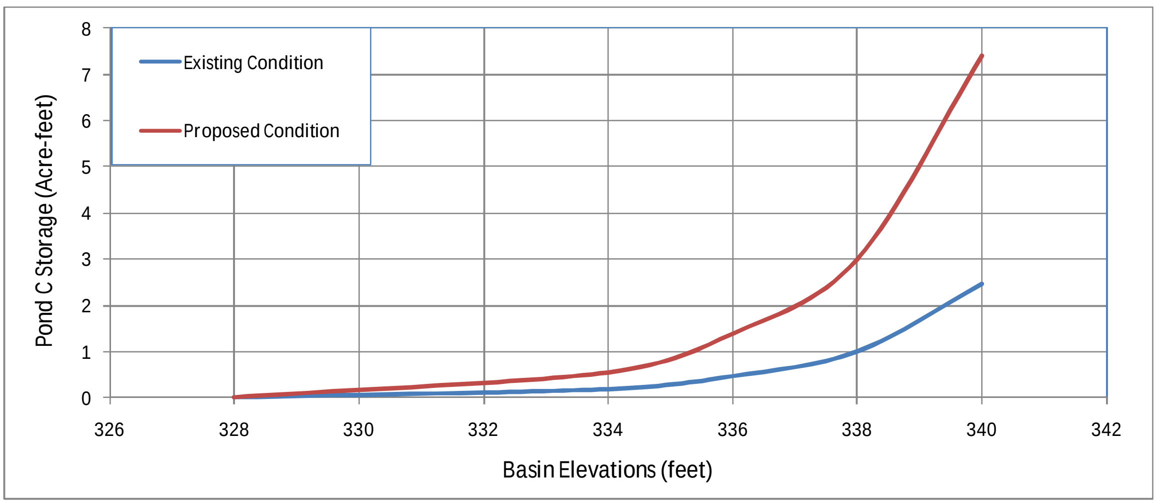


Figure 18 - Existing vs. Proposed Condition Elevation-Storage Relations for Pond B and C on Westchester County Airport

3.2.2 SUNY Purchase – The Blind Brook runs along the east side of the property. The proposed design shows two potential detention areas. The idea would be to build three stabilized earthen berms across the floodplain with openings at the channel. This would limit the construction disturbance to the footprint of a berm and back water up, similar to a bridge opening constricting flow and flooding the existing woods upstream. The spill through could be grouted riprap, with a riprap lined plunge pool on the downstream side. The table below shows the detailed information about the weir height and the maximum inundation area behind the berm. A hand sketch was provided below showing a plan, profile and a couple of sections.

Table 14 - Detailed Information about Pond 1 and 2 on SUNY Property

Ponds	Weir Height (ft)	Weir Length (ft)	Maximum Inundation Area (acre)	Maximum Pond Storage (acre-ft)
SUNY Pond 1	8	280	15.50	53.18
SUNY Pond 2	13	820	15.07	65.62

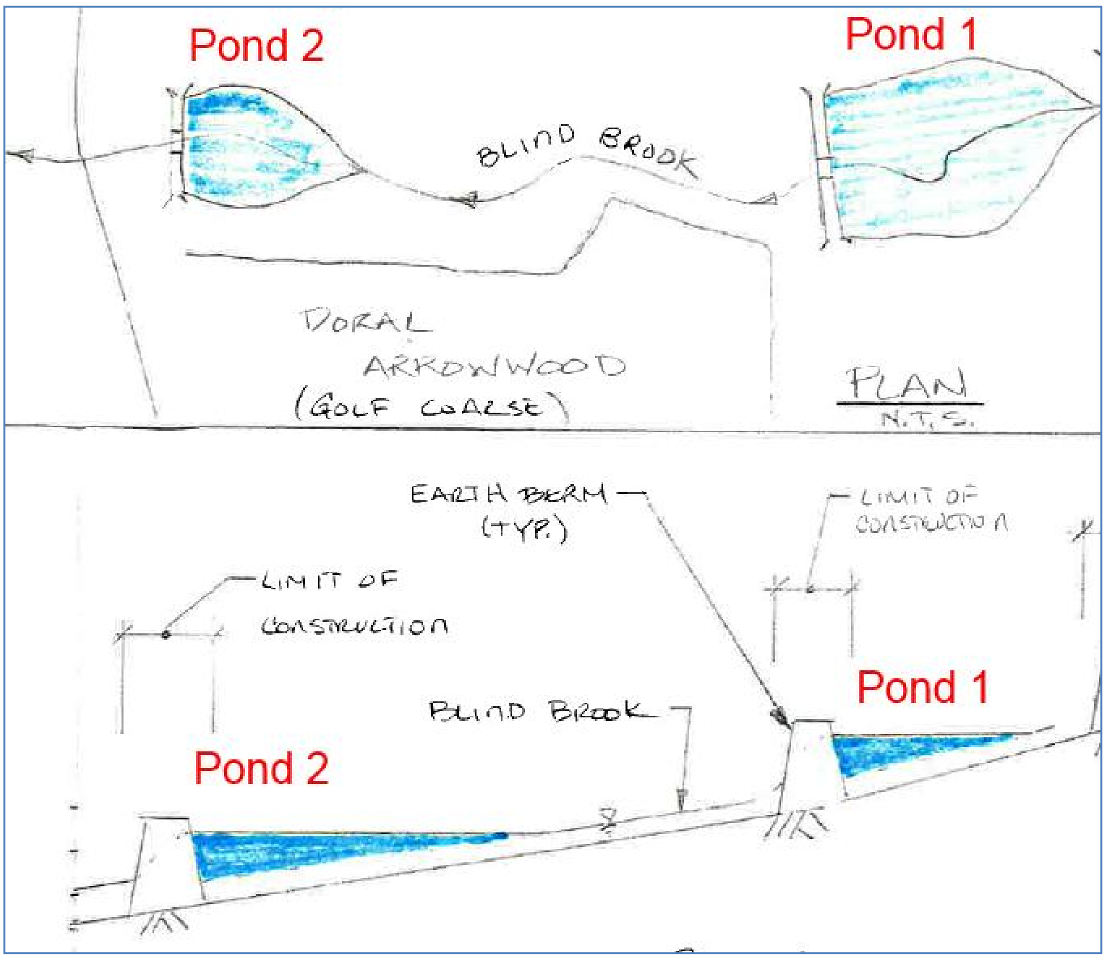


Figure 19 - Sketch of SUNY Detention Pond 1 and Pond 2

3.2.3 PepsiCo – There is an area along the east side of the PepsiCo property where a low stabilized earthen berm could provide additional detention volume. PepsiCo manages most all of their stormwater utilizing the existing pond on their property to manage stormwater. Detention along the brook would be independent of their system.

Table 15 - Detailed Information about Pond on PepsiCo Property

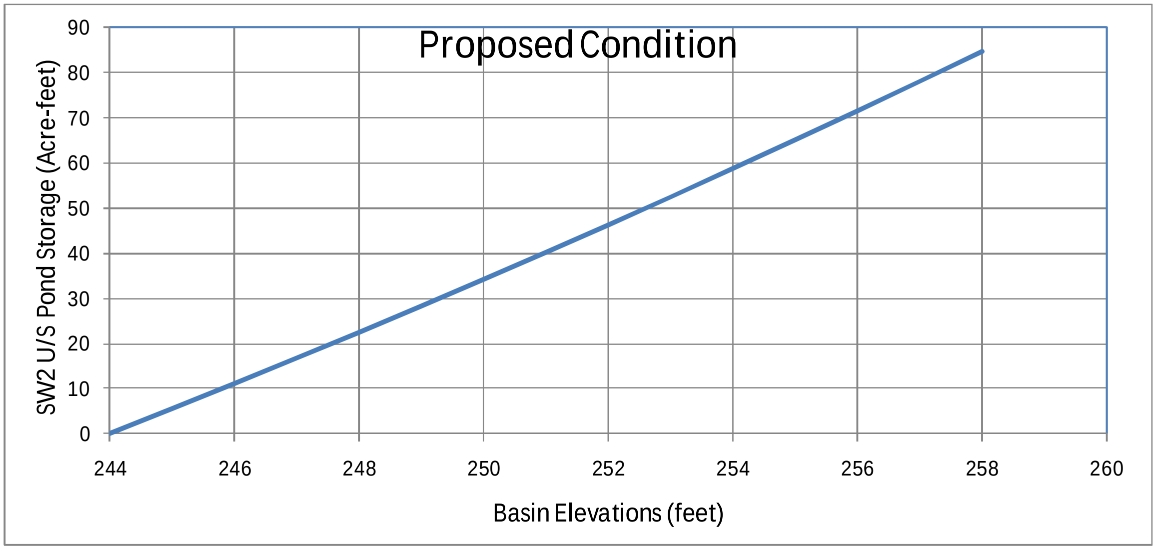
Pond	Weir Height (ft)	Weir Length (ft)	Maximum Inundation Area (acre)	Maximum Pond Storage (acre-ft)
PepsiCo Pond	6	200	2.96	13.44

3.2.4 Hutchinson River Parkway Right-of-Way – The Blind Brook crosses under the parkway a number of times before heading south through the Village of Rye Brook. It appears the brook is fairly channelized through this section. The areas shown in the sketches could be simply opened up, and the additional volume will flood out and detain water simply due to the sinuosity of brook and the numerous culvert headwalls directing flow. Refer to Appendix B for detailed information of pond storage elevation relationship.

Table 16 - Detailed Information about Pond 1, 2 and 3 along Hutchinson River Parkway (HRP) Right-of-way

Ponds	Weir Height (ft)	Weir Length (ft)	Maximum Inundation Area (acre)	Maximum Pond Storage (acre-ft)
HRP Pond 1	4	70	1.32	4.66
HRP Pond 2	2	100	1.81	2.67
HRP Pond 3	4	100	1.44	4.17

3.2.5 Sub-watershed SW2 – The west tributary of Blind Brook as shown in Figure 17 belongs to SW2 watershed which has a contributing area of 3.28 square miles. There are some online reservoirs on the main stem of west tributary of Blind Brook. Based on the aerial image, U/S pond and Edgar Bronfman Lake are proposed at the existing reservoir location. The proposed ponds will be constructed by expanding the existing ponds. The potential storage volumes of those two ponds are listed below. The heights of the dams are 14’ for both ponds based on existing contours. The slide slope for the proposed ponds is assumed to be 3:1.



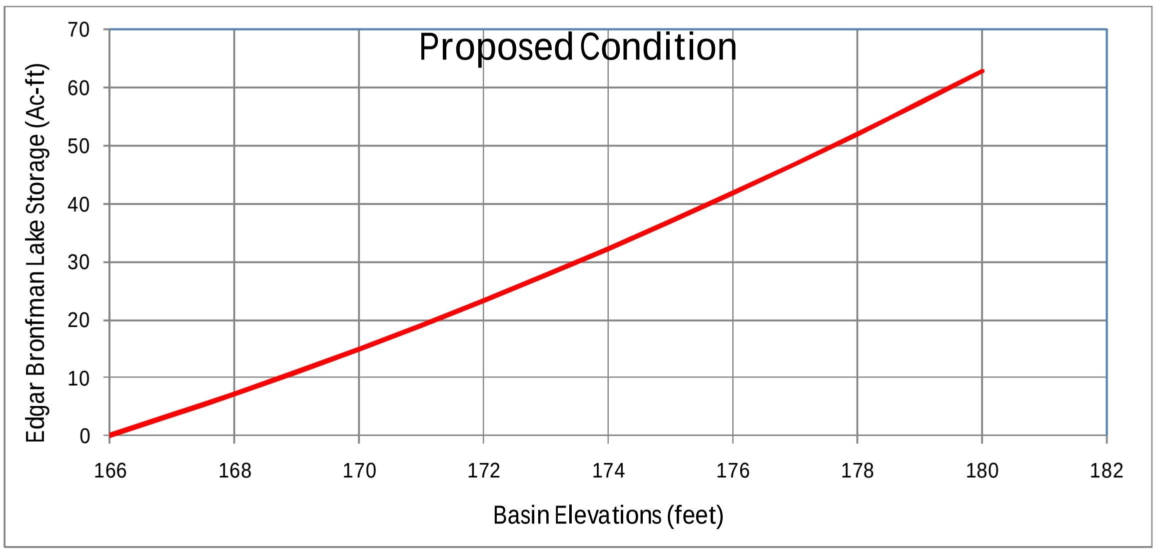


Figure 20 - Proposed Condition Elevation-Storage Relations for SW2 U/S Pond and Edgar Bronfman Lake

3.3 Hydrologic and Hydraulic Analysis Results of Added Detentions

PB first modeled results and calculations for each of the detention ponds as if they were installed as an individual detention pond within the Blind Brook Watershed. This gave us a snap shot of the results of each of the detention areas and how it specifically functions. The hydrologic model, HydroCAD, for each scenario was created first to obtain the reduced peak discharges, then the full hydrographs were used to input into hydraulic model, HEC-RAS as the upstream boundary conditions. Due to the nature of the long computational time required for the unsteady flow simulation, the storms modeled will be limited to 2, 10, 50 and 100-year storm events. For comparison in water surfaces elevations were compared at five locations as shown in Figure 14. Table 17 below shows the comparison of the Future 2011 condition vs. five proposed condition detention analysis results.

As it can be seen from this table, only the SW1-SUNY detention and SW2 detention provide significant water surface elevation reductions at downstream locations. For SUNY detentions, the maximum water surface elevation reduction is 1.70 ft just upstream of I-95 for 10-year flood. For SW2 detentions, the maximum water surface elevation reduction is 1.46 ft at upstream of I-95 for 10-year flood. The reason is that the detention areas in those regions have significant volumes to reduce flood peaks and then water surface elevations downstream.

Table 17 - Water Surface Elevations for Future 2011 PB Study vs. Five Detention Analysis

Return Periods	Locations	Future 2011 Six Sub-watersheds (ft)	(1) Airport Detention (ft)	(2) SUNY Detention (ft)	(3) PepsiCo Detention (ft)	(4) Hutchinson River Parkway (HRP) Detention (ft)	(5) SW2 Detention (ft)	Difference 1 Future 2011 vs. Airport Detention (ft)	Difference 2 Future 2011 vs. SUNY Detention (ft)	Difference 3 Future 2011 vs. PepsiCo Detention (ft)	Difference 4 Future 2011 vs. HRP Detention (ft)	Difference 5 Future 2011 vs. SW2 Detention (ft)
2-Year Storm	D/S I-287	33.28	33.25	33.18	33.26	33.24	33.16	-0.03	-0.10	-0.02	-0.04	-0.12
	Purchase St	27.74	27.71	27.65	27.71	27.69	27.62	-0.03	-0.09	-0.03	-0.05	-0.12
	Mendota Avenue	24.45	24.42	24.36	24.42	24.40	24.33	-0.03	-0.09	-0.03	-0.05	-0.12
	Highland Road	23.88	23.85	23.79	23.86	23.84	23.77	-0.03	-0.09	-0.02	-0.04	-0.11
	U/S I-95	22.95	22.92	22.88	22.93	22.92	22.86	-0.03	-0.07	-0.02	-0.03	-0.09
10-Year Storm	D/S I-287	35.31	35.18	34.40	35.29	35.22	34.61	-0.13	-0.91	-0.02	-0.09	-0.70
	Purchase St	31.22	31.02	29.62	31.18	31.08	30.00	-0.20	-1.60	-0.04	-0.14	-1.22
	Mendota Avenue	27.86	27.67	26.50	27.82	27.73	26.73	-0.19	-1.36	-0.04	-0.13	-1.13
	Highland Road	27.77	27.58	26.37	27.73	27.65	26.61	-0.19	-1.40	-0.04	-0.12	-1.16
	U/S I-95	26.23	25.97	24.53	26.18	26.06	24.77	-0.26	-1.70	-0.05	-0.17	-1.46
50-Year Storm	D/S I-287	36.37	36.29	35.85	36.36	36.33	35.92	-0.08	-0.52	-0.01	-0.04	-0.45
	Purchase St	33.24	33.11	32.34	33.22	33.17	32.46	-0.13	-0.90	-0.02	-0.07	-0.78
	Mendota Avenue	30.93	30.71	29.51	30.89	30.80	29.59	-0.22	-1.42	-0.04	-0.13	-1.34
	Highland Road	30.87	30.65	29.44	30.83	30.75	29.52	-0.22	-1.43	-0.04	-0.12	-1.35
	U/S I-95	29.83	29.60	28.32	29.78	29.69	28.4	-0.23	-1.51	-0.05	-0.14	-1.43
100-Year Storm	D/S I-287	36.59	36.53	36.23	36.58	36.56	36.21	-0.06	-0.36	-0.01	-0.03	-0.38
	Purchase St	33.75	33.6	33.03	33.74	33.68	32.98	-0.15	-0.72	-0.01	-0.07	-0.77
	Mendota Avenue	31.89	31.67	30.67	31.85	31.77	30.53	-0.22	-1.22	-0.04	-0.12	-1.36
	Highland Road	31.84	31.62	30.62	31.81	31.72	30.47	-0.22	-1.22	-0.03	-0.12	-1.37
	U/S I-95	30.87	30.65	29.56	30.84	30.76	29.42	-0.22	-1.31	-0.03	-0.11	-1.45

3.4 Cumulative Implementation of the Detention Areas

Once an understanding was established of the potential improvements from each of the detention areas, PB looked at the two most effective detention areas, SW1-SUNY detention and SW2 detention, and evaluate the cumulative detention effect to better understand the overall improvements associated over time. The reason of choosing these two detentions is that they provide the significant water surface elevation reductions. This effort included the development of the sixth proposed condition scenario. The improvements at SUNY Purchase detention together with SW2 detention were evaluated to determine the overall downstream flood peak reductions at five downstream locations.

As it can be seen from the table, the flood water surface elevation reduction ranges from 0.2 ft to 3.25 ft, and these values are significant when compared with individual detention areas. The maximum water surface elevation reduction is 3.25 ft at Highland Road for the 10-year flood event.

Table 18 - Water Surface Elevations for Future 2011 PB Study vs. Cumulative Detention Analysis

Return Periods	Locations	Future 2011 Six Sub-Watersheds in SW1 and SW2 (ft)	SUNY Detention + SW2 Detention (ft)	Difference 6 Future 2011 vs. SUNY+SW2 Detentions (ft)
2-Year Storm	D/S I-287	33.28	33.08	-0.20
	Purchase St	27.74	27.54	-0.20
	Mendota Avenue	24.45	24.25	-0.20
	Highland Road	23.88	23.68	-0.20
	U/S I-95	22.95	22.81	-0.14
10-Year Storm	D/S I-287	35.31	33.58	-1.73
	Purchase St	31.22	28.12	-3.10
	Mendota Avenue	27.86	24.9	-2.96
	Highland Road	27.77	24.52	-3.25
	U/S I-95	26.23	23.26	-2.97
50-Year Storm	D/S I-287	36.37	35.45	-0.92
	Purchase St	33.24	31.58	-1.66
	Mendota Avenue	30.93	28.36	-2.57
	Highland Road	30.87	28.29	-2.58
	U/S I-95	29.83	26.95	-2.88
100-Year Storm	D/S I-287	36.59	35.91	-0.68
	Purchase St	33.75	32.36	-1.39
	Mendota Avenue	31.89	29.39	-2.50
	Highland Road	31.84	29.32	-2.52
	U/S I-95	30.87	28.19	-2.68

4. Hydraulic Analysis of Resize and Maximize Upper Pond at Bowman Dam

Located within the Village of Rye Brook immediately upstream of I-287, the Bowman Avenue Dam was constructed originally in the 1900' s. The site is the only regional flood control facility owned and operated by the City. The dam and the Upper Pond were once used for ice production. In 1941, the dam collapsed and was rebuilt.

The existing dam is 119 feet long by 13 feet high (measured to the spillway), with a reinforced concrete gravity dam founded on ledge rock. Currently the dam has a 15-foot wide by 11.5-foot high outlet at the bottom of the dam and a 20-foot wide by 2-foot high spillway at the top. The orifice opening is 15-feet wide by 2.5-foot high due the presence of a fixed timber gate. Based on aerial photographs from 1925 and 2013, the Bowman Avenue Dam site has changed considerably. Over the past 88 years, the Upper Pond has been significantly reduced in size due to siltation. It has been estimated that the Upper Pond is approximately 1/4 of its original size.

Chas. H Sell, Inc. in 2008 outlined the automated sluice gate as a recommendation that benefit flood mitigation measures. The City of Rye installed the sluice gate in 2013, and the detailed information of optimal operation of this gate will be discussed in Chapter 5.

Figure 21 - Existing Condition Aerial Image of Bowman Avenue Dam and Upper Pond and Lower Pond (Scale 1:200')

In the report entitled "Hydrologic and Hydraulic Analysis: Bowman Avenue Dam Project, Study for Resizing the Upper Pond Reservoir", by Paul C. RIZZO Engineering in September 2012 (RIZZO report), RIZZO studied:

- Case C: resizing the Upper Pond by excavating 104,000 cubic yards of material (i.e. 96,000 cubic yards of soil and 14,000 cubic yards of rock);
- Case D: a maximized resized alternative aiming to remove approximately 130,000 cubic yards of material (i.e. 109,000 cubic yards of soil and 21,000 cubic yards of rock).

Case C and D can be accomplished by excavating along the banks of the pond, in particular the north side, combined in some instances with dredging of the pond bottom to remove silt. The bottom of the pond itself was taken as an average elevation of approximately 41 feet for case C, and 39 feet for case D. The following figures showed the proposed excavation map for Case C and Case D.

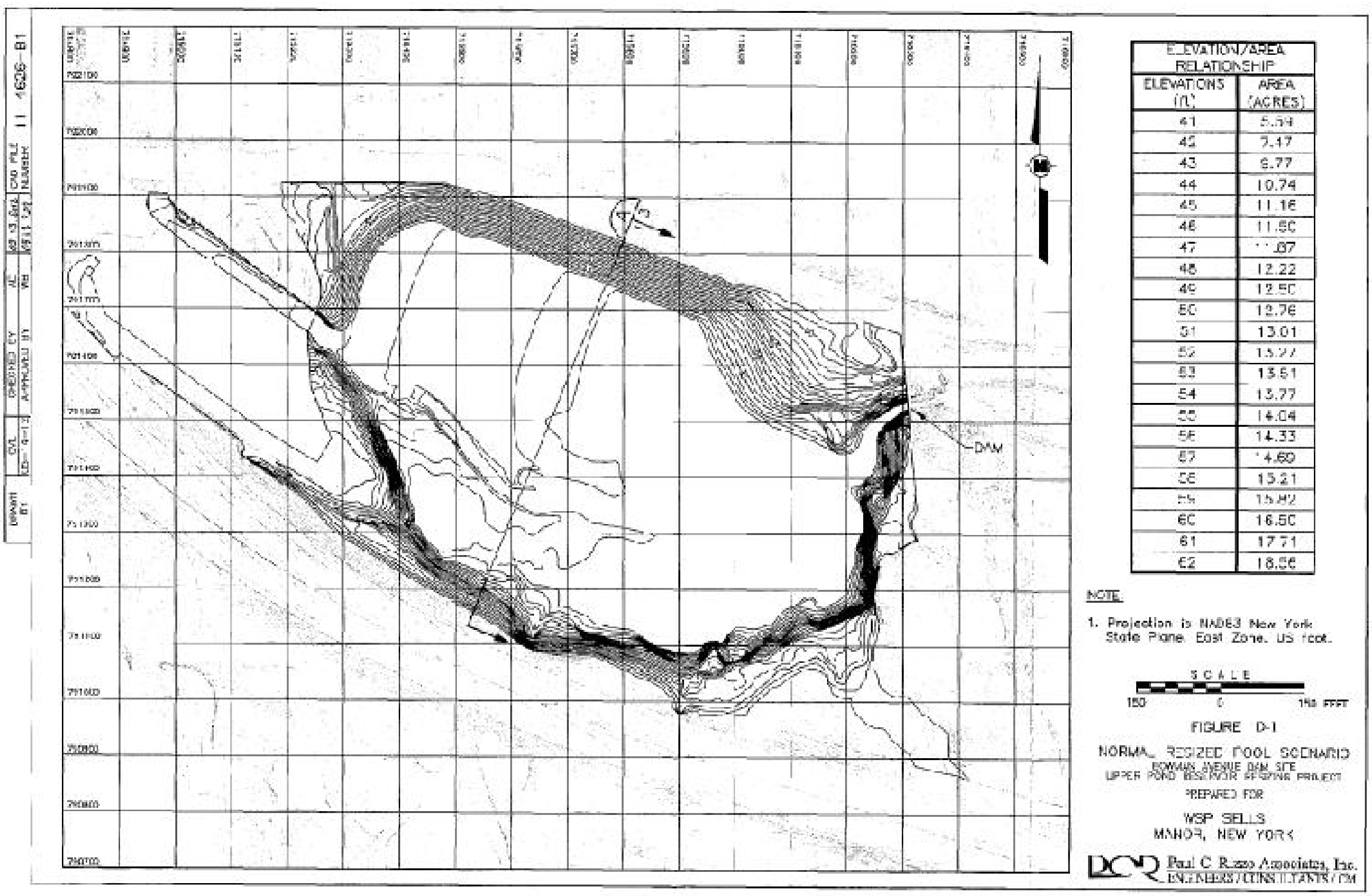


Figure 22-1 - Resize Upper Pond Elevation Area Map

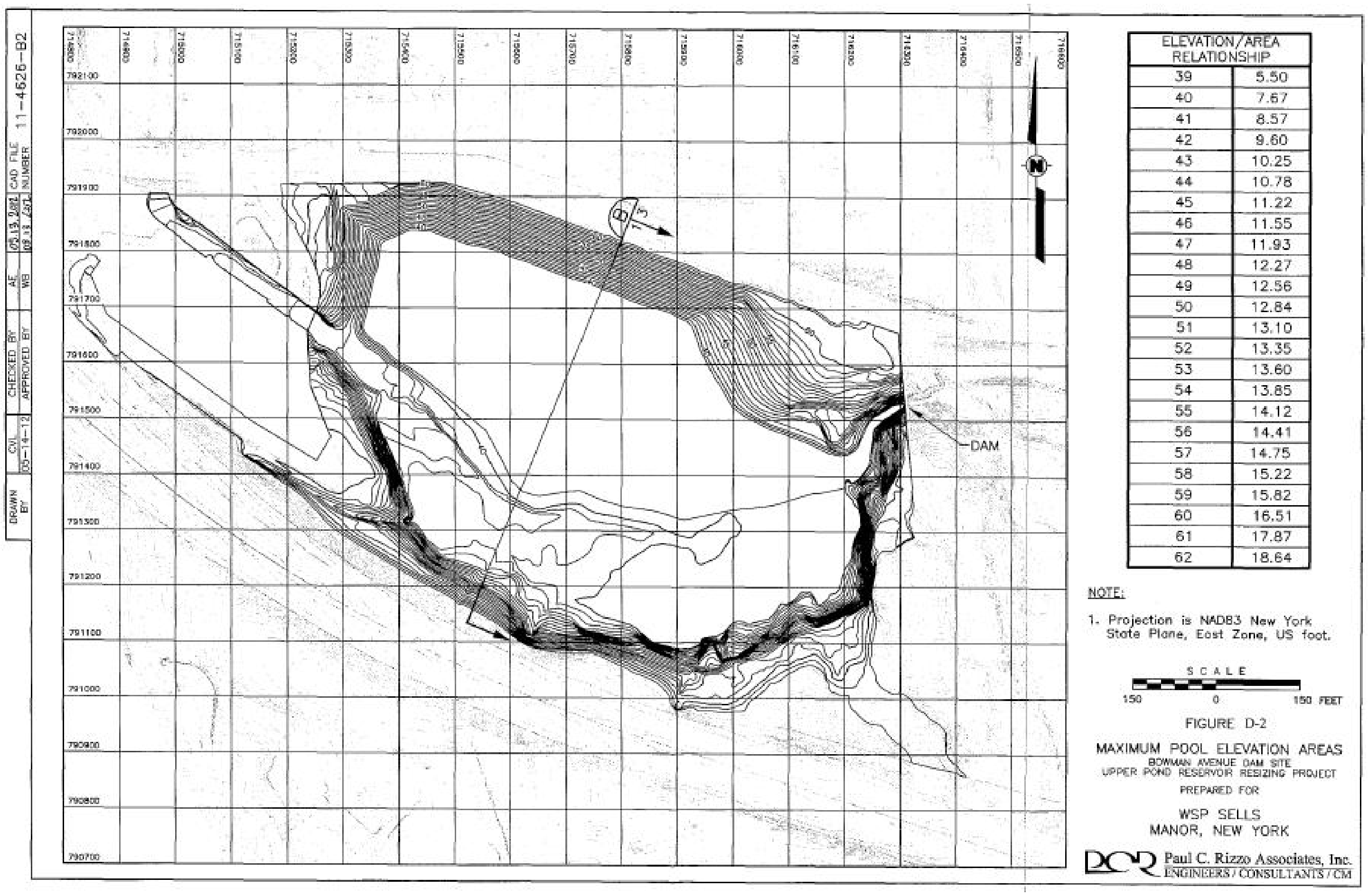


Figure 22-2 - Maximum Upper Pond Elevation Area Map

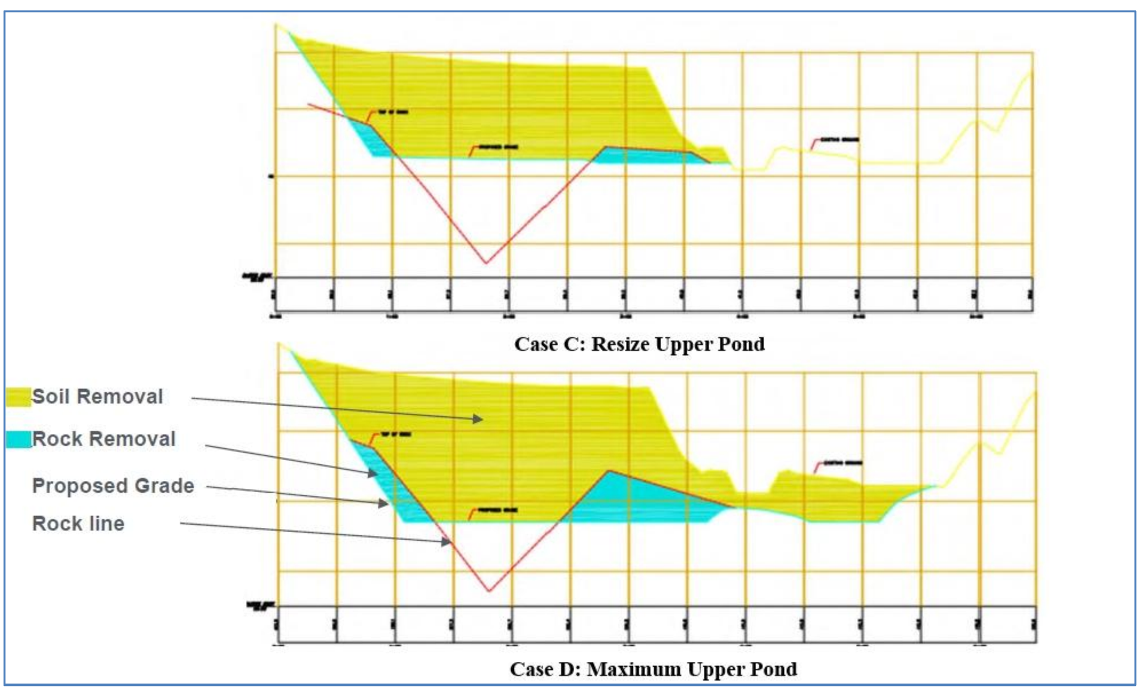


Figure 23 - Cross Section View of Resize Upper Pond and Max Upper Pond

The two alternatives analyzed under this chapter examined the effects of increasing the storage volume of the Upper Pond on the downstream water surface elevation for various storm events. The revised HEC-RAS geometry files for the resized upper pond and maximize upper pond were input into HEC-RAS model, and run for the proposed condition 2, 10, 50 and 100-year flood event.

Hydraulic analysis results showed that between the two resized pond alternatives, Cases C and D, the incremental benefit gained with the maximized resized alternative (Case D) is insignificant. By implementing the smaller resized pond alternative (Case C), potential water elevations are 0.1~ ft lower for 2-year flood, 0.4 to 1.0 ft lower for 10-year floods, 0.3 to 1.3 ft lower for 50-year flood, 0.1 to 0.6 ft lower for 100-year flood.

Table 19 - Future 2011 PB Study vs. Resize and Maximized Upper Pond Scenarios

Water Surface Elevations	Locations	Future 2011 PB Study - TRC Airport DA , SW1 (less Airport) and SW2 (ft)	Case C: RIZZO-Resized Upper Pond (ft)	Difference 7 Future 2011 vs. Case C: RIZZO-Resize Upper Pond	Case D: RIZZO-Max Upper Pond (ft)	Difference 8 Future 2011 vs. Case D: RIZZO-Max Upper Pond
2-Year Flood	D/S I-287	33.28	33.14	-0.14	33.09	-0.19
	Purchase St	27.74	27.60	-0.14	27.51	-0.23
	Mendota Avenue	24.45	24.32	-0.13	24.23	-0.22
	Highland Road	23.88	23.75	-0.13	23.66	-0.22
	U/S I-95	22.95	22.85	-0.10	22.79	-0.16
10-Year Flood	D/S I-287	35.31	34.84	-0.47	34.77	-0.54
	Purchase St	31.22	30.42	-0.80	30.29	-0.93
	Mendota Avenue	27.86	27.12	-0.74	27.01	-0.85
	Highland Road	27.77	27.02	-0.75	26.90	-0.87
	U/S I-95	26.23	25.23	-1.00	25.10	-1.13
50-Year Flood	D/S I-287	36.37	36.04	-0.33	36.02	-0.35
	Purchase St	33.24	32.60	-0.64	32.56	-0.68
	Mendota Avenue	30.93	29.70	-1.23	29.61	-1.32
	Highland Road	30.87	29.64	-1.23	29.54	-1.33
	U/S I-95	29.83	28.53	-1.30	28.43	-1.40
100-Year Flood	D/S I-287	36.59	36.49	-0.10	36.48	-0.11
	Purchase St	33.75	33.43	-0.32	33.40	-0.35
	Mendota Avenue	31.89	31.29	-0.60	31.21	-0.68
	Highland Road	31.84	31.24	-0.60	31.16	-0.68
	U/S I-95	30.87	30.24	-0.63	30.15	-0.72

5. Optimal Sluice Gate Operations

Reports 2 and 3 (2008), prepared by Chas. H Sell, Inc., outlined the installation of an automated sluice gate at the Bowman Avenue Dam as a flood mitigation measure that showed a benefit and was affordable to the City. Whereas Alternative D in Report 3 showed comparable flow reductions for the 2, 5 and 10-year storms, the gate had a greater impact on the 25, 50 and 100-year events (10%, 23% and 9% reductions respectively). This was primarily attributed to the influence of the sluice gate. In Report 4 (2012), the modified gate operation rules proposed by RIZZO showed that a greater potential reduction in downstream water elevations resulting from the sluice gate installation for large storm events (i.e. floods with return periods between 25 and 100 years). Overall, water elevations are projected to be approximately 6 inches lower after sluice gate installation for the 50- and 100-year return period floods.

The City of Rye installed the sluice gate in 2013, and has requested that PB evaluate the operations of the gate to determine if the gate could be operated more efficiently. New parameters were developed and analyzed to determine if a more efficient functional rule parameter existed for the operation of the gate. This chapter analyses the operations of the sluice gate and recommends changes in the location of the stream gauge used to control the sluice gate and the operational rules used to operate the gate.

5.1 Review of the Recommended Sluice Gate Elevations

In the preferred alternatives, Alternative A of Report 2 and Alternative D of Report 3, the installation of an automated sluice gate at the Bowman Avenue Dam was recommeneded to prodive some immediate relief downstream of the dam. An automated sluice gate has the ability to vary the outlet opening, thus providing the optimum orifice size for the flow rate in the stream. Report 2 and 3 analyzed the 2-year to 100-year flood events, during which the orficice diameter should vary from 1.3 ft to 8.3 ft. The sluice gate would be automatically controlled based on water surface elevations measured at a gauge mounted at the dam.

Based on the analysis performed by Chas. H Sell, Inc, this alternative provides the most cost effective means to reduce water surface elevations downstream. The location of the gauge in the HEC-RAS model is Cross Section 2988.114 which is shown in the figure below. As it can be seen from this figure, the gauge of measuing water suface elevations was located in the center of the upper pond, 660 ft upstream of the dam, rather than at the immediate upstream face of the Bowman Avenue Dam as mention in Reports 2 and 3.

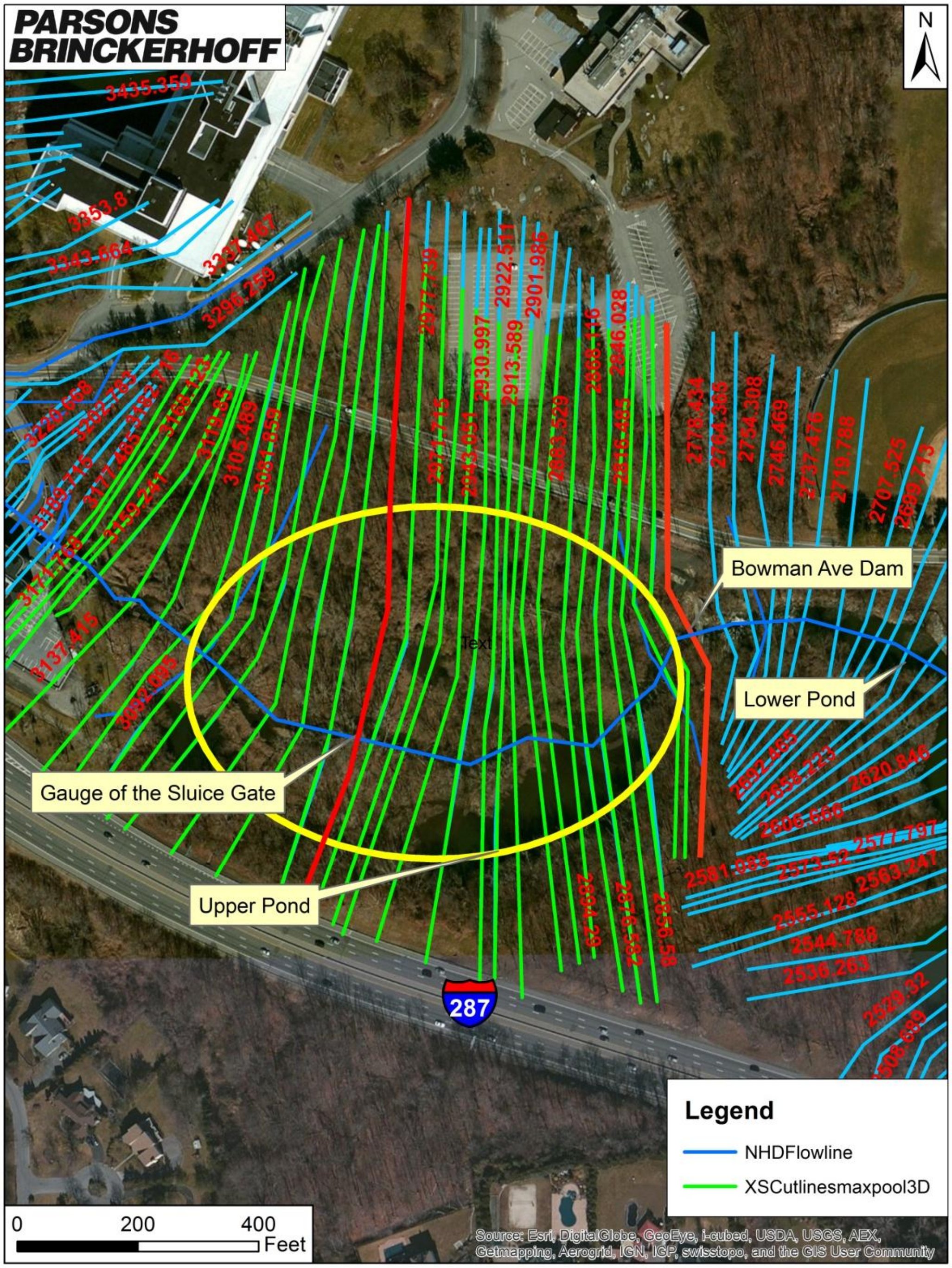


Figure 24 - Location of Gauge of the Sluice Gate in Sell's Model

The Sell's operational rule for the gate is described in the HEC-RAS unsteady flow simulation gate rule as the figure shown below.

```
'WS Elevation' = Cross Sections:WS Elevation(Blindbrook,Reach1,2988.114,Value at cu...
If ('WS Elevation' > 56.57) Then
    Gate.Opening = 8.3
ElseIf ('WS Elevation' < 56.57) And ('WS Elevation' > 55.7) Then
    Gate.Opening = 1.3
ElseIf ('WS Elevation' < 55.7) And ('WS Elevation' > 55.1) Then
    Gate.Opening = 2.3
ElseIf ('WS Elevation' < 55.1) And ('WS Elevation' > 55.06) Then
    Gate.Opening = 2
ElseIf ('WS Elevation' < 55.06) And ('WS Elevation' > 54.74) Then
    Gate.Opening = 3
ElseIf ('WS Elevation' < 54.74) Then
    Gate.Opening = 0.1
ElseIf ('WS Elevation' > 35.6) Then
    Gate.Opening = 3
End If
```

Figure 25 - Gate Operation Rule Developed by Sells (2010)

The results from Sell's study are shown in Table 20 below with the potential water surface elevation reductions. As noted in this table, there is a 4.15 ft reduction of 50-year water surface elevation just upstream of I-95. Due to the change of the overtopping situation at I-95, the compuational methoed for the I-95 bridge has swithed from energey flow to pressure and/or weir flow. Under pressure flow condition, the bridge has more capacity of passing flows, and some of the backwater effect has been removed. That's the reason the water surface elevation reduction was significant upstream of the bridge.

Table 20 - Optimal Gate Operations of Sells Results (2008)

Return Periods	Locations	W.S. Elevation (ft)		Difference (ft)
		Existing	Alternative A	
2-Year Storm	I-95 (U/S)	20.77	20.08	-0.69
	Highland Rd. (U/S)	21.41	21.43	0.02
	Purchase St. (U/S)	25.65	25.65	0
	I-287 (D/S)	31.07	31.07	0
5-Year Storm	I-95 (U/S)	22.95	22.36	-0.59
	Highland Rd. (U/S)	24.19	23.35	-0.84
	Purchase St. (U/S)	27.20	26.61	-0.59
	I-287 (D/S)	32.15	31.62	-0.53
10-Year Storm	I-95 (U/S)	24.59	23.79	-0.80
	Highland Rd. (U/S)	25.88	25.24	-0.64
	Purchase St. (U/S)	28.33	27.73	-0.6
	I-287 (D/S)	32.73	32.27	-0.46
25-Year Storm	I-95 (U/S)	26.93	26.19	-0.74
	Highland Rd. (U/S)	27.78	27.20	-0.58
	Purchase St. (U/S)	30.06	29.21	-0.85
	I-287 (D/S)	33.44	32.87	-0.57
50-Year Storm	I-95 (U/S)	30.56	26.41	-4.15
	Highland Rd. (U/S)	31.01	27.39	-3.62
	Purchase St. (U/S)	31.91	30.18	-1.73
	I-287 (D/S)	34.11	33.66	-0.45
100-Year Storm	I-95 (U/S)	32.17	31.12	-1.05
	Highland Rd. (U/S)	32.60	31.57	-1.03
	Purchase St. (U/S)	33.44	32.55	-0.89
	I-287 (D/S)	34.97	34.54	-0.43

In RIZZO's 2012 report, a Hydrologic and Hydraulic (H&H) analysis was performed to optimize the sluice gate operation and to increase potential benefits from the new sluice gate. The proposed condition Case B represents RIZZO's proposed optimized gate sequence operation consisting or keeping sluice gate closed for the 5-year storm, adopting the Sells gate operation procedure for return period ranging from 5 to 10 years and setting the sluice gate fully open for floods greater than a 10-year event.

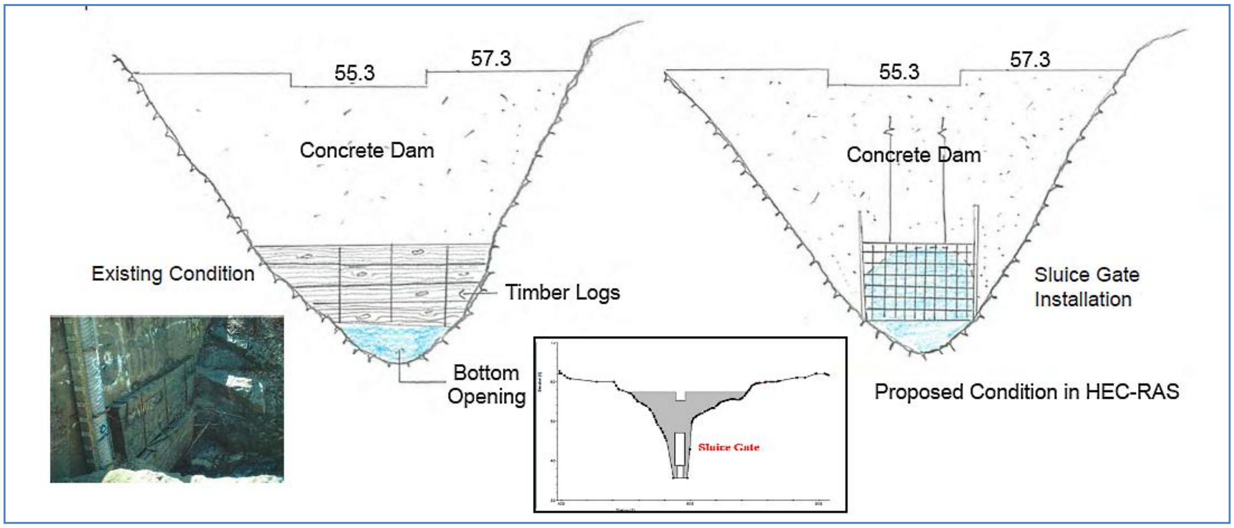


Figure 26 - Graphical Representation of Sluice Gate on Bowman Dam

Results from the RIZZO study showed that there is a potential reduction in water surface elevation resulting from sluice gate installation for large storm events (i.e. floods with return periods between 25 and 100 years). Overall, water elevations are projected to be approximately 0.5 ft lower after sluice gate installation for the 50- and 100-year return period floods. For locations to compare water surface elevations between existing and proposed condition, please prefer to Figure 16 on Page 45.

Table 21 - Optimal Gate Operations of RIZZO Results (2012)

Return Periods	Locations	Existing Condition (CASE A)	Sluice Gate Installation (CASE B)	Difference (ft)
2-Year Storm	D/S of I-287	33.8	33.8	0
	Purchase Street	28.3	28.3	0
	Mendota Avenue	24.9	24.9	0
	Highland Road	24.5	24.5	0
	U/S I-95	23.4	23.4	0
5-Year Storm	D/S of I-287	34.5	*	*
	Purchase Street	29.8	*	*
	Mendota Avenue	26.6	*	*
	Highland Road	26.5	*	*
	U/S I-95	24.7	*	*
10-Year Storm	D/S of I-287	35.1	*	*
	Purchase Street	31	*	*
	Mendota Avenue	27.8	*	*
	Highland Road	27.7	*	*
	U/S I-95	26.1	*	*
25-Year Storm	D/S of I-287	35.5	35.4	-0.1
	Purchase Street	31.7	31.6	-0.1
	Mendota Avenue	28.7	28.6	-0.1
	Highland Road	28.6	28.5	-0.1
	U/S I-95	27.3	27.2	-0.1
50-Year Storm	D/S of I-287	35.9	35.7	-0.2
	Purchase Street	32.5	32.1	-0.4
	Mendota Avenue	29.8	29.4	-0.4
	Highland Road	29.8	29.3	-0.5
	U/S I-95	28.7	28.2	-0.5
100-Year Storm	D/S of I-287	36.3	36.1	-0.2
	Purchase Street	33.2	33.0	-0.2
	Mendota Avenue	31.2	30.8	-0.4
	Highland Road	31.2	30.7	-0.5
	U/S I-95	30.2	29.7	-0.5

*Refer to Sells gate operation sequence (Reference 5)

5.2 Hydraulic Analysis for Optimal Gate Operations Based on Water Surface Elevations at Indian Village

After consulting with engineers from the City of Rye, PB stuided the operational rule of sluice gate on Bowman Avenue Dam. PB also looked at alternate locations for the gauge that measure water surface elevations for controlling the sluice gate. As suggested by the City, the gauge would be moved to a location downstream of the dam, and upstream of the flood prone areas of Indian Village (upstream of I-95). The reason for this is that by using this location, the maximum water surface elevations would be reduced at all downstream locations. The new gauge location is located at Cross Section 890.597, approximately 1,600 ft upstream of I-95 southbound, which is shown in the figure below.

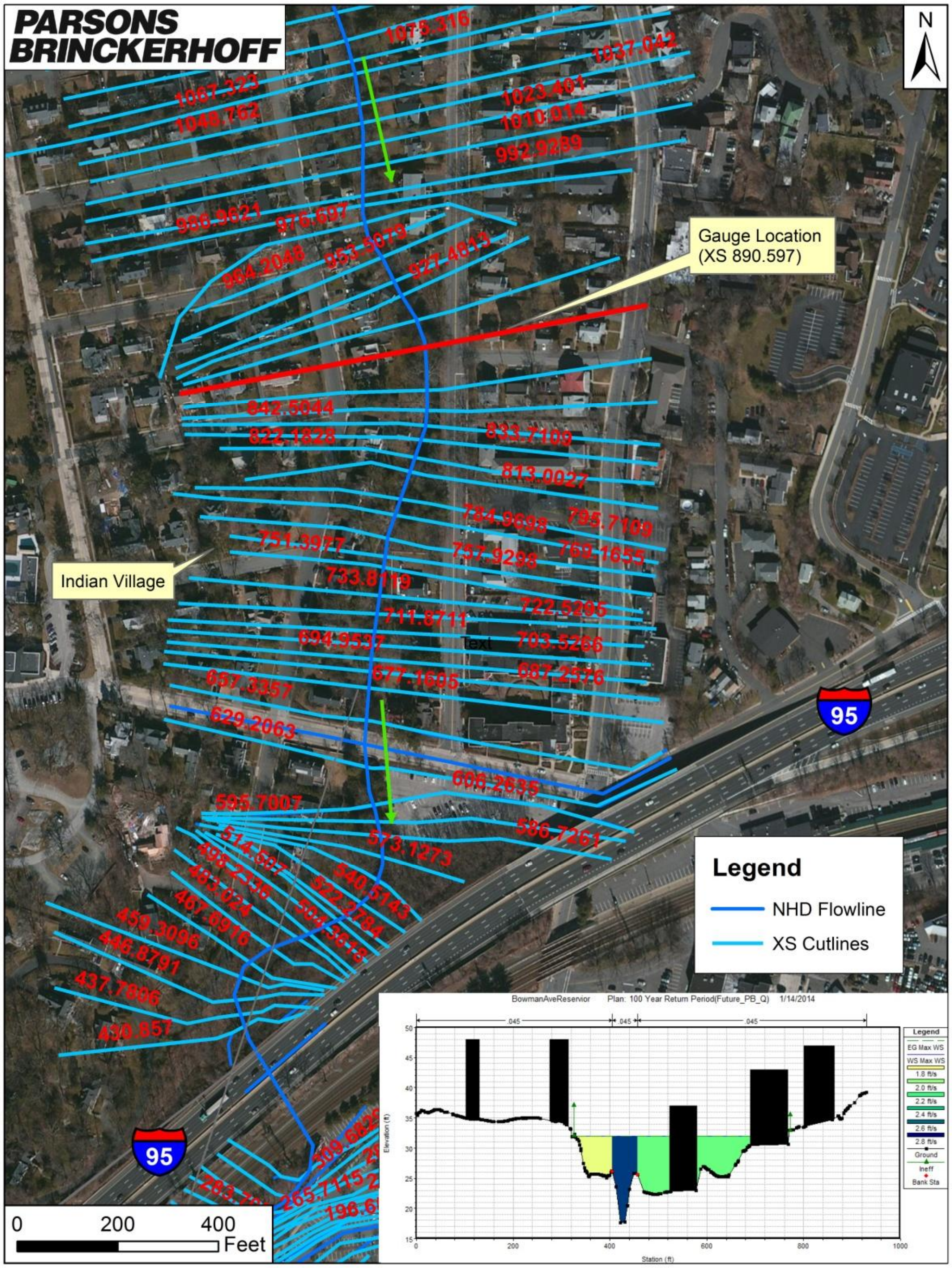


Figure 27 - Location of the Gauge at Indian Village and Cross Section Plot of Cross Section 890.597

The optimal elevation to close the gate for each storm event is obtained by analyzing the existing condition maximum water surface elevation for the corresponding storms at XS 890.597. For example, the 100-year flood water surface elevation vs. simulation time plot for gauge location Cross Section 890.597 was obtained and is shown below. To reduce the peak water surface elvation of 31.95', the gate has to be closed earlier, before the water surface reaches this elevation. The final optimal water surface elevations when the gate is closed was set to be elevation 30'. This value was achived through a trail-and-error process by varying the trigger elevation from 3' below the peak existing elevation (28.95') to 1' below existing peak water surface elevation (30.95') as shown in between the two light green lines in the figure below.

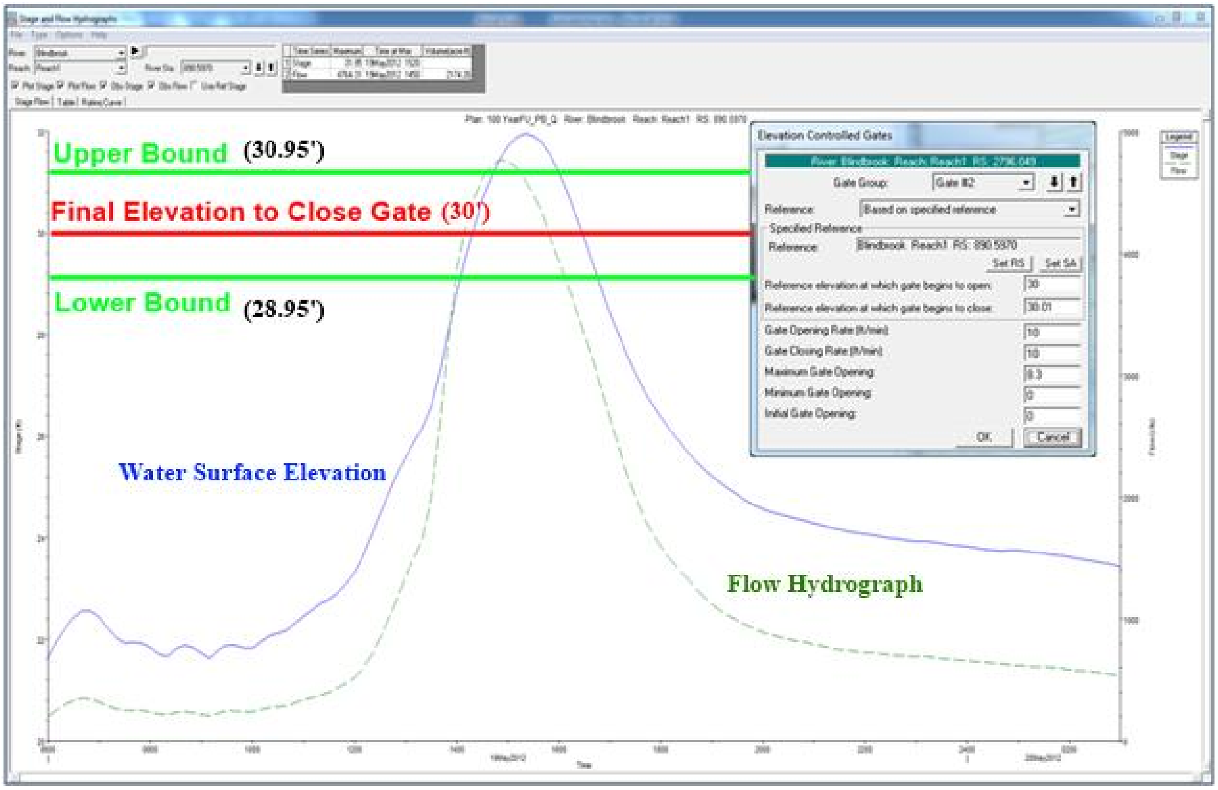


Figure 28 - Optimal Elevation to Close Sluice Gate at Cross Section 890.597 for 100-Year Storm with Trail-and-Error Upper and Lower Bounds

The final optimal elevations to close sluice gate for each storm event was analyzed was listed below in Table 22. As it should be noted, at the beginning of each storm event the sluice gate is fully open. The bottom opeing (Gate #1 in HEC-RAS) is always fully open under all circumstances.

Figure 29 shows the relationship between peak discharges at Cross Section 890.597 and optimal water surface elevations to close the sluice gate. This figure can be used to operate the gate based on peak discharges predicted at this cross section location.

Table 22 - Optimal Water Surface Elevations at XS 890.597 to Close Sluice Gate

Return Periods	Maximum Water Surface Elevation of Existing Condition at XS 890.597 (ft)	Optimal Water Surface Elevation at XS 890.597 to Close Sluice Gate (ft)	Peak Discharge at XS 890.597 (cfs)
2-Year Storm	25.26	23.69	848
5-Year Storm	26.68	24.09	1,844
10-Year Storm	28.14	25.69	2,622
25-Year Storm	29.18	27.19	3,140
50-Year Storm	31.02	29.09	3,672
100-Year Storm	31.95	30.01	4,354

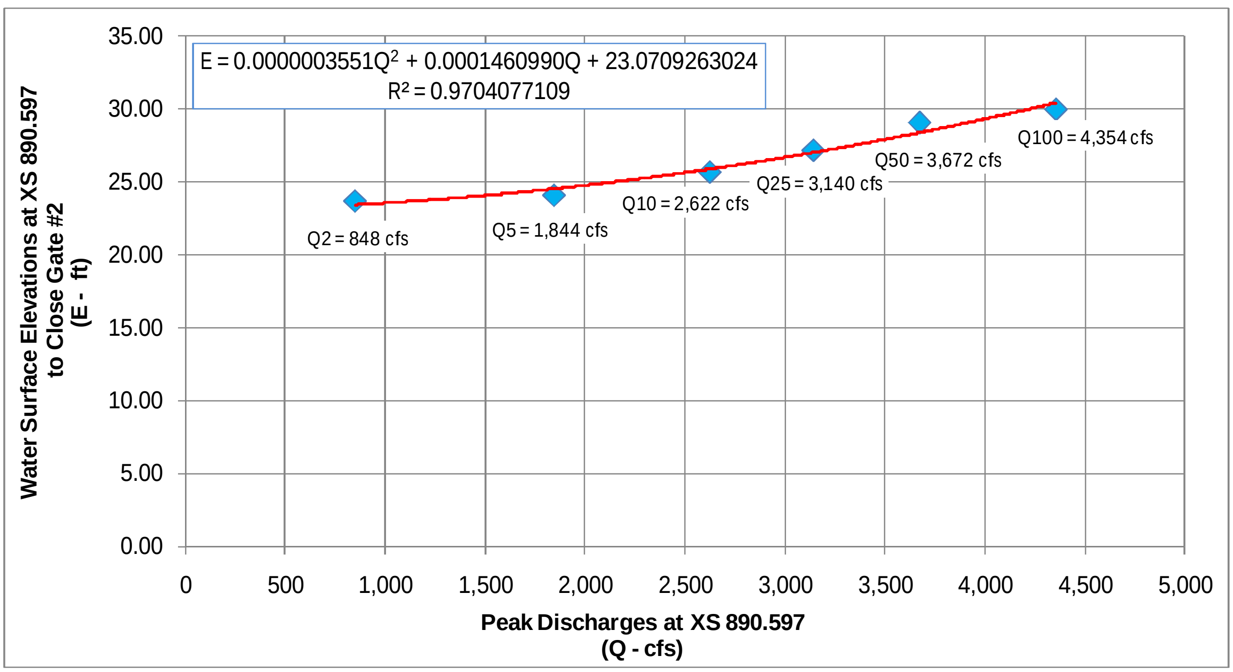


Figure 29 - Optimal Elevation to Close Sluice Gate Based Peak Discharges at Cross Section 890.597

The results of this alternate gauge location at Indian Village are listed below with the potential water surface elevation reductions are shown in the table below. As shown on the following table, when compared with previous RIZZO's study, the water surface elevation reductions have been increased for 10, 25, 50 and 100-year storm. Espiecially for 50-year flood, at Mendota Avenue, Highland Road and U/S of I-95, the reduction in water surface elevation almost trippled when compared with RIZZO's study at the same locations. The maximum water surface elevation reduction is 1.48 ft at both Mendota Avenue and Highland Road for 50-year flood. The benefits of operating the gate utilizing rule developed by PB for storm events smaller than 10-year, i.e., 2 and 5-year storm, are not signifcant.

Table 23 - Results of PB's Optimal Gate Operations Based on Water Surface Elevation at Indian Village (2014)

Return Periods	Locations	Future 2011 PB Study - TRC Airport DA , SW1 and SW2 (ft)	Modified Gate Operation Rules at Indian Village (ft)	Difference 9 Future 2011 vs. Modified Gate Operation Rules (ft)
2-Year Storm	D/S I-287	33.28	33.27	-0.01
	Purchase St	27.74	27.73	-0.01
	Mendota Avenue	24.45	24.44	-0.01
	Highland Road	23.88	23.88	0.00
	U/S I-95	22.95	22.95	0.00
5-Year Storm	D/S I-287	34.42	34.46	0.04
	Purchase St	29.54	29.49	-0.05
	Mendota Avenue	26.26	26.21	-0.05
	Highland Road	26.12	26.06	-0.06
	U/S I-95	24.31	24.26	-0.05
10-Year Storm	D/S I-287	35.31	35.10	-0.21
	Purchase St	31.22	30.88	-0.34
	Mendota Avenue	27.86	27.55	-0.31
	Highland Road	27.77	27.46	-0.31
	U/S I-95	26.23	25.80	-0.43
25-Year Storm	D/S I-287	35.82	35.53	-0.29
	Purchase St	32.15	31.64	-0.51
	Mendota Avenue	28.98	28.38	-0.60
	Highland Road	28.91	28.30	-0.61
	U/S I-95	27.73	26.96	-0.77
50-Year Storm	D/S I-287	36.37	35.90	-0.47
	Purchase St	33.24	32.38	-0.86
	Mendota Avenue	30.93	29.45	-1.48
	Highland Road	30.87	29.39	-1.48
	U/S I-95	29.83	28.26	-1.57
100-Year Storm	D/S I-287	36.59	36.41	-0.18
	Purchase St	33.75	33.22	-0.53
	Mendota Avenue	31.89	31.16	-0.73
	Highland Road	31.84	31.11	-0.73
	U/S I-95	30.87	30.09	-0.78

5.3 Hydraulic Analysis for Optimal Gate Operations Based on Water Surface Elevations at Downstream of I-287

After meeting with engineers from the City of Rye on January 30 2013, PB also studied moving the location of measurement gauge of water surface elevations to a location downstream of the dam, and upstream of the flood prone areas upstream of I-287. Compared with the previous scenario where the gauge is located at Indian village (HEC-RAS Cross Section 890.597), this scenario has the gauge located 170 ft downstream of I-287, and 5,800 ft upstream of I-95 (HEC-RAS Cross Section 2230.179, Figure 30). The advantage of this gauge location is to protect a larger flood-prone residential areas along Blind Brook, and provide greater water surface elevation reductions downstream of Bowman Avenue Dam.

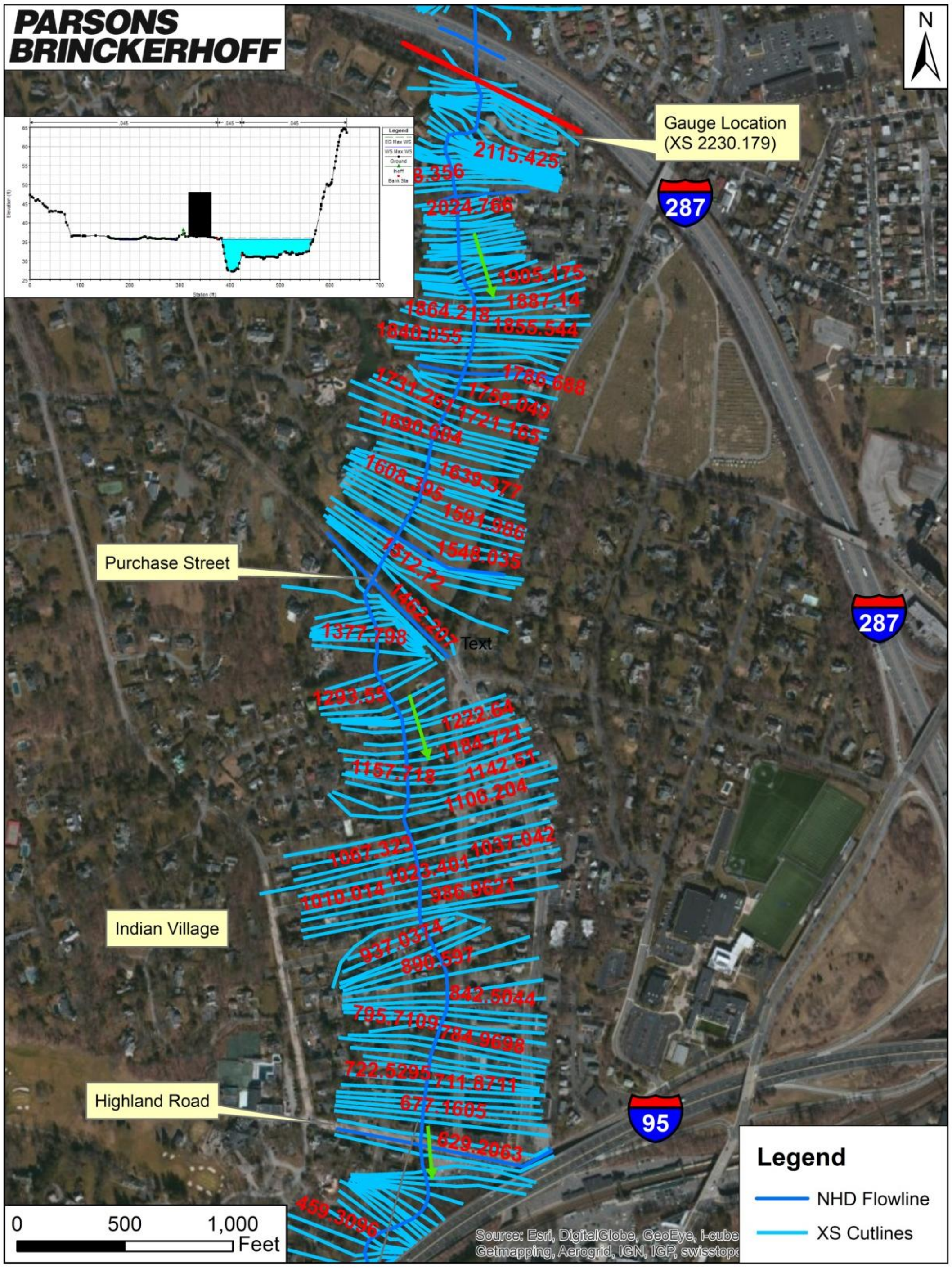


Figure 30 - Location of the Gauge at Downstream of I-287 and Plot of Cross Section 2230.179

The optimal elevation to close the gate for each storm event is obtained by analyzing the existing condition maximum water surface elevation for the corresponding storms at Cross Section 2230.179. Simlar to the approach discussed in Section 5.2, the final optimal water surface elevations when the gate is closed was set using a trail-and-error process by varying the trigger elevation from 3' below the peak existing elevation to 1' below existing peak water surface elevation.

The final optimal elevations to close sluice gate for each storm event analyzed was listed below in Table 24. Figure 31 shows the relationship between peak discharges at Cross Section 2230.179 and optimal water surface elevations to close sluice gate. This figure can be used to operate the gate based on any peak discharges predicted at this cross section location.

Table 24 - Optimal Water Surface Elevations at Cross Section 2230.179 to Close Sluice Gate

Return Periods	Maximum Water Surface Elevation of Existing Condition at XS 2230.179 (ft)	Optimal Water Surface Elevation at XS 2230.179 to Close Sluice Gate (ft)	Peak Discharge at XS 2230.179 (cfs)
2-Year Storm	33.28	31.72	777
5-Year Storm	34.42	33.24	1,813
10-Year Storm	35.31	34.62	2,636
25-Year Storm	35.82	35.13	3,255
50-Year Storm	36.37	35.68	3,931
100-Year Storm	36.59	36.00	4,790

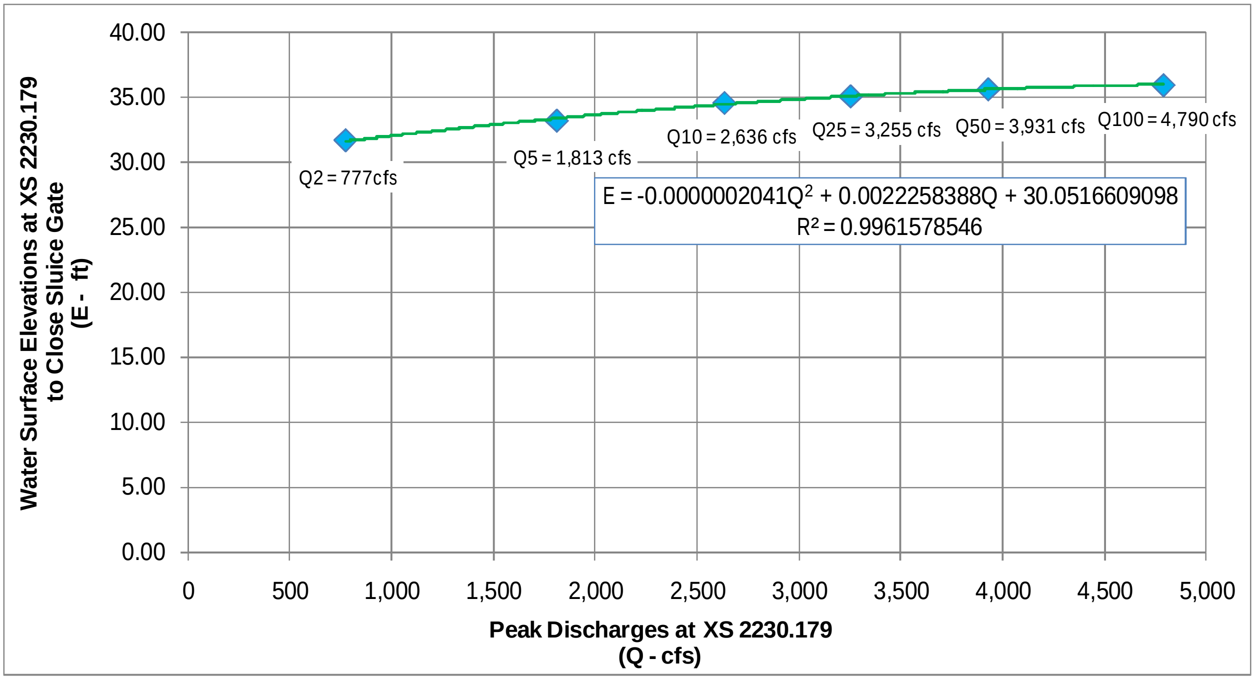


Figure 31 - Optimal Elevation to Close Sluice Gate Based Peak Discharges at Cross Section 2230.179

The results from this scenario are listed below with the potential water surface elevation reductions shown below.

Table 25 - Results of PB's Optimal Gate Operations Based on Water Surface Elevation at Downstream of I-287 (2014)

Return Periods	Locations	Future 2011 PB Study - TRC Airport DA , SW1 and SW2 (ft)	Modified Gate Operation Rules at D/S of I-287 (ft)	Difference 10 Future 2011 vs. Modified Gate Operation Rules (ft)
2-Year Storm	D/S I-287	33.28	33.28	0.00
	Purchase St	27.74	27.74	0.00
	Mendota Avenue	24.45	24.45	0.00
	Highland Road	23.88	23.88	0.00
	U/S I-95	22.95	22.95	0.00
5-Year Storm	D/S I-287	34.42	34.28	-0.14
	Purchase St	29.54	29.23	-0.31
	Mendota Avenue	26.26	26.01	-0.25
	Highland Road	26.12	25.85	-0.27
	U/S I-95	24.31	24.08	-0.23
10-Year Storm	D/S I-287	35.31	34.87	-0.44
	Purchase St	31.22	30.37	-0.85
	Mendota Avenue	27.86	27.25	-0.61
	Highland Road	27.77	27.15	-0.62
	U/S I-95	26.23	25.40	-0.83
25-Year Storm	D/S I-287	35.82	35.36	-0.46
	Purchase St	32.15	31.28	-0.87
	Mendota Avenue	28.98	28.18	-0.80
	Highland Road	28.91	28.11	-0.80
	U/S I-95	27.73	26.70	-1.03
50-Year Storm	D/S I-287	36.37	35.85	-0.52
	Purchase St	33.24	32.27	-0.97
	Mendota Avenue	30.93	29.56	-1.37
	Highland Road	30.87	29.50	-1.37
	U/S I-95	29.83	28.38	-1.45
100-Year Storm	D/S I-287	36.59	36.37	-0.22
	Purchase St	33.75	33.32	-0.43
	Mendota Avenue	31.89	31.23	-0.66
	Highland Road	31.84	31.17	-0.67
	U/S I-95	30.87	30.17	-0.70

As shown in the table, when compared with previous gauge operation based on water surface elevation at Indian Village, the water surface elevation reductions have been all increased for 5, 10, 25-year storms. Especially for 5 and 10-year flood, at Mendota Avenue, Highland Road and upstream of I-95, the water surface elevation reductions almost trippled when compared with Section 5.2's results at the same locations. The water surface elevation reductions for the 50-year and 100-year have been slightly decreased for Mendota Avenue, Highland Road and upstream of I-95, since the gauge is located much further away from those locations. However, the water surface elevation reductions are still considered to be the same manitude as the previous scenario.

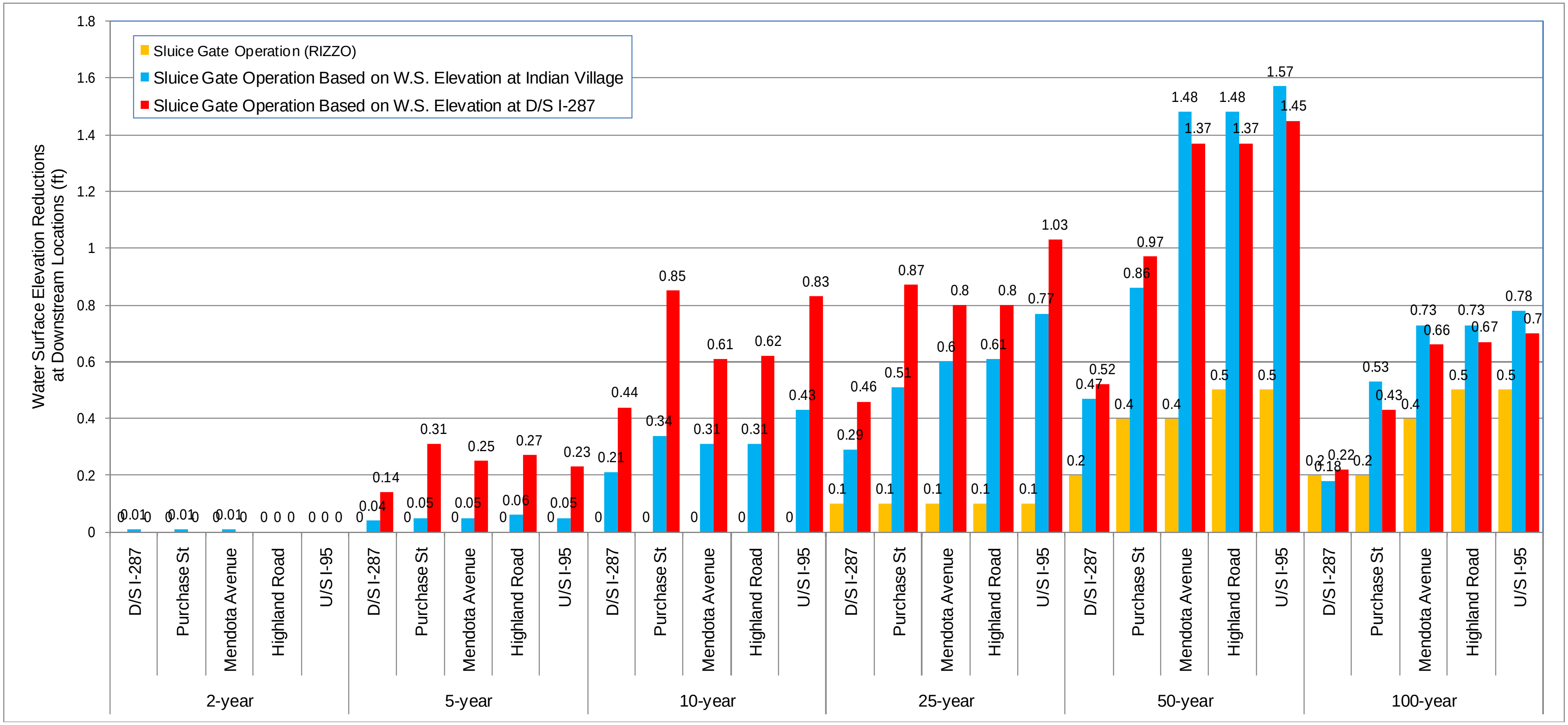


Figure 32 - Comparison of Water Surface Elevation Reduction at 5 Downstream Loation for 2, 5, 10, 25, 50 and 100-year Flood between RIZZO Study (2012) and PB Study (two gauge loaction scenarios, 2014)

5.4 Hydraulic Analysis for Resize/Max Upper Pond with Optimal Gate Operations

PB analyzed the cumulative effect for resize/max Upper Pond and optimal gate operations based on water surface elevations at Indian Village/downstream of I-287 for the following four scenarios:

- Resize Upper Pond scenario (Chapter 4) + optimal gate operations based on water surface elevations at Indian Village developed in Section 5.2;
- Maximize Upper Pond scenario (Chapter 4) + optimal gate operations based on water surface elevations at Indian Village developed in Section 5.2;
- Resize Upper Pond scenario + optimal gate operations based on water surface elevations at downstream of I-287 developed in Section 5.3;
- Maximize Upper Pond scenario + optimal gate operations based on water surface elevations at downstream of I-287 developed in Section 5.3.

The results of the analysis for the scenarios listed above are shown in the Table 26 and 27. With the optimal gate operations based on the water surface elevations at Indian Village or I-287, we can obtain additional reductions in water surface elevation at all downstream locations (difference 11 and 12, difference 13 and 14). However, it should be noted that difference 11 is not the linear summation of difference 7 (resize Upper Pond ONLY) and difference 9 (optimal gate operations at Indian Village ONLY). Difference 11 is the individual unsteady HEC-RAS run with resizing upper pond as RAS geometries, and optimal gate operation rules developed by PB as the unsteady flow condition set up simultaneously in the model. Due the nature of unsteady flow simulation and hydraulic flood routings, the result of the scenario will not equal to the summations of two separated HEC-RAS runs, resize/max upper pond and optimal gate operation. In short, Difference 11 ≠ Difference 7 + Difference 9, Difference 12 ≠ Difference 8 + Difference 9, Difference 13 ≠ Difference 7 + Difference 10, Difference 14 ≠ Difference 8 + Difference 10.

Table 26 - Water Surface Elevation Difference: Resize/Max Upper Pond with Gate with Gauge at Indian Village

Water Surface Elevations	Locations	(1) Future 2011	(2) Gate with Gauge at Indian Village	(3) Difference	(4) Resized Upper Pond	(5) Difference	(6) Max Upper Pond	(7) Difference	(8) Resized Upper Pond with Gauge at Indian Village	(9) Difference 11	(10) Max Upper Pond with Gauge at Indian Village	(11) Difference 12
2-year	D/S I-287	33.28	33.27	-0.01	33.14	-0.14	33.09	-0.19	33.14	-0.14	33.08	-0.20
	Purchase St	27.74	27.73	-0.01	27.60	-0.14	27.51	-0.23	27.6	-0.14	27.51	-0.23
	Mendota Avenue	24.45	24.44	-0.01	24.32	-0.13	24.23	-0.22	24.32	-0.13	24.24	-0.21
	Highland Road	23.88	23.88	0.00	23.75	-0.13	23.66	-0.22	23.75	-0.13	23.66	-0.22
	U/S I-95	22.95	22.95	0.00	22.85	-0.10	22.79	-0.16	22.85	-0.10	22.79	-0.16
10-year	D/S I-287	35.31	35.10	-0.21	34.84	-0.47	34.77	-0.54	35.08	-0.23	35.08	-0.23
	Purchase St	31.22	30.88	-0.34	30.42	-0.80	30.29	-0.93	30.24	-0.98	30.23	-0.99
	Mendota Avenue	27.86	27.55	-0.31	27.12	-0.74	27.01	-0.85	26.73	-1.13	26.60	-1.26
	Highland Road	27.77	27.46	-0.31	27.02	-0.75	26.90	-0.87	26.61	-1.16	26.47	-1.30
	U/S I-95	26.23	25.80	-0.43	25.23	-1.00	25.10	-1.13	24.78	-1.45	24.63	-1.60
50-year	D/S I-287	36.37	35.90	-0.47	36.04	-0.33	36.02	-0.35	35.87	-0.50	35.83	-0.54
	Purchase St	33.24	32.38	-0.86	32.60	-0.64	32.56	-0.68	32.22	-1.02	32.15	-1.09
	Mendota Avenue	30.93	29.45	-1.48	29.70	-1.23	29.61	-1.32	29.09	-1.84	29.00	-1.93
	Highland Road	30.87	29.39	-1.48	29.64	-1.23	29.54	-1.33	29.02	-1.85	28.93	-1.94
	U/S I-95	29.83	28.26	-1.57	28.53	-1.30	28.43	-1.40	27.87	-1.96	27.76	-2.07
100-year	D/S I-287	36.59	36.41	-0.18	36.49	-0.10	36.48	-0.11	36.18	-0.41	36.18	-0.41
	Purchase St	33.75	33.22	-0.53	33.43	-0.32	33.40	-0.35	32.89	-0.86	32.89	-0.86
	Mendota Avenue	31.89	31.16	-0.73	31.29	-0.60	31.21	-0.68	30.27	-1.62	30.25	-1.64
	Highland Road	31.84	31.11	-0.73	31.24	-0.60	31.16	-0.68	30.21	-1.63	30.20	-1.64
	U/S I-95	30.87	30.09	-0.78	30.24	-0.63	30.15	-0.72	29.14	-1.73	29.12	-1.75

(1) Future 2011: existing condition with various modernization and improvement projects proposed at Westchester County Airport in 2011.
(2) Gate with Gauge at Indian Village: optimal gate operations based on water surface elevations measured by a gauge located at Indian Village.
(3) Difference between (2) and (1).
(4) Resized Upper Pond: resized the Upper Pond by excavating 104,000 cubic yards of material (i.e. 96,000 cubic yards of soil and 14,000 cubic yards of rock).
(5) Difference between (4) and (1).
(6) Max Upper Pond: maximized scenario aiming to remove 130,000 cubic yards of material (i.e. 109,000 cubic yards of soil and 21,000 cubic yards of rock).
(7) Difference between (6) and (1).
(8) Resized Upper Pond with Gauge at Indian Village: cumulative effect for resize Upper Pond and Gate with Gauge at Indian Village, Scenario (2) and (4).
(9) Difference 11: difference between (8) and (1).
(10) Max Upper Pond with Gauge at Indian Village: cumulative effect for resize Upper Pond and Gate with Gauge at Indian Village, Scenario (2) and (6).
(11) Difference 12: difference between (10) and (1).

Table 27 - Water Surface Elevation Difference: Resize/Max Upper Pond with Gate with Gauge at Downstream of I-287

Water Surface Elevations	Locations	(1) Future 2011	(2) Gate with Gauge at D/S I-287	(3) Difference	(4) Resized Upper Pond	(5) Difference	(6) Max Upper Pond	(7) Difference	(8) Resized Upper Pond with Gauge at D/S I-287	(9) Difference 13	(10) Max Upper Pond with Gauge at D/S I-287	(11) Difference 14
2-year	D/S I-287	33.28	33.28	0.00	33.14	-0.14	33.09	-0.19	33.14	-0.14	33.09	-0.19
	Purchase St	27.74	27.74	0.00	27.60	-0.14	27.51	-0.23	27.6	-0.14	27.51	-0.23
	Mendota Avenue	24.45	24.45	0.00	24.32	-0.13	24.23	-0.22	24.32	-0.13	24.23	-0.22
	Highland Road	23.88	23.88	0.00	23.75	-0.13	23.66	-0.22	23.75	-0.13	23.66	-0.22
	U/S I-95	22.95	22.95	0.00	22.85	-0.10	22.79	-0.16	22.85	-0.10	22.79	-0.16
10-year	D/S I-287	35.31	34.87	-0.44	34.84	-0.47	34.77	-0.54	34.70	-0.61	34.70	-0.61
	Purchase St	31.22	30.37	-0.85	30.42	-0.80	30.29	-0.93	30.17	-1.05	30.14	-1.08
	Mendota Avenue	27.86	27.25	-0.61	27.12	-0.74	27.01	-0.85	27.06	-0.80	27.03	-0.83
	Highland Road	27.77	27.15	-0.62	27.02	-0.75	26.90	-0.87	26.95	-0.82	26.92	-0.85
	U/S I-95	26.23	25.40	-0.83	25.23	-1.00	25.10	-1.13	25.16	-1.07	25.12	-1.11
50-year	D/S I-287	36.37	35.85	-0.52	36.04	-0.33	36.02	-0.35	35.69	-0.68	35.68	-0.69
	Purchase St	33.24	32.27	-0.97	32.60	-0.64	32.56	-0.68	32.07	-1.17	32.06	-1.18
	Mendota Avenue	30.93	29.56	-1.37	29.70	-1.23	29.61	-1.32	29.23	-1.70	29.26	-1.67
	Highland Road	30.87	29.50	-1.37	29.64	-1.23	29.54	-1.33	29.16	-1.71	29.19	-1.68
	U/S I-95	29.83	28.38	-1.45	28.53	-1.30	28.43	-1.40	28.02	-1.81	28.06	-1.77
100-year	D/S I-287	36.59	36.37	-0.22	36.49	-0.10	36.48	-0.11	36.04	-0.55	36.04	-0.55
	Purchase St	33.75	33.32	-0.43	33.43	-0.32	33.40	-0.35	32.79	-0.96	32.78	-0.97
	Mendota Avenue	31.89	31.23	-0.66	31.29	-0.60	31.21	-0.68	30.53	-1.36	30.51	-1.38
	Highland Road	31.84	31.17	-0.67	31.24	-0.60	31.16	-0.68	30.48	-1.36	30.46	-1.38
	U/S I-95	30.87	30.17	-0.70	30.24	-0.63	29.14	-1.73	29.42	-1.45	29.40	-1.47

(1) Future 2011: existing condition with various modernization and improvement projects proposed at Westchester County Airport in 2011.
(2) Gate with Gauge at D/S I-287: optimal gate operations based on water surface elevations measured by a gauge located at downstream of I-287.
(3) Difference between (2) and (1).
(4) Resized Upper Pond: resized the Upper Pond by excavating 104,000 cubic yards of material (i.e. 96,000 cubic yards of soil and 14,000 cubic yards of rock).
(5) Difference between (4) and (1).
(6) Max Upper Pond: maximized scenario aiming to remove 130,000 cubic yards of material (i.e. 109,000 cubic yards of soil and 21,000 cubic yards of rock).
(7) Difference between (6) and (1).
(8) Resized Upper Pond with Gauge at D/S I-287: cumulative effect for resize Upper Pond and Gate with Gauge at downstream of I-287, Scenario (2) and (4).
(9) Difference 13: difference between (8) and (1).
(10) Max Upper Pond with Gauge at D/S I-287: cumulative effect for resize Upper Pond and Gate with Gauge at downstream of I-287, Scenario (2) and (6).
(11) Difference 14: difference between (10) and (1).

5.5 Hydraulic Analysis for SW1-SUNY Detention, Resize/Max Upper Pond with Optimal Gate Operations

PB analyzed the cumulative effect for SW1-SUNY detention, resize/max Upper Pond and optimal gate operations based on water surface elevations at Indian Village/downstream of I-287 for the following four scenarios:

- SW1-SUNY detention (Section 3.3) + resize Upper Pond scenario (Chapter 4) + optimal gate operations based on water surface elevations at Indian Village developed in Section 5.2;
- SW1-SUNY detention (Section 3.3) + maximize Upper Pond scenario (Chapter 4) + optimal gate operations based on water surface elevations at Indian Village developed in Section 5.2;
- SW1-SUNY detention (Section 3.3) + resize Upper Pond scenario + optimal gate operations based on water surface elevations at downstream of I-287 developed in Section 5.3;
- SW1-SUNY detention (Section 3.3) + maximize Upper Pond scenario + optimal gate operations based on water surface elevations at downstream of I-287 developed in Section 5.3.

The results of the above scenarios are shown in the Table 28 and 29 below. With SW1-SUNY detention, resize/max Upper Pond and the optimal gate operations based on the water surface elevations at Indian Village or I-287, we can obtain additional water surface elevation reductions at all downstream locations (difference 15 and 16, difference 17 and 18).

Table 28 - SW1-SUNY Detention, Plus RIZZO Resize/Max Upper Pond with Optimal Gate Operation Based on Water Surface Elevations at Indian Village

Water Surface Elevations	Locations	Future 2011 PB Study - TRC Airport DA , SW1 (less Airport) and SW2 (ft)	SW1-SUNY+RIZZO-Resized Upper Pond with Gate Operation at Indian Village (ft)	SW1-SUNY+RIZZO-Max Upper Pond with Gate Operation at Indian Village (ft)	Difference 15 Future 2011 vs. SUNY Detention + RIZZO-Resize Upper Pond and Gate Operation at Indian Village	Difference 16 Future 2011 vs. SUNY Detention + RIZZO-Max Upper Pond and Gate Operation at Indian Village
2-year	D/S I-287	33.28	33.09	33.03	-0.19	-0.25
	Purchase St	27.74	27.55	27.45	-0.19	-0.29
	Mendota Avenue	24.45	24.27	24.18	-0.18	-0.27
	Highland Road	23.88	23.7	23.6	-0.18	-0.28
	U/S I-95	22.95	22.82	22.75	-0.13	-0.20
10-year	D/S I-287	35.31	35.08	35.05	-0.23	-0.26
	Purchase St	31.22	30.30	30.25	-0.92	-0.97
	Mendota Avenue	27.86	26.39	26.36	-1.47	-1.50
	Highland Road	27.77	26.25	26.22	-1.52	-1.55
	U/S I-95	26.23	24.42	24.40	-1.81	-1.83
50-year	D/S I-287	36.37	35.65	35.64	-0.72	-0.73
	Purchase St	33.24	31.33	31.33	-1.91	-1.91
	Mendota Avenue	30.93	27.73	27.65	-3.20	-3.28
	Highland Road	30.87	27.65	27.56	-3.22	-3.31
	U/S I-95	29.83	26.06	25.94	-3.77	-3.89
100-year	D/S I-287	36.59	35.72	35.71	-0.87	-0.88
	Purchase St	33.75	32.21	32.19	-1.54	-1.56
	Mendota Avenue	31.89	29.52	29.49	-2.37	-2.40
	Highland Road	31.84	29.45	29.42	-2.39	-2.42
	U/S I-95	30.87	28.33	28.30	-2.54	-2.57

Table 29 - SW1-SUNY Detention, Plus RIZZO Resize/Max Upper Pond with Optimal Gate Operation Based on Water Surface Elevations at Downstream of I-287

Water Surface Elevations	Locations	Future 2011 PB Study - TRC Airport DA , SW1 (less Airport) and SW2 (ft)	SW1-SUNY+RIZZO-Resized Upper Pond with Gate Operation at Downstream of I-287 (ft)	SW1-SUNY+RIZZO-Max Upper Pond with Gate Operation at Downstream of I-287 (ft)	Difference 17 Future 2011 vs. SUNY Detention + RIZZO-Resize Upper Pond and Gate Operation at Indian Village	Difference 18 Future 2011 vs. SUNY Detention + RIZZO-Max Upper Pond and Gate Operation at Indian Village
2-year	D/S I-287	33.28	33.09	33.03	-0.19	-0.25
	Purchase St	27.74	27.55	27.45	-0.19	-0.29
	Mendota Avenue	24.45	24.27	24.18	-0.18	-0.27
	Highland Road	23.88	23.7	23.6	-0.18	-0.28
	U/S I-95	22.95	22.82	22.75	-0.13	-0.20
10-year	D/S I-287	35.31	34.53	34.49	-0.78	-0.82
	Purchase St	31.22	29.97	29.87	-1.25	-1.35
	Mendota Avenue	27.86	26.85	26.78	-1.01	-1.08
	Highland Road	27.77	26.73	26.66	-1.04	-1.11
	U/S I-95	26.23	24.91	24.83	-1.32	-1.40
50-year	D/S I-287	36.37	35.38	35.38	-0.99	-0.99
	Purchase St	33.24	31.60	31.58	-1.64	-1.66
	Mendota Avenue	30.93	28.59	28.56	-2.34	-2.37
	Highland Road	30.87	28.52	28.49	-2.35	-2.38
	U/S I-95	29.83	27.25	27.21	-2.58	-2.62
100-year	D/S I-287	36.59	35.72	35.71	-0.87	-0.88
	Purchase St	33.75	32.21	32.19	-1.54	-1.56
	Mendota Avenue	31.89	29.52	29.49	-2.37	-2.40
	Highland Road	31.84	29.45	29.42	-2.39	-2.42
	U/S I-95	30.87	28.33	28.30	-2.54	-2.57

6. Cost Estimate for Resizing Upper Pond and SUNY-Purchase Ponds

6.1 Introduction

On March 27, 2014, a meeting was held between the City of Rye (City)'s Flood Mitigation Committee and Parsons Brinckerhoff (PB). During this meeting, the City requested PB to perform additional work, as detailed below, which involves expanding the scope of the Blind Brook Watershed Study currently being performed by PB:

- Task 1 – Review of Upper Pond Cost Estimate: A review of the existing cost estimate and soil survey prepared by Rizzo for the proposed improvements to the Upper Pond at the Bowman Avenue Dam will be performed. For this task quantities and unit prices used in the original cost estimate will be reviewed and updated as needed. Unit prices will be obtained from current New York State Department of Transportation (NYSDOT) item costs.

- Task 2 – Cost Estimate for the Proposed State University of New York (SUNY) Purchase Ponds: Cost estimates for the proposed retention ponds to be constructed on the SUNY Purchase property will be prepared and will be an order of magnitude estimate which will not include engineering design or permitting costs. NYSDOT item costs will be used in the preparation of this estimate.

- Task 3 – Additional Report Tables: Additional table will be prepared detailing the costs and water surface elevation reductions as a result of resizing the Upper Pond and the creation of the two SUNY Purchase Ponds

6.2 Review and Verify Cost Estimate of Resizing Upper Pond

The upper pond of the Bowman Avenue dam has been previously studied by Paul C. RIZZO Engineering (RIZZO) in a report titled "Hydrologic and Hydraulic Analysis: Bowman Avenue Dam Project, Study for Resizing the Upper Pond Reservoir", (RIZZO report) dated September 2012. Within the Rizzo study, two scenarios pertaining to Upper pond as found on page 10 of this report included:

- Case C: The proposed resizing of the Upper Pond by excavating 104,000 cubic yards of material (i.e. 96,000 cubic yards of soil and 14,000 cubic yards of rock). The actual quantity should be 110,000 cubic yards based on summarizing the soil and rock quantity, not the 104,000 cubic yards mentioned above.

- Case D: This alternative maximized the Upper Pond by proposing to remove approximately 130,000 cubic yards of material (i.e. 109,000 cubic yards of soil and 21,000 cubic yards of rock).

Hydraulic analysis completed within this report shows that between the two resized pond alternatives, Cases C and D, the incremental benefit gained by implementing (Case D) would be insignificant. By implementing the smaller resized pond alternative (Case C), potential water elevations reductions are shown in the table below;

Storm Event (Yr.)	Water surface elevation reduction (ft.)
2	0.1
10	0.4 to 1.0
50	0.3 to 1.3
100	0.1 to 0.6

Benefits of Case C can be accomplished by excavating along the banks of the pond, in particular the north side, combined in some instances with dredging of the pond bottom to remove existing silt deposits. The average elevation used for the bottom of the pond was 41 feet. Resizing of the Upper pond is shown in Figure 22-1 with elevation and surface area values listed on the right hand side of this figure. As noted, the Resized Upper Pond scenario will involve excavation of the non-hazardous contaminated soil located in the southern part of the pond (Figure 33) to varying depths. RIZZO has estimated a removal of 5,100 tons of contaminated soil for Case C.

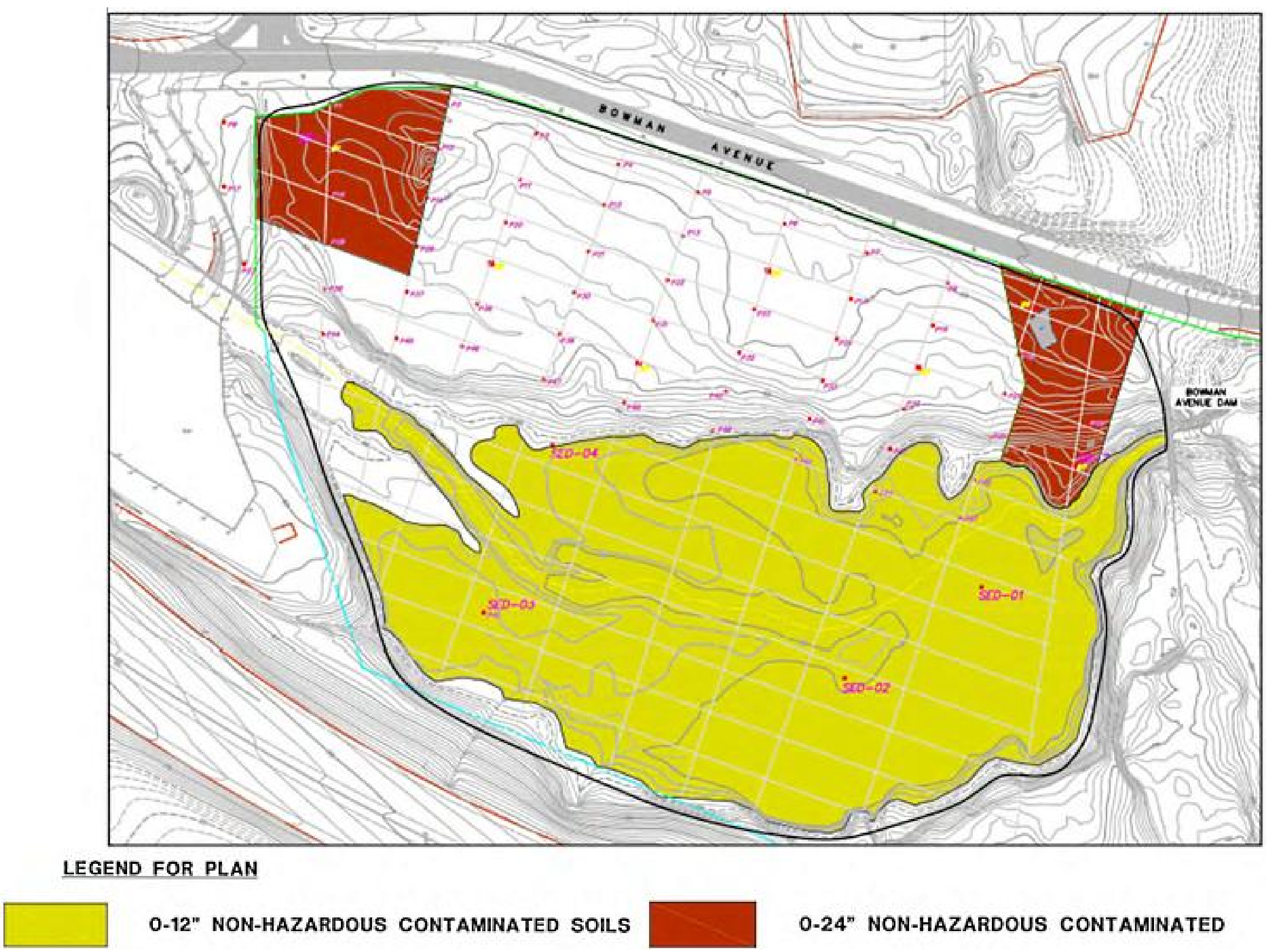


Figure 33 - Case C: Location of Contaminated Soils in Upper Pond

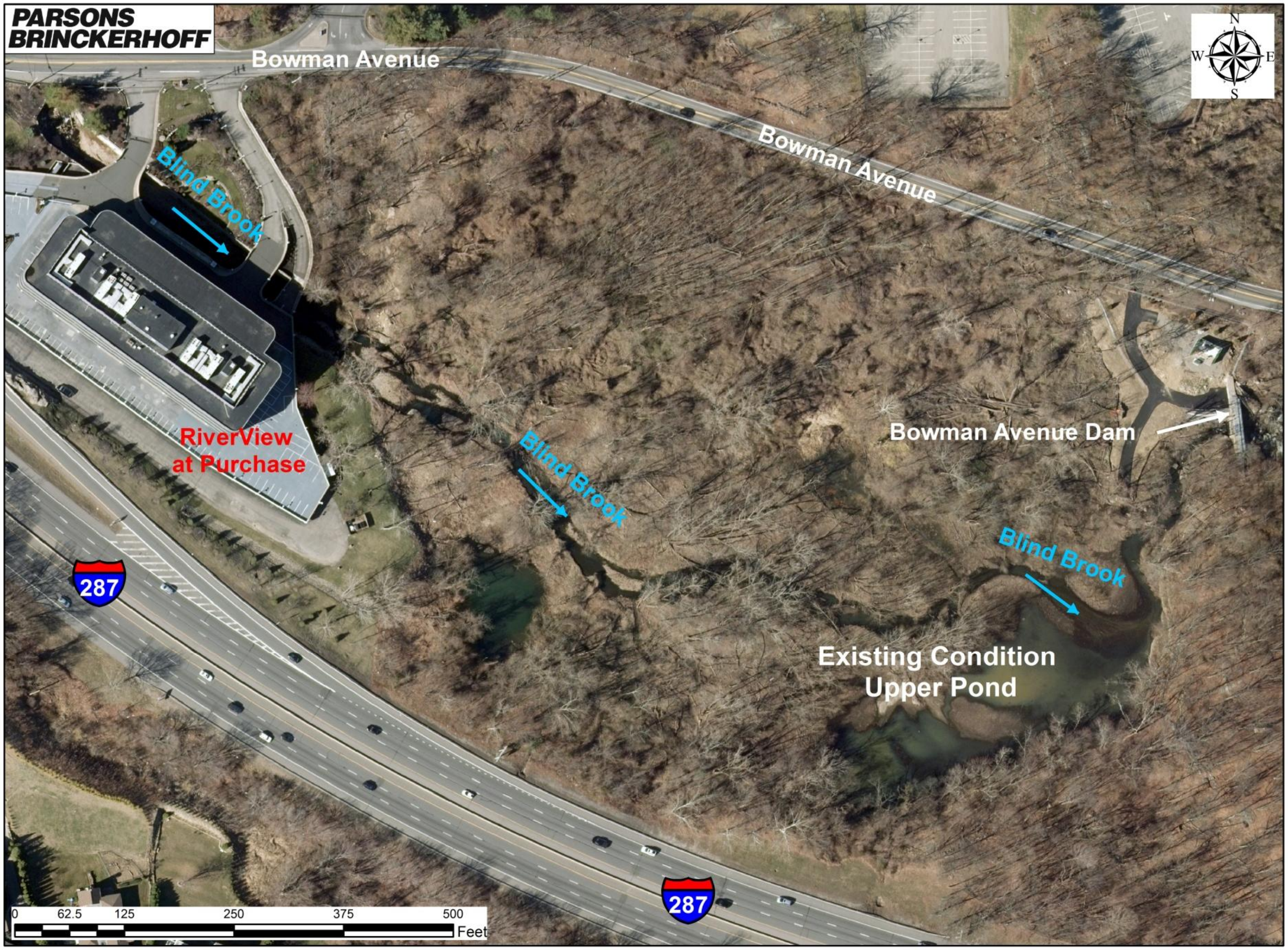


Figure 34 - Aerial Image of Existing Condition Upper Pond

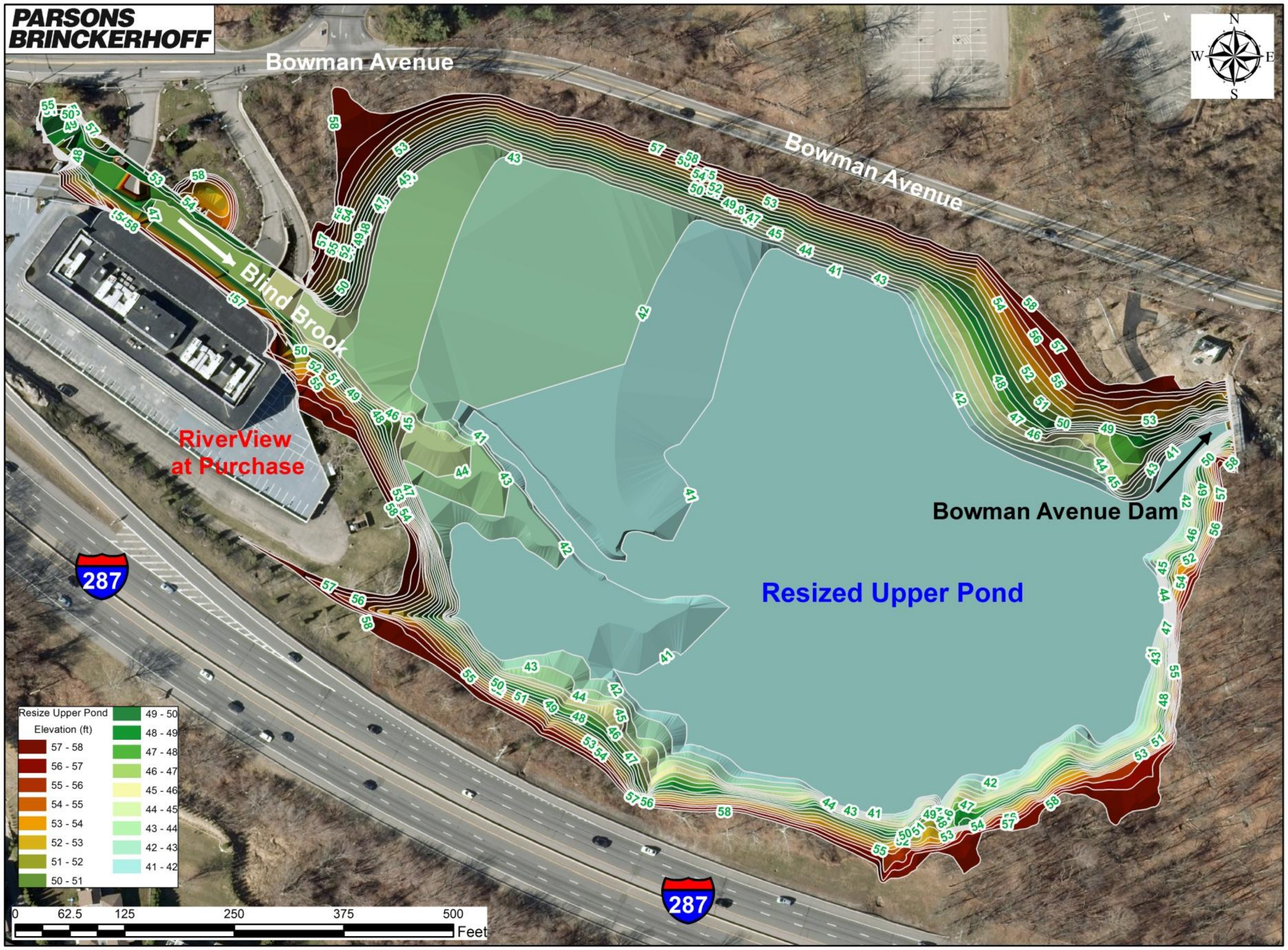


Figure 35 - Proposed TIN and 1-foot Interval Contours of Resize Upper Pond

By comparing figures 35 and 22-1 (RIZZO's study completed in July, 2012) the enclosed areas of 1-foot interval contours, an elevation-area-storage relationship was developed. The area and volume comparisons are shown to be similar between the two separate studies. The maximum difference occurs at Elevation 42 feet, where the percentage area difference and volume difference is 7% and 6% between RIZZO's and PB's computations, respectively as shown in Table 30.

Table 30 - Comparison between Areas & Volume of the Proposed Resized Upper Pond

Elevation	Area			Volume		
	RIZZO	PB	% Difference	RIZZO	PB	% Difference
41	5.59	5.89	5%	0.00	0.00	0
42	7.47	8.00	7%	6.51	6.91	6%
43	9.77	9.81	0%	15.10	15.80	5%
44	10.74	10.76	0%	25.35	26.09	3%
45	11.16	11.18	0%	36.30	37.05	2%
46	11.5	11.50	0%	47.63	48.40	2%
47	11.87	11.88	0%	59.32	60.08	1%
48	12.22	12.28	0%	71.36	72.16	1%
49	12.5	12.62	1%	83.72	84.61	1%
50	12.76	12.89	1%	96.35	97.37	1%
51	13.01	13.13	1%	109.24	110.38	1%
52	13.27	13.42	1%	122.37	123.66	1%
53	13.51	13.68	1%	135.76	137.21	1%
54	13.77	13.97	1%	149.40	151.03	1%
55	14.04	14.26	2%	163.31	165.15	1%
56	14.33	14.58	2%	177.49	179.57	1%
57	14.69	14.94	2%	192.00	194.33	1%
58	15.21	15.50	2%	206.95	209.54	1%

As stated in Sells' 2008 Report, the storage volume for the existing condition of the Upper Pond is 145 acre-feet. By calculating the difference of storage volumes between the existing and proposed ponds gives an estimated excavation quantity as shown in Table 31. This table shows that the excavation quantities are in close agreement between RIZZO's study and PB's study, with only 4% of variance. It should also be noted that the total excavation quantities in RIZZO's presentation and report in 2012 are slightly different than the numbers listed here, the quantities noted in Table 31 were computed based on the elevation vs. area as shown in Figure 22-1.

Table 31 - Excavation Quantity of the Upper Pond (RIZZO vs. PB)

Volume	Existing Pond	Case C : Resize Pond		Excavation Quantity		% Excavation Difference
	Sell Report	RIZZO	PB	RIZZO	PB	
Total (Ac-Ft)	145	206.95	209.54	61.95	64.54	4%
Total (Yd^3)	233,885	333,815	337,992	99,930	104,107	

The total excavation quantity for the Upper pond consists of two major components, rock excavation and soil excavation. For this study, the rock line at only one location was provided as surveyed by RIZZO in 2012; this section is represented by the red line in the upper half of Figure 36. The percentage of the rock was estimated by summarizing the light blue area vs. the total light blue plus the yellow area of the cross section shown in the lower half of the same figure, which includes both soil and rock. An estimated 6% of the total excavation would be rock, while RIZZO's estimation was 12.5%. The deviation of percentages may be due to the use of cross section in a different location other than the one provided.

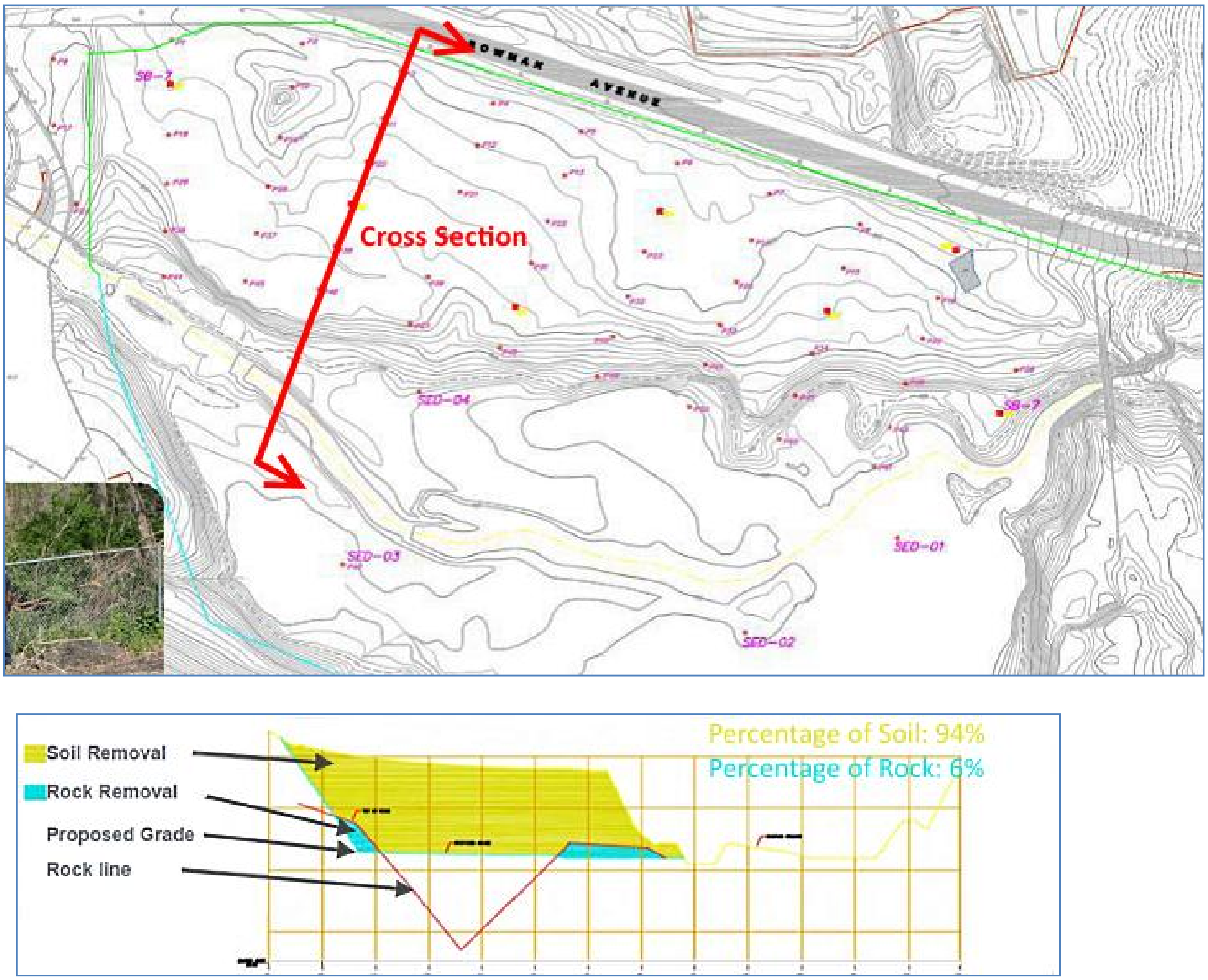


Figure 36 - Cross Section Location and View in Geotechnical Survey of Upper Pond

By using the information at the given location from the geotechnical survey, 6% total of rock excavation, it was estimated that the rock quantity equates to 3.87 acre-feet, which is 50% less than the 7.74 acre-feet computed by RIZZO using the 12.5% rock to total excavation ratio.

Table 32 - Rock/Soil Excavation Quantities of the Upper Pond (RIZZO 12.5% Rock vs. PB 6% Rock)

Volume	Rock to Total Percantage		Rock Quantity		% Rock Difference	Soil Quantity		% Soil Difference
	RIZZO	PB	RIZZO	PB		RIZZO	PB	
Total (Ac-Ft)	12.50%	6.00%	7.74	3.87	-50%	54.21	60.67	12%
Total (Yd^3)			12,491	6,246		87,439	97,861	

The construction costs associated with the resizing the Upper Pond has been provided in tables 33-1 and 33-2. Costs for this study have been obtained from the "*Weighted Average Item Price Report By Item Region and Quarter" (US Customary Contract Let, July 2012 – June 2013)* provided from the Office of Engineering, Design Quality Assurance Bureau, New York State Department of Transportation (NYSDOT) website. This study estimates a total construction cost of 6.1 million dollars for the resizing of the upper pond, which includes major work items such as mobilization, clearing and grubbing, rock/soil excavation, water handling, soil erosion and sediment control. Compared this cost with RIZZO's cost estimate of approximately 7 million dollars, this reports cost estimate is 11% less due to the difference in the amount of rock excavation.

Table 33-1 and Table 33-2. Construction Cost Estimate for the Upper Pond (RIZZO 12.5% Rock vs. PB 6% Rock)

RIZZO Cost Estimate Based on Presentation

Item	Description	Unit	Quantity	Unit Cost	Cost
1	Mobilization	LS	1	100,000	100,000
2	Clearing and Grubbing	AC	15.5	7,800	120,900
3	Rock Excavation	CY	13,660	100	1,366,000
4	Soil Excavation	CY	95,000	40	3,800,000
5	Water Handling	LS	1	100,000	100,000
6	Soil Erosion and Sediment Control	LS	1	200,000	200,000
				Total	5,686,900
				contingencey 20%	1,137,380
				Total	**6,824,280**

PB Cost Estimate

Item	Description	Unit	Quantity	Unit Cost	Cost
1	Mobilization	LS	1	100,000	100,000
2	Clearing and Grubbing	AC	15.5	7,800	120,900
3	Rock Excavation	CY	6,246	100	624,642
4	Soil Excavation	CY	97,861	40	3,914,424
5	Water Handling	LS	1	100,000	100,000
6	Soil Erosion and Sediment Control	LS	1	200,000	200,000
				Total	5,059,966
				contingencey 20%	1,011,993
				Total	**6,071,960**
				Percentage Difference	-11%

If we utilize 12.5% of rock for the total excavation ratio, the estimation of the rock quantity increases to 8.07 acre-feet, which results in a 4% increase in the amount computed by RIZZO. This result is shown in Table 34.

Table 34 - Rock/Soil Excavation Quantities of the Upper Pond (RIZZO vs. PB, Both 12.5% Rock)

Volume	Rock to Total Percantage		Rock Quantity		% Rock Difference	Soil Quantity		% Soil Difference
	RIZZO	PB	RIZZO	PB		RIZZO	PB	
Total (Ac-Ft)	12.50%	12.50%	7.74	8.07	4%	54.21	56.47	4%
Total (Yd^3)			12,491	13,013		87,439	91,094	

The total cost estimate with 12.5% of rock excavation is listed in table 35, the difference between the studies estimate and RIZZO's estimate is 4%.

Table 35 - Construction Cost Estimate for the Upper Pond (RIZZO vs. PB, Both 12.5% Rock)

Item	Description	Unit	Quantity	Unit Cost	Cost
1	Mobilization	LS	1	100,000	100,000
2	Clearing and Grubbing	AC	15.5	7,800	120,900
3	Rock Excavation	CY	13,013	100	1,301,338
4	Soil Excavation	CY	91,094	40	3,643,746
5	Water Handling	LS	1	100,000	100,000
6	Soil Erosion and Sediment Control	LS	1	200,000	200,000
				Total	5,465,984
				contingency 20%	1,093,197
				Total	6,559,181
				Percentage Difference	-4%

In summary, the total cost of resizing the Upper Pond to an average elevation of 41 feet should range between 6.1 million dollars to 6.6 million dollars. This cost estimate included a 20% contingency, and a total rock excavation ratio ranging between 6% to 12.5%.

6.3 Cost Estimate for Detention Ponds at SUNY-Purchase

6.3.1 Introduction

The cost estimate for the two proposed detention ponds at SUNY-Purchase has been performed using the following procedure. 2011 LiDAR data with 1/9 arc resolution (10 feet) was downloaded for the proposed pond area from the USGS National Viewer Website. The vertical elevation of the LiDAR data was converted from meters to feet. By using the *Spatial Analyst* tool, 1-foot interval contour has been created at the location of the proposed ponds. For Pond 1 and Pond 2 on the site of SUNY-Purchase campus, Figure 37 shows the location of two ponds, the maximum inundation areas and the close vicinity areas in three dimensional (3D) view of ArcScene Program.

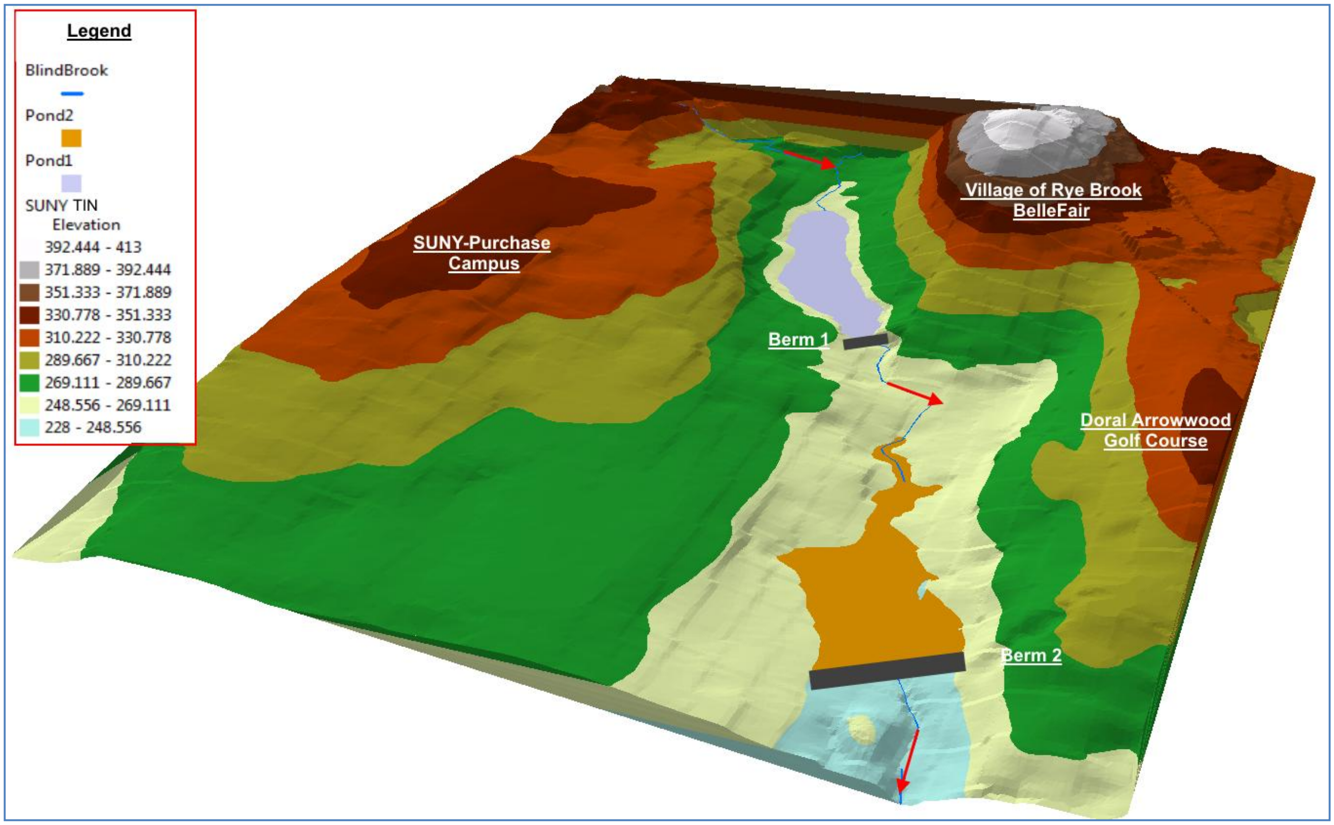


Figure 37 - Location of Two Proposed Ponds in Vicinity of SUNY-Purchase

Due the maximum height of the proposed earth berms for the two ponds, which are 8 feet (pond 1) and 13 feet (pond 2) respectively, Mechanically Stabilized Earth (MSE) Walls will be used to construct the berm. MSE Walls are cost-effective soil-retaining structures that can tolerate much larger settlements than reinforced concrete walls (NYSDOT Geotechnical Engineering Manual, Gem-16 Revision #2, 2007). By placing tensile reinforcing elements (inclusions) in the soil, the strength of the soil can be improved significantly such that the vertical face of the soil/reinforcement system is essentially self supporting. Use of a facing system to prevent soil raveling between the reinforcing elements allows very steep slopes and vertical walls to be constructed safely. In some cases, the inclusions can also withstand bending from shear stresses, providing additional stability to the system.

MSE Walls offer significant technical and cost advantages over conventional reinforced concrete retaining structures at sites with poor foundation conditions. In such cases, the elimination of costs for foundation improvements such as piles and pile caps, that may be required to support of conventional structures, which result in cost savings of greater than 50 percent on completed projects. Some additional successful uses of MSE walls include:

- Temporary structures, which have been especially cost-effective for temporary detours necessary for highway reconstruction projects.
- Reinforced soil dikes, which have been used for containment structures for water and waste impoundments around oil and liquid natural gas storage tanks. (The use of reinforced soil containment dikes is economical and can also result in savings of land because a vertical face can be used, which reduces construction time). This is used for present study.
- Dams and seawalls, including increasing the height of existing dams.
- Bulk materials storage using sloped walls.

Detailed information such as maximum wall height, wall length, maximum inundation area, maximum pond storage volumes of proposed Pond 1 and Pond 2 on the SUNY property are listed in Table 36 below. As it should be noted, both ponds will be dry ponds under normal base flow condition. Pond will only store water for storm events greater than 2-years.

Table 36 - Detailed Information about Pond 1 and 2 on SUNY-Purchase

	Wall Features		Pond Features			
	Wall Length (feet)	Maximum Wall Height (feet)	Lowest Ground Elevation in Pond (feet)	Maximum Water Surface Elevation in Pond (feet)	Maximum Inundation Area (acre)	Maximum Pond Storage (acre-feet)
Pond 1	280	8	258	266	15.50	53.18
Pond 2	820	13	236	249	15.07	65.62

6.3.2 Construction Cost Estimate of Pond 1

Pond 1 will be constructed at an upstream location in close vicinity to the SUNY-Purchase campus. The location of the MSE wall and the maximum inundation area are shown in the Figure 38. As it can be seen from this figure, Pond 1 is located in between SUNY-Purchase and Village of Rye Brook - BelleFair. On the west side of the maximum inundation boundary of the pond, there is a support facility which houses central air conditioner fan units for an office complex, this facility requires flood protection from the inundation. A 3 foot high, 300 foot long proposed flood wall will be constructed to protect the facility.

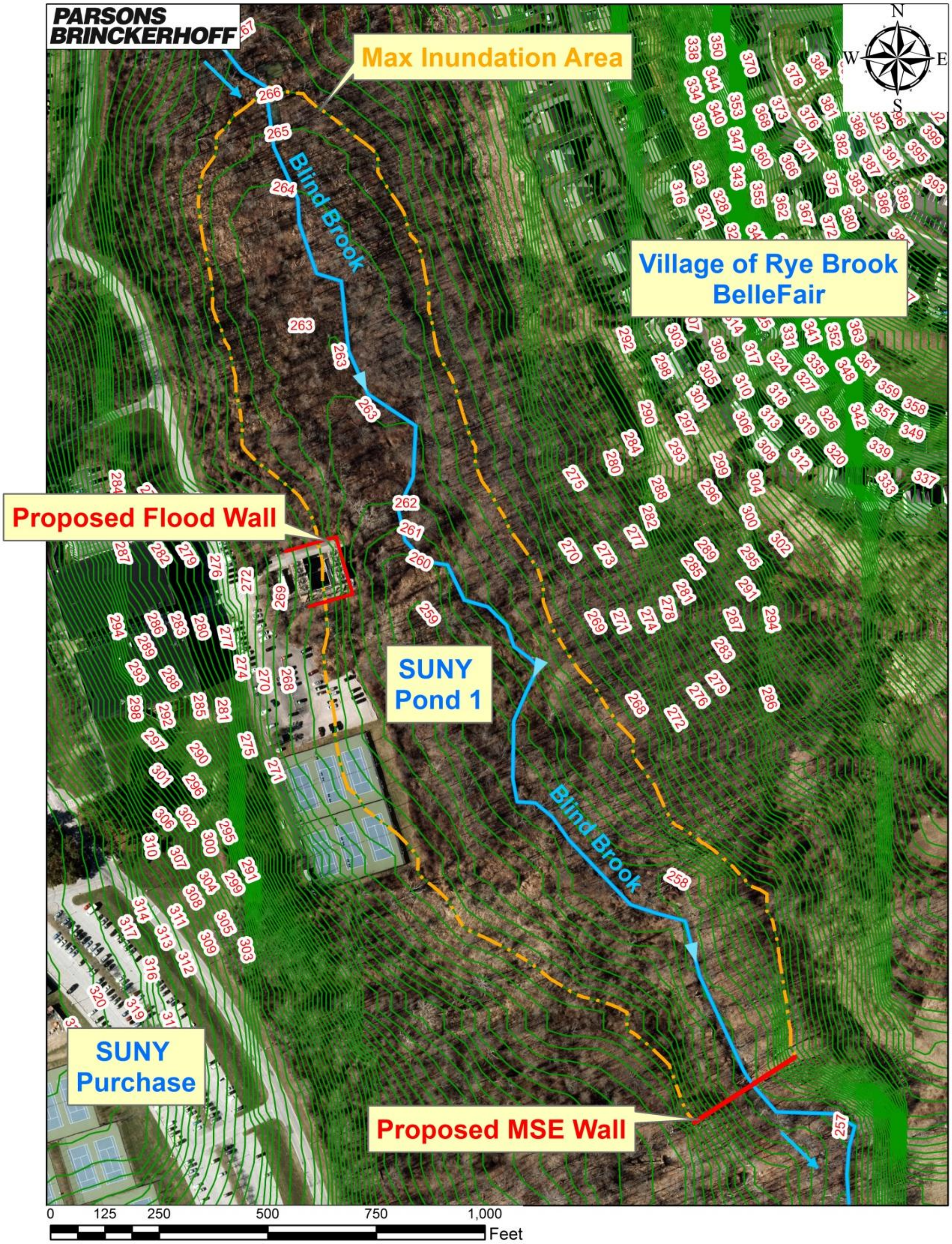


Figure 38 - Proposed Detention Pond 1 (Upstream) in SUNY-Purchase

Using the conic volume computational method, the elevation area and storage volume table for Pond 1 was developed and is provided in Table 37, as well as the graphical representation in Figure 39.

Table 37 - Elevation-Area-Volume Computation for SUNY Pond 1

Elevation	Area	ΔV	Total Volume
(ft)	(acre)	(ac-ft)	(ac-ft)
258	0.31	0.00	0.00
259	1.76	0.94	0.94
260	3.05	2.38	3.31
261	4.21	3.62	6.93
262	5.47	4.83	11.76
263	7.25	6.34	18.10
264	10.63	8.89	26.98
265	13.17	11.88	38.86
266	15.50	14.32	53.18

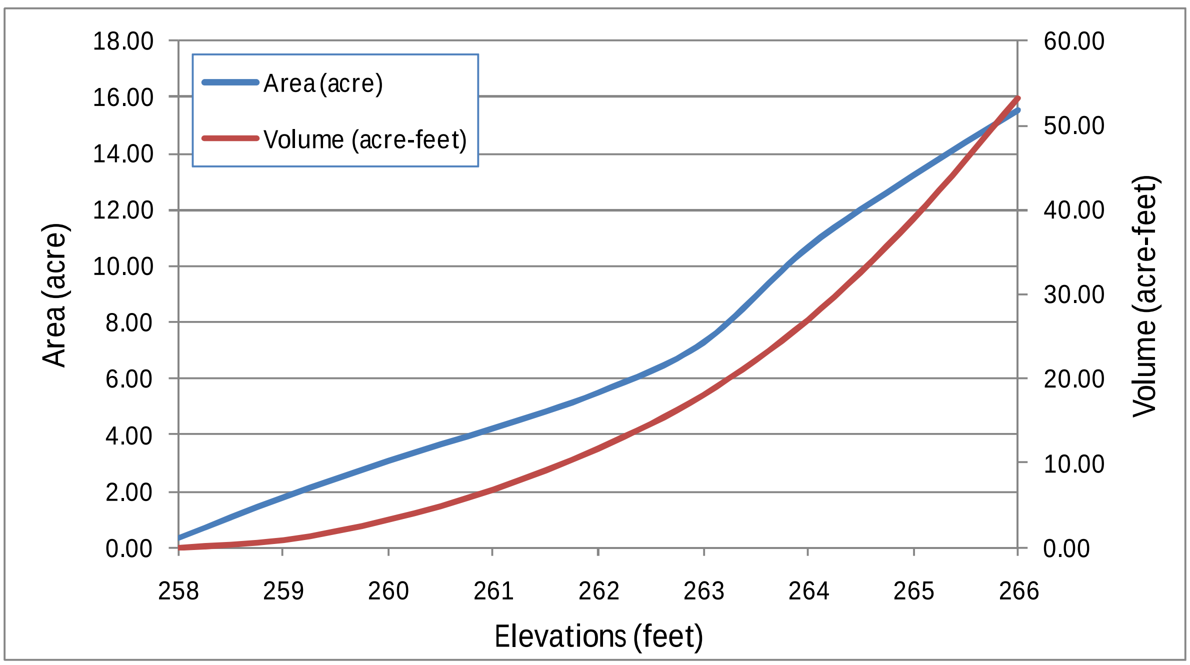


Figure 39 - Elevation-Area & Elevation-Storage Curves for Proposed Detention Pond 1

It should be noted that the height of the MSE Wall is not a constant. The height of the wall will gradually decrease from 8 feet in the vicinity of the main channel to zero on the outer edge of the floodplain. By assuming a 1:1 side slope of the MSE Wall, the face area can be computed based on the cross section plot of the ground surface and the top of the wall elevation of 266 feet. The front surface area of the wall is computed to be 2,084 square feet, or 193.71 square meters. In close vicinity to the main channel, there will be a

3 feet high by 20 feet wide opening in the wall to allow the low flow of Blind Brook to pass downstream.

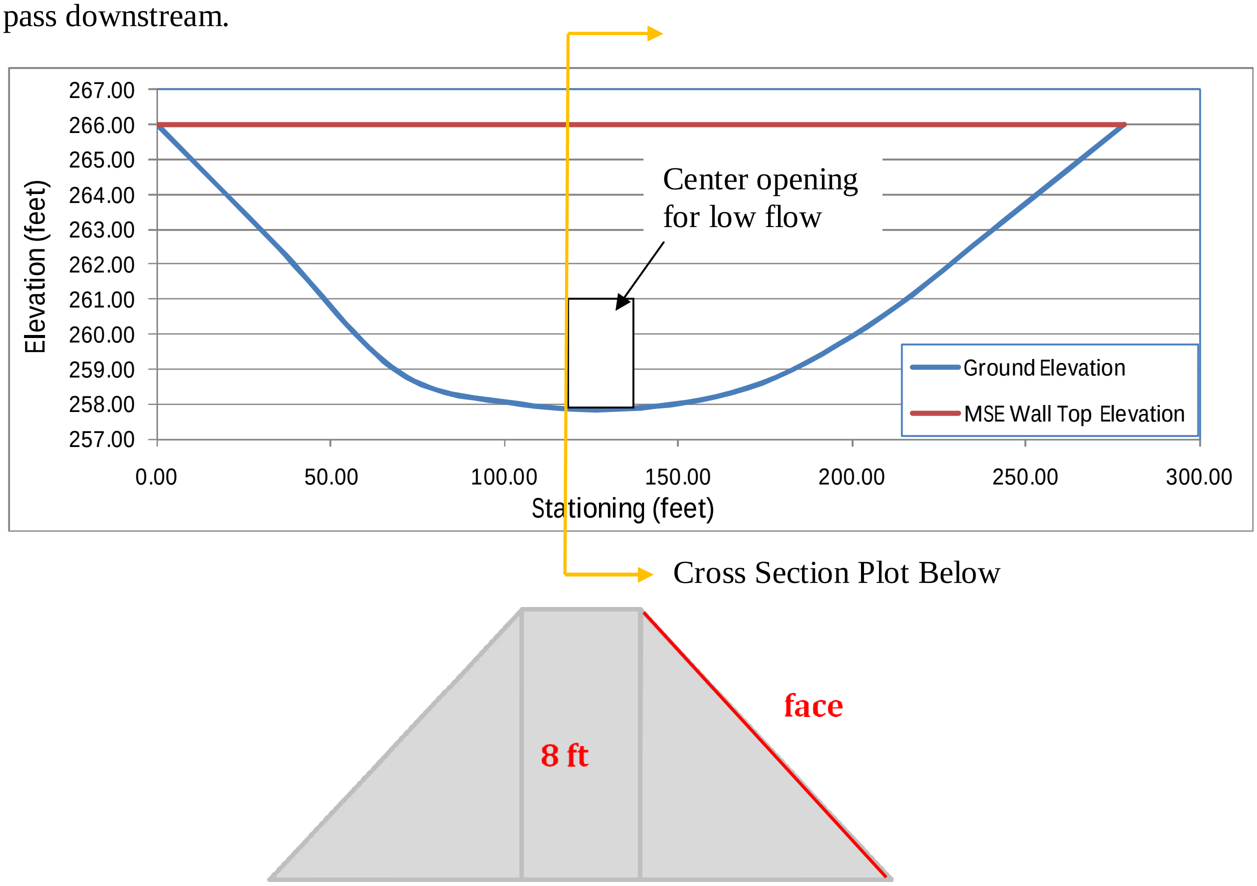


Figure 40 - Face View of the MSE Wall and Cross Section of Proposed Detention Pond 1

Table 38 - Computation of the Face Area of MSE Wall of Pond 1

X	Z_{ground}	Z_{wall}	ΔArea	Face Area
ft	ft	ft	ft^2	ft^2
0.00	266.00	266	0.00	0
34.81	262.49	266	86.42	86
69.62	258.92	266	260.76	347
104.43	258.01	266	371.07	718
139.24	257.90	266	396.18	1,114
174.05	258.60	266	381.57	1,496
208.87	260.51	266	317.14	1,813
243.68	263.24	266	203.01	2,016
278.49	266.00	266	67.90	2,084

In NYSDOT's Geotechnical Manual (2007), site specific costs of a MSE Wall is a function of many factors, including cut-fill requirements, wall/slope size and type, in-situ soil type, available backfill materials, facing finish, and if the wall is a temporary or a permanent application. It has been found that MSE Walls with precast concrete facings are usually less expensive than reinforced concrete retaining walls for heights greater than about 3 m (10 ft) and average foundation conditions. Modular Block Walls (MBW) is competitive with concrete walls at heights of less than 4.5 m (15 ft).

In general, the use of MSE walls results in savings on the order of 25 to 50 percent and possibly more in comparison with a conventional reinforced concrete retaining structure, especially when the latter is supported on a deep foundation system (poor foundation condition). A substantial savings is obtained by the elimination of the deep foundations, which is usually possible because reinforced soil structures can accommodate relatively large amounts of total and differential settlements. Other cost saving features included ease and speed of construction. A comparison of wall material and erection costs for several reinforced soil retaining walls and other retaining wall systems, based on a survey of state and federal transportation agencies, is shown in Figure 41. Typical total costs for MSE Walls range from $200 to $400 per square meters, or $19 to $37 per square feet of face area, generally as function of height, size of project and cost of select fill.

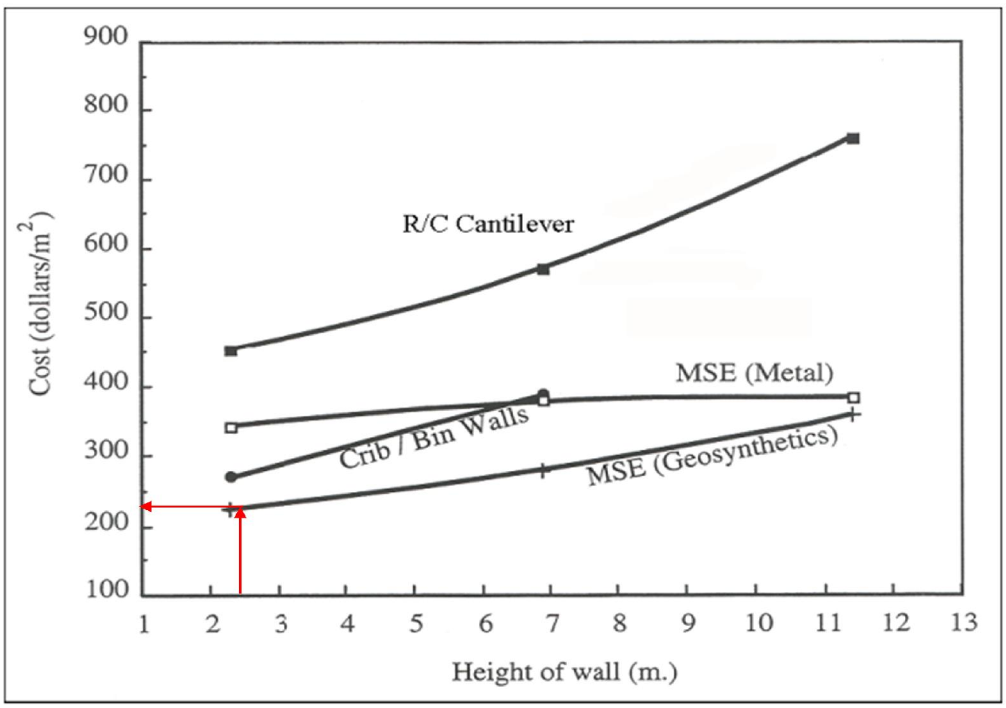


Figure 41 - Cost Estimate Based on Height of MSE Wall for Pond 1

From the figure above, the unit cost based on the height of the wall is 235 dollars per square meters, or 22 dollars per square feet. The cost of constructing the MSE wall will be: $22 / ft^2 x 2,084 ft^2 = $45,848. The total cost was computed in the following table. The total cost for constructing detention Pond 1 would be approximately $143,000.

Table 39 - Cost Estimate for Detention Pond 1 in SUNY-Purchase

Item	Description	Unit	Quantity	Unit Cost	Cost
1	Mobilization	LS	1	10,000	10,000
2	Clearing and Grubbing	AC	0.5	7,800	3,900
3	MSE Wall	LS	1	45,848	45,848
4	Soil Erosion and Sediment Control	LS	1	20,000	20,000
5	Flood Wall	SF	900	44	39,285
				Total	119,033
				Contingencey 20%	23,807
				Total	142,840

6.3.3 Construction Cost Estimate of Pond 2

Pond 2 will be constructed approximately 0.76 mile downstream of Pond 1 on the main stem of Blind Brook. The location of the MSE Wall and the maximum inundation area is shown in the Figure 42. As it can be seen from the figure, Pond 2 is located in between SUNY-Purchase and Doral Arrowwood Golf Course, in the Village of Rye Brook. The main channel of Blind Brook is also the property boundary line for the two properties mentioned above.

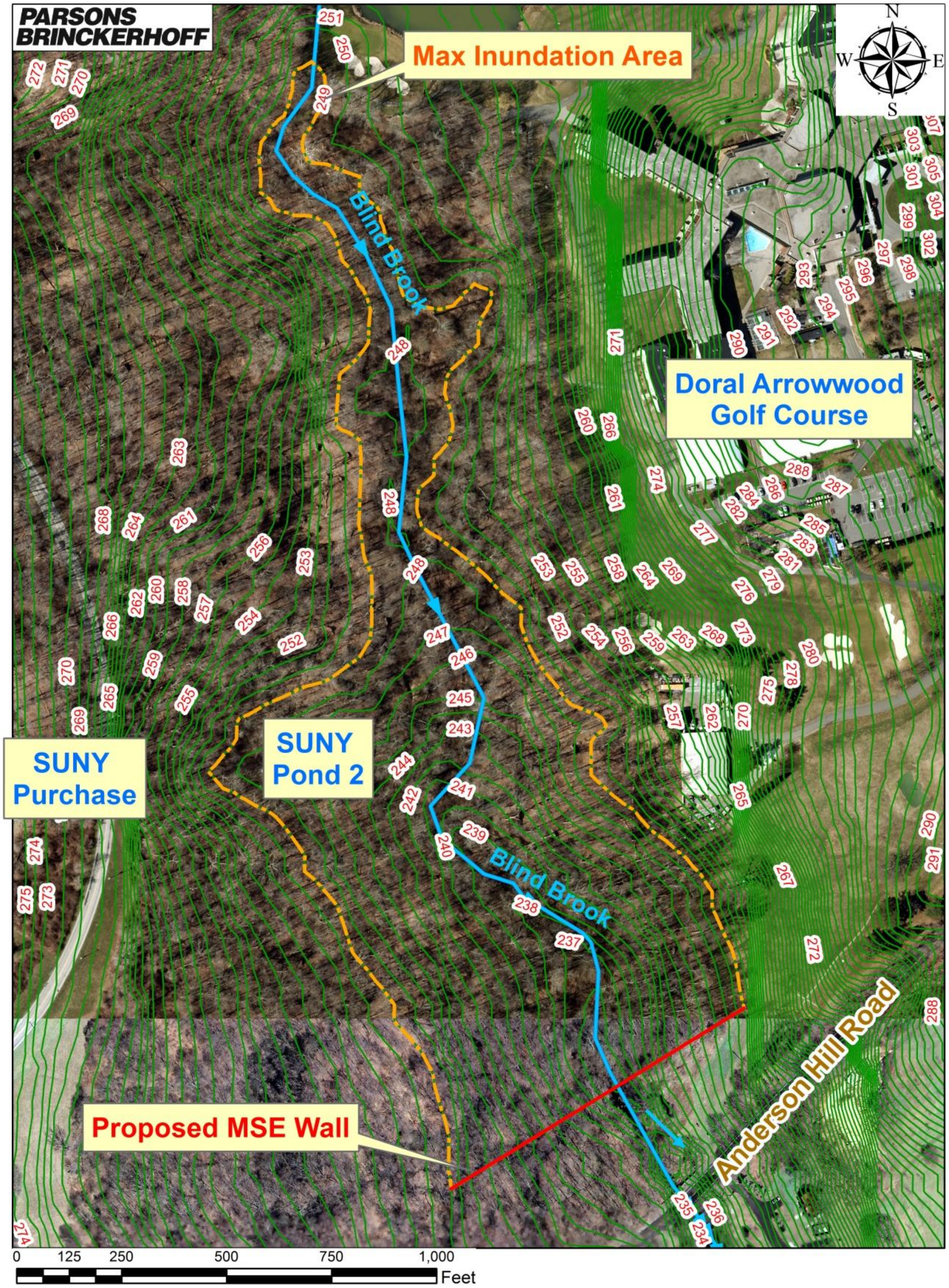


Figure 42 - Proposed Detention Pond 2 (Downstream) in SUNY-Purchase

Using the conic volume computational method, the elevation area and storage table for Detention Pond 2 was developed and provided in Table 40 below, as well as the graphical representation in Figure 43.

Table 40 - Elevation-Area-Volume Computation for SUNY Pond 2

Elevation	Area	ΔV	Total Volume
(ft)	(acre)	(ac-ft)	(ac-ft)
236	0.20	0.00	0.00
237	0.56	0.36	0.36
238	0.94	0.74	1.11
239	1.61	1.26	2.36
240	2.28	1.93	4.30
241	2.97	2.62	6.92
242	3.73	3.34	10.26
243	4.59	4.15	14.41
244	5.49	5.03	19.44
245	6.49	5.98	25.42
246	7.75	7.11	32.53
247	9.05	8.39	40.92
248	12.70	10.83	51.75
249	15.07	13.87	65.62

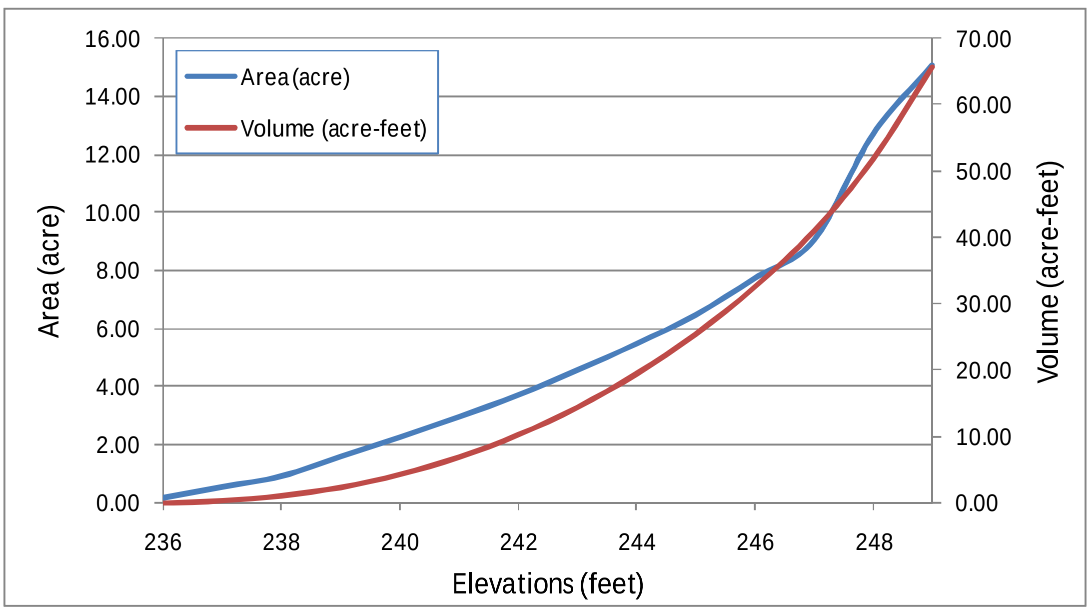


Figure 43 - Elevation-Area and Elevation-Storage Curves for Proposed Detention Pond 2

It should be noted that the height of the MSE Wall is not a constant. The height of the wall will gradually decrease from 13 feet in close vicinity of the main channel to zero on the outer edge of the floodplain. At station 410 feet measured from the center line of the main channel, the wall will taper into the existing ground.

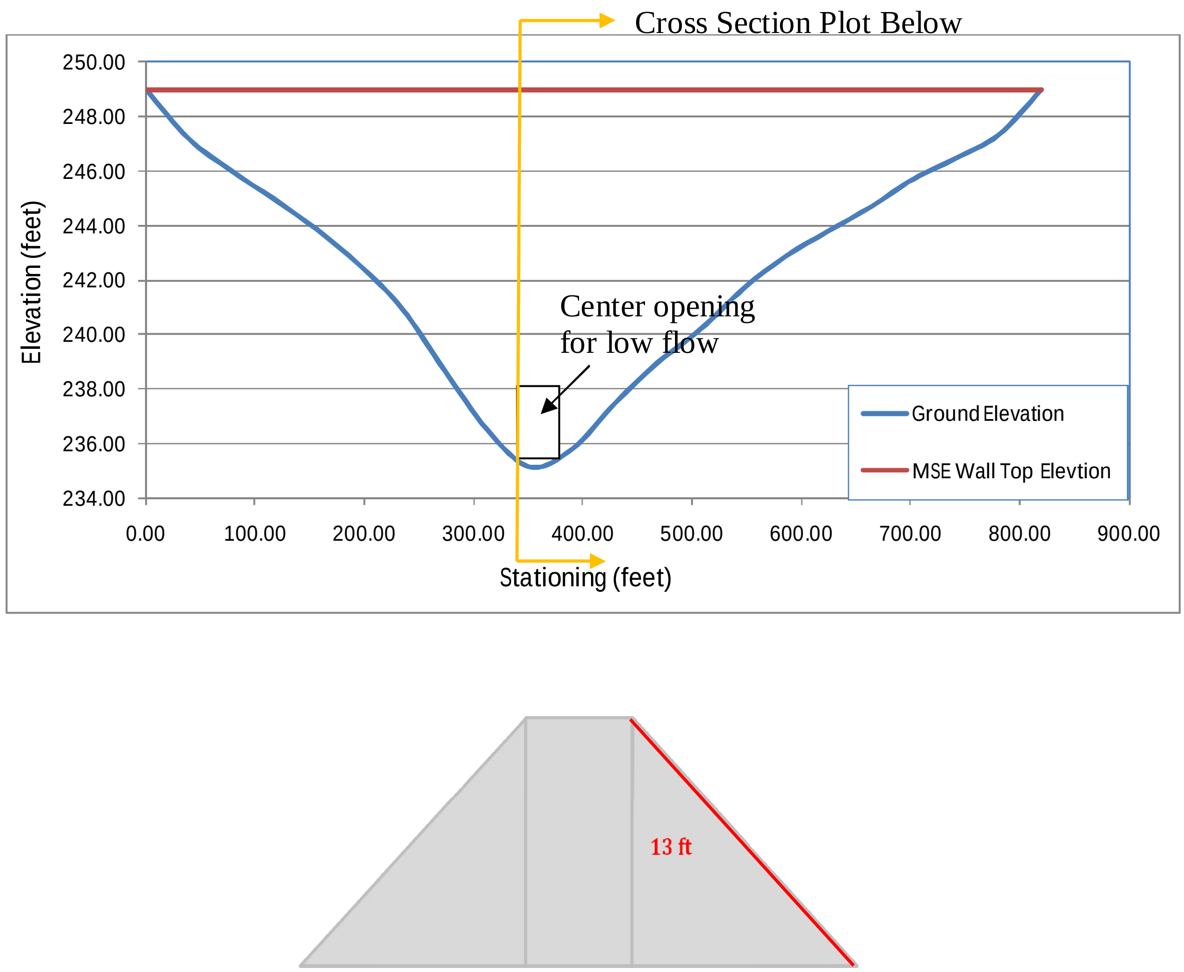


Figure 44 - Face View of the MSE Wall & Cross Section of Proposed Detention Pond 2

By assuming a 1:1 side slope of the MSE wall, the face area of wall is computed based on the cross section plot and the top of the wall elevation of 249 feet. The front surface area of the wall was computed to be 7,683 square feet, or 714 square meters. In close vicinity to the main channel, there will be a 3 feet high by 20 feet wide opening in the wall to allow the low flow of Blind Brook to pass downstream.

Table 41 - Computation of the Face Area of MSE Wall of Pond 2

X	Z_{ground}	Z_{wall}	**Δ**Area	Face Area
ft	ft	ft	ft^2	ft^2
0.00	249.00	249	0.00	0
39.04	247.20	249	49.81	50
78.08	246.01	249	132.22	182
117.12	244.98	249	193.37	375
156.16	243.86	249	252.94	628
195.20	242.56	249	319.81	948
234.24	240.95	249	399.97	1,348
273.28	238.68	249	507.14	1,855
312.32	236.49	249	630.23	2,485
351.36	235.12	249	728.30	3,214
390.40	235.80	249	747.51	3,961
429.44	237.47	249	682.70	4,644
468.48	238.96	249	595.35	5,239
507.52	240.18	249	520.49	5,760
546.56	241.67	249	445.81	6,206
585.60	242.88	249	371.44	6,577
624.63	243.80	249	312.57	6,890
663.67	244.69	249	262.49	7,152
702.71	245.72	249	209.48	7,362
741.75	246.48	249	160.14	7,522
780.79	247.35	249	115.29	7,637
819.83	249.00	249	45.69	7,683

Based on NYSDOT Geotechnical Manual (2007), from the same figure used to complete the cost of Pond 1, the unit cost of Pond 2 with 13 feet wall height is 250 dollars per square meters, or 23 dollars per square feet. The cost of constructing the MSE Wall will be: \$23 / ft^2 x 7,683 ft^2 = \$178,508. The total cost was computed in the following table. The total cost for constructing detention Pond 2 would be approximately \$368,000.

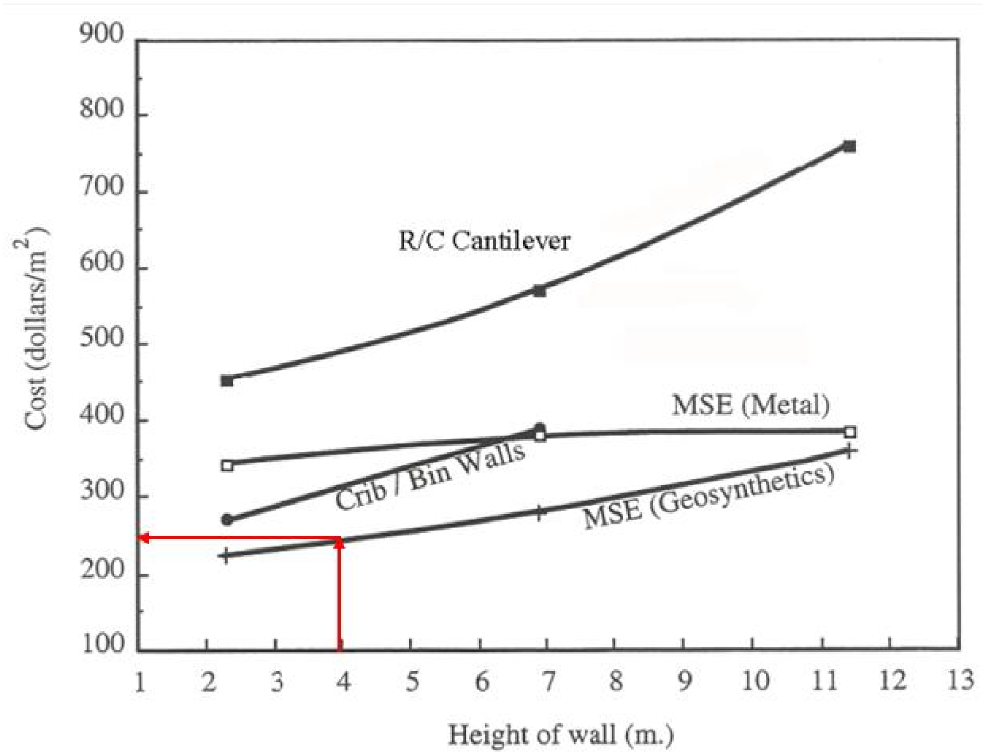


Figure 45 - Cost Estimate Based on Height of MSE Wall of Pond 2

Table 42 - Cost Estimate for Pond 2 in SUNY-Purchase

Item	Description	Unit	Quantity	Unit Cost	Cost
1	Mobilization	LS	1	40,000	40,000
2	Clearing and Grubbing	AC	1	7,800	7,800
3	MSE Wall	LS	1	178,508	178,508
4	Soil Erosion and Sediment Control	LS	1	80,000	80,000
				Total	306,308
				Contingencey 20%	61,262
				Total	367,570

6.3.4 Construction Cost Summary for SUNY-Purchase Ponds

By using the *Weighted Average Item Price Report By Item Region and Quarter (US Customary Contract Let, July 2012 – June 2013)* from the Office of Engineering, Design Quality Assurance Bureau, New York State Department of Transportation (NYSDOT) website, the total cost for constructing the two detention ponds on the SUNY-Purchase will be approximately \$143,000 + \$368,000 = \$511,000. The major construction work includes building two MSE Walls with maximum height ranging from 8 feet to 13 feet, and the length of 280 feet and 820 feet respectively. For Pond 1, construction of the 300 ft long, 3 ft high flood wall along the a support facility located on the SUNY Purchase campus is needed to protect the property from being flood during larger storm events.

6.4 Summary Table of Cost and Water Surface Elevation Reduction

Based on the hydraulic analysis in the previous chapters and the cost estimate in this chapter, the cost and water surface elevation reduction comparison table is listed below. The water surface elevation reduction refers to the comparison between proposed improvements of the ponds vs. the existing condition with gate and gauge at the dam, but assume no operation of the gate (gate is closed). Additional water surface elevation reductions would be realized if relocation of the gauge at Bowman Avenue Dam is considered. No cost estimate has been developed for this alternative yet.

Table 43 - Cost and Water Surface Elevation Reductions of Two Proposed Options

Return Periods	Locations	SUNY Detention Ponds		Resize Upper Pond	
		Cost	Water Surface Elevation Reduction	Cost	Water Surface Elevation Reduction
2-Year Storm	D/S I-287	0.51 Million Dollars	-0.10	6.1 ~ 6.6 Million Dollars	-0.14
	Purchase St		-0.09		-0.14
	Mendota Avenue		-0.09		-0.13
	Highland Road		-0.09		-0.13
	U/S I-95		-0.07		-0.10
10-Year Storm	D/S I-287		-0.91		-0.47
	Purchase St		-1.60		-0.80
	Mendota Avenue		-1.36		-0.74
	Highland Road		-1.40		-0.75
	U/S I-95		-1.70		-1.00
50-Year Storm	D/S I-287		-0.52		-0.33
	Purchase St		-0.90		-0.64
	Mendota Avenue		-1.42		-1.23
	Highland Road		-1.43		-1.23
	U/S I-95		-1.51		-1.30
100-Year Storm	D/S I-287		-0.36		-0.10
	Purchase St		-0.72		-0.32
	Mendota Avenue		-1.22		-0.60
	Highland Road		-1.22		-0.60
	U/S I-95		-1.31		-0.63

7. Conclusions & Recommended Next Step

From the review of the reports and our analysis of the hydrology, river hydraulics, and structure hydraulics, Parsons Brinckerhoff (PB) provided the following conclusions summarizing our findings:

1. PB reviewed and analyzed six reports previously submitted to the City of Rye. For each report, PB summarized the purpose of objectives, findings and recommendations, hydrologic and hydraulic modeling method are provided for each report. A summary of the improvements with a list tabulating the hydraulic parameters, such as flow discharges and water surface elevations for a given storm event, was also provided at the end of each report review.

2. Based on TRC's 2010 hydrologic model of the Westchester County Airport and RIZZO's 2012 hydrologic model of Blind Brook watershed, a set of new hydrologic models for the 5.38 square miles Upper Blind Brook Watershed were created based on ten potential detention areas. This model used further subdivided watershed into seven sub-watershed new areas (SW1-Airport, SW1-SUNY, SW1-PepsiCo, SW2-U/S, SW2-Edgar Bronfman Lake, SW2-D/S and SW2-Hutchinson River Parkway) to better represent each contributing area for the potential detention basin. The models were first run for the Existing Conditions (2010) as noted in TRC report, and the results were found to be in good agreement with those results from the hydrologic models in RIZZO report with two sub-watersheds approach.

3. By using GIS mapping and the information obtained from a field visit to the Blind Brook, PB selected ten potential detention areas to study the detention effect on the flood peak discharges in the following five regions, SW1-Airport (2 detentions), SW1-SUNY (2 detentions), SW1-PepsiCo (1 detention), SW2-Hutchinson River Parkway (3 detentions) and SW2 (2 detentions). The addition of the identified detention areas along the Blind Brook were evaluated first individually; it was found that the SW1-SUNY detention basins and SW2 detention basins would provide most significant water surface elevation reductions at five downstream locations. For SW1-SUNY (2 detentions) basin, the maximum water surface elevation reduction is 1.70 ft, upstream of I-95 for the 10-year flood. For SW2 (2 detentions) basin, the maximum water surface elevation reduction is 1.46 ft upstream of I-95 for the 10-year flood. Then two most effective detention regions were also evaluated cumulatively to provide a sense of incremental benefits of implementation over time. The maximum water surface elevation reduction is 3.25 ft at Highland Road for the 10-year flood for cumulative detention analysis.

4. Based on the request from the City of Rye at the meeting on January 31, 2013, PB studied the effect of resize/max the upper pond at the Bowman Avenue Dam. Two alternatives were analyzed that examined the effects of increasing the storage volume of the upper pond on the downstream water surface elevation for various storm events. The revised HEC-RAS geometry files for the resized upper pond

and maximize upper pond were input into the HEC-RAS model, and run for the proposed condition 2, 10, 50 and 100-year flood. Hydraulic analysis results showed that between the two resized pond alternatives, Cases C and D, the incremental benefit gained with the maximized resized alternative (Case D) is insignificant. By implementing the smaller resized pond alternative (Case C), potential water elevations are 0.1~ ft lower for 2-year flood, 0.4 to 1.0 ft lower for 10-year floods, 0.3 to 1.3 ft lower for 50-year flood, 0.1 to 0.6 ft lower for 100-year flood.

5. After consulting with engineers from the City of Rye, PB studied two scenarios of a new operational rule of the sluice gate. The location of the gauge for measuring water surface elevations used to determine opening or closing of the sluice gate would be moved to a point downstream of the dam, and upstream of the flood prone areas of Indian Village (upstream of I-95) and downstream of I-287. By using this approach, the maximum water surface elevations would be reduced at all downstream locations. The optimal elevation to close sluice gate for each storm event is obtained by analyzing the existing conditions maximum water surface elevation for the corresponding storms at the gauge location. The final optimal water surface elevations when the gate is closed was found by using a trail-and-error process for a range of value varying between 3 ft below the peak existing elevation to 1 ft below existing peak water surface elevation. The final result showed that the water surface elevation reductions have been increased for 10, 25, 50 and 100-year storm when compared with RIZZO's study. Especially for 50-year flood, at Mendota Avenue, Highland Road and U/S of I-95, the water surface elevation reductions almost tripled when compared with RIZZO's results at the same locations. The maximum water surface elevation reduction is 1.48 ft at both Mendota Avenue and Highland Road for the 50-year flood. Operation of Gauge based on water surface elevations at downstream of I-287 provide more water surface elevation reductions at all downstream locations for 5, 10 and 25-year storms than the gauge location at Indian Village.

6. PB studied the cumulative effects of the resize/max Upper Pond with the optimal sluice gate operations, SW1-SUNY detention plus resize/max Upper Pond with the optimal sluice gate operations respectively based on water surface elevations at downstream of I-287 and Indian Village. With SW1-SUNY detention, resize/max Upper Pond and the optimal gate operations based on the water surface elevations at Indian Village or downstream of I-287, we can obtain additional water surface elevation reductions at all downstream locations for all storm events studied.

7. PB also performed the construction cost estimate for resizing the Upper Pond and two detention ponds on SUNY-Purchase. The cost for resizing Upper Pond is ranging from 6.1 million dollars to 6.6 million dollars. The cost for two detention ponds on SUNY-Purchase is approximately 0.51 million dollars. The cost and water surface elevation reductions table is provided for those two proposed improvements.

The recommended next steps are:

1. Obtain surveyed stream cross section survey to improve the accuracy of the hydraulic model, since currently the topographic data was taken from LiDAR and doesn't contain the detailed geometry of the stream cross section below the water surface.

2. Install the stream gauges along Blind Brook main stem. Hydrologic and hydraulic models could then be calibrated more precisely with measured discharges and water surface elevation data to better represent the existing condition.

3. Develop detailed detention pond grading plans, outfall structures, and elevation-storage-discharge relationships for the selected potential detention areas.

References

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Appendix A

Watershed Map

Web Soil Survey

Appendix B

Hydrologic Model – HydroCAD Model

Appendix B.1

- **Existing Condition (2010)**
- **Future Condition (2011)**

Appendix B.2

- **SW1-Airport Detention**
- **SW1-SUNY Detention**
- **SW1-PepsiCo Detention**
- **SW1- Hutchinson River Parkway Detention**
- **SW2 Detention**

Appendix B.3

- **SW1-SUNY + SW2 Detention**

Appendix C

Hydraulic Model – HEC-RAS Model

Appendix C.1

Existing Condition

Appendix C.2

- **SW1-Airport Detention**
- **SW1-SUNY Detention**
- **SW1-PepsiCo Detention**
- **SW1- Hutchinson River Parkway Detention**
- **SW2 Detention**
- **SW1-SUNY + SW2 Detention**

Appendix C.3

- **RIZZO Resize Upper Pond**
- **RIZZO Max Upper Pond**

Appendix C.4

Optimal Gate Operations Based on Water Surface Elevations at Indian Village

Appendix C.5

Optimal Gate Operations Based on Water Surface Elevations at Downstream of I-287

Appendix C.6

- **RIZZO Resize Upper Pond + Optimal Gate Operations Based on Water Surface Elevations at Indian Village**
- **RIZZO Max Upper Pond + Optimal Gate Operations Based on Water Surface Elevations at Indian Village**
- **SW1-SUNY + RIZZO Resize Upper Pond + Optimal Gate Operations Based on Water Surface Elevations at Indian Village**
- **SW1-SUNY + RIZZO Max Upper Pond + Optimal Gate Operations Based on Water Surface Elevations at Indian Village**

Appendix C.7

- **RIZZO Resize Upper Pond + Optimal Gate Operations Based on Water Surface Elevations at Downstream of I-287**
- **RIZZO Max Upper Pond + Optimal Gate Operations Based on Water Surface Elevations at Downstream of I-287**
- **SW1-SUNY + RIZZO Resize Upper Pond + Optimal Gate Operations Based on Water Surface Elevations at Downstream of I-287**
- **SW1-SUNY + RIZZO Max Upper Pond + Optimal Gate Operations Based on Water Surface Elevations at Downstream of I-287**

Appendix D

Proposed Detention Areas