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WATERSHED MODELING USING HEC-RAS, HEC-HMS, AND GIS MODELS – A CASE STUDY OF THE WRECK POND BROOK WATERSHED IN MONMOUTH COUNTY, NEW JERSEY

by

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And approved by

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ABSTRACT OF THE DISSERTATION

WATERSHED MODELING USING HEC-RAS, HEC-HMS, AND GIS MODELS – A CASE STUDY OF THE WRECK POND BROOK WATERSHED IN MONMOUTH COUNTY, NEW JERSEY

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Dissertation Director:

Dr. Monica A. Mazurek

Managing stormwater on a watershed basis is considered the best strategy to address flooding. A watershed model should provide temporal and spatial distribution of runoff response for a given storm. GIS applicability in watershed modeling is increasing due to the availability of spatial information, fast processors and interfaces such as ArcHydro, HEC-GeoHMS, and HEC-GeoRAS linking hydrologic and hydraulic models to the ArcGIS environment.

Soil Conservation Service methods are used widely in hydrologic models. Several parameters inherent to these empirical methods are average values derived from various watershed conditions. These average values overestimate peak flows for flat, low-lying coastal terrains. The design of flood control structures based on these flow values allow more post-development discharge, make the system more hydraulically efficient, increase project costs, and cause flooding for areas downstream. In this study, Wreck Pond Brook Watershed (WPBW), a coastal New Jersey area was used for

sensitivity studies of the initial abstraction ratio and peak rate factor. The HEC-HMS modeling results indicated use of a lower peak rate factor (e.g. 284) and 5% initial abstraction ratio provided better characterization of stream response. These updated parameters provide new technical information for improving stormwater management in coastal areas.

An important limitation in hydraulic modeling is the economic constraint on cross-section spacing for surveying channels and floodplains. Applying GIS techniques in hydraulic modeling eliminated this constraint. Floodplain analysis was done using ArcGIS, HEC-GeoRAS and HEC-RAS. Detailed elevation data (LIDAR information from Monmouth County) was incorporated into the HEC-RAS using GIS models. This innovation was important for improving model efficiency. The modeled floodplain demonstrated close agreement to the observed floodplain for the October 2005 storm and showed greater accuracy compared to the FEMA floodplain for the 100-year storm. This study validated use of LIDAR elevation data in floodplain analysis for the second-order streams in coastal NJ.

Finally, an approach was demonstrated using modeled floodplain and HEC-HMS for flood control analysis. This study presents an innovative watershed modeling approach using GIS models while addressing the limitations of traditional hydrologic and hydraulic methods using WPBW as an example.

ACKNOWLEDGEMENT

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I would also like to thank colleagues at New Jersey Department of Environmental Protection and the members of the Wreck Pond Brook Regional Stormwater Management Planning Committee for their direct and indirect contribution to the project.

I express my sincere gratitude to my family for their continuous love and encouragement during my dissertation research.

DISCLAIMER

The tidal effect of the Atlantic Ocean is not considered in this study. It also should be noted that some of the analyses in this study are based on limited information and hypothetical elements (e.g. outlet structures for online ponds). The intention here is to provide an example for approaches that can be used to address flooding issues in WPBW. Any design and construction projects should not rely solely on this information and should be based on its own detailed investigation. Watershed characteristics and stream routing parameters in the existing model (Part of the Wreck Pond Brook Watershed Regional stormwater management plan) were re-evaluated up to the confluence of Wreck Pond Brook and Hannabrand Brook at the Old Mill Road culvert. The floodplains developed in this study should not be used for any regulatory requirements and only official NJ State and FEMA flood insurance rate maps should be used.

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GLOSSARY

BMP:	Best Management Practices
CN:	Curve Number
DEM:	Digital Elevation Model
DFIRM:	Digital Flood Insurance Rate Maps
DTM:	Digital Terrain Model
DUH:	Dimensionless Unit Hydrograph
FEMA:	Federal Emergency Management Agency
HEC-HMS:	Hydrologic Engineering Centers Hydrologic Modeling System
HEC-RAS:	Hydrologic Engineering Centers River Analysis System
HRU:	Hydrologic Response Unit
HSG:	Hydrologic Soil Group
Ia:	Initial Abstraction
n:	Manning's roughness coefficient
NEXRAD:	Next Generation Radar
NJDA:	New Jersey Department of Agriculture
NJDEP:	New Jersey Department of Environmental Protection
NOAA:	National Oceanic and Atmospheric Administration
NRCS:	Natural Resources Conservation Service
NWS:	National Weather Service
P:	Precipitation
PRF:	Peak Rate Factor
LMDUH:	Lower Monmouth Dimensionless Unit Hydrograph

RSWMP:	Regional Stormwater Management Plan
S:	Watershed Storage
SCS:	Soil Conservation Service
Tc:	Time of Concentration
TIN:	Triangulated Irregular Network
Tl:	Lag Time
Tp:	Time to Peak
UH:	Unit Hydrograph
USACE:	United States Army Corps of Engineers
USGS:	United States Geological Survey
WPBW:	Wreck Pond Brook Watershed

1. INTRODUCTION

1.1 Research Objectives and Importance

"Although loss of life to floods during the past half-century has declined, mostly because of improved warning systems, economic losses have continued to rise due to increased urbanization and coastal development." (Source: USGS Fact sheet, 2006-3026)

The population of New Jersey has grown from 4.8 million to 8.4 million from 1950 to 2000 (U.S. Bureau of Census, 2003) causing increased stress on the State's water resources (USGS, 2005). This stress is compounded further by global warming impacts including increased precipitation and storm intensity and rising seawater levels for coastal and low-lying areas (IPCC, Assessment Report 4, 2007). Due to such global and local changes in the hydrologic cycle and sea-levels, the need for accurate and reliable predictions of the water environment responses is a pressing societal need (SEDS, 2008). Flooding is described as New Jersey's number one natural hazard (FEMA, 2004). Hence, reliable hydrologic models are necessary to predict flood levels and damages in light of increased population growth in low-lying coastal areas and a warming global climate. This study has developed and evaluated models for the Wreck Pond Brook Watershed (WPBW) under high and low flow events. The storm event of October 2005 was used to model high flow conditions. The model was calibrated and validated using Antecedent Moisture Condition II and III. Hence, the developed model encompasses various scenarios of rainfall and watershed conditions.

NJ is at the forefront of water quantity and quality issues with its streams and lakes due to its high population density and strong development pressure. Consequently, the problems in NJ's watersheds demand more detailed and relevant studies and also greater scientific innovation as far as addressing water quantity and quality issues. Identification and verification of the regional parameters for coastal watersheds in NJ are the first result of this research.

One of the most important parameters in the design of the flood control structures and bridges and culverts, is the design flow value. Many government agencies either mandate or recommend using Soil Conservation Service (SCS) methods for calculation of design flows. For new construction projects, many agencies require matching the pre-development peak flow values or some percentage reduction of it. The SCS dimensionless unit hydrograph and the SCS runoff curve number method are used widely for calculating the peak flow value and total runoff volume, respectively. The peak rate coefficient in the SCS dimensionless unit hydrograph, also called the peak rate factor (PRF), affects the peak design flow value, thereby, affecting the sizing of flood control structures. The PRF value of 484 is inherent in the use of the SCS standard dimensionless unit hydrograph method. This PRF value overestimates the peak flows for flat, low-lying coastal terrains. This overestimation allows more post-development discharge and requires over sizing of drainage systems. Such designs increase the cost of the project, make the system more hydraulically efficient, and cause flooding for the downstream areas. A goal of this research was to investigate several PRF values, including those used widely in hydrologic field studies in estimating water flow. Field measurements of the WPBW were performed to compare measured and modeled results for the PRF values. The outcome of this aspect of the study was directed to improving the design of flood control structures and to provide a scientific basis for generating effective government policies for stormwater management in coastal areas.

Few studies have investigated the effect of an important hydrological parameter called the initial abstraction ratio (Ia/S). Currently, the ratio is set as a constant value of 0.2 in the SCS runoff curve number method. The constant initial abstraction ratio is regarded at best, an ambiguous assumed value and requires considerable refinement (Shi et al, 2009). The relationship developed by Woodward et al, (2003) allows application of a 5% Ia/S ratio using conjugate curve numbers. In this research, we investigate the effect of 5% and 20% initial abstraction ratio using the HEC-HMS model.

It is important to implement a multidisciplinary approach across spatial and temporal scales to understand the hydrologic environment. Technical advances in environmental sensing, modeling and cyberinfrastructure offer opportunities to cross disciplinary boundaries and scales to provide solutions that society and applied scientists and engineers can implement (Science, Education and Design Strategy for the WATERS Network, 2008). GIS is a powerful tool for storing, managing, analyzing and visualizing information in a spatial context. However, in the past water resource engineers had little training if any to work within a GIS information and modeling environment. The cost-benefit analysis for learning and applying GIS techniques often overcame the benefits. However, with the advent of the interfaces connecting GIS and water resources models, GIS applications in water resources have increased. Dodson and Li (1999) found the automated floodplain delineation was more efficient and accurate compared to the traditional approach. The availability of the spatial data in digital formats acceptable for GIS analysis from government agencies has significant cost savings in terms of the initial data requirement for the GIS application. Seamless 10 and 30-meter digital raster elevation data and land use/land cover data are available from the United States Geological Survey (USGS) for the contiguous United States. The soils information can be obtained from the Natural Resources Conservation Service (NRCS). Also, many state and local government agencies maintain their own spatial data, which generally is available at higher resolution (e.g., data from Monmouth County office of GIS was photogrammetric certified at +/- 1 foot accuracy). One of the important objectives of this research is the development and analysis of a state-of-the-art watershed model applying GIS techniques into traditional hydrologic and hydraulic modeling.

Hydrologic Engineering Center's River Analysis System (HEC-RAS) is a widely used simulation model for floodplain analysis. The most important limitation in this analysis is the economic constraint for surveying the required cross-sections of stream channels and associated floodplains. This surveying task would require cross-sections to be spaced farther depending on budget and schedule, which reduces the accuracy of the computed water surface profiles. Applying GIS techniques in the HEC-RAS simulation eliminates this limitation and offers the possibility of generating closely spaced cross-sections from LIDAR survey information that can be imported into HEC-RAS. Another important source of error is the subjective selection of the loss parameters, especially the Manning's roughness coefficient 'n' (Dodson and Li, 1999). Again, extracting the Manning's roughness coefficient values from Landuse/Landcover information using GIS techniques helps to assign 'n' values to the channel cross-sections. This study applies GIS techniques into HEC-RAS modeling using the HEC-GeoRAS toolbar and uses LIDAR information for the required elevation data. Another important outcome of this study is the validation of the LIDAR data in the floodplain analysis for the second order streams. This is particularly important due to upcoming LIDAR information that will be available for the State of New Jersey.

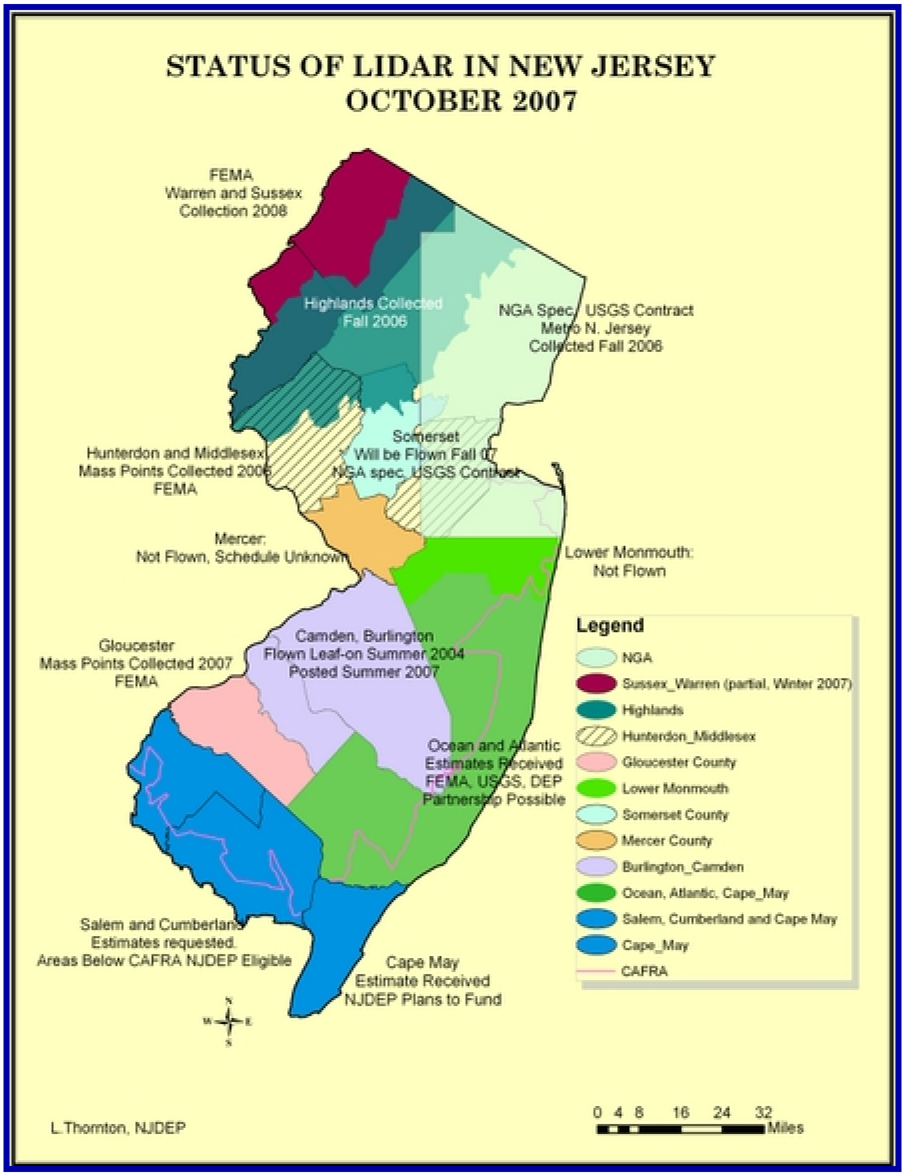

Figure 1: Status of LIDAR in New Jersey
(Source: https://lidarbb.cr.usgs.gov/index.php?showtopic=1777)

Finally, the traditional stormwater management approach (site-by-site basis) does not adequately address flooding as it fails to account for increased runoff volume (Emerson et al, 2005). Stormwater should be managed on a regional scale to predict and control flooding more appropriately. For example, the New Jersey Department of Environmental Protection (NJDEP) encourages regional scale planning and implementation strategies to identify and resolve water quality and quantity issues in the watershed. One of the important objectives of this research was to identify an appropriate flood control strategy for WPBW and to provide updated field validated and modeling results to use in regional stormwater management plans (RSWMP) implementation for the WPBW.

1.2 Study Area

WPBW was selected as one of the Governor's Coastal Initiative projects in NJ for 2005 because of its significant economic and recreational value and the multiple beach closures due to excessive bacterial loadings. WPBW is about 12 mi^2 and contains portions of Wall Township, Spring Lakes, Spring Lake Heights, and Sea Girt municipalities. NJDEP funded a RSWMP for WPBW to address beach closures impacted by high fecal counts. However, the storm event in October 2005, caused flooding at several locations in the watershed. The flooding caused by this storm event prompted mandatory evacuations of the roughly 115 homes in Spring Lake and Loch Harbor (NYTimes, Oct 14, 2005). Hence, the RSWMP focused on addressing both water quality and quantity issues in the watershed. The photographs in Figures 2-4 were collected during the development of the RSWMP. A local resident attending the monthly RSWMP meetings provided the flooding photographs.

Figure 2: Flooding near Station W3 - Old Mill Road

Figure 3: Stream stage before flooding

Figure 4: Stream stage after flooding

The following figures show the watershed location, digital elevation model (DEM) for WPBW, Landuse/Landcover data, and soils information obtained from the Monmouth County office of GIS.

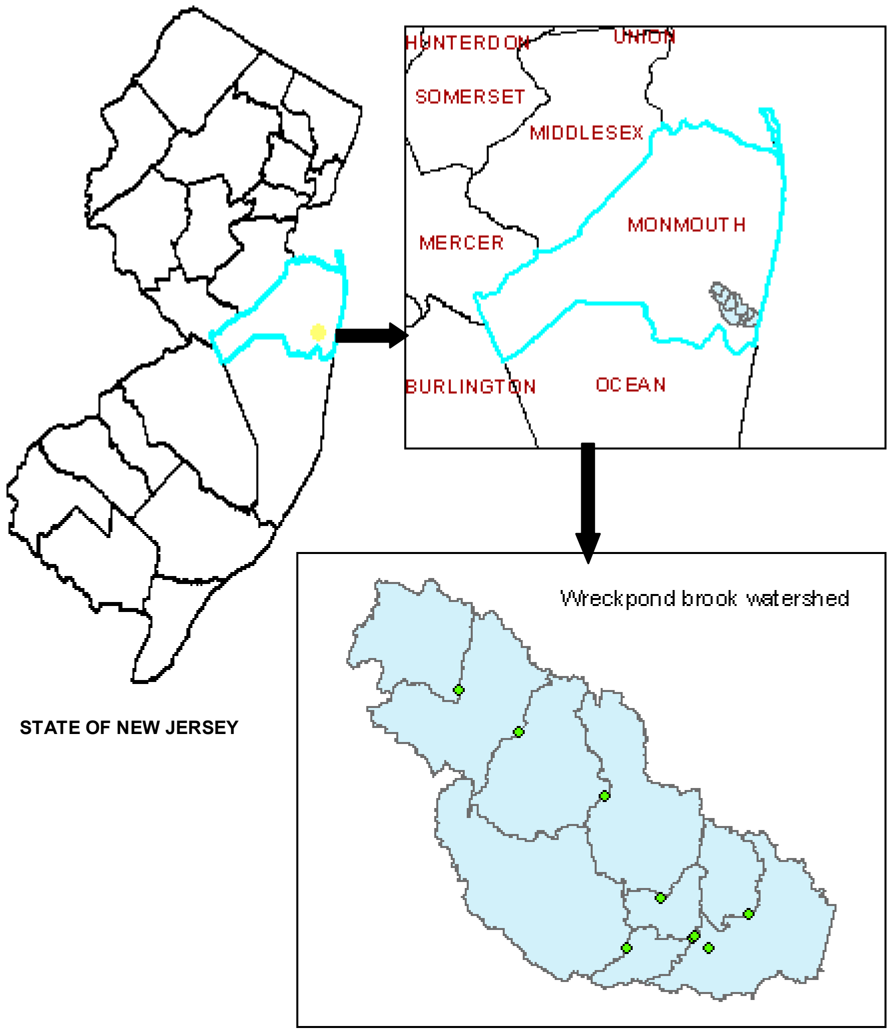

Figure 5: Location of the study area

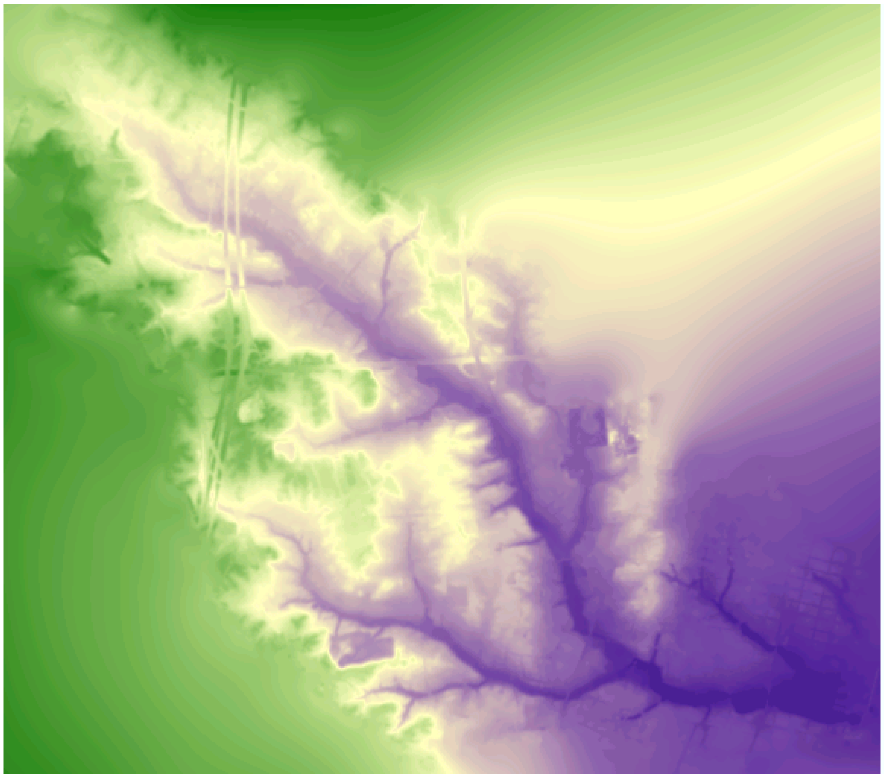

Figure 6: Digital Elevation Model for WPBW
(Source: Monmouth County Office of GIS)

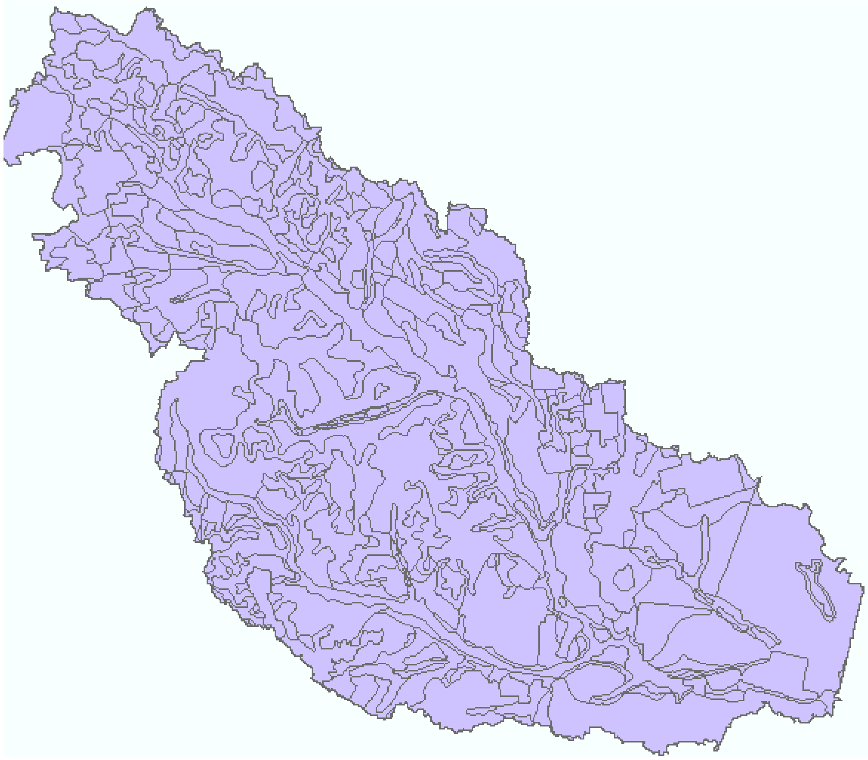

Figure 7: Soils information for WPBW
(Source: Monmouth County Office of GIS)

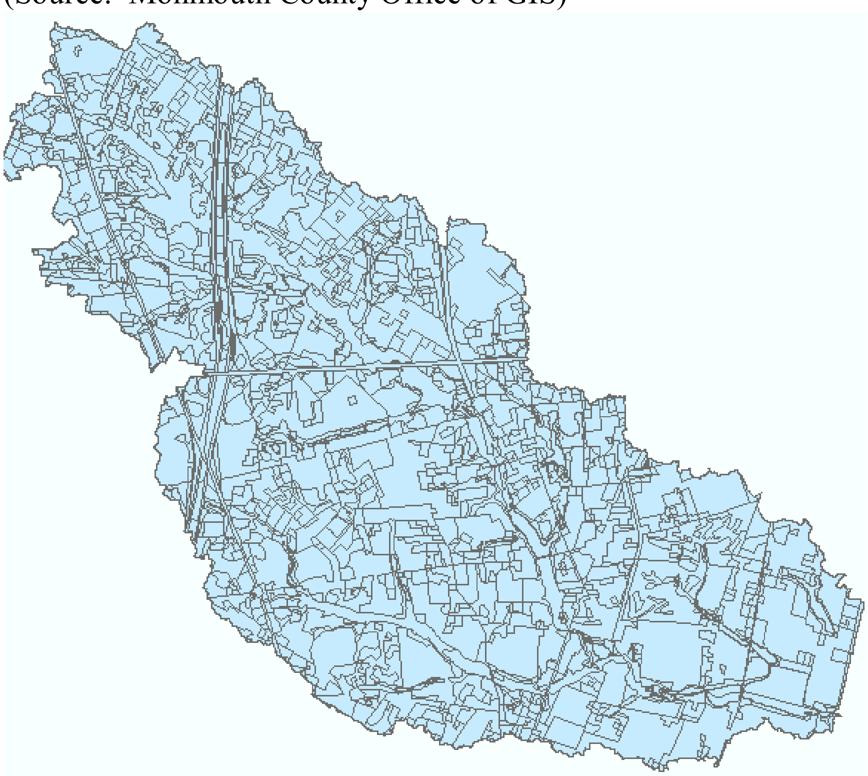

Figure 8: Landuse/Land cover information for WPBW
(Source: Monmouth County Office of GIS)

1.3 Key Science Questions

The questions addressed by this research are the following:

- What are the regional parameters for characterizing the low-lying relief of coastal watersheds in NJ?
- What is the contribution of GIS in the watershed modeling due to advances in computational speed and availability of spatial and temporal information?
- Is the general approach of a uniform detention strategy for the entire State good for WPBW?
- Is distributed modeling using NEXRAD rainfall better than lumped modeling using rain gage information?
- What revisions to WPBW Regional Stormwater management plan can be beneficial?

Stream response to rainfall events were studied for Wreck Pond Brook and inflow tributaries at 7 locations, from spring 2004 to fall 2006. The locations were the outlet of each hydrologic response unit, commonly known as subwatersheds. The real-time elevation data were collected and processed into discharge hydrographs using stream rating curves. Stream rating curves were developed using field measured average velocities and average cross sectional areas. These discharge hydrographs (observed) were compared against the modeled hydrographs (computed). The hydrologic model was developed using GIS techniques and Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS). Peak rate factor (PRF) and initial abstraction ratio were evaluated using HEC-HMS. Sensitivity study was performed for the modified parameters. This was important as any change to this value adopted by the government agencies would require significant overhaul of prevailing practice. The peak discharge values computed using HEC-HMS was used to compute water surface profiles by Hydrologic Engineering Center's River Analysis System (HEC-RAS). The necessary geometry file for HEC-RAS was developed in ArcGIS environment using HEC-GeoRAS. The computed water surface profiles were exported to GIS environment for floodplain delineation and analysis. The floodplain delineated in this study was compared with the draft Digital Flood Insurance Rate Maps (DFIRM). The flow value and loss coefficient parameters used in HEC-RAS were compared with NJ state supplemental flood studies. The necessary elevation information for floodplain delineation and analysis was obtained from the DEM and TIN files provided by the Monmouth County Office of GIS.

1.4 Work Acknowledgement

Table 1 lists the involvement of individuals who contributed to the development of the technical portion of the RSWMP. The table acknowledges persons involved and delineates the work performed as a part of this study of WPBW.

Table 1: Work Acknowledgement

Work	**Name**
• Watershed and Subwatershed delineation using ArcHydro and ArcGIS.	J. Brockwell, J. Showler and K. Patel
• DEM and TIN development for WPBW.	J. Brockwell
• Developing Curve Number grid.	K. Patel

• Creating stream geometry files at subwatershed outlets using surveyed data and HEC-GeoRAS. • Verification of stream geometry files.	J. Brockwell and K. Patel J. Showler and K. Patel
• Real time elevation data collection, rating curves development and Bridge surveys.	J. Showler, H. Birckhead and K. Patel
• Hydrologic model development using HEC-HMS (Existing).	J. Showler and K. Patel
• 5% Initial abstraction ratio investigation.	K. Patel
• Peak rate factor sensitivity investigation.	K. Patel
• Hydraulic model development using HEC-RAS.	K. Patel
• Floodplain Delineation and Analysis using HEC-RAS, HEC-GeoRAS and ArcGIS.	K. Patel
• Flood control analysis for WPBW.	K. Patel
• Hydrologic model development using HEC-HMS (Revised).	K. Patel

2. LITERATURE REVIEW

2.1 Stormwater Management

The appropriate management of stormwater is an international concern (Goff and Gentry, 2006). Traditionally, stormwater management was practiced on a site-by-site basis and stopped typically at the municipal border (Ecologic, 2005). The approach maintained the peak runoff rates for pre-and post-development conditions. This design approach failed to account for increased volume of runoff and actually could increase flooding (Emerson et al, 2005). Also, the detention provided in the lower portion of a large watershed could delay the peak flow and could coincide with the hydrograph peak from the upper watershed, thus, increasing peak flows (Corbitt, 1990).

The NJDEP adopted Stormwater Management Rules (N.J.A.C 7:8) in February, 2004 which required the reduction of peak flows for post development site conditions to address increased volumes. This approach, while reducing peak discharges immediately downstream of the development could fail to reduce peak flows at a watershed scale (Goff and Gentry, 2006). Ferguson (1998) described watershed-wide uniform detention strategy a failure. He further stated such methods did not have favorable effects on baseflow or water quality and failed also to control flooding. Hence, this generalized concept applied at a state-wide level throughout NJ may be ineffective to prevent flooding for certain watersheds. NJDEP identified regional stormwater management planning as a water resource management strategy to address existing issues or to anticipate and avoid future issues on a regional basis. The product of this planning process was the Regional Stormwater Management Plan (RSWMP) (NJDEP, 2004).

One of the important components of RSWMP was the hydrologic and hydraulic characterization of the watershed. The limitations and problems created by a uniform detention strategy can be overcome by providing detention on the main channel or within subbasins of the upper portion of large watersheds. To determine the optimum combination of controls to reduce flood damages is a complex problem (Corbitt, 1990). A calibrated and validated watershed model is necessary to successfully implement the RSWMP and to achieve desired flood control or water quality improvement. The models chosen to address water quantity issues in WPBW are the United States Army Corps of Engineers (USACE) Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS) and River Analysis System (HEC-RAS). These are public domain models and are used widely by private and public entities throughout the U.S and other parts of the world. HEC-HMS is the standard model used by the private sector in the U.S for the study of drainage design and for analyzing the impact of land use changes on flooding (Singh and Woolhiser, 2002). Several studies done using HEC-HMS have shown promising results (Anderson et. al, 2002, Knebl et al, 2005, McColl et al, 2007). HEC-2 (now HEC-RAS) has been applied extensively for the past 35 years in floodplain delineation work (Yang et al, 2006).

2.2 Runoff Curve Number Method

The SCS Runoff Curve Number method is used in this study to compute the runoff. It was developed by the United States Department of Agriculture (USDA) Soil Conservation Service (SCS) and is a method of estimating rainfall excess from rainfall (Hjelmfelt, 1991). The method is described in detail in National Engineering Handbook (2004). The chapter was prepared originally by Mockus (1964), and was revised by

Hjelmfelt (1998) with assistance from the NRCS Curve Number work group and H.F. Moody. Despite the wide use of the curve number procedure, documentation of its origin and derivation are incomplete (Hjelmfelt, 1991).

The conceptual basis of the curve number method has been the object of both support and criticism (Ponce and Hawkins, 1996). The major disadvantages of the method are sensitivity of the method to Curve Number (CN) values, fixing the initial abstraction ratio, and lack of clear guidance on how to vary Antecedent Moisture Conditions (AMC). However, the method is used widely and is accepted in numerous hydrologic studies. The SCS method originally was developed for agricultural watersheds in the mid-western United States; however it has been used throughout the world far beyond its original developers would have imagined.

The basis of the curve number method is the empirical relationship between the retention (rainfall not converted into runoff) and runoff properties of the watershed and the rainfall. Mockus found equation 1 appropriate to describe the curves of the field measured runoff and rainfall values (National Engineering Handbook, 2004). Equation 1 describes the conditions in which no initial abstraction occurs.

$$\frac{F}{S} = \frac{Q}{P} \qquad \text{Equation 1}$$

where $F = P - Q$ = actual retention after runoff begins;

Q = actual runoff

S = potential maximum retention after runoff begins ($S \geq F$)

P = potential maximum runoff (i.e., total rainfall if no initial abstraction).

For most applications, a certain amount of rainfall is abstracted. The three important abstractions for any single storm event are rainfall interception (Meteorological rainfall minus throughfall, stem flow and water drip), depression storage (topographic undulations), and infiltration into the soil. The curve number method lumps all three abstractions into one term, the Initial abstraction (*Ia*), and subtracts this calculated value from the rainfall total volume. The total rainfall must exceed this initial abstraction before any runoff is generated. This gives the potential maximum runoff (rainfall available for runoff) as $P - Ia$. Substituting this value in equation 1 yields following equation

$$\frac{P - Ia - Q}{S} = \frac{Q}{P - Ia} \qquad \text{Equation 2}$$

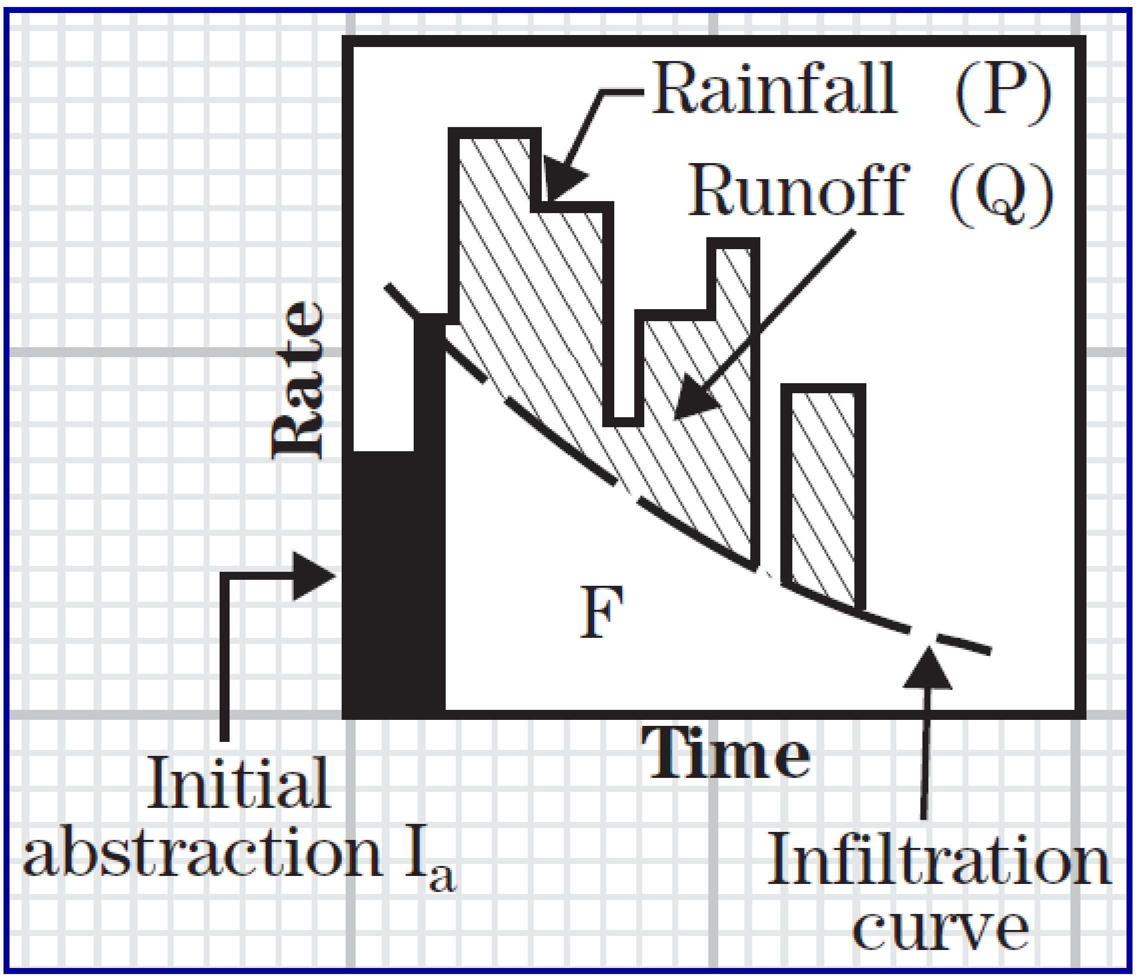

Figure 9: Components of SCS Runoff equation
(Source: NEH July 2004)

It is important to note the potential maximum retention term, "*S*", excludes *Ia*. Hence, for a given storm, maximum loss of rainfall is *S* plus *Ia*. Rearranging terms in Equation 2 for Q gives

$$Q = \frac{(P - Ia)^2}{(P - Ia) + S}$$ Equation 3

Establishing the relation to estimate *Ia* was challenging. The SCS provided the following empirical Equation 4 based on the assumption *Ia* was a function of the potential maximum retention *S*.

$$Ia = 0.2S$$ Equation 4

The potential maximum retention *S* is related to the dimensionless parameter CN in the range of 0 ≤ CN ≤ 100 by Equation 5.

$$S = \left(\frac{1000}{CN}\right) - 10)$$ Equation 5

Substituting Equation 4 into Equation 3 yields,

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)}$$ Equation 6

Equation 6 has only one parameter that needs to be evaluated (i.e., S) which can be determined by using Equation 5 and curve number tables published by the SCS.

2.3 Initial Abstraction Ratio

Figure 10 shows the justification for the relationship given in Equation 4.

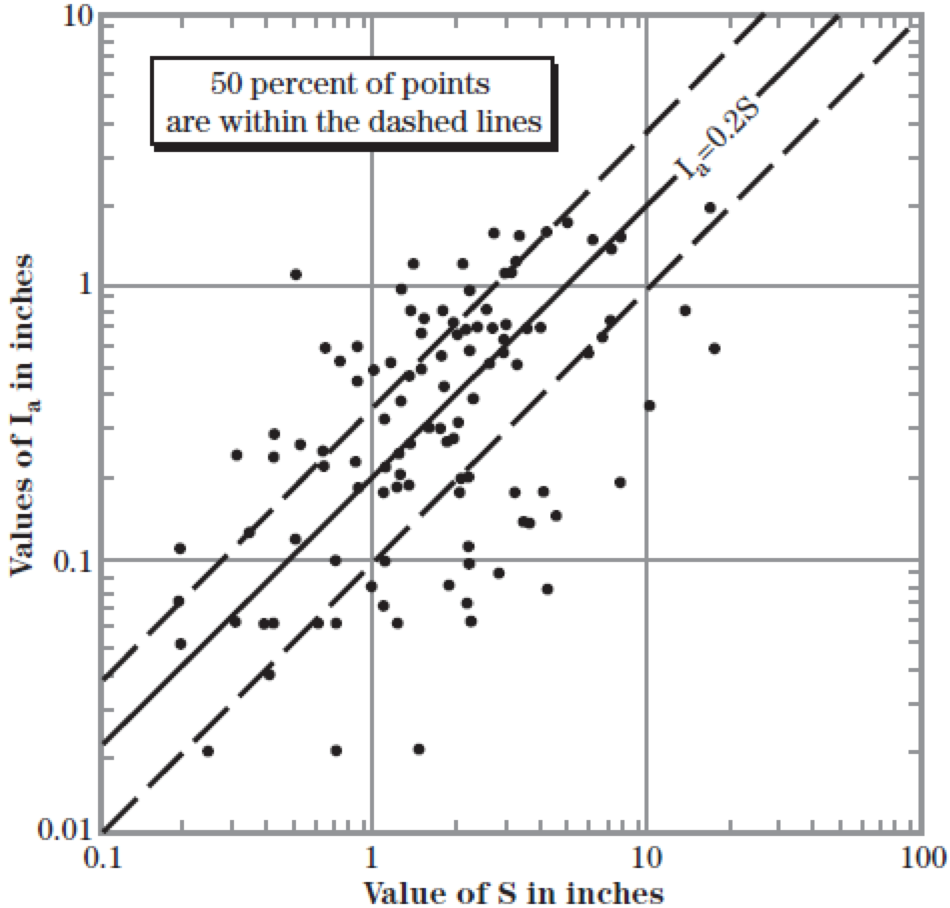

Figure 10: Relation between Ia and S
(Source: NEH, July 2004)

It can be seen from Figure 10 there is a significant spread in the data and 50% of the data lies within the two dashed lines representing the initial abstraction ratio of 0.095 and 0.38. Woodward et al., (2003) analyzed rainfall and runoff data for 307 watersheds using event fitting and model fitting techniques. He found 252 of the 307 cases (5 out of every 6) produced higher correlation and lower SE (standard error) with the initial abstraction ratio (Ia/S) of 5%. Lim et al., (2006) indicated use of a 5% Ia/S and modifying CN values to account for urbanization improved the model's predicted runoff. However, utilizing a 5% initial abstraction ratio required modifying the standard SCS curve number values. Woodward et. al., (2003) termed these values the "conjugate curve numbers" and generated a relation between $CN_{0.20}$ and $CN_{0.05}$ (based on relation between $S_{0.20}$ and $S_{0.05}$ developed by Hawkins, 1985). They found a 5% (Ia/S) ratio produced higher direct runoff than a 20% (Ia/S) ratio only up to certain threshold rainfall value called the "critical precipitation". For precipitation higher than the critical precipitation, the 20% (Ia/S) ratio produced more direct runoff, which was attributed to the fact that $S_{0.05}$ was different than $S_{0.20}$. The response of WPBW to a 5% (Ia/S) ratio and a 20%(Ia/S) ratio was analyzed in this research for 2, 10, 25, 50 and 100-year NRCS storm events. A few storm events were analyzed to verify the applicability of a 5% Ia/S ratio based on field measured rainfall and stream gage information. The 'Event analysis method' (Woodward et. al, 2003) was used for this analysis.

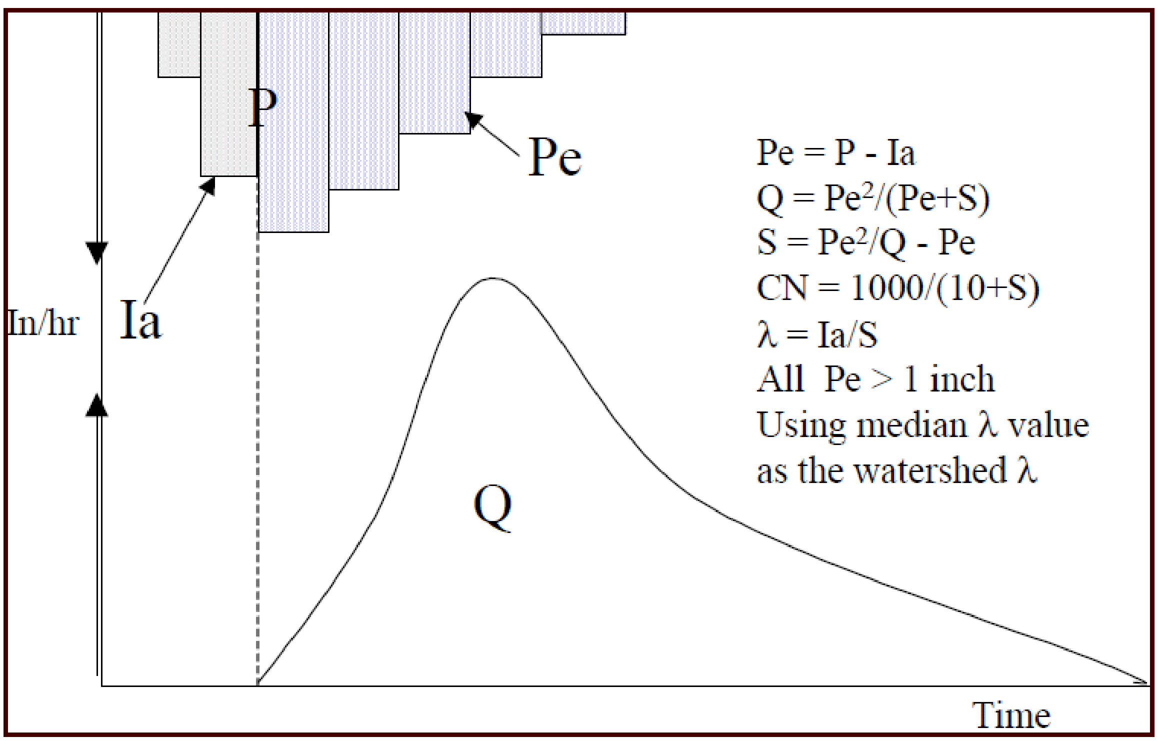

Figure 11: Event Analysis Method (Source: Woodward et. al., 2003)

The conjugate curve numbers and potential retention relations (Woodward et al, 2003; Hawkins, 1985) were used for the computation of curve numbers with the use of

5% initial abstraction ratio. It is necessary to change the SCS curve number values if any relation other than 20% initial abstraction ratio is used.

$$S_{0.05} = 1.33 S_{0.20}^{1.15}$$ Equation 7

$$CN_{0.05} = \frac{100}{1.879[100/CN_{0.20} - 1]^{1.15} + 1}$$ Equation 8

Table 2 shows the conjugate curve numbers, initial abstraction ratio, and the potential retention values developed using the above equations. These values were used in HEC-HMS model to utilize 5%Ia/S ratio.

Table 2: Curve number and S values for 5% Ia ratio

CN_{20}	$CN_{0.05}$	$S_{0.05}$	$Ia_{0.05}$
98	97.91	0.21	0.011
96	95.36	0.49	0.024
94	92.65	0.79	0.040
92	89.83	1.13	0.057
90	86.94	1.50	0.075
88	84.03	1.90	0.095
86	81.10	2.33	0.116
84	78.18	2.79	0.140
82	75.27	3.29	0.164
80	72.38	3.82	0.191
78	69.52	4.38	0.219
76	66.70	4.99	0.250
74	63.93	5.64	0.282
72	61.19	6.34	0.317
70	58.51	7.09	0.355
68	55.88	7.90	0.395
66	53.30	8.76	0.438
64	50.77	9.70	0.485
62	48.31	10.70	0.535
60	45.90	11.79	0.589

58	43.55	12.96	0.648
56	41.26	14.24	0.712
54	39.02	15.63	0.781
52	36.85	17.14	0.857
50	34.73	18.79	0.940
48	32.68	20.60	1.030
46	30.68	22.59	1.130
44	28.74	24.80	1.240
42	26.86	27.24	1.362
40	25.03	29.95	1.498
38	23.26	32.99	1.650
36	21.54	36.42	1.821

2.4 Antecedent Moisture Condition

Ponce (1989) reported for the case of low curve numbers or rainfall depths, the curve number method is very sensitive to the curve number and antecedent moisture conditions. NEH-4 (SCS, 1985) showed an appropriate Antecedent Moisture Condition (AMC) level based on a 5-day antecedent rainfall for dormant and growing seasons. However, due to concerns such as not accounting for regional differences or scale effects, it was removed from the NEH-4 release of 1993 (Ponce and Hawkins, 1996). During the CN development, the natural scatter of points around the median CN was interpreted as a measure of the natural variability of soil moisture and associated rainfall-runoff relationships. The average curve numbers are the median values used to represent the average response of a site for given soil, cover and surface conditions. The theoretical limits of CN values are 0 and 100. The enveloping CN values were defined using P-Q plots for each site. This provided practical limits of CN values including accounting for the observed variability of CN values. Hence, the observed variability of rainfall-runoff response was represented by the AMC parameter (Ponce and Hawkins, 1996). Table 3 shows the curve numbers developed by SCS for AMC I (dry conditions), AMC II (average conditions) and AMC III (wet conditions).

Table 3: CN values for AMC I and III corresponding to CN values for AMC II

CN FOR AMC II	CN FOR AMC I	CN FOR AMCIII
98	98	98
95	87	98
90	78	96
85	70	94
80	63	91
75	57	88
70	51	85
65	45	82
60	40	78
55	35	74
50	31	70
45	26	65
40	22	60
35	18	55
30	15	50

(Source: Viessman and Lewis, 2003)

2.5 Peak Rate Factor

Another common suspect parameter is the Peak Rate Factor (PRF). PRF is a coefficient that ranges from nearly 600 for steep mountainous conditions to 300 for flat swampy conditions (Viessman and Lewis, 2003). The standard SCS unit hydrograph is a dimensionless unit hydrograph and often is solved using triangulation as shown in Figure 12.

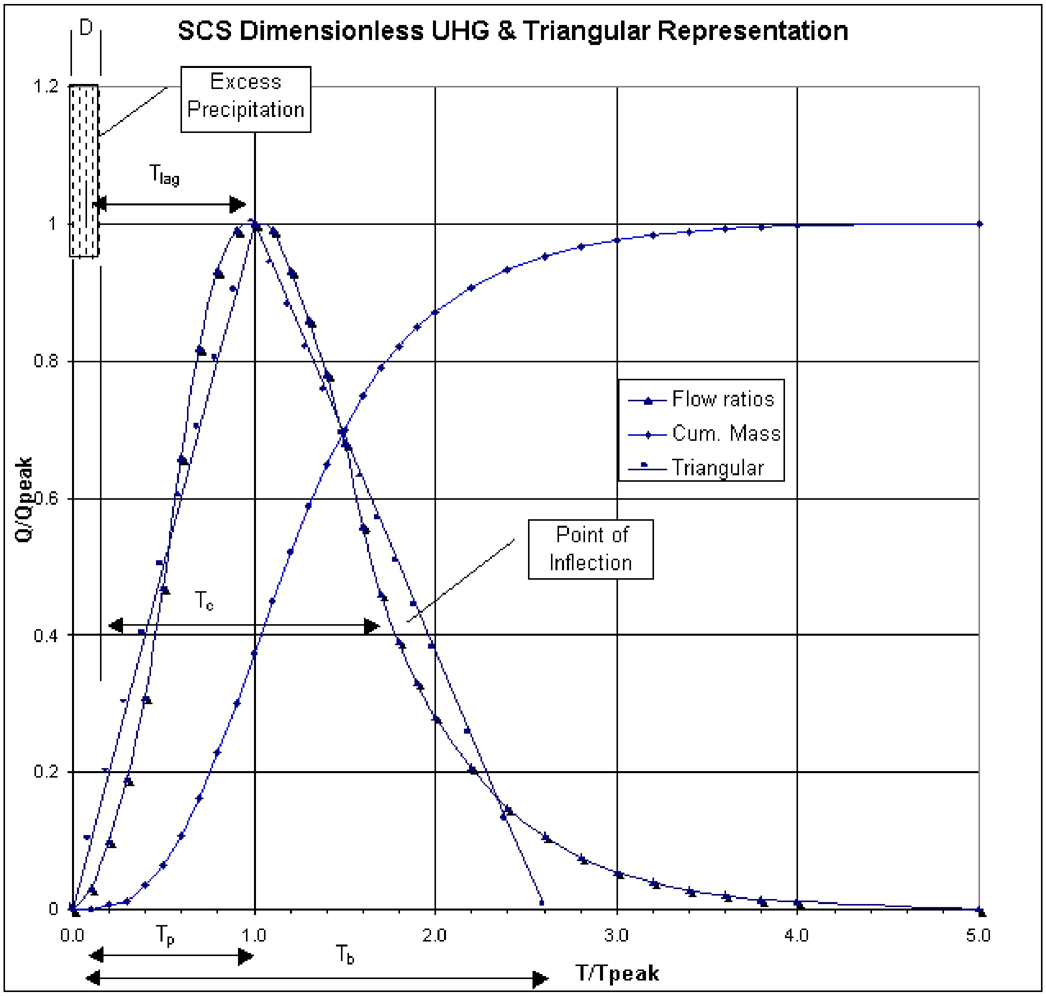

Figure 12: SCS Dimensionless unit hydrograph and its triangular representation
(Source: http://www.nohrsc.noaa.gov/technology/gis/uhg_manual.html)

From Figure 12, the total volume Q under the triangle is given by:

$$Q = \frac{1}{2} T_p q_p + \frac{1}{2} T_r q_p \qquad \text{Equation 9}$$

$$Q = \frac{q_p}{2}(T_p + T_r) \qquad \text{Equation 10}$$

$$q_p = \frac{2Q}{T_p + T_r} \qquad \text{Equation 11}$$

Converting the above equation so as to express the area under the unit hydrograph as 1 inch from the drainage area of 1 square mile in 1 hour, we get following equation:

$$q_p = \frac{645.33 * 2 * Q}{T_p + T_r}$$ Equation 12

Now the time base of the unit hydrograph is $T_b = 2.67T_p$ and $T_b = T_p + T_r$ hence

$T_r = 1.67T_p$

Substituting this value in the above equation, yields,

$$q_p = \frac{484AQ}{T_p}$$ Equation 13

Where,

A is the drainage area in square miles

Q is the runoff volume in inches

T_p is the time to peak in hours, and

q_p is the peak flow rate in cfs.

As shown above, the peak rate factor of 484 has the inherent assumption that 3/8 of the volume under the unit hydrograph is under the rising limb and the remaining 5/8 of the volume is under the recession limb. This may not be true if the study area has characteristics requiring a lower or higher peak factor. The National Engineering Handbook by NRCS (March 2007) acknowledged several studies after 1972 depicted the variation in peak rate factors from below 100 to more than 600. State and Soil Conservation District officials in New Jersey were concerned the use of the NRCS standard Dimensionless Unit Hydrograph (DUH) with a PRF of 484 resulted in unrealistically high peaks, specifically in the WPBW. Hence, a study was performed by Dewberry (Fairfax, Virginia) for the New Jersey Department of Agriculture (NJDA) and funded by the New Jersey Department of Environmental Protection (NJDEP) to address this concern. The study recommended using a PRF value of 230 instead of 484. Also, it recommended using the relationship between hydrograph time parameters as $T_p = 0.5T_c$

instead of the standard NRCS relation of $T_p = 2/3T_c$ for large watersheds in Lower Monmouth County, NJ. This change in parameter relations was attributed to the fact that a gamma distribution technique was used to develop the Lower Monmouth Dimensionless Unit Hydrograph (LMDUH).

LMDUH was derived from a USGS gage data from the Shark River and Manasquan stream gages (bounding the study area by the north and south, respectively). HMS includes the SCS unit hydrograph as a "built in" feature, however it was converted so that the same time units (hours) used for other dimensionless hydrographs could be used as well. The Delmarva unit hydrograph is recommended for modeling watersheds in the coastal plain region of New Jersey, which is characterized, by flat topography (average watershed slope less than 5 percent), low relief and significant surface storage in swales and depressions. Figure 13 shows the WPBW study area falling in New Jersey the coastal plain. Hence, the Delmarva unit hydrograph was converted to a percentage curve for modeling the study area. This research analyzed the applicability of the PRF of 230 and compared the results with the PRF of 284 (Delmarva unit hydrograph) and PRF of 484 (standard SCS unit hydrograph) for WPBW.

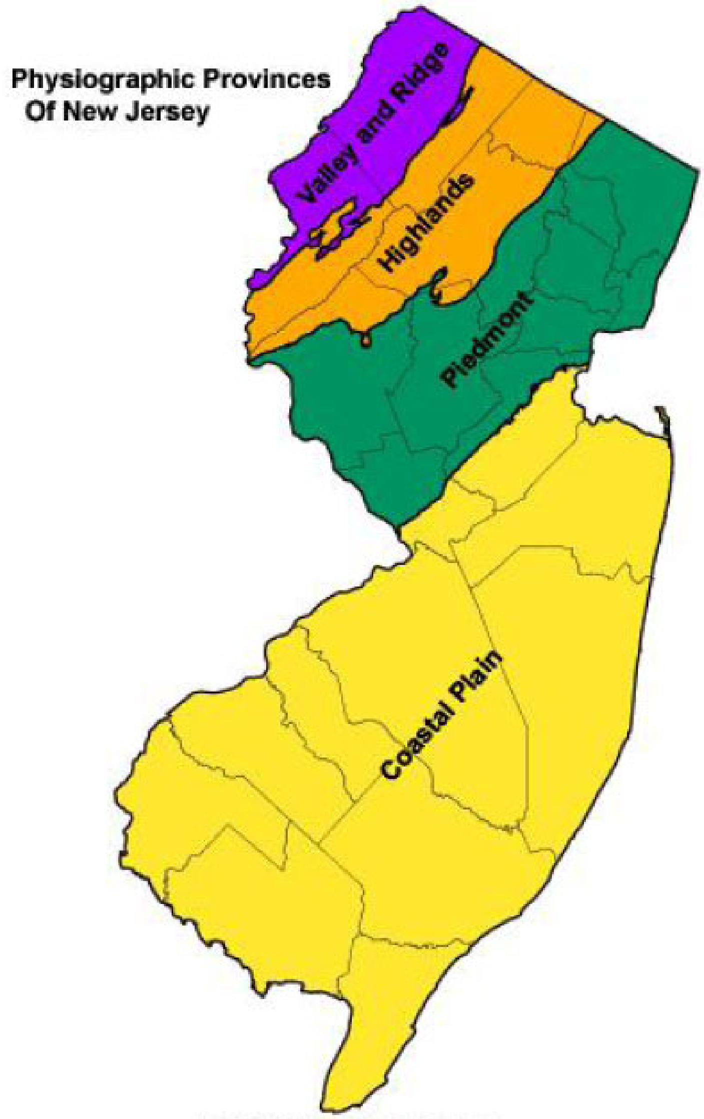

Figure 13: Physiographic provinces of New Jersey

(Source: NJDEP, http://www.state.nj.us/dep/njgs/geodata/dgs02-7.htm)

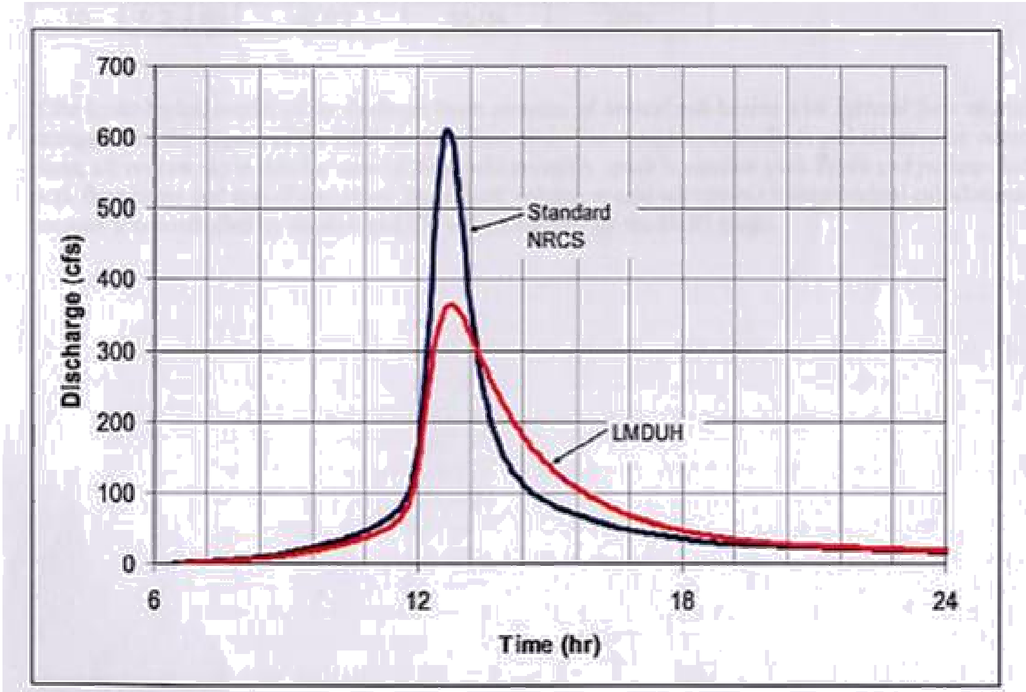

Figure 14: Dimensionless unit hydrographs
(Source: WPBW RSWMP, 2008)

2.6 GIS techniques in Hydrologic and Hydraulic Modeling

The increasing availability of spatial data (terrain and rainfall), GIS software to manage spatial data, faster processors, and the availability of interfaces to connect simulation models with GIS, have increased use of GIS in watershed modeling (Carpenter et al, 2001; Singh and Woolhiser, 2002; Vieux, 1991, Whiteaker et al, 2006: Garbrecht et al, 2001). HEC-GeoRAS is the geospatial tool used in this study, which serves as the interface between GIS and the simulation model HEC-RAS. Figure 15 shows the flow diagram for using HEC-GeoRAS (HEC-GeoRAS User's manual, ver 4.0, Sep 2005). HEC-GeoRAS allows engineers to concentrate on hydraulic model development and analysis rather than GIS mechanics. The user environment provides engineers an opportunity to view real-world systems of interest, which in turn assists them to rectify errors and make informed decisions in the model development (Ackerman et al, 1999). Tate et al, (2002) applied HEC-GeoRAS successfully to create a terrain model for floodplain mapping. A widely used approach is watershed modeling that divides the drainage basin into discrete units possessing similar rainfall-runoff and physical characteristics. This approach reduces model complexity and spatially distributed data requirements in basin-scale models (Beighley et al, 2005). An ArcGIS data model – ArcHydro- is used to structure spatial data and to develop Hydrologic Response Units (HRU) for WPBW.

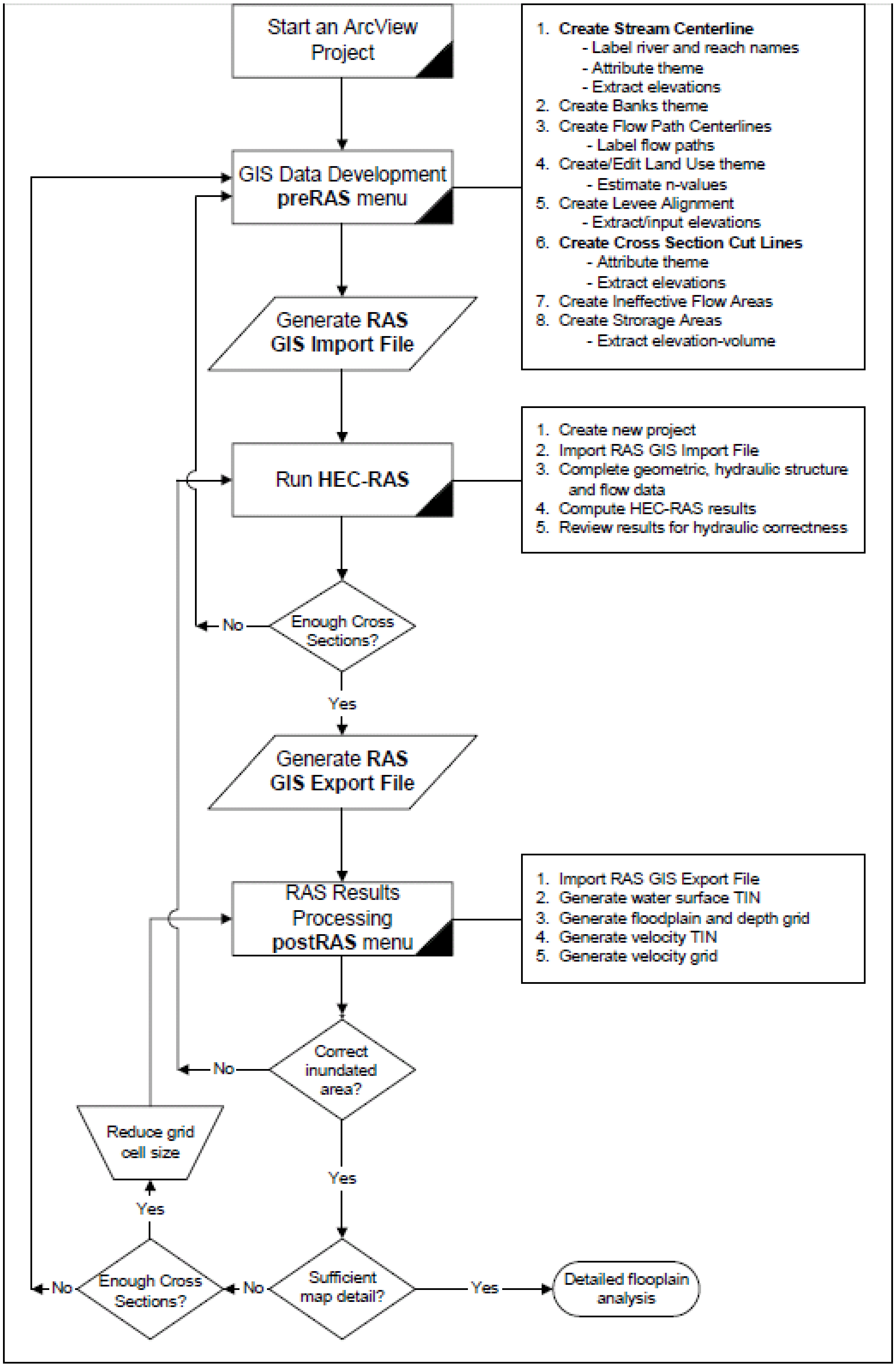

Figure 15: Flow diagram for using HEC-GeoRAS
(Source: HEC-GeoRAS User's Manual, 2005)

The two major digital formats in which surfaces are represented are Digital Elevation Models (DEMs) and Triangular Irregular Networks (TINs). DEMs are commonly referred to as gridded data and contain terrain information in its cells. The resolution of the DEM refers to the cell size and each cell is assigned one elevation value. TINs represent the terrain surface in triangles connecting points and break lines. Points can be used to represent valleys, high points, and any abrupt changes. Break lines can represent line features such as banks, roads, drainage divides etc. TINs generally are considered more precise than DEMs, however processing data in DEM format is faster than TIN format. Commonly TIN is developed through a photogrammetric method. However, the limitation of this method is its inability to provide terrain information below the water surface of the hydrographic feature. This may be a problem for large rivers where significant low flow volumes exists (Long, 1999). Wreck Pond Brook and its tributaries comprise a small watershed for this to be an issue, especially for high flow event modeling. The following figure 16 shows example of LIiDAR instrument collecting elevation data using a Twin Engine Aircraft. More information regarding data collection and data processing can be found at the website: http://www.csc.noaa.gov/products/sccoasts/html/tutlid

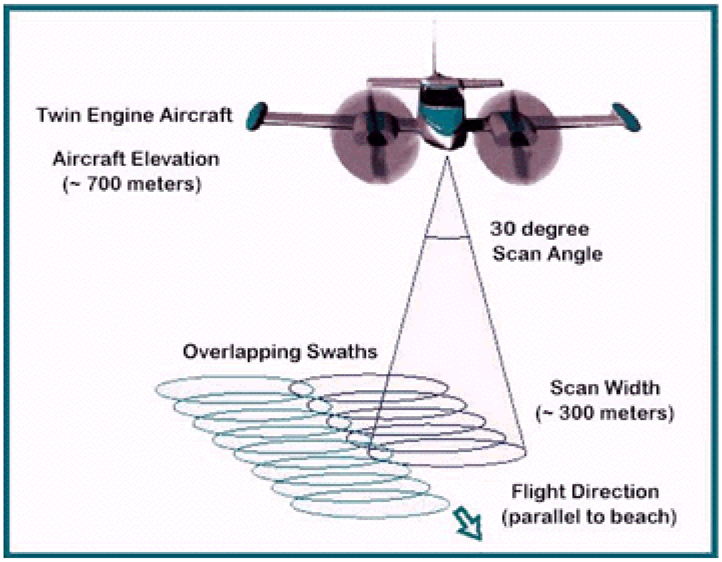

Figure 16: Elevation data collection using LIDAR technique
(Source: http://www.csc.noaa.gov/products/sccoasts/html/tutlid.htm)

Precipitation measures have the most critical influence on the hydrological model performance (Neary et al, 2004). Mixed results have been observed with the use of radar data in hydrological modeling. Bedient et al, (2000) found model performance using radar data as accurate as rain gauge data. Moreda et al, (2002) observed simulated volume using radar rainfall data as low as 30 percent of the observed volume. Sun et al, (2000) suggested using radar data in combination with gauge data to improve model performance. However, "*The hypothesis that higher resolution data will lead to more accurate hydrograph simulations remains largely untested* (www.nws.noaa.gov/oh)." When a polygon grid of NEXRAD precipitation data was overlaid on WPBW, only 8 cells contained the entire watershed out of which 4 cells encompassed most of the watershed.

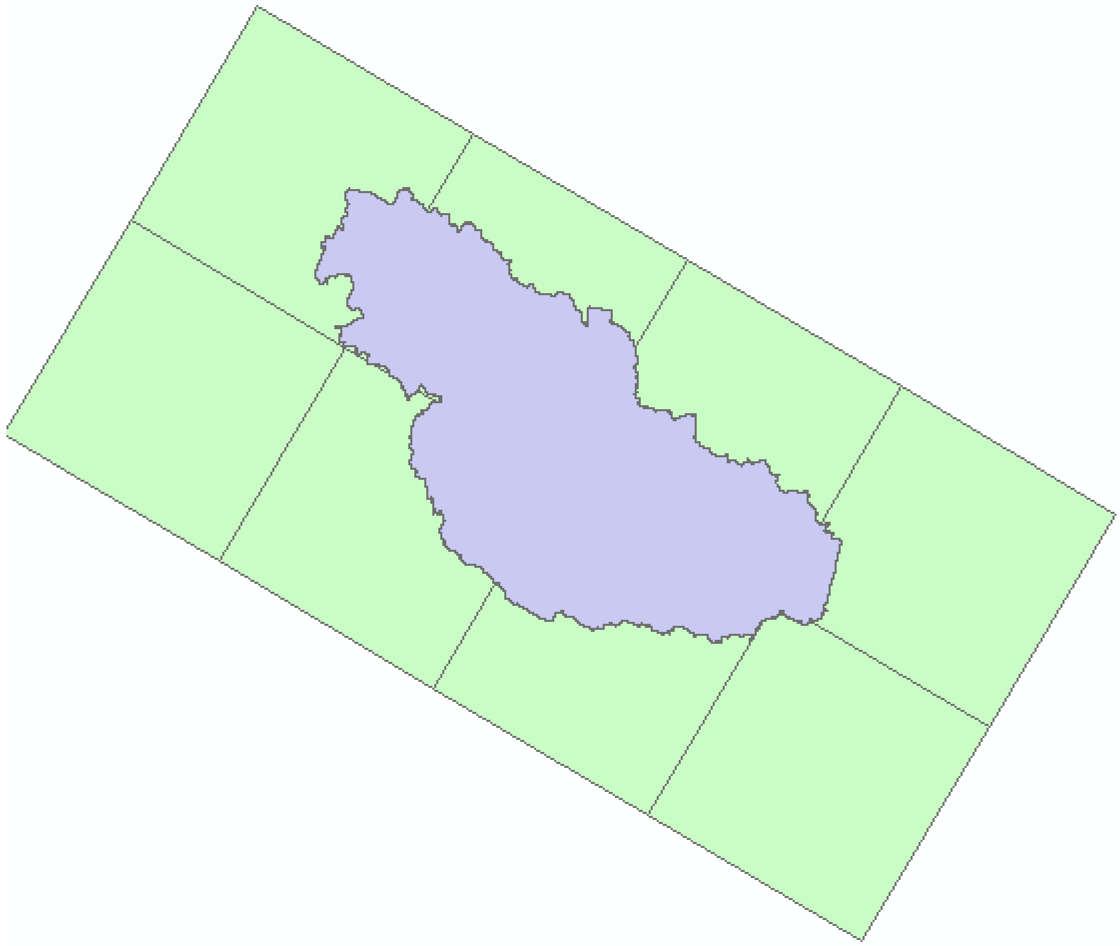

Figure 17: WPBW overlaying NEXRAD XMRG polygons

Hence, precipitation data were utilized for modeling from the NJ Mesonet station location at Sea Girt, NJ and from the weather station installed specifically for RSWMP development behind the library in Wall Township, NJ.

3. THEORY AND METHODOLOGY

The overall modeling sequence can be depicted as follows:

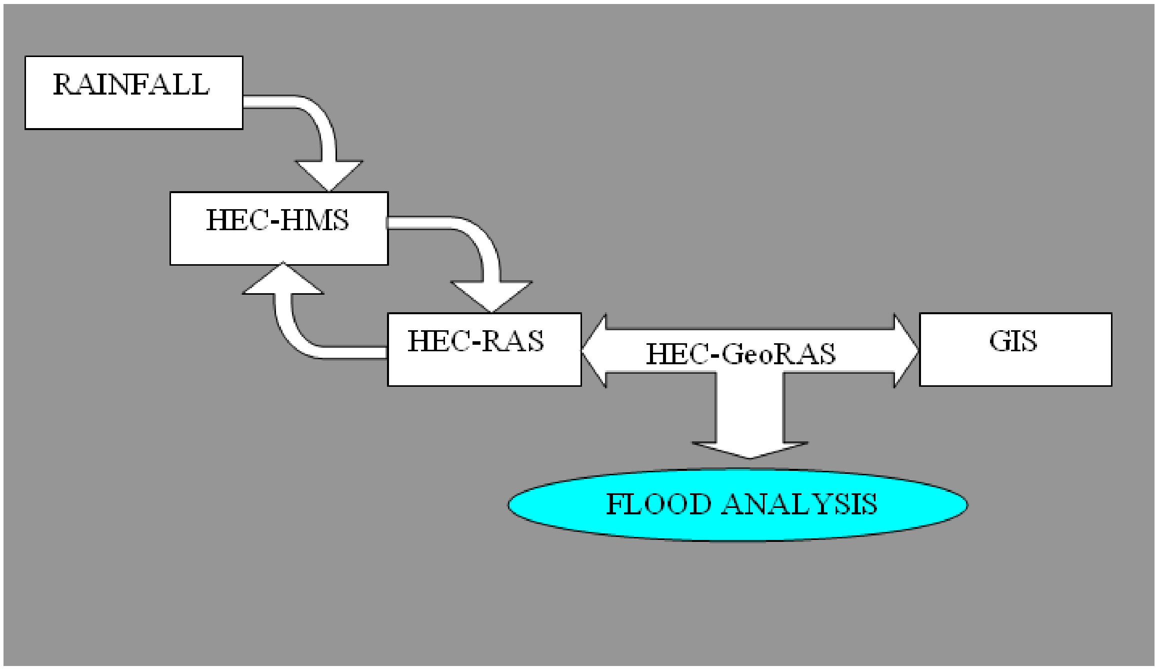

Figure 18: Overall Modeling Sequence

The forcing condition for any hydrologic model is the rainfall. Several rainfall sources are available in variety of formats and can be used as is or in combination. Rainfall data could be field monitored gage data or hypothetical statistical storm or NEXRAD rainfall data. HEC-HMS simulates rainfall-runoff process. From the given precipitation, it deducts losses and convolutes the excess rainfall with specified unit hydrograph and routes it through the channel to generate a runoff hydrograph. This flow information is used then in HEC-RAS to estimate water surface elevations. HEC-RAS simulates one-dimensional flow and generates water surface profiles for given flow conditions. It is capable of modeling both steady and unsteady flow conditions. The geometry file necessary for HEC-RAS simulation can be created in a GIS environment using HEC-GeoRAS toolbar and can be imported to a RAS environment. The output from HEC-RAS simulation can be exported into a GIS environment for floodplain delineation using HEC-GeoRAS.

For the purpose of RSWMP development and this study, the real time elevation data were collected using Win-Situ 4.5 Instrument control software with pressure sensor device miniTROLL, standard by In-situ Inc (www.in-situ.com). Velocity readings were taken using AquaCalc Pro by JBS instruments (www.jbsinstruments.com). AquaCalc is an electronic instrument that automates open-channel discharge measurements. Real time depth data was converted to the elevation data and velocity readings. These values were converted to flow data with the AquaCalc computer by multiplying channel cross-sectional area and velocity. Precipitation data was obtained from the weather station located at the intersection of Bailey's Corner Road and Allaire Road. The station is in the mid-eastern section of the watershed. The Monmouth County Office of GIS provided GIS terrain data. The GIS data had horizontal accuracy that was certified photogrammetrically as +/- one foot. Surveying was conducted at all the stations to accurately measure the channel depths and major grade breaks such as banks and other variations in channel and floodplain.

The models chosen for hydrologic and hydraulic simulations were the United States Army Corps of Engineers Hydrologic Engineering Center's HEC-HMS and HEC-RAS models. The models were used together with the GIS tool HEC-GeoRAS and the ArcGIS data model - ArcHydro to provide an interface with GIS. Drainage features and Hydrologic Response Units (HRUs) were delineated using the Digital Terrain Model (DTM) and ArcHydro. Stage-Discharge rating curves were generated at each HRU outlet using the simulation model HEC-RAS. The necessary geometry files for HEC-RAS simulation were developed using the Triangulated Irregular Network (TIN) model and the GIS tool HEC-GeoRAS in ArcGIS 9.1 with the spatial analyst and 3D analyst extension. The rating curves were combined with the real time stream elevation data to develop the observed stream hydrograph. HEC-HMS was used to simulate the watershed response to rainfall and was calibrated to match the observed stream hydrograph. Notably, the Peak Rate Factor of 230 was used in HEC-HMS simulation for specifying transformation hydrograph. After calibrating and validating the HMS model for WPBW, the NRCS statistical storm events for 2, 10, 25, 50 and 100-year recurrence intervals were run to determine the flows through various hydrologic elements of the model. The flows for the 100-year storm event were input in HEC-RAS to generate water surface profiles. The water surface profiles from HEC-RAS were exported to the GIS environment and the floodplain was delineated using the HEC-GeoRAS interface. The resulting floodplain was compared against the D-FIRM maps currently under update and refinement by FEMA.

3.1 Model Description

3.1.1 HEC-HMS

HEC-HMS is an abbreviation for Hydrologic Engineering Center's – Hydrologic Modeling System. HEC-HMS is hydrologic simulation software for modeling precipitation-runoff processes for a dendritic watershed. The basic components of the HEC-HMS are the basin model, meteorologic model, control specifications, time-series data, and paired data.

The basin model was used for physical representation of the watershed. Available elements in the basin model were: subbasin, reach, junction, reservoir, diversion, source, and sink. Computation proceeded from the upstream elements in a downstream direction. The meteorologic model performed meteorologic data analysis and included precipitation, evapotranspiration, and snowmelt. The control specification specified the time span of the simulation. Control specifications included a starting date and time, ending date and time, and a time interval. Combining a basin model, meteorologic model, and control specifications formed a simulation run.

Rainfall-runoff simulation using HMS consisted of modeling four basic components of the hydrologic cycle. Each component was modeled using one of several available models. A few limitations on the model combinations are described in the HMS users manual (version 3.3, September 2008).

1. Models to compute runoff volume
   a. Initial and constant-rate
   b. SCS curve number (CN)
   c. Gridded SCS CN
   d. Green and Ampt
   e. Deficit and constant rate
   f. Soil moisture accounting (SMA)
   g. Gridded SMA
2. Models of direct runoff (overland flow and interflow)
   a. User-specified unit hydrograph (UH)
   b. Clark's UH

   c. Snyder's UH
   d. SCS UH
   e. ModClark
   f. Kinematic wave
3. Models of baseflow
   a. Constant monthly
   b. Exponential recession
   c. Linear reservoir
4. Models of channel flow
   a. Kinematic wave
   b. Lag
   c. Modified Puls
   d. Muskingum
   e. Muskingum-Cunge Standard Section
   f. Muskingum-cunge 8-point Section
   g. Confluence
   h. Bifurcation

All of the mathematical models in the HMS uses constant parameter values, are uncoupled and are deterministic. Hence, the parameters are assumed to be time stationary and the models are solved independently and every time a simulation is computed it will yield exactly same results computed in the previous run.

3.1.2 HEC-RAS

HEC-RAS is an abbreviation for Hydrologic Engineering Center's – River Analysis System. HEC-RAS is a hydraulic simulation model for calculating water surface profiles for natural and man-made channels. It is a simulation model that has one-dimensional steady and unsteady flow analysis options. It also has components for movable boundary sediment transport computations and water quality analysis.

Steady flow analysis allows calculating water surface profiles for steady, gradually varied flow. It can model subcritical, supercritical and mixed flow regime water surface profiles. It also can model hydraulic structures such as bridges, culverts, weirs and spillways. Due to its extensive modeling capabilities, HEC-RAS is used widely for channel and flood plain management and flood insurance studies to evaluate floodway encroachments. Water surface profiles are calculated by an iterative procedure solving a one dimensional energy equation. A momentum equation is used for computing rapidly varying profiles. The input requirements for the model are channel and flood-plain elevations, right and left channel bank locations, Manning's roughness coefficients for channel and overbanks, contraction and expansion coefficients, downstream cross-section distances, flow data and any one-boundary condition such as known water surface elevation, critical depth, normal depth, or rating curve.

3.1.3 Arc-Hydro

Arc Hydro is a geospatial and temporal data model for water resources that operate within ArcGIS (Maidment, ESRI, 2002). Arc Hydro is not a simulation model in itself. It helps to structure data to support the hydrologic simulation model. HRUs can be delineated automatically using the digital elevation model (DEM) or triangular irregular network (TIN) together with ArcHydro. The drainage path is defined based on the eight-direction pour point model, hence the creation of a flow direction grid. This model is discussed in detail (Arc Hydro, Maidment, 2002). The flow accumulation grid is developed from the flow direction grid. By defining the threshold drainage area (flow accumulation value), the stream links are defined. The catchment grid is developed by combining the flow direction grid and stream links. These raster grids - stream links, outlet cells and catchment grid were vectorized to drainage lines, drainage points and catchment polygons respectively and stored in Arc Hydro. Watershed and subwatersheds were delineated by selecting outlet points of interest.

3.1.4 HEC-GeoRAS

HEC-GeoRAS is a set of tools (toolbar) for use in a GIS environment. It provides an interface between ArcGIS and HEC-RAS software. It is designed specifically to process geospatial data for use in RAS modeling and to process RAS results into a GIS environment. Processing terrain information and other GIS data in ArcGIS using GeoRAS allows creating and exporting a geometry file for RAS analysis. The geometry file created contains river, reach and station information, cross-section cutlines, bank stations, reach lengths for left and right overbanks and channel, roughness coefficients and also can contain blocked obstructions, limited culvert and bridge information, ineffective flow areas and storage areas. The results from RAS simulation (e.g. water surface profiles) can be exported into a GIS environment and further analysis can be performed using the GeoRAS toolbar. The GIS data exchange between RAS and ArcGIS is done through a specifically formatted GIS data exchange (*.sdf) file. To utilize current capabilities of GeoRAS requires both 3D Analyst and Spatial Analyst extensions. Also, a

Microsoft Windows operating system is necessary to use the GeoRAS toolbar. Microsoft XML 4.0, ESRI ApFramework and ApUtilities, and ESRI XML Data Exchange are the required installations prior to installation of HEC-GeoRAS 4.0 or HEC-GeoRAS 4.1. GeoRAS 4.0 is for ArcGIS 8.3 and GeoRAS 4.1 is for ArcGIS 9.1 and newer versions.

3.2 Model Development

3.2.1 HEC-HMS

The models included in the HMS are mathematical models. They are the equations that represent the behavior of the hydrologic system components. The equations are solved for known inputs (e.g. precipitation, upstream hydrograph) to predict the physical system's response i.e. output (e.g. runoff, downstream hydrograph). The hydrologic system could comprise several components such as drainage basin, reservoirs, lakes, bridges, junctions, streams etc. The Wreck Pond Brook watershed is divided into eight subwatersheds and is further divided into several smaller drainage areas where necessary. For example, lake surfaces or directly connected impervious areas are input as subareas with very little lag time. All the subareas of the Wreck Pond Brook watershed are represented as separate but connected areas within the model. Stream reaches, impoundments or junctions connect these subareas. Model inputs consist of the aerial extent of the subareas (square miles), runoff coefficients (curve number), time parameters (lag time) and hydrograph information (percentage curve). In addition, lakes and ponds are represented in the model by describing the storage-discharge and elevation-discharge relationships into the paired data component. The precipitation is input into the meteorological model component. The meteorological model allows to input one of the several available precipitation products including SCS storm or user specified hyetograph. User specified hyetograph input was chosen for calibrating and validating the model. Rainfall data (precipitation input) was collected from the South Jersey Resource Conservation and Development Council R.I.S.E. network along with data collected from the New Jersey Mesonet weather station network. Once the HEC-HMS model was developed, SCS storm data was input to analyze the flows for 2, 10, 25, 50 and 100-year storm events.

HMS allows the user to input customized data. Therefore, data developed specifically for the study area can be entered to model the watershed. For example, the LMDUH developed for the study area was converted to S-hydrograph and used in modeling.

HMS allows the user to input observed flow values or observed stages in the field. Observed flow values appear in the summary tables, time-series tables and graphs for the modeled elements. HMS computes runoff depth and volume for observed and modeled hydrographs and can be compared in the summary table of the element. Graphical results provide a mechanism for visual analysis by superimposing the observed hydrograph over the modeled hydrograph. For Wreck Pond Brook watershed, real-time stream elevations were converted to stream flow hydrographs using HEC-RAS and were input as observed flow into time-series data. This step allowed calibration of the HMS model by comparing the observed hydrograph against the modeled hydrograph.

The simulation run computes the runoff response from the basin model using rainfall input from the meteorological model for the time interval and duration selected in the control specifications. All three components (basin model, meteorological model and control specifications) are required for the simulation run computation. Combinations of parameters can be modeled and tested by creating multiple simulation runs. A total of 15 simulation runs are required to compare the Wreck Pond watershed response to 2, 10, 25, 50 and 100-year storm events using the standard SCS, Delmarva and Lower Monmouth unit hydrographs. Results of a run can be accessed from the main toolbar or by right clicking the element in the basin model window or from the ‘results’ tab in the watershed explorer window.

3.2.1.1 Basin Model Development

The basin model provides several methods to choose from for modeling rainfall runoff processes. These models are listed in section 3.1. Below are the screen captures from HEC-HMS showing available methods for Basin model development.

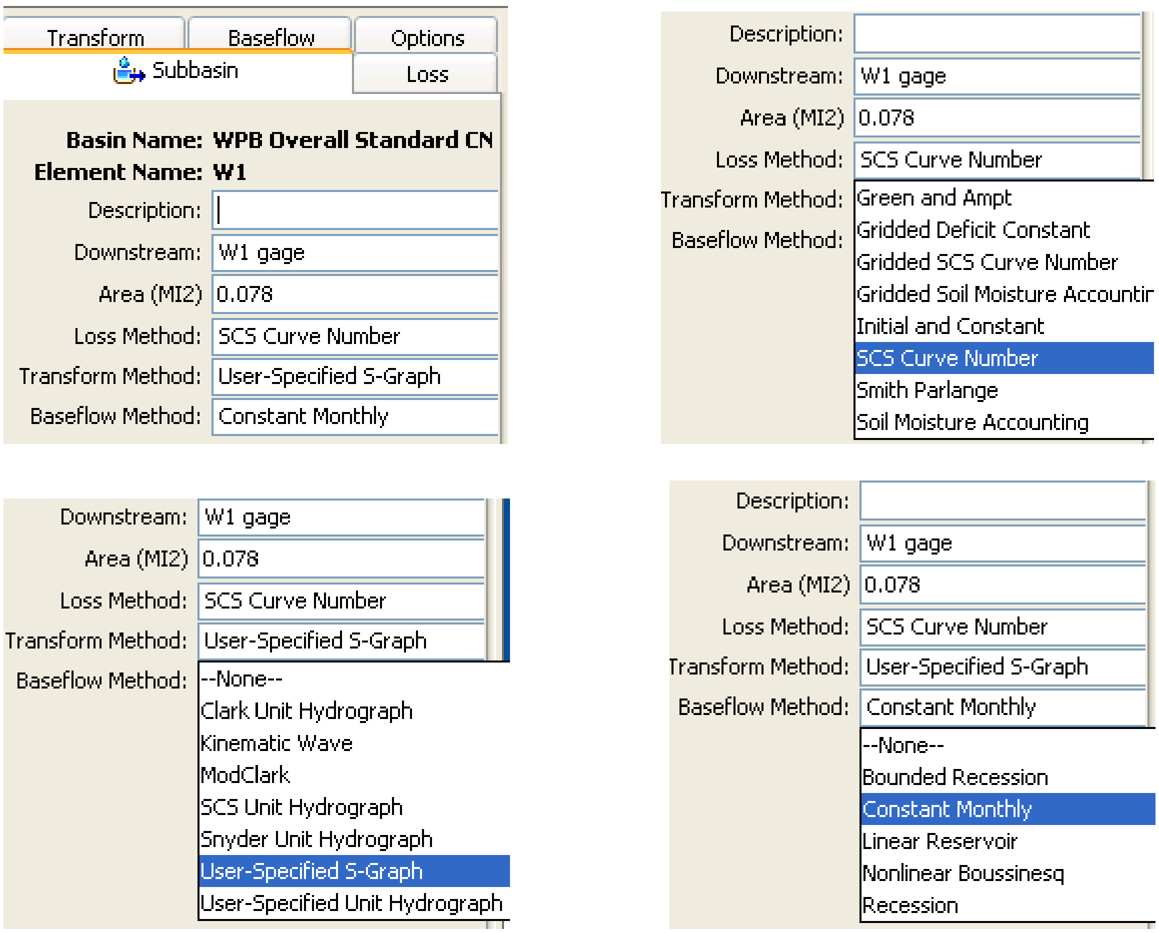

Figure 19: Methods for Basin Model development

3.2.1.2 Loss Method:

The SCS curve number (CN) method was chosen to compute runoff volume. The method is described in detail in '*Urban Hydrology for Small Watersheds',* Technical Release No. 55, commonly referred to as TR55 (SCS, 1986).

The solution of the SCS runoff equation is shown below in Figure 20.

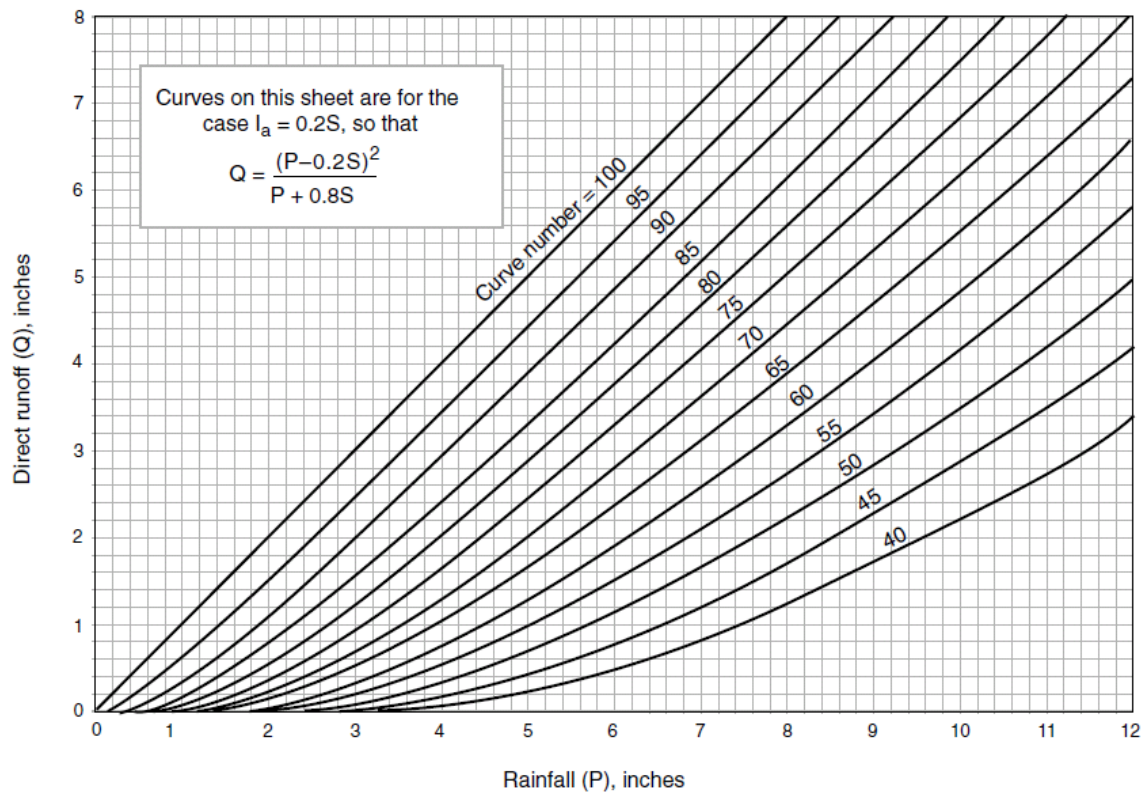

Figure 20: Solution of SCS runoff equation
(Source: TR-55, 1986)

The following tables (4 to7) from TR-55 (SCS, 1986) provide curve number values.

Table 4: Runoff curve numbers for urban areas

Cover Type and Hydrologic Condition	**Average Percent Impervious Area**	**A**	**B**	**C**	**D**
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.)					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm drains (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only)		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84

2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation)		77	86	91	94

(Source: TR-55, SCS 1986)

Table 5: Runoff curve numbers for cultivated agricultural lands

Cover Type	Treatment	Hydrologic Condition	A	B	C	D
Fallow	Bare soil		77	86	91	94
	Crop residue cover (CR)	Poor	76	85	90	93
		Good	74	83	88	90
Row Crops	Straight row (SR)	Poor	72	81	88	91
		Good	67	78	85	89
	SR + CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C + CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured & terraced (C&T)	Poor	66	74	80	82
		Good	62	71	78	81
	C&T + CR	Poor	65	73	79	81
		Good	61	70	77	80
Small grain	SR	Poor	65	76	84	88
		Good	63	75	83	87
	SR + CR	Poor	64	75	83	86
		Good	60	72	80	84
	C	Poor	63	74	82	85
		Good	61	73	81	84
	C + CR	Poor	62	73	81	84
		Good	60	72	80	83
	C&T	Poor	61	72	79	82
		Good	59	70	78	81
	C&T + CR	Poor	60	71	78	81

		Good	58	69	77	80
Close-seeded or broadcast Legumes or Rotation Meadow	SR	Poor	66	77	85	89
		Good	58	72	81	85
	C	Poor	64	75	83	85
		Good	55	69	78	83
	C&T	Poor	63	73	80	83
		Good	51	67	76	80

(Source: TR-55, SCS 1986)

Table 6: Runoff curve numbers for other agricultural lands

Cover Type	Hydrologic Condition	A	B	C	D
Pasture, grassland, or range-continuous forage for grazing	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow – continuous grass, protected from grazing and generally mowed for hay		30	58	71	78
Brush – brush-weed-grass mixture, with brush the major element	Poor				
	Fair	35	56	70	77
	Good	30	48	65	73
Woods – grass combination (orchard or tree farm)	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30	55	70	77
Farmsteads – buildings, lanes, driveways, and surrounding lots		59	74	82	86

(Source: TR-55, SCS 1986)

Table 7: Runoff curve numbers for arid and semi-arid rangelands

Cover Type	Hydrologic Condition	A	B	C	D
Herbaceous—mixture of grass, weeds, and low-growing brush, with brush the minor element	Poor		80	87	93
	Fair		71	81	89
	Good		62	74	85
Oak-aspen—mountain brush mixture of oak brush, aspen, mountain mahogany, bitter brush, maple, and other brush	Poor		66	74	79
	Fair		48	57	63
	Good		30	41	48
Pinyon-juniper—pinyon, juniper, or both; grass understory	Poor		75	85	89
	Fair		58	73	80
	Good		41	61	71
Sagebrush with grass understory	Poor		67	80	85
	Fair		51	63	70
	Good		35	47	55
Desert shrutb - major plants include saltbush, greasewood, creosote bush, blackbrush, bursage, palo verde, mesquite, and cactus	Poor	63	77	85	88
	Fair	55	72	81	86
	Good	49	68	79	84

(Source: TR-55, SCS 1986)

The curve numbers in tables 4 through 7 are for average runoff conditions and an initial abstraction ratio of 0.2.

3.2.1.3 Transform Method

The user specified S-graph method was chosen for transforming excess precipitation into a runoff hydrograph. Three dimensionless unit hydrographs were converted to a percentage curve and were input manually for comparison. The standard

SCS dimensionless unit hydrograph, Delmarva unit hydrograph and Lower Monmouth dimensionless unit hydrograph were converted and used in the study for comparison.

Several parameters inherent in SCS unit hydrograph are different for the Delmarva unit hydrograph and the Lower Monmouth unit hydrograph. They are summarized as shown in the table 8 below.

Table 8: Unit hydrograph Characteristics

Parameters	**SCS unit hydrograph**	**LM unit hydrograph**	**Delmarva**
T_l (lag time)	$0.6T_c$	$0.5T_c$	
T_p (time to peak)	$0.67T_c$	$0.5T_c$	
T_r (time for recession limb)	$4T_p$	$12T_p$	$9T_p$
T_b	$5T_p$	$13T_p$	$10T_p$
D (excess rainfall duratio)	$0.133T_c$	$0.042T_c$	
PRF	484	230	284
% of Volume under rising limb	37.5	24.6	22

(Source: Dewberry Inc, 2004)

The table below shows the coordinates of the unit hydrograph and its mass curve parameters.

Table 9: SCS dimensionless unit hydrograph

t/Tp	**Qa/Q**	**Q/Qp**	**t/Tp**	**Qa/Q**	**Q/Qp**
0	0	0	170	78.8	0.46
10	0.1	0.03	180	82	0.39
20	0.6	0.1	190	84.7	0.33
30	1.2	0.19	200	87	0.28
40	3.6	0.31	220	90.6	0.207
50	6.5	0.47	240	93.3	0.147
60	10.7	0.66	260	95.2	0.107
70	16.2	0.82	280	96.6	0.077
80	22.8	0.93	300	97.5	0.055
90	30	0.99	320	98.3	0.04

100	37.5	1	340	98.8	0.029
110	44.9	0.99	360	99.1	0.021
120	52.1	0.93	380	99.4	0.015
130	58.8	0.86	400	99.6	0.011
140	64.9	0.78	450	99.9	0.005
150	70.4	0.68	500	100	0
160	75	0.56			

(Source: http://www.nohrsc.noaa.gov/technology/gis/uhg_manual.html)

Table 10: Delmarva unit hydrograph and mass curve co-ordinates

t/Tp	Qa/Q	Q/Qp	t/Tp	Qa/Q	Q/Qp
0	0	0	500	93.5	0.109
20	0.5	0.111	520	94.4	0.097
40	2.5	0.356	540	95.2	0.086
60	6.9	0.655	560	95.9	0.076
80	13.7	0.896	580	96.5	0.066
100	22	1	600	97	0.057
120	30.4	0.929	620	97.5	0.049
140	38.1	0.828	640	97.9	0.041
160	45	0.737	660	98.2	0.033
180	51	0.656	680	98.5	0.027
200	56.5	0.584	700	98.7	0.024
220	61.3	0.521	720	98.9	0.021
240	65.6	0.465	740	99.1	0.018
260	69.4	0.415	760	99.2	0.015
280	72.9	0.371	780	99.3	0.013
300	76	0.331	800	99.4	0.012
320	78.7	0.296	820	99.5	0.011
340	81.1	0.265	840	99.6	0.009
360	83.3	0.237	860	99.7	0.008
380	85.3	0.212	880	99.7	0.008
400	87.1	0.19	900	99.8	0.006
420	88.6	0.17	920	99.8	0.006
440	90	0.153	940	99.9	0.005
460	91.3	0.138	960	99.9	0.005

480	92.5	0.123	980	100	0
			1000	100	0

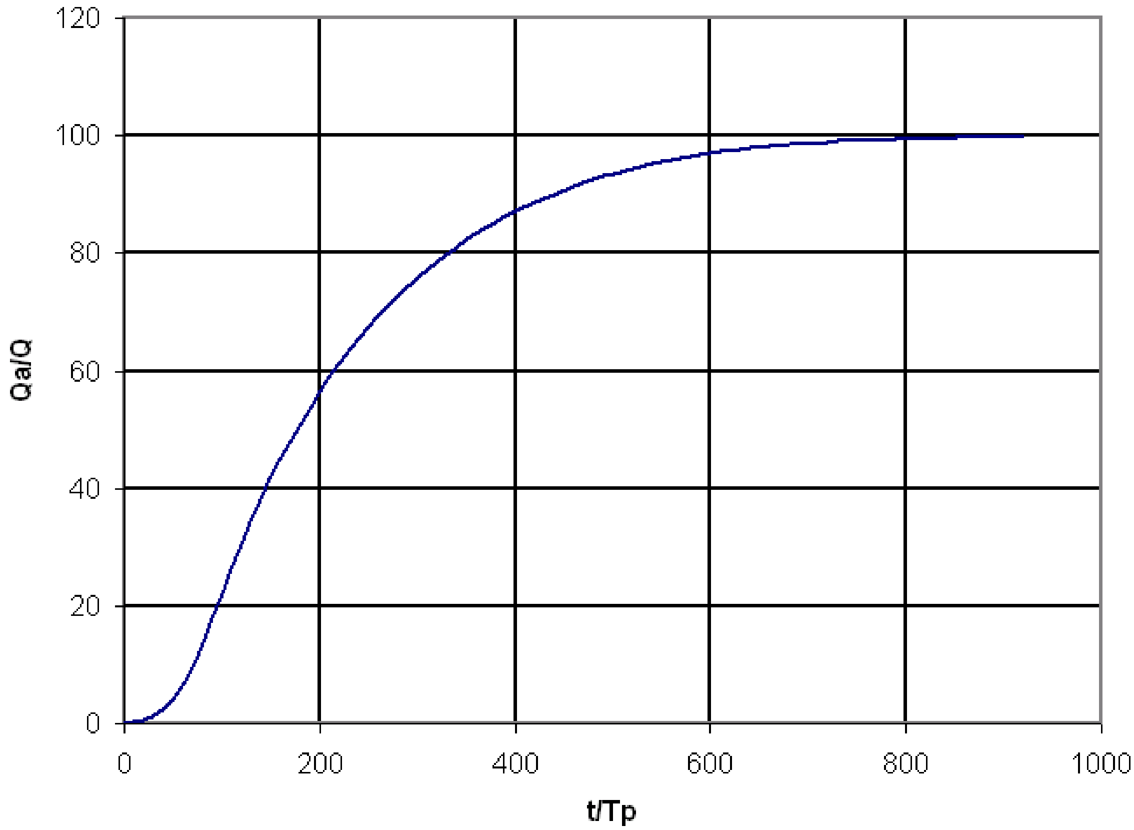

Figure 21: Delmarva Mass curve

Table 11: Lower Monmouth dimensionless unit hydrograph and mass curve co-ordinates

t/Tp	Qa/Q	Q/Qp	t/Tp	Qa/Q	Q/Qp
0	0	0	220	60.6	0.622
10	0.3	0.191	240	64.8	0.648
20	1.4	0.385	260	68.6	0.686
30	3	0.552	280	71.9	0.719
40	5.2	0.689	300	74.9	0.749
50	7.9	0.796	340	80	0.8
60	10.9	0.877	360	82.1	0.821
70	14.1	0.935	380	84	0.84
80	17.5	0.973	400	85.7	0.857
90	21	0.994	450	89.2	0.892

100	24.6	1	500	91.9	0.919
110	28.1	0.994	550	93.9	0.939
120	31.6	0.979	600	95.4	0.954
130	35.1	0.957	650	96.5	0.965
140	38.5	0.928	700	97.4	0.974
150	41.7	0.894	750	98	0.98
160	44.8	0.858	800	98.5	0.985
170	47.8	0.819	850	98.9	0.989
180	50.7	0.778	900	99.2	0.992
185	52	0.758	1000	99.5	0.995
192	53.9	0.729	1100	99.7	0.997
200	55.9	0.697	1200	99.9	0.999

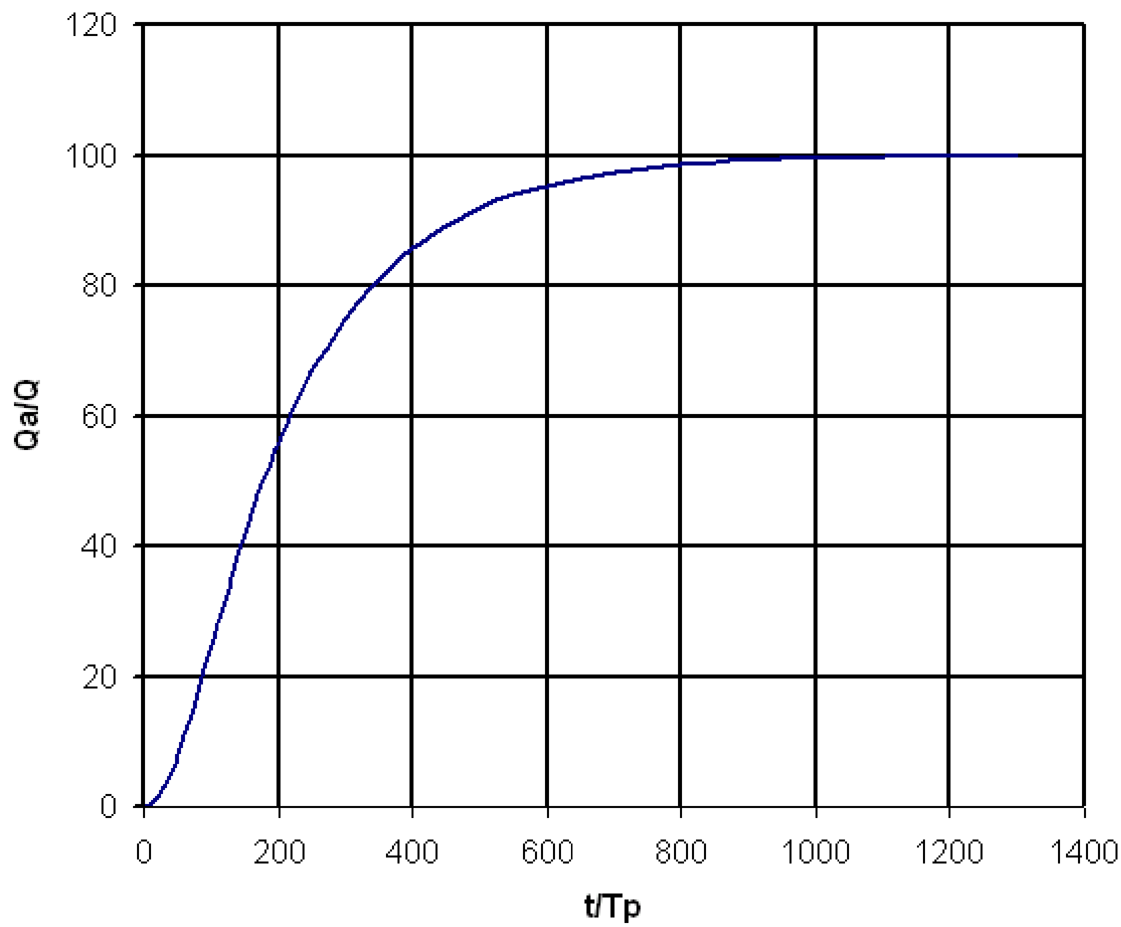

Figure 22: Lower Monmouth mass curve

The user specified S-graph transform method in HMS requires the input in the form of percentage of unit flow versus percentage of time lag. Therefore, the same S-graph can be used in different subbasins with different time lags. Thus, in the tables above the ratio t/T_p needs to be converted to t/T_l.

For the SCS dimensionless unit hydrograph,

$T_p = 0.67T_c$ and $T_l = 0.6T_c$, hence we get $T_p = 1.12T_l$.

Hence, percentages of the time lag ordinates are computed by multiplying the ratio (t/T_p) in the table above by 1.12.

For the Lower Monmouth dimensionless unit hydrograph,

$T_p = 0.5T_c$ and $T_l = 0.5T_c$, therefore we get $T_p = T_l$. Hence the ratio (t/T_p) in the table can be used as percentage time lag ordinates.

3.2.1.4 Base flow Method

During the runoff process, a certain portion of the precipitation enters the shallow ground water table and later is transmitted to the stream as base flow. Base flow may be observed during non-precipitation events as the shallow flow, which is characterized by a constant flow rate over time. During a precipitation event, base flow may increase while the stream hydrograph (runoff hydrograph) is flowing through the channel. For large streams and rivers, this base flow component may be a significant portion of the overall hydrograph. An internet based utility for base flow separation known as, "WHAT" (Web-based Hydrograph Analysis Tool, produced by Purdue University, (2002) http://cobweb.ecn.purdue.edu/~what/) was used during the RSWMP development to examine the relative degree of base flow in the observed (gauged) hydrographs. Most of the Wreck Pond Watershed is underlain by sandy, gravely soils which have a very high transmissivity. The hydrographs rose quickly and fell quickly, indicating that the watershed itself was not retaining runoff in the soils for long periods. This was observed in the stream gage recordings where the after-storm base flow values were frequently the same as or only slightly higher than pre-storm values. NJDA utilized a constant monthly base flow value and separated base flow hydrographs were developed through the WHAT website. NJDA was able to use both methods to satisfactorily reproduce both runoff volume and peak flow values. No changes are proposed in this study for baseflow computations.

3.2.2 HEC-RAS

The hydraulic model for WPBW was developed using the steady flow water surface profile computation method found in HEC-RAS. Steady flow suggests that flows at a given cross section are time invariant flows. The computations are based on energy balance from one cross section to the next. The energy equation is based on a fundamental law of physics, conservation of energy, which states “Energy can neither be created nor be destroyed, but can be transformed from one form to another”.

$$Z_2 + Y_2 + \frac{\alpha_2 V_2^2}{2g} = Z_1 + Y_1 + \frac{\alpha_1 V_1^2}{2g} + h_e \qquad \text{Equation 14}$$

Where,

Subscripts 1 and 2 represent properties at downstream and upstream cross sections respectively,

Z_1, Z_2 are the main channel inverts,

Y_1, Y_2 are the depth of water at cross sections

V_1, V_2 are the average velocity in the cross sections

α_1, α_2 are the velocity weighting coefficients

g = acceleration due to gravity (9.81 m/s^2)

h_e = energy head loss

The following figure 23 from the RAS reference manual explains graphically the above-mentioned terms (Version 4.0, March 2008).

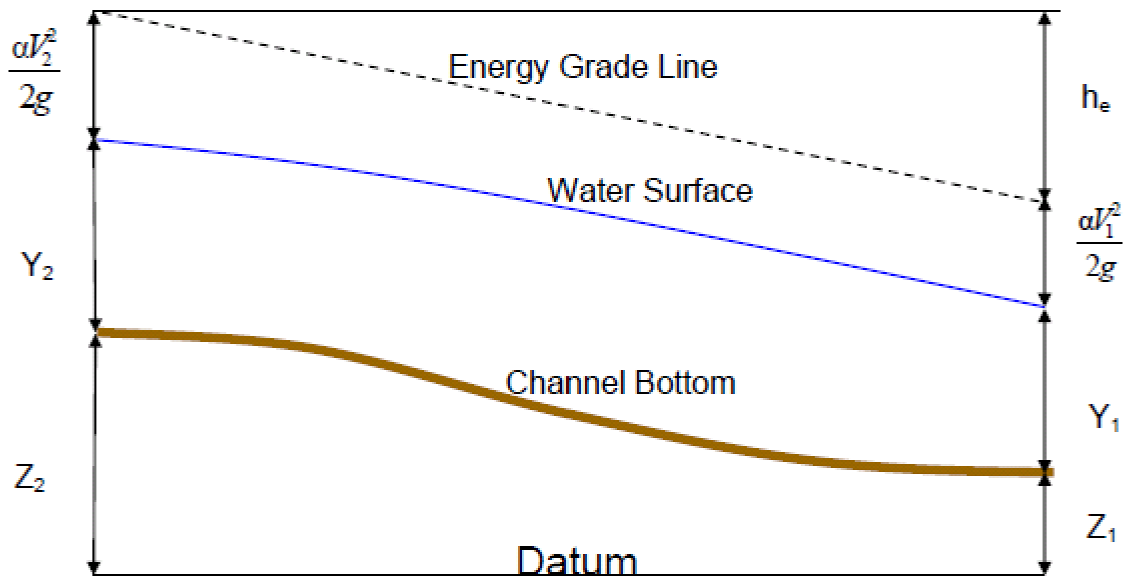

Figure 23: Energy equation terms
(Source: RAS reference manual, 2008)

The term h_e is the energy head loss between two cross sections as water travels downstream from cross section 2 to cross section 1. This energy is lost due to friction and to contraction or expansion losses. The energy lost is the energy converted from mechanical energy (potential or kinetic) to heat energy. The energy head loss is computed using the following equation:

$$h_e = LS_f + C\left[\frac{\alpha_2 V_2^2}{2g} - \frac{\alpha_1 V_1^2}{2g}\right] \qquad \text{Equation 15}$$

Where,

L = discharge weighted reach length and is expressed as:

$$L = \frac{L_{LOB}Q_{LOB} + L_{CH}Q_{CH} + L_{ROB}Q_{ROB}}{Q_{LOB} + Q_{CH} + Q_{ROB}}$$ Equation 16

S_f = representative friction slope between two sections

C = expansion or contraction loss coefficient

The flow resistance in the above equation expressed as S_f is related to flow velocity by using a resistance coefficient (Mays, 1996). The commonly used relationships are the Manning's formula, Darcy-Weisbach formula and the Chezy formula. HEC-RAS uses Manning's relationship for computations.

$$Q = KS_f^{1/2}$$ Equation 17

$$K = \frac{1.486}{n} AR_h^{2/3}$$ = Conveyance for subdivision Equation 18

n = Manning's roughness coefficient for subdivision

A = flow area of subdivision

R = hydraulic radius for subdivision (area/wetted perimeter)

To calculate total conveyance and velocity coefficient, RAS divides the flow in the overbank areas using the input cross-section as n-value break points. This is the default approach. The alternative conveyance method, which computes between every coordinate point in the overbanks, can be specified if needed. The later method was used in HEC-2 program.

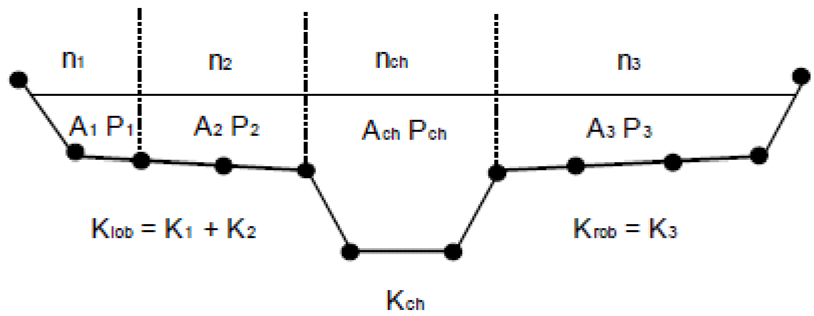

Figure 24: HEC-RAS default conveyance subdivision approach
(Source: RAS reference manual, 2008)

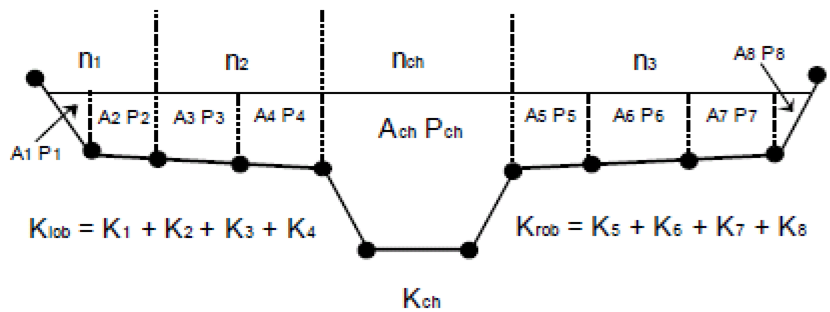

Figure 25: Alternative conveyance subdivision method
(Source: RAS reference manual, 2008)

It is not known which method is more accurate (HEC-RAS reference manual, Version 4.0, March 2008). For the WPBW, the default conveyance subdivision method was used as it produced higher water surface elevation than the alternative conveyance subdivision method.

RAS computes single mean energy at each cross section, i.e., only one water surface elevation is computed for each cross section. Hence it is necessary to compute the velocity head weighting coefficient, α, to compute the mean kinetic energy. It is computed based on conveyance in overbanks and main channel. The representative friction slope is calculated by default as per the average conveyance equation.

$$a = \frac{(A_t)^2 \left[\frac{K_{lob}^3}{A_{lob}^2} + \frac{K_{ch}^3}{A_{ch}^2} + \frac{K_{rob}^3}{A_{rob}^2} \right]}{K_t^3}$$

Equation 19

$$\overline{S}_f = \left(\frac{Q_1 + Q_2}{K_1 + K_2} \right)^2$$

Equation 20

An iterative procedure can be found in details in HEC-RAS reference manual (Version 4.0, March 2008).

The Manning's n values used in the RAS model for WPBW were based on the cross section intersection with landuse/landcover polygon and are shown in the following Table 12:

Table 12: Manning's 'n' values and Landuse/Landcover

Landuse/Landcover	**Manning's n value**
Forest	0.15
Urban	0.05
Wetlands	0.1
Agriculture	0.05
Barren land	0.03
Channel	0.035

Table 13 shows recommended contraction and expansion coefficients for subcritical flow. As many cross sections were digitized for WPBW, the change in cross sections was small and flow transitions were assumed to be gradual. Hence, contraction and expansion coefficients of 0.1 and 0.3 were used. These coefficients represent the value for 'C' in equation 15. When the velocity head upstream is greater than the velocity head downstream, the program assumed expansion was occurring between channel cross sections and used the expansion coefficient to compute the energy loss.

When the velocity head downstream was greater than velocity head upstream, the program assumed contraction was occurring between channel cross sections and used the contraction coefficient to compute the energy loss.

Table 13: Sub critical flow contraction and expansion coefficients

	Contraction	Expansion
No transition loss	0.0	0.0
Gradual transitions	0.1	0.3
Typical Bridge sections	0.3	0.5
Abrupt transitions	0.6	0.8

(Source: RAS reference manual, 2008)

Equation 14 is valid for gradually varying flows. However, there were several times when the flow changed from subcritical to supercritical and vice-versa. Such changing flows are rapidly varying flows. When rapidly varying flows occurred such as near bridges and culverts, stream junctions etc or due to significant change in channel slope, RAS uses a momentum equation for computing water surface profiles.

$$\frac{\partial Q}{\partial t} + \frac{\partial QV}{\partial x} + gA\left(\frac{\partial z}{\partial x} + S_f\right) = 0 \qquad \text{Equation 21}$$

The river reach schematic for WPBW created in the GIS environment is shown in figure below. RAS requires boundary conditions at the upstream and downstream end of the reach for the subcritical flow and at the upstream end for the supercritical flow. Critical depth and junctions were used as the boundary conditions for WPBW. Junctions were formed while creating the geometry file using HEC-GeoRAS. Junctions were formed at the confluence of three or more reaches if the endpoints were snapped properly in GIS. Snapping can be done using the Snapping feature from the Editor toolbar. If needed, snapping tolerance can be changed using Options from the Editor toolbar and changing the snapping tolerance under the General tab.

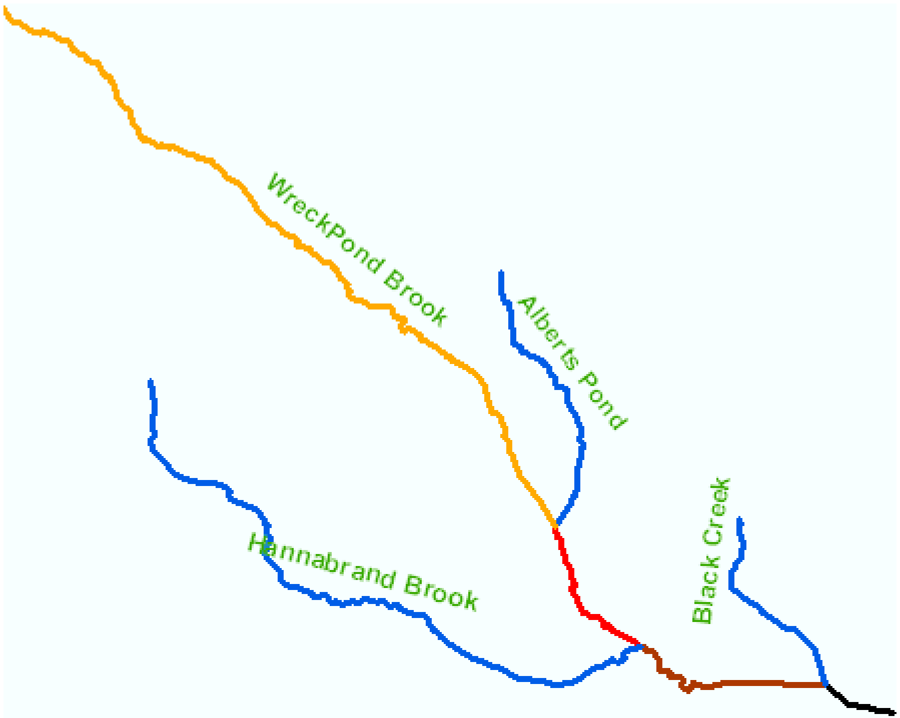

Figure 26: River reach schematic for WPBW

Table 14: Junctions for Reaches in WPBW

Reach Connectivity

	River	Reach	Up Junction	Up Storage Area	Dn Junction	Dn Storage Area
1	Alberts Pond	Tributary			2	
2	Black Creek	Tributary			4	
3	Hannabrand Brook	Tributary			3	
4	WreckPond Brook	Upper Reach			2	
5	WreckPond Brook	Middle Reach	2		3	
6	WreckPond Brook	Pond Reach	3		4	
7	WreckPond Brook	Outlet Reach	4			

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The input requirement for the RAS model setup is the channel and floodplain information in form of geometric data and hydrologic information in form of steady flow data. The geometric data was developed in the GIS environment using the HEC-GeoRAS interface, which was described in detail in the section '*Creating Geometry file in GIS environment using HEC-GeoRAS*'. The information flow between the GIS and RAS environments is shown in the flow diagram below.

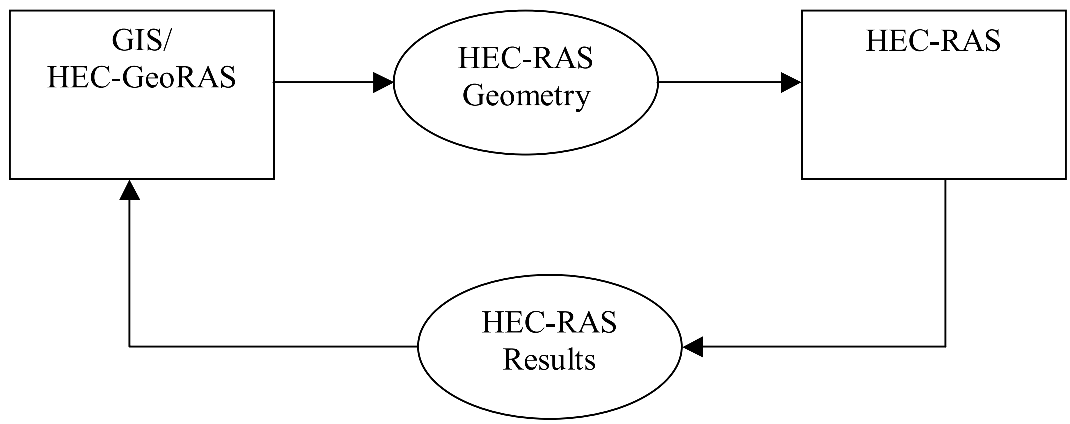


Figure 27: Information flow between GIS environment and HEC-RAS environment

HEC-RAS allows importing the geometry file in the GIS format. The geometry file created in the GIS environment does not capture enough information to model bridges/culverts. Hence, the bridges/culverts in WPBW were identified and surveyed to determine the size of the culvert openings. Figure 28 shows the bridges/culverts identified in the WPBW for hydraulic modeling.

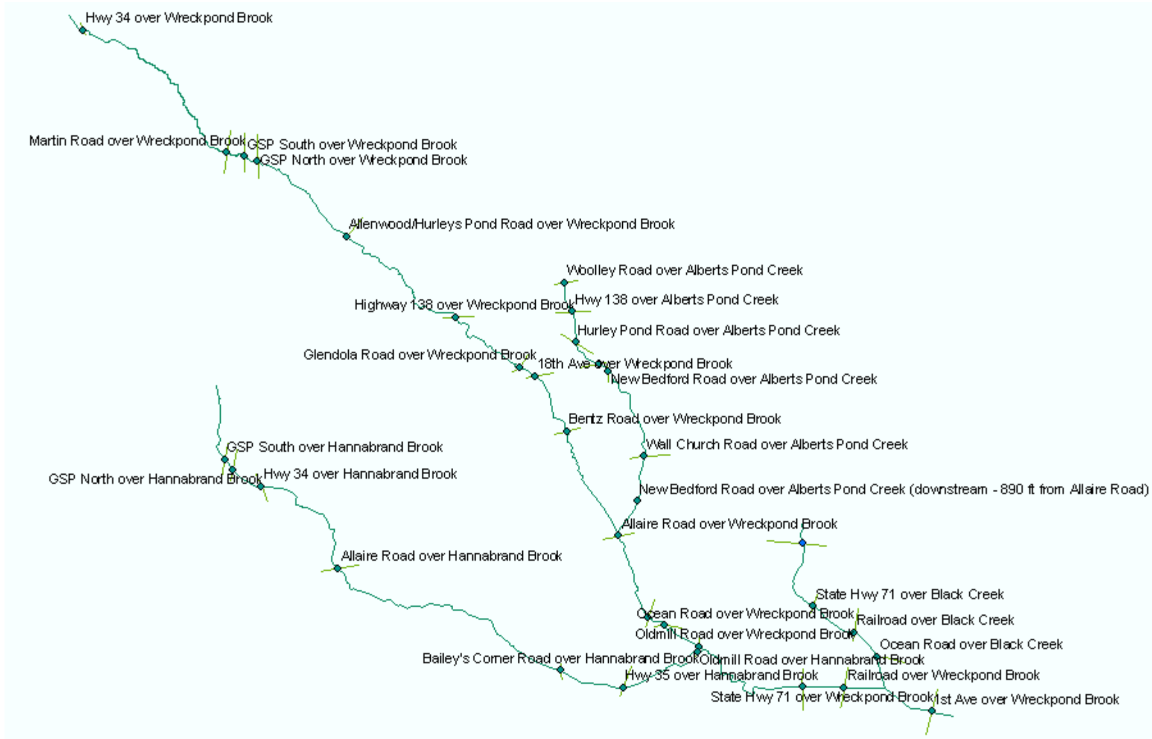


Figure 28: Bridge locations in WPBW

Table 15: Bridges in Wreck Pond Brook watershed: Name, Station and Deck width information.

WRECK POND BROOK			
	Reach	River Station	Deck Width (ft)
1	Upper Reach	39532.78 Hwy 34 over WPB	85
2	Upper Reach	32843.42 Martins Rd over WPB	35
3	Upper Reach	32217.32 GSP South over WPB	86
4	Upper Reach	31767.26 GSP North over WPB	86
5	Upper Reach	28078.39 Hurley's Pond Rd over WPB	30
6	Upper Reach	23283.20 Hwy 138 over WPB	130

7	Upper Reach	20256.59 Glendola Rd over WPB	24
8	Upper Reach	19697.75 18th Ave over WPB	27
9	Upper Reach	17661.51 Bentz Rd over WPB	18.2
10	Middle Reach	13885.32 Allaire Rd over WPB	32
11	Middle Reach	11078.51 Hwy 35 over WPB	54
12	Middle Reach	10472.65 Ocean Rd over WPB	47
13	Middle Reach	9134.251 Old mill Rd over WPB	46.5
14	Pond Reach	4733.073 Hwy 71 over WPB	54
15	Pond Reach	3475.023 Railroad over WPB	28
16	Outlet Reach	662.6741 1st Ave over WPB	57
HANNABRAND BROOK			
	Reach	River Station	Bridge Width
1	Tributary	18730.83 GSP South over HB	220
2	Tributary	18346.01 GSP North over HB	84
3	Tributary	17328.38 Hwy 34 over HB	138
4	Tributary	13190.01 Allaire Rd over HB	34
5	Tributary	5039.090 Baileys Corner Rd over HB	25
6	Tributary	3013.816 Hwy 35 over HB	49
7	Tributary	211.1309 Old mill Rd over HB	45
BLACK CREEK			
	Reach	River Station	Bridge Width
1	Tributary	5743.606 Warren Rd over BC	35

2	Tributary	3467.920 Hwy 71 over BC	48
3	Tributary	1980.029 Railroad over BC	37.5
4	Tributary	958.3540 Ocean Rd over BC	27
ALBERS POND CREEK			
	Reach	River Station	Bridge Width
1	Tributary	8940.879 Woolley Rd over APC	24.5
2	Tributary	7931.318 Hwy 138 over APC	130
3	Tributary	7006.292 Hurley's Pond Rd over APC	26
4	Tributary	5967.974 18th Ave over APC	31
5	Tributary	5611.098 New Bedford Rd over APC	32.83
6	Tributary	2472.996 Wall Church Rd over APC	26
7	Tributary	1100	26.33

Figure 29: 1st Avenue on Wreck Pond Brook (Source: H. Birckhead, 2008)

Figure 30: Glendola Road on Wreck Pond Brook (Source: K. Patel, 2007)

Figure 31: Hurley's Pond Road over Wreck Pond Brook (Source: K. Patel, 2007)

Figure 32: Highway 35 over Wreck Pond Brook (Source: K. Patel, 2008)

Figure 33: Highway 71 over Wreck Pond Brook (Source: K. Patel, 2008)

Figure 34: Ocean Avenue over Black Creek (Source: K. Patel, 2008)

Figure 35: Ocean Road over Wreck Pond Brook (Source: K. Patel, 2008)

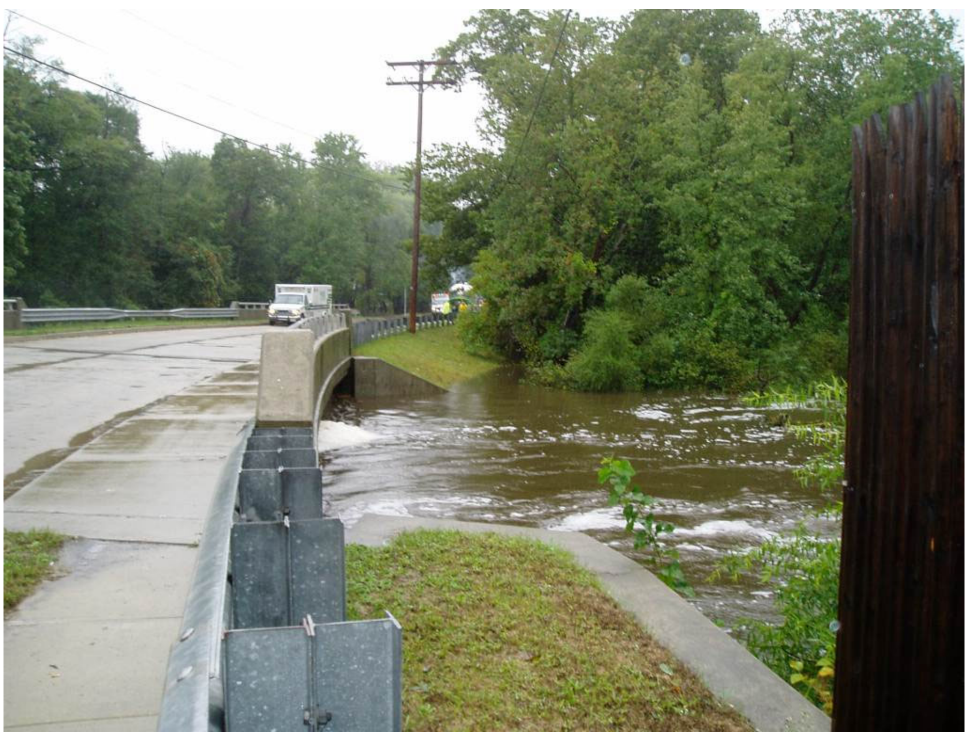


Figure 36: Old Mill Road over Hannabrand Brook (Source: Resident, WPBW)

Figure 37: Old Mill Road over Wreck Pond Brook (Source: K. Patel, 2008)

Figure 38: Railroad over Black Creek (Source: K. Patel, 2008)

Figure 39: Railroad over Wreck Pond Brook (Source: H. Birckhead, 2008)

Once the geometry data and steady flow data were entered, the model was ready to run. The RAS model was calibrated using observed data for the October 2005 storm event. The calibrated model was run then for the NRCS 100-year storm event flow values to compute water surface profiles and to delineate and compare the floodplain with DFIRM (draft maps provided by Mr. H. Rimawi, Medina consultant - contractor for FEMA).

4. CASE STUDY

4.1 Precipitation

Precipitation is the driving condition for rainfall-runoff process. For WPBW, precipitation data were available from two different sources to provide the storm events used in developing the hydrologic model. The storm events were analyzed for rainfall distribution and depth. Ideally the storm event during the summer season with the distribution pattern similar to the NRCS statistical storm distribution was the preferred choice for modeling. However, very few storm events will have the exact same pattern, if any. The summer storm event is preferable as it represents the average channel, floodplain and watershed conditions commonly used by water resources engineering consultants. A decision matrix was developed for storm event selection process for hydrologic modeling. It is shown in the table below.

Table 16: Wreck Pond Brook storm selection matrix

Wreck Pond Brook Storm Selection Matrix

Date	W1	W2	W3	W5	W6	W7	W8	W9	P, in	
3/28/2005	?	X	-	X	?	?	?	?	1.9	Data from NJ Network; seagirt gage, hrly record
4/2/2005	?	?	-	?	?	?	?	?	1.24	Data from NJ Network; seagirt gage, hrly record
5/20/2005	?	?	?	?	?	?	?	?	1.17	Data from NJ Network; seagirt gage, hrly record
6/27/2005	?	?	?	?	?	?	?	?	1.13	
10/15/2005	?	?	?	?	?	?	?	?	11	
1/3/2006	?	?	?	?	?	?	-	-	2.43	
1/8/2006	?	-	-	?	?	-	?	?	1.66	
4/22/2006	?	?	?	-	?	-	-	-	1.59	
6/24/2006	X	X	X	X	X	-	X	X	1.93	
7/6/2006	X	-	X	?	X	-	?	X	1.41	Nice Storm
8/26/2006	?	`	?	?	?	-	X	?	1.53	
9/2/2006	?	-	?	?	?	-	-	?	3.25	two storms?
11/8/2006	?	-	?	?	?	-	-	?	1.51	
2/14/2007	?	-	-	?	?	-	?	?	2.2	
3/2/2007	?	-	-	?	?	-	?	?	1.68	
4/12/2007	?	-	-	?	?	-	?	?	2.61	Nice Storm
4/15/2007	?	-	-	ok	?	-	-	?	2.24	long duration storm

?	stream gage data available for this station
?	Stream and storm data available and are acceptable
X	Both data sets available and are chosen for modeling
-	No stream gage data
?	Stream gage data is questionable
OK	Stream data available and could be used if none other is used
OK	data is 2nd choice, maybe 3rd
?	data available but is 2nd choice

three systems of gages from headwaters to outlet:
Wreck Pond Brook= W6->W9->W7->W1-W3
Hanabrand Brook= W5->W2
Black Creek = W8

(Source: J. E. Showler, New Jersey Department of Agriculture)

The storm events outside of early spring to late fall season were eliminated. Also, the storm events with very long spreads were second choice options for modeling if needed. The next consideration was for the precipitation depth. Based on field observations for an average antecedent moisture condition, at least 1 inch of precipitation would be needed to observe any significant change on the stream gages. Hence, only the storm events were analyzed with more than one inch of rainfall depth. The following table 17 shows the storms selected for model development based on the above-mentioned considerations. The description for the storm events also is found in the regional stormwater management plan report for WPBW (October, 2008), however it was felt important to describe here the storm conditions used for modeling. The rainfall hyetographs were created using public domain software HEC-DSSVue (version 1.2.10, September 2006), which can be downloaded from the USACE website (http://www.hec.usace.army.mil/software/hec-dss/hecdssvue-dssvue.htm).

Table 17: Summary of storm events used for model development

Storm Event	Depth, in	Duration, hours	Gage Source	Watershed
March 28, 2005	1.93	24	Sea Girt/ NJ Mesonet	Hannabrand Brook
June 24, 2006	1.94	8.5	Wall Twp./RISE	Wreck Pond Brook, Hannabrand Brook, Black Creek
July 6, 2006	1.41	7	Wall Twp./RISE	Wreck Pond Brook
April 12, 2007	2.55	7.5	Wall Twp./RISE	Black Creek

(Source: WPBW RSWMP report, 2008)

Figure 40: Weather station installed behind the library in Wall Township, NJ
(Source: http://www.wallnj.com/news/2004.fall.pdf)

Figure 41: NJ Mesonet weather station at Sea Girt, NJ
(Source: http://climate.rutgers.edu/njwxnet/dataviewer-netpt.php)

Table 18: NJ Mesonet Weather Station information:

City	Sea Girt
State	NJ
Alt. Name	KQ52
County	Monmouth
Elevation	2 m
Latitude	40.1203
Longitude	-74.0327
Type	Campbell Scientific
Network	Mesonet
Activation	2003-11-14

(Source: http://climate.rutgers.edu/njwxnet/dataviewer-netpt.php)

March 28, 2005. Precipitation data was retrieved from the New Jersey Mesonet weather station network gage located in Sea Girt, NJ. The Wall Township RISE network gage was not online at the time. The Sea Girt gage data was measured in 1-hour increments, while all RISE network data was measured at 6-minute intervals. There was approximately 0.34 inches of precipitation on March 23rd, 5 days prior to the modeled storm event. This storm event originally was considered "marginal" due to the time of year – at the beginning of leaf out. Vegetation would not be fully expanded and conditions might not be comparable to other events, which took place later in the growing season. However, the storm depth of 1.93 inches met the criteria for sufficient rainfall and there was an adequate gage response at the Hannabrand Brook gages (W5 and W2).

Additionally, the gage at W2 stopped working later in the project, which limited available data for use in modeling. Due to the time of year and prior rain event, it was assumed that an antecedent moisture condition above 'average" might be present which could be modeled with higher than normal curve numbers and/or wet soils being construed as connected impervious cover to imitate quicker watershed responses.

Table 19: Rainfall for Storm event of March 28, 2005

Date & Time	P (in)
28Mar2005, 00:00	0.02
28Mar2005, 01:00	0.02
28Mar2005, 02:00	0.06
28Mar2005, 03:00	0.06
28Mar2005, 04:00	0.1
28Mar2005, 05:00	0.06
28Mar2005, 06:00	0
28Mar2005, 07:00	0.03
28Mar2005, 08:00	0.06
28Mar2005, 09:00	0.09
28Mar2005, 10:00	0.05
28Mar2005, 11:00	0.07
28Mar2005, 12:00	0.08
28Mar2005, 13:00	0.08
28Mar2005, 14:00	0.15
28Mar2005, 15:00	0.02
28Mar2005, 16:00	0.02
28Mar2005, 17:00	0.11
28Mar2005, 18:00	0.04
28Mar2005, 19:00	0.36
28Mar2005, 20:00	0.44
28Mar2005, 21:00	0
28Mar2005, 22:00	0.01

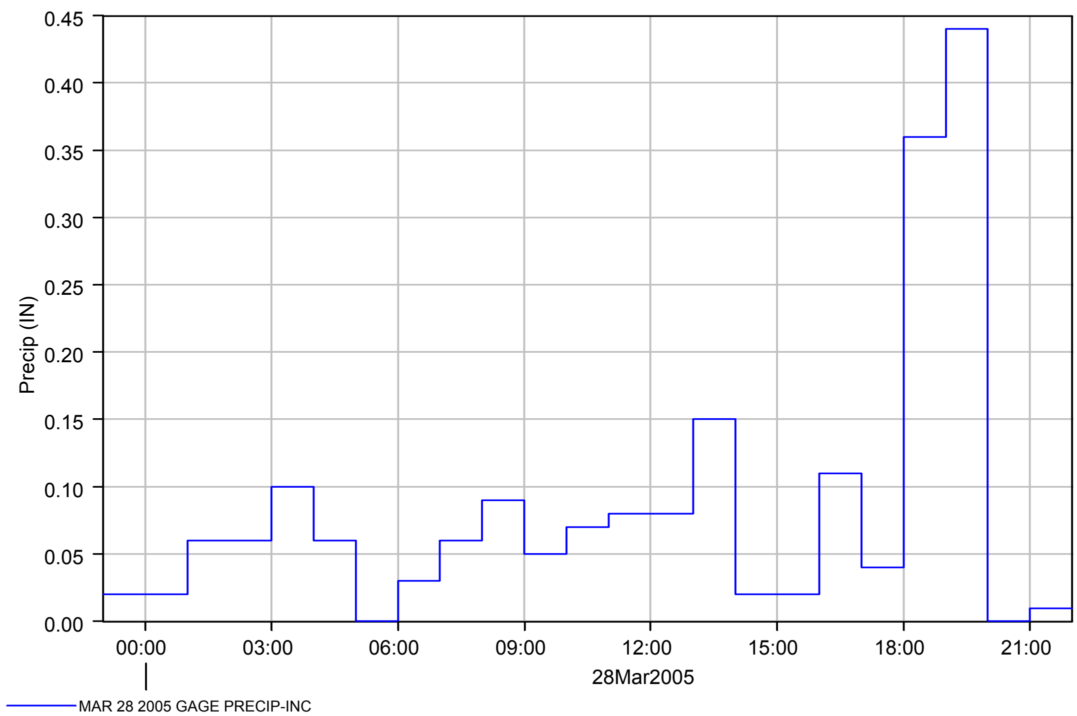


Figure 42: Hyetograph for the storm event of March 28, 2005

October 14, 2005. The storm event of October 14, 2005 was spread over a one-week period with total measured precipitation of 12.86 inch. As can be seen from the hyetograph, most of the precipitation occurred from October 12 to October 14. The peak flows and runoff volume generated were comparable to the NRCS 100-year storm event. Hence this storm event was used to model high flow events.

Table 20: Rainfall for Storm event of October 14, 2005

Date & Time	**P (in)**	Date & Time	**P (in)**	Date & time	**P (in)**
12Oct2005, 01:12	0.01	13Oct2005, 16:12	0.48	15Oct2005, 08:12	0.02
12Oct2005, 02:12	0.03	13Oct2005, 17:12	0.65	15Oct2005, 09:12	0.02
12Oct2005, 03:12	0.11	13Oct2005, 18:12	0.68	15Oct2005, 10:12	0.01
12Oct2005, 04:12	0.11	13Oct2005, 19:12	0.86	15Oct2005, 11:12	0.01
12Oct2005, 05:12	0.49	13Oct2005, 20:12	0.75	15Oct2005, 12:12	0.03
12Oct2005, 06:12	0.21	13Oct2005, 21:12	0.59	15Oct2005, 13:12	0.02

12Oct2005, 07:12	0.18	13Oct2005, 22:12	0.29	15Oct2005, 14:12	0.01
12Oct2005, 08:12	0.12	13Oct2005, 23:12	0.03	15Oct2005, 15:12	0.02
12Oct2005, 09:12	0.21	14Oct2005, 00:12	0.03	15Oct2005, 16:12	0.02
12Oct2005, 10:12	0.01	14Oct2005, 01:12	0.02	15Oct2005, 17:12	0.01
12Oct2005, 11:12	0.26	14Oct2005, 02:12	0.05	15Oct2005, 18:12	0.02
12Oct2005, 12:12	0.19	14Oct2005, 03:12	0.08	15Oct2005, 19:12	0.02
12Oct2005, 13:12	0.18	14Oct2005, 04:12	0.07	15Oct2005, 20:12	0.02
12Oct2005, 14:12	0.34	14Oct2005, 05:12	0.11	15Oct2005, 21:12	0.01
12Oct2005, 15:12	0.12	14Oct2005, 06:12	0.18	15Oct2005, 22:12	0.02
12Oct2005, 16:12	0.1	14Oct2005, 07:12	0.26	15Oct2005, 23:12	0.02
12Oct2005, 17:12	0.23	14Oct2005, 08:12	0.15	16Oct2005, 00:12	0.02
12Oct2005, 18:12	0.02	14Oct2005, 09:12	0.1	16Oct2005, 01:12	0.02
12Oct2005, 19:12	0.25	14Oct2005, 10:12	0.05	16Oct2005, 02:12	0.02
12Oct2005, 20:12	0.38	14Oct2005, 11:12	0.04	16Oct2005, 03:12	0.04
12Oct2005, 21:12	0.34	14Oct2005, 12:12	0.02	16Oct2005, 04:12	0.08
12Oct2005, 22:12	0.1	14Oct2005, 13:12	0.02	16Oct2005, 05:12	0.08
12Oct2005, 23:12	0.06	14Oct2005, 14:12	0.06	16Oct2005, 06:12	0.04
13Oct2005, 00:12	0.01	14Oct2005, 15:12	0.04	16Oct2005, 07:12	0.02
13Oct2005, 01:12	0.02	14Oct2005, 16:12	0.04	16Oct2005, 08:12	0.02
13Oct2005, 02:12	0.03	14Oct2005, 17:12	0.04	16Oct2005, 09:12	0.02
13Oct2005, 03:12	0.05	14Oct2005, 18:12	0.01	16Oct2005, 10:12	0.01
13Oct2005, 04:12	0.03	14Oct2005, 19:12	0.01	16Oct2005, 11:12	0.01
13Oct2005, 05:12	0.04	14Oct2005, 20:12	0.01	16Oct2005, 12:12	0.01
13Oct2005, 06:12	0.03	14Oct2005, 21:12	0.01	16Oct2005, 13:12	0.02
13Oct2005, 07:12	0.11	14Oct2005, 22:12	0.02	16Oct2005, 14:12	0.02
13Oct2005, 08:12	0.31	14Oct2005, 23:12	0.01	16Oct2005, 15:12	0.02
13Oct2005, 09:12	0.47	15Oct2005, 00:12	0.01	16Oct2005, 16:12	0.02
13Oct2005, 10:12	0.02	15Oct2005, 01:12	0.05	16Oct2005, 17:12	0.02
13Oct2005, 11:12	0.01	15Oct2005, 02:12	0.07	16Oct2005, 18:12	0.01
13Oct2005, 12:12	0.1	15Oct2005, 03:12	0.06	16Oct2005, 19:12	0.01
13Oct2005, 13:12	0.05	15Oct2005, 04:12	0.02	16Oct2005, 20:12	0.01
13Oct2005, 14:12	0.01	15Oct2005, 05:12	0.01	16Oct2005, 21:12	0.01
13Oct2005, 15:12	0.66	15Oct2005, 06:12	0.02	16Oct2005, 22:12	0.01
		15Oct2005, 07:12	0.03	16Oct2005, 23:12	0.01

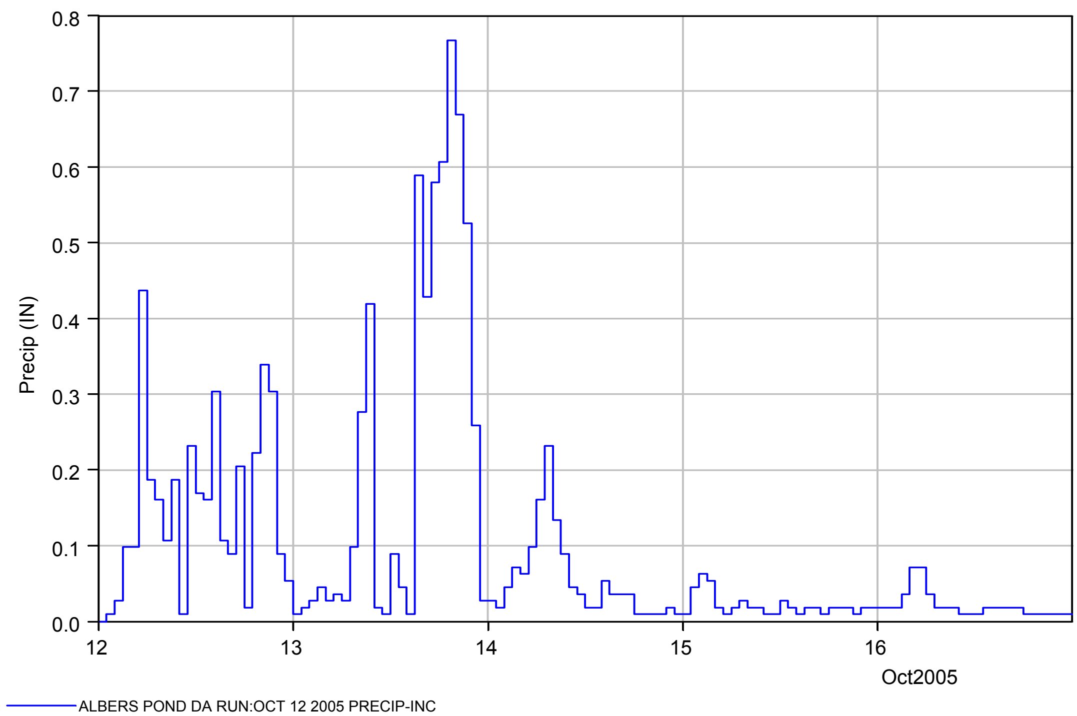


Figure 43: Hyetograph for Storm event of October 14, 2005

June 24, 2006. The storm had heavy rains totaling about 2 inches over a 24-hour period. However, there was an initial rainfall event, followed by several hours of no precipitation, subsequently followed by the "main" storm event, which constituted 1.84 inches of rain. Several attempts using the full 1.94 inches of precipitation failed to produce a good match to the gage hydrographs. Therefore, it was decided to use the 1.84 inch precipitation event as the storm event, and to account for the prior rainfall as an antecedent moisture condition. The assumption was that this prior rainfall was sufficient to load vegetation, fill voids and sufficiently wet soils such that the watershed response would be more characteristic of a higher curve number and/or impervious areas that were directly connected to the stream, since soils might be saturated and would convey runoff rather than contain it.

Table 21: Rainfall for storm event of June 24, 2006

Date & Time	Rainfall (in)	Date & Time	Rainfall (in)
6/24/06 3:18 AM	0.05	6/24/06 7:42 PM	0.01
6/24/06 3:24 AM	0.01	6/24/06 7:48 PM	0.01
6/24/06 3:30 AM	0.01	6/24/06 7:54 PM	0.02
6/24/06 3:36 AM	0.01	6/24/06 8:00 PM	0.02
6/24/06 6:42 AM	0.01	6/24/06 8:06 PM	0.02
6/24/06 3:06 PM	0.1	6/24/06 8:12 PM	0.02
6/24/06 3:12 PM	0.18	6/24/06 8:18 PM	0.02
6/24/06 3:18 PM	0.1	6/24/06 8:24 PM	0.02
6/24/06 3:36 PM	0.08	6/24/06 8:30 PM	0.02
6/24/06 3:42 PM	0.09	6/24/06 8:36 PM	0.01
6/24/06 3:48 PM	0.01	6/24/06 8:42 PM	0.03
6/24/06 3:54 PM	0.01	6/24/06 8:48 PM	0.01
6/24/06 5:06 PM	0.01	6/24/06 8:54 PM	0.01
6/24/06 5:42 PM	0.05	6/24/06 9:00 PM	0.01
6/24/06 5:48 PM	0.05	6/24/06 9:06 PM	0.01
6/24/06 5:54 PM	0.02	6/24/06 9:18 PM	0.01
6/24/06 6:00 PM	0.11	6/24/06 9:30 PM	0.01
6/24/06 6:06 PM	0.08	6/24/06 9:42 PM	0.01
6/24/06 6:12 PM	0.02	6/24/06 9:48 PM	0.01
6/24/06 6:18 PM	0.01	6/24/06 9:54 PM	0.02
6/24/06 6:24 PM	0.01	6/24/06 10:00 PM	0.02
6/24/06 6:30 PM	0.03	6/24/06 10:06 PM	0.01
6/24/06 6:36 PM	0.02	6/24/06 10:12 PM	0.01
6/24/06 6:42 PM	0.08	6/24/06 10:18 PM	0.03
6/24/06 6:48 PM	0.11	6/24/06 10:24 PM	0.02
6/24/06 6:54 PM	0.04	6/24/06 10:30 PM	0.03
6/24/06 7:00 PM	0.02	6/24/06 10:36 PM	0.02
6/24/06 7:06 PM	0.02	6/24/06 10:42 PM	0.02
6/24/06 7:12 PM	0.01	6/24/06 10:48 PM	0.01
6/24/06 7:18 PM	0.01	6/24/06 10:54 PM	0.01
6/24/06 7:24 PM	0.02	6/24/06 11:00 PM	0.01

6/24/06 7:30 PM	0.02	6/24/06 11:06 PM	0.01
6/24/06 7:36 PM	0.02	6/24/06 11:30 PM	0.01

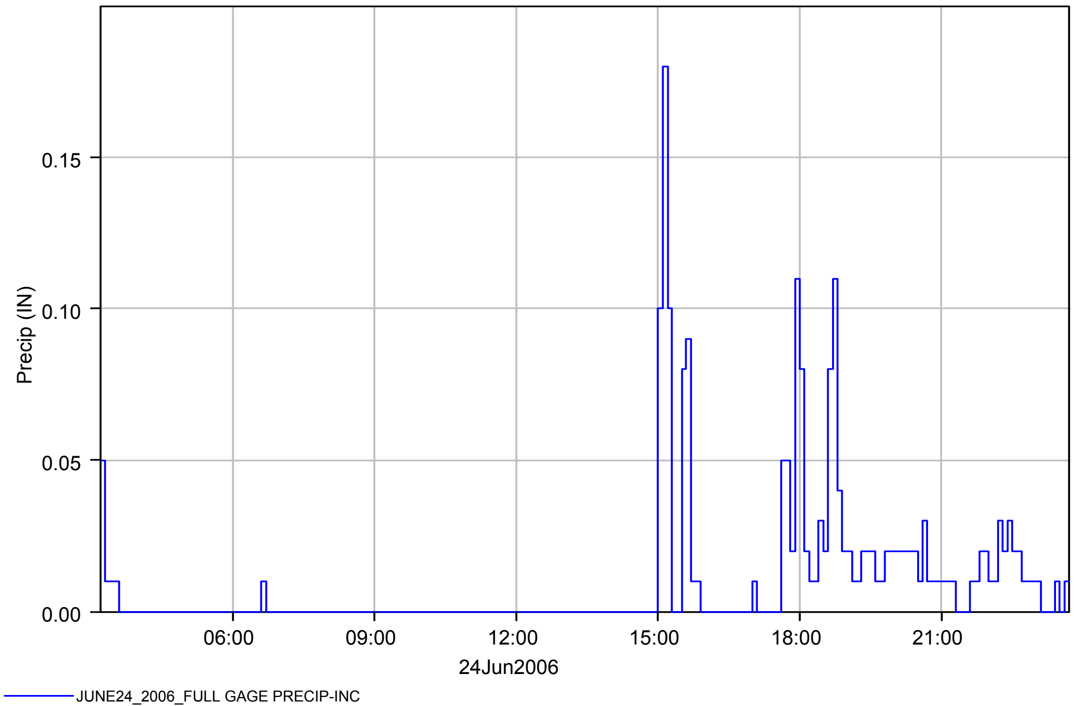


Figure 44: Hyetograph for June 24, 2006 showing small burst followed by storm several hours later

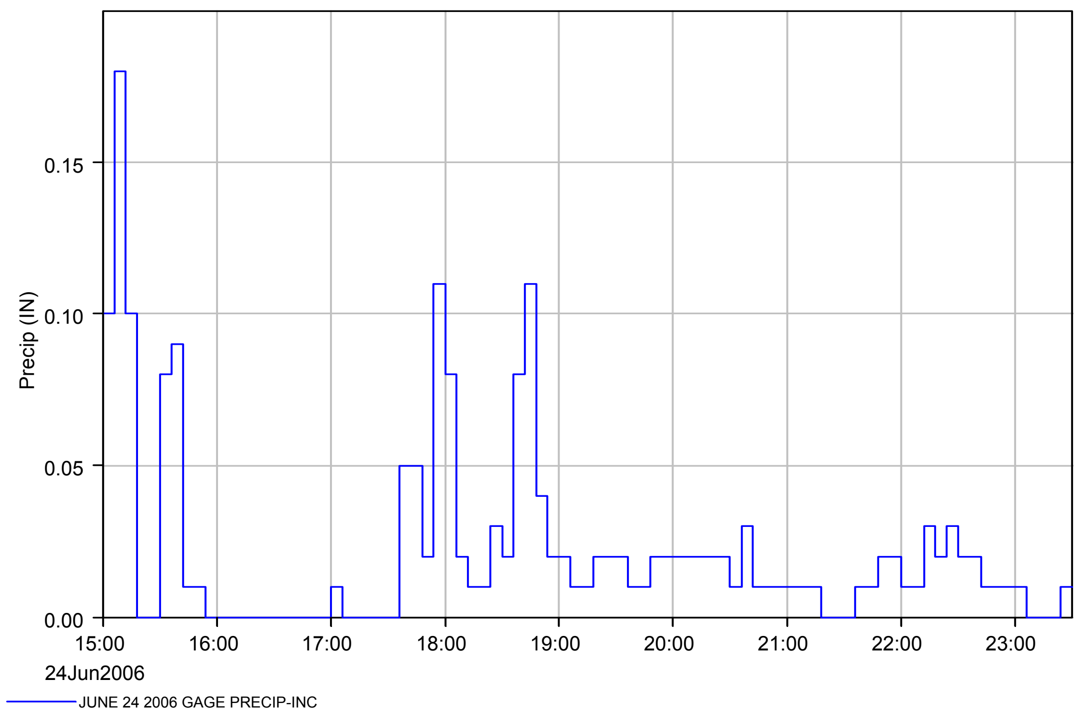


Figure 45: Hyetograph for the storm event of June 24, 2006

July 6, 2006. This event was preceded by several days of precipitation totaling 1.35 inches. Typically storm events during July would be considered to be either average or below average antecedent conditions. However, there was sufficient rainfall prior to the selected event to maintain either average or above-average antecedent conditions. The July 6th event total was 1.41 inches, which was somewhat lower than desirable, however the gage provided a sufficient distribution pattern and the stream gage response was adequate.

Table 22: Rainfall for storm event of July 6, 2006

Date & Time	P (in)	Date & Time	P (in)
06Jul2006, 00:48	0.01	06Jul2006, 04:18	0.01
06Jul2006, 00:54	0.01	06Jul2006, 04:24	0.02
06Jul2006, 01:00	0	06Jul2006, 04:30	0.02
06Jul2006, 01:06	0.02	06Jul2006, 04:36	0.03
06Jul2006, 01:12	0.01	06Jul2006, 04:42	0.03

06Jul2006, 01:18	0.01	06Jul2006, 04:48	0.02
06Jul2006, 01:24	0.01	06Jul2006, 04:54	0.03
06Jul2006, 01:30	0.01	06Jul2006, 05:00	0.03
06Jul2006, 01:36	0.01	06Jul2006, 05:06	0.01
06Jul2006, 01:42	0.01	06Jul2006, 05:12	0.01
06Jul2006, 01:48	0.02	06Jul2006, 05:18	0.01
06Jul2006, 01:54	0.01	06Jul2006, 05:24	0.02
06Jul2006, 02:00	0.03	06Jul2006, 05:30	0.04
06Jul2006, 02:06	0.01	06Jul2006, 05:36	0.03
06Jul2006, 02:12	0.03	06Jul2006, 05:42	0.03
06Jul2006, 02:18	0.03	06Jul2006, 05:48	0.01
06Jul2006, 02:24	0.02	06Jul2006, 05:54	0.02
06Jul2006, 02:30	0.03	06Jul2006, 06:00	0.01
06Jul2006, 02:36	0.03	06Jul2006, 06:06	0.02
06Jul2006, 02:42	0.04	06Jul2006, 06:12	0.03
06Jul2006, 02:48	0.04	06Jul2006, 06:18	0.01
06Jul2006, 02:54	0.06	06Jul2006, 06:24	0.02
06Jul2006, 03:00	0.04	06Jul2006, 06:30	0.02
06Jul2006, 03:06	0.05	06Jul2006, 06:36	0.02
06Jul2006, 03:12	0.04	06Jul2006, 06:42	0.01
06Jul2006, 03:18	0.02	06Jul2006, 06:48	0.02
06Jul2006, 03:24	0.02	06Jul2006, 06:54	0
06Jul2006, 03:30	0.02	06Jul2006, 07:00	0.01
06Jul2006, 03:36	0.06	06Jul2006, 07:06	0.01
06Jul2006, 03:42	0.04	06Jul2006, 07:12	0
06Jul2006, 03:48	0.04	06Jul2006, 07:18	0.01
06Jul2006, 03:54	0.01	06Jul2006, 07:24	0.01
06Jul2006, 04:00	0.02	06Jul2006, 07:30	0
06Jul2006, 04:06	0.01	06Jul2006, 07:36	0
06Jul2006, 04:12	0.01	06Jul2006, 07:42	0.01

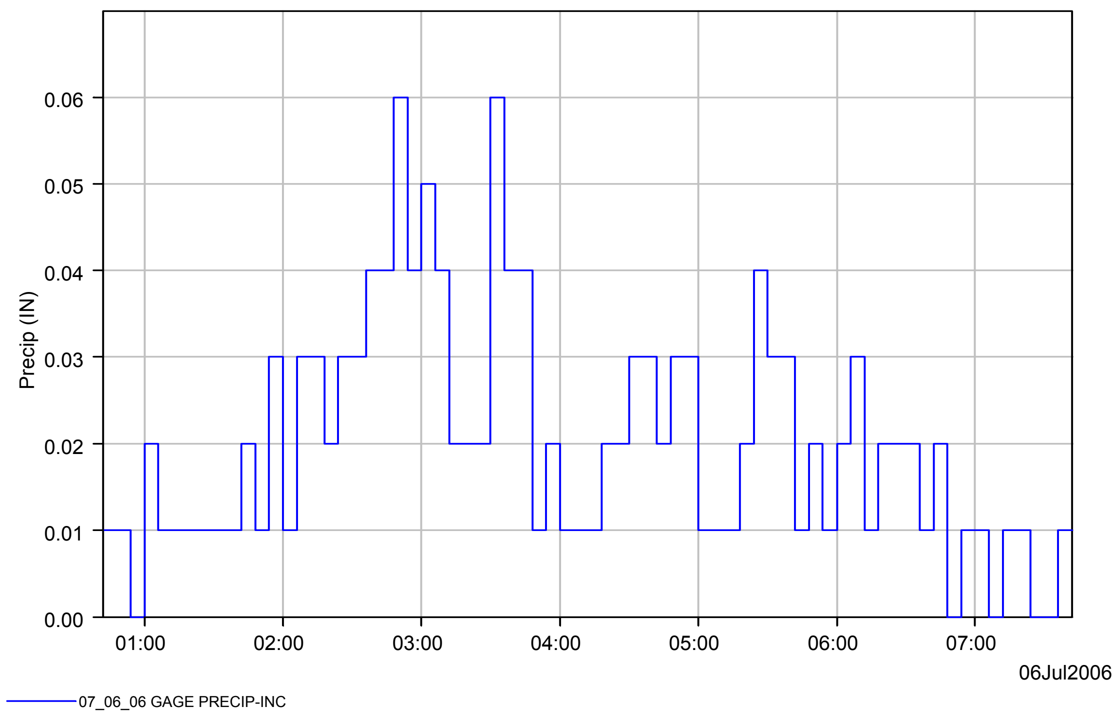


Figure 46: Hyetograph for the storm event of July 06, 2006

April 12, 2007. This event totaled 2.61 inches, however the initiation of the storm was "spotty" for several hours. Therefore, the storm was modeled as a 2.55" event to account for the main body of the storm. A storm event of 1.25" occurred approximately one week prior to the modeled event. Given the time of year (early in the growing season, cooler temperatures), a slightly higher than "average' runoff condition should exist in the watershed. Soils were assumed saturated and the base flow in streams as elevated.

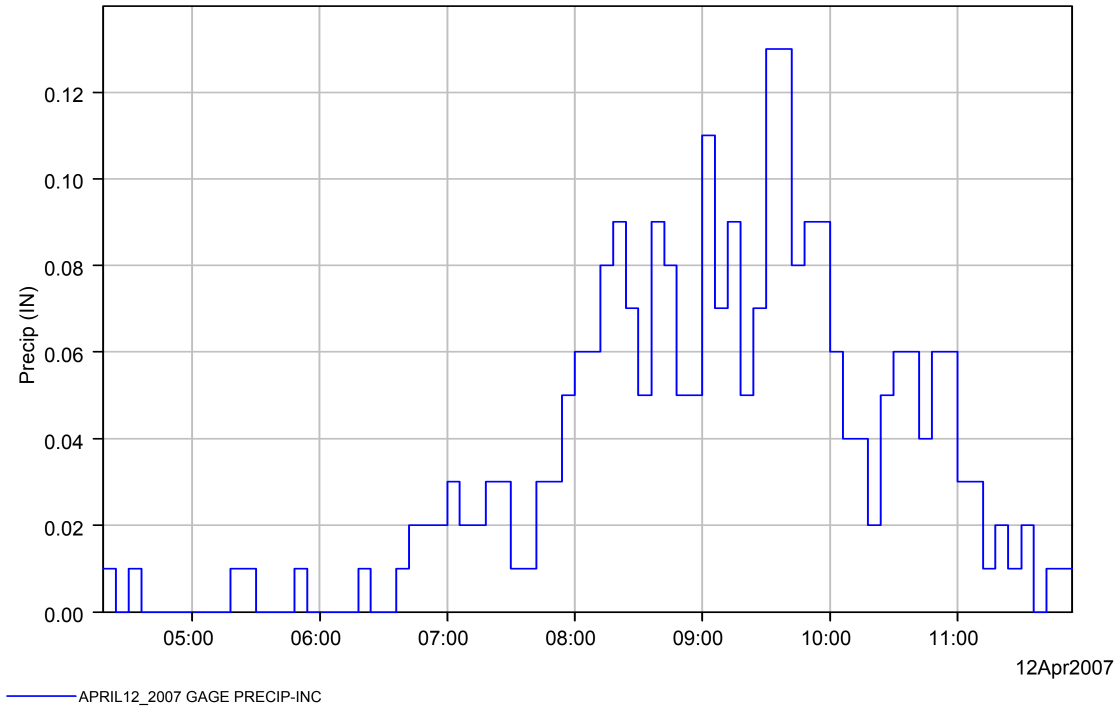


Figure 47: Rainfall hyetograph for the storm event of April 12, 2007

4.2 GIS based Hydrologic Modeling

4.2.1 Watershed delineation

The DEM provided by Monmouth County office of GIS is shown below. The accuracy of the GIS elevation data was photogrammetric certified as +/- one foot vertical and horizontal. The flow pattern and some road network are visualized clearly in the figure.

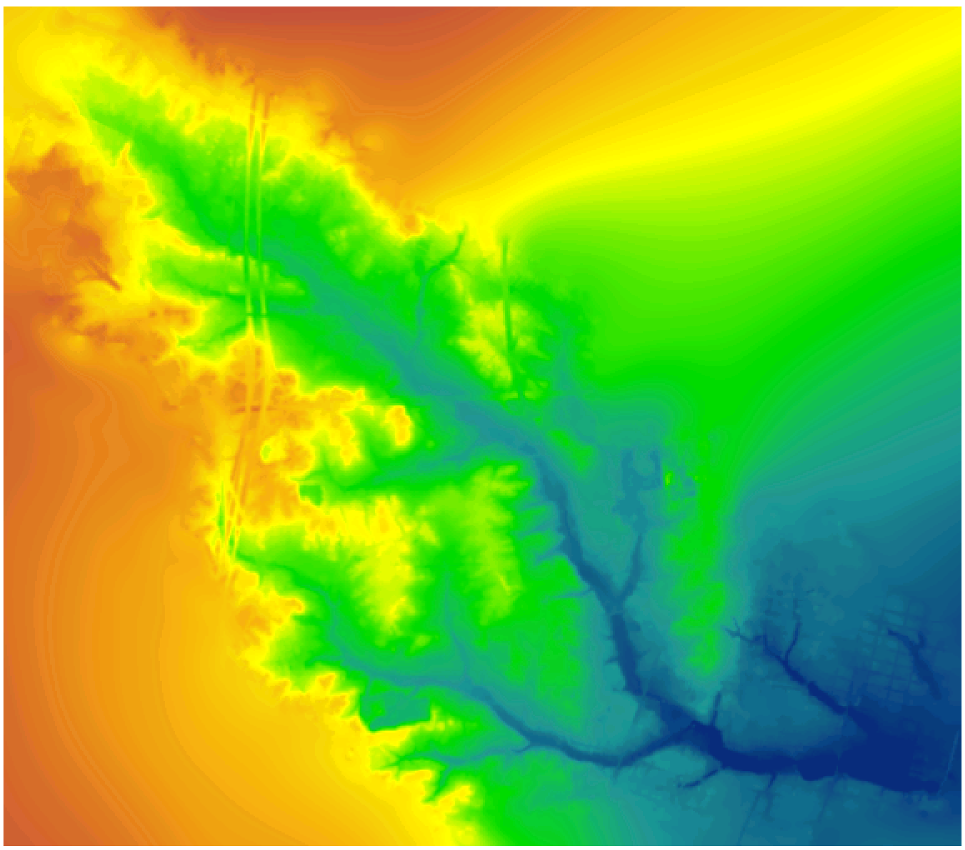


Figure 48: Digital Elevation Model for WPBW
(Source: Monmouth County Office of GIS)

All the features stored in Arc Hydro are called Hydrofeatures. Hydrofeatures are uniquely identified by attributes HydroID and HydroCode. The HydroID label uniquely identifies features within Arc Hydro. The HydroCode helps in linking Arc Hydro with other information systems by labeling a feature of interest with an identifier used by external systems. The Arc Hydro data model divides water resources data as shown in the figure below into five components: i) network; ii) drainage; iii) channel; iv) hydrography; and v) Time series.. This is described in detail in (Arc Hydro, GIS for Water Resources, Maidment, 2002).

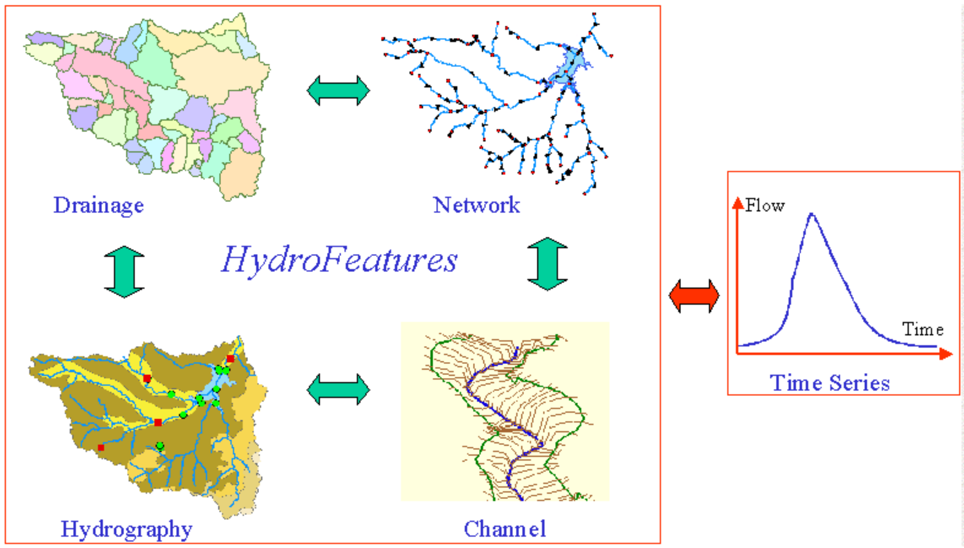


Figure 49: Arc Hydro data model
(Source: Arc Hydro for ArcGIS 9.2 - help files)

Arc Hydro tools are the set of public domain utilities that facilitates hydrologic analysis using data from Arc Hydro data model. The following shows the Arc Hydro toolbar.

Arc Hydro tools facilitated the automated generation of watershed boundaries in the GIS environment. However, Arc Hydro required slight manual editing for WPBW due to man-made drainage features at the drainage divides. The important steps during watershed delineation were preparing the DEM data for processing followed by creating a flow direction grid. The flow direction grid uses the elevation data from the elevation grid and computes and assigns the value of the direction of steepest descent to each cell.

This direction is calculated with an eight-direction pour point model. As an example, the following Figure 51 shows the value assigned to cells and the number of cells (count) with the assigned value.

Symbol	<VALUE>	Label	Count
	1	1	170407
	2	2	307519
	4	4	238816
	8	8	106456
	16	16	66399
	32	32	43768
	64	64	75873
	128	128	161907

Figure 50: Table showing counts for direction

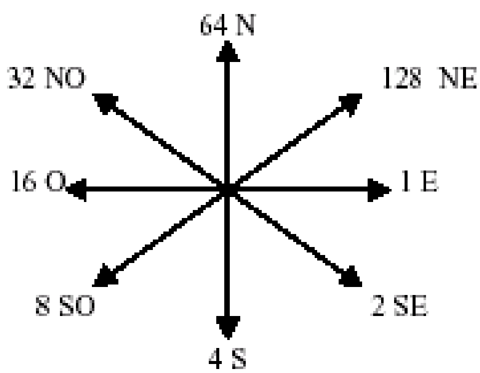


Figure 51: Values assigned in flow direction grid
(Source: Maidment, 2002)

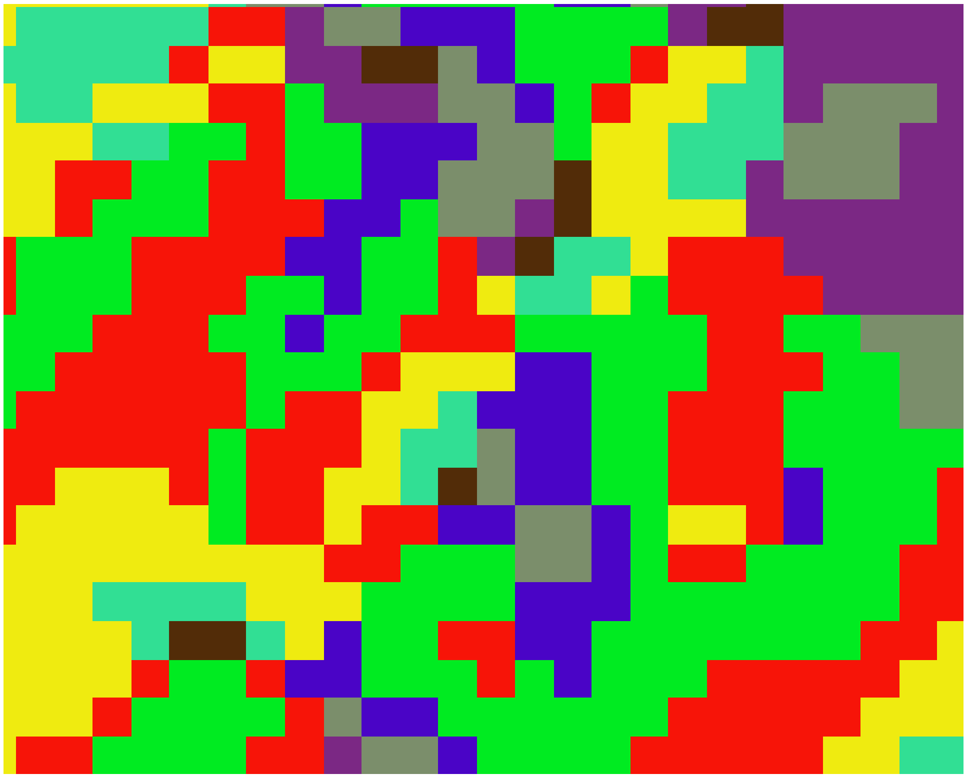


Figure 52: Flow direction grid zoomed in by a factor of 1:1000, and showing values assigned to each cell by quantitative color representation.

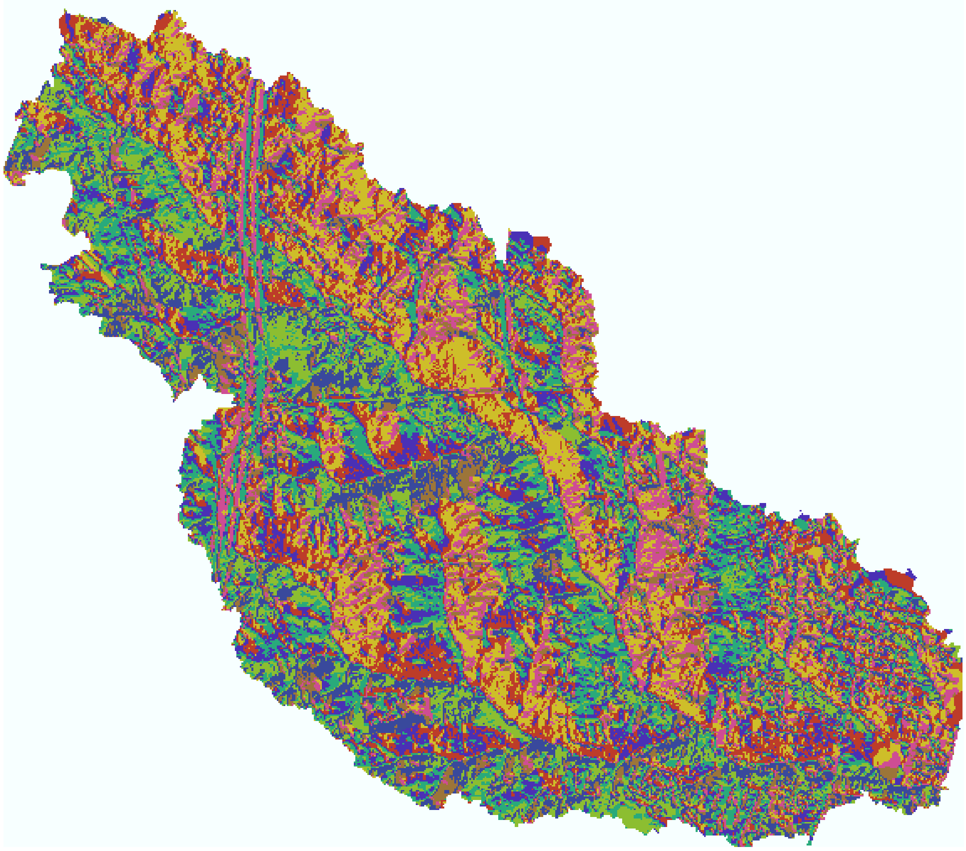


Figure 53: Flow direction grid created from the DEM

The next step is computing the flow accumulation grid. This uses flow direction grid values as input and computes the number of cells draining into each cell. The resulting grid is called the flow accumulation grid. The next task is to create the stream grid. Specifying the number of accumulated cells defines the stream grid. This grid has cell value of 1 for all cells. The following step is to create stream links from the stream grid by assigning a value of 1 for the first link, 2 for the second link and so on rather than a single value as assigned in stream grid raster. Figure 54 shows this outcome graphically.

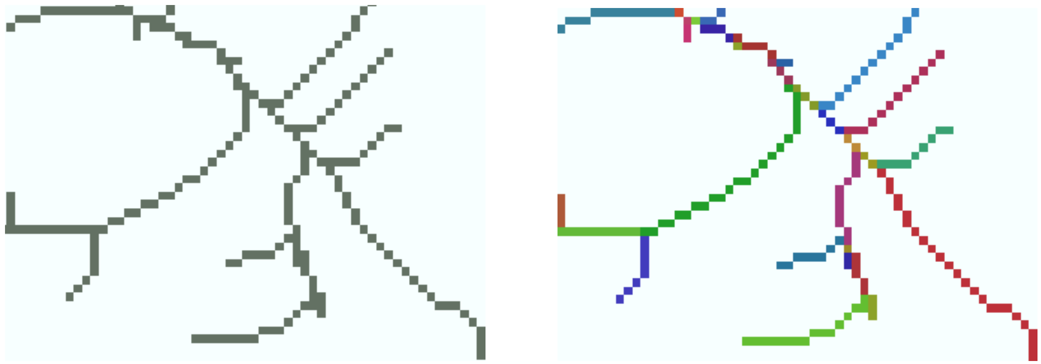


Figure 54: Stream grid and Stream link grid

The next task is to define the catchment grid. It is created using the flow direction grid to define the cells that drain into the stream link.

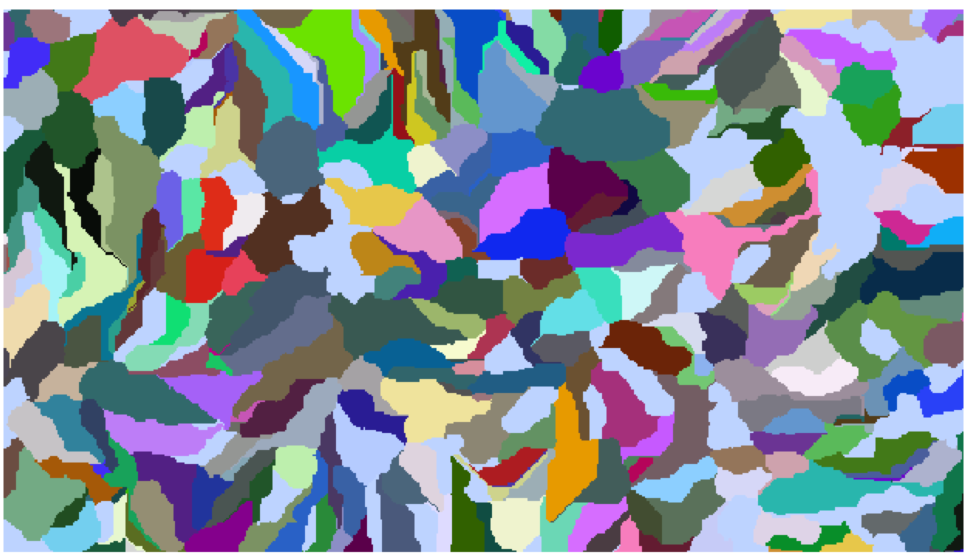


Figure 55: Catchments for WPBW

Finally, catchments are generated from the raster catchment grid by vectorizing into the catchment polygon. Creation of the catchment polygons, completes the preprocessing steps for delineation of the watershed and subwatersheds. Delineation of the watershed at an outlet point includes the entire upstream drainage area, whereas subwatershed delineation at an outlet point includes the incremental area between upstream of this outlet point and downstream of others.

Figure 56: Subwatersheds for WPBW
(Source: WPBW RSWMP, 2008)

The watershed was subdivided into 8 subwatersheds as shown in the figure above. Each of the sub watershed boundaries was field verified and corrected to account for factors such as underground piping that might transfer drainage from one sub area to another crossing the drainage divide.

4.2.2 Computing Subwatersheds CN values

As described in the model development section, the SCS curve number method was used as a loss model for rainfall-runoff computations. Hence, the curve number grid was developed using the HEC-GeoHMS toolbar and ArcGIS 9.2. HEC-GeoHMS is the toolbar developed by USACE to provide an interface between HEC-HMS and the GIS environment. The necessary data to develop the curve number grid described here is land use layer, soils layer and spatial analyst extension. Soil and land cover data was available for most of the country from NRCS (http://soildatamart.nrcs.usda.gov/) and USGS (http://eros.usgs.gov/products/landcover/lulc.php), respectively. For modeling WPBW, the necessary data was obtained from Monmouth County office of GIS (provided by. J. Brockwell).

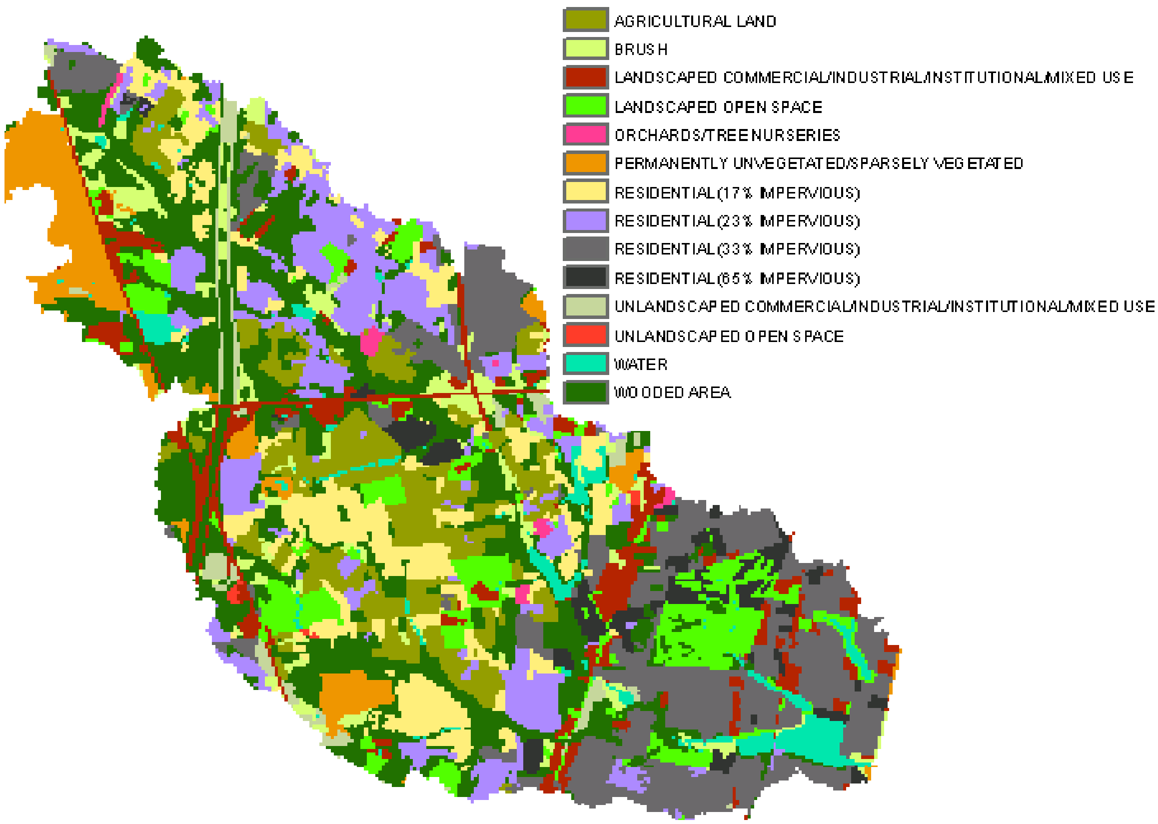


Figure 57: Land use data for WPBW

The first step in generating a curve number grid is to prepare land use data for analysis. The land use polygon for WPBW was rasterized using the spatial analyst toolbar to 'convert features to raster' option. The land use grid was reclassified to contain 14 categories as shown in the table below.

Table 23: Land use classification for developing CN grid

VALUE *	COUNT	SCS06
1	8120	WOODED AREA
2	3124	RESIDENTIAL (17% IMPERVIOUS)
3	691	UNLANDSCAPED COMMERCIAL/INDUSTRIAL/INSTITUTIONAL/MIXED USE
4	1441	BRUSH
5	3411	AGRICULTURAL LAND
6	1815	PERMANENTLY UNVEGETATED/SPARSELY VEGETATED
7	3548	RESIDENTIAL (23% IMPERVIOUS)
8	1151	RESIDENTIAL (65% IMPERVIOUS)
9	2324	LANDSCAPED COMMERCIAL/INDUSTRIAL/INSTITUTIONAL/MIXED USE
10	1011	WATER
11	2544	LANDSCAPED OPEN SPACE
12	159	ORCHARDS/TREE NURSERIES
13	5554	RESIDENTIAL (33% IMPERVIOUS)
14	44	UNLANDSCAPED OPEN SPACE

A revised classification scheme containing greater or fewer categories can be developed using the spatial analyst toolbar option ‘Reclassify’. After this reclassification, the land use grid was converted to land use polygons using the spatial analyst toolbar option ‘convert rasters to feature’. Every time a feature was converted to a raster or vice-versa, a small amount of information was lost in the conversion. Hence, such conversions should be done only when necessary. At this point the land use data was prepared for use in the task involving generation of the curve number grid.

Soils data also were prepared for analysis. The soils feature for WPBW had a hydrologic soil group assigned to each polygon. However, several polygons had assigned values as B/D or C/D. Soils data in this format were inadequate for completion of the computational analysis. A field ‘SoilCode’ was added into the soils layer and was assigned values from the hydrologic soil group from field ‘Hydgrp’. The B/D was replaced with soil type B and C/D was replaced with soil type C. If a significant amount of such classification exists, then care should be taken during this replacement to check whether the soils are drained or not. This replacement is done easily using the ‘field calculator'. The field calculator was accessed by opening attribute table of the feature and right clicking on the field heading for the calculation needed (Figure 58).

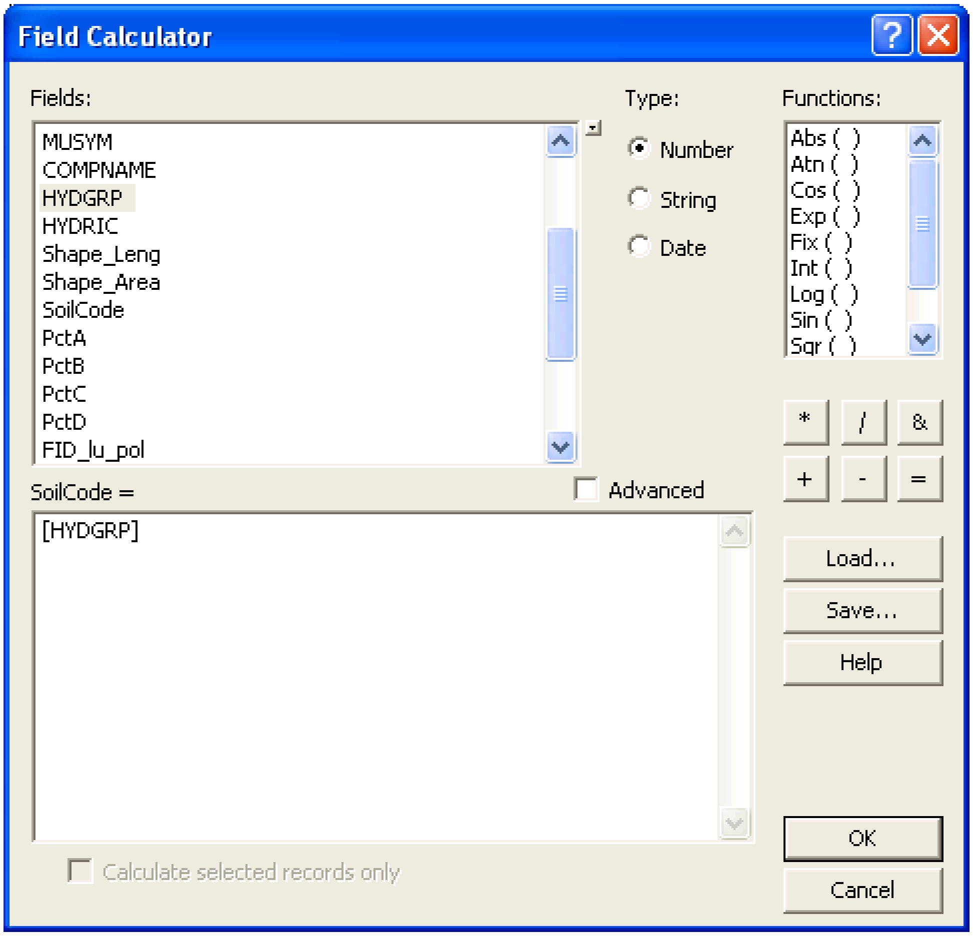


Figure 58: Field calculator – assigning HSG to soil polygons

Most polygons in the soils layer for WPBW had one assigned soil type. A few B/D and C/D assignments were refined as described above. At this point, the soils polygon in the soils layer had one assigned hydrologic soils group. Four additional fields were then created in the soils layer. They were PctA, PctB, PctC and PctD (percentage of A, B, C and D). One of these four fields was assigned a value of 100 based on the soil code and remaining 3 fields were assigned value=e 0 (because each polygon had only one soil code assigned to it). The percentage value can be assigned using field calculator (Figure 53).

Attributes of Soils1

	COMPNAME	HYDGRP	SoilCode	PctA	PctB	PctC	PctD
	LAKEWOOD	A	A	100	0	0	0
	PITS	A	A	100	0	0	0
	WOODSTOWN	C	C	0	0	100	0
	WOODSTOWN	C	C	0	0	100	0
	DOWNER	B	B	0	100	0	0
	EVESBORO	A	A	100	0	0	0
	SASSAFRAS	B	B	0	100	0	0
	SASSAFRAS	B	B	0	100	0	0
	SASSAFRAS	B	B	0	100	0	0
	EVESBORO	A	A	100	0	0	0
	EVESBORO	A	A	100	0	0	0
	EVESBORO	A	A	100	0	0	0
	LAKEWOOD	A	A	100	0	0	0
	EVESBORO	A	A	100	0	0	0
	EVESBORO	A	A	100	0	0	0
	EVESBORO	A	A	100	0	0	0
	EVESBORO	A	A	100	0	0	0
	WATER (LESS THAN		D	0	0	0	100
	WATER (LESS THAN		D	0	0	0	100
	WATER (LESS THAN		D	0	0	0	100

Record: 0 Show: All Selected Records

Figure 59: Percentage A, B, C and D fields created and assigned values

Once the soil layer attributes were created and values assigned, the soils and land use layers were combined. The ‘union’ tool from ArcTool box was used for this operation and is as shown in figure below.

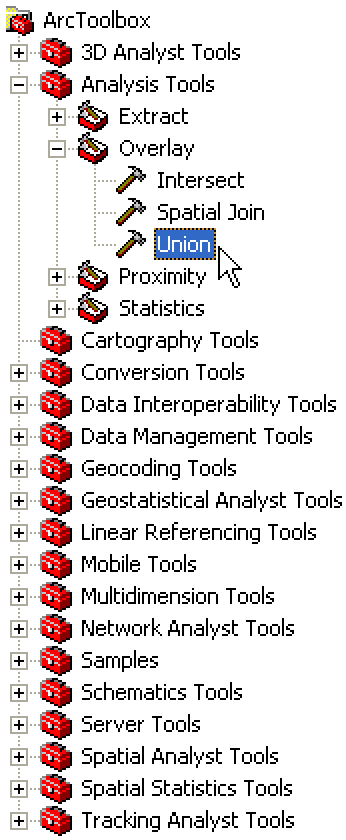


Figure 60: Accessing 'Union' tool from Arc Toolbox

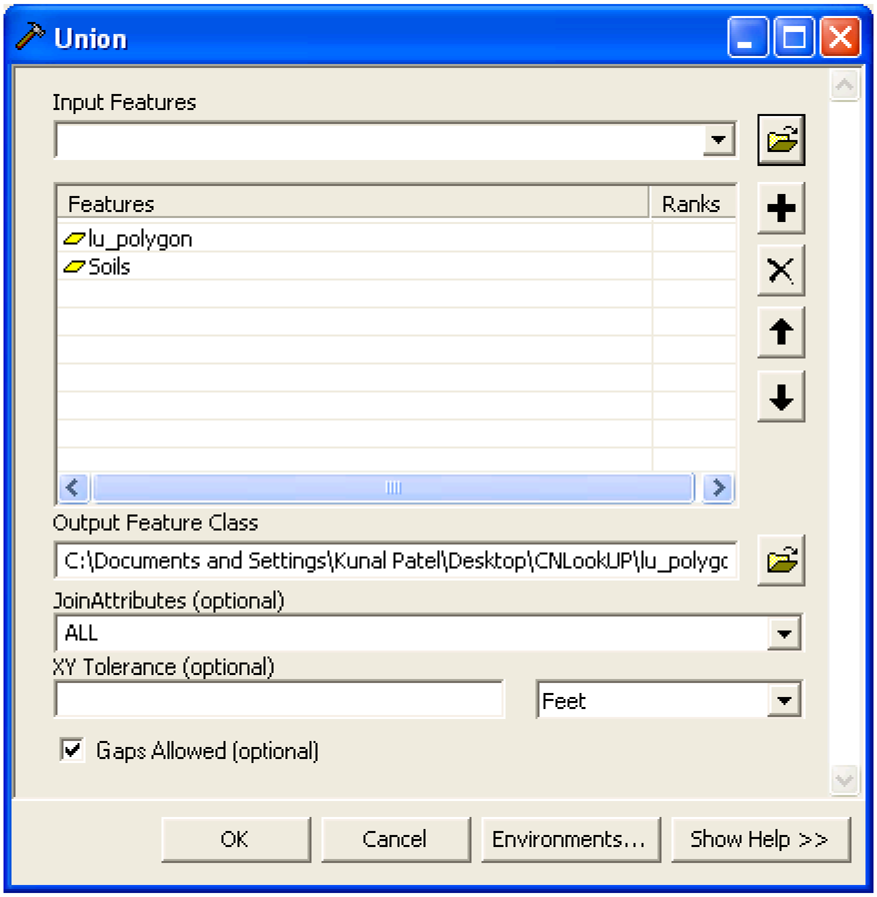


Figure 61: 'Union' tool window for combining land use and soils layers

During this step, the polygons created did not have information corresponding to one of the merged features for the case where the outer boundaries of features did not merge exactly. For example, in the figure shown below some information was lost at the watershed boundary. The resulting difference in CN values and drainage area due to this limitation was checked and was found insignificant for WPBW.

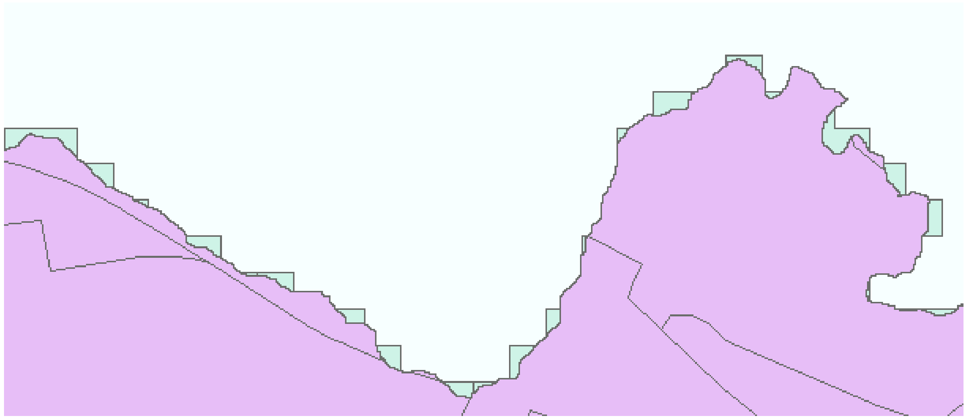


Figure 62: Sliver polygon issue

The next step in preparing the CN grid was to create the CNLookUp table. This was done in ArcCatalog by performing the following sequence to create the table: select File→New→Table. The required fields are LUValue, Description, A, B, C and D. All these field types are integer values except the description field, which obviously is text type. Entering the 14 classifications from the land use grid created previously populated the LUValue and the Description field in CNLookUp table. The remaining fields A, B, C and D were populated with curve number values by matching soil-landuse combinations from the NRCS Technical Release 55 tables (June 1986). Also, an additional field 'LandUse' was created and populated with land use categories under the 'LUValue' field. This field is a required field by HEC-GeoHMS. The resulting CNLookUp table is shown below.

Table 24: CNLookUp table for WPBW

Attributes of CNLookUp

OID	Field1	LUValue	Descriptio	A	B	C	D	Value	LandUse
10	1	5	AGRICULTURAL LAND	39	61	74	80	5	5
0	2	4	BRUSH	30	48	65	73	4	4
12	3	13	RESIDENTIAL (33% IMPERVIOUS)	57	72	81	86	13	13
1	4	11	LANDSCAPED OPEN SPACE	39	61	74	80	11	11
2	5	12	ORCHARDS/TREE NURSERIES	32	58	72	79	12	12
3	6	6	PERMANENTLY UNVEGETATED/SPARSELY	68	79	86	89	6	6
4	7	2	RESIDENTIAL (17% IMPERVIOUS)	51	68	79	84	2	2
5	8	7	RESIDENTIAL (23% IMPERVIOUS)	54	70	80	85	7	7
11	9	9	LANDSCAPED COMMERCIAL/INDUSTRIAL/IN	81	88	91	93	9	9
6	10	8	RESIDENTIAL (65% IMPERVIOUS)	77	85	90	92	8	8
7	11	3	UNLANDSCAPED COMMERCIAL/INDUSTRIAL	89	92	94	95	3	3
8	12	14	UNLANDSCAPED OPEN SPACE	68	79	86	89	14	14
13	13	10	WATER	10	10	10	10	10	10
9	14	1	WOODED AREA	30	55	70	77	1	1

Record: 0 Show: All Selected Records (0 out of 14 Selected)

The final step was to create the CN Grid by selecting Utility → Create Parameter Grids in the HEC-GeoHMS project view toolbar. Selecting the appropriate features and tables created in the steps above generated the curve number grid shown below.

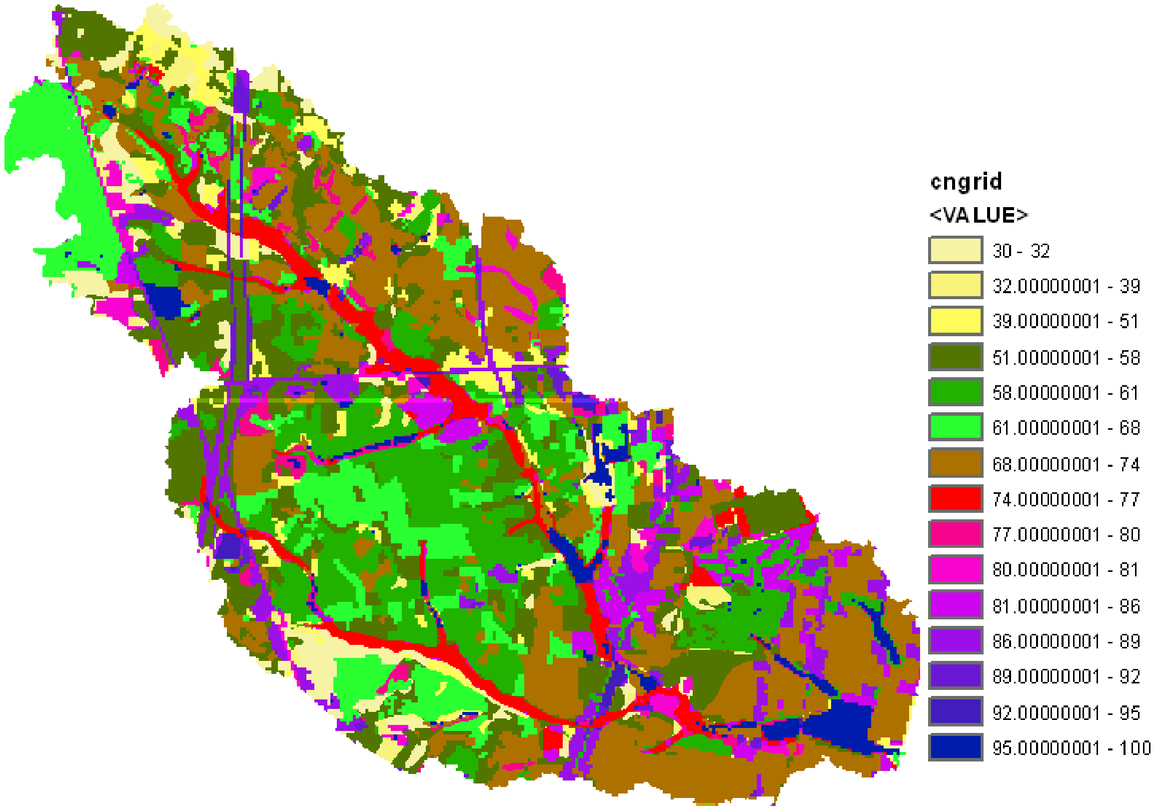


Figure 63: Curve Number grid for WPBW

Using the TR-55 weighted average curve number method (June, 1986), the average curve numbers were computed for each subwatershed (Table 25).

Table 25: Weighted average Curve Numbers for WPBW

Subwatershed	**Area (acres)**	**Area (Sq. Miles)**	**Weighted CN**
W6 (Martins Rd)	810.00	1.27	60.3
W9 (Hurleys Pond Rd)	1152.40	1.80	65.2
W7 (Glendola Rd)	1124.50	1.76	67
W1 (Waterford Glen)	1230.70	1.92	68.7
W5 (Baileys corner Rd)	1711.00	2.67	62.5
W2 (Hannabrand Brook culvert)	259.20	0.41	70.1
W3 (Old mill Dam culvert)	298.70	0.47	72.2
W8 (Spring Lake golf course)	415.50	0.65	65.8
Wreck pond outlet subwatershed	1125.60	1.76	74.8
Total	8127.60	12.70	

The layers created in this section can be used for several other analyses too. For example, the runoff analysis for 100-year storm event for WPBW is shown below (Figure 59). The curve number value of each polygon was converted to the runoff depth for the specified rainfall depth of 8.9 inches for Monmouth County, NJ. It can be seen clearly that the majority of the watershed generates runoff of 4 to 6 inches for the 100-year storm. The equation used to compute runoff (Q) from the curve number and precipitation (P) is shown below (Ponce and Hawkins, 1996).

$$Q = \frac{[CN(P+2)-200]^2}{CN[CN(P-8)+800]} \qquad \text{Equation 22}$$

The runoff values shown in the figure 64 below is in inches.

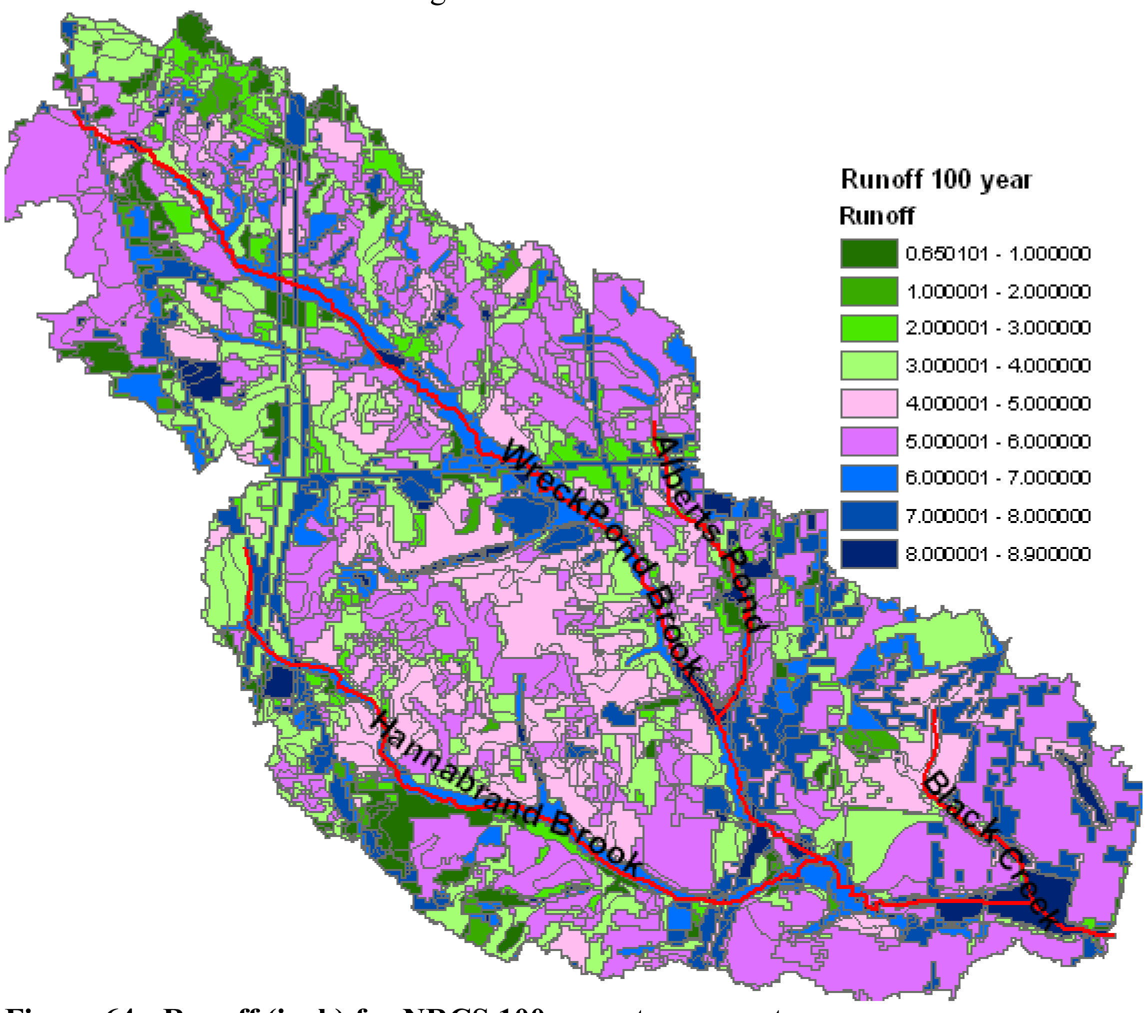


Figure 64: Runoff (inch) for NRCS 100 year storm event

4.2.3 Computing Basin Lag time

The basin lag time often is defined as the difference in time between the center of mass of effective (net) rainfall and the center of mass of direct runoff produced by the net rain (Viessman and Lewis, 2003). It locates the hydrograph's position relative to the causative storm pattern. HMS uses the basin lag time in the user specified S-graph to transform method computations. Most computer programs, especially those using the SCS procedures, compute lag time as 60% of the time of concentration. For the WPBW study, an initial value of Tc was measured along the main channel of the sub-drainage area using stream data from GIS. This gave an initial starting point for the time parameter. Physical characteristics of the watershed have significant effects on actual lag time. Slope, landcover, vegetation, urbanization and sewerage infrastructure, impoundments etc. all act to either detain or accelerate runoff velocity. Once an initial lag time was computed, the lag was varied by informed trial and error by comparing the computed hydrograph with the observed (gauged) hydrograph. Adjustments in lag time have the effect of increasing or decreasing the magnitude of the peak flow as well as moving the occurrence of the peak backwards or forwards in time. Therefore, the lag can be adjusted within reason until the peak magnitude and timing agree with gauged data.

4.2.4 Reservoir Modeling

The reservoir modeling in HMS is capable of modeling any configuration of outlets and ponds. However, it is not recommended to use a reservoir model if the outflow is under a backwater effect or if the reservoir is gated and the outflow is controlled by gate operation. For WPBW all reservoirs except the outlet from the Wreck Pond itself is inlet controlled, i.e., the outflow is function of upstream water surface elevation. A combination of field surveys, GIS data and existing reports were used to develop reservoir-rating tables. Existing reports were available for Old Mill Pond and Hurley's Pond from Monmouth County Engineering Department and Hatch-Mott McDonald for Monmouth County, respectively. The rating table data was developed by NJDA for the Albert's Pond, Osborne Pond, Mc Dowel Pond (18th Ave), Fairway Mews detention basins, and the Spring Lake Golf Course impoundments. Reservoir information for Albert's Pond, Kellers Pond and Mc Dowel Pond were updated to include more stage – storage –discharge information to accommodate flows for modeling the NRCS 100-year storm event using the standard SCS dimensionless unit hydrograph (PRF 484). Starting elevations were based on the Monmouth County Digital Elevation Data on the lake surface. Field observations allowed assuming this elevation close to the impoundment outlets. Incremental elevation and volumes were determined by measuring contours around each impoundment using GIS. Outlet hydraulics was developed from limited field measurements of weirs, outlets, pipes etc. Excel spreadsheets were developed to compute stage-storage-discharge tables. In the following figure, yellow line shows the stream and cyan polygon shows the contour around the reservoir.

Figure 65: Kellers Pond information extracted using GIS

The Elevation – Storage relationship was developed using an elevation-area method. In this method, the volume contained between two contours is obtained as the product of vertical distance between two contours and the average of the area occupied by each contour. Each volume computed is an incremental volume and the cumulative volume is obtained by adding incremental volumes. The volume between two contours can be expressed in equation form as shown below:

$$V = \frac{(A_1 + A_2)}{2} * (E_2 - E_1) \qquad \text{Equation 23}$$

Where,

V = storage volume (acre-feet)

A_1,A_2 = Area occupied by contour 1 and 2 respectively (acres)

E_2, E_2 = Elevation of contour 1 and 2 respectively ($E_2 > E_1$) (feet)

Table 18: Elevation – Area method for Keller's Pond

Elevation (feet)	Surface Area (acres)	Incremental Storage (ac-ft)	Cumulative Storage (ac-ft)
86.0	2.7	0	0
86.5	3.86	1.64	1.64
87.0	4.83	2.1725	3.8125
87.5	5.8	2.6575	6.47

88.0	8	3.45	9.92
90.0	12.8	20.8	30.72

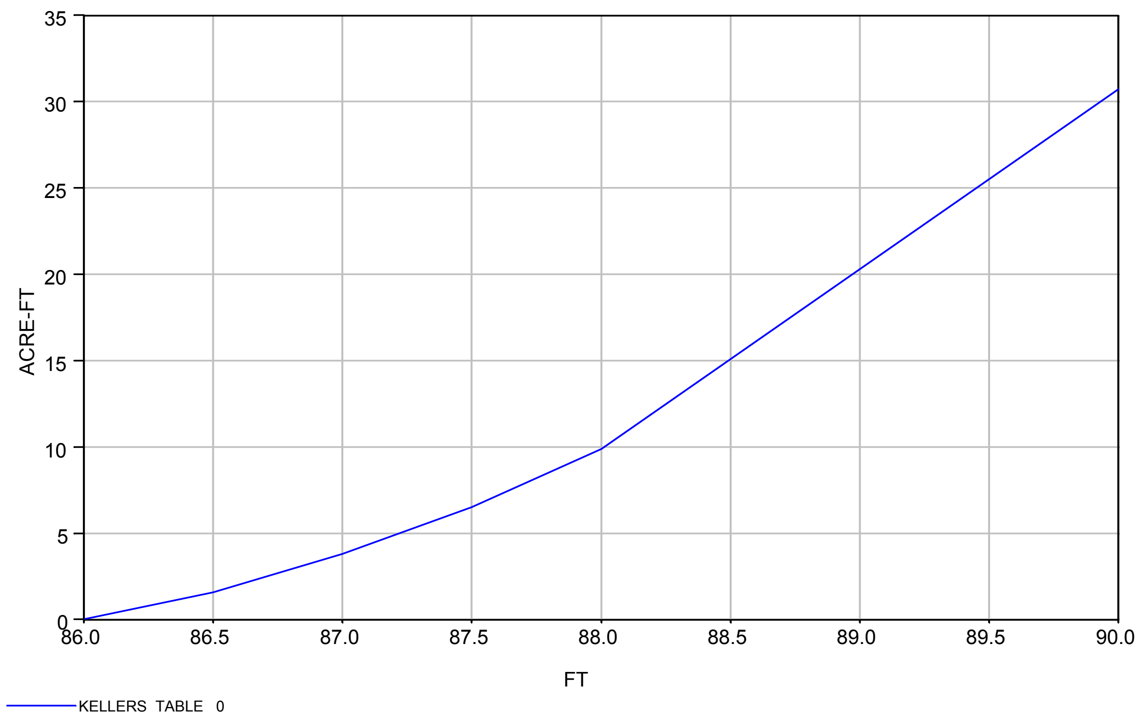


Figure 66: Keller's pond Elevation-Storage curve

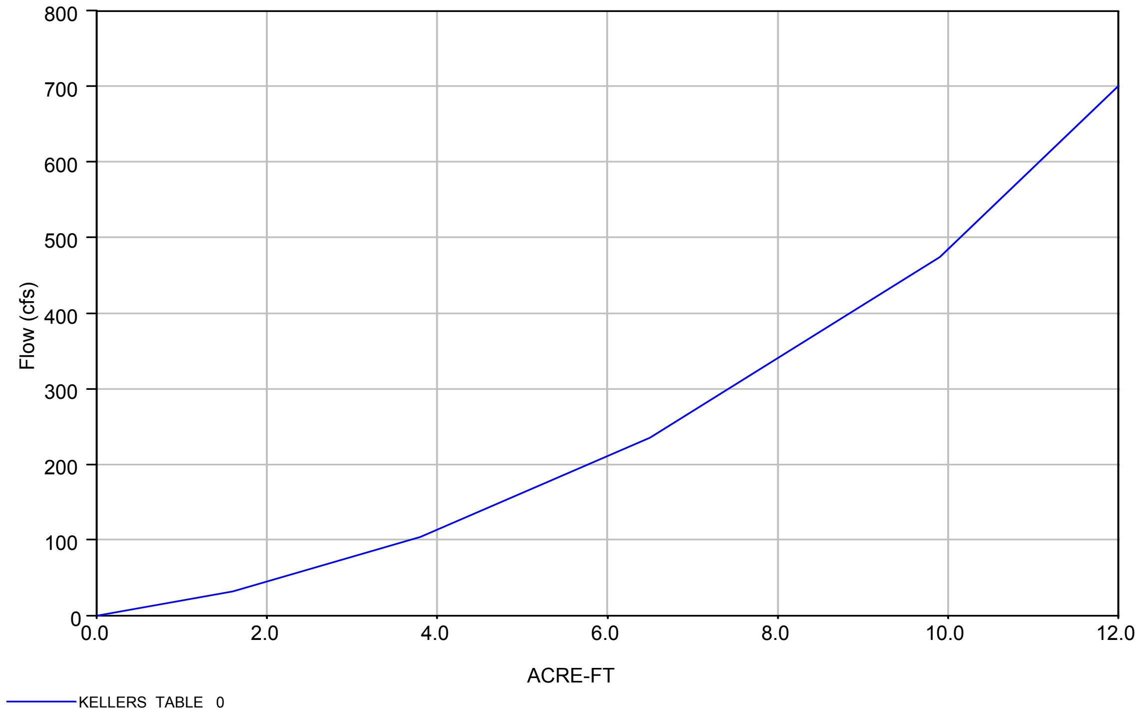


Figure 67: Keller's pond Storage – Discharge curve

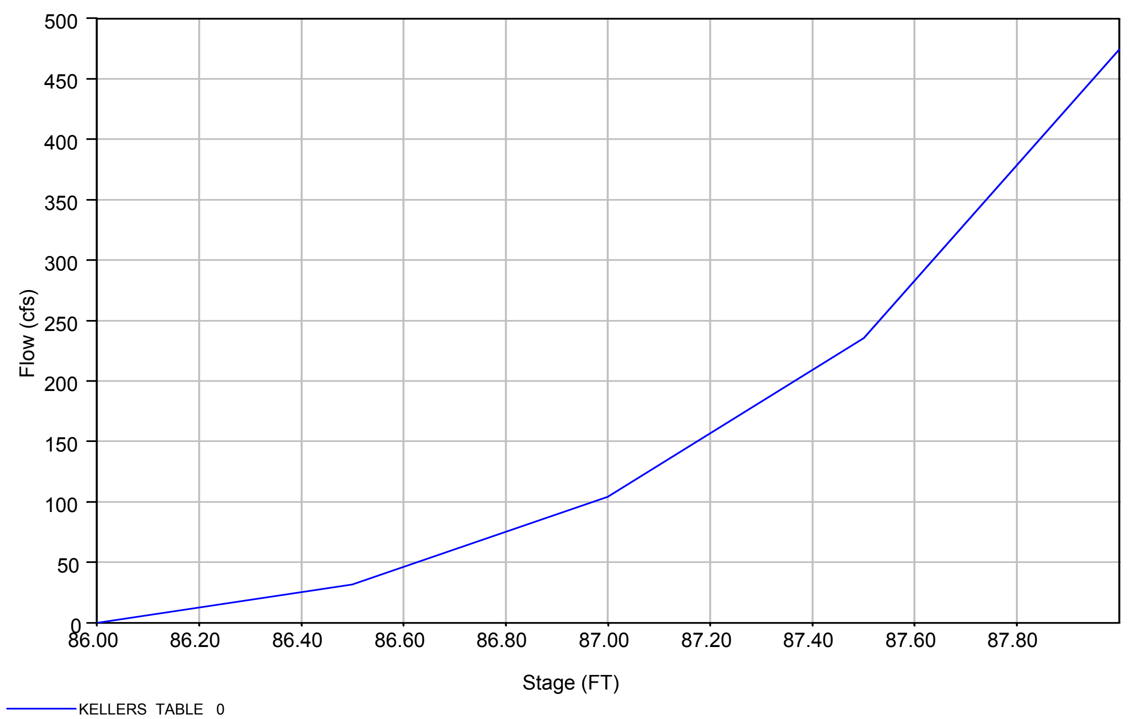


Figure 68: Keller's pond Stage – Discharge curve

The information for other reservoirs in WPBW can be found in Appendix A (Storage – Discharge), Appendix B (Elevation – Storage) and Appendix C (Stage – Discharge).

Figure 69: J. E. Showler and K.P. Patel at the outlet of Wreck Pond to the Atlantic Ocean. (Source: S. Jacobus, 2008)

4.2.5 Reach Routing

As the flood runoff travels through the channel reach it becomes attenuated due to channel storage effects. The routing models available in HEC-HMS account for this attenuation and were described briefly in section 3.1.1. Each of these models computes the downstream hydrograph, given an upstream hydrograph as a boundary condition. Several stream channel reaches were modeled using the Muskingum routing method (HMS Technical Reference Manual, March 2000). The Muskingum routing model uses a simple finite difference approximation of the continuity equation. This method attempts to modify the hygrograph shape using a timing factor (K) and an attenuation factor (X). The Muskingum method is based on the following equations:

Continuity of mass equation:

$$I - O = \frac{dS}{dt} \approx \frac{\Delta S}{\Delta t}$$ Equation 24

Where,

I and O are the inflow and outflow in the time increment dt (or Δt) and S is the storage.

Also, expressed as following if $\Delta t = t - t_{t-1}$;

$$\left(\frac{I_{t-1} + I_t}{2}\right) - \left(\frac{O_{t-1} + O_t}{2}\right) = \left(\frac{S_t - S_{t-1}}{\Delta t}\right)$$ Equation 25

For reach routing, I and O represent the average value of the upstream hydrograph and the downstream hydrograph, respectively.

Weighted average storage (S) function:

$S = KO + KX[I - O] = K[XI + (1 - X)O]$ Equation 26

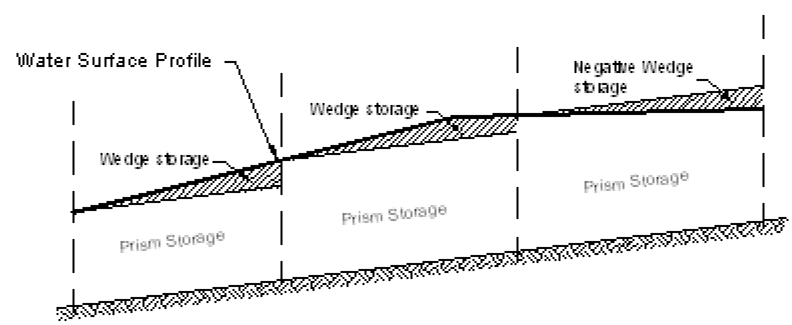


Figure 70: Channel storage (Wedge and Prism)
(Source: RAS reference manual, 2008)

The term *KO* (outflow rate multiplied by the travel time through reach) represents the volume of prism storage and the term *KX[I – O]* represents the volume of wedge storage (weighted difference between inflow and outflow multiplied by the travel time).

Substituting the storage equation into the continuity equation gives following equation:

$$\frac{I_1 + I_2}{2} - \frac{O_1 + O_2}{2} = \frac{K[XI_2 + (1-X)O_2] - K[XI_1 + (1-X)O_1]}{\Delta t} \qquad \text{Equation 27}$$

The above equation can be rearranged into the following:

$$O_2 = C_o I_2 + C_1 I_1 + C_2 O_1 \qquad \text{Equation 28}$$

Where,

$$C_o = -(KX - 0.5\Delta t) / (K - KX + 0.5\Delta t) \qquad \text{Equation 29}$$

$$C_1 = (KX + 0.5\Delta t) / (K - KX + 0.5\Delta t) \qquad \text{Equation 30}$$

$$C_2 = (K - KX - 0.5\Delta t) / (K - KX + 0.5\Delta t) \qquad \text{Equation 31}$$

The sum of coefficients in equations 29, 30, and 31 should yield a value of 1.

For storage in the channel that is controlled by the downstream conditions, X = 0. It suggests that the inflow has little effect and reflects reservoir storage type relation (S = KO) with maximum attenuation.

For X = 0.5, inflow and outflow have equal weight and provide no attenuation and the hydrograph will be translated by one time increment (Δt). If Δt is small, I_1 is approximately equal to I_2 and will cancel each other. The value of X = 0.2 commonly is assumed. The value of X is smaller for streams with mild slopes and wide floodplains and will be higher for streams with steep slopes. Values of K were estimated from the channel length and an assumed flow velocity. Values of X were obtained by trial and error within acceptable ranges.

4.2.6 Diversions

During the hydrologic model development for RSWMP, it was found that peak flows and volumes were lower downstream than upstream for certain storm events for Hannabrand Brook. The stream section from W5 to W2 was walked to determine if the data was erroneous or if there was a physical basis for the difference. The survey revealed numerous locations where debris dams lay across the stream, forcing higher flows into the floodplain. In some cases, the stream banks were slightly depressed which would allow flows to be diverted out of the channel. These conditions may be attributed to the flat topography, wide floodplains and heavily vegetated stream banks. A further examination of stream flow data during selected modeling events indicated for this subwatershed, base flow conditions at the upper and lower gage stations were recorded accurately and indicating gages functioned correctly. HMS provides generalized options for including diversions of flow from a channel system. This was used for a calibration event for the Hannabrand Brook subwatershed. Computed results compared favorably with observed hydrographs (see Section 4.2.8.3).

4.2.7 WPBW Model Schematic and Model output

The HMS symbols for hydrologic elements such as basins, reaches, junctions, and reservoirs were overlaid on the background map of the WPBW. The watershed was divided into eight sub-watersheds and again further divided into sub-drainage areas. These steps allowed the model to account for directly connected impervious areas and areas draining to reservoirs and to provide homogenous drainage areas for the CN method application. Figure 71 shows the model schematic for WPBW. HMS provides the option to lock the locations of the elements over the background map.

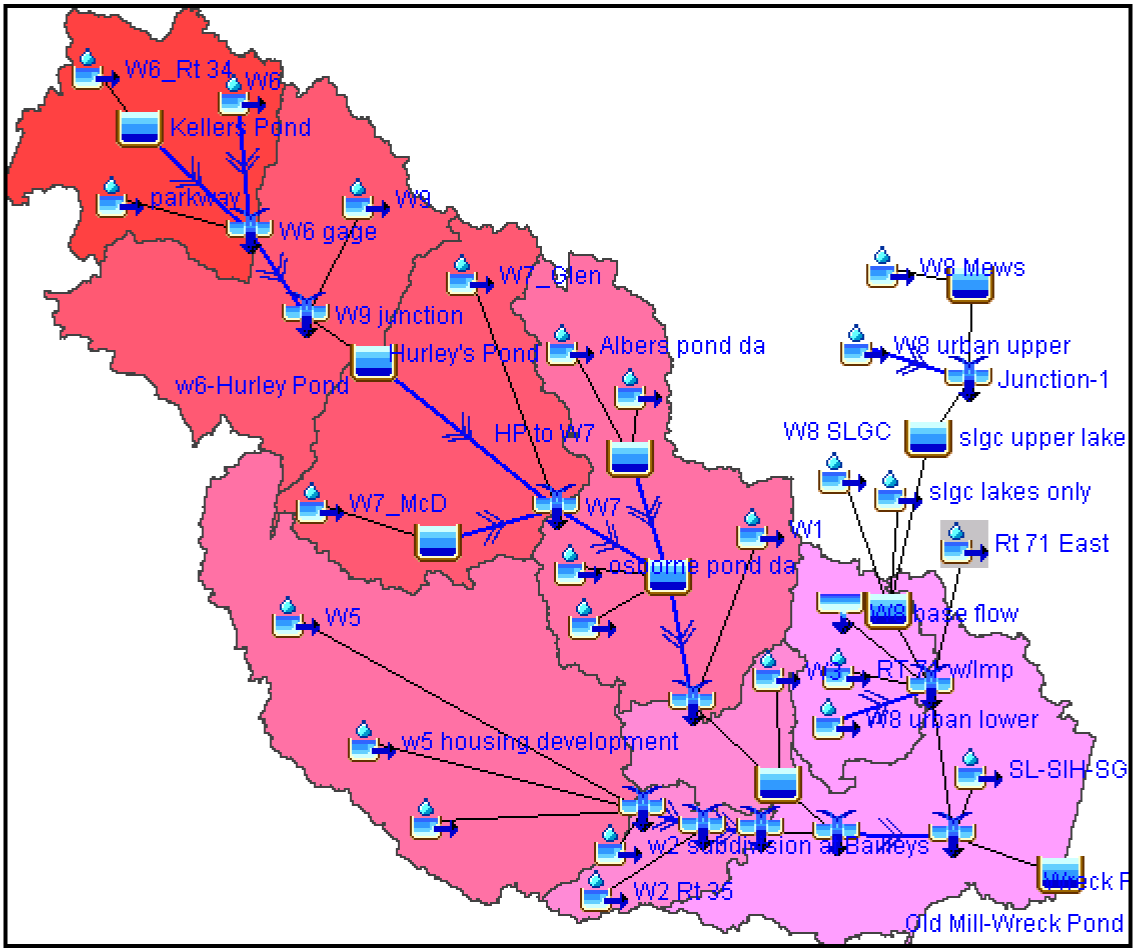


Figure 71: WPBW Hydrologic Model schematic

4.2.8 Model Calibration and Validation

The goal in this study was to develop the model, which takes into account varying conditions such as antecedent moisture conditions, low flow conditions and high flow conditions and to identify regional parameters for a coastal watershed in New Jersey. The storm event of June 24, 2006 modeled AMC III, the storm event of July 06, 2006 modeled AMC II, and the storm event of October 2005 modeled a high flow event. The regional parameters such as the SCS, Delmarva and Lower Monmouth unit hydrographs and initial abstraction ratio values of 20% and 5% were modeled and compared for WPBW. The modeled storm events for calibration and verification during the RSWMP development were reevaluated due to changes proposed by this study based on the analysis described in sections for 'Modeling October 2005 storm event' and 'Event Analysis method for WPBW'.

4.2.8.1 Modeling June 24, 2006 storm event using AMC III conditions for model calibration

Hawkins (1985) reported the following equations to describe the relation between potential maximum retention values for varying soil moisture conditions.

$$S_1 = 2.281 S_2 \qquad \text{Equation 32}$$

$$S_3 = 0.427 S_2 \qquad \text{Equation 33}$$

Ponce and Hawkins (1996) derived following equations using above relationship.

$$CN_1 = \frac{CN_2}{2.281 - 0.01281 CN_2} \qquad \text{Equation 34}$$

$$CN_3 = \frac{CN_2}{0.427 + 0.00573 CN_2} \qquad \text{Equation 35}$$

Mishra et al, (2008) reported the formula in Equation 35 to perform best among all the available CN conversion formulas for AMC III.

Table 3 in the literature review section shows the CN values for different soil moisture conditions and these values were used in this study.

The storm event of June 24, 2006, was selected for calibrating the HMS model. During modeling, the effect of antecedent moisture became apparent and the model was calibrated using AMC III. The 0.09-inch rainfall (highlighted in rainfall table) was deducted from the total rainfall (1.93 – 0.09 = 1.84 inch) and AMC III was used. The following table 26 shows the curve number used to model AMC III for various sub-basins in the WPBW.

Table 26: Curve numbers used to model AMC III for June 24, 2006 storm event

Subbasin	CN (AMCII)	% Impervious	CN (AMCIII)	CN used for AMCIII
Albers pond da	70	0	85	85
Albers Pond Surface	98	100	98	98
Osborne pond da	66	0	83	83
Osborne Pond surface	98	100	98	98
Parkway	98	100	98	98
W1	71	0	86	86
W3	70	0	85	79
W6	61	0	79	75
W6_Rt 34	72	0	92	82
W7_Glen	66	0	83	83
W7_McD	66	0	83	82
W9	70	0	85	81

In Figure 72, it can be seen the response of WPBW for the June 24, 2006 storm event using different soil moisture conditions. The blue colored hydrograph is the response using AMC II and red colored hydrograph is the response using AMC III conditions. The green colored hydrograph is the observed hydrograph.

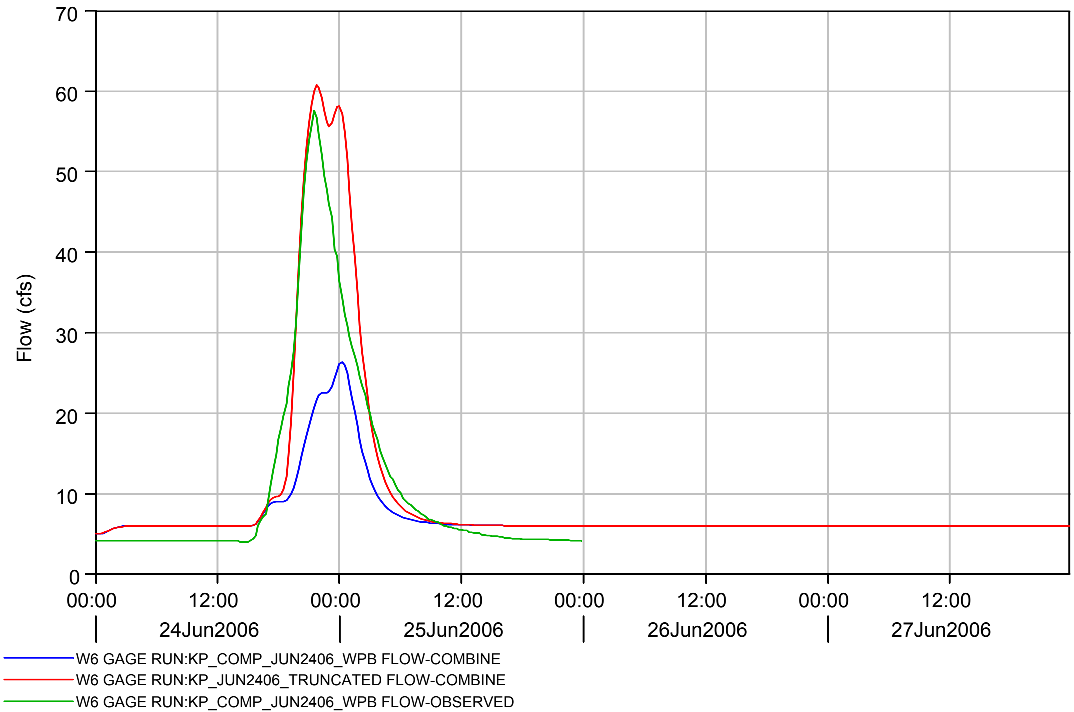


Figure 72: Effects of June 24, 2006 modeling using AMC II and III at station W6

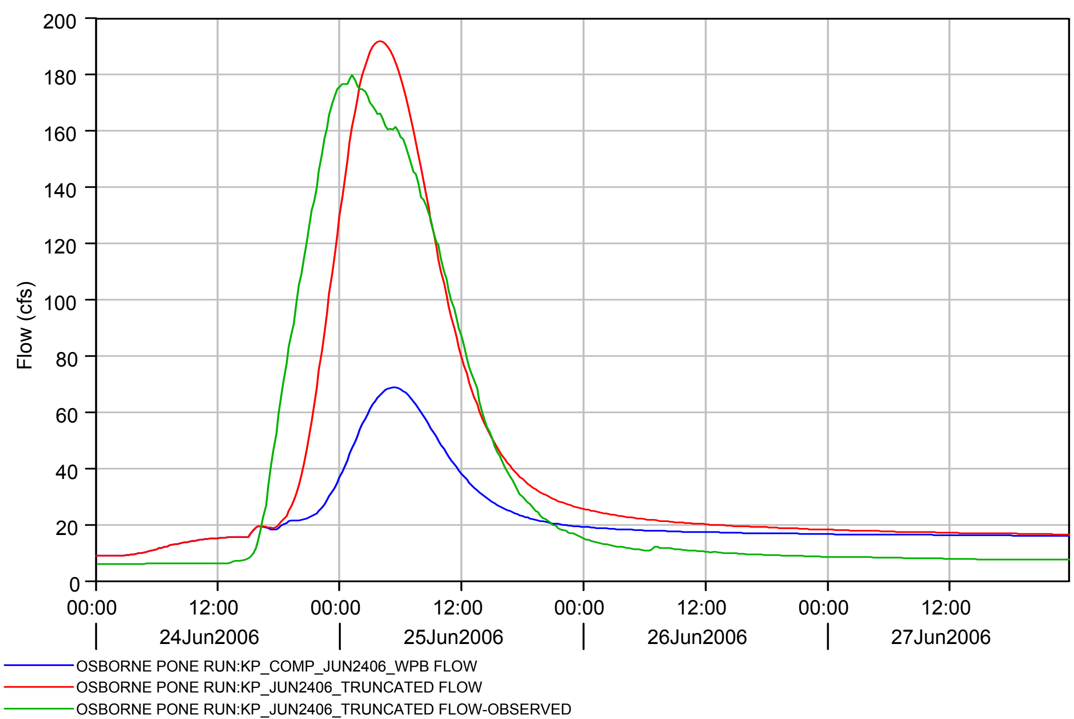


Figure 73: Effects of June 24, 2006 modeling using AMC II and III at Osborne Pond

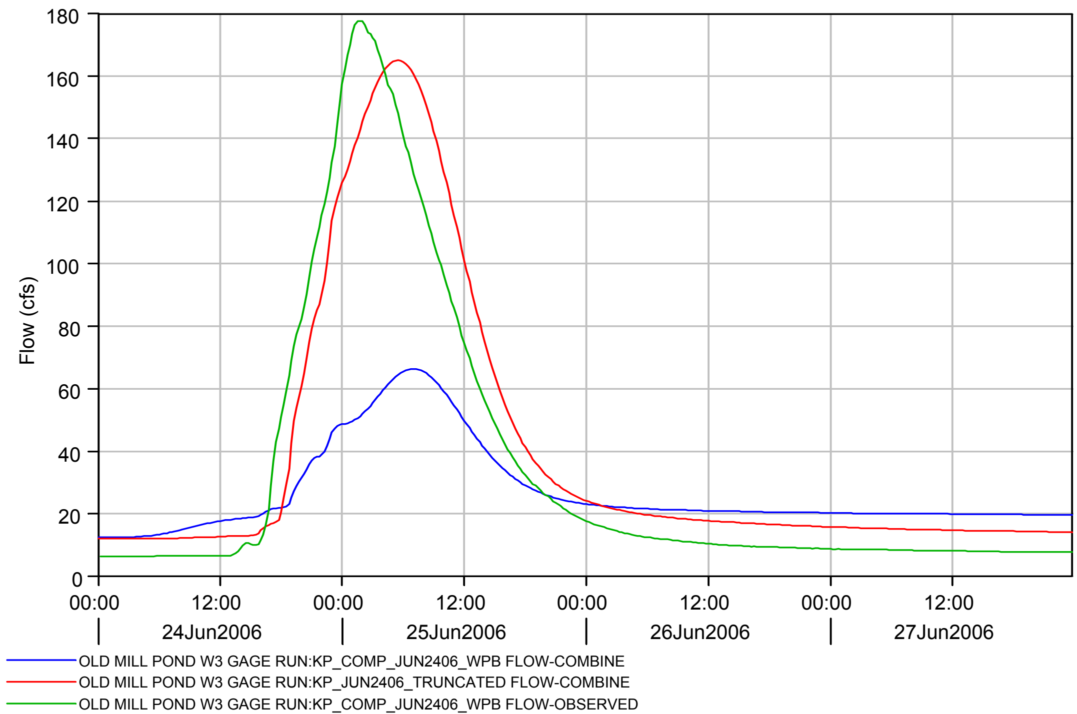


Figure 74: Effects of June 24, 2006 modeling using AMC II and III at Station W3

4.2.8.2 Modeling July 06, 2006 storm event using AMC II for model verification

The storm event of July 06, 2006 was used to verify the model. The following figures show the comparison of the modeled and observed hydrographs for the verification run. In figure 76, verification run at Old Mill Pond - W3 gage, it can be seen that the peak flow and volume under the hydrograph is in good agreement. However, the time factor is off by approximately 12 hours. This difference cannot be verified but could be caused due to difference in timing in terms of AM and PM for the gage W3 for that time period.

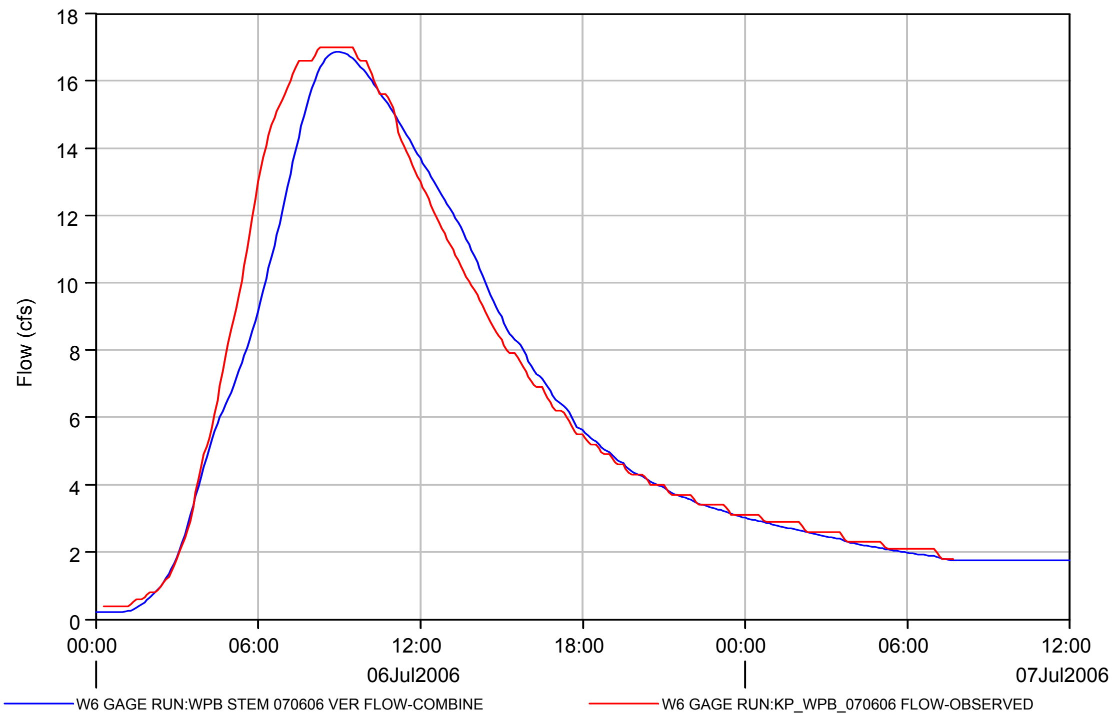


Figure 75: Hydrograph comparison for the Verification run for July 06, 2006 storm event at station W6

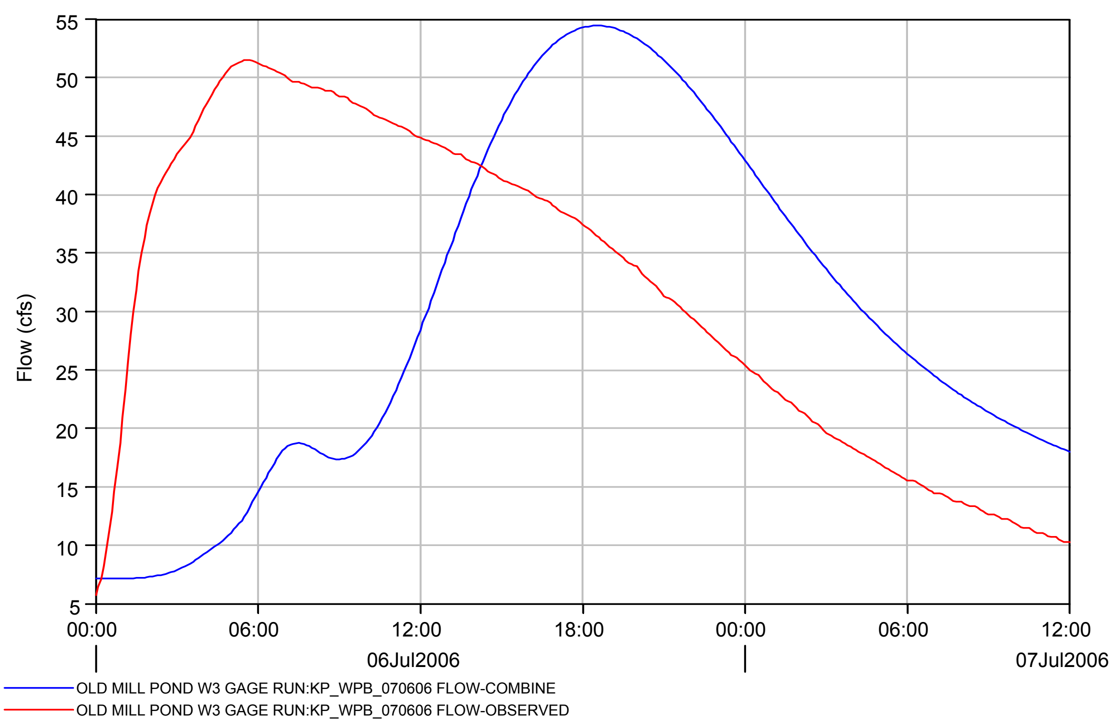


Figure 76: Hydrograph comparison for the Verification run for July 06, 2006 storm event at station W3

4.2.8.3 Model calibration and verification for Hannabrand Brook

The storm event of March 28, 2005 and June 24, 2006 were used for calibration and verification of Hannabrand Brook hydrologic model.

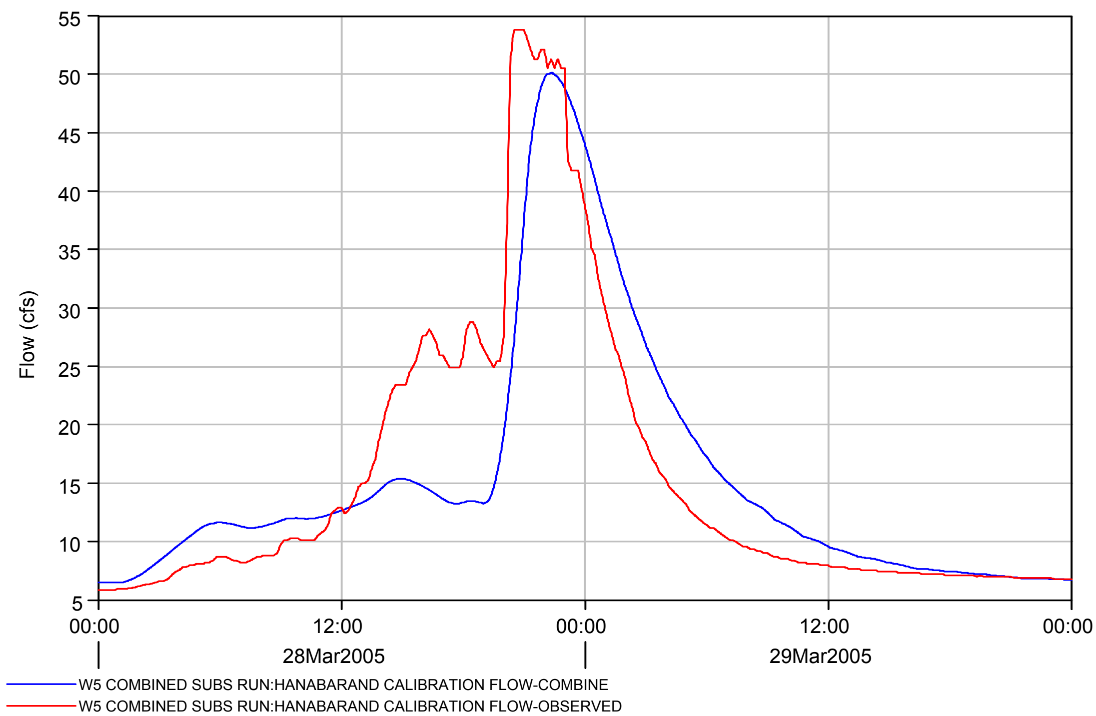


Figure 77: Hydrograph comparison for the Calibration run for March 28, 2005 storm event at station W5

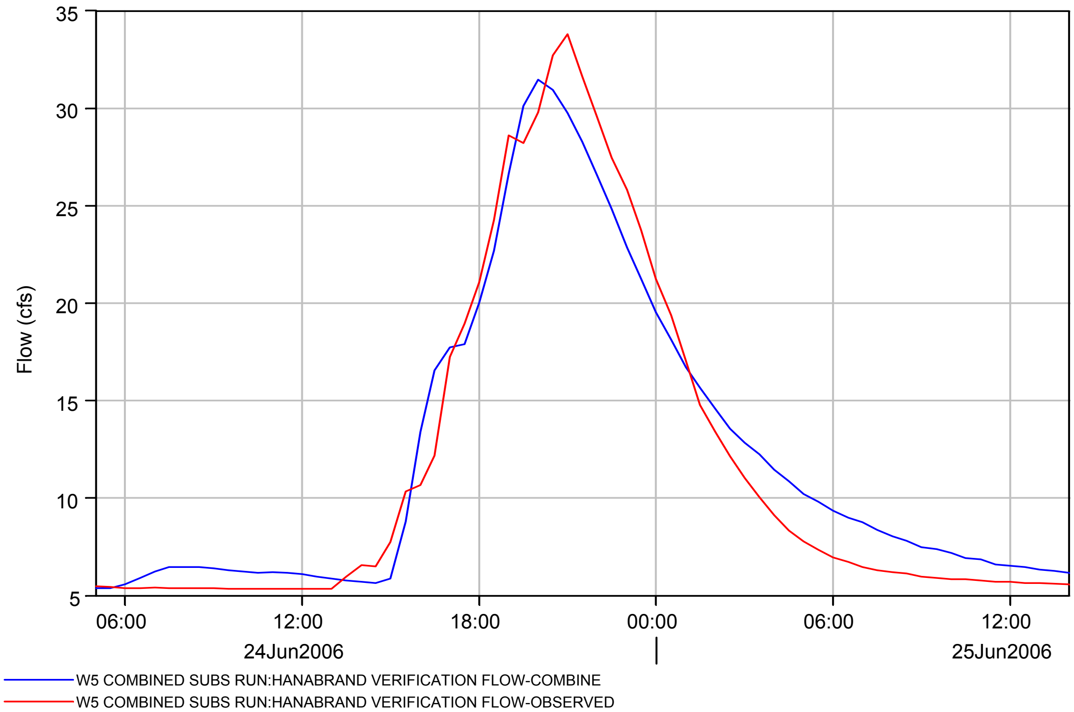


Figure 78: Hydrograph comparison for the Verification run for June 24, 2006 storm event at station W5

4.2.8.4 Modeling October 2005 storm event – High flow event

For modeling the October 2005 storm event, the stage data was available, however conversion to flow values to obtain the observed hydrograph was not utilized, as it would have required using the modeled rating curves at the upper end. Hence, peak flow timing was chosen for modeling this event. The stations chosen to analyze the peak flow timing were the outlet of Wreck Pond Brook at station W3 and outlet of Hannabrand Brook at station W2. Flow from the two stations merge downstream and soon below that location, the stream is under tidal influence. The peak flow timing analysis method required investigating the difference between the peak timings at these two stations. All the rainfall events within the time period of 2 years were chosen to investigate the peak timings. The total of 6 storm events were found where the data was available at both stations. Table 27 summarizes the findings of this analysis.

Table 27: Stream gage data analysis for peak flow times at Station W3 and Station W2

Storm	P (inch)	W3 (Peak Time)	W2 (Peak Time)	Peak at W3 later than W2 by (hr)
5/20/2005	1.32	05/20/2005@17:32	05/20/2005@16:22	1.17
6/27/2005	1.13	06/27/2005@14:32	06/27/2005@13:22	1.17
10/14/2005	11	10/14/2005@1:44	10/14/2005@0:38	1.1
1/3/2006	2.43	01/03/2006@9:06	01/03/2006@8:22	0.63
4/22/2006	1.59	04/23/2006@10:14	04/23/2006@10:02	0.20
6/24/2006	1.93	06/25/2006@1:25	06/25/2006@1:02	0.38

From the above table it can be seen the peak flow timings at both stations were close with an average difference of 45 minutes. The peak flow at Station W3 arrived an average of 45 minutes later than Station W2. The observed peak timings for October 2005 storm event was 1:44 AM on October 14th at station W3 and 0:38 AM on October 14th. The existing RSWMP model predicted the peak flow timings at 8:00 AM on October 14th at station W3 and 5:00 AM on October 14th at station W2. Hence, there was a difference of 6.27 hours and 4.37 hours between observed and modeled peaks, with the modeled peak occurring later than observed peaks. This study reevaluates the lag time for all the sub-basins and the reach routing parameters for all the reaches. As a starting estimate, the Arc Hydro tool bar was used to delineate the longest flow paths in the subwatersheds. The longest flow paths was divided by assuming 1 ft/s velocity and used as an estimate for the time of concentration estimate and as a starting point in the model redevelopment. The time of concentration was multiplied by 0.5 to compute the basin lag times. The SCS standard procedure used 0.6 times the time of concentration as the basin lag time, however the LMDUH used 0.5 times the time of concentration as basin lag time.

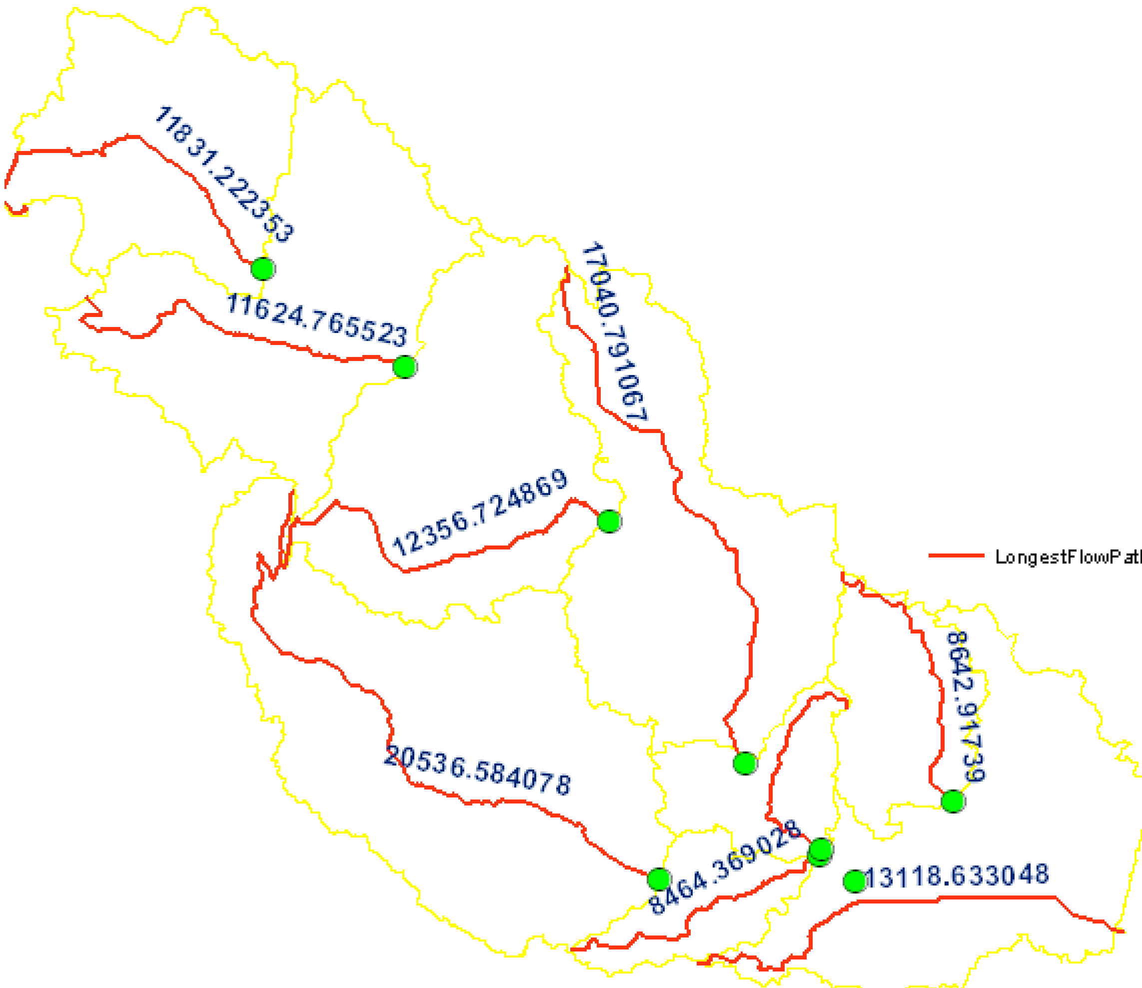


Figure 79: Longest flow path delineation for subbasins using Arc Hydro toolbar

Table 28 shows the time of concentration and lag time for all the subwatersheds in WPBW used as starting estimates derived from the Arc Hydro toolbar.

Table 28: Initial estimate of T_c and T_l using Arc Hydro toolbar

Subwatershed	Length (ft)	T_c (hr)	T_l(hr)
W6	11831	3.29	1.64
W9	11624	3.23	1.61

W7	12356	3.43	1.72
W1	17040	4.73	2.37
W3	6868	1.91	0.95
W5	20536	5.70	2.85
W2	8464	2.35	1.18
W8	8642	2.40	1.20
Lower Watershed	13118	3.64	1.82

Using the initial estimates shown above, several calibration and verification runs were computed. The final values obtained for the Muskingum K (hr) and the X values for channel routing and subbasin lag times are shown in the following tables 29-30.

Table 29: Channel routing parameters for Muskingum method

Hydrologic Element (Reach)	Existing Muskingum K (hr)	Revised Muskingum K (hr)	Existing Muskingum X	Revised Muskingum X
Albers to Osborne	2	1	0	0.05
HP to W7	4	2.5	0	0.1
McD to W7	2	1	0	0.1
Old Mill-Wreck Pond	4	N/A	0	N/A
Osbonre to W1	4	0.5	0	0.1
rt34-W6	2	1.5	0	0.1
Rt 35 to Outlet	1.5	1.5	0	0
W1-W3	Absent	1	Absent	0.05
W5-W2	6	2	0	0.1
W6 disconnected travel	1.5	Deleted	0.001	Deleted
W6-Hurley Pond	1.5	2	0.001	0.05
W7 to Osborne Pond	2	1.25	0.001	0.1

Table 30: Lag time (hr) for subbasins in WPBW

Hydrologic Element Subbasin	Existing Lag time (hr)	Revised Lag time (hr)
Albers Pond da	1.8	1
Albers Pond Surface	0.1	0.1

Osborne pond da	2.4	0.5
Osborne Pond surface	0.1	0.1
Parkway	1.65	1
SL-SlH-SG	1	N/A
W2 Rt 35	0.5	1.5
W2 subdivision at Baileys	2	2
W3	1	0.75
W5	2.3	2.75
W5 add/l housing	0.1	1
W5 housing development	0.5	1
W6	1	1.5
W6_Rt 34	0.5	0.5
W7_Glen	2	2
W7_McD	1.2	1
Rt 71 East	0.1	N/A
Rt 71 w/ Imp	0.1	N/A
Slgc lakes only	0.1	N/A
W8 Mews	0.75	N/A
W8 SLGC	0.75	N/A
W8 urban lower	0.4	N/A
W8 urban upper	1	N/A
W9	2	1.6

4.2.8.5 Event Analysis Method for WPBW

Using the "Event Analysis Method" described by Woodward et. al, 2003, several rainfall-runoff data were analyzed to determine the suitability of using a 5% (Ia/S) ratio for modeling WPBW.

i) Event analysis method for storm event of March 28, 2005 at gage W7

The measured real-time stream elevation data were used to determine the start of the runoff time. For the storm event of March 28, 2005, runoff start time was obtained as

2:00 AM on March 28, 2005. This also is seen in the figure below (Figure 75). The precipitation at the onset of runoff hydrograph was recorded as 0.1 inch and the data is available in the 'Precipitation' section of the thesis. Analysis of rainfall and stream gage elevation information provided following values:

Ia = 0.1 inch

P = 1.93 inch

Pe = 1.83 inch

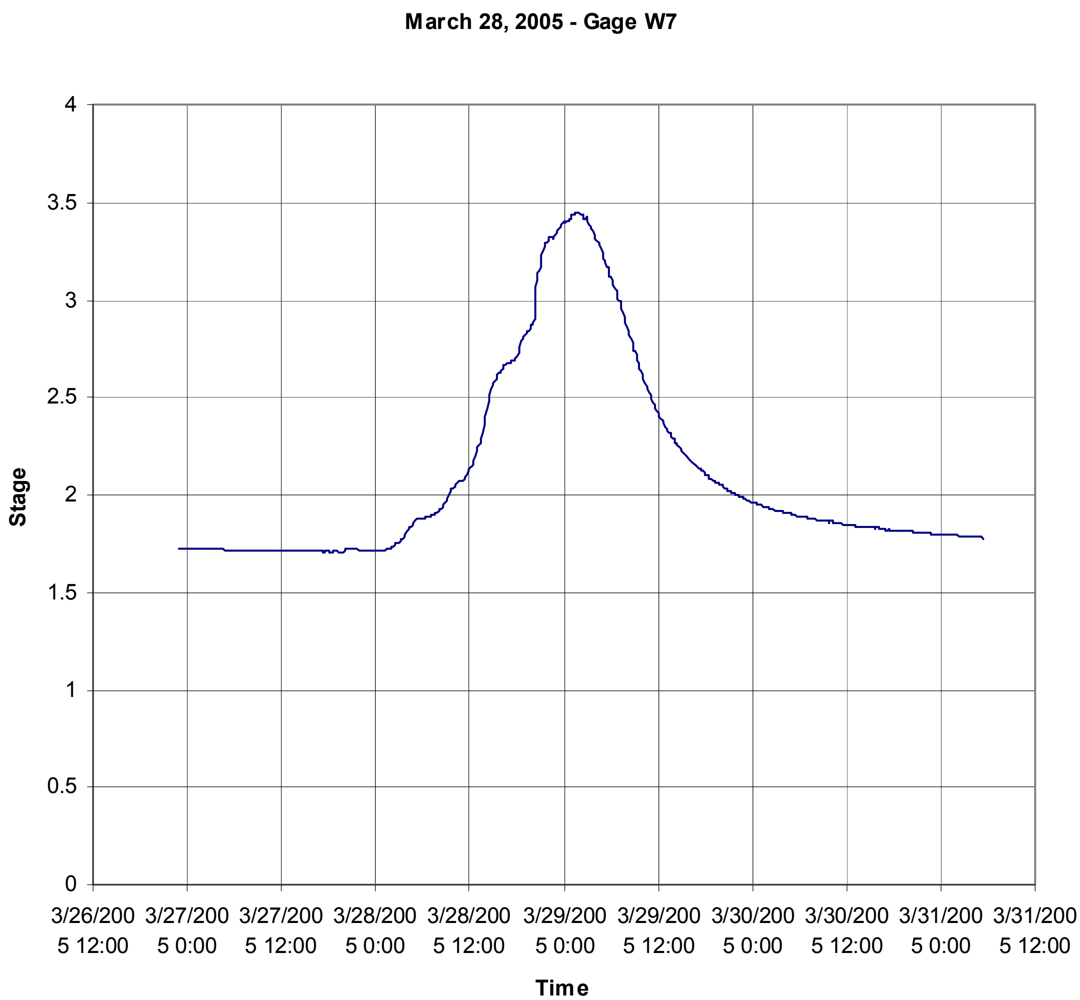


Figure 80: Stream gage data at station W7 for March 28, 2005 storm

The value of Q was computed from the observed hydrograph at station W7 for the March 28, 2005 storm event. The analysis of observed stream flow provided Q = 0.53 inch.

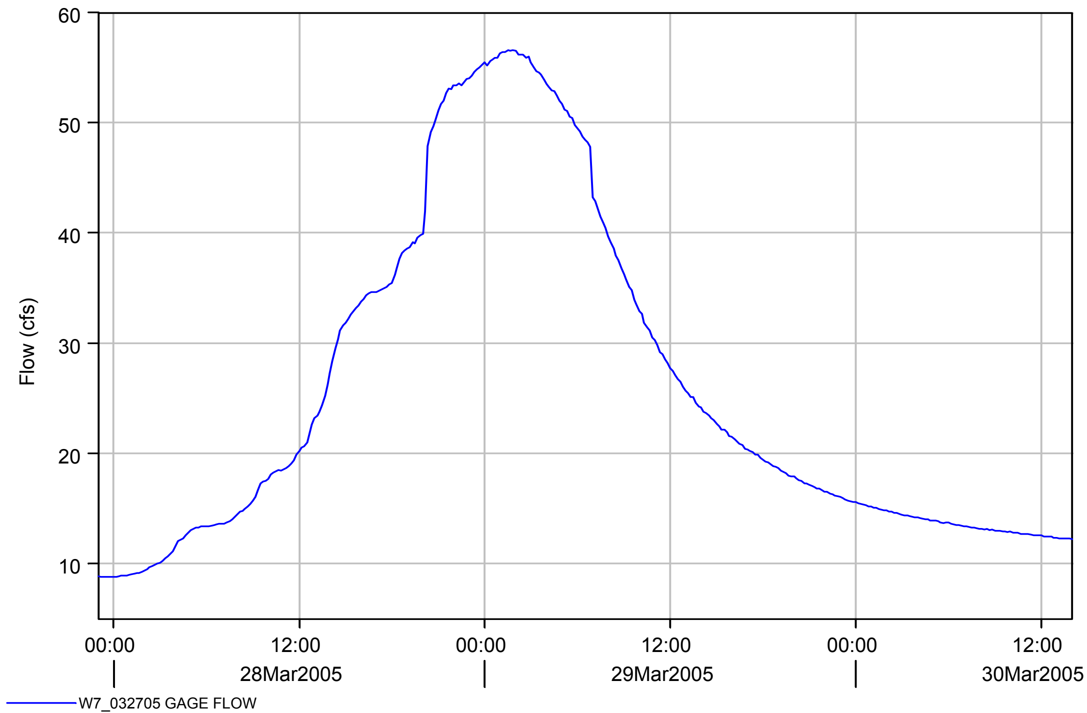


Figure 81: Observed hydrograph at station W7 for March 28, 2005 storm

Therefore, using equations described in 'Event Analysis Method', values of S and λ were computed as shown below:

$$S = \frac{P_e^{\ 2}}{Q} - P_e = \frac{(1.83)^2}{0.53} - 1.83 = 4.49 inch \qquad \text{Equation 36}$$

$$\lambda = \frac{I_a}{S} = \frac{0.1}{4.49} = 0.022 \qquad \text{Equation 37}$$

ii) Event analysis method for storm event of July 08, 2005 at gage W6

The measured real-time stream elevation data were used to determine the start of runoff time. For the storm event of July 08, 2005, runoff start time was obtained as 3:30 AM on July 08, 2005. This also is seen in the figure below. The precipitation at the onset of the runoff hydrograph was recorded as 0.12 inch and the data are available in the 'Precipitation' section of the thesis. The analysis of rainfall and stream gage elevation information provided following values:

Ia = 0.12 inch

P = 1.2 inch

Pe = P – Ia = 1.08 inch

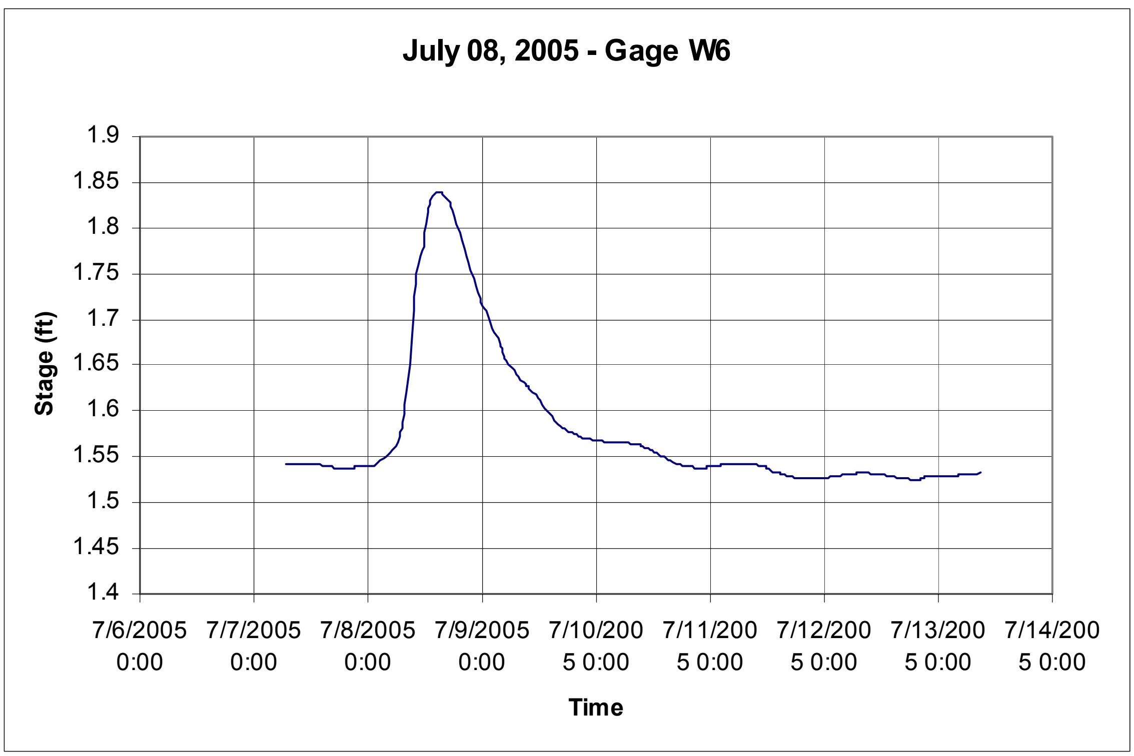


Figure 82: Stream gage data at station W6 for July 08, 2005 storm event

The value of Q was computed from the observed hydrograph data at station W6 for the July 08, 2005 storm event. The analysis of the observed stream flow provided Q = 0.23 inch.

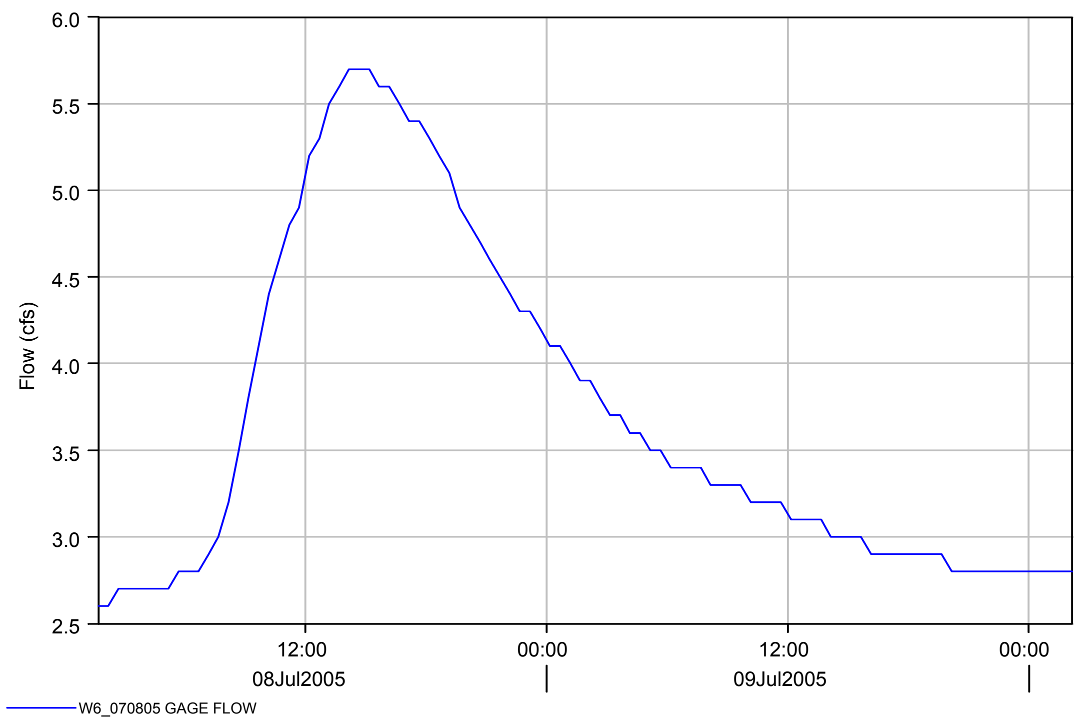


Figure 83: Observed hydrograph at station W6 for July 08, 2005 storm

Therefore, using equations described in ‘Event Analysis Method’, values of S and λ were computed as shown below:

$$S = \frac{P_e^{\,2}}{Q} - P_e = \frac{(1.08)^2}{0.23} - 1.08 = 4.0 inch$$ Equation 38

$$\lambda = \frac{I_a}{S} = \frac{0.12}{4.0} = 0.03$$ Equation 39

iii) Event analysis method for the storm event of July 06, 2006 at gage W6

The measured real-time stream elevation data were used to determine the start of runoff time. For the storm event of July 06, 2006, runoff start time was obtained as 2:17 AM on July 06, 2006. This also is seen in the figure 84. The precipitation at the onset of runoff hydrograph was recorded as 0.15 inch and the data are available in the 'Precipitation' section of the thesis. Analysis of rainfall and stream gage elevation information provided following values:

$Ia = 0.15$ inch

$P = 1.41$ inch

$Pe = P - Ia = 1.26$ inch

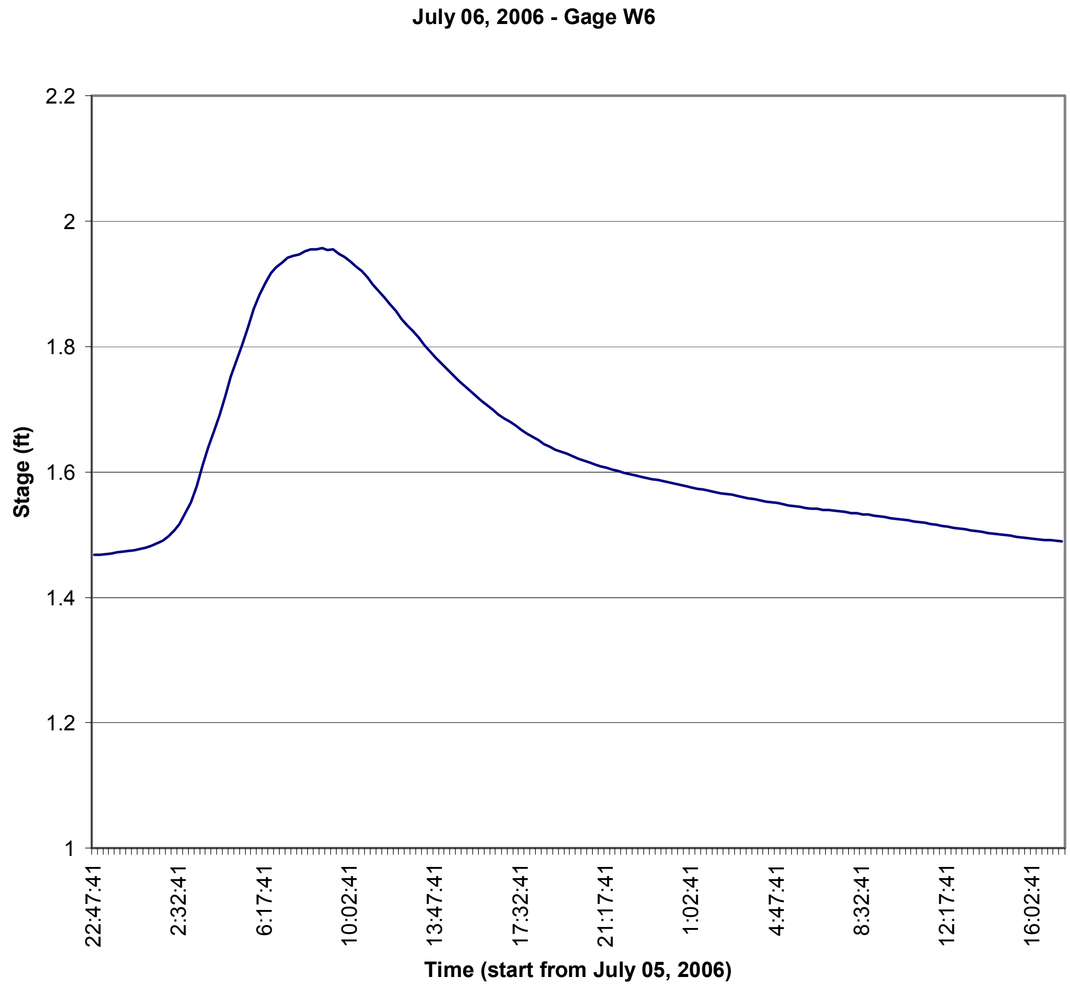


Figure 84: Stream gage data at station W6 for July 06, 2006 storm event

The value of Q was computed from the observed hydrograph data at station W6 for the July 06, 2006 storm event. The analysis of the observed stream flow provided Q = 0.22

inch.

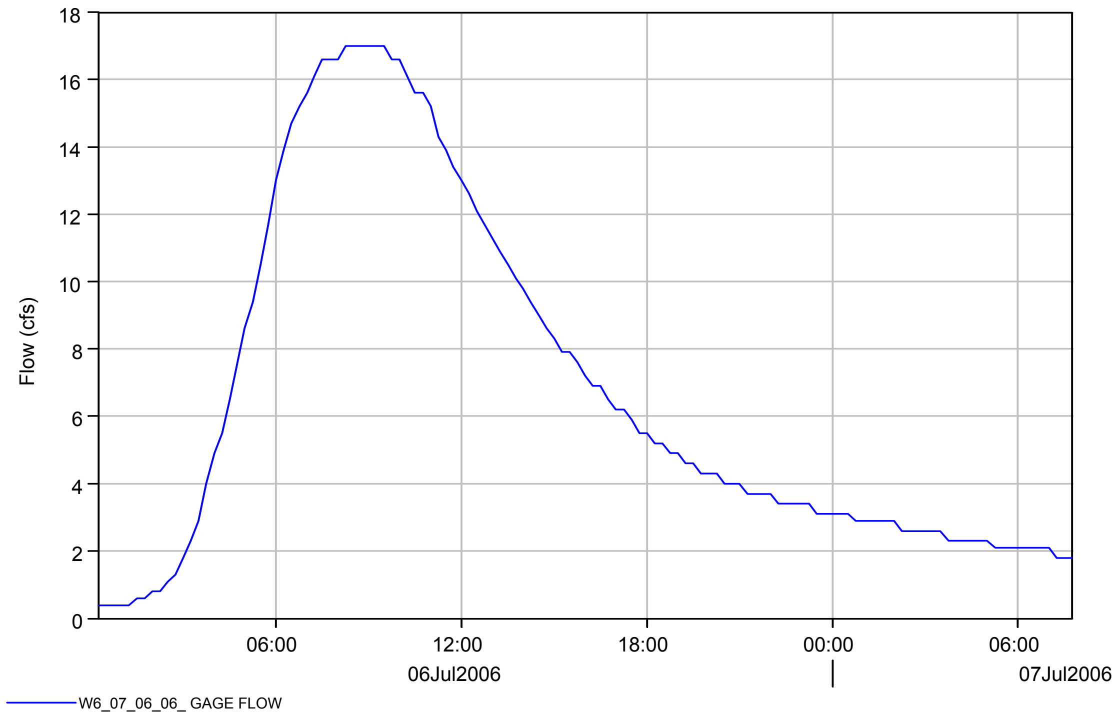


Figure 85: Observed hydrograph at station W6 for July 06, 2006 storm

Therefore using equations described in 'Event Analysis Method', values of S and λ are computed as shown below:

$$S = \frac{{P_e}^2}{Q} - P_e = \frac{(1.26)^2}{0.22} - 1.26 = 5.95 inch \qquad \text{Equation 40}$$

$$\lambda = \frac{I_a}{S} = \frac{0.15}{5.95} = 0.025 \qquad \text{Equation 41}$$

4.2.8.6 Hydrograph comparison for calibration and verification runs using different approaches

This section shows the hydrograph comparisons for the model calibration and validation runs with existing and revised reach routing and with subbasin lag time parameters. The modeled response is compared against the stream gage response. Also shown in the figures (86-90) is the response of Wreck Pond Brook using a 5% initial abstraction ratio with revised reach routing and subbasin lag time parameters.

In the following figures (86-90) (station W6, W1 and W3), the black line represents the observed hydrograph, the green line represents the existing RSWMP modeled hydrograph, the red line shows hydrograph for the revised model with 20% Ia ratio, and the blue line shows the hydrograph for revised model with 5% Ia ratio. It can be seen from the figures the revised model simulates well the stream response for high frequency storms.

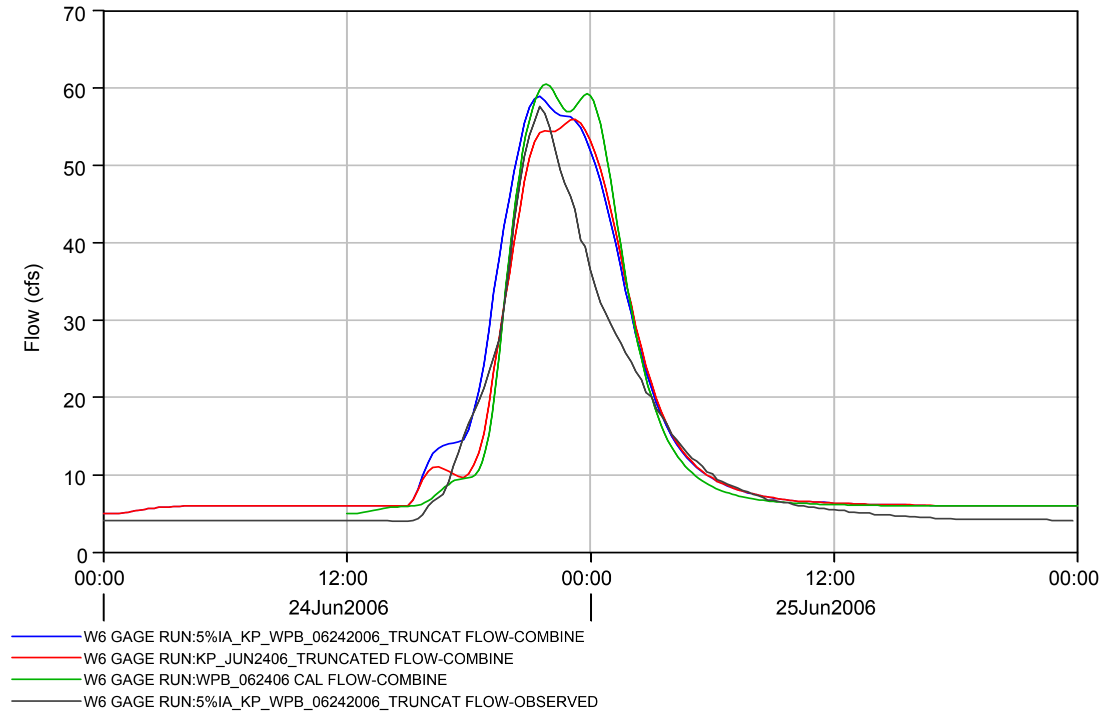


Figure 86: Hydrograph comparison for the calibration run for June 24, 2006 storm (1.93 – 0.09 = 1.84 inch) at station W6

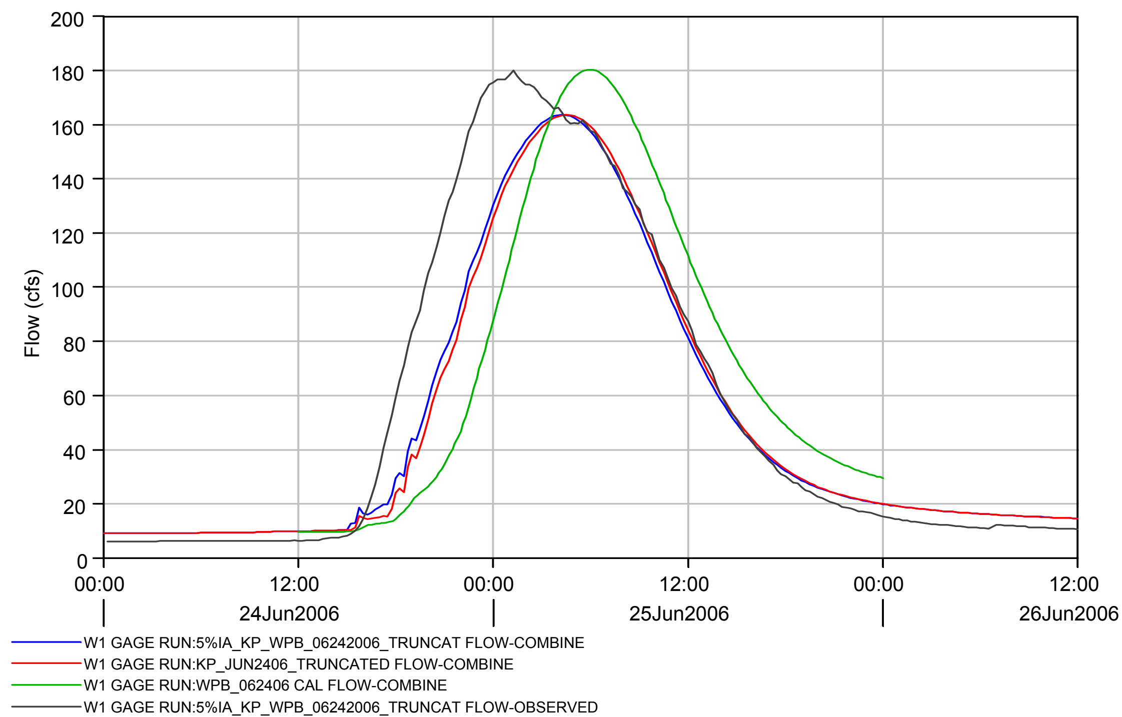


Figure 87: Hydrograph comparison for the calibration run for June 24, 2006 storm (1.93 – 0.09 = 1.84 inch) at station W1

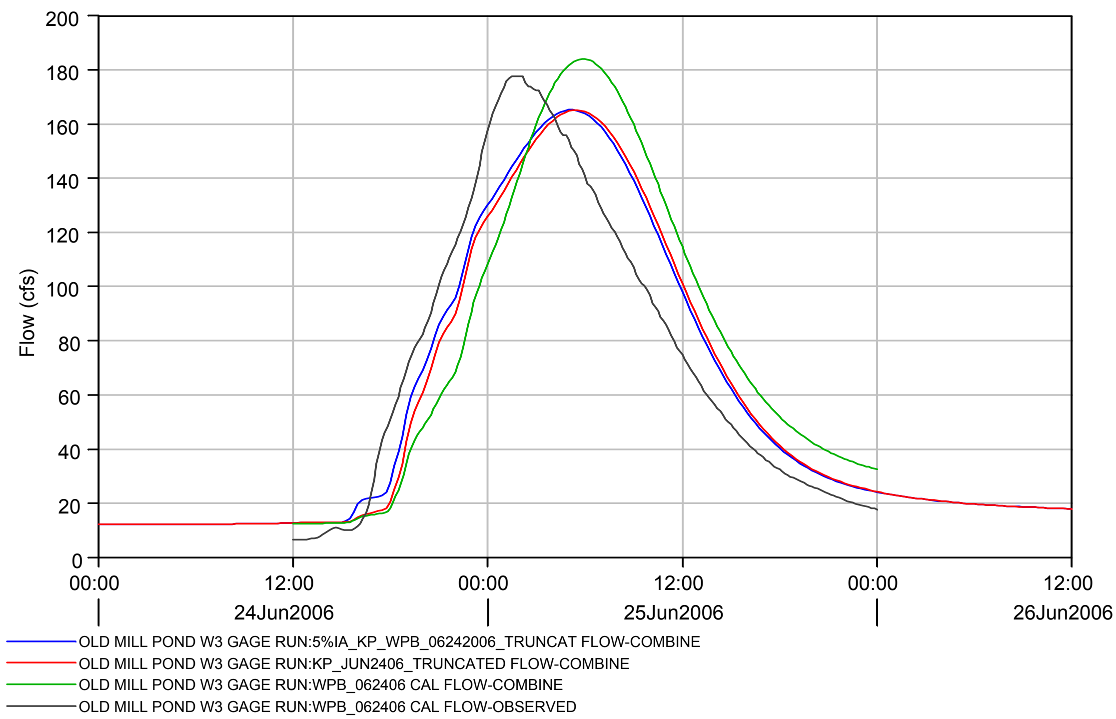


Figure 88: Hydrograph comparison for the calibration run for June 24, 2006 storm (1.93 – 0.09 = 1.84 inch) at station W3

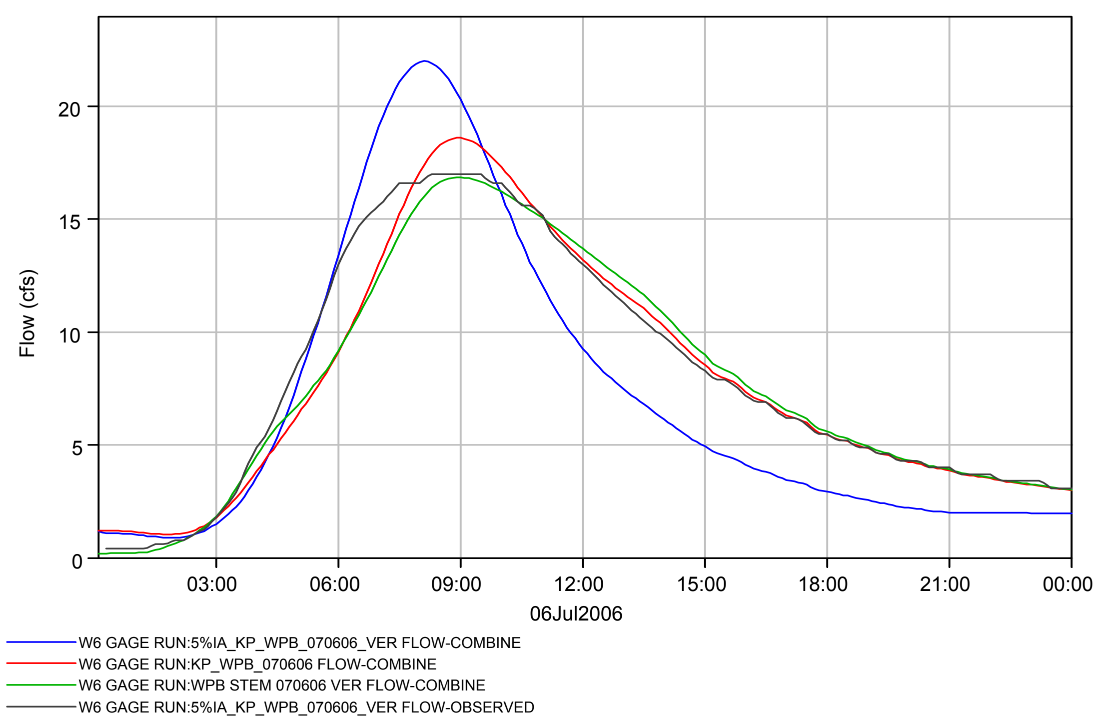


Figure 89: Hydrograph comparison for the verification run for July 06, 2006 storm event at station W6

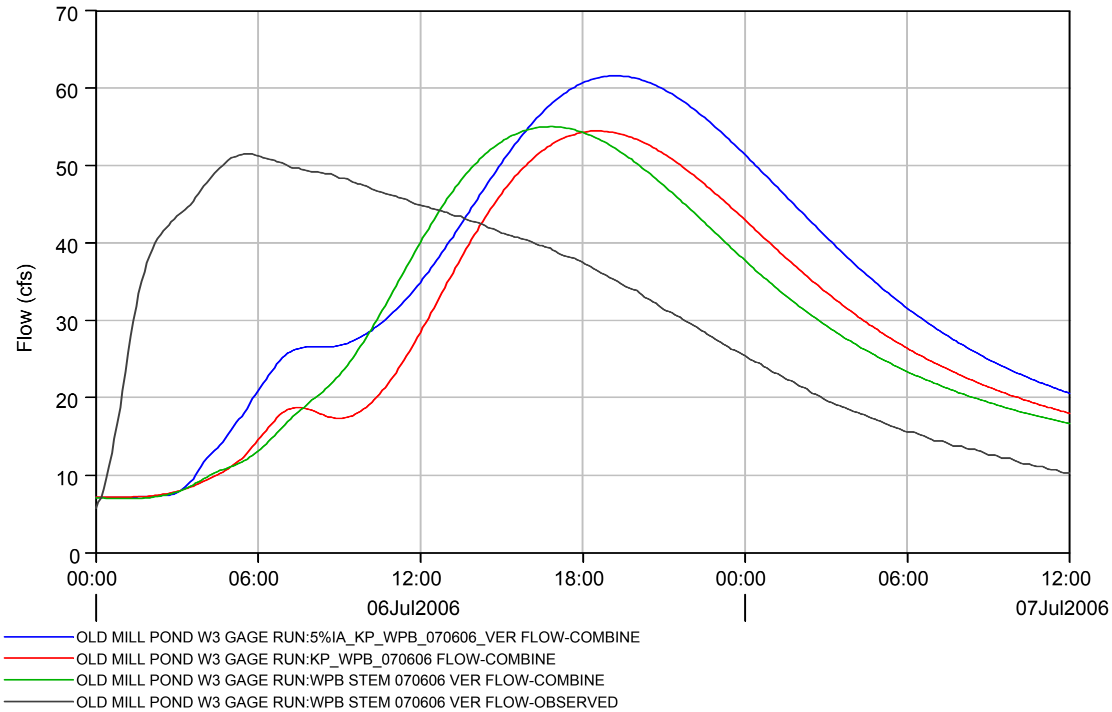


Figure 90: Hydrograph comparison for the verification run for July 06, 2006 storm event at station W3

In Figure 91 for station W2 (Hannabrand outlet), the green line shows the hydrograph from the existing model, the red line shows the observed hydrograph and the blue line shows the hydrograph from the revised model. In Figure 92 for station W5, the green line shows the observed hydrograph, the blue line shows the hydrograph from the existing model and the red line shows the hydrograph from the revised model.

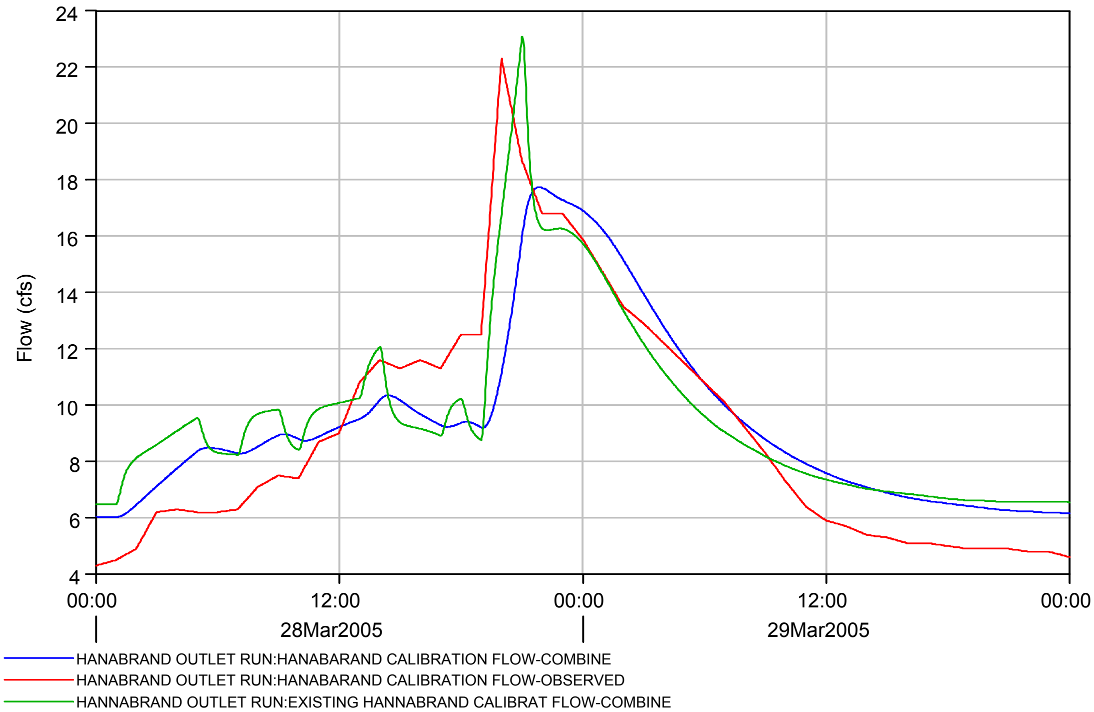


Figure 91: Hydrograph comparison for the calibration run for March 28, 2005 storm event at station W2

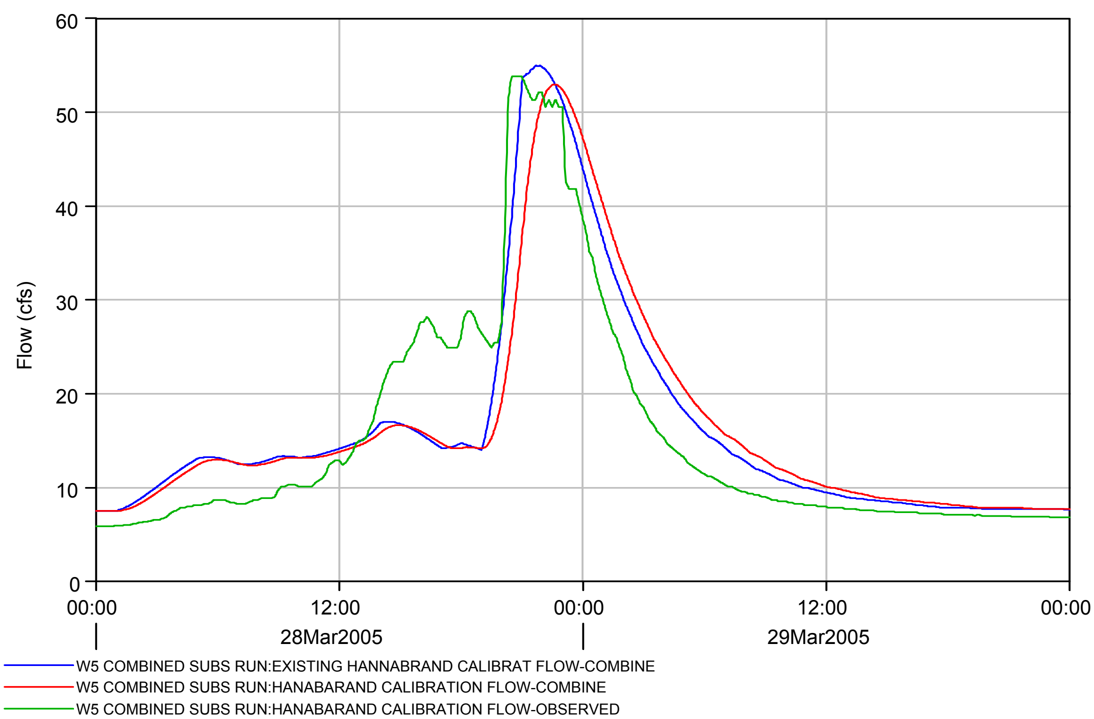


Figure 92: Hydrograph comparison for the calibration run for March 28, 2005 storm event at station W5

The following figures (93-94) show comparisons of the hydrograph for the October 2005 storm event using different approaches at the outlet of Wreck Pond Brook (station W3 – Old Mill pond) and Hannabrand Brook (station W2 – Hannabrand outlet). The approaches vary in the choice of unit hydrograph. The comparison shows revised model (modified basing lag times and stream routing parameters) output using three hydrographs, i.e. using SCS (green colored), Delmarva (blue colored) and Lower Monmouth (red colored) unit hydrographs against the existing RSWMP model using LMDUH (black colored).

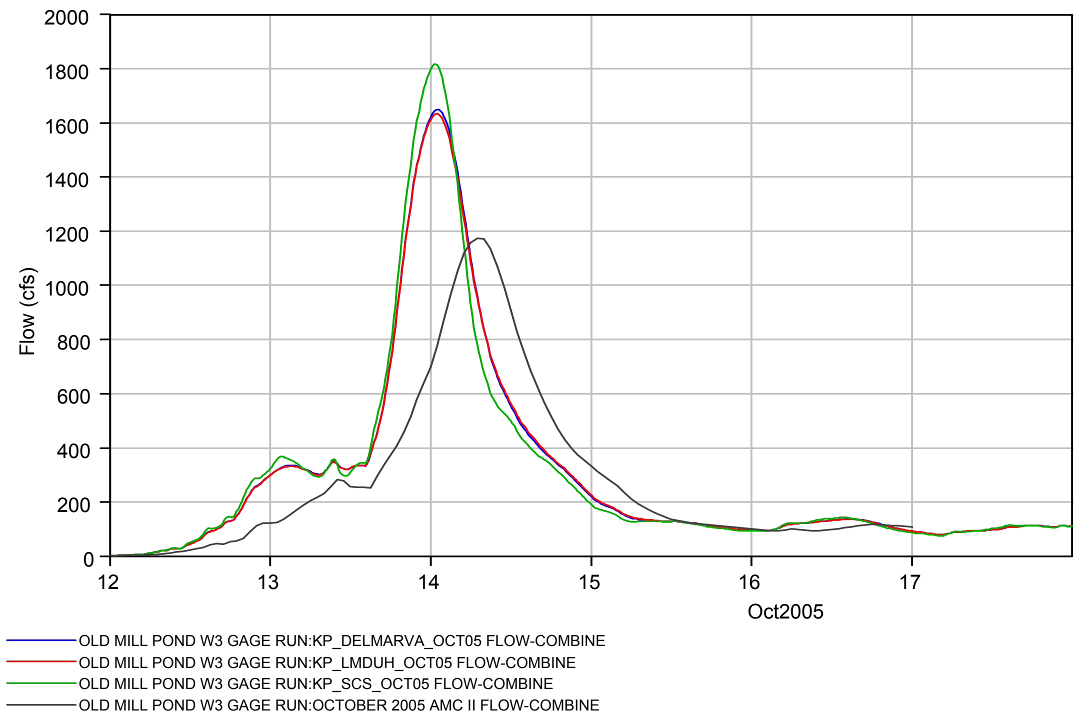


Figure 93: Hydrograph comparison using revised models (SCS, Delmarva and LMDUH) with existing RSWMP model using LMDUH for October 2005 storm event at stationW3

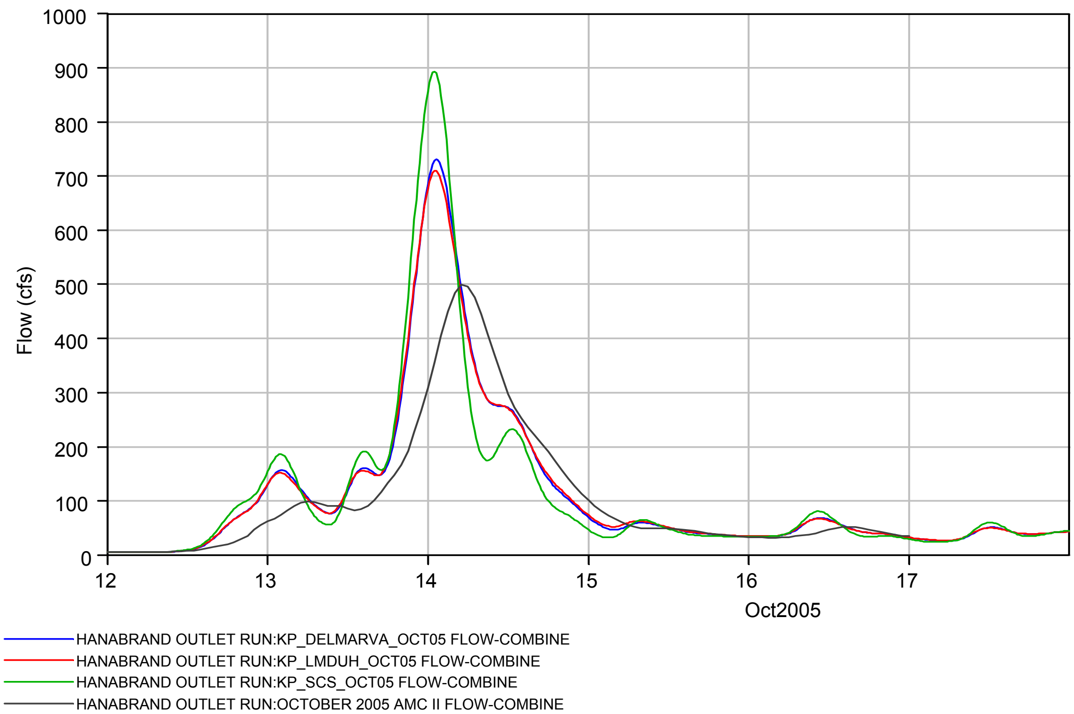


Figure 94: Hydrograph comparison using revised models (SCS, Delmarva and LMDUH) with existing RSWMP model using LMDUH for October 2005 storm event at stationW2

In the above comparison, it can be seen the existing model from RSWMP predicted a delayed peak that was confirmed by comparison to the field-measured values. The standard 484 SCS dimensionless unit hydrograph compares well for peak time, however peak flow values were computed very high. The modified model with the Delmarva and Lower Monmouth dimensionless unit hydrographs provided a good match to the observed peak time. The computed peak flows are less than the standard SCS and higher than the existing RSWMP model. The revised model using the Delmarva or LMDUH values provided good matches with peak flow timing; however the peak flow values were higher tending towards the standard SCS unit hydrograph rather than the LMDUH.

Another parameter tested besides the peak rate factor and AMC conditions was the initial abstraction ratio. The study on initial abstraction ratio was performed by several researchers and was summarized earlier in the literature review section. In the following figure 95, there is another response of WPBW using the LMDUH and a 5% initial abstraction ratio. This combination provided a better match to field measured peak timings and also the peak flows were closer to the flows using the LMDUH rather than the SCS unit hydrograph. In Figure 95, the red colored hydrograph shows the watershed response using revised reach routing and subbasin lag time values, LMDUH/Delmarva unit hydrograph and a 5% initial abstraction ratio. This combination of watershed input values models the WPBW response well in terms of peak timings and flow values.

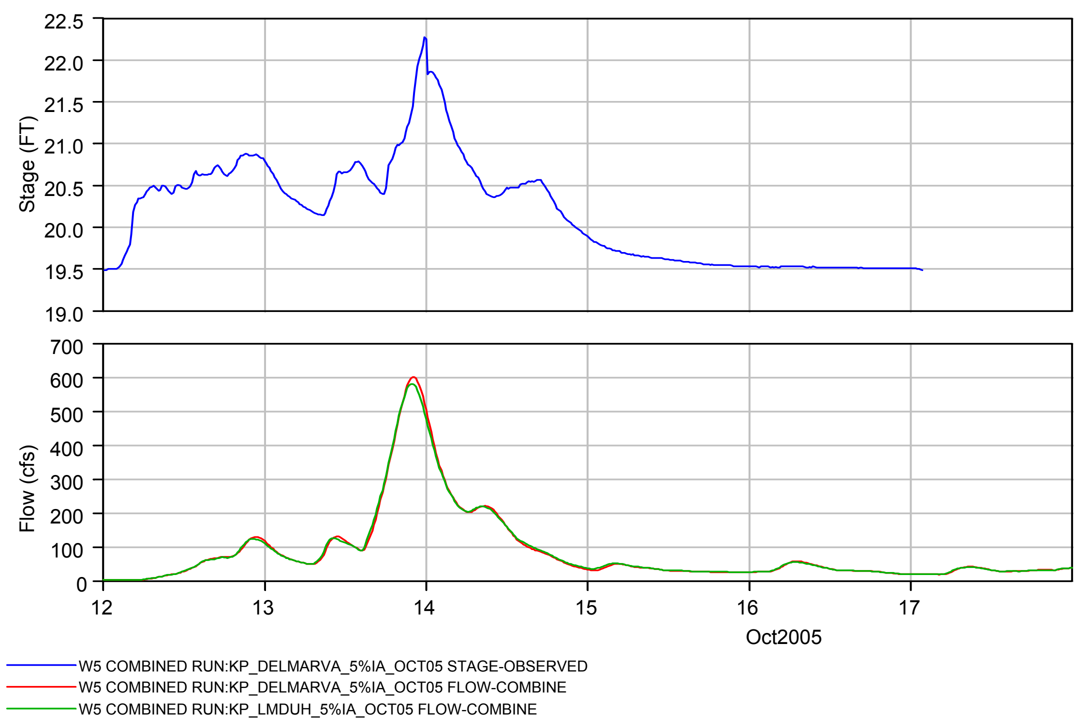


Figure 95: Modeled hydrograph comparison with the observed stream gage information at station W5

In Figure 96, the peak flow timing for observed stream response (blue colored stage hydrograph) matched well with revised model using a 5% initial abstraction ratio (red colored hydrograph). However, it can be seen that the stream gage at station W3 malfunctioned for almost one day after reaching the peak. Hence, the modeled response cannot be verified for recession limb at station W3.

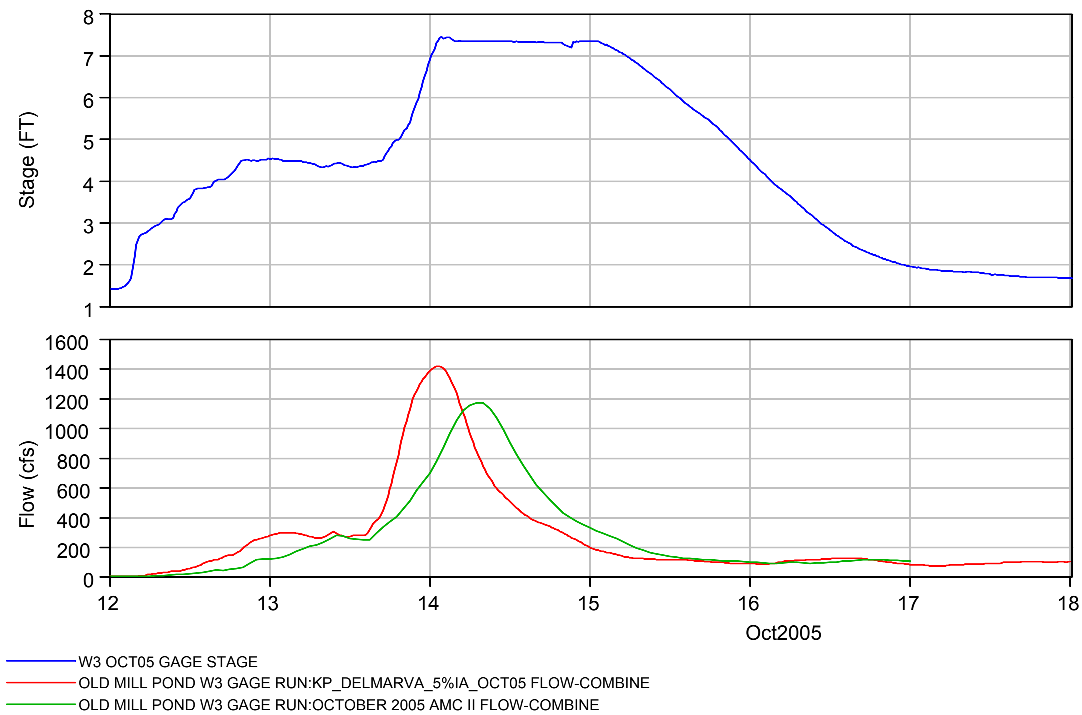


Figure 96: Modeled hydrograph comparison with the observed stream gage information at station W3

As described above, the use of the revised model with 5% initial abstraction ratio provides good match to the observed stream response for the low flow events. For the high flow events the revised model provides a better match than the existing RSWMP model. Figures (97-98) show the hydrographs at the outlet of Hannabrand Brook and Wreck Pond Brook just before the merge point using the revised model and SCS UH (black colored), Delmarva UH (blue colored) and, LMDUH (green colored). Figures

(97-98) show the response of the revised model with 5% initial abstraction ratio (red colored) and the response of the existing RSWMP model (pink colored). The existing RSWMP model was developed using PRF of 230 and 20% initial abstraction ratio. The blue colored hydrograph show the response of the revised model with PRF of 284 and 20% initial abstraction ratio. The green colored hydrograph show the response of the revised model with PRF of 230 and 20% initial abstraction ratio. It can be seen that the use of lower peak rate factor (e.g. 230) and 5% initial abstraction ratio (red colored hydrograph) provides a better match for peak flow timings and also the peak discharges.

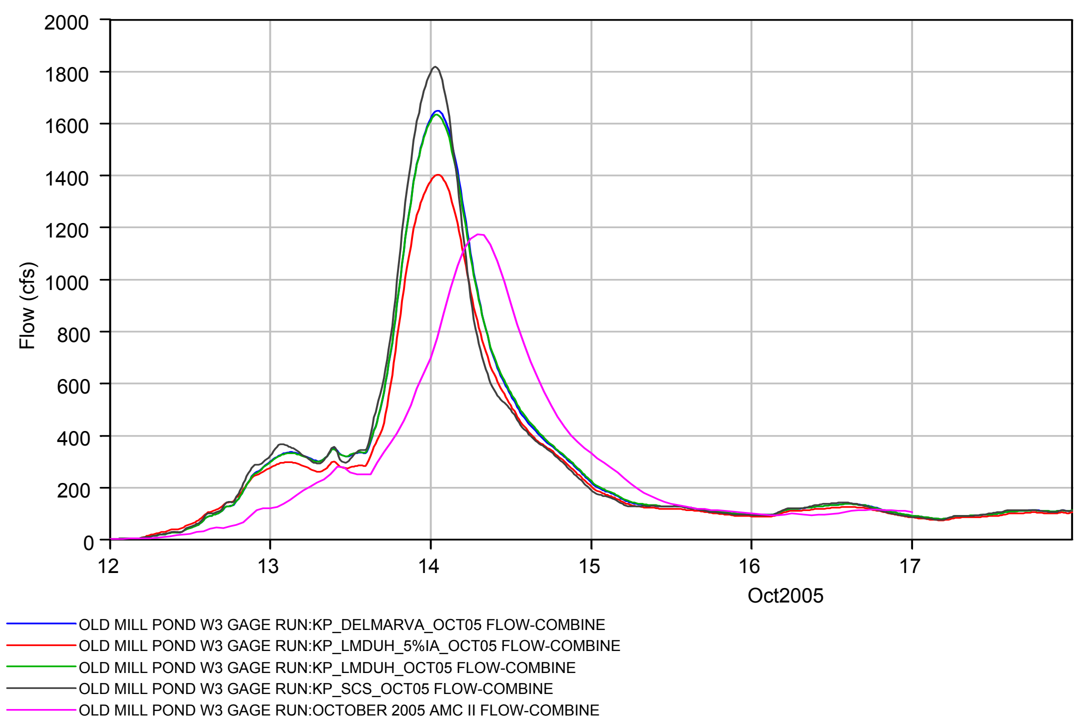


Figure 97: Hydrograph comparison for revised model with existing RSWMP model for October 2005 storm event at stationW3

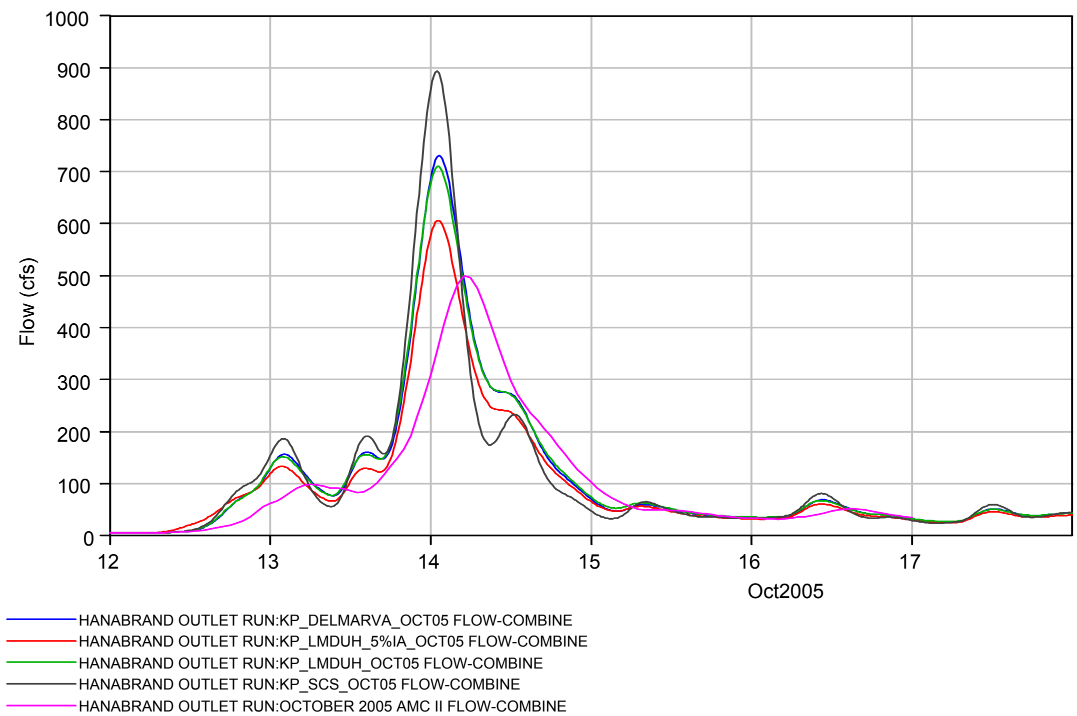


Figure 98: Hydrograph comparison for revised model with existing RSWMP model for October 2005 storm event at stationW2

The response of the watershed to Delmarva and LMDUH with a 5% initial abstraction ratio was compared for October 2005 storm event. This comparison is important, as the current regulations by NJDEP require use of Delmarva for coastal areas in New Jersey where appropriate. It needs to be justified to change this procedure to use a PRF of 230 for WPBW or for the lower Monmouth County region. However, this study as shown below finds it is not necessary to make this change as it seems to provide little benefit, if any, in doing a major overhaul of existing methodology at NJDEP and the municipalities in WPBW.

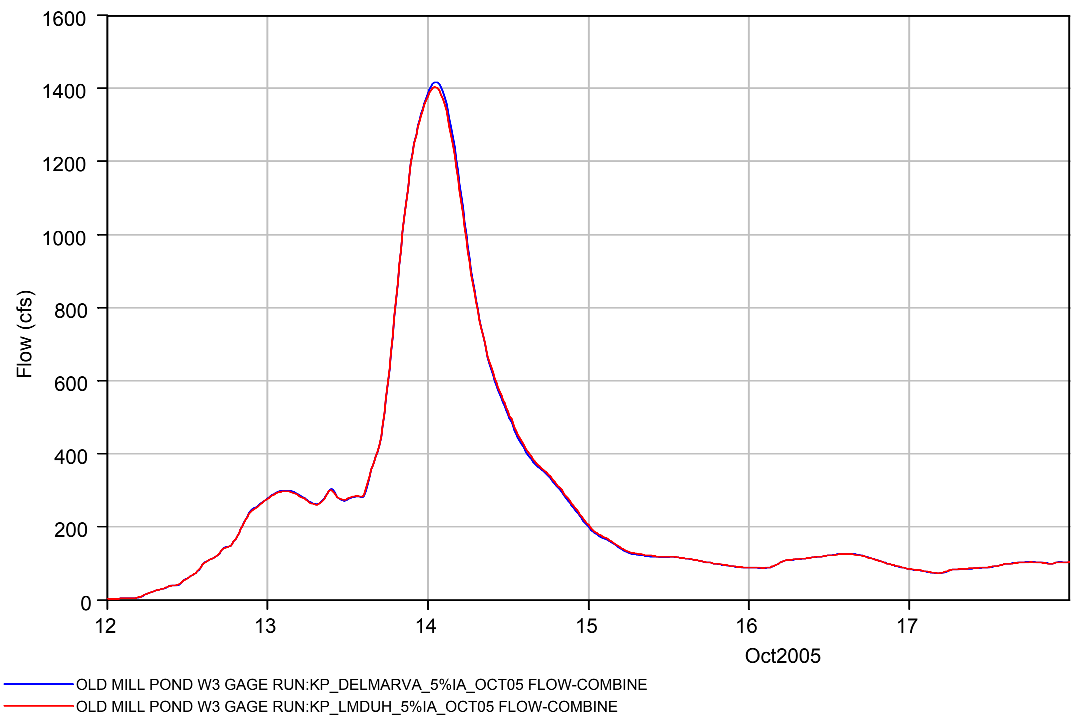


Figure 99: Delmarva vs. LMDUH with 5%Ia at station W3 for October 2005 storm

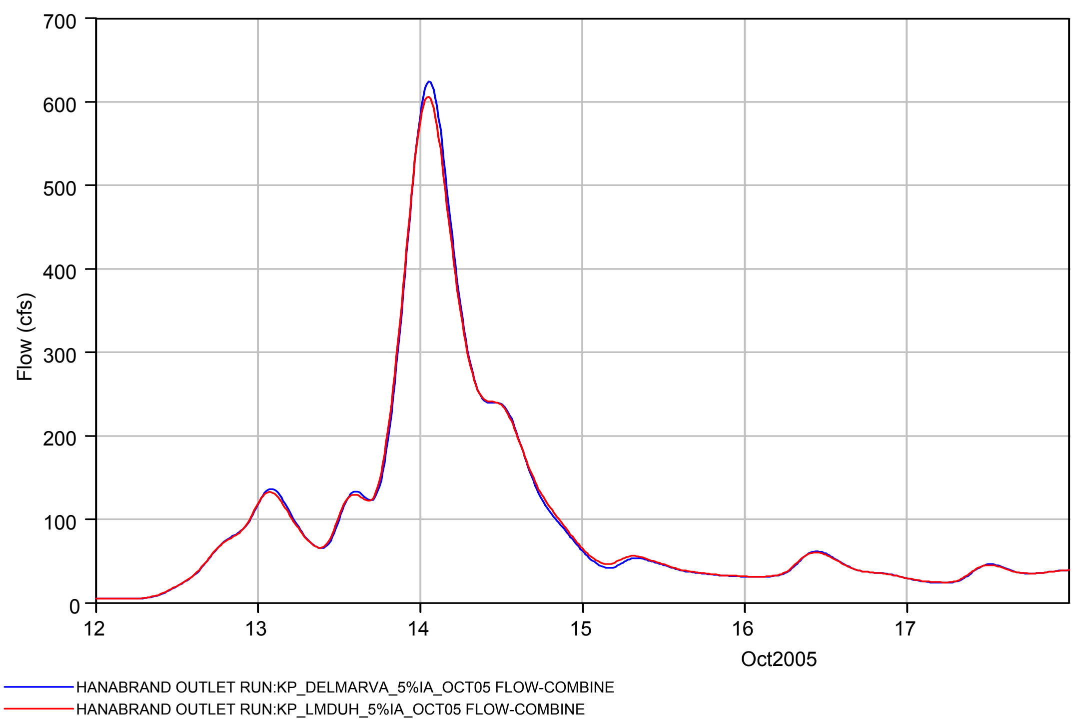


Figure 100: Delmarva vs. LMDUH with 5%Ia at station W2 for October 2005 storm

4.2.9 Modeling NRCS statistical storms

Table 31 lists the rainfall values for 1, 2, 5, 10, 25, 50 and 100-year storm events for all the counties in New Jersey. The values from this table are used to model 2, 10, 25, 50 and 100-year storm events for the Wreck Pond Brook watershed.

Table 31: New Jersey 24 hour rainfall frequency data

County	1 year	2 year	5 year	10 year	25 year	50 year	100 year
Atlantic	2.72	3.31	4.3	5.16	6.46	7.61	8.9
Bergen	2.75	3.33	4.26	5.06	6.25	7.29	8.43
Burlington	2.77	3.36	4.34	5.18	6.45	7.56	8.81
Camden	2.73	3.31	4.25	5.06	6.28	7.33	8.51
Cape May	2.68	3.27	4.24	5.08	6.37	7.5	8.77
Cumberland	2.69	3.27	4.25	5.09	6.37	7.5	8.77
Essex	2.84	3.44	4.4	5.22	6.44	7.5	8.67
Gloucester	2.71	3.29	4.23	5.04	6.27	7.34	8.52
Hudson	2.74	3.31	4.24	5.02	6.18	7.19	8.3
Hunterdon	2.8	3.38	4.26	5	6.09	7.02	8.03
Mercer	2.74	3.31	4.23	5.02	6.2	7.21	8.35
Middlesex	2.76	3.35	4.3	5.12	6.36	7.43	8.63
Monmouth	2.79	3.38	4.37	5.23	6.52	7.66	8.93
Morris	2.94	3.54	4.46	5.23	6.36	7.31	8.35
Ocean	2.81	3.42	4.45	5.33	6.68	7.87	9.2
Passaic	2.87	3.47	4.42	5.24	6.45	7.49	8.64
Salem	2.69	3.26	4.2	5.01	6.23	7.29	8.47
Somerset	2.77	3.34	4.25	5.02	6.17	7.16	8.24
Sussex	2.68	3.22	4.02	4.7	5.72	6.6	7.58
Union	2.79	3.38	4.34	5.17	6.41	7.48	8.68
Warren	2.78	3.34	4.18	4.89	5.93	6.83	7.82

The NRCS 100-year and 50-year storm events are used in the flooding analysis. The 25-year and 10-year storm events are used frequently in the sizing of conveyance structures. The 2-year storm event provides information on bankfull flow values, which is important for sediment discharge and stream erosion analysis. Hence, for WPBW the model was run for 2, 10, 25, 50 and 100-year storm events.

Table 32 shows the response of various hydrologic elements of the WPBW hydrologic model in terms of peak flow rate, time of peak, and total volume of runoff for the NRCS 100-year storm event. The table shows the results for the Delmarva (PRF 284) unit hydrograph. The table also provides information for the drainage area (square miles) and runoff volume (inch) for the corresponding element. The blank cells indicate the model did not calculate the runoff volume for that element. The model could not calculate the runoff volume for subsequent junctions because the drainage area for the base flow source element for subwatershed W8 was not specified. However, since this study is limited to the confluence of Wreck Pond Brook and Hannabrand Brook downstream of Old Mill Pond Road, it was decided not to address the issue. The HEC-HMS simulation results can be found in Appendix E for all the hydrologic elements in WPBW model for 100, 50, 25, 10 and 2-year NRCS storm events using percentage curves created from the SCS, Delmarva and Lower Monmouth dimensionless unit hydrographs.

Table 32: HMS simulation results for 100-Year NRCS storm event using percentage curve created from Delmarva unit hydrograph (PRF 284) and 5% initial abstraction ratio

HYDROLOGIC ELEMENT	DRAINAGE AREA (SQ. MILE)	PEAK FLOW (CFS)	TIME OF PEAK [a]	VOL. (IN)
Albers Pond	0.97000	438.3	24Jun2006, 14:30	5.01
Albers pond da	0.93000	597.9	24Jun2006, 13:15	4.88
Albers Pond Surface	0.04000	127.5	24Jun2006, 12:15	8.93
Albers to Osborne	0.97000	414.8	24Jun2006, 15:45	5.01
Hanabrand Outlet	2.94550	653.7	24Jun2006, 19:00	4.15
HP to W7	3.11250	921.6	24Jun2006, 18:45	4.57

Hurley's Pond	3.11250	1008.9	24Jun2006, 16:15	4.57
Junction-1	0.42200	330.6	24Jun2006, 13:15	5.50
Junction-3	Not Specified	500.5	24Jun2006, 13:15	
Kellers Pond	0.34250	358.2	24Jun2006, 13:00	5.62
McDowel Pond	0.59000	327.9	24Jun2006, 13:15	4.26
McD to W7	0.59000	259.9	24Jun2006, 14:15	4.26
Mews Combined Basin	0.17200	151.6	24Jun2006, 13:15	5.66
Mews Storm sewer	0.25000	180.3	24Jun2006, 13:30	5.39
Old Mill Pond W3 gage	7.27050	1496.9	24Jun2006, 21:15	4.61
Old Mill-Wreck Pond	10.21600	1946.1	25Jun2006, 00:30	4.47
Osbonre to W1	6.71250	1507.6	24Jun2006, 19:45	4.54
Osborne Pond	6.71250	1510.8	24Jun2006, 19:15	4.54
Osborne Pond da	0.85000	775.9	24Jun2006, 12:45	4.50
Osborne Pond surface	0.03000	95.6	24Jun2006, 12:15	8.93
Parkway	0.04000	45.2	24Jun2006, 13:15	8.93
Rt34-W6	0.34250	251.0	24Jun2006, 14:30	5.62
Rt 35 culvert	2.94550	696.1	24Jun2006, 17:15	4.15
Rt 35 to Outlet	2.94550	653.7	24Jun2006, 19:00	4.15
Rt 71 East	0.00512	13.8	24Jun2006, 12:15	6.67
RT 71 w/Imp	0.02000	49.4	24Jun2006, 12:15	6.04
Slgc flowpath	0.02519	21.5	24Jun2006, 12:45	3.71
Slgc lake	0.64040	462.2	24Jun2006, 13:15	5.07
Slgc lakes only	0.00440	14.0	24Jun2006, 12:15	8.93
Slgc upper lake	0.42200	329.7	24Jun2006, 13:15	5.50
SL-SlH-SG	1.76000	1110.1	24Jun2006, 13:30	5.37
W1	0.07800	160.2	24Jun2006, 12:15	5.78
W1 gage	6.79050	1513.1	24Jun2006, 19:45	4.56
W1-W3	6.79050	1484.2	24Jun2006, 20:30	4.55
W2 Rt 35	0.13750	100.2	24Jun2006, 13:15	7.13
W2 subdivision at Baileys	0.16250	59.7	24Jun2006, 14:30	5.95
W3	0.48000	372.5	24Jun2006, 13:00	5.50
W5	2.33000	553.6	24Jun2006, 15:15	3.84
W5 add/l housing	0.07800	42.3	24Jun2006, 13:15	4.06

W5 combined	2.64550	659.4	24Jun2006, 15:00	3.98
W5 housing development	0.23750	131.2	24Jun2006, 13:15	5.37
W5-W2	2.80800	663.2	24Jun2006, 17:15	4.01
W6	0.93000	333.4	24Jun2006, 13:45	4.27
W6_Rt 34	0.34250	380.5	24Jun2006, 12:45	5.62
W6 gage	1.27250	568.7	24Jun2006, 14:15	4.63
W6-Hurley Pond	1.31250	515.9	24Jun2006, 16:30	4.76
W7	4.86250	1255.6	24Jun2006, 18:15	4.45
W7_Glen	1.16000	392.9	24Jun2006, 14:30	4.21
W7_McD	0.59000	328.8	24Jun2006, 13:15	4.26
W7 to Osborne Pond	4.86250	1237.4	24Jun2006, 19:30	4.45
W8 base flow	Not Specified	12.0	24Jun2006, 00:00	
W8 Mews	0.17200	159.9	24Jun2006, 13:00	5.66
W8 SLGC	0.21400	144.0	24Jun2006, 13:00	4.15
W8 urban lower	0.02519	21.7	24Jun2006, 12:30	3.71
W8 urban upper	0.25000	182.8	24Jun2006, 13:15	5.39
W9	1.80000	683.4	24Jun2006, 14:00	4.44
W9 junction	3.11250	1015.3	24Jun2006, 15:45	4.57
WPB-HAN Combined	10.21600	2084.5	24Jun2006, 20:15	4.48
WP Junction	Not Specified	2098.6	25Jun2006, 00:00	
Wreck Pond	Not Specified	607.7	25Jun2006, 10:30	
Albers Pond	0.97000	438.3	24Jun2006, 14:30	5.01

a : Storm event is from the hypothetical start time of 24 June 2006 @ 00:00 hours.

4.3 GIS based Hydraulic Modeling

4.3.1 Developing rating curves for each sub watershed outlets

Water surface elevation, average cross sectional area and average velocity are the necessary data to develop a rating curve (Water surface Elevation - *feet* vs. Flow - *cfs* relationship) for the stream at the selected gage locations. The real-time elevation data were collected using the miniTROLL pressure sensor device with Win-Situ 4.5 Instrument control software manufactured by In-situ Inc (www.in-situ.com). Velocity readings were taken using a USGS-type current meter with AquaCalc Pro software manufactured by JBS instruments (www.jbsinstruments.com). Velocity readings were converted to flow data with the AquaCalc computer by multiplying channel cross-sectional area to the velocity.

The pressure sensor devices were installed at the outlet of each sub watershed. The Mini-Troll unit was approximately seven-eighths of an inch in diameter and twelve inches long and contains a pressure transducer, circuitry, memory and storage for two AA sized batteries. It was housed inside a two inch PVC pipe and connected via a cable that extended to the surface. The PVC pipe was anchored to a concrete block to avoid sensor displacement by high flow events. The end of the cable was vented to the atmosphere and was housed inside a four inch PVC pipe. The equipment was attached to a tree or bridge to resist stream flows during the low frequency storm events. The PVC tube itself was vented to allow for changes in water level in the stream to rise and fall within the PVC piping without creating "back pressure" on the end of the vented cable contained inside the four-inch PVC housing. Also, a staff gage was installed as a secondary check for the data from a pressure sensor device and as a field backup in case the miniTroll unit was not working properly. The following figures show the staff gage and the miniTroll unit housed inside the PVC pipe.

Figure 101: Staff gage and miniTroll instrument housed inside PVC pipe anchored in a concrete block

Figure 102: Staff gage and miniTroll instrument housed inside PVC pipe and anchored to a culvert

The elevation information stored in the miniTroll unit was downloaded to a "Rugged Reader" TM, a robust PDA device running the Windows Pocket PC operating system. Software from In-Situ Inc was installed on the PDA and was connected to the miniTroll vented cable. The retrieved data was stored in ASCII format and later downloaded to a PC and exported to Excel for editing. The PDA software, "Pocket-Situ" allowed viewing both the text data and small graphs directly on the PDA itself giving an opportunity to quality check the data while still in the field.

Table 34 shows the data collected by the miniTroll pressure sensor devices. It provides the start and stop time and the time interval between the data collection. It also provides the total number of data points and date the report was generated. The elevation information collected was represented in terms of channel pressure (feet of water) above the pressure transducer. This information was converted into water surface elevation by adding the recording gage elevation (datum) to the feet of water. For example, in the data shown below (downloaded from Aquacalc into PC) 12.42 feet of gage elevation was added to the channel pressure which then gave the stream elevation at gage W1 in feet at the recorded time. Najarian Associates (Eatontown, NJ) surveyed the gage elevations.

Table 33: Gage elevations (Datum) – Staff gage and Recording gage

ELEV	Item	Station
15.77	staff gage	W1
12.42	recording gage pvc	W1
6.45	staff gage	W2
2.80	recording gage pvc	W2
1.83	recording gage pvc	W3
5.33	staff gage	W3
18.10	recording gage pvc	W5
21.58	staff gage	W5
62.82	recording gage pvc	W6
65.96	staff gage	W6
27.98	recording gage pvc	W7
32.34	staff gage	W7
2.29	recording gage pvc	W8
5.36	staff gage	W8
54.18	weir	W9
54.18	weir	W9
62.88	recording gage pvc	W9
58.21	staff gage	W9

Table 34: Sample file of real time elevation data downloaded into PC

In-Situ Inc.	MiniTroll Std P		
Report generated:	10/25/2005	9:34:24	
Report from file:	...\SN17908 2005-06-29 104618 w1.bin		
Win-Situ Version	4.51		
Serial number:	17908		
Firmware Version	3.09		
Unit name:	w1		
Test name:		w1	
Test defined on:	6/29/2005	10:46:13	
Test started on:	6/29/2005	10:46:18	
Test stopped on:	N/A	N/A	
Data gathered using Linear testing			
Time between data points:			
1800.0	Seconds.		
Number of data samples:	5470		
TOTAL DATA SAMPLES	5470		
Channel number [2]			
Measurement type:	Pressure		
Channel name:	OnBoard Pressure		
Sensor Range:	5 PSIG.		
Specific gravity:	1		

Date	Time	ET (sec)	ET (hrs)	Chan[2] Pressure Feet H2O
-------	-------	---------		-----------
6/29/2005	10:46:18	0	0	0.701
6/29/2005	11:16:18	1800	0.5	0.692
6/29/2005	11:46:18	3600	1	0.692
6/29/2005	12:16:18	5400	1.5	0.685
6/29/2005	12:46:18	7200	2	0.689
6/29/2005	13:16:18	9000	2.5	0.686
6/29/2005	13:46:18	10800	3	0.687
6/29/2005	14:16:18	12600	3.5	0.693
6/29/2005	14:46:18	14400	4	0.692
6/29/2005	15:16:18	16200	4.5	0.68
6/29/2005	15:46:18	18000	5	0.687
6/29/2005	16:16:18	19800	5.5	0.679
6/29/2005	16:46:18	21600	6	0.682

To develop the data points for the lower portion of the rating curve, the water flow was measured in the channel and the corresponding water depth was recorded. One such measurement provided one data point on the stream rating curve. Multiple data points were collected by measuring stream flow for varying water depths. The flow measurements in the field were limited to bankfull flows as safety was warranted for measuring flows higher than bankfull flow. The USGS – type current meter mounted on a wading rod was used to measure velocities across the stream channel. The meter consists of cups mounted on an axis which rotate in the direction of flow when submerged in water. These rotations generate electric current. A device connected to the cups via a cable converts the current into velocity units.

The current meter measures the velocity at a depth where the cups intersect the water flow. The six-tenths depth method was used to estimate average velocity at a vertical. Actual observations and mathematical theory has shown that the 0.6 depth method gives reliable results (Buchanan and Somers, 1976). This method is acceptable whenever the water depth is between 0.3 feet and 2.5 feet. In this method, flow velocity is measured at the 0.6 of the depth below the surface and is used as an average velocity for that vertical. Several such average vertical velocities measured across the channel cross-section give the average channel velocity for that cross-section. The channel cross-section was established using a 100-foot flexible tape reel. The tape was anchored at both banks and average vertical velocity was measured at one-foot intervals or less. AquaCalc Pro attached to a current meter recorded measurements at each station. The software provides the option to select velocity measurement intervals at 0.6 depths. The velocity readings were averaged over a forty-second time period to determine average flow velocity. The attached computer calculated the cross-sectional area for each section by multiplying the width of each section by the depth. It then multiplied the area with average velocity for that section giving the average flow rate through the section. The first reading (near bank) and last reading (far bank) had depth and velocity set to 0.0 which signaled the beginning and end of a section measurement. Adding average flow rates for all the sections across the cross section gave the average flow rate through the channel cross-section.

Once the data points for the lower portion of a rating curve were collected as explained above, additional data points for higher elevations were obtained using HEC-RAS modeling. The RAS model was calibrated for flows up to bankfull flows using data points collected in the field. The out-of-bank flows and corresponding elevations were computed using this calibrated model. The GIS elevation data was used with surveyed channel geometry to create TIN and established the necessary accuracy for the sections of stream channels at the sub-watershed outlets. The accuracy of the GIS elevation data was photogrammetric certified as +/- one foot vertical and horizontal. The GIS elevation data and the channel surveys provided the DTM necessary for the hydraulic simulation to compute water surface profiles for the surveyed length of the stream. WPBW was divided into eight sub-watersheds based on the outlets where flow and water quality measurements were taken. TIN was developed for each sub-watershed to facilitate the calculations.

HEC-GeoRAS provided the connectivity between GIS and HEC-RAS. Stream centerline, bank lines, flow path centerlines, cross-section cutlines and ineffective flow areas layers were created in the ArcGIS environment using HEC-GeoRAS. The created layers were verified based on field inspection. HEC-GeoRAS enables automated calculation of stream and station lengths. The 3-D cross-sections were developed using XS cutlines layer together with TIN.

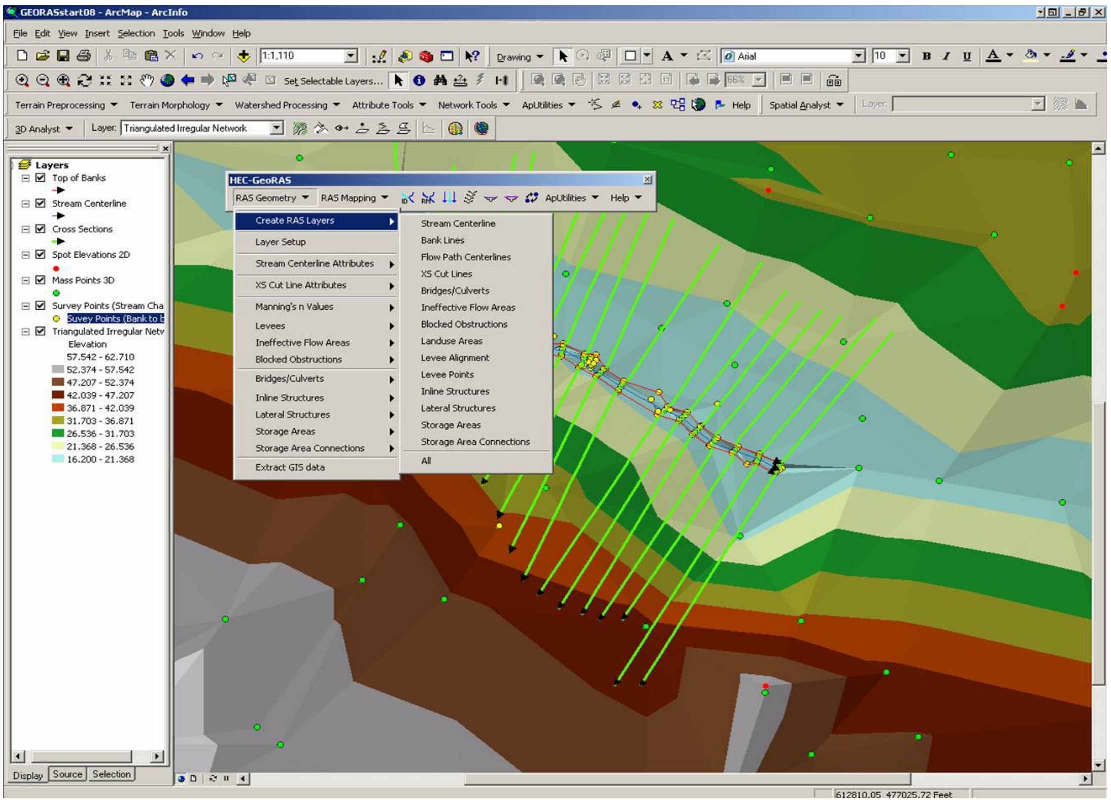


Figure 103: Geometric data overlaying TIN and HEC-GeoRAS toolbar open

HEC-RAS has a built-in option to import geometry data in GIS format. The channel and floodplain elevations, right and left channel bank locations and downstream cross-section distance were derived from geometry files exported from GIS. The Manning's roughness coefficient for the channel and overbanks were optimized using field verification and model calibration. The normal depth parameter was specified as the boundary condition. The model was calibrated for known water surface elevations, velocities and flows measured in the field. The tables below (Tables 35-41) show the flow and elevation data obtained from the field measurements for the gages at sub-watershed outlets. All units of elevations are in feet.

Table 35: Station W6 – Measured Elevation and Flow Data

Date	Staff Gage	Recording Gage	WS Elevation Staff Gage	WS Elevation Recording Gage	Flow (cfs)
5/6/2005	1.56	1.59	64.19	64.41	2.75
7/8/2005	1.76	1.826	64.39	64.65	6.86
10/13/2005	2.8	2.876	65.43	65.70	81.85

Table 36: Station W9 – Measured Elevation and Flow Data

Date	Staff Gage	Recording Gage	WS Elevation Staff Gage	WS Elevation Recording Gage	Flow (cfs)
10/13/2005			58.21		68.96

Table 37: Station W7 – Measured Elevation and Flow Data

Date	Staff Gage	Recording Gage	WS Elevation Staff Gage	WS Elevation Recording Gage	Flow (cfs)
07/20/2005	0.58	1.51	29.60	29.49	3.72
06/10/2005	1.60	1.59	30.62	29.58	5.66
11/30/2005	2.60	3.07	31.62	31.05	45.37

Table 38: Station W1 – Measured Elevation and Flow Data

Date	Staff Gage	Recording Gage	WS Elevation Staff Gage	WS Elevation Recording Gage	Flow (cfs)
07/20/2005	0.48	0.55	12.93	12.97	7.77
06/10/2005	0.50	0.61	12.95	13.03	9.34
07/08/2005	1.87	1.87	14.32	14.29	54.23
11/30/2005	2.32	2.27	14.77	14.69	58.73

Table 39: Station W3 – Measured Elevation and Flow Data

Date	Staff Gage	Recording Gage	WS Elevation Staff Gage	WS Elevation Recording Gage	Flow (cfs)
06/07/2005	0.95	1.48	2.96	3.11	11.60
07/20/2005	0.82	1.28	2.83	2.91	7.43

10/26/2005	2.18	2.65	4.19	4.28	46.71

Table 40: Station W5 – Measured Elevation and Flow Data

Date	Staff Gage	Recording Gage	WS Elevation Staff Gage	WS Elevation Recording Gage	Flow (cfs)
09/20/2005	1.16	1.37	19.42	19.34	1.85
05/10/2005	1.43	1.73	19.69	19.69	3.35
06/07/2005	1.44	1.69	19.70	19.66	3.19
08/09/2005	1.55	1.96	19.81	19.93	6.03

Table 41: Station W2 – Measured Elevation and Flow Data

Date	Staff Gage	Recording Gage	WS Elevation Staff Gage	WS Elevation Recording Gage	Flow (cfs)
05/10/2005	1.27	1.47	4.40	4.27	4
07/20/2005	1.16	1.32	4.29	4.12	2.92
10/13/2005	2.75	2.85	5.88	5.65	34.54
10/26/2005	1.32	1.74	4.45	4.54	5.52

The following figures (Figures 101-111) show the rating curves developed for each station at the subwatershed outlets using field measured values and HEC-RAS as described above.

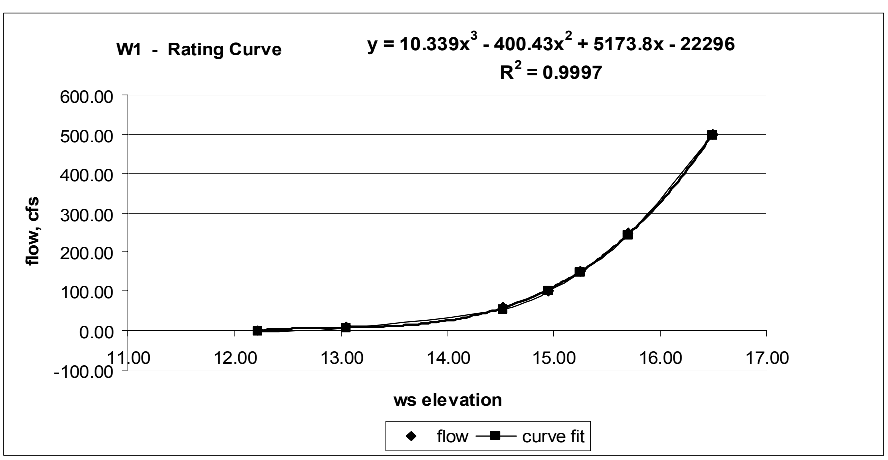


Figure 104: Rating curve for station W1

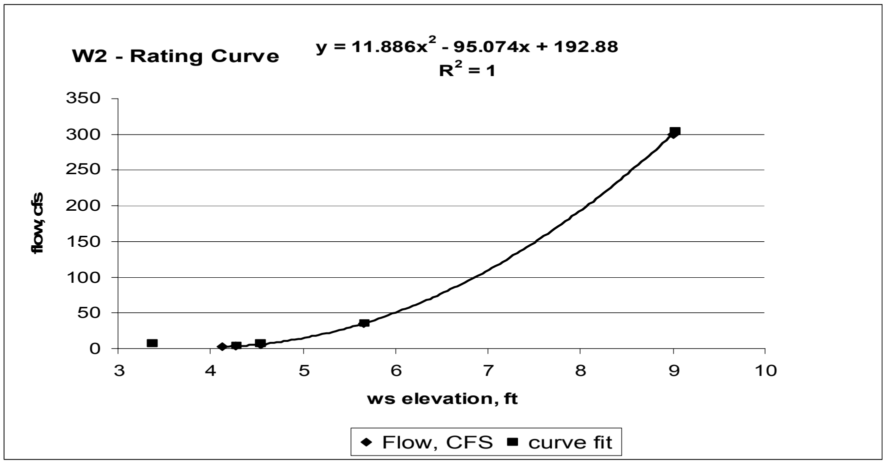


Figure 105: Rating curve for station W2

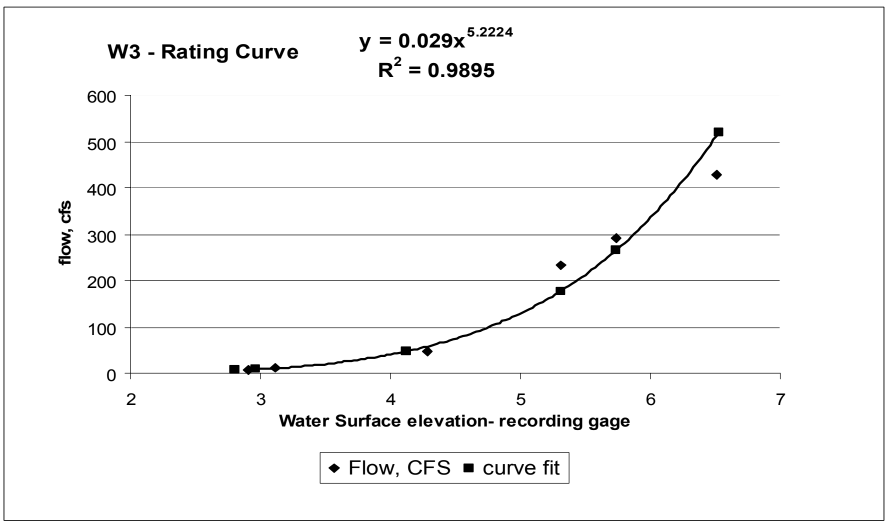
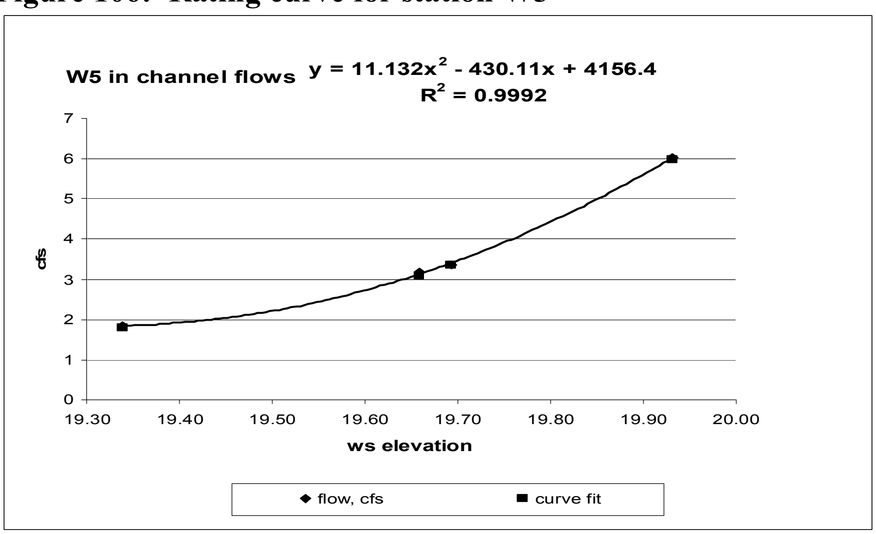


Figure 106: Rating curve for station W3



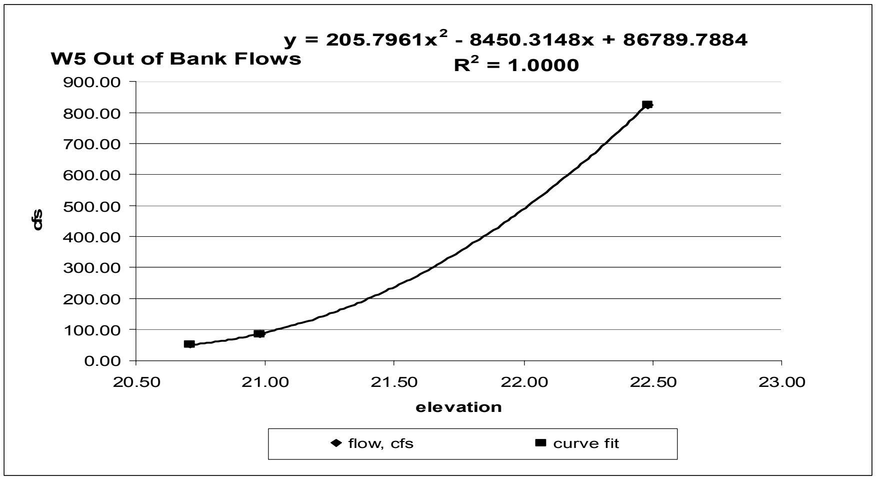


Figure 107: Rating curve for station W5

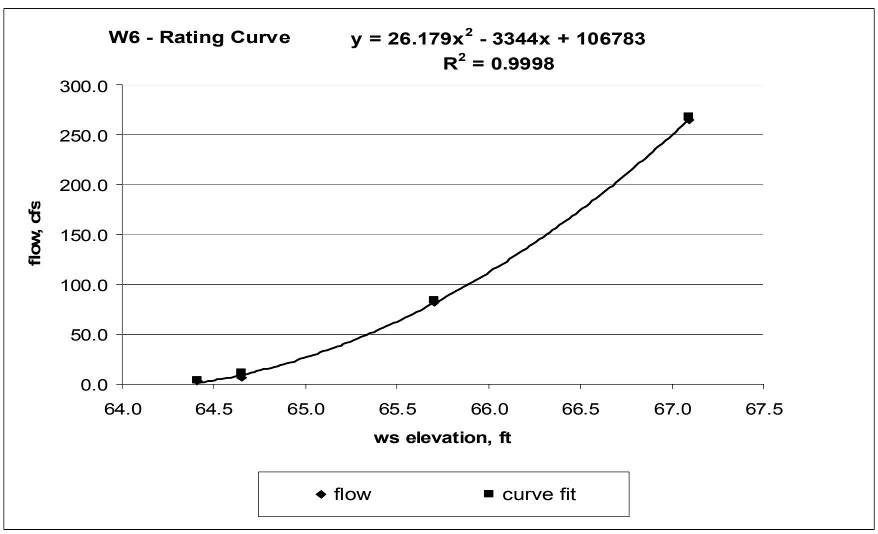


Figure 108: Rating curve for station W6

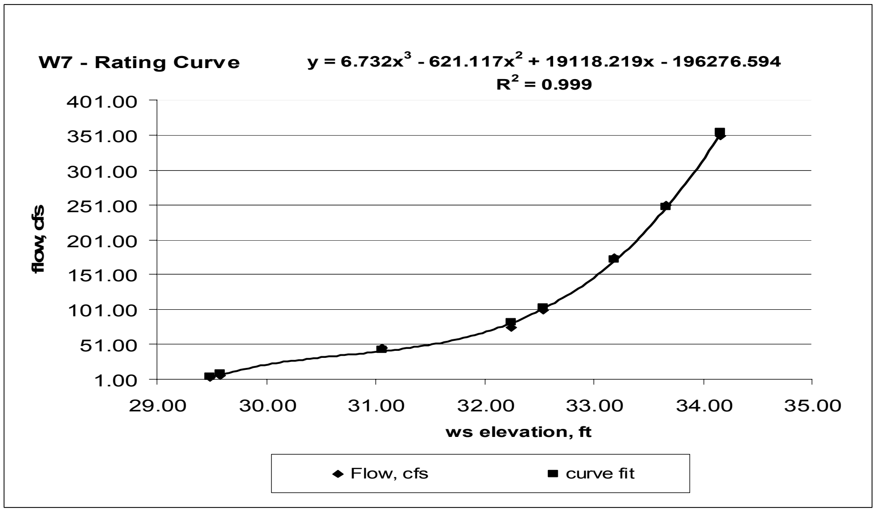


Figure 109: Rating curve for station W7

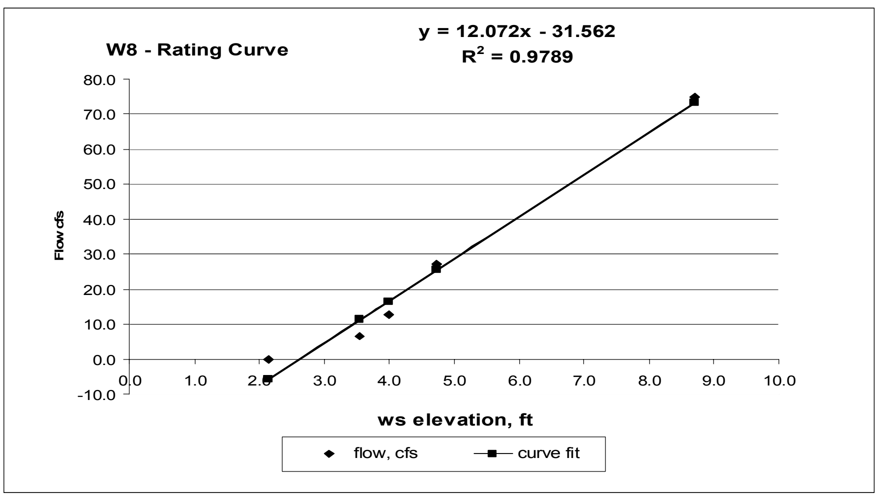


Figure 110: Rating curve for station W8

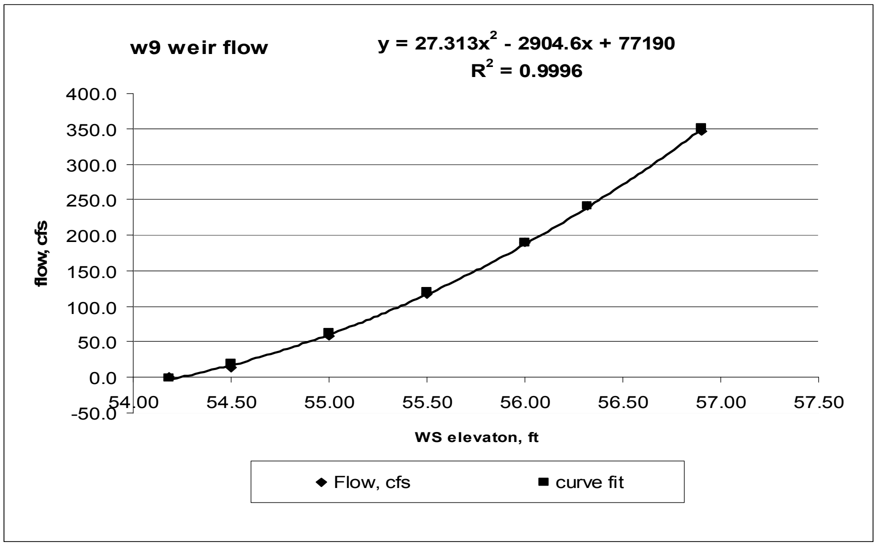


Figure 111: Rating curve for stationW9

4.3.2 Creating Geometry File in GIS environment using HEC-GeoRAS

In this section the procedure is described in detail for creating the geometry file for WPBW in the GIS environment for export into RAS environment. Below are the steps for pre-processing of GIS data to create the geometry file for use in HEC-RAS modeling.

- Open ArcMap and check extensions 3D Analyst and Spatial Analyst from Tools → Extensions. These are the necessary extensions for this procedure.
- Bring in HEC-GeoRAS toolbar from View → Toolbars → HEC-GeoRAS.
- Save the ArcMap document (mxd file) into working folder.
- Two separate data frames are created in ArcMap, one for each flowchart direction, i.e. pre-processing of data for creating RAS geometry and post-processing of RAS results for the flood inundation map.
- Create new dataframe using ApUtilities → Add New Map.
- Name this data frame as WPBW_Geometry.

- Bring in terrain information (elevation data) by adding TIN file (provided by Monmouth County Office of GIS).
- Right click data frame and click on properties to check for coordinate system. (Co-ordinates of TIN file are assigned automatically to data frame, if not already assigned then set it to New Jersey State Plane Coordinate system as most of the data are in this projected coordinate system).
- Create empty layers using path: RAS Geometry → Create RAS Layers → All. (Changed the name of River to River1 and Banks to Banks1, as River and Banks layers are available from Monmouth County). This creates layers for all possible attributes in RAS modeling. Any desired layer can be created and exported. After all the empty layers (14 totals) were created, they were added automatically to the ArcMap. (A message "Complete layers by digitizing features" pops-up).

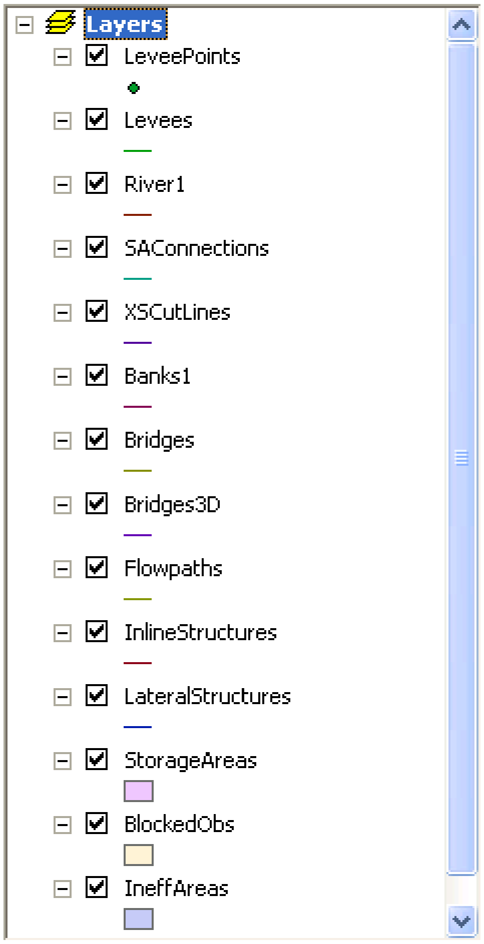


Figure 112: Empty layers created for geometry file

- A database named Wreck Pond.mdb is created automatically in a folder where the ArcMap document Wreck Pond.mxd is saved. This database contains all the 14 layers created above into a personal geodatabase feature dataset: WPBW_Geometry.
- (To add previously created Banks and River layer to this personal geodatabase and name them Banks1 and River1: Save and close ArcMap and open ArcCatalog. Delete empty Banks1 and River1 feature class created in previous steps. Right click on WPBW_Geometry, Import → Feature Class (single). Navigate to location for existing River layer as input features and write River1 as

the output features class. Click OK. This adds River1 into WPBW_Geometry. Repeat the steps for adding Bank1 into WPBW_Geometry.)

- Bring in River1 and Banks1 layer.
- River and Banks were created previously using aerial photos. Banks for online ponds were used as banks.
- Bring in Aerial photos on background to visualize the work. This visual benefit of working in the GIS environment helps prevent/reduce error while creating geometric features of WPBW.
- To digitize the stream centerline, click on Editor → Start Editing and choose location for Wreck Pond.mdb personal geodatabase. After digitizing the reaches, the River Code/Reach Code is assigned using a button on HEC-GeoRAS toolbar.
- Wreck Pond Brook has three tributaries. Alberts Pond Creek, Hannabrand Brook and Black Creek.

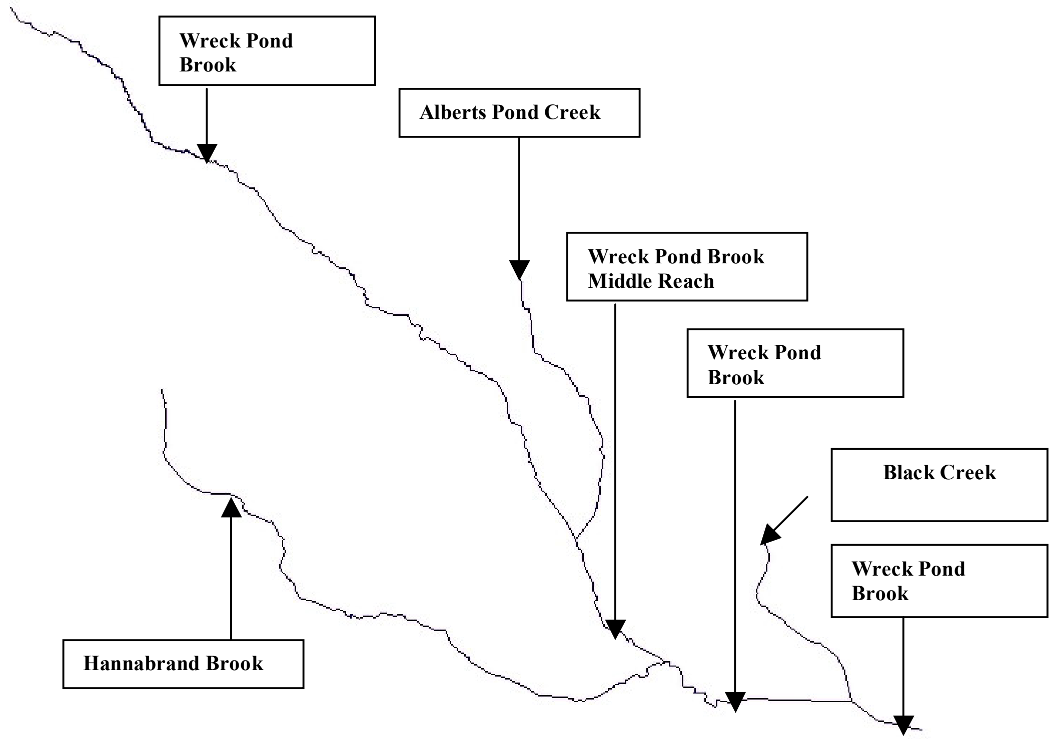


Figure 113: Wreck Pond Brook and its tributaries (River and Reach names)

The River and Reach names are assigned as follows:

Table 42: River and Reach names for WPBW

River	**Reach**
Wreck Pond Brook	Upper Reach
Alberts Pond Creek	Tributary
Wreck Pond Brook	Middle Reach
Hannabrand Brook	Tributary
Wreck Pond Brook	Pond Reach
Black Creek	Tributary
Wreck Pond Brook	Outlet Reach

To populate the fields in River1 attribute table, two steps are required.

- Click on RAS Geometry → Stream Centerline Attributes → Topology (Use River1 as Stream centerline and TIN file provided by the Monmouth County Office of GIS, as Terrain TIN). This step populates FromNode and ToNode attributes.
- Click on RAS Geometry → Stream Centerline Attributes → Lengths/Stations.

Attributes of River1

OBJECTID *	Shape *	HydroID	Reach	River	FromNode	ToNode	ArcLength	FromSta	ToSta	Shape_Len
1	Polyline	87	Tributary	Alberts Pond	1	2	9050.7988	0	9050.7988	9050.79897
2	Polyline	89	Middle Reach	WreckPond Brook	2	3	5064.9043	8959.2734	14024.179	5064.904168
3	Polyline	90	Pond Reach	WreckPond Brook	3	4	6688.5576	2270.7163	8959.2734	6688.558137
4	Polyline	91	Outlet Reach	WreckPond Brook	4	5	2270.7163	0	2270.7163	2270.716214
5	Polyline	92	Tributary	Black Creek	6	4	6358.3711	0	6358.3711	6358.371131
6	Polyline	94	Tributary	Hannabrand Brook	7	3	21070.619	0	21070.619	21070.61839
7	Polyline	191	Upper Reach	WreckPond Brook	8	2	26161.783	14024.179	40185.961	26161.78297

Record: 1 Show: All Selected Records (0 out of 7 Selected) Options

Table 43: Attribute table of layer 'River1'

In the above attribute table, the value of field 'FromSta' is the river station at the downstream end of the Reach and 'ToSta' is the river station at the upstream end of the reach. The numbers assigned are useful in checking if the junctions are formed properly. If junctions are not formed at the confluence of the three reaches, then the river network is not created properly and should be fixed before proceeding further. In case of WPBW, merging of the reaches forms all junctions. Hence, RAS computes the flow in the downstream reach after the merge and computes the corresponding stage and assigns this stage to the upstream reaches. The detailed discussion on boundary conditions for reach connections can be found in the RAS reference manual (Version 4.0, March 2008).

River banks were created using aerial photos on background and digitizing the left bank first followed by the right bank, starting from the upstream end to the downstream end. Banks of ponds were digitized as river banks for online ponds as shown in the figure below.

Figure 114: Banks delineation for online ponds

Flowpaths are used to define the hydraulic flow paths of the stream in the left and right overbanks and the main channel. They also represent the path for measuring the reach lengths between cross-sections. Reach lengths measure the distance between cross sections and are used in energy loss calculations. While digitizing the flowpaths, it is important that the flow path line points downstream and is continuous for each river. The stream centerline is used to define the flowpath centerline, as they essentially are the same. Flowpaths indicate the center of mass of flow in the left and right overbanks and also in the main channel. The center of mass of the flow could be significantly different between low flow and high flow events. Hence, different flow path lines may need to be delineated for different flow events.

The following figure 115 from the HEC-RAS reference manual shows flowpaths as dashed lines.

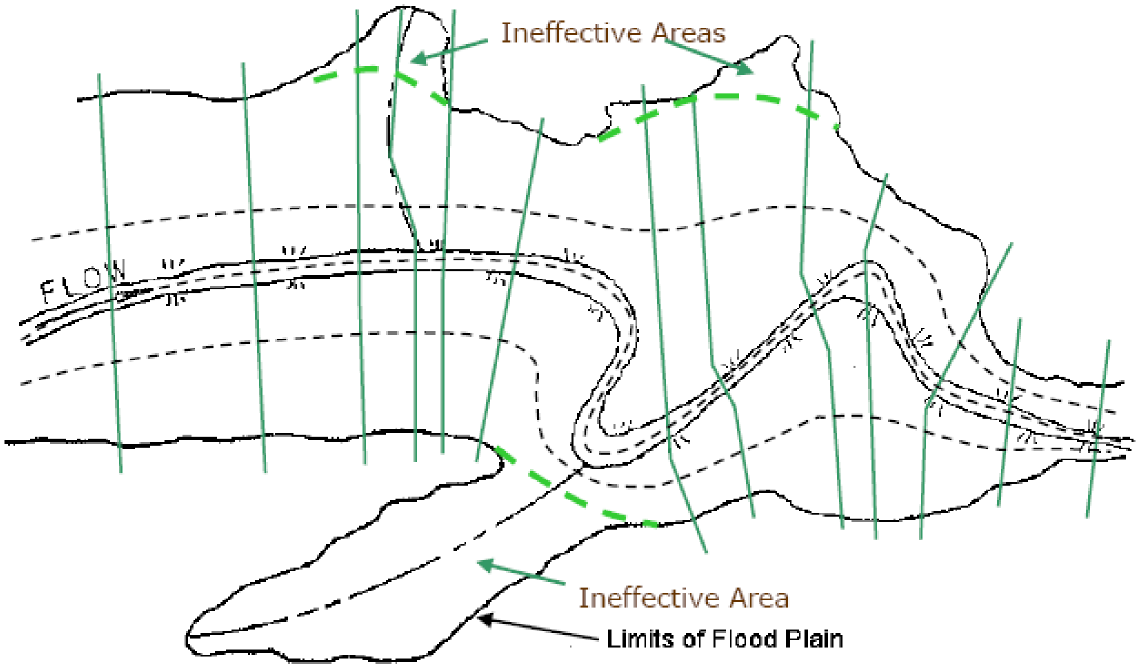
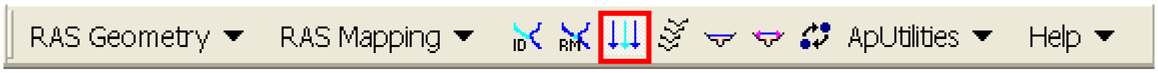


Figure 115: Flowpaths and ineffective area concept (RAS reference manual, 2008)

An ineffective flow area is an area where the water will pond but the velocity is essentially zero. These areas do not actively convey water and are excluded from flow area and wetter perimeter calculations. The next step is to label the flowpaths by clicking the *Select Flowpath and Assign LineType Attribute* button as highlighted below and then click on the flowpath feature, for example channel as shown in the figure below.



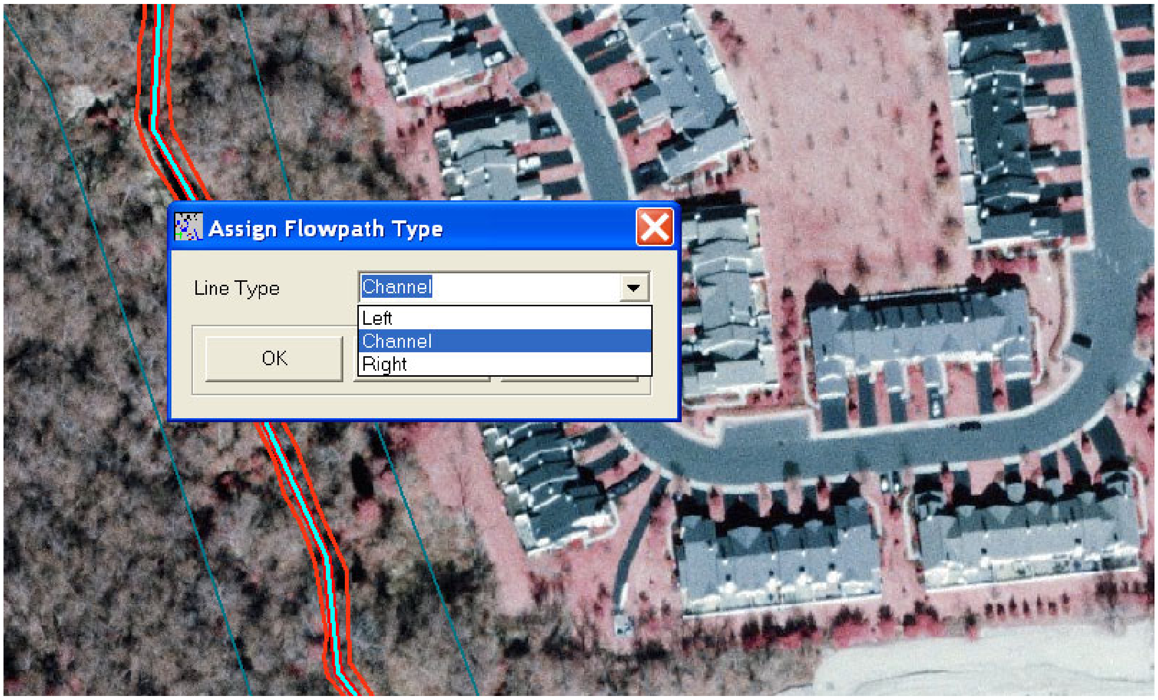


Figure 116: Assigning flowpath type to digitized line feature

Check the attribute table for Flowpaths feature and make sure LineType field has an entry for all rows.

Attributes of Flowpaths

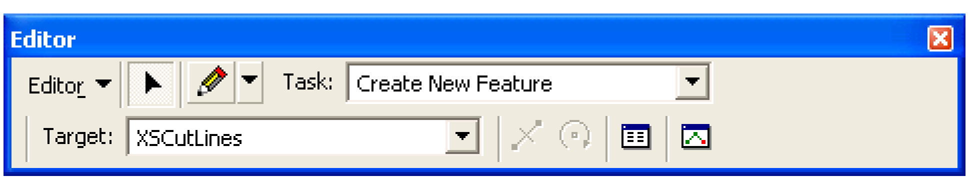
	Shape *	OID *	Shape_Length	LineType
▸	Polyline	3	37295.782389	Left
	Polyline	4	36984.107731	Right
	Polyline	5	6307.359281	Left
	Polyline	6	6290.080102	Right
	Polyline	7	20098.230989	Left
	Polyline	8	20353.156262	Right
	Polyline	9	8909.734892	Left
	Polyline	10	8408.771836	Right
	Polyline	11	9050.79897	Channel
	Polyline	12	26161.782972	Channel
	Polyline	13	21070.618393	Channel
	Polyline	14	5064.904168	Channel
	Polyline	15	6688.558137	Channel
	Polyline	16	2270.716214	Channel
	Polyline	19	6358.371131	Channel

Record: 1 Show: All Selected

Table 44: Flowpaths attribute table

The next task is to create cross-sections for the reaches created in the previous steps. Cross-sections characterize the flow carrying capacity of the channel and the floodplain. Cross-sections should be perpendicular to the flow in the main channel as well as the overbank areas. XSCutlines should be drawn from the left overbank to the right overbank facing downstream. XSCutlines can intersect the main channel only once.

- Click on Editor → Start Editing and select personal geodatabase where empty XSCutlines feature is created. Select *Create New Feature* as Task and *XSCutLines* as Target as shown below.



HEC-GeoRAS allows automated creation of the stream cross-section. The automated procedure does not follow general one-dimensional modeling guidelines. Figure 117 shows the automated X-sections created at a 500 ft distance and a 1500 ft width. It can be seen from the figure that several X-sections intersect each other, which is not acceptable.

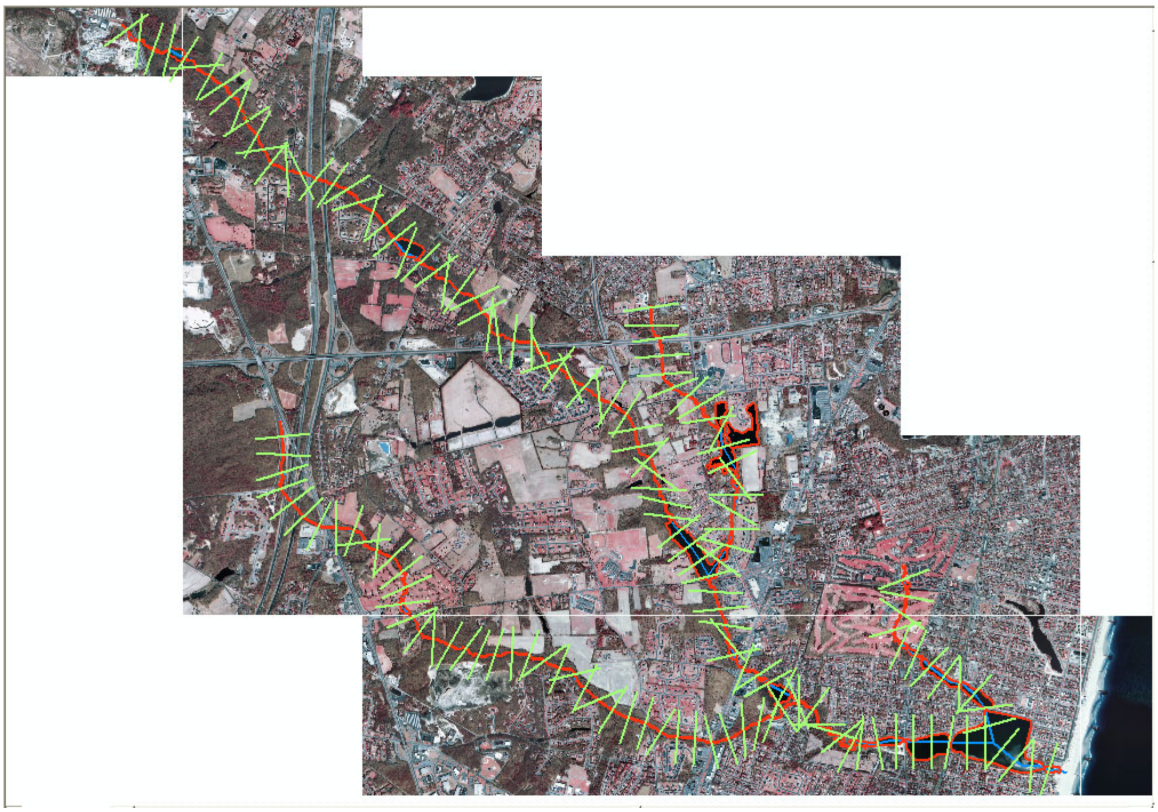
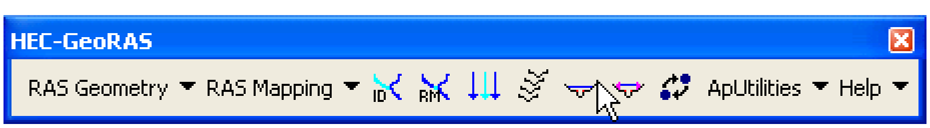


Figure 117: Automated cross section creation

Hence, manual digitization is utilized. While digitizing Cross-sections, the output can be plotted and checked manually for needed edits. The following HEC-GeoRAS tool can be used for checking a cross section:



Also, while digitizing it is important to digitize one cross-section upstream and downstream near roads where bridge/culvert information will be entered later. Below is the figure of X-sections digitized for streams in the Wreck Pond Watershed.

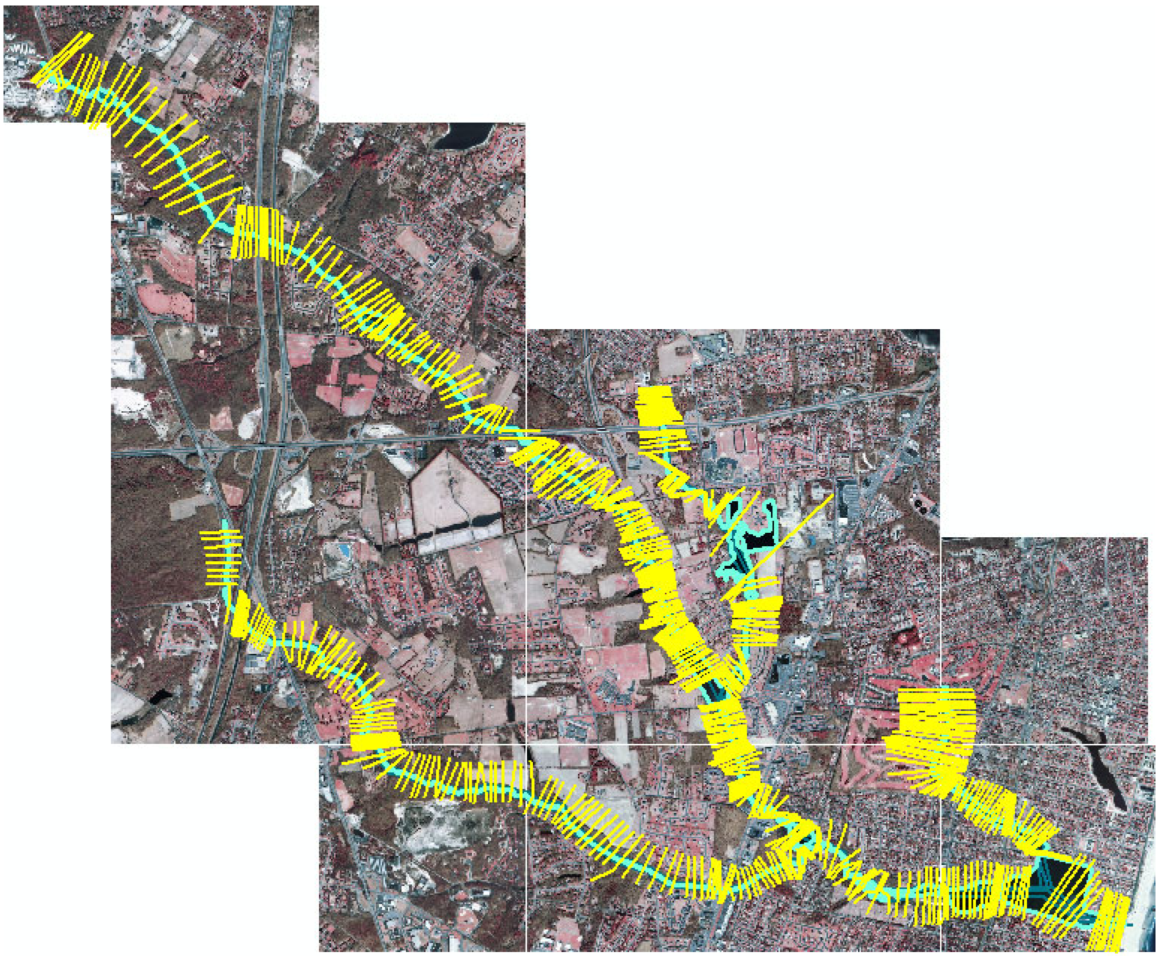


Figure 118: Cross sections created manually for WPBW

There are several important fields in the attribute table of the above created cross sections that need input values. This is done as: Click on RAS Geometry → XS Cut Line Attributes → All. Select the fields as shown below.

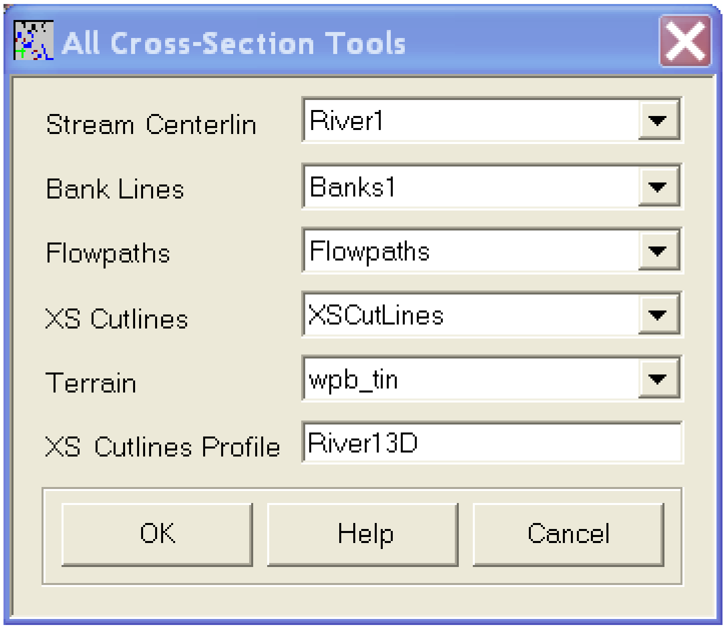
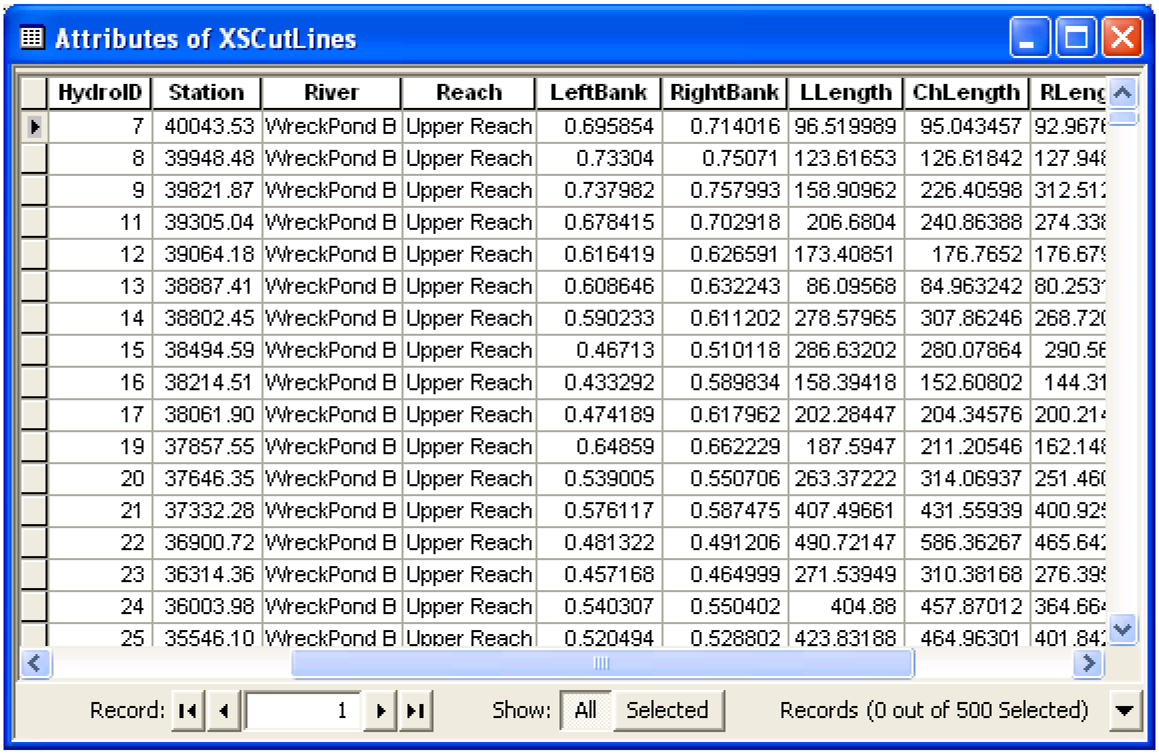


Figure 119: Extracting information for XS cutlines

This will extract the required information in XS cutlines for the attribute table as shown below.

Table 45: Cross section cutlines attribute table



Attributes of XSCutLines

HydroID	Station	River	Reach	LeftBank	RightBank	LLength	ChLength	RLeng
7	40043.53	WreckPond B	Upper Reach	0.695854	0.714016	96.519989	95.043457	92.967
8	39948.48	WreckPond B	Upper Reach	0.73304	0.75071	123.61653	126.61842	127.94
9	39821.87	WreckPond B	Upper Reach	0.737982	0.757993	158.90962	226.40598	312.51
11	39305.04	WreckPond B	Upper Reach	0.678415	0.702918	206.6804	240.86388	274.33
12	39064.18	WreckPond B	Upper Reach	0.616419	0.626591	173.40851	176.7652	176.67
13	38887.41	WreckPond B	Upper Reach	0.608646	0.632243	86.09568	84.963242	80.253
14	38802.45	WreckPond B	Upper Reach	0.590233	0.611202	278.57965	307.86246	268.72
15	38494.59	WreckPond B	Upper Reach	0.46713	0.510118	286.63202	280.07864	290.5
16	38214.51	WreckPond B	Upper Reach	0.433292	0.589834	158.39418	152.60802	144.31
17	38061.90	WreckPond B	Upper Reach	0.474189	0.617962	202.28447	204.34576	200.21
19	37857.55	WreckPond B	Upper Reach	0.64859	0.662229	187.5947	211.20546	162.14
20	37646.35	WreckPond B	Upper Reach	0.539005	0.550706	263.37222	314.06937	251.46
21	37332.28	WreckPond B	Upper Reach	0.576117	0.587475	407.49661	431.55939	400.92
22	36900.72	WreckPond B	Upper Reach	0.481322	0.491206	490.72147	586.36267	465.64
23	36314.36	WreckPond B	Upper Reach	0.457168	0.464999	271.53949	310.38168	276.39
24	36003.98	WreckPond B	Upper Reach	0.540307	0.550402	404.88	457.87012	364.66
25	35546.10	WreckPond B	Upper Reach	0.520494	0.528802	423.83188	464.96301	401.84

Record: 1 Show: All Selected Records (0 out of 500 Selected)

The channel information was available for short lengths where stream gages were installed. Hence, the cross-sections were created using the TIN with detail channel information and were compared with cross-sections created using TIN without detailed channel information. The following figures 120 and 121 shows the resulting difference between the two methods. More cross-sections are shown in Appendix D. The difference is considered small for high flow events.

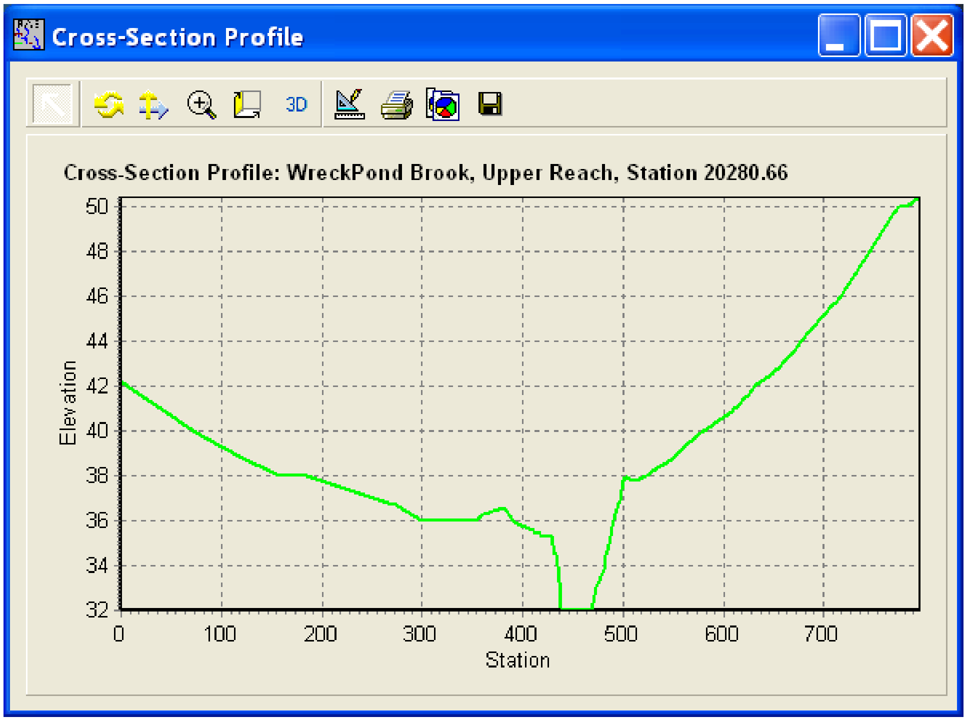


Figure 120: Cross-section at station 20280.66 for WPB using TIN without channel detail

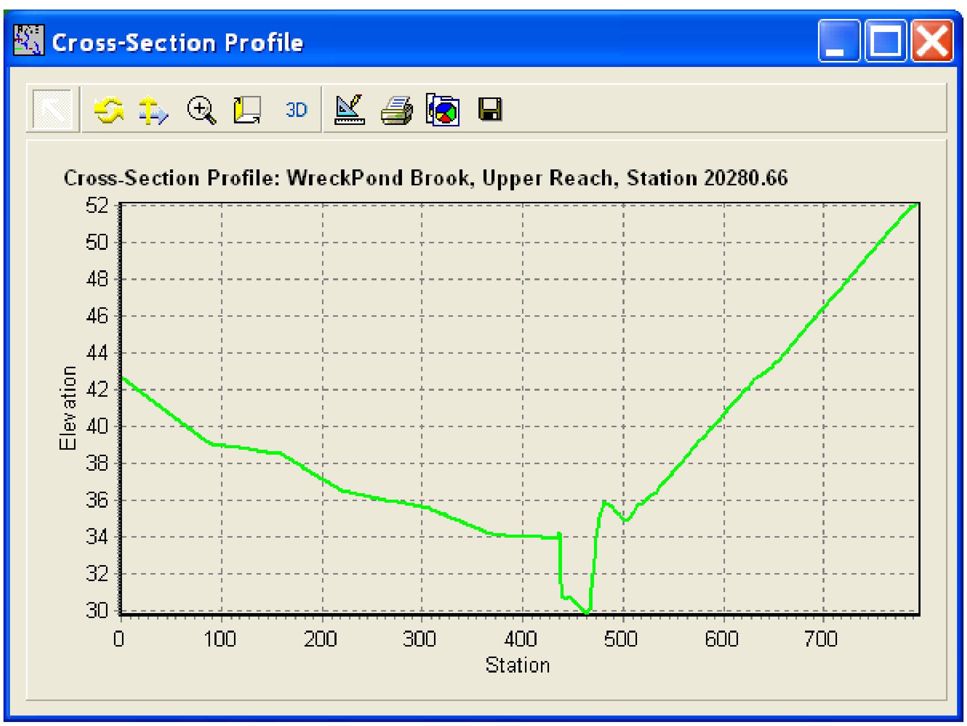


Figure 121: Cross-section at station 20280.66 for WPB using TIN with channel detail

Bridges were digitized after the cross sections were created. The distance to the upstream X-section was calculated from aerial photos. The top width was calculated from aerial photos or surveyed in the field. A combination of both methods was used for WPBW. Additional fields must be populated after the bridges are digitized. This involves the following steps: Click on RAS Geometry → Bridges/Culverts → All. Choose appropriate features as shown below and click OK.

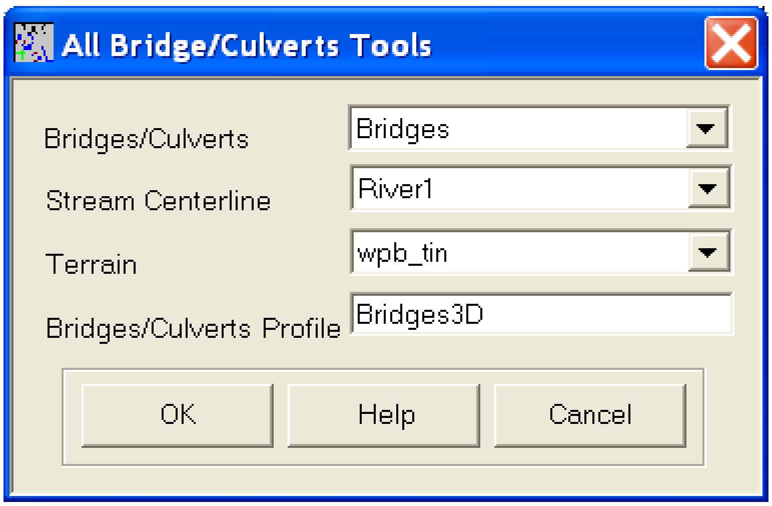


Figure 122: Extracting information for Bridges

This step will populate the remaining fields in the attribute table and is shown below.

Table 46: Attribute table of Bridges

Attributes of Bridges

River	Reach	Station	USDistance	TopWidth	NodeName
WreckPond Brook	Upper Reach	39532.789	15	85	Hwy 34 over WPB
WreckPond Brook	Upper Reach	32843.422	16.5	35	Martins Rd over WPB
WreckPond Brook	Upper Reach	32217.326	26	86	GSP South over WPB
WreckPond Brook	Upper Reach	31767.264	13	86	GSP North over WPB
WreckPond Brook	Upper Reach	28078.395	19	30	Hurleys Pond Rd over W
WreckPond Brook	Upper Reach	23283.205	29	130	Hwy 138 over WPB
WreckPond Brook	Upper Reach	20256.592	10.5	24	Glendola Rd over WPB
WreckPond Brook	Upper Reach	19697.752	35	44	18th Ave over WPB
WreckPond Brook	Upper Reach	17661.514	11.5	18	Bentz Rd over WPB
WreckPond Brook	Middle Reach	13885.325	11.5	32	Allaire Rd over WPB
WreckPond Brook	Middle Reach	11078.511	10.5	54	Hwy 35 over WPB
WreckPond Brook	Middle Reach	10472.655	8.5	47	Ocean Rd over WPB
WreckPond Brook	Middle Reach	9134.252	9.5	47	Oldmill Rd over WPB

Record: 0 Show: All Selected Records (0 out of 33

The next key step is to create the Manning's 'n' value table. The 'n' values were assigned by using the feature class having an 'n' value corresponding to a discrete land use. The Manning's_n_LU feature class was created for this purpose and is shown in table 47. The Manning's 'n' values represent the roughness classifications for the channel and overbank areas, and are used to calculating frictional energy losses between cross sections.

Table 47: Manning's 'n' value table

Attributes of Mannings_n_LU

LU06	TYPE06	ACRES	Shape_Leng	N_Value	Shape_Length	Shape_Ar
4120	FOREST	0.666435	1121.450234	0.15	1121.450078	29029.9
4322	FOREST	0.850249	1406.478217	0.15	661.645541	14934.9
1140	URBAN	0.955714	1978.645925	0.05	1978.646091	41630.8
1200	URBAN	3.96136	2293.811412	0.05	2344.969774	167035.0
4440	FOREST	3.4503	3171.759805	0.15	969.547647	18338.4
6231	WETLANDS	7.67087	4442.893684	0.1	1957.063964	39087.
1140	URBAN	1.2025	963.171752	0.05	963.171589	52380.7
4120	FOREST	2.82371	1645.760499	0.15	1645.76045	123000.8
1140	URBAN	5.29216	2636.251172	0.05	2636.250675	230526.3
2100	AGRICULTURE	4.50319	2016.102748	0.05	2016.102751	196158.
4312	FOREST	112.27	35042.259552	0.15	5605.680132	417022.6
4120	FOREST	56.0243	13922.742555	0.15	8157.475239	693456.9
4220	FOREST	5.18618	2590.273225	0.15	384.223303	2380.5

Record: 0 Show: All Selected Records (0 out of 1470

The landuse feature must have a field for the land use description and corresponding Manning's n value, which is TYPE06 and N_Value, respectively, in the table above. The table is created by the following procedure: Click RAS Geometry → Manning's n Values → Extract n Values, to assign Manning's n value. The fields were selected as shown below.

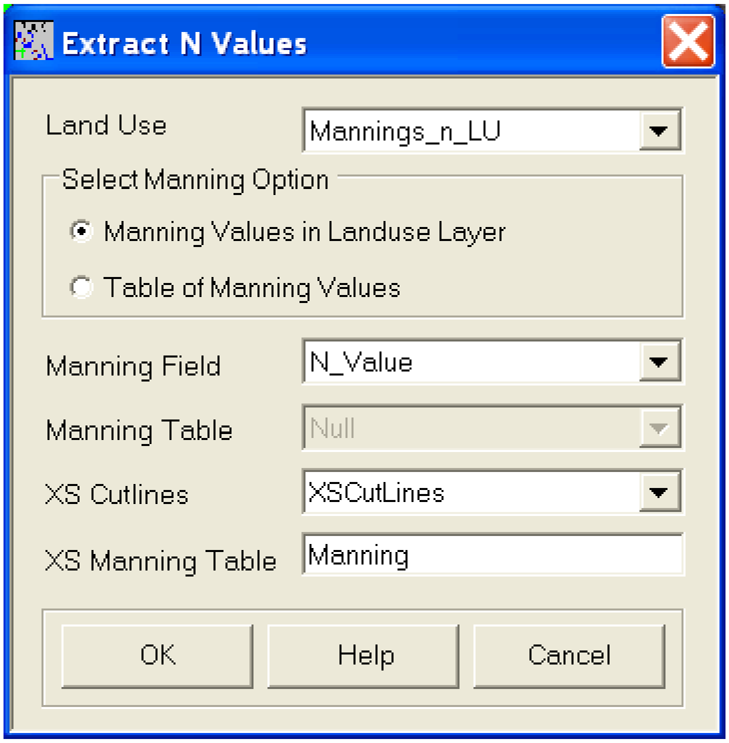
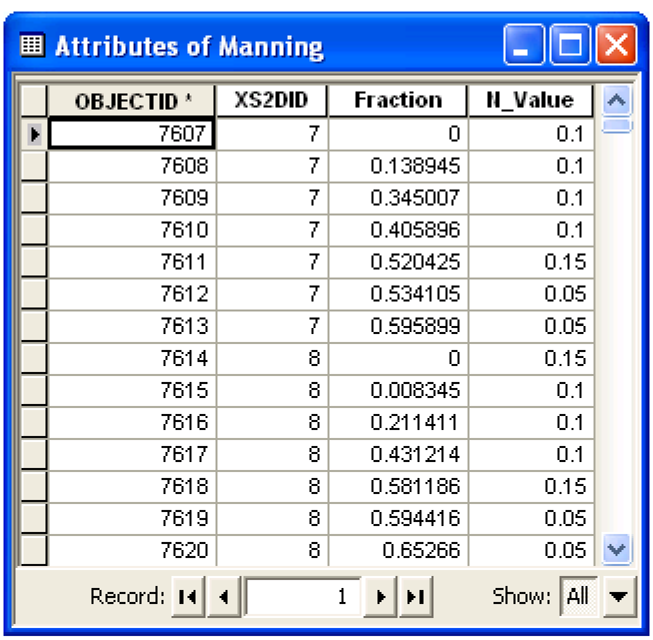


Figure 123: Extracting Manning's 'n' value

This task generates a XS Manning Table and assigns Manning's n value to the X-sections.

Table 48: Manning's 'n' value assigned to channel and floodplain

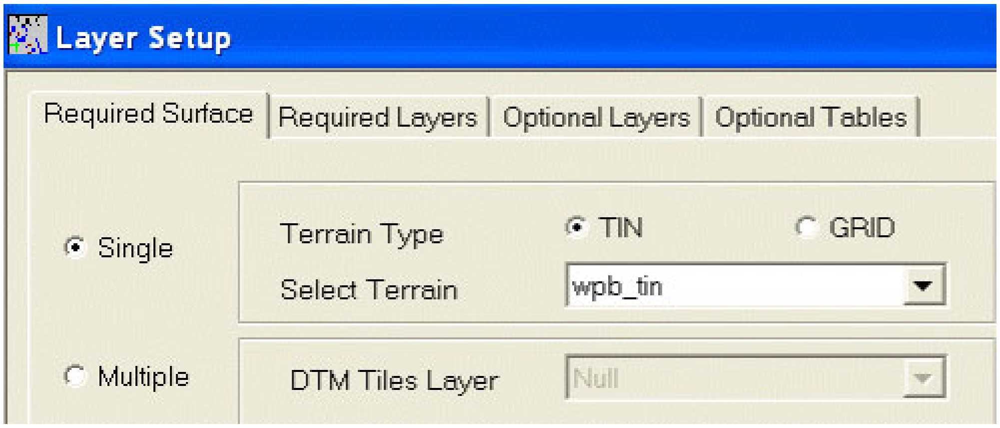


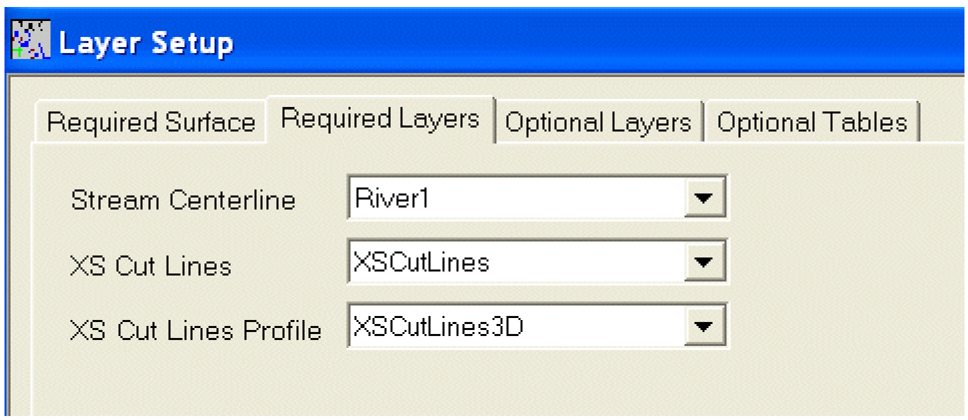
Attributes of Manning

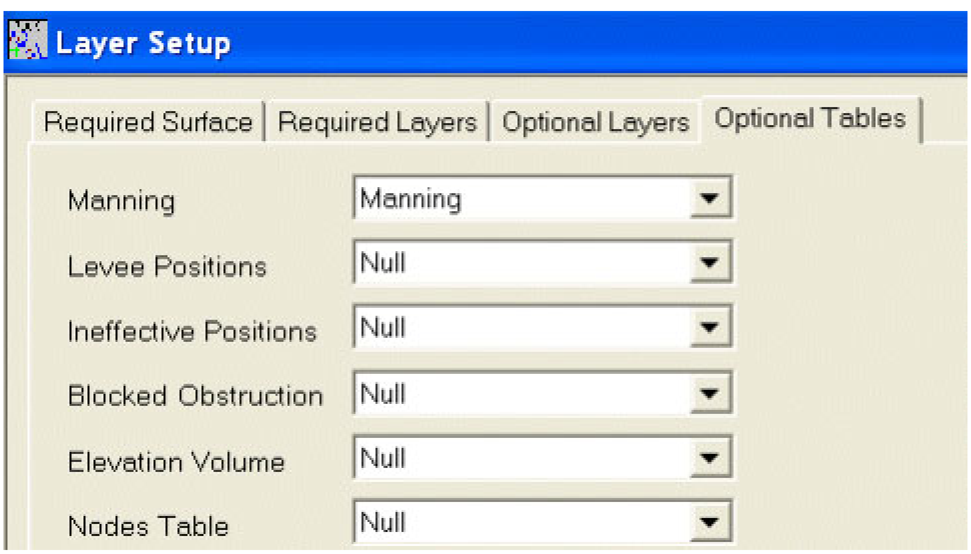
OBJECTID *	XS2DID	Fraction	N_Value
7607	7	0	0.1
7608	7	0.138945	0.1
7609	7	0.345007	0.1
7610	7	0.405896	0.1
7611	7	0.520425	0.15
7612	7	0.534105	0.05
7613	7	0.595899	0.05
7614	8	0	0.15
7615	8	0.008345	0.1
7616	8	0.211411	0.1
7617	8	0.431214	0.1
7618	8	0.581186	0.15
7619	8	0.594416	0.05
7620	8	0.65266	0.05

Record: 1 Show: All

Before exporting the data to HEC-RAS, a final check was performed by clicking on RAS Geometry → Layer Setup and following layers were verified in each tab.







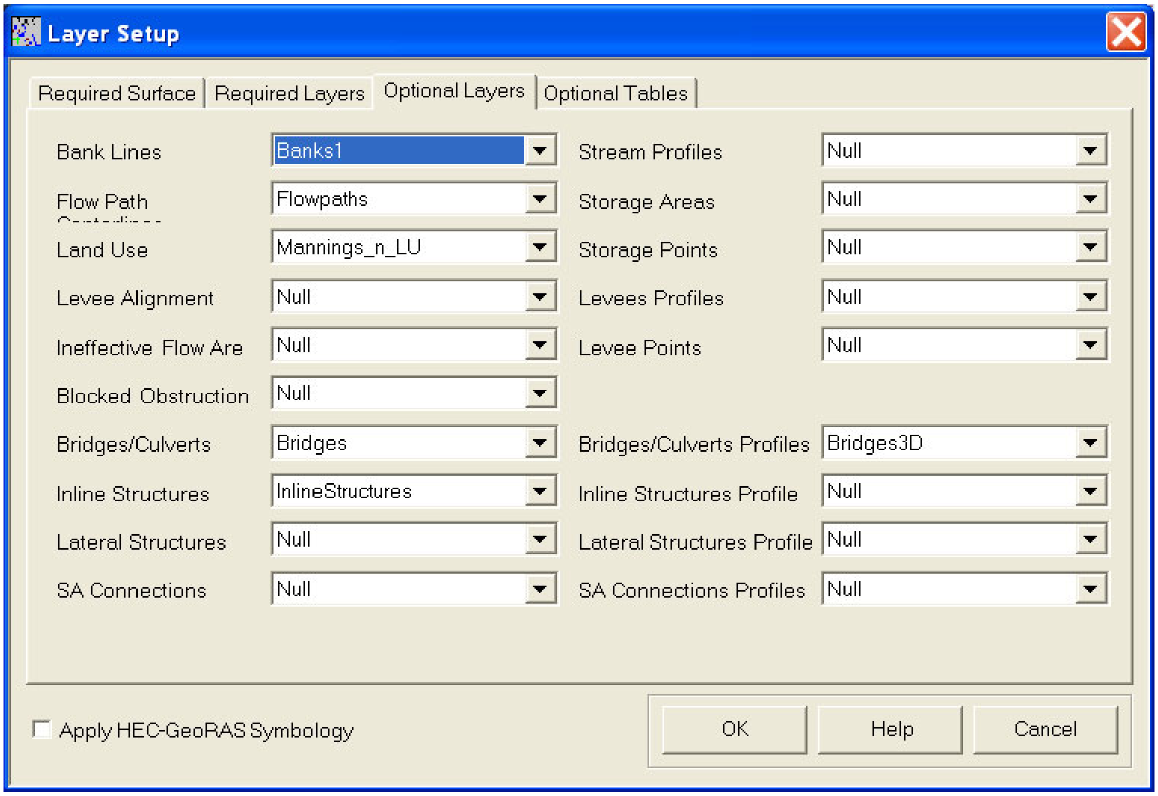


Figure 124: Layer verification prior to geometry file export

The last step was to export the GIS data to HEC-RAS for further modeling. This was done by clicking RAS Geometry → Extract GIS Data. The name and location were selected for the export file and then the "click OK" was executed as shown below. This step created two files: GIS2RAS.xml and GIS2RASImport.sdf.

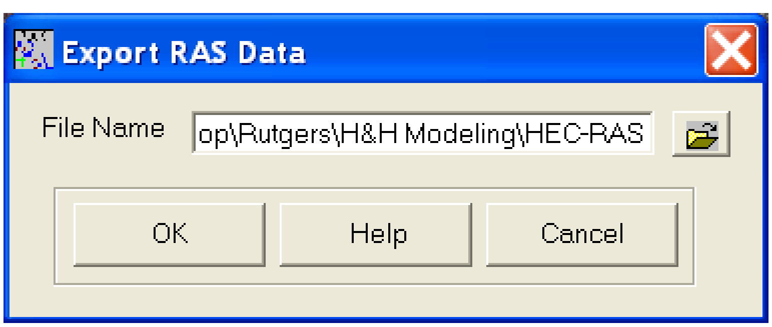


Figure 125: Exporting RAS data to HEC-RAS

4.3.3 Importing geometric data to HEC-RAS

HEC-RAS was used to process the WPBW parameters by the following steps: Open the HEC-RAS model → Create new project →click on Edit/Enter geometric data. In the Geometric Data window, click on File → Import Geometric Data → GIS Format. Navigate to GIS2RAS.RASImport.sdf file created using HEC-GeoRAS and click OK. This will bring geometric data created in the GIS environment to the RAS environment as shown below.

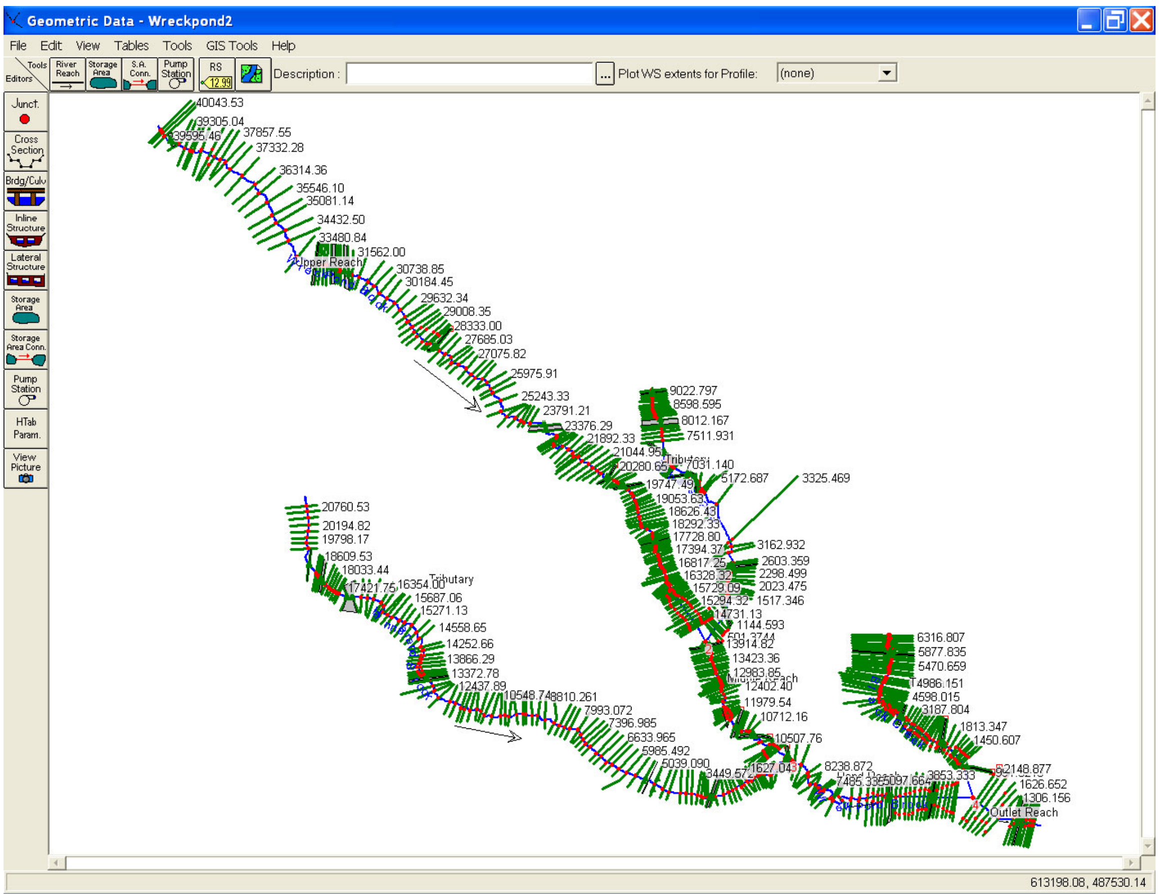


Figure 126: Imported geometric data into HEC-RAS

4.3.4 Preparing/Refining imported geometric data

BRIDGES/CULVERTS

The RAS reference manual provides values of Manning's 'n' value for closed conduits flowing partly full. The Manning's n value for culverts was chosen as 0.011 as most of the culverts surveyed were concrete culverts, straight and free from debris

Table 49: Manning's 'n' value for concrete culverts

Concrete	**Manning's 'n' value**		
Culvert, straight and free of debris	0.010	0.011	0.013
Culvert with bends, connections, and some debris	0.011	0.013	0.014
Finished	0.011	0.012	0.014
Sewer with manholes, inlet, etc., straight	0.013	0.015	0.017
Unfinished, steel form	0.012	0.013	0.014
Unfinished, smooth wood form	0.012	0.014	0.016
Unfinished, rough wood form	0.015	0.017	0.020

CROSS-SECTIONS

Sometimes it was necessary to edit manually the X-sections imported from the GIS environment to the RAS environment. One such example is shown below. In this case the cross-section was digitized very close to a bridge structure and therefore, failed to pick the channel information from the terrain model. Hence, care should be exercised when digitizing cross sections near structures such as culverts and bridges. The following is the figure of an X-section generated using GIS.

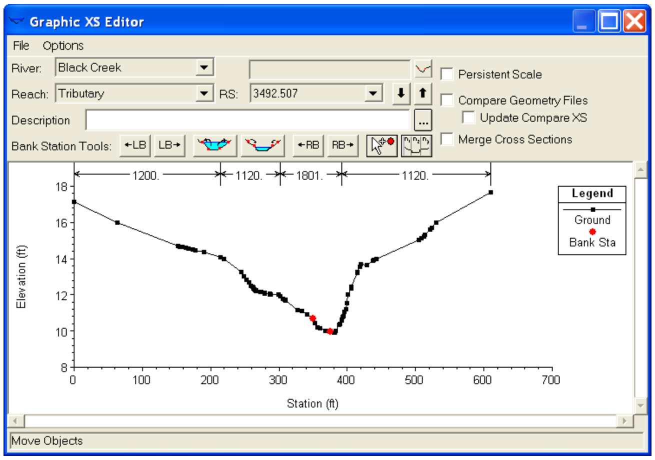


Figure 127: Cross section digitized very close to culvert

The bottom elevation was extracted as 10 ft from the terrain information. However, it actually was at 4 ft. This error can be corrected easily using the Graphical XS Editor in HEC-RAS as shown in figure below.

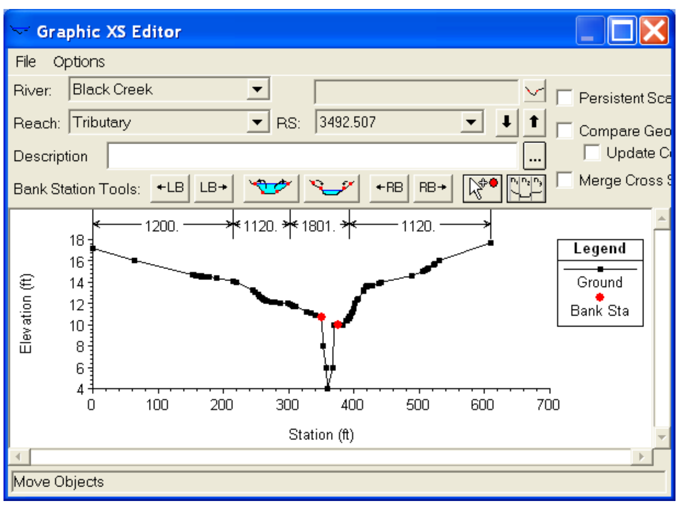


Figure 128: Cross section refined using Graphical cross section Editor in HEC-RAS

MANNING'S 'N':

The Manning's 'n' values were extracted from tabulated values which were calculated using landuse data. However, landuse polygons at most of the locations had combined stream and riparian areas as woods. Consequently, several X-sections generated using the GIS attributed ‘n’ values for woods to channel areas resulted in higher ‘n’ values for the channel.

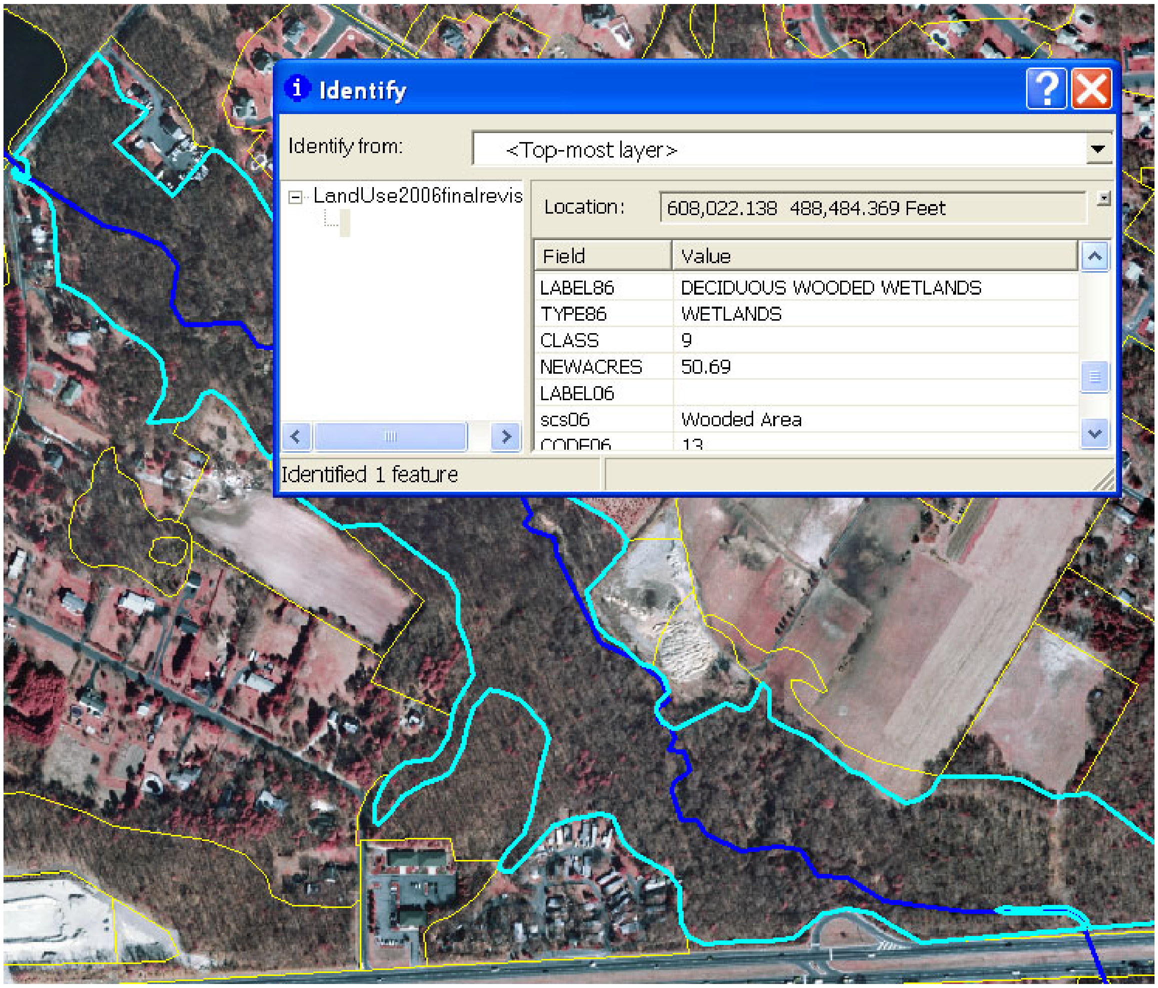


Figure 129: Landuse polygon selected showing deciduous woods attributed to channel and riparian areas

In the figure above it can be seen the selected landuse polygon (cyan polygon) was assigned a single attribute as ‘Deciduous Wooded Wetland’ for the channel and riparian areas. When the Manning’s ‘n’ value table is created it assigns this polygon a single ‘n’ value of 0.1. As a result, when the X-section passes through this polygon and the Manning’s ‘n’ value is extracted and is assigned an ‘n’ value of 0.1 for the channel and overbanks areas instead of an 'n' value of 0.035 for the channel and 0.1 for overbank areas.

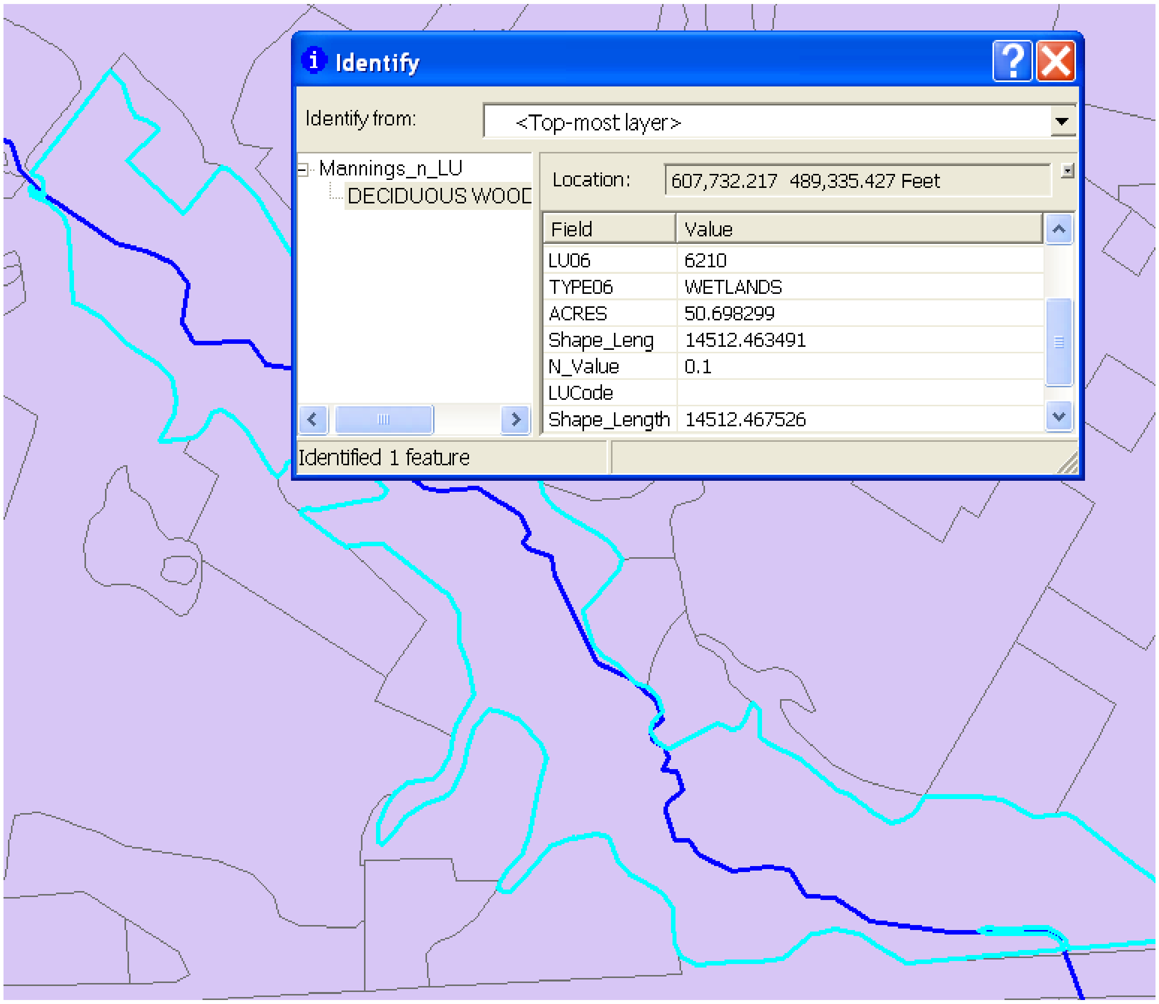
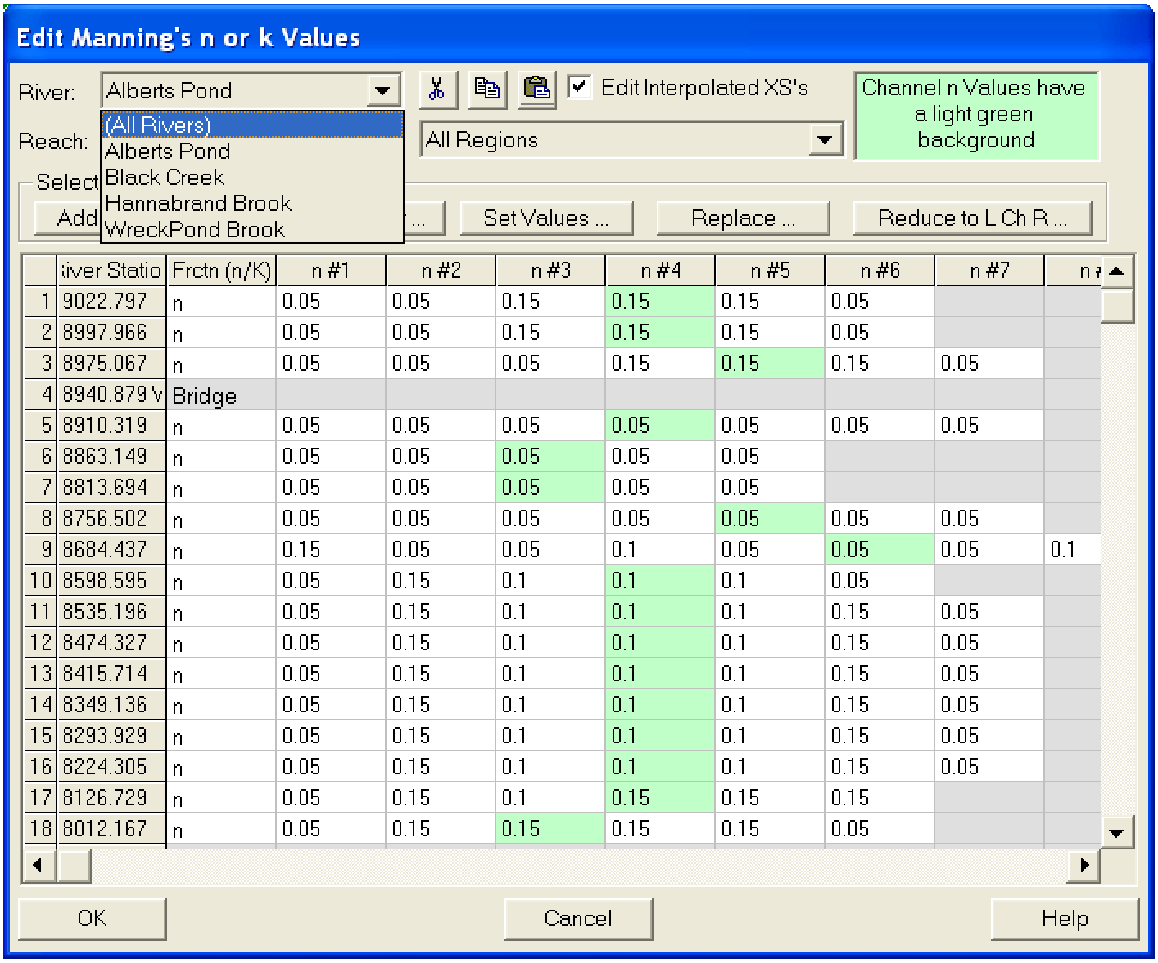


Figure 130: Manning's 'n' issue

The Manning's 'n' value selection for such polygon is corrected in the HEC-RAS environment by the following procedure: In the geometry window, click on → Tables → Manning's n or k values. As shown in the figure below, several 'n' values were assigned to the channel portion of the X-sections.

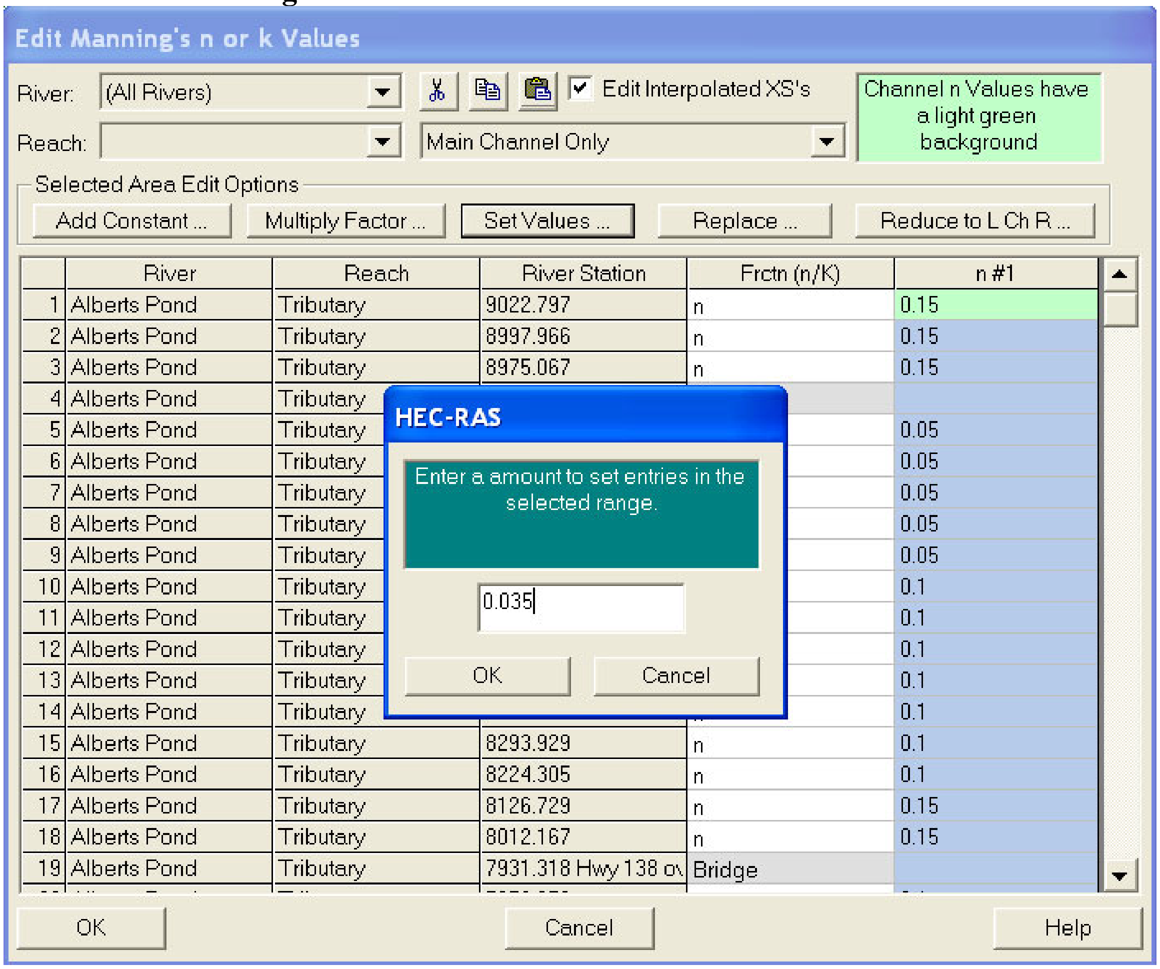
Table 50: Manning's 'n' table in RAS with incorrect channel 'n' values



	iver Statio	Frctn (n/K)	n #1	n #2	n #3	n #4	n #5	n #6	n #7	n #
1	9022.797	n	0.05	0.05	0.15	0.15	0.15	0.05		
2	8997.966	n	0.05	0.05	0.15	0.15	0.15	0.05		
3	8975.067	n	0.05	0.05	0.05	0.15	0.15	0.15	0.05	
4	8940.879 V	Bridge								
5	8910.319	n	0.05	0.05	0.05	0.05	0.05	0.05	0.05	
6	8863.149	n	0.05	0.05	0.05	0.05	0.05			
7	8813.694	n	0.05	0.05	0.05	0.05	0.05			
8	8756.502	n	0.05	0.05	0.05	0.05	0.05	0.05	0.05	
9	8684.437	n	0.15	0.05	0.05	0.1	0.05	0.05	0.05	0.1
10	8598.595	n	0.05	0.15	0.1	0.1	0.1	0.05		
11	8535.196	n	0.05	0.15	0.1	0.1	0.1	0.15	0.05	
12	8474.327	n	0.05	0.15	0.1	0.1	0.1	0.15	0.05	
13	8415.714	n	0.05	0.15	0.1	0.1	0.1	0.15	0.05	
14	8349.136	n	0.05	0.15	0.1	0.1	0.1	0.15	0.05	
15	8293.929	n	0.05	0.15	0.1	0.1	0.1	0.15	0.05	
16	8224.305	n	0.05	0.15	0.1	0.1	0.1	0.15	0.05	
17	8126.729	n	0.05	0.15	0.1	0.15	0.15	0.15		
18	8012.167	n	0.05	0.15	0.15	0.15	0.15	0.05		

Select All Rivers in the table and select 'Main channel only' instead of All Regions. Highlight n#1 column and Click on Set Values and enter 0.035 to replace the channel's 'n' value to 0.035. The corrected channel 'n' value gets entered into Table 51.

Table 51: Correcting channel 'n' values

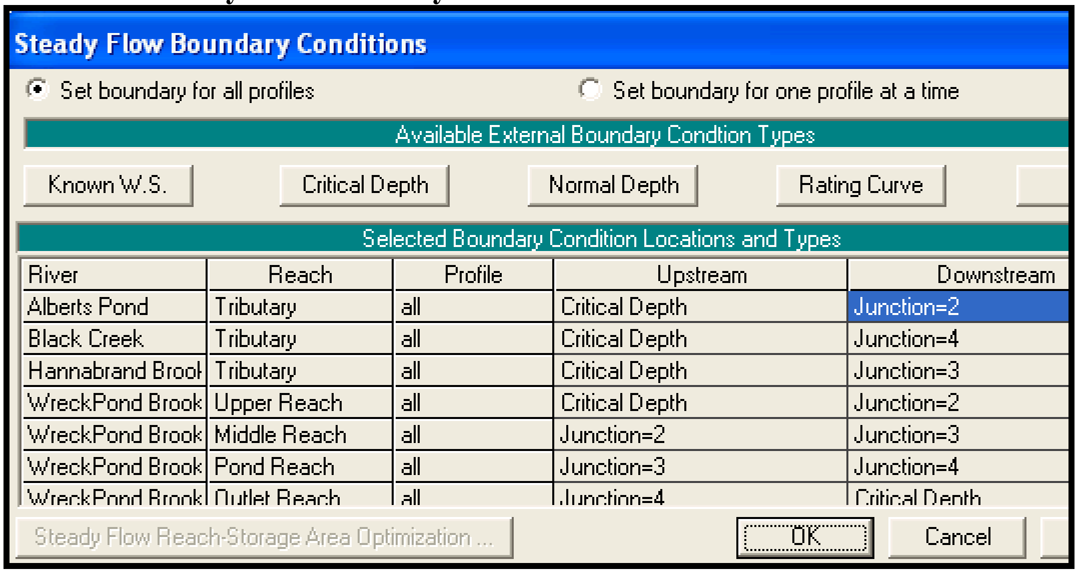


4.3.5 Preparing steady flow data

STEADY FLOW BOUNDARY CONDITIONS

The tributaries and main stream were snapped appropriately in the GIS environment. The resulting junctions are available in the HEC-RAS geometry file when imported in GIS format. Junctions are locations where two or more streams join together or split apart and are specified as boundary conditions. The following boundary conditions were selected for the RAS analysis

Table 52: Steady flow boundary conditions



The flow information for a desired storm event is entered as follows. The flow change locations were input where the flow information was available from the HEC-HMS model. The cross-section was located corresponding to this flow change location in the GIS environment by overlaying the cross-sections over aerial photos.

Table 53: Steady flow data

Steady flow data for WPBW using Delmarva UH and 5% Ia ratio					
	River	Reach	RS	PF 1 (cfs)	PF2 (cfs)
1	Alberts Pond	Tributary	9022.797	438	338
2	Black Creek	Tributary	6316.807	330	168
3	Black Creek	Tributary	5788.683	330	168
4	Black Creek	Tributary	3492.507	501	274
5	Hannabrand Brook	Tributary	20760.53	660	601
6	Hannabrand Brook	Tributary	5144.909	660	601
7	Hannabrand Brook	Tributary	183.3193	654	614
8	Wreck Pond Brook	Upper Reach	40043.53	358	145
9	Wreck Pond Brook	Upper Reach	37857.55	358	145
10	Wreck Pond Brook	Upper Reach	32878.73	569	393
11	Wreck Pond Brook	Upper Reach	28104.53	1009	862
12	Wreck Pond Brook	Upper Reach	20280.65	1256	1151

13	Wreck Pond Brook	Middle Reach	13914.82	1510	1422
14	Wreck Pond Brook	Middle Reach	12650.27	1513	1423
15	Wreck Pond Brook	Middle Reach	9164.05	1497	1401
16	Wreck Pond Brook	Pond Reach	8881.883	2085	2008
17	Wreck Pond Brook	Outlet Reach	2148.877	2098	2011
PF1: NRCS 100 year storm					
PF2: October 2005 storm					

The flow change locations were added by clicking on the button ‘Add A Flow Change Location’ in the “steady flow data” editor window.

STEADY FLOW ANALYSIS:

The steady flow analysis was performed according to the following procedure: Select Run → Steady Flow Analysis in the HEC-RAS window. Specify the geometry file, steady flow file, and plan name. Three options are available for flow regime selection. Select Mixed and click on ‘Compute’.

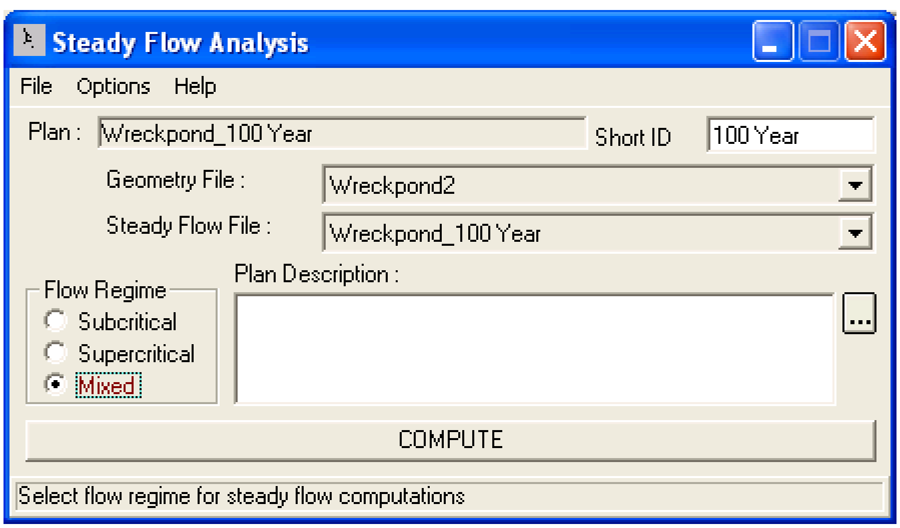


Figure 131: HEC-RAS model ready to run

4.4 Floodplain Delineation using GIS

4.4.1 Delineating Flood inundation area for October 2005 storm

EXPORT HEC-RAS OUTPUT:

Click on File → Export GIS Data. Choose the profile to be exported. Check box for any additional properties to be exported to the GIS environment. Click Export Data. This step creates the SDF file at a specified location.

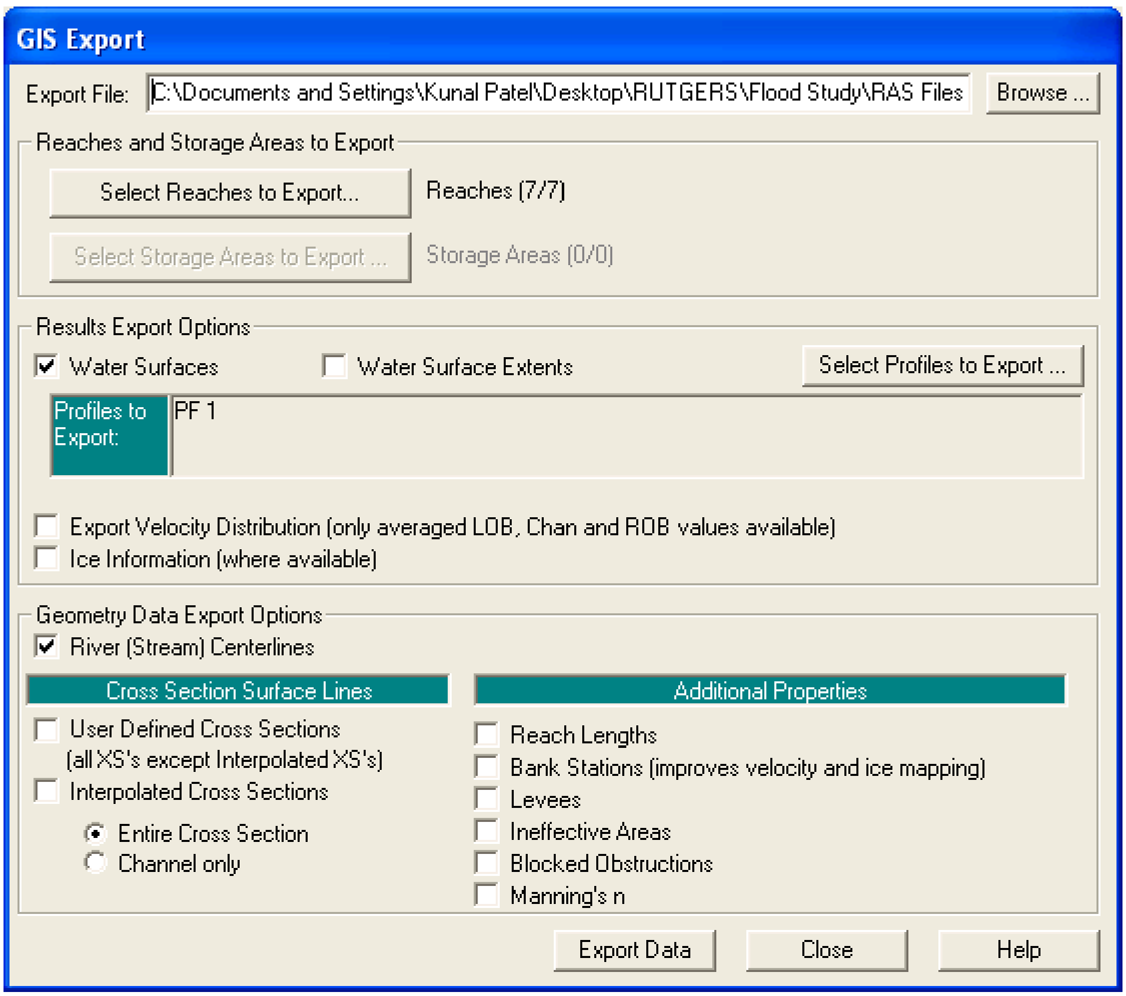
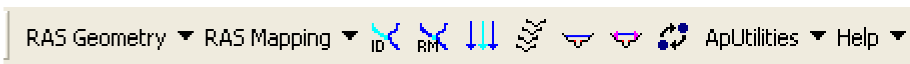


Figure 132: Exporting water surface profiles from RAS to GIS

FLOODPLAIN MAPPING:

Open ArcGIS 9.2 and click on Tools → Customize and check the box for HEC-GeoRAS, then click the Close button. This step adds the HEC-GeoRAS toolbar in the GIS window.



Click on Import RAS SDF file button. . Select the file exported from the RAS environment. This step \converts the SDF file to the XML format. After successful conversion the following message pops-up. Click ok.

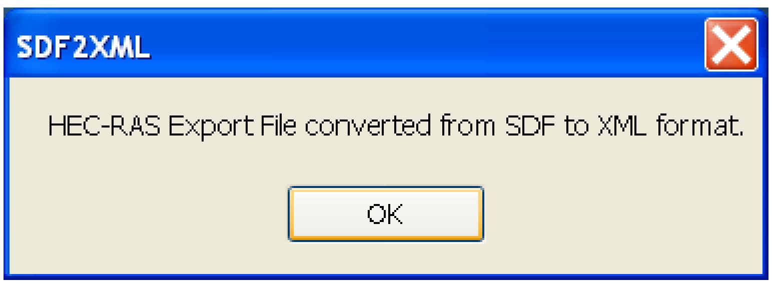


Figure 133: RAS file conversion from SDF to XML

Now click on RAS Mapping → Read RAS GIS Export File.

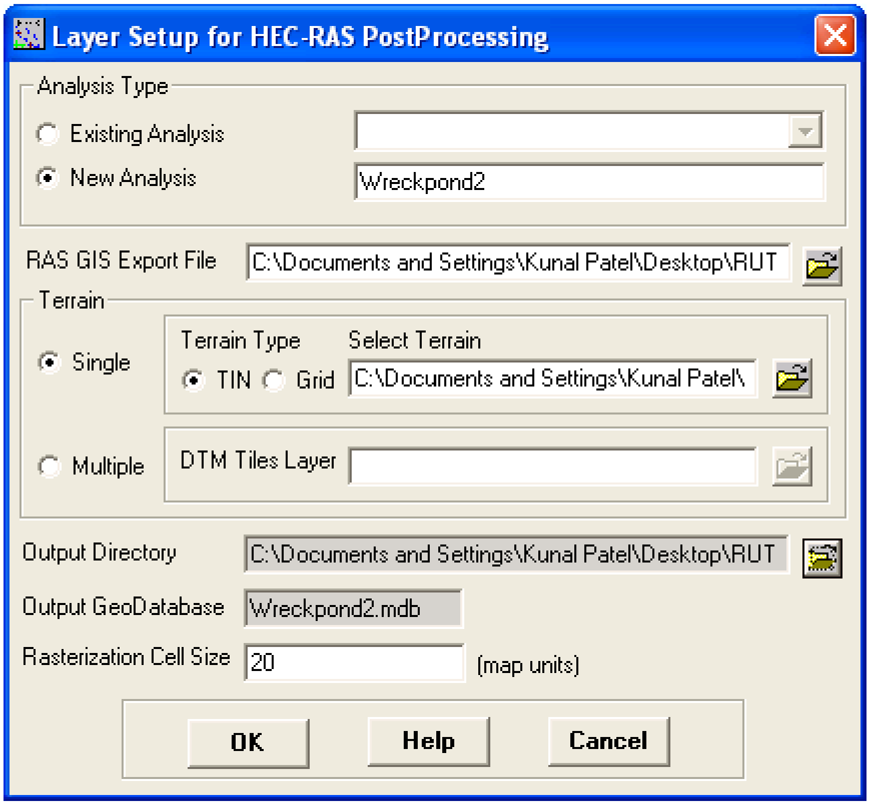


Figure 134: Layer setup for RAS post processing in GIS

Select the New Analysis option and specify the name. Browse to the location for the XML file created above. Also, specify the TIN file to be used for terrain information. Specify the output location and click ok. This creates the geodatabase with the name specified in the New Analysis tab. A DEM also is generated from the specified TIN file and is added to this folder. A new data frame with analysis name is added to ArcMap automatically.

Now click on RAS Mapping → Read RAS GIS Export File. This step creates and adds the Bounding Polygon into ArcMap. The bounding polygon defines the extent of the floodplain mapping by connecting the endpoints of the X-sections. This limits the floodplain to the area modeled in HEC-RAS.

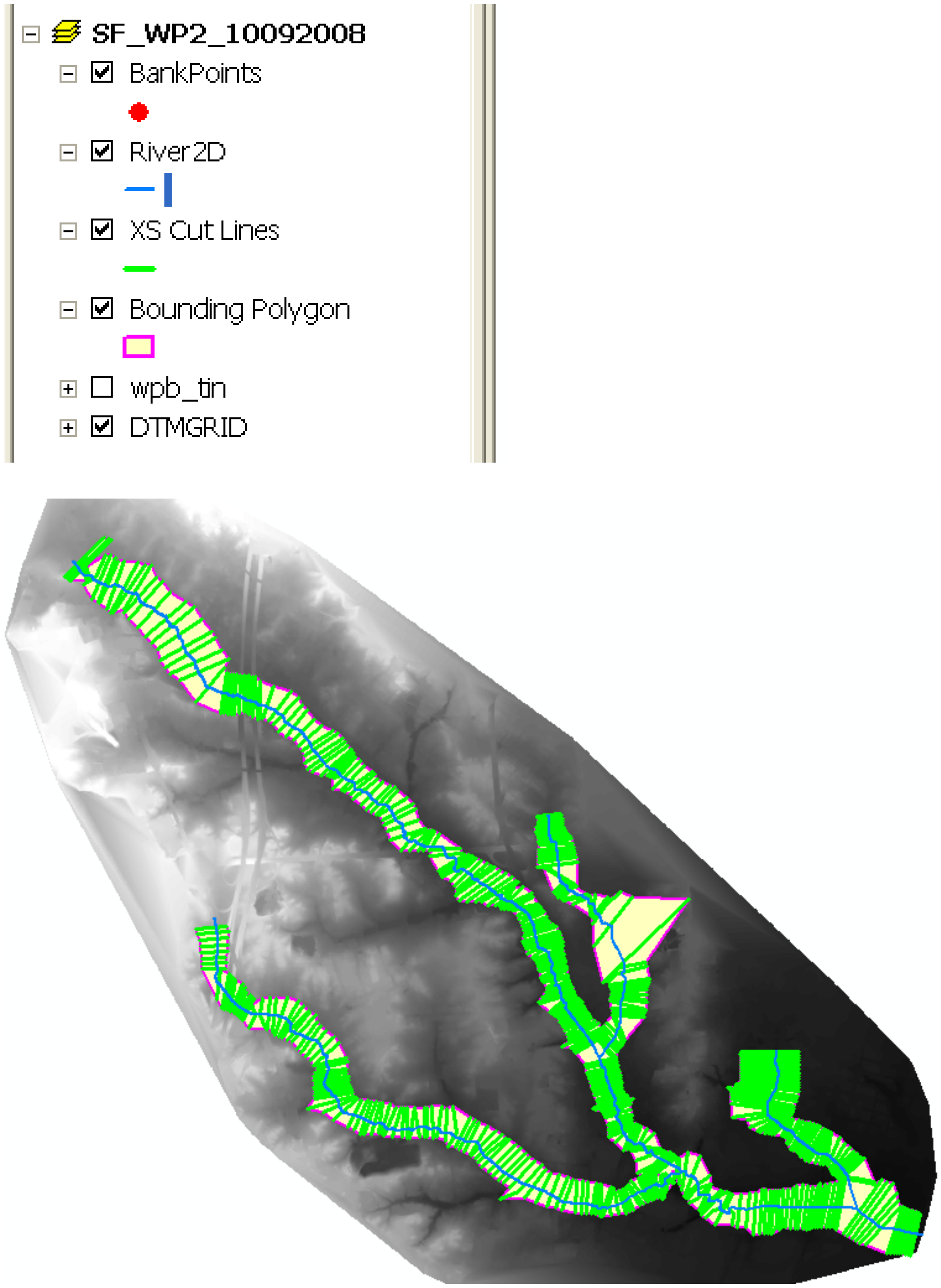


Figure 135: Bounding polygon and DTM grid for WPBW

Now click on RAS Mapping → Inundation Mapping → Water Surface Generation. Select the profile and click ok as shown below. If there are multiple profiles, they can be selected by holding the ctrl and shift key together with a left mouse click.

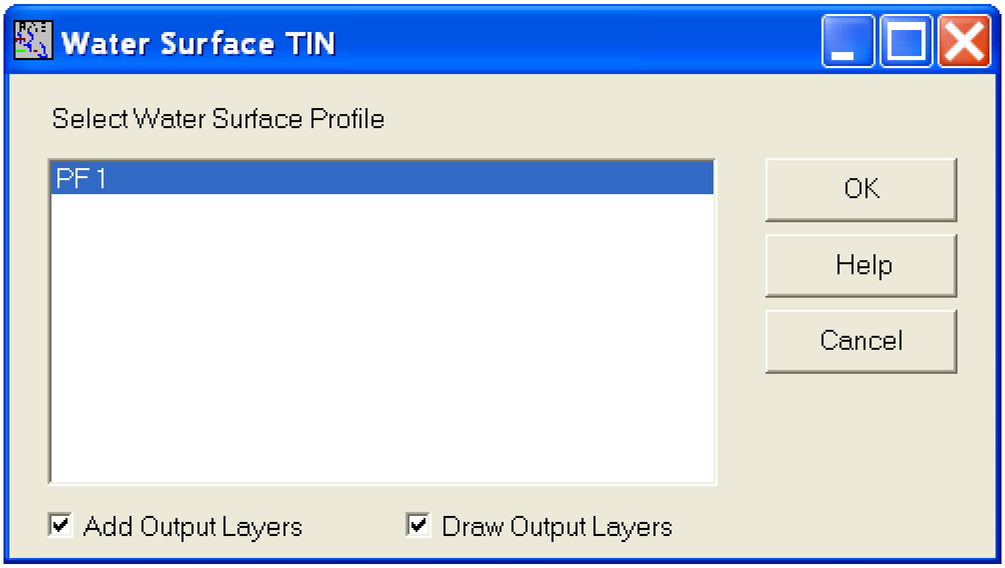


Figure 136: Water surface profile selection for TIN creation

This step creates the water surface TIN for each profile selected irrespective of the terrain model. The ArcGIS triangulation method creates the surface using X-sections as the hard breaklines with constant elevation. This elevation corresponds to the elevation calculated by HEC-RAS for a particular profile. The water surface TIN is stored in the analysis directory using the profile name with prefix ‘t’ for TIN (tPF1 in this case). The TIN created includes the areas outside possible inundation and is generated by connecting the farthest points on the bounding polygon.

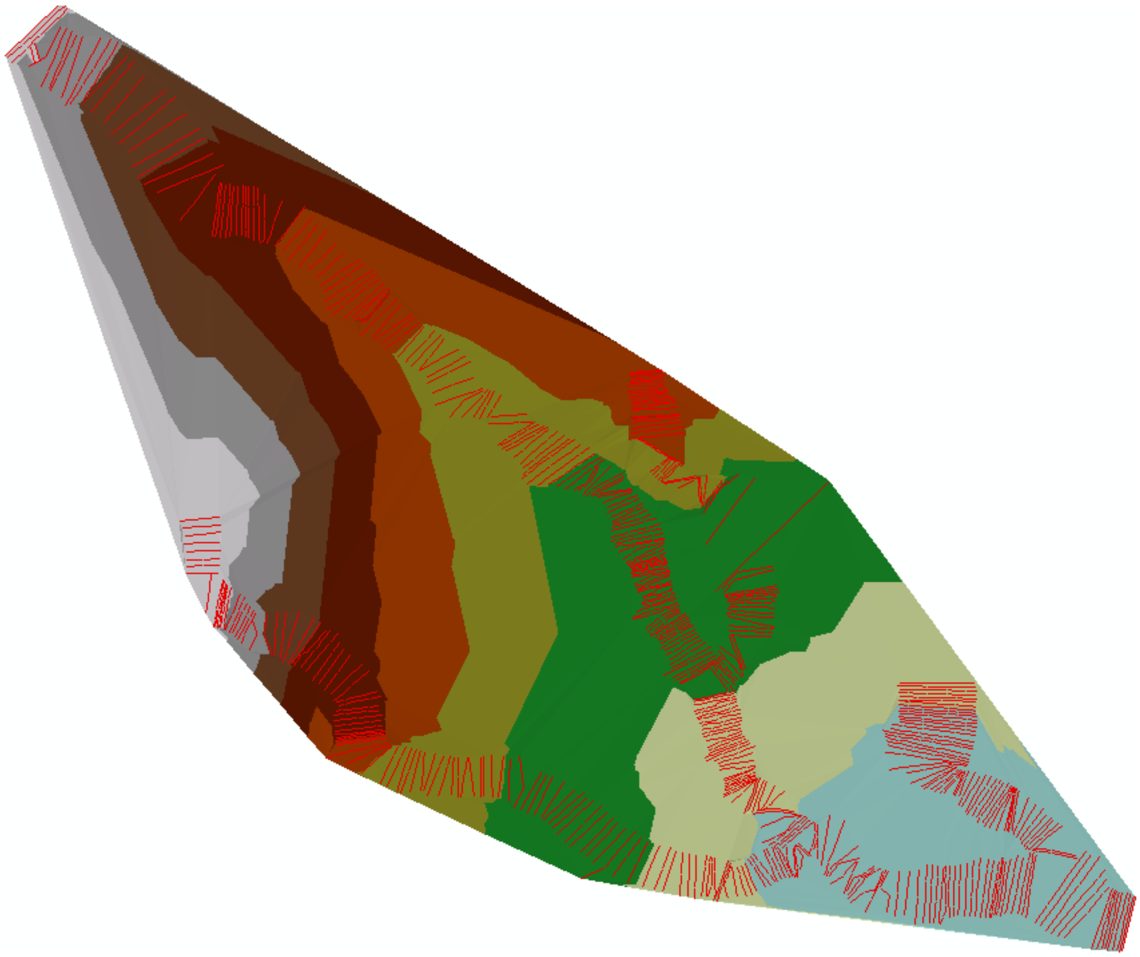


Figure 137: Water surface TIN with cross sections as hard breaklines

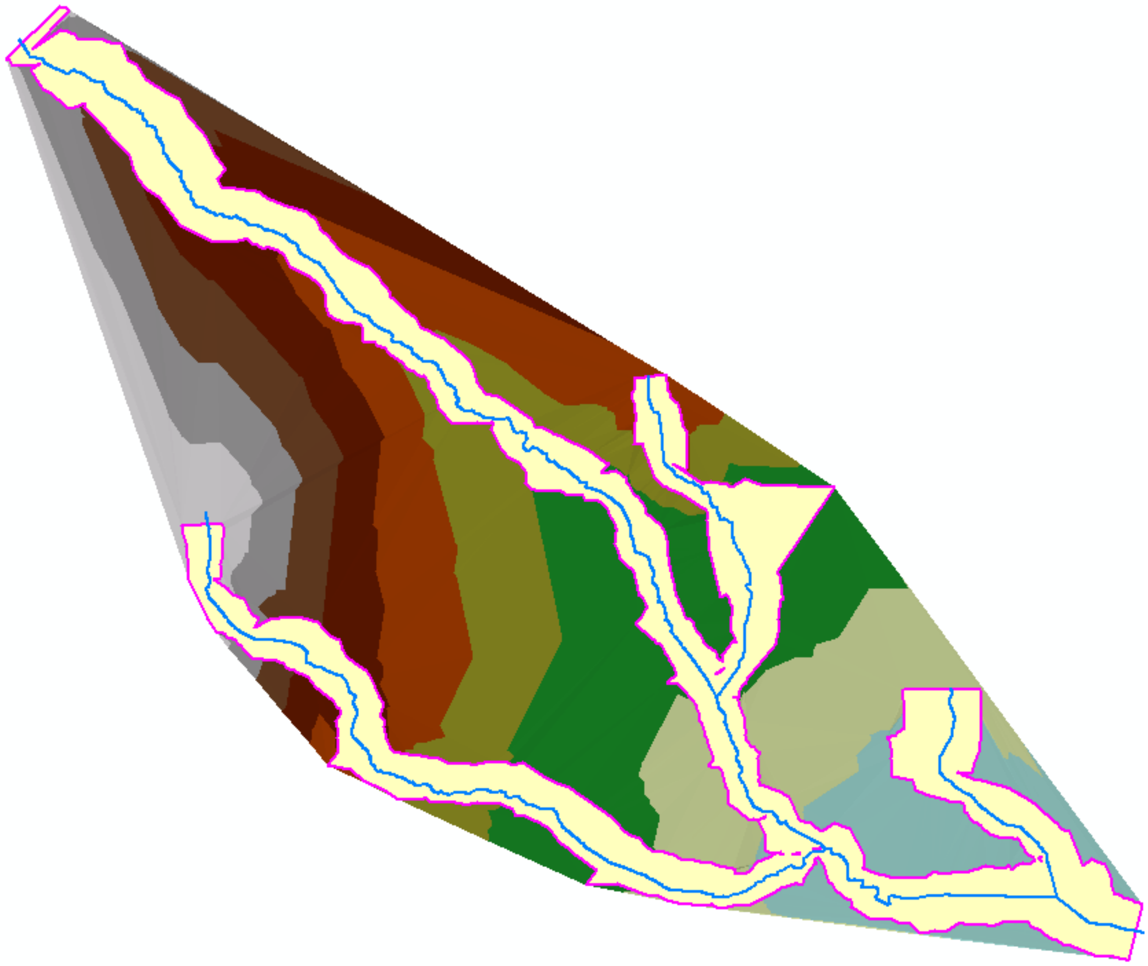


Figure 138: Bounding polygon overlaying on Water surface TIN

FLOODPLAIN DELINEATION:

This step uses Water surface TIN and DTMGRID (created from TIN for Wreck Pond watershed) to calculate the floodplain boundary and inundation depth. The following procedure was used: Click on RAS Mapping → Inundation → Floodplain Delineation. Select profile and click ok.

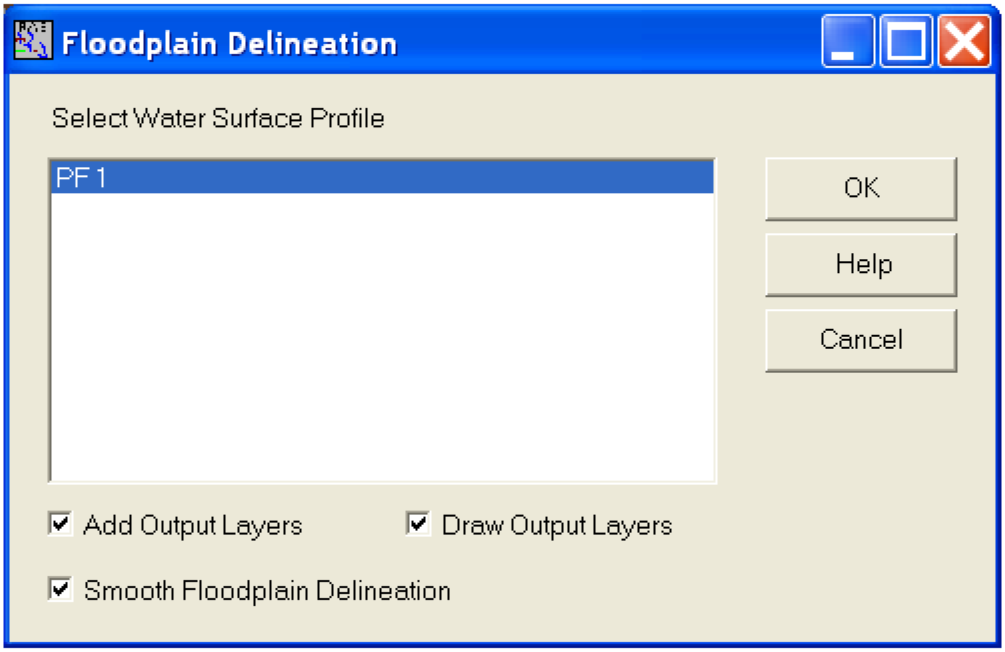
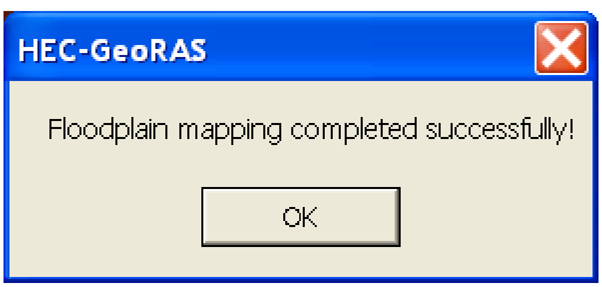


Figure 139: Water surface profile selection for Floodplain delineation

This process first converts the Water surface TIN into Grid. This raster version of the water surface TIN is stored in the analysis directory as 'wsgridP001' DTMGRID and is subtracted from the water surface grid.



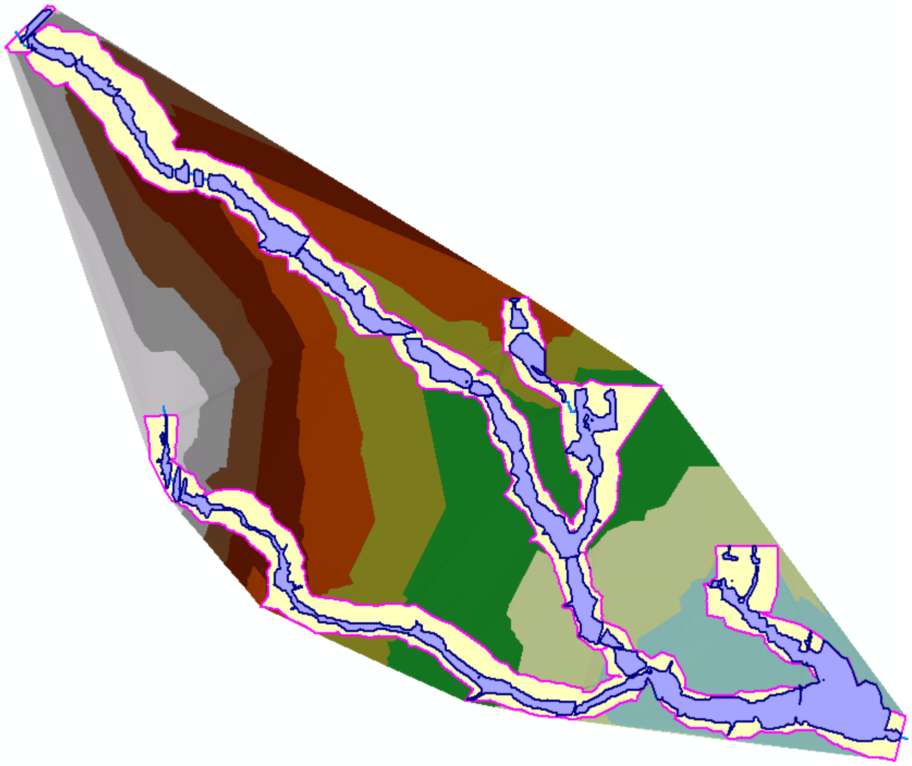


Figure 140: Floodplain overlaying Water surface TIN

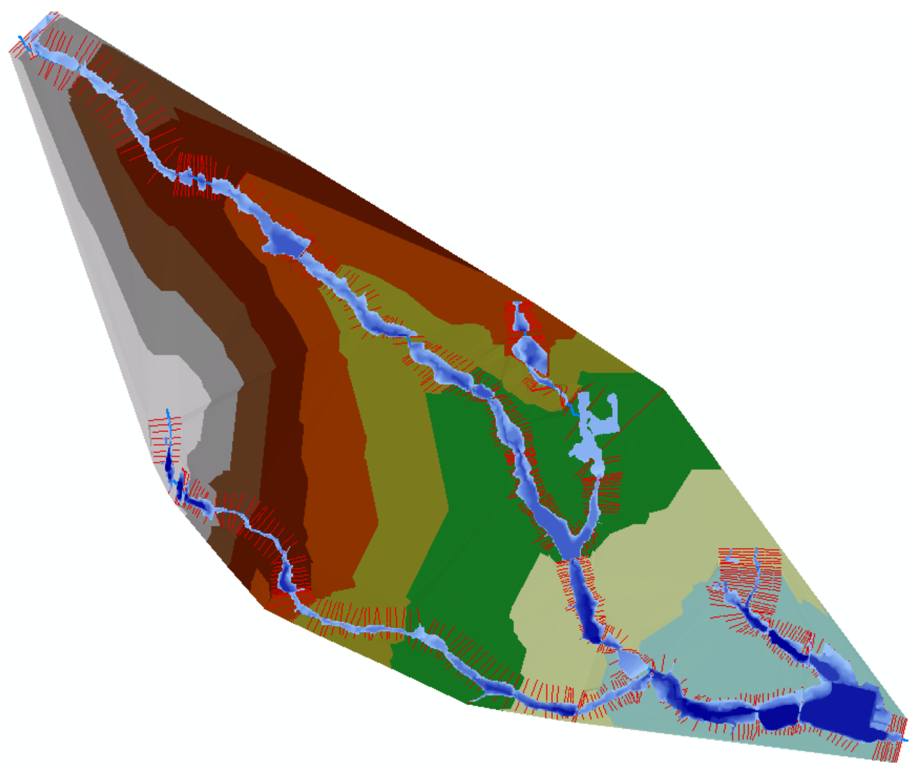


Figure 141: Water depth grid overlaying Water surface TIN

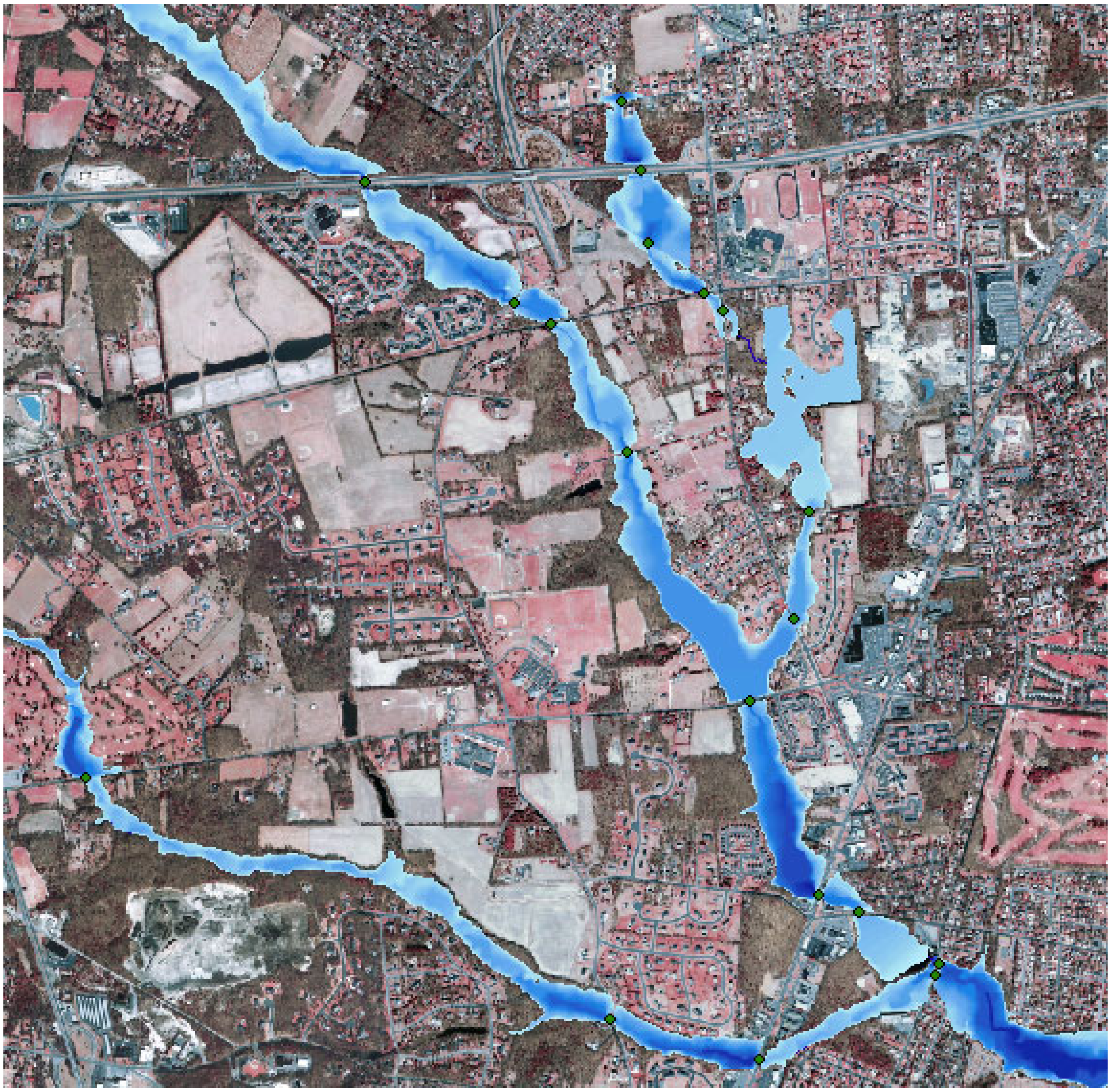


Figure 142: Water depth grid overlaying Aerial photos

4.4.2 Flood inundation area for SCS 100 year storm and DFIRM

Using the steps described in earlier sections, the 100-year floodplain was delineated for WPBW by entering the 100-year flow values in the steady flow data window in HEC-RAS. The following figures (Figures 143–146) show the 100-year floodplain (red-colored polygon) computed in ArcGIS and is overlaid by the draft DFIRM (1% annual chance - yellow line) for Wreck Pond Brook and Hannabrand Brook.

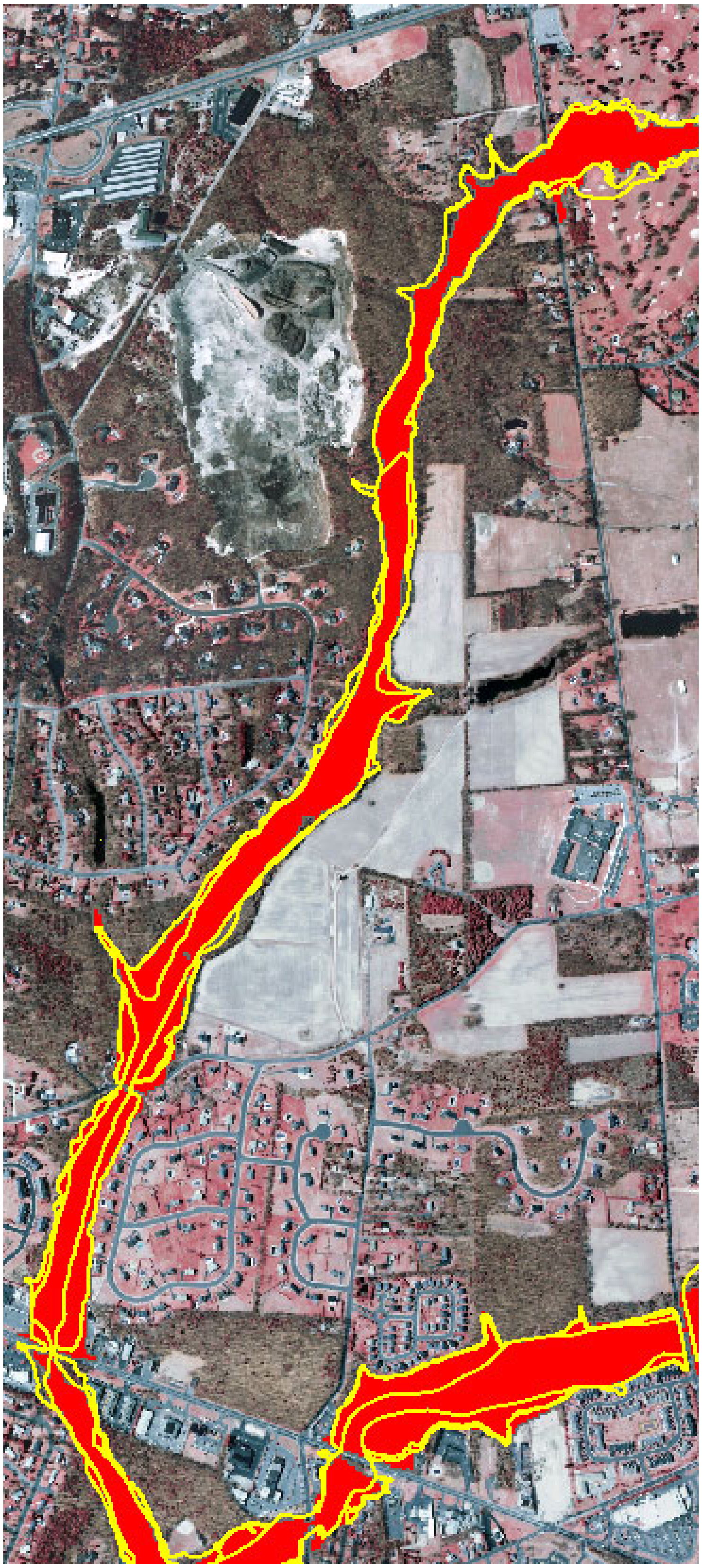


Figure 143: Modeled and DFIRM 100-year floodplain for Hannabrand Brook

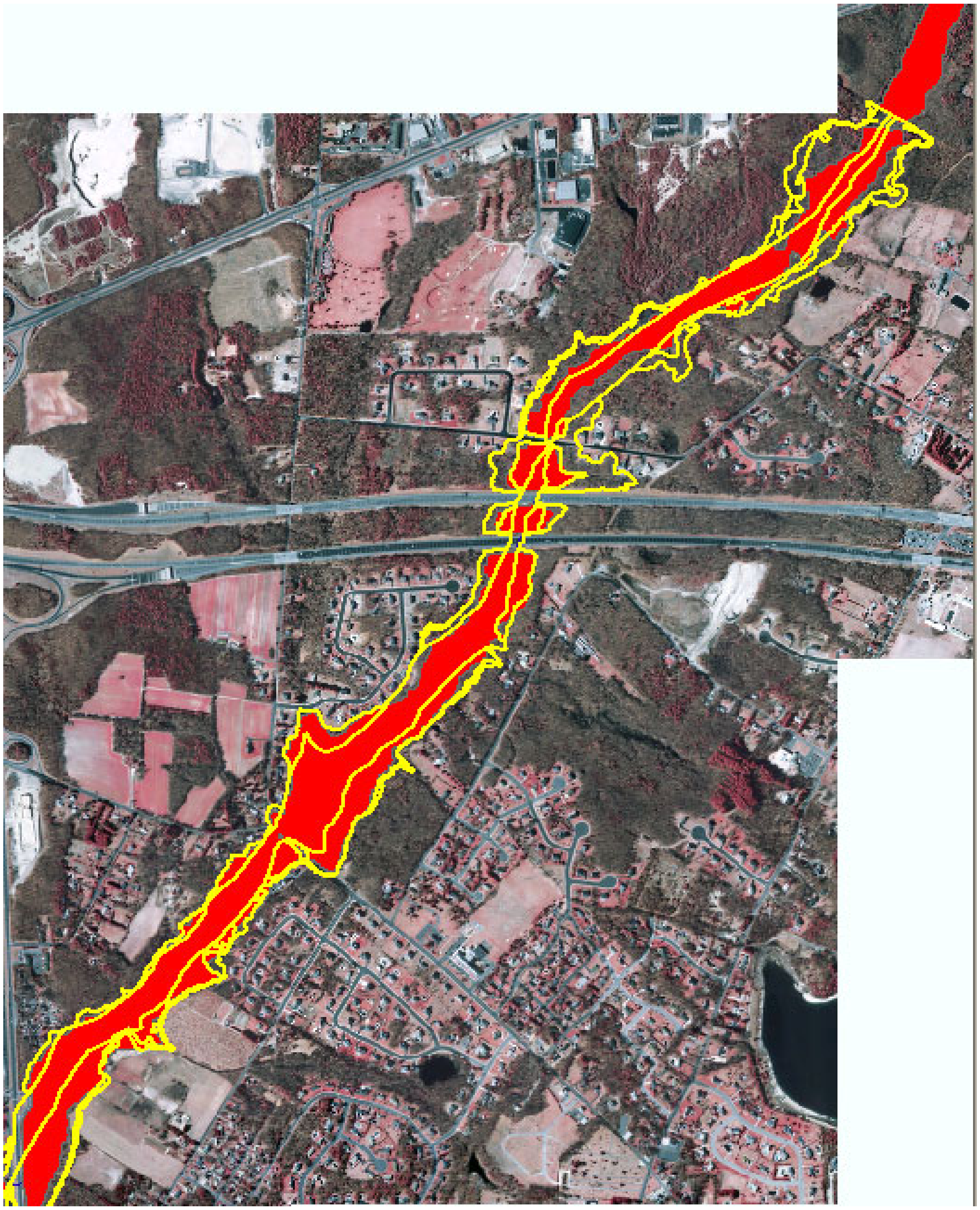


Figure 144: Modeled and DFIRM 100-year floodplain for Wreck Pond Brook (up to Hwy 138)

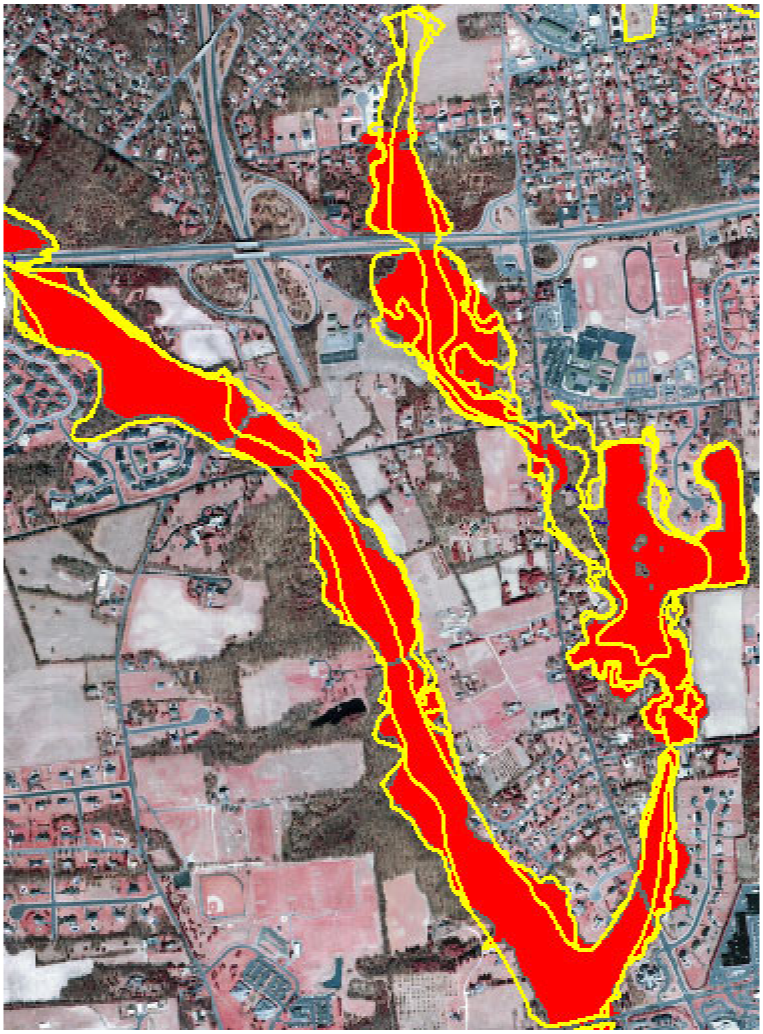


Figure 145: Modeled and DFIRM 100-year floodplain for Wreck Pond Brook (Hwy 138 to Allaire Rd) and Albers Pond Creek

Figure 146: Modeled and DFIRM 100-year floodplain for Wreck Pond Brook (Allaire Rd to just downstream of Old Mill Pond)

4.4.3 Flood inundation area for October 2005 storm and SCS 100 year storm

Figures 147-149 show the mapped flood inundation area using ArcGIS for the October 2005 storm event (blue) overlaying on 100-year floodplain (red). It can be seen from the figure that the extent of flooding for October 2005 for a majority of the watershed was nearly equal to the 100-year flood.

Figure 147: October 2005 floodplain (blue) overlaying 100-year floodplain (red)

Figure 148: Zoomed in view of October 2005 flood map (blue) overlaying 100-year floodplain (red) (section on Wreck Pond Brook between Allaire Rd and Hwy 35)

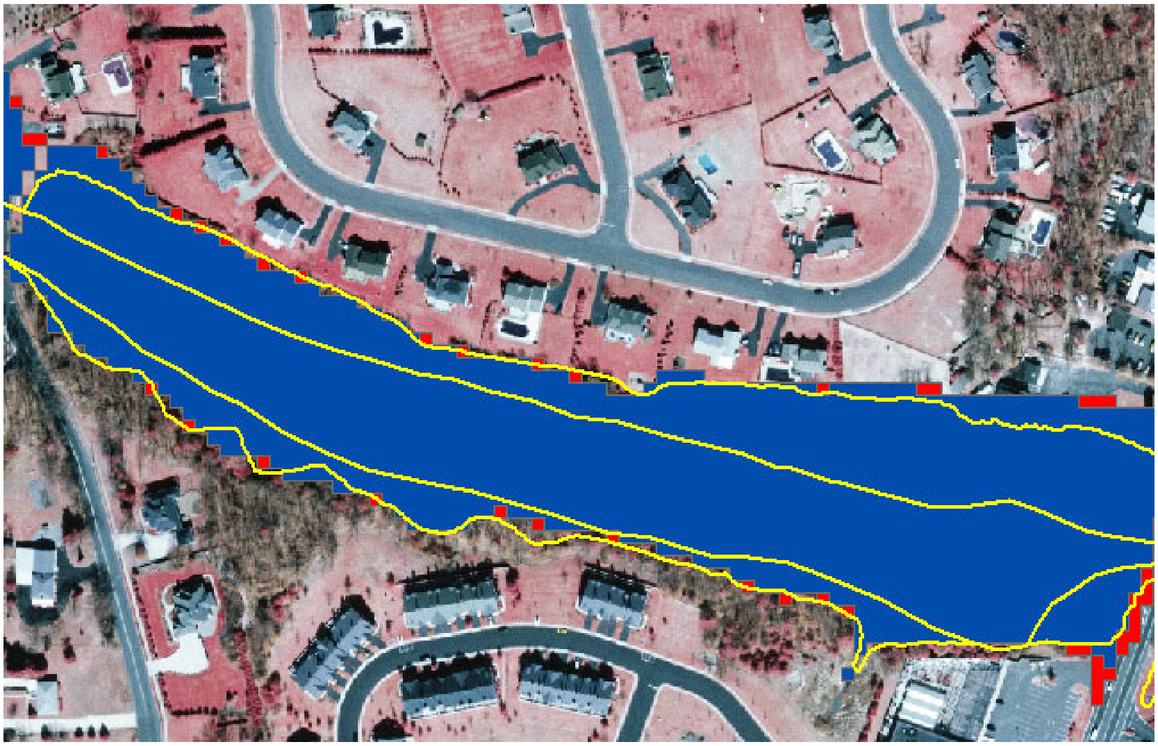


Figure 149: Section on Hannabrand Brook between Bailey's Corner Rd and Hwy 35 (October 2005 floodplain (blue), Modeled 100-year floodplain (red) and DFIRM line (yellow))

5 RESULTS AND DISCUSSION

5.1 Flows using different approaches

This section shows the results of flows developed using the industry standard method and flows developed using the 5%Ia/S ratio and percentage curves from three different unit hydrographs, i.e., Standard SCS dimensionless unit hydrograph, Delmarva unit hydrograph, and Lower Monmouth dimensionless unit hydrograph.

5.1.1 Modeling Results for NRCS 100-Year Storm

Table 39 shows the flow values computed using different parameters for the NRCS 100 year design storm event. The specified rainfall was 8.9 inch in 24 hour as shown in Table 31.

Table 39: Flows for different hydrologic elements using different unit hydrographs and abstraction ratio for 100-year storm event

Hydrologic Element	**Peak Discharge (CFS) for NRCS 100 Year storm event**			
	SCS - PRF 484	**SCS**	**Delmarva**	**LMDUH**
	20% Ia Ratio	**5% Ia Ratio**		
Wreck Pond Brook				
W6 – Martin's Rd	938	791	569	544
W9 – Hurley's Pond	1538	1309	1015	989
W7 – Glendola Rd	1675	1427	1256	1227
W1 – Waterford Glen	1975	1692	1513	1484
W3 – Old Mill Rd	1926	1674	1497	1474
Hannabrand Brook				
W5 – Baileys Corner Rd	1102	932	659	606
W2 – Old Mill Rd	1028	869	654	633

Combined discharge below Old Mill Rd	2880	2494	2085	2049
Sea Girt/Spring Lake/Spring Lake Heights	1869	1650	1110	1010
W8 – Golf Courses	805	700	501	478
Combined inflows to Wreck Pond	2731	2369	2097	2077

Figures 150-151 compare the hydrographs at the outlet of two streams (Wreck Pond Brook and Hannabrand Brook) before they merge. The outlet of Wreck Pond Brook before the merge is labeled "Old Mill Pond W3 gage" and outlet of Hannabrand Brook is labeled "Hannabrand outlet". The comparison of hydrographs after the merge point also is shown and is labeled "WPB-Han Combined". The hydrographs show the stream response for NRCS 100-year storm and provides a graphical comparison of peak discharges using the industry standard approach and the modified regional parameters. The green colored hydrograph shows the response modeled using the industry standard approach (PRF of 484 and 20% Ia ratio), whereas the black colored hydrograph shows the response using one industry standard parameter (PRF – 484) and one modified parameter (5% Ia ratio). The blue colored hydrograph (PRF – 284 and 5% Ia ratio) and red colored hydrograph (PRF – 230 and 5% Ia ratio) show the response using both modified parameters.

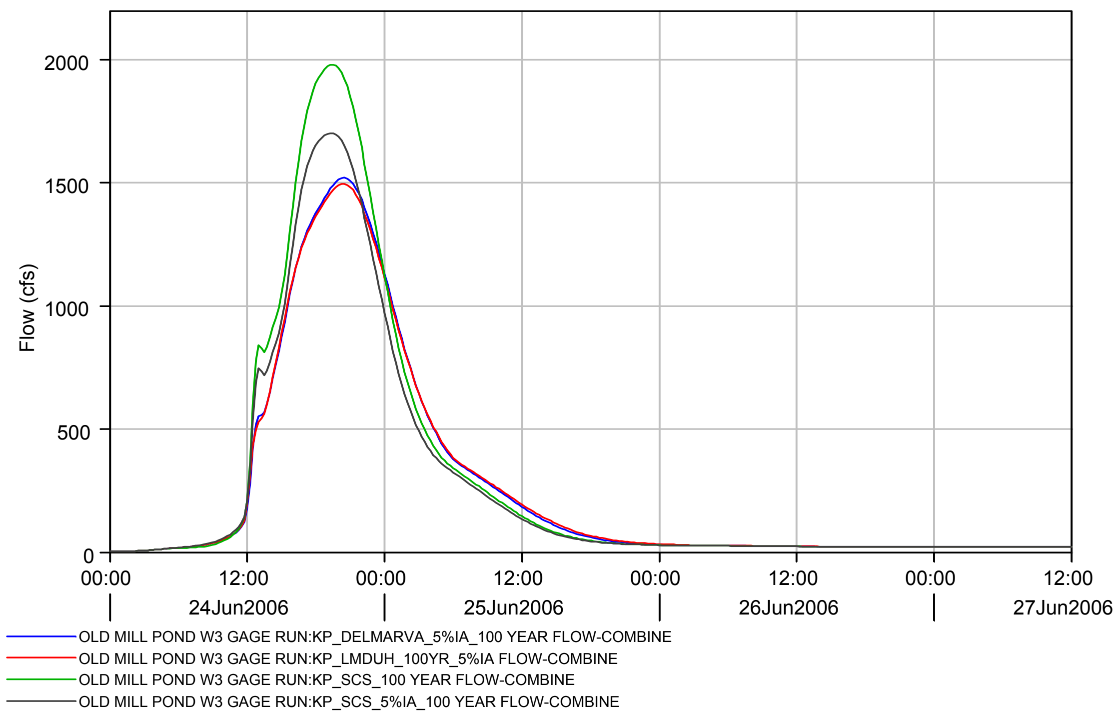


Figure 150: Hydrograph comparison for Wreck Pond Brook at Station W3 for NRCS 100-year storm event

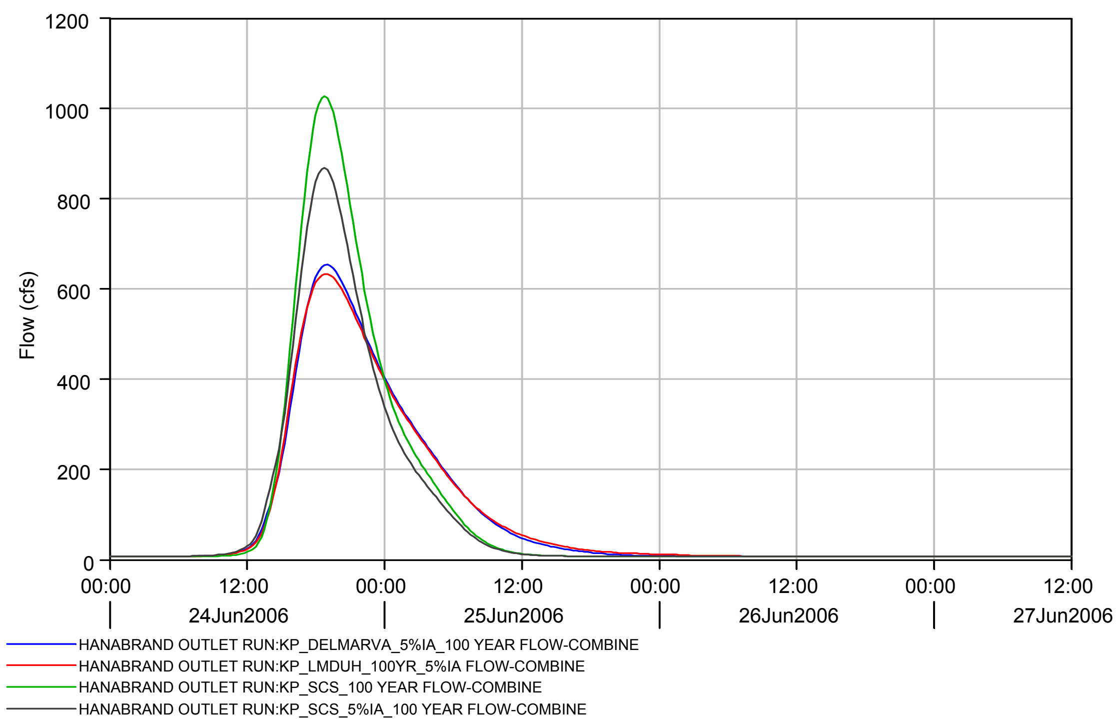


Figure 151: Hydrograph comparison for Hannabrand Brook at Station W2 for NRCS 100-year storm event

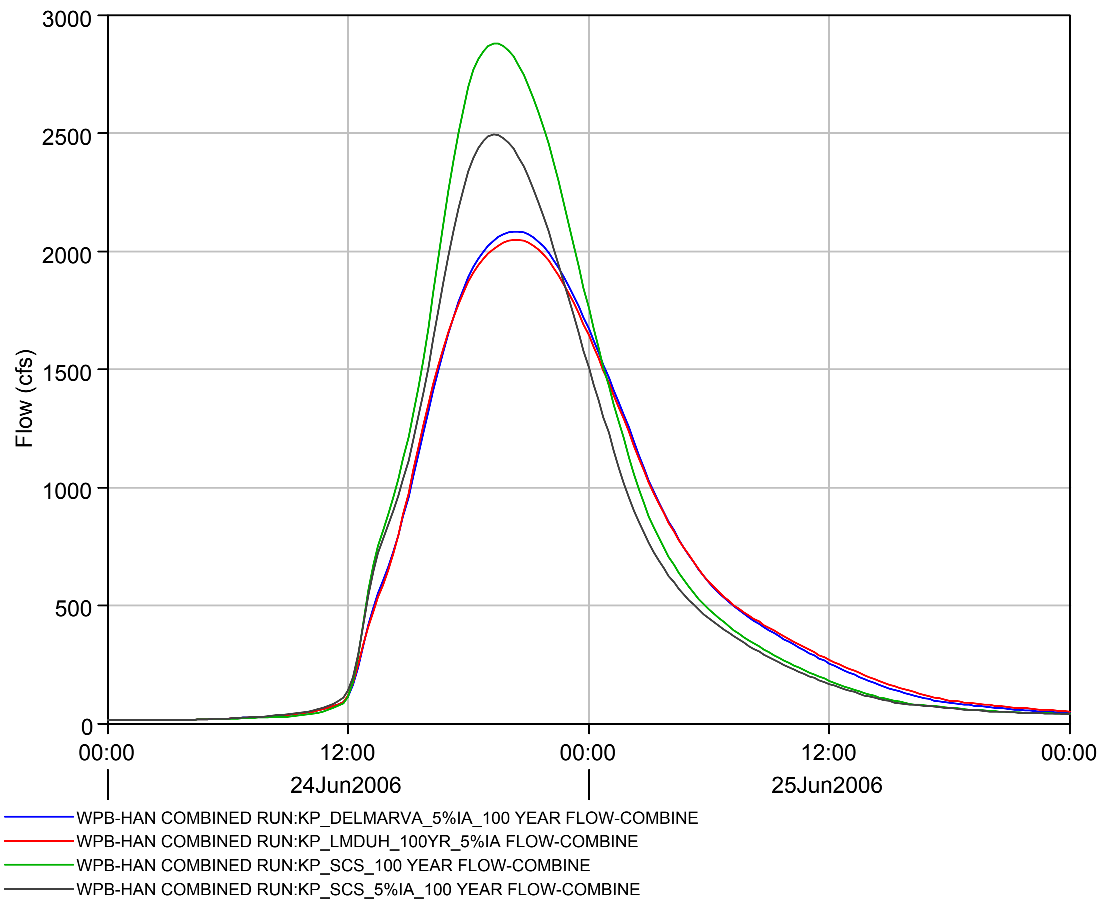


Figure 152: Hydrograph comparison for combined flows of Wreck Pond Brook and Hannabrand Brook for NRCS 100-year storm event

Figure 153 shows the inflow into Wreck Pond for NRCS 100-year storm event. The modeled hydrograph is simulated using the revised modeled in this study (up to confluence of Hannabrand Brook and Wreck Pond Brook) and the existing RSWMP model below this confluence. None of these models takes into account the tidal effect of Atlantic Ocean.

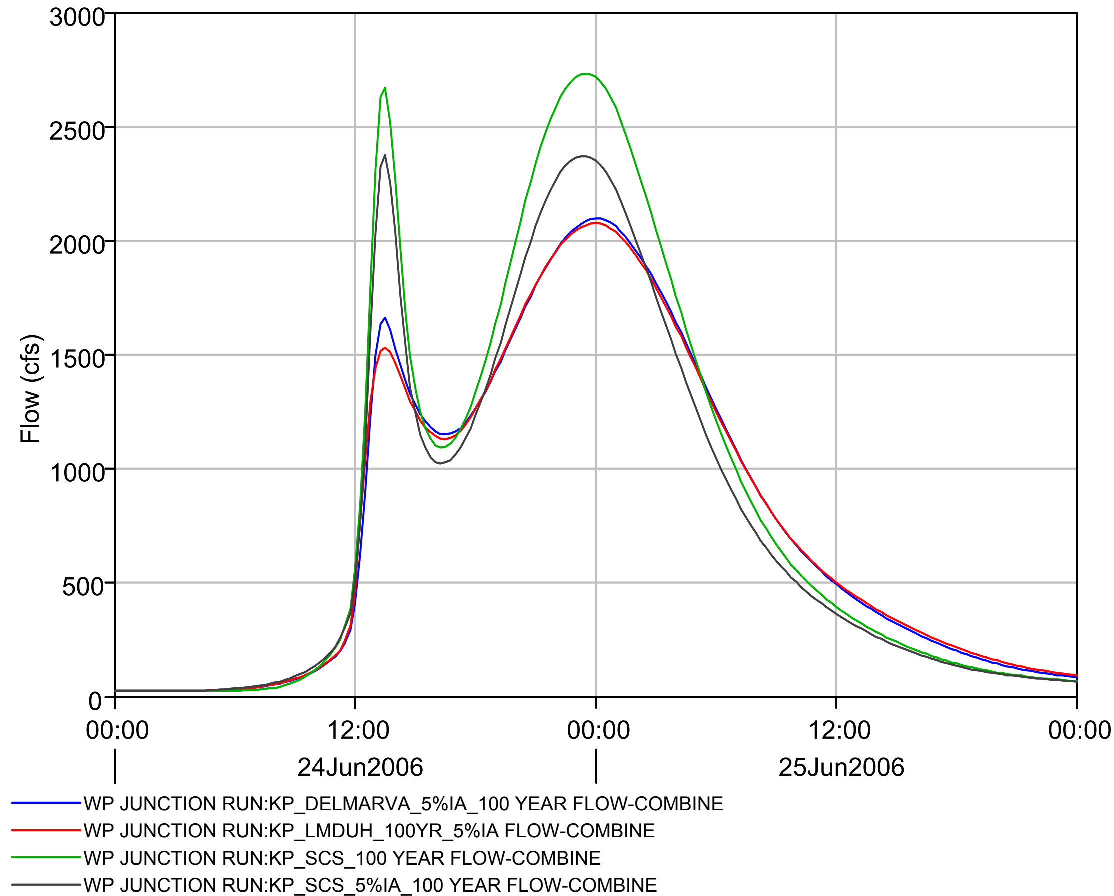


Figure 153: Hydrograph comparison for flows into Wreck Pond for NRCS 100-year storm event

Table 54 shows the difference in flow values expressed as percentages using the industry standard approach of the SCS dimensionless unit hydrograph and a 20%Ia ratio against the Delmarva and Lower Monmouth unit hydrographs and a 5%Ia ratio.

Table 54: Percentage difference in flows using different unit hydrographs

Hydrologic Element	**Percent difference (%) in flow values for NRCS 100-Year storm event**		
	SCS (20%Ia) Vs. Delmarva (5%Ia)	**SCS (20%Ia) Vs. LMDUH (5%Ia)**	**Delmarva(5%Ia) Vs. LMDUH (5%Ia)**
Wreck Pond Brook			
W6 – Martin's Rd	39.34	42.00	4.39
W9 – Hurley's Pond	34.01	35.70	2.56
W7 – Glendola Rd	25.01	26.75	2.31
W1 – Waterford Glen	23.39	24.86	1.92
W3 – Old Mill Rd	22.27	23.47	1.54
Hannabrand Brook			
W5 – Baileys Corner Rd	40.20	45.01	8.04
W2 – Old Mill Rd	36.38	38.42	3.21
Combined discharge below Old Mill Rd	27.60	28.85	1.73
Sea Girt/Spring Lake/Spring Lake Heights	40.61	45.96	9.01
W8 – Golf Courses	37.76	40.62	4.59
Combined inflows to Wreck Pond	23.21	23.95	0.95

5.1.2 Modeling Results for NRCS 50-Year Storm

The hydrographs (Figures 154-157) show the stream responses for NRCS 50-year storm and provide graphical comparisons of peak discharges using the industry standard approach and the modified regional parameters. The black-colored hydrograph shows the response modeled using the industry standard approach (PRF of 484 and 20% Ia ratio), whereas the green-colored hydrograph shows the response using one industry standard parameter (PRF – 484) and one modified parameter (5% Ia ratio). The blue-colored hydrograph (PRF – 284 and 5% Ia ratio) and red-colored hydrograph (PRF – 230 and 5% Ia ratio) show the responses using both modified parameters.

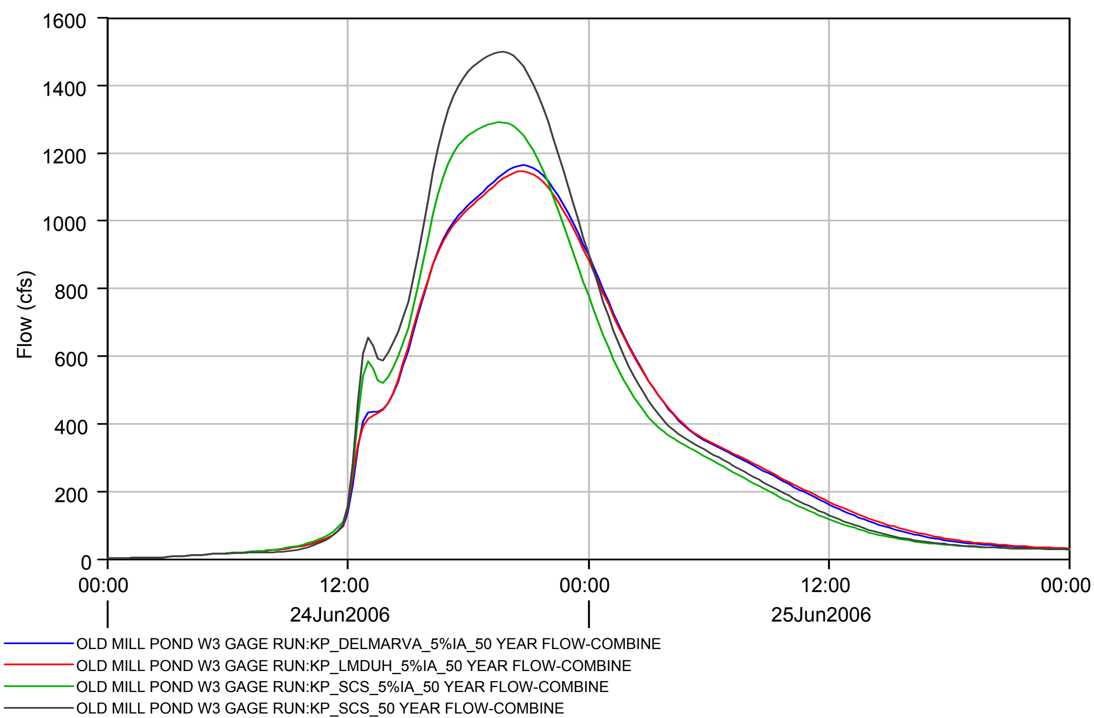


Figure 154: Hydrograph comparison for Wreck Pond Brook at Station W3 for NRCS 50-year storm event

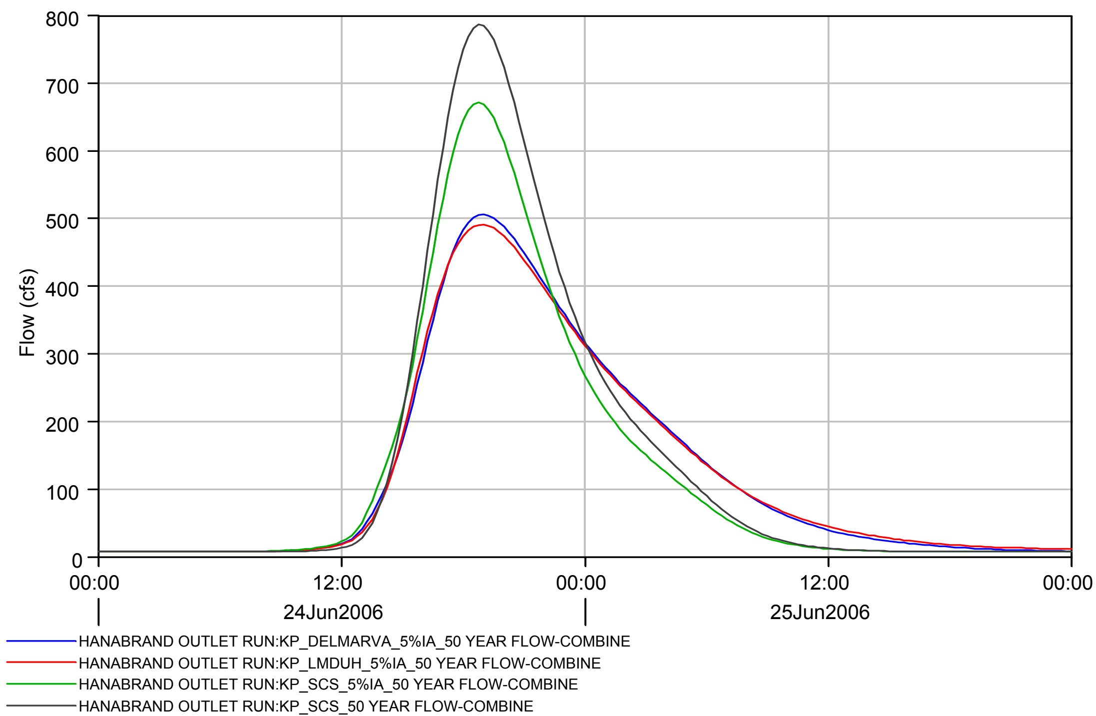


Figure 155: Hydrograph comparison for Hannabrand Brook at Station W2 for NRCS 50-year storm event

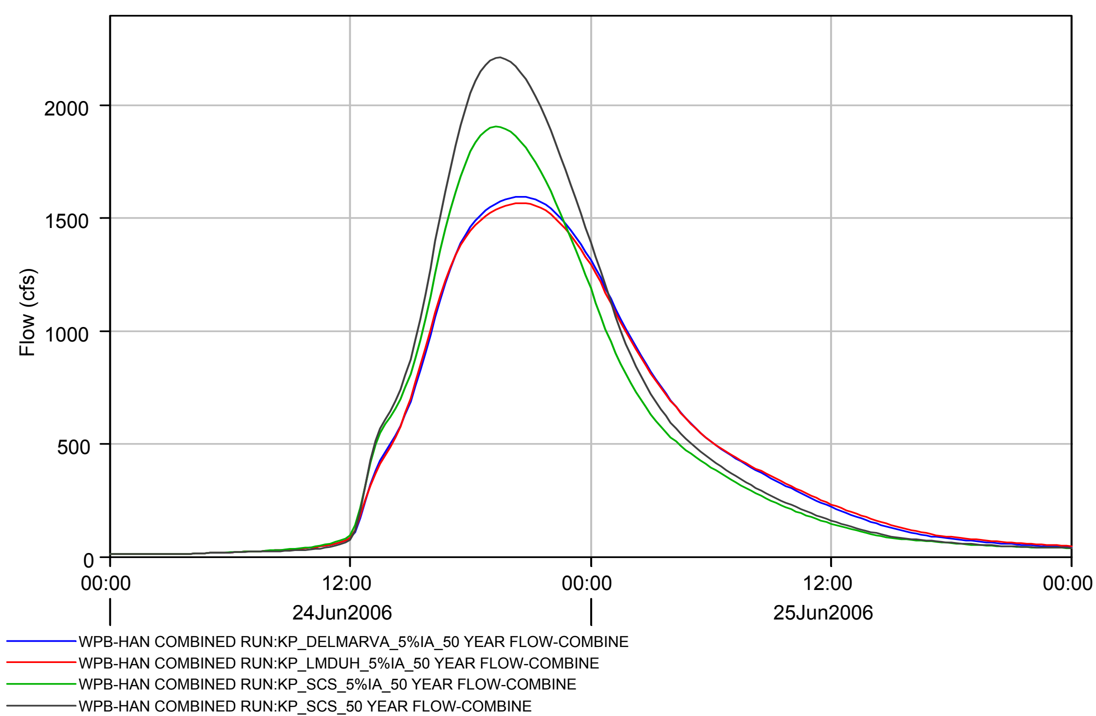


Figure 156: Hydrograph comparison for combined flows of Wreck Pond Brook and Hannabrand Brook for NRCS 50-year storm event

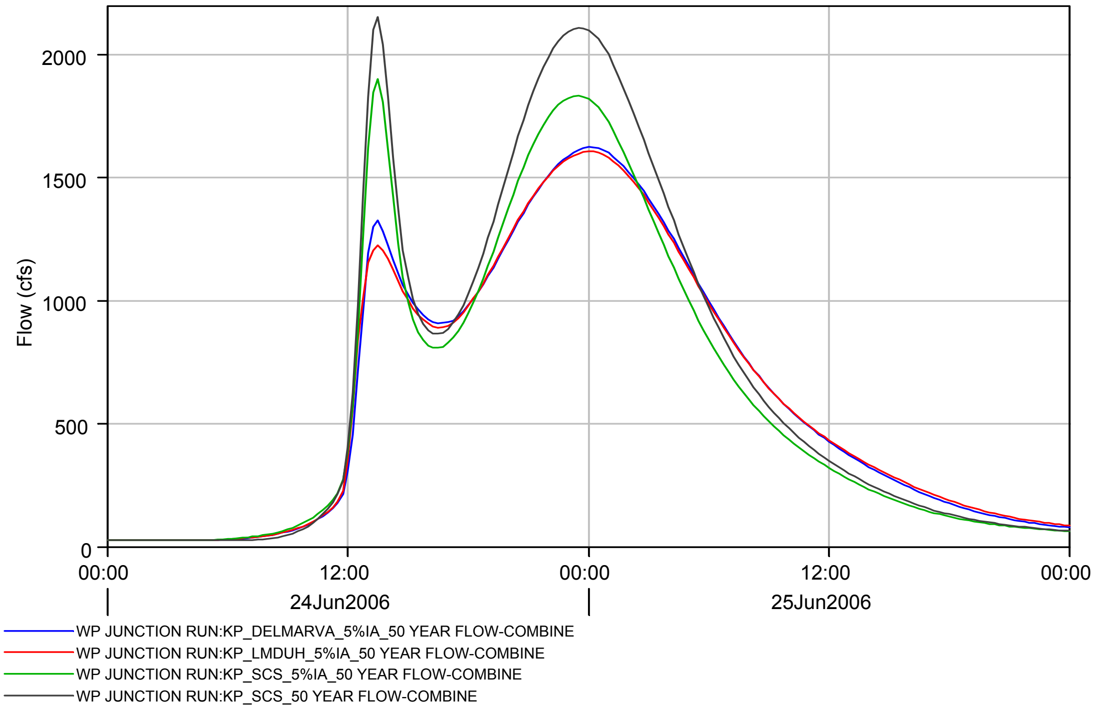


Figure 157: Hydrograph comparison for flows into Wreck Pond for NRCS 50-year storm event

5.1.3 Modeling Results for NRCS 25-Year Storm

The hydrographs (Figures 158-161) show the stream responses for the NRCS 25-year storm and provides graphical comparison of peak discharges using industry standard approach and modified regional parameters. The green-colored hydrograph shows the response modeled using the industry standard approach (PRF of 484 and 20% Ia ratio), whereas the black-colored hydrograph shows the response using one industry standard parameter (PRF – 484) and one modified parameter (5% Ia ratio). The blue-colored hydrograph (PRF – 284 and 5% Ia ratio) and red-colored hydrograph (PRF – 230 and 5% Ia ratio) show the responses using both modified parameters.

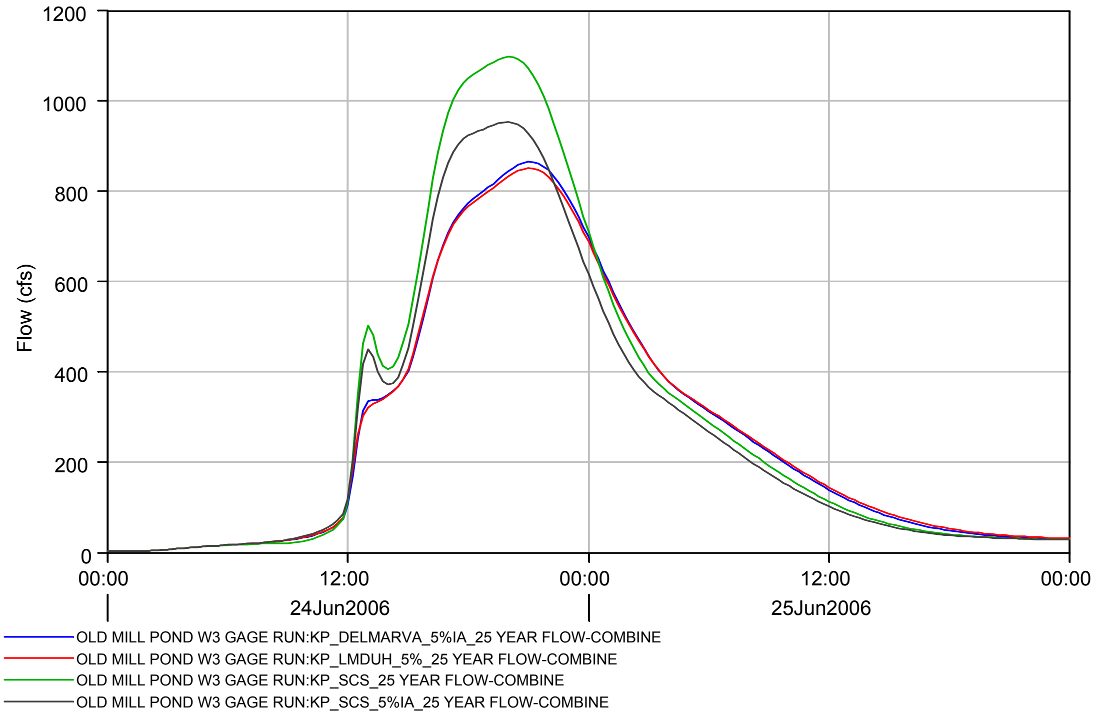


Figure 158: Hydrograph comparison for Wreck Pond Brook at Station W3 for NRCS 25-year storm event

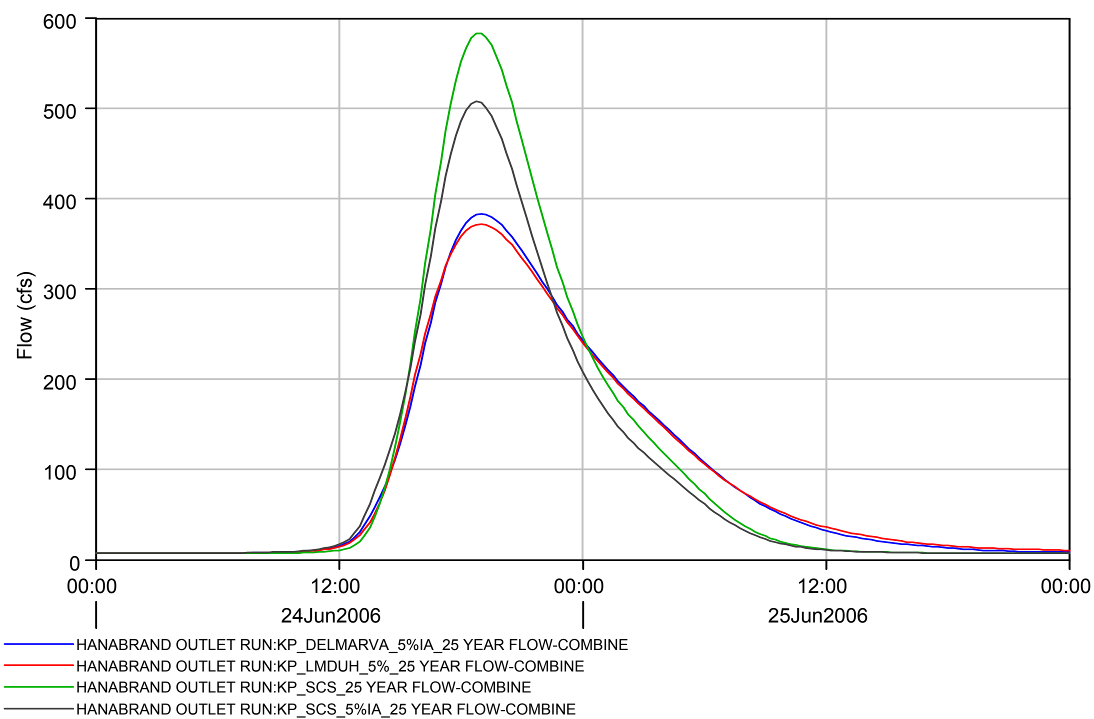


Figure 159: Hydrograph comparison for Hannabrand Brook at Station W2 for NRCS 25-year storm event

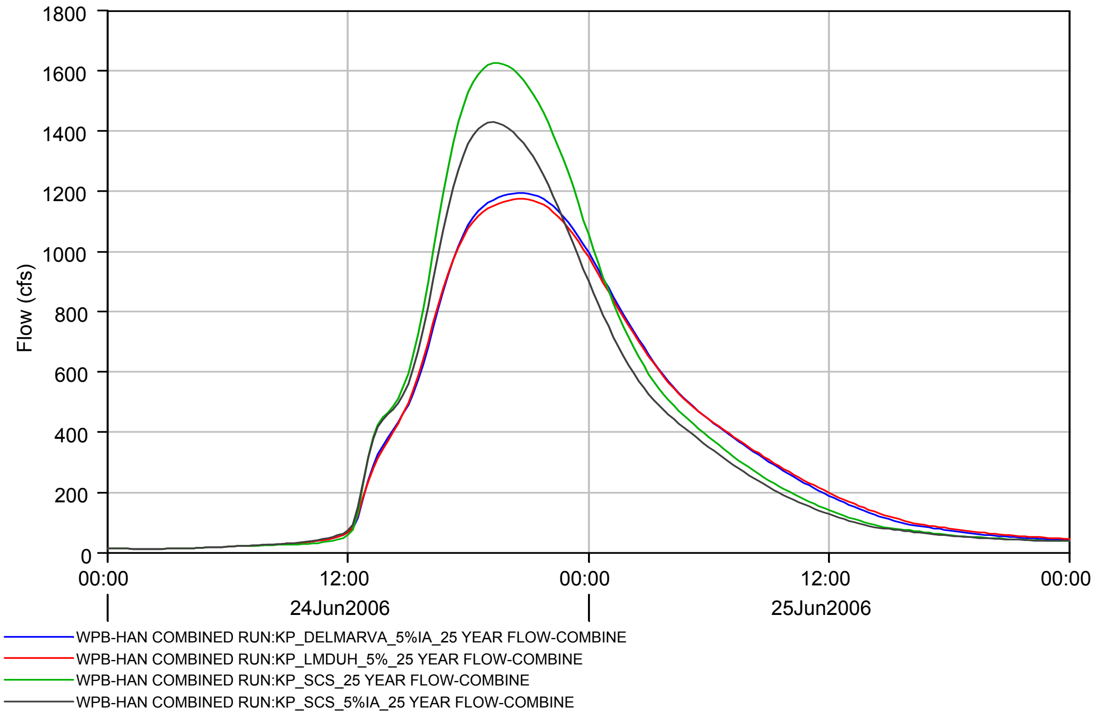


Figure 160: Hydrograph comparison for combined flows of Wreck Pond Brook and Hannabrand Brook for NRCS 25-year storm event

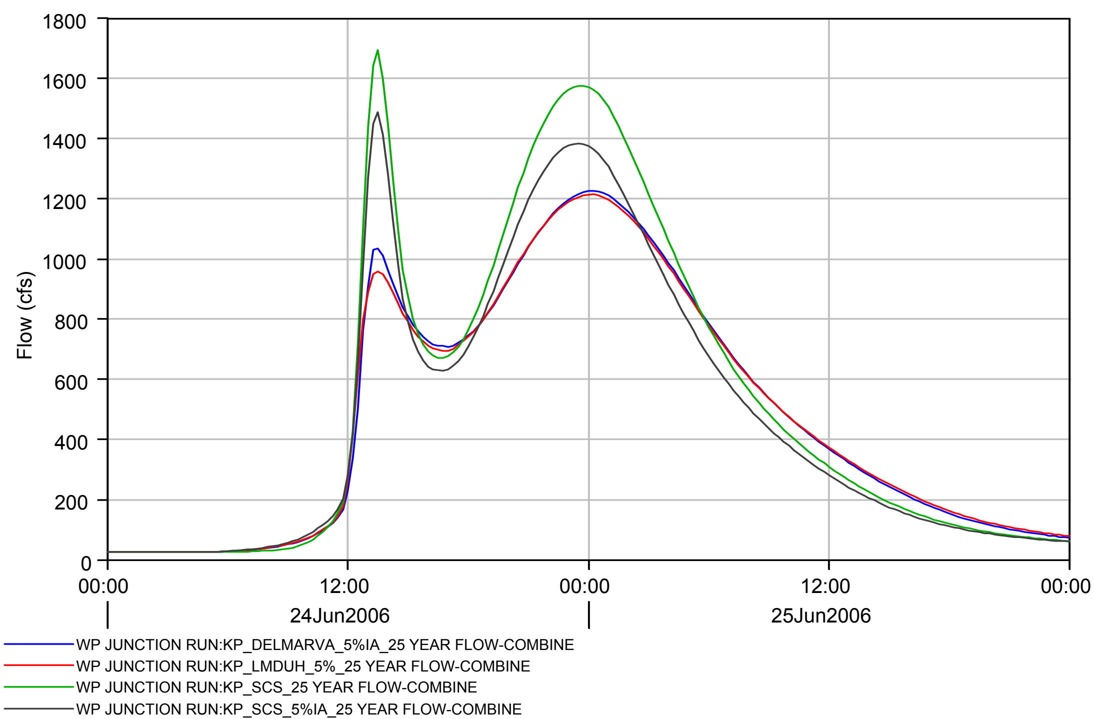


Figure 161: Hydrograph comparison for flows into Wreck Pond for NRCS 25-year storm event

5.1.4 Modeling Results for NRCS 10-Year Storm

The hydrographs (Figures 162-165) show the stream response for the NRCS 10-year storm and provide graphical comparisons of peak discharges using the industry standard approach and the modified regional parameters. The green-colored hydrograph shows the response modeled using the industry standard approach (PRF of 484 and 20% Ia ratio), whereas the black-colored hydrograph shows the response using one industry standard parameter (PRF – 484) and one modified parameter (5% Ia ratio). The blue-colored hydrograph (PRF – 284 and 5% Ia ratio) and the red-colored hydrograph (PRF – 230 and 5% Ia ratio) show the responses using both modified parameters.

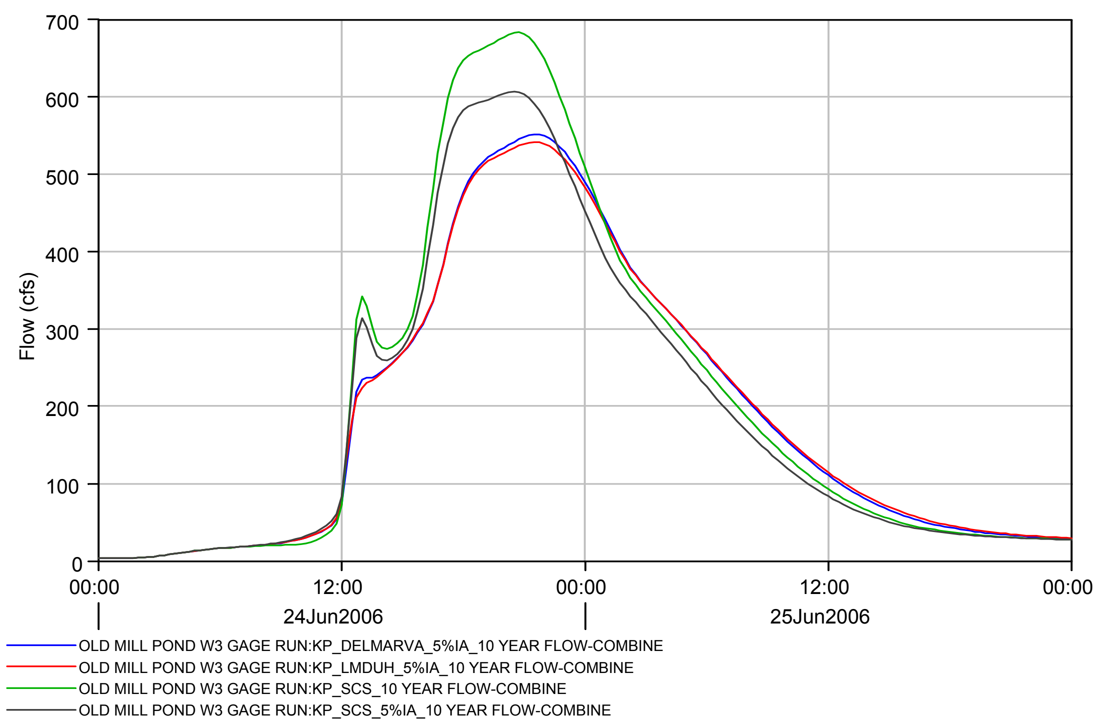


Figure 162: Hydrograph comparison for Wreck Pond Brook at Station W3 for NRCS 10-year storm event

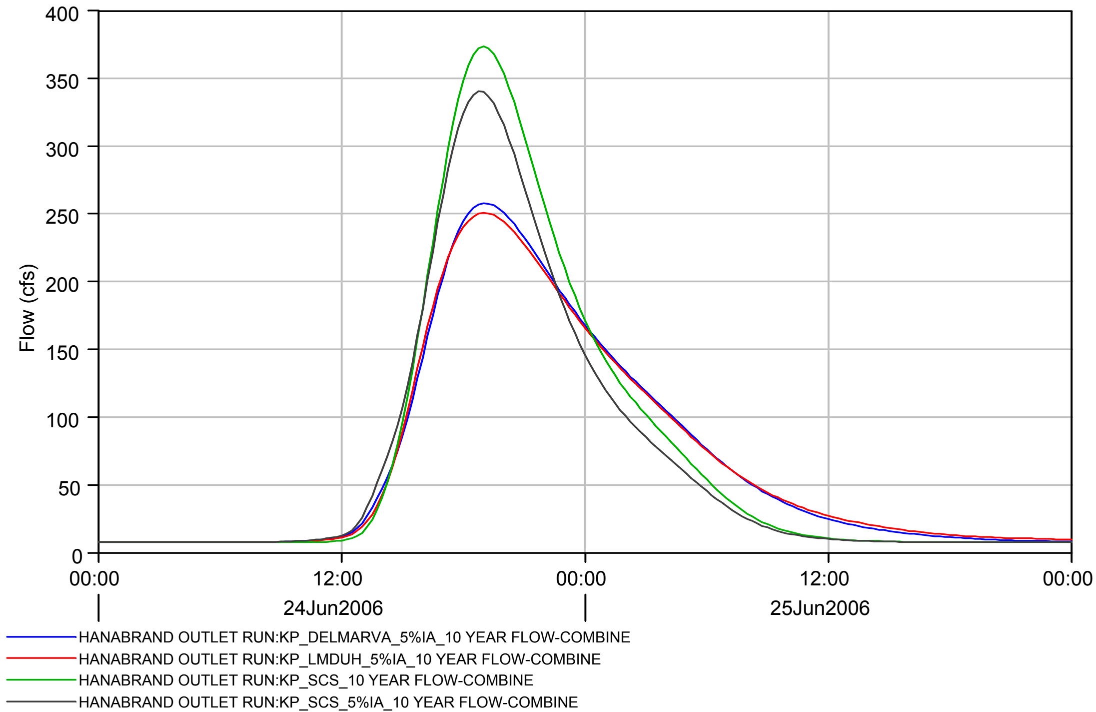


Figure 163: Hydrograph comparison for Hannabrand Brook at Station W2 for NRCS 10-year storm event

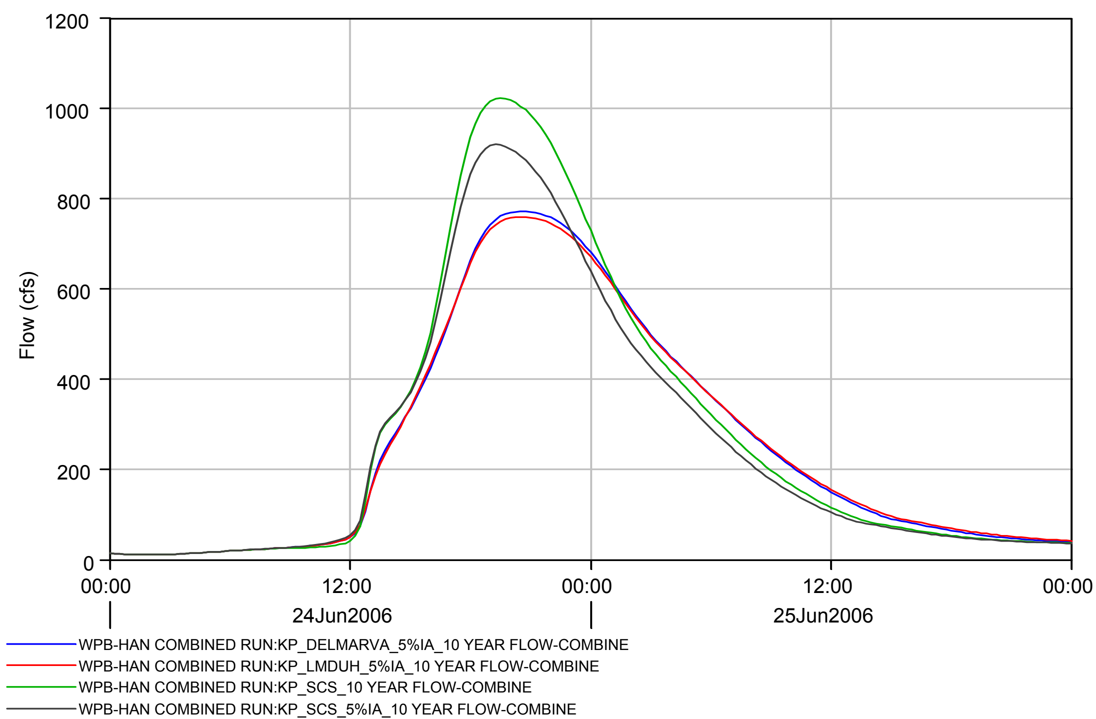


Figure 164: Hydrograph comparison for combined flows of Wreck Pond Brook and Hannabrand Brook for NRCS 10-year storm event

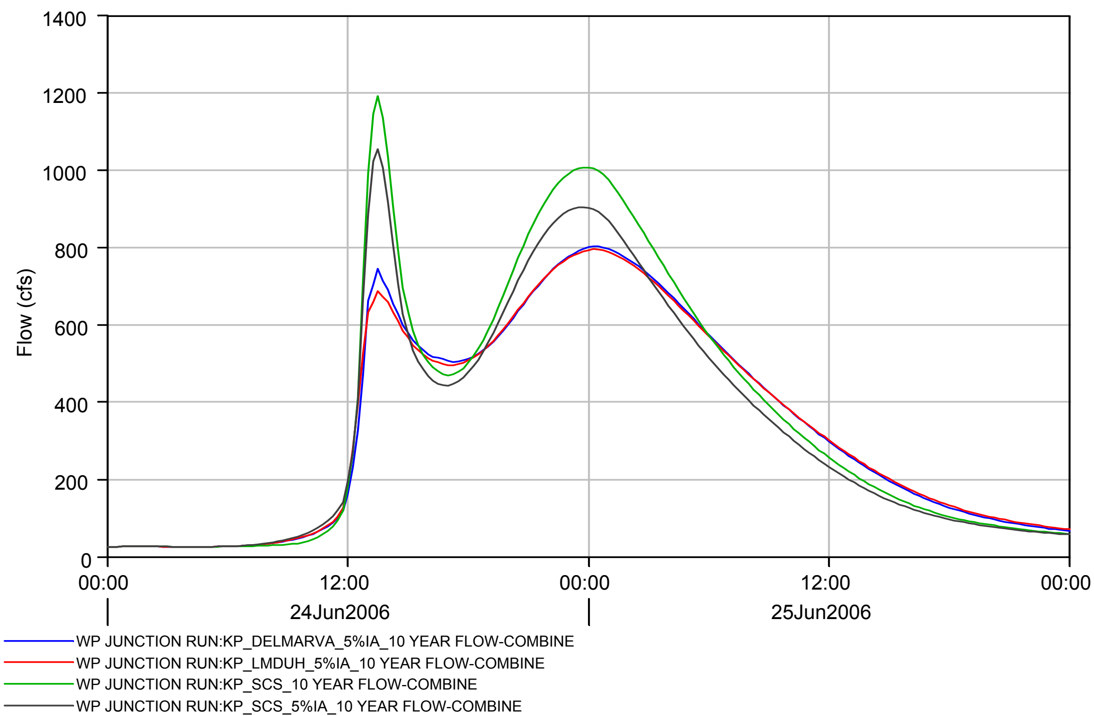


Figure 165: Hydrograph comparison for flows into Wreck Pond for NRCS 10- year storm event

5.1.5 Modeling Results for NRCS 2-Year Storm

The hydrographs (Figures 166-169) show the stream response for the NRCS 2-year storm and provide graphical comparisons of peak discharges using the industry standard approach and the modified regional parameters. The green-colored hydrograph shows the response modeled using the industry standard approach (PRF of 484 and 20% Ia ratio,) whereas the black-colored hydrograph shows the response using one industry standard parameter (PRF – 484) and one modified parameter (5% Ia ratio). The blue-colored hydrograph (PRF – 284 and 5% Ia ratio) and the red-colored hydrograph (PRF – 230 and 5% Ia ratio) show the responses using both modified parameters.

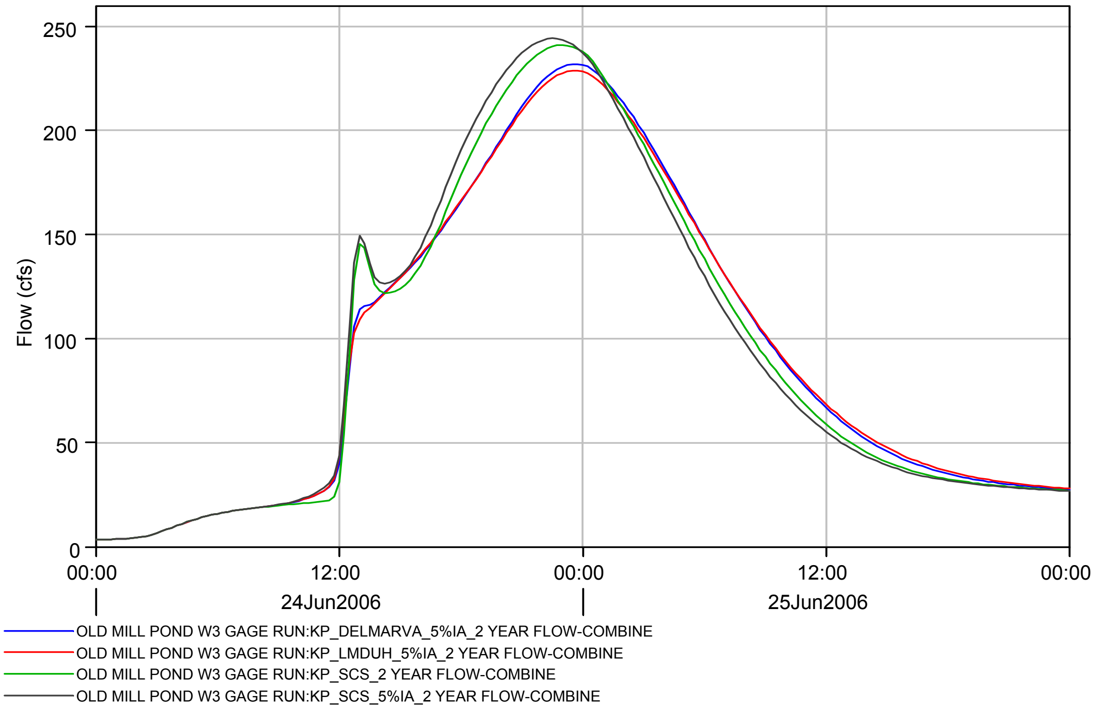


Figure 166: Hydrograph comparison for Wreck Pond Brook at Station W3 for NRCS 2-year storm event

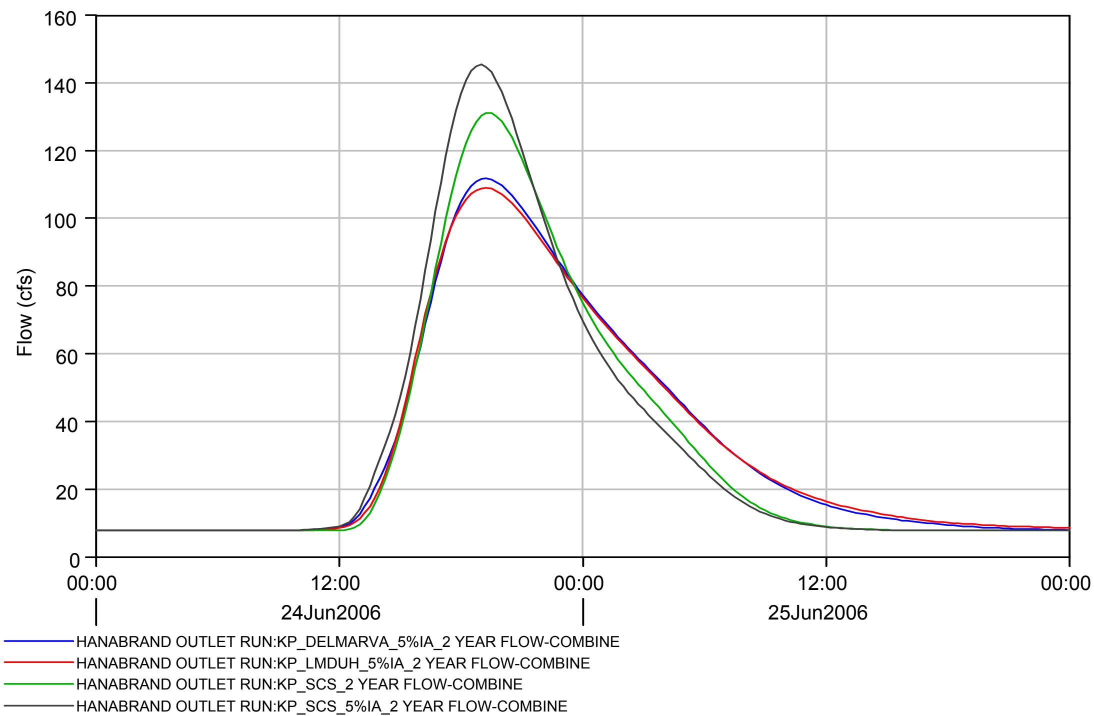


Figure 167: Hydrograph comparison for Hannabrand Brook at Station W2 for NRCS 2-year storm event

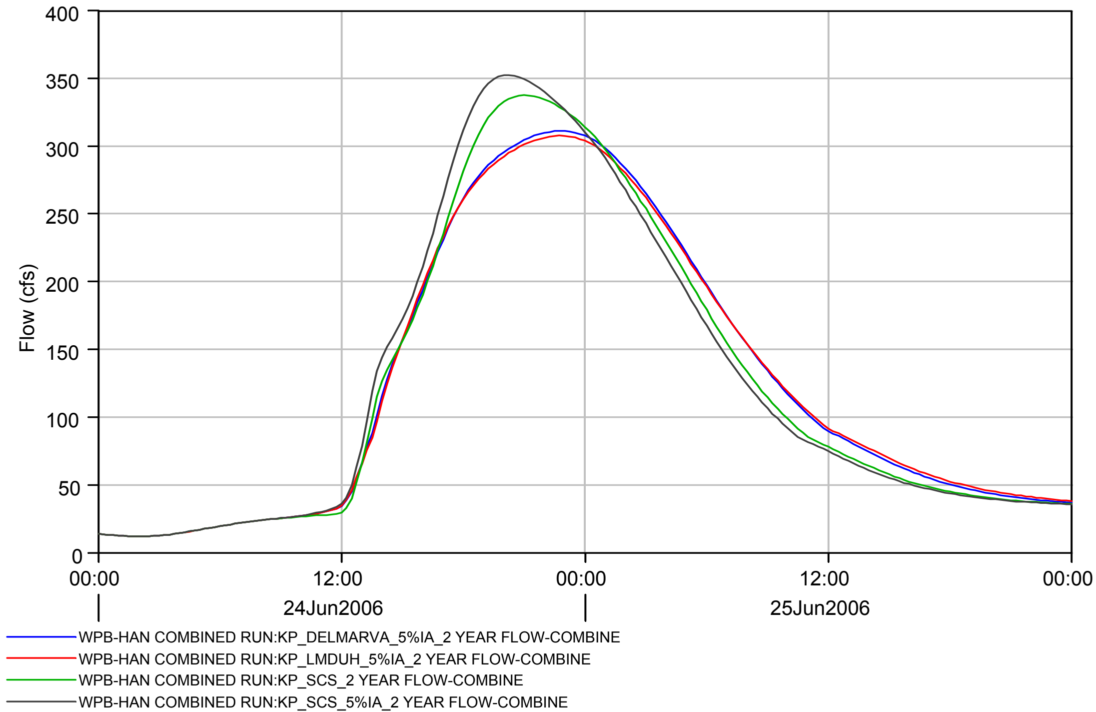


Figure 168: Hydrograph comparison for combined flows of Wreck Pond Brook and Hannabrand Brook for NRCS 2-year storm event

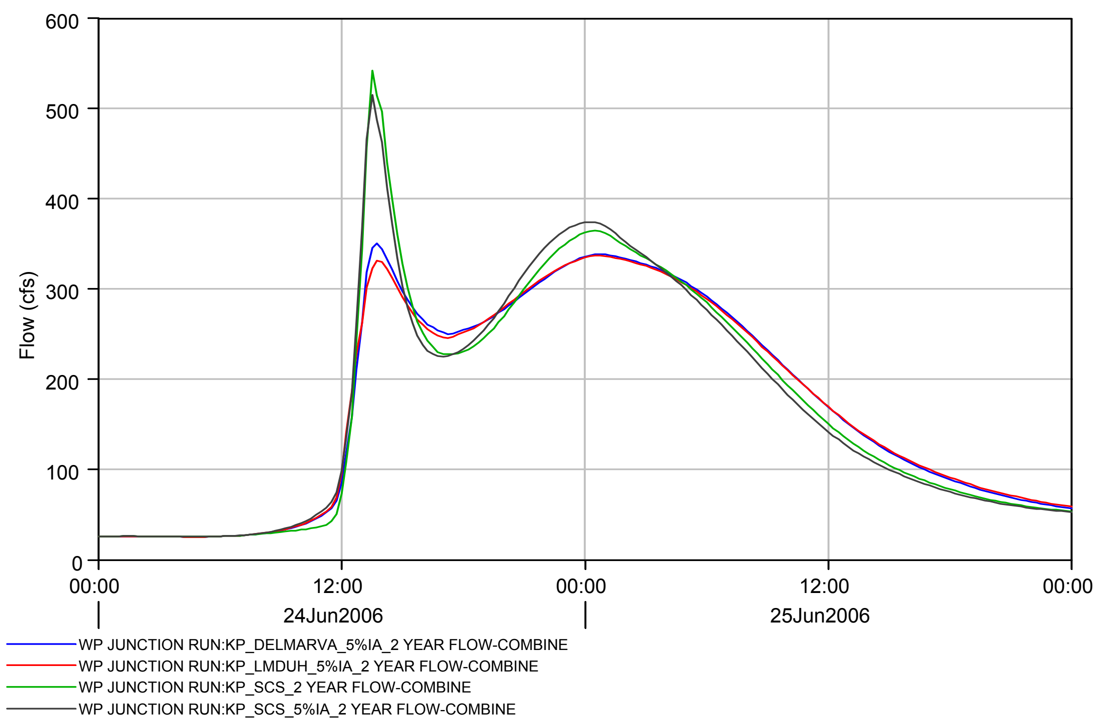


Figure 169: Hydrograph comparison for combined flows into Wreck Pond for NRCS 2-year storm event

5.1.6 Critical precipitation for subwatersheds in WPBW

As described by Woodward et. al, (2003) and Lim et. al,(2006), the 5% Ia/S ratio does not always produce higher direct runoff. The 5% Ia/S ratio produces higher runoff within a certain range of precipitation for a given land use and soil combination (CN Value). If the precipitation level exceeds the upper bound (critical precipitation value), the 20% Ia/S ratio produces higher direct runoff. As can be seen from the table below, all the subwatersheds (except the lower subwatershed) within WPBW have critical precipitation values between the 2-year and 10-year storm event.

Table 55: Percentage difference in flows for 20% Ia/S ratio vs. 5% Ia/S ratio

Hydrologic Element	**Percent difference (%) in flow values 20% Initial abstraction Vs. 5% Initial abstraction**				
	LMDUH 100 year	**LMDUH 50 year**	**LMDUH 25 year**	**LMDUH 10 year**	**LMDUH 2 year**
W6 – Martin's Rd	14.26	13.57	11.76	8.55	-8.05
W9 – Hurley's Pond	13.99	13.31	12.08	8.94	-6.55
W7 – Glendola Rd	13.93	13.88	13.39	11.27	-2.40
W1 – Waterford Glen	13.25	13.12	12.63	10.54	-1.05
W3 – Old Mill Rd	13.07	12.61	12.52	10.63	-1.03
W5 – Baileys Corner Rd	14.62	13.63	12.21	7.64	-13.83
W2 – Old Mill Rd	13.85	13.10	11.95	8.52	-8.43
Combined discharge below Old Mill Rd	13.34	12.53	12.53	10.51	-1.94
Sea Girt/Spring Lake/Spring Lake Heights	11.22	11.64	11.66	11.09	5.05
W8 – Golf Courses	11.42	12.13	11.73	8.33	0
Combined inflows to Wreck Pond	13.00	12.65	11.45	10.05	2.62

5.1.7 Peak flow analysis

The following Figure 170 shows the runoff hydrograph for Hannabrand Brook and Wreck Pond Brook before the merge point. The combined hydrograph (green-colored) after the merge point also is shown for the NRCS 100-year storm event using a 5% Ia/S and the percentage curve from the Delmarva unit hydrograph. As can be seen, the hydrographs from the Hannabrand Brook and Wreck Pond Brook peak within close time periods. The time interval bracketing the peak flow is sufficient to cause the peaks to merge, making the situation worse.

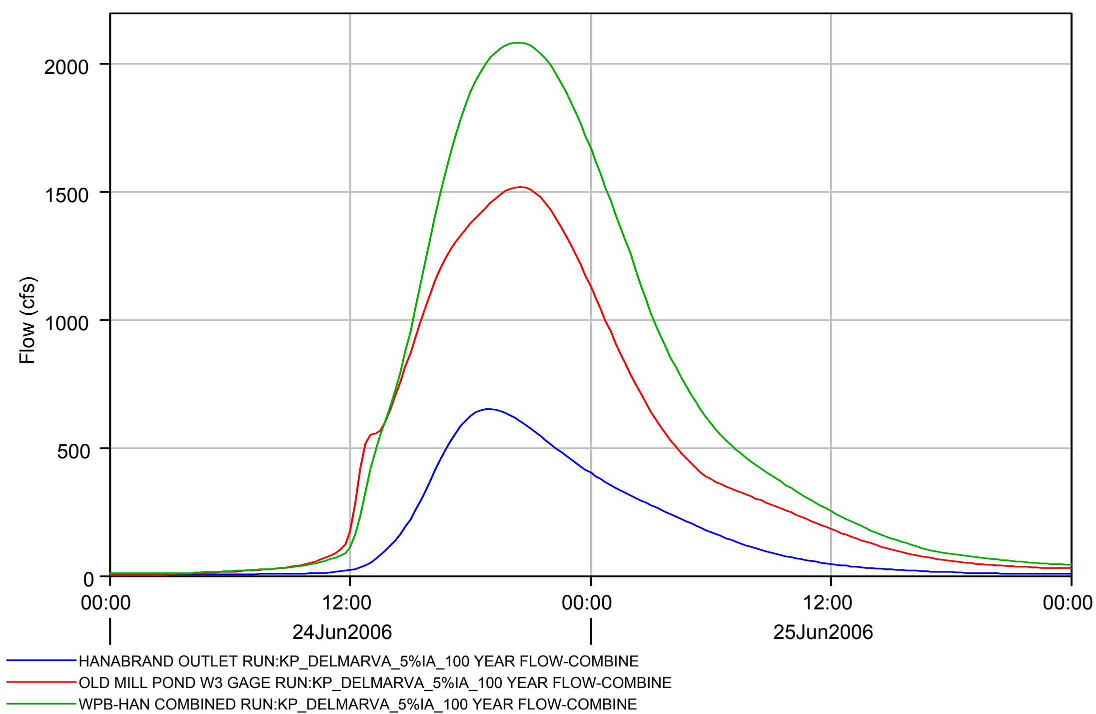


Figure 170: Hannabrand Brook and Wreck Pond Brook before merge and combined flow after merge.

Figure 171 shows the runoff hydrograph for several hydrologic elements contributing to Wreck Pond using a 5% Ia/S and percentage curve from the Delmarva unit hydrograph. The pink-colored hydrograph for WP Junction shows the cumulative effect of all the hydrographs contributing to flows into Wreck Pond. It can be seen from the following hydrograph that flows into Wreck Pond have two peaks for 100-year storm event with the second peak higher than the first peak. Also, the time interval between the two peaks is more than 10 hours. Therefore, it is likely the residents will be affected twice for the one flooding event. It can be seen the first peak was caused due to flows from the subwatershed W8 and lower watershed (Spring lake, Spring lake heights, and Sea Girt). The second peak was caused due to flows from the Hannabrand Brook and Wreck Pond Brook draining the upper watershed (Wall Township).

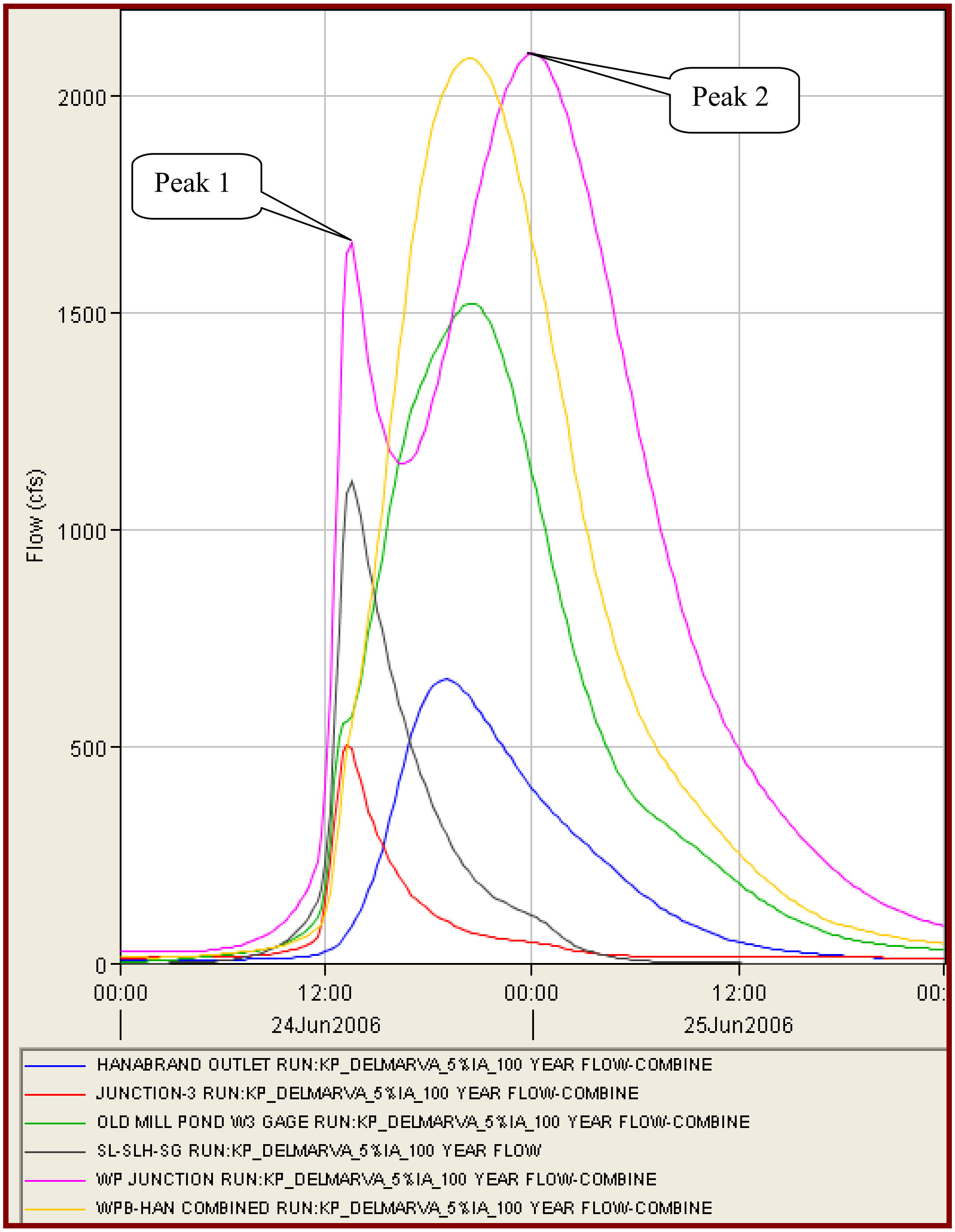


Figure 171: Hydrographs contributing to flows into Wreck Pond

5.2 Floodplain Analysis

The floodplain developed for Wreck Pond Brook and Hannabrand Brook (red-colored floodmap) was overlaid with the FEMA Q3 data (green line). The Q3 data was produced to roughly approximate (in digital format) the floodplain limits of the effective hardcopy FIRM maps (email communication, Mr. H. Rimawi, Medina consultants). The FEMA floodplain limits were noticeably different at several locations. Figures 172 and 173 show screenshots of several locations where the FEMA Q3 data and this project's floodplain data do not coincide.

Figure 172: FEMA Q3 data (green line) overlaying floodplain delineated (red polygon) - location on Hannabrand Brook

Figure 173: FEMA Q3 data (green line) overlaying floodplain (red polygon) for Albers Pond

However, in discussions with Mr. H. Rimawi (Medina Consultants) and Mr. R. Einhorn (Department of Homeland Security, FEMA), it was found the floodplain limits were being updated and remapped using existing flood studies and 2-ft contours provided by Monmouth County Office of GIS (Mr. J. Brockwell). As a result, the DFIRM was created using detailed elevation information based on 2-ft contours and provided a better match to modeled floodplain compared to the FEMA Q3 data. No new riverine study was conducted for the streams in WPBW. A copy of the existing database was sent to office of Dam Safety and Flood Control at NJDEP and was reviewed. The review of this copy (FEMA, Flood Insurance Study Number 34025CV001A – Revised January 11, 2008) from NJDEP's office of Dam Safety and Flood Control program revealed the flow of 2940 cfs was used as a peak discharge of 1-percent chance upstream of Old Mill Road. Some hand written flow information was available which matched the flows from supplemental studies for the Wreck Pond Brook and Hannabrand Brook conducted by the NJDEP. The flow value in the flood insurance studies (FIS) was based on the hydrologic and hydraulic analyses in the FIS report created in August 1976 by Tippetts-Abbett-McCarthy-Stratton for the FIA under contract No. H-3733.

Figure 174: DFIRM (yellow line) overlaying modeled floodplain (red polygon)-location on Hannabrand Brook

Figure 175: DFIRM (yellow line) overlaying modeled floodplain (red polygon) for Albers Pond

Supplemental studies were conducted by NJ State for the study area. The existing flood maps and flood hazard studies were obtained from the NJDEP. The existing profiles were determined using HEC-2 and the floodway was determined by Method 1 using a 0.2 foot rise criterion. Tables 56 and 57provides the comparison of Manning's roughness values and contraction and expansion coefficients for the existing study and this research study. Manning's roughness value was used for computing friction loss and the contraction and expansion loss coefficients were used for computing the transition loss in Equation 15.

Table 56: Friction loss and Transition loss coefficients ('n' and 'C') for Wreck Pond Brook

Wreck Pond Brook		N_{CH}	N_L	N_R	C_C	C_E
Station 0 - 6270	Existing flood study	0.03	0.045	0.045	0.3	0.5
	Current study	0.035	0.05 to 0.15	0.05 to 0.15	0.1	0.3
Station 6270 – 16288	Existing flood study	0.03	0.055	0.055	0.3	0.5
	Current study	0.035	0.05 to 0.15	0.05 to 0.15	0.1	0.3
Station 16288 – 25173	Existing flood study	0.03	0.045	0.045	0.3	0.5
	Current study	0.035	0.05 to 0.15	0.05 to 0.15	0.1	0.3

Table 57: Friction loss and Transition loss coefficients ('n' and 'C') for Hannabrand Brook

Hannabrand Brook		N_{CH}	N_L	N_R	C_C	C_E
Station 2946 - 13822	Existing flood study	0.035	0.075	0.075	0.1	0.3
	Current study	0.035	0.05 to 0.15	0.05 to 0.15	0.1	0.3
Station 13822 – 15573	Existing flood study	0.035	0.06	0.06	0.1	0.3
	Current study	0.035	0.05 to 0.15	0.05 to 0.15	0.1	0.3
Station 15573 – 16828	Existing flood study	0.035	0.055	0.055	0.1	0.3
	Current study	0.035	0.05 to 0.15	0.05 to 0.15	0.1	0.3

The flow values from the supplemental flood studies are compared with flow values used in this study. The floodway and flood hazard area maps for the supplemental studies of the Wreck Pond Brook and Hannabrand Brook are available in Appendix F.

Table 58: Flow comparison for supplemental flood study and current study for Wreck Pond Brook

	Supplemental study		**Current study**	
Location	**Q_{100} (cfs)**	**Q_{50} (cfs)**	**Q_{100} (cfs)**	**Q_{50} (cfs)**
WPB at Martins Road	430	350	569	441
WPB at Downstream of Garden State Parkway North	1020	850	569	441
WPB at Hurleys pond Dam	1020	850	1009	792
WPB at Route 138	1300	1070	1009	792
WPB at Glendola Road	1430	1180	1256	965
WPB at Bentz Road	1430	1180	1256	965
WPB at downstream of Allaire Road	1440	1200	1513	1153
WPB at Route 35	1440	1200	1513	1153
WPB at Old Mill road	1440	1200	1497	1147

5.3 *Flood Control Analysis*

Figure 176 compares the inflow hydrographs into Wreck Pond for the existing RSWMP model and the revised model developed in this study. The revised model predicts an early peak compared to the existing RSWMP model. The analysis is explained in the model calibration and verification section in Chapter 4.

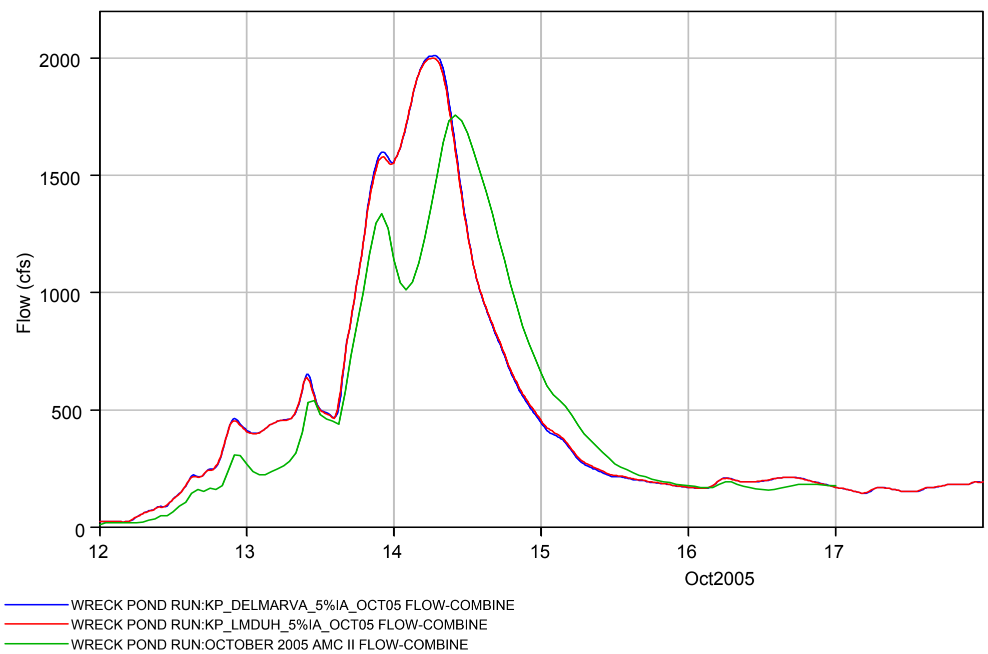


Figure 176: Hydrograph comparing existing RSWMP model and revised model output for October 2005 storm event in Wreck Pond

When zoomed near the peak in Figure 176, the difference in the peak flow timing prediction was 3.5 hours using the existing model and revised model in this study. This difference in peak flow is shown in Figure 177.

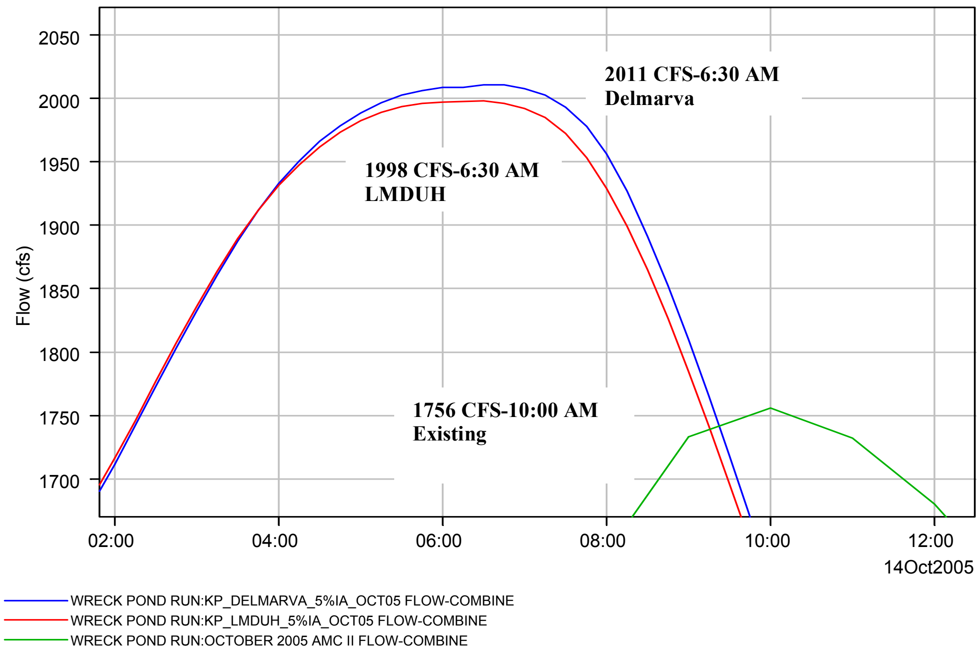


Figure 177: Hydrographs zoomed near peak for October 2005 storm for existing and revised hydrologic model

5.3.1 Online Pond detention for Wreck Pond Brook

As discussed earlier in the Dissertation, the difference in peak flow timing for Hannabrand Brook and Wreck Pond Brook is an average of 45 minutes, with the peak from Hannabrand Brook arriving earlier than peak from Wreck Pond Brook. Consequently, any attenuation provided from Wreck Pond Brook will allow more time between the peaks and will result in lower peak flows into Wreck Pond. The analysis was performed for the online ponds on Wreck Pond Brook. These results are discussed below.

Hurley's Pond:

Figure 178 shows the water surface profiles computed by RAS in Hurley's Pond just before Hurley's Pond Road. PF1 is the water surface elevation for the NRCS 100-year storm (1009 cfs @ station 28104.53) and PF2 is the water surface elevation for the

October 2005 storm (862 cfs @ station 28104.53). PF1 gives WSE as 58.86 feet and PF2 gives WSE as 58.80 feet. Hence, there is a difference of 0.06 feet for the flow difference of 147 cfs.

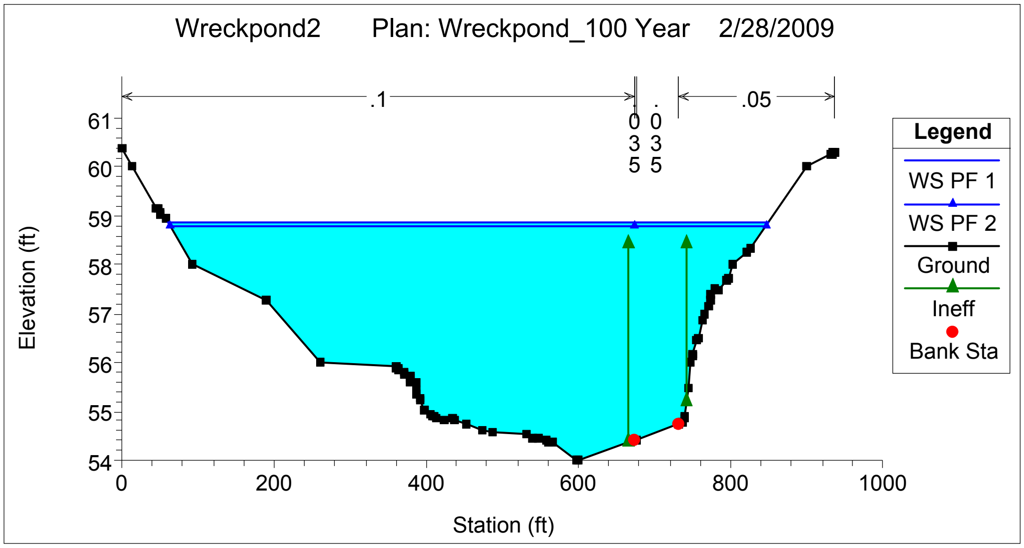


Figure 178: Water surface profiles at station 28104.53

Figure 179, yellow lines show the draft DFIRM lines for a 100-year flood. The blue line shows the contour for an elevation of 60 feet. Therefore, a feasible option for runoff volume management would be to use storage available from 58.86 ft to 60 ft by modifying the outlet structure of Hurley's pond. However, it will be necessary to increase the roadway elevation to achieve increased storage capacity.

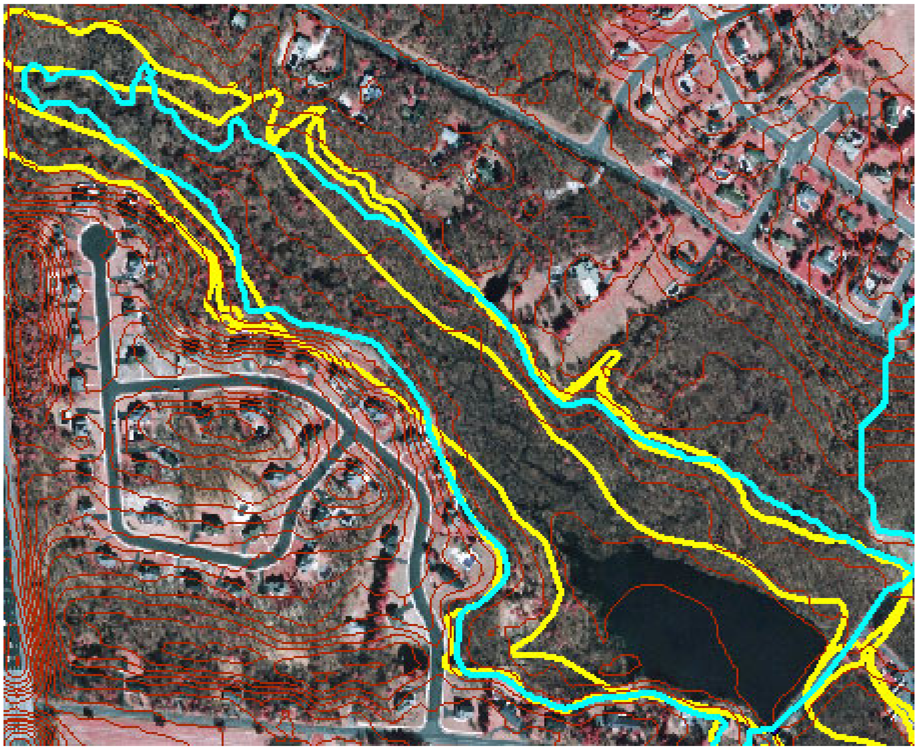


Figure 179: DFIRM line for 1% chance and storage analysis at Hurley's pond

Keller's Pond:

Figure 180 shows the water surface profiles computed by RAS for Keller's Pond. PF1 is the water surface elevation predicted by the NRCS 100-year storm (358 cfs @ station 37857.55) and PF2 is the water surface elevation for the October 2005 storm (145 cfs @ station 37857.55). PF1 gives WSE as 86.75 feet and PF2 gives WSE as 86.51 feet. Therefore, there is a difference of 0.24 feet for the flow difference of 213 cfs.

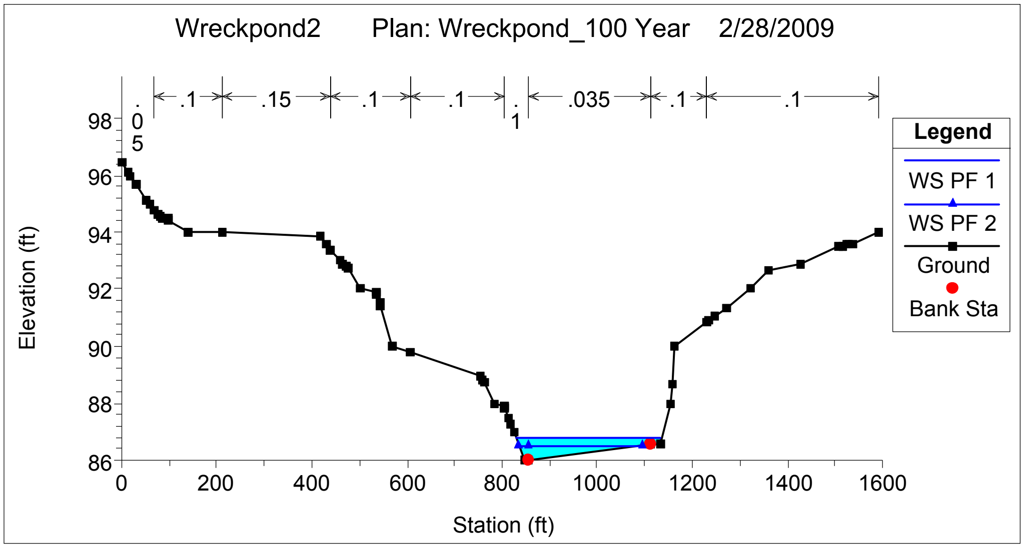


Figure 180: Water surface profiles at station 37857.55

In the following figure 181, the yellow lines show the draft DFIRM lines for the 100-year flood. The blue line shows the contour for the 90-foot elevation. A feasible stormwater management option would be to use storage available from 86.75 ft to 90 ft by modifying the outlet structure of Keller's Pond. Currently, there is an ongoing feasibility study at the Keller's Pond site performed by Wall Township, NJ to modify the pond outlet for water quality benefits.

Figure 181: DFIRM line for 1% chance and storage analysis at Keller's pond

Osborne Pond:

Figure 182, shows the water surface profiles computed by RAS for Osborne Pond just before the Allaire Road intersection. PF1 is the water surface elevation for NRCS 100-year storm (1510 cfs @ station 13194.82) and PF2 is the water surface elevation for the October 2005 storm (1422 cfs @ station 13194.82). PF1 gives the WSE as 28.40 feet and PF2 gives the WSE as 28.23 feet. The difference is 0.17 feet for the flow difference of 88 cfs.

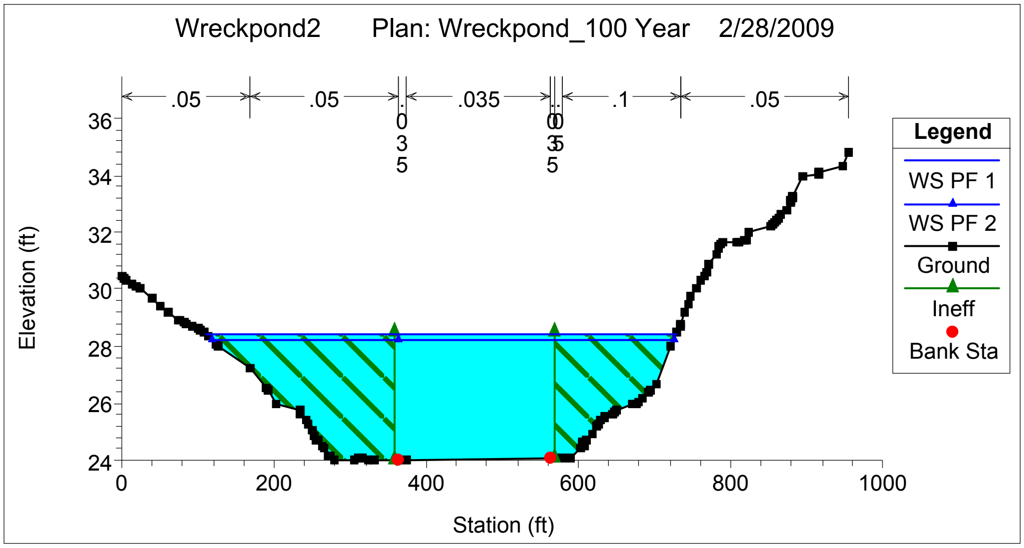


Figure 182: Water surface profiles at station 13194.82

In Figure 183, yellow lines show the draft DFIRM lines for 100-year flood. The blue line shows the contour for an elevation of 28 feet. As can be seen from the figure below, there are houses built on left bank of Wreck Pond Brook and Albert's Pond Creek (almost up to the edge of the 100-year floodplain). In this outcome, there is not enough flexibility to increase the available storage without increasing the risk of flooding to residents. Therefore, any future flood control project at this location would need to proceed with significant caution.

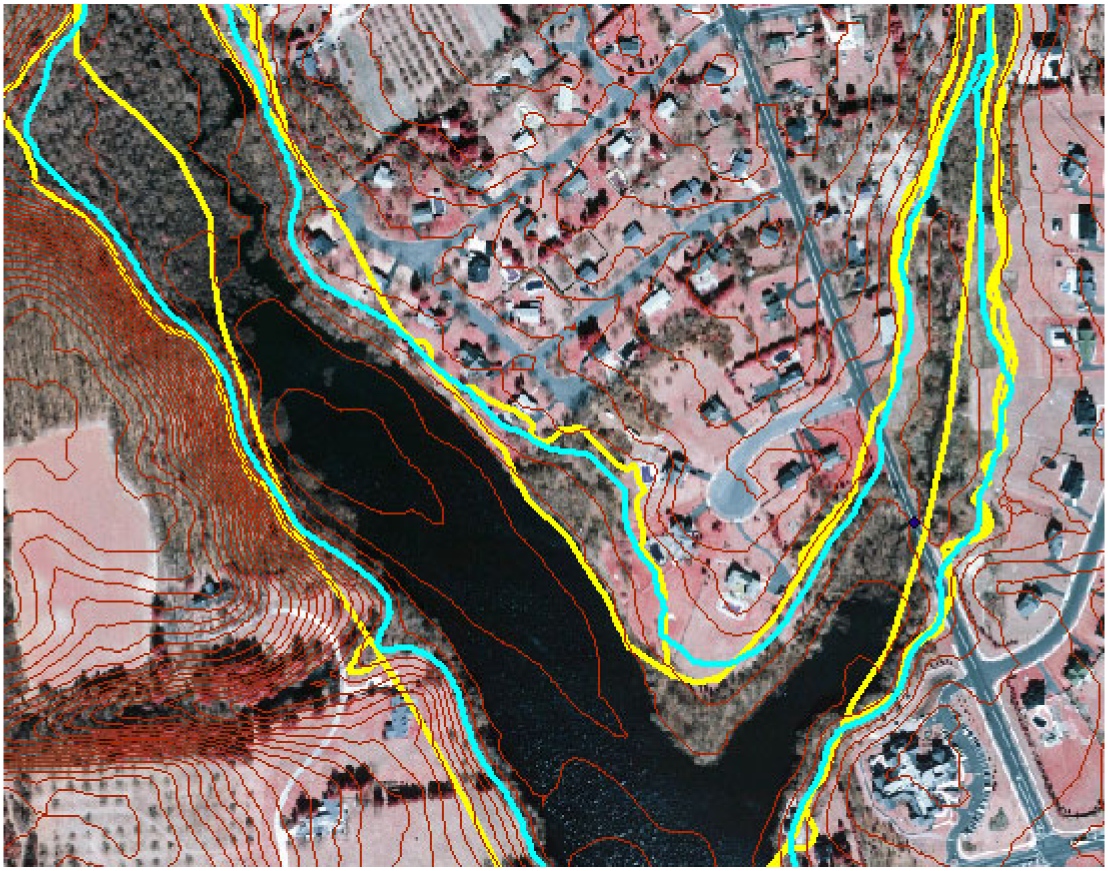


Figure 183: DFIRM line for 1% chance and storage analysis at Osborne pond

Old Mill Pond:

Figure 184 shows the water surface profiles computed by RAS at the inlet of Old Mill Pond just downstream of Ocean Road. PF1 is the water surface elevation for the NRCS 100-year storm (1513 cfs @ station 10437.18) and PF2 is the water surface elevation for the October 2005 storm (1423 cfs @ station 10437.18). PF1 gives WSE as 14.79 feet and PF2 gives WSE as 14.70 feet. There is a difference of 0.09 feet for the flow difference of 90 cfs.

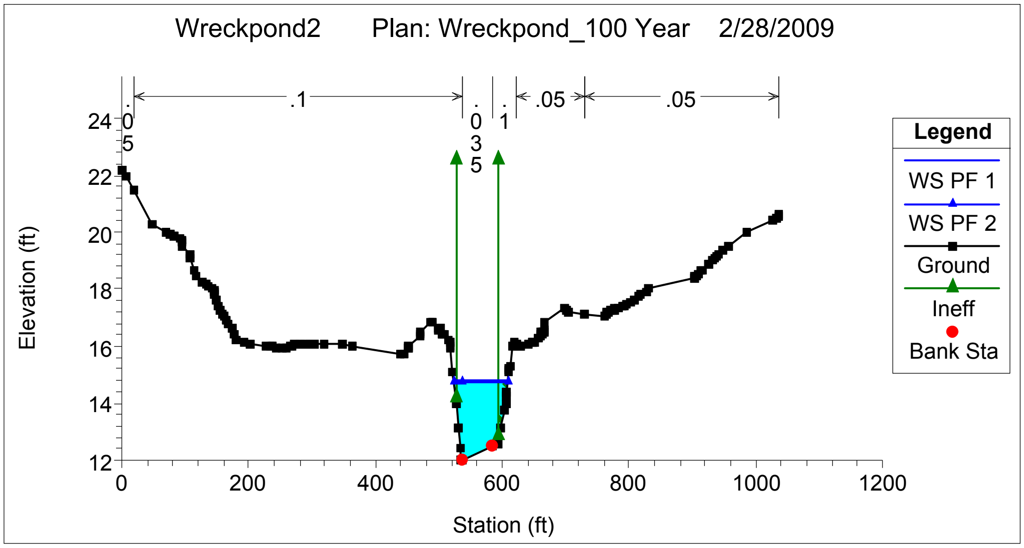


Figure 184: Water surface Profile at station 10437.18

In Figure 185, the yellow lines show the draft DFIRM lines for the 100-year flood. The blue line shows the contour for the 14-foot elevation. As can be seen from Figure 185 below, the restaurant built on the left lower corner of Old Mill Pond and several houses upstream of Ocean Road are at the perimeter of the 100-year floodplain. Any additional proposed storage increases would add significant risk of flooding to these properties. As a result, there is not sufficient flexibility to increase the available storage without increasing the risk of flooding to properties. Any future flood control project at this location would need to be evaluated with significant caution.

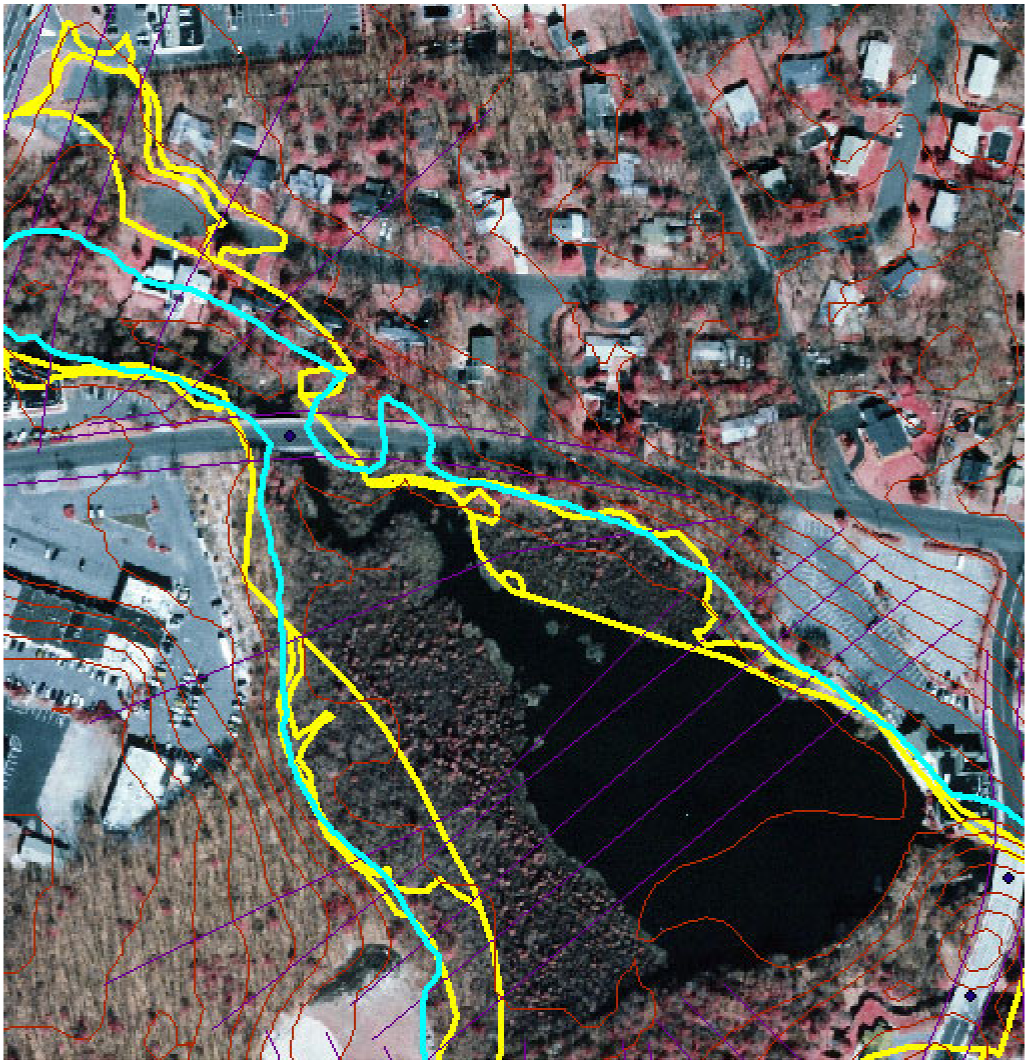


Figure 185: DFIRM line for 1% chance and storage analysis at Old Mill Pond

5.3.2 Pumping

Figures 186-187, shows the effects of a model simulation using 4 hypothetical pumps, each with a capacity of 50 cfs and set to begin at an elevation corresponding to 1000 cfs in the Wreck Pond Watershed. Figure 186 shows the simulation for a 50-year NRCS storm event and Figure 187 shows a simulation for a 100-year NRCS storm event. The 1000 cfs is the flow between the 10-year and 25-year storm events.

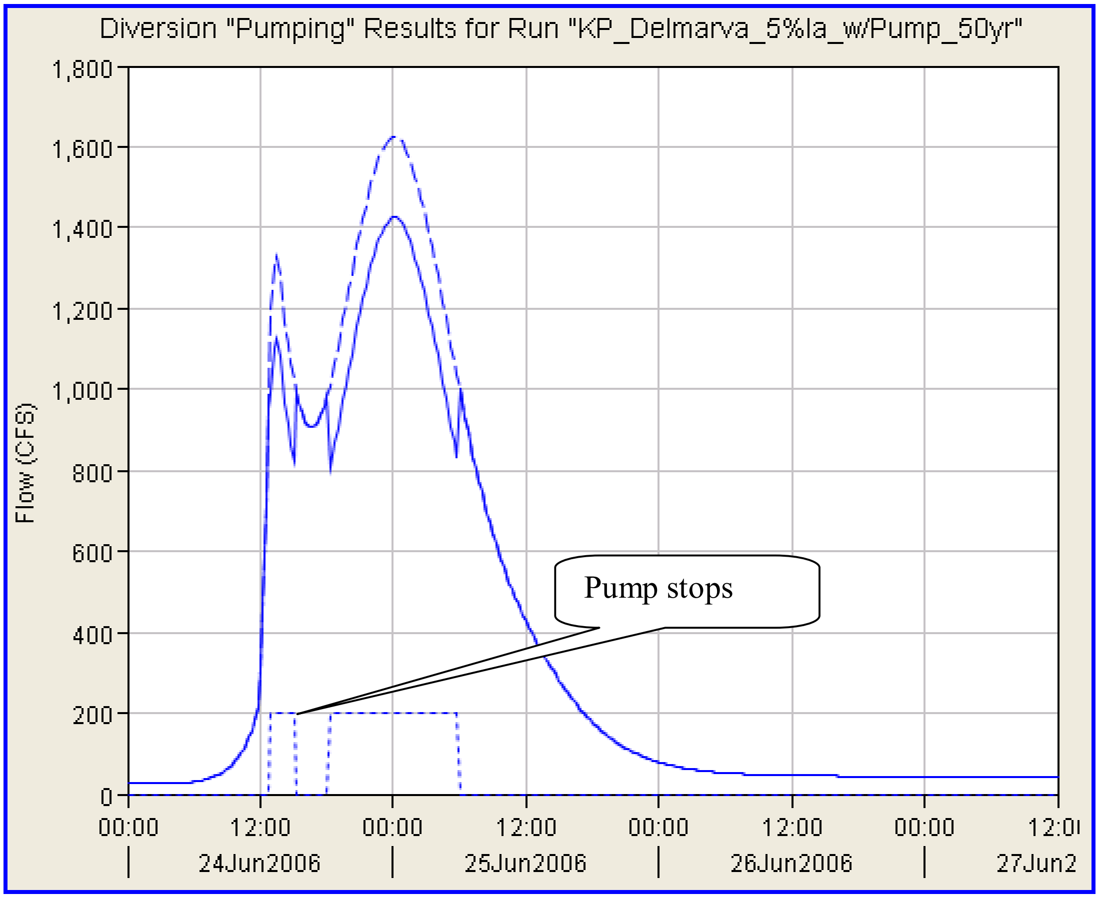


Figure 186: Pumping effect for 50-year NRCS storm event

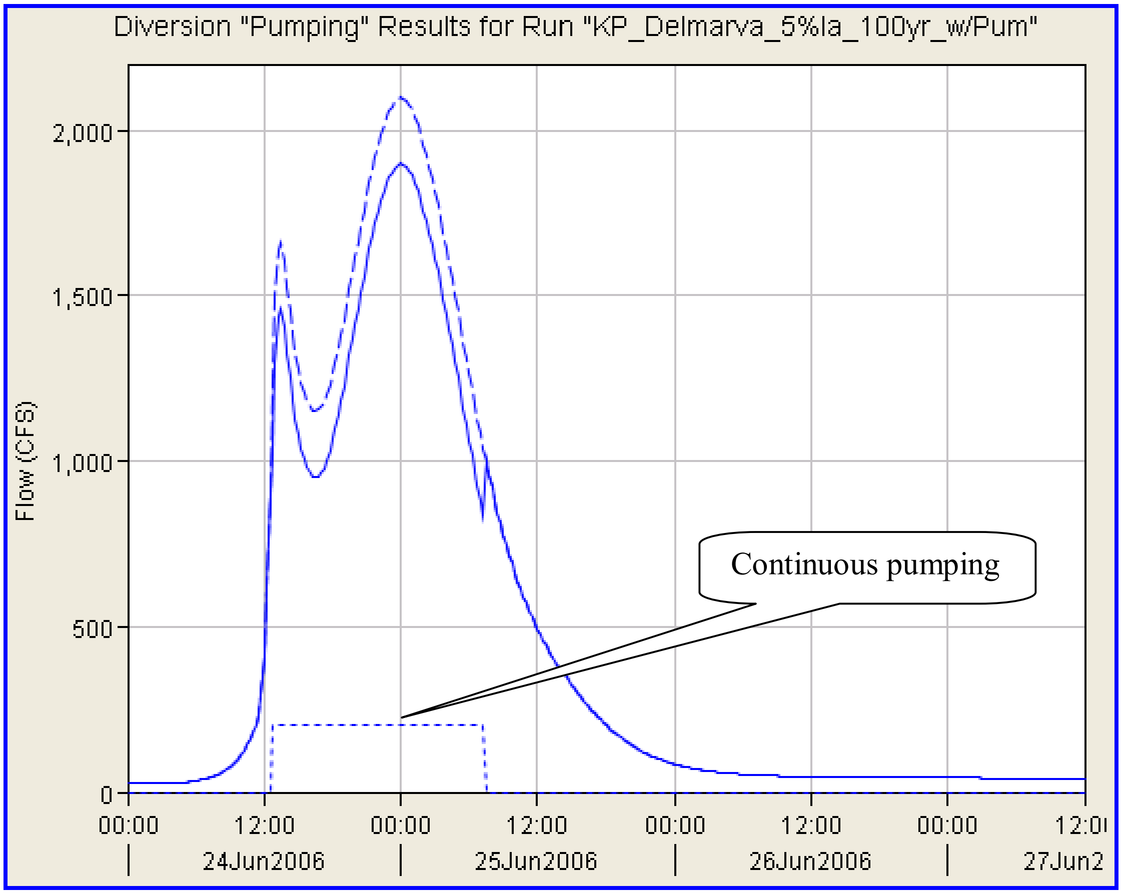


Figure 187: Pumping effect for 100-year storm event

As can be seen in Figures 186-187, once the pumps are selected based on cost and final design requirements, the continuous pumping system also can be used as a flood warning system if the pumping stops at a specific elevation. In these figures it can be seen the pumping ceases for the 50-year storm event, whereas pumping is continuous for the 100-year storm event. It is possible to use such a mechanism in watersheds like Wreck Pond because there are two peaks for real storm events.

5.3.3 Combined effect of Detention and Pumping

The feasible detention location was simulated using a hypothetical outlet structure at Keller’s Pond and Hurley’s Pond as discussed in section 5.3.1 and focusing on flood control using online ponds. The modified stage-storage-discharge relationship is provided in Appendix A, B and C. The result of individual detention is shown in the Figures 188-189. The following hydrographs (Figures 188-190) were determined for the NRCS 100-year storm event.

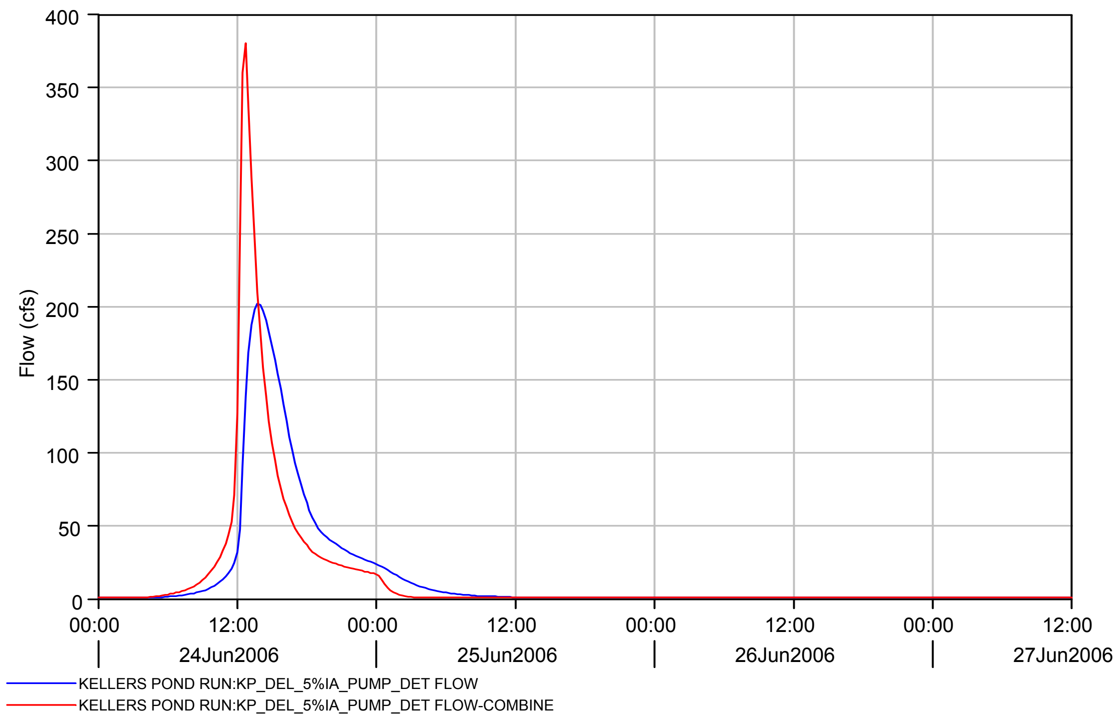


Figure 188: Hydrograph attenuation by modifying outlet structure of Keller's Pond

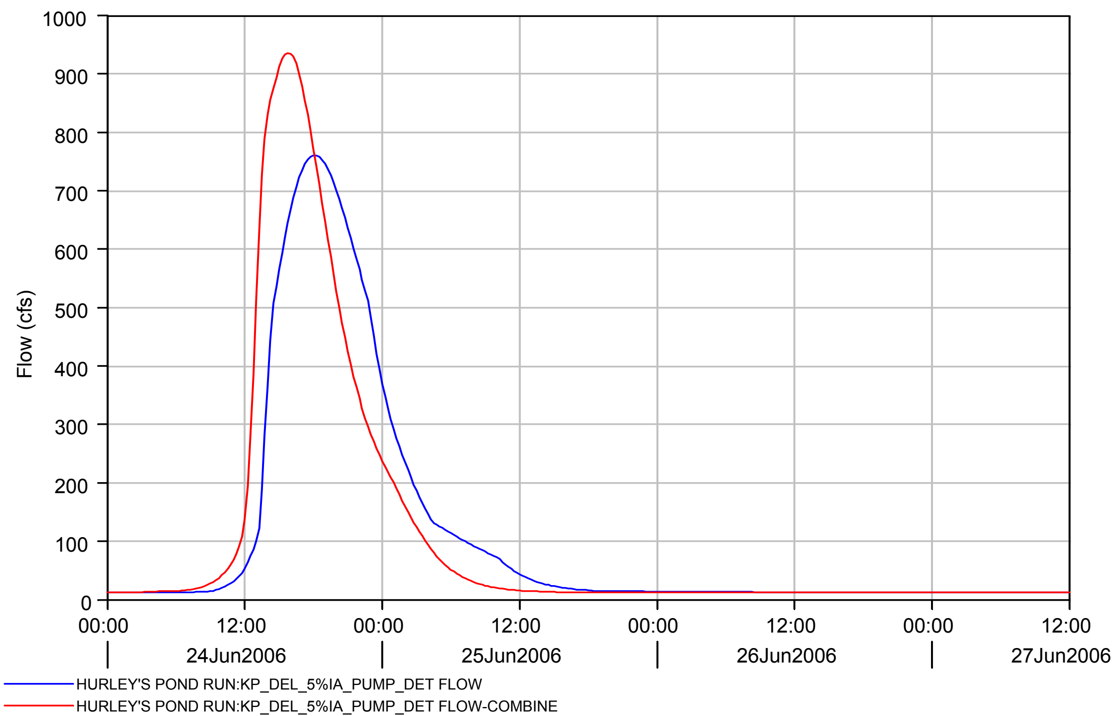


Figure 189: Hydrograph attenuation by modifying outlet structure of Hurley's Pond

Figure 190, shows the combined effect of attenuation using detention and pumping for Wreck Pond. In Figure 190 the attenuation provided using detention and pumping was 16.8 %, i.e., reduction in peak flow from 2098 cfs to 1746 cfs. This methodology is intended to serve as an example for preliminary investigations on hydrograph attenuation achieved in Wreck Pond by work performed anywhere in the watershed. This analysis is based on limited information and any future design and construction projects should include detailed investigation of potential modifications to current flow characteristics in Wreck Pond Watershed.

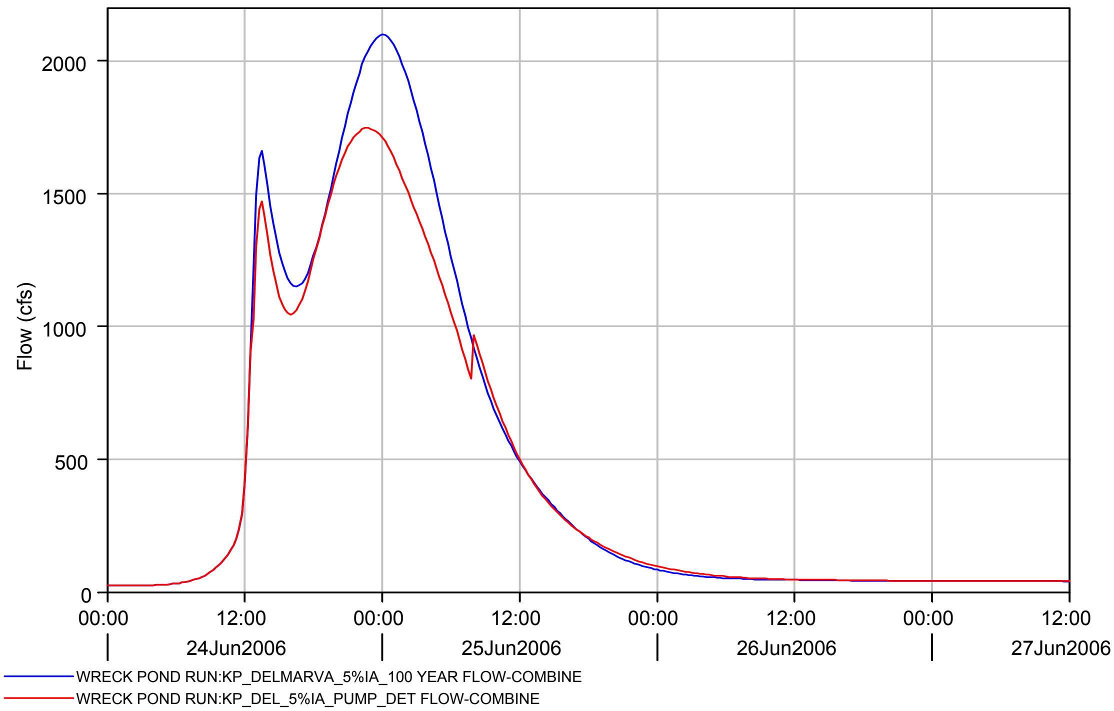


Figure 190: Combined effect of detention and pumping on Hydrograph in Wreck Pond

6 CONTEXT

The first section of this chapter summarizes the findings of this study. A review of the key questions guiding this research is provided in the subsequent section. Control strategies to reduce flooding in WPBW are discussed. Finally, recommendations for future research and implementation are discussed in the final section of this chapter.

6.1 Conclusion

Use of detailed elevation information (e.g. LIDAR) via ArcGIS in HEC-RAS and HEC-HMS can improve the quality of the watershed model in coastal areas of NJ. The modeling approach used in this study was a novel combination of available interfaces (ArcHydro, HEC-GeoHMS, and HEC-GeoRAS) which linked HEC-HMS and HEC-RAS to the GIS environment. Hydrological parameters (flow, rainfall) and physical properties (elevation, soil type, land use) of the Wreck Pond Watershed were added to the watershed model to improve the model output with field verified spatial (e.g., Landuse/Landcover, drainage divides etc) and temporal (e.g., Stream stage) information in the watershed. This modeling approach can result in substantial cost savings compared to other field measurement and modeling studies for the watershed carried out by other consulting groups and FEMA. The combination of HEC-HMS, HEC-RAS and GIS models provided a new, accurate method for modeling and visualizing the spatial and temporal distribution of the watershed response for a given storm event in terms of runoff and flood inundation. The modeled watershed response depended on several inherent parameters in the methods chosen in HEC-HMS and HEC-RAS applications.

SCS methods are widely used in hydrologic studies. Specifically, the SCS curve number method for runoff calculations and the SCS dimensionless unit hydrograph for the convolution of rainfall excess are necessary to compute the runoff hydrograph for a storm event. It is important to understand that standard values chosen as a 20% initial abstraction ratio in the SCS curve number method and peak rate factor of 484 in the SCS dimensionless unit hydrograph are the values used to represent a variety of rainfall-runoff responses for several watersheds studied by SCS. Significant deviation from this average response characterization was observed in the WPBW. The watershed model developed to simulate the observed characteristics of the rainfall-runoff relation for WPBW provided a better fit by using a 5% initial abstraction ratio and a lower peak rate factor (e.g. 284). Therefore, low-lying coastal watersheds of NJ are more accurately modeled by the lower initial abstraction ratio and peak rate factor compared to the industry standard parameters.

Analysis of hydrograph features such as the rising limb and the peak and recession limb provided insight into catchment release time and initial abstraction and storage properties (Ia/S). Stream gage information and rainfall information were available for several locations in WPBW from spring, 2004 to fall, 2006. The rainfall-runoff analysis using the 'Event analysis method' described by Woodward et al., (2003), indicated lower values for the initial abstraction ratio for WPBW. The 5% initial abstraction ratio provided a better fit for peak discharges and peak timings for the October 2005 storm event. Use of a 5% initial abstraction ratio required the method of calculating conjugate curve numbers developed by Woodward et al., (2003) which were based on relations between $S_{0.20}$ and $S_{0.05}$ developed by Hawkins (1985). The difference in CN values resulted from the difference in potential maximum retention (S) values for the corresponding 20% and 5% initial abstraction ratios. There is a specific value of precipitation called "critical precipitation", below which the 5% initial abstraction ratio will result in higher calculated peak discharges. Accordingly, for a precipitation amount above the critical precipitation value, a 5% initial abstraction ratio will result in lower peak discharges compared to a 20% initial abstraction ratio. This value is in between the 2-year and 10-year NRCS storm event for all subwatersheds in WPBW.

Dewberry Inc, VA (2004), developed the peak rate factor of 230 for WPBW for NJDA under a study funded by NJDEP (personal communication, Steven Jacobus, NJDEP). This study verified use of the standard SCS dimensionless unit hydrograph with a peak rate factor of 484 results in unrealistically higher peak flow rates for WPBW. The discrepancy in the predicted and observed flow rates was due to the low-lying coastal relief within the watershed. Use of the Delmarva unit hydrograph with a PRF of 284 currently is recommended by NJDEP for coastal areas of NJ where applicable. Comparison of the watershed response using the unit hydrograph with PRF of 230 and 484 with the Delmarva unit hydrograph with PRF of 284 was done for 2, 10, 25, 50 and 100-year NRCS storm events. The result of the analysis indicated minor differences in peak discharges with the use of the 230 and 284 PRF values. Hence, even though the PRF of 230 is the more accurate representation for WPBW, this study does not recommend a change from the current recommendation of 284 unless future studies warrant otherwise. Any change in the PRF value from the present value would require significant overhaul of the prevailing practice at NJDEP and member municipalities. The results of this study have not indicated a change in the PRF value is warranted.

The combination of lower PRF (e.g., 284) and 5% initial abstraction ratio provided a better match to the observed response in terms of peak discharges and peak timings. The difference in peak discharges due to the use of standard parameters and the modified/recommended parameters were analyzed with varying storm recurrence intervals. The storms used in the sensitivity analysis were the 2, 10, 25, 50 and 100-year storms. No trend was observed in differences due to the recurrence interval.

The model next was calibrated and validated for varying storm and watershed conditions. The model could predict different antecedent moisture conditions, high frequency storms and low frequency storms. The storm event of June 24, 2006 was used to model AMC III and the storm event of October 2005 was used to model a low frequency storm.

The flood inundation area for the October 2005 storm event was developed using HEC-RAS version 4.0 Beta, HEC-GeoRAS 4.1.1 and ArcGIS 9.2. The flow values were obtained from the HEC-HMS model developed for WPBW. Both the flow values and flood inundation area for the October 2005 storm event were compared with the NRCS 100-year storm event. The results indicated a small difference in peak discharge values (with October 2005 storm with lesser values) and in flood inundation areas. Overall, an additional 43.2 acres were inundated for NRCS 100 year storm than for the October 2005 storm. The total inundation area predicted for the NRCS 100-year storm was 727.2 acres. Very little difference was observed in the flood stage between two storm events. This finding was attributed to the flat floodplain in the WPBW, which allowed the streams to connect and allowed floodwaters to move laterally without substantial increase in flood stage. Further investigation is necessary of the stage – discharge relationship within the Wreck Pond, including study of the tidal effect.

The stream response at the outlet of the Hannabrand Brook (station W2) and Wreck Pond Brook (station W3) within the two-year time period was available for six storm events that exceeded 1 inch. The analysis of the real-time elevation data provided peak flow timings at the outlets just before they merged downstream of the Old Mill Road culvert. The results indicated the peak flow from the Hannabrand Brook and Wreck Pond Brook occurred within a close time interval ranging from 0.2 to 1.17 hour with an average difference of 45 minutes. The peak discharges at Hannabrand Brook outlet occurred sooner than Wreck Pond Brook outlet at Old Mill Pond for all the storm events analyzed. However, due to the peak flow duration, it can reasonably be assumed the peak flows for Wreck Pond Brook and Hannabrand Brook merged at the same time and exacerbated the flooding for the areas downstream of the confluence. The peak flows from the lower portion of the watershed occurred much earlier in time and only the lower portion of the recession limb of the hydrograph added to the peak discharges from the upper watershed. As a result, the lower watershed did not contribute to peak discharges originating from the upper watershed. The combination of flows from streams within WPBW produced the two flow peaks in Wreck Pond for a single storm event. Consequently, two peak flows may cause flooding to occur two times for one storm event. However, due to the slightly lower value of the 1st peak, this peak occurrence can be utilized as an early warning of more floodwater to arrive in a few hours from the upper watershed. The time lag between the two flow peaks may provide a critical period of several hours for implementation of local emergency management plans.

The results from this study indicate that a uniform detention strategy may not be adequate to address flooding in WPBW. The control strategies may need to be redefined for lower watershed versus subwatershed for Hannabrand Brook and upper reaches of Wreck Pond Brook to reduce flood impacts in WPBW. One example of a management strategy is to construct online detention ponds on Wreck Pond Brook that are combined with pumping systems.

6.2 Key Questions

Five key questions guiding this research were discussed in Chapter 1. They are given here with responses gained from this research.

What are the regional parameters for characterizing the low-lying relief of coastal watersheds in NJ?

This study confirmed the use of the standard dimensionless unit hydrograph with a PRF value of 484 produced unrealistically high peak discharges. The Delmarva unit hydrograph with a PRF value of 284 was an acceptable value and should be used. Even though a PRF of 230 was a more accurate value for WPBW, this study did not find benefits justifying the change from 284 to 230. Such a changed would cause major overhaul of prevailing practices at NJDEP and member municipalities.

A 5% initial abstraction ratio should be used instead of the standard 20% ratio, as it provided a better match to the observed characteristics of the WPBW, specifically, the peak flow timings and peak discharges for flood events.

What is the contribution of GIS in the watershed modeling due to advances in computational speed and availability of spatial and temporal information?

This study demonstrated methodology is now ready as a prototype model for watershed analysis in coastal areas of NJ. This study validated the use of detailed elevation information (LIDAR) for first and second-order streams in coastal areas of NJ by using WPBW as an example. Results from this study do not support amending the FEMA DFIRM lines for WPBW because these lines yielded more conservative floodplain widths.

Is the general approach of a uniform detention strategy for the entire State good for WPBW?

The results from this research indicated a uniform detention strategy is not a good approach for WPBW. On the contrary, it may exacerbate the flooding in areas surrounding Wreck Pond and downstream of Old Mill Pond. Flood control strategies should be redefined for the lower watershed, the subwatershed for Hannabrand Brook, and the upper reach of Wreck Pond Brook. The model developed in this study should be included in any future development of flood control strategies for low-lying coastal areas in New Jersey.

Is distributed modeling using NEXRAD rainfall better than lumped modeling using rain gage information?

It cannot be determined from the existing literature if distributed modeling is applicable at this scale. Also, due to limited scope of this research it was decided to not to investigate to address this question.

What revisions to WPBW Regional stormwater management plan can be beneficial?

The study identified and recommended the following changes to the WPBW RSWMP that should benefit future stormwater analysis and implementation of projects.

- An initial abstraction ratio is recommended at 5% instead of 20% along with Delmarva unit hydrograph for future hydrologic analysis.
- Currently, there is insufficient justification based on model outcomes to adopt a PRF of 230. A change of PRF would cause a significant overhaul of prevailing practices at NJDEP and member municipalities. Future studies and data might otherwise change this recommendation.
- The study confirmed the need for flood control measures in WPBW, including but not limited to detention on Wreck Pond Brook and pumping from Wreck Pond.

6.3 Future Recommendations

The methods and approach developed in this study can be used for other watershed evaluations and can be combined with NEXRAD rainfall information to address unanswered key questions. WPBW, if found applicable, also can be used as a study area as significant amount of data is available for analysis.

Future hydrologic studies in coastal watersheds in NJ should analyze applicability of a 5% initial abstraction ratio together with the Delmarva unit hydrograph to further strengthen the findings of this modeling approach and to confirm its applicability for a much larger area outside of WPBW.

Further evaluation of elevation information (LIDAR) for streams higher than second order streams is necessary and should be extended to different watershed conditions.

For unmapped streams where no floodplain is delineated by FEMA, the floodplain may be delineated using the methods and approach described in this research as it provides faster results and substantial cost savings.

Member municipalities of WPBW should use the hydrograph generated using the model developed in this study combined with an overlay pre and post development hydrographs. This information is important in evaluating the impact of future projects in WPBW, especially if a proposed project might exacerbate the flooding at downstream locations.

Further investigation is necessary to develop the stage-discharge relation for Wreck Pond, including the tidal effect from the Atlantic Ocean. This study identified the importance and need for WPBW managers to design and implement the flood control measures and an emergency response plan using a regional approach.

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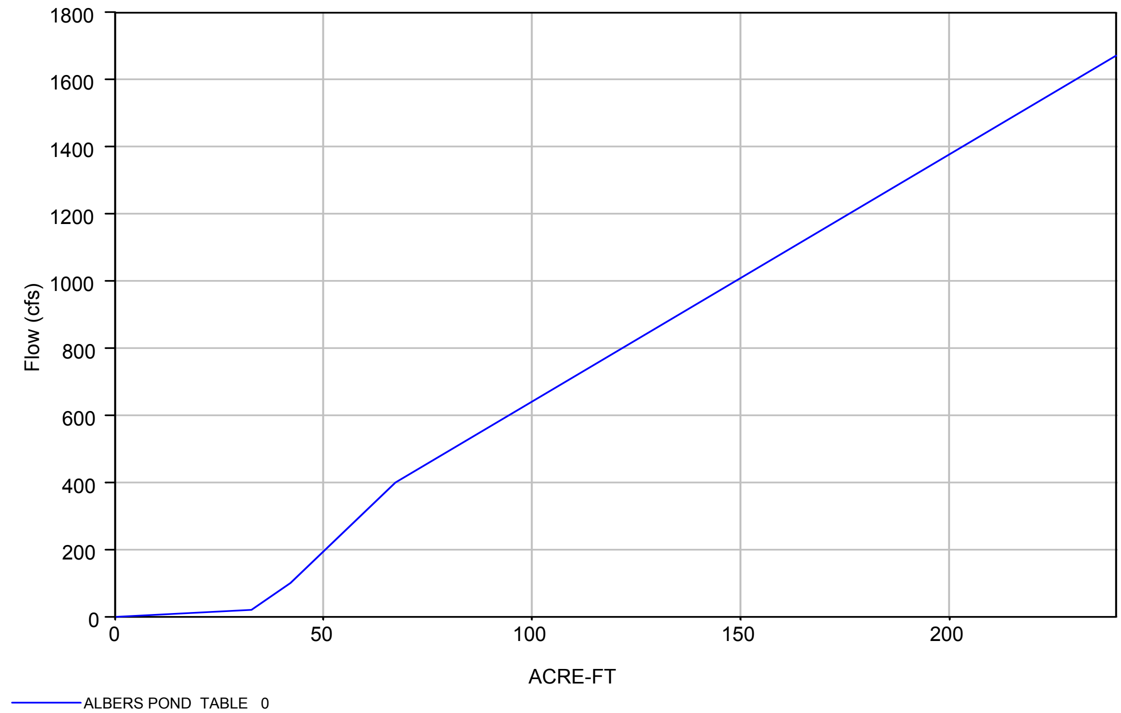
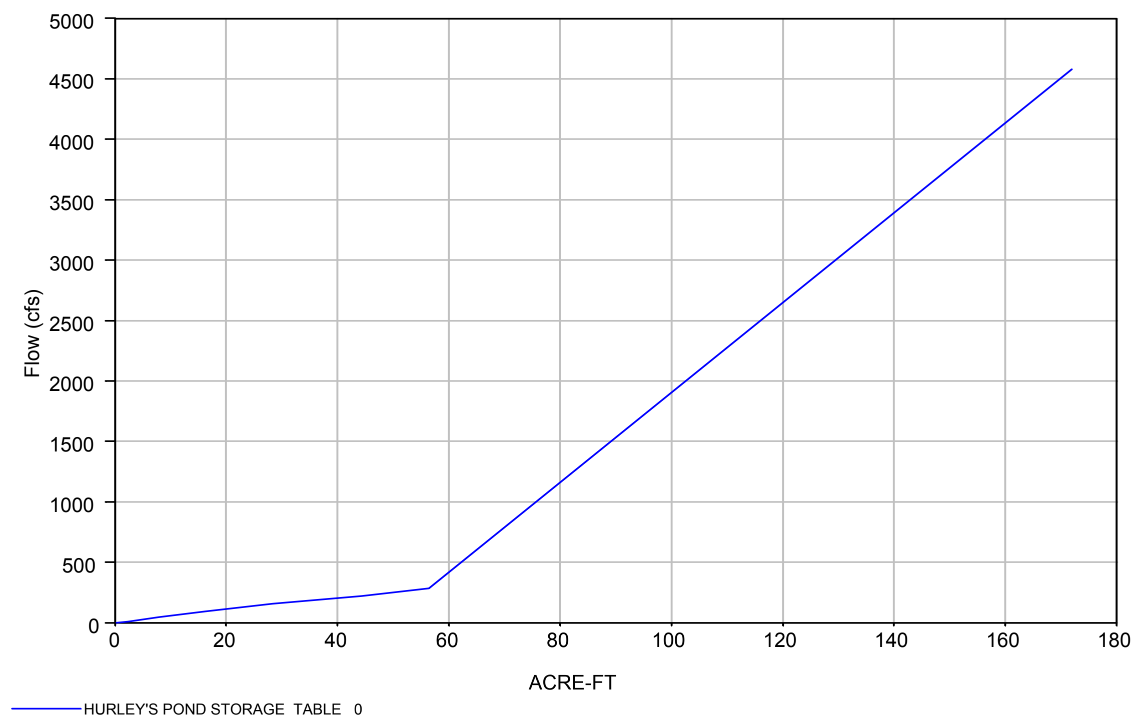
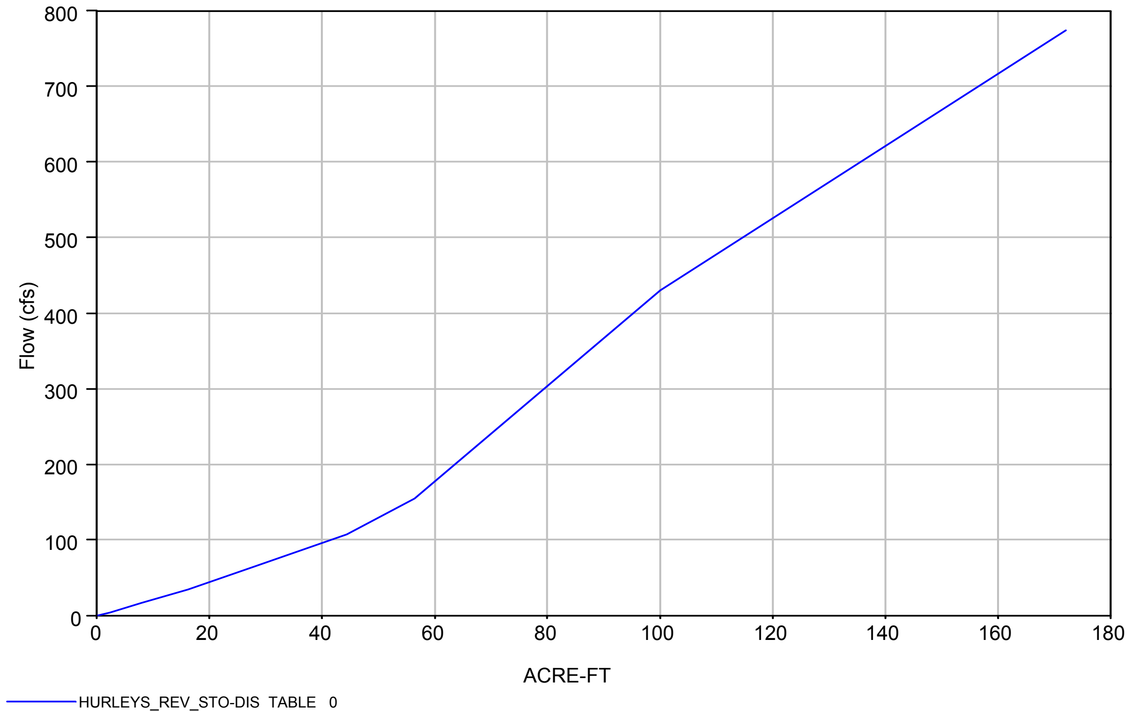
APPENDIX A: STORAGE – FLOW RELATION FOR RESERVOIRS



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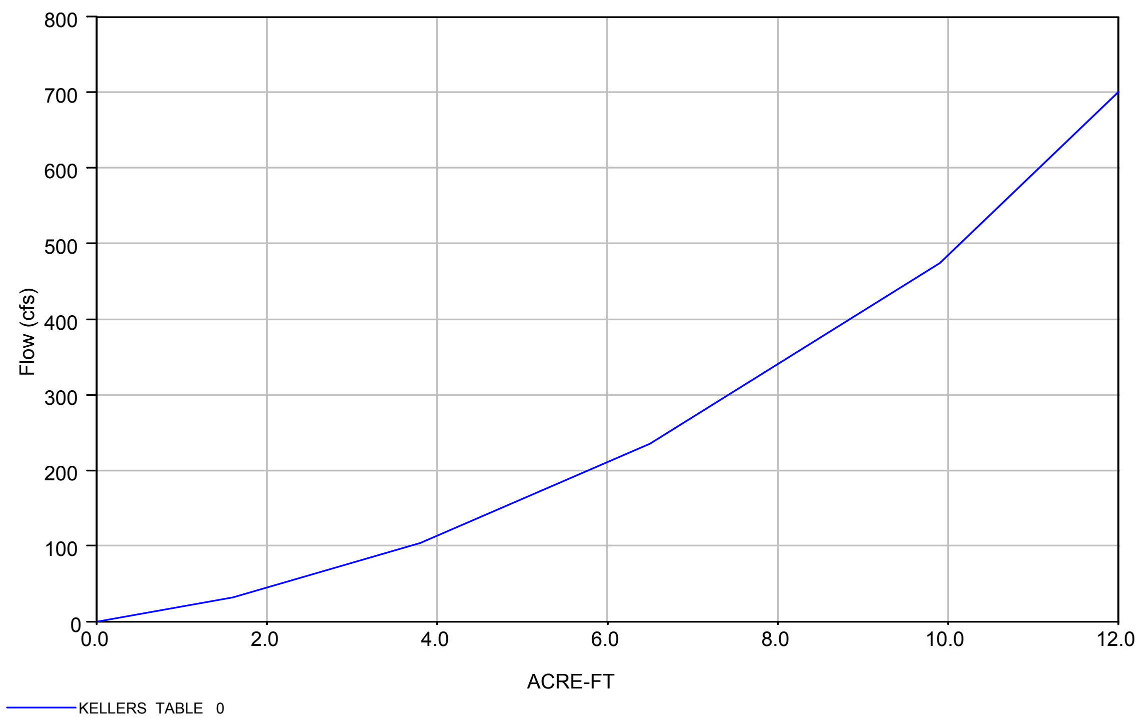
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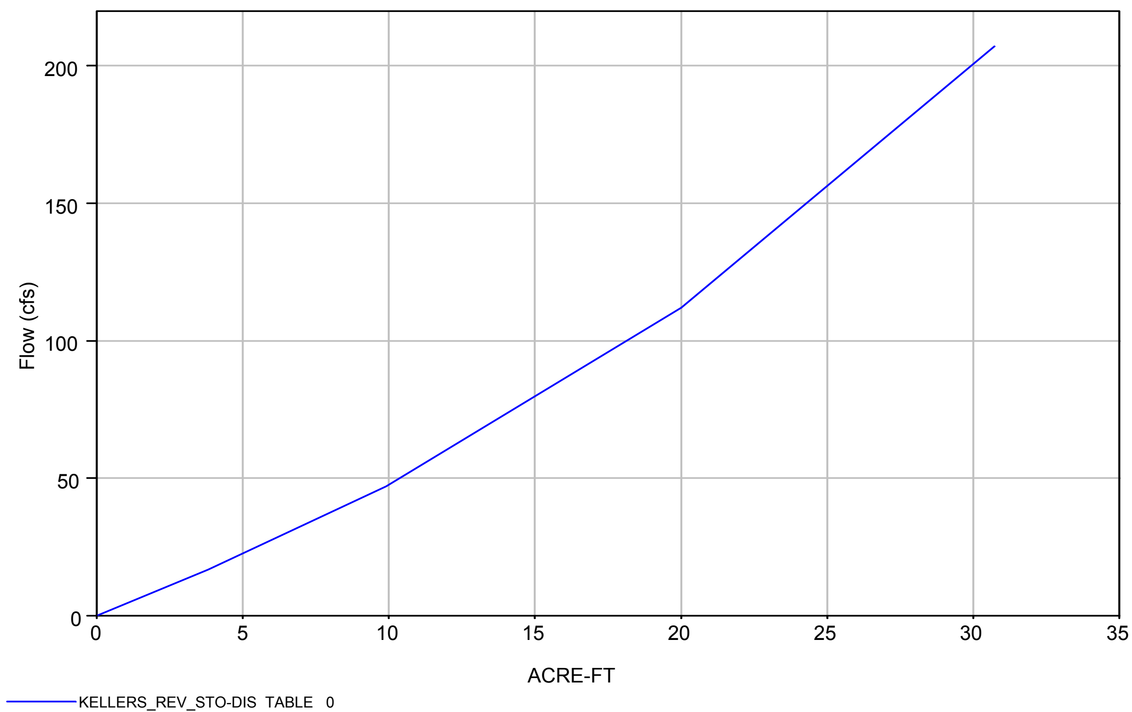
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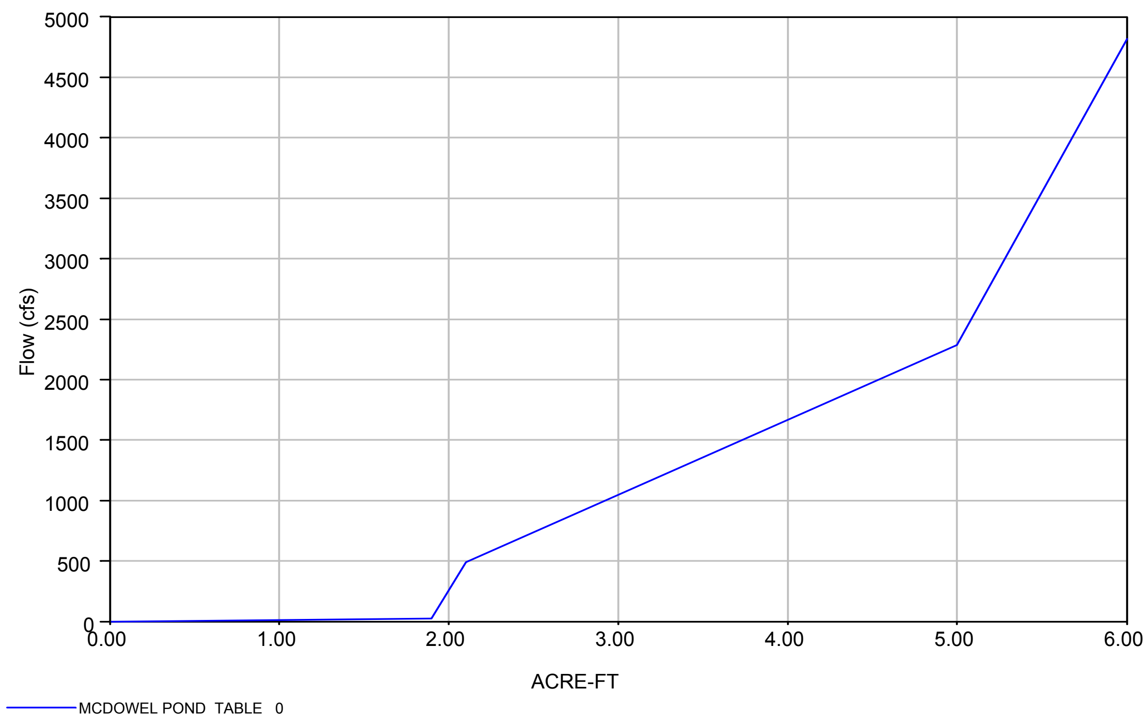
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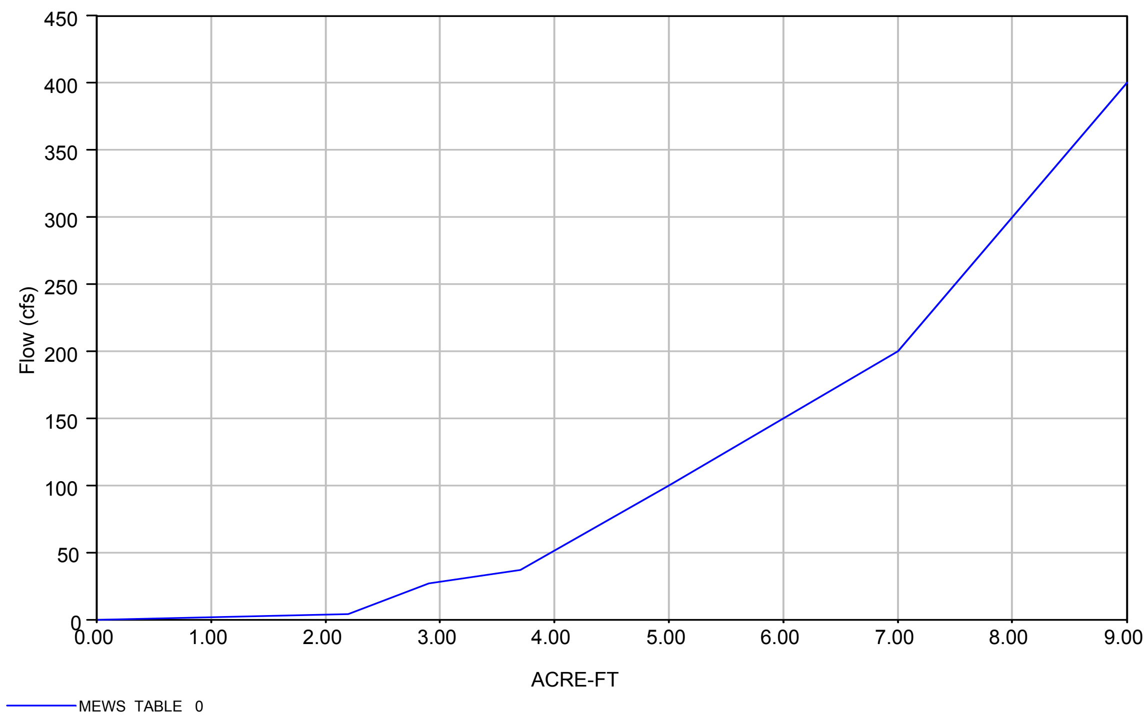
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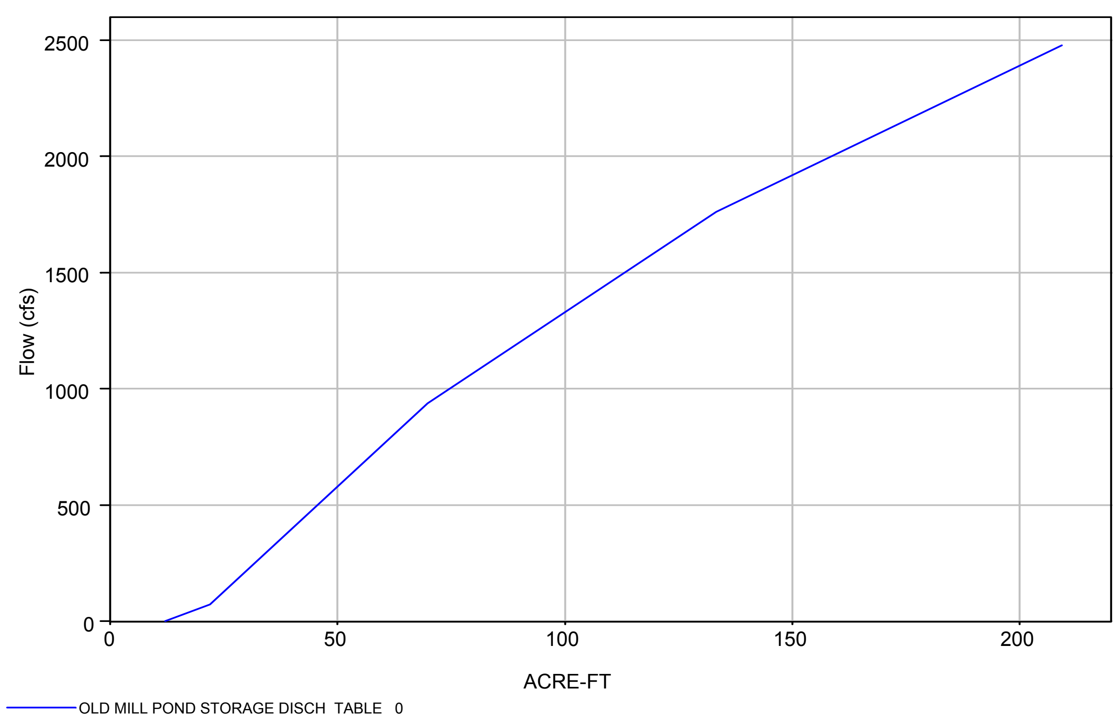
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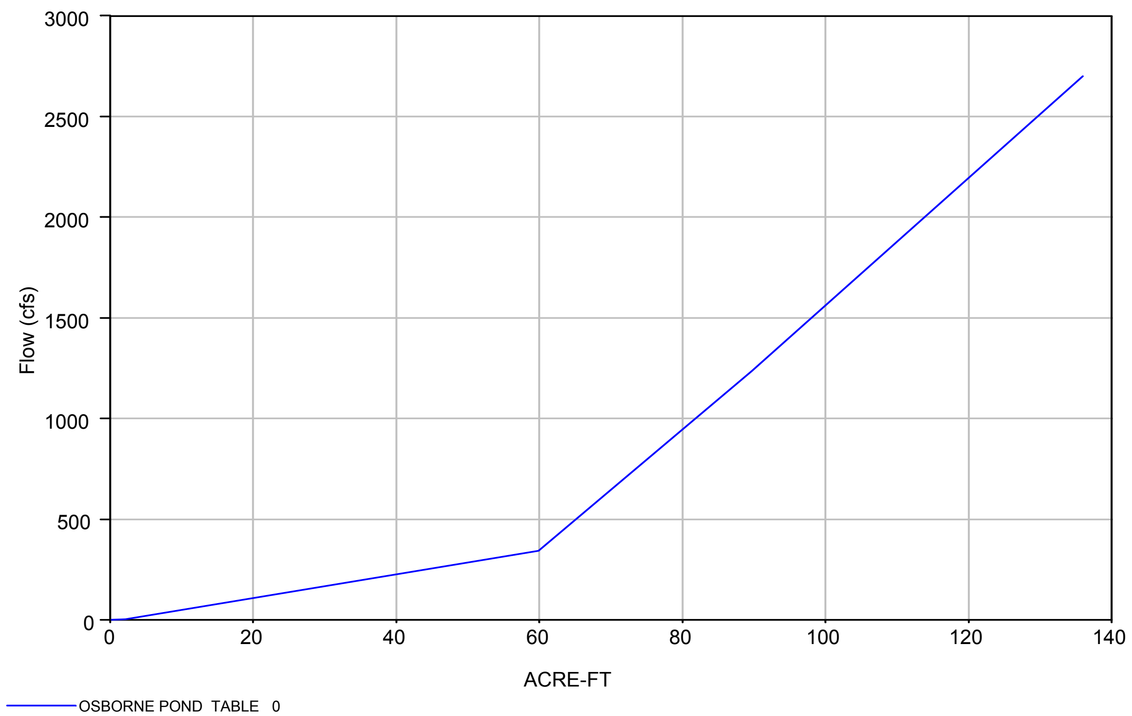
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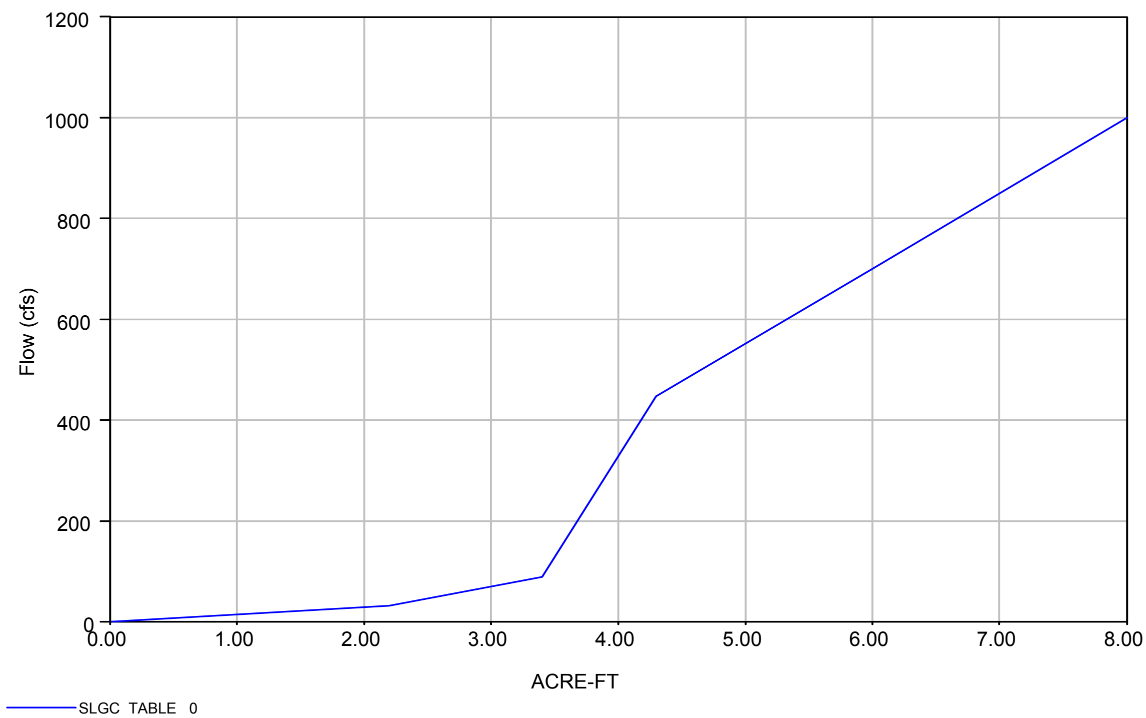
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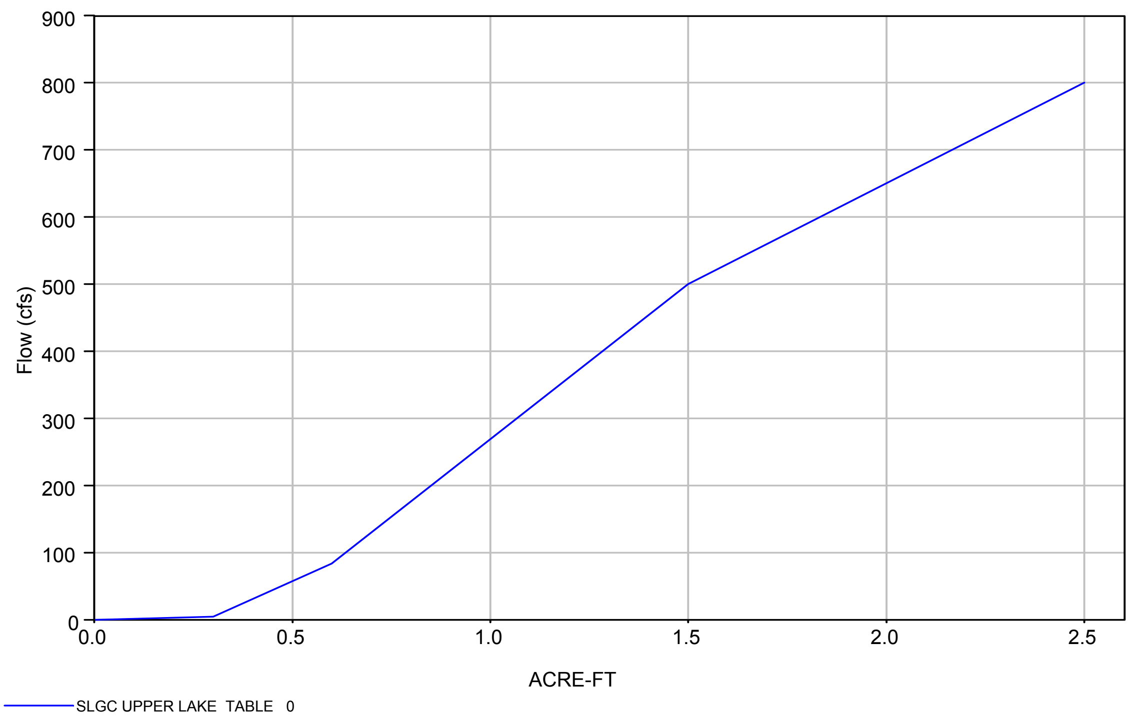
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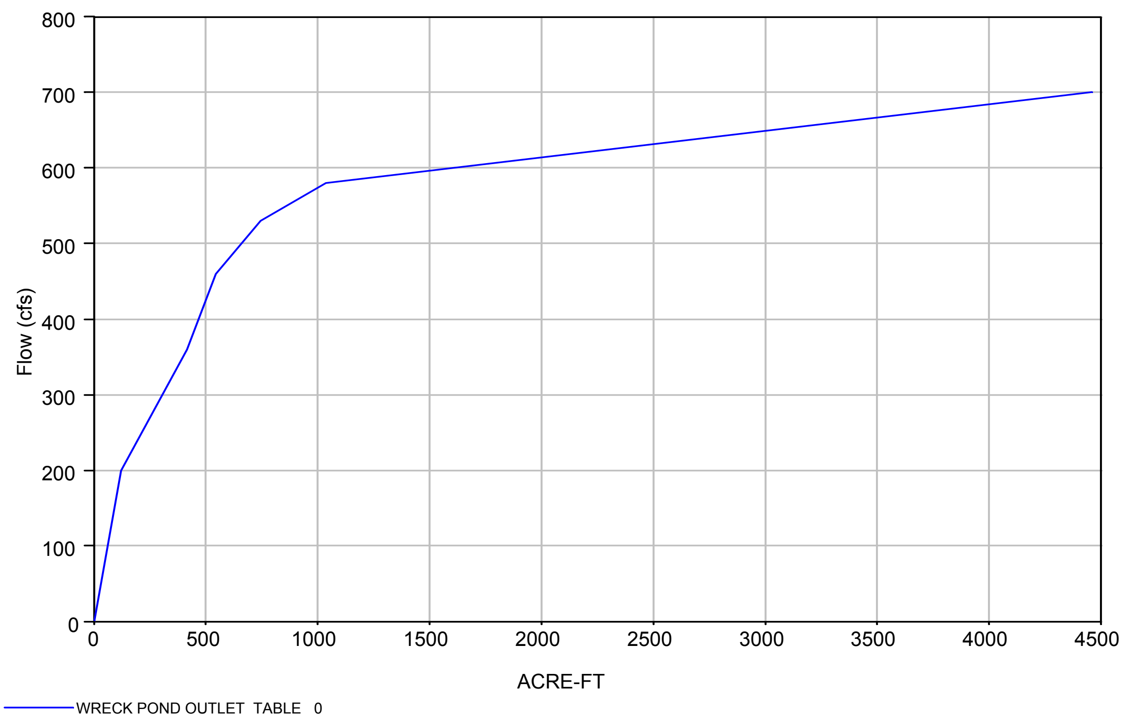
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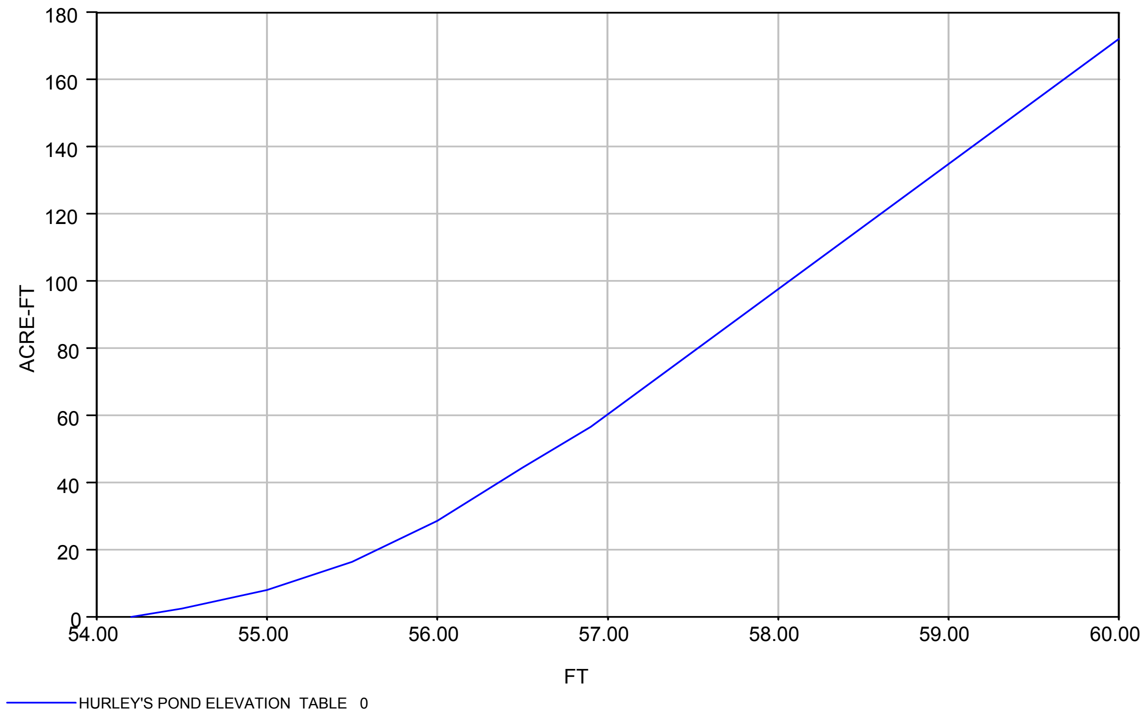
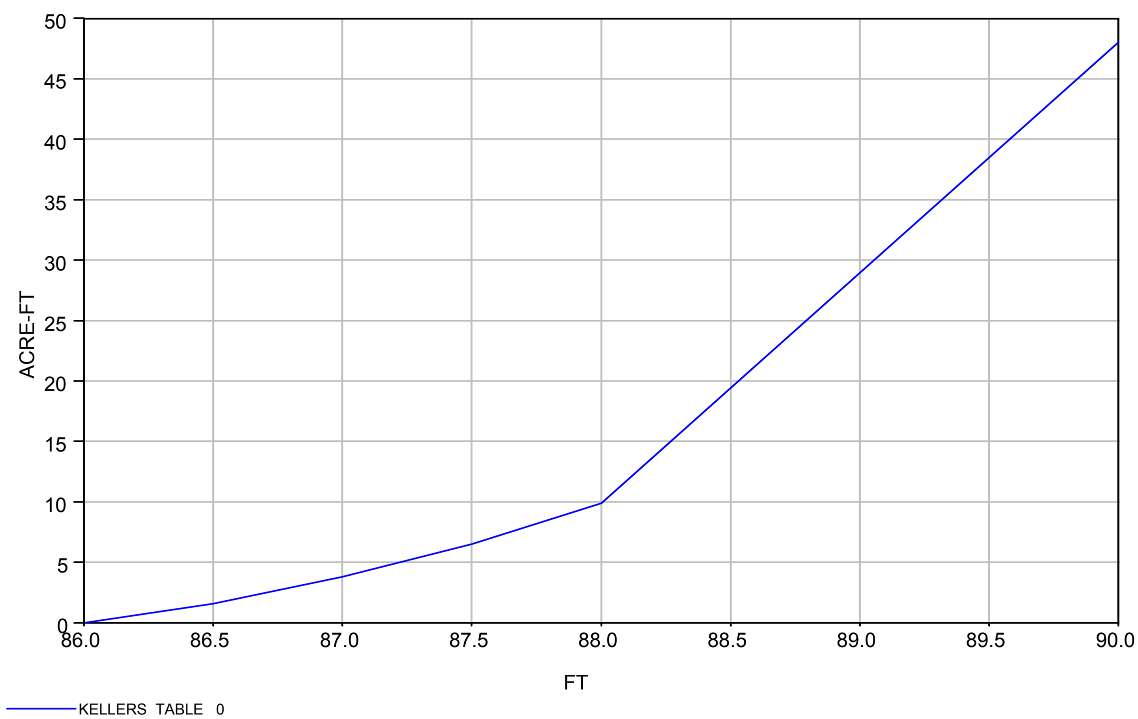
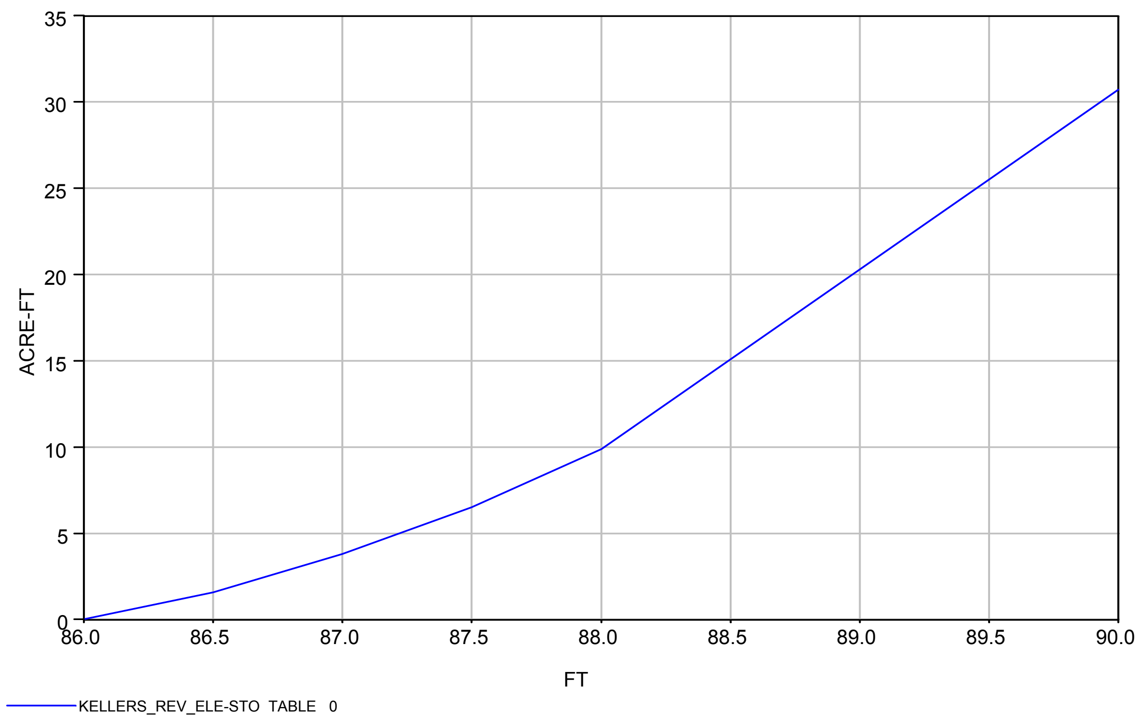
WRECK POND OUTLET TABLE 0

APPENDIX B: ELEVATION–STORAGE RELATION FOR RESERVOIRS







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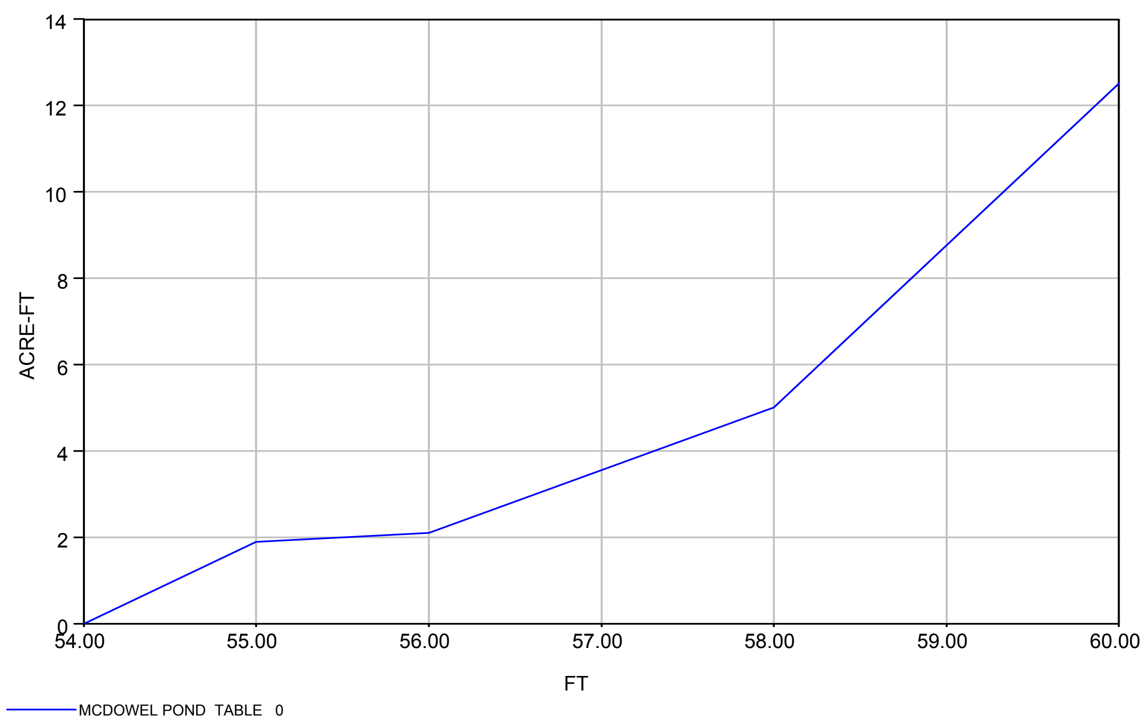

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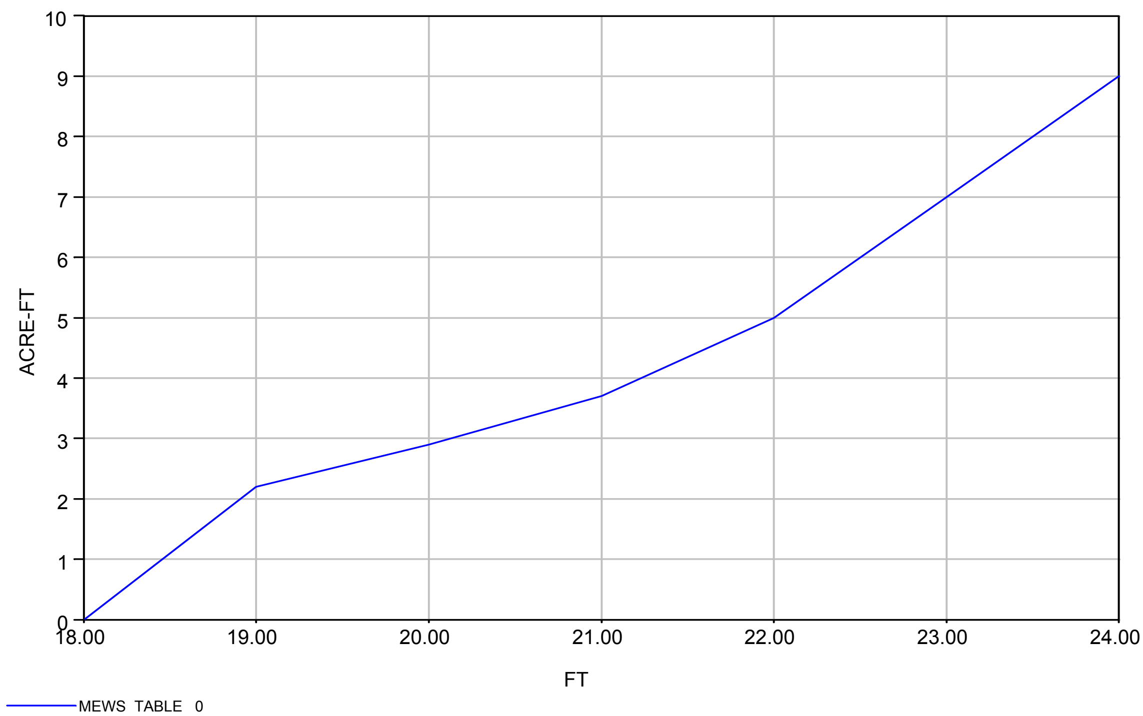

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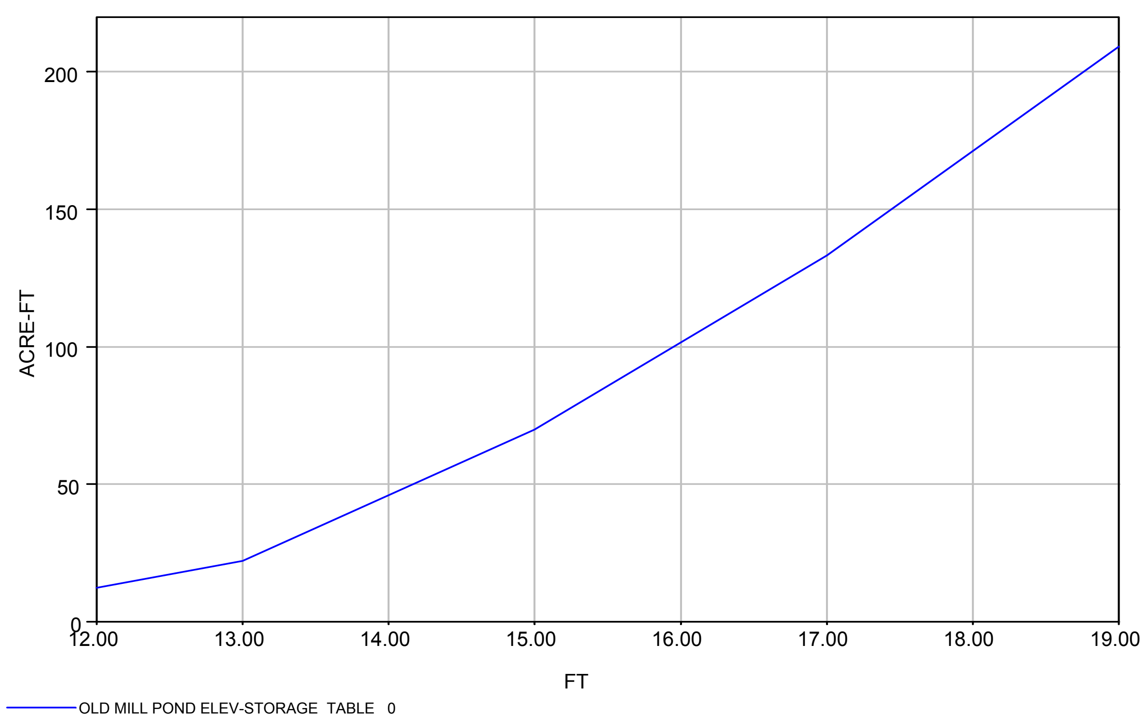

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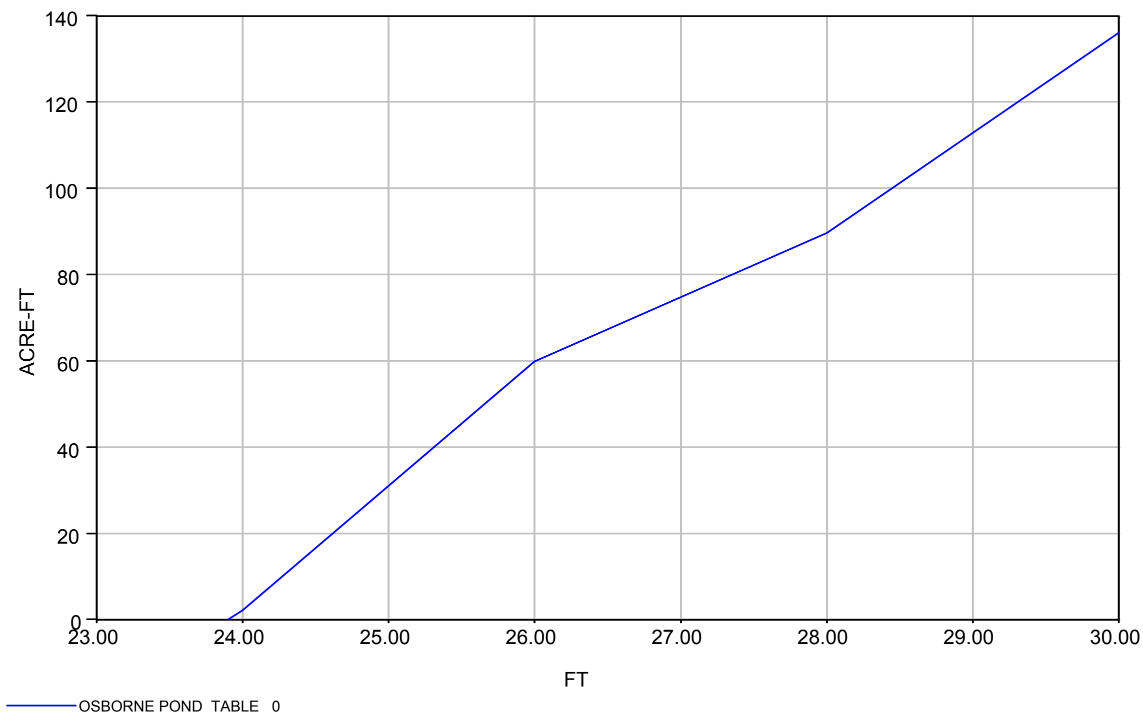

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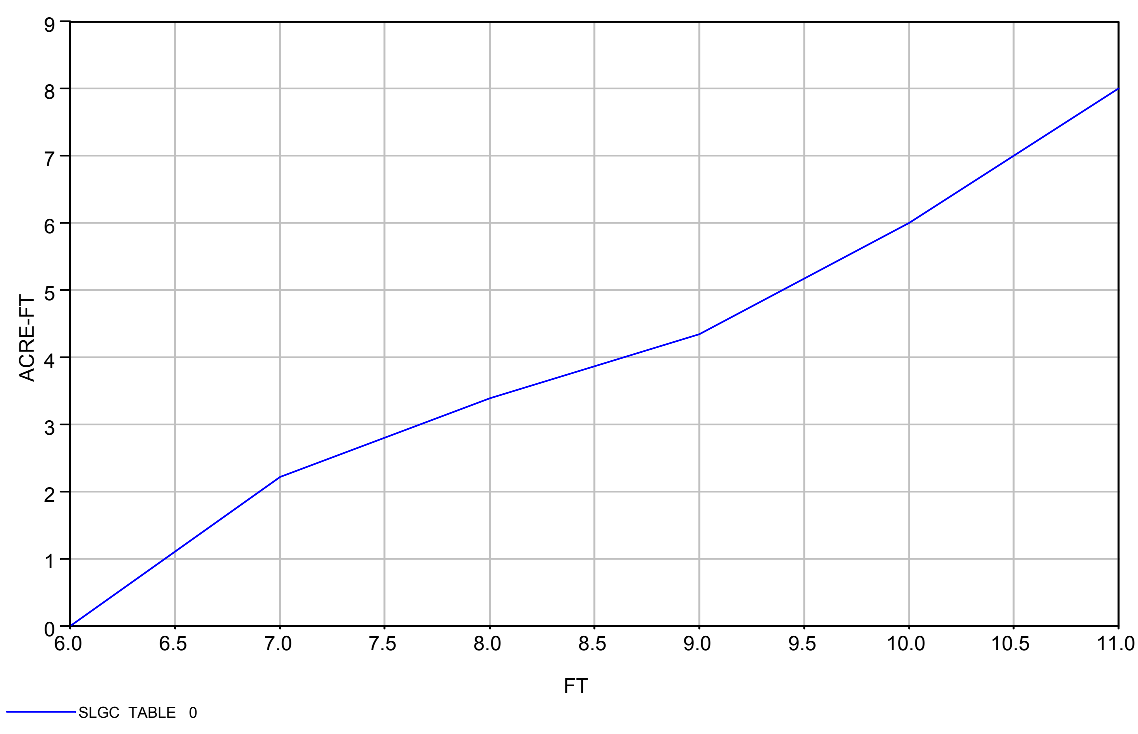

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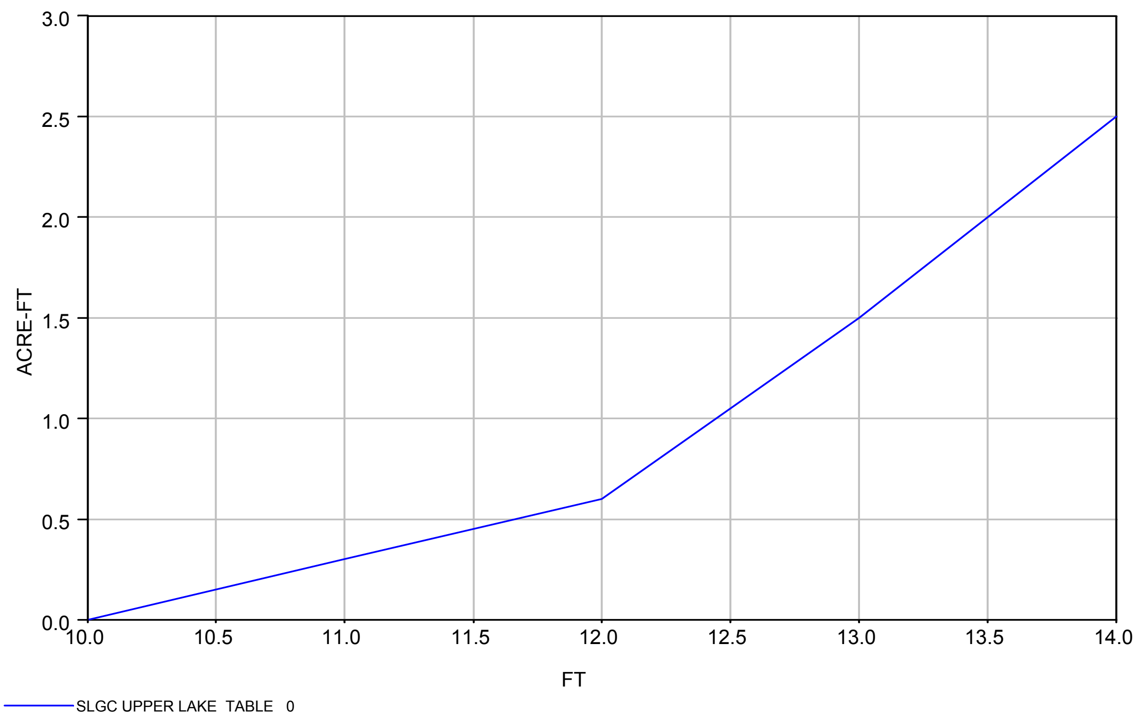
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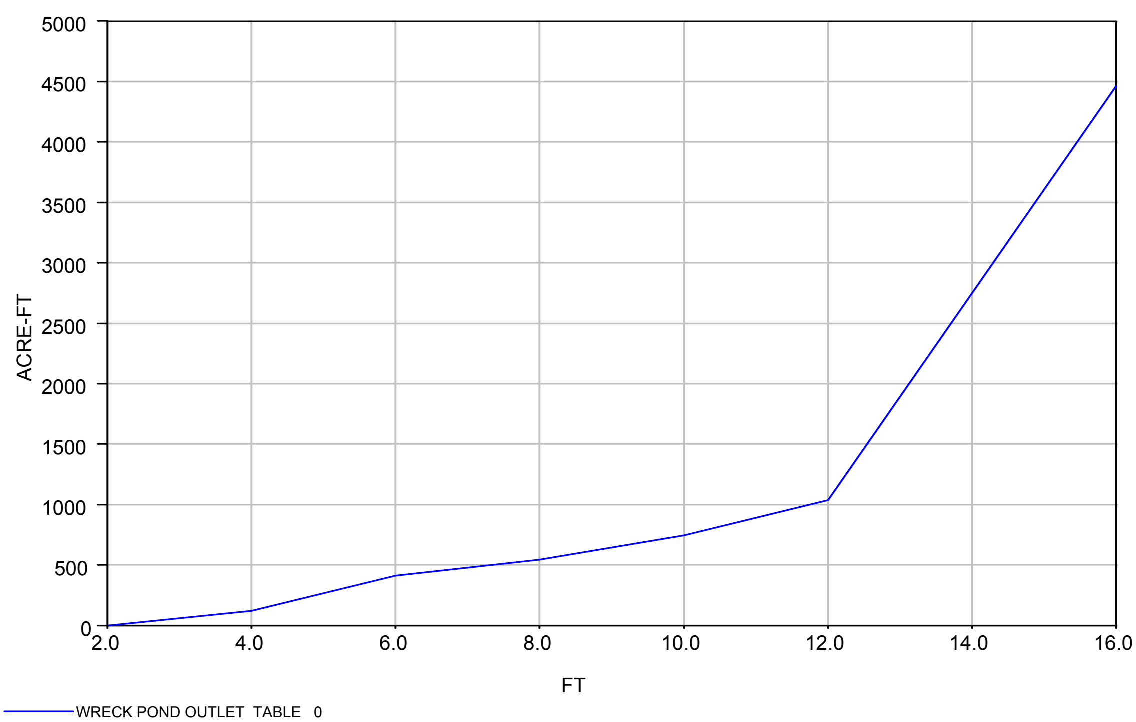
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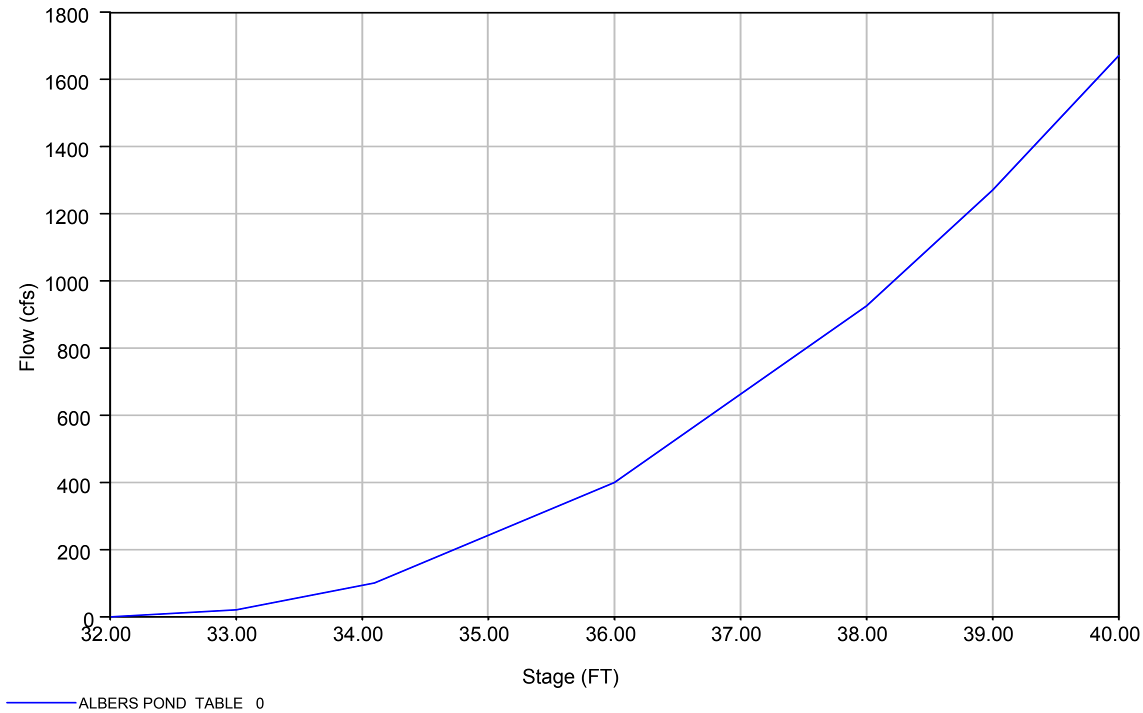
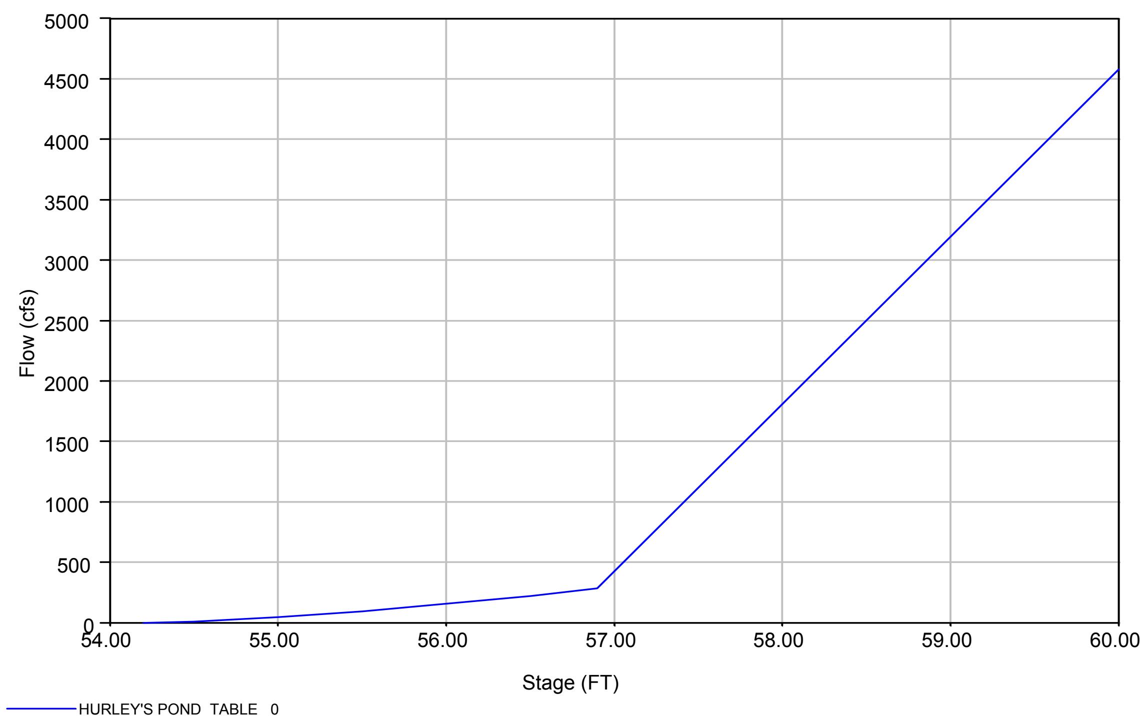
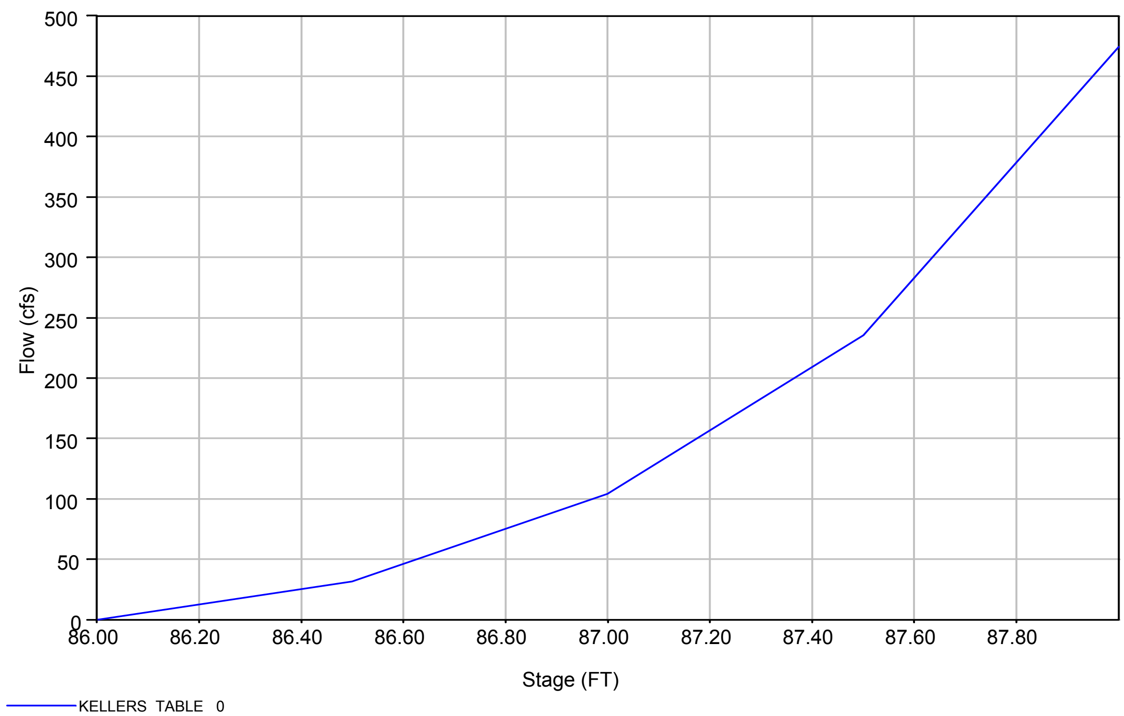
WRECK POND OUTLET TABLE 0

APPENDIX C: STAGE – FLOW RELATION FOR RESERVOIRS







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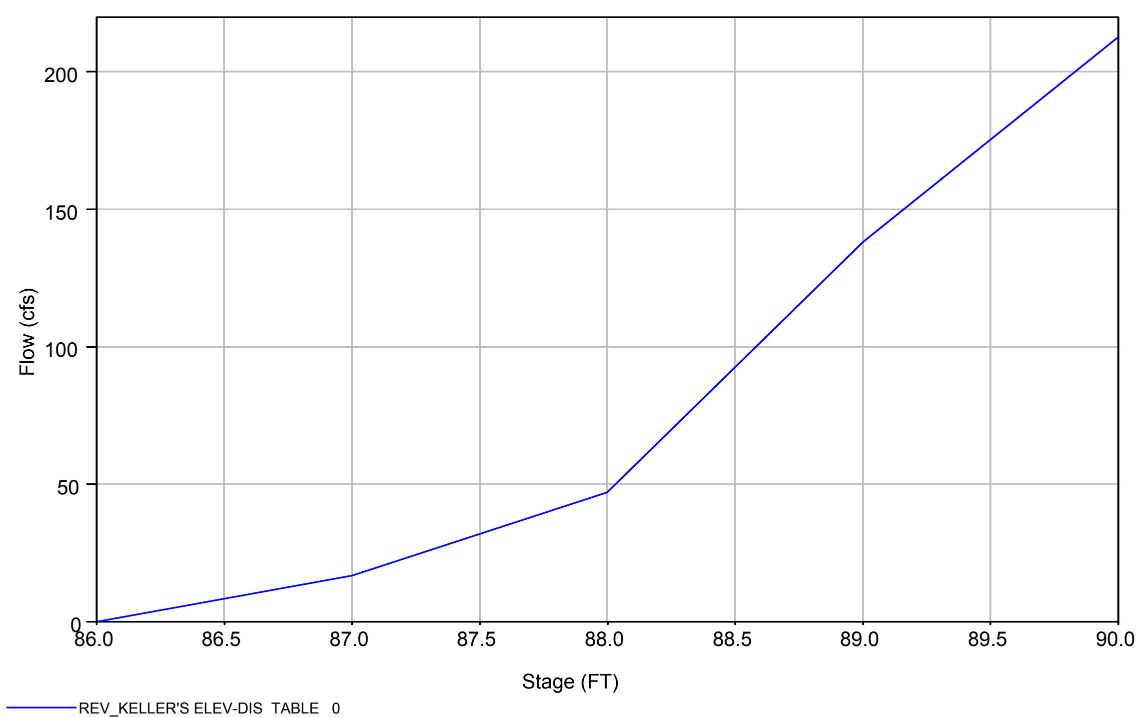
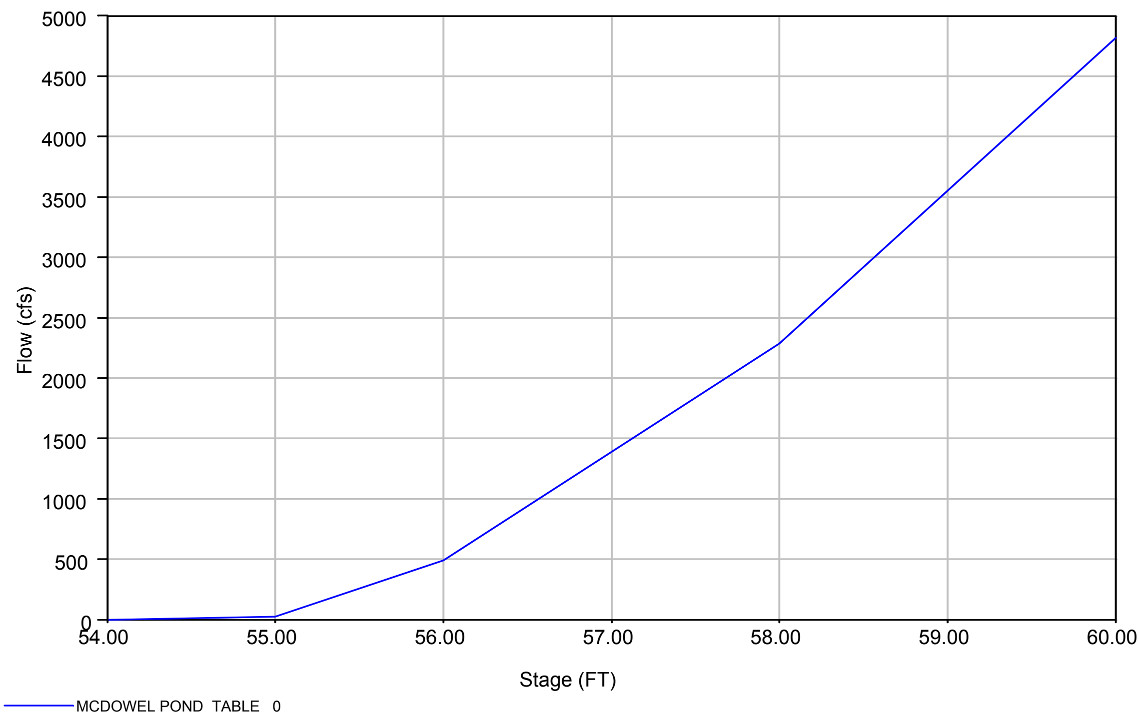
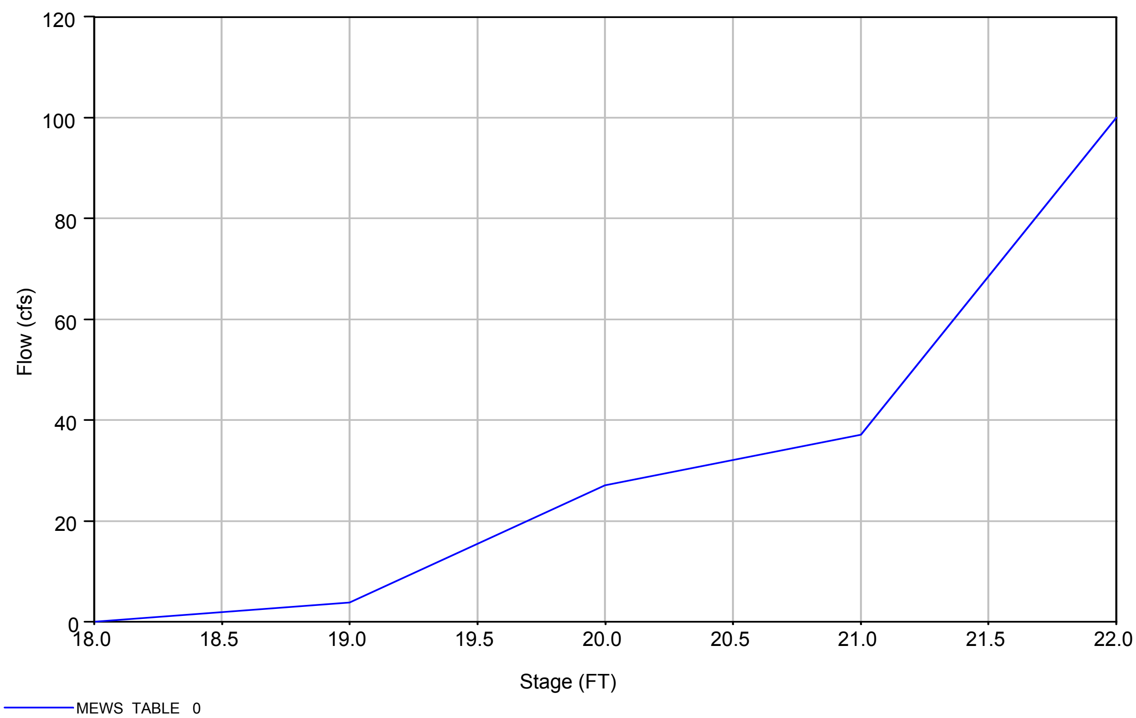
86.20
86.40
86.60
86.80
87.00
87.20
87.40
87.60
87.80
Stage (FT)
KELLERS TABLE 0



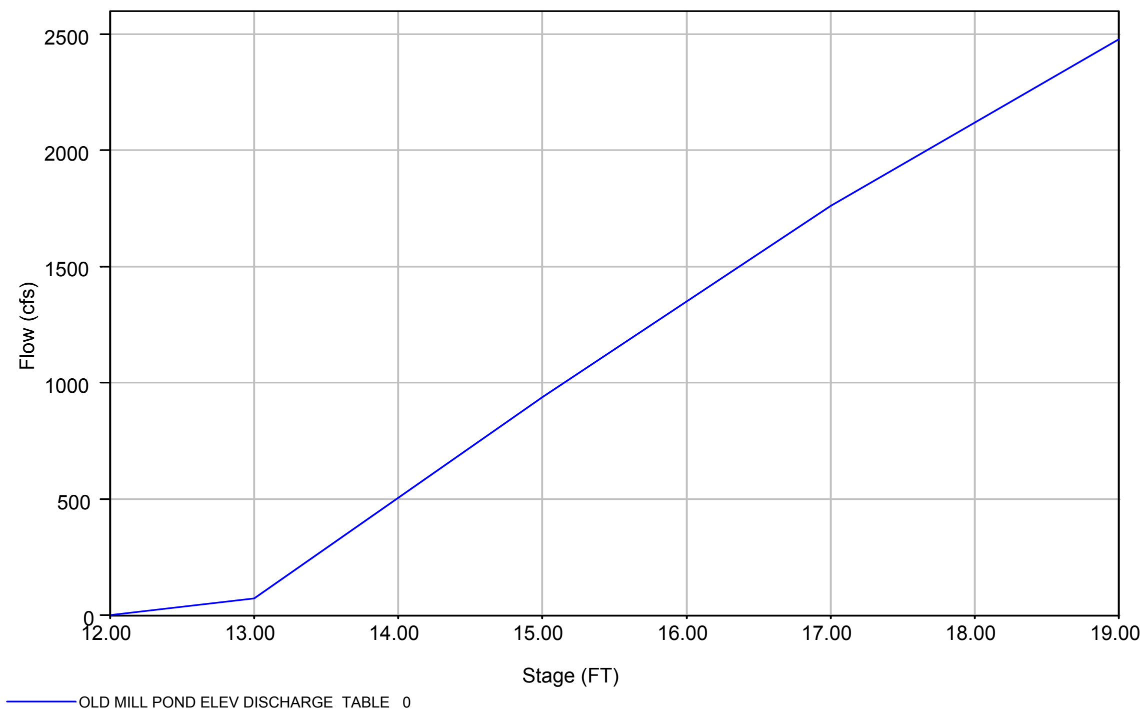
200
150
100
50
0
Flow (cfs)
86.0
86.5
87.0
87.5
88.0
88.5
89.0
89.5
90.0
Stage (FT)
REV_KELLER'S ELEV-DIS TABLE 0

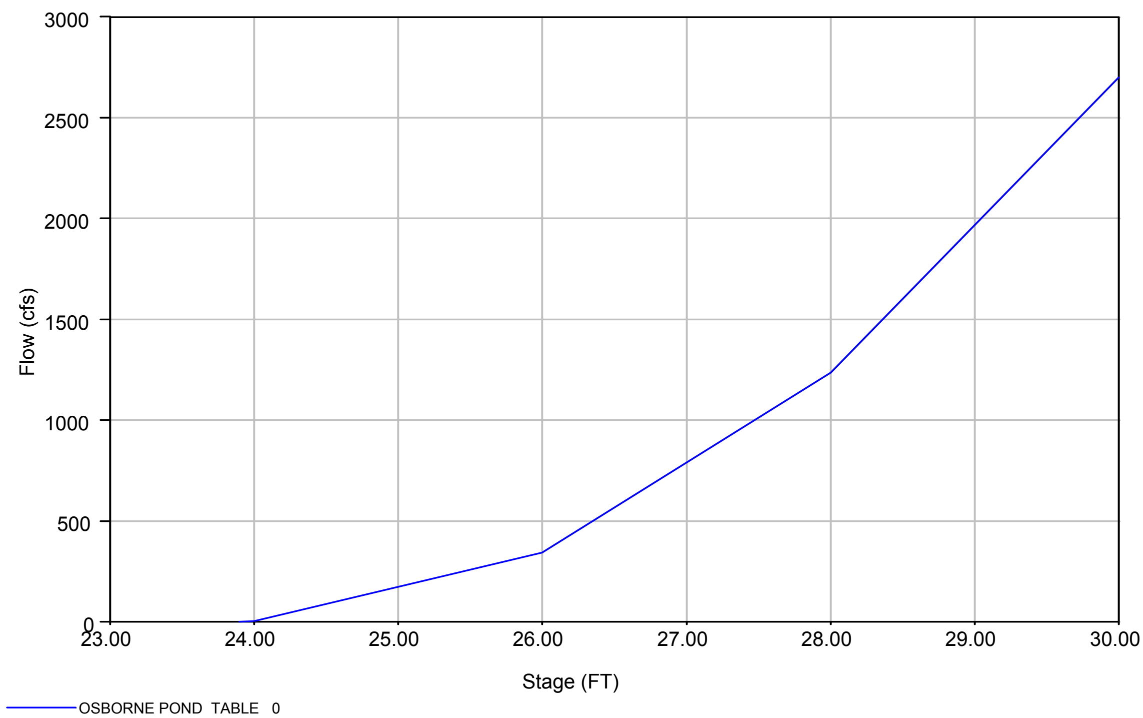


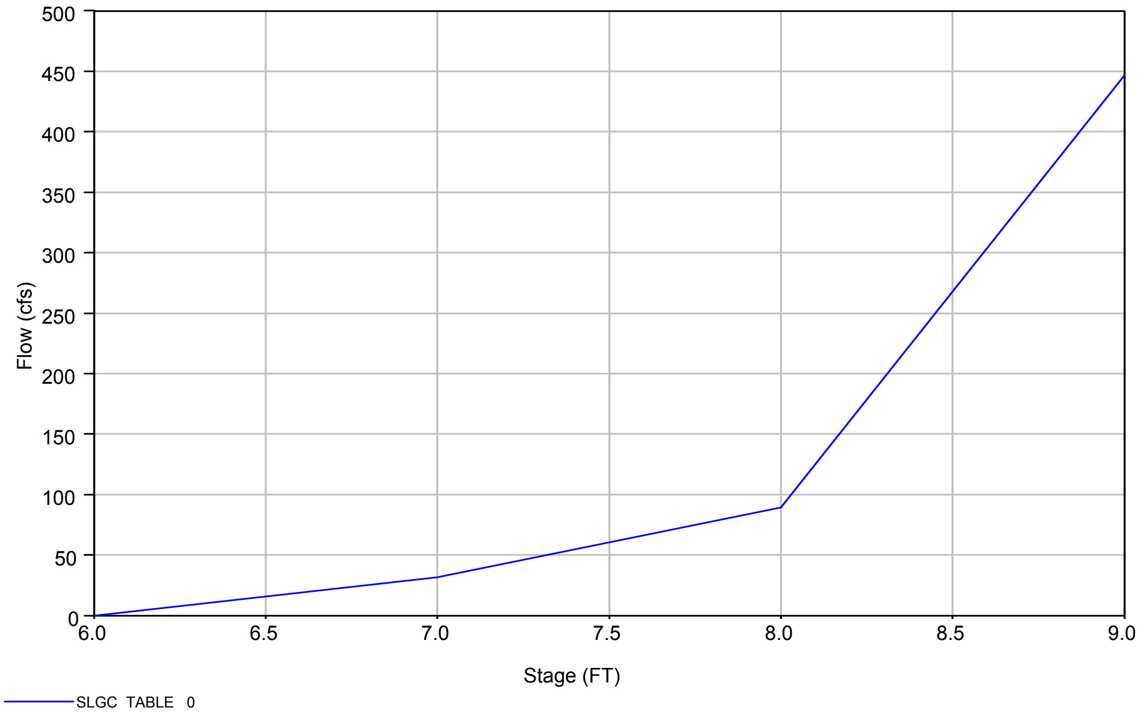
5000
4500
4000
3500
3000
2500
2000
1500
1000
500
0
Flow (cfs)
54.00
55.00
56.00
57.00
58.00
59.00
60.00
Stage (FT)
MCDOWEL POND TABLE 0



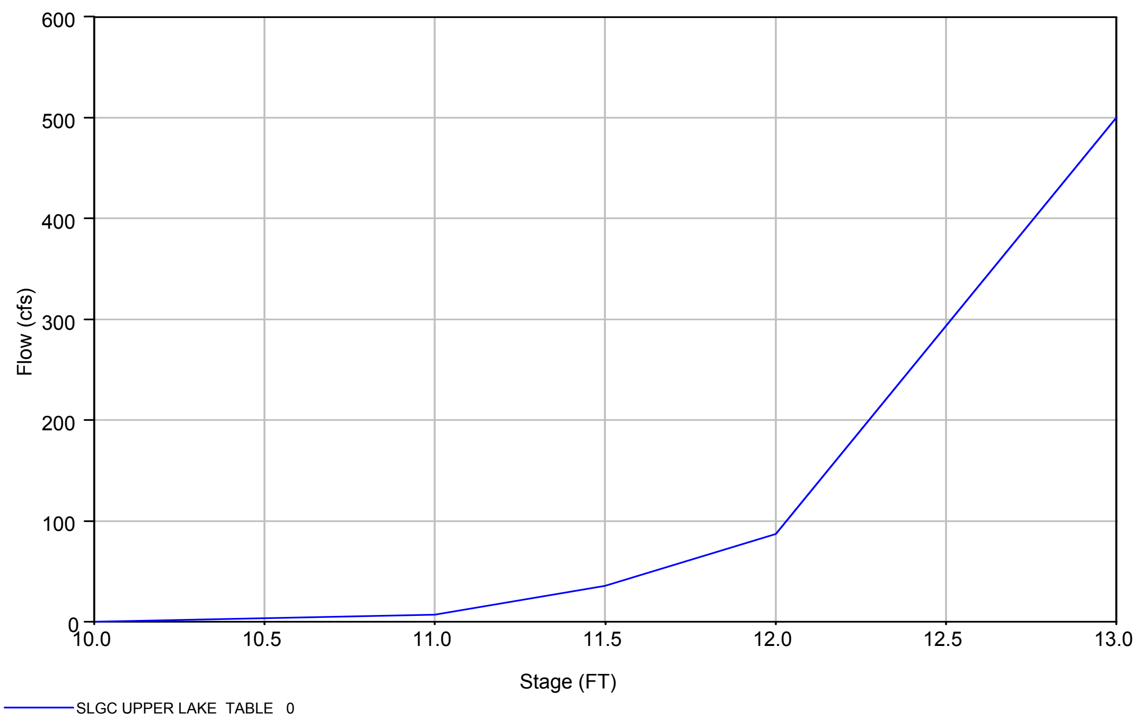
120
100
80
60
40
20
0
Flow (cfs)
18.0
18.5
19.0
19.5
20.0
20.5
21.0
21.5
22.0
Stage (FT)
MEWS TABLE 0


2500
2000
1500
1000
500
0
Flow (cfs)
12.00
13.00
14.00
15.00
16.00
17.00
18.00
19.00
Stage (FT)
OLD MILL POND ELEV DISCHARGE TABLE 0

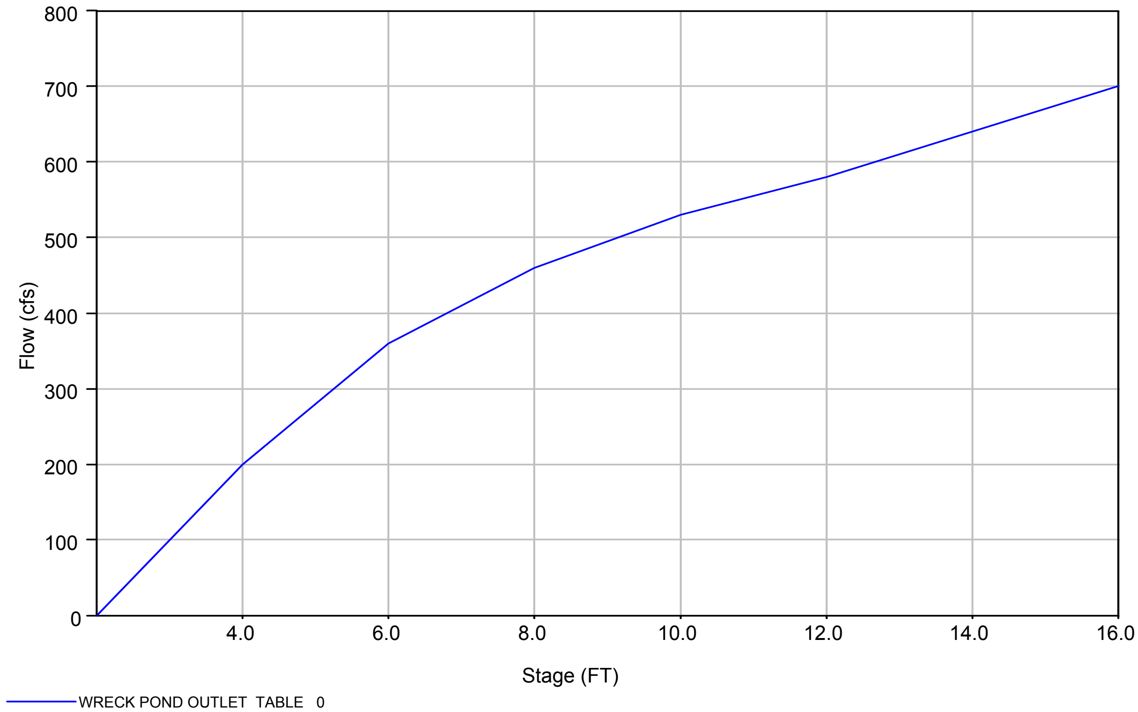

3000
2500
2000
1500
1000
500
0
Flow (cfs)
23.00
24.00
25.00
26.00
27.00
28.00
29.00
30.00
Stage (FT)
OSBORNE POND TABLE 0



500
450
400
350
300
250
200
150
100
50
0
Flow (cfs)
6.0
6.5
7.0
7.5
8.0
8.5
9.0
Stage (FT)
SLGC TABLE 0

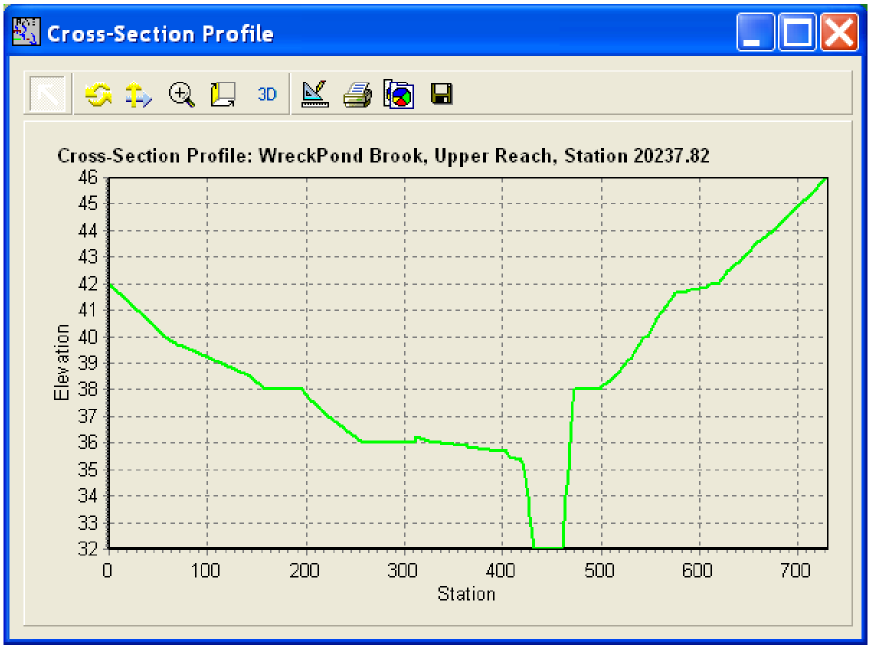


600
500
400
300
200
100
0
Flow (cfs)
10.0
10.5
11.0
11.5
12.0
12.5
13.0
Stage (FT)
SLGC UPPER LAKE TABLE 0

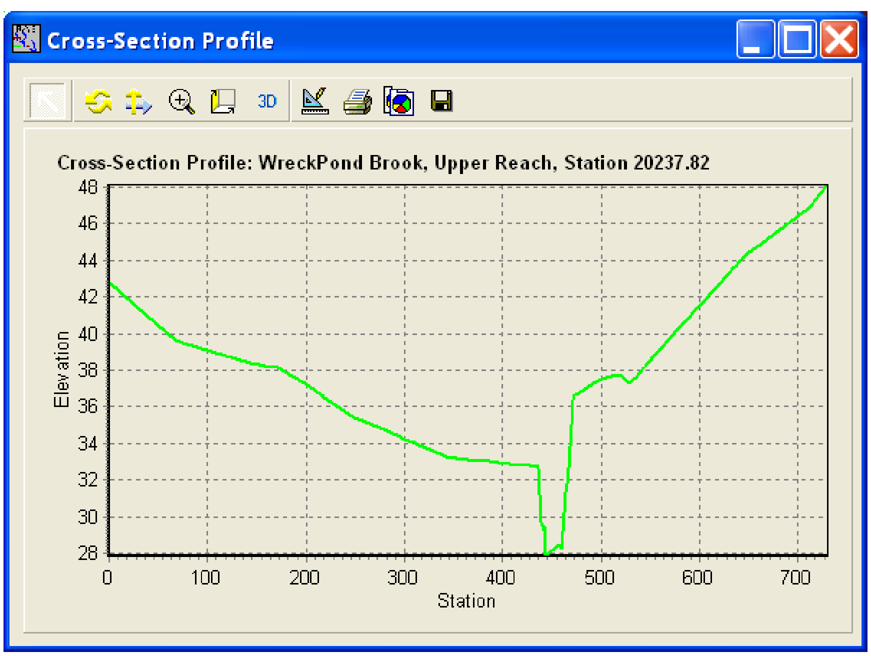

800
700
600
500
400
300
200
100
0
Flow (cfs)
4.0
6.0
8.0
10.0
12.0
14.0
16.0
Stage (FT)
WRECK POND OUTLET TABLE 0

APPENDIX D: CROSS – SECTIONS COMPARISON

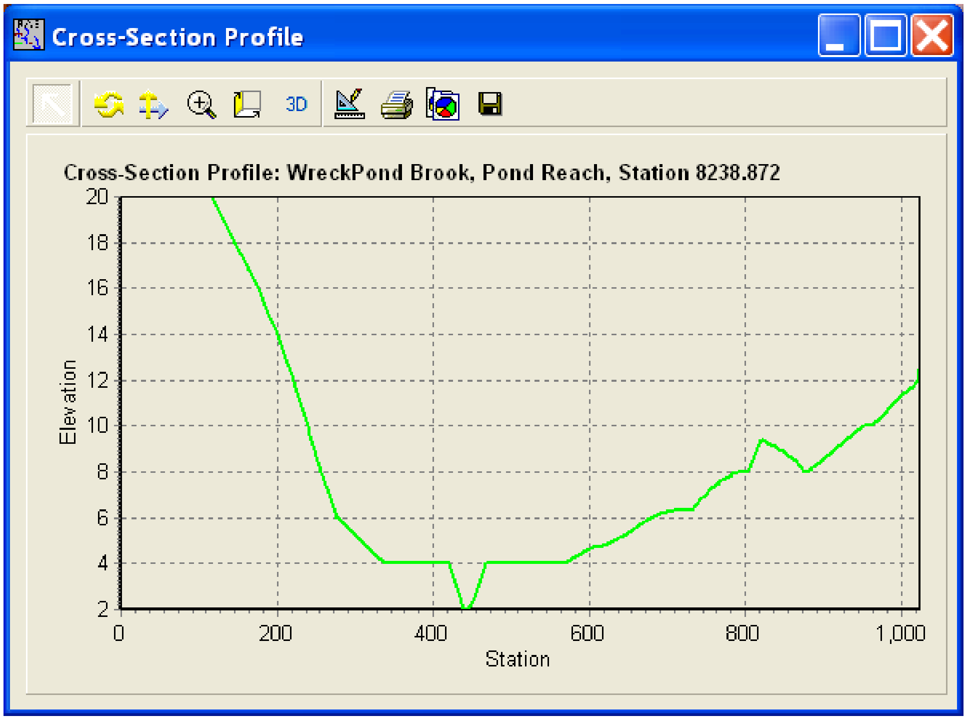
CROSS-SECTIONS COMPARISON USING TIN'S CREATED WITH (METHOD 1) AND WITHOUT (METHOD 2) DETAILED CHANNEL INFORMATION.



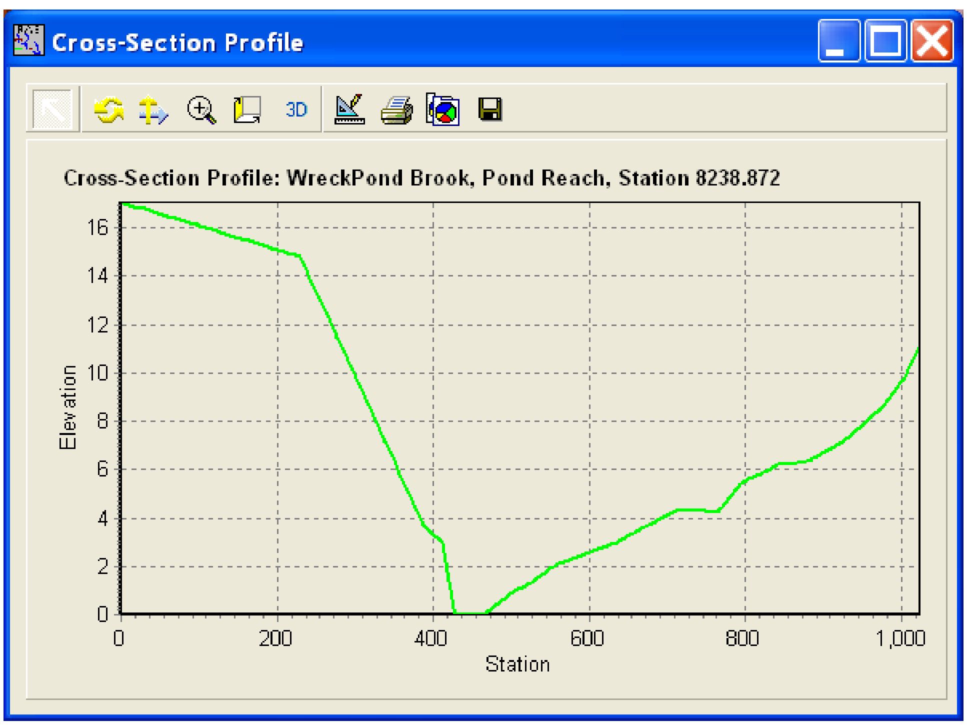
Cross section at station 20237.82 for WPB with method 2



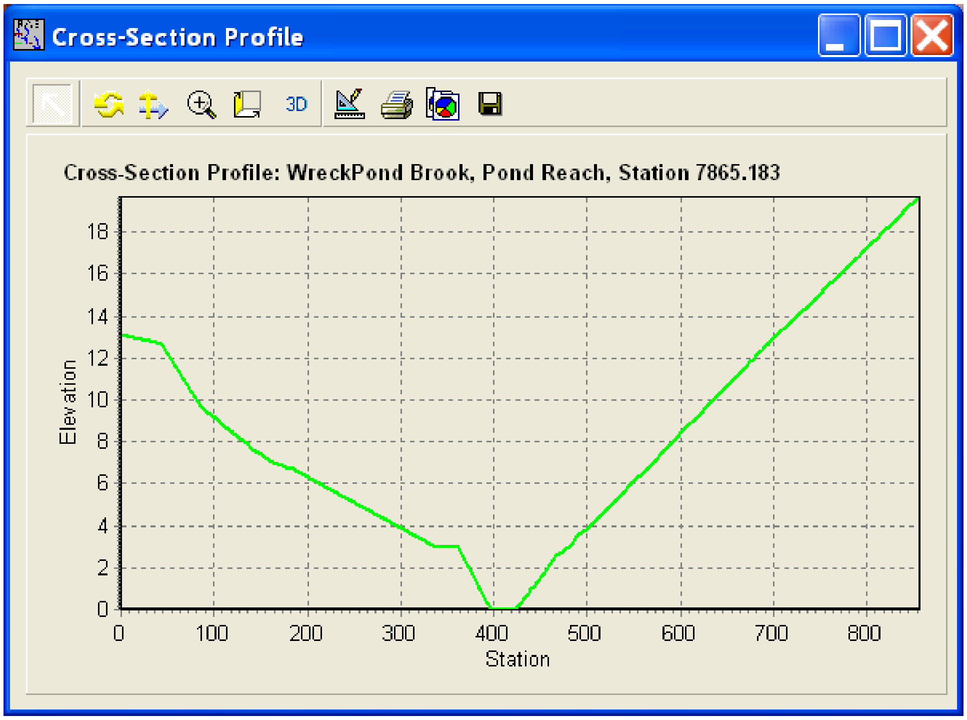
Cross section at station 20237.82 for WPB with method 1



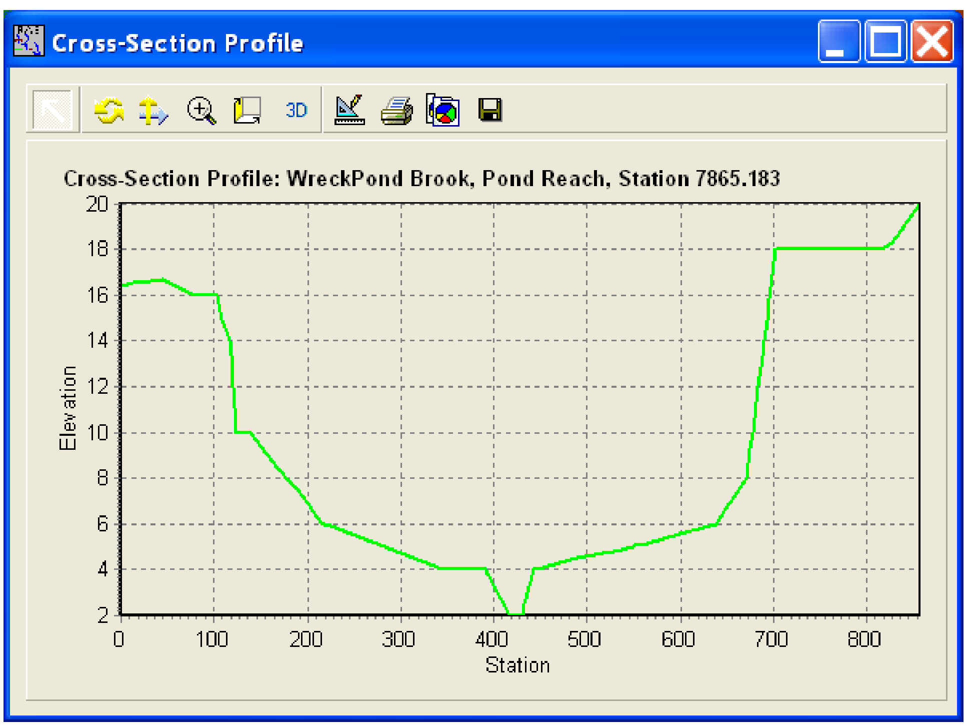
Cross section at station 8238.87 for WPB with method 2



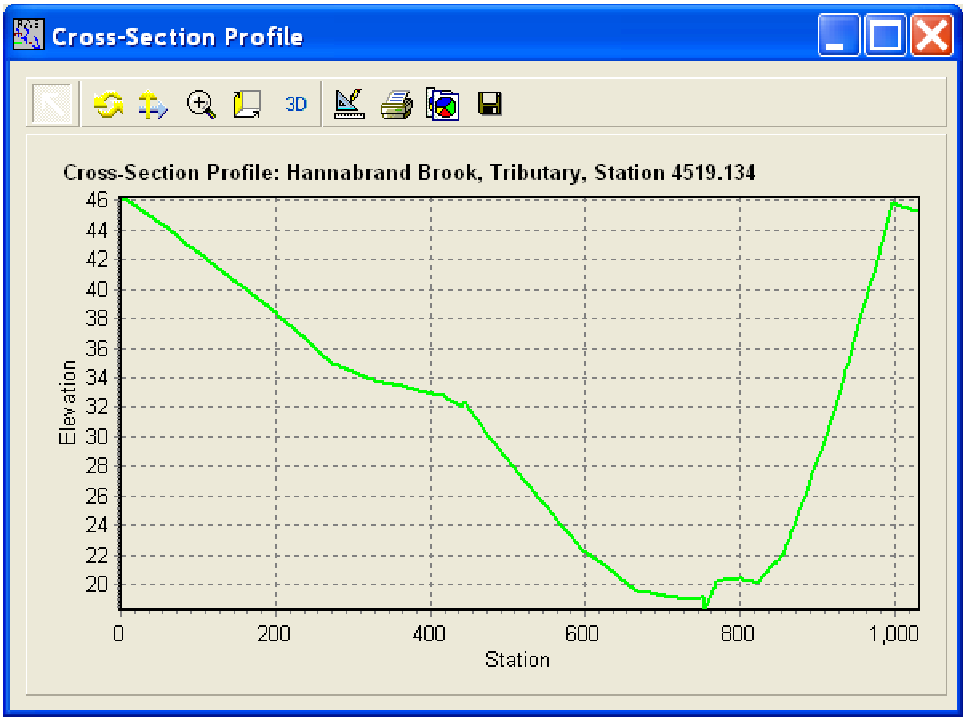
Cross section at station 8238.87 for WPB with method 1



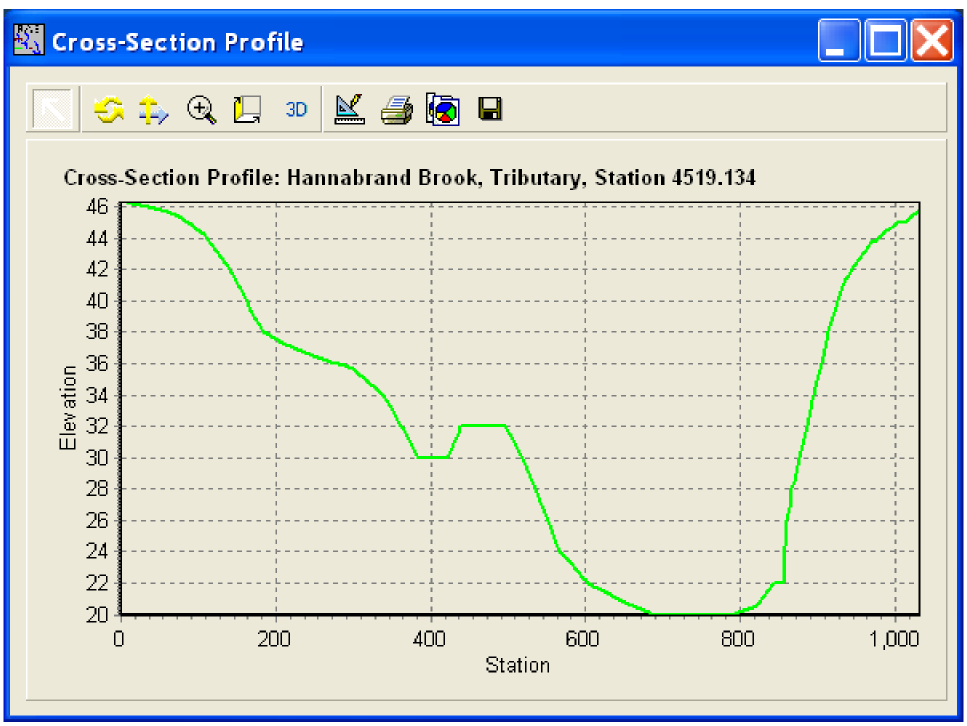
Cross section at station 7865.18 for WPB with method 1



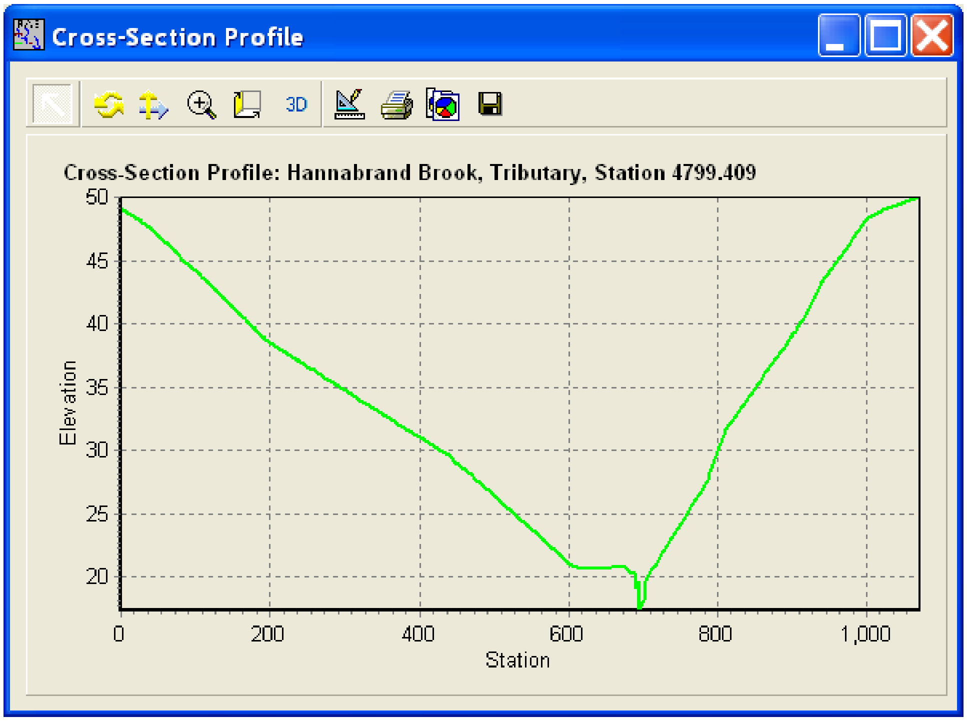
Cross section at station 7865.18 for WPB with method 2



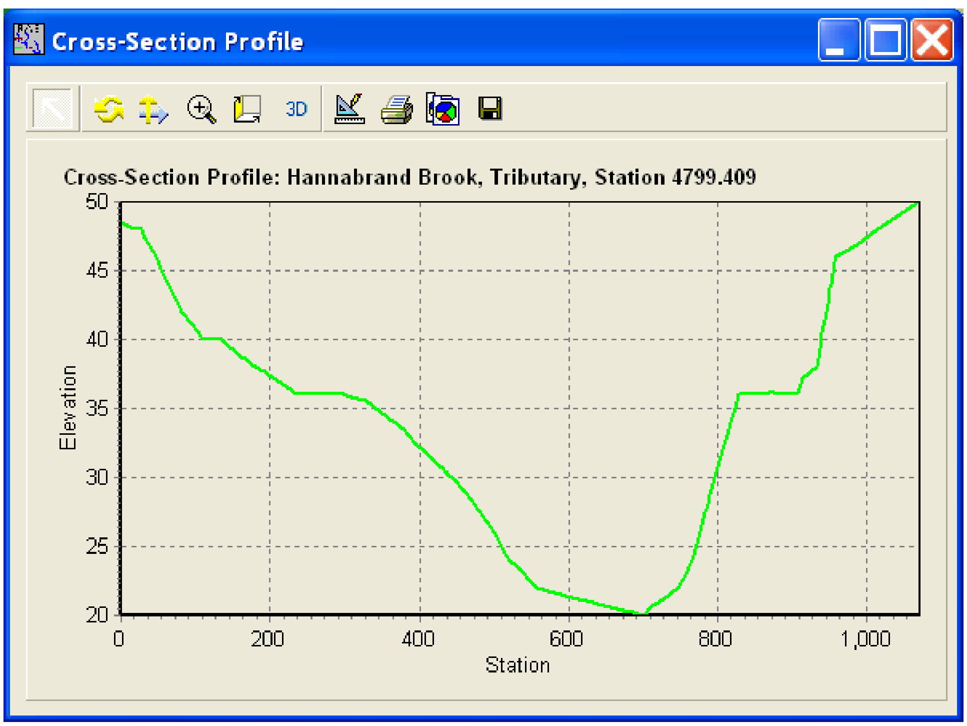
Cross section at station 4519.13 for HB with method 1



Cross section at station 4519.13 for HB with method 2



Cross section at station 4799.40 for HB with method 1



Cross section at station 4799.40 for HB with method 2

APPENDIX E: RESULTS FOR 100, 50, 25, 10 AND 2-YEAR STORM

Table 1: HMS simulation results for 100-Year NRCS storm event using percentage curves created from the SCS standard dimensionless unit hydrograph (PRF 484)

HYDROLOGIC ELEMENT	DRAINAGE AREA (SQ. MILE)	PEAK FLOW (CFS)	TIME OF PEAK [a]	VOL. (IN)
Albers Pond	0.97000	619.5	24Jun2006, 14:15	5.52
Albers pond da	0.93000	1026.3	24Jun2006, 13:15	5.41
Albers Pond Surface	0.04000	132.8	24Jun2006, 12:15	8.93
Albers to Osborne	0.97000	559.2	24Jun2006, 15:15	5.52
Hanabrand Outlet	2.94550	1027.6	24Jun2006, 18:45	4.73
HP to W7	3.11250	1296.2	24Jun2006, 18:00	5.16
Hurley's Pond	3.11250	1506.6	24Jun2006, 15:00	5.16
Junction-1	0.42200	537.5	24Jun2006, 13:15	6.00
Junction-3	Not Specified	805.3	24Jun2006, 13:15	
Kellers Pond	0.34250	584.6	24Jun2006, 12:45	6.15
McDowel Pond	0.59000	594.0	24Jun2006, 13:15	4.89
McD to W7	0.59000	426.1	24Jun2006, 14:00	4.89
Mews Combined Basin	0.17200	249.6	24Jun2006, 13:00	6.16
Mews Stormsewer	0.25000	298.4	24Jun2006, 13:30	5.89
Old Mill Pond W3 gage	7.27050	1925.7	24Jun2006, 20:30	5.19
Old Mill-Wreck Pond	10.21600	2577.7	24Jun2006, 23:30	5.05
Osbonre to W1	6.71250	1967.8	24Jun2006, 18:45	5.12
Osborne Pond	6.71250	1971.0	24Jun2006, 18:15	5.12
Osborne Pond da	0.85000	1269.8	24Jun2006, 12:45	5.06
Osborne Pond surface	0.03000	99.6	24Jun2006, 12:15	8.93
Parkway	0.04000	64.8	24Jun2006, 13:15	8.93
Rt34-W6	0.34250	336.7	24Jun2006, 14:15	6.15
Rt 35 culvert	2.94550	1120.7	24Jun2006, 17:15	4.73
Rt 35 to Outlet	2.94550	1027.6	24Jun2006, 18:45	4.73
Rt 71 East	0.00512	15.4	24Jun2006, 12:15	6.97
RT 71 w/Imp	0.02000	57.2	24Jun2006, 12:15	6.46
Slgc flowpath	0.02519	36.0	24Jun2006, 12:45	4.38
Slgc lake	0.64040	764.3	24Jun2006, 13:15	5.60

Slgc lakes only	0.00440	14.6	24Jun2006, 12:15	8.93
slgc upper lake	0.42200	536.7	24Jun2006, 13:15	6.00
SL-SlH-SG	1.76000	1866.8	24Jun2006, 13:30	5.89
W1	0.07800	199.0	24Jun2006, 12:15	6.35
W1 gage	6.79050	1974.5	24Jun2006, 18:30	5.14
W1-W3	6.79050	1940.0	24Jun2006, 19:30	5.13
W2 Rt 35	0.13750	127.8	24Jun2006, 13:45	7.66
W2 subdivision at Baileys	0.16250	104.7	24Jun2006, 14:30	6.51
W3	0.48000	634.8	24Jun2006, 13:00	6.09
W5	2.33000	1015.2	24Jun2006, 15:30	4.42
W5 add/l housing	0.07800	75.6	24Jun2006, 13:15	4.66
W5 combined	2.64550	1101.8	24Jun2006, 15:30	4.57
W5 housing development	0.23750	232.6	24Jun2006, 13:15	5.97
W5-W2	2.80800	1088.1	24Jun2006, 17:15	4.59
W6	0.93000	607.2	24Jun2006, 14:00	4.87
W6_Rt 34	0.34250	595.5	24Jun2006, 12:45	6.15
W6 gage	1.27250	937.5	24Jun2006, 14:00	5.21
W6-Hurley Pond	1.31250	751.0	24Jun2006, 16:00	5.33
W7	4.86250	1674.7	24Jun2006, 17:30	5.05
W7_Glen	1.16000	713.8	24Jun2006, 14:30	4.84
W7_McD	0.59000	588.3	24Jun2006, 13:15	4.89
W7 to Osborne Pond	4.86250	1642.2	24Jun2006, 18:45	5.05
W8 base flow	Not Specified	12.0	24Jun2006, 00:00	
W8 Mews	0.17200	259.8	24Jun2006, 13:00	6.16
W8 SLGC	0.21400	251.7	24Jun2006, 13:00	4.74
W8 urban lower	0.02519	37.5	24Jun2006, 12:30	4.38
W8 urban upper	0.25000	305.2	24Jun2006, 13:15	5.89
W9	1.80000	1233.2	24Jun2006, 14:00	5.04
W9 junction	3.11250	1537.8	24Jun2006, 14:30	5.16
WPB-HAN Combined	10.21600	2880.0	24Jun2006, 19:30	5.06
WP Junction	Not Specified	2731.3	24Jun2006, 23:30	
Wreck Pond	Not Specified	621.9	25Jun2006, 09:30	

a : Storm event is from the hypothetical start time of 24 June 2006 @ 00:00 hours.

Table 2: HMS simulation results for 100-Year NRCS storm event using percentage curve created from Delmarva unit hydrograph (PRF 284)

HYDROLOGIC ELEMENT	DRAINAGE AREA (SQ. MILE)	PEAK FLOW (CFS)	TIME OF PEAK [a]	VOL. (IN)
Albers Pond	0.97000	485.8	24Jun2006, 14:45	5.52
Albers pond da	0.93000	686.0	24Jun2006, 13:15	5.41
Albers Pond Surface	0.04000	127.5	24Jun2006, 12:15	8.93
Albers to Osborne	0.97000	460.8	24Jun2006, 16:00	5.52
Hanabrand Outlet	2.94550	768.3	24Jun2006, 19:00	4.73
HP to W7	3.11250	1073.5	24Jun2006, 18:45	5.16
Hurley's Pond	3.11250	1175.1	24Jun2006, 16:15	5.16
Junction-1	0.42200	371.4	24Jun2006, 13:15	6.00
Junction-3	Not Specified	571.4	24Jun2006, 13:15	
Kellers Pond	0.34250	406.9	24Jun2006, 13:00	6.15
McDowel Pond	0.59000	386.6	24Jun2006, 13:15	4.89
McD to W7	0.59000	307.1	24Jun2006, 14:15	4.89
Mews Combined Basin	0.17200	169.7	24Jun2006, 13:15	6.16
Mews Storm sewer	0.25000	202.8	24Jun2006, 13:30	5.89
Old Mill Pond W3 gage	7.27050	1733.9	24Jun2006, 21:15	5.19
Old Mill-Wreck Pond	10.21600	2255.2	25Jun2006, 00:30	5.05
Osbonre to W1	6.71250	1753.2	24Jun2006, 19:45	5.12
Osborne Pond	6.71250	1756.9	24Jun2006, 19:15	5.12
Osborne Pond da	0.85000	910.4	24Jun2006, 12:45	5.06
Osborne Pond surface	0.03000	95.6	24Jun2006, 12:15	8.93
Parkway	0.04000	45.2	24Jun2006, 13:15	8.93
Rt34-W6	0.34250	285.2	24Jun2006, 14:30	6.15
Rt 35 culvert	2.94550	819.0	24Jun2006, 17:15	4.73
Rt 35 to Outlet	2.94550	768.3	24Jun2006, 19:00	4.73
Rt 71 East	0.00512	14.6	24Jun2006, 12:15	6.97
RT 71 w/Imp	0.02000	54.0	24Jun2006, 12:15	6.46
Slgc flowpath	0.02519	26.4	24Jun2006, 12:45	4.38
Slgc lake	0.64040	528.7	24Jun2006, 13:15	5.60
Slgc lakes only	0.00440	14.0	24Jun2006, 12:15	8.93

Slgc upper lake	0.42200	370.6	24Jun2006, 13:15	6.00
SL-SlH-SG	1.76000	1251.0	24Jun2006, 13:30	5.89
W1	0.07800	185.6	24Jun2006, 12:15	6.35
W1 gage	6.79050	1759.1	24Jun2006, 19:45	5.14
W1-W3	6.79050	1723.5	24Jun2006, 20:30	5.13
W2 Rt 35	0.13750	85.3	24Jun2006, 13:45	7.66
W2 subdivision at Baileys	0.16250	68.5	24Jun2006, 14:30	6.51
W3	0.48000	433.2	24Jun2006, 13:00	6.09
W5	2.33000	648.3	24Jun2006, 15:15	4.42
W5 add/l housing	0.07800	50.1	24Jun2006, 13:15	4.66
W5 combined	2.64550	771.8	24Jun2006, 15:00	4.57
W5 housing development	0.23750	155.0	24Jun2006, 13:15	5.97
W5-W2	2.80800	774.4	24Jun2006, 17:30	4.59
W6	0.93000	395.6	24Jun2006, 13:45	4.87
W6_Rt 34	0.34250	432.4	24Jun2006, 12:45	6.15
W6 gage	1.27250	664.9	24Jun2006, 14:15	5.21
W6-Hurley Pond	1.31250	595.6	24Jun2006, 16:30	5.33
W7	4.86250	1464.8	24Jun2006, 18:15	5.05
W7_Glen	1.16000	462.1	24Jun2006, 14:30	4.84
W7_McD	0.59000	390.7	24Jun2006, 13:15	4.89
W7 to Osborne Pond	4.86250	1443.7	24Jun2006, 19:30	5.05
W8 base flow	Not Specified	12.0	24Jun2006, 00:00	
W8 Mews	0.17200	179.0	24Jun2006, 13:00	6.16
W8 SLGC	0.21400	170.2	24Jun2006, 13:00	4.74
W8 urban lower	0.02519	26.7	24Jun2006, 12:30	4.38
W8 urban upper	0.25000	205.6	24Jun2006, 13:15	5.89
W9	1.80000	805.0	24Jun2006, 14:00	5.04
W9 junction	3.11250	1182.5	24Jun2006, 15:45	5.16
WPB-HAN Combined	10.21600	2426.6	24Jun2006, 20:30	5.06
WP Junction	Not Specified	2415.4	25Jun2006, 00:00	
Wreck Pond	Not Specified	619.4	25Jun2006, 11:15	
Albers Pond	0.97000	485.8	24Jun2006, 14:45	5.52

a : Storm event is from the hypothetical start time of 24 June 2006 @ 00:00 hours.

Table 3: HMS simulation results for 100-Year NRCS storm event using percentage curve created from Lower Monmouth dimensionless unit hydrograph (PRF 230)

HYDROLOGIC ELEMENT	DRAINAGE AREA (SQ. MILE)	PEAK FLOW (CFS)	TIME OF PEAK [a]	VOL. (IN)
Albers Pond	0.97	473.7	24Jun2006, 14:45	5.52
Albers pond da	0.93	629.6	24Jun2006, 13:15	5.41
Albers Pond Surface	0.04	125.8	24Jun2006, 12:15	8.93
Albers to Osborne	0.97	450.3	24Jun2006, 15:45	5.52
Hanabrand Outlet	2.9455	735.8	24Jun2006, 19:00	4.73
HP to W7	3.1125	1046.1	24Jun2006, 18:45	5.16
Hurley's Pond	3.1125	1142.5	24Jun2006, 16:00	5.16
Junction-1	0.422	346	24Jun2006, 13:15	6
Junction-3	Not Specified	540.4	24Jun2006, 13:15	
Kellers Pond	0.3425	386.9	24Jun2006, 13:00	6.15
McDowel Pond	0.59	367.6	24Jun2006, 13:30	4.89
McD to W7	0.59	296.9	24Jun2006, 14:15	4.89
Mews Combined Basin	0.172	159.7	24Jun2006, 13:15	6.16
Mews Stormsewer	0.25	187.5	24Jun2006, 13:30	5.89
Old Mill Pond W3 gage	7.2705	1707.2	24Jun2006, 21:15	5.19
Old Mill-Wreck Pond	10.216	2214.4	25Jun2006, 00:30	5.05
Osbonre to W1	6.7125	1719.4	24Jun2006, 19:30	5.12
Osborne Pond	6.7125	1722.6	24Jun2006, 19:00	5.12
Osborne Pond da	0.85	855.1	24Jun2006, 12:45	5.06
Osborne Pond surface	0.03	94.4	24Jun2006, 12:15	8.93
Parkway	0.04	41.7	24Jun2006, 13:15	8.93
Rt34-W6	0.3425	278.4	24Jun2006, 14:30	6.15
Rt 35 culvert	2.9455	774.9	24Jun2006, 17:15	4.73
Rt 35 to Outlet	2.9455	735.8	24Jun2006, 19:00	4.73
Rt 71 East	0.00512	14.4	24Jun2006, 12:15	6.97
RT 71 w/Imp	0.02	53.3	24Jun2006, 12:15	6.46
Slgc flowpath	0.02519	25.1	24Jun2006, 12:45	4.38
Slgc lake	0.6404	497.9	24Jun2006, 13:15	5.6
Slgc lakes only	0.0044	13.8	24Jun2006, 12:15	8.93

Slgc upper lake	0.422	345.6	24Jun2006, 13:15	6
SL-SlH-SG	1.76	1137.2	24Jun2006, 13:30	5.89
W1	0.078	183.1	24Jun2006, 12:15	6.35
W1 gage	6.7905	1725.6	24Jun2006, 19:30	5.14
W1-W3	6.7905	1692.2	24Jun2006, 20:30	5.13
W2 Rt 35	0.1375	77.8	24Jun2006, 14:00	7.66
W2 subdivision at Baileys	0.1625	62.8	24Jun2006, 14:45	6.51
W3	0.48	401	24Jun2006, 13:00	6.09
W5	2.33	597.4	24Jun2006, 15:45	4.42
W5 add/l housing	0.078	46	24Jun2006, 13:15	4.66
W5 combined	2.6455	708.2	24Jun2006, 15:15	4.57
W5 housing development	0.2375	142.4	24Jun2006, 13:15	5.97
W5-W2	2.808	731.2	24Jun2006, 17:30	4.59
W6	0.93	364.5	24Jun2006, 14:00	4.87
W6_Rt 34	0.3425	406.4	24Jun2006, 12:45	6.15
W6 gage	1.2725	635.6	24Jun2006, 14:15	5.21
W6-Hurley Pond	1.3125	577.4	24Jun2006, 16:30	5.33
W7	4.8625	1431.5	24Jun2006, 18:15	5.05
W7_Glen	1.16	422.9	24Jun2006, 14:45	4.84
W7_McD	0.59	358.5	24Jun2006, 13:15	4.89
W7 to Osborne Pond	4.8625	1410.3	24Jun2006, 19:30	5.05
W8 base flow	Not Specified	12	24Jun2006, 00:00	
W8 Mews	0.172	165.7	24Jun2006, 13:00	6.16
W8 SLGC	0.214	157.4	24Jun2006, 13:00	4.74
W8 urban lower	0.02519	25.4	24Jun2006, 12:30	4.38
W8 urban upper	0.25	188.8	24Jun2006, 13:15	5.89
W9	1.8	735.8	24Jun2006, 14:00	5.04
W9 junction	3.1125	1151.5	24Jun2006, 15:45	5.16
WPB-HAN Combined	10.216	2375.4	24Jun2006, 20:30	5.06
WP Junction	Not Specified	2383.6	25Jun2006, 00:00	
Wreck Pond	Not Specified	618.7	25Jun2006, 11:15	

a : Storm event is from the hypothetical start time of 24 June 2006 @ 00:00 hours.

Table 4: HMS simulation results for 100-Year NRCS storm event using percentage curve created from SCS standard dimensionless unit hydrograph (PRF 484) and 5% initial abstraction ratio

HYDROLOGIC ELEMENT	DRAINAGE AREA (SQ. MILE)	PEAK FLOW (CFS)	TIME OF PEAK [a]	VOL. (IN)
Albers Pond	0.97000	553.3	24Jun2006, 14:15	5.01
Albers pond da	0.93000	889.0	24Jun2006, 13:15	4.88
Albers Pond Surface	0.04000	132.8	24Jun2006, 12:15	8.93
Albers to Osborne	0.97000	500.7	24Jun2006, 15:15	5.01
Hanabrand Outlet	2.94550	869.0	24Jun2006, 18:45	4.15
HP to W7	3.11250	1104.5	24Jun2006, 18:00	4.57
Hurley's Pond	3.11250	1282.7	24Jun2006, 15:00	4.57
Junction-1	0.42200	478.2	24Jun2006, 13:15	5.50
Junction-3	Not Specified	700.4	24Jun2006, 13:15	
Kellers Pond	0.34250	496.1	24Jun2006, 12:45	5.62
McDowel Pond	0.59000	483.4	24Jun2006, 13:15	4.26
McDowel to W7	0.59000	356.0	24Jun2006, 14:00	4.26
Mews Combined Basin	0.17200	217.1	24Jun2006, 13:15	5.66
Mews Stormsewer	0.25000	264.7	24Jun2006, 13:30	5.39
Old Mill Pond W3 gage	7.27050	1674.3	24Jun2006, 20:15	4.61
Old Mill-Wreck Pond	10.21600	2226.3	24Jun2006, 23:30	4.47
Osborne to W1	6.71250	1685.9	24Jun2006, 18:30	4.54
Osborne Pond	6.71250	1687.5	24Jun2006, 18:00	4.54
Osborne Pond da	0.85000	1078.2	24Jun2006, 12:45	4.50
Osborne Pond surface	0.03000	99.6	24Jun2006, 12:15	8.93
Parkway	0.04000	64.8	24Jun2006, 13:15	8.93
Rt34-W6	0.34250	296.0	24Jun2006, 14:15	5.62
Rt 35 culvert	2.94550	946.0	24Jun2006, 17:15	4.15
Rt 35 to Outlet	2.94550	869.0	24Jun2006, 18:45	4.15
Rt 71 East	0.00512	14.6	24Jun2006, 12:15	6.67
RT 71 w/Imp	0.02000	52.6	24Jun2006, 12:15	6.04
Slgc flowpath	0.02519	29.1	24Jun2006, 12:45	3.71
Slgc lake	0.64040	663.6	24Jun2006, 13:15	5.07
Slgc lakes only	0.00440	14.6	24Jun2006, 12:15	8.93

Slgc upper lake	0.42200	477.2	24Jun2006, 13:15	5.50
SL-SlH-SG	1.76000	1650.0	24Jun2006, 13:30	5.37
W1	0.07800	172.4	24Jun2006, 12:15	5.78
W1 gage	6.79050	1692.1	24Jun2006, 18:30	4.56
W1-W3	6.79050	1666.0	24Jun2006, 19:30	4.55
W2 Rt 35	0.13750	147.7	24Jun2006, 13:15	7.13
W2 subdivision at Baileys	0.16250	90.2	24Jun2006, 14:30	5.95
W3	0.48000	543.7	24Jun2006, 13:00	5.50
W5	2.33000	856.5	24Jun2006, 15:30	3.84
W5 add/l housing	0.07800	63.2	24Jun2006, 13:15	4.06
W5 combined	2.64550	932.0	24Jun2006, 15:15	3.98
W5 housing development	0.23750	194.9	24Jun2006, 13:15	5.37
W5-W2	2.80800	924.4	24Jun2006, 17:15	4.01
W6	0.93000	502.9	24Jun2006, 14:00	4.27
W6_Rt 34	0.34250	524.5	24Jun2006, 12:45	5.62
W6 gage	1.27250	791.4	24Jun2006, 14:00	4.63
W6-Hurley Pond	1.31250	644.0	24Jun2006, 16:00	4.76
W7	4.86250	1427.4	24Jun2006, 17:30	4.45
W7_Glen	1.16000	600.4	24Jun2006, 14:30	4.21
W7_McD	0.59000	490.6	24Jun2006, 13:15	4.26
W7 to Osborne Pond	4.86250	1400.3	24Jun2006, 18:45	4.45
W8 base flow	Not Specified	12.0	24Jun2006, 00:00	
W8 Mews	0.17200	232.1	24Jun2006, 13:00	5.66
W8 SLGC	0.21400	211.3	24Jun2006, 13:00	4.15
W8 urban lower	0.02519	30.3	24Jun2006, 12:30	3.71
W8 urban upper	0.25000	270.4	24Jun2006, 13:15	5.39
W9	1.80000	1034.1	24Jun2006, 14:00	4.44
W9 junction	3.11250	1309.4	24Jun2006, 14:30	4.57
WPB-HAN Combined	10.21600	2494.0	24Jun2006, 19:15	4.48
WP Junction	Not Specified	2376.4	24Jun2006, 13:30	
Wreck Pond	Not Specified	610.0	25Jun2006, 09:00	
Albers Pond	0.97000	553.3	24Jun2006, 14:15	5.01

a : Storm event is from the hypothetical start time of 24 June 2006 @ 00:00 hours.

Table 5: HMS simulation results for 100-Year NRCS storm event using percentage curve created from Delmarva unit hydrograph (PRF 284) and 5% initial abstraction ratio

HYDROLOGIC ELEMENT	DRAINAGE AREA (SQ. MILE)	PEAK FLOW (CFS)	TIME OF PEAK [a]	VOL. (IN)
Albers Pond	0.97000	438.3	24Jun2006, 14:30	5.01
Albers pond da	0.93000	597.9	24Jun2006, 13:15	4.88
Albers Pond Surface	0.04000	127.5	24Jun2006, 12:15	8.93
Albers to Osborne	0.97000	414.8	24Jun2006, 15:45	5.01
Hanabrand Outlet	2.94550	653.7	24Jun2006, 19:00	4.15
HP to W7	3.11250	921.6	24Jun2006, 18:45	4.57
Hurley's Pond	3.11250	1008.9	24Jun2006, 16:15	4.57
Junction-1	0.42200	330.6	24Jun2006, 13:15	5.50
Junction-3	Not Specified	500.5	24Jun2006, 13:15	
Kellers Pond	0.34250	358.2	24Jun2006, 13:00	5.62
McDowel Pond	0.59000	327.9	24Jun2006, 13:15	4.26
McD to W7	0.59000	259.9	24Jun2006, 14:15	4.26
Mews Combined Basin	0.17200	151.6	24Jun2006, 13:15	5.66
Mews Storm sewer	0.25000	180.3	24Jun2006, 13:30	5.39
Old Mill Pond W3 gage	7.27050	1496.9	24Jun2006, 21:15	4.61
Old Mill-Wreck Pond	10.21600	1946.1	25Jun2006, 00:30	4.47
Osborne to W1	6.71250	1507.6	24Jun2006, 19:45	4.54
Osborne Pond	6.71250	1510.8	24Jun2006, 19:15	4.54
Osborne Pond da	0.85000	775.9	24Jun2006, 12:45	4.50
Osborne Pond surface	0.03000	95.6	24Jun2006, 12:15	8.93
Parkway	0.04000	45.2	24Jun2006, 13:15	8.93
Rt34-W6	0.34250	251.0	24Jun2006, 14:30	5.62
Rt 35 culvert	2.94550	696.1	24Jun2006, 17:15	4.15
Rt 35 to Outlet	2.94550	653.7	24Jun2006, 19:00	4.15
Rt 71 East	0.00512	13.8	24Jun2006, 12:15	6.67
RT 71 w/Imp	0.02000	49.4	24Jun2006, 12:15	6.04
Slgc flowpath	0.02519	21.5	24Jun2006, 12:45	3.71
Slgc lake	0.64040	462.2	24Jun2006, 13:15	5.07
Slgc lakes only	0.00440	14.0	24Jun2006, 12:15	8.93

Slgc upper lake	0.42200	329.7	24Jun2006, 13:15	5.50
SL-SlH-SG	1.76000	1110.1	24Jun2006, 13:30	5.37
W1	0.07800	160.2	24Jun2006, 12:15	5.78
W1 gage	6.79050	1513.1	24Jun2006, 19:45	4.56
W1-W3	6.79050	1484.2	24Jun2006, 20:30	4.55
W2 Rt 35	0.13750	100.2	24Jun2006, 13:15	7.13
W2 subdivision at Baileys	0.16250	59.7	24Jun2006, 14:30	5.95
W3	0.48000	372.5	24Jun2006, 13:00	5.50
W5	2.33000	553.6	24Jun2006, 15:15	3.84
W5 add/l housing	0.07800	42.3	24Jun2006, 13:15	4.06
W5 combined	2.64550	659.4	24Jun2006, 15:00	3.98
W5 housing development	0.23750	131.2	24Jun2006, 13:15	5.37
W5-W2	2.80800	663.2	24Jun2006, 17:15	4.01
W6	0.93000	333.4	24Jun2006, 13:45	4.27
W6_Rt 34	0.34250	380.5	24Jun2006, 12:45	5.62
W6 gage	1.27250	568.7	24Jun2006, 14:15	4.63
W6-Hurley Pond	1.31250	515.9	24Jun2006, 16:30	4.76
W7	4.86250	1255.6	24Jun2006, 18:15	4.45
W7_Glen	1.16000	392.9	24Jun2006, 14:30	4.21
W7_McD	0.59000	328.8	24Jun2006, 13:15	4.26
W7 to Osborne Pond	4.86250	1237.4	24Jun2006, 19:30	4.45
W8 base flow	Not Specified	12.0	24Jun2006, 00:00	
W8 Mews	0.17200	159.9	24Jun2006, 13:00	5.66
W8 SLGC	0.21400	144.0	24Jun2006, 13:00	4.15
W8 urban lower	0.02519	21.7	24Jun2006, 12:30	3.71
W8 urban upper	0.25000	182.8	24Jun2006, 13:15	5.39
W9	1.80000	683.4	24Jun2006, 14:00	4.44
W9 junction	3.11250	1015.3	24Jun2006, 15:45	4.57
WPB-HAN Combined	10.21600	2084.5	24Jun2006, 20:15	4.48
WP Junction	Not Specified	2098.6	25Jun2006, 00:00	
Wreck Pond	Not Specified	607.7	25Jun2006, 10:30	
Albers Pond	0.97000	438.3	24Jun2006, 14:30	5.01

a : Storm event is from the hypothetical start time of 24 June 2006 @ 00:00 hours.

Table 6: HMS simulation results for 100-Year NRCS storm event using percentage curve created from LMDUH (PRF 230) and 5% initial abstraction ratio

HYDROLOGIC ELEMENT	DRAINAGE AREA (SQ. MILE)	PEAK FLOW (CFS)	TIME OF PEAK [a]	VOL. (IN)
Albers Pond	0.97	428.2	24Jun2006, 14:30	5.01
Albers Pond da	0.93	549.4	24Jun2006, 13:15	4.88
Albers Pond Surface	0.04	125.8	24Jun2006, 12:15	8.93
Albers to Osborne	0.97	405.8	24Jun2006, 15:45	5.01
Hanabrand Outlet	2.9455	633.3	24Jun2006, 19:00	4.15
HP to W7	3.1125	898.2	24Jun2006, 18:45	4.57
Hurley's Pond	3.1125	981.4	24Jun2006, 16:00	4.57
Junction-1	0.422	308.3	24Jun2006, 13:15	5.5
Junction-3	Not Specified	478.2	24Jun2006, 13:15	
Kellers Pond	0.3425	340.7	24Jun2006, 13:00	5.62
McDowel Pond	0.59	304.5	24Jun2006, 13:15	4.26
McD to W7	0.59	251	24Jun2006, 14:30	4.26
Mews Combined Basin	0.172	142.7	24Jun2006, 13:15	5.66
Mews Stormsewer	0.25	166.9	24Jun2006, 13:30	5.39
Old Mill Pond W3 gage	7.2705	1474	24Jun2006, 21:15	4.61
Old Mill-Wreck Pond	10.216	1916.7	25Jun2006, 00:30	4.47
Osbonre to W1	6.7125	1478.7	24Jun2006, 19:45	4.54
Osborne Pond	6.7125	1481.5	24Jun2006, 19:15	4.54
Osborne Pond da	0.85	729.3	24Jun2006, 12:45	4.5
Osborne Pond surface	0.03	94.4	24Jun2006, 12:15	8.93
Parkway	0.04	41.7	24Jun2006, 13:15	8.93
Rt34-W6	0.3425	245.2	24Jun2006, 14:30	5.62
Rt 35 culvert	2.9455	666.6	24Jun2006, 17:15	4.15
Rt 35 to Outlet	2.9455	633.3	24Jun2006, 19:00	4.15
Rt 71 East	0.00512	13.6	24Jun2006, 12:15	6.67
RT 71 w/Imp	0.02	48.7	24Jun2006, 12:15	6.04
Slgc flowpath	0.02519	20.5	24Jun2006, 12:45	3.71
Slgc lake	0.6404	440	24Jun2006, 13:15	5.07
Slgc lakes only	0.0044	13.8	24Jun2006, 12:15	8.93

Slgc upper lake	0.422	308.1	24Jun2006, 13:15	5.5
SL-SlH-SG	1.76	1010	24Jun2006, 13:30	5.37
W1	0.078	158.1	24Jun2006, 12:15	5.78
W1 gage	6.7905	1484.2	24Jun2006, 19:45	4.56
W1-W3	6.7905	1457.5	24Jun2006, 20:30	4.55
W2 Rt 35	0.1375	69.3	24Jun2006, 14:00	7.13
W2 subdivision at Baileys	0.1625	54.9	24Jun2006, 14:30	5.95
W3	0.48	345.2	24Jun2006, 13:00	5.5
W5	2.33	510	24Jun2006, 15:45	3.84
W5 add/l housing	0.078	38.9	24Jun2006, 13:15	4.06
W5 combined	2.6455	605.7	24Jun2006, 15:15	3.98
W5 housing development	0.2375	120.8	24Jun2006, 13:15	5.37
W5-W2	2.808	627	24Jun2006, 17:15	4.01
W6	0.93	307.1	24Jun2006, 14:00	4.27
W6_Rt 34	0.3425	357.8	24Jun2006, 12:45	5.62
W6 gage	1.2725	544.4	24Jun2006, 14:15	4.63
W6-Hurley Pond	1.3125	500.5	24Jun2006, 16:30	4.76
W7	4.8625	1226.7	24Jun2006, 18:15	4.45
W7_Glen	1.16	360.1	24Jun2006, 14:45	4.21
W7_McD	0.59	302.1	24Jun2006, 13:15	4.26
W7 to Osborne Pond	4.8625	1208.8	24Jun2006, 19:30	4.45
W8 base flow	Not Specified	12	24Jun2006, 00:00	
W8 Mews	0.172	148.2	24Jun2006, 13:00	5.66
W8 SLGC	0.214	133.3	24Jun2006, 13:00	4.15
W8 urban lower	0.02519	20.7	24Jun2006, 12:30	3.71
W8 urban upper	0.25	167.9	24Jun2006, 13:15	5.39
W9	1.8	626	24Jun2006, 14:00	4.44
W9 junction	3.1125	988.8	24Jun2006, 15:45	4.57
WPB-HAN Combined	10.216	2048.5	24Jun2006, 20:15	4.48
WP Junction	Not Specified	2076.8	25Jun2006, 00:00	
Wreck Pond	Not Specified	607.1	25Jun2006, 10:45	

a : Storm event is from the hypothetical start time of 24 June 2006 @ 00:00 hours.

Table 7: HMS simulation results for 50 Year NRCS storm event using percentage curve created from SCS standard dimensionless unit hydrograph (PRF 484)

HYDROLOGIC ELEMENT	DRAINAGE AREA (SQ. MILE)	PEAK FLOW (CFS)	TIME OF PEAK [a]	VOLUME (INCH)
Albers Pond	0.97000	491.7	24Jun2006, 14:15	4.42
Albers pond da	0.93000	810.5	24Jun2006, 13:15	4.31
Albers Pond Surface	0.04000	113.9	24Jun2006, 12:15	7.66
Albers to Osborne	0.97000	440.9	24Jun2006, 15:30	4.42
Hanabrand Outlet	2.94550	786.9	24Jun2006, 18:45	3.73
HP to W7	3.11250	987.4	24Jun2006, 18:00	4.13
Hurley's Pond	3.11250	1153.3	24Jun2006, 15:00	4.13
Junction-1	0.42200	431.6	24Jun2006, 13:15	4.84
Junction-3	Not Specified	638.4	24Jun2006, 13:15	
Kellers Pond	0.34250	448.6	24Jun2006, 12:45	5.00
McDowel Pond	0.59000	474.6	24Jun2006, 13:15	3.84
McD to W7	0.59000	329.0	24Jun2006, 14:00	3.84
Mews Combined Basin	0.17200	194.5	24Jun2006, 13:15	4.99
Mews Stormsewer	0.25000	240.7	24Jun2006, 13:30	4.74
Old Mill Pond W3 gage	7.27050	1476.3	24Jun2006, 20:30	4.14
Old Mill-Wreck Pond	10.21600	1978.8	24Jun2006, 23:45	4.01
Osbonre to W1	6.71250	1482.5	24Jun2006, 19:00	4.07
Osborne Pond	6.71250	1485.0	24Jun2006, 18:30	4.07
osborne pond da	0.85000	995.5	24Jun2006, 12:45	3.99
Osborne Pond surface	0.03000	85.4	24Jun2006, 12:15	7.66
parkway	0.04000	55.6	24Jun2006, 13:15	7.66
rt34-W6	0.34250	268.4	24Jun2006, 14:15	5.00
Rt 35 culvert	2.94550	857.4	24Jun2006, 17:15	3.73
Rt 35 to Outlet	2.94550	786.9	24Jun2006, 18:45	3.73
Rt 71 East	0.00512	12.9	24Jun2006, 12:15	5.75
RT 71 w/Imp	0.02000	47.3	24Jun2006, 12:15	5.27
slgc flowpath	0.02519	27.6	24Jun2006, 12:45	3.38
slgc lake	0.64040	603.2	24Jun2006, 13:15	4.48
slgc lakes only	0.00440	12.5	24Jun2006, 12:15	7.66

slgc upper lake	0.42200	430.3	24Jun2006, 13:15	4.84
SL-SlH-SG	1.76000	1502.0	24Jun2006, 13:30	4.74
W1	0.07800	160.4	24Jun2006, 12:15	5.23
W1 gage	6.79050	1487.9	24Jun2006, 19:00	4.09
W1-W3	6.79050	1466.7	24Jun2006, 19:45	4.08
W2 Rt 35	0.13750	102.7	24Jun2006, 13:45	6.52
w2 subdivision at Bailieys	0.16250	81.8	24Jun2006, 14:30	5.44
W3	0.48000	503.0	24Jun2006, 13:00	4.99
W5	2.33000	770.8	24Jun2006, 15:30	3.43
W5 add/l housing	0.07800	58.3	24Jun2006, 13:15	3.62
w5 combined	2.64550	840.7	24Jun2006, 15:30	3.57
w5 housing development	0.23750	179.8	24Jun2006, 13:15	4.94
W5-W2	2.80800	830.2	24Jun2006, 17:15	3.60
W6	0.93000	460.0	24Jun2006, 14:00	3.89
W6_Rt 34	0.34250	480.6	24Jun2006, 12:45	5.00
W6 gage	1.27250	720.3	24Jun2006, 14:00	4.19
w6-Hurley Pond	1.31250	581.1	24Jun2006, 16:00	4.29
W7	4.86250	1277.2	24Jun2006, 17:30	4.02
W7_Glen	1.16000	552.4	24Jun2006, 14:30	3.79
W7_McD	0.59000	456.0	24Jun2006, 13:15	3.84
W7 to Osborne Pond	4.86250	1251.0	24Jun2006, 18:45	4.02
W8 base flow	Not Specified	12.0	24Jun2006, 00:00	
W8 Mews	0.17200	211.3	24Jun2006, 13:00	4.99
W8 SLGC	0.21400	195.3	24Jun2006, 13:00	3.70
W8 urban lower	0.02519	28.6	24Jun2006, 12:30	3.38
W8 urban upper	0.25000	245.9	24Jun2006, 13:15	4.74
W9	1.80000	946.4	24Jun2006, 14:00	4.02
W9 junction	3.11250	1179.6	24Jun2006, 14:30	4.13
WPB-HAN Combined	10.21600	2211.6	24Jun2006, 19:30	4.02
WP Junction	Not Specified	2152.2	24Jun2006, 13:30	
Wreck Pond	Not Specified	600.3	25Jun2006, 08:45	
Albers Pond	0.97000	491.7	24Jun2006, 14:15	4.42

a : Storm event is from the hypothetical start time of 24 June 2006 @ 00:00 hours.

Table 8: HMS simulation results for 50 Year NRCS storm event using percentage curve created from Delmarva unit hydrograph (PRF 284)

HYDROLOGIC ELEMENT	DRAINAGE AREA (SQ. MILE)	PEAK FLOW (CFS)	TIME OF PEAK [a]	VOLUME (INCH)
Albers Pond	0.97000	390.4	24Jun2006, 14:45	4.42
Albers pond da	0.93000	539.0	24Jun2006, 13:15	4.31
Albers Pond Surface	0.04000	109.3	24Jun2006, 12:15	7.66
Albers to Osborne	0.97000	362.2	24Jun2006, 15:45	4.41
Hanabrand Outlet	2.94550	589.6	24Jun2006, 19:00	3.73
HP to W7	3.11250	824.9	24Jun2006, 19:00	4.13
Hurley's Pond	3.11250	911.2	24Jun2006, 16:15	4.13
Junction-1	0.42200	298.4	24Jun2006, 13:15	4.84
Junction-3	Not Specified	454.5	24Jun2006, 13:15	
Kellers Pond	0.34250	326.3	24Jun2006, 13:00	5.00
McDowel Pond	0.59000	299.3	24Jun2006, 13:30	3.84
McD to W7	0.59000	236.1	24Jun2006, 14:15	3.84
Mews Combined Basin	0.17200	137.0	24Jun2006, 13:15	4.99
Mews Stormsewer	0.25000	162.8	24Jun2006, 13:30	4.74
Old Mill Pond W3 gage	7.27050	1325.4	24Jun2006, 21:30	4.14
Old Mill-Wreck Pond	10.21600	1730.1	25Jun2006, 00:45	4.01
Osbonre to W1	6.71250	1337.6	24Jun2006, 20:00	4.07
Osborne Pond	6.71250	1340.7	24Jun2006, 19:30	4.07
osborne pond da	0.85000	709.0	24Jun2006, 12:45	3.99
Osborne Pond surface	0.03000	82.0	24Jun2006, 12:15	7.66
parkway	0.04000	38.8	24Jun2006, 13:15	7.66
rt34-W6	0.34250	227.3	24Jun2006, 14:30	5.00
Rt 35 culvert	2.94550	627.4	24Jun2006, 17:15	3.73
Rt 35 to Outlet	2.94550	589.6	24Jun2006, 19:00	3.73
Rt 71 East	0.00512	12.2	24Jun2006, 12:15	5.75
RT 71 w/Imp	0.02000	44.4	24Jun2006, 12:15	5.27
slgc flowpath	0.02519	20.1	24Jun2006, 12:45	3.38
slgc lake	0.64040	418.2	24Jun2006, 13:15	4.48
slgc lakes only	0.00440	12.0	24Jun2006, 12:15	7.66

slgc upper lake	0.42200	297.1	24Jun2006, 13:15	4.84
SL-SlH-SG	1.76000	1002.2	24Jun2006, 13:30	4.74
W1	0.07800	148.9	24Jun2006, 12:15	5.23
W1 gage	6.79050	1342.6	24Jun2006, 20:00	4.08
W1-W3	6.79050	1314.8	24Jun2006, 20:45	4.08
W2 Rt 35	0.13750	68.4	24Jun2006, 13:45	6.52
w2 subdivision at Bailieys	0.16250	53.5	24Jun2006, 14:30	5.44
W3	0.48000	341.5	24Jun2006, 13:00	4.99
W5	2.33000	491.5	24Jun2006, 15:30	3.43
W5 add/l housing	0.07800	38.4	24Jun2006, 13:15	3.62
w5 combined	2.64550	587.2	24Jun2006, 15:15	3.57
w5 housing development	0.23750	119.4	24Jun2006, 13:15	4.94
W5-W2	2.80800	591.9	24Jun2006, 17:30	3.60
W6	0.93000	297.6	24Jun2006, 14:00	3.89
W6_Rt 34	0.34250	347.2	24Jun2006, 12:45	5.00
W6 gage	1.27250	512.7	24Jun2006, 14:15	4.19
w6-Hurley Pond	1.31250	463.6	24Jun2006, 16:30	4.29
W7	4.86250	1123.9	24Jun2006, 18:30	4.02
W7_Glen	1.16000	356.4	24Jun2006, 14:30	3.79
W7_McD	0.59000	301.2	24Jun2006, 13:15	3.84
W7 to Osborne Pond	4.86250	1106.9	24Jun2006, 19:45	4.02
W8 base flow	Not Specified	12.0	24Jun2006, 00:00	
W8 Mews	0.17200	145.0	24Jun2006, 13:00	4.99
W8 SLGC	0.21400	131.2	24Jun2006, 13:00	3.70
W8 urban lower	0.02519	20.2	24Jun2006, 12:30	3.38
W8 urban upper	0.25000	164.9	24Jun2006, 13:15	4.74
W9	1.80000	615.7	24Jun2006, 14:00	4.02
W9 junction	3.11250	917.1	24Jun2006, 16:00	4.13
WPB-HAN Combined	10.21600	1849.9	24Jun2006, 20:45	4.02
WP Junction	Not Specified	1864.0	25Jun2006, 00:15	
Wreck Pond	Not Specified	598.0	25Jun2006, 10:15	
Albers Pond	0.97000	390.4	24Jun2006, 14:45	4.42

a : Storm event is from the hypothetical start time of 24 June 2006 @ 00:00 hours

Table 9: HMS simulation results for 50 Year NRCS storm event using percentage curve created from Lower Monmouth dimensionless unit hydrograph (PRF 230)

HYDROLOGIC ELEMENT	DRAINAGE AREA (SQ. MILE)	PEAK FLOW (CFS)	TIME OF PEAK [a]	VOLUME (INCH)
Albers Pond	0.97000	379.6	24Jun2006, 14:45	4.42
Albers pond da	0.93000	494.6	24Jun2006, 13:15	4.31
Albers Pond Surface	0.04000	107.9	24Jun2006, 12:15	7.66
Albers to Osborne	0.97000	353.1	24Jun2006, 15:45	4.41
Hanabrand Outlet	2.94550	565.2	24Jun2006, 19:00	3.73
HP to W7	3.11250	804.1	24Jun2006, 19:00	4.13
Hurley's Pond	3.11250	886.3	24Jun2006, 16:15	4.13
Junction-1	0.42200	278.1	24Jun2006, 13:15	4.84
Junction-3	Not Specified	431.3	24Jun2006, 13:00	
Kellers Pond	0.34250	310.3	24Jun2006, 13:00	5.00
McDowel Pond	0.59000	292.1	24Jun2006, 13:15	3.84
McD to W7	0.59000	229.8	24Jun2006, 14:30	3.84
Mews Combined Basin	0.17200	129.0	24Jun2006, 13:15	4.99
Mews Stormsewer	0.25000	150.5	24Jun2006, 13:30	4.74
Old Mill Pond W3 gage	7.27050	1305.8	24Jun2006, 21:30	4.14
Old Mill-Wreck Pond	10.21600	1699.1	25Jun2006, 00:45	4.01
Osbonre to W1	6.71250	1313.1	24Jun2006, 20:00	4.07
Osborne Pond	6.71250	1315.9	24Jun2006, 19:30	4.07
osborne pond da	0.85000	665.7	24Jun2006, 12:45	3.99
Osborne Pond surface	0.03000	81.0	24Jun2006, 12:15	7.66
parkway	0.04000	35.7	24Jun2006, 13:15	7.66
rt34-W6	0.34250	222.0	24Jun2006, 14:30	5.00
Rt 35 culvert	2.94550	594.4	24Jun2006, 17:15	3.73
Rt 35 to Outlet	2.94550	565.2	24Jun2006, 19:00	3.73
Rt 71 East	0.00512	12.0	24Jun2006, 12:15	5.75
RT 71 w/Imp	0.02000	43.8	24Jun2006, 12:15	5.27
slgc flowpath	0.02519	19.1	24Jun2006, 12:45	3.38
slgc lake	0.64040	392.3	24Jun2006, 13:30	4.48
slgc lakes only	0.00440	11.9	24Jun2006, 12:15	7.66

slgc upper lake	0.42200	277.4	24Jun2006, 13:15	4.84
SL-SlH-SG	1.76000	910.8	24Jun2006, 13:30	4.74
W1	0.07800	146.9	24Jun2006, 12:15	5.23
W1 gage	6.79050	1318.0	24Jun2006, 20:00	4.08
W1-W3	6.79050	1292.0	24Jun2006, 20:45	4.08
W2 Rt 35	0.13750	62.6	24Jun2006, 14:00	6.52
w2 subdivision at Baileys	0.16250	49.2	24Jun2006, 14:45	5.44
W3	0.48000	316.2	24Jun2006, 13:00	4.99
W5	2.33000	454.4	24Jun2006, 15:45	3.43
W5 add/l housing	0.07800	35.2	24Jun2006, 13:15	3.62
w5 combined	2.64550	540.9	24Jun2006, 15:30	3.57
w5 housing development	0.23750	109.7	24Jun2006, 13:15	4.94
W5-W2	2.80800	559.5	24Jun2006, 17:30	3.60
W6	0.93000	275.2	24Jun2006, 14:00	3.89
W6_Rt 34	0.34250	326.3	24Jun2006, 12:45	5.00
W6 gage	1.27250	491.8	24Jun2006, 14:30	4.19
w6-Hurley Pond	1.31250	449.7	24Jun2006, 16:30	4.29
W7	4.86250	1099.2	24Jun2006, 18:30	4.02
W7_Glen	1.16000	327.0	24Jun2006, 14:45	3.79
W7_McD	0.59000	276.3	24Jun2006, 13:15	3.84
W7 to Osborne Pond	4.86250	1081.6	24Jun2006, 19:45	4.02
W8 base flow	Not Specified	12.0	24Jun2006, 00:00	
W8 Mews	0.17200	134.2	24Jun2006, 13:00	4.99
W8 SLGC	0.21400	121.4	24Jun2006, 13:00	3.70
W8 urban lower	0.02519	19.2	24Jun2006, 12:30	3.38
W8 urban upper	0.25000	151.4	24Jun2006, 13:15	4.74
W9	1.80000	564.3	24Jun2006, 14:15	4.02
W9 junction	3.11250	892.5	24Jun2006, 16:00	4.13
WPB-HAN Combined	10.21600	1812.2	24Jun2006, 20:45	4.02
WP Junction	Not Specified	1839.6	25Jun2006, 00:15	
Wreck Pond	Not Specified	597.5	25Jun2006, 10:15	
Albers Pond	0.97000	379.6	24Jun2006, 14:45	4.42

a : Storm event is from the hypothetical start time of 24 June 2006 @ 00:00 hours.

Table 10: HMS simulation results for 25 Year NRCS storm event using percentage curve created from SCS standard dimensionless unit hydrograph (PRF 484)

HYDROLOGIC ELEMENT	DRAINAGE AREA (SQ. MILE)	PEAK FLOW (CFS)	TIME OF PEAK [a]	VOLUME (INCH)
Albers Pond	0.97000	380.4	24Jun2006, 14:15	3.46
Albers pond da	0.93000	622.1	24Jun2006, 13:15	3.36
Albers Pond Surface	0.04000	96.9	24Jun2006, 12:15	6.52
Albers to Osborne	0.97000	327.3	24Jun2006, 15:30	3.46
Hanabrand Outlet	2.94550	583.2	24Jun2006, 19:00	2.88
HP to W7	3.11250	725.6	24Jun2006, 18:15	3.26
Hurley's Pond	3.11250	852.5	24Jun2006, 15:15	3.26
Junction-1	0.42200	340.2	24Jun2006, 13:15	3.83
Junction-3	Not Specified	496.9	24Jun2006, 13:15	
Kellers Pond	0.34250	347.8	24Jun2006, 12:45	4.01
McDowel Pond	0.59000	363.4	24Jun2006, 13:30	2.94
McD to W7	0.59000	245.0	24Jun2006, 14:15	2.94
Mews Combined Basin	0.17200	154.3	24Jun2006, 13:15	3.97
Mews Stormsewer	0.25000	189.8	24Jun2006, 13:30	3.74
Old Mill Pond W3 gage	7.27050	1083.7	24Jun2006, 20:45	3.23
Old Mill-Wreck Pond	10.21600	1467.0	24Jun2006, 23:45	3.12
Osbonre to W1	6.71250	1081.2	24Jun2006, 19:30	3.17
Osborne Pond	6.71250	1083.5	24Jun2006, 19:00	3.17
osborne pond da	0.85000	757.1	24Jun2006, 12:45	3.08
Osborne Pond surface	0.03000	72.7	24Jun2006, 12:15	6.52
parkway	0.04000	47.3	24Jun2006, 13:15	6.52
rt34-W6	0.34250	208.5	24Jun2006, 14:15	4.01
Rt 35 culvert	2.94550	634.1	24Jun2006, 17:30	2.88
Rt 35 to Outlet	2.94550	583.2	24Jun2006, 19:00	2.88
Rt 71 East	0.00512	10.6	24Jun2006, 12:15	4.67
RT 71 w/Imp	0.02000	38.4	24Jun2006, 12:15	4.22
slgc flowpath	0.02519	20.3	24Jun2006, 12:45	2.53
slgc lake	0.64040	466.7	24Jun2006, 13:15	3.51
slgc lakes only	0.00440	10.7	24Jun2006, 12:15	6.52

slgc upper lake	0.42200	338.0	24Jun2006, 13:15	3.83
SL-SlH-SG	1.76000	1180.0	24Jun2006, 13:30	3.73
W1	0.07800	126.3	24Jun2006, 12:15	4.26
W1 gage	6.79050	1085.5	24Jun2006, 19:30	3.19
W1-W3	6.79050	1070.6	24Jun2006, 20:00	3.18
W2 Rt 35	0.13750	80.6	24Jun2006, 13:45	5.52
w2 subdivision at Baileys	0.16250	62.1	24Jun2006, 14:30	4.53
W3	0.48000	387.8	24Jun2006, 13:00	4.04
W5	2.33000	565.6	24Jun2006, 15:45	2.59
W5 add/l housing	0.07800	43.4	24Jun2006, 13:15	2.74
w5 combined	2.64550	619.2	24Jun2006, 15:30	2.72
w5 housing development	0.23750	134.5	24Jun2006, 13:15	4.05
W5-W2	2.80800	613.4	24Jun2006, 17:30	2.75
W6	0.93000	335.3	24Jun2006, 14:00	3.06
W6_Rt 34	0.34250	378.9	24Jun2006, 12:45	4.01
W6 gage	1.27250	534.8	24Jun2006, 14:00	3.32
w6-Hurley Pond	1.31250	436.4	24Jun2006, 16:15	3.41
W7	4.86250	937.5	24Jun2006, 17:45	3.13
W7_Glen	1.16000	413.8	24Jun2006, 14:30	2.89
W7_McD	0.59000	341.8	24Jun2006, 13:15	2.94
W7 to Osborne Pond	4.86250	916.9	24Jun2006, 19:00	3.13
W8 base flow	Not Specified	12.0	24Jun2006, 00:00	
W8 Mews	0.17200	168.3	24Jun2006, 13:00	3.97
W8 SLGC	0.21400	146.7	24Jun2006, 13:00	2.81
W8 urban lower	0.02519	21.0	24Jun2006, 12:30	2.53
W8 urban upper	0.25000	193.5	24Jun2006, 13:15	3.74
W9	1.80000	701.3	24Jun2006, 14:00	3.15
W9 junction	3.11250	875.8	24Jun2006, 14:45	3.26
WPB-HAN Combined	10.21600	1627.0	24Jun2006, 19:30	3.13
WP Junction	Not Specified	1694.3	24Jun2006, 13:30	
Wreck Pond	Not Specified	582.7	25Jun2006, 07:45	

a : Storm event is from the hypothetical start time of 24 June 2006 @ 00:00 hours.

Table 11: HMS simulation results for 25 Year NRCS storm event using percentage curve created from Delmarva unit hydrograph (PRF 284)

HYDROLOGIC ELEMENT	DRAINAGE AREA (SQ. MILE)	PEAK FLOW (CFS)	TIME OF PEAK [a]	VOLUME (INCH)
Albers Pond	0.97000	284.0	24Jun2006, 14:45	3.46
Albers pond da	0.93000	411.4	24Jun2006, 13:15	3.36
Albers Pond Surface	0.04000	93.1	24Jun2006, 12:15	6.52
Albers to Osborne	0.97000	263.9	24Jun2006, 16:00	3.46
Hanabrand Outlet	2.94550	438.9	24Jun2006, 19:15	2.88
HP to W7	3.11250	610.6	24Jun2006, 19:15	3.26
Hurley's Pond	3.11250	684.7	24Jun2006, 16:30	3.26
Junction-1	0.42200	233.5	24Jun2006, 13:15	3.83
Junction-3	Not Specified	366.9	24Jun2006, 13:15	
Kellers Pond	0.34250	252.2	24Jun2006, 13:00	4.01
McDowel Pond	0.59000	222.6	24Jun2006, 13:30	2.94
McD to W7	0.59000	175.9	24Jun2006, 14:30	2.94
Mews Combined Basin	0.17200	107.5	24Jun2006, 13:15	3.97
Mews Stormsewer	0.25000	127.6	24Jun2006, 13:30	3.74
Old Mill Pond W3 gage	7.27050	982.2	24Jun2006, 21:45	3.23
Old Mill-Wreck Pond	10.21600	1288.0	25Jun2006, 00:45	3.12
Osbonre to W1	6.71250	985.5	24Jun2006, 20:15	3.17
Osborne Pond	6.71250	987.8	24Jun2006, 19:45	3.17
osborne pond da	0.85000	534.9	24Jun2006, 12:45	3.08
Osborne Pond surface	0.03000	69.8	24Jun2006, 12:15	6.52
parkway	0.04000	33.0	24Jun2006, 13:15	6.52
rt34-W6	0.34250	176.5	24Jun2006, 14:30	4.01
Rt 35 culvert	2.94550	466.2	24Jun2006, 17:30	2.88
Rt 35 to Outlet	2.94550	438.9	24Jun2006, 19:15	2.88
Rt 71 East	0.00512	10.0	24Jun2006, 12:15	4.67
RT 71 w/Imp	0.02000	35.9	24Jun2006, 12:15	4.22
slgc flowpath	0.02519	14.7	24Jun2006, 12:45	2.53
slgc lake	0.64040	336.0	24Jun2006, 13:15	3.51
slgc lakes only	0.00440	10.2	24Jun2006, 12:15	6.52

slgc upper lake	0.42200	232.7	24Jun2006, 13:30	3.83
SL-SlH-SG	1.76000	783.6	24Jun2006, 13:30	3.73
W1	0.07800	116.5	24Jun2006, 12:15	4.26
W1 gage	6.79050	989.6	24Jun2006, 20:15	3.18
W1-W3	6.79050	969.7	24Jun2006, 21:15	3.18
W2 Rt 35	0.13750	53.6	24Jun2006, 13:45	5.52
w2 subdivision at Baillieys	0.16250	40.7	24Jun2006, 14:30	4.53
W3	0.48000	261.8	24Jun2006, 13:00	4.04
W5	2.33000	359.9	24Jun2006, 15:45	2.59
W5 add/l housing	0.07800	28.4	24Jun2006, 13:15	2.74
w5 combined	2.64550	432.1	24Jun2006, 15:15	2.72
w5 housing development	0.23750	88.9	24Jun2006, 13:15	4.05
W5-W2	2.80800	437.7	24Jun2006, 17:30	2.75
W6	0.93000	216.4	24Jun2006, 14:00	3.06
W6_Rt 34	0.34250	272.1	24Jun2006, 12:45	4.01
W6 gage	1.27250	384.1	24Jun2006, 14:30	3.32
w6-Hurley Pond	1.31250	350.5	24Jun2006, 16:30	3.41
W7	4.86250	830.9	24Jun2006, 18:45	3.13
W7_Glen	1.16000	266.1	24Jun2006, 14:30	2.89
W7_McD	0.59000	224.5	24Jun2006, 13:15	2.94
W7 to Osborne Pond	4.86250	817.1	24Jun2006, 20:00	3.13
W8 base flow	Not Specified	12.0	24Jun2006, 00:00	
W8 Mews	0.17200	114.9	24Jun2006, 13:00	3.97
W8 SLGC	0.21400	97.9	24Jun2006, 13:00	2.81
W8 urban lower	0.02519	14.7	24Jun2006, 12:30	2.53
W8 urban upper	0.25000	129.2	24Jun2006, 13:15	3.74
W9	1.80000	455.1	24Jun2006, 14:00	3.15
W9 junction	3.11250	691.0	24Jun2006, 16:00	3.26
WPB-HAN Combined	10.21600	1370.1	24Jun2006, 20:45	3.13
WP Junction	Not Specified	1398.1	25Jun2006, 00:15	
Wreck Pond	Not Specified	580.9	25Jun2006, 09:15	

a : Storm event is from the hypothetical start time of 24 June 2006 @ 00:00 hours.

Table 12: HMS simulation results for 25 Year NRCS storm event using percentage curve created from Lower Monmouth dimensionless unit hydrograph (PRF 230)

HYDROLOGIC ELEMENT	DRAINAGE AREA (SQ. MILE)	PEAK FLOW (CFS)	TIME OF PEAK [a]	VOLUME (INCH)
Albers Pond	0.97000	276.9	24Jun2006, 14:45	3.46
Albers pond da	0.93000	377.4	24Jun2006, 13:15	3.36
Albers Pond Surface	0.04000	91.9	24Jun2006, 12:15	6.52
Albers to Osborne	0.97000	257.8	24Jun2006, 16:00	3.46
Hanabrand Outlet	2.94550	420.9	24Jun2006, 19:00	2.88
HP to W7	3.11250	595.2	24Jun2006, 19:15	3.26
Hurley's Pond	3.11250	666.3	24Jun2006, 16:15	3.26
Junction-1	0.42200	217.8	24Jun2006, 13:15	3.83
Junction-3	Not Specified	341.9	24Jun2006, 13:15	
Kellers Pond	0.34250	238.3	24Jun2006, 13:00	4.01
McDowel Pond	0.59000	242.5	24Jun2006, 13:30	2.94
McD to W7	0.59000	172.1	24Jun2006, 14:45	2.94
Mews Combined Basin	0.17200	101.4	24Jun2006, 13:15	3.97
Mews Stormsewer	0.25000	118.0	24Jun2006, 13:30	3.74
Old Mill Pond W3 gage	7.27050	967.8	24Jun2006, 21:45	3.23
Old Mill-Wreck Pond	10.21600	1265.3	25Jun2006, 00:45	3.12
Osbonre to W1	6.71250	967.2	24Jun2006, 20:15	3.17
Osborne Pond	6.71250	969.3	24Jun2006, 19:45	3.17
osborne pond da	0.85000	502.1	24Jun2006, 12:45	3.08
Osborne Pond surface	0.03000	68.9	24Jun2006, 12:15	6.52
Parkway	0.04000	30.4	24Jun2006, 13:15	6.52
rt34-W6	0.34250	172.5	24Jun2006, 14:30	4.01
Rt 35 culvert	2.94550	442.0	24Jun2006, 17:30	2.88
Rt 35 to Outlet	2.94550	420.9	24Jun2006, 19:00	2.88
Rt 71 East	0.00512	9.9	24Jun2006, 12:15	4.67
RT 71 w/Imp	0.02000	35.4	24Jun2006, 12:15	4.22
slgc flowpath	0.02519	14.0	24Jun2006, 12:45	2.53
slgc lake	0.64040	311.1	24Jun2006, 13:15	3.51
slgc lakes only	0.00440	10.1	24Jun2006, 12:15	6.52

slgc upper lake	0.42200	218.4	24Jun2006, 13:30	3.83
SL-SlH-SG	1.76000	711.8	24Jun2006, 13:30	3.73
W1	0.07800	114.9	24Jun2006, 12:15	4.26
W1 gage	6.79050	971.3	24Jun2006, 20:15	3.18
W1-W3	6.79050	952.3	24Jun2006, 21:15	3.18
W2 Rt 35	0.13750	49.2	24Jun2006, 14:00	5.52
w2 subdivision at Baileys	0.16250	37.6	24Jun2006, 14:45	4.53
W3	0.48000	242.3	24Jun2006, 13:00	4.04
W5	2.33000	333.8	24Jun2006, 15:45	2.59
W5 add/l housing	0.07800	26.1	24Jun2006, 13:15	2.74
w5 combined	2.64550	399.7	24Jun2006, 15:30	2.72
w5 housing development	0.23750	81.8	24Jun2006, 13:15	4.05
W5-W2	2.80800	414.4	24Jun2006, 17:30	2.75
W6	0.93000	200.4	24Jun2006, 14:15	3.06
W6_Rt 34	0.34250	255.6	24Jun2006, 12:45	4.01
W6 gage	1.27250	370.6	24Jun2006, 14:30	3.32
w6-Hurley Pond	1.31250	340.3	24Jun2006, 16:30	3.41
W7	4.86250	812.4	24Jun2006, 18:45	3.13
W7_Glen	1.16000	245.0	24Jun2006, 14:45	2.89
W7_McD	0.59000	205.9	24Jun2006, 13:15	2.94
W7 to Osborne Pond	4.86250	798.1	24Jun2006, 20:00	3.13
W8 base flow	Not Specified	12.0	24Jun2006, 00:00	
W8 Mews	0.17200	106.3	24Jun2006, 13:00	3.97
W8 SLGC	0.21400	90.5	24Jun2006, 13:00	2.81
W8 urban lower	0.02519	14.0	24Jun2006, 12:30	2.53
W8 urban upper	0.25000	118.5	24Jun2006, 13:15	3.74
W9	1.80000	419.1	24Jun2006, 14:15	3.15
W9 junction	3.11250	673.0	24Jun2006, 16:00	3.26
WPB-HAN Combined	10.21600	1343.9	24Jun2006, 20:45	3.13
WP Junction	Not Specified	1381.2	25Jun2006, 00:15	
Wreck Pond	Not Specified	580.5	25Jun2006, 09:15	

a : Storm event is from the hypothetical start time of 24 June 2006 @ 00:00 hours.

Table 13: HMS simulation results for 10 Year NRCS storm event using percentage curve created from SCS standard dimensionless unit hydrograph (PRF 484)

HYDROLOGIC ELEMENT	DRAINAGE AREA (SQ. MILE)	PEAK FLOW (CFS)	TIME OF PEAK [a]	VOLUME (INCH)
Albers Pond	0.97000	219.9	24Jun2006, 14:30	2.44
Albers pond da	0.93000	418.9	24Jun2006, 13:15	2.35
Albers Pond Surface	0.04000	77.8	24Jun2006, 12:15	5.23
Albers to Osborne	0.97000	191.7	24Jun2006, 15:45	2.43
Hanabrand Outlet	2.94550	373.6	24Jun2006, 19:00	2.00
HP to W7	3.11250	446.7	24Jun2006, 18:45	2.34
Hurley's Pond	3.11250	536.1	24Jun2006, 16:00	2.34
Junction-1	0.42200	238.2	24Jun2006, 13:15	2.74
Junction-3	Not Specified	344.5	24Jun2006, 13:15	
Kellers Pond	0.34250	232.3	24Jun2006, 13:00	2.93
McDowel Pond	0.59000	263.7	24Jun2006, 13:15	2.00
McD to W7	0.59000	158.1	24Jun2006, 14:00	1.99
Mews Combined Basin	0.17200	108.4	24Jun2006, 13:15	2.85
Mews Stormsewer	0.25000	133.8	24Jun2006, 13:30	2.66
Old Mill Pond W3 gage	7.27050	678.4	24Jun2006, 21:15	2.28
Old Mill-Wreck Pond	10.21600	924.6	25Jun2006, 00:00	2.19
Osbonre to W1	6.71250	670.4	24Jun2006, 17:00	2.23
Osborne Pond	6.71250	675.3	24Jun2006, 16:30	2.23
osborne pond da	0.85000	501.4	24Jun2006, 12:45	2.12
Osborne Pond surface	0.03000	58.3	24Jun2006, 12:15	5.23
Parkway	0.04000	37.9	24Jun2006, 13:15	5.23
rt34-W6	0.34250	142.3	24Jun2006, 14:15	2.93
Rt 35 culvert	2.94550	405.0	24Jun2006, 17:30	2.00
Rt 35 to Outlet	2.94550	373.6	24Jun2006, 19:00	2.00
Rt 71 East	0.00512	8.0	24Jun2006, 12:15	3.48
RT 71 w/Imp	0.02000	28.3	24Jun2006, 12:15	3.07
slgc flowpath	0.02519	12.8	24Jun2006, 12:45	1.66
slgc lake	0.64040	319.9	24Jun2006, 13:15	2.47
slgc lakes only	0.00440	8.6	24Jun2006, 12:15	5.23

slgc upper lake	0.42200	238.1	24Jun2006, 13:15	2.74
SL-SlH-SG	1.76000	826.5	24Jun2006, 13:30	2.64
W1	0.07800	88.7	24Jun2006, 12:15	3.21
W1 gage	6.79050	675.2	24Jun2006, 17:00	2.24
W1-W3	6.79050	663.8	24Jun2006, 20:45	2.23
W2 Rt 35	0.13750	56.6	24Jun2006, 14:00	4.44
w2 subdivision at Baileys	0.16250	41.3	24Jun2006, 14:45	3.57
W3	0.48000	263.2	24Jun2006, 13:00	3.02
W5	2.33000	354.4	24Jun2006, 15:45	1.72
W5 add/l housing	0.07800	27.7	24Jun2006, 13:15	1.81
w5 combined	2.64550	390.2	24Jun2006, 15:45	1.85
w5 housing development	0.23750	86.7	24Jun2006, 13:15	3.12
W5-W2	2.80800	389.2	24Jun2006, 17:30	1.88
W6	0.93000	206.6	24Jun2006, 14:00	2.21
W6_Rt 34	0.34250	266.9	24Jun2006, 12:45	2.93
W6 gage	1.27250	345.6	24Jun2006, 14:15	2.40
w6-Hurley Pond	1.31250	286.6	24Jun2006, 16:15	2.49
W7	4.86250	574.9	24Jun2006, 18:30	2.21
W7_Glen	1.16000	268.0	24Jun2006, 14:45	1.94
W7_McD	0.59000	220.9	24Jun2006, 13:15	2.00
W7 to Osborne Pond	4.86250	563.0	24Jun2006, 19:30	2.21
W8 base flow	Not Specified	12.0	24Jun2006, 00:00	
W8 Mews	0.17200	120.6	24Jun2006, 13:00	2.85
W8 SLGC	0.21400	95.1	24Jun2006, 13:00	1.88
W8 urban lower	0.02519	13.0	24Jun2006, 12:30	1.66
W8 urban upper	0.25000	136.1	24Jun2006, 13:15	2.66
W9	1.80000	445.7	24Jun2006, 14:15	2.24
W9 junction	3.11250	564.5	24Jun2006, 14:45	2.34
WPB-HAN Combined	10.21600	1023.2	24Jun2006, 19:30	2.20
WP Junction	Not Specified	1192.0	24Jun2006, 13:30	
Wreck Pond	Not Specified	499.6	25Jun2006, 07:00	

a : Storm event is from the hypothetical start time of 24 June 2006 @ 00:00 hours.

Table 14: HMS simulation results for 10 Year NRCS storm event using percentage curve created from Delmarva unit hydrograph (PRF 284)

HYDROLOGIC ELEMENT	DRAINAGE AREA (SQ. MILE)	PEAK FLOW (CFS)	TIME OF PEAK [a]	VOLUME (INCH)
Albers Pond	0.97000	170.8	24Jun2006, 15:30	2.43
Albers pond da	0.93000	274.8	24Jun2006, 13:15	2.35
Albers Pond Surface	0.04000	74.6	24Jun2006, 12:15	5.23
Albers to Osborne	0.97000	159.4	24Jun2006, 16:45	2.43
Hanabrand Outlet	2.94550	283.1	24Jun2006, 19:15	2.00
HP to W7	3.11250	376.8	24Jun2006, 19:45	2.34
Hurley's Pond	3.11250	437.6	24Jun2006, 17:00	2.34
Junction-1	0.42200	162.4	24Jun2006, 13:30	2.74
Junction-3	Not Specified	253.8	24Jun2006, 13:30	
Kellers Pond	0.34250	171.1	24Jun2006, 13:00	2.93
McDowel Pond	0.59000	148.4	24Jun2006, 13:15	2.00
McD to W7	0.59000	112.9	24Jun2006, 14:30	1.99
Mews Combined Basin	0.17200	73.1	24Jun2006, 13:30	2.85
Mews Stormsewer	0.25000	89.3	24Jun2006, 13:30	2.66
Old Mill Pond W3 gage	7.27050	618.3	24Jun2006, 22:15	2.28
Old Mill-Wreck Pond	10.21600	815.4	25Jun2006, 01:15	2.19
Osbonre to W1	6.71250	611.6	24Jun2006, 21:00	2.23
Osborne Pond	6.71250	613.0	24Jun2006, 20:30	2.23
osborne pond da	0.85000	349.4	24Jun2006, 12:45	2.12
Osborne Pond surface	0.03000	56.0	24Jun2006, 12:15	5.23
Parkway	0.04000	26.5	24Jun2006, 13:15	5.23
rt34-W6	0.34250	121.5	24Jun2006, 14:45	2.93
Rt 35 culvert	2.94550	299.5	24Jun2006, 17:30	2.00
Rt 35 to Outlet	2.94550	283.1	24Jun2006, 19:15	2.00
Rt 71 East	0.00512	7.5	24Jun2006, 12:15	3.48
RT 71 w/Imp	0.02000	26.3	24Jun2006, 12:15	3.07
slgc flowpath	0.02519	9.1	24Jun2006, 12:45	1.66
slgc lake	0.64040	230.8	24Jun2006, 13:30	2.47
slgc lakes only	0.00440	8.2	24Jun2006, 12:15	5.23

slgc upper lake	0.42200	163.8	24Jun2006, 13:30	2.74
SL-SlH-SG	1.76000	545.2	24Jun2006, 13:30	2.64
W1	0.07800	80.9	24Jun2006, 12:15	3.21
W1 gage	6.79050	614.5	24Jun2006, 21:00	2.24
W1-W3	6.79050	604.3	24Jun2006, 21:45	2.23
W2 Rt 35	0.13750	37.6	24Jun2006, 13:45	4.44
w2 subdivision at Baillieys	0.16250	27.2	24Jun2006, 14:30	3.57
W3	0.48000	176.2	24Jun2006, 13:00	3.02
W5	2.33000	225.9	24Jun2006, 15:45	1.72
W5 add/l housing	0.07800	18.0	24Jun2006, 13:15	1.81
w5 combined	2.64550	272.6	24Jun2006, 15:15	1.85
w5 housing development	0.23750	57.2	24Jun2006, 13:15	3.12
W5-W2	2.80800	279.1	24Jun2006, 17:45	1.88
W6	0.93000	133.6	24Jun2006, 14:00	2.21
W6_Rt 34	0.34250	189.9	24Jun2006, 12:45	2.93
W6 gage	1.27250	250.7	24Jun2006, 14:30	2.40
w6-Hurley Pond	1.31250	232.7	24Jun2006, 16:45	2.49
W7	4.86250	511.4	24Jun2006, 19:15	2.21
W7_Glen	1.16000	171.7	24Jun2006, 14:30	1.94
W7_McD	0.59000	143.9	24Jun2006, 13:15	2.00
W7 to Osborne Pond	4.86250	502.4	24Jun2006, 20:30	2.21
W8 base flow	Not Specified	12.0	24Jun2006, 00:00	
W8 Mews	0.17200	81.8	24Jun2006, 13:00	2.85
W8 SLGC	0.21400	62.8	24Jun2006, 13:00	1.88
W8 urban lower	0.02519	9.2	24Jun2006, 12:45	1.66
W8 urban upper	0.25000	90.2	24Jun2006, 13:15	2.66
W9	1.80000	288.7	24Jun2006, 14:00	2.24
W9 junction	3.11250	455.4	24Jun2006, 16:15	2.34
WPB-HAN Combined	10.21600	863.4	24Jun2006, 21:00	2.20
WP Junction	Not Specified	895.0	25Jun2006, 00:30	
Wreck Pond	Not Specified	488.7	25Jun2006, 08:30	

a : Storm event is from the hypothetical start time of 24 June 2006 @ 00:00 hours

Table 15: HMS simulation results for 10 Year NRCS storm event using percentage curve created from Lower Monmouth dimensionless unit hydrograph (PRF 230)

HYDROLOGIC ELEMENT	DRAINAGE AREA (SQ. MILE)	PEAK FLOW (CFS)	TIME OF PEAK [a]	VOLUME (INCH)
Albers Pond	0.97000	166.8	24Jun2006, 15:30	2.43
Albers pond da	0.93000	252.1	24Jun2006, 13:15	2.35
Albers Pond Surface	0.04000	73.7	24Jun2006, 12:15	5.23
Albers to Osborne	0.97000	155.5	24Jun2006, 16:30	2.43
Hanabrand Outlet	2.94550	272.2	24Jun2006, 19:15	2.00
HP to W7	3.11250	366.1	24Jun2006, 19:45	2.34
Hurley's Pond	3.11250	424.7	24Jun2006, 17:00	2.34
Junction-1	0.42200	152.2	24Jun2006, 13:30	2.74
Junction-3	Not Specified	237.4	24Jun2006, 13:30	
Kellers Pond	0.34250	163.8	24Jun2006, 13:00	2.93
McDowel Pond	0.59000	174.0	24Jun2006, 13:15	2.00
McD to W7	0.59000	112.4	24Jun2006, 14:30	1.99
Mews Combined Basin	0.17200	69.7	24Jun2006, 13:30	2.85
Mews Stormsewer	0.25000	82.5	24Jun2006, 13:30	2.66
Old Mill Pond W3 gage	7.27050	607.7	24Jun2006, 22:15	2.28
Old Mill-Wreck Pond	10.21600	800.7	25Jun2006, 01:15	2.19
Osbonre to W1	6.71250	598.9	24Jun2006, 21:00	2.23
Osborne Pond	6.71250	600.2	24Jun2006, 20:30	2.23
osborne pond da	0.85000	327.9	24Jun2006, 12:45	2.12
Osborne Pond surface	0.03000	55.3	24Jun2006, 12:15	5.23
parkway	0.04000	24.4	24Jun2006, 13:15	5.23
rt34-W6	0.34250	118.6	24Jun2006, 14:45	2.93
Rt 35 culvert	2.94550	284.9	24Jun2006, 17:30	2.00
Rt 35 to Outlet	2.94550	272.2	24Jun2006, 19:15	2.00
Rt 71 East	0.00512	7.4	24Jun2006, 12:15	3.48
RT 71 w/Imp	0.02000	25.9	24Jun2006, 12:15	3.07
slgc flowpath	0.02519	8.7	24Jun2006, 12:45	1.66
slgc lake	0.64040	214.4	24Jun2006, 13:30	2.47
slgc lakes only	0.00440	8.1	24Jun2006, 12:15	5.23

slgc upper lake	0.42200	152.1	24Jun2006, 13:30	2.74
SL-SlH-SG	1.76000	496.4	24Jun2006, 13:45	2.64
W1	0.07800	79.8	24Jun2006, 12:15	3.21
W1 gage	6.79050	601.9	24Jun2006, 21:00	2.24
W1-W3	6.79050	592.5	24Jun2006, 21:45	2.23
W2 Rt 35	0.13750	34.7	24Jun2006, 14:00	4.44
w2 subdivision at Bailieys	0.16250	25.3	24Jun2006, 14:45	3.57
W3	0.48000	163.1	24Jun2006, 13:00	3.02
W5	2.33000	210.5	24Jun2006, 16:15	1.72
W5 add/l housing	0.07800	16.7	24Jun2006, 13:30	1.81
w5 combined	2.64550	254.0	24Jun2006, 15:30	1.85
w5 housing development	0.23750	53.1	24Jun2006, 13:30	3.12
W5-W2	2.80800	265.1	24Jun2006, 17:45	1.88
W6	0.93000	124.9	24Jun2006, 14:15	2.21
W6_Rt 34	0.34250	178.3	24Jun2006, 12:45	2.93
W6 gage	1.27250	242.1	24Jun2006, 14:30	2.40
w6-Hurley Pond	1.31250	226.4	24Jun2006, 16:45	2.49
W7	4.86250	498.1	24Jun2006, 19:30	2.21
W7_Glen	1.16000	159.0	24Jun2006, 14:45	1.94
W7_McD	0.59000	133.0	24Jun2006, 13:30	2.00
W7 to Osborne Pond	4.86250	489.1	24Jun2006, 20:30	2.21
W8 base flow	Not Specified	12.0	24Jun2006, 00:00	
W8 Mews	0.17200	75.6	24Jun2006, 13:00	2.85
W8 SLGC	0.21400	58.1	24Jun2006, 13:00	1.88
W8 urban lower	0.02519	8.8	24Jun2006, 12:45	1.66
W8 urban upper	0.25000	82.8	24Jun2006, 13:15	2.66
W9	1.80000	268.1	24Jun2006, 14:15	2.24
W9 junction	3.11250	442.8	24Jun2006, 16:00	2.34
WPB-HAN Combined	10.21600	845.9	24Jun2006, 21:00	2.20
WP Junction	Not Specified	884.2	25Jun2006, 00:30	
Wreck Pond	Not Specified	486.5	25Jun2006, 08:30	

a : Storm event is from the hypothetical start time of 24 June 2006 @ 00:00 hours

Table16: HMS simulation results for 2 Year NRCS storm event using percentage curve created from SCS standard dimensionless unit hydrograph (PRF 484)

HYDROLOGIC ELEMENT	DRAINAGE AREA (SQ. MILE)	PEAK FLOW (CFS)	TIME OF PEAK [a]	VOLUME (INCH)
Albers Pond	0.97000	37.1	24Jun2006, 17:15	1.14
Albers pond da	0.93000	161.8	24Jun2006, 13:15	1.07
Albers Pond Surface	0.04000	50.3	24Jun2006, 12:15	3.38
Albers to Osborne	0.97000	35.4	24Jun2006, 18:15	1.14
Hanabrand Outlet	2.94550	131.1	24Jun2006, 19:15	0.95
HP to W7	3.11250	154.8	24Jun2006, 20:15	1.24
Hurley's Pond	3.11250	162.9	24Jun2006, 17:30	1.24
Junction-1	0.42200	95.9	24Jun2006, 13:30	1.30
Junction-3	Not Specified	154.4	24Jun2006, 13:30	
Kellers Pond	0.34250	93.0	24Jun2006, 13:00	1.54
McDowel Pond	0.59000	83.4	24Jun2006, 13:15	0.85
McD to W7	0.59000	51.2	24Jun2006, 14:30	0.84
Mews Combined Basin	0.17200	36.1	24Jun2006, 13:45	1.38
Mews Stormsewer	0.25000	60.0	24Jun2006, 13:30	1.25
Old Mill Pond W3 gage	7.27050	239.6	24Jun2006, 23:45	1.12
Old Mill-Wreck Pond	10.21600	321.0	25Jun2006, 01:45	1.06
Osbonre to W1	6.71250	232.2	24Jun2006, 22:00	1.07
Osborne Pond	6.71250	232.5	24Jun2006, 21:30	1.07
osborne pond da	0.85000	181.5	24Jun2006, 12:45	0.94
Osborne Pond surface	0.03000	37.7	24Jun2006, 12:15	3.38
parkway	0.04000	24.5	24Jun2006, 13:15	3.38
rt34-W6	0.34250	60.0	24Jun2006, 14:30	1.54
Rt 35 culvert	2.94550	140.1	24Jun2006, 17:45	0.95
Rt 35 to Outlet	2.94550	131.1	24Jun2006, 19:15	0.95
Rt 71 East	0.00512	4.4	24Jun2006, 12:15	1.84
RT 71 w/Imp	0.02000	14.4	24Jun2006, 12:15	1.53
slgc flowpath	0.02519	3.8	24Jun2006, 12:45	0.62
slgc lake	0.64040	137.8	24Jun2006, 13:30	1.13
slgc lakes only	0.00440	5.5	24Jun2006, 12:15	3.38
slgc upper lake	0.42200	94.9	24Jun2006, 13:30	1.30

SL-SlH-SG	1.76000	363.1	24Jun2006, 13:45	1.22
W1	0.07800	38.9	24Jun2006, 12:15	1.88
W1 gage	6.79050	234.0	24Jun2006, 22:00	1.08
W1-W3	6.79050	231.2	24Jun2006, 23:00	1.08
W2 Rt 35	0.13750	25.7	24Jun2006, 14:00	3.05
w2 subdivision at Bailieys	0.16250	16.3	24Jun2006, 14:45	2.39
W3	0.48000	104.4	24Jun2006, 13:00	1.75
W5	2.33000	113.7	24Jun2006, 16:00	0.69
W5 add/l housing	0.07800	9.2	24Jun2006, 13:30	0.69
w5 combined	2.64550	129.5	24Jun2006, 16:00	0.81
w5 housing development	0.23750	30.3	24Jun2006, 13:30	2.01
W5-W2	2.80800	131.9	24Jun2006, 18:00	0.85
W6	0.93000	62.9	24Jun2006, 14:15	1.22
W6_Rt 34	0.34250	118.4	24Jun2006, 12:45	1.54
W6 gage	1.27250	122.2	24Jun2006, 14:30	1.31
w6-Hurley Pond	1.31250	108.7	24Jun2006, 16:30	1.37
W7	4.86250	201.5	24Jun2006, 19:15	1.09
W7_Glen	1.16000	94.5	24Jun2006, 14:45	0.79
W7_McD	0.59000	75.9	24Jun2006, 13:30	0.85
W7 to Osborne Pond	4.86250	199.9	24Jun2006, 20:30	1.09
W8 base flow	Not Specified	12.0	24Jun2006, 00:00	
W8 Mews	0.17200	56.7	24Jun2006, 13:00	1.38
W8 SLGC	0.21400	32.4	24Jun2006, 13:00	0.74
W8 urban lower	0.02519	4.0	24Jun2006, 12:45	0.62
W8 urban upper	0.25000	60.6	24Jun2006, 13:15	1.25
W9	1.80000	149.9	24Jun2006, 14:15	1.15
W9 junction	3.11250	207.1	24Jun2006, 15:45	1.24
WPB-HAN Combined	10.21600	337.6	24Jun2006, 21:00	1.07
WP Junction	Not Specified	542.2	24Jun2006, 13:30	
Wreck Pond	Not Specified	247.2	25Jun2006, 07:45	

a : Storm event is from the hypothetical start time of 24 June 2006 @ 00:00 hours

Table 17: HMS simulation results for 2 Year NRCS storm event using percentage curve created from Delmarva unit hydrograph (PRF 284)

HYDROLOGIC ELEMENT	DRAINAGE AREA (SQ. MILE)	PEAK FLOW (CFS)	TIME OF PEAK [a]	VOLUME (INCH)
Albers Pond	0.97000	34.8	24Jun2006, 18:30	1.14
Albers pond da	0.93000	105.3	24Jun2006, 13:30	1.07
Albers Pond Surface	0.04000	48.2	24Jun2006, 12:15	3.38
Albers to Osborne	0.97000	33.3	24Jun2006, 19:45	1.14
Hanabrand Outlet	2.94550	102.7	24Jun2006, 19:45	0.95
HP to W7	3.11250	139.6	24Jun2006, 21:15	1.24
Hurley's Pond	3.11250	145.3	24Jun2006, 18:30	1.24
Junction-1	0.42200	66.9	24Jun2006, 13:30	1.30
Junction-3	Not Specified	99.5	24Jun2006, 14:00	
Kellers Pond	0.34250	68.5	24Jun2006, 13:15	1.54
McDowel Pond	0.59000	57.7	24Jun2006, 13:30	0.85
McD to W7	0.59000	38.9	24Jun2006, 15:15	0.84
Mews Combined Basin	0.17200	28.6	24Jun2006, 14:00	1.38
Mews Stormsewer	0.25000	39.4	24Jun2006, 13:30	1.25
Old Mill Pond W3 gage	7.27050	227.8	25Jun2006, 00:30	1.11
Old Mill-Wreck Pond	10.21600	296.0	25Jun2006, 03:15	1.06
Osbonre to W1	6.71250	221.2	24Jun2006, 23:15	1.07
Osborne Pond	6.71250	221.4	24Jun2006, 22:45	1.07
osborne pond da	0.85000	122.3	24Jun2006, 12:45	0.94
Osborne Pond surface	0.03000	36.2	24Jun2006, 12:15	3.38
parkway	0.04000	17.1	24Jun2006, 13:15	3.38
rt34-W6	0.34250	51.7	24Jun2006, 15:00	1.54
Rt 35 culvert	2.94550	107.2	24Jun2006, 18:00	0.95
Rt 35 to Outlet	2.94550	102.7	24Jun2006, 19:45	0.95
Rt 71 East	0.00512	4.0	24Jun2006, 12:15	1.84
RT 71 w/Imp	0.02000	13.1	24Jun2006, 12:15	1.53
slgc flowpath	0.02519	2.8	24Jun2006, 13:00	0.62
slgc lake	0.64040	83.6	24Jun2006, 14:00	1.13
slgc lakes only	0.00440	5.3	24Jun2006, 12:15	3.38

slgc upper lake	0.42200	66.9	24Jun2006, 13:30	1.30
SL-SlH-SG	1.76000	235.8	24Jun2006, 13:30	1.22
W1	0.07800	34.5	24Jun2006, 12:15	1.88
W1 gage	6.79050	222.7	24Jun2006, 23:15	1.08
W1-W3	6.79050	220.0	25Jun2006, 00:15	1.08
W2 Rt 35	0.13750	17.2	24Jun2006, 14:00	3.05
w2 subdivision at Baileys	0.16250	11.1	24Jun2006, 15:00	2.39
W3	0.48000	69.0	24Jun2006, 13:00	1.75
W5	2.33000	74.5	24Jun2006, 16:30	0.69
W5 add/l housing	0.07800	5.9	24Jun2006, 13:30	0.69
w5 combined	2.64550	92.2	24Jun2006, 16:00	0.81
w5 housing development	0.23750	20.4	24Jun2006, 13:30	2.01
W5-W2	2.80800	97.2	24Jun2006, 18:15	0.85
W6	0.93000	42.6	24Jun2006, 14:30	1.22
W6_Rt 34	0.34250	82.2	24Jun2006, 12:45	1.54
W6 gage	1.27250	94.1	24Jun2006, 15:00	1.31
w6-Hurley Pond	1.31250	92.4	24Jun2006, 17:00	1.37
W7	4.86250	190.7	24Jun2006, 20:00	1.09
W7_Glen	1.16000	60.9	24Jun2006, 15:00	0.79
W7_McD	0.59000	49.2	24Jun2006, 13:30	0.85
W7 to Osborne Pond	4.86250	189.5	24Jun2006, 21:30	1.09
W8 base flow	Not Specified	12.0	24Jun2006, 00:00	
W8 Mews	0.17200	37.8	24Jun2006, 13:00	1.38
W8 SLGC	0.21400	21.2	24Jun2006, 13:15	0.74
W8 urban lower	0.02519	2.9	24Jun2006, 12:45	0.62
W8 urban upper	0.25000	39.6	24Jun2006, 13:15	1.25
W9	1.80000	98.8	24Jun2006, 14:15	1.15
W9 junction	3.11250	176.6	24Jun2006, 16:30	1.24
WPB-HAN Combined	10.21600	305.8	24Jun2006, 23:30	1.07
WP Junction	Not Specified	359.0	24Jun2006, 13:45	
Wreck Pond	Not Specified	242.5	25Jun2006, 09:00	

a : Storm event is from the hypothetical start time of 24 June 2006 @ 00:00 hours

Table 18: HMS simulation results for 2 Year NRCS storm event using percentage curve created from Lower Monmouth dimensionless unit hydrograph (PRF 230)

HYDROLOGIC ELEMENT	DRAINAGE AREA (SQ. MILE)	PEAK FLOW (CFS)	TIME OF PEAK [a]	VOLUME (INCH)
Albers Pond	0.97000	33.4	24Jun2006, 18:45	1.14
Albers pond da	0.93000	98.6	24Jun2006, 13:30	1.07
Albers Pond Surface	0.04000	47.6	24Jun2006, 12:15	3.38
Albers to Osborne	0.97000	32.1	24Jun2006, 20:00	1.14
Hanabrand Outlet	2.94550	99.4	24Jun2006, 19:45	0.95
HP to W7	3.11250	136.9	24Jun2006, 21:15	1.24
Hurley's Pond	3.11250	142.4	24Jun2006, 18:30	1.24
Junction-1	0.42200	64.1	24Jun2006, 13:45	1.30
Junction-3	Not Specified	96.4	24Jun2006, 14:00	
Kellers Pond	0.34250	66.2	24Jun2006, 13:15	1.54
McDowel Pond	0.59000	55.9	24Jun2006, 13:30	0.85
McD to W7	0.59000	38.0	24Jun2006, 15:15	0.84
Mews Combined Basin	0.17200	28.0	24Jun2006, 14:00	1.38
Mews Stormsewer	0.25000	36.4	24Jun2006, 13:30	1.25
Old Mill Pond W3 gage	7.27050	224.9	25Jun2006, 00:30	1.11
Old Mill-Wreck Pond	10.21600	291.9	25Jun2006, 03:15	1.06
Osbonre to W1	6.71250	218.2	24Jun2006, 23:15	1.07
Osborne Pond	6.71250	218.4	24Jun2006, 22:45	1.07
osborne pond da	0.85000	114.8	24Jun2006, 12:45	0.94
Osborne Pond surface	0.03000	35.7	24Jun2006, 12:15	3.38
parkway	0.04000	15.8	24Jun2006, 13:15	3.38
rt34-W6	0.34250	50.6	24Jun2006, 15:00	1.54
Rt 35 culvert	2.94550	103.0	24Jun2006, 18:00	0.95
Rt 35 to Outlet	2.94550	99.4	24Jun2006, 19:45	0.95
Rt 71 East	0.00512	4.0	24Jun2006, 12:15	1.84
RT 71 w/Imp	0.02000	12.9	24Jun2006, 12:15	1.53
slgc flowpath	0.02519	2.7	24Jun2006, 13:00	0.62
slgc lake	0.64040	80.6	24Jun2006, 14:00	1.13
slgc lakes only	0.00440	5.2	24Jun2006, 12:15	3.38

slgc upper lake	0.42200	64.0	24Jun2006, 13:45	1.30
SL-SlH-SG	1.76000	217.8	24Jun2006, 13:45	1.22
W1	0.07800	34.0	24Jun2006, 12:15	1.88
W1 gage	6.79050	219.7	24Jun2006, 23:15	1.08
W1-W3	6.79050	217.1	25Jun2006, 00:15	1.08
W2 Rt 35	0.13750	16.0	24Jun2006, 14:15	3.05
w2 subdivision at Bailieys	0.16250	10.6	24Jun2006, 15:15	2.39
W3	0.48000	64.2	24Jun2006, 13:15	1.75
W5	2.33000	71.4	24Jun2006, 16:45	0.69
W5 add/l housing	0.07800	5.6	24Jun2006, 13:45	0.69
w5 combined	2.64550	87.4	24Jun2006, 16:00	0.81
w5 housing development	0.23750	19.3	24Jun2006, 13:45	2.01
W5-W2	2.80800	93.3	24Jun2006, 18:15	0.85
W6	0.93000	41.2	24Jun2006, 14:45	1.22
W6_Rt 34	0.34250	77.2	24Jun2006, 12:45	1.54
W6 gage	1.27250	91.6	24Jun2006, 15:00	1.31
w6-Hurley Pond	1.31250	90.2	24Jun2006, 17:00	1.37
W7	4.86250	187.2	24Jun2006, 20:00	1.09
W7_Glen	1.16000	57.7	24Jun2006, 15:30	0.79
W7_McD	0.59000	46.1	24Jun2006, 13:30	0.85
W7 to Osborne Pond	4.86250	186.1	24Jun2006, 21:30	1.09
W8 base flow	Not Specified	12.0	24Jun2006, 00:00	
W8 Mews	0.17200	35.0	24Jun2006, 13:00	1.38
W8 SLGC	0.21400	20.0	24Jun2006, 13:15	0.74
W8 urban lower	0.02519	2.7	24Jun2006, 12:45	0.62
W8 urban upper	0.25000	36.6	24Jun2006, 13:30	1.25
W9	1.80000	94.5	24Jun2006, 14:45	1.15
W9 junction	3.11250	172.0	24Jun2006, 16:30	1.24
WPB-HAN Combined	10.21600	301.4	24Jun2006, 23:30	1.07
WP Junction	Not Specified	340.5	24Jun2006, 14:00	
Wreck Pond	Not Specified	241.6	25Jun2006, 08:45	

a : Storm event is from the hypothetical start time of 24 June 2006 @ 00:00 hours.

Table 19: HMS simulation results for 50 Year NRCS storm event using percentage curve created from Standard SCS dimensionless unit hydrograph (PRF 484) and 5% initial abstraction ratio.

Albers Pond	0.97000	442.6	24Jun2006, 14:15	4.01
Albers pond da	0.93000	700.2	24Jun2006, 13:15	3.89
Albers Pond Surface	0.04000	113.9	24Jun2006, 12:15	7.66
Albers to Osborne	0.97000	395.6	24Jun2006, 15:15	4.01
Hanabrand Outlet	2.94550	671.9	24Jun2006, 18:45	3.30
HP to W7	3.11250	845.9	24Jun2006, 18:00	3.69
Hurley's Pond	3.11250	990.3	24Jun2006, 15:00	3.69
Junction-1	0.42200	380.8	24Jun2006, 13:15	4.42
Junction-3	Not Specified	552.9	24Jun2006, 13:15	
Kellers Pond	0.34250	389.1	24Jun2006, 12:45	4.57
McDowel Pond	0.59000	371.7	24Jun2006, 13:30	3.35
McD to W7	0.59000	275.2	24Jun2006, 14:00	3.35
Mews Combined Basin	0.17200	172.2	24Jun2006, 13:15	4.57
Mews Stormsewer	0.25000	212.1	24Jun2006, 13:30	4.33
Old Mill Pond W3 gage	7.27050	1273.4	24Jun2006, 20:30	3.69
Old Mill-Wreck Pond	10.21600	1710.7	24Jun2006, 23:30	3.56
Osbonre to W1	6.71250	1272.5	24Jun2006, 19:00	3.62
Osborne Pond	6.71250	1274.5	24Jun2006, 18:30	3.62
osborne pond da	0.85000	843.6	24Jun2006, 12:45	3.56
Osborne Pond surface	0.03000	85.4	24Jun2006, 12:15	7.66
parkway	0.04000	55.6	24Jun2006, 13:15	7.66
rt34-W6	0.34250	234.5	24Jun2006, 14:15	4.57
Rt 35 culvert	2.94550	731.0	24Jun2006, 17:15	3.30
Rt 35 to Outlet	2.94550	671.9	24Jun2006, 18:45	3.30
Rt 71 East	0.00512	12.1	24Jun2006, 12:15	5.49
RT 71 w/Imp	0.02000	43.0	24Jun2006, 12:15	4.91
slgc flowpath	0.02519	22.4	24Jun2006, 12:45	2.87
slgc lake	0.64040	521.2	24Jun2006, 13:15	4.05
slgc lakes only	0.00440	12.5	24Jun2006, 12:15	7.66
slgc upper lake	0.42200	379.4	24Jun2006, 13:15	4.42

SL-SlH-SG	1.76000	1318.0	24Jun2006, 13:30	4.30
W1	0.07800	137.4	24Jun2006, 12:15	4.76
W1 gage	6.79050	1277.4	24Jun2006, 19:00	3.64
W1-W3	6.79050	1261.4	24Jun2006, 19:45	3.63
W2 Rt 35	0.13750	118.1	24Jun2006, 13:15	6.08
w2 subdivision at Bailieys	0.16250	70.8	24Jun2006, 14:30	5.01
W3	0.48000	428.7	24Jun2006, 13:00	4.52
W5	2.33000	658.0	24Jun2006, 15:30	2.99
W5 add/l housing	0.07800	48.9	24Jun2006, 13:15	3.16
w5 combined	2.64550	717.3	24Jun2006, 15:15	3.13
w5 housing development	0.23750	151.4	24Jun2006, 13:15	4.47
W5-W2	2.80800	712.9	24Jun2006, 17:15	3.16
W6	0.93000	386.0	24Jun2006, 14:00	3.44
W6_Rt 34	0.34250	419.6	24Jun2006, 12:45	4.57
W6 gage	1.27250	612.5	24Jun2006, 14:00	3.75
w6-Hurley Pond	1.31250	502.7	24Jun2006, 16:00	3.87
W7	4.86250	1091.2	24Jun2006, 17:30	3.55
W7_Glen	1.16000	466.2	24Jun2006, 14:30	3.30
W7_McD	0.59000	380.9	24Jun2006, 13:15	3.35
W7 to Osborne Pond	4.86250	1069.6	24Jun2006, 18:45	3.55
W8 base flow	Not Specified	12.0	24Jun2006, 00:00	
W8 Mews	0.17200	187.2	24Jun2006, 13:00	4.57
W8 SLGC	0.21400	164.4	24Jun2006, 13:00	3.25
W8 urban lower	0.02519	23.3	24Jun2006, 12:30	2.87
W8 urban upper	0.25000	216.5	24Jun2006, 13:15	4.33
W9	1.80000	799.6	24Jun2006, 14:00	3.56
W9 junction	3.11250	1014.6	24Jun2006, 14:30	3.69
WPB-HAN Combined	10.21600	1905.4	24Jun2006, 19:15	3.57
WP Junction	Not Specified	1902.0	24Jun2006, 13:30	
Wreck Pond	Not Specified	591.5	25Jun2006, 08:00	
Albers Pond	0.97000	442.6	24Jun2006, 14:15	4.01

a : Storm event is from the hypothetical start time of 24 June 2006 @ 00:00 hours.

Table 20: HMS simulation results for 50 Year NRCS storm event using percentage curve created from Delmarva (PRF 284) and 5% initial abstraction ratio

Albers Pond	0.97000	343.7	24Jun2006, 14:30	4.01
Albers pond da	0.93000	470.0	24Jun2006, 13:15	3.89
Albers Pond Surface	0.04000	109.3	24Jun2006, 12:15	7.66
Albers to Osborne	0.97000	319.1	24Jun2006, 15:45	4.01
Hanabrand Outlet	2.94550	506.2	24Jun2006, 19:00	3.30
HP to W7	3.11250	710.8	24Jun2006, 19:00	3.69
Hurley's Pond	3.11250	786.9	24Jun2006, 16:15	3.69
Junction-1	0.42200	264.3	24Jun2006, 13:15	4.42
Junction-3	Not Specified	399.1	24Jun2006, 13:15	
Kellers Pond	0.34250	284.1	24Jun2006, 13:00	4.57
McDowel Pond	0.59000	271.1	24Jun2006, 13:15	3.35
McD to W7	0.59000	202.3	24Jun2006, 14:30	3.35
Mews Combined Basin	0.17200	121.5	24Jun2006, 13:15	4.57
Mews Stormsewer	0.25000	144.2	24Jun2006, 13:30	4.33
Old Mill Pond W3 gage	7.27050	1146.9	24Jun2006, 21:30	3.68
Old Mill-Wreck Pond	10.21600	1498.4	25Jun2006, 00:30	3.56
Osbonre to W1	6.71250	1152.9	24Jun2006, 20:00	3.62
Osborne Pond	6.71250	1155.5	24Jun2006, 19:30	3.62
osborne pond da	0.85000	605.3	24Jun2006, 12:45	3.56
Osborne Pond surface	0.03000	82.0	24Jun2006, 12:15	7.66
parkway	0.04000	38.8	24Jun2006, 13:15	7.66
rt34-W6	0.34250	199.2	24Jun2006, 14:30	4.57
Rt 35 culvert	2.94550	538.4	24Jun2006, 17:15	3.30
Rt 35 to Outlet	2.94550	506.2	24Jun2006, 19:00	3.30
Rt 71 East	0.00512	11.4	24Jun2006, 12:15	5.49
RT 71 w/Imp	0.02000	40.3	24Jun2006, 12:15	4.91
slgc flowpath	0.02519	16.5	24Jun2006, 12:45	2.87
slgc lake	0.64040	366.3	24Jun2006, 13:15	4.05
slgc lakes only	0.00440	12.0	24Jun2006, 12:15	7.66
slgc upper lake	0.42200	263.0	24Jun2006, 13:15	4.42
SL-SlH-SG	1.76000	884.9	24Jun2006, 13:30	4.30

W1	0.07800	127.3	24Jun2006, 12:15	4.76
W1 gage	6.79050	1157.4	24Jun2006, 20:00	3.64
W1-W3	6.79050	1135.0	24Jun2006, 20:45	3.63
W2 Rt 35	0.13750	80.1	24Jun2006, 13:15	6.08
w2 subdivision at Bailieys	0.16250	46.9	24Jun2006, 14:30	5.01
W3	0.48000	293.1	24Jun2006, 13:00	4.52
W5	2.33000	424.3	24Jun2006, 15:15	2.99
W5 add/l housing	0.07800	32.7	24Jun2006, 13:15	3.16
w5 combined	2.64550	507.0	24Jun2006, 15:00	3.13
w5 housing development	0.23750	101.9	24Jun2006, 13:15	4.47
W5-W2	2.80800	511.5	24Jun2006, 17:30	3.16
W6	0.93000	255.1	24Jun2006, 13:45	3.44
W6_Rt 34	0.34250	303.5	24Jun2006, 12:45	4.57
W6 gage	1.27250	441.3	24Jun2006, 14:15	3.75
w6-Hurley Pond	1.31250	404.4	24Jun2006, 16:30	3.87
W7	4.86250	965.4	24Jun2006, 18:30	3.55
W7_Glen	1.16000	304.6	24Jun2006, 14:30	3.30
W7_McD	0.59000	254.7	24Jun2006, 13:15	3.35
W7 to Osborne Pond	4.86250	950.2	24Jun2006, 19:45	3.55
W8 base flow	Not Specified	12.0	24Jun2006, 00:00	
W8 Mews	0.17200	128.7	24Jun2006, 13:00	4.57
W8 SLGC	0.21400	111.8	24Jun2006, 13:00	3.25
W8 urban lower	0.02519	16.6	24Jun2006, 12:30	2.87
W8 urban upper	0.25000	146.0	24Jun2006, 13:15	4.33
W9	1.80000	527.8	24Jun2006, 14:00	3.56
W9 junction	3.11250	791.7	24Jun2006, 16:00	3.69
WPB-HAN Combined	10.21600	1595.0	24Jun2006, 20:30	3.57
WP Junction	Not Specified	1625.1	25Jun2006, 00:00	
Wreck Pond	Not Specified	589.4	25Jun2006, 09:45	
Albers Pond	0.97000	343.7	24Jun2006, 14:30	4.01

a : Storm event is from the hypothetical start time of 24 June 2006 @ 00:00 hours.

Table21: HMS simulation results for 50 Year NRCS storm event using percentage curve created from LMDUH (PRF 230) and 5% initial abstraction ratio

HYDROLOGIC ELEMENT	DRAINAGE AREA (SQ. MILE)	PEAK FLOW (CFS)	TIME OF PEAK [a]	VOLUME (INCH)
Albers Pond	0.97000	334.2	24Jun2006, 14:30	4.01
Albers pond da	0.93000	431.8	24Jun2006, 13:15	3.89
Albers Pond Surface	0.04000	107.9	24Jun2006, 12:15	7.66
Albers to Osborne	0.97000	311.2	24Jun2006, 15:45	4.01
Hanabrand Outlet	2.94550	490.8	24Jun2006, 19:00	3.30
HP to W7	3.11250	692.9	24Jun2006, 19:00	3.69
Hurley's Pond	3.11250	765.5	24Jun2006, 16:15	3.69
Junction-1	0.42200	246.6	24Jun2006, 13:15	4.42
Junction-3	Not Specified	378.4	24Jun2006, 13:00	
Kellers Pond	0.34250	270.2	24Jun2006, 13:00	4.57
McDowel Pond	0.59000	244.8	24Jun2006, 13:15	3.35
McD to W7	0.59000	195.2	24Jun2006, 14:30	3.35
Mews Combined Basin	0.17200	114.4	24Jun2006, 13:15	4.57
Mews Stormsewer	0.25000	133.4	24Jun2006, 13:30	4.33
Old Mill Pond W3 gage	7.27050	1129.7	24Jun2006, 21:30	3.68
Old Mill-Wreck Pond	10.21600	1476.4	25Jun2006, 00:30	3.56
Osbonre to W1	6.71250	1131.6	24Jun2006, 20:00	3.62
Osborne Pond	6.71250	1134.0	24Jun2006, 19:30	3.62
osborne pond da	0.85000	568.9	24Jun2006, 12:45	3.56
Osborne Pond surface	0.03000	81.0	24Jun2006, 12:15	7.66
parkway	0.04000	35.7	24Jun2006, 13:15	7.66
rt34-W6	0.34250	194.8	24Jun2006, 14:30	4.57
Rt 35 culvert	2.94550	516.1	24Jun2006, 17:15	3.30
Rt 35 to Outlet	2.94550	490.8	24Jun2006, 19:00	3.30
Rt 71 East	0.00512	11.3	24Jun2006, 12:15	5.49
RT 71 w/Imp	0.02000	39.7	24Jun2006, 12:15	4.91
slgc flowpath	0.02519	15.7	24Jun2006, 12:45	2.87
slgc lake	0.64040	343.9	24Jun2006, 13:30	4.05
slgc lakes only	0.00440	11.9	24Jun2006, 12:15	7.66

slgc upper lake	0.42200	245.7	24Jun2006, 13:15	4.42
SL-SlH-SG	1.76000	805.0	24Jun2006, 13:30	4.30
W1	0.07800	125.6	24Jun2006, 12:15	4.76
W1 gage	6.79050	1136.1	24Jun2006, 20:00	3.64
W1-W3	6.79050	1115.2	24Jun2006, 20:45	3.63
W2 Rt 35	0.13750	55.6	24Jun2006, 14:00	6.08
w2 subdivision at Baileys	0.16250	43.2	24Jun2006, 14:45	5.01
W3	0.48000	271.7	24Jun2006, 13:00	4.52
W5	2.33000	392.1	24Jun2006, 15:45	2.99
W5 add/l housing	0.07800	30.0	24Jun2006, 13:15	3.16
w5 combined	2.64550	466.6	24Jun2006, 15:15	3.13
w5 housing development	0.23750	93.8	24Jun2006, 13:15	4.47
W5-W2	2.80800	483.8	24Jun2006, 17:15	3.16
W6	0.93000	235.6	24Jun2006, 14:00	3.44
W6_Rt 34	0.34250	285.3	24Jun2006, 12:45	4.57
W6 gage	1.27250	424.2	24Jun2006, 14:30	3.75
w6-Hurley Pond	1.31250	392.5	24Jun2006, 16:30	3.87
W7	4.86250	943.5	24Jun2006, 18:30	3.55
W7_Glen	1.16000	279.6	24Jun2006, 14:45	3.30
W7_McD	0.59000	234.0	24Jun2006, 13:15	3.35
W7 to Osborne Pond	4.86250	928.3	24Jun2006, 19:45	3.55
W8 base flow	Not Specified	12.0	24Jun2006, 00:00	
W8 Mews	0.17200	119.2	24Jun2006, 13:00	4.57
W8 SLGC	0.21400	103.5	24Jun2006, 13:00	3.25
W8 urban lower	0.02519	15.9	24Jun2006, 12:30	2.87
W8 urban upper	0.25000	134.2	24Jun2006, 13:15	4.33
W9	1.80000	483.7	24Jun2006, 14:00	3.56
W9 junction	3.11250	770.8	24Jun2006, 15:45	3.69
WPB-HAN Combined	10.21600	1568.5	24Jun2006, 20:30	3.57
WP Junction	Not Specified	1608.8	25Jun2006, 00:00	
Wreck Pond	Not Specified	588.9	25Jun2006, 09:45	

a : Storm event is from the hypothetical start time of 24 June 2006 @ 00:00 hours.

Table 22: HMS simulation results for 25 Year NRCS storm event using percentage curve created from Standard SCS dimensionless unit hydrograph (PRF 484) and 5% initial abstraction ratio

Albers Pond	0.97000	334.5	24Jun2006, 14:15	3.15
Albers pond da	0.93000	539.7	24Jun2006, 13:15	3.04
Albers Pond Surface	0.04000	96.9	24Jun2006, 12:15	6.52
Albers to Osborne	0.97000	288.3	24Jun2006, 15:15	3.15
Hanabrand Outlet	2.94550	507.8	24Jun2006, 18:45	2.58
HP to W7	3.11250	628.6	24Jun2006, 18:15	2.94
Hurley's Pond	3.11250	742.6	24Jun2006, 15:15	2.94
Junction-1	0.42200	299.3	24Jun2006, 13:15	3.50
Junction-3	Not Specified	437.8	24Jun2006, 13:15	
Kellers Pond	0.34250	293.0	24Jun2006, 12:45	3.66
McDowel Pond	0.59000	288.6	24Jun2006, 13:15	2.58
McD to W7	0.59000	208.3	24Jun2006, 14:00	2.58
Mews Combined Basin	0.17200	135.9	24Jun2006, 13:15	3.62
Mews Stormsewer	0.25000	166.7	24Jun2006, 13:30	3.41
Old Mill Pond W3 gage	7.27050	945.9	24Jun2006, 20:30	2.91
Old Mill-Wreck Pond	10.21600	1281.6	24Jun2006, 23:30	2.80
Osbonre to W1	6.71250	935.9	24Jun2006, 19:30	2.85
Osborne Pond	6.71250	939.6	24Jun2006, 16:15	2.85
osborne pond da	0.85000	645.4	24Jun2006, 12:45	2.77
Osborne Pond surface	0.03000	72.7	24Jun2006, 12:15	6.52
parkway	0.04000	47.3	24Jun2006, 13:15	6.52
rt34-W6	0.34250	181.5	24Jun2006, 14:15	3.66
Rt 35 culvert	2.94550	552.0	24Jun2006, 17:15	2.58
Rt 35 to Outlet	2.94550	507.8	24Jun2006, 18:45	2.58
Rt 71 East	0.00512	9.9	24Jun2006, 12:15	4.44
RT 71 w/Imp	0.02000	34.5	24Jun2006, 12:15	3.92
slgc flowpath	0.02519	16.8	24Jun2006, 12:45	2.18
slgc lake	0.64040	410.4	24Jun2006, 13:15	3.18
slgc lakes only	0.00440	10.7	24Jun2006, 12:15	6.52
slgc upper lake	0.42200	297.5	24Jun2006, 13:15	3.50

SL-SlH-SG	1.76000	1031.5	24Jun2006, 13:30	3.38
W1	0.07800	107.3	24Jun2006, 12:15	3.89
W1 gage	6.79050	940.6	24Jun2006, 16:45	2.86
W1-W3	6.79050	929.4	24Jun2006, 20:00	2.86
W2 Rt 35	0.13750	92.6	24Jun2006, 13:15	5.18
w2 subdivision at Bailieys	0.16250	54.4	24Jun2006, 14:30	4.21
W3	0.48000	330.9	24Jun2006, 13:00	3.68
W5	2.33000	493.4	24Jun2006, 15:30	2.29
W5 add/l housing	0.07800	37.0	24Jun2006, 13:15	2.40
w5 combined	2.64550	539.8	24Jun2006, 15:30	2.42
w5 housing development	0.23750	115.0	24Jun2006, 13:15	3.72
W5-W2	2.80800	537.0	24Jun2006, 17:15	2.45
W6	0.93000	289.4	24Jun2006, 14:00	2.76
W6_Rt 34	0.34250	328.8	24Jun2006, 12:45	3.66
W6 gage	1.27250	462.6	24Jun2006, 14:00	3.00
w6-Hurley Pond	1.31250	384.2	24Jun2006, 16:00	3.11
W7	4.86250	807.1	24Jun2006, 17:45	2.80
W7_Glen	1.16000	353.7	24Jun2006, 14:30	2.53
W7_McD	0.59000	288.9	24Jun2006, 13:15	2.58
W7 to Osborne Pond	4.86250	790.2	24Jun2006, 19:00	2.80
W8 base flow	Not Specified	12.0	24Jun2006, 00:00	
W8 Mews	0.17200	148.2	24Jun2006, 13:00	3.62
W8 SLGC	0.21400	125.1	24Jun2006, 13:00	2.49
W8 urban lower	0.02519	17.5	24Jun2006, 12:30	2.18
W8 urban upper	0.25000	170.0	24Jun2006, 13:15	3.41
W9	1.80000	604.1	24Jun2006, 14:00	2.82
W9 junction	3.11250	768.9	24Jun2006, 14:30	2.94
WPB-HAN Combined	10.21600	1429.2	24Jun2006, 19:15	2.81
WP Junction	Not Specified	1486.5	24Jun2006, 13:30	
Wreck Pond	Not Specified	565.3	25Jun2006, 07:15	
Albers Pond	0.97000	334.5	24Jun2006, 14:15	3.15

a : Storm event is from the hypothetical start time of 24 June 2006 @ 00:00 hours.

Table 23: HMS simulation results for 25 Year NRCS storm event using percentage curve created from Delmarva (PRF 284) and 5% initial abstraction ratio

Albers Pond	0.97000	250.8	24Jun2006, 14:45	3.15
Albers pond da	0.93000	361.4	24Jun2006, 13:15	3.04
Albers Pond Surface	0.04000	93.1	24Jun2006, 12:15	6.52
Albers to Osborne	0.97000	233.1	24Jun2006, 16:00	3.15
Hanabrand Outlet	2.94550	383.3	24Jun2006, 19:00	2.58
HP to W7	3.11250	531.1	24Jun2006, 19:15	2.94
Hurley's Pond	3.11250	599.3	24Jun2006, 16:15	2.94
Junction-1	0.42200	206.5	24Jun2006, 13:15	3.50
Junction-3	Not Specified	323.1	24Jun2006, 13:15	
Kellers Pond	0.34250	216.5	24Jun2006, 13:00	3.66
McDowel Pond	0.59000	211.2	24Jun2006, 13:30	2.58
McD to W7	0.59000	153.4	24Jun2006, 14:15	2.58
Mews Combined Basin	0.17200	94.8	24Jun2006, 13:15	3.62
Mews Stormsewer	0.25000	113.1	24Jun2006, 13:30	3.41
Old Mill Pond W3 gage	7.27050	857.8	24Jun2006, 21:45	2.90
Old Mill-Wreck Pond	10.21600	1123.1	25Jun2006, 00:45	2.80
Osbonre to W1	6.71250	854.1	24Jun2006, 20:15	2.85
Osborne Pond	6.71250	856.0	24Jun2006, 19:45	2.85
osborne pond da	0.85000	461.5	24Jun2006, 12:45	2.77
Osborne Pond surface	0.03000	69.8	24Jun2006, 12:15	6.52
parkway	0.04000	33.0	24Jun2006, 13:15	6.52
rt34-W6	0.34250	154.4	24Jun2006, 14:30	3.66
Rt 35 culvert	2.94550	407.2	24Jun2006, 17:15	2.58
Rt 35 to Outlet	2.94550	383.3	24Jun2006, 19:00	2.58
Rt 71 East	0.00512	9.3	24Jun2006, 12:15	4.44
RT 71 w/Imp	0.02000	32.2	24Jun2006, 12:15	3.92
slgc flowpath	0.02519	12.4	24Jun2006, 12:45	2.18
slgc lake	0.64040	294.8	24Jun2006, 13:15	3.18
slgc lakes only	0.00440	10.2	24Jun2006, 12:15	6.52
slgc upper lake	0.42200	206.1	24Jun2006, 13:30	3.50
SL-SlH-SG	1.76000	691.0	24Jun2006, 13:30	3.38

W1	0.07800	99.1	24Jun2006, 12:15	3.89
W1 gage	6.79050	857.8	24Jun2006, 20:15	2.86
W1-W3	6.79050	841.8	24Jun2006, 21:15	2.86
W2 Rt 35	0.13750	62.8	24Jun2006, 13:15	5.18
w2 subdivision at Bailieys	0.16250	36.2	24Jun2006, 14:30	4.21
W3	0.48000	225.8	24Jun2006, 13:00	3.68
W5	2.33000	317.8	24Jun2006, 15:30	2.29
W5 add/l housing	0.07800	24.6	24Jun2006, 13:15	2.40
w5 combined	2.64550	380.9	24Jun2006, 15:15	2.42
w5 housing development	0.23750	77.4	24Jun2006, 13:15	3.72
W5-W2	2.80800	385.8	24Jun2006, 17:30	2.45
W6	0.93000	190.6	24Jun2006, 13:45	2.76
W6_Rt 34	0.34250	237.1	24Jun2006, 12:45	3.66
W6 gage	1.27250	335.2	24Jun2006, 14:30	3.00
w6-Hurley Pond	1.31250	310.5	24Jun2006, 16:30	3.11
W7	4.86250	718.7	24Jun2006, 18:45	2.80
W7_Glen	1.16000	230.6	24Jun2006, 14:30	2.53
W7_McD	0.59000	192.7	24Jun2006, 13:15	2.58
W7 to Osborne Pond	4.86250	706.2	24Jun2006, 20:00	2.80
W8 base flow	Not Specified	12.0	24Jun2006, 00:00	
W8 Mews	0.17200	101.6	24Jun2006, 13:00	3.62
W8 SLGC	0.21400	84.8	24Jun2006, 13:00	2.49
W8 urban lower	0.02519	12.5	24Jun2006, 12:30	2.18
W8 urban upper	0.25000	114.4	24Jun2006, 13:15	3.41
W9	1.80000	398.5	24Jun2006, 14:00	2.82
W9 junction	3.11250	605.1	24Jun2006, 16:00	2.94
WPB-HAN Combined	10.21600	1194.6	24Jun2006, 20:45	2.81
WP Junction	Not Specified	1226.7	25Jun2006, 00:15	
Wreck Pond	Not Specified	557.4	25Jun2006, 08:45	
Albers Pond	0.97000	250.8	24Jun2006, 14:45	3.15

a : Storm event is from the hypothetical start time of 24 June 2006 @ 00:00 hours.

Table 24: HMS simulation results for 25 Year NRCS storm event using percentage curve created from LMDUH (PRF 230) and 5% initial abstraction ratio

HYDROLOGIC ELEMENT	DRAINAGE AREA (SQ. MILE)	PEAK FLOW (CFS)	TIME OF PEAK [a]	VOLUME (INCH)
Albers Pond	0.97000	244.5	24Jun2006, 14:45	3.15
Albers pond da	0.93000	332.1	24Jun2006, 13:15	3.04
Albers Pond Surface	0.04000	91.9	24Jun2006, 12:15	6.52
Albers to Osborne	0.97000	227.6	24Jun2006, 16:00	3.15
Hanabrand Outlet	2.94550	372.0	24Jun2006, 19:00	2.58
HP to W7	3.11250	517.4	24Jun2006, 19:15	2.94
Hurley's Pond	3.11250	583.3	24Jun2006, 16:15	2.94
Junction-1	0.42200	193.0	24Jun2006, 13:15	3.50
Junction-3	Not Specified	301.2	24Jun2006, 13:15	
Kellers Pond	0.34250	207.1	24Jun2006, 13:00	3.66
McDowel Pond	0.59000	187.1	24Jun2006, 13:30	2.58
McD to W7	0.59000	147.0	24Jun2006, 14:15	2.58
Mews Combined Basin	0.17200	89.6	24Jun2006, 13:15	3.62
Mews Stormsewer	0.25000	104.6	24Jun2006, 13:30	3.41
Old Mill Pond W3 gage	7.27050	844.1	24Jun2006, 21:30	2.90
Old Mill-Wreck Pond	10.21600	1106.2	25Jun2006, 00:45	2.80
Osbonre to W1	6.71250	837.8	24Jun2006, 20:15	2.85
Osborne Pond	6.71250	839.5	24Jun2006, 19:45	2.85
osborne pond da	0.85000	433.7	24Jun2006, 12:45	2.77
Osborne Pond surface	0.03000	68.9	24Jun2006, 12:15	6.52
parkway	0.04000	30.4	24Jun2006, 13:15	6.52
rt34-W6	0.34250	150.9	24Jun2006, 14:30	3.66
Rt 35 culvert	2.94550	390.8	24Jun2006, 17:15	2.58
Rt 35 to Outlet	2.94550	372.0	24Jun2006, 19:00	2.58
Rt 71 East	0.00512	9.2	24Jun2006, 12:15	4.44
RT 71 w/Imp	0.02000	31.8	24Jun2006, 12:15	3.92
slgc flowpath	0.02519	11.8	24Jun2006, 12:45	2.18
slgc lake	0.64040	273.0	24Jun2006, 13:15	3.18
slgc lakes only	0.00440	10.1	24Jun2006, 12:15	6.52

slgc upper lake	0.42200	193.3	24Jun2006, 13:30	3.50
SL-SlH-SG	1.76000	628.6	24Jun2006, 13:30	3.38
W1	0.07800	97.7	24Jun2006, 12:15	3.89
W1 gage	6.79050	841.4	24Jun2006, 20:15	2.86
W1-W3	6.79050	826.4	24Jun2006, 21:00	2.86
W2 Rt 35	0.13750	43.8	24Jun2006, 14:00	5.18
w2 subdivision at Bailieys	0.16250	33.4	24Jun2006, 14:45	4.21
W3	0.48000	209.4	24Jun2006, 13:00	3.68
W5	2.33000	294.3	24Jun2006, 15:45	2.29
W5 add/l housing	0.07800	22.6	24Jun2006, 13:15	2.40
w5 combined	2.64550	351.5	24Jun2006, 15:30	2.42
w5 housing development	0.23750	71.3	24Jun2006, 13:15	3.72
W5-W2	2.80800	365.2	24Jun2006, 17:30	2.45
W6	0.93000	176.6	24Jun2006, 14:00	2.76
W6_Rt 34	0.34250	223.0	24Jun2006, 12:45	3.66
W6 gage	1.27250	323.6	24Jun2006, 14:30	3.00
w6-Hurley Pond	1.31250	301.5	24Jun2006, 16:30	3.11
W7	4.86250	701.5	24Jun2006, 18:45	2.80
W7_Glen	1.16000	212.1	24Jun2006, 14:45	2.53
W7_McD	0.59000	177.1	24Jun2006, 13:15	2.58
W7 to Osborne Pond	4.86250	689.4	24Jun2006, 20:00	2.80
W8 base flow	Not Specified	12.0	24Jun2006, 00:00	
W8 Mews	0.17200	94.1	24Jun2006, 13:00	3.62
W8 SLGC	0.21400	78.6	24Jun2006, 13:00	2.49
W8 urban lower	0.02519	11.9	24Jun2006, 12:30	2.18
W8 urban upper	0.25000	105.2	24Jun2006, 13:15	3.41
W9	1.80000	366.0	24Jun2006, 14:15	2.82
W9 junction	3.11250	589.2	24Jun2006, 16:00	2.94
WPB-HAN Combined	10.21600	1174.4	24Jun2006, 20:45	2.81
WP Junction	Not Specified	1214.5	25Jun2006, 00:15	
Wreck Pond	Not Specified	555.8	25Jun2006, 08:45	

a : Storm event is from the hypothetical start time of 24 June 2006 @ 00:00 hours.

Table 25: HMS simulation results for 10 Year NRCS storm event using percentage curve created from Standard SCS dimensionless unit hydrograph (PRF 484) and 5% initial abstraction ratio

Albers Pond	0.97000	196.0	24Jun2006, 14:30	2.25
Albers pond da	0.93000	371.5	24Jun2006, 13:15	2.15
Albers Pond Surface	0.04000	77.8	24Jun2006, 12:15	5.23
Albers to Osborne	0.97000	170.6	24Jun2006, 15:45	2.25
Hanabrand Outlet	2.94550	340.4	24Jun2006, 18:45	1.85
HP to W7	3.11250	398.0	24Jun2006, 18:45	2.18
Hurley's Pond	3.11250	480.2	24Jun2006, 16:00	2.18
Junction-1	0.42200	210.7	24Jun2006, 13:15	2.50
Junction-3	Not Specified	309.6	24Jun2006, 13:15	
Kellers Pond	0.34250	202.0	24Jun2006, 13:00	2.70
McDowel Pond	0.59000	204.8	24Jun2006, 13:15	1.79
McD to W7	0.59000	139.3	24Jun2006, 14:00	1.79
Mews Combined Basin	0.17200	95.4	24Jun2006, 13:15	2.61
Mews Stormsewer	0.25000	118.3	24Jun2006, 13:30	2.43
Old Mill Pond W3 gage	7.27050	603.1	24Jun2006, 21:00	2.10
Old Mill-Wreck Pond	10.21600	827.7	25Jun2006, 00:00	2.01
Osbonre to W1	6.71250	604.7	24Jun2006, 17:00	2.04
Osborne Pond	6.71250	609.4	24Jun2006, 16:30	2.05
osborne pond da	0.85000	439.4	24Jun2006, 12:45	1.94
Osborne Pond surface	0.03000	58.3	24Jun2006, 12:15	5.23
parkway	0.04000	37.9	24Jun2006, 13:15	5.23
rt34-W6	0.34250	125.2	24Jun2006, 14:15	2.70
Rt 35 culvert	2.94550	369.5	24Jun2006, 17:15	1.85
Rt 35 to Outlet	2.94550	340.4	24Jun2006, 18:45	1.85
Rt 71 East	0.00512	7.4	24Jun2006, 12:15	3.30
RT 71 w/Imp	0.02000	25.2	24Jun2006, 12:15	2.85
slgc flowpath	0.02519	11.2	24Jun2006, 12:45	1.46
slgc lake	0.64040	286.7	24Jun2006, 13:15	2.25
slgc lakes only	0.00440	8.6	24Jun2006, 12:15	5.23
slgc upper lake	0.42200	210.7	24Jun2006, 13:15	2.50

SL-SlH-SG	1.76000	725.8	24Jun2006, 13:30	2.40
W1	0.07800	75.3	24Jun2006, 12:15	2.97
W1 gage	6.79050	609.0	24Jun2006, 17:00	2.06
W1-W3	6.79050	589.4	24Jun2006, 20:30	2.05
W2 Rt 35	0.13750	65.6	24Jun2006, 13:15	4.21
w2 subdivision at Baileys	0.16250	37.5	24Jun2006, 14:30	3.39
W3	0.48000	228.4	24Jun2006, 13:00	2.80
W5	2.33000	326.3	24Jun2006, 15:30	1.57
W5 add/l housing	0.07800	24.7	24Jun2006, 13:15	1.63
w5 combined	2.64550	359.3	24Jun2006, 15:30	1.70
w5 housing development	0.23750	77.8	24Jun2006, 13:15	2.94
W5-W2	2.80800	358.0	24Jun2006, 17:30	1.73
W6	0.93000	191.7	24Jun2006, 14:00	2.07
W6_Rt 34	0.34250	231.9	24Jun2006, 12:45	2.70
W6 gage	1.27250	310.4	24Jun2006, 14:15	2.24
w6-Hurley Pond	1.31250	262.1	24Jun2006, 16:00	2.33
W7	4.86250	507.9	24Jun2006, 18:15	2.02
W7_Glen	1.16000	238.0	24Jun2006, 14:30	1.74
W7_McD	0.59000	194.3	24Jun2006, 13:15	1.79
W7 to Osborne Pond	4.86250	497.4	24Jun2006, 19:30	2.02
W8 base flow	Not Specified	12.0	24Jun2006, 00:00	
W8 Mews	0.17200	106.1	24Jun2006, 13:00	2.61
W8 SLGC	0.21400	84.6	24Jun2006, 13:00	1.70
W8 urban lower	0.02519	11.6	24Jun2006, 12:30	1.46
W8 urban upper	0.25000	120.4	24Jun2006, 13:15	2.43
W9	1.80000	404.5	24Jun2006, 14:00	2.07
W9 junction	3.11250	518.4	24Jun2006, 14:30	2.18
WPB-HAN Combined	10.21600	920.1	24Jun2006, 19:15	2.02
WP Junction	Not Specified	1054.5	24Jun2006, 13:30	
Wreck Pond	Not Specified	473.1	25Jun2006, 06:45	
Albers Pond	0.97000	196.0	24Jun2006, 14:30	2.25

a : Storm event is from the hypothetical start time of 24 June 2006 @ 00:00 hours.

Table 26: HMS simulation results for 10 Year NRCS storm event using percentage curve created from DELMARVA (PRF 284) and 5% initial abstraction ratio

Albers Pond	0.97000	152.1	24Jun2006, 15:15	2.25
Albers pond da	0.93000	248.0	24Jun2006, 13:15	2.15
Albers Pond Surface	0.04000	74.6	24Jun2006, 12:15	5.23
Albers to Osborne	0.97000	142.0	24Jun2006, 16:30	2.25
Hanabrand Outlet	2.94550	257.9	24Jun2006, 19:00	1.85
HP to W7	3.11250	333.6	24Jun2006, 19:45	2.18
Hurley's Pond	3.11250	388.4	24Jun2006, 17:00	2.18
Junction-1	0.42200	144.7	24Jun2006, 13:30	2.50
Junction-3	Not Specified	227.6	24Jun2006, 13:30	
Kellers Pond	0.34250	149.5	24Jun2006, 13:00	2.70
McDowel Pond	0.59000	163.3	24Jun2006, 13:15	1.79
McD to W7	0.59000	103.9	24Jun2006, 14:30	1.79
Mews Combined Basin	0.17200	64.7	24Jun2006, 13:30	2.61
Mews Stormsewer	0.25000	80.0	24Jun2006, 13:30	2.43
Old Mill Pond W3 gage	7.27050	547.9	24Jun2006, 22:15	2.10
Old Mill-Wreck Pond	10.21600	727.7	25Jun2006, 01:00	2.01
Osbonre to W1	6.71250	539.4	24Jun2006, 21:00	2.04
Osborne Pond	6.71250	540.5	24Jun2006, 20:30	2.05
osborne pond da	0.85000	312.7	24Jun2006, 12:45	1.94
Osborne Pond surface	0.03000	56.0	24Jun2006, 12:15	5.23
parkway	0.04000	26.5	24Jun2006, 13:15	5.23
rt34-W6	0.34250	107.3	24Jun2006, 14:45	2.70
Rt 35 culvert	2.94550	273.4	24Jun2006, 17:15	1.85
Rt 35 to Outlet	2.94550	257.9	24Jun2006, 19:00	1.85
Rt 71 East	0.00512	6.9	24Jun2006, 12:15	3.30
RT 71 w/Imp	0.02000	23.4	24Jun2006, 12:15	2.85
slgc flowpath	0.02519	8.2	24Jun2006, 12:45	1.46
slgc lake	0.64040	205.9	24Jun2006, 13:30	2.25
slgc lakes only	0.00440	8.2	24Jun2006, 12:15	5.23
slgc upper lake	0.42200	145.7	24Jun2006, 13:30	2.50
SL-SlH-SG	1.76000	484.8	24Jun2006, 13:30	2.40

W1	0.07800	69.1	24Jun2006, 12:15	2.97
W1 gage	6.79050	542.0	24Jun2006, 21:00	2.05
W1-W3	6.79050	534.7	24Jun2006, 21:45	2.05
W2 Rt 35	0.13750	44.5	24Jun2006, 13:15	4.21
w2 subdivision at Bailieys	0.16250	25.1	24Jun2006, 14:30	3.39
W3	0.48000	155.6	24Jun2006, 13:00	2.80
W5	2.33000	210.0	24Jun2006, 15:30	1.57
W5 add/l housing	0.07800	16.4	24Jun2006, 13:15	1.63
w5 combined	2.64550	253.4	24Jun2006, 15:15	1.70
w5 housing development	0.23750	52.4	24Jun2006, 13:15	2.94
W5-W2	2.80800	257.8	24Jun2006, 17:30	1.73
W6	0.93000	126.2	24Jun2006, 14:00	2.07
W6_Rt 34	0.34250	166.6	24Jun2006, 12:45	2.70
W6 gage	1.27250	227.0	24Jun2006, 14:30	2.24
w6-Hurley Pond	1.31250	213.4	24Jun2006, 16:30	2.33
W7	4.86250	448.8	24Jun2006, 19:30	2.02
W7_Glen	1.16000	154.8	24Jun2006, 14:30	1.74
W7_McD	0.59000	129.2	24Jun2006, 13:15	1.79
W7 to Osborne Pond	4.86250	441.5	24Jun2006, 20:30	2.02
W8 base flow	Not Specified	12.0	24Jun2006, 00:00	
W8 Mews	0.17200	72.5	24Jun2006, 13:00	2.61
W8 SLGC	0.21400	57.2	24Jun2006, 13:00	1.70
W8 urban lower	0.02519	8.2	24Jun2006, 12:30	1.46
W8 urban upper	0.25000	80.8	24Jun2006, 13:15	2.43
W9	1.80000	266.7	24Jun2006, 14:00	2.07
W9 junction	3.11250	413.1	24Jun2006, 16:00	2.18
WPB-HAN Combined	10.21600	771.8	24Jun2006, 20:30	2.02
WP Junction	Not Specified	802.6	25Jun2006, 00:15	
Wreck Pond	Not Specified	463.8	25Jun2006, 08:15	
Albers Pond	0.97000	152.1	24Jun2006, 15:15	2.25

a : Storm event is from the hypothetical start time of 24 June 2006 @ 00:00 hours.

Table 27: HMS simulation results for 10 Year NRCS storm event using percentage curve created from LMDUH (PRF 230) and 5% initial abstraction ratio

HYDROLOGIC ELEMENT	DRAINAGE AREA (SQ. MILE)	PEAK FLOW (CFS)	TIME OF PEAK [a]	VOLUME (INCH)
Albers Pond	0.97000	148.4	24Jun2006, 15:15	2.25
Albers pond da	0.93000	227.9	24Jun2006, 13:15	2.15
Albers Pond Surface	0.04000	73.7	24Jun2006, 12:15	5.23
Albers to Osborne	0.97000	138.5	24Jun2006, 16:30	2.24
Hanabrand Outlet	2.94550	250.7	24Jun2006, 19:00	1.85
HP to W7	3.11250	323.2	24Jun2006, 19:45	2.18
Hurley's Pond	3.11250	374.5	24Jun2006, 17:00	2.18
Junction-1	0.42200	135.7	24Jun2006, 13:30	2.50
Junction-3	Not Specified	212.3	24Jun2006, 13:30	
Kellers Pond	0.34250	143.1	24Jun2006, 13:00	2.70
McDowel Pond	0.59000	130.4	24Jun2006, 13:15	1.79
McD to W7	0.59000	98.9	24Jun2006, 14:30	1.79
Mews Combined Basin	0.17200	61.7	24Jun2006, 13:30	2.61
Mews Stormsewer	0.25000	74.0	24Jun2006, 13:30	2.43
Old Mill Pond W3 gage	7.27050	538.3	24Jun2006, 22:00	2.10
Old Mill-Wreck Pond	10.21600	716.8	25Jun2006, 01:00	2.01
Osbonre to W1	6.71250	527.5	24Jun2006, 21:00	2.04
Osborne Pond	6.71250	528.5	24Jun2006, 20:30	2.05
osborne pond da	0.85000	293.9	24Jun2006, 12:45	1.94
Osborne Pond surface	0.03000	55.3	24Jun2006, 12:15	5.23
parkway	0.04000	24.4	24Jun2006, 13:15	5.23
rt34-W6	0.34250	104.8	24Jun2006, 14:45	2.70
Rt 35 culvert	2.94550	262.9	24Jun2006, 17:15	1.85
Rt 35 to Outlet	2.94550	250.7	24Jun2006, 19:00	1.85
Rt 71 East	0.00512	6.8	24Jun2006, 12:15	3.30
RT 71 w/Imp	0.02000	23.1	24Jun2006, 12:15	2.85
slgc flowpath	0.02519	7.8	24Jun2006, 12:45	1.46
slgc lake	0.64040	190.6	24Jun2006, 13:30	2.25
slgc lakes only	0.00440	8.1	24Jun2006, 12:15	5.23

slgc upper lake	0.42200	135.7	24Jun2006, 13:30	2.50
SL-SlH-SG	1.76000	441.1	24Jun2006, 13:45	2.40
W1	0.07800	68.2	24Jun2006, 12:15	2.97
W1 gage	6.79050	530.1	24Jun2006, 21:00	2.05
W1-W3	6.79050	523.8	24Jun2006, 21:45	2.05
W2 Rt 35	0.13750	31.2	24Jun2006, 14:00	4.21
w2 subdivision at Baileys	0.16250	23.3	24Jun2006, 14:45	3.39
W3	0.48000	144.3	24Jun2006, 13:00	2.80
W5	2.33000	195.1	24Jun2006, 15:45	1.57
W5 add/l housing	0.07800	15.1	24Jun2006, 13:15	1.63
w5 combined	2.64550	234.6	24Jun2006, 15:30	1.70
w5 housing development	0.23750	48.4	24Jun2006, 13:15	2.94
W5-W2	2.80800	244.4	24Jun2006, 17:30	1.73
W6	0.93000	117.2	24Jun2006, 14:00	2.07
W6_Rt 34	0.34250	156.6	24Jun2006, 12:45	2.70
W6 gage	1.27250	219.4	24Jun2006, 14:30	2.24
w6-Hurley Pond	1.31250	207.5	24Jun2006, 16:30	2.33
W7	4.86250	435.5	24Jun2006, 19:30	2.02
W7_Glen	1.16000	142.8	24Jun2006, 14:45	1.74
W7_McD	0.59000	118.8	24Jun2006, 13:15	1.79
W7 to Osborne Pond	4.86250	428.8	24Jun2006, 20:30	2.02
W8 base flow	Not Specified	12.0	24Jun2006, 00:00	
W8 Mews	0.17200	67.2	24Jun2006, 13:00	2.61
W8 SLGC	0.21400	53.0	24Jun2006, 13:00	1.70
W8 urban lower	0.02519	7.8	24Jun2006, 12:30	1.46
W8 urban upper	0.25000	74.3	24Jun2006, 13:15	2.43
W9	1.80000	245.9	24Jun2006, 14:15	2.07
W9 junction	3.11250	402.5	24Jun2006, 16:00	2.18
WPB-HAN Combined	10.21600	759.6	24Jun2006, 20:30	2.02
WP Junction	Not Specified	795.4	25Jun2006, 00:15	
Wreck Pond	Not Specified	462.1	25Jun2006, 08:15	

a : Storm event is from the hypothetical start time of 24 June 2006 @ 00:00 hours.

Table 28: HMS simulation results for 2 Year NRCS storm event using percentage curve created from SCS standard dimensionless unit hydrograph (PRF 484) and 5% initial abstraction ratio

Albers Pond	0.97000	37.5	24Jun2006, 16:45	1.13
Albers pond da	0.93000	166.3	24Jun2006, 13:15	1.06
Albers Pond Surface	0.04000	50.3	24Jun2006, 12:15	3.38
Albers to Osborne	0.97000	35.5	24Jun2006, 17:45	1.13
Hanabrand Outlet	2.94550	145.4	24Jun2006, 19:00	0.98
HP to W7	3.11250	163.9	24Jun2006, 19:45	1.26
Hurley's Pond	3.11250	172.9	24Jun2006, 17:15	1.26
Junction-1	0.42200	91.9	24Jun2006, 13:30	1.24
Junction-3	Not Specified	145.8	24Jun2006, 13:30	
Kellers Pond	0.34250	88.1	24Jun2006, 13:00	1.48
McDowel Pond	0.59000	86.6	24Jun2006, 13:15	0.84
McD to W7	0.59000	55.3	24Jun2006, 14:15	0.84
Mews Combined Basin	0.17200	34.9	24Jun2006, 13:30	1.30
Mews Stormsewer	0.25000	57.0	24Jun2006, 13:30	1.19
Old Mill Pond W3 gage	7.27050	242.9	24Jun2006, 23:15	1.12
Old Mill-Wreck Pond	10.21600	331.1	25Jun2006, 01:00	1.07
Osbonre to W1	6.71250	236.5	24Jun2006, 21:45	1.08
Osborne Pond	6.71250	236.7	24Jun2006, 21:15	1.08
osborne pond da	0.85000	191.8	24Jun2006, 12:45	0.94
Osborne Pond surface	0.03000	37.7	24Jun2006, 12:15	3.38
parkway	0.04000	24.5	24Jun2006, 13:15	3.38
rt34-W6	0.34250	57.4	24Jun2006, 14:30	1.48
Rt 35 culvert	2.94550	157.1	24Jun2006, 17:30	0.98
Rt 35 to Outlet	2.94550	145.4	24Jun2006, 19:00	0.98
Rt 71 East	0.00512	4.0	24Jun2006, 12:15	1.76
RT 71 w/Imp	0.02000	12.9	24Jun2006, 12:15	1.45
slgc flowpath	0.02519	4.7	24Jun2006, 12:45	0.63
slgc lake	0.64040	129.6	24Jun2006, 13:30	1.09
slgc lakes only	0.00440	5.5	24Jun2006, 12:15	3.38
slgc upper lake	0.42200	91.0	24Jun2006, 13:30	1.24

SL-SlH-SG	1.76000	340.9	24Jun2006, 13:30	1.15
W1	0.07800	35.2	24Jun2006, 12:15	1.82
W1 gage	6.79050	238.0	24Jun2006, 21:30	1.09
W1-W3	6.79050	235.3	24Jun2006, 22:30	1.08
W2 Rt 35	0.13750	31.6	24Jun2006, 13:15	2.99
w2 subdivision at Bailieys	0.16250	17.3	24Jun2006, 14:30	2.39
W3	0.48000	103.5	24Jun2006, 13:00	1.72
W5	2.33000	134.1	24Jun2006, 15:45	0.73
W5 add/l housing	0.07800	10.4	24Jun2006, 13:15	0.70
w5 combined	2.64550	149.5	24Jun2006, 15:30	0.85
w5 housing development	0.23750	33.9	24Jun2006, 13:15	2.02
W5-W2	2.80800	150.6	24Jun2006, 17:30	0.88
W6	0.93000	79.4	24Jun2006, 14:00	1.26
W6_Rt 34	0.34250	109.7	24Jun2006, 12:45	1.48
W6 gage	1.27250	133.5	24Jun2006, 14:15	1.32
w6-Hurley Pond	1.31250	118.1	24Jun2006, 16:15	1.39
W7	4.86250	208.0	24Jun2006, 19:00	1.10
W7_Glen	1.16000	101.1	24Jun2006, 14:45	0.79
W7_McD	0.59000	82.4	24Jun2006, 13:15	0.84
W7 to Osborne Pond	4.86250	206.2	24Jun2006, 20:15	1.10
W8 base flow	Not Specified	12.0	24Jun2006, 00:00	
W8 Mews	0.17200	52.2	24Jun2006, 13:00	1.30
W8 SLGC	0.21400	36.6	24Jun2006, 13:00	0.76
W8 urban lower	0.02519	4.8	24Jun2006, 12:30	0.63
W8 urban upper	0.25000	57.9	24Jun2006, 13:15	1.19
W9	1.80000	171.1	24Jun2006, 14:00	1.17
W9 junction	3.11250	227.2	24Jun2006, 14:45	1.26
WPB-HAN Combined	10.21600	352.5	24Jun2006, 20:15	1.08
WP Junction	Not Specified	514.7	24Jun2006, 13:30	
Wreck Pond	Not Specified	248.8	25Jun2006, 07:15	
Albers Pond	0.97000	37.5	24Jun2006, 16:45	1.13

a : Storm event is from the hypothetical start time of 24 June 2006 @ 00:00 hours.

Table 29: HMS simulation results for 2 Year NRCS storm event using percentage curve created from DELMARVA (PRF 284) and 5% initial abstraction ratio

Albers Pond	0.97000	34.7	24Jun2006, 18:15	1.13
Albers pond da	0.93000	110.4	24Jun2006, 13:15	1.06
Albers Pond Surface	0.04000	48.2	24Jun2006, 12:15	3.38
Albers to Osborne	0.97000	33.2	24Jun2006, 19:15	1.12
Hanabrand Outlet	2.94550	111.8	24Jun2006, 19:15	0.98
HP to W7	3.11250	146.8	24Jun2006, 20:45	1.26
Hurley's Pond	3.11250	153.0	24Jun2006, 18:00	1.26
Junction-1	0.42200	65.5	24Jun2006, 13:30	1.24
Junction-3	Not Specified	99.0	24Jun2006, 13:45	
Kellers Pond	0.34250	65.4	24Jun2006, 13:15	1.48
McDowel Pond	0.59000	62.7	24Jun2006, 13:15	0.84
McD to W7	0.59000	41.4	24Jun2006, 15:00	0.84
Mews Combined Basin	0.17200	27.6	24Jun2006, 13:45	1.30
Mews Stormsewer	0.25000	38.3	24Jun2006, 13:30	1.19
Old Mill Pond W3 gage	7.27050	230.5	25Jun2006, 00:15	1.12
Old Mill-Wreck Pond	10.21600	301.9	25Jun2006, 02:45	1.07
Osbonre to W1	6.71250	224.6	24Jun2006, 22:45	1.08
Osborne Pond	6.71250	224.8	24Jun2006, 22:15	1.08
osborne pond da	0.85000	135.1	24Jun2006, 12:45	0.94
Osborne Pond surface	0.03000	36.2	24Jun2006, 12:15	3.38
parkway	0.04000	17.1	24Jun2006, 13:15	3.38
rt34-W6	0.34250	49.5	24Jun2006, 14:45	1.48
Rt 35 culvert	2.94550	117.9	24Jun2006, 17:30	0.98
Rt 35 to Outlet	2.94550	111.8	24Jun2006, 19:15	0.98
Rt 71 East	0.00512	3.7	24Jun2006, 12:15	1.76
RT 71 w/Imp	0.02000	11.9	24Jun2006, 12:15	1.45
slgc flowpath	0.02519	3.4	24Jun2006, 12:45	0.63
slgc lake	0.64040	83.3	24Jun2006, 14:00	1.09
slgc lakes only	0.00440	5.3	24Jun2006, 12:15	3.38
slgc upper lake	0.42200	65.4	24Jun2006, 13:30	1.24
SL-SlH-SG	1.76000	226.3	24Jun2006, 13:30	1.15

W1	0.07800	32.0	24Jun2006, 12:15	1.82
W1 gage	6.79050	226.0	24Jun2006, 22:45	1.09
W1-W3	6.79050	223.2	24Jun2006, 23:45	1.08
W2 Rt 35	0.13750	21.7	24Jun2006, 13:15	2.99
w2 subdivision at Bailieys	0.16250	11.9	24Jun2006, 14:30	2.39
W3	0.48000	70.4	24Jun2006, 13:00	1.72
W5	2.33000	86.4	24Jun2006, 15:45	0.73
W5 add/l housing	0.07800	6.8	24Jun2006, 13:15	0.70
w5 combined	2.64550	105.9	24Jun2006, 15:15	0.85
w5 housing development	0.23750	23.1	24Jun2006, 13:15	2.02
W5-W2	2.80800	109.3	24Jun2006, 17:45	0.88
W6	0.93000	52.9	24Jun2006, 14:00	1.26
W6_Rt 34	0.34250	78.2	24Jun2006, 12:45	1.48
W6 gage	1.27250	99.7	24Jun2006, 14:45	1.32
w6-Hurley Pond	1.31250	98.0	24Jun2006, 16:45	1.39
W7	4.86250	196.8	24Jun2006, 19:45	1.10
W7_Glen	1.16000	65.5	24Jun2006, 14:30	0.79
W7_McD	0.59000	54.4	24Jun2006, 13:15	0.84
W7 to Osborne Pond	4.86250	195.4	24Jun2006, 21:00	1.10
W8 base flow	Not Specified	12.0	24Jun2006, 00:00	
W8 Mews	0.17200	35.5	24Jun2006, 13:00	1.30
W8 SLGC	0.21400	24.6	24Jun2006, 13:00	0.76
W8 urban lower	0.02519	3.4	24Jun2006, 12:45	0.63
W8 urban upper	0.25000	38.6	24Jun2006, 13:15	1.19
W9	1.80000	113.5	24Jun2006, 14:00	1.17
W9 junction	3.11250	186.3	24Jun2006, 16:15	1.26
WPB-HAN Combined	10.21600	311.2	24Jun2006, 22:45	1.08
WP Junction	Not Specified	350.9	24Jun2006, 13:45	
Wreck Pond	Not Specified	243.7	25Jun2006, 08:30	
Albers Pond	0.97000	34.7	24Jun2006, 18:15	1.13

a : Storm event is from the hypothetical start time of 24 June 2006 @ 00:00 hours.

Table 30: HMS simulation results for 2 Year NRCS storm event using percentage curve created from LMDUH (PRF 230) and 5% initial abstraction ratio

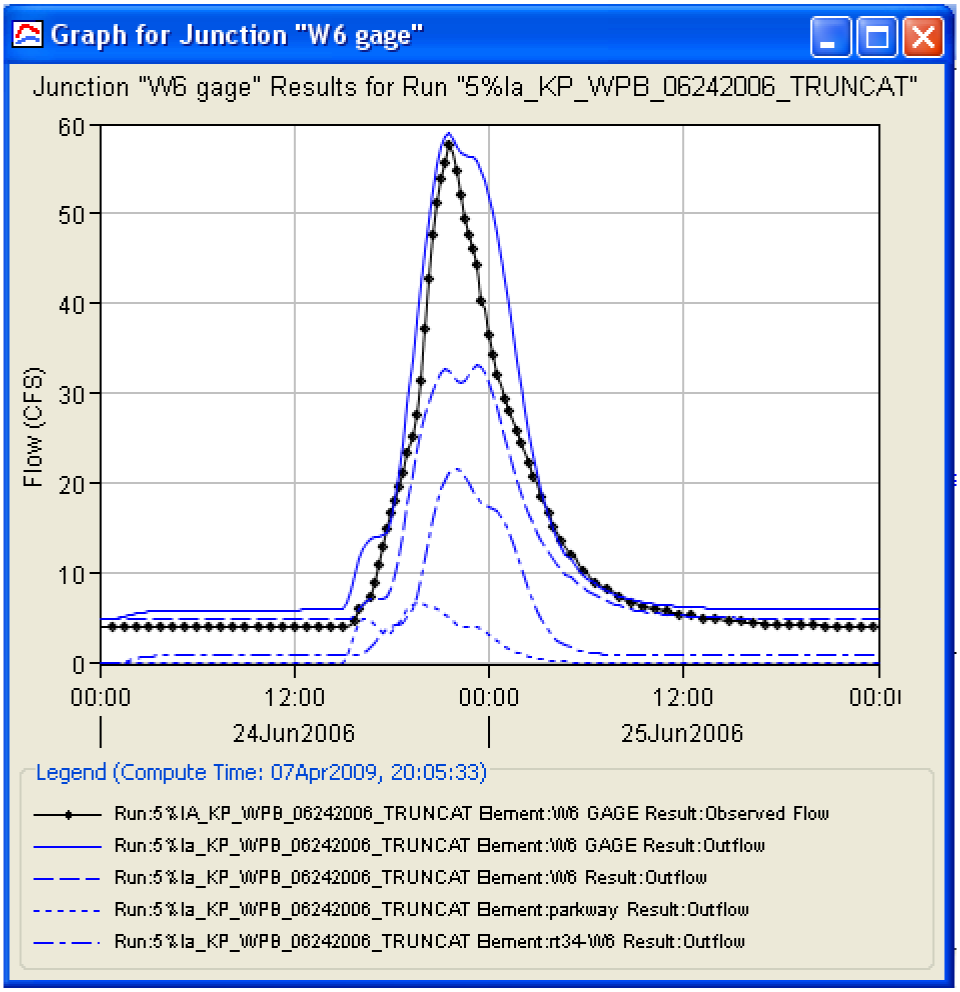
HYDROLOGIC ELEMENT	DRAINAGE AREA (SQ. MILE)	PEAK FLOW (CFS)	TIME OF PEAK [a]	VOLUME (INCH)
Albers Pond	0.97000	33.1	24Jun2006, 18:15	1.13
Albers pond da	0.93000	101.6	24Jun2006, 13:30	1.06
Albers Pond Surface	0.04000	47.6	24Jun2006, 12:15	3.38
Albers to Osborne	0.97000	31.8	24Jun2006, 19:30	1.12
Hanabrand Outlet	2.94550	109.0	24Jun2006, 19:15	0.98
HP to W7	3.11250	143.8	24Jun2006, 20:45	1.26
Hurley's Pond	3.11250	149.9	24Jun2006, 18:00	1.26
Junction-1	0.42200	62.0	24Jun2006, 13:45	1.24
Junction-3	Not Specified	95.8	24Jun2006, 14:00	
Kellers Pond	0.34250	63.1	24Jun2006, 13:15	1.48
McDowel Pond	0.59000	65.0	24Jun2006, 13:15	0.84
McD to W7	0.59000	40.8	24Jun2006, 15:00	0.84
Mews Combined Basin	0.17200	27.2	24Jun2006, 13:45	1.30
Mews Stormsewer	0.25000	35.4	24Jun2006, 13:30	1.19
Old Mill Pond W3 gage	7.27050	227.5	25Jun2006, 00:15	1.12
Old Mill-Wreck Pond	10.21600	298.5	25Jun2006, 02:30	1.07
Osbonre to W1	6.71250	221.4	24Jun2006, 22:45	1.08
Osborne Pond	6.71250	221.6	24Jun2006, 22:15	1.08
osborne pond da	0.85000	127.0	24Jun2006, 12:45	0.94
Osborne Pond surface	0.03000	35.7	24Jun2006, 12:15	3.38
parkway	0.04000	15.8	24Jun2006, 13:15	3.38
rt34-W6	0.34250	48.4	24Jun2006, 14:45	1.48
Rt 35 culvert	2.94550	113.9	24Jun2006, 17:30	0.98
Rt 35 to Outlet	2.94550	109.0	24Jun2006, 19:15	0.98
Rt 71 East	0.00512	3.6	24Jun2006, 12:15	1.76
RT 71 w/Imp	0.02000	11.7	24Jun2006, 12:15	1.45
slgc flowpath	0.02519	3.2	24Jun2006, 12:45	0.63
slgc lake	0.64040	80.2	24Jun2006, 14:00	1.09
slgc lakes only	0.00440	5.2	24Jun2006, 12:15	3.38

slgc upper lake	0.42200	62.1	24Jun2006, 13:45	1.24
SL-SlH-SG	1.76000	207.3	24Jun2006, 13:45	1.15
W1	0.07800	31.6	24Jun2006, 12:15	1.82
W1 gage	6.79050	222.8	24Jun2006, 22:45	1.09
W1-W3	6.79050	220.1	24Jun2006, 23:45	1.08
W2 Rt 35	0.13750	15.6	24Jun2006, 14:00	2.99
w2 subdivision at Bailieys	0.16250	11.2	24Jun2006, 14:45	2.39
W3	0.48000	65.4	24Jun2006, 13:00	1.72
W5	2.33000	80.7	24Jun2006, 16:00	0.73
W5 add/l housing	0.07800	6.3	24Jun2006, 13:30	0.70
w5 combined	2.64550	98.9	24Jun2006, 15:30	0.85
w5 housing development	0.23750	21.6	24Jun2006, 13:30	2.02
W5-W2	2.80800	104.1	24Jun2006, 17:45	0.88
W6	0.93000	49.6	24Jun2006, 14:15	1.26
W6_Rt 34	0.34250	73.5	24Jun2006, 12:45	1.48
W6 gage	1.27250	96.9	24Jun2006, 14:45	1.32
w6-Hurley Pond	1.31250	95.5	24Jun2006, 16:45	1.39
W7	4.86250	193.0	24Jun2006, 19:45	1.10
W7_Glen	1.16000	60.8	24Jun2006, 14:45	0.79
W7_McD	0.59000	50.4	24Jun2006, 13:30	0.84
W7 to Osborne Pond	4.86250	191.7	24Jun2006, 21:00	1.10
W8 base flow	Not Specified	12.0	24Jun2006, 00:00	
W8 Mews	0.17200	32.9	24Jun2006, 13:00	1.30
W8 SLGC	0.21400	22.8	24Jun2006, 13:00	0.76
W8 urban lower	0.02519	3.2	24Jun2006, 12:45	0.63
W8 urban upper	0.25000	35.5	24Jun2006, 13:15	1.19
W9	1.80000	105.8	24Jun2006, 14:15	1.17
W9 junction	3.11250	181.4	24Jun2006, 16:15	1.26
WPB-HAN Combined	10.21600	307.6	24Jun2006, 22:45	1.08
WP Junction	Not Specified	336.8	25Jun2006, 00:30	
Wreck Pond	Not Specified	242.8	25Jun2006, 08:30	

a : Storm event is from the hypothetical start time of 24 June 2006 @ 00:00 hours.

APPENDIX F: MODEL CALIBRATION AND VALIDATION RESULTS

Calibration Run – June 24, 2006 – AMC III – Station W6



Summary Results for Junction "W6 gage"

Project: wpb overall rebuild
Simulation Run: 5%Ia_KP_WPB_06242006_TRUNCAT Junction: W6 gage

Start of Run:	24Jun2006, 00:00	Basin Model:	5%Ia_KP_WPB_062406
End of Run:	26Jun2006, 00:00	Meteorologic Model:	KP_062406_TRUNCATED
Compute Time:	07Apr2009, 20:05:33	Control Specifications:	jun242006

Volume Units: (•) IN ( ) AC-FT

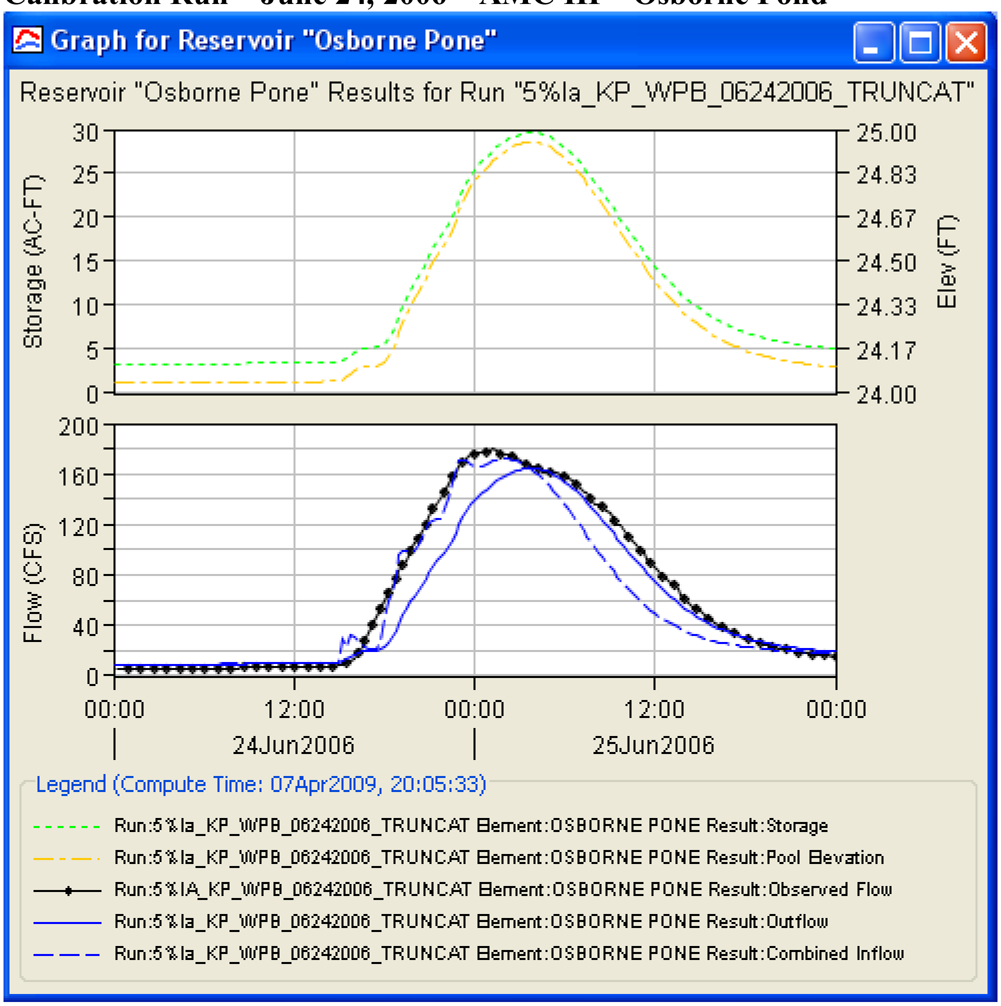
Computed Results

Peak Outflow : 59.0 (CFS) Date/Time of Peak Outflow : 24Jun2006, 21:30
Total Outflow : 0.78 (IN)

Observed Hydrograph at Gage w6_062406

Peak Discharge :	57.60 (CFS)	Date/Time of Peak Discharge :	24Jun2006, 21:30
Avg Abs Residual :	2.67 (CFS)		
Total Residual :	0.14 (IN)	Total Obs Q :	0.64 (IN)

Calibration Run – June 24, 2006 – AMC III – Osborne Pond



Summary Results for Reservoir "Osborne Pone"

Project: wpb overall rebuild
Simulation Run: 5%Ia_KP_WPB_06242006_TRUNCAT Reservoir: Osborne Pone

Start of Run:	24Jun2006, 00:00	Basin Model:	5%Ia_KP_WPB_062406
End of Run:	26Jun2006, 00:00	Meteorologic Model:	KP_062406_TRUNCATED
Compute Time:	07Apr2009, 20:05:33	Control Specifications:	jun242006

Volume Units: (•) IN ( ) AC-FT

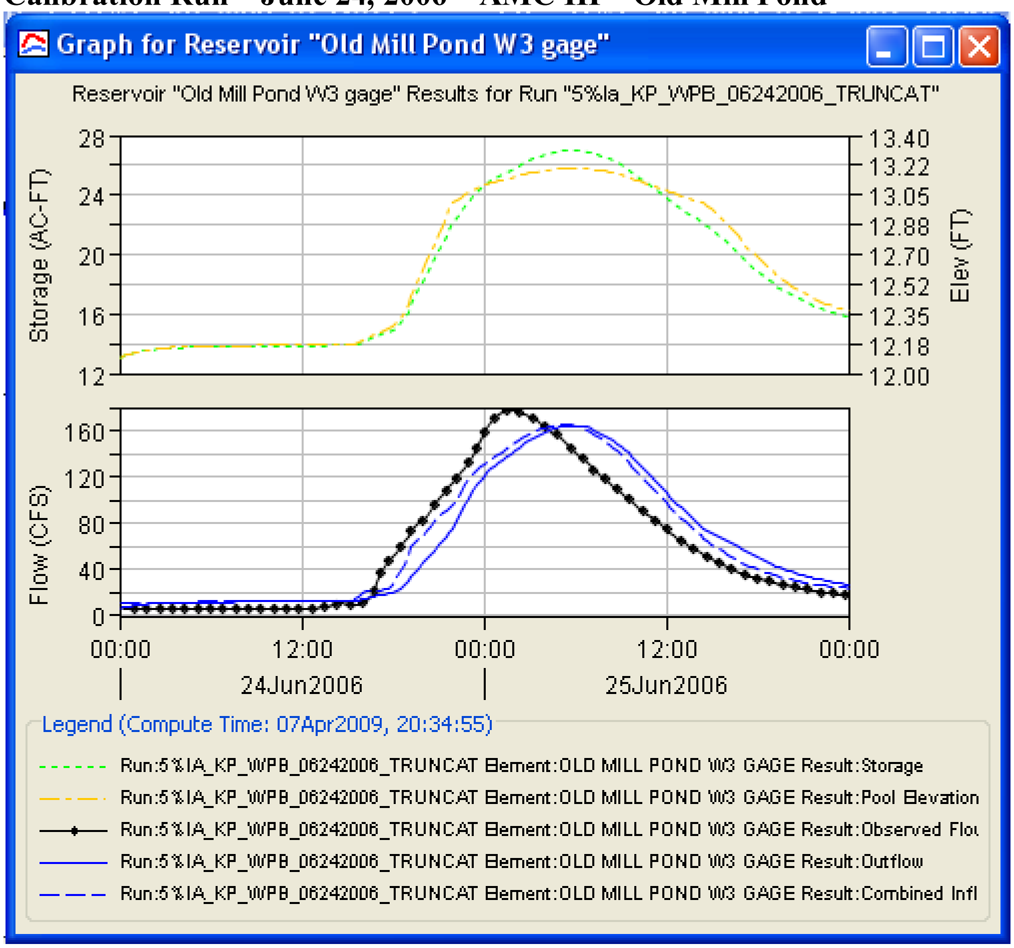
Computed Results

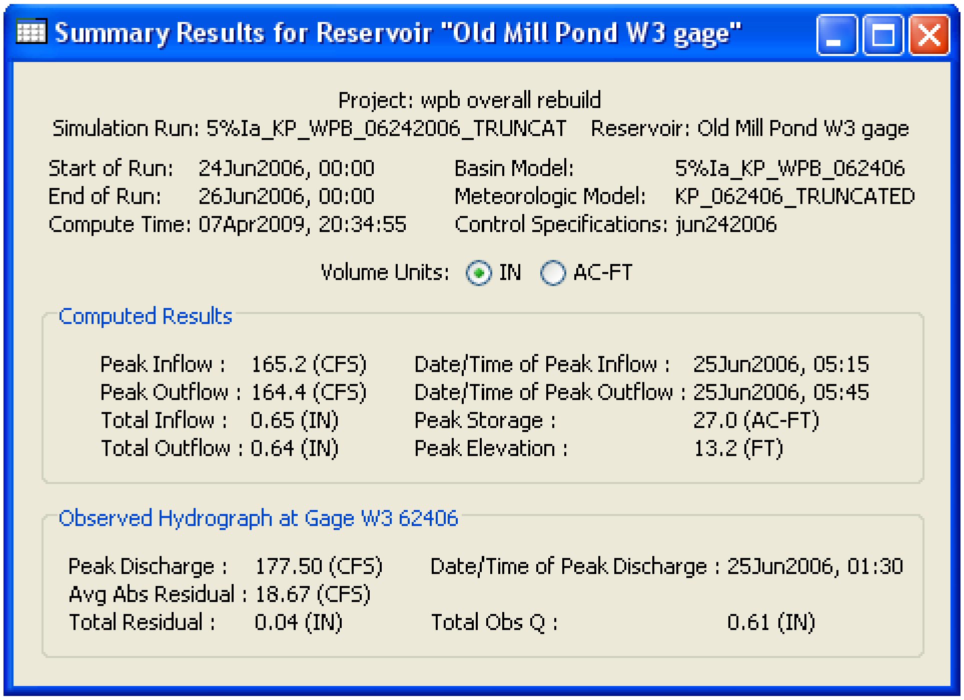
Peak Inflow :	171.9 (CFS)	Date/Time of Peak Inflow :	25Jun2006, 02:00
Peak Outflow :	164.1 (CFS)	Date/Time of Peak Outflow :	25Jun2006, 03:45
Total Inflow :	0.64 (IN)	Peak Storage :	29.7 (AC-FT)
Total Outflow :	0.63 (IN)	Peak Elevation :	25.0 (FT)

Observed Hydrograph at Gage W1_062406

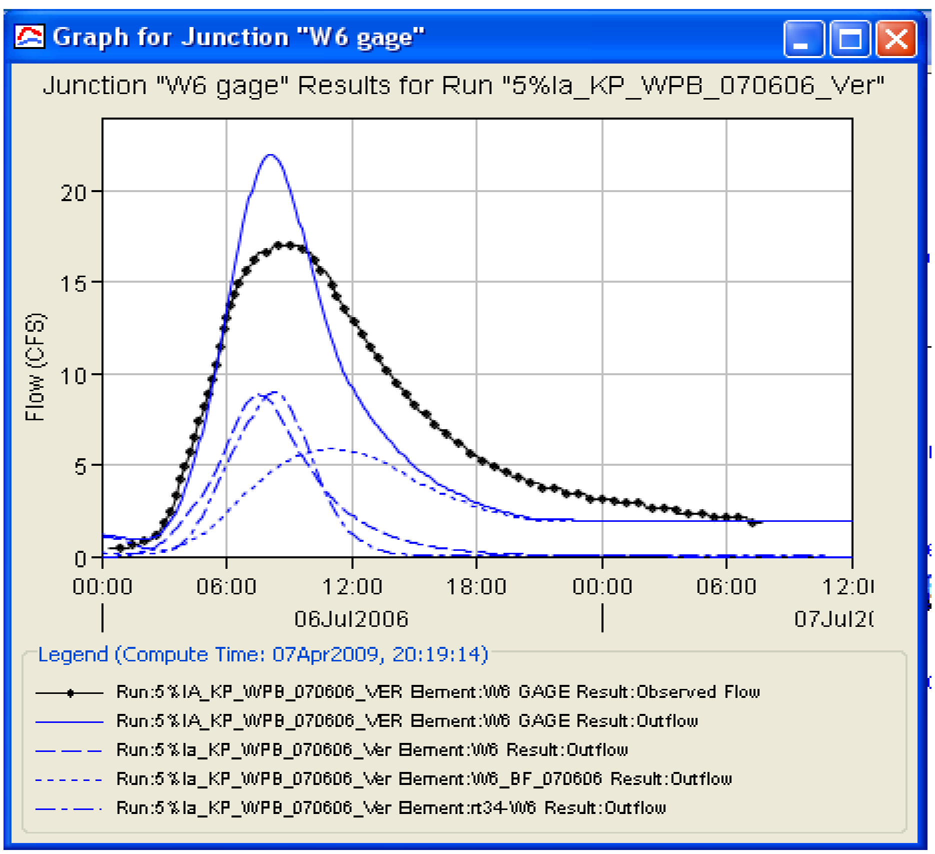
Peak Discharge :	179.85 (CFS)	Date/Time of Peak Discharge :	25Jun2006, 01:15
Avg Abs Residual :	10.79 (CFS)		
Total Residual :	-0.09 (IN)	Total Obs Q :	0.72 (IN)

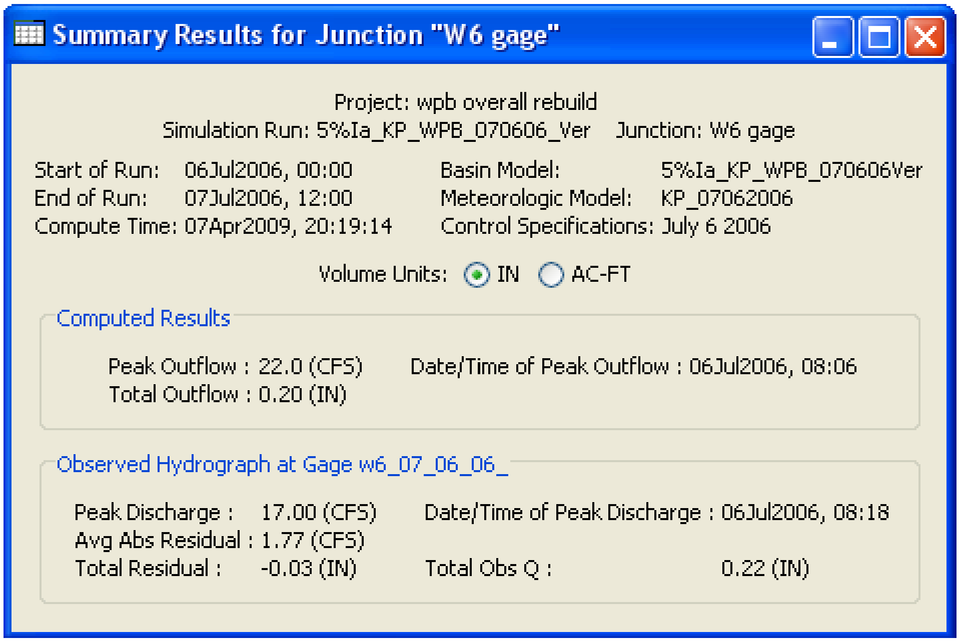
Calibration Run – June 24, 2006 – AMC III – Old Mill Pond





Verification Run – July 06, 2006 – AMC II – Station W6




Summary Results for Junction "W6 gage"

Project: wpb overall rebuild
Simulation Run: 5%Ia_KP_WPB_070606_Ver Junction: W6 gage

Start of Run:	06Jul2006, 00:00	Basin Model:	5%Ia_KP_WPB_070606Ver
End of Run:	07Jul2006, 12:00	Meteorologic Model:	KP_07062006
Compute Time:	07Apr2009, 20:19:14	Control Specifications:	July 6 2006

Volume Units: IN AC-FT

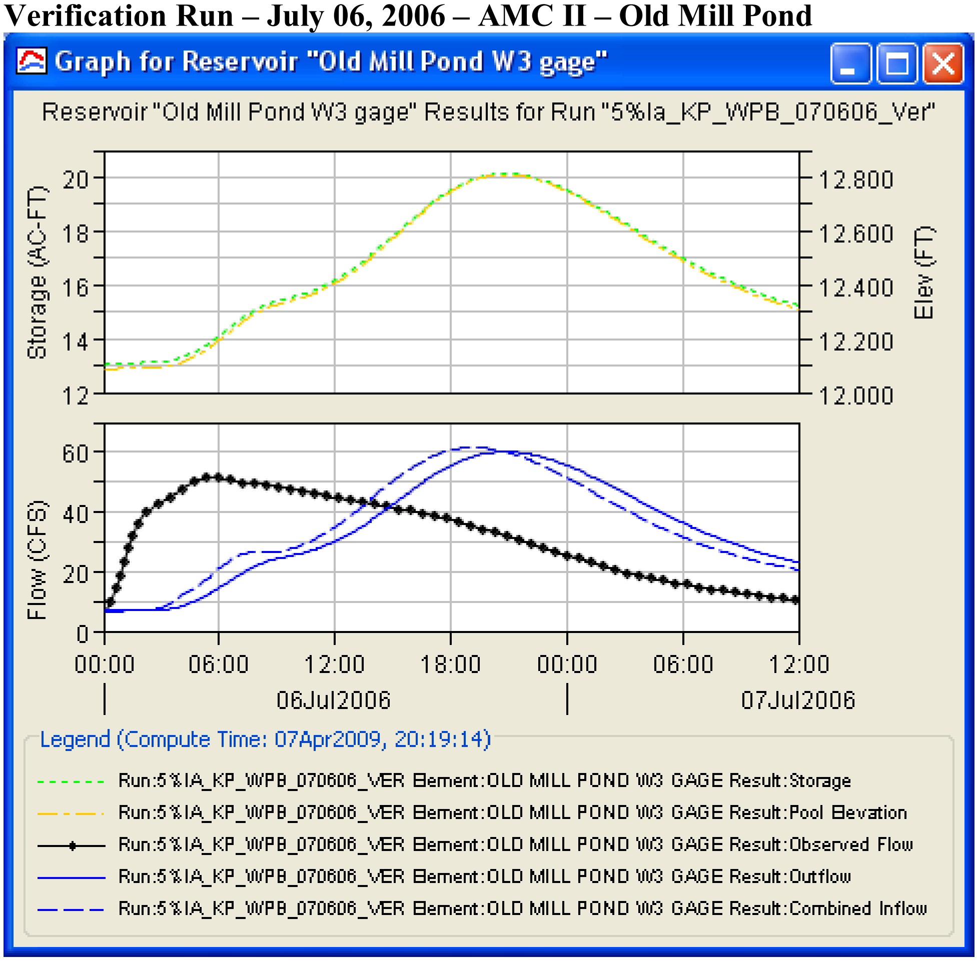
Computed Results

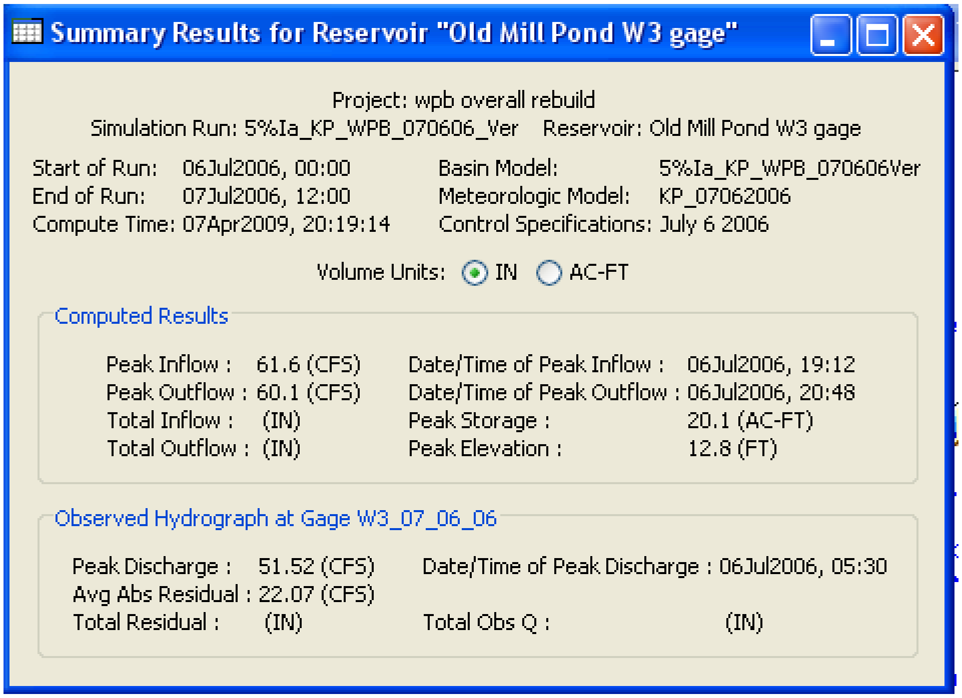
Peak Outflow : 22.0 (CFS) Date/Time of Peak Outflow : 06Jul2006, 08:06
Total Outflow : 0.20 (IN)

Observed Hydrograph at Gage w6_07_06_06_

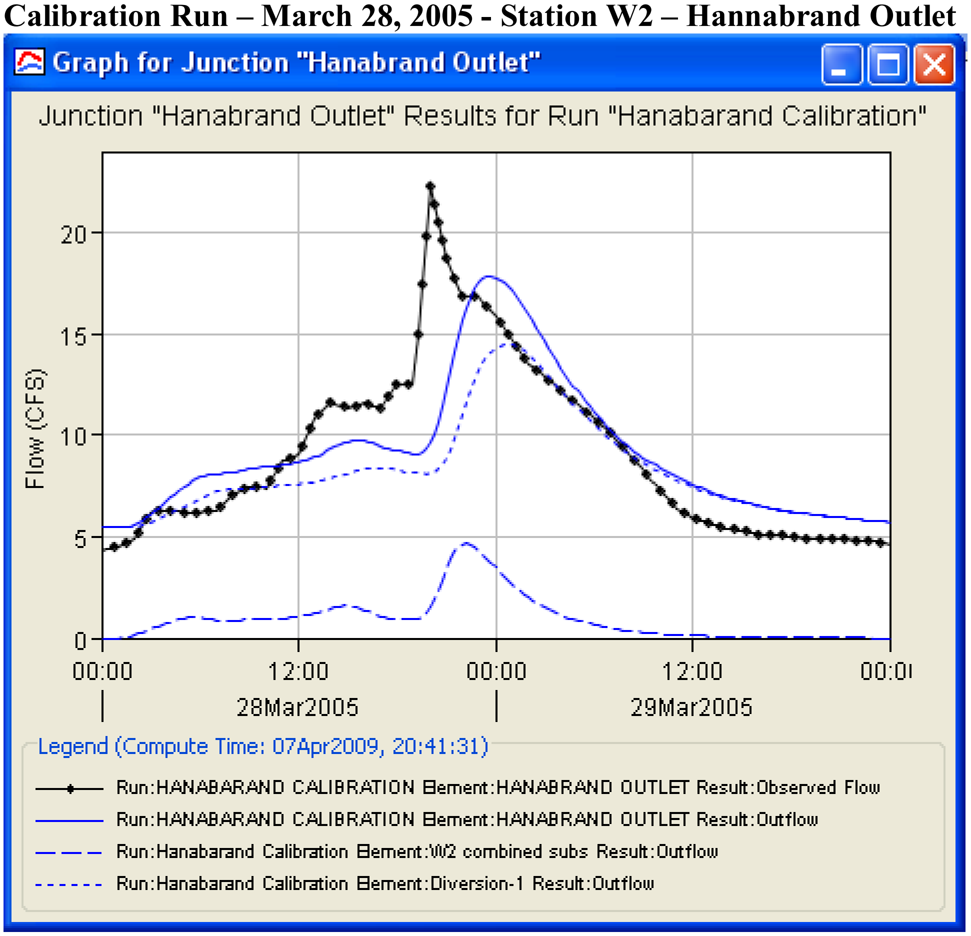
Peak Discharge :	17.00 (CFS)	Date/Time of Peak Discharge :	06Jul2006, 08:18
Avg Abs Residual :	1.77 (CFS)		
Total Residual :	-0.03 (IN)	Total Obs Q :	0.22 (IN)

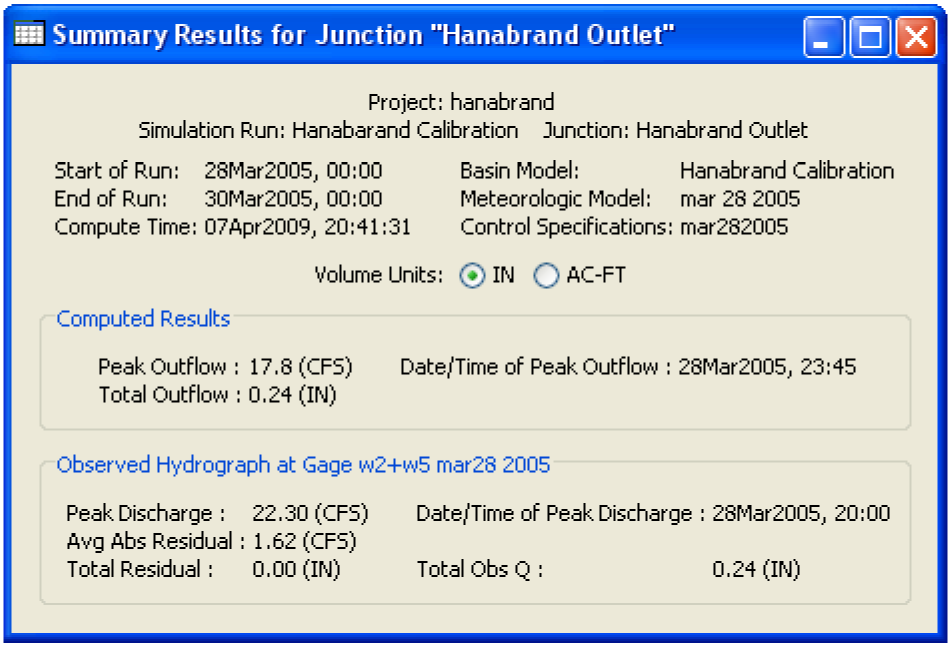
Verification Run – July 06, 2006 – AMC II – Old Mill Pond



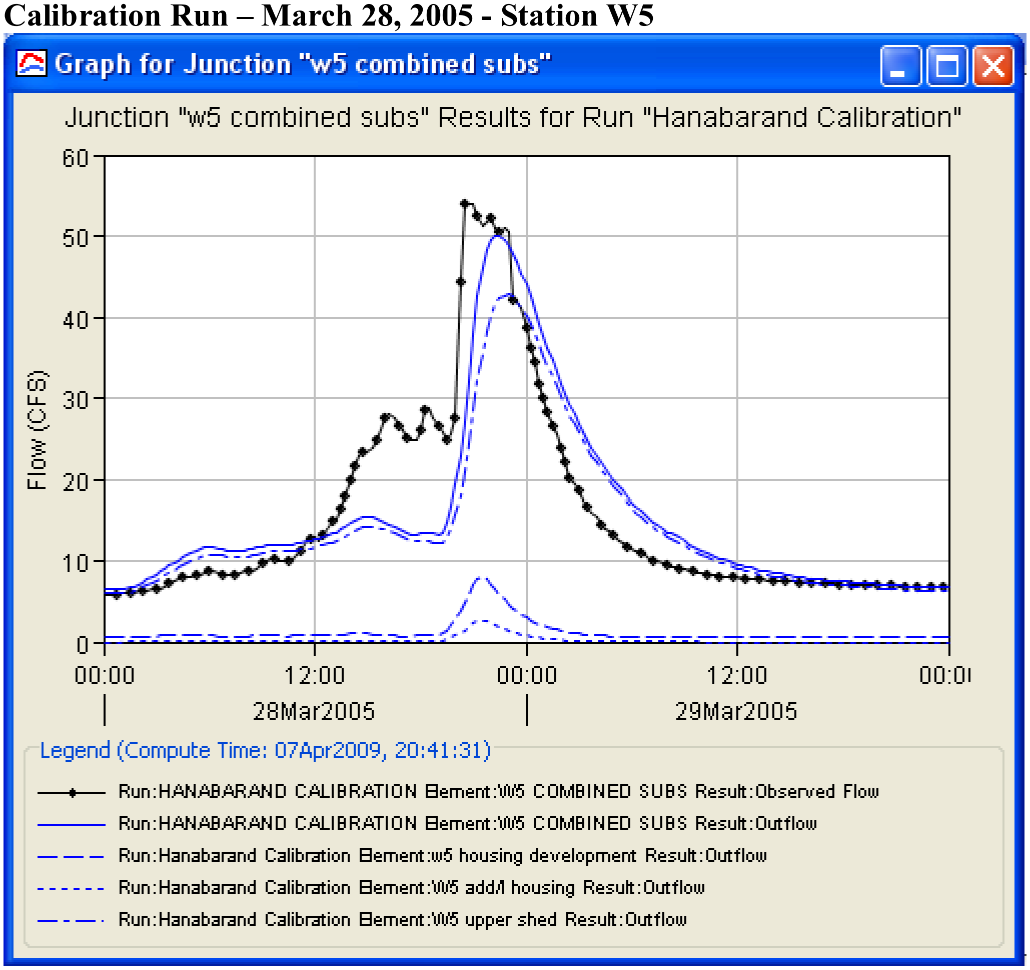


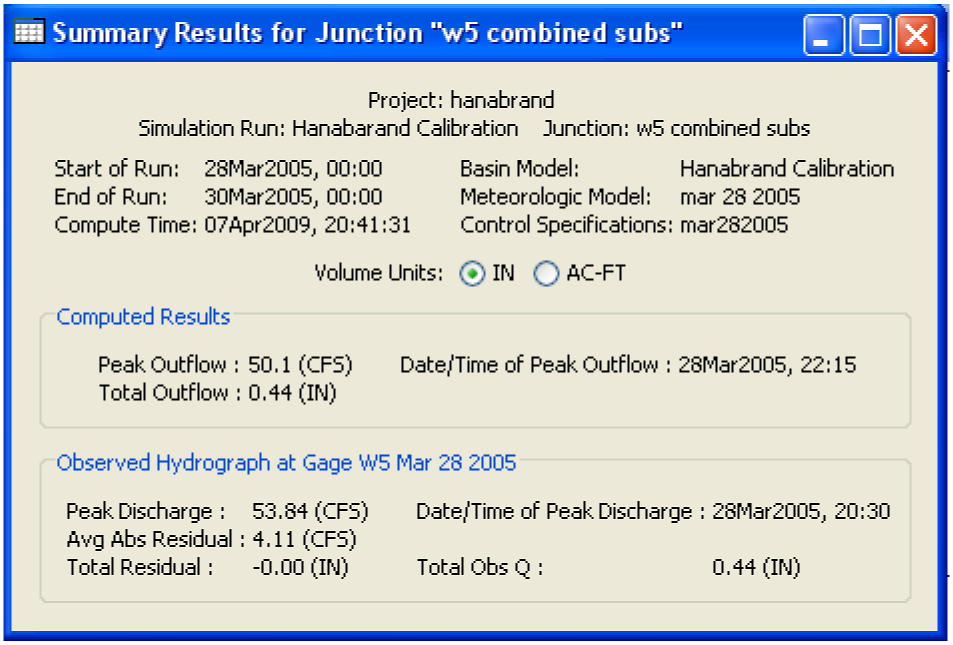
Calibration Run – March 28, 2005 - Station W2 – Hannabrand Outlet





Calibration Run – March 28, 2005 - Station W5





Summary Results for Junction "w5 combined subs"

Project: hanabrand
Simulation Run: Hanabarand Calibration Junction: w5 combined subs

Start of Run:	28Mar2005, 00:00	Basin Model:	Hanabrand Calibration
End of Run:	30Mar2005, 00:00	Meteorologic Model:	mar 28 2005
Compute Time:	07Apr2009, 20:41:31	Control Specifications:	mar282005

Volume Units: IN AC-FT

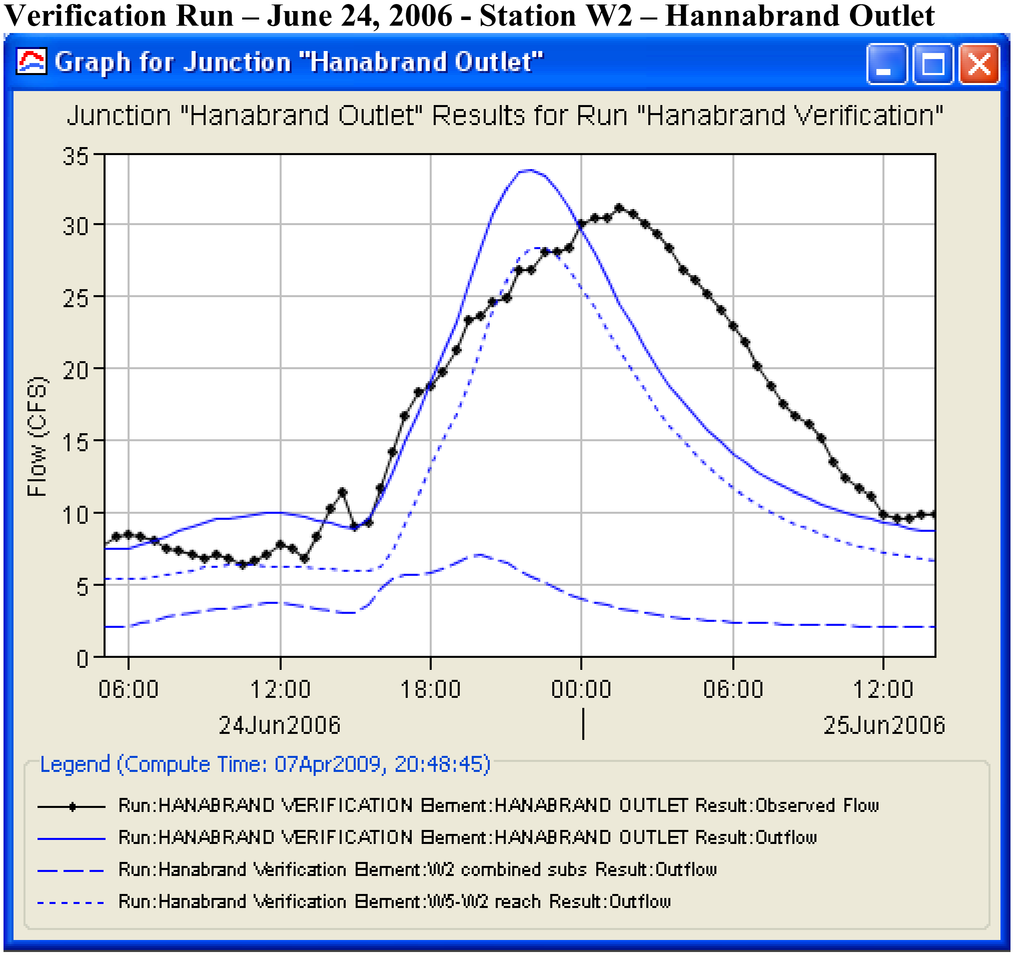
Computed Results

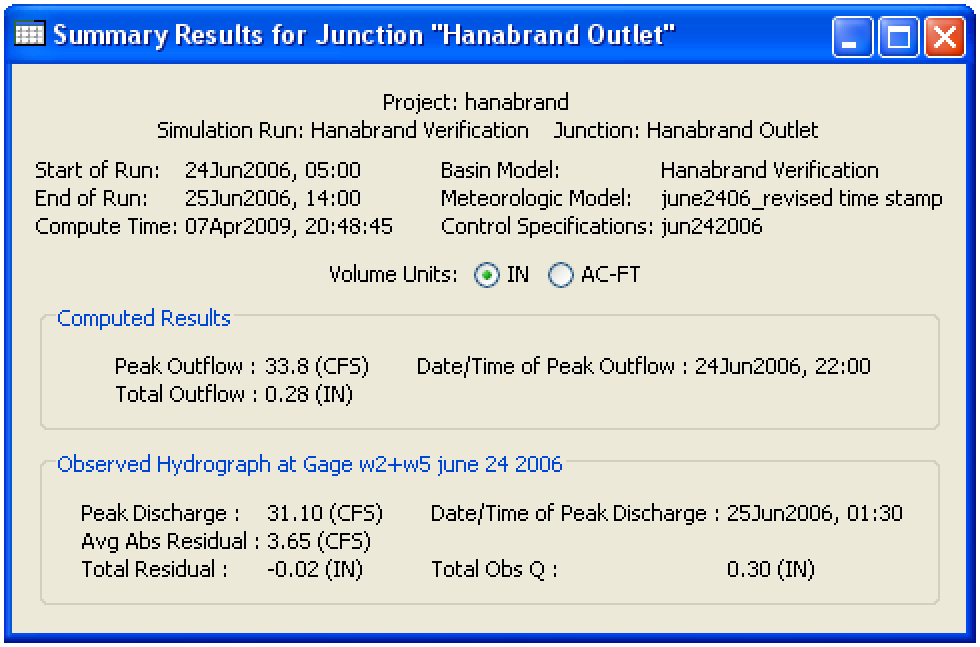
Peak Outflow : 50.1 (CFS) Date/Time of Peak Outflow : 28Mar2005, 22:15
Total Outflow : 0.44 (IN)

Observed Hydrograph at Gage W5 Mar 28 2005

Peak Discharge :	53.84 (CFS)	Date/Time of Peak Discharge :	28Mar2005, 20:30
Avg Abs Residual :	4.11 (CFS)		
Total Residual :	-0.00 (IN)	Total Obs Q :	0.44 (IN)

Verification Run – June 24, 2006 - Station W2 – Hannabrand Outlet




Summary Results for Junction "Hanabrand Outlet"

Project: hanabrand
Simulation Run: Hanabrand Verification   Junction: Hanabrand Outlet

Start of Run:	24Jun2006, 05:00	Basin Model:	Hanabrand Verification
End of Run:	25Jun2006, 14:00	Meteorologic Model:	june2406_revised time stamp
Compute Time:	07Apr2009, 20:48:45	Control Specifications:	jun242006

Volume Units: IN AC-FT

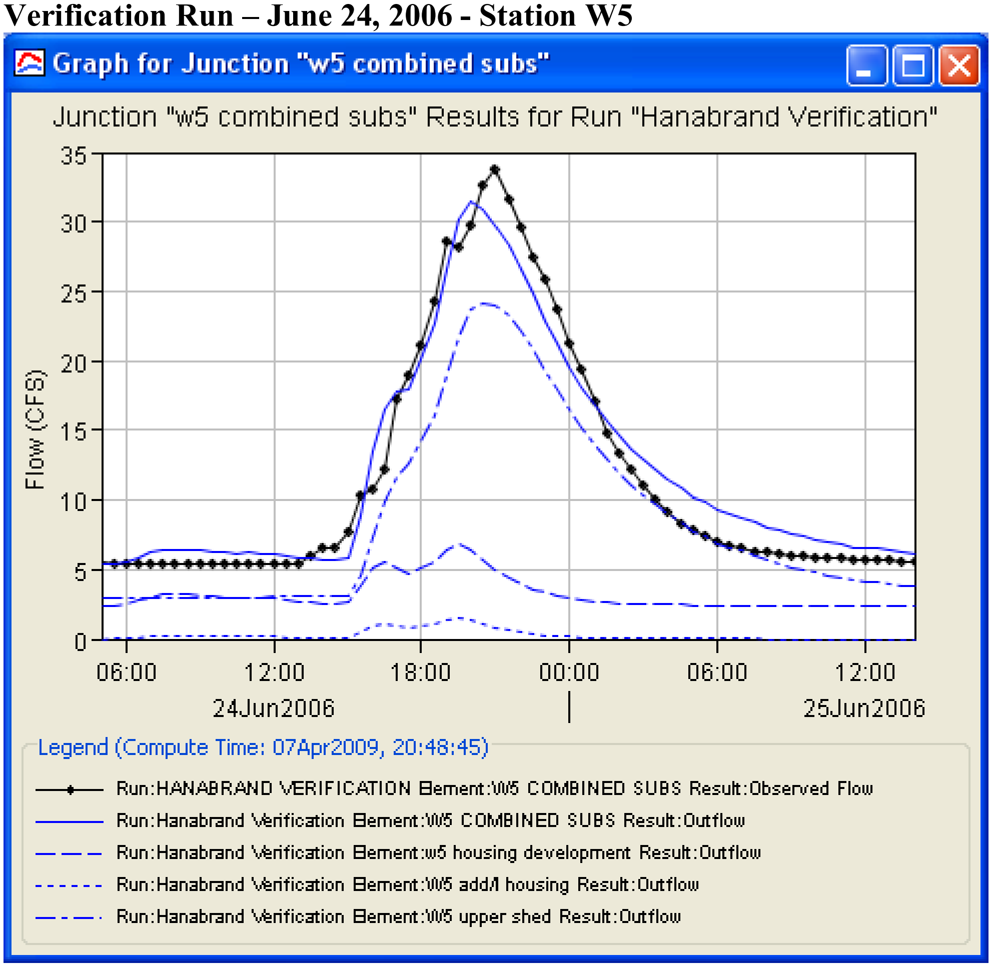
Computed Results

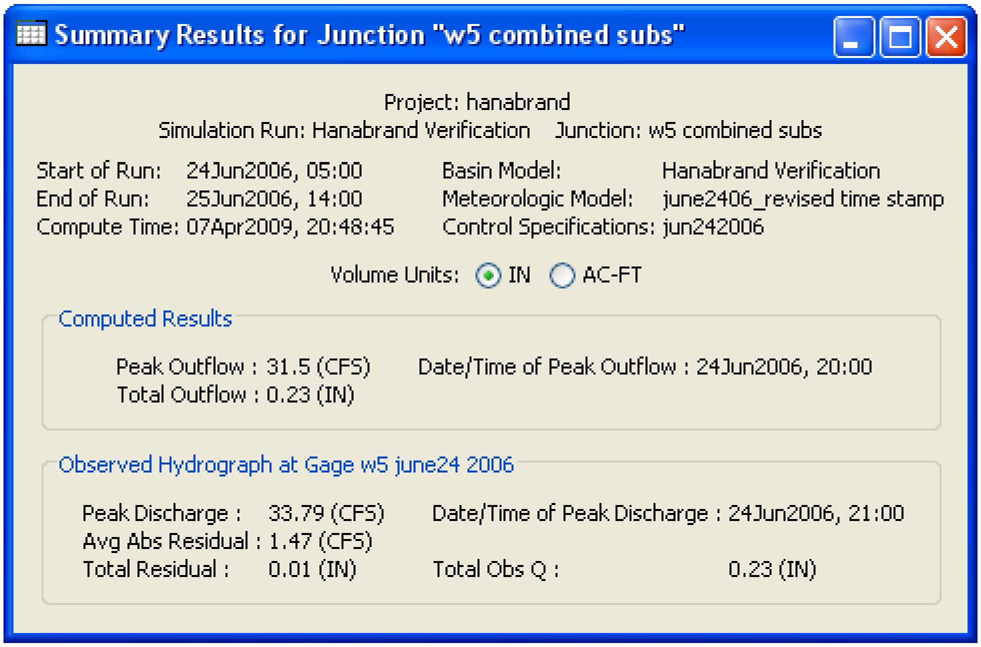
Peak Outflow : 33.8 (CFS)   Date/Time of Peak Outflow : 24Jun2006, 22:00
Total Outflow : 0.28 (IN)

Observed Hydrograph at Gage w2+w5 june 24 2006

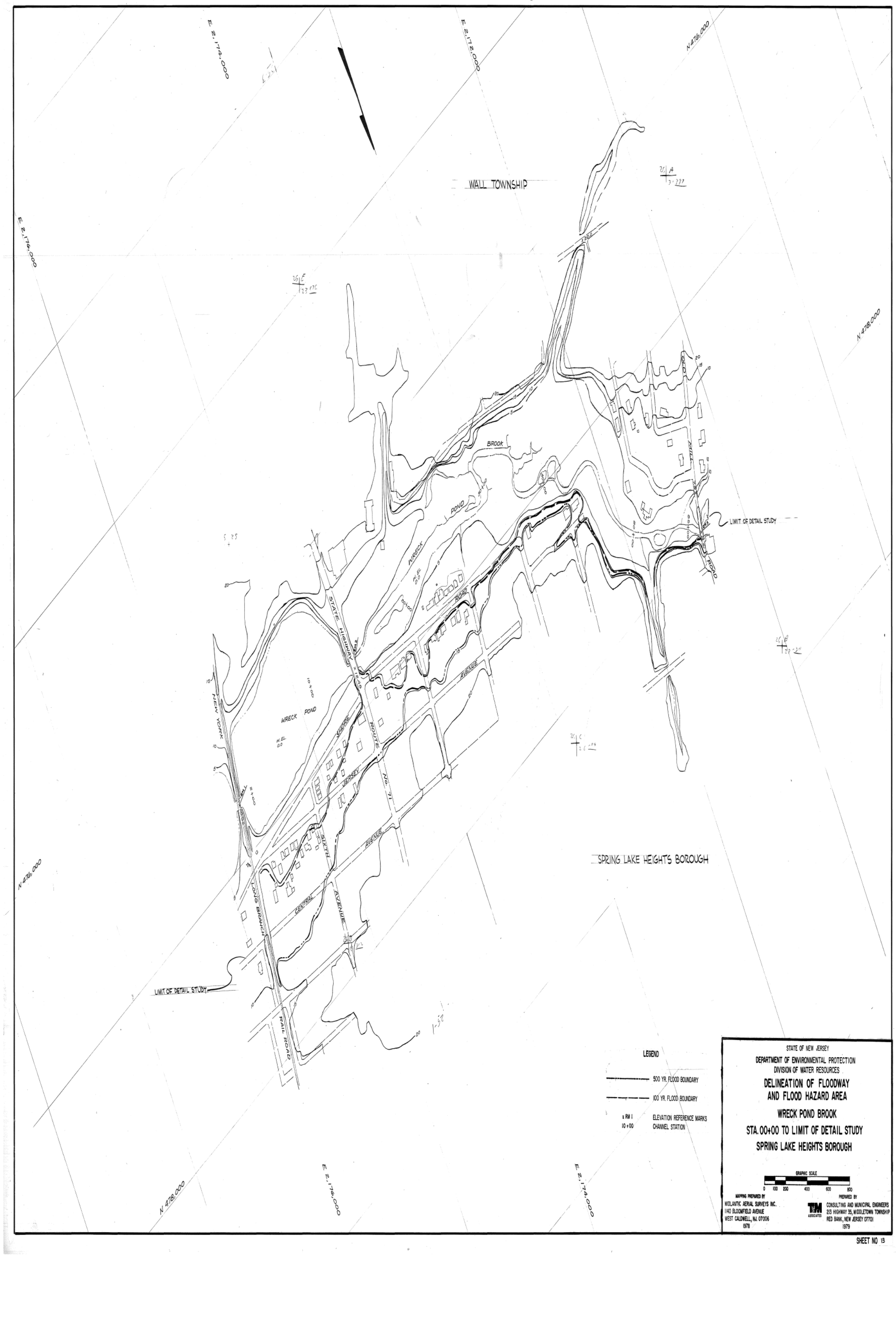
Peak Discharge : 31.10 (CFS)   Date/Time of Peak Discharge : 25Jun2006, 01:30
Avg Abs Residual : 3.65 (CFS)
Total Residual : -0.02 (IN)   Total Obs Q : 0.30 (IN)

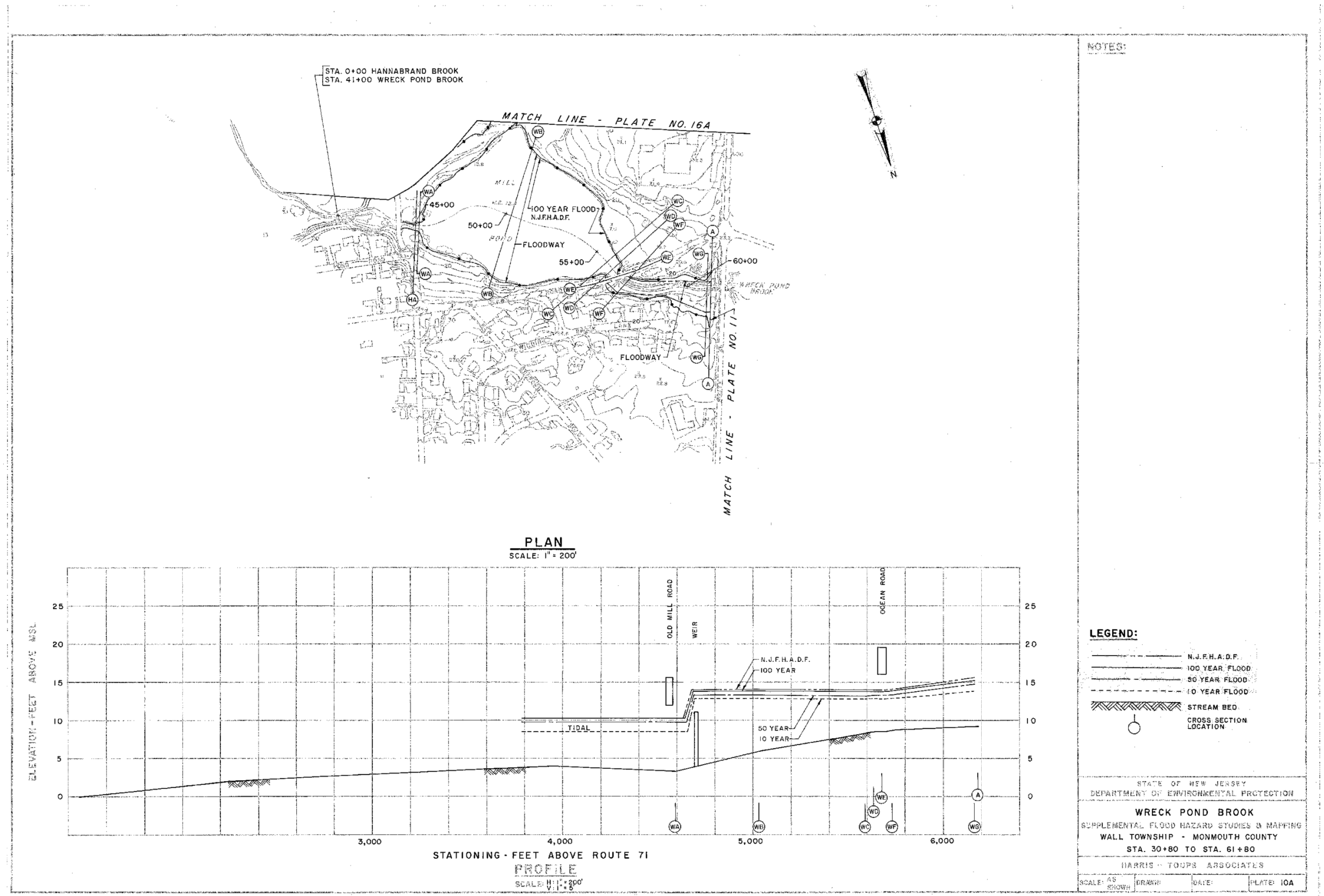
Verification Run – June 24, 2006 - Station W5

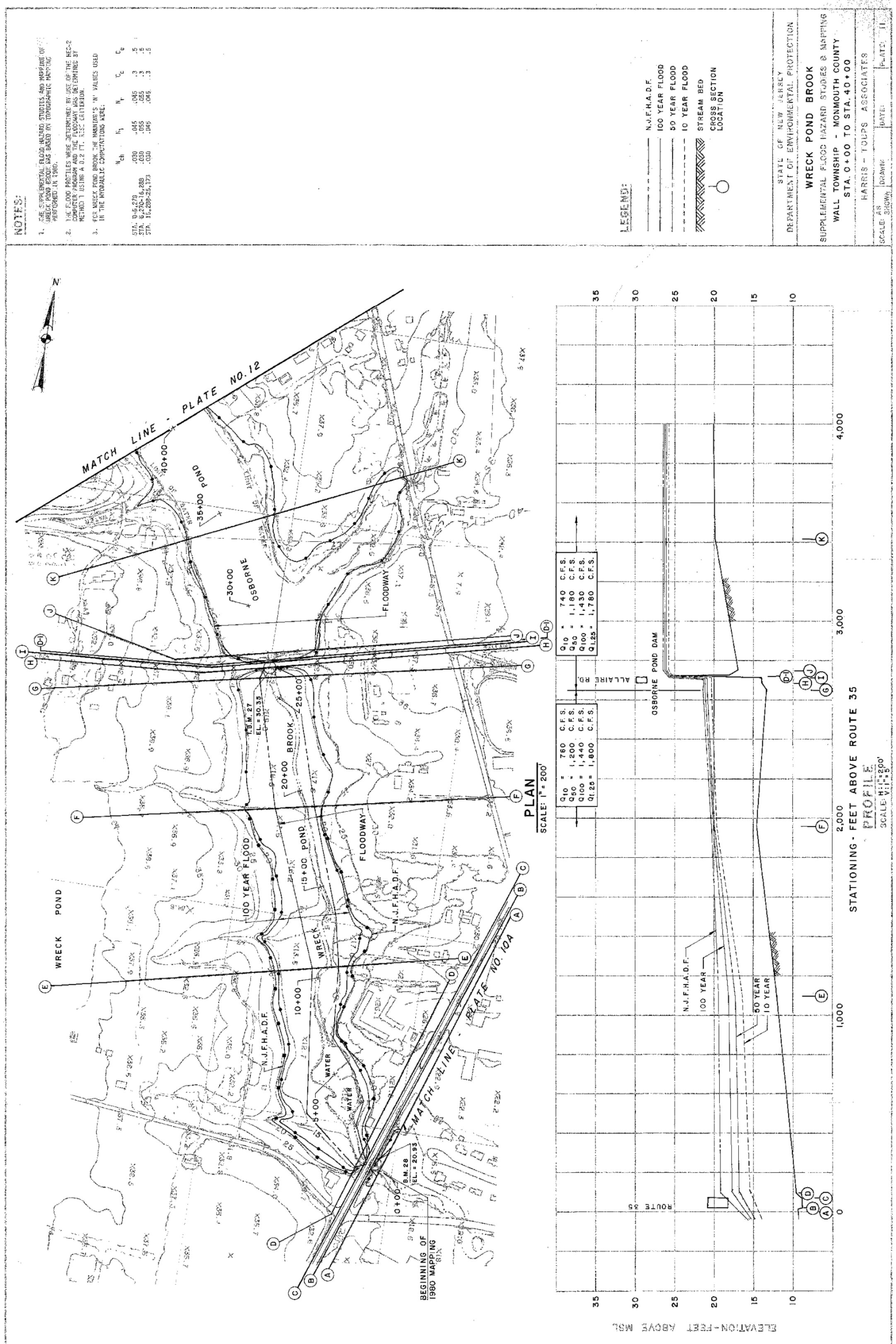


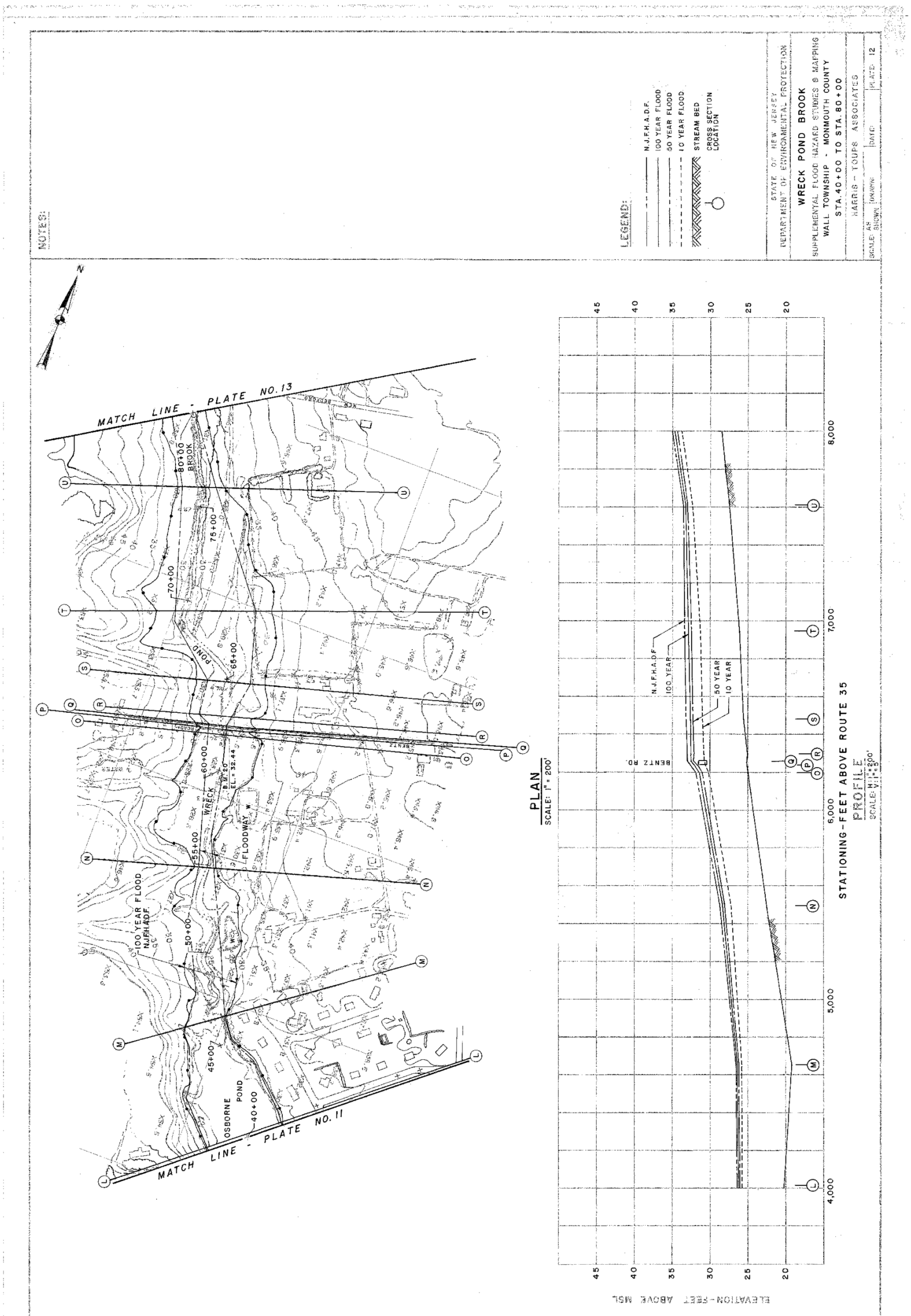

Summary Results for Junction "w5 combined subs"
Project: hanabrand
Simulation Run: Hanabrand Verification Junction: w5 combined subs
Start of Run: 24Jun2006, 05:00
End of Run: 25Jun2006, 14:00
Compute Time: 07Apr2009, 20:48:45
Basin Model: Hanabrand Verification
Meteorologic Model: june2406_revised time stamp
Control Specifications: jun242006
Volume Units: IN AC-FT
Computed Results
Peak Outflow : 31.5 (CFS)
Date/Time of Peak Outflow : 24Jun2006, 20:00
Total Outflow : 0.23 (IN)
Observed Hydrograph at Gage w5 june24 2006
Peak Discharge : 33.79 (CFS)
Date/Time of Peak Discharge : 24Jun2006, 21:00
Avg Abs Residual : 1.47 (CFS)
Total Residual : 0.01 (IN)
Total Obs Q : 0.23 (IN)

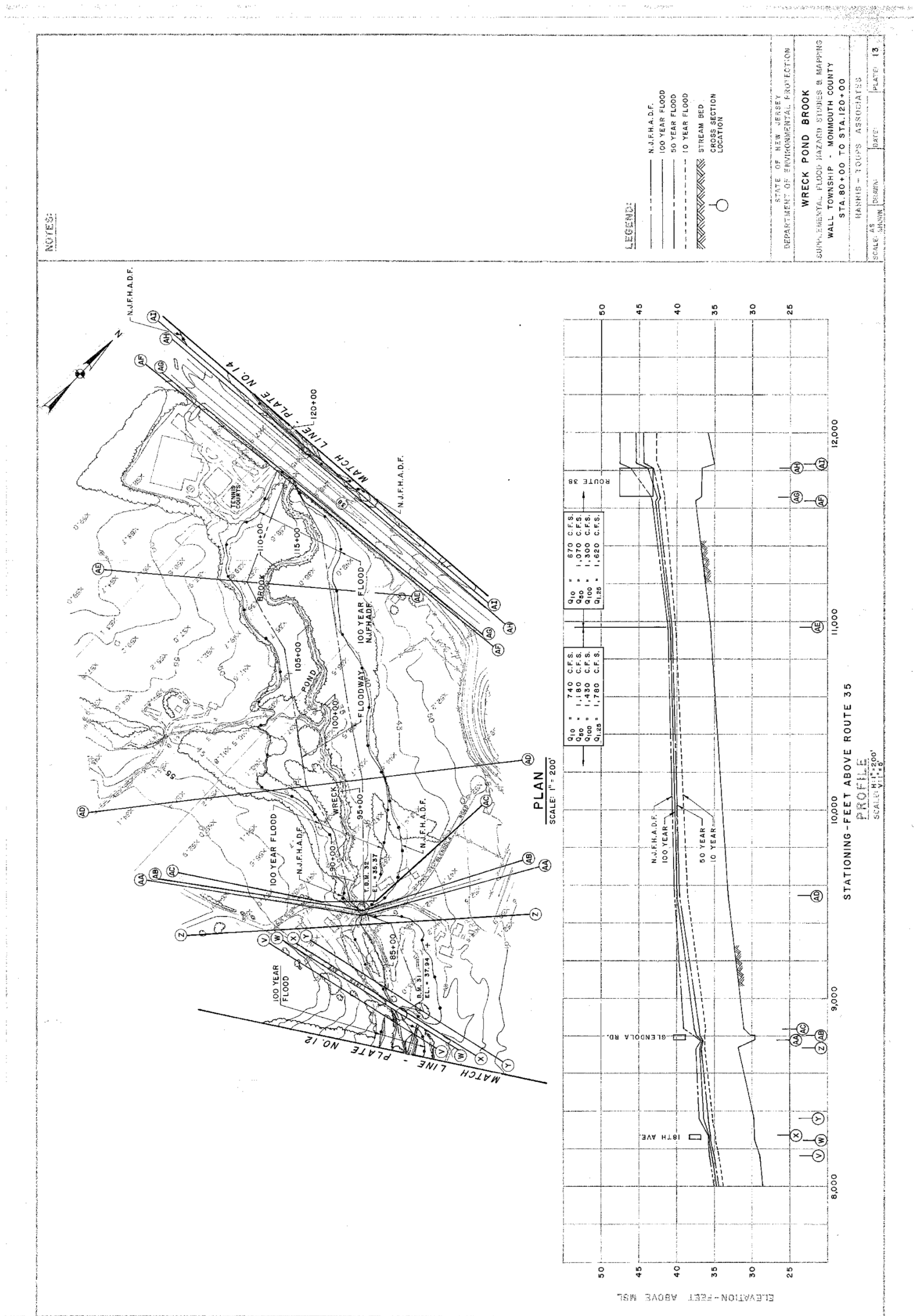
APPENDIX G: SUPPLEMENTAL FLOOD HAZARD STUDIES AND MAPPING FOR WRECK POND BROOK AND HANNABRAND BROOK

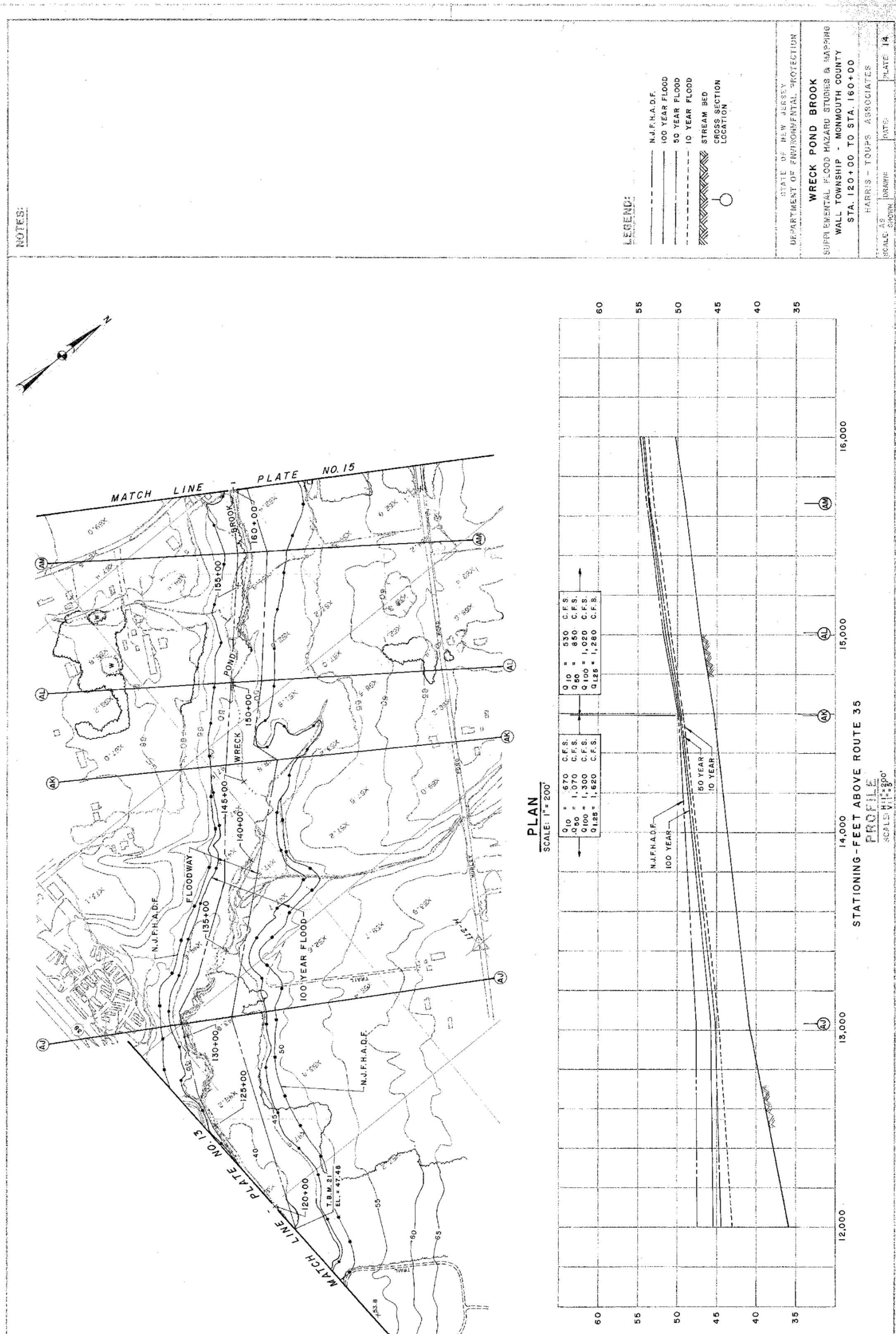

WALL TOWNSHIP
SPRING LAKE HEIGHTS BOROUGH
WRECK POND
LIMIT OF DETAIL STUDY
LIMIT OF DETAIL STUDY
E 2,174,000
E 2,172,000
E 2,176,000
N 478,000
N 476,000
LEGEND
500 YR. FLOOD BOUNDARY
100 YR. FLOOD BOUNDARY
x RM 1 ELEVATION REFERENCE MARKS
10+00 CHANNEL STATION
STATE OF NEW JERSEY
DEPARTMENT OF ENVIRONMENTAL PROTECTION
DIVISION OF WATER RESOURCES
DELINEATION OF FLOODWAY
AND FLOOD HAZARD AREA
WRECK POND BROOK
STA. 00+00 TO LIMIT OF DETAIL STUDY
SPRING LAKE HEIGHTS BOROUGH
GRAPHIC SCALE
0 100 200 400 600 800
MAPPING PREPARED BY
MIDLANTIC AERIAL SURVEYS INC.
1140 BLOOMFIELD AVENUE
WEST CALDWELL, N.J. 07006
1978
PREPARED BY
T&M ASSOCIATES
CONSULTING AND MUNICIPAL ENGINEERS
213 HIGHWAY 35, MIDDLETOWN TOWNSHIP
RED BANK, NEW JERSEY 07701
1979
SHEET NO 13

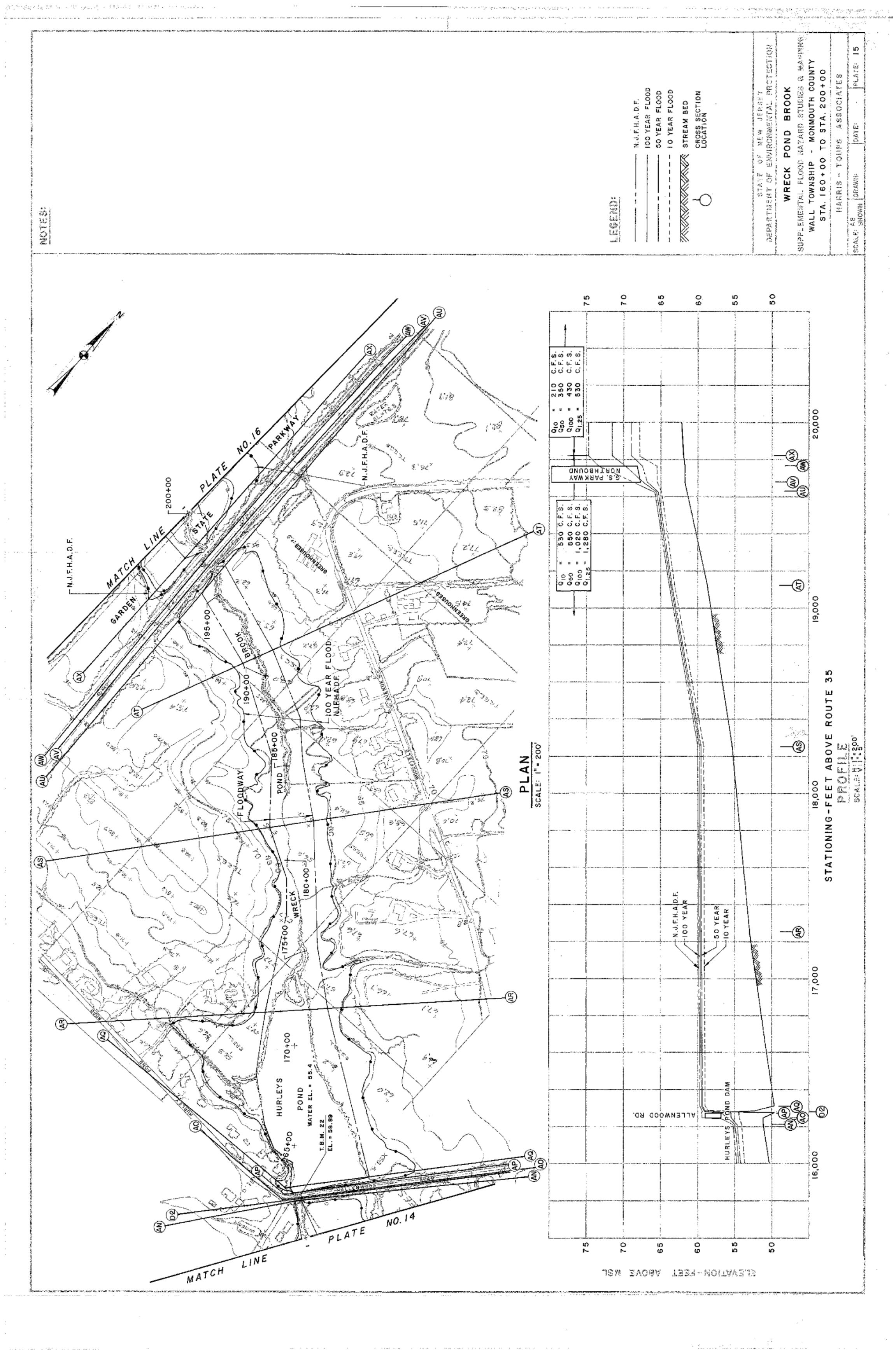

NOTES:
STA. 0+00 HANNABRAND BROOK
STA. 41+00 WRECK POND BROOK
MATCH LINE - PLATE NO. 16A
MATCH LINE - PLATE NO. 11
100 YEAR FLOOD
N.J.F.H.A.D.F.
FLOODWAY
45+00
50+00
55+00
60+00
MILL POND
WRECK POND BROOK
N
PLAN
SCALE: 1" = 200'
OLD MILL ROAD
WEIR
OCEAN ROAD
N.J.F.H.A.D.F.
100 YEAR
50 YEAR
10 YEAR
TIDAL
ELEVATION - FEET ABOVE MSL
STATIONING - FEET ABOVE ROUTE 71
PROFILE
SCALE: H: 1" = 200' V: 1" = 5'
LEGEND:
N.J.F.H.A.D.F.
100 YEAR FLOOD
50 YEAR FLOOD
10 YEAR FLOOD
STREAM BED
CROSS SECTION LOCATION
STATE OF NEW JERSEY
DEPARTMENT OF ENVIRONMENTAL PROTECTION
WRECK POND BROOK
SUPPLEMENTAL FLOOD HAZARD STUDIES & MAPPING
WALL TOWNSHIP - MONMOUTH COUNTY
STA. 30+80 TO STA. 61+80
HARRIS - TOUPS ASSOCIATES
SCALE: AS SHOWN
DRAWN
DATE:
PLATE: 10A

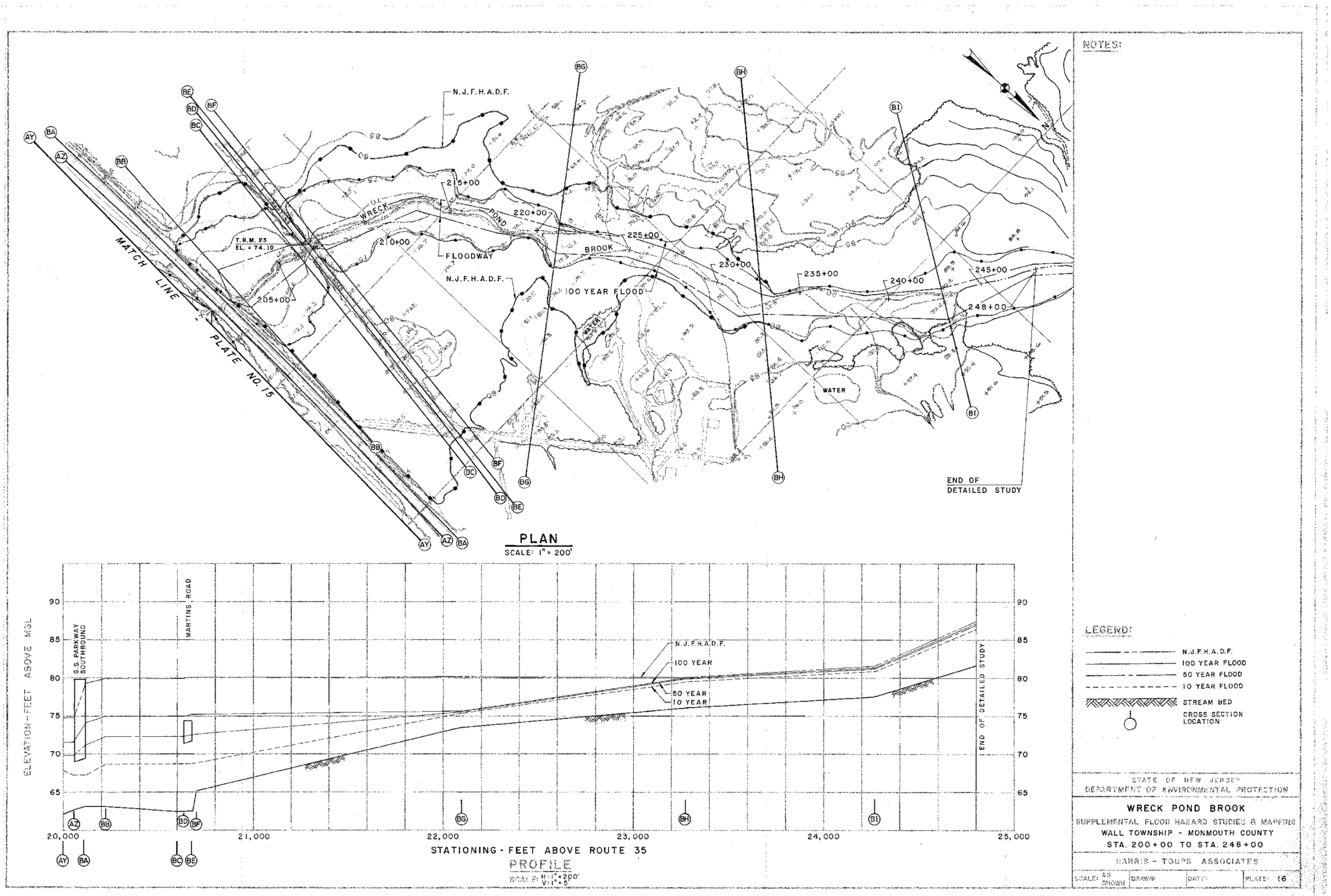

STATE OF NEW JERSEY
DEPARTMENT OF ENVIRONMENTAL PROTECTION
WRECK POND BROOK
SUPPLEMENTAL FLOOD HAZARD STUDIES & MAPPING
WALL TOWNSHIP - MONMOUTH COUNTY
STA. 0+00 TO STA. 40+00
HARRIS - TOUPS ASSOCIATES
LEGEND:
N.J.F.H.A.D.F.
100 YEAR FLOOD
50 YEAR FLOOD
10 YEAR FLOOD
STREAM BED
CROSS SECTION LOCATION
MATCH LINE - PLATE NO. 12
WRECK POND
OSBORNE POND DAM
ALLAIRE RD.
ROUTE 35
PLAN
PROFILE
STATIONING - FEET ABOVE ROUTE 35
ELEVATION-FEET ABOVE MSL
BEGINNING OF 1980 MAPPING

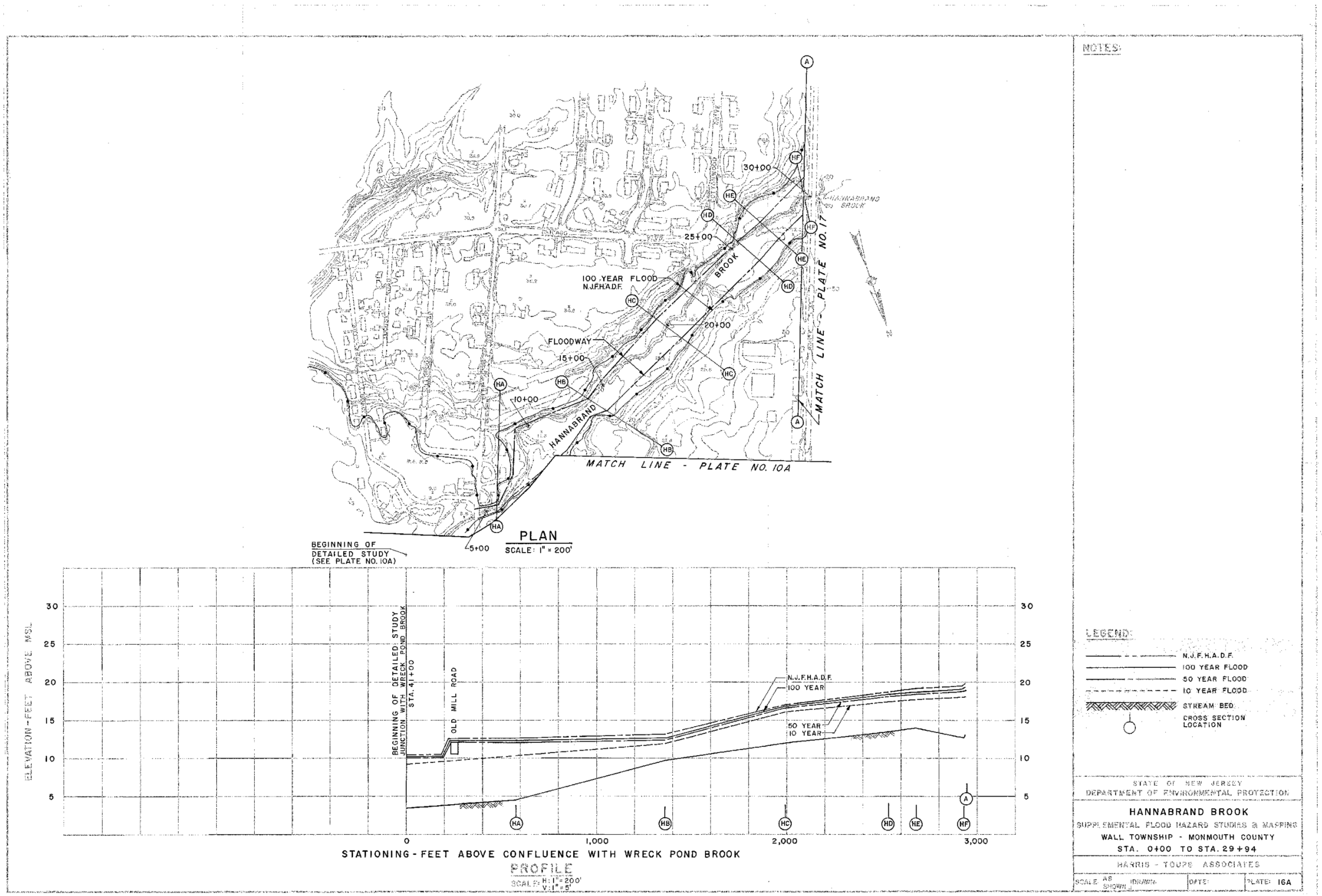

NOTES:
LEGEND:
N.J.F.H.A.D.F.
100 YEAR FLOOD
50 YEAR FLOOD
10 YEAR FLOOD
STREAM BED
CROSS SECTION LOCATION
STATE OF NEW JERSEY
DEPARTMENT OF ENVIRONMENTAL PROTECTION
WRECK POND BROOK
SUPPLEMENTAL FLOOD HAZARD STUDIES & MAPPING
WALL TOWNSHIP - MONMOUTH COUNTY
STA. 40+00 TO STA. 80+00
HARRIS - TOUPS ASSOCIATES
SCALE AS SHOWN
DRAWN
DATE
PLATE 12
MATCH LINE - PLATE NO. 13
MATCH LINE - PLATE NO. 11
WRECK POND BROOK
FLOODWAY
OSBORNE POND
100 YEAR FLOOD
NJFHADF
BENTZ RD.
PLAN
SCALE: 1" = 200'
PROFILE
SCALE: H: 1" = 200' V: 1" = 5'
STATIONING-FEET ABOVE ROUTE 35
ELEVATION-FEET ABOVE MSL
N.J.F.H.A.D.F.
100 YEAR
50 YEAR
10 YEAR
4,000
5,000
6,000
7,000
8,000


NOTES:
LEGEND:
N.J.F.H.A.D.F.
100 YEAR FLOOD
50 YEAR FLOOD
10 YEAR FLOOD
STREAM BED
CROSS SECTION LOCATION
STATE OF NEW JERSEY
DEPARTMENT OF ENVIRONMENTAL PROTECTION
WRECK POND BROOK
SUPPLEMENTAL FLOOD HAZARD STUDIES & MAPPING
WALL TOWNSHIP - MONMOUTH COUNTY
STA.80+00 TO STA.120+00
HARRIS - TOUPS ASSOCIATES
SCALE: AS SHOWN
DRAWN:
DATE:
PLATE: 13
PLAN
SCALE: 1" = 200'
MATCH LINE - PLATE NO.12
MATCH LINE - PLATE NO.14
PROFILE
STATIONING-FEET ABOVE ROUTE 35
ELEVATION-FEET ABOVE MSL
ROUTE 35
GLENDOLA RD.
18TH AVE.

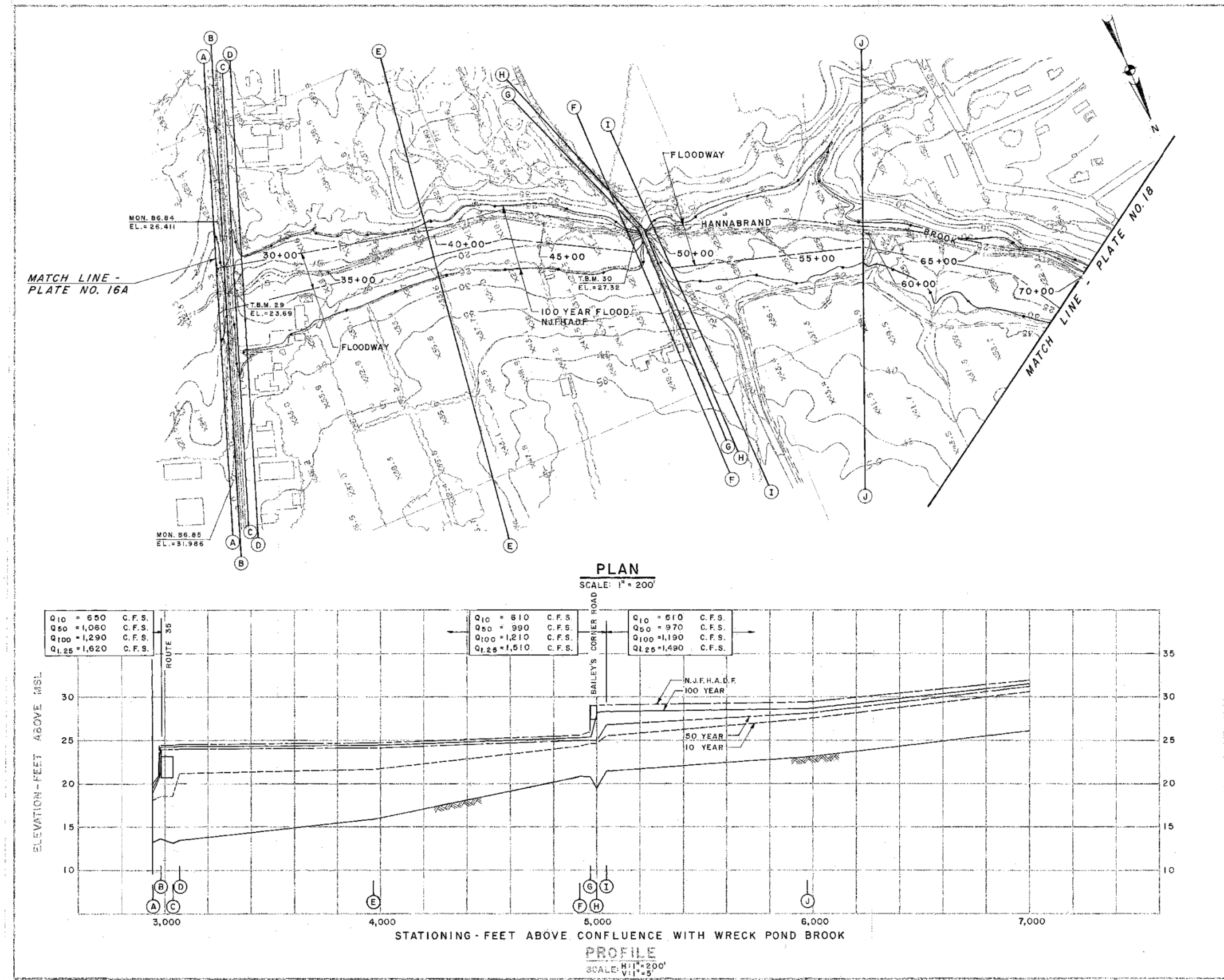

NOTES:
MATCH LINE - PLATE NO. 13
MATCH LINE - PLATE NO. 15
N.J.F.H.A.D.F.
FLOODWAY
100 YEAR FLOOD
WRECK
POND
BROOK
120+00
125+00
130+00
135+00
140+00
145+00
150+00
155+00
160+00
T.B.M. 21
EL. = 47.48
PLAN
SCALE: 1" = 200'
Q10 = 670 C.F.S.
Q50 = 1,070 C.F.S.
Q100 = 1,300 C.F.S.
Q1.25 = 1,620 C.F.S.
Q10 = 530 C.F.S.
Q50 = 850 C.F.S.
Q100 = 1,020 C.F.S.
Q1.25 = 1,280 C.F.S.
N.J.F.H.A.D.F.
100 YEAR
50 YEAR
10 YEAR
ELEVATION-FEET ABOVE MSL
STATIONING-FEET ABOVE ROUTE 35
PROFILE
LEGEND:
N.J.F.H.A.D.F.
100 YEAR FLOOD
50 YEAR FLOOD
10 YEAR FLOOD
STREAM BED
CROSS SECTION LOCATION
STATE OF NEW JERSEY
DEPARTMENT OF ENVIRONMENTAL PROTECTION
WRECK POND BROOK
SUPPLEMENTAL FLOOD HAZARD STUDIES & MAPPING
WALL TOWNSHIP - MONMOUTH COUNTY
STA. 120+00 TO STA. 160+00
HARRIS - TOUPS ASSOCIATES
SCALE: AS SHOWN
DRAWN:
DATE:
PLATE: 14


NOTES:
LEGEND:
N.J.F.H.A.D.F.
100 YEAR FLOOD
50 YEAR FLOOD
10 YEAR FLOOD
STREAM BED
CROSS SECTION LOCATION
STATE OF NEW JERSEY
DEPARTMENT OF ENVIRONMENTAL PROTECTION
WRECK POND BROOK
SUPPLEMENTAL FLOOD HAZARD STUDIES & MAPPING
WALL TOWNSHIP - MONMOUTH COUNTY
STA. 160+00 TO STA. 200+00
HARRIS - TOUPS ASSOCIATES
PLATE: 15
MATCH LINE - PLATE NO. 16
MATCH LINE - PLATE NO. 14
GARDEN STATE PARKWAY
N.J.F.H.A.D.F.
FLOODWAY
100 YEAR FLOOD N.J.F.H.A.D.F.
WRECK POND BROOK
GREENHOUSES
HURLEYS POND
WATER EL. = 55.4
T.B.M. 22
EL. = 58.89
PLAN
SCALE: 1" = 200'
Q10 = 530 C.F.S.
Q50 = 850 C.F.S.
Q100 = 1,020 C.F.S.
Q1.25 = 1,280 C.F.S.
Q10 = 210 C.F.S.
Q50 = 350 C.F.S.
Q100 = 430 C.F.S.
Q1.25 = 530 C.F.S.
G.S. PARKWAY NORTHBOUND
ALLENWOOD RD.
HURLEYS POND DAM
N.J.F.H.A.D.F.
100 YEAR
50 YEAR
10 YEAR
STATIONING - FEET ABOVE ROUTE 35
PROFILE
ELEVATION - FEET ABOVE MSL


NOTES:
N.J.F.H.A.D.F.
WRECK
POND
BROOK
FLOODWAY
100 YEAR FLOOD
WATER
T.B.M. 23
EL. = 74.10
MATCH LINE - PLATE NO. 15
205+00
210+00
215+00
220+00
225+00
230+00
235+00
240+00
245+00
248+00
END OF DETAILED STUDY
PLAN
SCALE: 1" = 200'
ELEVATION-FEET ABOVE MSL
G.S. PARKWAY SOUTHBOUND
MARTINS ROAD
100 YEAR
50 YEAR
10 YEAR
20,000
21,000
22,000
23,000
24,000
25,000
STATIONING - FEET ABOVE ROUTE 35
PROFILE
SCALE: H: 1" = 200' V: 1" = 5'
LEGEND:
N.J.F.H.A.D.F.
100 YEAR FLOOD
50 YEAR FLOOD
10 YEAR FLOOD
STREAM BED
CROSS SECTION LOCATION
STATE OF NEW JERSEY
DEPARTMENT OF ENVIRONMENTAL PROTECTION
WRECK POND BROOK
SUPPLEMENTAL FLOOD HAZARD STUDIES & MAPPING
WALL TOWNSHIP - MONMOUTH COUNTY
STA. 200+00 TO STA. 248+00
HARRIS - TOUPS ASSOCIATES
SCALE: AS SHOWN
DRAWN
DATE
PLATE 16



NOTES:
PLAN
SCALE: 1" = 200'
MATCH LINE - PLATE NO. 17
MATCH LINE - PLATE NO. 10A
HANNABRAND BROOK
100 YEAR FLOOD
N.J.F.H.A.D.F.
FLOODWAY
BEGINNING OF DETAILED STUDY (SEE PLATE NO. 10A)
BEGINNING OF DETAILED STUDY JUNCTION WITH WRECK POND BROOK STA. 41+00
OLD MILL ROAD
N.J.F.H.A.D.F.
100 YEAR
50 YEAR
10 YEAR
ELEVATION-FEET ABOVE MSL
STATIONING - FEET ABOVE CONFLUENCE WITH WRECK POND BROOK
PROFILE
SCALE: H: 1" = 200' V: 1" = 5'
LEGEND:
N.J.F.H.A.D.F.
100 YEAR FLOOD
50 YEAR FLOOD
10 YEAR FLOOD
STREAM BED
CROSS SECTION LOCATION
STATE OF NEW JERSEY
DEPARTMENT OF ENVIRONMENTAL PROTECTION
HANNABRAND BROOK
SUPPLEMENTAL FLOOD HAZARD STUDIES & MAPPING
WALL TOWNSHIP - MONMOUTH COUNTY
STA. 0+00 TO STA. 29+94
HARRIS - TOUPS ASSOCIATES
SCALE AS SHOWN
PLATE: 16A



NOTES:

1. THE SUPPLEMENTAL FLOOD HAZARD STUDIES AND MAPPING OF HANNABRAND BROOK WAS BASED ON TOPOGRAPHIC MAPPING PERFORMED IN 1980.
2. THE FLOOD PROFILES WERE DETERMINED BY USE OF THE HEC-2 COMPUTER PROGRAM AND THE FLOODWAY WAS DETERMINED BY METHOD 1 USING A 0.2 FT. RISE CRITERION.
3. FOR HANNABRAND BROOK THE MANNING'S 'N' VALUES USED IN THE HYDRAULIC COMPUTATIONS WERE:

	N_{ch}	N_l	N_r	C_c	C_e
STA. 2,946-13,822	.035	.075	.075	.1	.3
STA. 13,822-15,573	.035	.050	.060	.1	.3
STA. 15,573-16,828	.035	.055	.055	.1	.3

LEGEND:

- N.J.F.H.A.D.F.
- 100 YEAR FLOOD
- 50 YEAR FLOOD
- 10 YEAR FLOOD
- STREAM BED
- CROSS SECTION LOCATION

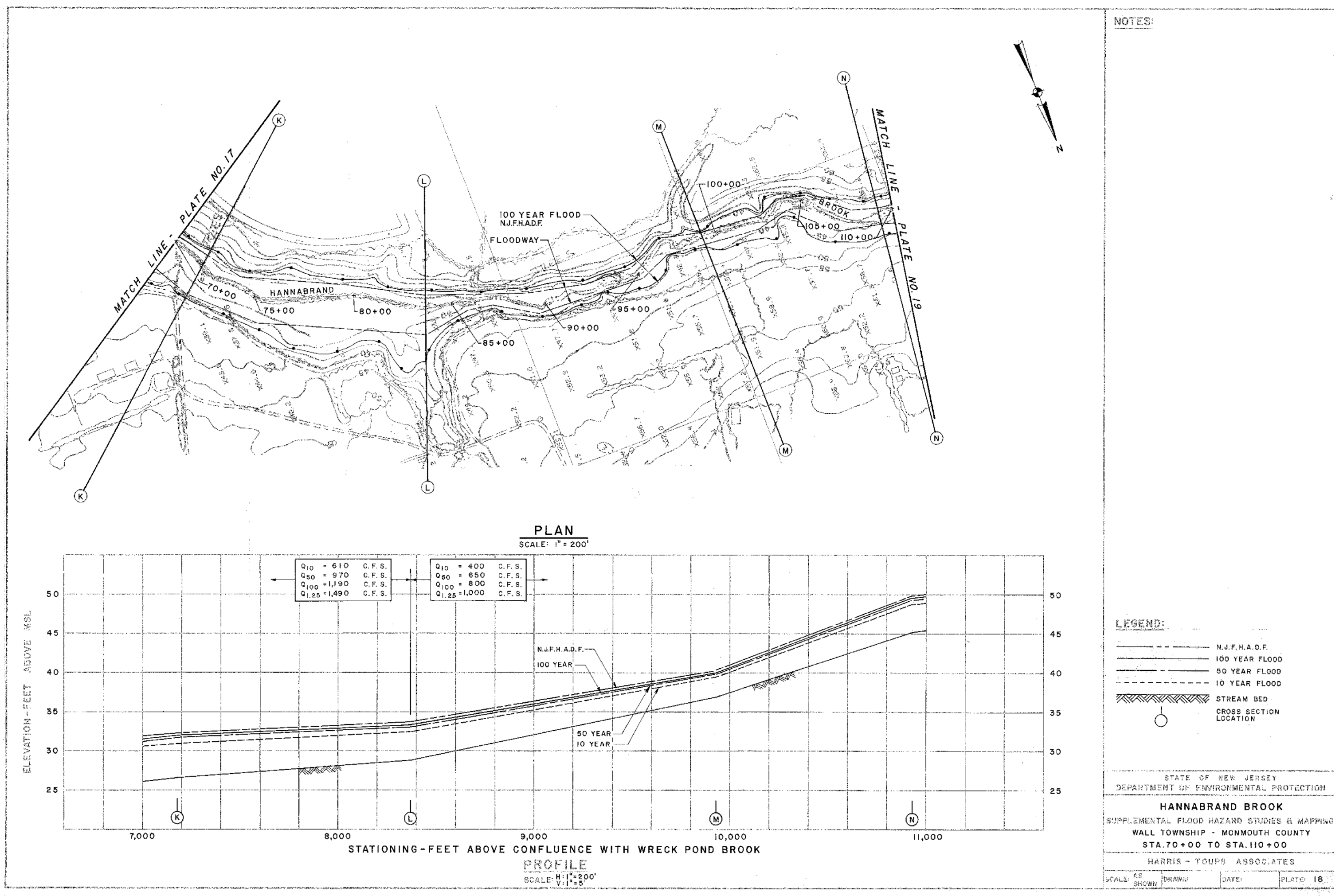
STATE OF NEW JERSEY
DEPARTMENT OF ENVIRONMENTAL PROTECTION

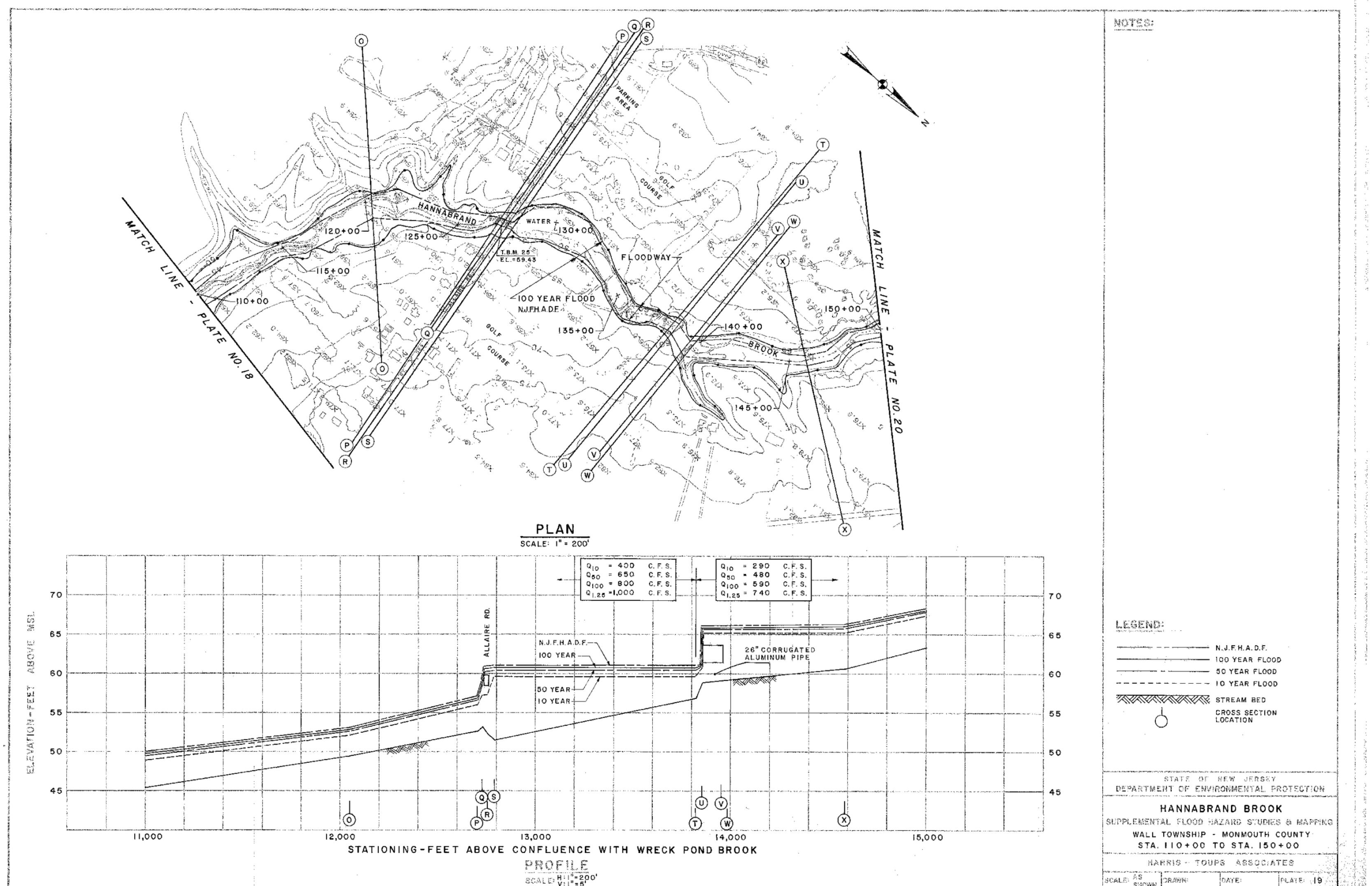
HANNABRAND BROOK

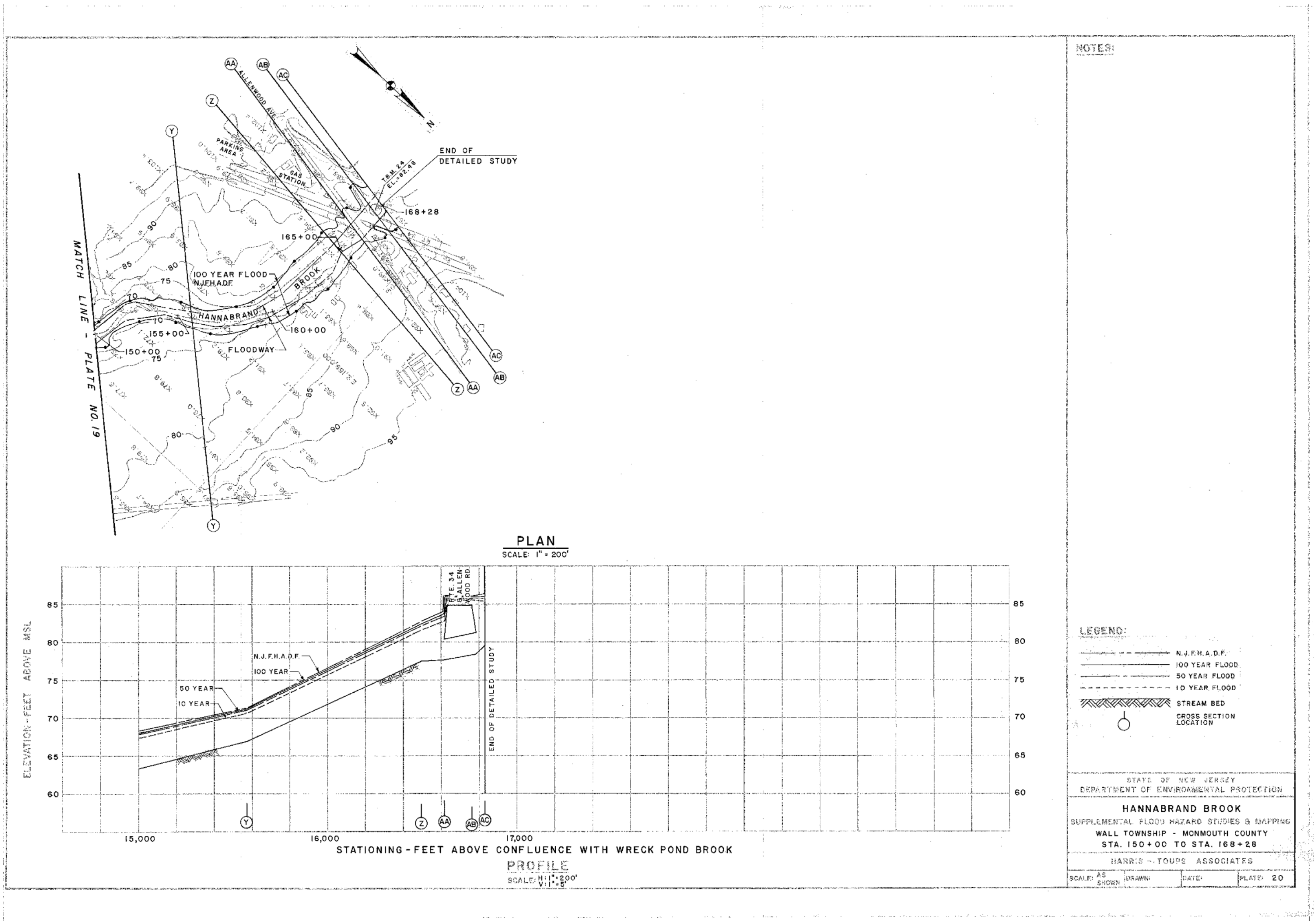
SUPPLEMENTAL FLOOD HAZARD STUDIES & MAPPING
WALL TOWNSHIP - MONMOUTH COUNTY
STA. 29+94 TO STA. 70+00

HARRIS - TOUPS ASSOCIATES

SCALE: AS SHOWN	DRAWN:	DATE:	PLATE: 17


NOTES:
MATCH LINE - PLATE NO. 17
MATCH LINE - PLATE NO. 19
100 YEAR FLOOD
N.J.F.H.A.D.F.
FLOODWAY
HANNABRAND
BROOK
70+00
75+00
80+00
85+00
90+00
95+00
100+00
105+00
110+00
N
PLAN
SCALE: 1" = 200'
Q10 = 610 C.F.S.
Q50 = 970 C.F.S.
Q100 = 1,190 C.F.S.
Q1.25 = 1,490 C.F.S.
Q10 = 400 C.F.S.
Q50 = 650 C.F.S.
Q100 = 800 C.F.S.
Q1.25 = 1,000 C.F.S.
N.J.F.H.A.D.F.
100 YEAR
50 YEAR
10 YEAR
ELEVATION-FEET ABOVE MSL
STATIONING-FEET ABOVE CONFLUENCE WITH WRECK POND BROOK
PROFILE
SCALE: H: 1" = 200' V: 1" = 5'
LEGEND:
N.J.F.H.A.D.F.
100 YEAR FLOOD
50 YEAR FLOOD
10 YEAR FLOOD
STREAM BED
CROSS SECTION LOCATION
STATE OF NEW JERSEY
DEPARTMENT OF ENVIRONMENTAL PROTECTION
HANNABRAND BROOK
SUPPLEMENTAL FLOOD HAZARD STUDIES & MAPPING
WALL TOWNSHIP - MONMOUTH COUNTY
STA. 70+00 TO STA. 110+00
HARRIS - TOUPS ASSOCIATES
SCALE: AS SHOWN
DRAWN
DATE:
PLATE: 18


NOTES:
MATCH LINE - PLATE NO.18
MATCH LINE - PLATE NO.20
HANNABRAND
BROOK
WATER
FLOODWAY
100 YEAR FLOOD
NJFHADF
T.B.M. 25
EL.=59.43
PARKING AREA
GOLF COURSE
110+00
115+00
120+00
125+00
130+00
135+00
140+00
145+00
150+00
PLAN
SCALE: 1" = 200'
Q10 = 400 C.F.S.
Q50 = 650 C.F.S.
Q100 = 800 C.F.S.
Q1.25 = 1,000 C.F.S.
Q10 = 290 C.F.S.
Q50 = 480 C.F.S.
Q100 = 590 C.F.S.
Q1.25 = 740 C.F.S.
ALLAIRE RD.
N.J.F.H.A.D.F.
100 YEAR
50 YEAR
10 YEAR
26" CORRUGATED ALUMINUM PIPE
ELEVATION-FEET ABOVE MSL
STATIONING-FEET ABOVE CONFLUENCE WITH WRECK POND BROOK
PROFILE
SCALE: H: 1"=200' V: 1"=5'
LEGEND:
N.J.F.H.A.D.F.
100 YEAR FLOOD
50 YEAR FLOOD
10 YEAR FLOOD
STREAM BED
CROSS SECTION LOCATION
STATE OF NEW JERSEY
DEPARTMENT OF ENVIRONMENTAL PROTECTION
HANNABRAND BROOK
SUPPLEMENTAL FLOOD HAZARD STUDIES & MAPPING
WALL TOWNSHIP - MONMOUTH COUNTY
STA. 110+00 TO STA. 150+00
HARRIS - TOUPS ASSOCIATES
SCALE: AS SHOWN
DRAWN:
DATE:
PLATE: 19


NOTES:
END OF DETAILED STUDY
168+28
165+00
160+00
155+00
150+00
100 YEAR FLOOD NJFHADF
HANNABRAND BROOK
FLOODWAY
ALLENWOOD AVE
PARKING AREA
GAS STATION
MATCH LINE - PLATE NO. 19
PLAN
SCALE: 1" = 200'
N.J.F.H.A.D.F.
100 YEAR
50 YEAR
10 YEAR
END OF DETAILED STUDY
ELEVATION-FEET ABOVE MSL
STATIONING-FEET ABOVE CONFLUENCE WITH WRECK POND BROOK
PROFILE
SCALE: H:1" = 200' V:1" = 5'
LEGEND:
N.J.F.H.A.D.F.
100 YEAR FLOOD
50 YEAR FLOOD
10 YEAR FLOOD
STREAM BED
CROSS SECTION LOCATION
STATE OF NEW JERSEY
DEPARTMENT OF ENVIRONMENTAL PROTECTION
HANNABRAND BROOK
SUPPLEMENTAL FLOOD HAZARD STUDIES & MAPPING
WALL TOWNSHIP - MONMOUTH COUNTY
STA. 150+00 TO STA. 168+28
HARRIS - TOUPS ASSOCIATES
SCALE: AS SHOWN
DRAWN
DATE:
PLATE: 20

CURRICULUM VITA

Kunal P. Patel, PhD

1/2004 – 5/2009	Rutgers, The State University of New Jersey, Civil & Environmental Engineering Department, PhD
1/2004 - 5/2009	Environmental Engineer, New Jersey Department of Environmental Protection, NJ
11/2001-9/2003	Lecturer, Department of Chemical Engineering, Dharmsinh Desai University (Previously DDIT), Gujarat, India
8/2000-09/2002	Institute of Science and Technology for Advanced Studies and Research, Post Graduate Programme, Gujarat, India, MS (Environmental Engineering)
8/1996-8/2000	G.H.Patel College of Engineering & Technology, Chemical Engineering Department, Gujarat, India, BS (Chemical Engineering)